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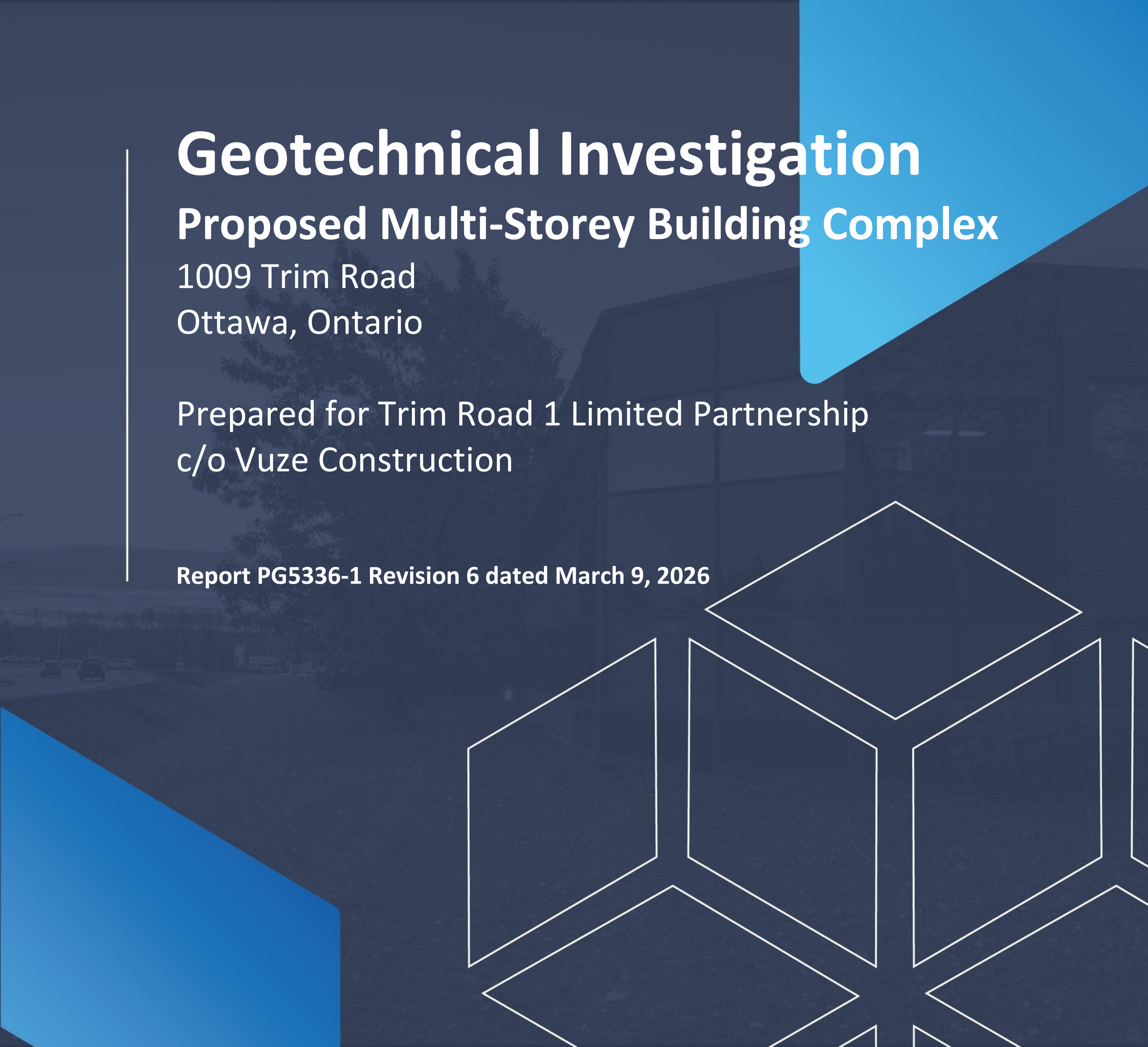

Geotechnical Investigation

Proposed Multi-Storey Building Complex

1009 Trim Road
Ottawa, Ontario

Prepared for Trim Road 1 Limited Partnership
c/o Vuze Construction

Report PG5336-1 Revision 6 dated March 9, 2026

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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by Trim Road 1 Limited Partnership, care of Vuze Construction to carry out a geotechnical investigation for the proposed multi-storey building complex to be located at 1009 Trim Road, in the City of Ottawa (refer to Figure 1 - Key Plan in Appendix 2 of this report).

The objective of the geotechnical investigation was to:

- Determine the subsoil and groundwater conditions at this site by means of a test hole program.

- Provide geotechnical recommendations pertaining to the design of the proposed development including construction considerations which may affect the design.

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

Investigating the presence or potential presence of contamination on the subject site was not part of the scope of work of the present investigation. Therefore, the present report does not address environmental issues.

2.0 Proposed Development

Based on available drawings, the proposed complex will consist of four high rise residential buildings. It is understood that each tower will be constructed over a common podium consisting of an underground parking structure extending 3 levels under finished grade along Trim Road. The podium levels of Residential Tower B2 and B3 will be connected by a 1-storey podium level that will be constructed within the area between the two buildings. The development will also include associated asphalt covered parking areas, access lanes and landscaped areas. It is further anticipated that the site will be municipally serviced.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the current investigation was carried out on June 29 to July 2, 2020, and consisted of a total of 4 boreholes drilled and sampled to a maximum depth of 15.9 m below the existing grade. A dynamic cone penetration test (DCPT) was carried out at two boreholes (BH 3 and BH 4) to determine inferred bedrock depth which ranged from 34.0 to 41.8 m below the existing grade. A previous field program was carried out by others in 2016. At that time a total of 6 boreholes and 4 test pits were advanced to a maximum depth of 47.9 m below the existing grade. These locations of these test holes are illustrated on Drawing PG5336- 1 - Test Hole Location Plan included in Appendix 2.

The borehole locations for the current investigation were determined in the field by Paterson personnel taking into consideration existing borehole coverage and existing site features. The locations of the boreholes are illustrated on Drawing PG5336-1 - Test Hole Location Plan included in Appendix 2.

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

Sampling and In Situ Testing

Soil samples from the boreholes were recovered from the auger flights or a 50 mm diameter split-spoon sampler. All soil samples were classified on site, placed in sealed plastic bags and transported to the laboratory for further review. The depths at which the auger and split spoon samples were recovered from the test holes are presented as AU and SS, respectively, on the Soil Profile and Test Data sheets presented in Appendix 1.

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

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

Undrained shear strength testing was carried out at regular depth intervals in cohesive soils. This testing was done in general accordance with ASTM D2573-08 - Standard Test Method for Field Vane Shear Test in Cohesive Soil. Reference should be made to the Soil Profile and Test Data Sheets provided in Appendix 1.

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

Sample Storage

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

3.2 Field Survey

The test hole locations were selected by Paterson to provide general coverage of the proposed development taking into consideration existing site features and underground utilities. The test hole locations and ground surface elevations at each test hole location were surveyed by Paterson personnel. The ground surface elevations at the borehole locations were referenced to a geodetic datum. The test hole locations are presented on Drawing PG5336-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

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

3.4 Analytical Testing

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

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

4.0 Observations

4.1 Surface Conditions

The majority of the subject site is gravel covered with large boulders. Small to medium sized trees are present on the property boundaries of the subject site that border Trim Road and Inlet Private. The southern portion of the site is relatively flat and slightly above grade from Inlet Private. The site slopes towards the Ottawa river to the north, following Trim Road. An approximately 2 m high pile of boulders was observed at the northwestern portion of the site. The existing grade within the site drops down by approximately 3 to 4 m, at the west end of the site towards Tweddle Road and the existing grade slopes up approximately 2 to 3 m at the east side towards the neighbouring property at 8899 Jeanne d'Arc Boulevard. The ground surface within the subject site slopes down gradually towards the northern portion of the site. The northern portion of the site is wet land from the Ottawa River. The site is bordered to the north by the Ottawa River, to the east by vacant treed land, to the west by Tweddle Road, and to the south by Trim Road.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile at the test hole locations consists of topsoil underlain by a fill consisting of silty sand mixed with clay and/or gravel. Fill consisting of boulders and blast rock were also noted on site. A very stiff brown silty clay deposit was encountered under the fill layer. The brown silty clay was underlain by a stiff grey silty clay layer. Practical refusal to DCPT was encountered in BH3 and BH4 between 34.0 and 41.8 m below existing grade.

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

Bedrock

Based on available geological mapping, the subject site is located in an area where the bedrock consists of interbedded limestone and dolomite of the Gull River formation. The overburden drift thickness is estimated to be between 20 to 35 m.

4.3 Groundwater

The groundwater level readings are presented in Table 1. It is important to note that groundwater level readings from piezometers and monitoring wells could be influenced by surface water infiltrating the backfilled boreholes within low permeability soils, such as at the subject site. Groundwater conditions can also be estimated based on the observed colour, moisture levels and consistency of the recovered soil samples. Based on Paterson's review of the recovered soil samples, the long-term groundwater level is expected to be at a depth ranging between 4 to 5 m below existing ground surface.

Table 1 – Summary of Groundwater Levels

Test Hole Number	Ground Surface Elevation (m)	Measured Groundwater Levels		Dated Recorded
		Depth (m)	Elevation (m)	
BH 1-20	46.87	4.72	42.15	July 17, 2020
BH 2-20	47.73	4.51	43.22	July 17, 2020
BH 3-20	49.31	4.93	44.38	July 17, 2020
MW 16-1	47.30	3.29	44.01	July 17, 2020
		1.50	45.80	April 7, 2016
MW 16-2	47.20	2.83	44.37	July 17, 2020
		5.50	41.70	April 7, 2016
MW 16-3	48.80	4.10	44.70	July 17, 2020
		5.02	43.78	April 7, 2016
MW 16-4	47.10	2.83	44.27	July 17, 2020
		2.00	45.10	April 7, 2016
MW 16-5	43.60	4.80	38.80	April 7, 2016
MW 16-6	43.00	1.10	41.90	July 17, 2020
		0.70	42.30	April 7, 2016

Notes: The ground surface elevations at the borehole locations are referenced to a geodetic datum.
- "*" indicates monitoring well installed within borehole.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is suitable for the proposed high-rise buildings. It is expected that the proposed high-rise buildings will be founded on end bearing piled foundations extending to the bedrock surface. It is also expected that the underground parking structure beyond the towers’ extent will be founded on conventional spread footings placed on an undisturbed, very stiff to stiff silty clay bearing surface.

A control joint between the piled foundation and the underground parking foundation can be considered to avoid differential settlement. The structural design will dictate if this is required.

Permissible Grade Raise

Due to the presence of a silty clay layer, the subject site is subjected to a permissible grade restriction. Our permissible grade raise recommendations are discussed in Subsection 5.3.

5.2 Site Grading and Preparation

Stripping Depth

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

Fill Placement

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

Non-specified existing fill along with site-excavated soil can be used as general landscaping fill where settlement of the ground surface is of minor concern. These materials should be spread in thin lifts and be compacted at minimum by the tracks of the spreading equipment to minimize voids. If these materials are to be used to build up the subgrade level for areas to be paved, they should be compacted in thin lifts to a minimum density of 95% of their respective SPMDD. Non-specified existing fill and site-excavated soils are not suitable for placement as backfill against foundation walls due to the frost heave potential of the site excavated soils below settlement sensitive areas, such as concrete sidewalks and exterior concrete entrance areas.

Fill used for grading beneath the base and subbase layers of paved areas should consist, unless otherwise specified, of clean imported granular fill, such as OPSS Granular A, Granular B Type II or select subgrade material. This material should be tested and approved prior to delivery to the site. The fill should be placed in lifts no greater than 300 mm thick and compacted using suitable compaction equipment for the lift thickness. Fill placed beneath the paved areas should be compacted to at least 95% of its SPMDD.

5.3 Foundation Design

Conventional Shallow Footings

Strip footings, up to 3 m wide, and pad footings, up to 5 m wide, placed over an undisturbed, very stiff brown silty clay bearing surface can be designed using bearing resistance value at serviceability limit states (SLS) of **200 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **300 kPa**. A geotechnical resistance factor of 0.5 was applied to the reported bearing resistance values at ULS.

Strip footings, up to 3 m wide, and pad footings, up to 5 m wide, placed over an undisturbed, stiff grey silty clay bearing surface can be designed using bearing resistance value at serviceability limit states (SLS) of **150 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **250 kPa**. A geotechnical resistance factor of 0.5 was applied to the reported bearing resistance values at ULS.

An undisturbed soil bearing surface consists of one from which all topsoil and deleterious materials, such as loose, frozen or disturbed soil, have been removed prior to the placement of concrete for footings.

For the parking garage, the bearing resistance value given for footings at SLS will be subjected to potential post construction total and differential settlements of 20 and 10 mm, respectively.

Footings on Lean Concrete

Where the underside of footings is located within the existing fill layer, consideration should be given to lower the footings to a native bearing surface.

Alternatively, footings can be placed over lean concrete in-filled trenches extending from design underside of footing level to the native bearing surface. The bearing surface surface should be reviewed and approved by the geotechnical consultant at the time of excavation. The near vertical, zero entry trench should extend at least 300 mm beyond the outside face of the footing and be in-filled with minimum 15 MPa lean concrete. It should be noted that the zero-entry trenches would be excavated through silty sand and therefore, the sidewalls could become unstable. Precautions should be taken during construction to ensure personnel and equipment are kept away from the top of the trenches (see Subsection 6.3).

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 when a plane extending down and out from the bottom edge of the footing at a minimum of 1H:1V passes only through in situ soil or engineered fill.

Piled Foundation

It is expected that the buildings will be constructed over concrete filled steel pipe piles driven to refusal on the bedrock surface.

For deep foundations, concrete-filled steel pipe piles are generally utilized in the Ottawa area. Applicable pile resistance at SLS values and factored pile resistance at ULS values are given in Table 2. A resistance factor of 0.4 has been incorporated into the factored ULS values. Note that these are all geotechnical axial resistance values.

The geotechnical pile resistance values were estimated using the Hiley dynamic formula, to be confirmed during pile installation with a program of dynamic monitoring. For this project, the dynamic monitoring of two (2) to four (4) piles would be recommended. This is considered to be the minimum monitoring program, as the piles under shear walls may be required to be driven using the maximum recommended driving energy to achieve the greatest factored resistance at ULS values. Re-striking of all piles at least once will also be required after at least 48 hours have elapsed since initial driving.

Table 2 – Pile Foundation Design Data

Pile Outside Diameter (mm)	Pile Wall Thickness (mm)	Geotechnical Axial Resistance		Final Set (blows/12mm)	Transferred Hammer Energy (kJ)
		SLS (kN)	Factored at ULS (kN)		
245	9	925	1110	6	27
245	11	1050	1260	6	31
245	13	1200	1440	6	35

Permissible Grade Raise Restrictions

Based on the results of our field investigation, a permissible grade raise restriction for the subject site of **2.0 m** can be used for design purposes. If higher than permissible grade raises are required, preloading with or without a surcharge, lightweight fill, and/or other measures should be investigated to reduce the risks of unacceptable long-term post construction total and differential settlements.

5.4 Design for Earthquakes

The subject site can be taken as seismic site response **Class D** as defined in Table 4.1.8.4.A of the Ontario Building Code (OBC) 2024 for foundations considered at this site.

Further to the above, it should be noted that liquefaction potential is assessed as part of the seismic design considerations. The silty clay deposit encountered at the subject site has been encountered during numerous geotechnical investigations completed by Paterson across the greater Ottawa area. Based on our experience, and supported by multiple laboratory testing results, this material would typically be considered highly plastic with a plasticity index (PI) greater than 20. Figure 6.15 of the Canadian Foundation Manual (2006) provides criteria for liquefaction assessment of fine-grained soils from Bray et al. (2004) as shown in Figure 1 below.

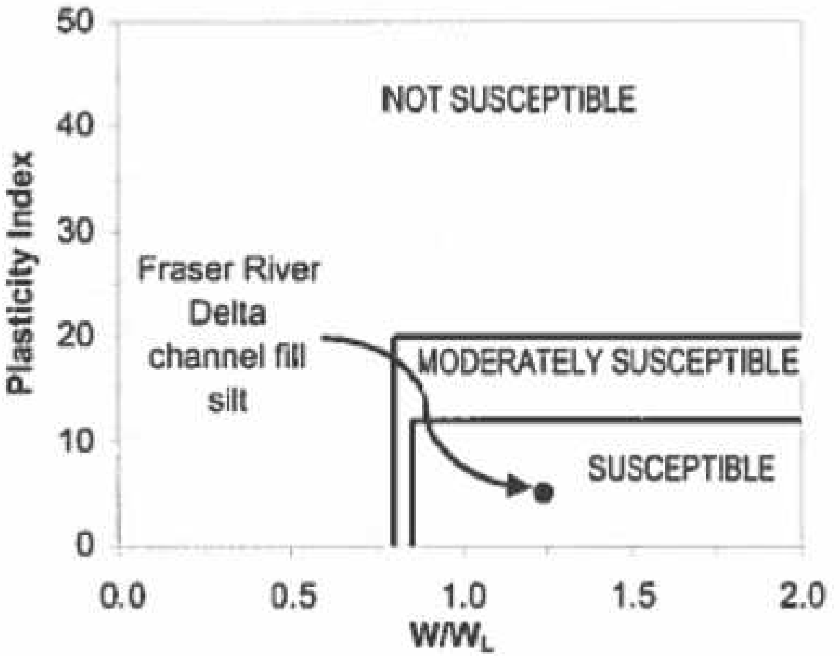


Figure 1 – Bray et al. (2004) criteria for liquefaction assessment of fine-grained soils

Based on the Atterberg Limits testing results conducted on the representative soils samples at the subject site resulting in Plasticity Index (PI) above 20 in conjunction with the site-specific shear wave velocity test results, the underlying soils at the subject site not considered susceptible to liquefaction or subsequent ‘earth flows’ from a geotechnical perspective.

Additionally, cyclic softening analyses were carried out using site-specific parameters following established methodologies (e.g., Idriss and Boulanger 2004, 2007, 2008) while considering the pertinent information in the Canadian Foundation Engineering Manual 2023 (CFEM 2023). The analysis results yielded factors of safety greater than 1.1, indicating that the soils within the local silty clay deposit are not expected to experience cyclic softening under the design seismic loading conditions.

5.5 Basement Slab

With the removal of all topsoil and deleterious fill, such as those containing organic materials, within the footprint of the proposed building, the in-situ soil or engineered fill surface will be considered to be an acceptable subgrade on which to commence backfilling for floor slab construction.

Any soft areas should be removed and backfilled with appropriate backfill material prior to placing any fill. OPSS Granular B Type II compacted to a minimum of 98% of the material’s SPMDD are recommended for backfilling below the floor slab.

It is expected that the basement area for the proposed building will be mostly parking, and the recommended pavement structure noted in Subsection 5.7 will be applicable. However, if storage or other uses of the lower level are proposed where a concrete floor slab will be used, it is recommended that the upper 200 mm of sub-slab fill consist of OPSS Granular A crushed stone compacted to 98% of the materials SPMDD.

A sub-slab drainage system, consisting of lines of perforated drainage pipe subdrains connected to a positive outlet, should be provided under the lowest level floor slab where a basement level is provided. The spacing of the sub-slab drainage pipes should be advised by Paterson during the design phase and once the footing and sump pit locations re known. The footprint would be confirmed at the time of construction once groundwater infiltration can be best assessed, if any. This is discussed further in Subsection 6.1.

5.6 Basement Wall

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

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

Lateral Earth Pressures

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

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

An additional pressure having a magnitude equal to $K_o \cdot q$ and acting on the entire height of the wall should be added to the above diagram for any surcharge loading, q (kPa), that may be placed at ground surface adjacent to the wall.

The surcharge pressure will only be applicable for static analyses and should not be used in conjunction with the seismic loading case.

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

Seismic Earth Pressures

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

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

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

The peak ground acceleration, (a_{max}), specific for the site is 0.405 g according to OBC 2024. Note that the vertical seismic coefficient is assumed to be zero.

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

The total earth force (P_{AE}) is considered to act at a height, h (m), from the base of the wall, where:
$h = \{P_o \cdot (H/3) + \Delta P_{AE} \cdot (0.6 \cdot H)\}/P_{AE}$

The earth forces calculated are unfactored. For the ULS case, the earth loads should be factored as live loads, as per OBC 2024.

5.7 Pavement Structure

The recommended pavement structures for the subject site are shown in Table 3, Table 4 and Table 5.

Table 3 – Recommended Pavement Structure – Car Only Parking Areas

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

Table 4 – Recommended Pavement Structure – Access Lanes and Ramp

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

Table 5 – Recommended Rigid Pavement Structure – Lower Parking Level

Thickness (mm)	Material Description
Specified by Others	**32 MPa Concrete**
300	**BASE** - OPSS Granular A Crushed Stone
SUBGRADE - Either fill, in situ soil, or OPSS Granular B Type I or II material placed over in situ soil.	

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 Type II material.

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

Pavement Structure Drainage

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

Where silty clay is anticipated at subgrade level, consideration should be given to installing subdrains during the pavement construction. The sub-drain inverts should be approximately 300 mm below subgrade level and run longitudinal along the curblines. The subgrade surface should be crowned to promote water flow to the drainage lines.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Foundation Drainage/Flood Proofing

Based on the available information, the lower parking level will be located below the 100-year flood level. To limit long-term groundwater infiltration, it is recommended that a flood proofing system be designed for the proposed building. The system should consist of a water suppression system to lessen the infiltration volumes and manage discharge. Also, a perimeter foundation drainage system will be required as a secondary system to account for any groundwater which breaches the primary groundwater infiltration control system.

The groundwater infiltration control system should extend above the 100-year flood level and the following is suggested for preliminary design purposes:

- Pour a concrete mud slab at the base of the excavation to create a horizontal hydraulic barrier. Typically, the minimum thickness of the concrete mud slab is 150 mm.
- Place a composite drainage layer, such as Delta Drain 6000 or equivalent, over the foundation wall (as a secondary system). The composite drainage layer should extend from finished grade to underside of footing level.
- Place a suitable waterproofing membrane on the drainage layer, such as a bentomat liner system or equivalent. The membrane liner should extend down to footing level. The membrane liner should tie into the concrete mud slab.
- Pour foundation wall against the composite drainage system.

It is recommended that the composite drainage system (such as Delta Drain 6000 or equivalent) extend down to the footing level. It is recommended that 150 mm diameter sleeves at 3-6 m centres be cast in the footing or at the foundation wall/footing interface to allow the infiltration of water to flow to the interior perimeter drainage pipe. The perimeter drainage pipe and underfloor drainage system should direct water to sump pit(s) within the lower basement area.

It is important to note that the building's sump pit and elevator pit be considered for waterproofing in a similar fashion. A detail can be provided by Paterson once the design drawings are available for the elevator and sump pits.

Underfloor Drainage

It is anticipated that underfloor drainage will be required to control water infiltration. For design purposes, we recommend that 150 mm diameter perforated pipes be placed at 6 m centres. The spacing of the underfloor drainage system should be confirmed at the time of completing the excavation when water infiltration can be better assessed.

Foundation Backfill

Backfill against the exterior sides of the foundation walls should consist of free draining non frost susceptible granular materials compacted in lifts as per Subsection 5.2 for areas where frost susceptible structures, such as the site access lane, are to be located. A frost taper should also be provided at the transition between the building face and the native, silty clay subgrade for the access lane.

The greater part of the site excavated materials will be frost susceptible and, as such, are acceptable for foundation wall backfill within landscaped finished areas only.

6.2 Protection of Footings Against Frost Action

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

A minimum of 2.1 m thick soil cover (or equivalent) should be provided for exterior unheated footings, not thermally connected to a heated space, such as exterior columns and/or wing walls.

The parking garage may require protection against frost action depending on the founding depth. Unheated structures, such as the access ramp wall footings, may be required to be insulated against the deleterious effect of frost action. A minimum of 2.1 m of soil cover alone, or a minimum of 0.6 m of soil cover, in conjunction with foundation insulation, should be provided.

It has been our experience that insufficient soil cover is typically provided to footings located in areas where minimal soil cover is available, such as entrance ramps to underground parking garages. Paterson requests permission to review design drawings prior to construction to ensure proper frost protection is provided.

6.3 Retaining Wall Design

It is expected that retaining walls will be required to grade the property. Retaining walls higher than 1.0 m should be designed by a professional engineer. The bearing resistance values provided in Section 5.3 are applicable to the proposed retaining walls.

The soil parameters presented in Tables 6 should be used for the design of the retaining walls. The design should also include a global stability analysis of the system.

Global stability analysis should include static and seismic analysis of the system and present the minimum factor of safety. The system should be design for a factor of safety of 1.5 under static conditions and 1.1 for seismic conditions.

Backfill Material

The retaining wall should be backfilled with free-draining granular backfill materials and incorporate longitudinal drains and weep holes to provide positive drainage of the backfill. For the purpose of this report, it is recommended that the wall be backfilled with either OPSS Granular B Type II or Granular A materials. The backfill should be placed within a wedge-shaped zone defined by a line drawn up and back from the back edge of the base block of the wall at an inclination of 1H:1V or a minimum of 1 m behind the back of the blocks. All material should be compacted to a minimum of 98% of the material’s SPMDD.

Based on the proposed preliminary landscaping plans provided, the proposed grades within multiple areas adjacent to the retaining walls exceed our permissible grade raise recommendations. Where significant grade raise exceedances have occurred, lightweight fill (LWF), such as expanded polystyrene (EPS) geofoam blocks, is recommended for specific areas adjacent to the proposed retaining walls. The designer is to consider the maximum grade raise and provide equivalent LWF backfill to mitigate possible differential settlement.

Lateral Earth Pressures

It is recommended that a minimum of 1 m of the backfill material to consist of clean imported engineered crushed stone such as OPSS Granular A or Granular B Type II. The soil parameters presented in Table 6 should be used for the design of the retaining wall.

Table 6 – Geotechnical Parameters for Backfill and Bedding Materials

Material Description	Unit Weight (kN/m^3)		Friction Angle () φ'	Friction Factor, tan δ	Earth Pressure Coefficients		
	Drained γ_{dr}	Effective γ'			Active K_a	At-Rest K_o	Passive K_P
OPSS Granular A (Crushed Stone)	22	13.7	36	0.6	0.26	0.41	3.85
OPSS Granular B Type II (Crushed Stone)	22	13.7	36	0.6	0.26	0.41	3.85
OPSS Granular B Type I (Sand-Gravel)	21	13	32	0.52	0.31	0.47	3.25

Notes:
1. Properties for fill materials are for condition of 98% of standard Proctor maximum dry density.
2. The earth pressure coefficients provided are for horizontal backfill profile.
3. For soil above the groundwater level the “drained” unit weight should be used and below groundwater level the “effective” unit weight should be used.

Retaining Wall Types

Where the retaining wall is to be higher than 1 m and or support a roadway or slope consideration can be given to using large precast concrete segmental block retaining wall system, such as Redi-Rock and Stone Strong. Quality precast products are designed to resist large load under gravity and may not require as much excavation or reinforcement. Typical products vary in size from 0.6 to over 2.4 m in depth depending on the total height of the wall. The size of these supporting structures should be considered when drafting site plans and grading plans, especially where they will be located between structures.

6.4 Excavation Side Slopes

The side slopes of excavations in the soil and fill overburden materials for the proposed building should be retained by temporary shoring systems from the start of the excavation until the structure is backfilled. However, some temporary shallow excavations can be expected for the installation of site services. This is further discussed in the following sections. Reference should also be made to File 125-1 - Landslide Hazard Assessment Addendum, dated July 6, 2023, for recommendations on temporary excavation and shoring.

Unsupported Excavations

Temporary shallow open cut excavations where expected should be completed at a maximum of 2H:1V or shallower slope above groundwater level. A shallower slope is required for excavation below groundwater level.

Where temporary shallow open cut slopes are expected along the south side of the underground parking structure, the excavation should be completed in the following sequential order.

- ☐ The initial excavation should be excavated at a 2H:1V or flatter slope.
- ☐ The excavation should then be completed in smaller sections - maximum of 3 to 5 m wide at a time.
- ☐ Each section should be fully supported and braced prior to excavating additional sections and no more than 20% of the section should be unsupported at a time.

This will help maintain the lateral support zone of the open cut excavations and mitigate any minor slope failures, which could potentially progress into a larger failure.

The subsurface soils are considered to be a Type 2 and Type 3 soil according to the Occupational Health and Safety Act and Regulations for Construction Projects.

Excavated soil should not be stockpiled directly at the top of excavations and heavy equipment should be kept away from the excavation sides. A trench box is recommended to protect personnel working in trenches with steep or vertical sides. Services are expected to be installed by “cut and cover” methods and excavations should not remain open for extended periods of time.

Temporary Shoring

Temporary shoring will be required to retain the overburden soil to complete the required excavations for the underground parking structure. The shoring requirements designed by a structural engineer specializing in those works, or Paterson, will depend on the depth of the excavation, the proximity of the adjacent structures and the elevation of the adjacent building foundations and underground services. The design and implementation of these temporary systems will be the responsibility of the excavation contractor and their design team.

Inspections and approval of the temporary system will also be the responsibility of the designer. Geotechnical information provided below is to assist the designer in completing a suitable and safe shoring system. The designer should take into account the impact of a significant precipitation event and designate design measures to ensure that a precipitation will not negatively impact the shoring system, or soils supported by the system.

The temporary shoring system should also be periodically monitored with inclinometers during the construction phase of the project, to confirm that no movement occurs. The temporary shoring system should be additionally braced is more than 2 to 3 mm of movement is noted at depths below 2 m. The monitoring program should be in place until the foundation walls are poured up to the ground surface.

Any changes to the approved shoring design system should be reported immediately to the owner's structural design prior to implementation. The temporary system could consist of soldier pile and lagging system or interlocking steel sheet piling. Any additional loading due to street traffic, construction equipment, adjacent structures, and facilities, etc., should be included to the earth pressures described below.

These systems could be cantilevered, anchored, or braced. Given the sandy nature of the fill material present within the site, the designer should consider provisions to mitigate the potential for excessive losses of retained soil during the lagging installation process if consideration is given to using a soldier pile and lagging system.

Generally, it is expected that the shoring systems will be provided with tie-back rock anchors to ensure their stability. The shoring system is recommended to be adequately supported to resist toe failure and inspected to ensure that the sheet piles extend well below the excavation base.

It should be noted if consideration is being given to utilizing a raker style support for the shoring system that lateral movements can occur, and the structural engineer should ensure that the design selected minimizes these movements to tolerable levels.

The earth pressures acting on the shoring system may be calculated using the parameters provided in Table 7.

Table 7 - Soil Parameters for Calculating Earth Pressures Acting on Shoring System

Parameter	Value
Active Earth Pressure Coefficient (K_a)	0.33
Passive Earth Pressure Coefficient (K_p)	3
At-Rest Earth Pressure Coefficient (K_o)	0.5
Unit Weight (γ), kN/m^3	20
Submerged Unit Weight (γ'), kN/m^3	13

The active earth pressure should be calculated where wall movements are permissible while the at-rest pressure should be calculated if no movement is permissible. Due to the concern of potential landslide risk, it is understood that the temporary shoring system will be designed to resist at-rest earth pressure, so as to prevent any soil movement. The dry unit weight should be calculated above the groundwater level while the effective unit weight should be calculated below the groundwater level.

The hydrostatic groundwater pressure should be included to the earth pressure distribution wherever the effective unit weight is calculated for earth pressures.
If the groundwater level is lowered, the dry unit weight for the soil should be calculated full weight, with no hydrostatic groundwater pressure component.

For design purposes, the minimum factor of safety of 1.5 should be calculated.

6.5 Pipe Bedding and Backfill

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

At least 150 mm of OPSS Granular A crushed stone should be used for pipe bedding for sewer and water pipes. The bedding should extend to the spring line of the pipe.

Cover material, from the spring line to at least 300 mm above the obvert of the pipe, should consist of OPSS Granular A (concrete or PSM PVC pipes) or sand (concrete pipe). The bedding and cover materials should be placed in maximum 225 mm thick lifts compacted to 95% of the material’s SPMDD.

Where hard surface areas are considered above the trench backfill, the trench backfill material within the frost zone (about 1.8 m below finished grade) should consist of the soils exposed at the trench walls to minimize differential frost heaving. The trench backfill should be placed in maximum 300 mm thick loose lifts and compacted to a minimum of 95% of the SPMDD.

To reduce long-term lowering of the groundwater level at this site, clay seals should be provided in the service trenches where a clay subgrade is encountered. The seals should be at least 1.5 m long and should extend from trench wall to trench wall. Generally, the seals should extend from the frost line and fully penetrate the bedding, sub-bedding and cover material. The barriers should consist of relatively dry and compactable brown silty clay placed in maximum 225 mm thick loose layers and compacted to a minimum of 95% of the material’s SPMDD. The clay seals should be placed at the site boundaries and at strategic locations at no more than 60 m intervals in the service trenches.

6.6 Groundwater Control

It is anticipated that groundwater infiltration into the excavations should be low through the sides of the excavation and controllable using open sumps.

Pumping from open sumps should be sufficient to control the groundwater influx through the sides of shallow excavations. The contractor should be prepared to direct water away from all bearing surfaces and subgrades, regardless of the source, to prevent disturbance to the founding medium.

Groundwater Control for Building Construction

Under the current regulations enacted by the Ministry of Environment, Conservation and Parks (MECP), any dewatering in excess of 50,000 L/day requires a registration on the Environmental Activity and Sector Registry (EASR), so long as that dewatering is related to construction. If the dewatering is not related to construction, a Permit to Take Water obtained from the MECP will be required.

In the event that an EASR is required to facilitate dewatering of the proposed development, a minimum of three to four weeks should be allotted for completion [ML1.1] of the EASR registration and the Water Taking and Discharge Plan, to be prepared by a Qualified Person as stipulated under O.Reg. 63/16. Should a Permit to Take Water be required, a minimum of five to six months should be allotted for completion of the permit, due to the minimum review period imposed by the MECP.

Long-term Groundwater Control

Our recommendations for the proposed building's long-term groundwater control are presented in Subsection 6.1. Any groundwater which encounters the building's perimeter groundwater infiltration control system will be directed to the proposed building's sump pit. It is expected that groundwater flow will be low (i.e. less than 25,000 L/day with peak periods noted after rain events. It is anticipated that the groundwater flow will be controllable using conventional open sumps.

Impacts on Neighboring Structures

Based on observations, the long-term groundwater level is anticipated at depths 4-5 m below the existing ground surface. A local groundwater lowering is anticipated under short-term conditions due to construction of the proposed building. The extent of any significant groundwater lowering should occur within a limited range of the subject site due to the minimal temporary groundwater lowering.

The neighboring structures are expected to be founded within the brown silty clay crust bearing surface. No issues are expected, with respect to groundwater lowering, that would cause long term damage to adjacent structures surrounding the proposed building.

6.7 Winter Construction

The subsurface soil conditions contain 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 base of the excavations should be insulated from sub-zero temperatures immediately upon exposure and until such time as heat is adequately supplied to the building and the footings are protected with sufficient soil cover to prevent freezing at founding level.

The trench excavations should be constructed to avoid the introduction of frozen materials, snow or ice into the trenches. Also, pavement construction is difficult during winter. The subgrade consists of frost susceptible soils which will experience total and differential frost heaving during construction.

Also, the introduction of frost, snow or ice into the pavement materials or fill used to backfill the lower basement level, which is difficult to avoid during winter conditions, will greatly negatively affect the performance of the fill and impact construction schedules.

6.8 Corrosion Potential and Sulphate

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

6.9 Slope Stability Analysis

The analysis of the stability of the slope was carried out using SLIDE, a computer program which permits a two-dimensional slope stability analysis using several methods including the Bishop's method and Morgenstein Price 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 the variability of the subsoil and groundwater conditions, a factor of safety greater than one is usually required to ascertain than the risks of failure are acceptable.

Both slope cross-sections were analysed utilizing the latest topographic mapping prepared by Annis, O'Sullivan, Vollebekk Ltd., and our interpretation of the conceptual plans provided by the client by incorporating three levels of underground parking structure.

The slope stability analysis was completed at each slope cross-section along the watercourse bordering the north boundary of the site under worst-case-scenario by assigning cohesive soils under fully saturated conditions.

The existing drainage feature located within a confined channel located in excess of 50 m from the east property boundary does not pose any concerns from a geotechnical perspective to the proposed development at the aforementioned site.

The effective strength soil parameters used for static analysis were chosen based on the subsoil information recovered during the geotechnical investigation which also happens to reflect the soil parameters that were used during the slope stability assessment completed for the previous slope stability assessments within the immediate area of the subject site. The effective strength soil parameters used for static analysis are presented in Table 8 below.

Table 8 – Soil Parameters Static Analysis

Soil Layer	Unit Weight (kN/m^3)	Friction Angle (degrees)	Cohesion (kPa)
Fill	18	28	2
Brown Silty Clay Crust	16	32	10
Grey Silty Clay	16	28	8
Bedrock	22	Infinite Strength	

The total strength parameters for seismic analysis were chosen based on the in situ, undrained shear strengths recovered within the open boreholes completed at the time of our geotechnical investigation and based on our general knowledge of the area's geology. The strength parameters used for seismic analysis at the slope cross-sections are presented in Table 9 below.

Table 9 – Soil Parameters Static Analysis

Soil Layer	Unit Weight (kN/m^3)	Friction Angle (degrees)	Cohesion (kPa)
Fill	18	28	2
Brown Silty Clay Crust	16	-	150
Grey Silty Clay	16	-	100
Bedrock	22	Infinite Strength	

The location of the three cross-sections analyzed are presented on Drawing PG5336-1 - Test Hole Location Plan enclosed.

Static Loading Analysis

The results of the static analysis for the proposed slope are shown in Figure 1A and 2A attached to the current report. The minimum analysed slope stability factor of safety under fully saturated conditions (worst-case-scenario) were calculated to be greater than 1.5.

As a result, the three slope cross-sections analyzed were all above the recommended Factor of Safety of 1.5 and are considered stable under static conditions.

Seismic Loading Analysis

An analysis considering seismic loading was also completed as part of our slope stability assessment. A horizontal seismic acceleration, Kh, of 0.201G was considered for the analysed section. A factor of safety of 1.1 is considered to be satisfactory for stability analysis including seismic loading.

The results of the seismic analysis are shown in Figure 1B and 2B attached to the current report. The overall slope stability factor of safety at the three slope cross-sections when considering seismic loading was found to be greater than 1.1 which is considered to be stable under seismic loading.

Limit of Hazard Lands

The limit of hazard lands includes allowances for a geotechnical stable slope, the potential for future toe erosion and access for equipment to remediate a potential slide. This is discussed further in the following sections.

Stable Slope Allowance

The stable slope limit is usually defined by the extent of the lowest slip circle (failure slip plains) analyzed behind the top of slope where the minimum factor of safety calculated is less than 1.5.

The minimum factor of safety was calculated for all the three slope sections analysed to be above the recommended 1.5 under static conditions and above the recommended factor of safety of 1.1 under seismic loading and therefore defined as stable and no stable slope allowance is required from a geotechnical perspective.

Toe Erosion Allowance

The toe erosion allowance for the valley corridor wall slope are based on the cohesive nature of the top layers of the subsoils, the observed current erosional activities and the width and location of the current watercourse.

Since the existing watercourse (Ottawa River) is located greater than 20 m from the toe of the slope and no evidence of erosional activities were observed along the toe of the slope during our site visit. As per “River and Stream System: Erosion Hazard Limit prepared by Ontario Ministry of Natural Resources”*,* confined systems where the toe of the slope located more than 15 m from the watercourse do not require setback for toe erosion allowance.

Based on the measured distance between the toe of the slope and the watercourse, slope geometry and slope stability analysis results, in our opinion, no toe erosion allowance is required for the subject section of the site.

Erosion Access Allowance

Based on the City of Ottawa guidelines for slope stability, as a general rule, where the development precludes an access for construction equipment such as a parking lot, access lanes, rear yards, etc, a 6 m erosion access allowance must be provided. However, due to the overall stability of the slope in conjunction with the proximity of the watercourse to the toe of the slope, it’s considered acceptable to omit the requirement for the 6 m erosion access allowance for the subject section of slope. However, as a conservative approach, a **6 m** erosion access allowance was provided from the top of slope identified during our site visit on June 2, 2020, which is presented as the Limit of Hazard Lands setback identified on drawing PG5336-1- Test Hole Location Plan attached to the current report.

Conclusion

The use of piles / micropiles founded on bedrock will not impose any additional lateral loading on the deep clay deposit that might significantly affect the stability of the slope. The depth of the structure extends below any deep failure plains and helps to stabilise the slope. Therefore, the proposed development is considered acceptable, from a slope stability perspective.

The recommendations provided in this letter report are in accordance with Paterson’s present understanding of the project. Should any conditions at the site be encountered which differ from the site observations, Paterson requests immediate notification to permit reassessment for the recommendations.

6.10 Landslide Hazard Assessment

The following summarizes the recommendations provided on the report – File 125 - 1 - Landslide Hazard Assessment Addendum prepared by McQuarrie Geotechnical Consultants Limited dated July 6, 2023.

The report recommends the use of a temporary shoring system to facilitate the deep excavation for the underground parking structure, along with recommendations on completing open-cut excavations that could help mitigate the loss of lateral support of the excavations, which will reduce the risk of localized failures that could progress into larger failures. In addition, the report included recommendations for temporary slope monitoring during construction using inclinometers, as well as considerations for long-term monitoring through LiDAR surveys and change-detection analysis. The report recommends that the temporary shoring systems and the foundation walls should be designed to resist as-rest earth pressure to mitigate even minor movements that could progress into larger failures. Based on our review of the report, Paterson is satisfied with the findings and concur with the conclusions presented in the report.

7.0 Recommendations

It is recommended that the following be carried out by Paterson once preliminary and future details of the proposed development have been prepared:

- Review preliminary and detailed grading, servicing and structural plan(s) from a geotechnical perspective.
- Review of the geotechnical aspects of the excavation contractor's shoring design, prior to construction, if applicable.
- Review of architectural plans pertaining to foundation and underfloor drainage systems and waterproofing details for elevator shafts and pools.
- Complete detailed retaining wall structural and geotechnical design.

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

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

☐ Inspection of all foundation drainage and groundwater infiltration control systems.

☐ Sampling and testing of the concrete and fill materials used.

☐ Observation of the placement of the foundation insulation, if applicable.

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

☐ Observation of all subgrades prior to backfilling.

☐ Field density tests to determine the level of compaction achieved.

☐ Sampling and testing of the bituminous concrete including mix design reviews.

A report confirming the work has been conducted in general accordance with the recommendations could be issued, upon request, following the completion of a satisfactory materials testing and observation program by Paterson.

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

8.0 Statement of Limitations

The recommendations provided are in accordance with the present understanding of the project. Paterson requests permission to review the recommendations when the drawings and specifications are completed.

A soils investigation is a limited sampling of a site. Should any conditions at the site be encountered which differ from those at the test locations, Paterson requests immediate notification to permit reassessment of our recommendations.

The recommendations provided herein should only be used by the design professionals associated with this project. They are not intended for contractors bidding on or undertaking the work. The latter should evaluate the factual information provided in this report and determine the suitability and completeness for their intended construction schedule and methods. Additional testing may be required for their purposes.

The present report applies only to the project described in this document. Use of this report for purposes other than those described herein or by person(s) other than Trim Road 1 Limited Partnership and Vuze Construction, or their agents, is not authorized without review by Paterson for the applicability of our recommendations to the alternative use of the report.

Paterson Group Inc.

LICENSED PROFESSIONAL ENGINEER
March 9, 2026
J. R. VILLENEUVE
100504344
PROVINCE OF ONTARIO

Pratheep Thirumoolan, M.Eng., P.Eng.

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

Report Distribution:

☐ Vuze Construction
☐ Trim Road 1 Limited Partnership (1 email copy)
☐ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TESTING RESULTS

SOIL PROFILE BY OTHERS

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Building Complex - 1009 Trim Road
Ottawa, Ontario

DATUM Geodetic

FILE NO. PG5336

REMARKS

HOLE NO. BH 1-20

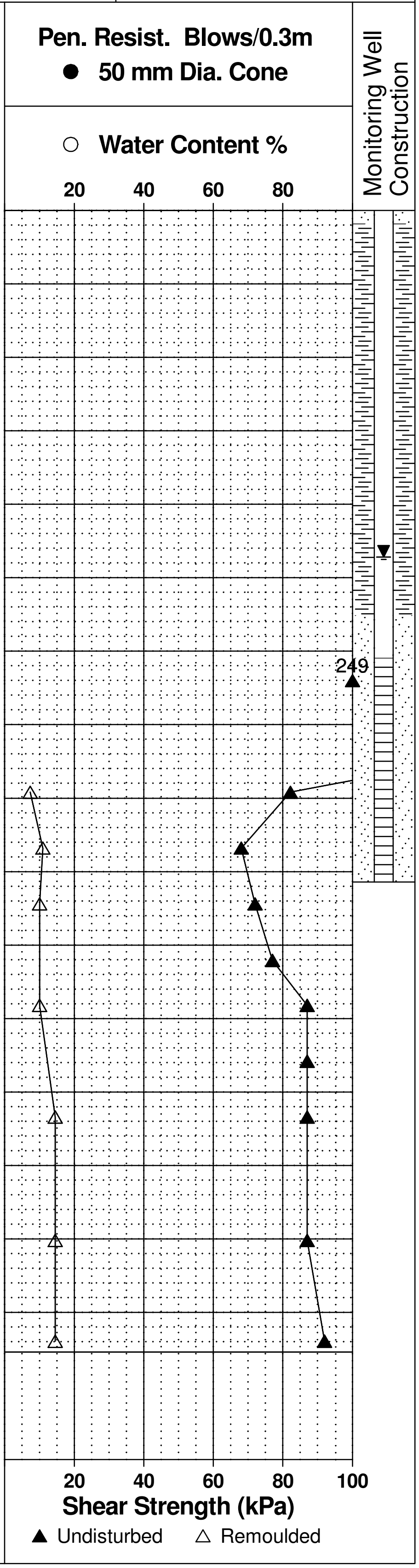
BORINGS BY Track-Mount Power Auger

DATE June 29, 2020

SOIL DESCRIPTION	Depth to base (m)
GROUND SURFACE	
FILL: Brown silty sand with gravel, crushed stone, trace cobbles, boulders and blast rock	1.12
FILL: Brown/black silty clay with gravel, trace sand, brick, occasional cobbles and boulders	2.13
FILL: Brown/black sand with crushed stone, shale fragments, some clay, occasional cobbles and boulders	3.05
FILL: Grey-brown silty clay, some sand, gravel, trace cobbles, plastic, organics	6.10
Hard to very stiff, brown **SILTY CLAY** - stiff and grey by 8.3m depth	15.54
End of Borehole (GWL @ 4.72m - July 17, 2020)	

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	57	50+
SS	3	58	11
SS	4	50	38
SS	5	79	14
SS	6	75	17
SS	7	50	P
SS	8	54	P
SS	9	88	
SS	10	92	
SS	11	83	
SS	12	100	
SS	13	100	
SS	14	50	P

DEPTH (m)	ELEV. (m)
0	46.87
1	45.87
2	44.87
3	43.87
4	42.87
5	41.87
6	40.87
7	39.87
8	38.87
9	37.87
10	36.87
11	35.87
12	34.87
13	33.87
14	32.87
15	31.87



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Building Complex - 1009 Trim Road
Ottawa, Ontario

DATUM Geodetic
REMARKS
BORINGS BY Track-Mount Power Auger
DATE June 30, 2020
FILE NO. PG5336
HOLE NO. BH 2-20

SOIL DESCRIPTION

GROUND SURFACE

FILL: Brown silty sand with crushed stone, trace organics, occasional boulders and blast rock — 1.37

FILL: Brown to grey silty clay, some sand and crushed stone, occasional boulders

- some wood from 3.8 to 4.1m depth

— 6.10

Hard to very stiff, brown **SILTY CLAY**

- stiff and grey by 8.2m depth

— 15.54

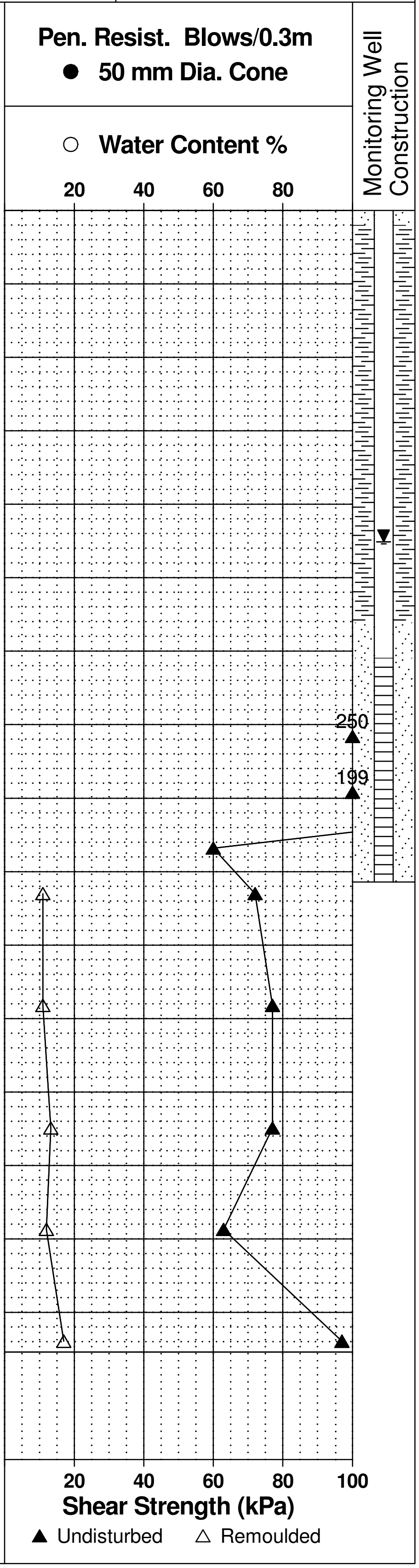
End of Borehole

(GWL @ 4.51m - July 17, 2020)

SAMPLE

TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	10	13
SS	3	42	18
SS	4	50	7
SS	5	50	43
SS	6	17	18
SS	7	33	3
SS	8	17	3
SS	9	100	9
SS	10	75	P
SS	11	100	P
SS	12	100	P
SS	13	100	P

DEPTH (m)	ELEV. (m)
0	47.73
1	46.73
2	45.73
3	44.73
4	43.73
5	42.73
6	41.73
7	40.73
8	39.73
9	38.73
10	37.73
11	36.73
12	35.73
13	34.73
14	33.73
15	32.73



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SOIL PROFILE AND TEST DATA

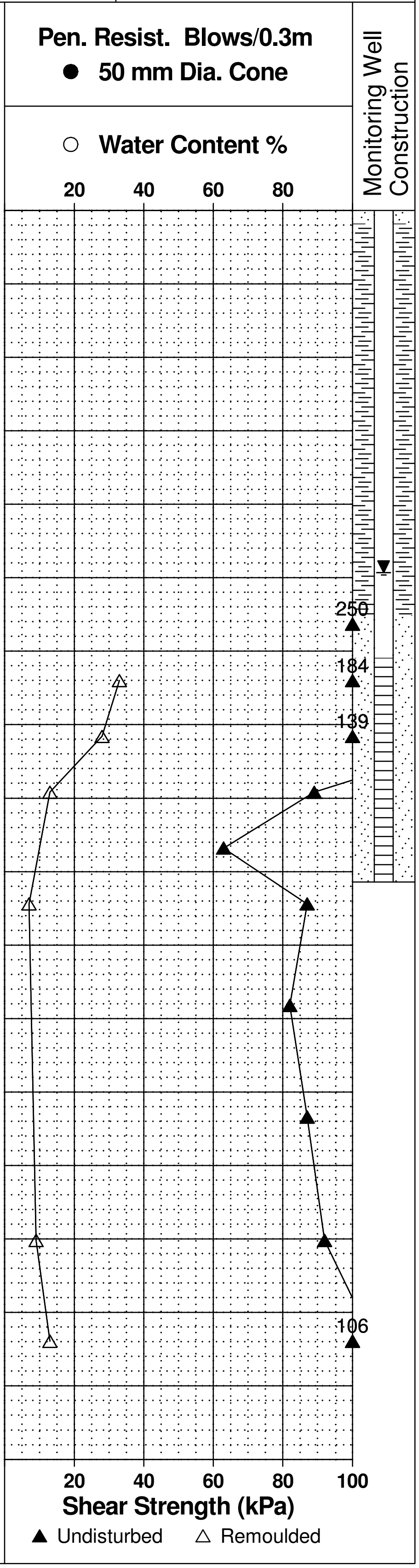
Geotechnical Investigation
Prop. Multi-Storey Building Complex - 1009 Trim Road
Ottawa, Ontario

DATUM Geodetic
REMARKS
BORINGS BY Track-Mount Power Auger
DATE July 2, 2020
FILE NO. PG5336
HOLE NO. BH 3-20

SOIL DESCRIPTION	Depth (m)
GROUND SURFACE	
FILL: Brown silty sand with crushed stone, occasional cobbles, boulders and blast rock	1.52
FILL: Brown sity clay, trace sand, gravel and wood	3.96
Hard to very stiff, brown **SILTY CLAY** - stiff and grey by 8.4m depth	15.54
Dynamic Cone Penetration Test commenced at 15.54m depth. Cone pushed to 31.7m depth.	

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	33	50+
SS	3	0	10
SS	4	58	3
SS	5	83	18
SS	6	75	19
SS	7	92	16
SS	8	92	P
SS	9	92	P
SS	10	96	P
SS	11	92	P
SS	12	96	P
SS	13	100	P
SS	14	96	P

DEPTH (m)	ELEV. (m)
0	49.31
1	48.31
2	47.31
3	46.31
4	45.31
5	44.31
6	43.31
7	42.31
8	41.31
9	40.31
10	39.31
11	38.31
12	37.31
13	36.31
14	35.31
15	34.31
16	33.31
17	32.31



SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Building Complex - 1009 Trim Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

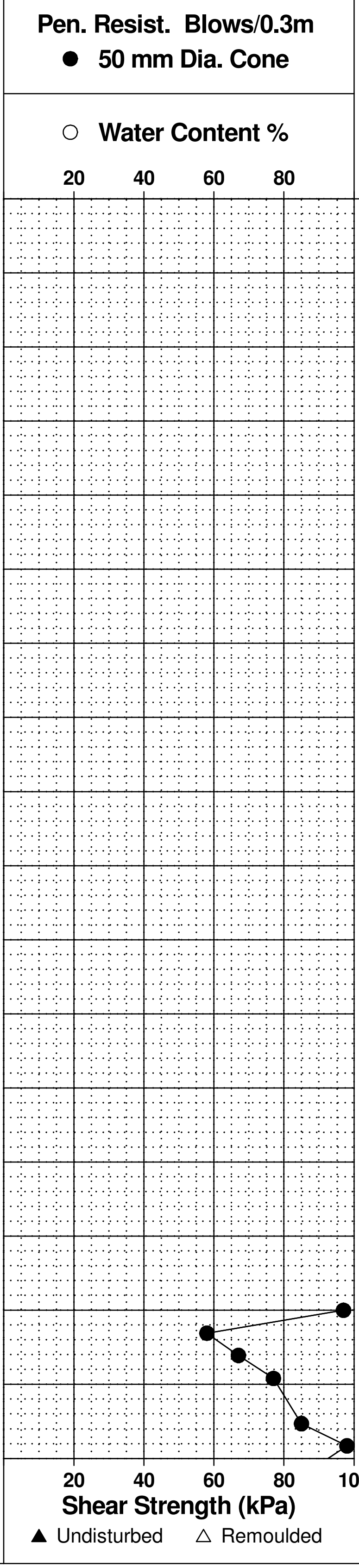
BORINGS BY Track-Mount Power Auger

DATE July 2, 2020

FILE NO. PG5336

HOLE NO. BH 3-20

SOIL DESCRIPTION GROUND SURFACE	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Monitoring Well Construction
						17	32.31	
						18	31.31	
						19	30.31	
						20	29.31	
						21	28.31	
						22	27.31	
						23	26.31	
						24	25.31	
						25	24.31	
						26	23.31	
						27	22.31	
						28	21.31	
						29	20.31	
						30	19.31	
						31	18.31	
						32	17.31	
						33	16.31	
						34	15.31	



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Building Complex - 1009 Trim Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

DATE July 2, 2020

FILE NO. PG5336

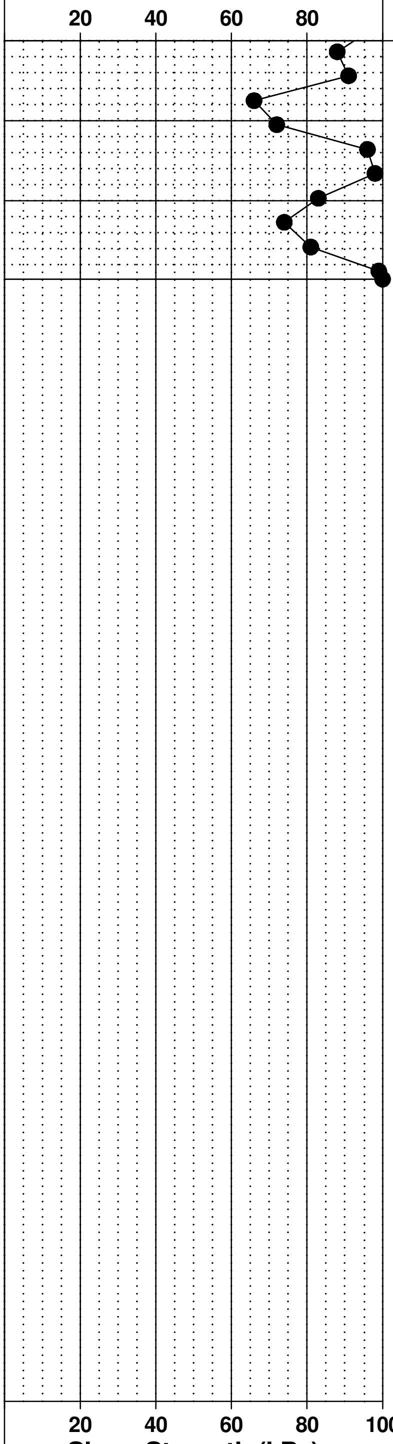
HOLE NO. BH 3-20

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE							
						34	15.31
						35	14.31
						36	13.31
36.98							
End of Borehole Practical DCPT refusal at 36.98m depth. (GWL @ 4.93m - July 17, 2020)							

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

○ Water Content %

Monitoring Well Construction



Shear Strength (kPa)
▲ Undisturbed △ Remoulded

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Building Complex - 1009 Trim Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

DATE June 30, 2020

FILE NO. PG5336

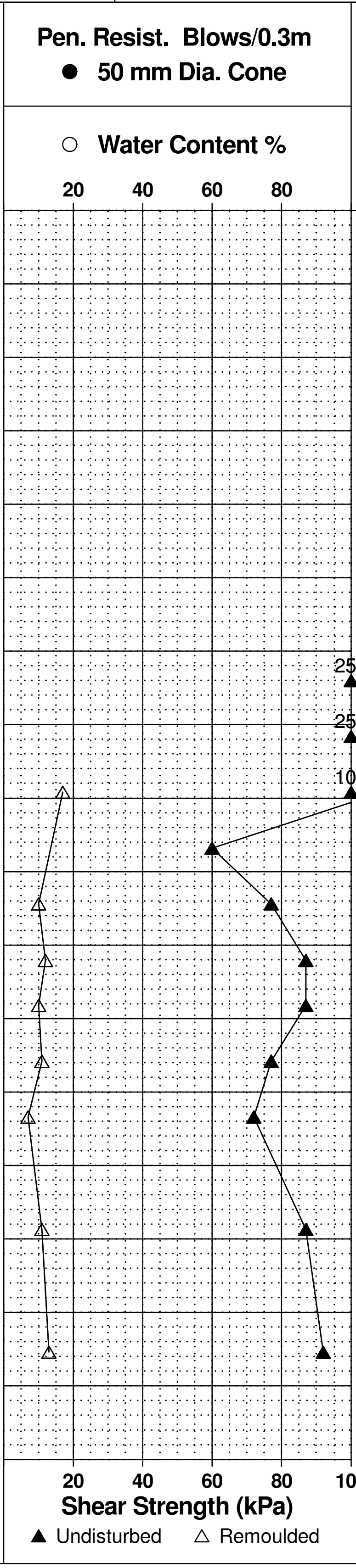
HOLE NO. BH 4-20

GROUND SURFACE

SOIL DESCRIPTION	Depth (m)
FILL: Brown silty sand, some gravel, trace organics, cobbles and boulders	0.60
FILL: Brown sand with crushed stone	0.76
FILL: Grey-brown silty clay, trace sand, gravel and topsoil - trace wood by 5.2m depth	5.69
Hard to very stiff, brown **SILTY CLAY** - stiff and grey by 8.7m depth	15.85
Dynamic Cone Penetration Test commenced at 15.85m depth. Cone pushed to 41.1m depth.	

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	33	14
SS	3	4	9
SS	4	75	5
SS	5	29	P
SS	6	92	7
SS	7	62	4
SS	8	88	6
SS	9	100	P
SS	10	100	P
SS	11	100	P
SS	12	100	P
SS	13	83	P
SS	14	100	P
SS	15	100	P

DEPTH (m)	ELEV. (m)
0	48.90
1	47.90
2	46.90
3	45.90
4	44.90
5	43.90
6	42.90
7	41.90
8	40.90
9	39.90
10	38.90
11	37.90
12	36.90
13	35.90
14	34.90
15	33.90
16	32.90
17	31.90



Monitoring Well Construction

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Building Complex - 1009 Trim Road
Ottawa, Ontario

DATUM Geodetic

FILE NO. PG5336

REMARKS

HOLE NO. BH 4-20

BORINGS BY Track-Mount Power Auger

DATE June 30, 2020

SOIL DESCRIPTION GROUND SURFACE	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content % 20 40 60 80	Monitoring Well Construction
						17	31.90		
						18	30.90		
						19	29.90		
						20	28.90		
						21	27.90		
						22	26.90		
						23	25.90		
						24	24.90		
						25	23.90		
						26	22.90		
						27	21.90		
						28	20.90		
						29	19.90		
						30	18.90		
						31	17.90		
						32	16.90		
						33	15.90		
						34	14.90		

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

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Building Complex - 1009 Trim Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

DATE June 30, 2020

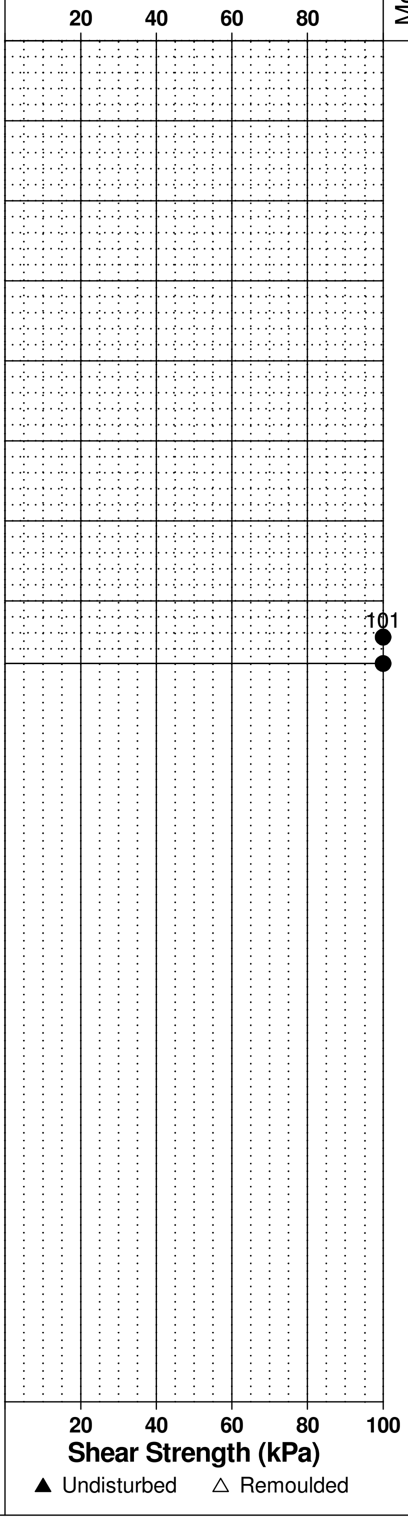
FILE NO. PG5336

HOLE NO. BH 4-20

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						34	14.90
						35	13.90
						36	12.90
						37	11.90
						38	10.90
						39	9.90
						40	8.90
						41	7.90
41.78							
End of Borehole Practical DCPT refusal at 41.78m depth.							

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

○ Water Content %



Shear Strength (kPa)
▲ Undisturbed △ Remoulded

Monitoring Well Construction

SYMBOLS AND TERMS

SOIL DESCRIPTION

Behavioural properties, such as structure and strength, take precedence over particle gradation in describing soils. Terminology describing soil structure are as follows:

Desiccated	-	having visible signs of weathering by oxidation of clay minerals, shrinkage cracks, etc.
Fissured	-	having cracks, and hence a blocky structure.
Varved	-	composed of regular alternating layers of silt and clay.
Stratified	-	composed of alternating layers of different soil types, e.g. silt and sand or silt and clay.
Well-Graded	-	Having wide range in grain sizes and substantial amounts of all intermediate particle sizes (see Grain Size Distribution).
Uniformly-Graded	-	Predominantly of one grain size (see Grain Size Distribution).

The standard terminology to describe the relative strength of cohesionless soils is the compactness condition, usually inferred from the results of the Standard Penetration Test (SPT) ‘N’ value. The SPT N value is the number of blows of a 63.5 kg hammer, falling 760 mm, required to drive a 51 mm O.D. split spoon sampler 300 mm into the soil after an initial penetration of 150 mm. An SPT N value of “P” denotes that the split-spoon sampler was pushed 300 mm into the soil without the use of a falling hammer.

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

The standard terminology to describe the strength of cohesive soils is the consistency, which is based on the undisturbed undrained shear strength as measured by the in situ or laboratory shear vane tests, unconfined compression tests, or occasionally by the Standard Penetration Test (SPT). Note that the typical correlations of undrained shear strength to SPT N value (tabulated below) tend to underestimate the consistency for sensitive silty clays, so Paterson reviews the applicable split spoon samples in the laboratory to provide a more representative consistency value based on tactile examination.

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

SYMBOLS AND TERMS (continued)

SOIL DESCRIPTION (continued)

Cohesive soils can also be classified according to their “sensitivity”. The sensitivity, S_t, is the ratio between the undisturbed undrained shear strength and the remoulded undrained shear strength of the soil. The classes of sensitivity may be defined as follows:

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

ROCK DESCRIPTION

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

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

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

SAMPLE TYPES

SS - Split spoon sample (obtained in conjunction with the performing of the Standard Penetration Test (SPT))

TW - Thin wall tube or Shelby tube, generally recovered using a piston sampler

G - "Grab" sample from test pit or surface materials

AU - Auger sample or bulk sample

WS - Wash sample

RC - Rock core sample (Core bit size BQ, NQ, HQ, etc.). Rock core samples are obtained with the use of standard diamond drilling bits.

SYMBOLS AND TERMS (continued)

PLASTICITY LIMITS AND GRAIN SIZE DISTRIBUTION

WC% - Natural water content or water content of sample, %
LL - Liquid Limit, % (water content above which soil behaves as a liquid)
PL - Plastic Limit, % (water content above which soil behaves plastically)
PI - Plasticity Index, % (difference between LL and PL)

Dxx - Grain size at which xx% of the soil, by weight, is of finer grain sizes
These grain size descriptions are not used below 0.075 mm grain size
D10 - Grain size at which 10% of the soil is finer (effective grain size)
D60 - Grain size at which 60% of the soil is finer

Cc - Concavity coefficient = $(D30)^2$ / (D10 x D60)
Cu - Uniformity coefficient = D60 / D10

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

CONSOLIDATION TEST

p'_o - Present effective overburden pressure at sample depth
p'_c - Preconsolidation pressure of (maximum past pressure on) sample
Ccr - Recompression index (in effect at pressures below p'_c)
Cc - Compression index (in effect at pressures above p'_c)

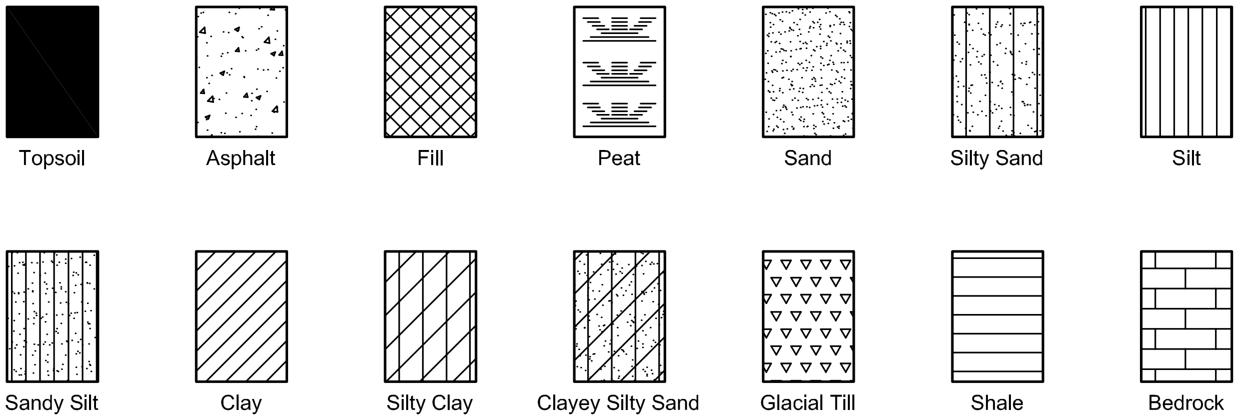
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

PERMEABILITY TEST

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

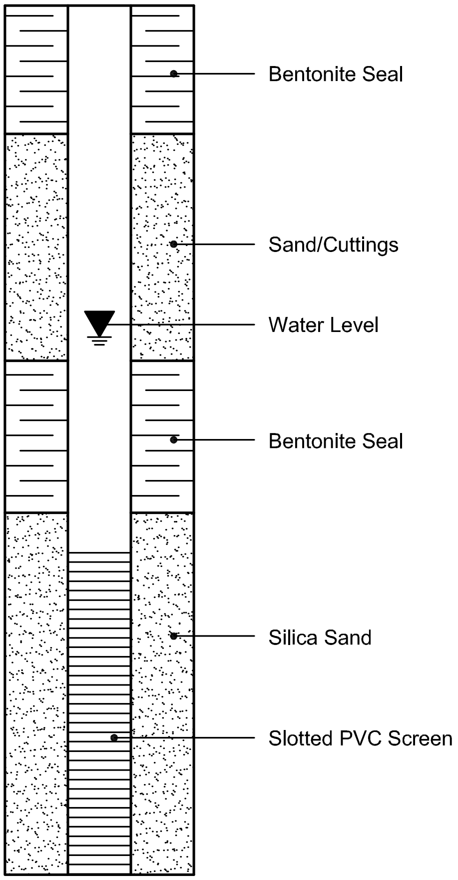
SYMBOLS AND TERMS (continued)

STRATA PLOT

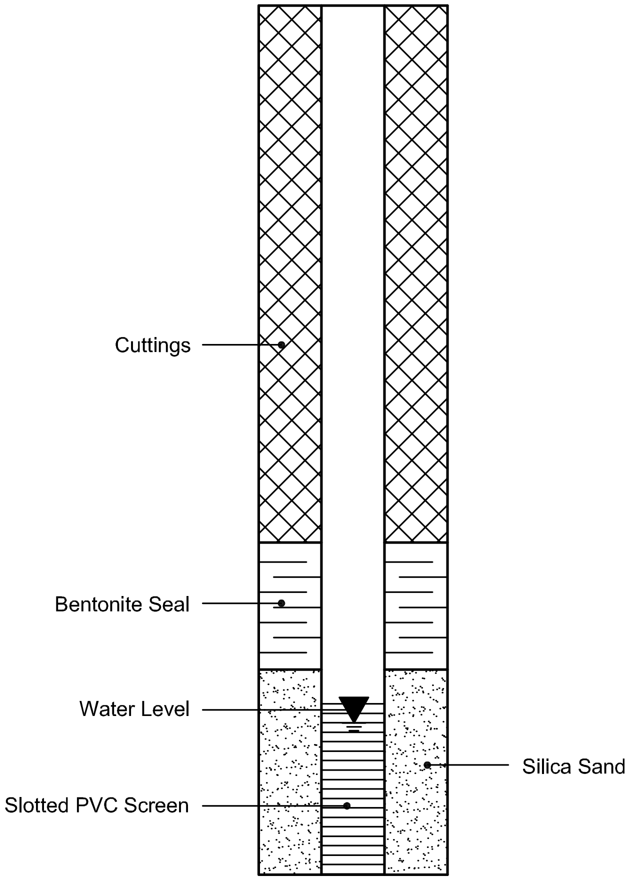


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Certificate of Analysis

Client: Paterson Group Consulting Engineers

Client PO: 29948

Report Date: 13-Jul-2020

Order Date: 7-Jul-2020

Project Description: PG5336

	Client ID:	BH4-SS8B	-	-	-
	Sample Date:	29-Jun-20 12:00	-	-	-
	Sample ID:	2028144-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	73.0	-	-	-
General Inorganics					
pH	0.05 pH Units	7.44	-	-	-
Resistivity	0.10 Ohm.m	45.0	-	-	-
Anions					
Chloride	5 ug/g dry	32	-	-	-
Sulphate	5 ug/g dry	38	-	-	-

LOG OF BOREHOLE MW16-1

Project: Geotechnical Investigation
Client: Grandmaître Family
Project Location: Part Lot 30, Concession 1, Parts 1 & 2, Cumberland, ON
Datum: Approximate
BH Location: See borehole location plan N 5038380 E 462237

DRILLING DATA
Rig Type:
Method: Hollow Stem Auger
Borehole Diameter: 203 mm
Core Diameter: 76 mm

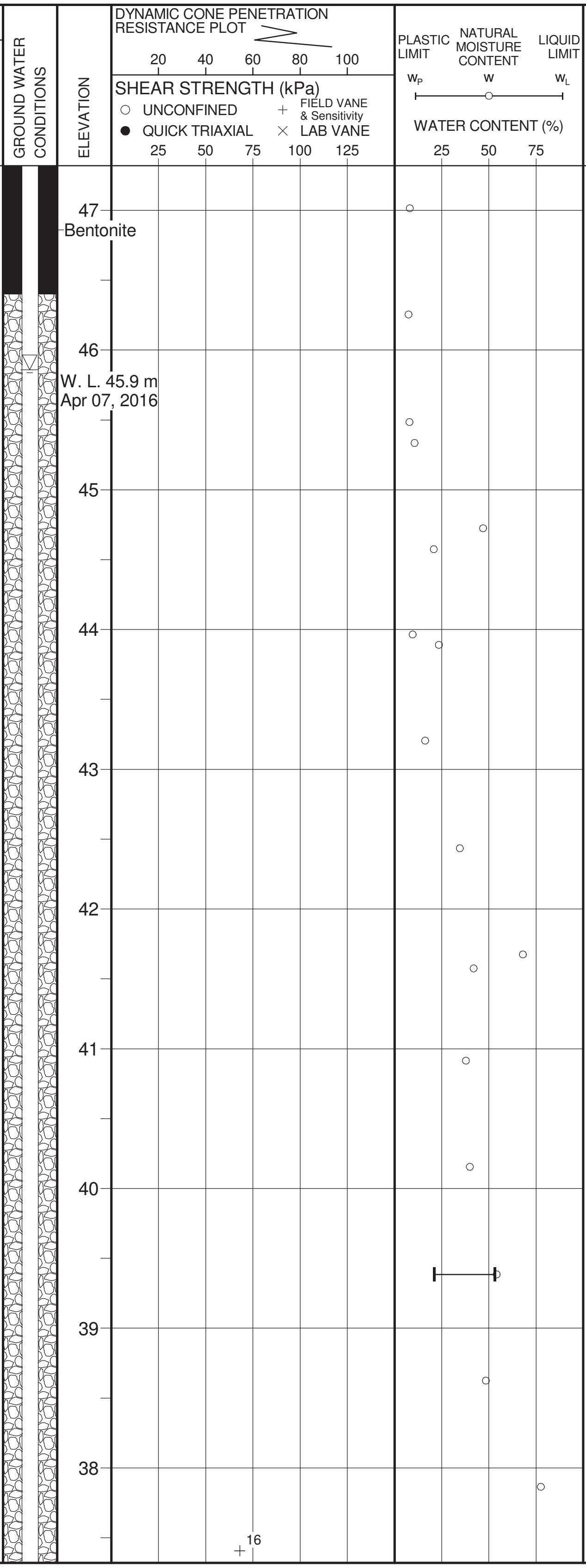
Project No.: 161-03361-00
Date Started: 3/24/2016
Supervisor:
Reviewer:

SOIL PROFILE

(m) ELEV DEPTH	DESCRIPTION
47.3 0.0	**CRUSHED SAND AND GRAVEL** trace to some silt, trace to some clay, grey, wet, compact (FILL) - loose below 0.75 m
45.5 1.8	**CLAYEY SILT** some sand, some gravel, dark brown, moist, loose to compact (FILL)
44.7 2.7	**SILTY SAND** some gravel,brown, moist (FILL)
44.3 43.1	**SAND AND GRAVEL** brown, mosit (FILL)
3.2	**CLAYEY SILT** some sand, trace gravel, grey-brown, wet, loose to compact (FILL) - Shale Fragments below 3.8 m - very loose below 4.8 m in depth
42.0 5.3	**SILTY SAND ORGANIC SOIL** trace wood, light brown, wet
41.8 5.5	**CLAYEY SILT**: trace gravel, grey-brown, wet (FILL)
41.2 6.1	**SILTY CLAY** brown-grey, wet, very stiff (WEATHERED CRUST) - stiff
38.9 8.4	**SILTY CLAY** grey, wet, stiff

SAMPLES

NUMBER	TYPE	"N" BLOWS 0.3 m	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	28	
2	SS	5	
3A 3B	SS	10	
4 4B	SS	8	19 56 (25)
5A 5B	SS	19	
6	SS	8	
7	SS	2	
8A 8B	SS	2	
9	SS	19	
10	SS	20	
11	SS	4	0 0 40 60
12	SS	2	
13	SS	WH	
	VANE		



POCKET PEN. (Cu) (kPa)
NATURAL UNIT WT (KN/m³)

Continued Next Page

GROUNDWATER ELEVATIONS
Shallow/ Single Installation ▽ ▼ Deep/Dual Installation ▽ ▼

GRAPH NOTES $+^3, \times^3$: Numbers refer to Sensitivity ○ ε=3% Strain at Failure

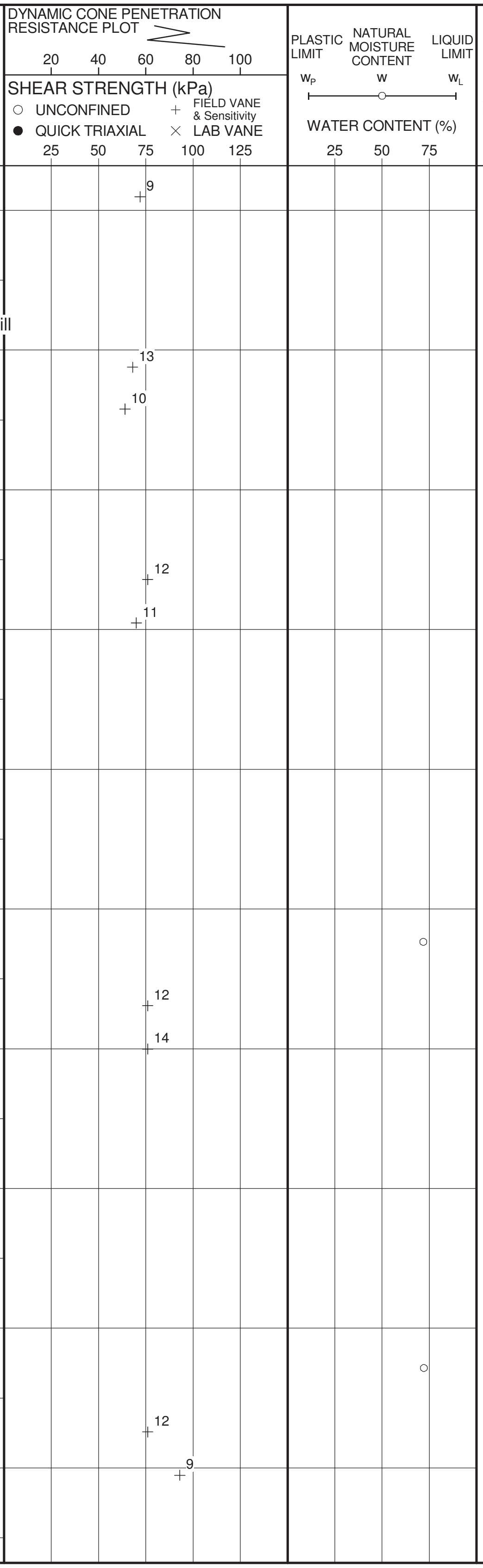
LOG OF BOREHOLE MW16-1

Project: Geotechnical Investigation
Client: Grandmaître Family
Project Location: Part Lot 30, Concession 1, Parts 1 & 2, Cumberland, ON
Datum: Approximate
BH Location: See borehole location plan N 5038380 E 462237

DRILLING DATA
Rig Type:
Method: Hollow Stem Auger
Borehole Diameter: 203 mm
Core Diameter: 76 mm

Project No.: 161-03361-00
Date Started: 3/24/2016
Supervisor:
Reviewer:

SOIL PROFILE: (m) ELEV DEPTH	DESCRIPTION	STRATA PLOT	SAMPLES: NUMBER	TYPE	"N" BLOWS/0.3 m	GROUND WATER CONDITIONS	ELEVATION
	SILTY CLAY grey, wet, stiff(Continued)			VANE			37
			14	TW		Backfill	
				VANE			36
				VANE			
			15	TW			35
				VANE			
				VANE			34
							33
			16	SS	WH		32
				VANE			
				VANE			31
							30
			17	SS	2		29
				VANE			
				VANE			28



POCKET PEN. (Cu) (kPa)

NATURAL UNIT WT (KN/m³)

REMARKS AND GRAIN SIZE DISTRIBUTION (%)
GR SA SI CL

Continued Next Page

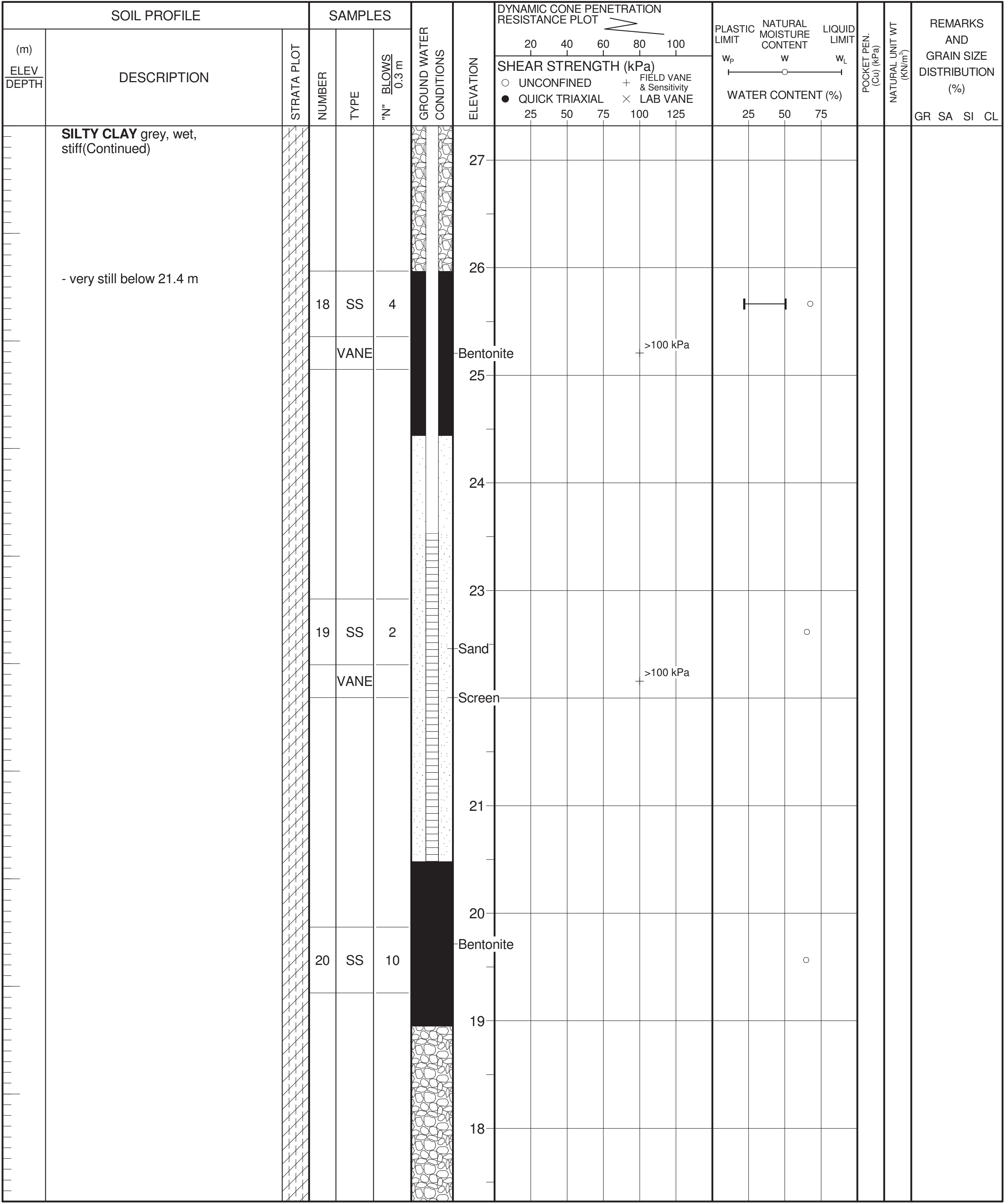
GROUNDWATER ELEVATIONS

Shallow/ Single Installation ▽ ▼ Deep/Dual Installation ▼ ▽

GRAPH NOTES: $+^3, \times^3$: Numbers refer to Sensitivity ○ ε=3% Strain at Failure

LOG OF BOREHOLE MW16-1

Project: Geotechnical Investigation	**DRILLING DATA**	
Client: Grandmaître Family	Rig Type:	Project No.: 161-03361-00
Project Location: Part Lot 30, Concession 1, Parts 1 & 2, Cumberland, ON	Method: Hollow Stem Auger	Date Started: 3/24/2016
Datum: Approximate	Borehole Diameter: 203 mm	Supervisor:
BH Location: See borehole location plan N 5038380 E 462237	Core Diameter: 76 mm	Reviewer:



SILTY CLAY grey, wet, stiff(Continued)

- very still below 21.4 m

NUMBER	TYPE	"N" BLOWS/0.3 m	GROUND WATER CONDITIONS	SHEAR STRENGTH (kPa)
18	SS	4		
	VANE		Bentonite	>100 kPa
19	SS	2	Sand	
	VANE		Screen	>100 kPa
20	SS	10	Bentonite	

Continued Next Page

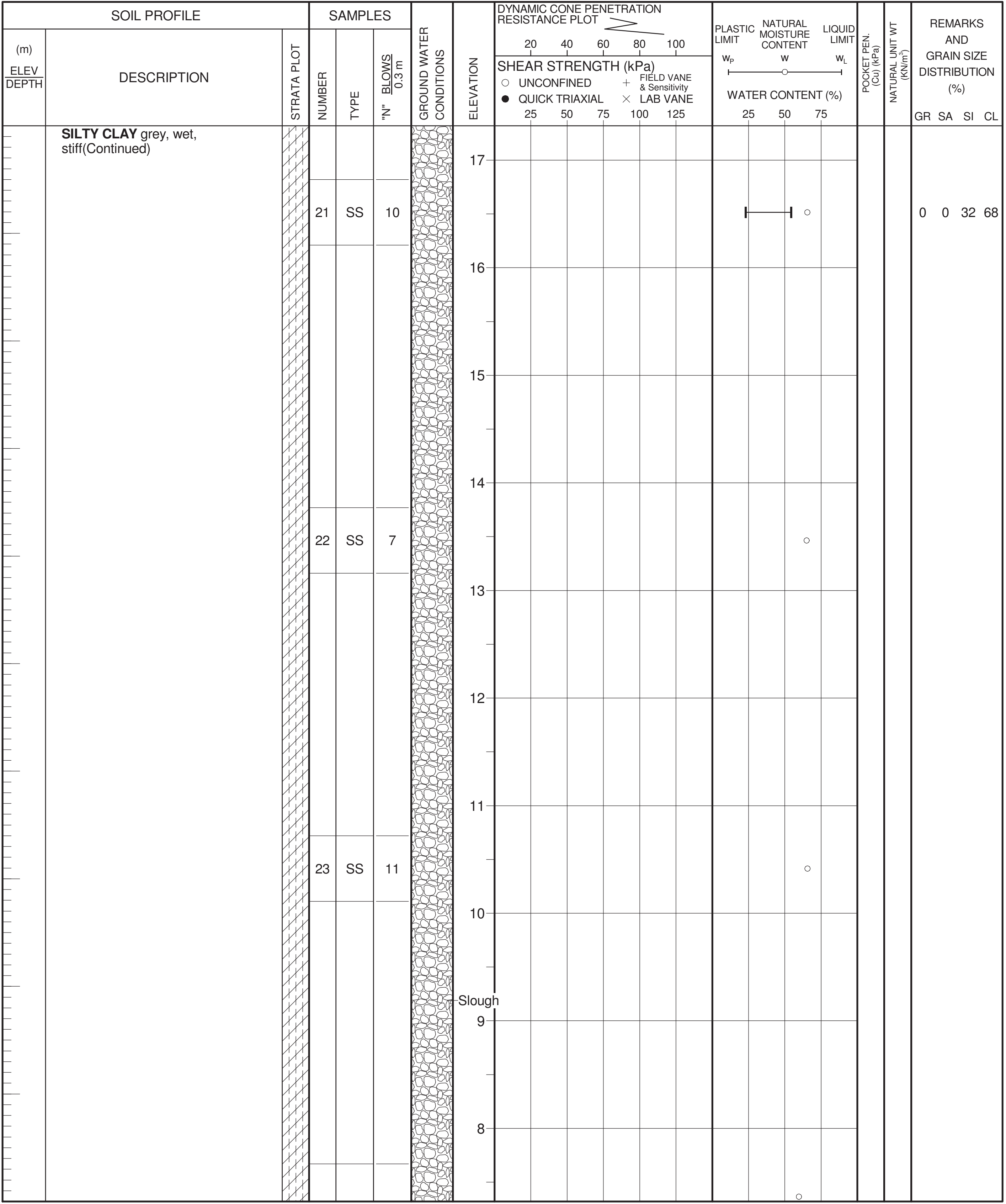
GROUNDWATER ELEVATIONS

Shallow/ Single Installation ▽ ▼ Deep/Dual Installation ▽ ▼

GRAPH NOTES + 3, × 3: Numbers refer to Sensitivity ○ ε=3% Strain at Failure

LOG OF BOREHOLE MW16-1

Project: Geotechnical Investigation	**DRILLING DATA**	
Client: Grandmaître Family	Rig Type:	Project No.: 161-03361-00
Project Location: Part Lot 30, Concession 1, Parts 1 & 2, Cumberland, ON	Method: Hollow Stem Auger	Date Started: 3/24/2016
Datum: Approximate	Borehole Diameter: 203 mm	Supervisor:
BH Location: See borehole location plan N 5038380 E 462237	Core Diameter: 76 mm	Reviewer:



SILTY CLAY grey, wet, stiff(Continued)

NUMBER	TYPE	"N" BLOWS 0.3 m	GR	SA	SI	CL
21	SS	10	0	0	32	68
22	SS	7				
23	SS	11				

Continued Next Page

GROUNDWATER ELEVATIONS

Shallow/ Single Installation ▽ ▼ Deep/Dual Installation ▽ ▼

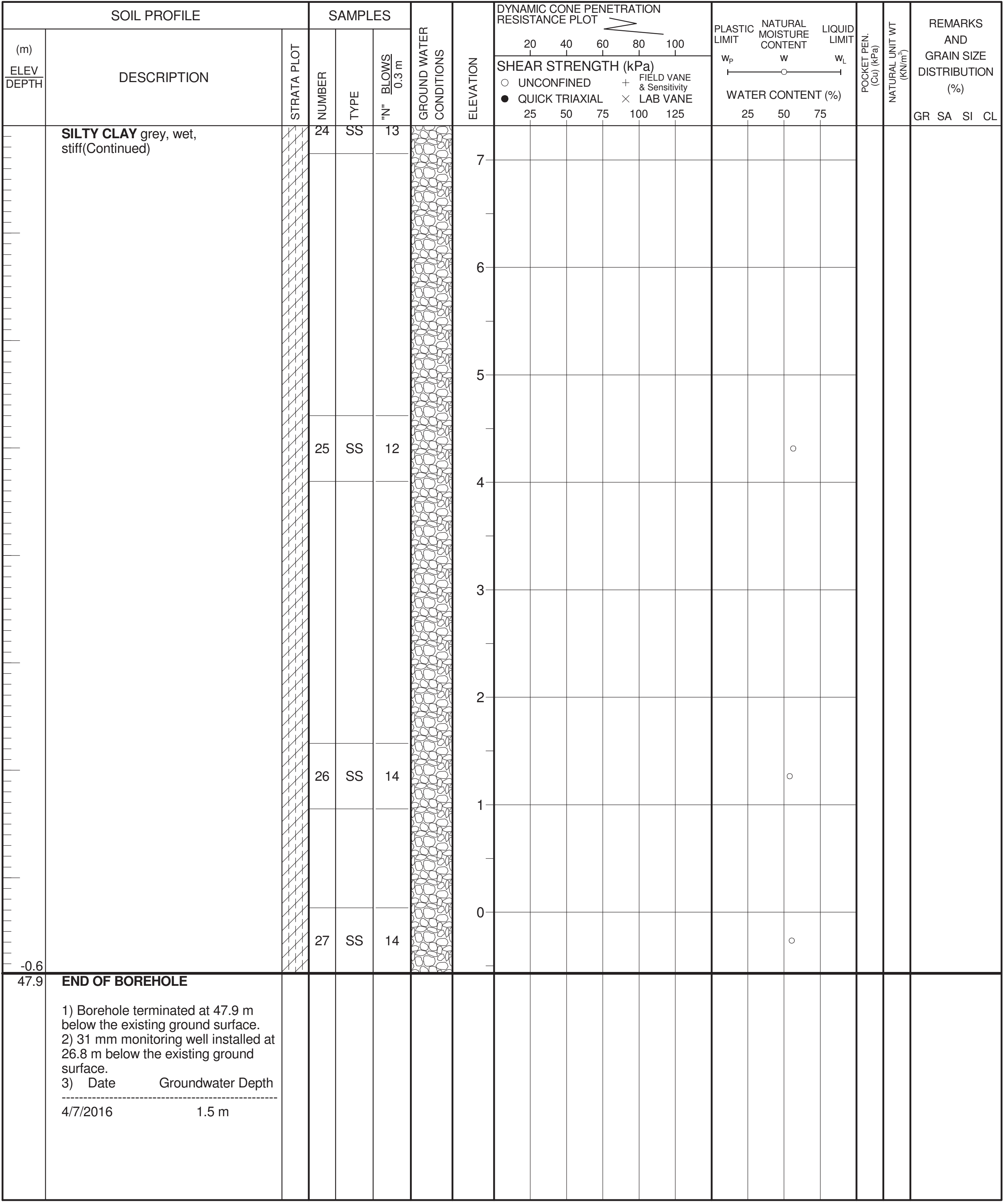
GRAPH NOTES $+^3, \times^3$: Numbers refer to Sensitivity ○ ε=3% Strain at Failure

LOG OF BOREHOLE MW16-1

Project: Geotechnical Investigation
Client: Grandmaître Family
Project Location: Part Lot 30, Concession 1, Parts 1 & 2, Cumberland, ON
Datum: Approximate
BH Location: See borehole location plan N 5038380 E 462237

DRILLING DATA
Rig Type:
Method: Hollow Stem Auger
Borehole Diameter: 203 mm
Core Diameter: 76 mm

Project No.: 161-03361-00
Date Started: 3/24/2016
Supervisor:
Reviewer:



(m) ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" BLOWS / 0.3 m
	SILTY CLAY grey, wet, stiff(Continued)	24	SS	13
		25	SS	12
		26	SS	14
		27	SS	14
-0.6 47.9	**END OF BOREHOLE**			

1) Borehole terminated at 47.9 m below the existing ground surface.
2) 31 mm monitoring well installed at 26.8 m below the existing ground surface.
3) Date Groundwater Depth

Date	Groundwater Depth
4/7/2016	1.5 m

GROUNDWATER ELEVATIONS

Shallow/ Single Installation ▽ ▼ Deep/Dual Installation ▽ ▽

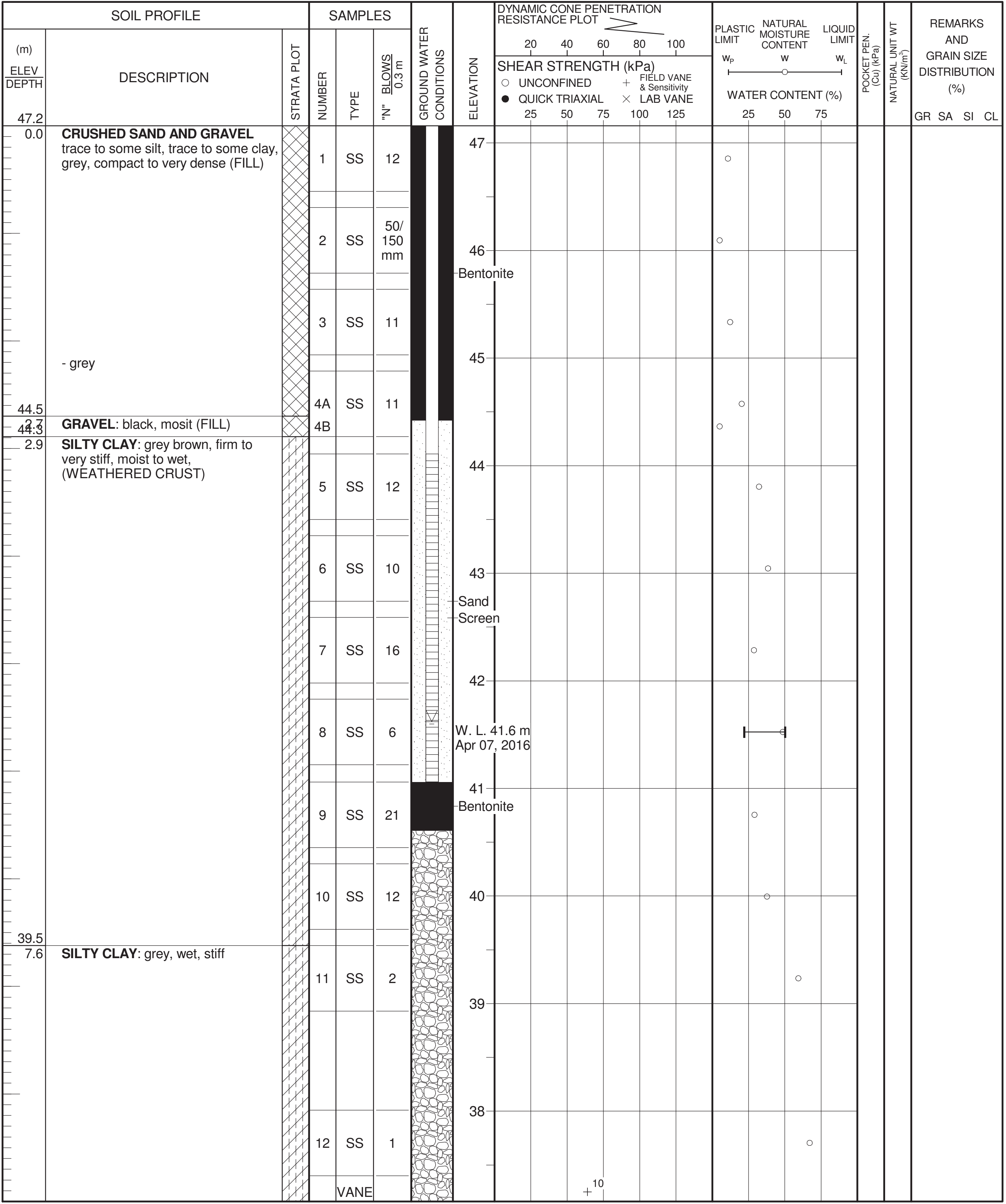
GRAPH NOTES + 3, × 3: Numbers refer to Sensitivity ○ ε=3% Strain at Failure

LOG OF BOREHOLE MW16-2

Project: Geotechnical Investigation
Client: Grandmaître Family
Project Location: Part Lot 30, Concession 1, Parts 1 & 2, Cumberland, ON
Datum: Approximate
BH Location: See borehole location plan N 462330 E 5038430

DRILLING DATA
Rig Type:
Method: Hollow Stem Auger
Borehole Diameter: 203 mm
Core Diameter:

Project No.: 161-03361-00
Date Started: 3/22/2016
Supervisor:
Reviewer:



ELEV (m)	DEPTH (m)	DESCRIPTION	NUMBER	TYPE	"N" BLOWS/0.3 m
47.2	0.0	**CRUSHED SAND AND GRAVEL** trace to some silt, trace to some clay, grey, compact to very dense (FILL)	1	SS	12
			2	SS	50/ 150 mm
			3	SS	11
		- grey	4A	SS	11
44.5	2.7	**GRAVEL**: black, mosit (FILL)	4B		
44.3	2.9	**SILTY CLAY**: grey brown, firm to very stiff, moist to wet, (WEATHERED CRUST)	5	SS	12
			6	SS	10
			7	SS	16
			8	SS	6
			9	SS	21
			10	SS	12
39.5	7.6	**SILTY CLAY**: grey, wet, stiff	11	SS	2
			12	SS	1
				VANE	

Continued Next Page

GROUNDWATER ELEVATIONS

Shallow/ Single Installation ▽ ▼ Deep/Dual Installation ▼ ▽

GRAPH NOTES

$+^3$, $\times^3$: Numbers refer to Sensitivity

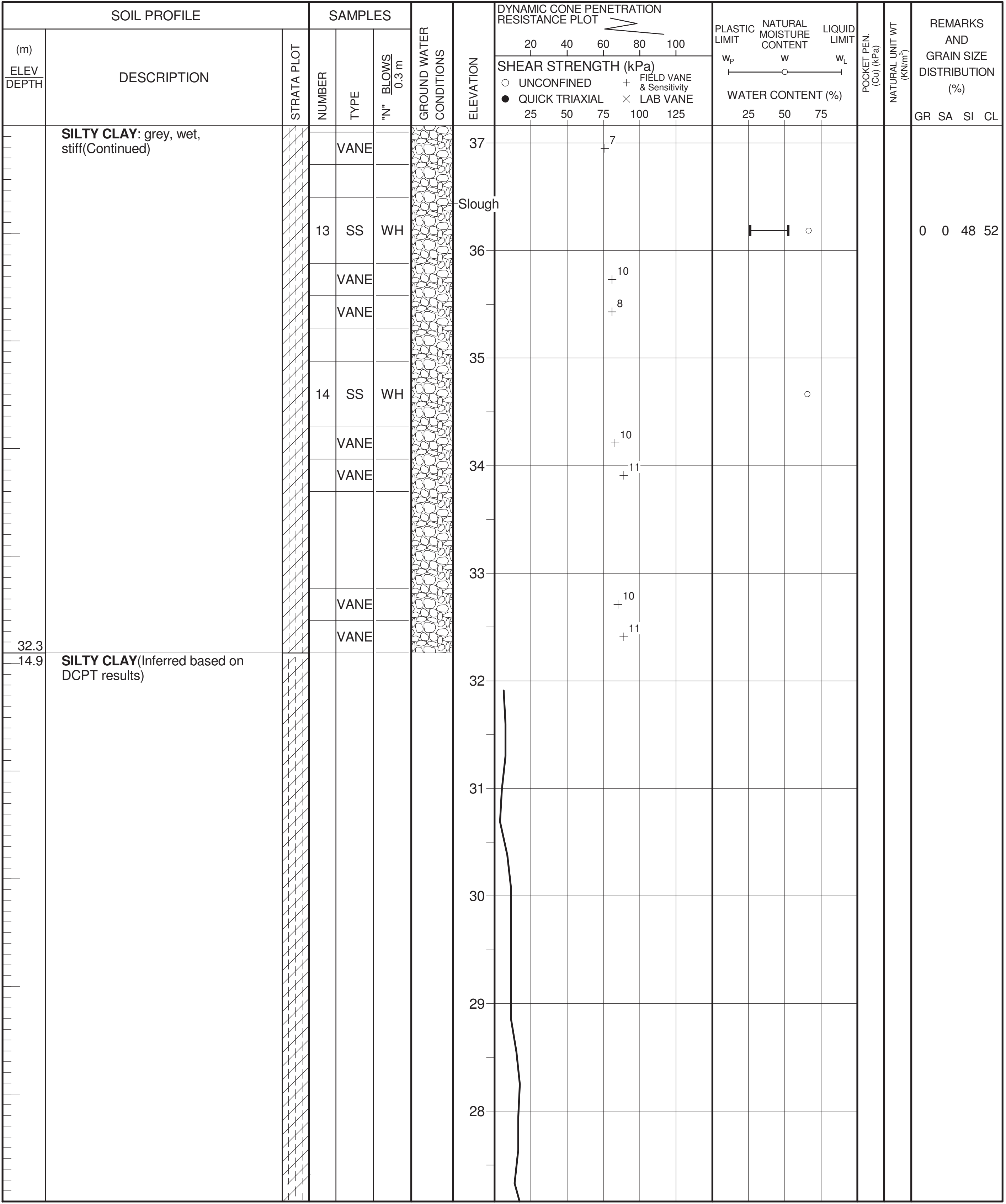
○ ε=3% Strain at Failure

LOG OF BOREHOLE MW16-2

Project: Geotechnical Investigation
Client: Grandmaître Family
Project Location: Part Lot 30, Concession 1, Parts 1 & 2, Cumberland, ON
Datum: Approximate
BH Location: See borehole location plan N 462330 E 5038430

DRILLING DATA
Rig Type:
Method: Hollow Stem Auger
Borehole Diameter: 203 mm
Core Diameter:

Project No.: 161-03361-00
Date Started: 3/22/2016
Supervisor:
Reviewer:



ELEV DEPTH (m)	DESCRIPTION	NUMBER	TYPE	"N"	FIELD VANE & Sensitivity	REMARKS (GR SA SI CL)
	SILTY CLAY: grey, wet, stiff(Continued)		VANE		7	
		13	SS	WH		0 0 48 52
			VANE		10	
			VANE		8	
		14	SS	WH		
			VANE		10	
			VANE		11	
			VANE		10	
			VANE		11	
32.3 14.9	**SILTY CLAY**(Inferred based on DCPT results)					

Continued Next Page

GROUNDWATER ELEVATIONS

Shallow/ Single Installation ▽ ▼ Deep/Dual Installation ▼ ▽

GRAPH NOTES $+^3, \times^3$: Numbers refer to Sensitivity ○ ε=3% Strain at Failure

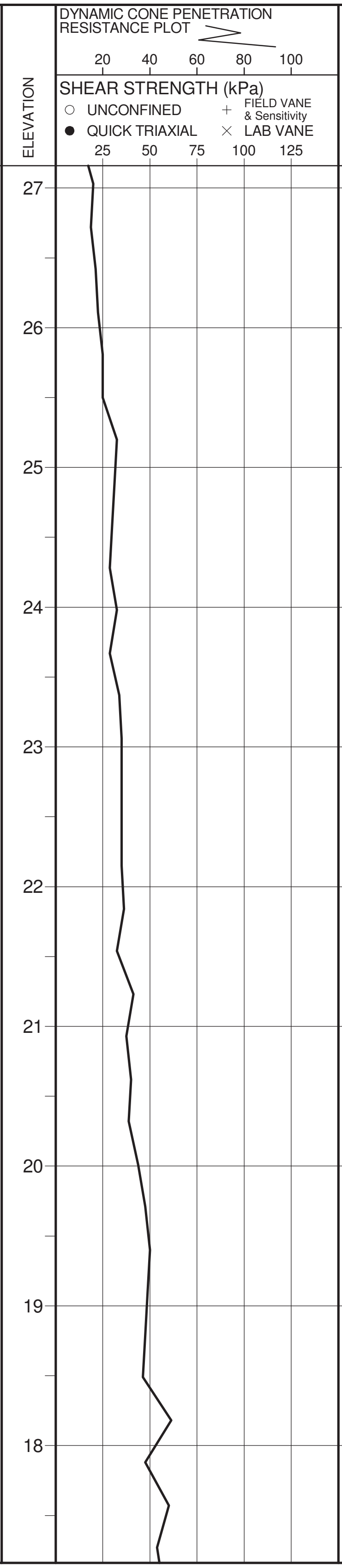
LOG OF BOREHOLE MW16-2

Project: Geotechnical Investigation
Client: Grandmaître Family
Project Location: Part Lot 30, Concession 1, Parts 1 & 2, Cumberland, ON
Datum: Approximate
BH Location: See borehole location plan N 462330 E 5038430

DRILLING DATA
Rig Type:
Method: Hollow Stem Auger
Borehole Diameter: 203 mm
Core Diameter:

Project No.: 161-03361-00
Date Started: 3/22/2016
Supervisor:
Reviewer:

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) 25 50 75	POCKET PEN. (Cu) (kPa)	NATURAL UNIT WT (KN/m³)	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
(m) ELEV DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" BLOWS/0.3 m					
	SILTY CLAY(Inferred based on DCPT results)(Continued)									



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GROUNDWATER ELEVATIONS

Shallow/ Single Installation ▽ ▼ Deep/Dual Installation ▼ ▽

GRAPH NOTES: $+^3$, $\times^3$: Numbers refer to Sensitivity ○ ε=3% Strain at Failure

Sheet No. 3 of 4

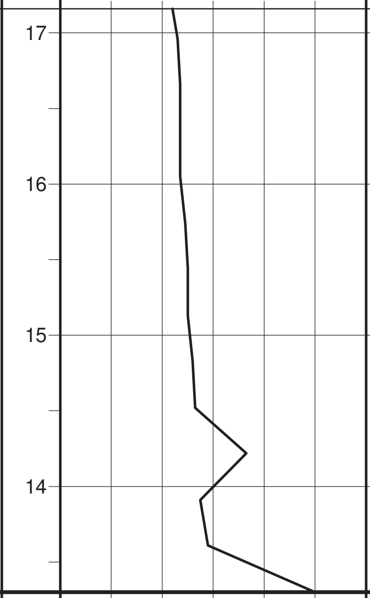
LOG OF BOREHOLE MW16-2

Project: Geotechnical Investigation
Client: Grandmaître Family
Project Location: Part Lot 30, Concession 1, Parts 1 & 2, Cumberland, ON
Datum: Approximate
BH Location: See borehole location plan N 462330 E 5038430

DRILLING DATA
Rig Type:
Method: Hollow Stem Auger
Borehole Diameter: 203 mm
Core Diameter:

Project No.: 161-03361-00
Date Started: 3/22/2016
Supervisor:
Reviewer:

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH (kPa) ○ UNCONFINED + FIELD VANE & Sensitivity ● QUICK TRIAXIAL × LAB VANE 25 50 75 100 125	PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L WATER CONTENT (%) 25 50 75	POCKET PEN. (Cu) (kPa)	NATURAL UNIT WT (KN/m³)	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
(m) ELEV DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" BLOWS/0.3 m							
	SILTY CLAY(Inferred based on DCPT results)(Continued)						17 16 15 14					
13.3 33.9	**END OF BOREHOLE** 1) Augering 14.9 m below the existing ground surface, switch to DCPT. 2) Borehole dry at completion of augering. 3) DCPT refusal at 33.9 m below the existing ground surface. 4) 31 mm monitoring well installed at 6.1 m below the existing ground surface. 5) Date Groundwater Depth 4/7/2016 5.5 m											



GROUNDWATER ELEVATIONS

Shallow/ Single Installation ▽ ▼ Deep/Dual Installation ▼ ▽

GRAPH NOTES $+^3, \times^3$: Numbers refer to Sensitivity ○ ε=3% Strain at Failure

LOG OF BOREHOLE MW16-3

Project: Geotechnical Investigation
Client: Grandmaître Family
Project Location: Part Lot 30, Concession 1, Parts 1 & 2, Cumberland, ON
Datum: Approximate
BH Location: See borehole location plan N 462249 E 5038342

DRILLING DATA
Rig Type:
Method: Hollow Stem Auger
Borehole Diameter: 203 mm
Core Diameter:

Project No.: 161-03361-00
Date Started: 3/22/2016
Supervisor:
Reviewer:

SOIL PROFILE

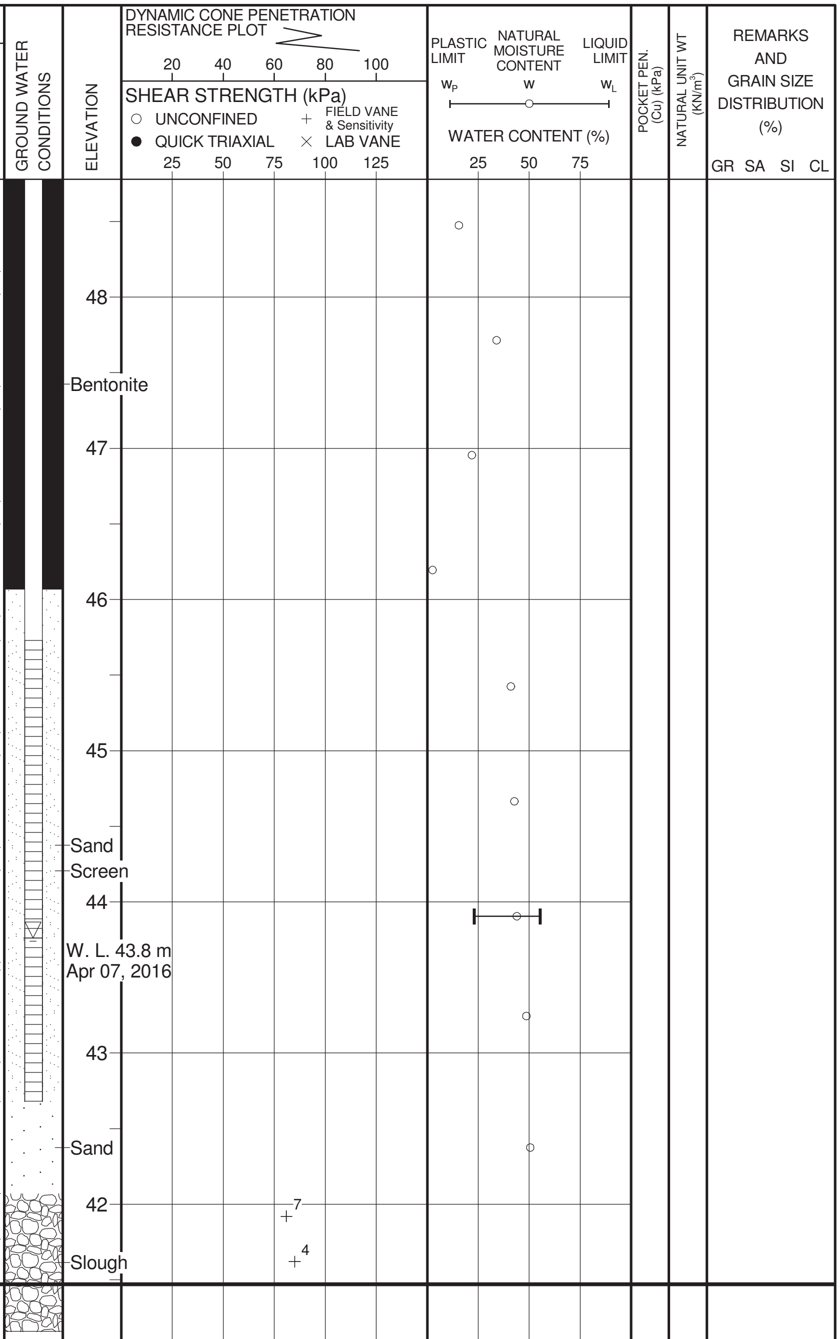
(m) ELEV DEPTH	DESCRIPTION
48.8	
0.0	**CRUSHED SAND AND GRAVEL** trace to some silt, trace to some clay, grey, compact to very dense (FILL)
45.7 3.1	**SILTY CLAY**: grey, moist, firm to stiff
44.8 4.0	**SILTY CLAY**: grey, moist, firm to stiff
41.5 7.3	**END OF BOREHOLE**

1) Borehole terminated at 7.62 m below the existing ground surface.
2) Borehole dry at the completion of augering.
3) 31 mm monitoring well installed at 6.1 m below the existing ground surface.
4) Date Groundwater Depth

Date	Groundwater Depth
4/7/2016	5.02 m

SAMPLES

NUMBER	TYPE	"N" BLOWS 0.3 m
1	SS	14
2	SS	
3	SS	6
4	SS	15
5	SS	9
6	SS	8
7	SS	7
8	SS	4
9	SS	3
	VANE	
	VANE	



GROUNDWATER ELEVATIONS
Shallow/ Single Installation ▽ ▼ Deep/Dual Installation ▽ ▼

GRAPH NOTES + 3, × 3: Numbers refer to Sensitivity ○ ε=3% Strain at Failure

LOG OF BOREHOLE MW16-4

Project: Geotechnical Investigation
Client: Grandmaître Family
Project Location: Part Lot 30, Concession 1, Parts 1 & 2, Cumberland, ON
Datum: Approximate
BH Location: See borehole location plan N 462344 E 5038407

DRILLING DATA
Rig Type:
Method: Hollow Stem Auger
Borehole Diameter: 203 mm
Core Diameter:

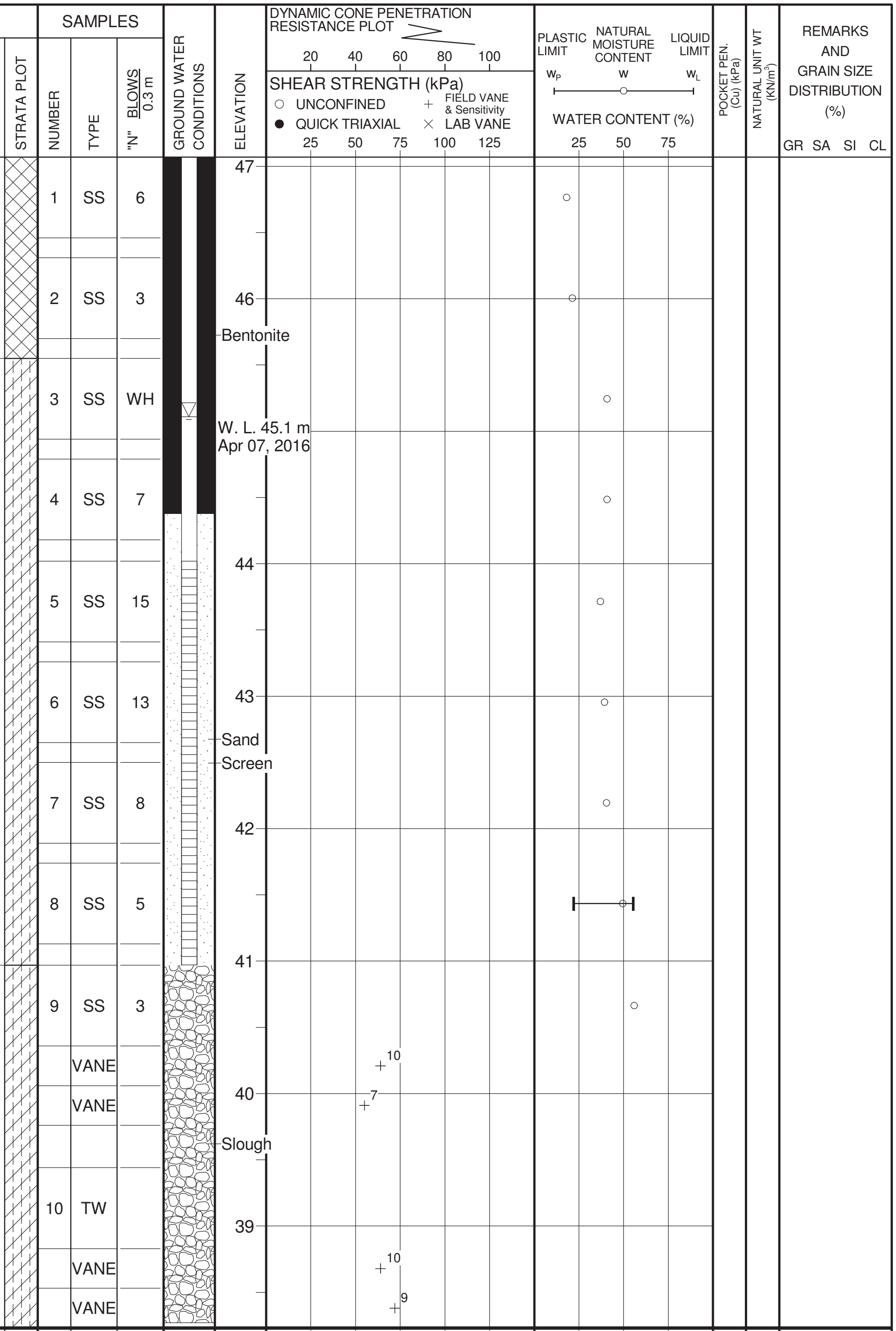
Project No.: 161-03361-00
Date Started: 3/22/2016
Supervisor:
Reviewer:

SOIL PROFILE

(m) ELEV DEPTH	DESCRIPTION
47.1	
0.0	**CRUSHED SAND AND GRAVEL** trace to some silt, trace to some clay, grey, loose to very loose (FILL)
45.6 1.5	**SILTY CLAY**: brown, moist, stiff to stiff very (WEATHERED CRUST) 1.5 m - 2.1 m : trace to some organics - becoming wet below 5.2 m
41.0 6.1	**SILTY CLAY**: grey, moist, stiff to stiff very
38.2 8.8	**END OF BOREHOLE** 1) Borehole terminated at 8.8 m below the existing ground surface. 2) Seepage noted upon completion of borehole at 7.8 m below the existing ground surface. 3) 31 mm monitoring well installed at

SAMPLES

NUMBER	TYPE	"N" BLOWS 0.3 m
1	SS	6
2	SS	3
3	SS	WH
4	SS	7
5	SS	15
6	SS	13
7	SS	8
8	SS	5
9	SS	3
	VANE	
	VANE	
10	TW	
	VANE	
	VANE	



Continued Next Page

GROUNDWATER ELEVATIONS

Shallow/ Single Installation ▽ ▼ Deep/Dual Installation ▽ ▼

GRAPH NOTES + 3, × 3: Numbers refer to Sensitivity ○ ε=3% Strain at Failure

LOG OF BOREHOLE MW16-4

Project: Geotechnical Investigation
Client: Grandmaître Family
Project Location: Part Lot 30, Concession 1, Parts 1 & 2, Cumberland, ON
Datum: Approximate
BH Location: See borehole location plan N 462344 E 5038407

DRILLING DATA
Rig Type:
Method: Hollow Stem Auger
Borehole Diameter: 203 mm
Core Diameter:

Project No.: 161-03361-00
Date Started: 3/22/2016
Supervisor:
Reviewer:

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH (kPa) ○ UNCONFINED + FIELD VANE & Sensitivity ● QUICK TRIAXIAL × LAB VANE 25 50 75 100 125	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 25 50 75	POCKET PEN. (Cu) (kPa)	NATURAL UNIT WT (KN/m^3)	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
(m) ELEV DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" BLOWS/0.3 m							
	6.1 m below the existing ground surface. 4) Date Groundwater Depth --- 4/7/2016 2.0 m											

GROUNDWATER ELEVATIONS

Shallow/ Single Installation ▽ ▼ Deep/Dual Installation ▼ ▽

GRAPH NOTES + ^{3}, × ^{3}: Numbers refer to Sensitivity ○ ε=3% Strain at Failure

LOG OF BOREHOLE MW16-5

Project: Geotechnical Investigation
Client: Grandmaître Family
Project Location: Part Lot 30, Concession 1, Parts 1 & 2, Cumberland, ON
Datum: Approximate
BH Location: See borehole location plan N 462379 E 5038450

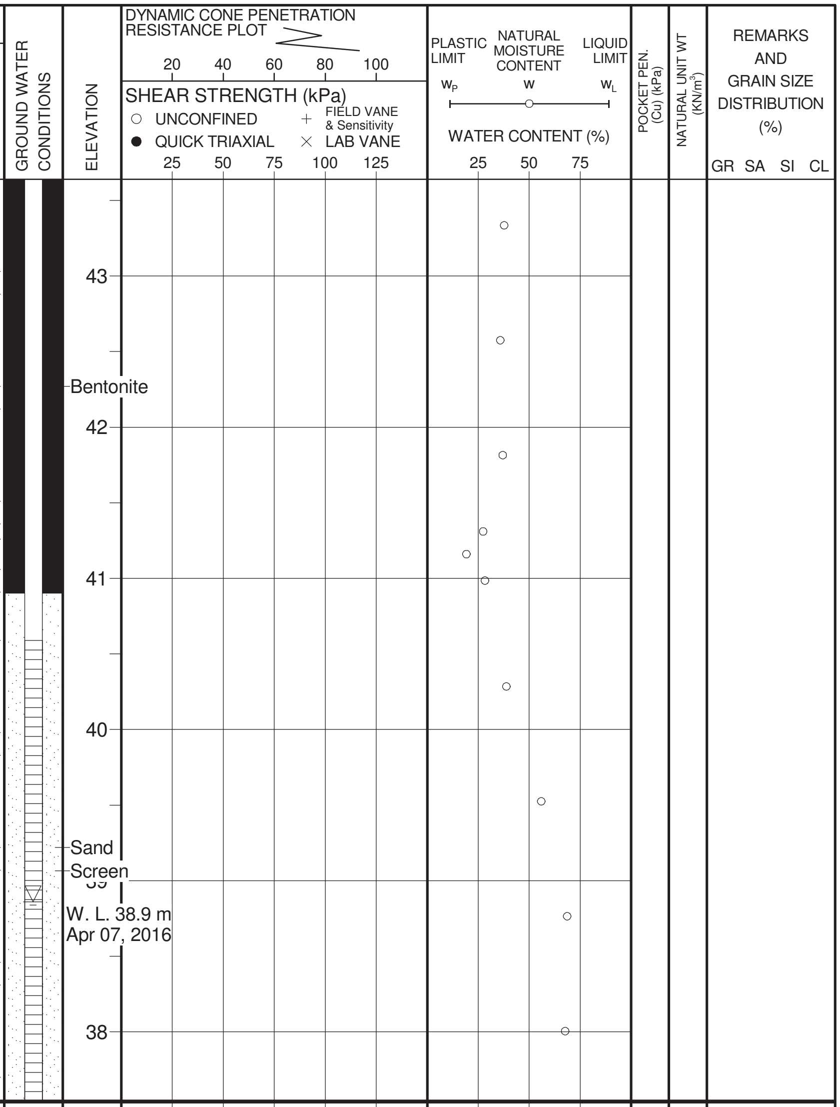
DRILLING DATA
Rig Type:
Method: Hollow Stem Auger
Borehole Diameter: 203 mm
Core Diameter:

Project No.: 161-03361-00
Date Started: 3/22/2016
Supervisor:
Reviewer:

(m) ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	"N" BLOWS/0.3 m
43.6				
0.0	**SILTY CLAY** brown-grey, moist, soft to firm (FILL)	1	SS	2
		2	SS	6
42.1 1.5	**SILTY CLAY** some organic deposits, brown-grey, moist, stiff	3	SS	5
41.3 2.4	**SILTY SAND** grey-brown, moist	4A 4B	SS	21 21
41.0 2.6	**SILTY CLAY**: grey brown, wet, stiff to very stiff (WEATHERED CRUST)	4C		21
		5	SS	15
		6	SS	5
39.1 4.6	**SILTY CLAY**: grey, wet, stiff	7	SS	2
		8	SS	1
37.5 6.1	**END OF BOREHOLE**			

1) Borehole terminated at 6.1 m below the existing ground surface.
2) 31 mm monitoring well installed at 6.1 m below the existing ground surface.
3) Date Groundwater Depth

Date	Groundwater Depth
4/7/2016	4.8 m



GROUNDWATER ELEVATIONS

Shallow/ Single Installation ▽ ▼ Deep/Dual Installation ▽ ▽

GRAPH NOTES $+^3, \times^3$: Numbers refer to Sensitivity ○ ε=3% Strain at Failure

Sheet No. 1 of 1

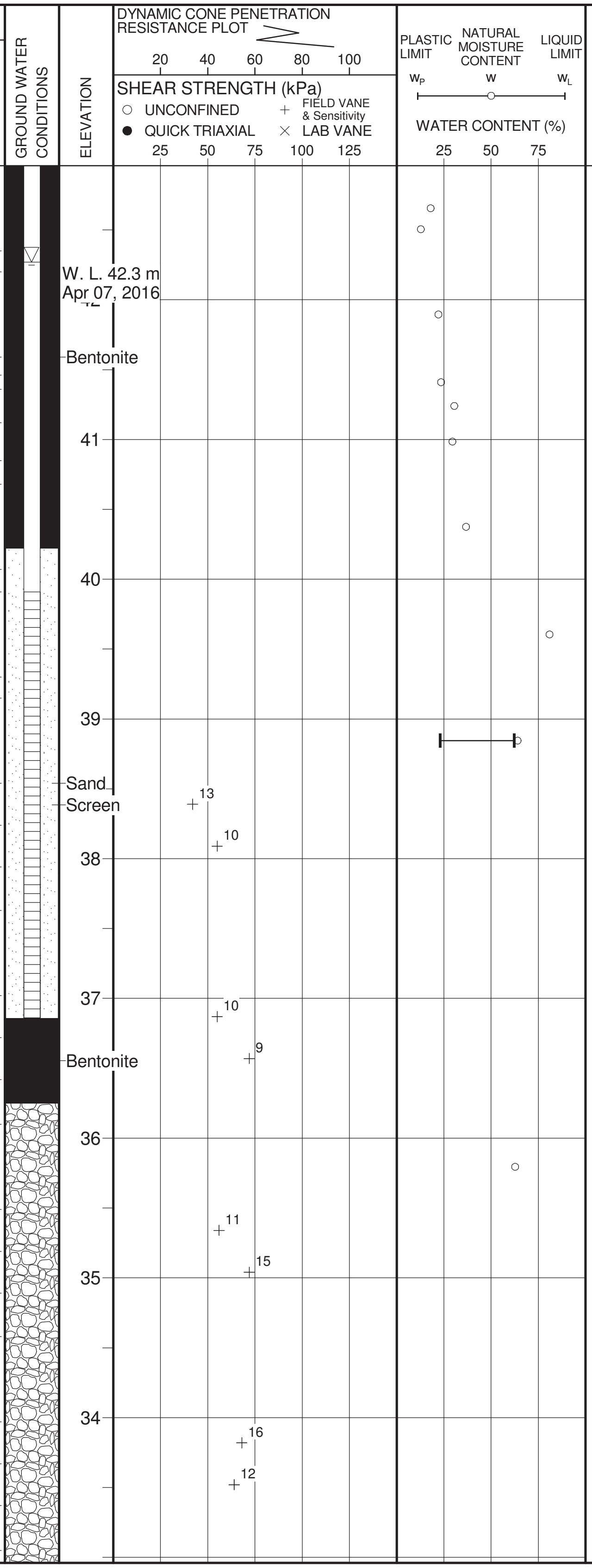
LOG OF BOREHOLE MW16-6

Project: Geotechnical Investigation
Client: Grandmaître Family
Project Location: Part Lot 30, Concession 1, Parts 1 & 2, Cumberland, ON
Datum: Approximate
BH Location: See borehole location plan N 462225 E 5038410

DRILLING DATA
Rig Type:
Method: Hollow Stem Auger
Borehole Diameter: 203 mm
Core Diameter:

Project No.: 161-03361-00
Date Started: 3/23/2016
Supervisor:
Reviewer:

SOIL PROFILE		SAMPLES			REMARKS AND GRAIN SIZE DISTRIBUTION (%)
(m) ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" BLOWS 0.3 m	GR SA SI CL
43.0					
42.9 0.0	**TOPSOIL - 20 mm** **CRUSHED SAND AND GRAVEL** trace silt, brown, wet, compact (FILL)	1A 1B	SS	28	39 52 (9)
42.2 0.8	**CLAYEY SILT** some sand, trace gravel, trace brick, dark brown, moist, compact (FILL)	2	SS	10	
41.4 1.6	**SANDY SILT** trace gravel, dark brown, moist, loose (FILL)	3A 3B	SS		
41.1 1.8	**SILTY CLAY**: trace to some gravel, trace to some sand, brown, moist, firm (FILL)	3C			
		4A	SS		
39.9 3.1	**SILTY CLAY**: grey brown, moist, stiff (WEATHERED CRUST)	5	SS	7	
39.2 3.8	**SILTY CLAY**: grey, wet, stiff	6	SS	WH	0 0 26 74
			VANE		
			VANE		
		A	TW		
			VANE		
			VANE		
		7	SS	WH	
			VANE		
			VANE		
		B	TW		
			VANE		
			VANE		



POCKET PEN. (Cu) (kPa)

NATURAL UNIT WT (KN/m³)

Continued Next Page

GROUNDWATER ELEVATIONS

Shallow/ Single Installation ▽ ▼ Deep/Dual Installation ▽ ▼

GRAPH NOTES + 3, × 3: Numbers refer to Sensitivity ○ ε=3% Strain at Failure

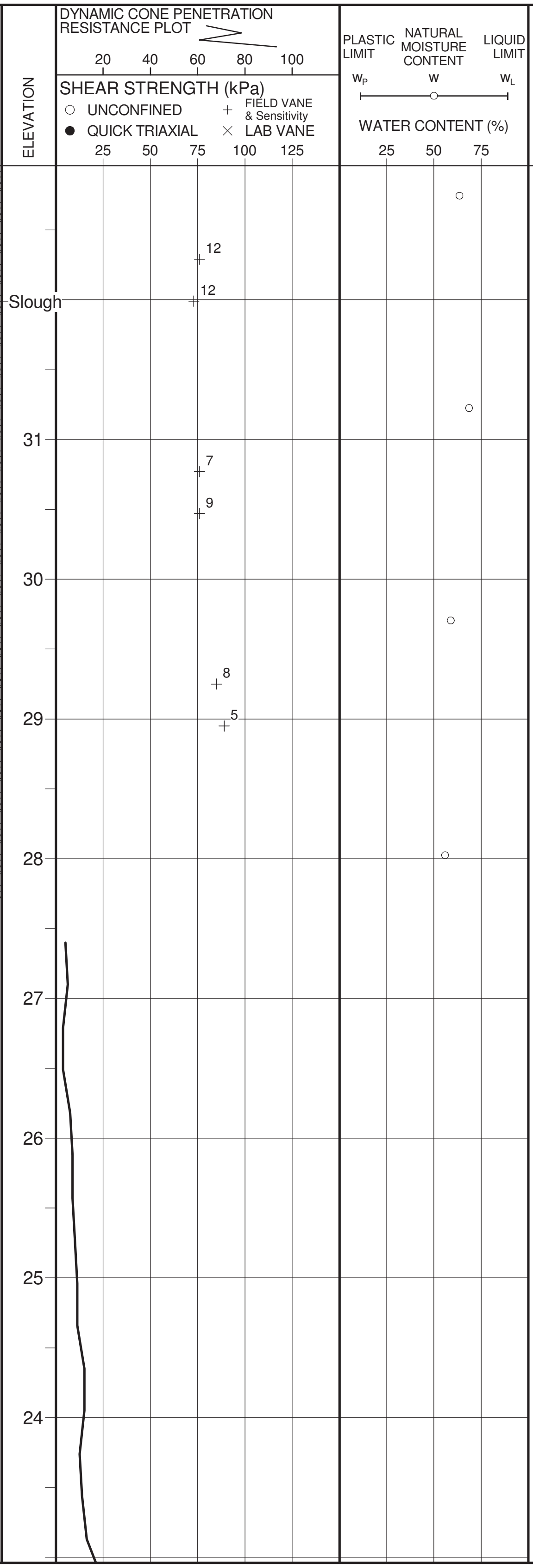
LOG OF BOREHOLE MW16-6

Project: Geotechnical Investigation
Client: Grandmaître Family
Project Location: Part Lot 30, Concession 1, Parts 1 & 2, Cumberland, ON
Datum: Approximate
BH Location: See borehole location plan N 462225 E 5038410

DRILLING DATA
Rig Type:
Method: Hollow Stem Auger
Borehole Diameter: 203 mm
Core Diameter:

Project No.: 161-03361-00
Date Started: 3/23/2016
Supervisor:
Reviewer:

(m) ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" BLOWS / 0.3 m	GROUND WATER CONDITIONS	FIELD VANE & Sensitivity	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	SILTY CLAY: grey, wet, stiff(Continued)	8	SS	WH			
			VANE			12	
			VANE		Slough	12	
		9	SS	WH			
			VANE			7	
			VANE			9	
		10	SS	WH			
			VANE			8	
			VANE			5	
		11	SS	4			
27.7 15.2	**SILTY CLAY**: grey, wet, stiff (Inferred based on DCPT results)						



POCKET PEN. (Cu) (kPa)
NATURAL UNIT WT (KN/m^3)

Continued Next Page

GROUNDWATER ELEVATIONS

Shallow/ Single Installation ▽ ▼ Deep/Dual Installation ▼ ▽

GRAPH NOTES $+^3, \times^3$: Numbers refer to Sensitivity ○ ε=3% Strain at Failure

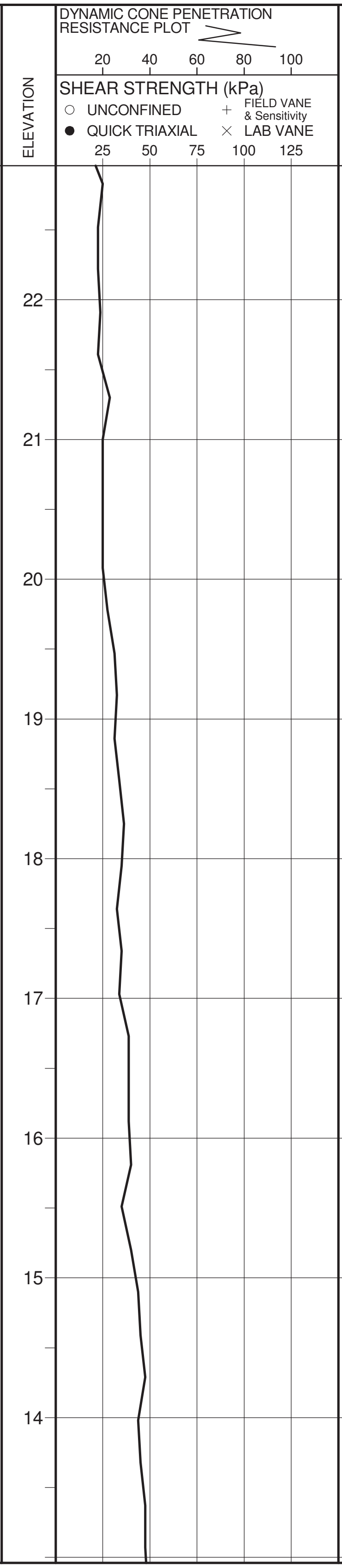
LOG OF BOREHOLE MW16-6

Project: Geotechnical Investigation
Client: Grandmaître Family
Project Location: Part Lot 30, Concession 1, Parts 1 & 2, Cumberland, ON
Datum: Approximate
BH Location: See borehole location plan N 462225 E 5038410

DRILLING DATA
Rig Type:
Method: Hollow Stem Auger
Borehole Diameter: 203 mm
Core Diameter:

Project No.: 161-03361-00
Date Started: 3/23/2016
Supervisor:
Reviewer:

SOIL PROFILE			SAMPLES										
(m) ELEV DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" BLOWS/0.3 m	GROUND WATER CONDITIONS	ELEVATION	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH (kPa)	PLASTIC LIMIT / NATURAL MOISTURE CONTENT / LIQUID LIMIT (WATER CONTENT (%))	POCKET PEN. (Cu) (kPa)	NATURAL UNIT WT (KN/m³)	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	SILTY CLAY: grey, wet, stiff (Inferred based on DCPT results)(Continued)											



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GROUNDWATER ELEVATIONS

Shallow/ Single Installation ▽ ▼ Deep/Dual Installation ▼ ▽

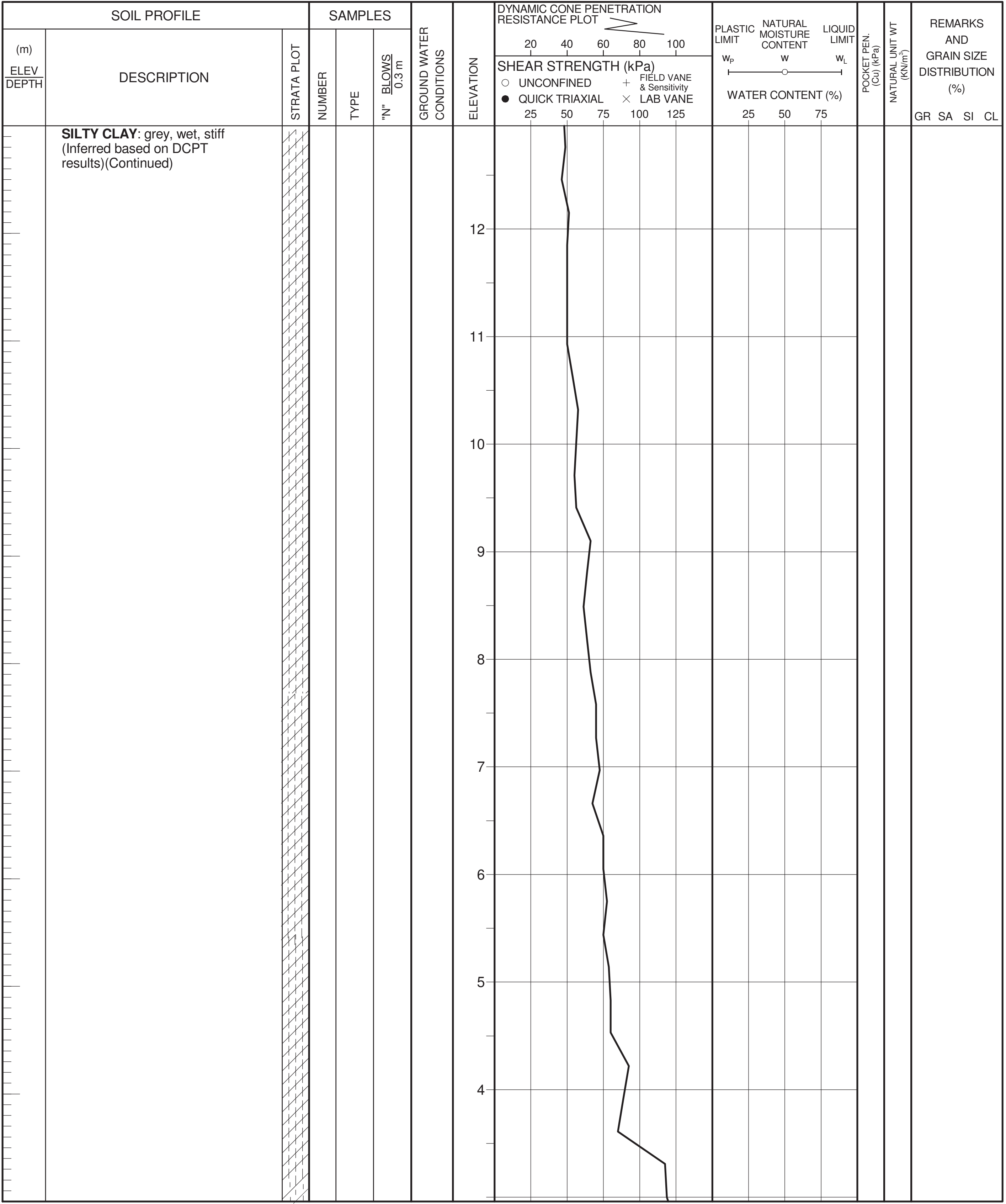
GRAPH NOTES $+^3, \times^3$: Numbers refer to Sensitivity ○ ε=3% Strain at Failure

LOG OF BOREHOLE MW16-6

Project: Geotechnical Investigation
Client: Grandmaître Family
Project Location: Part Lot 30, Concession 1, Parts 1 & 2, Cumberland, ON
Datum: Approximate
BH Location: See borehole location plan N 462225 E 5038410

DRILLING DATA
Rig Type:
Method: Hollow Stem Auger
Borehole Diameter: 203 mm
Core Diameter:

Project No.: 161-03361-00
Date Started: 3/23/2016
Supervisor:
Reviewer:



SILTY CLAY: grey, wet, stiff (Inferred based on DCPT results)(Continued)

Continued Next Page

GROUNDWATER ELEVATIONS

Shallow/ Single Installation ▽ ▼ Deep/Dual Installation ▼ ▽

GRAPH NOTES $+^3, \times^3$: Numbers refer to Sensitivity ○ ε=3% Strain at Failure

Sheet No. 4 of 5

WSP SOIL LOG - OTTAWA GEOTECHNICAL BH LOGS - 1009 TRIM ROAD, OTTAWA, ONTARIO.GPJ SPL.GDT 5/5/16

LOG OF BOREHOLE MW16-6

Project: Geotechnical Investigation	**DRILLING DATA**	
Client: Grandmaître Family	Rig Type:	Project No.: 161-03361-00
Project Location: Part Lot 30, Concession 1, Parts 1 & 2, Cumberland, ON	Method: Hollow Stem Auger	Date Started: 3/23/2016
Datum: Approximate	Borehole Diameter: 203 mm	Supervisor:
BH Location: See borehole location plan N 462225 E 5038410	Core Diameter:	Reviewer:

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH (kPa) ○ UNCONFINED + FIELD VANE & Sensitivity ● QUICK TRIAXIAL × LAB VANE 25 50 75 100 125	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 25 50 75	POCKET PEN. (Cu) (kPa)	NATURAL UNIT WT (kN/m^3)	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
(m) ELEV DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" BLOWS 0.3 m							
2.7 40.3	**END OF BOREHOLE** 1) End of augering at 15.2 m below the existing ground surface. Switch to DCPT. 2) Seepage noted at the bottom of borehole upon completion of augering. 3) DCPT refusal at 40.3 m below the existing ground surface. 4) 31 mm monitoring well installed at 6.1 m below the existing ground surface. 5) Date Groundwater Depth -- 4/7/2016 0.7 m											

WSP SOIL LOG - OTTAWA GEOTECHNICAL BH LOGS - 1009 TRIM ROAD, OTTAWA, ONTARIO.GPJ SPL.GDT 5/5/16

GROUNDWATER ELEVATIONS

Shallow/ Single Installation ▽ ▼ Deep/Dual Installation ▼ ▽

GRAPH NOTES $+^3, \times^3$: Numbers refer to Sensitivity ○ ε=3% Strain at Failure

Appendix B: Test-pit Logs

TEST PIT NUMBER (ELEVATION)	**DEPTH (METRES)**	**DESCRIPTION**
TP 16-1 (44.4 m)	0.0 – 1.2 1.2 – 1.8 1.8 – 4.0 4.0 – 6.7 6.7	Crushed Sand and Gravel, black, moist (FILL) Silty Clay some sand, trace to some gravel, dark brown, moist (Fill) Silt Clay, trace roots and organics, brown-grey, moist (WEATHERED CRUST) Silty Clay, grey, moist End of Test Pit

<table>
<tr><th>TEST PIT NUMBER (ELEVATION)</th><th>DEPTH (METRES)</th><th>DESCRIPTION</th></tr>
<tr><td>TP 16-2
(45.7 m)</td><td>0.0 – 2.1</td><td>Silty Sand and Crushed Gravel with boulders/cobbles, trace to some clay, brown, moist (FILL)</td></tr>
<tr><td></td><td>2.1 – 3.4</td><td>Silty Clay mixed with organic deposits, brown, moist</td></tr>
<tr><td></td><td>3.4 – 6.7</td><td>Silty Clay. grey-brown, moist (WEATHERED CRUST)</td></tr>
<tr><td></td><td>6.7</td><td>End of Test Pit</td></tr>
<tr><td colspan="3"></td></tr>
</table>

TEST PIT NUMBER (ELEVATION)	DEPTH (METRES)	DESCRIPTION
TP 16-3 (45.9 m)	0.0 – 0.9	Crushed Sand and Gravel, with boulders/cobbles, grey, moist (FILL)
	0.9 - 2.6	Silty Clay, trace sand, trace to some gravel, brown, moist (FILL) - Roots 1.7 m in depth
	2.6 – 4.3	Silty Clay, some gravel, trace to some roots and organic material, grey-brown, moist
	4.3 – 7.3	Silty Clay, grey-brown, moist (Weathered Crust)
	7.3	End of Test Pit
		
		

<table>
<tr><th>TEST PIT NUMBER (ELEVATION)</th><th>DEPTH (METRES)</th><th>DESCRIPTION</th></tr>
<tr><td>TP 16-4
(47.3 m)</td><td>0.0 – 1.8
1.8 – 4.0
4.0 – 6.4
6.4 – 7.3
7.3</td><td>Crushed Sand and Gravel with boulders/cobbles, grey, moist (FILL)
Silty Sand and Gravel, some clay to clayey, brown, moist (FILL)
Silty Clay, trace to some gravel, trace roots, grey-brown, moist (FILL)
Organic Soil mixed with roots, black, moist
End of Test Pit</td></tr>
<tr><td>Sample
1</td><td>Depth
0 – 0.6 m</td><td><table><tr><th><u>% Gravel</u></th><th><u>% Sand</u></th><th><u>% Fines</u></th></tr><tr><td>84</td><td>15</td><td>1</td></tr></table></td></tr>
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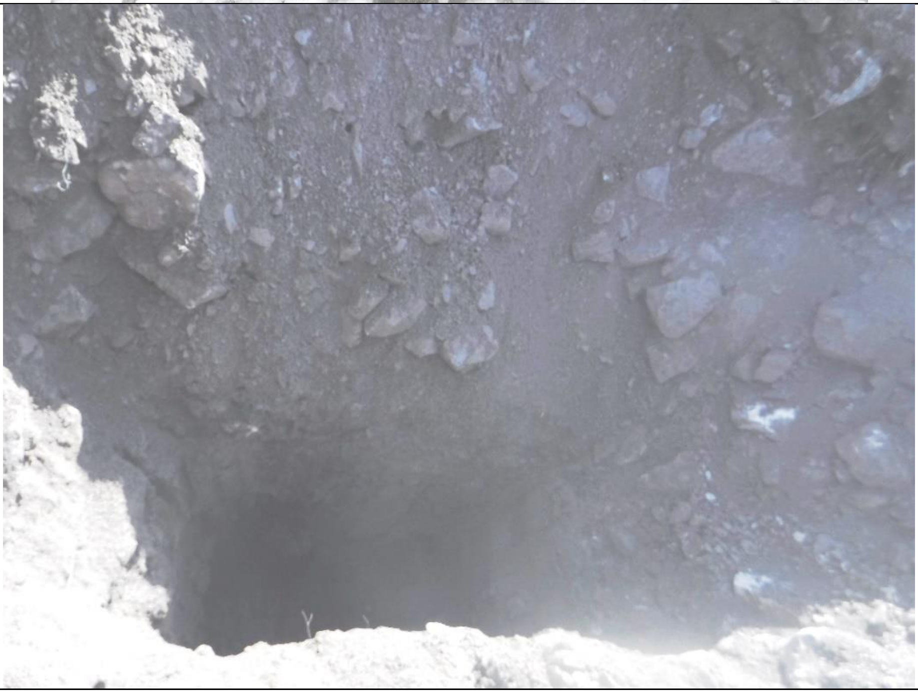
<tr><td colspan="3"></td></tr>
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APPENDIX 2

FIGURE 1 – KEY PLAN

GLOBAL STABILITY ANALYSIS - FIGURES 1A TO 2B

DRAWING PG5336-1 – TEST HOLE LOCATION PLAN

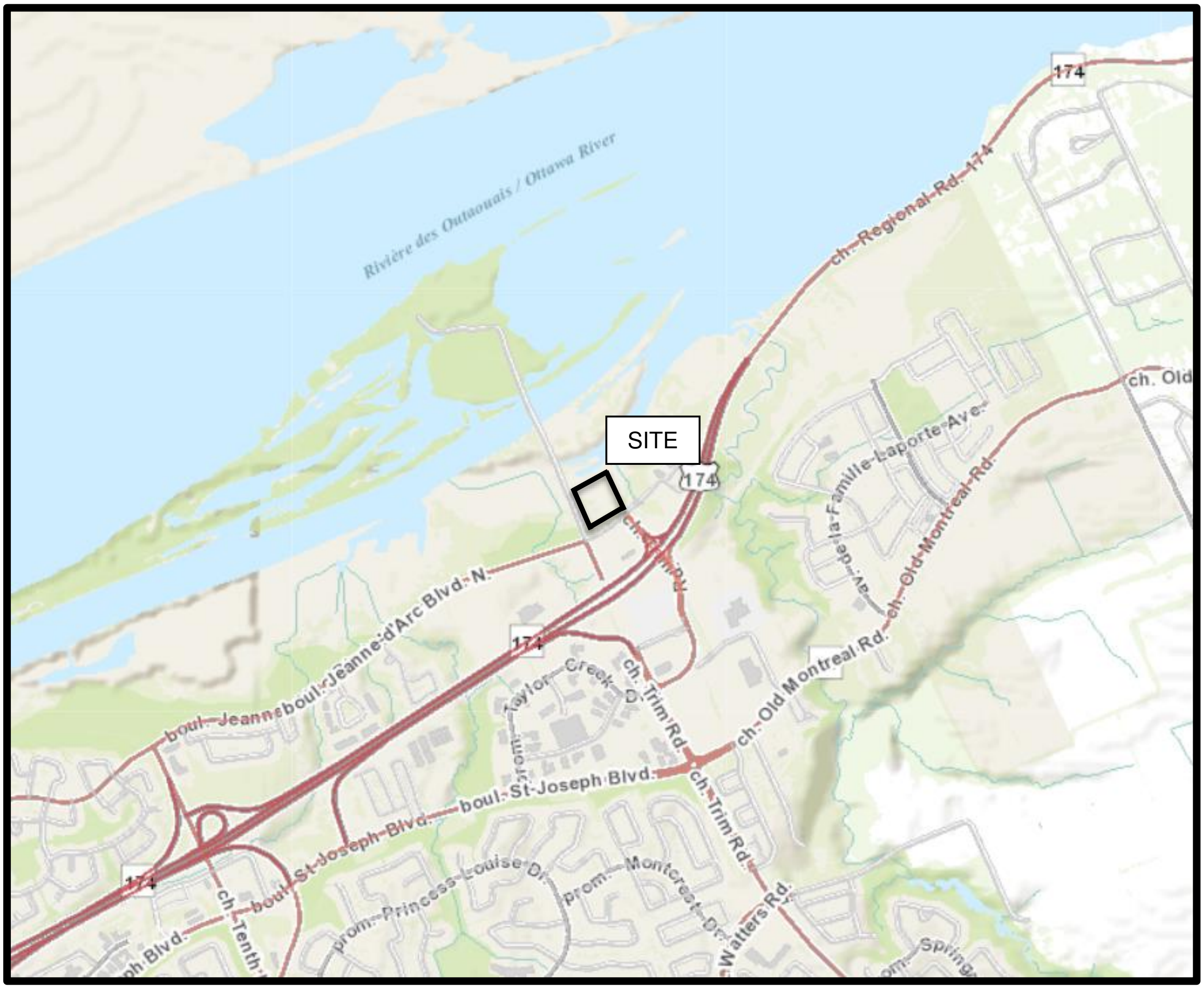
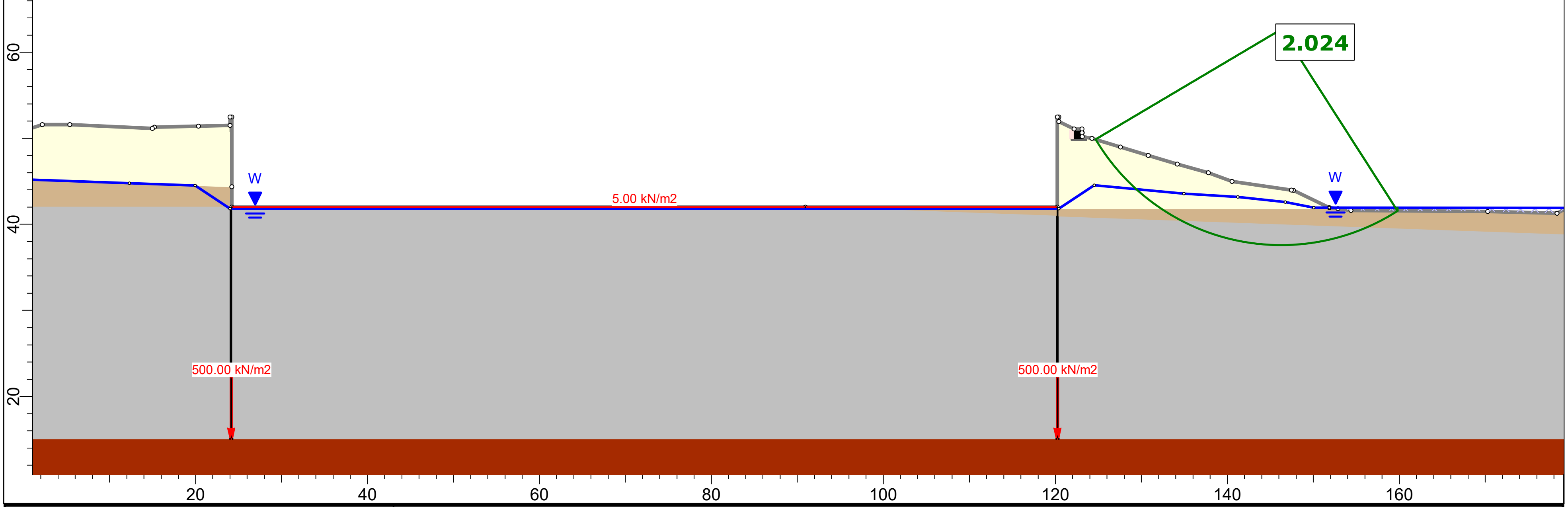


FIGURE 1

KEY PLAN

Material Name	Color	Unit Weight (kN/ m3)	Strength Type	Cohesion (kPa)	Phi (deg)
Topsoil	🟩	15	Mohr-Coulomb	7	30
Fill	🟨	18	Mohr-Coulomb	2	28
Engineered Fill	◼	22	Mohr-Coulomb	0	34
Lightweight Fill (LWF)	◻	0.3	Undrained	10	
Silty Clay Crust	🟫	16	Mohr-Coulomb	10	33
Concrete	⬛	22	Infinite strength		
Grey Silty Clay	⬜	16	Mohr-Coulomb	7	27
Bedrock	🟥	22	Infinite strength		

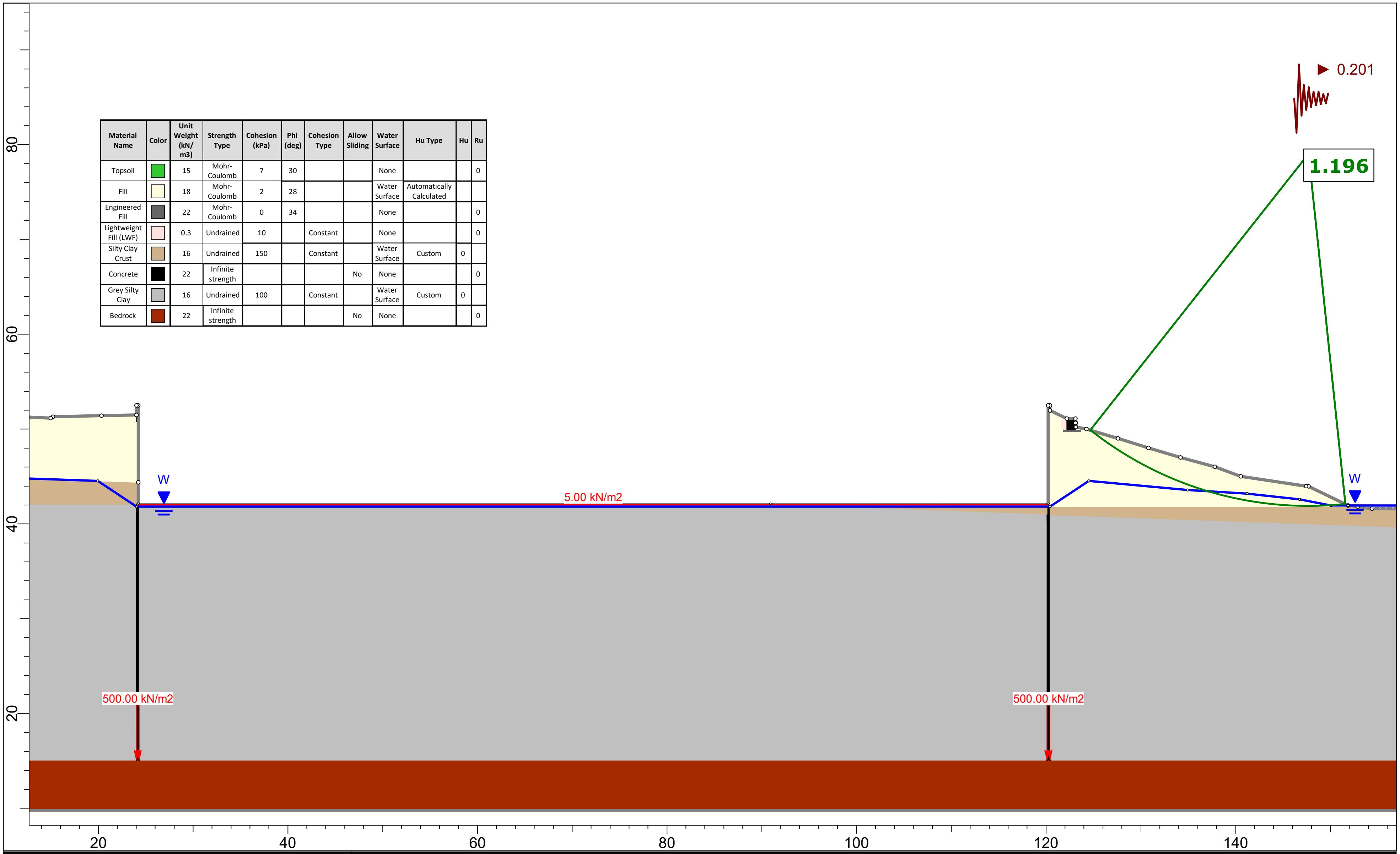


PATERSON GROUP

SLIDEINTERPRET 9.025

Project	PG5336 - 1015 Tweddle Road
Group	Figure 1A - Section A - Static Analysis

Drawn By	PT	*Company*	Vuze Construction
Date	02/2026	*File Name*	Slope Stability Assessment



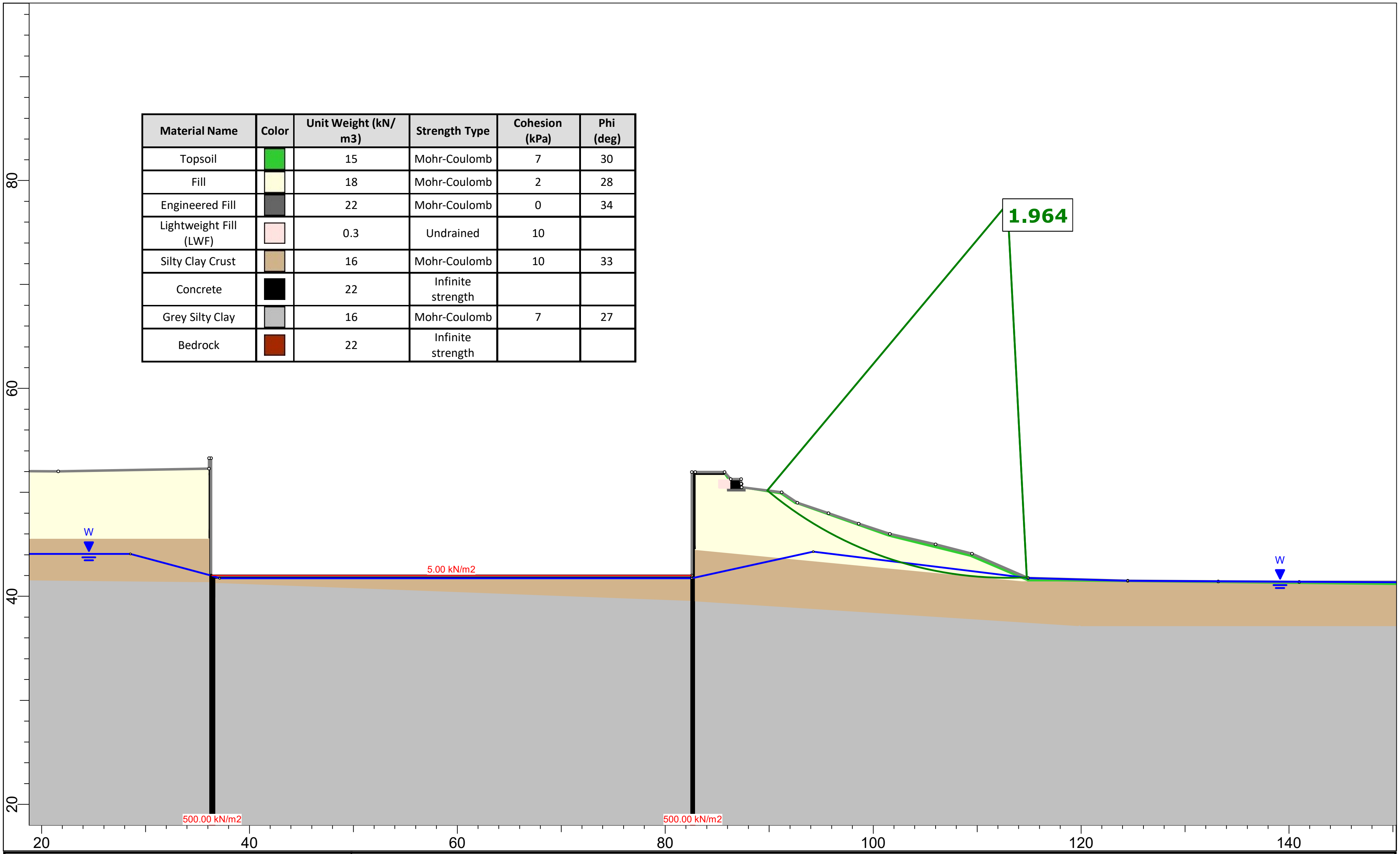
Material Name	Color	Unit Weight (kN/ m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Cohesion Type	Allow Sliding	Water Surface	Hu Type	Hu	Ru
Topsoil	🟩	15	Mohr-Coulomb	7	30			None			0
Fill	⬜	18	Mohr-Coulomb	2	28			Water Surface	Automatically Calculated		
Engineered Fill	⬛	22	Mohr-Coulomb	0	34			None			0
Lightweight Fill (LWF)	⬜	0.3	Undrained	10		Constant		None			0
Silty Clay Crust	🟫	16	Undrained	150		Constant		Water Surface	Custom	0	
Concrete	⬛	22	Infinite strength				No	None			0
Grey Silty Clay	⬜	16	Undrained	100		Constant		Water Surface	Custom	0	
Bedrock	🟥	22	Infinite strength				No	None			0

PATERSON GROUP

SLIDEINTERPRET 9.025

Project	PG5336 - 1015 Tweddle Road
Group	Figure 1B - Section A - Seismic Analysis

Drawn By	PT	*Company*	Vuze Construction
Date	02/2026	*File Name*	Slope Stability Assessment

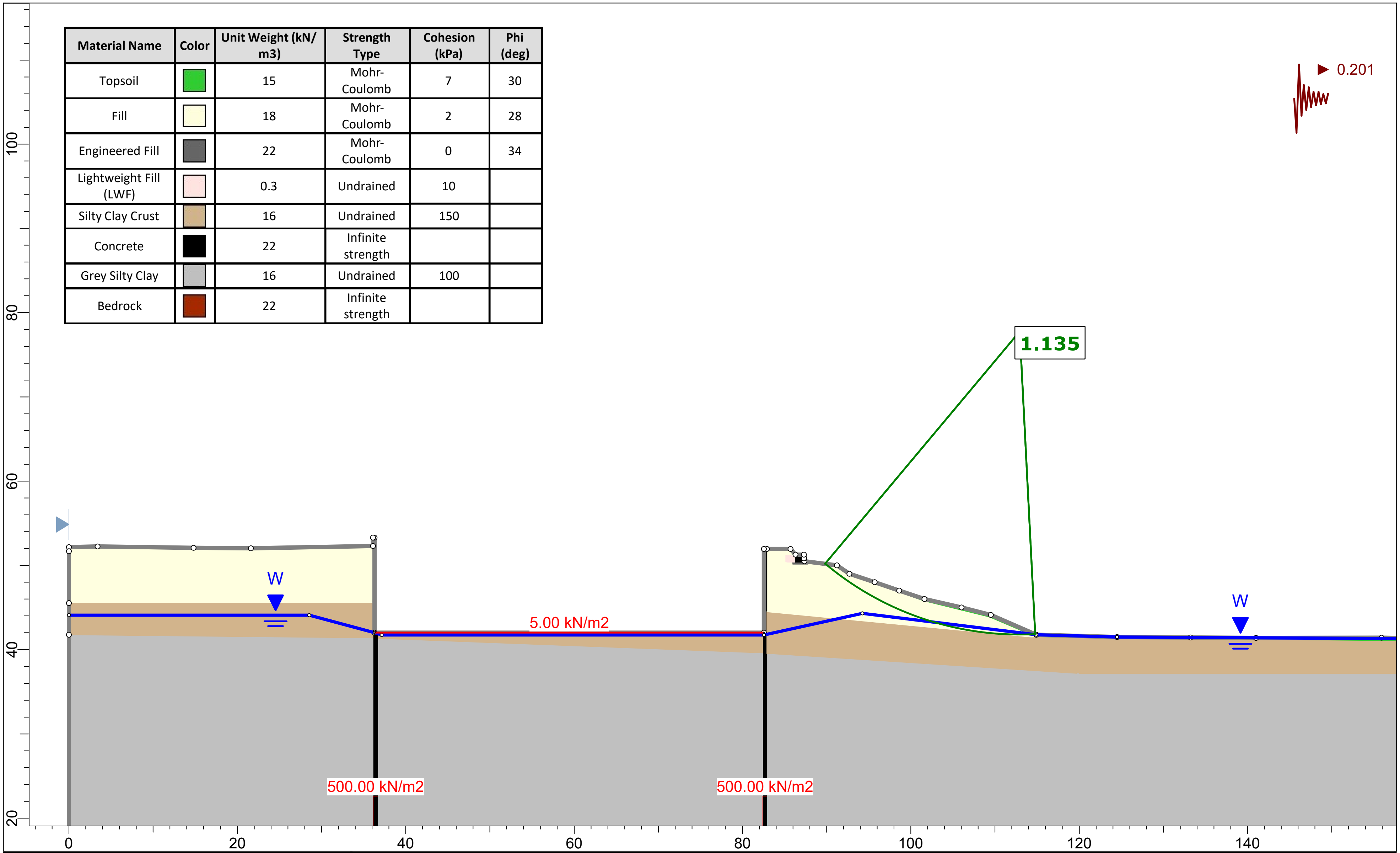


Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)
Topsoil	🟩	15	Mohr-Coulomb	7	30
Fill	🟨	18	Mohr-Coulomb	2	28
Engineered Fill	⬛	22	Mohr-Coulomb	0	34
Lightweight Fill (LWF)	⬜	0.3	Undrained	10	
Silty Clay Crust	🟫	16	Mohr-Coulomb	10	33
Concrete	⬛	22	Infinite strength		
Grey Silty Clay	⬜	16	Mohr-Coulomb	7	27
Bedrock	🟥	22	Infinite strength		

PATERSON GROUP

SLIDEINTERPRET 9.025

Project	PG5336 - 1015 Tweddle Road
Group	Figure 2A - Section B - Static Analysis
Drawn By	PT
Company	Vuze Construction
Date	02/2026
File Name	Slope Stability Assessment

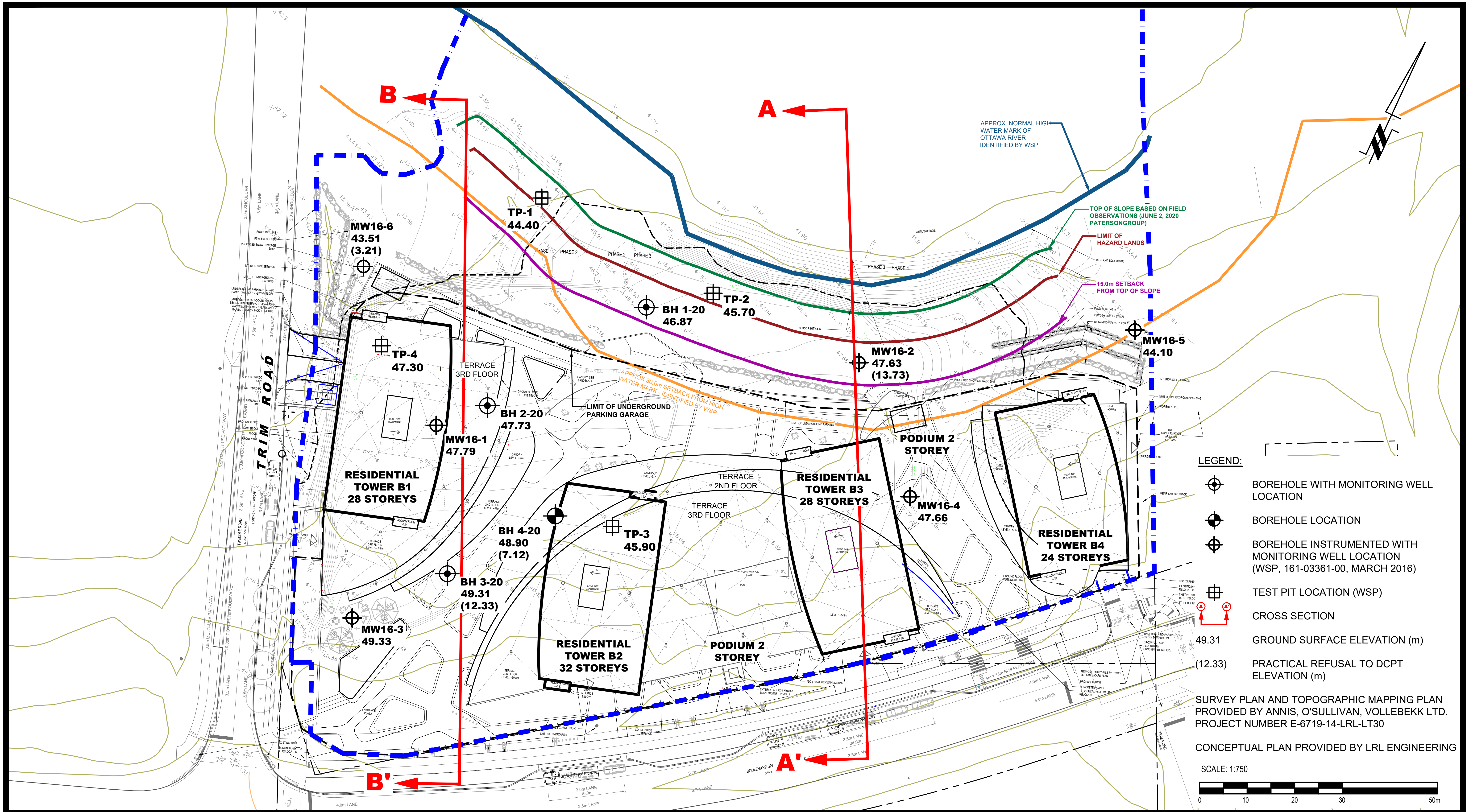


Material Name	Color	Unit Weight (kN/ m3)	Strength Type	Cohesion (kPa)	Phi (deg)
Topsoil	🟩	15	Mohr-Coulomb	7	30
Fill	🟨	18	Mohr-Coulomb	2	28
Engineered Fill	⬛	22	Mohr-Coulomb	0	34
Lightweight Fill (LWF)	⬜	0.3	Undrained	10	
Silty Clay Crust	🟫	16	Undrained	150	
Concrete	⬛	22	Infinite strength		
Grey Silty Clay	⬜	16	Undrained	100	
Bedrock	🟥	22	Infinite strength		

PATERSON GROUP

SLIDEINTERPRET 9.025

Project	PG5336 - 1015 Tweddle Road
Group	Figure 2B - Section B - Seismic Analysis
Drawn By	PT
Company	Vuze Construction
Date	02/2026
File Name	Slope Stability Assessment



LEGEND:

⌖	BOREHOLE WITH MONITORING WELL LOCATION
●	BOREHOLE LOCATION
⌖	BOREHOLE INSTRUMENTED WITH MONITORING WELL LOCATION (WSP, 161-03361-00, MARCH 2016)
⊞	TEST PIT LOCATION (WSP)
A A'	CROSS SECTION
49.31	GROUND SURFACE ELEVATION (m)
(12.33)	PRACTICAL REFUSAL TO DCPT ELEVATION (m)

SURVEY PLAN AND TOPOGRAPHIC MAPPING PLAN PROVIDED BY ANNIS, O'SULLIVAN, VOLLEBEKK LTD. PROJECT NUMBER E-6719-14-LRL-LT30

CONCEPTUAL PLAN PROVIDED BY LRL ENGINEERING

SCALE: 1:750

0 10 20 30 50m

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

NO.	REVISIONS	DATE	INITIAL
7	UPDATED TO LATEST CONCEPTUAL PLAN	2805/2025	PT
6	UPDATED TO LATEST CONCEPTUAL PLAN	10/02/2024	JV
5	UPDATED TO LATEST CONCEPTUAL PLAN	10/12/2021	JV
4	SLOPE STABILITY INFORMATION ADDED	08/19/2020	RG
3	UPDATED TO LATEST CONCEPTUAL PLAN	08/18/2020	RG

STARWOOD GROUP INC.
GEOTECHNICAL INVESTIGATION
PROPOSED MULTI-STOREY BUILDING COMPLEX
1009 TRIM ROAD

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:750
Date: 04/2020
Drawn by: RCG
Report No.: PG5336-1
Checked by: RG
Dwg. No.: PG5336-1
Approved by: DJG
Revision No.: 7

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