S. LLEWELLYN & ASSOCIATES LIMITED
CONSULTING ENGINEERS

Functional Servicing & Stormwater Management Report

1110 LORNE PARK ROAD

CITY OF MISSISSAUGA

JACON CONSTRUCTION LIMITED

MARCH 2019

SLA File: 18091

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1.0 INTRODUCTION AND BACKGROUND

1.1 OVERVIEW

S. Llewellyn & Associates Limited has been retained by Jacan Construction Ltd. to provide consulting engineering services for the proposed development at 1110 Lorne Park Road in the City of Mississauga (see Figure 1.0 for location plan). This report will outline the functional servicing and the stormwater management strategy for the proposed development.

The proposed development consists of constructing a 2-storey 7 unit townhouse building with a gross floor area of approximately 1468m^2. The proposed site will also include asphalt parking, concrete walkways and landscaped areas.

This report will provide detailed information of the proposed servicing scheme for this development. Please refer to the site engineering plans prepared by S. Llewellyn and Associates Limited and the site plan prepared by Eugene Kuan Architects for additional information.

1.2 BACKGROUND INFORMATION

The following documents were referenced in the preparation of this report:

Ref. 1: MOE Stormwater Management Practices Planning and Design Manual (Ministry of Environment, March 2003)

Ref. 2: Development Requirements Manual (City of Mississauga, September 2016)

Ref. 3: Green Development Standards (City Mississauga, December 2010)

Ref. 4: Public Works Design, Specifications & Procedures Manual – Sanitary Sewer Design Criteria (Region of Peel, March 2017)

Ref. 5: Public Works Design, Specifications & Procedures Manual – Watermain Design Criteria (Region of Peel, June 2010)

Figure 1.0 – Location Plan

2.0 STORMWATER MANAGEMENT

The following stormwater management (SWM) criteria will be applied to the site, in accordance with the City of Mississauga requirements:

Quantity Control
The stormwater discharge rate from the proposed site shall be controlled to the 2 year event pre-development conditions discharge rate for all storm events up to and including the 100-year event.

Quality Control
The stormwater runoff from the proposed condition site must meet Enhanced (Level 1) stormwater quality control (80% TSS removal, 90% average annual runoff treatment).

Erosion Control
Erosion and sediment control measures will be implemented in accordance with the standards of the City of Mississauga.

Water Balance
5mm of storm water run off from entire site must infiltrated on-site.

2.1 EXISTING CONDITIONS

In the existing condition, the site consists entirely of pervious areas. The site is bound by existing residential buildings to the west, Lorne Park Road to the north, Albertson Crescent to the east and by Bramblewood Lane to the south. In the existing condition, the entire site sheet drains toward Bramblewood Lane where it is captured by the existing ditch within the right-of-way.

One catchment area, Catchment 101, has been identified in the existing condition. Catchment 101 represents the drainage area for the entire site, which sheet drains to Bramblewood Lane. See Table 2.1 below and the Pre-Development Storm Drainage Area Plan in Appendix A for details.

Table 2.1 – Existing Conditions Catchment Areas

Catchment ID	Description	Area (ha)	Percent Impervious	Run-off Coefficient
101	Entire site	0.14	0%	0.25

An analysis was performed on Catchment 101 using the Rational Method for the 2-year to 100-year City of Mississauga design storms. A summary of the results can be found in the Table 2.2.

Table 2.2 – Existing Condition Site Discharge

Storm Event	Catchment 101 Discharge (m^3/s)
2-Yr Event	0.007
100-Yr Event	0.017

2.2 PROPOSED CONDITIONS

It is proposed to develop the site by constructing a 2-storey 7 unit townhouse building with associated asphalt driveway/parking lot, concrete curb/sidewalk and landscaped areas. It is proposed to service the site with a private storm sewer system designed and constructed in accordance with the standards and specifications of the City of Mississauga.

Two catchment areas, Catchment 201 and 202, have been identified in the proposed condition. Catchment 201 represents the drainage area for the small portion of the site that will remain uncontrolled, sheet draining to Albertson Crescent. Catchment 202 represents the drainage area for the majority of the site which will be captured and controlled by the proposed storm sewer system, ultimately discharging to the existing 250mmø storm sewer along Lorne Park Road. See Table 2.3 below and the Post-Development Drainage Area Plan in Appendix A for details.

Table 2.3 – Proposed Condition Catchment Areas				
Catchment ID	Description	Area (ha)	Percent Impervious	Run-off Coefficient
201	Uncontrolled to Albertson Lane	0.02	43%	0.53
202	Controlled to Lorne Park Road	0.12	53%	0.60

An analysis was performed on the Proposed Condition site using the Rational Method to determine the uncontrolled flows during the 2-year to 100-year City of Mississauga design storms. A summary of the results can be found in Table 2.4 and detailed Modified Rational Method can be found in Appendix B.

Table 2.4 – Proposed Condition with no Stormwater Management (Uncontrolled)			
Storm Event	Catchment 201 Discharge (m^3/s)	Catchment 202 Discharge (m^3/s)	Total
2-Yr	0.002	0.015	0.017
100-Yr	0.005	0.035	0.040

Water Quantity Control

With the installation of on-site quantity control measures for Catchment 202, it will be required to provide stormwater storage during storm events up to and including the 100-year event. To provide the required storage, it is proposed to install StormTech SC-740 tanks under the proposed asphalt parking lot which will provide 46m^3 of active storage.

The proposed 75mm orifice will be installed downstream of the proposed CBMH2 to control the flow to the pre-development level and provide storage within the proposed underground tanks and pipes.

The proposed pipe connecting CBMH2 to the StromTech tanks will be reversed (i.e. sloped towards the tanks) to allow for all the run-off to entre the tanks first. Once the tanks became full then will discharge the stormwater through the proposed 75mm orifice plate installed downstream of CBMH2.

Details of the proposed storage tanks can be found on the Site Servicing Plan. The stage-storage-discharge characteristics can be seen in Table 2.5 below and Appendix A for details.

Table 2.5 – Proposed Condition Stage-Storage-Discharge (Catchment 202)			
Storm Event	Total Storage (m^3)	Active Storage (m^3)	Discharge (m^3/s)
92.62 (Orifice Invert)	0	0	0.0000
92.62 (Bottom of Tank)	0	0	0.0000
92.62 (Orifice Invert)	0	0	0.0000
93.68 (Top of tank)	44	4	0.0121
93.95 (Max Ponding)	45	5	0.0135

An analysis was performed on the Proposed Condition site using the Modified Rational Method to determine the required volume of stormwater storage required during the 2-year and 100-year City of Mississauga design storms. A summary of the results can be found in Table 2.6 and Modified Rational Method found in Appendix B.

Table 2.6 – Proposed Condition Stormwater Discharge (Controlled)			
Storm Event	Catchment 202 Controlled Discharge (m^3/s)	Allowable Discharge (m^3/s)	Required Storage (m^3)
2-Yr	0.007	0.007	8
100-Yr	0.014 *(0.0062)	0.007	32

*Used minimum orifice size 50mm to meet MECP requirements

Table 2.7 – Proposed Condition Stormwater Discharge entire site				
Storm Event	Catchment 201 Uncontrolled Discharge (m^3/s)	Catchment 202 Controlled Discharge (m^3/s)	Total (m^3/s)	Allowable Discharge (m^3/s)
2-Yr	0.002	0.007	0.009	0.007
100-Yr	0.005	0.014	0.019	0.007

This analysis determined the following:

- The proposed condition discharge rates will not exceed the existing condition discharge rates during the 2-year storm events with the installation of a 75mm orifice plate and SC-740 underground storage tank. However, with 50mm the proposed condition discharge rate will not exceed the allowable rate but 75mm orifice is used to meet the minimum size required by MECP

- 32m^3 of stormwater storage is required for Catchment 202 during the 100-year event, which can be accommodated by the proposed storage tanks, having a volume of 40m^3.

Water Balance

As per the City of Mississauga requirements 5 mm of the storm water runoff from the entire site should infiltrate into the ground. The total volume required for the water balance for the site is 7m^3 (5mm x 1413m^2). This volume added to the total storage required for quantity control for the site within the StormTech SC-740 tanks with a total of 39m^3 (32 + 7). The storage tank SC-740 can provide the infiltration required for the site. See details in the engineering plans.

Water Quality Control

The proposed development is required to achieve an "Enhanced" (80% TSS removal) level of water quality protection. To achieve these criteria, discharge from Catchment 202 will be subject to treatment from a Hydroguard oil/grit separator before ultimately discharging to the existing 250mmø storm sewer along Lorne Park Road. The Hydroguard sizing software was used to determine the required size of oil/grit separator unit for the site. It was determined that a Hydroguard HG4 will provide 98% TSS removal and 100% average annual runoff treatment, which satisfies the requirements for an "Enhanced" level for quality control. See Hydroguard unit sizing procedures in Appendix C for details.

Hydroguard units require regular inspection and maintenance as per the manufacture's specifications to ensure the unit operates properly. See Hydroguard Maintenance Manual in Appendix C for details.

As part of a "treatment train" approach, the subject lands will also provide quality control practices by the use of an Isolator Row.

Isolator Row has been proposed to be installed within a chamber of the Stormtech SC-740 system. Stormwater entering the system will capture debris and sediment within the Isolator Row chamber. Isolator Rows require regular inspection and maintenance as per the manufacture's specifications to ensure the Stormtech chamber operates properly. The technical information regarding the Isolator Row have been included in Appendix C of this report.

Low Impact Development

The proposed storage tank will provide underground infiltration practices. The tank has been designed with the bottom of the tank sitting 92.62m below the outlet invert, allowing for approximately 7m^3 of infiltration storage, which is equal to 5mm of runoff over the entire site. Rainfall runoff will pass through the isolator rows prior to infiltrating. The chambers inside of the tank will have open bottoms underlying a granular stone reservoir, where stormwater will saturate into an ultimately infiltrate into the native soil.

Storage/Infiltration Volume=7m^3
Site Areas=1413m^2
Rainfall Depth=5mm
Draw Down Time (hrs) = 5

According to the Geotech report completed by Soil Probe on Feb. 9, 2011 for the subject site, is shown that the test composite sample is fine sand with some silt, also the native soil below fill is sand texture. The recommended layer infiltration rate is 20mm/hr.

2.3 SEDIMENT AND EROSION CONTROL

In order to minimize erosion during the grading and site servicing period of construction, the following measures will be implemented:

- Install silt fencing along the outer boundary of the site to ensure that sediment does not migrate to the adjacent properties;
- Install sediment control (silt sacks) in the proposed catchbasins as well as the nearby existing catchbasins to ensure that no untreated runoff enters the existing conveyance system
- Stabilize all disturbed or landscaped areas with hydro seeding/sodding to minimize the opportunity for erosion.

To ensure and document the effectiveness of the erosion and sediment control structures, an appropriate inspection and maintenance program is necessary.
The program will include the following activities:

- Inspection of the erosion and sediment controls (e.g. silt fences, sediment traps, outlets, vegetation, etc.) with follow up reports to the governing municipality; and
- The developer and/or his contractor shall be responsible for any costs incurred during the remediation of problem areas.

Details of the proposed erosion & sediment control measures can be seen on the Grading & Erosion Control Plan.

3.0 SANITARY SEWER SERVICING

3.1 EXISTING CONDITIONS

The site is located on Lorne Park Road with an existing 250mmø sanitary sewer located along Lorne Park Road and an existing 250mmø sanitary sewer located along Bramblewood Lane.

3.2 SANITARY DEMAND

The proposed development consists of a 2-storey 7 unit townhouse building with a gross floor area of approximately 1468m^2. Wastewater generation for the site was calculated based on the Region of Peel Sanitary Design Criteria. Table 3.1 summarizes the sanitary sewer discharge rates from the proposed site.

Table 3.1- Proposed Sanitary Sewer Discharge

Site Area (ha)	Population [A]	Avg. Demand [B] (l/s)	Infiltration [C] (l/s)	**Peak Flow [D] (l/s)**
0.14	25	0.087	0.028	**0.115**

[A] 175 persons/hectare x 0.14 hectares
[B] Region of Peel STD. DWG. 2-9-2, design flow based upon sewage flow of 302.8L/cap/day
[D] Infiltration flow based on 0.20 l/ha/sec infiltration rate
[E] Peak Flow = Average Flow + Infiltration

3.3 PROPOSED SANITARY SERVICING AND CAPACITY ANALYSIS

The proposed townhouse building will be serviced by a 200mmø sanitary sewer, designed and constructed in accordance with the City of Mississauga standards. Drainage from this sewer will discharge to the existing 250mmø sanitary sewer along Bramblewood Lane.

The minimum grade of the proposed 200mmø sanitary sewer will be 2.0%. At this minimum grade, the proposed sanitary sewer will have a capacity of 0.0408 m^3/s (40.8 l/s). Therefore, the proposed 200mmø sanitary sewer at 2.0% grade is adequately sized to service the proposed development.

4.0 DOMESTIC AND FIRE WATER SUPPLY SERVICING

4.1 EXISTING CONDITIONS

The existing municipal water distribution system consists of a 300mmø watermain located along Lorne Park Road, a 200mmø watermain located along Albertson Crescent, and a 200mmø watermain located along Bramblewood Lane. The nearest existing fire hydrant is located in front of the subject lands on Lorne Park Road.

4.2 DOMESTIC WATER DEMAND

The following is an estimate of the water usage for the proposed townhouse building. Water usage for the site was calculated in accordance with the Region of Peel Watermain Design Criteria. Table 4.1 summarizes the domestic water demand requirements for the Average Daily, Maximum Daily and Peaking Hourly demand scenarios for the subject land.

Table 4.1 – Proposed Domestic Water Demand

Site Area (ha)	Population ^{A}	Average Daily Demand ^{B} (l/s)	Max. Daily Peaking Factor ^{C}	Max. Hourly Peaking Factor ^{D}	**Max. Daily Demand ^{E} (l/s)**	**Max. Hourly Demand ^{F} (l/s)**
0.14	25	0.081	0.162	0.243	**0.013**	**0.0197**

^{A} 175 persons/hectare x 0.14 hectares
^{B} Average Daily Demand = 280 l/person/day x population
^{C} Max. Daily Peaking Factor = 2.0
^{D} Max. Hourly Peaking Factor = 3.0
^{E} Max. Daily Demand = Average Daily Demand x Max. Daily Peaking Factor
^{F} Max. Hourly Demand = Average Daily Demand x Max. Hourly Peaking Factor

4.3 FIRE FLOW DEMAND

Fire flow demands for development are governed by a number of guidelines and criteria, such as the Water Supply for Public Fire Protection (Fire Underwriters Survey, 1999), Ontario Building Code (OBC), and various codes and standards published by the National Fire Protection Association (NFPA).

The proposed 2-storey 7 unit townhouse building will be constructed of non-combustible construction (C=1.0), with limited combustible occupancy (-15% correction). The exposure corrections for the building are based on the following:

North face: 10% correction (20.1 to 30m)
South face: 5% correction (30.1 to 45m)
East face: 10% correction (20.1 to 30m)
West face: 20% correction (3.1 to 10m)
Total: 45%

The resulting required flow rate as determined in accordance with the Fire Underwriters Survey – 1999 Water Supply for Public Fire Protection, as specified by the City of Mississauga is **8,000 l/min (133 l/s**). Refer to the Fire Flow Demand Requirements in Appendix D for calculations and details.

Hydrant flow tests for the public fire hydrant in close proximity to the proposed development has been analyzed to determine if the municipal system adjacent to the subject site is adequate to provide the required fire flow, with a minimum pressure of 20 psi. Table 4.2 summarizes the hydrant flow test data completed by Jackson Waterworks. See Appendix D for fire hydrant flow test results.

Table 4.2 - Hydrant Flow Test Data	
Location	LORNE PARK ROAD
Static Pressure	74 psi
Residual Pressure During Test Flow	59 psi
Test Flow Rate	1,580 IGPM (120 l/s)
Theoretical Flow @ 20 psi	**2,918 IGPM (221 l/s)**

Based on the above hydrant flow test data, the theoretical maximum available flow rate is **221 l/s**, while the maximum required fire flow for the proposed development is only **133 l/s**. Therefore, the water distribution system has adequate pressure and capacity to service the subject site.

4.4 PROPOSED WATER SERVICING AND ANALYSIS

Proposed water servicing for the site consists of connecting a 150mmø water service from the existing 200mmø adjacent to the site on Bramblewood Lane. The proposed 150mmø water service will provide domestic water service for the proposed townhouse building. Water services for the site are to be designed and constructed in accordance with the Region of Peel standards.

5.0 CONCLUSIONS AND RECOMMENDATIONS

Based on the information provided herein, it is concluded that the proposed development on Lorne Park Road can be constructed to meet the requirements of the City of Mississauga and the Region of Peel. Therefore, it is recommended that:

- The development be graded and serviced in accordance with the Grading & Erosion Control Plan and the Site Servicing Plan prepared by S.Llewellyn & Associates Limited;
- StormTech SC-740 tanks be installed as per the Site Servicing Plan and this report to provide additional storage for quality control and water balance.
- A Hydroguard oil/grit separator be installed as per the Site Servicing Plan and this report to provide effective stormwater quality control;
- Erosion and sediment controls be installed as described in this report to meet the City of Mississauga requirements;
- The proposed sanitary and water servicing system be installed as per this report to adequately service the proposed development;

We trust the information enclosed herein is satisfactory. Should you have any questions please do not hesitate to contact our office.

Prepared by:
S. LLEWELLYN & ASSOCIATES LIMITED

LICENSED PROFESSIONAL ENGINEER
A. GANEM MOHAMED
100110800
March 20/19
PROVINCE OF ONTARIO

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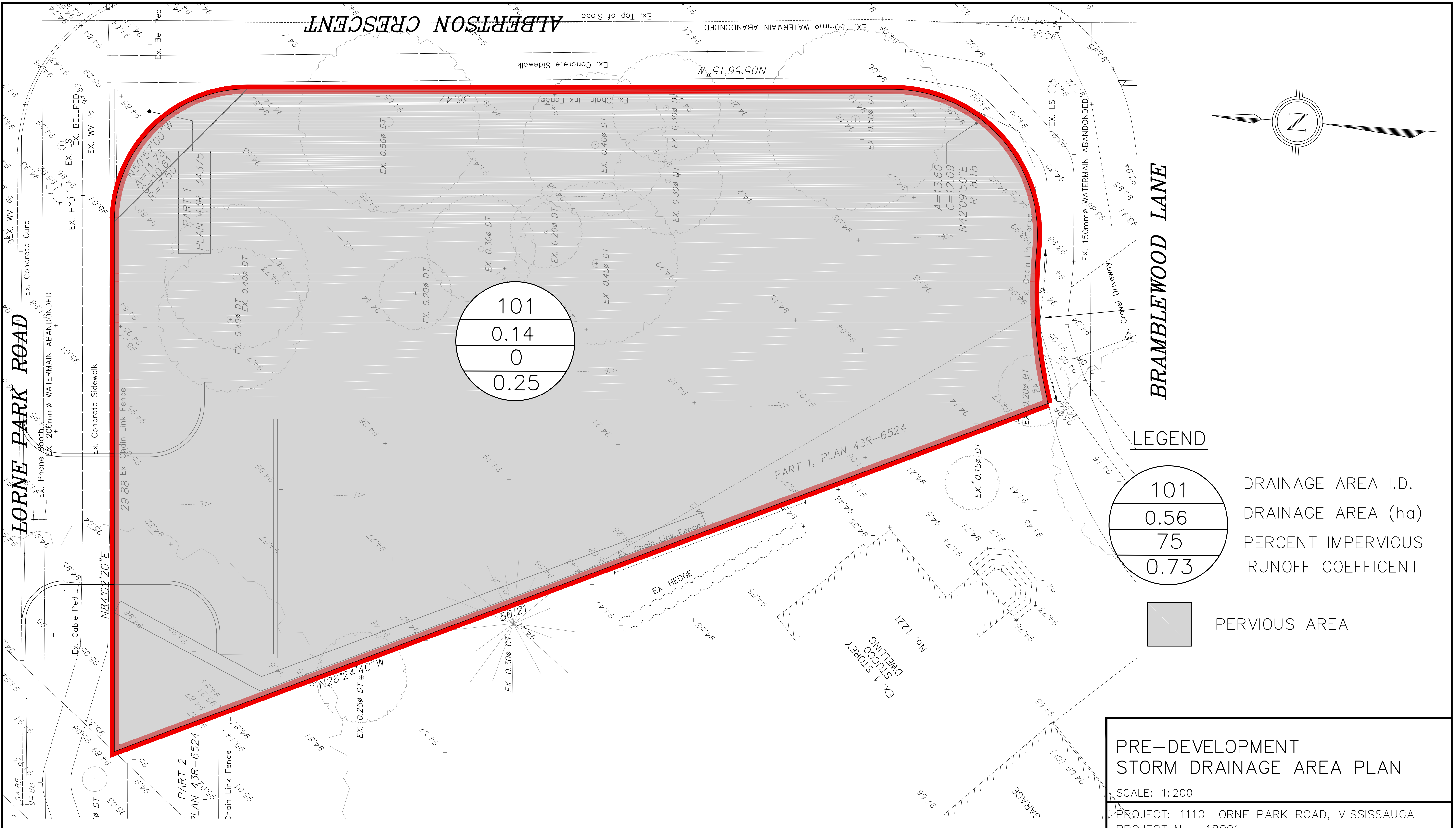
APPENDIX A

STORMWATER MANAGEMENT INFORMATION



PRE–DEVELOPMENT
STORM DRAINAGE AREA PLAN

SCALE: 1:200

PROJECT: 1110 LORNE PARK ROAD, MISSISSAUGA
PROJECT No.: 18091

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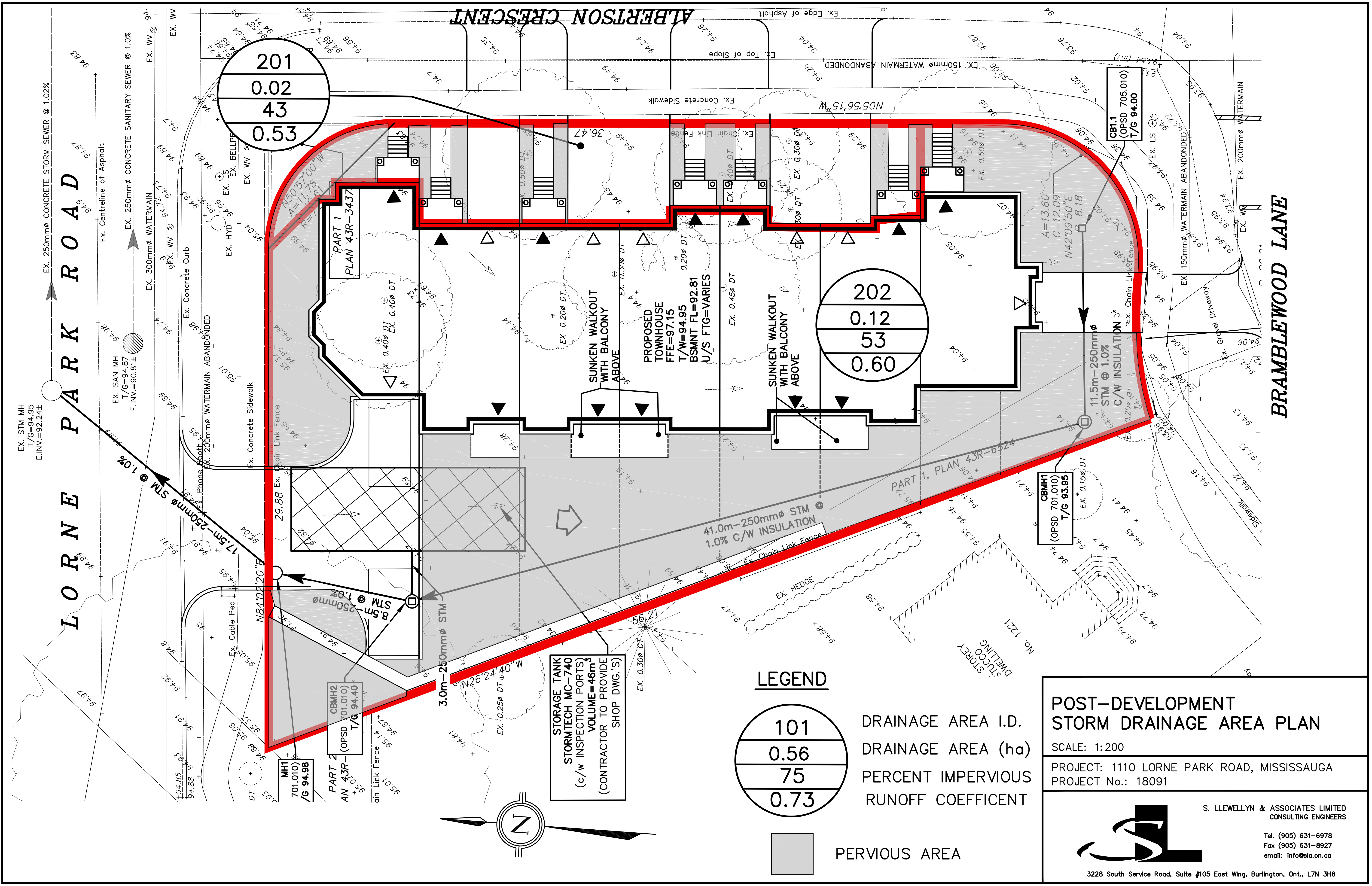

LEGEND

101	DRAINAGE AREA I.D.
0.56	DRAINAGE AREA (ha)
75	PERCENT IMPERVIOUS
0.73	RUNOFF COEFFICENT
▨	PERVIOUS AREA

POST–DEVELOPMENT STORM DRAINAGE AREA PLAN

SCALE: 1:200

PROJECT: 1110 LORNE PARK ROAD, MISSISSAUGA
PROJECT No.: 18091

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1110 Lorne Park Road,
Mississauga Ontario

S. LLEWELLYN & ASSOCIATES LIMITED
CONSULTING ENGINEERS

STAGE-STORAGE-DISCHARGE CALCULATIONS

Outlet Device No. 1 (Quantity)

Type:	Orifice Plate
Diameter (mm)	**75**
Area (m^2)	0.00442
Invert Elev. (m)	92.62
Disch. Coeff. (C_d)	0.6
Discharge (Q) =	$C_d A (2 g H)^{0.5}$

				Outlet No. 1	
	Elevation m	Tank Volume m^3	Structures/Pipe/ Surface Volume m^3	H m	Discharge m^3/s
Orifice Invert	92.62	0	0	0.000	0.0000
Bottom of Tank	92.62	0	0	0.000	0.0000
Orifice Invert	92.62	0	0	0.000	0.0000
Top of Tank	93.68	40	4	1.060	0.0121
Max Ponding	93.95	40	5	1.330	0.0135

Project: 18091

APPENDIX B

MODIFIED RATIONAL METHOD INFORMATION

2-Year Storm - Modified Rational Method

Stormwater Storage Volume and Orifice Sizing

Determination of required storage volume under proposed conditions. Storage volume calculated using the Modified Rational Method.
Catchment 201

Mississauga Storm Rainfall Information		
City/Town/Region:	**Mississauga**	
Return Period:	**2**	Years
A =	**610.000**	
B =	**4.600**	
C=	**0.780**	
Tc =	**10**	minutes
	600	seconds

Area of site being investigated (ha) =	**0.14**	(Lot Area)
Composite Runoff Coeff. (C) =	**0.6**	(Post-development "C")
Release Rate = Q_{ALLOW} (m^3/s) =	**0.007**	(Allowable discharge)

Flows from Lot area calculated from area indicated above

Roof flows (Q_{ROOF}) added in as a constant flow rate into the orifice controlled system (if applicable)

Duration (T_D)		Rainfall Intensity		Post-Development Runoff			Runoff Volume	Release Volume	Storage Volume
				Site	Roof	Total "Q_{POST}"			
(min)	(sec)	(mm/hr)	(m/s)	(m^3/s)	(m^3/s)	(m^3)	(m^3)	(m^3)	(m^3)
5	300	104.510	0.0000290	0.025	0.0	0.0246	7.37	3.15	4.22
10	600	75.359	0.0000209	0.018	0.0	0.0177	10.63	4.20	6.43
15	900	59.892	0.0000166	0.014	0.0	0.0141	12.67	5.25	7.42
20	1200	50.165	0.0000139	0.012	0.0	0.0118	14.15	6.30	7.85
25	1500	43.423	0.0000121	0.010	0.0	0.0102	15.31	7.35	7.96
30	1800	38.446	0.0000107	0.009	0.0	0.0090	16.26	8.40	7.86
35	2100	34.604	0.0000096	0.008	0.0	0.0081	17.08	9.45	7.63
40	2400	31.539	0.0000088	0.007	0.0	0.0074	17.79	10.50	7.29
45	2700	29.030	0.0000081	0.007	0.0	0.0068	18.42	11.55	6.87
50	3000	26.935	0.0000075	0.006	0.0	0.0063	18.99	12.60	6.39
55	3300	25.156	0.0000070	0.006	0.0	0.0059	19.51	13.65	5.86
60	3600	23.624	0.0000066	0.006	0.0	0.0056	19.99	14.70	5.29

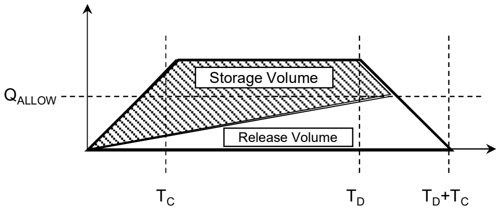
Max. required storage volume = 7.96 m^3

Q_{POST} = (C i A) x 10000 m^2/ha (Rational Method)

Runoff Volume = Area under trapezoidal hydrograph
= $(T_D - T_C)Q_{POST} + (T_C\,Q_{POST})$

Release Volume = Area under triangular outflow hydrograph
= $\frac{1}{2}(T_D + T_C)\,Q_{ALLOW}$

Storage Volume = Runoff Volume - Release Volume



100-Year Storm - Modified Rational Method

Stormwater Storage Volume and Orifice Sizing

Determination of required storage volume under proposed conditions. Storage volume calculated using the Modified Rational Method.
Catchment 201

MississaugaStorm Rainfall Information

City/Town/Region: **Mississauga**
Return Period: **100** Years
A = **1450.000**
B = **4.900**
C= **0.780**
Tc = **10** minutes
600 seconds

Area of site being investigated (ha) = **0.14** (Lot Area)
Composite Runoff Coeff. (C) = **0.6** (Post development "C")
Release Rate = Q_{ALLOW} (m^3/s) = **0.007** (Allowable discharge)

Flows from Lot area calculated from area indicated above

Roof flows (Q_{ROOF}) added in as a constant flow rate into the orifice controlled system (if applicable)

				Post-Development Runoff			Runoff	Release	Storage
Duration (T_D)		Rainfall Intensity		Site	Roof	Total "Q_{POST}"	Volume	Volume	Volume
(min)	(sec)	(mm/hr)	(m/s)	(m^3/s)	(m^3/s)	(m^3)	(m^3)	(m^3)	(m^3)
5	300	242.534	0.0000674	0.057	0.0	0.0566	16.98	3.15	13.83
10	600	176.312	0.0000490	0.041	0.0	0.0411	24.68	4.20	20.48
15	900	140.690	0.0000391	0.033	0.0	0.0328	29.54	5.25	24.29
20	1200	118.122	0.0000328	0.028	0.0	0.0276	33.07	6.30	26.77
25	1500	102.410	0.0000284	0.024	0.0	0.0239	35.84	7.35	28.49
30	1800	90.775	0.0000252	0.021	0.0	0.0212	38.13	8.40	29.73
35	2100	81.773	0.0000227	0.019	0.0	0.0191	40.07	9.45	30.62
40	2400	74.579	0.0000207	0.017	0.0	0.0174	41.76	10.50	31.26
45	2700	68.683	0.0000191	0.016	0.0	0.0160	43.27	11.55	31.72
50	3000	63.753	0.0000177	0.015	0.0	0.0149	44.63	12.60	32.03
55	3300	59.563	0.0000165	0.014	0.0	0.0139	45.86	13.65	32.21
60	3600	55.952	0.0000155	0.013	0.0	0.0131	47.00	14.70	32.30

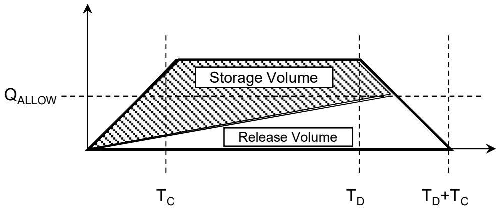
Max. required storage volume = 32.30 m^3

Q_{POST} = (C i A) x 10000 m^2/ha (Rational Method)

Runoff Volume = Area under trapezoidal hydrograph
= $(T_D - T_C)Q_{POST} + (T_C\,Q_{POST})$

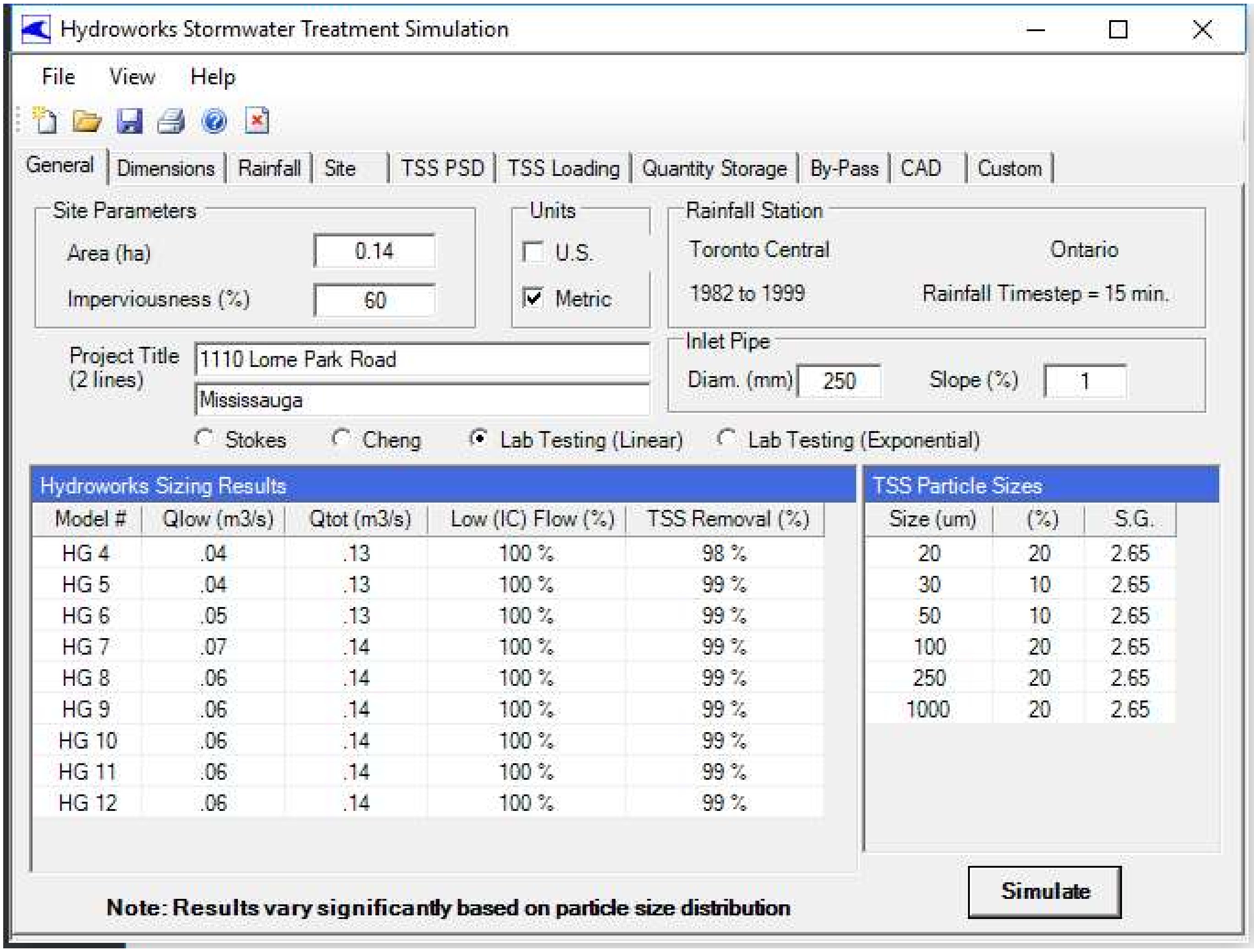
Release Volume = Area under triangular outflow hydrograph
= $½\,(T_D + T_C)\,Q_{ALLOW}$

Storage Volume = Runoff Volume - Release Volume



APPENDIX C

OIL/GRIT SEPARATOR INFORMATION



Site Parameters

Area (ha): 0.14

Imperviousness (%): 60

Units

☐ U.S.

☑ Metric

Rainfall Station

Toronto Central Ontario

1982 to 1999 Rainfall Timestep = 15 min.

Project Title (2 lines): 1110 Lorne Park Road
Mississauga

Inlet Pipe

Diam. (mm): 250 Slope (%): 1

○ Stokes ○ Cheng ● Lab Testing (Linear) ○ Lab Testing (Exponential)

Hydroworks Sizing Results

Model #	Qlow (m3/s)	Qtot (m3/s)	Low (IC) Flow (%)	TSS Removal (%)
HG 4	.04	.13	100 %	98 %
HG 5	.04	.13	100 %	99 %
HG 6	.05	.13	100 %	99 %
HG 7	.07	.14	100 %	99 %
HG 8	.06	.14	100 %	99 %
HG 9	.06	.14	100 %	99 %
HG 10	.06	.14	100 %	99 %
HG 11	.06	.14	100 %	99 %
HG 12	.06	.14	100 %	99 %

TSS Particle Sizes

Size (um)	(%)	S.G.
20	20	2.65
30	10	2.65
50	10	2.65
100	20	2.65
250	20	2.65
1000	20	2.65

Note: Results vary significantly based on particle size distribution

Hydroworks® Hydroguard

Maintenance Manual

Version 1.3

Introduction

The Hydroguard is a state of the art hydrodynamic separator. Hydrodynamic separators remove solids, debris and lighter than water (oil, trash, floating debris) pollutants from stormwater. Hydrodynamic separators and other water quality measures are mandated by regulatory agencies (Town/City, State, Federal Government) to protect storm water quality from pollution generated by urban development (traffic, people) as part of new development permitting requirements.

As storm water treatment structures fill up with pollutants they become less and less effective in removing new pollution. Therefore it is important that storm water treatment structures be maintained on a regular basis to ensure that they are operating at optimum performance. The Hydroguard is no different in this regard and this manual has been assembled to provide the owner/operator with the necessary information to inspect and coordinate maintenance of their Hydroguard.

Hydroworks® HG Operation

The Hydroworks HG separator is unique since it treats both high and low flows in one device, but maintains separate flow paths for low and high flows. Accordingly, high flows do not scour out the fines that are settled in the low flow path since they are treated in a separate area of the device as shown in Figure 1.

The HG separator consists of three chambers:

1. an inner chamber that treats low or normal flows
2. a middle chamber that treats high flows
3. an outlet chamber where water is discharged to the downstream storm system

Under normal or low flows, water enters the middle chamber and is conveyed into the inner chamber by momentum. Since the inner chamber is offset to one side of the structure the water strikes the wall of the inner chamber at a tangent creating a vortex within the inner chamber. The vortex motion forces solids and floatables to the middle of the inner chamber. The water spirals down the inner chamber to the outlet of the inner chamber which is located below the inlet of the inner chamber and adjacent to the wall of the structure but above the floor of the structure. Floatables are trapped since the outlet of the inner chamber is submerged. The design maximizes the retention of settled solids since solids are forced to the center of the inner chamber by the vortex motion of water while the outlet of the inner chamber draws water from the wall of the inner chamber.

The water leaving the inner chamber continues into the middle chamber, again at a tangent to the wall of the structure. The water is then conveyed through an outlet baffle wall (high and low baffle). This enhances the collection of any floatables or solids not removed by the inner chamber. Water flowing through the baffles then enters the outlet chamber and is discharged into the downstream storm drain.

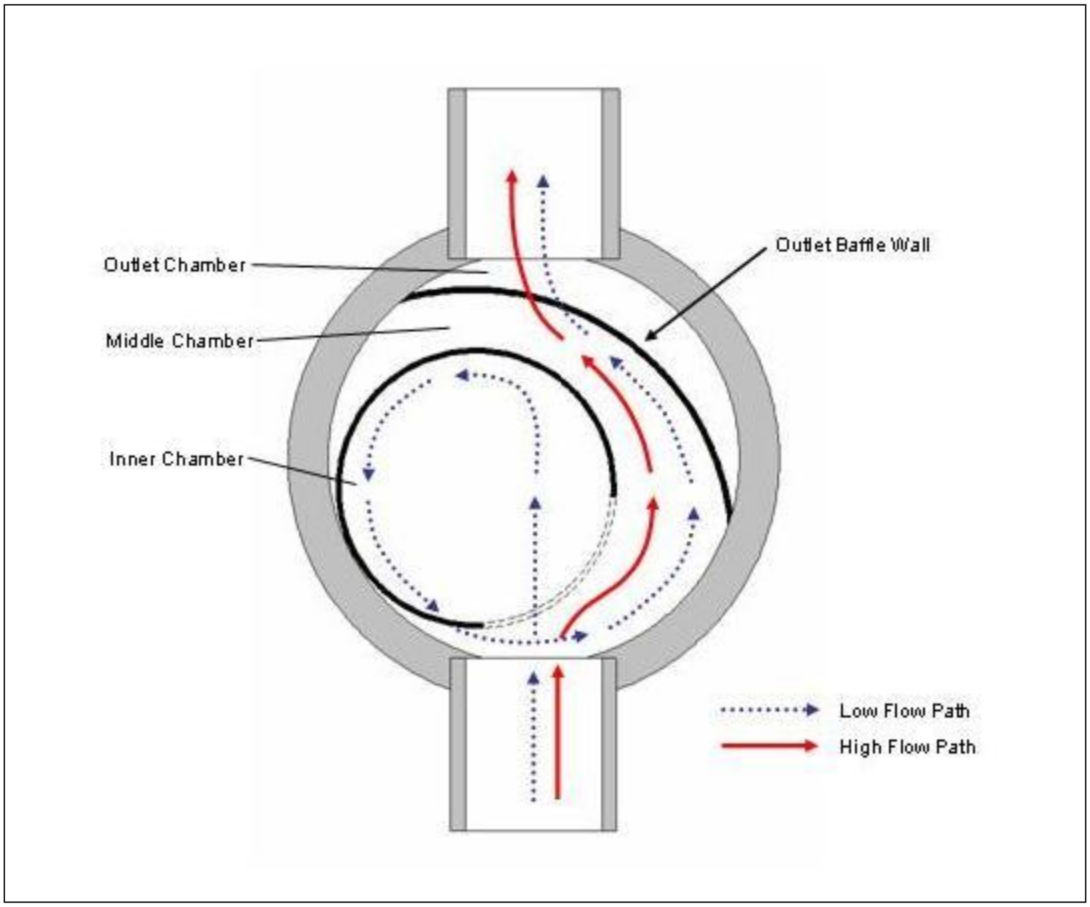


Figure 1. Hydroworks HG Operation – Plan View

During high flows, the flow rate entering the inner chamber is restricted by the size of the inlet opening to the inner chamber. This restriction of flow rate into the inner chamber prevents scour and re-suspension of solids from the inner chamber during periods of high flow. This is important since fines, which are typically considered highly polluted, are conveyed during low/normal flows.

The excess flow is conveyed directly into the middle chamber where it receives treatment for floatables and solids via the baffle system. This treatment of the higher flow rates is important since trash and heavier solids are typically conveyed during periods of higher flow rates. The Hydroworks HG separator is revolutionary since it incorporates low and high flow treatment in one device while maintaining separate low and high flow paths to prevent the scour and re-suspension of fines.

Figure 2 is a profile view of the HG separator showing the flow patterns for low and high flows.

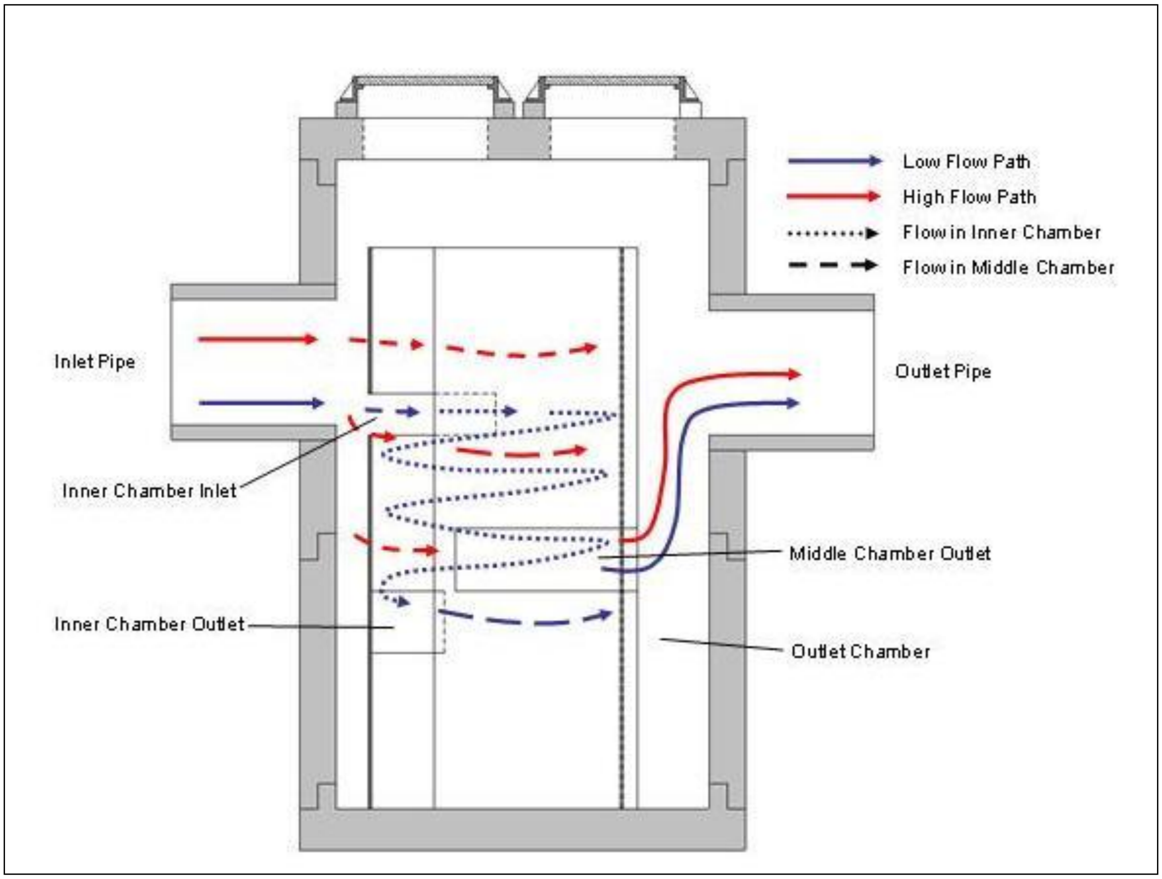
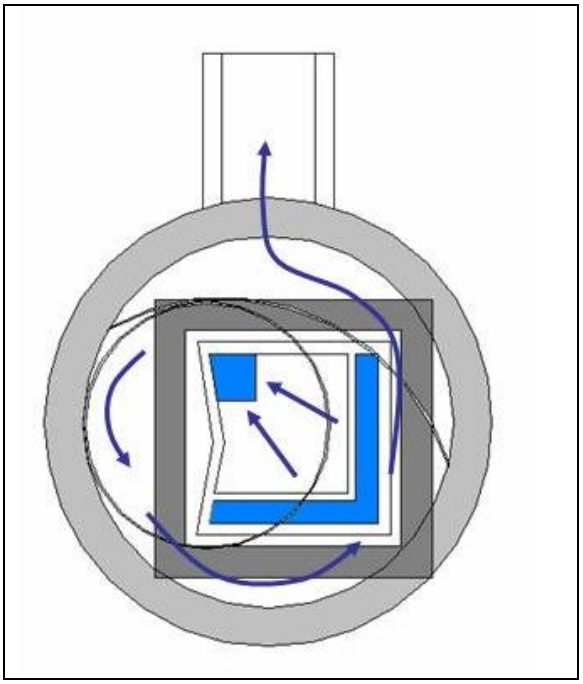


Figure 2. Hydroworks HG Operation – Profile View

The HG 4i is an inlet version of the HG 4 separator. There is a catch-basin grate on top of the HG 4i. Water flows directly into the inner chamber of the HG 4i through the catch-basin grate on top of the structure. The grate is oversized to allow maintenance of the entire structure. A funnel that sits underneath the grate on the top cap of the concrete itself directs the water into the inner chamber during normal flows and the middle chamber during high flows. Figures 3 and 4 show the flow paths for the HG 4i separator.

The inlet funnel is sloped towards the corner inlet and hence the wall of the inner chamber. Water moves in a circular direction in the inner chamber since water enters tangentially along the wall of the inner chamber due to the sloping funnel.

Water continues moving in a circular motion (vortex) through the rest of the structure (through the middle chamber and baffle wall) until it is discharged from the separator.



During periods of peak flow the water will back up from the corner inlet and overflow into two side overflow troughs which discharge directly into the middle chamber. These overflow troughs are covered from the surface such that water cannot directly fall through them (i.e. water must back up to enter the overflow troughs).

Accordingly this funnel provides the same separate flow paths for low and high flow as the other Hydroguard separators.

The whole funnel is removed for inspection and cleaning providing.

Figure 3. Hydroworks Hydroguard HG 4i Normal Flow Path

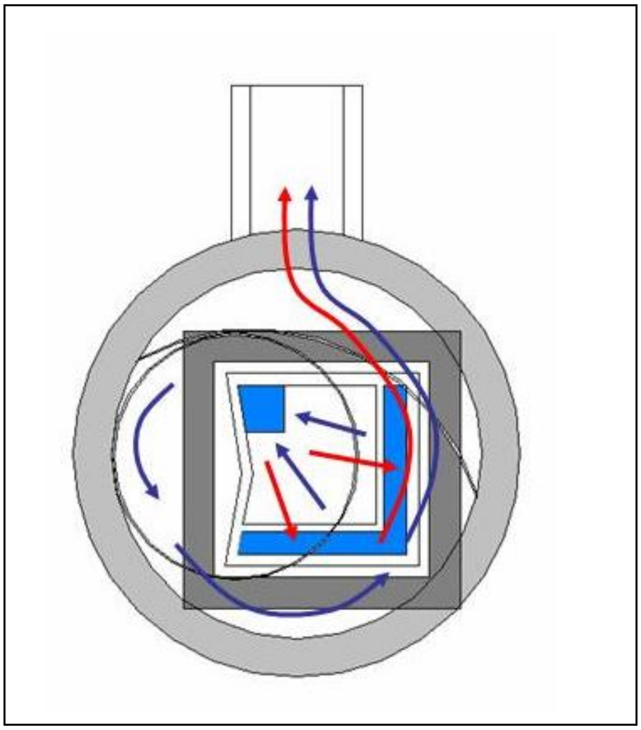


Figure 4. Hydroworks Hydroguard HG 4i Peak Flow Path

Inspection

Procedure

Although all parts of the Hydroguard should be inspected, inspection and maintenance should focus on the inner and middle chambers since this is where the pollutants (floatable and sinking) will accumulate.

Floatables

A visual inspection can be conducted for floatables by removing the covers and looking down into the separator. Multiple covers are provided on Hydroworks HG units to access all areas of the separator (The HG 4 may have a single larger 32” (800mm) cover due to the lack of space for multiple 24” (600mm) covers).

TSS/Sediment

Inspection for TSS build-up can be conducted using a Sludge Judge®, Core Pro®, AccuSludge® or equivalent sampling device that allows the measurement of the depth of TSS/sediment in the unit. These devices typically have a ball valve at the bottom of the tube that allows water and TSS to flow into the tube when lowering the tube into the unit. Once the unit touches the bottom of the device, it is quickly pulled upward such that the water and TSS in the tube forces the ball valve closed allowing the user to see a full core of water/TSS in the unit. The unit should be inspected for TSS through each of the access covers. Several readings (2 or 3) should be made at each access cover to ensure that an accurate TSS depth measurement is recorded.

Frequency

Construction Period

The HG separator should be inspected every two weeks and after every large storm (over 0.5” (12.5 mm) of rain) during the construction period.

Post-Construction Period

The Hydroworks HG separator should be inspected once per year for normal stabilized sites (grassed or paved areas). If the unit is subject to oil spills or runoff from unstabilized (storage piles, exposed soils) areas the HG separator should be inspected more frequently (4 times per year). An initial annual inspection will indicate the required future frequency of maintenance if the unit was maintained after the construction period.

Reporting

Reports should be prepared as part of each inspection and include the following information:

1. Date of inspection
2. GPS coordinates of Hydroworks unit
3. Time since last rainfall
4. Date of last inspection
5. Installation deficiencies (missing parts, incorrect installation of parts)
6. Structural deficiencies (concrete cracks, broken parts)
7. Operational deficiencies (leaks, blockages)
8. Presence of oil sheen or depth of oil layer
9. Estimate of depth/volume of floatables (trash, leaves) captured
10. Sediment depth measured
11. Recommendations for any repairs and/or maintenance for the unit
12. Estimation of time before maintenance is required if not required at time of inspection

A sample inspection checklist is provided at the end of this manual.

Maintenance

Procedure

The Hydroworks HG unit is typically maintained using a vactor truck or clam shell bucket. There are numerous companies that can maintain the HG separator. Envirocalm, LLC, an affiliate company of Hydroworks offers inspection and maintenance services and can inspect and maintain the HG separator. (www.envirocalm.com).

Disposal of the contents of the separator depend on local requirements. Maintenance of a Hydroworks HG unit will typically take 1 to 2 hours.

Frequency

Construction Period

A HG separator can fill with construction sediment quickly during the construction period. The construction sediment will have a much coarser particle size distribution than the suspended solids during the post-development period. Accordingly, scour is not so much of a concern during the construction period compared to the separator filling up with solids. The Hydroguard must be maintained during the construction period when the depth of TSS/sediment reaches 27” (675 mm). This represents 75% of the maximum sediment storage capacity. It must also be maintained during the construction period if there is an appreciable depth of oil in the unit (more than a sheen) or if floatables other than oil cover over 50% of the open water surface on the inlet side of the outlet baffle wall.

The HG separator should be maintained at the end of the construction period, prior to utilization for the post-construction period.

Post-Construction Period

The Hydroguard was independently tested by Alden Research Laboratory in 2008. A HG6 was tested for scour with initial sediment loads of 4.6 ft^3 and 9.3 ft^3. The results from these tests were almost identical. Therefore, the 9.3 ft^3 sediment load was used as 50% of the maximum sediment depth for maintenance in the calculation of the maintenance interval for the HG6 separator based on the NJDEP maintenance interval equation.

Maintenance Interval (months) = 3.565 x (Sediment Storage) / (MTFR x TSS Removal)

Maintenance Interval (HG6) = 3.565 x 9.3 / (1.67x 0.55) = 36 months

All values (flow, sediment storage) can be scaled by the surface area making the sediment depths and maintenance intervals equal for all separators.

The separator was loaded with the sediment in the inner chamber and middle chamber with the majority of sediment (80%) located in the inner chamber. The inner chamber for area represents approximately 44% of the separator surface area. The inner chamber is 4 ft (1200 mm) in diameter in the HG6. Therefore the 50% sediment depth for the HG6 in the inner chamber would be:

9.3 ft^3 x 0.80 / (3.14 x 4 ft^2) x 12 in/ft = 7.1 inches (175 mm)

Accordingly the 100% sediment volume would represent 14.2” (350 mm) of sediment depth in the inner chamber.

The HG separator must be maintained if there is an appreciable depth of oil in the unit (more than a sheen) or if floatables other than oil cover over 50% of the open water surface on the inlet side of the outlet baffle wall. It should also be maintained once the accumulated TSS/sediment depths are greater than 14” (350 mm) in the inner chamber. For typical stabilized post-construction sites (parking lots, streets) it is anticipated that maintenance will be required annually or once every two years. More frequent or less frequent maintenance will be required depending on individual site conditions (traffic use, stabilization, storage piles, etc.). The long term maintenance frequency can be established based on the maintenance requirements during the first several years of operation if site conditions do not change.

Hydroworks

HYDROGUARD INSPECTION SHEET

Date ______________________________
Date of Last Inspection ______________________________

Site ______________________________
City ______________________________
State ______________________________
Owner ______________________________

GPS Coordinates ______________________________

Date of last rainfall ______________________________

Site Characteristics	**Yes**	**No**
Soil erosion evident	☐	☐
Exposed material storage on site	☐	☐
Large exposure to leaf litter (lots of trees)	☐	☐
High traffic (vehicle) area	☐	☐

Hydroguard	**Yes**	**No**
Incorrect access orientation	☐ ***	☐
Obstructions in the inlet or outlet	☐ *	☐
Missing internal components	☐ **	☐
Improperly installed internal components	☐ **	☐
Improperly installed inlet or outlet pipes	☐ ***	☐
Internal component damage (cracked, broken, loose pieces)	☐ **	☐
Floating debris in the separator (oil, leaves, trash)	☐	☐
Large debris visible in the separator	☐ *	☐
Concrete cracks/deficiencies	☐ ***	☐
Exposed rebar	☐ **	☐
Water seepage (water level not at outlet pipe invert)	☐ ***	☐

Water level depth below outlet pipe invert _______“

Routine Measurements

Floating debris depth	**< 0.5” (13mm)**	☐	**>0.5” 13mm)**	☐*
Floating debris coverage	**< 25% of surface area**	☐	**> 25% surface area**	☐ *
Sludge depth	**< 14” (350mm)**	☐	**> 14” (350mm)**	☐ *

Other Comments: ____
____
____

*	Maintenance required
**	Repairs required
***	Further investigation is required

Please call Hydroworks at 888-290-7900 or email us at support@hydroworks.com if you have any questions regarding the Inspection Checklist. Please fax a copy of the completed checklist to Hydroworks at 888-783-7271 for our records.

APPENDIX D

WATER ANALYSIS INFORMATION

FIRE FLOW DEMAND REQUIREMENTS - FIRE UNDERWRITERS SURVEY (FUS GUIDELINES)

Project Number: **18091**
Project Name: **1110 Lorne Park Road**
Date: **Mar-19**

Fire flow demands for the FUS method is based on information and guidance provided in "Water Supply for Public Protection" (Fire Underwriters Survey, 1999).

An estimate of the fire flow required is given by the following formula:

$$F = 220\,C\sqrt{A} \qquad \textbf{(1)}$$

where:

the required fire flow in litres per minute
coefficient related to the type of construction
= 1.5 for wood frame construction (structure essentially all combustible).
= 1.0 for ordinary construction (brick or other masonry walls, combustible floor and interior)
= 0.8 for non-combustible construction (unprotected metal structural components, masonry or metal walls)
= 0.6 for fire-resistive construction (fully protected frame, floors, roof)
Total floor area in square metres

				(1)		(2)			(3)		(4)		**Final Adjusted Fire Flow**	
Building / Location	# of Storeys	**Total GFA (m^2)**	Type of Construction	***Fire Flow "F"***		***Occupancy***			***Sprinkler***		***Exposure***			
				(l/min)	(l/s)	%	Adjustment (l/min)	**Adjusted Fire Flow (l/min)**	%	Adjustment (l/min)	%	Adjustment (l/min)	**(l/min)**	**(l/s)**
Townhouse	**2**	**527**	**1.0**	5100	85.0	**0**	0.0	**5100.0**	**10**	510.0	45	2295.0	**8000**	**133**

(2) Occupancy

Non-Combustible	-25%
Limited Combustible	-15%
Combustible	No Charge
Free Burning	15%
Rapid Burning	25%

(3) Sprinkler

Minimum credit for systems designed to NFPA 13 is 30%.

If the domestic and fire services are supplied by the same municipal water system, then take an additional 10%.

If the sprinkler system is fully supervised (ie. annunciator panel that alerts the Fire Dept., such as a school), then an additional 10% can be taken. Maximum credit = 50%.

(4) Exposure

0 to 3m	25%
3.1 to 10m	20%
10.1 to 20m	15%
20.1 to 30m	10%
30.1 to 45m	5%

Calculate for all sides. Maximum charge shall not exceed 75%

Side	Exposure (m)	Charge (%)
North =	**28**	**10**
South =	**39.1**	**5**
East =	**28.4**	**10**
West =	**7.8**	**20**
Total Expoure =		***45***

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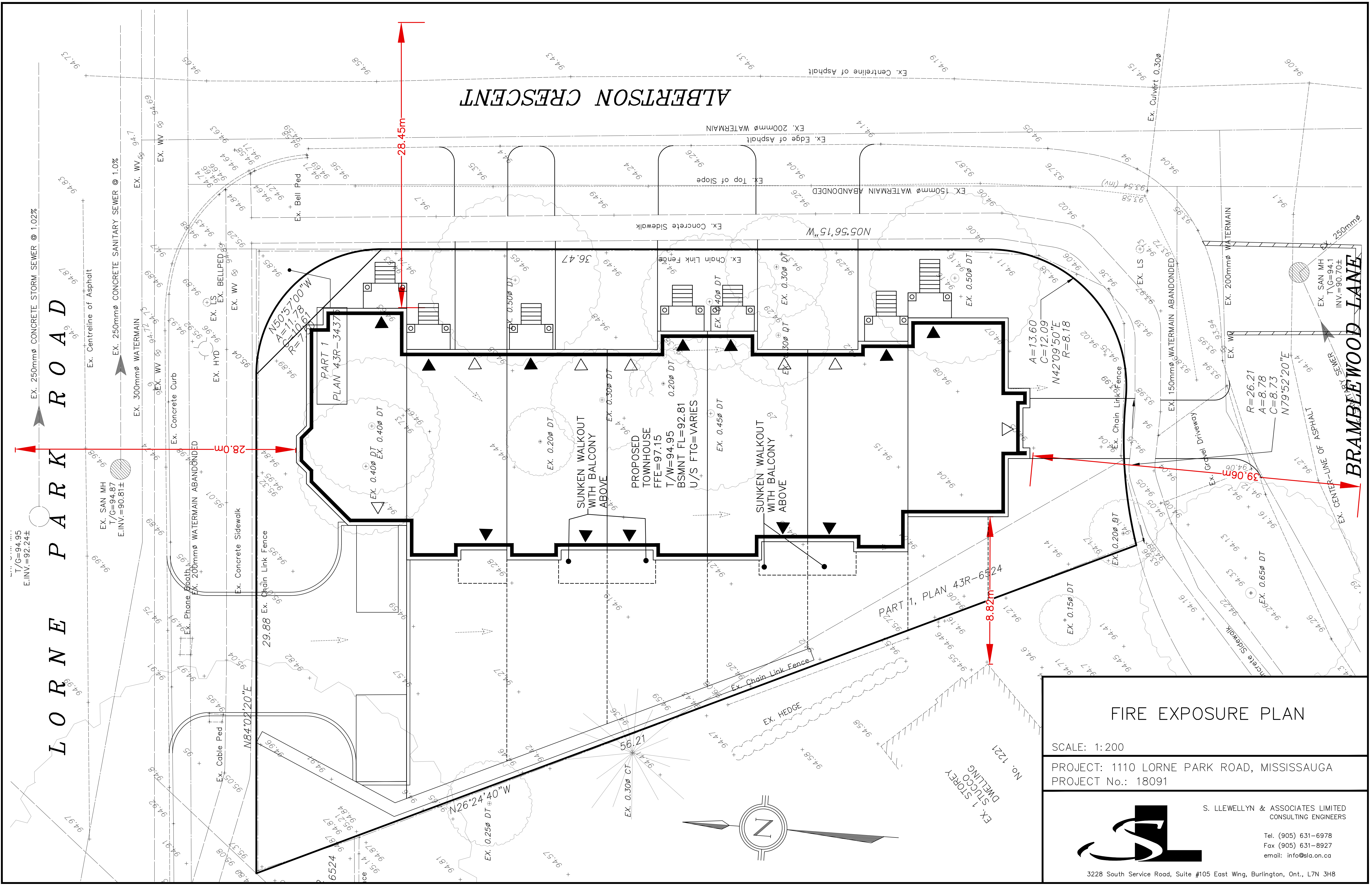

FIRE EXPOSURE PLAN

SCALE: 1:200

PROJECT: 1110 LORNE PARK ROAD, MISSISSAUGA
PROJECT No.: 18091

S. LLEWELLYN & ASSOCIATES LIMITED
CONSULTING ENGINEERS

Tel. (905) 631-6978
Fax (905) 631-8927
email: info@sla.on.ca

3228 South Service Road, Suite #105 East Wing, Burlington, Ont., L7N 3H8

3/20/2019 4:46:32 PM - Stephanie Duffin

APPENDIX E

SANITARY SEWER DESIGN SHEET

Project No. 18091
Sheet No. 1
Checked by:
Computed by: SD
Date: March 7, 2019

CITY OF MISSISSAUGA

SANITARY SEWER DESIGN

Project: 1110 Lorne Park Road

Design flow factor = 302.8 l/day per person

$M = 1 + \frac{14}{4 + p^{0.5}}$ P is in thousands

Infiltration factor = 0.20 l/s/ha

Vmin = 0.75m/s
Vmax = 3.5m/s
n=0.013 (>600mm)
n=0.015 (<600mm)

Area No.	Street Name	From MH	To MH	Pop. Density [per/ha]	Incremental Area [ha]	Cumulative Area [ha]	Population Increment [per]	Cumulative Population [per]		Peaking Factor M	Average Flow l/s	Peak Flow l/s	Infil-tration l/s	Total Flow l/s	Proposed Sewer Design					% Full	Actual Velocity (m/s)	Remarks
															Diameter [mm]	Material	Grade %	Capacity l/s	Velocity m/s			
1	No Street	MH1A	Ex. MH	175	0.14	0.14	25	25		4.37	0.10	0.45	0.03	0.47	200	PVC	2.00%	40.82	1.28	1.2	0.35	
2	Bramblewood Lane	Ex. MH1	Ex. MH	70	0.48	0.62	33	58		4.30	0.24	1.04	0.12	1.16	250	PVC	1.00%	52.35	1.05	2.2	0.37	
3	Albertson Crescent	Ex. MH		70	0.82	1.44	57	115		4.23	0.48	2.03	0.29	2.32	250	PVC	0.56%	39.17	0.79	5.9	0.31	

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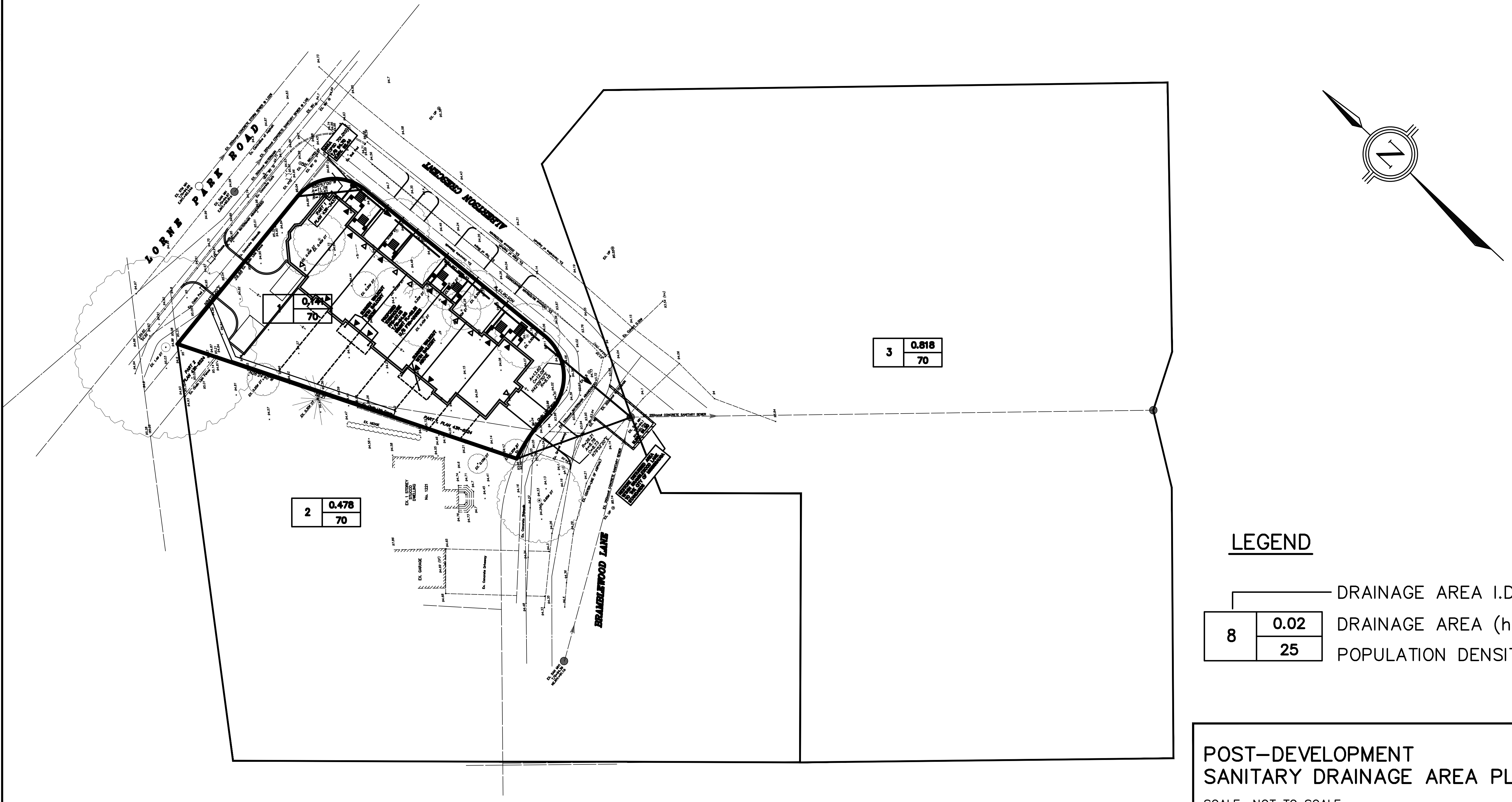


LEGEND

8	0.02	DRAINAGE AREA I.D.
		DRAINAGE AREA (ha)
	25	POPULATION DENSITY

POST-DEVELOPMENT SANITARY DRAINAGE AREA PLAN

SCALE: NOT TO SCALE

PROJECT: 1110 LORNE PARK ROAD, MISSISSAUGA
PROJECT No.: 18091

S. LLEWELLYN & ASSOCIATES LIMITED
CONSULTING ENGINEERS

Tel. (905) 631-6978
Fax (905) 631-8927
email: info@sla.on.ca

3228 South Service Road, Suite #105 East Wing, Burlington, Ont., L7N 3H8

3/20/2019 4:45:53 PM – Stephanie Duffin

APPENDIX F

GEOTECHNICAL REPORT

Jacan Construction Ltd.
5400 Bimini Court
Mississauga, Ontario
L5M 6G9

Attention: Mr. Liaquat J. Mian

RE: GEOTECHNICAL INVESTIGATION FOR THE PROPOSED OFFICE BUILDING AT 1110 LORNE PARK ROAD CITY OF MISSISSAUGA, ONTARIO

Report No. 2011-22768

February 9, 2011

DISTRIBUTION

3 Copies: **Jacan Construction Ltd.**

Original: **(File No. SP-3179)**

SOIL PROBE LTD.

TABLE OF CONTENT

ENCLOSURES

APPENDIX A

SOIL PROBE LTD.
CONSULTING GEOTECHNICAL, INSPECTION & TESTING ENGINEERS
110 IRONSIDE CRESCENT, UNIT 20, SCARBOROUGH, ONTARIO, M1X 1M2
TEL: (416) 754-7055 FAX: (416) 754-1259 e-mail: info@soilprobe.ca

DATE: February 9, 2011

REPORT NO.: 2011-22768
FILE NO.: SP-3179

1.0 INTRODUCTION

Mr. Liaquat Mian of Jacan Construction Ltd., authorized Soil Probe Ltd. (SPL) to carry out a geotechnical investigation for the proposed 2 storied Office Building with a basement (for Storage and Mechanical use only) at 1110 Lorne Park Road, City of Mississauga, Ontario.

As per the terms of reference, the purpose of this geotechnical investigation was to collect information on the subsoil and groundwater conditions at the subject site and to make recommendations for the design and construction of the foundations, basement, etc., for the proposed building as well as driveway pavement design and also to check the suitability of the site for the construction of a Permeable Pavement for the parking lot and to review the structural competency of a proposed design submitted to us.

2.0 PROJECT AND SITE DESCRIPTION

The subject site is located at the south-west corner of Lorne Park Road and Albertson Crescent; it is roughly a trapezium-shaped, and bounded by Bramblewood Lane on the south and a residential dwelling on the west. There are some trees within the site, and at the time of our field work for this investigation the site was covered by a layer of snow.

The existing ground surface within the site is more or less level. The maximum difference in existing grade elevations between the borehole locations is about 70 cm, with the highest elevation of 94.78 m being at the location of Borehole No. 3, drilled near the north-east corner of the site and the lowest elevation of 94.07 m at the location of Borehole No. 7, drilled in the south-western part of the site.

3.0 FIELDWORK

3.1 ITEMS OF FIELDWORK

The fieldwork was carried out on January 21, 2011. A total of seven (7) boreholes were drilled at the locations shown on Plot Plan of Enclosure No. 9. Four of these boreholes were drilled around the proposed building locations and 3 in the parking lot area.

The boreholes were drilled to depths of 6.55 m each (Building B.H. Nos. 1 through 4) and to 2.45 m (Parking Lot Boreholes 5, 6 & 7), all below the existing grade.

3.2 METHOD OF FIELDWORK

The boreholes were advanced using a truck mounted, 115 mm diameter, and solid stem auger machine (CME 45), equipped for soil sampling. Standard penetration tests (SPTs) were conducted according to ASTM Method D1586 at a depth interval of 0.76 m in the top 3.5 m and at 1.5 m at lower levels, in each borehole (Building Boreholes) and continuous sampling (Parking Lot Boreholes). Representative soil samples were recovered from the split spoon sampler used in these SPTs. The results of the SPT, in terms of the number of blows per 0.3 m of penetration after 1st 15 cm, designated as "N-value", have been used to estimate the relative density of native cohesionless soils (No cohesive soil was hit within the investigated depths).

A soil technologist from Soil Probe (under the direction of a Senior Engineer) supervised the fieldwork. The locations of the boreholes were decided by us and our field personnel laid out the boreholes and also determined the existing grade elevations at the borehole locations using **"THE TOP OF THE EXISTING SANITARY MANHOLE, LOCATED ON BRAMBLEWOOD LANE, SOUTH OF THE SITE"** as a Temporary Bench Mark (TBM); the Geodetic Elevation of the TBM was obtained as 94.12 m from a Site Plan Drawing received from the client.

4.0 LABORATORY TESTS

The soil samples recovered from the SPT spoon were properly sealed, labelled and brought to our laboratory. They were visually examined to classify each sample. The natural moisture content of each sample was determined by drying in the oven, in the laboratory.

The natural moisture contents, the description and classification of each sample and the N-values (of SPTs) are presented in the borehole logs on Enclosure Nos. 1 through 7, while the terms and symbols used to describe the soils on these logs are summarized on Enclosure No. 8.

A composite soil sample was prepared through mixing of SPT samples from depth 0.76 m to 2.3 m of Parking Lot B.H. Nos. 5, 6 & 7 and subjected to grading test. The results of the grading analysis are presented on Enclosure No. 10.

5.0 SUBSOIL CONDITIONS

The investigations reported herein indicate that the site is underlain by a surficial topsoil/fill cover followed by native soils comprising sand to silty sand.

Detailed soil descriptions at the borehole locations are given in the borehole logs (Enclosure Nos. 1 through 7) while generalized descriptions of the different subsoil units encountered within the investigated depths are given in the following subsections.

5.1 TOPSOIL (SURFICIAL AND BURIED)

A surficial layer of topsoil, about 100 mm to 800 mm thick was found at existing grade at the locations of B.H. Nos. 1, 2, 3, 5, 6 & 7; also a buried topsoil layers, about 300 to 400 mm thick was found between two fill layers (discussed next) at B.H. Nos. 4 & 6.

5.2 FILL

Fill was encountered at existing grade at B.H. No. 4 and below topsoil at B.H. Nos. 1, 2, 3, 5, 6 & 7. The fill layers extend to depths in the range of about 1.8 m (B.H. No. 5) to 2.9 m (B.H. No. 4) below the existing grade.

The fill layers included materials varying from mixed dark brown and grey sand with some gravel, through reddish brown fine sand, occasionally with trace of silt, greyish brown fine to medium sand, fine sand with trace to some organics, dark brown/greyish brown fine sand occasionally with trace to some organics, grey silty sand with trace of organics dark brown medium sand.

The natural moisture contents of the fill layers are in the range of about 4.8% (B.H. No. 6) to 17% (B.H. No 4), with some of the higher values reflecting organic/topsoil inclusions. The N-values (from SPT) of these layers are in the range of 4 (B.H. Nos. 5 & 7) to 19 (B.H. Nos. 2 & 5), the relatively higher N-values, being generally associated with the gravel-rich fill layers.

The grading curve obtained from grain-size analysis of the composite fill sample from parking lot B.H. Nos. 5, 6 & 7 is presented on Enclosure No. 10; it confirms that the fill layer is fine sand with some silt.

5.3 NATIVE GRANULAR SOIL

Granular soils are the only ones encountered within the investigated depths, occurring at all the building boreholes and at B.H. No. 5; these occur below the fill layers at depths varying from about 1.8 m to 2.9 m, and comprise materials varying from fine sand with trace of silt to silty fine sand to medium sand. The N-values of these layers are in the range of 10 (B.H. No. 2) to 50 (B.H. No. 4), suggesting these layers to be in loose to very dense conditions.

The natural moisture content of the granular soils are in the range of about 8% (B.H. No. 5) to 24% (B.H. No. 2), indicating their moist to wet conditions.

6.0 GROUNDWATER CONDITIONS

The boreholes were advanced using dry augering, and ground seepage water was found at a depth below existing grade of about 4.2 m in Borehole No. 2, 4.0 m in Borehole No. 3, and 3.8 m (B.H. No. 4).

Based on the above information and visual examination of the soil samples obtained, in our opinion, the ground seepage water encountered in the above boreholes represents true water table in the locality.

7.0 DISCUSSIONS & RECOMMENDATIONS

As per the design drawings received from the client, the proposed office building will be a 2-storied structure with a basement (for storage and mechanical use only) and a permeable parking lot on its south and driveway entrance from Albertson Crescent. Based on the above information and the geotechnical data collected through our investigation and presented in the preceding chapters, our comments and recommendations are as follows.

7.1 FOUNDATION DESIGN (REF. B.H. NOS. 1, 2, 3 & 4)

The boring data of the above-noted boreholes have indicated that the undisturbed native ground is suitable for supporting the proposed building through conventional spread, circular and/or strip footing foundations. The footings can be founded at a minimum depth of 2.25 m (B.H. Nos. 1 & 2), 2.75m (B.H. No. 3) and 3.05 m (B.H. No. 4) below the existing grade. Allowable soil bearing pressures of 300 kPa (SLS) and 400 kPa (ULS) are recommended for footing design.

For the above soil bearing pressure, it is assumed that the footings will have a minimum width of 600 mm and a minimum depth/width ratio of 0.5. For footings in the basements, the depth of footing should be considered from the top of finished basement floor. For footings of smaller width or smaller depth/width ratio, the allowable soil bearing pressure should be decreased proportionately. For frost protection, external footings should be covered with at least 1.2 m of soil. Also if the basement is un-heated the interior footings should also be placed at least 1.2 m below finished basement level.

Prior to pouring concrete footings, the subsoil at the footing founding levels should be inspected by a soils engineer from this office.

7.2 BASEMENT CONSTRUCTION (FOR STORAGE & MECHANICAL EQUIPMENTS)

The installation of perimeter weepers enclosed in filter socks around exterior footings would be required as per the Ontario Building Code requirement. The weeping tiles should be connected to a sump, as there is no storm sewer along the streets adjacent to the subject site. Furthermore, in view of permeable parking lot pavement (discussed next) proposed to be constructed adjacent to the building, the exterior faces of the foundation walls should be water proofed.

Basement floor slabs can rest on undisturbed natural ground. For bedding and to serve as a moisture barrier under the basement floor slabs, a minimum of 150 mm thick layer of crushed stone should be placed.

7.3 PARKING LOT PAVEMENT (Ref. B.H. Nos. 5, 6 & 7)

It is proposed to construct the parking lot pavement as a permeable pavement, for which a design has been proposed by the client's consultants, and Soil Probe has been requested to check if the subject site is suitable for this type of pavement and also to assess the structural adequacy of the proposed design. This has been carried out as discussed below.

A) Site Suitability: The suitability of the site depends on the infiltration characteristics of the subgrade fill/native soil and the position of the water table. In this context, reference is made to the grading curve of the composite fill layer, Enclosure No. 10; it shows that the tested composite sample is fine sand with some silt; also the native soil below fill is sandy in texture. The infiltration rate for the tested fill material is estimated to be about 30 mm/hour; however, as infiltration performance is affected by clogging over time (as fine particles invade the permeable pathways), allowance for clogging must be considered. As such a long term infiltration rate of 20 mm/hour is recommended for the pavement subgrade. The minimum depth of water table at this site is 3.8 m below existing grade (Ref. log of BH. No. 4, Enclosure No. 4).

Based on the above data, in our opinion, permeable pavement construction is feasible at this site.

B) Review of Proposed Pavement Structure: We have reviewed the proposed Permeable Pavement Structure Detail (Drawing No. 208147-SK2), dated February, 2011, prepared by EMC Group Limited for this site (Copy attached). As per this drawing, the proposed permeable pavement structure will be as follows:

Pavement Component	Thickness (mm)
Permeable Concrete Pavers	80
5 mm Gravel leveling Course	50
20 mm Clear crushed Granular	100
50 mm Clear Crushed Granular	300
Total Thickness	**530 mm**

The design also includes installation of 100 mm diameter perforated pipe subdrain. Based on our estimate of Granular Base Equivalency of the above design, in our opinion, the above-listed pavement structure would be adequate for a normal parking lot from a structural point of view, provided the subgrade, consisting of a relatively fine-textured fill of marginal compactness condition is proof rolled, and any soft areas removed.

Prior to placing the filter cloth (to separate the pavement granulars from the subgrade soil) all topsoil (surficial/buried) should be removed (or salvaged for landscaping) and the subgrade should be compacted to obtain a minimum of 98% Standard Proctor Maximum Dry Density (SPMDD). The granular materials should then be placed in thin layers and compacted with a heavy smooth drum roller (as per related City/CVC Document) to eliminate any inter-layer voids.

7.4 DRIVEWAY PAVEMENT (REF. B.H. NOS. 4 & 6)

It is understood that the project envisages one driveway entrance from Albertson Crescent. For the construction of this driveway all the topsoil and organic-rich fill, should be completely removed (or saved for landscaping).

Based on the geotechnical data from the above boreholes, the undisturbed native ground as well as relatively clean existing fill soils can support the proposed driveway pavement. Accordingly, in view of the frost susceptibility and drainage characteristics of the on-site soils and the expected volume of traffic for an office development the following pavement design will perform satisfactorily.

Recommended Driveway Pavement Design

PAVEMENT COMPONENTS	HEAVY DUTY DRIVEWAY
Asphalt Wearing Course (OPSS 1150) HL-3	40 mm
Asphalt Base Course (OPSS1150) HL8	60 mm
OPSS Granular 'A' Base (OPSS 1010)	150 mm
OPSS Granular 'B' Sub-base (OPSS 1010)	400 mm
Alternatively	
20 mm Crusher Run Limestone (CRL)	150 mm
50 mm Crusher Run Limestone	300 mm

The 20 mm diameter CRL shall meet the Ontario Provincial Standard Specification (OPSS) Granular "A" gradation specification. The 50 mm diameter CRL shall meet the OPSS Granular B "Type I" gradation specification. The stone bases should be compacted to at least 100% of their SPMDD.

The asphaltic concretes are to be hot-mixed, hot-laid in accordance with current OPSS specifications, Forms 310 and 1150 (Ontario PGAC grades PG 58-28equivalency), and compacted to a minimum of 92.5 – 96% of maximum Relative Density (mRD).

Prior to placing the granular bases, the final subgrade should be proof-rolled to identify soft spots, if any, and rectified as required.

In order to intercept infiltrating water and provide drainage of the subgrade and pavement material, we recommend that subdrains, wrapped in filter cloth, be provided along both sides of the driveways in the proposed subgrade. Also, the subgrade should be crowned to promote flow of water towards the subdrains and catch basins.

7.5 EARTH PRESSURES

The following equation should be used to estimate the intensity of the lateral earth pressure acting against any earth retaining structure, such as the walls of the basement.

$P = K(\gamma h+q)$

Where K = Appropriate coefficient of earth pressure;
γ = Unit weight of compacted backfill, adjacent to the walls;
h = Depth (below adjacent highest grade) at which P is calculated;
q = intensity of any surcharge distributed uniformly over the backfill surface.

The coefficient of the earth pressure at rest (K_o) should be used in the calculation of the earth pressure on the basement walls, which are expected to be rather rigid and not to deflect.

For the on-site soils, the following geotechnical parameters may be assumed:

i) Wet unit weight (γ) kN/m^3 = 19.0

ii) Coefficients of Earth Pressure:

at Rest (K_o)	=	0.5
Active (K_a)	=	0.3
Passive (K_p)	=	3.0

7.6 EXCAVATION AND DEWATERING

Excavations for construction of footings, basement, etc. are not anticipated to pose any problem. Any excavation deeper than 1.2 m should be sloped back or shored to conform to the latest version of the Occupational Health and Safety Act (OHSA) and applicable regulations for construction projects.

The existing fill and native granular soils are considered as a Type 3 soils in accordance with the OHSA; according to this Act the sides of open excavations should temporarily be stable with a slope of 1 horizontal to 1 vertical.

No ground water problems are anticipated for excavation above ground water table; any seepage from wet pockets in fill/native soil can be drained out by conventional sump pumping.

7.7 EARTHQUAKE CONSIDERATIONS

In accordance with the Ontario Building Code 2006 (O. Reg. 350/06, as amended) (OBC), the proposed building should be designed to resist earthquake loads.

Based on the OBC, the subject site should be classed as "Site Class D" for designing against earthquake forces.

7.8 SITE PREPARATION

As pointed out earlier in Section 5.0, topsoil and or topsoil-mixed fill have been found at a few borehole locations as also some trees within the site. It is recommended that prior to starting construction the following site preparation should be carried out:

i) all the topsoil and topsoil-mixed fill should be removed (or salvaged for re-use in landscaping),

ii) The trees which interfere with the proposed development should be cut and removed (including their major root systems).

8.0 STATEMENT OF LIMITATIONS

The comments and recommendations presented in this report are based on the geotechnical data gathered from the boreholes at the locations indicated on the plot plan of Enclosure No. 9 and are intended as a guide for the design engineers of the project. Soil and groundwater conditions between and beyond the borehole locations may differ from those encountered at the time of our soil investigation and may become apparent during construction. Our responsibility is limited to an accurate interpretation of the soil and groundwater conditions prevailing at the locations investigated.

9.0 CLOSURE

We feel honoured to be involved in this project. It would be appreciated if we are given the opportunity to ensure that our recommendations are implemented as intended.

Respectfully Submitted
SOIL PROBE LTD.

LICENSED PROFESSIONAL ENGINEER
D. HOSSAIN
PROVINCE OF ONTARIO

Delwar Hossain, Ph.D, P.Eng.
Senior Vice President

for Anwar Memon, M.Phil, DIC., P.Eng.
President

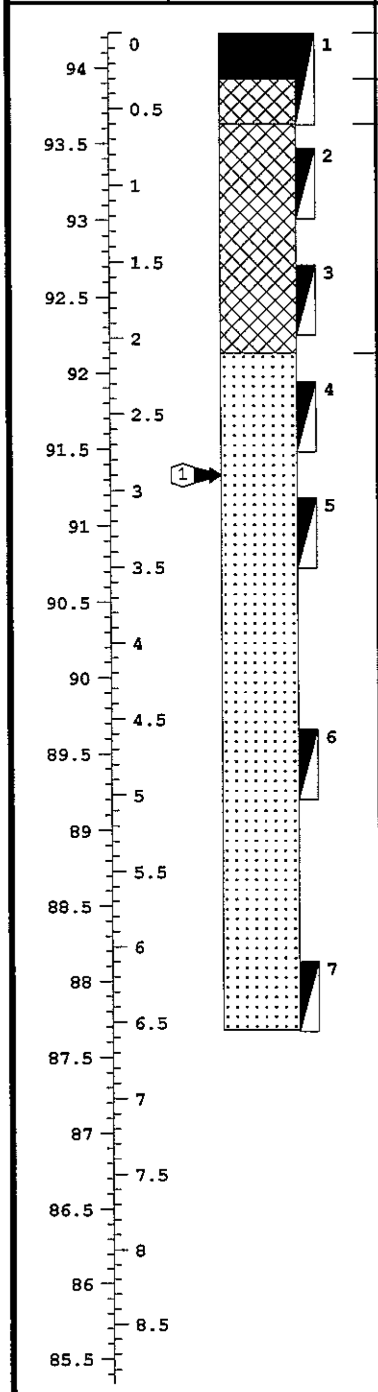
DH-AM\dh-am\td\SHARE2011\SRP 2011\S3179768-Jacan Consturction-Geotech-1110 Lorne Park Rd-Mississauga-Feb 2011

Encls.
Appendix A.

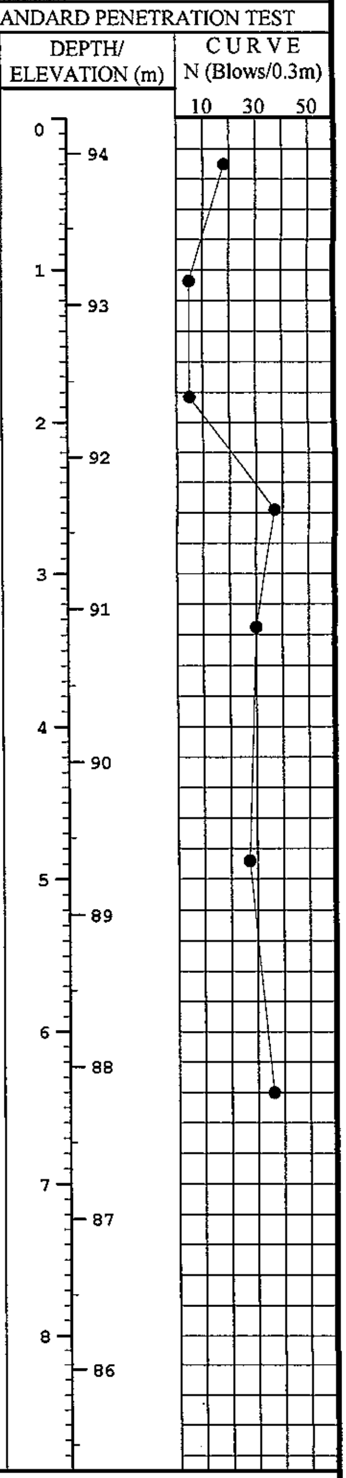
BOREHOLE LOG
BOERHOLE NO.: 1

PROJECT: Proposed Office Building
LOCATION: 1110 Lorne Park Road, City of Mississauga, Ontario
ELEVATION (m) 94.23
CAVED AT DEPTH (m): 2.90
N=Blow Count in Standard Penetration Test (Blows/0.3m)

PROJECT NO.: SP-3179
DATE: January 21, 2011
WATER LEVEL DEPTH (m):
M.C. = Natural Moisture Content



DESCRIPTION	M.C. %	N
TOPSOIL- mixed with fine sand, about 300 mm thick.		
FILL - mixed dark brown and grey sand with some gravel, wet (frozen)	11.8	18
FILL - reddish brown fine sand, moist	6.2	5
- layer of greyish brown, moist, fine to medium sand below about 1.4 m	6.9	5
FINE SAND - trace of silt, greyish brown, moist to very moist, dense.	13.2	37
	18.3	30
- becomes wet and compact to dense below about 4.0 m		
	22.2	27
	21.6	36
End of Borehole @ 6.55 m Cave in at 2.9 m No water in borehole on completion		



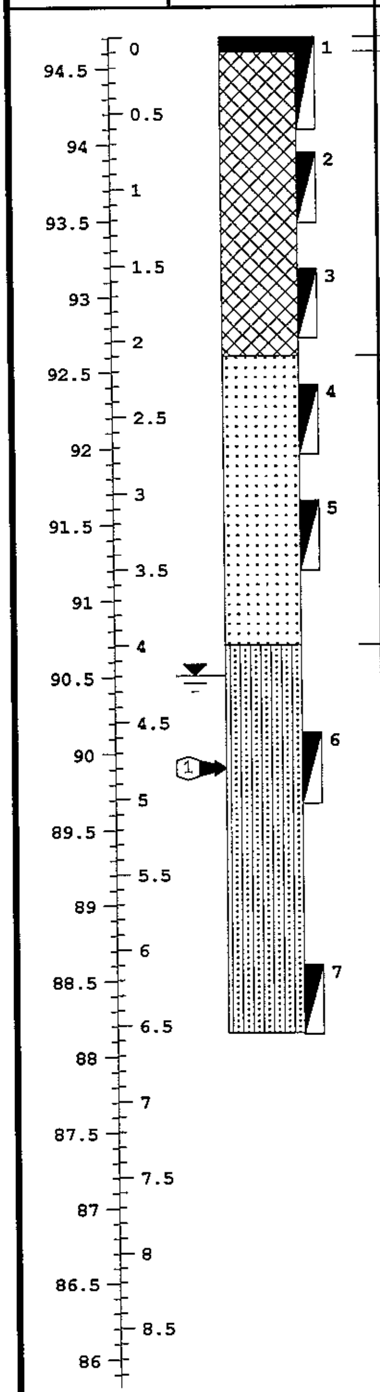
Enclosure No. 1

SOIL PROBE LTD.

BOREHOLE LOG
BOERHOLE NO.: 2

PROJECT: Proposed Office Building
LOCATION: 1110 Lorne Park Road, City of Mississauga, Ontario
ELEVATION (m) 94.71
CAVED AT DEPTH (m): 4.8
N=Blow Count in Standard Penetration Test (Blows/0.3m)

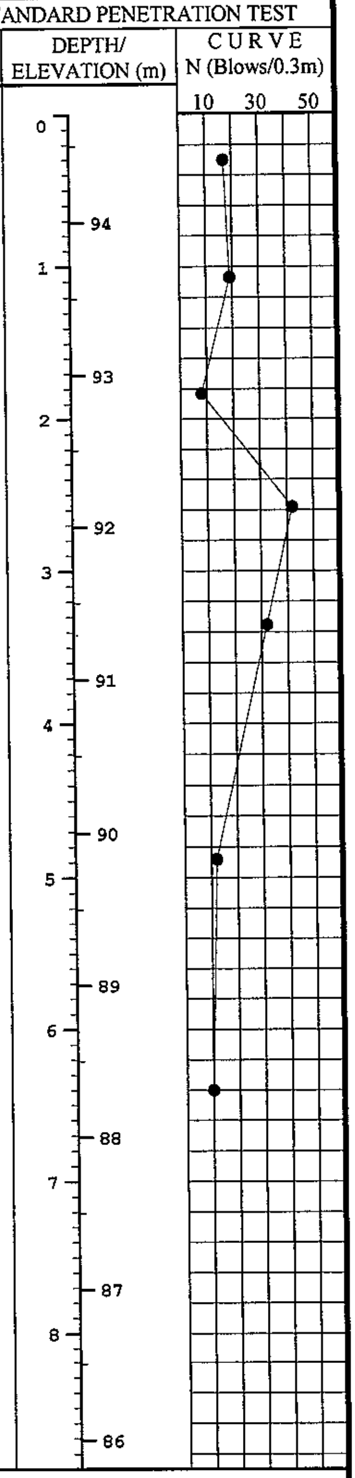
PROJECT NO.: SP-3179
DATE: January 21, 2011
WATER LEVEL DEPTH (m): 4.2
M.C. = Natural Moisture Content



DESCRIPTION	M.C. %	N
TOPSOIL - about 100 mm thick		
FILL - dark brown fine sand with trace to some organics, moist	10.8	17
- layer of dark brown fine sand below about 0.6 m	15.7	19
- occasional root below about 1.4 m.	12.3	8
FINE SAND - trace of silt, greyish brown, moist, dense.	11.1	42
	16.2	32
SILTY FINE SAND - brown, wet, compact to loose.	16.7	12
	23.5	10

End of borehole at 6.55 m
Water level at 4.2 m on completion of borehole
Cave in at 4.8 m.

STANDARD PENETRATION TEST



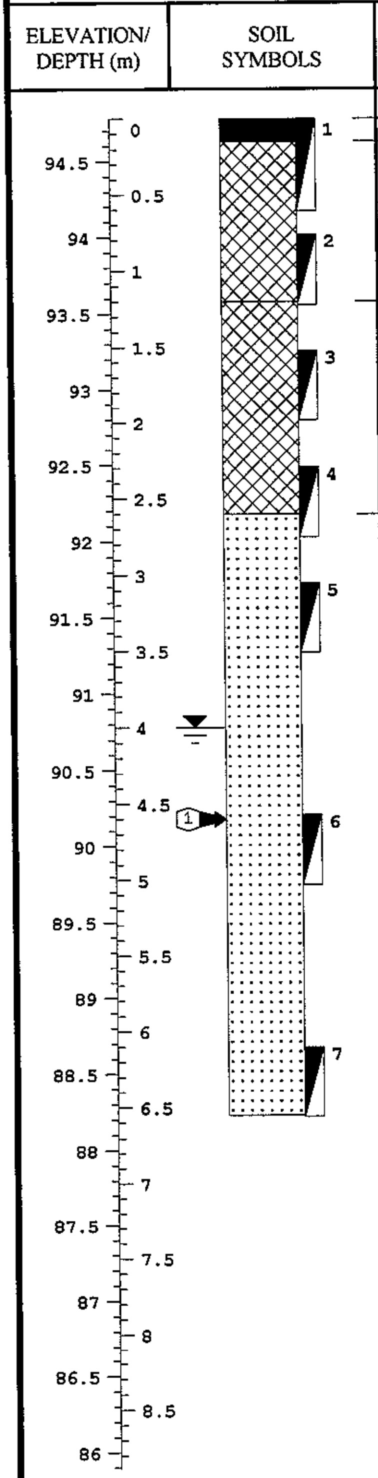
Enclosure No. 2

SOIL PROBE LTD.

BOREHOLE LOG
BOERHOLE NO.: 3

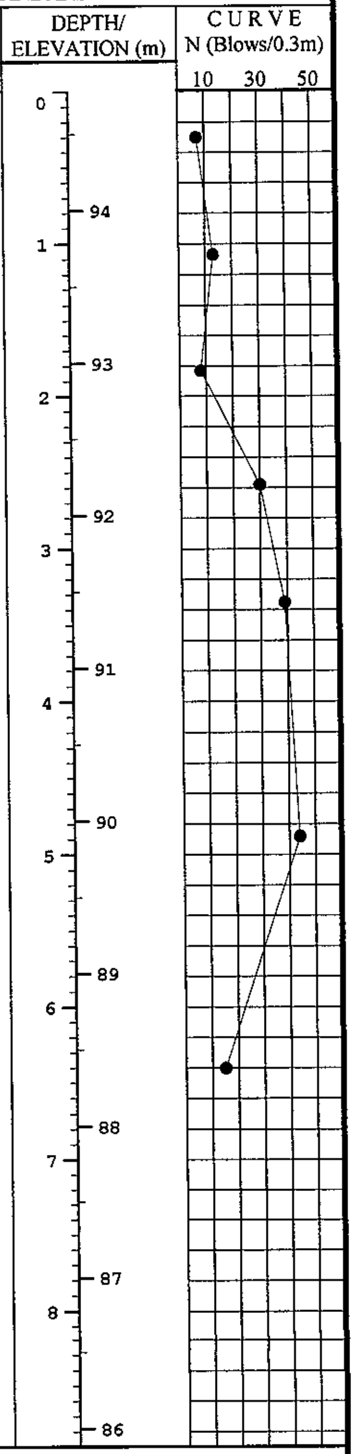
PROJECT: Proposed Office Building
LOCATION: 1110 Lorne Park Road, City of Mississauga, Ontario
ELEVATION (m) 94.78
CAVED AT DEPTH (m): 4.6
N=Blow Count in Standard Penetration Test (Blows/0.3m)

PROJECT NO.: SP-3179
DATE: January 21, 2011
WATER LEVEL DEPTH (m): 4.0
M.C. = Natural Moisture Content



DESCRIPTION	M.C. %	STANDARD PENETRATION TEST N
TOPSOIL - about 150 mm thick		
FILL - dark brown fine sand with trace to some organics, moist	9.9	7
	7.7	13
FILL - dark brown fine sand, moist	7.8	8
	11.1	30
FINE SAND - trace of silt greyish brown, moist, compact to dense.	10.5	39
- wet below about 4.0 m		
	19.8	44
- compact below about 5.5 m.		
	21.6	15

End of borehole at 6.55 m
Water level at 4.0 m on completion of borehole
Cave in at 4.6 m



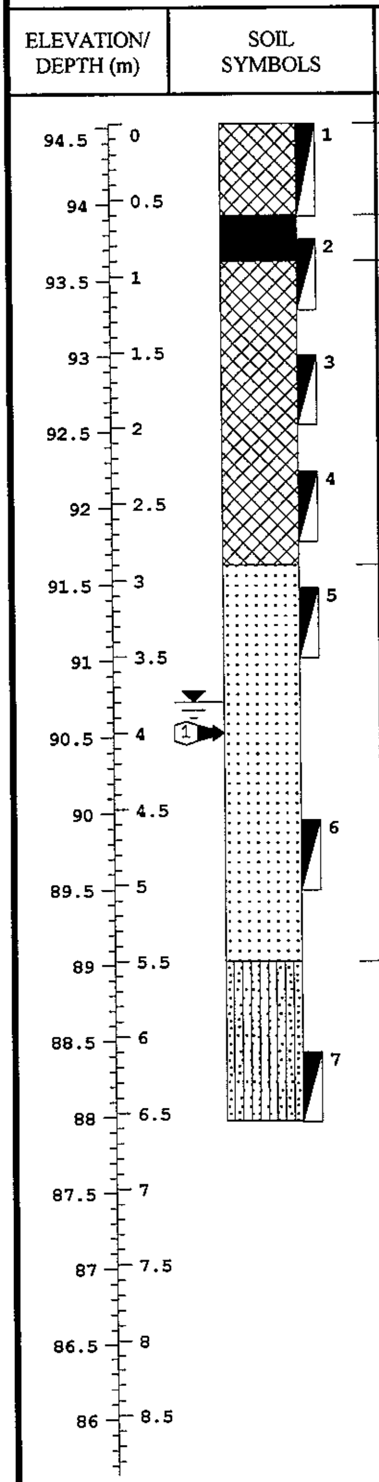
Enclosure No. 3

SOIL PROBE LTD.

BOREHOLE LOG
BOERHOLE NO.: 4

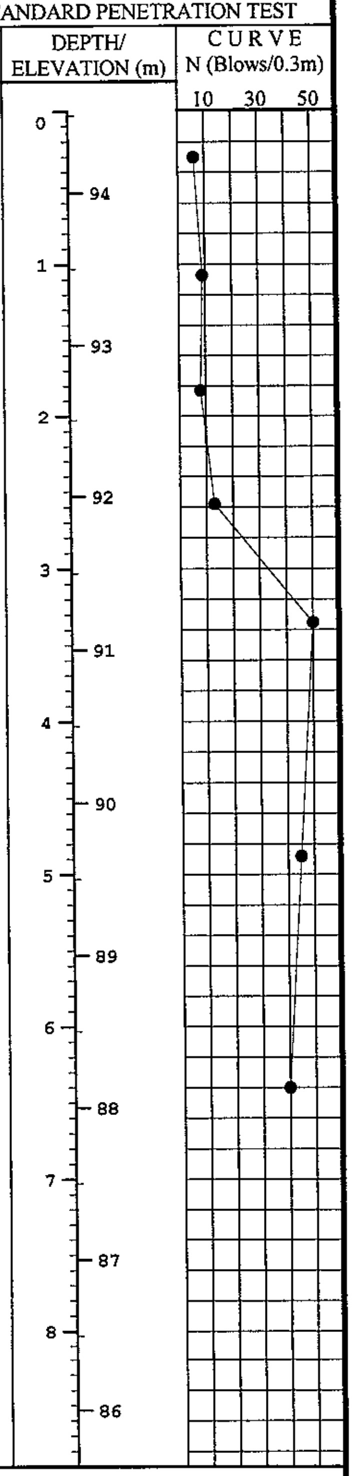
PROJECT: Proposed Office Building
LOCATION: 1110 Lorne Park Road, City of Mississauga, Ontario
ELEVATION (m) 94.53
CAVED AT DEPTH (m): 4.0
N=Blow Count in Standard Penetration Test (Blows/0.3m)

PROJECT NO.: SP-3179
DATE: January 21, 2011
WATER LEVEL DEPTH (m): 3.8
M.C. = Natural Moisture Content



DESCRIPTION	M.C. %	N
FILL - greyish brown fine sand mixed with some topsoil/organics, moist	17.0	6
TOPSOIL - mixed with some fine sand, about 300 mm thick		
FILL - reddish brown fine sand, moist	9.2	9
- layer of dark brown medium sand at about 1.4 m	6.9	8
	10.4	13
MEDIUM SAND - brownish grey, moist, dense to very dense.	12.6	50
- becomes wet & texture changes to fine to medium below about 3. 8 m		
	18.0	45
SANDY SILT - brownish grey, very moist to wet, dense.		
	18.5	40
End of borehole at 6.55 m Water level at 3.8 m Cave in at 4.0 m		

STANDARD PENETRATION TEST

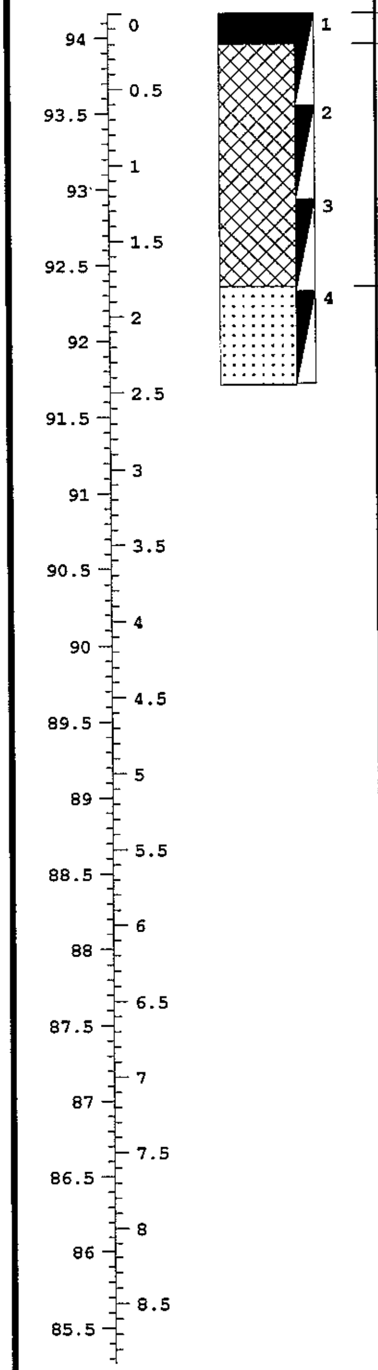


Enclosure No. 4

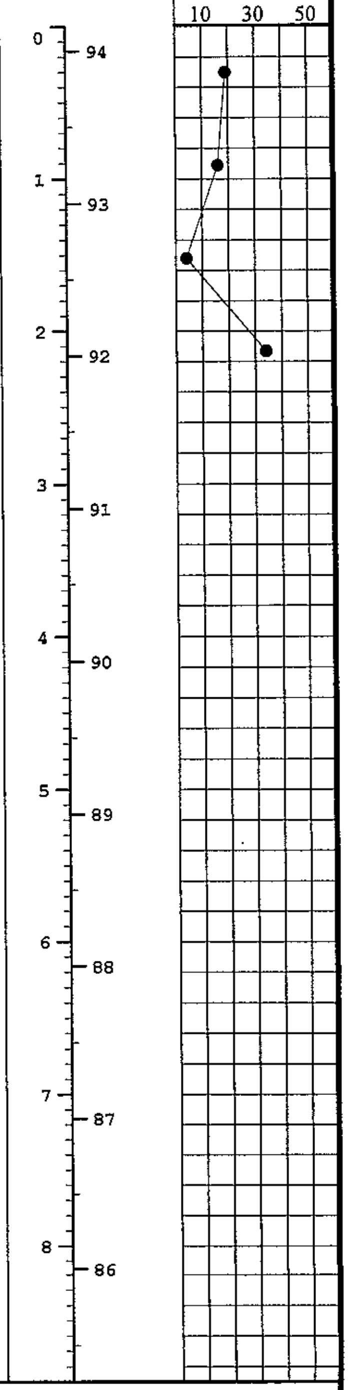
BOREHOLE LOG
BOERHOLE NO.: 5

PROJECT: Proposed Office Building
LOCATION: 1110 Lorne Park Road, City of Mississauga, Ontario
ELEVATION (m) 94.16
CAVED AT DEPTH (m):
N=Blow Count in Standard Penetration Test (Blows/0.3m)

PROJECT NO.: SP-3179
DATE: January 21, 2011
WATER LEVEL DEPTH (m):
M.C. = Natural Moisture Content



DESCRIPTION	M.C. %	STANDARD PENETRATION TEST N
TOPSOIL - about 200 mm thick		
FILL - grey silty sand with trace of organics, moist.	8.6	19
- layer of moist greyish brown fine sand below about 0.4 m		
- layer of reddish brown fine sand with trace of silt below about 0.6 m, moist.	6.6	16
	11.1	4
FINE SAND - trace of silt, brown to greyish brown, moist.	7.9	34
End of borehole at 2.45 m Borehole dry on completion.		



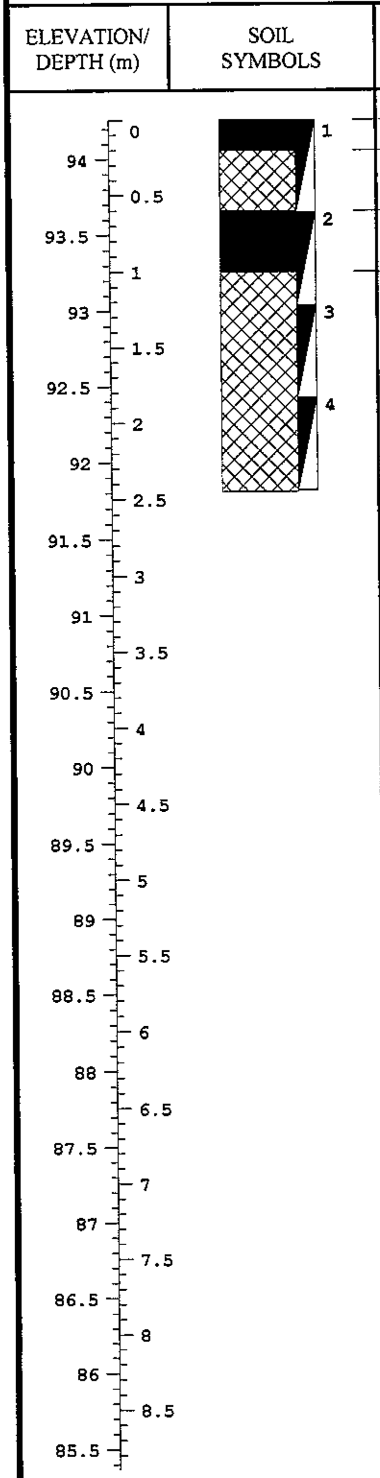
Enclosure No. 5

SOIL PROBE LTD.

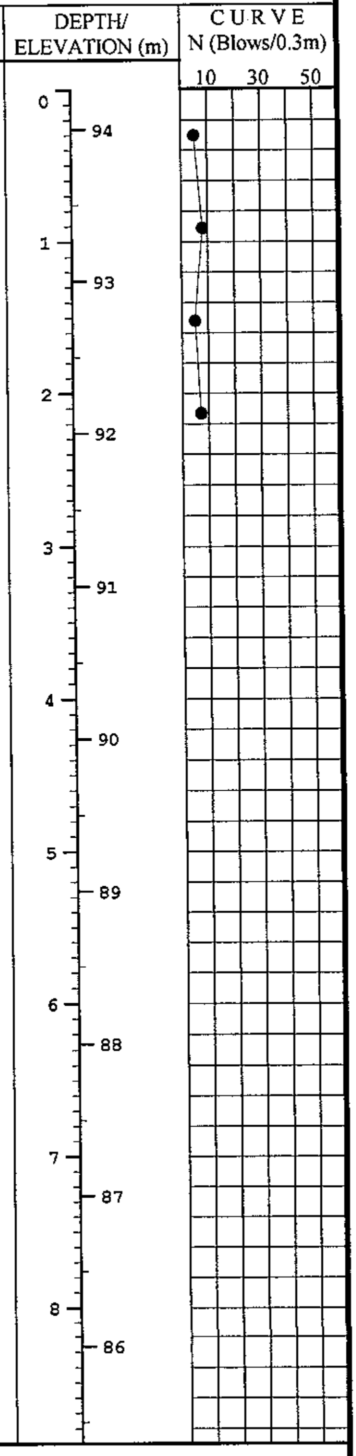
BOREHOLE LOG
BOERHOLE NO.: 6

PROJECT: Proposed Office Building
LOCATION: 1110 Lorne Park Road, City of Mississauga, Ontario
ELEVATION (m) 94.26
CAVED AT DEPTH (m):
N=Blow Count in Standard Penetration Test (Blows/0.3m)

PROJECT NO.: SP-3179
DATE: January 21, 2011
WATER LEVEL DEPTH (m):
M.C. = Natural Moisture Content



DESCRIPTION	M.C. %	STANDARD PENETRATION TEST N
TOPSOIL - about 200 mm thick		
FILL - brown fine sand, trace of gravel, moist.	7.5	5
TOPSOIL - about 400 mm thick		
	8.9	8
FILL - reddish brown fine sand, moist		
	4.8	5
- occasional root below about 1.8 m		
	5.8	7
End of borehole at 2.45 m Borehole dry on completion.		



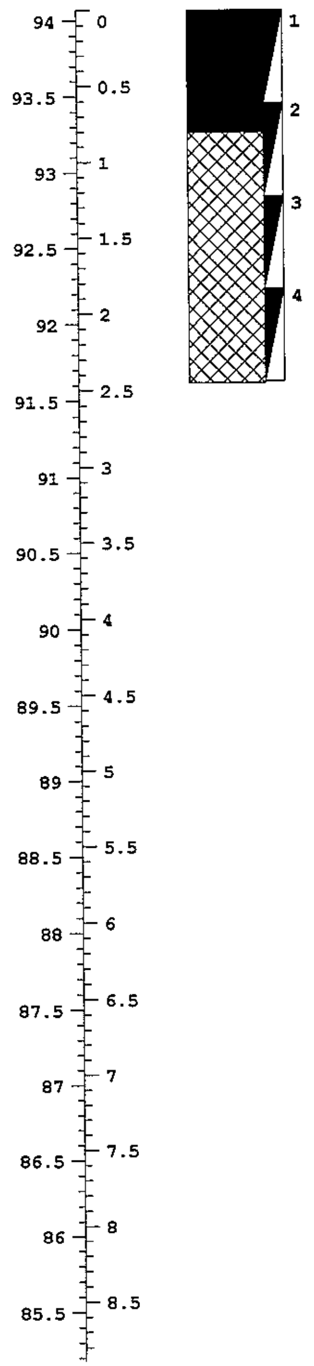
Enclosure No. 6

SOIL PROBE LTD.

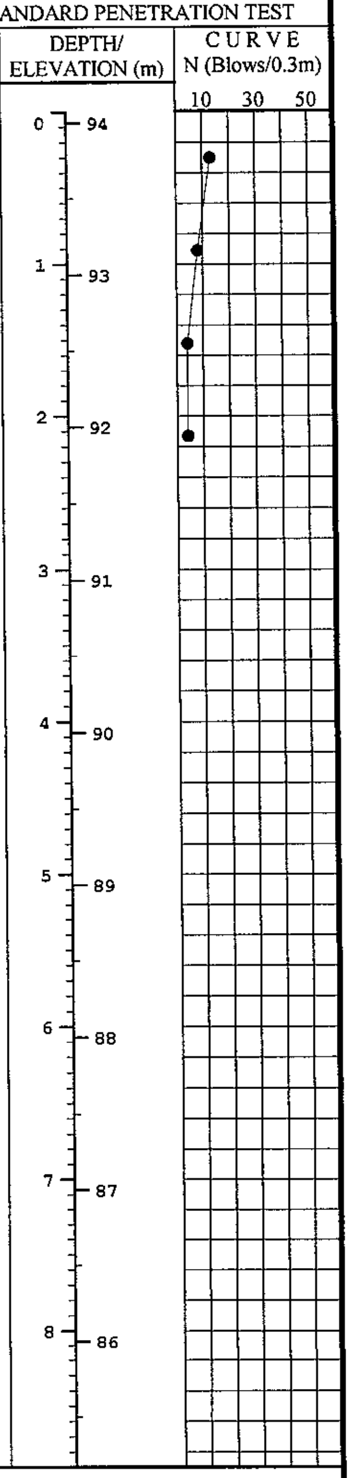
BOREHOLE LOG
BOERHOLE NO.: 7

PROJECT: Proposed Office Building
LOCATION: 1110 Lorne Park Road, City of Mississauga, Ontario
ELEVATION (m) 94.07
CAVED AT DEPTH (m):
N=Blow Count in Standard Penetration Test (Blows/0.3m)

PROJECT NO.: SP-3179
DATE: January 21, 2011
WATER LEVEL DEPTH (m):
M.C. = Natural Moisture Content



DESCRIPTION	M.C. %	N
TOPSOIL - mixed with some sand and trace of gravel size stones, moist (frozen) about 800 mm thick.	9.2	13
FILL - reddish brown fine sand, moist.	7.1	8
- includes occasional roots, below about 1.2 m	6.1	4
	6.8	4
End of borehole at 2.45 m Borehole dry on completion.		



Enclosure No. 7

SOIL PROBE LTD.

KEY TO SYMBOLS

Enclosure No. 8
Report No.: 2011 – 22768

Symbol	Description
Strata symbols	
▩	Fill
░	Sand
▥	Silty sand
▦	Sandy silt
■	Topsoil

Notes:

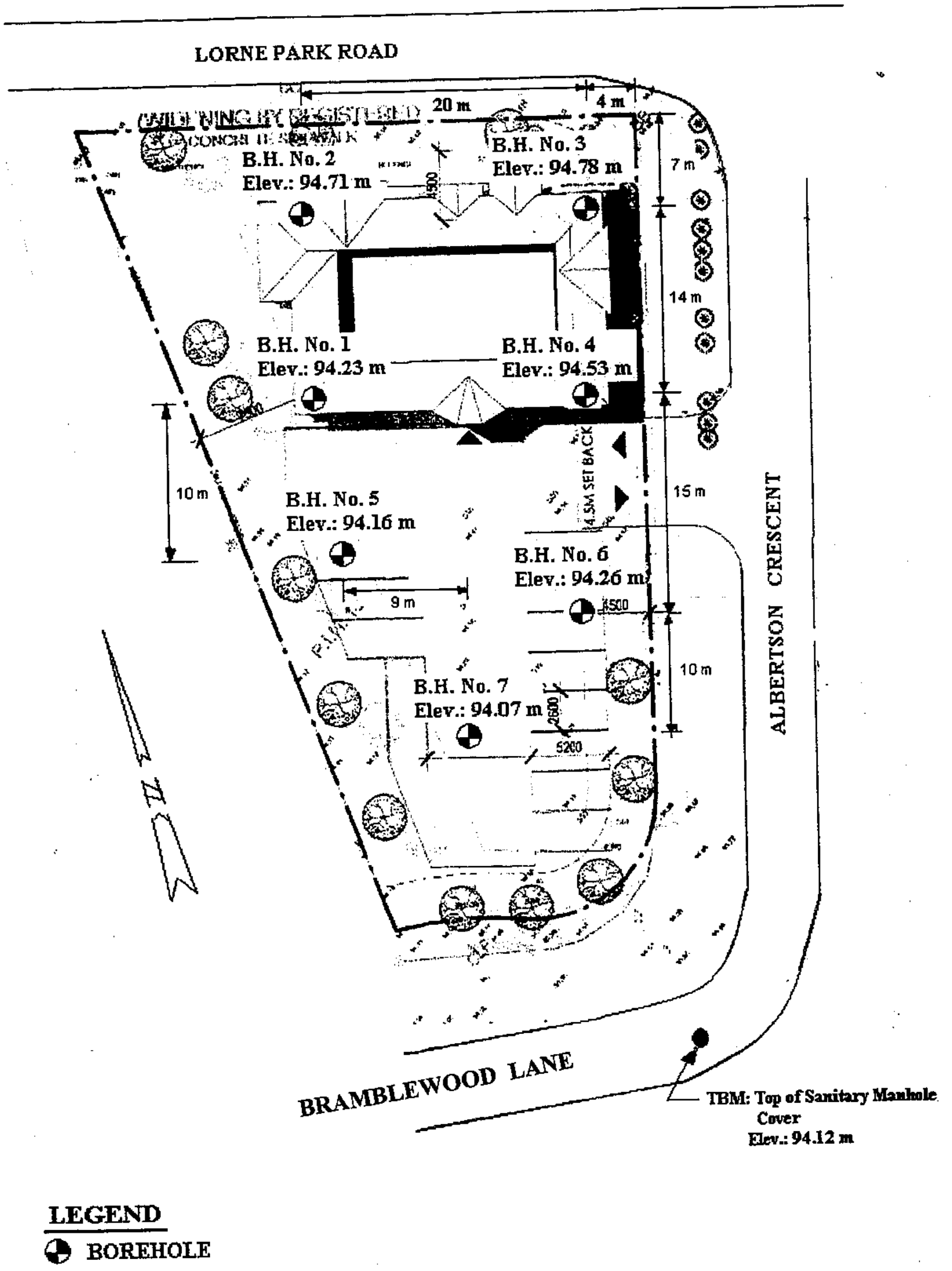
TERMS DESCRIBING RELATIVE DENSITY, BASED ON STANDARD PENETRATION TEST N-VALUE FOR COARSE GRAINED SOILS (major portion retained on No.200 sieve).

DESCRIPTIVE TERM	"N"-VALUE (blows/0.3m)	RELATIVE DENSITY (%)
Very Loose	< 4	< 15
Loose	4 to 10	15 to 35
Compact or Medium	10 to 30	35 to 65
Dense	30 to 50	65 to 85
Very Dense	> 50	> 85

TERMS DESCRIBING CONSISTENCY, BASED ON STANDARD PENETRATION TEST N-VALUE, FOR FINE GRAINED SOILS (major portion passing No. 200 sieve)

DESCRIPTIVE TERM	UNCONFINED COMPRESSIVE STRENGTH (kPa)	"N"-VALUE (blows/0.3m)
Very Soft	< 25	< 2
Soft	25 to 50	2 to 4
Firm	50 to 100	4 to 8
Stiff	100 to 200	8 to 15
Very Stiff	200 to 400	15 to 30
Hard	> 400	> 30

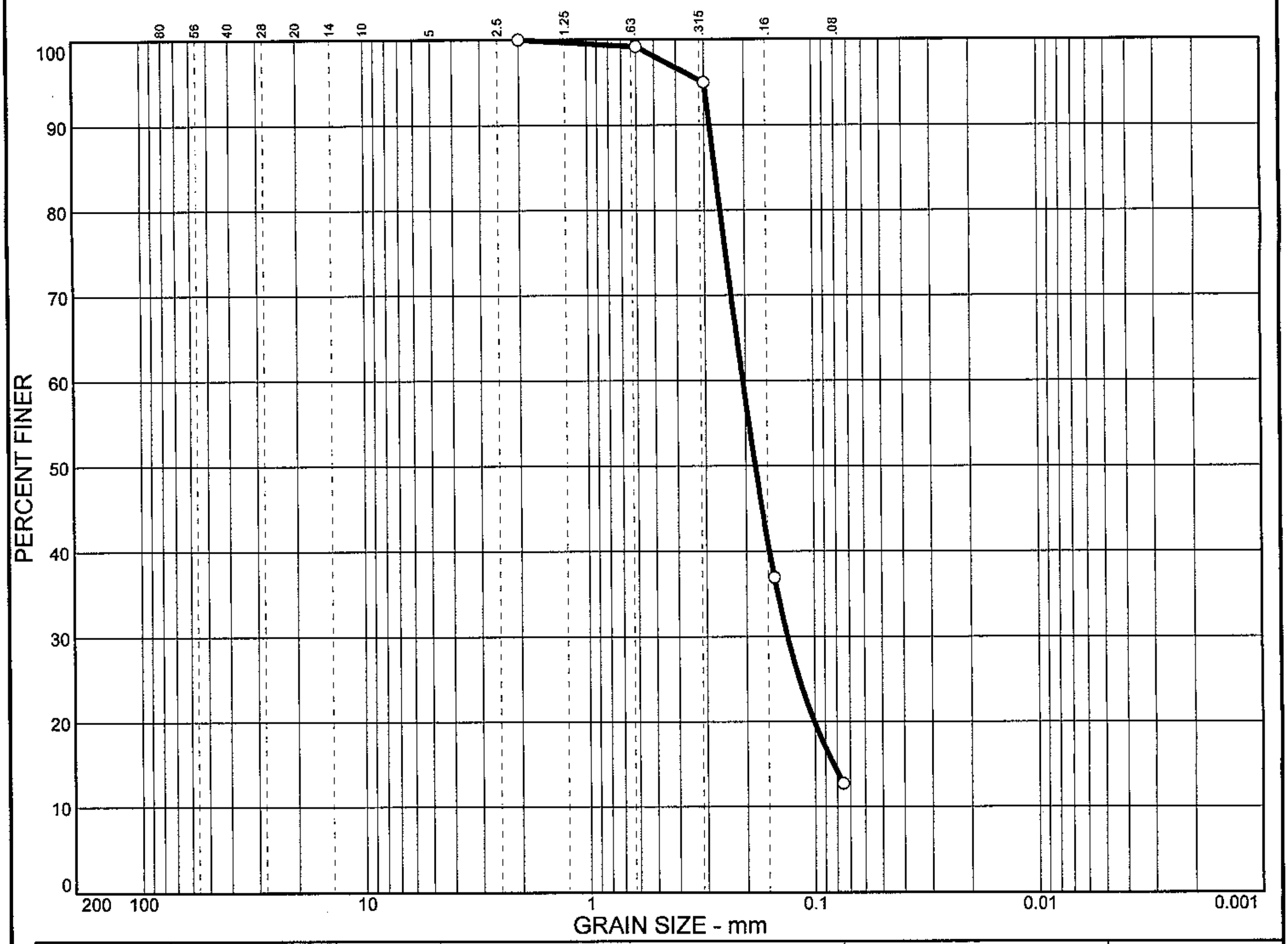
Enclosure No. 9
Report No.: 2011 - 22768



PLOT PLAN SHOWING THE BOREHOLE LOCATIONS FOR THE PROPOSED OFFICE BUILDING AT 1110 LORNE PARK ROAD, CITY OF MISSISSAUGA, ONTARIO.

(NOT TO SCALE)

GRAIN SIZE DISTRIBUTION TEST REPORT



	% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
○	0.0	0.0	87.3	12.7	

☒	LL	PL	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○			0.270	0.205	0.182	0.132	0.0829			

MATERIAL DESCRIPTION	USCS	AASHTO
○ On-site material		

Project No. SP-3179 **Client:** Jacan Construction Ltd.

Project: Proposed Office Building at 1110 Lorne Park Road, City of Mississauga, Ontario

○ **Location:** B.H. No. 5 (Depth:0.6 m - 1.8 m), B.H. Nos. 6 & 7: from below topsoil

Remarks:

○ Sampled by: Ehtesham
On January 21, 2011

Report No.: 2011-22768

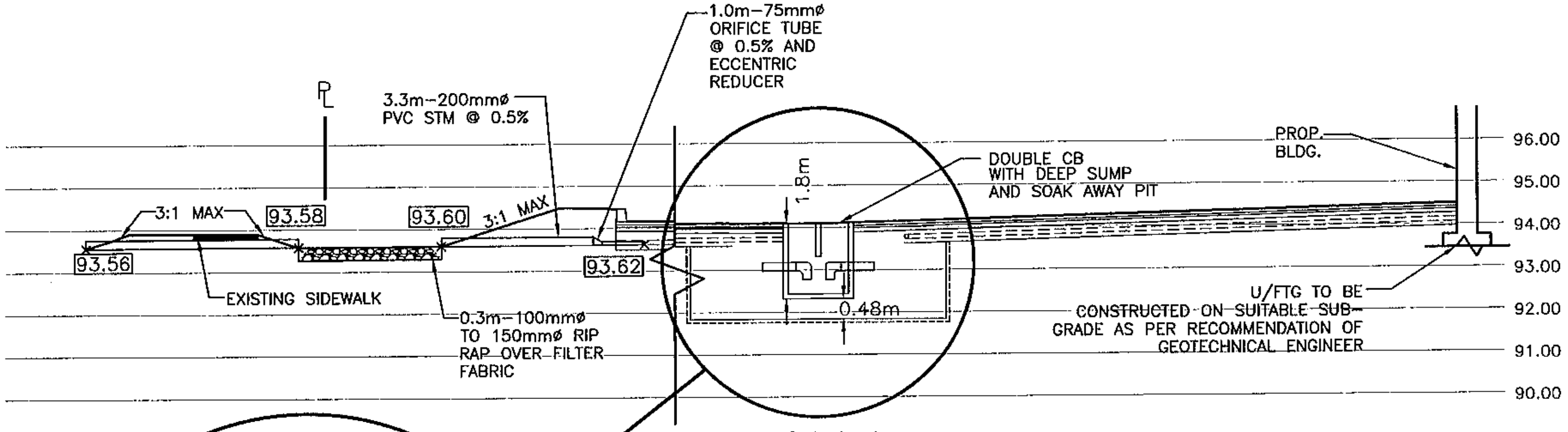
GRAIN SIZE DISTRIBUTION TEST REPORT

SOIL PROBE LTD.

Enclosure No. 10

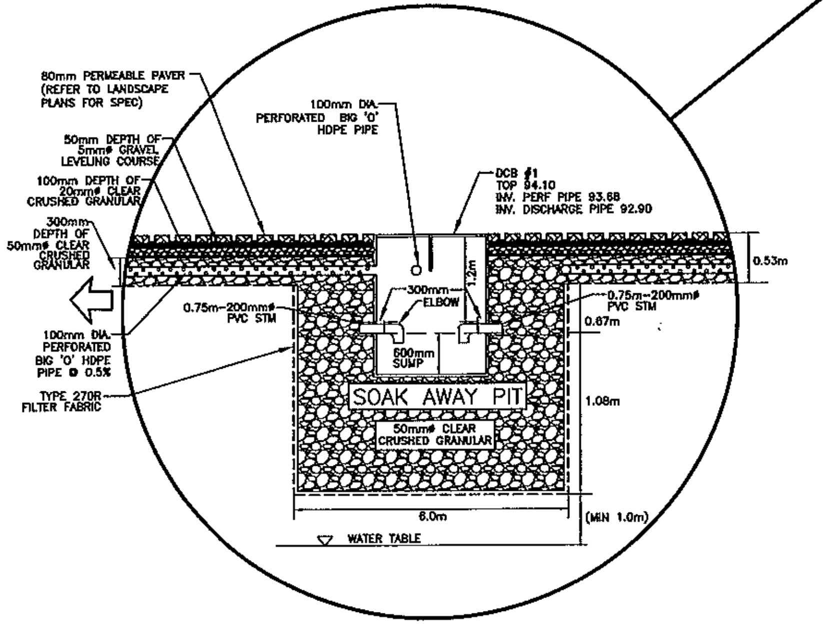
APPENDIX A

PROPOSED PERMEABLE PAVEMENT DESIGN
(PREPARED BY EMC GROUP LIMITED)



SECTION A–A

1:100



PERMEABLE PAVEMENT AND SOAK AWAY PIT DETAIL

N.T.S.

EMC GROUP LIMITED
Engineers Planners Project Managers
Vaughan Ontario (905) 738-3939

TITLE:
PROPOSED 2-STOREY OFFICE BLDG.
1110 LORNE PARK ROAD
PERMEABLE PAVEMENT DETAIL

DESIGN: S.P.	CHECK: M.A.Z.	DATE: FEB. 2011
SCALE: 1 : 100 METRIC		DWG. No. 208147–SK2

APPENDIX E

ENGINEERING DRAWINGS