Geotechnical Investigation

Proposed Residential Development

2506 Innes Road
Ottawa, Ontario

Prepared for Concorde Properties

Report PG6818-1 dated October 5, 2023

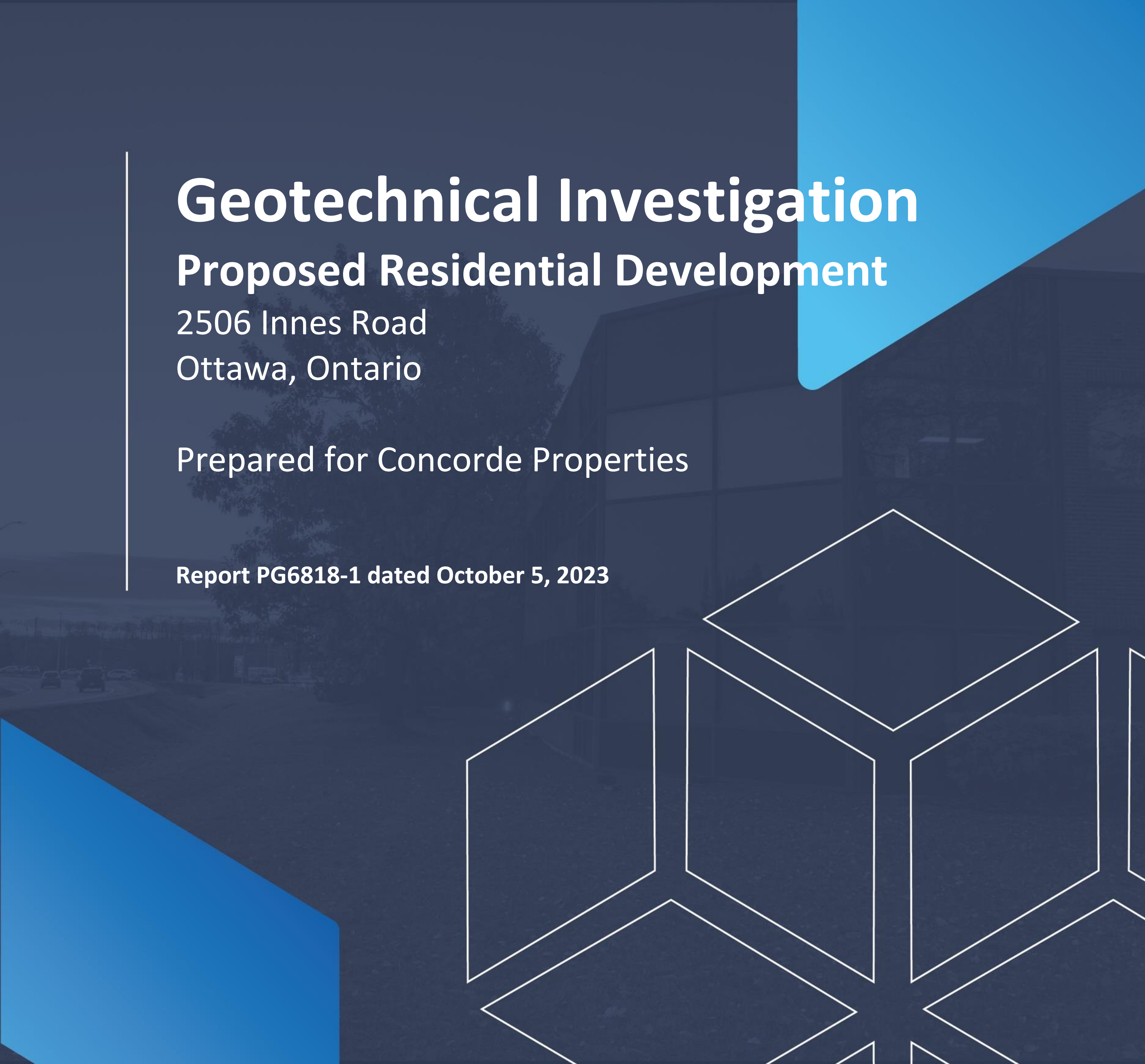


Table of Contents

Appendices

Appendix 1	Soil Profile and Test Data Sheets Symbols and Terms Analytical Testing Results
Appendix 2	Figure 1 - Key Plan Drawing PG6818-1 - Test Hole Location Plan

1.0 Introduction

Paterson Group (Paterson) was commissioned by Concorde Properties to conduct a geotechnical investigation for the proposed residential development to be located at 2506 Innes Road in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2 of this report for the general site location).

The objectives of the geotechnical investigation were to:

☐ Determine the subsoil and groundwater conditions at this site by means of boreholes.

☐ Provide geotechnical recommendations pertaining to 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.

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 2 townhouse structures, each with a basement level. At finished grade, the proposed buildings will generally be surrounded by asphalt-paved access lanes and parking areas with landscaped margins. It is also anticipated that the proposed residential development will be municipally serviced.

It is expected that the existing commercial building will need to be demolished to accommodate the construction of the proposed buildings.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the geotechnical investigation was carried out on August 10, 2023, and consisted of advancing a total of 4 boreholes to a maximum depth of 6.7 m. The borehole locations were distributed in a manner to provide general coverage of the subject site, taking into consideration underground utilities and site features. The borehole locations are shown on Drawing PG6818-1 - Test Hole Location Plan included in Appendix 2.

The boreholes were advanced using a low-clearance auger 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. The drilling procedure consisted of augering to the required depths at the selected borehole locations, and 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 initially classified on site and subsequently placed in sealed plastic bags and transported to our laboratory for further examination and classification. The depths at which the auger and split spoon were recovered from the boreholes are shown as AU and SS, respectively, on the Soil Profile and Test Data Sheets 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.

Undrained shear strength testing was conducted at regular intervals in cohesive soils (silty clay) using a field vane apparatus.

The subsurface conditions observed in the boreholes 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

Groundwater monitoring wells were installed in all boreholes to permit monitoring of the groundwater levels upon the completion of field investigation. The groundwater level readings were recorded after a suitable stabilization period. Groundwater level observations are discussed in Section 4.3 and are presented in the Soil Profile and Test Data sheets in Appendix 1.

3.2 Field Survey

The borehole locations, and the ground surface elevation at each borehole location, were surveyed by Paterson using a handheld GPS unit with respect to a geodetic datum. The locations of the boreholes, and ground surface elevation at each borehole location, are presented on Drawing PG6818-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Review

Soil samples were recovered from the subject site and visually examined in our laboratory to review the results of the field logging. All samples will be stored in the laboratory for a period of one month after issuance of this report. They will then be discarded unless we are otherwise directed.

3.4 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. The sample was tested to determine the concentration of sulphate and chloride, the resistivity and the pH of the sample. The results are presented in Appendix 1 and are discussed further in Section 6.7.

4.0 Observations

4.1 Surface Conditions

The subject site is currently occupied by an existing commercial building located within the eastern portion of the property. A number of storage containers are located at the rear of the building. The remainder of the site generally consists of asphalt-surfaced access lanes and parking areas with landscaped margins.

The site is bordered to the north by Innes Road, to the east by a commercial property, and to the south and west by residential properties. The ground surface across the site is relatively level at approximate elevation of 75 m.

4.2 Subsurface Profile

Generally, the subsurface profile at the borehole locations consists of an approximate 50 mm thickness of asphaltic concrete underlain by a layer of fill material. The fill material was generally observed to consist of loose to dense, brown silty sand with trace amounts of gravel, extending to approximate depths ranging from 3.1 to 3.8 m below the existing ground surface.

A stiff to firm, grey silty clay was encountered under the fill layer at all borehole locations, extending to the maximum depths of the boreholes at 6.1 to 6.7 m below the existing ground surface.

Reference should be made to the Soil Profile and Test Data Sheets in Appendix 1 for details of the soil profile encountered at each borehole location.

Bedrock

Based on available geological mapping, the bedrock in the subject area consists of shale of the Billings Formation with an approximate drift thickness of 25 to 50 m.

4.3 Groundwater

The groundwater levels were measured in the installed monitoring wells on August 17, 2023. The observed groundwater levels are shown in Table 1 on the next page.

Table 1 - Summary of Groundwater Level Readings

Test Hole Number	Ground Surface Elevation (m)	Groundwater Levels (m) Below Existing Surface	Groundwater Elevation (m)	Recording Date
BH1-23	74.91	2.60	72.31	August 17, 2023
BH2-23	74.93	2.82	72.11	August 17, 2023
BH3-23	74.93	2.76	72.17	August 17, 2023
BH4-23	75.16	2.78	72.38	August 17, 2023

Notes: Ground surface elevations at borehole locations were surveyed by Paterson and are referenced to a geodetic datum.

It is important to note that groundwater level readings could be influenced by surface water infiltrating the backfilled boreholes. Groundwater levels can be estimated based on the observed colour, moisture content and consistency of the recovered soil samples. Based on these observations, the static groundwater levels are expected to range between a 2.5 to 3 m depth.

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

5.0 Discussion

5.1 Geotechnical Assessment

The subject site is considered suitable for the proposed development, from a geotechnical perspective. It is recommended that the proposed buildings be founded on conventional spread footings placed on an undisturbed, stiff to firm silty clay, or on the existing fill surface which is prepared in accordance with the "Subgrade Improvement Program for Foundations" procedure provided in Section 5.2.

Due to the presence of a silty clay deposit, the site will be subjected to grade raise restrictions. The permissible grade raise recommendations are discussed 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 any fill containing significant amounts of deleterious or organic materials, should be stripped from under any buildings and other settlement sensitive structures. The existing fill material, where free of significant amounts of organic material and reviewed and approved by Paterson Group at the time of construction, can be left in place. Care should be taken not to disturb adequate bearing soils below the founding level during site preparation activities.

Any soft areas should be removed and backfilled with OPSS Granular B Type II, with a maximum particle size of 50 mm and compacted to 98% of the material's SPMDD.

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

Fill Placement

Engineered fill used for grading beneath the proposed building areas, where required, 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 buildings and paved areas should be compacted to at least 98% of the material's 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 with a maximum thickness of 300 mm and compacted by the tracks of the spreading equipment to minimize voids. If this material is to be used to build up the subgrade for areas to be paved, it must 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 placement as backfill against foundation walls or settlement-sensitive structures such as concrete sidewalks and exterior concrete entrance areas, unless a composite drainage blanket connected to a perimeter drainage system is provided.

Subgrade Improvement Program for Foundations

The following subgrade improvement program is recommended for areas where fill, free of significant amounts of deleterious materials, is encountered at the underside of footing elevation for the proposed buildings:

- ☐ The bearing surface at design underside of footing level should be sub-excavated at least 500 mm below footing level, extending at least 1 m beyond the outside faces of the footing.

- ☐ The footing subgrade should be proof-compacted with a vibratory drum roller or large vibratory plate compactor. Any poor performing areas should be removed and replaced with an OPSS Granular B Type II material placed in maximum 300 mm loose lifts and compacted by a vibratory drum roller making several passes and witnessed by the geotechnical consultant.

- ☐ The sub-excavated area should be in-filled up to the design underside of footing elevation with engineered fill, such as OPSS Granular B Type II, placed in maximum 300 mm loose lifts and compacted to at least 98% of the material's SPMDD.

5.3 Foundation Design

Bearing Resistance Values

Strip footings up to 3 m wide, and pad footings up to 5 m wide, placed on an undisturbed, stiff to firm silty clay bearing surface, or on an approved existing fill subgrade which is prepared in accordance with the “Subgrade Improvement Programs for Foundations” procedure above, can be designed using a bearing resistance value at serviceability limit states (SLS) of **100 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **150 kPa**. A geotechnical resistance factor of 0.5 was applied to the above noted bearing resistance value at ULS.

An undisturbed soil bearing surface consists of a surface from which all topsoil and deleterious materials, such as loose, frozen or disturbed soil, whether in situ or not, have been removed, in the dry, prior to the placement of concrete for footings.

Footings designed using the bearing resistance value at SLS given 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 silty clay or fill above the groundwater table, when a plane extending horizontally and vertically from the footing perimeter 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 Restriction

Due to the presence of the silty clay deposit, a permissible grade raise restriction is recommended for the proposed development at the subject site. Based on the testing results, a permissible grade raise restriction of **1.5 m** above the existing ground surface is recommended.

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

Based on the subsurface profile encountered across the subject site, the site class for seismic site response can be taken as **Class D**. If a higher seismic site class is required, a site specific shear wave velocity test needs to be completed to accurately determine the applicable seismic site classification for foundation design of the proposed buildings, according to in Table 4.1.8.4.A of the Ontario Building Code (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 Floor Slab

With the removal of all topsoil and fill, containing significant amounts of deleterious or organic materials, the undisturbed, stiff to firm silty clay or existing fill, approved by the geotechnical consultant at the time of construction, will be considered an acceptable subgrade surface on which to commence backfilling for floor slab construction.

Where the lowest level slab subgrade consists of the existing fill, a vibratory drum roller should complete several passes over the subgrade surface as a proof-rolling program. Any poor performing areas should be removed and reinstated with an engineered fill, such as OPSS Granular B Type II, which is compacted to at least 98% of its SPMDD.

It is recommended that the upper 200 mm of sub-floor fill consist of 19 mm clear crushed stone. In consideration of the anticipated groundwater conditions, an underslab drainage system, consisting of lines of perforated drainage pipe sub-drains connected to a positive outlet, should be provided in the clear crushed stone layer under the lower basement floor of the proposed residential building. This is discussed further in Section 6.1.

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.

Two distinct conditions, static and seismic, should be reviewed for design calculations. The corresponding parameters are presented below.

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:

Lateral Earth Pressure

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 Pressure

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 $0.375 \cdot a \cdot H^2/g$ 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 $P_O = .5\, K_O\, \gamma\, H^2$, where K = 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.

5.6 Pavement Design

The pavement structure presented in Tables 2 and 3 should be used for car only parking areas, at-grade access lanes and heavy truck parking and loading areas.

Table 2 - Recommended Pavement Structure - Car Only Parking Areas

Thickness (mm)	Material Description
50	**Wear Course** - 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 placed over in-situ soil or fill	

Table 3 - Recommended Pavement Structure - Access Lanes and Heavy Loading 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 placed over in-situ soil or fill	

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

The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 99% of its SPMDD using suitable vibratory equipment.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Foundation Drainage

It is recommended that a perimeter foundation drainage system be provided for the proposed buildings. The system should consist of a 100 to 150 mm diameter, perforated and corrugated plastic pipe which is surrounded on all sides by 150 mm of 19 mm clear crushed stone, placed at the footing level around the exterior perimeter of the structure. The pipe should have a positive outlet, such as a gravity connection to the storm sewer.

Underslab Drainage System

Underslab drainage will be required to control water infiltration for the underground parking levels. For preliminary design purposes, we recommend that 100 or 150 mm perforated pipes be placed at approximate 6 m centres underlying the lowest level floor slab. The spacing of the underslab drainage system should be confirmed at the time of completing the excavation when water infiltration can be better assessed.

Foundation Backfill

Backfill against the exterior sides of the foundation walls should consist of free-draining, non-frost susceptible granular materials. 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 composite drainage system, 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 of Footings Against Frost Action

Perimeter footings of heated structures are recommended to be insulated against the deleterious effects of frost action. A minimum 1.5 m thick soil cover, or an equivalent combination of soil cover and foundation insulation, should be provided in this regard.

Exterior unheated footings, such as isolated 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 an equivalent combination of soil cover and foundation insulation.

6.3 Excavation Side Slopes

The side slopes of excavations in the overburden 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.

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 is used 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

Due to the anticipated depth of excavation of the proposed buildings and the proximity to the property boundaries, temporary shoring may be required to support the overburden soils of the adjacent properties where insufficient room is available for open cut methods. 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.

The designer should also take into account the impact of a significant precipitation event and designate design measures to ensure that a precipitation event 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 designer prior to implementation.

The temporary shoring system may consist of a soldier pile and lagging system 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 pressure acting on the shoring system may be calculated using the following parameters.

Table 4 - 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 , kN/m_3	21
Submerged Unit Weight , 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 table.

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.

A minimum of 150 mm of OPSS Granular A should be placed for bedding for sewer or water pipes when placed on a soil subgrade. The bedding should extend to the spring line of the pipe. Cover material, from the spring line to a minimum of 300 mm above the obvert of the pipe, should consist of OPSS Granular A (concrete or PSM PVC pipes) or sand (concrete pipe). The bedding and cover materials should be placed in maximum 225 mm thick lifts and compacted to 98% of the SPMDD.

It should generally be possible to re-use the site materials above the cover material if the operations are carried out in dry weather conditions. Wet silty clay should not be used as backfill material 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 1.2 m below finished grade) and above the cover material should match the soils exposed at the trench walls to minimize differential frost heaving. The trench backfill should be placed in maximum 225 mm thick loose lifts and compacted to a minimum of 98% 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 (in the trench direction) and should extend from trench wall to trench wall. Generally, the seals should extend from the frost line and fully penetrate the bedding, subbedding, and cover material. The barriers should consist of relatively dry and compactable brown silty clay placed in maximum 225 mm thick loose layers compacted to a minimum of 95% of the materials standard Proctor maximum dry density. The clay seals should be placed at the site boundaries.

6.5 Groundwater Control

Based on the geotechnical investigation, it is anticipated that groundwater infiltration into excavations should be low to moderate and controllable using open sumps. The contractor should be prepared to direct water away from all subgrades, regardless of the source, to prevent disturbance to the founding medium.

Permit to Take Water

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. At least 4 to 5 months should be allowed for completion of the application 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 Persons as stipulated under O.Reg. 63/16.

Impacts to Neighbouring Properties

The excavation for the proposed buildings is not anticipated to extend significantly below the groundwater level, therefore any dewatering will be localized to the subject site, and will not impact neighbouring properties.

6.6 Winter Construction

Precautions must be taken if winter construction is considered for this project. The subsoil conditions at this site 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 the event of construction during below zero temperatures, the founding stratum should be protected from freezing temperatures by the use of straw, propane heaters and tarpaulins or other suitable means. In this regard, 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 footings are protected with sufficient soil cover to prevent freezing at founding level.

Trench excavations and pavement construction are also difficult activities to complete during freezing conditions without introducing frost into the subgrade or in the excavation walls and bottoms. Precautions should be taken if such activities are to be carried out during freezing conditions.

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 an aggressive to very aggressive corrosive environment.

6.8 Landscaping Considerations

Due to the presence of the silty clay deposit at the site, the following tree planting setbacks are recommended. Tree planting setback limits are 7.5 m for small (mature height up to 7.5 m) and medium size trees (mature tree height 7.5 to 14 m), provided that the following conditions are met:

- ☐ The underside of footing (USF) is 2.1 m or greater below the lowest finished grade for footings within 10 m from the tree, as measured from the centre of the tree trunk and verified by means of the Grading Plan.
- ☐ A small tree must be provided with a minimum of 25 m^3 of available soils volume while a medium tree must be provided with a minimum of 30 m^3 of available soil volume, as determined by the Landscape Architect. The developer is to ensure that the soil is generally un-compacted when backfilling in street tree planting locations.
- ☐ The tree species must be small (mature tree height up to 7.5 m) to medium size (mature tree height 7.5 m to 14 m) as confirmed by the Landscape Architect.
- ☐ The foundation walls are to be reinforced at least nominally (minimum of two upper and two lower 15M bars in the foundation wall).
- ☐ Grading surrounding the tree must promote drainage to the tree root zone (in such a manner as not to be detrimental to the tree), as noted on the subdivision Grading Plan.

It is well documented in the literature, and is our experience, that fast-growing trees located near buildings founded on cohesive soils that shrink on drying can result in long-term differential settlements of the structures. Tree varieties that have the most pronounced effect on foundations are seen to consist of poplars, willows and some maples (i.e. Manitoba Maples) and, as such, they should not be considered in the landscaping design.

It should be noted that plants such as shrubs and bushes in which root growth is typically limited to the upper 1 m of overburden soils, may be planted within the 7.5 m setback limit.

7.0 Recommendations

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

☐ Review of the grading plan, from a geotechnical perspective.

☐ Review of the geotechnical aspects of the excavation contractor's temporary shoring system, if required, prior to construction

☐ Observation of all bearing surfaces prior to the placement of concrete.

☐ Sampling and testing of the concrete and fill materials.

☐ Observation of all subgrades prior to backfilling.

☐ Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.

☐ 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 soils, with the exception of engineered crushed stone fill, generated by construction activities that will be transported on-site or off-site should be handled as per *Ontario Regulation 406/19: On-Site and Excess Soil Management*.

8.0 Statement of Limitations

The recommendations provided herein 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 Concorde Properties, or their agents, is not authorized without review by Paterson for the applicability of our recommendations to the alternative use of the report.

Paterson Group Inc.

Kevin A. Pickard, P.Eng.

LICENSED PROFESSIONAL ENGINEER
Oct. 5, 2023
S. S. DENNIS
100519516
Scott Dennis
PROVINCE OF ONTARIO

Scott Dennis
Scott S. Dennis, P.Eng.

Report Distribution:

❑ Concorde Properties (e-mail copy)
❑ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TESTING RESULT

SOIL PROFILE AND TEST DATA

GEOTECHNICAL INVESTIGATION

2506 Innes Road

DATUM: Geodetic **EASTING:** 377603.672 **NORTHING:** 5032559.801 **ELEVATION:** 74.91

PROJECT: Geotechnical Investigation

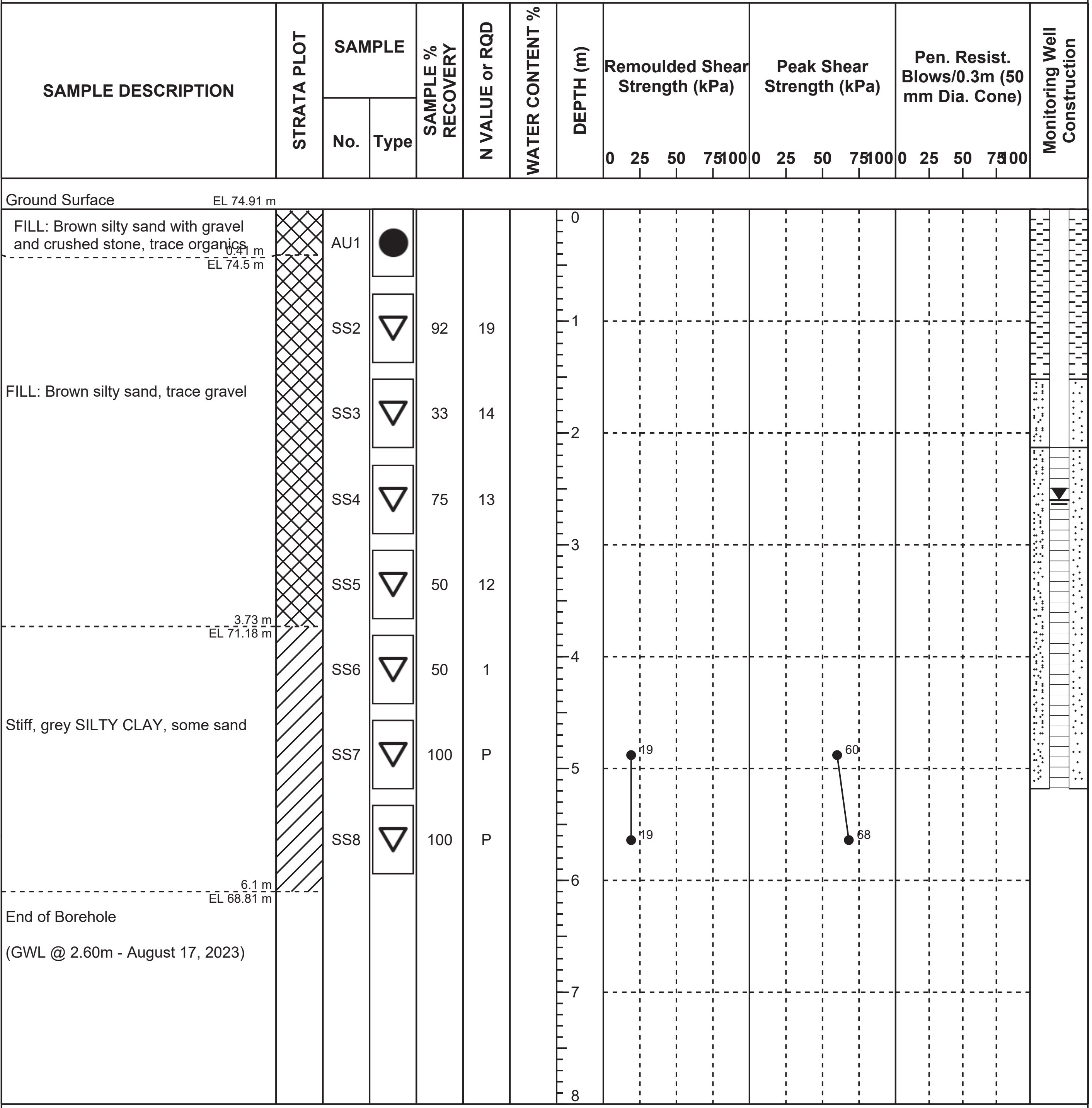
BORINGS BY: CME Low Clearance Drill

REMARKS:

DATE: August 10, 2023

FILE NO. PG6818

HOLE NO. BH 1-23



SAMPLE DESCRIPTION	SAMPLE No.	SAMPLE Type	SAMPLE % RECOVERY	N VALUE or RQD	WATER CONTENT %	Remoulded Shear Strength (kPa)	Peak Shear Strength (kPa)	Pen. Resist. Blows/0.3m (50 mm Dia. Cone)
Ground Surface EL 74.91 m								
FILL: Brown silty sand with gravel and crushed stone, trace organics	AU1	●						
0.41 m EL 74.5 m								
FILL: Brown silty sand, trace gravel	SS2	▽	92	19				
	SS3	▽	33	14				
	SS4	▽	75	13				
	SS5	▽	50	12				
3.73 m EL 71.18 m								
Stiff, grey SILTY CLAY, some sand	SS6	▽	50	1				
	SS7	▽	100	P		19	60	
	SS8	▽	100	P		19	68	
6.1 m EL 68.81 m								
End of Borehole								
(GWL @ 2.60m - August 17, 2023)								

PATERSON GROUP

SOIL PROFILE AND TEST DATA

GEOTECHNICAL INVESTIGATION

2506 Innes Road

DATUM: Geodetic **EASTING:** 377593.755 **NORTHING:** 5032568.463 **ELEVATION:** 74.93

PROJECT: Geotechnical Investigation

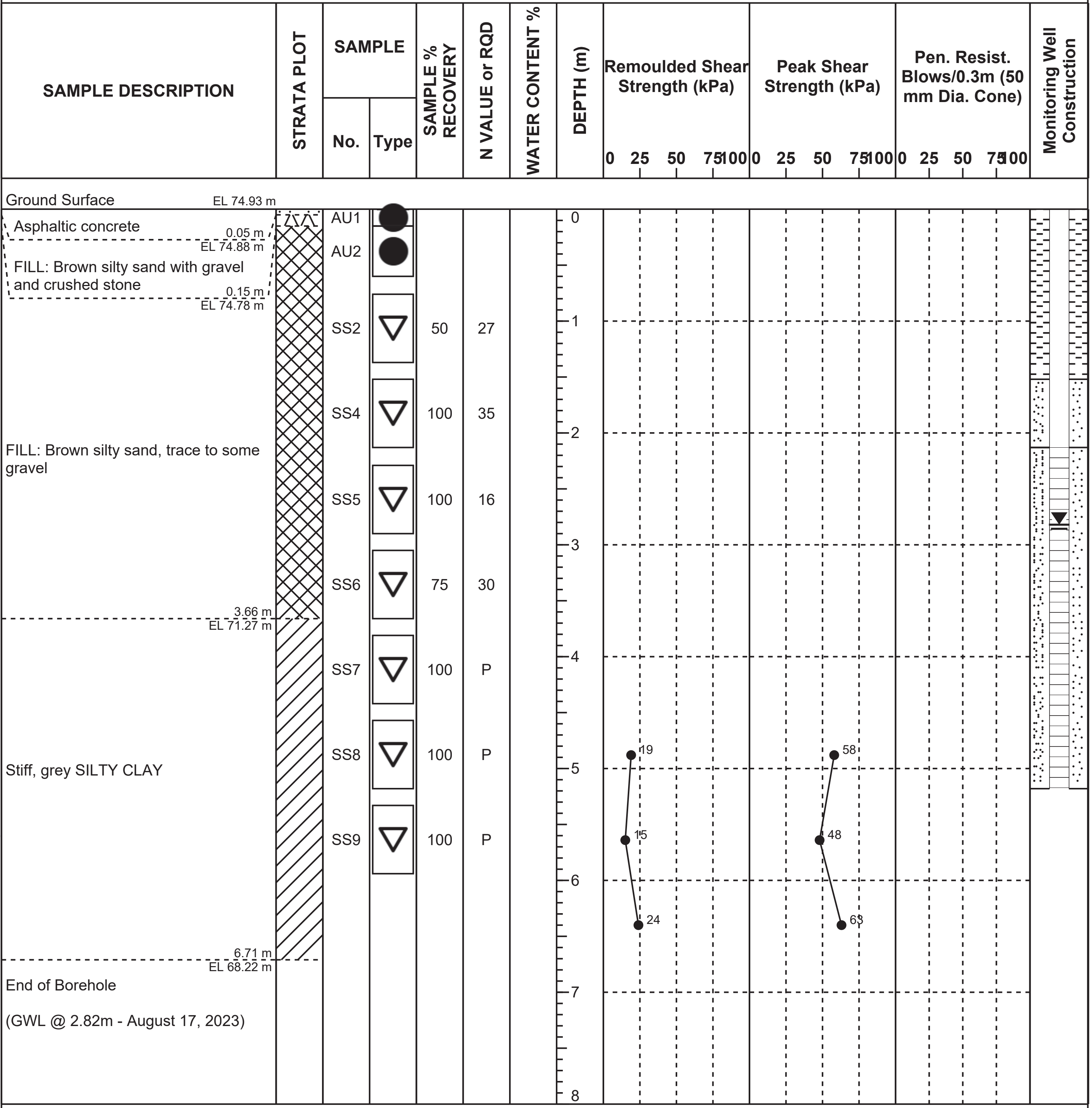
BORINGS BY: CME Low Clearance Drill

REMARKS:

DATE: August 10, 2023

FILE NO. **PG6818**

HOLE NO. BH 2-23



SAMPLE DESCRIPTION

Ground Surface EL 74.93 m

Asphaltic concrete 0.05 m EL 74.88 m

FILL: Brown silty sand with gravel and crushed stone 0.15 m EL 74.78 m

FILL: Brown silty sand, trace to some gravel 3.66 m EL 71.27 m

Stiff, grey SILTY CLAY 6.71 m EL 68.22 m

End of Borehole

(GWL @ 2.82m - August 17, 2023)

SAMPLE No.	SAMPLE Type	SAMPLE % RECOVERY	N VALUE or RQD	WATER CONTENT %
AU1	●			
AU2	●			
SS2	▽	50	27	
SS4	▽	100	35	
SS5	▽	100	16	
SS6	▽	75	30	
SS7	▽	100	P	
SS8	▽	100	P	
SS9	▽	100	P	

DEPTH (m): 0 1 2 3 4 5 6 7 8

SOIL PROFILE AND TEST DATA

GEOTECHNICAL INVESTIGATION

2506 Innes Road

DATUM: Geodetic **EASTING:** 377594.113 **NORTHING:** 5032589.393 **ELEVATION:** 74.93

PROJECT: Geotechnical Investigation

BORINGS BY: CME Low Clearance Drill

REMARKS:

DATE: August 10, 2023

FILE NO. **PG6818**

HOLE NO. BH 3-23

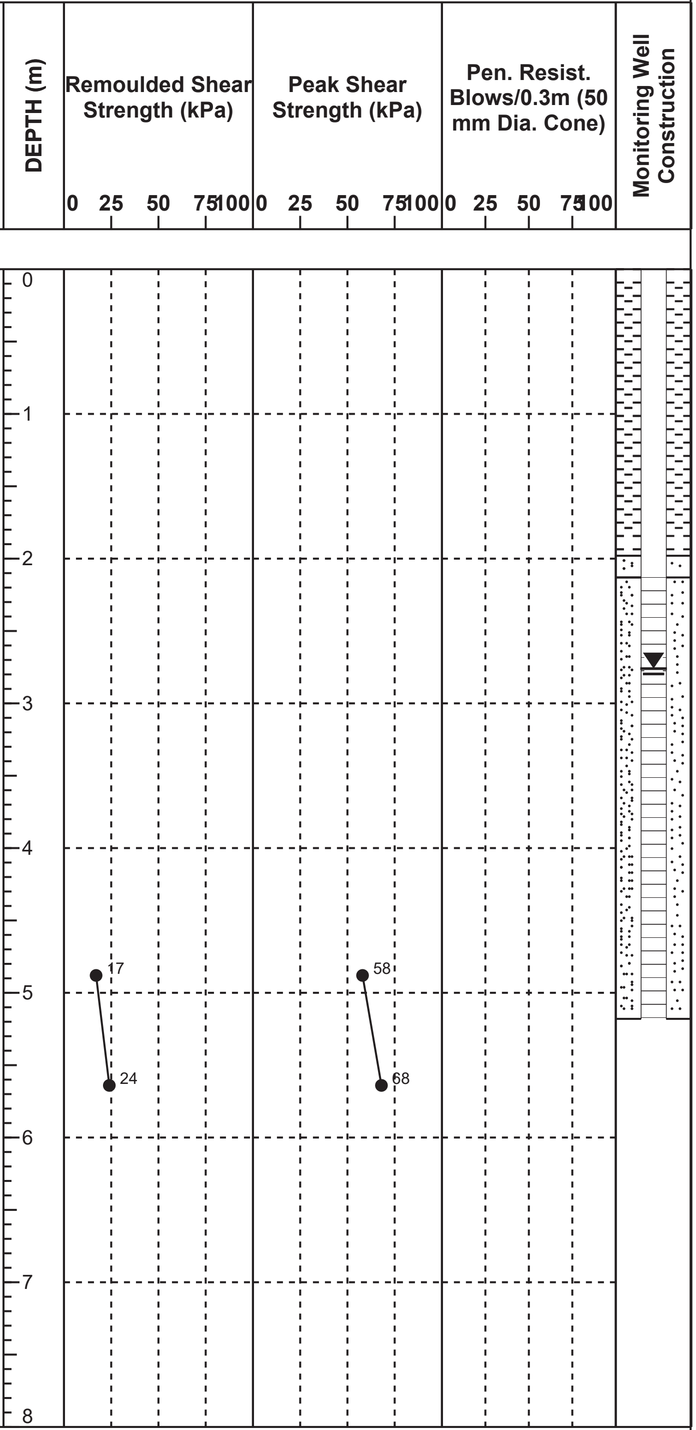
Ground Surface EL 74.93 m

SAMPLE DESCRIPTION	Depth	Elevation
Asphaltic concrete	0.05 m	EL 74.88 m
FILL: Brown silty sand with gravel and crushed stone	0.1 m	EL 74.83 m
FILL: Brown silty sand, trace gravel	3.81 m	EL 71.12 m
Stiff, grey SILTY CLAY, trace sand	6.1 m	EL 68.83 m

End of Borehole

(GWL @ 2.76m - August 17, 2023)

SAMPLE No.	SAMPLE Type	SAMPLE % RECOVERY	N VALUE or RQD	WATER CONTENT %
AU1	●			
SS2	▽	67	8	
SS3	▽	33	6	
SS4	▽	83	3	
SS5	▽	33	P	
SS6	▽	100	P	
SS7	▽	100	P	
SS8	▽	100	P	



PATERSON GROUP

SOIL PROFILE AND TEST DATA

GEOTECHNICAL INVESTIGATION

2506 Innes Road

DATUM: Geodetic **EASTING:** 377625.371 **NORTHING:** 5032581.632 **ELEVATION:** 75.16

PROJECT: Geotechnical Investigation

BORINGS BY: CME Low Clearance Drill

REMARKS:

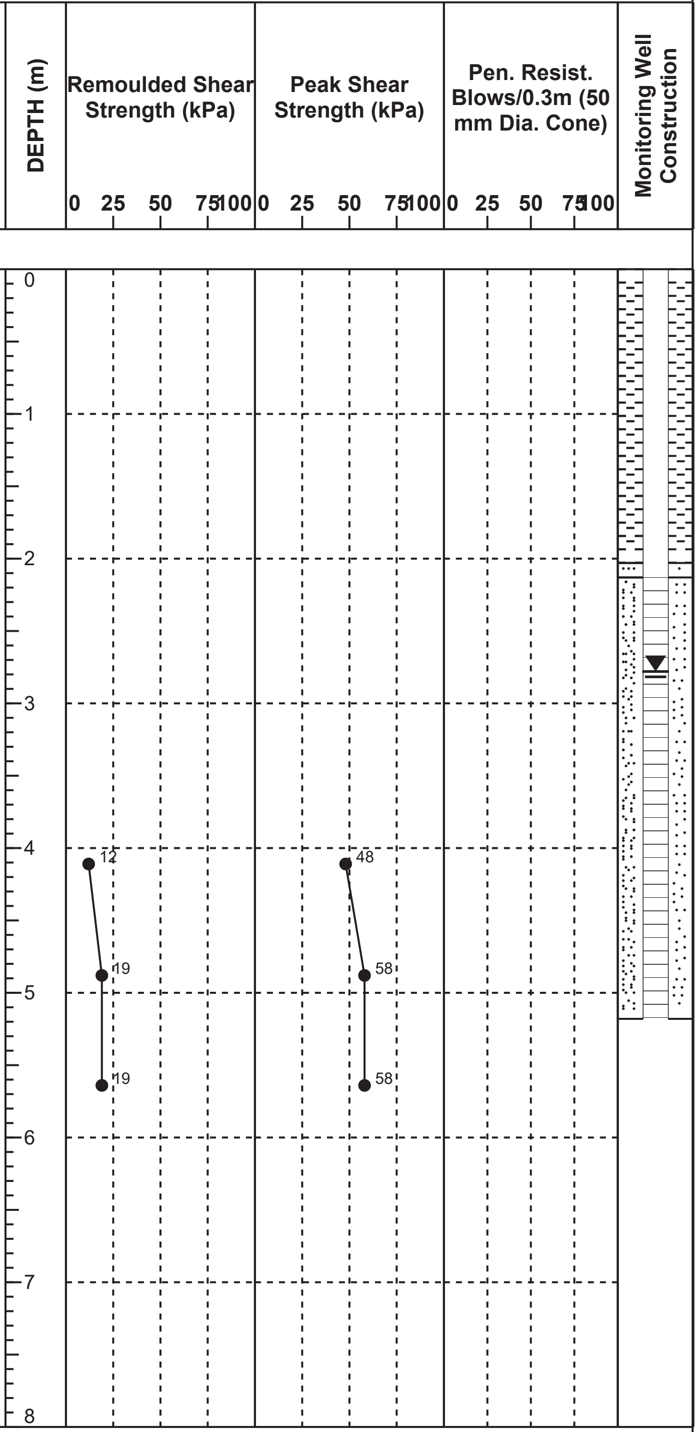
DATE: August 10, 2023

FILE NO. **PG6818**

HOLE NO. BH 4-23

SAMPLE DESCRIPTION	Depth	Elevation
Ground Surface		EL 75.16 m
Asphaltic concrete	0.05 m	EL 75.11 m
FILL: Brown silty sand with gravel and crushed stone	0.15 m	EL 75.01 m
FILL: Brown silty sand, trace gravel	3.05 m	EL 72.11 m
Stiff, grey SILTY CLAY	6.1 m	EL 69.06 m
End of Borehole		
(GWL @ 2.78m - August 17, 2023)		

SAMPLE No.	SAMPLE Type	SAMPLE % RECOVERY	N VALUE or RQD	WATER CONTENT %
AU1	●			
SS2	▽	50	9	
SS3	▽	25	4	
SS4	▽	25	3	
SS5	▽	75	1	
SS6	▽	100	P	
SS7	▽	100	P	
SS8	▽	100	P	



RSLog / Geotechnical Borehole - Geodetic - MW / paterson-group / admin / October 03, 2023 01:54 PM

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 relative strength of cohesionless soils is the compactness condition, 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. An SPT N value of “P” denotes that the split-spoon sampler was pushed 300 mm into the soil without the use of a falling hammer.

Compactness Condition	‘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 shear vane tests, unconfined compression tests, or occasionally by the Standard Penetration Test (SPT). Note that the typical correlations of undrained shear strength to SPT N value (tabulated below) tend to underestimate the consistency for sensitive silty clays, so Paterson reviews the applicable split spoon samples in the laboratory to provide a more representative consistency value based on tactile examination.

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, S_t, is the ratio between the undisturbed undrained shear strength and the remoulded undrained shear strength of the soil. The classes of sensitivity may be defined as follows:

Low Sensitivity:	$S_t < 2$
Medium Sensitivity:	$2 < S_t < 4$
Sensitive:	$4 < S_t < 8$
Extra Sensitive:	$8 < S_t < 16$
Quick Clay:	$S_t > 16$

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 NQ or larger size core. However, it can be used on smaller core sizes, such as BQ, 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, generally recovered using a piston sampler
G	-	"Grab" sample from test pit or surface materials
AU	-	Auger sample or bulk sample
WS	-	Wash sample
RC	-	Rock core sample (Core bit size BQ, NQ, HQ, etc.). Rock core samples are obtained with the use of standard diamond drilling bits.

SYMBOLS AND TERMS (continued)

PLASTICITY LIMITS AND GRAIN SIZE DISTRIBUTION

WC% - Natural water 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 at 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)

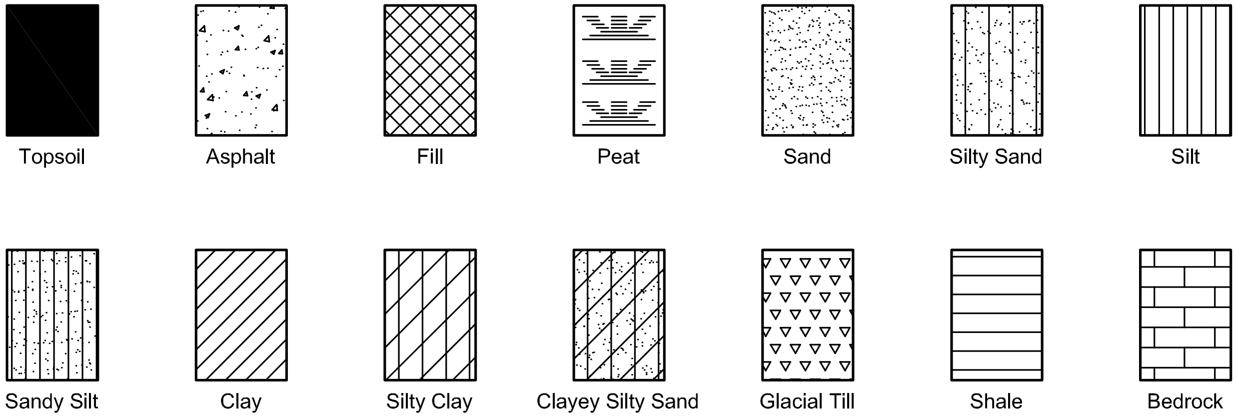
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

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.

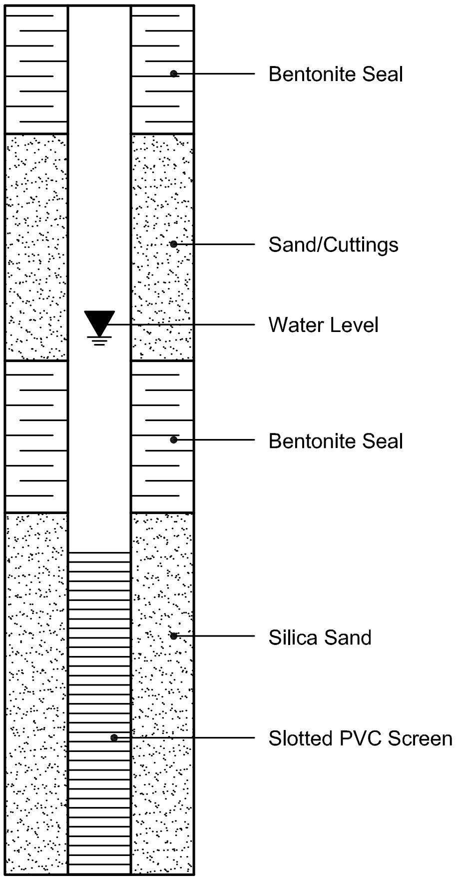
SYMBOLS AND TERMS (continued)

STRATA PLOT

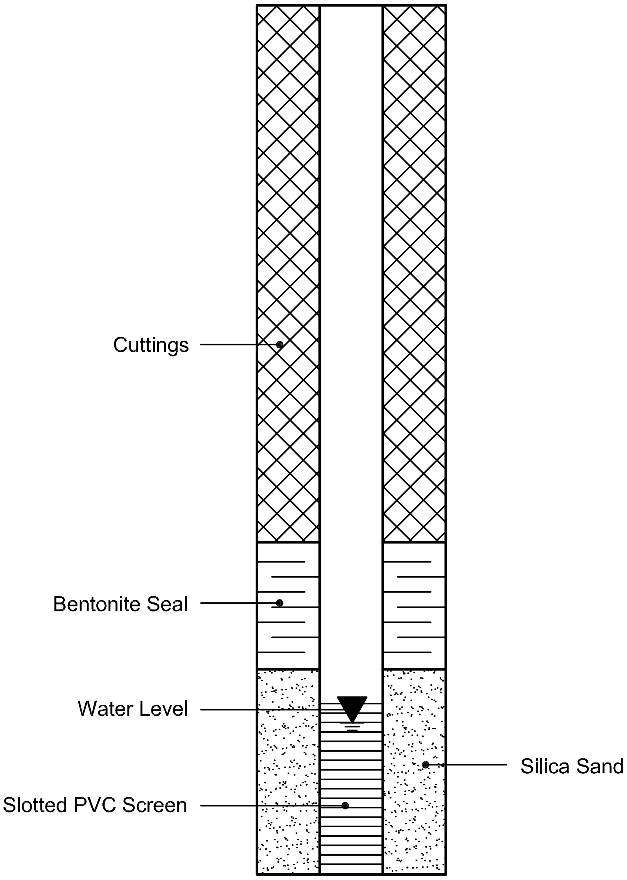


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Certificate of Analysis

Report Date: 22-Aug-2023

Client: **Paterson Group Consulting Engineers**

Order Date: 17-Aug-2023

Client PO: 58165

Project Description: PG6818

	Client ID:	BH1-23 / SS6	-	-	-		
	Sample Date:	10-Aug-23 09:00	-	-	-	-	-
	Sample ID:	2333439-01	-	-	-		
	Matrix:	Soil	-	-	-		
	MDL/Units						
Physical Characteristics							
% Solids	0.1 % by Wt.	63.7	-	-	-	-	-
General Inorganics							
pH	0.05 pH Units	7.59	-	-	-	-	-
Resistivity	0.1 Ohm.m	26.2	-	-	-	-	-
Anions							
Chloride	10 ug/g	60	-	-	-	-	-
Sulphate	10 ug/g	57	-	-	-	-	-

APPENDIX 2

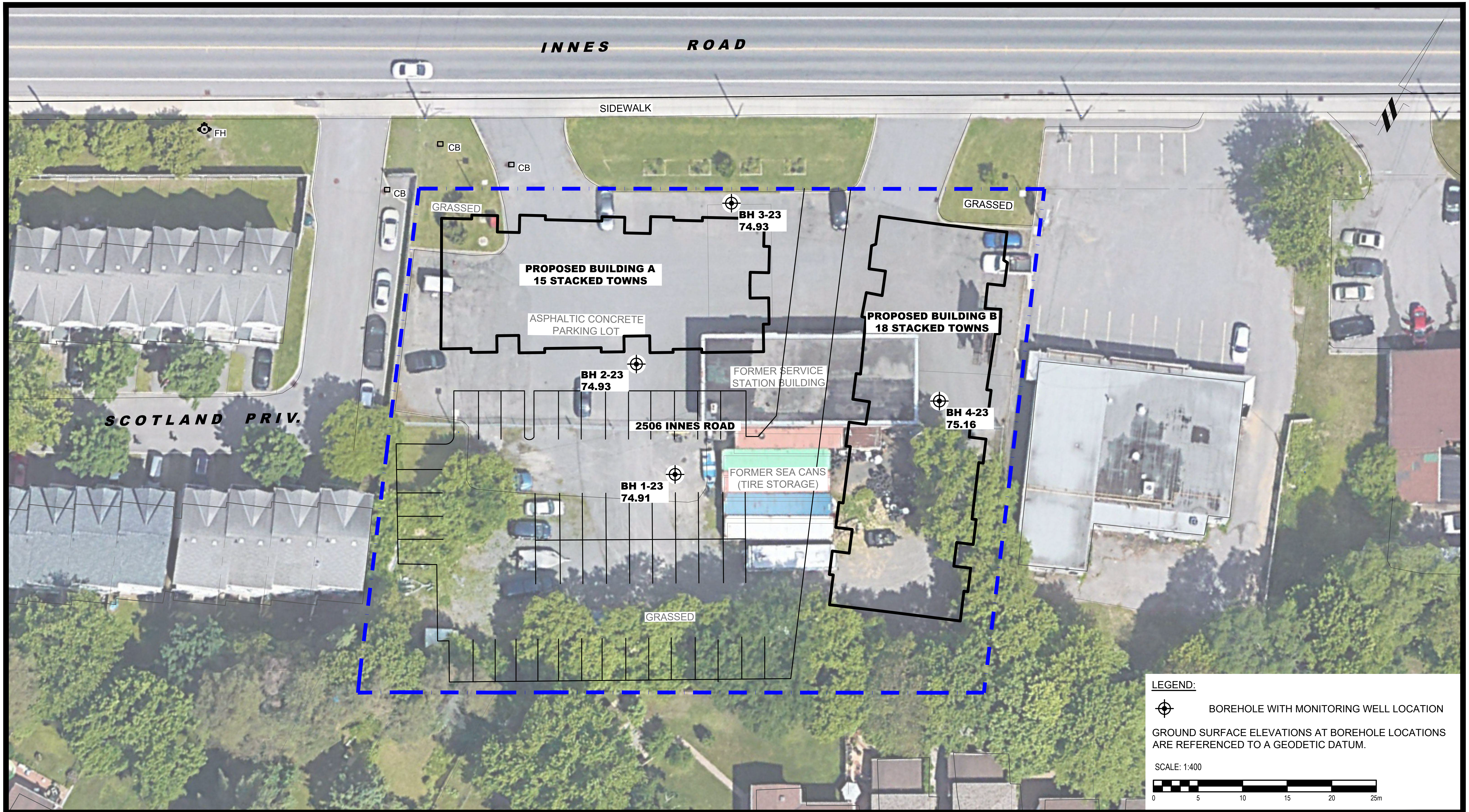
FIGURE 1 – KEY PLAN

DRAWING PG6818-1 – TEST HOLE LOCATION PLAN

FIGURE 1

KEY PLAN

PATERSON GROUP



PATERSON GROUP

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

NO.	REVISIONS	DATE	INITIAL

CONCORDE PROPERTIES
GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL DEVELOPMENT
2506 INNES ROAD

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:400	Date: 08/2023
Drawn by: GK	Report No.: PG6818-1
Checked by: SK	Dwg. No.: **PG6818-1**
Approved by: KP	Revision No.:

p:\autocad drawings\geotechnical\pg68xx\pg6818\pg6818-1 test hole location plan.dwg

11x17