S. LLEWELLYN & ASSOCIATES LIMITED
CONSULTING ENGINEERS

Functional Servicing & Stormwater Management Report

1110 LORNE PARK ROAD

CITY OF MISSISSAUGA

JACON CONSTRUCTION LIMITED

JANUARY 2020

SLA File: 18091

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TABLE OF CONTENTS

Page

TABLES

FIGURES

APPENDICES

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 driveways, concrete walkways and landscaped areas.

This report will provide detailed information of the proposed servicing scheme for this development has been specifically to address issues raised by the City of Mississauga with respect to the storm drainage outlet for the site. 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)

Ref. 6: Geotechnical Investigation for the Proposed Office Building at 1110 Lorne Park Road, City of Mississauga, Ontario (Soil Probe Ltd., February 9, 2011)

Figure 1.0 – Location Plan

2.0 STORMWATER MANAGEMENT

The following stormwater management (SWM) criteria is required by the City of Mississauga:

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 ditches and then drains easterly towards Albertson Crescent. An existing 300mm diameter culvert crossing is located under Albertson Crescent that directs flow to the existing ditch on the east side of road that then drains southerly.

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	Area (ha)	Intensity (mm/hr) ^{A}	Discharge (m^3/s)^{A}
2-Yr Event	0.14	75.359	0.007
100-Yr Event	0.14	176.312	0.017

^{A} based on time-of-concentration = 10 min.

2.2 PROPOSED CONDITIONS

It is proposed to develop the site by constructing a 2-storey 7 unit townhouse building with associated driveways, 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. 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	65%	0.69
202	Infiltrated on-site	0.12	53%	0.60

The original servicing design for this developed included a storm sewer outlet to the existing 250 mm diameter storm sewer on Lorne Park Road along with on-site quantity control in the form of underground storage. Upon review of the original submission, the City of Mississauga indicated that there was no available capacity in the Lorne Park Road storm sewer for any additional flow. As such, two other alternative were explored for a storm outlet for the development. The first alternative involved the extension of the existing storm located on Bramblewood Lane which is located approximately 60 m southwest of the development. In addition to the cost of extending municipal storm sewers, a capacity review of the existing system indicated that downstream sections of existing storm sewer within the public right-of-way and side yards of private residences were already operating over-capacity. This option was rejected due to the cost and disruption to existing residents in the area. The second potential outlet reviewed was the ditch on the east side of Albertson Crescent which is better defined and deeper than the ditch on the west side. However, upon review it was determined that it was too shallow to provide an outlet invert that would allow for sufficient cover on the road crossing.

A review of the geotechnical information (ref. 6) shows that starting at approximately 2m below existing grade, particularly along the west side of the site (boreholes 1, 2 and 5), the site is underlain with fine sand with traces of silt. Since provision for a typical storm sewer outlet pipe is problematic for this development and the geotechnical conditions are favourable, it is proposed to provide full capture and on-site infiltration of all storm events up to and including the 100-year storm from runoff generated by Catchment 202.

The City of Mississauga 100-year (4 hour) storm event generates approximately 79.4 mm of precipitation (see SWMHYMO input/output info in Appendix B). Catchment 202, which includes the entire roof area and back and side lawn area has an imperious coverage of 53%. Assuming a conservative (generating higher runoff) Curve Number (CN) of 75 and initial abstraction values or 0.5 mm and 1.0 mm for the impervious and pervious surfaces respectively, Catchment 202 generates a runoff volume (RV) of 58.514 mm (see SWMHYMO output in Appendix B).

Catchment 202 has drainage area of 0.12 ha. The volumes associated with this area based on the information presented above as is follows:

- Total volume of rainfall on catchment = 0.12 ha x 79.4 mm = 96 m^3
- Runoff volume generated on catchment = 0.12 ha x 58.514 mm = 71 m^3

Based on the surface characteristics, the runoff volume to be captured is 71 m^3. However, to provide a conservative design, the average between the rainfall and runoff volume, or 83 m^3, is the target underground storage volume that will be used in the design.

The geotechnical report provided a grain size distribution curve for the on-site sandy material from a number of boreholes. Hazen developed a relationship for approximating the hydraulic conductivity of sand based on the grain size distribution given by:

$$k = 10^{-2} \times D_{10}^{2}$$ (source: Soil Mechanics, 4th Ed. R.F. Craig)

where: k = hydraulic conductivity (m/s)
D_{10} = effective size at 10% passing (mm)

Based on the grain size curve provided in the report, the D_{10} size, with a slight extrapolation of the curve is approximately 0.07 mm. Using the Hazen equation, the hydraulic conductivity is approximated at 4.9×10^{-5} m/s, which places it in the range of fine sands.

It is proposed to use the StormBrixx stormwater tanks. The bottom of the tank will be located within the fine sand layer and will have a footprint area of approximately 63 m^2. Based the hydraulic conductivity of 4.9×10^{-5} m/s calculated earlier, the exfiltration rate from the tank will be approximately: $63m^2 \times 4.9 \times 10^{-5} m/s = 0.00308\ m^3/s$ (3.08 l/s). Table 2.3 summarizes the underground storage stage-storage-discharge characteristics.

Based on an exfiltration rate of 3.08 l/s, the 85 m^3 of tank storage volume would theoretically draw down in approximately 8 hours. Even with a factor of safety incorporated into the calculations, draw down for a full tank (100-year storm) would be less than 24 hours.

Table 2.4 – Proposed Underground Storage Characteristics

Elevation	Storage Volume (m^3)	Discharge (m^3/s)
92.08 (Bottom of Tank)	0	Exfiltration rate = 0.00308 m^3/s
93.47 (Top of Tank)	85	

The 2-year and 100-year site runoff was calculated for the post-development condition using the Rational Method and is shown in Table 2.5.

Table 2.5 – Proposed Condition Discharge

Storm Event	Intensity (mm/hr) A	Catchment 201 Discharge (m^3/s)	Catchment 202 Discharge (m^3/s)	Total Site Discharge (m^3/s)
2-Yr	75.359	0.003	0 (infiltrated on-site)	0.003
100-Yr	176.312	0.007	0 (infiltrated on-site)	0.007
A based on time-of-concentration = 10 min.				

This analysis determined the following:

- The 85 m^3 of underground storage will be able to store and infiltrate up to and including the 100-year storm event within a 24 hour period.

- Since runoff from Catchment 202 will be retained and infiltrated on-site, the proposed conditions site discharge to the off-site storm sewer or surface conveyance systems will be less than the existing conditions.

Storm Sewer Servicing
Based on the information presented above, the proposed development will not have a piped storm sewer outlet from the site. All storm sewers will be internal and further information with respect to their functioning is provided below

- To capture the entire roof area and direct it to the infiltration gallery, it is proposed to connect the front downspouts to a 150 mm diameter storm sewer system that will carry the flows around the building. One-half of the roof area is approximately 200 m^2 in area and will generate a 100-year flow of 10 l/s. A 150mm diameter storm sewer at 1.0% will convey 15 l/s.

- Overflow – If the tank become full, the system will overflow first at CBMH 4 at the southeast corner of the site which has a top-of-grate elevation of 93.80 m. Minor ponding (0.20 m or less) will occur at CBMH 4 and CBMH 1 at which point spilling will occur at approximately elevation 94.0 m at the Bramblewood Lane right-of-way near the driveway for the end unit. Surface flow within the right-of-way will flow east and be conveyed through the existing 300 mm diameter culvert cross Albertson Crescent or flow overland along Albertson Crescent.

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). The Stormbrixx tanks will provide 85 m^3 of storage for infiltration.

Water Quality Control
The proposed development is required to achieve an “Enhanced” (80% TSS removal) level of water quality protection. As indicated previously, with exception of the front yard and driveway area (Catchment 201), runoff from the remainder of the site (Catchment 202) will remain within the property and be infiltrated. Since the area consists of rooftop and landscaping which is considered to be relatively clean water, no specific end-of-pipe water quality control devices will be incorporated into the design. However, the StormBrixx tank will included a geotextile wrapped isolator row to facilitate filtering and debris capture within a specific location for ease of future maintenance.

Low Impact Development
The zero discharge infiltration approach to the site stormwater management is consistent with a LID design approach. No other LID measures will be implement on-site.

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 individual 25mmø copper water services from the existing 200mmø adjacent to the site on Bramblewood Lane. 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;
- ACO StormBrixx tanks be installed to infiltration a large portion of the site and eliminate the need for the piped storm sewer outlet;
- 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;

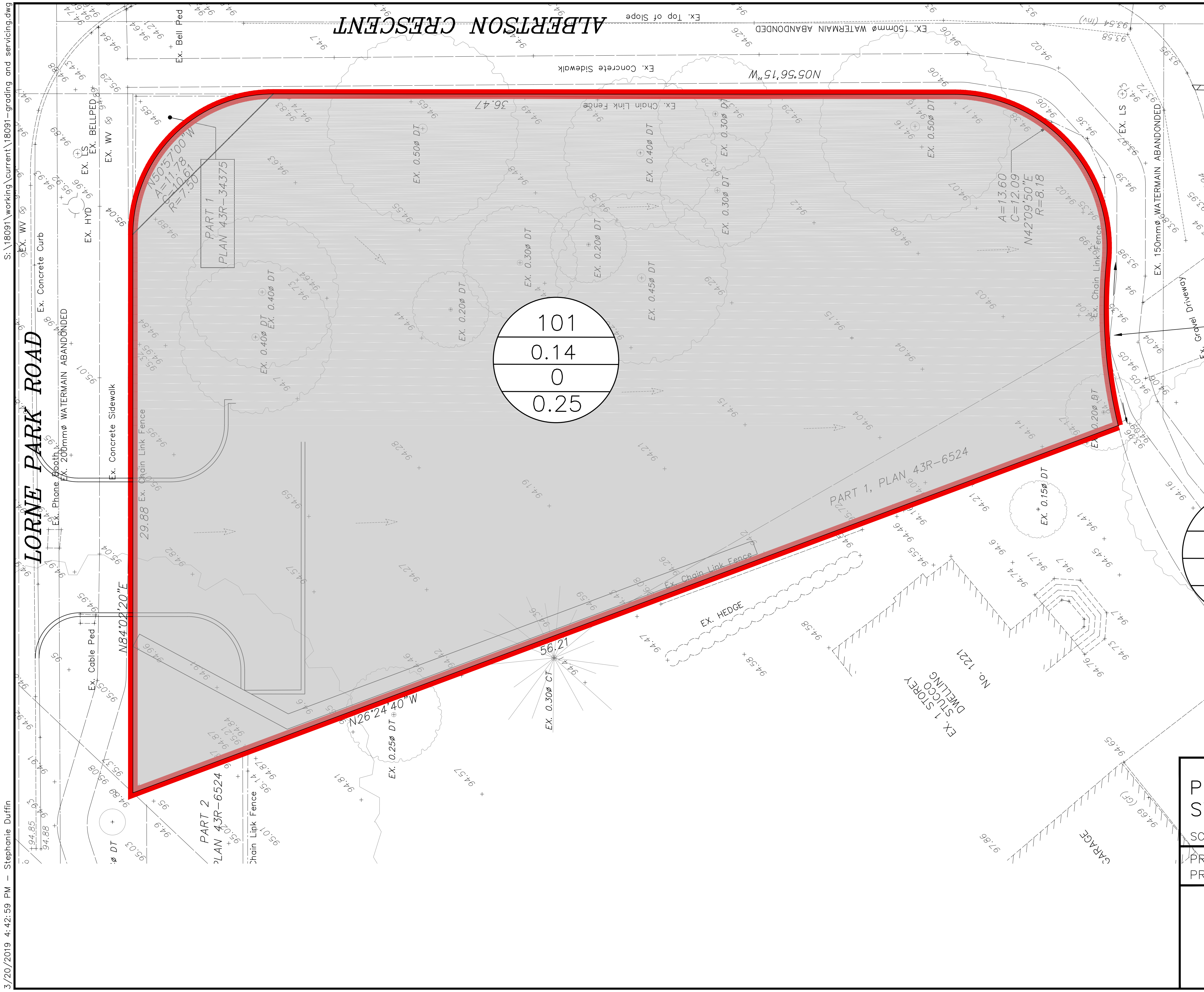
We trust the information enclosed herein is satisfactory. Should you have any questions please do not hesitate to contact our office.

Prepared by:
S. LLEWELLYN & ASSOCIATES LIMITED

John Oreskovic, P.Eng.

APPENDIX A

FIGURES



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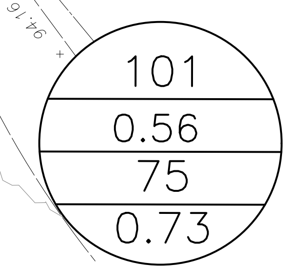

DRAINAGE AREA I.D.
DRAINAGE AREA (ha)
PERCENT IMPERVIOUS
RUNOFF COEFFICENT

PERVIOUS AREA

PRE-DEVELOPMENT STORM DRAINAGE AREA PLAN

SCALE: 1:200

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

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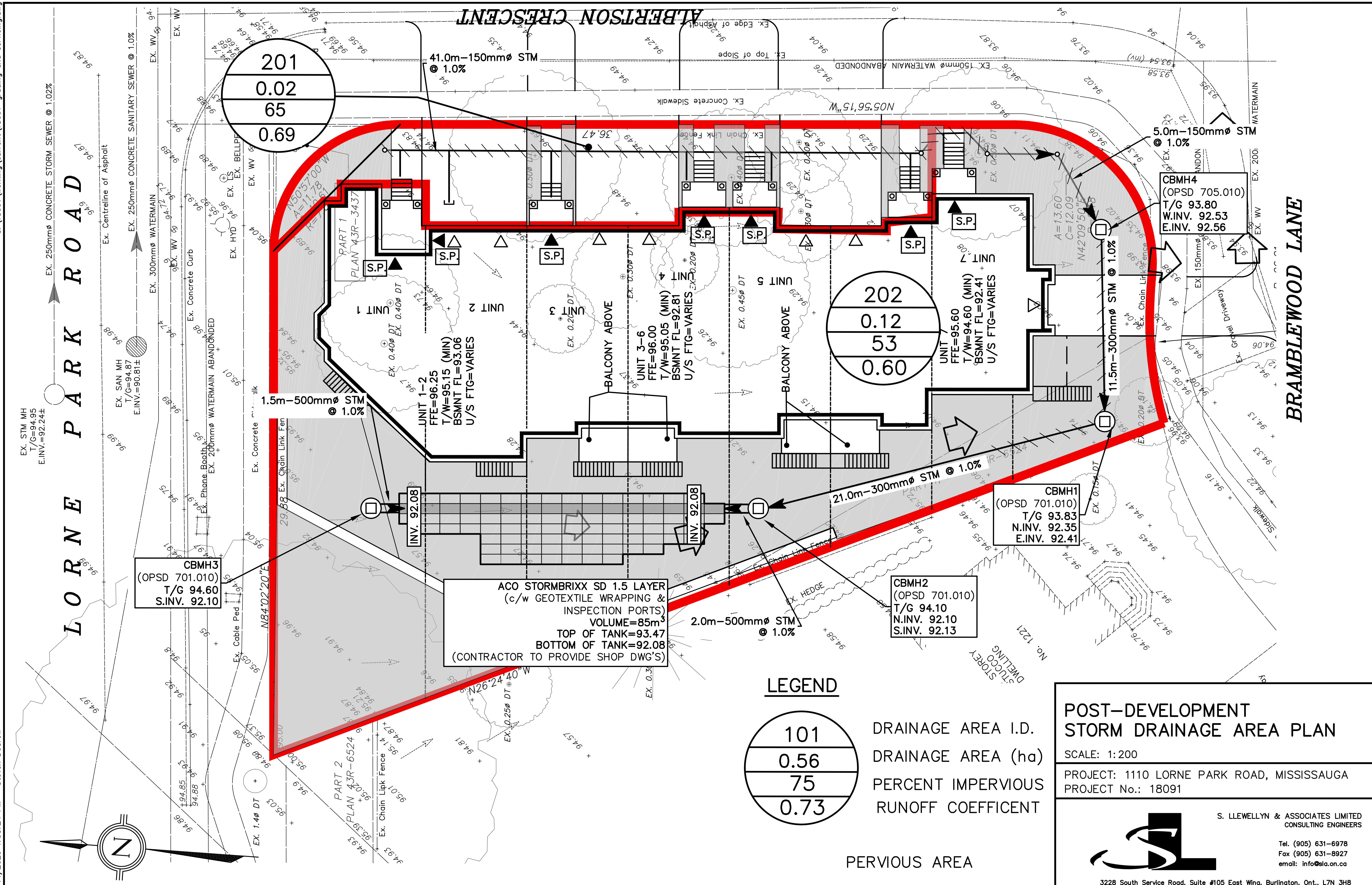
LORNE PARK ROAD
ALBERTSON CRESCENT
BRAMBLEWOOD LANE
201
0.02
65
0.69
202
0.12
53
0.60
41.0m–150mmø STM @ 1.0%
5.0m–150mmø STM @ 1.0%
11.5m–300mmø STM @ 1.0%
21.0m–300mmø STM @ 1.0%
1.5m–500mmø STM @ 1.0%
2.0m–500mmø STM @ 1.0%
CBMH4 (OPSD 705.010) T/G 93.80 W.INV. 92.53 E.INV. 92.56
CBMH1 (OPSD 701.010) T/G 93.83 N.INV. 92.35 E.INV. 92.41
CBMH2 (OPSD 701.010) T/G 94.10 N.INV. 92.10 S.INV. 92.13
CBMH3 (OPSD 701.010) T/G 94.60 S.INV. 92.10
ACO STORMBRIXX SD 1.5 LAYER (c/w GEOTEXTILE WRAPPING & INSPECTION PORTS) VOLUME=85m³ TOP OF TANK=93.47 BOTTOM OF TANK=92.08 (CONTRACTOR TO PROVIDE SHOP DWG'S)
INV. 92.08
UNIT 1
UNIT 2
UNIT 3
UNIT 4
UNIT 5
UNIT 7
UNIT 1–2 FFE=96.25 T/W=95.15 (MIN) BSMNT FL=93.06 U/S FTG=VARIES
UNIT 3–6 FFE=96.00 T/W=95.05 (MIN) BSMNT FL=92.81 U/S FTG=VARIES
UNIT 7 FFE=95.60 T/W=94.60 (MIN) BSMNT FL=92.41 U/S FTG=VARIES
BALCONY ABOVE
S.P.
EX. 250mmø CONCRETE STORM SEWER @ 1.02%
EX. 250mmø CONCRETE SANITARY SEWER @ 1.0%
EX. 300mmø WATERMAIN
EX. 200mmø WATERMAIN ABANDONDED
EX. 150mmø WATERMAIN ABANDONDED
Ex. Centreline of Asphalt
Ex. Concrete Curb
Ex. Concrete Sidewalk
Ex. Edge of Asphalt
Ex. Top of Slope
Ex. Chain Link Fence
EX. HEDGE
EX. STM MH T/G=94.95 E.INV.=92.24±
EX. SAN MH T/G=94.87 E.INV.=90.81±
PART 1 PLAN 43R-3437
PART 2 PLAN 43R-6524
No. 1221 2 STOREY STUCCO DWELLING
N05°56'15"W
N84°02'20"E
N26°24'40"W
36.47
29.88
LEGEND
101
0.56
75
0.73
DRAINAGE AREA I.D.
DRAINAGE AREA (ha)
PERCENT IMPERVIOUS
RUNOFF COEFFICENT
PERVIOUS AREA
POST–DEVELOPMENT STORM DRAINAGE AREA PLAN
SCALE: 1:200
PROJECT: 1110 LORNE PARK ROAD, MISSISSAUGA
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S:\18091\working\current\18091-grading and servicing.dwg
1/14/2020 1:55:24 PM – Steven DeGrow

APPENDIX B

STORMWATER MANAGEMENT INFORMATION

```
2     Metric units
*#*******************************************************************************
*#  Project Name: [1110 LORNE PARK]    Project Number: [18091]
*#  Date         : NOV 2019
*#  Company      : S. Llewellyn & Associates
*#  License #    :  3053466
*#*******************************************************************************
*#
*# City of Mississauga 100-yr IDF storm event analyzed to determined
*# runoff volume (RV) from catchment 202 directed to infiltration gallery
*#
*#-----------------|-----------------------------------------------------------|
START              TZERO=[0.0],  METOUT=[2],  NSTORM=[0],  NRUN=[0]
*%                  [  ] <--storm filename, one per line for NSTORM time
*%-----------------|-----------------------------------------------------------|
CHICAGO STORM      IUNITS=[2],  TD=[4](hrs), TPRAT=[0.38],  CSDT=[-10](min),
                   ICASEcs=[1],
                    A=[1450],  B=[4.9], and C=[0.780],
*%-----------------|-----------------------------------------------------------|
CALIB STANDHYD     ID= 1  NHYD=["202"], DT=[1](min), AREA=[0.12](ha),
                   XIMP=[.53], TIMP=[.53], DWF=[0](cms), LOSS=[2],
                   SCS curve number CN=[75],
                   Pervious   surfaces: IAper=[4.0](mm), SLPP=[2.0](%),
                                        LGP=[20](m), MNP=[0.025], SCP=[0](min),
                   Impervious surfaces: IAimp=[.5](mm), SLPI=[2.0](%),
                                        LGI=[20](m), MNI=[.013], SCI=[0](min),
                   RAINFALL=[ ,  ,  ,  , ](mm/hr) ,  END=-1
*%-----------------|-----------------------------------------------------------|
FINISH
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   SSSSS  W W W  M M M  HHHHH    Y    M M M  O   O  ## 9   9  9   9  Ver  4.05
       S   W W   M   M  H   H    Y    M   M  O   O      9999   9999  Sept 2011
   SSSSS   W W   M   M  H   H    Y    M   M   OOO          9      9  =========
                                                       9   9  9   9  # 3902680
        StormWater Management HYdrologic Model          999    999   =========

 ******************************************************************************
 ***************************** SWMHYMO Ver/4.05  *****************************
 *********  A single event and continuous hydrologic simulation model  *********
 *********     based on the principles of HYMO and its successors      *********
 *********                OTTHYMO-83 and OTTHYMO-89.                   *********
 ******************************************************************************
 ********* Distributed by:  J.F. Sabourin and Associates Inc.          *********
 *********                  Ottawa,  Ontario: (613) 836-3884           *********
 *********                  Gatineau, Quebec: (819) 243-6858           *********
 *********                  E-Mail: swmhymo@jfsa.Com                   *********
 ******************************************************************************

 ++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++
 +++++++++ Licensed user: S. Llewellyn & Associates Ltd               +++++++++
 +++++++++                 in any City          SERIAL#:3902680       +++++++++
 ++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++

 ******************************************************************************
 *********           ++++++ PROGRAM ARRAY DIMENSIONS ++++++            *********
 *********           Maximum value for ID numbers  :     10            *********
 *********           Max. number of rainfall points: 105408            *********
 *********           Max. number of flow points    : 105408            *********
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 * Output  filename: T:\projects\18091\18091stm.out                           *
 * Summary filename: T:\projects\18091\18091stm.sum                           *
 * User comments:                                                             *
 * 1:_________________________________________________________________________*
 * 2:_________________________________________________________________________*
 * 3:_________________________________________________________________________*
 ******************************************************************************


--------------------------------------------------------------------------------
001:0001------------------------------------------------------------------------
*#******************************************************************************
*#  Project Name: [1110 LORNE PARK]    Project Number: [18091]
*#  Date        : NOV 2019
*#  Company     : S. Llewellyn & Associates
*#  License #   :  3053466
*#******************************************************************************
*#
*# City of Mississauga 100-yr IDF storm event analyzed to determined
*# runoff volume (RV) from catchment 202 directed to infiltration gallery
```

```
*#
*#-----------------|--------------------------------------------------------------|
--------------------
| START             |  Project  dir.: T:\projects\18091\
--------------------  Rainfall dir.: T:\projects\18091\
    TZERO =   .00 hrs on        0
    METOUT=   2 (output = METRIC)
    NRUN  = 001
    NSTORM=   0
--------------------------------------------------------------------------------
001:0002------------------------------------------------------------------------
--------------------
| CHICAGO STORM     |    IDF curve parameters: A=1450.000
| Ptotal= 79.40 mm |                           B=   4.900
--------------------                           C=    .780
                        used in:   INTENSITY =  A / (t + B)^C

                        Duration of storm  =  4.00 hrs
                        Storm time step    = 10.00 min
                        Time to peak ratio =    .38

              TIME     RAIN |   TIME     RAIN |   TIME     RAIN |   TIME     RAIN
               hrs    mm/hr |    hrs    mm/hr |    hrs    mm/hr |    hrs    mm/hr
               .17    5.290 |   1.17   19.074 |   2.17   14.280 |   3.17    6.468
               .33    5.930 |   1.33   43.028 |   2.33   11.722 |   3.33    5.972
               .50    6.777 |   1.50  176.312 |   2.50    9.999 |   3.50    5.554
               .67    7.959 |   1.67   51.864 |   2.67    8.754 |   3.67    5.198
               .83    9.734 |   1.83   26.882 |   2.83    7.810 |   3.83    4.889
              1.00   12.738 |   2.00   18.503 |   3.00    7.068 |   4.00    4.619

--------------------------------------------------------------------------------
001:0003------------------------------------------------------------------------
----------------------
| CALIB STANDHYD      |   Area    (ha)=     .12
| 01:202    DT= 1.00 |   Total Imp(%)=   53.00   Dir. Conn.(%)=   53.00
----------------------
                              IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=        .06          .06
     Dep. Storage     (mm)=        .50         4.00
     Average Slope     (%)=       2.00         2.00
     Length            (m)=      20.00        20.00
     Mannings n           =       .013         .025

     Max.eff.Inten.(mm/hr)=     176.31        96.07
                over (min)        1.00         2.00
     Storage Coeff.  (min)=        .63 (ii)    1.82 (ii)
     Unit Hyd. Tpeak (min)=       1.00         2.00
     Unit Hyd. peak  (cms)=       1.35          .59
                                                            *TOTALS*
     PEAK FLOW       (cms)=        .03          .01           .045 (iii)
     TIME TO PEAK    (hrs)=       1.47         1.50          1.500
     RUNOFF VOLUME    (mm)=      78.90        35.52         58.514
     TOTAL RAINFALL   (mm)=      79.40        79.40         79.404
     RUNOFF COEFFICIENT   =        .99          .45           .737

       (i) CN PROCEDURE SELECTED FOR PERVIOUS LOSSES:
           CN* =   75.0    Ia = Dep. Storage  (Above)
      (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
           THAN THE STORAGE COEFFICIENT.
```

Total 100-yr rainfall

100-yr runoff volume depth from catchment

```
     (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

--------------------------------------------------------------------------------
001:0004------------------------------------------------------------------------
      FINISH
--------------------------------------------------------------------------------
********************************************************************************
     WARNINGS / ERRORS / NOTES
     -------------------------
   Simulation ended on 2019-11-25     at 18:34:40
================================================================================
```

APPENDIX C

ACO STORMBRIXX PRODUCT INFORMATION

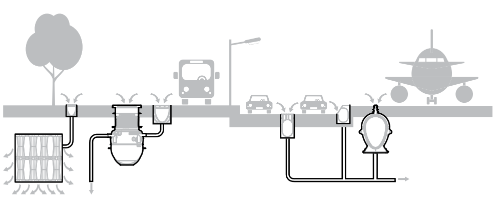
ACO Storm Water Management

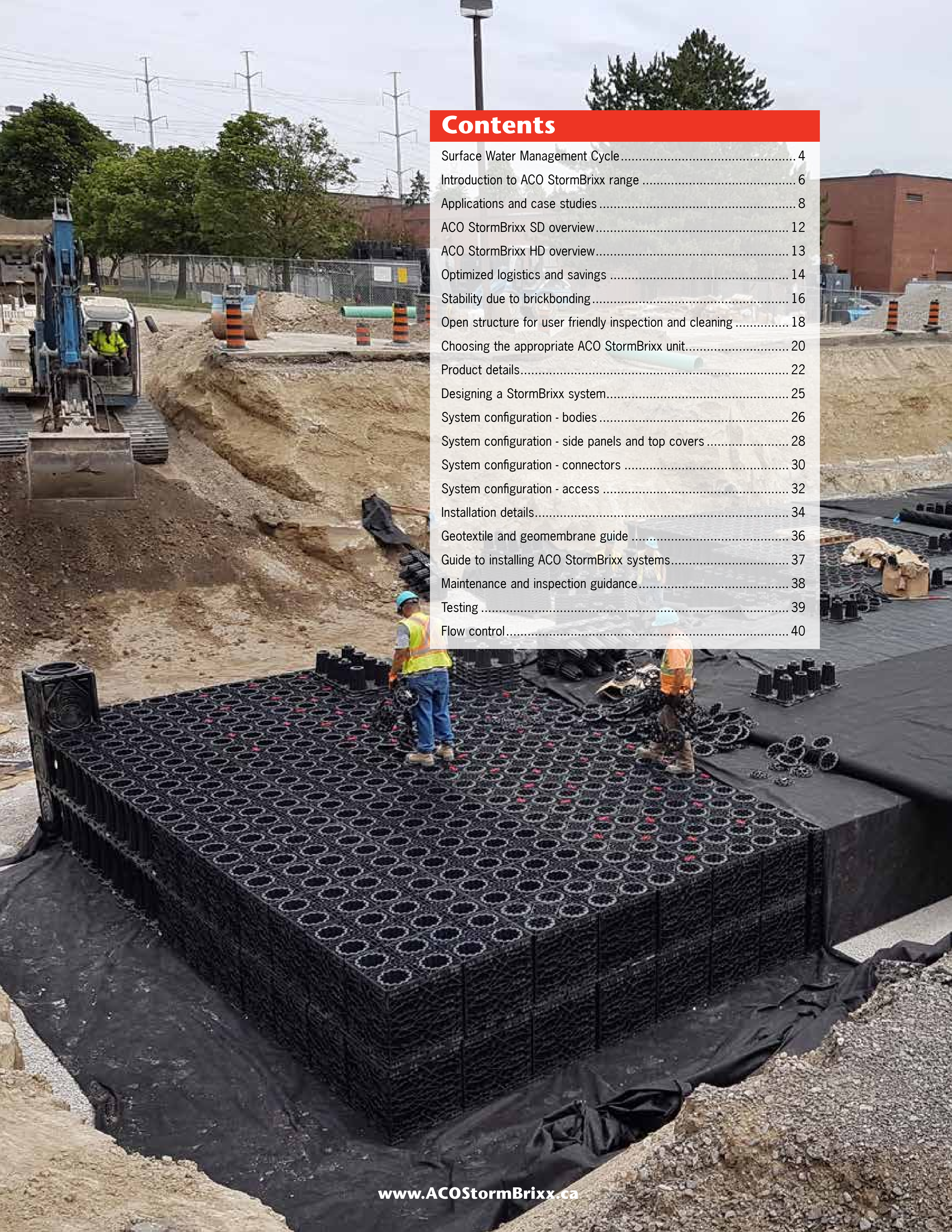
ACO STORMBRIXX BROCHURE

Stormwater retention

Stormwater detention

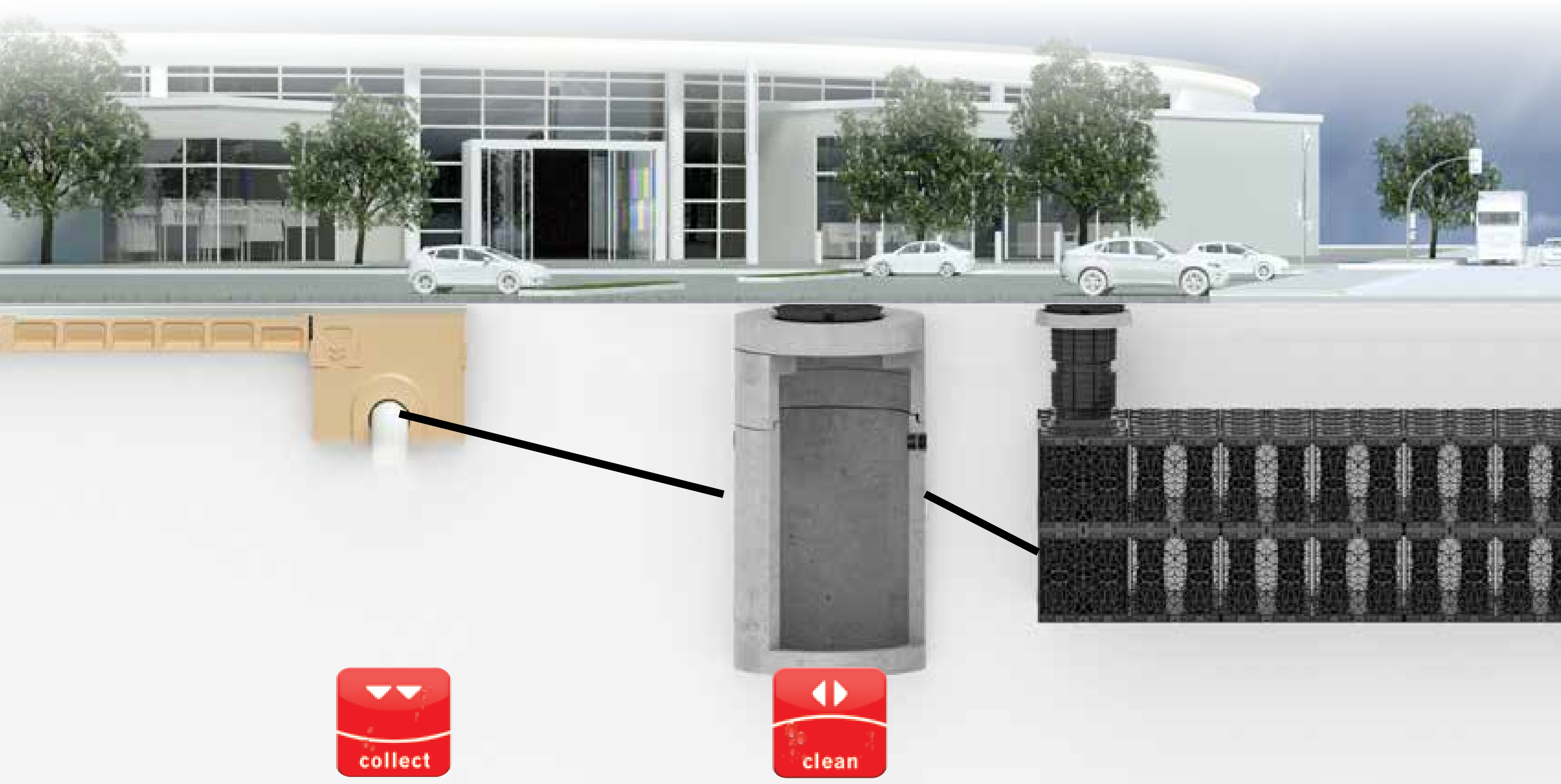
Stormwater infiltration





Contents

The Surface Water Management Cycle



Where surface water management and water protection begins

Surface water management begins with an assessment of the hydrological demands of the project landscape. The rainfall and topography determine the surface water solution devised. ACO provides expertise in both the assessment, product solutions and optimum layouts to collect surface water across the site.

In hard surfaced areas, the extensive range and capacity of ACO trench drainage products offer a high intercept performance along the total length of the trench run, and reduce the occurrence of ponding or run off on the site

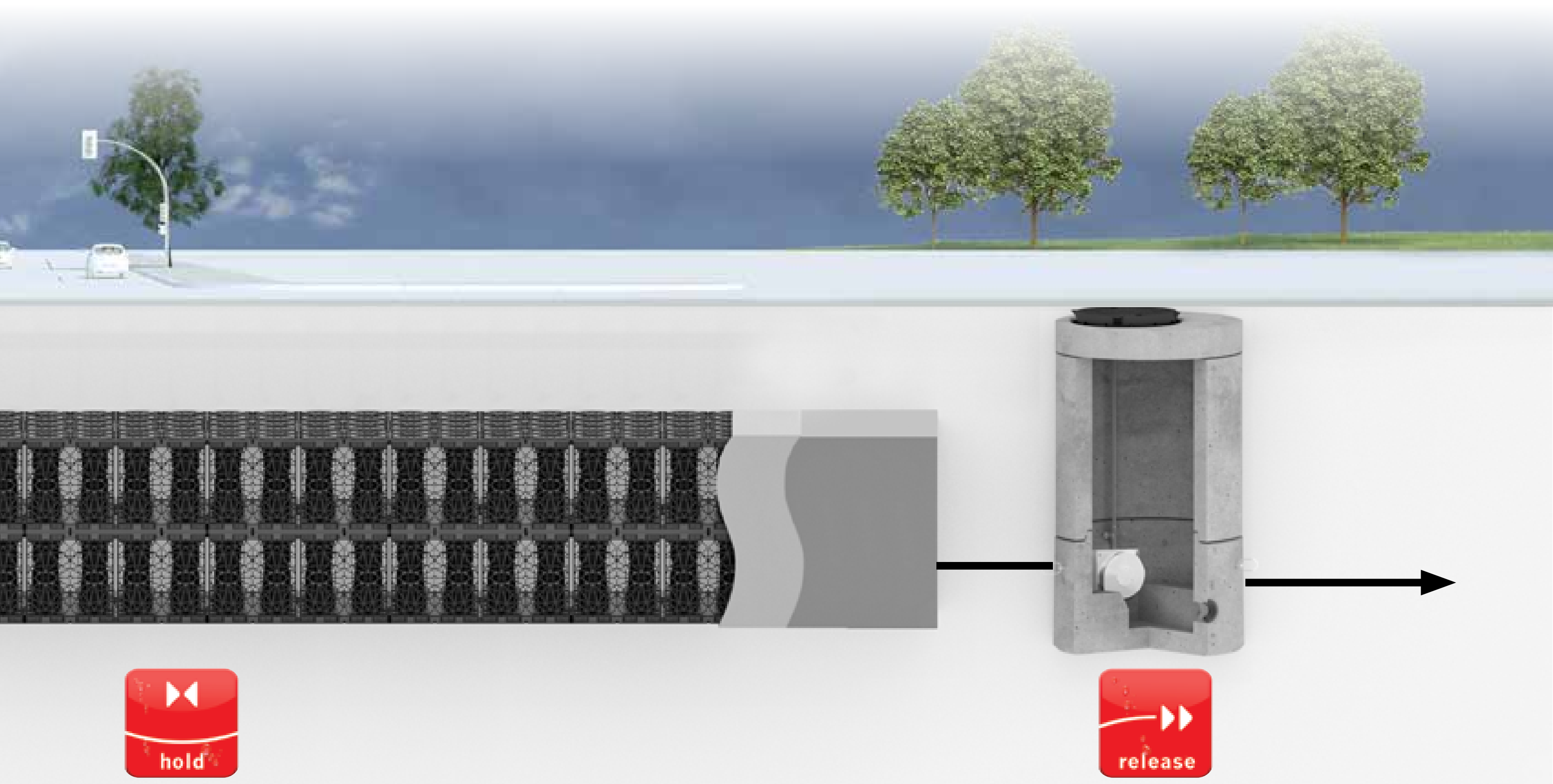
The safety and convenience of people, buildings and traffic is assured and surface water is managed on to its next stage in the water management process.

Achieving the right water quality

Water quality is an important factor when designing a surface water management solution because surface water run-off is at increasing risk of contamination from greater urbanisation and transportation demands on the environment. Policy and planning guidelines require water quality is taken into account to prevent contamination of surface and groundwater and if untreated water is discharged into the natural surroundings it could endanger plant and wildlife, therefore preventative methods should be put in place.

Contaminations come in many forms, such as siltation containing suspended hydrocarbons and heavy metals, tire wear, brake dust, soot and sediments, as well as de-icing products used during winter months.

ACO offer a number of treatment units to deal with water quality including heavy metal and suspended solids separators, and gas/oil interceptors. These can be combined with swales, so clean water can nourish an onsite wildlife area and allow wildlife and biodiversity to flourish.



Reducing surface runoff to a natural level

With increasing urbanisation larger areas of landscape are being covered with impermeable surfacing, and so the risk of flooding increases. The natural water cycle of infiltration, evaporation and evapotranspiration is hindered and solutions such as ACO StormBrixx can be used to store and control surface water runoff rate down to more natural levels. Geocellular system can be used for infiltration and attenuation, as well as Low Impact Development (LID).

ACO StormBrixx can help support the sewer network by providing capacity to meet these high risk flooding scenarios, and its use in LID schemes has allowed it to protect surrounding water networks and inhabited areas through a controlled discharge into the groundwater that mimics natural infiltration.

hold

Control discharge rate to the required level

To meet the struggling capacity of storm sewer networks and natural waterways, water discharge rates are addressed on each site, by either Orifice plate or Vortex flow controllers.

ACO have solutions for both, with units usually sized to match the previous run-off rates, or a greenfield equivalent to ensure that the infrastructure and environment are not put under strain.

release

Introduction to ACO StormBrixx Range

ACO StormBrixx is a unique, and patented, plastic geocellular stormwater management system.Designed for surface water infiltration and storage, its versatility allows it to be used in applications across all construction environments as a standalone solution or as part of an integrated sustainable urban drainage (LID) scheme.

What is ACO StormBrixx?

Plastic geocellular systems are a widely accepted method of creating infiltration and attenuation systems. They have been installed in a variety of applications for a number of years. A drawback of some types of systems is a lack of accessibility for maintenance.

StormBrixx was developed to address accessibility issues and allow maintenance requirements to be completed easily.

Sustainable surface water management is becoming an integral part of most major planning applications. Consideration should be given to management of both quantity and quality of water discharged off-site, along with ongoing maintainability.

ACO StormBrixx addresses the ongoing maintenance requirements by providing true 3D access for inspection and maintenance, while retaining the structural integrity of the installation.

The ACO StormBrixx system

The ACO StormBrixx system consists of a single, recyclable, polypropylene body that can be assembled in a variety of ways to form an open bonded structure.

StormBrixx's unique pillar configuration gives a high void ratio of 95%-97%. This minimizes excavation required to achieve a specified storage capacity, reduces the aggregate needed for backfilling, and improves the flow characteristics of runoff through the tank.

Side panels are added to the perimeter of the system for lateral support, and top covers are added to ensure consistent vertical support for cover fill material.

ACO StormBrixx benefits from a patented cell brick and cross bonding feature, which provides unparalleled stability in the construction of the tank. Where brickbonding is not used, or for multilayered tank structures, connectors are available to support the integrity of the structure.

Additional accessories available include inspection point and pipe connectors, as well as a range of chambers including man access for full inspection and maintenance.

StormBrixx can be configured to minimize silt accumulation and has the added feature of a low flow and drain down facility ensuring that the system can be properly maintained throughout its life.

Why choose ACO StormBrixx?

Structural Integrity

The StormBrixx system has been independently tested to certify structural integrity and the long term life expectancy of the system.

The patented brickbonding and cross bonding feature provides a strong, long life installation and helps improve the construction speed of the tank.

Access and maintenance

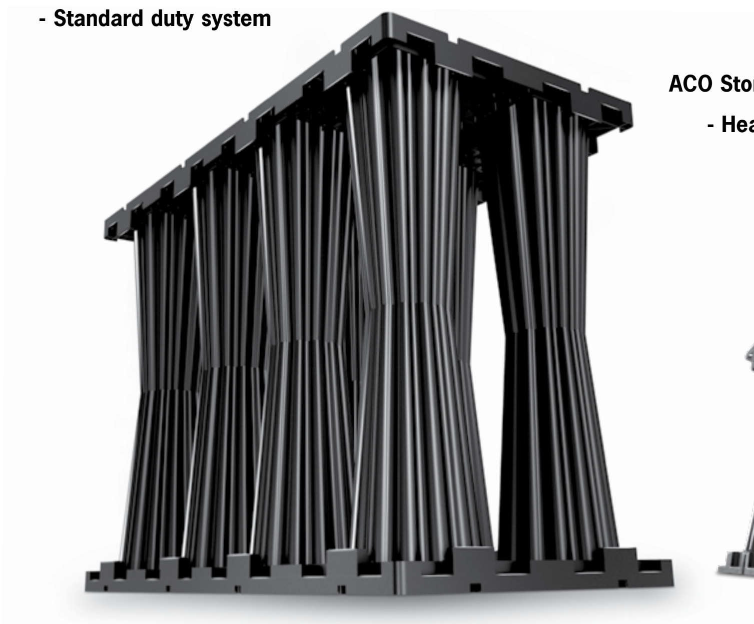
StormBrixx addresses the fundamental requirement of access and maintenance for LID Approval and water utility companies. The open cell structure permits completely free access for CCTV and jetting equipment which allows the whole system, including all the extremities, to be inspected and maintained from a few access points.

Simplified handling and logistics

StormBrixx simplifies delivery, site logistics and installation as a result of its stackable design. Each single injection moulded body nestles, optimizing logistical and installation cost significantly, thus helping to reduce the carbon footprint of the system.

ACO StormBrixx SD

- Standard duty system



ACO StormBrixx HD

- Heavy duty system

System benefits

- Brick bonded and cross bonding stacking for optimal stability
- Low flow, draindown and silt management features
- Man access and 3D inspection access to tank interior
- Environmentally efficient solution, minimizing carbon emissions in manufacture, transportation and on-site assembly
- High void ratio minimizes excavation volume
- Fully certified performance
- Manufactured from recyclable polypropylene
- Suitable for all industrial, commercial and residential applications including highways

ACO StormBrixx Applications and Case Studies

ACO StormBrixx SD

The StormBrixx SD (Standard Duty) range broadens the scope of installations to a more varied retention and infiltration requirements for general purpose use. System benefits such as stackability can reduce congestion on site.

Typical applications

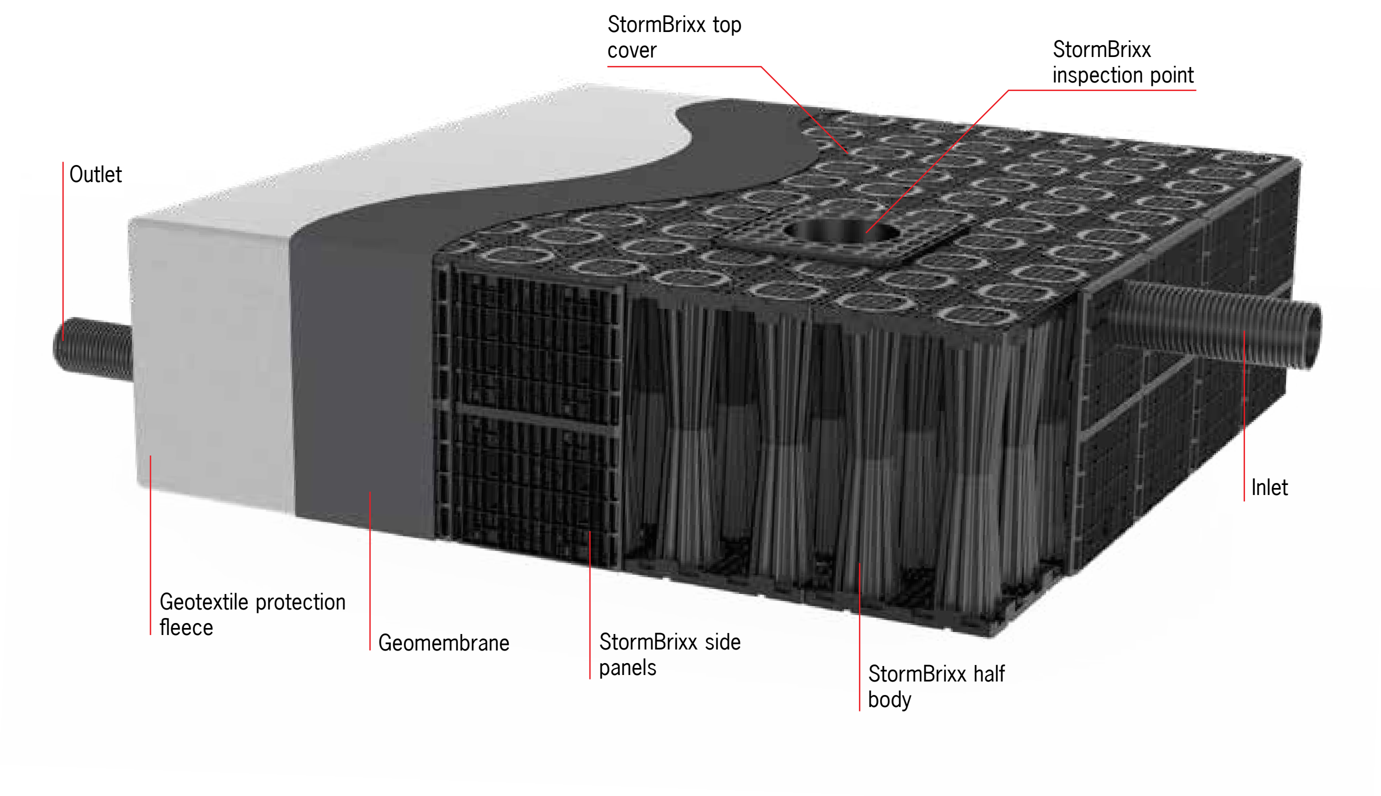
Parking lots

Educational installations

Housing developments

LID applications

ACO StormBrixx SD arrangement



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

Building / Location	# of Storeys	**Total GFA (m^2)**	Type of Construction	(1) ***Fire Flow "F"*** (l/min)	(l/s)	(2) ***Occupancy*** %	Adjustment (l/min)	**Adjusted Fire Flow (l/min)**	(3) ***Sprinkler*** %	Adjustment (l/min)	(4) ***Exposure*** %	Adjustment (l/min)	**Final Adjusted Fire Flow (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%.

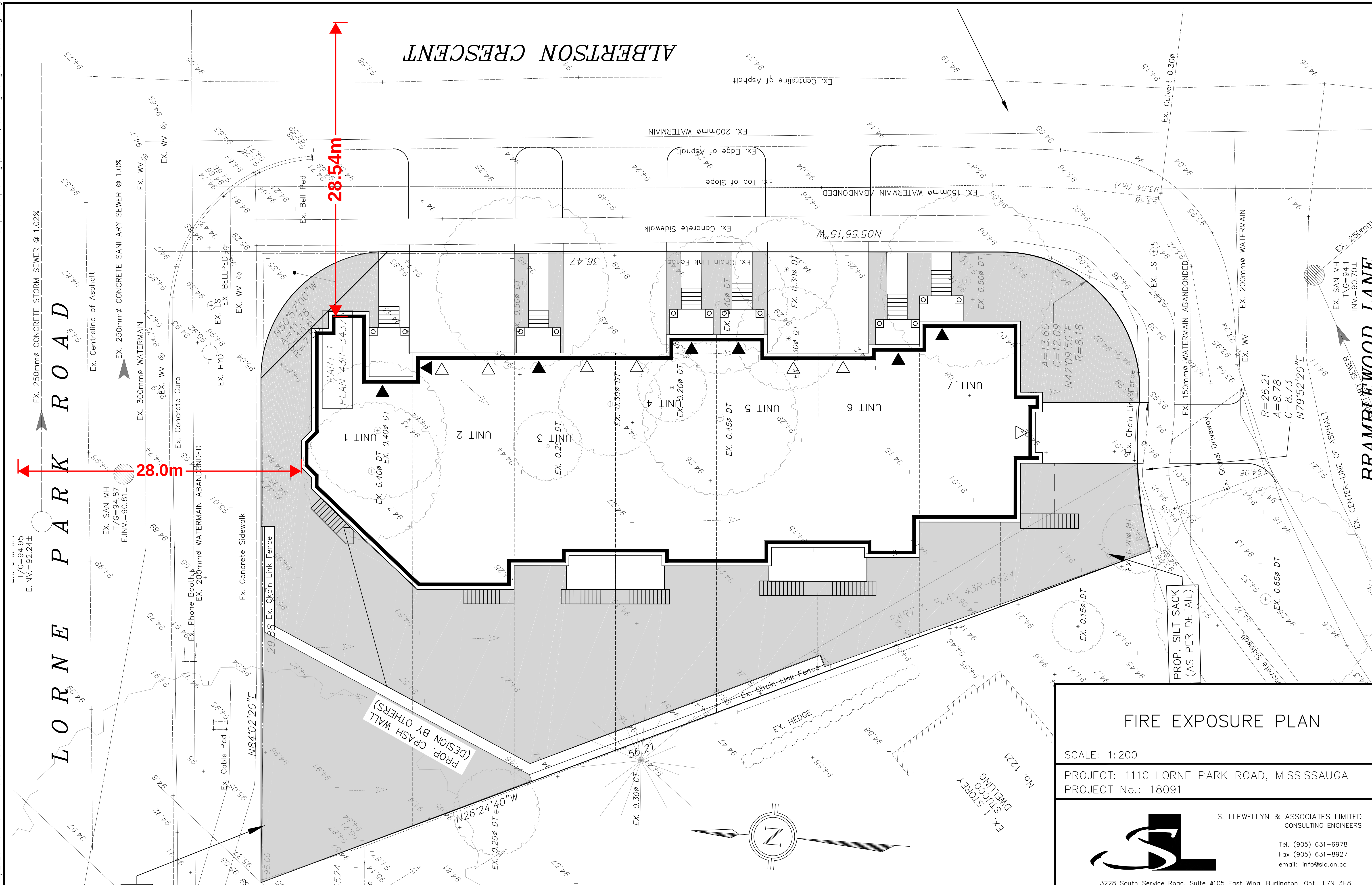
If the sprinkler system is fully supervised (ie. annunciator panel that alerts the Fire Dept., such as a school), then an additional 10% can be taken. Maximum credit = 50%.

(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***


ALBERTSON CRESCENT
LORNE PARK ROAD
BRAMBLEWOOD LANE
28.54m
28.0m
UNIT 1
UNIT 2
UNIT 3
UNIT 4
UNIT 5
UNIT 6
UNIT 7
EX. 250mmø CONCRETE STORM SEWER @ 1.02%
EX. 250mmø CONCRETE SANITARY SEWER @ 1.0%
EX. 300mmø WATERMAIN
EX. 200mmø WATERMAIN ABANDONDED
EX. 200mmø WATERMAIN
EX. 150mmø WATERMAIN ABANDONDED
Ex. Concrete Sidewalk
Ex. Chain Link Fence
PROP. CRASH WALL (DESIGN BY OTHERS)
PROP. SILT SACK (AS PER DETAIL)
PART 1, PLAN 43R-6724
EX. 1 STOREY STUCCO DWELLING
No. 1221
EX. HEDGE
N26°24'40"W
N84°02'20"E
N05°56'15"W
N79°52'20"E
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
S:\18091\working\current\18091-grading and servicing.dwg
1/14/2020 1:57:49 PM - Steven DeGrow

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

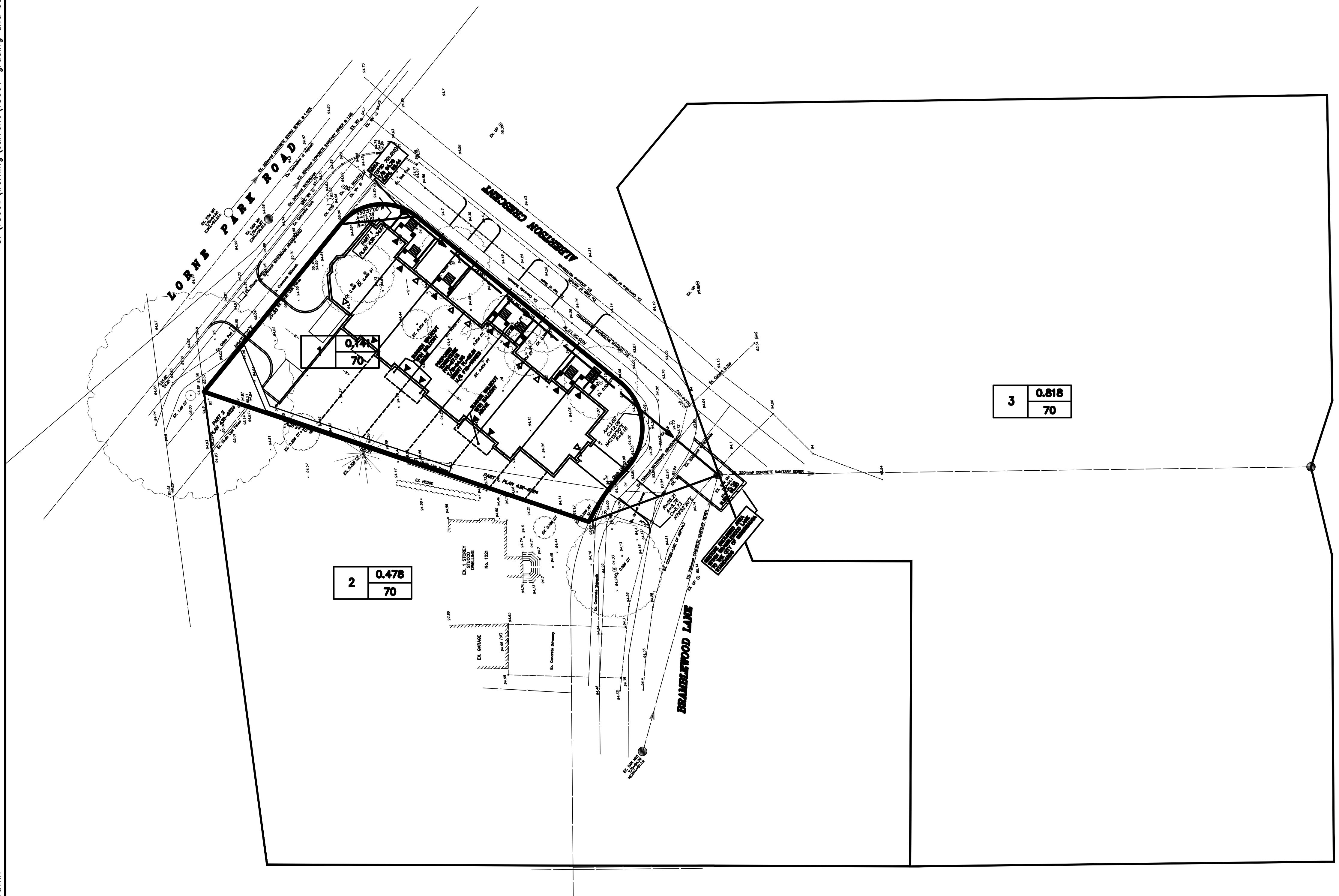
$M = 1 + \frac{14}{4 + p^{0.5}}$ P is in thousands

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: Diameter [mm]	Material	Grade %	Capacity l/s	Velocity m/s	% Full	Actual Velocity (m/s)	Remarks
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	

S:\18091\working\current\18091-grading and servicing.dwg



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

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

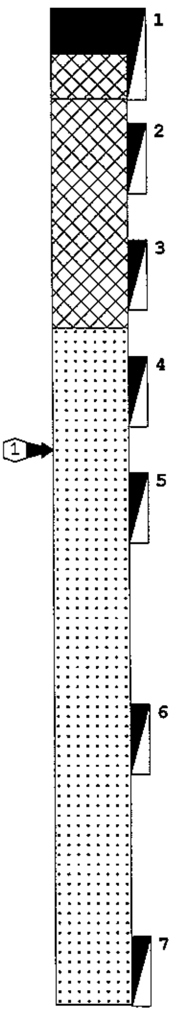
Encls.
Appendix A.

BOREHOLE LOG
BOERHOLE NO.: 1

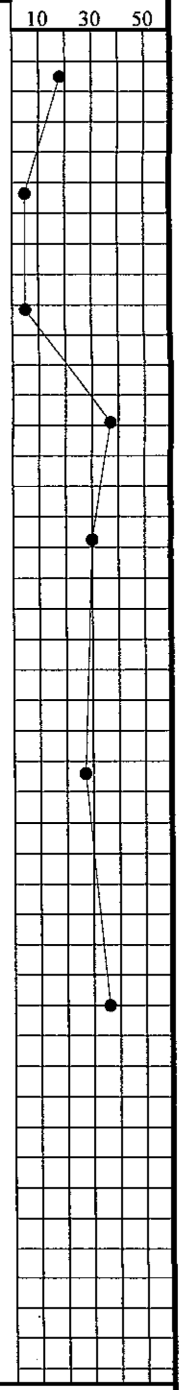
PROJECT: Proposed Office Building
LOCATION: 1110 Lorne Park Road, City of Mississauga, Ontario
ELEVATION (m) 94.23
CAVED AT DEPTH (m): 2.90
N=Blow Count in Standard Penetration Test (Blows/0.3m)

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

Sample	DESCRIPTION	M.C. %	N
1	TOPSOIL- mixed with fine sand, about 300 mm thick.		
	FILL - mixed dark brown and grey sand with some gravel, wet (frozen)	11.8	18
2	FILL - reddish brown fine sand, moist	6.2	5
3	- layer of greyish brown, moist, fine to medium sand below about 1.4 m	6.9	5
4	FINE SAND - trace of silt, greyish brown, moist to very moist, dense.	13.2	37
5		18.3	30
	- becomes wet and compact to dense below about 4.0 m		
6		22.2	27
7		21.6	36



STANDARD PENETRATION TEST — CURVE N (Blows/0.3m)



End of Borehole @ 6.55 m
Cave in at 2.9 m
No water in borehole on completion

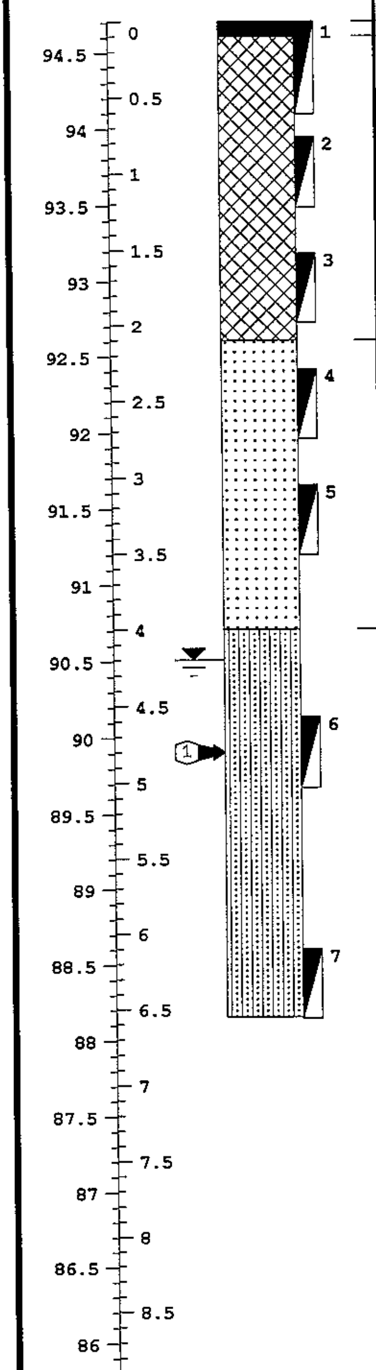
Enclosure No. 1

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)

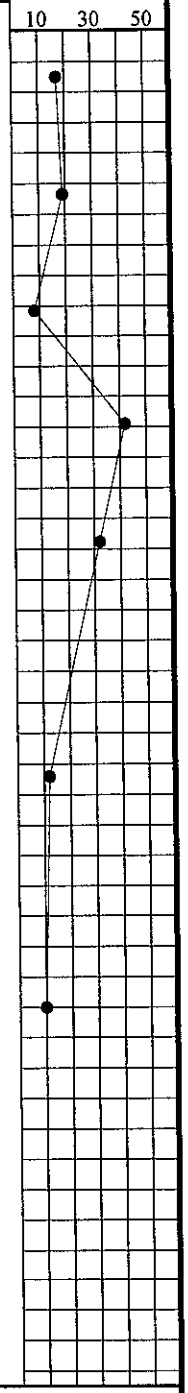
PROJECT NO.: SP-3179
DATE: January 21, 2011
WATER LEVEL DEPTH (m): 4.2
M.C. = Natural Moisture Content



DESCRIPTION	M.C. %	N	Sample
TOPSOIL - about 100 mm thick			
FILL - dark brown fine sand with trace to some organics, moist	10.8	17	1
- layer of dark brown fine sand below about 0.6 m	15.7	19	2
- occasional root below about 1.4 m.	12.3	8	3
FINE SAND - trace of silt, greyish brown, moist, dense.	11.1	42	4
	16.2	32	5
SILTY FINE SAND - brown, wet, compact to loose.	16.7	12	6
	23.5	10	7

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 — CURVE N (Blows/0.3m)



Enclosure No. 2

SOIL PROBE LTD.

BOREHOLE LOG
BOERHOLE NO.: 3

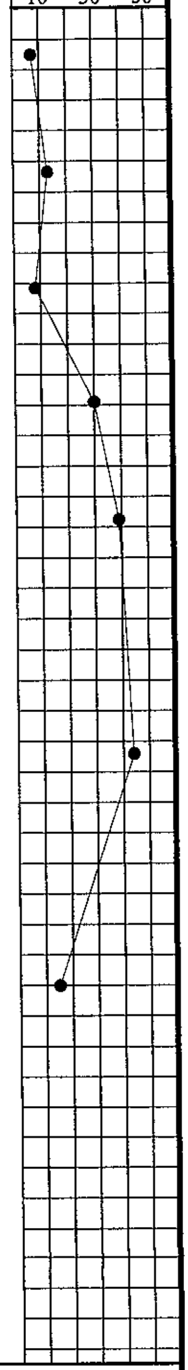
PROJECT: Proposed Office Building
LOCATION: 1110 Lorne Park Road, City of Mississauga, Ontario
ELEVATION (m) 94.78
CAVED AT DEPTH (m): 4.6
N=Blow Count in Standard Penetration Test (Blows/0.3m)

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

Sample	DESCRIPTION	M.C. %	N
1	TOPSOIL - about 150 mm thick FILL - dark brown fine sand with trace to some organics, moist	9.9	7
2		7.7	13
3	FILL - dark brown fine sand, moist	7.8	8
4		11.1	30
5	FINE SAND - trace of silt greyish brown, moist, compact to dense.	10.5	39
	- wet below about 4.0 m		
6		19.8	44
	- compact below about 5.5 m.		
7		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

STANDARD PENETRATION TEST – CURVE N (Blows/0.3m)



Enclosure No. 3

SOIL PROBE LTD.

BOREHOLE LOG
BOERHOLE NO.: 4

SOIL PROBE

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)

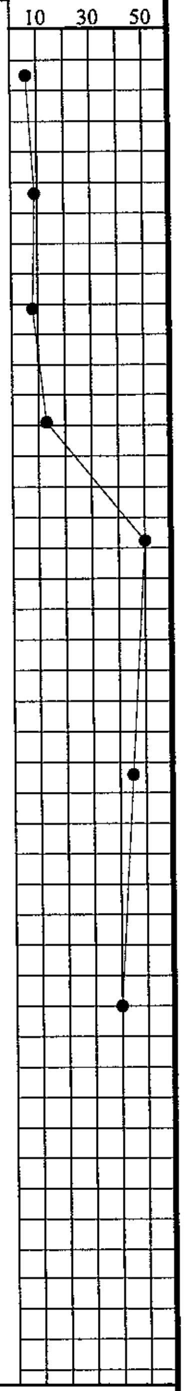
PROJECT NO.: SP-3179
DATE: January 21, 2011
WATER LEVEL DEPTH (m): 3.8
M.C. = Natural Moisture Content

Sample	DESCRIPTION	M.C. %	N
1	FILL - greyish brown fine sand mixed with some topsoil/organics, moist	17.0	6
2	TOPSOIL - mixed with some fine sand, about 300 mm thick FILL - reddish brown fine sand, moist	9.2	9
3	- layer of dark brown medium sand at about 1.4 m	6.9	8
4		10.4	13
5	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		
6		18.0	45
7	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

ELEVATION/DEPTH (m): 94.5 / 0; 94 / 0.5; 93.5 / 1; 93 / 1.5; 92.5 / 2; 92 / 2.5; 91.5 / 3; 91 / 3.5; 90.5 / 4; 90 / 4.5; 89.5 / 5; 89 / 5.5; 88.5 / 6; 88 / 6.5; 87.5 / 7; 87 / 7.5; 86.5 / 8; 86 / 8.5

STANDARD PENETRATION TEST — CURVE N (Blows/0.3m)



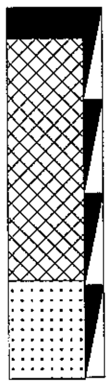
Enclosure No. 4

BOREHOLE LOG
BOERHOLE NO.: 5

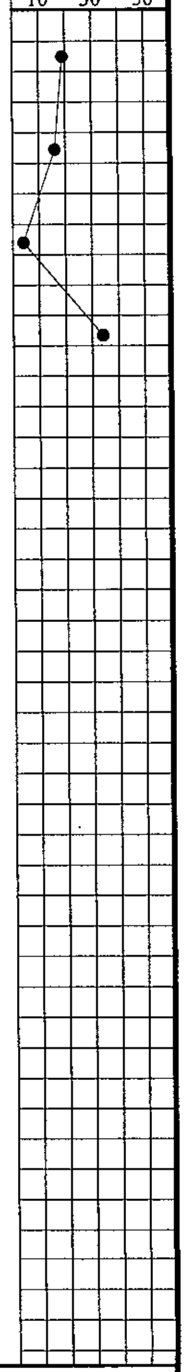
PROJECT: Proposed Office Building
LOCATION: 1110 Lorne Park Road, City of Mississauga, Ontario
ELEVATION (m) 94.16
CAVED AT DEPTH (m):
N=Blow Count in Standard Penetration Test (Blows/0.3m)

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

ELEVATION/ DEPTH (m)	SOIL SYMBOLS	DESCRIPTION	M.C. %	STANDARD PENETRATION TEST N
94.16 / 0	1	TOPSOIL - about 200 mm thick		
		FILL - grey silty sand with trace of organics, moist.	8.6	19
	2	- 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
	3		11.1	4
	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.		



STANDARD PENETRATION TEST — DEPTH/ ELEVATION (m) — CURVE N (Blows/0.3m)



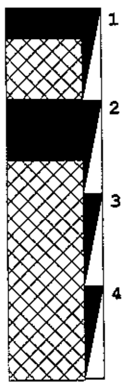
Enclosure No. 5

SOIL PROBE LTD.

BOREHOLE LOG
BOERHOLE NO.: 6

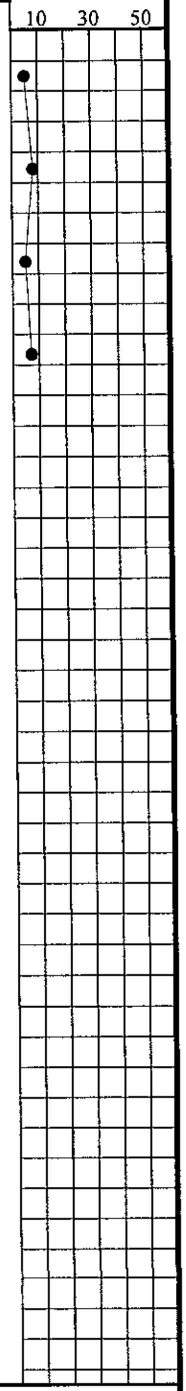
PROJECT: Proposed Office Building
LOCATION: 1110 Lorne Park Road, City of Mississauga, Ontario
ELEVATION (m) 94.26
CAVED AT DEPTH (m):
N=Blow Count in Standard Penetration Test (Blows/0.3m)

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



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

STANDARD PENETRATION TEST — CURVE N (Blows/0.3m)



Enclosure No. 6

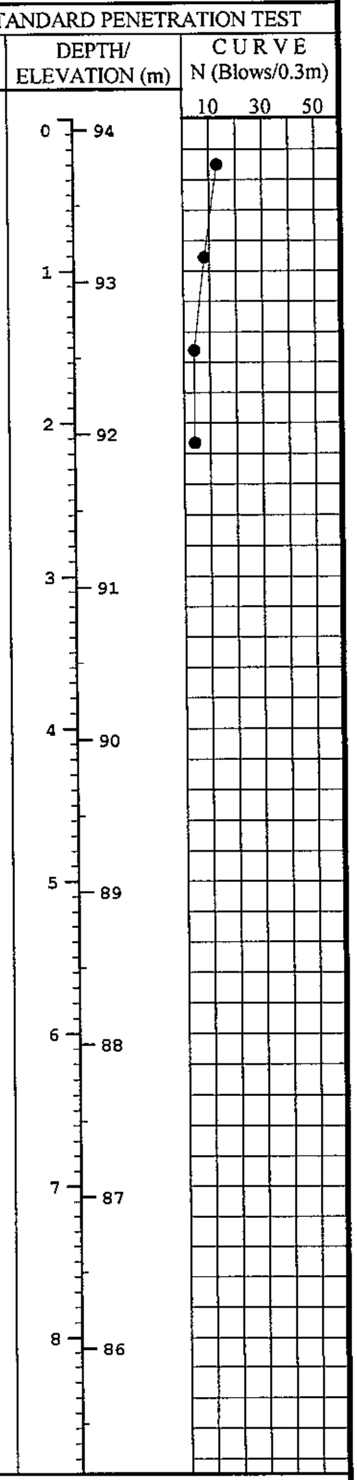
SOIL PROBE LTD.

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

ELEVATION/ DEPTH (m)	SOIL SYMBOLS	DESCRIPTION	M.C. %	N
94 / 0	1	TOPSOIL - mixed with some sand and trace of gravel size stones, moist (frozen) about 800 mm thick.	9.2	13
93 / 1	2	FILL - reddish brown fine sand, moist.	7.1	8
	3	- includes occasional roots, below about 1.2 m	6.1	4
92 / 2	4		6.8	4
		End of borehole at 2.45 m Borehole dry on completion.		



Enclosure No. 7

SOIL PROBE LTD.

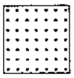
KEY TO SYMBOLS

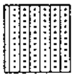
Enclosure No. 8
Report No.: 2011 – 22768

Symbol Description

Strata symbols

 Fill

 Sand

 Silty sand

 Sandy silt

 Topsoil

Notes:

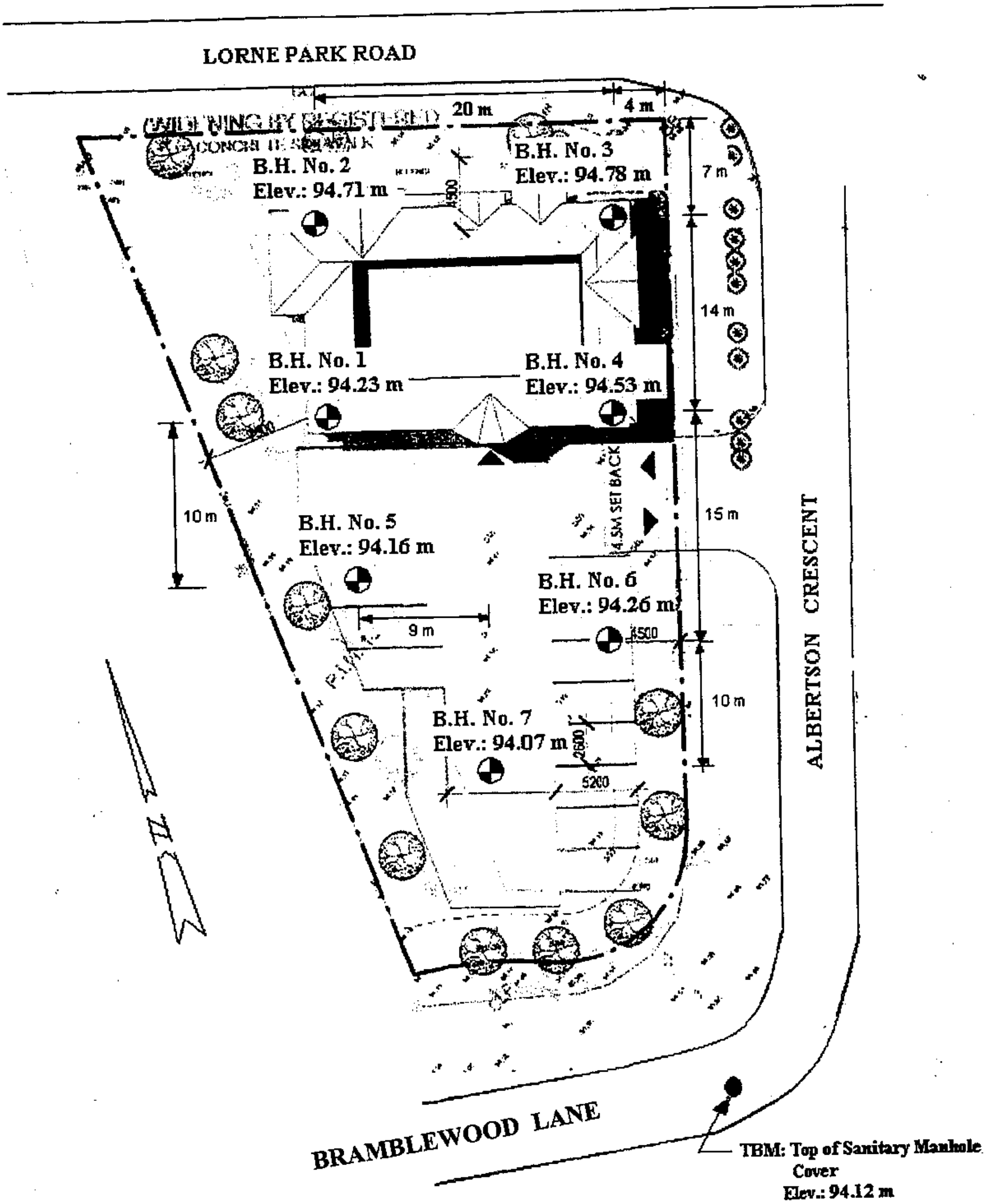
TERMS DESCRIBING RELATIVE DENSITY, BASED ON STANDARD PENETRATION TEST N-VALUE FOR COARSE GRAINED SOILS (major portion retained on No.200 sieve).

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



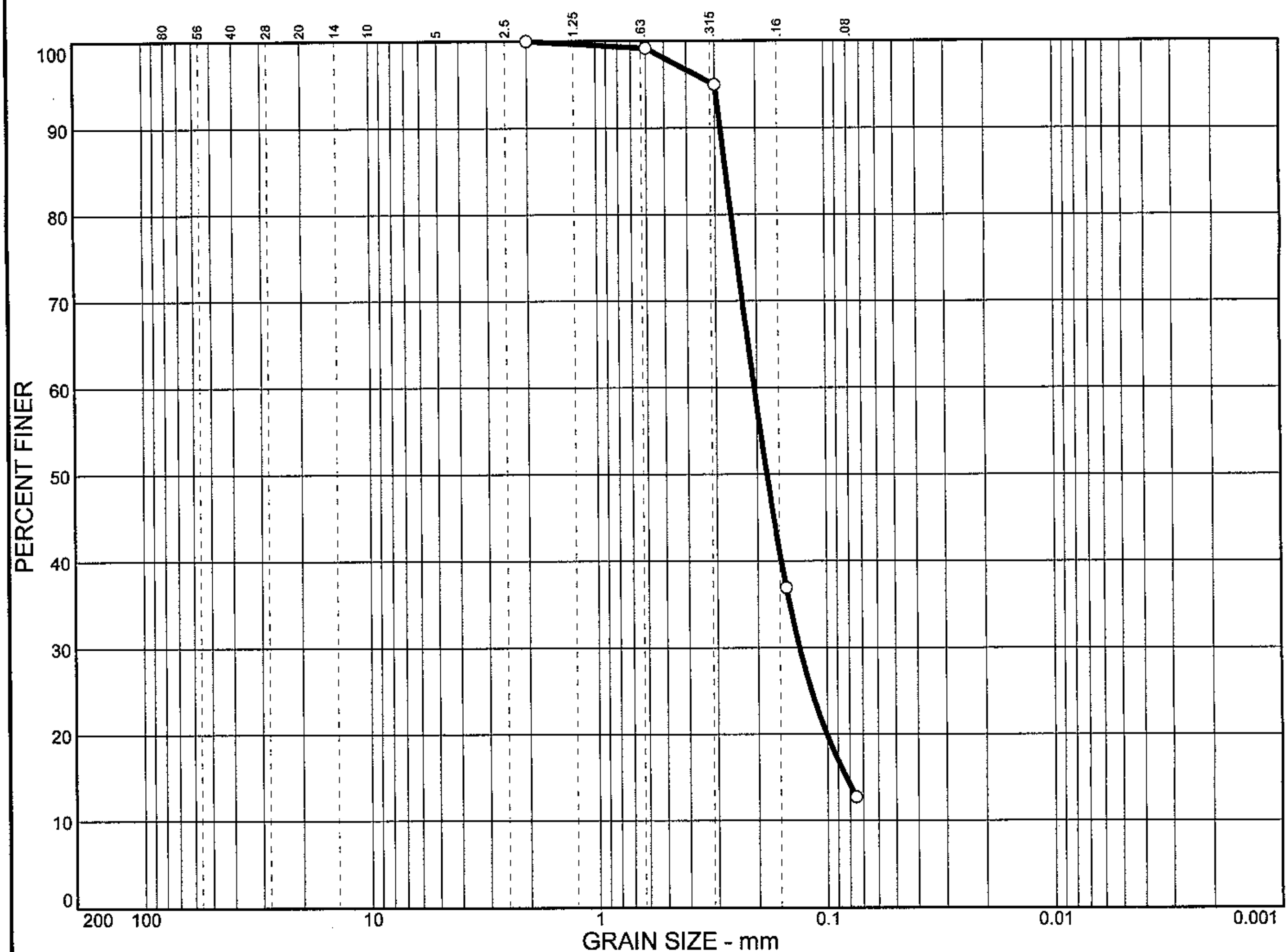
LEGEND

BOREHOLE

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

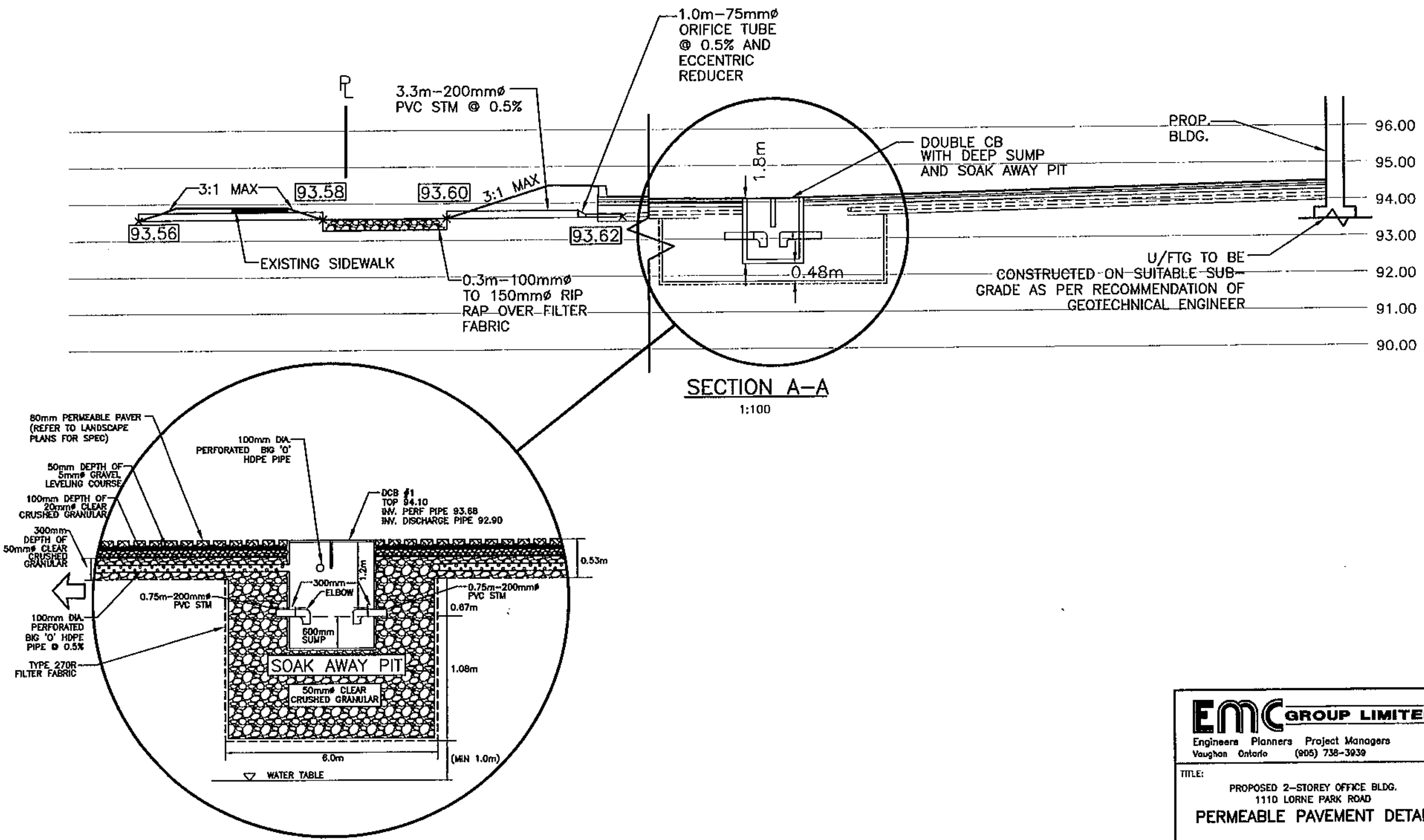
GRAIN SIZE DISTRIBUTION TEST REPORT

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