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PROPOSED REDEVELOPMENT
1755 & 1805 Pickering Parkway,
City of Pickering, Ontario

MASTER SERVICING AND STORM WATER MANAGEMENT REPORT

Prepared For:

Pickering Ridge Lands Inc.
&
Bayfield Realty Advisors

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1. INTRODUCTION

Site Description

The subject development has a total area of 9.484 ha and is bound by Pickering Parkway to the north, Highway 401 to the south, Notion Road and Saint Francis De Sales Cemetery to the east and Brock Road to the west. Currently, the site is developed with multi-tenant, “big box” and smaller commercial retail establishments with associated asphalt parking lots. The topography of the site is relatively flat sloping northeast. The subject site known as First Pickering Place (FPP) is currently designated as “Mixed Use Areas – Specialty Retailing Node” in the Pickering Official Plan; the lands with this designation are intended to have the widest variety of uses and highest levels of activities. An aerial view of the site can be found in Appendix A showing surrounding uses. Refer to Exhibit 1 below for the site location. Exhibit 2 shows the plan of the redeveloped site and location of Phase 1 within the site.

Background

The objective of this report is to define a feasible servicing plan focusing on the Full Development Build out to allow Phase 1 of the development to proceed, which includes Towers A1 and A2. This report will evaluate servicing schemes for the proposed redevelopment at the full build out with respect to sanitary, water and storm servicing and evaluate the stormwater management (SWM) strategy to meet the SWM requirements set out by regulatory agencies.

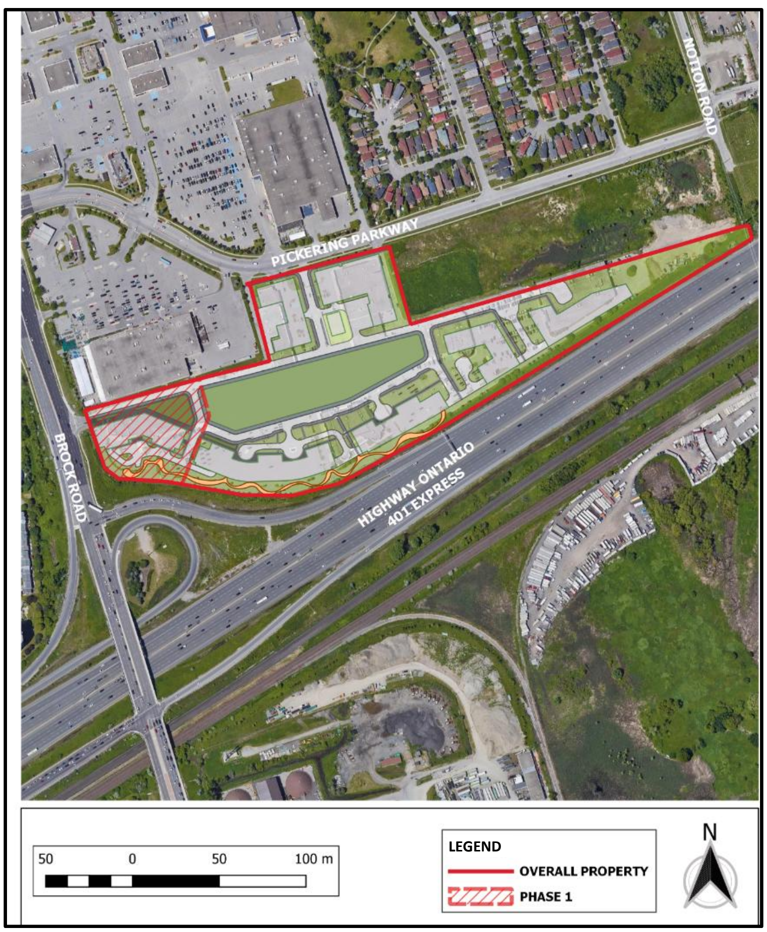


Exhibit 1 Location of the project site

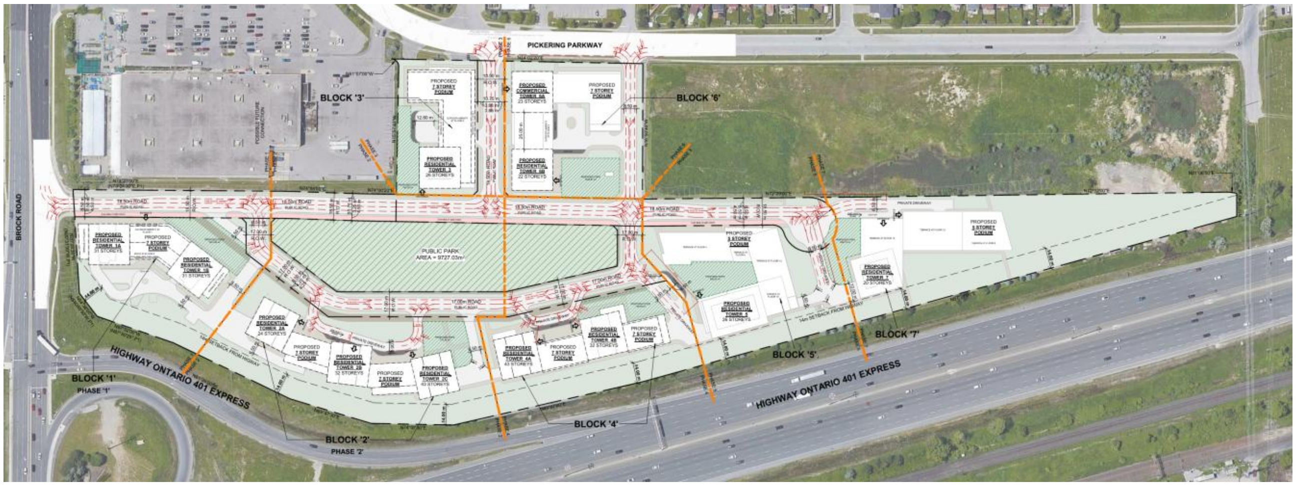


Exhibit 2 Full build out layout and location of Phase 1

2. SCOPE OF WORK

The Odan/Detech Group Inc. was retained by the owners, **Pickering Ridge Lands Inc. & Bayfield Realty Advisors** to propose a servicing scheme(s) for the Redevelopment of 1755 & 1805 Pickering Parkway (Pickering Design Centre). The scope of work in brief involves the following:

a) Gather information on the existing services for the Site and surrounding the Site.

b) Work with or assemble a team of Consultants and Vendors to perform specialized tasks required for the global servicing assessment.

c) Meetings/conversations with consulting team and landowners to coordinate developments.

d) Produce Servicing Schemes that will allow for the development of the intensified site at full build out with focus on the development of Phase 1. The servicing analysis entails a review for sanitary wastewater, water distribution, storm water management and grading.

Currently, the proposed development area is divided into 7 blocks (Block '1' to Block '7'), of which Phase one corresponds to Block '1'. The proposed redevelopment in Phase 1 will consist of a mixed-use development with two towers of 31 storeys. The proposed building will have retail at grade (1,663m2) and 678 apartment dwelling units, 4 levels of underground parking and surface parking. Refer to site plan and site statistics prepared by Turner Fleischer Architects Inc. in Appendix A for additional information related to Phase 1 and the Ultimate Development.

3. SANITARY SERVICING

Existing Sanitary Sewer Infrastructure

As constructed and design drawings obtained from the Region of Durham and the Town of Pickering show that an existing 250 mm diameter sanitary sewer in Pickering Parkway are located as the main sanitary outlet of the subject site.

There are two existing sanitary sewer connections to the site, a 250mm sanitary outlet toward Pickering Parkway at the north of the site and a 150 mm sanitary outlet toward Notion Road at the east of the site.

Refer to Exhibit 3 for the location of the Site and the layout of the existing sanitary sewers in the area.

The majority of sanitary flow from the existing commercial site is conveyed through an existing 250 mm diameter sanitary sewer west to east along Pickering Parkway. Then connected to a 250 mm diameter sanitary sewer at the intersection with Marshcourt Dr, which conveys the sanitary flow to the north. The 250 mm diameter sanitary sewer on Marshcourt Dr then increases to a 375 mm diameter sewer at the Region's easement and the sanitary sewer conveys the collected sanitary flow to a 375 mm diameter sanitary sewer on Notion Road. The 375 mm diameter sanitary sewer on Notion Road is connected to a 750 mm sanitary sewer on Orchard Road that conveys the collected flow to the east. The 750mm pipe is the outlet for the subject site. There is a site located to the south of the Region Easement located on the east side of Notion Road that is currently service via this existing sanitary sewer. The Site is service with a 300mm dia. Sanitary sewer south of the 375mm dia. Sanitary sewer on Notion Rd. and subsequently with a 200mm lateral.

The sanitary analysis will be conducted considering the flow from all sites that presently flow to Orchard Road and the future flow from the redevelopment of 1899 Brock Road and surrounding tributary areas which have been provided by the Region. Refer to Region sanitary maps and correspondence in Appendix B for additional information.

In completing the analysis, the following information will be used or relied upon:

- Drawings from City of Pickering.
- Drawings from The Regional Municipality of Durham.
- Sanitary system Maps from The Regional Municipality of Durham
- Design guidelines for sanitary sewers systems from The Regional Municipality of Durham
- Master Servicing & Stormwater Management Report -1899 Brock Road, SCHAEFFERS Consulting Engineers, May 2021
- Functional Servicing & Stormwater Management Report Residential Townhouse Development - 1856 Notion Road, GHD, Jan 2018

EXISTING SYSTEM REVIEW

Based on review of the existing sanitary sewer sheets in Appendix B, the redeveloped site cannot be routed through the existing sewer system along Pickering Pkwy, Marshcourt Drive, easement between homes to Notion Road to Orchard Drive. Due to limitations of the existing sanitary sewer capacity, it would mean replacing a relatively deep sewer between two existing homes. The recommended and preferred routing would be along Pickering Pkwy to Notion Road to Orchard Drive.

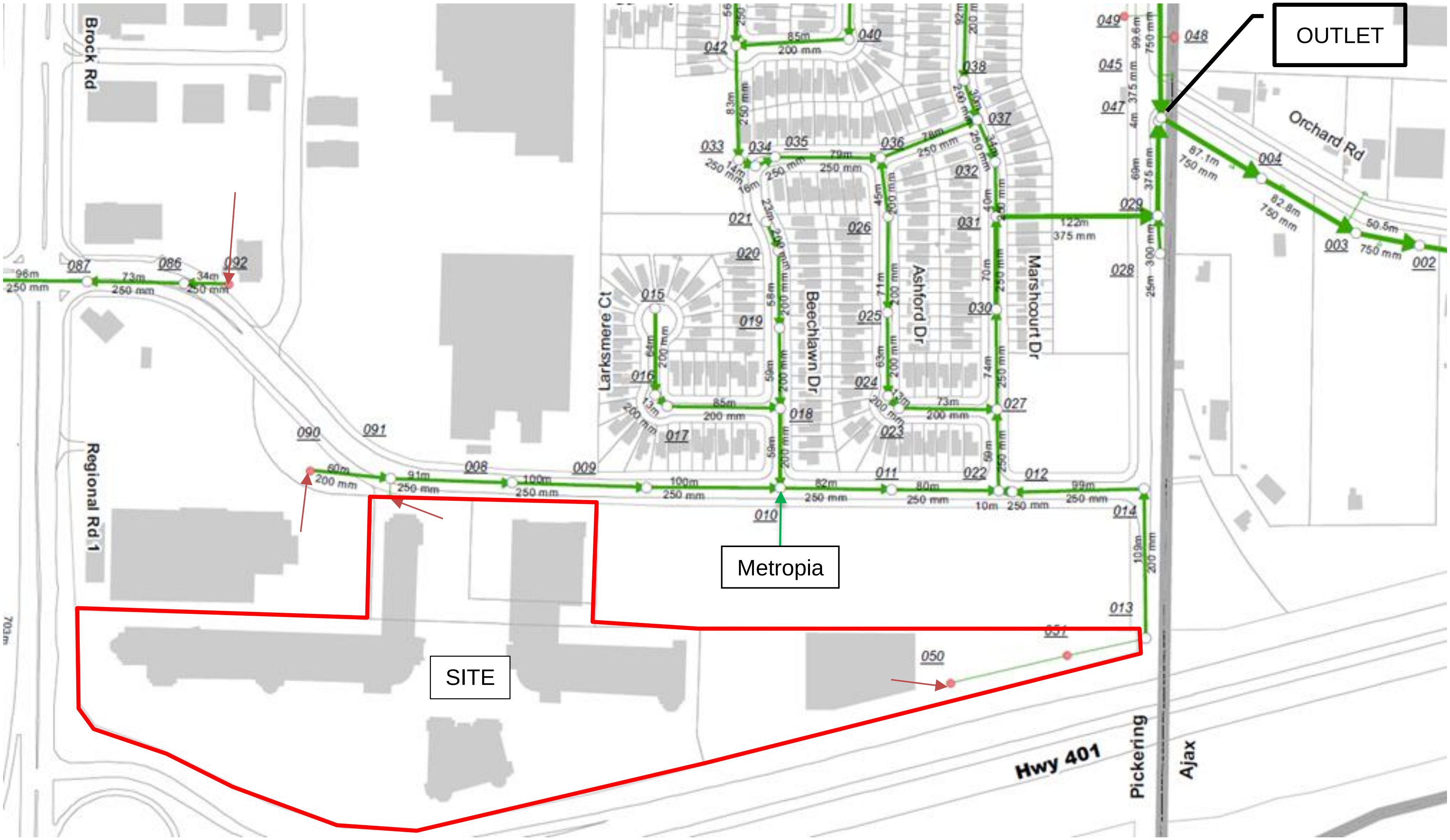


Exhibit 3 Durham Region layout of existing sanitary sewers

REGION OF DURHAM PREFERRED SYSTEM

Discussion with the Region of Durham (Aaron Christie), regarding redevelopment of the subject site and intensification of future development lands, can be summarized as follows:

1) The Region solution for the intensification is to provide a sewage pump station (SP) on the south side of HWY 401. From this SP a large trunk sewer will be extended North under HWY 401 to Notion Road, then continue North on Notion Road. This pump station is outlined within the current Region's Capital Budget and 9-year forecast; however, this will be subject to further study as part of a Class Environmental Assessment. The applicant shall note that the timing for this future project cannot be determined at this time as indicated by the Region.

2) For the early Phases of this development, a new sanitary sewer is proposed along Pickering Parkway to Notion Road. This section of sewer will be sized for full build-out of Brock Precinct service area.

3) The applicant is proposing to construct a sanitary sewer along Notion Road to Orchard Road to utilise the remaining capacity inf the Orchard Road Trunk Sanitary Sewer on an interim bases for the early phases of this proposed development.

4) Any cost sharing for works constructed by the developer will be determined as per the Region's cost shar policy. Generally, the application will be responsible for the minimum size required to service their development along the full length of the constructed sanitary sewer.

5) Sanitary mapping has been provided by the Region which indicates proposed future development lands and the associated tributary areas which will ultimately discharge to the SP on the south side of HWY 401 via Pickering Parkway and the Notion Road trunk sewer. Population densities for these proposed development lands were provided by the Region. Refer to Exhibits 4, 5 & 6 below for the Region's sanitary mapping and related population densities.

Region of Durham Sanitary Maps & Correspondence indicating population densities

Exhibit 4 – Region Map 1 North [1899 Brock Rd & Mixed-use Lands]

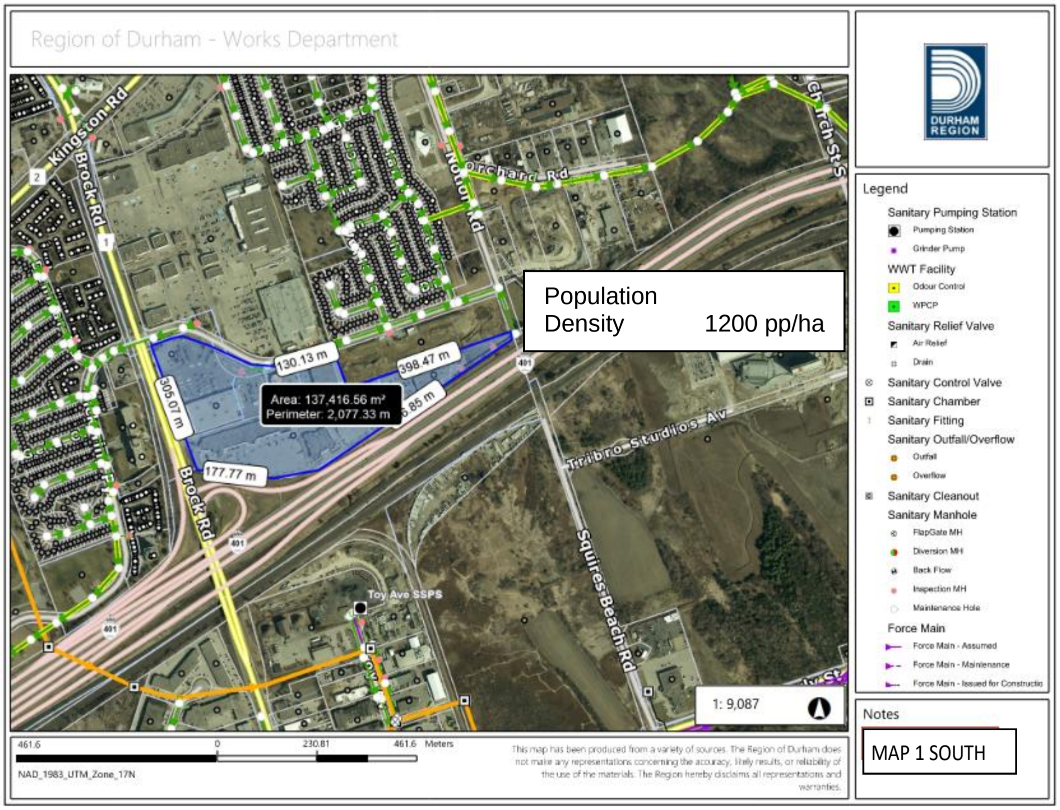


Exhibit 5 – Region Map 1 South [Subject site and 1731/1735 Pickering Pkwy]

Exhibit 6 – Region Map 2 South [Metropia Lands]

6) The Region has allowed for the Phase 1 of 1899 Brock Road to be discharged west ward to Brock Road and will therefore not be included in our Phase 1 downstream sanitary analysis.

7) The Phase 1 for the subject site will be allowed to discharge to Orchard Drive North on Notion Road, for the interim condition. Phase 1 will be serviced via local sanitary sewers along Notion Road from Pickering Parkway to the existing 750mm dia. Sanitary sewer on Orchard Road. Based upon flow monitoring, the remaining capacity within Orchard Road 750mm diameter sanitary sewer is estimated to be 150 l/sec as provided by the Region.

8) The existing Sanitary Service servicing the existing Food Store located on the east side of the development will remain until such time that the Region proceeds with improvements to their Wastewater infrastructure via the addition of wastewater main to pump stations south of the 401 on Notion Rd. As this portion of the site has been included at the intersection of Notion Rd. and Pickering Parkway it's removal will not alter the Ultimate Design. Should the Region improvements proceed in advance of Block 7 (Phase 7) an Interim reconnection of this sewer would be required.

9) In the full build out condition the temporary sewers on Notion Road will be replaced by the Region with a trunk sewer. Thus, all the sewage from the existing and redeveloped sites will flow south in the Notion Road trunk, under HWY 401 to the new Region SP.

10) The Region prefers that the Sewer to Notion Road along Pickering Parkway be installed to accommodate the fully built out sites and the existing sites along the way.

11) The Region will allow a smaller sewer diameter pipe on Notion Road than on Pickering Parkway for the interim condition since the trunk sewer will replace this to flow South under the HWY 401.

12) Sanitary Capacity is assigned upon execution of a development agreement with the Region of Durham.

The Region has also given us the approximate reserve capacity of the Orchard Drive sewer from where we show it on Exhibit 3 eastward. See the following e-mail from Aaron Christie.

Hello Mark,

At this time base your study on the assumption that there could be up to a capacity of 150 l/s available within the 750mm sanitary sewer at Orchard Road. This is based on preliminary input received from the Region of Durham and is subject to change as your application and development of the surrounding lands moves forward.

Based on my interpretation of the mapping, the 600mm watermain on Brock Road has a 300mm dia. tee to the west and then there is a 300mm x 300mm dia. tee and 90 degree bend providing the 300mm dia. watermain to the east across Brock Road to Pickering Parkway.

Thanks,

Aaron Christie, P.Eng. | Manager, Engineering Planning & Studies
Works Department
The Regional Municipality of Durham
Aaron.Christie@durham.ca | 905-668-7711 extension 3608 | durham.ca
My pronouns are he/his

The analysis will proceed to:

- Provide an existing sewer system analysis to show that the Marshcourt Drive route cannot be used.
- Provide a redeveloped site Phase 1 sanitary sewer design sheet (with Required pipe sizes for context only as this sewer would not be sufficient for future developments and full build out)
- Provide a redeveloped site Phase 1 sanitary sewer design sheet (with Proposed pipe sizes)
- Provide a redeveloped sites (subject, 1899 Brock Road and future tributaries) fully developed sanitary sewer design sheet to Notion Road.

Design Criteria

Sanitary flows for the subject site are calculated based on the Regional Municipality of Durham design specifications for sanitary sewers. The summary is as follows:

Residential

- Average flow: 364 L/person/day
- Infiltration: 22.5 m^3 gross ha/day (0.26 l/s/ha/day) – when foundation drains are not connected to the sanitary sewer.
- Peaking Factor:

$$K = 1 + \frac{14}{4 + P^{1/2}}$$

Where K=Harmon Peaking Factor, P = Population in thousands.
K-Maximum= 3.8m, K-Minimum= 1.5

- When the number and type of housing units within a proposed development are known, the calculation of population for the proposed development shall be based on the following:

Type of Housing	**Persons/ha**
Single Family Dwelling,	60
Semi-detached & Duplex	100
Townhouses	125
Apartment(s)	
1. Low density (62 u/ha)	150
2. Medium to low density (86 u/ha)	210
3. Medium density (124 u/ha)	300
4. High Density (274 u/ha)	600
u/ha = units per hectare	

Type of Housing	**Persons/Unit**
Single Family Dwelling, Semi-Detached and Links	3.5
Townhouses/Stacked Townhouses	3.0
Apartment(s)	
5. 1 Bedroom or smaller (Bachelor)	1.5
6. 1 Bedroom and Den	2.5
7. 2 Bedroom	2.5
8. 3 Bedroom	3.5
9. 4 Bedroom or larger	4.5

<u>Commercial</u>

Design Flow: 180 m^3/gross floor area ha/day (2.08 l/s/day) including infiltration and peaking effect.

EXISTING SANITARY SEWER CAPACITY CALCULATION

The capacity of the existing sanitary sewer system located from the subject site to Orchard Rd was evaluated using a sanitary sewer design sheet based on the above parameters. The design sheet for the existing conditions has been completed based on the drainage areas and existing sewer information provided by the Region of Durham and the City of Pickering. Refer to Appendix B for the existing conditions sewer spread sheet and further details. The existing sanitary tributary areas are found in Appendix E.

PROPOSED SANITARY SEWER DESIGN CONSIDERATION

Based on our discussion with the Region of Durham (Aaron Christie), that they (the Region) want the redeveloped flow from 1899 Brock Road and the updated tributary areas, provided by the Region, to flow from their Site east on Pickering Parkway to Notion Road.

Metropia is planning to develop a new townhouse development at 1856 Notion Road known as the Metropia Site. The details are contained within the "Functional Servicing and Stormwater Management Report", by GHD, Jan 2018. The sanitary flow (6.78 L/s) from the development will be routed to the existing manhole (MH-H9-0010) on Pickering Parkway.

Since four existing retail buildings will remain operational within the site for Phase 1 construction. The construction of new sewers will need to be phased to ensure drainage is maintained to the existing buildings.

Table 1 is a summary of the flows generated by the Site during Phase 1.

Table 1 – Proposed population and sanitary peak flow estimate (Phase 1)

Unit Type /Land Use	Number of Units /Gross floor Area	Persons/ Unit	Population	Peaking Factor	Infiltration (L/sec)	Sanitary Flow (L/sec)
North Sanitary Outlet to Pickering Parkway						
Commercial (Ex.)	0.79 ha	-	-	1	-	1.65
Commercial (Prop.)	0.17 ha	-	-	1		0.35
Apartments (Prop.)	678 Units 126- 1 Bedroom 337- 2 Bedroom 207-3 Bedroom 8 -4 Bedroom	1.5 2.5 3.5 4.5	1793	3.62	0.31	27.28
Total	-	-	-	-		29.28
East Sanitary Outlet to Notion Road						
Commercial (Ex.)	0.425 ha	-	0.425 ha	1		0.88
Total	-	-	-	-		0.88

Table 2 – Proposed population and sanitary peak flow estimate (Full Build out)

Unit Type /Land Use	**Number of Units /Gross floor Area**	**Persons/ Unit**	**Population**	**Peaking Factor**	**Infiltration (L/sec)**	**Sanitary Flow (L/sec)**
North Sanitary Outlet to Pickering Parkway						
Commercial (Prop.)	2.67 ha	-	-	1		5.56
Apartments (Prop.)	5298 Units	2.5	13,245	2.83	2.46	158.06
Total						166.09

The total flow to the Pickering Parkway sanitary sewer at full build out of the subject site is 166.09 L/sec.

We will show **3 Scenarios** to evaluate the improvements required to accommodate the redevelopment. The scenarios are as follows:

1. Existing conditions
2. Phase 1 of subject site
3. Full development of subject site and full development of 1899 Brock Road and future tributaries

Find enclosed in **Appendix B**, spread sheets for each scenario. Sanitary tributary plan maps are included in **Appendix E** for reference.

The purpose of **Scenario 1** (existing conditions) is to establish the base rate into MH 17 at Orchard Road. Durham Region has suggested that the excess capacity in the Orchard Road sewer system is approximately 150 L/sec. The reason for the existing condition is to establish the flow into existing MH 17 from the south side. MH 17 is located at the south side intersection of Orchard Road and Notion Road. If the **Scenario 2** flow into the south side of MH 17 is less than **Scenario 1** plus 150 L/sec, then Phase 1 of First Pickering Place can be accommodated.

The purpose for **Scenario 2** is to establish the flow rate to size the pipes from Pickering Parkway to Orchard Road along Notion Road. These pipes along Notion Road are interim for Phase 1 until the Region replaces them with a trunk sewer along Notion Road. Essentially these pipes will be a throw away along Notion Road.

The purpose for **Scenario 3** is to establish the flow rate to size the pipes from 1899 Brock Road along Pickering Parkway to Notion Road. These pipes will be sized to handle the existing flows and the full future build out of the development sites proposed in the Region's sanitary mapping provided and included in Appendix B for reference.

SUMMARY AND RECOMMENDATION

Based on the above review and analysis we offer the following summary and recommendations:

1) Phase 1 of First Pickering Place cannot be accommodated by the existing sanitary sewer system and present routing path. Refer to spread sheet for existing conditions.

2) The present path would require the replacement of a sewer between two existing homes. This is not recommended.

3) The 750 mm sanitary sewer on Orchard Road has sufficient capacity to accommodate Phase 1 of First Pickering Place and the existing uses.

4) We recommend that the owners of First Pickering Place build the sanitary sewer on Pickering Parkway from 1899 Brock Road site to Notion Road to accommodate the full build out of all future development sites and the existing flows. This section of sanitary sewer will be subject to development charges as discussed with the Region of Durham.

5) The sanitary pipe on Notion Road (from Pickering Parkway to Orchard Rd) will be sized to convey existing flows and flows from Phase 1 (First Pickering Place) to the existing Orchard Road sanitary sewer. The Region will allow this interim condition at limited capacity until such time that the Ultimate Trunk Sewer is constructed in the future to convey flows to the South SP. The interim pipe will be downsized from that on Pickering Parkway, the Region will allow this, since it is a temporary measure until the Region replaces it with a trunk sewer on Notion Road.

Table 3 – Phase 1 - Offsite sewer improvements

Sewer location	Upstream MH	Downstream MH	Sewer size, length and slope	Comments
Pickering Parkway	1899 Brock Road	EX MH H8-0091	525mm – 116m @ 1.0%	New pipe
Pickering Parkway	EX MH H8-0091	Prop MH9A	675mm – 49.4m @ 0.45%	Replacement pipe
Pickering Parkway	Prop MH9A	EX MH H9-0018	675mm – 41.8m @ 0.45%	Replacement pipe
Pickering Parkway	EX MH H9-0018	EX MH H9-0019	675mm – 100m @ 0.45%	Replacement pipe
Pickering Parkway	EX MH H9-0019	EX MH H9-0010	675mm – 100m @ 0.45%	Replacement pipe
Pickering Parkway	EX MH H9-0010	EX MH H9-0011	675mm – 83m @ 0.45%	Replacement pipe
Pickering Parkway	EX MH H9-0011	EX MH H9-0022	675mm – 80m @ 0.45%	Replacement pipe
Pickering Parkway	EX MH H9-0022	EX MH H9-0014	675mm – 110m @ 0.45%	Replacement pipe
Pickering Parkway	EX MH H9-0014	Prop MH 13A	450mm – 15m @ 0.22%	Interim Pipe Phase 1
Notion Road	Prop MH 13A	Prop MH 14A	450mm – 100m @ 0.22%	Interim Pipe Phase 1
Notion Road	Prop MH 14A	SAN MH H9-0029	450mm – 102m @ 0.22%	Interim Pipe Phase 1
Notion Road	Prop MH H9-0029	Prop MH H9-0045	450mm – 72m @ 0.22%	Replacement pipe
Notion Road	Prop MH H9-0045	Prop MH 17	450mm – 4m @ 0.23%	Replacement pipe

Note: Notion Road pipes are temporary and will be replaced by the Ultimate Regional Trunk sewer that will be directed South on Notion Road to the downstream SP. At that time the 450mm from Pickering Parkway to Notion Road will be upsized to a 675mm dia. Sewer since at this time the final layout of the Ultimate sewer on Notion Road is unknown.

CONCLUSION

Based on the above findings the sanitary pipe along Pickering Parkway will be replaced and constructed to accommodate the full build out of all future development sites and the existing flows.

The sanitary pipe on Notion Road (from Pickering Parkway to Orchard Rd) will be sized and constructed to temporarily convey existing flows and flows from Phase 1 (First Pickering Place) to the existing Orchard Road sanitary sewer.

Notion Road pipes will be removed when the Region constructs their SP and Trunk Sewer and the lateral to Pickering Parkway will be upsized at that time.

4. WATER SUPPLY AND DISTRIBUTION

EXISTING SYSTEM:

First Pickering Place (FPP) existing water service is fed from a 300 mm Ø City main on Pickering Parkway. The Plaza has a 300mm Ø service main off of Pickering Parkway with a series of hydrants and lateral services inside the Plaza to feed the multiple buildings. Refer to Exhibit 7 for the Regions existing water system.

The purpose of this report is not to evaluate the existing water distribution system, but to evaluate if the existing system can accommodate the proposed intensification. The rest of this section will deal with the intensified site.

REDEVELOPED SITE:

The unit rate and peaking factors of water consumption, minimum pipe size and allowable pressure in line were established from the Durham Region Criteria. Refer to table 4 for the domestic at demand nodes. The fire demand for First Pickering Place is unknown at this stage, since the building designs are not advanced enough. KYPIPE has a unique algorithm to calculate the available fire flows at all hydrants or selected nodes. We will report the available fire flow + maximum day demand at all hydrants and at the block nodes. The demand from Beechlawn Drive and Marshcourt Drive was taken from the homes from the intersection of Beechlawn Drive and Marshcourt Drive to Pickering Parkway. The below table is consistent with the current site though the total Units are less the model still remains valid for the reduce unit counts.

Table 4 – Demand Calculations at Select Nodes.

NODE	DESCRIPTION OF DEVELOPMENT	NUMBER OF UNITS	Shopping (m2)	POPULATION	Average Day (RESIDENTIAL + ICI) (L/sec)	Peak Day (L/sec)	Peak Hour (L/sec)	Assummed Fire flow required (L/sec)	Total Flow required (Fire + max day) (L/sec)
A-100	BLOCK 1	678	1669	1695	7.24	13.75	20.63	190	203.8
B-200	BLOCK 2	1090	1006	2725	11.54	21.92	32.88	190	211.9
C-300	BLOCK 3	446	920	1115	4.75	9.03	13.54	190	199.0
D-400	BLOCK 4	1022	696	2555	10.80	20.53	30.79	190	210.5
E-500	BLOCK 5	617	665	1543	6.54	12.42	18.63	190	202.4
F-600	BLOCK 6	762	21737	1905	9.28	17.64	26.46	190	207.6
G-700	BLOCK 7	659	0	1648	6.94	13.19	19.78	190	203.2
1899 Brock Rd	Mix use		49522	3366	17.05	32.39	48.58	n/a	n/a
Metropia	Residential		0	672	2.83	5.38	8.07	n/a	n/a
Beechlawn Dr	ex residential	104	0	364	1.53	2.91	4.37	n/a	n/a
Marshcourt Dr	ex residential	120	0	420	1.77	3.36	5.04	n/a	n/a
CTC	ex Canadian Tire		7900	0	0.46	0.87	1.30	n/a	n/a
GAS	ex Gas Bar		600	0	0.03	0.07	0.10	n/a	n/a
TOTALS		**5498**	**84715**	**18007**	**80.77**	**153.45**	**230.18**	-	-

PEAK DAY FACTOR	1.9	APARTMENT UNITS (average 2 Bedroom)	2.5 ppu
PEAK HOUR FACTOR	2.85	SINGLE FAMILY HOMES	3.5 ppu
		TOWNHOME UNITS	3.0 ppu
		Shopping DEMAND IS	5000 L/Day/1000m2
AVERAGE DAY (364 L/CAP/DAY)			

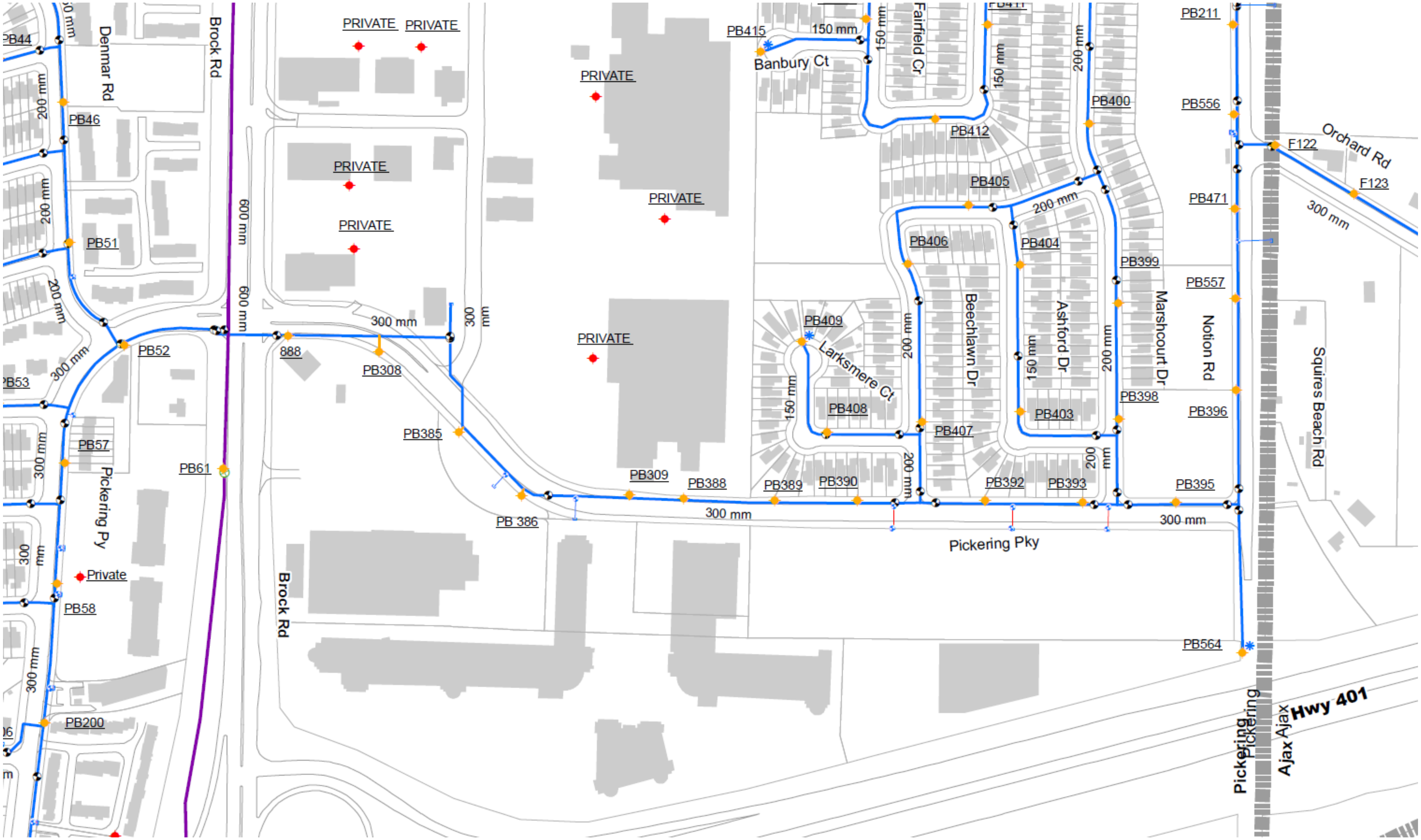


Exhibit 7 Durham Region layout of existing water system

PRESSURE CRITERIA:

The pressures and volumes must be sufficient for Peak hour conditions and under fire conditions as established by the Ontario Building Code. The MOE minimal residual pressure under fire conditions is 140 kPa (20.3 psi). According to the Durham Region, Design Criteria for Water mains the allowable pressures are as per table 5.

Table 5 – Allowable pressures

SCENERIO	DURHAM REGION CRITERIA Allowable Pressure (kPa)		MOE Allowable Pressure (kPa)	
	min	max	max	max
Min. Hour	275	700	275	700
Average Day	275	700	275	700
Max Day	275	700	275	700
Max Hour	275	700	275	700
Maximum Day + Fire	140	700	140	700

Note, the subject development will have development blocks that will require booster pumps to deliver domestic supply to upper levels. Where the pressure is greater than 550 kPa at the development blocks, a pressure reducing valve shall be installed to limit the maximum static pressure to not more than 550 kPa as per The Ontario Building Code.

The maximum allowable velocity in the pipes will be limited to 5.0 m/sec for all flow conditions.

First Pickering Place is located Durham Region pressure district A1. The supply points were derived by data from hydrant flow tests. Refer to Appendix C.

The existing First Pickering Place water supply will be kept intact with minor variation. The existing ring main will be moved out of the future ROW and future park and town easements for the sewers.

SYSTEM MODEL:

The hydraulic model KYPIPE was used to analyze the water distribution system for the redeveloped Site. See below the KYPIPE model with explanation. The Elevation information within the redeveloped Site is provided by the Odan/Detech Group Inc. Pressure district A1 will continue to provide service to the mall. In addition, a new 300 mm main will be introduced to service the new development contemplated within First Pickering Place. See model in Exhibit 8.

1.0 Friction Factors

The water mains have been designed using a Hazen-Williams C-factor as follows:

Pipe size (mm)	Hazen-Williams C-factor
150 or smaller	100
200 and 300	110
>300 to 600	120
Over 600	130

2.0 Pipe Diameters

The diameter of the proposed water main is 300 mm which is at and above the minimum diameter for water mains that provide fire protection and that are required under the Regions criteria. The 300mm diameter water mains are also capable of providing a flushing velocity of 0.8 m/s during cleaning and flushing procedures.

3.0 Pipe System Design and Minimum Pipe Cover

Fire hydrants have been located at dead end location to provide a means for adequate flushing. The minimum cover over the water main shall be 1.8 m which provides for adequate frost protection. The junction elevations were taken from finished grade elevations at the centerline of the road.

4.0 Service Pipes

Suitable Water services will be provided for domestic and for fire at the detailed design stage. The material provided will be acceptable material under Part 7, Division B of the Building Code (O. Reg. 350/13) and AWWA Standards. Water services were not modelled.

5.0 Source node

Water supply for the site is based on the hydrant flow tests as given in Appendix C. The KYPIPE model will use a Variable pressure supply node at the pipe elevation. The input data is summarized in table 6.

Table 6 – Supply Pressure/flow table

Supply (Notion Road) VP-1		Supply (Brock Road) VP-2		Supply (Brock Road) VP-3	
Pressure (kPa)	Flow (L/s)	Pressure (kPa)	Flow (L/s)	Pressure (kPa)	Flow (L/s)
621	0	552	0	552	0
517	121.9	510	109.0	510	109.0
310	219.7	310	269.0	310	269.0
138	285.3	138	358.8	138	358.8

The above data will be entered into KYPIPE variable pressure supply node. KYPIPE will take the above data points. The raw data is shown in Appendix C. In addition, the Region in the e-mail below has stated that the VP-2 and VP-3 will have similar supply curves.

From: Aaron Christie <Aaron.Christie@durham.ca>
Sent: Tuesday, April 2, 2024 1:31 PM
To: Mark Harris - Odan Detech Group <mark@odandetech.com>
Cc: Peter Castellan <Peter.Castellan@Durham.ca>; Amy Nugent <Amy.Nugent@durham.ca>
Subject: RE: Bayfield - Pickering Parkway - URGENT - Water Flows & Pressures -

Hello Mark,

The Top Water Level for Pressure Zone 1 in Pickering is 144.5m.

The existing 300m dia. watermain on Pickering Parkway is also connected to the 600mm dia. watermain on Brock Road.

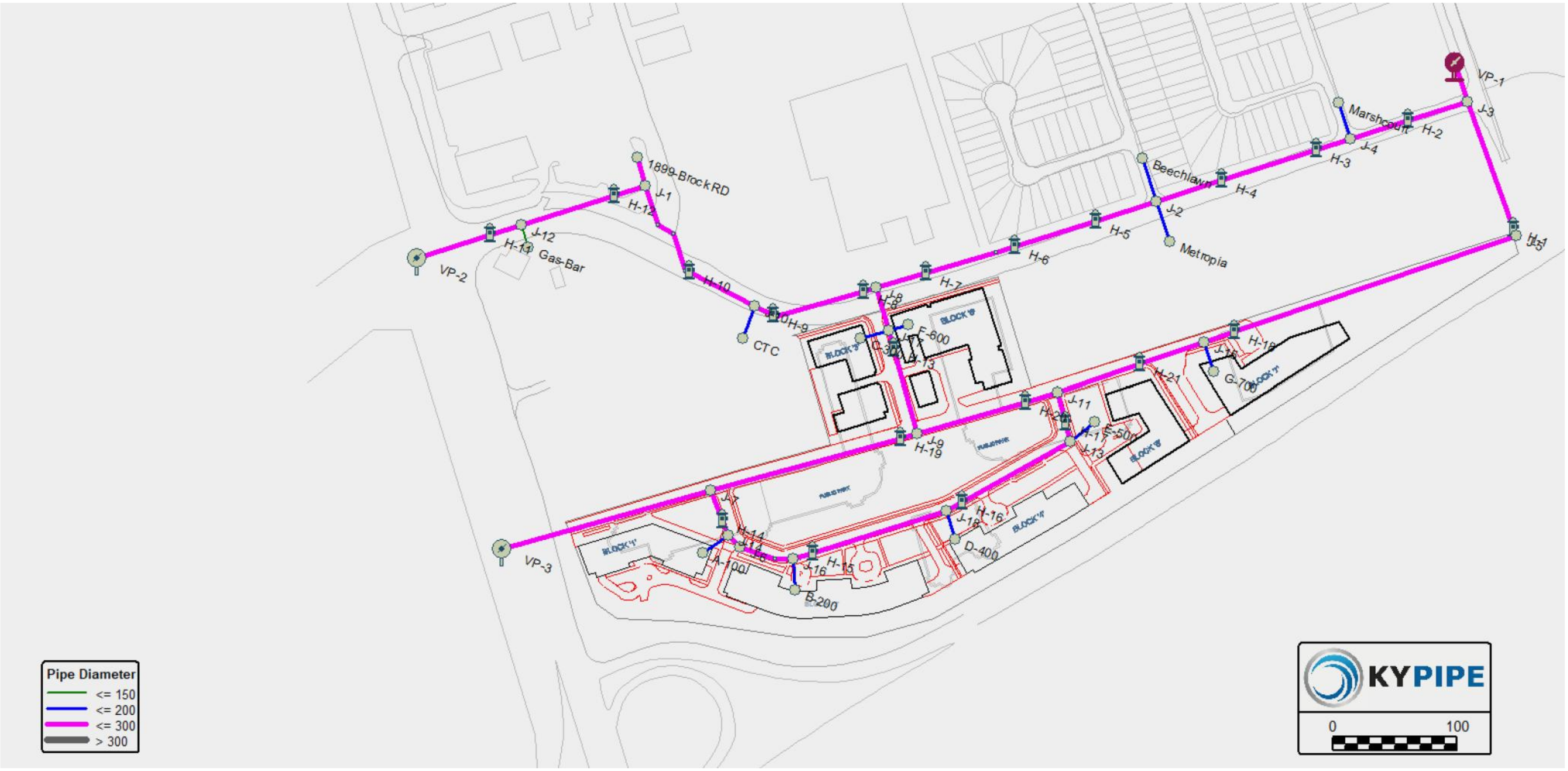
Flow tests along the existing 300mm dia. main between the Walmart driveway and Brock Road should give you similar results as the proposed 300mm dia.

If not, the instructions for ordering a fire flow test can be found here: https://www.durham.ca/en/living-here/fire-flow-test.aspxhydrant

Aaron Christie, P.Eng.| Senior Project Engineer
Water & Wastewater Infrastructure Planning |Works Department
The Regional Municipality of Durham | **Celebrating 50 years!**
Aaron.Christie@durham.ca | 905-668-4113 extension 3608 | durham.ca
My pronouns are he/him. | durham.ca/50years

Exhibit 8 shows the KYPIPE node numbering and hydrant numbering used in the output tables.

Exhibit 8 - KYPIPE MODEL**:** node number and hydrants numbers



INPUT INFORMATION

U N I T S   S P E C I F I E D

```
    FLOWRATE ............ = liters/second
    HEAD (HGL) .......... = meters
    PRESSURE ............ = kpa
```

P I P E L I N E   D A T A

STATUS CODE: XX -CLOSED PIPE   CV -CHECK VALVE

P I P E N A M E	NODE NAMES #1	#2	LENGTH (m)	DIAMETER (mm)	ROUGHNESS COEFF.	MINOR LOSS COEFF.
P-1	VP-2	J-12	88.18	300.00	110.0000	0.00
P-2	VP-1	J-3	32.86	300.00	110.0000	0.00
P-3	J-1	J-10	140.53	300.00	110.0000	0.00
P-4	J-2	J-4	163.07	300.00	110.0000	0.00
P-5	J-4	J-3	99.18	300.00	110.0000	0.00
P-6	J-2	Metropia	33.89	200.00	110.0000	0.00
P-7	J-2	Beechlawn	36.42	200.00	110.0000	0.00
P-8	J-4	Marshcourt	30.17	200.00	110.0000	0.00
P-9	J-8	J-2	234.78	300.00	110.0000	0.00
P-10	J-8	J-17	36.36	250.00	110.0000	0.00
P-11	J-10	J-8	101.91	300.00	110.0000	0.00
P-12	J-10	CTC	27.23	200.00	110.0000	0.00
P-13	J-12	J-1	103.93	300.00	110.0000	0.00
P-14	J-12	Gas-Bar	18.48	100.00	100.0000	0.00
P-15	J-1	1899-Brock	23.64	300.00	110.0000	0.00
P-16	J-3	J-5	113.84	300.00	110.0000	0.00
P-17	J-9	J-7	171.04	300.00	110.0000	0.00
P-18	J-7	J-14	38.49	300.00	110.0000	0.00
P-19	J-6	J-16	44.21	300.00	110.0000	0.00
P-20	J-13	J-11	40.11	300.00	110.0000	0.00
P-21	J-11	J-9	117.10	300.00	110.0000	0.00
P-22	J-11	J-15	124.65	300.00	110.0000	0.00
P-23	J-14	J-6	13.13	300.00	110.0000	0.00
P-24	J-14	A-100	24.49	200.00	110.0000	0.00
P-25	J-16	J-18	128.74	300.00	110.0000	0.00
P-26	J-16	B-200	24.98	200.00	110.0000	0.00
P-27	J-18	J-13	113.76	300.00	110.0000	0.00
P-28	J-18	D-400	23.46	200.00	110.0000	0.00
P-29	J-13	E-500	24.93	200.00	110.0000	0.00
P-30	J-15	J-5	263.98	300.00	110.0000	0.00
P-31	J-15	G-700	24.80	200.00	110.0000	0.00
P-32	J-17	J-9	84.95	250.00	110.0000	0.00
P-33	J-17	C-300	23.33	200.00	110.0000	0.00
P-34	J-17	F-600	16.55	200.00	110.0000	0.00
P-35	VP-3	J-7	174.28	300.00	110.0000	0.00

P U M P/L O S S   E L E M E N T   D A T A

THERE IS A DEVICE AT NODE   VP-1 DESCRIBED BY THE FOLLOWING DATA: (ID= 1)

HEAD (m)	FLOWRATE (l/s)	EFFICIENCY (%)
63.32	0.00	75.00 (Default)
52.72	122.00	75.00 (Default)
31.61	220.00	75.00 (Default)
14.07	285.00	75.00 (Default)

THERE IS A DEVICE AT NODE   VP-2 DESCRIBED BY THE FOLLOWING DATA: (ID= 2)

HEAD (m)	FLOWRATE (l/s)	EFFICIENCY (%)
56.29	0.00	75.00 (Default)
52.00	109.00	75.00 (Default)
31.61	269.00	75.00 (Default)

14.07	359.00	75.00 (Default)

THERE IS A DEVICE AT NODE VP-3 DESCRIBED BY THE FOLLOWING DATA: (ID= 3)

HEAD (m)	FLOWRATE (l/s)	EFFICIENCY (%)
56.29	0.00	75.00 (Default)
52.00	109.00	75.00 (Default)
31.61	269.00	75.00 (Default)
14.07	359.00	75.00 (Default)

N O D E D A T A

NODE NAME	NODE TITLE	EXTERNAL DEMAND (l/s)	JUNCTION ELEVATION (m)	EXTERNAL GRADE (m)
Beechlawn		1.53	84.40	
CTC		0.46	85.90	
Marshcourt		1.77	83.30	
Metropia		2.83	84.40	
1899-Brock		17.05	86.40	
A-100		7.24	89.50	
B-200		11.54	88.90	
C-300		4.75	85.80	
D-400		10.80	87.50	
E-500		6.54	86.50	
F-600		9.28	85.80	
G-700		6.94	86.20	
Gas-Bar		0.03	88.80	
J-1		0.00	86.30	
J-2		0.00	84.10	
J-3		0.00	83.40	
J-4		0.00	83.30	
J-5		0.00	87.00	
J-6		0.00	89.20	
J-7		0.00	88.50	
J-8		0.00	85.50	
J-9		0.00	86.00	
J-10		0.00	85.90	
J-11		0.00	86.00	
J-12		0.00	88.70	
J-13		0.00	86.20	
J-14		0.00	89.00	
J-15		0.00	85.80	
J-16		0.00	88.50	
J-17		0.00	85.70	
J-18		0.00	87.10	
VP-1		----	83.40	83.40
VP-2		----	88.50	88.50
VP-3		----	88.50	88.50

SYSTEM ANALYSIS AND RESULTS

SIMULATION RESULTS: Fully connected

Table 7- Average Day

Table 8- Maximum Day

Table 9- Peak Hour

Table 10- Maximum Day + Fire Flow

Table 7 – Average Day: At Select Nodes

N O D E   R E S U L T S

NODE NAME	NODE TITLE	EXTERNAL DEMAND lps	HYDRAULIC GRADE m	NODE ELEVATION m	PRESSURE HEAD m	NODE PRESSURE kPa
Beechlawn		1.53	144.71	84.40	60.31	591.41
CTC		0.46	144.62	85.90	58.72	575.89
Marshcourt		1.77	144.79	83.30	61.49	603.04
Metropia		2.83	144.71	84.40	60.31	591.39
1899-Brock		17.05	144.62	86.40	58.22	570.91
A-100		7.24	144.57	89.50	55.07	540.04
B-200		11.54	144.54	88.90	55.64	545.65
C-300		4.75	144.59	85.80	58.79	576.57
D-400		10.80	144.55	87.50	57.05	559.43
E-500		6.54	144.57	86.50	58.07	569.52
F-600		9.28	144.59	85.80	58.79	576.49
G-700		6.94	144.62	86.20	58.42	572.94
Gas-Bar		0.03	144.66	88.80	55.86	547.75
J-1		0.00	144.62	86.30	58.32	571.97
J-2		0.00	144.71	84.10	60.61	594.36
J-3		0.00	144.86	83.40	61.46	602.67
J-4		0.00	144.79	83.30	61.49	603.06
J-5		0.00	144.79	87.00	57.79	566.72
J-6		0.00	144.58	89.20	55.38	543.07
J-7		0.00	144.60	88.50	56.10	550.12
J-8		0.00	144.63	85.50	59.13	579.82
J-9		0.00	144.60	86.00	58.60	574.64
J-10		0.00	144.62	85.90	58.72	575.90
J-11		0.00	144.60	86.00	58.60	574.64
J-12		0.00	144.66	88.70	55.96	548.74
J-13		0.00	144.58	86.20	58.38	572.56
J-14		0.00	144.58	89.00	55.58	545.06
J-15		0.00	144.63	85.80	58.83	576.97
J-16		0.00	144.57	88.50	56.07	549.86
J-17		0.00	144.60	85.70	58.90	577.60
J-18		0.00	144.57	87.10	57.47	563.59
VP-1		----	144.93	83.40	61.53	603.37
VP-2		----	144.68	88.50	56.18	550.95
VP-3		----	144.66	88.50	56.16	550.73

Table 8 – Maximum Day: At Select Nodes

N O D E   R E S U L T S

NODE NAME	NODE TITLE	EXTERNAL DEMAND lps	HYDRAULIC GRADE m	NODE ELEVATION m	PRESSURE HEAD m	NODE PRESSURE kPa
Beechlawn		2.91(1.90)	143.64	84.40	59.24	580.90
CTC		0.87(1.90)	143.57	85.90	57.67	565.59
Marshcourt		3.36(1.90)	143.75	83.30	60.45	592.84
Metropia		5.38(1.90)	143.63	84.40	59.23	580.84
1899-Brock		32.40(1.90)	143.58	86.40	57.18	560.72
A-100		13.76(1.90)	143.39	89.50	53.89	528.45
B-200		21.93(1.90)	143.29	88.90	54.39	533.38
C-300		9.02(1.90)	143.46	85.80	57.66	565.42
D-400		20.52(1.90)	143.30	87.50	55.80	547.23
E-500		12.43(1.90)	143.38	86.50	56.88	557.83
F-600		17.63(1.90)	143.43	85.80	57.63	565.18
G-700		13.19(1.90)	143.45	86.20	57.25	561.47
Gas-Bar		0.06(1.90)	143.82	88.80	55.02	539.52
J-1		0.00	143.60	86.30	57.30	561.96
J-2		0.00	143.64	84.10	59.54	583.88
J-3		0.00	143.85	83.40	60.45	592.77
J-4		0.00	143.76	83.30	60.46	592.87
J-5		0.00	143.74	87.00	56.74	556.41
J-6		0.00	143.42	89.20	54.22	531.68
J-7		0.00	143.49	88.50	54.99	539.24
J-8		0.00	143.56	85.50	58.06	569.34
J-9		0.00	143.47	86.00	57.47	563.60
J-10		0.00	143.57	85.90	57.67	565.60
J-11		0.00	143.45	86.00	57.45	563.40
J-12		0.00	143.82	88.70	55.12	540.50
J-13		0.00	143.42	86.20	57.22	561.09
J-14		0.00	143.43	89.00	54.43	533.74
J-15		0.00	143.49	85.80	57.69	565.75
J-16		0.00	143.38	88.50	54.88	538.23
J-17		0.00	143.47	85.70	57.77	566.57
J-18		0.00	143.38	87.10	56.28	551.93
VP-1		----	143.95	83.40	60.55	593.84
VP-2		----	144.00	88.50	55.50	544.23
VP-3		----	143.89	88.50	55.39	543.19

Table 9 – Peak Hour: At Select Nodes

N O D E   R E S U L T S

NODE NAME	NODE TITLE	EXTERNAL DEMAND lps	HYDRAULIC GRADE m	NODE ELEVATION m	PRESSURE HEAD m	NODE PRESSURE kPa
Beechlawn		4.36(2.85)	141.72	84.40	57.32	562.10
CTC		1.31(2.85)	141.69	85.90	55.79	547.07
Marshcourt		5.04(2.85)	141.89	83.30	58.59	574.60
Metropia		8.07(2.85)	141.70	84.40	57.30	561.97
1899-Brock		48.59(2.85)	141.73	86.40	55.33	542.61
A-100		20.63(2.85)	141.28	89.50	51.78	507.82
B-200		32.89(2.85)	141.07	88.90	52.17	511.60
C-300		13.54(2.85)	141.42	85.80	55.62	545.41
D-400		30.78(2.85)	141.09	87.50	53.59	525.55
E-500		18.64(2.85)	141.25	86.50	54.75	536.92
F-600		26.45(2.85)	141.36	85.80	55.56	544.89
G-700		19.78(2.85)	141.37	86.20	55.17	540.99
Gas-Bar		0.09(2.85)	142.31	88.80	53.51	524.74
J-1		0.00	141.78	86.30	55.48	544.12
J-2		0.00	141.72	84.10	57.62	565.11
J-3		0.00	142.04	83.40	58.64	575.07
J-4		0.00	141.90	83.30	58.60	574.68
J-5		0.00	141.86	87.00	54.86	538.00
J-6		0.00	141.34	89.20	52.14	511.36
J-7		0.00	141.50	88.50	53.00	519.80
J-8		0.00	141.62	85.50	56.12	550.37
J-9		0.00	141.45	86.00	55.45	543.75
J-10		0.00	141.69	85.90	55.79	547.08
J-11		0.00	141.39	86.00	55.39	543.18
J-12		0.00	142.31	88.70	53.61	525.72
J-13		0.00	141.32	86.20	55.12	540.55
J-14		0.00	141.37	89.00	52.37	513.54
J-15		0.00	141.44	85.80	55.64	545.68
J-16		0.00	141.27	88.50	52.77	517.49
J-17		0.00	141.45	85.70	55.75	546.74
J-18		0.00	141.26	87.10	54.16	531.11
VP-1		----	142.22	83.40	58.82	576.81
VP-2		----	142.75	88.50	54.25	532.04
VP-3		----	142.49	88.50	53.99	529.48

Table 10 – Maximum Day + Fire: At Hydrants

```
================================================================================

          Fireflow/Hydrant Report:
         __________________________

Scenario: No Title
Global Demand Factor for this Scenario: 1.900

Specified Minimum Pressure(kPa):  140.0
Minimum Static Pressure(kPa)   :  140.0

 Flow-1: Flowrate to maintain the specified
         pressure at (hydrant) node
 Node-2: Node that has a lower pressure than
         specified value at Flow-1
 Flow-2: Flowrate to maintain the specified
         pressure at Node-2

 Hose Constant =    0.00
```

Hydrant Node	Hydrant Constant	Elevation	Static Pressure	Flow-1 lps	Flow-2 lps	Node-2 lps	Flow Capacity	NFPA Color
H-11	0.0	88.7	541.0	548.9			548.9	ORANGE
H-10	0.0	86.1	563.8	555.8			555.8	ORANGE
H-4	0.0	83.8	587.2	543.7			543.7	ORANGE
H-3	0.0	83.5	590.7	558.5			558.5	ORANGE
H-2	0.0	83.3	593.3	587.2			587.2	ORANGE
H-7	0.0	85.3	571.4	574.7			574.7	ORANGE
H-6	0.0	84.8	576.6	550.6			550.6	ORANGE
H-5	0.0	84.4	580.8	541.0			541.0	ORANGE
H-9	0.0	85.9	565.6	569.5			569.5	ORANGE
H-8	0.0	85.6	568.4	593.2			593.2	ORANGE
H-12	0.0	86.4	561.5	550.3			550.3	ORANGE
H-1	0.0	87.0	556.5	534.0			534.0	ORANGE
H-19	0.0	86.1	562.6	578.4			578.4	ORANGE
H-14	0.0	88.9	534.9	537.7	533.4	A-100	533.4	ORANGE
H-17	0.0	86.2	561.2	545.4			545.4	ORANGE
H-20	0.0	86.0	563.4	562.9			562.9	ORANGE
H-21	0.0	85.9	564.6	536.7			536.7	ORANGE
H-15	0.0	88.5	538.2	501.1			501.1	ORANGE
H-16	0.0	87.1	552.0	502.7			502.7	ORANGE
H-18	0.0	85.8	566.0	521.5			521.5	ORANGE
H-13	0.0	85.7	566.6	561.8			561.8	ORANGE

At development blocks:

```
================================================================================
FireFlow/Hydrant Report
          Fireflow/Hydrant Report:
         __________________________

Scenario: No Title
Global Demand Factor for this Scenario: 1.900

Specified Minimum Pressure(kPa):  140.0
Minimum Static Pressure(kPa)   :  140.0

 Flow-1: Flowrate to maintain the specified
         pressure at (hydrant) node
 Node-2: Node that has a lower pressure than
         specified value at Flow-1
 Flow-2: Flowrate to maintain the specified
         pressure at Node-2

 Hose Constant =    0.00

   Hydrant    Hydrant  Elevation  Demand    Static       Flow-1    Flow-2    Node-2    Flow
NFPA
    Node     Constant             lps      Pressure      lps       lps      lps      Capacity
Color

_________________________________________________________________________________________________
____
     A-100       0.0      89.5      13.8     528.4       374.4
     B-200       0.0      88.9      21.9     533.4       370.3
     C-300       0.0      85.8       9.0     565.4       401.3
     D-400       0.0      87.5      20.5     547.2       377.0
     E-500       0.0      86.5      12.4     557.8       383.2
     F-600       0.0      85.8      17.6     565.2       440.9
     G-700       0.0      86.2      13.2     561.5       381.0
```

The following table 11 provides a summary of the flows:

Table 11 – Summary of required flows and available flows at Select Nodes.

NODE	**Description of development**	**Total Flow Assumed required (L/sec) (Fire + max day)**	**Available Flow from KYPIPE (L/sec) (Fire + max day)**
A-100	BLOCK 1	203.8	374
B-200	BLOCK 2	211.9	370
C-300	BLOCK 3	199.0	401
D-400	BLOCK 4	210.5	377
E-500	BLOCK 5	202.4	383
F-600	BLOCK 6	207.6	440
G-700	BLOCK 7	203.2	381

Note, fire flows to building blocks are based on a 200mm service pipe to the building. A bigger pipe would allow larger flows.

DISCUSSION OF RESULTS:

- The pipe sizes shown are required for the fire flows and to ensure velocities are below 5.0 m/sec for fire flows.

- First Pickering Place will require new mains and hydrants. Some will be relocated to suit the development.

- Pressures for normal operation (average, maximum and peak day) have been achieved in the fully connected scenario.

- The pipe sizes chosen are adequate.

- Where pressures are greater than 80 psi (550 kPa) buildings will require pressure reducing valves prior to meter connection. Hydrant tests prior to permit stage will confirm this.

- Looping to Notion Road or Brock Road is required to provide redundancy to the development since many buildings are taller than 84 m. The OBC requires a second connection to a public system when buildings are greater than 84 m.

DESIGN CRITERIA: The following data was used in the design of the system:

1.0 ***Transient Pressures***

The proposed water main will be designed to withstand pressures up to 1034 kPa (150 psi) which is sufficient to withstand the maximum operating pressure of 150 psi, plus any transient pressure it may be subjected to.

The pipes and joints have also been designed to withstand the maximum operating pressure plus the surge pressure that would be created by stopping a water column moving at 0.6 m/s.

The transient pressure surge in a PVC Class 150 DR 18 pipe is 0.6m/s water column is 35 psi.

2.0 ***Pipe Strength***

The proposed water main pipe material is PVC Class 150 DR18 conforming to CSA B137.3 and AWWA C900.

Loading calculations for pipe strength are based on the internal pressures of the system. The pipe stiffness values for the specified pipe class are relatively high; therefore, deflection from static and/or live loads is not a critical design factor.

For water main pipe material consisting of PVC Class 150 DR18 conforming to CSA B137.3 and AWWA C900, the maximum internal pressure is 150 psi with a long-term FS of 2.5 and 4 for short term surge pressures.

3.0 Fire Hydrants

All hydrants shall be 3-way hydrants and shall be spaced as detailed on the engineering drawings. All hydrants shall

- Be in accordance with the approved water main materials list
- Be dry-barrel type in accordance with AWWA C502: Dry-Barrel Fire Hydrants
- Be 3-way, two nozzles which are 180˚ to each other and parallel to the street and a 100mm pumper "STORZ" connection facing the street
- Open as per Region Standard.
- Have a 25mm top operating nut size
- Be painted as per Region Standard
- Be controlled by a secondary valve close-coupled to the hydrant
- Have a hydrant lead of 150mm from the water main to the hydrant
- Be installed plumb and in accordance with the Region Standard drawing which provides adequate thrust blocking to prevent movement caused by thrust forces.

The water table is not expected to rise above the hydrant drain ports.

4.0 Valves

The water main will be designed such that there are a minimum of 3 gate valves at each T-intersection and 4 at cross intersections.

All gate valves shall be in accordance with the Regions approved water main materials list which conforms to AWWA standards.

5.0. Air Release and Vacuum Release Valves

Not applicable

6.0 Valve, Meter and Blow-off Chambers

Not applicable

7.0 Separation Distances from Contamination Sources

The water main has been located such that there is a minimum of 2.5m horizontal separation from the nearest sewer. Crossing of sewers as per MOE criteria.

8.0 Restraints

Mechanical joint restraints are to be installed on bell and spigot joints for all water mains constructed in fill material and at all tees, horizontal bends, vertical bends, hydrants, end of mains and valves. Concrete thrust blocks are not permitted unless expressly approved by the Region. All mechanical restraint systems shall be installed with cathodic protection.

9.0 ***Additional Design Considerations***

The water mains within the proposed site shall be installed in accordance with the current Region specifications and requirements.

If there is a crossing of the water main and a sewer, the water main shall cross above the sewer with sufficient vertical separation to allow for proper bedding and structural support of the water main, (0.5m minimum).

In cases where there is a conflict with the elevation of the sewer and the water main such that the water main cannot cross above the sewer, the water main has been designed to cross below the sewer subject to the following conditions.

a) There shall be a minimum vertical separation of 0.5m between the bottom of the sewer pipe and the top of the water main,
b) The water main shall be lowered below the sewer using vertical thrust blocks and restraining joints,
c) The length of the water main pipe shall be centered at the point of crossing so that the joints are equidistant and as far as possible from the sewer, and
d) The sewer shall be adequately supported to prevent joint deflection and settling.

DEVELOPMENT PHASING:

The Site will be developed in phases. Refer to Appendix A. The above analysis assumes a completed development of the First Pickering Place. The waterman around the Mall is looped and will remain that way.

The Developer has indicated that the new development will proceed with Block 1 first. The following is the watermain staging.

Phase	Block	Install
Phase 1	Block 1	New 300mm to Pickering Parkway and new 300 mm redundant line from Pickering Parkway with valve on Pickering Parkway main line. Refer to Figure S-1b for details
Phase 2	Block 2	New 300mm to Pickering Parkway and new 300 mm redundant line from Pickering Parkway with valve on Pickering Parkway main line. Refer to Figure S-1b for details
Phase 3 to 7	Block 3 to 7	As per final layout. 300mm to Pickering Parkway looped and redundant new 300mm line from Notion Road. Refer to Figure S-1a for details.

Refer to Figure S-1b in Appendix E for the proposed water service system for Phase 1 and 2. Refer to Figure S-1a in Appendix E for the proposed water service system for Phase 3 to 7. Based on the recent KYPIPE model results, the system can deliver the above noted service.

5. STORMWATER MANAGEMENT & FOUNDATION WATERPROOFING

Design Criteria

Stormwater management for the proposed development will follow the stormwater management criteria set out by the City of Pickering, Toronto and Region Conservation Authority and the Ontario Ministry of the Environment, Conservation and Parks.

A summary of the stormwater management criteria applicable to the site are as follows:

Quantity Control:

The City of Pickering requires quantity control of Blocks 1 to 7 to a post development allowable flow based on a 5 year Design Storm to a runoff coefficient of C=0.50 during this event. All storms up to and including the 100 Year Design storm must be controlled to this criterion.

Quality Control:

Quality control measures are to be designed to provide Enhanced Protection - long term average removal of 80% of Total Suspended Solids (TSS) on an annual loading basis from all runoff leaving the proposed development site based on the post-development level of imperviousness.

This can be achieved via filtration many methods and Low Impact Development Techniques (LID). To ensure that 80% TSS removal is achieved the use of a Jellyfish Filtration Oil Grit Separator (JFOGS) or similar approved equivalent would accomplish this.

Water Balance:

Retention of the runoff from up to a 5mm storm event on site for reuse, evaporation or infiltration.

- Rain Harvesting
- Green Roofs
- Downspout Disconnection
- Soakaway Pits, Infiltration Trenches (Galleries) and Chambers
- Bioretention Facilities
- Vegetated Filter Strips
- Permeable Pavers
- Enhanced Grass Swales
- Dry Swales
- Perforated Pipe Systems

These techniques help to promote water quality and quantity and water reuse as it relates to stormwater management techniques. At the Stie Plan development stage these techniques will be reviewed in detailed to determine the ideal strategy for each development Block.

Existing Storm Servicing and Drainage Patterns

As-constructed and design plans and profiles drawings obtained from the Region of Durham and the City of Pickering show that the following storm sewers are located within and around the site.

Refer to Exhibit 9 for the existing storm sewer system in and around the Site.

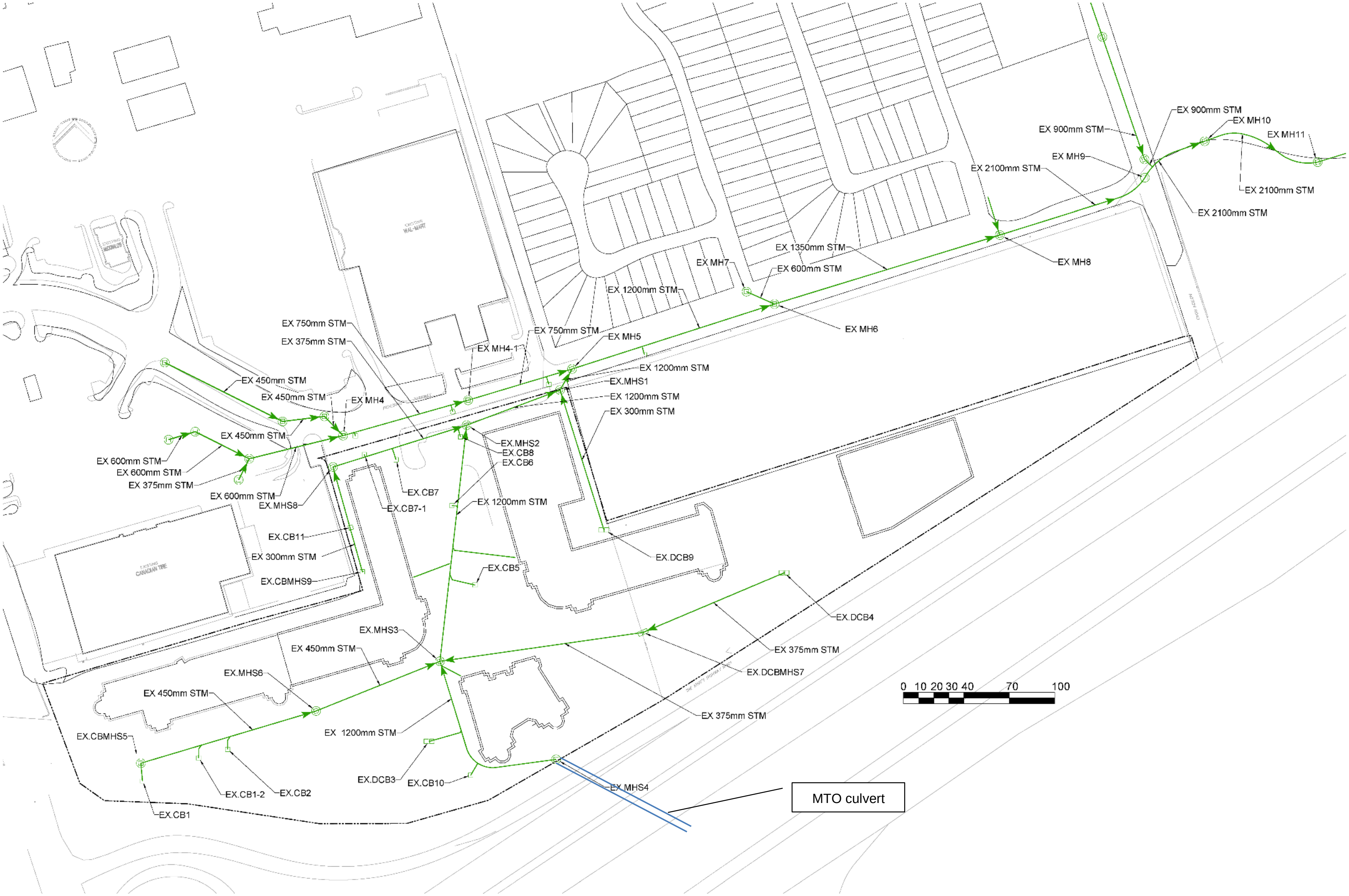


Exhibit 9 - City layout of existing Storm sewers and Site sewers

Exhibit 10 - City layout of existing Storm sewers and Site sewers

The subject site is located within the Duffin's Creek watershed. Refer to Exhibits 9 and 10 for the existing storm drainage infrastructure for the subject site and surrounding area. Note Exhibit 10 is from the AECOM Class EA environmental report for the Notion Road overpass of Hwy 401.

The drainage from the subject site can be summarized as follow:

1. MTO box culvert discharges flow from Hwy # 401 to a short ditch on the south side of the subject site. The flow is captured by an inlet structure attached to an existing 1200mmø storm sewer system which is routed north to Pickering Parkway where it discharges to a 1200mmø existing storm on Pickering Parkway. The pipe continues east on Pickering parkway, changes pipe sizes as shown on Exhibit 10, crosses Notion Road, continues east and discharges via a head wall to a drainage channel which empties into Duffin's Creek.

2. The subject site drains via a series of catch basins and sewers which connect to the 1200mmø storm from the 401 to Pickering Parkway as described in 1 above. Ultimately discharging to the same location.

3. The overland flow from the site is conveyed via 2 locations, from the south through the lands onto the Pickering Parkway and ultimately conveyed via Pipes and existing channel, east of the Notion Road, to the Duffin's Creek and via overflow to the east within the MTO drainage ditch.

4. Currently, there is no stormwater quantity, quality control measures implemented within the existing site.

A pre-development tributary plan has been prepared based on a drainage pattern analysis of the site's digital terrain model created from existing topographic survey and information obtained from the Region, City and MTO records. The pre-development storm tributary plan is included in Appendix E.

5.1 PREDEVELOPMENT ALLOWABLE FLOW ASSESSMENT

Pre-development/Allowable Flow Rates

The post-development flows from the site will be limited to the 5-year design storm event at an allowable rate based on a runoff coefficient of C=0.50 up to the 100-year design storm event. Please note that the actual runoff coefficient for the existing site condition is much higher than C=0.5. The flows were calculated using both rational method. The City of Pickering's Intensity Duration Frequency (IDF) curve values were used for rational method calculation.

The allowable flows for the site are presented in Table 13.

Table 12 – Summary Table of Allowable Flows

Block #	Area	$Q_{5yr\ Pre}$	Volume $Q_{100\ Post}$	Volume Factor $Q_{100\ Post\ (x1.5)}$
Block 1	0.88	0.105	200	300
Block 2	1.29	0.144	302	453
Block 3	0.51	0.055	121	182
Block 4	0.89	0.104	204	306
Block 5	0.92	0.135	186	279
Block 6	0.87	0.121	182	273
Block 7	1.02	0.118	235	352

The Parkland and City Right of Ways are not included in the above as they will remain uncontrolled and have been included within the storm sewer design sheets.

The post-development flows from the site will be limited to the pre-development flows for the 5-year design storm event. The pre-development flows were calculated based on pre-development tributary areas with runoff coefficient of 0.5. Please note that the actual runoff coefficient for the existing site condition is much higher than 0.5. The flows were calculated using the rational method. The City of Pickering's Intensity Duration Frequency (IDF) curve values were used for rational method calculation.

5.2 POST DEVELPOMENT

1. The SWM for the redeveloped First Pickering Place will establish/analyse the following:
2. Flows to the Pickering Parkway storm sewer based on the criteria established above.
3. Establish SWM criteria for the redeveloped First Pickering Place in order to limit the flows.
4. Evaluate the flows entering the down-stream sewer system.
5. Evaluate the water quality requirements.
6. Evaluate the water balance for the Site.
7. Make recommendations as to the implementation of the SWM.
8. Evaluate the staging of construction.

City of Pickering Storm water management criteria for this development has been outlined as follows:

Stormwater Management Criteria that must be included in the FSSR are as follows:

- Control of post-development peak flow rates from the 100 year design storm to 5 Year Design Storm Event at a runoff coefficient equal to C=0.50 for development Blocks.
- A maximum runoff coefficient of 0.5 should be used to represent pre-development conditions for Blocks 1 to 7.
- Follow Stormwater Management Design Guidelines, prepared by City of Pickering. Runoff Conveyance will be as follows, the minor system is to be designed to accommodate the 5-year storm, while the major overland system is to be designed for the 100-year storm event. Where there is no suitable overland flow route, the minor system must convey the 100-year storm after on site attenuation.

The following Table establishes the allowable flows from each of the proposed Blocks based on a runoff coefficient of C=0.50 for the 5 year design storm event and provides for the required storage volumes of each block. In order to establish required storage volumes a conservative approach was taken at this stage using a runoff coefficient of C=0.90 for post development.

In general a C=0.85 is used for apartment type developments. It is therefore likely that the runoff coefficient will be reduced further from C=0.90 through implementation of various Low Impact Design Techniques and Water Reuse at the time of Detailed Design.

Table 13 – Summary Table of SWMM Quantity Pre Development Allowable Flows and Storage

Block #	Area	Q5yr Pre	Volume Q100 Post	Volume Factor Q100 Post (x1.5)
Block 1	0.88	0.105	200	300
Block 2	1.29	0.144	302	453
Block 3	0.51	0.055	121	182
Block 4	0.89	0.104	204	306
Block 5	0.92	0.135	186	279
Block 6	0.87	0.121	182	273
Block 7	1.02	0.118	235	352

Refer to Appendix D for detailed calculations related to storage volumes and orifice sizes based on the Rationale Method related to the above Table values.

The Tank Size and related storage techniques including locations will be finalized for each development at the detailed design stage during Site Plan approval based on the finalized build form.

SUMMMARY OF SWM Quantity Control Features:

Refer to table 16 for the SWM used for quantity control on the redeveloped Site.

Table 14 – Summary Table of SWMM Quantity Features for Redeveloped Site

BLOCK OR DESCRIPTION AND FLOW AREA TO TANK (ha)	SWMM FEATURE DESCRIPTION & FOOTPRINT (m2)	VOLUME REQUIRED 100-year flow (m3) max of 4 hr Chicago or AES With 1.5x Safety	ORIFICE CONTROL C=0.80	ORIFICE max head (m)	Maximum 100-year flow (L/sec)
BLOCK 1 (0.88 ha)	1-Storage Tank (206)	* TANK 1 – 314	175 mm	1.52	105
BLOCK 2 (1.29 ha)	1-Storage Tank (289)	*TANK 1 – 482	200 mm	1.67	144
BLOCK 3 (0.50 ha)	1-Storage Tank (125)	*TANK 1 – 196	125 mm	1.59	55
BLOCK 4 (0.89 ha)	1-Storage Tank (206)	*TANK 1 – 308	175 mm	1.49	104
BLOCK 5 (0.92 ha)	1-Storage Tank (206)	*TANK 1 – 304	200 mm	1.48	135
BLOCK 6 (0.87 ha)	1-Storage Tank (248)	*TANK 1 – 295	200 mm	1.19	121
BLOCK 7 (1.01 ha)	1-Storage Tank (330)	*TANK 1 – 371	200 mm	1.13	118

All Maximum Volumes created by 4-hour Chicago storm.

Max volumes calculated using the modified rational method and City of Pickering IDF parameters.

*Note – Tank Sizes have been provided with a safety factor of 1.5x and will be adjusted during the Site Plan approval stage based on detailed design. The safety factor has been applied to account for maximizing tank volumes should the system require pumping in order to minimize the footprint of the tank within the proposed building and underground parking.

Table 17 summarizes the allowable flows for each Block.

Table 15 – Target Release rates from development Blocks to Pickering Parkway sewer

Block #	**Area (ha)**	**Allowable Release Rate** (m^3/s) 5 year Storm	**Post-development Flows** (m^3/s) 100 Year Storm
Block 1	0.88	0.130	0.105
Block 2	1.29	0.190	0.144
Block 3	0.51	0.075	0.055
Block 4	0.89	0.131	0.104
Block 5	0.92	0.136	0.135
Block 6	0.87	0.128	0.121
Block 7	1.02	0.151	0.118
Total Site (Excluding Park & Private Roads)	6.38	0.941	0.782

Rational method uses C = 0.5 for 5 year event, Tc = 10 min

As per City criteria for; 100-year storm - Ca = 1.25

FOUNDATION WATERPROOFING STRATEGY

Dewatering discharge during construction and long term will be as follows:

At the Pre-consultation for 1755 & 1805 Pickering Parkway the City of Pickering made the following statement:

Please note that the City will not accept discharge of foundation drainage to the storm system due to the potential for adverse impacts.

Please note that Region of Durham will not accept discharge of foundation drainage to the sanitary sewers. This statement is part of their sewer bylaw.

Based on the above we recommend the Architect, Structural Engineer, Geotechnical Engineer and Mechanical Engineer devise a waterproofing system with the shoring and foundation design.

Based on the above we have not incorporated any allowance for foundation drainage in the SWM for the site.

6. WATER QUALITY

Based on the type of developments proposed for Block 1 to 7 and the limitations within these developments it is anticipated that the use of a Jellyfish Filtration OGS will be necessary to achieve 80% TSS removal as there is limited opportunity to implement LID Techniques where the majority above grade features are located over an underground parking structure.

During the Site Plan approval stage and upon such time that the build form is finalized the use of LID techniques will be reviewed to determine if water quality can also be achieved via a train treatment approach in which a standard OGS unit without filtration would be provided with a TSS removal efficiency of 50% with the remaining 30% being achieved through train treatment using LID techniques if determined to be achievable.

As the exact type of development is not finalized a standard approach has been applied to all sites at this time to be conservative and use of Jellyfish Filtration system is currently proposed to achieve water quality.

Adjustment to the strategy and OGS Jellyfish sizing will be finalized during the Site Plan Approval development stage. The provided sizing of the proposed OGS Jellyfish Units is provided for reference only at this time to show that water quality can be achieved.

The use of a Jellyfish Filtration OGS and the use of a Train Treatment approach where achieved during detailed design. During the Site Plan approval stage the above assumptions will be adjusted and OGS unit sizes modified to meet the exact parameters of the final Site Plan properties.

7. WATER BALANCE

The City Pickering and TRCA will require that the first 5mm of any storm be captured and reused or returned to the environment. Water balance can be addressed by various methods including, infiltration by use of soak-away pit or permeable pavers, irrigation, vegetative cover and water reuse for toilet flushing of commercial uses. It is recommended that a separate cistern be installed for storm water reuse adjacent to the SWM tank with an overflow baffle to the SWM tank for when the site has reached its reuse potential.

LIDS:

- Imbrium Filterra Bioretention System
- Silva Cells
- Soak away pits
- Bio swales
- others

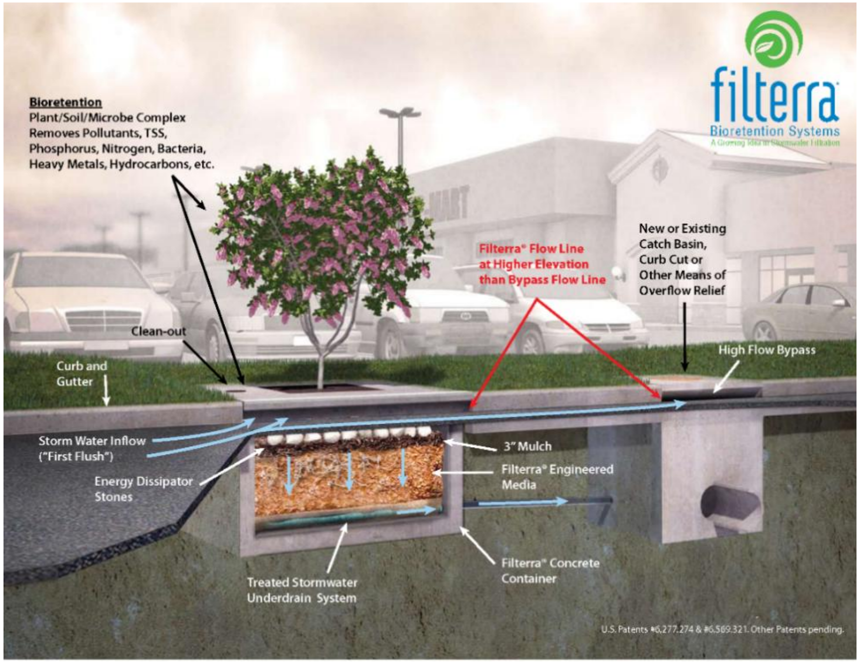
Infiltration gallery footprints are to be designed considering in-situ percolation rates.

A drain-down time of 48 hours should be applied to any infiltration gallery calculations.

Other Options:

1. Imbrium Filterra Bio retention System

This is an appropriate method for water quality treatment in a train treatment environment. Storm water runoff enters the Filterra system through a curb-inlet opening and flows through a specially designed filter media mixture contained in a landscaped modular container. The following photos show the installed Filterra unit and a section through the unit.



2. Silva Cells

The Silva Cell is a modular suspended pavement system that uses soil volumes to support large tree growth and provide powerful on-site storm water management through absorption, evapotranspiration, and interception. The system is typically installed under pavement applications and can be configured in several different ways:

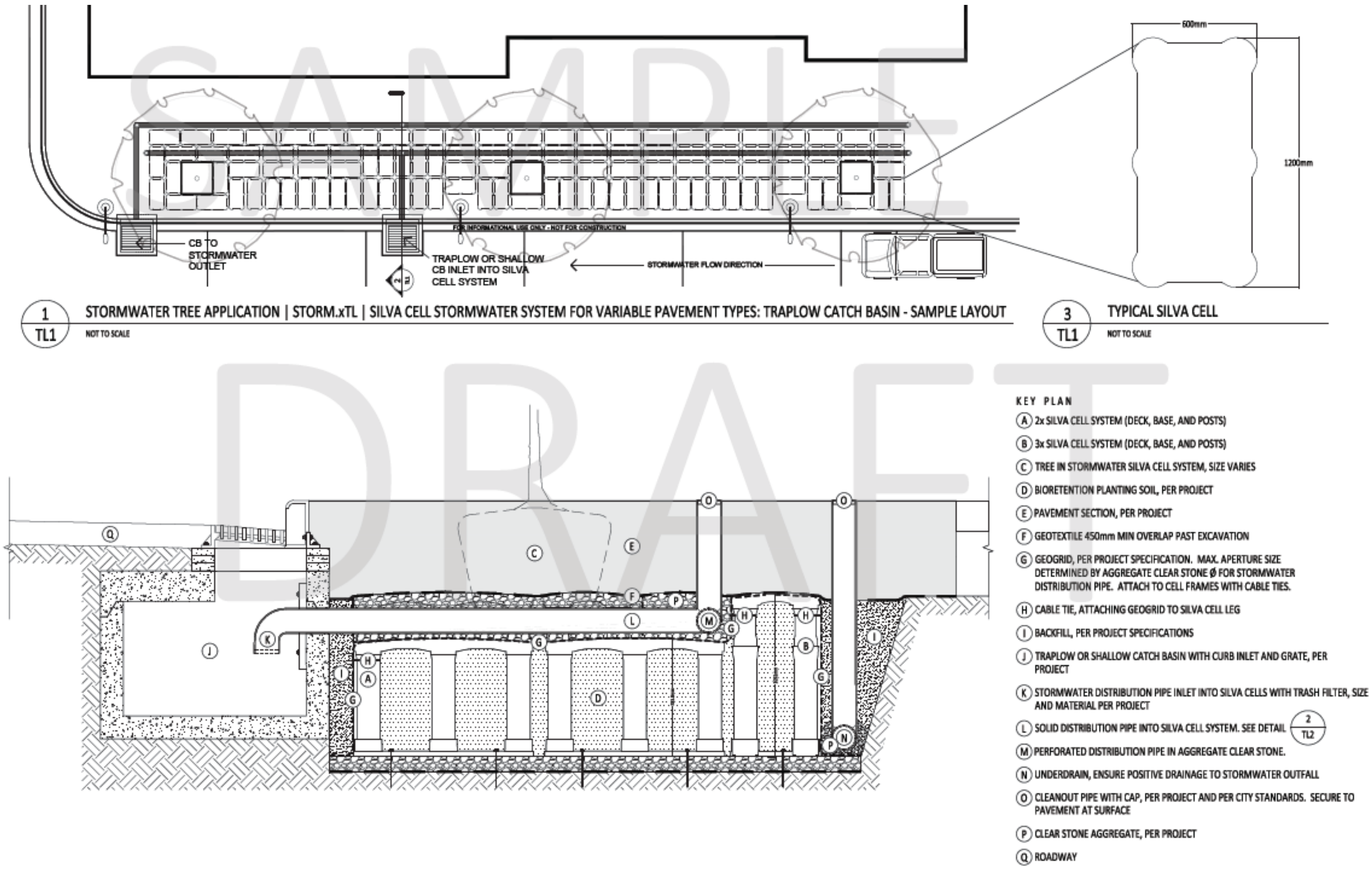
Streetscapes
Adjacent to or under sidewalks
Between buildings and streets.

Parking Areas
Under parking stalls adjacent
to medians or islands.

Public Spaces
Under plazas, promenades, courtyards, or other public spaces at office buildings, museums, schools, and transit centers

The Region of York is using Silva Cells on the widening and reconstruction of Yonge Street.

The following detail is a typical Silva Cell application.



3. Bio swales

We have reviewed the possibility of bio swales. The use of boulevard bio swales was dismissed due to lack of room. The opportunity to deploy bios swales in the blocks is a possibility but based on our past experience not practical. The Park area could have soak away pits rather than bio swales.

The individual quality control and water balance methods can be determined at the site plan stage.

Water Balance Targets

The primary objective of the Water Balance Targets/Criteria is to capture and manage annual rainfall on the development site itself to preserve the pre-development hydrology (or “water balance”, which typically consists of three components: runoff, infiltration, and evapotranspiration) through a combination of infiltration, evapotranspiration, landscaping, rainwater reuse and/or other low impact development practices.

The City of Pickering Stormwater Management Design Guidelines’ target for water balance is to provide runoff reduction from the site through infiltration, evapotranspiration and reuse of a minimum of 5mm of rainfall depth across all impervious surfaces.

The site area and 5mm rainfall depth will be used to calculate the water balance target. The water balance targets for each block are as follows.

Table 16 – Summary Table of Water Balance Targets 5mm Retention

BLOCK OR DESCRIPTION	AREA (m2)	Water Balance Volume (m3) Area x 5mm	Initial Abstraction Volume (m) Area x 1.5mm	Stormwater Re-use / Retention Required (m3)
BLOCK 1	8,800	44.0	-13.2	**30.8**
BLOCK 2	12,900	64.5	-19.4	**45.1**
BLOCK 3	5,000	25.0	-7.5	**17.5**
BLOCK 4	8,900	44.5	-13.4	**31.1**
BLOCK 5	9,200	46.0	-13.8	**32.2**
BLOCK 6	8,700	43.5	-13.0	**30.5**
BLOCK 7	10,100	50.5	-15.2	**35.3**

*Initial abstraction based on 1.5mm depth for Urban Residential

The above water balance requirements will be adjusted and reviewed at the Site Plan approval stage based on the finalized site plan and surface features.

8. SITE SERVICING PHASING CONSIDERATIONS

In order to maintain the operation of the existing Mall during various Phases it is recommended that the during Phase 1 an Interim Municipal and Regional Servicing Easement is provided from the Private development in Phase 1 through the existing Mall Lands. This easement would be in favour of the Region and Municipality during an interim condition until the future Phases are developed and future Right-of-Way is constructed.

Providing a Municipal and Regional Servicing Easement during Phase 1 allows for the proposed Phase 1 to proceed while allowing for connection of the existing Mall to the new Municipal and Regional Easement during this Phase.

The proposed conceptual Phase 1 Servicing Schematic is provided in Appendix E - Figure PH1- Phase 1 Site Servicing and Easement Plan.

In general, the following is proposed for allowing Phase 1 Servicing to proceed:

- Construct sanitary storm and water servicing within Phase 1 through Municipal and Regional Servicing Easement.
- Maintain Existing Mall servicing within Mall lands and reconnect to Interim Municipal and Regional Servicing Easement.
- Relocate existing Storm sewer within Mall to align with future ROW and maintain existing storm sewer located on north property line during Phase 1 and until such time that Phase 6 proceeds.
- Subsequent Phasing will be reviewed at such time that the Phases proceed and will generally follow similar Phasing as identified during Phase 1 in which existing Mall services will be reconnect and adjusted to connect to the Municipal and Regional Servicing Easement. Refer to Figure PH1 Phase 1 Siet Servicing and Easement Plan for general layout and notes related to Future Phasing.

In general the addition of a Municipal and Regional Servicing Easement during Phase 1 will allow for Private Mall services and Phase 1 Private Development to connect to a Municipal and Regional Servicing Easement.

As the site develops to future Phases connection to the Municipal and Regional Servicing Easement can continue in future Phases and be adjusted as required to maintain existing Mall function.

Detailed Phasing Plans will be provided at the Detailed Design stage for each Phase of the development as they are submitted.

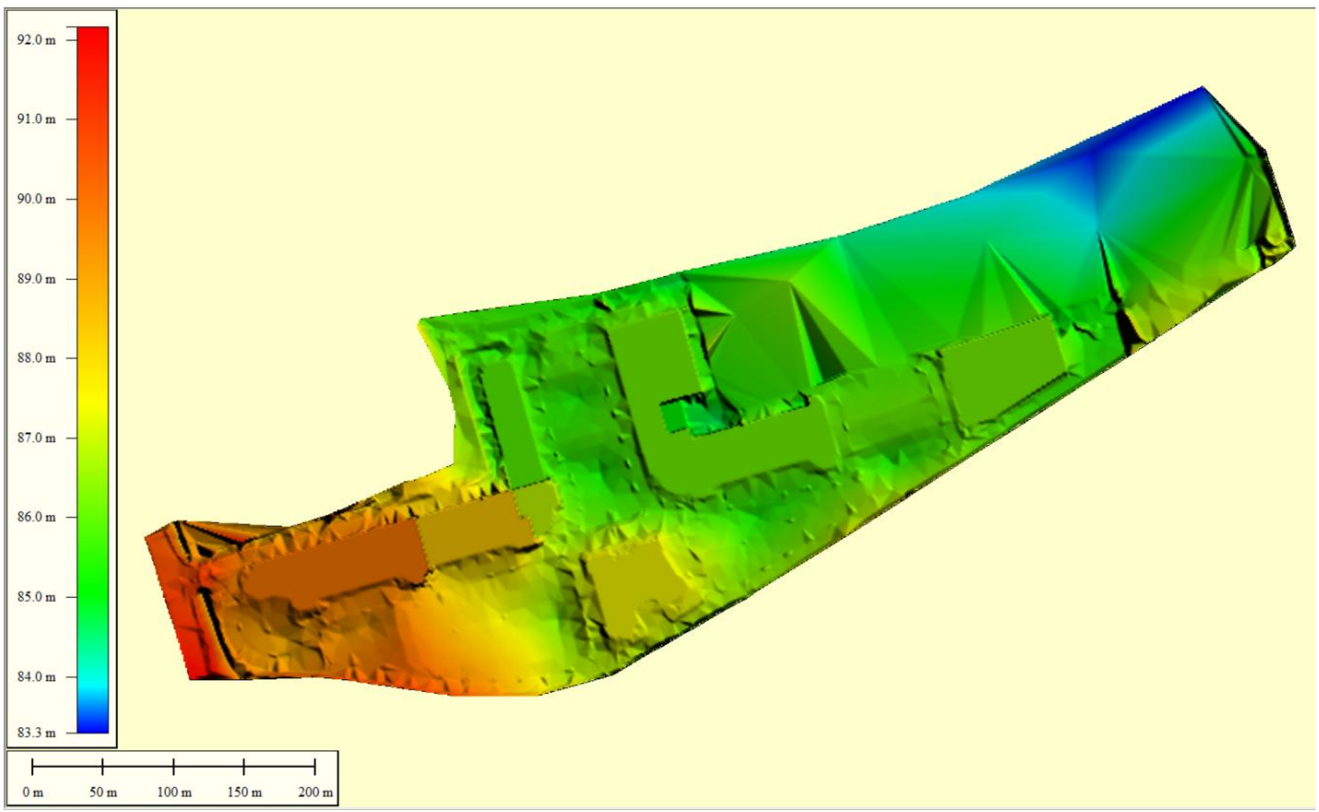
9. GRADING CONSIDERATIONS

The existing topography of the site generally slopes from west to northeast towards the low point of the site located on the east side of the Site. Under the new development and existing adjacent developments there will be several grading constraints for this development to match. The constraints are the existing commercial buildings, Metropia new residential, Pickering Parkway and Notion Road right of ways.

Refer to Exhibit 11 for the color-coded existing topography for the Site.

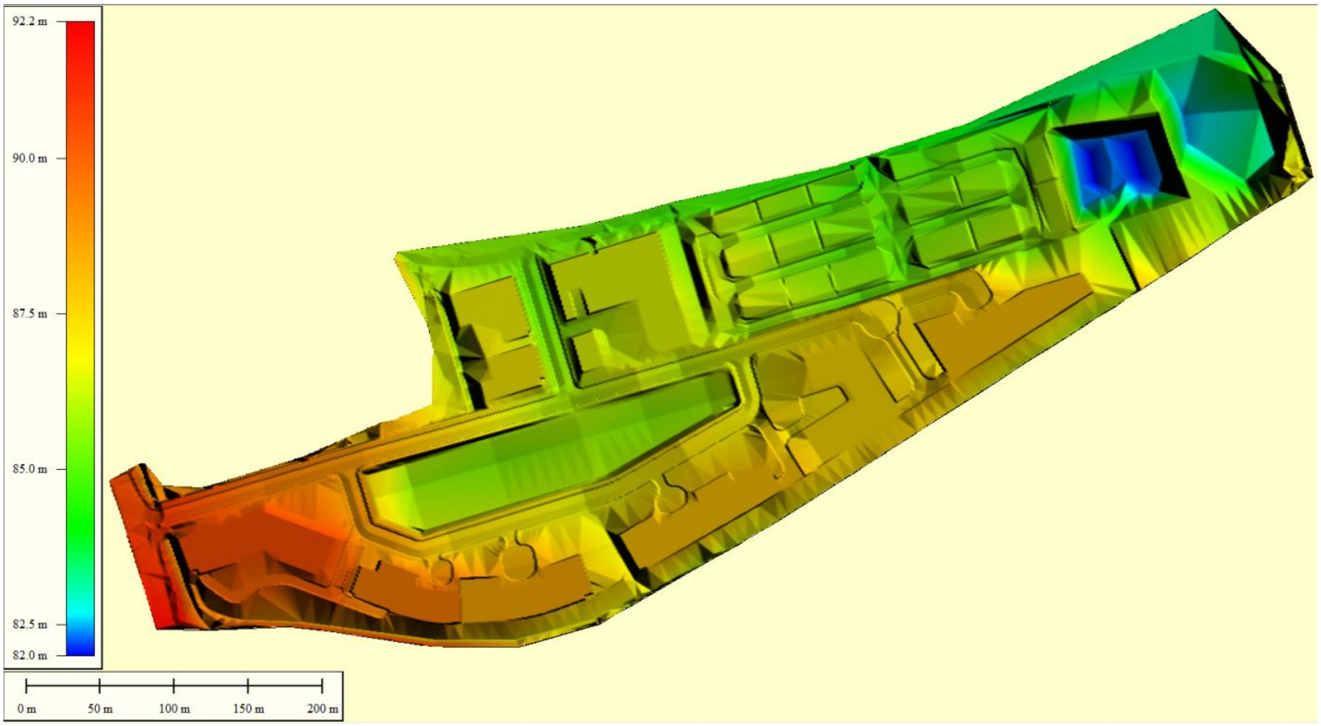
Refer to the concept grading plan in Appendix E for details of the proposed grades for the fully developed site. The proposed grading of the redeveloped site will follow the general lay of the land as shown in Exhibit 11.

Exhibit 11 – Colour coded existing Site topography



Refer to Exhibit 11 for the colour coded proposed Site topography.

Exhibit 12 – Colour coded Proposed Site topography



Refer to the Conceptual Grading Plan included in Appendix E for additional detail.

Grading for each parcel will be detailed at the Site Plan stage of development.

10. EROSION AND SEDIMENT CONTROL

Erosion and sediment controls for the site will be implemented according to the Golden Horseshoe Area Conservation Authorities' Erosion and Sediment Control Guidelines for Urban Construction. A detailed erosion control plan will be prepared upon final design and each Site Plan Stage.

11. SOILS REPORT AND HYDROGEOLOGY:

A preliminary Geotechnical investigation has been completed for the site. The purpose of the study is to characterize hydrogeological conditions and determine permitting requirements for the proposed development at the First Pickering Place. The study was completed by Terraprobe dated May 27, 2021 for Pickering Ridge Lands Inc. & Bayfield Realty Advisors.

Native clayey silt glacial till, underlying dense to very dense matrix of sandy silt to silty sand till is the typical soil underlying the site. The soils have some infiltration capacity. The water table underneath varies from 4 to 6 m below grade. Based on the grading it may be possible to provide infiltration galleries. The water table should be monitored further in order to get a wide range of potential water table levels. Monitoring will provide better confidence in the potential maximum ground water levels.

12. RECOMMENDATIONS:

1) We recommend that the owners of First Pickering Place build the sanitary sewer on Pickering Parkway from 1899 Brock Road site to Notion Road to accommodate the full build out of all future development sites and the existing flows. This section of sanitary sewer will be subject to development charges as discussed with the Region of Durham.

2) The sanitary pipe on Notion Road (from Pickering Parkway to Orchard Rd) will be sized to convey existing flows and flows from Phase 1 (First Pickering Place) to the existing Orchard Road sanitary sewer. The Region will allow this interim condition at limited capacity until such time that the Ultimate Trunk Sewer is constructed in the future to convey flows to the South SP. The interim pipe will be downsized from that on Pickering Parkway, the Region will allow this, since it is a temporary measure until the Region replaces it with a trunk sewer on Notion Road.

3) We recommend looping the watermain to Notion Road, Pickering Parkway and Brock Road to provide redundancy to the development since many buildings are taller than 84 m. The OBC requires a second connection to a public system when buildings are greater than 84 m.

13. CONCLUSIONS

The findings of our investigation and analysis can be concluded as follows:

The proposed site is serviceable with the added density with respect to sanitary, water and storm by connecting to the existing infrastructure surrounding the site as outlined in this report.

14. REFERENCES

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12. **ROAD AND BRIDGE DECK DRAINAGE SYSTEMS** by MTO, November 1982.
13. **XPSWMM users Guide** by INNOVYZE 2021.
14. **EPA SWMM 5**, Build 5.1.012, Manual.
15. **LOW IMPACT DEVELOPMENT STORMWATER MANAGEMENT MANUAL**, 2008, by Credit Valley Conservation Authority and Toronto Town Conservation Authority.
16. **Master Servicing and Stormwater Management Report**, for 1899 Brock Road, City of Pickering, May 2021 by SCHAEFFERS.
17. **Functional Servicing & Stormwater Management Report Residential Townhouse Development – 1856 Notion Road Durham Region – City of Pickering**, January 19, 2018, by GHD.
18. City of Pickering and Pickering Developments Inc. – **New Highway 401 Road Crossing (from Notion Road to Squires Beach Road) Schedule "C' Municipal Class Environmental Assessment**, October 2019, by AECOM.

Respectfully Submitted:
The Odan Detech Group Inc.

January 29th, 2025

Paul Hecimovic, P.Eng.

Mark Harris

Digitally signed by Mark Harris
DN: C=CA,
E=mark@odandetech.com,
O=The Odan/Detech Group Inc.,
CN=Mark Harris
Date: 2025.01.29 11:27:07-05'00'

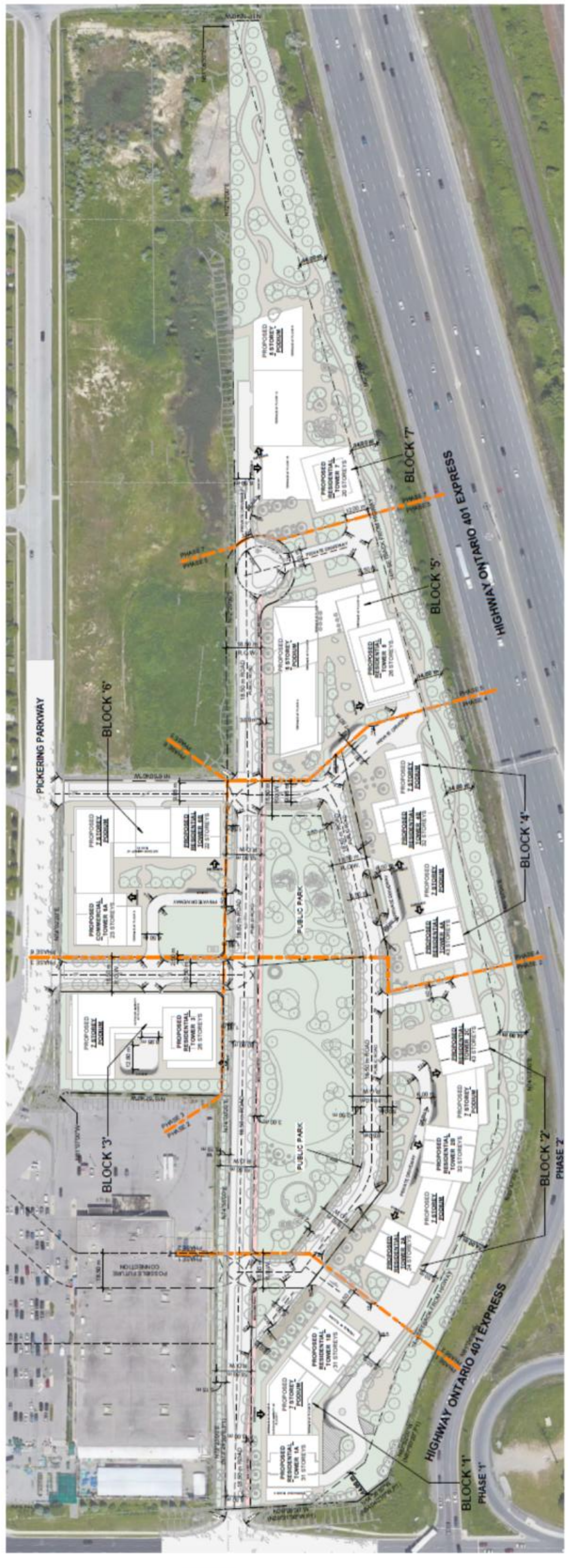
Mark Harris, Dipl. Tech.

APPENDIX A

Aerial Photo of Existing Site
Site Plan of the Proposed Development (reduced)

Aerial Photo of Existing Site

Site Plan of the Proposed Development (reduced)



APPENDIX B

Existing condition sanitary sewer design sheet

Redeveloped site Phase 1 sanitary sewer design sheet - REQUIRED SIZES

Redeveloped site Phase 1 sanitary sewer design sheet - PROPOSED SIZES

Redeveloped sites (subject, 1899 Brock Road and surrounding tributaries) sanitary sewer design sheet

Region of Durham Tributary Maps & Correspondence indicating population densities

FIGURE S-3

SCENARIO 1: EXISTING CONDITIONS
1755 and 1805 Pickering Parkway Existing Conditions Sanitary Flow Calculation

DESIGNED BY: S. Ahonen
CHECKED BY: M. Al-Awad
DATE: 2025-01-17

STREET	TRIB ID	UPSTREAM MH	DOWNSTREAM MH	RESIDENTIAL: LOT AREA UNIT (ha)	LOT AREA ACCUM. (ha)	POP. DENSITY (Persons/ha)	POP. DENSITY (Persons/Unit)	# OF UNITS	POP.	PEAK FLOW FACTOR, K_H	COMMERCIAL: LOT AREA (Ha)	FLOOR SPACE INDEX	GROSS FLOOR AREA GFA (ha)	GROSS FLOOR AREA ACCUM. (ha)	FLOW (L/s): RESIDENTIAL FLOW INFIL. 0.26 (L/s)	RESIDENTIAL FLOW SEWAGE 0.0042 (L/s)	COMM. 2.08 l/s	TOTAL FLOW l/s	EXISTING SEWER: Length L (m)	Size D (mm)	Slope S (%)	Full Flow Capacity Qcap (L/s)	Full Flow Velocity V (m/s)	PRESENT CONDITION % Full Q(d)/Qcap	NOTES
Canadian Tire Site	4	EX.MH090	EX MH H9-0091										0.79	0.79			1.65	1.65	59.8	200	0.30	17.96	0.57	9.2	
Subject Site	1	CAP-3	EX.MH4A										0.28	0.28			0.59	0.59	12.0	150	1.00	15.23	0.86	3.9	
Subject Site		EX.MH4A	EX.MH3A											0.28			0.59	0.59	53.9	150	0.96	14.92	0.84	3.9	
Subject Site	2	EX.MH5A	EX.MH3A										0.92	0.92			1.91	1.91	51.8	150	0.97	15.00	0.85	12.8	
Subject Site	3	CAP-1	EX.MH3A										0.94	0.94			1.96	1.96	13.0	150	0.92	14.61	0.83	13.4	
Subject Site		EX.MH3A	EX.MH2A											2.15			4.46	4.46	85.0	250	0.53	43.29	0.88	10.3	
Subject Site		EX.MH2A	EX.MH1A											2.15			4.46	4.46	38.1	250	0.45	39.89	0.81	11.2	
Subject Site		EX.MH1A	EX MH H9-0091											2.15			4.46	4.46	14.0	250	0.50	42.05	0.86	10.6	
Pickering Parkway	13	EX MH H9-0091	EX MH H9-0018	0.25	0.25									2.94	0.07		6.11	6.17	91.2	250	0.35	35.18	0.72	17.5	
Pickering Parkway	14	EX MH H9-0018	EX MH H9-0019	0.24	0.49									2.94	0.13		6.11	6.23	100.0	250	0.49	41.63	0.85	15.0	
Pickering Parkway	15	EX MH H9-0019	EX MH H9-0010	0.28	0.77									2.94	0.20		6.11	6.31	99.8	250	0.48	41.20	0.84	15.3	
BEECHLAWN DR	7	EX MH018	EX MH H9-0010	2.89	2.89		3.5	63	221	3.80					0.75	3.52		4.27	59.0	200	0.95	31.97	1.02	13.4	
METROPIA	6	EX MH3A	EX MH H9-0010	2.09	2.09		3	130	390	3.80					0.54	6.22		6.77	38.2	200	1.00	32.80	1.04	20.6	
Pickering Parkway	16	EX MH H9-0010	EX MH H9-0011	0.22	5.97				611	3.80				2.94	1.55	9.74	6.11	17.40	82.5	250	0.36	35.68	0.73	48.8	
Pickering Parkway	17	EX MH H9-0011	EX MH H9-0022	0.24	6.21				611	3.80				2.94	1.61	9.74	6.11	17.47	80.1	250	0.46	40.33	0.82	43.3	
Subject Site	5	EX MH 35-34	EX MH 35-33										0.42	0.42			0.88	0.88	84.7	150	1.00	15.23	0.86	5.8	
Notion Road	20	EX MH 35-33	EX MH H9-0014	0.20	0.20									0.42	0.05		0.88	0.93	109.4	200	1.82	44.25	1.41	2.1	
Pickering Parkway	19	EX MH H9-0014	EX MH 35-9	0.22	0.42									0.42	0.11		0.88	0.99	99.7	250	0.58	45.29	0.92	2.2	
Pickering Parkway	18	EX MH 35-9	EX MH H9-0022	0.01	0.43									0.42	0.11		0.88	0.99	10.4	250	1.12	62.93	1.28	1.6	
MARSHCOURT DR		EX MH H9-0022	EX MH 35-25	0.00	6.64				611	3.80				3.36	1.73	9.74	6.99	18.46	58.7	250	0.41	38.08	0.78	48.5	
ASHFORD DR	8	EX.MH023	EX MH 35-25	1.93	1.93		3.5	44	154	3.80					0.50	2.46		2.96	73.0	200	0.10	10.37	0.33	28.5	
MARSHCOURT DR	9	SAN MH 35-25	EX MH 35-26	0.29	8.86		3.5	8	28	3.80				3.36	2.30	0.45	6.99	9.74	72.8	250	0.54	43.70	0.89	22.3	
MARSHCOURT DR	10	SAN MH 35-26	EX MH 35-27	0.60	9.46		3.5	14	49	3.80				3.36	2.46	0.78	6.99	10.23	70.3	250	0.55	44.10	0.90	23.2	
MARSHCOURT DR	11, 12	EX MH 032	EX MH 35-27	17.39	17.39		3.5	262	917	3.80			0.67	0.67	4.52	14.64	1.39	20.55	40.5	250	0.27	30.90	0.63	66.5	
EASEMENT		SAN MH 35-27	SAN MH H9-0029		26.85				966	3.80				4.03	6.98	15.42	8.38	30.78	124.0	375	0.16	70.13	0.63	43.9	
NOTION ROAD		SAN MH H9-0029	SAN MH H9-0045															30.78	71.8	375	0.22	82.24	0.74	37.4	
NOTION ROAD		SAN MH H9-0045	SAN MH 17															30.78	4.0	375	0.23	84.09	0.76	36.6	
ORCHARD ROAD																		**30.78**		750	Available capacity at Orchard Rd 750mm dia. pipe is 150 L/s. Total flow calculated here does not include the existing sanitary flows south on Notion Rd.				see note below about capacity*

Design Criteria as per The Regional Municipality of Durham '*Design Specifications for Sanitary Sewers*'
Average daily per capita flow = 364 L/cap/day (Residential)
Average daily per capita flow = 180,000 L/GFA hectares/day (commercial&industrial)
I = Unit of peak extraneous flow when foundation drains are NOT connected to the storm sewer = 0.26 L/s/Ha
Q(p) = peak population flow (L/s) Q(I) = peak extraneous flow (L/s)
Q(d) = peak design flow (L/s)
PEAKING FACTOR (Harmon; Residential) M =1 + 14/(4+(P/1000^0.5))
PEAK POPULATION FLOW, Q (p) = q*P*M / 86400 L / Sec.
PEAK EXTRANEOUS FLOW, Q(i) = I*A L / Sec.
PEAK DESIGN FLOW, Q(d) = Q(p) + Q(i) L / Sec.
PIPE ROUGHGNESS, n = 0.013 For Manning's Equation

NOTES:
1) MINIMUM VELOCITY = 0.60 m/s
2) MAXIMUM VELOCITY = 3.65 m/s
3) INFILTRATION 0.26 l/s = 22.5 m3/Ha/DAY
INFILTRATION 0.52 l/s = 45.0 m3/Ha/DAY (Foundation DraIn Connections)
4) COMMERCIAL 2.08 l/s (local sewers) 1.04 l/s (trunk sewers)
5) EXISTING CONDITION INCLUDES COMMITTED DEVELOPMENT
6) USE ACTUAL METRIC I.D. PIPE SIZE IN mm
7) COMMERCIAL FLOOR SPACE INDEX=50% UNLESS OTHERWISE KNOWN

Population Density by Land Use

Housing Type	Density
Single & Semi Detached	3.5 P/u
Townhouse	3.0 P/u
Apartment-2Bdrm	2.5 P/u

Housing Type	Density
Single Family	60 persons/ha
Semi Detached & Duplex	100 persons/ha

* ASSUMED 150 L/s AVAILABLE EXCESS FLOW CAPACITY AT ORCHARD ROAD as per correspondence with Durham Region

MUNICIPALITY: REGIONAL MUNICIPALITY OF DURHAM
PROJECT: 20266

20266 - Phase 1 - FSR - Sanitary Design Sheets January 2025
ODAN DETECH GROUP INC.

FIGURE S4

NOTE: THIS DESIGN IS PROVIDED FOR REFERENCE ONLY, AND INTENDED TO PROVIDE CONTEXT OF REQUIRED PHASE 1 PIPE SIZES WITHOUT NO FUTURE BUILD OUT

SCENARIO 2: PHASE 1 CONDITIONS
Redeveloped subject site Phase 1 sanitary sewer design sheet **REQUIRED PIPE SIZES**

DESIGNED BY: S. Ahonen
CHECKED BY: M. Al-Awad

DATE: 2025-01-17

STREET	TRIB ID	UPSTREAM MH	DOWNSTREAM MH	RESIDENTIAL: LOT AREA UNIT (ha)	RESIDENTIAL: LOT AREA ACCUM. (ha)	RESIDENTIAL: POP. DENSITY (Persons/ha)	RESIDENTIAL: POP. DENSITY (Persons/Unit)	RESIDENTIAL: # OF UNITS	RESIDENTIAL: POP.	RESIDENTIAL: PEAK FLOW FACTOR, K_H	COMMERCIAL: LOT AREA (Ha)	COMMERCIAL: FLOOR SPACE INDEX	COMMERCIAL: GROSS FLOOR AREA GFA (ha)	COMMERCIAL: GROSS FLOOR AREA ACCUM. (ha)	INDUST. GROSS FLOOR AREA UNIT (ha)	FLOW (L/s): RESIDENTIAL FLOW INFIL. 0.26 (L/s)	FLOW (L/s): RESIDENTIAL FLOW SEWAGE 0.0042 (L/s)	FLOW (L/s): COMM. 2.08 l/s	FLOW (L/s): INDUS. 2.08 l/s see note 4	FLOW (L/s): INSTIT. 1.30 l/s	FLOW (L/s): TOTAL FLOW l/s	EXISTING SEWER: Length L (m)	EXISTING SEWER: Size D (mm)	EXISTING SEWER: Slope S (%)	EXISTING SEWER: Full Flow Capacity Qcap (L/s)	EXISTING SEWER: Full Flow Velocity V (m/s)	PRESENT CONDITION: % Full Q(d)/Qcap	NOTES
Canadian Tire Site	4	EX.MH090	SAN MH H9-0091										0.79	0.79				1.65			1.65	59.8	200	0.30	17.96	0.57	9.2	
Pickering Parkway		SAN MH 34-82	Prop MH9A	0.52	0.52									0.79		0.14	0.00	1.65			1.78	49.0	250	0.37	36.17	0.74	4.9	required pipe see following sheet
Subject Site	P1	Prop MH1A	Prop MH2A	1.18	1.18		1.5	126	1793	3.62			0.17	0.17		0.31	27.28	0.35			27.94	20.7	300	0.87	90.20	1.28	31.0	required pipe see following sheet
							2.5	337																				
							3.5	207																				
							4.5	8																				
Subject Site		Prop MH2A	Prop MH3A		1.18				1793	3.62				0.17		0.31	27.28	0.35			27.94	60.8	300	0.70	80.91	1.14	34.5	required pipe see following sheet
Subject Site	P2	Prop MH3A	Prop MH4A		1.18				1793	3.62			0.28	0.45		0.31	27.28	0.93			28.52	90.0	300	0.72	82.05	1.16	34.8	required pipe see following sheet
Subject Site		Prop MH4A	Prop MH1A-1		1.18				1793	3.62				0.45		0.31	27.28	0.93			28.52	96.4	300	0.55	71.72	1.01	39.8	Interim pipe Phase 1
Subject Site	P3,2	Prop MH1A-1	Prop MH7A		1.18				1793	3.62			1.50	1.95		0.31	27.28	4.05			31.64	45.4	300	0.31	53.84	0.76	58.8	Interim pipe Phase 1
Subject Site		Prop MH7A	Prop MH8A		1.18				1793	3.62				1.95		0.31	27.28	4.05			31.64	29.9	300	0.52	69.73	0.99	45.4	required pipe see following sheet
Subject Site		Prop MH8A	Prop MH9A		1.18				1793	3.62				1.95		0.31	27.28	4.05			31.64	14.3	300	0.72	82.05	1.16	38.6	required pipe see following sheet
Pickering Parkway	13	Prop MH9A	SAN MH H9-0018	0.25	1.95				1793	3.62				2.74		0.51	27.28	5.70			33.48	42.0	300	0.42	62.67	0.89	53.4	required pipe see following sheet
Pickering Parkway	14	SAN MH H9-0018	SAN MH H9-0019	0.24	2.19				1793	3.62				2.74		0.57	27.28	5.70			33.55	100.0	300	0.47	66.29	0.94	50.6	required pipe see following sheet
Pickering Parkway	15	SAN MH H9-0019	SAN MH H9-0010	0.28	2.47				1793	3.62				2.74		0.64	27.28	5.70			33.62	99.8	300	0.48	67.00	0.95	50.2	required pipe see following sheet
BEECHLAWN DR	7	EX MH018	SAN MH H9-0010	2.89	2.89		3.5	63	221	3.80						0.75	3.52				4.27	59.0	200	0.95	31.97	1.02	13.4	
METROPIA	6	EX MH3A	SAN MH H9-0010	2.09	2.09		3	130	390	3.80						0.54	6.22				6.77	38.2	200	1.00	32.80	1.04	20.6	
Pickering Parkway	16	SANMH H9-0010	SAN MH H9-0011	0.22	7.67				2404	3.52				2.74		1.99	35.56	5.70			43.25	82.5	300	0.38	59.61	0.84	72.6	required pipe see following sheet
Pickering Parkway	17	SAN MH H9-0011	SAN MH 35-8	0.24	7.91				2404	3.52				2.74		2.06	35.56	5.70			43.31	80.0	300	0.46	65.59	0.93	66.0	required pipe see following sheet
Pickering Parkway	18	SAN MH 35-8	SAN MH H9-0014	0.22	8.13				2404	3.52				2.74		2.11	35.56	5.70			43.37	110.1	300	0.57	73.01	1.03	59.4	required pipe see following sheet
Subject Site	5	SAN MH 35-34	SAN MH 35-33										0.42	0.42		0.00		0.88			0.88	145.7	150	1.00	15.23	0.86	5.8	
Notion Road	20	SAN MH 35-33	SAN MH H9-0014	0.50	0.50									0.42		0.13		0.88			1.01	109.4	200	1.82	44.25	1.41	2.3	
~~MARSHCOURT DR~~		~~EX MH 35-8~~	~~EX MH 35-25~~		~~0.00~~					~~0.00~~						~~0.00~~	~~0.00~~	~~0.00~~			~~0.00~~	~~58.9~~	~~250~~	~~0.41~~	~~38.08~~	~~0.78~~	~~0.0~~	pipe to remain as cleanout access
ASHFORD DR	8	EX.MH023	SAN MH 35-25	1.93	1.93		3.5	44	154	3.80						0.50	2.46				2.96	73.0	200	0.10	10.37	0.33	28.5	
MARSHCOURT DR	9	SAN MH 35-25	SAN MH 35-26	0.29	2.22		3.5	8	28	3.80						0.58	0.45	0.00			1.02	72.8	250	0.54	43.70	0.89	2.3	
MARSHCOURT DR	10	SAN MH 35-26	SAN MH 35-27	0.60	2.82		3.5	14	49	3.80						0.73	0.78	0.00			1.51	70.3	250	0.55	44.10	0.90	3.4	
MARSHCOURT DR	11, 12	EX MH 032	SAN MH 35-27	17.39	17.39		3.5	262	917	3.80			0.67	0.67		4.52	14.64	1.39			20.55	40.5	250	0.27	30.90	0.63	66.5	
EASEMENT		SAN MH 35-27	SAN MH 35-29	0.00	20.21				966	3.80				0.67		5.25	15.42	1.39			22.06	124.0	375	0.16	70.13	0.63	31.5	
NOTION ROAD		SAN MH H9-0014	Prop MH 13A	0.01	8.64				2404	3.52				3.16		2.25	35.56	6.58			44.39	14.5	450	0.22	133.73	0.84	33.2	Interim pipe Phase 1
NOTION ROAD		Prop MH 13A	Prop MH 14A	0.25	8.89				2404	3.52				3.16		2.31	35.56	6.58			44.45	100.0	450	0.22	133.73	0.84	33.2	Interim pipe Phase 1
NOTION ROAD	21,23	Prop MH 14A	SAN MH H9-0029	0.29	9.18				2404	3.52				3.16	0.66	2.39	35.56	6.58	1.3728		45.90	101.8	450	0.22	133.73	0.84	34.3	Interim pipe Phase 1
NOTION ROAD	22	SAN MH H9-0029	SAN MH H9-0045	0.30	9.48				3370	3.40				3.83	0.66	2.46	48.10	7.98	1.3728		59.92	71.8	450	0.20	127.50	0.80	47.0	Interim pipe Phase 1
NOTION ROAD		SAN MH H9-0045	SAN MH 17		9.48				3370	3.40				3.83	0.66	2.46	48.10	7.98	1.3728		59.92	4.0	450	0.20	127.50	0.80	47.0	Interim pipe Phase 1
ORCHARD ROAD		SAN MH 17	SAN MH 18		9.48																**59.92**		750	Available capacity at Orchard Rd 750mm dia. pipe is 150 L/s. Total flow calculated here does not include the existing sanitary flows conveyed south on Notion Rd to Orchard Rd.				see note below about capacity*

Design Criteria as per The Regional Municipality of Durham '*Design Specifications for Sanitary Sewers*'
Average daily per capita flow = 364 L/cap/day (Residential)
Average daily per capita flow = 180,000 L/GFA hectares/day (commercial&industrial)
I = Unit of peak extraneous flow when foundation drains are NOT connected to the storm sewer = 0.26 L/s/Ha
Q(p) = peak population flow (L/s) Q(I) = peak extraneous flow (L/s)
Q(d) = peak design flow (L/s)
PEAKING FACTOR (Harmon; Residential) M =1 + 14/(4+(P/1000^0.5))
PEAK POPULATION FLOW, Q (p) = q*P*M / 86400 L / Sec.
PEAK EXTRANEOUS FLOW, Q(i) = I*A L / Sec.
PEAK DESIGN FLOW, Q(d) = Q(p) + Q(i) L / Sec.
PIPE ROUGHGNESS, n = 0.013 For Manning's Equation

NOTES:
1) MINIMUM VELOCITY = 0.60 m/s
2) MAXIMUM VELOCITY = 3.65 m/s
3) INFILTRATION 0.26 l/s = 22.5 m3/Ha/DAY
   INFILTRATION 0.52 l/s = 45.0 m3/Ha/DAY (Foundation DraIn Connections)
4) COMMERCIAL 2.08 l/s (local sewers) 1.04 l/s (trunk sewers)
5) EXISTING CONDITION INCLUDES COMMITTED DEVELOPMENT
6) USE ACTUAL METRIC I.D. PIPE SIZE IN mm
7) COMMERCIAL FLOOR SPACE INDEX=50% UNLESS OTHERWISE KNOWN

Population Density by Land Use

Housing Type	Density
Single & Semi Detached	3.5 P/u
Townhouse	3.0 P/u
1 Bedroom	1.5 P/u
2 Bedroon and 1 Bedroom+Den	2.5 P/u
3 Bedroom	3.5 P/u
4 Bedroom	4.5 P/u

Housing Type	Density
Single Family	60 persons/ha
Semi Detached & Duplex	100 persons/ha

ODAN·DETECH CONSULTING ENGINEERS

* ASSUMED 150 L/s AVAILABLE EXCESS FLOW CAPACITY AT ORCHARD ROAD as per correspondence with Durham Region

MUNICIPALITY: REGIONAL MUNICIPALITY OF DURHAM
PROJECT: 20266

20266 - Phase 1 - FSR - Sanitary Design Sheets January 2025
ODAN DETECH GROUP INC.

SCENARIO 2: PHASE 1 CONDITIONS
Redeveloped subject site Phase 1 sanitary sewer design sheet **PROPOSED PIPE SIZES**

DESIGNED BY: S. Ahonen
CHECKED BY: M. Al-Awad

FIGURE S-4

DATE: 2025-01-17

STREET	TRIB ID	UPSTREAM MH	DOWNSTREAM MH	RESIDENTIAL: LOT AREA UNIT (ha)	LOT AREA ACCUM. (ha)	POP. DENSITY (Persons/ha)	POP. DENSITY (Persons/Unit)	# OF UNITS	POP.	PEAK FLOW FACTOR, K_H	COMMERCIAL: LOT AREA (Ha)	FLOOR SPACE INDEX	GROSS FLOOR AREA GFA (ha)	GROSS FLOOR AREA ACCUM. (ha)	INDUST.: GROSS FLOOR AREA UNIT (ha)	ACCUM. (ha)	FLOW (L/s): RESIDENTIAL FLOW INFIL. 0.26 (L/s)	SEWAGE 0.0042 (L/s)	COMM. 2.08 l/s	INDUS. 2.08 l/s see note	INSTIT. 1.30 l/s	TOTAL FLOW l/s	EXISTING SEWER: Length L (m)	Size D (mm)	Slope S (%)	Full Flow Capacity Qcap (L/s)	Full Flow Velocity V (m/s)	PRESENT CONDITION % Full Q(d)/Qcap	NOTES
Canadian Tire Site	4	EX.MH090	SAN MH H9-0091										0.79	0.79					1.65			1.65	59.8	200	0.30	17.96	0.57	9.2	
Pickering Parkway		SAN MH 34-82	Prop MH9A	0.52	0.52									0.79			0.14	0.00	1.65			1.78	49.0	675	0.45	563.88	1.58	0.3	pipe sized for full build-out
Subject Site	P1	Prop MHBK1	Prop MH2A	1.18	1.18			678	1793	3.62			0.17	0.17			0.31	27.28	0.35			27.94	11.3	300	2.00	136.76	1.93	20.4	pipe sized for full build-out
Subject Site		Prop MH2A	Prop MH3A		1.18				1793	3.62				0.17			0.31	27.28	0.35			27.94	60.8	300	0.70	80.91	1.14	34.5	pipe sized for full build-out
Subject Site	P2	Prop MH3A	Prop MH4A		1.18				1793	3.62			0.28	0.45			0.31	27.28	0.93			28.52	90.0	300	0.70	80.91	1.14	35.2	pipe sized for full build-out
Subject Site		Prop MH4A	Prop MH9A-1		1.18				1793	3.62				0.45			0.31	27.28	0.93			28.52	41.1	300	0.70	80.91	1.14	35.2	Interim pipe Phase 1
Subject Site	P3,2	Prop MH9A-1	Prop MH6A		1.18				1793	3.62			1.50	1.95			0.31	27.28	4.05			31.64	35.1	300	0.70	80.91	1.14	39.1	Interim pipe Phase 1
Subject Site		Prop MH6A	Prop MH1A-1		1.18				1793	3.62				1.95			0.31	27.28	4.05			31.64	22.8	300	0.70	80.91	1.14	39.1	pipe sized for full build-out
Subject Site		Prop MH1A-1	Prop MH7A		1.18				1793	3.62				1.95			0.31	27.28	4.05			31.64	45.4	450	0.70	238.54	1.50	13.3	pipe sized for full build-out
Subject Site		Prop MH7A	Prop MH8A		1.18				1793	3.62				1.95			0.31	27.28	4.05			31.64	29.9	450	0.70	238.54	1.50	13.3	pipe sized for full build-out
Subject Site		Prop MH8A	Prop MH9A		1.18				1793	3.62				1.95			0.31	27.28	4.05			31.64	14.3	450	0.70	238.54	1.50	13.3	pipe sized for full build-out
Pickering Parkway	13	Prop MH9A	SAN MH H9-0018	0.25	1.95				1793	3.62				2.74			0.51	27.28	5.70			33.48	42.0	675	0.45	563.88	1.58	5.9	pipe sized for full build-out
Pickering Parkway	14	SAN MH H9-0018	SAN MH H9-0019	0.24	2.19				1793	3.62				2.74			0.57	27.28	5.70			33.55	100.0	675	0.45	563.88	1.58	5.9	pipe sized for full build-out
Pickering Parkway	15	SAN MH H9-0019	SAN MH H9-0010	0.28	2.47				1793	3.62				2.74			0.64	27.28	5.70			33.62	99.8	675	0.45	563.88	1.58	6.0	pipe sized for full build-out
BEECHLAWN DR	7	EX MH018	EX MH H9-0010	2.89	2.89		3.5	63	221	3.80							0.75	3.52				4.27	59.0	200	0.95	31.97	1.02	13.4	
METROPIA	6	EX MH3A	EX MH H9-0010	2.09	2.09		3	130	390	3.80							0.54	6.22				6.77	38.2	200	1.00	32.80	1.04	20.6	
Pickering Parkway	16	EX MH H9-0010	EX MH H9-0011	0.22	7.67				2404	3.52				2.74			1.99	35.56	5.70			43.25	82.5	675	0.45	563.88	1.58	7.7	pipe sized for full build-out
Pickering Parkway	17	EX MH H9-0011	EX MH-H9-0022	0.24	7.91				2404	3.52				2.74			2.06	35.56	5.70			43.31	80.0	675	0.45	563.88	1.58	7.7	pipe sized for full build-out
Pickering Parkway	18	EX MH-H9-0022	EX MH H9-0014	0.22	8.13				**2404**	3.52				2.74			2.11	35.56	5.70			43.37	110.1	675	0.45	563.88	1.58	7.7	pipe sized for full build-out
Subject Site	5	SAN MH 35-34	SAN MH 35-33										0.42	0.42			0.00		0.88			0.88	145.7	150	1.00	15.23	0.86	5.8	
Notion Road	20	SAN MH 35-33	SAN MH H9-0014	0.50	0.50									0.42			0.13		0.88			1.01	109.4	200	1.82	44.25	1.41	2.3	
~~MARSHCOURT DR~~		~~EX MH 35-8~~	~~EX MH 35-25~~		~~0.00~~					~~0.00~~							~~0.00~~	~~0.00~~	~~0.00~~			~~0.00~~	~~58.9~~	~~250~~	~~0.41~~	~~38.08~~	~~0.78~~	~~0.0~~	pipe to remain as cleanout access
ASHFORD DR	8	EX.MH023	SAN MH 35-25	1.93	1.93		3.5	44	154	3.80							0.50	2.46				2.96	73.0	200	0.10	10.37	0.33	28.5	
MARSHCOURT DR	9	SAN MH 35-25	SAN MH 35-26	0.29	2.22		3.5	8	28	3.80							0.58	0.45	0.00			1.02	72.8	250	0.54	43.70	0.89	2.3	
MARSHCOURT DR	10	SAN MH 35-26	SAN MH 35-27	0.60	2.82		3.5	14	49	3.80							0.73	0.78	0.00			1.51	70.3	250	0.55	44.10	0.90	3.4	
MARSHCOURT DR	11, 12	EX MH 032	SAN MH 35-27	17.39	17.39		3.5	262	917	3.80			0.67	0.67			4.52	14.64	1.39			20.55	40.5	250	0.27	30.90	0.63	66.5	
EASEMENT		SAN MH 35-27	SAN MH H9-0029	0.00	20.21				966	3.80				0.67			5.25	15.42	1.39			22.06	124.0	375	0.16	70.13	0.63	31.5	
NOTION ROAD		SAN MH H9-0014	Prop MH 13A	0.01	0.51				2404	3.52				3.16			0.13	35.56	6.58			42.27	14.5	450	0.40	180.32	1.13	23.4	**Interim pipe Phase 1**
NOTION ROAD		Prop MH 13A	Prop MH 14A	0.25	0.76				2404	3.52				3.16			0.20	35.56	6.58			42.34	100.0	450	0.22	133.73	0.84	31.7	**Interim pipe Phase 1**
NOTION ROAD	21,23	Prop MH 14A	SAN MH H9-	0.29	1.05				2404	3.52				3.16	0.66		0.27	35.56	6.58	1.3728		43.79	101.8	450	0.22	133.73	0.84	32.7	**Interim pipe Phase 1**
NOTION ROAD	22	SAN MH H9-0029	SAN MH H9-0045	0.30	21.56				3370	3.40				3.83	0.66		5.60	48.10	7.98	1.3728		63.06	71.8	450	0.22	133.73	0.84	47.2	Interim pipe Phase 1
NOTION ROAD		SAN MH H9-0045	SAN MH 17		21.56				3370	3.40				3.83	0.66		5.60	48.10	7.98	1.3728		63.06	3.5	450	0.23	136.73	0.86	46.1	Interim pipe Phase 1
ORCHARD ROAD		SAN MH 17	SAN MH 18		21.56																	**63.06**		750	Available capacity at Orchard Rd 750mm dia. pipe is 150 L/s. Total flow calculated here does not include the existing sanitary flows conveyed south on Notion Rd to Orchard Rd.				see note below about capacity

Design Criteria as per The Regional Municipality of Durham '*Design Specifications for Sanitary Sewers*'
Average daily per capita flow = 364 L/cap/day (Residential)
Average daily per capita flow = 180,000 L/GFA hectares/day (commercial&industrial)
I = Unit of peak extraneous flow when foundation drains are NOT connected to the storm sewer = 0.26 L/s/Ha
Q(p) = peak population flow (L/s) Q(I) = peak extraneous flow (L/s)
Q(d) = peak design flow (L/s)
PEAKING FACTOR (Harmon; Residential) M =1 + 14/(4+(P/1000^0.5))
PEAK POPULATION FLOW, Q (p) = q*P*M / 86400 L / Sec.
PEAK EXTRANEOUS FLOW, Q(i) = I*A L / Sec.
PEAK DESIGN FLOW, Q(d) = Q(p) + Q(i) L / Sec.
PIPE ROUGHGNESS, n = 0.013 For Manning's Equation

NOTES:
1) MINIMUM VELOCITY = 0.60 m/s
2) MAXIMUM VELOCITY = 3.65 m/s
3) INFILTRATION 0.26 l/s = 22.5 m3/Ha/DAY
   INFILTRATION 0.52 l/s = 45.0 m3/Ha/DAY (Foundation Drain Connections)
4) COMMERCIAL 2.08 l/s (local sewers) 1.04 l/s (trunk sewers)
5) EXISTING CONDITION INCLUDES COMMITTED DEVELOPMENT
6) USE ACTUAL METRIC I.D. PIPE SIZE IN mm
7) COMMERCIAL FLOOR SPACE INDEX=50% UNLESS OTHERWISE KNOWN

Population Density by Land Use

Housing Type	Density
Single & Semi Detached	3.5 P/u
Townhouse	3.0 P/u
1 Bedroom	1.5 P/u
2 Bedroon and 1 Bedroom+Den	2.5 P/u
3 Bedroom	3.5 P/u
4 Bedroom	4.5 P/u

Housing Type	Density
Single Family	60 persons/ha
Semi Detached & Duplex	100 persons/ha

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* ASSUMED 150 L/s AVAILABLE EXCESS FLOW CAPACITY AT ORCHARD ROAD as per correspondence with Durham Region

MUNICIPALITY: REGIONAL MUNICIPALITY OF DURHAM
PROJECT: 20266

20266 - Phase 1 - FSR - Sanitary Design Sheets January 2025
ODAN DETECH GROUP INC.

SCENARIO 3: CONCEPTUAL FULL BUILDOUT CONDITIONS
Full development of subject site and future tributary sanitary design sheet

DESIGNED BY: S. Ahon 0.013
CHECKED BY: M. Al-Aw 09/09/2021

DATE: 2025-01-17

FIGURE S-5

STREET	TRIB ID	UPSTREAM MH	DOWNSTREAM MH	RESIDENTIAL LOT AREA UNIT (ha)	RESIDENTIAL LOT AREA ACCUM. (ha)	POP. DENSITY (Persons/ha)	POP. DENSITY (Persons/ Unit)	# OF UNITS	POP.	PEAK FLOW FACTOR, K_H	COMMERCIAL LOT AREA (Ha)	FLOOR SPACE INDEX	GROSS FLOOR AREA GFA (ha)	GROSS FLOOR AREA ACCUM. (ha)	INDUST. GROSS FLOOR AREA UNIT (ha)	INDUST. GROSS FLOOR AREA ACCUM. (ha)	FLOW (L/s) RESIDENTIAL FLOW INFIL. 0.26 (L/s)	RESIDENTIAL FLOW SEWAGE 0.0042 (L/s)	COMM. 2.08 l/s	TOTAL FLOW l/s	EXISTING SEWER Length L (m)	Size D (mm)	Slope S (%)	Full Flow Capacity Qcap (L/s)	Full Flow Velocity V (m/s)	PRESENT CONDITION % Full Q(d)/Qcap	NOTES
1899 Brock Road	P9	Prop MH16A	SAN MH H9-0091	29.50	29.50	800			23600	2.58							7.67	255.78	0.00	263.45	116.0	525	1.00	430.06	1.99	61.3	FUTURE PROPOSED
Canadian Tire Lands	P10	EX.MH090	SAN MH H9-0091	4.10	4.10	1200			4920	3.25							1.07	67.19	0.00	68.25	59.8	450	0.30	156.16	0.98	43.7	EX PIPE OUTSIDE SCOPE OF WORK
Pickering Parkway	13	SAN MH H9-0091	Prop MH9A	0.25	33.85				28520	2.50				0.00			8.80	299.32	0.00	308.12	49.0	675	0.45	563.88	1.58	54.6	PROPOSED
Subject Site	P1	Prop MHBK1	Prop MH2A	1.18	1.18			678	1793	3.62			0.17	0.17			0.31	27.28	0.35	27.94	11.3	300	2.00	136.76	1.93	20.4	PROPOSED
Subject Site		Prop MH2A	Prop MH3A		1.18				1793	3.62				0.17			0.31	27.28	0.35	27.94	60.8	300	0.70	80.91	1.14	34.5	PROPOSED
Subject Site	P2	Prop MH3A	Prop MH4A	1.28	2.46		2.5	1090	4518	3.29			0.10	0.27			0.64	62.34	0.56	63.54	90.0	300	0.70	80.91	1.14	78.5	PROPOSED
Subject Site		Prop MH4A	Prop MH5A		2.46				4518	3.29				0.27			0.64	62.34	0.56	63.54	32.8	300	0.70	80.91	1.14	78.5	PROPOSED
Subject Site	P3,P4	Prop MH5A	Prop MH6A	3.01	5.47		2.5	1022	7073	3.10			0.07	0.34			1.42	92.16	0.71	94.28	35.1	300	0.70	80.91	1.14	116.5	PROPOSED
Subject Site	P5,P6	Prop MH6A	Prop MH1A-1	2.63	8.10		2.5	1403	10581	2.93			0.07	0.41			2.11	130.22	0.85	133.17	22.8	450	0.70	238.54	1.50	55.8	PROPOSED
Subject Site	P7,P8	Prop MH1A-1	Prop MH7A	1.45	9.55		2.5	1208	13601	2.82			2.26	2.67			2.48	161.14	5.55	169.18	45.4	450	0.70	238.54	1.50	70.9	PROPOSED
Subject Site		Prop MH7A	Prop MH8A		9.55				13601	2.82				2.67			2.48	161.14	5.55	169.18	29.9	450	0.70	238.54	1.50	70.9	PROPOSED
Subject Site		Prop MH8A	Prop MH9A		9.55				13601	2.82				2.67			2.48	161.14	5.55	169.18	14.3	450	0.70	238.54	1.50	70.9	PROPOSED
Pickering Parkway	13	Prop MH9A	EX MH H9-0018	0.25	43.65				42121	2.33				2.67			11.35	413.01	5.55	429.91	42.1	675	0.45	563.88	1.58	76.2	PROPOSED
Pickering Parkway	14	EX MH H9-0018	EX MH H9-0019	0.24	43.89				42121	2.33				2.67			11.41	413.01	5.55	429.97	100.0	675	0.45	563.88	1.58	76.3	PROPOSED
Pickering Parkway	15	EX MH H9-0019	EX MH H9-0010	0.28	44.17				42121	2.33				2.67			11.48	413.01	5.55	430.04	99.8	675	0.45	563.88	1.58	76.3	PROPOSED
BEECHLAWN DR	7	EX MH018	EX MH H9-0010	2.89	2.89		3.5	63	221	3.80							0.75	3.52		4.27	59.0	200	0.95	31.97	1.02	13.4	EX
METROPIA	20	SAN MH3A	SAN MH H9-	2.09	2.09		3	130	390	3.80							0.54	6.22		6.77	38.2	200	1.00	32.80	1.04	20.6	EX
~~MARSHCOURT DR~~		~~EX MH H9-0022~~	~~EX MH 35-25~~							~~0.00~~							~~0.00~~	~~0.00~~	~~0.00~~	~~0.00~~	~~58.9~~	~~250~~	~~0.41~~	~~38.08~~	~~0.78~~	~~0.0~~	pipe to remain as cleanout access
ASHFORD DR	8	EX.MH023	SAN MH 35-25	1.93	1.93		3.5	44	154	3.80							0.50	2.46		2.96	73.0	200	0.10	10.37	0.33	28.5	EX
MARSHCOURT DR	9	SAN MH 35-25	SAN MH 35-26	0.29	2.22		3.5	8	28	3.80							0.58	0.45	0.00	1.02	72.8	250	0.54	43.70	0.89	2.3	EX
MARSHCOURT DR	10	SAN MH 35-26	SAN MH 35-27	0.60	2.82		3.5	14	49	3.80							0.73	0.78	0.00	1.51	70.3	250	0.55	44.10	0.90	3.4	EX
MARSHCOURT DR	11	EX MH 032	SAN MH 35-27	17.39	17.39		3.5	262	917	3.80							4.52	14.64	0.00	19.16	40.5	250	0.27	30.90	0.63	62.0	EX
EASEMENT		SAN MH 35-27	SAN MH H9-0029		20.21				966	3.80							5.25	15.42	0.00	20.67	124.0	375	0.16	70.13	0.63	29.5	outlet to Region Trunk on Notion Rd*
Pickering Parkway	16	SAN MH H9-0010	SAN MH H9-0011	0.22	2.31				42731	2.33				0.00			0.60	417.93	0.00	418.53	82.5	675	0.45	563.88	1.58	74.2	PROPOSED
Pickering Parkway	17	SAN MH H9-0011	SAN MH H9-0022	0.24	2.55				42731	2.33				0.00			0.66	417.93	0.00	418.59	80.1	675	0.45	563.88	1.58	74.2	PROPOSED
Pickering Parkway	18	SAN MH H9-0022	SAN MH H9-0014	0.22	2.77				42731	2.33				0.00			0.72	417.93	0.00	418.65	110.1	675	0.45	563.88	1.58	74.2	PROPOSED
Notion Road		SAN MH H9-0014	MH 13A																	418.65	14.5	675	0.20	375.92	1.05	111.4	outlet to Region Trunk on Notion Rd

Design Criteria as per The Regional Municipality of Durham '*Design Specifications for Sanitary Sewers*'
Average daily per capita flow = 364 L/cap/day (Residential)
Average daily per capita flow = 180,000 L/GFA hectares/day (commercial&industrial)
I = Unit of peak extraneous flow when foundation drains are NOT connected to the storm sewer = 0.26 L/s/Ha
Q(p) = peak population flow (L/s) Q(I) = peak extraneous flow (L/s)
Q(d) = peak design flow (L/s)
PEAKING FACTOR (Harmon; Residential) M =1 + 14/(4+(P/1000^0.5))
PEAK POPULATION FLOW, Q (p) = q*P*M / 86400 L / Sec.
PEAK EXTRANEOUS FLOW, Q(I) = I*A L / Sec.
PEAK DESIGN FLOW, Q(d) = Q(p) + Q(I) L / Sec.
PIPE ROUGHGNESS, n = 0.013 For Manning's Equation

NOTES:
1) MINIMUM VELOCITY = 0.60 m/s
2) MAXIMUM VELOCITY = 3.65 m/s
3) INFILTRATION 0.26 l/s = 22.5 m3/Ha/DAY
INFILTRATION 0.52 l/s = 45.0 m3/Ha/DAY (Foundation Drain Connections)
4) COMMERCIAL 2.08 l/s (local sewers) 1.04 l/s (trunk sewers)
5) EXISTING CONDITION INCLUDES COMMITTED DEVELOPMENT
6) USE ACTUAL METRIC I.D. PIPE SIZE IN mm
7) COMMERCIAL FLOOR SPACE INDEX=50% UNLESS OTHERWISE KNOWN

Population Density by Land Use

Housing Type	Density
Single & Semi Detached	3.5 P/u
Townhouse	3.0 P/u
1 Bedroom	1.5 P/u
2 Bedroom and 1 Bedroom+Den	2.5 P/u
3 Bedroom	3.5 P/u
4 Bedroom	4.5 P/u

Housing Type	Density
Single Family	60 persons/ha
Semi Detached & Duplex	100 persons/ha

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*ASSUMED FLOW FROM EASEMENT SEWER AND PICKERING PARKWAY WILL OUTLET TO REGION TRUNK ON NOTION RD

SANITARY FLOW CALCULATIONS

SCENARIO: Full Phase - JANUARY 2025

Sanitary discharge calculations as per Regional Municipality of Durham *Design Specifications For Sanitary Sewers April 2017.*

LOT AREA (ha)= 9.484

TOTAL Approx. Number of Residential Units = 5,298

UPDATED MARCH 2022

Block	Number of Units	GROSS FLOOR AREA (m^2)	TOTAL POPULATION	TOTAL DAILY FLOW (LITERS)	AVERAGE DAILY FLOW (L/sec)	PEAKING FACTOR, K_H	TOTAL FLOW FROM LAND USE (L/sec)
Block 1 Using 2.5 Persons/Unit	678		1,695	616,980	7.1	3.64	26.00
Block 2 Using 2.5 Persons/Unit	1090		2,725	991,900	11.5	3.48	39.92
Block 3 Using 2.5 Persons/Unit	446		1,115	405,860	4.7	3.77	17.70
Block 4 Using 2.5 Persons/Unit	1022		2,555	930,020	10.8	3.50	37.68
Block 5 Using 2.5 Persons/Unit	641		1,603	583,310	6.8	3.66	24.70
Block 6 Using 2.5 Persons/Unit	762		1,905	693,420	8.0	3.60	28.91
Block 7 Using 2.5 Persons/Unit	659		1,648	599,690	6.9	3.65	25.33
Total Residential	**5,298**		**13,245**	4,821,180	55.8	2.83	**158.06**
Block 1		1,669		30,042	0.3	1	0.348
Block 2		1,006		18,108	0.2	1	0.210
Block 3		920		16,560	0.2	1	0.192
Block 4		696		12,528	0.1	1	0.145
Block 5		665		11,970	0.1	1	0.139
Block 6		21,737		391,266	4.5	1	4.529
Total Non-Residential		**26,693**		480,474	5.6	1	5.561

Q = (MqP/86400) + A * I (L/sec)

Q1= total flow from Residential Land Use (L/sec)
Q2= total flow from Commercial Land Use (L/sec)
Qinfil = total flow from infiltration (L/sec)
Qtot = total flow (Land use + infiltration)

V1= Total Volume from Land Use in liters

V1=	5,301,654	Q1=	158.06
		Q2=	5.56
		Qinfil	2.47
		Qtot	166.09

where : P is population
q(res) = 364 L/cap/day
q(ICI) = 180,000 L/floor hectres/day

A = gross site area
i = 0.26 L/sec/ha (infiltration rate)
Peaking Factor K_H = 1 + [14 / (4 + (P/1000)$^{1/2}$))]

<u>Region of Durham Tributary Maps & Correspondence indicating population densities</u>

Hello Mark,

I have attached some maps showing the approximate areas. If you can get more precise areas from your base, please use them, otherwise just use the numbers below:

- Map 1 North +/- 30 ha @ 800 people/ha = 24,000
- Map 1 South +/- 14 ha @ 1200 people/ha = 16,800
- Map 2 South – approved application for 130 units x 3 people per unit = population of 390 (connection at Beachlawn)

Be sure that the pipe on Pickering Parkway is sized to be at no more than 80% capacity based on these populations.

Let me know if you have any questions.

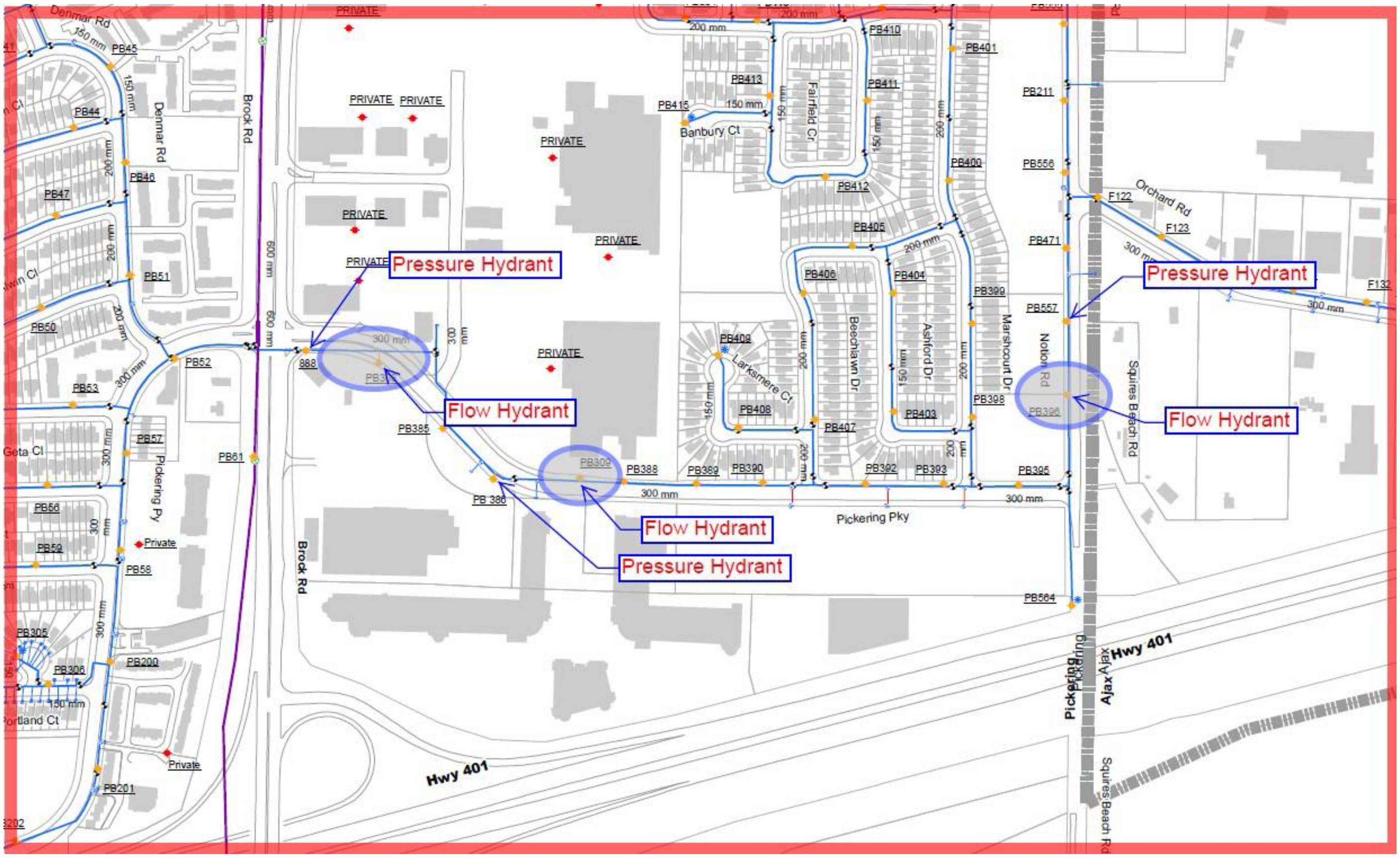
Thanks,

Aaron Christie, P.Eng. | Manager, Engineering Planning & Studies
Works Department
The Regional Municipality of Durham
Aaron.Christie@durham.ca | 905-668-7711 extension 3608 | durham.ca
My pronouns are he/his

APPENDIX C

Location of hydrant flow tests
Hydrant flow tests

Location of hydrant flow tests



SCG | FLOWMETRIX INDU-TECH PROCESS WESTCAN

Fire Flow Testing Report

Residual Hydrant #	**PB557**
NFPA Colour Code	**BLUE**

DATE	September 8, 2021
TIME	10:30 AM
ADDRESS	1972 Notion Rd
	Pickering, ON
SIZE-inches/mm	12 300
MATERIAL	PVC
CONTACT INFO	The Odan/Detech Group Inc.
	Mark Harris
	C: (905) 632-3811 ext.122
	E: mark@odandetech.com

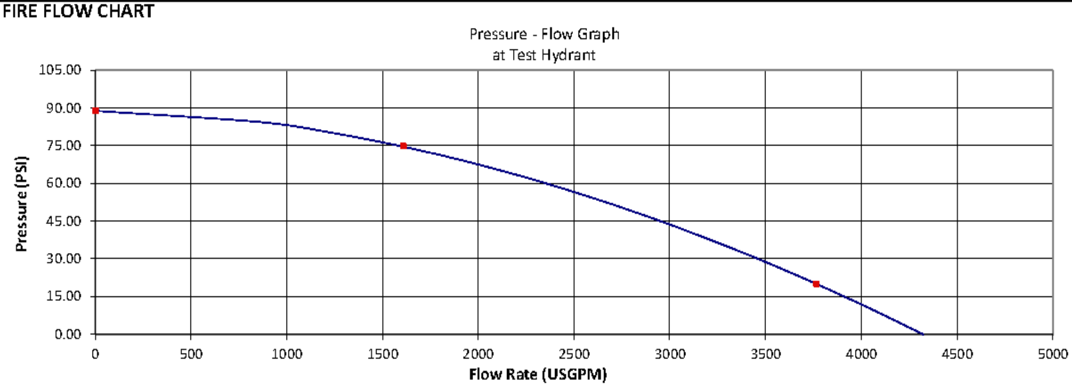
RESIDUAL HYDRANT INFO.

HYDRANT #	PB557	
N.F.P.A. COLOUR CODE	BLUE	
STATIC PRESSURE	88.9	psi
RESIDUAL PRESSURE	74.6	psi
PRESSURE DROP	14.3	psi
% PRESSURE DROP	16.0	% psi
Flow on Water Main At Test Hydrant -	20 psi	3766 USGPM

FLOW HYDRANT(S) INFO.

HYDRANT ASSET ID	HYD. # PORTS	OUTLET DIAMETER (INCHES)	NOZZLE COEFFICIENT	DIFFUSER TYPE	DIFFUSER COEFFICIENT	PITOT READING (psi)	PITOT FLOW (USGPM)	FLOW METER (USGPM)
PB396	2	2.5	Round	LPD250	0.90	**28.4**	804	0
		2.5	Round	LPD250	0.90	**28.4**	804	0
				Total Flow (USGPM)			1609	0
				Total Flow (USGPM)			1609	

FIRE FLOW CHART



COMMENTS

OPERATOR	FMX	Jordan Whitlock
OPERATOR	FMX	Denis Kriventsev
OPERATOR		Region of Durham

ODG_FireFlowTestingReport_Pickering

"If we don't measure it, how do you manage it?"

FLOWMETRIX
INDU-TECH
PROCESS
WESTCAN

Fire Flow Testing Report

Residual Hydrant #	**PB386**
NFPA Colour Code	**BLUE**

DATE	September 8, 2021	
TIME	10:45 AM	
ADDRESS	1735 Pickering Pkwy	
	Pickering, ON	
SIZE-Inches/mm	12	300
MATERIAL		PVC
CONTACT INFO	The Odan/Detech Group Inc.	
	Mark Harris	
	C: (905) 632-3811 ext.122	
	E: mark@odandetech.com	

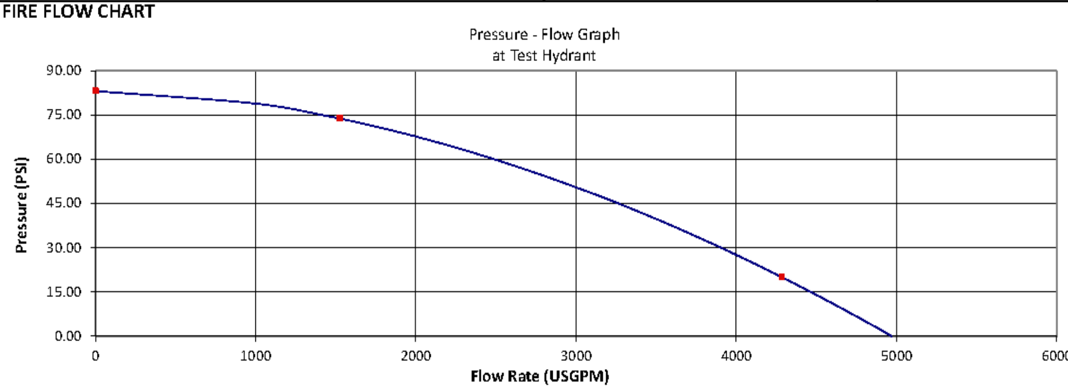
RESIDUAL HYDRANT INFO.

HYDRANT #	PB386	
N.F.P.A. COLOUR CODE	BLUE	
STATIC PRESSURE	83.2	psi
RESIDUAL PRESSURE	73.8	psi
PRESSURE DROP	9.3	psi
% PRESSURE DROP	11.2	% psi
Flow on Water Main At Test Hydrant -	20 psi	4283 USGPM

FLOW HYDRANT(S) INFO.

HYDRANT ASSET ID	HYD. # PORTS	OUTLET DIAMETER (INCHES)	NOZZLE COEFFICIENT	DIFFUSER TYPE	DIFFUSER COEFFICIENT	PITOT READING (psi)	PITOT FLOW (USGPM)	FLOW METER (USGPM)
PB309	2	2.5	Round	LPD250	0.90	**25.5**	762	0
		2.5	Round	LPD250	0.90	**25.5**	762	0
					Total Flow (USGPM)		1525	0
					Total Flow (USGPM)		1525	

FIRE FLOW CHART



COMMENTS

OPERATOR	FMX	Jordan Whitlock
OPERATOR	FMX	Denis Kriventsev
OPERATOR		Region of Durham

ODG_FireFlowTestingReport_Pickering

"If we don't measure it, how do you manage it?"

FLOWMETRIX
INDU-TECH
PROCESS
WESTCAN

Fire Flow Testing Report

Residual Hydrant # **PB888**
NFPA Colour Code **BLUE**

DATE	September 8, 2021
TIME	11:00 AM
ADDRESS	1785 Pickering Pkwy
	Pickering, ON
SIZE-inches/mm	12 300
MATERIAL	PVC
CONTACT INFO	The Odan/Detech Group Inc.
	Mark Harris
	C: (905) 632-3811 ext.122
	E: mark@odandetech.com

RESIDUAL HYDRANT INFO.

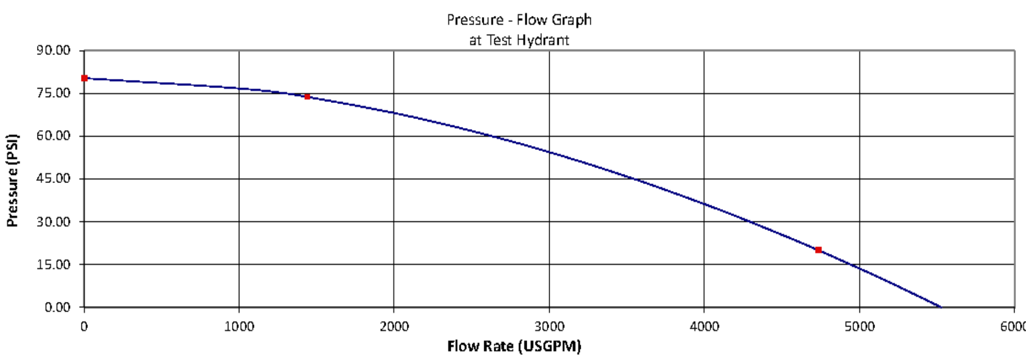
HYDRANT #	PB888	
N.F.P.A. COLOUR CODE	BLUE	
STATIC PRESSURE	80.3	psi
RESIDUAL PRESSURE	73.7	psi
PRESSURE DROP	6.7	psi
% PRESSURE DROP	8.3	% psi

Flow on Water Main At Test Hydrant - 20 psi 4735 USGPM

FLOW HYDRANT(S) INFO.

HYDRANT ASSET ID	HYD. # PORTS	OUTLET DIAMETER (INCHES)	NOZZLE COEFFICIENT	DIFFUSER TYPE	DIFFUSER COEFFICIENT	PITOT READING (psi)	PITOT FLOW (USGPM)	FLOW METER (USGPM)
PB308	2	2.5	Round	LPD250	0.90	**22.7**	720	0
		2.5	Round	LPD250	0.90	**22.7**	720	0
				Total Flow (USGPM)			1439	0
				Total Flow (USGPM)			1439	

FIRE FLOW CHART



COMMENTS

OPERATOR	FMX	Jordan Whitlock
OPERATOR	FMX	Denis Kriventsev
OPERATOR		Region of Durham

ODG_FireFlowTestingReport_Pickering "If we don't measure it, how do you manage it?"

APPENDIX D

Storm Sewer Design Sheets
Rational Formula Stormwater Calculations
Low Impact Development Strategies
Jellyfish Filter ETV Certification

FIGURE S-7

PROPOSED STORM TRIBUTARY PLAN

PICKERING BRIDGE LANDS INC.
PROPOSED MIX-USE DEVELOPMENT
1755 + 1805 PICKERING PARKWAY
PICKERING, ON

DATE:	PROJ. NO.:	SCALE:
JAN 2025	20266	NTS

ODAN·DETECH CONSULTING ENGINEERS

P:\2020\20266\2020\Drawings\01_Functional\02_FD_Production Dwgs\20266 PROP STM TRIB PLAN

STORM SEWER DESIGN

ODAN·DETECH CONSULTING ENGINEERS

5YEAR Mount Hope Storm $i = 1082.901/(Tc+6.007)^{0.837}$

100 YEAR Mount Hope Storm $i = 2096.425/(Tc+6.485)^{0.863}$

20266-PROPOSED MIXED-USE DEVELOPMENT
1755 & 1805 PICKERING PARKWAY
PICKERING, ON

DESIGN BY: P.T.
CHECKED BY: M.H.
DATE: January 20, 2025

TC= 10 Vmin = 0.80m/s Vmax = 5.0m/s
n = 0.013

SEWER DESIGN: PIPE ROUGHNESS: n = 0.013 For Manning's Equation
% of Full Flow: Peak Flow / Full Flow Capacity

LOCATION			STORMWATER ANALYSIS								STORM SEWER DATA					
Tributary ID No.	From Manhole	To Manhole	A Area (ha)	C Runoff Coeff.	A*C	Accumulated A*C	Time of Concentration (min)	Flow Time (min)	5-yr-Rainfall Intensity (mm/hr)	5-yr-Peak Flow (l/s)	Pipe Length (m)	Pipe Size (mm)	Pipe Slope (%)	Pipe Full Flow Capacity (l/s)	Pipe Full Flow Velocity (m/s)	5-yr.-Percent of Full Flow Capacity (%)
1	-	2	0.120	0.75	0.090	0.090	10.00	-	106.31	27	-	-	-	-	-	-
17	-	2	0.090	0.75	0.068	0.068	10.00	-	106.31	20	-	-	-	-	-	-
-	1	2	-	-	-	0.158	10.00	0.77	106.31	47	62.8	300	0.98	96	1.35	49%
26	-	3	0.030	0.75	0.023	0.023	10.00	-	106.31	7	-	-	-	-	-	-
-	2	3	-	-	-	0.180	10.77	0.26	102.20	51	29.4	375	1.43	210	1.90	24%
4	-	5	0.100	0.75	0.075	0.075	10.00	-	106.31	22	-	-	-	-	-	-
BLOCK 1	-	5	0.880	0.50	0.440	0.440	10.00	-	106.31	130	-	-	-	-	-	-
-	3	5	-	-	-	0.695	11.03	0.56	100.90	195	61.4	525	0.85	396	1.83	49%
6	-	7	0.120	0.75	0.090	0.090	10.00	-	106.31	27	-	-	-	-	-	-
BLOCK 2	-	7	1.290	0.50	0.645	0.645	10.00	-	106.31	191	-	-	-	-	-	-
-	5	7	-	-	-	1.430	11.59	0.83	98.21	390	96.6	600	0.8	549	1.94	71%
A1	EX MH S4	6	41.900	0.20	8.380	8.380	30.00	0.17	53.94	1257	41.0	1200	1.37	4563	4.03	28%
	6	7	0.000	-	-	8.380	30.17	0.19	53.73	1252	48.2	1200	1.54	4838	4.28	26%
24	-	7	0.100	0.75	0.075	0.075	10.00	-	106.31	22	-	-	-	-	-	-
BLOCK 4	-	7	0.890	0.50	0.445	0.445	10.00	-	106.31	132	-	-	-	-	-	-
-	4	7	-	-	-	0.520	10.00	0.11	106.31	154	13.5	375	1.54	218	1.97	71%
-	7	18	-	-	-	10.330	30.17	0.15	53.73	1543	32.5	1200	1.16	4199	3.71	37%
-	18	14	-	-	-	10.330	30.32	0.15	53.55	1538	33.8	1200	1.21	4289	3.79	36%
9	8	9	0.170	0.75	0.128	0.128	10.00	0.38	106.31	38	39.6	525	0.78	380	1.75	10%
5	-	12	0.110	0.75	0.083	0.083	10.00	-	106.31	24	-	-	-	-	-	-
-	9	12	-	-	-	0.210	10.38	1.06	104.26	61	89.7	525	0.5	304	1.40	20%
12A	-	11	0.400	0.25	0.100	0.100	10.00	-	106.31	30	-	-	-	-	-	-
BLOCK 7	-	11	1.016	0.50	0.508	0.508	10.00	-	106.31	150	-	-	-	-	-	-
13	-	11	0.160	0.50	0.080	0.080	10.00	-	106.31	24	-	-	-	-	-	-
-	B7	11	-	-	-	0.688	10.00	1.07	106.31	203	90.0	525	0.5	304	1.40	67%
14	-	12	0.120	0.75	0.090	0.090	10.00	-	106.31	27	-	-	-	-	-	-
BLOCK 5	-	12	0.920	0.50	0.460	0.460	10.00	-	106.31	136	-	-	-	-	-	-
-	11	12	-	-	-	1.238	11.07	0.52	100.72	347	51.1	600	0.56	459	1.63	75%
25	-	14	0.190	0.75	0.143	0.143	10.00	-	106.31	42	-	-	-	-	-	-
-	12	14	-	-	-	1.590	11.59	0.64	98.20	434	83.1	750	0.74	958	2.17	45%
27	-	14	0.170	0.75	0.128	0.128	10.00	-	106.31	38	-	-	-	-	-	-
20	-	14	0.100	0.75	0.075	0.075	10.00	-	106.31	22	-	-	-	-	-	-
-	13	14	-	-	-	0.203	10.00	0.68	106.31	60	68.7	300	1.5	118	1.68	51%
15	-	15	0.280	0.25	0.070	0.070	10.00	-	106.31	21	-	-	-	-	-	-
2	-	15	0.690	0.25	0.173	0.173	10.00	-	106.31	51	-	-	-	-	-	-
-	14	15	-	-	-	12.365	30.46	0.28	53.36	1834	60.9	1200	1.14	4163	3.68	44%
23	-	16	0.190	0.75	0.143	0.143	10.00	-	106.31	42	-	-	-	-	-	-
BLOCK 3	-	16	0.510	0.50	0.255	0.255	10.00	-	106.31	75	-	-	-	-	-	-
BLOCK 6	-	16	0.870	0.50	0.435	0.435	10.00	-	106.31	129	-	-	-	-	-	-
-	15	16	-	-	-	13.198	30.74	0.17	53.03	1946	40.5	1200	1.26	4376	3.87	44%
-	16	17	-	-	-	13.198	30.91	0.07	52.82	1938	14.8	1200	1.12	4126	3.65	47%
EX-RD	-	4	0.891	0.75	0.668	0.668	10.00	-	106.31	197	-	-	-	-	-	-
EX-LOT	-	4	4.412	0.75	3.309	3.309	10.00	-	106.31	978	-	-	-	-	-	-
-	17	4	-	-	-	17.175	30.98	0.16	52.74	2518	29.4	1200	0.78	3443	3.04	73%
-	4	5	-	-	-	17.175	31.14	0.39	52.55	2509	71.8	1200	0.78	3443	3.04	73%
-	5	6	-	-	-	17.175	31.54	0.96	52.09	2487	140.7	1200	0.5	2757	2.44	90%

SANITARY FLOW CALCULATIONS

SCENARIO: Full Phase - JANUARY 2025

Sanitary discharge calculations as per Regional Municipality of Durham *Design Specifications For Sanitary Sewers April 2017.*

LOT AREA (ha)= **9.484**
TOTAL Approx. Number of Residential Units = **5,298**

UPDATED MARCH 2022

ODAN·DETECH CONSULTING ENGINEERS

Block	Number of Units	GROSS FLOOR AREA (m^2)	TOTAL POPULATION	TOTAL DAILY FLOW (LITERS)	AVERAGE DAILY FLOW (L/sec)	PEAKING FACTOR, K_H	TOTAL FLOW FROM LAND USE (L/sec)
Block 1 Using 2.5 Persons/Unit	678		1,695	616,980	7.1	3.64	26.00
Block 2 Using 2.5 Persons/Unit	1090		2,725	991,900	11.5	3.48	39.92
Block 3 Using 2.5 Persons/Unit	446		1,115	405,860	4.7	3.77	17.70
Block 4 Using 2.5 Persons/Unit	1022		2,555	930,020	10.8	3.50	37.68
Block 5 Using 2.5 Persons/Unit	641		1,603	583,310	6.8	3.66	24.70
Block 6 Using 2.5 Persons/Unit	762		1,905	693,420	8.0	3.60	28.91
Block 7 Using 2.5 Persons/Unit	659		1,648	599,690	6.9	3.65	25.33
Total Residential	**5,298**		**13,245**	4,821,180	55.8	2.83	**158.06**
Block 1		1,669		30,042	0.3	1	0.348
Block 2		1,006		18,108	0.2	1	0.210
Block 3		920		16,560	0.2	1	0.192
Block 4		696		12,528	0.1	1	0.145
Block 5		665		11,970	0.1	1	0.139
Block 6		21,737		391,266	4.5	1	4.529
Total Non-Residential		**26,693**		480,474	5.6	1	5.561

Q = (MqP/86400) + A * I (L/sec)

Q1= total flow from Residential Land Use (L/sec)
Q2= total flow from Commercial Land Use (L/sec)
Qinfil = total flow from infiltration (L/sec)
Qtot = total flow (Land use + infiltration)

V1= Total Volume from Land Use in liters

V1=	5,301,654	Q1=	158.06
		Q2=	5.56
		Qinfil	2.47
		Qtot	166.09

where : P is population
q(res) = 364 L/cap/day
q(ICI) = 180,000 L/floor hectres/day

A = gross site area
i = 0.26 L/sec/ha (infiltration rate)
Peaking Factor $K_H = 1 + [14 / (4 + (P/1000)^{1/2})]$

Modified Rational Method

Project:	1755 & 1805 Pickering PKWY	Date:	1/15/2025
Project No.:	20266		
Municipality:	Pickering		
Catchment No.	Block 1		

Area (ha):	0.880		100-year Rainfall	
Runoff Coefficient:	0.500		Intensity (I) :	A/(T+B)^C
100-Yr Runoff Coefficient:		0.900	A:	2096.43
*Target Flow (m3/s):	0.105	Note: Adjusted to Orifice	B:	6.485
(5-yr Allowable)	0.130		C:	0.863

Initial Time: 10 min
Increment: 5 min

Time	I	Peak Flow	Runoff Vol.	Discharge Vol.	Storage
min	mm/hr	m3/s	m3	m3	m3
10	186.7	0.411	246.6	63	183.6
15	148.5	0.327	294.3	94.5	199.8
20	**124.0**	**0.273**	**327.6**	**126**	**201.6**
25	106.8	0.235	352.8	157.5	195.3
30	94.1	0.207	372.8	189	183.8
35	84.2	0.185	389.3	220.5	168.8
40	76.3	0.168	403.2	252	151.2
45	69.9	0.154	415.4	283.5	131.9
50	64.5	0.142	426.0	315	111.0
55	59.9	0.132	435.6	346.5	89.1
60	56.0	0.123	444.2	378	66.2
65	52.6	0.116	452.0	409.5	42.5
70	49.7	0.109	459.2	441	18.2
75	47.0	0.104	465.8	472.5	-6.7
80	44.7	0.098	472.0	504	-32.0

* Target Flow is calculated based on 5-year storm event-Rational Method

I5= 1082.901/(T+6.007)^0.837
Tc= 10 min
I5=106.4 mm/hr.

ORIFICE DISCHARGE CALCULATOR - SWM TANK - BLK 1

This program calculates the discharge from a circular orifice when given elevations and orifice diameters by the user.

Discharge based on orifice equ.: Q = CA x sqrt(2gh)

Tank Area
137.5 m2

Q-allowable
130 l/sec

Stall Area	Stalls	Total Area
13.75	10	137.5

Orifice Diameter = 0.1750 m
Orifice Area = 0.0241 m2
Discharge Coeff. = 0.8000

	Head (m)	Discharge(m3/s)	Discharge (L/s)	Vol (m3)	
	0	0.0000	0	0	
	0.20	0.0381	38	28	
	0.40	0.0539	54	55	
	0.80	0.0762	76	110	
	1.00	0.0852	85	138	
100-year	1.52	0.1051	105	209	
Top of Tank (free board)	1.80	0.1144	114	248	
				@1.5x	@x1.5 Area
				314	206

Modified Rational Method

Project:	1755 & 1805 Pickering PKWY	Date:	1/15/2025
Project No.:	20266		
Municipality:	Pickering		
Catchment No.	Block 2		

Area (ha):	1.290		100-year Rainfall	
Runoff Coefficient:	0.500		Intensity (I) :	A/(T+B)^C
100-Yr Runoff Coefficient:		0.900	A:	2096.43
*Target Flow (m3/s):	0.144	Note: Adjusted to Orifice	B:	6.485
(5-yr Allowable)	0.190		C:	0.863

Initial Time: 10 min.
Increment: 5 min

Time	I	Peak Flow	Runoff Vol.	Discharge Vol.	Storage
min	mm/hr	m3/s	m3	m3	m3
10	186.7	0.603	361.5	86.4	275.1
15	148.5	0.479	431.5	129.6	301.9
20	**124.0**	**0.400**	**480.3**	**172.8**	**307.5**
25	106.8	0.345	517.1	216	301.1
30	94.1	0.304	546.4	259.2	287.2
35	84.2	0.272	570.6	302.4	268.2
40	76.3	0.246	591.1	345.6	245.5
45	69.9	0.226	608.9	388.8	220.1
50	64.5	0.208	624.5	432	192.5
55	59.9	0.193	638.5	475.2	163.3
60	56.0	0.181	651.1	518.4	132.7
65	52.6	0.170	662.6	561.6	101.0
70	49.7	0.160	673.1	604.8	68.3
75	47.0	0.152	682.8	648	34.8
80	44.7	0.144	691.9	691.2	0.7

* Target Flow is calculated based on 5-year storm event-Rational Method

I5= 1082.901/(T+6.007)^0.837
Tc= 10 min
I5=106.4 mm/hr.

ORIFICE DISCHARGE CALCULATOR - SWM TANK - BLK 2

This program calculates the discharge from a circular orifice when given elevations and orifice diameters by the user.

Discharge based on orifice equ.: Q = CA x sqrt(2gh)

Tank Area
192.5 m2

Stall Area	Stalls	Total Area
13.75	14	192.5

Q-allowable
190 l/sec

Orifice Diameter = 0.2000 m
Orifice Area = 0.0314 m2
Discharge Coeff. = 0.8000

	Head (m)	Discharge(m3/s)	Discharge (L/s)	Vol (m3)		
	0	0.0000	0	0		
	0.20	0.0498	50	39		
	0.40	0.0704	70	77		
	0.80	0.0996	100	154		
	1.00	0.1113	111	193	x1.5	x1.5 Area
100-year	1.67	0.1439	144	321	482.213	288.75
Top of Tank (free board)	2.00	**0.1574**	157	385		

@1.5x	@x1.5 Area
482	289

Modified Rational Method

Project:	1755 & 1805 Pickering PKWY	Date:	1/15/2025
Project No.:	20266		
Municipality:	Pickering		
Catchment No.	Block 3		

Area (ha):	0.510		100-year Rainfall	
Runoff Coefficient:	0.500		Intensity (I) :	A/(T+B)^C
100-Yr Runoff Coefficient:		0.900	A:	2096.43
*Target Flow (m3/s):	0.055	Note: Adjusted to Orifice	B:	6.485
(5-yr Allowable)	0.190		C:	0.863

Initial Time: 10 min
Increment: 5 min

Time	I	Peak Flow	Runoff Vol.	Discharge Vol.	Storage
min	mm/hr	m3/s	m3	m3	m3
10	186.7	0.238	142.9	33	109.9
15	148.5	0.190	170.6	49.5	121.1
20	**124.0**	**0.158**	**189.9**	**66**	**123.9**
25	106.8	0.136	204.4	82.5	121.9
30	94.1	0.120	216.0	99	117.0
35	84.2	0.107	225.6	115.5	110.1
40	76.3	0.097	233.7	132	101.7
45	69.9	0.089	240.7	148.5	92.2
50	64.5	0.082	246.9	165	81.9
55	59.9	0.076	252.4	181.5	70.9
60	56.0	0.072	257.4	198	59.4
65	52.6	0.067	261.9	214.5	47.4
70	49.7	0.063	266.1	231	35.1
75	47.0	0.060	270.0	247.5	22.5
80	44.7	0.057	273.5	264	9.5

* Target Flow is calculated based on 5-year storm event-Rational Method

I5= 1082.901/(T+6.007)^0.837
Tc= 10 min
I5=106.4 mm/hr.

ORIFICE DISCHARGE CALCULATOR - SWM TANK - BLK 3

This program calculates the discharge from a circular orifice when given elevations and orifice diameters by the user.

Discharge based on orifice equ.: Q = CA x sqrt(2gh)

Tank Area
82.5 m2

Stall Area	Stalls	Total Area
13.75	6	82.5

Q-allowable
75 l/sec

Orifice Diameter = 0.1250 m
Orifice Area = 0.0123 m2
Discharge Coeff. = 0.8000

	Head (m)	Discharge(m3/s)	Discharge (L/s)	Vol (m3)		
	0	0.0000	0	0		
	0.20	0.0194	19	17		
	0.40	0.0275	28	33		
	0.80	0.0389	39	66		
	1.00	0.0435	43	83		
	1.20	0.0476	48	99	x1.5	x1.5 Area
100-year	1.59	0.0548	55	131	196.763	123.75
Top of Tank (free board)	1.90	**0.0599**	60	157		

@1.5x	@x1.5 Area
196.763	123.75

Modified Rational Method

Project:	1755 & 1805 Pickering PKWY	Date:	1/15/2025
Project No.:	20266		
Municipality:	Pickering		
Catchment No.	Block 4		

Area (ha):	0.890		100-year Rainfall	
Runoff Coefficient:	0.500		Intensity (I) :	A/(T+B)^C
100-Yr Runoff Coefficient:	0.900		A:	2096.43
*Target Flow (m3/s):	0.104	Note: Adjusted to Orifice	B:	6.485
(5-yr Allowable)	0.131		C:	0.863

Initial Time:	10 min
Increment:	5 min

Time	I	Peak Flow	Runoff Vol.	Discharge Vol.	Storage
min	mm/hr	m3/s	m3	m3	m3
10	186.7	0.416	249.4	62.4	187.0
15	148.5	0.331	297.7	93.6	204.1
20	**124.0**	**0.276**	**331.4**	**124.8**	**206.6**
25	106.8	0.238	356.8	156	200.8
30	94.1	0.209	377.0	187.2	189.8
35	84.2	0.187	393.7	218.4	175.3
40	76.3	0.170	407.8	249.6	158.2
45	69.9	0.156	420.1	280.8	139.3
50	64.5	0.144	430.9	312	118.9
55	59.9	0.133	440.5	343.2	97.3
60	56.0	0.125	449.2	374.4	74.8
65	52.6	0.117	457.1	405.6	51.5
70	49.7	0.111	464.4	436.8	27.6
75	47.0	0.105	471.1	468	3.1
80	44.7	0.099	477.3	499.2	-21.9

* Target Flow is calculated based on 5-year storm event-Rational Method

I5= 1082.901/(T+6.007)^0.837
Tc= 10 min
I5=106.4 mm/hr.

ORIFICE DISCHARGE CALCULATOR - SWM TANK - BLK 4

This program calculates the discharge from a circular orifice when given elevations and orifice diameters by the user.

Discharge based on orifice equ.: Q = CA x sqrt(2gh)

Tank Area 137.5 m2

Stall Area	Stalls	Total Area
13.75	10	137.5

Q-allowable 131 l/sec

Orifice Diameter = 0.1750 m
Orifice Area = 0.0241 m2
Discharge Coeff. = 0.8000

	Head (m)	Discharge(m3/s)	Discharge (L/s)	Vol (m3)	x1.5	x1.5 Area
	0	0.0000	0	0		
	0.20	0.0381	38	28		
	0.40	0.0539	54	55		
	0.80	0.0762	76	110		
	1.00	0.0852	85	138		
	1.20	0.0934	93	165		
100-year	1.49	0.1040	104	205	307.313	206.25
Top of Tank (free board)	1.80	0.1144	114	248		

@1.5x	@x1.5 Area
307.313	206.25

Modified Rational Method

Project:	1755 & 1805 Pickering PKWY	Date:	1/15/2025
Project No.:	20266		
Municipality:	Pickering		
Catchment No.	Block 5		

Area (ha):	0.920		100-year Rainfall	
Runoff Coefficient:	0.500		Intensity (I) :	A/(T+B)^C
100-Yr Runoff Coefficient:		0.900	A:	2096.43
*Target Flow (m3/s):	0.135	Note: Adjusted to Orifice	B:	6.485
(5-yr Allowable)	0.136		C:	0.863

Initial Time: 10 min
Increment: 5 min

Time	I	Peak Flow	Runoff Vol.	Discharge Vol.	Storage
min	mm/hr	m3/s	m3	m3	m3
10	186.7	0.430	257.8	81.12	176.7
15	**148.5**	**0.342**	**307.7**	**121.68**	**186.0**
20	124.0	0.285	342.5	162.24	180.3
25	106.8	0.246	368.8	202.8	166.0
30	94.1	0.216	389.7	243.36	146.3
35	84.2	0.194	406.9	283.92	123.0
40	76.3	0.176	421.6	324.48	97.1
45	69.9	0.161	434.2	365.04	69.2
50	64.5	0.148	445.4	405.6	39.8
55	59.9	0.138	455.4	446.16	9.2
60	56.0	0.129	464.4	486.72	-22.4
65	52.6	0.121	472.5	527.28	-54.7
70	49.7	0.114	480.0	567.84	-87.8
75	47.0	0.108	487.0	608.4	-121.4
80	44.7	0.103	493.4	648.96	-155.5

* Target Flow is calculated based on 5-year storm event-Rational Method

I5= 1082.901/(T+6.007)^0.837
Tc= 10 min
I5=106.4 mm/hr.

ORIFICE DISCHARGE CALCULATOR - SWM TANK - BLK 5

This program calculates the discharge from a circular orifice when given elevations and orifice diameters by the user.

Discharge based on orifice equ.: Q = CA x sqrt(2gh)

Tank Area
137.5 m2

Stall Area	Stalls	Total Area
13.75	10	137.5

Q-allowable
136 l/sec

Orifice Diameter = 0.2000 m
Orifice Area = 0.0314 m2
Discharge Coeff. = 0.8000

	Head (m)	Discharge(m3/s)	Discharge (L/s)	Vol (m3)		
	0	0.0000	0	0		
	0.20	0.0498	50	28		
	0.40	0.0704	70	55		
	0.80	0.0996	100	110		
	1.00	0.1113	111	138	x1.5	x1.5 Area
100-year	1.48	0.1352	135	203	304.219	206.25
Top of Tank (free board)	1.80	0.1494	149	248		

@1.5x	@x1.5 Area
304	206

Modified Rational Method

Project:	1755 & 1805 Pickering PKWY	Date:	1/15/2025
Project No.:	20266		
Municipality:	Pickering		
Catchment No.	Block 6		

Area (ha):	0.870		100-year Rainfall	
Runoff Coefficient:	0.500		Intensity (I) :	A/(T+B)^C
100-Yr Runoff Coefficient:	0.900		A:	2096.43
*Target Flow (m3/s):	0.121	Note: Adjusted to Orifice	B:	6.485
(5-yr Allowable)	0.128		C:	0.863

Initial Time: 10 min
Increment: 5 min

Time	I	Peak Flow	Runoff Vol.	Discharge Vol.	Storage
min	mm/hr	m3/s	m3	m3	m3
10	186.7	0.406	243.8	72.84	171.0
15	**148.5**	**0.323**	**291.0**	**109.26**	**181.7**
20	124.0	0.270	323.9	145.68	178.2
25	106.8	0.233	348.8	182.1	166.7
30	94.1	0.205	368.5	218.52	150.0
35	84.2	0.183	384.8	254.94	129.9
40	76.3	0.166	398.7	291.36	107.3
45	69.9	0.152	410.6	327.78	82.9
50	64.5	0.140	421.2	364.2	57.0
55	59.9	0.130	430.6	400.62	30.0
60	56.0	0.122	439.1	437.04	2.1
65	52.6	0.115	446.9	473.46	-26.6
70	49.7	0.108	454.0	509.88	-55.9
75	47.0	0.102	460.5	546.3	-85.8
80	44.7	0.097	466.6	582.72	-116.1

* Target Flow is calculated based on 5-year storm event-Rational Method

I5= 1082.901/(T+6.007)^0.837
Tc= 10 min
I5=106.4 mm/hr.

ORIFICE DISCHARGE CALCULATOR - SWM TANK - BLK 6

This program calculates the discharge from a circular orifice when given elevations and orifice diameters by the user.

Discharge based on orifice equ.: Q = CA x sqrt(2gh)

Tank Area
165 m2

Stall Area	Stalls	Total Area
13.75	12	165

Q-allowable
128 l/sec

Orifice Diameter = 0.2000 m
Orifice Area = 0.0314 m2
Discharge Coeff. = 0.8000

	Head (m)	Discharge(m3/s)	Discharge (L/s)	Vol (m3)	x1.5	x1.5 Area
	0	0.0000	0	0		
	0.20	0.0498	50	33		
	0.40	0.0704	70	66		
	0.80	0.0996	100	132		
	1.00	0.1113	111	165		
100-year	**1.19**	**0.1214**	**121**	**196**	294.525	247.5
Top of Tank (free board)	1.50	0.1363	136	248		

@1.5x	@x1.5 Area
295	248

Modified Rational Method

Project:	1755 & 1805 Pickering PKWY	Date:	1/15/2025
Project No.:	20266		
Municipality:	Pickering		
Catchment No.	Block 7		

Area (ha):	1.020		100-year Rainfall	
Runoff Coefficient:	0.500		Intensity (I) :	A/(T+B)^C
100-Yr Runoff Coefficient:	0.900		A:	2096.43
*Target Flow (m3/s):	0.118	Note: Adjusted to Orifice	B:	6.485
(5-yr Allowable)	0.151		C:	0.863
Initial Time:	10 min			
Increment:	5 min			

Time	I	Peak Flow	Runoff Vol.	Discharge Vol.	Storage
min	mm/hr	m3/s	m3	m3	m3
10	186.7	0.476	285.9	70.8	215.1
15	148.5	0.379	341.2	106.2	235.0
20	**124.0**	**0.316**	**379.8**	**141.6**	**238.2**
25	106.8	0.273	408.9	177	231.9
30	94.1	0.240	432.1	212.4	219.7
35	84.2	0.215	451.2	247.8	203.4
40	76.3	0.195	467.4	283.2	184.2
45	69.9	0.178	481.4	318.6	162.8
50	64.5	0.165	493.8	354	139.8
55	59.9	0.153	504.9	389.4	115.5
60	56.0	0.143	514.8	424.8	90.0
65	52.6	0.134	523.9	460.2	63.7
70	49.7	0.127	532.2	495.6	36.6
75	47.0	0.120	539.9	531	8.9
80	44.7	0.114	547.1	566.4	-19.3

* Target Flow is calculated based on 5-year storm event-Rational Method

I5= 1082.901/(T+6.007)^0.837
Tc= 10 min
I5=106.4 mm/hr.

ORIFICE DISCHARGE CALCULATOR - SWM TANK - BLK 7

This program calculates the discharge from a circular orifice when given elevations and orifice diameters by the user.

Discharge based on orifice equ.: Q = CA x sqrt(2gh)

Tank Area
220 m2

Stall Area	Stalls	Total Area
13.75	16	220

Q-allowable
151 l/sec

Orifice Diameter = 0.2000 m
Orifice Area = 0.0314 m2
Discharge Coeff. = 0.8000

	Head (m)	Discharge(m3/s)	Discharge (L/s)	Vol (m3)	
	0	0.0000	0	0	
	0.20	0.0498	50	44	
	0.40	0.0704	70	88	
	0.80	0.0996	100	176	
	1.00	0.1113	111	220	
100-year	1.13	0.1181	118	248	
Top of Tank (free board)	1.43	0.1331	133	315	
				@1.5x	@x1.5 Area
				371	330

Block #	Area	$Q_{5yr\ Pre}$	Volume $Q_{100\ Post}$	Volume Factor $Q_{100\ Post\ (x1.5)}$
Block 1	0.88	0.105	200	300
Block 2	1.29	0.144	302	453
Block 3	0.51	0.055	121	182
Block 4	0.89	0.104	204	306
Block 5	0.92	0.135	186	279
Block 6	0.87	0.121	182	273
Block 7	1.02	0.118	235	352

PRESERVING IMPORTANT HYDROLOGIC FEATURES AND FUNCTION

There are many features in the natural landscape that provide the important hydrologic functions of retention, detention, infiltration, and filtering of stormwater. These features include, but are not limited to; highly permeable soils, pocket wetlands, significant small (headwater) drainage features, riparian buffers, floodplains, undisturbed natural vegetation, and tree clusters. All areas of hydrologic importance should be delineated at the earliest stage in the development planning process.

STRATEGIES

1. ***Buffers provide filtration, infiltration, flood management, and bank stability benefits.*** Unlike stormwater ponds and other structural infrastructure, buffers are essentially a no capital cost and low maintenance form of "green" infrastructure. The benefits of buffers diminish when slopes are greater than 25%; therefore steep slopes should not be counted as buffer.
2. ***Preserve areas of undisturbed soil and vegetation cover.*** Typical construction practices, such as topsoil stripping and stockpiling, and site grading and compaction by construction equipment, can considerably reduce the infiltration capacity (and treatment capacity) of soils. During construction, natural heritage features and locations where stormwater infiltration practices will be constructed should be delineated and not subject to construction equipment or other vehicular traffic, nor stockpiling of topsoil.
3. ***Avoid development on permeable soils.*** Highly permeable soils (i.e., hydrologic soil groups A and B) function as important groundwater recharge areas. To the greatest extent possible, these areas should be preserved in an undisturbed condition or set aside for stormwater infiltration practices. Where avoiding development on permeable soils is not possible, stormwater management should focus on mitigation of reduced groundwater recharge through application of stormwater infiltration practices.
4. ***Preserve existing trees and, where possible, tree clusters.*** Mature stands of deciduous trees will intercept 10 to 20% of annual precipitation falling on them, and a stand of evergreens will intercept 15 to 40%. Preserving mature trees will provide immediate benefits in new developments, whereas newly planted trees will take 10 years or more to provide equivalent benefits. Tree clusters can be incorporated into parking lot interiors or perimeters, private lawns, open space areas, road buffers, and median strips. An uncompacted soil volume of 15 to 28 m3 is recommended to achieve a healthy mature tree with a long lifespan.

SITING AND LAYOUT OF DEVELOPMENT

The location and configuration of elements, such as streets, sidewalks, driveways, and buildings, within the framework of the natural heritage system provides many opportunities to reduce stormwater runoff.

STRATEGIES

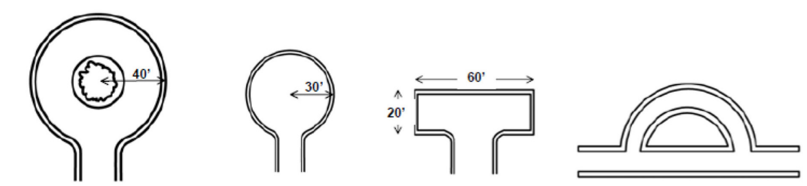
5. ***Fit the design to the terrain.*** Using the terrain and natural drainage as a design element will reduce the amount of clearing and grading required and the extent of necessary underground drainage infrastructure. This helps to preserve predevelopment drainage boundaries.
6. ***Use open space or clustered development.*** Clustering development increases the development density in less sensitive areas of the site while leaving the rest of the site as protected community open space. Some features of open space or clustered development are smaller lots, shared driveways, and shared parking. Clustered development also reduces the amount of impervious surfaces and stormwater runoff to be managed, reduces pressure on buffer areas, reduces the construction footprint, and provides more area and options for stormwater controls.
7. ***Use innovative street network designs.*** Certain roadway network designs (e.g., loops, cul-de-sacs, fused grids) create less impervious area than others. These layouts by themselves may not achieve the many goals of urban design. However, used in a hybrid form together or with other street patterns, they can meet multiple urban design objectives and reduce the necessary street area, thereby reducing the amount of impervious surfaces and stormwater runoff to be managed.
8. ***Reduce roadway setbacks and lot frontages.*** The lengths of setbacks and frontages are a determinant for the area of pavement, street, driveways, and walkways, needed to service a development. Municipal zoning regulations for setbacks and frontages have been found to be a significant influence on the production of stormwater runoff.

REDUCING IMPERVIOUS AREA

Many of the strategies described previously are primarily for the purpose of reducing impervious area on a macro scale. The following strategies provide examples of how to reduce impervious area on a micro or lot level scale.

STRATEGIES

9. ***Reduce street width.*** Streets constitute the largest percentage of impervious area and contribute proportionally to the urban runoff. Streets widths are sized for the free flow of traffic and movements of large emergency vehicles. In many cases, such as low density residential, these widths are oversized for the typical function of the street. Amending urban design standards to allow alternative, narrower street widths might be appropriate in some situations. There are a variety of ways to accommodate emergency vehicle movements and traffic flow on narrower streets, including alternative street parking configurations, vehicle pullout space, connected street networks, prohibiting parking near intersections, and reinforced turf or gravel edges.
10. ***Reduce building footprints.*** Reduce the building footprint by using taller multi-story buildings and taking advantage of opportunities to consolidate services into the same space.
11. ***Reduce parking footprints.*** Excess parking not only results in greater stormwater impacts and greater stormwater management costs but also adds unnecessary construction and maintenance costs and uses space that could be used for a revenue generating purpose.
    - Keep the number of parking spaces to the minimum required. Parking ratio requirements are often set to meet the highest hourly parking demand during the peak season. The parking space requirement should instead consider an average parking demand and other factors influencing demand like access to mass transit.
    - Take advantage of opportunities for shared parking. For example, businesses with daytime parking peaks can be paired with evening parking peaks, such as offices and a theatre, or land uses with weekday peak demand can be paired with weekend peak demand land uses, such as a school and church.
    - Reductions in impervious surface can also be found in the geometry of the parking lot. One way aisles when paired with angled parking will require less space than a two way aisle. Other reductions can be found in using unpaved end-of-stall overhangs, setting aside smaller stalls for compact vehicles, and configuring or overlapping common areas like fire lanes, collectors, loading, and drop off areas.
    - More costly approaches to reducing the parking footprint include parking structures or underground parking.
12. ***Consider alternative cul-de-sac designs.*** Using alternatives to the standard 15 metre radius cul-de-sac can further reduce the impervious area required to service each dwelling. Ways to reduce the impervious areas of cul-de-sacs include a landscaped or bioretention centre island, T-shaped turnaround, or by using a loop road instead.
13. ***Eliminate unnecessary sidewalks and driveways.*** A flexible design standard for sidewalks is recommended to allow for unnecessary sidewalks to be eliminated. Sidewalks that are not needed for pedestrian circulation or connectivity should be removed. Often sidewalks are only necessary on one side of the street. Driveway impervious area can be reduced through the use of shared driveways or alley accessed garages



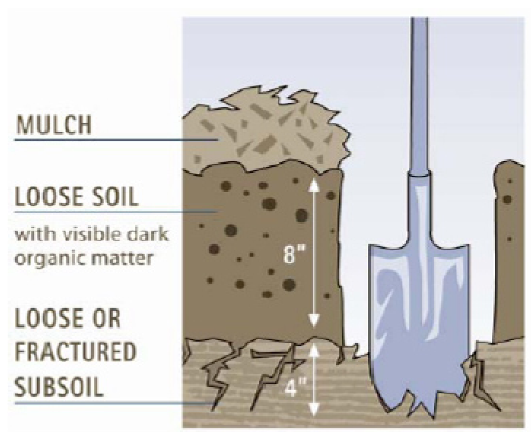
40 ft cul-de sac with landscaped island | 30 ft radius cul-de-sac | 60 by 20 ft T-shaped turnaround | Loop road

Source: CWP, 1998

	Square grid (Miletus, Houston, Portland, etc.)	Oblong grid (most cities with a grid)	Oblong grid 2 (some cities or in certain areas)	Loops (Subdivisions – 1950 to now)	Culs-de-sac (Radburn – 1932 to now)
Percentage of area for streets	36.0%	35.0%	31.4%	27.4%	23.7%
Percentage of buildable area	64.0%	65.0%	68.6%	72.6%	76.3%

Source: CMHC, 2002

Soil Amendment Guidelines



Soil amendment sizing criteria:

impervious area / soil area = 1
- use 100 mm compost, till to 300 - 450 mm depth

impervious area / soil area = 2
- use 200 mm compost, till to 300 - 450 mm depth

impervious area / soil area = 3
- use 300 mm compost, till to 450 - 600 mm depth

Compost should consist of well-aged (at least one year) leaf compost. Amended soil should have an organic content of 8-15% by weight or 30-40% by volume.

Source: Soils for Salmon, 2005

Open Drainage Applied in a Medium Density Neighbourhood

Source: U.S. EPA

USING NATURAL DRAINAGE SYSTEMS

Rather than collect and move stormwater rapidly to a centralized location for detention and treatment, the goal of these strategies is to take advantage of undisturbed vegetated areas and natural drainage patterns (e.g., small headwater drainage features). These strategies will extend runoff flow paths and slow down flow to allow soils and vegetation to treat and retain it. Using natural systems or green infrastructure is often more cost effective than traditional drainage systems, and they provide more ancillary benefits.

STRATEGIES

14. ***"Disconnect" impervious areas.*** Roof leaders or downspouts, parking lots, driveways, sidewalks, and patios should be disconnected from the storm sewer and directed towards stabilized pervious areas as sheet flow where possible. In cases of concentrated flow, the flow can be broken up with level spreaders or flow dissipating riprap. With the proper treatment, the landscaped areas of a site can accept runoff from impervious areas. Deep tilling or soil aeration is recommended for topsoil that has been replaced or compacted by construction equipment. Soil amendments can be applied to hydrologic soil group C and D soils to encourage runoff absorption. Use deep rooting vegetation in landscaped areas when possible which will maintain and possibly improve soil infiltration rate over time:
    - ***Undisturbed densely vegetated areas and buffers*** - A hydrologist and/or ecologist should be consulted before designing a site to drain to sensitive natural heritage features like pocket wetlands.
    - ***Landscaped and disturbed areas*** - With the proper treatment, the landscaped areas of the site can accept runoff from impervious areas. Deep tilling or soil aeration is recommended for topsoil that has been replaced or compacted by construction equipment. Soil amendments can be applied to hydrologic soil group C and D soils to encourage runoff absorption. Use deep rooting vegetation in landscaped areas when possible which will maintain and possibly improve the infiltration rates over time.
15. ***Preserve or create micro-topography.*** Undisturbed lands have a micro-topography of dips, hummocks and mounds which slow and retain runoff. Site grading smoothes out these topographic features. Micro-topography can be restored in areas of ornamental landscaping or naturalization.
16. ***Extend drainage flow paths.*** Slowing down flows and lengthening flow paths allow more opportunities for stormwater to be filtered and infiltrated. Extending the travel time can also delay and lower peak flows. Where suitable, flows should be conveyed using vegetated open channels (e.g., enhanced grass swales).

CVC/TRCA LOW IMPACT DEVELOPMENT
PLANNING AND DESIGN GUIDE - FACT SHEET

LOW IMPACT DEVELOPMENT SITE DESIGN STRATEGIES

TORONTO AND REGION Conservation for The Living City

CVC CREDIT VALLEY CONSERVATION

FOR FURTHER DETAILS SEE SECTION 3.2 OF THE CVC/TRCA LID SWM GUIDE

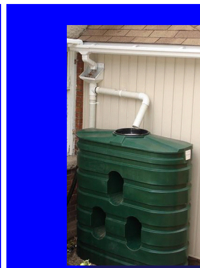
RAINWATER HARVESTING

CVC/TRCA LOW IMPACT DEVELOPMENT PLANNING AND DESIGN GUIDE - FACT SHEET

GENERAL DESCRIPTION

Rainwater harvesting is the process of intercepting, conveying and storing rainfall for future use. The rain that falls upon a catchment surface, such as a roof, is collected and conveyed into a storage tank. Storage tanks range in size from rain barrels for residential land uses (typically 190 to 400 litres in size), to large cisterns for industrial, commercial and institutional land uses. A typical pre-fabricated cistern can range from 750 to 40,000 litres in size.

With minimal pretreatment (e.g., gravity filtration or first-flush diversion), the captured rainwater can be used for outdoor non-potable water uses such as irrigation and pressure washing, or in the building to flush toilets or urinals. It is estimated that these applications alone can reduce household municipal water consumption by up to 55%. The capture and use of rainwater can, in turn, significantly reduce stormwater runoff volume and pollutant load. By providing a reliable and renewable source of water to end users, rainwater harvesting systems can also help reduce demand on municipal treated water supplies. This helps to delay expansion of treatment and distribution systems, conserve energy used for pumping and treating water and lower consumer water bills.



DESIGN GUIDANCE

CATCHMENT AREA
The catchment area is simply the surface from which rainfall is collected. Generally, roofs are the catchment area, although rainwater from low traffic parking lots and walkways, may be suitable for some non-potable uses (e.g., outdoor washing). The quality of the harvested water will vary according to the type of catchment area and material from which it is constructed. Water harvested from parking lots, walkways and certain types of roofs, such as asphalt shingle, tar and gravel, and wood shingle roofs, should only be used for irrigation or toilet flushing due to potential for contamination with toxic compounds.

COLLECTION AND CONVEYANCE SYSTEM
The collection and conveyance system consists of the eavestroughs, downspouts and pipes that channel runoff into the storage tank. Eavestroughs and downspouts should be designed with screens to prevent large debris from entering the storage tank. For dual use cisterns (used year-round for both outdoor and indoor uses), the conveyance pipe leading to the cistern should be buried at a depth no less than the local maximum frost penetration depth and have a minimum 1% slope. If this is not possible, conveyance pipes should either be located in a heated indoor environment (e.g., garage, basement) or be insulated or equipped with heat tracing to prevent freezing. All connections between downspouts, conveyance pipes and the storage tank must prevent entry of small animals or insects into the storage tank.

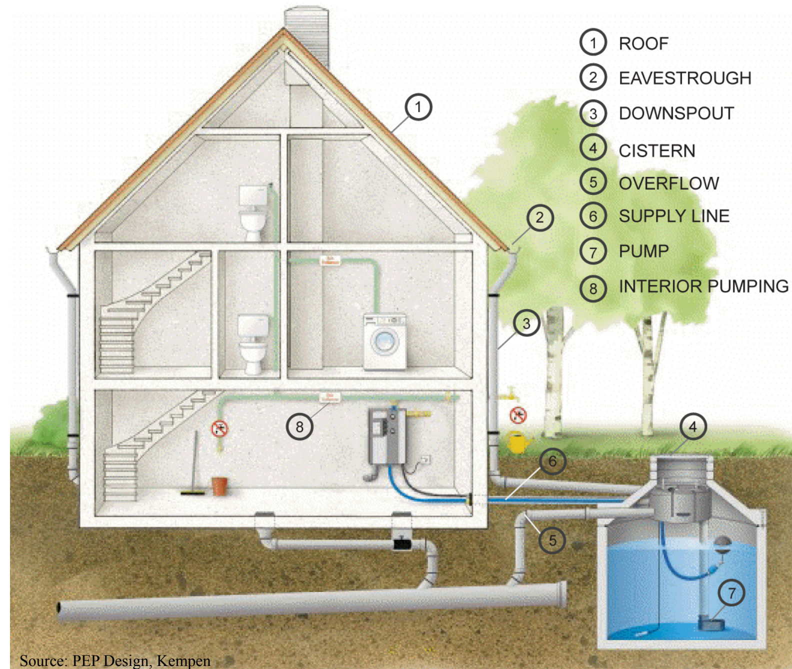
PRE-TREATMENT
Pretreatment is needed to remove debris, dust, leaves, and other debris that accumulates on roofs and prevents clogging within the rainwater harvesting system. For dual use cisterns that supply water for irrigation and toilet flushing only, filtration or first-flush diversion pretreatment is recommended. To prevent ice accumulation and damage during winter, first-flush diverters or in-ground filters should be in a temperature controlled environment, buried below the local frost penetration depth, insulated or equipped with heat tracing.

STORAGE TANKS
The storage tank is the most important and typically the most expensive component of a rainwater harvesting system. The required size of storage tank is dictated by several variables: rainfall and snowfall frequencies and totals, the intended use of the harvested water, the catchment surface area, aesthetics, and budget. In the Greater Toronto Area, an initial target for sizing the storage tank could be the predicted rainwater usage over a 10 to 12 day period.

DISTRIBUTION SYSTEM
Most distribution systems are gravity fed or operated using pumps to convey harvested rainwater from the storage tank to its final destination. Typical outdoor systems use gravity to feed hoses via a tap and spigot. For underground cisterns, a water pump is needed. Indoor systems usually require a pump, pressure tank, back-up water supply line and backflow preventer. The typical pump and pressure tank arrangement consists of a multistage centrifugal pump, which draws water out of the storage tank into the pressure tank, where it is stored for distribution.

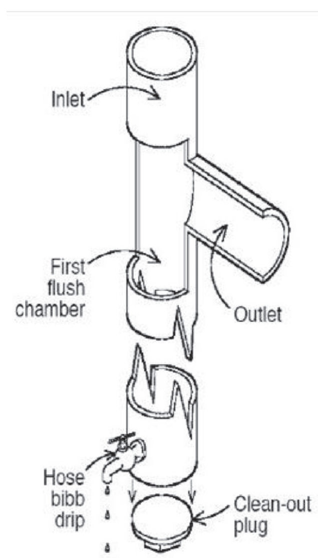
OVERFLOW SYSTEM
An overflow system must be included in the design. Overflow pipes should have a capacity equal to or greater than the inflow pipe(s). The overflow system may consist of a conveyance pipe from the top of the cistern to a pervious area down gradient of the storage tank, where suitable grading exists. The overflow discharge location should be designed as simple downspout disconnection to a pervious area, vegetated filter strip, or grass swale. The overflow conveyance pipe should be screened to prevent small animals and insects from entering. Where site grading does not permit overflow discharge to a pervious area, the conveyance pipe may either be indirectly connected to a storm sewer (discharge to an impervious area connected to a storm sewer inlet) or directly connected to a storm sewer with incorporation of a backflow preventer.

ACCESS AND MAINTENANCE
For underground cisterns, a standard size manhole opening should be provided for maintenance purposes. This access point should be secured with a lock to prevent unwanted access.

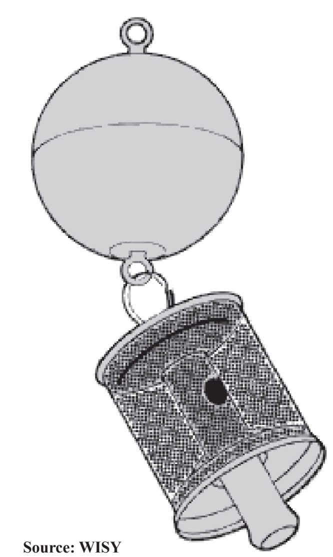


Source: PEP Design, Kempen

OVERVIEW



FIRST FLUSH DIVERTER



Source: WISY

FLOATING SUCTION FILTER

ABILITY TO MEET SWM OBJECTIVES

BMP	Water Balance Benefit	Water Quality Improvement	Stream Channel Erosion Control Benefit
Rainwater Harvesting	Yes - magnitude depends on water usage	Yes - size for the water quality storage requirement	Partial - can be used in series with other practices

GENERAL SPECIFICATIONS

Component	Specification	Quantity
Eavestroughs and Downspouts	Materials commonly used for eavestroughs and downspouts include polyvinylchloride (PVC) pipe, vinyl, aluminum and galvanized steel. Lead should not be used as solder as rainwater can dissolve the lead and contaminate the water supply.	Length of eavestroughs and downspouts is determined by the size and layout of the catchment and the location of the storage tanks.
Pretreatment	At least one of the following: • leaf and mosquito screens (1 mm mesh size); • first-flush diverter; • in-ground filter; • in-tank filter. Large tanks (10 m3 or larger) should have a settling compartment for removal of sediments.	1 per inlet to the collection system
Storage Tanks	• Materials used to construct storage tanks should be structurally sound. • Tanks should be installed in locations where native soils or building structures can support the load associated with the volume of stored water. • Storage tanks should be water tight and sealed using a water safe, non-toxic substance. • Tanks should be opaque to prevent the growth of algae • Previously used containers to be converted to rainwater storage tanks should be fit for potable water or food-grade products. • Cisterns above- or below ground must have a lockable opening of at least 450 mm diameter.	The size of the cistern(s) is determined during the design calculations.

Note: This table does not address indoor systems or pumps.

SITE CONSIDERATIONS

Available Space
Storage tanks can be placed underground, indoors, on roofs, or adjacent to buildings depending on intended uses of the rainwater.

Site Topography
Influences the placement of the storage tank and design of the distribution and overflow systems.

Soil
Underground cisterns should be placed on or in native, rather than fill soil.

Head
Rain barrels or above ground cisterns with gravity distribution systems should be sited up-gradient from landscaping areas to which rainwater is to be applied.

Pollution Hot Spot Runoff
Can be an effective BMP for roof runoff from sites where land uses or activities at ground level have the potential to generate highly contaminated runoff.

Winter Operation
Can be used throughout the winter if tanks are located below the local frost penetration depth or indoors.

Underground Utilities
Presence of underground utilities may constrain the location of underground storage tanks.

Plumbing Code
Code allows the use of harvested rainwater for toilet and urinal flushing, but systems require installation of backflow prevention devices.

Standing Water and Mosquitoes
If improperly managed, tanks can create habitat suitable for mosquito breeding, so screens should be placed on inlets and outlets to prevent entry.

Child Safety
Above and below ground cisterns with openings large enough for children to enter must have lockable covers.

Setback
Tanks should be water tight to avoid ponding or saturation of soils within 4 metres of building foundations.

Vehicle Loading
Underground tanks should be sited in areas without vehicular traffic.

Drawdown Between Storms
A suggested target for sizing the storage tank to ensure drawdown between storms is the predicted rainwater demand over a 10 to 12 day period.

OPERATION AND MAINTENANCE

Maintenance requirements for rainwater harvesting systems vary according to use. Systems that are used to provide supplemental irrigation water have relatively low maintenance requirements, while systems designed for indoor uses have much higher maintenance requirements. All rainwater harvesting system components should undergo regular inspections every six months during the spring and fall seasons to keep leaf screens, eavestroughs and downspouts free of leaves and other debris; check screens and patch holes or gaps; clean and maintain first flush diverters and filters, especially those on drip irrigation systems; inspect and clean storage tank lids, paying special attention to vents and screens on inflow and outflow spigots; and replace damaged system components as needed.

MOSQUITO CONTROL
If screening is not sufficient to deter mosquitoes, vegetable oil can be used to smother larvae. Alternatively mosquito dunks or pellets containing larvicide can be used.

WINTER OPERATION
Rainwater harvesting systems have a number of components that can be affected by freezing winter temperatures. For above-ground systems, winter-time operation may not be possible. Prior to the onset of freezing temperatures, above-ground systems should be disconnected and drained. For below-ground and indoor systems, downspouts and overflow components should be checked for ice blockages during snowmelt events.

FOR FURTHER DETAILS SEE SECTION 4.1 OF THE CVC/TRCA LID SWM GUIDE

GENERAL DESCRIPTION

Green roofs, also known as "living roofs" or "rooftop gardens", consist of a thin layer of vegetation and growing medium installed on top of a conventional flat or sloped roof. Green roofs are touted for their benefits to cities, as they improve energy efficiency, reduce urban heat island effects, and create greenspace for passive recreation or aesthetic enjoyment. They are also attractive for their water quality, water balance, and peak flow control benefits. The green roof acts like a lawn or meadow by storing rainwater in the growing medium and ponding areas. Excess rainfall enters underdrains and overflow points and is conveyed in the building drainage system. After the storm, a large portion of the stored water is evapotranspired by the plants, evaporates or slowly drains away.

There are two types of green roofs: intensive and extensive. Intensive green roofs contain greater than 15 cm depth of growing medium, can be planted with deeply rooted plants and are designed to handle pedestrian traffic. Extensive green roofs consist of a thinner growing medium layer (15 cm depth or less) with herbaceous vegetative cover. Guidance here focuses on extensive green roofs.

DESIGN GUIDANCE

ROOF STRUCTURE

The load bearing capacity of the roof structure must be sufficient to support the soil and plants of the green roof assembly, as well as the live load associated with maintenance staff accessing the roof. A green roof assembly weighing more than 80 kg per square metre, when saturated, requires consultation with a structural engineer. Green roofs may be installed on roofs with slopes up to 10%. As a fire resistance measure, non-vegetative materials, such as stone or pavers should be installed around all roof openings and at the base of all walls that contain openings.

WATERPROOFING SYSTEM

The first layer above the roof surface is a waterproofing membrane. Two common waterproofing techniques are monolithic and thermoplastic sheet membranes. Another option is a liquid-applied inverted roofing membrane assembly system in which the insulation is placed over the waterproofing, which adheres to the roof structure. An additional protective layer is generally placed on top of the membrane followed by a physical or chemical root barrier. Once the waterproofing system has been installed it should be fully tested prior to construction of the drainage system. Electronic leak detection systems should also be installed at this time if desired.

DRAINAGE LAYER

The drainage system includes a porous drainage layer and a geosynthetic filter mat to prevent fine growing medium particles from clogging the porous media. The drainage layer can be made up of gravels or recycled-polyethylene materials that are capable of water retention and efficient drainage. The depth of the drainage layer depends on the load bearing capacity of the roof structure and the stormwater retention requirements. The porosity of the drainage layer should be greater than or equal to 25%.

CONVEYANCE AND OVERFLOW

Once the porous media is saturated, all runoff (infiltrate or overland flow) should be directed to a traditional roof storm drain system. Landscaping style catch basins should be installed with the elevation raised to the desired ponding elevation. Alternately, roof drain flow restrictors can be used. Excess runoff can be directed through roof leaders to another stormwater BMP such as a rain barrel, soakaway, bioretention area, swale or simply drain to a pervious area.

GROWING MEDIUM

The growing medium is usually a mixture of sand, gravel, crushed brick, compost, or organic matter combined with soil. The medium ranges between 40 and 150 mm in depth and increases the roof load by 80 to 170 kg per square metre when fully saturated. The sensitivity of the receiving water to which the green roof ultimately drains should be taken into consideration when selecting the growing medium mix. Green roof growing media with less compost in the mix will leach less nitrogen and phosphorus. Low nutrient growing media also promotes the dominance of stress-tolerant native plants. Fertilizer applied to the growing medium during production and the period during which vegetation is becoming established should be coated controlled release fertilizer to reduce the risk of damage to vegetation and leaching of nutrients into overflowing runoff. Fertilizer applications should not exceed 5 g of nitrogen per square metre.

MODULAR SYSTEMS

Modular systems are trays of vegetation in a growing medium that are prepared and grown off-site and placed on the roof for complete coverage. There are also pre-cultivated vegetation blankets that are grown in flexible growing media structures, allowing them to be rolled out onto the green roof assembly. The advantage of these systems is that they can be removed for maintenance.

Green Rooftops are composed of:

- A roof structure capable of supporting the weight of a green roof system;
- A waterproofing system designed to protect the building and roof structure;
- A drainage layer that consists of a porous medium capable of water storage for plant uptake;
- A geosynthetic layer to prevent fine soil media from clogging the porous media;
- Soil with appropriate characteristics to support selected green roof plants;
- Plants with appropriate tolerance for harsh rooftop conditions and shallow rooting depths.

GREEN ROOF LAYERS
(Source: Great Lakes Water Institute)

ABILITY TO MEET SWM OBJECTIVES

BMP	Water Balance Benefit	Water Quality Improvement	Stream Channel Erosion Control Benefit
Green Rooftops	Yes	Yes	Yes

GENERAL SPECIFICATIONS

ASTM International released the following Green Roof standards in 2005:

- E2396-05 Standard Test Method for Saturated Water Permeability of Granular Drainage Media;
- E2397-05 Standard Determination of Dead Loads and Live Loads associated with Green Roof Systems;
- E2398-05 Standard test method for water capture and media retention of geocomposite drain layers for green roof systems;
- E2399-05 Standard Test Method for Maximum Media Density for Dead Load Analysis of Green Roof Systems; and
- E2400-06 Standard Guide for Selection, Installation, and Maintenance of Plants for Green Roof Systems.

Although the Ontario Building Code (2006) does not specifically address the construction of green roofs, requirements from the Building Code Act and Division B may apply to components of the construction. Further requirements from sections 2.4 and 2.11 of the 1997 Ontario Fire Code also require consideration.

COMMON CONCERNS

WATER DAMAGE TO ROOF

While failure of waterproofing elements may present a risk of water damage, a warranty can ensure that any damage to the waterproofing system will be repaired. Leak detection systems can also be installed to minimize or prevent water damage.

VEGETATION MAINTENANCE

Extreme weather conditions can have an impact on plant survival. Appropriate plant selection will help to ensure plant survival during weather extremes. Irrigation during the first year may be necessary in order to establish vegetation. Vegetation maintenance costs decrease substantially after the first two years.

COLD CLIMATE

Green roofs are a feasible BMP for cold climates. Snow can protect the vegetation layer and once thawed, will percolate into the growing medium and is either absorbed or drained away just as it would during a rain event. No seasonal adjustments in operation are needed.

COST

An analysis to determine cost effectiveness for a given site should include the roof lifespan, energy savings, stormwater management requirements, aesthetics, market value, tax and other municipal incentives. It is estimated that green roofs can extend the life of a roof structure by as long as 20 years by reducing exposure of the materials to sun and precipitation. They can also reduce energy demand by as much as 75% .

ON PRIVATE PROPERTY

Property owners or managers will need to be educated on their routine operation and 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 could be used to encourage property owners or managers to maintain existing practices.

CONSTRUCTION CONSIDERATIONS

An experienced professional green roof installer should install the green roof. The installer must work with the construction contractor to ensure that the waterproofing membrane installed is appropriate for use under a green roof assembly. Conventional green roof assemblies should be constructed in sections for easier inspection and maintenance access to the membrane and roof drains. Green roofs can be purchased as complete systems from specialized suppliers who distribute all the assembly components, including the waterproofing membrane. Alternatively, a green roof designer can design a customized green roof and specify suppliers for each component of the system.

OPERATION AND MAINTENANCE

- Green roof maintenance is typically greatest in the first two years as plants are becoming established. Vegetation should be monitored to ensure dense coverage. A warranty on the vegetation should be included in the construction contract.
- Regular operation of a green roof includes irrigation and leak detection. Watering should be based on actual soil moisture conditions as plants are designed to be drought tolerant. Electronic leak detection is recommended. This system, also used with traditional roofs, must be installed prior to the green roof.
- Ongoing maintenance should occur at least twice per year and should include weeding to remove volunteer seedlings of trees and shrubs and debris removal. In particular, the overflow conveyance system should be kept clear.

SITE CONSIDERATIONS

Roof Slope
Green roofs may be installed on roofs with slopes up to 10%.

Drainage Area & Runoff Volume
Green roofs are designed to capture precipitation falling directly onto the roof surface. They are not designed to receive runoff diverted from other source areas.

Structural Requirements
Load bearing capacity of the building structure and selected roof deck need to be sufficient to support the weight of the soil, vegetation and accumulated water or snow, and may also need to support pedestrians, concrete pavers, etc.

GREEN ROOFS

CVC/TRCA LOW IMPACT DEVELOPMENT
PLANNING AND DESIGN GUIDE - FACT SHEET

TORONTO AND REGION Conservation for The Living City

CVC CREDIT VALLEY CONSERVATION

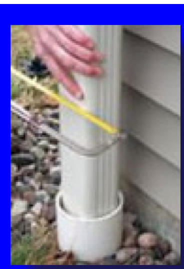
FOR FURTHER DETAILS SEE SECTION 4.2 OF THE CVC/TRCA LID SWM GUIDE

DOWNSPOUT DISCONNECTION

CVC/TRCA LOW IMPACT DEVELOPMENT
PLANNING AND DESIGN GUIDE - FACT SHEET

GENERAL DESCRIPTION

Simple downspout disconnection involves directing flow from roof downspouts to a pervious area that drains away from the building. This prevents stormwater from directly entering the storm sewer system or flowing across a "connected" impervious surface, such as a driveway, that drains to a storm sewer. Simple downspout disconnection requires a minimum flow path length across the pervious area of 5 metres.


DESIGN GUIDANCE

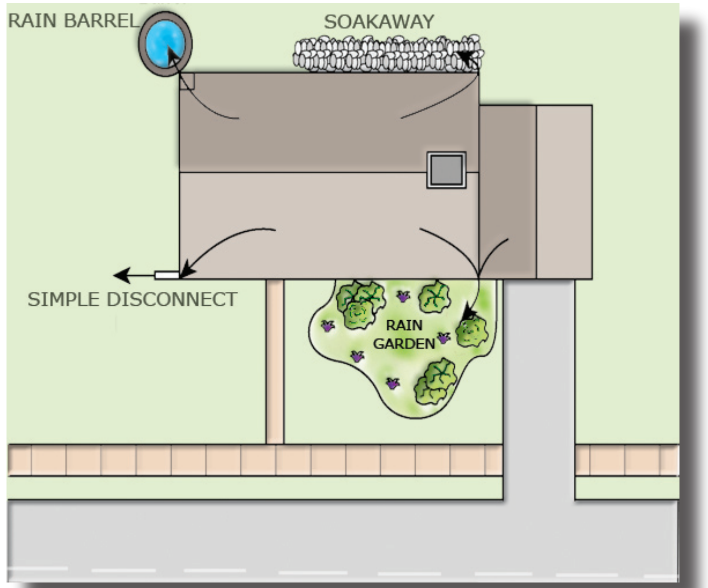
Roof downspout disconnections should meet the following criteria:

- Pervious areas used for downspout disconnection should be graded to have a slope of between 1 to 5%;
- Pervious areas should slope away from the building;
- The flow path length across the pervious area should be 5 metres or greater;
- The infiltration rate of soils in the pervious area should be 15 mm/hr or greater (i.e., hydraulic conductivity of 1x10-6 cm/s or greater);
- If infiltration rate of the soil in the pervious area is less than 15 mm/hr, it should be tilled to a depth of 300 mm and amended with compost to achieve a ratio of 8 to 15% organic content by weight or 30 to 40% by volume;
- If the flow path length across the pervious area is less than 5 metres and the soils are hydrologic soil group C or D, roof runoff should be directed to another LID practice (e.g., rainwater harvesting system, bioretention area, swale, soakaway, perforated pipe system);
- The total roof area contributing drainage to any single downspout discharge location should not exceed 100 square metres; and,
- A level spreading device (e.g., pea gravel diaphragm) or energy dissipating device (e.g., splash pad) should be placed at the downspout discharge location to distribute runoff as evenly as possible over the pervious area.

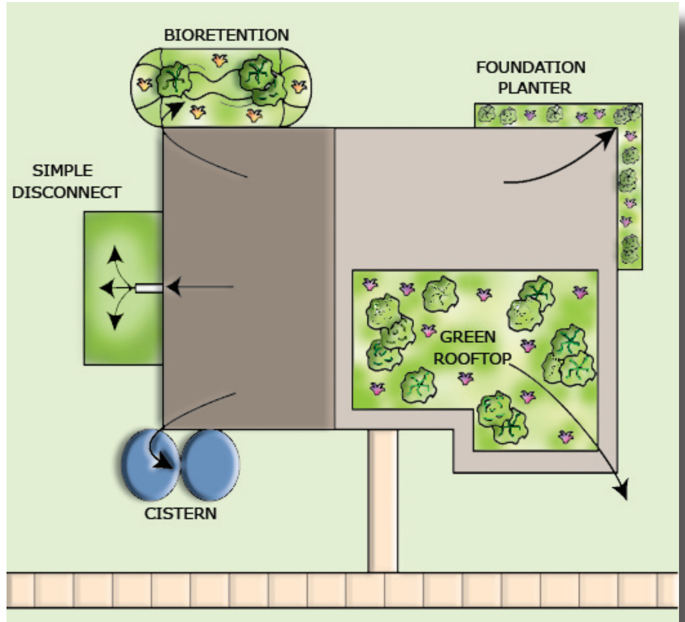
APPLICATIONS

There are many options for keeping roof runoff out of the storm sewer system. Some of the options are as follows:

- Simple roof downspout disconnection to a pervious area or vegetated filter strip, where sufficient flow path length across the pervious area and suitable soil conditions exist;
- Roof downspout disconnection to a pervious area or vegetated filter strip that has been tilled and amended with compost to improve soil infiltration rate and moisture storage capacity;
- Directing roof runoff to an enhanced grass swale, dry swale, bioretention area, soakaway or perforated pipe system;
- Directing roof runoff to a rainwater harvesting system (e.g., rain barrel or cistern) with overflow to a pervious area, vegetated filter strip, swale, bioretention area, soakaway or permeable pavement.



RESIDENTIAL



COMMERCIAL

ABILITY TO MEET SWM OBJECTIVES

BMP	Water Balance Benfit	Water Quality Improvement	Stream Channel Erosion Control Benefit
Downspout Disconnec-tion	Partial - depends on soil infiltration rate and length of flow path over the pervious area	Partial - depends on soil infiltration rate and length of flow path over the pervious area	Partial - depends on combination with other practices

Downspout disconnection is primarily a practice used to help achieve water balance benefits, although it can also contribute to water quality improvement. Very limited research has been conducted on the runoff reduction benefits of downspout disconnection, so initial estimates are drawn from research on filter strips, which operate in a similar manner. The research indicates that runoff reduction is a function of soil type, slope, vegetative cover and filtering distance. A conservative runoff reduction rate is 25% for hydrologic soil group (HSG) C and D soils and 50% for HSG A and B soils.* These values apply to disconnections that meet the feasibility criteria outlined in this section, and do not include any further runoff reduction due to the use of compost amendments along the filter path.

*Hydrologic soil group (HSG) classifications are based on the ability of the soil to transmit water. Soil groups are ranked from A to D. Group A soils are sandy, loamy sand, or sandy loam types. Group B soils are silt loam or loam types, Group C soils are sandy clay loam types. Group D soils are clay loam, silty clay loam, sandy clay, silty clay or clay types


OVERVIEW

SITE CONSIDERATIONS

Site Topography
Disconnected downspouts should discharge to a gradual slope that conveys runoff away from the building. The slope should be between 1% and 5%. Grading should discourage flow from reconnecting with adjacent impervious surfaces.

Water Table
Roof downspouts should only be disconnected where the minimum depth to the seasonally high water table is at least one (1) metre below the surface.

Pollution Hot Spot Runoff
Downspout disconnection can be used where land uses or activities at ground-level 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) as long as the roof runoff is kept separate from runoff from ground-level impervious surfaces.

COMMON CONCERNS

- ON PRIVATE PROPERTY
Property owners or managers will need to be educated on its function and maintenance needs, 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 could be used to encourage property owners or managers to maintain existing practices.
- STANDING WATER AND PONDING
Downspout disconnection is not intended to pond water, so any standing water should be infiltrated or evaporated within 24 hours of the end of each runoff event. If ponding for longer than 24 hours occurs, mitigation actions noted under Operation and Maintenance should be undertaken.

CONSTRUCTION CONSIDERATIONS

SOIL DISTURBANCE AND COMPACTION
Only vehicular traffic necessary for construction should be allowed on the pervious areas to which roof downspouts will be discharged. If vehicle traffic is unavoidable, then the pervious area should be tilled to a depth of 300 mm to loosen the compacted soil.

EROSION AND SEDIMENT CONTROL
If possible, construction runoff should be directed away from the proposed downspout discharge location. After the contributing drainage area and the downspout discharge location are stabilized and vegetated, erosion and sediment control structures can be removed.

OPERATION AND MAINTENANCE

Maintenance of disconnected downspouts will generally be no different than for lawns or landscaped areas. A maintenance agreement with property owners or managers may be required to ensure that downspouts remain disconnected and the pervious area remains pervious. For long-term efficacy, the pervious area should be protected from compaction. One method is to plant shrubs or trees along the perimeter of the pervious area to prevent traffic. On commercial sites, the pervious area should not be an area with high foot traffic. If ponding of water for longer than 24 hours occurs, the pervious area should be dethatched and aerated. If ponding persists, regrading or tilling to reverse compaction and/or addition of compost to improve soil moisture retention may be required.

TORONTO AND REGION Conservation for The Living City

CVC CREDIT VALLEY CONSERVATION

FOR FURTHER DETAILS SEE SECTION 4.3 OF THE CVC/TRCA LID SWM GUIDE

SOAKAWAYS, INFILTRATION TRENCHES AND CHAMBERS

CVC/TRCA LOW IMPACT DEVELOPMENT PLANNING AND DESIGN GUIDE - FACT SHEET

GENERAL DESCRIPTION

Soakaways are rectangular or circular excavations lined with geotextile fabric and filled with clean granular stone or other void forming material that receive runoff from a perforated pipe inlet and allow it to infiltrate into the native soil. They typically service individual lots and receive only roof and walkway runoff but can also be designed to receive overflows from rainwater harvesting systems. Soakaways can also be referred to as infiltration galleries, dry wells or soakaway pits.

Infiltration trenches are rectangular trenches lined with geotextile fabric and filled with clean granular stone or other void forming material. Like soakaways, they typically service an individual lot and receive only roof and walkway runoff. This design variation on soakaways is well suited to sites where available space for infiltration is limited to narrow strips of land between buildings or properties, or along road rights-of-way. They can also be referred to as infiltration galleries or linear soakaways.

Infiltration chambers are another design variation on soakaways. They include a range of proprietary manufactured modular structures installed underground, typically under parking or landscaped areas that create large void spaces for temporary storage of stormwater, allowing it to infiltrate into the underlying native soil. Structures typically have open bottoms, perforated side walls and optional underlying granular stone reservoirs. They can be installed individually or in series in trench or bed configurations. They can infiltrate roof, walkway, parking lot and road runoff with adequate pretreatment. Due to the large volume of underground void space they create in comparison to a soakaway of the same dimensions, and the modular nature of their design, they are well suited to sites where available space for other types of BMPs is limited, or where it is desirable for the facility to have little or no surface footprint (e.g., high density development contexts). They can also be referred to as infiltration tanks.

DESIGN GUIDANCE

MONITORING WELLS

Capped vertical non-perforated pipes are recommended to the inlet and outlet pipes are recommended to provide a means of inspecting and flushing them out as part of routine maintenance. A capped vertical standpipe consisting of an anchored 100 to 150 mm diameter perforated pipe with a lockable cap installed to the bottom of the facility is also recommended for monitoring the length of time required to fully drain the facility between storms. Manholes and inspection ports should be installed in infiltration chambers to provide access for monitoring and maintenance activities.

PRE-TREATMENT

It is important to prevent sediment and debris from entering infiltration facilities because they could contribute to clogging and failure of the system. The following pretreatment devices are options:

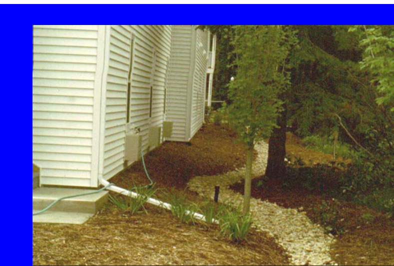
- Leaf screens: Leaf screens are mesh screens installed either on the building eavestroughs or roof downspouts and are used to remove leaves and other large debris from roof runoff.
- In-ground devices: Devices placed between a conveyance pipe and the facility (e.g., oil and grit separators, sedimentation chamber or goss traps), that can be designed to remove both large and fine particulate from runoff. A number of proprietary stormwater filter designs are available
- Vegetated filter strips or grass swales: Road and parking lot runoff can be pretreated with vegetated filter strips or grass swales prior to entering the infiltration practice

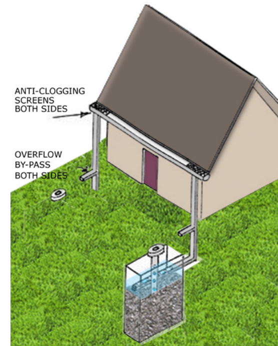
FILTER MEDIA

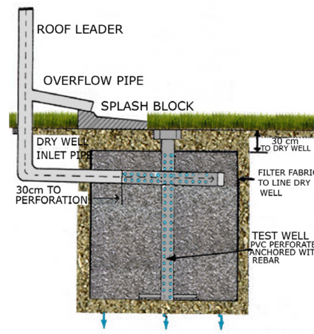
- Stone reservoir: Soakaways and infiltration trenches should be filled with uniformly-graded, washed stone that provides 30 to 40% void space. Granular material should be 50 mm clear stone
- Geotextile: A non-woven needle punched, or woven monofilament geotextile fabric should be installed around the stone reservoir of soakaways and infiltration trenches with a minimum overlap at the top of 300 mm. Woven slit film and non-woven heat bonded fabrics should not be used as they are prone to clogging. Specification of geotextile fabrics should consider the apparent opening size (AOS) for non-woven fabrics, or percent open area (POA) for woven fabrics, which affect the long term ability to maintain water flow. Other factors that need consideration include maximum forces to be exerted on the fabric, and the load bearing ratio, texture (i.e., grain size distribution) and permeability of the native soil in which they will be installed.


Source: Lanark Consultants Ltd

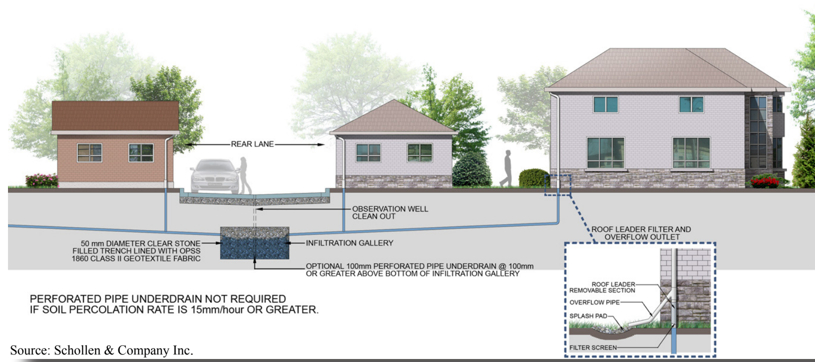

Source: Pennsylvania Department of Environmental Protection







DRY WELL SYSTEM



Source: Schollen & Company Inc.

INFILTRATION TRENCH BELOW A LANEWAY

Source: Schollen & Company Inc.

INFILTRATION CHAMBER SYSTEM UNDER A PARKING LOT

GEOMETRY AND SITE LAYOUT

Soakaways and infiltration chambers can be designed in a variety of shapes, while infiltration trenches are typically rectangular excavations with a bottom width generally between 600 and 2400 mm. Facilities should have level or nearly level bed bottoms.

CONVEYANCE AND OVERFLOW

Inlet pipes to soakaways and infiltration trenches are typically perforated pipe connected to a standard non-perforated pipe or eavestrough that conveys runoff from the source area to the facility. The inlet and overflow outlet to the facility should be installed below the maximum frost penetration depth to prevent freezing. The overflow outlet can simply be the perforated pipe inlet that backs up when the facility is at capacity and discharges to a splash pad and pervious area at grade or can be a pipe that is at the top of the gravel layer and is connected to a storm sewer. Outlet pipes must have capacity equal to or greater than the inlet.

ABILITY TO MEET SWM OBJECTIVES

BMP	Water Balance Benefit	Water Quality Improvement	Stream Channel Erosion Control Benefit
Soakaways, Infiltration Trenches and Chambers	Yes	Yes	Partial, depends on soil infiltration rate

CONSTRUCTION CONSIDERATIONS

SOIL DISTURBANCE AND COMPACTION: Before site work begins, locations of facilities should be clearly marked. Only vehicular traffic used for construction of the infiltration facility should be allowed close to the facility location.

EROSION AND SEDIMENT CONTROL: Infiltration practices should never serve as a sediment control device during construction. Construction runoff should be directed away from the proposed facility location. After the site is vegetated, erosion and sediment control structures can be removed.

COMMON CONCERNS

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. To minimize risk of groundwater contamination the following management approaches are recommended:

- 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;
- 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 10 years of operation.

ON PRIVATE PROPERTY

Property owners or managers will need to be educated on their routine maintenance needs, understand the long-term maintenance plan, and 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 could be used to encourage property owners or managers to maintain existing practices. Alternatively, infiltration practices 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.

WINTER OPERATION

Soakaways, infiltration trenches and chambers will continue to function during winter months if the inlet pipe and top of the facility is located below the local maximum frost penetration depth.

OPERATION AND MAINTENANCE

Maintenance typically consists of cleaning out leaves, debris and accumulated sediment caught in pretreatment devices, inlets and outlets annually or as needed. Inspection via an monitoring well should be performed to ensure the facility drains within the maximum acceptable length of time (typically 72 hours) at least annually and following every major storm event (>25 mm). If the time required to fully drain exceeds 72 hours, drain via pumping and clean out the perforated pipe underdrain, if present. If slow drainage persists, the system may need removal and replacement of granular material and/or geotextile fabric.

SITE CONSIDERATIONS

Wellhead Protection

Facilities receiving road or parking lot runoff should not be located within two (2) year time-of-travel wellhead protection areas.

Site Topography

Facilities cannot be located on natural slopes greater than 15%.

Water Table

The bottom of the facility should be vertically separated by one (1) metre from the seasonally high water table or top of bedrock elevation.

Soil

Soakaways, infiltration trenches and chambers can be constructed over any soil type, but hydrologic soil group A or B soils are best for achieving water balance and channel erosion control objectives. If possible, facilities should be located in portions of the site with the highest native soil infiltration rates. Designers should verify the soil infiltration rate at the proposed location and depth through field measurement of hydraulic conductivity under field saturated conditions.

Drainage Area

Typically are designed with an impervious drainage area to treatment facility area ratio of between 5:1 and 20:1. A maximum ratio of 10:1 is recommended for facilities receiving road or parking lot runoff.

Pollution Hot Spot Runoff

To protect groundwater from possible contamination, runoff from pollution hot spots should not be treated by soakaways, infiltration trenches or chambers.

Setback from Buildings

Facilities should be setback a minimum of four (4) metres from building foundations.

Proximity to Underground Utilities

Local utility design guidance should be consulted to define the horizontal and vertical offsets. Generally, requirements for underground utilities passing near the practice will be no different than for utilities in other pervious areas. However, the designer should consider the need for long term maintenance when locating infiltration facilities near other underground utilities.

TORONTO AND REGION Conservation for The Living City

CVC CREDIT VALLEY CONSERVATION

FOR FURTHER DETAILS SEE SECTION 4.4 OF THE CVC/TRCA LID SWM GUIDE

BIORETENTION

CVC/TRCA LOW IMPACT DEVELOPMENT
PLANNING AND DESIGN GUIDE - FACT SHEET

Source: Minnesota Businesses for Clean Water

Source: City of Portland

GENERAL 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 (i.e., a biofilter). The primary component of the practice is the filter bed which is a mixture of sand, fines and organic material. Other elements include a mulch ground cover and plants adapted to the conditions of a stormwater practice. 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 provide a convenient area for snow storage and treatment.

DESIGN GUIDANCE

SOIL CHARACTERISTICS

Bioretention can be constructed over any soil type, but hydrologic soil group A and B are best for achieving water balance goals. If possible, bioretention should be sited in the areas of the development with the highest native soil infiltration rates. Bioretention in soils with infiltration rates less than 15 mm/hr will require an underdrain. Designers should verify the native soil infiltration rate at the proposed location and depth through measurement of hydraulic conductivity under field saturated conditions.

GEOMETRY & SITE LAYOUT

Key geometry and site layout factors include:

- 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 is 0.8 hectares. Typical ratios of impervious drainage area to treatment facility area range from 5:1 to 15:1.
- Bioretention can be configured to fit into many locations and shapes. However, cells that are narrow 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.

PRE-TREATMENT

Pretreatment prevents premature clogging by capturing coarse sediment particles before they reach the filter bed. Where the runoff source area produces little sediment, such as roofs, bioretention can function effectively without pretreatment. To treat parking area or road runoff, a two-cell design that incorporates a forebay is recommended. Pretreatment practices that may be feasible, depending on the method of conveyance and the availability of space include:

- *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.
- *Vegetated filter strip (sheet flow):* Should be a minimum of three (3) metres in width. If smaller strips are used, more frequent maintenance of the filter bed can be anticipated.
- *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 and maintain sheet flow into the facility. 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):* Suitable for small bioretention cells with drainage areas less than 100 square metres.

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.

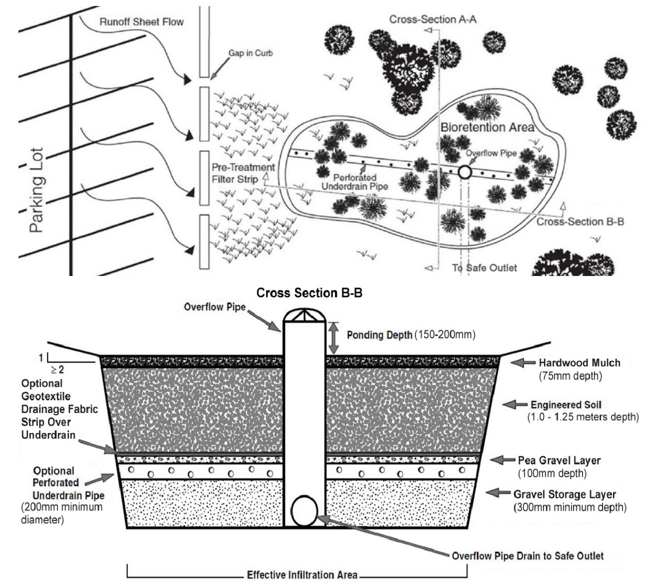
FILTER MEDIA

- COMPOSITION: To ensure a consistent and homogeneous bed, filter media should come pre-mixed from an approved vendor.
- DEPTH: Recommended depth is between 1.0 and 1.25 m. However in constrained applications, pollutant removal benefits may be achieved in beds as shallow as 500 mm. If trees are to be included in the design, bed depth must be at least 1.0 m.
- MULCH: A 75 mm layer of mulch on the surface of the filter bed enhances plant survival, suppresses weed growth and pretreats runoff before it reaches the filter bed.

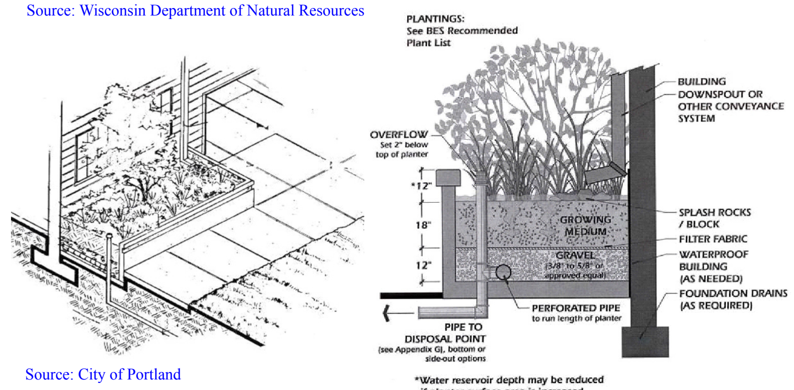
CONVEYANCE AND OVERFLOW

Bioretention can be designed to be inline or offline from the drainage system. Inline bioretention accepts all flow from a drainage area and conveys larger event flows through an overflow outlet. Overflow structures must be sized to safely convey larger storm events out of the facility. The invert of the overflow should be placed at the maximum water surface elevation of the bioretention area, which is typically 150-250 mm above the filter bed surface.

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 maintenance frequency.



Source: Wisconsin Department of Natural Resources



Source: City of Portland

ABILITY 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

UNDERDRAIN

- Only needed where native soil infiltration rate is less than 15 mm/hr (hydraulic conductivity of less than 1x10-6 cm/s).
- Should consist of a perforated pipe embedded in the coarse gravel storage layer at least 100 mm above the bottom.
- 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.
- A vertical standpipe connected to the underdrain can be used as a cleanout and monitoring well.

MONITORING WELLS

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

GENERAL SPECIFICATIONS

Material	Specification	Quantity
Filter Media Composition	Filter Media Soil Mixture to contain: • 85 to 88% sand • 8 to 12% soil fines • 3 to 5% organic matter (leaf compost) Other Criteria: • Phosphorus soil test index (P-Index) value between 10 to 30 ppm • Cationic exchange capacity (CEC) greater than 10 meq/100 g • Free of stones, stumps, roots and other large debris • pH between 5.5 to 7.5 • Infiltration rate greater than 25 mm/hr	Recommended depth is between 1.0 and 1.25 metres.
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. For further guidance see CVC/TRCA LID SWM Planning and Design Guide, Table 4.5.5.	Strip over the perforated pipe underdrain (if present) between the filter media bed and gravel storage layer (stone reservoir)
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. Locations should not be used as sediment basins during construction, as the concentration of fines will prevent post-construction infiltration. To prevent sediment from clogging the surface of a bioretention cell, stormwater should be diverted away from the bioretention until the drainage area is fully stabilized.

For further guidance regarding key steps during construction, see the CVC/TRCA LID SWM Planning and Design Guide, Section 4.5.2 - Construction Considerations)

OPERATION 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. Regular watering may be required during the first two years until vegetation is established.

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. 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.

Trash and debris should be removed from pretreatment devices, the bioretention area surface and inlet and outlets at least twice annually. Other maintenance activities include reapplying mulch, pruning, weeding replacing dead vegetation and repairing eroded areas as needed. Remove accumulated sediment on the bioretention area surface when dry and exceeding 25 mm depth.

SITE CONSIDERATIONS

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
Reserve open areas of about 10 to 20% of the size of the contributing drainage area.

Site Topography
Contributing slopes should be between 1 to 5%. The surface of the filter bed should be flat to allow flow to spread out. A stepped multi-cell design can also be used.

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.

Water Table
A minimum of one (1) metre separating the seasonally high water table or top of bedrock elevation and the bottom of the practice is necessary.

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 1x10-6 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.

Drainage Area & Runoff Volume
Typical contributing drainage areas are between 100 m2 to 0.5 hectares. The maximum recommended contributing drainage area is 0.8 hectares. Typical ratios of impervious drainage area to treatment facility area range from 5:1 to 15:1.

Pollution Hot Spot Runoff
To protect groundwater from possible contamination, runoff from pollution hot spots 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.

Overhead Wires
Check whether the future tree canopy height in the bioretention area will interfere with existing overhead phone and power lines.

Setback from Buildings
If an impermeable liner is used, no setback is needed. If not, a four (4) metre setback from building foundations should be applied.

TORONTO AND REGION Conservation for The Living City

CVC CREDIT VALLEY CONSERVATION

FOR FURTHER DETAILS SEE SECTION 4.5 OF THE CVC/TRCA LID SWM GUIDE

VEGETATED FILTER STRIPS

CVC/TRCA LOW IMPACT DEVELOPMENT
PLANNING AND DESIGN GUIDE - FACT SHEET

GENERAL DESCRIPTION

Vegetated filter strips (a.k.a. buffer strips and grassed filter strips) are gently sloping, densely vegetated areas that treat runoff as sheet flow from adjacent impervious areas. They slow runoff velocity and filter out suspended sediment and associated pollutants, and provide some infiltration into underlying soils. Originally used as an agricultural treatment practice, filter strips have evolved into an urban SWM practice. Vegetation may be comprised of a variety of trees, shrubs and native plants to add aesthetic value as well as water quality benefits. With proper design and maintenance, filter strips can provide relatively high pollutant removal benefits. Maintaining sheet flow into the filter strip through the use of a level spreading device (e.g., pea gravel diaphragm) is essential. Using vegetated filter strips as pretreatment practices to other best management practices is highly recommended. They also provide a convenient area for snow storage and treatment, and are particularly valuable due to their capacity for snowmelt infiltration.

DESIGN GUIDANCE

GEOMETRY AND SITE LAYOUT

The maximum contributing flow path length across adjacent impervious surfaces should not exceed 25 metres. The impervious surfaces draining to a filter strip should not have slopes greater than 3%.

The filter strip should have a flow path length of at least five (5) metres to provide substantial water quality benefits; however, some pollutant removal benefits are realized with three (3) metres of flow path length.

PRETREATMENT

A pea gravel diaphragm at the top of the slope is recommended to act as a pretreatment device and level spreader to maintain sheet flow into the filter strip.

CONVEYANCE AND OVERFLOW

Level spreaders are recommended to ensure runoff draining into the filter strip does so as sheet flow (e.g., pea gravel diaphragms, concrete curbs with cutouts). When filter strip slopes are greater than 5%, a series of level spreaders should be used to help maintain sheet flow.

When designed as a stand alone water quality BMP (i.e., not pretreatment to another BMP) the vegetated filter strip should be designed with a pervious berm at the toe of the slope for shallow ponding of runoff. The berm should be 150 to 300 millimetres in height above the bottom of the depression and should contain a perforated pipe underdrain connected to the storm sewer. The volume ponded behind the berm should be equal to the water quality storage requirement. During larger storms, runoff overtops the berm and flows directly into a storm sewer inlet.

SOIL AMENDMENTS

If soils on the filter strip site are highly compacted, or of such low fertility that vegetation cannot become established, they should be tilled to a depth of 300 mm and amended with compost to achieve an organic content of 8 to 15% by weight or 30 to 40% by volume.

OPERATION AND MAINTENANCE

Generally, routine maintenance will be the same as for any other landscaped area; weeding, pruning, and litter removal. Regular watering may be required during the first two years until vegetation is established. Routine inspection is very important to ensure that dense vegetation cover is maintained and inflowing runoff does not become concentrated and short circuit the practice. Vehicles should not be parked or driven on filter strips. For routine mowing of grassed filter strips, the lightest possible mowing equipment should be used to prevent soil compaction.

For the first two years following construction the filter strip 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. 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.

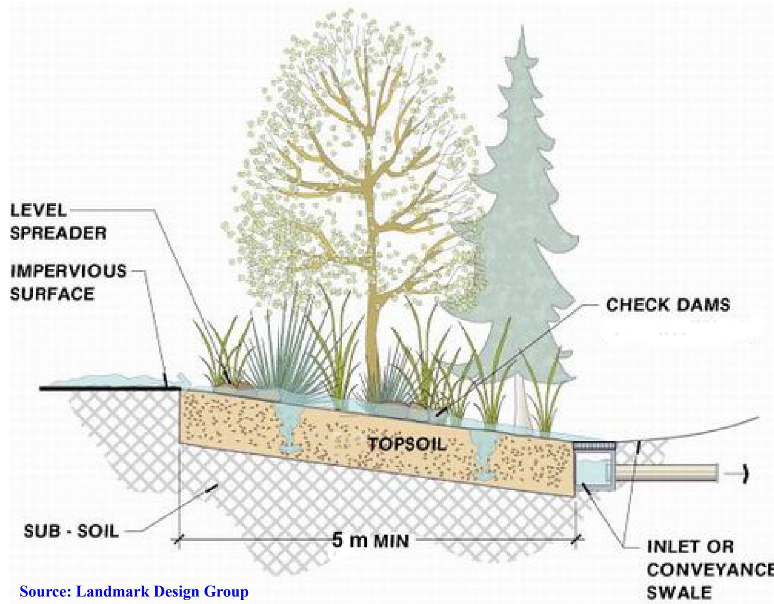
Trash and debris should be removed from pretreatment devices and the filter strip surface at least twice annually. Other maintenance activities include weeding, replacing dead vegetation, repairing eroded areas, dethatching and aerating as needed. Remove accumulated sediment on the filter strip surface when dry and exceeding 25 mm depth


Source: Trinkaus Engineering


Source: Wisconsin Department of Natural Resources

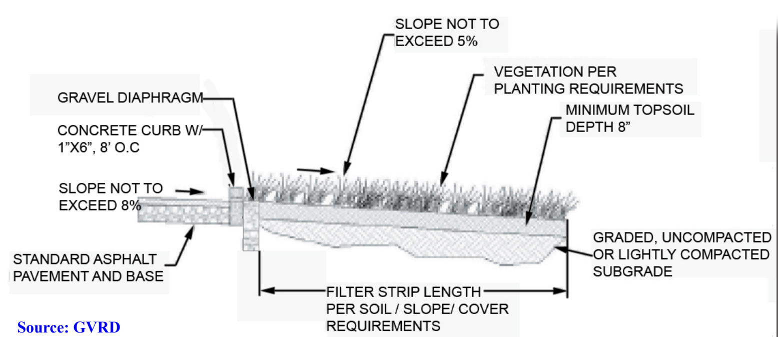

Source: Wisconsin Department of Natural Resources


Source: Washington State Department of Transportation

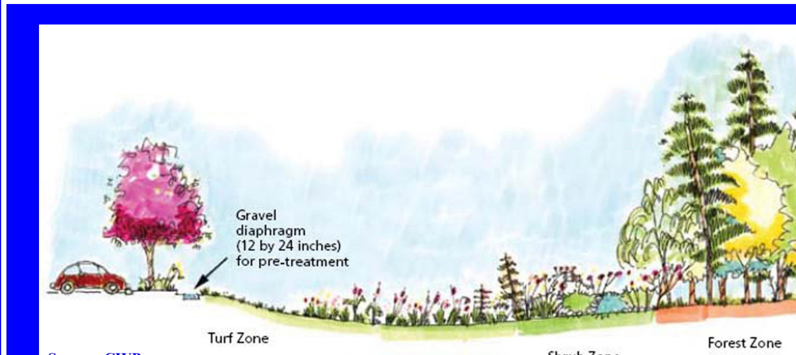


Source: Landmark Design Group

VEGETATED FILTER STRIPS



Source: GVRD



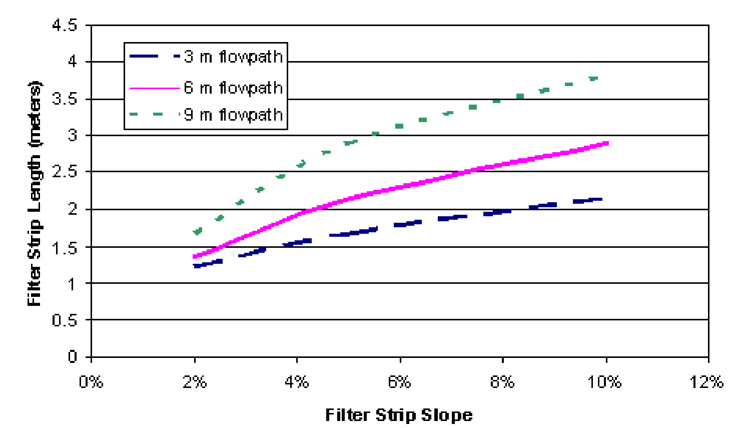
Source: CWP

ABILITY TO MEET SWM OBJECTIVES

BMP	Water Balance Benefit	Water Quality Improvement	Stream Channel Erosion Control Benefit
Vegetated Filter Strips	Partial - depends on soil infiltration rate	Partial - depends on soil infiltration rate and flow path length	Partial - depends on soil infiltration rate

GENERAL SPECIFICATIONS

Material	Specification	Quantity
Gravel Diaphragm	Washed 3 to 10 mm diameter stone	Diaphragm should be a minimum of 300 mm wide and 600 mm deep (MDE, 2000).
Gravel/ Earthen Berm	Berm should be composed of sand (35 to 60%), silt (30 to 55%), and gravel (10 to 25%) (MDE, 2000) Gravel should be 15 to 25 mm in diameter.	N/A



Source: Pennsylvania Department of Environmental Protection

CONSTRUCTION CONSIDERATIONS

Soil Disturbance and Compaction

The limits of disturbance should be clearly shown on all construction drawings. Before site work begins, areas for filter strips should be clearly marked and protected by acceptable signage and silt fencing. Only vehicular traffic used for construction should be allowed within three metres of the filter strip.

Erosion and Sediment Control

Construction runoff should be directed away from the proposed filter strip site. If used for sediment control during construction, it should be regraded and revegetated after construction is finished.

SITE CONSIDERATIONS

Available Space

The flow path length across the vegetated filter strip should be at least 5 metres to provide substantial water quality benefits. Vegetated filter strips incorporated as pretreatment to another BMP may be designed with shorter flow path lengths.

Site Topography

Filter strips are best used to treat runoff from ground-level impervious surfaces that generate sheet flow (e.g., roads and parking areas). The recommended filter strip slope is between 1 to 5%.

Flow Path Length Across Impermeable Surface

The maximum flow path length across the contributing impermeable surface should be less than 25 metres.

Soil

Filter strips are a suitable practice on all soil types. If soils are highly compacted, or of such low fertility that vegetation cannot become established, they should be tilled to a depth of 300 mm and amended with compost to achieve an organic content of 8 to 15% by weight or 30 to 40% by volume.

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 vegetated filter strips.

Water table

Filter strips should only be used where depth to the seasonally high water table is at least one (1) metre below the ground surface.

TORONTO AND REGION Conservation for The Living City

CVC CREDIT VALLEY CONSERVATION

PERMEABLE PAVEMENT

CVC/TRCA LOW IMPACT DEVELOPMENT
PLANNING AND DESIGN GUIDE - FACT SHEET

GENERAL DESCRIPTION

Permeable pavements, an alternative to traditional impervious pavement, allow stormwater to drain through them and into a stone reservoir where it is infiltrated into the underlying native soil or temporarily detained. They can be used for low traffic roads, parking lots, driveways, pedestrian plazas and walkways. Permeable pavement is ideal for sites with limited space for other surface stormwater BMPs. Examples of permeable pavement types include:

- permeable interlocking concrete pavers (i.e., block pavers);
- plastic or concrete grid systems (i.e., grid pavers);
- pervious concrete; and
- porous asphalt.

Depending on the native soils and physical constraints, the system may be designed with no underdrain for full infiltration, with an underdrain for partial infiltration, or with an impermeable liner and underdrain for a no infiltration or detention and filtration only practice.

DESIGN GUIDANCE

GEOMETRY & SITE LAYOUT

Permeable pavement systems can be used for entire parking lot areas or driveways or can be designed to receive runoff from adjacent impervious pavement. For example, the parking spaces of a parking lot or road can be permeable pavers while the drive lanes are impervious asphalt. In general, the impervious area should not exceed 1.2 times the area of the permeable pavement which receives the runoff (GVRD, 2005).

PRE-TREATMENT

In most permeable pavement designs, the pavement bedding layer acts as pretreatment to the stone reservoir below. Periodic vacuum sweeping and preventative measures like not storing snow or other materials on the pavement are critical to prevent clogging. An optional pretreatment element can be a pea gravel choking layer above the coarse gravel storage reservoir.

CONVEYANCE AND OVERFLOW

All designs require an overflow outlet connected to a storm sewer with capacity to convey larger storms. One option is to set storm drain inlets slightly above the surface elevation of the pavement, which allows for temporary shallow ponding above the surface. Another design option is an overflow edge, which is a gravel trench along the downgradient edge of the pavement surface that drains to the stone reservoir below.

Pavements designed for full infiltration, where native soil infiltration rate is 15 mm/hr or greater, do not require incorporation of a perforated pipe underdrain. Pavements designed for partial infiltration, where native soil infiltration rate is less than 15 mm/hr, should incorporate a perforated pipe underdrain placed near the top of the granular stone reservoir. Partial infiltration designs can also include a flow restrictor assembly on the underdrain to optimize infiltration with desired drawdown time between storm events.

MONITORING WELLS

A capped vertical standpipe consisting of an anchored 100 to 150 mm 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.

STONE RESERVOIR

The stone reservoir must be designed to meet both runoff storage and structural support requirements. Clean washed stone is recommended as any fines in the aggregate material will migrate to the bottom and may prematurely clog the native soil. The bottom of the reservoir should be flat so that runoff will be able to infiltrate evenly through the entire surface. If the system is not designed for infiltration, the bottom should be sloped at 1 to 5% toward the underdrain.

GEOTEXTILE

A non-woven needle punched, or woven monofilament geotextile fabric should be installed between the stone reservoir and native soil to maintain separation.

EDGE RESTRAINTS

Pavers must abut tightly against the restraints to prevent rotation under load and any consequent spreading of joints. The restraints must be able to withstand the impact of temperature changes, vehicular traffic and snow removal equipment. Metal or plastic stripping is acceptable in some cases, but concrete edges are preferred. Concrete edge restraints should be supported on a minimum base of 150 mm of aggregate.

LANDSCAPING

Adjacent landscaping areas should drain away from permeable pavement to prevent sediments from running onto the surface. Urban trees also benefit from being surrounded by permeable pavement rather than impervious cover, because their roots receive more air and water.

OPERATION AND MAINTENANCE

Annual inspections of permeable pavement should be conducted in the spring to ensure continued infiltration performance. Check for deterioration and whether water is draining between storms. The pavement reservoir should drain completely within 72 hours of the end of the storm event. The following maintenance procedures and preventative measures should be incorporated into a maintenance plan:

Surface Sweeping: Sweeping should occur once or twice a year with a commercial vacuum sweeping unit. Permeable pavement should not be washed with high pressure water systems or compressed air units.

Inlet Structures: Drainage pipes and structures within or draining to the subsurface bedding beneath permeable pavement should be cleaned out on regular intervals.

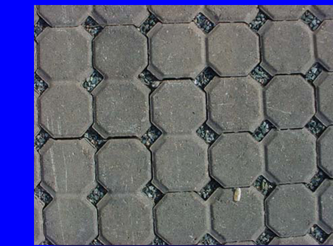
Heavy Vehicles: Trucks and other heavy vehicles should be prevented from tracking or spilling dirt onto the permeable pavement.

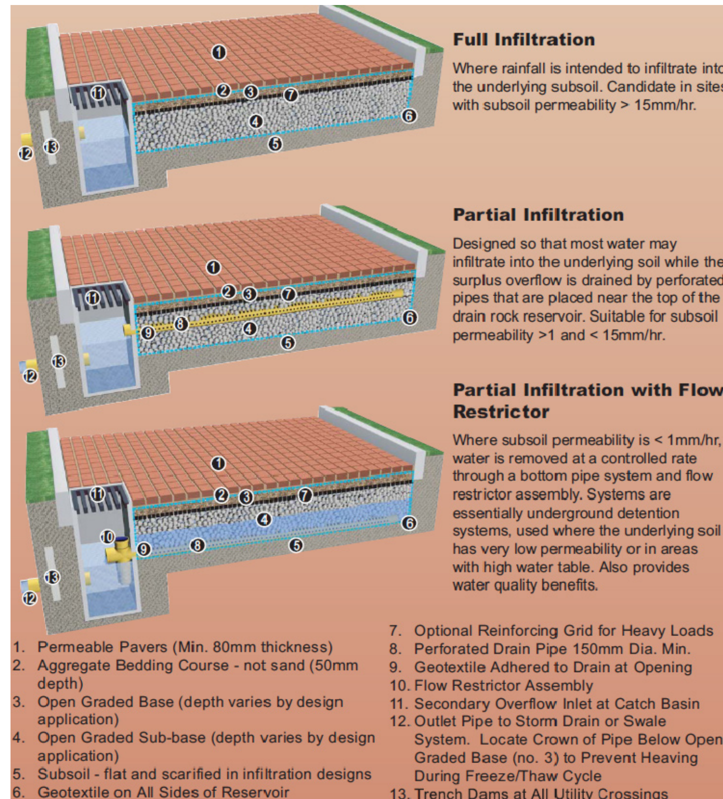
Construction and Hazardous Materials: Due to the potential for groundwater contamination, all construction or hazardous material carriers should be prohibited from entering a permeable pavement site.

Drainage Areas: Impervious areas contributing to the permeable pavement should be regularly swept and kept clear of litter and debris. Flows from any landscaped areas should be diverted away from thepavement or be well stabilized with vegetation.

Grid Pavers: Grid paver systems that have been planted with grass should be mowed regularly with the clippings removed. Grassed grid pavers may require periodic watering and fertilization to establish and maintain healthy vegetation.

Winter Maintenance: Sand should not be spread on permeable pavement as it can quickly lead to clogging. Deicers should only be used in moderation and only when needed. Pilot studies have found that permeable pavement requires 75% less de-icing salt than conventional pavement over the course of a typical winter season. Permeable pavement is plowed for snow removal like any other pavement. Plowed snow piles should not be stored on permeable pavement systems.





Source: GVRD

ABILTY TO MEET SWM OBJECTIVES

BMP	Water Balance Benefit	Water Quality Improvement	Stream Channel Erosion Control Benefit
Permeable pavement with no underdrain	Yes	Yes - size for water quality storage requirement	Partial - based on available storage volume and soil infiltration rate
Permeable pavement with underdrain	Moderate - based on native soil infiltration rates and storage beneath the underdrain	Yes - size for water quality storage requirement	Partial - based on available storage volume and soil infiltration rate
Permeable pavement with underdrain and liner	No - some volume reduction occurs through evapotranspiration	Moderate - limited filtering and settling of sediments	Partial - based on available storage volume and soil infiltration rate

GENERAL SPECIFICATIONS

Material	Specification	Quantity
Pervious Concrete	• NO4-RG-S7 mix with air entrainment proven to have the best freeze-thaw durability after 300 freeze-thaw cycles. • 28 day compressive strength = 5.5 to 20 MPa • Void ratio = 14% - 31% • Permeability = 900 to 21,500 mm/hr	Thickness will range from 100mm - 150 mm depending on the expected loads
Porous Asphalt	• Open-graded asphalt mix with a minimum of 16% air voids • Polymers can be added to provide additional strength for heavy loads • The University of New Hampshire Stormwater Center has detailed design specifications for porous asphalt on their web-page: http://www.unh.edu/erg/cstev/pubs_specs_info	Thickness will range from 50 mm to 100 mm depending on the expected loads.
Permeable Pavers	• Permeable pavers should conform to manufacturer specifications. • ASTM No. 8 (5 mm dia.) crushed aggregate is recommended for fill material in the paver openings. For narrow joints between interlocking shapes, a smaller sized aggregate may be used (Smith, 2006). • Pavers shall meet the minimum material and physical properties set forth in CAN 3-A231.2, Standard Specification for Precast Concrete Pavers. • Pigment in concrete pavers shall conform to ASTM C 979. • Maximum allowable breakage of product is 5%.	For vehicular applications, the minimum paver thickness is 80 mm and for pedestrian applications is 60 mm. Joint widths should be no greater than 15 mm for pedestrian applications.
Stone Reservoir	All aggregates should meet the following criteria: • Maximum wash loss of 0.5% • Minimum durability index of 35 • Maximum abrasion of 10% for 100 revolutions and maximum of 50% for 500 revolutions <u>Granular Subbase</u> The granular subbase material shall consist of granular material graded in accordance with ASTM D 2940. Material should be clear crushed 50 mm diameter stone with void space ratio of 0.4. <u>Granular Base</u> The granular base material shall be crushed stone conforming to ASTM C 33 No 57. Material should be clear crushed 20 mm diameter stone. <u>Bedding</u> The granular bedding material shall be graded in accordance with the requirements of ASTM C 33 No 8. The typical bedding thickness is between 40 mm and 75 mm. Material should be 5 mm diameter stone or as determined by the Design Engineer (Smith, 2006).	See BMP Sizing section for aggregate bed depth and multiply by application are to get total volume.
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; • Maximum forces that will be exerted on the fabric (i.e., what tensile, tear and puncture strength ratings are required?); • Load bearing ratio of the underlying native soil (i.e., is geotextile needed to prevent downward migration of aggregate into the native soil?); • Texture (i.e., grain size distribution) of the overlying aggregate material; and • Permeability of the native soil. For further guidance see CVC/TRCA LID SWM Planning and Design Guide, Table 4.7.3.	Between stone reservoir and native soil.
Underdrain (optional)	• HDPE or equivalent material, continuously perforated with smooth interior and a minimum inside diameter of 100 mm. • Perforations in pipes should be 10 mm in diameter. • A standpipe from the underdrain to the pavement surface can be used for monitoring and maintenance of the underdrain. The top of the standpipe should be covered with a screw cap and a vandal-proof lock.	Pipes should terminate 0.3 m short from the sides of the base.

SITE CONSIDERATIONS

Wellhead Protection

Permeable pavement should not be used for road or parking surfaces within two (2) year time-of-travel wellhead protection areas.

Site Topography

Permeable pavement surface should be at least 1% and no greater than 5%.

Water Table

The base of permeable pavement stone reservoir should be at least one (1) metre above the seasonally high water table or top of bedrock elevation.

Soil

Systems located in native soils with an infiltration rate of less than 15 mm/hr (i.e., hydraulic conductivity of less than 1x10-6 cm/s) require a perforated pipe underdrain. Native soil infiltration rate at the proposed location and depth should be confirmed through measurement of hydraulic conductivity under field saturated conditions.

Drainage Area & Runoff Volume

In general, the impervious area treated should not exceed 1.2 times the area of permeable pavement which receives the runoff.

Setback from Buildings

Should be located downslope from building foundations. If the pavement does not receive runoff from other surfaces, no setback is required. If the pavement receives runoff from other surfaces a minimum setback of four (4) metres downgradient is recommended.

Pollution Hot Spot Runoff

To protect groundwater from possible contamination, runoff from pollution hot spots should not be treated by permeable pavement.

CONSTRUCTION CONSIDERATIONS

SEDIMENT CONTROL

The treatment area should be fully protected during construction so that no sediment reaches the permeable pavement system. Construction traffic should be blocked from the permeable pavement and its drainage areas once the pavement has been installed.

BASE CONSTRUCTION

In parking lots, the stone aggregate should be placed in 100 mm to 150 mm lifts and compacted with a minimum 9,070 kg (10 ton) steel drum roller.

WEATHER

Porous asphalt and pervious concrete will not properly pour and set in extremely high and low temperatures.

PAVEMENT PLACEMENT

Properly installed permeable pavement requires trained and experienced producers and construction contractors.

TORONTO AND REGION Conservation for The Living City

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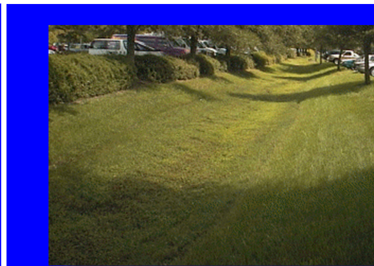
FOR FURTHER DETAILS SEE SECTION 4.7 OF THE CVC/TRCA LID SWM GUIDE

ENHANCED GRASS SWALES

CVC/TRCA LOW IMPACT DEVELOPMENT
PLANNING AND DESIGN GUIDE - FACT SHEET




GENERAL DESCRIPTION

Enhanced grass swales are vegetated open channels designed to convey, treat and attenuate stormwater runoff (also referred to as enhanced vegetated swales). Check dams and vegetation in the swale slows the water to allow sedimentation, filtration through the root zone and soil matrix, evapotranspiration, and infiltration into the underlying native soil. Simple grass channels or ditches have long been used for stormwater conveyance, particularly for roadway drainage. Enhanced grass swales incorporate design features such as modified geometry and check dams that improve the contaminant removal and runoff reduction functions of simple grass channel and roadside ditch designs.

Where development density, topography and depth to water table permit, enhanced grass swales are a preferred alternative to both curb and gutter and storm drains as a stormwater conveyance system. When incorporated into a site design, they can reduce impervious cover, accent the natural landscape, and provide aesthetic benefits.

DESIGN GUIDANCE

GEOMETRY AND SITE LAYOUT

- Shape: Should be designed with a trapezoidal or parabolic cross section. Trapezoidal swales will generally evolve into parabolic swales over time, so the initial trapezoidal cross-section design should be checked for capacity and conveyance assuming it is a parabolic cross-section. Swale length between culverts should be 5 metres or greater.
- Bottom Width: Should be designed with a bottom width between 0.75 and 3.0 metres. Should allow for shallow flows and adequate water quality treatment, while preventing flows from concentrating and creating gullies.
- Longitudinal Slope: Slopes should be between 0.5% and 4%. Check dams should be incorporated on slopes greater than 3%.
- Length: When used to convey and treat road runoff, the length simply parallels the road, and therefore should be equal to, or greater than the contributing roadway length.
- Flow Depth: A maximum flow depth of 100 mm is recommended during a 4 hour, 25 mm Chicago storm event.
- Side Slopes: Should be as flat as possible to aid in providing pretreatment for lateral incoming flows and to maximize the swale filtering surface. Steeper side slopes are likely to have erosion gullying from incoming lateral flows. A maximum slope of 2.5:1 (H:V) is recommended and a 4:1 slope is preferred where space permits.

PRE-TREATMENT

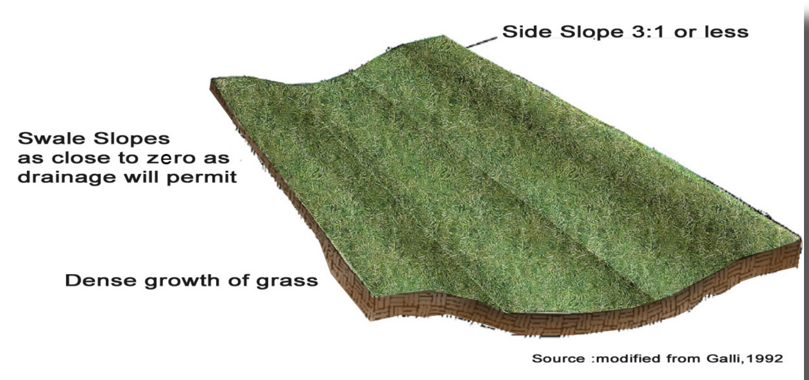
A pea gravel diaphragm located along the top of each bank can be used to provide pretreatment of any runoff entering the swale laterally along its length. Vegetated filter strips or mild side slopes (3:1) also provide pretreatment for any lateral sheet flow entering the swale. Sedimentation forebays at inlets to the swale are also a pretreatment option.

CONVEYANCE AND OVERFLOW

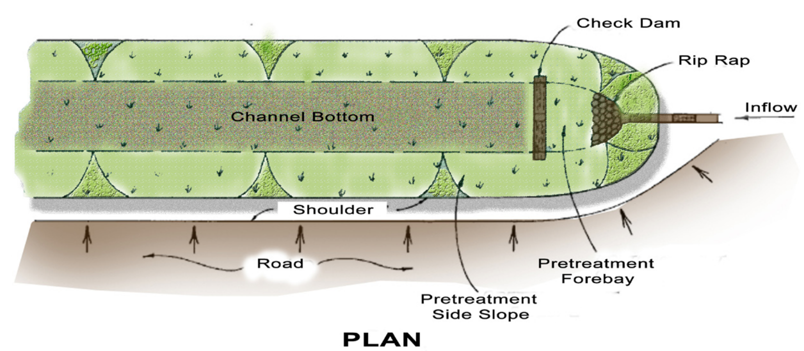
Grass swales must be designed for a maximum velocity of 0.5 m/s or less for the 4 hour 25 mm Chicago storm event. The swale should also convey the locally required design storm (usually the 10 year storm) at non-erosive velocities.

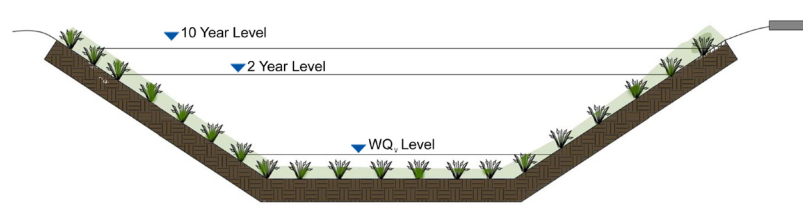
SOIL AMENDMENTS

If soils along the location of the swale are highly compacted, or of such low fertility that vegetation cannot become established, they should be tilled to a depth of 300 mm and amended with compost to achieve an organic content of 8 to 15% by weight or 30 to 40% by volume.



PLAN VIEW OF A GRASS SWALE





PLAN AND PROFILE VIEWS

ABILITY TO MEET SWM OBJECTIVES

BMP	Water Balance Benefit	Water Quality Improvement	Stream Channel Erosion Control Benefit
Enhanced Grass Swale	Partial - depends on soil infiltration rate	Yes, if design velocity is 0.5 m/s or less for a 4 hour, 25 mm Chicago storm	Partial - depends on soil infiltration rate

GENERAL SPECIFICATIONS

Component	Specification	Quantity
Check Dams	Constructed of a non-erosive material such as suitably sized aggregate, wood, gabions, riprap, or concrete. All check dams should be underlain with geotextile filter fabric. Wood used for check dams should consist of pressure treated logs or timbers, or water-resistant tree species such as cedar, hemlock, swamp oak or locust.	Spacing should be based on the longitudinal slope and desired ponding volume.
Gravel Diaphragm	Washed stone between 3 and 10 mm in diameter.	Minimum of 300 mm wide and 600 mm deep.

CONSTRUCTION CONSIDERATIONS

Grass swales should be clearly marked before site work begins to avoid disturbance during construction. No vehicular traffic, except that specifically used to construct the facility, should be allowed within the swale site. Any accumulation of sediment that does occur within the swale must be removed during the final stages of grading to achieve the design cross-section. Final grading and planting should not occur until the adjoining areas draining into the swale are stabilized. Flow should not be diverted into the swale until the banks are stabilized.

Preferably, the swale should be planted in the spring so that the vegetation can become established with minimal irrigation. Installation of erosion control matting or blanketing to stabilize soil during establishment of vegetation is highly recommended. If sod is used, it should be placed with staggered ends and secured by rolling the sod. This helps to prevent gullies.

OPERATION AND MAINTENANCE

Generally, routine maintenance will be the same as for any other landscaped area; weeding, pruning, and litter removal. Grassed swales should be mown at least twice yearly to maintain grass height between 75 and 150 mm. The lightest possible mowing equipment should be used to prevent soil compaction. Routine roadside ditch maintenance practices such as scraping and re-grading should be avoided. Regular watering may be required during the first two years until vegetation is established. Routine inspection is very important to ensure that dense vegetation cover is maintained and inlets and pretreatment devices are free of debris.

For the first two years following construction the swale 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. Inspect for vegetation density (at least 80% coverage), damage by foot or vehicular traffic, accumulation of debris, trash and sediment, and structural damage to pretreatment devices.

Trash and debris should be removed from pretreatment devices and the surface of the swale at least twice annually. Other maintenance activities include weeding, replacing dead vegetation, repairing eroded areas, dethatching and aerating as needed. Remove accumulated sediment on the swale surface when dry and exceeding 25 mm depth.

SITE CONSIDERATIONS

Available Space
Grass swales usually consume about 5 to 15% of their contributing drainage area. A width of at least 2 metres is needed.

Site Topography
Site topography constrains the application of grass swales. Longitudinal slopes between 0.5 and 6% are allowable. This prevents ponding while providing residence time and preventing erosion. On slopes steeper than 3%, check dams should be used.

Drainage Area & Runoff Volume
The conveyance capacity should match the drainage area. Sheet flow to the grass swale is preferable. If drainage areas are greater than 2 hectares, high discharge through the swale may not allow for filtering and infiltration, and may create erosive conditions. Typical ratios of impervious drainage area to treatment facility area range from 5:1 to 10:1.

Soil
Grass swales can be applied on sites with any type of soils.

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 grass swales.

Proximity to Underground Utilities
Utilities running parallel to the grass swale should be offset from the centerline of the swale. Underground utilities below the bottom of the swale are not a problem.

Water Table
The bottom of the swale should be separated from the seasonally high water table or top of bedrock elevation by at least one (1) metre.

Setback from Buildings
Should be located a minimum of four (4) metres from building foundations to prevent water damage.

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FOR FURTHER DETAILS SEE SECTION 4.8 OF THE CVC/TRCA LID SWM GUIDE

CVC/TRCA LOW IMPACT DEVELOPMENT PLANNING AND DESIGN GUIDE - FACT SHEET

DRY SWALES

GENERAL DESCRIPTION

A dry swale can be thought of as an enhanced grass swale that incorporates an engineered filter media bed and optional perforated pipe underdrain or a bioretention cell configured as a linear open channel. They can also be referred to as infiltration swales or bio-swales. Dry swales are similar to enhanced grass swales in terms of the design of their surface geometry, slope, check dams and pretreatment devices. They are similar to bioretention cells in terms of the design of the filter media bed, gravel storage layer and optional underdrain. In general, they are open channels designed to convey, treat and attenuate stormwater runoff. Vegetation or aggregate material on the surface of the swale slows the runoff water to allow sedimentation, filtration through the root zone and engineered soil bed, evapotranspiration, and infiltration into the underlying native soil.

DESIGN GUIDANCE

GEOMETRY AND SITE LAYOUT

- SHAPE: A parabolic shape is preferable for aesthetic, maintenance and hydraulic reasons. However, design may be simplified with a trapezoidal cross-section as long as the engineered soil (filter media) bed boundaries lay in the flat bottom areas. Swale length between culverts should be 5 metres or greater.
- BOTTOM WIDTH: For the trapezoidal cross section, the bottom width should be between 0.75 and 2 metres. When greater widths are desired, bioretention cell designs should be used.
- SIDE SLOPES: Should be no steeper than 3:1 for maintenance considerations (mowing). Flatter slopes are encouraged where adequate space is available to provide pretreatment for sheet flows entering the swale.
- LONGITUDINAL SLOPE: Should be as gradual as possible to permit the temporary ponding of the water quality storage requirement. Should be designed with longitudinal slopes generally ranging from 0.5 to 4%, and no greater than 6%. On slopes steeper than 3%, check dams should be used. Check dam spacing should be based on the slope and desired ponding volume. They should be spaced far enough apart to allow access for maintenance equipment (e.g., mowers).

PRE-TREATMENT

Pretreatment prevents premature clogging by capturing coarse sediments before they reach the filter bed. Where runoff source areas produce little sediment, such as roofs, dry swales can function effectively without pretreatment. To treat parking area or road runoff, a two-cell design that incorporates a forebay is recommended. Pretreatment practices that may be feasible, depending on conveyance method and availability of space include:

- SEDIMENTATION FOREBAY (TWO-CELL DESIGN): Forebay ponding volume should account for 25% of the water quality storage requirement and be designed with a 2:1 length to width ratio.
- VEGETATED FILTER STRIP (SHEET FLOW): Should ideally be a minimum of three (3) metres in width. If smaller strips are used, more frequent maintenance of the filter bed can be anticipated.
- 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 dry swale will promote settling out of sediment and maintain sheet flow into the facility. 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): Suitable for small dry swales with drainage areas less than 100 square metres.

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: To ensure a consistent and homogeneous bed, filter media should come pre-mixed from an approved vendor.
- DEPTH: Recommended depth is between 1.0 and 1.25 m. However in constrained applications, pollutant removal benefits may be achieved in beds as shallow as 500 mm. If trees are to be included in the design, bed depth must be at least 1.0 m.
- MULCH: A 75 mm layer of mulch on the surface of the filter bed enhances plant survival, suppresses weed growth and pretreats runoff before it reaches the filter bed.

UNDERDRAIN

- Only needed where native soil infiltration rate is less than 15 mm/hr (hydraulic conductivity of less than 1x10-6 cm/s).
- Should consist of a perforated pipe embedded in the coarse gravel storage layer at least 100 mm above the bottom.
- 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.
- A vertical standpipe connected to the underdrain at the furthest downstream end of the swale can be used as a cleanout and monitoring well.

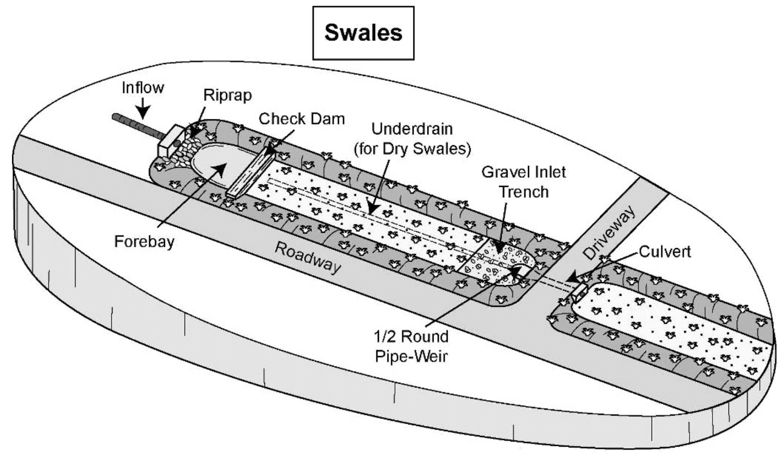
Source: TRCA

Source: Seattle Public Utilities

Source: City of Portland

Source: Lynn Richards

Source: Portland Public Schools



ABILITY TO MEET SWM OBJECTIVES

BMP	Water Balance Benefit	Water Quality Improvement	Stream Channel Erosion Control Benefit
Dry swale with no underdrain or full infiltration	Yes	Yes - size for water quality storage requirement	Partial - based on available storage volume and soil infiltration rate
Dry swale with underdrain or partial infiltration	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
Dry swale with underdrain and impermeable liner or no infiltration	Partial - some volume reduction through evapotranspiration	Yes - size for water quality storage requirement	Partial - some volume reduction through evapotranspiration

OPERATION AND MAINTENANCE

Dry swales require 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. Regular watering may be required during the first two years until vegetation is established.

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. 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.

Trash and debris should be removed from pretreatment devices, the dry swale surface and inlet and outlets at least twice annually. Other maintenance activities include reapplying mulch, pruning, weeding replacing dead vegetation and repairing eroded areas as needed. Remove accumulated sediment on the dry swale surface when dry and exceeding 25 mm depth.

CONVEYANCE AND OVERFLOW

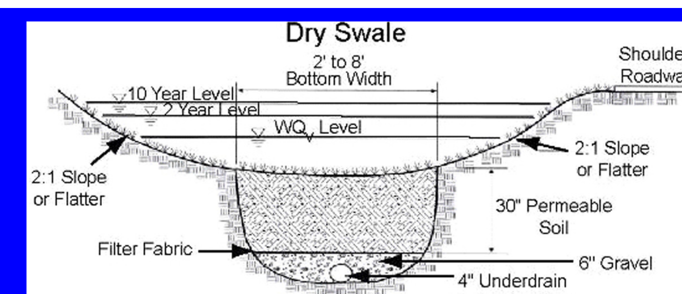
Should be designed for a maximum velocity of 0.5 m/s or less for a 4 hour 25 mm Chicago storm event. The swale should also convey the locally required design storm (usually the 10 year storm) at non-erosive velocities with freeboard provided above the required design storm water level.

MONITORING WELLS

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

GENERAL SPECIFICATIONS

Material	Specification	Quantity
Filter Media Composition	Filter Media Soil Mixture to contain: • 85 to 88% sand • 8 to 12% soil fines • 3 to 5% organic matter (leaf compost) Other Criteria: • Phosphorus soil test index (P-Index) value between 10 to 30 ppm • Cationic exchange capacity (CEC) greater than 10 meq/100 g • Free of stones, stumps, roots and other large debris • pH between 5.5 to 7.5 • Infiltration rate greater than 25 mm/hr.	Volumetric computation based on surface area and depth used in design computations
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. For further guidance see CVC/TRCA LID SWM Planning and Design Guide, Table 4.9.4.	Strip over the perforated pipe underdrain (if present) between the filter media bed and gravel storage layer (stone reservoir).
Gravel	Washed 50 mm diameter clear stone with void space ratio of 0.4 should be used to surround the underdrain.	Volumetric computation based on depth.
Underdrain (optional)	Perforated HDPE or equivalent material, minimum 100 mm dia., 200 mm dia. recommended. Set pipe invert at least 100 mm above bottom of the gravel layer.	• Perforated pipe for length of dry swale. • Non-perforated pipe to connect with storm drain system. • One or more caps. • T's for underdrain
Check Dams	• Should be constructed of a non-erosive material such as wood, gabions, riprap, or concrete and underlain with filter fabric. • Wood used should consist of pressure treated logs or timbers, or water-resistant tree species such as cedar, hemlock, swamp oak or locust.	Computation of check dam material needed based on surface area and depth used in design computations
Mulch or Matting	• Shredded hardwood bark mulch • Where flow velocities dictate, use erosion and sediment control matting - coconut fiber or equivalent.	Mulch - A 75 mm layer on the surface of the filter bed. Matting - based on filter bed area.



SITE CONSIDERATIONS

Available Space
Footprints are 5 to 15% of their contributing drainage area. Swale length between culverts should be 5m or greater.

Site Topography
Longitudinal slopes ranging from 0.5 to 4%. On slopes steeper than 3%, check dams should be used.

Drainage Area and Runoff Volume to Site
Typically treat drainage areas of two hectares or less. Typical ratios of impervious drainage area to treatment facility area range from 5:1 to 15:1.

Soil
Dry swales 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 1x10-6 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.

Wellhead Protection
Facilities receiving road or parking lot runoff should not be located within two (2) year time-of-travel wellhead protection areas.

Water Table
The bottom of the swale should be separated from the seasonally high water table or top of bedrock elevation by at least one (1) metre to prevent groundwater contamination.

Pollution Hot Spot Runoff
To protect groundwater from possible contamination, runoff from pollution hot spots should not be treated dry swales 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.

Setback from Buildings
Should be set back four (4) metres from building foundations unless an impermeable liner and underdrain system is used.

Proximity to Underground Utilities
Designers should consult local utility design guidance for the horizontal and vertical clearance between storm drains, ditches, and surface water bodies.

CONSTRUCTION CONSIDERATIONS

Ideally, dry swale sites should remain outside the limit of disturbance until construction of the swale begins to prevent soil compaction by heavy equipment. Dry swale locations should never be used as the site of sediment basins during construction, as the concentration of fines will prevent post-construction infiltration. To prevent clogging, stormwater should be diverted away from the practice until the drainage area is fully stabilized.

Source: Lake County, Illinois

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FOR FURTHER DETAILS SEE SECTION 4.9 OF THE CVC/TRCA LID SWM GUIDE

PERFORATED PIPE SYSTEMS

CVC/TRCA LOW IMPACT DEVELOPMENT PLANNING AND DESIGN GUIDE - FACT SHEET

GENERAL DESCRIPTION

Perforated pipe systems can be thought of as long infiltration trenches or linear soak-aways that are designed for both conveyance and infiltration of stormwater runoff. They are composed of perforated pipes installed in gently sloping granular stone beds that are lined with geotextile fabric that allow infiltration of runoff into the gravel bed and underlying native soil while it is being conveyed from source areas or other BMPs to an end-of-pipe facility or receiving waterbody. Perforated pipe systems can be used in place of conventional storm sewer pipes, where topography, water table depth, and runoff quality conditions are suitable. They are suitable for treating runoff from roofs, walkways, parking lots and low to medium traffic roads, with adequate pretreatment. Perforated pipe systems can also be referred to as pervious pipe systems, exfiltration systems, clean water collector systems and percolation drainage systems.

DESIGN GUIDANCE

SOIL CHARACTERISTICS

Perforated pipe systems can be constructed over any soil type, but hydrologic soil group A or B soils are best for achieving water balance objectives. If possible, facilities should be located in portions of the site with the highest native soil infiltration rates. Designers should verify the native soil infiltration rate at the proposed location and depth through measurement of hydraulic conductivity under field saturated conditions.

GEOMETRY AND SITE LAYOUT

Gravel beds in which the perforated pipes are installed are typically rectangular excavations with a bottom width between 600 and 2400 mm. The gravel beds should have gentle slopes between 0.5 to 1%.

PRE-TREATMENT

It is important to prevent sediment and debris from entering infiltration facilities because they could contribute to clogging and failure of the system. The following pre-treatment devices are options:

- *In-ground devices:* Devices placed between a conveyance pipe and the facility (e.g., oil and grit separators, sedimentation chambers, goss traps), that can be designed to remove both large and fine particulate from runoff. A number of proprietary filter designs are also available.
- *Vegetated filter strips or grass swales:* Road and parking lot runoff can be pretreated with vegetated filter strips or grass swales prior to entering the infiltration practice.

CONVEYANCE AND OVERFLOW

Collection and conveyance of runoff into the perforated pipe system can be accomplished through conventional catchbasins and non-perforated pipes leading from foundation drains and roof downspouts. Perforated pipes should be smooth walled to reduce the potential for clogging and facilitate clean out. The gravel filled trench should be 75 to 150 mm deep above the perforated pipe. On-line concrete, clay or plastic trench baffles or other barriers can be installed across the granular filled trench to reduce flow along the system, thereby increasing the potential for infiltration. Overflows from the granular filled trench should either back up into manholes that are also connected to conventional storm sewers or be conveyed to a storm sewer or receiving waterbody by overland flow.

FILTER MEDIA

- *Gravel filled trench:* Should be filled with uniformly-graded, washed, 50 mm clear stone that provides 40% void space.
- *Geotextile:* A non-woven needle punched, or woven monofilament geotextile fabric should be installed around the stone reservoir of perforated pipe systems with a minimum overlap at the top of 300 mm.

COMMON CONCERNS

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. To minimize risk of groundwater contamination the following management approaches are recommended:

- 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;
- 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.

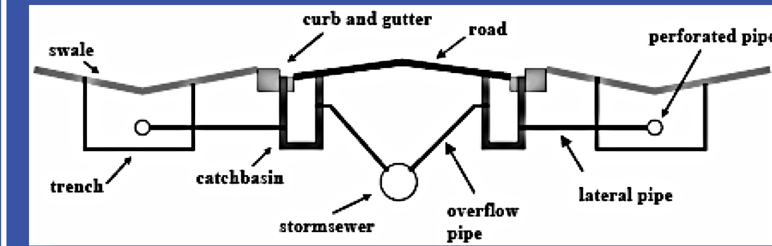
Standing Water and Mosquitoes
Complete drawdown should occur within 72 hours after a storm event, before mosquitoes have an opportunity to breed.

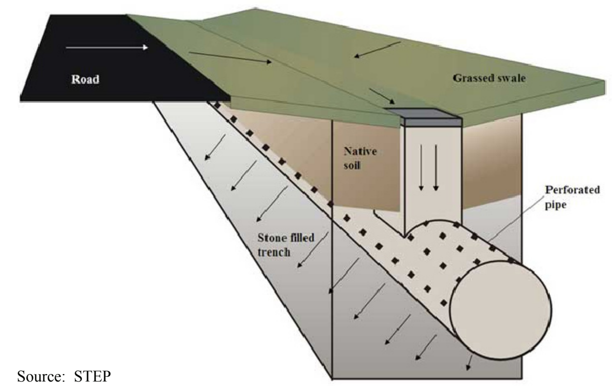
Foundations and Seepage
Should be setback at least four (4) metres from building foundations to prevent basement flooding and damage during freeze/thaw cycles.

Winter Operation
Perforated pipe systems will continue to function during winter months if the inlet pipe and top of the gravel bed is located below the local maximum frost penetration depth.

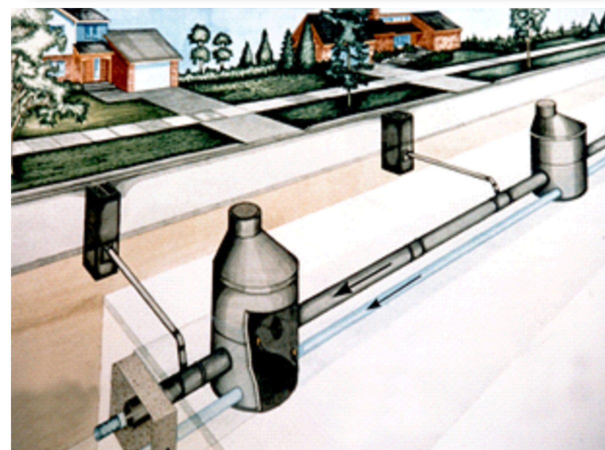
Risk of Soil Contamination
Available evidence from monitoring studies indicates that small distributed stormwater infiltration practices do not contaminate underlying soils, even after 10 years of operation.

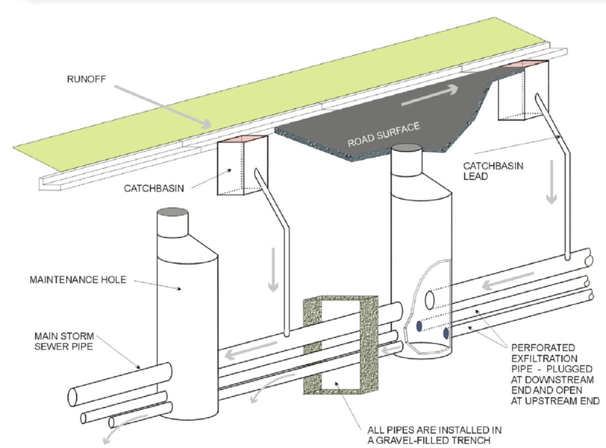
Maintenance
With proper location and adequate pretreatment, perforated pipe systems can continue to function effectively with very low levels of maintenance activities. Like conventional stormwater conveyance infrastructure (i.e., catchbasins and storm sewers), perforated pipe systems are typically located on public property (e.g., within road rights-of-way). An advantage to incorporating these systems in stormwater management systems is that legal agreements with property owners or managers, to ensure long term operation and maintenance, are not needed.





Source: STEP





CONSTRUCTION CONSIDERATIONS

Soil Disturbance and Compaction
Before site work begins, locations of facilities should be clearly marked. Only vehicular traffic used for construction of the infiltration facility should be allowed close to the facility location.

Erosion and Sediment Control
Infiltration practices should never serve as a sediment control device during construction. Construction runoff should be directed away from the proposed facility location. After the site is vegetated, erosion and sediment control structures can be removed.

GENERAL SPECIFICATIONS

Material	Specification	Quantity
Perforated pipe	Pipe should be continuously perforated, smooth interior, with a minimum inside diameter of 200 millimetres.	Perforated pipe should run lengthwise through the facility at least 100 mm above the bottom of the gravel filled trench. Non-perforated pipe should be used for conveyance to the facility.
Stone	The trench in which perforated pipes are installed should be filled with washed 50 mm clear stone with a 40% void ratio.	Volume is based on trench dimensions and a void ratio of 40%.
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; • Maximum forces that will be exerted on the fabric (i.e., what tensile, tear and puncture strength ratings are required?); • Load bearing ratio of the underlying native soil (i.e., is geotextile needed to prevent downward migration of aggregate into the native soil?); • Texture (i.e., grain size distribution) of the overlying native soil, filter media soil or aggregate material; and • Permeability of the native soil. For further guidance see CVC/TRCA LID SWM Planning and Design Guide, Table 4.10.4	Around the gravel filled trench (stone reservoir).

ABILITY TO MEET SWM OBJECTIVES

BMP	Water Balance Benefit	Water Quality Improvement	Stream Channel Erosion Control Benefit
Perforated Pipe Systems	Yes	Yes	Partial, depends on soil infiltration rate

OPERATION AND MAINTENANCE

Maintenance typically consists of cleaning out leaves, debris and accumulated sediment caught in pretreatment devices annually or as needed. Inspection via manholes should be performed to ensure the facility drains within the maximum acceptable length of time (typically 72 hours) at least annually and following every major storm event (>25 mm). If the time required to fully drain exceeds 72 hours, drain via pumping and clean out the perforated pipe by flushing. If slow drainage persists, the system may need removal and replacement of granular material and/or geotextile liner. Perforated pipe systems should be located below shoulders of roadways, pervious boulevards or grass swales where they can be readily excavated for servicing.


Source: STEP


Source: Plastream

SITE CONSIDERATIONS

Site Topography
Systems cannot be located on natural slopes greater than 15%. The gravel bed should be designed with gentle slopes between 0.5. to 1%.

Drainage Area
Typically designed with an impervious drainage area to treatment facility area ratio of between 5:1 to 10:1.

Soil
Perforated pipe systems 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. Native soil infiltration rate at the proposed facility location and depth should be confirmed through measurement of hydraulic conductivity under field saturated conditions.

Wellhead Protection
Facilities receiving road or parking lot runoff should not be located within two (2) year time-of-travel wellhead protection areas.

Water Table
The bottom of the gravel bed should be separated from the seasonally high water table or top of bedrock elevation by at least one (1) metre to prevent groundwater contamination.

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 perforated pipe systems.

Setback from Buildings
Facilities should be setback a minimum of four (4) metres from building foundations.

Proximity to Underground Utilities
Local utility design guidance should be consulted to define the horizontal and vertical offsets. Generally, requirements for underground utilities passing near the practice will be no different than for utilities in other pervious areas.

TORONTO AND REGION Conservation for The Living City

CVC CREDIT VALLEY CONSERVATION

FOR FURTHER DETAILS SEE SECTION 4.10 OF THE CVC/TRCA LID SWM GUIDE

◆ Verification Statement ◆ Verification Statement ◆ Verification Statement ◆ Verification Statement ◆

VERIFICATION STATEMENT

GLOBE Performance Solutions

Verifies the performance of

Jellyfish® Filter

Developed by Imbrium Systems, Inc.,
Whitby, Ontario, Canada

Registration: GPS-ETV_VR2023-08-31_Imbrium-JF

In accordance with

ISO 14034:2016

Environmental Management —
Environmental Technology Verification (ETV)

John D. Wiebe, PhD
Executive Chairman
GLOBE Performance Solutions

GLOBE PERFORMANCE SOLUTIONS

August 15, 2023
Vancouver, BC, Canada

Verification Body
GLOBE Performance Solutions
404 – 999 Canada Place | Vancouver, B.C | Canada |V6C 3E2

Technology description and application

The Jellyfish® Filter is an engineered stormwater quality treatment technology designed to remove a variety of stormwater pollutants including floatable trash and debris, oil, coarse and fine suspended sediments, and particulate-bound pollutants such as nutrients, heavy metals, and hydrocarbons. The Jellyfish Filter combines gravitational pre-treatment (sedimentation and floatation) and membrane filtration in a single compact structure. The system utilizes membrane filtration cartridges comprised of multiple pleated filter elements ("filtration tentacles") that provide high filtration surface area with the associated advantages of high flow rate, high sediment capacity, and low filtration flux rate.

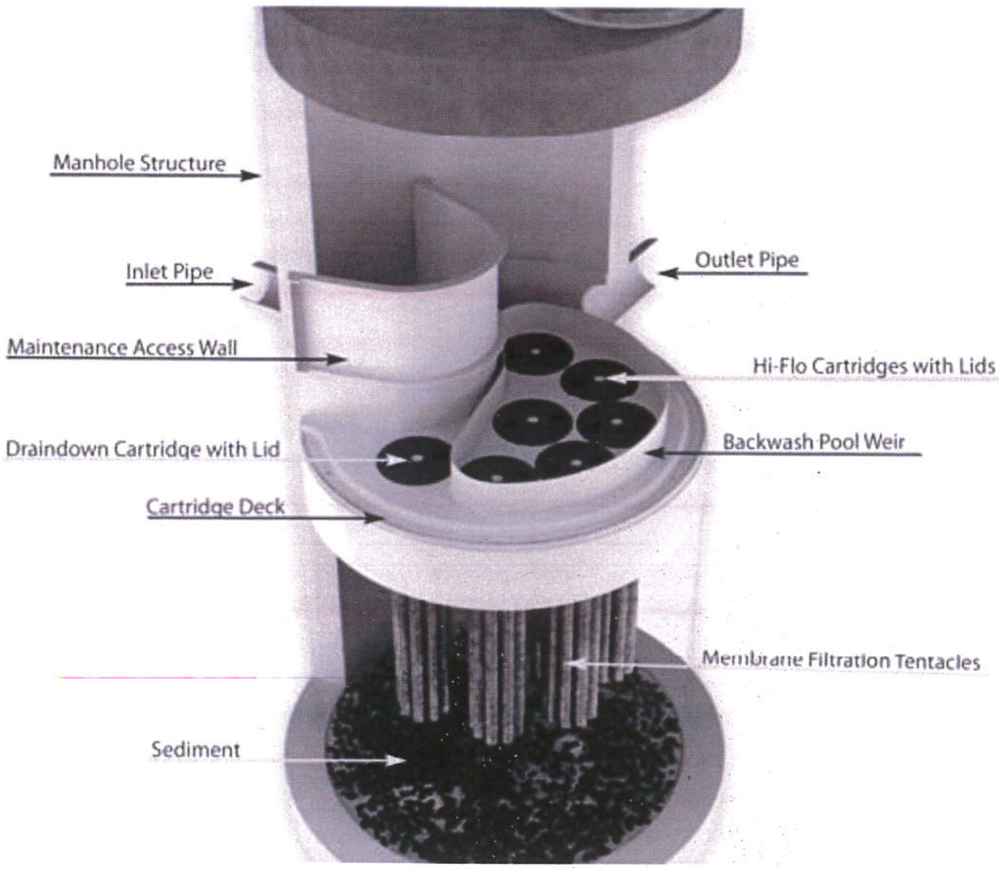


Figure 1. Cut-away graphic of a Jellyfish® Filter manhole with 6 hi-flo cartridges and 1 draindown cartridge

Figure 1 depicts a cut-away graphic of a typical 6-ft diameter Jellyfish® Filter manhole with 6 hi-flo cartridges and 1 draindown cartridge (JF6-6-1). Stormwater influent enters the system through the inlet pipe and builds a pond behind the maintenance access wall, with the pond elevation providing driving head. Flow is channeled downward into the lower chamber beneath the cartridge deck. A flexible separator skirt (not shown in the graphic) surrounds the filtration zone where the filtration tentacles of each cartridge are suspended, and the volume between the vessel wall and the outside surface of the separator skirt comprises a pretreatment channel. As flow spreads throughout the pretreatment channel, floatable pollutants accumulate at the surface of the pond behind the maintenance access wall and also beneath the cartridge deck in the pretreatment channel, while coarse sediments settle to the sump. Flow proceeds under the separator skirt and upward into the filtration zone, entering each filtration tentacle and depositing fine suspended sediment and associated particulate-bound pollutants on the outside surface of the membranes. Filtered water proceeds up the center tube of each tentacle, with the flow from each tentacle combining under the cartridge lid, and discharging to the top of the cartridge deck through the cartridge lid orifice. Filtered effluent from the hi-flo cartridges enters a pool enclosed by a 15-cm high weir, and if storm intensity and resultant driving head is sufficient, filtered water overflows the weir and proceeds across the cartridge deck to the outlet pipe. Filtered effluent discharging from the draindown cartridge(s) passes directly to the outlet pipe, and requires only a minimal amount of driving head (2.5 cm) to provide forward flow. As storm intensity subsides and driving head drops below 15 cm, filtered water within the backwash pool reverses direction and passes backward through the hi-flo cartridges, and thereby dislodges sediment from the membranes which subsequently settles to the sump below the filtration zone. During this passive backwashing process, water in the lower chamber is displaced only through the draindown cartridge(s). Additional self-cleaning processes include gravity, as well as vibrational pulses emitted when flow exits the orifice of each cartridge lid, and these combined processes significantly extend the cartridge service life and maintenance cleaning interval. Sediment removal from the sump by vacuum is required when sediment depths reach 30 cm, and cartridges are typically removed, externally rinsed, and recommissioned on an annual basis, or as site-specific maintenance conditions require. Filtration tentacle replacement is typically required every 3 – 5 years.

Performance conditions

The data and results published in this Technology Fact Sheet were obtained from a field monitoring program conducted on a Jellyfish® Filter JF4-2-1 (4-ft diameter manhole with 2 hi-flo cartridges and 1 draindown cartridge), in accordance with the provisions of the TARP Tier II Protocol (TARP, 2003) and New Jersey Tier II Stormwater Test Requirements—Amendments to TARP Tier II Protocol (NJDEP, 2009). Testing was completed by researchers led by Dr. John Sansalone at the University of Florida's Engineering School of Sustainable Infrastructure and Environment. The drainage area providing stormwater runoff to the test unit varied between 502 m^2 and 799 m^2 (5400 ft^2 to 8600 ft^2) depending on storm intensity and wind direction. The unit was monitored for a total of 25 TARP qualifying storm events (i.e. $\geq$ 2.5 mm of rainfall) contributing cumulative rainfall of 381 mm (15 in) over the 13-month period between May 28, 2010 and June 27, 2011. Only TARP-qualified storms were routed through the unit, and maintenance was not required during the testing period based on sediment accumulation less than the depth indicated for maintenance, and also based on hydraulic testing performed on the system after the conclusion of monitoring.

Table 1 shows the specified and achieved amended TARP criteria for storm selection and sampling. **Table 2** shows the observed ranges of operational conditions that occurred over the testing period.

Table 1. Specified and achieved amended TARP criteria for storm selection and sampling

Description	Criteria value	Achieved value
Total rainfall	$\geq$ 2.5 mm (0.1 in)	> 2.5 mm (0.1 in)
Minimum inter-event period	6 hrs	10 hrs
Minimum flow-weighted composite sample storm coverage	70% including as much of the first 20% of the storm	100%
Minimum influent/effluent samples	10, but a minimum of 5 subsamples for composite samples	Minimum of 8 subsamples for composite samples
Total sampled rainfall	Minimum 381 mm (15 in)	384 mm (15.01 in)
Number of storms	Minimum 20	25

Table 2. Observed operational conditions for events monitored over the study period

Operational condition	Observed range
Storm durations	26 – 691 min
Previous dry hours	10 - 910 hrs
Rainfall depth	3 – 50 mm
Initial rainfall to runoff lag time	1 – 34 min
Runoff volume	206 – 13,229 L
Peak rainfall intensity	5 – 137 mm/hr
Peak runoff flow rate	0.5 – 14.3 L/s
Event median flow rate	0.01 – 5.5 L/s

The 4-ft diameter test unit has sedimentation surface area of 1.17 m^2 (12.56 ft^2). Each of the three filter cartridges employed in the test unit uses filtration tentacles of 137 cm (54 in) length, with filter surface area of 35.4 m^2 (381 ft^2) per cartridge, and total filter surface area of 106.2 m^2 (1143 ft^2) for the three cartridges combined. The design treatment flow rate is 5 L/s (80 gal/min) for each of the two hi-flo cartridges and 2.5 L/s (40 gal/min) for the single draindown cartridge, for a total design treatment flow rate of 12.6 L/s (200 gal/min) at design driving head of 457 mm (18 in). This translates to a filtration flux rate (flow rate per unit filter surface area) of 0.14 L/s/m^2 (0.21 gal/min/ft^2) for each hi-flo cartridge and 0.07 L/s/m^2 (0.11 gal/min/ft^2) for the draindown cartridge. The design flow rate for each cartridge is controlled by the sizing of the orifice in the cartridge lid. The distance from the bottom of the filtration tentacles to the sump is 61 cm (24 in).

Performance claims

The Jellyfish® Filter demonstrated the removal efficiencies indicated in **Table 3** for respective constituents during field monitoring of 25 TARP qualified storm events with cumulative rainfall of 381 mm, conducted in accordance with the provisions of the TARP Tier II Protocol (TARP, 2003) and New Jersey Tier II Stormwater Test Requirements—Amendments to TARP Tier II Protocol (NJDEP, 2009), and using the following design parameters:

- System hydraulic loading rate (system treatment flow rate per unit of sedimentation surface area) of 10.8 L/s/m^2 (15.9 gal/min/ft^2) or lower
- Filtration flux rate (flow rate per unit filter surface area) of 0.14 L/s/m^2 (0.21 gal/min/ft^2) or lower for each hi-flo cartridge and 0.07 L/s/m^2 (0.11 gal/min/ft^2) or lower for each draindown cartridge
- Distance from the bottom of the filtration tentacles to the sump of 61 cm (24 in) or greater
- Driving head of 457 mm (18 in) or greater

Table 3. Mean, median and 95% confidence interval (median) for removal efficiencies of selected stormwater constituents

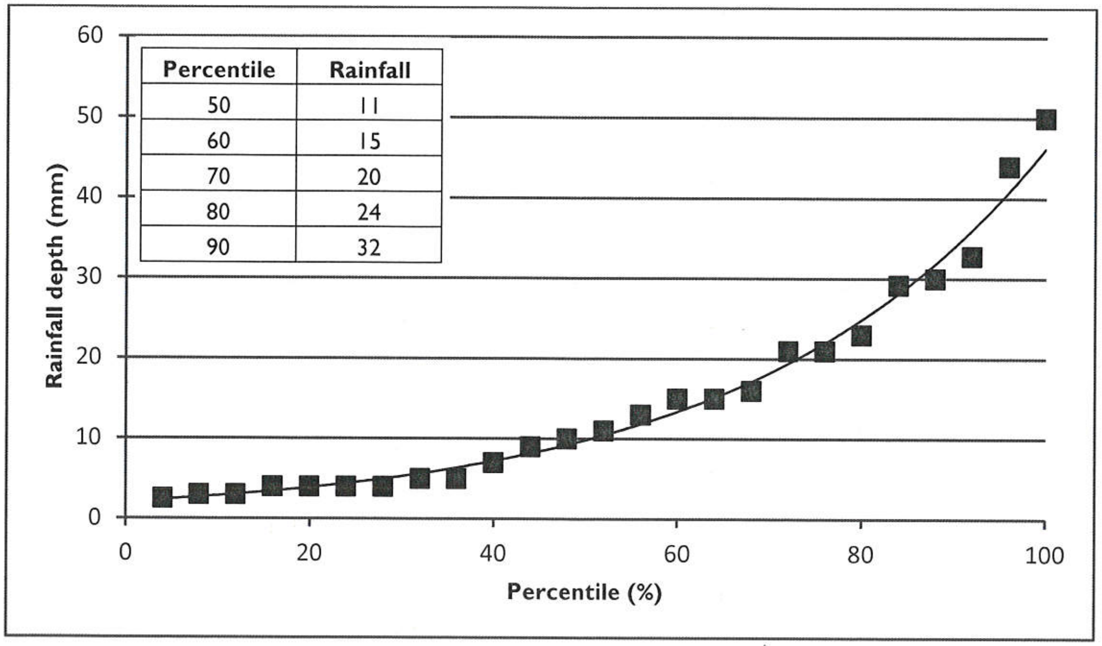
Parameter	Mean	Median	Median - 95% Lower Limit	Median - 95% Upper Limit
TSS	84.7	85.6	82.8	89.8
SSC	97.5	98.3	97.1	98.7
Total phosphorus	48.8	49.1	43.3	60.1
Total nitrogen	37.9	39.3	31.2	54.6
Zinc	55.3	69	39	75
Copper	83.0	91.7	75.1	98.9
Oil and grease	60.1	60	42.7	100

N.B. As with any field test of stormwater treatment devices, removal efficiencies will vary based on pollutant influent concentrations and other site specific conditions.

❖The performance claims can be applied to other Jellyfish® Filter models smaller or larger than the tested model as long as the untested models are designed in accordance with the design parameters specified in the performance claims.

Performance results

The frequency of rainfall depths monitored during the study is presented in **Figure 2**. The median and 90th percentile rainfall depths were 11 mm and 31.7 mm, respectively. These values represent the depth of rainfall that is not exceeded in 50 and 90 percent of the monitored rainfall events.



Percentile	Rainfall
50	11
60	15
70	20
80	24
90	32

Figure 2. Rainfall depth frequency curve

Sediment removal performance was assessed by measuring the event mean concentration and mass of suspended sediment entering and leaving the unit during runoff events. This involved sampling the full cross-section of influent and effluent flows manually at 2 - 10 minute intervals for the full duration of each storm event and combining discrete samples into flow-weighted composites. Comparing the theoretical mass recovery from the sump calculated by the difference between the influent and effluent mass to the actual dry weight of the recovered sump mass showed an overall mass balance recovery of 94.5% over the study period.

The median d50 particle size (i.e. 50th percentile particle size) of the influent and effluent was 82 and 3 µm, respectively (**Figure 3**). The median influent particles sizes ranged between 22 and 263 µm, whereas median effluent particle sizes ranged between 1 and 11 µm.

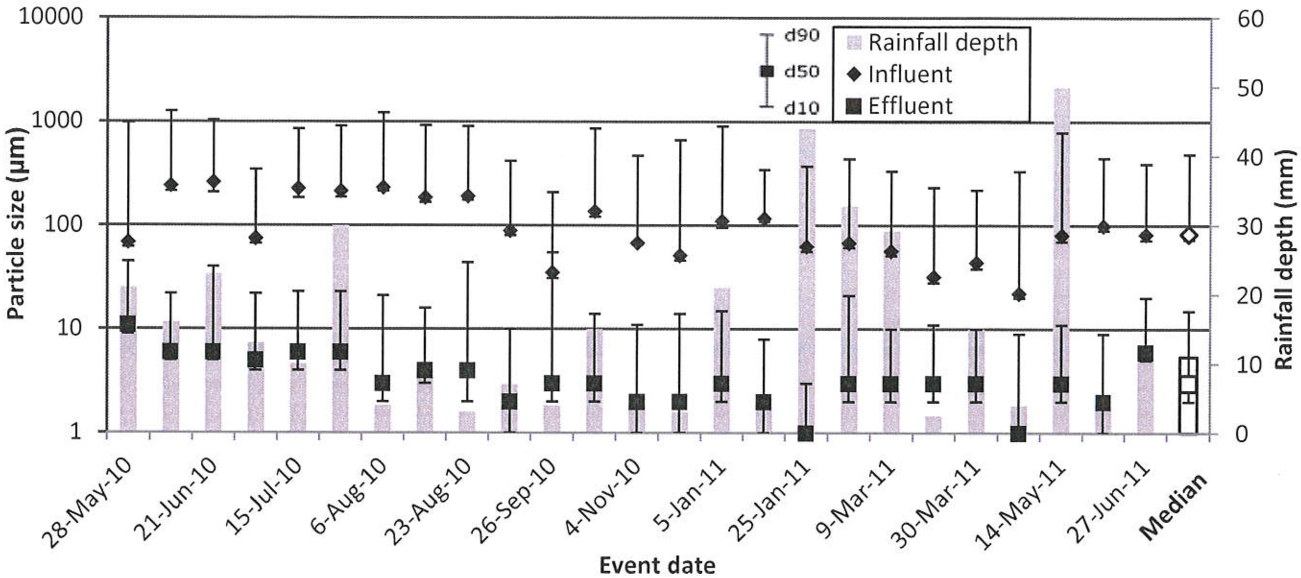


Figure 3. The rainfall depth and d10, d50, and d90 particle sizes of the influent and effluent composite samples for each monitored storm event over the 13-month testing period

Sampling of flows into and out of the Jellyfish Filter over the testing period showed statistically significant reductions ($p < 0.05$; Wilcoxon signed-rank test) in influent event mean concentrations for all selected stormwater constituents (**Table 4** and **Figure 4**). Effluent event mean Suspended Sediment Concentrations (SSC) were below 19 mg/L during all monitored events. Load-based removal rates were also calculated based on the sum of loads over the study period. These removal rages ranged from 46.3 for Total Nitrogen to 98.6 for SSC (**Table 4**).

Table 4. Summary statistics for influent and effluent event mean concentrations for selected constituents

Water Quality Variable	Sampling Location	Min	Max	Median	Range	Mean	SD	Load based removal efficiency (%)
TSS	Influent (mg/L)	16.30	261.00	79.30	244.70	86.26	51.37	87.2
	Effluent (mg/L)	3.20	21.70	11.80	18.50	10.99	4.79	
SSC	Influent (mg/L)	78.20	1401.70	444.50	1323.50	482.26	338.34	98.6
	Effluent (mg/L)	2.80	18.10	7.30	15.30	7.88	3.77	
TP	Influent (µg/L)	887.00	8793.00	3063.00	7906.00	3550.20	1914.50	64.2
	Effluent (µg/L)	472.00	4769.00	1480.00	4297.00	1688.08	1059.98	
TN	Influent (µg/L)	1170.00	10479.00	3110.00	9309.00	3519.32	2161.47	46.3
	Effluent (µg/L)	553.00	6579.00	1610.00	6026.00	2091.76	1613.61	
Zn	Influent (µg/L)	0.005	7600.00	1500.00	7600.00	1792.00	1852.91	76.1
	Effluent (µg/L)	0.005	2760.00	450.00	2760.00	561.64	594.70	
Cu	Influent (µg/L)	0.001	880.40	79.50	880.40	171.28	229.33	92.1
	Effluent (µg/L)	0.001	51.30	6.90	51.30	14.36	17.22	
Oil and Grease	Influent (mg/L)	0.20	4.06	0.93	3.86	1.07	0.82	46.4
	Effluent (mg/L)	0.00	2.32	0.35	2.32	0.50	0.60	

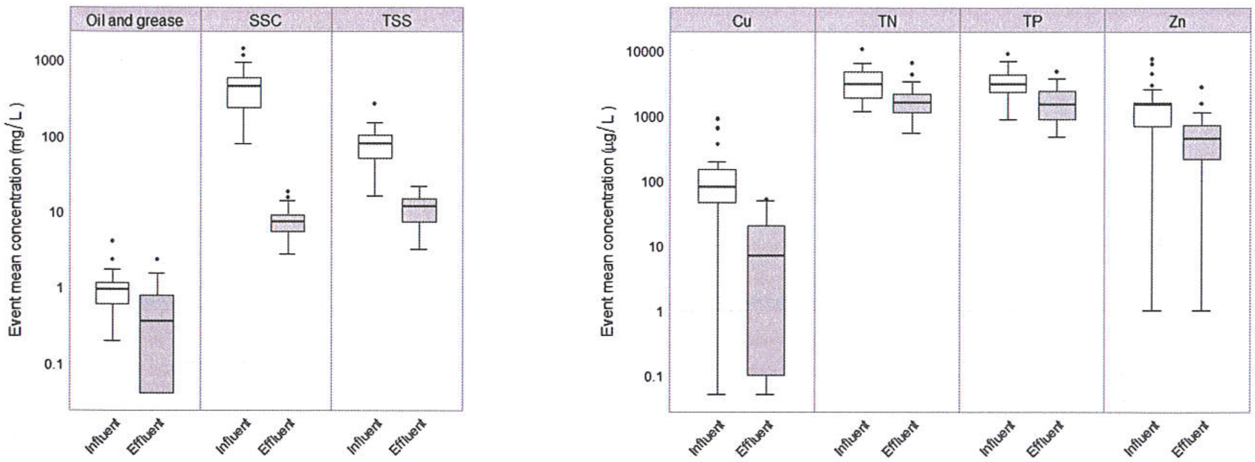


Figure 4. Boxplots showing the distribution of influent and effluent event mean concentrations (EMC) for selected stormwater constituents over the study period

Verification

The verification was completed by the Verification Expert, Toronto and Region Conservation Authority, contracted by GLOBE Performance Solutions, using the International Standard ***ISO 14034:2016 Environmental Management -- Environmental Technology Verification (ETV)***. Data and information provided by Imbrium Systems to support the performance claim included the performance monitoring report prepared by University of Florida, Engineering School of Sustainable Infrastructure and Environment, and dated November 2011. This report is based on testing completed in accordance with the Technology Acceptance Reciprocity Partnership (TARP) Tier II Protocol (2003) and New Jersey Tier II Stormwater Test Requirements--Amendments to TARP Tier II Protocol (NJDEP, 2009).

What is ISO14034:2016 Environmental Management – Environmental Technology Verification (ETV)?

ISO 14034:2016 specifies principles, procedures and requirements for Environmental Technology Verification (ETV), and was developed and published by the *International Organization for Standardization (ISO)*. The objective of ETV is to provide credible, reliable and independent verification of the performance of environmental technologies. An environmental technology is a technology that either results in an environmental added value or measures parameters that indicate an environmental impact. Such technologies have an increasingly important role in addressing environmental challenges and achieving sustainable development.

For more information on the Jellyfish® Filter please contact:

Imbrium Systems, Inc.
407 Fairview Drive
Whitby, ON
L1N 3A9, Canada
Tel: 416-960-9900
info@imbriumsystems.com

For more information on ISO 14034:2016 / ETV please contact:

GLOBE Performance Solutions
404 – 999 Canada Place
Vancouver, BC
V6C 3E2 Canada
Tel: 604-695-5018 / Toll Free: 1-855-695-5018
etv@globeperformance.com

Limitation of verification - Registration: GPS-ETV_VR2023-08-31_Imbrium-JF
GLOBE Performance Solutions and the Verification Expert provide the verification services solely on the basis of the information supplied by the applicant or vendor and assume no liability thereafter. The responsibility for the information supplied remains solely with the applicant or vendor and the liability for the purchase, installation, and operation (whether consequential or otherwise) is not transferred to any other party as a result of the verification.

APPENDIX E

Figure S-1a – Conceptual Servicing Layout Plan
Figure S-1b – Conceptual Phase 1 Servicing Plan

Figure S-2 – Conceptual Grading Plans

Figure S-3 – Existing conditions Sanitary Tributary Plan
Figure S-4 – Phase 1 conditions Sanitary Tributary Plan
Figure S-5 – Conceptual conditions Sanitary Tributary Plan

Figure S-6 – Existing Storm Drainage Boundary Plan
Figure S-7 – Conceptual Storm Tributary Area Plan

Figure S-8 – Notion Road Profile – 450mm Interim Sanitary Sewers
Figure S-9 – Pickering Parkway – Profile 1/2
Figure S-10 – Pickering Parkway – Profile 2/2

City of Pickering Storm Tributary Area Plans
MTO Plan & Profiles

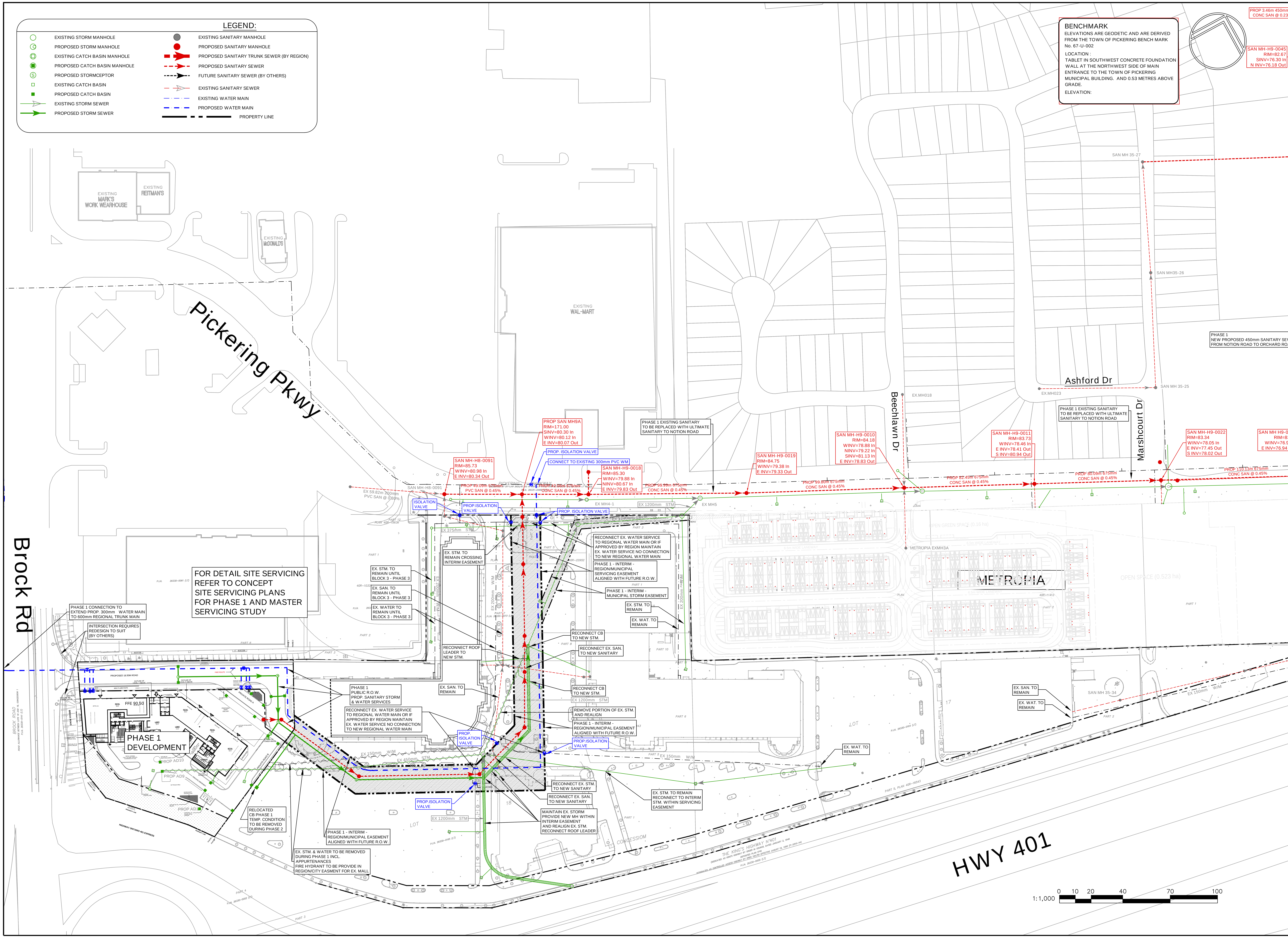


FIGURE PH1

PHASE 1 SITE SEVICING & EASEMENT PLAN

PICKERING BRIDGE LANDS INC.
PROPOSED MIX-USE DEVELOPMENT
1755 + 1805 PICKERING PARKWAY
PICKERING, ON

DATE:	PROJ. NO.:	SCALE:
JAN 2025	20266	1:1000

ODAN·DETECH CONSULTING ENGINEERS

The Odan/Detech Group Inc. P: (905) 632-3811 F: (905) 632-3363
5230 SOUTH SERVICE ROAD, BURLINGTON, ONTARIO, L7L 5K2

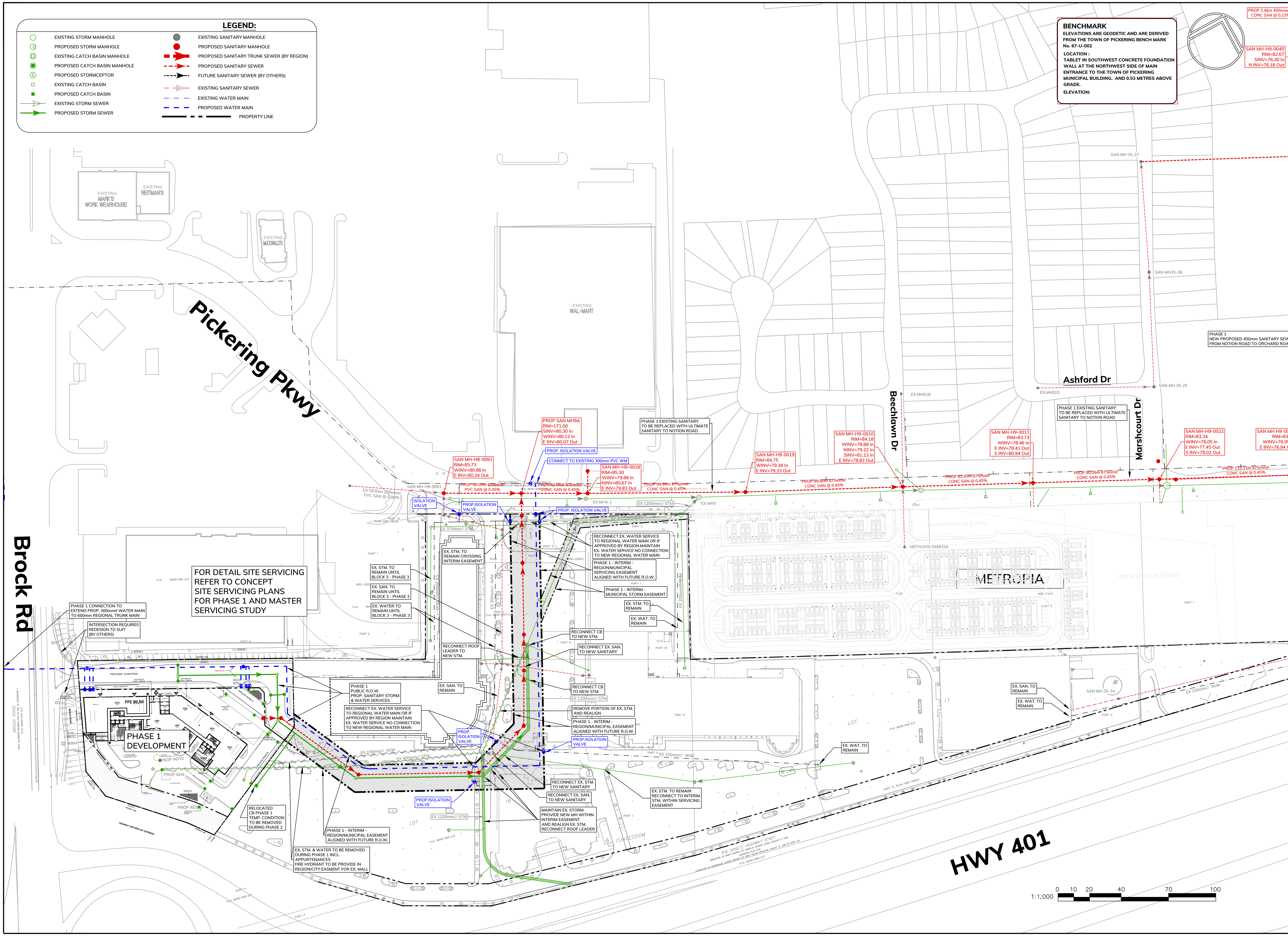


FIGURE PH1

PHASE 1 SITE SEVICING & EASEMENT PLAN

PICKERING BRIDGE LANDS INC.
PROPOSED MIX-USE DEVELOPMENT
1755 + 1805 PICKERING PARKWAY
PICKERING, ON

DATE:	PROJ. NO.:	SCALE:
JAN 2025	20266	1:1000

ODAN·DETECH CONSULTING ENGINEERS

BENCHMARK
ELEVATIONS ARE GEODETIC AND ARE DERIVED FROM THE TOWN OF PICKERING BENCH MARK No. 67-U-002

LOCATION :
TABLET IN SOUTHWEST CONCRETE FOUNDATION WALL AT THE NORTHWEST SIDE OF MAIN ENTRANCE TO THE TOWN OF PICKERING MUNICIPAL BUILDING. AND 0.53 METRES ABOVE GRADE.

ELEVATION:
PUBLISHED ELEVATION =87.581 metres.

LEGEND:

EXISTING STORM MANHOLE	EXISTING SANITARY MANHOLE
PROPOSED STORM MANHOLE	PROPOSED SANITARY MANHOLE
EXISTING CATCH BASIN MANHOLE	PROPOSED SANITARY TRUNK SEWER (BY REGION)
PROPOSED CATCH BASIN MANHOLE	PROPOSED SANITARY SEWER
PROPOSED STORMCEPTOR	FUTURE SANITARY SEWER (BY OTHERS)
EXISTING CATCH BASIN	EXISTING SANITARY SEWER
PROPOSED CATCH BASIN	EXISTING WATER MAIN
EXISTING STORM SEWER	PROPOSED WATER MAIN
PROPOSED STORM SEWER	PROPERTY LINE

1:1,000 0 10 20 40 70 100

FIGURE S-1a
CONCEPTUAL SERVICING LAYOUT PLAN

PICKERING BRIDGE LANDS INC.
PROPOSED MIX-USE DEVELOPMENT
1755 + 1805 PICKERING PARKWAY
PICKERING, ON

DATE:	PROJ. NO.:	SCALE:
SEPT 2021	20266	1:1000

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5230 SOUTH SERVICE ROAD, BURLINGTON, ONTARIO, L7L 5K2

FIGURE S-1b

SITE SERVICING PHASE 1
LAYOUT PLAN

PICKERING BRIDGE LANDS INC.
PROPOSED MIX-USE DEVELOPMENT
1755 + 1805 PICKERING PARKWAY
PICKERING, ON

DATE:	PROJ. NO.:	SCALE:
SEPT 2021	20266	1:1000

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5230 SOUTH SERVICE ROAD, BURLINGTON, ONTARIO, L7L 5K2

GRADING LEGEND:

○	EXISTING STORM MANHOLE	⊠	EXISTING WATER VALVE & BOX	× (100.00)	PROPOSED SWALE INVERT ELEVATION
	PROPOSED STORM MANHOLE		PROPOSED WATER VALVE & BOX	1.0%	PROPOSED FLOW ARROW AND SLOPE
	EXISTING CATCH BASIN MANHOLE	○	EXISTING VALVE CHAMBER	➡	PROPOSED EMERGENCY OVERLAND FLOW
	PROPOSED CATCH BASIN MANHOLE		BOREHOLE LOCATION	100.50	EXISTING CONTOUR
	PROPOSED STORMCEPTOR		PROPOSED VALVE CHAMBER		PROPOSED SLOPE (3:1 OR HIGHER)
□	EXISTING CATCH BASIN	× (100.00)	FUTURE ELEVATION		DENOTES PROPOSED PROPERTY LINE
■	PROPOSED CATCH BASIN	× 100.00	EXISTING SPOT ELEVATION		DENOTES PROPOSED LIMIT OF CONSTRUCTION
	PROPOSED AREA DRAIN	× (100.00)	PROPOSED ELEVATION		DENOTES PROPOSED HEAVY DUTY ASPHALT AREA
●	EXISTING SANITARY MANHOLE	× (100.00)TC	PROPOSED TOP OF CURB ELEVATION		
●	PROPOSED SANITARY MANHOLE	× (100.00)GL	PROPOSED GUTTER LINE ELEVATION		
	EXISTING HYDRANT	× (100.00)HP	PROPOSED HIGH POINT		
	PROPOSED HYDRANT	× (100.00)LP	PROPOSED LOW POINT		

FIGURE S-2

CONCEPTUAL GRADING PLAN (ULTIMATE)

PICKERING BRIDGE LANDS INC.
PROPOSED MIX-USE DEVELOPMENT
1755 + 1805 PICKERING PARKWAY
PICKERING, ON

DATE:	PROJ. NO.:	SCALE:
SEPT 2021	20266	1:1000

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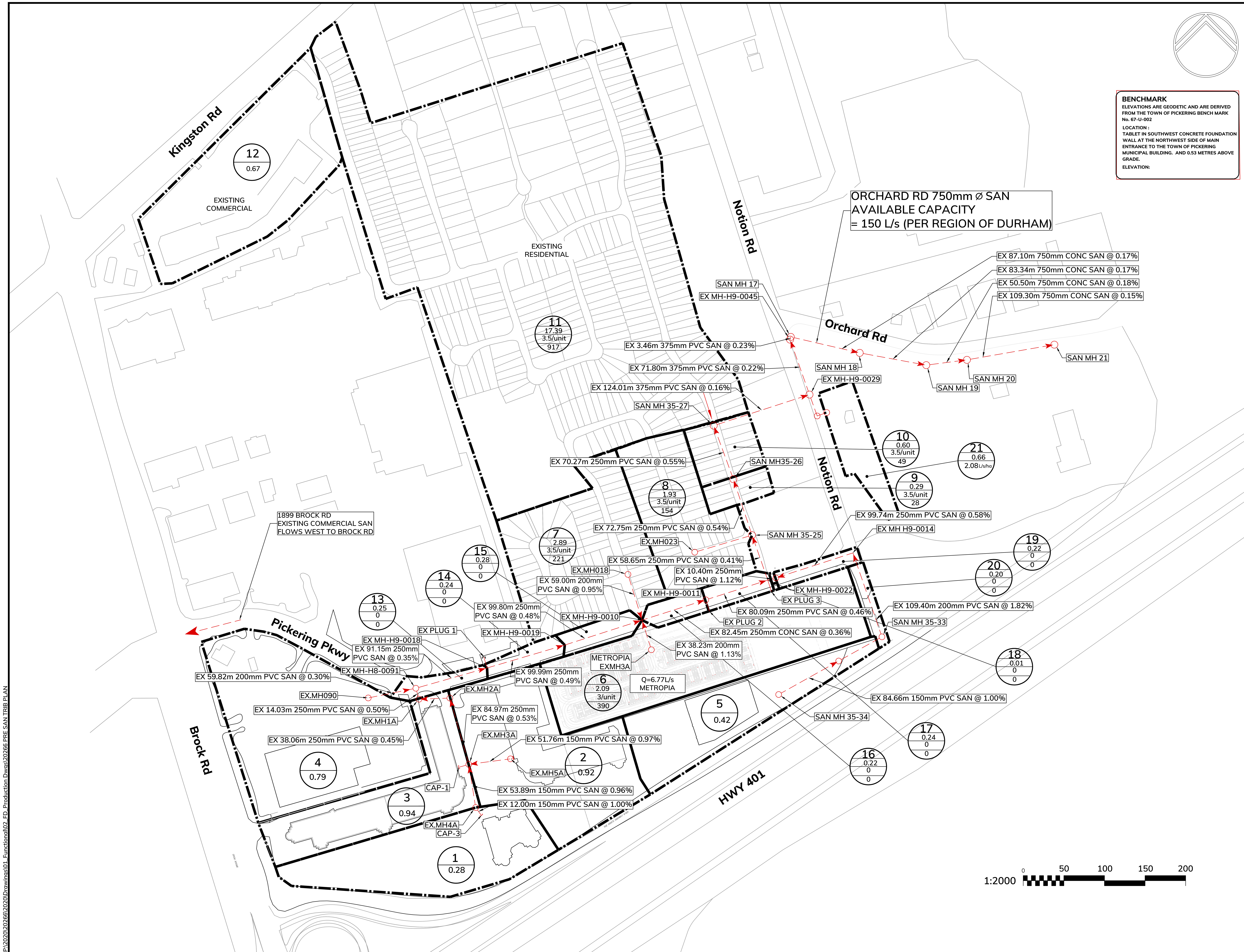
CONSULTING ENGINEERS



LEGEND

- EXISTING SANITARY MANHOLE
- EXISTING SANITARY SEWER
- EXISTING DRAINAGE AREA

COMMERCIAL

5 — TRIBUTARY AREA ID NO.
0.42 — GROSS FLOOR AREA (ha)

RESIDENTIAL

9 — TRIBUTARY AREA ID NO.
0.29 — TRIBUTARY AREA (ha)
60 — POPULATION DENSITY (Persons/ha)
17 — EQUIVALENT POPULATION

INDUSTRIAL

9 — TRIBUTARY AREA ID NO.
0.29 — TRIBUTARY AREA (ha)
60 — L/s
17 — EQUIVALENT POPULATION

FIGURE S-3

EXISTING CONDITIONS SANITARY TRIBUTARY PLAN

PICKERING BRIDGE LANDS INC.
PROPOSED MIX-USE DEVELOPMENT
1755 + 1805 PICKERING PARKWAY
PICKERING, ON

DATE:	PROJ. NO.:	SCALE:
SEPT 2021	20266	1:2000

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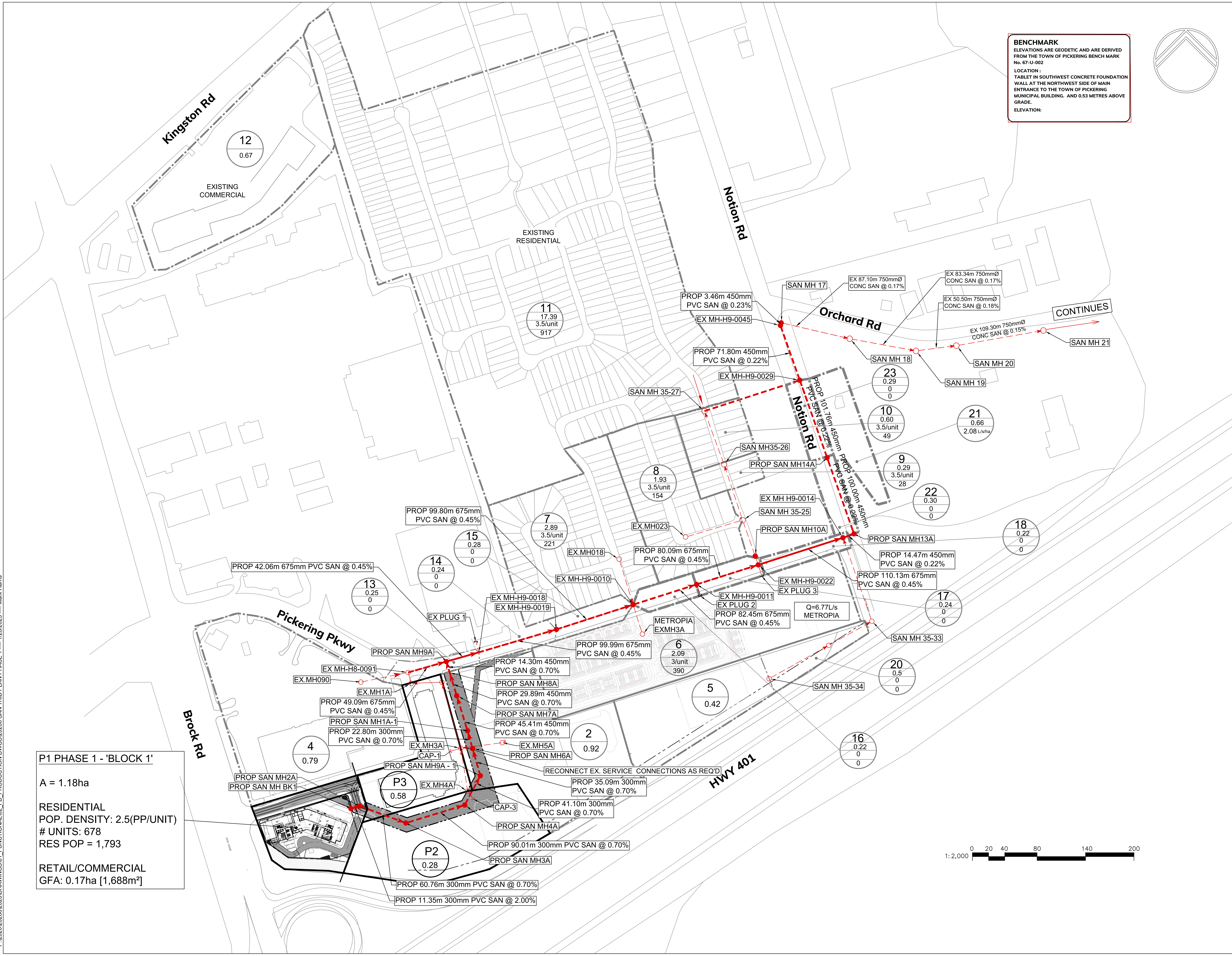

FIGURE S-4
PHASE 1 CONDITIONS
SANITARY TRIBUTARY PLAN

PICKERING BRIDGE LANDS INC.
PROPOSED MIX-USE DEVELOPMENT
1755 + 1805 PICKERING PARKWAY
PICKERING, ON

DATE:	PROJ. NO.:	SCALE:
SEPT 2021	20266	1:2000

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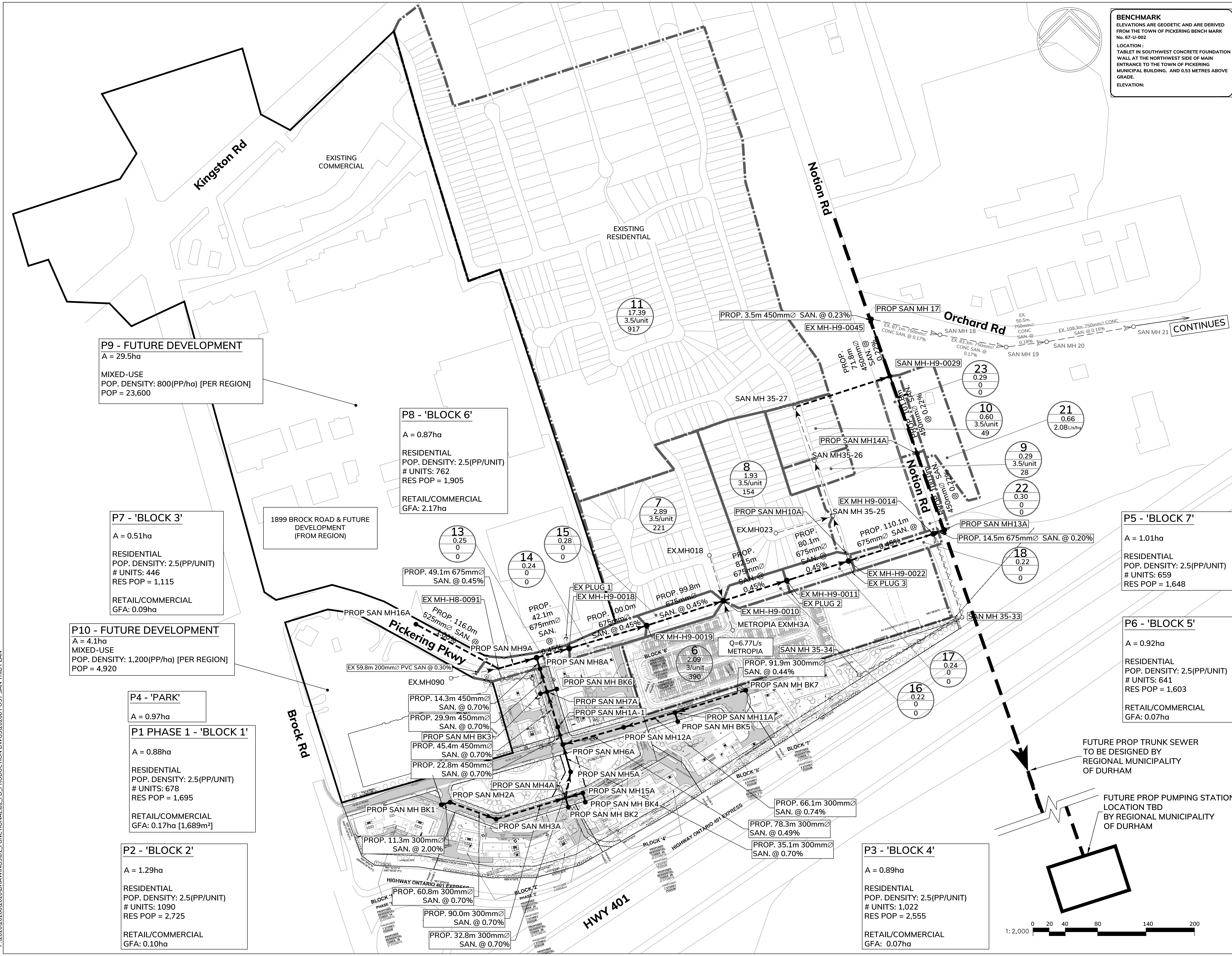
P:\2020\20266\2020\DRAWINGS\01_FUNCTIONAL\02_FD_PRODUCTION DWGS\20266 SAN TRIB PLAN PHASE 1 — 1/28/2025 — Mark Harris



LEGEND

- EXISTING SANITARY MANHOLE
- PROPOSED SANITARY MANHOLE
- EXISTING SANITARY SEWER
- PROPOSED SANITARY SEWER
- REGION TRUNK SEWER
- SANITARY SEWER REMOVALS
- EXISTING DRAINAGE AREA
- PHASED DRAINAGE AREA

COMMERCIAL

5 — TRIBUTARY AREA ID NO.
0.42 — GROSS FLOOR AREA (ha)

RESIDENTIAL

9 — TRIBUTARY AREA ID NO.
0.29 — TRIBUTARY AREA (ha)
60 — POPULATION DENSITY (Persons/ha)
17 — EQUIVALENT POPULATION

FIGURE S-5
CONCEPTUAL CONDITIONS SANITARY TRIBUTARY PLAN

PICKERING BRIDGE LANDS INC.
PROPOSED MIX-USE DEVELOPMENT
1755 + 1805 PICKERING PARKWAY
PICKERING, ON

DATE:	PROJ. NO.:	SCALE:
SEPT 2021	20266	1:2000

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EX MH10
EX MH11
EX MH9
EX MH7
EX PLUG 2
EX MH8
EX PLUG 1
EX MH6
EX MH5
EX MH4-1
EX MH4
EX.MHS1
METROPIA
EX.MHS2
EX.CB8
EX.CB6
EX.CB7
EX.CB7-1
EX.MHS8
EX.CB11
EX.CBMHS9
EX.CB5
EX.DCB9
EX.DCB4
EX.MHS3
EX.MHS6
EX.DCBMHS7
EX.CBMHS5
EX.CB1-2
EX.CB2
EX.DCB3
EX.CB10
EX.MHS4
EX.CB1
EXISTING WAL-MART
EXISTING McDONALD'S
EXISTING CANADIAN TIRE
EX 900mm∅ STM
EX 2100mm∅ STM
EX 1350mm∅ STM
EX 600mm∅ STM
EX 675mm∅ STM
EX 450mm∅ STM
EX 1200mm∅ STM
EX 750mm∅ STM
EX 375mm∅ STM
EX 16.05m 1200mm∅ CONC STM @ 1.26%
EX 66.01m 1200mm∅ CONC STM @ 0.50%
EX 97.02m 300mm∅ CONC STM @ 2.25%
EX 157.21m 1200mm∅ CONC STM @ 0.46%
EX 72.04m 300mm∅ CONC STM @ 1.19%
EX 88.26m 450mm∅ PVC STM @ 1.31%
EX 120.97m 450mm∅ PVC STM @ 1.36%
EX 101.42m 375mm∅ CONC STM @ 0.80%
EX 135.61m 375mm∅ CONC STM @ 0.41%
EX 1200mm∅ STM
EX 39.86m 1200mm∅ CONC STM @ 2.40%
EL.85.92
EL.86.99
EL.86.48
EL.86.91
EL.90.43
EL.88.05
BENCHMARK
ELEVATIONS ARE GEODETIC AND ARE DERIVED FROM THE TOWN OF PICKERING BENCH MARK No. 67-U-002
LOCATION :
TABLET IN SOUTHWEST CONCRETE FOUNDATION WALL AT THE NORTHWEST SIDE OF MAIN ENTRANCE TO THE TOWN OF PICKERING MUNICIPAL BUILDING. AND 0.53 METRES ABOVE GRADE.
ELEVATION:
LEGEND
EXISTING STORM MANHOLE
EXISTING CATCH BASIN
EXISTING DRAINAGE AREA
EXISTING STORM SEWER
SURFACE FLOW DIRECTION
EXISTING OVERLAND FLOW DIRECTION
NOTE:
PLAN PROVIDED FOR REFERENCE ONLY
AS PER CITY OF PICKERING FLOWS ARE TO BE CONTROLLED
TO C=0.50 FOR BLOCKS 1 TO 7 POST DEVELOPMENT
THEREFORE PREDEVELOPMENT RUNOFF COEFFICIENTS AND
IMPERVIOUSNESS DO NOT DETERMINE ALLOWABLE FLOWS
PLAN PROVIDED TO SHOW BOUNDARY OF DEVLEPOMENT ONLY
0 10 20 30 40 70 100
FIGURE S-6
EXISTING STORM TRIBUTARY PLAN
PICKERING BRIDGE LANDS INC.
PROPOSED MIX-USE DEVELOPMENT
1755 + 1805 PICKERING PARKWAY
PICKERING, ON
DATE: SEPT 2021
PROJ. NO.: 20266
SCALE: NTS
ODAN·DETECH CONSULTING ENGINEERS

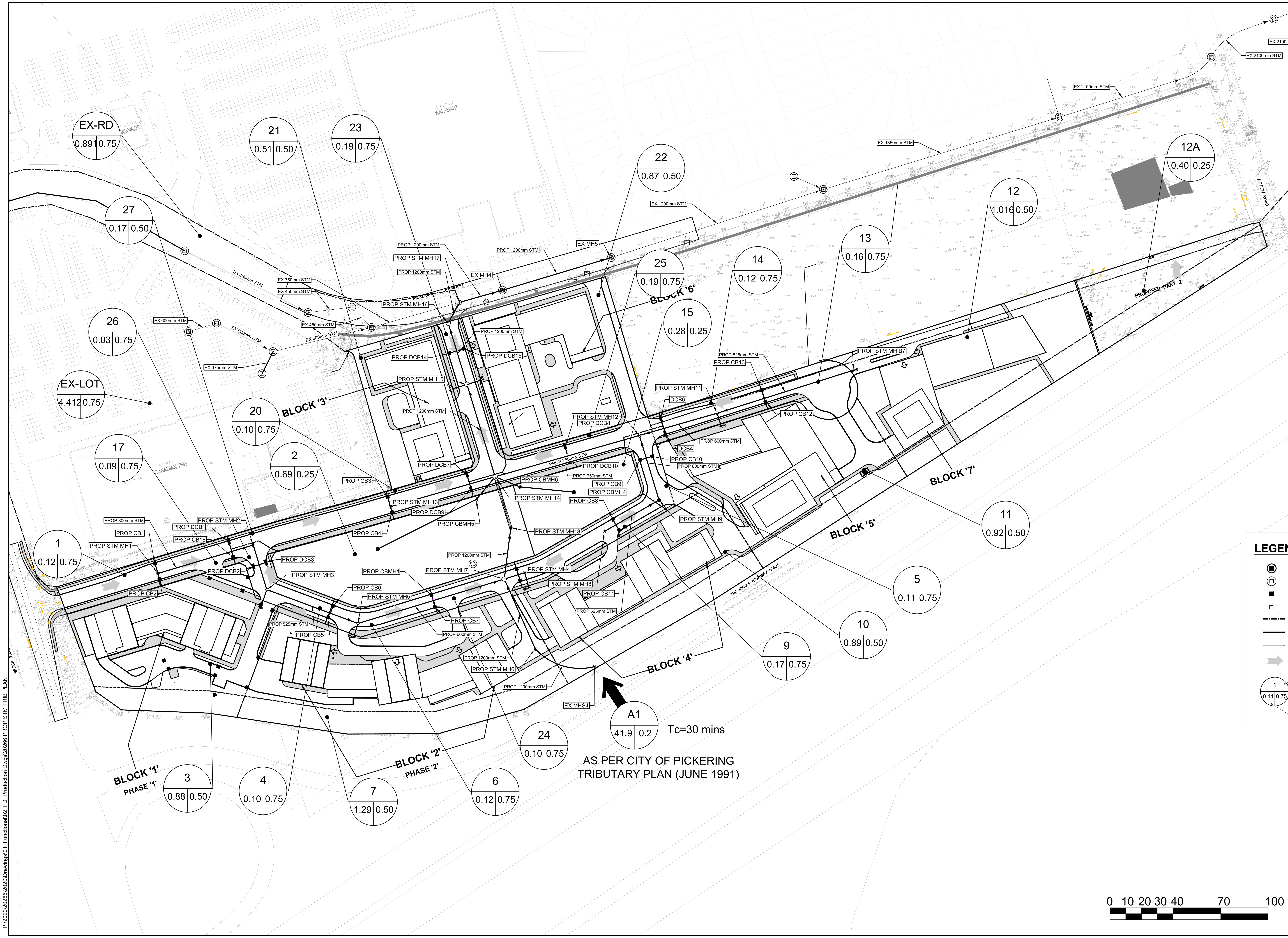

FIGURE S-7

PROPOSED STORM TRIBUTARY PLAN

PICKERING BRIDGE LANDS INC.
PROPOSED MIX-USE DEVELOPMENT
1755 + 1805 PICKERING PARKWAY
PICKERING, ON

DATE:	PROJ. NO.:	SCALE:
JAN 2025	20266	NTS

ODAN·DETECH CONSULTING ENGINEERS

SCALES
HOR 1:400
VER 1:80

FIGURE S-8
NOTION ROAD PROFILE (0+000 to 0+320)

PICKERING BRIDGE LANDS INC.
PROPOSED MIX-USE DEVELOPMENT
1755 + 1805 PICKERING PARKWAY
PICKERING, ON

DATE:	PROJ. NO.:	SCALE:
APR 2024	20266	AS NOTED

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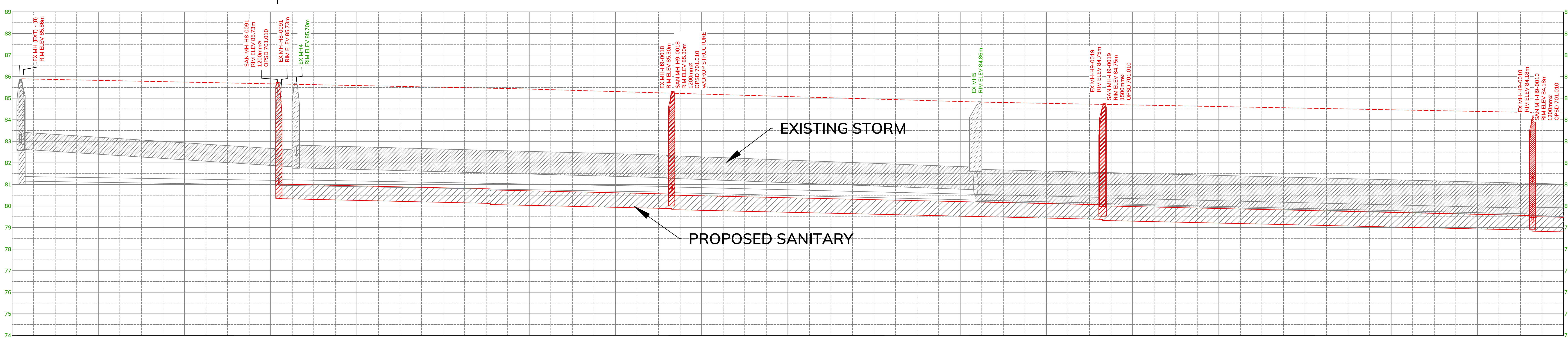
PIPES TO NOTION ROAD TO BE SIZED AND CONSTRUCTED FOR FULL BUILD OUT OF SUBJECT SITE, 1899 BROCK ROAD AND FUTURE DEVELOPMENTS (PER REGION)

SEE FIGURE S-10



EXISTING ELEVATION	0+000; 0+020; 0+040 85.85; 0+060 85.76; 0+080 85.65; 0+100 85.54; 0+120; 0+140; 0+160; 0+180; 0+200; 0+220; 0+240; 0+260; 0+280; 0+300; 0+320; 0+340; 0+360	EXISTING ELEVATION
PROPOSED SANITARY	E 80.34 W 80.98; PROP 49.09m 675mm CONC SAN @ 0.45% PVC-SDR 35 ASTM D-3034 CSA B182.2 BEDDING AS PER S-200.010; PROP 42.06m 675mm CONC SAN @ 0.45% PVC-SDR 35 ASTM D-3034 CSA B182.2 BEDDING AS PER S-200.010; PROP 99.99m 675mm CONC SAN @ 0.45% PVC-SDR 35 ASTM D-3034 CSA B182.2 BEDDING AS PER S-200.010; W 79.38 E 79.33; PROP 99.80m 675mm CONC SAN @ 0.45% PVC-SDR 35 ASTM D-3034 CSA B182.2 BEDDING AS PER S-200.010; W 78.88 E 78.83 N 79.22 S 81.13	PROPOSED SANITARY SEWER INVERT
EXISTING SANITARY	EX 59.82m 200mm SAN @ 0.30%; E 80.96 S 81.07 W 80.98; EX 91.15m 250mm SAN @ 0.35% (TO BE REMOVED); W 79.88 E 79.83 N 80.67; EX 99.99m 250mm SAN @ 0.49% (TO BE REMOVED); W 80.14 E 80.12; EX 99.80m 250mm SAN @ 0.48% (TO BE REMOVED); PROP 82.45m 675mm CONC SAN @ 0.45% PVC-SDR 35 ASTM D-3034 CSA B182.2 BEDDING AS PER S-200.010; W 79.64 E 79.62 N 79.92 S 81.13	EXSTING SAN SEWER INVERT
EXISTING STORM	E 82.73 SW 82.93 NW 82.83; W 81.90 NW 82.34; EX 86.05m 750mm STM @ 0.54%; EX 71.76m 750mm STM @ 0.75%; W 80.90 E 80.36 SW 80.44; EX 140.65m 1200mm STM @ 0.50%; EX 82.45m 250mm SAN @ 0.36% (TO BE REMOVED)	EXSTING STORM SEWER INVERT

SCALES
HOR 1:400
VER 1:80

FIGURE S-9
PICKERING PARKWAY PROFILE (0+000 to 0+360)

PICKERING BRIDGE LANDS INC.
PROPOSED MIX-USE DEVELOPMENT
1755 + 1805 PICKERING PARKWAY
PICKERING, ON

DATE:	PROJ. NO.:	SCALE:
APR 2024	20266	AS NOTED

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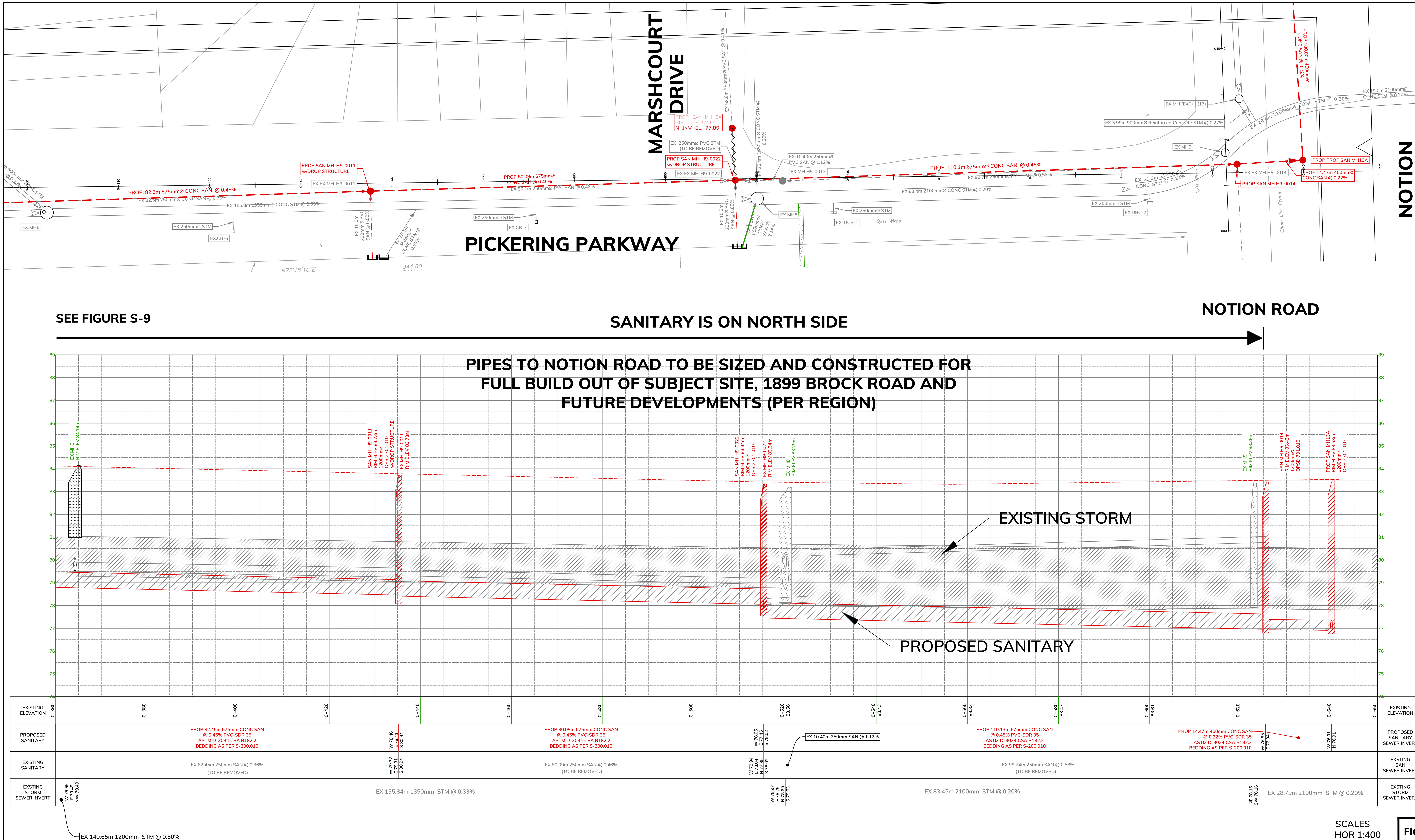

SCALES
HOR 1:400
VER 1:80

FIGURE S-10
PICKERING PARKWAY PROFILE
(0+360 to 0+650)

PICKERING BRIDGE LANDS INC.
PROPOSED MIX-USE DEVELOPMENT
1755 + 1805 PICKERING PARKWAY
PICKERING, ON

DATE:	PROJ. NO.:	SCALE:
APR 2024	20266	AS NOTED

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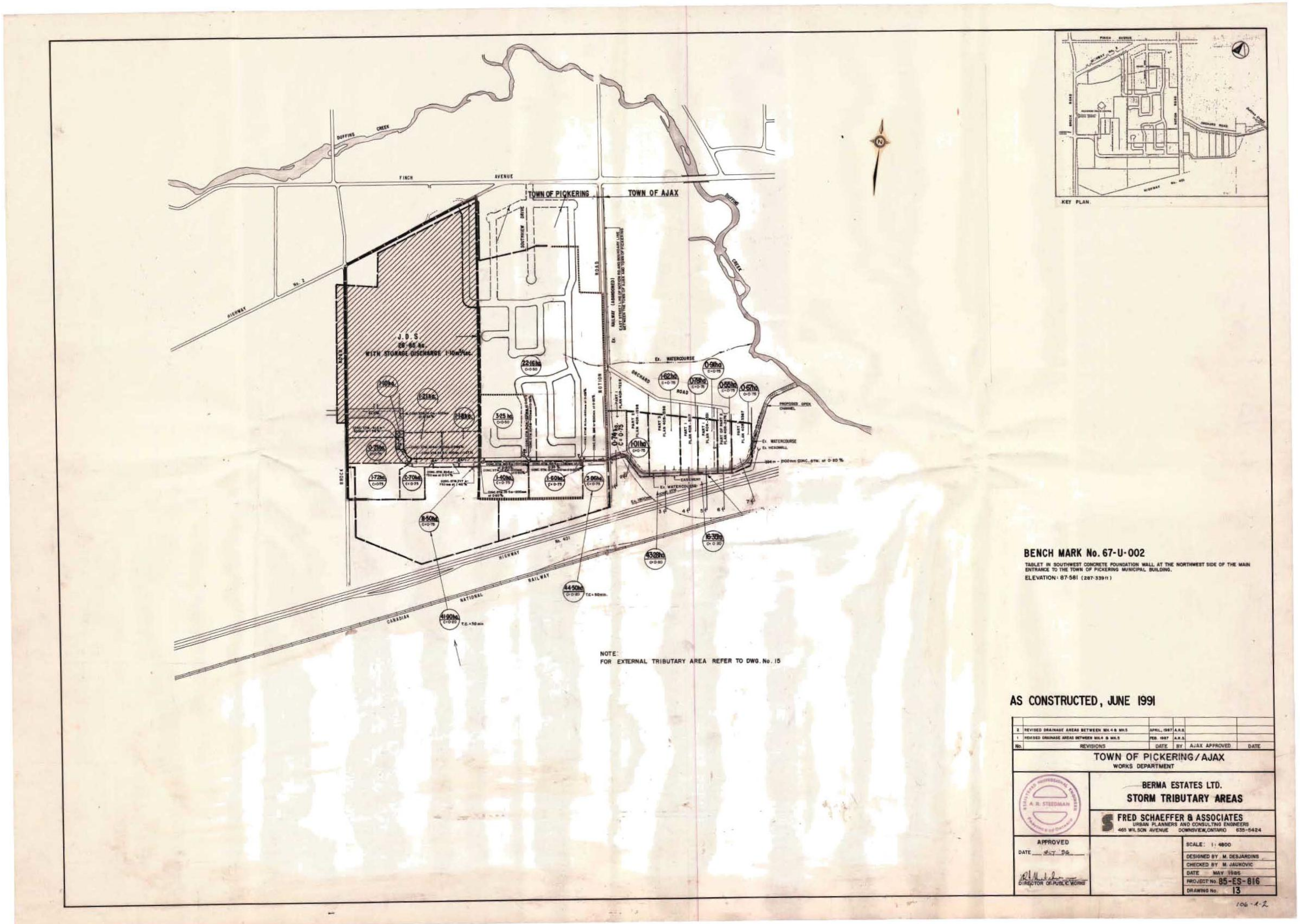
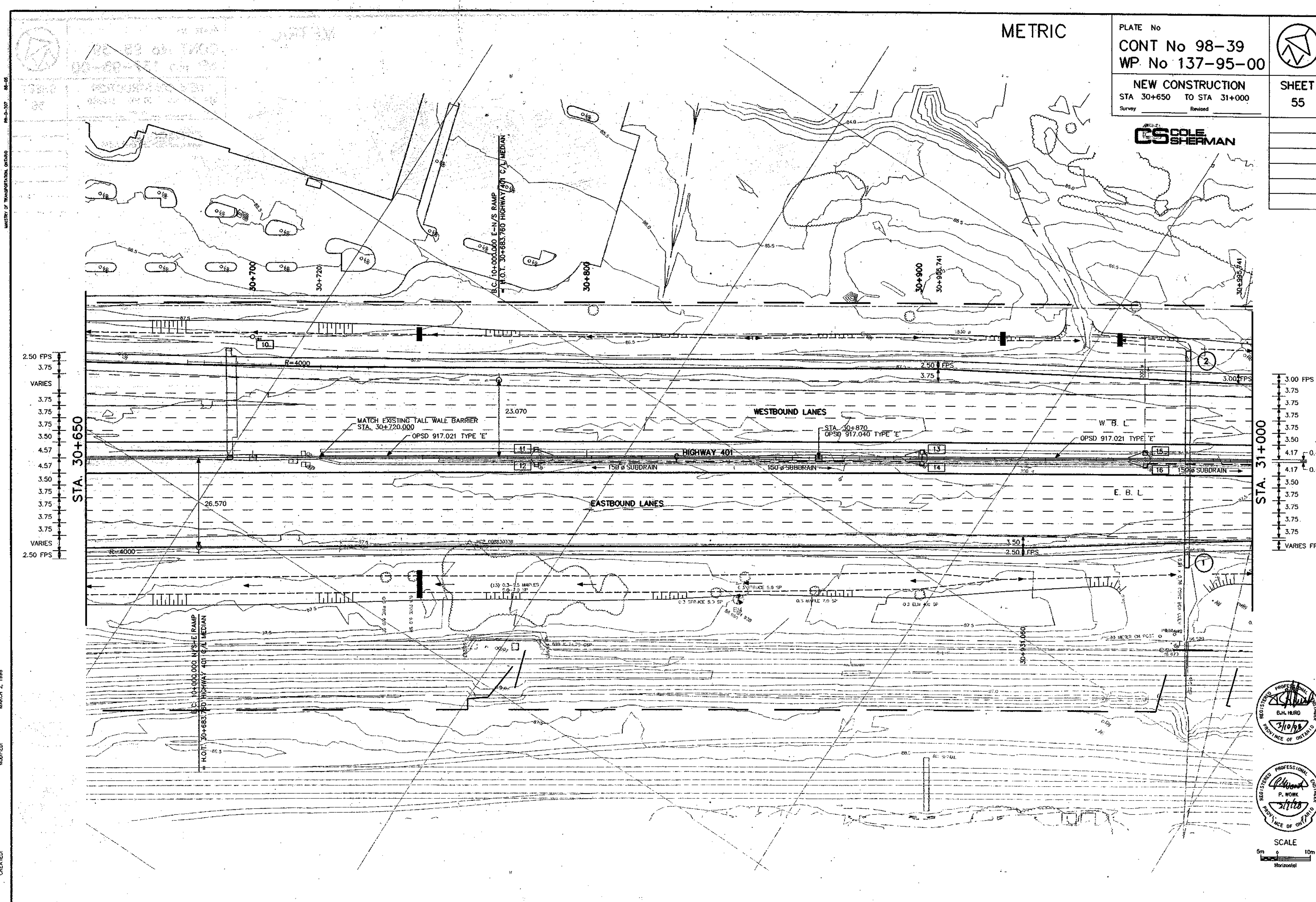
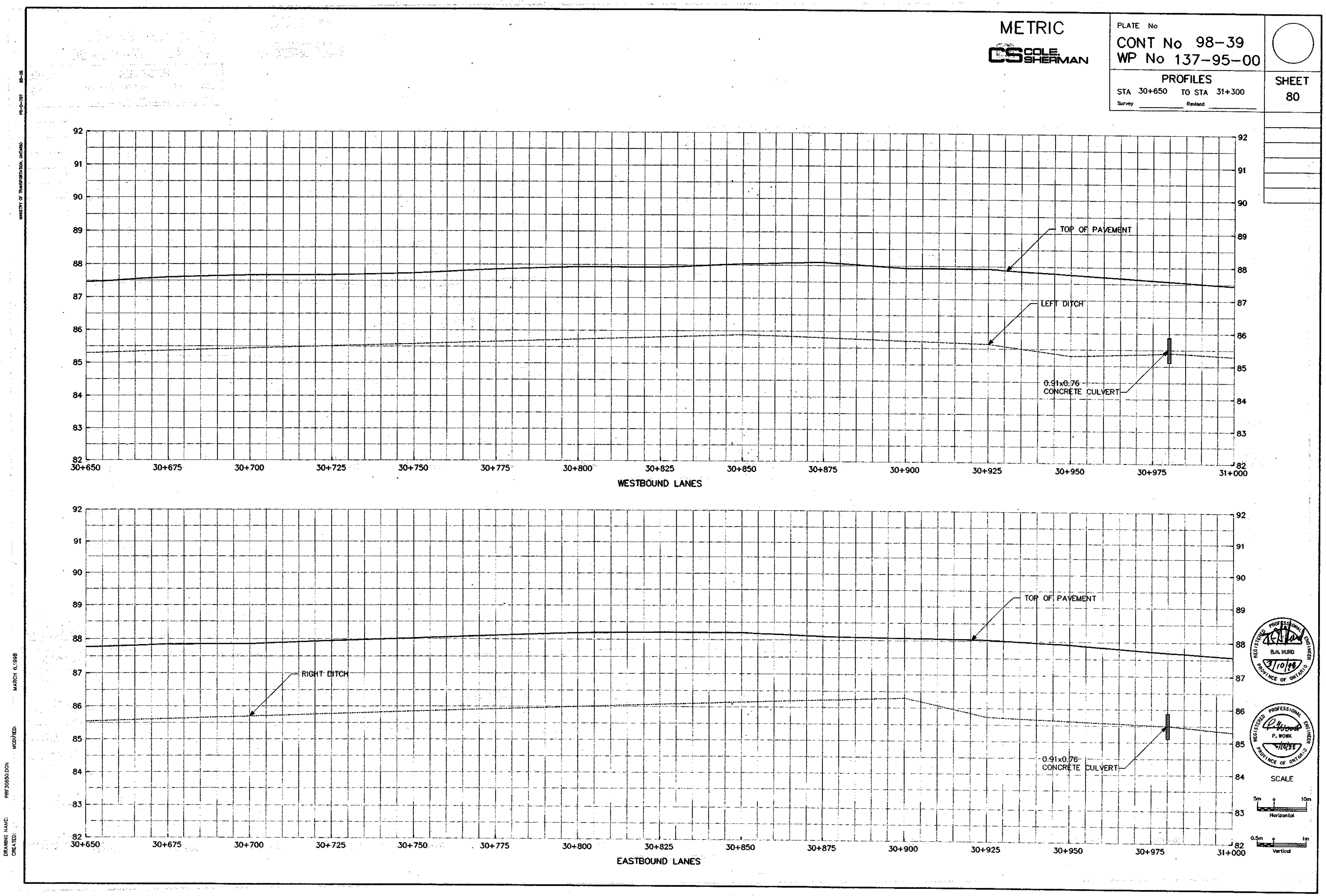

BENCH MARK No. 67-U-002
TABLET IN SOUTHWEST CONCRETE FOUNDATION WALL AT THE NORTHWEST SIDE OF THE MAIN ENTRANCE TO THE TOWN OF PICKERING MUNICIPAL BUILDING.
ELEVATION: 87·581 (287·339ft)
NOTE:
FOR EXTERNAL TRIBUTARY AREA REFER TO DWG. No. 15
AS CONSTRUCTED, JUNE 1991
TOWN OF PICKERING
TOWN OF AJAX
FINCH AVENUE
KEY PLAN
TOWN OF PICKERING/AJAX
WORKS DEPARTMENT
BERMA ESTATES LTD.
STORM TRIBUTARY AREAS
FRED SCHAEFFER & ASSOCIATES
URBAN PLANNERS AND CONSULTING ENGINEERS
APPROVED
DIRECTOR OF PUBLIC WORKS
SCALE: 1:4800
DESIGNED BY M. DESJARDINS
CHECKED BY M. JAUKOVIC
PROJECT No. 85-ES-816
DRAWING No. 13


METRIC
PLATE No
CONT No 98-39
WP No 137-95-00
NEW CONSTRUCTION
STA 30+650 TO STA 31+000
SHEET
55
COLE SHERMAN
WESTBOUND LANES
EASTBOUND LANES
HIGHWAY 401
W. B. L.
E. B. L.
STA. 30+650
STA. 31+000
MATCH EXISTING TALL WALL BARRIER STA. 30+720.000
OPSD 917.021 TYPE 'E'
STA. 30+870 OPSD 917.040 TYPE 'E'
150 ø SUBDRAIN
R=4000
SCALE


METRIC
COLE SHERMAN
PLATE No
CONT No 98-39
WP No 137-95-00
PROFILES
STA 30+650 TO STA 31+300
SHEET
80
TOP OF PAVEMENT
LEFT DITCH
0.91x0.76
CONCRETE CULVERT
WESTBOUND LANES
RIGHT DITCH
EASTBOUND LANES
SCALE
Horizontal
Vertical
B.H. HURD
P. WORK
PROVINCE OF ONTARIO
MARCH 6,1998