Geotechnical Investigation

Proposed Warehouse Complex

6165 Thunder Road
Ottawa, Ontario

Prepared for HP Urban Inc.

Report PG6430-1 dated February 22, 2023

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1.0 Introduction

Paterson Group (Paterson) was commissioned by HP Urban Inc. to conduct a geotechnical investigation for the proposed development to be located at 6165 Thunder Road in the City of Ottawa (refer to Figure 1 - Key Plan in Appendix 2 of this report).

The objective of the geotechnical investigation was 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.

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 available site plans, it is understood that the future development will generally consist of a warehouse building and three self-storage buildings of slab-on-grade construction. Associated parking areas, access lanes, accessory structures for storage and garbage, and landscaped margins are also anticipated for the development. It is further understood that the subject site will be serviced by private well and septic services.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the current investigation was carried out on October 25, 2022. At that time, 4 boreholes were advanced to a maximum depth of 7.6 m below the existing ground surface. The test hole locations were placed in a manner to provide general coverage of the subject site taking into consideration site features and underground utilities. The test hole locations for the current investigation are presented on Drawing PG6430-1 - Test Hole Location Plan included in Appendix 2.

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

Sampling and In Situ Testing

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.

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

It should be noted that overburden thickness evaluation by completing dynamic cone penetration testing (DCPT) was not completed within the subject site. The DCPT consists of driving a steel drill rod, equipped with a 50 mm diameter cone at its tip, using a 63.5 kg hammer falling from a height of 760 mm. The number of blows required to drive the cone into the soil is recorded for each 300 mm increment. Based on our experience and knowledge around the vicinity of the site, existing subsurface gas pockets are induced to venting while removing the steel drill rods upon completion of DCPT testing. Therefore, for safety reasons, a DCPT has not been completed at the subject site as part of this geotechnical investigation. Overburden thickness evaluation for the subject site was completed based on review of available geological mapping, as well as our knowledge and experience within the immediate vicinity of the subject site.

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

All boreholes were fitted with a flexible polyethylene standpipe to allow groundwater level monitoring subsequent to the completion of the field 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 subject site. The test hole locations and ground surface elevation at each test hole location were surveyed by Paterson using a high precision GPS and referenced to a geodetic datum. The location of the boreholes is presented on Drawing PG6430-1 - 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. A total of one (1) shrinkage test, one (1) grain size distribution analysis, and two (2) Atterberg limit tests were completed on selected soil samples. Moisture content testing was completed on all recovered soil samples. The results are presented in Subsection 4.2 and on Grain Size Distribution and Hydrometer Testing, and Atterberg Limit Results and Shrinkage Test Results presented in Appendix 1.

Sample Storage

All samples from the recent investigation will be stored in the laboratory for a period of one month after issuance of this report. The samples will then be discarded unless directed otherwise.

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 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 Subsection 6.7.

4.0 Observations

4.1 Surface Conditions

The majority of the subject site currently consists of grass covered, undeveloped land with some small scattered trees present. The north-west portion of the site was observed to be covered by cattails with standing water present at the time of the field investigation. Some relatively small fill piles were observed to be present at the site at the time of the field investigation.

The ground surface across the subject site is relatively flat with an approximate geodetic elevation of 76 to 77 m. The site was generally observed to be lower than the adjacent roadways by up to 1 m.

The subject site is bordered by a Highway 417 ramp to the north, and by Boundary Road and further by an Amazon Warehouse facility to the east. The site is directly bordered by Thunder Road to the south and west, and further by a gas station to the south, and further by undeveloped land to the west.

4.2 Subsurface Profile

Overburden

Generally, the subsurface soil profile encountered at the test hole locations consisted of a layer of fill underlain by a silty sand to sandy silt layer, followed by a deep deposit of silty clay. The fill material was generally observed to consist of brown silty sand with varying amounts of clay, gravel, cobbles, organics and concrete fragments. The fill layer was observed to extend to depths ranging between 0.7 to 1.2 m below the existing surface.

The silty sand to sandy silt layer generally consisted of compact, brown silty sand, with trace amounts of clay, gravel and organics. The silty sand layer was observed to extend up to a depth of 1.2 to 1.5 m below ground surface.

The silty clay deposit consisted of a stiff to firm brown silty clay crust underlain by a soft to firm grey silty clay. The weathered silty clay crust was observed in all boreholes, extending to depths ranging between 3.0 to 3.7 m below the ground surface.

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

Bedrock
Based on available geological mapping, the bedrock in the subject area consists of Paleozoic shale of the Carlsbad formation, with an overburden drift thickness ranging between 25 to 50 m. Practical refusal to DCPT testing on inferred bedrock has been previously encountered on nearby sites at geodetic elevations ranging between approximately 60 to 50 m.

It should be noted that subsurface gas pockets have previously been encountered within the upper fractured bedrock in the vicinity of the subject site.

4.3 Laboratory Testing

Atterberg Limit Tests

Atterberg limits testing, as well as associated moisture content testing, was completed on select silty clay samples where encountered. The results of the Atterberg limits test are presented in Table 1 and on the Atterberg Limits Results sheet in Appendix 1. The results of the moisture content test are presented on the Soil Profile and Test Data Sheet in Appendix 1. The tested silty clay samples classify as inorganic clays of high plasticity (CH) in accordance with the Unified Soil Classification System.

Table 1 - Atterberg Limits Results

Sample	Depth (m)	LL (%)	PL (%)	PI (%)	w (%)	Classification
BH 1-22	1.83	69	32	37	56.84	CH
BH 2-22	1.83	60	28	32	45.99	CH

Notes: LL: Liquid Limit; PL: Plastic Limit; PI: Plasticity Index; w: Water Content,
CH: Inorganic Clay of High Plasticity

Grain Size Distribution and Hydrometer Testing

Grain size distribution analysis was completed on one select recovered silty clay sample. The results of the grain size distribution analysis are presented in Table 2 and on the Grain Size Distribution sheets in Appendix 1.

Table 2 – Grain Size Distribution Results					
Sample	**Depth (m)**	**Gravel (%)**	**Sand (%)**	**Silt (%)**	**Clay (%)**
BH 3-22	1.83	0.4	1.8	31.3	66.5
Note: The ground surface elevation at each borehole location was surveyed using a handheld GPS using a geodetic datum.					

4.4 Groundwater

Groundwater levels were recorded at each piezometer location on November 2, 2022. The groundwater level readings are presented in the Soil Profile and Test Data sheets in Appendix 1. The measured groundwater levels are presented in Table 3 below.

Table 3 – Summary of Groundwater Levels				
Borehole Number	**Ground Surface Elevation (m)**	**Measured Groundwater Level**		**Date Recorded**
		Depth (m)	**Elevation (m)**	
BH 1-22	77.05	0.89	76.16	November 2, 2022
BH 2-22	77.01	7.53	69.48	November 2, 2022
BH 3-22	76.89	1.09	75.80	November 2, 2022
BH 4-22	76.71	5.07	71.64	November 2, 2022
Note: The ground surface elevation at each borehole location was surveyed using a handheld GPS using a geodetic datum.				

It should be noted that surface water can become trapped within a backfilled borehole that can lead to higher than typical groundwater level observations.

The long-term groundwater levels can also be estimated based on the observed colour, consistency, and moisture content of the recovered soil samples. Based on these observations, the long-term groundwater table can be expected at a depth of approximately 1 to 2 m below the existing ground surface. Groundwater levels are subject to seasonal fluctuations. Therefore, the groundwater levels could vary at the time of construction.

Shrinkage Testing

Linear shrinkage testing was completed on samples recovered from 1.5 m depth from boreholes BH 1-22. The shrinkage limit and shrinkage ratio of the tested silty clay sample (BH 1-22) were found to be 19.93% and 1.741, respectively.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is considered suitable for the proposed development. The proposed buildings can be founded on conventional shallow footings placed on an undisturbed, stiff to firm, brown silty clay crust or firm to soft grey silty clay provided that the design loads can be achieved based on the bearing resistance values provided. For buildings where design loads exceed the bearing resistance values, end bearing piles will be required to handle the design building loads. End bearing pile design capacities and uplift resistance values have been provided in Subsection 5.3. Bearing capacities for conventional shallow footings have been provided in Subsection 5.3 for any lightweight structure to be constructed at the subject site.

Due to the presence of a deep silty clay deposit, a permissible grade raise restriction will be applied for the subject site.

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

5.2 Site Grading and Preparation

Stripping Depth

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

Care should be taken not to disturb subgrade soils during site preparation activities. Disturbance of the subgrade may result in having to sub-excavate the disturbed material and the placement of additional suitable fill material.

Fill Placement

Fill used for grading beneath the proposed development should consist, unless otherwise specified, of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A, Granular B Type II. The imported fill material should be tested and approved prior to delivery. The fill should be placed in maximum 300 mm thick loose lifts and compacted by suitable compaction equipment. Fill placed beneath the building should be compacted to a minimum of 98% of the standard Proctor maximum dry density (SPMDD).

Non-specified existing fill along with site-excavated soil could be placed as general landscaping fill where settlement of the ground surface is of minor concern. These materials should be spread in lifts with a maximum thickness of 300 mm and compacted by the tracks of the spreading equipment to minimize voids. If these materials are to be used to build up the subgrade level for areas to be paved, they should be compacted in thin lifts to a minimum density of 95% of the SPMDD.

Non-specified existing fill and site-excavated soils are not suitable for use as backfill against foundation walls unless a composite drainage blanket connected to a perimeter drainage system is provided.

5.3 Foundation Design

Conventional Shallow Foundation – Bearing Resistance Values

Strip footings, up to 3 m wide, and pad footings, up to 5 m wide, placed over an undisturbed, stiff to firm brown silty clay bearing surface can be designed using a bearing resistance value at serviceability limit states (SLS) of **120 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **180 kPa** incorporating a geotechnical factor of 0.5 for ULS..

Strip footings, up to 3 m wide, and pad footings, up to 5 m wide, placed over an undisturbed, firm to soft grey silty clay bearing surface can be designed using a bearing resistance value at serviceability limit states (SLS) of **60 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **90 kPa** incorporating a geotechnical factor of 0.5 for ULS..

Footings placed on undisturbed brown silty sand to sandy silt, can be designed using a bearing resistance value at SLS of **120 kPa** and a factored bearing resistance value at ULS of **180 kPa** incorporating a geotechnical factor of 0.5 for ULS. If the silty sand layer is found to be in a loose state of compaction, the contractor is recommended to proof roll the subgrade under dry conditions and above freezing temperatures and approved by Paterson at the time of construction.

The bearing resistance values are provided on the assumption that the footings will be placed on an undisturbed soil bearing surfaces. An undisturbed soil bearing surface consists of one from which all topsoil and deleterious materials, such as loose, frozen, or disturbed soil, have been removed, 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 stiff silty clay above the groundwater table when a plane extending down and out from the bottom edges of the footing, at a minimum of 1.5H:1V, passes only through in situ soil of the same or higher capacity as that of the bearing medium.

Settlement

The total and differential settlements will be dependent on characteristics of the proposed buildings. For design purposes, the total and differential settlements are estimated to be 25 and 20 mm, respectively. A post-development groundwater lowering of 0.5 m was assumed.

The potential post construction total and differential settlements are dependent on the position of the long-term groundwater level when buildings are situated over deposits of compressible silty clay. Efforts can be made to reduce the impacts of the proposed development on the long-term groundwater level by placing clay dykes in the service trenches, reducing the size of paved areas, leaving green spaces to allow for groundwater recharge or limiting planting of trees to areas away from the buildings. However, it is not economically possible to control the groundwater level.

5.4 Permissible Grade Raise Recommendations

Based on the undrained shear strength values of the silty clay deposit encountered throughout the subject site, a permissible grade raise restriction of **0.7 m** above existing ground surface is applicable for the subject site.

A post-development groundwater lowering of 0.5 m was considered in our permissible grade raise restriction calculations. If higher than permissible grade raises are required, preloading with or without a surcharge, lightweight fill and/or other measures should be investigated to reduce the risks of unacceptable long-term post construction total and differential settlements.

The total and differential settlements will be dependent on characteristics of the proposed buildings. For design purposes, the total and differential settlements are estimated to be 25 and 20 mm, respectively. A post-development groundwater lowering of 0.5 m was assumed for our calculations.

To reduce potential long term liabilities, consideration should be given to provide means to reduce long term groundwater lowering (e.g. clay dykes, restriction on planting around the structures, etc.).

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. Provided sufficient time is available to induce the required settlements, consideration could be given to surcharging the subject site.

5.5 Design for Earthquakes

The site class for seismic site response can be taken as **Class E** for design of the proposed buildings at the subject site in accordance with the OBC 2012. The soils underlying the subject site are not susceptible to liquefaction. Reference should be made to the latest revision of the 2012 Ontario Building Code for a full discussion of the earthquake design requirements.

5.6 Slab on Grade Construction

With the removal of all topsoil and fill, containing deleterious or organic materials, the native soil will be considered an acceptable subgrade surface on which to commence backfilling for slab-on-grade construction. It is recommended that the upper 200 mm of sub-slab fill should consist of an OPSS Granular A crushed stone. All backfill material within the footprint of the proposed building should be placed in maximum 300 mm thick loose lifts and compacted to at least 98% of its SPMDD.

Any soft areas should be removed and backfilled with appropriate backfill material. OPSS Granular A or Granular B Type II, with a maximum particle size of 50 mm, or approved granular alternative material are recommended for backfilling below the floor slab.

Passive Ventilation System

It is recommended that a passive ventilation system be placed beneath the floor slab(s) to ventilate any potential subsurface emanating gases. A vapor barrier and a poly liner should also be considered to block any has egress into the confines of the buildings.

5.7 Pavement Design

Minimum Pavement Structure Recommendations

Car only parking areas, heavy truck parking areas and access lanes are anticipated at this site. The proposed pavement structures are presented in Tables 4 and 5.

Table 4 - Recommended Pavement Structure - Car Only Parking Areas

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

Table 5 - Recommended Pavement Structure - Access Lanes and Heavy Truck Parking Areas

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

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

If soft spots develop in the subgrade during compaction or due to construction traffic, the affected areas should be excavated and replaced with OPSS Granular B Type II material. The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 100% of the material's SPMDD using suitable vibratory equipment.

Pavement Structure Drainage

The pavement structure performance is dependent on the moisture condition at the contact zone between the subgrade material and granular base. Failure to provide adequate drainage under conditions of heavy wheel loading could result in the subgrade fines being pumped into the stone subbase voids, thereby reducing the load bearing capacity.

Due to the low permeability of the subgrade materials consideration should be provided to installing subdrains during the pavement construction as per City of Ottawa standards. The subdrains should be provided for catch basins and extend at least 3 m in four orthogonal directions. The clear crushed stone surrounding the drainage lines should be wrapped with suitable filter cloth. The subdrain inverts should be approximately 300 mm below subgrade level. The subgrade surface should be shaped to promote water flow to the drainage lines.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Foundation Drainage

It is recommended that a perimeter foundation drainage system be designed for the future development. The system should consist of a 150 mm diameter, geotextile-wrapped, 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.

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 composite drainage system, such as Delta Drain 6000 or an approved equivalent. Imported granular materials, such as clean sand or OPSS Granular B Type I granular material, should otherwise be used for this purpose.

Concrete Sidewalks Adjacent to Buildings

To avoid differential settlements within the proposed sidewalks adjacent to the proposed buildings, it is recommended that the upper 600 mm of backfill placed below the concrete sidewalks adjacent to the building footprints to consist of non-frost susceptible material such as OPSS Granular A or Granular B Type II. The granular material should be placed in maximum 300 mm loose lifts and compacted to a minimum of 98% of the material's SPMDD using suitable compaction equipment. The subgrade material should be shaped to promote positive drainage towards the buildings perimeter drainage system. Consideration should be given to placing a layer of rigid insulation below the granular fill layer, however, should be detailed by the geotechnical consultant once design drawings are finalized if considered.

6.2 Protection Against Frost Action

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

Other exterior unheated footings, such as those for isolated exterior piers, wing walls and/or loading decks are more prone to deleterious movement associated with frost action than the exterior walls of the proper structure. These footings should be provided with a minimum 2.1 m thick soil cover (or insulation equivalent). It is recommended that Paterson review the proposed frost protection detail for any of the above-noted ancillary or unheated structures.

6.3 Excavation Side Slopes

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

Unsupported Side Slopes

The excavation side slopes above the groundwater level extending to a maximum depth of 3 m should be cut back at 1H:1V or flatter. The flatter slope is required for excavation below groundwater level. The subsoil at this site is considered to be mainly a Type 2 and 3 soil according to the Occupational Health and Safety Act and Regulations for Construction Projects.

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

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

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

It should be noted that based on our observations at the time of the field investigation, specifically at the locations of boreholes BH 26-22-PH18 and BH 30-22-PH18 within the glacial till layer containing silty sand, the soil was observed to be very sensitive when wet. Upon disturbance the soil was observed to be prone to running below the water table and should be cut back at 1.5H:1V or flatter. As a result, a dewatering program may be required as part of the deep service installation within the subject site. The dewatering program should consist of a series of well points designed and installed by a licensed contractor specializing in dewatering.

Excavation Base Stability

The base of supported excavations can fail by three 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

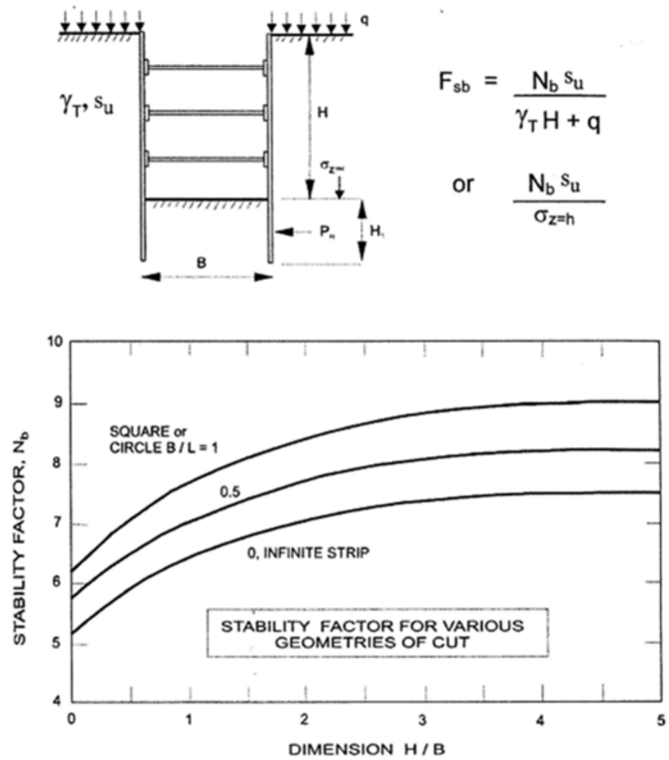

Figure 1 – Stability Factor for Various Geometries of Cut

A safety factor of 2 is generally recommended to be considered for excavation base stability.

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.

At least 150 mm of OPSS Granular A should be used for pipe bedding for sewer and water pipes. The bedding should extend to the spring line of the pipe. Cover material, from the spring line to at least 300 mm above the obvert of the pipe, should consist of OPSS Granular A or Granular B Type II with a maximum size of 25 mm. The bedding and cover materials should be placed in maximum 225 mm thick lifts compacted to 99% of the material's standard Proctor maximum dry density.

It should generally be possible to re-use the upper portion of the dry to moist (not wet) silty clay above the cover material if the excavation and filling operations are carried out in dry weather conditions.

Where hard surfaces are considered above the trench backfill, the trench backfill material within the frost zone (about 1.8 m below finished grade) should match the soils exposed at the trench walls to reduce potential differential frost heaving. The backfill should be placed in maximum 225 mm thick loose lifts and compacted to a minimum of 95% of the material's SPMDD.

For areas where rigid insulation will be used to provide frost protection. It is recommended that the rigid insulation be placed at the pipe obvert to allow for the maximum amount of granular cover over the pipe. Having the insulation at the obvert will provide a more effective insulation detail.

Clay Seals

To reduce long-term lowering of the groundwater level at this site, clay seals should be provided in the service trenches. The seals should be at least 1.5 m long and should extend from trench wall to trench wall. Generally, the seals should extend from the frost line and fully penetrate the bedding, sub-bedding and cover material. The barriers should consist of relatively dry and 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.

6.5 Groundwater Control

Due to the relatively impervious nature of the silty clay materials, it is anticipated that groundwater infiltration into the excavations should be low to moderate and controllable using open sumps. Pumping from open sumps should be sufficient to control the groundwater influx through the sides of shallow excavations. The contractor should be prepared to direct water away from all bearing surfaces and subgrades, regardless of the source, to prevent disturbances to the founding medium.

Permit to Take Water

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

For typical ground or surface water volumes being pumped during the construction phase, typically between 50,000 to 400,000 L/day, it is required to register on the Environmental Activity and Sector Registry (EASR). A minimum of two to four weeks should be allotted for completion of the EASR registration and the Water Taking and Discharge Plan to be prepared by a Qualified Person as stipulated under O.Reg. 63/16. 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.

6.6 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 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.

6.7 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

Tree Planting Considerations

In accordance with the City of Ottawa Tree Planting in Sensitive Marine Clay Soils (2017 Guidelines), Paterson completed a soils review of the site to determine applicable tree planting setbacks. Atterberg limits testing was completed for the recovered silty clay samples at selected locations throughout the subject site. The soil samples were recovered from elevations below the anticipated design underside of footing elevation and 3.5 m depth below anticipated finished grade. The results of our testing are presented in Table 1 in Subsection 4.2 and in Appendix 1.

Based on the results of the Atterberg limit testing mentioned above, the plasticity index was found to be less than 40% in all the tested clay samples. In addition, based on the clay content found in the clay samples from the grain size distribution test results, moisture levels and consistency, the silty clay across the subject site is considered low to medium sensitivity clay and cannot be designated as sensitive marine clays.

The following tree planting setbacks are recommended for the low to medium sensitivity silty clay deposit throughout the subject site.

- ❑ Large trees (mature height over 14 m) can be planted within these areas provided that a tree to foundation setback equal to the full mature height of the tree can be provided.

- ❑ Tree planting setback limits may be reduced to **4.5 m** 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 conditions noted below are met.

- ❑ 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 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.

7.0 Recommendations

The following is recommended to be completed once the preliminary site plan and site development are determined:

- ❑ Review detailed grading plan(s) from a geotechnical perspective, once available.
- ❑ Review of architectural and structural drawings to ensure adequate frost protection is provided to the subsoil.
- ❑ Review underfloor passive ventilation system design, once available, and installation during construction
- ❑ Observation of all bearing surfaces prior to the placement of concrete.
- ❑ Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.
- ❑ Observation of all subgrades prior to backfilling.
- ❑ Field density tests to determine the level of compaction achieved.
- ❑ Sampling and testing of the bituminous concrete including mix design reviews.

All excess soils must 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 the completion of a satisfactory inspection program by the geotechnical consultant.

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 HP Urban Inc. 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.

Nicole R.L. Patey, B.Eng.

Faisal I. Abou-Seido, P.Eng.

Report Distribution:

- ❑ HP Urban Inc. (1 digital copy)
- ❑ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ATTERBERG LIMITS RESULTS

GRAIN SIZE ANALYSIS RESULTS

SHRINKAGE ANALYSIS RESULTS

ANALYTICAL TESTING RESULTS

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9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Warehouse Complex - 6165 Thunder Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

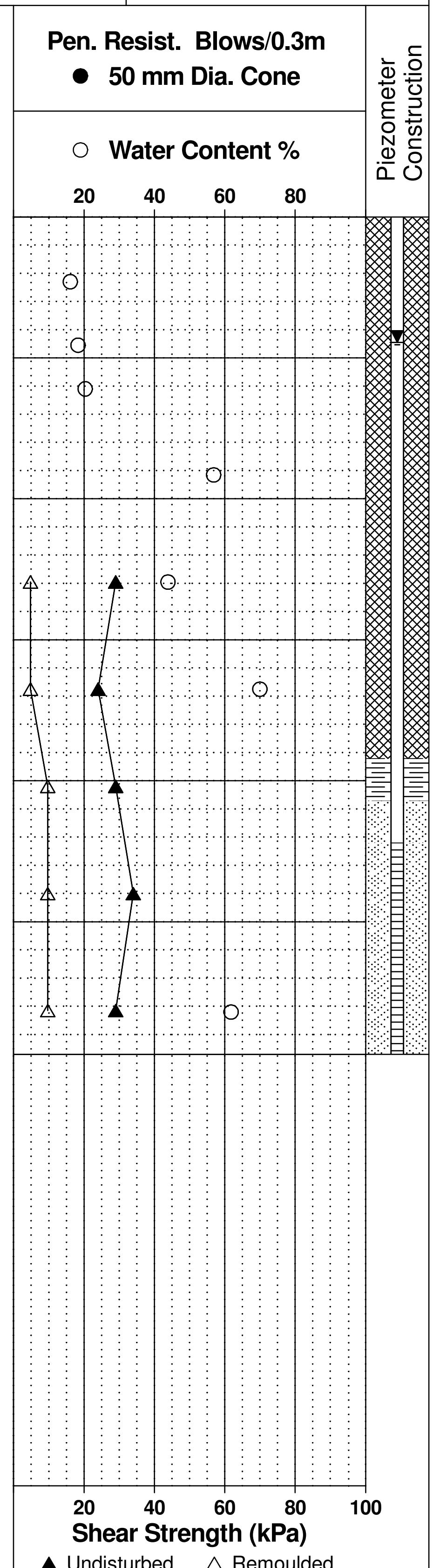
BORINGS BY Track-Mount Power Auger

DATE October 25, 2022

FILE NO. PG6430

HOLE NO. BH 1-22

SOIL DESCRIPTION	STRATA DEPTH	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	77.05
FILL: Brown silty sand with gravel, cobbles, trace clay and organics		AU	1				
		SS	2	67	6	1	76.05
	1.17						
Loose, dark brown **SILTY SAND,** trace clay and gravel	1.45						
		SS	3	83	3	2	75.05
Firm, reddish brown **SILTY CLAY**		SS	4	83	P		
						3	74.05
- grey by 3.0m depth		SS	5	75	P		
						4	73.05
						5	72.05
		SS	6	100	P		
	5.94						
End of Borehole							
(GWL @ 0.89m - Nov. 2, 2022)							



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Warehouse Complex - 6165 Thunder Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

DATE October 25, 2022

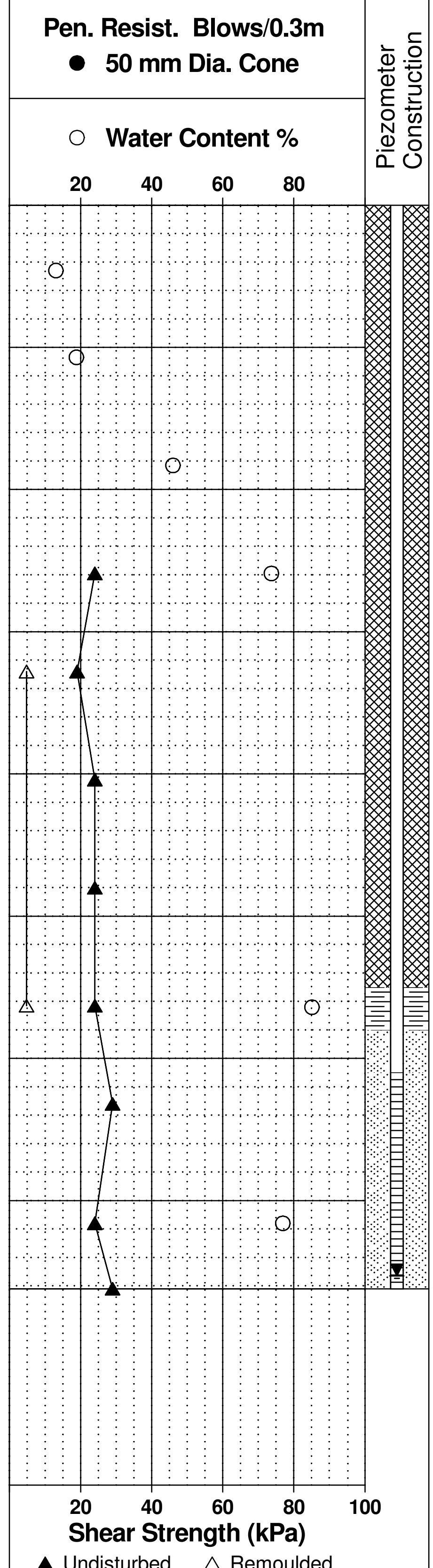
FILE NO. PG6430

HOLE NO. BH 2-22

SOIL DESCRIPTION	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE				
FILL: Brown silty sand with gravel, trace clay and organics (to 0.69)	AU	1		
Compact, brown **SILTY SAND,** trace clay, gravel, organics (0.69 – 1.45)	SS	2	50	12
Soft to firm, brown **SILTY CLAY** (1.45 – 7.62)	SS	3	100	2
	SS	4	100	P
- grey by 3.0m depth	SS	5	100	P
	SS	6	100	P
End of Borehole (7.62)				

(GWL @ 7.53m - Nov. 2, 2022)

DEPTH (m)	ELEV. (m)
0	77.01
1	76.01
2	75.01
3	74.01
4	73.01
5	72.01
6	71.01
7	70.01



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Warehouse Complex - 6165 Thunder Road
Ottawa, Ontario

DATUM Geodetic

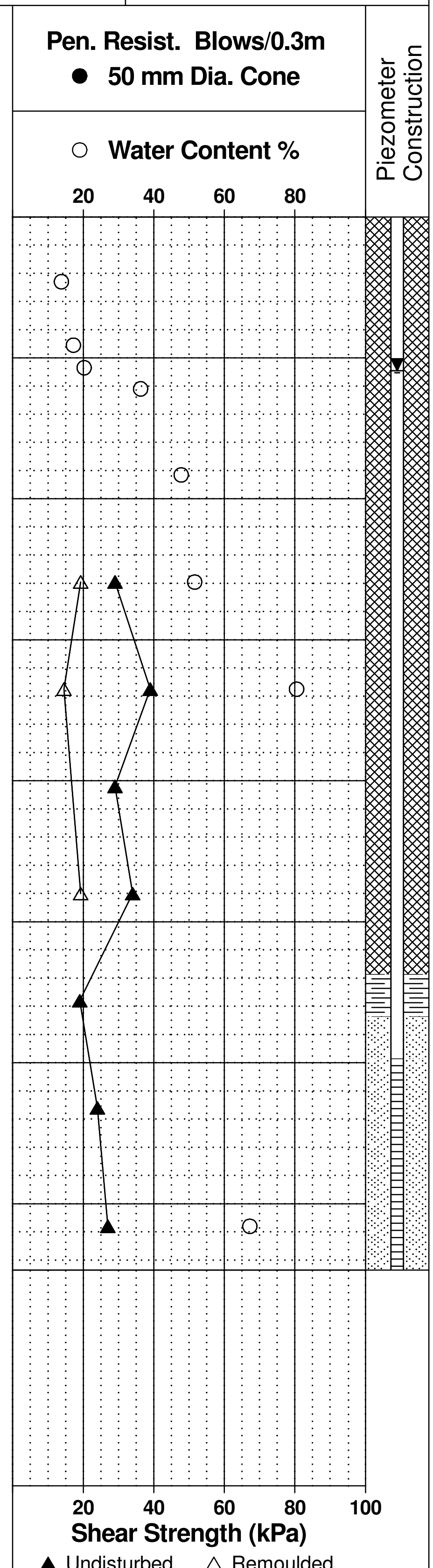
REMARKS

BORINGS BY Track-Mount Power Auger **DATE** October 25, 2022

FILE NO. PG6430

HOLE NO. BH 3-22

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	76.89
FILL: Brown silty sand with gravel, some clay, organics and concrete		AU	1				
1.07							
Compact, brown **SILTY SAND**		SS	2	75	14	1	75.89
1.27							
Firm, brown **SILTY CLAY**		SS	3	100	4	2	74.89
- grey by 3.7m depth		SS	4	100	P	3	73.89
		SS	5	100	P	4	72.89
- soft to firm by 5.3m depth						5	71.89
						6	70.89
		SS	6	8	P	7	69.89
7.47							
End of Borehole							
(GWL @ 1.09m - Nov. 2, 2022)							



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Warehouse Complex - 6165 Thunder Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

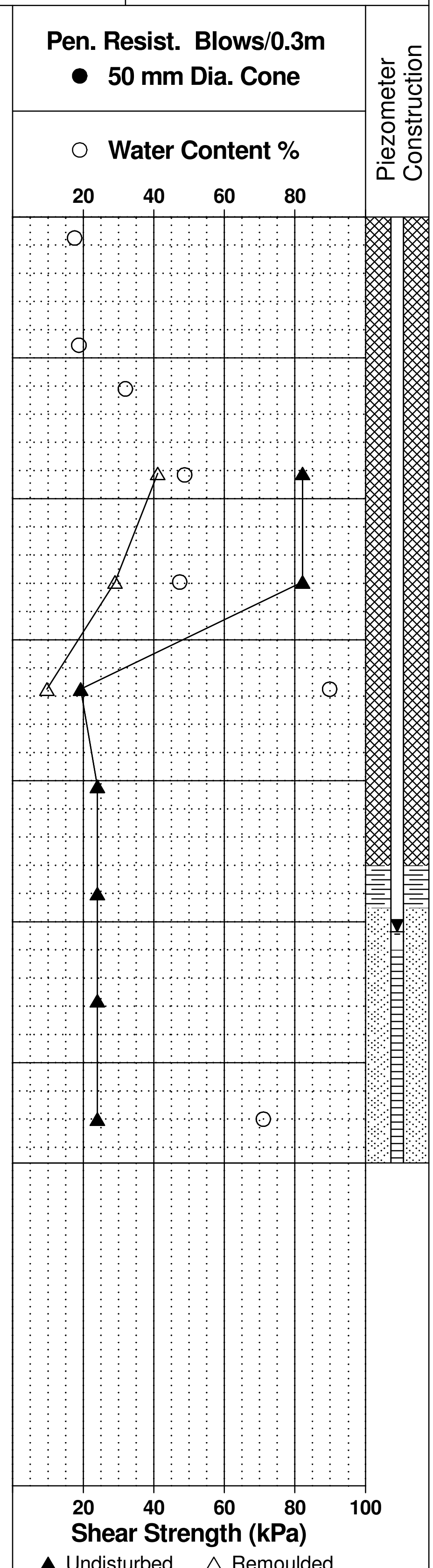
BORINGS BY Track-Mount Power Auger

DATE October 25, 2022

FILE NO. PG6430

HOLE NO. BH 4-22

SOIL DESCRIPTION	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	76.71
FILL: Brown silty sand with gravel, some clay, organics, cobbles, trace concrete (0.69)	AU	1				
Loose, brown **SILTY SAND to SANDY SILT** (1.17)	SS	2	75	5	1	75.71
Stiff to firm, brown **SILTY CLAY**	SS	3	17	P	2	74.71
	SS	4	17	P		
- soft and grey by 3.0m depth	SS	5	100	P	3	73.71
					4	72.71
					5	71.71
					6	70.71
(6.71)	SS	6	100	P		
End of Borehole						
(GWL @ 5.07m - Nov. 2, 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 strength of cohesionless soils is the relative density, usually inferred from the results of the Standard Penetration Test (SPT) 'N' value. The SPT N value is the number of blows of a 63.5 kg hammer, falling 760 mm, required to drive a 51 mm O.D. split spoon sampler 300 mm into the soil after an initial penetration of 150 mm.

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

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

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

SYMBOLS AND TERMS (continued)

SOIL DESCRIPTION (continued)

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

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

ROCK DESCRIPTION

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

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

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

SAMPLE TYPES

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

SYMBOLS AND TERMS (continued)

GRAIN SIZE DISTRIBUTION

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

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

CONSOLIDATION TEST

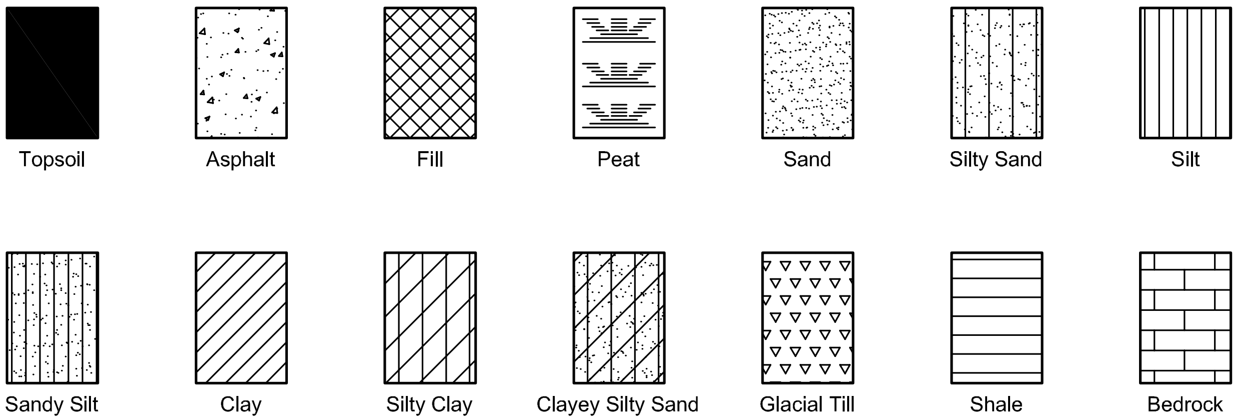
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void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

PERMEABILITY TEST

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

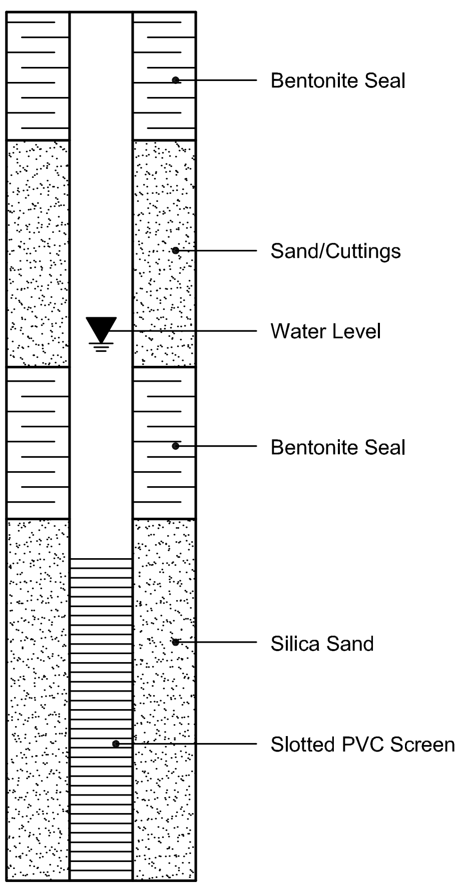
SYMBOLS AND TERMS (continued)

STRATA PLOT

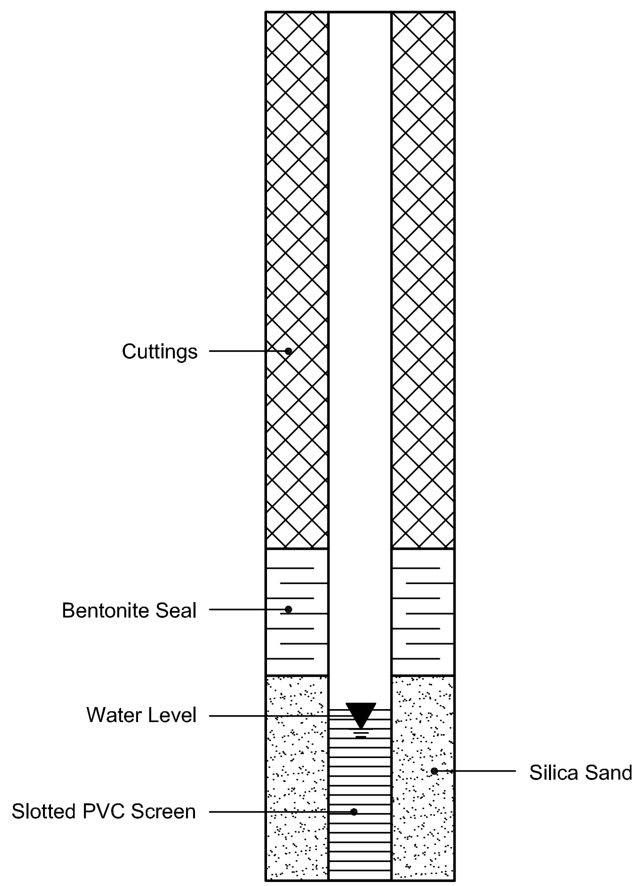


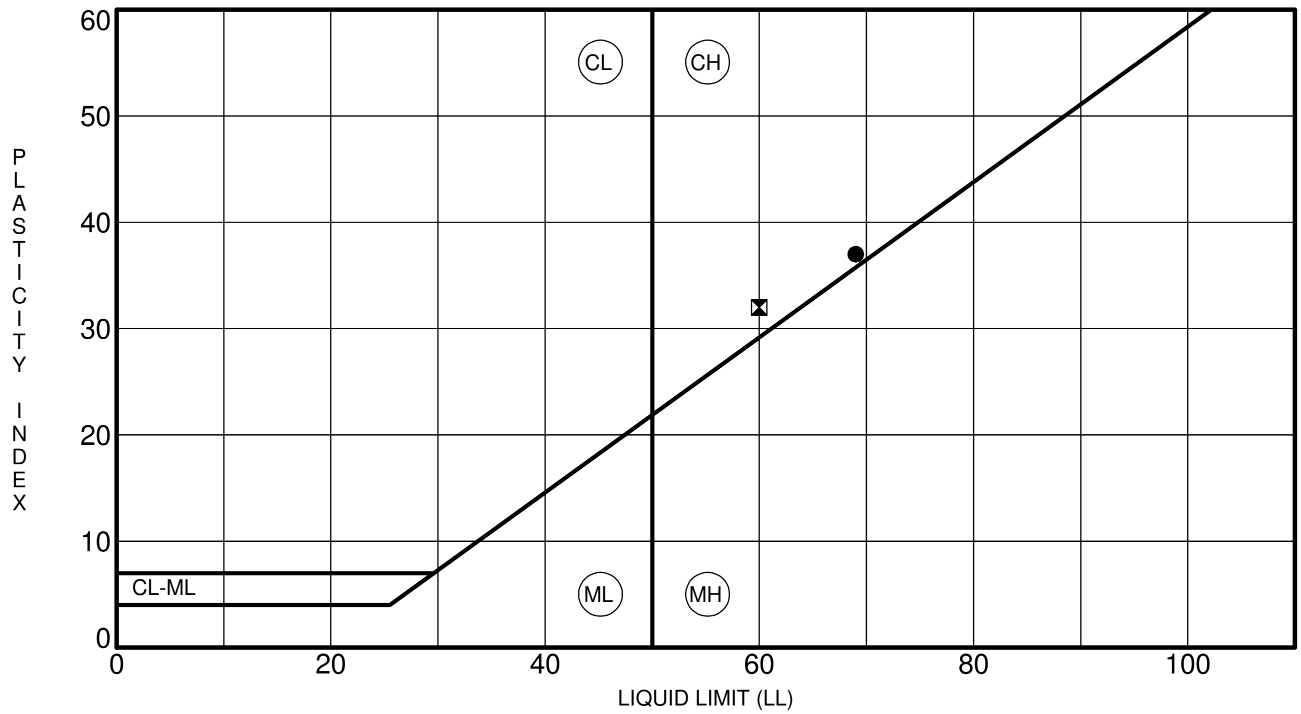
MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION





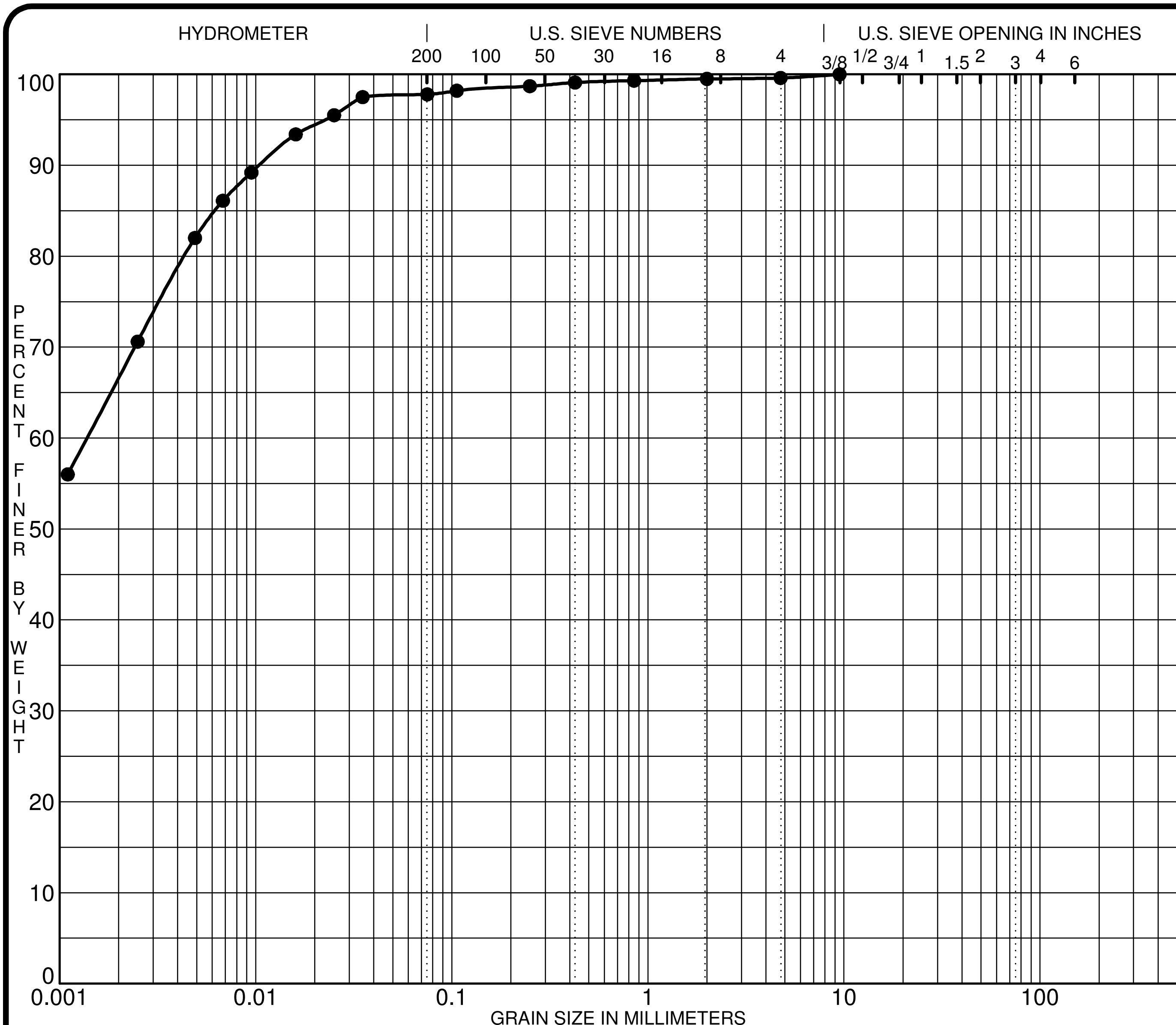
	Specimen Identification		LL	PL	PI	Fines	Classification
●	**BH 1-22**	**SS3**	**69**	**32**	**37**		**CH - Inorganic clay of high plasticity**
⧗	**BH 2-22**	**SS3**	**60**	**28**	**32**		**CH - Inorganic clay of high plasticity**

CLIENT	**HP Urban Inc.**	FILE NO.	**PG6430**
PROJECT	**Geotechnical Investigation - Proposed Warehouse Complex - 6165 Thunder Road**	DATE	**25 Oct 22**

patersongroup **Consulting Engineers**

9 Auriga Drive, Ottawa, Ontario K2E 7T9

ATTERBERG LIMITS' RESULTS



SILT OR CLAY	SAND			GRAVEL		COBBLES
	fine	medium	coarse	fine	coarse	

	Specimen Identification		Classification	MC%	LL	PL	PI	Cc	Cu
●	**BH 3-22**	**SS3**	**CH - Inorganic clay of high plasticity**						
⊠									
▲									
★									

	Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**BH 3-22**	**SS3**	**9.50**	**0.00**			**0.4**	**1.8**	**97.8**	
⊠										
▲										
★										

CLIENT **HP Urban Inc.**

FILE NO. **PG6430**

PROJECT **Geotechnical Investigation - Proposed Warehouse Complex - 6165 Thunder Road**

DATE **25 Oct 22**

patersongroup **Consulting Engineers**

9 Auriga Drive, Ottawa, Ontario K2E 7T9

GRAIN SIZE DISTRIBUTION

PATERSON GROUP

Linear Shrinkage
ASTM D4943-02

CLIENT:	HP Urban Inc.	DEPTH	5'-0" to 7'-0"	FILE NO.:	PG6430
PROJECT:	6165 Thunder Rd	BH OR TP No:	BH1-22	DATE SAMPLED	25-Oct
LAB No:	40451	TESTED BY:	CP/CS	DATE RECEIVED	28-Oct
SAMPLED BY:	J.P	DATE REPORTED:	7-Nov-22	DATE TESTED	2-Nov

LABORATORY INFORMATION & TEST RESULTS

Moisture	**No. of Blows(8)**	**Calibration (Two Trials)**	**Tin NO.(F)**	
Tare	4.93	Tin	4.76	4.76
Soil Pat Wet + Tare	62.26	Tin + Grease	4.93	4.93
Soil Pat Wet	57.33	Glass	48.97	48.97
Soil Pat Dry + Tare	36.71	Tin + Glass + Water	91.36	91.38
Soil Pat Dry	31.78	Volume	37.46	37.48
Moisture	**80.40**	**Average Volume**	**37.47**	

Soil Pat + String	31.92
Soil Pat + Wax + String in Air	36.42
Soil Pat + Wax + String in Water	13.11
Volume Of Pat (Vdx)	23.31

RESULTS:

Shrinkage Limit	**19.93**
Shrinkage Ratio	**1.741**
Volumetric Shrinkage	**105.272**
Linear Shrinkage	**21.313**

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

PARACEL LABORATORIES LTD.

Order #: 2244373

Certificate of Analysis

Report Date: 03-Nov-2022

Client: Paterson Group Consulting Engineers

Order Date: 27-Oct-2022

Client PO: 56074

Project Description: PG6430

	Client ID:	BH3-22-SS4	-	-	-		
	Sample Date:	25-Oct-22 09:00	-	-	-	-	-
	Sample ID:	2244373-01	-	-	-		
	Matrix:	Soil	-	-	-		
	MDL/Units						
Physical Characteristics							
% Solids	0.1 % by Wt.	59.2	-	-	-	-	-
General Inorganics							
pH	0.05 pH Units	7.75	-	-	-	-	-
Resistivity	0.1 Ohm.m	18.0	-	-	-	-	-
Anions							
Chloride	5 ug/g	53	-	-	-	-	-
Sulphate	5 ug/g	267	-	-	-	-	-

APPENDIX 2

FIGURE 1 – KEY PLAN

DRAWING PG6430-1 – TEST HOLE LOCATION PLAN

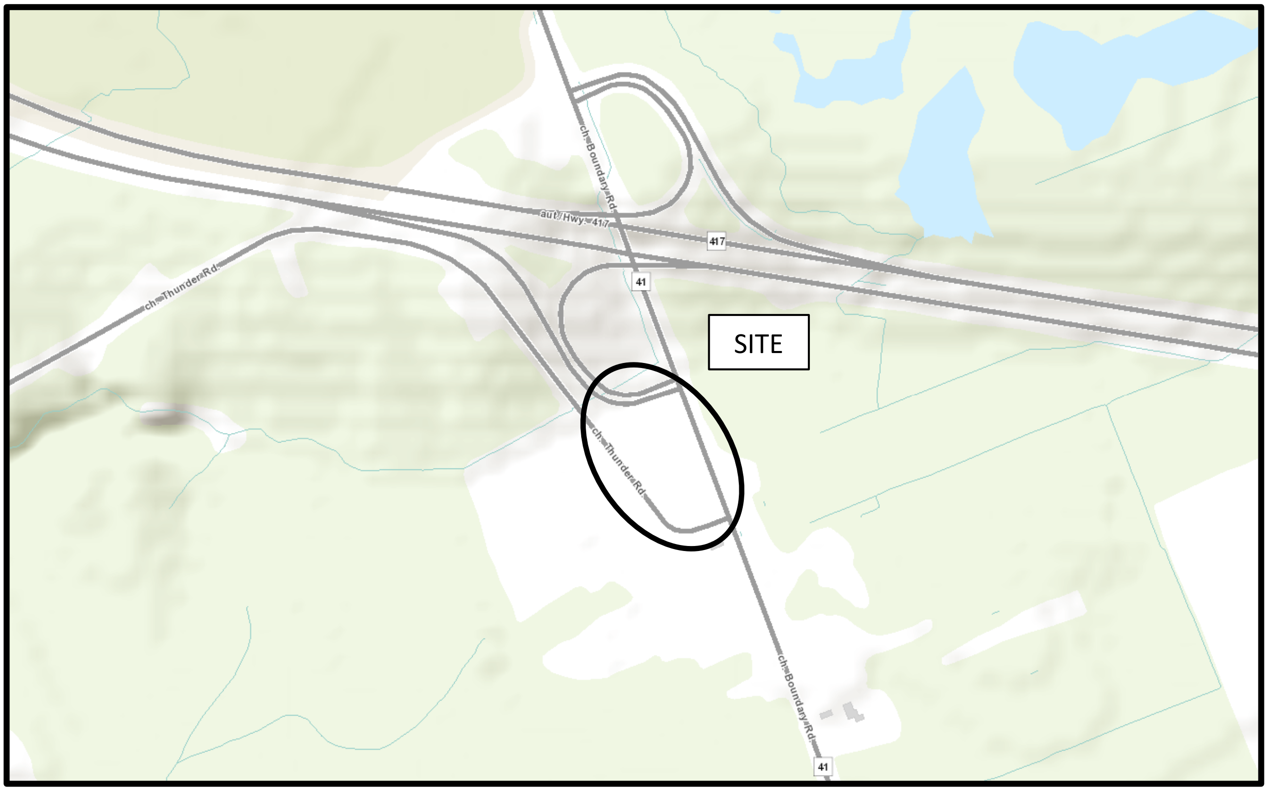
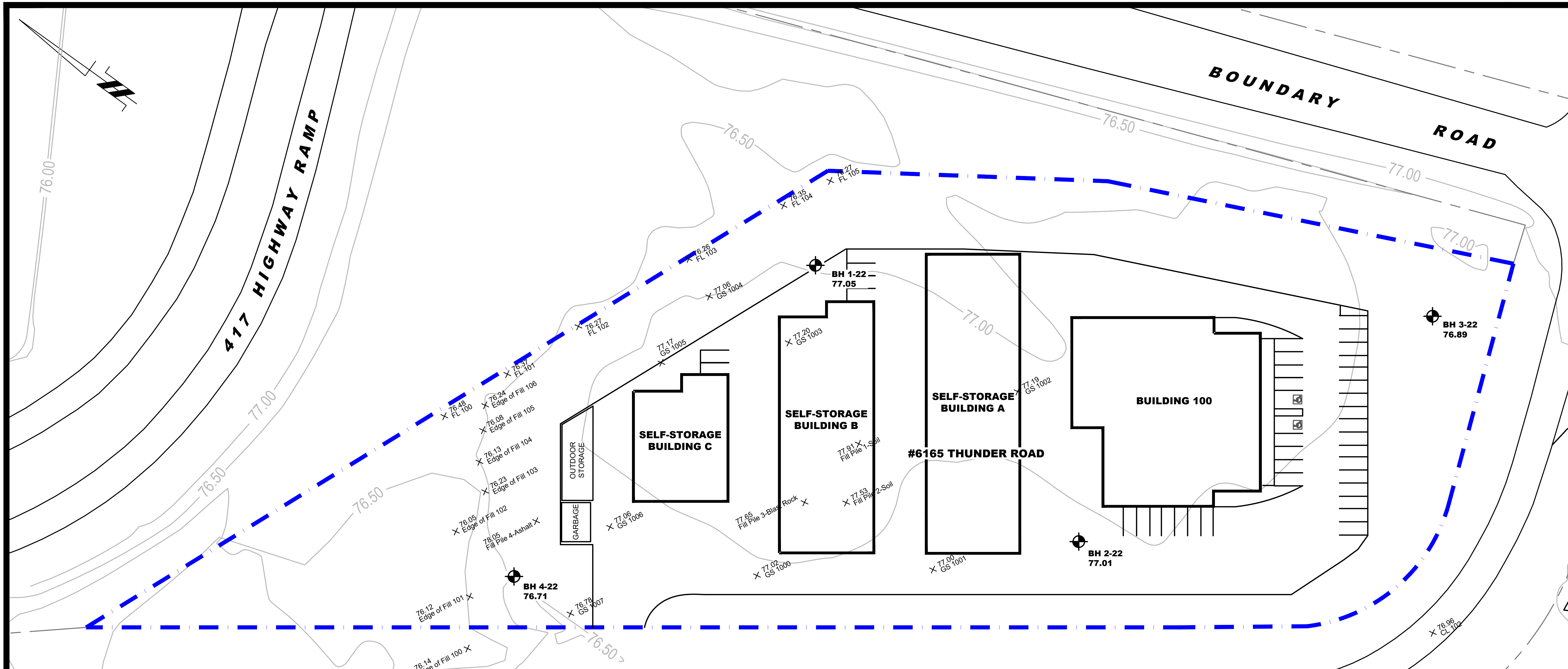


FIGURE 1

KEY PLAN



LEGEND:

BOREHOLE LOCATION

76.89 GROUND SURFACE ELEVATION (m)

CONCEPTUAL PLAN PROVIDED BY STEWART + TSAI ARCHITECTS INC.

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

SCALE: 1:750

0 10 20 30 50m

PATERSON GROUP

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

NO.	REVISIONS	DATE	INITIAL

HP URBAN INC.

GEOTECHNICAL INVESTIGATION
PROPOSED WAREHOUSE COMPLEX
6165 THUNDER ROAD

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:750	Date: 12/2022
Drawn by: JM	Report No.: PG6430-1
Checked by: NP	Dwg. No.: PG6430-1
Approved by: DJG	Revision No.:

p:\autocad drawings\geotechnical\pg64xx\pg6430\pg6430-1 test hole location plan r.dwg

11x17