Site Servicing and Stormwater Management Report 224 Preston Street Site Plan, Ottawa, ON

Type of Document:
Site Plan Submission

Client:
224 On Preston Inc.
101-337 Sunnyside Ave.
Ottawa, ON K1S 0Y6

Project Number: OTT-22019695-A0

Prepared By:

EXP Services Inc.
100-2650 Queensview Drive
Ottawa, ON   K2B 8H6
T: +1.613.688.1899
F: +1.613.225.7337

Date Submitted:
January 20, 2023

Site Servicing and Stormwater Management Report
224 Preston Street Site Plan, Ottawa, Ontario

Type of Document:
Site Plan Submission

Client:
224 On Preston Inc.
101-337 Sunnyside Ave.
Ottawa, ON K1S 0Y6

Project Number:
OTT-22019695-A0

Prepared By:

EXP Services Inc.
100-2650 Queensview Drive
Ottawa, ON K2B 8H6
Canada
T: 613 688-1899
F: 613 225-7337
www.exp.com

LICENSED PROFESSIONAL ENGINEER
Y. AMMOURI
100504164
JAN 20, 2023
PROVINCE OF ONTARIO

Chris Collins
Digitally signed by Chris Collins
DN: C=US, E=chris.collins@exp.com, O=EXP, OU=Land Development, CN=Chris Collins
Date: 2023.01.24 15:54:44-05'00'

Yasser Ammouri, M.Eng, P.Eng.
Project Engineer
Infrastructure Services

Chris Collins
Senior Land Development Manager
Infrastructure Services

Date Submitted:
January 20, 2023

Legal Notification

This report was prepared by EXP Services Inc. for the account of **224 On Preston Inc**.

Any use that a third party makes of this report, or any reliance on or decisions to be made based on it, are the responsibility of such third parties. EXP Services Inc. accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this project.

Table of Contents

List of Figures

List of Tables

List of Appendices

1 Introduction

224 On Preston Inc. (client) retained EXP Services Inc. (EXP) to undertake a site servicing and stormwater management study in support of a site plan application for the development of 224 Preston Street property located in Ottawa, ON.

The site is legally described as part of Block 123 of registered plan13 in the City of Ottawa. It is located in the southwest corner of Preston Street and Larch Street with frontage along Preston Street (major collector) and Larch Street. The client wishes to develop the site into a six-storey mix-use building. The site is within the Corso Italia District Secondary Plan Refer to Figure 1 for the site location.

This report will discuss the adequacy of the existing municipal sewers and watermains to convey the storm runoff, sanitary flows and water demands that will result from the proposed development. This report also provides a design brief in support of the engineering drawings, for the Site Plan Control Application submission and City of Ottawa approval.

2 Existing Conditions

There is an existing two-story commercial building located in the southeast corner of the site. The remainder of the property consists of hard surface, primarily asphalt paving. The site has frontage on both Preston Street and Larch Street with municipal services within the right of way (ROW) of both.

The subject site is relatively flat and sheet drains primarily towards Larch Street with a small portion of the site draining toward Preston Street.

3 Existing Infrastructure

Based on the information provided on the topographical survey prepared by Annis, O'sullivan, Vollebekk Ltd. Dated December 10, 2021, and the City of Ottawa GIS website, the following municipal infrastructure was identified.

Preston Street

- A 1800mm dia. Concrete Combined sewer pipe within the road.
- A 400mm dia. ductile iron water pipe on the east side of the road.
- A fire hydrant located at the northeast corner of the intersection between Preston Street and Balsam Street.

Larch Street

- A 600mm dia. Concrete Combined sewer pipe within the road.
- A 150mm dia. PVC water pipe within the road.
- A Fire Hydrant near the northwest corner of the subject property.

4 Proposed Development

The proposed development will consist of a six-story mix use building that includes 19 bachelor units, nine 1-bedroom units, 2 commercial units on the ground floor and 4 underground parking spaces in the basement level. The building will also have a garbage storage room and a bike storage room in the basement and ground levels. There is one underground parking access point located at the northwest corner of the property off Larch Street. Four pedestrian access points will connect the proposed building to the adjacent streets. One access point off Larch Street will be used to access the waste management room. The other three will be used to access the proposed commercial and residential units. Refer to the proposed site plan for more detail Figure 1 .

The proposed development will be serviced using the existing combined sewer and watermain along Larch Street. Stormwater management will be handled at the roof of the proposed building.

5 Referenced Guidelines

Various documents were referred to in preparing the current report including:

- Sewer Design Guidelines, Second Edition, Document SDG002, October 2012, City of Ottawa (Guidelines) including:
  - Technical Bulletin ISDTB-2012-4 (20 June 2012)
  - Technical Bulletin ISDTB-2014-01 (05 February 2014)
  - Technical Bulletin PIEDTB-2016-01 (September 6, 2016)
  - Technical Bulletin ISDTB-2018-01 (21 March 2018)
  - Technical Bulletin ISDTB-2018-04 (27 June 2018)
  - Technical Bulletin ISDTB-2019-02 (08 July 2019)
- Ottawa Design Guidelines – Water Distribution, July 2010 (WDG001), including:
  - Technical Bulletin ISDTB-2014-02 (May 27, 2014)
  - Technical Bulletin ISTB-2018-02 (21 March 2018)
- Ontario Ministry of Transportation (MTO) Drainage Manual, 1995-1997
- Stormwater Management Planning and Design Manual, Ontario Ministry of the Environment and Climate Change, March 2003 (SMPDM).
- Design Guidelines for Drinking-Water Systems, Ontario Ministry of the Environment and Climate Change, 2008 (GDWS).
- Fire Underwriters Survey, Water Supply for Public Fire Protection (FUS), 1999
- Ontario Building Code 2012, Ministry of Municipal Affairs and Housing

6 Watermain Servicing

A new 150mm dia. water service connection will be extended from the existing 150mm dia. watermain on Larch Street to the proposed building. Refer to the site servicing plan C100 for more details on the location of existing and proposed water services.

Fire protection demands have been calculated in subsequent sections using the latest version of the Fire Underwriter Survey. These fire demands will be provided using the existing fire hydrants located at the northwest corner of the property on Larch Street and the hydrant located at northeast corner of Preston Street and Balsam Street.

6.1 Domestic Water Demands

The domestic water demands are estimated below, utilizing parameters from the WDG001 and the GDWS. Table 6.1 summarizes the parameters used.

Table 6-1: Water System Design Criteria

Design Parameter	Value
Population Density – bachelor and 1-bedroom	1.4 persons/unit
City of Ottawa Average Day Demands	280 L/person/day
Commercial Average Day Demands	28,000 L/ha/day
Max Day Peaking Factor (MECP method when less than 500 persons)	9.12 x Average Day Demands
Peak Hour Factor (MECP method when less than 500 persons)	13.73 x Average Day Demands
City of Ottawa Commercial Max Day Peaking Factor	1.5 x Average Day Demands
City of Ottawa Commercial Peak Hour Factor	1.8 x Max Day Demands
Depth of Cover Required	2.4m
Maximum Allowable Pressure	690 kPa (100 psi)
Minimum Allowable Pressure	275.8 kPa (40 psi)
Minimum Allowable Pressure during fire flow conditions	137.9 kPa (20 psi)

Population:

19- Bachelor Apartments x 1.4 person/unit = 26.6 Persons
9-1 Bedroom Apartments x 1.4 person/unit = 12.6 Persons
Total = 40 Persons
191m^2 of Commercial space

Average daily water consumption = 280 L/person/day
Number of residents = 40
40 * 280 = 11,200 L/day
Maximum Day Factor = 9.12 x Avg. Day (from GDWS, Table 3-3)
Maximum Hour Factor = 13.73 x Avg. Day (from GDWS, Table 3-3)

Commercial area:

Total office space area of both buildings = 191m^2
Average Day Demand = 28,000L/ha/day
Average daily water consumption = 28000L/ha/day * (1hec/10000m^2) * 191m^2
= 534.8 L/day
Maximum Day Factor = 1.5 x Avg. Day (from WDG001)
Maximum Hour Factor = 1.8 x Max. Day (from WDG001)

The average, maximum day and peak hour domestic demands for the building are as follows:

Domestic Residential Water Demands:

Average Day 40 persons * 280 L/person/day = 11,200L/day
= 11,200 / 86,400 sec/day = 0.13 L/sec
Maximum Day = 9.12 x 0.13 = 1.185 L/sec
Peak Hour = 13.73 x 0.13 = 1.78 L/sec

Domestic Commercial Water Demands:

Average Day = 534.8L/day x (1 / 86,400) sec/day = 0.01L/sec
Maximum Day = 1.5 x 0.01 = 0.01 L/sec

Peak Hour = 1.8 x 0.01 = 0.02L/sec

Total Domestic Water Demands:

Average Day = 0.13+0.01 = 0.14L/s

Maximum Day = 1.185+0.01 = 1.195L/s

Peak Hour = 1.78+0.02 = 1.80 L/s

Detailed calculations of the domestic water demands are provided in Table B1 of Appendix B.

6.2 Fire Flow Requirements

The required fire flow for the proposed site was estimated based on the Fire Underwriters Survey. The following equation from the latest version of the Fire Underwriters Survey (2020) was used for calculation of the supply rates required to be supplied by the hydrant.

$F = 220 * C\sqrt{A}$ 2

where:

- F = the required fire flow in liters per minute
- C = coefficient related to the type of construction
- A = the total floor area in square meters

Table 6-2: Summary of Required Fire Flow Protection

Item	Design Value
Floors Above Grade	6 floors
Construction Coefficient	1.0
Fire Protection Type	Sprinkler System
Building Height (m)	21
Building Area (sq.m)	1628.6
$F=220C\sqrt{A}$ (L/sec)	8,878/min (9,000 rounded to closest 1,000)
Reduction due to low Occupancy	-0%
Reduction due to Sprinkler System	-50%
Increase due to separation	58%
Fire Flow Requirement (L/min)	9,720 or 10,000 L/min (rounded to closest 1,000) or 167 L/sec

The fire flow requirement for the proposed building was found to be 167L/sec. Refer to Table B2 in Appendix B for detailed calculations.

The fire flow required is expected to be accommodated by using the existing fire hydrants located at the northwest corner of the property on Larch Street and the hydrant located at northeast corner of Preston Street and Balsam Street intersection.

Moreover, boundary condition of the site were provided by the City of Ottawa staff. The boundaries state that the maximum HGL 115.3m, the minimum HGL is 107.2m and the Max daily + fire flow pressure is 98.0m. it is recommended to do a pressure test at the time of construction and install a pressure reducing valve if required to bring the water pressure down. The correspondence with the City staff regarding the boundary conditions can be found in Appendix.

7 Sanitary Sewer Design

The site will be serviced with a 200mm dia. PVC sanitary service connected to an existing 600mm dia. concrete combined sewer on Larch Street. Two manholes will be provide, one for the connection at the combined sewer and one at the property line for cleanout purposes.

The sanitary sewer system is designed based on a population flow, area-based infiltration allowance and foundation drain allowance. The flows were calculated using City of Ottawa design guidelines as follows:

Population:

19- Bachelor Apartments x 1.4 person/unit = 26.6 Persons
9-1 Bedroom Apartments x 1.4 person/unit = 12.6 Persons
Total = 40 Persons

Commercial Area:

$191m^2$ of Commercial space

Residential Sanitary Flow:

Average Domestic Flow = 350 L/person/day
Domestic Flow = 40 x 350 L/person/day x (1/86,400 sec/day)
= 0.162 L/sec
Peak Factor = $1 + 14 / (4 + (40/1000)^{0.5}) * K$ (K = 1)
= 4.333 (4.0 Max)

Q Peak Domestic = 0.162 L/sec x 4
= 0.648 L/sec

Commercial Sanitary Flow:

Average Domestic Flow = 50,000 L/gross ha/day
Domestic Flow = 0.0191 x 50,000 L/ha/day x (1/86,400 sec/day)
= 0.011 L/sec
Peak Factor = 1.5
Q Peak Domestic = 0.011 L/sec x1.5
= 0.0166 L/sec

Infiltration:

Q Infiltration = 0.28 L/ha/sec x 0.047 ha
= 0.013 L/sec

Foundation Drain Allowance:

Q Foundation = 5.0 L/ha/sec x 0.047 ha
= 0.235 L/sec

Total Peak Sewage Flow:

Total Sanitary Flow = 0.648+0.0166+0.013+0.235 **= 0.913 L/sec**

The proposed 200mm sanitary pipes having a slope of 1.00% and 2.00% will have a full flow capacity of 36.1 L/s and 51.0 L/s respectively. The proposed pipe capacity is sufficient to accommodate the anticipated sanitary flow from the proposed building. It is proposed that the existing 600mm dia. concrete combined sewer has enough capacity to accommodate the proposed development.

8 Stormwater Management

8.1 Design Criteria

The proposed stormwater system is designed in conformance with the latest version of the City of Ottawa Design Guidelines (October 2012). Section 5 "Storm and Combined Sewer Design" and Section 8 "Stormwater Management". A summary of the design criteria that relates to this design report is the proceeding sections below.

Minor System Design Criteria

- The storm sewers have been designed and sized based on the Rational Method and the Manning's Equation under free flow conditions for the 2-year storm using a 10-minute inlet time.
- The allowable release rate for the site is limited to a 2-year storm event using a time of concentration of 10 minutes and a runoff coefficient of 0.40. Flows in excess of the 2-year and up to the 100-year storm event will be detained onsite.

Major System Design Criteria

- The major system has been designed to accommodate onsite detention with sufficient capacity to attenuate the 100-year design storm. Excess runoff above the 100-year event will flow overland offsite.
- Onsite storage is provided for up to the 100-year design storm through surface ponding within the roof areas. Calculation of the required onsite storage volumes have been supported by calculations provided in Appendix D.
- Calculation of the required storage volumes has been prepared based on the Modified Rational Method as identified in Section 8.3.10.3 of the City's Sewer Guidelines.
- As noted in the pre-consultation meeting minutes dated June 2, 2022, the roof portion only will be controlled while the remainder of the site will go uncontrolled towards the right of ways (ROWs)

8.2 Pre-Development Conditions

There is an existing structure located in the southeast corner of the subject property. The remainder of the site is currently covered by paved parking and access areas. The calculated runoff coefficient for the site was found to be 0.9. however, based on the City of Ottawa requirements outlined in the pre-consultation meeting minutes, the maximum allowable runoff coefficient for the site will be limited to 0.4. This C value along with a time of concentration (Tc) of 10 minutes has been used to calculate the allowable release rate for the site. Table D1 to Table D3 in Appendix D provided detailed calculations under pre-development conditions.

8.3 Runoff Coefficients

Runoff coefficients used were based on actual areas taken from CAD. Runoff coefficients for impervious surfaces (roofs, asphalt, and concrete) were taken as 0.90, whereas those for pervious surfaces (grass/landscaping) were taken as 0.20. Average runoff coefficients for sub-catchments (or drainage areas) were calculated using the area weight. The runoff coefficients for pre-development and post-development catchments are provided in **Table D1 and D5** respectively, with a summary provided in in Table 8-1 below.

Table 8-1 – Summary of Runoff Coefficients

Location	Area (hectares)	Pre-Development Runoff Coefficient, C_{AVG}	Post-Development Runoff Coefficient, C_{AVG}
Entire Site	0.047	0.90	0.84

8.4 Calculation of Allowable Release Rate

The allowable release rate from the site is based on 2-year storm event with a runoff coefficient of 0.40 and a time of concentration of 10 minutes. To control runoff from the site to the allowable release rate, post-development flows from the building footprint will be restricted and on-site storage will be provided up to the 100-year storm event.

The following parameters will be used to determine the allowable release rates from the proposed site to the capped 375mm storm sewer at the property line, using the Rational method.

Q_{ALL} = 2.78 C I A

where:

Q_{ALL} = Peak Discharge (L/sec)
C = Runoff Coefficient (C=0.40)
I = Average Rainfall Intensity for return period (mm/hr)
= 732.951/(Tc+6.199)^0.810 (2-year)
Tc = Time of concentration (mins)
A = Drainage Area (hectares)

Q_{ALL} = 2.78 * 0.40 * 76.81mm/hr * 0.047 ha = 4.01 L/sec

The allowable discharge rate, based on the 2-year storm, was calculated to be 4.01 L/sec. To control runoff from the site it will be necessary to limit post-development flows for all storm return periods up to the 100-year event using flow control and detention of runoff, as noted in the following sections.

8.5 Calculation of Post-Development Runoff

To calculate the post-development runoff coefficient and required storage volumes, the site has been divided into three (3) catchment areas. The area labelled P1 represents the footprint of the building. This area will be controlled using roof drains that will restrict the flow to the allowable release limit. Area P2 will sheet drain towards Larch treet while area P3 will sheet drain towards Preston Street. Both areas P2 and P3 are considered uncontrolled. Refer to the post-development watershed plan C400 for more details on the site catchment areas.

The post-development average runoff coefficient for the site was calculated as 0.84. Based on the storm drainage areas the 2-year, 5-year and 100-year post-development peak flows are calculated based on the Rational Method and are summarized in Table 8-2 below with detailed calculations provided in **Table D6** of Appendix D.

Table 8-2: Summary of Post-Development Flows

Area No	Area (ha)	Storm = 2 yr			Storm = 5 yr			Storm = 100 yr		
		C_{AVG}	Q (L/sec)	Q_{CAP} (L/sec)	C_{AVG}	Q (L/sec)	Q_{CAP} (L/sec)	C_{AVG}	Q (L/sec)	Q_{CAP} (L/sec)
P1	0.0370	0.90	7.11	(1.53)	0.90	9.65	(2.08)	1.00	18.37	(4.00)
P2	0.0070	0.50	0.75	(0.75)	0.50	1.01	(1.01)	0.63	2.17	(2.17)
P3	0.0030	0.90	0.58	(0.58)	0.90	0.78	(0.78)	1.00	1.49	(1.49)
Total	0.0470		8.43	2.85		11.44	3.87		22.03	7.66

In summary, the building area P1 will be controlled to the allowable release rate calculated for the site using roof drains. Areas P2 and P3 will drain uncontrolled towards the ROW. The total release rate from the site during the 100-yr storm event will be 7.66 L/s.

8.6 Flow Control and Storage Method

It will be necessary to control runoff to the allowable rate; therefore, runoff will be detained using an inlet control device (ICDs) installed at the roof level. This will ensure that sufficient stormwater detention is provided and that the peak flows entering the storm sewer on Larch will be equal to or less than the allowable rate.

Based on the allowable release rate for the site, the required stormwater storage volume for the 100-year storm event will be 10 m^3. Detailed calculations using the Modified Rational Method of the onsite storage requirements are provided in Appendix D.

9 Geotechnical Recommendations

A geotechnical investigation was also carried out by EXP Services Inc., summarized in the report dated February 8th 2022. The subsurface condition of the site consists of fill material underlain by loose to very dense glacial till and gravel overlaying limestone bedrock at 5.8m depth. The Geotechnical investigation report notes that groundwater was encountered in the drilled boreholes at depths of 4 to 4.1m. A minimum of 1.5m of earth cover should be provided to the exterior foundations of heated structures to protect from damage against frost penetration.

For more geotechnical information, refer to the full geotechnical report by EXP Services Inc. found in Appendix F.

10 Erosion and Sediment Control

During all construction activities, erosion and sedimentation shall be controlled by the following techniques:

- extent of exposed soils shall be limited at any given time,
- exposed areas shall be re-vegetated as soon as possible,
- filter cloth shall be installed between frame and cover of all new catch basins and catch basin manholes,
- filter cloth shall be installed between frame and cover of the existing catch basins and catch basin manholes as identified on the site grading and erosion control plan,

- light duty silt fencing will be used to control runoff around the construction area. Silt fencing locations are identified on the erosion and sediment control plan.
- visual inspection shall be completed daily on sediment control barriers and any damage repaired immediately. Care will be taken to prevent damage during construction operations,
- In some cases, barriers may be removed temporarily to accommodate the construction operations. The affected barriers will be reinstated at night when construction is completed,
- Sediment control devices will be cleaned of accumulated silt as required. The deposits will be disposed of as per the requirements of the contract,
- during the course of construction, if the engineer believes that additional prevention methods are required to control erosion and sedimentation, the contractor will install additional silt fences or other methods as required to the satisfaction of the engineer, and
- Construction and maintenance requirements for erosion and sediment controls are to comply with Ontario Provincial Standard Specification (OPSS) OPSS 805, and City of Ottawa specifications.

11 Conclusions

This report addresses stormwater runoff from the proposed development located at 224 Preston Street, City of Ottawa, Ontario. The proposed 0.12-acre development will consist of a six-story mix use building that includes 19 bachelor units, nine 1-bedroom units, 2 commercial units on the ground floor and 4 underground parking spaces in the basement level. The following summarizes the servicing requirements for the site:

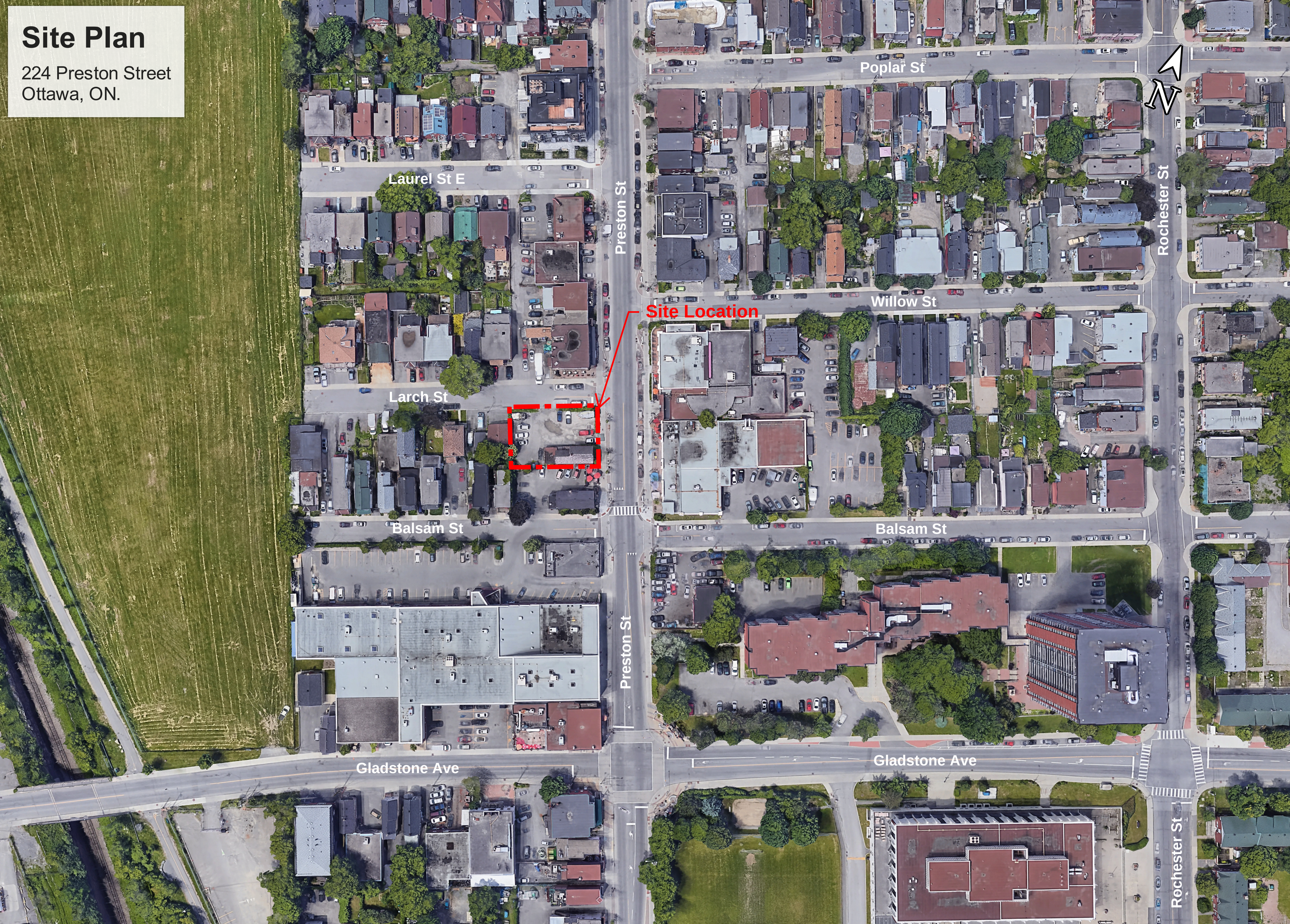
- The allowable capture rate from the proposed site was calculated based on a runoff coefficient of 0.84 and a time of concentration of 10 minutes for a 2-year storm event, connecting to the 600mm combined sewer pipe within Larch Street. The allowable release rate was calculated to be 4.01 L/sec. Runoff in excess of this will be detained onsite for up to the 100-year storm.
- Inlet control devices (ICDs) will be installed at the roof level to control the release rate from the site to the allowable 4.01L/s. The estimated storage required to control peak flows to the allowable release rate was 10 m^3 based on the Modified Rational Method.
- The proposed development has a peak sanitary flow of 0.913 L/sec based on City of Ottawa Guidelines. 200mm sewer lateral pipes will be installed with a slope of 1.0% and 2.0% having a full flow capacity of 36.1L/sec and 51.0L/s. this lateral will extend into the property and connect to the building.
- A new 150mm dia. water service connection will be extended from the existing 150mm dia. watermain on Larch Street to the proposed building. The required peak hour domestic water demand for the site was found to be 1.80L/s.
- The Maximum Required Fire Flow (RFF) based on the Fire Underwriter Survey (FUS) was calculated at 167 L/sec. The site fire demands will be provided using the existing fire hydrants located at the northwest corner of the property on Larch Street and the hydrant located at northeast corner of Preston Street and Balsam Street intersection.
- During all construction activities, erosion and sedimentation will be controlled on site.

Appendix A – Figures

Figure 1: Site Location Plan

Site Plan

224 Preston Street
Ottawa, ON.



Appendix B – Water Servicing

Table B1: Water Demand Chart

Table B2: Fire Flow Requirements Based on Fire Underwriters Survey (FUS) 2020

Correspondence with the City Regarding Boundary Conditions

TABLE B1
Water Demand Chart

Junction Number (Building)	No. of Units										Total Pop	Residential Demands					Commercial						Total Demands in (L/sec)		
	Singles/Semis/Towns				Apartments														Peaking Factors (x Avg Day)						
	Single Familty	Semi	Duplex	Townhome	Bachelor	1-Bed Apt	2-Bed Apt	3-Bed Apt	4-Bed Apt	Avg Apt.		Avg Day Demand (L/day)	Max Day Peaking Factor	Max Hour Peaking Factor	Max Day Demand (L/day)	Peak Hourly Demand (L/day)	Area (m^2)	Avg Demand (L/day)	Max Day	Peak Hour	Max Day Demand (L/day)	Peak Hour Demand (L/day)	Avg Day (L/s)	Max Day (L/s)	Peak Hour (L/s)
Building					19	9					40.0	11,200	9.12	13.73	102,107	153,720	191	535	1.5	2.7	802.2	1444.0	0.14	1.19	1.80
Totals =					19	9					40.0	11,200			102,107	153,720					802	1,444	0.14	1.19	1.80

Unit Densities	Persons/Unit
Singles	3.4
Semi-Detached	2.7
Duplex	2.3
Townhome	2.7
Bachelor Apt Unit	1.4
1-Bed Apt Unit	1.4
2-Bed Apt Unit	2.1
3-Bed Apt Unit	3.1
4-Bed Apt Unit	4.1
Avg. Apt Unit	1.8

Residential

Residential Consumption (L/pers/day) =	280	
Max Day Peaking Factor (* avg day) =	2.5	**9.12**
Peak Hour Factor (* avg day) =	5.5	**13.73**

Based on MECP Table 3-3. Less than 500 persons

Industrrial/Commercial/Institutional Water Consumption

Light Industrial (L/gross ha/day) =	35,000
Heavy Industrial (L/gross ha/day) =	55,000
Commer/Instit (L/m^2 floor/day) =	3
Max Day Peaking Factor (* avg day) =	1.5
Peak Hour Factor (* avg day) =	2.7

Project:

224 Preston Street Site Plan

Designed:	Location:
Y. Ammouri M.Eng, P.Eng	224 Preston Street, Ottawa, Ontario
Checked:	
Chris Collins	
File Reference:	Page No:
22019695 - Water - Demand Chart.xlsx	1 of 1

TABLE B2: FIRE FLOW REQURIEMENTS BASED ON FIRE UNDERWRITERS SURVEY(FUS) 2020

PROJECT: 224 Preston Street
Building No: **Mix Use**

An estimate of the Fire Flow required for a given fire area may be estimated by:

F = 220 * C * SQRT(A)

where: F = required fire flow in litres per minute
A = total floor area in m^2 (including all storeys, but excluding basements at least 50% below grade)
C = coefficient related to the type of construction

Task	Options	Multiplier	Input				Value Used	Fire Flow Total (L/min)
Choose Building Frame (C)	Wood Frame	1.5	Ordinary Construction				1	
	Ordinary Construction	1						
	Non-combustible Construction	0.8						
	Fire Resistive Construction	0.6						
Input Building Floor Areas (A)			Area	% Used	Area Used		1628.6 m²	
	Floor 6		191.61	100%	191.61			
	Floor 5		191.61	100%	191.61			
	Floor 4		317.73	100%	317.73			
	Floor 3		317.73	100%	317.73			
	Floor 2		317.73	100%	317.73			
	Floor 1		292.2	100%	292.2			
	Basement (At least 50% below grade, not included)			0%	0			
Fire Flow (F)	F = 220 * C * SQRT(A)							8,878
Fire Flow (F)	Rounded to nearest 1,000							**9,000**

Reductions/Increases Due to Factors Effecting Burning

Task	Options	Multiplier	Input	Value Used	Fire Flow Change (L/min)	Fire Flow Total (L/min)
Choose Combustibility of Building Contents	Non-combustible	-25%	Combustible	0%	0	9,000
	Limited Combustible	-15%				
	Combustible	0%				
	Free Burning	15%				
	Rapid Burning	25%				
Choose Reduction Due to Sprinkler System	**Adequate** Sprinkler Conforms to NFPA13	-30%	Adequate Sprinkler Conforms to NFPA13	-30%	-2,700	6,300
	No Sprinkler	0%				
	Standard Water Supply for Fire Department Hose Line and for Sprinkler System	-10%	Standard Water Supply for Fire Department Hose Line and for Sprinkler System	-10%	-900	5,400
	Not Standard Water Supply or Unavailable	0%				
	Fully Supervised Sprinkler System	-10%	Fully Supervised Sprinkler System	-10%	-900	4,500
	Not Fully Supervised or N/A	0%				

Task	Exposures	Separ-ation Dist (m)	Cond	Separation Conditon	Exposed Wall type	Exposed Wall Length								
						Length (m)	No of Storeys	Length-Height Factor	Sub-Conditon	Type IV-III (U)	Charge (%)	Total Charge (%)	Total Exposure Charge (L/min)	
Choose Structure Exposure Distance	North	20.8	4	20.1 to 30	Type IV-III (U)	24.75	6	148.5	4F	5%	5%	36%	3,240	7,740
	East	21.8	4	20.1 to 30	Type IV-III (U)	12.82	6	76.92	4D	3%	3%			
	South	9.1	2	3.1 to 10	Type IV-III (U)	24.75	6	148.5	2F	15%	15%			
	West	3.2	2	3.1 to 10	Type IV-III (U)	12.82	6	76.92	2D	13%	13%			

	Total Required Fire Flow, Rounded to the Nearest 1,000 L/min =	**8,000**
	Total Required Fire Flow (RFF), L/sec =	**133**
Obtain Required Fire Flow	Can the Total Fire Flow be Capped at 10,000 L/min (167 L/sec) based on "TECHNCAL BULLETIN ISTB-2018-02", (yes/no) =	**No**
	Total Required Fire Flow (RFF). If RFF < 167 use RFF (L/sec) =	**133**

Exposure Charges for Exposing Walls of Wood Frame Construciton (from Table G5)

Type V	Wood Frame
Type IV-III (U)	Mass Timber or Ordinary with Unprotected Openings
Type IV-III (P)	Mass Timber or Ordinary with Protected Openings
Type II-I (U)	Noncombustible or Fire Resistive with Unprotected Openings
Type II-I (P)	Noncombustible or Fire Resistive with Protected Openings

Conditons for Separation

Separation Dist	Condition
0m to 3m	1
3.1m to 10m	2
10.1m to 20m	3
20.1m to 30m	4
> 30.1m	5

Momen Siam

From: Jhamb, Nishant <nishant.jhamb@ottawa.ca>
Sent: Thursday, November 10, 2022 4:08 PM
To: Yasser Ammouri
Cc: Momen Siam
Subject: RE: 224 Preston Street. (PC2022-0118) water boundary conditions.
Attachments: 224 Preston Street October 2022.pdf

The following are boundary conditions, HGL, for hydraulic analysis at 224 Preston Street (zone 1W) assumed to be connected to the 152 mm watermain on Larch Street (see attached PDF for location).

Minimum HGL: 107.2 m

Maximum HGL: 115.3 m

Max Day + Fire flow (133 L/s): 98.0 m

The maximum pressure is estimated to be more than 80 psi. A pressure check at completion of construction is recommended to determine if pressure control is required.

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
Nishant

From: Yasser Ammouri <Yasser.Ammouri@exp.com>
Sent: November 10, 2022 3:23 PM
To: Jhamb, Nishant <nishant.jhamb@ottawa.ca>
Cc: Momen Siam <Momen.Siam@exp.com>
Subject: RE: 224 Preston Street. (PC2022-0118) water boundary conditions.

Hello Nishant,

I wanted to follow up with you regarding the boundary conditions.
Please let us know if you have received anything from the water department.

Thank you.

Yasser Ammouri, M.Eng., P.Eng.
EXP | Design Engineer
t : +1.343.804.4900 | e : yasser.ammouri@exp.com

From: Yasser Ammouri
Sent: Monday, October 24, 2022 9:37 AM
To: Jhamb, Nishant <nishant.jhamb@ottawa.ca>
Cc: Momen Siam <Momen.Siam@exp.com>
Subject: RE: 224 Preston Street. (PC2022-0118) water boundary conditions.

Hello Nishant,

I hope you had a good weekend.
Please find the 2020 FUS calculations attached here.

If you need anything else, please let us know.

Regards.

Yasser Ammouri, M.Eng., P.Eng.
EXP | Design Engineer
t : +1.343.804.4900 | e : yasser.ammouri@exp.com

From: Jhamb, Nishant <nishant.jhamb@ottawa.ca>
Sent: Thursday, October 20, 2022 11:52 AM
To: Yasser Ammouri <Yasser.Ammouri@exp.com>
Cc: Momen Siam <Momen.Siam@exp.com>
Subject: RE: 224 Preston Street. (PC2022-0118) water boundary conditions.

Hello Yasser, I have received the following comment from Water department.

Could you request the consultant adjust their request to use the 2020 FUS method for fire demand?

Thanks
Nishant

From: Yasser Ammouri <Yasser.Ammouri@exp.com>
Sent: October 20, 2022 11:37 AM
To: Jhamb, Nishant <nishant.jhamb@ottawa.ca>
Cc: Momen Siam <Momen.Siam@exp.com>
Subject: RE: 224 Preston Street. (PC2022-0118) water boundary conditions.

Thanks Nishant,

If you need anything else, please let me know.

Regards.

Yasser Ammouri, M.Eng., P.Eng.
EXP | Design Engineer
t : +1.343.804.4900 | e : yasser.ammouri@exp.com
exp.com | legal disclaimer
keep it green, read from the screen

From: Jhamb, Nishant <nishant.jhamb@ottawa.ca>
Sent: Thursday, October 20, 2022 11:11 AM
To: Yasser Ammouri <Yasser.Ammouri@exp.com>
Cc: Momen Siam <Momen.Siam@exp.com>
Subject: RE: 224 Preston Street. (PC2022-0118) water boundary conditions.

Thank you Yasser

I have sent the request to Water Resource team, please note it may take 2-3 weeks to get the BCs.

Thanks
Nishant

From: Yasser Ammouri <Yasser.Ammouri@exp.com>
Sent: October 19, 2022 2:56 PM
To: Jhamb, Nishant <nishant.jhamb@ottawa.ca>

Cc: Momen Siam <Momen.Siam@exp.com>
Subject: RE: 224 Preston Street. (PC2022-0118) water boundary conditions.

Hello Nishant,

Thank you for your prompt response.
Please find the FUS calculations attached here.

Regards.

Yasser Ammouri, M.Eng., P.Eng.
EXP | Design Engineer
t : +1.343.804.4900 | e : yasser.ammouri@exp.com

From: Jhamb, Nishant <nishant.jhamb@ottawa.ca>
Sent: Wednesday, October 19, 2022 1:54 PM
To: Yasser Ammouri <Yasser.Ammouri@exp.com>
Cc: Momen Siam <Momen.Siam@exp.com>
Subject: FW: 224 Preston Street. (PC2022-0118) water boundary conditions.

Hello Yasser

Thank you for the request, Can you please send us the FUS calculation as well.
Please ensure FUS calculations are as per latest FUS 2020 guide.

Have a Good Day
Nishant Jhamb, P.Eng
Project Manager |Gestionnaire de projet
Planning, Real Estate and Economic Development Department
Development Review - Central Branch
City of Ottawa | Ville d'Ottawa
110 Laurier Avenue West Ottawa, ON | 110, avenue. Laurier Ouest. Ottawa (Ontario) K1P 1J1
613.580.2424 ext./poste 23112, nishant.jhamb@ottawa.ca

From: Yasser Ammouri <Yasser.Ammouri@exp.com>
Sent: October 19, 2022 10:19 AM
To: Kadri, Nader <nader.kadri@ottawa.ca>; nishant.jhamb@ottawwa.ca; Saunders, Evan <evan.saunders@ottawa.ca>
Cc: Momen Siam <Momen.Siam@exp.com>
Subject: 224 Preston Street. (PC2022-0118) water boundary conditions.

Hello,

We are the civil consultant for the proposed project on 224 Preston Street. (PC2022-0118)
As noted in the meeting minutes (attached here), water boundary conditions are required to confirm adequate flow.

Could you please provide us with the boundary conditions?

- Type of Development and Units : 6 storey mix-use building. 2 commerical spaces on the ground floor, 19 bachelor units and nine (9) 1-berdoom units
- Site Address: 224 Preston Street, Ottawa, Ontario K1R 7R1
- A plan showing the proposed water service connection location. (attached is a preliminary servicing plan)
- Average Daily Demand (L/s) 0.136L/s
- Maximum Daily Demand (L/s) 1.194 L/s
- Peak Hour Demand (L/s) 1.791 L/s
- Fire Flow (L/min) 10,000 L/min

If you have any questions, please feel free to contact me.

Have a good day.

Yasser Ammouri, M.Eng., P.Eng.
EXP | Design Engineer
t : +1.343.804.4900 | e : yasser.ammouri@exp.com
2650 Queensview Drive
Suite 100
Ottawa, ON K2B 8H6
CANADA

Le présent courriel a été expédié par le système de courriels de la Ville d'Ottawa. Toute distribution, utilisation ou reproduction du courriel ou des renseignements qui s'y trouvent par une personne autre que son destinataire prévu est interdite. Je vous remercie de votre collaboration.

'
'

This e-mail originates from the City of Ottawa e-mail system. Any distribution, use or copying of this e-mail or the information it contains by other than the intended recipient(s) is unauthorized. Thank you.

Le présent courriel a été expédié par le système de courriels de la Ville d'Ottawa. Toute distribution, utilisation ou reproduction du courriel ou des renseignements qui s'y trouvent par une personne autre que son destinataire prévu est interdite. Je vous remercie de votre collaboration.

'
'

This e-mail originates from the City of Ottawa e-mail system. Any distribution, use or copying of this e-mail or the information it contains by other than the intended recipient(s) is unauthorized. Thank you.

Le présent courriel a été expédié par le système de courriels de la Ville d'Ottawa. Toute distribution, utilisation ou reproduction du courriel ou des renseignements qui s'y trouvent par une personne autre que son destinataire prévu est interdite. Je vous remercie de votre collaboration.

'
'

This e-mail originates from the City of Ottawa e-mail system. Any distribution, use or copying of this e-mail or the information it contains by other than the intended recipient(s) is unauthorized. Thank you.

Le présent courriel a été expédié par le système de courriels de la Ville d'Ottawa. Toute distribution, utilisation ou reproduction du courriel ou des renseignements qui s'y trouvent par une personne autre que son destinataire prévu est interdite. Je vous remercie de votre collaboration.

'

Appendix C – Sanitary Sewer Design Sheets

Table C1: Sanitary Sewer Calculation Sheet

TABLE C1 - SANITARY SEWER CALCULATION SHEET

LOCATION				RESEDENTIAL AREAS AND POPULAITONS												COMMERCIAL			INDUSTRIAL			INSTITUTIONAL			INFILTRATION			FOUNDATION DRAIN				SEWER DATA						
					NUMBER OF UNITS							POPULATION				AREA (ha)			AREA (ha)						AREA (ha)			AREA (ha)										
Street	U/S MH	D/S MH	Desc	Area (ha)	Singles	Semis	Towns	1-Bed Apt.	2-Bed Apt.	3-Bed Apt.	4-Bed Apt.	INDIV	ACCU	Peak Factor	Peak Flow (L/sec)	INDIV	ACCU	Peak Flow (L/sec)	INDIV	ACCU	Peak Factor (per	AREA (Ha)	ACCU AREA (Ha)	Peak Flow (L/sec)	INDIV	ACCU	INFILT FLOW (L/sec)	INDIV	ACCU	FOUND FLOW (L/sec)	TOTAL FLOW (L/s)	Nom Dia (mm)	Actual Dia (mm)	Slope (%)	Length (m)	Capacity (L/sec)	Q/Q_{CAP} (%)	Full Velocity (m/s)
Site	BLDG	SANMH 1		0.05				28				40	40	4.00	0.65	0.019	0.0191	0.01658							0.047	0.047	0.013	0.047	0.047	0.235	0.91	200	207.26	2.00	2.700	51.0	2%	1.72
	SANMH 1	SANMH 2											40	4.00	0.65		0.0191	0.01658								0.047	0.013		0.047	0.235	0.91	200	207.26	1.00	8.600	36.1	3%	1.21
	SANMH 2	Combined Sewer											40	4.00	0.65		0.0191	0.01658								0.047	0.013		0.047	0.235	0.91	600	607.26	0.63	-	503.2	0%	0.96
				0.047				28				40													0.047													

Residential Avg. Daily Flow, q (L/p/day) = 350
Commercial Avg. Daily Flow (L/gross ha/day) = 50,000
or L/gross ha/sec = 0.579
Institutianal Avg. Daily Flow (L/s/ha) = 50,000
or L/gross ha/sec = 0.579
Light Industrial Flow (L/gross ha/day) = 35,000
or L/gross ha/sec = 0.40509
Light Industrial Flow (L/gross ha/day) = 55,000
or L/gross ha/sec = 0.637
Extraneous Flows from Existing Areas (L/s/gross ha) = 5.00

Commercial Peak Factor = 1.5 (when area >20%)
1.0 (when area <20%)
Institutional Peak Factor = 1.5 (when area >20%)
1.0 (when area <20%)
Residential Correction Factor, K = 1.00
Manning N = 0.013
Peak extraneous flow, I (L/s/ha) = 0.28 (Total I/I)

Peak Population Flow, (L/sec) = P*q*M/86.4
Peak Extraneous Flow, (L/sec) = I*Ac
Residential Peaking Factor, M = 1 + (14/(4+P^0.5)) * K
A_c = Cumulative Area (hectares)
P = Population (thousands)
Sewer Capacity, Qcap (L/sec) = $1/N \ S^{1/2} R^{2/3} A_c$
(Manning's Equation)

Unti Type	Persons/Unit
Singles	3.0
Semi-Detached	2.7
Townhomes	2.7
Single Apt. Unit	1.4
2-bed Apt. Unit	2.1
3-bed Apt. Unit	3.1
4-bed Apt. Unit	3.8

Designed:	Project:
Y. Ammouri M.Eng, P.Eng	224 Preston Street
Checked:	Location:
Chris Collins	224 Preston Street, Ottawa, Ontario
File Reference:	Page No:
22019695 - SAN Design Sheet.xlsx	1 of 1

Appendix D – SWM Design Sheets

TABLE D1
CALCULATION OF AVERAGE RUNOFF COEFFICIENTS FOR PRE-DEVELOPMENT CONDTIONS

Area No.	Outlet Location	Asphalt Areas		Roof Areas		Concrete / Pavers		Grassed Areas		Sum AC	Total Area (m^2)	C_{AVG}
		C=0.90		C=0.90		C=0.90		C=0.20				
		Area (m^2)	A * C	Area (m^2)	A * C	Area (m^2)	A * C	Area (m^2)	A * C			
E1	ROW							470.000				
E2	ROW	470.00	423.0							423.0	470.00	0.90

TABLE D2
CALCULATION OF PEAK RUNOFF UNDER PRE-DEVELOPMENT CONDTIONS

Area No	Outlet Location	Area (ha)	Time of Conc, Tc (min)	Storm = 2 yr			Storm = 5 yr			Storm = 100 yr		
				I_2 (mm/hr)	Cavg	Q_2 (L/sec)	I_5 (mm/hr)	Cavg	Q_5 (L/sec)	I_{100} (mm/hr)	Cavg	Q_{100} (L/sec)
Site	ROW	0.04700	10	76.81	0.90	9.0	104.29	0.90	12.3	178.56	1.00	23.3

Notes

1) Intensity, $I = 732.951/(Tc+6.199)^{0.810}$ (2-year, City of 0ttawa)

2) Intensity, $I = 998.071/(Tc+6.035)^{0.814}$ (5-year, City of 0ttawa)

3) Intensity, $I = 1735.688/(Tc+6.014)^{0.820}$ (100-year, City of Ottawa)

4) Cavg for 100-year is increased by 25% to a maximum of 1.0

5) Allowable Capture Rate is based on 2-year storm at Tc=10 minutes, and discharging to combined sewer on Bronson Avenue

TABLE D3
ESTIMATION OF ALLOWABLE PEAK FLOWS (Based on Max C=0.40 with Tc=10mins & 2-yr Storm)

Area No	Outlet Location	Area (ha)	Time of Conc, Tc (min)	Storm = 2 yr		
				I_2 (mm/hr)	Cavg	Q_{2ALLOW} (L/sec)
Site	ROW	0.04700	10	76.81	0.40	4.01

Allowable Discharge (based on 2-yr storm)

Notes

1) Intensity, $I = 732.951/(Tc+6.199)^{0.810}$ (2-year, City of 0ttawa)

2) Allowable Capture Rate is based on 2-year storm at Tc=10 minutes, and discharging to combined sewer on Bronson Avenue

TABLE D4
AVERAGE RUNOFF COEFFICIENTS FOR POST-DEVELOPMENT CONDITIONS

$C_{ASPH/CONC}$ = 0.90 C_{ROOF} = 0.90 C_{GRASS} = 0.20

Area No.	Asphalt & Conc Areas (m^2)	A * C_{ASPH}	Roof Areas (m^2)	A * C_{ROOF}	Grassed Areas (m^2)	A * C_{GRASS}	Sum AC	Total Area (m^2)	C_{AVG}	Comment
P1		0.9	370	0.9		0.2	333.0	370	0.90	Parking area near CB8
P2	30	0.9		0.9	40	0.2	35.0	70	0.50	Parking area near CB7
P3	30	0.9		0.9		0.2	27.0	30	0.90	Roof & Parking area near CBMH6
Totals								470	0.84	

Notes

TABLE D5
SUMMARY OF POST-DEVELOPMENT PEAK FLOWS (Uncontrolled and Controlled)

Area No	Area (ha)	Time of Conc, Tc (min)	Storm = 2 yr				Storm = 5 yr				Storm = 100 yr			
			C_{AVG}	I_2 (mm/hr)	Q (L/sec)	Q_{CAP} (L/sec)	C_{AVG}	I_5 (mm/hr)	Q (L/sec)	Q_{CAP} (L/sec)	C_{AVG}	I_{100} (mm/hr)	Q (L/sec)	Q_{CAP} (L/sec)
P1	0.0370	10	0.90	76.81	7.11	**(1.53)**	0.90	104.19	9.65	**(2.08)**	1.00	178.56	18.37	**(4.00)**
P2	0.0070	10	0.50	76.81	0.75	**(0.75)**	0.50	104.19	1.01	**(1.01)**	0.63	178.56	2.17	**(2.17)**
P3	0.0030	10	0.90	76.81	0.58	**(0.58)**	0.90	104.19	0.78	**(0.78)**	1.00	178.56	1.49	**(1.49)**
total (storm)	0.0470				8.43	2.85			11.44	3.87			22.03	7.66
foundation drain														

Notes

1) Intensity, $I = 732.951/(Tc+6.199)^{0.810}$ (2-year, City of 0ttawa)

2) Intensity, $I = 998.071/(Tc+6.035)^{0.814}$ (5-year, City of 0ttawa)

3) Intensity, $I = 1735.688/(Tc+6.014)^{0.820}$ (100-year, City of Ottawa)

4) Cavg for 100-year is increased by 25% to a maximum of 1.0

5) Time of Concentration, Tc = ***10 mins***

6) For Flows under column Qcap which are shown in brackets ***(0.0)*** *, denotes flows that are uncontrolled)*

Table D6 Storage Volumes for 2-year, 5-Year and 100-Year Storms (MRM)

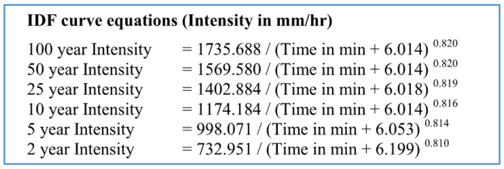
Area No: **P1**
C_{AVG} = 0.90 (2-yr)
C_{AVG} = 0.90 (5-yr)
C_{AVG} = 1.00 (100-yr, Max 1.0)
Time Interval = 10.00 (mins)
Drainage Area = 0.0370 (hectares)

Actual Release Rate (L/sec) = 4.00
Percentage of Actual Rate (City of Ottawa requirement) = 100%
Release Rate Used for Estimation of 100-year Storage (L/sec) = 4.0

Duration (mins)	Release Rate = 2.28 (L/sec) Return Period = 2 (years) IDF Parameters, A = 733.0 , B = 0.810 (I = A/(T_c+C) , C = 6.199					Release Rate = 3.09 (L/sec) Return Period = 5 (years) IDF Parameters, A = 998.1 , B = 0.814 (I = A/(T_c+C) , C = 6.053					Release Rate = 4.0 (L/sec) Return Period = 100 (years) IDF Parameters, A = 1735.7 , B = 0.820 (I = A/(T_c+C) , C = 6.014				
	Rainfall Intensity, I (mm/hr)	Peak Flow (L/sec)	Release Rate (L/sec)	Storage Rate (L/sec)	Storage (m^3)	Rainfall Intensity, I (mm/hr)	Peak Flow (L/sec)	Release Rate (L/sec)	Storage Rate (L/sec)	Storage (m^3)	Rainfall Intensity, I (mm/hr)	Peak Flow (L/sec)	Release Rate (L/sec)	Storage Rate (L/sec)	Storage (m^3)
0	167.2	15.5	2.3	13.2	0.0	230.5	21.3	3.1	18.2	0.0	398.6	41.0	4.0	37.0	0.0
10	76.8	7.1	2.3	4.8	2.9	104.2	9.6	3.1	6.6	3.9	178.6	18.4	4.0	14.4	8.6
20	52.0	4.8	2.3	2.5	3.0	70.3	6.5	3.1	3.4	4.1	120.0	12.3	4.0	8.3	10.0
30	40.0	3.7	2.3	1.4	2.6	53.9	5.0	3.1	1.9	3.4	91.9	9.4	4.0	5.4	9.8
40	32.9	3.0	2.3	0.8	1.8	44.2	4.1	3.1	1.0	2.4	75.1	7.7	4.0	3.7	9.0
50	28.0	2.6	2.3	0.3	1.0	37.7	3.5	3.1	0.4	1.2	64.0	6.6	4.0	2.6	7.7
60	24.6	2.3	2.3	0.0	0.0	32.9	3.0	3.1	0.0	-0.1	55.9	5.7	4.0	1.7	6.3
70	21.9	2.0	2.3	-0.2	-1.0	29.4	2.7	3.1	-0.4	-1.6	49.8	5.1	4.0	1.1	4.7
80	19.8	1.8	2.3	-0.4	-2.1	26.6	2.5	3.1	-0.6	-3.0	45.0	4.6	4.0	0.6	3.0
90	18.1	1.7	2.3	-0.6	-3.2	24.3	2.2	3.1	-0.8	-4.5	41.1	4.2	4.0	0.2	1.2
100	16.7	1.6	2.3	-0.7	-4.4	22.4	2.1	3.1	-1.0	-6.1	37.9	3.9	4.0	-0.1	-0.6
110	15.6	1.4	2.3	-0.8	-5.5	20.8	1.9	3.1	-1.2	-7.7	35.2	3.6	4.0	-0.4	-2.5
120	14.6	1.3	2.3	-0.9	-6.7	19.5	1.8	3.1	-1.3	-9.3	32.9	3.4	4.0	-0.6	-4.4
130	13.7	1.3	2.3	-1.0	-7.9	18.3	1.7	3.1	-1.4	-10.9	30.9	3.2	4.0	-0.8	-6.4
140	12.9	1.2	2.3	-1.1	-9.1	17.3	1.6	3.1	-1.5	-12.5	29.2	3.0	4.0	-1.0	-8.4
150	12.3	1.1	2.3	-1.1	-10.3	16.4	1.5	3.1	-1.6	-14.2	27.6	2.8	4.0	-1.2	-10.4
160	11.7	1.1	2.3	-1.2	-11.5	15.6	1.4	3.1	-1.6	-15.8	26.2	2.7	4.0	-1.3	-12.5
170	11.1	1.0	2.3	-1.2	-12.7	14.8	1.4	3.1	-1.7	-17.5	25.0	2.6	4.0	-1.4	-14.6
180	10.6	1.0	2.3	-1.3	-14.0	14.2	1.3	3.1	-1.8	-19.2	23.9	2.5	4.0	-1.5	-16.6
190	10.2	0.9	2.3	-1.3	-15.2	13.6	1.3	3.1	-1.8	-20.9	22.9	2.4	4.0	-1.6	-18.7
200	9.8	0.9	2.3	-1.4	-16.5	13.0	1.2	3.1	-1.9	-22.6	22.0	2.3	4.0	-1.7	-20.9
210	9.4	0.9	2.3	-1.4	-17.7	12.6	1.2	3.1	-1.9	-24.3	21.1	2.2	4.0	-1.8	-23.0
220	9.1	0.8	2.3	-1.4	-19.0	12.1	1.1	3.1	-2.0	-26.0	20.4	2.1	4.0	-1.9	-25.1
230	8.8	0.8	2.3	-1.5	-20.2	11.7	1.1	3.1	-2.0	-27.7	19.7	2.0	4.0	-2.0	-27.3
240	8.5	0.8	2.3	-1.5	-21.5	11.3	1.0	3.1	-2.0	-29.4	19.0	2.0	4.0	-2.0	-29.4
250	8.2	0.8	2.3	-1.5	-22.8	10.9	1.0	3.1	-2.1	-31.2	18.4	1.9	4.0	-2.1	-31.6
260	8.0	0.7	2.3	-1.5	-24.0	10.6	1.0	3.1	-2.1	-32.9	17.8	1.8	4.0	-2.2	-33.8
270	7.7	0.7	2.3	-1.6	-25.3	10.3	1.0	3.1	-2.1	-34.6	17.3	1.8	4.0	-2.2	-36.0
280	7.5	0.7	2.3	-1.6	-26.6	10.0	0.9	3.1	-2.2	-36.4	16.8	1.7	4.0	-2.3	-38.2
290	7.3	0.7	2.3	-1.6	-27.9	9.7	0.9	3.1	-2.2	-38.1	16.3	1.7	4.0	-2.3	-40.4
300	7.1	0.7	2.3	-1.6	-29.2	9.5	0.9	3.1	-2.2	-39.9	15.9	1.6	4.0	-2.4	-42.6
310	6.9	0.6	2.3	-1.6	-30.4	9.2	0.9	3.1	-2.2	-41.6	15.5	1.6	4.0	-2.4	-44.8
320	6.7	0.6	2.3	-1.7	-31.7	9.0	0.8	3.1	-2.3	-43.4	15.1	1.6	4.0	-2.4	-47.0
330	6.6	0.6	2.3	-1.7	-33.0	8.8	0.8	3.1	-2.3	-45.1	14.7	1.5	4.0	-2.5	-49.2
Max =					**3.0**					**4.1**					**10.0**

Notes
1) Peak flow is equal to the product of 2.78 x C x I x A
2) Rainfall Intensity, I = A/(Tc+C)^{B}
3) Release Rate = Min (Release Rate, Peak Flow)
4) Storage Rate = Peak Flow - Release Rate
5) Storage = Duration x Storage Rate
6) Maximium Storage = Max Storage Over Duration
7) Parameters a,b,c are for City of Ottawa

City of Ottawa IDF Data (from SDG002)



IDF curve equations (Intensity in mm/hr)

100 year Intensity = 1735.688 / (Time in min + 6.014)$^{0.820}$
50 year Intensity = 1569.580 / (Time in min + 6.014)$^{0.820}$
25 year Intensity = 1402.884 / (Time in min + 6.018)$^{0.819}$
10 year Intensity = 1174.184 / (Time in min + 6.014)$^{0.816}$
5 year Intensity = 998.071 / (Time in min + 6.053)$^{0.814}$
2 year Intensity = 732.951 / (Time in min + 6.199)$^{0.810}$

Table D7: 5-year & 100-year Roof Design Sheet - For Roof Drains using Flow Controlled Roof Drains

Project: 224 Preston Street
Location: Ottawa, ON.
Date: Jan 2023

Area #	Drain Type	Roof Drain Type	No Drains per Area	No of Weirs per Drain	Weir Position	Runoff Coeff (Cavg)		Drainage Area		5-year Event						100-year Event						Storage Required *(MRM)*		Maximium Storage Provided at Spill Elevation			
						5-year	100-year	m^2	ha	Runoff Rate (L/sec)	5yr Ponding Depth (mm)	Roof Drain Capacity Per Weir (gpm)	Roof Drain Capacity Per Drain per weir (gpm)	Roof Drain Capacity Per Drain (L/sec)	Total Flow From Roof Drains (L/sec)	Runoff Rate (L/sec)	100yr Ponding Depth (mm)	Roof Drain Capacity Per Weir (gpm)	Roof Drain Capacity Per Drain per weir (gpm)	Roof Drain Capacity Per Drain (L/sec)	Total Flow From Roof Drains (L/sec)	5-year (m^3)	100-year (m^3)	Area Available for Storage (m^2)	Max Prism Depth (mm)	Max Prisim Volume (m^3)	Total Volume (m3)
A1	RD	RD1	1	1	2-Closed	0.90	1.00	56.4	0.0056	1.471	99	5.0	5.0	0.315	0.315	2.800	133	5.0	5.0	0.315	0.315	0.82	1.97	56.4	150	2.8	2.82
A2	RD	RD1	1	1	2-Closed	0.90	1.00	57.7	0.0058	1.504	100	5.0	5.0	0.315	0.315	2.864	133	5.0	5.0	0.315	0.315	0.85	2.03	57.7	150	2.9	2.89
A3	RD	RD1	1	1	2-Closed	0.90	1.00	16.1	0.0016	0.419	69	5.0	5.0	0.315	0.315	0.798	107	5.0	5.0	0.315	0.315	0.08	0.29	16.1	150	0.8	0.80
A4	RD	RD1	1	1	2-Closed	0.90	1.00	37.5	0.0038	0.978	91	5.0	5.0	0.315	0.315	1.862	127	5.0	5.0	0.315	0.315	0.42	1.15	37.5	150	1.9	1.88
A5	RD	RD1	1	1	2-Closed	0.90	1.00	9.7	0.0010	0.253	55	5.0	5.0	0.315	0.253	0.482	90	5.0	5.0	0.315	0.315	0.02	0.11	9.7	150	0.5	0.49
A6	RD	RD1	1	1	2-Closed	0.90	1.00	19.3	0.0019	0.504	74	5.0	5.0	0.315	0.315	0.959	112	5.0	5.0	0.315	0.315	0.12	0.41	19.3	150	1.0	0.97
A7	RD	RD1	1	1	2-Closed	0.90	1.00	9.2	0.0009	0.240	55	5.0	5.0	0.315	0.240	0.456	88	5.0	5.0	0.315	0.315	0.02	0.09	9.2	150	0.5	0.46
A8	RD	RD1	1	1	2-Closed	0.90	1.00	11.2	0.0011	0.291	55	5.0	5.0	0.315	0.291	0.553	94	5.0	5.0	0.315	0.315	0.03	0.14	11.2	150	0.6	0.56
A9	RD	RD1	1	1	2-Closed	0.90	1.00	6.2	0.0006	0.162	55	5.0	5.0	0.315	0.162	0.309	68	5.0	5.0	0.315	0.309	0.02	0.03	6.2	150	0.3	0.31
Totals						0.9	0.9	**223**	**0.0223**	5.821		**45.00**		**2.84**	**2.52**	**11.08**		**45.00**		**2.84**	**2.83**	**2.37**	**6.22**	**223**		**11.2**	**11.2**
Min											**55**				`		**68**										
Max											**100**						**133**										

Runoff Based on the Following:

Storm Frequency (years) =	5	100
Time of Conc (mins) =	10	10
Storm Intensity (mm/hr) =	104.2	178.6

Qyr(cont) = 1.9
V2yr = 1.8

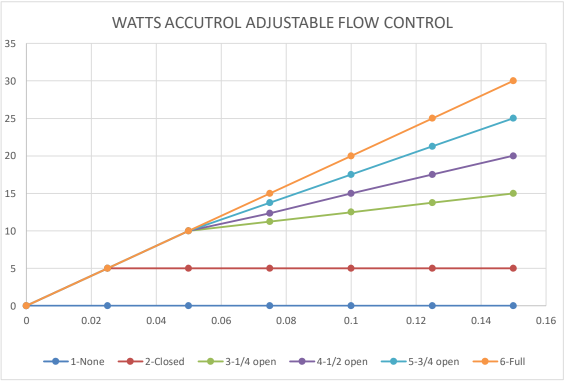
Roof Drain Types

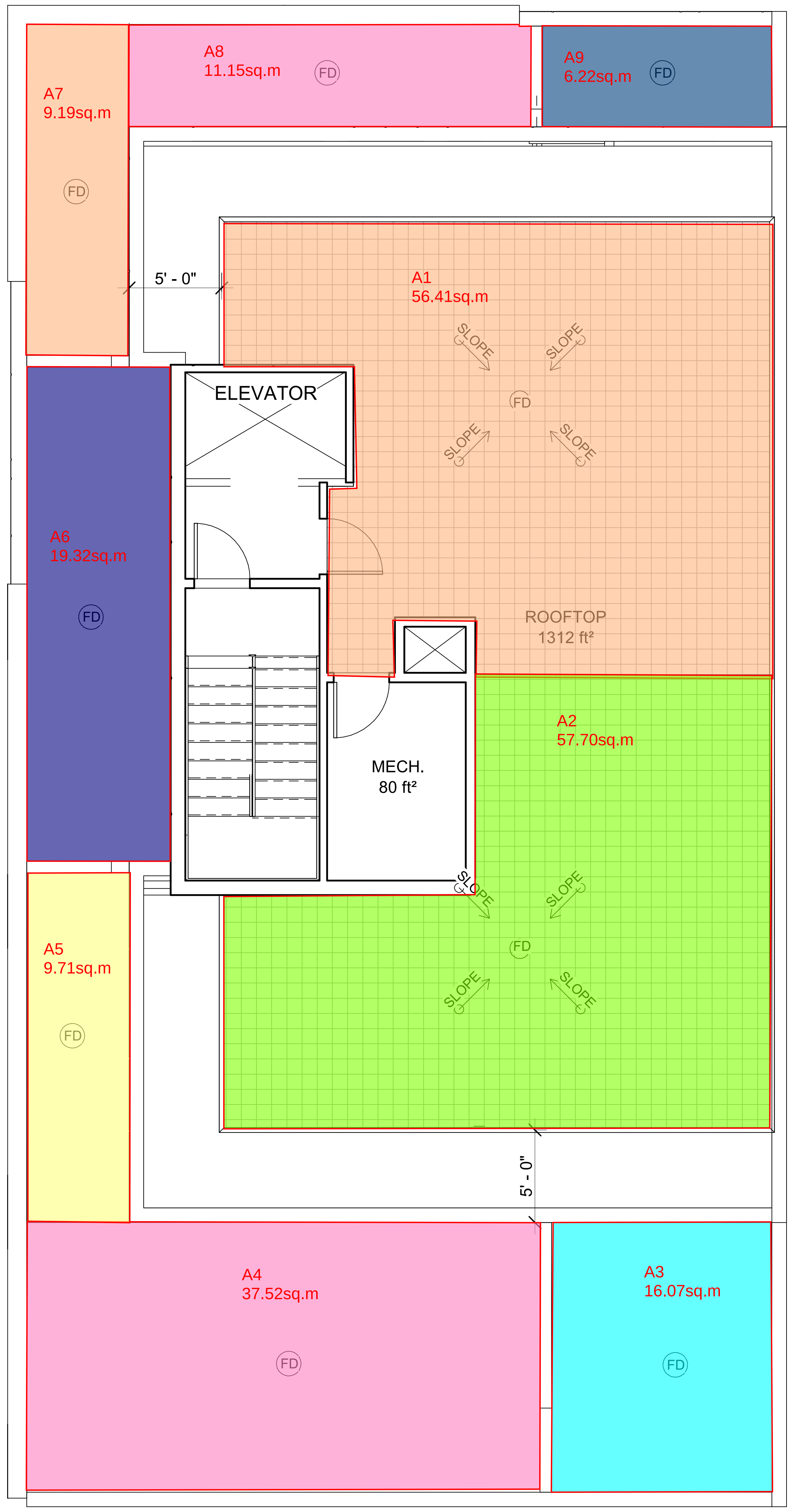
Drain Type =	RD1	RD2
Max Overflow Depth (mm	150 mm	150 mm
Flow Controlled (Yes/No)	Yes	No
Ponding	Yes	No
Weir Desc	Accutrol	n/a
No. Weirs	1	n/a

1.125

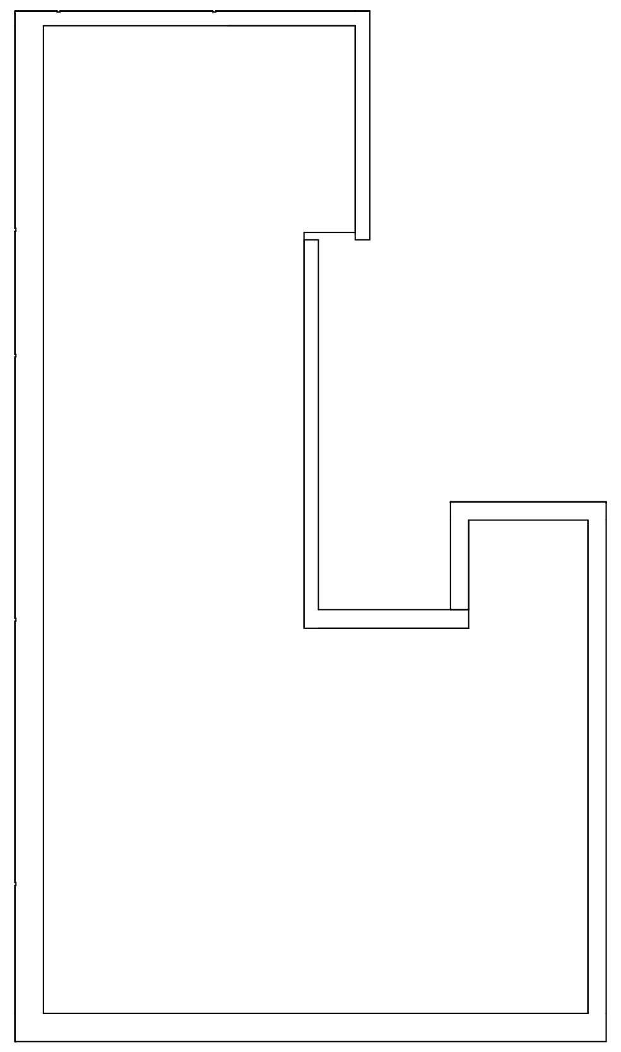
Roof Drains have Following Flow Rates: WATTS Flow Conttolled Drain

Weir Position		Flow (gpm) per depth							Max Flow Rate per Weir
		0	**25**	**50**	**75**	**100**	**125**	**150**	
		0	**0.025**	**0.05**	**0.075**	**0.1**	**0.125**	**0.15**	
1-None		0	0	0	0	0	0	0	0.000
2-Closed		0	5	5	5	5	5	5	0.315
3-1/4 open		0	5	10	11	13	14	15	0.946
4-1/2 open		0	5	10	12	15	18	20	1.262
5-3/4 open		0	5	10	14	18	21	25	1.577
6-Full		0	5	10	15	20	25	30	1.893





1 ROOF
3/16" = 1'-0"



2 UPPER ROOF
3/16" = 1'-0"

OTTAWA CARLETON CONSTRUCTION

OTTAWA CARLETON CONSTRUCTION GROUP LTD. - BCIN#: 112782
337 SUNNYSIDE AVE, SUITE 101,
OTTAWA, ON K1S 0R9

Fernando Matos

FERNANDO MATOS - BCIN#: 22431
613-884-4425

QUALIFICATION INFO
SMALL BUILDINGS

The undersigned has reviewed and takes responsibility for this design, and has the qualifications and meets the requirements set out in the Ontario Building Code to be a designer.

RESPONSIBILITIES:

DO NOT SCALE DRAWINGS

ALL DESIGN AND CONSTRUCTION TO BE IN ACCORDANCE WITH THE ONTARIO BUILDING CODE 2020

ALL CONTRACTORS MUST WORK IN ACCORDANCE WITH ALL LAWS, REGULATIONS AND BYLAWS HAVING JURISDICTION

IT IS THE RESPONSIBILITY OF THE APPROPRIATE CONTRACTOR TO CHECK AND VERIFY ALL DIMENSIONS ON SITE AND REPORT ALL ERRORS AND OMISSIONS TO THE ARCHITECT/DESIGNER

GENERAL NOTES:

224 PRESTON ST.
NEW 6 STOREY
MIXED-USE

CONSULTANTS
:
STRUCTURAL -
MECHANICAL -
ELECTRICAL -

		M/D/Y
9		
8		
7		
6		
5		
4		
3		
2		
1		
NO.	REVISION/ISSUE	DATE

PROJECT:

224 PRESTON ST.
NEW 6 STOREY MIXED-USE
224 PRESTON ST.
OTTAWA, ON K1R 7R1

FLOOR PLANS

DRAWN BY: L.T.
DATE:MARCH 26, 2022
SCALE: AS NOTED
SHEET: A2a

Appendix E – Drawings

(Included on a separate Cover)

Appendix F – Reports

Geotechnical Investigation
Proposed 6-Story Commercial and Residential Building
224 Preston Street, City of Ottawa, Ontario

Client:
Attn.: Fernando Matos
Ottawa Carleton Construction Ltd.
101-337 Sunnyside Ave.
Ottawa, Ontario K1S 0Y6

Type of Document:
Final

Project Number:
OTT-21019479-A0

Prepared By:
Matthew Zammit, P.Eng.
Geotechnical Engineer
Earth and Environment

Reviewed / Approved By:
Susan M. Potyondy, P.Eng.
Senior Geotechnical Engineer
Earth and Environment

Date Submitted:
February 8, 2022

100 - 2650 Queensview Drive, Ottawa, Ontario K2B 8H6
T: +1.613.688.1899 • F: +1.613.225.7337 • www.exp.com

Executive Summary

EXP Services Inc. (EXP) is pleased to present the results of the geotechnical investigation completed at the site registered by the street address of 224 Preston Street, Ottawa, Ontario (Figure 1). Terms and conditions of this assignment were outlined in EXP proposal number OTT-21019479-A0 dated September 30, 2021. This work was authorized by Fernando Matos on behalf of Ottawa Carleton Construction Ltd. via EXP's signed authorization form dated September 30, 2021.

Available preliminary architectural drawings indicate the proposed development at the site will comprise the construction of a new approximate 12.5 m by 23.0 m six-story mixed use commercial and residential building that will house 24-units (20 residential and 4 commercial units). The proposed building will have a one level underground parking garage. The floor slab for the underground parking garage will be constructed at a 3.05 m depth below finished grade. The proposed building will also have an elevator. The depth of the elevator pit below the lowest floor slab (parking garage slab) of the proposed building was not known at the time of this geotechnical investigation. It is anticipated that the final grades will match the existing grades of the site.

The fieldwork for the geotechnical investigation was undertaken on December 3,2021 and December 11, 2021 and consists of three (3) boreholes (Borehole Nos. 1, 2, and 3) advanced to auger refusal and termination depths of 4.7 m to 6.5 m below the existing ground surface. Monitoring wells were installed in two (2) boreholes for the long-term monitoring of groundwater levels.

Review of the borehole logs indicates the subsurface condition consists of fill underlain by loose to very dense glacial till and gravel overlying limestone bedrock at a 5.8 m depth (Elevation 52.1 m). The groundwater level is at 4.0 m and 4.1 m depths (Elevation 54.0 m and Elevation 53.8 m).

Based on the borehole information and Table 4.1.8.4.A in the 2012 Ontario Building Code (as amended May 2, 2019), the site classification for seismic site response is **Class C.** The subsurface soils are not considered to be liquefiable during a seismic event.

Grading plans were not available at the time of preparation of this report. However, based on a review of the existing grades at the site, the surrounding topography of the adjacent properties and streets and that the site is located within a well-established developed area of Ottawa, a grade raise is likely not required for this project. However, for design purposes, a grade raise of 0.5 m is considered acceptable at the site from a geotechnical point of view.

It is our understanding that the lowest floor slab of the proposed building for the underground parking garage will be placed at 3.05 m below the existing grade. In this case, it is assumed the footings will be placed 600 mm below the lowest floor at a 3.7 m depth. At this depth, the footings will be founded on the compact to very dense zone of the glacial till and on the compact zone of the gravel and slightly above the groundwater level. The native glacial till and gravel are considered suitable to support footings of the proposed building. The existing fill is not considered suitable for supporting the footings of the proposed building.

The depth of the base of the elevator pit was not known at the time of this geotechnical investigation. It is assumed however that the base of the elevator pit will be at a deeper depth than the footings and will likely be below the groundwater level.

It is anticipated that the base of the excavation for the footings and for the elevator pit may undergo basal instability or base type failure in the form of piping or heave due to the groundwater level located only 0.3 m and 0.4 m below the footings and the presence of the permeable gravel which extends below the groundwater level. To prevent base type failure, it is recommended that the groundwater level should be lowered by at least 1.0 m below the depth of the excavation for the proposed building prior to start of excavation and construction activities. This may be achieved by installing deep sumps and pumping with high-capacity pumps or by the use of well points. A specialist dewatering contractor must be consulted for this purpose.

Alternatively, the excavation may be undertaken within the confines of a shoring system that is also designed to cut-off groundwater flows towards the excavation and minimize groundwater flows into the shored excavation. In this regard, seepage of groundwater into the shored excavation should still be anticipated but may be removed by collecting the water at low points within the excavation and pumping from sumps. In areas of higher seepage, high-capacity pumps may be required to keep the excavation dry.

For an excavation where the groundwater level has been successfully lowered to at least 1.0 m below the base of the excavation for the proposed building, strip and spread footings may be founded at a 3.7 m depth below existing grade on the compact to very dense zone of the glacial till and on the compact zone of the gravel. Strip footings having a maximum width of 1.5 m may be designed for a bearing pressure at SLS of 150 kPa and factored geotechnical resistance at ULS of 225 kPa. Square pad footings having a maximum width and length of 3.0 m may be designed for a bearing pressure at SLS of 150 kPa and factored geotechnical resistance at ULS of 225 kPa. The factored geotechnical resistance includes a resistance factor of 0.5.

Settlement of footings designed for the above SLS bearing pressures are expected to be within tolerable limits of 25 mm total and 19 differential movement.

Consideration should be given to raising the lowest floor of the proposed building (the floor of the underground parking garage) to reduce the level of effort required for lowering the groundwater level as discussed above. Should the founding depth of the footings be different than the 3.7 m below existing grade, EXP should be contacted to review and provide updated SLS and factored ULS values for the revised footing depths.

If the SLS and factored ULS values provided in the report are not sufficient to support the proposed building, consideration may be given to placing the footings at a deeper depth on the bedrock. EXP can provide additional comments in this regard, if required.

A minimum of 1.5 m of earth cover should be provided to the exterior foundations of heated structures to protect them from damage due to frost penetration. The frost cover should be increased to 2.1 m for unheated structures if snow will not be removed from their vicinity and to 2.4 m if snow will be removed from the vicinity of the structure. When earth cover is less than the minimum required, an equivalent thermal combination of earth cover and rigid insulation or rigid insulation alone should be provided.

The subsurface basement walls of the proposed new building should be backfilled with free draining material, such as Ontario Provincial Standard Specification (OPSS) Granular B Type II and equipped with a perimeter drainage system to prevent the buildup of hydrostatic pressure behind the walls. The walls will be subjected to lateral static and dynamic (seismic) earth forces. These forces may be estimated by the equations provided in the report. The elevator pit should be designed as a water-tight structure. All subsurface walls should be properly waterproofed.

The floor of the parking garage of the new building may be designed as a slab-on-grade concrete floor or as a paved surface. Both options are presented in the report. Perimeter and underfloor drainage systems are required.

Excavations at the site may be undertaken using heavy equipment capable of removing cobbles and boulders within the glacial till and must be completed in accordance with the Occupational Health and Safety Act (OHSA), Ontario, Reg. 213/91. Based on the definitions provided in OHSA, the subsurface soils at the site are considered to be Type 3 soil and therefore must be sloped back at 1H:1V from the bottom of the excavation. If side slopes cannot be achieved due to space restrictions on site or due to proximity to adjacent structures, the excavations sides would require to be shored, designed and installed in accordance with latest edition of Ontario Regulation 213/91 under the OHSA and the 2006 Fourth Edition of the Canadian Foundation Engineering Manual (CFEM). Reference is made to the previous paragraph regarding lowering the groundwater level.

Seepage of the surface and subsurface water into the shored excavations is anticipated. However, it should be possible to collect water entering the excavations at low points and to remove it by conventional pumping techniques. In areas where more permeable soils may exist, a higher seepage rate should be anticipated and may require high-capacity pumps.

The materials to be excavated from the site will consist of asphalt, topsoil, sand and gravel fill, clayey silt with sand and gravel fill, and glacial till. The excavated soils are not considered suitable for use under structural elements and for backfilling purposes and therefore must be also disposed of off-site or used in landscaped areas. It is anticipated that the majority of the material required for underfloor fill and backfilling purposes would have to be imported as per the recommendations in the report.

The above and other related considerations are discussed in greater detail in the attached report.

Table of Contents:

Appendices:

Figures:

List of Tables:

1. Introduction

EXP Services Inc. (EXP) is pleased to present the results of the geotechnical investigation completed at the site located at 224 Preston Street, Ottawa, Ontario (Figure 1). Terms and conditions of this assignment were outlined in EXP proposal number OTT-21019479-A0 dated September 30, 2021. This work was authorized by Fernando Matos on behalf of Ottawa Carleton Construction Ltd. via EXP's signed authorization form dated September 30, 2021.

Available preliminary architectural drawings indicate the proposed development at the site will comprise the construction of a new approximate 12.5 m by 23.0 m six-story mixed use commercial and residential building that will house 24-units (20 residential and 4 commercial units). The proposed building will have a one level underground parking garage. The floor slab of the underground parking garage will be constructed at a 3.05 m depth below finished grade. The proposed building will also have an elevator. The depth of the elevator pit below the lowest floor slab of the proposed building (parking garage floor slab) was not known at the time of this geotechnical investigation. It is anticipated that the final grades will match the existing grades of the site.

This geotechnical investigation was undertaken to:

a) Establish the subsurface soil and groundwater conditions at the three (3) boreholes located at the site,

b) Provide classification of the site for seismic design in accordance with the requirements of the 2012 Ontario Building Code (OBC) as amended May 2,2019 and assess the liquefication potential of the subsurface soils in a seismic event,

c) Discuss grade raise restrictions,

d) Discuss foundation alternatives and provide the bearing pressure at Serviceability Limit State (SLS) and factored geotechnical resistance at Ultimate Limit State (ULS) for foundations as well as the anticipated total and differential settlements,

e) Comment on slab-on-grade construction for a concrete surface and for a paved surface for the floor of the parking garage of the proposed building and permanent drainage system requirements (perimeter and underfloor drainage systems),

f) Discuss lateral earth pressures against subsurface walls and provide lateral earth pressure parameters for static and seismic conditions,

g) Discuss excavation conditions and dewatering requirements during the construction of the proposed new building,

h) Comment on backfilling requirements and suitability of the on-site soils for backfilling purposes; and

i) Comment on subsurface concrete requirements and the corrosion potential of subsurface soils to buried metal structures/members.

The comments and recommendations given in this report assume that the above-described design concept will proceed into construction. If changes are made either in the design phase or during construction, this office must be retained to review these modifications. The result of this review may be a modification of our recommendations, or it may require additional field or laboratory work to check whether the changes are acceptable from a geotechnical viewpoint.

2. Site Description

The site is an approximate 15.5 m by 30.0 m deep rectangular shaped parcel of land bounded by Preston Street on the east side and Larch Street on the north side. It is bounded by a commercial property to the south and a residential property to the west. The neighboring lot to the south is developed with a paved driveway and less than 1 m high wooden deck along the adjacent property line. The neighboring lot to the west has a one-story garage constructed adjacent to the west property line of the site.

The site is currently occupied by a 2.5-story house with a basement located in the southeast corner of the site. This existing building will be demolished to allow the construction of the proposed new six story building. The site is flat with ground surface elevations ranging between Elevation 57.94 m and Elevation 58.08 m at the west end of the site to Elevation 57.79 m at the east end of the site based on the ground surface elevations of the boreholes.

3. Procedure

The fieldwork for the geotechnical investigation was undertaken on December 3,2021 and December 11, 2021 and consists of three (3) boreholes (Borehole Nos. 1, 2, and 3) advanced to auger refusal and termination depths of 4.7 m to 6.5 m below the existing ground surface. The locations of the boreholes are shown on the Borehole Location Plan, Figure 2. The fieldwork was supervised on a full-time basis by a representative from EXP.

The locations and geodetic elevations of the boreholes were surveyed by EXP. Prior to the fieldwork, the locations of the boreholes were cleared of any public and private underground services.

The boreholes were drilled using a CME-55 truck mounted drill rig equipped with continuous flight hollow stem augers and the capability to sample soil and bedrock. Auger samples were obtained in the three (3) boreholes from the ground surface to a 0.6 m depth below existing grade. Standard penetration tests (SPTs) were performed in the boreholes at 0.75 m and 1.5 m depth intervals with soil samples retrieved by the split-barrel sampler. The bedrock was cored in Borehole No. 2 by conventional rock coring method. A careful record of any sudden drops of the core barrel, colour of the wash water and wash water return were recorded during the rock coring operation.

A 32 mm diameter monitoring well with slotted section was installed in Borehole Nos. 1 and 3 for long-term monitoring of the groundwater level. The monitoring wells were installed in accordance with EXP standard practice, and the installation configuration is documented on the respective borehole log. The boreholes were backfilled upon completion of the field work and the installation of the monitoring wells.

All soil samples were visually examined in the field for textural classification, logged, preserved in plastic bags and identified. Similarly, the rock core was visually examined, placed in a core box, identified and logged. On completion of the fieldwork, all the soil samples and the rock core were transported to the EXP laboratory in Ottawa, Ontario, where they were visually examined by a geotechnical engineer and borehole logs were prepared. The engineer also assigned the laboratory testing program which is summarized in Table I.

Table I: Summary of Laboratory Testing Program

Type of Test	Number of Tests Completed
Soil Samples	
Moisture Content Determination	20
Grain Size Analysis	4
Atterberg Limit Determination	2
Corrosion Analysis (pH, sulphate, chloride and resistivity)	1
Bedrock Core	
Unit Weight Determination	1
Unconfined Compressive Strength Test	1

4. Subsurface Soil and Groundwater Conditions

A detailed description of the geotechnical conditions encountered in the boreholes is given on the borehole logs, Figures 3 to 5. The borehole logs and related information depict subsurface conditions only at the specific locations and times indicated. Subsurface conditions and water levels between the boreholes may differ from conditions at the locations where sampling was conducted. The passage of time may also result in changes in the conditions interpreted to exist at the locations where sampling was conducted.

The boreholes were drilled to provide representation of subsurface conditions as part of a geotechnical exploration program and are not intended to provide evidence of environmental conditions.

It should be noted that the soil boundaries indicated on the borehole logs are intended to reflect approximate transition zones for the purpose of geotechnical design and should not be interpreted as exact planes of geological change. The "Notes on Sample Descriptions" preceding borehole logs forms an integral part of this report and should be read in conjunction with this report.

A review of the borehole logs indicates the following subsurface soil conditions with depth and groundwater level measurements.

4.1 Pavement Structure

Borehole No. 3 is located in a paved area with a pavement structure consisting of 50 mm thick asphaltic concrete underlain by 350 mm thick granular base. The base material comprises of sand with silt and gravel. The moisture content of the granular fill base is 3 percent.

Grain size analysis was conducted on one (1) sample of the granular fill base material and the grain size distribution curve is shown in Figure 6. A summary of the results of the grain size distribution curve is shown in Table II.

Table II: Summary of Results from Grain-Size Analysis – Granular Fill Base Sample

Borehole No. (BH)– Auger Sample No. (AS)	Depth (m)	Grain-Size Analysis (%)			Soil Classification (USCS)
		Gravel	Sand	Fines (Silt and Clay)	
BH 3 – AS1	0.0 – 0.6	43	46	11	Poorly Graded Sand with Silt and Gravel (SP-SM)

Based on a review of the results from the grain size analysis, the granular fill base sample may be classified as a poorly graded sand with silt and gravel (SP-SM) in accordance with the Unified Soil Classification System (USCS).

4.2 Fill

A surficial granular fill layer was contacted in Borehole Nos. 1 and 2 extending to depths of 0.5 m and 0.7 m (Elevation 57.3 m and Elevation 57.2 m). The granular fill consists of silty sand and gravel. The moisture content of the granular fill is 10 percent.

The granular fill in Borehole Nos. 1 and 2 and the pavement structure in Borehole No. 3 are underlain by a mixed fill material to 1.4 m and 1.5 m depths (Elevation 56.7 m to Elevation 56.3 m). The fill consists of a heterogeneous mixture of silty clay and silty sand with gravel, cobbles and boulders. Based on standard penetration test (SPT) N-values of 12 to 29, the fill is in a compact state. The moisture content of the fill is 9 percent to 16 percent.

4.3 Buried Topsoil Layer

The fill in Borehole No. 1 is underlain by a 150 mm thick topsoil layer.

4.4 Sand and Gravel Layer

A 100 mm thick sand and gravel layer was contacted beneath the buried topsoil layer in Borehole No. 1.

4.5 Glacial Till

The thin sand and gravel layer in Borehole No. 1 and the fill in Borehole Nos. 2 and 3 are underlain by glacial till that extends to depths of 3.4 m to 4.6 m (Elevation 54.5 m to Elevation 53.2 m). The glacial till consists of silty sand with gravel, cobbles and boulders. The SPT N-values of 7 to 92 indicate the glacial till is in a loose to very dense state. The higher N-values may be a result of the sampler contacting a cobble and boulder within the glacial till. The presence of cobbles and boulders within the glacial till is also confirmed by the augers grinding as they drilled through the glacial till. The natural moisture content of the glacial till ranges from 7 percent to 11 percent.

Grain-size analysis was conducted on two (2) selected samples of the glacial till and the results are summarized in Table III. The grain-size distribution curves are shown in Figures 7 and 8.

Table III: Summary of Results from Grain-Size Analysis and Atterberg Limits - Glacial Till Samples

Borehole No. (BH) – Sample No. (SS)	Depth (m)	Grain-Size Analysis (%)			Atterberg Limits (%)				
		Gravel	Sand	Fines (Silt and Clay)	Moisture Content	Liquid Limit	Plastic Limit	Plasticity Index	Soil Classification (USCS)
BH 1 – SS4	3.0 – 3.6	25	46	29	7	NP	NP	NP	Silty Sand with Gravel (SM)
BH 3 – SS3	1.5 – 2.1	24	55	21	8	NP	NP	NP	Silty Sand with Gravel (SM)
Notes: NP = No Plastic									

Based on a review of the results of the grain-size analysis, the glacial till may be classified as a silty sand with gravel (SM) in accordance with the USCS. The glacial till contains cobbles and boulders.

4.6 Gravel

Gravel was contacted beneath the glacial till in Borehole Nos. 2 and 3 and extends to a 5.8 m depth (Elevation 52.1 m) in Borehole No. 2. Based on the SPT N-values of 9 to 30, the gravel is in a loose to dense state. The natural moisture content of the gravel is 6 percent to 13 percent.

Grain-size analysis was conducted on one (1) sample of the gravel and the results are summarized in Table IV. The grain-size distribution curve is shown in Figure 9.

Table IV: Summary of Results from Grain-Size Analysis – Gravel Sample

Borehole (BH) No. – Sample (SS) No.	Depth (m)	Grain-Size Analysis (%)			Soil Classification (USCS)
		Gravel	Sand	Fines (Silt and Clay)	
BH 2 – SS5	3.8 – 4.4	47	43	10	Poorly Graded Gravel with Silt and Sand (GP-GM)

Based on a review of the results of the grain-size analysis, the sand and gravel may be classified as a poorly graded gravel with silt and sand (GP-GM) in accordance with the USCS.

4.7 Limestone Bedrock

Refusal to auger was met in all three (3) boreholes at 4.7 m and 5.8 m depths (Elevation 53.2 m to Elevation 52.1 m). In Borehole Nos. 1 and 3 auger refusal was met on inferred bedrock or cobbles and boulders. The presence of bedrock was proven by coring an 800 mm length of the bedrock in Borehole No. 2. Photographs of the bedrock core are shown in Figure 10.

Based on a review of the published geology map (Map 1508A – Generalized Bedrock Geology, Ottawa-Hull, Ontario and Quebec, Geological Survey of Canada, printed by the Surveys and Mapping Branch, 1979), the bedrock is limestone of the Eastview formation. A review of the recovered rock core indicates the total core recovery (TCR) is 91 percent. The rock quality designation (RQD) value is 36 percent indicating the bedrock is of poor quality.

One (1) uniaxial compressive strength test was conducted on the rock core. The test results indicate a natural unit weight of 25.3 kN/m^3 and a uniaxial compressive strength of 77.9 MPa. Based on the uniaxial compressive strength test result, the bedrock may be classified as being strong in accordance with the 2006 Fourth Edition of the Canadian Foundation Engineering Manual (CFEM).

4.8 Groundwater Levels

A summary of the groundwater level measurements taken in the boreholes equipped with monitoring wells is shown in Table V.

Table V: Summary of Groundwater Level Measurements

Borehole No. (BH)	Ground Surface Elevation (m)	Date of Measurement (Elapsed Time in Days from Date of Installation)	Groundwater Depth Below Ground Surface (Elevation), m
BH 1	57.79	December 22, 2021 (19 days)	4.0 (53.8)
BH 3	58.08	December 22, 2021 (11 days)	4.1 (54.0)

The groundwater level is at 4.0 m and 4.1 m depths (Elevation 54.0 m and Elevation 53.8 m) in Borehole Nos. 1 and 3.

Groundwater levels were determined in the boreholes at the times and under the conditions stated in the scope of services. Note that fluctuations in the level of groundwater may occur due to a seasonal variation such as precipitation, snowmelt, rainfall activities, and other factors not evident at the time of measurement and therefore may be at a higher level during wet weather periods.

5. Seismic Site Classification and Liquefaction Potential of Soils

5.1 Site Classification for Seismic Site Response

Based on the borehole information and Table 4.1.8.4.A in the 2012 Ontario Building Code (as amended May 2, 2019), the site classification for seismic site response is **Class C.**

5.2 Liquefaction Potential of Soils

The subsurface soils are not considered to be liquefiable during a seismic event.

6. Grade Raise Restrictions

Grading plans were not available at the time of preparation of this report. However, based on a review of the existing grades at the site, the topography of the adjacent properties and streets and that the site is located within a well-established developed area of Ottawa, a grade raise is likely not required for this project. However, for design purposes, a grade raise of 0.5 m is considered acceptable at the site from a geotechnical point of view.

7. Foundation Considerations

It is our understanding that the lowest floor slab of the proposed building for the underground parking garage will be placed at 3.05 m below the existing grade. In this case, it is considered feasible to support the proposed building on footings placed 600 mm below the lowest floor at a 3.7 m depth. At this depth, the footings will be founded on the compact to very dense zone of the glacial till and on the compact zone of the gravel and slightly above the groundwater level. The native glacial till and gravel are considered suitable to support footings of the proposed building. The existing fill is not considered suitable for supporting the footings of the proposed building.

The depth of the base of the elevator pit was not known at the time of this geotechnical investigation. It is assumed however that the base of the elevator pit will be at a deeper depth than the footings and will likely be below the groundwater level.

It is anticipated that the base of the excavation for the footings and for the elevator pit may undergo basal instability or base type failure in the form of piping or heave due to the groundwater level located at 0.3 m and 0.4 m depth below the footing and the presence of the permeable gravel which extends below the groundwater level. To prevent base type failure, it is recommended that the groundwater level should be lowered by at least 1.0 m below the depth of the excavation for the proposed building prior to start of excavation and construction activities. This may be achieved by installing deep sumps and pumping with high-capacity pumps or by the use of well points. A specialist dewatering contractor must be consulted for this purpose.

Alternatively, the excavation may be undertaken within the confines of a shoring system that is also designed to cut-off groundwater flows towards the excavation and minimize groundwater flows into the shored excavation. In this regard, seepage of groundwater into the shored excavation should still be anticipated but may be removed by collecting the water at low points within the excavation and pumping from sumps. In areas of high infiltration, a higher seepage rate should be anticipated and high-capacity pumps may be required to keep the excavation dry.

For an excavation where the groundwater level has been successfully lowered to at least 1.0 m below the base of the excavation for the proposed building, strip and spread footings may be founded at a 3.7 m depth below existing grade on the compact to very dense zone of the glacial till and on the compact zone of the gravel. Strip footings having a maximum width of 1.5 m may be designed for a bearing pressure at SLS of 150 kPa and factored geotechnical resistance at ULS of 225 kPa. Square pad footings having a maximum width and length of 3.0 m may be designed for a bearing pressure at SLS of 150 kPa and factored geotechnical resistance at ULS of 225 kPa. The factored geotechnical resistance includes a resistance factor of 0.5.

Settlement of footings designed for the above SLS bearing pressures are expected to be within tolerable limits of 25 mm total and 19 differential movement.

Consideration should be given to raising the lowest floor of the proposed building (the floor of the underground parking garage) to reduce the level of effort required for lowering the groundwater level as discussed above. Should the founding depth of the footings be different than the 3.7 m below existing grade, EXP should be contacted to review and provide updated SLS and factored ULS values for the revised footing depths.

If the SLS and factored ULS values provided in this report are not sufficient to support the proposed building, consideration may be given to placing the footings at a deeper depth on the bedrock. EXP can provide additional comments in this regard, if required.

Foundation that are to be placed at different elevations should be located such that the higher footing is set below a line drawn up at 10H:7V from the near edge of the lower footing. The lower footing should be constructed before the upper footing to prevent the latter from being undermined during subsequent construction. This concept should also be applied to underground service excavations to ensure that foundations and underground services will not be undermined.

All footing beds should be examined by a geotechnical engineer to ensure that the founding surfaces are capable of supporting the design bearing pressure at SLS and the footing beds have been properly prepared.

It is recommended that a 50 mm thick concrete mud slab be placed on the approved glacial till and gravel subgrade to protect the subgrade from disturbance by construction equipment, workers (foot traffic) and the effects of the weather (such as precipitation).

A minimum of 1.5 m of earth cover should be provided to the exterior foundations of heated structures to protect them from damage due to frost penetration. The frost cover should be increased to 2.1 m for unheated structures if snow will not be removed from their vicinity and to 2.4 m if snow will be removed from the vicinity of the structure. When earth cover is less than the minimum required, an equivalent thermal combination of earth cover and rigid insulation or rigid insulation alone should be provided. EXP can provide additional comments in this regard, if required.

The recommended bearing pressure at SLS and factored geotechnical resistances at ULS have been calculated by EXP from the borehole information for the design stage only. The investigation and comments are necessarily on-going as new information of underground conditions becomes available. For example, more specific information is available with respect to conditions between boreholes when foundation construction is underway. The interpretation between boreholes and the recommendations of this report must therefore be checked through field monitoring provided by an experienced geotechnical engineer to validate the information for use during the construction stage.

8. Floor Slab and Drainage Requirements

The lowest floor level of the parking garage for the proposed building will be located at an approximate 3.05 m depth below the existing grade. Based on the borehole information, the lowest floor slab of the building will be founded on the glacial till and may be constructed as a concrete slab-on-grade or as a paved surface. The concrete and asphalt pavement structures indicated below are for light duty traffic only (cars). EXP can provide concrete and asphalt pavement structures for heavy duty traffic (cars and trucks), if required.

The lowest floor level for the parking garages is anticipated to be located within 1.0 m of the groundwater level. Therefore, underfloor and perimeter drainage systems will be required for the floor of the proposed below grade parking garage.

The underfloor drainage system may consist of 100 mm diameter perforated pipe or equivalent placed in parallel rows at 5 m to 6 m centres and at least 300 mm below the underside of the floor slab. The drains should be set on 100 mm thick bed of 19 mm sized clear stone and covered on top and sides with 150 mm thick clear stone that is fully wrapped or covered with an approved porous geotextile membrane, such as Terrafix 270R or equivalent. The perimeter drains may also consist of 100 mm diameter perforated pipe set on the footings and surrounded with 150 mm thick 19 mm sized clear stone that is fully wrapped or covered with an approved porous geotextile membrane, such as Terrafix 270R or equivalent. The perimeter and underfloor drains should be connected to separate sumps equipped with backup pumps and generators in case of mechanical failure and/or power outage, so that at least one system would be operational should the other fail.

The finished exterior grade should be sloped away from the building to prevent ponding of surface water close to the exterior walls of the buildings.

8.1 Lowest Floor Level as a Concrete Surface

The lowest floor slab of the parking garage may be designed as a slab-on-grade. The subgrade is anticipated to be glacial till. The exposed glacial till should be proofrolled and examined by a geotechnical engineer. Any loose/soft zones of the glacial till should be excavated and replaced with Ontario Provincial Standard Specification (OPSS) Granular B Type II compacted to 95 percent SPMDD.

The slab-on-grade should be set on the Granular A pad noted below that is placed on an engineered fill pad at least 300 mm thick placed on the approved glacial till subgrade. The required engineered fill pad should comprise of OPSS Granular B Type II placed on top of the approved glacial till in 300 mm thick lifts and each lift compacted to 98 percent standard Proctor maximum dry density (SPMDD).

Following the preparation of the engineered fill pad, the concrete slab for light duty traffic (cars only) may be constructed as follows:

- 150 mm thick concrete with 32 MPa compressive strength and air content of 5 percent to 8 percent, over
- 150 mm thick layer of OPSS Granular A base material compacted to 100 percent standard Proctor maximum dry density (SPMDD); over

- 300 mm minimum thick layer of OPSS Granular B Type II sub-base material compacted to 100 percent SMPDD.

The concrete slab should be reinforced and adequate saw cuts should be provided in the floor slab to control cracking. Additional recommendations can be provided once the final design of the floor of the parking garage has been determined.

8.2 Lowest Floor Level as a Paved Surface

The subgrade is anticipated to be glacial till. The exposed glacial till should be proofrolled and examined by a geotechnical engineer. Any loose/soft zones of the glacial till should be excavated and replaced with OPSS Granular B Type II compacted to 95 percent SPMDD.

Following approval and preparation of the glacial till subgrade, the asphalt pavement structure for light duty traffic (cars only) may be constructed on the approved glacial till subgrade as follow:

- 65 mm thick layer of asphaltic concrete consisting of HL3/SP12.5 – The asphaltic concrete should be placed and compacted as per OPSS 310 and 313 and should be designed in accordance with OPSS 1150/1151, over
- 150 mm thick layer of OPSS Granular A base material compacted to 100 percent SPMDD; over
- 450 mm thick layer of OPSS Granular B Type II sub-base material compacted to 100 percent SPMDD.

8.3 Additional Comment

Since the lowest floor slab of the proposed building will be above the groundwater level, the permanent drainage systems (perimeter and underfloor drainage systems) for the proposed building are not expected to adversely impact adjacent existing structures and infrastructure over the long-term.

9. Lateral Earth Pressure on Subsurface Walls

9.1 Basement Walls

The subsurface basement walls of the proposed new building should be backfilled with free draining material, such as OPSS Granular B Type II compacted to 95 percent SPMDD and equipped with a perimeter drainage system to prevent the buildup of hydrostatic pressure behind the walls. The walls will be subjected to lateral static and dynamic (seismic) earth forces. The expressions below assume free draining backfill material, a perimeter drainage system, level backfill surface behind the wall and vertical face on the back side of the wall.

For design purposes, the lateral static earth thrust against the subsurface walls may be computed from the following equation:

$$P = K_0 h (½ \gamma h + q)$$

where P = lateral earth thrust acting on the subsurface wall, kN/m

K_0 = lateral earth pressure at rest coefficient, assumed to be 0.5 for Granular B Type II backfill material

γ = unit weight of free draining granular backfill; Granular B Type II = 22 kN/m^3

h = depth of point of interest below top of backfill, m

q = surcharge load stress, kPa

The lateral dynamic thrust may be computed from the equation given below:

$$\Delta_{Pe} = \gamma H^2 \frac{a_h}{g} F_b$$

where Δ_{Pe} = dynamic thrust in kN/m of wall

H = height of wall, m

γ = unit weight of backfill material = 22 kN/m^3

$\frac{a_h}{g}$ = earth pressure coefficient = 0.32 for Ottawa area

F_b = thrust factor = 1.0

The dynamic thrust does not take into account the surcharge load. The resultant force acts approximately at 0.63H above the base of the wall.

9.2 Elevator Walls

The subsurface walls of the elevator pit should be designed as a water-tight structure with the groundwater level assumed to be at the top of the elevator pit wall. The elevator pit should be designed to resist buoyancy forces that may be resisted by the weight of the elevator structure or by installing rock anchors. EXP can provide comments and recommendations regarding rock anchors if required.

9.3 Additional Comments

All subsurface walls should be properly waterproofed.

10. Excavation and De-Watering Requirements

10.1 Excess Soil Management

A new Ontario Regulation 406/19 made under the Environmental Protection Act (November 28, 2019) was implemented as of January 1, 2021. The new regulation dictates the testing protocol that will be required for the management and disposal of excess soils. As set forth in the regulation, specific analytical testing protocols will need to be implemented and followed based on the volume of soil to be managed. The testing protocols are specific as to whether the soils are stockpiled or in situ. In either scenario, the testing protocols are far more onerous than have been historically carried out as part of standard industry practices. These decisions should be factored in and accounted for prior to the initiation of the project-defined scope of work. EXP would be pleased to assist with the implementation of a soil management and testing program that would satisfy the requirements of Ontario Regulation 406/19.

10.2 Excavations

Excavations for the construction of the proposed building is expected to extend to a maximum depth of 4.0 m below the existing ground surface. These excavations will extend through the fill and into the glacial till and gravel and are anticipated to be approximately at or slightly above the groundwater level.

It is also noted that the existing structures present within the footprint of the proposed new building will need to be demolished and all construction debris removed off site to allow for the construction of the new building.

Excavations through the fill and into the glacial till may be undertaken by heavy equipment capable of removing cobble and boulder sizes within the glacial till.

Excavations above the groundwater level may be undertaken using conventional equipment and should be completed in accordance with the Occupational Health and Safety Act (OHSA), Ontario, Reg. 213/91. Based on the definitions provided in OHSA, the subsurface soils at the site are considered to be Type 3 soil. As per OHSA, the sidewalls of open cut excavations undertaken within Type 3 soil, must be sloped back at 1H:1V from the bottom of the excavation. Within zones of seepage, the excavation side slopes are expected to slough and eventually stabilize at 2H:1V to 3H:1V from the bottom of the excavation. Open cut excavations below the groundwater level are anticipated to be more problematic and will require the lowering the groundwater level prior to the start of excavation and construction activities as discussed in this section of the report.

If side slopes cannot be achieved due to space restrictions on site such as the proximity of open cut excavations to the property limits, existing infrastructure or to foundations of adjacent existing buildings the new building construction would have to be undertaken within the confines of an engineered support system (shoring system). The need for a shoring system, the most appropriate type of shoring system and the design and installation of the shoring system should be determined by the contractors bidding on this project. The design of the shoring system should be undertaken by a professional engineer experienced in shoring design and the installation of the shoring system should be undertaken by a contractor experienced in the installation of shoring systems. The shoring system should be designed and installed in accordance with latest edition of Ontario Regulation 213/91 under the OHSA and the 2006 Fourth Edition of the Canadian Foundation Engineering Manual (CFEM). For tiebacks that may be required to laterally support the shoring system and will extend onto neighboring properties, permission may need to be obtained from the neighboring property owners.

As previously indicated, it is anticipated that the base of the excavation for the footings and the elevator pit may undergo basal instability or base type failure in the form of piping or heave due to the groundwater level located at 0.3 m and 0.4 m below the footing and the presence of the permeable gravel which extends below the groundwater level. To prevent base type failure, it is recommended that the groundwater level should be lowered by at least 1.0 m below the depth of the excavation for the proposed building prior to start of excavation and construction activities. This may be achieved by installing deep sumps and pumping with high-capacity pumps or by the use of well points. A specialist dewatering contractor must be consulted for this purpose.

Alternatively, the excavation may be undertaken within the confines of a shoring system that is also designed to cut-off groundwater flows towards the excavation and minimize groundwater flows into the shored excavation. In this regard, seepage of groundwater into the shored excavation should still be anticipated but may be removed by collecting the water at low points within the excavation and pumping from sumps. In areas of high infiltration, a higher seepage rate should be anticipated and high-capacity pumps may be required to keep the excavation dry.

The shoring system as well as adjacent settlement sensitive structures (buildings) and infrastructure should be monitored for movement (deflection) on a periodic basis during construction operations.

A pre-construction condition survey of buildings and infrastructure within the influence zone of the construction should be undertaken prior to start of construction activities including shoring installation activity.

It is recommended that vibration monitoring be conducted at the site and at adjacent existing buildings and infrastructure during the installation of the shoring system and during construction of the new building to ensure the existing structures and infrastructure are not damaged as a result of the construction activities and shoring installation.

Many geologic materials deteriorate rapidly upon exposure to meteorological elements. Unless otherwise specifically indicated in this report, walls and floors of excavations must be protected from moisture, desiccation, and frost action throughout the course of construction.

10.3 De-Watering Requirements

As discussed above, to prevent base type failure, it is recommended that the groundwater level should be lowered by at least 1.0 m below the depth of the excavation for the proposed building prior to start of excavation and construction activities. This may be achieved by installing a deep sumps and pumping with high-capacity pumps or by the use of well points. A specialist dewatering contractor must be consulted for this purpose.

Seepage of the surface and subsurface water into shored excavations is anticipated. However, it should be possible to collect water entering the excavations at low points and to remove it by conventional pumping techniques. In areas of high water infiltration or in areas where more permeable soils may exist, a higher seepage rate should be anticipated and may require high capacity pumps.

The dewatering of excavations on site during short-term construction operations is not expected to adversely impact adjacent existing structures and infrastructure.

It has been assumed that the maximum excavation depth at the site will be approximately 4.0 m below existing grade and would necessitate groundwater removal from the site. It is noteworthy to mention that legislation came into force in Ontario on March 29, 2016 to regulate groundwater takings for construction dewatering purposes. Prior to March 29, 2016, a Category 2 Permit to Take Water (PTTW) was required from the Ontario Ministry of the Environment and Climate Change (MOECC) for groundwater takings related to construction dewatering, where taking volumes in excess of 50 m3/day, but less than 400 m3/day, and the taking duration was no more than 30 consecutive days. The new legislation replaces the Category 2 PTTW for construction dewatering with a new process under the Environmental Activity and Sector Registry (EASR). The EASR is an on-line registry, which allows persons engaged in prescribed activities, such as water takings, to register with the Ministry of Environment, Conservation and Parks (MECP)instead of applying for a PTTW.

To be eligible for the new EASR process, the construction dewatering taking must be less than 400 m3/day under normal conditions. The water taking can be groundwater, storm water, or a combination of both. It should be noted that the 30-consecutive day limit on the water taking under the old Category 2 PTTW process has been removed in the new EASR process. Also, it should be noted that the EASR process requires two technical studies be prepared by a Qualified Person, prior to any water taking. These studies include a Water Taking Report, which provides assurance that the taking will not cause any unacceptable impacts, and a Discharge Plan, which provides assurance that the discharge will not result in any adverse impacts to the environment. A significant advantage of the new EASR process over the former Category 2 PTTW process, is that the groundwater taking may begin immediately after completing the on-line registration of the taking and paying the applicable fee, assuming the accompanying technical studies have been completed. The former PTTW process typically took more than 90 days, which had the potential to impact construction schedules.

Although this investigation has estimated the groundwater levels at the time of the fieldwork, and commented on dewatering and general construction problems, conditions may be present which are difficult to establish from standard boring and excavating techniques and which may affect the type and nature of dewatering procedures used by the contractor in practice. These conditions include local and seasonal fluctuations in the groundwater table, erratic changes in the soil profile, thin layers of soil with large or small permeabilities compared with the soil mass, etc. Only carefully controlled tests using pumped wells and observation wells will yield the quantitative data on groundwater volumes and pressures that are necessary to adequately engineer construction dewatering systems.

11. Backfilling Requirements and Suitability of On-Site Soils for Backfilling Purposes

The materials to be excavated from the site will consist of asphalt, topsoil, sand and gravel fill, clayey silt with sand and gravel fill, and glacial till. The excavated soils are not considered suitable for use under structural elements and for backfilling purposes and therefore must be also disposed of off-site or may be used in landscaped areas. However, subject to additional geotechnical testing at the start of construction, select portions of the glacial till (free of cobbles and boulders) above the groundwater level may be reused as backfill material outside the building.

It is anticipated that the majority of the material required for underfloor fill and backfilling purposes would have to be imported and should preferably conform to the following specifications:

- Engineered fill under the slab-on-grade area - OPSS 1010 Granular B Type II placed in 300 mm thick lifts and each lift compacted to 98 percent standard Proctor maximum dry density (SPMDD).

- Backfill against elevator pit walls and foundation walls outside the building – OPSS 1010 Granular B Type II placed in 300 mm thick lifts and each lift compacted to 98 percent of the SPMDD inside the building (elevator pit walls) and 95 percent SPMDD outside the building.

- Backfill in exterior services trenches – on-site approved excavated material or OPSS 1010 Select Subgrade Material (SSM) placed in 300 mm thick lifts and each lift compacted to 95 percent of the SPMDD.

12. Subsurface Concrete and Steel Requirements

Chemical tests limited to pH, sulphate, chloride and resistivity were undertaken on one (1) selected sample of the glacial till and a summary of the results is shown in Table VI. The laboratory certificate of analysis is attached in Appendix A.

Table VI: Corrosion Test Results on Soil Sample

Borehole – Sample No.	Soil Type	Depth (m)	pH	Sulphate (%)	Chloride (%)	Resistivity (ohm-cm)
BH 1 – SS5	Glacial Till	3.8 – 4.4	8.39	0.0091	0.0033	4270

The results indicate the soils have a negligible sulphate attack on subsurface concrete. The concrete should be designed in accordance with CSA A.23.1-14.

The results of the resistivity tests indicate that the glacial till is mildly corrosive to bare steel as per the National Association of Corrosion Engineers (NACE). Appropriate measures should be undertaken to protect the buried bare steel from corrosion.

13. General Comments

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

The information contained in this report is not intended to reflect on environmental aspects of the soils and groundwater. Should specific information be required, including for example the presence of pollutants, contaminants or other hazards in the soil, additional testing may be required.

We trust that the information contained in this report is satisfactory for your purposes. Should you have any questions, please contact this office.

Sincerely,

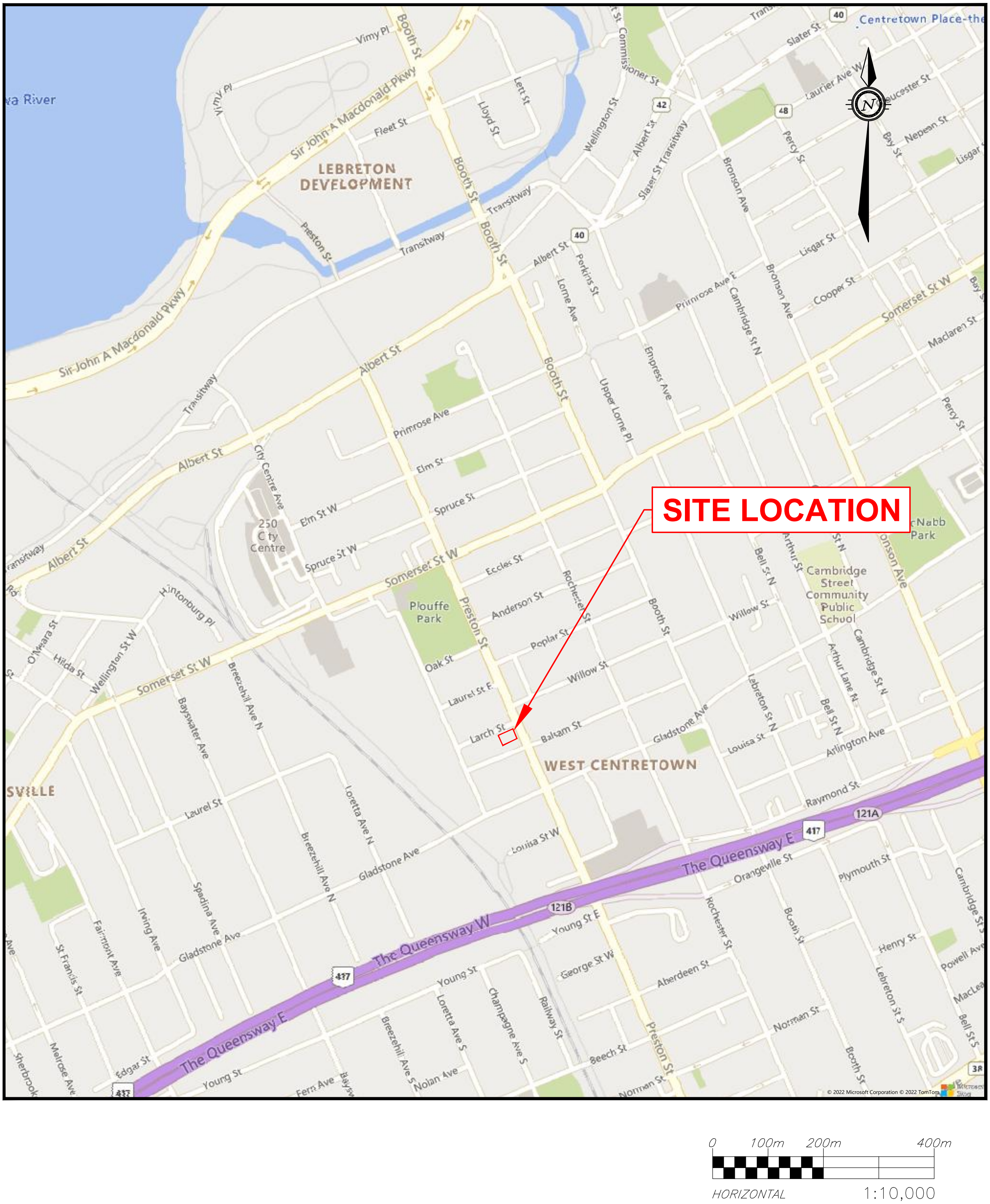
LICENSED PROFESSIONAL ENGINEER
M. S. ZAMMIT
100199988
Feb. 8, 2022
PROVINCE OF ONTARIO

Matt Zammit

Matthew Zammit, P.Eng.
Geotechnical Engineer
Earth and Environment

Susan M. Potyondy, P.Eng.
Senior Geotechnical Engineer
Earth and Environment

Figures



exp Services Inc.
100-2650 Queensview Drive
Ottawa, ON K2B 8H6

www.exp.com

DESIGN	I.T.
DRAWN	P.V.
DATE	JAN. 2022
FILE NO	OTT-21019479-A0

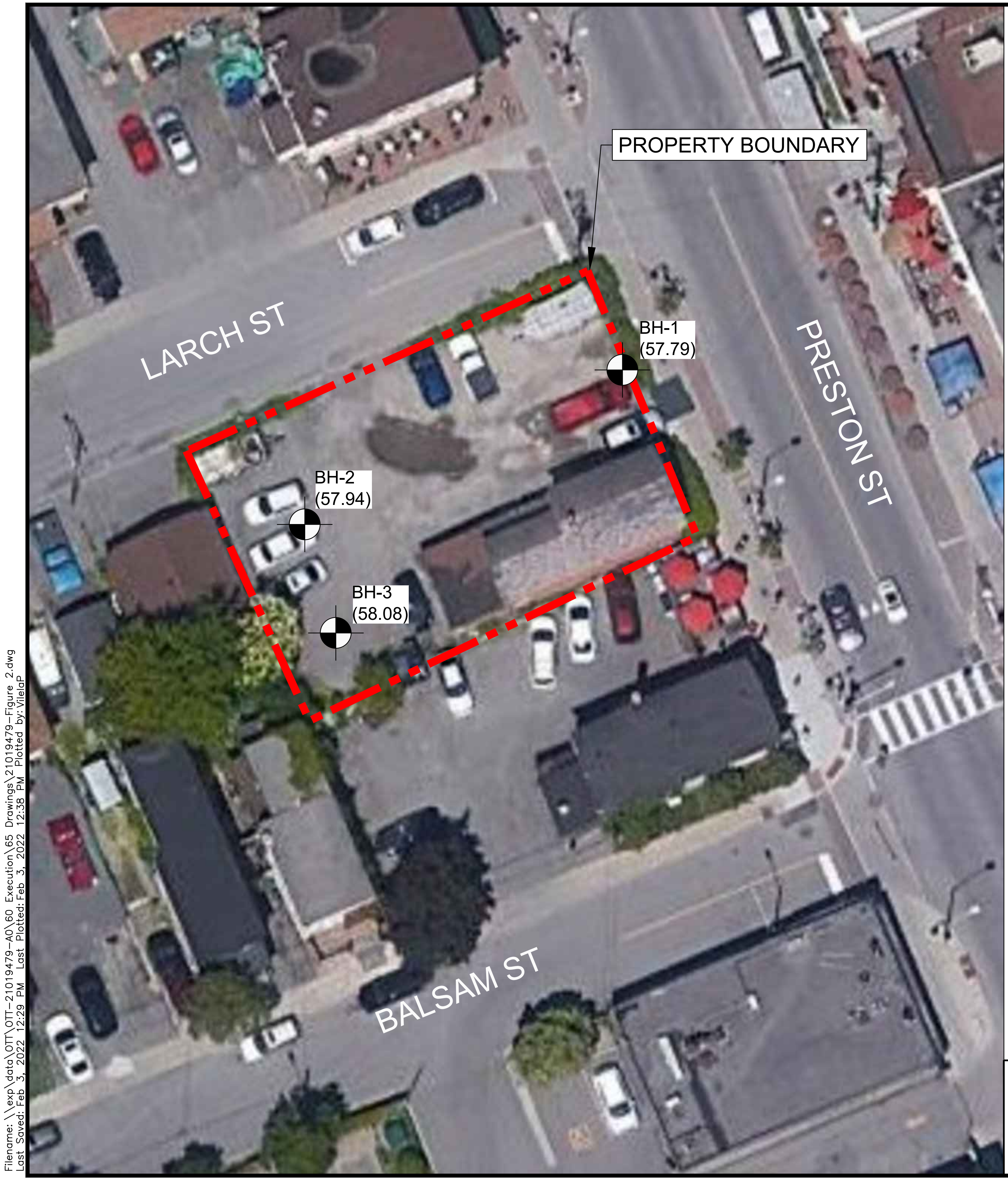
GEOTECHNICAL INVESTIGATION
PROPOSED SIX STOREY RESIDENTIAL DEVELOPMENT
224 PRESTON STREET, OTTAWA, ONTARIO

SITE LOCATION PLAN

SCALE
1:10,000

SKETCH NO
FIG 1

Filename: \\exp\data\OTT\OTT-21019479-A0\60 Execution\65 Drawings\21019479-Figure 1.dwg
Last Saved: Feb 3, 2022 11:33 AM Last Plotted: Feb 3, 2022 11:43 AM Plotted by: VilelaP



LEGEND

PROPERTY BOUNDARY	▬ ▪ ▬
BOREHOLE LOCATION	⊕

NOTES:

1. THE BOUNDARIES AND SOIL TYPES HAVE BEEN ESTABLISHED ONLY AT BOREHOLE LOCATIONS. BETWEEN BOREHOLES THEY ARE ASSUMED AND MAY BE SUBJECT TO CONSIDERABLE ERROR.
2. SOIL SAMPLES WILL BE RETAINED IN STORAGE FOR THREE MONTHS AND THEN DESTROYED UNLESS THE CLIENT ADVISES THAT AN EXTENDED TIME PERIOD IS REQUIRED.
3. ASPHALT AND TOPSOIL QUANTITIES SHOULD NOT BE ESTABLISHED FROM THE INFORMATION PROVIDED AT THE BOREHOLE LOCATIONS.
4. BOREHOLE ELEVATIONS SHOULD NOT BE USED TO DESIGN BUILDING(S) OR FLOOR SLABS OR PARKING LOTS GRADES.
5. THIS DRAWING FORMS PART OF THE REPORT PROJECT NUMBER AS REFERENCED AND SHOULD BE USED ONLY IN CONJUNCTION WITH THIS REPORT.

0 3m 6m 12m
HORIZONTAL 1:300

exp Services Inc.
100-2650 Queensview Drive
Ottawa, ON K2B 8H6

www.exp.com

DESIGN I.T.
DRAWN P.V.
DATE JAN. 2022
FILE NO OTT–21019479–A0

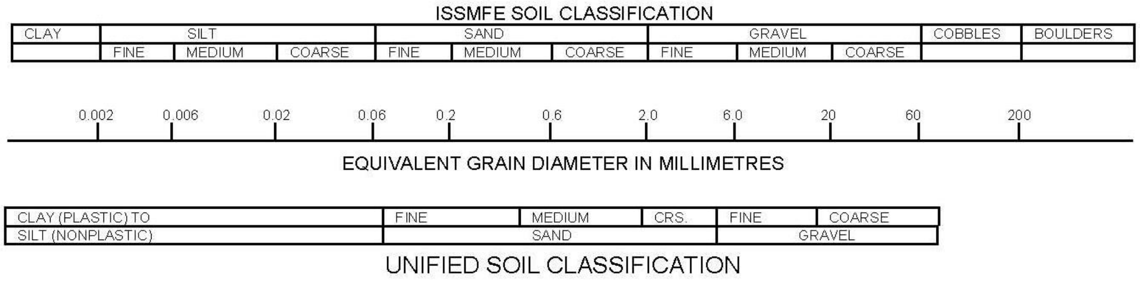
GEOTECHNICAL INVESTIGATION
PROPOSED SIX STOREY RESIDENTIAL DEVELOPMENT
224 PRESTON STREET, OTTAWA, ONTARIO

BOREHOLE LOCATION PLAN

SCALE 1:300
SKETCH NO
FIG 2

Notes On Sample Descriptions

1. All sample descriptions included in this report follow the Canadian Foundations Engineering Manual soil classification system. This system follows the standard proposed by the International Societv for Soil Mechanics and Foundation Engineering. Laboratory grain size analyses provided by **exp** Services Inc. also follow the same system. Different classification systems may be used by others; one such system is the Unified Soil Classification. Please note that, with the exception of those samples where a grain size analysis has been made, all samples are classified visually. Visual classification is not sufficiently accurate to provide exact grain sizing or precise differentiation between size classification systems.



2. Fill: Where fill is designated on the borehole log it is defined as indicated by the sample recovered during the boring process. The reader is cautioned that fills are heterogeneous in nature and variable in density or degree of compaction. The borehole description may therefore not be applicable as a general description of site fill materials. All fills should be expected to contain obstruction such as wood, large concrete pieces or subsurface basements, floors, tanks, etc., none of these may have been encountered in the boreholes. Since boreholes cannot accurately define the contents of the fill, test pits are recommended to provide supplementary information. Despite the use of test pits, the heterogeneous nature of fill will leave some ambiguity as to the exact composition of the fill. Most fills contain pockets, seams, or layers of organically contaminated soil. This organic material can result in the generation of methane gas and/or significant ongoing and future settlements. Fill at this site may have been monitored for the presence of methane gas and, if so, the results are given on the borehole logs. The monitoring process does not indicate the volume of gas that can be potentially generated nor does it pinpoint the source of the gas. These readings are to advise of the presence of gas only, and a detailed study is recommended for sites where any explosive gas/methane is detected. Some fill material may be contaminated by toxic/hazardous waste that renders it unacceptable for deposition in any but designated land fill sites; unless specifically stated the fill on this site has not been tested for contaminants that may be considered toxic or hazardous. This testing and a potential hazard study can be undertaken if requested. In most residential/commercial areas undergoing reconstruction, buried oil tanks are common and are generally not detected in a conventional geotechnical site investigation.

3. Till: The term till on the borehole logs indicates that the material originates from a geological process associated with glaciation. Because of this geological process the till must be considered heterogeneous in composition and as such may contain pockets and/or seams of material such as sand, gravel, silt or clay. Till often contains cobbles (60 to 200 mm) or boulders (over 200 mm). Contractors may therefore encounter cobbles and boulders during excavation, even if they are not indicated by the borings. It should be appreciated that normal sampling equipment cannot differentiate the size or type of any obstruction. Because of the horizontal and vertical variability of till, the sample description may be applicable to a very limited zone; caution is therefore essential when dealing with sensitive excavations or dewatering programs in till materials.

Log of Borehole BH-01

Project No: OTT-21019479-A0

Project: Proposed Six-Story Commercial and Residential Building

Location: 224 Preston Street, Ottawa, ON

Figure No. 3

Page. 1 of 1

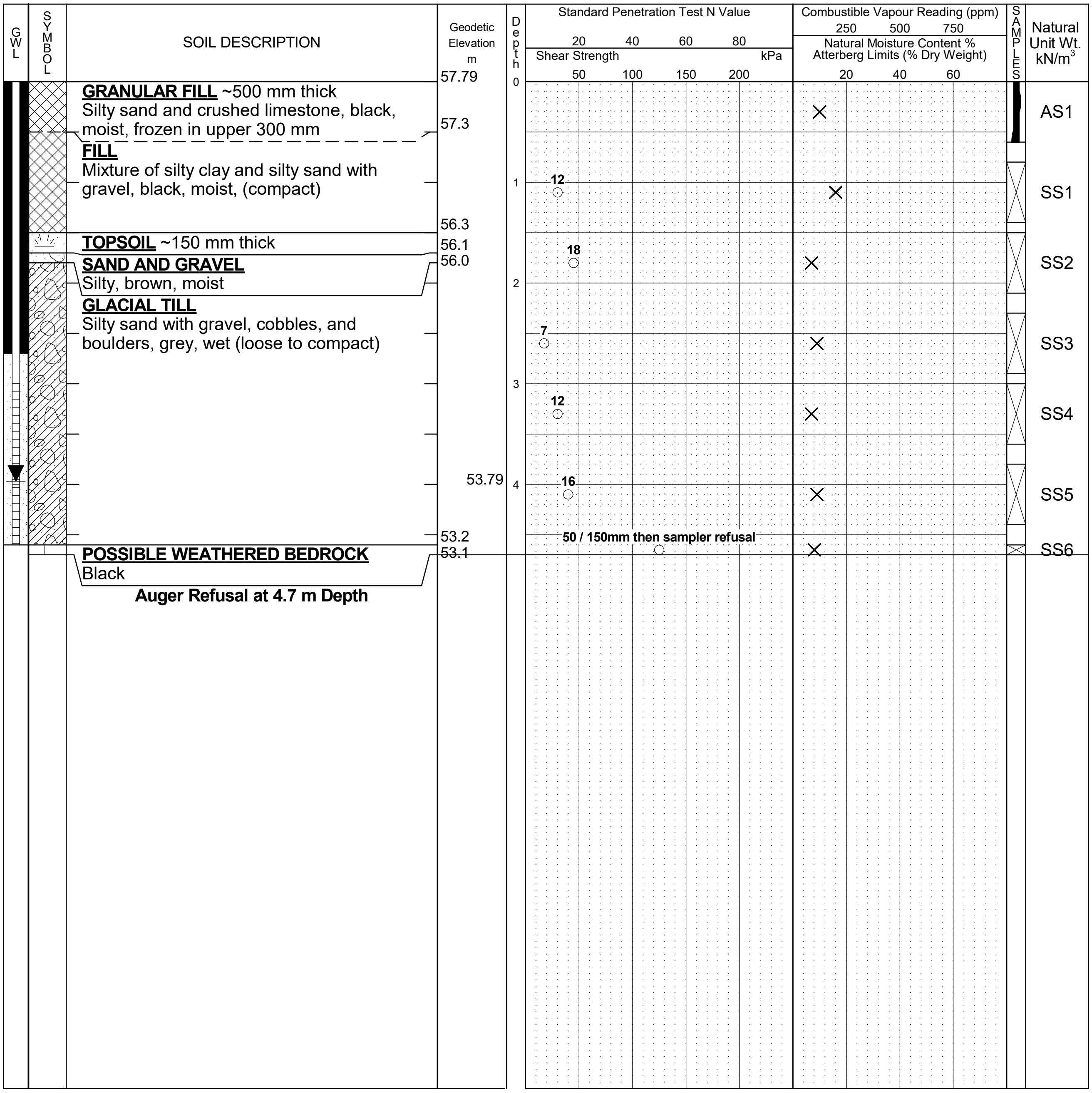
Date Drilled: 'Dec 3, 2021

Drill Type: CME 55 Truck Mounted Drill Rig

Datum: Geodetic Elevation

Logged by: M.Z. Checked by: I.T.

Split Spoon Sample	☒	Combustible Vapour Reading	□
Auger Sample	◨	Natural Moisture Content	×
SPT (N) Value	○	Atterberg Limits	├─○
Dynamic Cone Test	—	Undrained Triaxial at % Strain at Failure	⊕
Shelby Tube	■	Shear Strength by Penetrometer Test	▲
Shear Strength by Vane Test	+ S		



SOIL DESCRIPTION	Geodetic Elevation m
	57.79
GRANULAR FILL ~500 mm thick Silty sand and crushed limestone, black, moist, frozen in upper 300 mm	57.3
FILL Mixture of silty clay and silty sand with gravel, black, moist, (compact)	56.3
TOPSOIL ~150 mm thick	56.1
SAND AND GRAVEL Silty, brown, moist	56.0
GLACIAL TILL Silty sand with gravel, cobbles, and boulders, grey, wet (loose to compact)	53.2
POSSIBLE WEATHERED BEDROCK Black	53.1

Auger Refusal at 4.7 m Depth

Standard Penetration Test N Value	SAMPLES
	AS1
12	SS1
18	SS2
7	SS3
12	SS4
16	SS5
50 / 150mm then sampler refusal	SS6

NOTES:

1. Borehole data requires interpretation by EXP before use by others
2. A 32 mm diameter monitoring well installed as shown.
3. Field work supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-21019479-A0

WATER LEVEL RECORDS

Date	Water Level (m)	Hole Open To (m)
Upon Completion	3.6	4.4
Dec 22, 2021	4.0	

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %

Log of Borehole BH-02

Project No: OTT-21019479-A0

Project: Proposed Six-Story Commercial and Residential Building

Location: 224 Preston Street, Ottawa, ON

Figure No. 4

Page. 1 of 1

Date Drilled: 'Dec 11, 2021

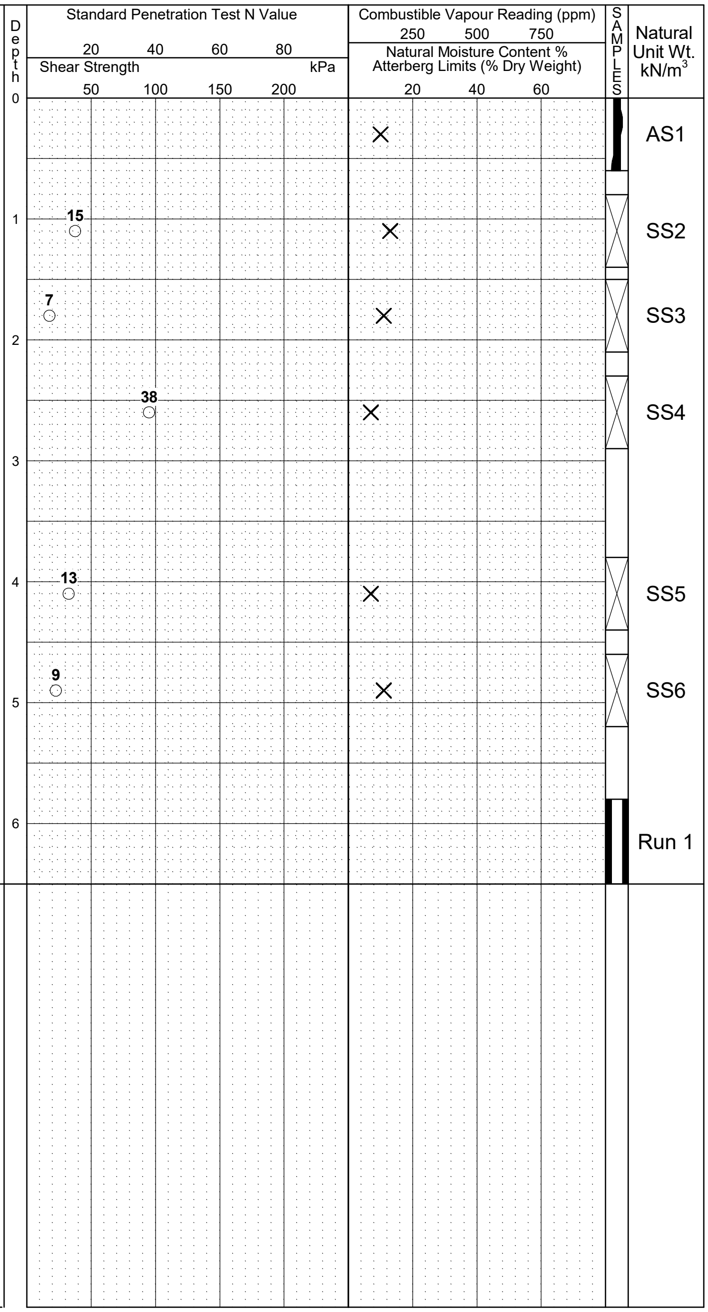
Drill Type: CME 55 Truck Mounted Drill Rig

Datum: Geodetic Elevation

Logged by: M.Z. Checked by: I.T.

Split Spoon Sample	☒	Combustible Vapour Reading	☐
Auger Sample	◧	Natural Moisture Content	×
SPT (N) Value	○	Atterberg Limits	⊢○
Dynamic Cone Test	—	Undrained Triaxial at % Strain at Failure	⊕
Shelby Tube	■	Shear Strength by Penetrometer Test	▲
Shear Strength by Vane Test	+ S		

SOIL DESCRIPTION	Geodetic Elevation m
	57.94
GRANULAR FILL Silty sand and gravel, brown, moist, (compact)	
	57.2
FILL Mixture of silty clay and silty sand with gravel, brown, moist, (compact)	
	56.5
GLACIAL TILL Silty sand with gravel, cobbles and boulders, grey, wet, (loose to dense) augers grinding from 2.3 m to 3.0 m depths	
	54.5
GRAVEL With silt and sand, brown, wet, (loose to compact)	
	52.1
LIMESTONE BEDROCK Black	
	51.4
Borehole Terminated at 6.5 m Depth	



NOTES:
1. Borehole data requires interpretation by EXP before use by others
2. Borehole backfilled upon completion of drilling.
3. Field work supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-21019479-A0

WATER LEVEL RECORDS

Date	Water Level (m)	Hole Open To (m)
Upon Completion	-	-

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %
1	5.8 - 6.5	91	36

Log of Borehole BH-03

Project No: OTT-21019479-A0

Project: Proposed Six-Story Commercial and Residential Building

Location: 224 Preston Street, Ottawa, ON

Figure No. 5

Page. 1 of 1

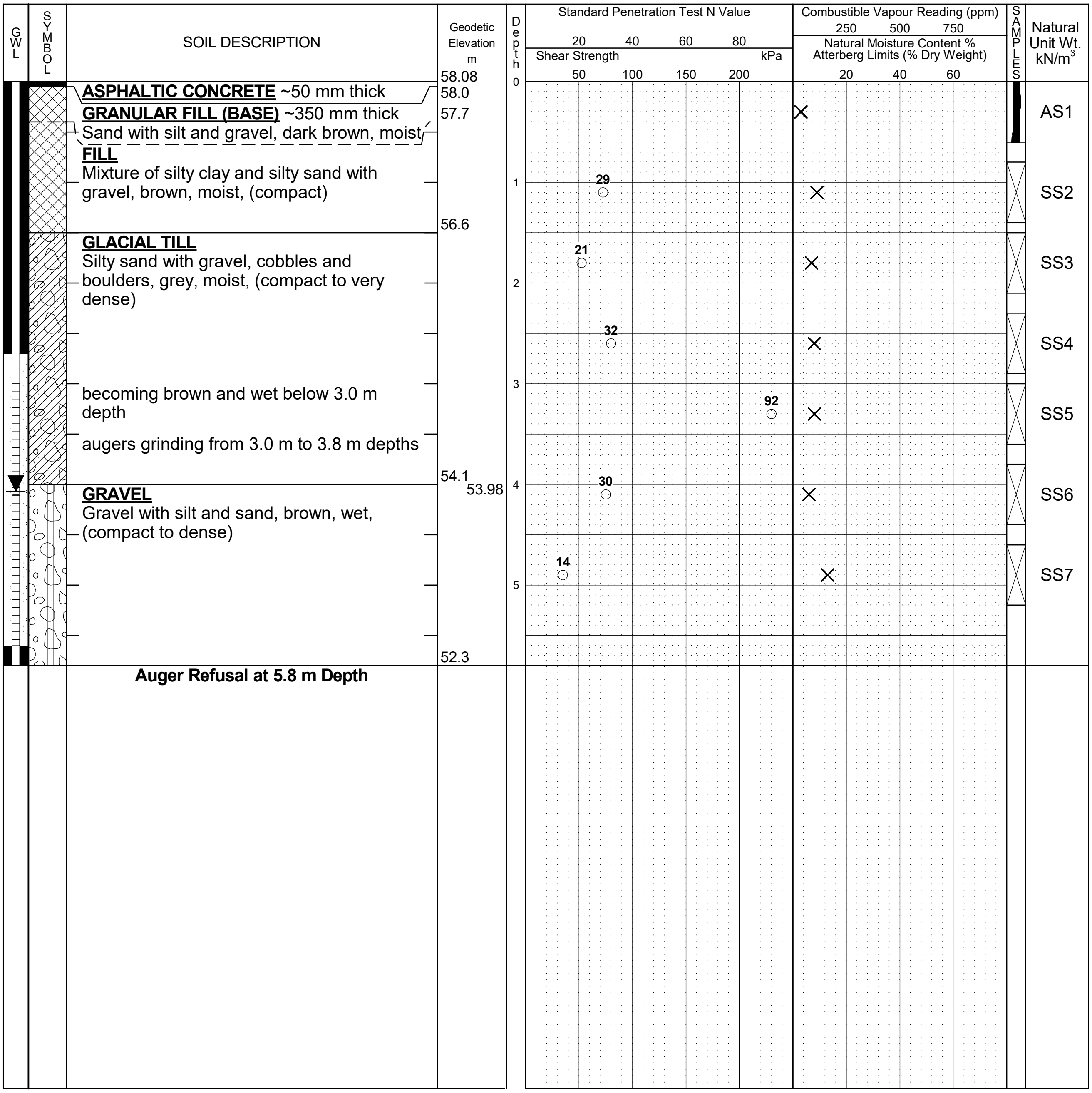
Date Drilled: 'Dec 3/11, 2021

Drill Type: CME 55 Truck Mounted Drill Rig

Datum: Geodetic Elevation

Logged by: M.Z. Checked by: I.T.

Split Spoon Sample	☒	Combustible Vapour Reading	☐
Auger Sample	◨	Natural Moisture Content	×
SPT (N) Value	○	Atterberg Limits	├─○
Dynamic Cone Test	───	Undrained Triaxial at % Strain at Failure	⊕
Shelby Tube	■	Shear Strength by Penetrometer Test	▲
Shear Strength by Vane Test	+ S		



ASPHALTIC CONCRETE ~50 mm thick

GRANULAR FILL (BASE) ~350 mm thick
Sand with silt and gravel, dark brown, moist

FILL
Mixture of silty clay and silty sand with gravel, brown, moist, (compact)

GLACIAL TILL
Silty sand with gravel, cobbles and boulders, grey, moist, (compact to very dense)

becoming brown and wet below 3.0 m depth

augers grinding from 3.0 m to 3.8 m depths

GRAVEL
Gravel with silt and sand, brown, wet, (compact to dense)

Auger Refusal at 5.8 m Depth

NOTES:
1. Borehole data requires interpretation by EXP before use by others
2. A 32 mm diameter monitoring well installed as shown.
3. Field work supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-21019479-A0

WATER LEVEL RECORDS

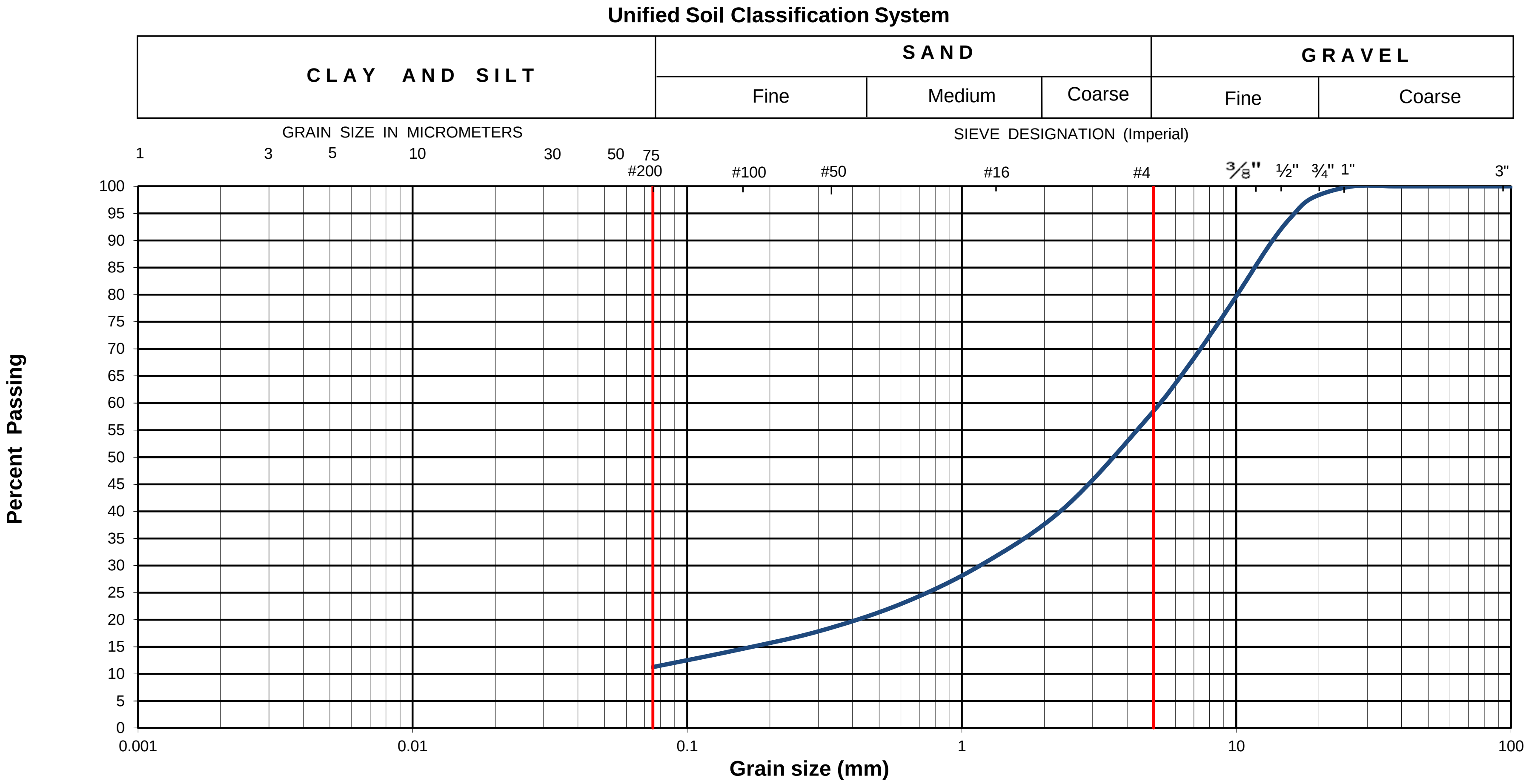
Date	Water Level (m)	Hole Open To (m)
Upon Completion	-	-
Dec 22, 2021	4.1	

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %

EXP Services Inc.
100-2650 Queensview Drive
Ottawa, ON   K2B 8H6

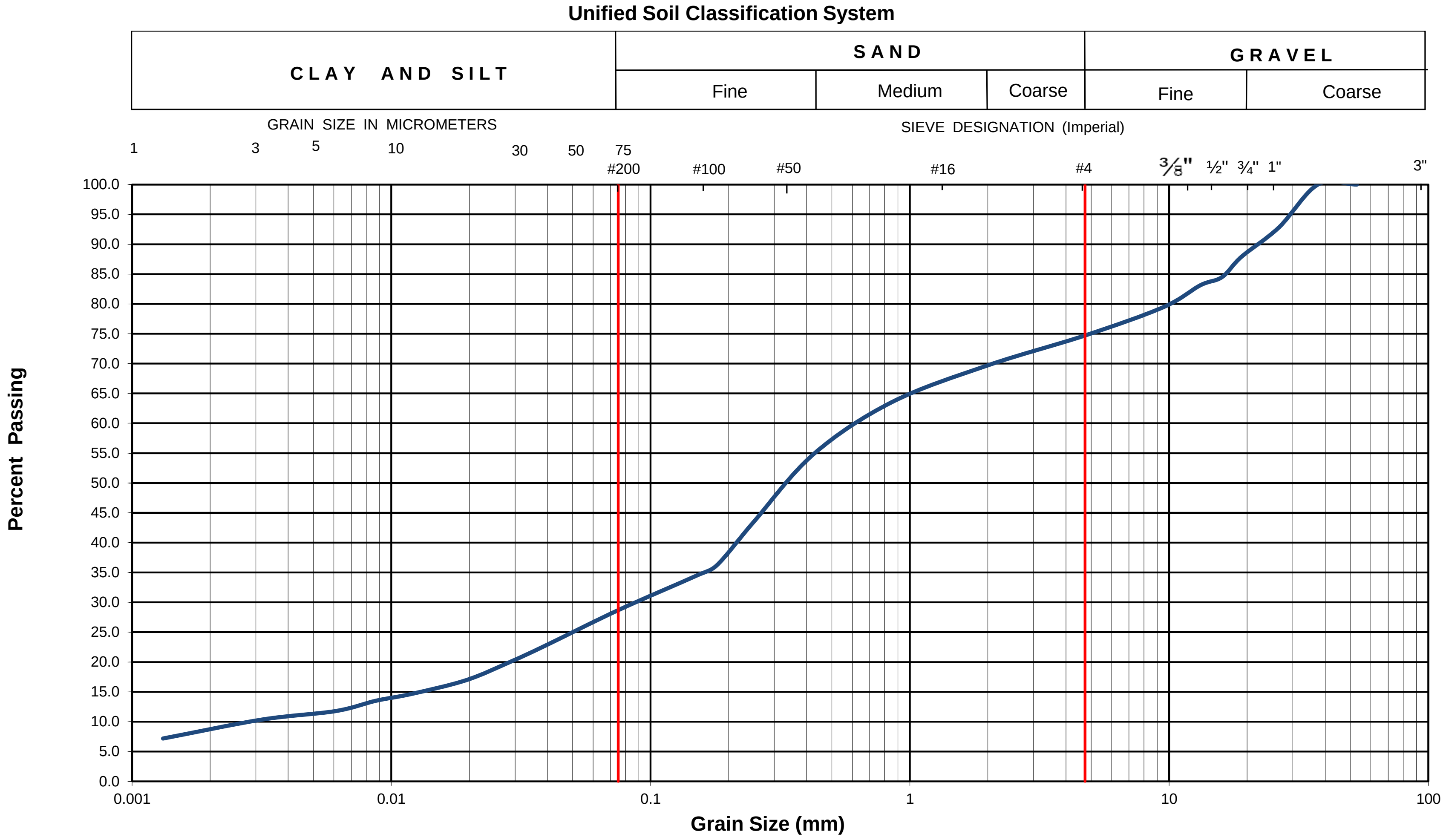
Grain-Size Distribution Curve
Method of Test For Sieve Analysis of Aggregate
ASTM C-136



EXP Project No.:	**OTT-21019479-A0**	**Project Name :**	**Proposed Six Story Commercial and Residential Building**						
Client :	**Ottawa Carleton Construction Group**	**Project Location :**	**224 Preston Street, Ottawa, Ontario**						
Date Sampled :	**December 3, 2021**	**Borehole No:**	**BH3**	**Sample:**	**AS1**			**Depth (m) :**	**0-0.6**
Sample Composition :		**Gravel (%)**	**43**	**Sand (%)**	**46**	**Silt & Clay (%)**	**11**	**Figure :**	**6**
Sample Description :	**GRANULAR FILL (BASE): Poorly Graded Sand with Silt and Gravel (SP-SM)**								

EXP Services Inc.
100-2650 Queensview Drive
Ottawa, ON   K2B 8H6

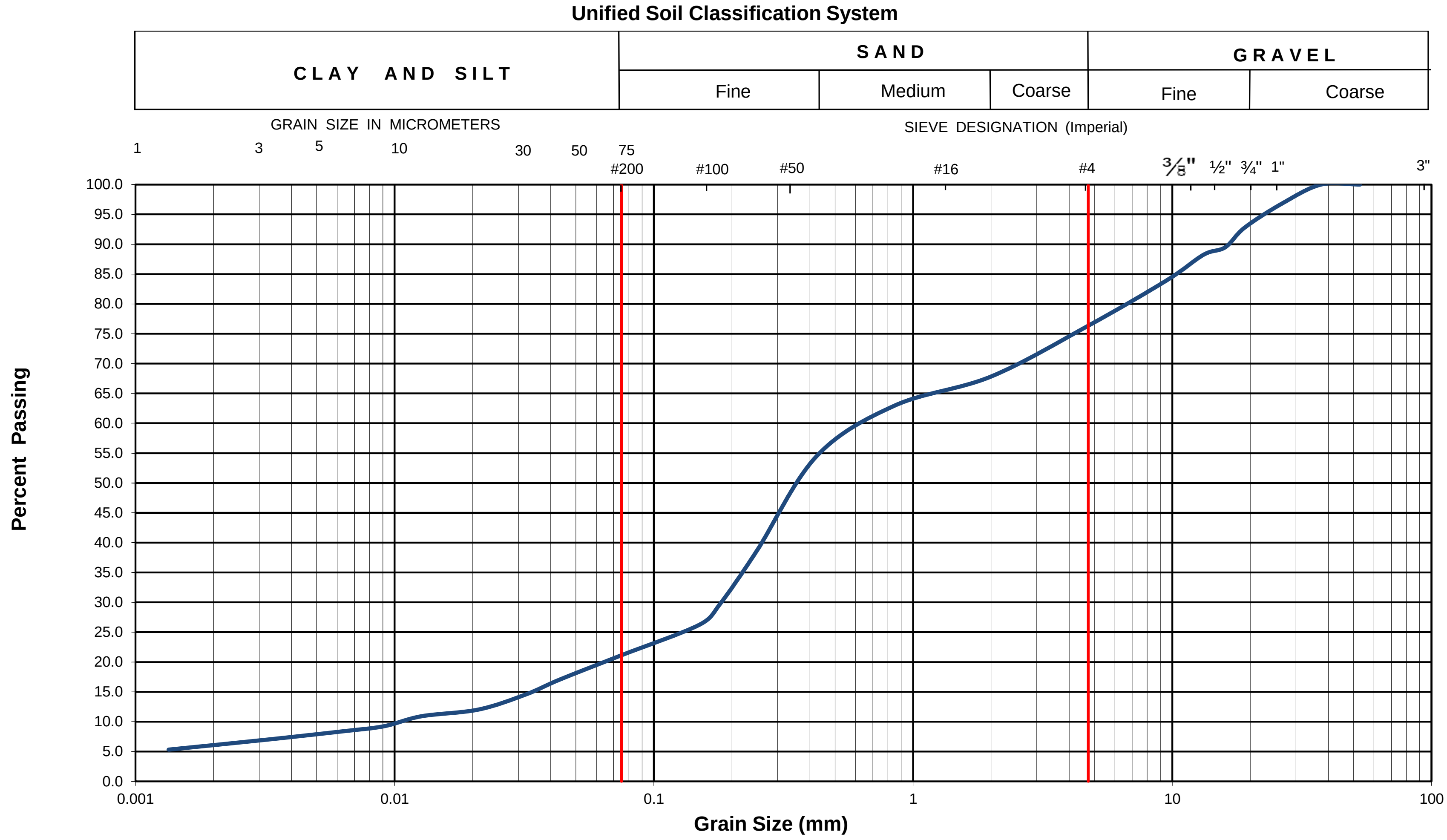
Grain-Size Distribution Curve
Method of Test For Particle Size Analysis of Soil
ASTM C-136/ASTM D422



EXP Project No.:	**OTT-21019479-A0**	**Project Name :**	**Proposed Six Story Commercial and Residential Building**						
Client :	**Ottawa Carleton Construction Group**	**Project Location :**	**224 Preston Street, Ottawa, Ontario**						
Date Sampled :	**December 3, 2021**	**Borehole No:**	**BH1**	**Sample No.:**	**SS4**			**Depth (m) :**	**3.0-3.6**
Sample Description :		**% Silt and Clay**	**29**	**% Sand**	**46**	**% Gravel**	**25**	**Figure :**	**7**
Sample Description :	**GLACIAL TILL: Silty Sand with Gravel (SM)**								

EXP Services Inc.
100-2650 Queensview Drive
Ottawa, ON K2B 8H6

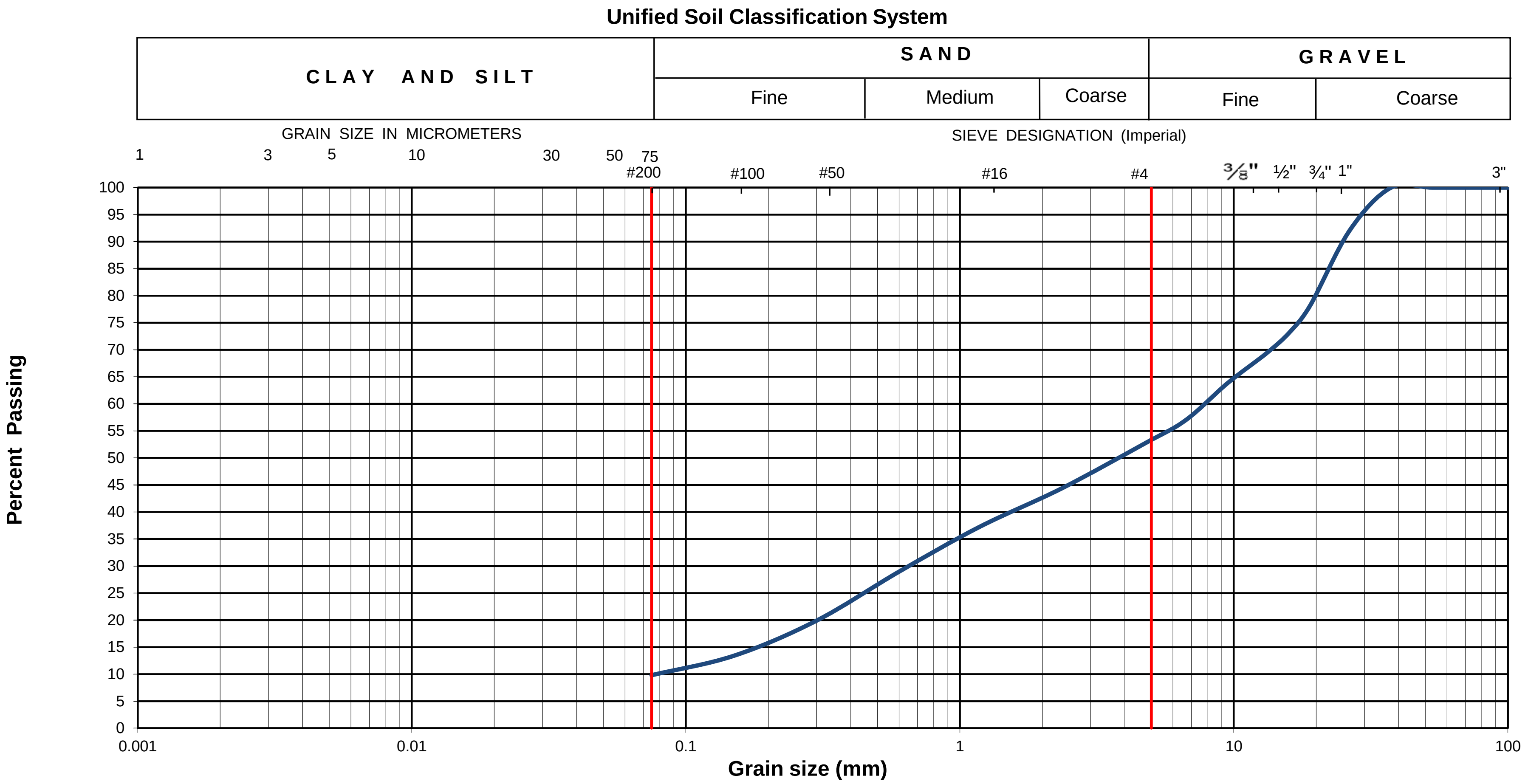
Grain-Size Distribution Curve
Method of Test For Particle Size Analysis of Soil
ASTM C-136/ASTM D422



EXP Project No.:	**OTT-21019479-A0**	**Project Name :**	**Proposed Six Story Commercial and Residential Building**				
Client :	**Ottawa Carleton Construction Group**	**Project Location :**	**224 Preston Street, Ottawa, Ontario**				
Date Sampled :	**December 3, 2021**	**Borehole No:**	**BH3**	**Sample No.:**	**SS3**	**Depth (m) :**	**1.5-2.1**
Sample Description :		**% Silt and Clay**	**21**	**% Sand**	**55**	**% Gravel**	**24**
Sample Description :	**GLACIAL TILL: Silty Sand with Gravel (SM)**					**Figure :**	**8**

EXP Services Inc.
100-2650 Queensview Drive
Ottawa, ON   K2B 8H6

Grain-Size Distribution Curve
Method of Test For Sieve Analysis of Aggregate
ASTM C-136



EXP Project No.:	**OTT-21019479-A0**	**Project Name :**	**Proposed Six Story Commercial and Residential Building**						
Client :	**Ottawa Carleton Construction Group**	**Project Location :**	**224 Preston Street, Ottawa, Ontario**						
Date Sampled :	**December 11, 2021**	**Borehole No:**	**BH2**	**Sample:**	**SS5**	**Depth (m) :**	**3.8-4.4**		
Sample Composition :		**Gravel (%)**	**47**	**Sand (%)**	**43**	**Silt & Clay (%)**	**10**	**Figure :**	**9**
Sample Description :	**Poorly Graded Gravel with Silt and Sand (GP-GM)**								

DRY BEDROCK CORES

WET BEDROCK CORES

EXP Services Inc. www.exp.com
t: +1.613.688.1899 | f: +1.613.225.7337
2650 Queensview Drive, Suite 100
Ottawa, ON K2B 8H6, Canada

borehole no. **BH2**	core runs Run 1: 5.8 m - 6.5 m	project Location: 224 Preston Street, Ottawa, ON	project no. OTT-21019479-A0
date cored Dec 11, 2021		Rock Core Photographs	FIG 10

Appendix A:
Laboratory Certificate of Analysis

5835 COOPERS AVENUE
MISSISSAUGA, ONTARIO
CANADA L4Z 1Y2
TEL (905)712-5100
FAX (905)712-5122
http://www.agatlabs.com

CLIENT NAME: EXP SERVICES INC
2650 QUEENSVIEW DRIVE, UNIT 100
OTTAWA, ON K2B8H6
(613) 688-1899

ATTENTION TO: Matthew Zammit

PROJECT: OTT-21019479

AGAT WORK ORDER: 21Z845489

SOIL ANALYSIS REVIEWED BY: Amanjot Bhela, Inorganic Lab Manager

DATE REPORTED: Dec 24, 2021

PAGES (INCLUDING COVER): 5

VERSION*: 1

Should you require any information regarding this analysis please contact your client services representative at (905) 712-5100

Notes

Disclaimer:

- *All work conducted herein has been done using accepted standard protocols, and generally accepted practices and methods. AGAT test methods may incorporate modifications from the specified reference methods to improve performance.*
- *All samples will be disposed of within 30 days after receipt unless a Long Term Storage Agreement is signed and returned. Some specialty analysis may be exempt, please contact your Client Project Manager for details.*
- *AGAT's liability in connection with any delay, performance or non-performance of these services is only to the Client and does not extend to any other third party. Unless expressly agreed otherwise in writing, AGAT's liability is limited to the actual cost of the specific analysis or analyses included in the services.*
- *This Certificate shall not be reproduced except in full, without the written approval of the laboratory.*
- *The test results reported herewith relate only to the samples as received by the laboratory.*
- *Application of guidelines is provided "as is" without warranty of any kind, either expressed or implied, including, but not limited to, warranties of merchantability, fitness for a particular purpose, or non-infringement. AGAT assumes no responsibility for any errors or omissions in the guidelines contained in this document.*
- *All reportable information as specified by ISO/IEC 17025:2017 is available from AGAT Laboratories upon request.*

Member of: Association of Professional Engineers and Geoscientists of Alberta (APEGA)
Western Enviro-Agricultural Laboratory Association (WEALA)
Environmental Services Association of Alberta (ESAA)

AGAT Laboratories is accredited to ISO/IEC 17025 by the Canadian Association for Laboratory Accreditation Inc. (CALA) and/or Standards Council of Canada (SCC) for specific tests listed on the scope of accreditation. AGAT Laboratories (Mississauga) is also accredited by the Canadian Association for Laboratory Accreditation Inc. (CALA) for specific drinking water tests. Accreditations are location and parameter specific. A complete listing of parameters for each location is available from www.cala.ca and/or www.scc.ca. The tests in this report may not necessarily be included in the scope of accreditation. Measurement Uncertainty is not taken into consideration when stating conformity with a specified requirement.

Certificate of Analysis

AGAT WORK ORDER: 21Z845489
PROJECT: OTT-21019479

CLIENT NAME: EXP SERVICES INC
SAMPLING SITE:224 Preston St., Ottawa

ATTENTION TO: Matthew Zammit
SAMPLED BY:EXP

Inorganic Chemistry (Soil)

DATE RECEIVED: 2021-12-16

DATE REPORTED: 2021-12-24

			SAMPLE DESCRIPTION:	BH1 SS5 12.5'-14.5'
			SAMPLE TYPE:	Soil
			DATE SAMPLED:	2021-12-03
Parameter	**Unit**	**G / S**	**RDL**	**3358461**
Chloride (2:1)	µg/g		2	33
Sulphate (2:1)	µg/g		2	91
pH (2:1)	pH Units		NA	8.39
Electrical Conductivity (2:1)	mS/cm		0.005	0.234
Resistivity (2:1) (Calculated)	ohm.cm		1	4270

Comments: RDL - Reported Detection Limit; G / S - Guideline / Standard

3358461 EC, pH, Chloride and Sulphate were determined on the extract obtained from the 2:1 leaching procedure (2 parts DI water: 1 part soil). Resistivity is a calculated parameter.

Analysis performed at AGAT Toronto (unless marked by *)

Certified By:

Amanjot Bhela

AMANJOT BHELA

Quality Assurance

CLIENT NAME: EXP SERVICES INC

PROJECT: OTT-21019479

SAMPLING SITE:224 Preston St., Ottawa

AGAT WORK ORDER: 21Z845489

ATTENTION TO: Matthew Zammit

SAMPLED BY:EXP

Soil Analysis

RPT Date: Dec 24, 2021			DUPLICATE			Method Blank	REFERENCE MATERIAL			METHOD BLANK SPIKE			MATRIX SPIKE		
PARAMETER	Batch	Sample Id	Dup #1	Dup #2	RPD		Measured Value	Acceptable Limits		Recovery	Acceptable Limits		Recovery	Acceptable Limits	
								Lower	Upper		Lower	Upper		Lower	Upper
Inorganic Chemistry (Soil)															
Chloride (2:1)	3358453		150	146	2.7%	< 2	99%	70%	130%	109%	80%	120%	106%	70%	130%
Sulphate (2:1)	3358453		6	6	NA	< 2	94%	70%	130%	105%	80%	120%	103%	70%	130%
pH (2:1)	3358453		7.15	7.19	0.6%	NA	100%	80%	120%	NA			NA		
Electrical Conductivity (2:1)	3347188		0.179	0.180	0.6%	< 0.005	104%	80%	120%	NA			NA		

Comments: NA signifies Not Applicable.
pH duplicates QA acceptance criteria was met relative as stated in Table 5-15 of Analytical Protocol document.

Duplicate NA: results are under 5X the RDL and will not be calculated.

Certified By: Amanjot Bhela

AMANJOT BHELA

5835 COOPERS AVENUE
MISSISSAUGA, ONTARIO
CANADA L4Z 1Y2
TEL (905)712-5100
FAX (905)712-5122
http://www.agatlabs.com

Method Summary

CLIENT NAME: EXP SERVICES INC

PROJECT: OTT-21019479

SAMPLING SITE:224 Preston St., Ottawa

AGAT WORK ORDER: 21Z845489

ATTENTION TO: Matthew Zammit

SAMPLED BY:EXP

PARAMETER	AGAT S.O.P	LITERATURE REFERENCE	ANALYTICAL TECHNIQUE
Soil Analysis			
Chloride (2:1)	INOR-93-6004	modified from SM 4110 B	ION CHROMATOGRAPH
Sulphate (2:1)	INOR-93-6004	modified from SM 4110 B	ION CHROMATOGRAPH
pH (2:1)	INOR 93-6031	modified from EPA 9045D and MCKEAGUE 3.11	PH METER
Electrical Conductivity (2:1)	INOR-93-6036	modified from MSA PART 3, CH 14 and SM 2510 B	EC METER
Resistivity (2:1) (Calculated)	INOR-93-6036	McKeague 4.12, SM 2510 B,SSA #5 Part 3	CALCULATION

AGAT Laboratories

5835 Coopers Avenue
Mississauga, Ontario L4Z 1Y2
Ph: 905.712.5100 Fax: 905.712.5122
webearth.agatlabs.com

Chain of Custody Record

If this is a Drinking Water sample, please use Drinking Water Chain of Custody Form (potable water consumed by humans)

Laboratory Use Only

Work Order #: 21Z845489

Cooler Quantity: one bag.

Arrival Temperatures: 20.6 | 20.3 | 20.0
8.4 | 8.5 | 8.7

Custody Seal Intact: ☐Yes ☐No ☐N/A

Notes: on icepack.

Report Information:

Company: ExP
Contact: Matthew Zammit
Address: 2650 Queensview drive Suite 100
Ottawa ON K2B 8H6
Phone: 613-688-1899 Fax:
Reports to be sent to:
1. Email: Matthew.Zammit@exp.com
2. Email:

Regulatory Requirements:

(Please check all applicable boxes)

☐ Regulation 153/04
Table ____ (Indicate One)
☐Ind/Com
☐Res/Park
☐Agriculture
Soil Texture (Check One)
☐Coarse
☐Fine

☐ Excess Soils R406
Table ____ (Indicate One)
☐ Regulation 558
☐ CCME

☐ Sewer Use
☐Sanitary ☐Storm
____ Region
☐ Prov. Water Quality Objectives (PWQO)
☐ Other
____ Indicate One

Is this submission for a **Record of Site Condition?** ☐ Yes ☐ No

Report Guideline on *Certificate of Analysis* ☐ Yes ☐ No

Turnaround Time (TAT) Required:

Regular TAT (Most Analysis) ☒ 5 to 7 Business Days

Rush TAT (Rush Surcharges Apply)

☐ 3 Business Days ☐ 2 Business Days ☐ Next Business Day

OR Date Required (Rush Surcharges May Apply): ____

Please provide prior notification for rush TAT
**TAT is exclusive of weekends and statutory holidays*

For 'Same Day' analysis, please contact your AGAT CPM

Project Information:

Project: OTT-21019479
Site Location: 224 Preston st, Ottawa
Sampled By: ExP
AGAT ID #: ____ PO: ____

Please note: If quotation number is not provided, client will be billed full price for analysis.

Invoice Information:

Bill To Same: Yes ☒ No ☐

Company:
Contact:
Address:
Email:

Sample Matrix Legend

B	Biota
GW	Ground Water
O	Oil
P	Paint
S	Soil
SD	Sediment
SW	Surface Water

Sample Identification	Date Sampled	Time Sampled	# of Containers	Sample Matrix	Comments/ Special Instructions	Field Filtered - Metals, Hg, CrVI, DOC Y / N	O. Reg 153			PAHs	Total PCBs ☐ Aroclor	VOC	O. Reg 558	O. Reg 406		Salt - EC/SAR	PH	Sulphate	Chloride	Resistivity				Potentially Hazardous or High Concentration (Y/N)
							Metals & Inorganics	Metals - ☐ CrVI, ☐ Hg, ☐ HWSB	BTEX, F1-F4 PHCs Analyze F4G if required ☐ Yes ☐ No				Landfill Disposal Characterization TCLP: TCLP: ☐M&I ☐VOCs ☐ABNs ☐B(a)P☐PCBs	Excess Soils SPLP Rainwater Leach SPLP: ☐Metals ☐VOCs ☐SVOCs	Excess Soils Characterization Package pH, ICPMS Metals, BTEX, F1-F4									
BH 1 SS5 12.5'-14.5'	Dec 3/21	AM PM	1														✓	/	/	/				
		AM PM																						
		AM PM																						
		AM PM																						
		AM PM																						
		AM PM																						
		AM PM																						
		AM PM																						
		AM PM																						
		AM PM																						
		AM PM																						
		AM PM																						

Samples Relinquished By (Print Name and Sign):	Date	Time	Samples Received By (Print Name and Sign):	Date	Time
Ryan DiGiuseppe	Dec 15/21	7:00pm	C. Griffith	21/12/16	08h00
Ca to Pulo	21/12/16	16h00		Dec 17	8:45

Page ____ of ____

Nº: T114907

Legal Notification

This report was prepared by EXP Services Inc. (EXP) for the account of Ottawa Carleton Construction Ltd.

Any use which a third party makes of this report, or any reliance on or decisions to be made based on it, are the responsibility of such third parties. EXP accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this project.

Report Distribution

Fernando Matos, Ottawa Carleton Construction Ltd., fernando@ottawacarletonconstruction.com

Frank Pocari, Ottawa Carleton Construction Ltd., frank.pocari @ottawacarletonconstruction.com