Geotechnical Engineering

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Preliminary Geotechnical Investigation

Proposed Multi-Storey Building
20 Mountain Crescent
Ottawa, Ontario

Prepared For

Surface Developments

August 25, 2020

Report PG5462-1

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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by Surface Developments to conduct a preliminary geotechnical investigation for the proposed multi-storey building to be located at 20 Mountain Crescent in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2 of this report).

The objectives of the geotechnical investigation were to:

- ❏ Determine the subsoil and groundwater conditions at this site by means of boreholes.
- ❏ Provide geotechnical recommendations for the design of the proposed development including construction considerations which may affect the design.

The following report has been prepared specifically and solely for the aforementioned project which is described herein. It contains our findings and includes geotechnical recommendations pertaining to the design and construction of the subject development as they are understood at the time of writing this report.

2.0 Proposed Development

Details of the proposed development were not available during the preparation of this report. However, it is understood that the preliminary concepts include a multi-storey building with one or more underground parking levels. It is anticipated that the footprint of the proposed building will occupy the majority of the subject site.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the preliminary geotechnical investigation was carried out on July 31, 2020. At that time, 3 boreholes were advanced to a maximum depth of 6.7 m. The borehole locations were distributed in a manner to provide general coverage of the subject site. The approximate locations of the boreholes are shown on Drawing PG5462-1 - Test Hole Location Plan included in Appendix 2.

All boreholes were advanced using a track-mounted auger drill rig, which was operated by a two-person crew. All fieldwork was conducted under the full-time supervision of our personnel under the direction of a senior engineer. The drilling procedure consisted of augering to the required depths at the selected locations, sampling and testing the overburden.

Sampling and In Situ Testing

Soil samples were collected from the boreholes using two different techniques, namely, sampled directly from the auger flights (AU) or collected using a 50 mm diameter split-spoon (SS) sampler. All samples were visually inspected and initially classified on site and subsequently placed in sealed plastic bags. All samples were transported to our laboratory for further examination and classification. The depths at which the auger and split spoon samples were recovered from the boreholes are shown as AU and SS, respectively, on the Soil Profile and Test Data sheets presented in Appendix 1.

A Standard Penetration Test (SPT) was conducted in conjunction with the recovery of the split spoon samples. The SPT results are recorded as "N" values on the Soil Profile and Test Data sheets. The "N" value is the number of blows required to drive the split spoon sampler 300 mm into the soil after a 150 mm initial penetration using a 63.5 kg hammer falling from a height of 760 mm.

The overburden thickness was evaluated by dynamic cone penetration tests (DCPT) at BH 1 and BH 2. The DCPT consists of driving a steel drill rod, equipped with a 50 mm diameter cone at the tip, using a 63.5 kg hammer falling from a height of 760 mm. The number of blows required to drive the cone into the soil is recorded for each 300 mm increment.

The subsurface conditions observed in the test holes were recorded in detail in the field. The soil profiles are presented on the Soil Profile and Test Data sheets in Appendix 1 of this report.

Groundwater

Monitoring wells were installed in all boreholes to permit monitoring of the groundwater levels subsequent to the completion of the current sampling program. All groundwater observations are noted on the Soil Profile and Test Data sheets presented in Appendix 1.

3.2 Field Survey

The test hole locations were selected by Paterson to provide general coverage of the proposed development, taking into consideration the existing site features and underground utilities. The test hole locations and ground surface elevation at each test hole location were surveyed by Paterson and referenced to a geodetic datum. The location of the test holes and ground surface elevation at each test hole location are presented on Drawing PG5462-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

Soil samples were recovered from the subject site and visually examined in our laboratory to review the results of the field logging. Soil samples will be stored for a period of one month after this report is completed, unless otherwise directed.

3.4 Analytical Testing

One (1) soil sample was submitted for analytical testing to assess the potential for exposed ferrous metals and the potential of sulphate attacks against subsurface concrete structures. The sample was analyzed to determine its concentration of sulphate and chloride along with its resistivity and pH. The laboratory test results are shown in Appendix 1 and are discussed in Section 6.5.

4.0 Observations

4.1 Surface Conditions

The subject site is currently occupied by a single-storey residential building fronting onto Mountain Crescent. The subject site is bordered by a 3-storey residential building to the north, Mountain Crescent to the east, a one storey commercial building to the south and Daze Street to the west. The existing ground surface across the site is relatively level at approximate geodetic elevations 90 to 91 m.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile at the test hole locations located within the subject site consists of an approximate 1.4 to 1.8 m thick fill layer underlying a topsoil layer. The fill was generally observed to consist of a brown silty clay with sand.

A silty clay deposit was encountered underlying the fill. The silty clay was generally observed to consist of a stiff to firm, brown silty clay with sand becoming a firm, grey silty clay at depths ranging from 2.3 to 3.4 m below the existing ground surface.

A glacial till deposit was observed underlying the silty clay at approximate depths of 6.1 m within BH 2 and BH 3. The glacial till deposit was generally observed to consist of a compact, grey silty sand with gravel, cobbles, boulders.

Practical refusal to the DCPT was encountered at approximate depths of 12.4 and 15.1 m at boreholes BH 1 and BH 2, respectively.

Bedrock

Based on available geological mapping, the bedrock in the area consists of shale of the Carlsbad formation as well as interbedded limestone and shale of the Verulam formation with a drift thickness of 25 to 50 m.

4.3 Groundwater

Groundwater levels were measured in the monitoring wells installed in the boreholes on August 10, 2020. The observed groundwater levels are summarized in Table 1 and are provided on the Soil Profile and Test Data sheets in Appendix 1.

Table 1 - Summary of Groundwater Levels

Borehole Number	Ground Surface Elev. (m)	Measured Groundwater Level		Recording Date
		Depth (m)	Elevation (m)	
BH 1	90.96	2.20	88.76	August 10, 2020
BH 2	89.96	4.15	85.81	August 10, 2020
BH 3	90.55	4.19	86.36	August 10, 2020

Note: The ground surface elevation at the test hole locations are referenced to a geodetic datum.

It should be noted that the groundwater level readings in the monitoring wells can be influenced by surface water becoming trapped in the backfill materials. Long-term groundwater levels can also be estimated based on the observed colour and consistency of the recovered soil samples. Based on these observations, the long-term groundwater table can be expected at approximately 3 to 4 m below ground surface.

However, it should be noted that groundwater levels are subject to seasonal fluctuations. Therefore, the groundwater level could vary at the time of construction.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is considered suitable for the proposed development. Based on the subsurface conditions encountered in the test holes and the anticipated building loads, it is recommended that foundation support for the proposed multi-storey building consist of:

- a raft foundation bearing on the undisturbed, stiff to firm silty clay or compact glacial till deposit, depending on the depth of the lowest basement level, or

- a deep foundation, such as end-bearing piles, which extends to the bedrock surface.

Due to the presence of the silty clay deposit, a permissible grade raise restriction will be required for the subject site. This is provided in Section 5.3.

The above and other considerations are further discussed in the following sections.

5.2 Site Grading and Preparation

Stripping Depth

Topsoil, and fill, containing significant amounts of deleterious or organic materials, should be stripped from under any building, paved areas, pipe bedding and other settlement sensitive structures.

Existing foundation walls and other construction debris should be entirely removed from within the building perimeter. Under paved areas, existing construction remnants, such as foundation walls, should be excavated to a minimum of 1 m below final grade.

Protection of Subgrade (Raft Foundation)

Where a raft foundation is utilized, it is recommended that a minimum 50 to 75 mm thick lean concrete mud slab be placed on the undisturbed, silty clay or glacial till subgrade shortly after the completion of the excavation. The main purpose of the mud slab is to reduce the risk of disturbance of the subgrade under the traffic of workers and equipment.

The final excavation to the raft bearing surface level and the placing of the mud slab should be done in smaller sections to avoid exposing large areas of the silty clay or glacial till to potential disturbance due to drying.

Compacted Granular Fill Working Platform (Pile Foundation)

Should the proposed multi-storey building be supported on a driven pile foundation, the use of heavy equipment would be required to install the piles (i.e. pile driving crane). It is conventional practice to install a compacted granular fill layer, at a convenient elevation, to allow the equipment to access the site without getting stuck and causing significant disturbance.

A typical working platform could consist of 0.6 m of OPSS Granular B, Type II crushed stone which is placed and compacted to a minimum of 98% of its standard Proctor maximum dry density (SPMDD) in lifts not exceeding 300 mm in thickness.

Once the piles have been driven and cut off, the working platform can be regraded, and soil tracked in, or soil pumping up from the pile installation locations, can be bladed off and the surface can be topped up, if necessary, and re-compacted to act as the substrate for further fill placement for the basement slab.

Pressure Relief Chamber

Should the proposed building have 2 or more basement levels with a raft foundation, a pressure relief chamber is recommended to be installed along with collection pipes within excavated within the glacial till deposit. The collection pipe trenching should extend along the proposed building perimeter and lead to the pressure relief chamber. Additional details can be provided, as required.

Fill Placement

Fill used for grading beneath the proposed buildings should consist of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. This material should be tested and approved prior to delivery to the site. The fill should be placed in lifts no greater than 300 mm thick and compacted using suitable compaction equipment for the lift thickness. Fill placed beneath the building and paved areas should be compacted to at least 98% of the material's standard Proctor maximum dry density (SPMDD).

Non-specified existing fill, along with site-excavated soil, can be used as general landscaping fill where settlement of the ground surface is of minor concern. This material should be spread in thin lifts and at least compacted by the tracks of the spreading equipment to minimize voids. If this material is to be used to build up the subgrade level for areas to be paved, it should be compacted in thin lifts to at least 95% of the material's SPMDD.

Non-specified existing fill and site-excavated soils are not suitable for use as backfill against foundation walls unless used in conjunction with a composite drainage membrane.

5.3 Foundation Design

Raft Foundation- One Basement Level

The proposed multi-storey building may be supported on a raft foundation where the contact pressure is within the values provided below. For 1 basement level, it is anticipated that the excavation will extend to a depth such that the underside of the raft slab would be placed between geodetic elevations of 87 to 86 m.

The amount of settlement of the raft slab will be dependent on the sustained raft contact pressure. The loading conditions for the contact pressure are based on sustained loads, that are generally taken to be 100% Dead Load and 50% Live Load. The contact pressure provided considers the stress relief associated with the soil removal required for 1 level of underground parking.

For 1 level of underground parking, a bearing resistance value at SLS (contact pressure) of **120 kPa** will be considered acceptable for a raft supported on the undisturbed, firm silty clay bearing surface. The factored bearing resistance (contact pressure) at ULS can be taken as **180 kPa**. For this case, the modulus of subgrade reaction was calculated to be **5 MPa/m** for a contact pressure of **120 kPa**.

The raft foundation design is required to consider the relative stiffness of the reinforced concrete slab and the supporting bearing medium. A geotechnical resistance factor of 0.5 was applied to the bearing resistance values at ULS.

Based on the following assumptions for the raft foundation, the proposed high-rise building can be designed using the above parameters with a total and differential settlement of 25 and 20 mm, respectively.

Raft Foundation - Two Basement Levels

For 2 basement levels, it is anticipated that the excavation will extend to a depth such that the underside of the raft slab would be placed between geodetic elevations of 84 to 83 m.

For this case, a bearing resistance value at SLS (contact pressure) of **250 kPa** will be considered acceptable for a raft supported on the undisturbed, compact glacial till bearing surface. The factored bearing resistance (contact pressure) at ULS can be taken as **500 kPa**. The modulus of subgrade reaction was calculated to be **8 MPa/m** for a contact pressure of **250 kPa**.

Based on the following assumptions for the raft foundation, the proposed high-rise building can be designed using the above parameters with a total and differential settlement of 25 and 20 mm, respectively.

Raft Foundation - Three Basement Levels

For 3 basement levels, it is expected that the excavation will extend to a depth, such that the underside of the raft slab would be placed between geodetic elevations of 81 to 80 m. The contact pressure provided considers the stress relief associated with the soil removal required for 3 levels of underground parking.

For this case, a bearing resistance value at SLS (contact pressure) of **300 kPa** will be considered acceptable for a raft supported on the undisturbed, compact glacial till bearing surface. The factored bearing resistance (contact pressure) at ULS can be taken as **600 kPa**. The modulus of subgrade reaction was calculated to be **10 MPa/m** for a contact pressure of **250 kPa**.

Based on the following assumptions for the raft foundation, the proposed multi-storey building can be designed using the above parameters with a total and differential settlement of 25 and 20 mm, respectively.

End Bearing Pile Foundation

If the raft slab bearing resistance values are insufficient for the proposed multi-storey building, a deep foundation system driven to refusal in the bedrock will be recommended for foundation support of the proposed building. For deep foundations, concrete-filled steel pipe piles are generally utilized in the Ottawa area. Applicable pile resistance values at SLS and ULS are given in Table 2. A resistance factor of 0.4 has been incorporated into the factored ULS values. Note that these are all geotechnical axial resistance values.

The geotechnical pile resistance values were estimated using the Hiley dynamic formula, to be confirmed during pile installation with a program of dynamic monitoring. Re-striking of all piles at least once will also be required after at least 48 hours have elapsed since initial driving.

Table 2 - Pile Foundation Design Data

Pile Outside Diameter (mm)	Pile Wall Thickness (mm)	Geotechnical Axial Resistance		Final Set (blows/ 12 mm)	Transferred Hammer Energy (kJ)
		SLS (kN)	Factored at ULS (kN)		
245	9	925	1110	6	27
245	11	1050	1260	6	31
245	13	1200	1440	6	35

The minimum centre-to-centre pile spacing is 2.5 times the pile diameter. The closer the piles are spaced, however, the more potential that the driving of subsequent piles in a group could have influence on piles in the group that have already been driven. These effects, primarily consisting of uplift of previously driven piles, are checked as part of the field review of the pile driving operations.

Prior to the commencement of production pile driving, a limited number of indicator piles should be installed across the site. It is recommended that each indicator pile be dynamically load tested to evaluate pile stresses, hammer efficiency, pile load transfer, and end-of-driving criteria for end-bearing in the bedrock.

Buildings founded on piles driven to refusal in the bedrock will have negligible post-construction settlement.

Permissible Grade Raise

Due to the presence of the silty clay deposit, a permissible grade raise restriction of **1.5 m** is recommended for grading at the subject site.

If higher than permissible grade raises are required, preloading with or without a surcharge, lightweight fill, and/or other measures should be investigated to reduce the risks of unacceptable long-term post construction total and differential settlements.

5.4 Design for Earthquakes

A seismic site response **Class C** should be used for the design of the proposed buildings at the subject site according to the OBC 2012. Soils underlying the subject site are not susceptible to liquefaction. Reference should be made to the latest revision of the OBC 2012 for a full discussion of the earthquake design requirements.

5.5 Basement Slab

Where a raft slab is utilized, a granular layer of OPSS Granular A will be required to allow for the installation of sub-floor services above the raft slab foundation. The thickness of the OPSS Granular A crushed stone will be dependent on the piping requirements.

For buildings founded on piles, it is recommended that the upper 200 mm of subfloor fill consists of 19 mm clear crushed stone. All backfill material within the footprint of the proposed building should be placed in maximum 300 mm thick loose layers and compacted to at least 98% of its SPMDD.

A sub-slab drainage system, consisting of lines of perforated drainage pipe sub-drains connected to a positive outlet, should be provided under the lowest level floor slab. The spacing of the sub-slab drainage pipes can be determined at the time of construction to confirm groundwater infiltration levels, if any.

5.6 Basement Wall

There are several combinations of backfill materials and retained soils that could be applicable for the basement walls of the subject structure. However, the conditions can be well-represented by assuming the retained soil consists of a material with an angle of internal friction of 30 degrees and a bulk (drained) unit weight of 20 kN/m^3.

Where undrained conditions are anticipated (i.e. below the groundwater level), the applicable effective (undrained) unit weight of the retained soil can be taken as 13 kN/m^3, where applicable. A hydrostatic pressure should be added to the total static earth pressure when using the effective unit weight.

Lateral Earth Pressures

The static horizontal earth pressure (p_o) can be calculated using a triangular earth pressure distribution equal to $K_o \cdot \gamma \cdot H$ where:

K_o = at-rest earth pressure coefficient of the applicable retained soil (0.5)
γ = unit weight of fill of the applicable retained soil (kN/m^3)
H = height of the wall (m)

An additional pressure having a magnitude equal to $K_o \cdot q$ and acting on the entire height of the wall should be added to the above diagram for any surcharge loading, q (kPa), that may be placed at ground surface adjacent to the wall. The surcharge pressure will only be applicable for static analyses and should not be used in conjunction with the seismic loading case.

Actual earth pressures could be higher than the “at-rest” case if care is not exercised during the compaction of the backfill materials to maintain a minimum separation of 0.3 m from the walls with the compaction equipment.

Seismic Earth Pressures

The total seismic force (P_{AE}) includes both the earth force component (P_o) and the seismic component (ΔP_{AE}).

The seismic earth force (ΔP_{AE}) can be calculated using $0.375 \cdot a_c \cdot \gamma \cdot H^2/g$ where:

a_c = $(1.45-a_{max}/g)a_{max}$
γ = unit weight of fill of the applicable retained soil (kN/m^3)
H = height of the wall (m)
g = gravity, 9.81 m/s^2

The peak ground acceleration, (a_{max}), for the Ottawa area is 0.32g according to OBC 2012. Note that the vertical seismic coefficient is assumed to be zero.

The earth force component (P_o) under seismic conditions can be calculated using $P_o = 0.5\ K_o\ \gamma\ H^2$, where $K_o = 0.5$ for the soil conditions noted above.
The total earth force (P_{AE}) is considered to act at a height, h (m), from the base of the wall, where:

$h = \{P_o \cdot (H/3) + \Delta P_{AE} \cdot (0.6 \cdot H)\}/P_{AE}$

The earth forces calculated are unfactored. For the ULS case, the earth loads should be factored as live loads, as per OBC 2012.

5.7 Pavement Structure

Car only parking areas, heavy truck parking areas and access lanes are anticipated at the subject site. The proposed pavement structures are presented in Tables 3 and 4.

Table 3 - Recommended Pavement Structure - Car Only Parking Areas

Thickness (mm)	Material Description
50	Wear Course - HL-3 or Superpave 12.5 Asphaltic Concrete
150	BASE - OPSS Granular A Crushed Stone
300	SUBBASE - OPSS Granular B Type II
SUBGRADE - Either fill, in situ soil, or OPSS Granular B Type I or II material placed over in situ soil or fill	

Table 4 - Recommended Pavement Structure - Access Lanes, Ramp and Heavy Truck Parking Areas

Thickness (mm)	Material Description
40	Wear Course - HL-3 or Superpave 12.5 Asphaltic Concrete
50	Binder Course - HL-8 or Superpave 19.0 Asphaltic Concrete
150	BASE - OPSS Granular A Crushed Stone
450	SUBBASE - OPSS Granular B Type II
SUBGRADE - Either fill, in situ soil, or OPSS Granular B Type I or II material placed over in situ soil or fill	

Minimum Performance Graded (PG) 58-34 asphalt cement should be used for this project.

If soft spots develop in the subgrade during compaction or due to construction traffic, the affected areas should be excavated and replaced with OPSS Granular B Type II material. The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 99% of the material's SPMDD using suitable vibratory equipment.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Waterproofing

Depending on the building configuration and the number of underground parking levels, a waterproofing program may be required. Foundation drainage and waterproofing details can be provided in a supplemental investigation upon finalization of the building design details.

6.2 Protection of Footings Against Frost Action

Perimeter foundations of heated structures are required to be insulated against the deleterious effects of frost action. Generally, a minimum of 1.5 m thick soil cover (or an equivalent combination of soil cover and foundation insulation) should be provided in this regard.

Exterior unheated foundations are more prone to deleterious movement associated with frost action than the exterior walls of the structure proper and require additional protection, such as soil cover of 2.1 m or a combination of soil cover and foundation insulation.

The foundations for the basement levels are expected to have sufficient frost protection due to the founding depth. However, it has been our experience that insufficient soil cover is typically provided to entrance ramps to underground parking garages. Paterson requests permission to review design drawings prior to construction to ensure proper frost protection is provided for these areas.

6.3 Excavation Side Slopes

The side slopes of excavations in the soil and fill overburden materials should either be cut back at acceptable slopes or should be retained by shoring systems from the start of the excavation until the structure is backfilled. Based on the depth of the proposed structure and the expected proximity to property lines, it is anticipated that a temporary shoring system will be required to support the excavation.

Unsupported Excavations

The excavation side slopes above the groundwater level extending to a maximum depth of 3 m should be excavated at 1H:1V or shallower. The shallower slope is required for excavation below groundwater level. The subsurface soils are considered to be a Type 2 and 3 soil according to the Occupational Health and Safety Act and Regulations for Construction Projects.

Excavated soil should not be stockpiled directly at the top of excavations and heavy equipment should be kept away from the excavation sides.

Slopes in excess of 3 m in height should be periodically inspected by the geotechnical consultant in order to detect if the slopes are exhibiting signs of distress.

A trench box is recommended to protect personnel working in trenches with steep or vertical sides. Services are expected to be installed by “cut and cover” methods and excavations should not remain open for extended periods of time.

Temporary Shoring

Temporary shoring is anticipated to be required to support the overburden soils during the proposed building excavation. The design and approval of the shoring system will be the responsibility of the shoring contractor and the shoring designer who is a licensed professional engineer and is hired by the shoring contractor. It is the responsibility of the shoring contractor to ensure that the temporary shoring is in compliance with safety requirements, designed to avoid any damage to adjacent structures and include dewatering control measures.

In the event that subsurface conditions differ from the approved design during the actual installation, it is the responsibility of the shoring contractor to commission the required experts to re-assess the design and implement the required changes. Furthermore, the design of the temporary shoring system should take into consideration a full hydrostatic condition which can occur during significant precipitation events.

The temporary shoring system may consist of a soldier pile and lagging system or steel sheet piles which could be cantilevered, anchored or braced. The shoring system is recommended to be adequately supported to resist toe failure.

Any additional loading due to street traffic, construction equipment, adjacent structures and facilities, etc., should be added to the earth pressures described below. The earth pressures acting on the shoring system may be calculated using the following parameters.

Table 5 - Soil Parameters	
Parameters	**Values**
Active Earth Pressure Coefficient (K_a)	0.33
Passive Earth Pressure Coefficient (K_p)	3
At-Rest Earth Pressure Coefficient (K_o)	0.5
Unit Weight (γ), kN/m^3	21
Submerged Unit Weight (γ), kN/m^3	13

The active earth pressure should be calculated where wall movements are permissible while the at-rest pressure should be calculated if no movement is permissible. The dry unit weight should be calculated above the groundwater level while the effective unit weight should be calculated below the groundwater level.

The hydrostatic groundwater pressure should be included to the earth pressure distribution wherever the effective unit weight are calculated for earth pressures. If the groundwater level is lowered, the dry unit weight for the soil/bedrock should be calculated full weight, with no hydrostatic groundwater pressure component.

For design purposes, the minimum factor of safety of 1.5 should be calculated.

Underpinning of Adjacent Structures

If the footings of the proposed building are to extend within the lateral support zone of adjacent building foundations, underpinning of the structures would be required. The depth of the underpinning will be dependent on the depth of the neighbouring foundations relative to the founding depth of the proposed building at the subject site. It is recommended that requirements for underpinning be evaluated prior to the start of construction or at the start of construction.

6.4 Groundwater Control

Depending on the building configuration and the number of underground parking levels, a waterproofing program may be required to mitigate the effects of groundwater table lowering.

A temporary Ministry of Environment, Conservation and Parks (MECP) permit to take water (PTTW) may be required if more than 400,000 L/day of ground and/or surface water are to be pumped during the construction phase.

For typical ground or surface water volumes being pumped during the construction phase, typically between 50,000 to 400,000 L/day, it is required to register on the Environmental Activity and Sector Registry (EASR). A minimum of two to four weeks should be allotted for completion of the EASR registration and the Water Taking and Discharge Plan to be prepared by a Qualified Person as stipulated under O.Reg. 63/16. If a project qualifies for a PTTW based upon anticipated conditions, an EASR will not be allowed as a temporary dewatering measure while awaiting the MECP review of the PTTW application.

6.5 Corrosion Potential and Sulphate

The results of analytical testing show that the sulphate content is less than 0.1%. This result is indicative that Type 10 Portland cement (normal cement) would be appropriate for this site. The chloride content and the pH of the sample indicate that they are not significant factors in creating a corrosive environment for exposed ferrous metals at this site, whereas the resistivity is indicative of a moderate to aggressive corrosive environment.

7.0 Recommendations

A materials testing and observation services program is a requirement for the provided foundation design data to be applicable. The following aspects of the program should be performed by the geotechnical consultant:

- ❑ Complete a supplemental investigation once design details are finalized.
- ❑ Review of the grading plan from a geotechnical perspective.
- ❑ Review of the Contractor's design of the temporary shoring system.
- ❑ Inspection and approval of the installation of the pressure relief chamber.
- ❑ Observation of all bearing surfaces prior to the placement of concrete.
- ❑ Sampling and testing of the concrete and fill materials used.
- ❑ Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.
- ❑ Observation of all subgrades prior to backfilling.
- ❑ Field density tests to determine the level of compaction achieved.
- ❑ Sampling and testing of the bituminous concrete including mix design reviews.

A report confirming that these works have been conducted in general accordance with our recommendations could be issued, upon request, following the completion of a satisfactory materials testing and observation program by the geotechnical consultant.

8.0 Statement of Limitations

The recommendations provided in this report are in accordance with our present understanding of the project. We request permission to review our recommendations when the drawings and specifications are completed.

A geotechnical investigation is a limited sampling of a site. Should any conditions at the site be encountered which differ from those at the test locations, we request immediate notification to permit reassessment of our recommendations.

The recommendations provided herein should only be used by the design professionals associated with this project. They are not intended for contractors bidding on or undertaking the work. The latter should evaluate the factual information provided in this report and determine its suitability and completeness for their intended construction schedule and methods. Additional testing may be required for their purposes.

The present report applies only to the project described in this document. Use of this report for purposes other than those described herein or by person(s) other than Surface Developments or their agents is not authorized without review by Paterson for the applicability of our recommendations to the altered use of the report.

Paterson Group Inc.

LICENSED PROFESSIONAL ENGINEER
August 25, 2020
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PROVINCE OF ONTARIO

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Report Distribution

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APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TESTING RESULTS

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Preliminary Geotechnnical Investigation
Proposed Redevelopment - 20 Mountain Crescent
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

DATE July 31, 2020

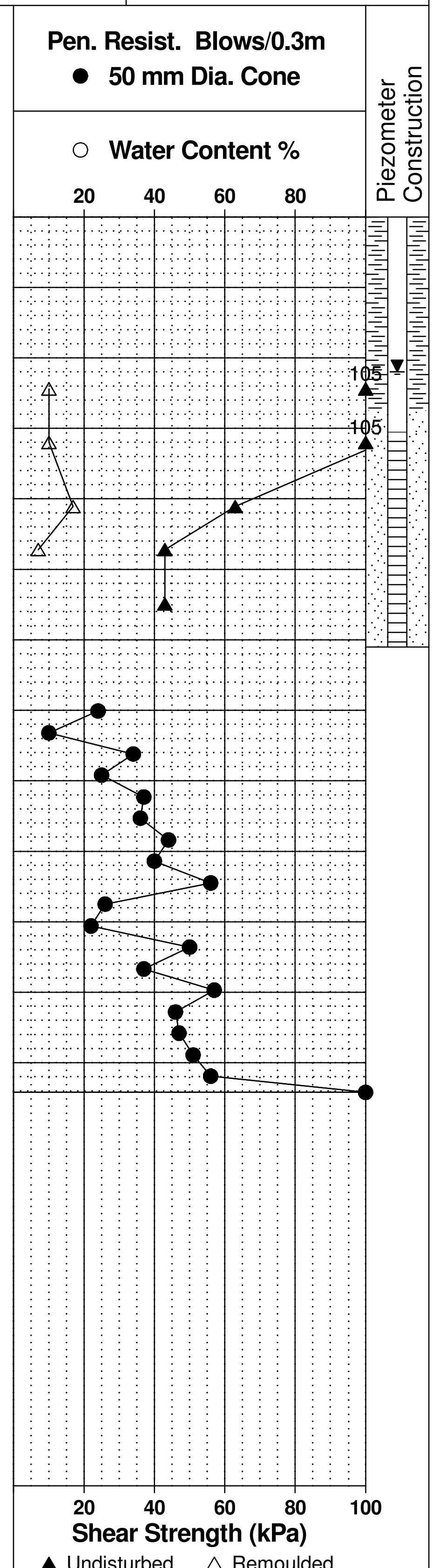
FILE NO. PG5462

HOLE NO. BH 1

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	90.96
TOPSOIL 0.05		AU	1				
FILL: Brown silty clay, some sand 1.37		SS	2	62	20	1	89.96
Stiff to firm, brown **SILTY CLAY,** some to trace sand		SS	3	8	5	2	88.96
						3	87.96
- grey by 3.4m depth		SS	4	100	P	4	86.96
						5	85.96
6.40						6	84.96
Dynamic Cone Penetration Test commenced at 6.40m depth.						7	83.96
						8	82.96
						9	81.96
						10	80.96
						11	79.96
12.42						12	78.96
End of Borehole							

Practical DCPT refusal at 12.42m depth.

(GWL @ 2.20m - Aug. 10, 2020)



patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Preliminary Geotechnnical Investigation
Proposed Redevelopment - 20 Mountain Crescent
Ottawa, Ontario

DATUM Geodetic

FILE NO. PG5462

REMARKS

HOLE NO. BH 2

BORINGS BY Track-Mount Power Auger

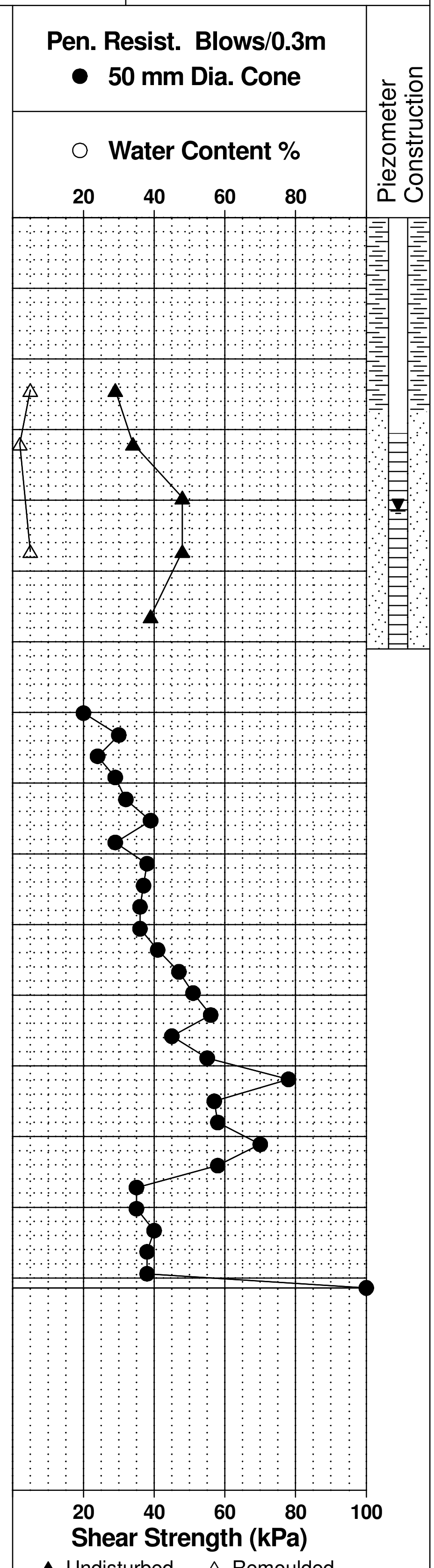
DATE July 31, 2020

SOIL DESCRIPTION	Strata depth (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	89.96
TOPSOIL	0.05	AU	1				
FILL: Brown silty clay, some sand	1.37	SS	2	75	12	1	88.96
Stiff to firm, brown **SILTY CLAY,** some to trace sand - grey by 2.3m depth	6.10	SS	3	100	3	2	87.96
						3	86.96
						4	85.96
						5	84.96
						6	83.96
GLACIAL TILL: Compact, grey silty sand, trace clay, gravel, cobbles	6.70	SS	4	71	24		
Dynamic Cone Penetration Test commenced at 6.70m depth.						7	82.96
						8	81.96
						9	80.96
						10	79.96
						11	78.96
						12	77.96
						13	76.96
						14	75.96
	15.14					15	74.96

End of Borehole

Practical DCPT refusal at 12.42m depth.

(GWL @ 4.15m - Aug. 10, 2020)



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SOIL PROFILE AND TEST DATA

Preliminary Geotechnnical Investigation
Proposed Redevelopment - 20 Mountain Crescent
Ottawa, Ontario

DATUM Geodetic

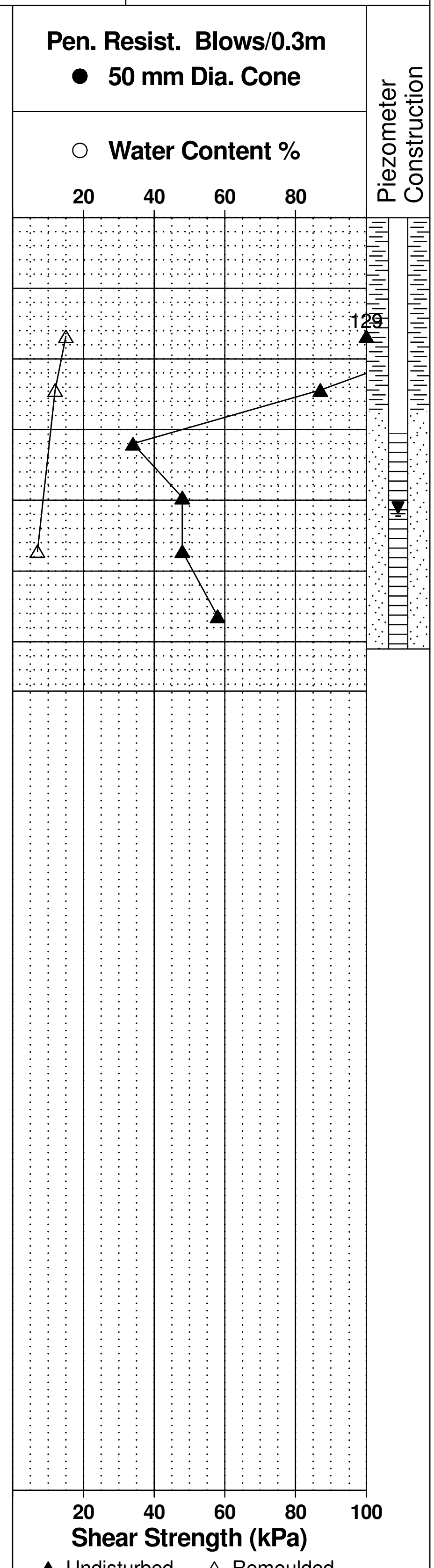
FILE NO. PG5462

REMARKS

HOLE NO. BH 3

BORINGS BY Track-Mount Power Auger **DATE** July 31, 2020

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	90.55
TOPSOIL 0.05		AU	1				
FILL: Brown silty clay, trace sand		SS	2	62	18	1	89.55
1.83							
Very stiff to stiff, brown **SILTY CLAY,** trace sand						2	88.55
						3	87.55
- grey by 2.3m depth						4	86.55
- stiff by 5.0m depth						5	85.55
6.10						6	84.55
GLACIAL TILL: Compact, grey silty sand, trace gravel and cobbles 6.70		SS	3	83	22		
End of Borehole							
(GWL @ 4.19m - Aug. 10, 2020)							



SYMBOLS AND TERMS

SOIL DESCRIPTION

Behavioural properties, such as structure and strength, take precedence over particle gradation in describing soils. Terminology describing soil structure are as follows:

Desiccated	-	having visible signs of weathering by oxidation of clay minerals, shrinkage cracks, etc.
Fissured	-	having cracks, and hence a blocky structure.
Varved	-	composed of regular alternating layers of silt and clay.
Stratified	-	composed of alternating layers of different soil types, e.g. silt and sand or silt and clay.
Well-Graded	-	Having wide range in grain sizes and substantial amounts of all intermediate particle sizes (see Grain Size Distribution).
Uniformly-Graded	-	Predominantly of one grain size (see Grain Size Distribution).

The standard terminology to describe the strength of cohesionless soils is the relative density, usually inferred from the results of the Standard Penetration Test (SPT) ‘N’ value. The SPT N value is the number of blows of a 63.5 kg hammer, falling 760 mm, required to drive a 51 mm O.D. split spoon sampler 300 mm into the soil after an initial penetration of 150 mm.

Relative Density	‘N’ Value	Relative Density %
Very Loose	<4	<15
Loose	4-10	15-35
Compact	10-30	35-65
Dense	30-50	65-85
Very Dense	>50	>85

The standard terminology to describe the strength of cohesive soils is the consistency, which is based on the undisturbed undrained shear strength as measured by the in situ or laboratory vane tests, penetrometer tests, unconfined compression tests, or occasionally by Standard Penetration Tests.

Consistency	Undrained Shear Strength (kPa)	‘N’ Value
Very Soft	<12	<2
Soft	12-25	2-4
Firm	25-50	4-8
Stiff	50-100	8-15
Very Stiff	100-200	15-30
Hard	>200	>30

SYMBOLS AND TERMS (continued)

SOIL DESCRIPTION (continued)

Cohesive soils can also be classified according to their “sensitivity”. The sensitivity is the ratio between the undisturbed undrained shear strength and the remoulded undrained shear strength of the soil.

Terminology used for describing soil strata based upon texture, or the proportion of individual particle sizes present is provided on the Textural Soil Classification Chart at the end of this information package.

ROCK DESCRIPTION

The structural description of the bedrock mass is based on the Rock Quality Designation (RQD).

The RQD classification is based on a modified core recovery percentage in which all pieces of sound core over 100 mm long are counted as recovery. The smaller pieces are considered to be a result of closely-spaced discontinuities (resulting from shearing, jointing, faulting, or weathering) in the rock mass and are not counted. RQD is ideally determined from NXL size core. However, it can be used on smaller core sizes, such as BX, if the bulk of the fractures caused by drilling stresses (called “mechanical breaks”) are easily distinguishable from the normal in situ fractures.

RQD %	**ROCK QUALITY**
90-100	Excellent, intact, very sound
75-90	Good, massive, moderately jointed or sound
50-75	Fair, blocky and seamy, fractured
25-50	Poor, shattered and very seamy or blocky, severely fractured
0-25	Very poor, crushed, very severely fractured

SAMPLE TYPES

SS	-	Split spoon sample (obtained in conjunction with the performing of the Standard Penetration Test (SPT))
TW	-	Thin wall tube or Shelby tube
PS	-	Piston sample
AU	-	Auger sample or bulk sample
WS	-	Wash sample
RC	-	Rock core sample (Core bit size AXT, BXL, etc.). Rock core samples are obtained with the use of standard diamond drilling bits.

SYMBOLS AND TERMS (continued)

GRAIN SIZE DISTRIBUTION

MC%	-	Natural moisture content or water content of sample, %
LL	-	Liquid Limit, % (water content above which soil behaves as a liquid)
PL	-	Plastic limit, % (water content above which soil behaves plastically)
PI	-	Plasticity index, % (difference between LL and PL)
Dxx	-	Grain size which xx% of the soil, by weight, is of finer grain sizes These grain size descriptions are not used below 0.075 mm grain size
D10	-	Grain size at which 10% of the soil is finer (effective grain size)
D60	-	Grain size at which 60% of the soil is finer
Cc	-	Concavity coefficient = $(D30)^2$ / (D10 x D60)
Cu	-	Uniformity coefficient = D60 / D10

Cc and Cu are used to assess the grading of sands and gravels:
Well-graded gravels have: 1 < Cc < 3 and Cu > 4
Well-graded sands have: 1 < Cc < 3 and Cu > 6
Sands and gravels not meeting the above requirements are poorly-graded or uniformly-graded.
Cc and Cu are not applicable for the description of soils with more than 10% silt and clay (more than 10% finer than 0.075 mm or the #200 sieve)

CONSOLIDATION TEST

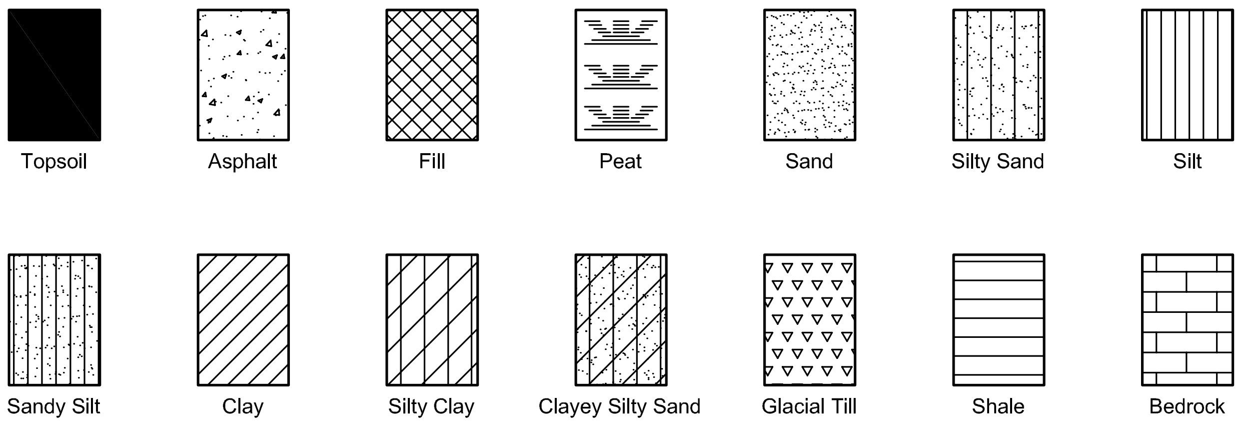
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void Ratio		Initial sample void ratio = volume of voids / volume of solids
Wo	-	Initial water content (at start of consolidation test)

PERMEABILITY TEST

k	-	Coefficient of permeability or hydraulic conductivity is a measure of the ability of water to flow through the sample. The value of k is measured at a specified unit weight for (remoulded) cohesionless soil samples, because its value will vary with the unit weight or density of the sample during the test.

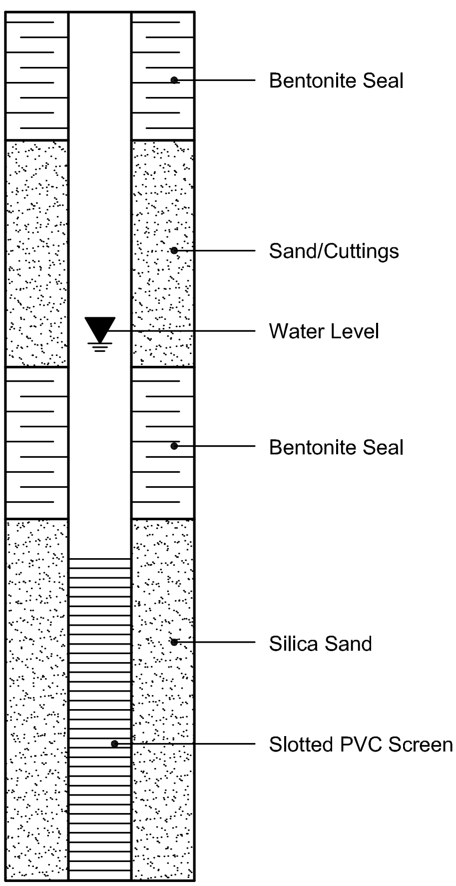
SYMBOLS AND TERMS (continued)

STRATA PLOT

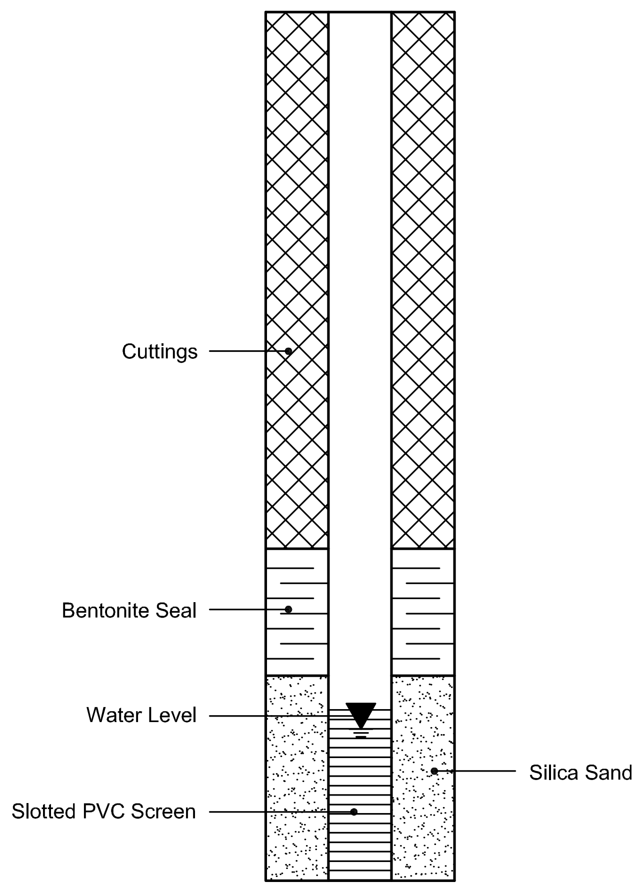


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Certificate of Analysis

Client: Paterson Group Consulting Engineers

Client PO: 30547

Report Date: 21-Aug-2020

Order Date: 20-Aug-2020

Project Description: PG5462

	Client ID:	BH2-SS4	-	-	-
	Sample Date:	31-Jul-20 12:00	-	-	-
	Sample ID:	2034481-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	86.5	-	-	-
General Inorganics					
pH	0.05 pH Units	7.96	-	-	-
Resistivity	0.10 Ohm.m	33.0	-	-	-
Anions					
Chloride	5 ug/g dry	86	-	-	-
Sulphate	5 ug/g dry	98	-	-	-

APPENDIX 2

FIGURE 1 - KEY PLAN

DRAWING PG5462-1 - TEST HOLE LOCATION PLAN

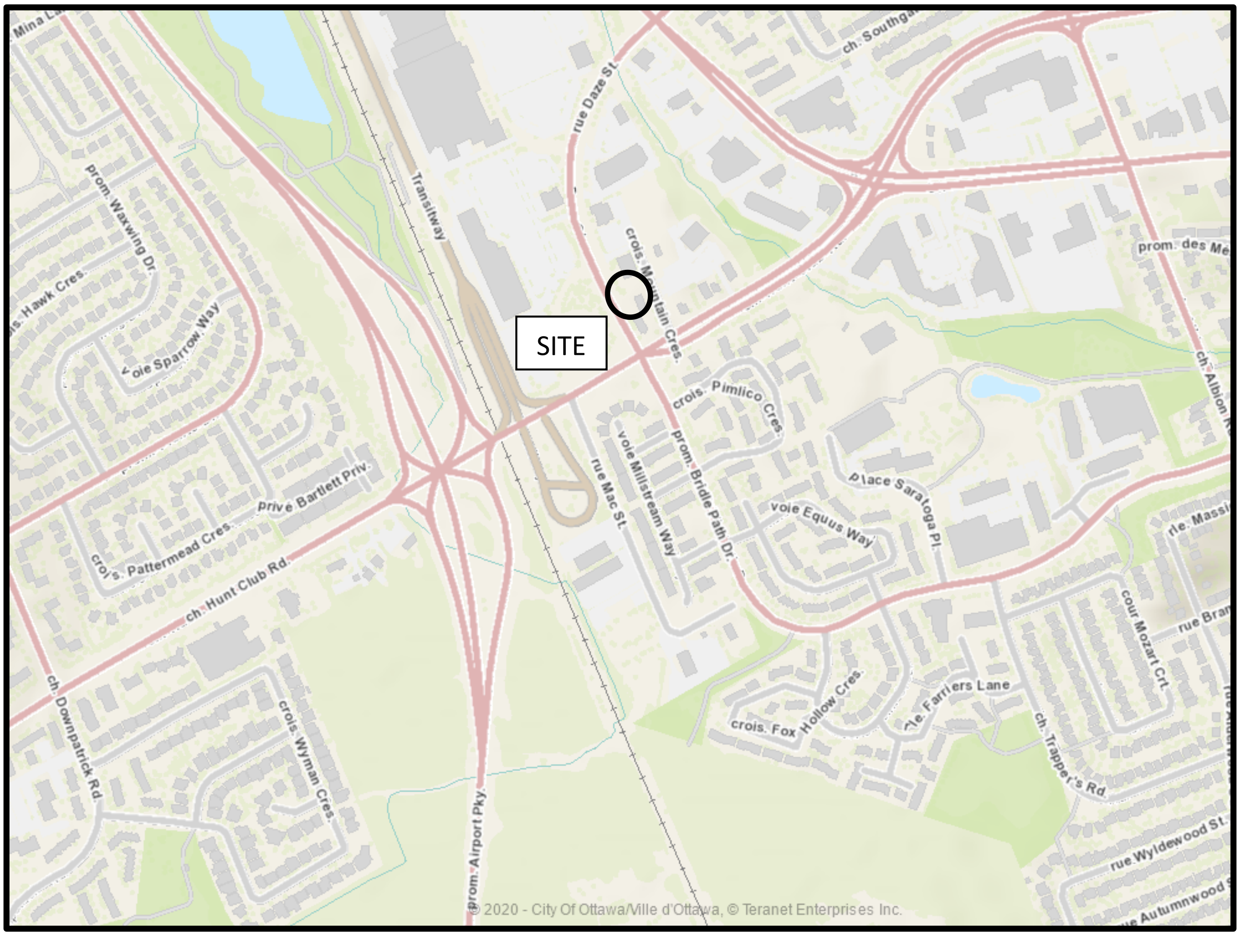


FIGURE 1

KEY PLAN

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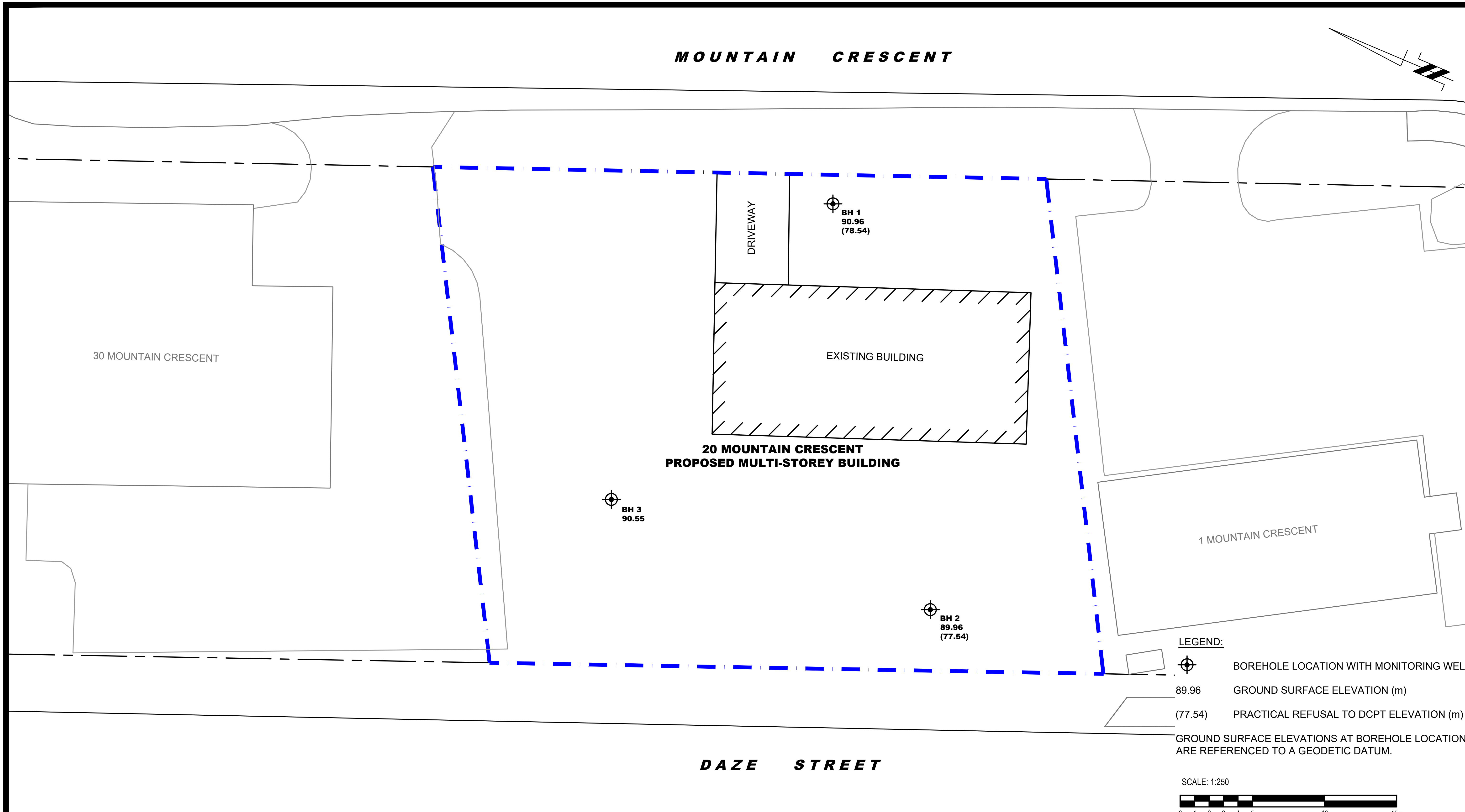

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NO.	REVISIONS	DATE	INITIAL

SURFACE DEVELOPMENTS

PRELIMINARY GEOTECHNICAL INVESTIGATION
PROPOSED MULTI-STOREY BUILDING - 20 MOUNTAIN CRESCENT

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:250	Date: 08/2020
Drawn by: YA	Report No.: PG5462-1
Checked by: KP	Dwg. No.: PG5462-1
Approved by: DJG	Revision No.:

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