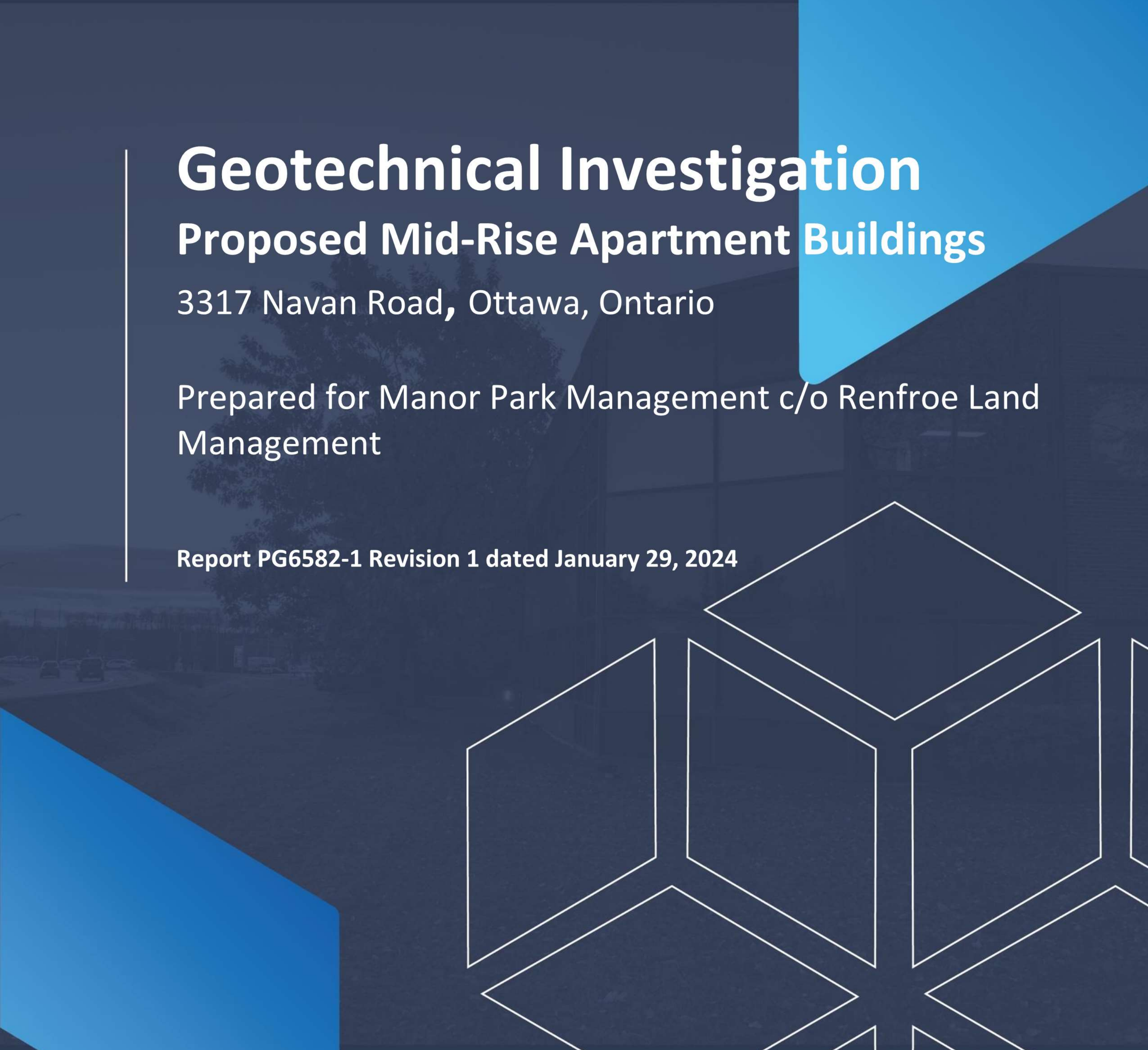

Geotechnical Investigation

Proposed Mid-Rise Apartment Buildings

3317 Navan Road, Ottawa, Ontario

Prepared for Manor Park Management c/o Renfroe Land Management

Report PG6582-1 Revision 1 dated January 29, 2024

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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by Renfroe Land Management to conduct a Geotechnical Investigation for the proposed development at 3317 Navan Road in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan presented in Appendix 2).

The objectives of the geotechnical investigation were to:

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

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

2.0 Proposed Development

Based on the available preliminary drawings, it is understood that the proposed development will consist of three mid-rise apartment buildings with one level of underground parking.

At grade access lanes, parking areas and landscaped areas are expected as part of the proposed development. It is further anticipated that the proposed development will be municipality serviced.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

Paterson conducted a geotechnical investigation at the subject site on March 17 and 20, 2023. The current investigation consisted of drilling 6 boreholes extending to a maximum depth of 8.1 m below the existing ground surface.

The test hole locations were distributed in a manner to provide general coverage of the subject site and taking into consideration underground utilities, site features and existing coverage. The locations of the test holes are shown on Drawing PG6582-1 - Test Hole Location Plan included in Appendix 2.

Previous Investigation were completed by others on July 24, 2007. The investigation consisted of advancing two boreholes within property line to a maximum depth of 7.3 m. The associated borehole logs are presented in Appendix 1. The locations of the test holes by others are shown on Drawing PG6582-1 - Test Hole Location Plan included in Appendix 2

The test holes were advanced using a Track Drill rig and operated by a two-person crew. The drilling procedure consisted of augering to the required depths at the selected locations, and sampling and testing the overburden. All fieldwork was conducted under the full-time supervision of Paterson personnel under the direction of a senior engineer.

Sampling and In Situ Testing

The soil samples were recovered from the auger flights and using a 50 mm diameter split-spoon sampler. The samples were initially classified on site, placed in sealed plastic bags and transported to our laboratory. The depths at which the auger and split-spoon samples were recovered from the boreholes are shown as AU and SS, respectively, on the Soil Profile and Test Data sheets 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.

The overburden thickness was evaluated by a dynamic cone penetration test (DCPT) at BH4-23. The DCPT consists of driving a steel drill rod, equipped with a 50 mm diameter cone at the tip, using a 63.5 kg hammer falling from a height of 760 mm. The number of blows required to drive the cone into the soil is recorded for each 300 mm increment.

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

Three (3) monitoring well was installed in BH 2-23, BH 4-23 and BH 6-23 to permit monitoring of the groundwater levels subsequent to the completion of the sampling program.

Flexible polyethylene standpipe was installed in all other boreholes to permit monitoring of the groundwater levels subsequent to the completion of the sampling program.

The groundwater observations are discussed in subsection 4.3 and presented in the Soil Profile and Test Data Sheets in Appendix 1

3.2 Field Survey

The test hole locations were selected by Paterson to provide general coverage of the proposed development taking into consideration the existing site features and underground utilities. The test hole locations and ground surface elevation at each test hole location were surveyed by Paterson with respect to a geodetic datum. The location of the test holes and ground surface elevation at each test hole location are presented on Drawing PG6582-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

Soil samples were recovered from the subject site and visually examined in our laboratory to review the results of the field logging. Moisture content testing were completed on selected soil samples. All samples 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.

3.4 Analytical Testing

One (1) soil sample was submitted for analytical testing during the current investigation to assess the corrosion potential for exposed ferrous metals and the potential of sulphate attacks against subsurface concrete structures. The sample was submitted 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 6.8.

4.0 Observations

4.1 Surface Conditions

Currently the site is an undeveloped area covered in grass and sparse tree area while some trees were removed for borehole access. A small ditch was observed to run east to west through the middle of the site. The ground surface at the subject site is relatively flat at an approx. elevation 85.70 m.

The site is bordered by residential buildings to the north, Navan Road to the south, industrial/commercial buildings, and Navan Road to the west and under construction residential development to the east.

4.2 Subsurface Profile

Overburden

Generally, the subsurface soil conditions encountered at the test hole locations consist of a layer of topsoil or fill material underlain by a loose to compact brown to grey silty sand followed by brown silty clay and grey silty clay.

Fill

At borehole BH 5-23 and BH 6-23, fill material was encountered at an approximate depth ranging from existing ground surface to 2.2 m. Fill generally observed to consist of brown silty clay to silty sand with trace of sand to clay, gravel and organics

Asphalt concrete was encountered in BH 5-23 at an approximate depth between 0.9 to 2.2 m.

Silty Sand

A loose to compact brown silty sand layer was encountered underlying the topsoil/fill at an approximate depth ranging from 0.3 to 4.6 m.

Silty clay

A very stiff to stiff brown to brown silty clay deposit was encountered underlying the brown silty sand layer at an approximate depth ranging from 0.9 to 3.0 m. in exemption to BH 4-23 and BH 6-23.

A stiff to firm grey silty clay deposit was encountered underlying the brown silty clay deposit at an approximate depth ranging from 1.8 to 8.1 m.

A DCPT was conducted at BH4-23, which encountered practical refusal at approximate depths of 41.2 m, respectively.

Bedrock

Based on available geological mapping, the bedrock within the subject site consists of Billings Formation with the approximate overburden thickness of 25 to 50 m which concur with the DCPT refusal at BH 4-23.

4.3 Groundwater

Groundwater levels measured in the standpipes are summarized in Table 1 on next page and are noted on the applicable Soil Profile and Test Data sheet presented in Appendix 1.

Table 1 - Summary of Groundwater Level Readings

Test Hole Number	Ground Surface Elevation (m)	Groundwater Depth (m)	Groundwater Elevation (m)	Recording Date
BH 1-23*	85.95	6.45	63.24	March 28, 2023
BH 2-23*	85.98	2.21	63.21	March 28, 2023
BH 3-23	85.37	0.94	61.77	March 28, 2023
BH 4-23*	86.40	1.29	63.14	March 28, 2023
BH 5-23	86.72	0.94	63.76	March 28, 2023
BH 6-23*	86.86	1.56	0.89	March 28, 2023

Note: The ground surface elevations from the current investigation are referenced to a geodetic datum.
*- Monitoring well Installed

It should be noted that the groundwater levels could be influenced by surface water infiltrating the backfilled boreholes. Long-term groundwater levels can also be estimated based on the observed color and consistency of the recovered soil samples. Based on these observations, the long-term groundwater table can be expected at approximately 1.5 to 2.5 m below the existing ground surface. It is expected that perched groundwater conditions may be encountered in the silty sand layer. The water is retained by the underlain low permeability silty clay layer.

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

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is suitable for the proposed building. It is expected that the footings may be placed on brown silty sand layer, brown silty clay crust or grey silty clay deposit.

Due to the presence of the silty clay layer, a permissible grade restriction is required for the proposed development where the silty clay is present below the building footprint. The permissible grade raise recommendations are further discussed in Subsection 5.3.

Excavation for the proposed services throughout the subject site will be completed mostly through OHSA Type 3 soils with a shallow groundwater table. It is anticipated that deep services may be placed at several locations across the subject site, and therefore the potential for basal heave should be reviewed for pipe placement.

The above and other considerations are discussed in the following paragraphs.

5.2 Site Grading and Preparation

Stripping Depth

Topsoil and fill, containing significant amount of deleterious material such as those containing organic materials should be stripped from under any buildings, paved areas and other settlement sensitive structures.

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

Fill Placement

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

Non-specified existing fill, along with site-excavated soil, can be used as general landscaping fill where settlement of the ground surface is of minor concern. This material should be spread in thin lifts and at least compacted by the tracks of the spreading equipment to minimize voids.

If approved site excavated fill 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 100% of its standard Proctor dry density, measured at in situ water content, or alternatively a minimum of 95% of the material’s SPMDD.

Non-specified existing fill and site-excavated soils are not suitable for use as backfill against foundation walls unless used in conjunction with a composite drainage membrane as discussed under subsection 6.1.

Ditch Backfilling Recommendations

It is understood that the section of the existing ditch is to be in-filled and the waterway diverted. Based on the completed test holes, the subsurface soils within the area to be in-filled consists of a topsoil layer underlain by loose to compact silty sand and stiff to firm silty clay.

In-filling of the existing ditch should be completed in accordance with the following procedure:

- The existing ditch side slopes should be stepped to provide a 1.5H:1V profile with maximum 600 mm high steps. All existing sediment and topsoil should be removed from the side slopes and bed of the creek.

- A well graded blast rock (maximum 300 mm diameter), or suitable alternative backfill to be approved by the geotechnical consultant, should be placed in maximum 500 mm loose lifts under dry conditions and compacted using suitable compaction equipment from the base of the creek up to 500 mm below the subbase level of the proposed pavement structure or to the surface of landscaped area.

- The blast rock fill layer, or suitable alternative, should be capped with a minimum 300mm thick layer of Granular B Type II. The cap layer should be placed in maximum 300 mm loose lifts and compacted to 98% of its SPMDD below the proposed roadway and design underside of footings for future residential units. The granular pad should be compacted to a minimum 98% of its SPMDD within the right-of-way.

The backfilling operations should be reviewed and approved by Paterson at the time of construction. If a portion of the ditch is to remain the design of the waterway should be reviewed by a specialized hydrologist.

Considerations should further be taken in regard to grade raise along the existing ditch. It is recommended that Paterson complete a grading plan review to comment on the installation of nearby footings if exceeding the permissible grade raise.

5.3 Foundation Design

Conventional Shallow Foundation

Strip footings, up to 2.5 m wide, and pad footings, up to 5 m wide, placed on an undisturbed, stiff brown silty clay bearing or on engineered fill placed directly over the undisturbed, stiff brown silty clay 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** incorporating a geotechnical factor of 0.5 at ULS.

Strip footings, up to 2 m wide, and pad footings, up to 4 m wide, placed on an undisturbed, firm silty clay bearing or on engineered fill placed directly over the undisturbed, firm silty clay can be designed using a bearing resistance value at SLS of **60 kPa** and a factored bearing resistance value at ULS of **100 kPa** incorporating a geotechnical factor of 0.5 at ULS.

Conventional footings placed on an undisturbed, compact silty sand bearing surface, or on engineered fill placed directly over the undisturbed, compact silty sand can be designed using a bearing resistance value at SLS of **100 kPa** and a factored bearing resistance value at ULS of **150 kPa** incorporating a geotechnical factor of 0.5 at ULS. Where silty sand is found in a loose state of compactness, it is recommended that the silty sand be proof rolled using suitable vibratory equipment, making several passes, under dry conditions and above freezing temperatures and approved by Paterson at the time of construction.

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

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.

Raft Foundation

If the above bearing resistance values provided for conventional style footings are insufficient for the proposed hotel building, consideration may be given to placing the proposed building on a raft foundation.

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

It is expected that the base of the slab is located at or below 3 m depth, the long term groundwater level will be at or below 3 m depth, the raft slab is impervious and the basement walls will be provided with a perimeter foundation drainage system.

The amount of settlement of the raft slab will be dependent on the sustained raft contact pressure. A bearing resistance value at SLS (contact pressure) of **150 kPa** can be used. The loading conditions for the contact pressure are based on sustained loads, that are generally taken to be 100% Dead Load and 50% Live Load. The factored bearing resistance (contact pressure) at ULS can be taken as **225 kPa**. A geotechnical resistance factor of 0.5 was applied to the bearing resistance value at ULS.

The preliminary modulus of subgrade reaction was calculated to be **3.8 MPa/m** for a contact pressure of **150 kPa**. The design of the raft foundation is required to consider the relative stiffness of the reinforced concrete slab and the supporting bearing medium.

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 footings.

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 sand, silty clay and engineered fill bearing media when a plane extending down and out from the bottom edges of the footing, at a minimum of 1.5H:1V, passes only through the in-situ soil or engineered fill of the same or higher capacity as that of the bearing medium.

Permissible Grade Raise Recommendation

A permissible grade raise restriction of **0.5 m** above original ground surface can be used for design purposes. If greater 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.

Based on the above discussion, if the proposed grading exceeds the permissible grade raise restrictions provided herein, several options could be considered for the foundation support of the proposed buildings:

Option 1 - Use of Lightweight Fill

Lightweight fill (LWF) can be used, consisting of EPS (expanded polystyrene) Type 12 or 15 blocks or other light weight materials which allow for raising the grade without adding a significant load to the underlying soils. However, these materials are expensive and, in the case of the EPS, are more difficult to use under the groundwater level, as they are buoyant, and must be protected against potential hydrocarbon spills. Use lightweight fill within the interior of the garage and porch areas to reduce the fill-related loads.

As an alternative to lightweight fill in the interior of the garage and porch, a structural slab can be designed to create a void beneath the floor slab and therefore reduce fill-related loads. Additional information can be provided once the design of the buildings is known.

Option 2 - Preloading or Surcharging

It is possible to preload or surcharge the subject site in localized areas provided sufficient time is available to achieve the desired settlements based on theoretical values from the settlement analysis. If this option is considered, a monitoring program using settlement plates and electronic piezometers will have to be implemented. This program will determine the amount of settlement in the preloaded or surcharged areas. Preloading to proposed finished grades will allow for consolidation of the underlying clays over a longer time period. Surcharging the site with additional fill above the proposed finished grade will add additional load to the underlying clays accelerating the consolidation process and allowing for accelerated settlements. Once the desired settlements are achieved, the site can be unloaded and the fill can be used elsewhere on site.
With both the preloading and surcharging methods, the loading period can be reduced by installing vertical wick drains or sand drains in the silty clay layer to promote the movement of groundwater towards the ground surface. However, vertical drains are expensive for this type of residential project.

5.4 Design for Earthquakes

The site class for seismic site response can be taken as **Class E** for foundations constructed at the subject site. 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

With the removal of all topsoil and deleterious fill from within the footprints of the proposed buildings, the native soil surface or approved engineered fill surface will be considered an acceptable subgrade on which to commence backfilling for floor slab construction.

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 OPSS Granular A crushed stone. Any soft areas should be removed and backfilled with appropriate backfill material prior to placing any fill. OPSS Granular B Type II, with a maximum particle size of 50 mm, is recommended for backfilling below the floor slab.

All backfill material within the footprint of the proposed buildings should be placed in maximum 300 mm thick loose layers and compacted to a minimum of 98% of the SPMDD.

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 (undrained) unit weight of the retained soil can be taken as 13 kN/m3, 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.

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 material (0.5)
γ = unit weight of fill of the applicable retained soil (kN/m^3)
H = height of the wall (m)

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

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

Seismic Earth Pressures

The total seismic force (P_{AE}) includes both the earth force component (P_o) and the seismic component (ΔP_{AE}). The seismic earth force (ΔP_{AE}) can be calculated using $0.375 \cdot a_c \cdot \gamma \cdot H^2/g$ where:

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

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

The earth force component (P_o) under seismic conditions can be calculated using $P_o = 0.5\ K_o\ \gamma\ H^2$, where $K_o = 0.5$ for the soil conditions noted above.

The total earth force (P_{AE}) is considered to act at a height, h (m), from the base of the wall, where:

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

The earth forces calculated are unfactored. For the ULS case, the earth loads should be factored as live loads, as per OBC 2020.

5.7 Pavement Structure

For design purposes, it is recommended that the rigid pavement structure for the underground parking level consist of Category C2, 32 MPa concrete at 28 days with air entrainment of 5 to 8%. The recommended rigid pavement structure is further presented in Table 2 below. The flexible pavement structure presented in Table 3 and Table 4 should be used for driveways and car only parking areas and at grade access lanes and heavy loading parking areas.

Table 2 - Recommended Rigid Pavement Structure - Lower Level

Thickness (mm)	Material Description
150	**Rigid Concrete Pavement -** 32 MPa concrete with air entrainment
300	**BASE** - OPSS Granular A Crushed Stone
SUBGRADE - Either fill, OPSS Granular B Type II material placed over in situ soil, fill or bedrock	

To control cracking due to shrinking of the concrete floor slab, it is recommended that strategically located saw cuts be used to create control joints within the concrete floor slab of the underground parking level. The control joints are generally recommended to be located at the center of the column lines and spaced at approximately 24 to 36 times the slab thickness (for example; a 0.15 m thick slab should have control joints spaced between 3.6 and 5.4 m).

The joints should be cut between 25 and 30% of the thickness of the concrete floor slab and completed as early as 4 hours after the concrete has been poured during warm temperatures and up to 12 hours during cooler temperatures.

Table 3 - Recommended Pavement Structure - Car Only Parking Areas and Fire 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 fill, in situ soil, or OPSS Granular B Type I or II material placed over in situ soil or fill	

Table 4 - Recommended Pavement Structure - Access Lanes

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
400	**SUBBASE** - OPSS Granular B Type II
SUBGRADE - Either fill, in situ soil, select subgrade material or OPSS Granular B Type I or II material placed over in situ soil or fill	

If soft spots develop in the subgrade during compaction or due to construction traffic, the affected areas should be excavated and replaced with OPSS Granular B Type I or Type II material.

The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 98% of the material's SPMDD using suitable compaction equipment.

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. Where silty clay is encountered at subgrade level, consideration should be given to installing subdrains during the pavement construction. These drains should be constructed according to City of Ottawa specifications. The drains should be connected to a positive outlet. The subgrade surface should be crowned to promote water flow to the drainage lines. The subdrains will help drain the pavement structure, especially in early Spring when the subgrade is saturated and weaker and, therefore, more susceptible to permanent deformation.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Waterproofing

A perimeter foundation drainage system in combination with a composite drainage board on the exterior foundation walls is recommended for the proposed building. The system should consist of a 150 mm diameter perforated, corrugated plastic pipe, 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 or sump pump.

Underslab Drainage

Underslab drainage is recommended to control water infiltration for the basement area. For preliminary design purposes, we recommend that 150 mm diameter perforated PVC pipes be placed at approximate 6 m spacing. The spacing of the underslab drainage system should be confirmed at the completion time of 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 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 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 required to be insulated against the deleterious effects of frost action. A minimum of 1.5 m of soil cover alone, or a minimum of 0.6 m of soil cover in conjunction with adequate foundation insulation, should be provided.

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 heated structure and require additional protection, such as soil cover of 2.1 m or an equivalent combination of soil cover and foundation insulation.

Unheated structures, such as the access ramp wall footings, may be required to be insulated against the deleterious effect of frost action. A minimum of 2.1 m of soil cover alone, or a minimum of 0.6 m of soil cover, in conjunction with foundation insulation, should be provided for the ramp wall.

6.4 Excavation Side Slopes

The side slopes of excavations in the overburden should be either 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 expected that sufficient room will be available for the greater part of the excavation to be undertaken by open-cut methods (i.e. 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. Excavations below the groundwater level should be cut back at a maximum slope of 1.5H:1V. The subsoil at this site is considered to be mainly a Type 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

Temporary shoring may be required for the overburden soil to complete the required excavations where insufficient room is available for open cut methods. 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 shoring system could consist of a soldier pile and lagging system. Any additional loading due to street traffic, construction equipment, adjacent structures and facilities, etc., should be included to the earth pressures described below. This system could be cantilevered, anchored or braced. The shoring system is recommended to be adequately supported to resist toe failure, if required, by means of extending the piles into the bedrock through pre-augered holes, if a soldier pile and lagging system is the preferred method.

The earth pressures acting on the temporary shoring system may be calculated with the following parameters.

Table 5 – Soils Parameter for Shoring System Design

Parameters	Values
Active Earth Pressure Coefficient (K_a)	0.33
Passive Earth Pressure Coefficient (K_p)	3
At-Rest Earth Pressure Coefficient (K_O)	0.5
Unit Weight (γ), kN/m^3	20
Submerged Unit Weight (γ), kN/m^3	13

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

The hydrostatic groundwater pressure should be included to the earth pressure distribution wherever the effective unit weights are 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.

Excavation Base Stability

The base of supported excavations can fail by three (3) general modes:

☐ Shear failure within the ground caused by inadequate resistance to loads imposed by grade difference inside and outside of the excavation,
☐ Piping from water seepage through granular soils, and
☐ Heave of layered soils due to water pressures confined by intervening low permeability soils.

Shear failure of excavation bases is typically rare in granular soils if adequate lateral support is provided. Inadequate dewatering can cause instability in excavations made through granular or layered soils. The potential for base heave in cohesive soils should be determined for stability of flexible retaining systems.

The factor of safety with respect to base heave, FS_b, is:

$$FS_b = N_b S_u / \sigma_z$$

where:

N_b - stability factor dependent upon the geometry of the excavation and given in Figure 1 on the following page.
S_u - undrained shear strength of the soil below the base level
σ_z - total overburden and surcharge pressures at the bottom of the excavation

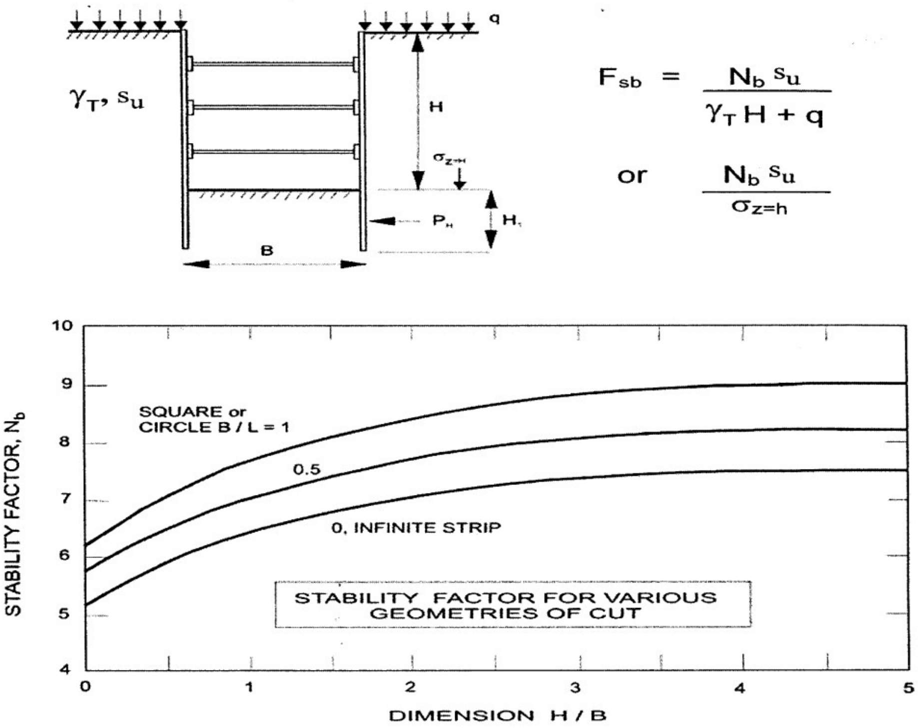


Figure 1 - Stability Factor for Various Geometries of Cut

The potential for basal heaving depends on the configuration of the excavation. A minimum factor of safety of 1.5 against basal heaving is recommended. In the case of soft to firm clays, **a factor of safety of 2 is recommended** for base stability.

Where the factor of safety is smaller than the required value, it is recommended that means to increase the factor of safety be provided such as stress relief cut at top of excavation and/or deepening the shoring system.

Excavation Precautions

For open cut situations, due to the subsoil conditions at the depth of the proposed services and the size and weight of the construction equipment and proposed pipe lengths, the following precautions should be taken while working in proximity to any open cut excavations:

☐ Where an engineered cut-off wall is to be used for shoring the deep excavations, the wall should be embedded at a sufficient depth to reduce the risk of failure at the base of the excavation.
☐ Trench boxes, where used, should be in direct contact with the trench wall soils. Periodic inspection by Paterson to verify the soil to wall contact is recommended.
☐ Deep excavations should be backfilled within 24 hours to mitigate risk of basal heaving and deformation of the trench walls (slope failure).
☐ Stockpiled soils are to be kept away from any deep excavations.
☐ Where possible, heavy construction traffic should be limited in proximity to the edge of excavation to limit vibration of the sensitive clay soil deposit.

6.5 Pipe Bedding and Backfill

Bedding and backfill materials should be in accordance with the most recent Material Specifications and Standard Detail Drawings from OPSD.

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. If the bedding is placed on bedrock, the thickness of the bedding should be increased to 300 mm for sewer pipes. Where deep services are installed in soft to firm grey silty clay, a non-woven geotextile such as Terrafix 270 R can be used to protect the silty clay surface and reinforce the base during compaction.

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 95% 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.

Where hard surface areas are considered above the trench backfill, the trench backfill material within the frost zone (about 1.5 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 95% of the material standard Proctor maximum dry density.

Clay Seals

To reduce long-term lowering of the groundwater level at this site, clay seals should be provided in the service trenches that are located in the areas underlain by sensitive silty clay. 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 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. Periodic inspection of the clay seal placement work should be completed by Paterson personnel during service installation operations.

6.6 Groundwater Control

It is anticipated that groundwater infiltration into the excavations should be minimum and controllable using open sumps. 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.

Groundwater Control for Building Construction

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 Person as stipulated under O.Reg. 63/16. If a project qualifies for a PTTW based upon anticipated conditions, an EASR will not be allowed as a temporary dewatering measure while awaiting the MECP review of the PTTW application.

6.7 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 in the subgrade or in the excavation walls and bottoms. Precautions should be taken if such activities are to be carried out during freezing conditions. Additional information could be provided, if required.

6.8 Corrosion Potential and Sulphate

The results of the analytical testing of one (1) soil sample show that the sulphate content is less than 0.1%. This result is indicative that Type 10 Portland cement (GU cement) would be appropriate. The results of the chloride content and pH indicate that they are not significant factors in creating a corrosive environment for exposed ferrous metals at this site while the resistivity tests yielded results indicative of a moderate to slightly aggressive corrosive environment.

6.9 Tree Planting Restrictions

Paterson completed a soils review of the subject development and neighboring development to determine applicable tree planting setbacks, in accordance with the City of Ottawa Tree Planting in Sensitive Marine Clay Soils (2017 Guidelines). Atterberg limits testing was completed for recovered silty clay samples during the historical geotechnical investigations. Grain size distribution analysis was also completed on 1 soil sample. The above-noted test results were completed on samples taken at depths between the anticipated design underside of footing elevation and 3.5 m depth below the anticipated finished grade. The soil profiles are presented on the Soil Profile and Test Data Sheet attached to this memorandum. The locations of test holes are shown Drawing PG6582-1 - Test Hole Location Plan in Appendix 2. The results of our testing are presented in Tables 6 and 7.

Table 6 – Atterberg Limits Results

Test Hole	Sample	Depth (m)	Liquid Limit (%)	Plastic Limit (%)	Plastic Index (%)	Moisture Content (%)	Classification
BH 13-22	SS3	1.8	69	19	50	58	CL
BH 1-14	TW3	2.6	77	26	52	82	CH

Notes: CL: Inorganic Clay of Low Plasticity; CH: Inorganic Clay of High Plasticity

Table 7 – Summary of Grain Size Distribution

Test Hole	Sample	Depth (m)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
BH 13-22	SS4	2.6	0.0	14.4	39.1	46.5

A medium to high sensitivity clay soil was encountered between the anticipated underside of footing elevations and 3.5 m below the preliminary finished grade as per City Guidelines at the subject site. Based on our Atterberg Limits' test results, the modified plasticity limits generally exceed 40% for the majority of the boreholes across the subject site. Therefore, the following tree planting setbacks are recommended for the medium to high sensitivity area.

Large trees (mature height over 14 m) can be planted within this area provided a tree to foundation setback equal to the full mature height of the tree can be provided (e.g. in a park or other green space). A tree planting setback limit of **7.5 m** is applicable for small (mature tree height up to 7.5m) and medium size trees (mature tree height 7.5 m 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 must be satisfied 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 as indicated procedural changes below. **It should be noted that where the footings are proposed at a shallower depth, a combination of engineered fill and/or root barrier system can be designed to accommodate a reduced footing depths which can be discussed in a separate report upon completion of the design grading plans.**

☐ A small tree must be provided with a minimum of 25 m^3 of available soil 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.

The tree planting restriction mentioned above be lowered to 4.5 m from foundation walls for any type of medium and large tree species, if the underside of footing (USF) is 3.5 m or greater below the lowest finished grade. Standard planting is applicable to small trees and bushes around the foundation.

7.0 Recommendations

It is a requirement for the foundation 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.
- 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.
- Periodic inspection of the installation of the underfloor and perimeter drainage and waterproofing systems.
- Observation of all subgrades prior to backfilling and placement of mud slabs.
- Field density tests to determine the level of compaction achieved.
- Sampling and testing of the bituminous concrete including mix design reviews.

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*.

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

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 Manor Park Management and Renfroe Land Management 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.

LICENSED PROFESSIONAL ENGINEER
Jan 29, 2024
J. R. VILLENEUVE
100504344
PROVINCE OF ONTARIO

Balaji Nirmala, M.Eng.

Joey R. Villeneuve, M.A.Sc., P.Eng

Report Distribution:

☐ Manor Park Management
☐ Renfroe Land Management
☐ Paterson Group

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

BOREHOLE BY OTHERS

ATTERBERG LIMITS TESTING RESULTS

GRAIN SIZE DISTRIBUTION ANALYSIS RESULTS

ANALYTICAL TESTING RESULTS

patersongroup Consulting Engineers
9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

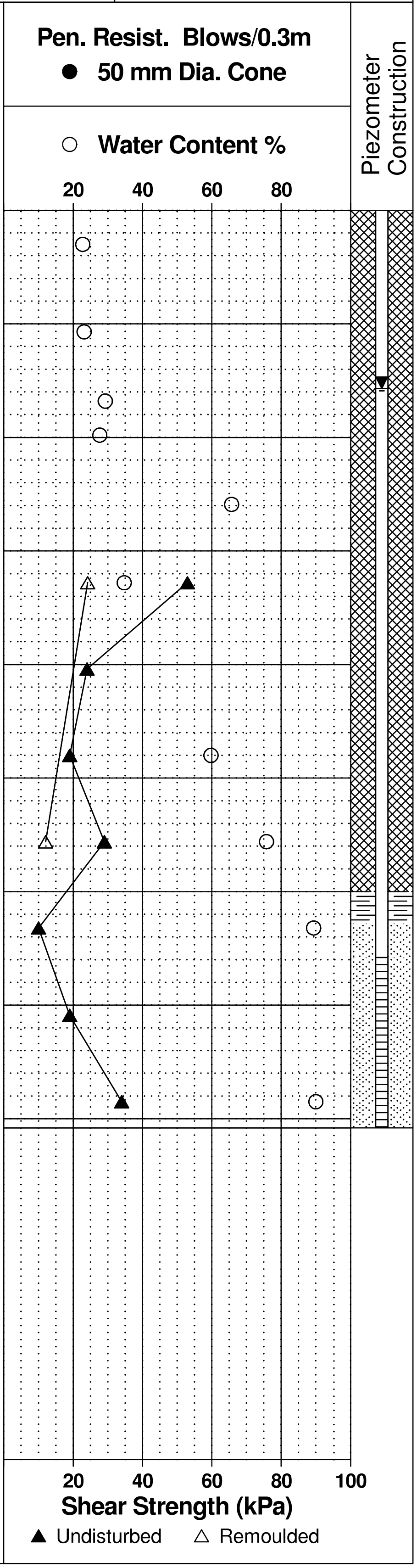
Geotechnical Investigation
Proposed Development - 3317 Navan Road
Ottawa, Ontario

DATUM Geodetic
REMARKS
BORINGS BY Track-Mount Power Auger
DATE March 17, 2023
FILE NO. **PG6582**
HOLE NO. **BH 1-23**

SOIL DESCRIPTION	
Ground Surface	
TOPSOIL	0.25
Loose to compact, brown **SILTY SAND**	1.68
Very stiff to stiff, brown **SILTY CLAY,** some sand	2.21
Stiff to firm, grey **SILTY CLAY**	8.08
End of Borehole (GWL @ 6.45m depth on March 28, 2023)	

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
SS	1	50	3
SS	2	83	11
SS	3	83	3
SS	4	100	1

DEPTH (m)	ELEV. (m)
0	85.95
1	84.95
2	83.95
3	82.95
4	81.95
5	80.95
6	79.95
7	78.95
8	77.95



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Development - 3317 Navan Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

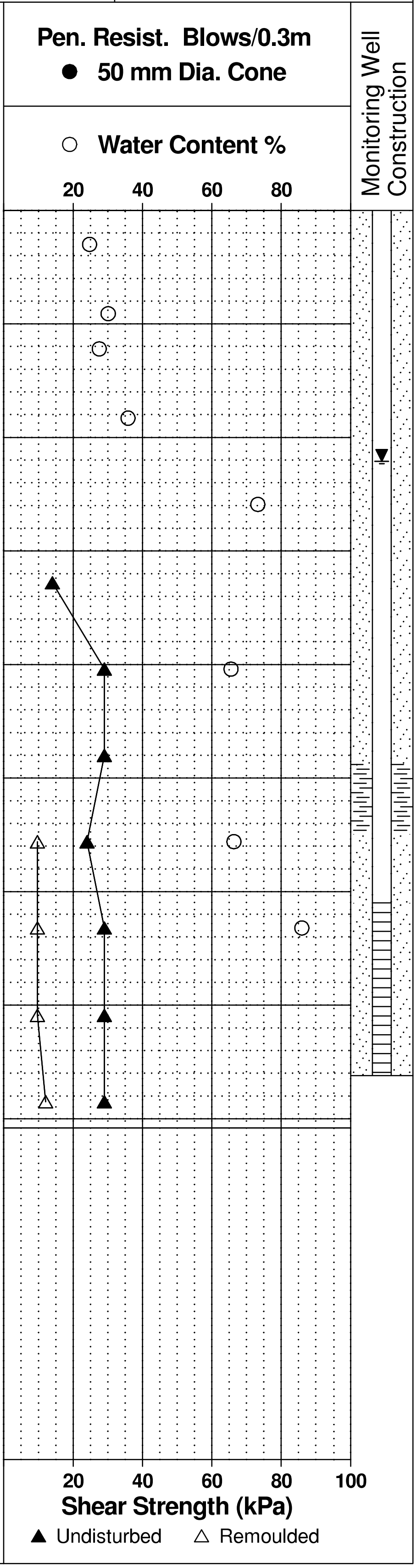
DATE March 17, 2023

FILE NO. **PG6582**

HOLE NO. **BH 2-23**

SOIL DESCRIPTION		SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD
GROUND SURFACE					
TOPSOIL	0.28	SS	1	62	7
Compact to loose, reddish brown to brown **SILTY SAND**		SS	2	83	10
	1.83	SS	3	33	4
Stiff, brown **SILTY CLAY,** trace sand	2.13				
Soft to firm, grey **SILTY CLAY**		SS	4	100	1
	8.08				
End of Borehole (GWL @ 2.21m depth on March 28, 2023)					

DEPTH (m)	ELEV. (m)
0	85.98
1	84.98
2	83.98
3	82.98
4	81.98
5	80.98
6	79.98
7	78.98
8	77.98



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Development - 3317 Navan Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

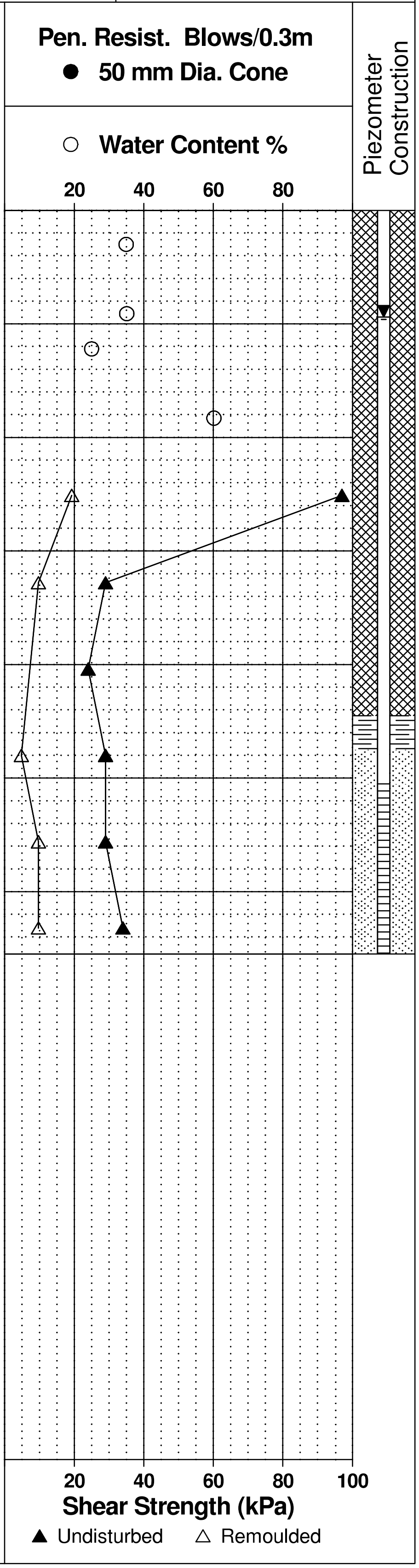
BORINGS BY Track-Mount Power Auger

DATE March 20, 2023

FILE NO. PG6582

HOLE NO. BH 3-23

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD		
Ground Surface						0	85.37
TOPSOIL 0.36		SS	1	25	6		
Loose, reddish brown **SILTY SAND** 0.91							
Very stiff to stiff, brown **SILTY CLAY** with sand seams		SS	2	75	4	1	84.37
1.83		SS	3	50	2		
						2	83.37
Stiff to firm, grey **SILTY CLAY**						3	82.37
						4	81.37
						5	80.37
						6	79.37
6.55							
End of Borehole (GWL @ 0.94m depth on March 28, 2023)							



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Development - 3317 Navan Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

DATE March 20, 2023

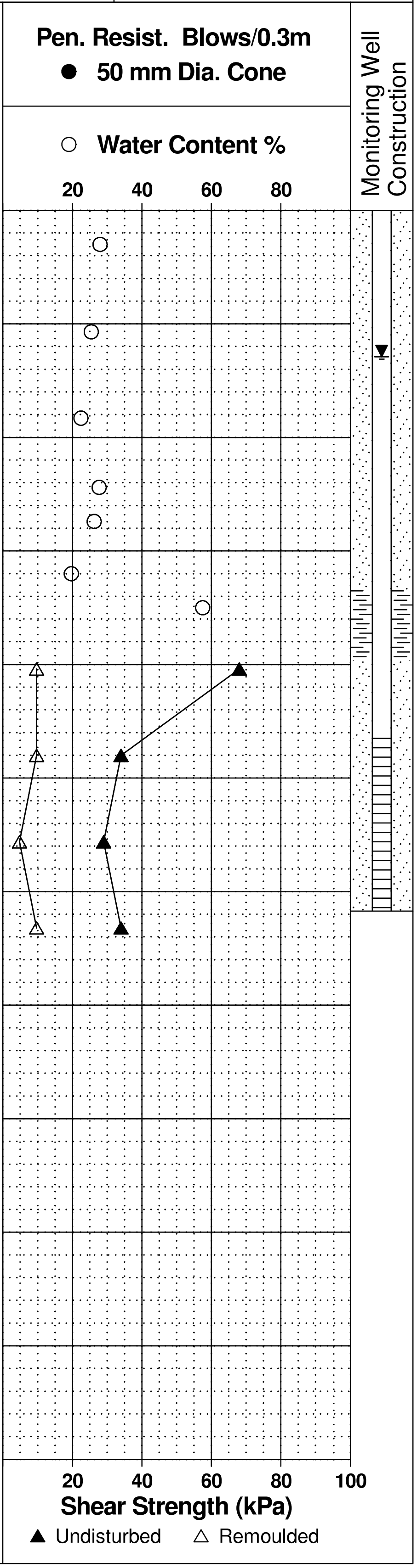
FILE NO. **PG6582**

HOLE NO. **BH 4-23**

SOIL DESCRIPTION	Depth to base (m)
GROUND SURFACE	
TOPSOIL	0.30
Loose to compact, reddish brown to brown **SILTY SAND**	2.74
Loose, grey **SILTY SAND** interbedded with silty clay layers	3.20
Stiff to firm, grey **SILTY CLAY**	6.55
Dynamic Cone Penetration Test commenced at 6.55m depth. Cone pushed to 34.7m depth.	

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
SS	1	83	3
SS	2	83	7
SS	3	100	16
SS	4	83	10
SS	5	100	4

DEPTH (m)	ELEV. (m)
0	86.40
1	85.40
2	84.40
3	83.40
4	82.40
5	81.40
6	80.40
7	79.40
8	78.40
9	77.40
10	76.40
11	75.40



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Development - 3317 Navan Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

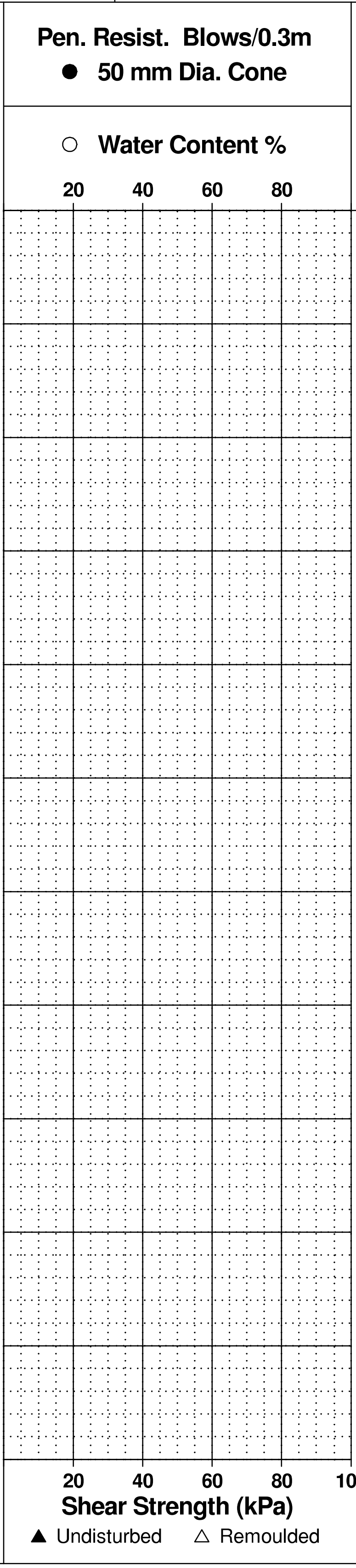
BORINGS BY Track-Mount Power Auger

DATE March 20, 2023

FILE NO. PG6582

HOLE NO. BH 4-23

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Piezometer Construction
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD			
Ground Surface								
Dynamic Cone Penetration Test commenced at 6.55m depth. Cone pushed to 34.7m depth.						11	75.40	
						12	74.40	
						13	73.40	
						14	72.40	
						15	71.40	
						16	70.40	
						17	69.40	
						18	68.40	
						19	67.40	
						20	66.40	
						21	65.40	
						22	64.40	



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Development - 3317 Navan Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

DATE March 20, 2023

FILE NO. PG6582

HOLE NO. BH 4-23

SOIL DESCRIPTION Ground Surface	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content % 20 40 60 80	Piezometer Construction
Dynamic Cone Penetration Test commenced at 6.55m depth. Cone pushed to 34.7m depth.						22	64.40		
						23	63.40		
						24	62.40		
						25	61.40		
						26	60.40		
						27	59.40		
						28	58.40		
						29	57.40		
						30	56.40		
						31	55.40		
						32	54.40		
						33	53.40		

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Development - 3317 Navan Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

DATE March 20, 2023

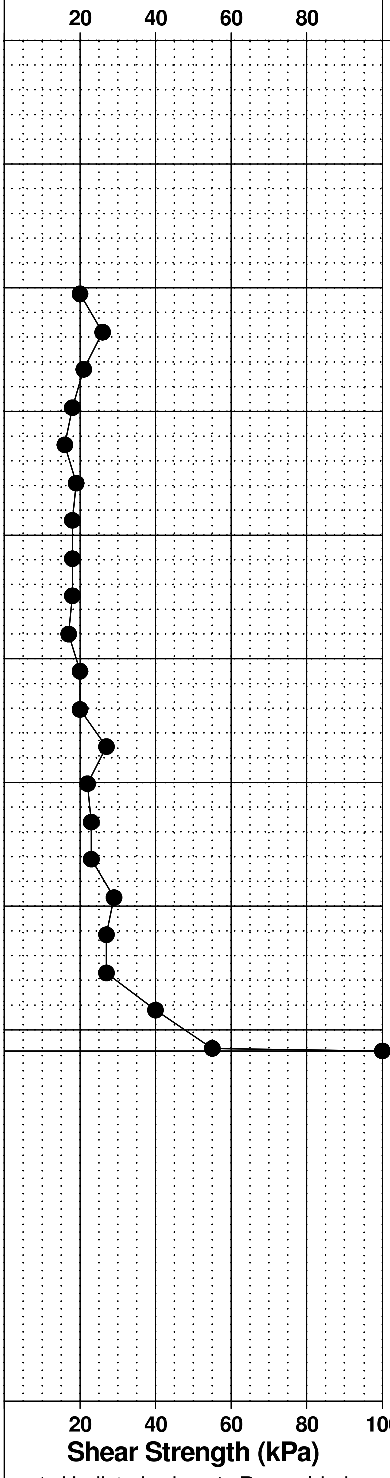
FILE NO. PG6582

HOLE NO. BH 4-23

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
Ground Surface							
Dynamic Cone Penetration Test commenced at 6.55m depth. Cone pushed to 34.7m depth.						33	53.40
						34	52.40
						35	51.40
						36	50.40
						37	49.40
						38	48.40
						39	47.40
						40	46.40
41.17						41	45.40
End of Borehole Practical DCPT refusal at 41.17m depth. (GWL @ 1.29m depth on March 28, 2023)							

Pen. Resist. Blows/0.3m
● 50 mm Dia. Cone

○ Water Content %



Shear Strength (kPa)
▲ Undisturbed △ Remoulded

Piezometer Construction

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Development - 3317 Navan Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

DATE March 20, 2023

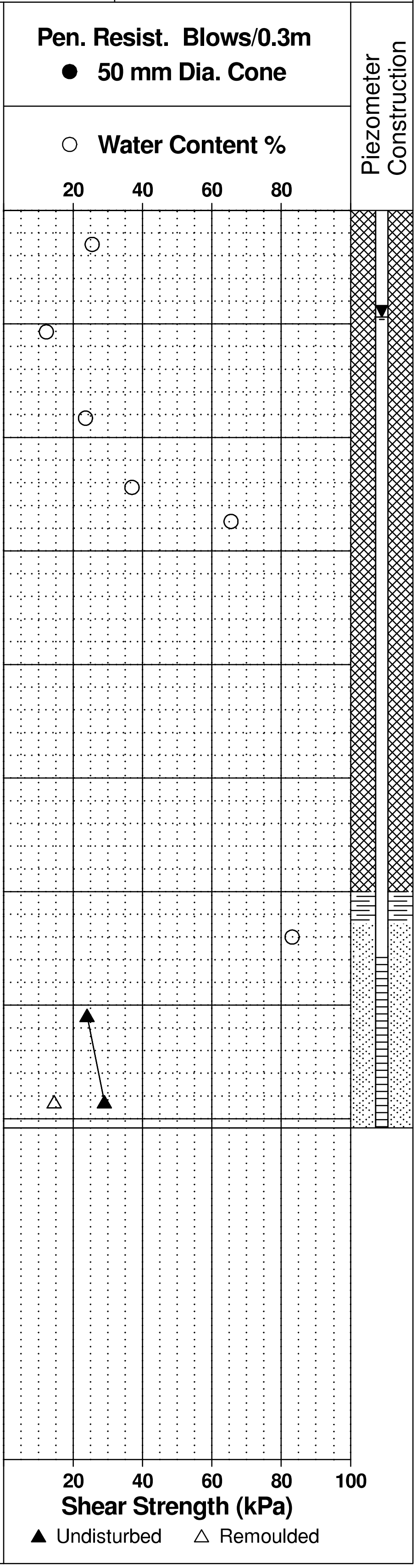
FILE NO. PG6582

HOLE NO. BH 5-23

SOIL DESCRIPTION	
Ground Surface	
FILL: Organics with silty sand	0.08
FILL: Brown silty sand with clay, gravel, trace organics and asphalt	0.91
Asphaltic concrete	1.45
FILL: Brown silty sand with clay, gravel, some asphalt	2.21
Very loose, brown **SILTY SAND**	2.59
Stiff, brown **SILTY CLAY**	2.97
Firm, grey **SILTY CLAY**	8.08
End of Borehole (GWL @ 0.94m depth on March 28, 2023)	

SAMPLE			
TYPE	NUMBER	% RECOVERY	N VALUE or RQD
SS	1	67	9
SS	2	75	35
SS	3	67	10
SS	4	83	3
SS	5	0	1
SS	6	0	1
SS	7	0	1
SS	8	0	1
SS	9	100	1

DEPTH (m)	ELEV. (m)
0	86.72
1	85.72
2	84.72
3	83.72
4	82.72
5	81.72
6	80.72
7	79.72
8	78.72



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Development - 3317 Navan Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

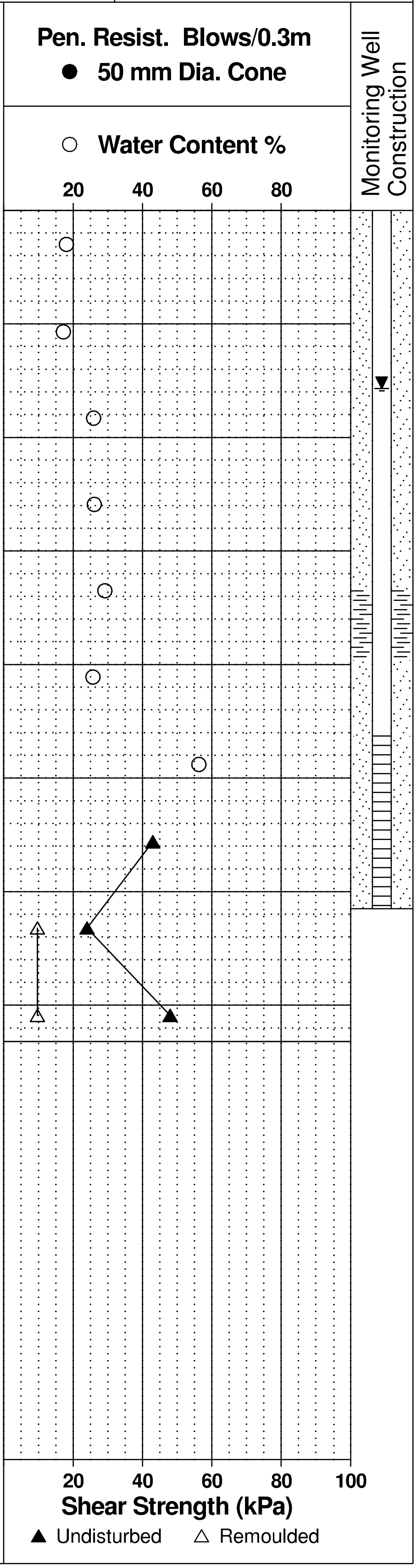
BORINGS BY Track-Mount Power Auger

DATE March 20, 2023

FILE NO. **PG6582**

HOLE NO. **BH 6-23**

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	86.86
FILL: Organics with silty sand 0.08							
FILL: Brown silty clay with sand, gravel, trace organics 0.81		SS	1	75	10		
		SS	2	67	16	1	85.86
		SS	3	75	11		
Compact, brown **SILTY SAND**						2	84.86
		SS	4	83	11		
						3	83.86
		SS	5	67	12		
- very loose by 3.8m depth						4	82.86
4.57		SS	6	67	1		
		SS	7	83	1		
						5	81.86
Firm, grey **SILTY CLAY**						6	80.86
						7	79.86
7.32							
End of Borehole (GWL @ 1.57m depth on March 28, 2023)							



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Dev.-Eastboro Phase 2-Navan Road
Ottawa, Ontario

DATUM Ground surface elevations provided by Annis, O'Sullivan, Vollebekk Limited.

FILE NO. **PG2444**

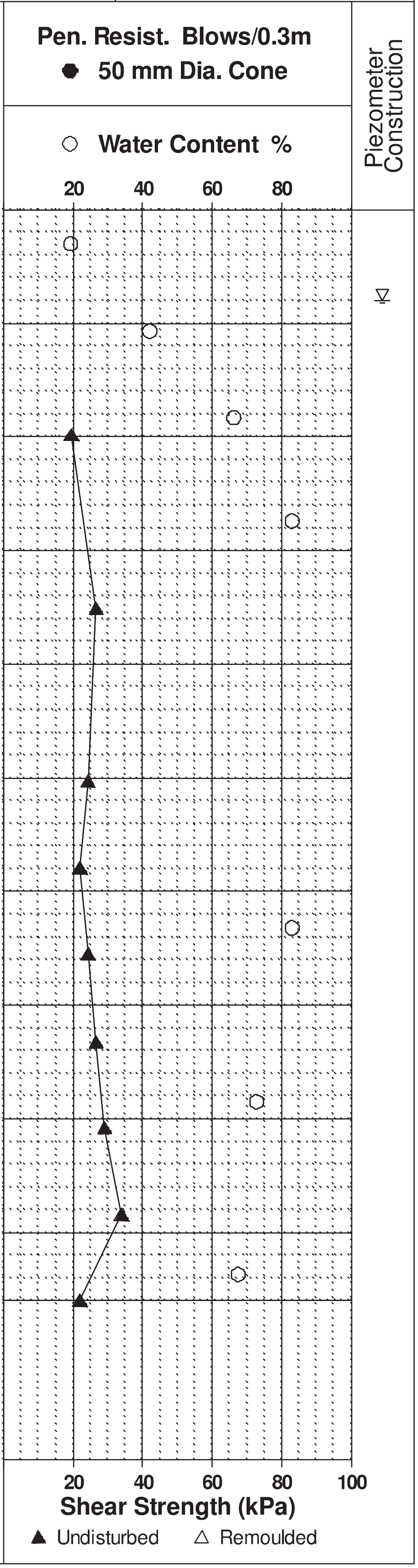
REMARKS

HOLE NO. **BH 1-14**

BORINGS BY CME 55 Power Auger

DATE June 17, 2014

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD		
GROUND SURFACE						0	87.74
Loose, brown **SILTY SAND**		AU	1				
		SS	2	67	4	1	86.74
1.37							
Soft to firm, brown **SILTY CLAY**						2	85.74
		TW	3	100			
- grey by 3.0m depth						3	84.74
		TW	4	100		4	83.74
						5	82.74
						6	81.74
						7	80.74
						8	79.74
						9	78.74
9.60							
End of Borehole (GWL @ 0.8m depth based on field observations)							



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9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Supplemental Geotechnical Investigation
Prop. Residential Development - Eastboro Phase 2
Navan Road, Ottawa, Ontario

DATUM Geodetic

REMARKS

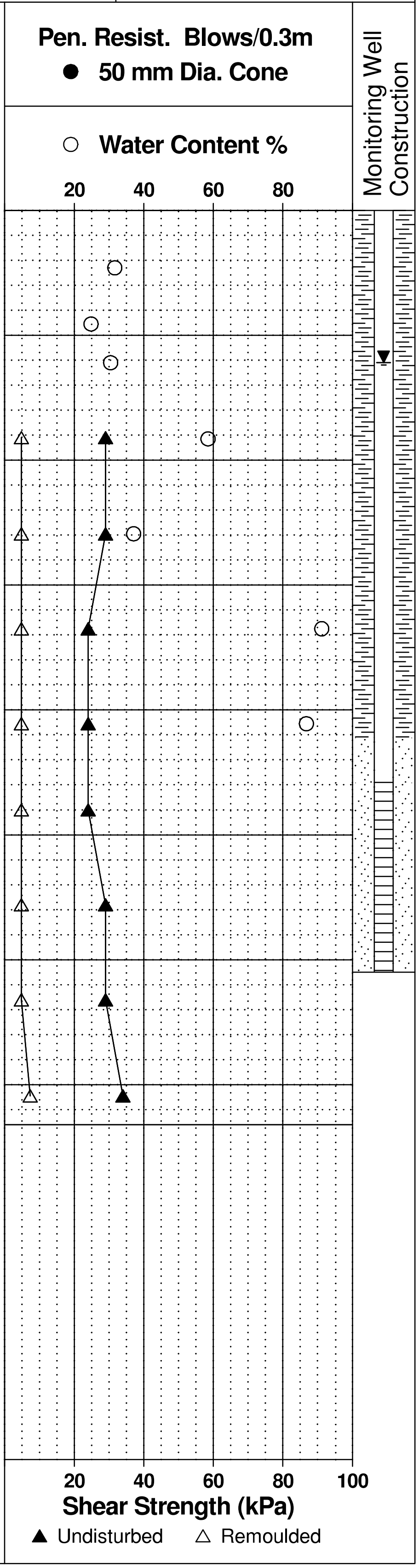
BORINGS BY Track-Mount Power Auger

DATE May 24, 2022

FILE NO. PG2444

HOLE NO. BH13-22

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	85.94
Loose to compact, brown **SILTY SAND**	AU	1				
- grey by 0.9m depth	SS	2	50	10	1	84.94
1.52						
Soft to firm, grey **SILTY CLAY**	SS	3	100		2	83.94
	SS	4	58			
					3	82.94
	SS	5	100			
	SS	6	100		4	81.94
					5	80.94
					6	79.94
					7	78.94
7.32						
End of Borehole (GWL @ 1.22m - June 17, 2022)						



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

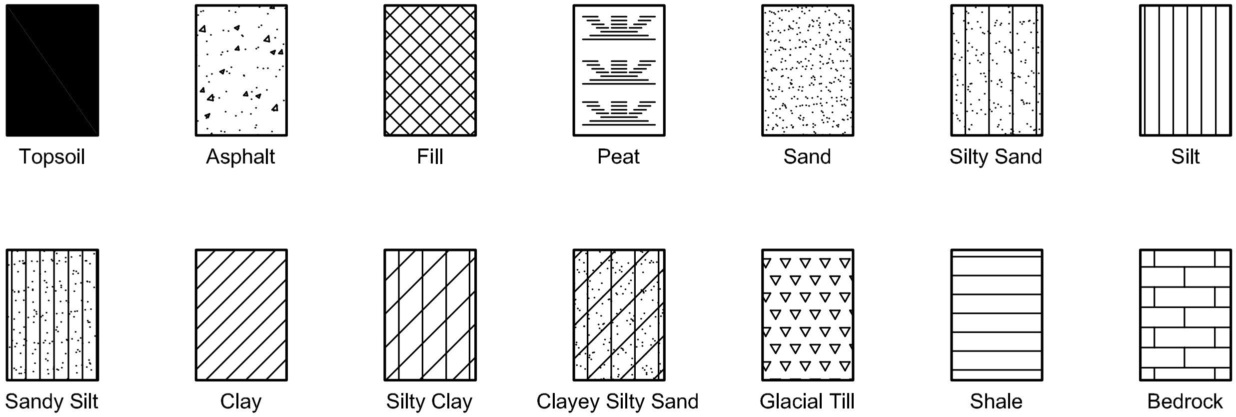
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void 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.

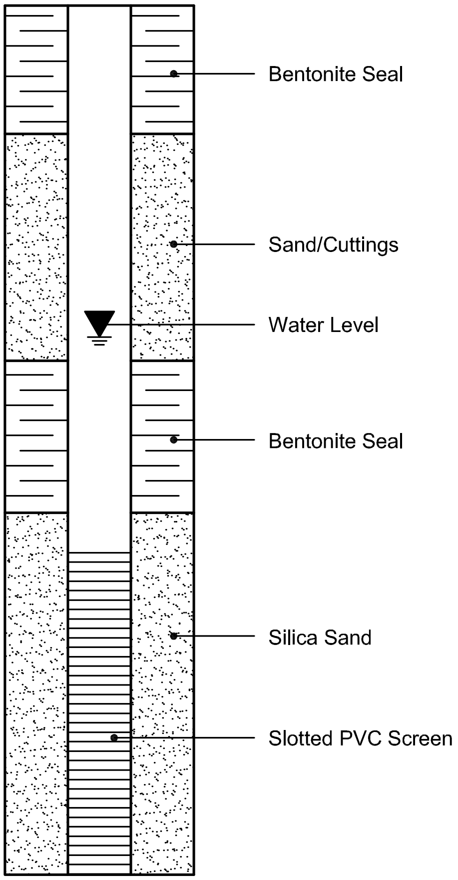
SYMBOLS AND TERMS (continued)

STRATA PLOT

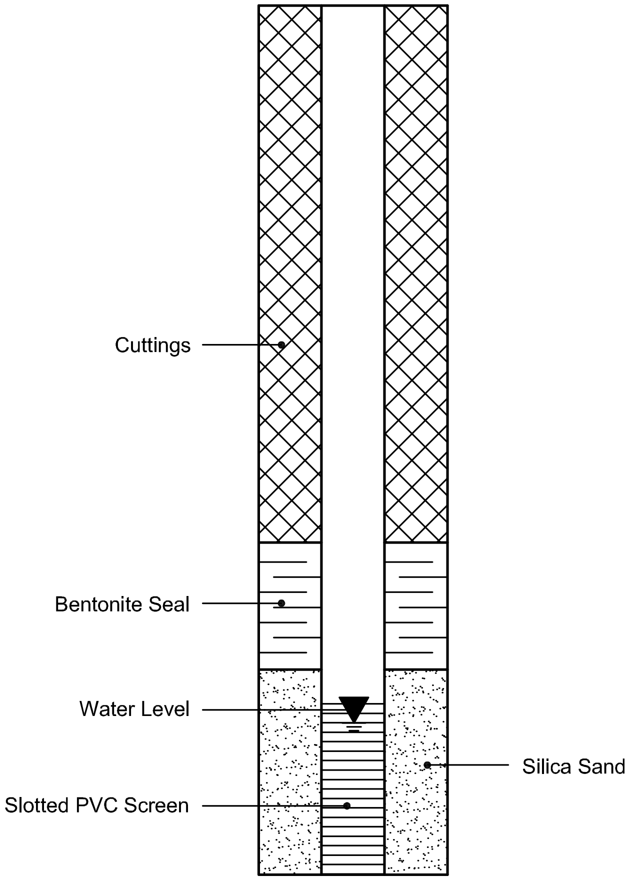


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



PROJECT: 07-1121-0129

RECORD OF BOREHOLE: 07-5

SHEET 1 OF 3

LOCATION: See Site Plan

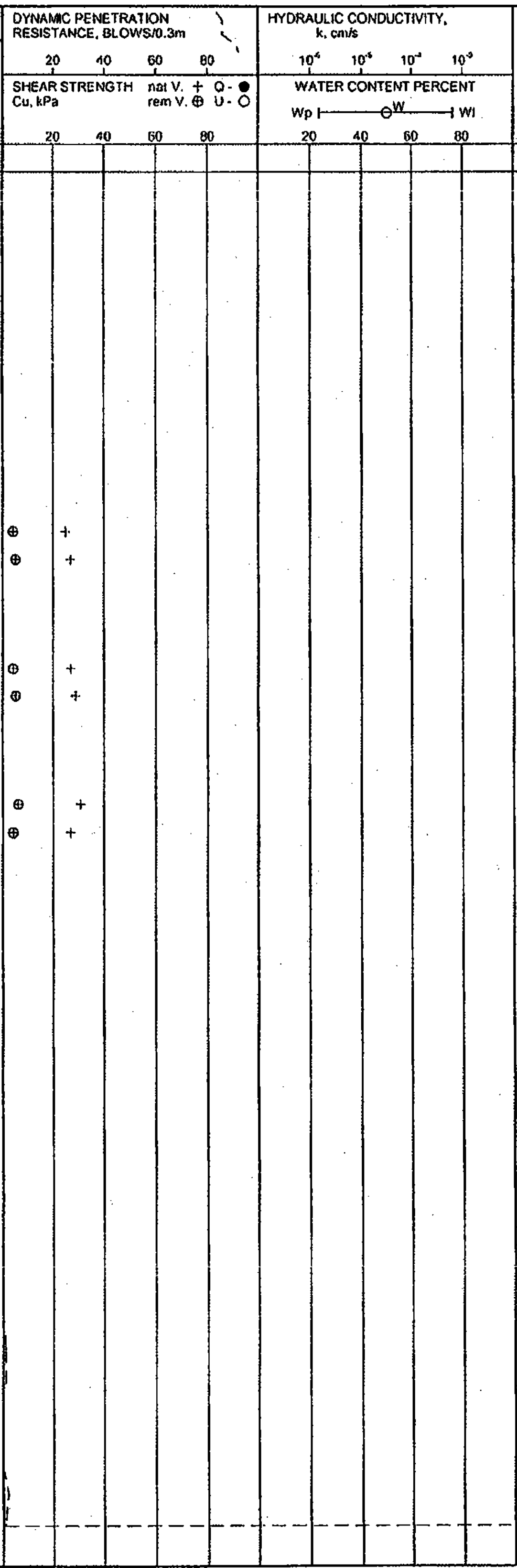
BORING DATE: July 24, 2007

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	ELEV. / DEPTH (m)	SAMPLES: NUMBER	TYPE	BLOWS/0.3m
		GROUND SURFACE	86.91			
0	Power Auger 200mm Diam. (Hollow Stem)	Brown and grey brown silty sand, trace organic matter and gravel (FILL)	0.00	1	A.S.	
		TOPSOIL	86.23 0.68			
1		Loose grey brown very fine SAND, occasional silt layers	0.81	2	50 DO	7
		Compact red orange to grey brown fine SAND, trace silt	85.69 1.22			
2				3	50 DO	14
				4	50 DO	13
3		Grey layered SILTY SAND, CLAYEY SILT and SILTY CLAY	84.01 2.90			
		Firm grey SILTY CLAY	83.56 3.35	5	50 DO	11
5				6	73 TP	PH
6				7	50 DO	PM
11	60° Cone Penetration	Probably grey Silty Clay	75.33 11.58			
15						



ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION

∇

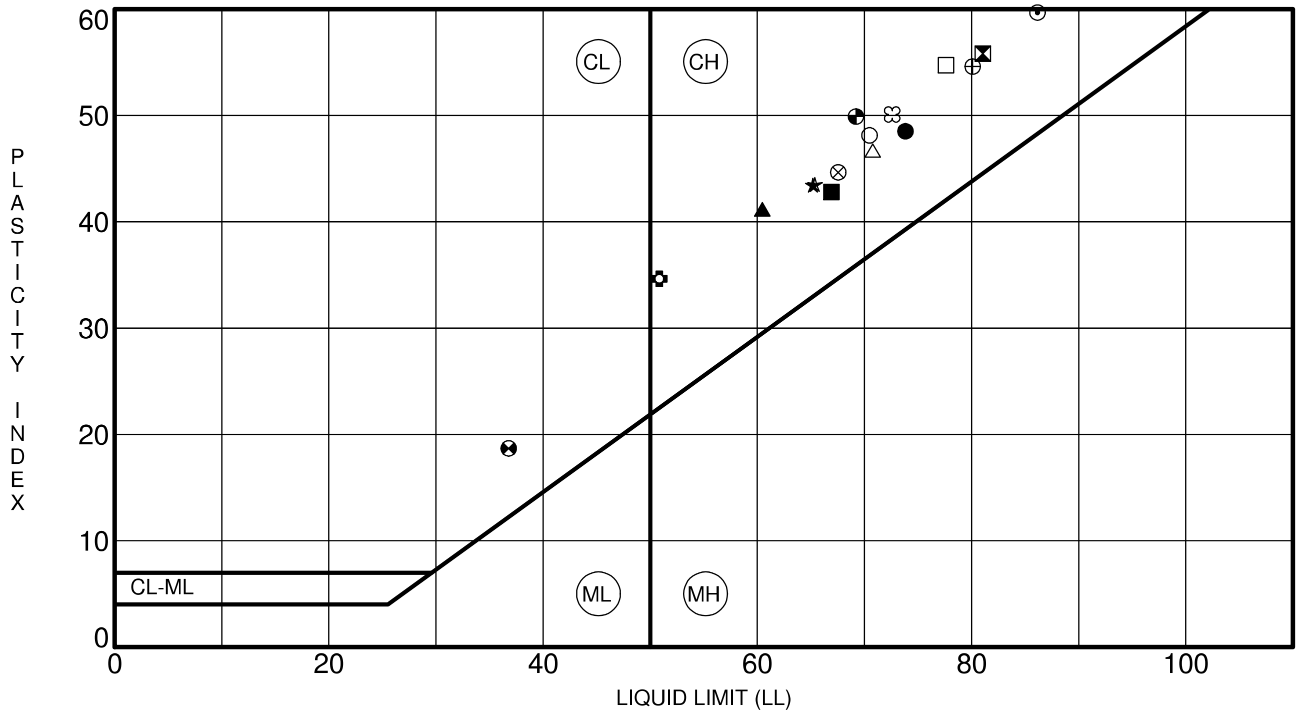
Water level in open hole at 1.83m depth below ground surface upon completion of drilling

CONTINUED NEXT PAGE

DEPTH SCALE
1 : 75

Golder Associates

LOGGED: P.A.H.
CHECKED: BDL



Specimen Identification			LL	PL	PI	Fines	Classification
●	BH 1-22	SS4	74	25	49		CH - Inorganic clays of high plasticity
⧗	BH 2-22	SS3	81	25	56		CH - Inorganic clays of high plasticity
▲	BH 3-22	SS2	60	19	41		CH - Inorganic clays of high plasticity
★	BH 4-22	SS4	65	22	43		CH - Inorganic clays of high plasticity
⊙	BH 5-22	SS5	86	26	60		CH - Inorganic clays of high plasticity
✿	BH 6-22	SS3	51	16	35		CH - Inorganic clays of high plasticity
○	BH 7-22	SS5	70	22	48		CH - Inorganic clays of high plasticity
△	BH 8-22	SS3	71	24	47		CH - Inorganic clays of high plasticity
⊗	BH 9-22	SS3	68	23	45		CH - Inorganic clays of high plasticity
⊕	BH10-22	SS3	80	25	55		CH - Inorganic clays of high plasticity
□	BH11-22	SS3	78	23	55		CH - Inorganic clays of high plasticity
◍	BH12-22	SS4	37	18	19		CL - Inorganic clays of low plasticity
◕	BH13-22	SS3	69	19	50		CH - Inorganic clays of high plasticity
☆	BH14-22	SS7	65	22	44		CH - Inorganic clays of high plasticity
✤	BH15-22	SS4	73	22	50		CH - Inorganic clays of high plasticity
■	BH16-22	SS2	67	24	43		CH - Inorganic clays of high plasticity

CLIENT **Ashcroft Homes**

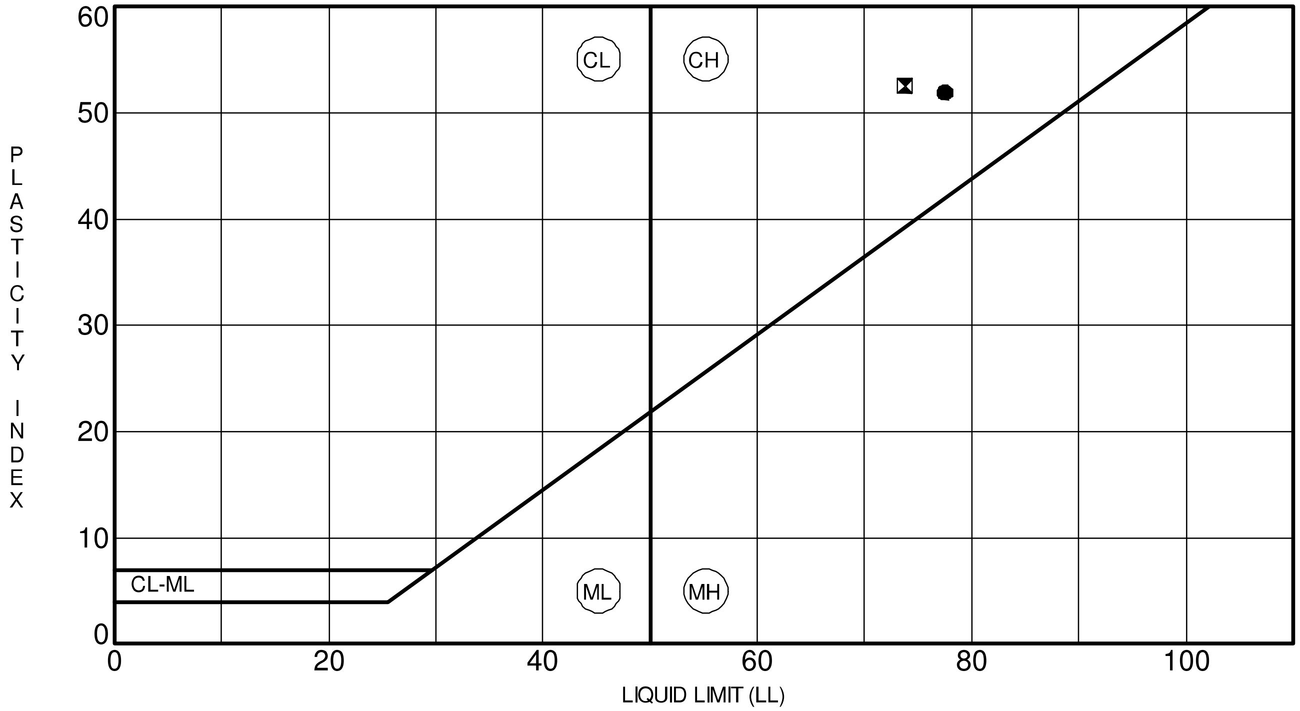
PROJECT **Supplemental Geotechnical Investigation - Prop.**
Residential Development - Eastboro Phase 2

FILE NO. **PG2444**

DATE **25 May 22**

patersongroup **Consulting Engineers**
154 Colonnade Road South, Ottawa, Ontario K2E 7J5

ATTERBERG LIMITS' RESULTS



	Specimen Identification		LL	PL	PI	Fines	Classification
●	**BH 1-14**	**TW 3**	**77**	**26**	**52**		**CH - High plasticity inorganic clay**
☒	**BH 4-14**	**TW 2**	**74**	**21**	**53**		**CH - High plasticity inorganic clay**

CLIENT **Ashcroft Homes**

FILE NO. **PG2444**

PROJECT **Geotechnical Investigation - Prop. Residential Dev.-Eastboro Phase 2-Navan Road**

DATE **13 Jun 14**

patersongroup **Consulting Engineers**

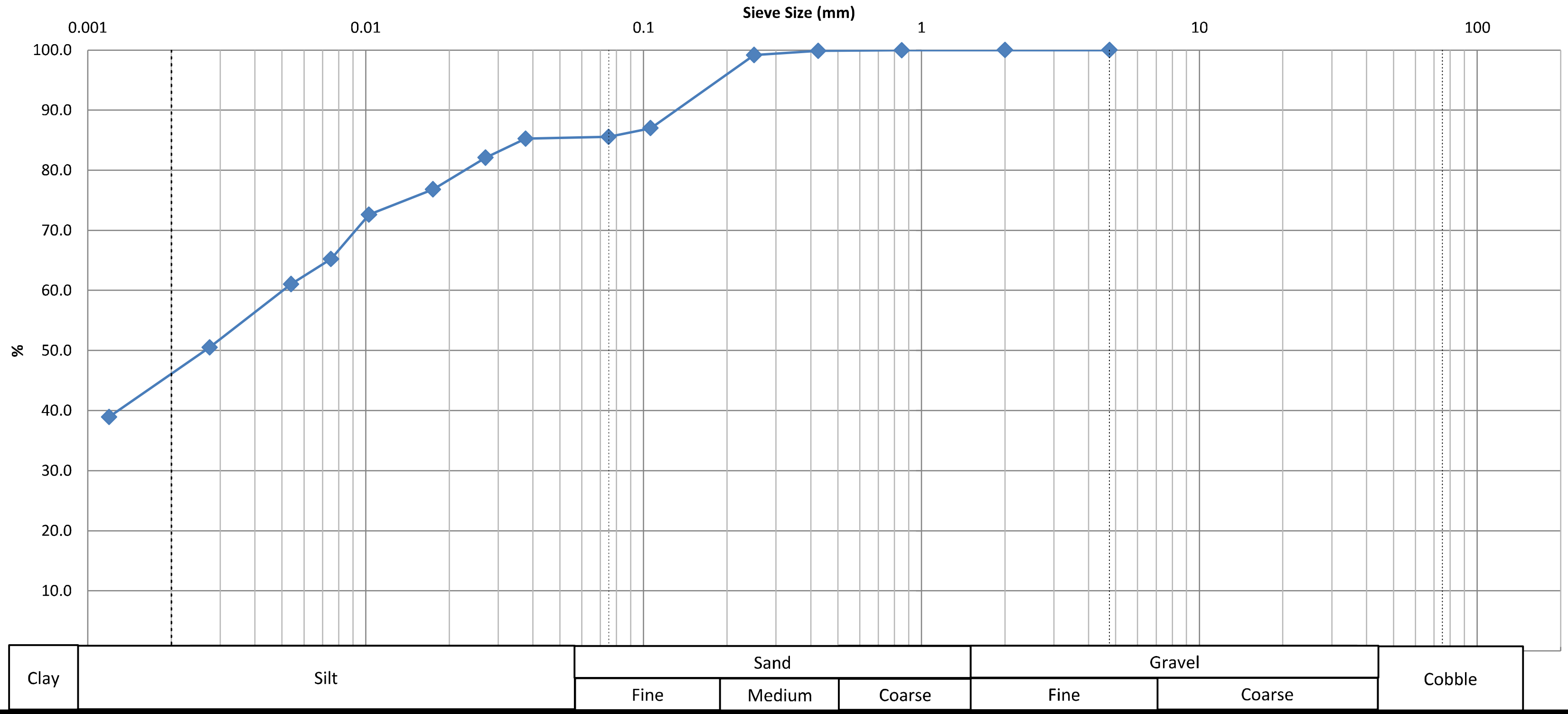
154 Colonnade Road South, Ottawa, Ontario K2E 7J5

ATTERBERG LIMITS' RESULTS

patersongroup
consulting engineers

SIEVE ANALYSIS
ASTM C136

CLIENT:	Ashcroft Homes	DEPTH:	7'-6" to 9'-6"	FILE NO:	PG2444
CONTRACT NO.:		BH OR TP No.:	BH13-22-PH2 SS4	LAB NO:	34115
PROJECT:	Eastboro - Phase 2			DATE RECEIVED:	30-May-22
				DATE TESTED:	31-May-22
DATE SAMPLED:	May 19-25			DATE REPORTED:	10-Jun-22
SAMPLED BY:	N.S			TESTED BY:	DK/CS



Identification	Soil Classification				MC(%)	LL	PL	PI	Cc	Cu
					46.6					
	D100	D60	D30	D10	Gravel (%)	Sand (%)		Silt (%)	Clay (%)	
					0.0	14.4		39.1	46.5	

Comments:

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

patersongroup
consulting engineers

HYDROMETER
LS-702 ASTM-422

CLIENT:	Ashcroft Homes	DEPTH:	7'-6" to 9'-6"	FILE NO.:	PG2444
PROJECT:	Eastboro - Phase 2	BH OR TP No.:	BH13-22-PH2 SS4	DATE SAMPLEI	May 19-25
LAB No. :	34115	TESTED BY:	DK/CS	DATE RECEIVE	30-May-22
SAMPLED BY:	N.S	DATE REPT'D:	10-Jun-22	DATE TESTED:	31-May-22

SAMPLE INFORMATION

SAMPLE MASS	SPECIFIC GRAVITY
114.6	2.700

INITIAL WEIGHT	50.00	**HYGROSCOPIC MOISTURE**		
WEIGHT CORRECTED	33.38	TARE WEIGHT	50.00	ACTUAL WEIGHT
WT. AFTER WASH BACK SIEVE	7.29	AIR DRY	127.90	77.90
SOLUTION CONCENTRATION	40 g/L	OVEN DRY	102.00	52.00
		CORRECTED	0.668	

GRAIN SIZE ANALYSIS

SIEVE DIAMETER (mm)	WEIGHT RETAINED (g)	PERCENT RETAINED	PERCENT PASSING
26.5			
19			
13.2			
9.5			
4.75	**0.00**	0.0	**100.0**
2.0	**0.00**	0.0	**100.0**
Pan	**102.00**		
0.850	**0.01**	0.0	**100.0**
0.425	**0.06**	0.1	**99.9**
0.250	**0.41**	0.8	**99.2**
0.106	**6.49**	13.0	**87.0**
0.075	**7.22**	14.4	**85.6**
Pan	**7.29**		

SIEVE CHECK	0.0	MAX = 0.3%	

HYDROMETER DATA

ELAPSED	TIME (24 hours)	Hs	Hc	Temp. (^oC)	DIAMETER	(P)	TOTAL PERCENT PASSING
1	8:30	46.5	6.0	23.0	0.0376	85.2	**85.2**
2	8:31	45.0	6.0	23.0	0.0270	82.1	**82.1**
5	8:34	42.5	6.0	23.0	0.0175	76.8	**76.8**
15	8:44	40.5	6.0	23.0	0.0103	72.6	**72.6**
30	8:59	37.0	6.0	23.0	0.0075	65.2	**65.2**
60	9:29	35.0	6.0	23.0	0.0054	61.0	**61.0**
250	12:39	30.0	6.0	23.0	0.0028	50.5	**50.5**
1440	8:29	24.5	6.0	23.0	0.0012	38.9	**38.9**

Moisture = 46.6%

REVIEWED BY:	**C. Beadow**	**Joe Forsyth, P. Eng.**

Certificate of Analysis
Client: **Paterson Group Consulting Engineers**
Client PO: 57056

Report Date: 24-Mar-2023
Order Date: 21-Mar-2023
Project Description: PG6582

	Client ID:	BH4-23-SS3 [5'-7']	-	-	-
	Sample Date:	20-Mar-23 09:00	-	-	-
	Sample ID:	2312239-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	81.1	-	-	-
General Inorganics					
pH	0.05 pH Units	6.78	-	-	-
Resistivity	0.1 Ohm.m	84.8	-	-	-
Anions					
Chloride	10 ug/g dry	<10	-	-	-
Sulphate	10 ug/g dry	48	-	-	-

APPENDIX 2

FIGURE 1 – KEY PLAN

DRAWING PG6582-1 – TEST HOLE LOCATION PLAN

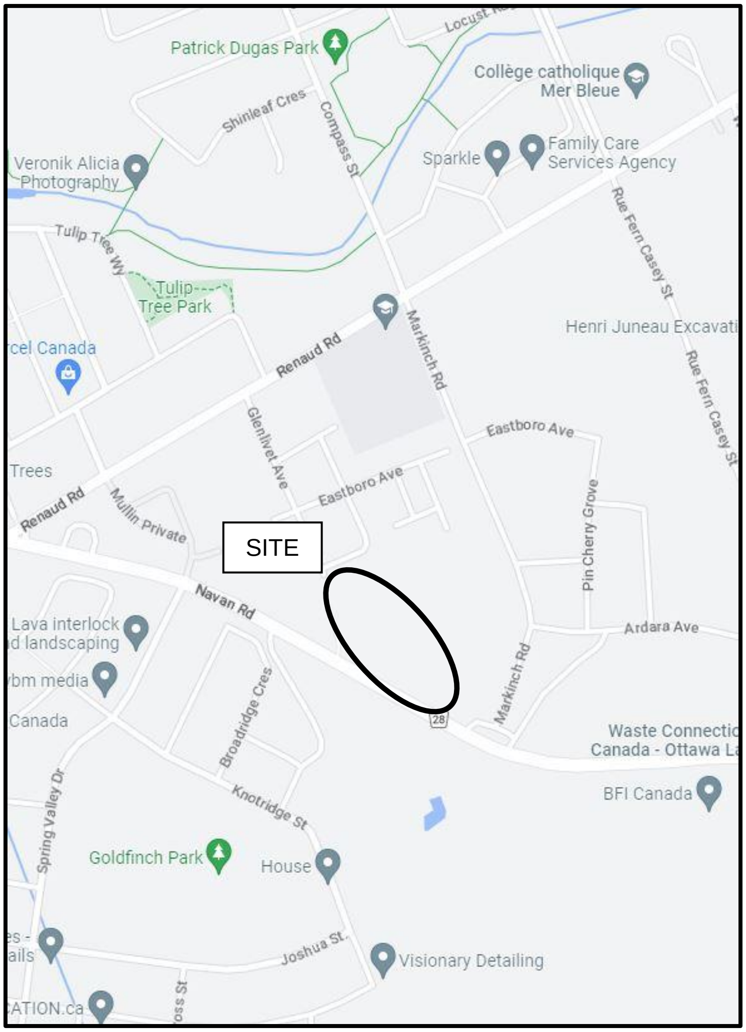
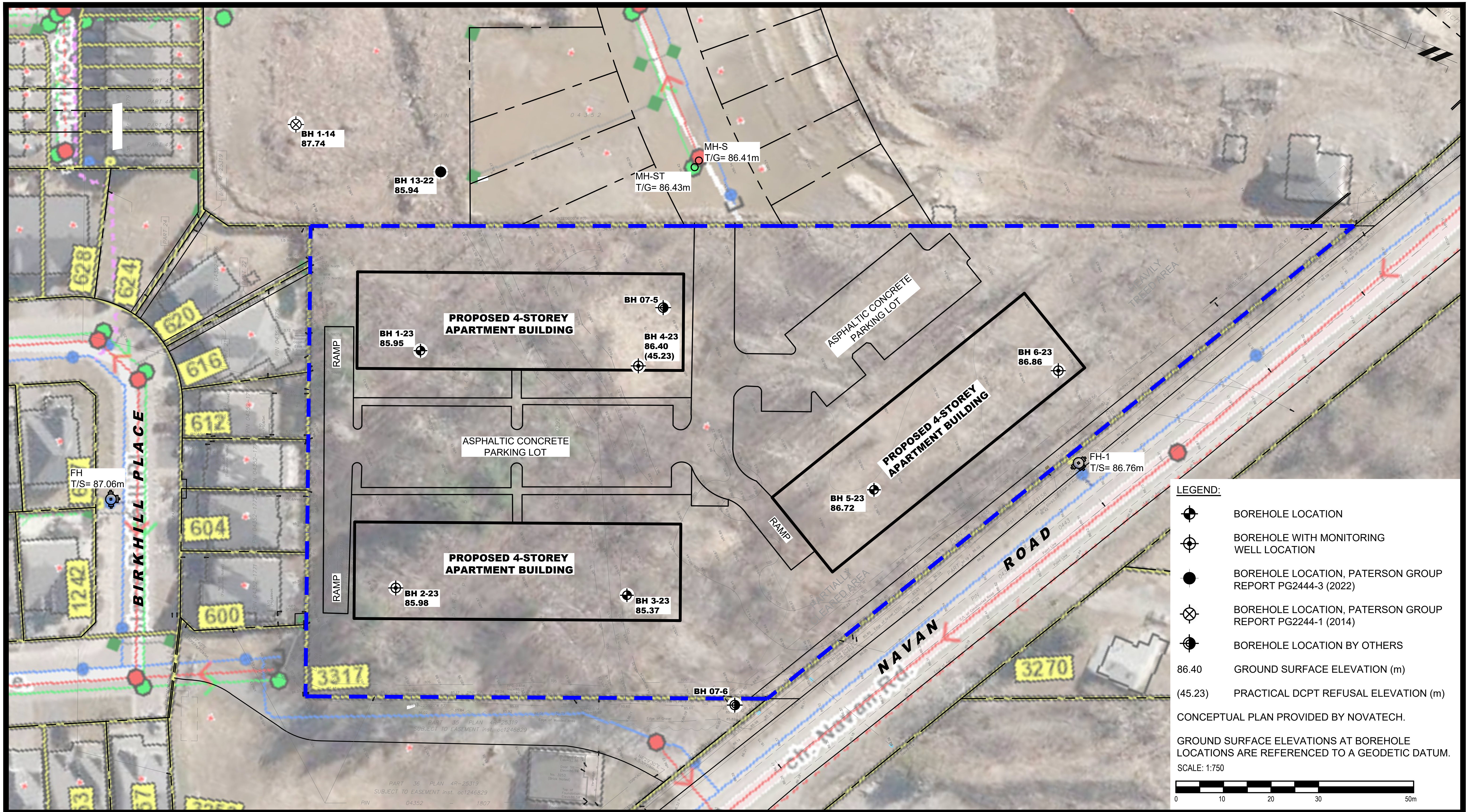


FIGURE 1

KEY PLAN

PATERSON GROUP



CONCEPTUAL PLAN PROVIDED BY NOVATECH.

GROUND SURFACE ELEVATIONS AT BOREHOLE LOCATIONS ARE REFERENCED TO A GEODETIC DATUM.

PATERSON GROUP

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

NO.	REVISIONS	DATE	INITIAL
1	ADJACENT BOREHOLES ADDED.	12/01/2024	BN

MANOR PARK MANAGEMENT c/o RENFROE LAND
GEOTECHNICAL INVESTIGATION
PROPOSED DEVELOPMENT
3317 NAVAN ROAD
OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:750	Date: 04/2023
Drawn by: YA	Report No.: PG6582-1
Checked by: BN	Dwg. No.: PG6582-1
Approved by: JV	Revision No.: 1

p:\autocad drawings\geotechnical\pg65xx\pg6582\pg6582-1-test hole location plan (rev.01).dwg

11x17