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Geotechnical Engineering
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July 7, 2023
File: PG5336-LET.03 Revision 1

9378-0633 Quebec Inc.
7 de Tellier
Gatineau, Quebec
J8T 8C2

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) reviewed the landslide hazard assessment report prepared by McQuarrie Geotechnical Consultants Limited dated February 8, 2021 for the proposed multi-storey building complex located at the aforementioned site.

It is understood that since the preparation of the aforementioned report, a fourth tower has been incorporated as part of the proposed multi-storey building complex located at the aforementioned site from the three towers previously considered. Given this addition, an updated evaluation of the landslide risk assessment has been completed and provided herein.

Paterson reviewed the landslide hazard assessment addendum prepared by McQuarrie Geotechnical Consultants Limited dated July 6, 2023 and considering the current proposed multi-storey building at the subject site. It should be noted that the aforementioned addendum updates and expands the previously submitted landslide risk assessment report prepared by McQuarrie Geotechnical Consultants Limited dated February 8, 2021.

Based on our review of the report, Paterson is satisfied with the findings and concur with the conclusions presented in the report. It should be noted that the current report supersedes the previously submitted reports and memorandums which are referred to in the aforementioned revised report and are attached to in Appendix D – *Previously Provided Report and Memorandums* of the present report.

We trust that this information satisfies your requirements.

Best Regards,

Paterson Group Inc.

LICENSED PROFESSIONAL ENGINEER
July 7, 2023
D. J. GILBERT
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July 6, 2023 File: 125-1

Starwood Group Inc.
c/o Paterson Group Inc.
9 Auriga Drive
Ottawa, Ontario K2E 7T9

1015 TWEDDLE ROAD, OTTAWA, ONTARIO
LANDSLIDE HAZARD ASSESSMENT
ADDENDUM

This report is an addendum to the Landslide Hazard Assessment dated February 8, 2021 and addresses the changes to the landslide hazards and risks associated with the addition of a fourth tower to the proposed development.

This report is subject to the attached Statement of General Conditions. These conditions should be clearly understood while reading or interpreting this report.

1 INTRODUCTION & BACKGROUND

McQuarrie Geotechnical summarized the results of a landslide hazard and partial risks analysis in a report dated February 8, 2021. The development plans were amended by adding a fourth residential tower. The purpose of this addendum is to update and expand the landslide risk assessment in light of the addition of the fourth tower. Specifically, the addendum includes:

i. more details regarding the individual risk assessment;
ii. analysis of the societal or group risk; and
iii. mitigation options to reduce the risk “as low as reasonably practicable” (ALARP).

2 PROPOSED DEVELOPMENT

The proposed development by Starwood Group includes four multi-storey residential towers connected by two levels of underground parking that will extend beyond the footprints of each tower and cover a majority of the site. The towers will range from 24 to 32 storeys high for a total of 1,006 one and two bedroom units. The final grade of the main floor is 52.40 m and the lower parking grade will be at 44.90 m elevation.

The existing grade along the south property line is approximately 50.0 m elevation; therefore, the temporary cutslope will be roughly 6.0 m deep, allowing for the depth of the pile caps. The existing grades across the site are highly variable but the parkade will generally result in removal of roughly 2 m of fill, on average. Final grades on the north side of the parkade are expected to be between 45.5 and 47 m elevation (LRL Associates, 2015), requiring landscaping fill outside of the building area ranging from 0 to 2 m thick.

3 RISK ANALYSIS METHODOLOGY

The probability of a landslide was estimated based on the ratio of bank affected by past landslides divided by the total length of bank comprised of sensitive clay. This analysis assumes that all slope and soil parameters are equal throughout the hazard area. Soil type or geology, and terrain conditions were considered in a secondary manner by:

i. only including the terrain mapped as sensitive clay or silt on the surficial geology maps;
ii. only including the terrain on the southwest (Ontario) side of the Ottawa River valley;
iii. only including the active bank of the Ottawa River and the lowermost bank of the proto-Ottawa River.

The older/upper terrace banks of the proto-Ottawa River were excluded because they are higher and have been deeply incised by tributary gullies. These gully banks are more prone to landslides but are not representative of the landslide probability at the subject property because of the vastly different terrain conditions. Similarly, the landslide frequency on the Quebec side of the Ottawa River is higher, mainly due to steeper terrain and many more tributary streams and rivers with deeper banks. Without factoring in the terrain conditions at each of the landslide locations, including the landslides on the Quebec side within the study would result in a bimodal relationship, with a much higher landslide probability on the Quebec side. So while bank height was not a direct factor in the probability analysis, it was considered and included indirectly by being selective of the area used to determine the probability.

Landslides along the upper proto-river terraces would be a factor if analyzing the probability of the subject property being impacted by a landslide initiating along the upper terrace. However, the upper terrace is located at least 1 km south of the subject property; therefore, any landslide would have to travel that far across gentle terrain including single-family residential developments, commercial developments, and the new light rail transit system. The probability of such a landslide reaching the subject property without warning is considered extremely remote.

In a multivariable risk analysis, this base probability would be adjusted using several other parameters. For landslides in sensitive clays, the other parameters would include bank height (as a direct factor), slope angle, and the presence of active toe erosion. Soil strength parameters would also be factored into the probability calculation by considering clay sensitivity, liquidity index, and remoulded shear strength. Unfortunately, such a detailed analysis is impossible without knowing the terrain and soil conditions at each of the past landslides within the study area. Such information is not available; therefore, instead of directly including these factors in the quantitative analysis, the base landslide probability was adjusted higher or lower using judgement by considering the soil strength and slope parameters at the

Tweddle Road site. These other critical factors affecting the landslide probability are described in Section 9 of the original report, and outlined below in Section 4.

4 LANDSLIDE HAZARD ASSESSMENT

4.1 Geology

Landslides in the sensitive clays in the Ottawa area have been found to occur more commonly where a surficial sand layer overlies the clay (Unit 2 on GSC OF352). The project site is mapped as clay without a surficial sand layer (Unit 1), as verified by the bore hole data from site. The base probability of landslide occurrence already takes into consideration the geology by differentiating Unit 1 from Unit 2. Only the Unit 1 polygons were included in the analysis.

4.2 Bank Height & Angle

Higher banks are associated with a much greater landslide occurrence. Various studies referenced in the original report found:

- banks less than 6 m high are rarely associated with landslides;
- modelling shows banks must be at least 10 m high to trigger an earth flow;
- higher banks are associated with larger earth flows.

For retrogressive flow slides, the bank height must be high enough to allow the initial slide debris to exit the depletion zone in order to create the over-steepened headscarp to allow retrogression (unless the bank is actively subject to toe erosion, as discussed below).

The upper slope along the south side of the property was originally between 3 and 5 m high, but has been supported for several decades by fill placed across the site. The local slope hazard maps for Ottawa did not even classify the subject property as being on a slope[1]. This fill will be excavated as part of the underground parkade but the cutslope will ultimately be fully supported by the parkade structure. The temporary excavation will be 5 to 6 m high and will create a short-term risk that should be mitigated. Mitigation measures should focus on maintaining the lateral support to the slope during construction by shoring or other means.

The north embankment above the river will be 3 to 4 m high but most of the fillslope has existed for decades without any instability. The excavation for the parkade will unload most of the property with the increased load limited to the exterior landscaping beyond the parkade footprint.

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[1] Klugman, M.A. and Chung, P. 1976. Slope stability study of the Regional Municipality of Ottawa-Carleton, Ontario Canada. Ontario Geological Survey Miscellaneous Paper MP68.

The permanent slope conditions both to the north and south should result in a much lower landslide probability than the base probability provided the temporary cutslopes are suitably shored or buttressed to prevent the development of any planes of weakness.

4.3 River Bank Erosion

A majority of the landslides in sensitive clays in Canada are triggered (or at least partly caused) by toe erosion. After a landslide has occurred, further erosion can remove the debris accumulation, creating conditions for retrogression.

Petrie Island is in a depositional environment and Tweddle Road further obstructs the river flow and any fluvial erosion along the north side of the subject property. This slope has not eroded in several decades if not longer; therefore, the landslide probability should be much less than the base probability.

4.4 Undrained Shear Strength, Clay Sensitivity & Liquidity Index

Earth flows occur where the remoulded shear strength is 1 kPa or less, the liquidity index is greater than 1.2 to 2.0, and the sensitivity is greater than 16 to 30. Earth spreads may occur where the remoulded shear strength is as high as 1.6 or perhaps even 2.0 kPa.

The site investigations at the project site found:
- The lowest remoulded shear strengths in the test holes by Paterson Group are typically 7 to 10 kPa, while WSP’s test holes found remoulded strengths between 3 and 8 kPa. No remoulded strengths were found to be 2 kPa or less.
- The sensitivities found in the test holes by Paterson Group are typically 10 or less. WSP’s test holes measured sensitivities typically between 10 and 12 but as high as 16.
- WSP measured liquidity indices between 0.66 and 1.62.

The sensitivities and liquidity indices are at the low end of the range associated with landslides in sensitive clays, while the remoulded strengths are much too high for an earth spread, let alone an earth flow. Based on these soil strength parameters, the landslide probability at this site should be significantly less than the base probability.

4.5 Earthquakes

Some of the landslides in the Ottawa area were very likely triggered by large earthquakes. However, the earthquake hazard is ubiquitous wherever the sensitive clay is located. For the most part, the seismically-induced landslide hazard should be affected by the same parameters that create the static landslide hazard (i.e. the same soil and slope conditions described above). Slopes that are marginally stable under static conditions are more likely to fail during an earthquake than slopes with a higher static factor of safety.

At Tweddle Road, the pseudo-static slope stability analysis determined the factor of safety under seismic conditions to be greater than 1.1 when subject to the 1:2,475 year earthquake. The application of limit equilibrium analysis to landslides in sensitive clay has been questioned by some researchers; however, it is still often used to analyze the initiating failure that triggers an earth flow. The stable factor of safety is consistent with both the terrain and soil conditions described above. Even under seismic conditions, the landslide probability should be less than the base probability.

5 RISK ANALYSIS

5.1 Individual Risk

The risk calculation estimates the probability of death to an individual using the following formula:

$$PDI = P(H) \times P(S{:}H) \times P(T{:}S) \times V(L{:}T)$$

where:

PDI	is the annual probability of death to a specific individual.
$P(H)$	is the annual probability of a landslide occurrence.
$P(S{:}H)$	is the probability of spatial interaction with subject property.
$P(T{:}S)$	is the probability of temporal interaction, which is separated into $P(T_R{:}S)$ the probability of someone being in the home at the time of the landslide (i.e. percentage of the day someone is in their home), and $P(T_W{:}S)$ the probability of insufficient warning to allow the occupant to escape.
$V(L{:}T)$	is the vulnerability, specifically the probability of fatality to persons in the building impacted by the landslide.

The values applied in the risk analysis must be reasonable and based on estimates while avoiding inherent and repeated conservatism. To quote Strouth and McDougall [2]:

> Engineers are trained to incorporate conservatism into their design assumptions. However, risk is overestimated when this conservative attitude is applied, perhaps unknowingly and to a number of different inputs, in a risk analysis. Inflated risk estimates are inappropriate for risk evaluation.

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[2] Strouth, A. & McDougall, S. (2022a). Individual risk evaluation for landslides: key details. Landslides 19: 977-991. http://dx.doi.org/10.1007/s10346-020-01547-8.

> Analysts should assess and present uncertainties transparently, while using best estimates for risk evaluation.

The base probability, as explained in Section 3, considered the southeast bank of the Ottawa River from Lower Allumette Lake to the east end of the mapping project downstream of Hawkesbury. Of this more than 240 km of bank, approximately 112 km is mapped as Unit 1 (clay) and less than 2.8 km is mapped as a landslide or crosses a landslide. This accounts for 7 or 8 landslides ranging in width from 100 m to 1,100 m, but more typically 300 to 500 m wide. The percentage of the river bank comprised of Unit 1 that has been directly affected by landslides is 2.5%.

Since some of these landslides were undoubtedly caused by the large earthquake 4550 years BP, the geologic record extends at least that far back and the landslides can be assumed to have occurred over at least that time period, resulting in an annual probability of a large landslide no greater than 1 in 182,000. If the landslide inventory is assumed to represent the full 8000 years of the clay deposit, the probability of a landslide occurring at this site would be estimated to be 1 in 320,000 per annum.

The resulting range of probabilities (1 in 182,000 to 1 in 320,000) is due to the unknown timeframe of the landslide record. Specifically, over what period can landslide scars still be delineated? The original surficial geology maps are from 1976, predating LiDAR; however, most of the study area used to estimate the landslide probability is also included in the recent mapping using LiDAR (GSC OF8600). With the added detail of LiDAR, the landslide record likely extends back the full 8,000 years, justifying a base probability of 1 in 320,000 per annum.

Since the probability already considers the width of the landslides, spatial interaction based on landslide width has already been factored ($P(H) \times P(S_W{:}H)$). Spatial interaction based on landslide length must still be considered. If a landslide occurs along the existing river bank or youngest of the proto-river banks along the south property line, the landslide is assumed to definitely affect the subject property, yielding a probability of spatial interaction $P(S_L{:}H)$ of 1.0. The steep bank along the next proto-river bank is at least 1 km south, far beyond the potential earth flow runout; therefore, such landslides were not considered in the risk analysis ($P(S_L{:}H)$ =0).

The probability of a landslide occurring at this site ($P(H) \times P(S_W{:}H)$) should be adjusted based on the hazard criteria, as discussed in Section 4. The factors critical to landslides in sensitive clay are: clay sensitivity, liquidity index, remoulded strength, bank height and angle, and active toe erosion (or loss of toe support by other means). In the absence of studies specifically relating landslide probability to each factor, adjustments must be based on professional judgement. The conditions at the subject property are positive with respect to all of these factors. None of the studies indicate any measureable hazard where the bank height is less than 6 m, there is no toe erosion, the remoulded strength is above 2 kPa, and the sensitivity is less than

10. Based on the actual site conditions, the estimated landslide probability at this specific site should be adjusted much lower than the base probability. At most, the probability would range between 1:320,000 and 1:500,000 per annum.

The probability of temporal interaction is based on an individual spending 12 hours per day inside their home on weekdays and 16 hours per day on weekends, averaging to $P(T_R{:}S) = 0.55$.

The probability of no warning ($P(T_W{:}S)$) is more complex. Some earth flows have occurred with merely a few hours of warning; however, most investigations of sensitive clay landslides in Canada and Norway describe ample warning signs. Most of the devastating landslides in sensitive clay are preceded by one or more precursor landslides and extensive river erosion, such as the Saint Jude landslide in 2010 and the Saint-Luc-de-Vincennes landslide in 2016. The area of the 1993 South Nation Landslide was evacuated years prior to the landslide due to evidence of pending failure.[3] The June 2022 earth flow in Saguenay, Quebec required evacuation of more than 50 homes. Several homes were lost in the landslide but no one was killed.

The deadliest landslide in Quebec history was the 1971 Saint-Jean-Vianney Landslide where 31 people died and 40 homes were destroyed. The landslide assessment[4] describes large tension cracks developing over a few weeks prior to the landslide, some houses settling 15 to 20 cm, and even cows refusing to go into the fields near the landslide. The main landslide movement began more than 3 hours prior to destruction of the first home. Despite the death toll, many people obviously evacuated the 40 homes as well as the surrounding area. The death toll is most likely due to a lack of knowledge at that time as to the potential for such a large, catastrophic and retrogressive landslide in sensitive clays.

Governments are far more aware of the landslide hazards today than in 1971, as evident by the significant reduction in deaths in the more recent landslides. Even without a formal emergency management system that includes evacuation alerts and evacuation orders, signs of a pending landslide would likely be readily noticed.

The Tweddle Road development will create a large, relatively rigid structure comprising the reinforced concrete parkade and four towers founded on piles. Although the piles cannot be designed to fully resist the landslide movement, they should resist movement enough to form large tension cracks between the foundation walls and the adjacent unreinforced ground. The most likely scenario is that precursor ground movement should be obvious in the hard landscaping and roadways, allowing ample warning to evacuate the buildings. A formal evacuation

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[3] S.G. Evans and G.R. Brooks. 2011. An earthflow in sensitive Champlain Sea sediments at Lemieux, Ontario, June 20, 1993, and its impact on the South Nation River. *Canadian Geotechnical Journal*. **31**(3): 384-394. https://doi.org/10.1139/t94-046

[4] F. Tavenas, J.-Y. Chagnon, and P. La Rochelle. 2011. The Saint-Jean-Vianney Landslide: Observations and Eyewitnesses Accounts. *Canadian Geotechnical Journal*. **8**(3): 463-478. https://doi.org/10.1139/t71-048

system managed by either the regional or provincial government could result in a $P(T_W{:}S)$ possibly as low as zero. In the absence of such a system, and allowing for less warning from some landslides, a reasonable value for $P(T_W{:}S)$ is estimated to be 0.3 (30% of residents fail to evacuate).

For comparison, PDI is also calculated for a $P(T_W{:}S)$ of 1.0, which assumes there is no warning. Since most of the recent landslides provided at least some warning and most residents were able to evacuate even without a government managed alert system, a $P(T_W{:}S)$ of 1.0 is not considered reasonable. However, the calculation is provided as a worst-case scenario merely to demonstrate the effect on PDI.

Vulnerability is equally challenging to estimate. The structure is larger and much more rigid than single-family houses. Considering the bank height and the potential magnitude of a landslide, the probability of any of the towers collapsing is considered to be quite low. However, as a worst-case scenario, this analysis applied FEMA's HAZUS natural hazard analysis tool for a building collapse due to an earthquake, which estimates the number of casualties to be 10% of the occupants.

The above values result in a PDI between 1:19 million to 1:30 million.

PDI = P(H) × P(S:H) × P(T:S) × V(L:T)				
$P(H) \times P(S{:}H)$	$P(T_R{:}S)$	$P(T_W{:}S)$	$V(L{:}T)$	PDI
1 : 320,000	0.55	0.30	0.10	**1 : 19,000,000**
1 : 500,000	0.55	0.30	0.10	**1 : 30,000,000**

Even if a landslide occurs without warning and no residents are able to evacuate prior to the landslide, the PDI would be less than 1 in 5 million per annum.

PDI = P(H) × P(S:H) × P(T:S) × V(L:T)				
$P(H) \times P(S{:}H)$	$P(T_R{:}S)$	$P(T_W{:}S)$	$V(L{:}T)$	PDI
1 : 320,000	0.55	1.0	0.10	**1 : 5,800,000**
1 : 500,000	0.55	1.0	0.10	**1 : 9,000,000**

Regardless, the PDI is several orders of magnitude less than the normal tolerable threshold of 1:10,000 and 1.5 to 2 orders of magnitude less than the more stringent threshold of 1:100,000 used for new structures by the District of North Vancouver. The PDI for this development meets all tolerable risk standards for individual risk.

PDI is the annual probability of death to a specific individual, usually the person most exposed to the hazard. Because PDI is the risk to a specific individual, it does not consider the number of people exposed or threatened by the hazard. Therefore, the increased density of the proposed development from three towers to four towers does not increase the individual risk or PDI.

5.2 Societal or Group Risk

When large groups of people are exposed to a potential landslide, societal or group risk analysis is more applicable. The differences between individual and societal risk analyses are explained by Strouth and McDougall (2022)[5]:

> "In short, individual risk tolerance thresholds are unrelated to, and need to be defined independently from, societal risk tolerance thresholds and reference lines. Individual and societal risk tolerance thresholds originated from different places and have different meanings. Societal risk tolerance thresholds refer to the probability of 'N' fatalities out of a larger population. They do not consider risk to any specific individual. The tolerable probability of one or more fatalities on a societal risk tool is not equivalent to an individual risk threshold."

Societal risk estimates are based on F-N curves that plot the estimated number of fatalities versus the probability of landslide occurrence. When multiple landslide scenarios exist, each is plotted individually to create a series of points. Different landslide scenarios could include different magnitudes of landslides or different structures where the probability of spatial interaction or the vulnerability differ. However, in this situation, only one landslide hazard was considered and all of the occupants of the four towers were considered to be equally exposed.

Using the same variables as described for individual risks in Section 4.1, the probability of a landslide occurrence at this property is estimated to be no greater than 1:320,000 and more likely 1:500,000 per annum.

Based on the total number of units being 1,006 and an average occupancy of roughly 1.5 people per unit, a total building population of 1,500 was assumed. Accordingly, a reasonable estimate of the number of deaths based on the same variables used in the PDI calculation would be 25.

$N_{fatalities} = P(T_R:S) \times P(T_W:S) \times V(L:T) \times N_{exposed}$				
$P(T_R:S)$	$P(T_W:S)$	$V(L:T)$	$N_{exposed}$	$N_{fatalities}$
0.55	0.3	0.10	1,500	**25**

In the unlikely event that a landslide occurs without warning and no residents are able to evacuate prior to the landslide, the number of deaths would be 83.

$N_{fatalities} = P(T_R:S) \times P(T_W:S) \times V(L:T) \times N_{exposed}$				
$P(T_R:S)$	$P(T_W:S)$	$V(L:T)$	$N_{exposed}$	$N_{fatalities}$
0.55	1.0	0.10	1,500	**83**

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[5] Strouth, A., McDougall, S. Individual Risk Evaluation For Landslides: Key Details. *Landslides* 19, 977–991 (2022). Https://Doi.Org/10.1007/S10346-021-01838-8

Both of these points are plotted on the F-N graph on Figure 1. Despite the vastly different estimates on probability and number of deaths, the two points still plot in the middle of the ALARP zone, which demonstrates the broad range of risks represented by this zone.

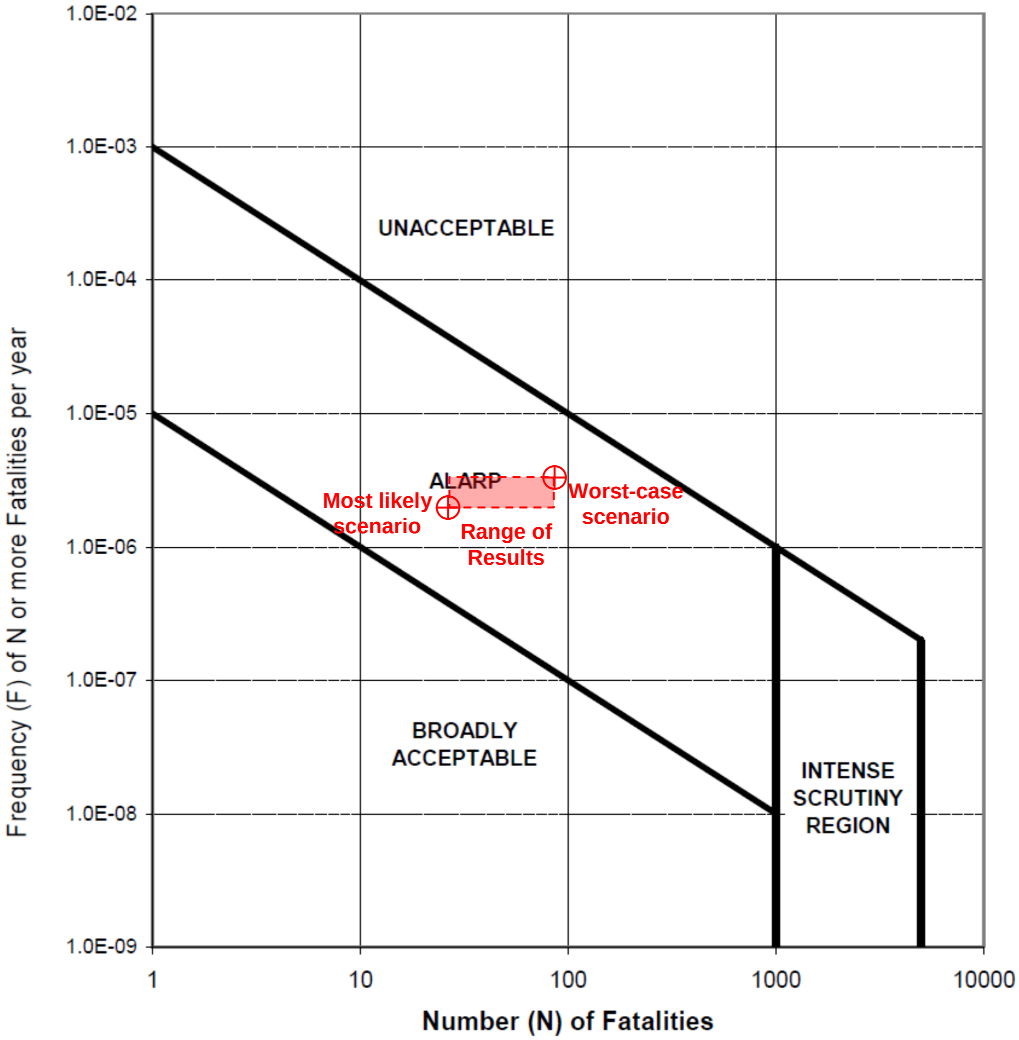

Figure 1: Societal Risk Frequency of Landslide vs Number of Fatalities

6 RISK MITIGATION

A large majority of sites selected for a detailed quantitative risk assessment invariably results in the societal risk plotting within the ALARP zone; the question is then, how much mitigation is necessary to be considered ALARP?[6] The ALARP zone is generally acceptable for development, provided reasonable measures are taken to reduce the risks. However, the costs of the risk reduction measures must be proportionate to the benefits in risk reduction.

6.1 Hazard Avoidance

Avoidance is not a practical mitigation method at this particular site considering the prevalence of the sensitive clay deposit in the Ottawa area. When the site characteristics (i.e. low bank height, lack of river erosion) and soil characteristics (high remoulded strength, low sensitivity) are taken into account, this site has a lower probability of being impacted by a landslide than much of the Ottawa area, particularly for river front property. The regional slope stability map for Ottawa-Carleton identifies this property as being practically the most stable along the Ottawa River east of Ottawa[7] (see Figure 2 of the original report). In this context, the property is relatively favourable and does not warrant avoidance.

6.2 Erosion Mitigation

Given the low probability of a landslide at this site, the most effective means of mitigating the landslide risks would be to prevent any reduction in slope stability resulting from the development. Most importantly, fluvial erosion must be prevented. Without erosion, the river bank is so short that a landslide initiating along the north side of the development is highly unlikely. Therefore, the first mitigation measure would be for a river processes expert to assess the foreshore slope to determine if erosion protection measures are needed and, if so, to design such measures.

6.3 Cutslope Stabilization

Stability of the slope along the south side of the property must be maintained during construction. The proposed cutslope should be designed and supported by shoring to prevent even a small failure that could initiate a larger retrogressive failure.

If an open cutslope is planned, even for a short period of time while shoring is installed, it must proceed sequentially. This could be achieved by initially excavating the cutslope no steeper than 2H:1V, then sequentially excavating panels 3 to 5 m

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[6] Strouth, A., McDougall, S. Societal risk evaluation for landslides: historical synthesis and proposed tools. *Landslides* **18**, 1071–1085 (2021). https://doi.org/10.1007/s10346-020-01547-8

[7] Klugman, M.A. and Chung, P. 1976. Slope stability study of the Regional Municipality of Ottawa-Carleton, Ontario Canada. Ontario Geological Survey Miscellaneous Paper MP68.

wide. Each panel must be fully supported and braced before proceeding with the adjacent panel, and no more than 20% of the panels should be unsupported at any one time. Shoring and bracing prevents the short-term loss of lateral support to the clay cutslope and, therefore, mitigates the landslide hazard along the south bank.

6.4 Increase Lateral Support

This south slope will be supported permanently by the parkade foundation wall. Walls are typically designed for active earth pressure conditions, which allows for minor movement of the wall. To mitigate the landslide risk, the south parkade wall should be design to support at-rest earth pressures, which are higher than active earth pressures. This mitigation measure increases the lateral support provided by the structure and should prevent any movement of the ground behind the wall.

6.5 Monitoring During Construction

Monitoring is the most common method of risk mitigation for landslides in sensitive clay and would be particularly effective in the short-term, during construction. Specifically, 4 or 5 slope inclinometers should be installed near the crest of the excavation and monitored regularly during construction to confirm that no ground movement occurs behind the south parkade wall. If more than 2 to 3 mm of movement occurs at depth (ignoring ice lensing in the upper 2 m), additional shoring support is recommended. The objective of cutslope monitoring would be to prevent the development of conditions that could lead to a landslide, thereby reducing the probability of a landslide.

6.6 Measures Not Considered Practicable

Other than the measures described above, mitigation options for landslides in sensitive clay are limited. The potential magnitude and depth preclude any structural means of stabilization. Increasing the pile size to stabilize the landslide is impossible because the potential depth and magnitude of the landslide would create bending moments that exceed the capacity of even large diameter steel pipe piles filled with concrete. Tie-back anchors to increase the lateral resistance are not an option because of the depth to bedrock, till, or any layer suitable to achieve pullout resistance.

If the river bank was higher, a toe buttress could be effective. However, the foreshore slope is so short and gentle that a toe buttress would be beneficial only to remediate river erosion. Therefore, the need for a toe buttress should be determined from the fluvial erosion assessment.

Long-term monitoring of landslide-prone areas has become the most common mitigation measure for sensitive clay landslides. However, the challenge with long-term monitoring is determining who is responsible for obtaining the information and who analyzes the results. At this particular site, with such a low probability of a landslide, there are many areas along the Ottawa River valley that would benefit much more from a monitoring program. With increased development in the area, a broad-based monitoring program using annual LiDAR surveys and change detection analysis will eventually become more viable and should be considered. The objective of long-term monitoring would be to allow evacuation of the area, thereby reducing the probability of no warning to zero. If $P(T_W{:}S) = 0$, there should be zero deaths.

6.7 Conclusion

The combination of recommended mitigation measures:

i. preventing erosion of the north bank,
ii. shoring and short-term monitoring of the south cutslope, and
iii. increasing the lateral resistance of the south parkade wall,

is considered an effective, reasonable and practicable, approach proportionate to the benefits in risk reduction. The risk reduction associated with these measures cannot be accurately quantified; however, by preventing any movement in the slopes both during construction and in the long-term, it is difficult to foresee how an earth flow or spread could possibly occur. The mitigated probability of a landslide occurrence is estimated to be reduced by 50%, reducing the estimated probability to the order of 1 in 10^6 per annum. With these measures, the risk can be considered to be “as low as reasonably practicable.”

CLOSING

Please contact me if you have any questions regarding this assessment or its conclusions.

McQuarrie Geotechnical Consultants Ltd.
EGBC Permit #1001716

EJ McQuarrie

E. J. McQUARRIE

E. J. McQUARRIE
16420
July 6, 2023

Eric J. McQuarrie, P.Eng., P.Geo. (BC)

APPENDIX A

TEST HOLE DATA

SUMMARY TABLE OF TEST HOLE & LAB TEST RESULTS

TEST HOLE LOCATION PLAN –PATERSON GROUP

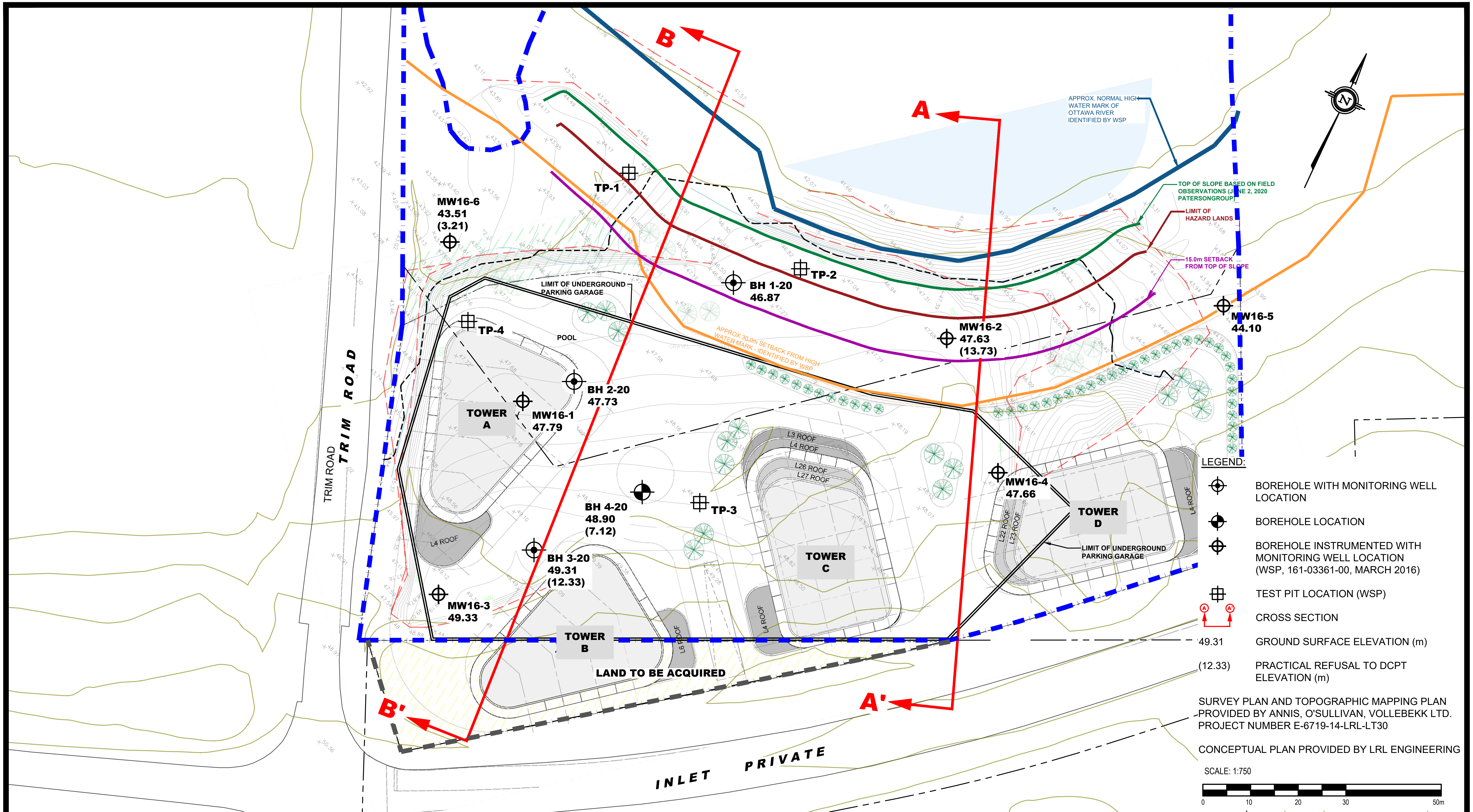
TEST HOLE LOGS – PATERSON GROUP

SYMBOLS & TERMS

TEST HOLE LOGS BY OTHERS (WSP)

SUMMARY OF TEST HOLE & LAB TEST RESULTS

Borehole	Top of Hole Elev. (m)	Gwt Elev. (m)	Fill Thickness (m)	Clay Crust Thickness (m)	Bedrock Depth (m)	Sample Depth (m)	Vane Shear		Sensitivity	Moisture Content %	Atterberg Limits			Liquidity Index
							Peak (kPa)	Remold (kPa)			LL %	PL %	PI %	
1-20	46.87	42.2	6.1	2.2	N/A	7.9	82	7	12					
						8.7	68	10	7					
						9.5	72	10	7					
						10.8	86	10	9					
						12.4	86	15	6					
						14.0	86	15	6					
						15.4	92	15	6					
2-20	47.73	43.2	6.1	2.1	N/A	9.3	72	11	7					
						10.8	76	11	7					
						12.5	76	13	6					
						13.9	64	12	5					
						15.4	96	16	6					
3-20	49.31	44.4	4	4.4	37	6.4	184	33	6					
						7.2	139	28	5					
						7.9	90	13	7					
						9.5	86	7	12					
						14	92	10	9					
						15.4	106	13	8					
4-20	48.9	No Piezo	5.7	3	41.8	8	109	16	7					
						9.5	77	10	8					
						10.3	87	12	7					
						10.9	87	10	9					
						11.6	77	11	7					
						12.4	72	7	10					
						13.9	87	11	8					
						15.5	92	13	7					
16-1	47.3	45.9	6.1	2.3	>47.9	8.0				54.0	53.2	21.0	32.2	1.02
						9.9	70	4	16					
						10.2	70	8	9					
						11.5	70	5	13					
						13.0	75	6	12					
						16.0	75	6	12					
						19.0	75	6	12					
						21.5				68.0	50.4	22.0	28.4	1.62
						22.0	>100							
						25.0	>100							
						31.0				66.0	54.5	23.0	31.5	1.37
16-2	47.2	41.6	2.9	4.7	33.9	5.5				49.0	50.2	22.0	28.2	0.96
						9.9	65	7	10					
						10.2	75	11	7					
						11.0				66.0	52.4	26.0	26.4	1.52
						11.5	80	8	10					
						11.8	80	11	7					
						13.0	80	8	10					
						14.5	85	9	10					
16-3	48.8	43.8	3.1	0	N/A	5.0				44.0	55.3	23.0	32.3	0.65
						6.9	80	11	7					
						7.2	85	21	4					
16-4	47.1	45.1	1.5	4.6	N/A	5.5				50.0	55.3	22.0	33.3	0.84
						7.0	65	7	10					
						7.2	55	8	7					
						8.4	65	7	10					
						8.7	70	8	9					
16-6	43.0	42.3	3.1	0.7	40.3	4.0				64.0	62.3	23.0	39.3	1.04
						4.6	40	3	13					
						4.9	55	6	10					
						6.0	55	6	10					
						6.4	70	8	9					
						7.6	55	5	11	62.0				
						8.0	70	5	15					
						9.2	65	4	16					
						9.5	65	5	12					
						10.7	75	6	12	65.0				
						11.0	75	6	12					
						12.2	75	11	7	70.0				
						12.5	75	8	9					
						13.7	85	11	8	60.0				
						14.0	90	18	5					



LEGEND:

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

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

CONCEPTUAL PLAN PROVIDED BY LRL ENGINEERING

SCALE: 1:750

0 10 20 30 50m

patersongroup
consulting engineers

154 Colonnade Road South
Ottawa, Ontario K2E 7J5
Tel: (613) 226-7381 Fax: (613) 226-6344

NO.	REVISIONS	DATE	INITIAL
5	UPDATED TO LATEST CONCEPTUAL PLAN	10/12/2021	JV
4	SLOPE STABILITY INFORMATION ADDED	08/19/2020	RG
3	UPDATED TO LATEST CONCEPTUAL PLAN	08/18/2020	RG
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.: 5

p:\autocad drawings\geotechnical\pg53xx\pg5336\pg5336-1 thlp rev5.dwg

SOIL PROFILE AND TEST DATA

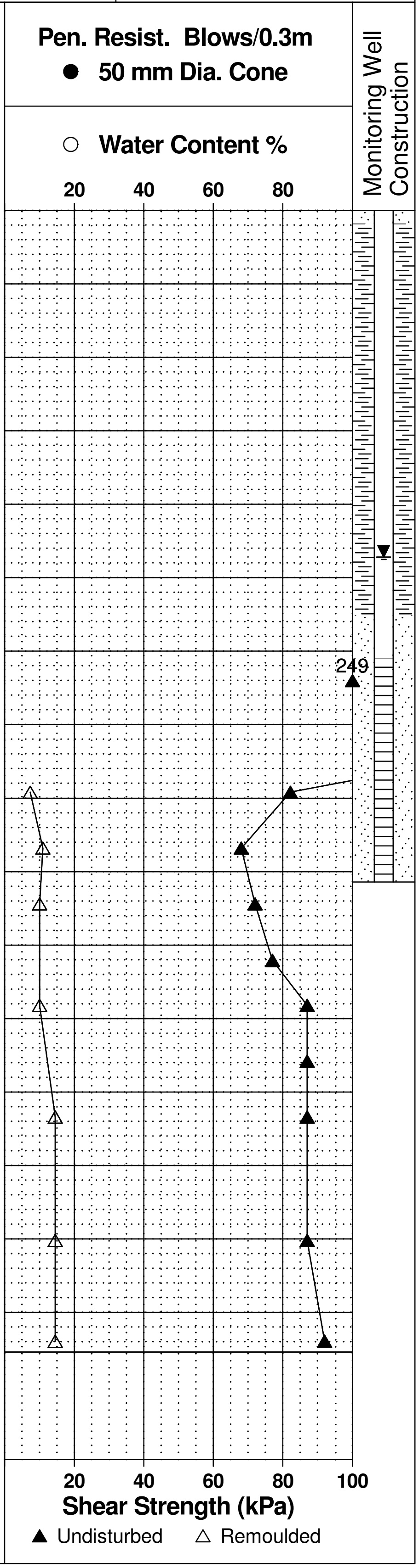
Geotechnical Investigation
Prop. Multi-Storey Building Complex - 1009 Trim Road
Ottawa, Ontario

DATUM Geodetic
REMARKS
BORINGS BY Track-Mount Power Auger
DATE June 29, 2020
FILE NO. PG5336
HOLE NO. BH 1-20

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

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

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



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SOIL PROFILE AND TEST DATA

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

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

DATE June 30, 2020

FILE NO. PG5336

HOLE NO. BH 2-20

SOIL DESCRIPTION

GROUND SURFACE

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

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

- some wood from 3.8 to 4.1m depth

6.10

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

- stiff and grey by 8.2m depth

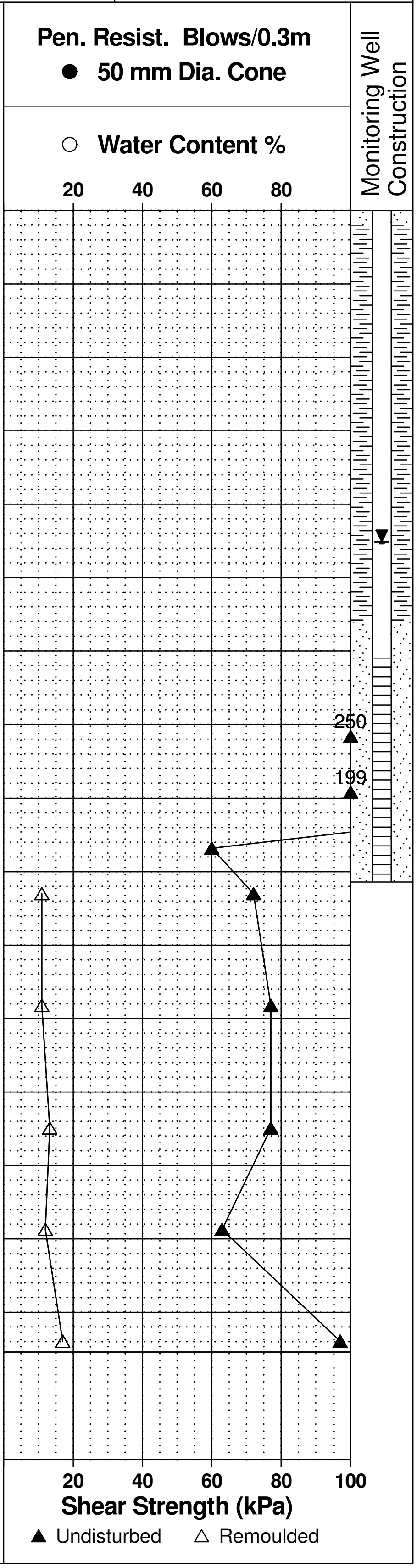
15.54

End of Borehole

(GWL @ 4.51m - July 17, 2020)

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

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



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SOIL PROFILE AND TEST DATA

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

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

DATE July 2, 2020

FILE NO. PG5336

HOLE NO. BH 3-20

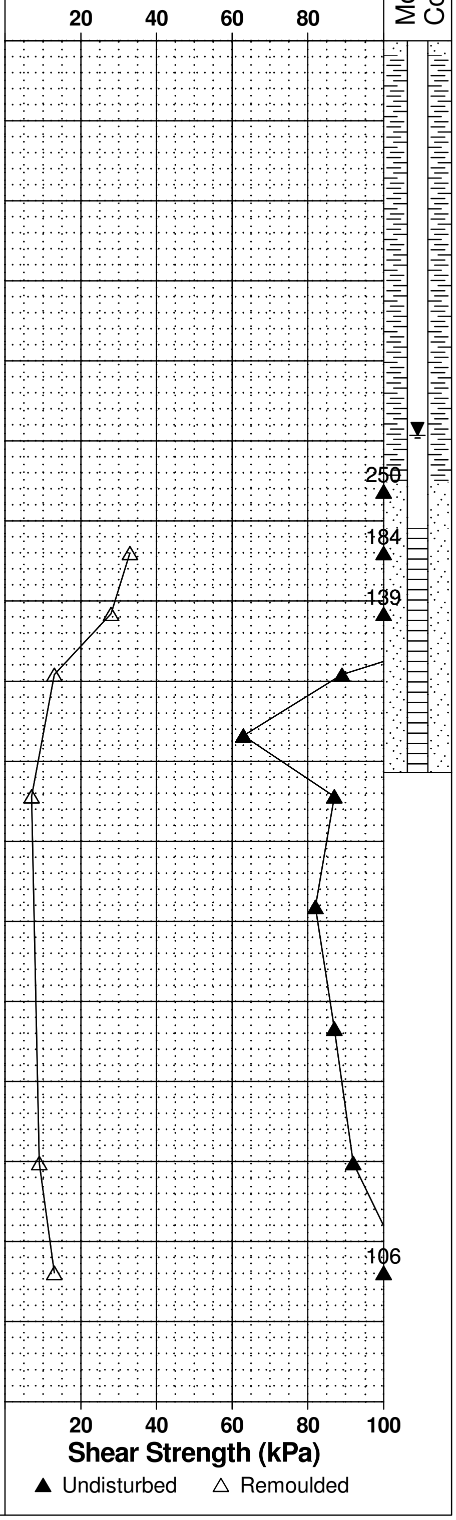
SOIL DESCRIPTION	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE				
FILL: Brown silty sand with crushed stone, occasional cobbles, boulders and blast rock	AU	1		
	SS	2	33	50+
1.52				
FILL: Brown sity clay, trace sand, gravel and wood	SS	3	0	10
	SS	4	58	3
	SS	5	83	18
3.96				
Hard to very stiff, brown **SILTY CLAY**	SS	6	75	19
	SS	7	92	16
	SS	8	92	P
	SS	9	92	P
	SS	10	96	P
	SS	11	92	P
- stiff and grey by 8.4m depth	SS	12	96	P
	SS	13	100	P
	SS	14	96	P
15.54				
Dynamic Cone Penetration Test commenced at 15.54m depth. Cone pushed to 31.7m depth.				

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

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

○ **Water Content %**

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

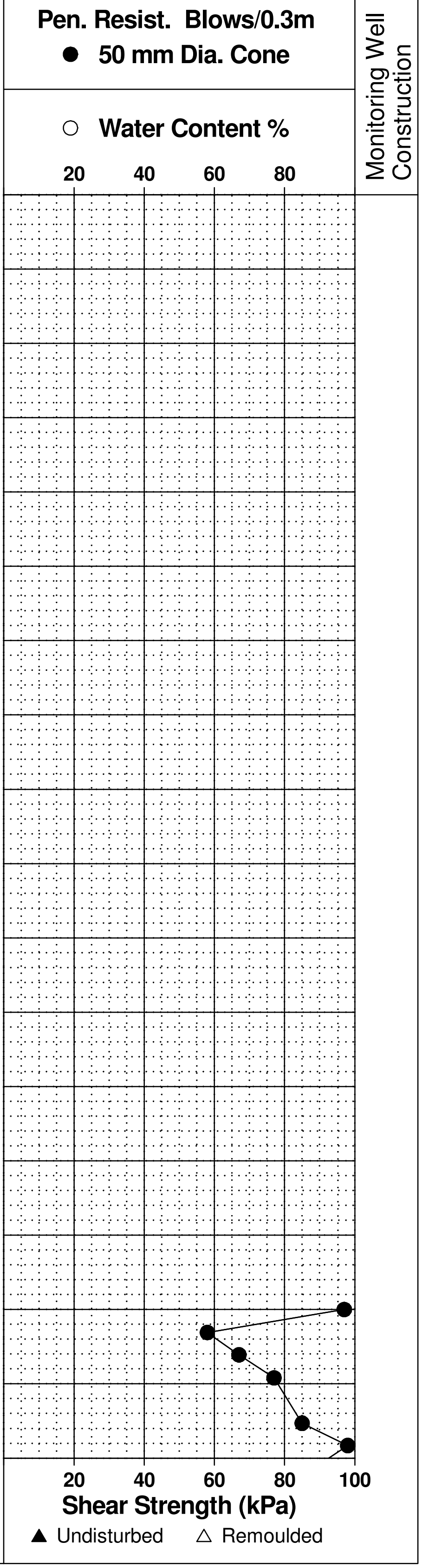
FILE NO. PG5336

REMARKS

HOLE NO. BH 3-20

BORINGS BY Track-Mount Power Auger **DATE** July 2, 2020

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



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SOIL PROFILE AND TEST DATA

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

DATUM Geodetic

REMARKS

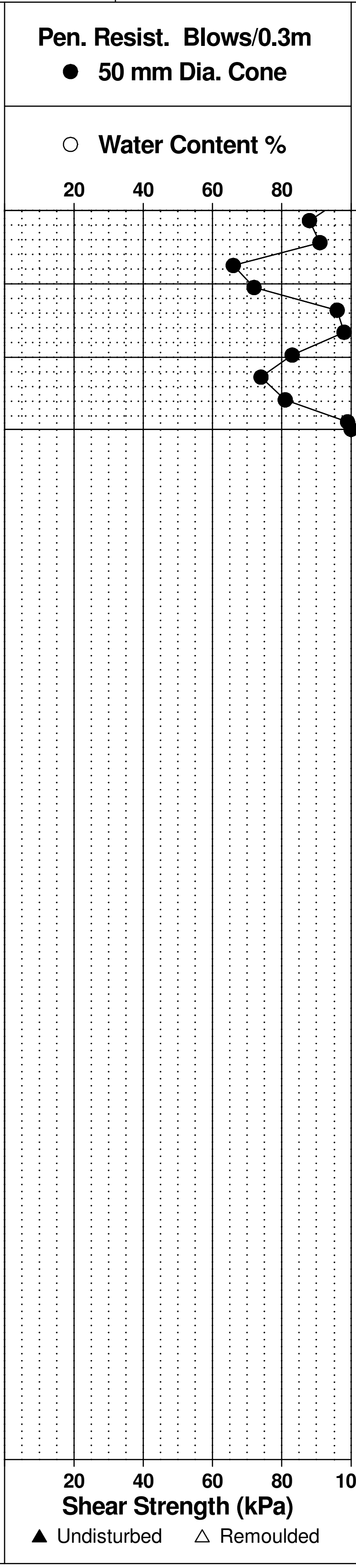
BORINGS BY Track-Mount Power Auger

DATE July 2, 2020

FILE NO. PG5336

HOLE NO. BH 3-20

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



Monitoring Well Construction

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SOIL PROFILE AND TEST DATA

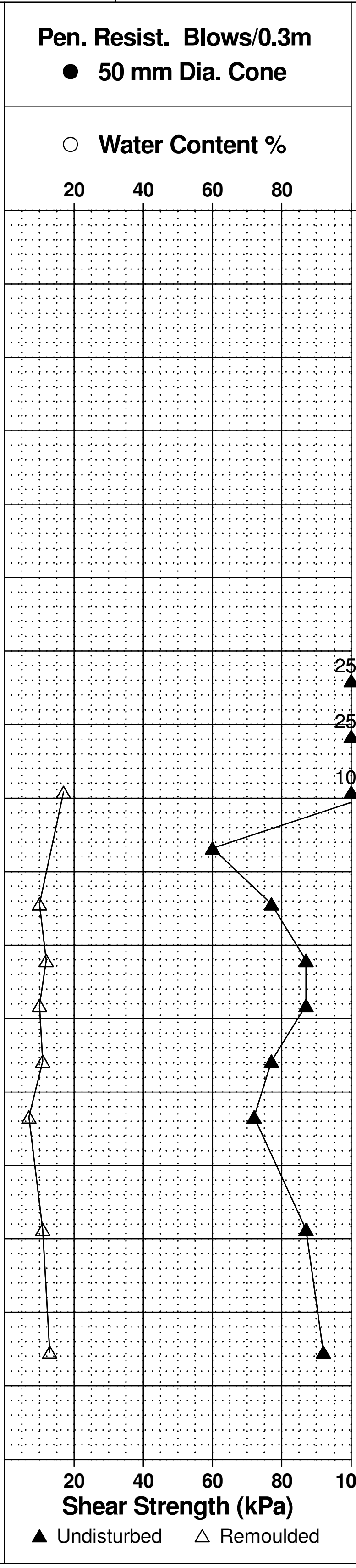
Geotechnical Investigation
Prop. Multi-Storey Building Complex - 1009 Trim Road
Ottawa, Ontario

DATUM Geodetic
REMARKS
BORINGS BY Track-Mount Power Auger
DATE June 30, 2020
FILE NO. PG5336
HOLE NO. BH 4-20

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

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

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



Monitoring Well Construction

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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
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD				
GROUND SURFACE						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

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SOIL PROFILE AND TEST DATA

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

DATUM Geodetic

REMARKS

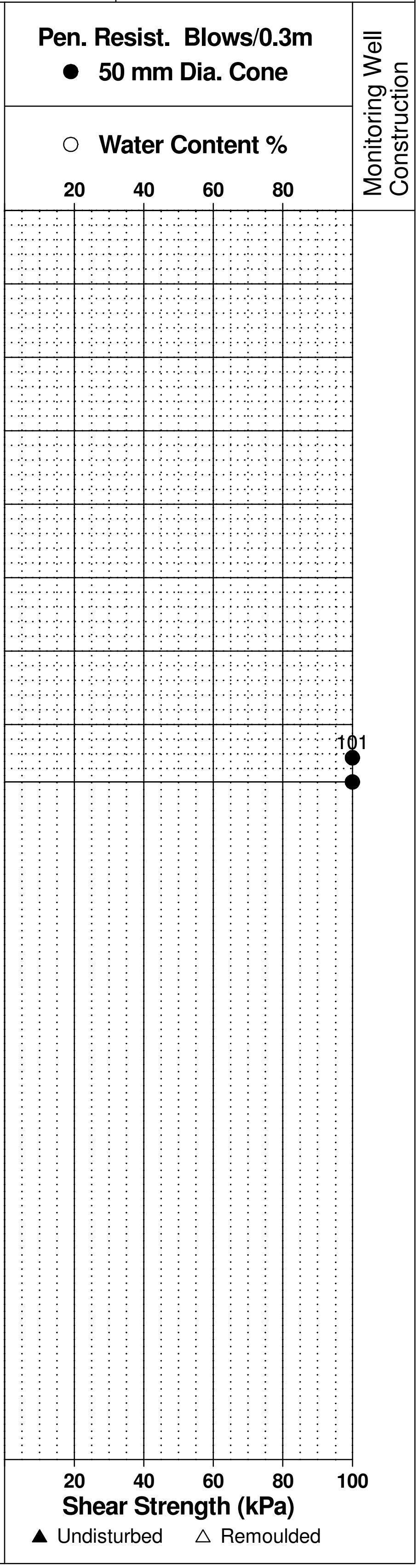
BORINGS BY Track-Mount Power Auger

DATE June 30, 2020

FILE NO. PG5336

HOLE NO. BH 4-20

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

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

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

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

SYMBOLS AND TERMS (continued)

SOIL DESCRIPTION (continued)

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

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

ROCK DESCRIPTION

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

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

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

SAMPLE TYPES

SS - Split spoon sample (obtained in conjunction with the performing of the Standard Penetration Test (SPT))
TW - Thin wall tube or Shelby tube, generally recovered using a piston sampler
G - "Grab" sample from test pit or surface materials
AU - Auger sample or bulk sample
WS - Wash sample
RC - Rock core sample (Core bit size BQ, NQ, HQ, etc.). Rock core samples are obtained with the use of standard diamond drilling bits.

SYMBOLS AND TERMS (continued)

PLASTICITY LIMITS AND GRAIN SIZE DISTRIBUTION

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

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

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

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

CONSOLIDATION TEST

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

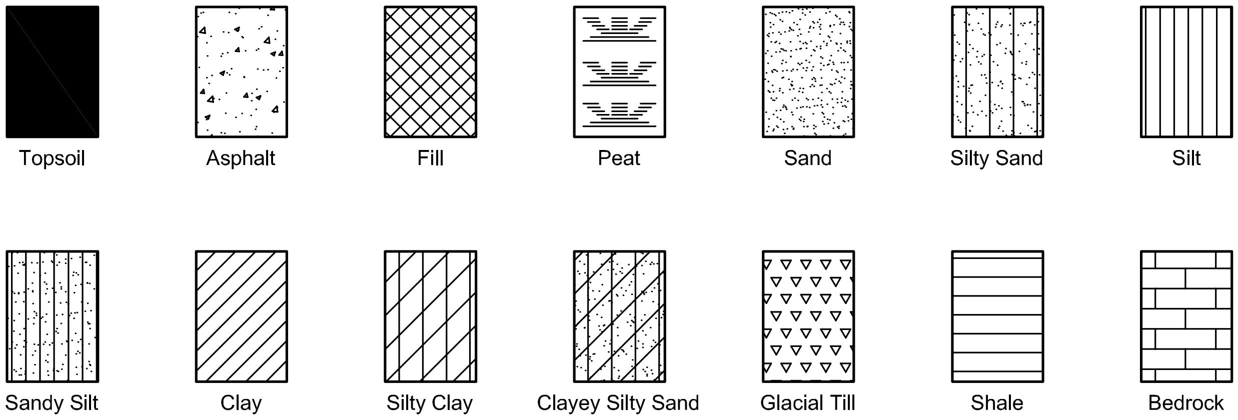
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

PERMEABILITY TEST

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

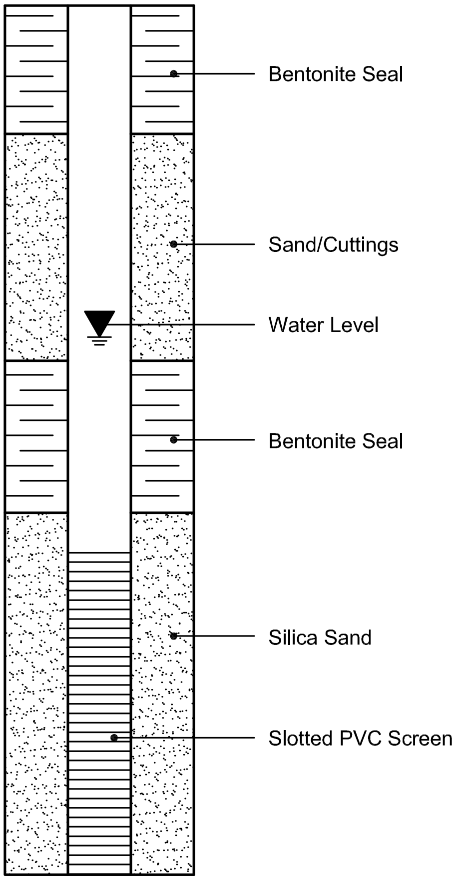
SYMBOLS AND TERMS (continued)

STRATA PLOT

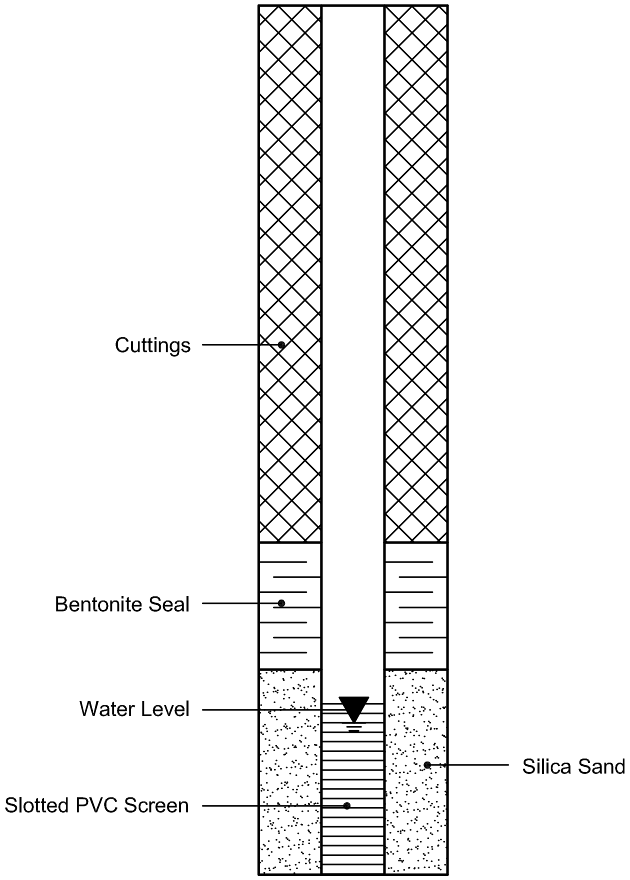


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



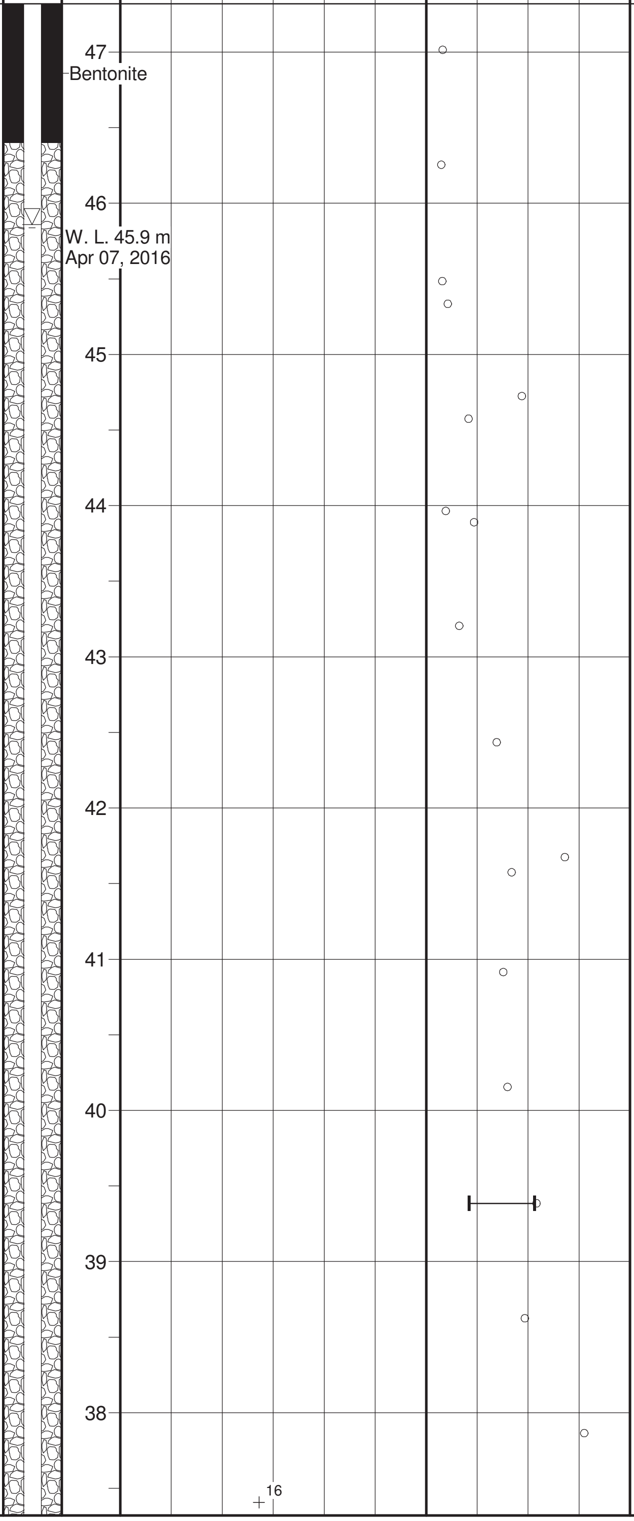
LOG OF BOREHOLE MW16-1

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:

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

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

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)



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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

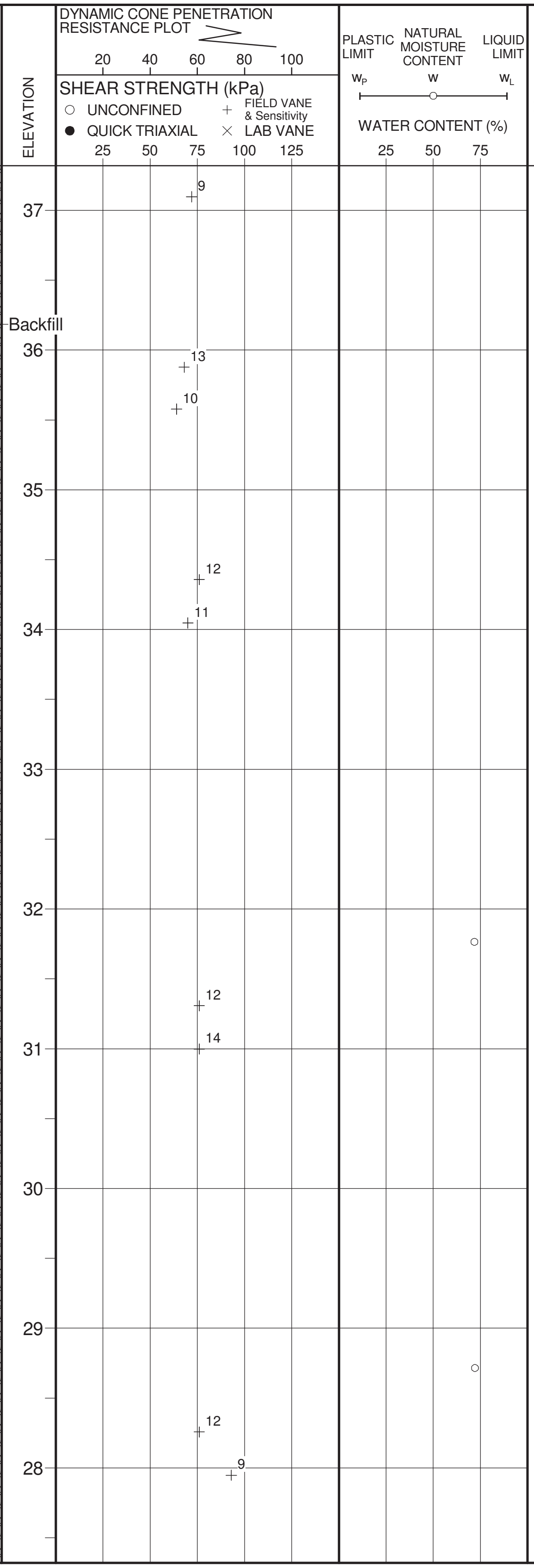
LOG OF BOREHOLE MW16-1

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

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

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

SOIL PROFILE (m) ELEV DEPTH	DESCRIPTION	STRATA PLOT	SAMPLES NUMBER	TYPE	"N" BLOWS 0.3 m	GROUND WATER CONDITIONS
	SILTY CLAY grey, wet, stiff(Continued)			VANE		Backfill
			14	TW		
				VANE		
				VANE		
			15	TW		
				VANE		
				VANE		
			16	SS	WH	
				VANE		
				VANE		
			17	SS	2	
				VANE		
				VANE		



POCKET PEN. (Cu) (kPa)	NATURAL UNIT WT (KN/m³)	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

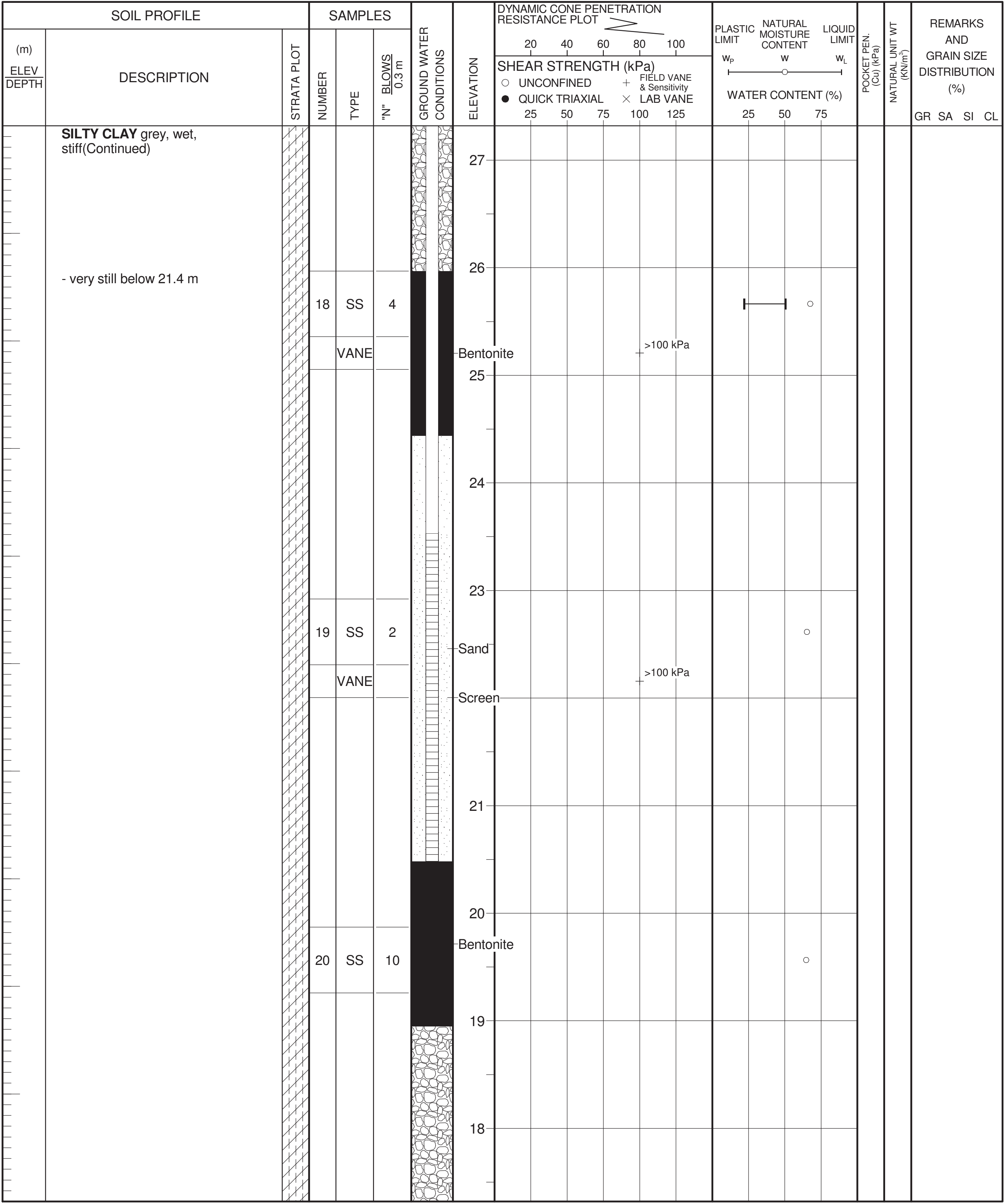
Continued Next Page

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

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

LOG OF BOREHOLE MW16-1

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



SILTY CLAY grey, wet, stiff(Continued)

- very still below 21.4 m

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

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

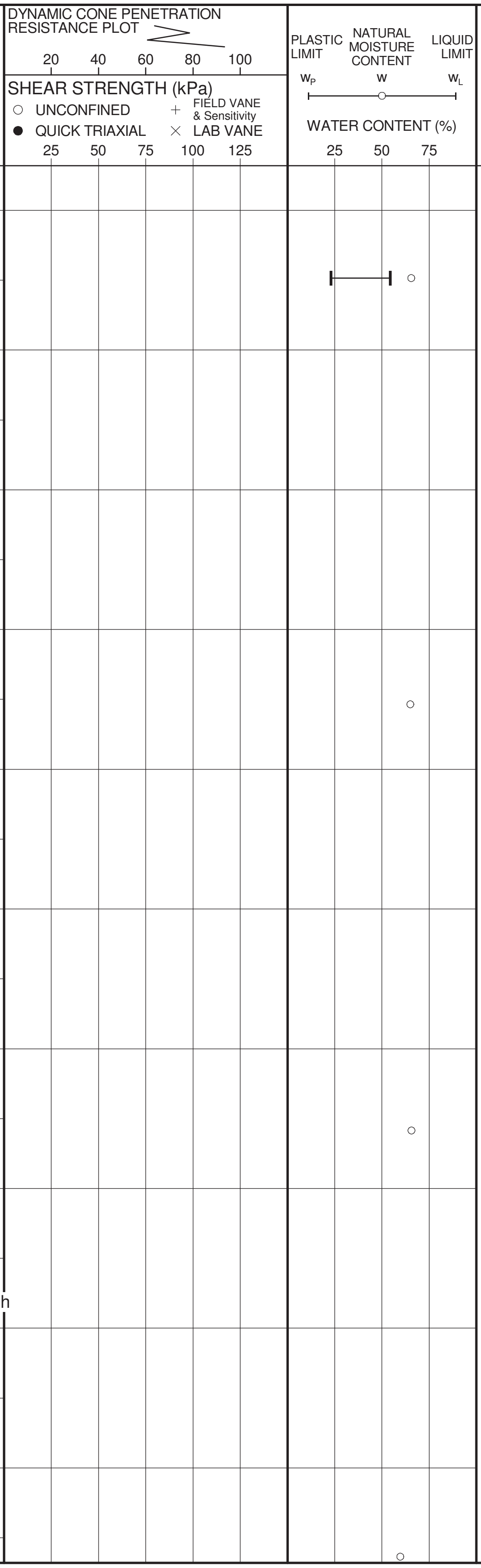
LOG OF BOREHOLE MW16-1

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	ELEVATION	POCKET PEN. (Cu) (kPa)	NATURAL UNIT WT (KN/m³)	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	SILTY CLAY grey, wet, stiff(Continued)						17			
			21	SS	10					0 0 32 68
							16			
							15			
							14			
			22	SS	7					
							13			
							12			
							11			
			23	SS	11					
							10			
						Slough	9			
							8			



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GROUNDWATER ELEVATIONS

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

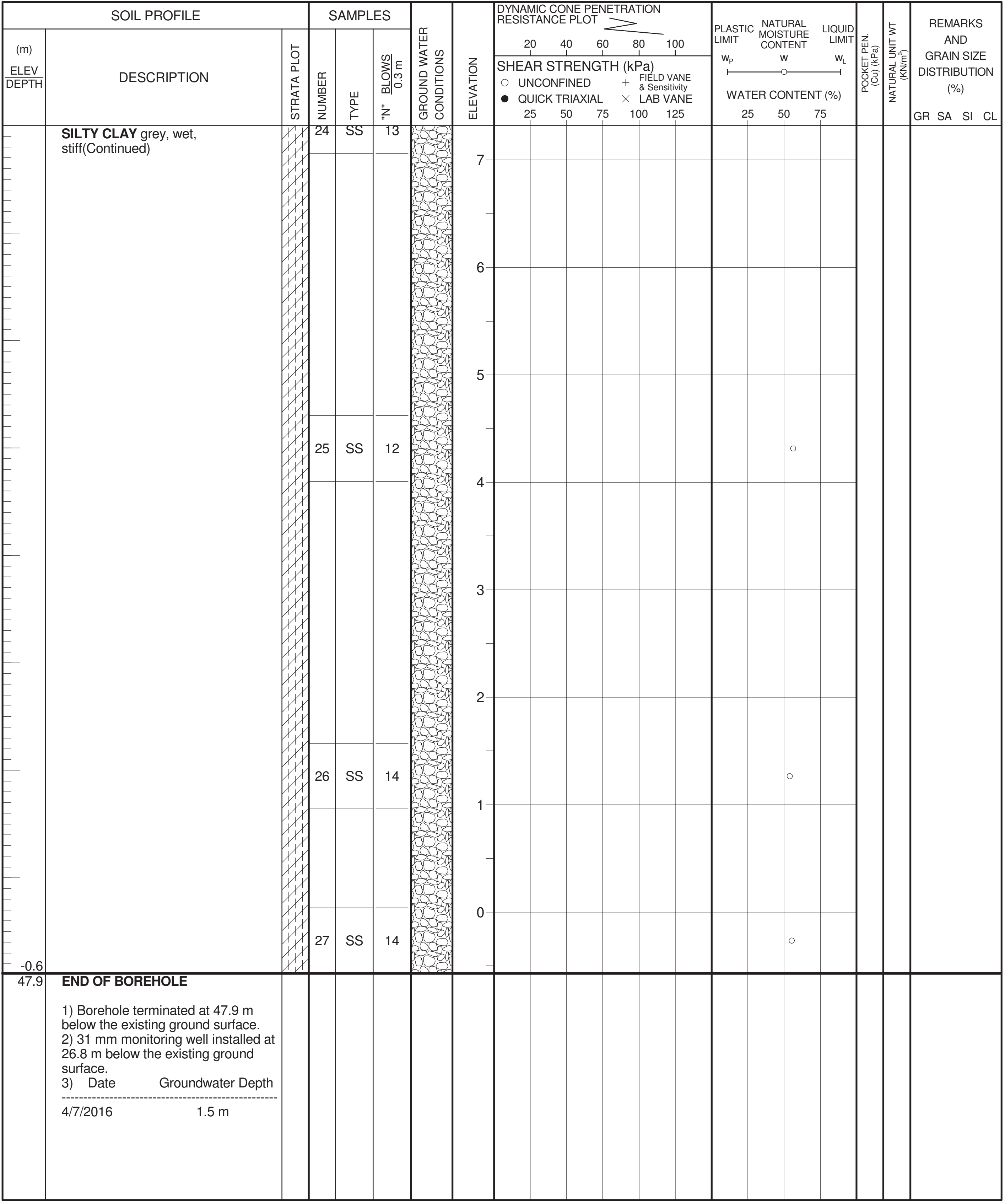
GRAPH NOTES + 3, × 3: Numbers refer to Sensitivity ○ ε=3% Strain at Failure

LOG OF BOREHOLE MW16-1

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

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

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



(m) ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" BLOWS 0.3 m
	SILTY CLAY grey, wet, stiff(Continued)	24	SS	13
		25	SS	12
		26	SS	14
-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 $+^3, \times^3$: Numbers refer to Sensitivity ○ ε=3% Strain at Failure

Sheet No. 5 of 5

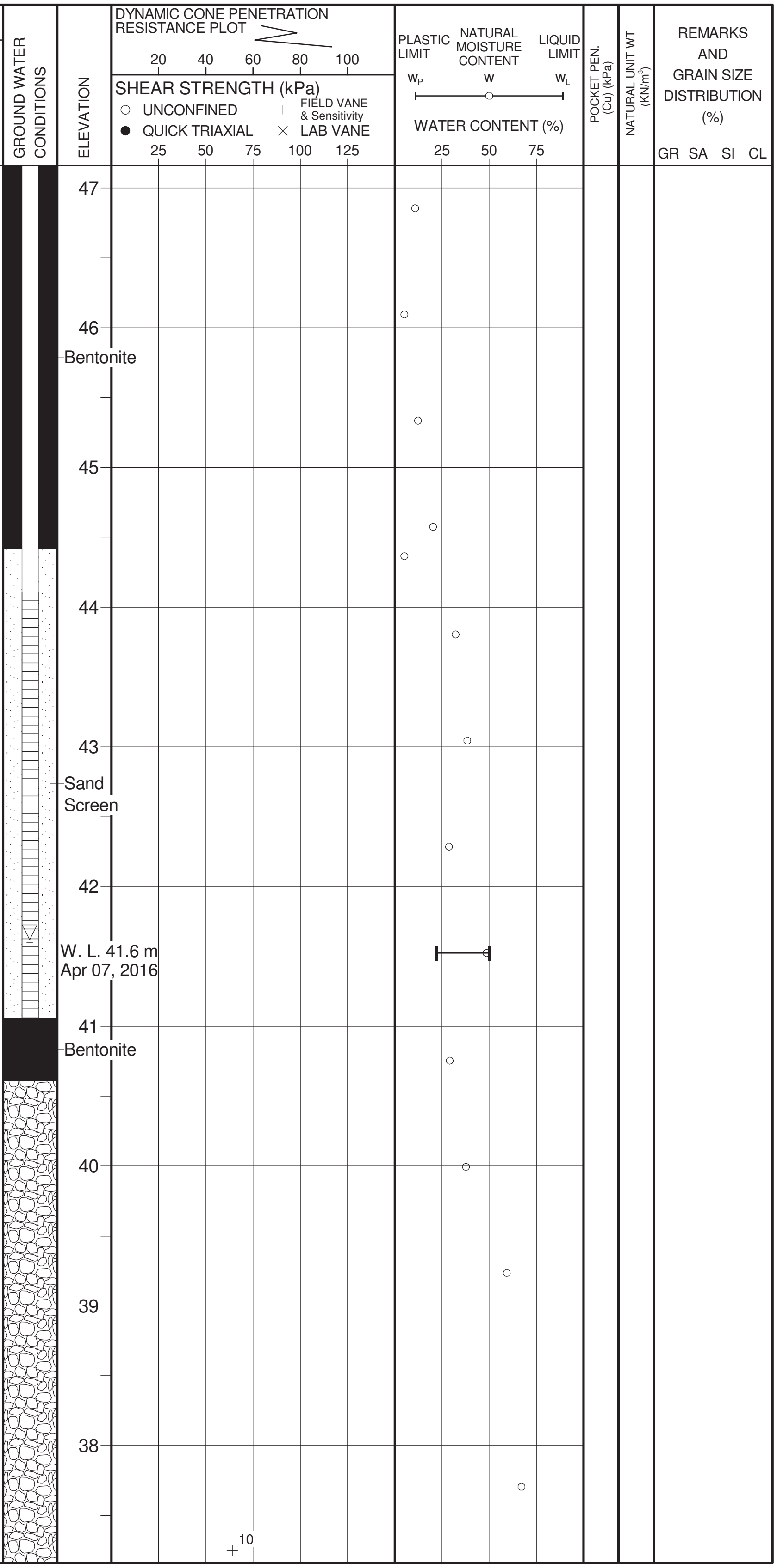
LOG OF BOREHOLE MW16-2

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		
(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
		3	SS	11
	- grey	4A	SS	11
44.5 2.7	**GRAVEL**: black, mosit (FILL)	4B		
44.3 2.9	**SILTY CLAY**: grey brown, firm to very stiff, moist to wet, (WEATHERED CRUST)	5	SS	12
		6	SS	10
		7	SS	16
		8	SS	6
		9	SS	21
		10	SS	12
39.5 7.6	**SILTY CLAY**: grey, wet, stiff	11	SS	2
		12	SS	1
			VANE	



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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 MW16-2

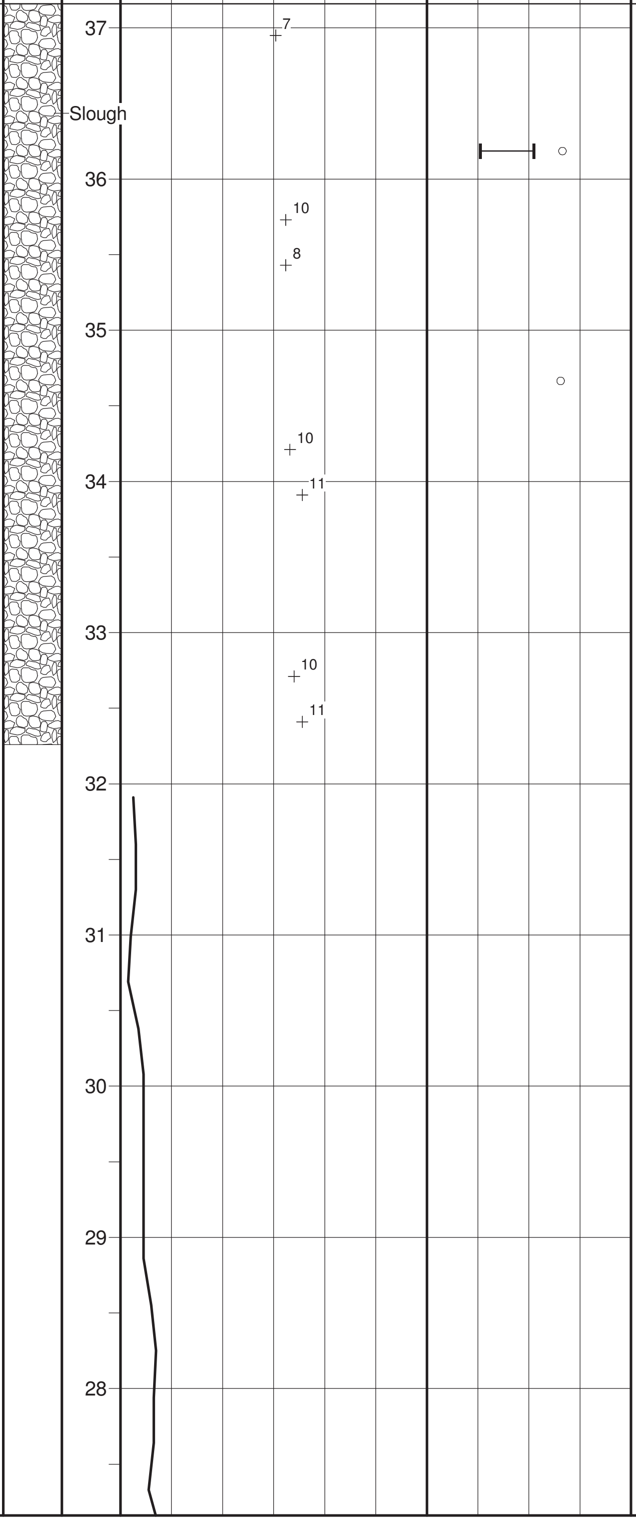
Project: Geotechnical Investigation
Client: Grandmaître Family
Project Location: Part Lot 30, Concession 1, Parts 1 & 2, Cumberland, ON
Datum: Approximate
BH Location: See borehole location plan N 462330 E 5038430

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			REMARKS AND GRAIN SIZE DISTRIBUTION (%)
(m) ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" BLOWS 0.3 m	GR SA SI CL
	SILTY CLAY: grey, wet, stiff(Continued)		VANE		
		13	SS	WH	0 0 48 52
			VANE		
			VANE		
		14	SS	WH	
			VANE		
			VANE		
			VANE		
			VANE		
32.3 14.9	**SILTY CLAY**(Inferred based on DCPT results)				

GROUND WATER CONDITIONS | ELEVATION | DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) | SHEAR STRENGTH (kPa): ○ UNCONFINED, ● QUICK TRIAXIAL, + FIELD VANE & Sensitivity, × 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)



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

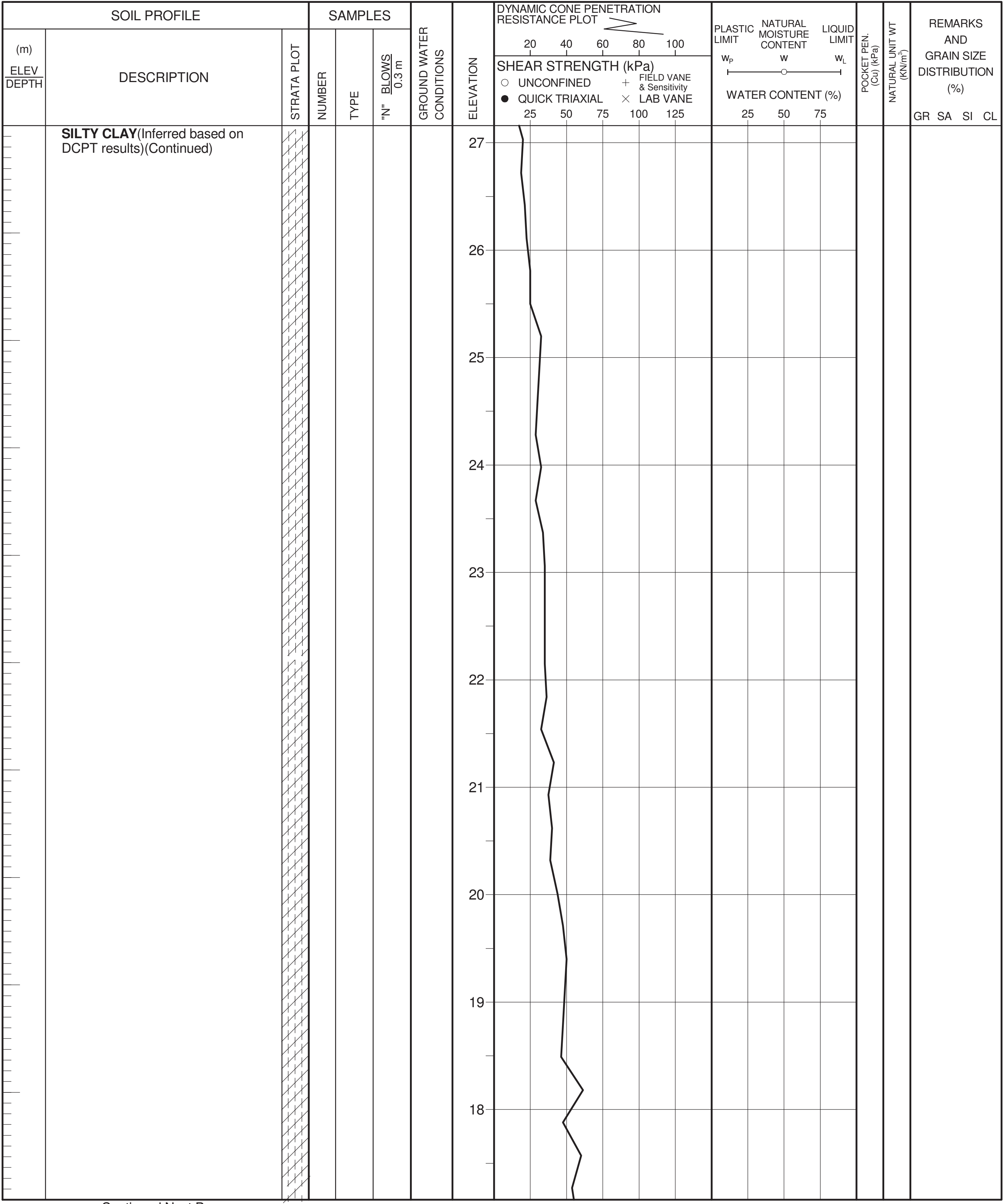
GRAPH NOTES + 3, × 3: Numbers refer to Sensitivity   ○ ε=3% Strain at Failure

LOG OF BOREHOLE MW16-2

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:



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GROUNDWATER ELEVATIONS

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

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

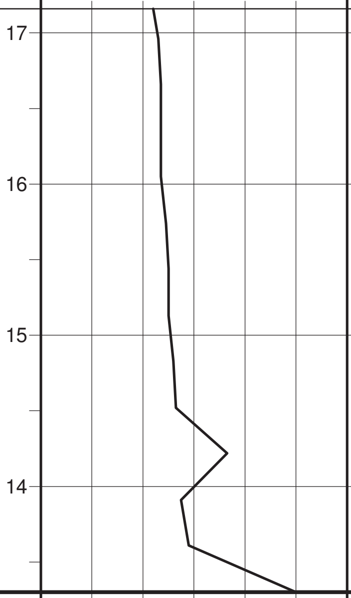
LOG OF BOREHOLE MW16-2

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/m^3)	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

Date	Groundwater Depth
4/7/2016	5.5 m

GROUNDWATER ELEVATIONS

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

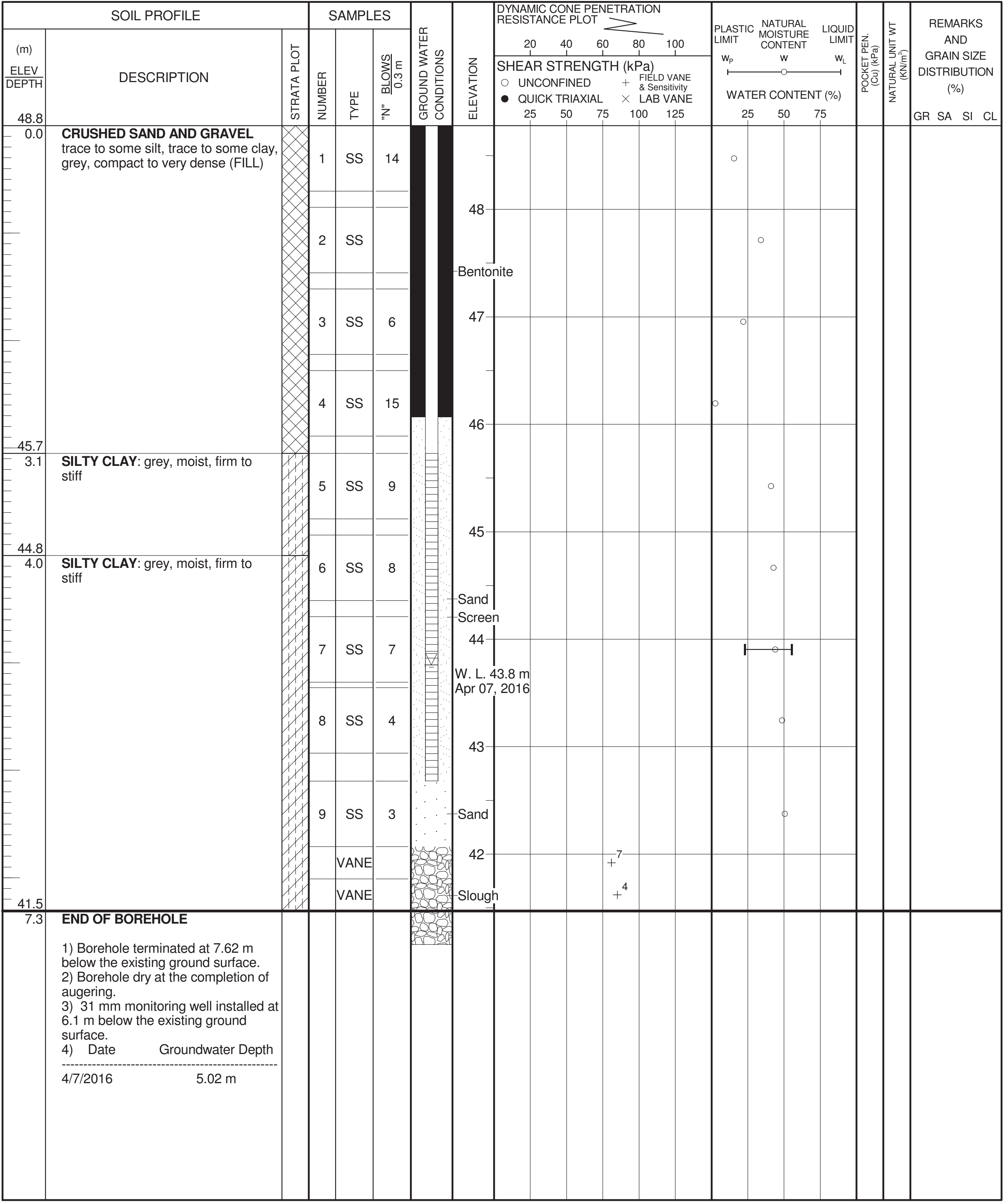
GRAPH NOTES + 3, × 3: Numbers refer to Sensitivity ○ ε=3% Strain at Failure

LOG OF BOREHOLE MW16-3

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

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

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



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

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

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

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

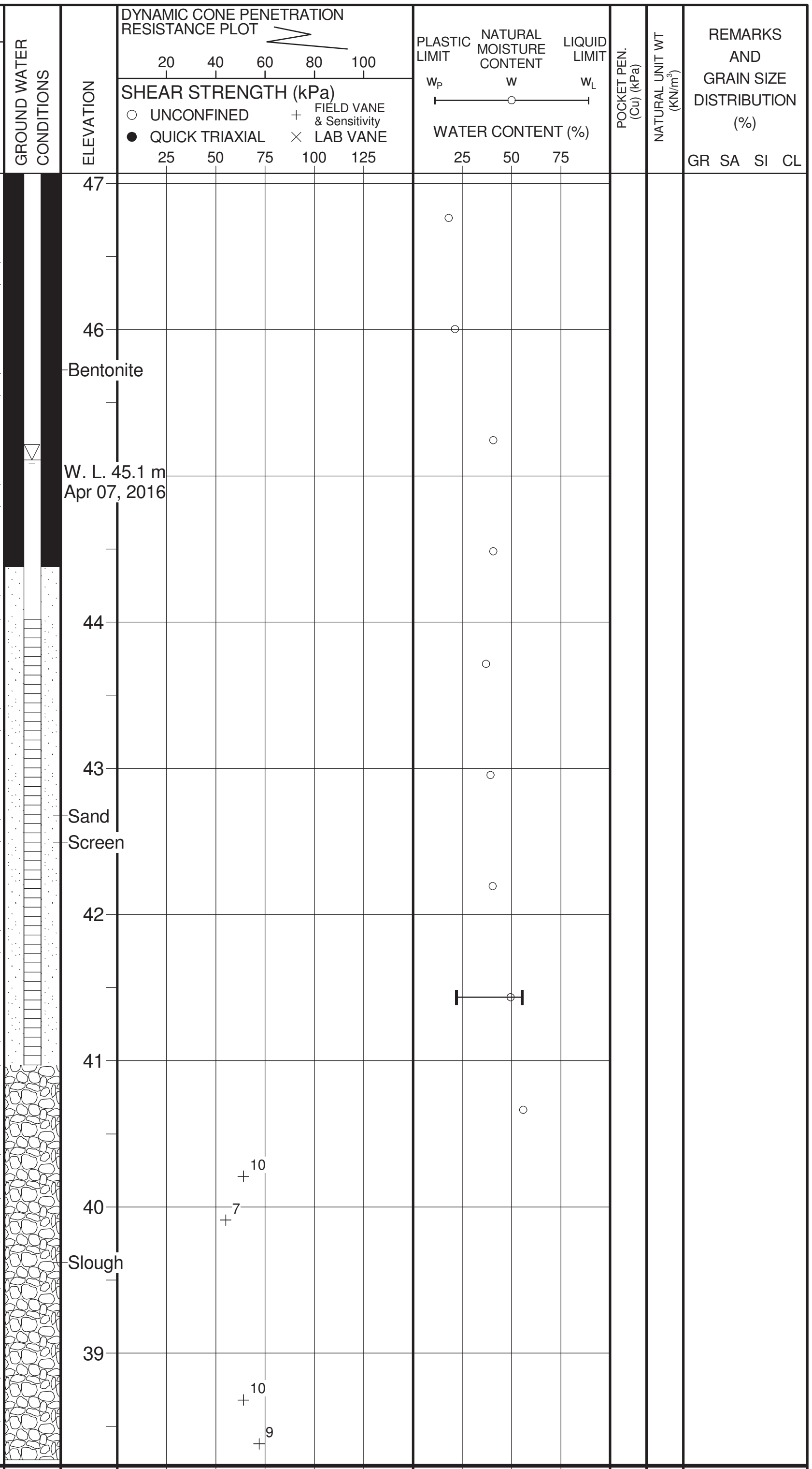
LOG OF BOREHOLE MW16-4

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:

ELEV (m)	DEPTH (m)	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									
(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/m^3)	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	6.1 m below the existing ground surface. 4) Date Groundwater Depth -- 4/7/2016 2.0 m											

GROUNDWATER ELEVATIONS

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

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

LOG OF BOREHOLE MW16-5

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

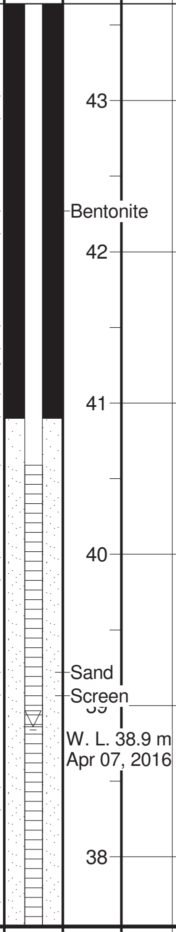
DRILLING DATA
Rig Type:
Method: Hollow Stem Auger
Borehole Diameter: 203 mm
Core Diameter:

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

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

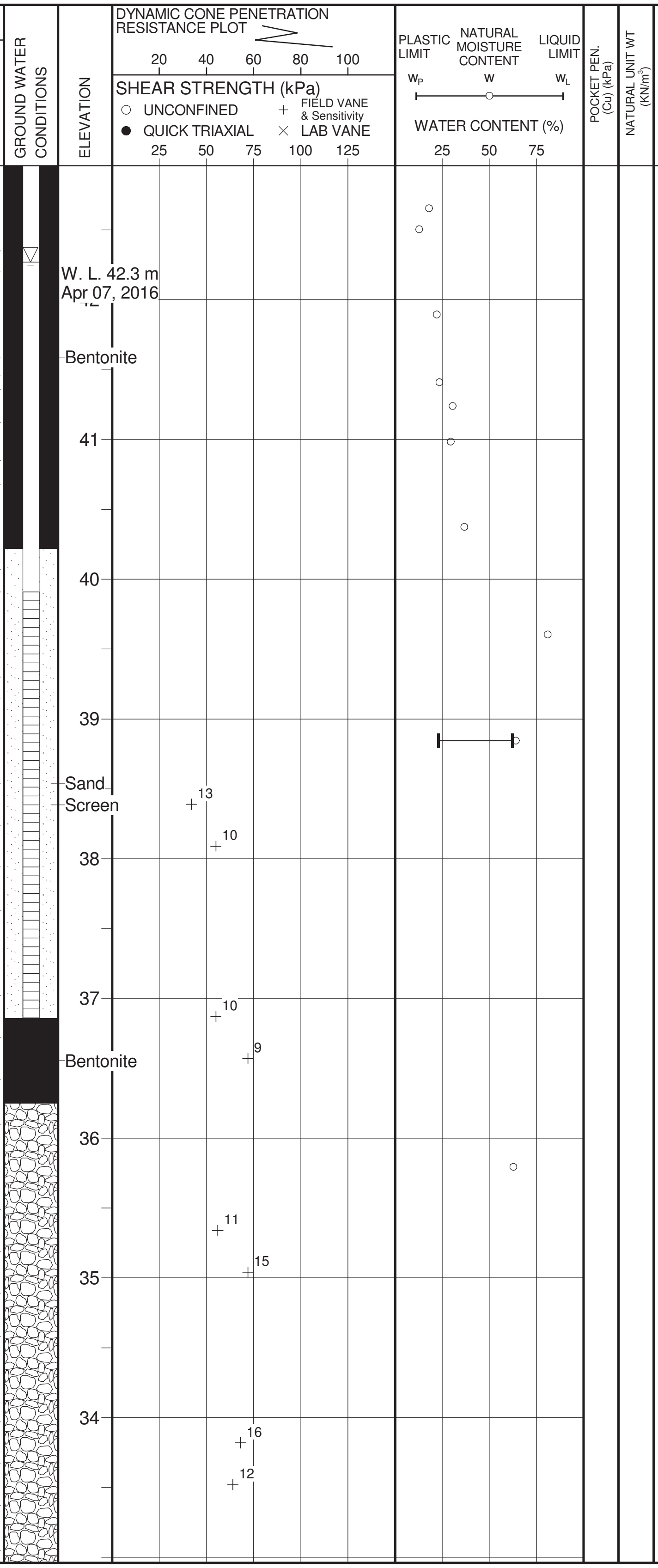
Shallow/ Single Installation ▽ ▼ Deep/Dual Installation ▽ ▼

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

LOG OF BOREHOLE MW16-6

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

SOIL PROFILE		SAMPLES			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		



Continued Next Page

GROUNDWATER ELEVATIONS

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

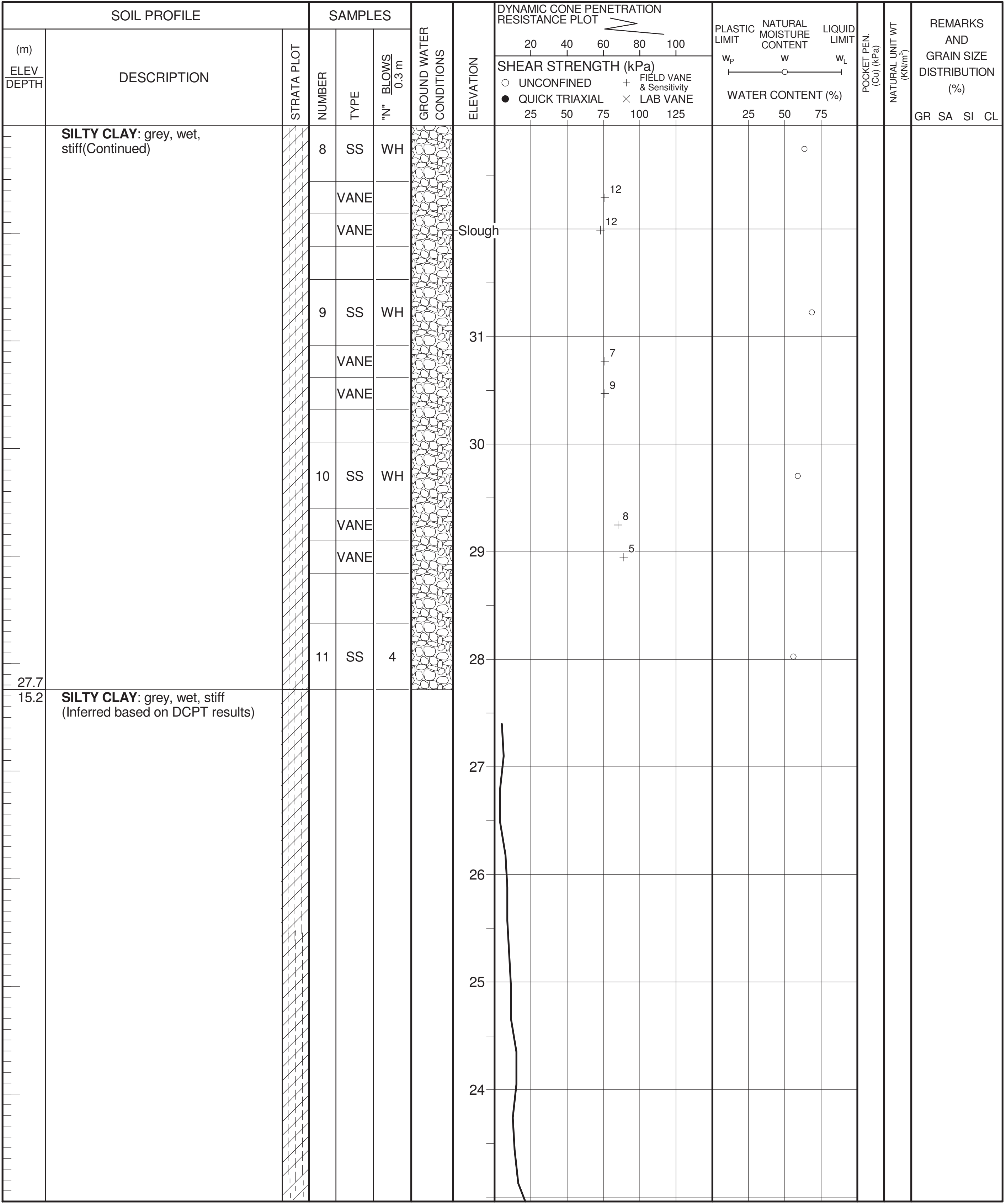
GRAPH NOTES + 3, × 3 : Numbers refer to Sensitivity ○ ε=3% Strain at Failure

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(Continued)

27.7
15.2
SILTY CLAY: grey, wet, stiff (Inferred based on DCPT results)

Continued Next Page

GROUNDWATER ELEVATIONS

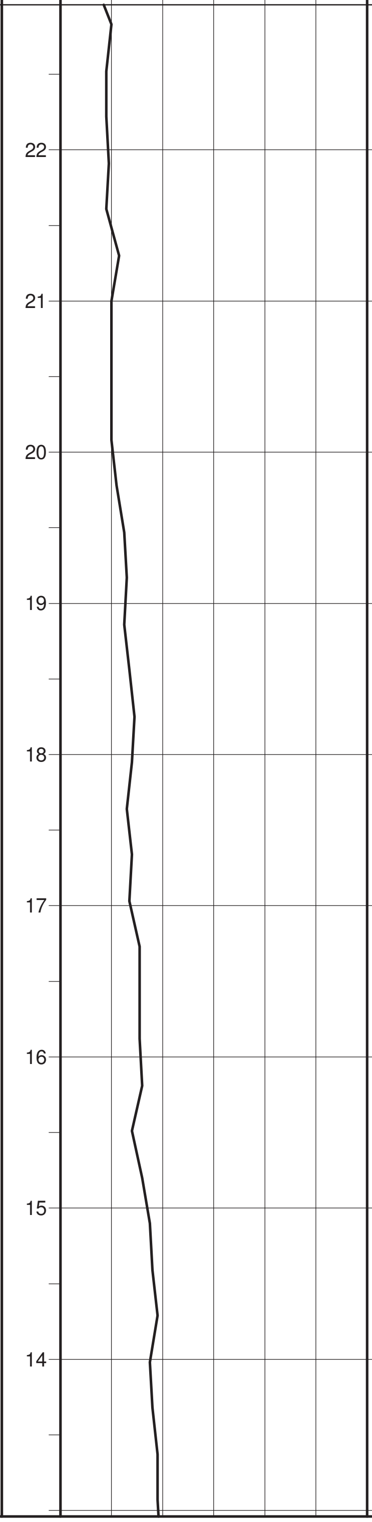
Shallow/ Single Installation ▽ ▼ Deep/Dual Installation ▽ ▽

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

LOG OF BOREHOLE MW16-6

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

SOIL PROFILE			SAMPLES					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	GROUND WATER CONDITIONS	ELEVATION					
	SILTY CLAY: grey, wet, stiff (Inferred based on DCPT results)(Continued)	▨										



Continued Next Page

GROUNDWATER ELEVATIONS

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

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

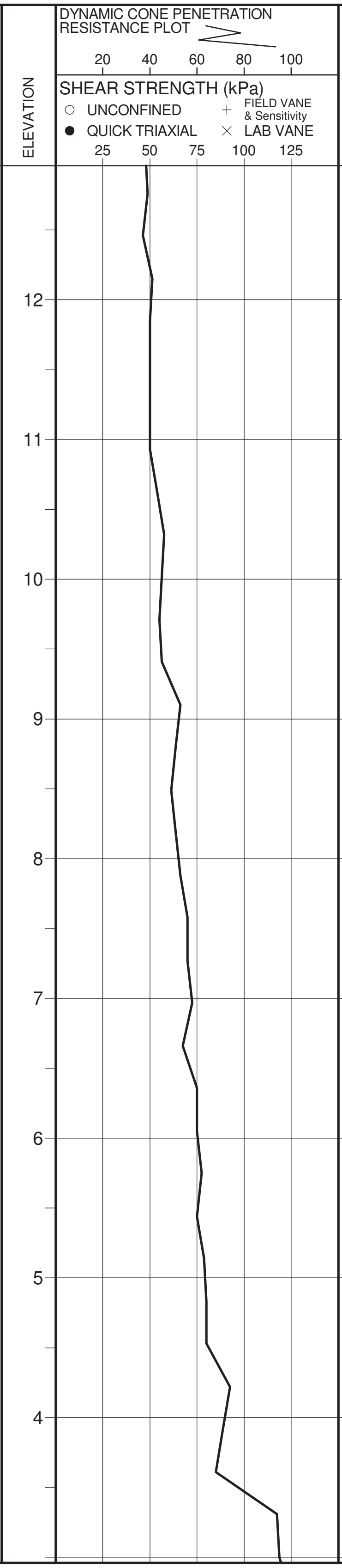
LOG OF BOREHOLE MW16-6

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							
	SILTY CLAY: grey, wet, stiff (Inferred based on DCPT results)(Continued)	▨					12 11 10 9 8 7 6 5 4					



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					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					
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											

GROUNDWATER ELEVATIONS

Shallow/ Single Installation ∇ ▼ Deep/Dual Installation ▼ ∇

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

APPENDIX B
SURFICIAL GEOLOGY

CROSS-SECTION & TEST HOLE LOCATION PLAN

CROSS-SECTIONS

patersongroup
consulting engineers

154 Colonnade Road South
Ottawa, Ontario K2E 7J5
Tel: (613) 226-7381 Fax: (613) 226-6344

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

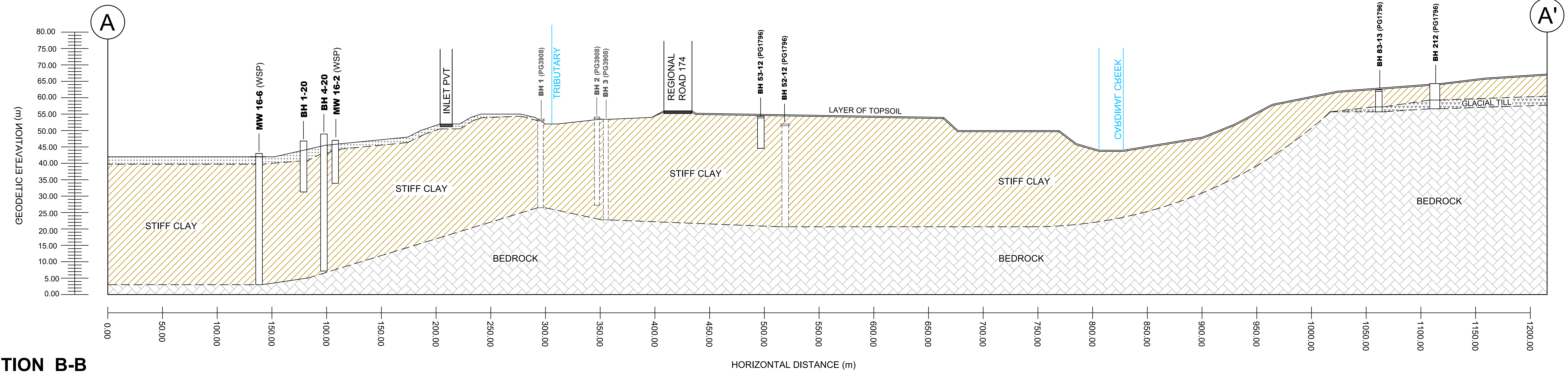
Title:
SURFICIAL GEOLOGICAL MAPPING AND HISTORICAL INFORMATION

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

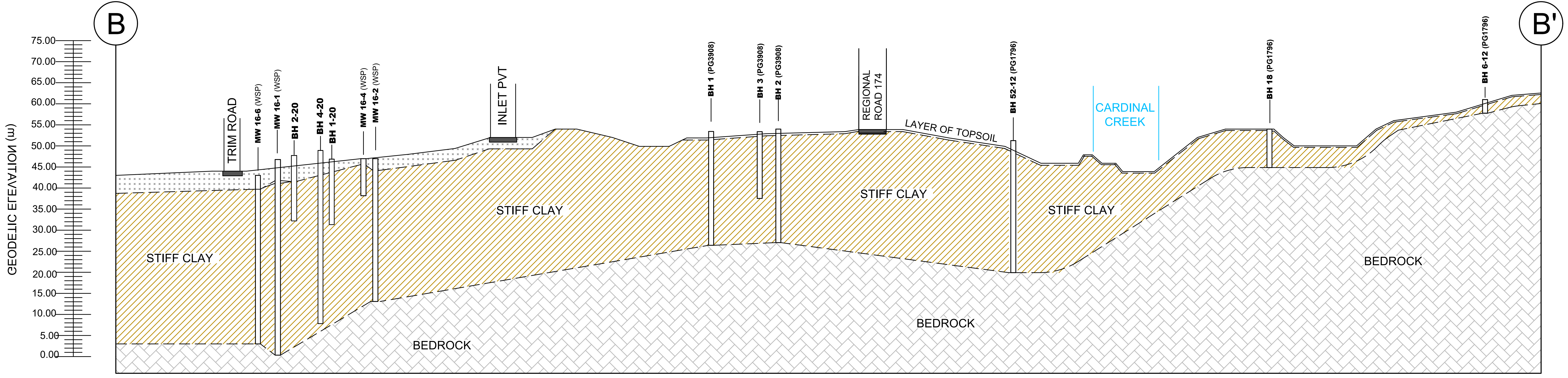
SECTION A-A

SCALE - H1:2000 , V : AS SHOWN



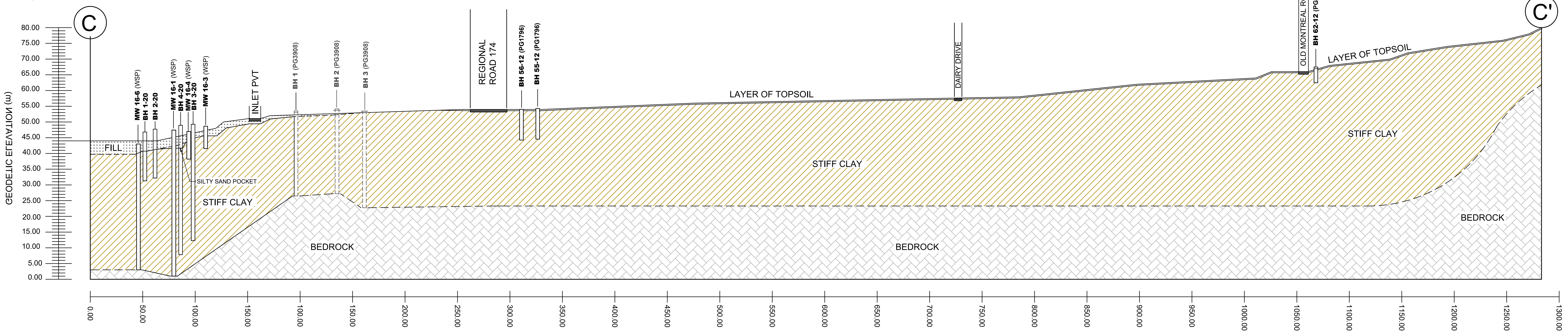
SECTION B-B

SCALE - H1:2000 , V : AS SHOWN



SECTION C-C

SCALE - H1:2000 , V : AS SHOWN



patersongroup
consulting engineers

154 Colonnade Road South
Ottawa, Ontario K2E 7J5
Tel: (613) 226-7381 Fax: (613) 226-6344

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 CROSS SECTIONS

Stamp:

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

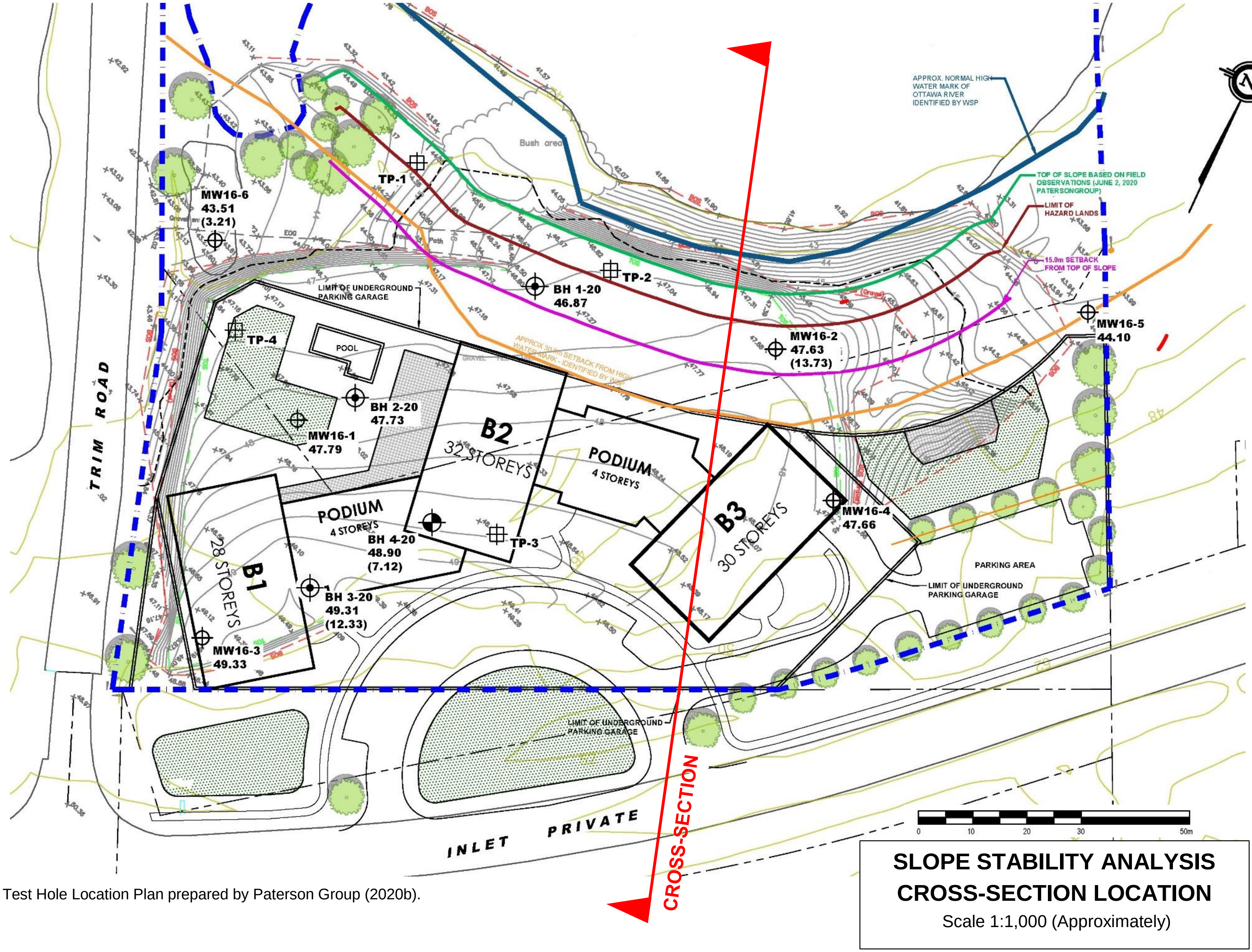
Report No.: PG5336-1
Drawing No.: PG5336-3
Revision No.: 1

p:\autocad drawings\geotechnical\pg53xx\pg5336\pg5336-surficial geology with cross sections.dwg

APPENDIX C
SLOPE STABILITY ANALYSIS RESULTS

CROSS-SECTION LOCATION PLAN

CROSS-SECTION PLOTS



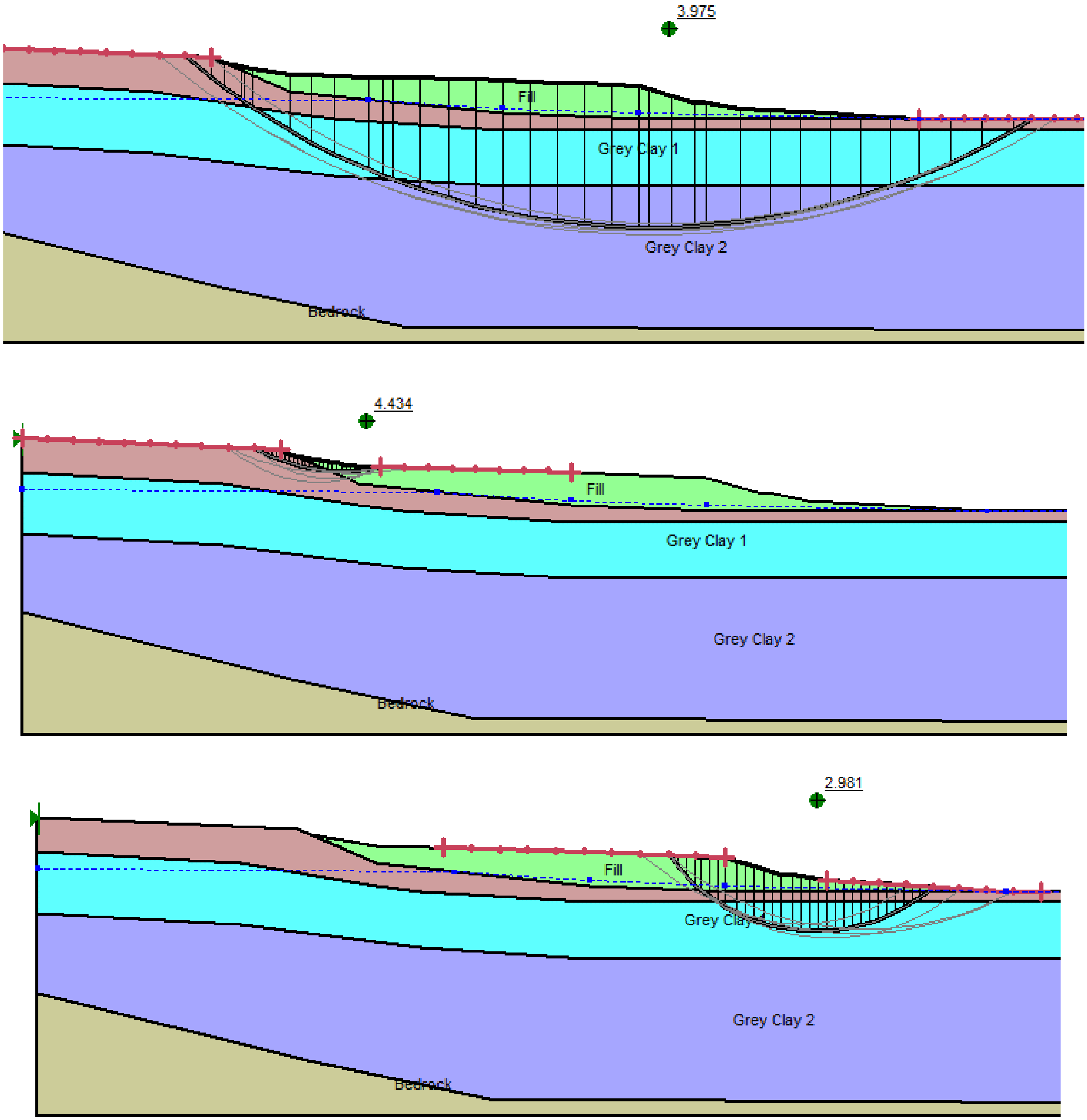
From Test Hole Location Plan prepared by Paterson Group (2020b).

SLOPE STABILITY ANALYSIS CROSS-SECTION LOCATION

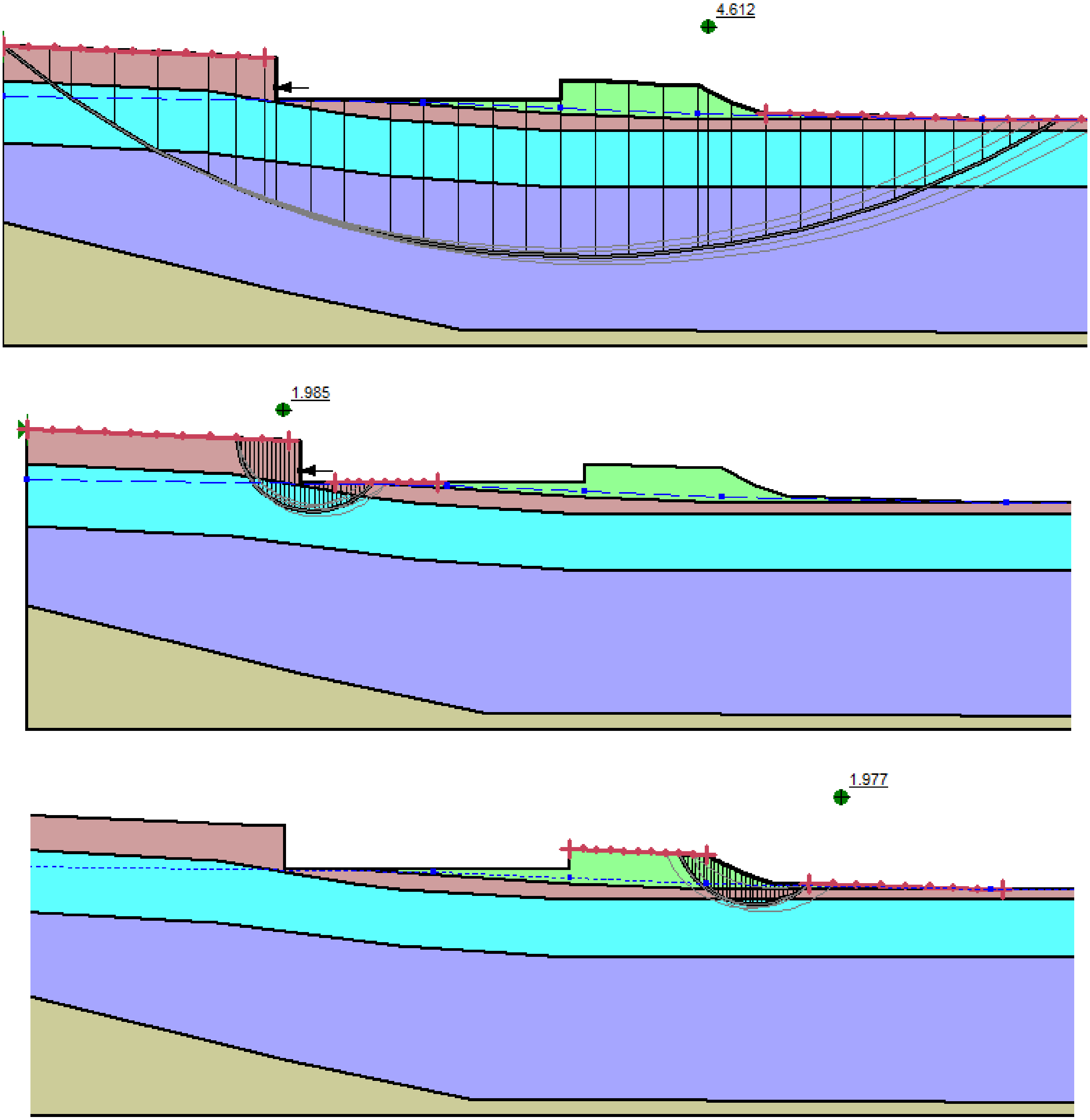
Scale 1:1,000 (Approximately)

McQuarrie Geotechnical Consultants Ltd.

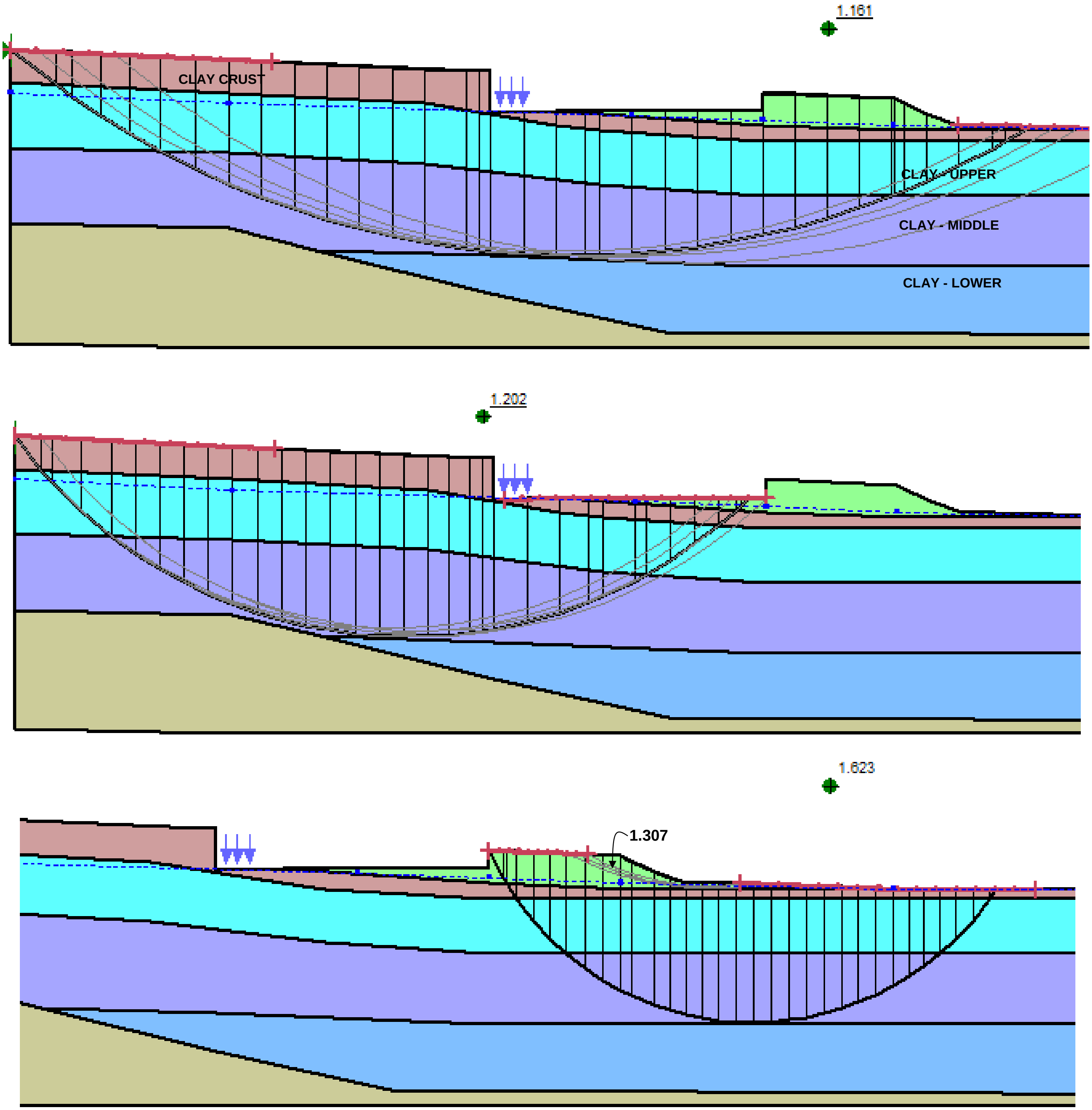
Geotechnical Engineering / Slope Stabilization / Landslide Hazard Management



SLOPE STABILITY RESULTS
EXISTING CONDITIONS
Scale 1:1,000 (Approximately)



SLOPE STABILITY RESULTS
FINAL CONDITIONS
Scale 1:1,000 (Approximately)



SLOPE STABILITY RESULTS
SEISMIC CONDITIONS

Scale 1:1,000 (Approximately)

APPENDIX D

PREVIOUSLY PROVIDED REPORTS AND MEMORANDUMS

LANDSLIDE HAZARD ASSESSMENT REPORT – McQUARRIE GEOTECHNICAL CONSULTANTS LIMITED DATED FEBRUARY 8, 2021

PATERSON GROUP REPORT PG5336-LET.04 SLOPE STABILITY REVIEW AND LANDSLIDE RISK ASSESSMENT

PATERSON GROUP PG5336-MEMO.05

McQuarrie
Geotechnical
Consultants Ltd.

Geotechnical Engineering
Slope Stabilization
Landslide Hazard Management

1009 TRIM ROAD
OTTAWA, ONTARIO
LANDSLIDE HAZARD ASSESSMENT

Prepared For

Starwood Group Inc. &
Paterson Group Inc.

Prepared by:

E. J. McQUARRIE
16420

E. J. McQUARRIE

Feb 8, 2021

E.J. McQuarrie, PEng, PGeo (BC)
Senior Geotechnical Engineer

February 8, 2021
Project #125-1

TABLE OF CONTENTS

ATTACHMENTS:

Statement of General Conditions

List of References

Figure 1.	Location of Project Site & Mapped Landslides
Figure 2.	Slope Stability Map
APPENDIX ‘A’	Test Hole Data
APPENDIX ‘B’	Summary of Subsurface Information
APPENDIX ‘C’	Slope Stability Analysis Results

1. INTRODUCTION

This report summarizes the results of a landslide hazard and partial risk assessment for the property at 1009 Trim Road, along the Ottawa River. Paterson Group Inc. had completed a slope stability assessment in accordance with the City of Ottawa's *Slope Stability Guidelines for Development Applications*. The Rideau Valley Conservation Authority (RVCA) is adopting a standard for landslide hazard and risk assessments similar to British Columbia's; therefore, McQuarrie Geotechnical Consultants Ltd. were retained by Starwood Group Inc. to assist Paterson Group in conducting a comprehensive study that meets both the City of Ottawa's standards plus the standards outlined in the *"Guidelines for Legislated Landslide Assessments for Proposed Residential Development in British Columbia"* (May 2010) prepared by the Engineers & Geoscientists British Columbia (EGBC).

This report is subject to the attached Statement of General Conditions, which should be read carefully and understood by all users of this report.

2. METHODOLOGY

2.1 Background Research

A literature review was conducted on landslides and landslide hazards in the sensitive clays in Ontario and Quebec. Many of these past studies researched the relationships between landslides and specific properties or parameters of the Champlain Clays; other papers summarized investigations of specific landslides.

Background research included geologic maps and landslide inventories of the Ottawa River valley. The landslide inventory of the Ottawa area was recently updated using LiDAR data to better delineate landslides. Both landslide inventories focused on the larger landslides but erosion and bank stability along the Ottawa River and nearby creek gullies had also been assessed.

The documents cited in this report are summarized on the attached List of References.

2.2 Field Investigation

The field component of the assessment was conducted by Paterson Group and is summarized in their geotechnical and slope stability reports (2020a & 2020b). Paterson Group conducted a site evaluation of the slopes as well as a detailed drilling investigation. Test hole logs from a 2016 investigation by WSP were also used. The test hole logs included soil descriptions, in-situ test results, and laboratory test results. Several of the test holes are instrumented with single standpipe piezometers.

Broader coverage beyond the boundaries of 1009 Trim Road was provided by test hole logs from nearby sites, including the proposed light rapid transit (LRT) station to be located roughly 250 m south of the project site.

2.3 Slope Stability Analysis

Slope stability analysis was conducted using the limit equilibrium software in order to analyze the slope under various conditions. Analysis procedures are discussed in Section 7.1.

2.4 Hazard Assessment

The results of the background research, the landslide inventory mapping, the subsurface investigation, and the slope stability analysis were all used to assess the potential for a landslide occurring at the subject property or upslope. Various means of assessing the hazard potential are discussed in Sections 9 and 10.

3. PROPOSED DEVELOPMENT

The development proposed by Starwood Group includes two multi-storey residential towers in Phase 1 and a third tower in Phase 2. The three towers will be connected by an underground parkade that will extend beyond the footprints of each tower and cover a majority of the site. Final ground surface grades are expected to slope gradually northward from 48.5 to 49 m elevation along the south edge of the property, grading to roughly 47 m elevation at the north edge (LRL Associates, 2015). The proposed grade of the lower parkade level is expected to be approximately 44.5 m elevation.

4. SITE & TERRAIN DESCRIPTION

The project site is located along the Ottawa River near Petrie Island. The site is bound by Trim Road to the west, Jeanne D'Arc Boulevard (or Inlet Private) to the south, and a shallow, still water channel to the north. Most of the property slopes gently to the north, towards wetlands along the river bank and has been filled over the previous decades. The fill creates an irregular surface with remnant piles scattered along the surface.

Trim Road accesses Petrie Island and the road embankment blocks the river flow directly adjacent to the subject property. The road is understood to have been constructed in the 1950s. GeoOttawa's airphotos for 1928, 1958, and 1965 do not extend far enough east to provide coverage of the project site; however, the airphoto from 1976 shows the road in place and already influencing fluvial processes. The 2019 geoOttawa imagery, taken during low water, shows wet lands extending more than 50 m beyond the shoreline, indicating a shallow river bed and a depositional or still water environment.

The project site is bound by slopes to the north and south; however, both are quite short and gentle. Based on site grading plans by LRL Associates (2015), the slope leading into the river has been altered by fill placement, is irregular and generally less than 3 m high. The slope to the south rises 3 to 4 m at roughly 15% grade up to Jeanne D'Arc Boulevard.

Farther south, the terrain is quite gentle with less than 1% grade extending more than 800 m from the subject property. This gentle terrace is drained by several gullied stream channels, with Cardinal Creek to the east and Taylors Creek to the west of the project site. Bilberry Creek and Green's Creek are located farther to the west. These streams are each gullied at least 6 to 8 m deep through the clay deposit, with Cardinal Creek and Green's Creek gullied 15 to 20 m deep. Other than the gully banks, the closest steep slope to the site is the bank of the proto-Ottawa River, 1 km south along the south side of St. Joseph Boulevard and Old Montreal Road.

The project site location is shown on both Figures 1 and 2. Figure 1 also shows the large landslides mapped in the general area (Brooks, 2019) while Figure 2 shows the gullied streams and their stability hazard ratings (Klugman & Chung, 1976). The closest landslides and gullies are described in Section 9.7.

5. SUBSURFACE CONDITIONS

Previous drilling by Paterson Group and WSP found that the soil conditions beneath the project site consist of the following units (Paterson Group, 2020a):

1.5 to 6.1 m thick	Fill – mixture of sand, gravel, clay, some blast rock and boulders.
0 to 4.7 m thick	Clay crust – very stiff, brown, silty.
>20 m thick	Grey silty clay – stiff
34 to > 48 m deep	Bedrock

The test hole location plan and logs are included in Appendix 'A', along with a table summarizing the critical data, such as shear strengths and sensitivity. The test hole logs indicate that the minimum peak undrained shear strengths in the grey clay are typically 65 kPa and increase with depth. WSP's test hole 16-6 found clay as weak as 40 kPa at 4.6 m depth, but the test hole is located north of the development area, within the flood zone where only 0.7 m of clay crust was found. Remoulded strengths are generally 7 kPa or higher, with some strengths as low as 4 kPa. Clay sensitivity is typically between 5 and 12, with two of WSP's test holes (16-1 and 16-6) recording localized sensitivities as high as 16. Based on the sensitivity classification system in the Canadian Foundation Engineering Manual (2018 Errata), the clay classifies as sensitive to extra sensitive.

The liquidity index (I_L), determined from moisture contents and Atterberg limits on WSP's test hole samples, ranged from 0.65 to 1.62. The lowest values for I_L were within the clay crust or upper grey clay while the highest values were at depth.

Test hole logs for the LRT station by Golder and AECOM found similar soil conditions, with a thinner fill layer overlying a brown silty clay crust and then stiff grey clay. Clay sensitivities typically ranged between 4 and 7 (Golder's TB-2, TB-4 & TB-25). The I_L ranged from 0.3 to 0.7 in the clay crust, increasing to greater than 1.0 near 9 m depth. AECOM's (2017a) summary of subsurface conditions north of OR174 also records similar conditions but with sensitivities up to 6 in the clay crust and 14 in the grey clay.

The surficial geology based on all of the subsurface investigations is summarized on cross-sections prepared by Paterson Group and included in Appendix 'B'.

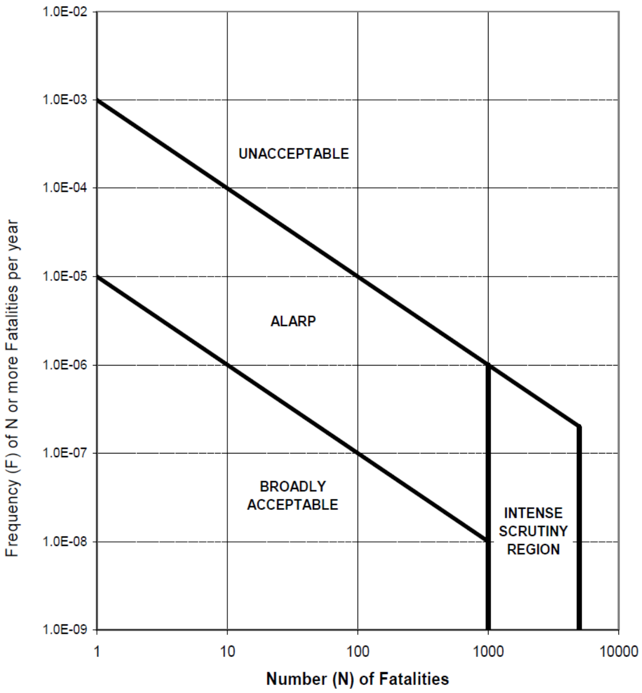
6. LEVEL OF LANDSLIDE SAFETY

Ontario's Planning Act includes specific "Natural Hazard Policies" that state that slope stability issues must be considered at the municipal planning level. Guidelines from the Ministry of Natural Resources (MNR) state that the "Limit of Slope Hazard Lands" should consider the potential for erosion along the toe of the slope over a period of 100 years, plus allowances for slope stability and equipment access. The City of Ottawa's *Slope Stability Guidelines for Development Applications* defines *hazard lands* as those with a factor of safety less than 1.5. Under seismic conditions, the *Guidelines* define an acceptable factor of safety as 1.1 while applying a seismic coefficient of one half of the peak ground acceleration (PGA) plus an increase to account for amplification based on the soil conditions. Since the design earthquake is based on the National and Ontario Building Codes, it is presumed to be the earthquake with a probability of exceedence of 2% in 50 years (annual probability of 1:2,475).

The RVCA are considering a probabilistic approach to landslide hazard and risk assessments, based on the Geological Survey of Canada's (GSC) Open File 7312 *Landslide Risk Evaluation, Canadian Technical Guidelines and Best Practices Related to Landslides* (Porter & Morgenstern, 2013). That document suggests the following tolerable risk criteria:

- <1:10,000 per annum probability for a landslide occurring and reaching the area of proposed development;
- <1:100,000 per annum risk of loss of life to individuals most at risk;
- group or societal risk of loss of life evaluated on an F-N curve, with ALARP (as low as reasonably possible) or broadly acceptable regions as the landslide safety criteria;
- tolerable slope deformation under seismic loading = 0.15 m (where it can be demonstrated that soils are not prone to earthquake-triggered liquefaction); and
- where appropriate, an allowance for 100 years of predicted toe erosion along river, lake, ocean, or reservoir shorelines.

The example F-N curve in GSC OF7312 is shown in the figure to the right.



7. SLOPE STABILITY ANALYSIS

7.1 Analysis Procedures

Slope stability analysis was conducted using the limit equilibrium software Slope/W. The following conditions were analyzed:

- existing slope configuration, drained conditions (effective stress analysis);
- temporary cutslope stability (undrained/total stress analysis);
- final slope configuration, drained conditions (effective stress analysis); and
- final slope configuration, seismic conditions (undrained/total stress analysis).

The analyses were conducted using the Morgenstern-Price method, with the soil stratigraphy determined based on the test holes at the project site and from the proposed LRT station to the south. Since the slope becomes gentler towards Trim Road (the west side of the site), the initial analysis showed that the critical cross-section is located in Phase 2 of the proposed development. All subsequent analysis focused on this single cross-section, shown on the Location Plan in Appendix ‘C’.

Several past studies (Locat et al, 2017, Tremblay-Auger et al, 2021) have applied strength parameters for the clay of c’=7.6 or 7.7 kPa and Ø’ = 30 to 32°, based on lab test by Lefebvre (1981). For this study, however, the effective stress strength parameters were estimated based on correlations with the plasticity index and are summarized in Table 1. The applied strength parameters are lower than the values used in past studies, with the values in the firm clay resembling the residual strength values used by Lefebvre. This conservative approach seemed prudent.

Table 1: Summary of Estimated Soil Strength Parameters

Soil Unit	**Unit Weight Ƴ (kN/m^3)**	**Effective Stress Analysis**		**Total Stress Analysis**	
		Cohesion c’ (kPa)	**Friction Angle Ø’ (°)**	**Undrained Shear Strength C_u (kPa)**	**Friction Angle Ø (°)**
Fill	18	0	34	-	34
Clay Crust	16	5	28	150	0
Grey Silty Clay - upper	16	0	28	70	0
Grey Silty Clay – middle	16	0	28	100	0
Grey Silty Clay - lower	17	0	28	200	0

Overall, the development creates a net unloading of the clay deposit, except to the north, between the parkade and the river. The buildings will be supported on piles bearing on bedrock; therefore, axial loads from the building were ignored in analysis. The analysis also ignored any lateral resistance provided by the piled foundation.

The final slope configuration includes a 6 to 7 m deep cut at the south edge of the site for the underground parkade, supported by a reinforced concrete foundation wall. A 320 kN/m horizontal line load was applied against this slope to simulate the resistance to the earth pressures, based on an at-rest earth pressure coefficient of 0.5, as previously determined by Paterson Group (2020a).

The applied piezometric pressures were based on the standpipe piezometers installed in the test holes on the project site, as well as the closest test holes at the proposed LRT station to the south (AECOM, 2017b). The groundwater level along the river bank was assumed to be at the toe of the bank at elevation 41.8 m, which simulates steady-state conditions. For flood conditions, the final slope is assumed to be armoured or reinforced in some way to prevent small-scale failures along this lower embankment.

The seismic (pseudo-static) analysis was conducted in accordance with the City of Ottawa's *Slope Stability Guidelines for Development Applications*, using undrained shear strengths in the clay (total stress analysis). The clay was divided into three units with the shear strengths in the upper two units based on the lower values for peak strength shown on the test hole logs, which is slightly conservative but accounts for some strain softening. The lowest clay unit was given an undrained shear strength of 200 kPa based on Dynamic Cone Penetration Test results in Test Hole 3-20, which showed a rapid change in resistance at 32 m depth.

The seismic coefficient (k) was taken as 0.5 times the PGA multiplied by 1.4 to account for amplification. The PGA for this site is 0.314g, based on the 2015 National Building Code Seismic Hazard Calculation for the design earthquake with a probability of exceedance of 2% in 50 years. The resulting seismic coefficient was 0.22g.

The seismic displacement during the design earthquake was estimated using the procedure developed by Bray and Travasarou (2007); however, the applicability of such methods in sensitive clays is not well understood.

7.2 Results

The results of the analyses of the various scenarios are summarized in Table 2.

The existing slope conditions are stable, as would be expected since there are no signs of past or present instability. The high factor of safety (FoS) of the upper slope is partly due to the fill placed against the embankment to the south. Despite the existing fill, the slope along the north edge of the site leading into the river is also stable.

The final slope configuration creates a net unloading; therefore, the FoS for a large failure encompassing the entire site actually increases compared to the existing conditions. Locally, the cut across the south edge of the site and the fill across the north edge, between the parkade and the river, decrease the FoS to approximately 2.0 in both cases. The FoS for all three failure configurations meets the City of Ottawa's minimum of 1.5.

Under earthquake conditions, using undrained shear strengths, the slip surface extends much deeper, to where the clay begins to stiffen, consistent with research (Aylsworth &

Lawrence, 2003). Still, the FoS exceeds the City of Ottawa's minimum of 1.1 for all failure sizes and configurations.

Based on the pseudo-static analysis, the anticipated seismic displacement during the design earthquake would be less than 10 cm; however, the analysis does not take into account liquefaction or the brittle behaviour of the clay. With a plasticity index greater than 20%, the clay does not meet the criteria for liquefaction; however, the sensitivity of the clay and the sudden loss of shear strength must be considered. This matter is discussed further in Section 9.8.

Table 2: Results of Slope Stability Analyses

Conditions	**Slip Surface**	**Factor of Safety**
Static Conditions – Effective Stress Parameters		
Existing Slope Conditions	Full slope failure	3.98
	Upper slope failure	4.43
	Lower slope failure	2.98
Final Slope Conditions	Full slope failure	4.61
	Upper slope failure	1.99
	Lower slope failure	1.98
Seismic Conditions – Total Stress Parameters		
Final Slope Conditions	Full slope failure	1.16
	Upper slope failure	1.20
	Lower slope failure – shallow slip	1.31
	Lower slope failure – reach building	1.62

8. LIMIT OF HAZARD LANDS

The City of Ottawa defines the "Limit of Hazard Lands" as those areas with a FoS less than 1.5 (static) or 1.1 (seismic), plus allowances for fluvial erosion, and equipment access.

The stability analysis determined the lowest FoS of 2.0 to be a slip surface extending 5 m back from the proposed crest, while the FoS for a larger failure impacting on the proposed structure exceeds 2.4. Since the "Limit of Hazard Lands" refers to areas with a FoS less than 1.5 under static conditions or 1.1 under seismic conditions, none of slope meets the stability criteria to be considered "Hazard Lands".

As explained in Section 4, the Trim Road embankment shelters the river bank, creating a still water environment. The river is located more than 20 m from the toe of the slope and no evidence of erosion was noted by Paterson Group. As such, no toe erosion allowance seems to be required. Still, the final design of the slope is expected to include some erosion control measures for flood conditions.

With a 30 m setback from the normal high water mark, the north edge of the parkade is to be located at least 24 m back from the final crest of the slope. With still water at the toe of the slope and more than 24 m of setback from the crest to the parkade structure, the typical erosion access allowance of 6 m seems adequate. This 6 m setback line is shown on Paterson Group's Plan in Appendix 'C'.

9. HAZARD ASSESSMENT

The published papers on landslides in the sensitive clays of Eastern Canada investigate and discuss several factors related to landslide occurrence and behaviour. Some factors are considered causal while others are merely related. The main factors are described briefly below, followed by a discussion on how they relate to the conditions at the project site.

9.1 Geology

Research

Landslides in the sensitive clays have been found to occur more commonly where a surficial sand layer overlies the clay (Fransham & Gadd, 1977). Subsequent studies have generally confirmed this original finding, including along Green's Creek where the landslides were commonly found where less than 3 m of sand overlies the clay (Hugenholtz & Lacelle, 2004).

Reasoning seems to be that the sand deposit is more prevalent in poorly drained areas where the high water table prevents formation of a clay crust. Another explanation may be that the freshwater environment under which the fluvial sand was deposited may have increased leaching in the clay, leading to higher sensitivities.

Project Site

The regional surficial geology maps for the Ottawa region (Fransham et al., 1976) created separate units for those areas mapped as having a surficial sand layer overlying the clay (Unit 2), versus sensitive clay without the surficial sand (Unit 1). The project site is mapped as Unit 1, without any sand layer. The site investigations confirm this finding; none of the test holes drilled on the project site or directly south of the site identified a sand layer overlying the clay. Accordingly, the landslide susceptibility is less than in those areas where the clay is overlain by sand.

9.2 Bank Height & Angle

Research

Both bank height and angle are primary geomorphic factors affecting landslide occurrence, magnitude and behaviour. When all other factors are held constant, the higher the bank, or the steeper the angle, the lower the factor of safety. The slope stability study of Ottawa-Carleton (Klugman & Chung, 1976) assessed slope heights and angles along both the Ottawa River and the tributary creeks, classifying the slopes based on their FoS. The study found that 3 m high slopes maintained stability with a FoS of 1.5 until the slope angle reached 40°, while or a 4 m high slope was stable to 32°.

Centrifugal modeling on samples of clays from the Rockliffe slide found that slope failures occurred only when the bank height exceeded 6 m, even when the slope angle was quite steep. For slope heights of 30° (1.7H:1V) or flatter, bank heights needed to be 10 m or greater (Goodings & Schofield, 1985).

Such findings are as expected and confirmed by the landslide inventory maps of the Ottawa area, which shows that a significant majority of the landslides have occurred along tributary streams or rivers feeding into the Ottawa River where the banks are higher and steeper. Higher banks are also associated with larger landslides (Quinn et al, 2011a), although not exclusively.

Furthermore, for retrogressive flow slides, the bank height must be high enough to allow the initial slide debris to exit the depletion zone in order to create the over-steepened headscarp to allow retrogression. However, slide debris can also exit the depletion zone as a result of river or creek erosion.

Project Site

An excerpt from the hazard map from the study by Klugman and Chung is shown on Figure 2. Although the project site was included in the study area, and the banks surrounding the site were classified, neither bank on the project site rated any classification. They simply were not mapped as slopes.

In fact, the project site does not have well-defined slopes, certainly not comparable to virtually any of the landslide locations. The upper slope along the south side of the property was perhaps 3 to 6 m high decades ago, prior to fill placement. After development, the bank will be fully supported by the foundation wall along the south side of the parkade. The stability analysis shows that, with the wall in place, the FoS will be approximately 2.0, meaning that the slope should not pose a landslide hazard.

Along the north side of the site, the embankment above the river will be 3 to 4 m high, again with a FoS close to 2.0 even without armouring or slope protection measures. The actual slope will be engineered better than assumed in the slope model, which should further increase the FoS. This slope should not pose a landslide hazard either.

9.3 River Bank Erosion

Research

Several landslides are at least partly caused by toe erosion, including Green's Creek where most of the landslides occur along the outside of meanders (Hugenholtz & Lacelle, 2004). After a landslide has occurred, further erosion can remove the debris accumulation, creating conditions for retrogression (Williams et al., 1979). Stream erosion has also been identified as the triggering factor for many earthflows (Karrow, 1972).

The seasonal distribution of landslides, with 60% of landslides occurring in April or May, also points towards the critical role of erosion in triggering landslides (Quinn et al., 2011b). Landslides also occur more frequently along those rivers with greater erosion rates (Quinn et al., 2012).

Project Site

With construction of Trim Road across to Petrie Island, any fluvial erosion along the north side of the subject property ceased. Airphotos clearly show that the current fluvial environment is depositional or static, and no signs of erosion were noted by Paterson Group. This slope has not been eroded in several decades if not longer.

9.4 Undrained Shear Strength

Research

While the initial landslide may occur in the drained condition, most papers relate retrogression to the undrained shear strength using Taylor's stability number:

$$N_s = \Upsilon H/C_u$$

Where: Υ= unit weight of the clay; H= bank height, and C_u =undrained shear strength.

Early research included analysis on forty landslides and concluded that for retrogression N_s must be equal to or exceed 6 (Mitchell & Markell, 1974).

Larger landslides have been found to depend on the remoulded strengths and, more specifically, earthflows seem to occur where the remoulded shear strength is 1 kPa or less (Quinn et al., 2011b).

Project Site

Based on a minimum undrained shear strength of 70 kPa, and a bank height of 4 m, Taylor's stability number is less than 1. The slope would be classified as stable in the short-term and any angle. More importantly, the stability number is much lower than the value 6 needed to cause a retrogressive failure.

The lowest remoulded shear strengths in the test holes by Paterson Group are typically 7 to 10 kPa while WSP's test holes found some remoulded strengths between 3 and 8 kPa in Test Hole 16-6 and 4 kPa in Test Hole 16-1. No remoulded strengths were found even close to 1 kPa, typically associated with earthflows.

9.5 Clay Sensitivity & Brittleness

Research

Past studies have found mixed results when relating landslide occurrences specifically to clay sensitivity; likely because of the wide range of other conditions and the different types of landslides. Earthflows have been reported in clays with sensitivities ranging from 10 to 1,000 (Mitchell & Markell, 1974). The sensitivity measured below the fissured crust is reportedly rarely less than 10 and sometimes several thousand (Penner & Burn, 1978).

Sensitivity is the primary determining factor of the remoulded strength of the clay, which is a factor in retrogressive flow slides (as discussed above in Section 9.4).

Project Site

The sensitivities found in the test holes by Paterson Group and WSP are typically 10 or less. WSP found the highest sensitivity on site to be 16. Although a direct relationship has not been found between sensitivity and landslide occurrence, the sensitivity of the clay beneath the project site is at the lower end of the range typical of the Ottawa area.

9.6 Groundwater

Research

Groundwater is a significant factor in most landslides. As noted in Section 9.3, landslides occur more frequently during spring freshet, which is also the period when groundwater levels tend to rise. Groundwater plays a larger role in landslides where down-cutting has created a downward gradient along the banks (Fransham & Gadd, 1977), most notably along the deep tributary gullies.

Several landslides in the sensitive clays of Eastern Canada have been found to occur where the groundwater gradient in the upper clay unit is downwards but is artesian or upwards in the lower clay unit (La Rochelle et al, 1970). This upward gradient in the lower clay unit can be caused by a rise in the bedrock surface (Quinn et al., 2010). The dual groundwater gradients are thought to increase leaching of the clay, which would increase sensitivity.

Project Site

None of the test holes were instrumented with multiple piezometers to determine the vertical piezometric gradient. However, the relatively low relief and gentle terrain across the project site creates a relatively low horizontal gradient.

9.7 Proximity to Other Landslides

Research

The landslide inventory map for Ottawa shows concentrations of landslides in specific areas, confirming what previous studies have found elsewhere in Quebec and Ontario (La Rochelle et al., 1970). For example, the Saint-Jude landslide on May 10, 2010 occurred close to sixteen older landslides (Penner & Burn, 1978), while the South Nation River has several groupings of landslides (Lawrence et al., 1996). Along Green’s Creek at least, most landslides have occurred in slopes exhibiting signs of earlier movement, such as in bowl-shaped concavities (Hugenholtz & Lacelle, 2004). The same likely applies to river banks where the slide debris can be eroded from the toe, leaving the over-steepened headscarp and the disturbed slip surface.

Past studies into landslide susceptibility in the Champlain Clays considered proximity to past landslides to be a significant factor. Quinn et al. even quantified the probability of a landslide based on the number of past landslides within 0.5 km, 1 km, and 2 km (2010). Landslides were found to be within 2 km of another landslide 96.7% of the time (Quinn et al., 2011a).

The concentration of landslides in given areas shows that they are not randomly or equally distributed; they occur where specific sets of terrain conditions exist. Most landslides occur in close proximity to other landslides because of shared terrain or subsurface conditions. Therefore, proximity may be a strong indicator of landslide susceptibility only where geomorphic and geologic conditions are similar.

Project Site

Landslides in the vicinity of the project site are shown on Figure 1. Working with the premise that 96.7% of landslides occur within 2 km of a pre-existing landslide, three landslides and another “probable landslide” are mapped within a 2 km radius of the site, as shown on Inset ‘N’ on Figure 1. These landslides all occurred between 1.1 and 1.6 km to the south-southeast of the project site; however, the three mapped landslides (Oln16 to Oln18) are along the deeply gullied banks of Cardinal Creek, where the banks are steep and 20 to 25 m high. The “probable landslide” (Oln15) occurred below the upper terrace on a 6 m high bank. This landslide is older, more obscured, and the deposition area is developed. These landslides occurred in conditions quite different than the project site.

One landslide (Oln14) is located at the 2 km limit west of the project site, along the lower terrace close to the Ottawa River. The remnant headscarp area is 6 m high but prior to failure, the river bank would have been 12 to 15 m high. The landslide inventory dates its occurrence as possibly “late Holocene” (Brooks, 2019), suggesting that it occurred shortly after the river level receded, meaning that fluvial erosion was likely a factor.

A large group of landslides is mapped 4 to 7 km to the southwest, south of St. Joseph Boulevard (Oln1 & Oln10-13). These landslides occurred along the 15 to 17 m high slope below the upper terrace; some along a steep gully.

Landslide Cmb 4 occurred 8 km northeast of the project site, on an 8 to 12 m high slope. This landslide is 1 km wide and is dated at roughly 2,000 years BP.

Landslide Cmb1, 4 km to the northeast of the project site, encompasses half of the golf course in Cumberland, and occurred on a 20 m high slope between 60 and 80 m elevation. The other landslides in the Cumberland area also appear to have occurred on this higher terrace with similar bank heights.

Taking into account the terrain conditions, most of these landslides share little with the slopes near the project site. Only Oln14 is located within 2 km of the project site and along the lower terrace of the proto-Ottawa River, and it occurred directly into the river, likely caused by toe erosion. While proximity to past landslides is a definite factor in landslide probability, the subject slope must also share terrain conditions with the nearby landslides. The lack of a defined bank at the project site and the depositional environment differentiate the terrain conditions at Landslide Oln14 and most other landslides along the Ottawa River.

9.8 Earthquakes

Research

Many of the largest landslides in the sensitive clay in Eastern Canada has been attributed to specific large earthquakes. The Charlevoix earthquake was M7 and is known to have triggered several large landslides. A series of large landslides date to roughly 4550 years BP and still another series to approximately 7060 years BP; both thought to be triggered by large landslides (Aylsworth & Lawrence, 2003). M5.9 to M6.0 is thought to be the lower threshold for inducing large landslides in the thick clay deposit.

Some larger landslides are thought to have been triggered by liquefaction of thin sand seams in the upper 20 to 25 m of the clay sequence, while two thick layers of sand were found at depth beneath the 7060 year old landslide in Lefaivre where the overlying sediments show signs of liquefaction (Aylsworth & Lawrence, 2003).

A more recent theory is that large earthquakes can induce or propagate failures along pre-existing weak zones or partially developed failure surfaces (Quinn et al., 2012). Such weak zones can develop due to the brittleness of the clay and past toe erosion. Therefore, these earthquake-induced landslides occur in conditions already susceptible to landslides.

Project Site

The 1:2,475 year earthquake for Ottawa has a PGA of 0.314g. Although the relationship between magnitude and PGA is not direct and depends on several other factors, a PGA of 0.314g would normally be associated with an earthquake exceeding M6.0. Therefore, earthquake-induced landslides are possible in the Ottawa area during the design earthquake.

The clays have already been subject to large historic earthquakes, which likely triggered many of the larger landslides. The closest landslides possibly triggered by either earthquake are 2 km to the west or 4 km to the east-northeast, both occurring in steeper terrain with greater relief. Earthquake-induced landslides occur where the potential for landslides already exists; they do not occur on gentle terrain with a high FoS. The project site has a FoS greater than 1.1 when subject to the 1:2,475 year earthquake, undoubtedly higher than where landslides have occurred in the general vicinity.

The behaviour of the sensitive clays during a large earthquake is uncertain; however, the sensitivity of the clays at the project site is at the low end of the range for the Champlain Clays. Both the geologic and geomorphic conditions are generally favourable at the project site and more stable than elsewhere along the banks of the proto-Ottawa River.

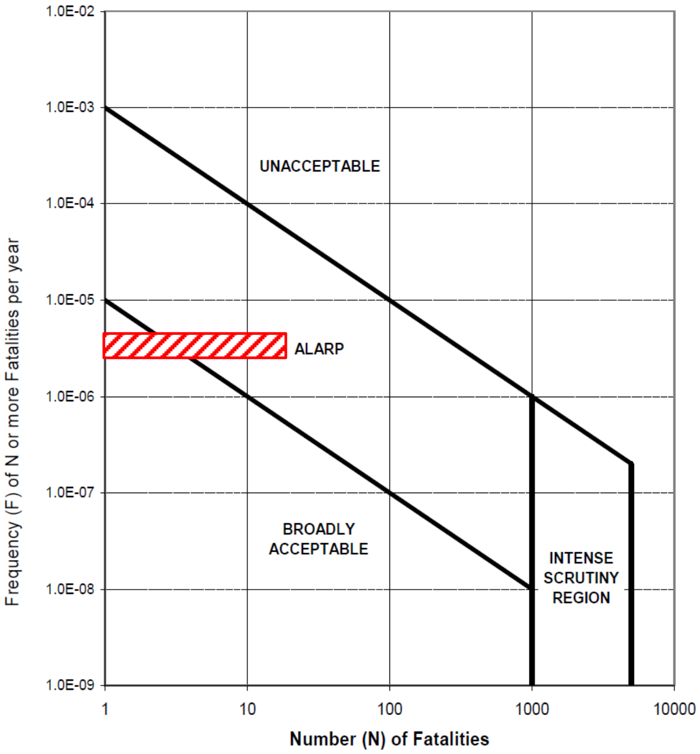
10. LANDSLIDE PROBABILITY

In multivariate probability analysis, the more variables considered, the smaller the ultimate sample size for each sub-group, which reduces both the power and the statistical reliability of the results. Therefore, quantitative hazard and risk analyses would have to cover a broad area with hundreds of landslides. An analysis to estimate the probability of a landslide impacting the project site would have to consider most, if not all, of the factors discussed in Section 9; however, the 2019 landslide inventory covers only the Ottawa area and the landslide database does not include relevant parameters such as slope height, angle, active erosion, clay sensitivity, etc. The study by Mitchell and Markell (1974) included some of these parameters in their database of 41 earthflows; however, the amount of missing data, inconsistencies in data collection, and the number of variables limits any statistical analysis. Adding the data from the more recent landslide studies provides some improvement but is still insufficient for multi-variate analysis.

The only consistent data with broad coverage is from the earlier landslide and geologic mapping by Fransham et al (1976); which provides sufficient data for a univariate analysis based solely on geology. Landslides are far more common along the tributary gullies and the upper terrace of the proto-Ottawa River valley, skewing the data. Since the project site is located along the existing river bank, the reliability of this analysis was strengthened by considering only the active bank of the Ottawa River and the lowermost bank of the proto-Ottawa River. Essentially, the south bank of the Ottawa River was treated as a lineal feature from Lower Allumette Lake to the east end of the mapping project downstream of Hawkesbury. Of this more than 240 km of bank, approximately 112 km is mapped as Unit 1 (clay) and less than 2.8 km is mapped as landslide or crosses a landslide. The result: less than 2.5% of Unit 1 along the south bank of the river is directly affected by landslides.

Since some of these landslides were undoubtedly caused by the large earthquake 4550 years BP, the slides can be assumed to have occurred over at least that time period, resulting in an annual probability of a large landslide of 1 in 182,000. If the landslide inventory is assumed to represent the full 8000 years of the deposit, the probability reduces to 1 in 320,000 per annum. Regardless, the probability of a landslide occurring at or directly impacting the project site is well below the suggested minimum of 1:10,000 per annum in GSC Open File 7312.

The risk to human life depends on how the landslide develops and progresses, and on the vulnerability of the structure. The proposed structure will be fairly rigid and founded on piles bearing on bedrock. Although the piles will not be designed to withstand the lateral forces exerted on them by the deep-seated movement of a landslide, the structure should resist movement far better than a structure bearing directly on the clay. The structure and surrounding ground should display obvious signs of movement long before a landslide occurs. Site investigations of landslides in the Champlain Clays indicate that small bank failures and even small landslides pre-date the larger events, often by several days (Lawrence et al., 1996, Tremblay-Auger et al, in print), or the toe of the bank was being actively eroded (Lefebvre et al. 1991). Given the size of the proposed development, the residents constitute a significant group exposed to the hazard; however, they also increase visibility, decreasing the likelihood that the precursor events and signs of a pending landslide go unnoticed. The warning time may not be sufficient to protect or save the structures, but should allow evacuation of the structures, reducing the human exposure to the hazard. The likelihood of warning signs of a developing landslide is quite high and, assuming these warning signs are heeded, the likelihood of anyone dying in a landslide should be quite low, if not avoidable. The risk to humans is difficult to quantify for a specific event at a specific site; however, since the probability of a large landslide is estimated between 1:182,000 to 1:320,000 (5E-06 to 3E-06). The PDI or PDG should plot within the ALARP to “Broadly Acceptable” Zones on the F-N. The ALARP zone is generally acceptable for development, provided reasonable measures are taken to reduce the risks. The reinforced concrete structure and piles bearing on bedrock would be considered reasonable measures.



This simplistic analysis considers only those landslides mapped and ignores the smaller landslides; however, smaller landslides pose a much lower risk to human life. Also, the estimated probability of a landslide assumes all of Unit 1 along the Ottawa River has an equal probability regardless of bank height. Virtually all of the site-specific details discussed in Section 9 are favourable for the project site and each should serve to further reduce the probability of a landslide if considered appropriately. The actual probability of the project site being involved or impacted by a landslide should be less than 1:182,000 to 1:320,000 per annum.

11. SUMMARY & CONCLUSIONS

The project site has two short slopes. The slope along the south edge of the development was partly created by fill placement for the adjacent roadway but, otherwise, was formed by the lowest bank of the proto-Ottawa River. This slope is now partly supported by fill placed on the site decades ago and will be fully supported by the foundation wall of the proposed parkade structure. Temporary stability must be managed during construction but the final grade configuration easily meets the City of Ottawa’s minimum factors of safety under both static and seismic conditions.

The lower slope leads into a sheltered still water area that drains into the Ottawa River. Toe erosion is not a hazard and the final embankment will be relatively short. The FoS meets the City of Ottawa's requirements, the bank is too short to trigger a retrogressive landslide and, without toe erosion, a primary landslide trigger is lacking.

With respect to a large-scale landslide in the sensitive clays, the FoS readily meets the City of Ottawa's requirements. The overall grade of the site is much gentler and the embankments much shorter than where any past landslides have occurred in the general area. This lower terrace bank has experienced a much lower landslide frequency than the upper terrace bank and the gullied banks of the tributaries. The sensitivity of the clay is at the low-end of the range for the Champlain Clays, and the remoulded strengths are significantly higher than those associated with retrogressive earthflows.

The gentle terrain extends far to the south where, again, the sensitivity and remoulded strengths of the clay indicate that a retrogressive earthflow is unlikely. The nearest steep slope to the south is more than 1 km away; therefore, the project site is well beyond the potential runout of any landslides initiating off the property, even a retrogressive earthflow.

The probability of a landslide occurring in the sensitive clay unit (Unit 1) near the edge of the lower terrace, is estimated to be between 1:182,000 to 1:320,000 per annum, but this assumes that all of the clay unit along the Ottawa River has an equal probability of a landslide. Landslides occur at specific locations due to specific geologic and geomorphic conditions. The site-specific conditions indicate that the project site has a lower probability of a landslide than most of the clay unit. The probability of a landslide capable of directly impacting the proposed structures is likely less than 1:320,000 per annum and much lower than the 1:10,000 per annum tolerable hazard level suggested by the GSC. Although the development will significantly increase human exposure to the hazard, it also increases the hazard visibility. The structure will be quite rigid and founded on piles bearing on bedrock; therefore, structural vulnerability should be quite low. The resulting risk to human life is difficult to estimate but should be well within the generally accepted societal levels. Based on these results, the proposed development is considered safe.

12. DETAILED DESIGN ISSUES

The landslide hazard assessment does not take into account short-term stability of the temporary cutslope. The temporary cutslope is expected to expose fill overlying very stiff clay crust, creating favourable conditions for temporary stability. Still, specific details of the temporary cutslope or shoring design should be determined by Paterson Group during the detailed design stage.

Similarly, the fillslope embankment along the north side of the development was based on an overall 2H:1V slope. The analysis focused on global stability of a failure extending into the underlying clay. Detailed design should consider minor failures within the fill, surface erosion, and fluvial erosion during spring flooding when specifying the material type, placement, compaction, and erosion control.

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STATEMENT OF GENERAL CONDITIONS

1. STANDARD OF CARE

This study and Report have been prepared in accordance with generally accepted engineering consulting practices at described in the Association of Professional Engineers and Geoscientists of BC's "*Guidelines for Legislated Landslide Assessments for Proposed Residential Development in British Columbia*" (Revised May 2010). No other warranty, expressed or implied, is made.

2. COMPLETE REPORT

All documents, records, data and files, whether electronic or otherwise, generated as part of this assignment are a part of the Report which is of a summary nature and is not intended to stand alone without reference to the instructions given to us by the Client, communications between us and the Client, and to any other reports, writings, proposals or documents prepared by us for the Client relative to the specific site described herein, all of which constitute the Report.

In order to properly understand the recommendations and opinions expressed herein, reference must be made to the whole of the report. We are not responsible for use by any party of portions of the report without reference to the whole report.

3. BASIS OF REPORT

The Report has been prepared for the specific site, development, design objectives and purpose that were described to us by the Client. The applicability and reliability of any of the findings, recommendations, suggestions, or opinions expressed in the document are only valid to the extent that there has been no material alteration to or variation from any of the said descriptions provided to us unless we are specifically requested by the Client to review and revise the Report in light of such alteration or variation.

4. USE OF THE REPORT

The information and opinions expressed in the Report, or any document forming part of the Report, are for the sole benefit of the Client. NO OTHER PARTY MAY USE OR RELY UPON THE REPORT OR ANY PORTION THEREOF WITHOUT OUR WRITTEN CONSENT. We will consent to any reasonable request by the client to approve the use of this report by other parties as "approved users. Any use that a third party makes of the Report, or any portion of the Report, are the sole responsibility of such third parties. We accept no responsibility for damages suffered by any third party resulting from unauthorized use of the Report.

5. INTERPRETATION OF THE REPORT

a) Nature and Exactness of Terrain Description: Identification of soils, rocks, terrain and geological units have been based on assessments performed in accordance with the standards set out in Paragraph 1. The field reconnaissance cannot practically cover the entire area and will only identify surface features and existing soil exposures. This type of assessment does not include subsurface investigation or measurement of soil strength properties. This assessment is qualitative, based on observed conditions and cannot be relied upon to identify conditions that may not be visible or instabilities caused by poor logging or road construction practices. Actual conditions may vary significantly between the points observed and all persons making use of such documents or records should be aware of, and accept, this risk. Some conditions change over time and those making use of the Report should be aware of this possibility and understand that the Report only presents the conditions at the time of assessment.

b) Reliance on Provided Information: The evaluation and conclusions contained in the Report have been prepared on the basis of conditions in evidence at the time of site inspections and on the basis of information provided to us. We have relied in good faith upon representations, information and instructions provided by the Client and others concerning the site. Accordingly, we cannot accept responsibility for any deficiency, misstatement or inaccuracy contained in the Report as a result of misstatements, omissions, misrepresentations, or fraudulent acts of persons providing information.

6. CONSTRUCTION INSPECTIONS

Our scope of work may include inspections of the work during construction or after completion. Such field reviews do not replace the need for appropriate construction inspection and supervision on the part of the client or his agents. We accept no responsibility for damages caused by unforeseen conditions unless we are on site during construction.

7. INHERENT RISKS

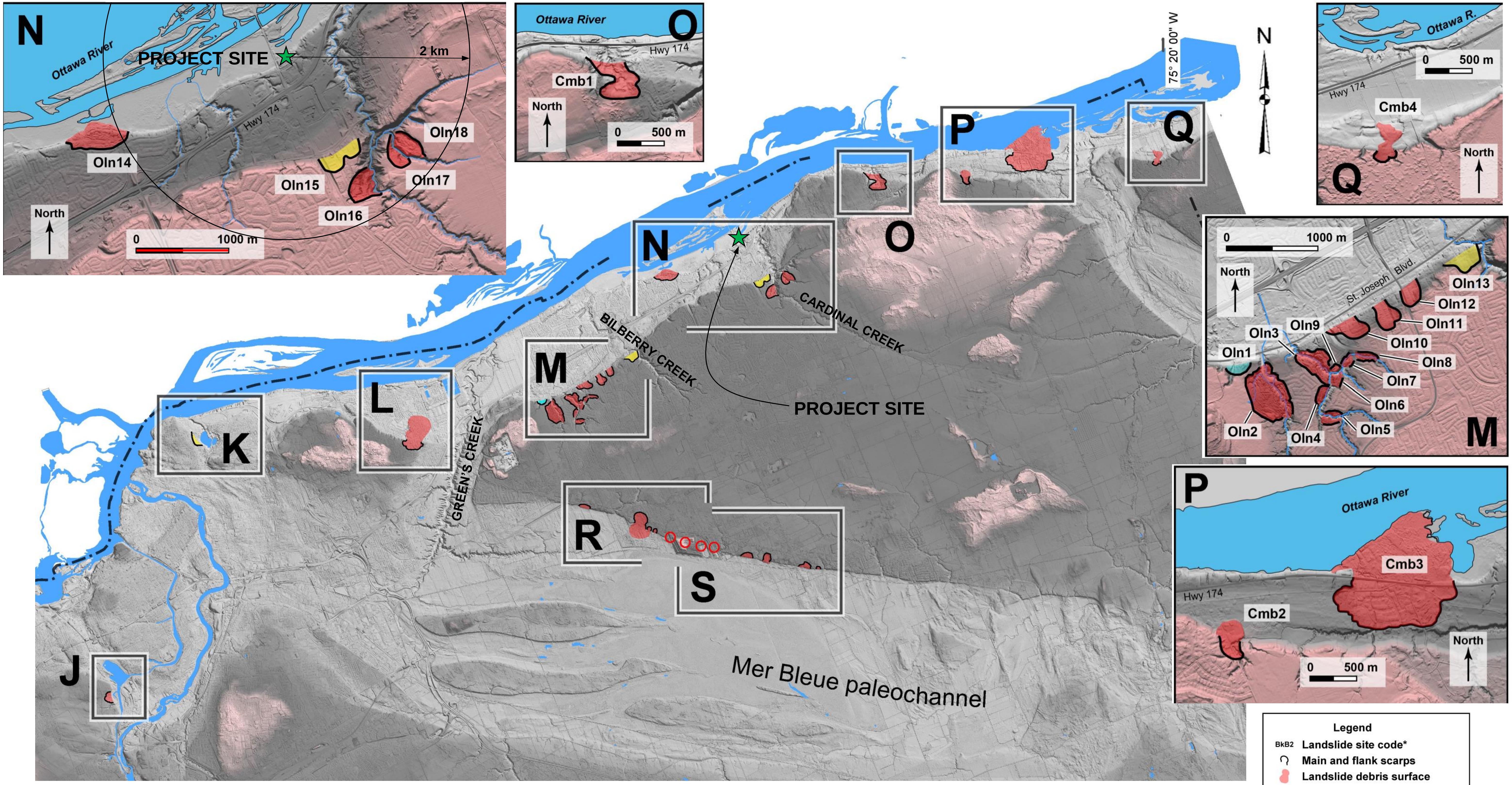
Landslide hazard assessments typically occur where there are risks of landslides. As such, inherent risks exist and landslides can occur even where the likelihood of instability has been identified as low. The client must operate with an understanding of this risk.

8. CONTROL OF WORK AND JOBSITE SAFETY

We are responsible only for the activities of our employees on the jobsite. The presence of our personnel on the site shall not be construed in any way to relieve the Client or any contractors on site from their responsibilities for site safety. The Client acknowledges that he, his representatives, contractors or others retain control of the site and that we never occupy a position of control of the site. The Client undertakes to inform us of all hazardous conditions, or other relevant conditions of which the Client is aware. The Client also recognizes that our activities may uncover previously unknown hazardous conditions and that such a discovery may require that certain regulatory bodies be informed and the Client agrees that notification to such bodies by us will not be a cause of action or dispute.

9. INDEPENDENT JUDGEMENTS OF CLIENT

The information, interpretations and conclusions in the Report are based on our interpretation of conditions revealed through limited assessment conducted within a defined scope of services. We cannot accept responsibility for independent conclusions, interpretations, interpolations and/or decisions of the Client, or others who may come into possession of the Report, or any part thereof, which may be based on information contained in the Report. This restriction of liability includes decisions made to either purchase or sell land.



From Geological Survey of Canada Open File 8600: *Sensitive Clay Landslide Inventory Map of Ottawa. (Brooks, 2019)*

0 5 10
Kilometres

Legend

BkB2	Landslide site code*
∩	Main and flank scarps
●	Landslide debris surface
🟥	Landslide*
🟨	Landslide, probable*
🟦	Landslide, possible*
○	Landslide, former site of*
- - -	Boundary between landslides
F	Letter keying inset map to location on main map

* See accompanying report for explanation

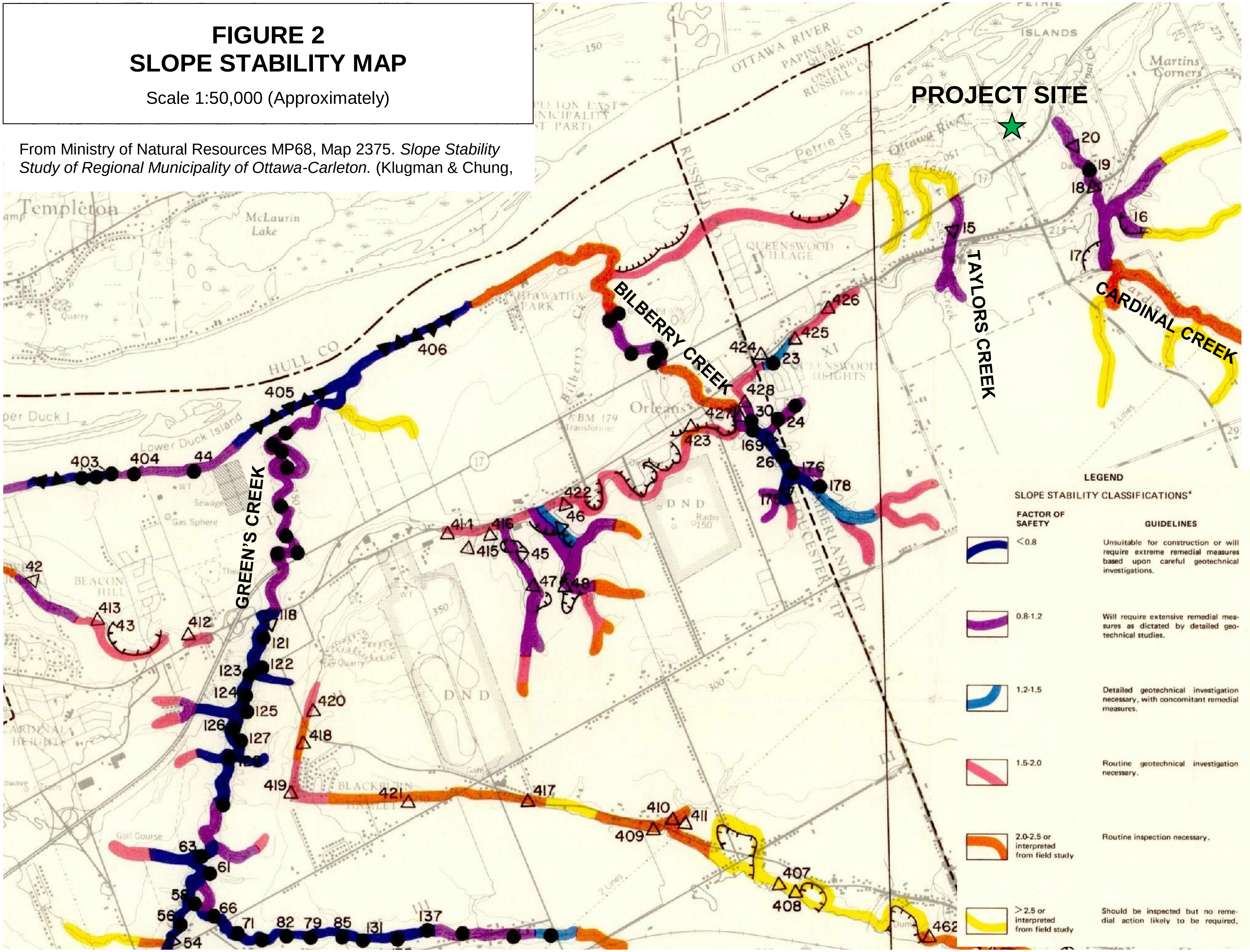
FIGURE 1
LOCATION OF PROJECT SITE & MAPPED LANDSLIDES

Scale 1:100,000 (Approximately)

FIGURE 2
SLOPE STABILITY MAP

Scale 1:50,000 (Approximately)

From Ministry of Natural Resources MP68, Map 2375. *Slope Stability Study of Regional Municipality of Ottawa-Carleton.* (Klugman & Chung,



LEGEND

SLOPE STABILITY CLASSIFICATIONS*

	FACTOR OF SAFETY	GUIDELINES
🟦	<0.8	Unsuitable for construction or will require extreme remedial measures based upon careful geotechnical investigations.
🟪	0.8-1.2	Will require extensive remedial measures as dictated by detailed geotechnical studies.
🟦	1.2-1.5	Detailed geotechnical investigation necessary, with concomitant remedial measures.
🟥	1.5-2.0	Routine geotechnical investigation necessary.
🟧	2.0-2.5 or interpreted from field study	Routine inspection necessary.
🟨	>2.5 or interpreted from field study	Should be inspected but no remedial action likely to be required.

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December 9, 2021
Report: PG5336-LET.04

9378-0633 Quebec Inc.
7 de Tellier
Gatineau, Quebec
J8T 8C2

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) reviewed the landslide hazard assessment report prepared by McQuarrie Geotechnical Consultants Limited dated February 8, 2021 for the proposed multi-storey buildings complex located at the aforementioned site. Since then, a fourth tower has been added with a slight change in the configuration of the underground parking garage.

The added tower means more people exposed to the landslide hazard. However, the number of occupants was not considered directly in the landslide risk calculation, so it does not directly increase the landslide risk.

The added excavation in the southwest corner increases the height of the cutslope by approximately 2 m because the existing ground is climbing southward towards Jeanne D'Arc Boulevard. This assumes that the lower parkade grade has not changed. Since the cutslope will be supported by the parkade, once constructed, this should not affect the long-term stability of the slope. Monitoring of the cutslope for movement will be carried out to confirm the above statement.

Based on our review of the report, Paterson is satisfied with the findings and concur with the conclusions presented in the report.

We trust that this information satisfies your requirements.

Best Regards,

Paterson Group Inc.

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

LICENSED PROFESSIONAL ENGINEER
December 9, 2021
C. P. DA SILVA
PROVINCE OF ONTARIO

PATERSON GROUP

memorandum

re: **Geotechnical Review - Response to Engineering Comments**
Proposed Multi-Storey Building Complex
1009 Trim Road - Ottawa

to: **Mr. John Smit** - john.smit@rogers.com

to: Starwood – **Mr. Martin Chénier** - chenierm@live.ca

date: June 22, 2022

file: PG5336-MEMO.05

Further to your request and authorization, Paterson Group (Paterson) prepared the following memorandum to provide responses to the Rideau Valley Conservation Authority comments regarding the landslide hazard evaluation at the aforementioned site. This memorandum should be read in conjunction with Paterson Group Geotechnical Report PG5336-1 Revision 2 dated December 10, 2021; Slope Stability Review; Landslide Risk Assessment Letter Report PG5336-LET.03 dated February 9, 2021.

It should be noted that the previously completed group risk analysis has been updated and is now superseded with the current review based on the currently proposed building configuration layout (ie.- 4 towers (current), instead of 3 towers (original)). It should be further noted that Paterson Letter Report PG5336-LET.04 dated December 9, 2021 should be considered to be superseded by the current memorandum report.

1.0 Background Information

It is understood that a fourth tower has been incorporated as part of the proposed multi-storey building complex located at the aforementioned site from the three towers previously considered. Given this addition, an updated evaluation of the group risk considered as part of the landslide risk assessment has been requested by the RVCA and is provided herein.

2.0 Landslide Risk Assessment Update

Considering the addition of a fourth high-rise tower to the proposed development, the group risk discussed in the Slope Stability Review and Landslide Risk Assessment Letter Report would be considered to increase. This is due to the number of occupants exposed to a hazard to increase accordingly. However, the probability of the hazard to occur throughout the subject site would remain the same (i.e., less than a 1 in 100,000 per year).

Since the number of occupants would increase, the visibility of warning signs of a landslide to occur would also increase and subsequently result in a proportionate reduction in human exposure to the hazard. Given this, although the group size would increase with the addition of a fourth tower, the associated risk would be expected to increase marginally from the previously established values in the aforementioned report. The increase would result in the number of fatalities remaining less than 100 such that the group risk would remain between the *Broadly Acceptable* and *ALARP* zones.

3.0 Mitigation Measures for ALARP Category

It has previously been stated that the temporary excavation side slope will be permanently supported by the parkade which would be further supported by a piled foundation support system. Since the foundation will transfer the overlying building loads to the underlying bedrock surface and bypass the clay deposit the foundation support system is inherently considered a risk mitigation measure against the probability of a landslide occurring within the clay deposit to impact the occupants.

It has also expected that the temporary shoring systems will be used to support excavation side slopes and that monitoring will occur on the shoring and surrounding soils at the time of construction. This would be considered an appropriate measure to mitigate risk associated with slope failures and landslides as generated by construction activities. This measure should be undertaken during the installation of piles and temporary shoring systems. It is further recommended to undertake the excavation in a staged approach that would result in continuous support of temporary excavations throughout the subject site.

Should the contractor consider undertaking an open-cut excavation, the excavation would be undertaken with suitable side-slopes and sequencing to minimize the potential for a landslide to occur. Slopes in excess of 3 m or below the groundwater table would be monitored periodically by the geotechnical consultant at the time of construction. As such, sufficient mitigations measures are considered to be in place to minimize the potential for a landslide occur throughout the construction and post-construction phases of the proposed development considering the additional tower.

We trust that this information satisfies your requirements.

Best Regards,

Paterson Group Inc.

Drew Petahtegoose, B.Eng.

David J. Gilbert, P.Eng.

LICENSED PROFESSIONAL ENGINEER
June 21, 2022
D. J. GILBERT
100116130
PROVINCE OF ONTARIO