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Proposed Multi-Storey Building
100 Weeping Willow Lane
Ottawa, Ontario

Prepared For

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Revision 1

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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by Homestead Land Holdings Ltd. to conduct a geotechnical investigation for the proposed multi-storey development to be located at 100 Weeping Willow Lane in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2 of this report). The objective of the investigation was to:

- ❑ Determine the existing subsoil and groundwater conditions at this site by means of boreholes.
- ❑ Provide geotechnical recommendations pertaining to 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 proposed development as they are understood at the time of this report.

2.0 Proposed Development

Based on the available drawings, it is understood that the proposed development will consist of a multi-storey building with 1 level of underground parking. The underground parking level is also proposed to extend to the east beyond the limits of the multi-storey building. Asphalt-paved parking areas, walkways and landscaped areas are proposed at finished grades surrounding the multi-storey building.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field investigation was carried out from June 25 to June 29, 2021, and consisted of advancing a total of 5 boreholes to a maximum depth of 12.7 m. The borehole locations were determined in the field by Paterson personnel taking into consideration site features and underground services. The locations of the boreholes are shown on Drawing PG5862-1 - Test Hole Location Plan in Appendix 2.

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

Sampling and In Situ Testing

Soil samples were recovered from the boreholes using two different techniques, namely, sampled directly from the auger flights (AU) or collected using a 50 mm diameter split-spoon (SS) sampler. Rock cores (RC) were obtained using a 47.6 mm inside diameter coring equipment. All samples were visually inspected and initially classified on site. The auger and split-spoon samples were placed in sealed plastic bags, and rock cores were placed in cardboard boxes. All samples were transported to our laboratory for further examination and classification. The depths at which the auger, split spoon and rock core samples were recovered from the boreholes are shown as AU, SS and RC, respectively, on the Soil Profile and Test Data sheets presented in Appendix 1.

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

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

Bedrock samples were recovered at borehole BH 2-21 using a core barrel and diamond drilling techniques. The depths at which rock core samples were recovered from the boreholes are shown as RC on the Soil Profile and Test Data sheets in Appendix 1.

A recovery value and a Rock Quality Designation (RQD) value were calculated for each drilled section (core run) of bedrock and are shown on the borehole logs. The recovery value is the ratio, in percentage, of the length of the bedrock sample recovered over the length of the drilled section (core run). The RQD value is the ratio, in percentage, of the total length of intact rock pieces longer than 100 mm in one core run over the length of the core run. These values are indicative of the quality of the bedrock.

The overburden thickness was evaluated by a dynamic cone penetration test (DCPT) completed at borehole BH 5-21 of the field investigation. 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 record for each 300 mm increment.

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

Groundwater

Flexible standpipes were installed in select boreholes to permit monitoring of the groundwater levels subsequent to the completion of the sampling program.

Sample Storage

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

3.2 Field Survey

The borehole locations were selected by Paterson to provide general coverage of the proposed development, taking into consideration the existing site features and underground utilities. The borehole locations and ground surface elevations at each borehole location were surveyed by Paterson using a GPS unit with respect to a geodetic datum. The location of the boreholes and ground surface elevation at each borehole location are presented on Drawing PG5862-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

Soil and bedrock samples were recovered from the subject site and visually examined in our laboratory to review the results of the field logging.

3.4 Laboratory Testing

One 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 Section 6.7.

4.0 Observations

4.1 Surface Conditions

The subject site is located within the northwest portion of 100 Weeping Willow Lane. The majority of the subject site is occupied by an existing fill pile which peaks in elevation at the center of the site and is grass covered with landscaped areas. Mature trees are located with the eastern portion of the subject site. An existing multi-storey building is located within the central portion of 100 Weeping Willow Lane, to the east of the proposed multi-storey building.

The subject site is bordered to the north by Weeping Willow Lane, to the east by asphalt-paved access lanes and parking areas associated with the existing multi-storey building, to the south by mature trees and an unnamed tributary of the Kizell Municipal Drain, and to the west by Varley Drive. The ground surface across the subject site slopes gently downward in all directions from the top of the historic fill pile, toward either the surrounding roadways or towards the unnamed tributary at approximate geodetic elevation 94.0 to 90.5 m.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile encountered at the test hole locations consists of a topsoil layer underlain by an approximate 2.0 to 4.0 m thick fill layer. The fill material was generally observed to consist of silty sand to silty clay with gravel, trace and trace amounts of topsoil and organics.

A stiff, brown silty clay was observed underlying the fill material, becoming firm and grey at approximate depths ranging from 4.0 to 5.2 m below the existing ground surface. The thickness of the silty clay deposit generally increases from west to east across the site, extending to depths ranging from 7.7 m at borehole BH 2-21 at the west end of the site, to 16.2 m at borehole BH 5-21 at the east end of the site.

Bedrock

Practical refusal to augering on the bedrock surface was encountered at an approximate depth of 7.7 m at borehole BH 2-21, and practical refusal to the DCPT was encountered at an approximate depth of 16.2 m at borehole BH 5-21.

The bedrock was cored at borehole BH 2-21 and was observed to consist of interbedded reddish grey gneiss and granite and was generally of good to excellent quality based on the RQDs of the bedrock core. At borehole BH 2-21, the bedrock was cored to an approximate depth of 10.3 m below the existing ground surface.

Based on available geological mapping, bedrock in the area of the subject site consists of Precambrian paragneiss of granitic origin with an overburden thickness ranging from approximately 1 to 15 m.

4.3 Groundwater

Groundwater level readings were measured in the standpipes on July 15, 2021. The measured groundwater level (GWL) readings are presented in Table 1 below.

Table 1 - Summary of Groundwater Level Readings

Borehole Number	Ground Surface Elevation (m)	Groundwater Levels (m)	Groundwater Elevation (m)	Recording Date
BH 1-21	91.20	3.45	87.75	July 15, 2021
BH 3-21	93.87	Borehole Dry	n/a	July 15, 2021
BH 4-21	91.23	4.22	87.01	July 15, 2021
BH 5-21	92.33	4.49	87.84	July 15, 2021

It should be noted that groundwater levels could be influenced by surface water infiltrating the backfilled boreholes. Long-term groundwater levels can also be estimated based on the observed colour, moisture content and consistency of the recovered soil samples.

Based on these observations, the long-term groundwater level is expected between an approximate 4 to 5 m depth. However, it should be noted that groundwater levels are subject to seasonal fluctuations, therefore, the groundwater levels could vary at the time of construction.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is considered suitable for the proposed development. It is recommended that the proposed multi-storey building be founded on one of the following:

- A raft foundation bearing on an undisturbed, stiff to firm silty clay bearing surface, or

- Deep foundations, such as end-bearing piles, which extend to the bedrock surface.

Further, it is recommended that the portions of the underground parking level which extend beyond the footprint of the multi-storey building be supported on conventional spread footings bearing on an undisturbed, stiff to firm silty clay bearing surface.

Due to the presence of a silty clay layer, the proposed development will be subjected to grade raise restrictions. Our permissible grade raise recommendations are discussed in Subsection 5.3.

The existing fill pile observed at the site should be further assessed to determine if the fill material is suitable for reuse as part of the proposed development.

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.

Protection of Subgrade (Raft Foundation)

Should a raft foundation be used, the raft subgrade would consist of a silty clay deposit, and it is recommended that a minimum 75 mm thick lean concrete mud slab be placed on the undisturbed silty clay subgrade shortly after the completion of the excavation. The main purpose of the mudslab is to reduce the risk of disturbance of the subgrade under the traffic of workers and equipment.

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

Compacted Granular Fill Working Platform (Pile Foundation)

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

A typical working platform could consist of 600 mm of OPSS Granular B Type II material, placed and compacted to a minimum of 98% of its SPMDD in lifts not exceeding 300 mm in thickness.

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

Vibration Considerations

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

The following construction equipment could be the source of vibrations: pile driving crane, compactor, dozer, truck traffic, etc. Vibrations could be the source of detrimental vibrations on the nearby buildings and structures. Therefore, all vibrations are recommended to be limited.

Two parameters are used to determine the permissible vibrations, namely, the maximum peak particle velocity and the frequency. For low frequency vibrations, the maximum allowable peak particle velocity is less than that for high frequency vibrations. As a guideline, the peak particle velocity should be less than 15 mm/s between frequencies of 4 to 12 Hz, and 50 mm/s above a frequency of 40 Hz (interpolate between 12 and 40 Hz).

The guidelines are for current construction standards. Considering that these guidelines are above perceptible human level and, in some cases, could be very disturbing to some people, a pre-construction survey is recommended to be completed to minimize the risks of claims during or following the construction of the proposed building.

Fill Placement

Fill used for grading beneath the proposed building should consist of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. This material should be tested and approved prior to delivery to the site. The fill should be placed in maximum 300 mm thick loose lifts and compacted by suitable compaction equipment. Fill placed beneath the building and paved areas 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 can be used as general landscaping fill where settlement of the ground surface is of minor concern. This material should be spread in lifts and at least compacted by the tracks of the spreading equipment to minimize voids. If this material is to be used to build up the subgrade level for areas to be paved, it should be compacted in thin lifts to at least 95% of the material's SPMDD.

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

5.3 Foundation Design

Spread Footings for Underground Level beyond Multi-Storey Building

For the portion of the underground parking level located beyond the footprint of the proposed multi-storey building, it is recommended that conventional spread footings placed on an undisturbed, stiff to firm silty clay bearing surface can be designed using a bearing resistance value at serviceability limit states (SLS) of **100 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **150 kPa**. A geotechnical resistance factor of 0.5 was applied to the above noted bearing resistance value at ULS.

The 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.

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 an undisturbed, stiff to firm silty clay above the groundwater table, when a plane extending down and out from the bottom edge of the footing at a minimum of 1.5H:1V passes only through in situ soil of the same or higher capacity as the bearing medium soil.

Raft Foundation

The proposed multi-storey building may be supported on a raft foundation, where the contact pressure is within the values provided below. For 1 underground parking level, it is anticipated that the excavation will extend to a depth such that the underside of the raft slab would be placed between geodetic elevations of 86 to 85 m.

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

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

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

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

End Bearing Pile Foundation

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

Table 2 – Pile Foundation Design Data					
Pile Outside Diameter (mm)	Pile Wall Thickness (mm)	Geotechnical Axial Resistance		Final Set (blows/ 12 mm)	Transferred Hammer Energy (kJ)
		SLS (kN)	Factored at ULS (kN)		
245	9	925	1090	10	28.5
245	11	1050	1260	10	34.2
245	13	1200	1500	10	40.7

Re-striking of all piles, at least once, will also be required after at least 48 hours have elapsed since initial driving. A full-time field review program should be conducted during the pile driving operations to record the pile lengths, ensure that the refusal criteria is met and that piles are driven within the location tolerances (within 75 mm of proper location and within 2% of vertical).

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

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

Permissible Grade Raise

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

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

5.4 Design for Earthquakes

The site class for seismic site response can be taken as **Class D**. If a higher seismic site class is required (Class C), a site specific shear wave velocity test may be completed to accurately determine the applicable seismic site classification for foundation design of the proposed building, as presented in Table 4.1.8.4.A of the Ontario Building Code (OBC) 2012.

Soils underlying the subject site are not susceptible to liquefaction. Reference should be made to the latest revision of the OBC 2012 for a full discussion of the earthquake design requirements.

5.5 Basement Floor Slab

With the removal of all topsoil and deleterious fill from within the footprint of the proposed building, the native soil will be considered an acceptable subgrade on which to commence backfilling for floor slab construction. It is understood that the underground level for the proposed building will be mostly parking and the recommended pavement structures noted in Subsection 5.7 will be applicable. However, if storage or other uses of the lower level will involve the construction of a concrete floor slab, the upper 200 mm of sub-slab fill is recommended to consist of 19 mm clear crushed stone.

Any soft areas in the basement slab subgrade should be removed and backfilled with appropriate backfill material prior to placing fill. OPSS Granular A or Granular B Type II, with a maximum particle size of 50 mm, are recommended for backfilling below the floor slab. All backfill material within the footprint of the proposed building should be placed in maximum 300 mm thick loose layers and compacted to a minimum of 98% of the SPMDD.

In consideration of the groundwater conditions encountered during the field investigation, a sub-slab drainage system, consisting of lines of perforated drainage pipe subdrains connected to a sump pit, should be provided in the subfloor fill under the lower basement floor (discussed further in Subsection 6.1).

5.6 Basement Wall

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

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

Lateral Earth Pressures

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

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

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

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

Seismic Earth Pressures

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

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

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

The peak ground acceleration, (a_{max}), for the Ottawa area is 0.32 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 2012.

5.7 Pavement Design

For design purposes, it is recommended that the rigid pavement structure for the lower 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 3 below. The flexible pavement structure presented in Table 4 should be used for at grade access lanes and heavy loading parking areas.

Table 3 - Recommended Rigid Pavement Structure - Lower Parking Level

Thickness (mm)	Material Description
125	**Exposure Class C2 - 32 MPa Concrete** (5 to 8% Air Entrainment)
300	**BASE** - OPSS Granular A Crushed Stone
SUBGRADE - Existing imported fill, or OPSS Granular B Type I or II material placed over in situ soil.	

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 lower 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 hour after the concrete has been poured during warm temperatures and up to 12 hours during cooler temperatures.

Table 4 - Recommended Asphalt Pavement Structure - Access Lanes and Heavy Loading Parking Areas

Thickness (mm)	Material Description
40	**Wear Course** - Superpave 12.5 Asphaltic Concrete
50	**Binder Course** - Superpave 19.0 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
300	**SUBBASE** - OPSS Granular B Type II
SUBGRADE - OPSS Granular B Type II overlying the Concrete Podium Deck.	

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

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

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

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Foundation Drainage

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

Where insufficient room is available for exterior backfill, it is suggested that the composite drainage system (such as Delta Drain 6000 or equivalent) be secured against the temporary shoring system, extending to a series of drainage sleeve inlets through the building foundation wall at the footing/foundation wall interface. The drainage sleeves should be at least 150 mm diameter and be spaced 3 m along the perimeter foundation walls. An interior perimeter drainage pipe should be placed along the building perimeter along with the sub-slab drainage system. The perimeter drainage pipe and sub-slab drainage system should direct water to sump pit(s) within the underground level.

Foundation Raft Slab Construction Joints

If applicable, it is expected that the raft slab will be poured in sections. For the construction joint at each pour, a rubber water stop along with a chemical grout (Xypex or equivalent) should be applied to the entire vertical joint of the raft slab. Furthermore, a rubber water stop should be incorporated in the horizontal interface between the foundation wall and the raft slab.

Sub-slab Drainage

Sub-slab drainage will be required to control water infiltration below the underground parking level slab. For preliminary design purposes, we recommend that 150 mm perforated pipes be placed at approximate 6 m centres underlying the basement floor slab. The spacing of the sub-slab drainage system should be confirmed by the geotechnical consultant at the time of completing the excavation when water infiltration can be better assessed.

Foundation Backfill

Where sufficient space is available for conventional backfilling, the backfill material against the exterior sides of the foundation walls should consist of free-draining, non frost susceptible granular materials. The site materials will be frost susceptible and, as such, are not recommended for re-use as backfill unless a composite drainage system (such as Delta Drain 6000) connected to a drainage system is provided.

6.2 Protection of Footings Against Frost Action

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

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

However, the footings are generally not expected to require protection against frost action due to the founding depth. Unheated structures such as the access ramp may require insulation for protection against the deleterious effects of frost action.

6.3 Excavation Side Slopes

The side slopes of excavations in the overburden materials should either be cut back at acceptable slopes or should be retained by temporary shoring systems from the start of the excavation until the structure is backfilled.

Unsupported Excavations

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

Excavated soil should not be stockpiled directly at the top of excavations and heavy equipment should be kept away from the excavation sides. Slopes in excess of 3 m in height should be periodically inspected by the geotechnical consultant in order to detect if the slopes are exhibiting signs of distress.

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

Temporary Shoring

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

In the event that subsurface conditions differ from the approved design during the actual installation, it is the responsibility of the shoring contractor to commission the required experts to re-assess the design and implement the required changes.

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

The temporary shoring system may consist of a soldier pile and lagging system or steel sheet piles which could be cantilevered, anchored or braced. The shoring system is recommended to be adequately supported to resist toe failure. Any additional loading due to street traffic, construction equipment, adjacent structures and facilities, etc., should be added to the earth pressures described below.

The earth pressures acting on the temporary shoring system may be calculated using the parameters outlined in Table 6 below.

Table 5 - Soil Parameters for Calculating Earth Pressures Acting on Shoring System

Parameter	Value
Active Earth Pressure Coefficient (K_a)	0.33
Passive Earth Pressure Coefficient (K_p)	3
At-Rest Earth Pressure Coefficient (K_o)	0.5
Unit Weight (γ), kN/m^3	21
Submerged Unit Weight(γ’), kN/m^3	13

The active earth pressure should be calculated where wall movements are permissible while the at-rest pressure should be calculated if no movement is permissible.

The dry unit weight should be used above the groundwater level while the effective unit weight should be used below the groundwater level.

The hydrostatic groundwater pressure should be added to the earth pressure distribution wherever the effective unit weights are used for earth pressure calculations. If the groundwater level is lowered, the dry unit weight for the soil should be used full weight, with no hydrostatic groundwater pressure component. For design purposes, the minimum factor of safety of 1.5 should be calculated.

6.4 Pipe Bedding and Backfill

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

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

It should generally be possible to re-use the site materials above the cover material if the operations are carried out in dry weather conditions.

Where hard surface areas are considered above the trench backfill, the trench backfill material within the frost zone (about 1.8 m below finished grade) 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's SPMDD. All cobbles larger than 200 mm in their longest direction should be segregated from re-use as trench backfill.

6.5 Groundwater Control

It is anticipated that groundwater infiltration into the excavations should be 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.

Impacts on Neighbouring Properties

The proposed building is not anticipated to extend significantly below the groundwater level, therefore, any dewatering at the site will be minimal and should have no adverse effects to the surrounding buildings or structures. The short term dewatering during the excavation program will be managed by the excavation contractor.

6.6 Winter Construction

Precautions must be taken if winter construction is considered for this project. The subsoil conditions at this site consist of frost susceptible materials. In the presence of water and freezing conditions, ice could form within the soil mass. Heaving and settlement upon thawing could occur.

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

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

6.7 Corrosion Potential and Sulphate

The results of the 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 indicate of an non aggressive to slightly aggressive corrosive environment.

6.8 Slope Stability Assessment

The slope conditions at the southern limits of the site were reviewed by Paterson field personnel on July 15, 2021 as part of the geotechnical investigation. One (1) slope cross-section (Section A) was studied as the worst case scenario. This is considered to be the worst-case scenario, as it is the portion of the proposed building footprint which is nearest to the creek, and where the topography of the slope is the steepest. The cross-section location is presented on Drawing PG5862-1 - Test Hole Location Plan in Appendix 2.

The existing slope, which extends down to a creek which leads to the Kizell Municipal Drain, was observed to be heavily vegetated with mature trees. No significant signs of erosion were observed at the toe of the slope along the watercourse. However, signs of historical erosion such as sloughing was observed along portions of the slope face. Bedrock outcroppings were observed along the creek bed at the western portion of the slope. Photographs from our site visit are included in Appendix 2.

Slope Stability Assessment

The analyses of the stability of the slope were carried out using SLIDE, a computer program which permits a two-dimensional slope stability analysis using several methods including the Bishop's method, which is a widely used and accepted analysis method. The program calculates a factor of safety, which represents the ratio of the forces resisting failure to those favouring failure. Theoretically, a factor of safety of 1.0 represents a condition where the slope is stable. However, due to intrinsic limitations of the calculation methods and the variability of the subsoil and groundwater conditions, a factor of safety greater than one is usually required to ascertain that the risks of failure are acceptable.

A minimum factor of safety of 1.5 is generally recommended for conditions where the failure of the slope would endanger permanent structures.

The cross-section was analyzed based on the existing conditions observed during our site visit and review of the available topographic mapping, and also taking into account the proposed site conditions which includes the removal of the existing fill pile. The slope stability analysis was completed at the cross-section under worst-case-scenario by assigning cohesive soils under fully saturated groundwater conditions. Subsoil conditions at the cross-sections were inferred based on nearby boreholes and general knowledge of the area's geology.

The effective soil strength parameters used for static analysis were chosen based on the subsoil information recovered during the geotechnical investigation. The effective strength soil parameters used for static analysis are presented in Table 6 below.

Table 6 – Effective Soil and Material Parameters (Static Analysis)

Soil Layer	Unit Weight (kN/m^3)	Friction Angle (degrees)	Cohesion (kPa)
Fill	19	33	2
Brown Silty Clay Crust	17	33	5
Grey Silty Clay	16	33	10

The total strength parameters for seismic analysis were chosen based on the in situ, undrained shear strengths recovered within the open boreholes completed at the time of the geotechnical investigation based on our general knowledge of the geology in the area. The strength parameters used for seismic analysis at the slope cross-sections are presented in Table 7 below.

Table 7 – total Stress Soil and Material Parameters (Seismic Analysis)

Soil Layer	Unit Weight (kN/m^3)	Friction Angle (degrees)	Undrained Shear Strength (kPa)
Fill	19	33	2
Brown Silty Clay Crust	17	-	80
Grey Silty Clay	16	-	50

Static Loading Analysis

The results static analysis for the proposed site conditions at Section A are shown on Figure 2, in Appendix 2. For the proposed conditions, the factor of safety was found to be 3.1, therefore a stable slope allowance is not required along the subject slope.

Seismic Loading Analysis

An analysis considering seismic loading for proposed conditions was also completed. A horizontal acceleration of 0.16 g was considered for the subject slope. A factor of safety of 1.1 is considered to be satisfactory for stability analyses including seismic loading.

The results of the seismic analysis for proposed site conditions are shown on Figure 3 in Appendix 2. The results indicate that the factor of safety is 3.9 under seismic conditions. Based on these results, the slope is considered to be stable under seismic loading. Therefore, when considering seismic loading, no stable slope allowance is required from the top of the slope to achieve a factor of safety of 1.1 for the limit of the hazard lands.

Geotechnical Setback - Limit of Hazard Lands

The toe erosion allowance for the slope (Section A) was determined based on the cohesive nature of the soils, the width of the watercourse, and the observed current erosion activities, which were minimal. Therefore, a toe erosion allowance of 1 m, in addition to an erosion access allowance of 6 m, applied from the top of slope is considered appropriate.

The geotechnical limit of hazard lands is therefore setback 7 m from the geotechnical top of slope, as indicated on Drawing PG5862-1 Test Hole Location Plan, in Appendix 2.

However, it should be noted that other setbacks may be applicable from the top of slope, such as from the MVCA or other regulatory bodies, which may exceed the geotechnical limit of hazard lands setback provided in the preceding paragraph.

The existing vegetation on the slope face should not be removed as it contributes to the stability of the slope and reduces erosion. If the existing vegetation needs to be removed, it is recommended that a 100 to 150 mm of topsoil mixed with a hardy seed be placed across the exposed slope face. The use of an erosion control blanket, may be necessary to minimize rill-type erosion until the vegetation takes root.

7.0 Recommendations

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

- ❑ Review of the geotechnical aspects of the excavation contractor's shoring design, if required, prior to construction.
- ❑ Observation of all bearing surfaces prior to the placement of concrete.
- ❑ Sampling and testing of the concrete and fill materials.
- ❑ Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.
- ❑ Observation of all subgrades prior to backfilling.
- ❑ Field density tests to determine the level of compaction achieved.
- ❑ Sampling and testing of the bituminous concrete including mix design reviews.

A report confirming that these works have been conducted in general accordance with our recommendations could be issued upon 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 Homestead Land Holdings Ltd. or their agents is not authorized without review by Paterson for the applicability of our recommendations to the alternative use of the report.

Paterson Group Inc.

Kevin A. Pickard, EIT

LICENSED PROFESSIONAL ENGINEER
Aug. 8, 2023
S. S. DENNIS
100519516
PROVINCE OF ONTARIO

Scott S. Dennis, P.Eng.

Report Distribution:

- ❑ Homestead Land Holdings Ltd. (Digital copy)
- ❑ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TESTING RESULTS

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Multi-Storey Development
100 Weeping Willow Lane, Ottawa, Ontario

DATUM Geodetic

FILE NO. PG5862

REMARKS

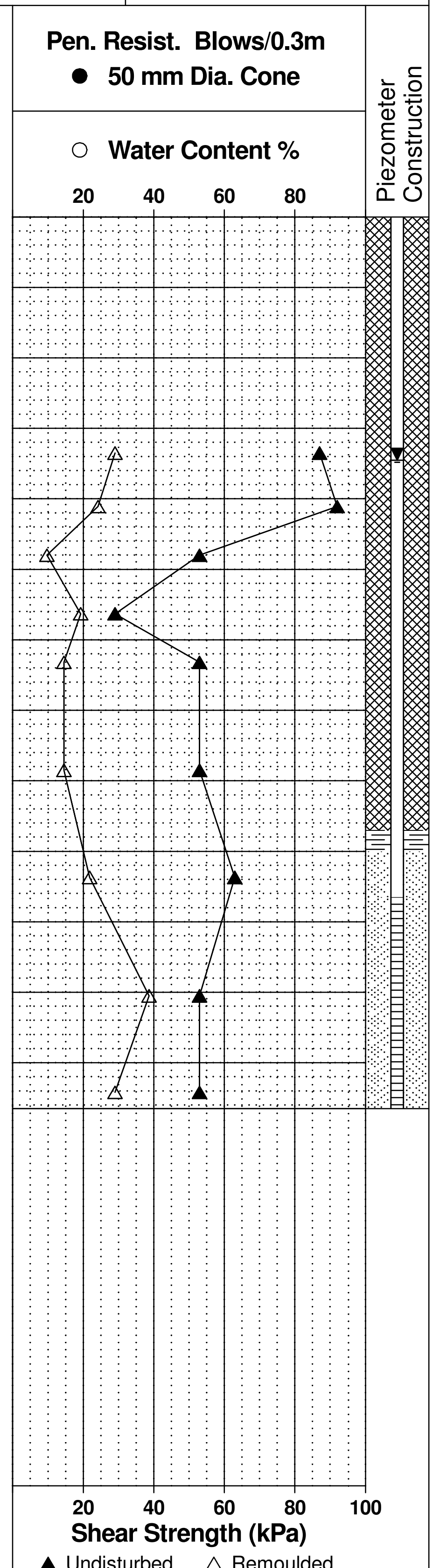
HOLE NO. BH 1-21

BORINGS BY CME-55 Low Clearance Drill

DATE June 25, 2021

SOIL DESCRIPTION	Depth to base (m)	TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
TOPSOIL	0.20				
FILL: Brown silty sand, some gravel	0.91	AU	1		
FILL: Brown silty clay with sand, trace topsoil	1.98	SS	2	83	21
		SS	3	8	8
Stiff to firm, brown **SILTY CLAY** - grey by 4.0m depth	12.65	SS	4	83	6
		SS	5	83	P
		SS	6	83	P
		SS	7	92	P
End of Borehole (GWL @ 3.45m - July 15, 2021)					

DEPTH (m)	ELEV. (m)
0	91.20
1	90.20
2	89.20
3	88.20
4	87.20
5	86.20
6	85.20
7	84.20
8	83.20
9	82.20
10	81.20
11	80.20
12	79.20



patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Multi-Storey Development
100 Weeping Willow Lane, Ottawa, Ontario

DATUM Geodetic

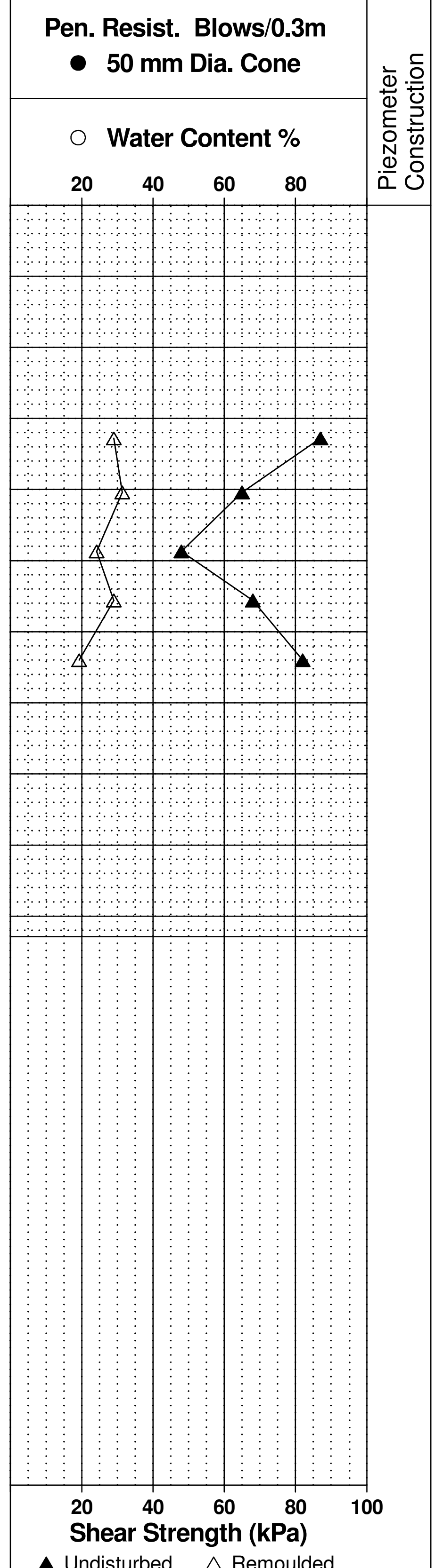
FILE NO. PG5862

REMARKS

HOLE NO. BH 2-21

BORINGS BY CME-55 Low Clearance Drill **DATE** June 25, 2021

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	90.72
TOPSOIL 0.30		AU	1				
FILL: Brown silty clay with sand, trace topsoil		SS	2	8	11	1	89.72
1.98		SS	3	8	6	2	88.72
Stiff to firm, brown **SILTY CLAY**		SS	4	100	5	3	87.72
						4	86.72
- grey by 4.7m depth		SS	5	100	P	5	85.72
		SS	6	100	P	6	84.72
7.72						7	83.72
		SS	7		50+		
BEDROCK: Good to excellent quality, reddish grey gneiss to granite		RC	1	100	80	8	82.72
						9	81.72
10.29		RC	2	100	96	10	80.72
End of Borehole							



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Multi-Storey Development
100 Weeping Willow Lane, Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE June 29, 2021

FILE NO. PG5862

HOLE NO. BH 3-21

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	93.87
TOPSOIL	0.20	AU	1				
FILL: Brown silty clay, some sand and organics, trace gravel		SS	2	17	18	1	92.87
		SS	3	33	9	2	91.87
		SS	4	25	16		
	3.50	SS	5	50	54	3	90.87
Stiff to firm, brown **SILTY CLAY,** trace sand		SS	6	100	7	4	89.87
		SS	7	100	4	5	88.87
		SS	8	100	1	6	87.87
	6.70	SS	9	100	1		
End of Borehole							
(BH dry - July 15, 2021)							

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

○ Water Content %
20 40 60 80

Piezometer Construction

20 40 60 80 100
Shear Strength (kPa)
▲ Undisturbed △ Remoulded

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Multi-Storey Development
100 Weeping Willow Lane, Ottawa, Ontario

DATUM Geodetic

FILE NO. PG5862

REMARKS

HOLE NO. BH 4-21

BORINGS BY CME-55 Low Clearance Drill **DATE** June 29, 2021

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
TOPSOIL	0.20	AU	1		
FILL: Brown silty sand, some clay and gravel	0.69	SS	2	42	9
FILL: Brown silty clay, some sand, trace gravel and organics	3.96	SS	3	50	5
		SS	4	67	2
		SS	5	50	3
Stiff to firm, brown **SILTY CLAY** - grey by 5.2m depth	5.94	SS	6	8	3
		SS	7	100	2
		SS	8	100	P
End of Borehole (GWL @ 4.22m - July 15, 2021)					

DEPTH (m)	ELEV. (m)
0	91.23
1	90.23
2	89.23
3	88.23
4	87.23
5	86.23

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

○ Water Content %

20 40 60 80

Piezometer Construction

Shear Strength (kPa)

20 40 60 80 100

▲ Undisturbed △ Remoulded

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Multi-Storey Development
100 Weeping Willow Lane, Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

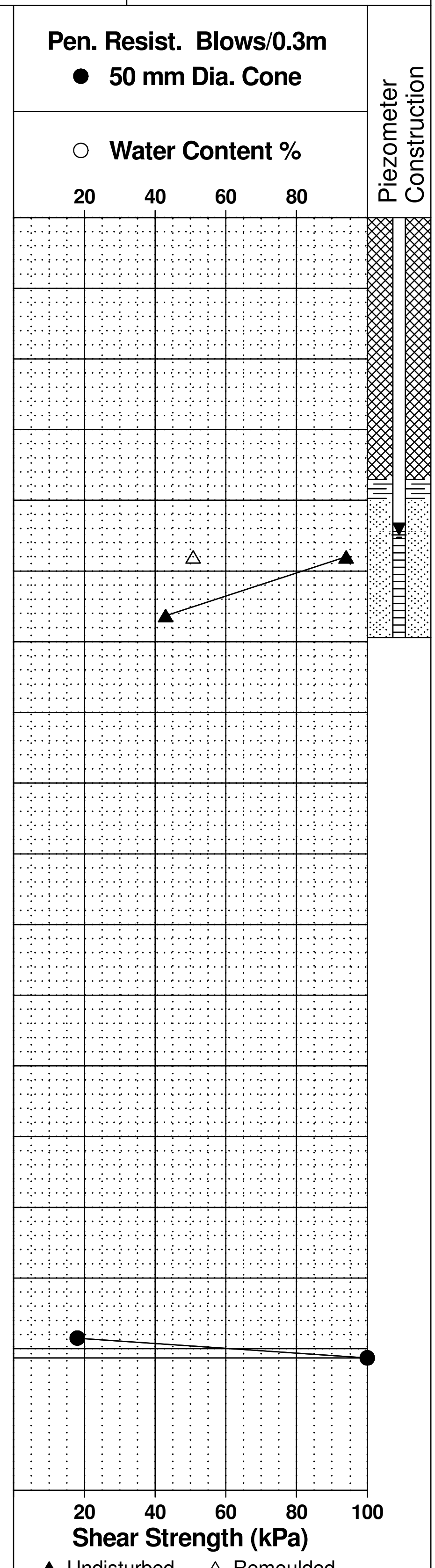
DATE June 29, 2021

FILE NO. PG5862

HOLE NO. BH 5-21

SOIL DESCRIPTION	Strata depth (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
TOPSOIL	0.20				
FILL: Brown silty clay, some sand, trace gravel and organics		AU	1		
		SS	2	33	9
	2.21	SS	3	42	5
Stiff to firm, brown **SILTY CLAY,** trace sand		SS	4	100	13
		SS	5	100	4
		SS	6	100	2
- firm and grey by 5.2m depth	5.94	SS	7	100	P
Dynamic Cone Penetration Test commenced at 5.94m depth. Cone pushed to 15.5m depth.					
	16.13				
End of Borehole Practical DCPT refusal at 16.13m depth (GWL @ 4.49m - July 15, 2021)					

DEPTH (m)	ELEV. (m)
0	92.33
1	91.33
2	90.33
3	89.33
4	88.33
5	87.33
6	86.33
7	85.33
8	84.33
9	83.33
10	82.33
11	81.33
12	80.33
13	79.33
14	78.33
15	77.33
16	76.33



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 = $(D30)^2 / (D10 \times D60)$
Cu	-	Uniformity coefficient = D60 / D10

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

CONSOLIDATION TEST

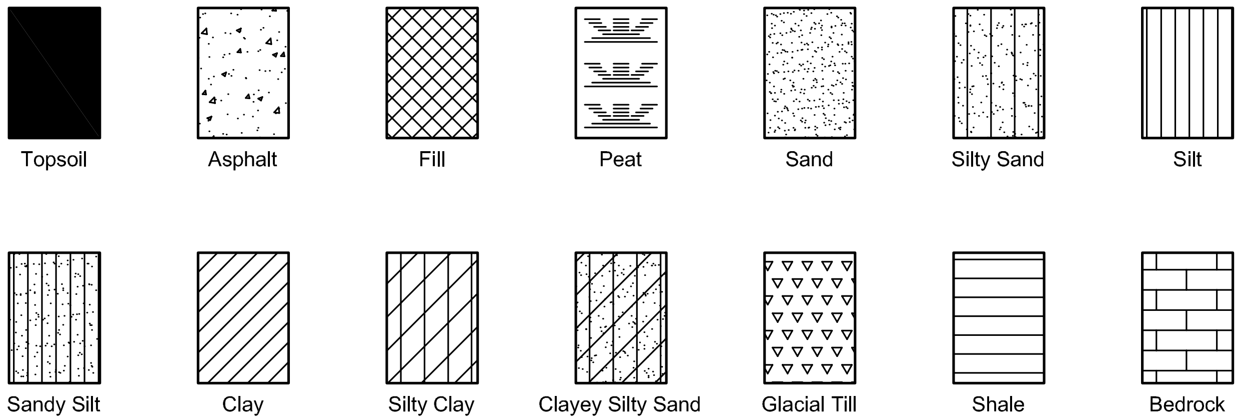
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void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

PERMEABILITY TEST

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

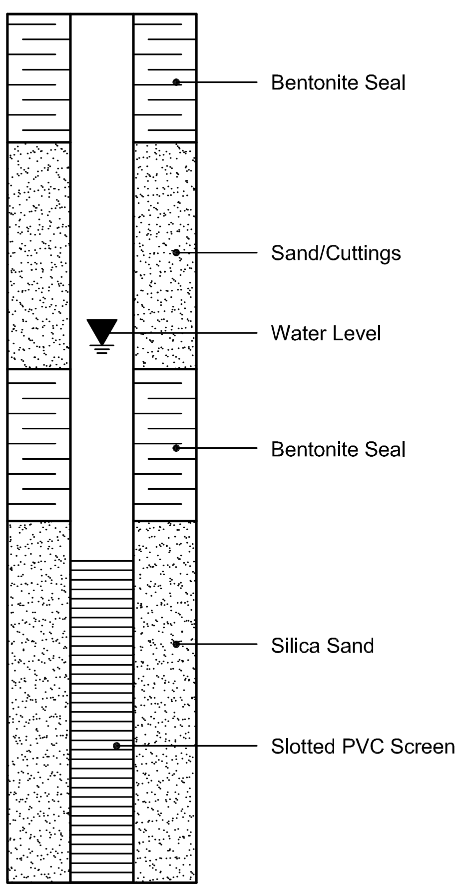
SYMBOLS AND TERMS (continued)

STRATA PLOT

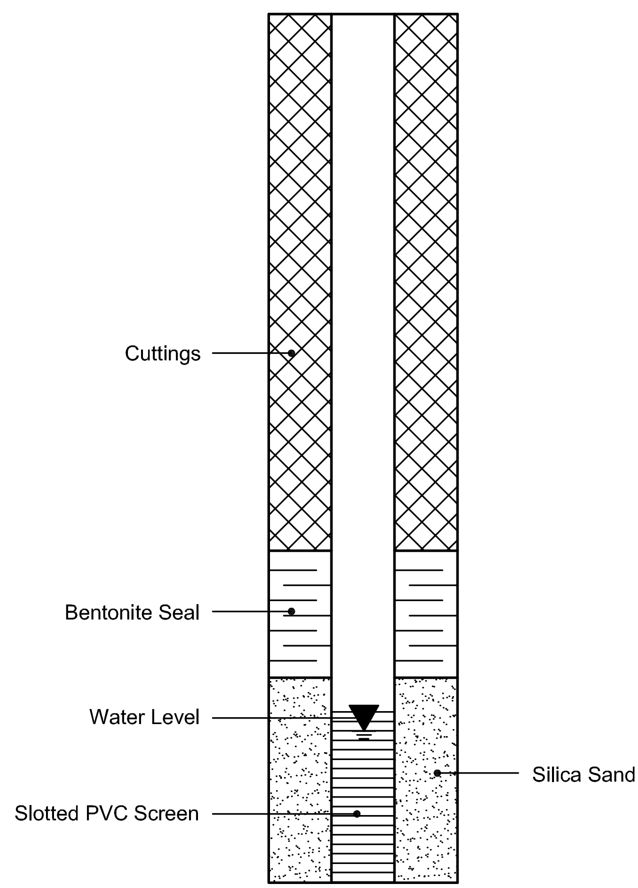


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Certificate of Analysis

Client: **Paterson Group Consulting Engineers**

Client PO: **32280**

Report Date: 05-Jul-2021

Order Date: 28-Jun-2021

Project Description: PG5862

	Client ID:	BH2-21 SS4	-	-	-
	Sample Date:	25-Jun-21 00:00	-	-	-
	Sample ID:	2127166-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	71.5	-	-	-
General Inorganics					
pH	0.05 pH Units	7.50	-	-	-
Resistivity	0.10 Ohm.m	161	-	-	-
Anions					
Chloride	5 ug/g dry	<5	-	-	-
Sulphate	5 ug/g dry	<5	-	-	-

APPENDIX 2

FIGURE 1 - KEY PLAN

FIGURES 2 & 3 - SECTIONS FOR SLOPE STABILITY ANALYSIS

DRAWING PG5862-1 - TEST HOLE LOCATION PLAN

PHOTOGRAPHS TAKEN DURING SITE VISIT

FIGURE 1

KEY PLAN

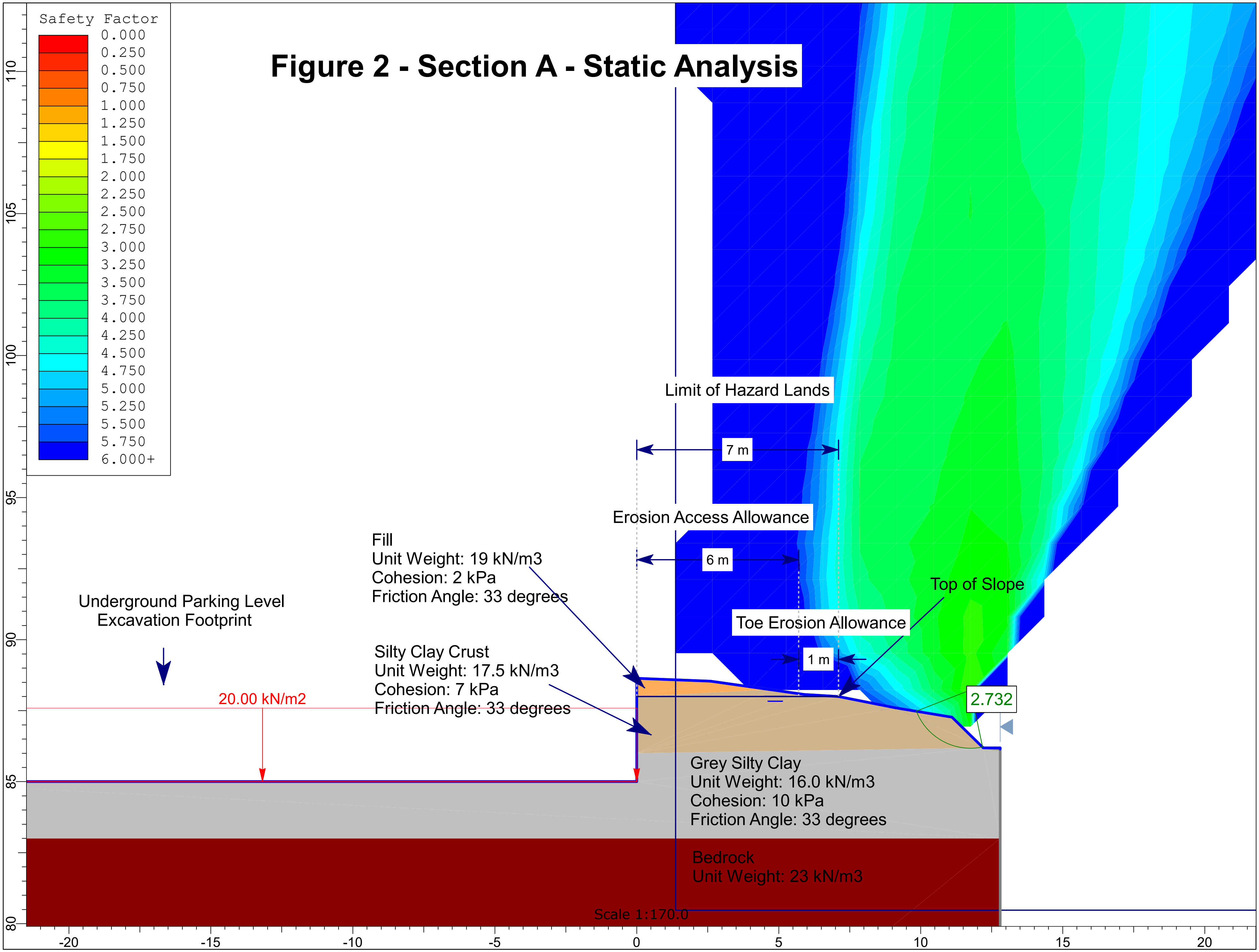
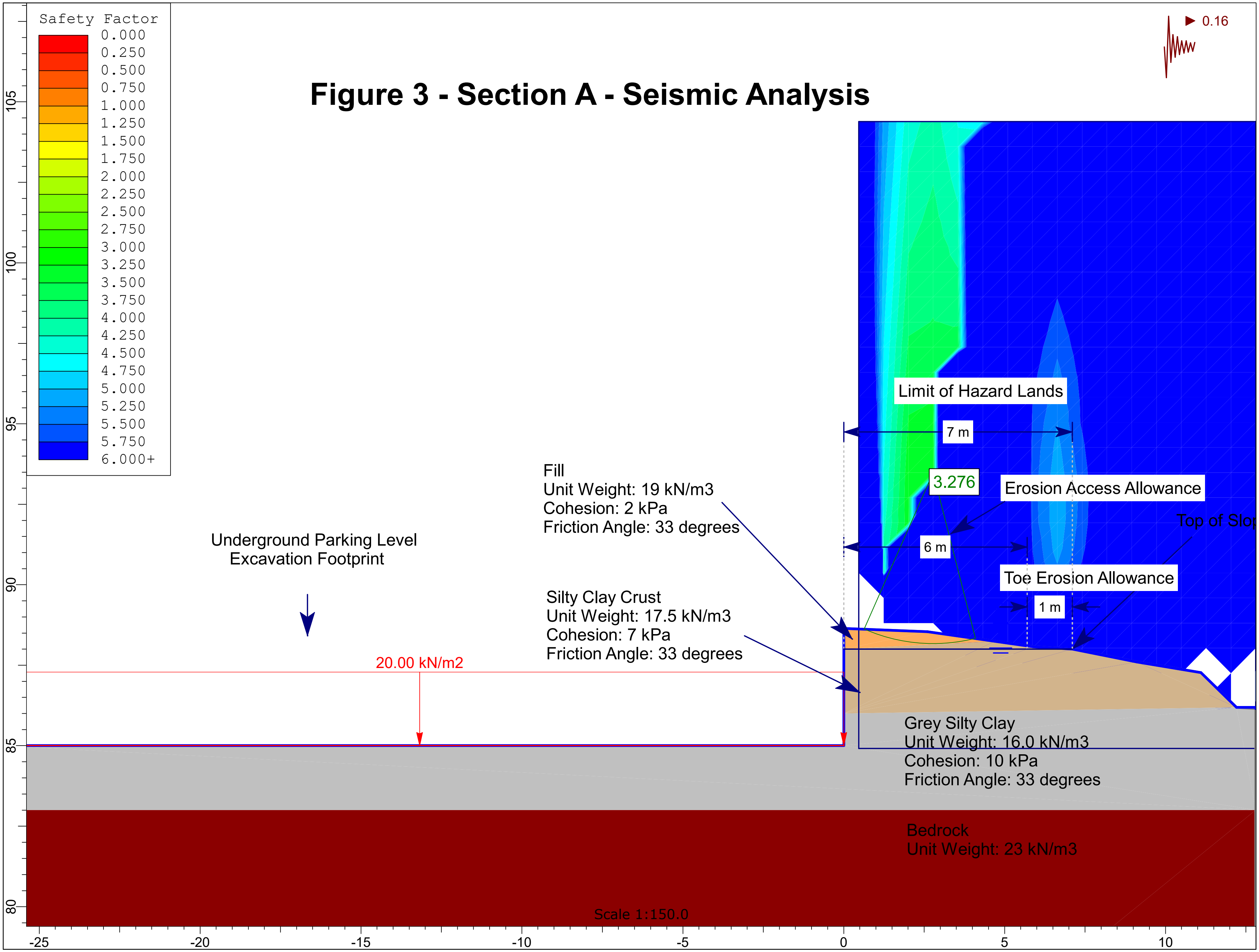
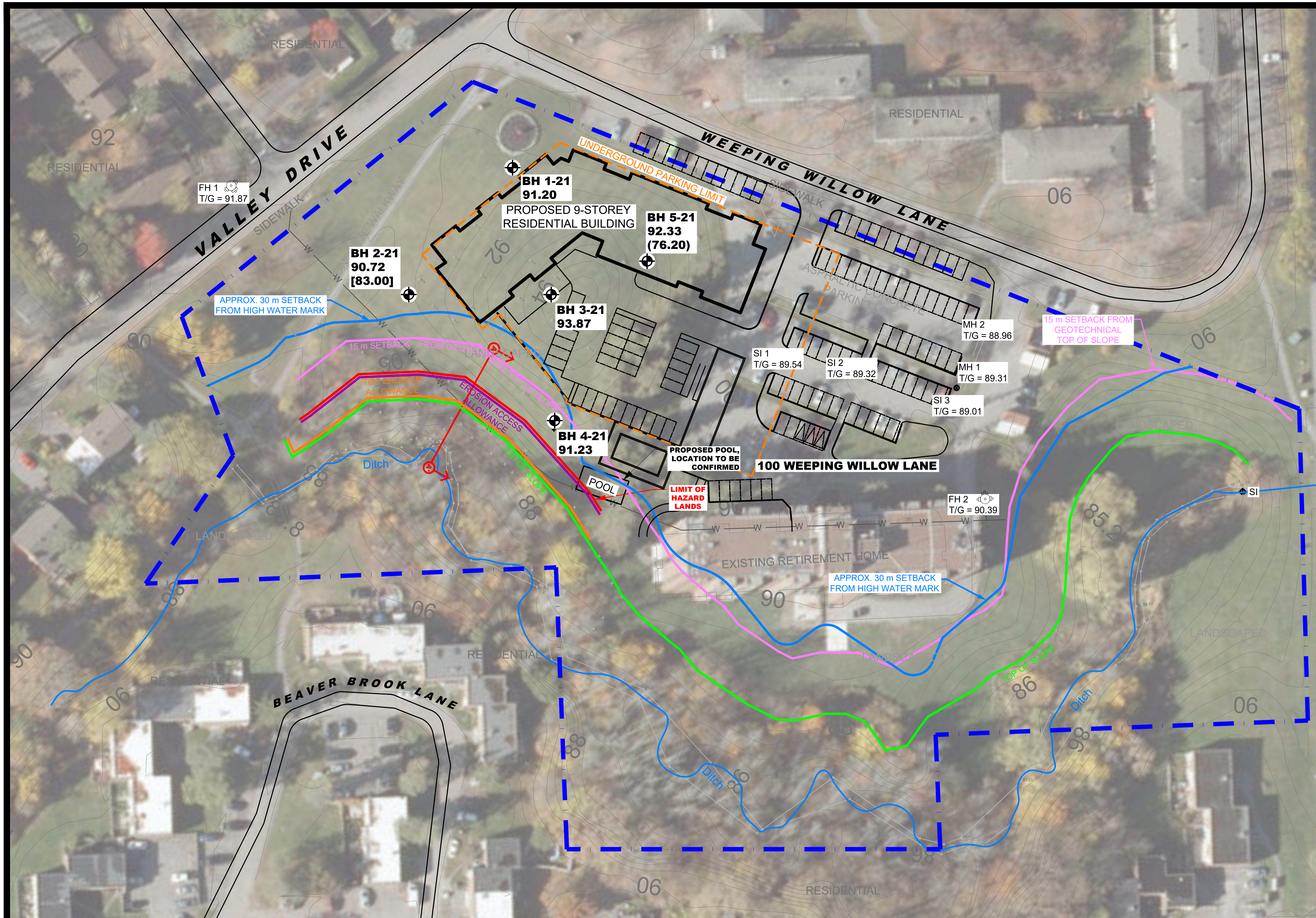


Figure 2 - Section A - Static Analysis
Safety Factor
0.000
0.250
0.500
0.750
1.000
1.250
1.500
1.750
2.000
2.250
2.500
2.750
3.000
3.250
3.500
3.750
4.000
4.250
4.500
4.750
5.000
5.250
5.500
5.750
6.000+
Limit of Hazard Lands
7 m
Erosion Access Allowance
6 m
Fill
Unit Weight: 19 kN/m3
Cohesion: 2 kPa
Friction Angle: 33 degrees
Top of Slope
Toe Erosion Allowance
1 m
2.732
Underground Parking Level
Excavation Footprint
20.00 kN/m2
Silty Clay Crust
Unit Weight: 17.5 kN/m3
Cohesion: 7 kPa
Friction Angle: 33 degrees
Grey Silty Clay
Unit Weight: 16.0 kN/m3
Cohesion: 10 kPa
Friction Angle: 33 degrees
Bedrock
Unit Weight: 23 kN/m3
Scale 1:170.0
110
105
100
95
90
85
80
-20
-15
-10
-5
0
5
10
15
20

Figure 3 - Section A - Seismic Analysis





LEGEND:

⊕	BOREHOLE LOCATION
A—A	SLOPE STABILITY SECTION
90.72	GROUND SURFACE ELEVATION (m)
[83.10]	BEDROCK SURFACE ELEVATION (m)
(76.20)	PRACTICAL REFUSAL TO DCPT ELEVATION (m)

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

30 m SETBACK FROM HIGH WATER MAKE BASED ON AVAILABLE DATA POINTS PROVIDED BY ANNIS O'SULLIVAN VOLBEKK LTD. WHERE NO DATA POINTS WERE AVAILABLE, 30 m SETBACK WAS DRAWN FROM EDGE OF CREEK

15 m SETBACK BASED ON PATERSON'S GEOTECHNICAL TOP OF SLOPE.

SCALE: 1:1000

0 10 20 30 40 50m 75m

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

NO.	REVISIONS	DATE	INITIAL
1	UPDATED CONCEPTUAL PLAN	04/08/2023	SD

HOMESTEAD LAND HOLDINGS
GEOTECHNICAL INVESTIGATION
PROPOSED MULTI-STOREY DEVELOPMENT
100 WEEPING WILLOW LANE

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:1000	Date: 06/2021
Drawn by: JM	Report No.: PG5862-1
Checked by: KP	Dwg. No.: PG5862-1
Approved by: SD	Revision No.:

p:\autocad drawings\geotechnical\pg58xx\pg5862\pg5862-1 thlp for slope analysis (rev.01).dwg

11x17

Photographs from Site Visit – July 15, 2021

Photo 1: Photograph taken of the slope face along the creek bed at southern limits of site, facing west. The slope was observed to be well vegetated. Signs of historical erosion such as oversteepening of the bank was noted.

Photo 2: Photograph taken of the slope face along the creek bed at southern limits of site, facing east. The slope was observed to be well vegetated with signs of historical erosion such as oversteepening of the banks

Photo 3: Photograph taken looking up the slope face along the creek, facing north. The slope face was observed to be well vegetated with mature trees. No visible signs of erosion were noted.

Photo 4: Photograph taken of bedrock outcropping located along the creek bed at the western limits of the subject site, face northeast. The slope face along this section of the creek was observed to be well vegetated with mature trees.