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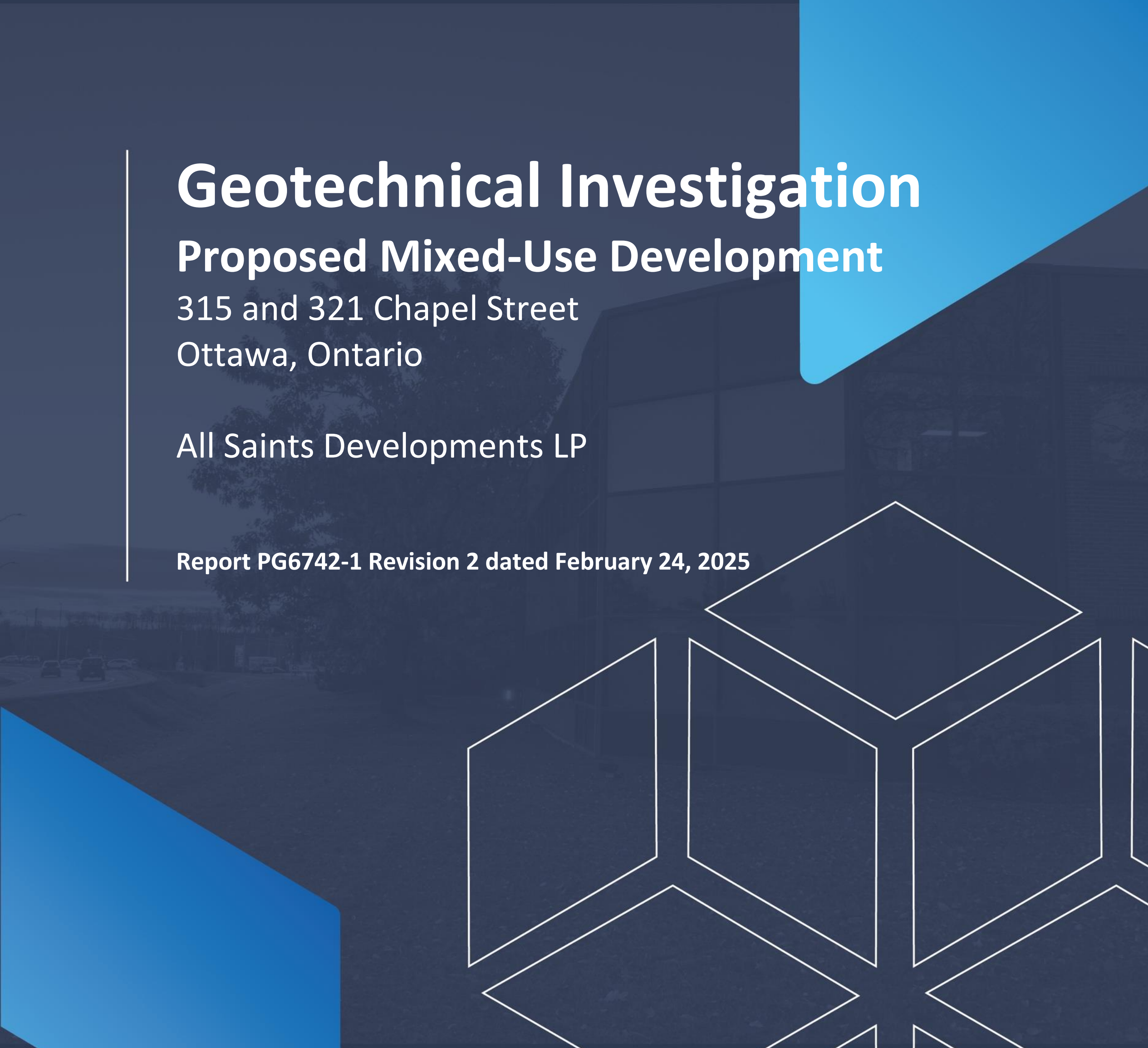


# Geotechnical Investigation

## Proposed Mixed-Use Development

315 and 321 Chapel Street
Ottawa, Ontario

All Saints Developments LP

**Report PG6742-1 Revision 2 dated February 24, 2025**

# Table of Contents

# Appendices

# 1.0 Introduction

Paterson Group (Paterson) was commissioned by All Saints Developments LP to complete a geotechnical investigation for the proposed development to be located at 315 and 321 Chapel Street, Ottawa, Ontario (refer to Figure 1 - Key Plan presented in Appendix 2).

The objective of the geotechnical investigation was to:

- ☐ determine the subsoil and groundwater conditions at the site by means of a test hole.
- ☐ provide geotechnical recommendations for the design of the proposed development including construction considerations which may affect its 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.

Investigating the presence or potential presence of contamination on the subject property was not part of the scope of work of the present investigation. Therefore, the present report does not address environmental issues.

# 2.0 Proposed Development

Based on the available conceptual drawings, it is understood that the proposed development will consist of a nine-storey building with two underground parking levels. It is expected the proposed building will cover the eastern portion of the subject site.

It is expected the existing church structure along Chapel Street will remain and be integrated into the proposed building while the remainder of the existing building's footprint will be demolished as part of the proposed development. It is further expected that the proposed development will be municipally serviced.

# 3.0 Method of Investigation

## 3.1 Field Investigation

### Field Program

The field program for the current investigation was carried out on July 14, 2023, and consisted of advancing two (2) boreholes to a maximum depth of 12.8 m. Previous test holes were advanced by others between April 10 & 11, 2023 and between June 13 & 14, 2023 and were advanced to a maximum depth of 17.4 m below ground surface. The test hole was placed in a manner to provide general coverage of the subject site taking into consideration site features and underground utilities. The test hole locations for the current investigation are presented on Drawing PG6742-1 - Test Hole Location Plan included in Appendix 2.

The borehole was completed using a low-clearance rubber-track drill rig operated by a two-person crew. All fieldwork was conducted under the full-time supervision of Paterson personnel under the direction of a senior engineer from the geotechnical division. The testing procedure consisted of augering to the required depths and at the selected locations sampling the overburden.

### Sampling and In Situ Testing

Soil samples were recovered during drilling from the auger flights or a 50 mm diameter split-spoon sampler. The split-spoon samples were classified on site and placed in sealed plastic bags. All samples were transported to our laboratory for further examination and classification. The split-spoon samples and auger grab-samples recovered from the boreholes are shown as SS and AU, respectively, on the Soil Profile and Test Data sheets presented in Appendix 1.

The 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.

Undrained shear strength testing was carried out in cohesive soils using a field vane apparatus.

The thickness of the overburden was evaluated at BH 2-23 by a dynamic cone penetration test (DCPT). The DCPT consists of driving a steel drill rod, equipped with a 50 mm diameter cone at its 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 logged on the Soil Profile and Test Data sheets in Appendix 1 of this report.

### Groundwater

Flexible polyethylene standpipes were installed in the boreholes to permit monitoring of the groundwater levels subsequent to the completion of the field investigations.

The groundwater observations are discussed in Subsection 4.3 and presented in the Soil Profile and Test Data sheets in Appendix 1.

### Sample Storage

All samples from the current investigation will be stored in the laboratory for a period of one (1) month after issuance of this report. They will then be discarded unless we are otherwise directed.

## 3.2 Field Survey

The test hole locations were selected to provide general coverage of the subject site. The test hole location and ground surface elevation at the test holes for the current investigation were surveyed by Paterson using a high precision, handheld GPS and referenced to a geodetic datum. The location of the test holes are presented in Drawing PG6742-1 - Test Hole Location Plan in Appendix 2.

## 3.3 Analytical Testing

One (1) soil sample was submitted for analytical testing to assess the corrosion potential for exposed ferrous metals and the potential of sulphate attacks against subsurface concrete structures by others. The sample was submitted to determine the concentration of sulphate and chloride, the resistivity, and the pH of the samples. The results are discussed further in Subsection 6.7.

# 4.0 Observations

## 4.1 Surface Conditions

The majority of the subject site is occupied by a two-storey church building with a basement level and some mature trees at the northeast corner of the site. The ground surface is relatively flat and an approximate geodetic elevation of 70.0 m. The subject site is bordered by Laurier Avenue East to the north, Chapel Street to the west, and Blackburn Avenue to the east. The ground surface throughout the subject site is approximately at-grade and up to 500 mm higher than the adjacent roadways. Additionally, there are two two-storey residential buildings located along the southern and southwestern property boundaries.

## 4.2 Subsurface Profile

### Overburden

Generally, the subsurface profile encountered at the test hole location consisted of either topsoil, concrete or asphalt underlain by fill and further by a deposit of silty clay. The silty clay deposit was observed to be underlain by a deposit of glacial till and further by the bedrock formation.

Topsoil was encountered at BH 1-23, BH 2-23 and BH 16-1 and was observed to be approximately 230, 200 and 410 mm thick, respectively.

Fill was encountered at all boreholes, apart from BH 101, as shallow as 75 mm and 0.2 m in areas with hardscaping and landscaping surfaces such as topsoil, respectively. The fill encountered at BH 102 and BH 16-2 below the hardscaping was observed to consist of crushed stone up to approximate depth of 0.8 and 0.5 m, respectively. The remaining fill up to a depth of 1.5 m at BH 16-2 was observed to consist of fine to medium sand with traces of silt. Fill encountered at BH 1-23, BH 2-23 and BH 16-1 throughout the landscaped portions of the subject site extended up to a depth of 1.6 m and generally consisted of brown silty sand with variable amounts of sand and gravel.

The fill layers were generally observed to be underlain by a deposit of silty clay which consisted of a layer of desiccated brown clay crust underlain by unweathered grey silty clay. The brown silty clay layer was observed at depths ranging between 0.7 to 4.4 m below the ground surface. It should be noted that the brown clay layer was observed as shallow as 0.3 m and directly below the concrete slab at BH 101 undertaken by others within the interior portion of the existing structure.

The brown silty clay was observed to be underlain by a layer of unweathered, firm to stiff grey silty clay which was observed to extend to depths ranging between 3.8 to 10.8 m below the existing ground surface.

The glacial till was encountered below the clay deposit and observed to be compact to very dense. The glacial till soil matrix comprised silty clay to silty sand with variable amounts of clay, sand, gravel, cobbles, and boulders and it was observed that the clay content was decreasing with depth. The glacial till was observed to extend to depths ranging between 9.7 to 14.4 m below the existing ground surface.

Practical refusal to the DCPT was encountered at a depth of 14.4 m below existing ground surface at BH 2-23. Practical refusal to augering was encountered at a depth of 11.4 and 11.9 at BH 1-23 and BH 16-2, respectively.

Reference should be made to the Soil Profile in Appendix 1 for specific details of the soil profiles encountered at each test hole location.

**Bedrock**

Bedrock was cored by others at BH102 and BH 16-1 at a depth of 13.1 and 14.2 m below ground surface, respectively. The bedrock was observed to consist of fair to excellent quality grey limestone. Based on available geological mapping, the site is located in an area where the bedrock consists of interbedded limestone and shale from the Verulam Formation with an overburden drift thickness ranging between 10 to 15 m.

## 4.3 Groundwater

Groundwater levels were measured in the installed piezometers during the current investigation. The measured groundwater level (GWL) readings are presented in Table 1 below and are shown on the Soil Profile and Test Data sheets in Appendix 1.

**Table 1 – Summary of Groundwater Levels**

| Test Hole | Ground Surface Elevation (m) | Measured Groundwater Level | | Date Recorded |
|---|---|---|---|---|
| | | Depth (m) | Elevation (m) | |
| BH 1-23 | 70.34 | 11.48 | 58.86 | July 21, 2023 |
| BH 2-23 | 70.00 | Dry and blocked at 7.34 m | N/A | July 21, 2023 |
| BH101 | N/A | 0.40 | N/A | April 10, 2023 |
| BH102 | N/A | 10.67 | N/A | April 11, 2023 |
| BH16-1 | 70.35 | 13.2 | 57.15 | September 26, 2016 |
| BH16-2 | 70.03 | Dry | N/A | September 26, 2016 |

**Note:** The ground surface elevation was surveyed using a handheld GPS and referenced to a geodetic datum.

It is important to note that groundwater level readings could be influenced by surface water infiltrating the backfilled borehole.

Long-term groundwater levels can also be estimated based on recovered soils samples moisture levels, soil sample coloring and consistency. Based on this methodology, the long-term groundwater level is estimated to be at **4** to **5 m** depth below the existing grade.

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

The subject site is considered suitable for the proposed development. It is expected that the proposed building may be supported using a raft foundation bearing upon an undisturbed, stiff, silty clay bearing surface.

Due to the presence of a silty clay layer, proposed grading throughout the subject site will be subjected to a permissible grade restriction. Our permissible grade raise recommendations are discussed in Subsection 5.3.

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

## 5.2 Site Grading and Preparation

### Stripping Depth

Asphalt, topsoil, and any deleterious fill, such as those containing organic materials, should be stripped from under any proposed building or other settlement sensitive structures. Care should be taken not to disturb adequate bearing soils below the founding level during site preparation activities. Disturbance of the subgrade may result in having to sub-excavate the disturbed material and the placement of additional suitable fill material.

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

### Vibration Considerations

Construction operations could cause vibrations, and possibly, sources of nuisance to the community. Therefore, means to reduce the vibration levels as much as possible should be incorporated in the construction operations to maintain a cooperative environment with the residents.

The following construction equipment could cause vibrations: piling equipment, hoe ram, compactor, dozer, crane, truck traffic, etc. The construction of a shoring system with soldier piles or sheet piling will require these pieces of equipment. Vibrations, caused by blasting or construction operations could cause detrimental vibrations on the adjoining buildings and structures. Therefore, it is recommended that all vibrations be limited.

Two parameters determine the recommended vibration limit, the maximum peak particle velocity and the frequency. For low frequency vibrations, the maximum allowable peak particle velocity is less than that for high frequency vibrations. As a guideline, the peak particle velocity should be less than 15 mm/s between frequencies of 4 to 12 Hz, and 50 mm/s above a frequency of 40 Hz (interpolate between 12 and 40 Hz). These guidelines are for current construction standards. Considering there are several sensitive buildings in close proximity to the subject site, consideration to lowering these guidelines is recommended. These guidelines are above perceptible human level and, in some cases, could be very disturbing to some people, a pre-construction survey is recommended to minimize the risks of claims during or following the construction of the proposed buildings.

### Fill Placement

Fill used for grading beneath the proposed building should consist of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. Granular material should be tested and approved prior to delivery to the site. The fill should be placed in loose lifts of 300 mm thick or less and compacted using suitable compaction equipment for the lift thickness. Fill placed beneath the building area should be compacted to at least 98% of the 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 geocomposite drainage membrane, such as Miradrain G100N or Delta Drain 6000.

### Protection of Subgrade (Raft Foundation)

Since the subgrade material for the building’s foundation is expected to consist of stiff, grey silty clay, it is recommended that a minimum 75 mm thick lean concrete mud slab be placed on the undisturbed silty clay 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 or workers and equipment.

The final excavation of the raft bearing surface level and the placing of the mud slab should be completed in smaller sections to avoid exposing large areas of the silty clay to potential disturbances due to drying. The bearing medium should be reviewed and approved by Paterson personnel prior to placing the mud slab layer.

## 5.3 Foundation Design

### Raft Foundation

Based on the expected loads from the proposed structure, a raft foundation bearing on the undisturbed stiff, grey silty clay bearing surface may be considered for foundation support for the proposed building. For design purposes, it was assumed that the base of the raft foundation would be located at an approximate depth of 6 to 7 m since it would be provided with two levels of underground parking.

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.

For the raft slab foundation, a bearing resistance value at SLS (contact pressure) of **200 kPa** will be considered acceptable for a raft supported on the undisturbed, firm silty clay. The factored bearing resistance (contact pressure) at ULS can be taken as **300 kPa**. For this case, the modulus of subgrade reaction was calculated to be **8.0 MPa/m** for a contact pressure of **200 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 high-rise portion of the proposed structure can be designed using the above parameters with a total and differential settlement of 25 and 15 mm, respectively.

### Conventional Shallow Foundations (Auxiliary Structures)

The following conventional spread footing bearing resistance values may be considered for portions of the underground parking garage structure located beyond the building footprint and other lightly loaded ancillary structures.

Strip footings, up to 3 m wide, and pad footings, up to 5 m wide, placed over an undisturbed, hard to stiff brown silty clay bearing surface can be designed using a bearing resistance value at serviceability limit states (SLS) of **150 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **225 kPa**.

Strip footings, up to 2 m wide, and pad footings, up to 4 m wide, placed over an undisturbed, stiff grey silty clay bearing surface can be designed using a bearing resistance value at serviceability limit states (SLS) of **90 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **135 kPa.** A geotechnical resistance factor of 0.5 was applied to the reported bearing resistance values at ULS.

An undisturbed soil bearing surface consists of one from which all topsoil and deleterious materials, such as loose, frozen or disturbed soil, have been removed prior to the placement of concrete for footings.

Footings placed on an undisturbed soil bearing surface and designed using the bearing resistance values at SLS provided above will be subjected to potential post-construction total and differential settlements of 25 and 20 mm, respectively.

### Lateral Support

The bearing medium under footing-supported structures is required to be provided with adequate lateral support with respect to excavations and different foundation levels. Adequate lateral support is provided to the encountered overburden material above the groundwater table when a plane extending down and out from the bottom edge of the footing at a minimum of 1.5H:1V passes only through in situ soil of the same or higher capacity as the bearing medium soil.

### Permissible Grade Raise Restrictions

Based on the undrained shear strength values of the silty clay deposit encountered throughout the subject site, a permissible grade raise restriction of **2.0 m** is recommended settlement sensitive structures will be located above or within the deposit. A post-development groundwater lowering of 0.5 m was considered in our permissible grade raise restriction calculations.

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

The site class for seismic site response can be taken as **Site Class C** for foundations constructed at the subject site, according to Table 4.1.8.4.A of the 2012 Ontario Building Code (OBC 2012). The soils underlying the subject site are not susceptible to liquefaction. Reference should be made to the latest revision of the 2012 Ontario Building Code 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. The recommended pavement structures noted in Subsection 5.7 will be applicable where the basement level underlying foundation support consists of a raft foundation. If storage or other uses of the lower level involve the construction of a concrete floor slab, the upper 200 mm of sub-slab fill is recommended to consist of 19 mm of 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 a minimum of 98% of the SPMDD. An engineered fill such as an OPSS Granular A or Granular B Type II compacted to 98% of its SPMDD could be placed around the proposed footings. Any soft areas should be removed and backfilled with appropriate backfill material prior to placing any fill. OPSS Granular A or Granular B Type II, with a maximum particle size of 50 mm, are recommended for backfilling below the floor slab.

A subfloor drainage system, consisting of lines of perforated drainage pipe subdrains connected to a positive outlet, should be provided in the clear stone backfill under the lowest basement floor. The spacing of the underfloor drainage system should be confirmed at the time of completing the excavation when water infiltration can be better assessed. This is discussed further in Section 6.1 of this report.

## 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$^3$. The applicable effective unit weight of the retained soil can be estimated as 13 kN/m$^3$, where applicable. A hydrostatic pressure should be added to the total static earth pressure when calculating the effective unit weight.

The total earth pressure ($P_{AE}$) includes the static earth pressure component ($P_o$) and the seismic component ($\Delta P_{AE}$).

### 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)
$\gamma$ = 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 ($\Delta P_{AE}$).

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

$a_c = (1.45 - a_{max}/g)a_{max}$
$\gamma$ = 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 the 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 the OBC 2012.

## 5.7 Pavement Design

### Pavement Structure Over Overburden

The following pavement structures may be considered for the access lane between the right-of-way and the access ramp as detailed in Tables 2, 3 and 4.

**Table 2 – Recommended Hard Landscaping – Pedestrian Walkways**

| Thickness (mm) | Material Description |
|---|---|
| Specified by Others | **Wear Course** – Interlocking Stones/Brick Pavers |
| 25 - 40 | **Levelling Course –** Stone Dust or Sand |
| 300 | **SUBBASE –** OPSS Granular A |
| **SUBGRADE** - Either fill, in situ soil or OPSS Granular B Type I or II material placed over in situ soil or fill. | |

**Table 3 - Recommended Pavement Structure - Car-Only Parking Areas and Fire-Truck Routes**

| 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 in situ soil, fill or OPSS Granular B Type I or II material placed over in situ soil. | |

**Table 4 - Recommended Pavement Structure - Heavy-Truck Traffic and Loading 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 in situ soil, fill or OPSS Granular B Type I or II material placed over in situ soil. | |

### Pavement Structure Over Raft Foundations

Based on the concrete raft slab subgrade for the underground parking level, the pavement structure indicated in the following tables may be considered for design purposes:

**Table 5 - Recommended Rigid Pavement Structure - Lower Level**

| Thickness (mm) | Material Description |
|---|---|
| Specified by Others | **Rigid Concrete Pavement –** Class C2 Exposure Class Reinforced Concrete |
| 300 | **BASE** - OPSS Granular A Crushed Stone |
| **SUBGRADE** – Reinforced Concrete Raft Slab | |

**Table 6 - Recommended Pavement Structure - Car-Only Parking Areas (Raft Slab)**

| Thickness (mm) | Material Description |
|---|---|
| 50 | **Wear Course** - HL-3 or Superpave 12.5 Asphaltic Concrete |
| 300 | **BASE** - OPSS Granular A Crushed Stone |
| **SUBGRADE** – Reinforced Concrete Raft Slab | |

**Table 7 - Recommended Pavement Structure – Access Lane, Fire Truck Lane, Ramp and Heavy Truck Parking Areas (Raft Slab)**

| 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 |
| 300 | **Base** - OPSS Granular A Crushed Stone |
| **SUBGRADE** – Reinforced Concrete Raft Slab | |

### Pavement Structure Drainage

Satisfactory performance of the pavement structure is largely dependent on keeping the contact zone between the subgrade material and the base stone in a dry condition. Failure to provide adequate drainage under conditions of heavy wheel loading can result in the fine subgrade soil being pumped into the voids in the stone subbase, thereby reducing its load carrying capacity.

For areas where silty clay is encountered at subgrade level and where overburden will be at the pavement structure subgrade, it is recommended that subdrains be installed during the pavement construction as per City of Ottawa standards.
The subdrain inverts should be approximately 300 mm below subgrade level. The subgrade surface should be crowned to promote water flow to the drainage lines.

# 6.0 Design and Construction Precautions

## 6.1 Foundation Drainage and Backfill

### Groundwater Suppression System

It is recommended that a groundwater suppression system be provided for the proposed structure. It is expected that insufficient room will be available for exterior backfill and the foundation wall will be cast as a blind-sided pour against a shoring system. It is recommended that the groundwater suppression system consist of the following:

❑ A waterproofing membrane should be placed against the shoring system between underside of the raft slab and a geodetic elevation of **67.0 m**. The membrane is recommended to overlap below and under the edge of the raft foundation footprint by a minimum of 600 mm inwards towards the building footprint and from the face of the raft foundation.

❑ A composite drainage membrane (DeltaDrain 6000, MiraDrain G100N or equivalent) should be placed against the HDPE face of the waterproofing membrane with the geotextile layer facing the waterproofing layer from finished ground surface to the top of the raft.

❑ The foundation drainage boards should be overlapped such that the bottom end of a higher board is placed in front of the top end of a lower board. All endlaps of the drainage board sheets should overlap abutting sheets by a minimum of 150 mm. All overlaps should be sealed with a suitable adhesive and/or sealant material approved by Paterson field personnel.

❑ It is recommended that 150 mm diameter PVC sleeves at 6 m centers be cast in the foundation wall at the foundation wall/raft interface to allow the infiltration of water to flow to the interior perimeter drainage pipe. The sleeves should be connected to openings in the HDPE face of the drainage board layer and should not cross the waterproofing membrane layer. The perimeter drainage pipe and underfloor drainage system should direct water to sump pit(s) within the lower basement area via an underfloor and interior drainage pipe system.

The top endlap of the foundation drainage board should be provided with a suitable termination bar against the foundation wall to mitigate the potential for water to perch between the drainage board and foundation wall.

### Interior Perimeter and Underfloor Drainage

The interior perimeter and underfloor drainage system will be required to control water infiltration below the lowest underground parking level slab and redirect water from the building's foundation drainage system to the buildings sump pit(s). The interior perimeter and underfloor drainage pipe should consist of a 150 mm diameter corrugated perforated plastic pipe sleeved with a geosock.

The underfloor drainage pipe should be placed in each direction of the basement floor span and connected to the perimeter drainage pipe. The interior drainage pipe should be provided with tee-connections to extend pipes between the perimeter drainage line and the HDPE-face of the composite foundation drainage board via the foundation wall sleeves. The spacing of the underfloor drainage system should be confirmed by Paterson once the foundation layout and sump system location has been finalized.

### Elevator Pit Waterproofing

The elevator shaft exterior foundation walls should be waterproofed to avoid any infiltration into the elevator pit. It is recommended that a waterproofing membrane, such as Colphene Torch'n Stick (or approved other) be applied to the exterior of the elavator shaft foundation wall. The Colphene Torch'n Stick waterproofing membrane should extend over the vertical portion of the raft slab and down to the top of the footing in accordance with the manufacturer's specifications. A continuous PVC waterstop such as Southern waterstop 14RCB or equivalent should be installed within the interface between the concrete base slab below the elevator shaft foundation walls.

The 150 mm diameter perforated corrugated pipe underfloor drainage should be placed along the perimeter of the exterior sidewalls and provided a gravity connection to the sump pump basin or the elevator sump pit.

### Foundation Raft Slab Construction Joints

It is anticipated the raft slab will be poured in several pour segments. For the construction joint at each pour, a rubber water stop along with a chemical grout (Xypex or equivalent) should be applied to the entire vertical joint of the slab.

### Foundation Backfilling

Backfill against the exterior sides of the foundation walls should consist of free-draining non frost susceptible granular materials. The greater part of the site excavated materials will be frost susceptible and, as such, are not recommended for re-use as backfill against the foundation walls, unless used in conjunction with a drainage geocomposite, such as Miradrain G100N or Delta Drain 6000, connected to the perimeter foundation drainage system. Imported granular materials, such as clean sand or OPSS Granular B Type I granular material, should otherwise be used for this purpose.

## 6.2 Protection Against Frost Action

Perimeter footings of heated structures are required to be insulated against the deleterious effect of frost action. A minimum of 1.5 m thick soil cover (or equivalent) should be provided in this regard.

Exterior unheated footings, such as those for isolated exterior piers, 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.

## 6.3 Excavation Side Slopes

The side slopes of excavations in the 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. It is assumed that sufficient room will be available for the greater part of the excavation to be undertaken by open-cut methods (i.e., unsupported excavations).

### Unsupported Excavations

The excavation side slopes above the groundwater level extending to a maximum depth of 3 m should be cut back at 1H:1V or flatter. The flatter slope is required for excavation below groundwater level. The subsoil at this site is considered to be mainly 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.

It is recommended that a trench box be used at all times to protect personnel working in trenches with steep or vertical sides. It is expected that services will be installed by “cut and cover” methods and excavations will not be left open for extended periods of time.

### Temporary Shoring

It is expected temporary shoring will be required for the overburden soil to complete the required excavations where insufficient room is available for open cut methods. This is expected based on the proximity of the existing structures and roadways. The shoring requirements designed by a structural engineer specializing in those works will depend on the depth of the excavation, the proximity of the adjacent structures and the elevation of the adjacent building foundations and underground services. The design and implementation of these temporary systems will be the responsibility of the excavation contractor and their design team.

Inspections and approval of the temporary system will also be the responsibility of the designer. Geotechnical information provided below is to assist the designer in completing a suitable and safe shoring system. The designer should take into account the impact of a significant precipitation event and designate design measures to ensure that a precipitation will not negatively impact the shoring system or soils supported by the system. Any changes to the approved shoring design system should be reported immediately to the owner’s structural design prior to implementation.

The temporary system could consist of soldier pile and lagging system or interlocking steel sheet piling. Any additional loading due to street traffic, construction equipment, adjacent structures, and facilities, etc., should be included to the earth pressures described below. These systems could be cantilevered, anchored, or braced.

Generally, it is expected that the shoring systems will be provided with tie-back rock anchors to ensure their stability. The shoring system is recommended to be adequately supported to resist toe failure and inspected to ensure that the sheet piles extend well below the excavation base. It should be noted if consideration is being given to utilizing a raker style support for the shoring system that lateral movements can occur and the structural engineer should ensure that the design selected minimizes these movements to tolerable levels.

The earth pressures acting on the shoring system may be calculated using the parameters provided in Table 8.

**Table 8 - Soil Parameters for Calculating Earth Pressures Acting on Shoring System**

| Parameter | Value |
|---|---|
| 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 ($\gamma$), kN/m$^3$ | 20 |
| Submerged Unit Weight ($\gamma$'), 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 is calculated for earth pressures. If the groundwater level is lowered, the dry unit weight for the soil 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.

## 6.4 Pipe Bedding and Backfill

Bedding and backfill materials should be in accordance with the most recent Material Specifications and Standard Detail Drawings from the Department of Public Works and Services, Infrastructure Services Branch of the City of Ottawa.

The pipe bedding for the sewer and water pipes should consist of at least 150 mm of OPSS Granular A. The bedding layer thickness should be increased to a minimum of 300 mm where the subgrade will consist of grey silty clay. The material should be placed in a maximum 225 mm thick loose lifts and compacted to a minimum of 99% of its SPMDD. The bedding material should extend at least to the spring line of the pipe.

The cover material, which should consist of OPSS Granular A, should extend from the spring line of the pipe to at least 300 mm above the obvert of the pipe. The material should be placed in maximum 225 mm thick lifts and compacted to a minimum of 99% of its SPMDD.

It should generally be possible to re-use the moist (not wet) site-generated fill above the cover material if the excavation and filling operations are carried out in dry weather conditions. Wet site-generated fill, such as the grey silty clay, will be difficult to re-use, as the high-water contents make compacting impractical without an extensive drying period.

Where hard surface areas are considered above the trench backfill, the trench backfill material within the frost zone (about 1.8 m below finished grade) should match the soils exposed at the trench walls to minimize differential frost heaving. The trench backfill should be placed in maximum 300 mm thick loose lifts and compacted to a minimum of 95% of the material's SPMDD.

To reduce long-term lowering of the groundwater level at this site, clay seals should be provided in the service trenches. The seals should be at least 1.5 m long and should extend from trench wall to trench wall. Generally, the seals should extend from the frost line and fully penetrate the bedding, sub bedding and cover material. The barriers should consist of relatively dry and compatible brown silty clay placed in maximum 225 mm thick loose layers and compacted to a minimum of 95% of the material's SPMDD. The clay seals should be placed at the site boundaries and at strategic locations at no more than 60 m intervals in the service trenches.

## 6.5 Groundwater Control

### Groundwater Control for Building Construction

Based on our observations, it is anticipated that groundwater infiltration into the excavations should be low and controllable using open sumps. Pumping from open sumps should be sufficient to control the groundwater influx through the sides of shallow excavations. The contractor should be prepared to direct water away from all bearing surfaces and subgrades, regardless of the source, to prevent disturbance to the founding medium.

### Permit to Take Water

A temporary Ministry of the Environment, Conservation and Parks (MECP) permit to take water (PTTW) may be required for this project if more than 400,000 L/day of ground and/or surface water is to be pumped during the construction phase. A minimum 4 to 5 months should be allowed for completion of the PTTW application package and issuance of the permit by the MECP.

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.

## 6.6 Winter Construction

Precautions must be taken if winter construction is considered for this project.

Where excavations are completed in proximity to existing structures which may be adversely affected due to the freezing conditions. The subsurface conditions mostly consist of frost susceptible materials. In the presence of water and freezing conditions, ice could form within the soil mass. Heaving and settlement upon thawing could occur.

In particular, where a shoring system is constructed, the soil behind the shoring system will be subjected to freezing conditions and could result in heaving of the structure(s) placed within or above frozen soil. Provisions should be made in the contract documents to protect the walls of the excavations from freezing, if and where applicable.

In the event of construction during below zero temperatures, the founding stratum should be protected from freezing temperatures by the installation of straw, propane heaters and/or glycol lines and tarpaulins or other suitable means. The base of the excavations should be insulated from sub-zero temperatures immediately upon exposure and until such time as heat is adequately supplied to the building and the foundation is protected with sufficient soil cover to prevent freezing at founding level.

Trench excavations, foundation construction and pavement construction are difficult activities to complete during freezing conditions without introducing frost in the subgrade or in the excavation walls and bottoms. Precautions should be considered if such activities are to be completed during freezing conditions. Additional information could be provided, if required.

Under winter conditions, if snow and ice is present within imported fill below future basement slabs, then settlement of the fill should be expected and support of a future basement slab and/or temporary supports for slab pours will be negatively impacted and could undergo settlement during spring and summer time conditions. Paterson should complete periodic inspections during fill placement to ensure that snow and ice quantities are minimized in settlement-sensitive areas.

## 6.7 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.

## 6.8 Landscaping Considerations

### Tree Planting Considerations

It is understood the proposed building will include two levels of underground parking and the structures will be founded at a minimum of 6 m below finished grade. Given the depth of foundations proposed for the structure, it is expected that the support of the foundations derives from soil located below the depth that dewatering by tree roots.

Therefore, foundation distress due to potential moisture depletion caused by trees is not expected to occur at the subject site. Since the proposed structure is not anticipated to be founded upon silty clay soils affected by the depth of root penetration, City approved trees within the subject site will not be subject to planting restrictions as based on the *City of Ottawa Tree Planting in Sensitive Marine Clay Soils (2017 Guidelines*) from a geotechnical perspective.

# 7.0 Recommendations

It is recommended that the following be carried out by Paterson once preliminary and future details of the proposed development have been prepared:

- ➢ Review preliminary and detailed grading, servicing and structural plan(s) from a geotechnical perspective.
- ➢ Review of the geotechnical aspects of the excavation contractor’s shoring design, prior to construction, if applicable.

It is a requirement for the foundation design data provided herein to be applicable that a material testing and observation program be performed by the geotechnical consultant. The following aspects of the program should be performed by Paterson:

- ➢ Review and inspection of the installation of the groundwater suppression system, including installation of underfloor drainage systems and waterproofing of elevator shafts.
- ➢ Observation of all bearing surfaces prior to the placement of concrete.
- ➢ Sampling and testing of the concrete and fill materials.
- ➢ 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 and follow-up field density tests to determine the level of compaction achieved.
- ➢ 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 the completion of a satisfactory inspection program by the geotechnical consultant.

All excess soil must be handled as per *Ontario Regulation 406/19: On-Site and Excess Soil Management*.

# 8.0 Statement of Limitations

The recommendations provided are in accordance with the present understanding of the project. Paterson requests permission to review the recommendations when the drawings and specifications are completed.

A soils investigation is a limited sampling of a site. Should any conditions at the site be encountered which differ from those at the test locations, Paterson requests 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 the 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 All Saints Developments LP or their agent(s) is not authorized without review by Paterson Group for the applicability of our recommendations to the altered use of the report.

**Paterson Group Inc.**

KBell

Killian Bell, B.Eng.

LICENSED PROFESSIONAL ENGINEER
**February 24, 2025**
D. J. PETAHTEGOOSE
100568013
PROVINCE OF ONTARIO

Drew Petahtegoose, P.Eng.

**Report Distribution:**

- ☐ All Saints Developments LP (email copy)
- ☐ Paterson Group (1 copy)

# APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TESTING RESULTS

# SOIL PROFILE AND TEST DATA

## GEOTECHNICAL INVESTIGATION

315 and 321 Chapel Street

**DATUM:** Geodetic **EASTING:** 369206.786 **NORTHING:** 5032236.578 **ELEVATION:** 70.34

**PROJECT:** Geotechnical Investigation - Proposed Mixed-Use Development

**BORINGS BY:** CME Low Clearance Drill

**REMARKS:**

**DATE:** July 14, 2023

**FILE NO.** **PG6742**

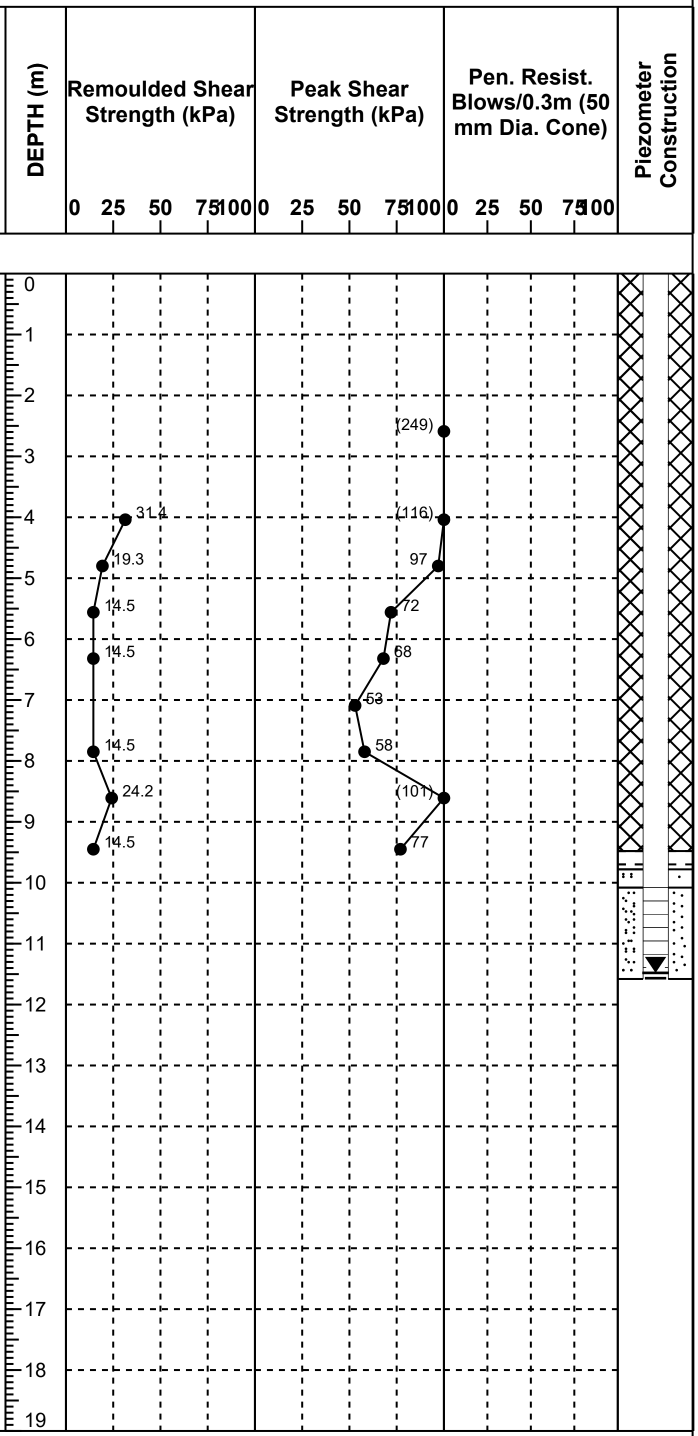
**HOLE NO.** BH 1-23

| SAMPLE DESCRIPTION | SAMPLE No. | SAMPLE Type | SAMPLE % RECOVERY | N VALUE or RQD | WATER CONTENT % |
|---|---|---|---|---|---|
| Ground Surface EL 70.34 m | | | | | |
| TOPSOIL 0.23 m EL 70.11 m | AU 1 | ● | | | 15 |
| FILL: Loose, brown silty sand, trace gravel 1.45 m EL 68.89 m | SS2 | ▽ | 100 | 6 | 15 |
| Hard to very stiff, brown SILTY CLAY 4.42 m EL 65.92 m | SS3 | ▽ | 100 | 8 | 41 |
| | SS4 | ▽ | 100 | P | 53 |
| | SS5 | ▽ | 100 | 4 | 62 |
| Stiff, grey SILTY CLAY 9.75 m EL 60.59 m | G6 | [#] | | | 77 |
| | SS7 | ▽ | 75 | P | 60 |
| | SS8 | ▽ | 67 | P | 38 |
| GLACIAL TILL: Compact, grey silty sand with gravel, cobbles and boulders, trace clay 11.58 m EL 58.76 m | SS9 | ▽ | 100 | 50+ | 26 |
| | SS10 | ▽ | 75 | 24 | 9 |

End of Borehole

Practical refusal to augering at 11.58m depth.

(GWL @ 11.48 - July 21, 2023)



PATERSON GROUP

# SOIL PROFILE AND TEST DATA

## GEOTECHNICAL INVESTIGATION

315 and 321 Chapel Street

**DATUM:** Geodetic **EASTING:** 369201.763 **NORTHING:** 5032260.979 **ELEVATION:** 70.00

**PROJECT:** Geotechnical Investigation - Proposed Mixed-Use Development

**BORINGS BY:** CME Low Clearance Drill

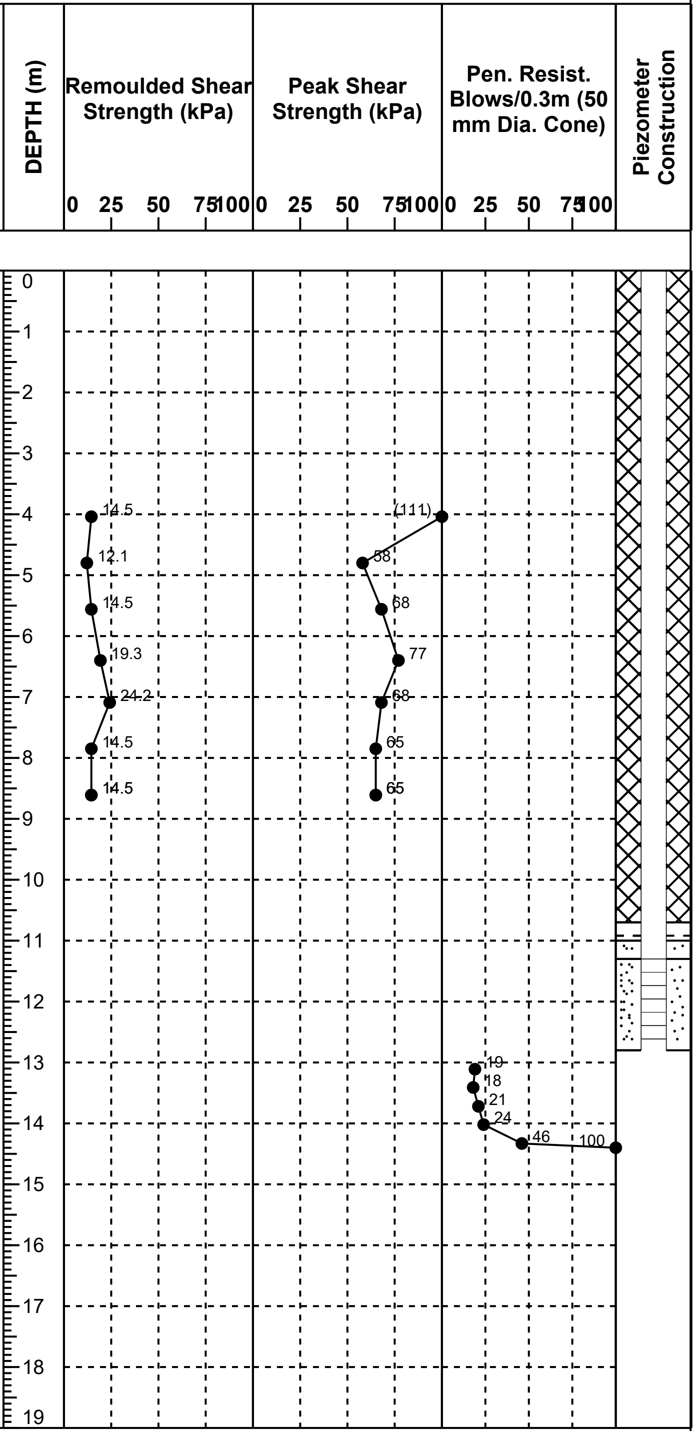
**REMARKS:**

**DATE:** July 14, 2023

**FILE NO.** **PG6742**

**HOLE NO.** BH 2-23

| SAMPLE DESCRIPTION | SAMPLE No. | SAMPLE Type | SAMPLE % RECOVERY | N VALUE or RQD | WATER CONTENT % |
|---|---|---|---|---|---|
| Ground Surface EL 70 m | | | | | |
| TOPSOIL 0.2 m EL 69.8 m | AU 1 | ● | | | 15 |
| FILL: Loose, brown silty sand 1.07 m EL 68.93 m | SS2 | ▽ | 100 | 7 | 48 |
| Very stiff, brown SILTY CLAY | SS3 | ▽ | 100 | 10 | 46 |
| | SS4 | ▽ | 100 | 4 | 50 |
| 4.42 m EL 65.58 m | SS5 | ▽ | 100 | 4 | 63 |
| Stiff to very stiff, grey SILTY CLAY | SS6 | ▽ | 100 | P | 62 |
| 9.83 m EL 60.17 m | SS7 | ▽ | 100 | 2 | 58 |
| GLACIAL TILL: Compact, grey silty sand with clay, gravel, cobbles and boulders | SS8 | ▽ | 58 | 13 | 16 |
| | SS9 | ▽ | 58 | 15 | 7 |
| | SS10 | ▽ | 67 | 21 | 8 |
| 12.8 m EL 57.2 m | SS11 | ▽ | 75 | 20 | 8 |
| Dynamic Cone Penetration Test commenced at 12.80m depth. 14.4 m EL 55.6 m | | | | | |
| End of Borehole | | | | | |
| Practical DCPT refusal at 14.40m depth. | | | | | |
| (BH dry and blocked at 7.42m - July 21, 2023) | | | | | |



RSLog / Geotechnical Borehole - Geodetic / paterson-group / admin / July 24, 2023 04:59 PM

CLIENT: All Saints Development LP
PROJECT NO.: CO923.00

**RECORD OF:**

# BH101

ADDRESS: 315 Chapel Street
CITY/PROVINCE: Ottawa
NORTHING (m):
EASTING (m):
ELEV. (m) 67.25
CONTRACTOR: Strata Drilling Group
METHOD: Direct Push
BOREHOLE DIAMETER (cm):
WELL DIAMETER (cm): -
SCREEN SLOT #: -
SAND TYPE: -
SEALANT TYPE: Bentonite

SAMPLE TYPE ◧ AUGER ◢ DRIVEN ⧓ CORING ⬒ DYNAMIC CONE ▯ SHELBY ⌶ SPLIT SPOON

| GWL (m) | SOIL SYMBOL | SOIL DESCRIPTION | DEPTH (m) | ELEVATION (m) | SHEAR STRENGTH (kPa) ● 40 80 120 160<br>N-VALUE (Blows/300mm) ▲ 20 40 60 80 | WATER CONTENT (%) PL W.C. LL 20 40 60 80 | SAMPLE NO. | SAMPLE TYPE | RECOVERY (%) | (new title) SV/TOV (ppm or %LEL) | LABORATORY TESTING | WELL INSTALLATION | REMARKS |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | ■ | 300 mm concrete slab | 0 | | | | | | | | | | |
| | ▨ | wet, brown, CLAY, trace sand | 0.5 | | | | 1 | ◢ | 100 | <10 | | ▼ | Borehole open and wet at 0.4 mbg upon completion |
| | | | 1 | | | | 2 | ◢ | 100 | <10 | | | |
| | ▨ | saturated, brown SILTY CLAY | 1.5 | | | | 3 | ◢ | 100 | <10 | | | |
| | | | 2 | | | | 4 | ◢ | 100 | <10 | | | |
| | | | 2.5 | | | | 5 | ◢ | 100 | <10 | | | |
| | | | 3 | | | | | | | | | | |
| | | END OF BOREHOLE | | | | | | | | | | | |

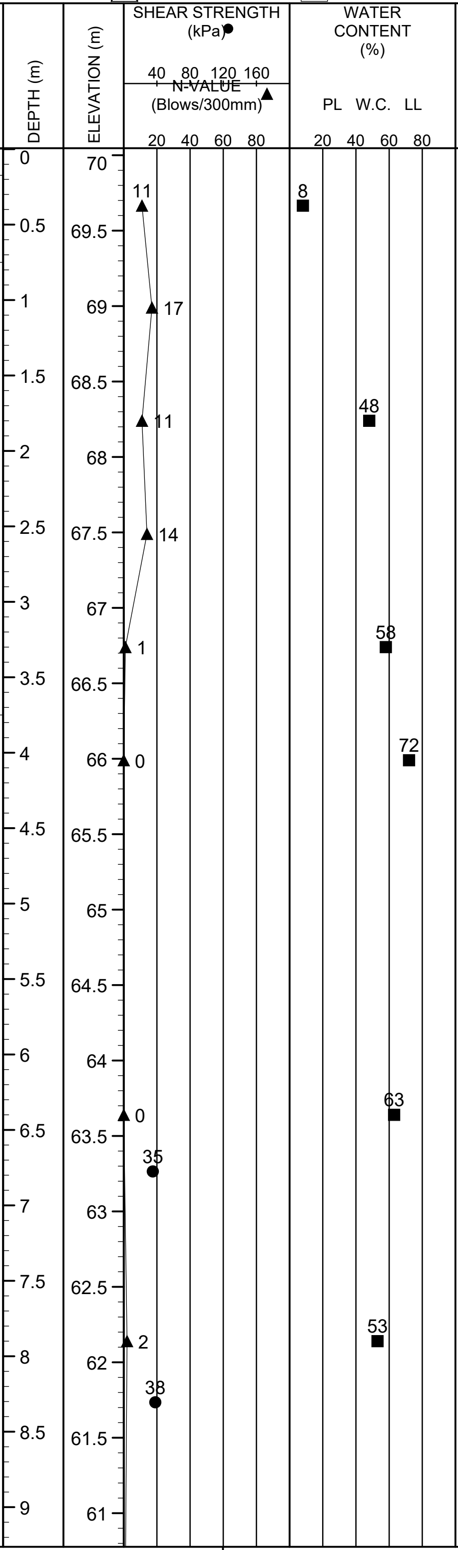
TERRAPEX

LOGGED BY: EB
INPUT BY: JZ
REVIEWED BY: JZ
DRILLING DATE: 10-APR-2023
MONITORING DATE: 10-APR-2023
PAGE 1 OF 1

CLIENT: All Saints Development LP
PROJECT NO.: CO923.00

# RECORD OF: BH102

ADDRESS: 315 Chapel Street
CITY/PROVINCE: Ottawa
NORTHING (m):
EASTING (m):
ELEV. (m) 70.04
CONTRACTOR: Strata Drilling Group
METHOD: Split Spoon Sampling
BOREHOLE DIAMETER (cm):
WELL DIAMETER (cm): -
SCREEN SLOT #: -
SAND TYPE: -
SEALANT TYPE:
SAMPLE TYPE ▮ AUGER ◩ DRIVEN ⧗ CORING ▀ DYNAMIC CONE ▯ SHELBY ⌶ SPLIT SPOON



| GWL (m) | SOIL SYMBOL | SOIL DESCRIPTION | SAMPLE NO. | SAMPLE TYPE | RECOVERY (%) | SV/TOV (ppm or %LEL) (new title) | LABORATORY TESTING | WELL INSTALLATION | REMARKS |
|---|---|---|---|---|---|---|---|---|---|
| | | 75 mm Concrete | | | | | | | |
| | | compact, wet, brown, gravel (FILL) | 1 | | 30 | | | | Borehole cave-in at 10.67 mbg and dry upon completion |
| | | very soft to very stiff, moist, grey SILTY CLAY, trace sand | 2 | | 0 | | | | |
| | | | 3 | | 80 | | | | |
| | | | 4 | | 0 | | | | |
| | | | 5 | | 100 | | | | |
| | | firm, very moist, grey SILT and CLAY | 6 | | 100 | | | | |
| | | | 7 | | 100 | | Grain Size & Atterberg | | |
| | | | | | | | | | Su= 35 KPa<br>Sr= 4 KPa |
| | | | 8 | | 100 | | | | |
| | | | | | | | | | Su= 38 KPa<br>Sr= 16 KPa |

TERRAPEX

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DRILLING DATE: 11-APR-2023
INPUT BY: JZ
MONITORING DATE: 11-APR-2023
REVIEWED BY: JZ
PAGE 1 OF 2

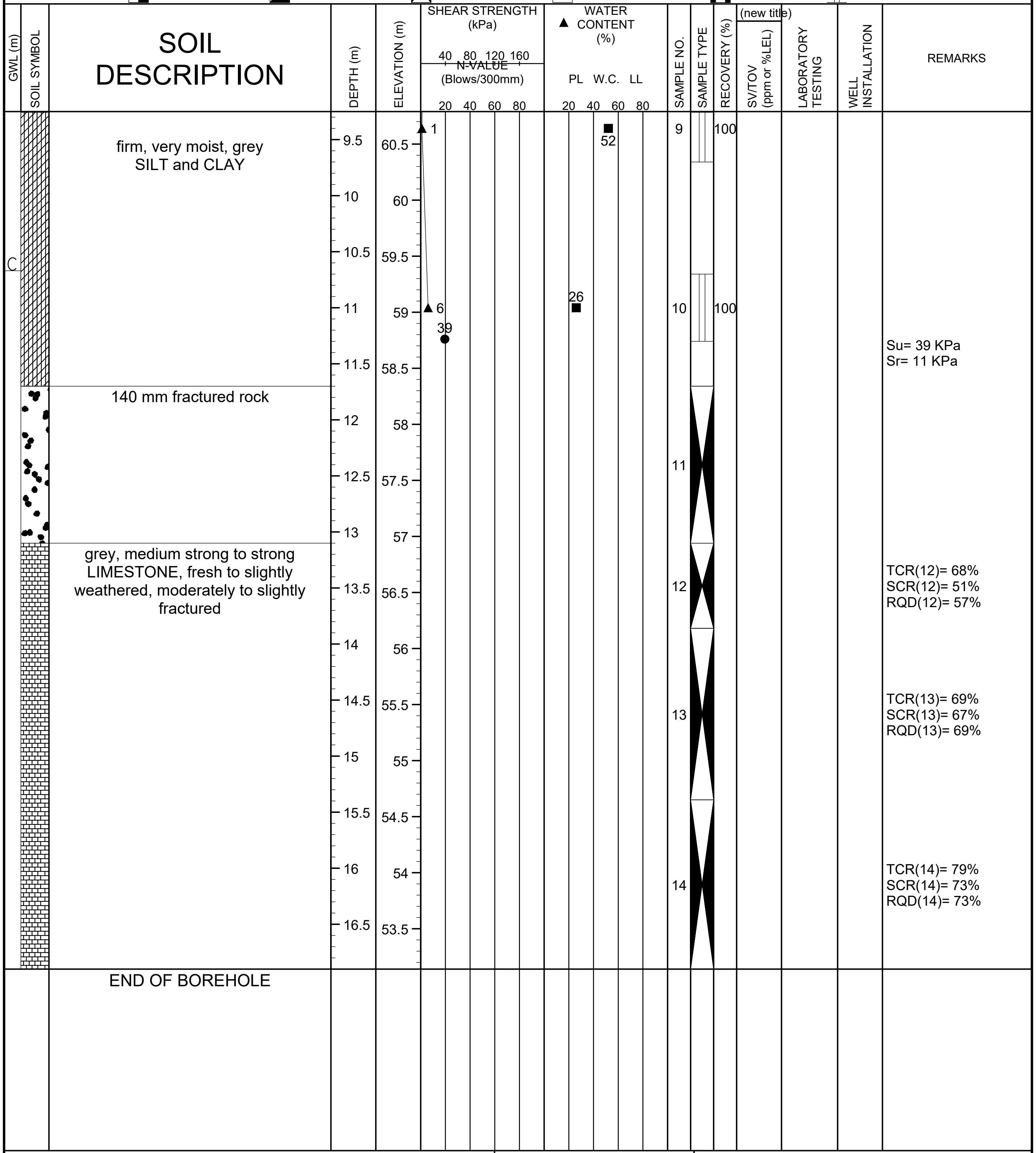
| | | |
|---|---|---|
| CLIENT: All Saints Development LP | PROJECT NO.: CO923.00 | **RECORD OF:**<br>**BH102** |
| ADDRESS: 315 Chapel Street | | |

| | | | |
|---|---|---|---|
| CITY/PROVINCE: Ottawa | NORTHING (m): | EASTING (m): | ELEV. (m) 70.04 |

| | |
|---|---|
| CONTRACTOR: Strata Drilling Group | METHOD: Split Spoon Sampling |

| | | | | |
|---|---|---|---|---|
| BOREHOLE DIAMETER (cm): | WELL DIAMETER (cm): - | SCREEN SLOT #: - | SAND TYPE: - | SEALANT TYPE: |

SAMPLE TYPE ▮ AUGER ◢ DRIVEN ⧗ CORING ▀ DYNAMIC CONE ▯ SHELBY ⌶ SPLIT SPOON



| SOIL DESCRIPTION | SAMPLE NO. | RECOVERY (%) | REMARKS |
|---|---|---|---|
| firm, very moist, grey SILT and CLAY | 9 | 100 | |
| | 10 | 100 | Su= 39 KPa<br>Sr= 11 KPa |
| 140 mm fractured rock | 11 | | |
| grey, medium strong to strong LIMESTONE, fresh to slightly weathered, moderately to slightly fractured | 12 | | TCR(12)= 68%<br>SCR(12)= 51%<br>RQD(12)= 57% |
| | 13 | | TCR(13)= 69%<br>SCR(13)= 67%<br>RQD(13)= 69% |
| | 14 | | TCR(14)= 79%<br>SCR(14)= 73%<br>RQD(14)= 73% |
| END OF BOREHOLE | | | |

TERRAPEX

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| LOGGED BY: HM | DRILLING DATE: 11-APR-2023 |
| INPUT BY: JZ | MONITORING DATE: 11-APR-2023 |
| REVIEWED BY: JZ | PAGE 2 OF 2 |

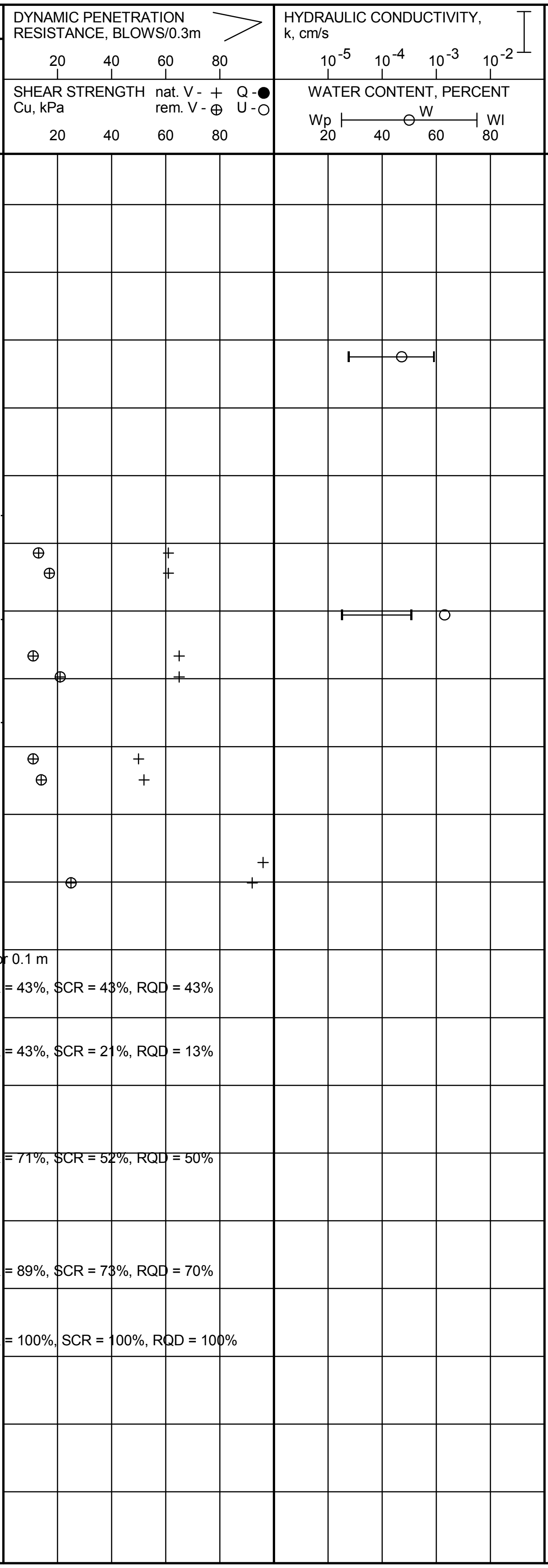
# RECORD OF BOREHOLE 16-1

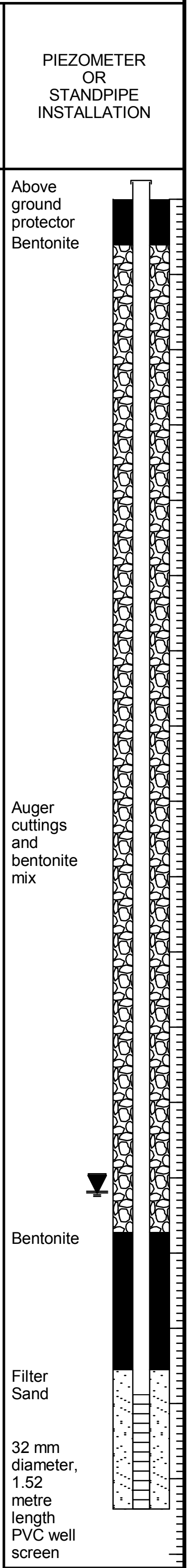
PROJECT: 63792.02
LOCATION: Refer to Borehole Location Plan, Figure 2
BORING DATE: June 13, 14, 2016

SHEET 1 OF 1
DATUM: Geodetic
SPT HAMMER: 63.5 kg; drop 0.76 m

| DEPTH SCALE METRES | BORING METHOD | SOIL PROFILE DESCRIPTION | ELEV. DEPTH (m) |
|---|---|---|---|
| 0 | Power Auger, 200 mm Diameter Hollow Stem | Ground Surface | 70.35 |
|  |  | Dark brown silty sand, trace organic material (TOPSOIL / FILL) | 69.94<br>0.41 |
| 1 |  | Brown, fine to medium grained sand, trace silt (FILL MATERIAL) | 68.72<br>1.63 |
| 2 |  | Very stiff, grey brown SILTY CLAY (WEATHERED CRUST) | 66.46<br>3.89 |
| 4 |  | Stiff, grey SILTY CLAY | 59.58<br>10.77 |
| 11 |  | Grey clayey silt, some sand and gravel with probable cobbles and boulders (GLACIAL TILL) | 58.16<br>12.19 |
| 12 | Power Rotary Diamond, NQ | Fractured, LIMESTONE bedrock | 56.13<br>14.22 |
| 14 |  | Slightly weathered, dark grey LIMESTONE bedrock | 52.95<br>17.40 |
| 17.4 |  | End of borehole |  |

| NUMBER | TYPE | BLOWS/0.3m | ADDITIONAL LAB. TESTING |
|---|---|---|---|
| 1 | 50 D.O. | 8 |  |
| 2 | 50 D.O. | 6 |  |
| 3 | 50 D.O. | 8 |  |
| 4 | 50 D.O. | 7 | (see Fig. A1) |
| 5 | 50 D.O. | 5 |  |
| 6 | 50 D.O. | 1 |  |
| 7 | 50 D.O. | W.H. |  |
| 8 | 50 D.O. | W.H. | (see Fig. A1) |
| 9 | 50 D.O. | W.H. |  |
| 10 | 50 D.O. | 3 |  |
| 11 | 50 D.O. | 5 |  |
| 12 | 50 D.O. | 50 for 0.1 m |  |
| 13 | R.C. | TCR = 43%, SCR = 43%, RQD = 43% |  |
| 14 | R.C. | TCR = 43%, SCR = 21%, RQD = 13% |  |
| 15 | R.C. | TCR = 71%, SCR = 52%, RQD = 50% |  |
| 16 | R.C. | TCR = 89%, SCR = 73%, RQD = 70% |  |
| 17 | R.C. | TCR = 100%, SCR = 100%, RQD = 100% |  |





GROUNDWATER OBSERVATIONS

| DATE | DEPTH (m) | ELEV. (m) |
|---|---|---|
| 16/06/20 | 13.18 ▽ | 57.17 |
| 16/09/26 | 13.20 ▼ | 57.15 |

DEPTH SCALE
1 to 100

Houle Chevrier Engineering

LOGGED: A.N.
CHECKED:

PROJECT: 63792.02

# RECORD OF BOREHOLE 16-2

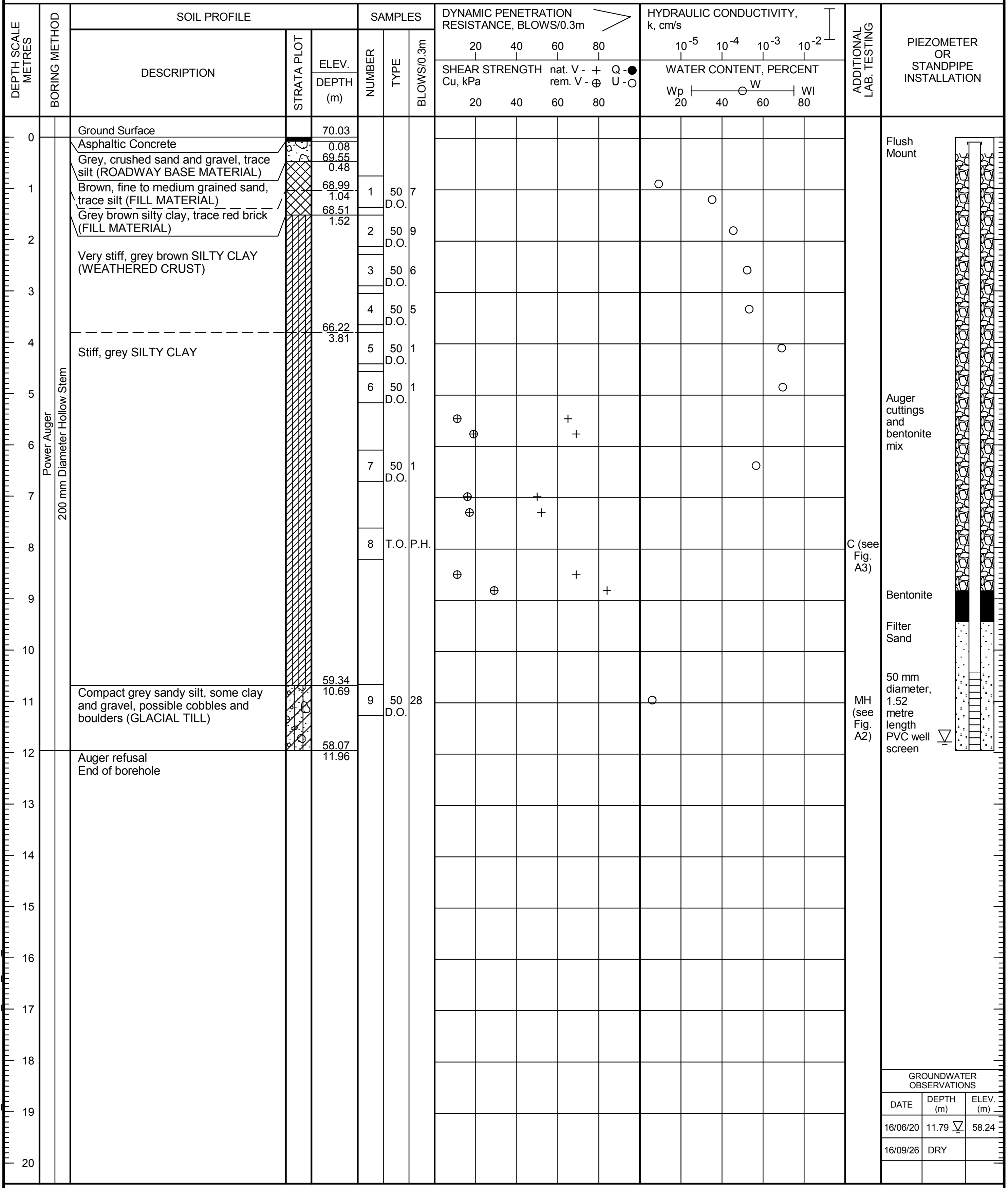
SHEET 1 OF 1

LOCATION: Refer to Borehole Location Plan, Figure 2

DATUM: Geodetic

BORING DATE: June 14, 2016

SPT HAMMER: 63.5 kg; drop 0.76 m



| DESCRIPTION | ELEV. DEPTH (m) |
|---|---|
| Ground Surface | 70.03 |
| Asphaltic Concrete | 0.08 |
| Grey, crushed sand and gravel, trace silt (ROADWAY BASE MATERIAL) | 69.55<br>0.48 |
| Brown, fine to medium grained sand, trace silt (FILL MATERIAL) | 68.99<br>1.04 |
| Grey brown silty clay, trace red brick (FILL MATERIAL) | 68.51<br>1.52 |
| Very stiff, grey brown SILTY CLAY (WEATHERED CRUST) | 66.22<br>3.81 |
| Stiff, grey SILTY CLAY | 59.34<br>10.69 |
| Compact grey sandy silt, some clay and gravel, possible cobbles and boulders (GLACIAL TILL) | 58.07<br>11.96 |
| Auger refusal<br>End of borehole | |

| NUMBER | TYPE | BLOWS/0.3m |
|---|---|---|
| 1 | 50 D.O. | 7 |
| 2 | 50 D.O. | 9 |
| 3 | 50 D.O. | 6 |
| 4 | 50 D.O. | 5 |
| 5 | 50 D.O. | 1 |
| 6 | 50 D.O. | 1 |
| 7 | 50 D.O. | 1 |
| 8 | T.O. | P.H. |
| 9 | 50 D.O. | 28 |

ADDITIONAL LAB. TESTING:

C (see Fig. A3)

MH (see Fig. A2)

PIEZOMETER OR STANDPIPE INSTALLATION:

Flush Mount

Auger cuttings and bentonite mix

Bentonite

Filter Sand

50 mm diameter, 1.52 metre length PVC well screen ▽

GROUNDWATER OBSERVATIONS

| DATE | DEPTH (m) | ELEV. (m) |
|---|---|---|
| 16/06/20 | 11.79 ▽ | 58.24 |
| 16/09/26 | DRY | |

DEPTH SCALE

1 to 100

Houle Chevrier Engineering

LOGGED: A.N.

CHECKED:

# 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 $p'_c$)

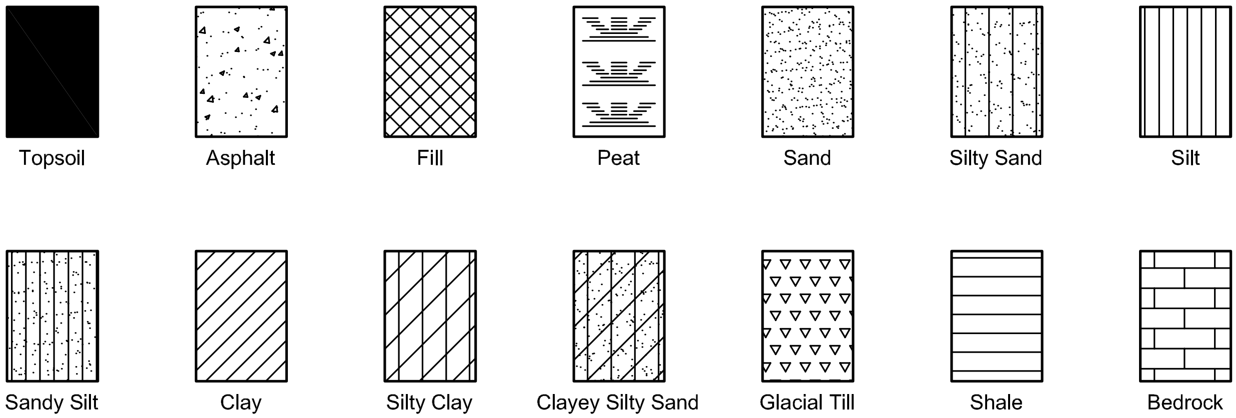
OC Ratio Overconsolidaton ratio = $p'_c / p'_o$
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

### 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.

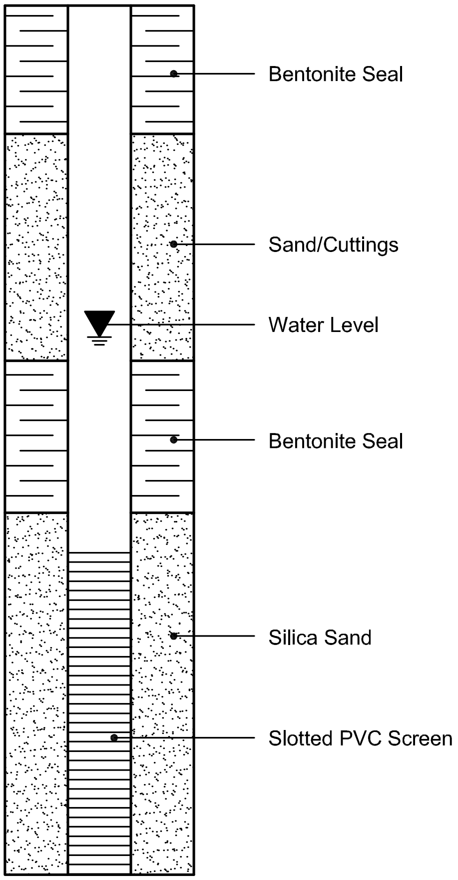
# SYMBOLS AND TERMS (continued)

## STRATA PLOT

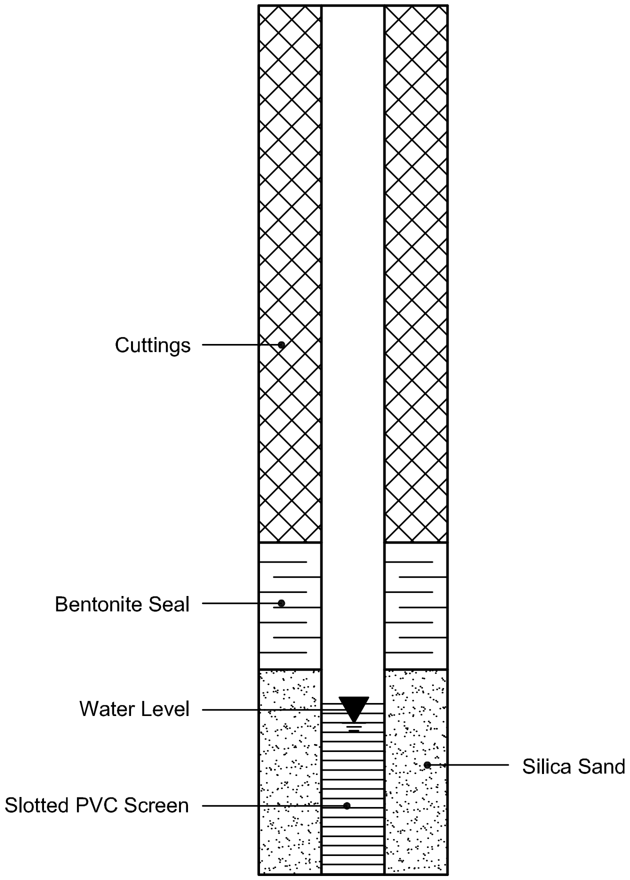


## MONITORING WELL AND PIEZOMETER CONSTRUCTION

### MONITORING WELL CONSTRUCTION



### PIEZOMETER CONSTRUCTION



Certificate of Analysis

Report Date: 21-Jul-2023

**Client:** **Paterson Group Consulting Engineers**

Order Date: 17-Jul-2023

**Client PO: 57915**

**Project Description: PG6742**

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| **Client ID:** | | BH2-23 SS6 | - | - | - | | |
| **Sample Date:** | | 14-Jul-23 09:00 | - | - | - | - | - |
| **Sample ID:** | | 2329067-01 | - | - | - | | |
| **Matrix:** | | Soil | - | - | - | | |
| | **MDL/Units** | | | | | | |
| **Physical Characteristics** | | | | | | | |
| % Solids | 0.1 % by Wt. | 63.0 | - | - | - | - | - |
| **General Inorganics** | | | | | | | |
| pH | 0.05 pH Units | 8.16 | - | - | - | - | - |
| Resistivity | 0.1 Ohm.m | 35.8 | - | - | - | - | - |
| **Anions** | | | | | | | |
| Chloride | 10 ug/g | 32 | - | - | - | - | - |
| Sulphate | 10 ug/g | 80 | - | - | - | - | - |

# APPENDIX 2

FIGURE 1 – KEY PLAN

DRAWING PG6742-1 – TEST HOLE LOCATION PLAN

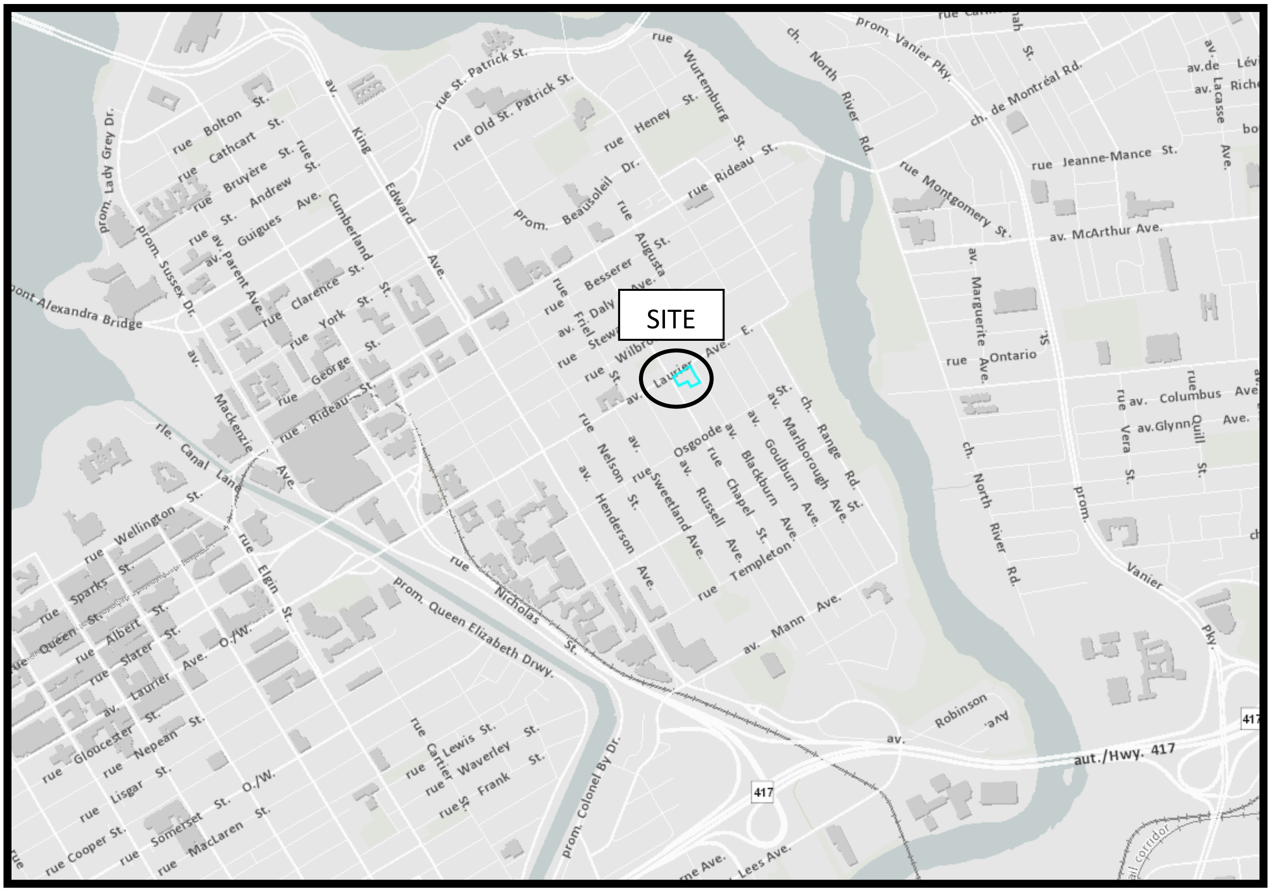
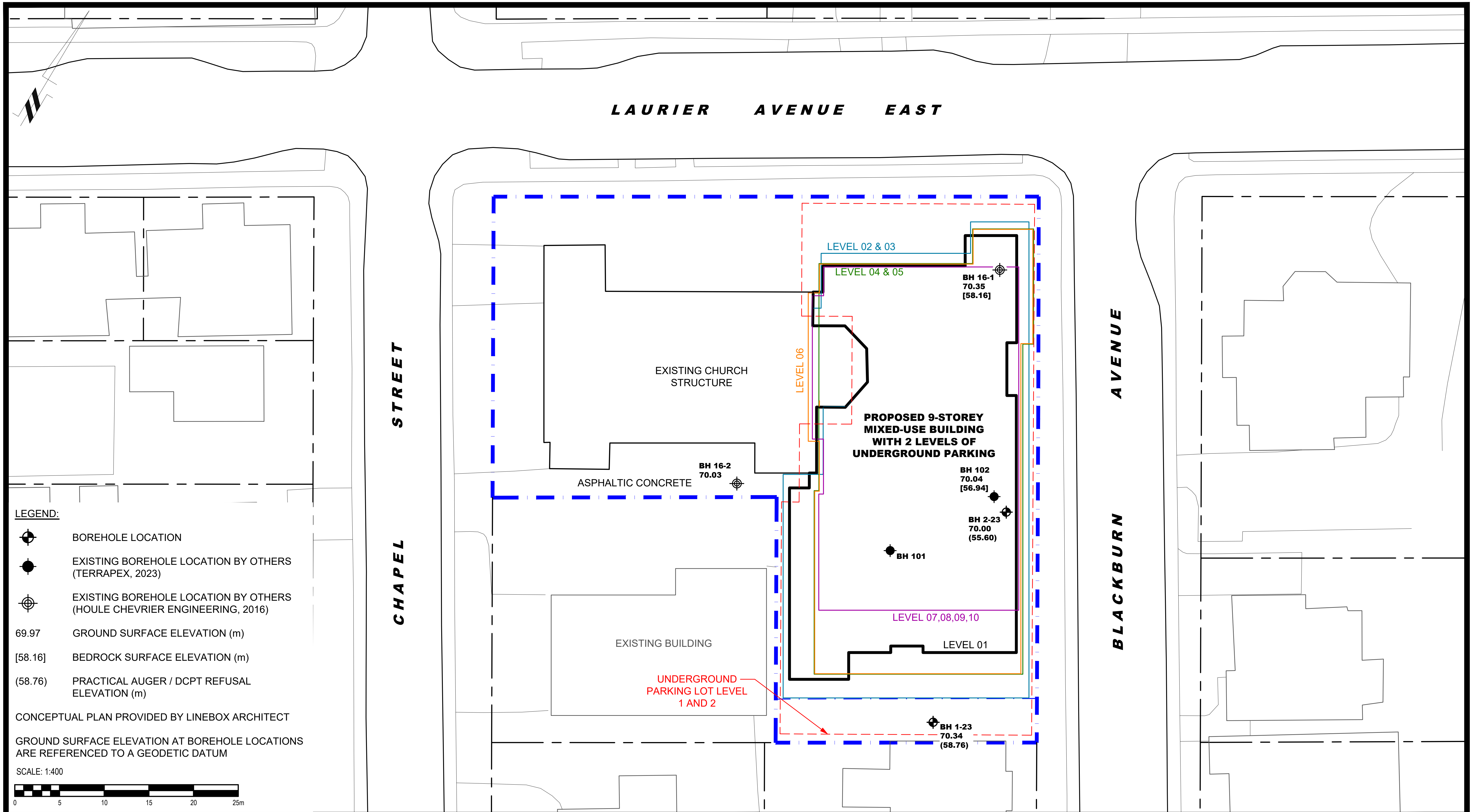


FIGURE 1

# KEY PLAN



LEGEND:

| | |
|---|---|
| ⊕ | BOREHOLE LOCATION |
| ● | EXISTING BOREHOLE LOCATION BY OTHERS (TERRAPEX, 2023) |
| ⌖ | EXISTING BOREHOLE LOCATION BY OTHERS (HOULE CHEVRIER ENGINEERING, 2016) |
| 69.97 | GROUND SURFACE ELEVATION (m) |
| [58.16] | BEDROCK SURFACE ELEVATION (m) |
| (58.76) | PRACTICAL AUGER / DCPT REFUSAL ELEVATION (m) |

CONCEPTUAL PLAN PROVIDED BY LINEBOX ARCHITECT

GROUND SURFACE ELEVATION AT BOREHOLE LOCATIONS ARE REFERENCED TO A GEODETIC DATUM

SCALE: 1:400

PATERSON GROUP
9 AURIGA DRIVE
OTTAWA, ON
K2E 7T9
TEL: (613) 226-7381

| NO. | REVISIONS | DATE | INITIAL |
|---|---|---|---|
| | | | |

ALL SAINTS DEVELOPMENTS
GEOTECHNICAL INVESTIGATION
PROPOSED MIXED-USE DEVELOPMENT
315 AND 321 CHAPEL STREET

OTTAWA, ONTARIO

Title:

# TEST HOLE LOCATION PLAN

Scale: 1:400
Drawn by: YA
Checked by: DP
Approved by: DJG
Date: 07/2023
Report No.: PG6742-1
Dwg. No.: PG6742-1
Revision No.:

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