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Slope Stability Review and Landslide Risk Assessment

Proposed Multi-Storey Building Complex
1009 Trim Road - Ottawa

Prepared For

Starwood Group Inc.

Paterson Group Inc.
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August 18, 2020

Report: PG3556-LET.01, Revision 2

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Report: PG5336-LET.01, Revision 2

Starwood Group Inc.
188 Eglington Avenue East, Suite 800
Toronto, Ontario,
M4P 2X7

Attention **Mr. Martin Chénier**

Subject: **Slope Stability Review and Landslide Risk Assessment**
Proposed Multi-Storey Building Complex
1009 Trim Road - Ottawa

Dear Sir,

Further to your request and authorization, Paterson Group (Paterson) prepared a slope stability review and landslide risk assessment as part of the proposed multi-storey building complex located at the aforementioned site.

The following slope stability review and landslide risk assessment was based on our evaluation of the existing slope bordering the south boundary of the site taking into consideration of the proposed development, available subsoil information within the immediate area of the site and our general knowledge of the areas geology.

The following slope stability review and landslide risk assessment was based on the available geotechnical information from WSP's Geotechnical Investigation Report, Project 161-03361-00 dated September, 2016.

1.0 Historical Information

As part of the current slope stability review, Paterson reviewed the available information recovered during the previous geotechnical investigations and slope stability assessments completed by Paterson within the immediate area of the subject site:

- ❏ Geotechnical Report, Proposed Hi-Rise Building, Towers 3, 4 and 5B, Petrie's Landing, 8900 Jean D'Arc Boulevard (Inlet Private), PG3908-2, Revision 6 dated March 2, 2020.

- ☐ Slope Stability Assessment Report (SSAR), Proposed Multi-Storey Buildings, Towers 3, 4, 5a and 5b, Petrie's Landing, Inlet Private, PG3908-LET.03 Revision 2 dated September 23, 2019.

- ☐ Landslide Risk Assessment - Proposed Multi-Storey Building Complex - Petrie's Landing - Inlet Private - Ottawa - Report: PG3908-MEMO.09 prepared by Paterson for Brigil Construction dated October 16, 2019.

In addition, Paterson reviewed the available information recovered during the geotechnical investigations from the neighbouring property to the southeast prepared by GHD Limited (which was formerly Inspec-Sol):

- ☐ Geotechnical Preliminary Investigation, Tower B, Highway 174 and Trim Road, Orleans, Ontario, Report T020548-A1 dated August 13, 2013 (Inspec-Sol)

- ☐ Additional Slope Stability Assessment, Petrie's Landing I - Tower II (Phase 2), Petrie Island, Ottawa, Ontario, Report T020548-A1 dated June 5, 2014 (Inspec-Sol)

- ☐ Geotechnical Investigation, Petrie's Landing Tower 2, 8900 Jeanne D'Arc Boulevard, Ottawa, Ontario, Report T020548-A2 dated June 22, 2016.

- ☐ Response to Golder Associates Ltd. Comments, Geotechnical Investigation Report (T020548-A1, dated August 13, 2013), dated June 29, 2016.

In addition, Paterson reviewed the following slope stability comments issued in the following peer review prepared by Golder Associates (Golder) for the existing slope bordering the south boundary of the subject site:

- ☐ Engineering Peer Review, Geotechnical Investigation and Slope Stability Assessment, Inspec-Sol Reports, 8900 Jeanne D'Arc Boulevard - Tower 2, Orleans, Ontario, Project 1650934 dated March 15, 2016.

2.0 Available Information

The current slope stability analysis was completed using the information recovered during our site visit in January 2019, topographic survey plan prepared by Annis, O'Sullivan, Vollebekk Ltd. dated September 24, 2014, Grading and Drainage Plan prepared by LRL Associate Engineers dated March 25, 2015, subsoil information recovered during the previous geotechnical investigations within the immediate area of the site as well as our general knowledge of the area's geology.

Subsoil Conditions

The subsoil and groundwater conditions used as part of the slope stability analysis was recovered from the geotechnical investigation report, Project 161-03361-00 dated September, 2016 prepared by others. Furthermore, a supplemental geotechnical investigation was completed by out firm and out findings have been summarized in our report, PG5336-1 dated August 18, 2020. Generally, the subsurface profile at the test hole locations within the subject site consists of varying thickness of fill consisting of a silty sand mixed with clay and/or crushed stone and gravel overlying a very stiff to stiff silty clay deposit extending to depths varying between 25 to 30 m below existing ground surface. The upper portion of the silty clay deposit was weathered to a very stiff brown silty clay crust extending to depths varying between 3 to 8 m below existing ground surface which in turn becomes stiff and grey at depth when overlying the bedrock surface.

In situ shear vane field testing and standard penetration tests carried out within the silty clay deposit were indicative of a very stiff weathered silty clay crust which was underlying by a stiff to firm consistency.

Six representative soil samples were submitted for grain size analysis from the test holes completed during the geotechnical investigation (by WSP) at the aforementioned site. The results of the grain size analysis are presented in Particle-Size Analysis of Soils (ASTM D422) attached to the current report.

In addition, seven representative soil samples were submitted for Atterberg Limits testing from the test holes completed during the geotechnical investigation (by WSP) at the aforementioned site. The Atterberg Limits test results are presented in Liquid Limit, Plastic Limit and Plasticity Index of Soils (ASTM D43158) attached to the current report.

Practical refusal to DCPT was encountered at MW16-2 and MW16-5 at geodetic elevations varying between 3 and 13 m on inferred bedrock.

Based on the 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.

Seismic Considerations

As part of the geotechnical investigation competed by GHD for the initial phase of the multi-storey development located within the immediate vicinity of the subject site, a geophysical (MASW) testing was completed to provide a site specific seismic site classification. Based on the results of the seismic testing, the average shear wave velocity Vs_{30}, was establish to be greater that **180 m/s**. Therefore, a **Site Class D** is applicable for foundation design within that area where similar soil conditions are encountered, as per Table 4.1.8.4.A of the OBC 2012. The results of the site specific geophysial (MASW) testing are presented in Table 1 - Summary of Shear Wave Velocity Measurements and in Figure 1 - Shear Wave Velocity Versus Depth attached to the current report.

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 2 below.

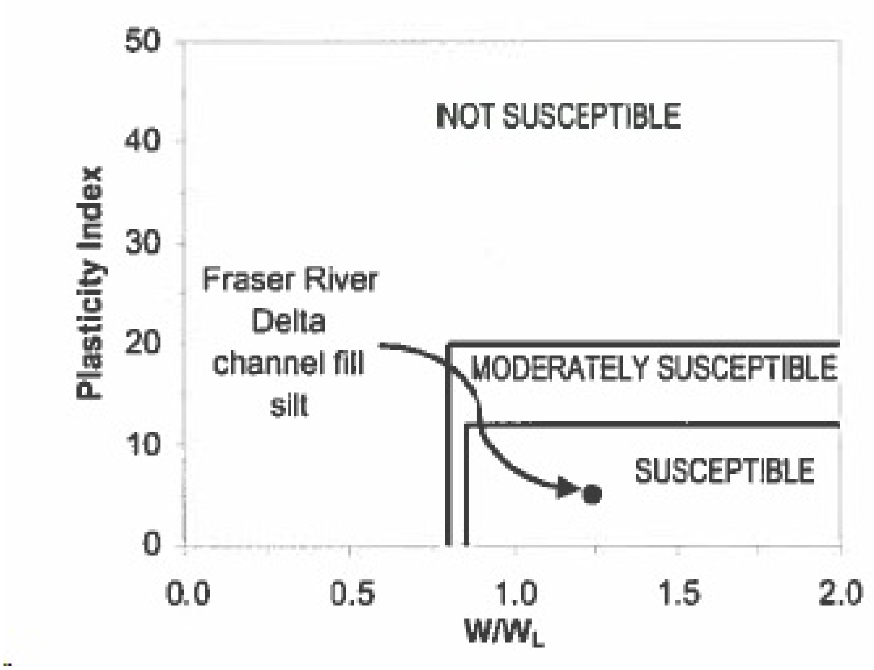


Figure 2 - Bray et al. (2004) criteria for liquefaction assessment of fine-grained soils

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.

3.0 Field Observations

The site is bordered to the west by Trim Road, to the south by Inlet Private and to the east by vacant l following by newly constructed multi-storey complex. The site is bordered to the north by the existing wetlands bordering the Ottawa River followed by a marina. The south and east portion of the site is densely treed with slopes measured at 4H:1V or flatter. No signs of active erosion was observed along the boundary of the wetlands that borders the north boundary of the subject site.

Sheet drainage from the table lands to the south of Inlet Private is directed by roadside ditches along Trim Road and through a drainage feature located within a confined channel located in excess of 50 m from the east property boundary.

A site inspection was carried out on June 2, 2020 to review and assess areas that have been historically in-filled. Based on our observations, the following commentary is provided:

- The majority of the subject site has been in-filled over the years creating a plateau where blast rock has been spread in the central portion of the site.

- To the north along the Ottawa River's edge, the natural slope is visible which is in close proximity to the existing wetland. The natural slope is similar to the slope observed on neighbouring properties to the west (across Trim Road) and to the east heading towards an existing development.

- The western boundary has been significantly raised from the natural grade primarily during the construction of Trim Road which created an embankment to reach Jeanne d'Arc Boulevard (Inlet Private) which is at a higher grade. To create a suitable grade for the roadway heading north to Petrie Island, a gradual slope was created which required a significant grade raise.

- The western boundary was also historically raised to match the grade along Trim Road and to create a higher plateau for the development site. Evidence of this grade raise is observed on the relatively undisturbed property immediately west of Trim Road which has low lying areas.

❏ The eastern portion of the site was also in-filled and an artificial drainage channel was created to direct any water accumulating along the northern side of Jeanne d'Arc Boulevard. The outlet of this channel has a berm which was most likely placed to manage the groundwater flow and avoid sediment run-off to the Ottawa River.

❏ The southern boundary of the subject site has a berm which appears to have been created during the installation of the trunk sewer and construction of Inlet Private. The surplus soil was most likely stockpiled along the tree line of the southern boundary and the drainage channel along the eastern boundary, was most likely created to prevent any water damming against the stockpiled soil and direct any water accumulation to the Ottawa River.

Based on the above observations, we can conclude that the subject site has been subjected to extensive historical in-filling activities at various periods.

4.0 Proposed Development

Based on the latest conceptual drawings provided, it's our understanding that the proposed two multi-storey structures identified will be founded on end-bearing piles driven to the underlying bedrock. It's also understood that the two level of underground parking structure will be shared between the two structures extending beyond the tower footprints and supported with conventional spread footings bearing on the undisturbed very stiff to stiff silty clay deposit.

Since the subject site will be lowered to accommodate two levels of underground parking which occupies most of the development area, the P-1 level will be exposed at the rear (northern boundary) and will match the roadway elevation along the southern boundary.

The P-2 level will be located below the natural grade and very close to the toe depth of the natural slope along the wetlands. Since the development will be set back a minimum of 30 m from the normal high water mark, there will be a gentle slope created along the northern boundary which will accommodate various potential landscaped features such as a pathway and reinstatement of natural features.

The excavated soil removed for the two level underground parking structure, which will occupy the majority of the subject site, will reduce the overall weight on the existing slope and stabilize the groundwater level to a lower elevation which will increase the overall stability of the existing slope.

5.0 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, 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 Starwood Group Inc. by incorporating the two level 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

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 1 below.

Table 1 - Effective Soil and Material Parameters (Static Analysis)

Soil Layer	Unit Weight (kN/m^3)	Friction Angle (degrees)	Cohesion (kPa)
Fill	18	28	2
Brown Silty Clay Crust	16	33	10
Grey Silty Clay	16	27	7
Bedrock	Impenetrable		

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 areas geology. The strength parameters used for seismic analysis at the slope cross-sections are presented in Table below.

Table 2 - Total Stress Soil and Material Parameters (Seismic Analysis)

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

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 under fully saturated conditions (worst-case-scenario) 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 2.1.

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, K_h, of 0.16G 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 analysis including seismic loading fully saturated conditions (worst-case-scenario) 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 1.5 which is considered to be stable under seismic loading.

6.0 Soil Sensitivity

The soil sensitivity is defined as the ratio of the undrained shear strength obtained by field vanes at in-situ state over the remoulded value. As per the Canadian Foundation Engineering Manual (April 1, 2018), the sensitivity results are categorized as below;

❏ Low sensitivity	$S<2$
❏ Medium sensitivity	$2<S<4$
❏ Sensitive	$4<S<8$
❏ Extra-sensitive	$8<S<16$
❏ Quick clay	$S>16$

Based on the boreholes completed by others, no field vane shear strengths were taken in the upper levels of the weathered silty clay crust. Where the sensitivity testing was undertaken in the deeper clay deposit below the weathered crust, the results ranged between 2.1 to 7.3, which is indicative of a medium sensitive to sensitive clay based on the Canadian Foundation Engineering Manual Addendum dated April 1, 2018. It's expected that the upper weathered silty clay crust will be considered as a low sensitivity clay deposit.

7.0 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. Generally, the erosion access allowance is taken as 6 m from the top of stable slope.

A slope stability assessment was carried out to determine the required stable slope allowance setback based on a factor of safety of 1.5 under static analysis and a factor of safety of 1.1 under seismic loading. A toe erosion and 6 m erosion access allowance were also considered in the determination of limits of hazard lands and are further discussed below.

Based on the proposed development, no significant slope stability issues are expected. Once the final grading is determined, a slope review will be carried out. However, since there is at least a 30 m set-back and a low natural slope near the edge of the wetland, a gentle slope will not pose any slope stability concerns.

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 three slope section 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 set back 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.

7.0 Landslide Risk Rationale and Justification

Paterson has evaluated the landslide risk of the overall area (beyond the boundaries of the subject site) which is approximately 500 m beyond the limit of the subject development. The land features identified within the 500 m radius are as follows:

- ❑ Low lying wetlands to the north which has created table lands to the south.
- ❑ Wetland channel along the Ottawa River between Petrie Islands and the shoreline.
- ❑ Causeway restricting water flow along the eastern portion of the channel.
- ❑ 4H:1V or flatter slope located within the south portion of the site which is heavily treed.
- ❑ Highway 174 and the future LRT station at Trim Road.
- ❑ Multi-storey building complex located to the east.
- ❑ The meandering Cardinal Creek valley and table lands to the south east.
- ❑ The residential development of Cardinal Creek Village southeast of the subject site.

The landslide risk assessment was rationalized as follows:

- ❑ As previously described, the silty clay deposit is considered as low sensitivity with elevated shear strengths and very deep weathered clay crust. Based on the thickness of the clay crust (average above 8 to 10 m) being significantly greater than the 6 m thickness stipulated in the paper presented by the Geological Survey of Canada, it is extremely unlikely to identify a landslide site. **Therefore, considering the natural state of the site, the subject site and the surrounding 500 m radius would not be susceptible to a landslide event**. There is no identified historical landslide flow documented in this area. The erosional failures along Cardinal Creek are not considered to be landslide flows.

- ❑ The proposed development will consist of building towers founded on end bearing piles extending to bedrock. The building towers will be connected by a two level underground parking garage. The founding depth of the development will be approximately 8 to 8.5 m below the existing grade of Inlet Private within the very stiff silty clay deposit. **Since the parking garage will occupy the bulk of the area, there is no potential of lateral movement of the structure which is designed to accommodate seismic conditions in accordance with the OBC and designed to the required seismic site classification.**

- ❑ The slope conditions meet the requirements of the MNR regulations and the slope stability assessment (including the seismic condition) was carried out in accordance with RVCA requirements for similar projects recently constructed in the Ottawa area. **Although a marine clay deposit, the subject site has a low sensitivity clay which permits a conventional slope stability assessment**.

❑ Based on the above, **the vulnerability estimation and the risk to life (safety risk) is considered negligible (less than 1:100,000)**. Although no official criteria exists in Ontario for risk tolerance, the lowest risk tolerance criteria established in other jurisdictions is 1:100,000 which is considered acceptable. In our opinion, there is no risk to life for this development.

We trust that this information satisfies your requirements.

Best Regards,

Paterson Group Inc.

Richard Groniger, C. Tech.

Carlos P. Da Silva, P.Eng., ing., QP_{ESA}

August 18, 2020
C. P. DA SILVA

Attachments

❑ Soil Profile and Test Data Sheets
❑ Symbols and Terms
❑ Soil Profile and Test Data Sheets (by others)
❑ Particle-Size Analysis of Soils (ASTM D422) (by others)
❑ Liquid Limit, Plastic Limit and Plasticity Index of Soils (ASTM D43158) (by others)
❑ Figure 1A - Section A - Proposed Conditions - Fully Saturated - Static Analysis.
❑ Figure 1B - Section A - Proposed Conditions - Fully Saturated - Seismic Loading
❑ Figure 2A - Section B - Proposed Conditions - Fully Saturated - Static Analysis.
❑ Figure 2B - Section B - Proposed Conditions - Fully Saturated - Seismic Loading
❑ Figure 3 - Aerial Photograph - 500 m Study Area
❑ Figure 4 - Area’s Geology
❑ Discussion Paper
❑ Drawing PG5336-1 (Rev.04) - Test Hole Location Plan
❑ Drawing PG5336-2 (Rev.01) - Surficial Geological Mapping and Historical Information
❑ Drawing PG5336-3 (Rev.01) - Surficial Geological Cross Sections

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

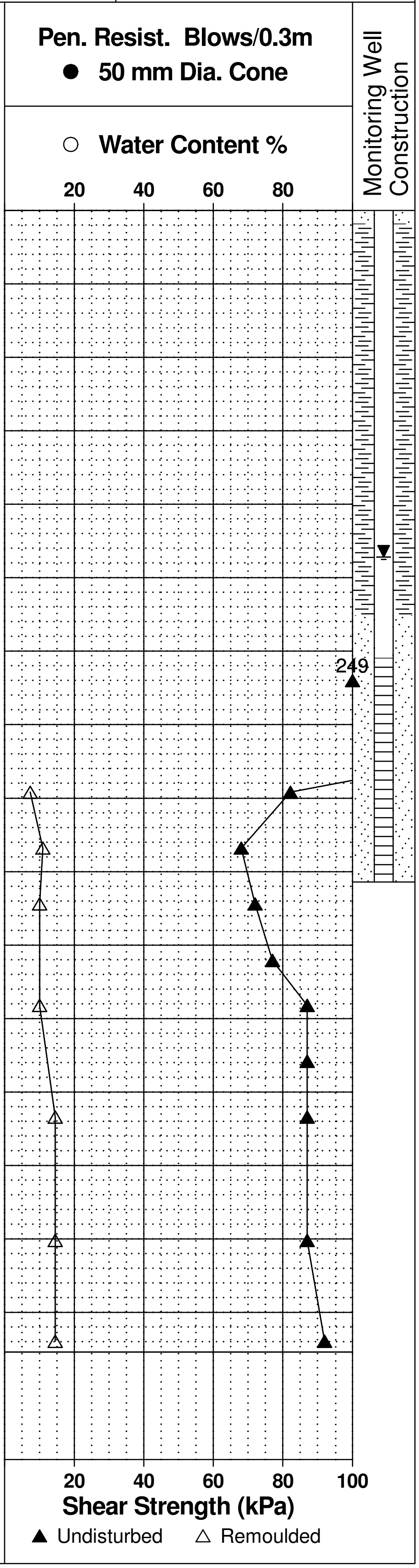
BORINGS BY Track-Mount Power Auger

DATE June 29, 2020

SOIL DESCRIPTION	
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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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

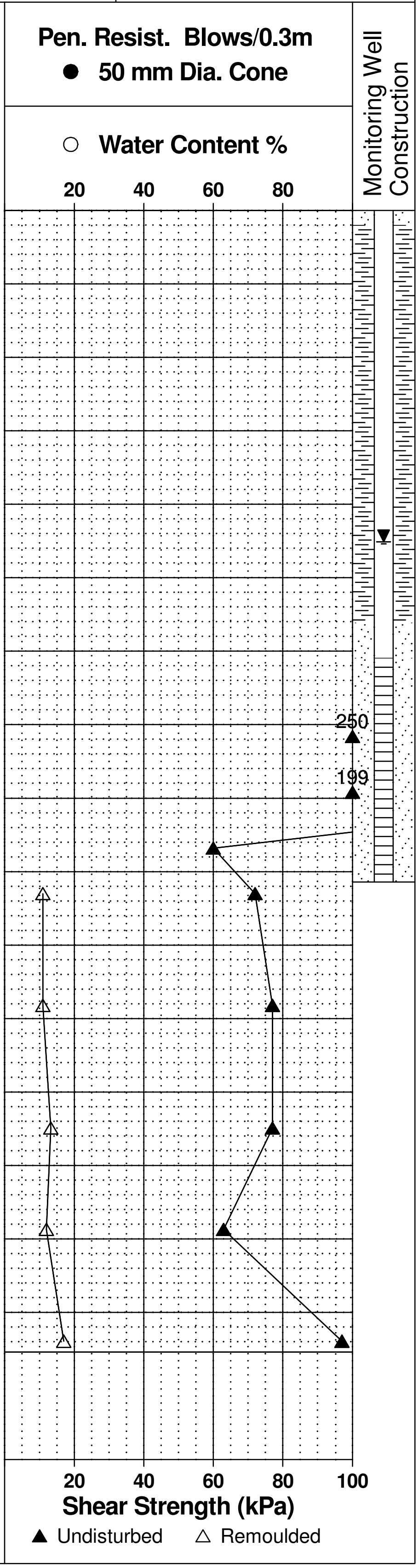
— 15.54

End of Borehole

(GWL @ 4.51m - July 17, 2020)

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

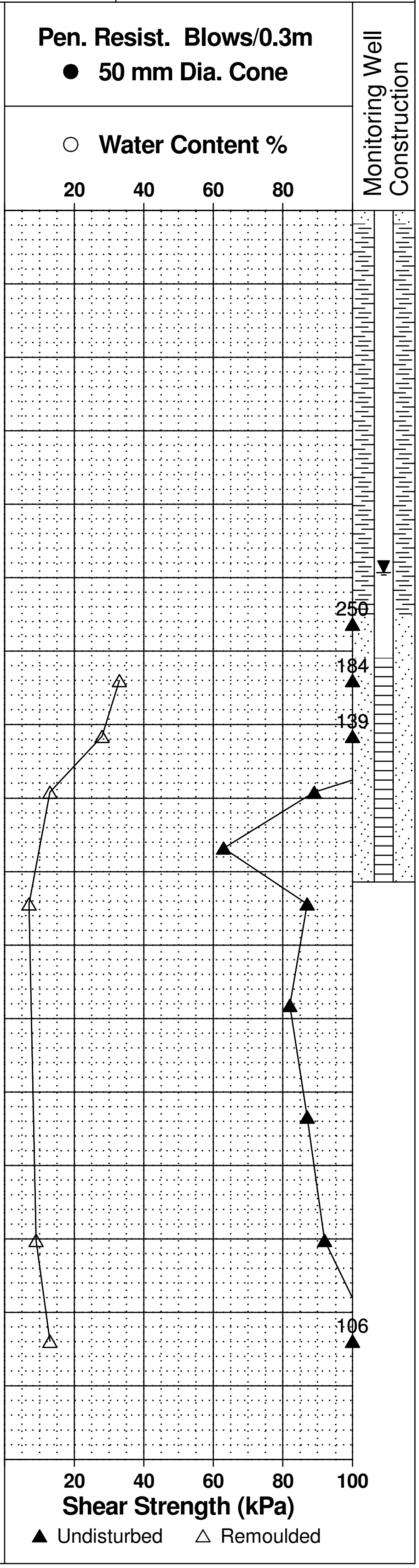
BORINGS BY Track-Mount Power Auger

DATE July 2, 2020

FILE NO. PG5336

HOLE NO. BH 3-20

SOIL DESCRIPTION						
GROUND SURFACE						
	TYPE	**NUMBER**	**% RECOVERY**	**N VALUE or RQD**	**DEPTH (m)**	**ELEV. (m)**
FILL: Brown silty sand with crushed stone, occasional cobbles, boulders and blast rock	AU	1			0	49.31
	SS	2	33	50+	1	48.31
1.52						
FILL: Brown sity clay, trace sand, gravel and wood	SS	3	0	10	2	47.31
	SS	4	58	3		
	SS	5	83	18	3	46.31
3.96						
Hard to very stiff, brown **SILTY CLAY**	SS	6	75	19	4	45.31
	SS	7	92	16	5	44.31
	SS	8	92	P		
	SS	9	92	P	6	43.31
	SS	10	96	P	7	42.31
	SS	11	92	P	8	41.31
- stiff and grey by 8.4m depth	SS	12	96	P	9	40.31
	SS	13	100	P		
					10	39.31
					11	38.31
					12	37.31
					13	36.31
	SS	14	96	P	14	35.31
					15	34.31
15.54						
Dynamic Cone Penetration Test commenced at 15.54m depth. Cone pushed to 31.7m depth.					16	33.31
					17	32.31



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

REMARKS

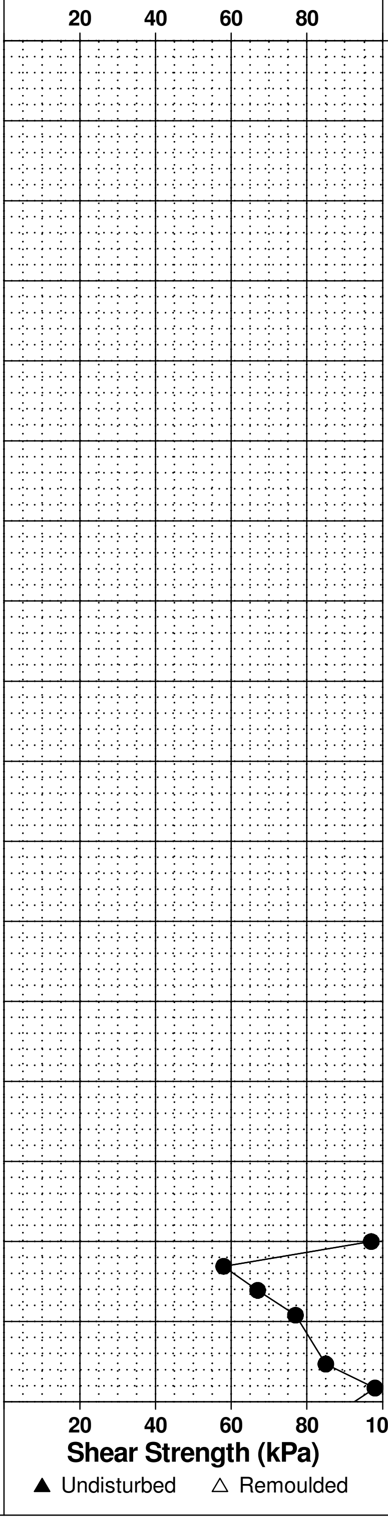
BORINGS BY Track-Mount Power Auger

DATE July 2, 2020

FILE NO. PG5336

HOLE NO. BH 3-20

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content %	Monitoring Well Construction
GROUND SURFACE		TYPE	NUMBER	% RECOVERY	N VALUE or RQD				
						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		



▲ Undisturbed △ Remoulded

SOIL PROFILE AND TEST DATA

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

DATUM Geodetic

REMARKS

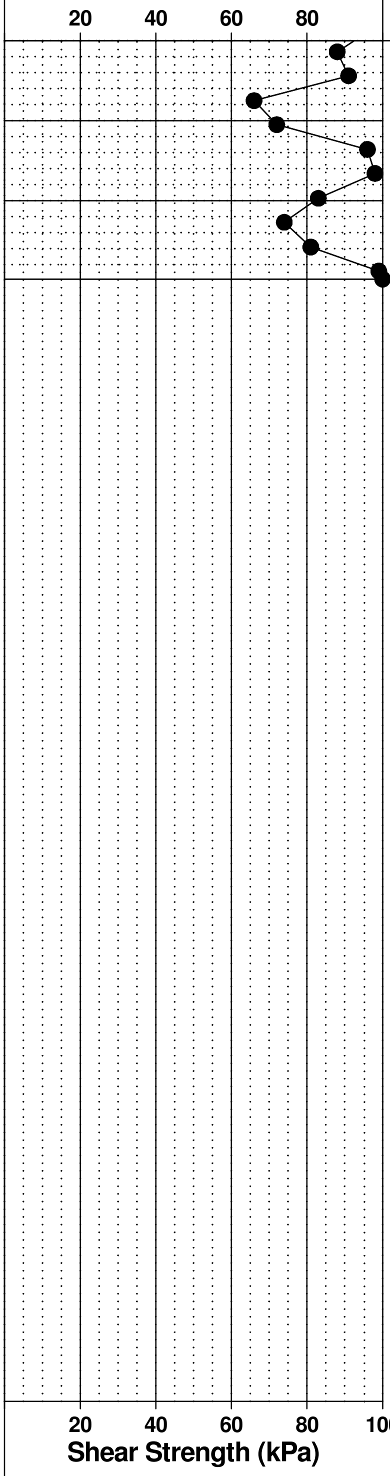
BORINGS BY Track-Mount Power Auger

DATE 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)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content %	Monitoring Well Construction
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)									



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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

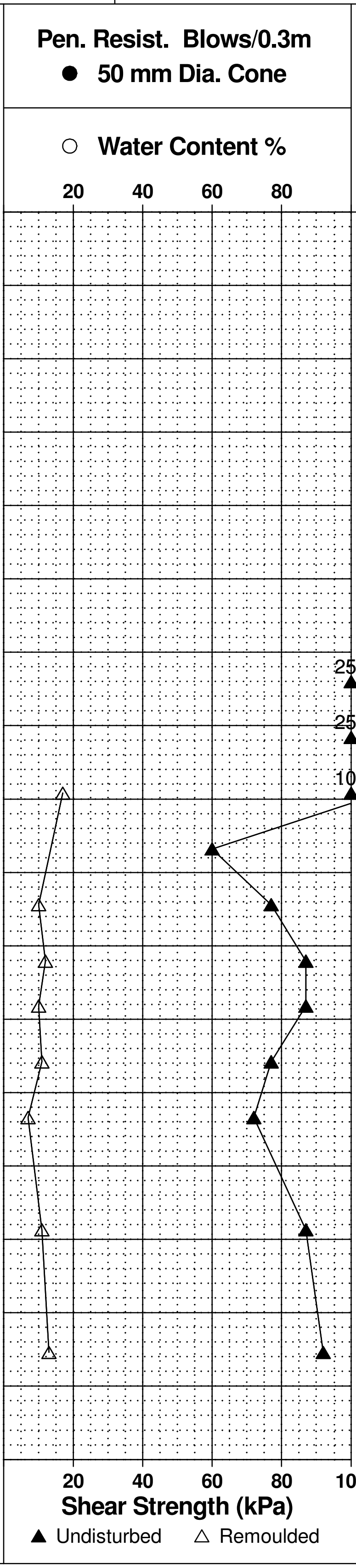
DATE June 30, 2020

FILE NO. PG5336

HOLE NO. BH 4-20

SOIL DESCRIPTION	STRATA PLOT (depth, m)	TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
FILL: Brown silty sand, some gravel, trace organics, cobbles and boulders	0.60	AU	1		
FILL: Brown sand with crushed stone	0.76	SS	2	33	14
FILL: Grey-brown silty clay, trace sand, gravel and topsoil		SS	3	4	9
		SS	4	75	5
		SS	5	29	P
		SS	6	92	7
		SS	7	62	4
- trace wood by 5.2m depth	5.69	SS	8	88	6
Hard to very stiff, brown **SILTY CLAY**		SS	9	100	P
		SS	10	100	P
		SS	11	100	P
- stiff and grey by 8.7m depth		SS	12	100	P
		SS	13	83	P
		SS	14	100	P
	15.85	SS	15	100	P
Dynamic Cone Penetration Test commenced at 15.85m depth. Cone pushed to 41.1m depth.					

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

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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 June 30, 2020

FILE NO. PG5336

HOLE NO. BH 4-20

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content % 20 40 60 80	Monitoring Well Construction
GROUND SURFACE		TYPE	NUMBER	% RECOVERY	N VALUE or RQD				
						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

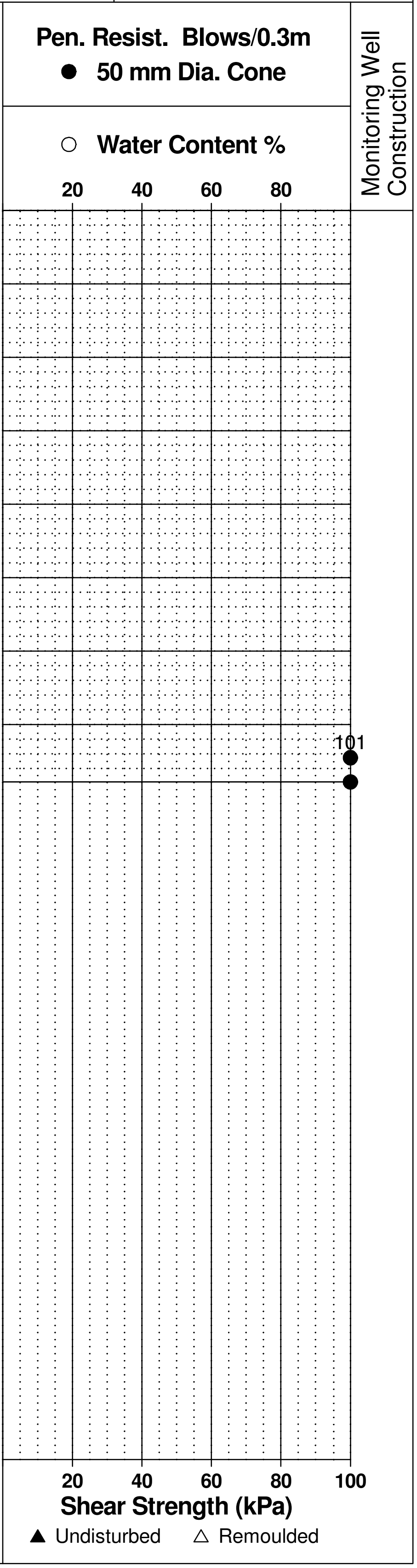
BORINGS BY Track-Mount Power Auger

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.							



SYMBOLS AND TERMS

SOIL DESCRIPTION

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

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

The standard terminology to describe the strength of cohesionless soils is the relative density, usually inferred from the results of the Standard Penetration Test (SPT) 'N' value. The SPT N value is the number of blows of a 63.5 kg hammer, falling 760 mm, required to drive a 51 mm O.D. split spoon sampler 300 mm into the soil after an initial penetration of 150 mm.

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

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

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

SYMBOLS AND TERMS (continued)

SOIL DESCRIPTION (continued)

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

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

ROCK DESCRIPTION

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

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

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

SAMPLE TYPES

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

SYMBOLS AND TERMS (continued)

GRAIN SIZE DISTRIBUTION

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

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

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

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

CONSOLIDATION TEST

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

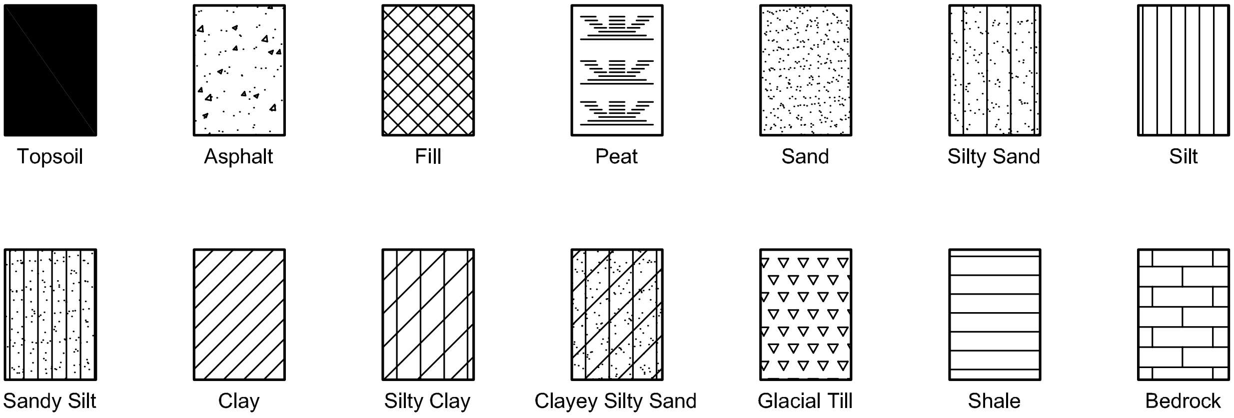
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

PERMEABILITY TEST

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

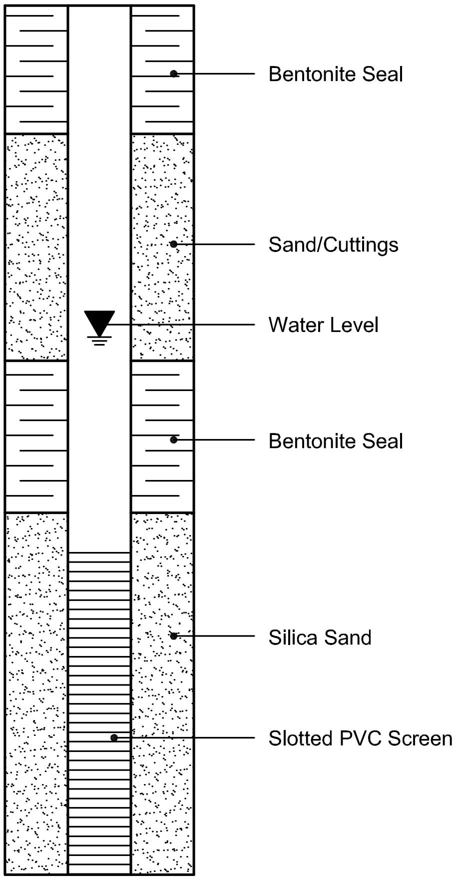
SYMBOLS AND TERMS (continued)

STRATA PLOT

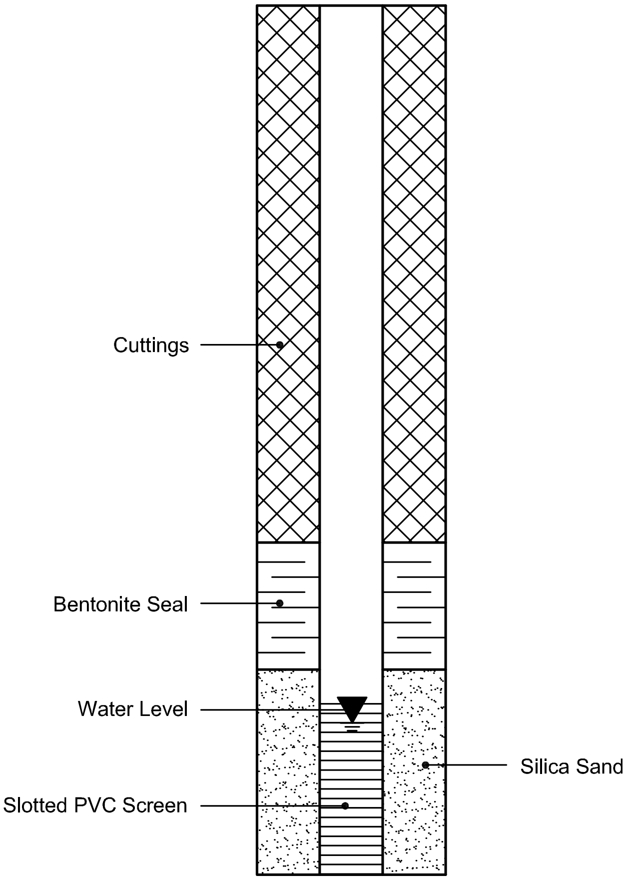


MONITORING WELL AND PIEZOMETER CONSTRUCTION

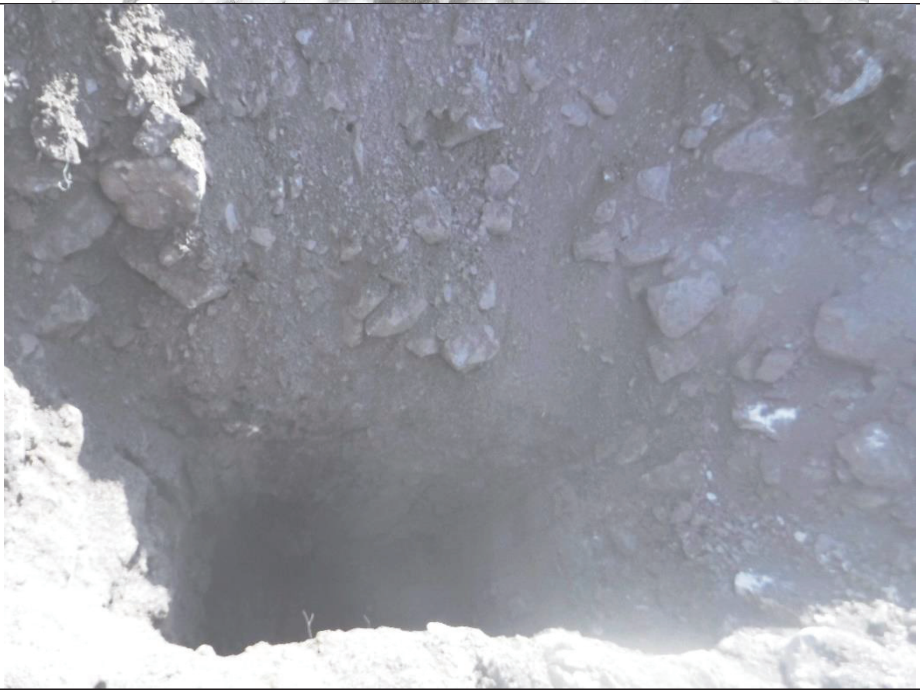
MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Test Pit Number (Elevation)	Depth (metres)	Description		
TP 16-4 (47.3 m)	0.0 – 1.8	Crushed Sand and Gravel with boulders/cobbles, grey, moist (FILL)		
	1.8 – 4.0	Silty Sand and Gravel, some clay to clayey, brown, moist (FILL)		
	4.0 – 6.4	Silty Clay, trace to some gravel, trace roots, grey-brown, moist (FILL)		
	6.4 – 7.3	Organic Soil mixed with roots, black, moist		
	7.3	End of Test Pit		
Sample	Depth	% Gravel	% Sand	% Fines
1	0 – 0.6 m	84	15	1



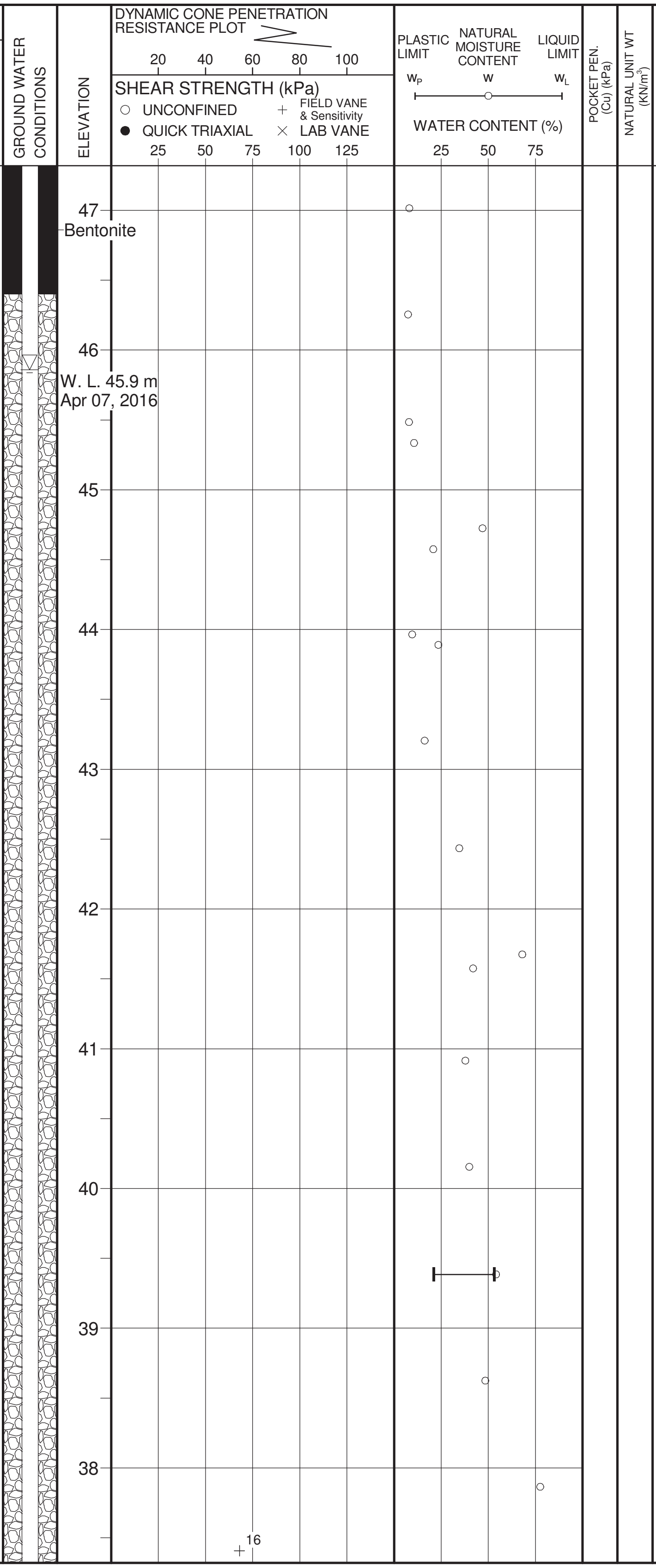
LOG OF BOREHOLE MW16-1

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	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	"N" BLOWS 0.3 m	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
47.3 0.0	**CRUSHED SAND AND GRAVEL** trace to some silt, trace to some clay, grey, wet, compact (FILL)	1	SS	28	
	- loose below 0.75 m	2	SS	5	
45.5 1.8	**CLAYEY SILT** some sand, some gravel, dark brown, moist, loose to compact (FILL)	3A 3B	SS	10	
44.7 2.7	**SILTY SAND** some gravel,brown, moist (FILL)	4 4B	SS	8	19 56 (25)
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	5A 5B	SS	19	
		6	SS	8	
	- very loose below 4.8 m in depth	7	SS	2	
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)	8A 8B	SS	2	
41.2 6.1	**SILTY CLAY** brown-grey, wet, very stiff (WEATHERED CRUST)	9	SS	19	
		10	SS	20	
	- stiff	11	SS	4	0 0 40 60
38.9 8.4	**SILTY CLAY** grey, wet, stiff	12	SS	2	
		13	SS	WH	
			VANE		



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GROUNDWATER ELEVATIONS

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

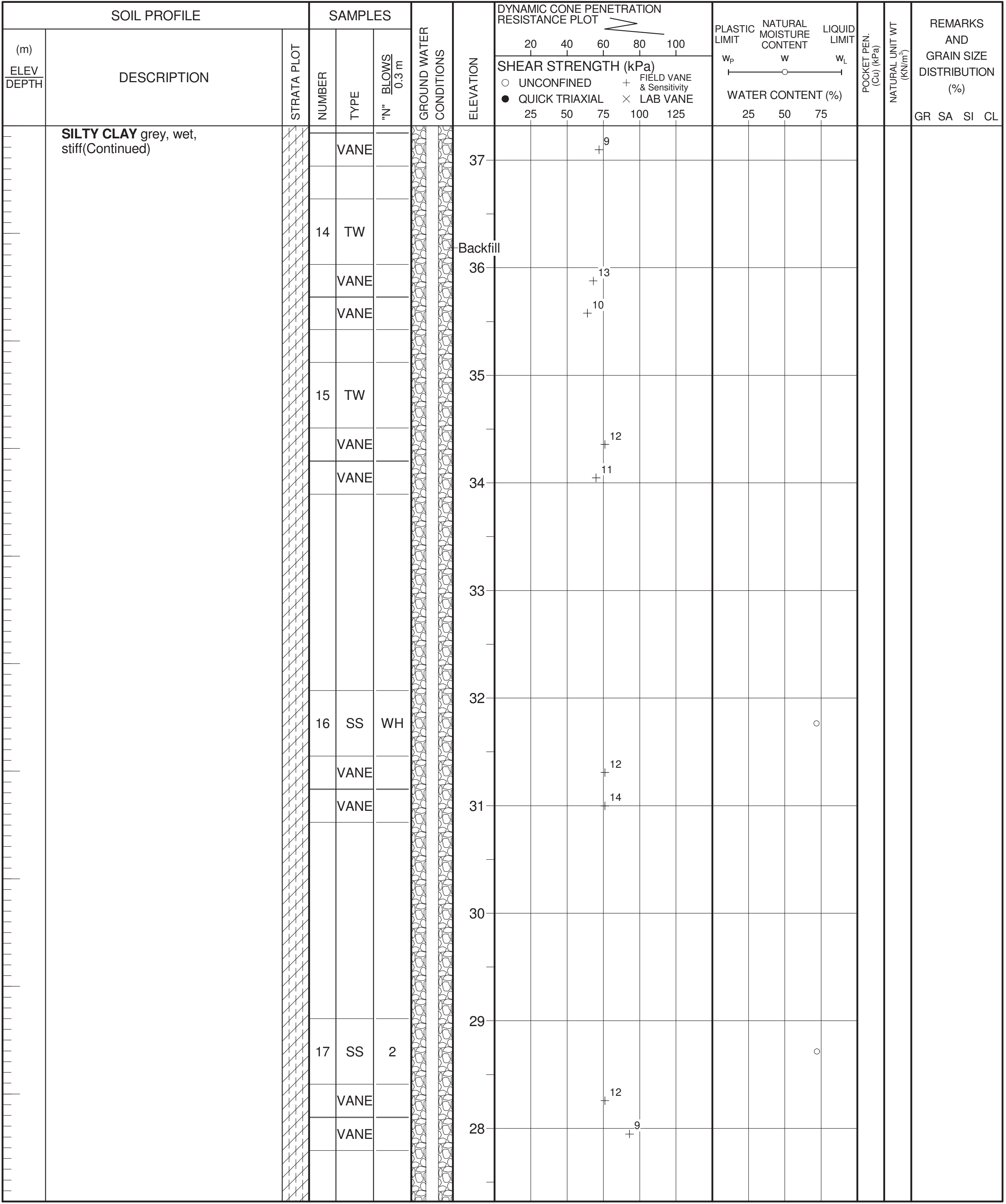
GRAPH NOTES + 3, × 3: Numbers refer to Sensitivity ○ ε=3% Strain at Failure

Sheet No. 1 of 5

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

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
	VANE	
14	TW	
	VANE	
	VANE	
15	TW	
	VANE	
	VANE	
16	SS	WH
	VANE	
	VANE	
17	SS	2
	VANE	
	VANE	

Continued Next Page

GROUNDWATER ELEVATIONS

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

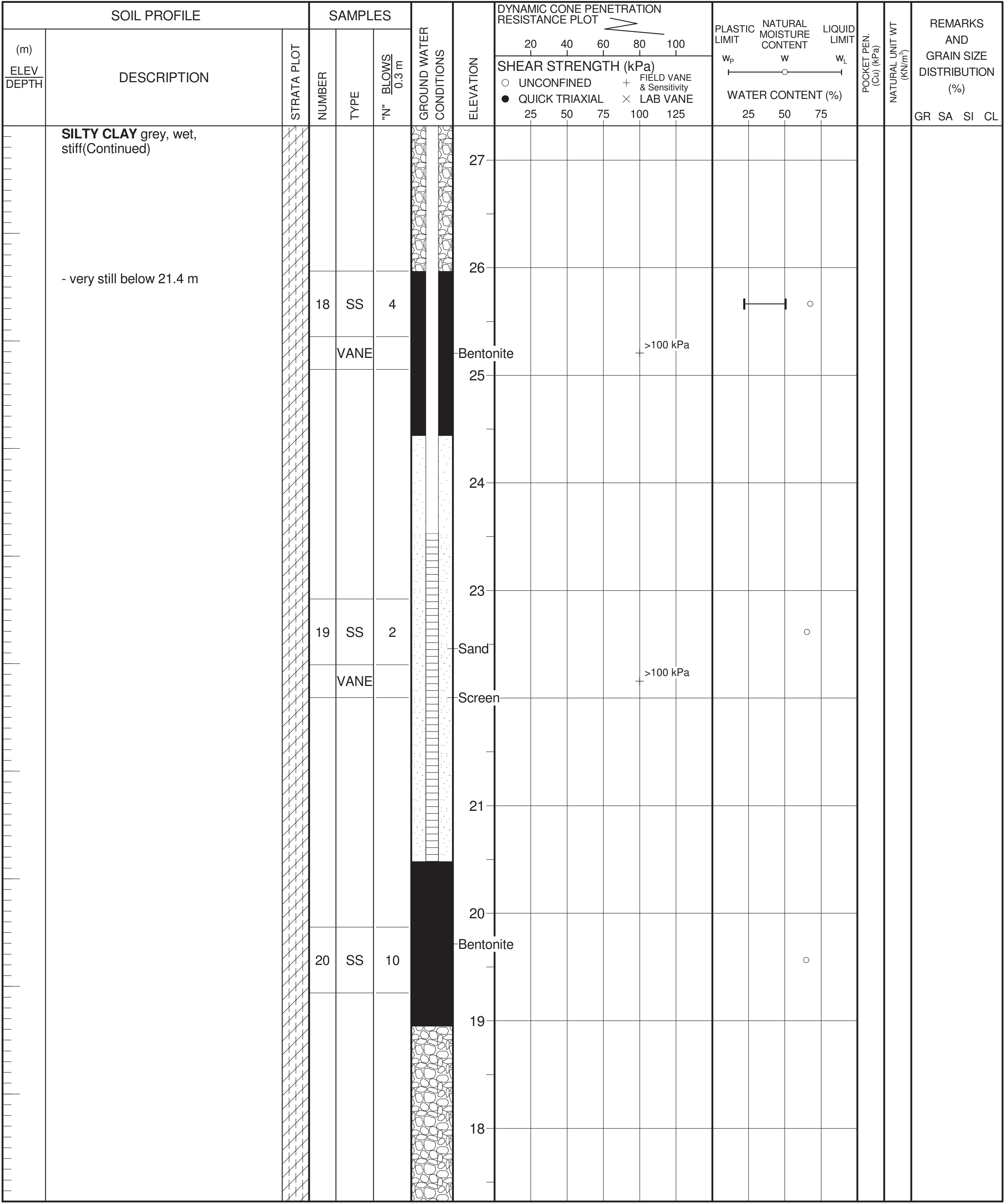
GRAPH NOTES $+^3$, $\times^3$: Numbers refer to Sensitivity ○ ε=3% Strain at Failure

LOG OF BOREHOLE MW16-1

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:



SILTY CLAY grey, wet, stiff(Continued)

- very still below 21.4 m

NUMBER	TYPE	"N" BLOWS/0.3 m
18	SS	4
	VANE	
19	SS	2
	VANE	
20	SS	10

Continued Next Page

GROUNDWATER ELEVATIONS

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

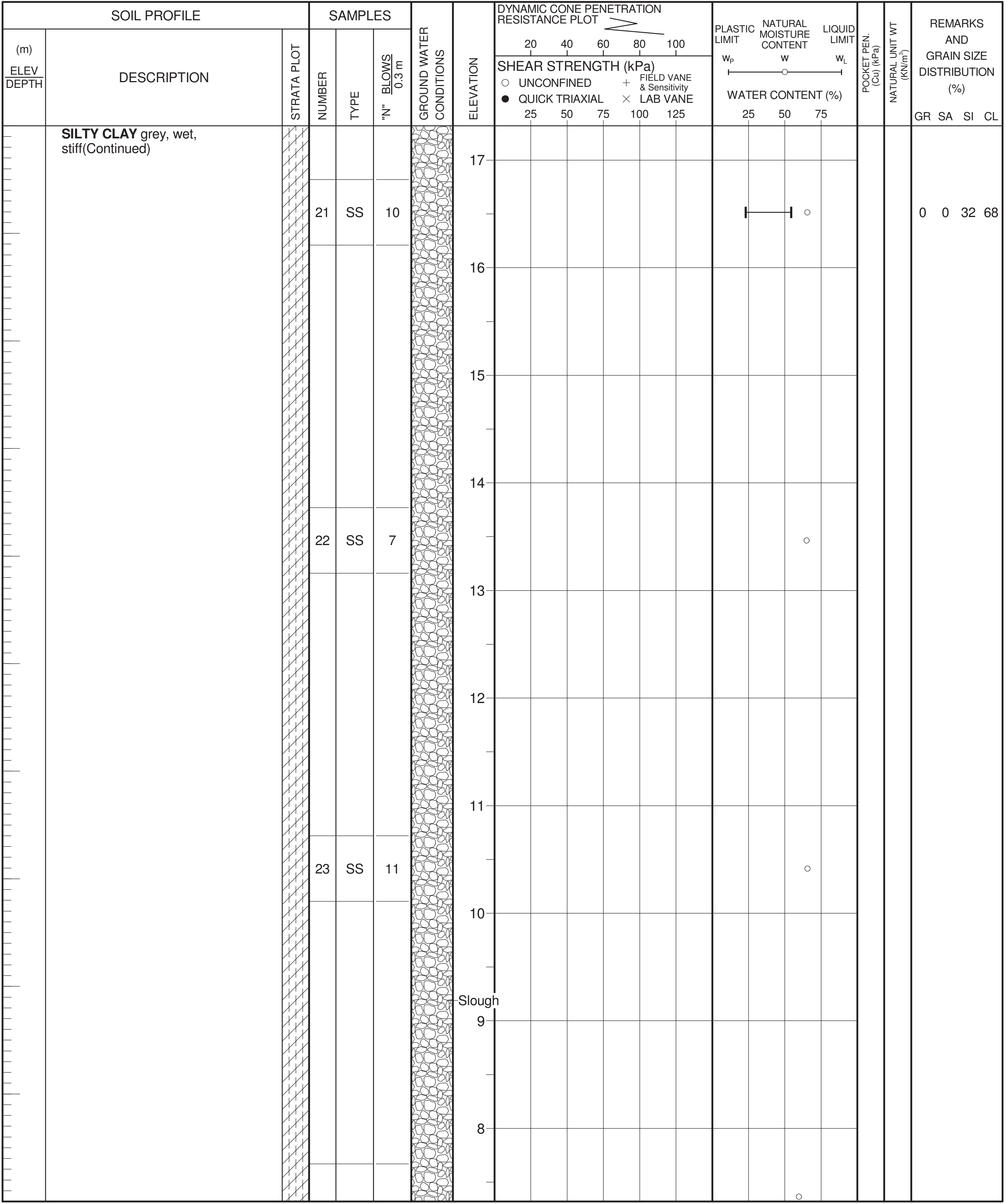
GRAPH NOTES + 3 , × 3 : Numbers refer to Sensitivity   ○ ε=3% Strain at Failure

LOG OF BOREHOLE MW16-1

Project: Geotechnical Investigation
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:



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

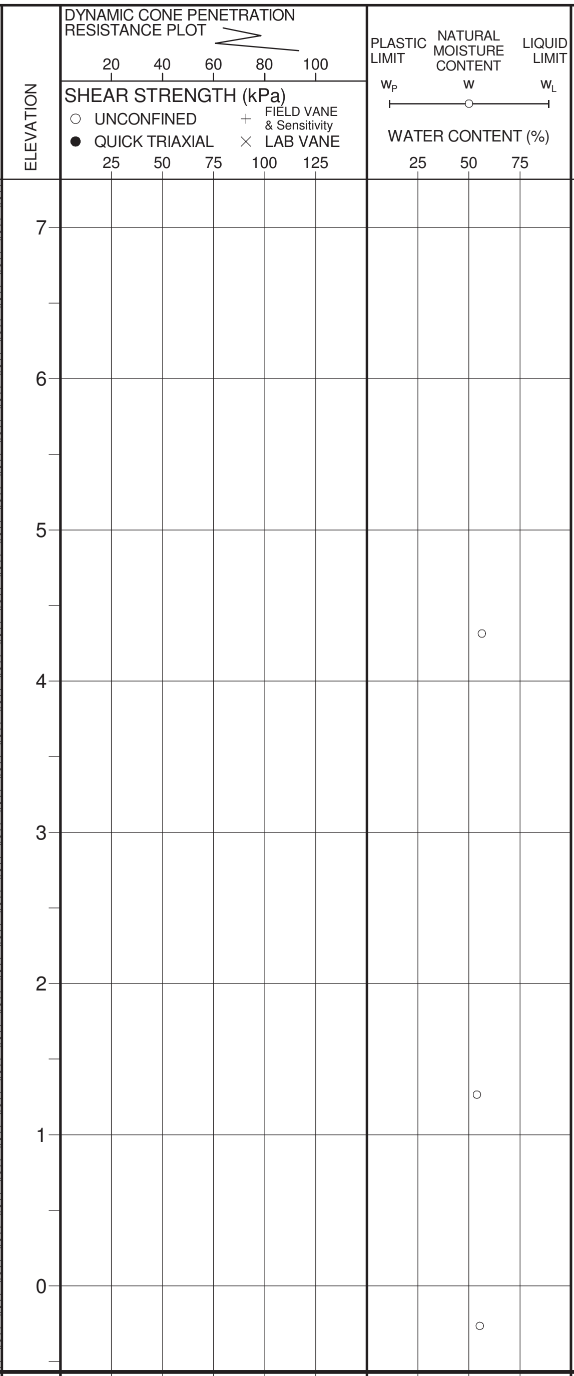
LOG OF BOREHOLE MW16-1

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			SAMPLES							
(m) ELEV DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" BLOWS 0.3 m	GROUND WATER CONDITIONS	POCKET PEN. (Cu) (kPa)	NATURAL UNIT WT (KN/m^3)	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	SILTY CLAY grey, wet, stiff(Continued)		24	SS	13				
			25	SS	12				
			26	SS	14				
-0.6			27	SS	14				
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
4/7/2016	1.5 m

GROUNDWATER ELEVATIONS

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

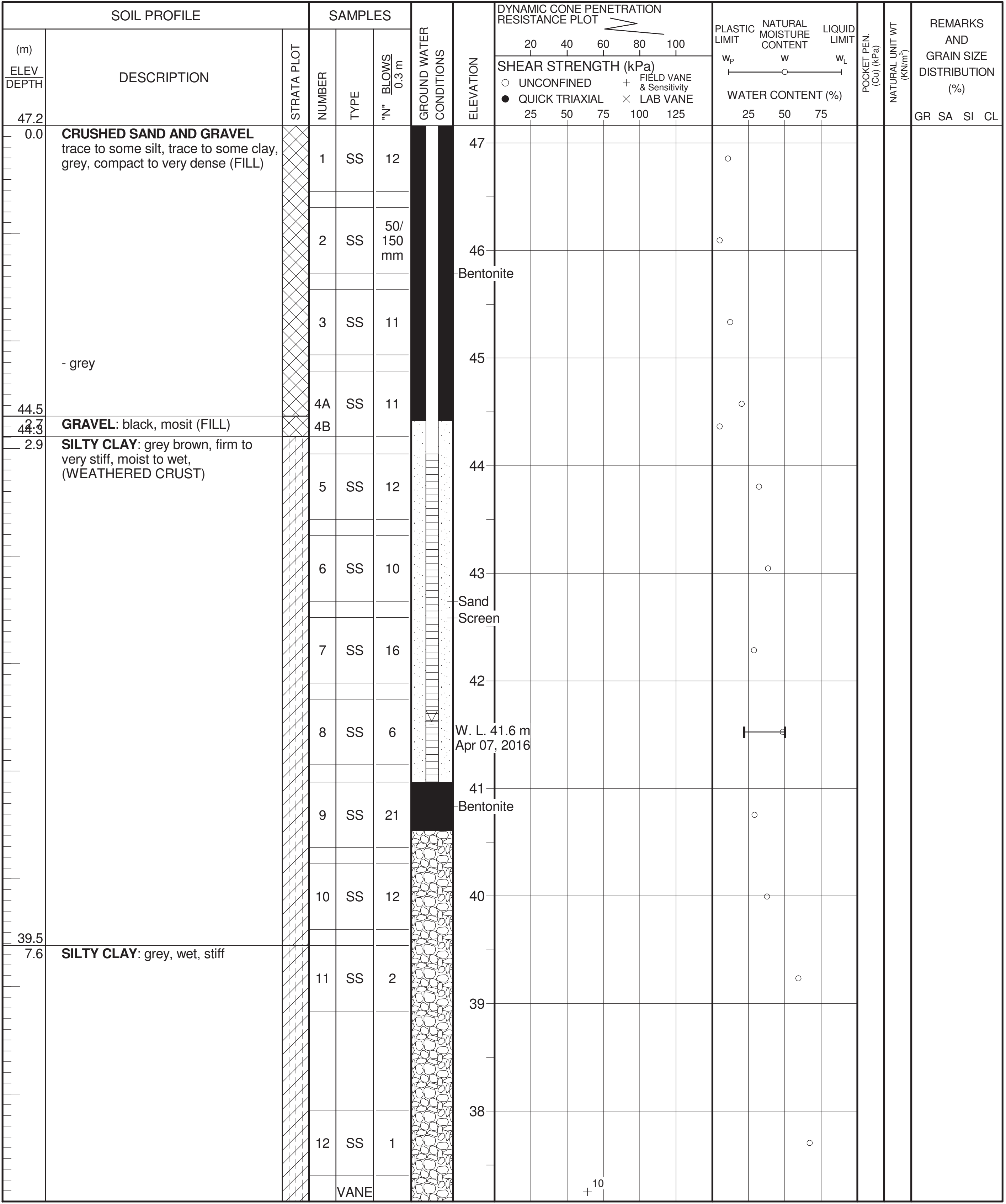
GRAPH NOTES + 3, × 3: Numbers refer to Sensitivity ○ ε=3% Strain at Failure

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:



SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS
(m) ELEV DEPTH	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	Bentonite
		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	Sand Screen
		7	SS	16	
		8	SS	6	W. L. 41.6 m Apr 07, 2016
		9	SS	21	Bentonite
		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 ▽ ▼

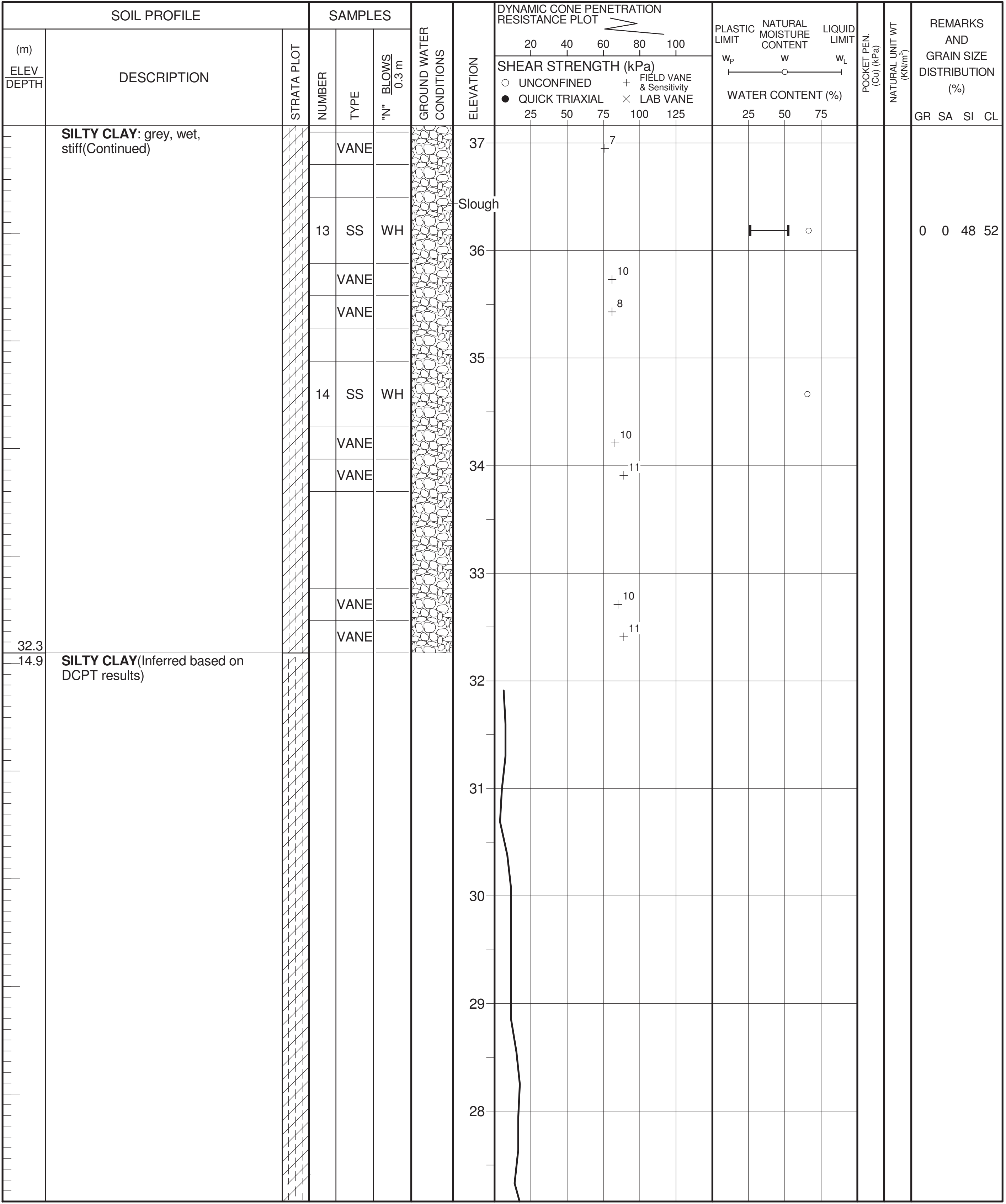
GRAPH NOTES $+^3$, $\times^3$: Numbers refer to Sensitivity ○ ε=3% Strain at Failure

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:



SILTY CLAY: grey, wet, stiff(Continued)

32.3
14.9 **SILTY CLAY**(Inferred based on DCPT results)

NUMBER	TYPE	"N" BLOWS 0.3 m	FIELD VANE & Sensitivity	GR	SA	SI	CL
	VANE		7				
13	SS	WH		0	0	48	52
	VANE		10				
	VANE		8				
14	SS	WH					
	VANE		10				
	VANE		11				
	VANE		10				
	VANE		11				

Continued Next Page

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

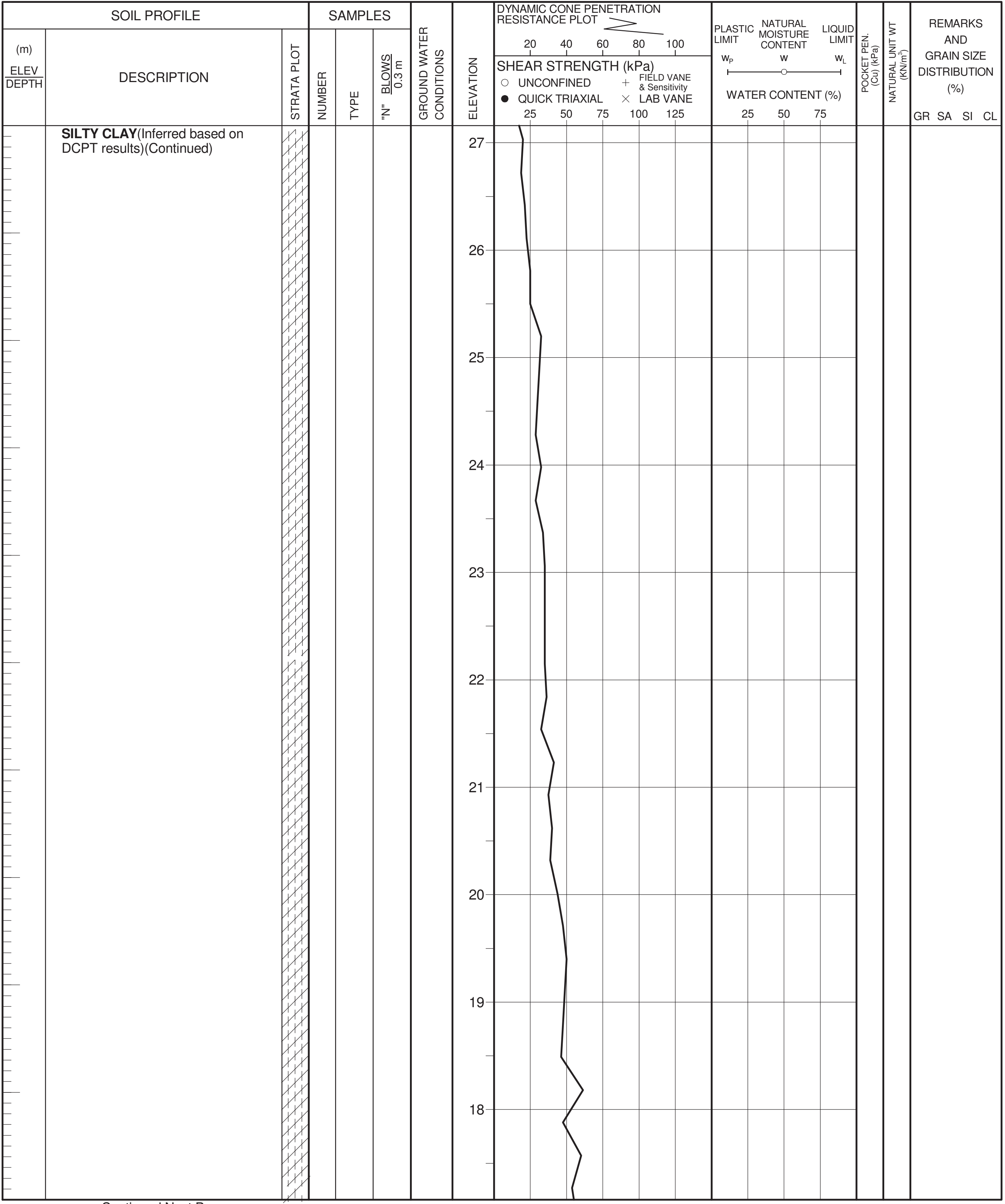
GRAPH NOTES
+ 3, × 3: Numbers refer to Sensitivity
○ ε=3% Strain at Failure

LOG OF BOREHOLE MW16-2

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:



SILTY CLAY(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

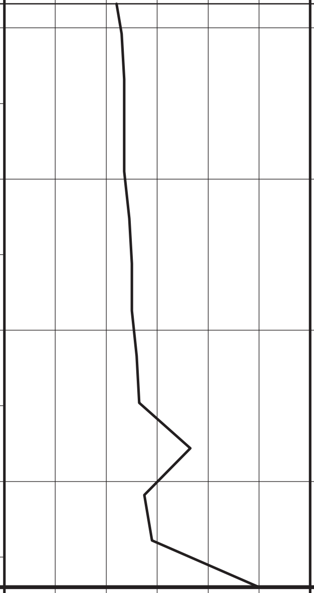
LOG OF BOREHOLE MW16-2

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					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	GROUND WATER CONDITIONS	ELEVATION					
	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

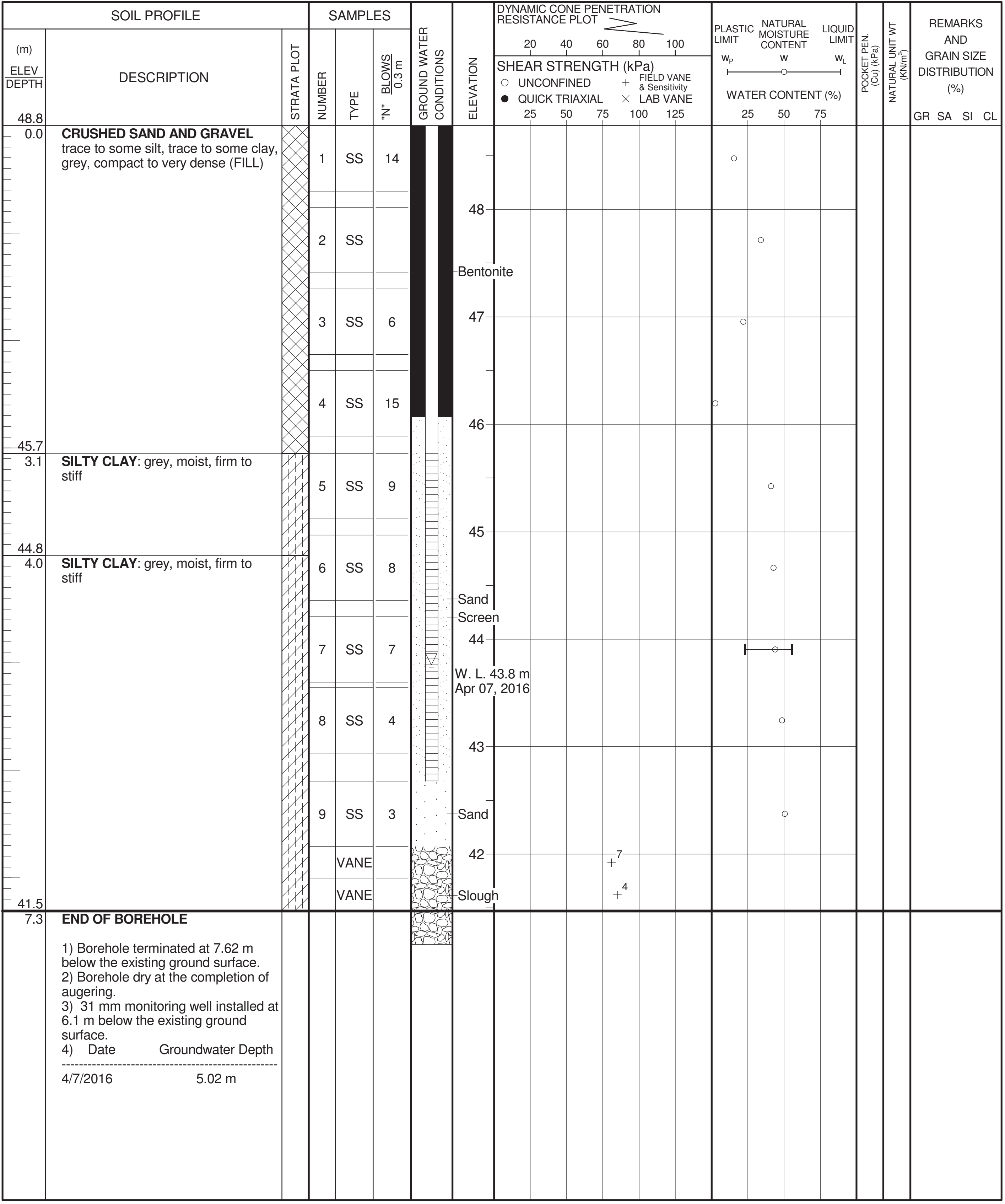
Shallow/ Single Installation ▽ ▼ Deep/Dual Installation ▼ ▽

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

WSP

LOG OF BOREHOLE MW16-3

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: 3/22/2016
Datum: Approximate	Borehole Diameter: 203 mm	Supervisor:
BH Location: See borehole location plan N 462249 E 5038342	Core Diameter:	Reviewer:



(m) ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" BLOWS 0.3 m
48.8 0.0	**CRUSHED SAND AND GRAVEL** trace to some silt, trace to some clay, grey, compact to very dense (FILL)	1	SS	14
		2	SS	
		3	SS	6
		4	SS	15
45.7 3.1	**SILTY CLAY**: grey, moist, firm to stiff	5	SS	9
44.8 4.0	**SILTY CLAY**: grey, moist, firm to stiff	6	SS	8
		7	SS	7
		8	SS	4
		9	SS	3
			VANE	
			VANE	
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

GROUNDWATER ELEVATIONS

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

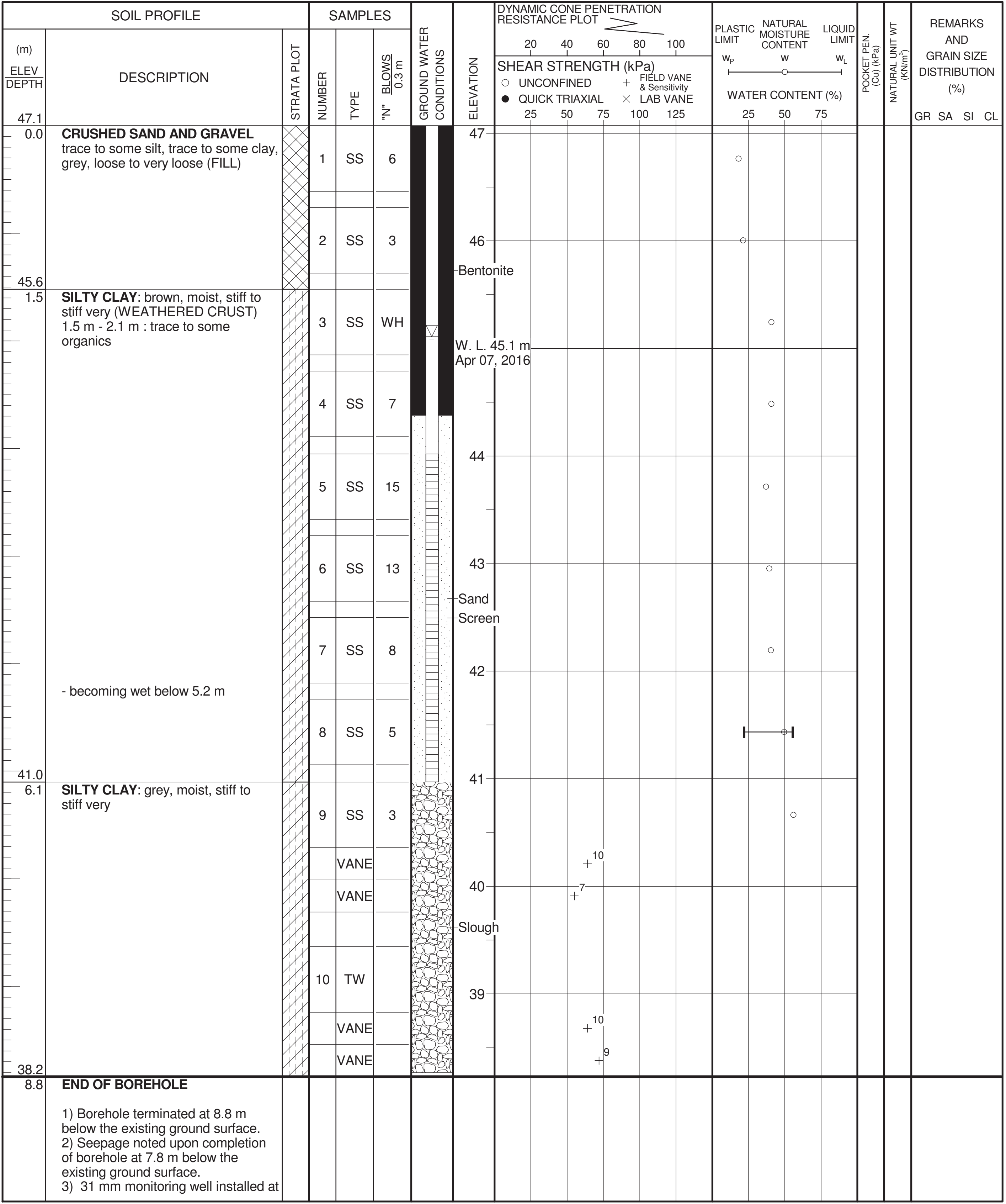
GRAPH NOTES $+^3, \times^3$: Numbers refer to Sensitivity ○ ε=3% Strain at Failure

LOG OF BOREHOLE MW16-4

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:



(m) ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" BLOWS 0.3 m
47.1				
0.0	**CRUSHED SAND AND GRAVEL** trace to some silt, trace to some clay, grey, loose to very loose (FILL)	1	SS	6
		2	SS	3
45.6 1.5	**SILTY CLAY**: brown, moist, stiff to stiff very (WEATHERED CRUST) 1.5 m - 2.1 m : trace to some organics	3	SS	WH
		4	SS	7
		5	SS	15
		6	SS	13
		7	SS	8
	- becoming wet below 5.2 m	8	SS	5
41.0 6.1	**SILTY CLAY**: grey, moist, stiff to stiff very	9	SS	3
			VANE	
			VANE	
		10	TW	
			VANE	
			VANE	
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			

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 MW16-4

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

Sheet No. 2 of 2

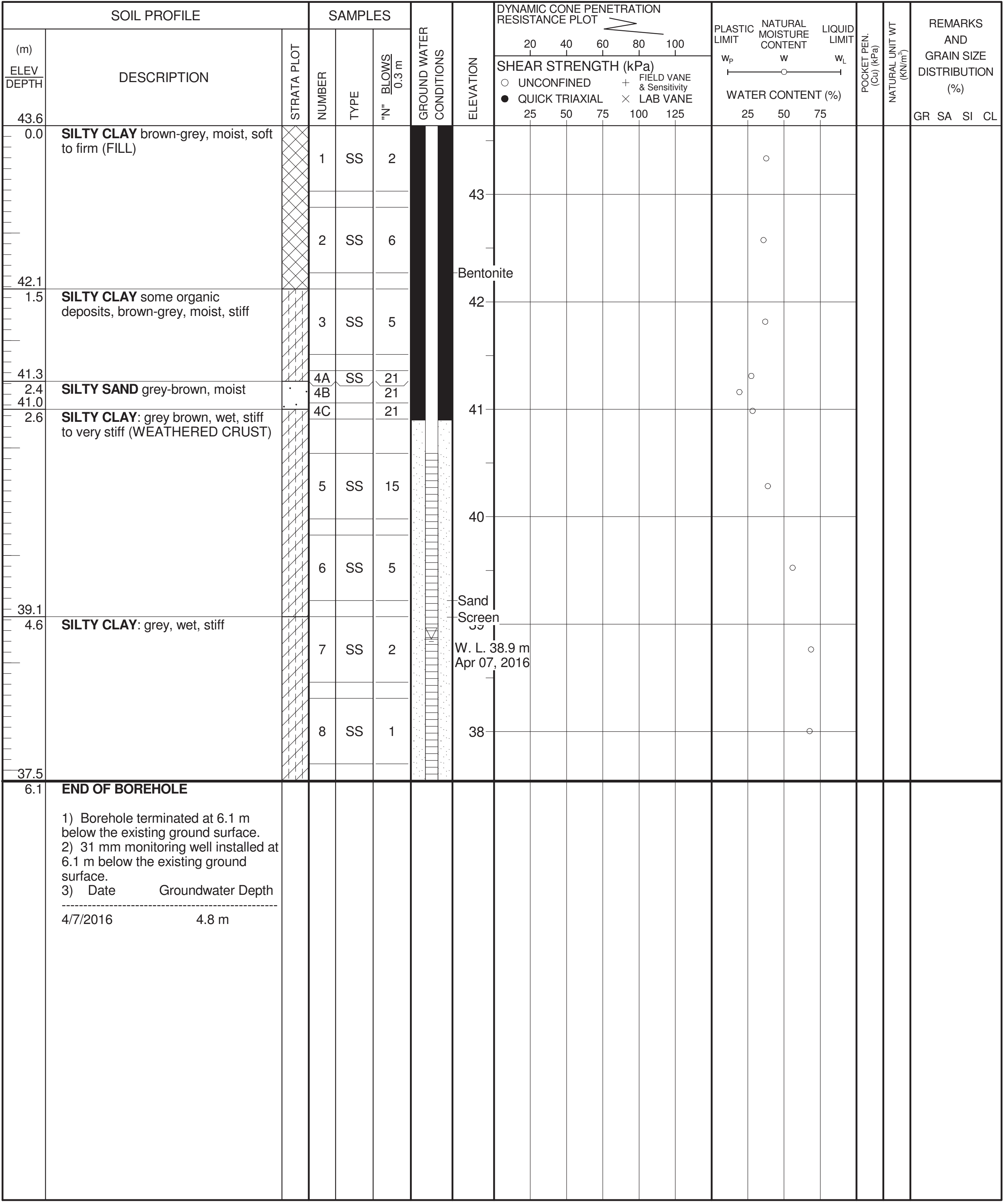
WSP SOIL LOG - OTTAWA GEOTECHNICAL BH LOGS - 1009 TRIM ROAD, OTTAWA, ONTARIO.GPJ SPL.GDT 5/5/16

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:



SOIL PROFILE		SAMPLES		
(m) ELEV DEPTH	DESCRIPTION	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	SS	21
		4B		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

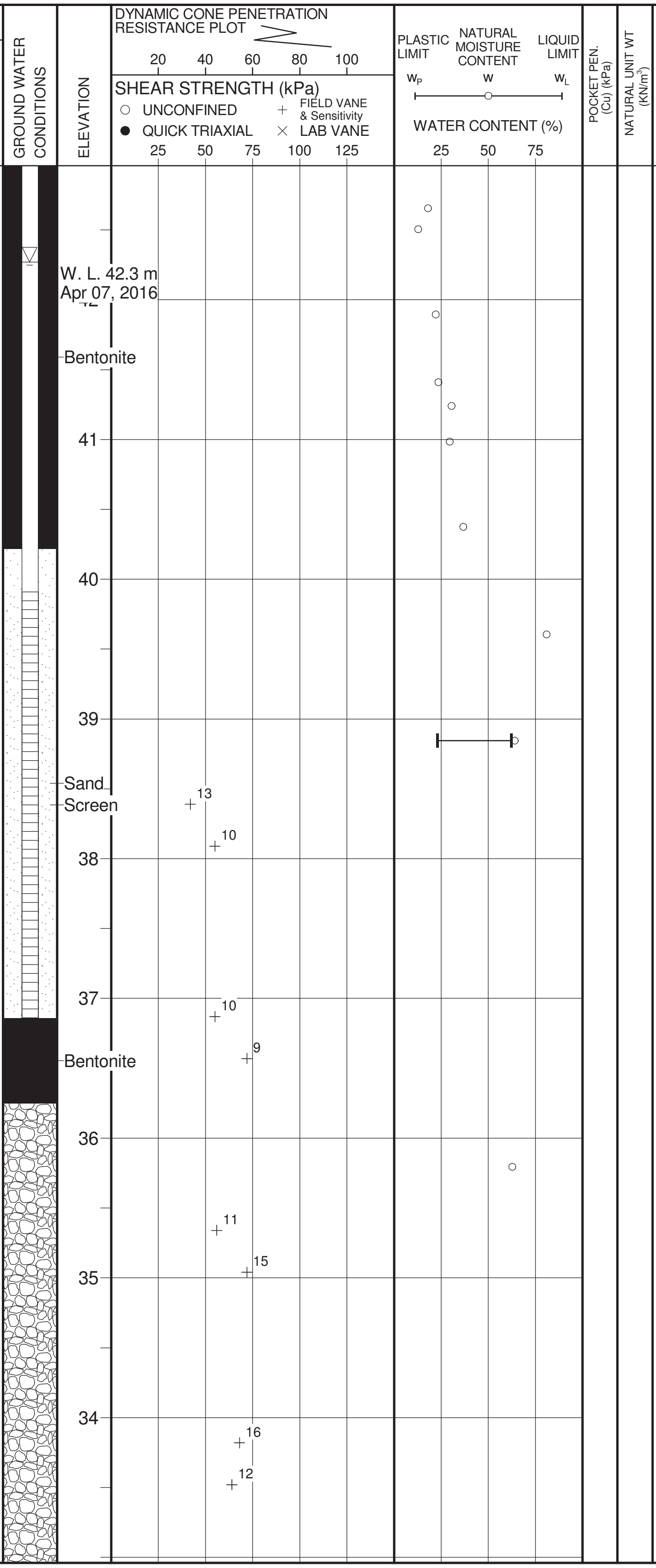
LOG OF BOREHOLE MW16-6

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					



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

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

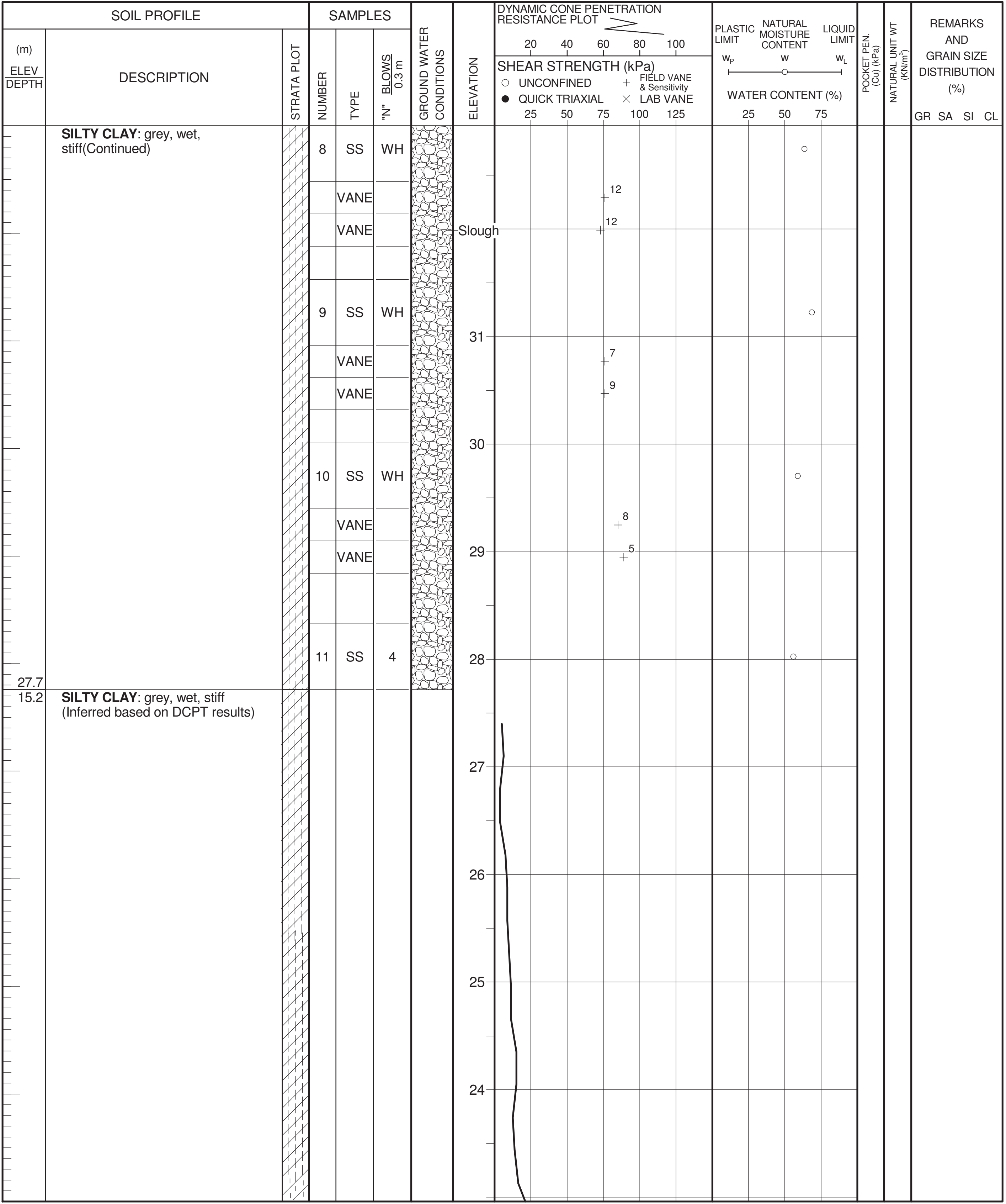
WSP SOIL LOG - OTTAWA GEOTECHNICAL BH LOGS - 1009 TRIM ROAD, OTTAWA, ONTARIO.GPJ SPL.GDT 5/5/16

LOG OF BOREHOLE MW16-6

Project: Geotechnical Investigation
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:



ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" BLOWS / 0.3 m
	SILTY CLAY: grey, wet, stiff(Continued)	8	SS	WH
			VANE	
			VANE	
		9	SS	WH
			VANE	
			VANE	
		10	SS	WH
			VANE	
			VANE	
		11	SS	4
27.7 15.2	**SILTY CLAY**: grey, wet, stiff (Inferred based on DCPT results)			

Continued Next Page

GROUNDWATER ELEVATIONS

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

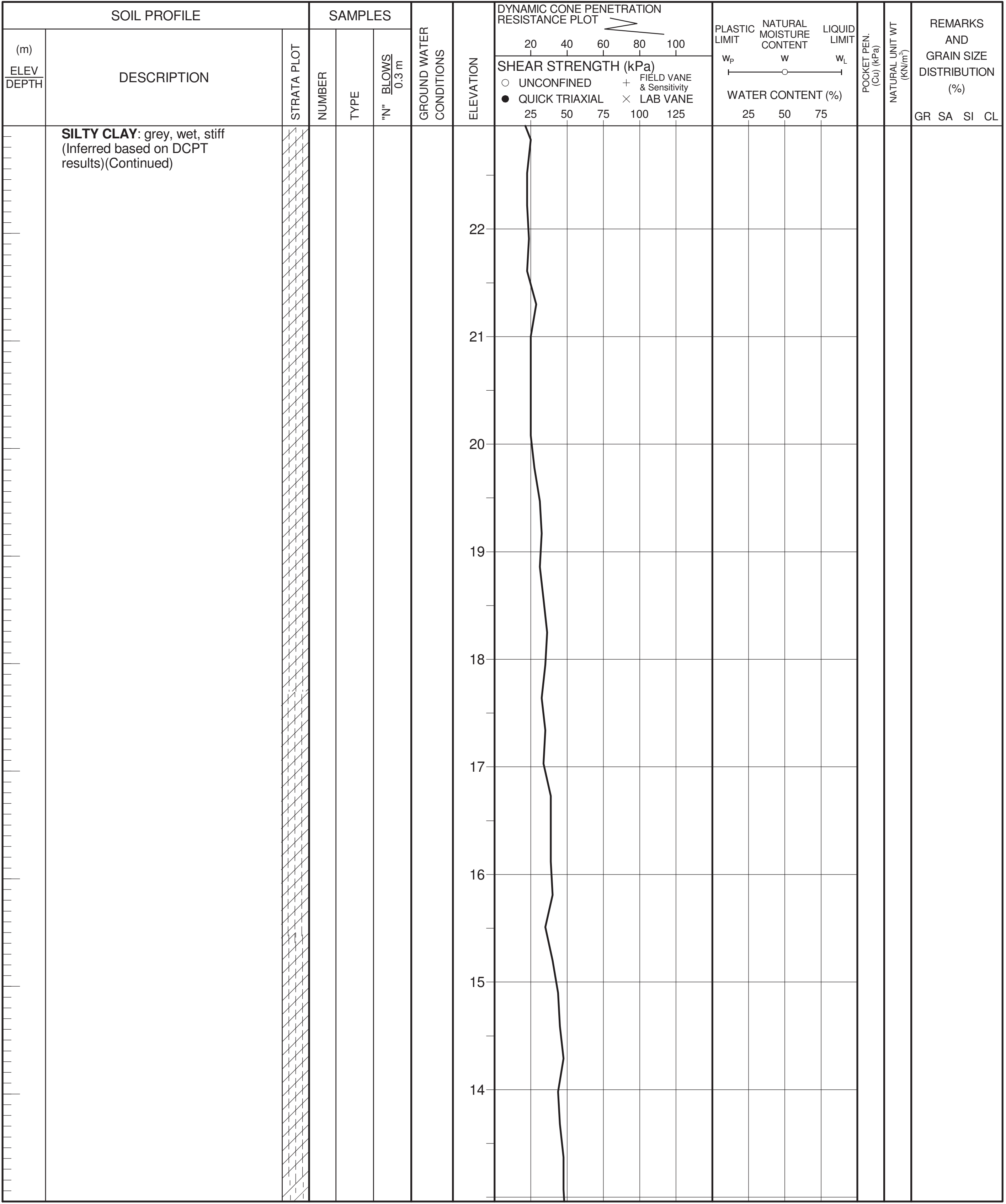
GRAPH NOTES + 3, × 3: Numbers refer to Sensitivity ○ ε=3% Strain at Failure

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:



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

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

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

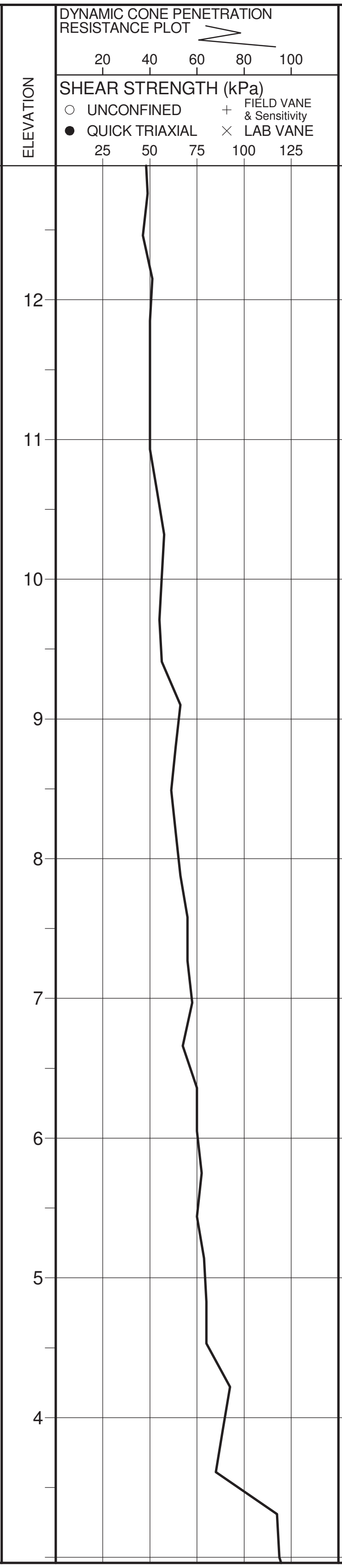
LOG OF BOREHOLE MW16-6

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 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
	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, × 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			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							
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

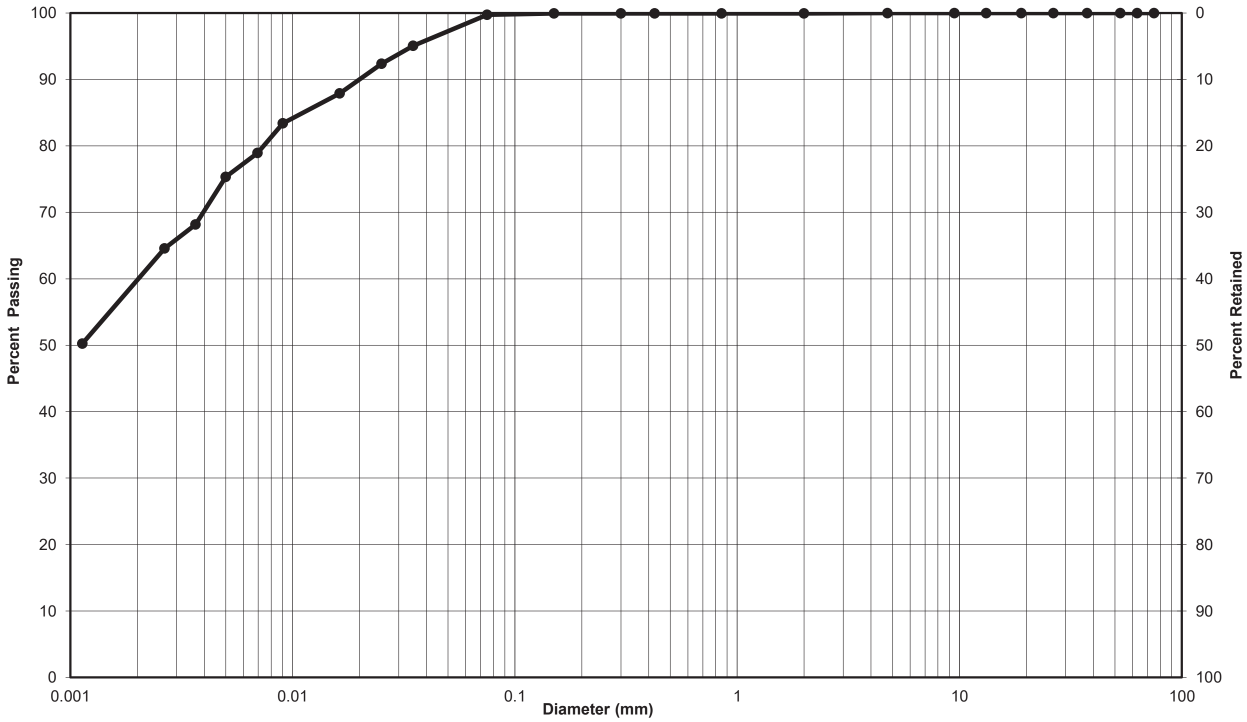
Particle-Size Analysis of Soils
(ASTM D422)

Client: Grandmaitre Estates
Lab no.: OL7-2

Project/Site: 1009 Trim Road, Ottawa
Project no.: 161-03361-00

Borehole no.: 16-1
Sample no.: SS11

Depth: 7.6-8.2m
Location: -



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<tr><td rowspan="2">Clay & Silt</td><td colspan="3">Sand</td><td colspan="2">Gravel</td></tr>
<tr><td>Fine</td><td>Medium</td><td>Coarse</td><td>Fine</td><td>Coarse</td></tr>
<tr><td colspan="6">Unified Soil Classification System</td></tr>
</table>

Percent Retained	Gravel	Sand	Clay & Silt	Silt	Clay
	0	0	100	40	60

Remarks:

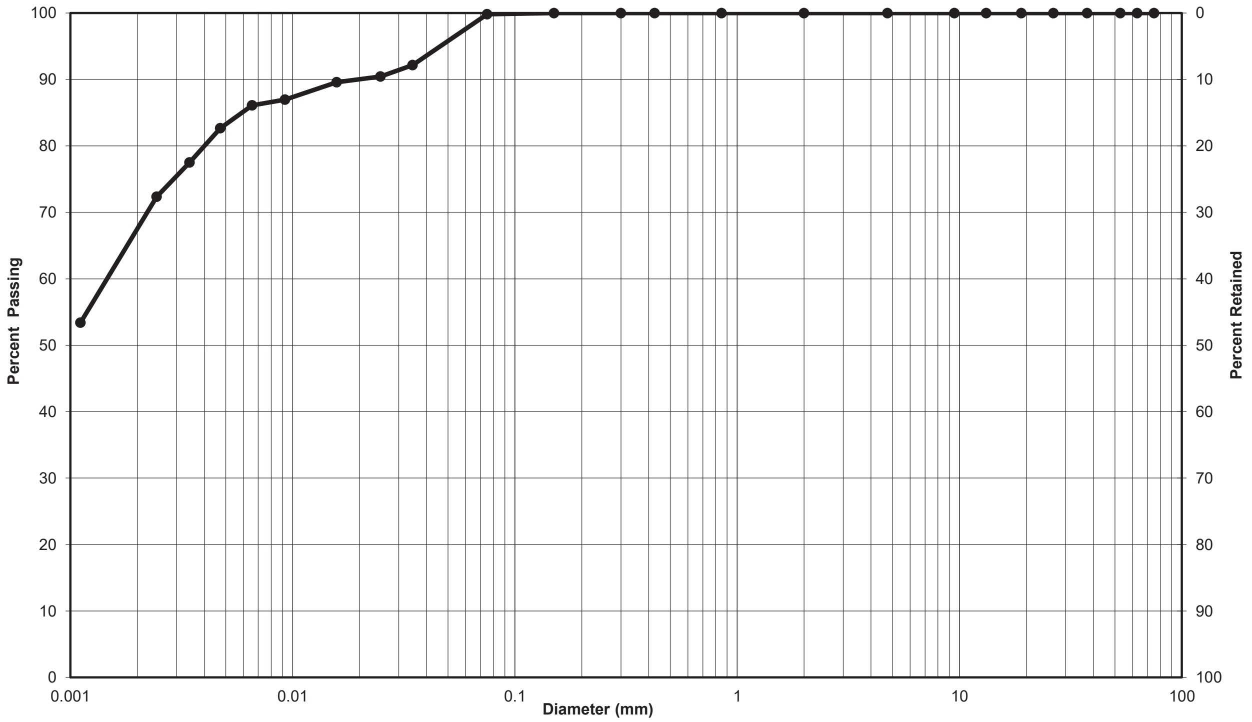
Performed by: S.Wheeler
Date: April 25, 2016

Verified by: N.Krebs
Date: May 3, 2016

Particle-Size Analysis of Soils
(ASTM D422)

Client:	Grandmaitre Estates	**Lab no.:**	OL7-4
Project/Site:	1009 Trim Road, Ottawa	**Project no.:**	161-03361-00

Borehole no.:	16-1	Sample no.:	SS21
Depth:	30.5-31.1m	Location:	-



Clay & Silt	Sand			Gravel	
	Fine	Medium	Coarse	Fine	Coarse
Unified Soil Classification System					

Percent Retained	**Gravel**	**Sand**	**Clay & Silt**	**Silt**	**Clay**
	0	0	100	32	68

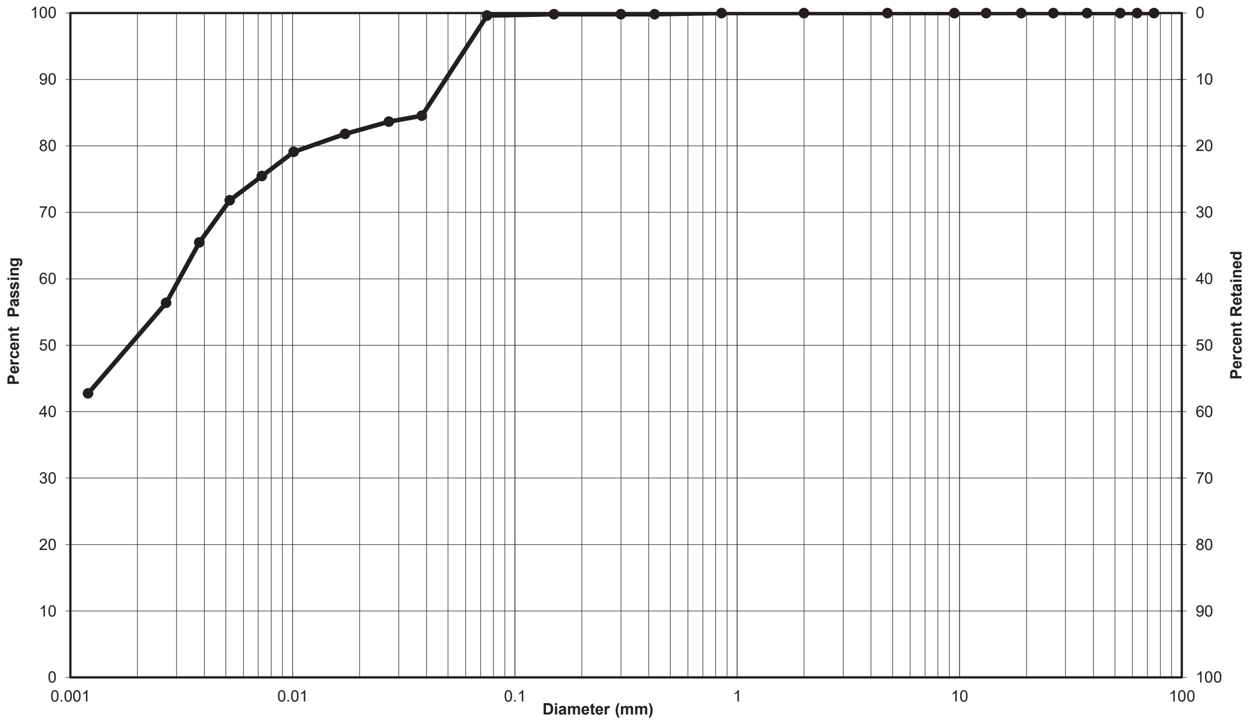
Remarks:

Performed by:	S.Wheeler	**Date:**	April 25, 2016
Verified by:	N.Krebs	**Date:**	May 3, 2016

Particle-Size Analysis of Soils
(ASTM D422)

Client:	Grandmaitre Estates	**Lab no.:**	OL7-7
Project/Site:	1009 Trim Road, Ottawa	**Project no.:**	161-03361-00

Borehole no.:	16-2	Sample no.:	SS13
Depth:	10.7-11.3m	Location:	-



Clay & Silt	Sand			Gravel	
	Fine	Medium	Coarse	Fine	Coarse
Unified Soil Classification System					

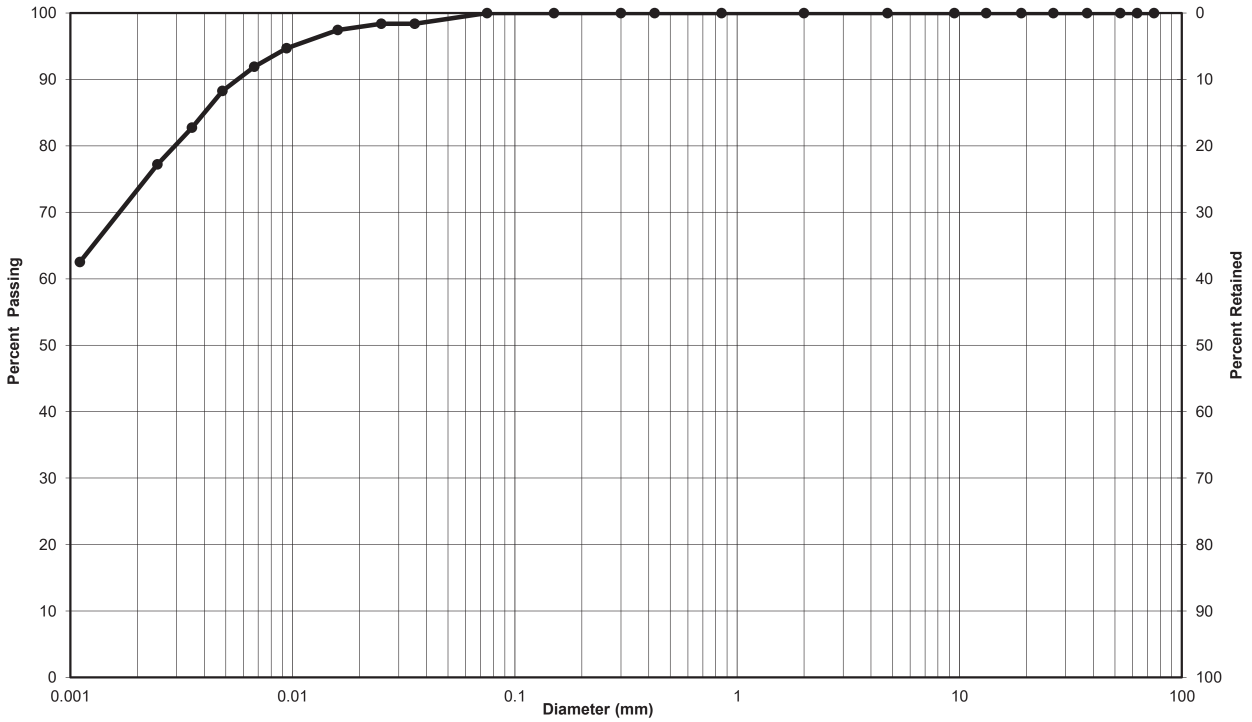
Percent Retained	Gravel	Sand	Clay & Silt	Silt	Clay
	0	0	100	48	52

Remarks:

Performed by:	D.Robertson	**Date:**	April 20, 2016
Verified by:	N.Krebs	**Date:**	May 3, 2016

Particle-Size Analysis of Soils
(ASTM D422)

Client:	Grandmaitre Estates	**Lab no.:**	OL7-12
Project/Site:	1009 Trim Road, Ottawa	**Project no.:**	161-03361-00
Borehole no.:	16-6	Sample no.:	SS6
Depth:	3.8-4.4m	Location:	-



<table>
<tr><td rowspan="2">Clay & Silt</td><td colspan="3">Sand</td><td colspan="2">Gravel</td></tr>
<tr><td>Fine</td><td>Medium</td><td>Coarse</td><td>Fine</td><td>Coarse</td></tr>
<tr><td colspan="6">Unified Soil Classification System</td></tr>
</table>

Percent Retained	Gravel	Sand	Clay & Silt	Silt	Clay
	0	0	100	26	74

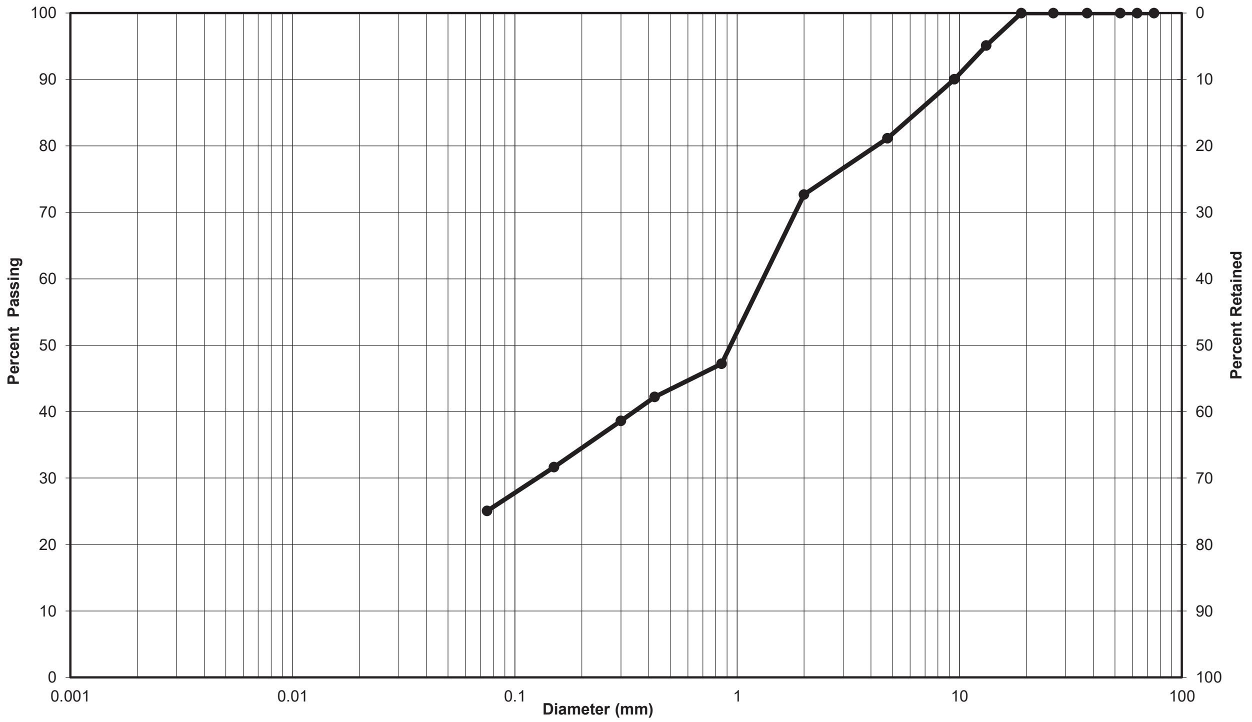
Remarks:

Performed by:	D.Robertson	**Date:**	April 29, 2016
Verified by:	N.Krebs	**Date:**	May 3, 2016

Particle-Size Analysis of Soils
(ASTM D422)

Client:	Grandmaitre Estates	**Lab no.:**	OL7-1
Project/Site:	1009 Trim Road, Ottawa	**Project no.:**	161-03361-00

Borehole no.:	16-1	Sample no.:	SS4B
Depth:	2.3-2.9m	Location:	-



<table>
<tr><td rowspan="2">Clay & Silt</td><td colspan="3">Sand</td><td colspan="2">Gravel</td></tr>
<tr><td>Fine</td><td>Medium</td><td>Coarse</td><td>Fine</td><td>Coarse</td></tr>
<tr><td colspan="6">Unified Soil Classification System</td></tr>
</table>

Percent Retained	Gravel	Sand	Clay & Silt	Silt	Clay
	19	56	25	-	-

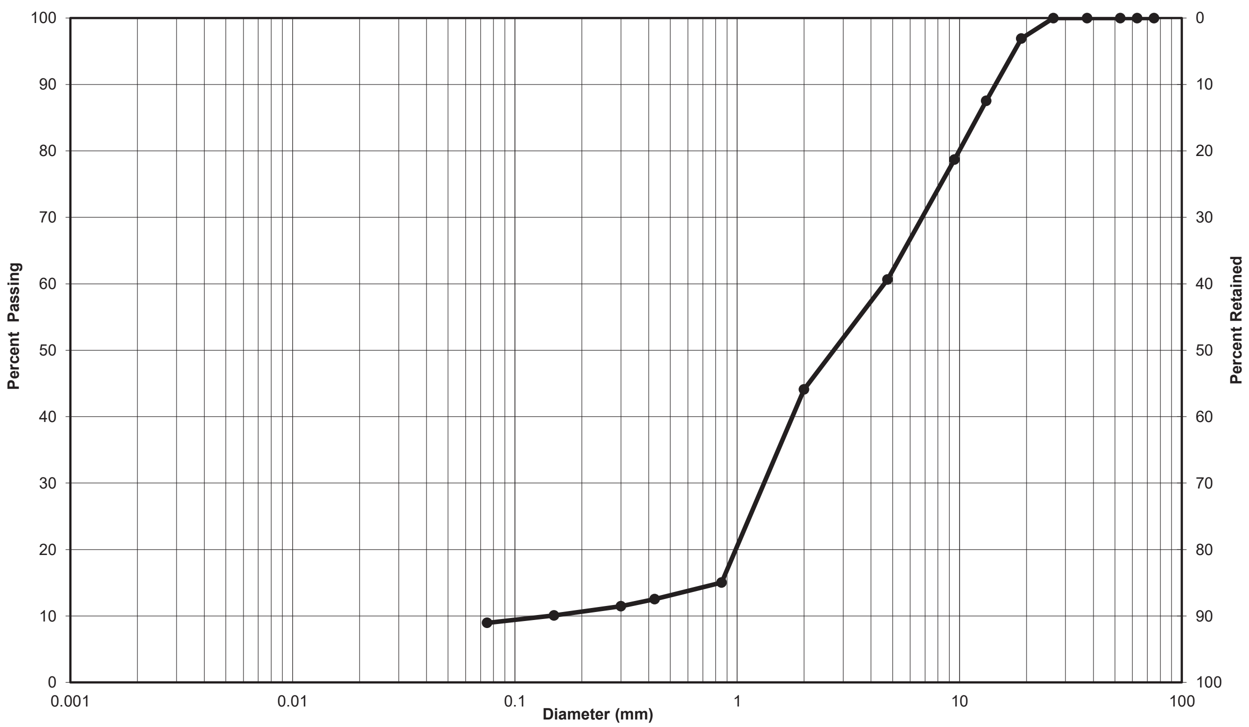
Remarks:

Performed by:	S.Wheeler	**Date:**	April 25, 2016
Verified by:	N.Krebs	**Date:**	May 3, 2016

Particle-Size Analysis of Soils
(ASTM D422)

Client: Grandmaitre Estates
Lab no.: OL7-13
Project/Site: 1009 Trim Road, Ottawa
Project no.: 161-03361-00

Borehole no.: 16-6
Sample no.: SS1B
Depth: 0-0.6m
Location: -



Clay & Silt	Sand			Gravel	
	Fine	Medium	Coarse	Fine	Coarse
Unified Soil Classification System					

Percent Retained	Gravel	Sand	Clay & Silt	Silt	Clay
	39	52	9	-	-

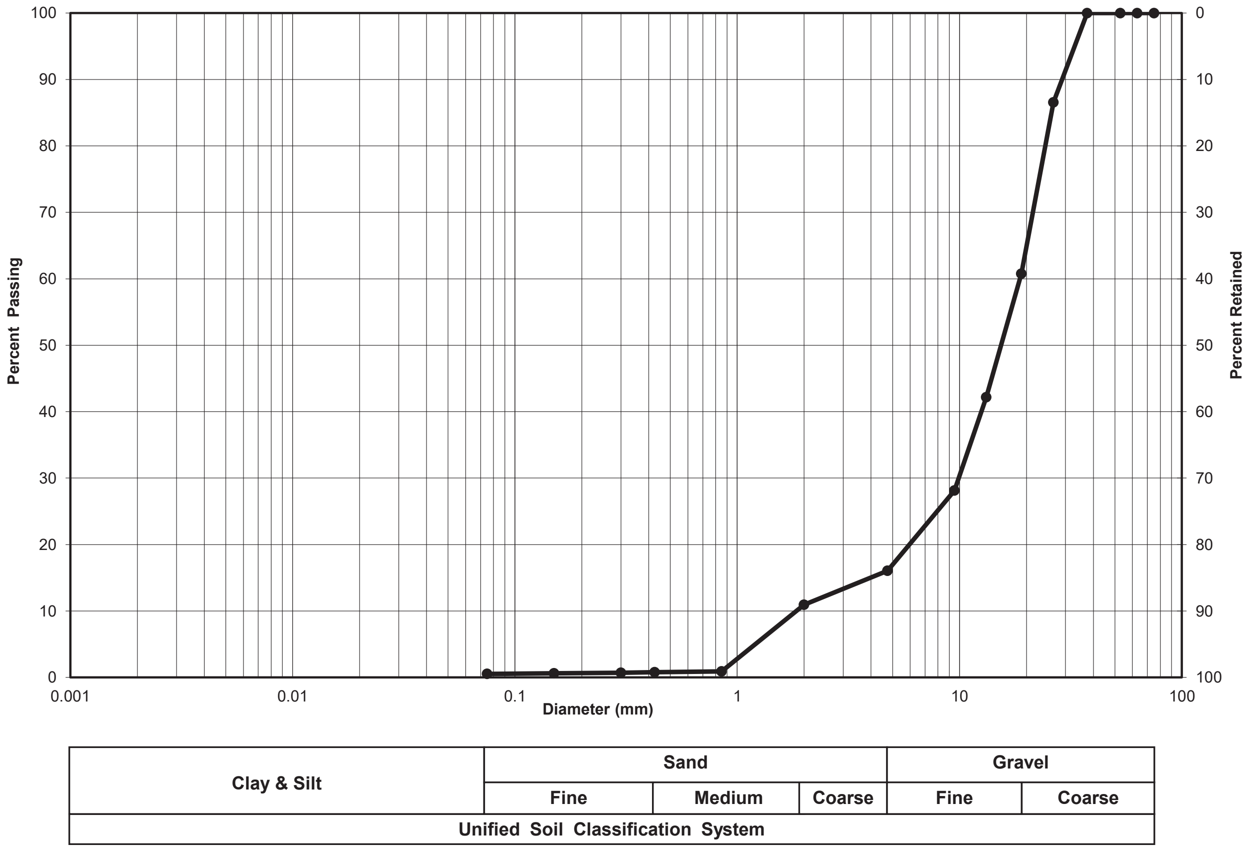
Remarks:

Performed by: S.Wheeler
Date: April 25, 2016
Verified by: N.Krebs
Date: May 3, 2016

Particle-Size Analysis of Soils
(ASTM D422)

Client:	Grandmaitre Estates	**Lab no.:**	OL7-14
Project/Site:	1009 Trim Road, Ottawa	**Project no.:**	161-03361-00

Borehole no.:	TP 4	Sample no.:	1
Depth:	0-0.6m	Location:	-



Percent Retained	**Gravel**	**Sand**	**Clay & Silt**	**Silt**	**Clay**
	83.9	15.5	0.6	-	-

Remarks:

Performed by:	S.Wheeler	**Date:**	April 25, 2016
Verified by:	N.Krebs	**Date:**	May 3, 2016

Liquid Limit, Plastic Limit and Plasticity Index of Soils
(ASTM D4318)

Client:	Grandmaitre Estates	**Lab no.:**	OL7-2
Project/Site:	1009 Trim Road, Ottawa	**Project no.:**	161-03361-00

Borehole no.:	16-1	Sample no.:	SS11	Depth:	7.6-8.25m
Soil description:	CH - High plasticity, inorganic clay			Date sampled:	-

Apparatus:	Hand Crank	Balance no.:	1	Porcelain bowl no.:	1
Liquid limit device no.:	1	Oven no.:	1	Spatula no.:	1
Sieve no.:	40	Glass plate no.:	1		

Liquid Limit (LL):

	Test No. 1	Test No. 2	Test No. 3
Number of blows	33	20	18
Water Content:			
Tare no.	12	9	80
Wet soil+tare, g	26.81	26.60	25.08
Dry soil+tare, g	22.64	22.48	21.16
Mass of water, g	4.17	4.12	3.92
Tare, g	14.56	14.84	14.00
Mass of soil, g	8.08	7.64	7.16
Water content %	51.6%	53.9%	54.7%

Plastic Limit (PL) - Water Content:

Tare no.	75	48
Wet soil+tare, g	20.58	21.18
Dry soil+tare, g	19.46	20.08
Mass of water, g	1.12	1.10
Tare, g	14.12	15.00
Mass of soil, g	5.34	5.08
Water content %	21.0%	21.7%
Average water content %	21.3%	

Natural Water Content (W^n):

Tare no.	B12	
Wet soil+tare, g	1012.80	
Dry soil+tare, g	689.20	
Mass of water, g	323.60	
Tare, g	90.70	
Mass of soil, g	598.50	
Water content %	54.1%	

Soil Preparation:

☑ Cohesive <425 µm

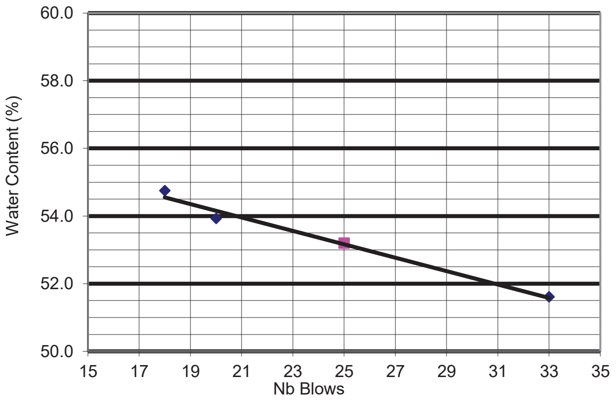
☐ Cohesive >425 µm

☐ Non-cohesive

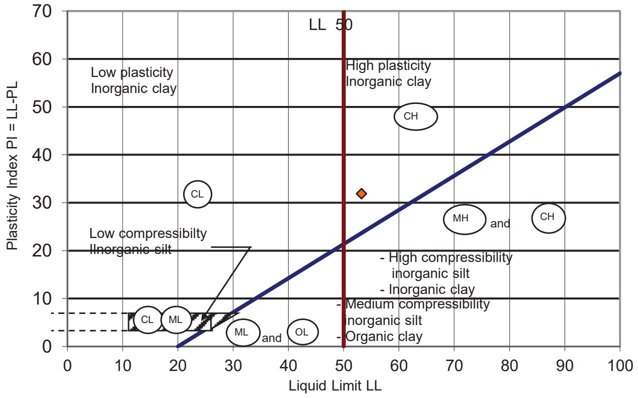
☑ Dry preparation

☐ Wet preparation

Results



Soil Plasticity Chart



Liquid Limit (LL)	Plastic Limit (PL)	Plasticity Index (PI)	Natural Water Content W^n
53.2	21	32	54

Remarks:

Performed by:	N.Krebs	**Date:**	April 22, 2016
Verified by:	N.Krebs	**Date:**	April 22, 2016

WSP

Liquid Limit, Plastic Limit and Plasticity Index of Soils
(ASTM D4318)

Client: Grandmaitre Estates
Lab no.: OL7-3

Project/Site: 1009 Trim Road, Ottawa
Project no.: 161-03361-00

Borehole no.: 16-1
Sample no.: SS18
Depth: 21.35-21.95m
Soil description: CH - High plasticity, inorganic clay
Date sampled: -

Apparatus: Hand Crank
Balance no.: 1
Porcelain bowl no.: 1
Liquid limit device no.: 1
Oven no.: 1
Spatula no.: 1
Sieve no.: 40
Glass plate no.: 1

Liquid Limit (LL):

	Test No. 1	Test No. 2	Test No. 3
Number of blows	32	25	18
Water Content:			
Tare no.	3	80	B68
Wet soil+tare, g	27.54	24.31	27.10
Dry soil+tare, g	23.41	20.84	22.59
Mass of water, g	4.13	3.47	4.51
Tare, g	14.95	14.00	13.86
Mass of soil, g	8.46	6.84	8.73
Water content %	48.8%	50.7%	51.7%

Plastic Limit (PL) - Water Content:

Tare no.	B67	14
Wet soil+tare, g	21.19	21.34
Dry soil+tare, g	19.91	19.97
Mass of water, g	1.28	1.37
Tare, g	13.96	13.88
Mass of soil, g	5.95	6.09
Water content %	21.5%	22.5%
Average water content %	22.0%	

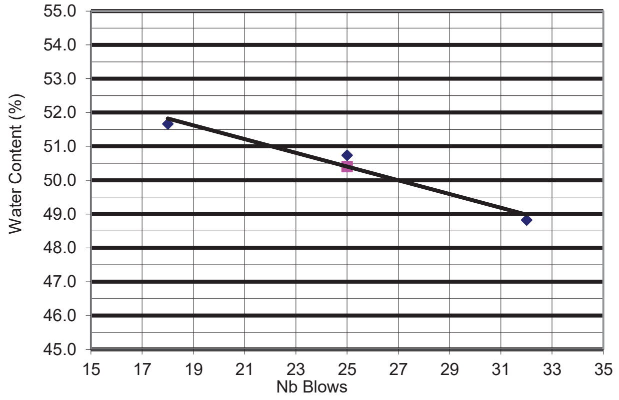
Natural Water Content (W^n):

Tare no.	B13	
Wet soil+tare, g	1017.10	
Dry soil+tare, g	641.30	
Mass of water, g	375.80	
Tare, g	84.60	
Mass of soil, g	556.70	
Water content %	67.5%	

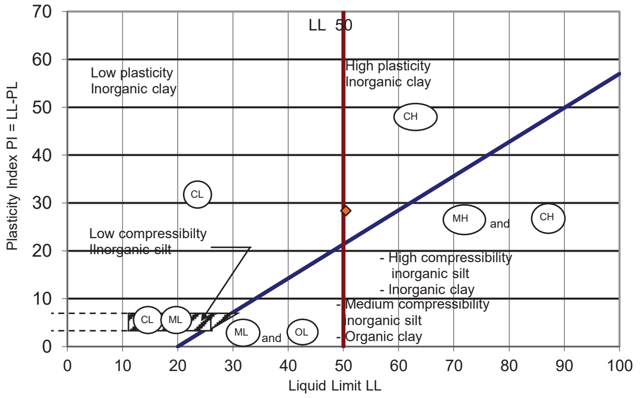
Soil Preparation:

☑ Cohesive <425 µm
☐ Cohesive >425 µm
☐ Non-cohesive
☑ Dry preparation
☐ Wet preparation

Results



Soil Plasticity Chart



Liquid Limit (LL)	Plastic Limit (PL)	Plasticity Index (PI)	Natural Water Content W^n
50.4	22	28	68

Remarks:

Performed by: N.Krebs
Date: April 20, 2016

Verified by: N.Krebs
Date: April 20, 2016

Liquid Limit, Plastic Limit and Plasticity Index of Soils
(ASTM D4318)

Client: Grandmaitre Estates
Lab no.: OL7-4
Project/Site: 1009 Trim Road, Ottawa
Project no.: 161-03361-00

Borehole no.: 16-1
Sample no.: SS21
Depth: 30.5-31.1m
Soil description: CH - High plasticity, inorganic clay
Date sampled: -

Apparatus: Hand Crank
Balance no.: 1
Porcelain bowl no.: 2
Liquid limit device no.: 1
Oven no.: 1
Spatula no.: 1
Sieve no.: 40
Glass plate no.: 1

Liquid Limit (LL):

	Test No. 1	Test No. 2	Test No. 3
Number of blows	25	22	19
Water Content:			
Tare no.	60	49	3
Wet soil+tare, g	24.20	23.89	25.76
Dry soil+tare, g	20.65	20.44	21.85
Mass of water, g	3.55	3.45	3.91
Tare, g	14.16	14.17	14.96
Mass of soil, g	6.49	6.27	6.89
Water content %	54.7%	55.0%	56.7%

Plastic Limit (PL) - Water Content:

Tare no.	12	9
Wet soil+tare, g	19.37	19.64
Dry soil+tare, g	18.48	18.73
Mass of water, g	0.89	0.91
Tare, g	14.58	14.87
Mass of soil, g	3.90	3.86
Water content %	22.8%	23.6%
Average water content %	23.2%	

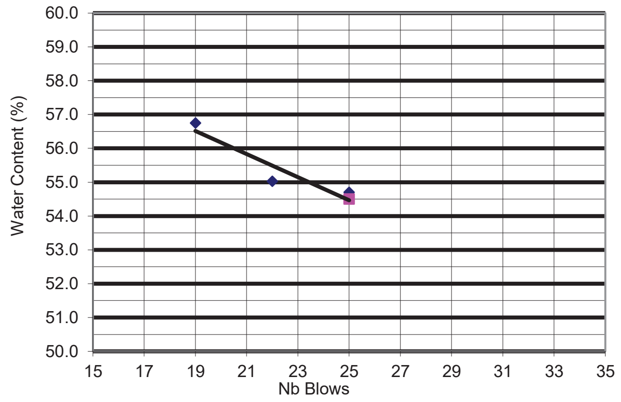
Natural Water Content (W^n):

Tare no.	B14	
Wet soil+tare, g	1014.80	
Dry soil+tare, g	653.10	
Mass of water, g	361.70	
Tare, g	100.90	
Mass of soil, g	552.20	
Water content %	65.5%	

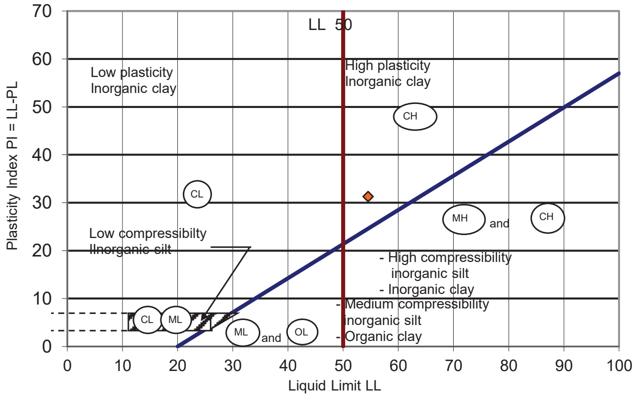
Soil Preparation:

☑ Cohesive <425 µm
☐ Cohesive >425 µm
☐ Non-cohesive
☑ Dry preparation
☐ Wet preparation

Results



Soil Plasticity Chart



Liquid Limit (LL)	Plastic Limit (PL)	Plasticity Index (PI)	Natural Water Content W^n
54.5	23	31	66

Remarks:

Performed by: N.Krebs
Date: April 20, 2016
Verified by: N.Krebs
Date: April 20, 2016

Liquid Limit, Plastic Limit and Plasticity Index of Soils
(ASTM D4318)

Client: Grandmaitre Estates
Lab no.: OL7-6
Project/Site: 1009 Trim Road, Ottawa
Project no.: 161-03361-00

Borehole no.: 16-2
Sample no.: SS8
Depth: 5.35-5.95m
Soil description: CH - High plasticity, inorganic clay
Date sampled: -

Apparatus: Hand Crank
Balance no.: 1
Porcelain bowl no.: 2
Liquid limit device no.: 1
Oven no.: 1
Spatula no.: 1
Sieve no.: 40
Glass plate no.: 1

Liquid Limit (LL):

	Test No. 1	Test No. 2	Test No. 3
Number of blows	41	28	16
Water Content:			
Tare no.	67	69	83
Wet soil+tare, g	22.72	23.71	25.00
Dry soil+tare, g	19.88	20.45	21.16
Mass of water, g	2.84	3.26	3.84
Tare, g	13.98	13.92	13.67
Mass of soil, g	5.90	6.53	7.49
Water content %	48.1%	49.9%	51.3%

Plastic Limit (PL) - Water Content:

Tare no.	78	8
Wet soil+tare, g	20.77	20.66
Dry soil+tare, g	19.74	19.55
Mass of water, g	1.03	1.11
Tare, g	14.87	14.84
Mass of soil, g	4.87	4.71
Water content %	21.1%	23.6%
Average water content %	22.4%	

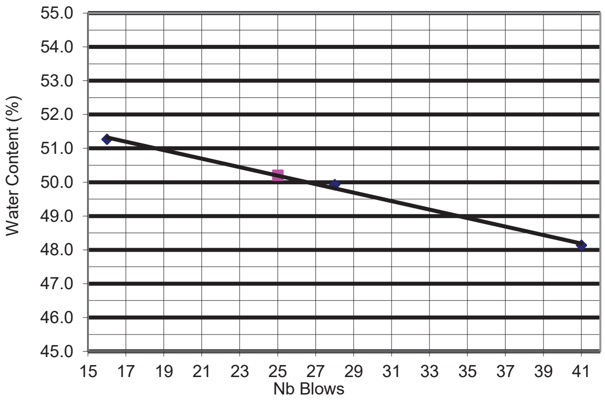
Natural Water Content (W^n):

Tare no.	B15	
Wet soil+tare, g	885.70	
Dry soil+tare, g	623.30	
Mass of water, g	262.40	
Tare, g	83.50	
Mass of soil, g	539.80	
Water content %	48.6%	

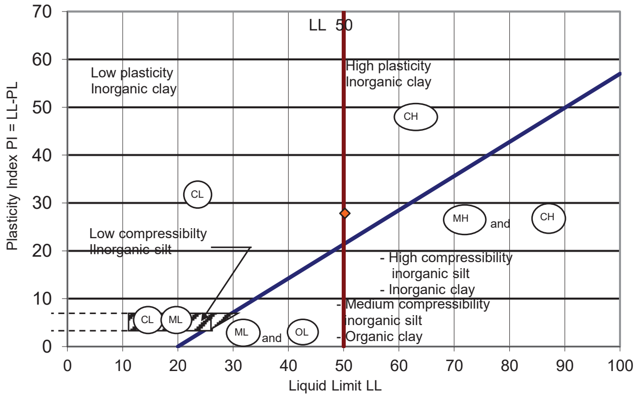
Soil Preparation:

☑ Cohesive <425 µm
☐ Cohesive >425 µm
☐ Non-cohesive
☑ Dry preparation
☐ Wet preparation

Results



Soil Plasticity Chart



Liquid Limit (LL)	Plastic Limit (PL)	Plasticity Index (PI)	Natural Water Content W^n
50.2	22	28	49

Remarks:

Performed by: S.Wheeler
Date: April 25, 2016
Verified by: N.Krebs
Date: April 25, 2016

Liquid Limit, Plastic Limit and Plasticity Index of Soils
(ASTM D4318)

Client: Grandmaitre Estates
Lab no.: OL7-7
Project/Site: 1009 Trim Road, Ottawa
Project no.: 161-03361-00

Borehole no.: 16-2
Sample no.: SS13
Depth: 10.65-11.25m
Soil description: CH - High plasticity, inorganic clay
Date sampled: -

Apparatus: Hand Crank
Balance no.: 1
Porcelain bowl no.: 4
Liquid limit device no.: 1
Oven no.: 1
Spatula no.: 1
Sieve no.: 40
Glass plate no.: 1

Liquid Limit (LL):

	Test No. 1	Test No. 2	Test No. 3
Number of blows	29	23	20
Water Content:			
Tare no.	B33	B39	146
Wet soil+tare, g	27.23	25.31	27.43
Dry soil+tare, g	22.54	21.41	23.06
Mass of water, g	4.69	3.90	4.37
Tare, g	13.52	13.94	14.82
Mass of soil, g	9.02	7.47	8.24
Water content %	52.0%	52.2%	53.0%

Plastic Limit (PL) - Water Content:

Tare no.	146	B39
Wet soil+tare, g	22.05	20.99
Dry soil+tare, g	20.57	19.50
Mass of water, g	1.48	1.49
Tare, g	14.86	13.95
Mass of soil, g	5.71	5.55
Water content %	25.9%	26.8%
Average water content %	26.4%	

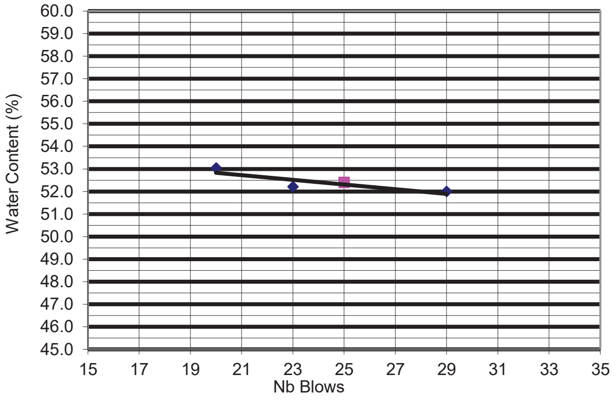
Natural Water Content (W^n):

Tare no.	B16	
Wet soil+tare, g	1331.40	
Dry soil+tare, g	836.10	
Mass of water, g	495.30	
Tare, g	90.10	
Mass of soil, g	746.00	
Water content %	66.4%	

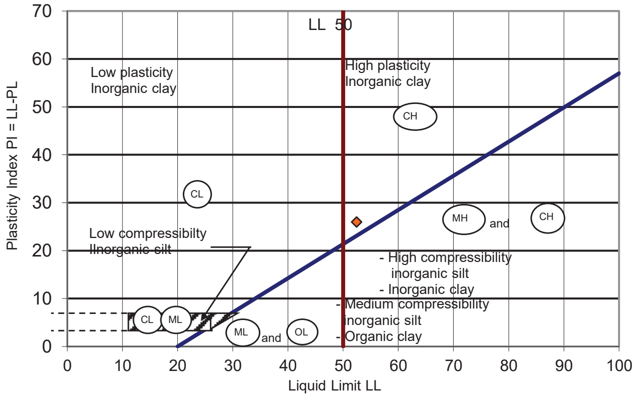
Soil Preparation:

☑ Cohesive <425 µm
☐ Cohesive >425 µm
☐ Non-cohesive
☑ Dry preparation
☐ Wet preparation

Results



Soil Plasticity Chart



Liquid Limit (LL)	Plastic Limit (PL)	Plasticity Index (PI)	Natural Water Content W^n
52.4	26	26	66

Remarks:

Performed by: N.Krebs
Date: May 4, 2016

Verified by: N.Krebs
Date: May 4, 2016

Liquid Limit, Plastic Limit and Plasticity Index of Soils
(ASTM D4318)

Client: Grandmaitre Estates
Lab no.: OL7-8
Project/Site: 1009 Trim Road, Ottawa
Project no.: 161-03361-00

Borehole no.: 16-3
Sample no.: SS7
Depth: 4.55-5.2m
Soil description: CH - High plasticity, inorganic clay
Date sampled: -

Apparatus: Hand Crank
Balance no.: 1
Porcelain bowl no.: 2
Liquid limit device no.: 1
Oven no.: 1
Spatula no.: 1
Sieve no.: 40
Glass plate no.: 1

Liquid Limit (LL):

	Test No. 1	Test No. 2	Test No. 3
Number of blows	35	19	15
Water Content:			
Tare no.	49	60	48
Wet soil+tare, g	26.68	25.54	27.24
Dry soil+tare, g	22.33	21.47	22.74
Mass of water, g	4.35	4.07	4.50
Tare, g	14.16	14.17	14.99
Mass of soil, g	8.17	7.30	7.75
Water content %	53.2%	55.8%	58.1%

Plastic Limit (PL) - Water Content:

Tare no.	3	B75
Wet soil+tare, g	21.11	20.30
Dry soil+tare, g	19.95	19.16
Mass of water, g	1.16	1.14
Tare, g	14.96	14.12
Mass of soil, g	4.99	5.04
Water content %	23.2%	22.6%
Average water content %	22.9%	

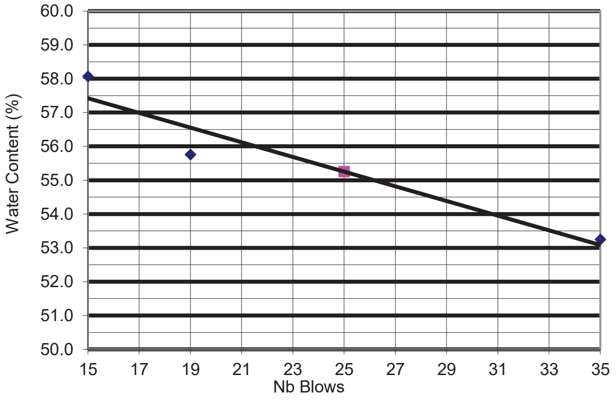
Natural Water Content (W^n):

Tare no.	B4	
Wet soil+tare, g	540.90	
Dry soil+tare, g	405.60	
Mass of water, g	135.30	
Tare, g	97.20	
Mass of soil, g	308.40	
Water content %	43.9%	

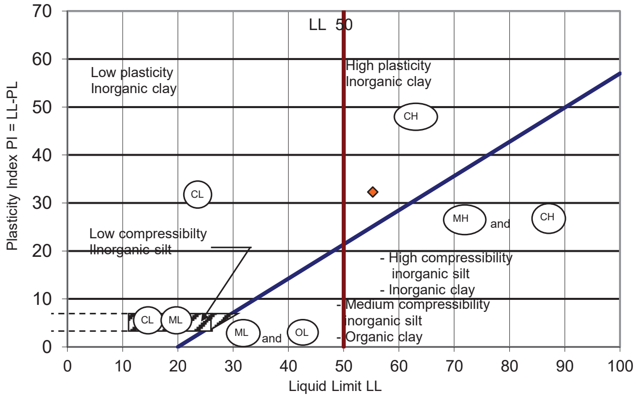
Soil Preparation:

☑ Cohesive <425 µm
☐ Cohesive >425 µm
☐ Non-cohesive
☑ Dry preparation
☐ Wet preparation

Results



Soil Plasticity Chart



Liquid Limit (LL)	Plastic Limit (PL)	Plasticity Index (PI)	Natural Water Content W^n
55.25	23	32	44

Remarks:

Performed by: N.Krebs
Date: April 25, 2016
Verified by: N.Krebs
Date: April 25, 2016

WSP

Liquid Limit, Plastic Limit and Plasticity Index of Soils
(ASTM D4318)

Client:	Grandmaitre Estates	**Lab no.:**	OL7-10
Project/Site:	1009 Trim Road, Ottawa	**Project no.:**	161-03361-00

Borehole no.:	16-4	Sample no.:	SS8	Depth:	5.35-5.95m
Soil description:	CH - High plasticity, inorganic clay			Date sampled:	-

Apparatus:	Hand Crank	Balance no.:	1	Porcelain bowl no.:	5
Liquid limit device no.:	1	Oven no.:	1	Spatula no.:	1
Sieve no.:	40	Glass plate no.:	1		

Liquid Limit (LL):

	Test No. 1	Test No. 2	Test No. 3
Number of blows	33	28	13
Water Content:			
Tare no.	9	19	54
Wet soil+tare, g	27.60	24.55	25.03
Dry soil+tare, g	23.20	20.65	21.09
Mass of water, g	4.40	3.90	3.94
Tare, g	14.85	13.50	14.41
Mass of soil, g	8.35	7.15	6.68
Water content %	52.7%	54.5%	59.0%

Plastic Limit (PL) - Water Content:

Tare no.	80	12
Wet soil+tare, g	20.18	21.16
Dry soil+tare, g	19.04	19.98
Mass of water, g	1.14	1.18
Tare, g	14.01	14.56
Mass of soil, g	5.03	5.42
Water content %	22.7%	21.8%
Average water content %	22.2%	

Natural Water Content (W^n):

Tare no.	B7	
Wet soil+tare, g	526.50	
Dry soil+tare, g	382.40	
Mass of water, g	144.10	
Tare, g	91.50	
Mass of soil, g	290.90	
Water content %	49.5%	

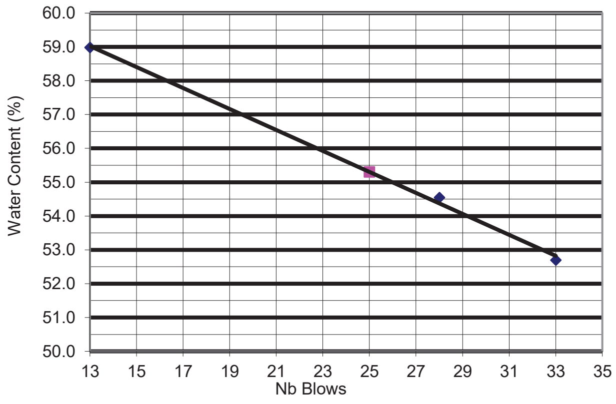
Soil Preparation:

☑ Cohesive <425 µm ☑ Dry preparation

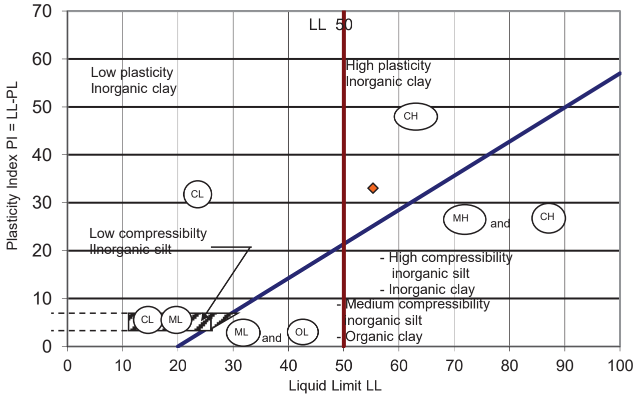
☐ Cohesive >425 µm ☐ Wet preparation

☐ Non-cohesive

Results



Soil Plasticity Chart



Liquid Limit (LL)	Plastic Limit (PL)	Plasticity Index (PI)	Natural Water Content W^n
55.3	22	33	50

Remarks:

Performed by:	N.Krebs	**Date:**	April 25, 2016
Verified by:	N.Krebs	**Date:**	April 25, 2016

Liquid Limit, Plastic Limit and Plasticity Index of Soils
(ASTM D4318)

Client:	Grandmaitre Estates	**Lab no.:**	OL7-12
Project/Site:	1009 Trim Road, Ottawa	**Project no.:**	161-03361-00

Borehole no.:	16-6	Sample no.:	SS6	Depth:	3.8-4.4m
Soil description:	CH - High plasticity, inorganic clay			Date sampled:	-

Apparatus:	Hand Crank	Balance no.:	1	Porcelain bowl no.:	4
Liquid limit device no.:	1	Oven no.:	1	Spatula no.:	1
Sieve no.:	40	Glass plate no.:	1		

Liquid Limit (LL):

	Test No. 1	Test No. 2	Test No. 3
Number of blows	40	28	14
Water Content:			
Tare no.	37	48	21
Wet soil+tare, g	24.74	26.29	23.76
Dry soil+tare, g	20.80	22.03	19.81
Mass of water, g	3.94	4.26	3.95
Tare, g	13.99	15.02	13.83
Mass of soil, g	6.81	7.01	5.98
Water content %	57.9%	60.8%	66.1%

Plastic Limit (PL) - Water Content:

Tare no.	26	150
Wet soil+tare, g	19.98	21.59
Dry soil+tare, g	18.81	20.36
Mass of water, g	1.17	1.23
Tare, g	13.72	14.94
Mass of soil, g	5.09	5.42
Water content %	23.0%	22.7%
Average water content %	22.8%	

Natural Water Content (W^n):

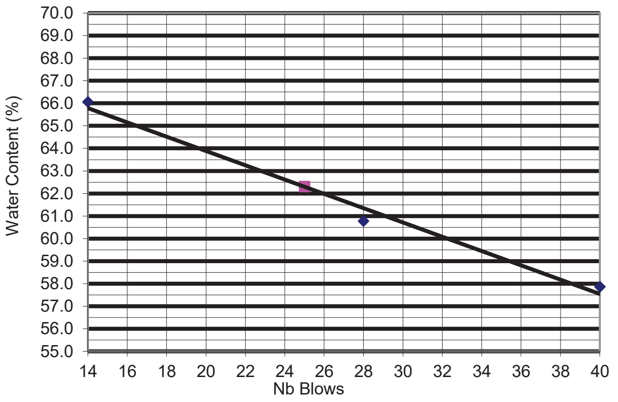
Tare no.	B6	
Wet soil+tare, g	1007.40	
Dry soil+tare, g	658.00	
Mass of water, g	349.40	
Tare, g	112.00	
Mass of soil, g	546.00	
Water content %	64.0%	

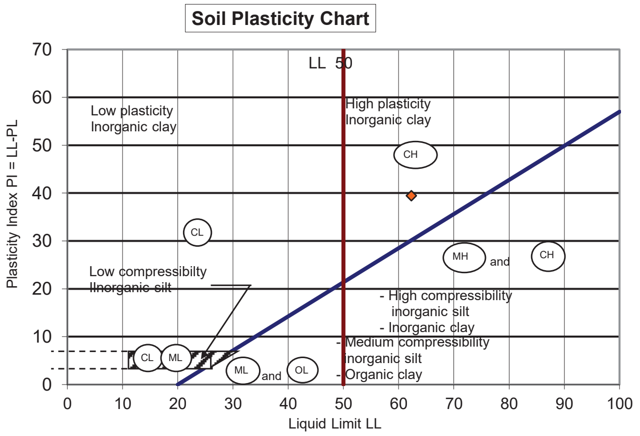
Soil Preparation:

☑ Cohesive <425 µm
☐ Cohesive >425 µm
☐ Non-cohesive

☑ Dry preparation
☐ Wet preparation

Results





Liquid Limit (LL)	Plastic Limit (PL)	Plasticity Index (PI)	Natural Water Content W^n
62.3	23	39	64

Remarks:

Performed by:	N.Krebs	**Date:**	May 4, 2016
Verified by:	N.Krebs	**Date:**	May 4, 2016

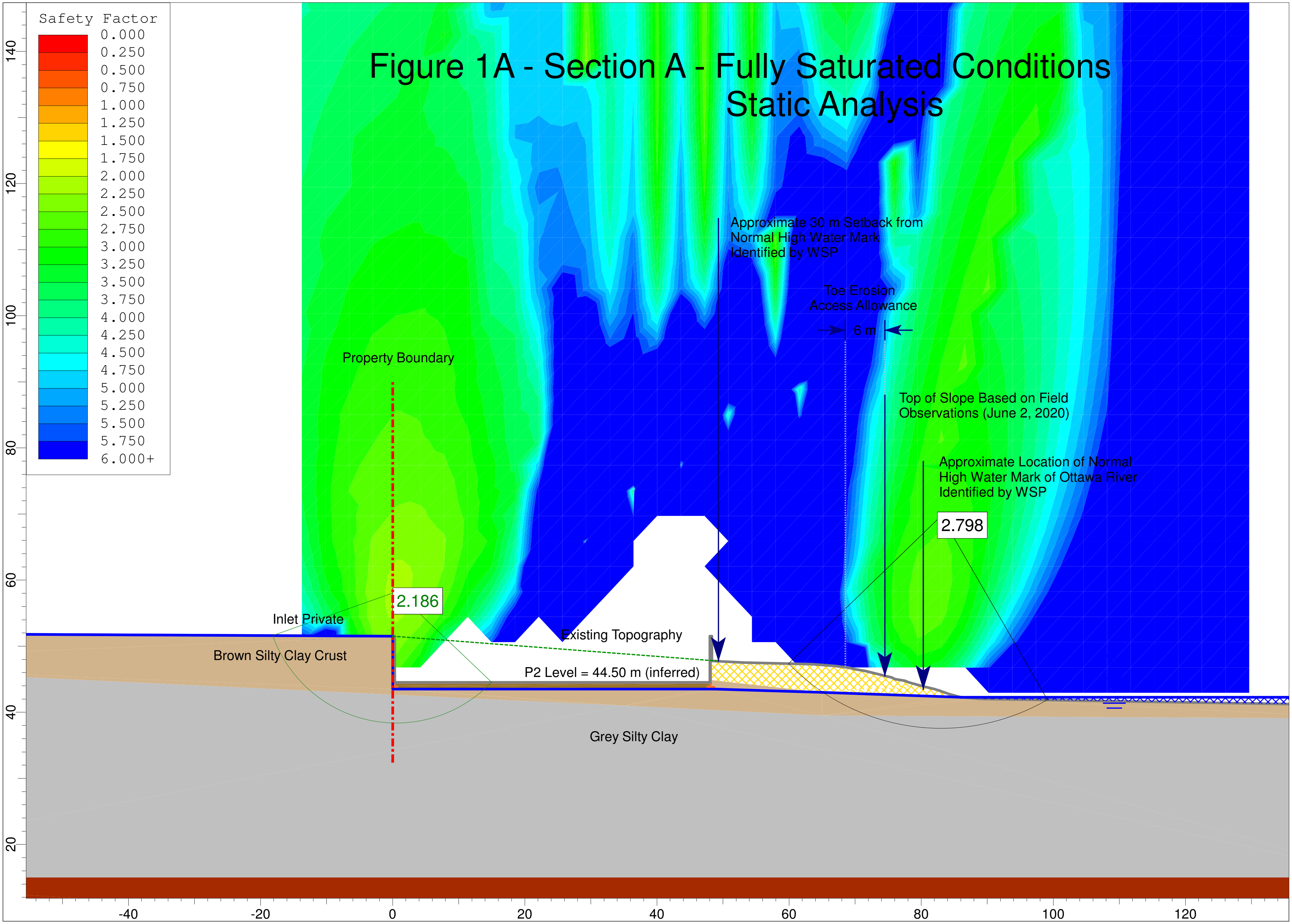


Figure 1A - Section A - Fully Saturated Conditions Static Analysis

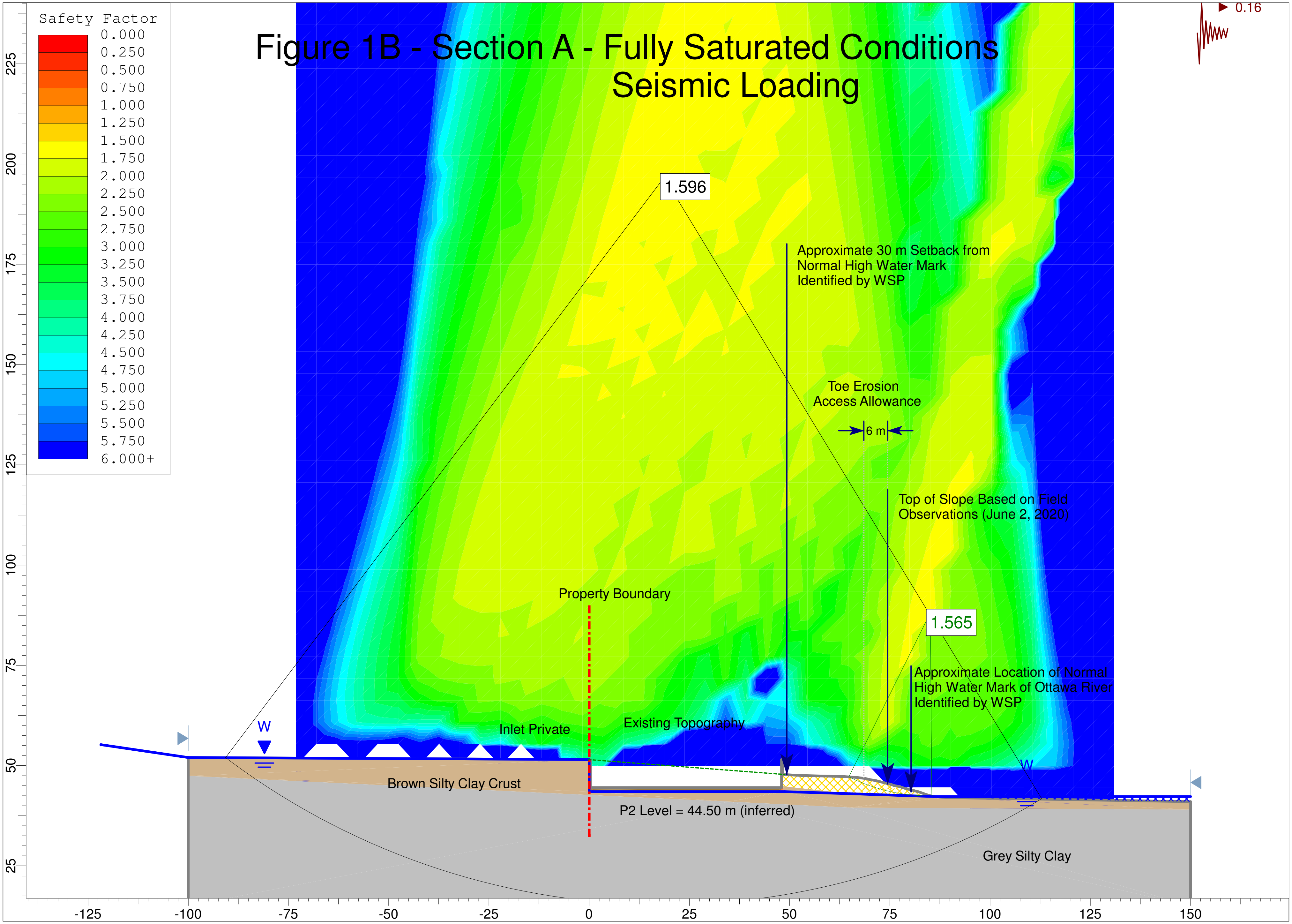


Figure 1B - Section A - Fully Saturated Conditions
Seismic Loading

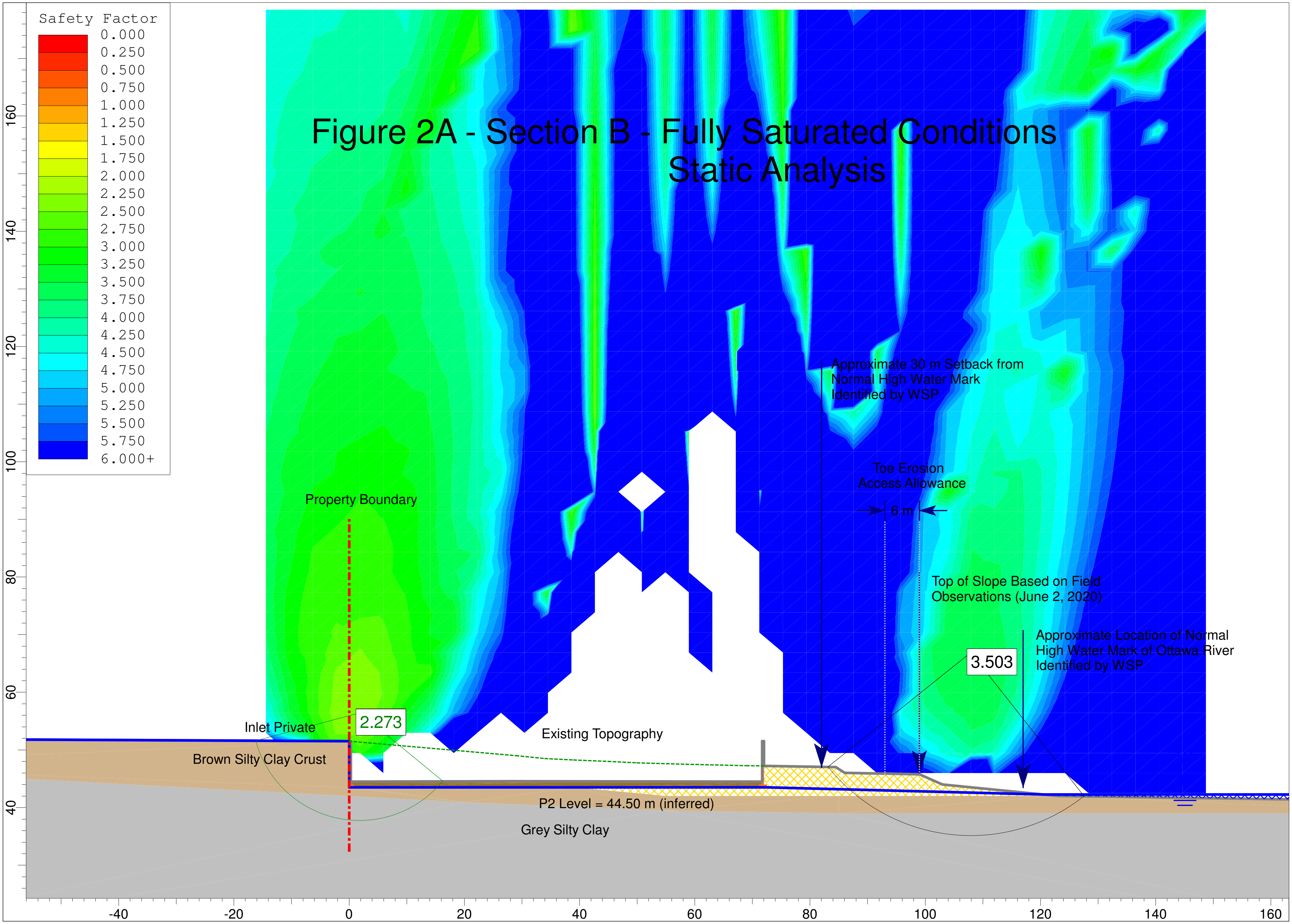


Figure 2A - Section B - Fully Saturated Conditions Static Analysis

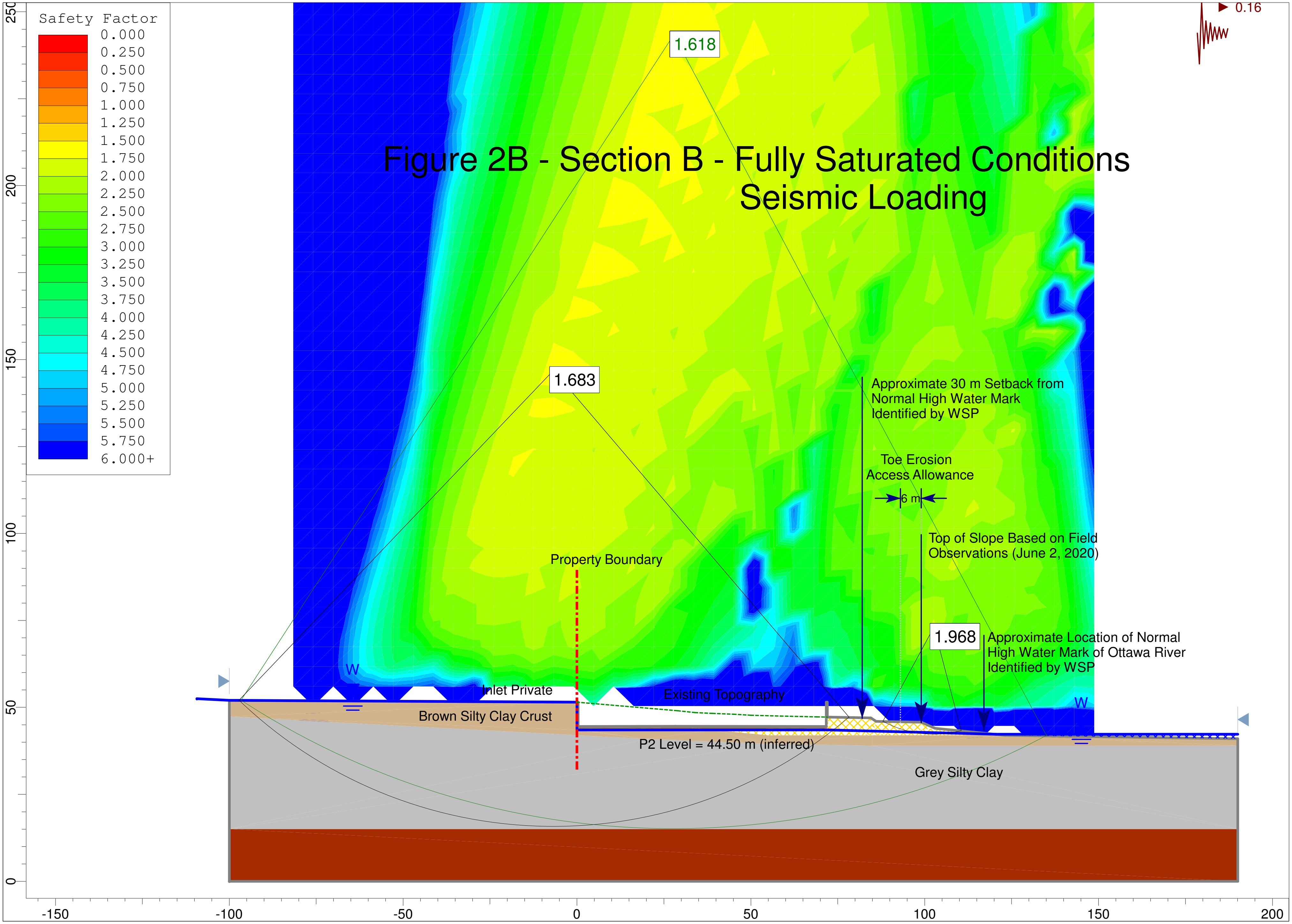
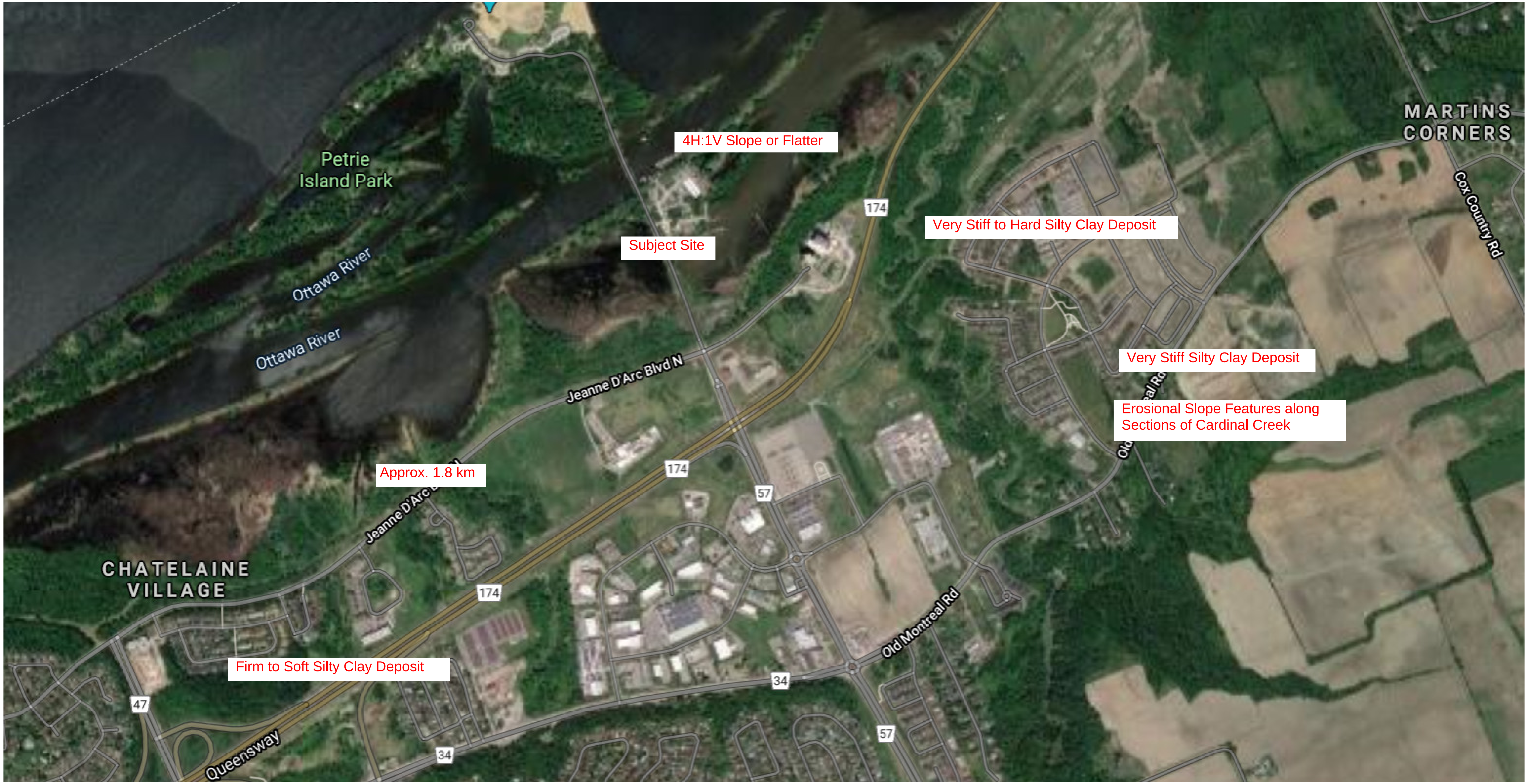


Figure 2B - Section B - Fully Saturated Conditions
Seismic Loading

FIGURE 3 - 500 m Study Area

FIGURE 4 - AREA'S GEOLOGY



Influence of surficial crusts on the development of spreads and flows in Eastern Canadian sensitive clays

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ABSTRACT

Spreads and flows are the two main types of large retrogressive landslides occurring in Eastern Canadian sensitive clays. In spreads, the soil mass mobilized during failure is dislocated in a succession of horsts and grabens leading to a typical ribbed topography in the landslide scar. The failure mode for flows, on the contrary, is characterized by a succession of rotational slides propagating rearward, which requires that clays liquefy during the movement. These mechanisms are now relatively well understood. However, conditions leading to the development of either a flow or a spread are not yet clearly identified. Some numerical results published in the literature suggest that spreads form preferentially when a non-sensitive crust overlying a sensitive clay deposit is present. We examine in this paper whether this result is supported by observations made on several spreads and flows that occurred in southeastern Ontario and Quebec. It is shown that the presence of a crust is likely not a discriminating factor.

RÉSUMÉ

Les étalements et les coulées sont les deux principaux types de grands glissements de terrain rétrogressifs se produisant dans les argiles sensibles de l'est du Canada. Dans le cas des étalements, la masse de sol mobilisée lors de la rupture est disloquée en une succession de horsts et de grabens, laissant une topographie nervurée typique à l'intérieur des cicatrices. Le mode de rupture pour les coulées est au contraire caractérisé par une succession de glissements rotationnels se propageant vers l'arrière, ce qui nécessite que l'argile se liquéfie lors du mouvement. Ces mécanismes sont maintenant assez bien compris. Cependant, les conditions conduisant au développement d'un étalement ou d'une coulée restent mal identifiées. Certains résultats de modélisations numériques publiés dans la littérature suggèrent que les étalements se forment surtout lorsqu'une croûte recouvrant les dépôts d'argiles sensibles est présente. Nous regardons dans cet article si ces résultats sont validés par l'observation de cas réels d'étalements et de coulées dans le sud-est de l'Ontario et au Québec. On montre que la présence d'une croûte n'est probablement pas un facteur discriminant.

1 INTRODUCTION

Large retrogressive landslides occurring in eastern Canadian sensitive clays can be grouped into two main types depending on the failure mode: spreads and flows (Fig. 1). In spreads, the soil mass mobilized during failure is dislocated in a rearward succession of horsts and grabens, also called prisms and wedges, leading to a characteristic ribbed topography in the landslide scar. The formation of horsts and grabens is the result of an extensional, active state of failure, and the overall movement along the basal failure surface is translational, as exemplified by the horizontal layering of strata often observed in intact horsts, indicating that no rotation occurred. The amount of soil remaining in a spread scar is variable but can be important, with only a localized remolding of clays. This failure mode was first identified by Odenstad (1951) and further analyzed by Carson (1977), Locat et al. (2011), Quinn et al. (2012), and Dey et al. (2015). The failure mode for flows, on the contrary, is characterized by a succession of rotational slides propagating rearward (Bjerrum, 1955; Tavenas et al., 1971; Gregersen, 1981; Tavenas, 1984; Demers et al. 2014). In each rotational failure, the displaced soil mass must reach a sufficiently fluid state to be able to be evacuated from the slope toe and to allow retrogression. Otherwise, the backscarp slope is buttressed by debris, which can stop the retrogressive movement. This implies that clays must have a low remolded shear strength to flow away from the slope toe. Typically, only a veneer of strongly remolded clays with patches of the upper crust of the soil profile are left in a flow scar. The floor topography is relatively smooth with no significant relief variations. The retrogression process for flows can be initiated by a first rotational slide along a slope resulting from various causes, such as a riverbank erosion, an excavation at the slope toe, an overloading at or near the slope crest, an increase in porewater pressures following rainfall or snowmelt, or by seismic shaking. For spreads, the current understanding is that any perturbation conducing to a horizontal unloading along a slope can trigger retrogression if appropriate conditions are met. These mechanisms may include as for flows an initial slide or seismic shaking. In some cases, field observations suggest that these two types of failures can occur successively during the same landslide event (Geertsema et al., 2006; Tremblay-Auger et al., 2018). Both spreads and flows can propagate rapidly on flat grounds or on very gentle slopes, over large distances often exceeding several hundreds of meters.

As mentioned by Demers et al. (2014), the geotechnical properties of soils involved in large retrogressive landslides are quite similar, and conditions leading to the development of either a flow or a spread are not yet well identified and understood. Several geometrical and mechanical factors can interact in a complex way, making it difficult to identify discriminating conditions. In this respect, numerical simulations are of interest because they allow parametric analyzes, which is obviously impossible to do with real landslides in the field (Dey et al., 2015; Locat et al. 2015; Wang et al., 2016a, 2016b; Wang and Hawlader, 2017; Zhang et al., 2018; Tran and Sołowski, 2019). In some of these simulations, it appears that spreads only develop when a crust having a significant thickness relative to the thickness of the mobilized soil mass overlies the sensitive clay unit in which the failure propagates. For example, Wang and Hawlader (2017) show that a 5-m thick crust with a shear strength of 50 kPa overlying sensitive clays with shear strengths linearly increasing with depth, from 40 to 100 kPa at a depth of 35 m, was needed to generate a spread. The initial slope was 30 m high with an angle of 26.6°. A simulation with no crust led to the development of a flow. However, not all the parameters of the analyses were exactly the same in the simulations, in particular for the post-peak properties of sensitive clays. The extent to which this affects the results of the simulations is not discussed in the paper, but the authors observe that the presence of a crust is a potential cause of the formation of horsts and grabens. In Dey et al. (2015), it is mentioned that a sufficiently high undrained shear strength is needed for a spread to occur but that this failure mode is prevented for a very thick crust representing, in the case analysed by the authors, 47% of the height above the failure surface. With all other parameters being equal, a very thin crust (5% of the height above the failure surface) only leads to the development of a horizontal shear band that initiates at the base of the slope. The slope height for the cases analysed was 19 m and the slope angle 30°. Similar results on the influence of a crust were obtained by Q.A. Tran (personal communication, Nov. 2018; Tran and Sołowski, 2019). In all these studies, the crust was modeled as a non-sensitive material in undrained conditions.

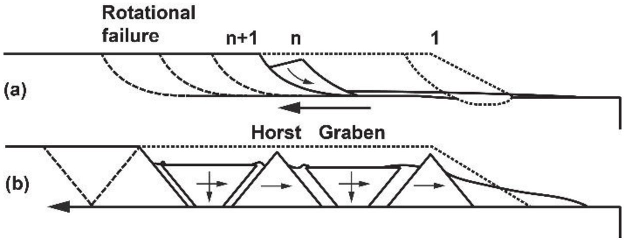

Figure 1. Conceptual sketch of the mode of failure (a) for a flow with successive rotational slides propagating rearward, and (b) for a spread involving the dislocation of the soil mass into horsts and grabens. Large arrows indicate the propagation direction of the failures. Small arrows indicate the main components of soil movement.

These simulation results are intriguing because crusts with variable thicknesses and strengths are almost always present on the top of sensitive clay deposits in Eastern Canada lowlands, including areas where large retrogressive landslides occurred. The goal of this paper is to examine whether these results are supported by observations made on several spreads and flows that occurred in southeastern Ontario and Quebec. First, the formation and the properties of crusts are reviewed. Then, criteria used to identify the failure mode are explained. Finally, results are presented and discussed.

2 CHARACTERSTICS OF CRUSTS OVERLYING SENSITIVE CLAY DEPOSITS

Surficial crusts overlying sensitive clay deposits are widespread in Eastern Canada lowlands due to the climatic and geological contexts of the region. In the literature, the terms ‘‘crust’’ most often implicitly refers to ‘‘clay crust’’ (e.g. Moum and Rosenqvist, 1957; Lefebvre et al., 1987). For this study, the definition of ‘‘surficial crust’’ is extended to any surficial unit having a shear strength or cone tip resistance well above values determined with in-situ field vane or piezocone tests immediately below in the intact clay deposit. This includes weathered clay crusts *sensu stricto*, and sandy units associated with regressive or fluvial-estuarine geological facies representing the final depositional stages of postglacial marine seas. According to this definition, a sandy cover does not need to be cemented to be considered as a crust. It was decided to include sandy crusts because their behavior during the failure propagation is probably not very different to the behavior of clay crusts as it will be seen.

2.1 Clay crusts

A surficial clay crust can develop from an unweathered clay deposit in response to a variety of chemical and physical processes. Seasonal cycles of freezing and thawing over millennia, along with fluctuations of the location of the groundwater table, are the major physical causes leading to a change in the mechanical properties of clays. Depending on the snow cover, the present-day average depth reached by freezing is about 0 to 2 m under the latitudes of the region of interest but could have been larger during colder periods in the last 10,000 years or so. Frost action causes water migration, which results in the formation of ice lenses and in a major change of the fabric of intact clays (Leroueil et al., 1991; Konrad et al., 1995).

The groundwater table fluctuations generate cycles of desiccation and wetting. These fluctuations can easily affect depths of two to four meters, especially close to slope crests where piezometric monitoring systematically indicates during dry periods deeper water tables than those on flat grounds. Infiltration of the surface waters, which usually have a chemistry different to the chemistry of the original pore water, is a secondary factor that can produce the oxidation and a partial cementation of the intact clays (Moum and Rosenqvist, 1957).

Trees also contribute to the weathering process by drawing groundwater from their roots. In addition, trees extract nutrients from the soil surrounding the roots, which can locally induce a strong chemical weathering. These effects are probably not negligible on regional scales if we consider the time period during which they were active. Most areas left free of water after the marine regression were rapidly colonized by trees (Dyke, 2005). By 6,000 years before present, a mixed forest with deciduous trees covered the entire St. Lawrence Valley and its tributaries up to the Saguenay region, while the lower north shore of the St. Lawrence River was occupied by a boreal forest. The maximum depth at which roots have a noticeable impact depends on the tree species, on the availability of water, and on the soil type. In studying the effect of trees on building settlement in the Ottawa and Montreal areas, Crawford (1968) and Silvestri et al. (1994) shown that changes in water content extended to depths exceeding three meters in clay deposits in the vicinity of some tree species common in the St. Lawrence Valley.

Another factor contributing to the development of a crust in clay deposits is the formation, by vertical and lateral erosion, of valleys, channels, and marine terraces. The associated stress relief results in the fracturing of clays in the eroded areas (Lefebvre and Morissette, 1984). It is often observed in fresh exposed lateral and back scarps of rotational slides that clays are traversed by a network of fissures reaching depths of two to three meters or more. These fissures are privileged paths for surface waters and tree roots which in turn facilitates weathering.

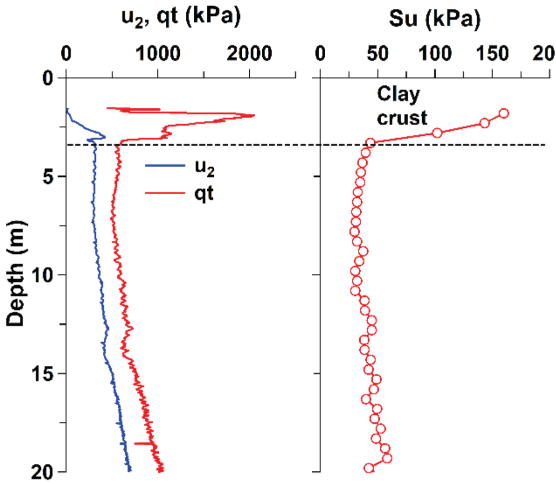

Figure 2. Piezocone and field vane tests profiles in a sensitive clay deposit overlain by a stiffer clay crust (case #32, Table 1). A hole was drilled to a depth of 1.5 m before beginning the piezocone test to avoid the desaturation of the porous element; u_2 is the pore pressure generated by cone penetration and qt is the cone tip resistance. Su is the soil shear strength determined from the field vane test.

As a result of a combination of these different processes, the thickness of the weathered clay crust generally varies from 1 to 6 m and is often of the order of 3 m on flat grounds (Lefebvre et al., 1987). It has long been recognized (e.g. Eden and Crawford, 1957) that the decrease of water content, and, possibly to a lesser extent, chemical weathering, lead to a significant increase of the undrained shear strength in the clay crust. Compared to unweathered sensitive clays, the plasticity indices and the remolded shears strengths in the crust are higher. Consequently, weathered clays are not sensitive to remolding. Typical resistance profiles are shown in Figure 2 for a site representative of the geotechnical conditions in Eastern Canada lowlands. This site is located just outside of a landslide scar that occurred along a small tributary of the Richelieu River about 30 km east of Montreal (case #32, Table 1). The undrained shear strength determined with the field vane test decreases from about 150 kPa close to the ground surface to about 40 kPa just below the crust. The cone tip resistance for a piezocone test done nearby follows the same trend.

2.2 Sandy crusts

As relative sea levels fell due to isostatic rebound following the last deglaciation, deltaic sands were deposited at the mouth of fluvial streams flowing into postglacial seas, like the Champlain Sea (e.g. Gadd, 1987; Parent and Occhietti, 1988). These deltaic units often cover marine clays and can reach thicknesses of a few tens of meters, particularly on the north shores of the Ottawa and St. Lawrence Rivers. They are generally organized in a complex succession of strata and channels of well sorted sands but can also locally contain finer grained sediments. Figure 3 shows a piezocone test result in a thick deltaic sand unit overlying a sensitive clay deposit at the rear of a spread (case #16, Table 1; see also Fig. 5). The cone tip resistance is much higher in the upper sandy unit than in the sensitive clays (note the difference in scales of Figs. 2 and 3) and shows strong variations over short vertical distances reflecting the heterogeneity of this sandy unit.

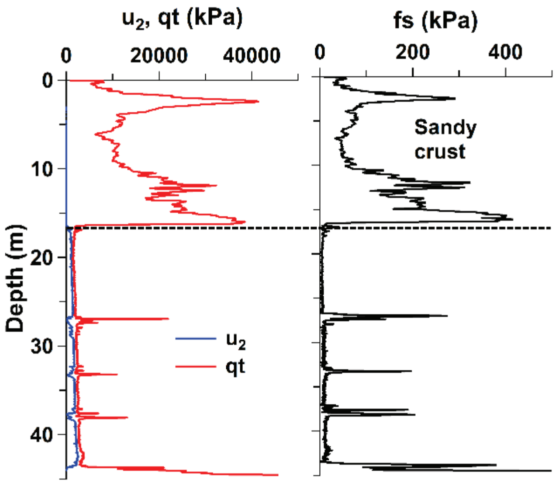

Figure 3. Piezocone test profiles in a sensitive clay deposit overlain by a stiffer sandy crust (case #16, Table 1); fs is the friction developed along the cone shaft during penetration.

During the last stage of the recession of marine waters, a fluvial system started to form in areas now approximately occupied by the Ottawa and the St. Lawrence Rivers (Gadd, 1987), while more open estuarine conditions existed farther downstream. Surficial sediments were reworked and redeposited as a thin blanket of sands or silts depending on the source material and on the distance over which they were transported. The thickness of these sandy fluvial or estuarine sediments is generally less than about 4 to 5 m and is on average 0.5 to 1.0 m. When this cover is relatively thin, a clay crust may have developed just below the sand cover once the lands emerged, as described previously.

2.3 Remarks on the behavior of surficial crusts

In the numerical simulations cited in introduction, the analyses are performed for undrained conditions because the few available eyewitnesses accounts indicate that large retrogressive landslides may occur very quickly, sometimes in less than a few minutes (e.g. Tavenas et al., 1971; Demers et al., 2014). In such rapid mass movements, the soils may not have time to drain and dissipate excess pore pressures during the propagation of failure. Although justified for the modeling of low permeability sensitive clay deposits, postulating that the whole soil profile behaves in an undrained manner is questionable when a clay or a sandy crust is present.

Field and laboratory observations show that clay crusts are highly fissured, both at the micro and macroscopic scales (Konrad et al., 1995). It has been reported by Lafleur and Lefebvre (1980) and Lafleur et al. (1987) that hydraulic conductivities are not controlled by the clay matrix but by these discontinuities. According to these authors, hydraulic conductivities in the clay crust can be higher by at least two to three orders of magnitude than in intact clays (10^{-8} - 10^{-7} m/s, compared to 10^{-10} - 10^{-9} m/s), which is likely enough to change its behavior from undrained to drained.

A related consequence of the presence of fissures is that the mass strength of the crust is similarly not controlled by the clay matrix. In a study on the back-analyses of several natural slope failures in Canadian soft clay deposits, Lefebvre (1981) underscored that ‘‘it is not reasonable to assume that the soil can resist tensile stresses under long term conditions, especially in the shallow superficial zone which is known to be fissured’’. In the limit-equilibrium slope stability analyses presented in that paper, a vertical tension crack full of water was considered in the crust. The crust was therefore modeled with a null strength.

Even if the crust behavior is not fully drained during the development of retrogressive landslides, the undrained shear strength as determined with the field vane test has been shown to be not compatible with the mobilized shear strength in back-analyses of failed embankments and excavations where a clay crust was involved (Silvestri, 1980; Lefebvre et al., 1987; Lafleur et al., 1988). Lefebvre et al. (1987) recommended to use, for practical purpose, the field vane strength measured in the intact clay immediately below the crust, which typically has a significantly lower value than the observed average shear strength in the crust (Fig. 2).

From the above, it appears that the behavior of clay crusts and sandy crusts may not be as different from what one might think at first glance. To support that view, it is interesting to recall 1) that Leroueil et al. (1991) observed that a sensitive clay subjected to freeze-thaw cycles had a sand-like behavior due to its micro-fissured nodular fabric; and 2) that hydraulic conductivities of sands, of the order of 10^{-7} -10^{-4} m/s, partly overlap those of clay crusts.

In accordance with this line of reasoning, it would thus be surprising that a clear relation exists between the presence of a crust and the landslide type.

3 SELECTED SPREADS AND FLOWS

A total of 37 large retrogressive landslide cases were selected for this study from case histories documented by the Ministère des transports du Québec (Fig. 4; Table 1). Most of these landslides are located between the Ottawa region and Quebec City, in areas that were covered about 10,500 to 12,000 years ago by the Champlain Sea (e.g. Parent and Occhietti, 1988). One landslide is in the Saguenay-Lake St. John region (case #1, Table 1), and three others are located east of Quebec City on the north shore of the St. Lawrence River (cases #2, 3, and 7, Table 1).

For a landslide to be selected, at least two piezocone tests had to be available, one outside the scar to determine the crust thickness, the other inside the scar to locate the failure surface and thus determine the thickness of the mobilized soil mass. For case #3, however, the thickness of the mobilized soil mass was estimated from visual observations.

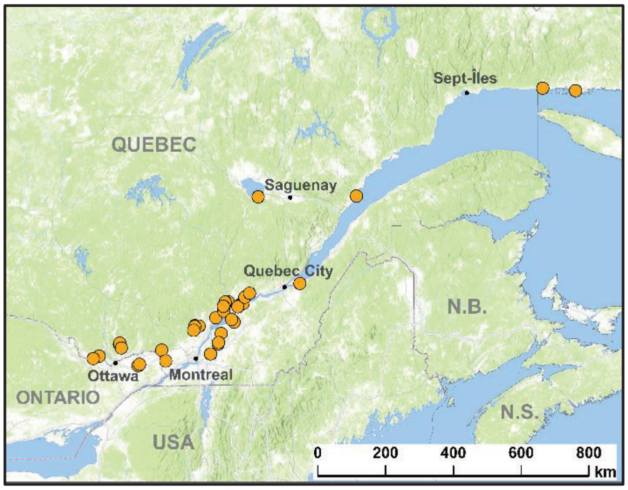

Figure 4. Location of large retrogressive landslides (orange symbols) considered in the present study.

3.1 Identification of landslide type

High resolution Lidar digital elevation models (DEMs) were systematically used to identify the type of large retrogressive landslides (see Demers et al., 2017, for an overview on the use of Lidar DEMs). In addition, geotechnical data and all other available information, including technical reports, historical documents, articles in newspapers, drawings and photographs, were considered to help determine or confirm the failure mode if necessary.

When a landslide scar showed a ribbed pattern on a Lidar DEM, the identification was easy and unambiguous, as illustrated in Figure 5 for the Notre-Dame-de-Lourdes landslide that occurred in the north shore of the St. Lawrence between Montreal and Quebec City (case #16, Table 1). The absence of a ribbed pattern or of a topography with horsts, however, is not a proof that the landslide was a flow and not a spread. In the rich farmlands of eastern Canada lowlands, the hummocky topography in landslide scars is often smoothed by earthworks to reclaim the land for agricultural purposes. A striking example is shown in Figure 6 for a landslide that occurred in 1975 near the town of St-Ambroise-de-Kildare, between Quebec City and Montreal on the north shore of the St. Lawrence River (case #20, Table 1). On a recent Lidar DEM (Fig. 6a), the bottom of the scar shows an even surface, and, in a first analysis, this lack of relief could be associated with a flow. On a vertical air photo taken in the days following the landslide, however, a succession of very well-defined horsts and grabens is clearly seen, indicating that this landslide is in fact a spread (Fig. 6b).

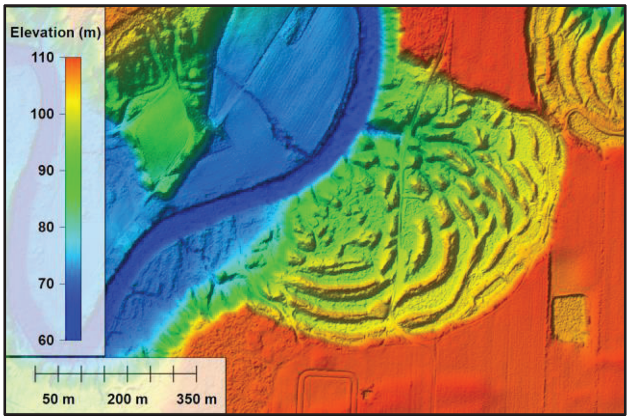

Figure 5. Lidar DEM of a landslide spread (case # 16, Table 1) showing a typical ribbed pattern with alternating horsts (protruding prisms) and grabens (sunken wedges). Part of another spread is visible in the upper right corner of the figure.

When only ambiguous visual information was available, identification was more challenging. In such cases, we considered the thickness of debris inside the scar, which is on average greater for spreads than for flows (Demers et al., 2014). In addition, the shape of the piezocone test profiles was taken into account. As the mobilized sensitive clays in flows can be almost entirely liquefied, the cone tip resistance and pore pressure profiles are typically shifted to much lower values in the debris, an attribute generally not observed in spreads. This is illustrated in Figure 7 for a flow that occurred in the Ottawa region in 2010. For old flows, a crust may have had time to develop in debris and the contrast in the profile shape may be less obvious.

In some landslides, the failure seems to have begun by a flow, which evolved into a spread (Tremblay-Auger et al., 2018). These cases are identified in Table 1 as compound landslides. We emphasize that this is a work in progress and that this classification is based on our current understanding. Some of these landslides could be reclassified in the future into either category as additional data become available or new conceptual models are proposed.

3.2 Determination of the thickness of the crust and of the mobilized soil mass

The determination of the thickness of crusts was most often straightforward. In the few cases where there was a gradual transition with alternating layers of sand and clay between the sensitive clays and a sandy crust, the determination was less easy, and some judgment was required to position the base of the crust.

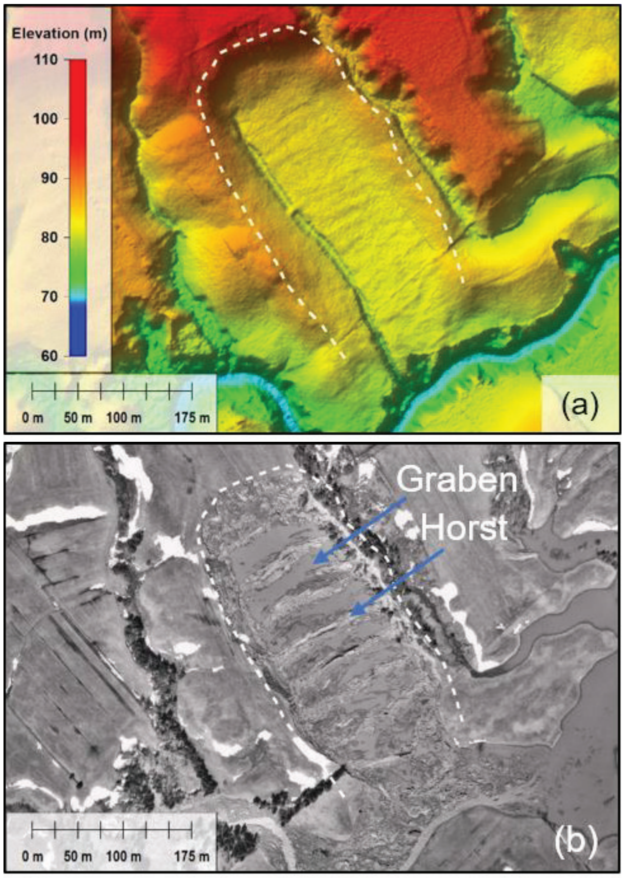

Figure 6. Effect of agricultural earthworks on the ground surface appearance at the site of the 1975 St-Ambroise-de-Kildare spread (case #20, Table 1). (a) Lidar DEM from a survey in 2008, (b) vertical air photo taken in 1975. The dashed white line circumscribes the landslide scar.

As shown in Figure 7, identifying the failure surface was generally not a problem for flows. The identification of the basal failure surface was trickier for spreads because this basal surface can sometimes be mistaken with the inclined interface between a horst and a graben and be positioned at a higher elevation (Demers et al., 2000). When more than one piezocone test was performed inside a scar, the consistency between the tests was used to detect these possible misinterpretations. The thickness of the mobilized soil mass was then estimated for each landslide and at each piezocone test location by reconstructing the pre-failure topography.

4 INFLUENCE OF CRUSTS ON LANDSLIDE DEVELOPMENT

The crust thickness is plotted in Figure 8 as a function of the thickness of the mobilized soil mass for the 37 cases listed in Table 1. For comparison, a few results of numerical simulations taken from the literature are also plotted in Figure 8. Different symbols are used for spreads and flows, and for retrogressive landslides involving these two modes of failure (compound landslides). A symbol without bars or with a bar in one direction means that only one piezocone test was available, either for the crust thickness or for the thickness of the mobilized soil, or for both. The symbols represent the average value in the case where more than two piezocone tests have been considered. Bar lengths illustrate the natural variability in crust thickness at a same site as well as the multi-stepped topography of the failure surface when detected or suspected. However, it cannot be ruled out that some of the variability in the thickness of the mobilized soil mass is due to a misidentification of the basal failure surface for some spreads, as mentioned in the previous section.

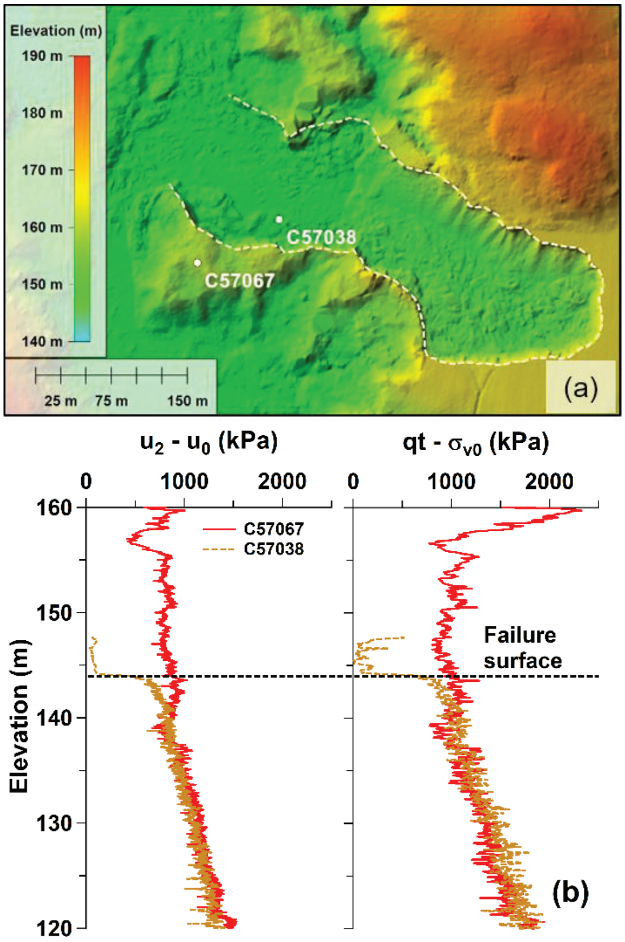

Figure 7. (a) Lidar DEM of the 2010 Notre-Dame-de-la-Salette flow (case #6, Table 1); (b) piezocone tests profiles showing the location of the failure surface. u_0 is the in-situ equilibrium pore pressure and σ_{v0} is the total vertical overburden pressure. The dashed white line in (a) delimits the boundary of the landslide scar.

Data are clustered into two groups, apparently irrespective of the thickness of the mobilized soil mass in each group: a first group with crust thicknesses lower than about 6 m, and a second group with crust thicknesses greater than about 10 m. Symbols for spreads, flows and compound landslides are intermixed with no discernible trend. Although not shown with a different symbol to avoid overloading the figure, all failures involving clay and sandy crusts in the first group overlap (Table 1). In the second group, which comprises spreads except for two cases, the crust consists mainly of sands. Interestingly, a spread in this second group occurred in a deposit with only a very small proportion of sensitive clay (case #16, Table 1). It is also worthwhile to highlight that the two flows in this group developed even if a thick sandy crust of the order of 40-50% of the thickness of the mobilized soil mass was present (cases #2 and 3, Table 1).

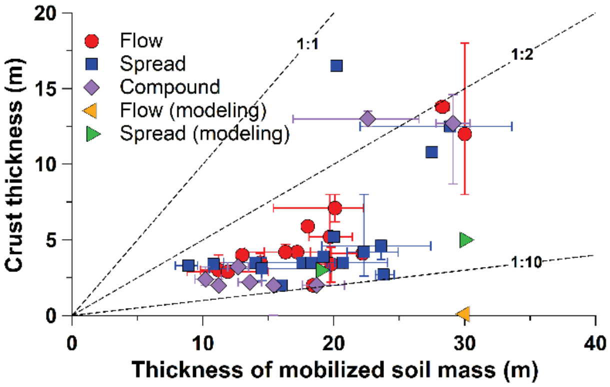

Figure 8. Crust thickness as a function of the thickness of the mobilized soil mass for the 37 landslide cases listed in Table 1. The bounds of horizontal and vertical bars, when displayed, correspond to the minimum and maximum observed values. The green and orange triangular symbols correspond to numerical simulation results (Saha, 2017; Wang and Hawlader, 2017). The different slope lines (1:1, 1:2 and 1:10) indicate the relative proportion of the crust thickness to the thickness of the mobilized soil mass.

5 DISCUSSIONS

As expected, no clear pattern emerges from the distribution of data points in Figure 8. The thickness of a crust, relative or not to the thickness of the mobilized soil mass, does not appear to be a discriminating factor controlling the development of either spreads or flows, or of retrogressive compound landslides in sensitive clay deposits. It was shown in Section 2 that crusts are almost ubiquitous in eastern Canada lowlands and that the nature of the crust, be it a clay crust or a sandy crust, is also probably irrelevant to explain why a landslide of a certain type can occur in a given area. Based on a brief review of field and laboratory observations, it was postulated that this absence of influence could be related to a similar behavior during the development of retrogressive landslides, both clay and sandy crusts sharing some common hydraulic and mechanical characteristics.

The clustering observed in Figure 8 into two apparently distinct groups may be an artefact due to under-sampling. The gap between the two groups would probably be reduced by investigating other landslide sites, particularly in geological environments where sands overlay sensitive clays. For reasons exposed in Section 2.1, it would be extremely unlikely to identify landslide sites with clay crusts much thicker than about 6 m. Incidentally, this is approximately the maximum clay crust thickness observed for the landslides listed in Table 1.

There is no evidence supporting that crusts can be modeled in numerical simulations as a unit having a higher undrained shear strength than the one in the underlying sensitive clays. On this subject, the point in Figure 8 corresponding to the simulation without a crust is unrealistic (but nevertheless useful in terms of parametric analyses), as no documented cases is characterized by an absence of crust. The fact that a flow can be followed by a spread during the same landslide event can be regarded as the best proof that the presence, the nature, and the thickness of a crust do not play a critical role in the development of a specific failure mode.

It is sometimes mentioned that the presence and the thickness of a crust tend to be correlated to the importance of salt leaching in the underlying clays, and indirectly to the mode of failure (e.g. Torrance, 2017). According to these views, leaching from below in response to ascending hydraulic gradients leads to a thin weathered clay crust while leaching from above by the downward infiltration of surface waters leads to a thick weathered clay crust. In the first scenario, the amount of highly sensitive clays with a low remolded shear strength is greater than in the second scenario, which enables the formation of flows. On the contrary, spreads would preferentially occur when a thicker crust is present because the amount of highly sensitive clays is proportionally less important. Although the amount of highly sensitive clays certainly does play a role, the crust thickness is not a good explanatory parameter as illustrated in Figure 8 by the distribution of data points in the first group of case histories.

An implicit assumption made in interpreting Figure 8 is that the investigations performed to characterize landslide sites are representative of prefailure conditions. The availability of high-resolution Lidar DEMs and, in several cases, of air photos taken before the landslides occurred allows for reconstructing the prefailure topography relatively easily. The thickness of the mobilized mass was therefore determined with reasonable accuracy. However, the thickness of the crust was measured outside landslide scars, in areas where the failure did not propagate further, meaning that conditions in these areas may not be representative of the prefailure situation. This is particularly possible in environments showing a high spatial variability of geological facies and of geotechnical properties. Although judgment has been used to exclude piezocone tests with unrepresentative profiles, this is an inherent limitation of such "after the fact" studies.

6 CONCLUSIONS

Among the many factors that may influence or control the occurrence of spreads and flows, results of numerical simulations presented in the literature suggest that the thickness of crusts overlying sensitive clay deposits is a possible discriminating factor. The analysis of the 37 landslide cases examined in this paper indicates that these numerical results are not consistent with field evidence, and that natural processes are probably more complex than currently recognized.

Crusts are present almost everywhere in regions that were inundated by postglacial seas. Based on field and laboratory observations, we argued that clay and sandy crusts behave similarly during large retrogressive landslides due to their hydraulic and mechanical properties, and that there is no obvious reason why they should play a critical role in the development of either flows or spreads, or compound landslides.

Other factors should better explain in which circumstances a flow or a spread can occur. To identify these factors and the way they may interact, numerical simulations with systematic parametric analyses should be pursued in conjunction with detailed field investigations of landslides that have occurred in as many different environments as possible. In particular, the role of the relative amount of potentially liquefiable clays with low remolded shear strength should be investigated in detail.

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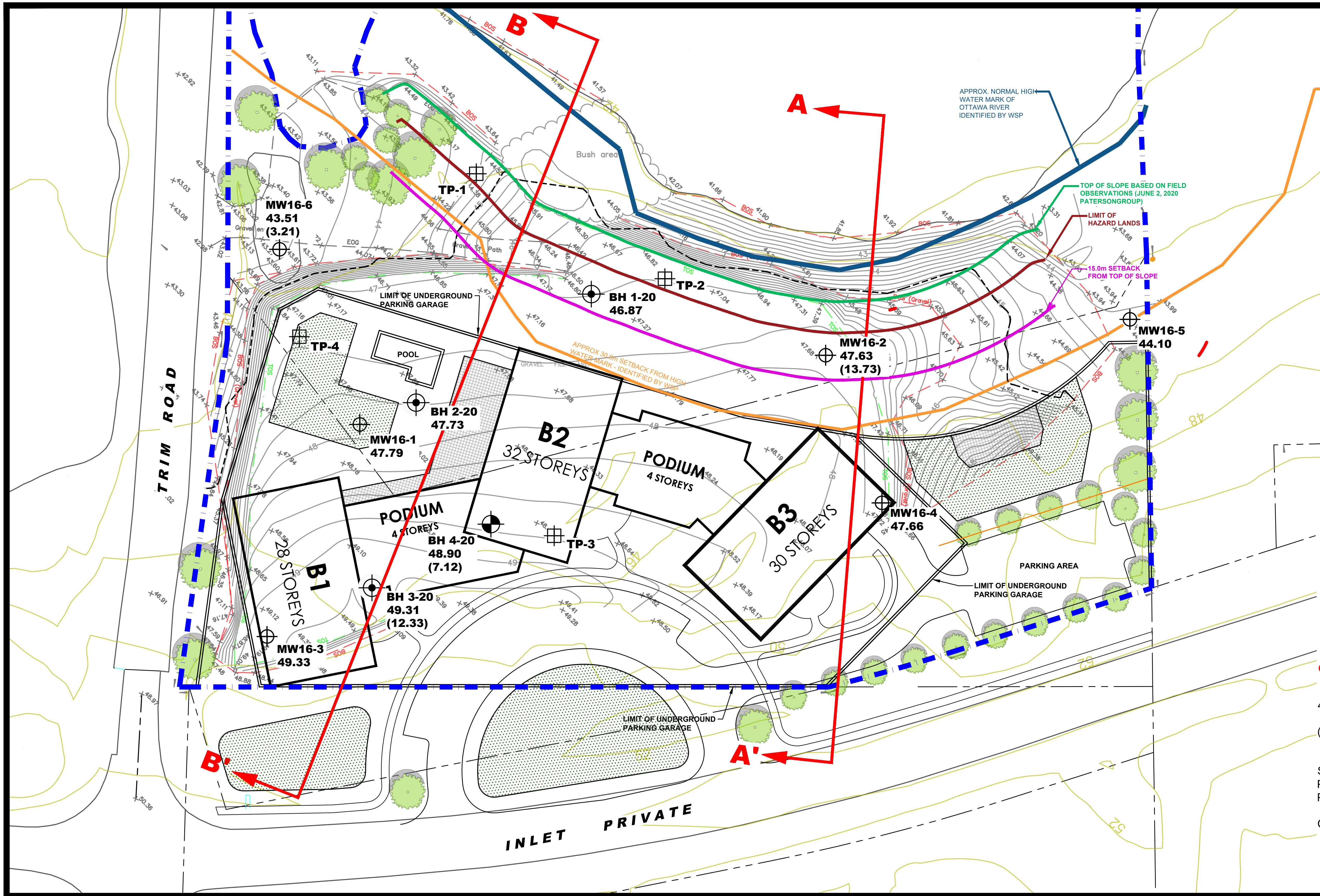
Table 1. Large retrogressive landslides considered for analysis

Site #	Name	Type	Date	Latitude	Longitude	Surface Area[1] (ha)	R[2] (m)	W[3] (m)	Th. Crust[4] (m)	Th. MSM[5] (m)	Crust Type
1	Desbiens	Flow	1989	48.4267	-71.9147	0.9	65	105	2.9	11.9	Clay
2	Havre-St-Pierre	Flow	Unknown	50.2552	-63.5000	13.5	350	540	13.8	28.3	Sand
3	Les Escoumins	Flow	1986	48.4394	-69.3101	1.4	70	140	12.0	30.0	Sand-Clay
4	Maskinongé	Flow	1840	46.2571	-73.0378	29.4	1000	320	3.0	11.2	Clay
5	Notre-Dame-de-la-Salette	Flow	1908	45.7687	-75.5925	6.5	125	460	4.1	22.0	Clay
6	Notre-Dame-de-la-Salette	Flow	2010	45.7941	-75.5837	5.6	425	150	4.2	16.3	Clay
7	Rivière-St-Jean	Flow	1970	50.2978	-64.3717	3.6	235	215	7.1	20.1	Sand
8	Shawinigan South	Flow	Unknown	46.5487	-72.7051	110.0	1400	1200	5.2	19.7	Sand
9	St-Boniface-de-Shawinigan	Flow	Unknown	46.5260	-72.7855	24.0	475	500	2.0	18.4	Clay
10	St-Boniface-de-Shawinigan	Flow	1924	46.5476	-72.7906	3.1	285	140	4.2	17.2	Clay
11	St-Jude	Flow	1954	45.7940	-72.9723	1.7	95	120	4.0	13.0	Sand
12	Ste-Geneviève-de-Batiscan	Flow	1870	46.5095	-72.3681	4.2	90.0	125.0	3.5	14.4	Clay
13	Ste-Marcelline	Flow	Unknown	46.1154	-73.5838	51.1	1160	390	3.4	19.8	Sand-Clay
14	Brownsburg	Spread	1988	45.6642	-74.4718	2.8	75	145	3.1	14.5	Clay
15	Eardley	Spread	Unknown	45.5543	-76.1281	11.8	210	490	3.9	19.2	Sand
16	Notre-Dame-de-Lourdes	Spread	Unknown	46.1074	-73.4783	23.5	410	630	16.5	20.2	Sand
17	Poupore	Spread	1903	45.7043	-75.5403	31.0	675	570	3.5	14.0	Clay
18	Quyon	Spread	~1000 BP[6]	45.5077	-76.2861	23.8	510	550	4.6	23.6	Sand-Clay
19	Rigaud	Spread	1978	45.4639	-74.3654	3.3	85	285	3.5	18.3	Clay
20	St-Ambroise	Spread	1975	46.099	-73.5954	5.7	350	170	4.2	22.3	Sand-Clay
21	St-Barnabé	Spread	2005	46.3802	-72.8239	3.6	110	175	12.5	28.9	Sand
22	St-Boniface-de-Shawinigan	Spread	1996	46.4691	-72.8369	27.7	200	955	10.8	27.5	Sand
23	St-David	Spread	2015	45.9716	-72.8932	0.7	130	80	3.3	8.9	Sand-Clay
24	St-Jude	Spread	1925	45.7790	-72.9757	7.9	105	525	3.5	17.6	Sand-Clay
25	St-Jude	Spread	2010	45.8046	-72.9641	3.8	75	265	2.7	23.8	Sand
26	St-Liguori	Spread	1989	46.0306	-73.6259	7.0	85	500	3.5	20.7	Clay
27	St-Luc-de-Vincennes	Spread	1986	46.4648	-72.4427	6.0	130	305	5.2	20.0	Sand
28	St-Vallier	Spread	1935	46.8841	-70.8096	4.3	230	150	3.4	10.8	Sand
29	Ste-Monique	Spread	1994	46.1785	-72.552	5.8	130	400	2.0	16.0	Sand-Clay
30	Casselman	Compound	1971	45.3767	-75.0992	27.7	400	770	13.0	22.6	Sand
31	Lemieux	Compound	1993	45.4010	-75.0584	16.9	560	275	12.7	29.1	Sand-Clay
32	Mont-St-Hilaire	Compound	1859	45.5920	-73.1772	4.4	240	160	3.2	12.7	Clay
33	Nicolet	Compound	1955	46.2268	-72.6198	2.2	170	110	2.0	11.2	Sand
34	Ste-Geneviève-de-Batiscan	Compound	1939	46.5190	-72.3091	5.3	120	395	2.2	13.6	Sand-Clay
35	St-Luc-de-Vincennes	Compound	2016	46.4612	-72.4529	1.8	145	160	2.4	10.2	Clay
36	St-Prosper	Compound	1953	46.6350	-72.2674	16.0	750	300	2.0	15.4	Sand
37	St-Thuribe	Compound	1898	46.7054	-72.1452	34.0	900	500	2.0	18.7	Clay

[1]Includes the surface area of the horizontally projected pre-landslide slope; [2]Retrogression distance, calculated from the slope crest to the farthest backscarp; [3]Width, calculated along a line normal to the retrogression axis; [4]Average thickness of crust; [5]Average thickness of mobilized soil mass; [6]Before Present.



LEGEND:

⊕ BOREHOLE WITH MONITORING WELL LOCATION

● BOREHOLE LOCATION

⊕ BOREHOLE INSTRUMENTED WITH MONITORING WELL LOCATION (WSP, 161-03361-00, MARCH 2016)

⊞ TEST PIT LOCATION (WSP)

Ⓐ Ⓐ 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

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NO.	REVISIONS	DATE	INITIAL
4	SLOPE STABILITY INFORMATION ADDED	08/19/2020	RG
3	UPDATED TO LATEST CONCEPTUAL PLAN	08/18/2020	RG
2	NEW BOREHOLES ADDED	07/07/2020	RG
1	UPDATED TO LATEST CONCEPTUAL PLAN	05/14/2020	RG

STARWOOD GROUP INC
SLOPE STABILITY REVIEW AND LANDSLIDE RISK ASSESSMENT
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.: 4

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NO.	REVISIONS	DATE	INITIAL
1	UPDATED TO INCLUDED 2020 TEST HOLE INFORMATION	08/19/2020	RG

STARWOOD GROUP INC
SLOPE STABILTY REVIEW AND LANDSLIDE RISK ASSESSMENT
PROPOSED MULTI-STOREY BUILDING COMPLEX
1009 TRIM ROAD

Title:
SURFICIAL GEOLOGICAL MAPPING AND HISTORICAL INFORMATION

Stamp:

Scale: 1:3000
Drawn by: RCG
Checked by: RG
Approved by: DJG
Date: 05/2020

Report No.: PG5336-1
Drawing No.: PG5336-2
Revision No.: 1

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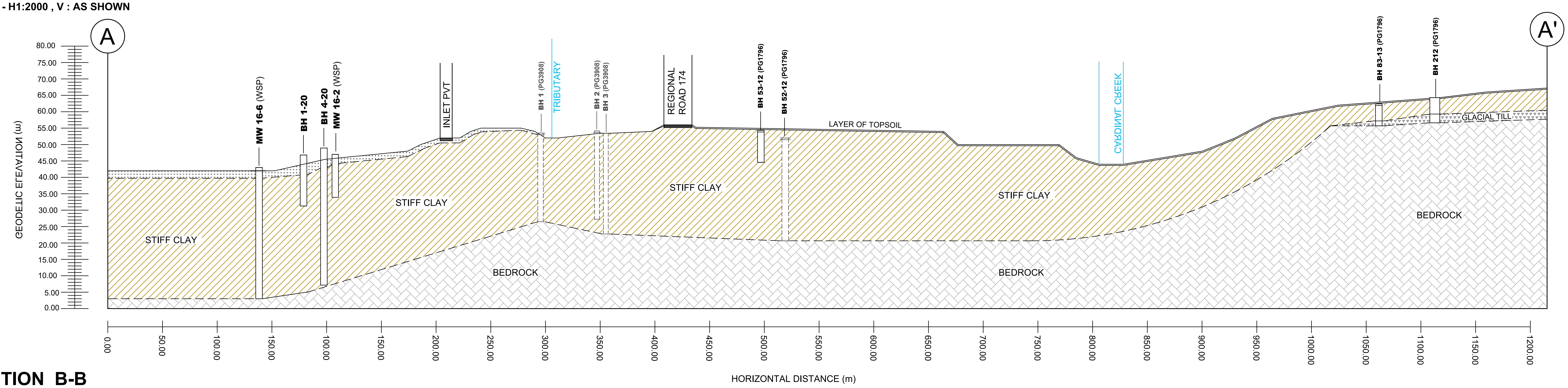
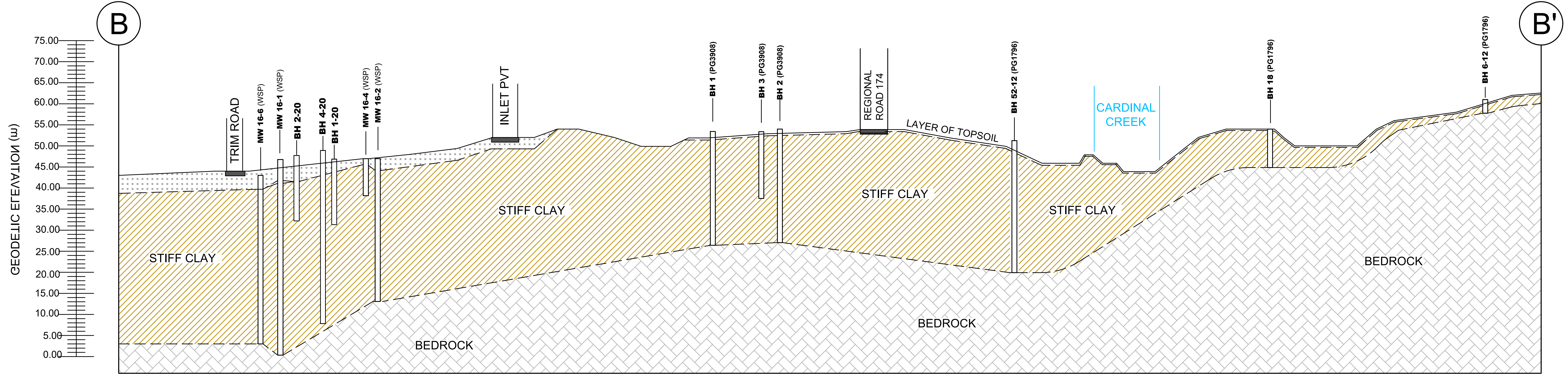
SECTION A-A

SCALE - H1:2000 , V : AS SHOWN



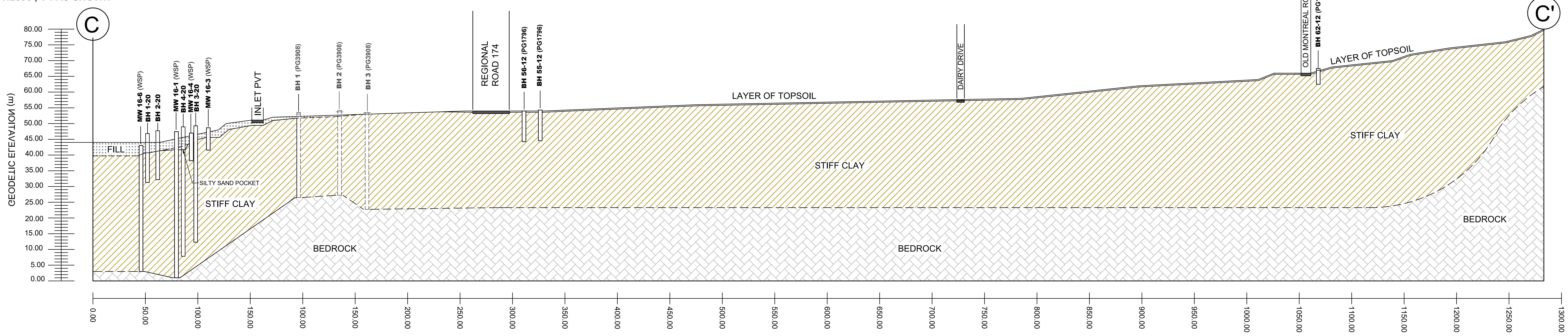
SECTION B-B

SCALE - H1:2000 , V : AS SHOWN



SECTION C-C

SCALE - H1:2000 , V : AS SHOWN



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NO.	REVISIONS	DATE	INITIAL
1	UPDATED TO INCLUDED 2020 TEST HOLE INFORMATION	08/19/2020	RG

STARWOOD GROUP INC
SLOPE STABITLY REVIEW AND LANDSLIDE RISK ASSESSMENT
PROPOSED MULTI-STOREY BUILDING COMPLEX
1009 TRIM ROAD

Title:
SURFICIAL GEOLOGICAL CROSS SECTIONS

Stamp:

Scale: AS SHOWN
Drawn by: RCG
Checked by: RG
Approved by: DJG
Date: 05/2020

Report No.: PG5336-1
Drawing No.: PG5336-3
Revision No.: 1