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PG6557-LET.01 Revision 4

Myers Automotive Group
1200 Baseline Road
Ottawa, Ontario
K2C 0A6

Attention: **David Johnston**

Subject: **Preliminary Geotechnical Investigation**
Proposed Commercial Development
2175 Prince of Wales Drive – Ottawa, Ontario

Dear David,

Further to your request, Paterson Group (Paterson) completed a preliminary geotechnical investigation at the aforementioned site. (Reference should be made to Figure 1 – Key Plan in the attachment of the current report). The current letter report presents the results of the geotechnical investigation and provides preliminary foundation design information and construction recommendations from a geotechnical perspective.

1.0 Proposed Development

At this time, concept plans are not available for the potential development of the subject site. However, Paterson understands there is a likelihood for the subject site to be developed to support a low- or mid-rise structure in close proximity to Prince of Wales Drive while the remainder of the subject site consist of either paved parking areas and access lanes, or landscaping. It is further understood that the proposed buildings will be connected to existing municipal services.

2.0 Field Observations

2.1 Field Investigation

The field program for the current investigation was conducted between the dates of April 6 and April 13, 2026. At that time, a total of six (6) boreholes were advanced to a maximum depth of 25.7 m below the existing ground surface. The previous field program for the investigation was conducted on May 11, 2017. At that time, a total of ten (10) test pits were excavated to a maximum depth 3.0 m below the existing ground surface.

It should be noted that previous investigations were completed by Paterson between November 2002 and May 2003 and consisted of advancing five (5) boreholes and four (4) hand augered test pits to maximum depths of 16.7 and 7.9 m below ground surface, respectively. Additionally, a previous historical geotechnical investigation for the subject site was completed on December 30, 1983, and consisted of advancing three (3) boreholes to a maximum depth of 12.2 m below ground surface.

The test holes from the supplemental investigation were distributed parallel to the top of the slope and taking into consideration site features, such as tree cover and vegetation, utilities, nearby slope conditions, comments provided by the City of Ottawa in previous rounds of development application submission and previous test hole coverage. The boreholes were completed at a maximum borehole spacing of 150 m. The supplemental 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.

Each borehole from the supplemental investigation was sampled using a combination of split-spoon samplers and MTO field vanes to measure the in-situ undrained and remoulded shear strength at the specified intervals. The portion of the deposit located beyond the grey clay layer was sampled using either split-spoons and/or split-spoons and field vanes in the brown silty clay layer.

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.

Wash-boring was implemented as of the depth which water was encountered within the test holes. Where split-spoon samples were undertaken at intervals that were previously tested by the field vane, the split-spoon sampler was pushed across the sampling interval and blow-counts were not obtained since the material was in a remoulded and disturbed state.

Split spoons within the brown clay layer were either advanced to confirm the presence of clay and underside of the overlying layer, or, to confirm the change in color from grey to brown after shear strength measurements would have been taken. The grey clay portion of the deposit was evaluated using a field vane. Vane refusal was obtained at the underside of the clay deposit which was subsequently sampled using a split-spoon sampler to confirm the underside of the clay deposit. All sampling using this methodology was undertaken at 750 mm depth intervals.

Following completion of the sampled boreholes from the supplemental investigation, an auxiliary test hole was undertaken directly adjacent to each borehole location (i.e., BH 1-26, BH 2-26 and BH 3-26) which consisted of sampling the grey portion of the clay deposit using 73 mm diameter thin-walled (TW) Shelby tubes at a 750 mm depth interval. The brown portion of the silty clay deposit was not sampled in this manner since the remoulded shear strength for this portion yields higher than able to be measured with the fall cone testing apparatus (i.e., 11 kPa).

All soil samples were initially classified on site and placed in sealed plastic bags or Shelby tubes, depending on the sampling requirements for the specified interval. Shelby tubes were sealed at both ends on site and handled in manner to minimize potential disturbances between the sample recovery and laboratory testing processes. All samples were transported by Paterson personnel to our laboratory. The depths at which the auger, split spoon and Shelby tube samples were recovered from the test holes are shown as AU and SS, and TW, respectively, on the Soil Profile and Test Data sheets appended to the present report.

All borehole locations are shown on Drawing PG6557-1 – Test Hole Location Plan, included in the appendix of the present report. The test hole locations and ground surface elevation at each test hole location were surveyed by Paterson using a high precision, handheld GPS and referenced to a geodetic datum. The subsurface conditions observed in the boreholes were recorded in detail in the field. The soil profiles are logged on the Soil Profile and Test Data sheets and attached to the present report.

2.2 Laboratory Testing

Fall Cone Remoulded Shear Strength Testing Results

Soil samples were recovered from the subject site and visually examined in our laboratory to review the results of the field logging. The middle third portion of all soil samples recovered with the Shelby tubes was extracted and two (2) samples were submitted for moisture testing, and one (1) sample was submitted for remoulded shear strength testing using the fall cone method. All samples will be stored in the laboratory for a period of one (1) month after issuance of this report. They will be discarded unless otherwise directed.

The results of the testing are discussed in the following sections and provided in the Soil Profile and Test Data sheets attached to the present report. Fall cone testing for remoulded shear strength and liquid limits, as well as moisture content testing, was undertaken in accordance with the following standards and references:

- Manufacturer Supplied Manual for Fall Cone Apparatus
- Bureau de normalization du Québec – CAN/BNQ 2501-110/2023 – Determination of Undrained Shear Strength and Sensitivity of Cohesive Soils Using a Fall Cone Penetrometer

The results of the remoulded shear strength testing are presented in Table 1 below and provided in the Soil Profile and Test Data sheets attached to the present report.

Table 1 – Summary of Fall Cone Remoulded Shear Strength Testing Results

Borehole Number	Sample Identifier	Sample Depth (m)	Undrained Shear Strength (kPa)	Remoulded Shear Strength (kPa)
BH 1-26	TW 1	6.1 – 6.7	N/A*	2.5
BH 1-26	TW 2	6.9 – 7.5	48	2.5
BH 1-26	TW 3	7.6 – 8.2	68	2.7
BH 3-26	TW 1	6.9 – 7.5	56	4.3
BH 3-26	TW 2	7.6 – 8.2	58	2.8
BH 3-26	TW 3	8.4 – 9.0	43	3.7
BH 5-26	TW 1	3.0 – 3.7	62	5.6
BH 5-26	TW 2	3.8 – 4.4	149	10.0
BH 5-26	TW 4	5.3 – 5.94	129	2.8
BH 6-26	TW 1	7.6 – 8.2	62	>11.0
BH 6-26	TW 2	8.4 – 9.0	62	3.3
BH 6-26	TW 3	9.1 – 9.8	62	6.5

Note: All remoulded shear strength values were measured using a fall cone and were taken on samples recovered from the middle third of the associated thin wall tube. All undrained shear strength measurements were measured using an MTO "B" or "N" field vane at the time of completing the boreholes. "*" refers to the interval being sampled with a split-spoon sampler such that a field vane could not be used at the time of the investigation.

In summary, the fall cone testing yielded remoulded shear strengths ranging between 2.5 kPa and greater than 11 kPa (11 kPa being the highest remould strength capable of being measured by the fall cone apparatus). No fall cone tests yielded less than 2.5 kPa of remoulded shear strength. All remaining samples (i.e., one third of the samples that were not extracted from the recovered Shelby tubes) remain sealed and will be stored in the laboratory for a period of one (1) month after issuance of this report. They will be discarded unless otherwise directed.

2.3 Surface Conditions

The subject site is undeveloped and generally covered with grass. The subject site is bordered by a treed ravine around a tributary to the Rideau River to the north, the Rideau River to the east, Waterbend Lane, followed by residential dwellings to the south, and Prince of Wales Drive to the west. The ground surface slopes gradually downward to the east towards the Rideau River. Paterson has conducted several rounds of field reconnaissance along the Rideau River as part of the slope stability assessment portion of this report, which is detailed further in that section.

2.4 Subsurface Soil Profile

Overburden

The subsurface profile encountered across the site generally consisted of topsoil underlain by variable amounts of silty sand, silty clay, sandy silt, and sand and further underlain by glacial till.

The uppermost native soils generally consisted of either silty sand or silty clay. Silty sand was encountered directly below the topsoil in TP 1-17, TP 2-17, TP 4-17, TP 5-17, TP 7-17, BH 3-03, BH 4-03, BH 5-03, HA 1, HA 3, and the 2026 boreholes BH 1-26 through BH 5-26. The silty sand layer generally extended to depths ranging from approximately 0.5 to 3.0 m below ground surface. In several locations, including BH 1-26, BH 3-26, and BH 4-26, the silty sand was underlain by brown sand deposits.

A silty clay deposit was encountered directly below the topsoil in TP 3-17, TP 6-17, TP 8-17, TP 9-17, TP 10-17, BH 1-02, BH 2-03, HA 2, and HA 4. The silty clay layer generally extended to depths ranging from approximately 2.7 to 6.0 m below ground surface. An intermittent layer of silty clay was encountered below the silty sand and/or sand layers at boreholes BH 1, BH 4-03, BH 5-03, BH 1-26, BH 2-26, BH 3-26, and BH 5-26. The intermittent silty clay layers generally extended to depths ranging from approximately 3.4 to 12.3 m below ground surface. The deposit was generally brown in colour, however grey silty clay was encountered locally in TP 1-27, BH 1, BH 5-03, HA 1, HA 4 and BH 1-26. The silty clay layer was observed to be underlain by granular soils consisting of silty sand, sandy silt, or sand in all test holes that were advanced past the silty clay deposit.

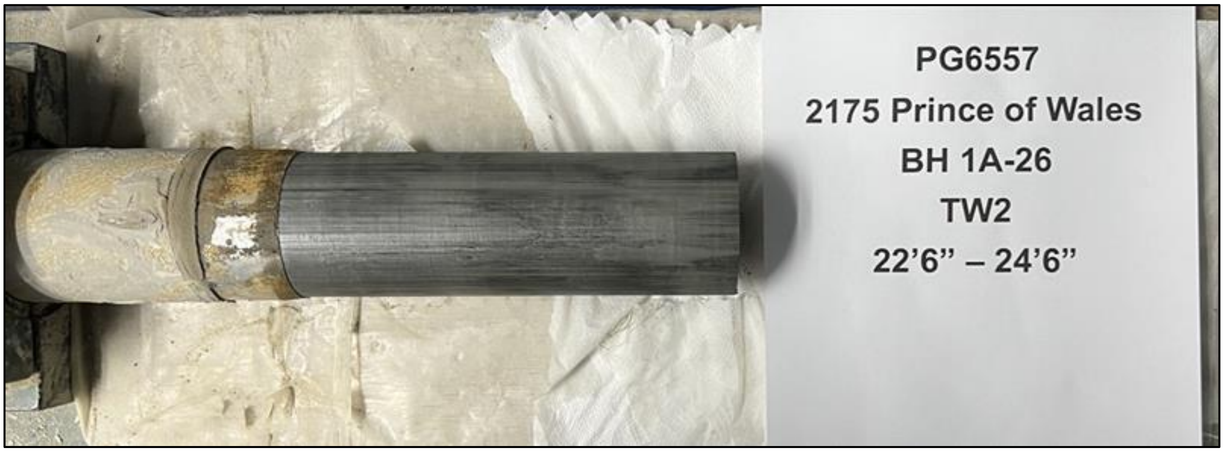
Localized sandy silt and clayey silt deposits were encountered intermittently throughout the investigated profile. Sandy silt was observed in BH 1-02, BH 1-26, BH 2-26, and BH 3-26, typically underlying silty clay deposits. Clayey silt was encountered in BH 5-26 between the silty clay and sand deposits. Interbedded layers of sandy silt, silty sand, and silty clay were also noted in the historical boreholes BH 1, BH 2, BH 3.

Sand deposits were encountered in several boreholes at intermediate to greater depths. Brown sand was observed in BH 1-26, BH 3-26, BH 4-26, and BH 5-26, while grey sand was encountered in BH 1, BH 1-26, and BH 4-26. These deposits were generally encountered below depths of approximately 4 to 11 m below ground surface and extended to the termination depth of several boreholes. Silty sand deposits were also encountered interbedded within the sand layers at numerous locations.

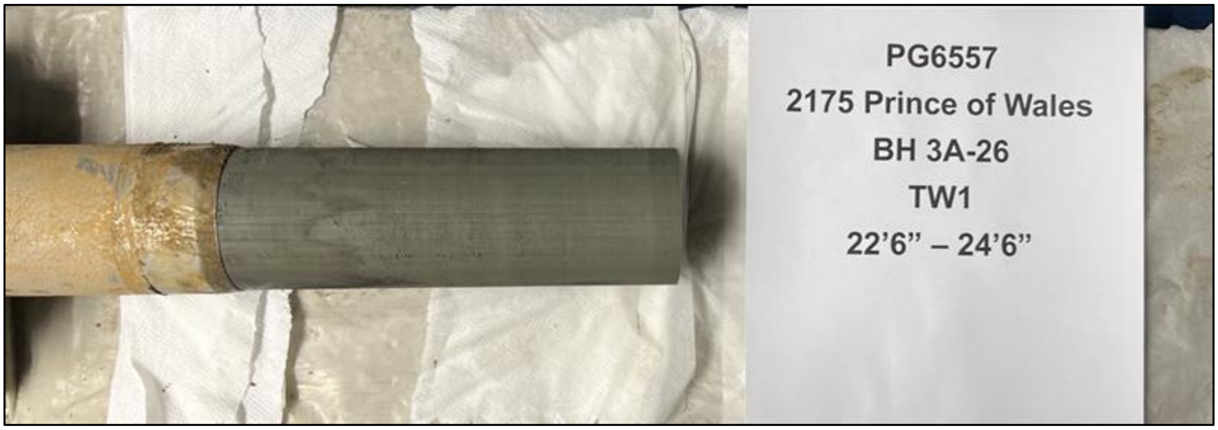
A glacial till deposit was encountered at the 2026 boreholes below the overburden soils. The glacial till deposit generally extended to depths ranging from approximately 12.9 to 25.7 m below ground surface.

Reference should be made to the Soil Profile and Test Data sheets attached to this present report for specific details of the soil profiles encountered at each test hole location.

Below are select photographs taken of samples of the in-situ silty clay soils from the recovered Shelby tubes:



Photograph 1 – Sample TW 2 from BH 1-26



Photograph 2 – Sample TW 1 from BH 3-26

Photograph 3 – Sample TW 3 from BH 3-26

Photograph 4 – Sample TW2 from BH 5-26

Photograph 5 – Sample TW1 from BH 6-26

Photograph 6 – Sample TW3 from BH 6-26

Bedrock

Based on available geological mapping, the bedrock surface in this area is encountered at depths varying between 15 and 25 m and consists of dolomite of the Oxford formation.

Groundwater

Groundwater infiltration levels were observed within the open test pit locations, as well as measured within the standpipe piezometer installed within the 2026 boreholes. The groundwater infiltration levels and groundwater measured at the 2026 borehole locations on April 24, 2026, and are presented in the Soil Profile and Test Data sheets attached.

Field Conditions

The subject site is relatively flat with a gradual slope (approximately 10H:1V) inclining downwards in a west to orientation along the eastern third of the property. The property is vacant and surfaced with grass, with the exception of the area supporting a free-standing billboard sign structure. The property borders Prince of Wales Drive to the west, partially Waterbend Lane to the south, the Rideau River to the south and east and a tributary to the Rideau River to the north.

The portion bordering the Rideau River consists of an approximately 6 to 10 m high slope inclined at approximately 1.5H:1V. The portion of the site bordering the northern tributary consists of a 6 to 9 m high slope with inclination ranging approximately between 0.8H:1V to 2.5H:1V.

The tributary formed part of a historically longer tributary which previously extended across Prince of Wales Drive and the current Antares Business Park and is visible on historical aerial imagery available through GeoOttawa. The headwall at the base of Prince of Wales Drive appears to receive surface water flows from sewer infrastructure for the same area that had been occupied by the previous tributary and additional segments of sewer systems. The subject tributary had not been backfilled and converted to a sewer alignment as the remainder of the historical tributary had been.

Surface water discharging from the stormwater sewer to the tributary contributes significantly to the existing and on-going active erosion. The area immediately beyond the headwall structure had been enhanced with erosion protection consisting of constructing rock protection in either 2018 or 2019 (and as recommended to be undertaken by Paterson under separate cover in 2017). However, the remainder of the area between the headwall and Rideau River remains similar to the historical condition and continues to be subject to erosion along the length of the tributary. Paterson completed several rounds of field reviews amongst the subject slopes, including April 2026, May 2025, October 2024, April 2017 and June 2009. Findings from these reviews do not indicate major alterations to the slopes with the exception of the erosion protection enhancement efforts in 2017/2018 at the headwall structure.

Active erosion is also present along the Rideau River. Based on our field reviews, the face of the sidewalls that are subject to erosion along the Rideau River side of the property is compact to dense sand to sandy silt, whereas it consists of either very stiff to stiff silty clay or compact to dense sand to sandy silty along the tributary.

Beyond the base of the slopes, the slopes are covered with vegetation consisting of mature trees and brush. The portion of the slopes in close contact with the water features are less vegetated due to the active erosion, with some undercutting present where the clay soils are present at the interface. Photographs of these visits are attached to the present report.

3.0 Geotechnical Discussion and Construction Precautions

3.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is considered suitable for the proposed development. It is expected that the proposed commercial buildings will consist of slab-on-grade construction and be supported by conventional spread footing foundations placed on an undisturbed compact silty sand or stiff silty clay bearing surface. Due to the presence of the silty clay deposit, a permissible grade raise restriction is required for the subject site.

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

3.2 Site Grading and Preparation

Topsoil and any deleterious fill, containing organics and/or construction debris, should be stripped from under any buildings, paved areas, pipe bedding and other settlement sensitive structures. Care should be taken not to disturb adequate bearing surfaces during site preparation activities.

3.3 Fill Placement

Engineered fill used for grading beneath the proposed building, where required, should consist of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. This material should be tested and approved prior to delivery to the site. The fill should be placed in a maximum 300 mm thick loose lifts and compacted using suitable compaction equipment for the lift thickness. Fill placed beneath the building should be compacted to at least 98% of the material’s Standard Proctor Maximum Dry Density (SPMDD).

Site-excavated soil can be used as general landscaping fill where settlement of the ground surface is of minor concern. The soil should be spread in thin lifts and at least compacted by the tracks of the spreading equipment to minimize voids and approved by Paterson personnel.

If the soil is to be used to build up the subgrade level for areas to be paved, it should be compacted in thin lifts to a minimum 95% of its SPMDD. The material should be placed under dry conditions and above freezing temperatures. Non-specified existing fill and site-excavated soils are not suitable for use as backfill against foundation walls unless used in conjunction with a composite drainage membrane.

3.4 Foundation Design

Strip footings, up to 3 m wide, and pad footings, up to 6 m wide, founded on an undisturbed stiff silty clay bearing surface can be designed using a bearing resistance value at serviceability limit states (SLS) of **150 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **225 kPa** incorporating a geotechnical resistance factor of 0.5 at ULS.

Footings placed on an undisturbed, compact silty sand bearing surface can be designed using a bearing resistance value at SLS of **150 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **225 kPa**.

An undisturbed soil bearing surface consists of one 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 resistance value at SLS will be subjected to potential post-construction total and differential settlements of 25 and 20 mm, respectively.

Permissible Grade Raise Restrictions

Due to the presence of the underlying silty clay layer, **a permissible grade raise restriction of 2.0 m** is recommended in the immediate area of settlement sensitive structures. 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.

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. Above the groundwater level, adequate lateral support is provided to a stiff silty clay or compact silty sand bearing medium 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.

3.5 Design for Earthquakes

The site class for seismic site response can be taken as **Site Designation X_D** for foundations constructed at the subject site in accordance with the 2024 Ontario Building Code (OBC 2024). If a higher seismic site class is required, a site-specific shear wave velocity test may be completed to accurately determine the applicable seismic site classification for foundation design of the proposed buildings.

Based on Paterson's review of the in-situ soils compactness and stiffness for non-cohesive and cohesive soils, respectively, the soils underlying the subject site are not considered susceptible to liquefaction or cyclic softening. Reference should be made to the latest revision of the Ontario Building Code for a full discussion of the earthquake design requirements.

3.6 Slab-On-Grade Construction

With the removal of all topsoil and deleterious materials within the footprint of the proposed buildings, the native soil surface, approved by Paterson personnel at the time of construction, will be considered an acceptable subgrade on which to commence backfilling for floor slab construction. The upper 200 mm of sub-slab fill is recommended to consist of OPSS Granular A crushed stone. All backfill material within the footprints of the proposed buildings should be placed in maximum 300 mm thick loose lifts and compacted to a minimum of 98% of its SPMDD. Any soft areas should be removed and backfilled with OPSS Granular B Type II, with a maximum particle size of 50 mm and compacted to 98% of the material's SPMDD.

3.7 Pavement Structure

If required, the pavement structure for car only parking, access lanes and heavy truck parking is presented in Table 2 and Table 3 below.

Table 2 - Recommended Pavement Structure - Driveways and Car Only Parking

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

Table 3 - Recommended Pavement Structure - Access Lane 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 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
400	**SUBBASE** - OPSS Granular B Type II
SUBGRADE - Either approved fill, in situ soil, or OPSS Granular B Type I or II material placed over in situ soil or fill.	

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 I or 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

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

Where silty clay is anticipated at subgrade level, consideration should be given to installing subdrains during the pavement construction. These drains should extend in four orthogonal directions or longitudinally when placed along a curb. The clear crushed stone surrounding the drainage lines or the pipe 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. Discharge of the subdrains should be directed by gravity to storm sewers or deeper drainage ditches.

3.8 Foundation Drainage

Since the buildings will consist of a slab-on-grade construction, a perimeter foundation drainage system is considered optional throughout the landscaped portions of the proposed building footprints. In areas where hardscaping or pavement structures will abut the building footprints, it is recommended to implement a foundation drainage system.

The system should consist of a minimum 100 to 150 mm diameter perforated corrugated plastic pipe wrapped in a geosock and surrounded on all sides by 150 mm of minimum 10 mm clear crushed stone. The clear stone should be wrapped in a non-woven geotextile. The pipe should have a positive outlet, such as a gravity connection to the storm sewer.

The pipe should be placed at the footing level around the exterior perimeter of the structure if the backfill between the founding depth and will consist of crushed stone fill or site-generated soil backfill in conjunction with a composite foundation drainage board, such as CCW MiraDRAIN 2000 or Delta-Teraxx. Alternatively, the perimeter drainage pipe may be placed up to 1 m below proposed finished grade and against the building footprint upon approved soil backfill to ensure adequate drainage of the granular fill layer is provided from precipitation events and/or spring meltwater. In this configuration, provided the backfill overlying the pipe consists of crushed stone fill associated with the pavement structure, a composite foundation drainage board will not be required in these areas.

3.9 Foundation Backfill

Backfill against the exterior sides of the foundation walls should consist of free-draining, non-frost susceptible imported crushed stone or clean sand fill. Alternatively, consideration may be given to placing site-generated soil fill as backfill against the foundation walls provided the material is compacted in 300 mm thick loose lifts and provided the foundation wall is covered with a composite foundation drainage board layer and associated perimeter drainage pipe with a gravity outlet. If the building's perimeter drainage pipe is located at footing level, a composite foundation drainage board should be placed against the foundation walls to ensure satisfactory drainage of the backfill layer to the perimeter drainage pipe.

All fill placed as foundation backfill should be placed in maximum 300 mm thick loose lifts, compacted using suitable compaction equipment (suitably sized smooth-drum roller for crushed stone fill, sheepsfoot roller for soil fill) and tested for compaction efforts at the time of construction by Paterson personnel.

3.10 Protection of Footings Against Frost Action

Perimeter footings of heated structures are required to be insulated against the deleterious effect of frost action. A minimum of 1.5 m thick soil cover should be provided for adequate frost protection of heated structures, or an equivalent combination of soil cover and foundation insulation.

Other exterior unheated footings, such as those for isolated exterior piers are more prone to deleterious movement associated with frost action than the exterior walls of the structure proper and require additional protection. These should be provided with a minimum 2.1 m thick soil cover or a combination of soil cover and foundation insulation.

3.11 Excavation Side Slopes

The side slopes of excavations in the soil and fill overburden materials should be either cut back to acceptable slopes or should be retained by shoring systems from the start of the excavation until the structure is backfilled. It is expected that sufficient room will be available for the greater part of the excavation to be undertaken by open-cut methods (i.e. unsupported excavations).

Unsupported Excavations

The excavation side slopes above the groundwater level extending to a maximum depth of 3 m should be cut back to 1H:1V or flatter. The flatter slope is required for excavation below groundwater level. The subsurface soils are considered to be Type 2 and Type 3 soil according to the Occupational Health and Safety Act and Regulations. 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 Paterson field personnel in order to detect if the slopes are exhibiting signs of distress.

Excavation side slopes around the building excavation should be protected from erosion by surface water and rainfall events by the use of secured tarpaulins spanning the length of the side slopes, or other means of erosion protection along their footprint. Efforts should also be made to maintain dry surfaces at the bottom of the excavation footprints and along the bottom of side slopes. Additional measures may be recommended at the time of construction by the geotechnical consultant.

A trench box is recommended 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.

3.12 Winter Construction

If winter construction is considered for this project, precautions should be provided for frost protection. The subsurface soil conditions mainly 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 installation of straw, propane heaters and tarpaulins or other suitable means. The excavation base 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.

3.13 Corrosion Potential and Sulphate

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

4.0 Geotechnical Slope Stability Analysis

4.1 Existing Slope Conditions and Subsoil Information

The south valley corridor wall of the drainage ravine along the north property boundary was noted to be vegetated with small brush and signs of active erosion occurring at several isolated locations within the watercourse/creek channel. A 2 to 3 m wide watercourse with water depths varying between 0.2 to 0.3 m in depth was noted to meander through the bottom of the valley corridor.

Along the east property boundary, the west valley corridor wall of the Rideau River is undergoing some active erosion with some subsequent undercutting of the toe of the slope. It is expected that historical erosional activities have resulted in a relatively steep slope along the subject section of the Rideau River. At the present time, the majority of banks are vegetated with brush and mature trees. It should be noted that a slope remedial program was initiated near the southeast corner of the subject site during the summer of 2003 and consisted of modifying the existing slope and reinstating it with blast rock. Reference should be made to Section C present in Figure 3A and Figure 3B, which is located on our attached drawing, PG6557-1 – Test Hole Location Plan.

The subsurface soil profile used for the slope stability analysis was based on the existing test hole information and available geological mapping in the immediate area of the subject site. Reference should be made to Subsection 2.4 of the present report for specific subsurface soil conditions.

4.2 Slope Stability Analysis

Following completion of the supplemental 2026 test hole investigation, Paterson revised the previous slope stability assessment to consider the most recent subsoils information. The slope stability analysis was completed using the most recently available topographic survey and LiDAR data while considering slope condition reviews conducted by Paterson field personnel.

Three cross-sections (Section A, Section B, and Section E) were studied as worst-case scenarios. “Worst-case” scenario implies the presence of factors such as steepness, height and/or subsoils conditions that impact the stability of the slope and would represent other areas that are similar in those properties but are more conservatively characterized by the those present at the “worst-case” location. Due to the proximity of the former slope failure located near the southeast corner of the site, a fourth cross-section (Section C) was analyzed as part of the slope stability analysis. Further, one additional cross-section was studied to review the stability of notable steepness (Section D). The cross-section locations are presented on Drawing PG6557-1 – Test Hole Location Plan attached to the present report.

The selected soil strength parameters utilized in the slope stability analysis were established based on our review and interpretation of all available site-specific subsurface information, including in-situ test hole observations, field testing, and available subsurface data from the current and previous geotechnical investigations completed within and adjacent to the subject site. The interpretation considered the encountered soil stratigraphy, groundwater observations, and the consistency of the native materials encountered across the site. The adopted soil parameters were selected to conservatively represent the interpreted site conditions and are considered representative of the governing materials influencing overall slope performance.

The slope stability analysis was carried out using SLIDE, a computer program which permits a two-dimensional slope stability analysis using several methods including 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 favoring 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 variability of the subsoil and groundwater conditions, a factor or safety greater than one is usually required to ascertain the risks of failure is 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 following analysis was undertaken in accordance and considering the guidance provided in the City of Ottawa Slope Stability Guidelines for Development Applications (2004) and the Ontario Ministry of Natural Resources Technical Guide – River and Stream Systems: Erosion Hazard Limit (2002).

Static Loading

The results of the existing slope conditions under static loading at Sections A, B, C, D, and E, are shown in Figures 1A, 2A, 3A, 4A, and 5A, respectively, and are attached to the present report. The overall slope stability factors of safety for the subject sections were found to be less than 1.5. The stable slope allowance from the top of slope required for slopes with a minimum factor of safety of 1.5 is identified for each profile in the attached figures.

Reference should be made to Table 4 below for a summary of the applicable setbacks applied under static loading conditions that would be considered in the resulting Limit of Hazard Lands designation line that would be advised at each slope cross-section location:

Table 4 – Summary of Static Analysis Stable Slope Allowance Setback

Section	Stable Slope Allowance (m)
A	12.5
B	8.0
C	1.0
D	7.4
E	6.4

Seismic Loading

An analysis considering seismic loading was also completed as part of the slope stability analysis. A horizontal seismic loading acceleration, K_h, of 0.186g was considered for the analyzed sections. A factor of safety of 1.1 is considered to be satisfactory for stability analysis including seismic loading. The results of the analysis including seismic loading, are shown in Figures 1B, 2B, 3B, 4B, and 5B for the slope sections. Where the minimum factor of safety is less than 1.1, the stable slope allowance from the top of slope required for the slope section is identified in the attached figures.

Reference should be made to Table 5 below for a summary of the applicable setbacks applied under seismic loading conditions that would be considered in the resulting Limit of Hazard Lands designation line that would be advised at each slope cross-section location:

Table 5 – Summary of Seismic Analysis Stable Slope Allowance Setback

Section	Stable Slope Allowance (m)
A	8.5
B	12.7
C	n/a
D	n/a
E	1.9

The associated Limits of Hazard Lands designation line considering these allowances are summarized in *Section 4.4 – Limit of Hazard Lands Designation* in this present report. Reference should be made to the slope stability analysis figures attached to the present report for additional information associated with the resulting analysis.

4.3 Susceptibility to Retrogressive Landslides

The supplemental 2026 investigation described in previous sections of the present report was undertaken in response to the City of Ottawa's Technical Guidance letter dated August 15, 2025. This Guidance letter recommends the following lab testing criteria for determining landslide susceptibility: clay soils with a remoulded shear strength less than 1.6 kPa and liquidity index greater than 1.0 measured using the fall cone testing methods.

Paterson undertook fall cone testing on all recovered Shelby tube samples containing clayey soils. In summary, the fall cone testing yielded remoulded shear strengths ranging between 2.5 kPa and greater than 11 kPa (11 kPa being the highest remould strength capable of being measured by the fall cone apparatus), and an average of 4 kPa.

The City of Ottawa's guidance advises that clays yielding a remoulded shear strength of less than 1.6 kPa (in conjunction with applicable liquidity indices) are susceptible to retrogressive landslides. While Atterberg limits testing was not undertaken, the remoulded shear strengths yielded higher than the advised threshold such that the subject clay soils are not considered to be susceptible to retrogressive landslides.

The likelihood for a retrogressive failure is further reduced by the limited presence of clay soils throughout the subject site as opposed to other parts of the Ottawa region where historical retrogressive landslides are more concentrated. The subject site is underlain by a clay deposit that transitions from being solely a weathered, very stiff layer of desiccated crust (BH 2-26) to slightly weathered/unweathered stiff grey silty clay reaching only up to 3 m in thickness. This limited layer thickness significantly reduces the potential for a retrogressive failure to occur should active erosion continue to be permitted at the base of the subject slopes.

The undrained shear strength associated with this layer provides resistance to the layer from shearing when underlying sand supporting this layer is eroded away from below it. While these failures have occurred historically, the laboratory testing results support the lack of observed retrogressive failures resulting from previous slip failures that have been triggered by on-going erosion at the base of the slopes.

Based on this information and these observations, the subject site is not considered to be susceptible to retrogressive landslides by the presence of the encountered clay deposit.

4.4 Limit of Hazard Lands Designation

Based on our review, the Limit of Hazards Lands designation line for the subject site will consider the combination of a stable slope allowance, toe erosion allowance and erosion access allowance. At this time, concept plans are not available for the potential development of the subject site. However, Paterson understands there is a likelihood for the subject site to be developed to support a low- or mid-rise structure in close proximity to Prince of Wales Drive while the remainder of the subject site consist of either paved parking areas and access lanes, or landscaping.

Based on this, Paterson has provided the following commentary and recommendations for these allowances considering the information discussed in the previous sections of the present report and potential development concepts.

Stable Slope Allowance

The stable slope allowances presented in Table 4 are recommended to comprise the stable slope allowance portion of the proposed Limit of Hazard Land designation line.

Erosion Access Allowance

Following the guidance provided in the aforementioned technical references for slope stability analysis, a 6 m erosion access allowance would be considered suitable for planning purposes. However, Paterson also understands the majority of the subject site is expected to be planned to be surfaced with either paved parking and access lanes, or landscaped. The Ontario Ministry of Natural Resources guidance document described the erosion access allowance to follow three principles to support its inclusion in planning:

- *Providing for emergency access to erosion prone areas;*
- *Providing for construction access for regular maintenance and access to the site in the event of an erosion event or failure of a structure;*
- *Providing protection against unforeseen or predicted external conditions which could have an adverse effect on the natural conditions or processes acting on or within an erosion prone area of provincial interest;*

For the subject site, the inclusion of a 6 m erosion access allowance in areas that would be otherwise planned to be either hardscaped or landscaped by the developer is considered superfluous from a geotechnical perspective.

Should the developer plan to include permanent structures, including any type of building, within a zone that would have otherwise been within the footprint of 6 m erosion access allowance, the allowance should remain and direct the structures footprint beyond that area.

Otherwise, the inclusion of paved parking areas and access lanes, or maintained landscaping such as grass and small plants, serves the function of the allowance while enhancing the area to serve the setbacks function as opposed to leaving these areas unimproved with potentially unkept vegetation.

Based on this, Paterson has prepared Drawing PG6557-1 – Test Hole Location Plan on the basis that an erosion access allowance will be integrated into the site plan. To mitigate the risk of planning for a structure to be located within the potential 6 m erosion access allowance that would otherwise be applicable in that scenario, the potential setback is depicted to guide planning decisions and mitigate encroachment of structures within this zone.

Toe Erosion Allowance

The toe erosion allowance for the slopes was based on the nature of the soils, observed current erosional activities, and the width and location of the current watercourses. Signs of active erosion were noted in areas where the existing watercourse has meandered in close proximity to the toe of the corridor wall of the tributary watercourse located to the north of the subject site. Two slope failures occurred between 2015 and 2020 were observed along the north property boundary; These failures are also shown on Figure 6 – LiDAR Mapping Surface Elevation Changes From 2015 to 2020, attached to the present report.

However, based on Paterson's field investigations performed in April 2026, March 2025, and October 2024, it was noted that these slopes contain high amounts of vegetation, indicating the slowdown of active erosion. Furthermore, the city of Ottawa has placed riprap stone along the culvert outlet, which discharges water into the tributary along the north property boundary. The addition of riprap stone has dissipated the energy of the outflowing water, reducing the active erosion within the tributary watercourse. Therefore, a toe erosion allowance of 5 m is considered appropriate for the tributary watercourse to the Rideau River.

Some erosional activities were noted along the toe of the subject valley corridor wall of the Rideau River. Furthermore, one slope failure was noted along the south property boundary, which is presented on Figure 6 – LiDAR Mapping Surface Elevation Changes From 2015 to 2020 attached to the present report. As a result, it is considered that a toe erosion allowance of 8 m is appropriate for the subject slope along the Rideau River.

Future Erosion Protection Considerations

Future consideration could be given to implementing erosion protection measures at the base of the slopes to reduce ongoing toe erosion and the associated *Toe Erosion* setback. Such measures could include protection of the slope toe by means of MTO rip-rap stone material. Should erosion protection measures be pursued in the future, Paterson requests to complete a subsequent review of the existing slope stability analysis to confirm the implications of the proposed works.

Summary and Proposed Limits of Hazard Lands Designation

Based on Paterson review and supporting rationale, Paterson suggests the following Limits of Hazard Lands be assigned at each cross-section location from a geotechnical perspective:

Table 6 – Summary of Slope Stability Setbacks

Section	Stable Slope (m)	Toe Erosion (m)	Erosion Access (m)	Limit of Hazard Lands (m)
A	12.5	5	0 or 6*	23.5
B	8	8	0 or 6*	22
C	1	8	0 or 6*	15
D	7.4	5	0 or 6*	18.4
E	6.4	5	0 or 6*	17.4

"*" refers to this limit being reduced to zero where a building structure is not planned to be within a potential 6 m zone that would be otherwise assigned to those structures. It is anticipated this 6 m would be otherwise occupied by either hardscaped (paved access lanes and parking) or landscaped areas that would serve the function of the setback in an improved manner than the limitations imposed by the setback.

Reference should be made to Drawing *PG6557-1 – Test Hole Location Plan* depicting the above-noted geotechnical Limit of Hazard Lands setback and previously mentioned 6 m erosion access allowance.

5.0 Recommendations

A materials testing and observation services program is a requirement for the provided foundation design recommendations to be applicable. The following aspects of the program should be performed by Paterson:

- ☐ A full geotechnical investigation should be completed once conceptual details of the proposed development are available.
- ☐ Review of the grading and servicing plans from a geotechnical perspective.
- ☐ Sampling and testing of the concrete and fill materials used.
- ☐ 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 the placement of the foundation insulation, if applicable.
- ☐ Observation of all subgrades prior to backfilling.
- ☐ Field density tests to determine the level of compaction achieved.
- ☐ Sampling and testing of bituminous concrete, including mix design reviews.

A report confirming that the construction has been conducted in general accordance with Paterson's recommendations could be issued upon the completion of a satisfactory inspection program by the Paterson consultant.

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

6.0 Statement of Limitations

The recommendations provided in the report are in accordance with Paterson's present understanding of the project. Paterson requests permission to review the recommendations when the drawings and specifications are completed. The client should be aware that any information pertaining to soils and all test pit logs is furnished as a matter of general information only, and test hole descriptions or logs are not to be interpreted as descriptive of conditions at locations other than those of the test holes. A soils investigation is a limited sampling of a site. Should any conditions at the site be encountered which differ from the test locations, Paterson requests immediate notification to permit reassessment of the recommendations.

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 Myers Automotive Group or their agents is not authorized without review by Paterson Group Inc. for the applicability of our recommendations to the altered use of the report.

We trust that the current submission meets your immediate requirements.

Best Regards,

Paterson Group Inc.

LICENSED PROFESSIONAL ENGINEER
June 19, 2026
D. J. PETAHTEGOOSE
100568013
PROVINCE OF ONTARIO

Nicholas F. R. Versolato P.Eng., ing.

Drew Petahtegoose, P.Eng.

Attachments

- ☐ Photographs – From Site Visit Captured on March 18, 2025, October 8, 2024, and April 17, 2017.
- ☐ Photographs – Comparison Photos.
- ☐ Photographs – From Site Visit Captured on April 23, 2026
- ☐ Soil Profile and Test Data Sheets.
- ☐ Symbols and Terms.
- ☐ Figure 1 – Key Plan.
- ☐ Figures 1A to 5B – Slope Stability Analysis.
- ☐ Figure 6 – LiDAR Mapping Surface Elevation Changes From 2015 to 2020.
- ☐ Drawing PG6557-1 - Test Hole Location Plan.

Report Distribution

- ☐ Myers Automotive Group (e-mail copy)
- ☐ Paterson Group (1 copy)

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9 Auriga Drive
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Ottawa Laboratory
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Materials Testing ◊ Retaining Wall Design ◊ Rural Development Design
Temporary Shoring Design ◊ Building Science ◊ Noise and Vibration Studies

patersongroup.ca

Photo 1: Photograph taken in a southeast direction illustrating some evidence of active erosion.

Photo 2: Photograph taken in a northeast direction illustrating some evidence of active erosion along the west bank of the Rideau River.

Photo 3: Photograph taken in a northeast direction illustrating some evidence of toe erosion along the west bank of the Rideau River.

Photo 4: Photograph taken in a southwest direction illustrating some evidence of active erosion along the west bank of the Rideau River.

Photo 5: Photograph taken in a southwest direction illustrating some evidence of active erosion along the west bank of the Rideau River.

Photo 6: Photograph taken in a southwest direction illustrating some evidence of toe erosion along the west bank of the Rideau River.

Photo 7: Photograph taken in a northeast direction illustrating some evidence of active erosion along the west bank of the Rideau River.

Photo 8: Photograph taken in a northeast direction illustrating the placement of riprap stone around the culvert outlet that discharges into the tributary to the Rideau River.

Photo 9: Photograph taken in a south direction along the tributary to the Rideau River illustrating some evidence of historical slip failures along the south banks of the valley corridor.

Photo 10: Photograph taken in a south direction along the tributary to the Rideau River illustrating some toe erosion along the base of the valley corridor where the watercourse meets the wall of the valley corridor.

Photo 11: Photograph taken in a north direction along the tributary to the Rideau River illustrating evidence of active erosion along the north bank of the valley corridor.

Photo 12: Photograph taken in a south direction along the tributary to the Rideau River illustrating evidence of historical slip failures along the south bank of the valley corridor and evidence of active erosion where the watercourse meets the wall of the valley corridor.

Photo 13: Photograph taken in a north direction along the tributary to the Rideau River illustrating evidence of active erosion.

Photo 14: Photograph taken in a south direction along the tributary to the Rideau River illustrating evidence of historical slip failures along the south bank of the valley corridor and evidence of active erosion where the watercourse meets the wall of the valley corridor.

Photo 15: Photograph taken in an east direction over the west bank of the Rideau River showing some active erosion.

Photo 16: Photograph taken in an east direction illustrating the active erosion and subsequent slip failures adjacent to the concrete box culvert.

Photo 17: Photograph taken in an east direction from the top of the concrete box culvert illustrating the active erosion and subsequent slip failures along the south bank of the tributary to the Rideau River.

Photo 18: Photograph taken in a west direction to further capture the active erosion and subsequently slip failure presented in the previous photograph.

Photo 19: Photograph taken in a south direction illustrating a relatively recent slope failure along the west bank of the Rideau River.

Photo 20: Similar to the previous photograph but taken in a north direction further illustrating the relatively recent slip failure along the west bank of the Rideau River.

Photo 21: Photograph taken in a west direction on April 17, 2017, illustrating the active erosion and subsequent slip failures adjacent to the concrete box culvert.

Photo 22: Photograph taken in a west direction on October 8, 2024, illustrating the placement of blast rock fill to mitigate the active erosion near the outlet of the concrete box culvert.

Photo 23: Photograph taken in an east direction on April 17, 2017, illustrating the active erosion and subsequent slip failure along the south bank of the Tributary to the Rideau River.

Photo 24: Photograph taken in southeast direction on October 8, 2024, illustrating the active erosion at the toe of the slope and the development of vegetation along the previously identified slip failure.

Photo 25: Photograph taken in an east direction on April 17, 2017, illustrating some gabion baskets used to mitigate the active erosion along the south bank of the tributary to the Rideau River.

Photo 26: Photograph taken in an east direction on October 8, 2024, validating that the gabion baskets that are continuing to perform as designed.

Photo 27: Photograph taken on April 17, 2017, in a northeast direction illustrating the active erosion and subsequent slip failures along the north bank of the tributary to the Rideau River.

Photo 28: Photograph taken on October 8, 2024, in a northeast direction further documenting the active erosion and historic slope failure that occurred along the north bank of the tributary to the Rideau River.

Photo 29: Photograph taken in a north direction illustrating the current state of the slope adjacent to the previously placed blast rock fill near the outlet of the concrete box culvert along the north bank of the tributary to the Rideau River.

Photo 30: Photograph taken in a west direction from where the tributary connects to the Rideau River, illustrating the active erosion along the northeastern corner slope face.

Photo 31: Photograph taken in an east direction illustrating the active erosion along the northeastern corner slope face (Rideau River beyond).

Photo 32: Photograph taken in a north direction at the base of the slope along the Rideau River, illustrating evidence of active erosion along the lower sections of the slope face.

Photo 33: Photograph taken in a southwest direction at the base of the slope along the Rideau River, illustrating evidence of active erosion along the lower sections of the slope face.

Photo 34: Photograph taken in a southwest direction midway up the slope along the Rideau River, illustrating evidence of active erosion along the lower sections and localized higher sections of the slope face.

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

2175 Prince of Wales Drive, Ottawa, Ontario

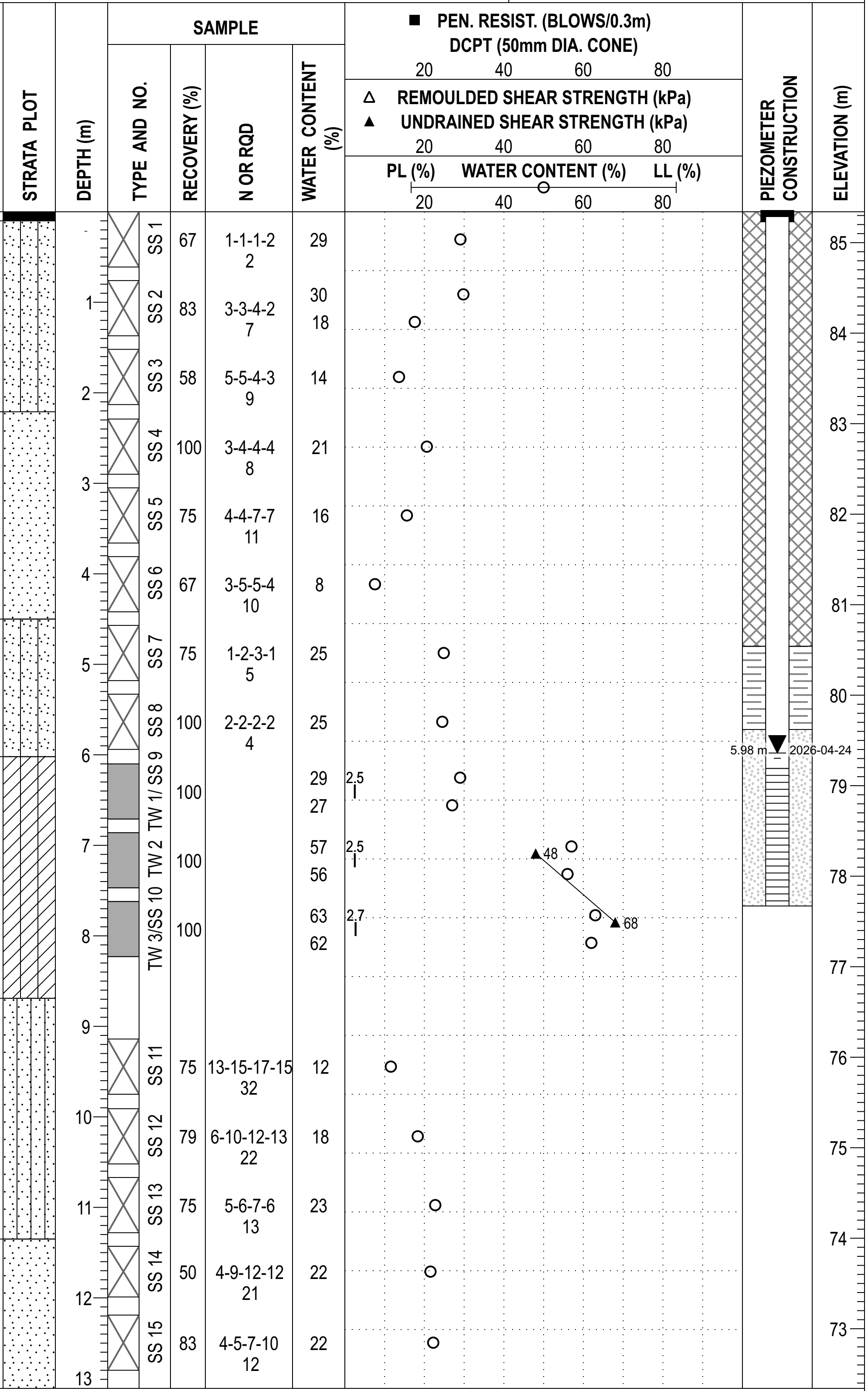
COORD. SYS.: MTM ZONE 9 **EASTING:** 367586.25 **NORTHING:** 5021681.20 **ELEVATION:** 85.34

PROJECT: Proposed Commercial Development
ADVANCED BY: Track Mounted Drill Rig
REMARKS: Implemented wash-boring technique as of 13.7 m depth
Datum: NAD1983
Geoid: HT2-2010
DATE: April 6, 2026

FILE NO. : **PG6557**
HOLE NO. : **BH 1-26**

SAMPLE DESCRIPTION	
GROUND SURFACE	
TOPSOIL	0.10m [85.24m]
Loose, brown **SILTY SAND**, some clay	2.21m [83.13m]
Loose to compact, brown **SAND**, trace clay	4.50m [80.84m]
Loose, brown **SILTY SAND**, trace clay	6.02m [79.32m]
Firm to stiff, grey **SILTY CLAY** to **CLAYEY SILT**	8.69m [76.65m]
Dense to compact, grey **SANDY SILT**, trace gravel	11.35m [73.99m]
Compact, grey **SAND**, trace silt	

TYPE AND NO.	RECOVERY (%)	N OR RQD	WATER CONTENT (%)
SS 1	67	1-1-1-2 2	29
SS 2	83	3-3-4-2 7	30 18
SS 3	58	5-5-4-3 9	14
SS 4	100	3-4-4-4 8	21
SS 5	75	4-4-7-7 11	16
SS 6	67	3-5-5-4 10	8
SS 7	75	1-2-3-1 5	25
SS 8	100	2-2-2-2 4	25
TW 1/ SS 9	100		29 27
TW 2	100		57 56
TW 3/SS 10	100		63 62
SS 11	75	13-15-17-15 32	12
SS 12	79	6-10-12-13 22	18
SS 13	75	5-6-7-6 13	23
SS 14	50	4-9-12-12 21	22
SS 15	83	4-5-7-10 12	22



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SOIL PROFILE AND TEST DATA

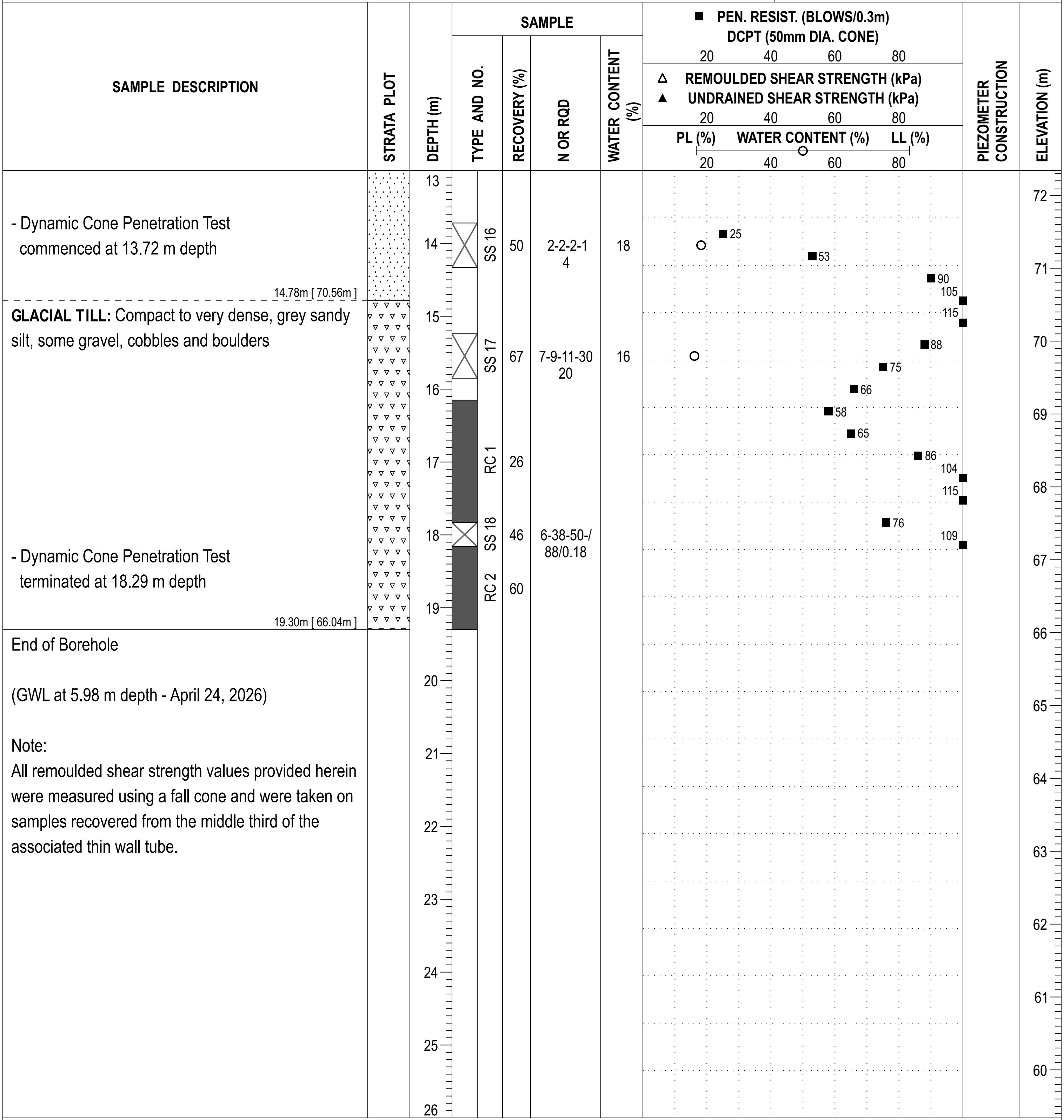
Geotechnical Investigation

2175 Prince of Wales Drive, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 367586.25 **NORTHING:** 5021681.20 **ELEVATION:** 85.34

PROJECT: Proposed Commercial Development
Datum: NAD1983
ADVANCED BY: Track Mounted Drill Rig
Geoid: HT2-2010
REMARKS: Implemented wash-boring technique as of 13.7 m depth
DATE: April 6, 2026

FILE NO. : PG6557
HOLE NO. : BH 1-26



SAMPLE DESCRIPTION

- Dynamic Cone Penetration Test commenced at 13.72 m depth

14.78m [70.56m]

GLACIAL TILL: Compact to very dense, grey sandy silt, some gravel, cobbles and boulders

- Dynamic Cone Penetration Test terminated at 18.29 m depth

19.30m [66.04m]

End of Borehole

(GWL at 5.98 m depth - April 24, 2026)

Note:
All remoulded shear strength values provided herein were measured using a fall cone and were taken on samples recovered from the middle third of the associated thin wall tube.

SAMPLE TYPE AND NO.	RECOVERY (%)	N OR RQD	WATER CONTENT (%)
SS 16	50	2-2-2-1 4	18
SS 17	67	7-9-11-30 20	16
RC 1	26		
SS 18	46	6-38-50-/ 88/0.18	
RC 2	60		

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation

2175 Prince of Wales Drive, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 367551.17 **NORTHING:** 5021752.27 **ELEVATION:** 86.14

PROJECT: Proposed Commercial Development

ADVANCED BY: Track Mounted Drill Rig

REMARKS: Implemented wash-boring technique as of 9.1 m depth

Datum: NAD1983

Geoid: HT2-2010

DATE: April 7, 2026

FILE NO. : **PG6557**

HOLE NO. : **BH 2-26**

SAMPLE DESCRIPTION

GROUND SURFACE

TOPSOIL

0.20m [85.94m]

Loose, brown **SILTY SAND**, trace clay

0.69m [85.45m]

Hard, brown **SILTY CLAY**

3.35m [82.79m]

Loose to compact, brown **SANDY SILT**

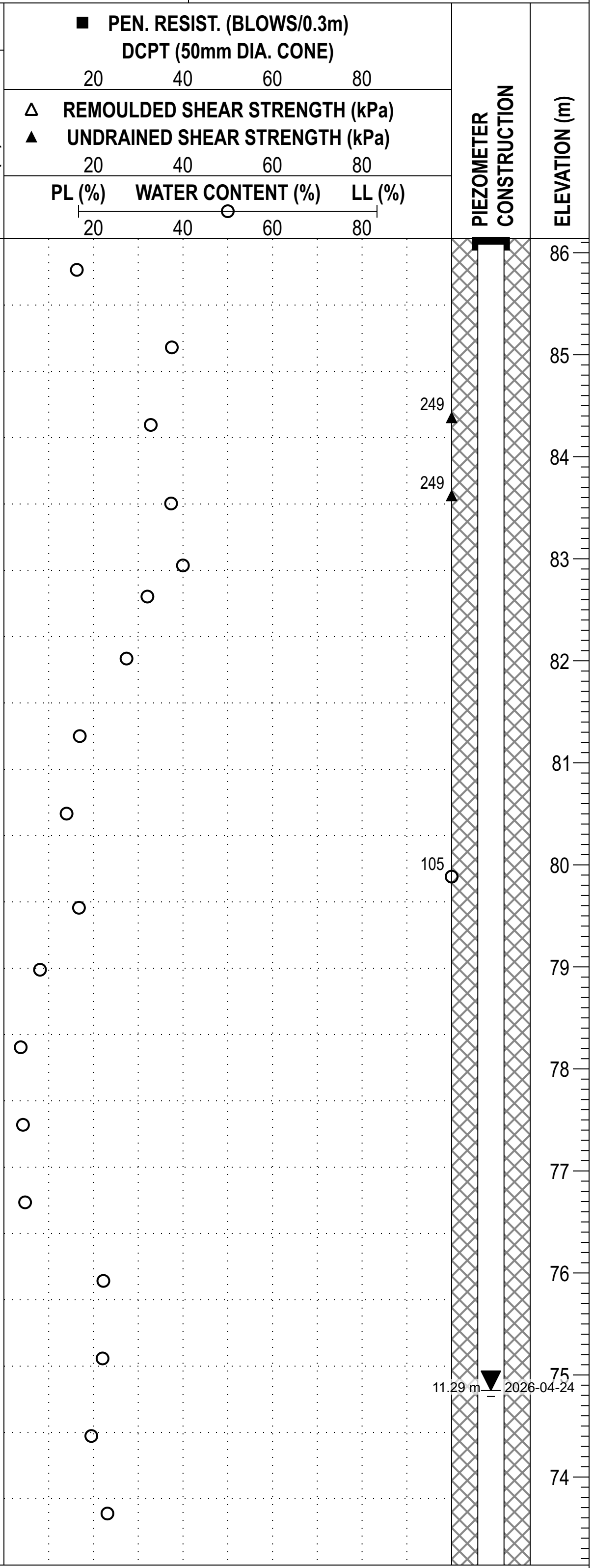
6.10m [80.04m]

Loose, grey **SANDY SILT**

6.71m [79.43m]

Compact to dense, brown **SAND**

TYPE AND NO.	RECOVERY (%)	N OR RQD	WATER CONTENT (%)
SS 1	75	1-2-2-2 4	16
SS 2	100	2-2-4-4 6	38
SS 3	100	P	33
SS 4	100	P	37
SS 5	83	2-2-4-6 6	40 32
SS 6	67	2-2-4-4 6	27
SS 7	58	2-2-8-14 10	17
SS 8	75	9-9-12-10 21	14
SS 9	75	2-2-6-10 8	105 17
SS 10	83	7-9-12-12 21	8
SS 11	67	8-15-13-13 28	4
SS 12	67	5-9-10-9 19	4
SS 13	75	4-8-9-9 17	5
SS 14	58	8-10-13-14 23	22
SS 15	50	10-10-12-15 22	22
SS 16	50	11-15-22-20 37	20
SS 17	50	9-13-17-17 30	23



DEPTH (m): 1 2 3 4 5 6 7 8 9 10 11 12 13

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation

2175 Prince of Wales Drive, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 367551.17 **NORTHING:** 5021752.27 **ELEVATION:** 86.14

PROJECT: Proposed Commercial Development Datum: NAD1983

ADVANCED BY: Track Mounted Drill Rig Geoid: HT2-2010

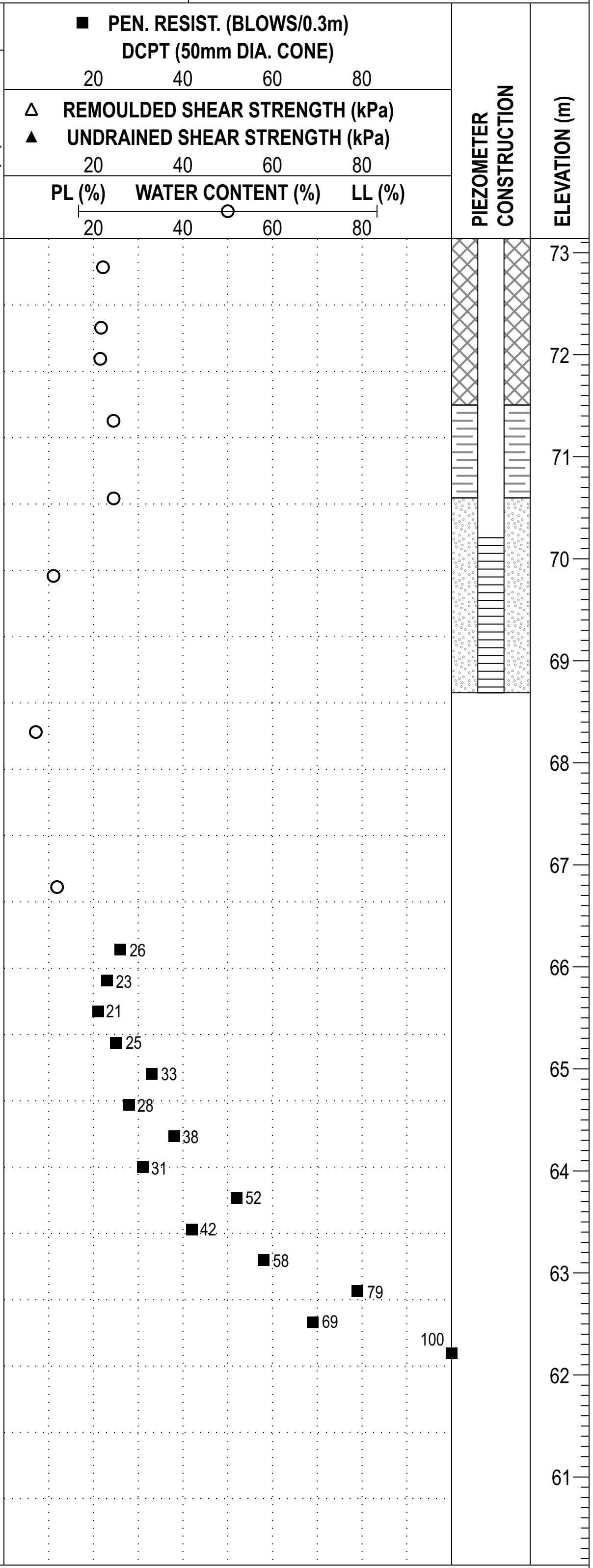
REMARKS: Implemented wash-boring technique as of 9.1 m depth **DATE:** April 7, 2026

FILE NO. : PG6557

HOLE NO. : BH 2-26

SAMPLE DESCRIPTION	Boundary
	14.02m [72.12m]
Dense, grey **SAND**	15.93m [70.21m]
GLACIAL TILL: Loose to compact, grey silty sand, some clay, gravel, cobbles and boulders	19.66m [66.48m]
Dynamic Cone Penetration Test commenced at 19.66 m depth	24.00m [62.14m]
End of Borehole Practical refusal to DCPT at 24.00 m depth (GWL at 11.29 m depth - April 24, 2026)	

TYPE AND NO.	RECOVERY (%)	N OR RQD	WATER CONTENT (%)
SS 18	50	8-12-14-15 26	22
SS 19	54	9-14-15-18 29	22 22
SS 20	58	13-15-20-21 35	24
SS 21	58	4-13-18-17 31	25
SS 22	92	4-1-3-3 4	11
SS 23	50	10-12-15-12 27	7
SS 24	42	12-14-15-18 29	12



DEPTH (m): 13 14 15 16 17 18 19 20 21 22 23 24 25 26

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation

2175 Prince of Wales Drive, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 367593.69 **NORTHING:** 5021776.12 **ELEVATION:** 85.34

PROJECT: Proposed Commercial Development

ADVANCED BY: Track Mounted Drill Rig

REMARKS: Implemented wash-boring technique as of 9.1 m depth

Datum: NAD1983

Geoid: HT2-2010

DATE: April 9, 2026

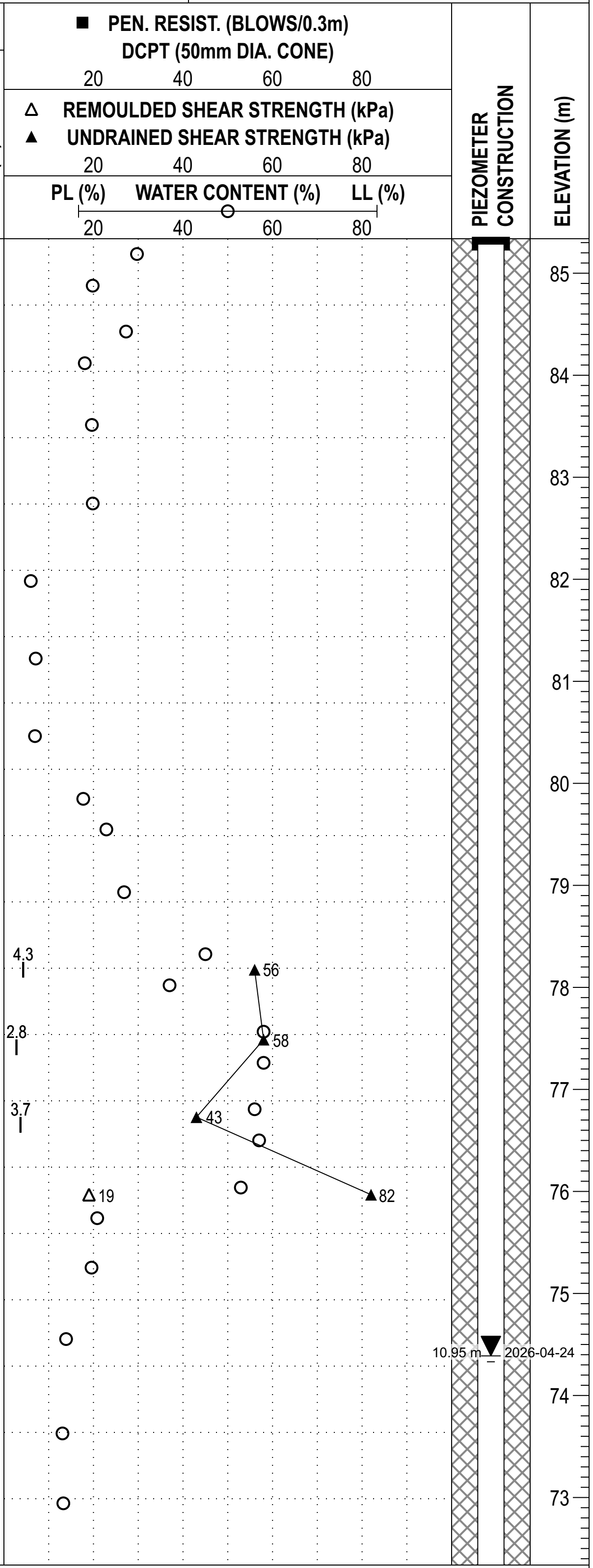
FILE NO. : **PG6557**

HOLE NO. : **BH 3-26**

SAMPLE DESCRIPTION	
GROUND SURFACE	
TOPSOIL	0.20m [85.14m]
Loose, brown **SILTY SAND**, trace clay	1.07m [84.27m]
Loose to compact, brown **SAND**, trace silt	5.79m [79.55m]
Loose, brown **SILTY SAND**, trace clay	6.78m [78.56m]
Stiff to firm, brown **SILTY CLAY**	9.53m [75.81m]
Compact to very dense, brown **SANDY SILT**, trace clay	10.36m [74.98m]
Very dense to dense, brown **SILTY SAND**, trace gravel	

SAMPLE TYPE AND NO.	RECOVERY (%)	N OR RQD	WATER CONTENT (%)
SS 1	83	1-3-4-4 7	30 20
SS 2	96	1-3-4-5 7	27 18
SS 3	75	2-5-6-6 11	20
SS 4	67	2-3-3-2 6	20
SS 5	75	2-4-9-9 13	6
SS 6	67	5-10-10-10 20	7
SS 7	67	4-7-6-7 13	7
SS 8	75	4-6-5-7 11	18 23
SS 9	100	3-2-2-2 4	27
SS10 TW 1	96		45 37
TW 2/SS11	100		58 58
TW 3	100		56 57
SS 12	100	0-1-8-14 9	53 21
SS 13	73	7-45-50-/ 95/0.2	20
SS 14	88	27-50-/-/ 50/0.08	14
SS 15	67	29-43-45-45 88	13
SS 16	74	32-48-50-/ 98/0.25	13

DEPTH (m): 1, 2, 3, 4, 5, 6, 7, 8, 9, 10, 11, 12, 13



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P:/AutoCAD Drawings/Test Hole Data Files/PG65xx/PG6557/data.sqlite 2026-05-13, 17:20 Paterson_Template JP

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

2175 Prince of Wales Drive, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 367593.69 **NORTHING:** 5021776.12 **ELEVATION:** 85.34

PROJECT: Proposed Commercial Development Datum: NAD1983

ADVANCED BY: Track Mounted Drill Rig Geoid: HT2-2010

REMARKS: Implemented wash-boring technique as of 9.1 m depth **DATE:** April 9, 2026

FILE NO. : **PG6557**

HOLE NO. : **BH 3-26**

SAMPLE DESCRIPTION

17.07m [68.27m]

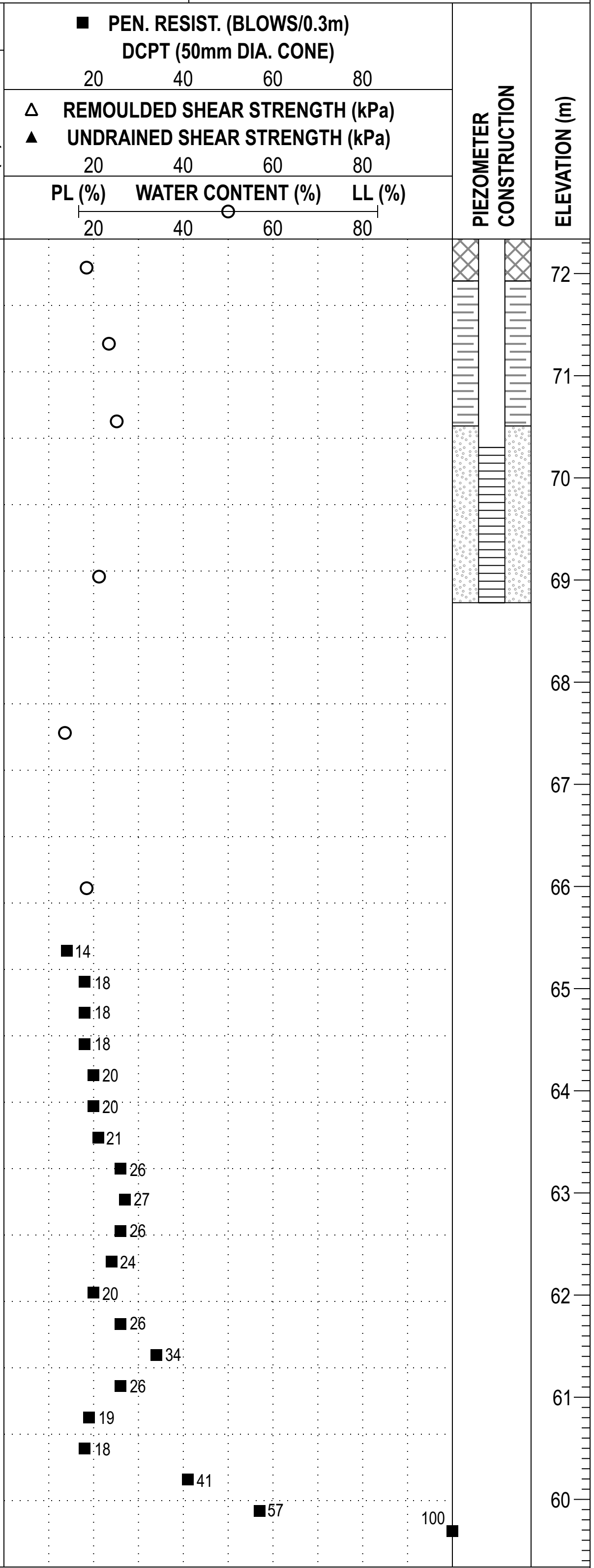
Dense to compact, brown **SAND**, some gravel and silt

19.66m [65.68m]

Dynamic Cone Penetration Test commenced at 19.66 m depth

25.70m [59.64m]

TYPE AND NO.	RECOVERY (%)	N OR RQD	WATER CONTENT (%)
SS 17	58	22-31-27-30 58	18
SS 18	42	15-26-24-25 50	23
SS 19	25	16-17-20-20 37	25
SS 20	42	13-16-14-16 30	21
SS 21	50	12-15-16-19 31	14
SS 22	17	12-11-10-11 21	18



P:/AutoCAD Drawings/Test Hole Data Files/PG65xx/PG6557/data.sqlite 2026-05-13, 17:20 Paterson_Template JP

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation

2175 Prince of Wales Drive, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 367593.69 **NORTHING:** 5021776.12 **ELEVATION:** 85.34

PROJECT: Proposed Commercial Development

ADVANCED BY: Track Mounted Drill Rig

REMARKS: Implemented wash-boring technique as of 9.1 m depth

Datum: NAD1983

Geoid: HT2-2010

DATE: April 9, 2026

FILE NO. : PG6557

HOLE NO. : BH 3-26

SAMPLE DESCRIPTION	STRATA PLOT	DEPTH (m)	SAMPLE TYPE AND NO.	SAMPLE RECOVERY (%)	SAMPLE N OR RQD	SAMPLE WATER CONTENT (%)	■ PEN. RESIST. (BLOWS/0.3m) DCPT (50mm DIA. CONE) 20 40 60 80 △ REMOULDED SHEAR STRENGTH (kPa) ▲ UNDRAINED SHEAR STRENGTH (kPa) 20 40 60 80 PL (%) WATER CONTENT (%) LL (%) 20 40 60 80	PIEZOMETER CONSTRUCTION	ELEVATION (m)
End of Borehole Practical refusal to DCPT at 25.70 m depth (GWL at 10.95 m depth - April 24, 2026) Note: Undisturbed shear strength value provided at TW1 and all remoulded shear strength values provided herein were measured using a fall cone and were taken on samples recovered from the middle third of the associated thin wall tube.		26 27 28 29 30 31 32 33 34 35 36 37 38 39							59 58 57 56 55 54 53 52 51 50 49 48 47

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PATERSON GROUP

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

2175 Prince of Wales Drive, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 367625.34 **NORTHING:** 5021715.45 **ELEVATION:** 82.75

PROJECT: Proposed Commercial Development

ADVANCED BY: Track Mounted Drill Rig

REMARKS: Implemented wash-boring technique as of 9.9 m depth

Datum: NAD1983

Geoid: HT2-2010

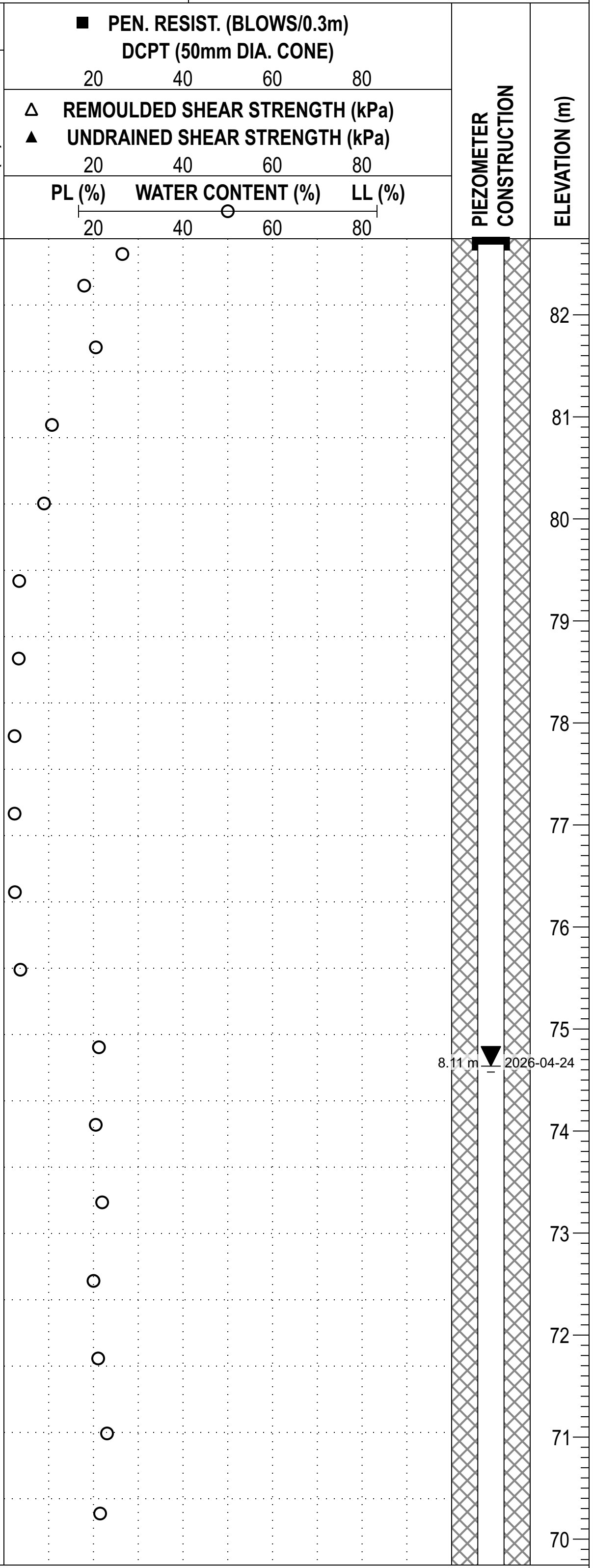
DATE: April 9, 2026

FILE NO. : PG6557

HOLE NO. : BH 4-26

SAMPLE DESCRIPTION	Bottom of stratum
GROUND SURFACE	
TOPSOIL	0.20m [82.55m]
Loose, brown **SILTY SAND**, some clay and gravel	1.37m [81.38m]
Compact, brown **SILTY SAND**	2.97m [79.78m]
Compact, brown **SAND**, trace silt	6.02m [76.73m]
Compact, brown **SILTY SAND**	7.54m [75.21m]
Compact, grey **SAND**, trace silt	11.35m [71.40m]
GLACIAL TILL: Loose to dense, grey silty sand, with clay, gravel, cobbles and boulders	12.88m [69.87m]

TYPE AND NO.	RECOVERY (%)	N OR RQD	WATER CONTENT (%)
SS 1	67	1-2-2-3 4	26 18
SS 2	67	2-2-2-5 4	21
SS 3	75	5-11-10-10 21	11
SS 4	75	6-11-12-12 23	9
SS 5	83	6-10-11-11 21	3
SS 6	75	7-9-9-10 18	3
SS 7	83	8-12-15-17 27	2
SS 8	75	8-12-14-13 26	2
SS 9	75	5-9-11-12 20	2
SS 10	75	5-10-12-14 22	4
SS 11	67	6-10-11-11 21	21
SS 12	67	7-9-10-10 19	21
SS 13	83	1-2-3-4 5	22
SS 14	58	5-6-9-11 15	20
SS 15	37	7-10-12-12 22	21
SS 16	42	6-9-11-9 20	23
SS 17	42	11-9-11-12 20	21



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation

2175 Prince of Wales Drive, Ottawa, Ontario

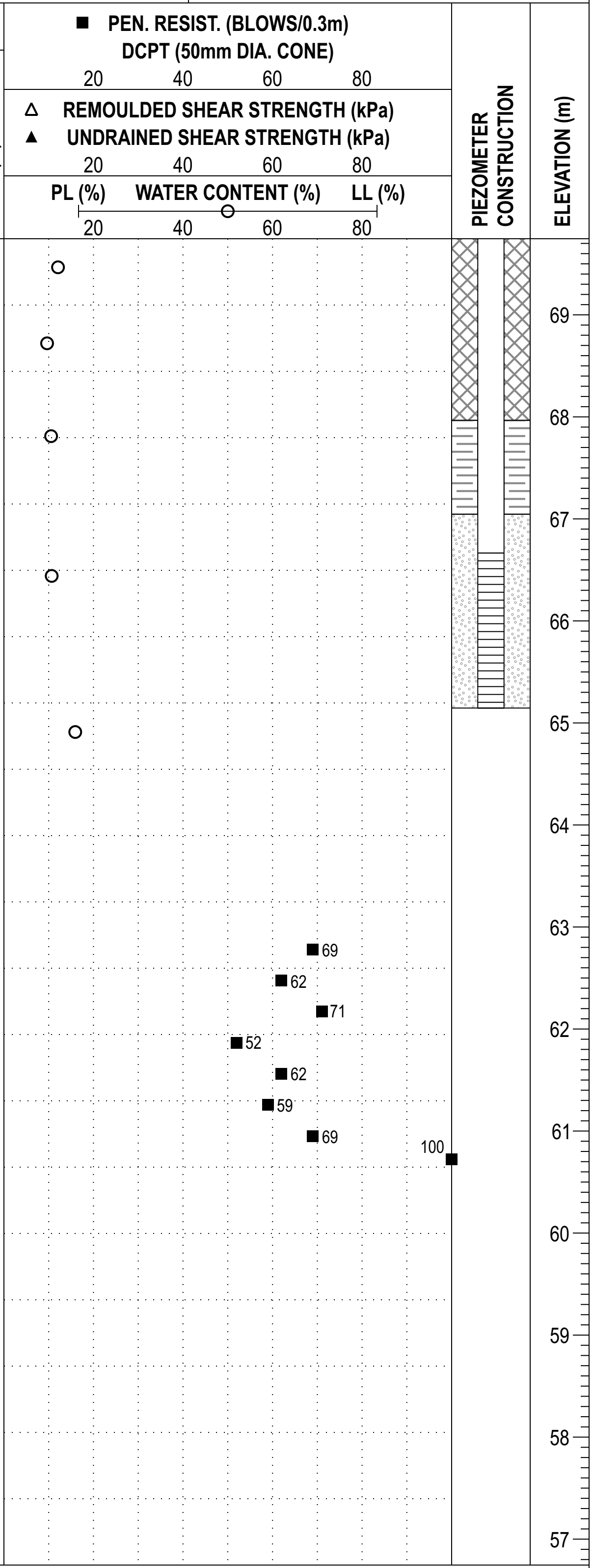
COORD. SYS.: MTM ZONE 9 **EASTING:** 367625.34 **NORTHING:** 5021715.45 **ELEVATION:** 82.75

PROJECT: Proposed Commercial Development
ADVANCED BY: Track Mounted Drill Rig
REMARKS: Implemented wash-boring technique as of 9.9 m depth

Datum: NAD1983
Geoid: HT2-2010
DATE: April 9, 2026

FILE NO. : PG6557
HOLE NO. : BH 4-26

SAMPLE DESCRIPTION	STRATA PLOT	DEPTH (m)	TYPE AND NO.	RECOVERY (%)	N OR RQD	WATER CONTENT (%)
Dynamic Cone Penetration Test Commenced at 12.88 m depth		13	SS 18	67	6-3-2-2 5	12
		14	SS 19	21	12-17-25-10 42	10
		15	SS 20	17	5-16-11-12 27	11
		16	SS 21	33	23-16-27-18 43	11
		17				
		18	SS 22	75	21-30-41-40 71	16
19.66m [63.09m]		19	SS 23	50	32-45-37-42 82	
End of Borehole Practical refusal to DCPT at 19.66 m depth (GWL at 8.11 m depth - April 24, 2026)		20				
		21				
		22				
		23				
		24				
		25				
		26				



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation

2175 Prince of Wales Drive, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 367644.54 **NORTHING:** 5021776.61 **ELEVATION:** 81.92

PROJECT: Proposed Commercial Development
Datum: NAD1983
ADVANCED BY: Track Mounted Drill Rig
Geoid: HT2-2010
REMARKS: Implemented wash-boring technique as of 6.1 m depth
DATE: April 10, 2026

FILE NO. : PG6557
HOLE NO. : BH 5-26

SAMPLE DESCRIPTION

GROUND SURFACE

TOPSOIL
0.20m [81.72m]

Loose, brown **SILTY SAND**, some clay, trace gravel
1.37m [80.55m]

Loose, brown **SILTY SAND**
2.97m [78.95m]

Very stiff, brown **SILTY CLAY**
6.02m [75.90m]

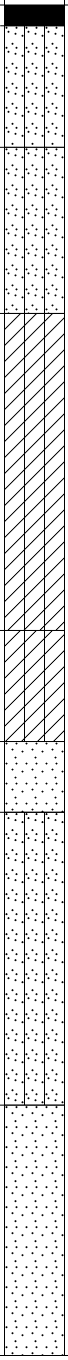
Loose, brown **CLAYEY SILT**, trace sand
7.09m [74.83m]

Dense, brown **SAND**, trace silt
7.77m [74.15m]

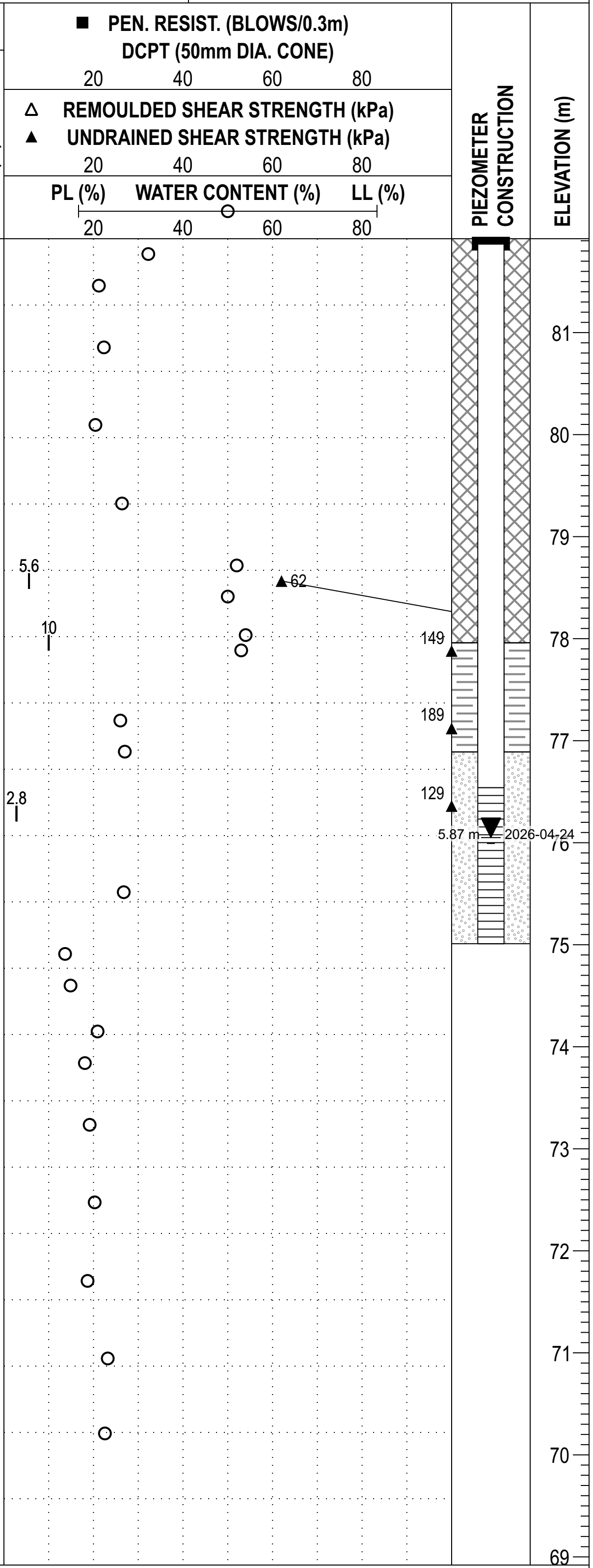
Very dense to dense, brown **SILTY SAND**
10.59m [71.33m]

Dense, brown **SAND**, trace silt

STRATA PLOT



TYPE AND NO.	RECOVERY (%)	N OR RQD	WATER CONTENT (%)
SS 1	62	1-1-1-3 2	32 21
SS 2	50	3-2-2-2 4	22
SS 3	67	2-3-4-2 7	20
SS 4	100	1-2-2-1 4	26
TW 1/SS5	100		52 50
TW 2	100		54 53
G 6			
TW 3	100		26 27
TW 4	100		
SS 7	83	3-2-5-7 7	27
SS 8	67	13-15-15-15 30	14 15
SS 9	58	9-23-29-22 52	21 18
SS 10	67	21-24-19-20 43	19
SS 11	67	13-19-20-20 39	20
SS 12	58	17-21-25-24 46	19
SS 13	42	15-19-25-22 44	23
SS 14	33	8-15-16-17 31	23



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation

2175 Prince of Wales Drive, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 367644.54 **NORTHING:** 5021776.61 **ELEVATION:** 81.92

PROJECT: Proposed Commercial Development

Datum: NAD1983

ADVANCED BY: Track Mounted Drill Rig

Geoid: HT2-2010

REMARKS: Implemented wash-boring technique as of 6.1 m depth

DATE: April 10, 2026

FILE NO. : PG6557

HOLE NO. : BH 5-26

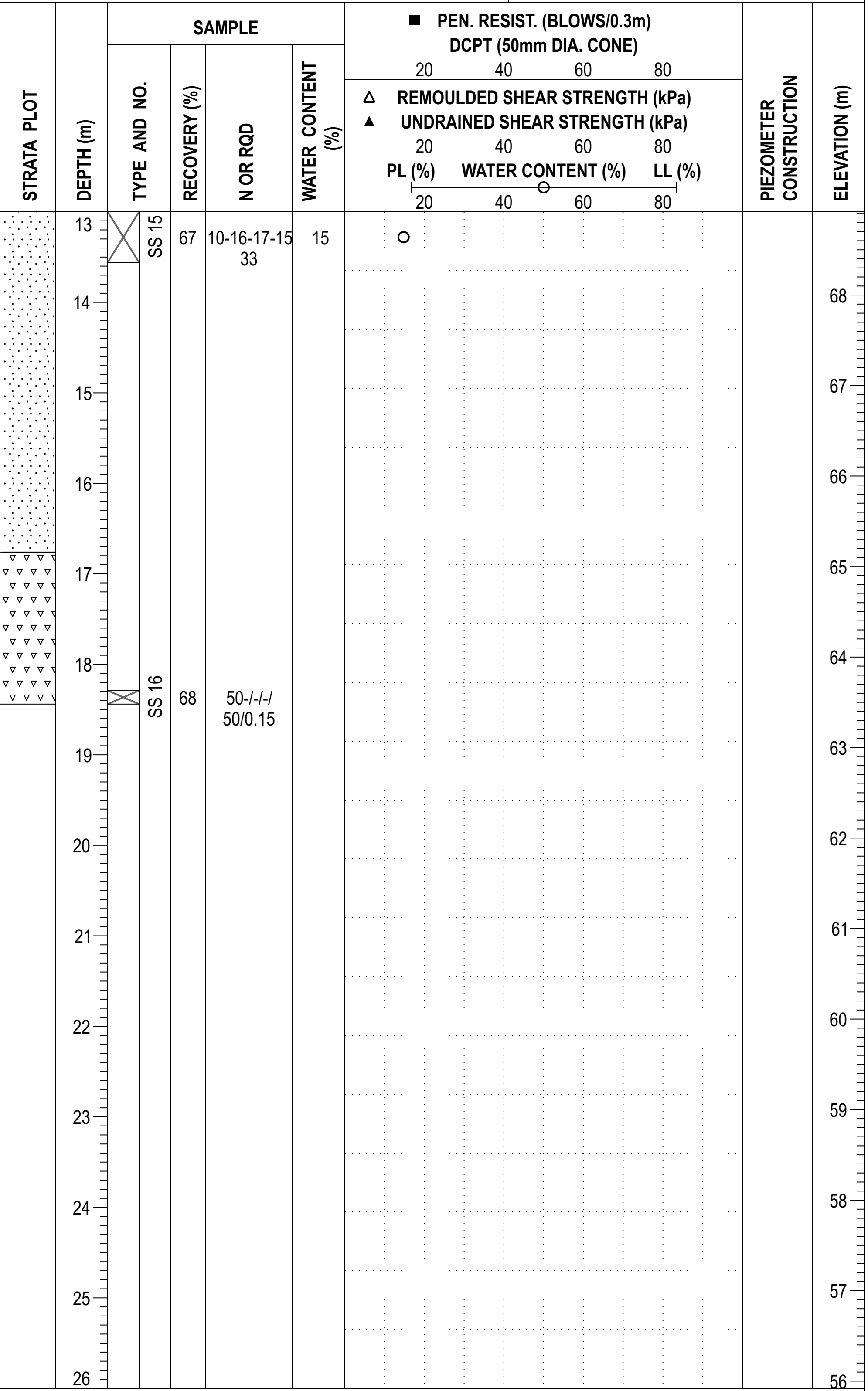
SAMPLE DESCRIPTION	DEPTH (m)	TYPE AND NO.	RECOVERY (%)	N OR RQD	WATER CONTENT (%)
	13	SS 15	67	10-16-17-15 33	15
16.76m [65.16m]					
GLACIAL TILL: Dense to very dense, brown silty sand, with gravel, cobbles and boulders					
	18	SS 16	68	50-/-/-/ 50/0.15	
18.44m [63.48m]					

End of Borehole

Practical refusal to augering at 18.44 m depth

(GWL at 5.87 m depth - April 24, 2026)

Note:
All remoulded shear strength values provided herein were measured using a fall cone and were taken on samples recovered from the middle third of the associated thin wall tube.



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SOIL PROFILE AND TEST DATA
Geotechnical Investigation
2175 Prince of Wales Drive, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 367544.02 **NORTHING:** 5021665.69 **ELEVATION:** 85.56

PROJECT: Proposed Commercial Development

ADVANCED BY: Track Mounted Drill Rig

REMARKS:

Datum: NAD1983

Geoid: HT2-2010

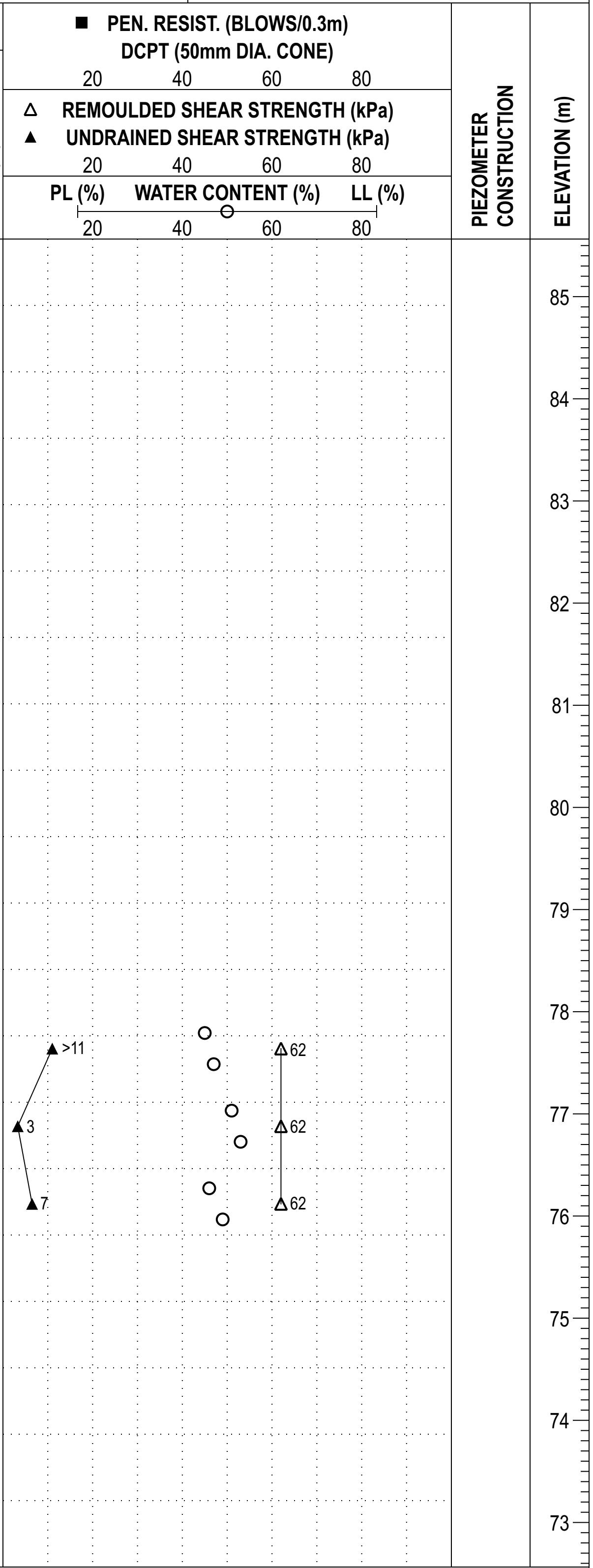
DATE: April 13, 2026

FILE NO. : **PG6557**

HOLE NO. : **BH 6-26**

SAMPLE DESCRIPTION	DEPTH (m)	TYPE AND NO.	RECOVERY (%)	N OR RQD	WATER CONTENT (%)
GROUND SURFACE	0				
7.62m [77.94m] Stiff to firm, grey **SILTY CLAY**	7.62	TW 1	100		45 47
		TW 2	100		51 53
		TW 3	100		46 49
9.91m [75.65m] Grey **SILTY SAND**	9.91	TW 4	92		
		TW 5	96		
		TW 6	92		
12.04m [73.52m] End of Borehole	12.04				

Note:
All remoulded shear strength values provided herein were measured using a fall cone and were taken on samples recovered from the middle third of the associated thin wall tube.



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Preliminary Geotechnical Investigation
2175 Prince of Wales Drive
Ottawa, Ontario

DATUM

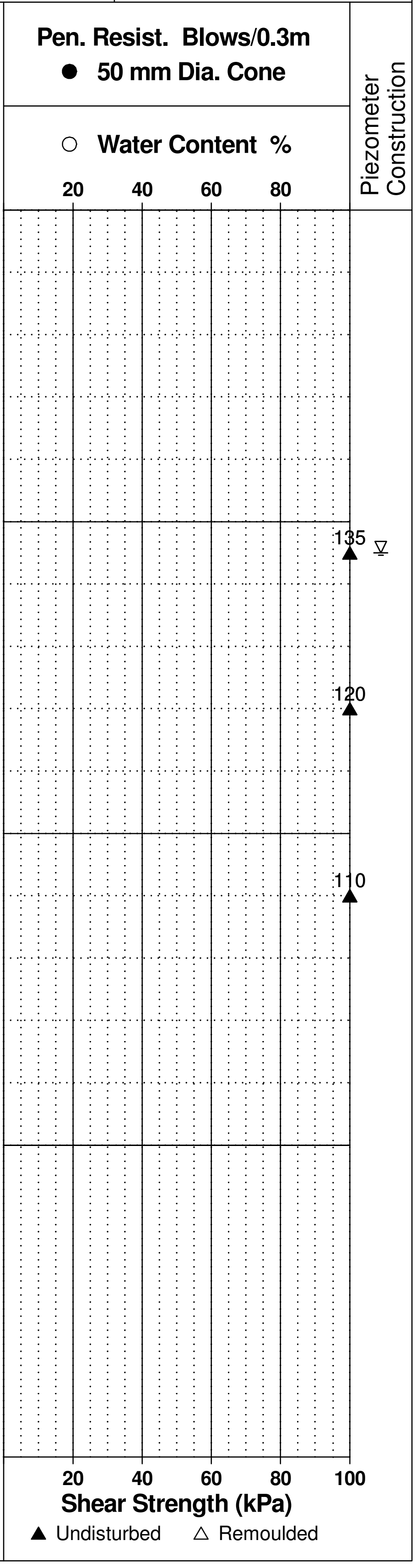
REMARKS Co-ordinates: 45.331482N, 75.700920W

BORINGS BY Rubber Tired Backhoe **DATE** May 11, 2017

FILE NO. PG4120

HOLE NO. TP 1-17

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	
Dark brown **TOPSOIL** with rootlets 0.24	■	G	1				
Compact, brown **SILTY SAND**, trace clay 0.70		G	2				
Very stiff, brown **SILTY CLAY**, trace sand - boulder noted at 1.0m depth 2.70	▨	G	3			1 2	
Stiff, grey **SILTY CLAY** with sand seams 3.00	▨	G	4			3	
End of Test Pit (Open hole GWL @ 1.1m depth) -rootlets extend to 0.5 m							



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Preliminary Geotechnical Investigation
2175 Prince of Wales Drive
Ottawa, Ontario

DATUM

REMARKS Co-ordinates: 45.331791N, 75.700262W

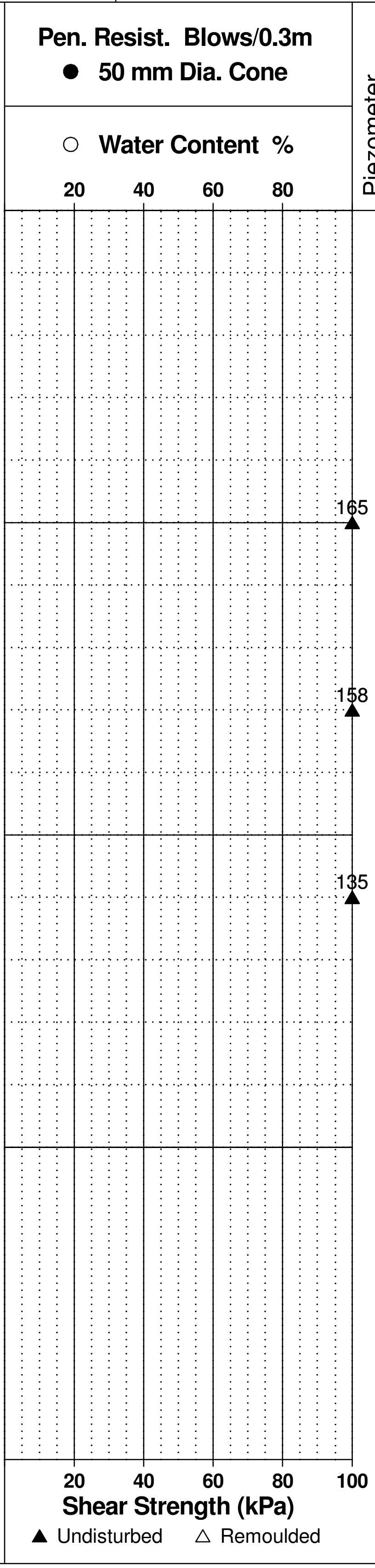
BORINGS BY Rubber Tired Backhoe

DATE May 11, 2017

FILE NO. PG4120

HOLE NO. TP 2-17

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Piezometer Construction
GROUND SURFACE						0		
Brown **TOPSOIL**, trace roots 0.27		G	1					
Very stiff, brown **SANDY CLAY** with silt, trace cobbles 0.70		G	2					
Very stiff to stiff, brown **SILTY CLAY**, trace sand 3.00		G G	3 4			1 2 3		
End of Test Pit								



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Preliminary Geotechnical Investigation
2175 Prince of Wales Drive
Ottawa, Ontario

DATUM

REMARKS Co-ordinates: 45.332297N, 75.699537W

BORINGS BY Rubber Tired Backhoe

DATE May 11, 2017

FILE NO. PG4120

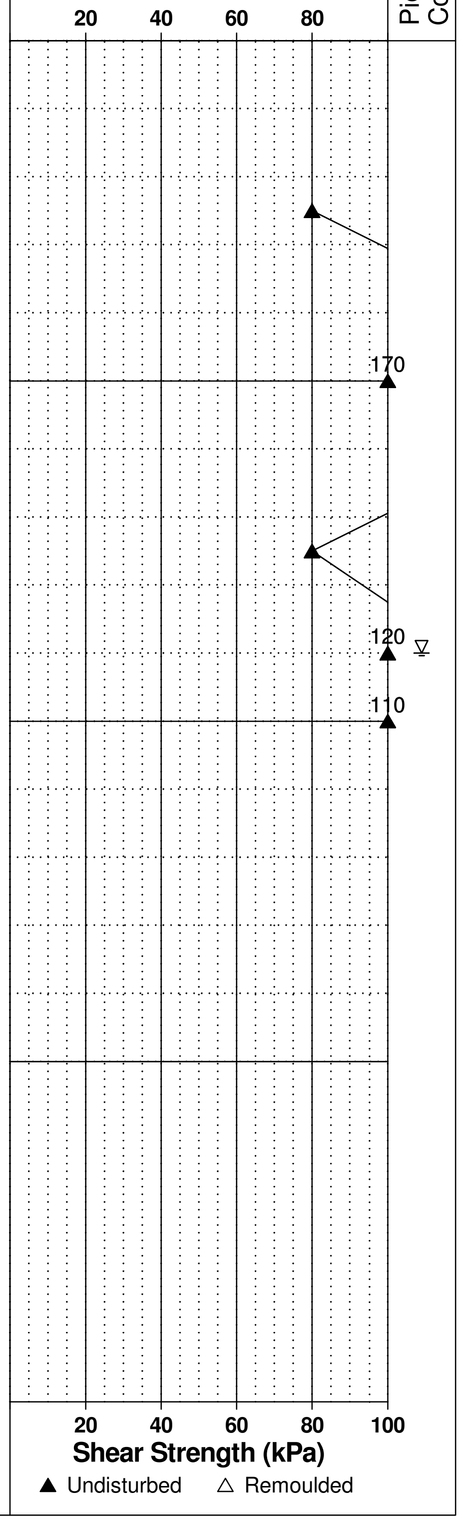
HOLE NO. TP 3-17

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	
Brown **TOPSOIL** with organics — 0.27	■	G	1				
Stiff, brown **SILTY CLAY** — 0.90	▨	G	2				
Very stiff to stiff, brown **SILTY CLAY** with silty sand seams — 3.00	▨	G G	3 4			1 2 3	
End of Test Pit (Open hole GWL @ 1.8m depth)							

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

○ Water Content %

Piezometer Construction



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Preliminary Geotechnical Investigation
2175 Prince of Wales Drive
Ottawa, Ontario

DATUM

REMARKS Co-ordinates: 45.332538N, 75.698942W

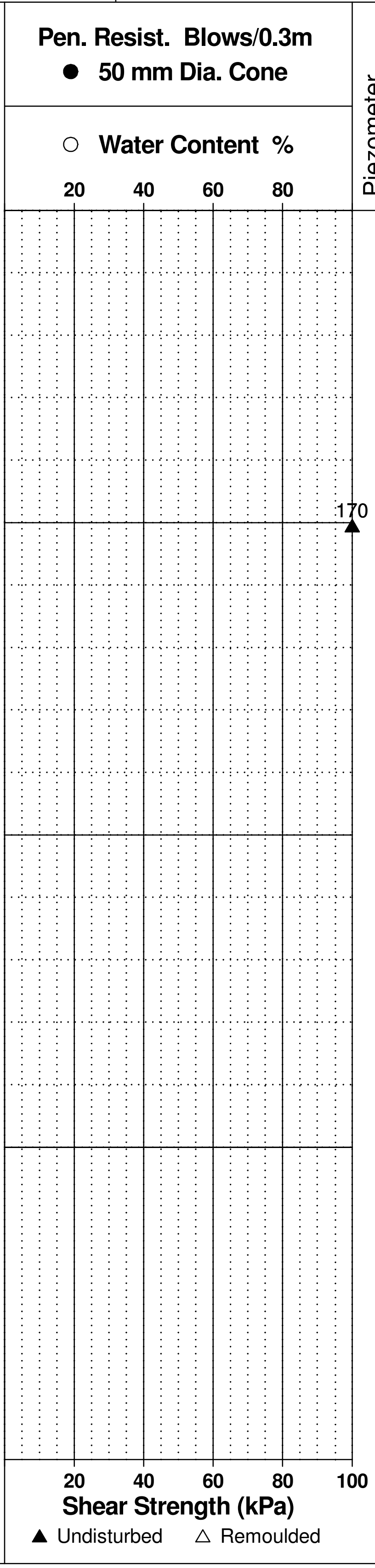
BORINGS BY Rubber Tired Backhoe

DATE May 11, 2017

FILE NO. PG4120

HOLE NO. TP 4-17

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Piezometer Construction
GROUND SURFACE						0		
Brown **TOPSOIL** with rootlets — 0.20	■	G	1					
Dense, brown **SILTY SAND**, some clay — 0.50		G	2					
Very stiff, brown **SILTY CLAY** with sand seams — 1.40	▨	G	3			1		
Compact, brown **SILTY SAND** — 3.00		G G	4 5			2 3		
End of Test Pit (TP dry upon completion)								



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Preliminary Geotechnical Investigation
2175 Prince of Wales Drive
Ottawa, Ontario

DATUM

REMARKS Co-ordinates: 45.332822N, 75.699174W

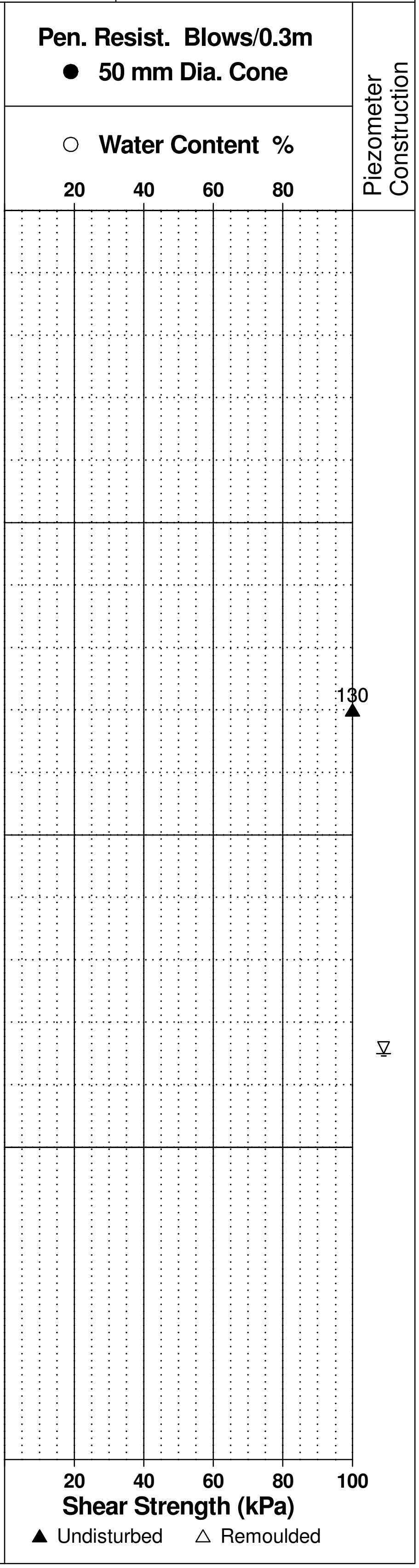
BORINGS BY Rubber Tired Backhoe

DATE May 11, 2017

FILE NO. PG4120

HOLE NO. TP 5-17

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	
Brown **TOPSOIL** with rootlets — 0.25	G	1				
Compact to dense, brown **SILTY SAND**, some clay, trace cobbles — 0.90	G	2				
Compact, brown **SILTY SAND**, trace gravel and boulders — 1.40	G	3			1	
Very stiff, brown **SILTY CLAY** with sand, trace cobbles — 2.20	G	4			2	
Compact, brown **SILTY SAND**, some clay — 3.00	G	5			3	
End of Test Pit (Open hole GWL @ 2.7m depth)						



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Preliminary Geotechnical Investigation
2175 Prince of Wales Drive
Ottawa, Ontario

DATUM

REMARKS Co-ordinates: 45.332822N, 75.699635W

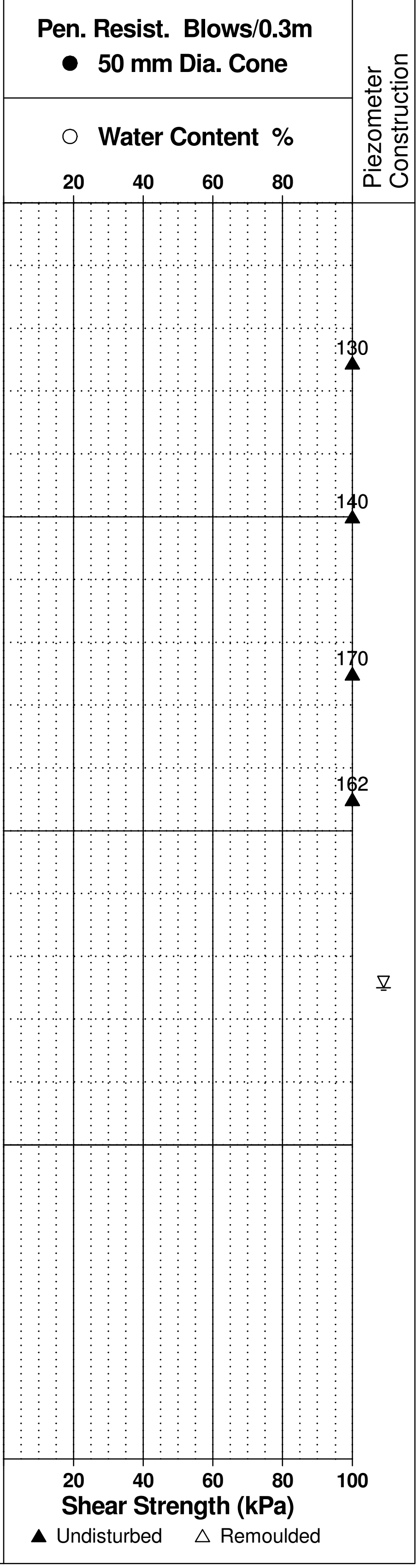
BORINGS BY Rubber Tired Backhoe

DATE May 11, 2017

FILE NO. PG4120

HOLE NO. TP 6-17

SOIL DESCRIPTION GROUND SURFACE	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
Brown **TOPSOIL** with rootlets 0.30	■	G	1			0	
Very stiff, brown **SILTY CLAY**, some sand seams, trace cobbles 1.20	▨	G	2			1	
Very stiff to stiff, brown **SILTY CLAY** - sand seams at 2.5m depth 3.00	▨	G G	3 4			2	
End of Test Pit (Open hole GWL @ 2.5m depth)						3	



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SOIL PROFILE AND TEST DATA

Preliminary Geotechnical Investigation
2175 Prince of Wales Drive
Ottawa, Ontario

DATUM

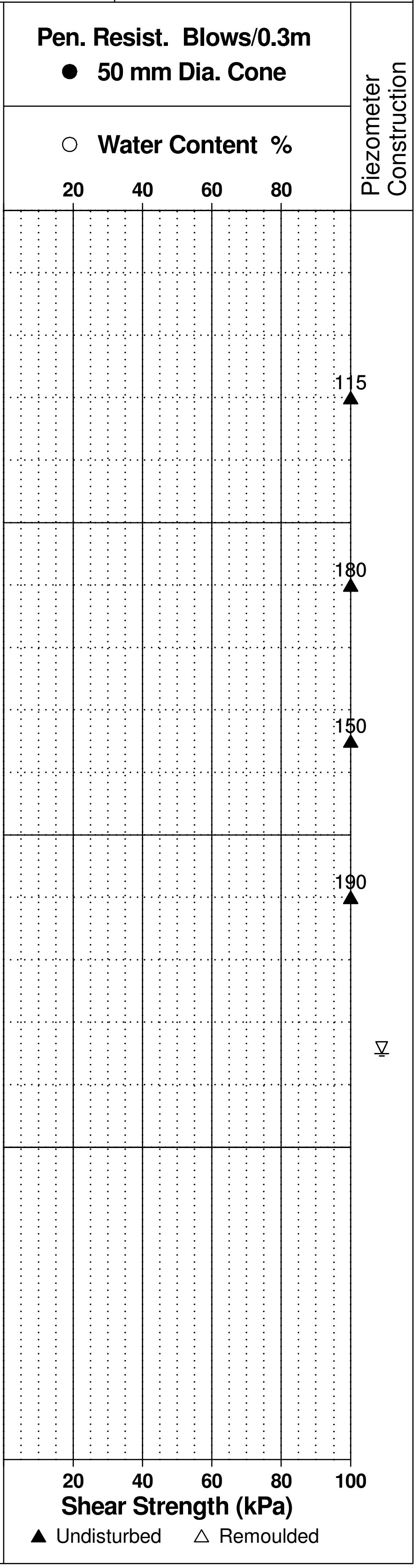
REMARKS Co-ordinates: 45.332959N, 75.700424W

BORINGS BY Rubber Tired Backhoe **DATE** May 11, 2017

FILE NO. PG4120

HOLE NO. TP 7-17

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	
Brown **TOPSOIL** 0.20	G	1				
Compact, brown **SILTY SAND**, some clay, trace gravel 0.60	G	2				
Very stiff, brown **SILTY CLAY**, some sand seams 3.00	G G	3 4			1 2 3	
End of Test Pit (Open hole GWL @ 2.7m depth)						



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SOIL PROFILE AND TEST DATA

Preliminary Geotechnical Investigation
2175 Prince of Wales Drive
Ottawa, Ontario

DATUM

REMARKS Co-ordinates: 45.332538N, 75.700326W

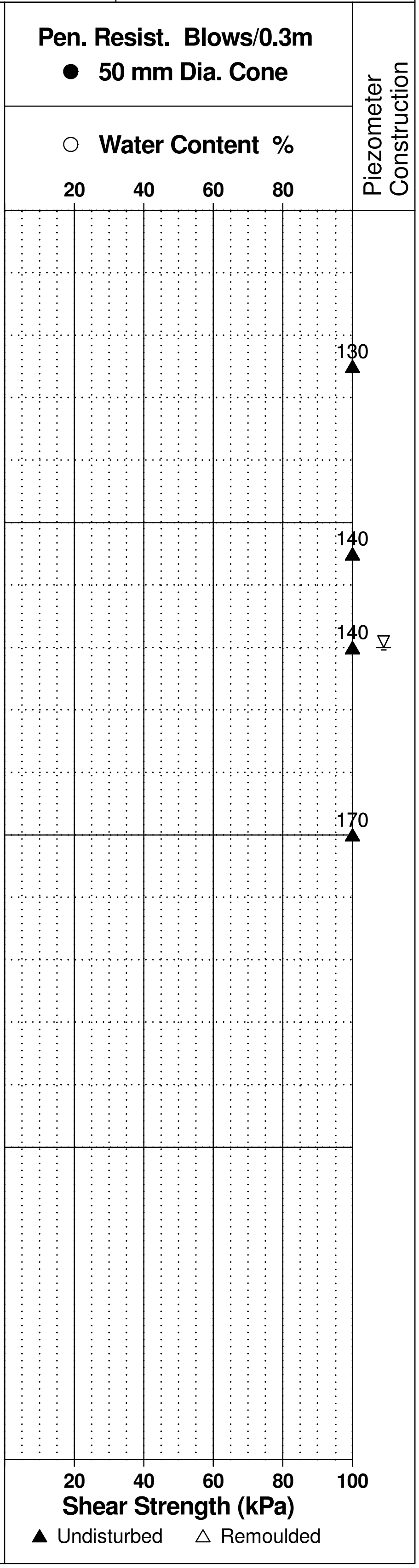
BORINGS BY Rubber Tired Backhoe

DATE May 11, 2017

FILE NO. PG4120

HOLE NO. TP 8-17

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	
Brown **TOPSOIL**	■	G	1				
0.25							
Very stiff, brown **SILTY CLAY**, trace sand and gravel	▨	G	2				
						1	
1.10							
Very stiff to stiff, brown **SILTY CLAY**, trace silty sand seams	▨						
						2	
		G	3				
3.00						3	
End of Test Pit							
(Open hole GWL @ 2 m depth)							



patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Preliminary Geotechnical Investigation
2175 Prince of Wales Drive
Ottawa, Ontario

DATUM

REMARKS Co-ordinates: 45.332153N, 75.700467W

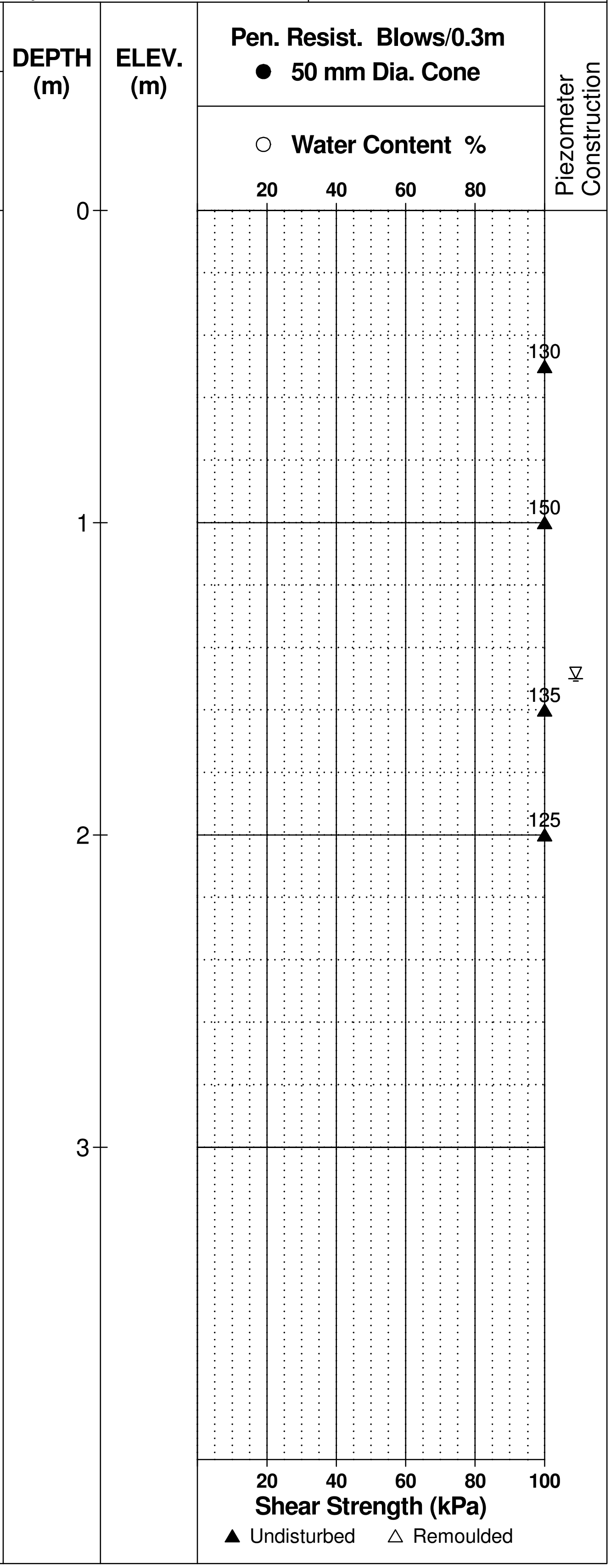
BORINGS BY Rubber Tired Backhoe

DATE May 11, 2017

FILE NO. PG4120

HOLE NO. TP 9-17

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD
GROUND SURFACE					
Brown **TOPSOIL**	■	G	1		
0.25					
Very stiff, brown **SILTY CLAY**, trace sand	▨	G	2		
3.00					
End of Test Pit (Open hole GWL @ 1.5 m depth)					



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Preliminary Geotechnical Investigation
2175 Prince of Wales Drive
Ottawa, Ontario

DATUM

REMARKS Co-ordinates: 45.332488N, 75.699615W

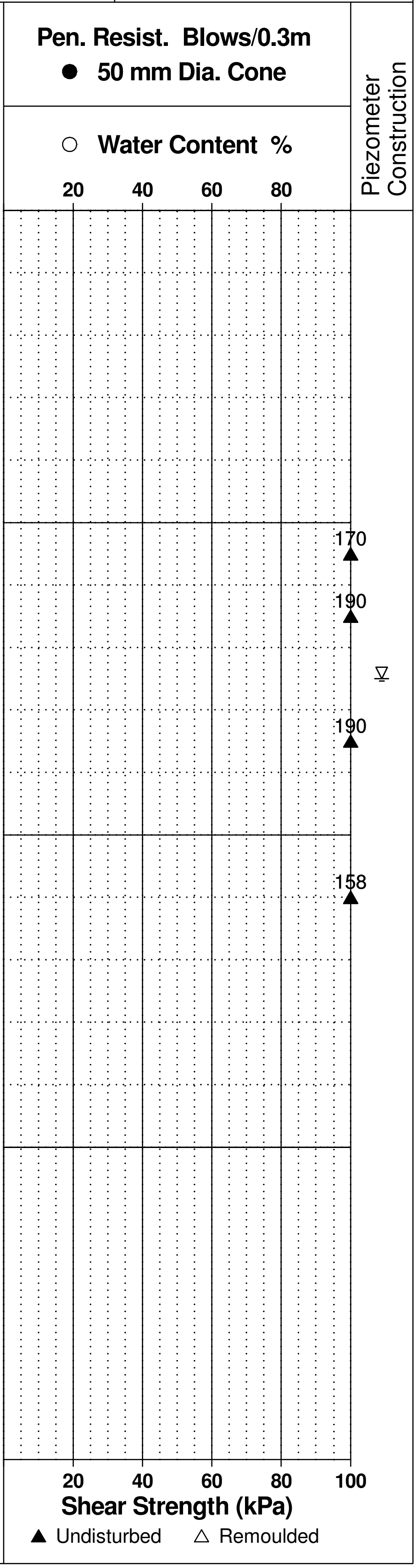
BORINGS BY Rubber Tired Backhoe

DATE May 11, 2017

FILE NO. PG4120

HOLE NO. TP10-17

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	
Black **TOPSOIL** with rootlets 0.32		G	1				
Very stiff, brown **SILTY CLAY** with sand, trace cobbles 1.00		G	2			1	
Very stiff, brown **SILTY CLAY**, some sand seams 3.00		G	3			2 3	
End of Test Pit (Open hole GWL @ 1.5 m depth)							



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28 Concourse Gate, Nepean, Ont. K2E 7T7

SOIL PROFILE & TEST DATA

Riverbank Failure Assessment
Waterbend Lane
Ottawa, Ontario

DATUM Geodetic, as provided by Cumming Cockburn Limited.

FILE NO. G8824

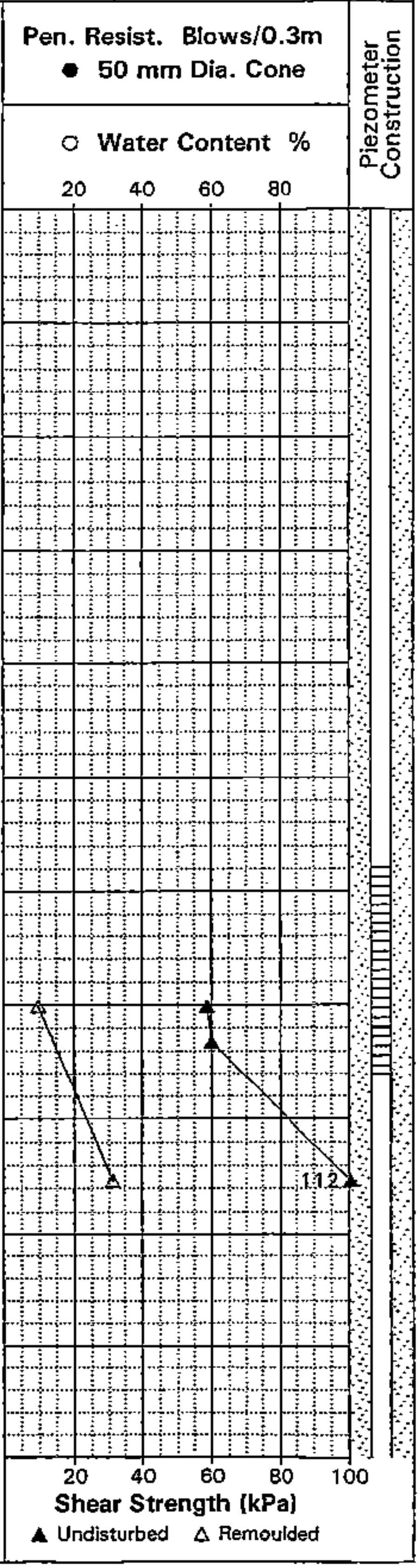
REMARKS

HOLE NO. BH 1

BORINGS BY CME 55 Power Auger **DATE** 12 DEC 02

SOIL DESCRIPTION		SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
75mm Topsoil over brown fine to coarse SAND and GRAVEL	0.18				
Brown SILTY CLAY interbedded with SILTY fine SAND		SS	1	67	10
		SS	2	83	11
	2.66	SS	3	100	9
Brown SILTY CLAY with occasional 25mm thick fine sand seams		SS	4	100	7
	4.34	SS	5	100	6
Brown SILTY fine to medium SAND with occasional 25 to 50mm thick layers of brown silty clay		SS	6	67	11
	6.02	SS	7	100	7
Grey SILTY fine SAND with clay, trace white shells	6.58	SS	8	100	1
Grey SILTY CLAY to CLAYEY SILT		SS	9	100	3
	9.50	SS	10	79	40

DEPTH (m)	ELEV. (m)
0	86.02
1	85.02
2	84.02
3	83.02
4	82.02
5	81.02
6	80.02
7	79.02
8	78.02
9	77.02
10	76.02
11	75.02



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SOIL PROFILE & TEST DATA

Riverbank Failure Assessment
Waterbend Lane
Ottawa, Ontario

DATUM Geodetic, as provided by Cumming Cockburn Limited.

REMARKS

BORINGS BY CME 55 Power Auger

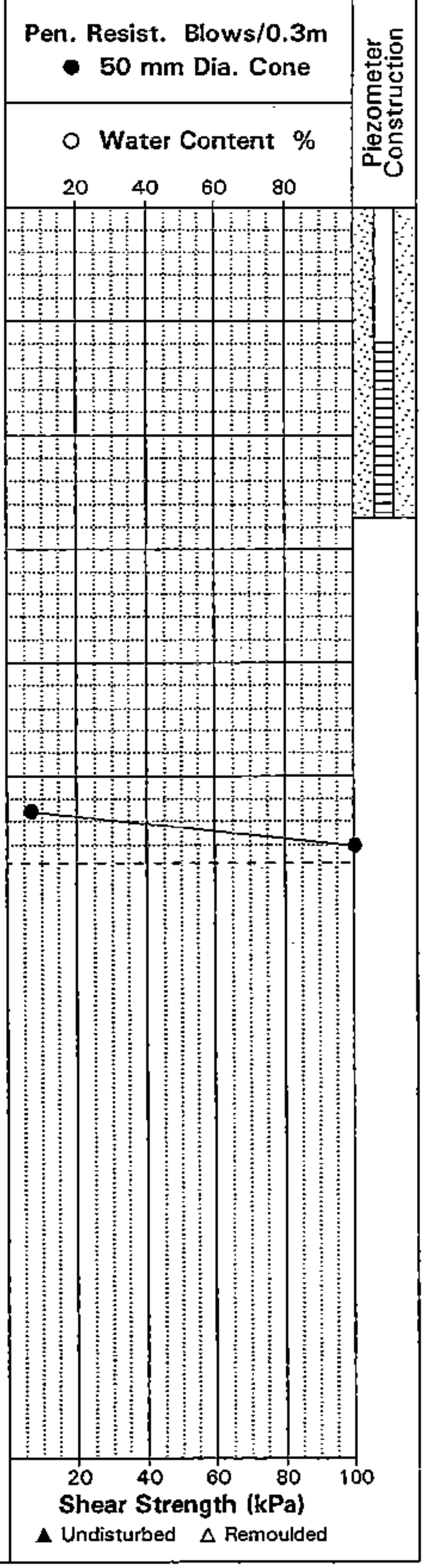
DATE 12 DEC 02

FILE NO. G8824

HOLE NO. BH 1

SOIL DESCRIPTION	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
Brown SILTY fine to medium SAND 380mm thick sandy silt layer @ 12.7m depth	SS	11	100	46
	SS	12	54	7
	SS	13	83	9
- gravel sizes by 15.5m depth - start of DCPT @ 16.3m depth	SS	14	100	47
16.76				
End of Borehole Refusal to DCPT at 16.8m depth (BH dry @ 12.8m on Dec. 16/02)				

DEPTH (m)	ELEV. (m)
11	75.02
12	74.02
13	73.02
14	72.02
15	71.02
16	70.02



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SOIL PROFILE & TEST DATA

Riverbank Failure Assessment
Waterbend Lane
Ottawa, Ontario

DATUM Geodetic, as provided by Cumming Cockburn Limited.

REMARKS

BORINGS BY Portable Drill

DATE 21 MAR 03

FILE NO. G8824

HOLE NO. BH 2-03

SOIL DESCRIPTION		SAMPLE				DEPTH (m)	ELEV. (m)
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD		
GROUND SURFACE						0	74.93
Brown SILTY fine SAND	0.30	G	1				
Brown SILTY CLAY, some sand	1.20	SS	2	71	2	1	73.93
Compact, brown SILTY fine SAND - 25mm thick silty clay seam at 2.12m depth - 50mm thick grey silt, trace clay @ 2.46m depth	2.74	SS	3	58	13	2	72.93
		SS	4	100	11		
		SS	5	62	14		
Compact, grey SILTY fine SAND - 10mm thick silty clay seam by 4.2m depth	4.57	SS	6	100	12	3	71.93
		SS	7	100	12	4	70.93
		SS	8	100	13		
End of Borehole							

(Open hole GWL @ 0.2m depth)

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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SOIL PROFILE & TEST DATA

Riverbank Failure Assessment
Waterbend Lane
Ottawa, Ontario

DATUM Geodetic, as provided by Cumming Cockburn Limited.

REMARKS

BORINGS BY Portable Drill

DATE 21 MAR 03

FILE NO. G8824

HOLE NO. BH 3-03

SOIL DESCRIPTION		SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	74.89
Brown SILTY fine to medium SAND	0.15	G	1				
Loose, grey SILTY fine SAND with thin grey silty clay layers		SS	2	67	4		
		SS	3	100	5	1	73.89
	2.13	SS	4	100	5	2	72.89
Loose to compact, grey to grey-brown SILTY fine to coarse SAND		SS	5	100	11		
		SS	6	100	8	3	71.89
		SS	7	100	13	4	70.89
	4.57	SS	8	92	22		
End of Test Pit (Open hole GWL @ 0.3m depth)							

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

○ Water Content %
20 40 60 80

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

Piezometer Construction

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SOIL PROFILE & TEST DATA

Riverbank Failure Assessment
Waterbend Lane
Ottawa, Ontario

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REMARKS

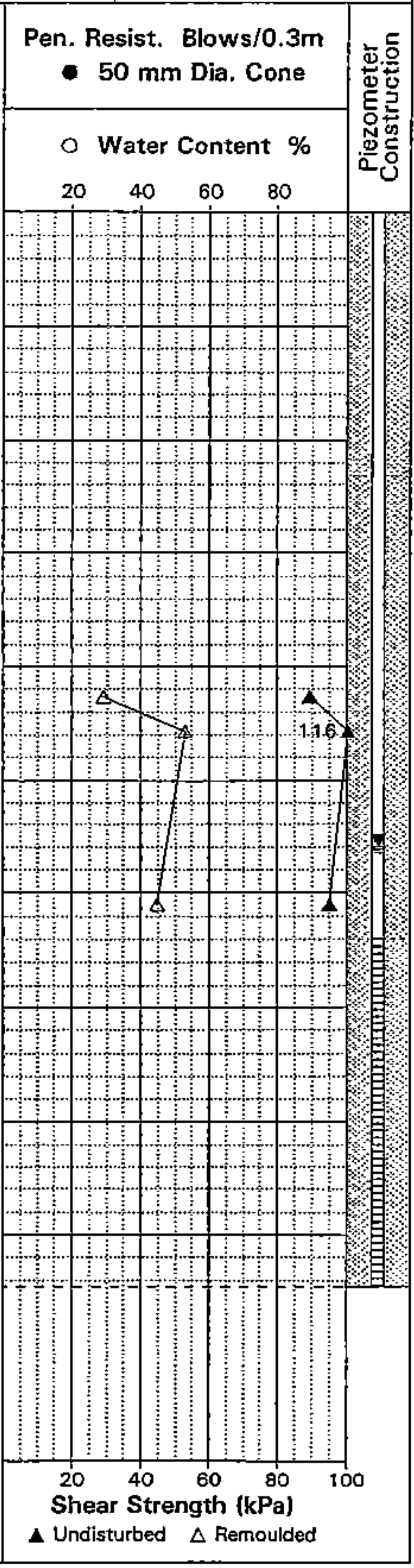
BORINGS BY CME 55 Power Auger

DATE 16 MAY 03

FILE NO. **G8824**

HOLE NO. **BH 4-03**

SOIL DESCRIPTION	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	81.82
TOPSOIL 0.38	SS	1	42	2		
Stiff, brown SILTY CLAY 0.56						
Very loose to loose, brown SILTY fine to coarse SAND	SS	2	75	3	1	80.82
	SS	3	71	3		
	SS	4	83	5	2	79.82
- brown-grey and trace clay by 2.6m depth 3.20	SS	5	67	4	3	78.82
Very stiff, brown-grey SILTY CLAY	SS	6	100	2		
	SS	7	83	P	4	77.82
	SS	8	100	3	5	76.82
	SS	9	100	2		
	SS	10	100	P	6	75.82
- with silt and sandy silt seams by 9.4m depth	SS	11		8		
7.60	SS	12	100	21	7	74.82
Compact, brown SILTY fine to coarse SAND	SS	13	67	22	8	73.82
	SS	14	100	11		
- grey by 8.8m depth 9.45	SS	15	100	30	9	72.82
End of Borehole						
(GWL @ 5.60m-May 22/03)						



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SOIL PROFILE & TEST DATA

Riverbank Failure Assessment
Waterbend Lane
Ottawa, Ontario

DATUM Geodetic, estimated.

REMARKS

BORINGS BY Portable Drill

DATE 8 MAY 03

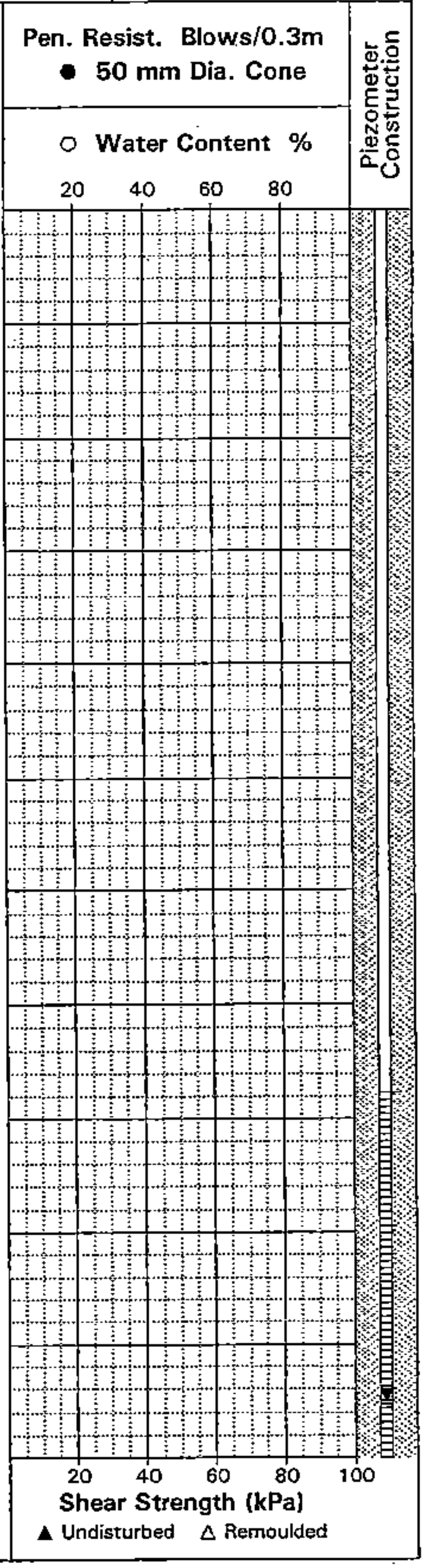
FILE NO. **G8824**

HOLE NO. **BH 5-03**

SOIL DESCRIPTION	Depth to base of layer (m)
GROUND SURFACE	
TOPSOIL	0.44
Very loose to loose, brown **SILTY** fine to medium **SAND**	1.47
Very stiff, brown-grey **SILTY CLAY** - occasional silty sand layers by 2.7m depth - grey by 3.1m depth	5.00
Very loose to loose, brown-grey SILTY SAND, trace clay - 0.3m thick silty clay at 6.1m depth	8.80
Firm to stiff, grey SILTY CLAY, trace sand	

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
SS	1	48	3
SS	2		3
SS	3		6
SS	4		10
SS	5	62	14
SS	6	88	10
SS	7	58	7
SS	8	71	10
SS	9	75	7
SS	10	96	8
SS	11	54	4
SS	12	100	3
SS	13	100	2
SS	14	100	2
SS	15		P
SS	16	100	3
SS	17	100	6

DEPTH (m)	ELEV. (m)
0	85.66
1	84.66
2	83.66
3	82.66
4	81.66
5	80.66
6	79.66
7	78.66
8	77.66
9	76.66
10	75.66
11	74.66



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SOIL PROFILE & TEST DATA

Riverbank Failure Assessment
Waterbend Lane
Ottawa, Ontario

DATUM Geodetic, estimated.

REMARKS

BORINGS BY Portable Drill

DATE 8 MAY 03

FILE NO. **G8824**

HOLE NO. **BH 5-03**

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
						11	74.66
Firm to stiff, grey SILTY **CLAY**, trace sand		SS	18	100	3		
		SS	19	100	10	12	73.66
12.34							
End of Borehole (GWL @ 10.5m-May 22/03)							

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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SOIL PROFILE & TEST DATA

Riverbank Failure Assessment
Waterbend Lane
Ottawa, Ontario

DATUM Geodetic, as provided by Cumming Cockburn Limited.

FILE NO. **G8824**

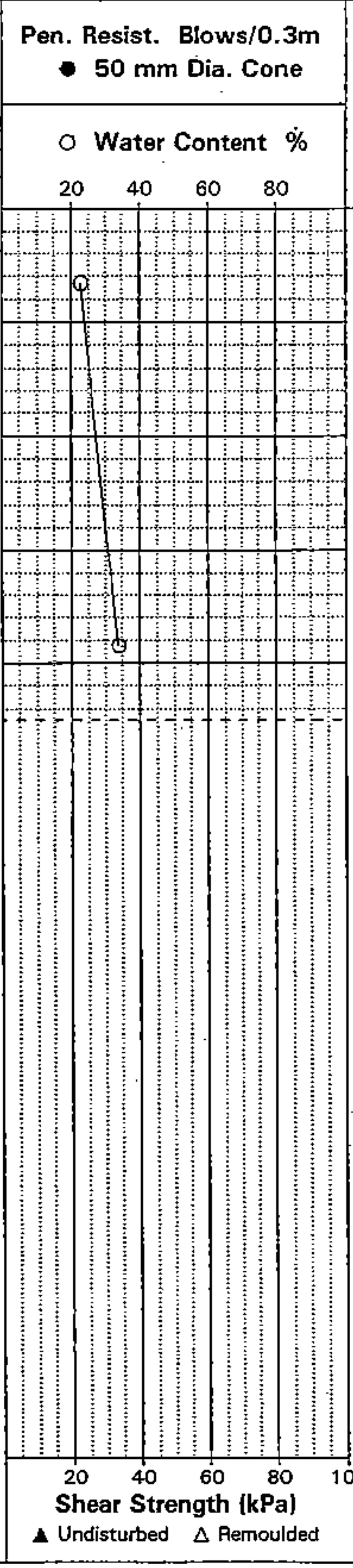
REMARKS

HOLE NO. **HA 1**

BORINGS BY Hand Auger

DATE 28 NOV 02

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	81.82
TOPSOIL 0.18						
Loose to compact, brown **SILTY** fine to medium **SAND**	G	1				
	G	2				
					1	80.82
- silty fine to medium sand with brown silty clay seams by 0.6m depth	G	3				
	G	4				
- brown-red silty fine to medium sand by 1.6m depth					2	79.82
- brown to light brown by 2.5m depth	G	5				
					3	78.82
- brown-grey by 3.4m depth 3.80	G	6				
Stiff, grey SILTY **CLAY/CLAYEY SILT**	G	7			4	77.82
4.50						
End of Hand Auger Hole						



Piezometer Construction

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SOIL PROFILE & TEST DATA

Riverbank Failure Assessment
Waterbend Lane
Ottawa, Ontario

DATUM Geodetic, as provided by Cumming Cockburn Limited.

FILE NO. **G8824**

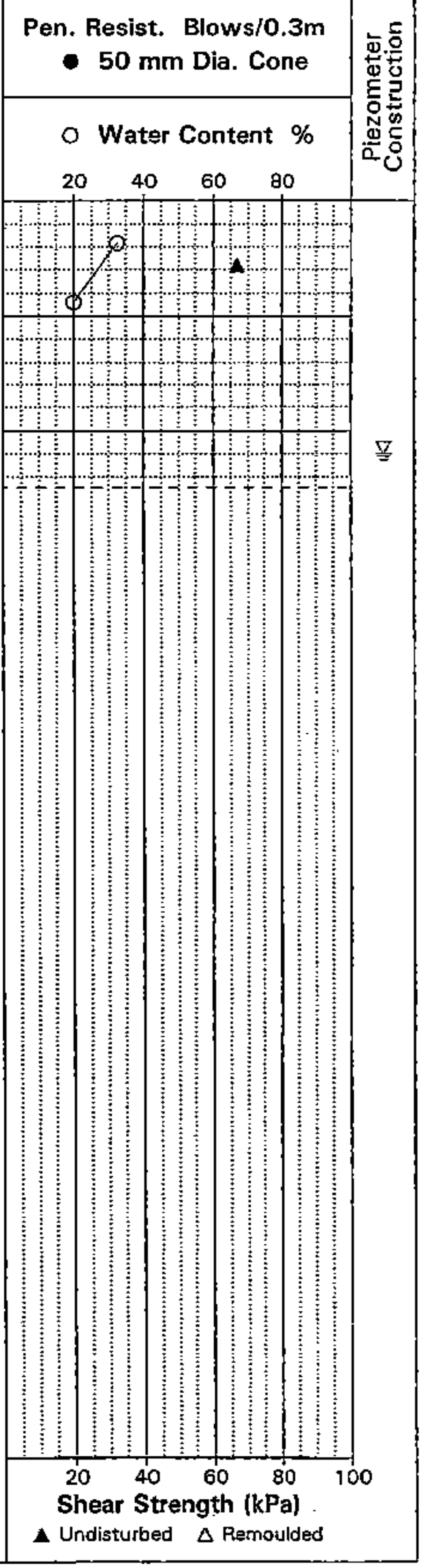
REMARKS

HOLE NO. **HA 2**

BORINGS BY Hand Auger

DATE 28 NOV 02

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	74.93
SILTY fine SAND 0.05							
Stiff, grey SILTY CLAY, trace sand 0.70		G	8				
Compact, grey SANDY SILT, trace clay		G	9			1	73.93
		G	10			2	72.93
2.49							
End of Hand Auger Hole							
(Open hole GWL @ 2.2m depth)							



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SOIL PROFILE & TEST DATA

Riverbank Failure Assessment
Waterbend Lane
Ottawa, Ontario

DATUM Geodetic, estimated.

REMARKS

BORINGS BY Hand Auger

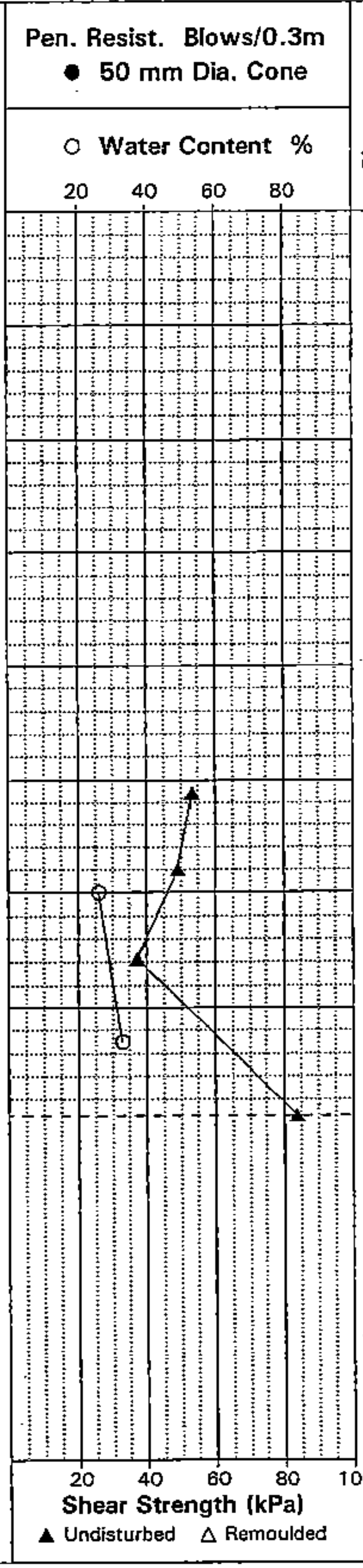
DATE 28 NOV 02

FILE NO. G8824

HOLE NO. HA 3

SOIL DESCRIPTION		SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
TOPSOIL	0.25				
Hard, brown **SILTY CLAY**	0.56	G	11		
Loose to compact, brown **SILTY** fine to coarse **SAND** with frequent silty clay seams		G	12		
- brown-grey silty fine to medium sand with silt by 4.4m depth		G	13		
- brown silty fine sand by 5.0m depth	5.10	G	14		
Stiff to firm, brown-grey **SANDY SILTY CLAY**, trace sand		G	15		
- stiff and grey by 7.2m depth	7.94	G	16		
End of Hand Auger Hole					

DEPTH (m)	ELEV. (m)
0	84.70
1	83.70
2	82.70
3	81.70
4	80.70
5	79.70
6	78.70
7	77.70



Piezometer Construction

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SOIL PROFILE & TEST DATA

Riverbank Failure Assessment
Waterbend Lane
Ottawa, Ontario

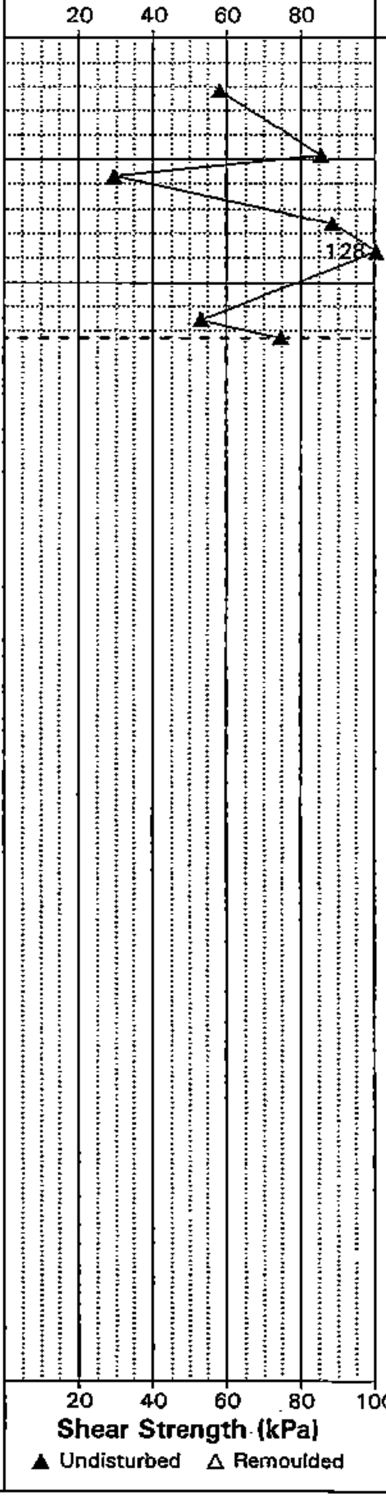
DATUM Geodetic, as provided by Cumming Cockburn Limited.
REMARKS
BORINGS BY Hand Auger
DATE 29 NOV 02

FILE NO. **G8824**
HOLE NO. **HA 4**

SOIL DESCRIPTION		STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE							0	74.89
SILTY fine SAND	0.08							
Stiff, grey SILTY CLAY, trace sand		▨	G	17				
		▨	G	18			1	73.89
- very stiff by 1.7m depth - clayey silt by 1.8m depth		▨	G	19			2	72.89
- stiff by 2.3m depth	2.46	▨						
End of Hand Auger Hole								

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

○ Water Content %



Shear Strength (kPa)
▲ Undisturbed △ Remoulded

Piezometer Construction

SOIL PROFILE AND TEST DATA

Proposed Residential Subdivision
South 1/2 Lot 26, Concession "A", R.F.
Nepean, Ontario

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Soil Investigations
Inspection & Testing Services
Damage Claims

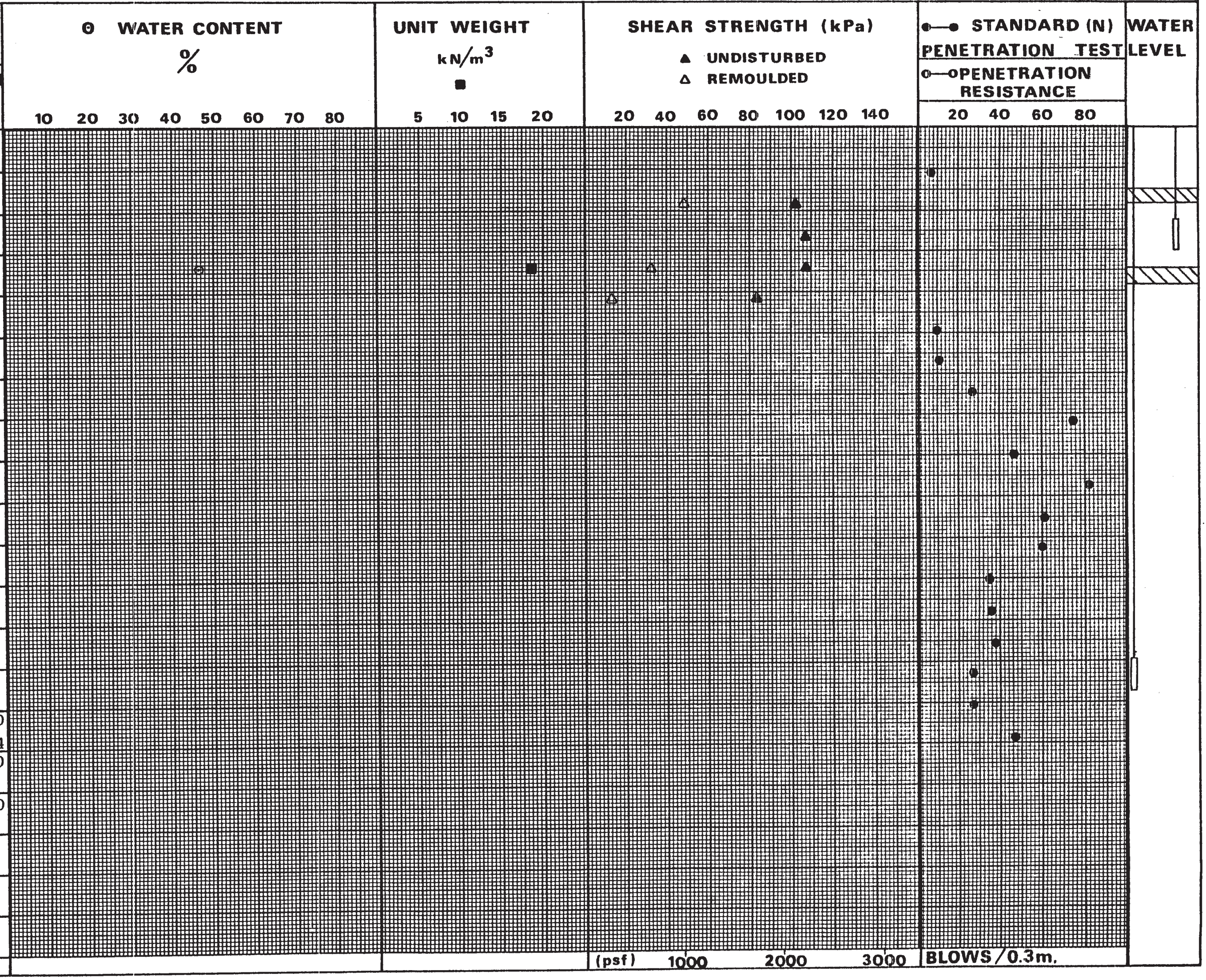
Offices & Laboratory
1479 Laperriere Ave.
Ottawa, Canada K1Z 7S8
Telephone (613) 728-3505

SHEET NO. 1 OF 3
HOLE NO. BH 1
GROUND SURFACE 86.04 BOTTOM HOLE 73.84
BEDROCK ______ GROUNDWATER DRY

DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER
Ground Surface		
250 mm TOPSOIL over a compact grey interbedded FINE SAND and SANDY SILT 0.8	G	1
Very stiff to stiff fissured olive grey SILTY CLAY with pinkish grey banding containing fine sand seams 3.6	SS	2
	TW	3
	TW	4
	TW	5
	TW	6
Compact brown SILTY FINE SAND containing clayey silt seams approximately 5 mm thick 5.5	SS	7
	SS	8
	SS	9
Very dense grey SANDY SILT containing silty fine sand seams 6.7	SS	10
	SS	11
Dense pale grey FINE SAND with some hairlike black banding. Becoming coarser with depth. 12.2	SS	12
	SS	13
	SS	14
	SS	15
	SS	16
	SS	17
	SS	18
	SS	19
	SS	20
Borehole terminated in sand		

ELEV. / DEPTH

86.04
0.00
0.80
1.60
2.40
3.20
82.04
4.00
4.80
5.60
6.40
7.20
78.04
8.00
8.80
9.60
10.40
11.20
74.04
12.00
12.80



SOIL PROFILE AND TEST DATA

Proposed Residential Subdivision
South 1/2 Lot 26, Concession "A", R.F.
Nepean, Ontario

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Inspection & Testing Services
Damage Claims

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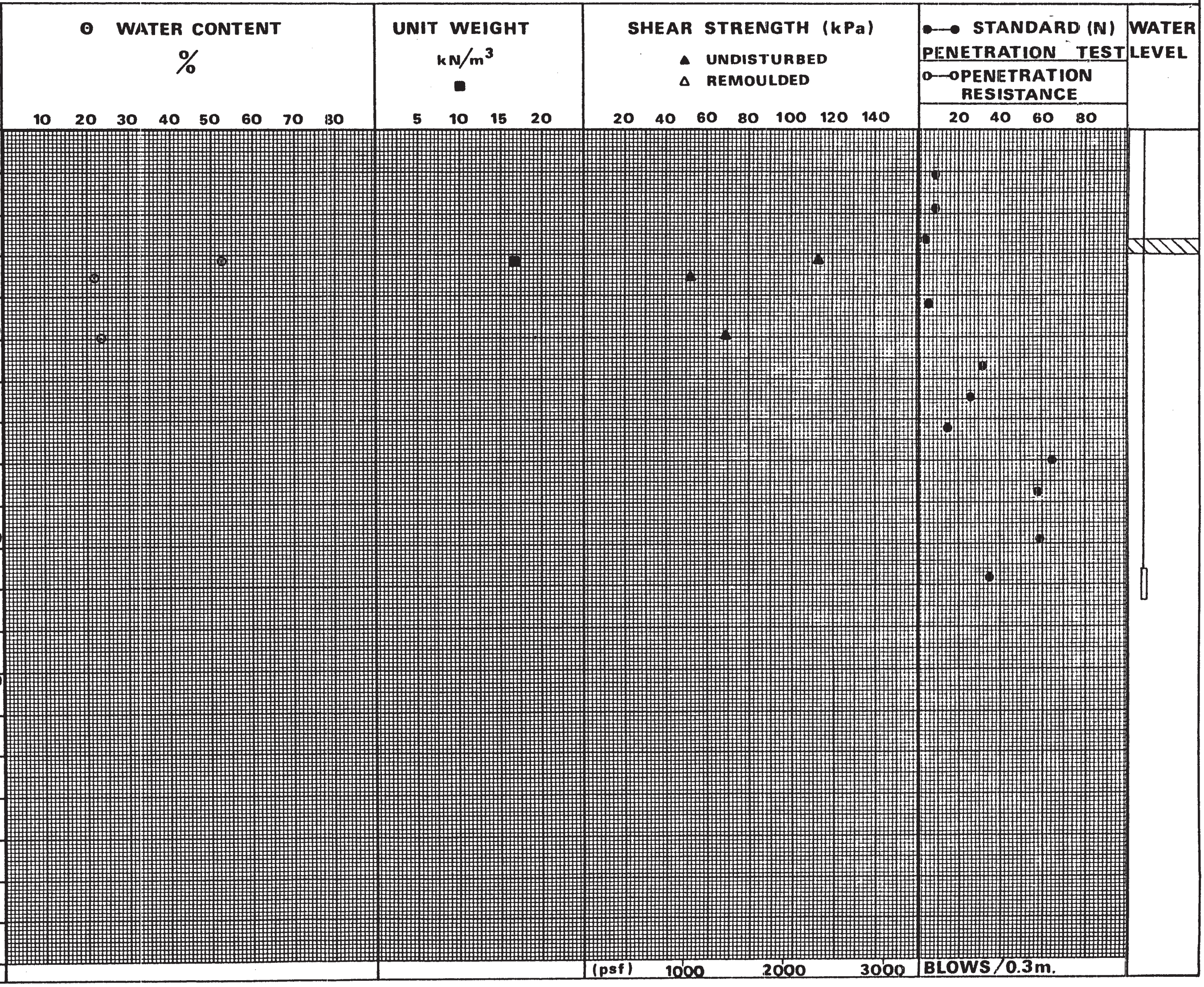
Telephone (613) 728-3505

SHEET NO. 2 OF 3
HOLE NO. BH 2
GROUND SURFACE 80.59 BOTTOM HOLE 71.59
BEDROCK ______ GROUNDWATER DRY

DESCRIPTION	LEGEND	SAMPLE TYPE	SAMPLE NUMBER	ELEV. DEPTH
Ground Surface				80.59
300 mm TOPSOIL over a loose brown SANDY SILT interbedded with layers of clayey silt and fine sand.		G	21	0.00
		SS	22	0.80
		SS	23	1.60
2.2		SS	24	
Stiff fissured olive grey SILTY CLAY 2.9		SS	25	2.40
Compact grey FINE SANDY SILT with a trace of clay		SS	26	3.20
				76.59
		TW	27	4.00
		SS	28	4.80
		SS	29	
6.1		SS	30	5.60
Dense light brown to grey SILTY FINE SAND		SS	31	6.40
		SS	32	7.20
		SS	33	72.59
				8.00
9.0		SS	34	8.80
Borehole terminated in silty fine sand				9.60
				10.40



SOIL PROFILE AND TEST DATA

Proposed Residential Subdivision
South 1/2 Lot 26, Concession "A", R.F.
Nepean, Ontario

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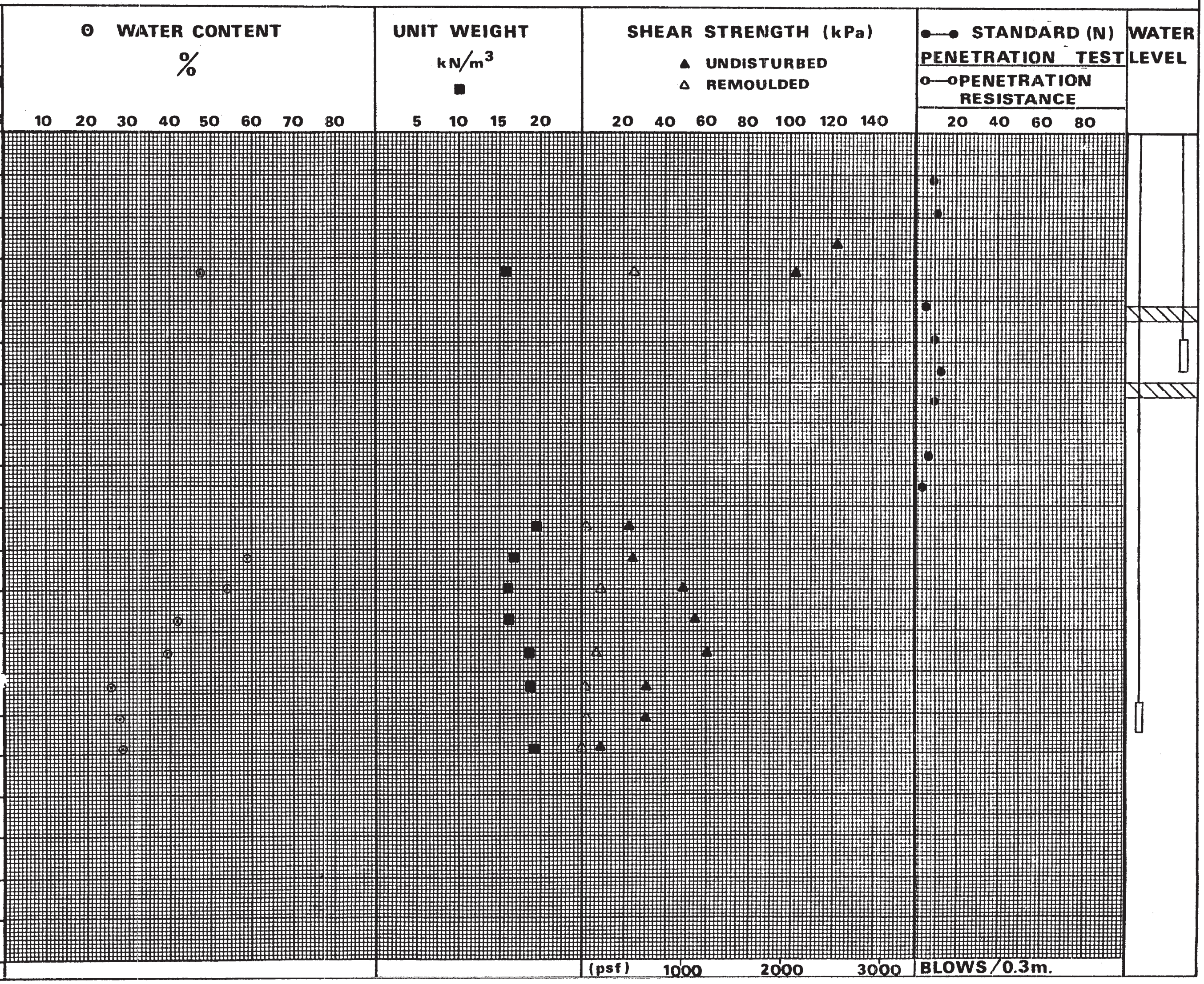
SHEET NO. 3 OF 3
HOLE NO. BH 3
GROUND SURFACE 85.53 BOTTOM HOLE 73.33
BEDROCK ______ GROUNDWATER DRY

DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	ELEV. DEPTH
Ground Surface			85.53
250 mm TOPSOIL over a loose brown SANDY SILT interbedded with clayey silt & sand	G	35	0.00
0.8	SS	36	0.80
Stiff olive grey fissured SILTY CLAY containing brown fine sand seams at 50 mm ± intervals	SS	37	1.60
	TW	38	
	TW	39	2.40
3.3	SS	40	3.20
	SS	41	81.53
	SS	42	4.00
	SS	43	4.80
			5.60
Compact brown SILTY FINE SAND containing clayey silt seams	SS	44	
	SS	45	6.40
	TW	46	7.20
7.9			77.53
Firm to stiff grey fissured SILTY CLAY with occasional fine sand lenses and containing fine sand seams	TW	47	8.00
	TW	48	8.80
	TW	49	
9.8	TW	50	9.60
STRATIFIED SILT: grey compact layers of silty sand, sandy silt and stiff silty clay	TW	51	10.40
	TW	52	11.20
	TW	53	73.53
12.2			12.00
Borehole terminated in silt			



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 = $(D30)^2 / (D10 \times D60)$
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)

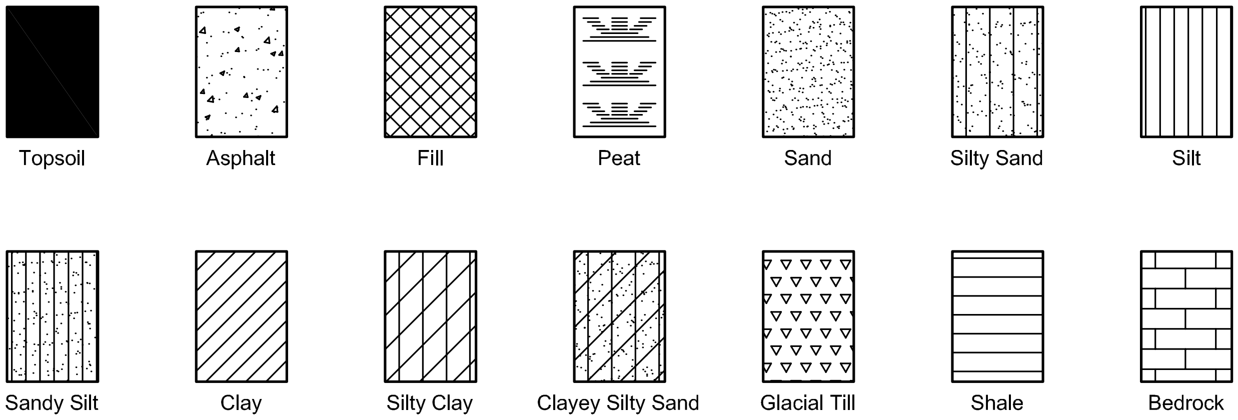
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

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.

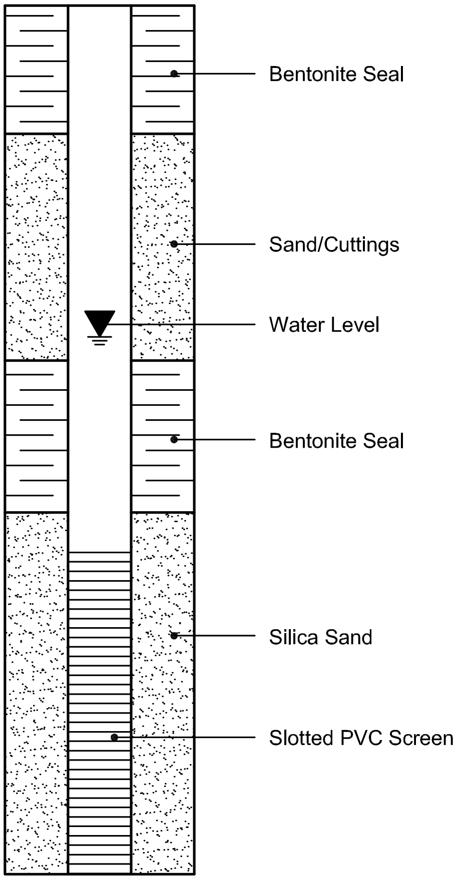
SYMBOLS AND TERMS (continued)

STRATA PLOT

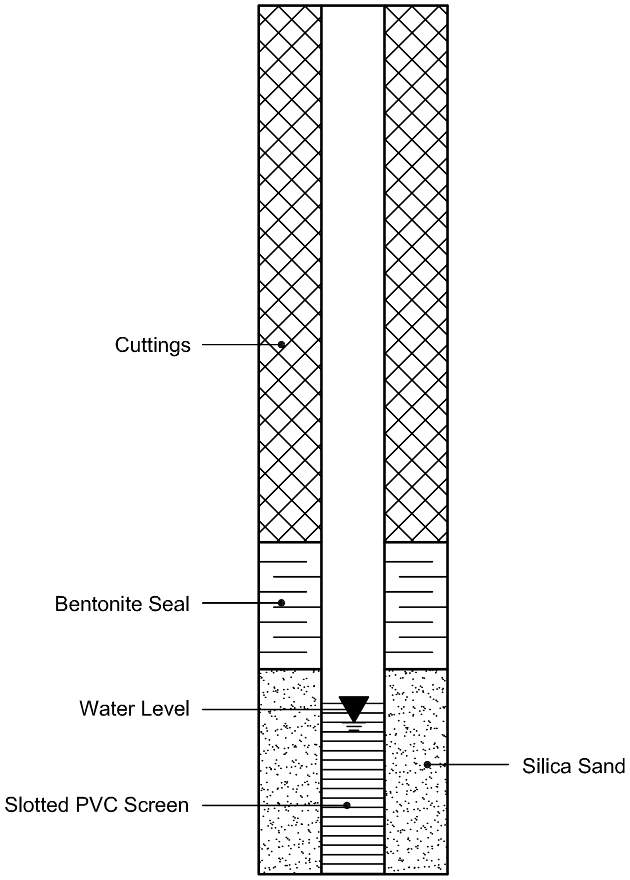


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



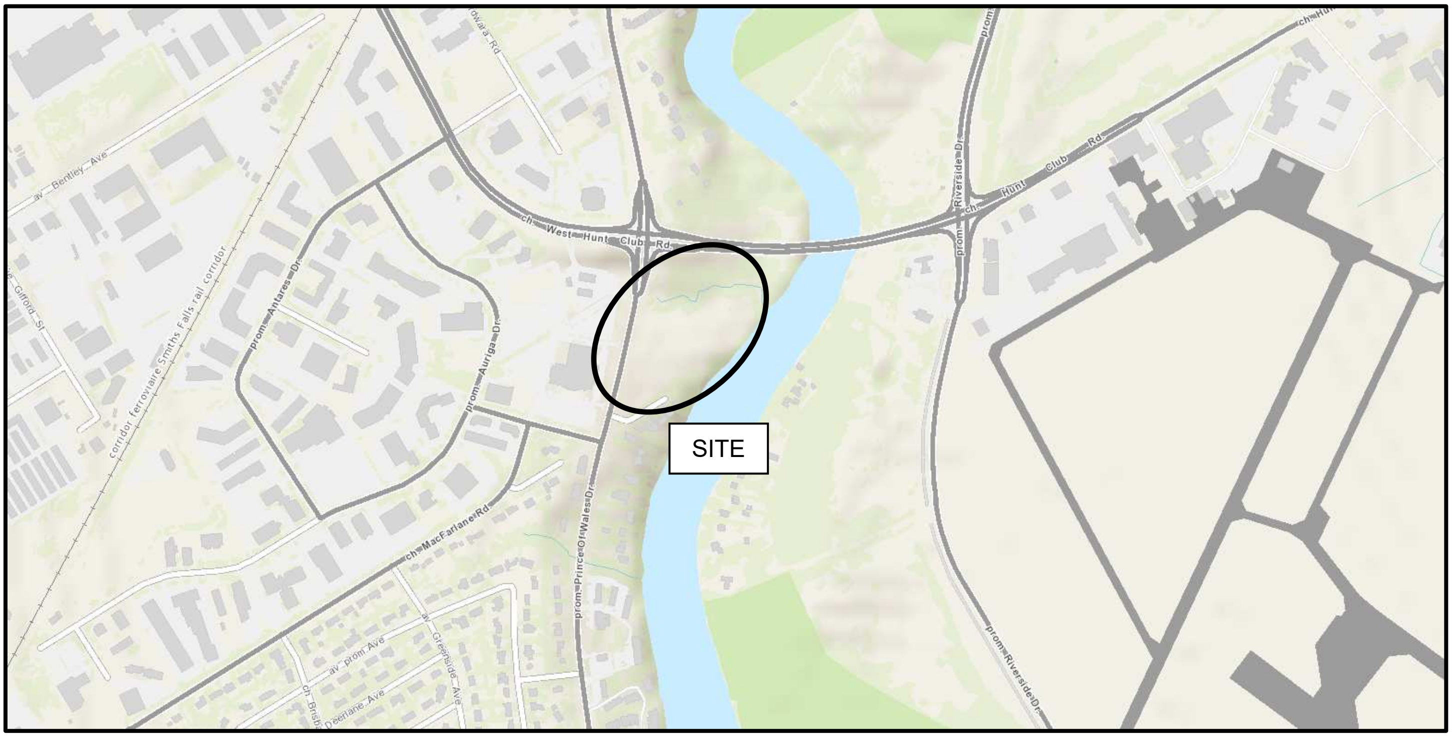
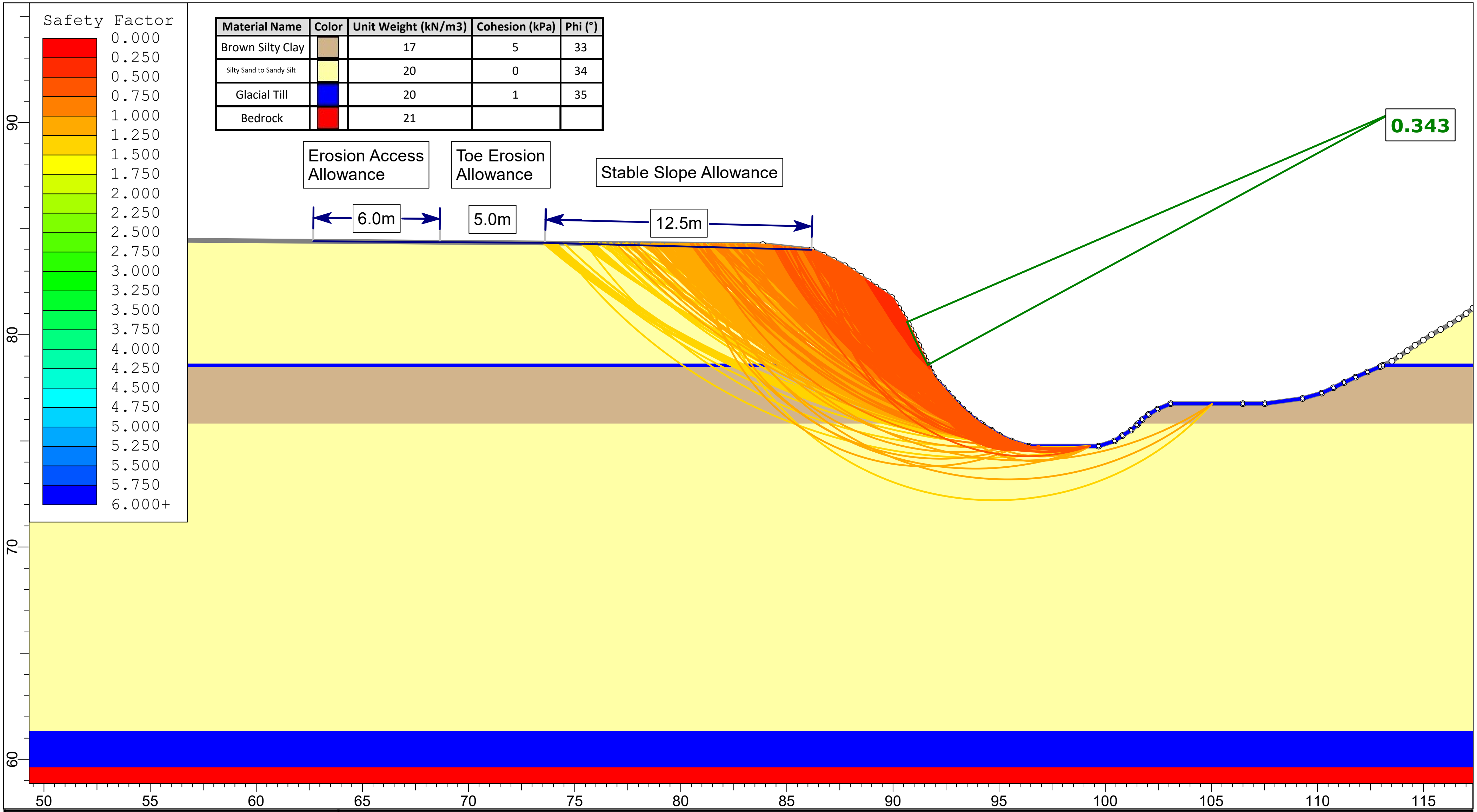


FIGURE 1

KEY PLAN

PATERSON
GROUP



Material Name	Color	Unit Weight (kN/m3)	Cohesion (kPa)	Phi (°)
Brown Silty Clay	🟫	17	5	33
Silty Sand to Sandy Silt	🟨	20	0	34
Glacial Till	🟦	20	1	35
Bedrock	🟥	21		

PATERSON GROUP

SLIDEINTERPRET 9.036

Project No.:

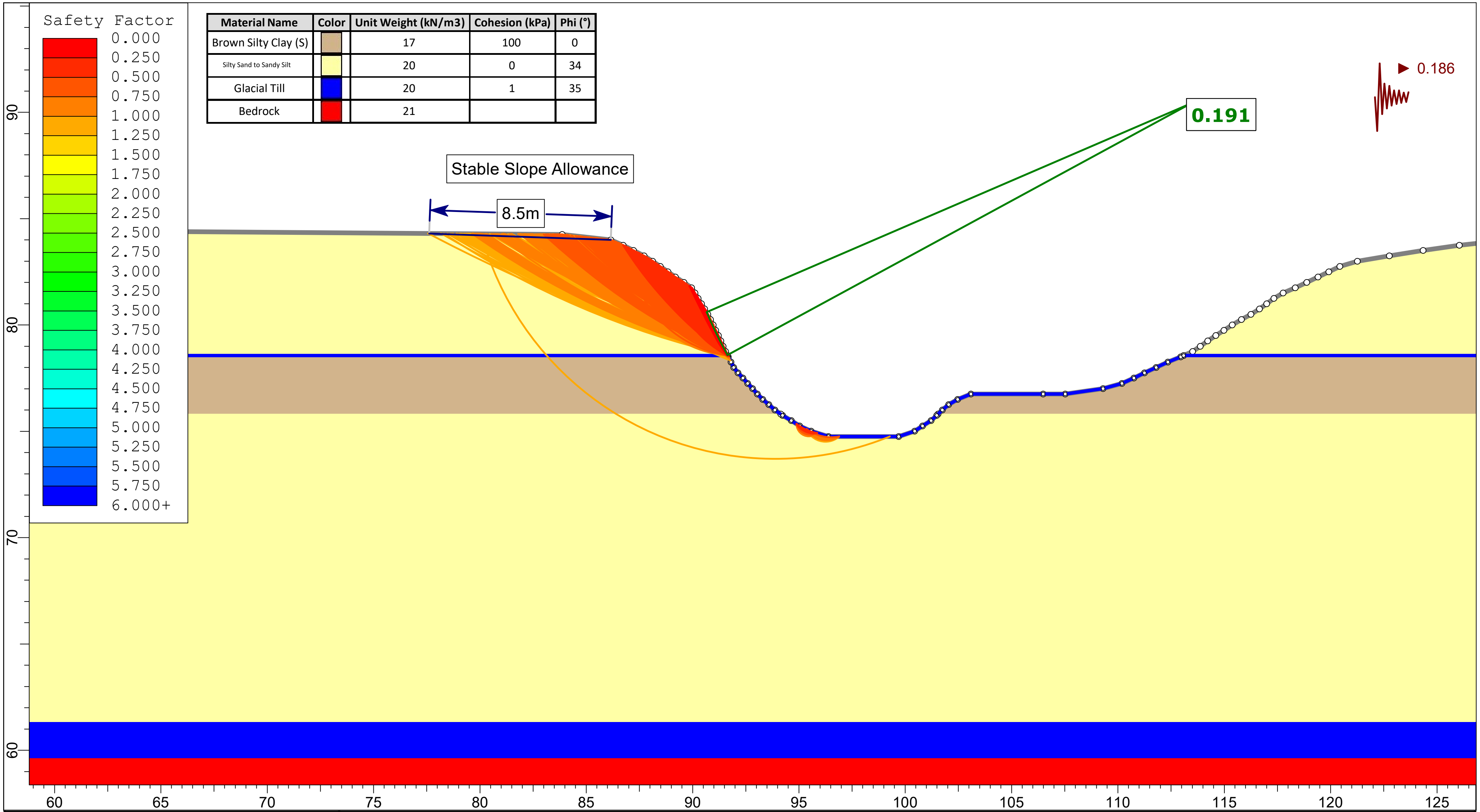
Myers Automotive Group
Slope Stability Analysis
2175 Prince of Wales Drive, Ottawa, Ontario

Drawing No.:

Figure 1A - Section A - Static Loading

Prepared By: NFRV

File No. PG6557



Material Name	Color	Unit Weight (kN/m3)	Cohesion (kPa)	Phi (°)
Brown Silty Clay (S)	🟫	17	100	0
Silty Sand to Sandy Silt	🟨	20	0	34
Glacial Till	🟦	20	1	35
Bedrock	🟥	21		

PATERSON GROUP

SLIDEINTERPRET 9.036

Project No.:

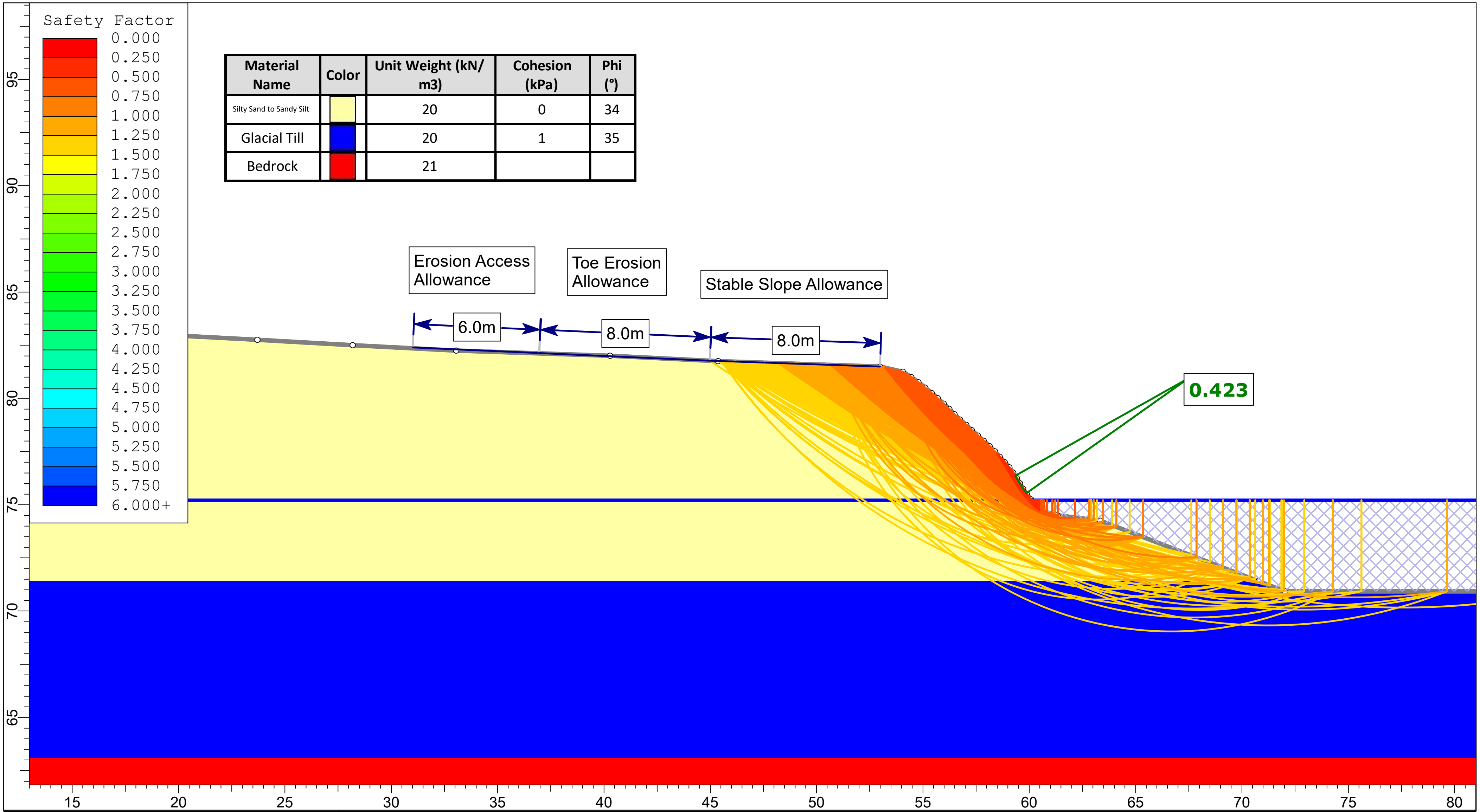
Myers Automotive Group
Slope Stability Analysis
2175 Prince of Wales Drive, Ottawa, Ontario

Drawing No.:

Figure 1B - Section A - Seismic Loading

Prepared By: NFRV

File No. PG6557



Material Name	Color	Unit Weight (kN/m3)	Cohesion (kPa)	Phi (°)
Silty Sand to Sandy Silt	🟨	20	0	34
Glacial Till	🟦	20	1	35
Bedrock	🟥	21		

PATERSON GROUP

SLIDEINTERPRET 9.036

Project No.:

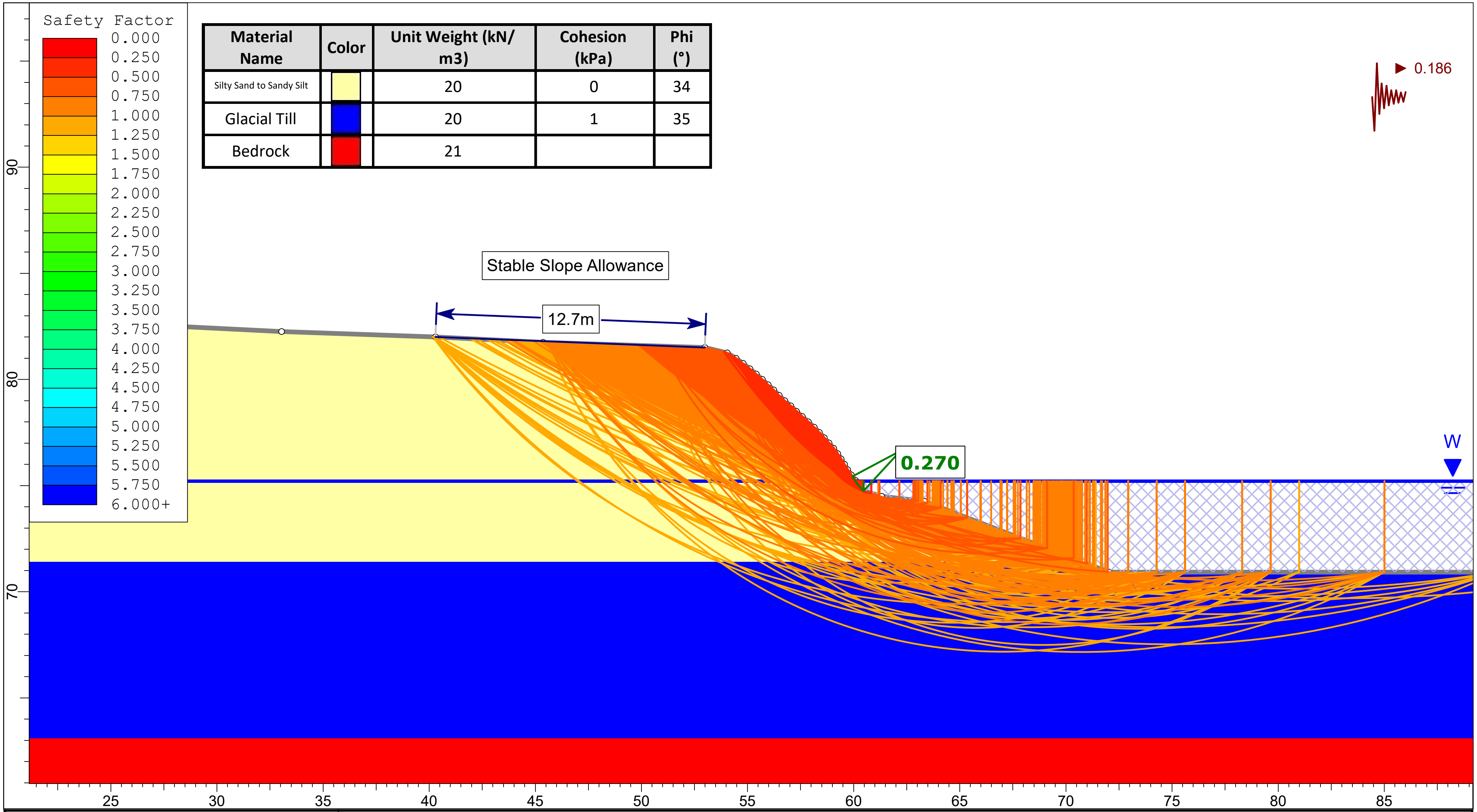
Myers Automotive Group
Slope Stability Analysis
2175 Prince of Wales Drive, Ottawa, Ontario

Drawing No.:

Figure 2A - Section B - Static Loading

Prepared By: NFRV

File No. PG6557



Material Name	Color	Unit Weight (kN/m3)	Cohesion (kPa)	Phi (°)
Silty Sand to Sandy Silt	🟨	20	0	34
Glacial Till	🟦	20	1	35
Bedrock	🟥	21		

PATERSON GROUP

SLIDEINTERPRET 9.036

Project No.:

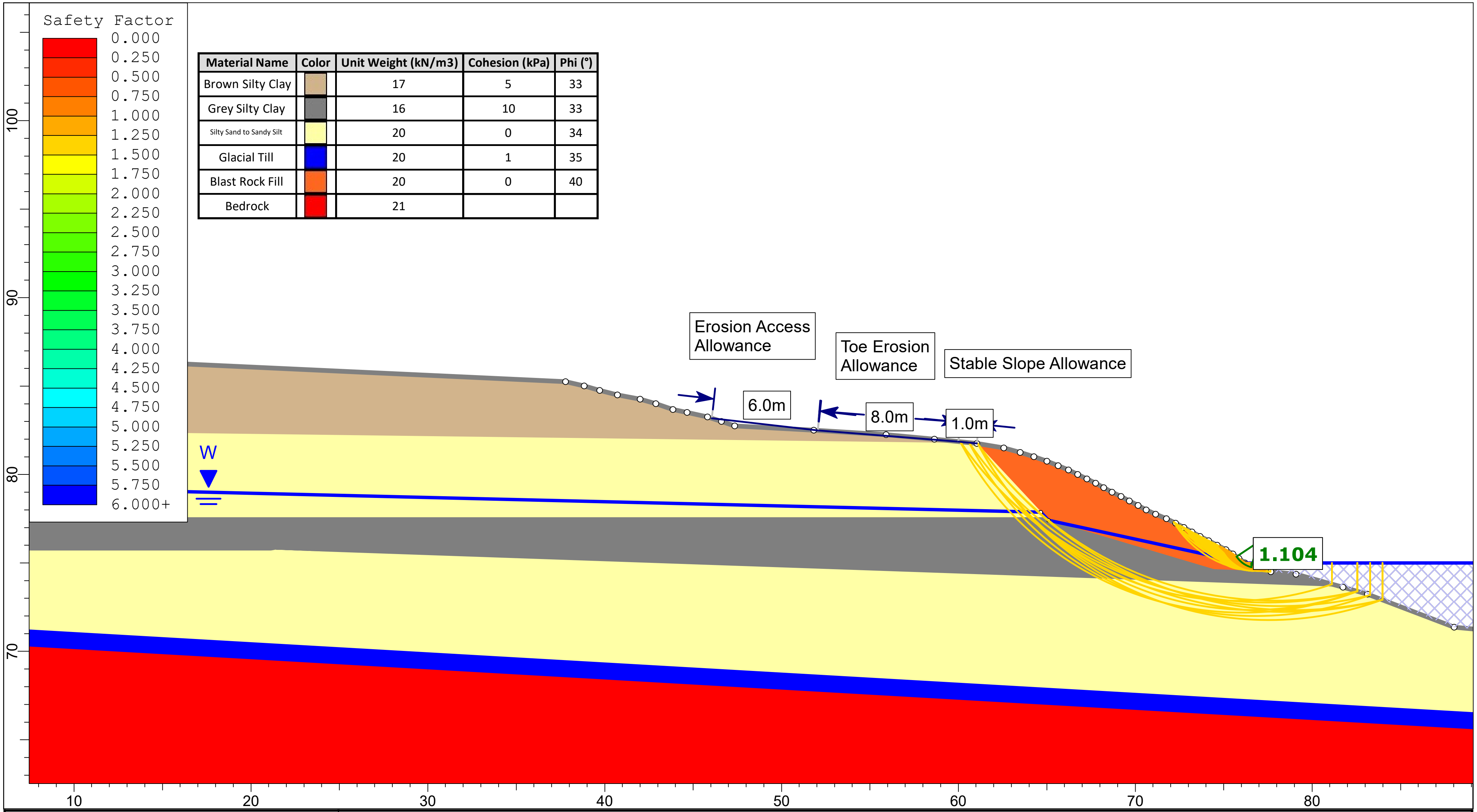
Myers Automotive Group
Slope Stability Analysis
2175 Prince of Wales Drive, Ottawa, Ontario

Drawing No.:

Figure 2B - Section B - Seismic Loading

Prepared By: NFRV

File No. PG6557



Material Name	Color	Unit Weight (kN/m3)	Cohesion (kPa)	Phi (°)
Brown Silty Clay	🟫	17	5	33
Grey Silty Clay	⬛	16	10	33
Silty Sand to Sandy Silt	🟨	20	0	34
Glacial Till	🟦	20	1	35
Blast Rock Fill	🟧	20	0	40
Bedrock	🟥	21		

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Project No.:

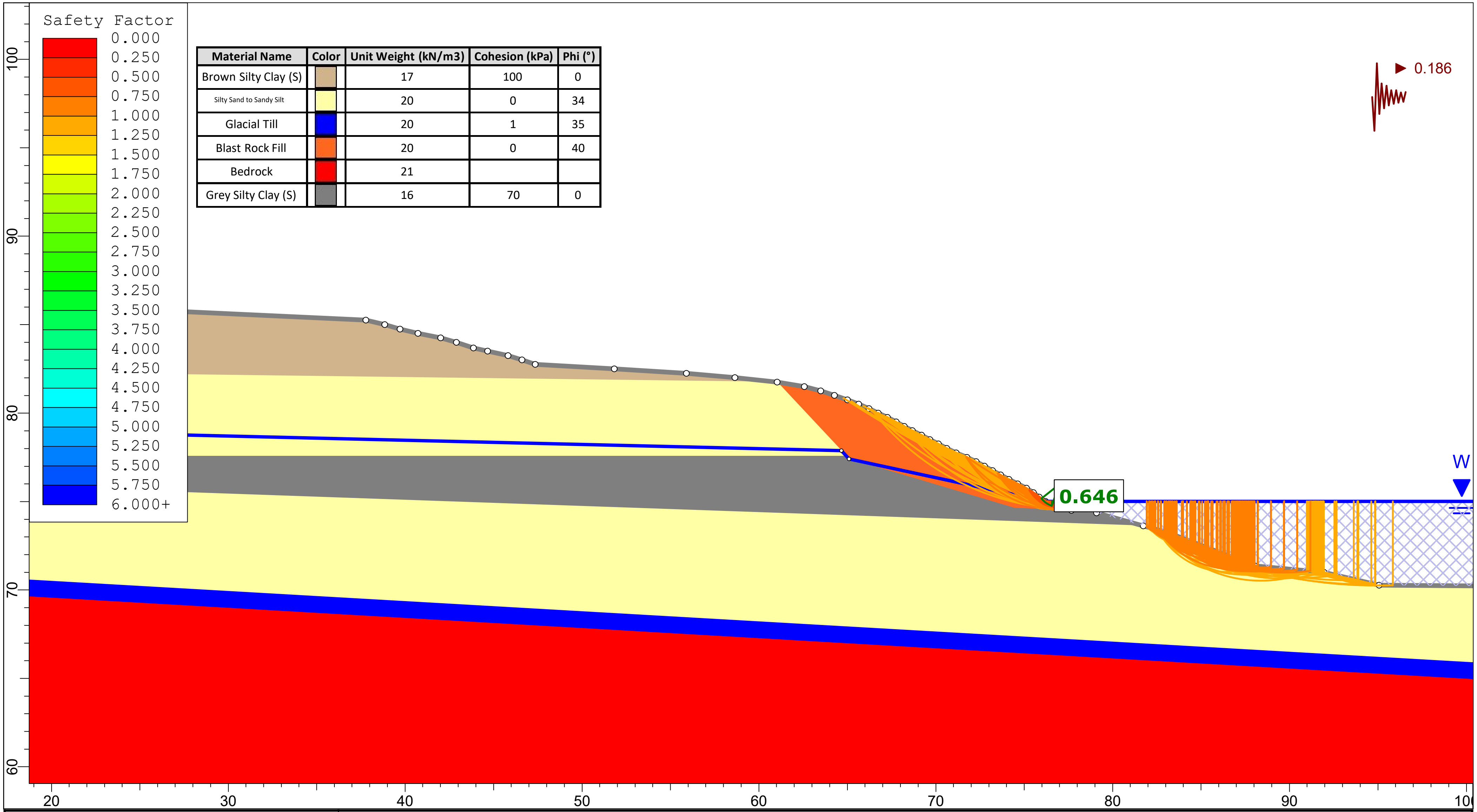
Myers Automotive Group
Slope Stability Analysis
2175 Prince of Wales Drive, Ottawa, Ontario

Drawing No.:

Figure 3A - Section C - Static Loading

Prepared By: NFRV

File No. PG6557



Material Name	Color	Unit Weight (kN/m3)	Cohesion (kPa)	Phi (°)
Brown Silty Clay (S)	🟫	17	100	0
Silty Sand to Sandy Silt	🟨	20	0	34
Glacial Till	🟦	20	1	35
Blast Rock Fill	🟧	20	0	40
Bedrock	🟥	21		
Grey Silty Clay (S)	▨	16	70	0

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Project No.:

Myers Automotive Group
Slope Stability Analysis
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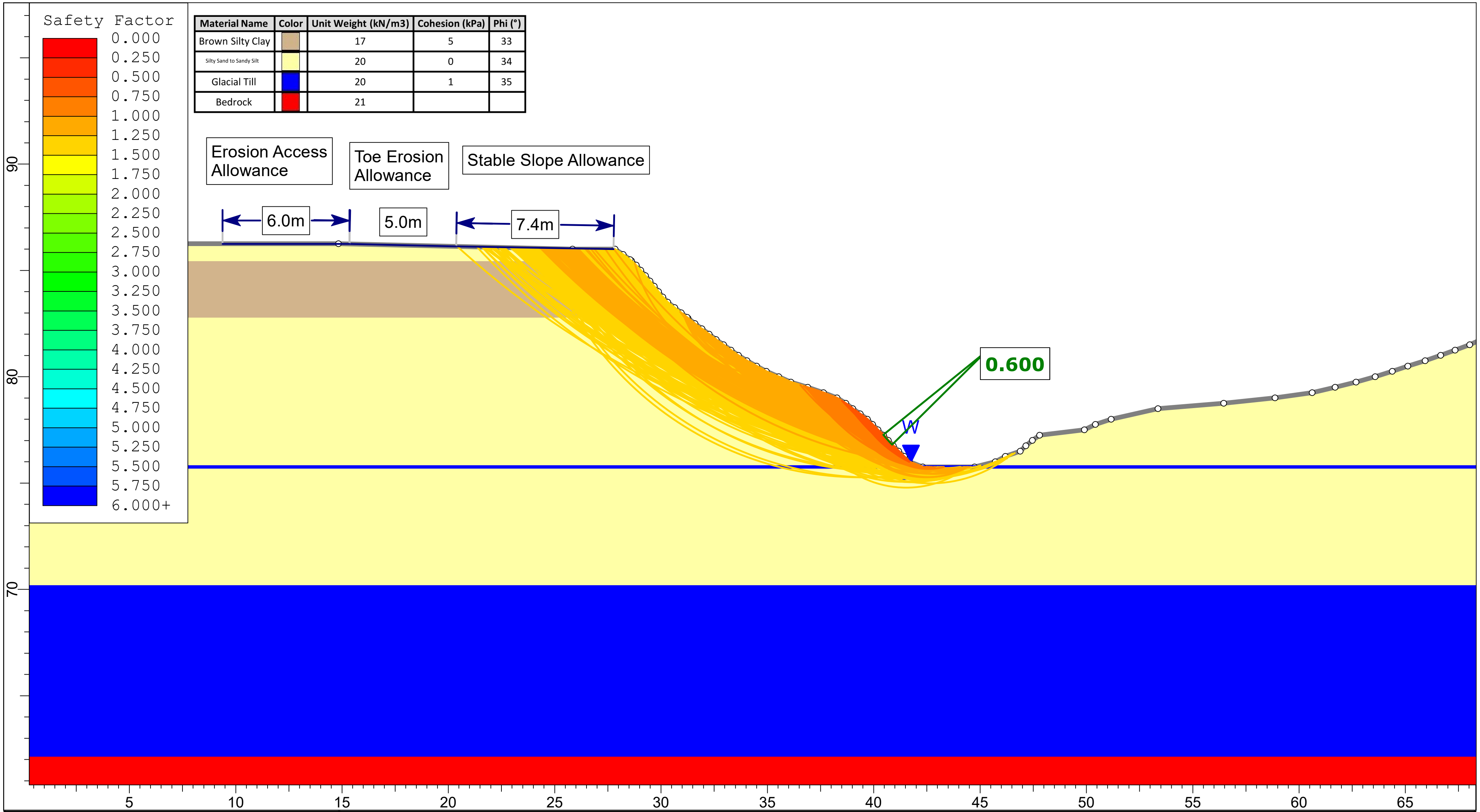
Drawing No.:

Figure 3B - Section C - Seismic Loading

Prepared By: NFRV

File No. PG6557



Material Name	Color	Unit Weight (kN/m3)	Cohesion (kPa)	Phi (°)
Brown Silty Clay	🟫	17	5	33
Silty Sand to Sandy Silt	🟨	20	0	34
Glacial Till	🟦	20	1	35
Bedrock	🟥	21		

PATERSON GROUP

SLIDEINTERPRET 9.036

Project No.:

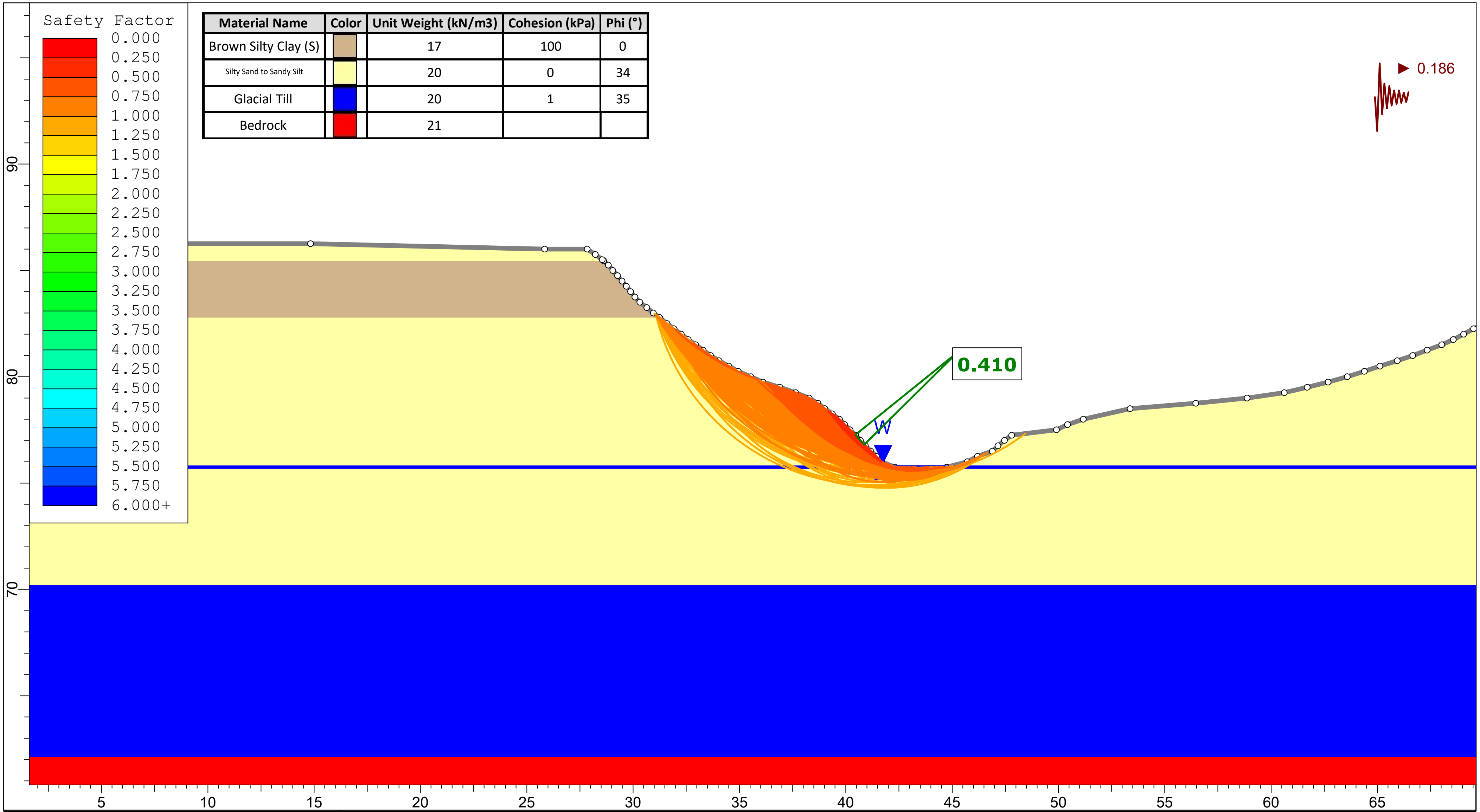
Myers Automotive Group
Slope Stability Analysis
2175 Prince of Wales Drive, Ottawa, Ontario

Drawing No.:

Figure 4A - Section D - Static Loading

Prepared By: NFRV

File No. PG6557



Material Name	Color	Unit Weight (kN/m3)	Cohesion (kPa)	Phi (°)
Brown Silty Clay (S)	🟫	17	100	0
Silty Sand to Sandy Silt	🟨	20	0	34
Glacial Till	🟦	20	1	35
Bedrock	🟥	21		

PATERSON GROUP

SLIDEINTERPRET 9.036

Project No.:

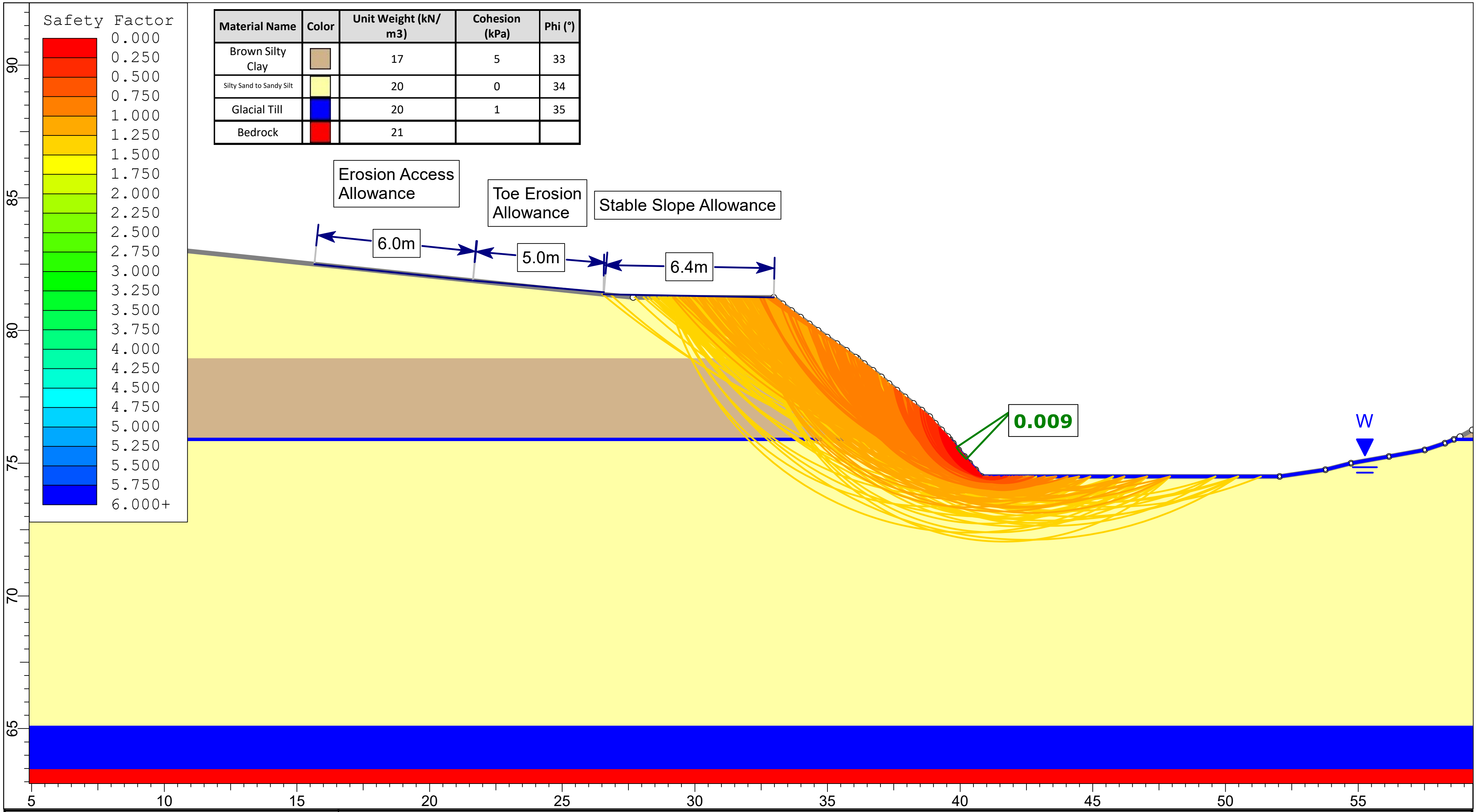
Myers Automotive Group
Slope Stability Analysis
2175 Prince of Wales Drive, Ottawa, Ontario

Drawing No.:

Figure 4B - Section D - Seismic Loading

Prepared By: NFRV

File No. PG6557

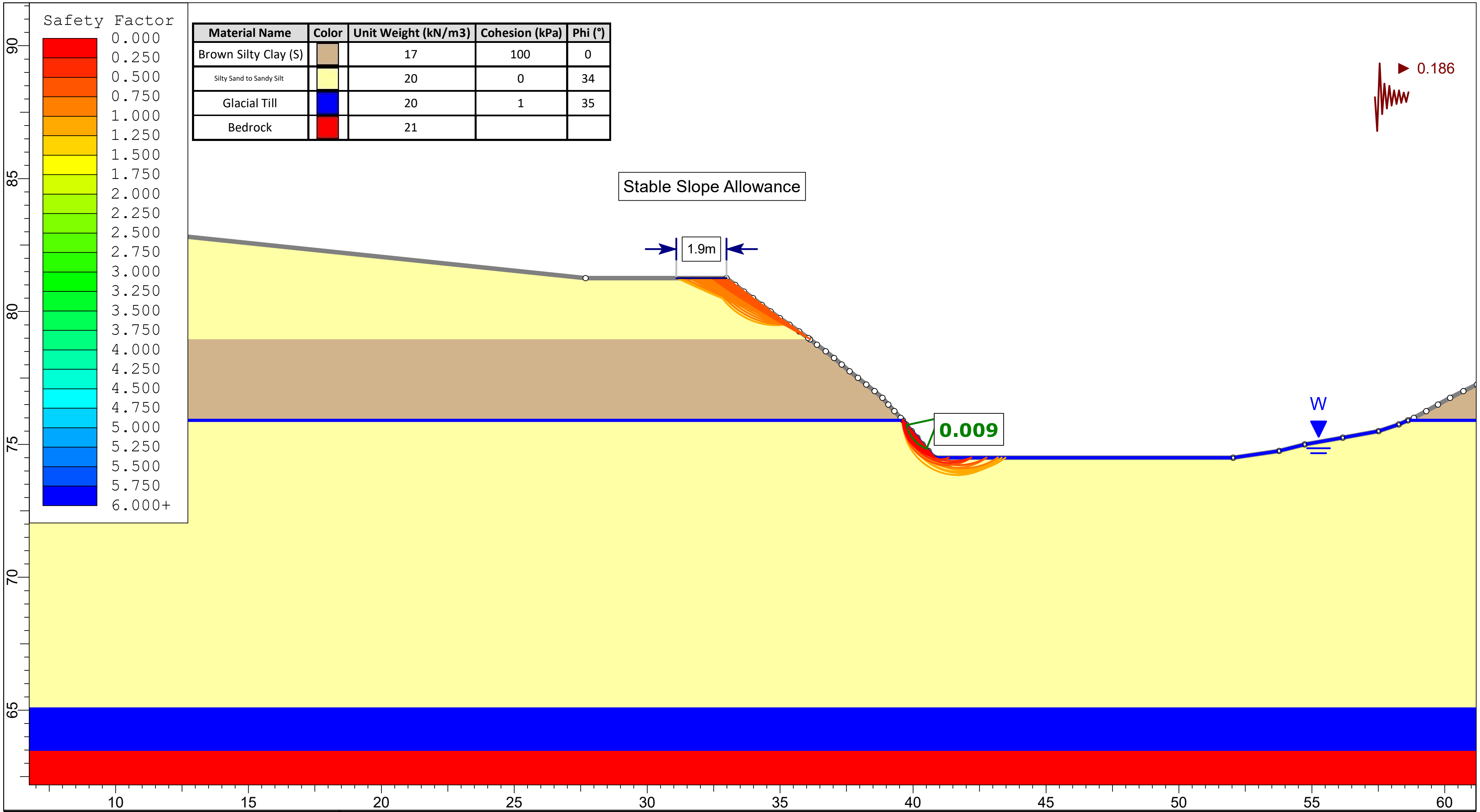


Material Name	Color	Unit Weight (kN/m3)	Cohesion (kPa)	Phi (°)
Brown Silty Clay	🟫	17	5	33
Silty Sand to Sandy Silt	🟨	20	0	34
Glacial Till	🟦	20	1	35
Bedrock	🟥	21		

PATERSON GROUP

SLIDEINTERPRET 9.036

Project No.:	**Myers Automotive Group** **Slope Stability Analysis** **2175 Prince of Wales Drive, Ottawa, Ontario**
Drawing No.:	**Figure 5A - Section E - Static Loading**
Prepared By: NFRV	*File No.* PG6557



Material Name	Color	Unit Weight (kN/m3)	Cohesion (kPa)	Phi (°)
Brown Silty Clay (S)	🟫	17	100	0
Silty Sand to Sandy Silt	🟨	20	0	34
Glacial Till	🟦	20	1	35
Bedrock	🟥	21		

PATERSON GROUP

SLIDEINTERPRET 9.036

Project No.:	**Myers Automotive Group** **Slope Stability Analysis** **2175 Prince of Wales Drive, Ottawa, Ontario**
Drawing No.:	**Figure 5B - Section E - Seismic Loading**
Prepared By: NFRV	*File No.* PG6557

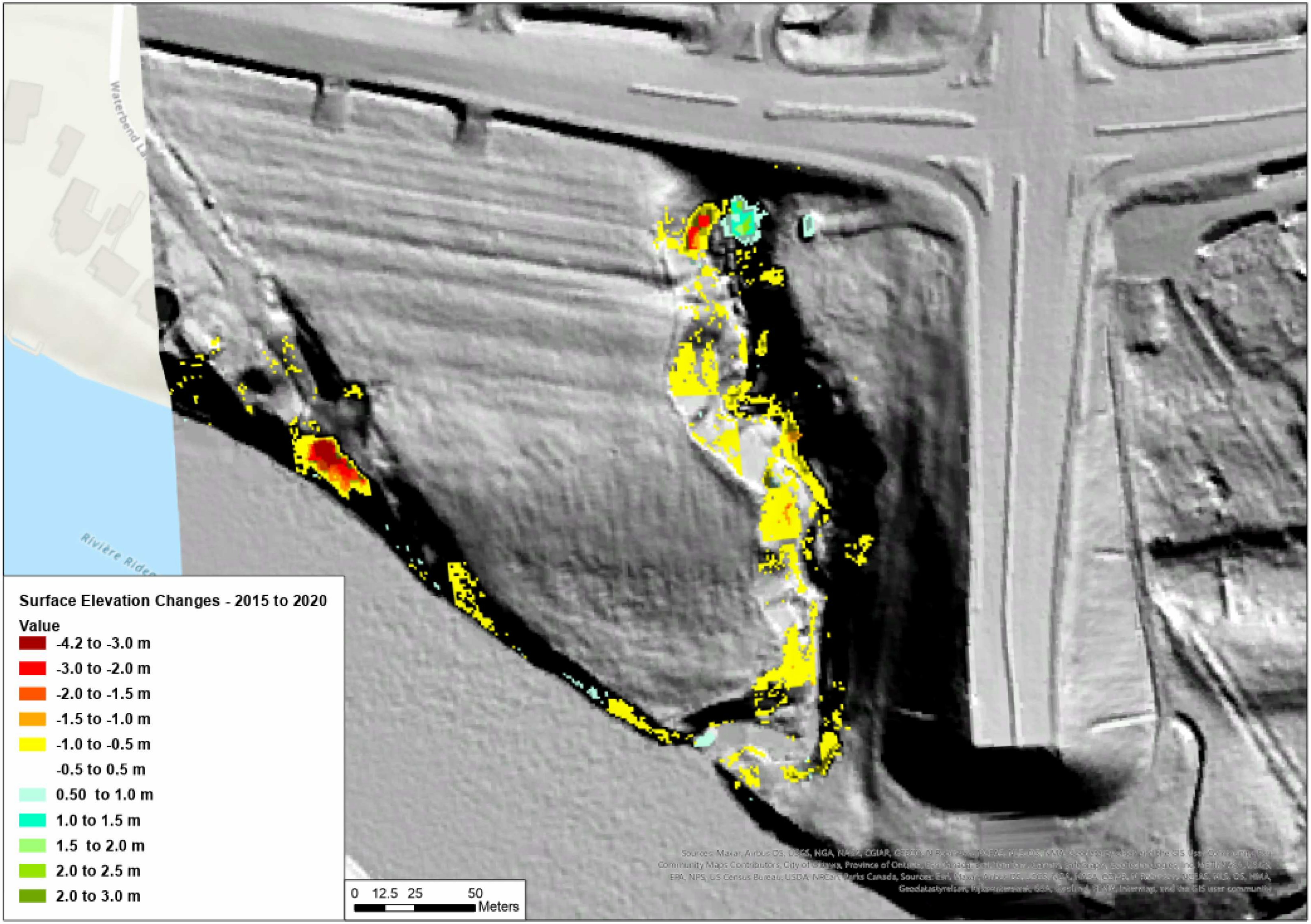
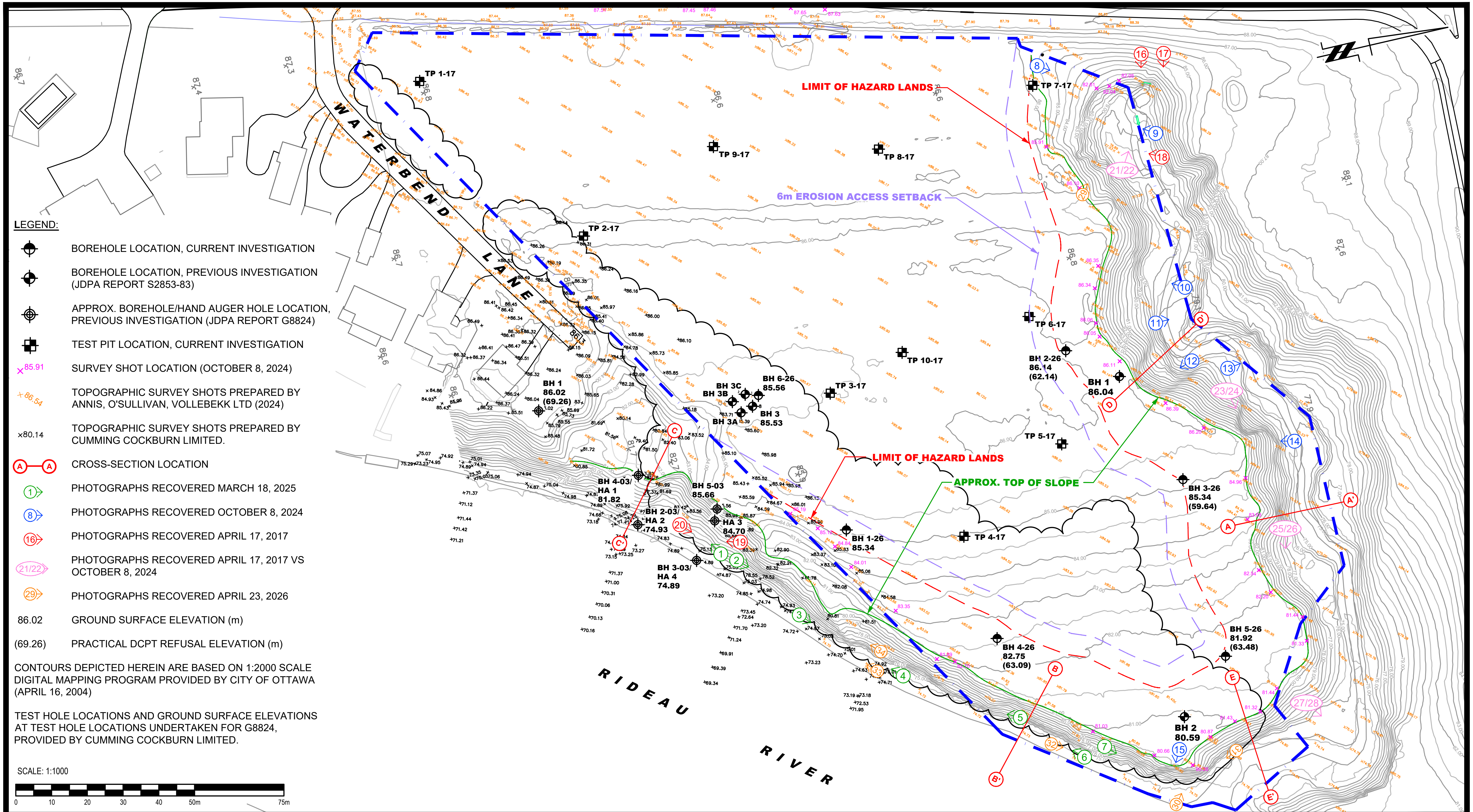


FIGURE 6 - LiDAR MAPPING - SURFACE ELVATION CHANGES FROM 2015 TO 2020

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



LEGEND:

⊕ BOREHOLE LOCATION, CURRENT INVESTIGATION

⊕ BOREHOLE LOCATION, PREVIOUS INVESTIGATION (JDPA REPORT S2853-83)

⊕ APPROX. BOREHOLE/HAND AUGER HOLE LOCATION, PREVIOUS INVESTIGATION (JDPA REPORT G8824)

■ TEST PIT LOCATION, CURRENT INVESTIGATION

×85.91 SURVEY SHOT LOCATION (OCTOBER 8, 2024)

×86.54 TOPOGRAPHIC SURVEY SHOTS PREPARED BY ANNIS, O'SULLIVAN, VOLLEBEKK LTD (2024)

×80.14 TOPOGRAPHIC SURVEY SHOTS PREPARED BY CUMMING COCKBURN LIMITED.

A—A CROSS-SECTION LOCATION

1 PHOTOGRAPHS RECOVERED MARCH 18, 2025

8 PHOTOGRAPHS RECOVERED OCTOBER 8, 2024

16 PHOTOGRAPHS RECOVERED APRIL 17, 2017

21/22 PHOTOGRAPHS RECOVERED APRIL 17, 2017 VS OCTOBER 8, 2024

29 PHOTOGRAPHS RECOVERED APRIL 23, 2026

86.02 GROUND SURFACE ELEVATION (m)

(69.26) PRACTICAL DCPT REFUSAL ELEVATION (m)

CONTOURS DEPICTED HEREIN ARE BASED ON 1:2000 SCALE DIGITAL MAPPING PROGRAM PROVIDED BY CITY OF OTTAWA (APRIL 16, 2004)

TEST HOLE LOCATIONS AND GROUND SURFACE ELEVATIONS AT TEST HOLE LOCATIONS UNDERTAKEN FOR G8824, PROVIDED BY CUMMING COCKBURN LIMITED.

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	DD/MM/YYYY	INITIAL
3	REVISED LIMIT OF HAZARD LANDS	18/06/2026	DP
2	ADDED 2026 BOREHOLE LOCATIONS	27/04/2026	DP
1	UPDATED BASED ON CITY COMMENTS	24/02/2026	YZ

MYERS AUTOMOTIVE GROUP
GEOTECHNICAL INVESTIGATION
PROPOSED COMMERCIAL DEVELOPMENT
2175 PRINCE OF WALES DRIVE

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:1000
Date: 05/2025
Drawn by: NFRV
Report No.: PG6557-LET.01
Checked by: NFRV
Dwg. No.: PG6557-1
Approved by: DP
Revision No.: 3

p:\autocad drawings\geotechnical\pg65xx\pg6557\pg6557-1 thlp (rev.03).dwg

11x17