Servicing Report – 1951 Carling Avenue

Project # 160401679

Prepared for:
2473493 Ontario Inc.

Prepared by:
Stantec Consulting Ltd.

June 18, 2021

Sign-off Sheet

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Prepared by ______________________________
(signature)

Thakshika Rathnasooriya, P.Eng.

LICENSED PROFESSIONAL ENGINEER
T.K. RATHNASOORIYA
100225159
2021-06-18
PROVINCE OF ONTARIO

Reviewed by ______________________________
(signature)

Kris Kilborn

Stantec

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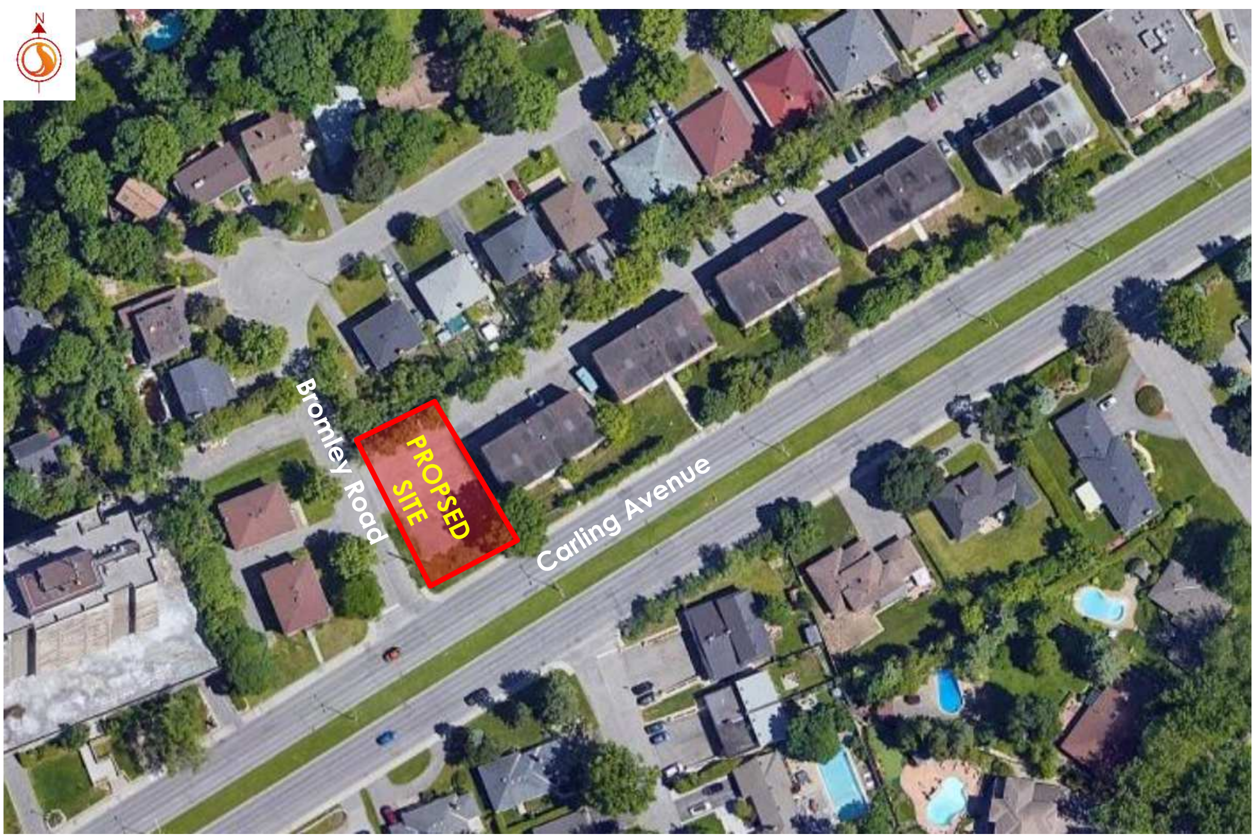
1.0 INTRODUCTION

Stantec Consulting Ltd. has been commissioned by 2473493 Ontario Inc. to prepare a servicing study in support of Site Plan Control submission of the proposed development located at 3368 Carling Avenue. The site is situated on the north side of Carling Avenue and east of the intersection of Bromley Road and Carling Avenue within the City of Ottawa. The proposed 0.09ha (0.22 acres) site would replace an existing parking area with a five-storey apartment complex comprising 27 total residential units. The proposed location of the site is shown in **Figure 1**. The site is presently zoned Arterial Mainstreet Zone, which permits the proposed site plan.

The intent of this report is to provide a servicing scenario for the site that is free of conflicts, provides on-site servicing in accordance with City of Ottawa design guidelines, and utilizes the existing local infrastructure in accordance with the guidelines outlined per consultation with City of Ottawa staff.

Figure 1: Location Plan



2.0 BACKGROUND

Documents referenced in preparation of the design for the 1951 Carling Avenue development include:

- Geotechnical Investigation – Proposed Residential Development – 1983 Carling Avenue, Pinchin Ltd., May 4, 2021.
- City of Ottawa Sewer Design Guidelines, City of Ottawa, October 2012.
- City of Ottawa Design Guidelines – Water Distribution, City of Ottawa, July 2010.
- Technical Bulletin ISDTB-2014-01, City of Ottawa, February 2014
- *Technical Bulletin ISTB-2018-01*, City of Ottawa, March 21, 2018
- *Technical Bulletin ISTB-2018-02*, City of Ottawa, March 21, 2018
- *Technical Bulletin ISTB-2018-03*, City of Ottawa, March 21, 2018

3.0 WATER SUPPLY SERVICING

3.1 BACKGROUND

The proposed development comprises one five storey residential apartment building, complete with associated infrastructure and access areas. The site is located on the north side of Carling Avenue and east of the intersection with Bromley Road. The site will be serviced via a 50mm building service connection to the existing 150mm dia. watermain within the Bromley Road ROW at the western boundary of the site. The property is located within the City's Pressure Zone 1W. Ground elevations of the site are approximately 82.2m. Under normal operating conditions, hydraulic gradelines vary from approximately 108.6m to 114.6m as confirmed through updated boundary conditions provided by the City of Ottawa (see **Appendix A.3**).

3.2 WATER DEMANDS

Water demands for the development were estimated using the Ministry of Environment's Design Guidelines for Drinking Water Systems (2008). A daily rate of 350 L/cap/day has been applied for the population of the proposed site. Population densities have been assumed as 1.4 pers./studio and single units and 2.1 pers/ two bedroom units. See **Appendix A.1** for detailed domestic water demand estimates.

The average day demand (AVDY) for the entire site was determined to be 0.2 L/s. The maximum daily demand (MXDY) is 2.5 times the AVDY (residential property), which equals 0.4 L/s. The peak hour demand (PKHR) is 2.2 times the MXDY, totaling 0.9 L/s.

Non-combustible construction was considered in the assessment for fire flow requirements. A sprinkler system was assumed for construction of the proposed building. Based on calculations per the OBC Guidelines (**Appendix A.2**), the maximum required fire flow for this development is 45 L/s (2,700L/min). The OBC calculations assumed 2-hour fire separation between the 3rd and 4th floor to maintain below the available City fire flows of 73L/s as per boundary conditions.

3.3 PROPOSED SERVICING

Boundary conditions provided by the City of Ottawa and based on an approximate elevation on-site of 82.2m, adequate domestic flows are available for the subject site, with pressures ranging from 26.4m (38 psi) to 32.4m (46 psi). This pressure range is slightly below the guidelines of 40-80 psi based on Ottawa's Design Guidelines for Water Distribution. As such, booster pumps will be required to meet the minimum pressures on all five floors of the proposed building.

Boundary conditions for the proposed development under maximum day demands and fire flow requirements demonstrate that the system will maintain a residual pressure of approximately the required 140 kPa (20 psi). The above demonstrates that the existing watermain within Bromley Road can provide adequate fire flows for the subject site.

A hydrant has been proposed on the City ROW along Bromley Road within 45m of the proposed building's primary entrance.

3.4 SUMMARY OF FINDINGS

The proposed development is located in an area of the City's water distribution system that has sufficient capacity to provide the required emergency fire flows. A two-hour fire separation is required between the 3rd and 4th floors to meet the minimum fire flow constraints. Booster pumps will be required to meet the domestic flows for the site and maintain a minimum pressure of 40psi for all five floors.

Based on the boundary conditions provided by City of Ottawa, the required fire flows are available for this development based on OFM guidelines and as per the City of Ottawa water distribution guidelines.

4.0 WASTEWATER SERVICING

4.1 BACKGROUND

The site will be serviced via an existing 225mm diameter sanitary sewer within Bromley Road immediately west of the subject site. A proposed 150 mm diameter service lateral connection is to be made directly to the existing 225 mm diameter concrete sanitary sewer along Bromley Road to service the proposed site (see **Drawing SSP-1**). The location of the existing sanitary service lateral shall be confirmed prior to construction and is to be abandoned as part of the servicing works.

4.2 DESIGN CRITERIA

As outlined in the City of Ottawa Sewer Design Guidelines and the MECP's Design Guidelines for Sewage Works, the following criteria were used to calculate estimated wastewater flow rates and to size the sanitary sewers:

- Minimum Velocity – 0.6 m/s (0.8 m/s for upstream sections)
- Maximum Velocity – 3.0 m/s
- Manning roughness coefficient for all smooth wall pipes – 0.013
- Minimum size – 200mm dia. for residential areas
- Average Wastewater Generation – 280L/cap/day
- Peak Factor – 4.0 (Harmon's)
- Extraneous Flow Allowance – 0.33 l/s/ha (conservative value)
- Manhole Spacing – 120 m
- Minimum Cover – 2.5m

4.3 PROPOSED SERVICING

The proposed site will be serviced by gravity sewers which will direct the wastewater flows (approx. 0.53 L/s with allowance for infiltration) to the proposed 225mm diameter sanitary sewer on Bromley Road. The proposed drainage pattern is detailed on **Drawing SSP-1**. A sanitary sewer design sheet for the proposed service lateral is included in **Appendix B.1**. A backwater valve is to be installed on the proposed sanitary service within the site and on all sanitary branches in the underground parking level to prevent any surcharge from the downstream sanitary sewer from impacting the proposed property.

5.0 STORMWATER MANAGEMENT

5.1 OBJECTIVES

The objective of this stormwater management plan is to determine the measures necessary to control the quantity/quality of stormwater released from the proposed development to criteria established during the pre-consultation/zoning process, and to provide sufficient detail for approval and construction.

5.2 SWM CRITERIA AND CONSTRAINTS

Criteria were established by combining current design practices outlined by the City of Ottawa Design Guidelines (2012), and through consultation with City of Ottawa staff. The following summarizes the criteria, with the source of each criterion indicated in brackets:

General

- Use of the dual drainage principle (City of Ottawa).
- Wherever feasible and practical, site-level measures should be used to reduce and control the volume and rate of runoff. (City of Ottawa)
- Assess impact of 100-year event outlined in the City of Ottawa Sewer Design Guidelines on major & minor drainage system (City of Ottawa)

Storm Sewer & Inlet Controls

- Size storm sewers to convey 5-year storm event under free-flow conditions using City of Ottawa I-D-F parameters (City of Ottawa).
- Proposed site to discharge to the existing 300mm diameter storm sewer within the Bromley Road at the western boundary of the subject site (City of Ottawa).
- All stormwater runoff from the site up to and including the 100-year storm event to be stored on site and released into the minor system at a maximum discharge equivalent to the 5-year storm predevelopment release rate to Bromley Road at a maximum runoff coefficient of 0.5.
- 100-year Storm HGL to be a minimum of 0.30 m below building foundation footing (City of Ottawa).

Surface Storage & Overland Flow

- Building openings to be a minimum of 0.30m above the 100-year water level (City of Ottawa)
- Maximum depth of flow under either static or dynamic conditions shall be less than 0.30m (City of Ottawa)
- Provide adequate emergency overflow conveyance off-site (City of Ottawa)

5.3 STORMWATER MANAGEMENT

The Modified Rational Method was employed to assess the rate and volume of runoff generated during post-development conditions. The site was subdivided into subcatchments (subareas) tributary to stormwater controls as defined by the location of inlet control devices. A summary of subareas and runoff coefficients is provided in **Appendix C**, and **Drawing SD-1** indicates the stormwater management subcatchments.

5.3.1 Allowable Release Rate

Based on consultation with City of Ottawa staff, restrictions on the peak post-development discharge rate from the subject site are not required should the post development peak flowrate not increase dramatically beyond the 5-year event pre-development scenario calculated with a maximum runoff coefficient of 0.5. The predevelopment release rate for the area has been determined using the rational method based on the criteria above. A time of concentration for the predevelopment area (10 minutes) was assigned based on the relatively small site and its proximity to the existing drainage outlet for the site. C coefficient values have been increased by 25% for the post-development 100-year storm event based on MTO Drainage Manual recommendations. Peak flow rates have been calculated using the rational method as follows:

$Q = 2.78\ CiA$

Where: Q = peak flow rate, L/s

A = drainage area, ha

I = rainfall intensity, mm/hr (per Ottawa IDF curves)

C = site runoff coefficient

Table 1: Target Release Rate

Design Storm	Target Flow Rate (L/s)
5-Year Event	13.3

5.3.2 Storage Requirements

It is proposed that rooftop storage via restricted roof release be used to reduce site peak outflow to reduce the impact on downstream infrastructure.

5.3.2.1 Rooftop Storage

It is proposed to retain stormwater on the building rooftops by installing restricted flow roof drains. The following calculations assume the roof will be equipped with standard Watts Model R1100 Accuflow Roof Drains.

Watts Drainage "Accutrol" roof drain weir data has been used to calculate a practical roof release rate and detention storage volume for the rooftops. It should be noted that the

"Accutrol" weir has been used as an example only, and that other products may be specified for use, provided that the total roof drain release rate is restricted to match the maximum rate of release indicated in **Table 2**, and that sufficient roof storage is provided to meet (or exceed) the resulting volume of detained stormwater. Proposed drain release rates have been calculated based on the Accutrol weir setting at ¼ open. Storage volume and controlled release rate are summarized in **Table 2**:

Table 2: Roof Control Area

Design Storm	Depth (mm)	Discharge (L/s)	Volume Stored (m^3)
5-Year	95	3.1	3.6
100-Year	132	3.6	9.5

Number of roof drains: 4

5.3.2.2 Subsurface Storage

Per the modified rational method calculations included as part of **Appendix C.2**, the remainder of the site is to be directed towards two catch basins (CB 200 and CB201) complete with IPEX Tempest LMF 100 ICD(CB200) to meet the target peak discharge rate during the 100-year event. In order to control peak discharge from the subject site to within target levels, a superpipe has been provided between catch basins to provide storage volume in the amount of approximately 7.1m3.

Controlled release rates and storage volumes required are summarized in **Table 5**.

Table 5: Subsurface Storage

Storm Return Period	Area ID	Design Head (m)	Discharge (L/s)	Orifice Type	$V_{required}$ (m^3)
5-year	CB-200A and CB-200B	0.32	5.0	IPEX Tempest LMF 100 ICD	2.8
100-year		0.70	7.5		6.4

5.3.2.3 Uncontrolled Release

Due to grading restrictions, one subcatchment area has been designed without a storage component. The UNC-1 catchment area discharges off-site uncontrolled to the adjacent Bromley Road ROW. Peak discharges from uncontrolled areas have been considered in the overall SWM plan and have been balanced through overcontrolling the proposed site discharge rates to meet target levels.

Table 3 summarizes the estimated uncontrolled storm release rates during the 5 and 100-year storm events.

Table 3: 5 and 100 Year Peak Uncontrolled Discharge Summary

Drainage Area	5-Year Event Discharge (L/s)	100-Year Event Discharge (L/s)
UNC-1	2.8	5.9

5.3.3 Results

Table 4 demonstrates that the proposed stormwater management plan provides adequate attenuation storage and demonstrates a minor increase (0.12 L/s) beyond the target 5-year storm peak discharge rate.

Table 4: Summary of Total 5 and 100-Year Event Release Rates

	5-Year Peak Discharge (L/s)	100-Year Peak Discharge (L/s)
Uncontrolled	2.8	5.9
Roof (enters subsurface storage pipe)	2.5	2.5
Controlled – Subsurface Storage	5.0	7.5
Total	**7.8**	**13.42**
Target	**13.3**	**13.33**

5.3.4 Quality Control

On-site quality control measures are expected for the proposed development per City of Ottawa staff. The Rideau Valley Conservation Authority (RVCA) has been contacted and at this time quality control measures have not been confirmed.

It is assumed that enhanced protection (80% removal of total suspended solids) will be required for the site before discharging to Bromley Road storm sewer and ultimately to the watercourse downstream. As a result, an oil grit separator (OGS) has been proposed to treat runoff from impervious areas. The OGS unit will be privately maintained and located upstream of the connection to the storm sewer as shown on **Drawing SD**. The OGS unit has been sized with tools provided by the manufacturer to validate whether the unit is appropriate for the contributing area. The analysis included as part of **Appendix C.3** indicates that the unit provides a minimum of 80% TSS removal for the site, meeting water quality objectives for the downstream watercourse. Stormceptor EF6 has been used as an example only, and that other products may be specified for use provided that the system removes 80% TSS given an overall site imperviousness of 68% (C = 0.68).

As the majority of impervious surfaces are directed to the on-site OGS unit, suspended solids within runoff generated by the site will not have a deleterious impact on downstream watercourses.

6.0 GRADING AND DRAINAGE

The proposed development site measures approximately 0.91ha in area. The site slopes from south to north, with grades at property corners varying by approximately 1.1m across the site. Overland flow is generally being directed to the adjacent Bromley Road ROW. A detailed grading plan (see **Drawing GP-1**) has been provided to satisfy any stormwater management requirements and provide for minimum cover requirements for storm and sanitary sewers where possible. Existing grades at the rear of the property have been maintained. Site grading has been established to provide emergency overland flow routes required for stormwater management in accordance with City of Ottawa requirements.

The subject site maintains emergency overland flow routes for flows deriving from storm events in excess of the maximum design event to the existing Carling Avenue and Bromley Road ROWs as depicted in **Drawing GP-1.**

7.0 UTILITIES

As the subject site lies within a developed residential community, Hydro, Bell, Gas and Cable servicing for the proposed development should be readily available. It is anticipated that existing infrastructure will be sufficient to provide a means of distribution for the proposed site. Exact size, location and routing of utilities, along with determination of any off-site works required for redevelopment, will be finalized after design circulation.

8.0 APPROVALS

It is not expected that Environmental Compliance Approvals (ECAs) under the Ontario Water Resources Act will be required by the Ontario Ministry of Environment conservations and Parks(MECP), as the proposed sewers will be approved under the building code act and the entirety of the site is maintained under one ownership.

The Rideau Valley Conservation Authority will need to be consulted in order to obtain municipal approval for site development. A Requirement for a MECP Permit to Take Water (PTTW) may be required and can be confirmed by the geotechnical consultant at the time of application.

9.0 EROSION CONTROL DURING CONSTRUCTION

Erosion and sediment controls must be in place during construction. The following recommendations to the contractor will be included in contract documents.

1. Implement best management practices to provide appropriate protection of the existing and proposed drainage system and the receiving water course(s).
2. Limit extent of exposed soils at any given time.
3. Re-vegetate exposed areas as soon as possible.
4. Minimize the area to be cleared and grubbed.
5. Protect exposed slopes with plastic or synthetic mulches.
6. Provide sediment traps and basins during dewatering.
7. Install sediment traps (such as SiltSack® by Terrafix) between catch basins and frames.
8. Plan construction at proper time to avoid flooding.

The contractor will, at every rainfall, complete inspections and guarantee proper performance. The inspection is to include:

9. Verification that water is not flowing under silt barriers.
10. Clean and change silt traps at catch basins.

Refer to **Drawing ECDS-1** for the proposed location of silt fences, straw bales and other erosion control structures.

10.0 GEOTECHNICAL INVESTIGATION AND ENVIRONMENTAL ASSESSMENT

A geotechnical Investigation report was prepared by Pinchin Ltd. on May 4, 2016. The report summarizes the existing soil conditions within the subject area and construction recommendations. For details which are not summarized below, please see the original Pinchin report.

A subsurface investigation was conducted with four borehole samples which concluded that the site is underlain by surficial asphalt or granular fill followed by glacial till and bedrock.

Bedrock was encountered within 0.8m to 0.9m below ground surface. Groundwater was not found within the open boreholes. Refer to File #289578 for additional Geotechnical information.

11.0 CONCLUSIONS

11.1 WATER SERVICING

Based on the supplied boundary conditions for existing watermain and estimated domestic and fire flow demands for the subject site, it is anticipated that two hour fire separation between the 3rd and 4th floor and booster pumps will be required to provide sufficient capacity to sustain both the required domestic demands and emergency fire flow demands of the proposed site.

11.2 SANITARY SERVICING

The proposed sanitary sewer network is sufficiently sized to provide gravity drainage of the site. The proposed site will be serviced by a gravity sewer service lateral which will direct wastewater flows (approx. 0.56 L/s) to a 225mm dia. sanitary sewer to be constructed within the Bromley Road ROW at the western boundary of the property. The proposed drainage outlet has sufficient capacity to receive sanitary discharge from the site.

11.3 STORMWATER SERVICING

The proposed stormwater management plan is in compliance with local and provincial standards. Rooftop storage with controlled roof drains, and subsurface storage via a large diameter storage pipe has been proposed to limit peak storm sewer inflows to the existing 300mm diameter storm sewers along Bromley Road ROW. The downstream receiving sewer has sufficient capacity to receive runoff volumes from the site.

11.4 GRADING

Grading for the site has been designed to provide an emergency overland flow route as per City requirements and reflects recommended in the Geotechnical Investigation Report prepared by Pinchin Ltd. on May 4, 2021. Erosion and sediment control measures will be implemented during construction to reduce the impact on existing facilities.

11.5 UTILITIES

Utility infrastructure exists within the existing Carling Avenue and Bromley Road ROWs at the southern and western boundaries of the proposed site. It is anticipated that existing infrastructure will be sufficient to provide a means of distribution for the proposed site. Exact size, location and routing of utilities will be finalized after design circulation.

11.6 APPROVALS/PERMITS

An MECP Environmental Compliance Approval is not expected to be required for the subject site as the on-site sewers are subject to the Building Code. A Permit to Take Water is anticipated to be required and will be confirmed by geotechnical consultant. The Rideau Valley Conservation Authority will need to be consulted in order to obtain municipal approval for site development. No other approval requirements from other regulatory agencies are anticipated.

Appendix A WATER SUPPLY SERVICING

A.1 DOMESTIC WATER DEMAND ESTIMATE

1951 Carling Avenue - Domestic Water Demand Estimates
Site Plan provided by Figurr Architects Collective (2021-03-23)
Project No. 160401679

Densities as per City Guidelines:		
Apartment Units		
1 Bedroom	1.4	ppu
2 Bedroom	2.1	ppu

Building ID	No. of Units	Population	Daily Rate of Demand [1]	Avg Day Demand		Max Day Demand [2]		Peak Hour Demand [2]	
			(L/m^2/day)	(L/min)	(L/s)	(L/min)	(L/s)	(L/min)	(L/s)
Apartment Units									
1 Bedroom / Studio	26	36	350	8.8	0.15	22.1	0.37	48.7	0.81
2 Bedroom	1	2	350	0.5	0.01	1.3	0.02	2.8	0.05
Total Site :	**27**	**39**		**9.4**	**0.2**	**23.4**	**0.4**	**51.5**	**0.9**

1 Average day water demand for residential areas: 350 L/cap/d
2 The City of Ottawa water demand criteria used to estimate peak demand rates for residential areas are as follows:
maximum day demand rate = 2.5 x average day demand rate for residential
peak hour demand rate = 2.2 x maximum day demand rate for residential

A.2 FIRE FLOW REQUIREMENTS PER OBC

Fire Flow Calculations as per Ontario Building Code 2006 (Appendix A)

Job#	160401679	Designed by:	TR
Date	28-May-21	Checked by:	
		Description:	5-Storey Res. Firewall between 3rd and 4th floor

$$Q = KVS_{tot}$$

Q = Volume of water required (L)
V = Total building volume (m3)
K = Water supply coefficient from Table 1
S_{tot} = Sotal of spatial coefficeint values from property line exposures on all sides as obtained from the formula

$S_{tot} = 1.0 + [S_{side1} + S_{side2} + S_{side3} + S_{side4}]$

1	Type of construction	Building Classification		Water Supply Coefficient
	Non-Combustible with Fire-Resistance Ratings	A-2, B-1, B-2, B-3, C, D		10
2	Area of one floor (m^2)	number of floors	Avg. height of ceiling (m)	Total Building Volume (m^3)
	345	3	3.13	3,236
3	Side	Exposure Distance (m)	Spatial Coefficient	Total Spatial Coeffiecient
	North	8.3	0.17	2
	East	4	0.5	
	South	6.7	0.33	
	West	3	0.5	
4	Established Fire Safety Plan?	Reduction in Volume (%)		Total Volume Reduction
	no	0%		0%
5				**Total Volume 'Q' (L)**
				64,720
				Minimum Required Fire Flow (L/min)
				2,700

A.3 BOUNDARY CONDITIONS

From: Surprenant, Eric
To: Rathnasooriya, Thakshika
Cc: Kilborn, Kris; Mott, Peter
Subject: Fw: 1951 Carling Avenue - Boundary Conditions Request
Date: Monday, May 17, 2021 2:11:21 PM
Attachments: 1951 Carling Avenue May 2021.pdf

Hello Thakshika,

See the following regarding your additional inquiry.

The following are boundary conditions, HGL, for hydraulic analysis at 1951 Carling (zone 1W) assumed to be connected to the 152mm on Bromley Road (see attached PDF for location).

Minimum HGL = 108.6 m

Maximum HGL = 114.6 m

Available Fire Flow @ 20 psi = 73 L/s, assuming a ground elevation of 82.7 m.

These are for current conditions and are based on computer model simulation.

Disclaimer: The boundary condition information is based on current operation of the city water distribution system. The computer model simulation is based on the best information available at the time. The operation of the water distribution system can change on a regular basis, resulting in a variation in boundary conditions. The physical properties of watermains deteriorate over time, as such must be assumed in the absence of actual field test data. The variation in physical watermain properties can therefore alter the results of the computer model simulation.

Thanks

Eric Surprenant, CET
Sr, Project Manager, Infrastructure Projects, West
Planning, Infrastructure & Economic Development
613 580-2424 ext.: 27794

From: Rathnasooriya, Thakshika <Thakshika.Rathnasooriya@stantec.com>

Sent: May 13, 2021 14:32
To: Surprenant, Eric <Eric.Surprenant@ottawa.ca>
Cc: Kilborn, Kris <kris.kilborn@stantec.com>; Mott, Peter <Peter.Mott@stantec.com>
Subject: RE: 1951 Carling Avenue - Boundary Conditions Request

Hi Eric ,

Are you able to also provide us with the boundary conditions for a maximum day plus fire flow demand of 105 L/s (6,300L/min).

Thank you,

Shika Rathnasooriya , P.Eng.

Direct: 613-668-9635
Thakshika.Rathnasooriya@stantec.com

Stantec
400 - 1331 Clyde Avenue
Ottawa ON K2C 3G4

From: Surprenant, Eric <Eric.Surprenant@ottawa.ca>
Sent: Wednesday, May 12, 2021 4:21 PM
To: Mott, Peter <Peter.Mott@stantec.com>; Kilborn, Kris <kris.kilborn@stantec.com>
Subject: Fw: 1951 Carling Avenue - Boundary Conditions Request

Hello Peter,

Please verify the fire demand as our water resources, Senior Engineer has noted that it seemed a bit low.

The following are boundary conditions, HGL, for hydraulic analysis at 1951 Carling (zone 1W) assumed to be connected to the 152mm on Bromley Road (see attached PDF for location).

Minimum HGL = 108.6 m
Maximum HGL = 114.6 m
Max Day + Fire Flow (67 L/s) = 98.8 m
These are for current conditions and are based on computer model simulation.
Disclaimer: The boundary condition information is based on current operation of the city water distribution system. The computer model simulation is based on the best information available at the time. The operation of the water distribution system can change on a regular basis, resulting in a variation in boundary conditions. The physical properties of watermains deteriorate over time, as such must be assumed in the absence of actual field test data. The variation in physical watermain properties can therefore alter the results of the computer model simulation.
Thanks
Eric Surprenant, CET
Sr, Project Manager, Infrastructure Projects, West
Planning, Infrastructure & Economic Development
613 580-2424 ext.: 27794

Please take note that due to current COVID situation, I am working remotely and Phone communication and messaging may not be reliable at this time. Preferred method of communications will be e-mails during this period. If your preference is telephone communication, please indicate this via e-mail and provide a contact telephone number.
Absence alert:

I apologize for any inconvenience.

From: Mott, Peter <Peter.Mott@stantec.com>
Sent: April 27, 2021 09:15
To: Surprenant, Eric <Eric.Surprenant@ottawa.ca>
Cc: Kilborn, Kris <kris.kilborn@stantec.com>
Subject: 1951 Carling Avenue - Boundary Conditions Request

I would like to request the hydraulic boundary conditions for the proposed site located at 1951 Carling Avenue. Please find attached the site plan, the key map showing the location of the proposed development, domestic water demand calculations, and fire flow calculations.

A summary of the proposed site is provided below:

We anticipate a connection to the existing watermain infrastructure to service the site. The following connection is expected for servicing:

➢Connection to existing 152 mm (UCI) watermain on Bromley Road.

*Existing fire hydrant adjacent to the property to the south along Carling Avenue.

For the purpose of the boundary conditions request, may you please provide us with the boundary conditions for the following servicing option:

i. Watermain connection to the existing 152 mm (UCI) watermain on Bromley Road; assuming a fire flow requirement of **4,000 L/min** for the site in addition to the domestic water demands provided below.

- The intended land use is residential, per the summary provided in the Domestic Demands spreadsheet. (See attached Site Plan with project stats)
- Estimated fire flow demand per the FUS methodology: 4000 L/min (67 L/s)
- Domestic water demands for the entire development:
  - **Average day: 9.9 L/min (0.20 L/s)**
  - **Maximum day: 24.7 L/min (0.40 L/s)**
  - **Peak hour: 54.3 L/min (0.90 L/s)**

Thank you for your time and please contact me at your earliest convenience if any additional information or clarification is required.

Best regards,

Peter Mott EIT
Engineering Intern, Community Development

Mobile: 613-897-0445
Peter.Mott@stantec.com
Stantec
400 - 1331 Clyde Avenue
Ottawa ON K2C 3G4

Appendix B WASTEWATER SERVICING

B.1 SANITARY SEWER DESIGN SHEET

Stantec

SUBDIVISION:

1951 Carling Avenue

DATE: 6/16/2021
REVISION: 2
DESIGNED BY: TR
CHECKED BY:

SANITARY SEWER DESIGN SHEET
(City of Ottawa)

FILE NUMBER: 160401679

DESIGN PARAMETERS

MAX PEAK FACTOR (RES.)=	4.0	AVG. DAILY FLOW / PERSON	280 l/p/day	MINIMUM VELOCITY	0.60 m/s
MIN PEAK FACTOR (RES.)=	2.0	COMMERCIAL	28,000 l/ha/day	MAXIMUM VELOCITY	3.00 m/s
PEAKING FACTOR (INDUSTRIAL):	2.4	INDUSTRIAL (HEAVY)	55,000 l/ha/day	MANNINGS n	0.013
PEAKING FACTOR (ICI >20%):	1.5	INDUSTRIAL (LIGHT)	35,000 l/ha/day	BEDDING CLASS	B
PERSONS / BACHELOR	1.4	INSTITUTIONAL	28,000 l/ha/day	MINIMUM COVER	2.50 m
PERSONS / 1 BEDROOM	1.4	INFILTRATION	0.33 l/s/Ha	HARMON CORRECTION FACTOR	0.8
PERSONS / 2 BEDROOM	2.1				
PERSONS / 2 BEDROOM	3.1				

LOCATION			RESIDENTIAL AREA AND POPULATION											COMMERCIAL		INDUSTRIAL (L)		INDUSTRIAL (H)	
AREA ID NUMBER	FROM M.H.	TO M.H.	AREA (ha)	BACHELOR	1 BEDROOM	2 BEDROOM	3 BEDROOM	POP.	CUMULATIVE AREA (ha)	CUMULATIVE POP.	PEAK FACT.	PEAK FLOW (l/s)		AREA (ha)	ACCU. AREA (ha)	AREA (ha)	ACCU. AREA (ha)	AREA (ha)	ACCU. AREA (ha)
BLDG	BLDG	TEE	0.034	17	9	1	0	39	0.03	39	4.00	0.50		0.000	0.000	0.00	0.00	0.00	0.00

AREA ID NUMBER	INSTITUTIONAL AREA (ha)	INSTITUTIONAL ACCU. AREA (ha)	GREEN / UNUSED AREA (ha)	GREEN / UNUSED ACCU. AREA (ha)	C+I+I PEAK FLOW (l/s)	INFILTRATION TOTAL AREA (ha)	INFILTRATION ACCU. AREA (ha)	INFILTRATION INFILT. FLOW (l/s)	TOTAL FLOW (l/s)	PIPE LENGTH (m)	PIPE DIA (mm)	PIPE MATERIAL	PIPE CLASS	PIPE SLOPE (%)	PIPE CAP. (FULL) (l/s)	PIPE CAP. V PEAK FLOW (%)	PIPE VEL. (FULL) (m/s)	PIPE VEL. (ACT.) (m/s)
BLDG	0.00	0.00	0.06	0.06	0.00	0.092	0.09	0.03	0.53	18.9	150	PVC	DR 28	1.00	15.3	**3.45%**	0.86	0.41
											225							

Appendix C STORMWATER MANAGEMENT

C.1 STORM SEWER DESIGN SHEET

Stantec	1951 Carling Avenue		STORM SEWER DESIGN SHEET (City of Ottawa)
	DATE:	2021-05-27	
	REVISION:	1	
	DESIGNED BY:	TR	FILE NUMBER: 160401679
	CHECKED BY:		

DESIGN PARAMETERS

$I = a / (t+b)^c$ (As per City of Ottawa Guidelines, 2012)

	1:2 yr	1:5 yr	1:10 yr	1:100 yr					
a =	732.951	998.071	1174.184	1735.688	MANNING'S n =	0.013		BEDDING CLASS	B
b =	6.199	6.053	6.014	6.014	MINIMUM COVER:	2.00	m		
c =	0.810	0.814	0.816	0.820	TIME OF ENTRY	10	min		

LOCATION			DRAINAGE AREA																
AREA ID NUMBER	FROM M.H.	TO M.H.	AREA (2-YEAR) (ha)	AREA (5-YEAR) (ha)	AREA (10-YEAR) (ha)	AREA (100-YEAR (ha)	AREA (ROOF) (ha)	C (2-YEAR) (-)	C (5-YEAR) (-)	C (10-YEAR) (-)	C (100-YEAR (-)	A x C (2-YEAR) (ha)	ACCUM AxC (2YR) (ha)	A x C (5-YEAR) (ha)	ACCUM. AxC (5YR) (ha)	A x C (10-YEAR) (ha)	ACCUM. AxC (10YR) (ha)	A x C (100-YEAR) (ha)	ACCUM. AxC (100YR) (ha)
BLDG, CB-200A, CB-200B	STM 101	STM 100	0.000	0.08	0.00	0.00	0.00	0.00	0.70	0.00	0.00	0.000	0.000	0.053	0.053	0.000	0.000	0.000	0.000

AREA ID NUMBER	T of C (min)	I_{2-YEAR} (mm/h)	I_{5-YEAR} (mm/h)	$I_{10-YEAR}$ (mm/h)	$I_{100-YEAR}$ (mm/h)	$Q_{CONTROL}$ (L/s)	ACCUM. $Q_{CONTROL}$ (L/s)	Q_{ACT} (CIA/360) (L/s)
BLDG, CB-200A, CB-200B	10.00	76.81	104.19	122.14	178.56	0.0	0.0	15.4
	10.22							

	PIPE SELECTION												
AREA ID NUMBER	LENGTH (m)	PIPE WIDTH OR DIAMETER (mm)	PIPE HEIGHT (mm)	PIPE SHAPE (-)	MATERIAL (-)	CLASS (-)	SLOPE %	Q_{CAP} (FULL) (L/s)	% FULL (-)	VEL. (FULL) (m/s)	VEL. (ACT) (m/s)	TIME OF FLOW (min)
BLDG, CB-200A, CB-200B	8.7	300	300	CIRCULAR	PVC	SDR 35	0.50	68.0	**22.7%**	0.97	0.65	0.22
		300	300									

C.2 RATIONAL METHOD CALCULATIONS

Stormwater Management Calculations

File No: **160401667**
Project: **971 Montreal Road**
Date: **16-Jun-21**

SWM Approach:
Post-development to Pre-development flows

Post-Development Site Conditions:

Overall Runoff Coefficient for Site and Sub-Catchment Areas

Runoff Coefficient Table								
Catchment Type	**Sub-catchment Area ID / Description**		**Area (ha) "A"**		**Runoff Coefficient "C"**	**"A x C"**		**Overall Runoff Coefficient**
Roof	ROOF	Hard	0.034		0.9	0.031		
		Soft	0.000		0.2	0.000		
	Subtotal			0.034498			0.0310482	0.90
Controlled - Tributary	CB-200A, CB-200B	Hard	0.020		0.9	0.018		
		Soft	0.021		0.2	0.004		
	Subtotal			0.04134			0.022212	0.54
Uncontrolled -Tributary	UNC-1	Hard	0.009		0.9	0.008		
		Soft	0.007		0.2	0.001		
	Subtotal			0.016233			0.0095775	0.59
Total				**0.092**			**0.063**	
Overall Runoff Coefficient= C:								**0.68**

Total Roof Areas	**0.034 ha**
Total Tributary Surface Areas (Controlled and Uncontrolled)	**0.058 ha**
Total Tributary Area to Outlet	**0.092 ha**
Total Uncontrolled Areas (Non-Tributary)	**0.000 ha**
Total Site	**0.092 ha**

Stormwater Management Calculations

Project #160401667, 971 Montreal Road
Modified Rational Method Calculatons for Storage

5 yr Intensity $I = a/(t + b)^c$
City of Ottawa

a =	998.071
b =	6.053
c =	0.814

t (min)	I (mm/hr)
10	104.19
20	70.25
30	53.93
40	44.18
50	37.65
60	32.94
70	29.37
80	26.56
90	24.29
100	22.41
110	20.82
120	19.47

5 YEAR Predevelopment Target Release from Portion of Site

Subdrainage Area: Predevelopment Tributary Area to Outlet
Area (ha): 0.0921
C: 0.50

Typical Time of Concentration

tc (min)	I (5 yr) (mm/hr)	Qtarget (L/s)
10	104.19	**13.33**

5 YEAR Modified Rational Method for Entire Site

Subdrainage Area: ROOF — Roof
Area (ha): 0.034 — Maximum Storage Depth: 150 mm
C: 0.90

tc (min)	I (5 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)	Depth (mm)	
10	104.19	8.99	3.08	5.91	3.55	94.3	0.00
20	70.25	6.06	3.09	2.98	**3.57**	94.6	0.00
30	53.93	4.65	3.01	1.65	2.97	88.2	0.00
40	44.18	3.81	2.90	0.91	2.19	79.9	0.00
50	37.65	3.25	2.77	0.48	1.45	69.3	0.00
60	32.94	2.84	2.61	0.23	0.84	56.8	0.00
70	29.37	2.54	2.42	0.11	0.48	48.0	0.00
80	26.56	2.29	2.21	0.08	0.40	43.8	0.00
90	24.29	2.10	2.03	0.06	0.34	40.3	0.00
100	22.41	1.93	1.89	0.05	0.29	37.4	0.00
110	20.82	1.80	1.76	0.04	0.24	34.9	0.00
120	19.47	1.68	1.65	0.03	0.20	32.7	0.00

Storage: Roof Storage

	Depth (mm)	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Discharge Check
5-year Water Level	95	0.09	3.09	3.57	13.80	0.00

Subdrainage Area: CB-200A, CB-200B — Controlled - Tributary
Area (ha): 0.041
C: 0.54

tc (min)	I (5 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)
10	104.19	9.52	4.92	4.60	2.76
20	70.25	7.42	5.04	2.38	**2.86**
30	53.93	6.34	4.84	1.49	2.69
40	44.18	5.63	4.60	1.03	2.48
50	37.65	5.09	4.34	0.75	2.25
60	32.94	4.64	4.08	0.56	2.02
70	29.37	4.24	3.81	0.42	1.78
80	26.56	3.85	3.53	0.32	1.55
90	24.29	3.53	3.28	0.25	1.35
100	22.41	3.27	3.07	0.20	1.19
110	20.82	3.05	2.89	0.16	1.06
120	19.47	2.85	2.72	0.13	0.95

Orifice Diameter: LMF100
Invert Elevation 80.12 m
T/G Elevation 82.04 m
Max Ponding Depth 0.32 m
Downstream W/L 79.80 m

	Stage	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Volume Check
5-year Water Level	69.26	0.32	5.04	2.86	7.24	OK

Subdrainage Area: UNC-1 — Uncontrolled -Tributary
Area (ha): 0.016
C: 0.59

tc (min)	I (5 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)
10	104.19	2.77	2.77		
20	70.25	1.87	1.87		
30	53.93	1.44	1.44		
40	44.18	1.18	1.18		
50	37.65	1.00	1.00		
60	32.94	0.88	0.88		
70	29.37	0.78	0.78		
80	26.56	0.71	0.71		
90	24.29	0.65	0.65		
100	22.41	0.60	0.60		
110	20.82	0.55	0.55		
120	19.47	0.52	0.52		

Project #160401667, 971 Montreal Road
Modified Rational Method Calculatons for Storage

100 yr Intensity $I = a/(t + b)^c$
City of Ottawa

a =	1735.688
b =	6.014
c =	0.820

t (min)	I (mm/hr)
10	178.56
20	119.95
30	91.87
40	75.15
50	63.95
60	55.89
70	49.79
80	44.99
90	41.11
100	37.90
110	35.20
120	32.89

100 YEAR Predevelopment Target Release from Portion of Site

Subdrainage Area: Predevelopment Tributary Area to Outlet
Area (ha): 0.0921
C: 0.50

100 YEAR Modified Rational Method for Entire Site

Subdrainage Area: ROOF — Roof
Area (ha): 0.034 — Maximum Storage Depth: 150 mm
C: 1.00

tc (min)	I (100 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)	Depth (mm)	
10	178.56	17.12	3.48	13.64	8.19	125.9	0.00
20	119.95	11.50	3.55	7.95	**9.54**	131.7	0.00
30	91.87	8.81	3.55	5.26	9.47	131.4	0.00
40	75.15	7.21	3.52	3.69	8.85	128.7	0.00
50	63.95	6.13	3.47	2.66	7.99	125.0	0.00
60	55.89	5.36	3.40	1.96	7.07	119.1	0.00
70	49.79	4.78	3.32	1.46	6.12	113.0	0.00
80	44.99	4.31	3.24	1.07	5.16	106.8	0.00
90	41.11	3.94	3.16	0.78	4.20	100.7	0.00
100	37.90	3.64	3.07	0.57	3.42	92.9	0.00
110	35.20	3.38	2.97	0.41	2.69	85.2	0.00
120	32.89	3.15	2.88	0.28	2.00	78.0	0.00

Storage: Roof Storage

	Depth (mm)	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Discharge Check
100-year Water Level	132	0.13	3.55	9.54	13.80	0.00

Subdrainage Area: CB-200A, CB-200B — Controlled - Tributary
Area (ha): 0.041
C: 0.67

tc (min)	I (100 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)
10	178.56	17.26	7.21	10.05	6.03
20	119.95	12.81	7.47	5.34	**6.41**
30	91.87	10.64	7.25	3.39	6.10
40	75.15	9.32	6.96	2.36	5.66
50	63.95	8.41	6.67	1.73	5.20
60	55.89	7.71	6.39	1.32	4.76
70	49.79	7.16	6.13	1.04	4.35
80	44.99	6.71	5.88	0.83	3.99
90	41.11	6.34	5.64	0.69	3.74
100	37.90	5.99	5.39	0.60	3.61
110	35.20	5.69	5.16	0.52	3.46
120	32.89	5.42	4.96	0.46	3.29

Orifice Diameter: LMF100
Invert Elevation 80.12 m
T/G Elevation 82.04 m
Max Storage Depth 0.70 m
Downstream W/L 79.80 m

Volume in CB-1 and CB-2 when head = 0.70 — 0.50
Max available volume in CB's — 0.72

	Stage	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Volume Check
100-year Water Level	69.65	0.70	7.47	6.41	7.24	OK
					0.83	

Subdrainage Area: UNC-1 — Uncontrolled -Tributary
Area (ha): 0.016
C: 0.74

tc (min)	I (100 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)
10	178.56	5.94	5.94		
20	119.95	3.99	3.99		
30	91.87	3.06	3.06		
40	75.15	2.50	2.50		
50	63.95	2.13	2.13		
60	55.89	1.86	1.86		
70	49.79	1.66	1.66		
80	44.99	1.50	1.50		
90	41.11	1.37	1.37		
100	37.90	1.26	1.26		
110	35.20	1.17	1.17		
120	32.89	1.09	1.09		

Stormwater Management Calculations

Project #160401667, 971 Montreal Road
Modified Rational Method Calculatons for Storage

SUMMARY TO OUTLET

			Vrequired	Vavailable*		
Tributary Area	0.092	ha				
Total 5yr Flow to Sewer	7.82	L/s	0	0	m^3	Ok
Non-Tributary Area	0.000	ha				
Total 5yr Flow Uncontrolled	0.00	L/s				
Total Area	0.092	ha				
Total 5yr Flow	7.82	L/s				
Target	13.33	L/s				

Project #160401667, 971 Montreal Road
Modified Rational Method Calculatons for Storage

SUMMARY TO OUTLET

			Vrequired	Vavailable*		
Tributary Area	0.092	ha				
Total 100yr Flow to Sewer	13.42	L/s	6.41	7.24	m^3	Ok
Non-Tributary Area	0.000	ha				
Total 100yr Flow Uncontrolled	0.00	L/s				
Total Area	0.092	ha				
Total 100yr Flow	13.42	L/s				
Target	13.33	L/s				

Project #160401667, 971 Montreal Road
Roof Drain Design Sheet, Area BLDG
Standard Watts Model R1100 Accutrol Roof Drain

Rating Curve				Volume Estimation				
Elevation (m)	Discharge Rate (cu.m/s)	Outlet Discharge (cu.m/s)	Storage (cu. m)	Elevation (m)	Area (sq. m)	Volume (cu. m) Increment	Volume (cu. m) Accumulated	Water Depth (m)
0.000	0.0000	0.0000	0	0.000	0	0	0	0.000
0.025	0.0003	0.0013	0	0.025	8	0	0	0.025
0.050	0.0006	0.0025	1	0.050	31	0	1	0.050
0.075	0.0007	0.0028	2	0.075	69	1	2	0.075
0.100	0.0008	0.0032	4	0.100	123	2	4	0.100
0.125	0.0009	0.0035	8	0.125	192	4	8	0.125
0.150	0.0009	0.0038	14	0.150	276	6	14	0.150

Drawdown Estimate			
Total Volume (cu.m)	Total Time (sec)	Vol (cu.m)	Detention Time (hr)
0.0	0.0	0.0	0
0.4	177.2	0.4	0.04922
1.7	427.5	1.2	0.16799
4.0	749.3	2.4	0.37613
7.9	1123.1	3.9	0.68809
13.7	1535.8	5.8	1.1147

Rooftop Storage Summary

Total Building Area (sq.m)		344.98
Assume Available Roof Area (sq.	80%	275.984
Roof Imperviousness		0.99
Roof Drain Requirement (sq.m/Notch)		232
Number of Roof Notches*		4
Max. Allowable Depth of Roof Ponding (m)		0.15
Max. Allowable Storage (cu.m)		14
Estimated 100 Year Drawdown Time (h)		0.8

* As per Ontario Building Code section OBC 7.4.10.4.(2)(c).

* Note: Number of drains can be reduced if multiple-notch drain used.

From Watts Drain Catalogue

Head (m) L/s

Head (m)	Open	75%	50%	**25%**	Closed
0.025	0.3155	0.3155	0.3155	**0.3155**	0.3155
0.050	0.6309	0.6309	0.6309	**0.6309**	0.3155
0.075	0.9464	0.8675	0.7886	**0.7098**	0.3155
0.100	1.2618	1.1041	0.9464	**0.7886**	0.3155
0.125	1.5773	1.3407	1.1041	**0.8675**	0.3155
0.150	0	1.5773	1.2618	**0.9464**	0.3155

Calculation Results

	5yr	100yr	Available
Qresult (cu.m/s)	0.003	0.004	-
Depth (m)	0.095	0.132	0.150
Volume (cu.m)	3.6	9.5	13.8
Draintime (hrs)	0.3	0.8	

C.3 OIL/ GRIT SEPERATOR SIZING

Project Summary Report: 1983 Carling Avenue Stormceptor Sizing

Project Information & Location			
Project Name	1983 Carling Avenue	**Project Number**	160401679
City	Ottawa	**State/ Province**	Ontario
Country	Canada	**Date**	5/30/2021
Designer Information		**EOR Information (optional)**	
Name	thakshika rathnasooriya	**Name**	
Company	stantec	**Company**	
Phone #	613-724-4081	**Phone #**	
Email	thakshika.rathnasooriya@stantec.com	**Email**	

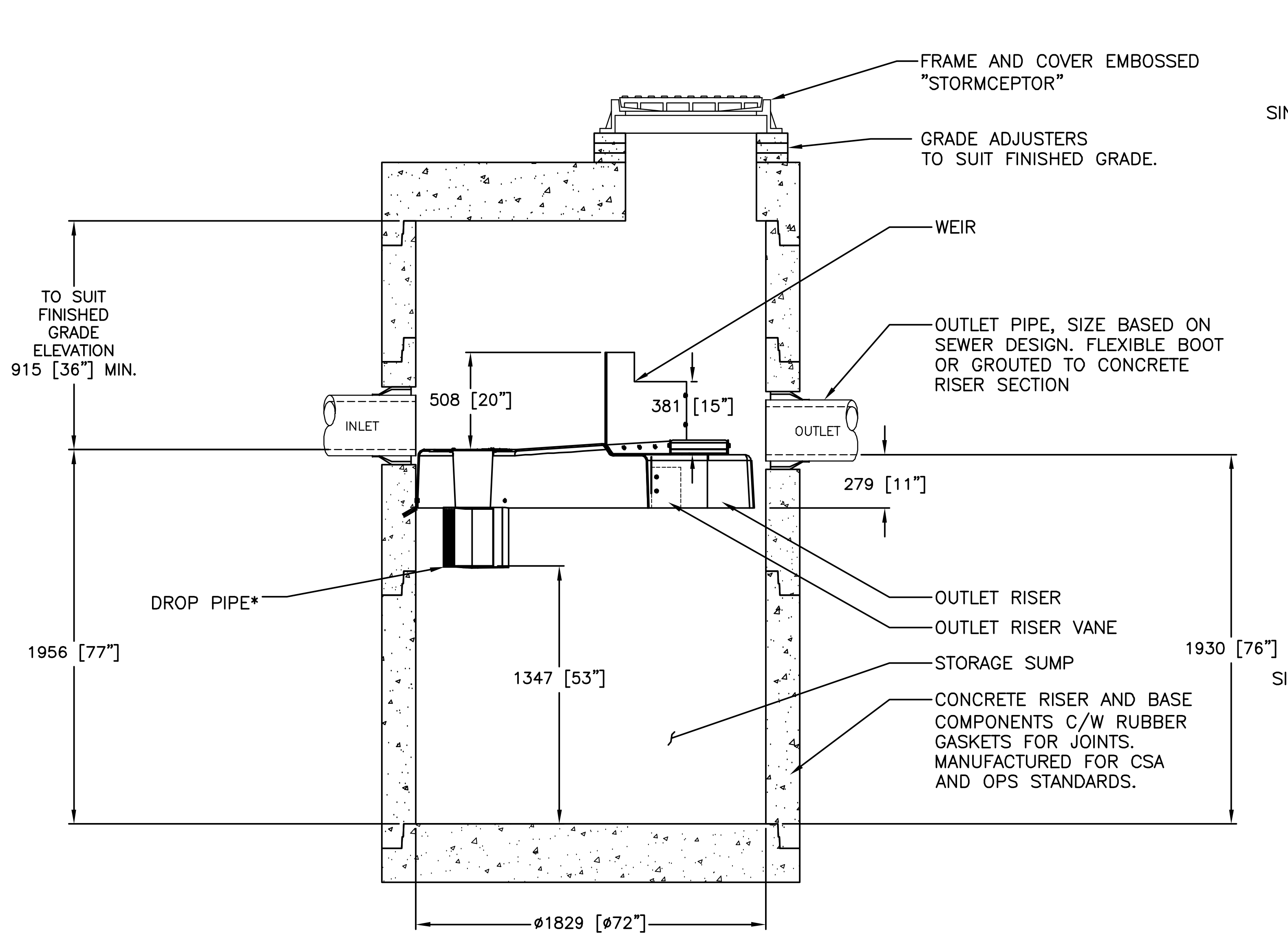
Stormwater Treatment Recommendation

The recommended Stormceptor Model(s) which achieve or exceed the user defined water quality objective for each site within the project are listed in the below Sizing Summary table.

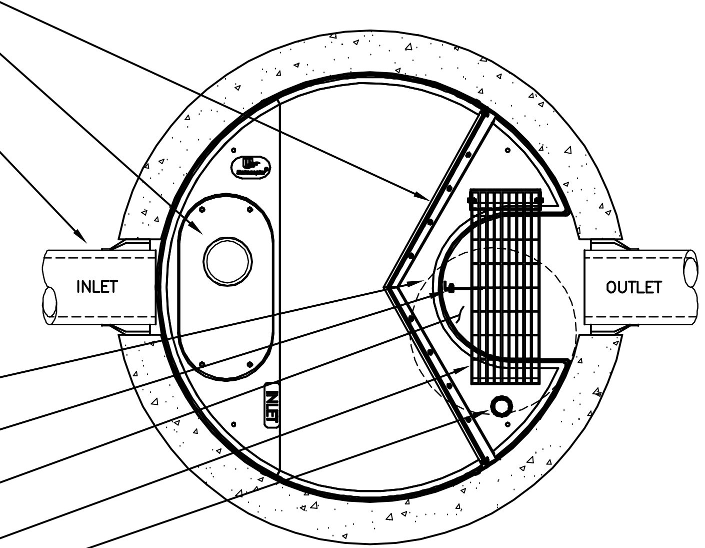
Project Summary						
Site Name	**Drainage Area (ha)**	**Imperviousness %**	**PSD**	**Target TSS Removal (%)**	**TSS Removal (%) Provided**	**Recommended Model**
1983 Carling Avenue	0.92	0.68		80	83	EF6

Notes
• Stormceptor performance estimates are based on simulations using PCSWMM for Stormceptor, which uses the EPA Rainfall and Runoff modules. • Design estimates listed are only representative of specific project requirements based on total suspended solids (TSS) removal defined by the selected PSD, and based on stable site conditions only, after construction is completed. • For submerged applications or sites specific to spill control, please contact your local Stormceptor representative for further design assistance.

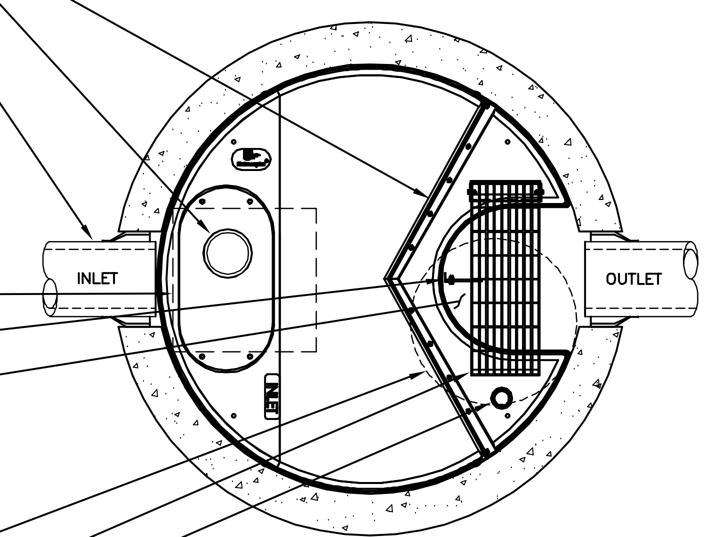
DRAWING NOT TO BE USED FOR CONSTRUCTION



SECTION VIEW



PLAN VIEW (STANDARD)



PLAN VIEW (INLET TOP)

GENERAL NOTES:

* MAXIMUM SURFACE LOADING RATE (SLR) INTO LOWER CHAMBER THROUGH DROP PIPE IS 1135 L/min/m^2 (27.9 gpm/ft^2) FOR STORMCEPTOR EF6 AND 535 L/min/m^2 (13.1 gpm/ft^2) FOR STORMCEPTOR EFO6 (OIL CAPTURE CONFIGURATION).

1. ALL DIMENSIONS INDICATED ARE IN MILLIMETERS (INCHES) UNLESS OTHERWISE SPECIFIED.
2. STORMCEPTOR STRUCTURE INLET AND OUTLET PIPE SIZE AND ORIENTATION SHOWN FOR INFORMATIONAL PURPOSES ONLY.
3. UNLESS OTHERWISE NOTED, BYPASS INFRASTRUCTURE, SUCH AS ALL UPSTREAM DIVERSION STRUCTURES, CONNECTING STRUCTURES, OR PIPE CONDUITS CONNECTING TO COMPLETE THE STORMCEPTOR SYSTEM SHALL BE PROVIDED AND ADDRESSED SEPARATELY.
4. DRAWING FOR INFORMATION PURPOSES ONLY. REFER TO ENGINEER'S SITE/UTILITY PLAN FOR STRUCTURE ORIENTATION.
5. NO PRODUCT SUBSTITUTIONS SHALL BE ACCEPTED UNLESS SUBMITTED 10 DAYS PRIOR TO PROJECT BID DATE, OR AS DIRECTED BY THE ENGINEER OF RECORD.

FOR SITE SPECIFIC DRAWINGS PLEASE CONTACT YOUR LOCAL STORMCEPTOR REPRESENTATIVE. SITE SPECIFIC DRAWINGS ARE BASED ON THE BEST AVAILABLE INFORMATION AT THE TIME. SOME FIELD REVISIONS TO THE SYSTEM LOCATION OR CONNECTION PIPING MAY BE NECESSARY BASED ON AVAILABLE SPACE OR SITE CONFIGURATION REVISIONS. ELEVATIONS SHOULD BE MAINTAINED EXCEPT WHERE NOTED ON BYPASS STRUCTURE (IF REQUIRED).

INSTALLATION NOTES

A. ANY SUB-BASE, BACKFILL DEPTH, AND/OR ANTI-FLOTATION PROVISIONS ARE SITE-SPECIFIC DESIGN CONSIDERATIONS AND SHALL BE SPECIFIED BY ENGINEER OF RECORD.
B. CONTRACTOR TO PROVIDE EQUIPMENT WITH SUFFICIENT LIFTING AND REACH CAPACITY TO LIFT AND SET THE STRUCTURE (LIFTING CLUTCHES PROVIDED)
C. CONTRACTOR WILL INSTALL AND LEVEL THE STRUCTURE, SEALING THE JOINTS, LINE ENTRY AND EXIT POINTS (NON-SHRINK GROUT WITH APPROVED WATERSTOP OR FLEXIBLE BOOT)
D. CONTRACTOR TO TAKE APPROPRIATE MEASURES TO PROTECT THE DEVICE FROM CONSTRUCTION-RELATED EROSION RUNOFF.
E. DEVICE ACTIVATION, BY CONTRACTOR, SHALL OCCUR ONLY AFTER SITE HAS BEEN STABILIZED AND THE STORMCEPTOR UNIT IS CLEAN AND FREE OF DEBRIS.

STANDARD DETAIL
NOT FOR CONSTRUCTION

SITE SPECIFIC DATA REQUIREMENTS

STORMCEPTOR MODEL	EF6				
STRUCTURE ID					*
WATER QUALITY FLOW RATE (L/s)					*
PEAK FLOW RATE (L/s)					*
RETURN PERIOD OF PEAK FLOW (yrs)					*
DRAINAGE AREA (HA)					*
DRAINAGE AREA IMPERVIOUSNESS (%)					*
PIPE DATA:	I.E.	MAT'L	DIA	SLOPE %	HGL
INLET #1	*	*	*	*	*
INLET #2	*	*	*	*	*
OUTLET	*	*	*	*	*

* PER ENGINEER OF RECORD

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MARK	DATE	REVISION DESCRIPTION	BY
####	####	####	####
####	####	####	####
####	####	####	####
1	6/8/18	OUTLET PLATFORM	JSK
0	05/26/17	INITIAL RELEASE	JSK

Stormceptor® EF

SCALE = NTS

imbrium®

DATE: 5/26/2017	
DESIGNED: JSK	DRAWN: JSK
CHECKED: BSF	APPROVED: SP
PROJECT No.: EF6	SEQUENCE No.: *
SHEET: 1 OF 1	

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Appendix D GEOTECHNICAL INVESTIGATION

FINAL

Geotechnical Investigation – Proposed Residential Development

1983 Carling Avenue, Ottawa, Ontario

Prepared for:

2473493 Ontario Inc.
485 Pinebush Road, Suite 102
Cambridge, ON N1T 0A6

May 4, 2021

Pinchin File: 289578

Issued to:	**2473493 Ontario Inc.**
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APPENDICES

1.0 INTRODUCTION AND SCOPE

Pinchin Ltd. (Pinchin) was retained by 2473493 Ontario Inc. (Client) to conduct a Geotechnical Investigation and provide subsequent geotechnical design recommendations for the proposed residential development to be located at 1983 Carling Avenue, Ottawa, Ontario (Site). The Site location is shown on Figure 1.

Based on information provided by the Client, it is Pinchin's understanding that the proposed development is to consist of a five-storey, slab-on-grade (i.e. no basement level) residential apartment building complete with new Site services. The proposed development does not include asphalt surfaced areas.

Pinchin's geotechnical comments and recommendations are based on the results of the Geotechnical Investigation and our understanding of the project scope.

The purpose of the Geotechnical Investigation was to delineate the subsurface conditions and soil engineering characteristics by advancing a total of four (4) sampled boreholes (Boreholes BH1 to BH4), at the Site. The information gathered from the Geotechnical Investigation will allow Pinchin to provide geotechnical design recommendations for the proposed development.

Based on a desk top review and the results of the Geotechnical Investigation, the following geotechnical data and engineering design recommendations are provided herein:

- A detailed description of the soil and groundwater conditions;
- Site preparation recommendations;
- Open cut excavations;
- Anticipated groundwater management;
- Site service trench design;
- Foundation design recommendations including bedrock bearing resistances at Ultimate Limit States (ULS) design;
- Potential total and differential settlements;
- Foundation frost protection and engineered fill specifications and installation;
- Seismic Site classification for seismic Site response;
- Concrete floor slab-on-grade support recommendations; and
- Potential construction concerns.

Abbreviations terminology and principle symbols commonly used throughout the report, borehole logs and appendices are enclosed in Appendix I.

2.0 SITE DESCRIPTION AND GEOLOGICAL SETTING

The Site is located on the north side of Carling Avenue, approximately 800 metres north of Highway 417 in Ottawa, Ontario. The Site is currently undeveloped and consists of an asphalt and gravel surfaced parking area with a small section of soft landscaping on the south portion of the Site. The lands adjacent to the Site are developed with a combination of single family and multi unit residential buildings.

Data obtained from the Ontario Geological Survey Maps, as published by the Ontario Ministry of Natural Resources, indicates that the Site is located on a Paleozoic bedrock. The underlying bedrock at this Site is of the Shadow Lake Formation consisting of limestone, dolostone, shale, arkose, and sandstone (Ontario Geological Survey Map 1972, published 1978).

3.0 GEOTECHNICAL FIELD INVESTIGATION AND METHODOLOGY

Pinchin completed a field investigation at the Site on March 29, 2021 by advancing a total of four sampled boreholes throughout the Site. The boreholes were advanced to depths ranging from approximately 0.8 to 0.9 metres below existing ground surface (mbgs), where refusal was encountered on probable bedrock. The approximate spatial locations of the boreholes advanced at the Site are shown on Figure 2.

The boreholes were advanced with the use of a Geoprobe 7822 DT direct push drill rig which was equipped with standard soil sampling equipment. Soil samples were collected at 0.76 m intervals using a 51 mm outside diameter (OD) split spoon barrel in conjunction with Standard Penetration Tests (SPT) “N” values (ASTM D1586). The SPT “N” values were used to assess the compactness condition of the non-cohesive soil.

Groundwater observations and measurements were obtained from the open boreholes during and upon completion of drilling. The groundwater observations and measurements recorded are included on the appended borehole logs.

The borehole locations and ground surface elevations were located at the Site by Pinchin personnel. The ground surface elevation at each borehole location was referenced to the following temporary benchmark as shown on Figure 2:

- TBM: Top of the southwest corner of the exposed portion of the foundation wall of the adjacent building to the east, at the approximate location shown on Figure 2; and
- Elevation: 100.0 metres (local datum).

The field investigation was monitored by experienced Pinchin personnel. Pinchin logged the drilling operations and identified the soil samples as they were retrieved. The recovered soil samples were sealed into plastic bags and carefully transported to an independent and accredited materials testing laboratory for detailed analysis and testing. All soil samples were classified according to visual and index properties by the project engineer.

The field logging of the soil and groundwater conditions was performed to collect geotechnical engineering design information. The borehole logs include textural descriptions of the subsoil in accordance with a modified Unified Soil Classification System (USCS) and indicate the soil boundaries inferred from non-continuous sampling and observations made during the borehole advancement. These boundaries reflect approximate transition zones for the purpose of geotechnical design and should not be interpreted as exact planes of geological change. The modified USCS classification is explained in further detail in Appendix I. Details of the soil and groundwater conditions encountered within the boreholes are included on the Borehole Logs within Appendix II.

Select soil samples collected from the boreholes were submitted to a material testing laboratory to determine the grain size distribution of the soil. A copy of the laboratory analytical reports is included in Appendix III. In addition, the collected samples were compared against previous geotechnical information from the area, for consistency and calibration of results.

4.0 SUBSURFACE CONDITIONS

4.1 Borehole Soil Stratigraphy

In general, the soil stratigraphy at the Site comprises either surficial asphalt or surficial granular fill overlying glacial till and probable bedrock to the maximum borehole refusal depth of approximately 0.9 mbgs. The appended borehole logs provide detailed soil descriptions and stratigraphies, results of SPT testing, and groundwater measurements.

The surficial asphalt was encountered within Boreholes BH2 and BH3 and was measured to be approximately 25 mm thick.

Granular fill was encountered at the surface in Boreholes BH1 and BH4 and underlying the surficial asphalt in Boreholes BH2 and BH3. The fill material was measured to range in thickness from approximately 0.5 to 0.8 m thick and ranged in soil matrix from sand and gravel containing trace silt, to gravelly sand containing trace silt. The non-cohesive material had a very loose to very dense relative density based SPT ‘N’ values of between 3 and greater than 50 blows per 300 mm penetration of a split spoon sampler. The results of two particle size distribution analyses completed on samples of the fill material indicate that the samples contained approximately 31 to 49% gravel, 43 to 59% sand, and 8 to 10% silt sized particles.

The glacial till was encountered underlying the granular fill in Borehole BH1 at approximately 0.5 mbgs and was measured to be approximately 0.4 m thick. The glacial till comprised silty clayey sand containing some gravel. The non-cohesive glacial till had a loose to very dense relative density based SPT 'N' values of 8 to greater than 50 blows per 300 mm penetration of a split spoon sampler. The result of one particle size distribution analysis completed on a sample of the glacial till indicates that the sample contains approximately 11% gravel, 38% sand, 28% silt, and 23% clay sized particles. The moisture content of the material tested was 24.5%, indicating the material was in a damp to moist condition at the time of sampling.

4.2 Groundwater Conditions

Groundwater observations and measurements were obtained in the open boreholes at the completion of drilling and are summarized on the appended borehole logs. Groundwater was not encountered within the open boreholes at drilling completion. Seasonal variations in the water table should be expected, with higher levels occurring during wet weather conditions in the spring and fall and lower levels occurring during dry weather conditions.

5.0 GEOTECHNICAL DESIGN RECOMMENDATIONS

5.1 General Information

The recommendations presented in the following sections of this report are based on the information available regarding the proposed construction, the results obtained from the geotechnical investigation, and Pinchin's experience with similar projects. Since the investigation only represents a portion of the subsurface conditions, it is possible that conditions may be encountered during construction that are substantially different than those encountered during the investigation. If these situations are encountered, adjustments to the design may be necessary. A qualified geotechnical engineer should be on-Site during the foundation preparation to ensure the subsurface conditions are the same/similar to what was observed during the investigation.

Based on information provided by the Client, it is Pinchin's understanding that the proposed development is to consist of a five-storey, slab-on-grade (i.e. no basement level) residential apartment building complete with new Site services. The proposed development does not include asphalt surfaced areas.

Probable bedrock was encountered between approximately 0.8 and 0.9 mbgs with the boreholes advanced at the Site. As such, Pinchin recommends to construct the proposed building on conventional shallow strip and spread footings founded on the underlying bedrock surface.

5.2 Site Preparation

Preparation of the Site for the proposed development will consist of removing all surficial and overburden materials down to the underlying bedrock surface in the vicinity of the proposed building footprint (below the foundations and floor slabs).

Prior to placing any fill material at the Site, the bedrock and/or subgrade soil should be inspected by a qualified geotechnical engineer and loosened/soft pockets should be sub excavated and replaced with an engineered fill. All fill material to raise grades below the floor slab is to be installed in maximum 200 mm thick loose lifts, compacted to 98% of its Standard Proctor Maximum Dry Density (SPMDD), within plus 2 to minus 4 of the optimum moisture contents. It is recommended that the floor slab subgrade fill comprise Ontario Provincial Standard Specification 1010 (OPSS 1010) Granular 'B' Type I or Type II material.

A qualified geotechnical engineering technician should be on site to observe fill placement operations and perform field density tests at random locations throughout each lift, to indicate the specified compaction is being achieved.

5.3 Open Cut Excavations and Anticipated Groundwater Management

Excavations for the building foundations will extend to an approximate depth of 0.8 to 0.9 mbgs, while excavations for the new Site services could potentially extend upwards of 2.1 mbgs, depending on the depth of the existing services in the vicinity of the Site that the new services will connect to. As such, a portion of the bedrock will need to be removed to accommodate the new Site services.

Based on the subsurface information obtained from within the boreholes, it is anticipated that the excavated material will predominately consist of granular fill and glacial till. Groundwater was not encountered within the boreholes at drilling completion and is not expected to be encountered in the overburden material during excavations. It is noted that the boreholes did not advance into the bedrock; as such, there is a potential for groundwater to be encountered during excavations into the bedrock.

Where workers must enter trench excavations deeper than 1.2 m, the trench excavations should be suitably sloped and/or braced in accordance with the Occupational Health and Safety Act (OHSA), Ontario Regulation 213/91, Construction Projects, July 1, 2011, Part III - Excavations, Section 226. Alternatively, the excavation walls may be supported by either closed shoring, bracing, or trench boxes complying with sections 235 to 239 and 241 under O. Reg. 231/91, s. 234(1). The use of trench boxes can most likely be used for temporary support of vertical side walls. The appropriate trench should be designed/confirmed for use in this soil deposit.

Based on the OHSA, the natural subgrade soils would be classified as Type 3 soil and temporary excavations in these soils must be sloped at an inclination of 1 horizontal to 1 vertical (H to V) from the base of the excavation.

The upper approximate 1.5 m of bedrock in this area is typically weathered and can usually be removed with mechanical equipment, such as a large excavator and hydraulic hammer (hoe ram) and where required, with line drilling on close centres. Often a hydraulic hammer can be utilized to create an initial opening for the excavator bucket to gain access of the layered rock. The bedrock is known to contain vertical joints and near horizontal bedding planes. Therefore, some vertical and horizontal over break of the bedrock should be expected.

Depending on the ability of the mechanical equipment to advance through the bedrock, drilling and blasting may be required. It is often difficult to blast “neat” lines using conventional drilling and blasting procedures, as such, problems with “over break” are common. This may affect quantities claimed by the contractor for rock excavations, as well as the potential for off-site disposal of the blasted rock, if necessary. Allowances should be made for over break conditions. Due consideration should also be given to controlled blasting procedures in order to prevent potential damage to the surrounding environment.

In addition, we recommend that a pre-blast survey of all neighbouring properties be undertaken prior to conducting drilling and blasting activities. The preconstruction survey will serve to protect the Client from claims unrelated to the construction activities in the development of this property.

Pinchin notes that, local contractors are familiar with excavating the local bedrock and have specialized knowledge and techniques for its removal. Depending on the block size and degree of weathering of the rock they may have a different approach than what is presented in the preceding paragraphs.

Construction slopes in intact bedrock should stand near vertical provided the “loose” rock is properly scaled off the face. Once the blasting is completed, if there are any permanent bedrock shear walls, they will have to be reviewed by a Rock Mechanics Specialist to determine if it is stable or if it needs reinforcing, such as rock bolting.

In addition to compliance with the OHSA, the excavation procedures must also comply to any potential other regulatory authorities, such as federal and municipal safety standards.

As previously mentioned, there is a potential for groundwater to be encountered during excavations into the bedrock. It is believed that this groundwater inflow can be controlled using a gravity dewatering system with perimeter interceptor ditches and high capacity pumps. It is noted that once the final grades have been set, Pinchin should review this recommendation and revise as necessary.

Seasonal variations in the water table should be expected, with higher levels occurring during wet weather conditions in the spring and fall and lower levels occurring during dry weather conditions. If construction commences during wet periods (typically spring or fall), there is a greater potential that the groundwater elevation could be higher and/or perched groundwater may be present. Any potential precipitation of perched groundwater should be able to be controlled from pumping from filtered sumps.

Prior to commencing excavations, it is critical that all existing surface water and potential surface water is controlled and diverted away from the Site to prevent infiltration and subgrade softening. At no time should excavations be left open for a period of time that will expose them to precipitation and cause subgrade softening.

All collected water is to discharge a sufficient distance away from the excavation to prevent re-entry. Sediment control measures, such as a silt fence should be installed at the discharge point of the dewatering system. The utmost care should be taken to avoid any potential impacts on the environment.

It is the responsibility of the contractor to propose a suitable dewatering system based on the groundwater elevation at the time of construction. The method used should not adversely impact any nearby structures. Excavations to conventional design depths for the building foundations are not expected to require a Permit to Take Water or a submission to the Environmental Activity and Sector Registry (EASR). It is the responsibility of the contractor to make this application if required.

5.4 Site Servicing

5.4.1 Pipe Bedding and Cover Materials for Flexible and Rigid Pipes

The subgrade conditions beneath the Site services will consist of bedrock. No support problems are anticipated for flexible or rigid pipes founded on the bedrock. Service pipes require an adequate base to ensure proper pipe connection and positive flow is maintained post construction. As such, pipe bedding should be placed to be of uniform thickness and compactness. The pipe bedding and cover material should conform to OPSD 802.010 and 802.013 specifications for flexible pipes and to OPSD 802.031 to 802.033 with Class ‘B’ bedding for rigid pipes.

For pipes installed within bedrock trenches, the following is recommended:

- Install 300 mm of 19 mm clear stone gravel (OPSS 1004) or Granular ‘A’ (OPSS 1010) below the pipe extending up the sides to the spring line;
- If clear stone is used as bedding material, then a non-woven geotextile (Terrafix 360R or equivalent) is to be placed over the clear stone and pipe extending up vertically along the side walls of the bedrock and pipe a minimum distance of 500 mm;

- The pipe cover material should consist of either a Granular ‘B’ Type I (OPSS 1010) with a maximum particle diameter size of 26.5 mm or bedding sand and should extend to a minimum of 300 mm above the top of the pipe; and
- If rock shatter is present a non-woven geotextile (Terrafix 360R or equivalent) may be required to prevent the migration of fines from the bedding material into the rock shatter. Where blasting is required for Site services, over blast of at least 600 mm of rock shatter should be performed. Over blast material may stay in the trench.

All granular fill material is to be placed in maximum 200 mm thick loose lifts compacted to a minimum of 98% SPMDD.

If constant groundwater infiltration becomes an issue, then an approximate 150 mm granular pad consisting of 19 mm clear stone gravel (OPSS 1004) wrapped in a non-woven geotextile (Terrafix 270R or equivalent) should be considered. The clear stone should contain a minimum of 50% crushed particles. Water collected within the stone should be controlled through sumps and filtered pumps.

5.4.2 Trench Backfill

Where the adjacent material consists of bedrock, the trench can be backfilled with well graded blast rock fill, with a gradation similar to OPSS 1010 Granular ‘B’ Type I. The soil should be placed to the underside of the granular subbase of the pavement structure and be compacted in maximum 300 mm thick lifts to 98% SPMDD within 4% of the optimum moisture content. This is recommended to provide soil compatibility and help minimize potential abrupt differential frost heave between surrounding natural materials similar in composition.

All stockpiled material should be protected from deleterious materials, additional moisture and be kept from freezing.

Quality control will be the utmost importance when selecting the material. The selection of the material should be done as early in the contract as possible to allow sufficient time for gradation and proctor testing on representative samples to ensure it meets the projects specifications.

It is anticipated that imported material will be required to backfill the trenches due to minimal amount of natural soil observed at the Site. Imported material should consist of a Granular ‘A’, Granular ‘B’ Type I, or Select Subgrade Material (OPSS 1010). Heavy construction equipment and truck traffic should not cross any pipe until at least 1 m of compacted soil is placed above the top of the pipe.

Post compaction settlement of finer grained soil can be expected, even when placed to compaction specifications. As such, fill materials should be installed as far in advance as possible before finishing the roadway in order to mitigate post compaction settlements.

5.4.3 Frost Protection

The frost penetration depth in Ottawa, Ontario is estimated to extend to approximately 1.8 mbgs in open roadways cleared of snow. As such, it is recommended to place water services at a minimum depth of 300 mm below this elevation with the top of the pipe located at 2.1 mbgs or lower as dictated by municipal service requirements. If a minimum of 2.1 m of soil cover cannot be provided, then the pipe should be insulated with a rigid polystyrene insulation (DOW Styrofoam HI40, or equivalent) or a pre-insulated pipe be utilized.

The insulation design configuration may either consist of placing horizontal insulation to a specified design distance beyond the outside edge of the pipe or an inverted "U" surrounding the top and sides of the pipe. Any method chosen requires suitable design and installation in accordance with the manufacture's recommendations. To accommodate the placement of horizontal insulation a wider excavation trench may be required.

5.5 Foundation Design

5.5.1 Shallow Foundations Bearing on Bedrock

For conventional shallow strip and spread footings established directly on the weathered bedrock surface, a factored geotechnical bearing resistance of 500 kPa may be used at ULS. Higher bearing resistances may be available on the unweathered bedrock; however, the bedrock should be cored to confirm this recommendation.

Prior to installing foundation formwork, the bedrock is to be reviewed by a geotechnical engineer. Serviceability Limit States (SLS) design does not apply to foundations bearing directly on bedrock, since the loads required for unacceptable settlements to occur would be much larger than the factored ULS and would be limited to the elastic compression of the bedrock and concrete.

The bearing resistance of 500 kPa assumes the bedrock is cleaned of all overburden material and any loose rock pieces. The bedrock should be cleaned with air or water pressure exposing clean sound bedrock. If construction proceeds during freezing weather conditions water should not be allowed to pool and freeze in bedrock depressions. All concrete should be installed and maintained above freezing temperatures as required by the concrete supplier.

The bedrock is to be relatively level with slopes not exceeding 10 degrees from the horizontal. Where the bedrock slope exceeds 10 degrees from the horizontal and does not exceed 25 degrees from the horizontal, shear dowels can be incorporated into the design to resist sliding. Where rock slopes are steeper, the bedrock is to be levelled and stepped as required. The change in vertical height will be a function of the rock quality at the proposed foundation location and will need to be determined at the time of construction.

As an alternative to levelling the bedrock, where the bedrock surface is irregular and jagged, it may be more practical to provide a level benching over these areas by pouring lean mix concrete (minimum 10 MPa) prior to constructing the foundations. This decision is made on Site since each situation will depend on the Site-specific bedrock conditions.

5.5.2 Site Classification for Seismic Site Response & Soil Behaviour

The following information has been provided to assist the building designer from a geotechnical perspective only. These geotechnical seismic design parameters should be reviewed in detail by the structural engineer and be incorporated into the design as required.

The seismic site classification has been based on the 2012 OBC. The parameters for determination of Site Classification for Seismic Site Response are set out in Table 4.1.8.4.A of the OBC. The site classification is based on the average shear wave velocity in the top 30 m of the site stratigraphy. If the average shear wave velocity is not known, the site class can be estimated from energy corrected Standard Penetration Resistance (N60) and/or the average undrained shear strength of the soil in the top 30 m.

The boreholes advanced at this Site extended to a maximum depth of approximately 0.9 mbgs where refusal was encountered on bedrock. SPT “N” values within the soil deposit ranged between 3 and greater than 50 blows per 300 mm. As such, based on Table 4.1.8.4.A of the OBC, this Site has been classified as Class C. A Site Class C has an average shear wave velocity (Vs) of between 360 and 760 m/s. It is recommended that shear wave velocity soundings be completed at the Site once the final design and depths of foundations are known as a higher Site Classification may be available.

5.5.3 Foundation Transition Zones

Where strip footings are founded at different elevations, the bedrock is to have a maximum slope of 2 H to 1 V, with the concrete footing having a maximum rise of 600 mm and a minimum run of 600 mm between each step, as detailed in the 2012 Ontario Building Code (OBC). The lower footing should be installed first to mitigate the risk of undermining the upper footing.

Individual spread footings are to be spaced a minimum distance of one and a half times the largest footing width apart from each other to avoid stress bulb interaction between footings. This assumes the footings are at the same elevation.

Foundations may be placed at a higher elevation relative to one another provided that the slope between the outside face of the foundations are separated at a minimum slope of 2H: 1V with an imaginary line drawn from the underside of the foundations. The lower footing should be installed first to mitigate the risk of undermining the upper footing.

5.5.4 Estimated Settlement

All individual spread footings should be founded on bedrock, reviewed, and approved by a licensed geotechnical engineer.

Foundations installed in accordance with the recommendations outlined in the preceding sections are not expected to exceed total settlements of 25 mm and differential settlements of 19 mm.

All foundations are to be designed and constructed to the minimum widths as detailed in the 2012 OBC.

5.5.5 Building Drainage

To assist in maintaining the building dry from surface water seepage, it is recommended that exterior grades around the buildings be sloped away at a 2% gradient or more, for a distance of at least 2.0 m. Roof drains should discharge a minimum of 1.5 m away from the structure to a drainage swale or appropriate storm drainage system.

Exterior perimeter foundations drains are not required, where the finished floor elevation is established a minimum of 150 mm above the exterior final grades or that the exterior gradient is properly sloped to divert surface water away from the building.

5.5.6 Shallow Foundations Frost Protection & Foundation Backfill

In the Ottawa, Ontario area, exterior perimeter foundations for heated buildings require a minimum of 1.8 m of soil cover above the underside of the footing to provide soil cover for frost protection.

It is noted that for foundations established on well-draining bedrock (i.e. no ponding adjacent to the foundation), frost protection is not required. This decision is typically made on Site since each situation will depend on Site specific bedrock conditions.

Where the foundations for heated buildings do not have the minimum 1.8 m of soil cover frost protection, they should be protected from frost with a combination of soil cover and rigid polystyrene insulation, such as Dow Styrofoam or equivalent product. If required, Pinchin can provide appropriate foundation frost protection recommendations as part of the design review.

To minimize potential frost movements from soil frost adhesion, the perimeter foundation backfill should consist of a free draining granular material, such as a Granular ‘B’ Type I (OPSS 1010) or an approved sand fill, extending a minimum lateral distance of 600 mm beyond the foundation. The backfill material used against the foundation must be placed so that the allowable lateral capacity is achieved. All granular material is to be placed in maximum 300 mm thick lifts compacted to a minimum of 100% SPMDD in hard landscaping areas and 95% SPMDD in soft landscaping areas. It is recommended that inspection and testing be carried out during construction to confirm backfill quality, thickness and to ensure compaction requirements are achieved.

5.5.7 Concrete Slab-on-Grade

Prior to the installation of the engineered fill material, all overburden and deleterious materials should be removed to the underlying bedrock surface. The underlying bedrock encountered within the boreholes is considered adequate for the support of a concrete slab-on-grade provided it is inspected and approved by an experienced geotechnical engineering consultant.

Based on the in-situ conditions, it is recommended to establish a concrete floor slab-on-grade on a minimum 200 mm thick layer of Granular ’A’ (OPSS 1010). The purpose of the Granular ’A’ is mainly to provide a level surfaced for the concrete formwork. Alternatively, consideration may also be given to using a 200 mm thick layer of uniformly compacted 19 mm clear stone. Any required up-fill should consist of a Granular ‘B’ Type I or Type II (OPSS 1010).

The installation of a vapour barrier may be required under the floor slab. If required, the vapour barrier should conform to the flooring manufacturer’s and designer’s requirements. Consideration may be given to carrying out moisture emission and/or relative humidity testing of the slab to determine the concrete condition prior to flooring installation. To minimize the potential for excess moisture in the floor slab, a concrete mixture with a low water-to-cement ratio (i.e. 0.5 to 0.55) should be used.

The following table provides the unfactored modulus of subgrade reaction values:

Material Type	Modulus of Subgrade Reaction (kN/m^3)
Granular A (OPSS 1010)	85,000
Granular “B” Type I (OPSS 1010)	75,000
Granular “B” Type II (OPSS 1010)	85,000

6.0 SITE SUPERVISION & QUALITY CONTROL

It is recommended that all geotechnical aspects of the project be reviewed and confirmed under the appropriate geotechnical supervision, to routinely check such items. This includes but is not limited to inspection and confirmation of the bedrock surface prior to pouring any foundations or footings, backfilling, or engineered fill installation to ensure that the actual conditions are not markedly different than what was observed at the borehole locations and geotechnical components are constructed as per Pinchin’s recommendations. Compaction quality control of engineered fill material (full-time monitoring) is recommended as standard practice, as well as regular sampling and testing of aggregates and concrete, to ensure that physical characteristics of materials for compliance during installation and satisfies all specifications presented within this report.

7.0 TERMS AND LIMITATIONS

This Geotechnical Investigation was performed for the exclusive use of 2473493 Ontario Inc. (Client) in order to evaluate the subsurface conditions at 1983 Carling Avenue, Ottawa, Ontario. Within the limitations of scope, schedule and budget, our services have been executed in accordance with generally accepted practises in the field of geotechnical engineering for the Site. Classification and identification of soil, and geologic units have been based upon commonly accepted methods employed in professional geotechnical practice. No warranty or other conditions, expressed or implied, should be understood. Conclusions derived are specific to the immediate area of study and cannot be extrapolated extensively away from sample locations.

Performance of this Geotechnical Investigation to the standards established by Pinchin is intended to reduce, but not eliminate, uncertainty regarding the subgrade soil at the Site, and recognizes reasonable limits on time and cost.

Regardless how exhaustive a Geotechnical Investigation is performed; the investigation cannot identify all the subsurface conditions. Therefore, no warranty is expressed or implied that the entire Site is representative of the subsurface information obtained at the specific locations of our investigation. If during construction, subsurface conditions differ from then what was encountered within our test location and the additional subsurface information provided to us, Pinchin should be contacted to review our recommendations. This report does not alleviate the contractor, owner, or any other parties of their respective responsibilities.

This report has been prepared for the exclusive use of the Client and their authorized agents. Any use which a third party makes of this report, or any reliance on or decisions to be made based on it, are the responsibility of the third parties. If additional parties require reliance on this report, written authorization from Pinchin will be required. Pinchin disclaims responsibility of consequential financial effects on transactions or property values, or requirements for follow-up actions and costs. No other warranties are implied or expressed. Furthermore, this report should not be construed as legal advice.

Pinchin makes no other representations whatsoever, including those concerning the legal significance of its findings, or as to other legal matters touched on in this report, including, but not limited to, ownership of any property, or the application of any law to the facts set forth herein. With respect to regulatory compliance issues, regulatory statutes are subject to interpretation and these interpretations may change over time. Please refer to Appendix IV, Report Limitations and Guidelines for Use, which pertains to this report.

Specific limitations related to the legal and financial and limitations to the scope of the current work are outlined in our proposal, the attached Methodology, and the Authorization to Proceed, Limitation of Liability and Terms of Engagement which accompanied the proposal.

Information provided by Pinchin is intended for Client use only. Pinchin will not provide results or information to any party unless disclosure by Pinchin is required by law. Any use by a third party of reports or documents authored by Pinchin or any reliance by a third party on or decisions made by a third party based on the findings described in said documents, is the sole responsibility of such third parties. Pinchin accepts no responsibility for damages suffered by any third party as a result of decisions made or actions conducted. No other warranties are implied or expressed.

289578 Geotechnical Investigation 1983 Carling Ave Ottawa ON 2473493 Ont Inc

Template: Master Geotechnical Investigation Report – Ontario, GEO, April 1, 2020

FIGURES

PINCHIN

PROJECT NAME	GEOTECHNICAL INVESTIGATION
CLIENT NAME	2473493 ONTARIO INC.
PROJECT LOCATION	1983 CARLING AVENUE, OTTAWA, ONTARIO
FIGURE NAME	KEY MAP
FIGURE NO.	1
APPROXIMATE SCALE	AS SHOWN
PROJECT NO.	289578
DATE	MAY 2021

LEGEND

BOREHOLE LOCATION
[XX.XX] GROUND SURFACE ELEVATION (m)
[XX.XX] BOREHOLE REFUSAL ELEVATION (m)
m METRES

PINCHIN

PROJECT NAME
GEOTECHNICAL INVESTIGATION

CLIENT NAME
2473493 ONTARIO INC.

PROJECT LOCATION
1983 CARLING AVENUE,
OTTAWA, ONTARIO

FIGURE NAME
BOREHOLE LOCATION PLAN

APPROXIMATE SCALE	PROJECT NO.
AS SHOWN	289578
DATE	FIGURE NO.
APRIL 2021	2

APPENDIX I

Abbreviations, Terminology and Principle Symbols used in Report and Borehole Logs

ABBREVIATIONS, TERMINOLOGY & PRINCIPAL SYMBOLS USED

Sampling Method

AS	Auger Sample	**w**	Washed Sample
SS	Split Spoon Sample	**HQ**	Rock Core (63.5 mm diam.)
ST	Thin Walled Shelby Tube	**NQ**	Rock Core (47.5 mm diam.)
BS	Block Sample	**BQ**	Rock Core (36.5 mm diam.)

In-Situ Soil Testing

Standard Penetration Test (SPT), "N" value is the number of blows required to drive a 51 mm outside diameter spilt barrel sampler into the soil a distance of 300 mm with a 63.5 kg weight free falling a distance of 760 mm after an initial penetration of 150 mm has been achieved. The SPT, "N" value is a qualitative term used to interpret the compactness condition of cohesionless soils and is used only as a very approximation to estimate the consistency and undrained shear strength of cohesive soils.

Dynamic Cone Penetration Test (DCPT) is the number of blows required to drive a cone with a 60 degree apex attached to "A" size drill rods continuously into the soil for each 300 mm penetration with a 63.5 kg weight free falling a distance of 760 mm.

Cone Penetration Test (CPT) is an electronic cone point with a 10 cm2 base area with a 60 degree apex pushed through the soil at a penetration rate of 2 cm/s.

Field Vane Test (FVT) consists of a vane blade, a set of rods and torque measuring apparatus used to determine the undrained shear strength of cohesive soils.

Soil Descriptions

The soil descriptions and classifications are based on an expanded Unified Soil Classification System (USCS). The USCS classifies soils on the basis of engineering properties. The system divides soils into three major categories; coarse grained, fine grained and highly organic soils. The soil is then subdivided based on either gradation or plasticity characteristics. The classification excludes particles larger than 75 mm. To aid in quantifying material amounts by weight within the respective grain size fractions the following terms have been included to expand the USCS:

Soil Classification		Terminology	Proportion
Clay	< 0.002 mm		
Silt	0.002 to 0.06 mm	"trace", trace sand, etc.	1 to 10%
Sand	0.075 to 4.75 mm	"some", some sand, etc.	10 to 20%
Gravel	4.75 to 75 mm	Adjective, sandy, gravelly, etc.	20 to 35%
Cobbles	75 to 200 mm	And, and gravel, and silt, etc.	>35%
Boulders	>200 mm	Noun, Sand, Gravel, Silt, etc.	>35% and main fraction

Notes:

- Soil properties, such as strength, gradation, plasticity, structure, etcetera, dictate the soils engineering behaviour over grain size fractions; and
- With the exception of soil samples tested for grain size distribution or plasticity, all soil samples have been classified based on visual and tactile observations. The accuracy of visual and tactile observation is not sufficient to differentiate between changes in soil classification or precise grain size and is therefore an approximate description.

The following table outlines the qualitative terms used to describe the compactness condition of cohesionless soil:

Cohesionless Soil	
Compactness Condition	**SPT N-Index (blows per 300 mm)**
Very Loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very Dense	> 50

The following table outlines the qualitative terms used to describe the consistency of cohesive soils related to undrained shear strength and SPT, N-Index:

Cohesive Soil		
Consistency	**Undrained Shear Strength (kPa)**	**SPT N-Index (blows per 300 mm)**
Very Soft	<12	<2
Soft	12 to 25	2 to 4
Firm	25 to 50	4 to 8
Stiff	50 to 100	8 to 15
Very Stiff	100 to 200	15 to 30
Hard	>200	>30

Note: Utilizing the SPT, N-Index value to correlate the consistency and undrained shear strength of cohesive soils is only very approximate and needs to be used with caution.

Soil & Rock Physical Properties

General

W	Natural water content or moisture content within soil sample
γ	Unit weight
γ'	Effective unit weight
γ_d	Dry unit weight
γ_{sat}	Saturated unit weight
ρ	Density
ρ_s	Density of solid particles
ρ_w	Density of Water
ρ_d	Dry density
ρ_{sat}	Saturated density e Void ratio
n	Porosity
S_r	Degree of saturation
E_{50}	Strain at 50% maximum stress (cohesive soil)

Consistency

W_L	Liquid limit
W_P	Plastic Limit
I_P	Plasticity Index
W_S	Shrinkage Limit
I_L	Liquidity Index
I_C	Consistency Index
e_{max}	Void ratio in loosest state
e_{min}	Void ratio in densest state
I_D	Density Index (formerly relative density)

Shear Strength

C_u, S_u	Undrained shear strength parameter (total stress)
C'_d	Drained shear strength parameter (effective stress)
r	Remolded shear strength
τ_p	Peak residual shear strength
τ_r	Residual shear strength
ø'	Angle of interface friction, coefficient of friction = tan ø'

Consolidation (One Dimensional)

C_C	Compression index (normally consolidated range)
C_r	Recompression index (over consolidated range)
C_S	Swelling index
m_V	Coefficient of volume change
c_V	Coefficient of consolidation
T_V	Time factor (vertical direction)
U	Degree of consolidation
σ'_o	Overburden pressure
σ'_p	Preconsolidation pressure (most probable)
OCR	Overconsolidation ratio

Permeability

The following table outlines the terms used to describe the degree of permeability of soil and common soil types associated with the permeability rates**:**

Permeability (k cm/s)	Degree of Permeability	Common Associated Soil Type
> 10^{-1}	Very High	Clean gravel
10^{-1} to 10^{-3}	High	Clean sand, Clean sand and gravel
10^{-3} to 10^{-5}	Medium	Fine sand to silty sand
10^{-5} to 10^{-7}	Low	Silt and clayey silt (low plasticity)
>10^{-7}	Practically Impermeable	Silty clay (medium to high plasticity)

Rock Coring

Rock Quality Designation (RQD) is an indirect measure of the number of fractures within a rock mass, Deere et al. (1967). It is the sum of sound pieces of rock core equal to or greater than 100 mm recovered from the core run, divided by the total length of the core run, expressed as a percentage. If the core section is broken due to mechanical or handling, the pieces are fitted together and if 100 mm or greater included in the total sum**.**

RQD is calculated as follows:

$$\text{RQD (\%)} = \frac{\Sigma \text{ Length of core pieces} > 100 \text{ mm} \times 100}{\text{Total length of core run}}$$

The following is the Classification of Rock with Respect to RQD Value:

RQD Classification	RQD Value (%)
Very poor quality	**<25**
Poor quality	25 to 50
Fair quality	50 to 75
Good quality	75 to 90
Excellent quality	90 to 100

APPENDIX II

Pinchin's Borehole Logs

Log of Borehole: BH1

Project #: 289578 **Logged By:** WT

Project: Geotechnical Investigation

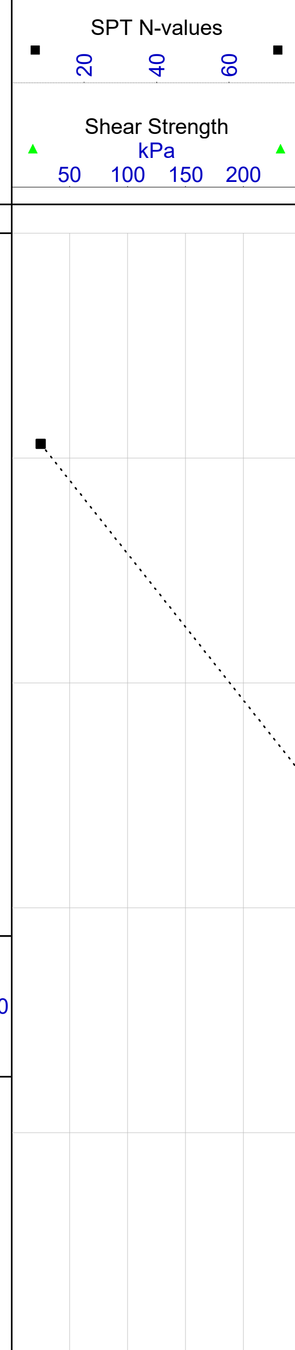
Client: 2473493 Ontario Inc.

Location: 1983 Carling Avenue, Ottawa, Ontario

Drill Date: March 29, 2021 **Project Manager:** WT

SUBSURFACE PROFILE / SAMPLE

Depth (m)	Symbol	Description	Elevation (m)	Monitoring Well Details	Sample Type	Sampler #	Recovery (%)	SPT N-values	Lab Analysis	Moisture (%)	Plasticity Index
		Ground Surface	99.24								
0		**Fill** Sand and gravel, trace silt, damp, brown, loose	98.78	No Monitoring Well Installed	SS	1	60	8	G.S.		
		Till Silty clayey sand, some gravel, damp, brown, loose to very dense	98.33		SS	2	80	>50	Hyd.	24.5	
1		End of Borehole Borehole terminated at approximately 0.91 m depth due to auger refusal on probable bedrock. Groundwater was not encountered at drilling completion.									



Contractor: Strata Drilling Group

Drilling Method: Hollow Stem / Split Spoon

Well Casing Size: N/A

Grade Elevation: 99.24 m

Top of Casing Elevation: N/A

Sheet: 1 of 1

Log of Borehole: BH2

Project #: 289578 **Logged By:** WT

Project: Geotechnical Investigation

Client: 2473493 Ontario Inc.

Location: 1983 Carling Avenue, Ottawa, Ontario

Drill Date: March 29, 2021 **Project Manager:** WT

Depth (m)	Symbol	Description	Elevation (m)	Monitoring Well Details	Sample Type	Sampler #	Recovery (%)	SPT N-values	SPT N-values (20, 40, 60) / Shear Strength kPa (50, 100, 150, 200)	Lab Analysis	Moisture (%)	Plasticity Index
		Ground Surface	98.99									
0		**Asphalt ~ 25 mm**		No Monitoring Well Installed								
		Fill Gravelly sand, trace silt, damp, brown, compact			SS	1	100	10	■	G.S.		
			98.23									
		End of Borehole Borehole terminated at approximately 0.76 m depth due to auger refusal on probable bedrock. Groundwater was not encountered at drilling completion.										
1												

Contractor: Strata Drilling Group

Drilling Method: Hollow Stem / Spilt Spoon

Well Casing Size: N/A

Grade Elevation: 98.99 m

Top of Casing Elevation: N/A

Sheet: 1 of 1

Log of Borehole: BH3

Project #: 289578 **Logged By:** WT

Project: Geotechnical Investigation

Client: 2473493 Ontario Inc.

Location: 1983 Carling Avenue, Ottawa, Ontario

Drill Date: March 29, 2021 **Project Manager:** WT

SUBSURFACE PROFILE					SAMPLE							
Depth (m)	Symbol	Description	Elevation (m)	Monitoring Well Details	Sample Type	Sampler #	Recovery (%)	SPT N-values	SPT N-values 20 40 60 / Shear Strength kPa 50 100 150 200	Lab Analysis	Moisture (%)	Plasticity Index
		Ground Surface	99.03									
0		**Asphalt ~ 25 mm**		No Monitoring Well Installed								
		Fill Sand and gravel, trace silt, damp, brown, compact			SS	1	80	3				
					SS	2	100	>50				
			98.12									
1		End of Borehole Borehole terminated at approximately 0.91 m depth due to auger refusal on probable bedrock. Groundwater was not encountered at drilling completion.										

Contractor: Strata Drilling Group **Grade Elevation:** 99.03 m

Drilling Method: Hollow Stem / Split Spoon **Top of Casing Elevation:** N/A

Well Casing Size: N/A **Sheet:** 1 of 1

Log of Borehole: BH4

Project #: 289578 **Logged By:** WT

Project: Geotechnical Investigation

Client: 2473493 Ontario Inc.

Location: 1983 Carling Avenue, Ottawa, Ontario

Drill Date: March 29, 2021 **Project Manager:** WT

SUBSURFACE PROFILE					SAMPLE							
Depth (m)	Symbol	Description	Elevation (m)	Monitoring Well Details	Sample Type	Sampler #	Recovery (%)	SPT N-values	SPT N-values (20, 40, 60) / Shear Strength kPa (50, 100, 150, 200)	Lab Analysis	Moisture (%)	Plasticity Index
		Ground Surface	99.41									
0		**Fill** Sand and gravel, trace silt, damp, brown, loose		No Monitoring Well Installed	SS	1	100	7	■			
			98.65									
		End of Borehole Borehole terminated at approximately 0.76 m depth due to auger refusal on probable bedrock. Groundwater was not encountered at drilling completion.										
1												

Contractor: Strata Drilling Group

Drilling Method: Hollow Stem / Split Spoon

Well Casing Size: N/A

Grade Elevation: 99.41 m

Top of Casing Elevation: N/A

Sheet: 1 of 1

APPENDIX III

Laboratory Testing Reports for Soil Samples

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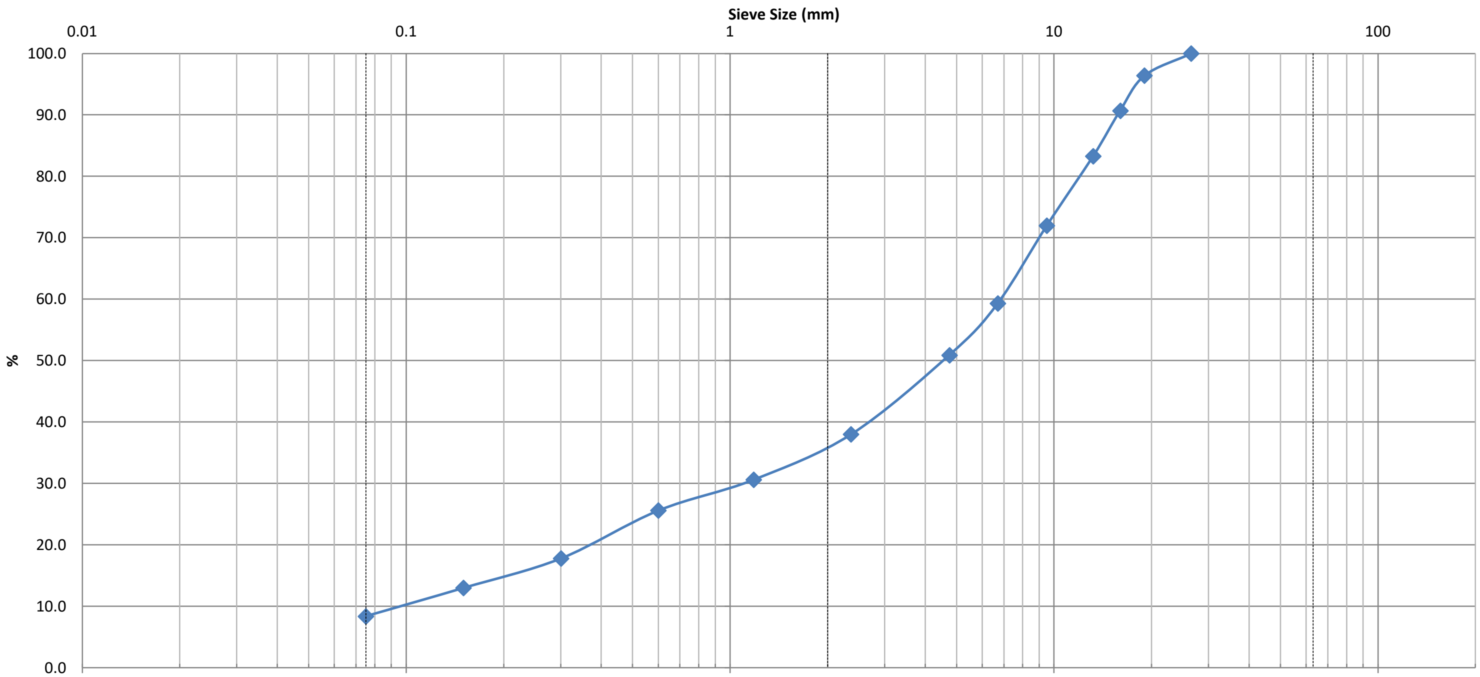
SIEVE ANALYSIS
ASTM C136

CLIENT:	Pinchin	DESCRIPTION:	Sand w Gravel	FILE NO:	PM4184
CONTRACT NO.:	-	SPECIFICATION:	**Sand w Gravel**	LAB NO:	23771
PROJECT:	289578	INTENDED USE:	-	DATE RECEIVED:	5-Apr-21
		PIT OR QUARRY:	-	DATE TESTED:	7-Apr-21
DATE SAMPLED:	5-Apr-21	SOURCE LOCATION:	BH1	DATE REPORTED:	9-Apr-21
SAMPLED BY:	Client	SAMPLE LOCATION:	0-1.5 '	TESTED BY:	DK



Silt and Clay	Sand			Gravel		Cobble
	Fine	Medium	Coarse	Fine	Coarse	

Identification	Soil Classification	MC(%)	LL	PL	PI	Cc	Cu
						2.26	70.5

D100	D60	D30	D10	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
26.5	6.7	1.2	0.095	49.1	42.5	8.4	

Comments:

REVIEWED BY:	**Curtis Beadow**	**Joe Fosyth, P. Eng.**

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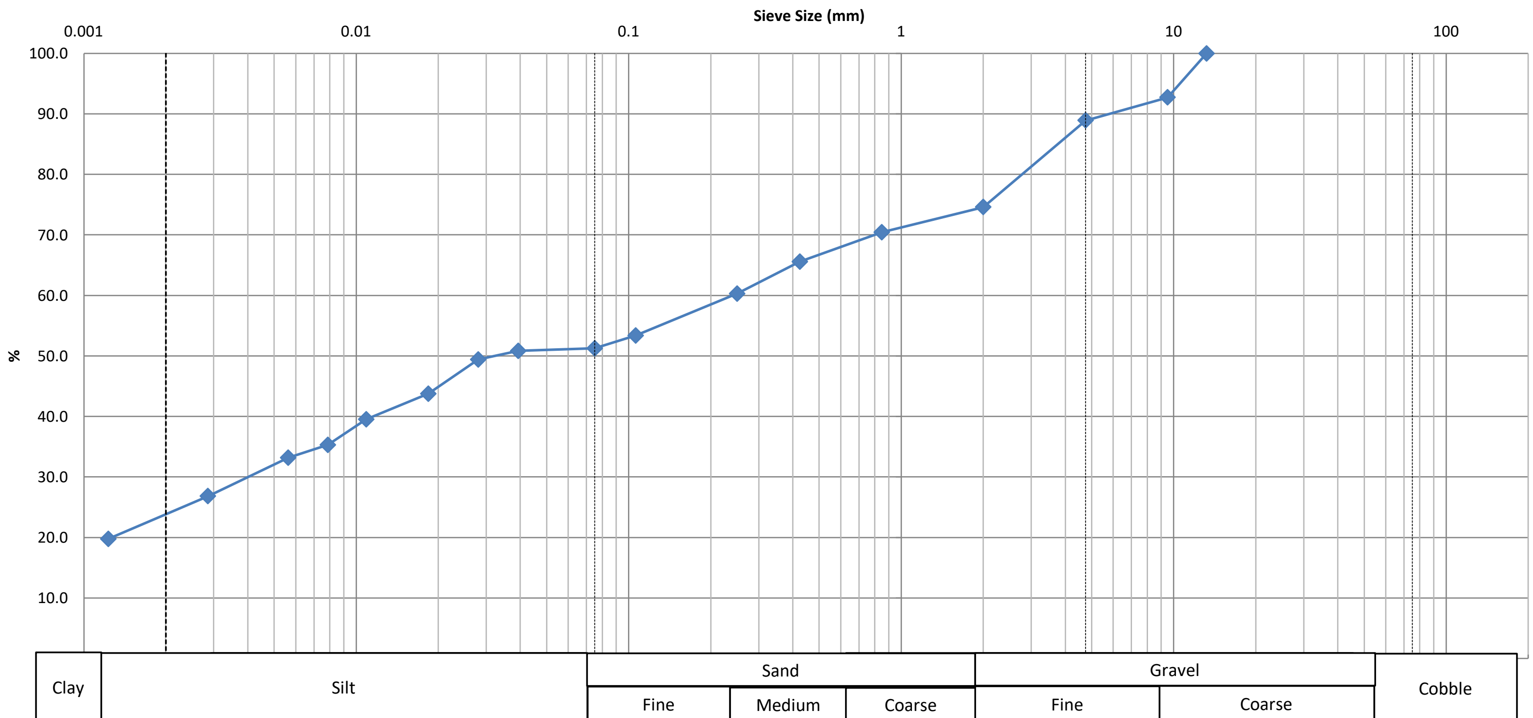
SIEVE ANALYSIS
ASTM C136

CLIENT:	Pinchin	DEPTH:	1.5 - 2 '	FILE NO:	PM4184
CONTRACT NO.:		BH OR TP No.:	BH1	LAB NO:	23769
PROJECT:	289578			DATE RECEIVED:	5-Apr-21
				DATE TESTED:	9-Apr-21
DATE SAMPLED:	5-Apr-21			DATE REPORTED:	9-Apr-21
SAMPLED BY:	Client			TESTED BY:	DB



Identification	Soil Classification					MC(%)	LL	PL	PI	Cc	Cu
						24.5					
	D100	D60	D30	D10	Gravel (%)	Sand (%)		Silt (%)		Clay (%)	
					11.1	37.7		28.2		23.0	

Comments:

REVIEWED BY:	**Curtis Beadow**	**Joe Fosyth, P. Eng.**

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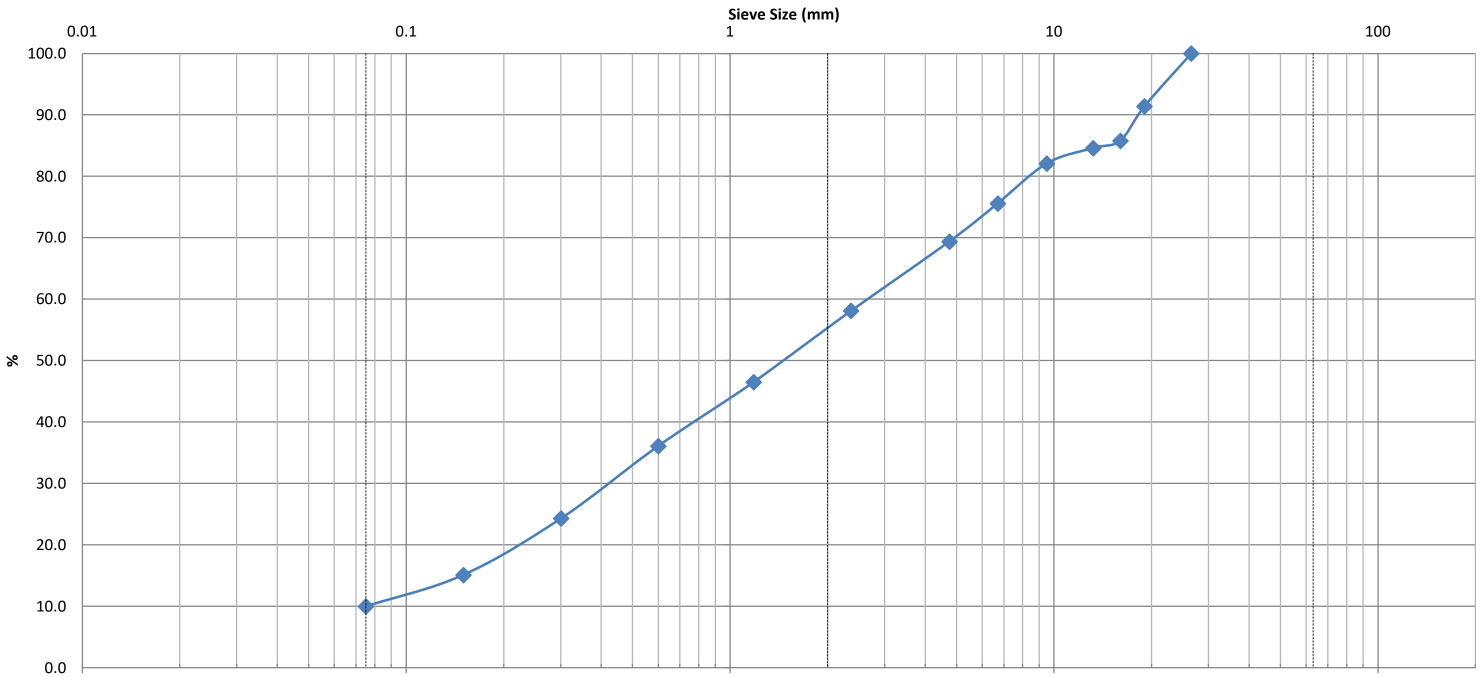
consulting engineers

SIEVE ANALYSIS
ASTM C136

CLIENT:	Pinchin	DESCRIPTION:	Silty Sand w Gravel	FILE NO:	PM4184
CONTRACT NO.:	-	SPECIFICATION:	**Silty Sand w Gravel**	LAB NO:	23770
PROJECT:	289578	INTENDED USE:	-	DATE RECEIVED:	5-Apr-21
		PIT OR QUARRY:	-	DATE TESTED:	7-Apr-21
DATE SAMPLED:	5-Apr-21	SOURCE LOCATION:	BH 2	DATE REPORTED:	9-Apr-21
SAMPLED BY:	Client	SAMPLE LOCATION:	0-2 '	TESTED BY:	DK



Silt and Clay	Sand			Gravel		Cobble
	Fine	Medium	Coarse	Fine	Coarse	

Identification	Soil Classification				MC(%)	LL	PL	PI	Cc	Cu
									0.86	36.7
	D100	D60	D30	D10	Gravel (%)	Sand (%)	Silt (%)		Clay (%)	
	26.5	2.75	0.42	0.075	30.6	59.4	10.0			

Comments:

REVIEWED BY:	**Curtis Beadow**	**Joe Fosyth, P. Eng.**

APPENDIX IV

Report Limitations and Guidelines for Use

REPORT LIMITATIONS & GUIDELINES FOR USE

This information has been provided to help manage risks with respect to the use of this report.

GEOTECHNICAL SERVICES ARE PERFORMED FOR SPECIFIC PURPOSES, PERSONS AND PROJECTS

This report was prepared for the exclusive use of the Client and their authorized agents, subject to the conditions and limitations contained within the duly authorized work plan. Any use which a third party makes of this report, or any reliance on or decisions to be made based on it, are the responsibility of the third parties. If additional parties require reliance on this report, written authorization from Pinchin will be required. Pinchin disclaims responsibility of consequential financial effects on transactions or property values, or requirements for follow-up actions and costs. No other warranties are implied or expressed. Furthermore, this report should not be construed as legal advice.

SUBSURFACE CONDITIONS CAN CHANGE

This geotechnical report is based on the existing conditions at the time the study was performed, and Pinchin's opinion of soil conditions are strictly based on soil samples collected at specific test hole locations. The findings and conclusions of Pinchin's reports may be affected by the passage of time, by manmade events such as construction on or adjacent to the Site, or by natural events such as floods, earthquakes, slope instability or groundwater fluctuations.

LIMITATIONS TO PROFESSIONAL OPINIONS

Interpretations of subsurface conditions are based on field observations from test holes that were spaced to capture a 'representative' snap shot of subsurface conditions. Site exploration identifies subsurface conditions only at points of sampling. Pinchin reviews field and laboratory data and then applies professional judgment to formulate an opinion of subsurface conditions throughout the Site. Actual subsurface conditions may differ, between sampling locations, from those indicated in this report.

LIMITATIONS OF RECOMMENDATIONS

Subsurface soil conditions should be verified by a qualified geotechnical engineer during construction. Pinchin should be notified if any discrepancies to this report or unusual conditions are found during construction.

Sufficient monitoring, testing and consultation should be provided by Pinchin during construction and/or excavation activities, to confirm that the conditions encountered are consistent with those indicated by the test hole investigation, and to provide recommendations for design changes should the conditions revealed during the work differ from those anticipated. In addition, monitoring, testing and consultation by Pinchin should be completed to evaluate whether or not earthwork activities are completed in accordance with our recommendations. Retaining Pinchin for construction observation for this project is the most effective method of managing the risks associated with unanticipated conditions. However, please be advised that any construction/excavation observations by Pinchin is over and above the mandate of this geotechnical evaluation and therefore, additional fees would apply.

MISINTERPRETATION OF GEOTECHNICAL ENGINEERING REPORT

Misinterpretation of this report by other design team members can result in costly problems. You could lower that risk by having Pinchin confer with appropriate members of the design team after submitting the report. Also retain Pinchin to review pertinent elements of the design team's plans and specifications. Contractors can also misinterpret a geotechnical engineering or geologic report. Reduce that risk by having Pinchin participate in pre-bid and preconstruction conferences, and by providing construction observation. Please be advised that retaining Pinchin to participation in any 'other' activities associated with this project is over and above the mandate of this geotechnical investigation and therefore, additional fees would apply.

CONTRACTORS RESPONSIBILITY FOR SITE SAFETY

This geotechnical report is not intended to direct the contractor's procedures, methods, schedule or management of the work Site. The contractor is solely responsible for job Site safety and for managing construction operations to minimize risks to on-Site personnel and to adjacent properties. It is ultimately the contractor's responsibility that the Ontario Occupational Health and Safety Act is adhered to, and Site conditions satisfy all 'other' acts, regulations and/or legislation that may be mandated by federal, provincial and/or municipal authorities.

SUBSURFACE SOIL AND/OR GROUNDWATER CONTAMINATION

This report is geotechnical in nature and was not performed in accordance with any environmental guidelines. As such, any environmental comments are very preliminary in nature and based solely on field observations. Accordingly, the scope of services do not include any interpretations, recommendations, findings, or conclusions regarding the, assessment, prevention or abatement of contaminants, and no conclusions or inferences should be drawn regarding contamination, as they may relate to this project. The term "contamination" includes, but is not limited to, molds, fungi, spores, bacteria, viruses, PCBs, petroleum hydrocarbons, inorganics, pesticides/insecticides, volatile organic compounds, polycyclic aromatic hydrocarbons and/or any of their by-products.

Pinchin will not be responsible for any consequential or indirect damages. Pinchin will only be held liable for damages resulting from the negligence of Pinchin. Pinchin will not be liable for any losses or damage if the Client has failed, within a period of two years following the date upon which the claim is discovered within the meaning of the Limitations Act, 2002 (Ontario), to commence legal proceedings against Pinchin to recover such losses or damage.

Appendix E DRAWINGS