REPORT

Geotechnical Report

Proposed New Building, Lees Campus

200 Lees Avenue, Ottawa, Ontario

Submitted to:

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1.0 INTRODUCTION

Golder Associates Ltd. (Golder) was retained by the University of Ottawa to conduct a geotechnical investigation in order to provide geotechnical input to the detailed design of the proposed new building at the 200 Lees Ave Campus in Ottawa, Ontario. A Site Location Plan showing the proposed building footprint is attached as Figure 1.

Golder completed two previous desktop studies; the first desktop study was to provide recommendations in support of the seismic retrofit of four buildings (Buildings A through D) at the site. Subsequent to that study the project plans changed to include the potential demolition of three of the buildings (Buildings B through D) and replacement with a new building up to six storeys in height (which will not have a basement level). A second desktop study was completed in June 2020 to provide preliminary engineering guidelines on the geotechnical design and foundation aspects of the project, including construction and environmental considerations which could influence design decisions. Additional fieldwork was proposed and carried out in general accordance with the scope of work provided in our proposal no. P20144766 dated April 2020.

The purpose of this current investigation was to assess the general subsurface conditions within the study area by means of a limited number of boreholes and associated laboratory testing. Based on an interpretation of the factual information obtained during the current investigation, along with the existing subsurface information available for the site from previous investigations, a general description of the soil and groundwater conditions is presented. These interpreted subsurface conditions and available project details were used to prepare engineering guidelines on the geotechnical design aspects of the project, including construction considerations which could influence design decisions.

The reader is referred to the ‘Important Information and Limitations of This Report’ which follows the text but forms an integral part of this document.

2.0 DESCRIPTION OF THE PROJECT AND SITE

The 200 Lees Avenue Campus was originally developed in the early 1960’s for Algonquin College, and was subsequently transferred to the University of Ottawa. Most of the main campus construction was completed in 1964, and included Buildings A to D. The construction of the Building E was started and completed in 1979.

The campus is bounded by the Rideau River to the south and east, Highway 417 to the north and the Transitway to the west.

The area was used by the City of Ottawa as a landfill between 1906 and 1947. Previous geotechnical and environmental investigations at this site indicate that up to approximately 8 metres of cinder and ash fill overlies the site. This fill was received from the former municipal waste incinerator on Lees Avenue. Below the fill, native soil consisting of mostly glacial till overlies shale bedrock.

From previous McRostie, Genest, St-Louis and Associates (MGS) records, the foundation elements for Buildings A, B, C and D, consist of uncased, cast-in-place, expanded base caissons (Franki piles).

The proposed new building is understood to be up to 6 storeys high with no basement. The proposed building footprint is shown on Figure 1.

Existing boreholes from previous investigations completed at this site by Golder Associates and McRostie & Associates have been used to supplement the current investigation. The locations of these previous boreholes are shown on the attached Site Plan (Figure 1). The results of the previous investigations are contained in the following reports:

1) Golder Associates, June 2020, Report 20144766 to the University of Ottawa titled: *"Preliminary Geotechnical Study, Proposed New Building, Lees Campus, 200 Lees Avenue, Ottawa, Ontario."*

2) Golder Associates, April 2020, Report 20140660 to the University of Ottawa titled: "*Geotechnical Study, Proposed Seismic Retrofits, 200 Lees Avenue, Ottawa, Ontario.*"

3) Golder Associates, 2012, Report 11-1121-0057 to the University of Ottawa titled: "*Geotechnical Investigation, Proposed Block A Redevelopment, 200 Lees Avenue, University of Ottawa, Ottawa, Ontario*"

4) Golder Associates, 2011, Report 11-1121-0057 to the University of Ottawa titled: "*Preliminary Geotechnical Investigation, Proposed Block A Redevelopment, 200 Lees Avenue, University of Ottawa, Ottawa, Ontario*"

5) Golder Associates, 2000. Report 001-2721 to the University of Ottawa titled: "*Final Report on Characterization of Subsurface/Material Condition, Geotechnical and Environmental Considerations, Algonquin College, Rideau Campus, Ottawa, Ontario*"

6) McRostie & Associates, 1962. Report SF-624 to Department of Public Works and Burgess, McLean & Mac Phadyen, Architects titled: "*Report on Foundation Investigation at Lees Avenue, Ottawa Site for Eastern Ontario Institute of Technology Buildings*"

Based on the results of previous investigations and the published geology maps available from the Geologic Survey of Canada (GSC) for this area, the subsurface conditions at the site are expected to consist of fill comprised of cinders and ashes in a matrix of sand to silty clay overlying silty clay to clayey silt and/or alluvium, over glacial till, over bedrock. The bedrock in the vicinity of the site is indicated to consist of shale of the Carlsbad formation.

3.0 PROCEDURE

The fieldwork for this investigation was carried out between July 6 and 13, 2020. During that time, a total of 3 boreholes (numbered 20-01 to 20-03) were advanced at the approximate locations shown on the attached Site Plan (Figure 1). The boreholes were advanced using a truck-mounted hollow-stem auger drill rig supplied and operated by Grenville Drilling from Grenville, Quebec. The boreholes were advanced to depths ranging between 14.8 and 17.1 m below the existing ground surface. Practical refusal to auger advancement was encountered in all of the boreholes which were then extended into the bedrock at two of the three locations using rotary diamond drilling techniques while retrieving HQ-sized core. Within these boreholes, the drilled lengths in the bedrock were about 3.2 metres.

Standard Penetration Tests (SPTs) were carried out within the overburden at regular intervals of depth. Samples of the soils encountered were recovered using 35 mm diameter split-spoon sampling equipment.

The fieldwork was supervised by technicians from our staff who located the boreholes, directed the drilling and in-situ testing operations, logged the boreholes and samples, and took custody of the soil and bedrock samples retrieved. On completion of the drilling operations, the soil and bedrock samples were transported to our laboratory for further examination by the project engineer and for laboratory testing, which included natural water content, grain size distribution, and Atterberg limit tests on selected soil samples.

One sample of soil from borehole 20-03 was submitted to Eurofins Environment Testing for basic chemical analyses related to potential sulphate attack on buried concrete elements and potential corrosion of buried ferrous elements. The results of the chemical analyses are still pending and will be included in the final version of the report.

Monitoring wells were installed in all boreholes. The wells were installed in bedrock in boreholes 20-01 and 20-02. Due to cave-in, the well was installed in the overburden about 1.4 m above the bedrock surface in borehole 20-03.

The borehole locations were selected in consultation with the University of Ottawa, marked in the field, and subsequently surveyed by Golder Associates personnel. The borehole coordinates and ground surface elevations were measured using a Trimble R8 GPS survey unit. The geodetic reference system used for the survey is the North American datum of 1983 (NAD83). The borehole coordinates are based on the Modified Transverse Mercator (MTM Zone 9) coordinate system. The elevations are referenced to Geodetic datum (CGVD28).

4.0 SUBSURFACE CONDITIONS

4.1 General

Information on the subsurface conditions is presented as follows:

- Borehole records from the current investigation are provided in Appendix A.
- Borehole records from previous investigations are provided in Appendix B.
- Photographs of the bedrock core are provided in Appendix C.
- Results of the basic chemical analyses will be provided in Appendix D in the final version of the report.
- Results of geophysical testing carried out in 2011 are provided in Appendix E.
- Results of the water content and Atterberg limit testing will be provided on the Record of Borehole Sheets in the final version of the report.
- Results of the grain size distribution testing will be provided in the final version of the report.

The Record of Borehole sheets describe the subsurface conditions at the borehole locations only.
The stratigraphic boundaries shown on the borehole records are inferred from non-continuous sampling, observations of drilling progress and results of Standard Penetration Tests and, therefore, represent transitions between soil types rather than exact planes of geological change. Furthermore, subsurface soil, bedrock and groundwater conditions will vary between and beyond the borehole locations.

The following sections present a more detailed overview of the subsurface conditions encountered in the boreholes advanced during the current 2020 investigation within the proposed new building footprint.

4.2 Overview of Subsurface Conditions

In general, the subsurface conditions at this site consists of surficial topsoil or pavement, over an extensive layer of cinders and ash in a matrix of sandy fill, underlain by glacial till comprising of clayey silt and sandy silt underlain by shale bedrock. A layer of silty clay was encountered beneath the fill in Borehole 20-03.

The following sections provide a more detailed description of the subsurface conditions encountered in the current borehole investigation that were advanced within the proposed building footprint.

4.3 Topsoil , Asphaltic Concrete/Concrete

An 80 mm thick layer of topsoil was encountered in Borehole 20-01 while a 40 mm thick layer of asphaltic concrete and a 130 mm concrete layer were encountered in Boreholes 20-02 and 20-03, respectively.

4.4 Fill

Fill was encountered in boreholes from previous investigations and in all boreholes in the current investigation. Where asphaltic concrete and concrete was encountered, the upper portion of the fill generally consists of brown, granular pavement structure comprised predominantly of varying amounts of sand and gravel. The pavement structure and/or granular fill extends to depths ranging from between about 0.4 to 2.4 m below the ground surface.

Beneath the topsoil in borehole 20-01 and beneath the pavement structure in boreholes 20-02 and 20-03, the fill consists of cinders and ashes within a matrix of predominantly sandy silt and silty sand. This fill extends to depths ranging from 5.2 m to 5.3 m below existing grade. In previous investigations this fill extended to depths ranging from 6.1 to 6.3 m below ground surface at the time of those investigations (the historical boreholes are not in exactly the same location, and the previous investigations were prior to the current site development).

The presence of organic matter and brick pieces was observed in some fill samples. Pieces of glass, wire and other materials were observed in the boreholes and test pits from previous investigations advanced within the vicinity of the proposed new building. Coal pieces were also observed in borehole 20-01 between about 3 and 5.2 m below grade and in borehole 20-03 between about 1.5 and 5.3 m below grade.

SPT “N” values measured within the pavement fill ranged from 11 to 36 blows per 0.3 m of penetration indicating a compact to dense state. The SPT “N” values for the sandy silt and silty sand fill ranged from 2 to 14 blows per 0.3 m of penetration indicating a compact to dense state. The measured moisture content of a sample of the pavement structure base/subbase measured about 6% while the moisture content of samples of the fill ranged from about 7 to 51 percent.

The results of a grain size distribution test carried out on a sample of the granular fill material is provided on Figures B1 in Appendix B.

4.5 Silty Clay

The cinder and ash fill is underlain by a silty clay layer that was encountered in borehole 20-03 and was also observed in previously drilled boreholes within the proposed building footprint. In this current investigation the clay extended from about 5.3 to 6.9 metres below existing grade. SPT ‘N’ values ranging from 7 to 12 blows per 0.3 metres of penetration were obtained, indicating a stiff consistency.

The moisture content of one sample of the clay deposit measured about 32%.

The results of Atterberg limit testing carried out on one select sample of the clay is shown on Figure B4 in Appendix B, which measured a plasticity index value of 9% and liquid limit value of 32% indicating the soil is of intermediate plasticity.

4.6 Glacial Till

A deposit of glacial till was encountered beneath the fill in boreholes 20-01 and 20-02 and beneath the silty clay in borehole 20-03. The glacial till typically consists of a heterogeneous mixture of gravel, cobbles, and boulders in a matrix of clayey (cohesive) silt, sandy silt and silty sand. The till layer was fully penetrated in all boreholes and extends to depths ranging between about 12.9 and 14.1 metres below existing ground surface. In previous investigations the till extended to depths ranging from about 10.7 to 13.3 metres within the vicinity of the proposed new building.

SPT “N” values within the cohesive glacial till layer gave ‘N’ values ranging from about 5 to 11 blows per 0.3 metres of penetration. SPT “N” values within the cohesionless glacial till (sandy silt and silty sand) layer gave ‘N’ values ranging from about “Weight of Hammer” to greater than 50 blows per 0.3 metres of penetration, indicating a very loose to very dense state of packing. Very high blow counts may also be indicative of the presence of cobbles and boulders in the till rather than the state of packing.

The moisture content of selected samples of the cohesive till measured between 26 and 30% and between 8 to 19% for the cohesionless till.

The results of grain size distribution testing carried out on one sample of the cohesive till and one sample of the cohesionless till deposit are provided on Figures B2 and B3, respectively, in Appendix B.

4.7 Bedrock

Bedrock was encountered in all boreholes below the glacial till. Boreholes 20-01 and 20-02 were extended about 3.2 metres into the bedrock and borehole 20-03 about 0.7 metres. The recovered bedrock cores from these locations consist of fresh, thinly to medium bedded, dark grey to black, fine grained shale bedrock.

The Total Core Recovery (TCR) of the cored bedrock was 100 percent and the Rock Quality Designation (RQD) ranged from about 10 to 90 percent, indicating a variable very poor to excellent quality rock.

Photographs of the bedrock core are presented in Appendix A.

Table 1 below summarizes the depths and elevations of the bedrock surface from the current and previous investigations. Based on this, the bedrock surface is anticipated to be between elevation 48.1 and 52.1 m.

Table 1: Summary of Bedrock Surface Depths and Elevations Near the Proposed New Building

Borehole No.	Report No.	Ground Surface Elevation$^{(1)}$ in Borehole (m)	Bedrock Depth $^{(2)}$ (m)	Bedrock Surface Elevation (m)
BH20-01	Current Report	62.3	12.9	49.4
BH20-02	Current Report	62.1	13.9	48.2
BH20-03	Current Report	62.5	14.1	48.4
BH-1	SF-624	61.7	13.6	48.1
BH-2	SF-624	62.1	13.1	49.0

Borehole No.	Report No.	Ground Surface Elevation$^{(1)}$ in Borehole (m)	Bedrock Depth $^{(2)}$ (m)	Bedrock Surface Elevation (m)
BH-3	SF-624	62.8	10.7	52.1
BH-6	SF-624	62.5	13.6	48.9
BH-8	SF-624	62.1	13.0	49.1
BH-10	SF-624	62.2	12.8	49.4
BH-11	SF-624	62.7	13.2	49.5
BH-101	SF-624	62.4	13.4	49.0
BH00-2	001-2721	62.1	11.9^{R}	50.2^{R}

Note: (1) Elevation – Geodetic.
(2) Depth below ground surface at borehole location.
^{R} – Auger refusal.

4.8 Groundwater Conditions

Monitoring wells were installed in boreholes 20-01 to 20-03. The groundwater levels observed in the monitoring wells on July 21, 2020 have been summarized in the following table:

Borehole	Geological Material Well Installed In	Ground Surface Elevation (m)	Groundwater Depth (m)	Groundwater Elevation (m)	Date of Measurement
20-01	Bedrock	62.3	7.4	54.9	July 21, 2020
20-02	Bedrock	62.1	7.3	54.8	July 21, 2020
20-03	Glacial Till	62.5	7.3	55.2	July 21, 2020

Monitoring wells installed as part of the Golder 2000 subsurface investigation in the vicinity of the proposed new building indicated that the groundwater was at about elevation 56.5 m east of the proposed building. Overnight water levels were measured in the 1962 McRostie & Associates Ltd. boreholes and ranged from elevation 54.6 m to 60.7 m. This groundwater level appears to be within the fill in most boreholes with the exception of BH00-3 and BH-2 in which the groundwater was encountered within the alluvium deposit and within the silt layer, respectively.

Groundwater levels are expected to fluctuate seasonally. Higher groundwater levels are expected during wet periods of the year, such as spring.

4.9 Corrosion Testing

One sample of soil from borehole 20-03 was submitted to Eurofins Environment Testing for basic chemical analysis related to potential sulphate attack on buried concrete elements and corrosion of buried ferrous elements. This results are shown in the table below.

Borehole / Sample Number	Sample Depth (m)	Chloride (%)	Sulphate (%)	pH	Conductivity (mS/cm)	Resistivity (Ohm-cm)
20-03 SA6	3.1 – 3.7	0.093	1.04	6.15	2.30	435

4.10 Environmental Concerns

The site is known to have previous environmental impacts. The environmental condition of the site was investigated separately (by others) and does not form part of the scope of the geotechnical investigation.

5.0 DISCUSSION

5.1 General

This section of the report provides preliminary engineering guidelines on the geotechnical design aspects of the project based on our interpretation of the available subsurface information described herein and our understanding of the project requirements. Reference should be made to the "*Important Information and Limitations of This Report*", which follows the text but forms part of this document.

The foundation engineering guidelines presented in this section have been developed in a manner consistent with the procedures outlined in Part 4 of the 2012 Ontario Building Code (OBC) for Limit States Design.

5.2 Foundation Options

It is understood that the existing buildings B, C and D will be demolished to facilitate the construction of the new multi-storey building which will be located approximately within the footprints of existing Buildings C and D based on preliminary drawings provided by the University of Ottawa. The building is not expected to have a basement level.

As part of this preliminary study, a number of different foundation options have been considered (from a geotechnical perspective). These include:

Shallow Foundations

Shallow foundations, such as a raft or mat foundation would be technically feasible but would require excavation and removal, disposal, and replacement (if there is not basement) of a significant amount of the existing fill material in order to avoid founding the building on the loose, uncontrolled, historical fill. These excavations would be below the water table in the sandy fill material and would require an active dewatering system as well as, potentially, shoring. In addition, the excavation would require removal of all of the existing pile foundations.

Overall, it is considered that there is no strong reason (from a geotechnical perspective) why a shallow foundation system would be beneficial to the currently proposed project unless there are basement levels. There are a number of reasons why shallow foundations are less desirable and it is likely that a deep foundation system is more appropriate.

Driven Steel Piles

Driven steel piles (H-piles or pipe piles) are a common foundation system in the area and are feasible for this project. Driven steel piles do not require extensive excavation or management of cuttings/spoil and are relatively fast and easy to install.

Drilled Caissons

Drilled Caissons are also feasible at the site. Caissons would typically be drilled through the fill and glacial till and socketed into rock. They would can typically generate very high capacities, but are typically more expensive than driven piles, due to the method of installation and the need to manage spoils and groundwater during construction.

Based on the fact that a basement level is not be required, the use of a deep foundation system (HP piles, steel pipe piles or caissons) is likely to be the most viable option from a geotechnical perspective. It is understood an assessment of the project from an environmental perspective is being completed by others concurrent with the geotechnical investigation.

5.3 Summary of Subsurface Conditions

The table bellow summarises the typical subsurface conditions in the north, south and central portions of the proposed new building footprint:

Table 2: Simplified Soil Stratigraphy for Proposed New Building[1]

Soil Stratigraphy	Proposed New Building		
Relevant Boreholes	North (BH20-01, BH-1, BH-8, BH00-2 and TP00-3)	Central (BH20-02, BH-6 and BH-101)	South (BH20-03, BH-2, BH-10, BH-11 and BH00-3)
Range of Ground Surface Elevation	62.1 to 62.3 m	62.1 to 62.5 m	62.2 to 62.5 m
Fill (cinder, ash and sand)	0 – 5.2 m	0 - 5.5 m	0 – 6.2 m
Native Soils - Silty Clay or Alluvium / Glacial Till (Clayey Silt and Sandy Silt)	5.2 – 13.0	5.5 – 13.9	6.2 – 14.1 m
Bedrock	> 13.0 m	>13.9 m	>14.1 m

Table Notes:

[1] The values provided above provide a typical or average stratigraphy for each building. Some variability of the strata depths within the building footprint should be expected. See individual borehole records for detailed information at the various borehole locations.

For preliminary design purposes, the groundwater level is assumed to be at elevation 55 m as measured in the monitoring wells installed in boreholes 20-01 to 20-03.

5.4 Frost Protection

All perimeter and exterior foundation elements (footings, etc.) or interior foundation elements in unheated areas should be provided with a minimum of 1.5 m of earth cover for frost protection purposes. Isolated, unheated exterior footings adjacent to surfaces which are cleared of snow cover during winter months should be provided with a minimum of 1.8 m of earth cover.

5.5 Seismic Design Considerations

The site falls within the Western Québec Seismic Zone (WQSZ) according to the Geological Survey of Canada. The WQSZ constitutes a large area that extends from Montréal to Témiscaming, and which encompasses the Ottawa area. Within the WQSZ recent seismic activity has been concentrated in two subzones; one along the Ottawa River and another more active subzone along the Montréal-Maniwaki axis. Historical seismicity within the

WQSZ from 1900 to 2000 includes the 1935 Témiscaming event which had a magnitude of 6.2, the 1944 Cornwall Massena event which had a magnitude 5.6, and the more recent 2010 Val-des-Bois (Québec) event which had a magnitude of 5.0 at about 55 kilometres north of Ottawa. In comparison to other seismically active areas in the world (e.g., California, Japan, New Zealand), the frequency of earthquake activity within the WQSZ is significantly lower but there still exists the potential for significant earthquake events to be generated.

The seismic design provisions of the 2012 OBC depend, in part, on the shear wave velocity of the upper 30 m of soil and/or bedrock below founding level.

Shear wave velocities were measured in borehole 11-1 during the Golder 2011 investigation. The seismic technical memorandum is provided in Appendix B of this report. The harmonic mean shear wave velocity of the subsurface soil and bedrock in the upper 30 metres depth was calculated by the following equation:

V_s = total thickness of all layers/ ∑ (each layer thickness/each layer shear wave velocity)

Based on the results of the vertical seismic profile (VSP) testing as well as a review of the subsurface conditions at the proposed building location, it is considered that a Site Class of C would be applicable to the design of the new building.

The soils present at the site are not considered to be liquefiable under the design earthquake.

5.6 Existing Foundations

The location of the new building falls within the footprints of Buildings C and D as shown on Figure 1. It is understood that Buildings A through D have a crawlspace in order to gain access to the building utilities. The height of the crawlspace is slightly more than one metre on average and there is evidence of the on-site fill having been left in place due to the visible presence of glass, cinder, etc.

The foundation elements for Buildings A, B, C and D, consist of uncased cast-in-place expanded base caisson piles of the Franki type. The 1962 piling specifications showed a 406-millimetre diameter shaft with Type 20 cement used in the concrete as a means of resisting the site-specific corrosive properties of existing fills. The piling records from Franki of Canada Limited indicate that a 5,500 pound hammer with a 20 foot drop was used to provide the energy (medium) to produce the base of the caisson piles. The specifications for the Franki piles were included in Appendix D of Golders April 2020 desktop report. All expanded base piles were made with a base having a volume of about 0.3 cubic metres (2 buckets).

The original piling records for Building A are included in Appendix D of Golders April 2020 report, however piling records for the remaining buildings (Buildings B, C and D) were not available.

The proposed building column/grid layout should be overlaid to the existing building grids/pile locations to identify which gridlines will be impacted by the existing foundations. The locations of the existing piles within the new building footprint should be located and surveyed during demolition. It has been assumed that it is unlikely that the proposed building can be matched to the existing building grids, such that consideration could be given to re-using the existing piles. As a result, it will be important that the proposed building foundations are designed such that the new piles should be located as far as possible away from the existing piles to avoid the new piles being obstructed by the old piles.

An estimate of the minimum distance can be calculated by determining the approximate size of the Franki pile base using pile records for Building A (it should be noted that none were available for the other buildings). This calculation would be a very rough estimate which will assume that the base is spherical and then applying a suitable factor of safety to account for the fact that the base is likely not perfectly spherical.

For instance, assuming that 2 buckets (equivalent to 0.3 cubic metres) of concrete was used for the base of the pile (as indicated in the Franki pile records), the diameter of the pile base would be about 0.85 m. Applying a FOS of 3, the minimum distance measured from the centre of an existing pile would be approximately 1.275 m. This minimum distance should be maintained for all existing piles.

5.7 New Foundations

New piles will be required for the proposed building. As previously noted, the use of driven or cast-in-place piles should be considered. The following sections discuss the geotechnical resistances for steel H-piles or steel pipe piles driven to refusal on dense till or on bedrock and caissons socketed in the shale bedrock.

5.7.1 Steel H-Pile or Steel Pipe Pile Foundations

5.7.1.1 Founding Elevations

The proposed structure may be supported on close-ended steel pipe (tube) piles or steel H-piles driven to refusal either within the lower, very dense portion of the till deposits or on the underlying bedrock.

Based on boreholes from previous studies, the following table provides an overview of the expected elevations of the very dense glacial till, as well as the bedrock surface elevations within the vicinity of the new building.

Table 3: Dense Glacial Till and Bedrock Surface Approximate Elevations

Approximate Location	Borehole Number (Report Number)	Approximate Elevation (m) of Surface of Dense Glacial Till	Approximate Bedrock Surface Elevation (m)
Northern Section of New Building	BH-8 (SF-624)	51.4	49.1
Central Section of Building	BH20-02	52.9	48.3
Southern Section of Building	BH20-03	51.9	48.4

H-piles should be reinforced at the tip with rock point driving shoes to improve seating of the piles on the bedrock and to reduce the potential for damage to the piles during driving through the overlying cobbles and boulders, in accordance with Ontario Provincial Standard Specification (OPSS) 903 *(Deep Foundations)*. To ensure adequate penetration into the hard and locally steeply sloping bedrock to provide fixity, a Titus HD Rock Injector rock point (or equivalent) driving shoe should be used.

As an alternative to driven piles (i.e. H-piles and/or closed-ended pipe piles), the use of an open-ended drilled pile advanced into the bedrock could also be considered. This pile type requires a specialized contractor and is generally more expensive than driven piles, but the use of drilled piles greatly reduces the risk of pile deflections, pile damage and piles ‘hanging up’ in the glacial till. For preliminary design purposes, the drilled pipe piles should be advanced to a minimum embedment depth of 1.5 metres into the bedrock.

5.7.1.2 Axial Geotechnical Resistance

For preliminary design purposes, HP 310x110 piles or 324 mm diameter closed-ended steel pipe piles driven to practical refusal within the very dense portions of the glacial till may be designed using factored axial geotechnical resistances at Ultimate Limit States (ULS) of 1,100 kN. The geotechnical reaction for an individual pile at SLS will not govern and may be higher than the factored geotechnical resistance at ULS; however, settlements of pile groups should be reviewed during the detailed design stage. Higher capacities would be achievable if larger pile sizes are used.

From past experience in this area, it is unlikely that the full factored structural capacity of 2,000 kN for an HP 310x110 or 1,500 kN for a 245 mm diameter closed-ended steel pipe pile with a 9 mm wall thickness driven to refusal on the shale bedrock can be achieved due to relaxation in the shale bedrock. Relaxation of the piles following the initial set could result from several processes, including:

- Softening of the shale bedrock into which the piles are driven;
- The dissipation of negative excess pore water pressures in the dense silty soil above the bedrock surface; and,
- The driving of adjacent piles.

Therefore, a reduced geotechnical capacity is recommended for piles installed in shale bedrock to account for the relaxation. For preliminary design of HP 310x110 piles driven to found on the shale bedrock, the factored axial geotechnical resistance at ULS may be taken as 1,500 kN. For design of 245 mm diameter pipe piles driven to bedrock, a factored geotechnical resistance at ULS may be taken as 1,050 kN. Serviceability Limit States (SLS) resistances do not apply to piles founded on the shale bedrock, since the SLS resistance for 25 mm of settlement is greater than the factored axial geotechnical resistance at ULS. As noted above, pre-drilling could be required to advance the piles through the lower, dense portions of the till if piles driven to bedrock are considered.

As an alternative to pre-drilling, the use of drilled pipe piles socketed into the bedrock may be considered. For a concrete-filled, 245 mm diameter steel pipe pile having a minimum wall thickness of 9 mm and at least 1.5 metre penetration into the bedrock, an axial geotechnical resistance at ULS of 1,050 kN may be used for assessment purposes. Serviceability Limit States (SLS) resistances would not govern for piles founded on the shale bedrock.

Provision should be made for restriking all of the piles several times to confirm the design set and/or the permanence of the set and to check for upward displacement due to driving adjacent piles. For the subsurface conditions at this site, it is expected that several rounds of restriking will be required on some or most of the piles. Our experience has shown that on similar sites five restrikes were required to obtain set.

The ULS pile capacities discussed herein have been based on static analyses and incorporate a geotechnical resistance factor of 0.4. Higher resistance values (0.5 for Pile Driver Analyzer or 0.6 for static pile load test methods) can be used where field testing is completed which would allow the use of higher design pile capacities. Given the large number of piles that will likely be required for the proposed building, consideration should be given to incorporating a pile testing program (at a minimum PDA testing which would justify increasing the resistance factor to 0.5; if a very large number of piles are required then a static load test could also be considered) into the contract requirements.

Pile installation should be in accordance with OPSS 903 *(Construction Specification for Deep Foundations).* For driven piles, the drawings should incorporate the appropriate note stating that the piles (both H-piles and pipe piles) should be equipped with a protective plate for the pipe piles or pile points/shoe for the H-piles (e.g. Titus Standard H Point, or similar) and should be driven to bedrock. The pile points / protective plates will provide additional protection to the pile tips against damage from boulders during driving. For piles driven to refusal on bedrock, and as described in OPSS 903, it is a generally accepted practice to reduce the hammer energy after abrupt peaking is met on the bedrock surface, and to then gradually increase the energy over a series of blows to seat the pile.

5.7.2 Caisson Foundations

As an alternative to driven pile foundations, the proposed building can be supported on caisson foundations socketed into the shale bedrock. The use of liners or casings will be required in order to advance the caissons through the overburden with minimal loss of ground. The casings should be extended so that they are “seated” a minimum of 500 mm into the bedrock.

Casing installation through the glacial till containing cobbles and boulders will be difficult and contractors should be prepared to deal with boulders and similar obstructions during drilling. Churn drilling and possibly rock coring techniques may be required to advance the caissons through the glacial till.

5.7.2.1 Axial Geotechnical Resistance

Due to the relatively high water table and the difficulty in socketing liners into shale bedrock to completely cut off the water infiltrations, it may not be feasible to dewater and clean the base of the caisson and, as such, end-bearing support may not be developed. The axial geotechnical resistance for rock socketed caissons is therefore recommended to be based primarily on the side-wall (shaft) resistance of the rock socket rather than end-bearing.

Rock-socketed caissons should be designed based on the side-wall (shaft) resistance of the rock socket and a factored geotechnical resistance at ULS of 900 kPa (i.e. a resistance factor of 0.4 has been applied), provided that the caisson socket is within competent bedrock (i.e., RQD greater than 75 percent) which was encountered from about elevation 49.5 m in borehole 20-01 and from about elevation 47.2 m in borehole 20-02. This value assumes that the side wall of the socket will be cleaned of any cuttings or smeared material.

To provide full fixity, the caissons should be provided with a minimum socket length equal to 2 times the caisson diameter. The structural engineer should check that the shear strength of the concrete is adequate to support these loads.

For a 0.9 metre diameter caisson socketed 4 m into the competent bedrock, a factored axial geotechnical resistance at ULS of about 10,200 kN is achievable. SLS resistances do not apply to caissons founded within the shale bedrock, because the SLS resistance for 25 mm of settlement is greater than the factored axial geotechnical resistance at ULS.

5.7.3 Uplift Resistance

It is understood that the piles could also be required to resist uplift forces. The uplift resistance of a pile is derived from friction along the shaft of the pile, and therefore depends on the length of the pile. It has been assumed that the pile will be installed to bedrock.

Driven Piles

For preliminary design, the ULS geotechnical resistance to uplift (factored) may be taken as 200 kilonewtons for an HP310X110 pile driven to a depth of about 12 m below the pile cap level.

Rock-Socketed Caissons

For preliminary design, the ULS geotechnical resistance to uplift (factored) may be calculated similarly to the axial geotechnical resistance discussed in Section 5.7.2.1, however a factor of 0.3 should be applied instead.

5.7.4 Lateral Resistance

The coefficient of horizontal subgrade reaction when applied over a specific area provides a spring constant that is commonly used to model load-deformation response of a pile subjected to lateral loading. The spring constant represents the stiffness of the ground and is controlled by the lateral resistance of the ground. The ultimate value of the lateral resistance developed by the ground in which the pile is embedded is controlled by the net passive pressure mobilized in the ground. Once the passive pressure resistance is fully mobilized, no further increase in lateral resistance is developed with additional lateral displacement of the pile. In most cases, the allowable or tolerable lateral displacement of the pile (i.e., Serviceability Limit States, SLS) is substantially lower than the movement required to fully mobilize the passive pressure (i.e., Ultimate Limit States, ULS).

5.7.4.1 Serviceability Limit States (SLS)

The soil parameter most used to determine the lateral resistance of piles at SLS is the coefficient of horizontal subgrade reaction. The coefficient of horizontal subgrade reaction is not a fundamental soil property and varies with geometry of the foundation. The suggested values for coefficient of horizontal subgrade reaction are summarized in Table 4 bellow:

For cohesionless soils:

$$k_h = \frac{n_h z}{B}$$

Where: n_h is the constant of horizontal subgrade reaction, as given below;

z is the depth (m); and,

B is the pile diameter/width (m).

The following ranges for the values of n_h may be used in the preliminary structural analysis. The ranges in values reflect the variability in the subsurface conditions, the soil properties and the approximate nature of the analysis and the non-linear nature of the soil behaviour (such that k_h is a function of deflection).

Table 4: Coefficients of Horizontal Subgrade Reaction

Depth (metres)	Soil Type	n_h (MN/m^3)
0 – 7.5	Fill and Native Soil (Above Water Table)	2.2
> 7.5	Glacial Till (Below Water Table)	4.4

Group action for lateral loading should be considered when the pile spacing in the direction of loading is less than eight pile diameters. Group action can be evaluated by reducing the coefficient of horizontal subgrade reaction in the direction of loading using a reduction factor, R, as follows:

Table 5: Pile Group Action Reduction Factors

Pile Spacing in Direction of Loading d = Pile Diameter or Width	Subgrade Reaction Reduction Factor, R
8d	1.00
6d	0.70
4d	0.40
3d	0.25

The coefficient of horizontal subgrade reaction values calculated as described above may then be used to calculate the lateral deflection of the pile (i.e., the SLS response of the pile), taking into the account the soil-structure interaction.

For establishing the ULS factored *structural* resistance, the shear force and bending moment distribution in the piles under factored loading can be established using these same procedures and parameters for evaluating the SLS response of the pile.

5.7.4.2 Ultimate Limit States (ULS)

The ULS *geotechnical* resistance to lateral loading may be calculated using passive earth pressure.

The ULS lateral passive resistance may be assumed to act over the pile shaft to a depth equal to six pile diameters below the underside of the pile cap (except where the silty clay thickness exceeds that depth) and the resistance per unit length of pile may be calculated as:

$$P_p(z) = 3dK_p \gamma D_w + 3dK_p (z - D_w) (\gamma - \gamma_w)$$

Where:

$P_p(z)$ = is the ULS lateral resistance at depth ‘z’ below the ground surface, i.e., underside of pile cap (kN/m)

γ = is average unit weight of overlying soil; use parameters provided in Lateral Earth Pressure Section

K_p = is the coefficient of passive earth pressure, use parameters provided in Lateral Earth Pressure Section

D_w = is the depth to groundwater table below the ground surface(m), assume at underside of pile cap level; use 2.4 m below the ground surface

γ_w = is the unit weight of water, use 9.81 kN/m^3

D = is the pile diameter or width (m)

The ULS lateral resistance of a pile group may be estimated as the sum of the individual resistances across the face of the group, perpendicular to the direction of the applied lateral force.

The ULS resistances obtained using the above parameters represent unfactored values; a resistance factor of 0.5 should be applied in calculating the horizontal resistance.

The lateral resistance of piles is a complex non-linear problem which involves not only soil mechanics, but soil-structure interaction. Furthermore, the modulus of subgrade reaction is not a material property but is a simplification to allow the soil resistance to be modelled as a linear elastic “spring”. For more complex or critical projects there are more sophisticated methods to analyze lateral pile capacity such as the method of p-y curves or finite element and finite difference modelling. Golder can provide additional guidance related to these methods if required.

5.7.5 Rock Anchors

Given the depth to the bedrock surface on this site, additional resistance to the uplift could be provided by the use of grouted rock anchors.

In designing grouted rock anchors, consideration should be given to four possible anchor failure modes.

i) failure of the steel tendon or top anchorage

ii) failure of the grout/tendon bond

iii) failure of the rock/grout bond

iv) failure within the rock mass, or rock cone pull-out

Potential failure modes i) and ii) are structural and are best addressed by the structural engineer. Adequate corrosion protection of the steel components should be provided to prevent potential premature failure due to steel corrosion.

For potential failure mode iii), the factored bond stress at the concrete/rock interface may be taken as 1,000 kilopascals for ULS design purposes. This value should be used in calculating the resistance under ULS conditions. If the response of the anchor under SLS conditions needs to be evaluated, for a preliminary assessment it may conservatively be taken as the elastic elongation of the unbonded portion of the anchor under the design loading.

For potential failure mode iv), the resistance should be calculated based on the buoyant weight of the potential mass of rock which could be mobilized by the anchor. This is typically considered as the mass of rock included within a cone (or wedge for a line of closely spaced anchors) having an apex at the tip of the anchor and having an apex angle of 60 degrees. For each individual anchor, the ULS factored geotechnical resistance can be calculated based on the following equation:

$$Q_r = \phi \frac{\pi}{3} \gamma' D^3 \tan^2(\theta)$$

where:

Q_r = factored uplift resistance of the anchor, kilonewtons

ϕ = resistance factor, 0.4

γ' = effective unit weight of rock, use 27 kilonewtons per cubic metre above groundwater level, 17 kilonewtons per cubic metre below the groundwater level

D = anchor length in metres

θ = ½ of the apex angle of the rock failure cone, use 30 degrees

Where the anchor load is applied at an angle to the vertical, the anchor and anchor group capacity should be reduced as follows:

$$q_{a'llow} = q_{allow} \cos(\alpha)$$

where:

$q_{a'llow}$ = allowable uplift capacity of anchor subject to inclined load in kilonewtons

q_{allow} = allowable uplift capacity of anchor subject to vertical load in kilonewtons

α = angle between the load direction and the vertical

For a group of anchors or for a line of closely spaced anchors the resistance must consider the potential overlap between the rock masses mobilized by individual anchors.

In the case of group effects for a series of rock anchors in a rectangle with width “a” and length “b” installed to a depth “D”, the equation for the volume of the truncated trapezoid failure zone would be as follows:

$$V = \frac{4}{3} D^3 sin^2\varphi + \ aD^2 \sin\varphi + bDH^2 \sin\varphi + abD$$

Where:

V = Volume of the truncated trapezoid failure zone

D = Depth of anchor group in metres

a = width of anchor group in metres

b = length of the anchor group in metres

φ = ½ of the apex angle of the rock failure cone, use 30 degrees

The ULS factored geotechnical resistance for the truncated trapezoid failure formed by the group of anchors can then be calculated based on the following equation:

$$Q_r = \phi \ \gamma' V$$

Where:

Q_r = Factored uplift resistance of the anchor, kN

f = Resistance factor, use 0.4

$\gamma^{/}$ = Effective unit weight of rock, use 17 kN per cubic metre below groundwater level

V = Volume of truncated trapezoid

It is suggested that pull-out tests be carried out on anchors to confirm their pull-out capacity. The pull-out tests should be carried out to 1.3 times the anchor service loads, and at least 10 percent of the anchors should be tested in this manner.

It is suggested that the installation and testing of the anchors be supervised by the geotechnical engineer. Care must be taken during grouting to ensure that the grouting pressure is sufficient to bond the entire length of the grout area with a minimum of voids. Probing of the holes should be carried out by the geotechnical engineer to ensure that the anchors are being installed in rock of adequate quality. It is also suggested that the anchor holes be thoroughly flushed with water to remove all debris and rock flour. It is essential that rock flour be completely removed from the holes to be grouted to ensure an adequate bond between the grout and the rock.

5.8 Site Servicing

At least 150 millimetres of OPSS Granular A should be used as pipe bedding for sewer and water pipes. Where unavoidable disturbance to the subgrade surface occurs during construction, it may be necessary to place a sub-bedding layer consisting of 300 millimetres of compacted OPSS Granular B Type II beneath the Granular A. The bedding material should, in all cases, extend to the spring line of the pipe and should be compacted to at least 95 percent of the material's standard Proctor maximum dry density. The use of clear crushed stone as a bedding layer should not be permitted anywhere on this project since fine particles from the sandy backfill materials and native soils could potentially migrate into the voids in the clear crushed stone and cause loss of lateral pipe support.

Cover material, from the spring line of the pipe to at least 300 millimetres above the top of pipe, should consist of OPSS Granular A or Granular B Type I with a maximum particle size of 25 millimetres. The cover material should be compacted to at least 95 percent of the material's standard Proctor maximum dry density.

It should generally be possible to re-use the existing fill, silty clay, and glacial till as trench backfill. Where the trench will be covered with hard surfaced areas, the type of material placed in the frost zone (between subgrade level and 1.8 metres depth) should match the soil exposed on the trench walls for frost heave compatibility. Trench backfill should be placed in maximum 300 millimetre thick lifts and should be compacted to at least 95 percent of the material's standard Proctor maximum dry density using suitable vibratory compaction equipment.

5.9 Pavement Design

In preparation for pavement construction, all topsoil, unsuitable fill, disturbed, or otherwise deleterious materials (i.e., those materials containing organic material) should be removed from the pavement areas. Some of the existing fill could remain provided that it is free of organic matter, and that the subgrade be subjected to a proof roll with a loaded tandem truck to reveal weak or soft areas prior to the construction of the new pavement structure. Soft or weak areas should be removed and repaired with acceptable earth borrow or OPSS Select Subgrade Material (SSM).

Pavement areas requiring grade raising to proposed subgrade level should be brought to grade using acceptable (compactable and inorganic) earth borrow or OPSS SSM. These materials should be placed in maximum 300 mm thick lifts and should be compacted to at least 95% of the materials standard Proctor maximum dry density using suitable compaction equipment.

The surface of the pavement subgrade should be crowned or sloped to promote drainage of the pavement granular structure towards perimeter swales or subdrains placed at the subgrade level

The following light-duty pavement design is recommended for the parking lot for this project and the following heavy-duty pavement design is recommended for any loading docks (if required):

Material		Light Duty Pavement Thickness of Pavement Elements (mm)	Heavy Duty Pavement Thickness of Pavement Elements (mm)
	Superpave 12.5 mm	60	40

Material		Light Duty Pavement Thickness of Pavement Elements (mm)	Heavy Duty Pavement Thickness of Pavement Elements (mm)
Bituminous Concrete OPSS 1150	Superpave 19.0 mm	-	50
Granular Material OPSS 1010	Granular A Base	150	150
	Granular B, Type II Subbase	300	450
	Prepared and Approved Subgrade		

The granular base and subbase materials should be uniformly compacted as per OPSS 310, Method A. The asphaltic concrete should be compacted in accordance with the procedures outlined in OPSS 310.

The asphaltic cement should consist of PG 58-34 and the design of the mixes should be based on a Traffic Category B.

The above pavement designs are based on the assumption that the pavement subgrade has been acceptably prepared (i.e., grade raise fill has been adequately compacted to the required density and the subgrade surface not disturbed by construction operations or precipitation). Depending on the actual conditions of the pavement subgrade at the time of construction, it could be necessary to increase the thickness of the subbase and/or to place a woven geotextile beneath the granular materials.

Where the new pavements will connect to existing pavements, the new pavement structures should be continued at least to the limits of construction, with any longitudinal transitions and/or tapers occurring thereafter. At these locations, the longitudinal transitions should be constructed by cutting the existing pavement structure vertically to the bottom of the existing subbase. The new granular layers should then be tapered up or down, as required, at a slope of 5 horizontal to 1 vertical to match the existing pavement structure. The asphaltic concrete does not need to be tapered between the new construction and the existing pavement. However, the asphaltic concrete of the existing pavement should be milled back an additional 300 mm to a depth of about 60 mm in areas where its thickness is greater than 100 mm, or matching the proposed surface course of the new asphaltic concrete. A tack coat should be provided, and the new surface course asphaltic concrete placed over the milled surface to form the new pavement joint. Where the existing pavement is less than 100 mm, then a butt joint on a vertical saw cut surface is acceptable. A tack coat should be placed on the vertical saw cut surface. The tack coat should be in accordance with the City SP F-3107.

5.10 Corrosion and Cement Type

One sample of soil from Borehole 20-03 was submitted to Eurofins Scientific for chemical analysis related to potential corrosion of exposed buried steel and concrete elements (corrosion and sulphate attack). The results of this testing are provided in Appendix C. The results indicate that there is a very high potential for sulphate attack. Concrete made with Type HS or HSb Portland cement should be used for concrete substructures.

The results also indicate a very high potential for corrosion of buried ferrous elements, which should be considered in the design of substructures and pile foundations.

6.0 SLOPE STABILITY ASSESSMENT

In general, six main components are typically involved in assessing the stability of a slope:

7) The geometry of the slope;
8) The geology of the slope (i.e., the composition of the various soil layers within the slope and their depth, thickness, and orientation);
9) The groundwater conditions (the groundwater levels and the hydraulic gradient/flow conditions);
10) The strength parameters for the soils;
11) The unit weights (i.e., densities) of the soils and waste within the slope; and,
12) External loading (i.e., surcharge, seismic forces).

For this assessment, the slope geometries used in the analyses were based on a topographic survey provided by the University of Ottawa. The two overall cross-sections selected for analysis are shown on Figure 1 (denoted as A-A’, B-B’).

The stability of the slope was evaluated using the SLOPE/W computer program. The Morgernstern-Price method, which satisfies both moment and force equilibrium, was used to compute a factor of safety. The factor of safety is defined as the ratio of the magnitude of the forces tending to resist failure to the magnitude of the forces tending to cause failure.

Theoretically, a slope with a factor of safety of less than 1.0 will undergo movement and one with a factor of safety of 1.0 or greater will not undergo movement. For analyses of the stability of slopes under static loading conditions, a factor of safety of greater than about 1.3 can be considered acceptable for this project and reflects inherent uncertainties related to the potential variability of the existing fill material and other subsurface variabilities, geometric imprecision, strain incompatibilities, and other risk factors.

The seismic loads imposed on a slope are modelled in a simplified manner by applying a horizontal “pseudo static” force to the soil mass. The “pseudo-static” force, F_s, is calculated as:

$$F_s = k_s \text{ x } M$$

Where: k_s = horizontal seismic coefficient; and,

M = mass of soil contained within the failure surface.

Since the site is used for recreational purposes, a minimum factor of safety of 1.1 is recommended under seismic loading conditions.

The seismic slope stability evaluations were carried out assuming that the design earthquake would correspond to an event with a 10% probability of occurrence in 50 years (i.e. the 475-year design earthquake). Based on the methodology outlined in 2012 OBC and VSP testing carried out in 2011, the Site Class was determined to be a Site Class C. Considering a site coefficient, F(PGA) = 1.0 and the firm ground PGA of 0.102 g, the ground surface PGA was calculated to be about 0.102 g. Therefore, a k_h value of 0.05 g, equal to one-half the ground surface PGA, was used in the slope stability analyses.

6.1 Material Properties

In general, the slopes range from approximately 5 to 6 metres in height, and the overall slope angle ranges from approximately 26 to 32 degrees from the horizontal 2H:1V and 1.6H:1V respectively.

The key material properties required to complete a stability analysis are the unit weight and shear strength of the materials. The shear strength of soil or waste is conventionally described using a Mohr-Coulomb criterion. This criterion describes the shear strength of a soil in terms of cohesive and frictional components. The magnitude of the frictional component depends on the stress acting perpendicular to the potential failure plane. From this criterion, the strength of a soil to resist shear stress (i.e., to resist sliding) is described by:

$$\tau = c' + \sigma' \tan \phi'$$

τ =Strength of the soil;

c' =Effective cohesion of the soil;

σ' =Effective normal stress (i.e., stress acting perpendicular to the shear plane); and,

ϕ' =Effective internal friction angle.

The characteristics of the soil stratigraphy within the slope was inferred from the results of the boreholes put down at the Site as part of the current and past investigations by Golder Associates Ltd. The borehole data indicates that the subsurface conditions on this site consist of surficial topsoil over a layer of cinders and ash in a matrix of sandy and silty fill, underlain by silty clay or alluvium over clayey silt and sandy silt glacial till.

The soil parameters used for the cinder and ash fill layer were based on results from previous investigations, and a visual examination of the samples from this fill layer. Upon a detailed visual inspection of all cinder and ash fill samples, it was observed that the fill consisted predominantly of silty sand and sandy silt with trace to some gravel and gravel sized pieces of brick and other miscellaneous gravel sized debris. On this basis, the fill layer is modelled as a silty sand with trace to some gravel.

The soil parameters used for the silty clay in the analyses were based on experience with similar soil in eastern Ontario.

A water level reading taken on July 21, 2020 in the wells installed in boreholes 20-01 to 20-03 provided a ground water elevation of about 55.0 m, indicating that the ground water level was within the glacial till deposit.

The material parameters adopted for the analysis are summarized in the table below.

Material	Bulk Unit Weight (kN/m^3)	Drained Parameters		Undrained Parameters
		Effective Cohesion (kPa)	Effective Internal Friction Angle (°)	Cohesion (kPa)
Existing Fill	19	0	33	-

Material	Bulk Unit Weight (kN/m^3)	Drained Parameters		Undrained Parameters
		Effective Cohesion (kPa)	Effective Internal Friction Angle (°)	Cohesion (kPa)
Silty Clay (Very Stiff to Stiff)	16	7.4	28.7	75
Glacial Till (Clayey Silt)	19	3	32	50
Glacial Till (Silty Sand or Sandy Silt)	19	0	33	-

6.2 Slope Stability Analysis Results

Two overall cross sections (identified as A-A’ and B-B’) were analyzed. The locations of the cross-sections are shown on Figure 1. The SLOPE/W outputs are shown in Appendix E.

The following table indicates the global factors of safety obtained for both static and dynamic analyses for the existing slopes.

Section	Global Factor of Safety		
	Static Drained	Static Undrained	Seismic
A-A’	1.4	1.4	1.3
B-B’	1.8	1.8	1.7

‘Hazard Lands’ associated with unstable slopes are defined as the table land adjacent to the slope for which there would be an inadequate ‘factor of safety’ against the land being affected by a slope failure. The Hazard Lands, as defined by Ministry of Natural Resources (MNR) guidelines and provincial planning policies, are unsuitable for development with buildings, roadways, parking areas or other infrastructure. In accordance with the MNR guidelines, the setback distance from the crest of an unstable slope to the Limit of Hazard Lands includes three components, as appropriate, namely:

1) A “Stable Slope Allowance”, which is determined as the limit beyond which there is an acceptable factor of safety (i.e., greater than about 1.5 static or 1.1 seismic) against slope instability.
2) An “Erosion Allowance”, to account for future movement of the slope toe, in the table land direction, as a result of erosion along the slope toe/creek bank.
3) An “Erosion Access Allowance” of 6 metres, to allow a corridor by which equipment could travel to access and repair a future slope failure. This Access Allowance is included in the determination of the Limit of Hazard Lands wherever the development could restrict future slope access.

Stable Slope Allowance must be applied to slopes that do not have an acceptable factor of safety (i.e., greater than about 1.5 static or 1.1 seismic). The static slope stability analysis for the slopes of the Rideau River indicates a factor of safety lower than 1.5 at section A-A’ and higher than 1.5 at section B-B’. The “pseudo-static” (or seismic) factors of safety at sections A-A’ and B-B are greater than 1.1.

Based on these analyses, the slopes in the area of section A-A’ will require a *Stable Slope Allowance* of 4 m as determined by the global slope stability analysis and the slopes in the area of section B-B are considered stable, and no *Stable Slope Allowance* is required.

An *Erosion Allowance* needs to be applied wherever there is active erosion, or the potential for active erosion based on the flow velocities. The width of the *Erosion Allowance* is described in the MNR guidelines and is a function of the soil type, state of erosion, and water course characteristics. Using Table 3 of the MNR Technical Guide, it was determined that an *Erosion Allowance* setback of 4 m is required. The following assumptions were made:

- The native soil type is assumed to be stiff/hard cohesive soil (clays, silt), coarse granular (gravels) tills,
- Based on an examination of the slope toe it was observed that the toe of the slope is adequately protected from erosion by previously placed open rip rap consisting of boulders, cobbles and broken pieces of concrete as shown on the photographs of the shoreline in Appendix E.
- The bankfull width was assumed to be greater than 30 m.

An *Erosion Access Allowance* needs to be applied when access to the slopes in the event of a slope failure is difficult. One example of a difficult slope access would be a row of semi-detached residential structures which back onto a slope. In the event of a failure, access would be provided by this 6 m access route behind the structures. At this site, there is a thick line of trees that could make access to the slope difficult, therefore a 6 m *Erosion Access Allowance* is required to the total setback distance from the top of the slope.

Based on the above, the setbacks shown in the following table should be applied to the top of the slopes:

Section	Stable Slope Allowance (m)	Erosion Allowance (m)	Erosion Access Allowance (m)	Setback Distance from Top of Slope (m)
A-A’	3	4	6	13
B-B’	0	4	6	10

7.0 ADDITIONAL CONSIDERATIONS

At the time of writing this report, only conceptual details related to the building were available. This information suggests this building will consist of up to 6 storeys with no basement levels. Golder Associates should review the final drawings and specifications for this project prior to tendering to confirm that the guidelines in this report have been adequately interpreted.

The construction activities could impact the existing adjacent structures and buildings. Appropriate damage assessments (pre and post condition surveys for example) should be carried out as necessary.

During construction, sufficient foundation inspections, subgrade inspections, in-situ density tests, materials testing, pile and rock anchor installation monitoring should be carried out to confirm that the conditions exposed are consistent with those encountered in the boreholes, and to monitor conformance to the pertinent project specifications. Concrete testing should be carried out in a CCIL certified laboratory.

The soils at this site are sensitive to disturbance from ponded water, construction traffic and frost. All bearing surfaces must be inspected by Golder prior to filling or concreting to ensure that strata having adequate bearing capacity have been reached and that the bearing surfaces have been properly prepared.

8.0 CLOSURE

We trust that this report provides sufficient geotechnical engineering information to facilitate the design of this project. If you have any questions regarding the contents of this report or require additional information, please do not hesitate to contact this office.

Signature Page

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The report is of a summary nature and is not intended to stand alone without reference to the instructions given to Golder by the Client, communications between Golder and the Client, and to any other reports prepared by Golder for the Client relative to the specific site described in the report. In order to properly understand the suggestions, recommendations and opinions expressed in this report, reference must be made to the whole of the report. Golder cannot be responsible for use of portions of the report without reference to the entire report.

Unless otherwise stated, the suggestions, recommendations and opinions given in this report are intended only for the guidance of the Client in the design of the specific project. The extent and detail of investigations, including the number of test holes, necessary to determine all of the relevant conditions which may affect construction costs would normally be greater than has been carried out for design purposes. Contractors bidding on, or undertaking the work, should rely on their own investigations, as well as their own interpretations of the factual data presented in the report, as to how subsurface conditions may affect their work, including but not limited to proposed construction techniques, schedule, safety and equipment capabilities.

Soil, Rock and Groundwater Conditions: Classification and identification of soils, rocks, and geologic units have been based on commonly accepted methods employed in the practice of geotechnical engineering and related disciplines. Classification and identification of the type and condition of these materials or units involves judgment, and boundaries between different soil, rock or geologic types or units may be transitional rather than abrupt. Accordingly, Golder does not warrant or guarantee the exactness of the descriptions.

IMPORTANT INFORMATION AND LIMITATIONS OF THIS REPORT (cont'd)

Special risks occur whenever engineering or related disciplines are applied to identify subsurface conditions and even a comprehensive investigation, sampling and testing program may fail to detect all or certain subsurface conditions. The environmental, geologic, geotechnical, geochemical and hydrogeologic conditions that Golder interprets to exist between and beyond sampling points may differ from those that actually exist. In addition to soil variability, fill of variable physical and chemical composition can be present over portions of the site or on adjacent properties. **The professional services retained for this project include only the geotechnical aspects of the subsurface conditions at the site, unless otherwise specifically stated and identified in the report.** The presence or implication(s) of possible surface and/or subsurface contamination resulting from previous activities or uses of the site and/or resulting from the introduction onto the site of materials from off-site sources are outside the terms of reference for this project and have not been investigated or addressed.

Soil and groundwater conditions shown in the factual data and described in the report are the observed conditions at the time of their determination or measurement. Unless otherwise noted, those conditions form the basis of the recommendations in the report. Groundwater conditions may vary between and beyond reported locations and can be affected by annual, seasonal and meteorological conditions. The condition of the soil, rock and groundwater may be significantly altered by construction activities (traffic, excavation, groundwater level lowering, pile driving, blasting, etc.) on the site or on adjacent sites. Excavation may expose the soils to changes due to wetting, drying or frost. Unless otherwise indicated the soil must be protected from these changes during construction.

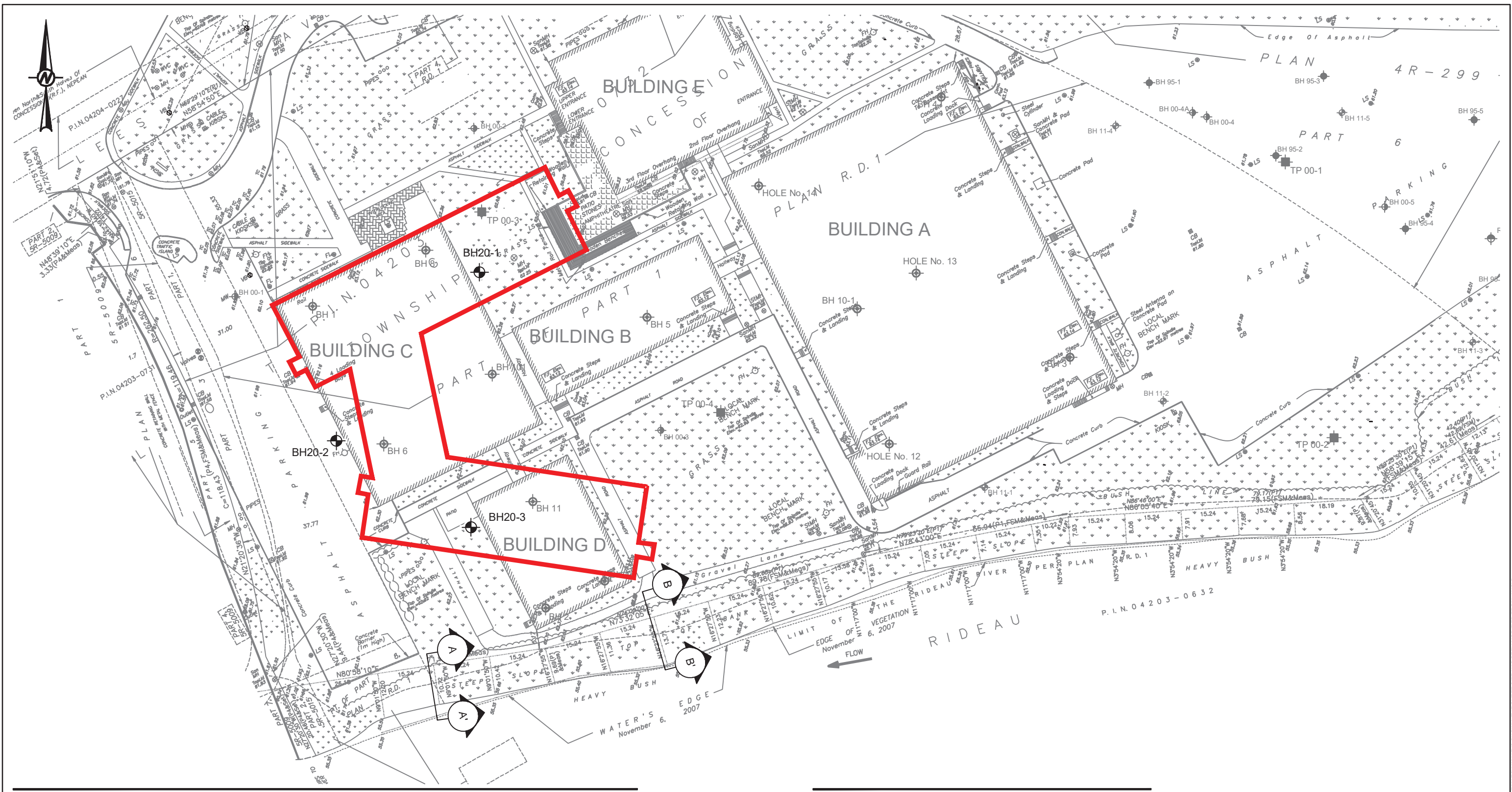
Sample Disposal: Golder will dispose of all uncontaminated soil and/or rock samples 90 days following issue of this report or, upon written request of the Client, will store uncontaminated samples and materials at the Client's expense. In the event that actual contaminated soils, fills or groundwater are encountered or are inferred to be present, all contaminated samples shall remain the property and responsibility of the Client for proper disposal.

Follow-Up and Construction Services: All details of the design were not known at the time of submission of Golder's report. Golder should be retained to review the final design, project plans and documents prior to construction, to confirm that they are consistent with the intent of Golder's report.

During construction, Golder should be retained to perform sufficient and timely observations of encountered conditions to confirm and document that the subsurface conditions do not materially differ from those interpreted conditions considered in the preparation of Golder's report and to confirm and document that construction activities do not adversely affect the suggestions, recommendations and opinions contained in Golder's report. Adequate field review, observation and testing during construction are necessary for Golder to be able to provide letters of assurance, in accordance with the requirements of many regulatory authorities. In cases where this recommendation is not followed, Golder's responsibility is limited to interpreting accurately the information encountered at the borehole locations, at the time of their initial determination or measurement during the preparation of the Report.

Changed Conditions and Drainage: Where conditions encountered at the site differ significantly from those anticipated in this report, either due to natural variability of subsurface conditions or construction activities, it is a condition of this report that Golder be notified of any changes and be provided with an opportunity to review or revise the recommendations within this report. Recognition of changed soil and rock conditions requires experience and it is recommended that Golder be employed to visit the site with sufficient frequency to detect if conditions have changed significantly.

Drainage of subsurface water is commonly required either for temporary or permanent installations for the project. Improper design or construction of drainage or dewatering can have serious consequences. Golder takes no responsibility for the effects of drainage unless specifically involved in the detailed design and construction monitoring of the system.



LEGEND

- ⊕ GOLDER 2020 PROPOSED BOREHOLE LOCATIONS
- ⊕ APPROXIMATE BOREHOLE LOCATION IN PLAN, PREVIOUS INVESTIGATION BY BY GOLDER ASSOCIATES LTD. ' REPORT No. 11-1121-0057
- ▨ APPROXIMATE TEST PIT LOCATION IN PLAN, PREVIOUS INVESTIGATION BY BY GOLDER ASSOCIATES LTD. ' REPORT No. 11-1121-0057
- ⊕ APPROXIMATE BOREHOLE LOCATION IN PLAN, PREVIOUS INVESTIGATION BY GOLDER ASSOCIATES LTD., REPORT No. 10-1121-0117
- ● APPROXIMATE BOREHOLE LOCATION IN PLAN, PREVIOUS INVESTIGATION BY McROSTIE GENEST ST-LOUIS, REPORT No. SF-4306
- ⊕ APPROXIMATE BOREHOLE LOCATION IN PLAN, PREVIOUS INVESTIGATION BY McROSTIE GENEST ST-LOUIS, REPORT No. SF-624
- ⊕ APPROXIMATE BOREHOLE LOCATION IN PLAN, PREVIOUS INVESTIGATION BY GOLDER ASSOCIATES LTD., REPORT No. 001-2721
- ■ APPROXIMATE TEST PIT LOCATION IN PLAN, PREVIOUS INVESTIGATION BY GOLDER ASSOCIATES LTD., REPORT No. 001-2721
- ⊕ APPROXIMATE MONITORING WELL LOCATION IN PLAN, PREVIOUS INVESTIGATION BY CH2M HILL, REPORT No. 120209
- ⊕ APPROXIMATE BOREHOLE LOCATION IN PLAN, PREVIOUS INVESTIGATION BY CANVIRO CONSULTANTS, 1988
- 🟥 APPROXIMATE PROPOSED NEW BUILDING FOOTPRINT

REFERENCE(S)

1. BASE PLAN SUPPLIED IN ELECTRONIC FORMAT BY BARRY J. HOBIN AND ASSOCIATES ARCHITECHTS INC. DATED JULY 19, 2011.

0 25 50
1:1,000 METRES

CLIENT
UNIVERSITY OF OTTAWA

PROJECT
GEOTECHNICAL INVESTIGATION

CONSULTANT
GOLDER

YYYY-MM-DD	2020-09-04
DESIGNED	----
PREPARED	ABD/JM
REVIEWED	BB
APPROVED	CH

TITLE
200 LEES AVENUE,
OTTAWA, ONTARIO

PROJECT NO.	CONTROL	REV.	FIGURE
20144766	0001	0	1

25 mm
0

IF THIS MEASUREMENT DOES NOT MATCH WHAT IS SHOWN, THE SHEET SIZE HAS BEEN MODIFIED FROM: ANSI B

APPENDIX A

- Current Investigation Borehole Records

- Bedrock Core Photographs (Boreholes 20-01 to 20-03)

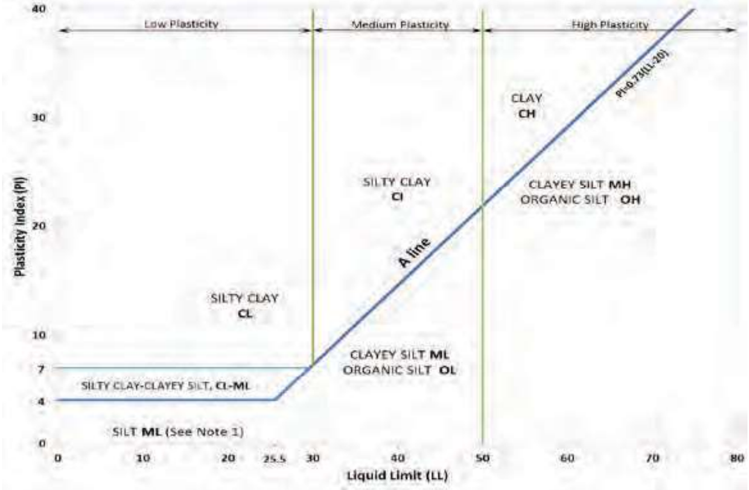
- Previous Borehole Records

METHOD OF SOIL CLASSIFICATION

The Golder Associates Ltd. Soil Classification System is based on the Unified Soil Classification System (USCS)

Organic or Inorganic	Soil Group	Type of Soil		Gradation or Plasticity	$Cu = \frac{D_{60}}{D_{10}}$	$Cc = \frac{(D_{30})^2}{D_{10}xD_{60}}$	Organic Content	USCS Group Symbol	Group Name
INORGANIC (Organic Content ≤30% by mass)	COARSE-GRAINED SOILS (>50% by mass is larger than 0.075 mm)	GRAVELS (>50% by mass of coarse fraction is larger than 4.75 mm)	Gravels with ≤12% fines (by mass)	Poorly Graded	<4	≤1 or ≥3	≤30%	GP	GRAVEL
				Well Graded	≥4	1 to 3		GW	GRAVEL
			Gravels with >12% fines (by mass)	Below A Line	n/a			GM	SILTY GRAVEL
				Above A Line	n/a			GC	CLAYEY GRAVEL
		SANDS (≥50% by mass of coarse fraction is smaller than 4.75 mm)	Sands with ≤12% fines (by mass)	Poorly Graded	<6	≤1 or ≥3		SP	SAND
				Well Graded	≥6	1 to 3		SW	SAND
			Sands with >12% fines (by mass)	Below A Line	n/a			SM	SILTY SAND
				Above A Line	n/a			SC	CLAYEY SAND

Organic or Inorganic	Soil Group	Type of Soil	Laboratory Tests	Field Indicators					Organic Content	USCS Group Symbol	Primary Name
				Dilatancy	Dry Strength	Shine Test	Thread Diameter	Toughness (of 3 mm thread)			
INORGANIC (Organic Content ≤30% by mass)	FINE-GRAINED SOILS (≥50% by mass is smaller than 0.075 mm)	SILTS (Non-Plastic or PI and LL plot below A-Line on Plasticity Chart below)	Liquid Limit <50	Rapid	None	None	>6 mm	N/A (can't roll 3 mm thread)	<5%	ML	SILT
				Slow	None to Low	Dull	3mm to 6 mm	None to low	<5%	ML	CLAYEY SILT
				Slow to very slow	Low to medium	Dull to slight	3mm to 6 mm	Low	5% to 30%	OL	ORGANIC SILT
			Liquid Limit ≥50	Slow to very slow	Low to medium	Slight	3mm to 6 mm	Low to medium	<5%	MH	CLAYEY SILT
				None	Medium to high	Dull to slight	1 mm to 3 mm	Medium to high	5% to 30%	OH	ORGANIC SILT
		CLAYS (PI and LL plot above A-Line on Plasticity Chart below)	Liquid Limit <30	None	Low to medium	Slight to shiny	~ 3 mm	Low to medium	0% to 30% (see Note 2)	CL	SILTY CLAY
			Liquid Limit 30 to 50	None	Medium to high	Slight to shiny	1 mm to 3 mm	Medium		CI	SILTY CLAY
			Liquid Limit ≥50	None	High	Shiny	<1 mm	High		CH	CLAY
HIGHLY ORGANIC SOILS (Organic Content >30% by mass)		Peat and mineral soil mixtures							30% to 75%	PT	SILTY PEAT, SANDY PEAT
		Predominantly peat, may contain some mineral soil, fibrous or amorphous peat							75% to 100%		PEAT



Note 1 – Fine grained materials with PI and LL that plot in this area are named (ML) SILT with slight plasticity. Fine-grained materials which are non-plastic (i.e. a PL cannot be measured) are named SILT.

Note 2 – For soils with <5% organic content, include the descriptor "trace organics" for soils with between 5% and 30% organic content include the prefix "organic" before the Primary name.

Dual Symbol — A dual symbol is two symbols separated by a hyphen, for example, GP-GM, SW-SC and CL-ML.

For non-cohesive soils, the dual symbols must be used when the soil has between 5% and 12% fines (i.e. to identify transitional material between "clean" and "dirty" sand or gravel.

For cohesive soils, the dual symbol must be used when the liquid limit and plasticity index values plot in the CL-ML area of the plasticity chart (see Plasticity Chart at left).

Borderline Symbol — A borderline symbol is two symbols separated by a slash, for example, CL/CI, GM/SM, CL/ML.

A borderline symbol should be used to indicate that the soil has been identified as having properties that are on the transition between similar materials. In addition, a borderline symbol may be used to indicate a range of similar soil types within a stratum.

ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES AND TEST PITS

PARTICLE SIZES OF CONSTITUENTS

Soil Constituent	Particle Size Description	Millimetres	Inches (US Std. Sieve Size)
BOULDERS	Not Applicable	>300	>12
COBBLES	Not Applicable	75 to 300	3 to 12
GRAVEL	Coarse Fine	19 to 75 4.75 to 19	0.75 to 3 (4) to 0.75
SAND	Coarse Medium Fine	2.00 to 4.75 0.425 to 2.00 0.075 to 0.425	(10) to (4) (40) to (10) (200) to (40)
SILT/CLAY	Classified by plasticity	<0.075	< (200)

MODIFIERS FOR SECONDARY AND MINOR CONSTITUENTS

Percentage by Mass	Modifier
>35	Use 'and' to combine major constituents (*i.e.,* SAND and GRAVEL)
> 12 to 35	Primary soil name prefixed with "gravelly, sandy, SILTY, CLAYEY" as applicable
> 5 to 12	some
≤ 5	trace

PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

The number of blows by a 63.5 kg (140 lb) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) split-spoon sampler for a distance of 300 mm (12 in.). Values reported are as recorded in the field and are uncorrected.

Cone Penetration Test (CPT)

An electronic cone penetrometer with a 60° conical tip and a project end area of 10 cm^2 pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (q_t), porewater pressure (u) and sleeve frictions are recorded electronically at 25 mm penetration intervals.

Dynamic Cone Penetration Resistance (DCPT); N_d:

The number of blows by a 63.5 kg (140 lb) hammer dropped 760 mm (30 in.) to drive uncased a 50 mm (2 in.) diameter, 60° cone attached to "A" size drill rods for a distance of 300 mm (12 in.).

PH:	Sampler advanced by hydraulic pressure
PM:	Sampler advanced by manual pressure
WH:	Sampler advanced by static weight of hammer
WR:	Sampler advanced by weight of sampler and rod

SAMPLES

AS	Auger sample
BS	Block sample
CS	Chunk sample
DD	Diamond Drilling
DO or DP	Seamless open ended, driven or pushed tube sampler – note size
DS	Denison type sample
GS	Grab Sample
MC	Modified California Samples
MS	Modified Shelby (for frozen soil)
RC	Rock core
SC	Soil core
SS	Split spoon sampler – note size
ST	Slotted tube
TO	Thin-walled, open – note size (Shelby tube)
TP	Thin-walled, piston – note size (Shelby tube)
WS	Wash sample

SOIL TESTS

w	water content
PL , w_p	plastic limit
LL , w_L	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test[1]
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement[1]
D_R	relative density (specific gravity, Gs)
DS	direct shear test
GS	specific gravity
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
SO_4	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V (FV)	field vane (LV-laboratory vane test)
γ	unit weight

1. Tests anisotropically consolidated prior to shear are shown as CAD, CAU.

NON-COHESIVE (COHESIONLESS) SOILS

Compactness[2]

Term	SPT 'N' (blows/0.3m)[1]
Very Loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very Dense	>50

1. SPT 'N' in accordance with ASTM D1586, uncorrected for the effects of overburden pressure.
2. Definition of compactness terms are based on SPT 'N' ranges as provided in Terzaghi, Peck and Mesri (1996). Many factors affect the recorded SPT 'N' value, including hammer efficiency (which may be greater than 60% in automatic trip hammers), overburden pressure, groundwater conditions, and grainsize. As such, the recorded SPT 'N' value(s) should be considered only an approximate guide to the soil compactness. These factors need to be considered when evaluating the results, and the stated compactness terms should not be relied upon for design or construction.

Field Moisture Condition

Term	Description
Dry	Soil flows freely through fingers.
Moist	Soils are darker than in the dry condition and may feel cool.
Wet	As moist, but with free water forming on hands when handled.

COHESIVE SOILS

Consistency

Term	Undrained Shear Strength (kPa)	SPT 'N'[1,2] (blows/0.3m)
Very Soft	<12	0 to 2
Soft	12 to 25	2 to 4
Firm	25 to 50	4 to 8
Stiff	50 to 100	8 to 15
Very Stiff	100 to 200	15 to 30
Hard	>200	>30

1. SPT 'N' in accordance with ASTM D1586, uncorrected for overburden pressure effects; approximate only.
2. SPT 'N' values should be considered ONLY an approximate guide to consistency; for sensitive clays (e.g., Champlain Sea clays), the N-value approximation for consistency terms does NOT apply. Rely on direct measurement of undrained shear strength or other manual observations.

Water Content

Term	Description
w < PL	Material is estimated to be drier than the Plastic Limit.
w ~ PL	Material is estimated to be close to the Plastic Limit.
w > PL	Material is estimated to be wetter than the Plastic Limit.

LIST OF SYMBOLS

Unless otherwise stated, the symbols employed in the report are as follows:

I. GENERAL

π	3.1416
ln x	natural logarithm of x
$\log_{10}$	x or log x, logarithm of x to base 10
g	acceleration due to gravity
t	time

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. in stress: $\Delta\,\sigma$
ε	linear strain
ε_v	volumetric strain
η	coefficient of viscosity
υ	Poisson’s ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma - u$)
σ'_{vo}	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stress (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

$\rho(\gamma)$	bulk density (bulk unit weight)*
$\rho_d(\gamma_d)$	dry density (dry unit weight)
$\rho_w(\gamma_w)$	density (unit weight) of water
$\rho_s(\gamma_s)$	density (unit weight) of solid particles
γ'	unit weight of submerged soil ($\gamma' = \gamma - \gamma_w$)
D_R	relative density (specific gravity) of solid particles ($D_R = \rho_s / \rho_w$) (formerly G_s)
e	void ratio
n	porosity
S	degree of saturation

(a) Index Properties (continued)

w	water content
w_l or LL	liquid limit
w_p or PL	plastic limit
I_p or PI	plasticity index = $(w_l - w_p)$
NP	non-plastic
w_s	shrinkage limit
I_L	liquidity index = $(w - w_p) / I_p$
I_C	consistency index = $(w_l - w) / I_p$
e_{max}	void ratio in loosest state
e_{min}	void ratio in densest state
I_D	density index = $(e_{max} - e) / (e_{max} - e_{min})$ (formerly relative density)

(b) Hydraulic Properties

h	hydraulic head or potential
q	rate of flow
v	velocity of flow
i	hydraulic gradient
k	hydraulic conductivity (coefficient of permeability)
j	seepage force per unit volume

(c) Consolidation (one-dimensional)

C_c	compression index (normally consolidated range)
C_r	recompression index (over-consolidated range)
C_s	swelling index
C_α	secondary compression index
m_v	coefficient of volume change
c_v	coefficient of consolidation (vertical direction)
c_h	coefficient of consolidation (horizontal direction)
T_v	time factor (vertical direction)
U	degree of consolidation
σ'_p	pre-consolidation stress
OCR	over-consolidation ratio = σ'_p / σ'_{vo}

(d) Shear Strength

τ_p, τ_r	peak and residual shear strength
ϕ'	effective angle of internal friction
δ	angle of interface friction
μ	coefficient of friction = $\tan \delta$
c'	effective cohesion
c_u, s_u	undrained shear strength ($\phi = 0$ analysis)
p	mean total stress $(\sigma_1 + \sigma_3)/2$
p′	mean effective stress $(\sigma'_1 + \sigma'_3)/2$
q	$(\sigma_1 - \sigma_3)/2$ or $(\sigma'_1 - \sigma'_3)/2$
q_u	compressive strength $(\sigma_1 - \sigma_3)$
S_t	sensitivity

Notes: 1 $\tau = c' + \sigma' \tan \phi'$

2 shear strength = (compressive strength)/2

* Density symbol is ρ. Unit weight symbol is γ where $\gamma = \rho g$ (i.e. mass density multiplied by acceleration due to gravity)

LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

WEATHERINGS STATE

Fresh: no visible sign of rock material weathering.

Faintly weathered: weathering limited to the surface of major discontinuities.

Slightly weathered: penetrative weathering developed on open discontinuity surfaces but only slight weathering of rock material.

Moderately weathered: weathering extends throughout the rock mass but the rock material is not friable.

Highly weathered: weathering extends throughout rock mass and the rock material is partly friable.

Completely weathered: rock is wholly decomposed and in a friable condition but the rock and structure are preserved.

BEDDING THICKNESS

Description	Bedding Plane Spacing
Very thickly bedded	Greater than 2 m
Thickly bedded	0.6 m to 2 m
Medium bedded	0.2 m to 0.6 m
Thinly bedded	60 mm to 0.2 m
Very thinly bedded	20 mm to 60 mm
Laminated	6 mm to 20 mm
Thinly laminated	Less than 6 mm

JOINT OR FOLIATION SPACING

Description	Spacing
Very wide	Greater than 3 m
Wide	1 m to 3 m
Moderately close	0.3 m to 1 m
Close	50 mm to 300 mm
Very close	Less than 50 mm

GRAIN SIZE

Term	Size*
Very Coarse Grained	Greater than 60 mm
Coarse Grained	2 mm to 60 mm
Medium Grained	60 microns to 2 mm
Fine Grained	2 microns to 60 microns
Very Fine Grained	Less than 2 microns

Note: * Grains greater than 60 microns diameter are visible to the naked eye.

CORE CONDITION

Total Core Recovery (TCR)

The percentage of solid drill core recovered regardless of quality or length, measured relative to the length of the total core run.

Solid Core Recovery (SCR)

The percentage of solid drill core, regardless of length, recovered at full diameter, measured relative to the length of the total core run.

Rock Quality Designation (RQD)

The percentage of solid drill core, greater than 100 mm length, as measured along the centerline axis of the core, relative to the length of the total core run. RQD varies from 0% for completely broken core to 100% for core in solid segments.

DISCONTINUITY DATA

Fracture Index

A count of the number of naturally occuring discontinuities (physical separations) in the rock core. Mechanically induced breaks caused by drilling are not included.

Dip with Respect to Core Axis

The angle of the discontinuity relative to the axis (length) of the core. In a vertical borehole a discontinuity with a 90^{o} angle is horizontal.

Description and Notes

An abbreviation description of the discontinuities, whether naturally occurring separations such as fractures, bedding planes and foliation planes and mechanically separated bedding or foliation surfaces. Additional information concerning the nature of fracture surfaces and infillings are also noted.

Abbreviations

JN	Joint	PL	Planar
FLT	Fault	CU	Curved
SH	Shear	UN	Undulating
VN	Vein	IR	Irregular
FR	Fracture	K	Slickensided
SY	Stylolite	PO	Polished
BD	Bedding	SM	Smooth
FO	Foliation	SR	Slightly Rough
CO	Contact	RO	Rough
AXJ	Axial Joint	VR	Very Rough
KV	Karstic Void		
MB	Mechanical Break		

PROJECT: 20144766

RECORD OF BOREHOLE: 20-01

SHEET 1 OF 3

LOCATION: N 5031000.8 ;E 369909.0

BORING DATE: July 6, 2020

DATUM: Geodetic

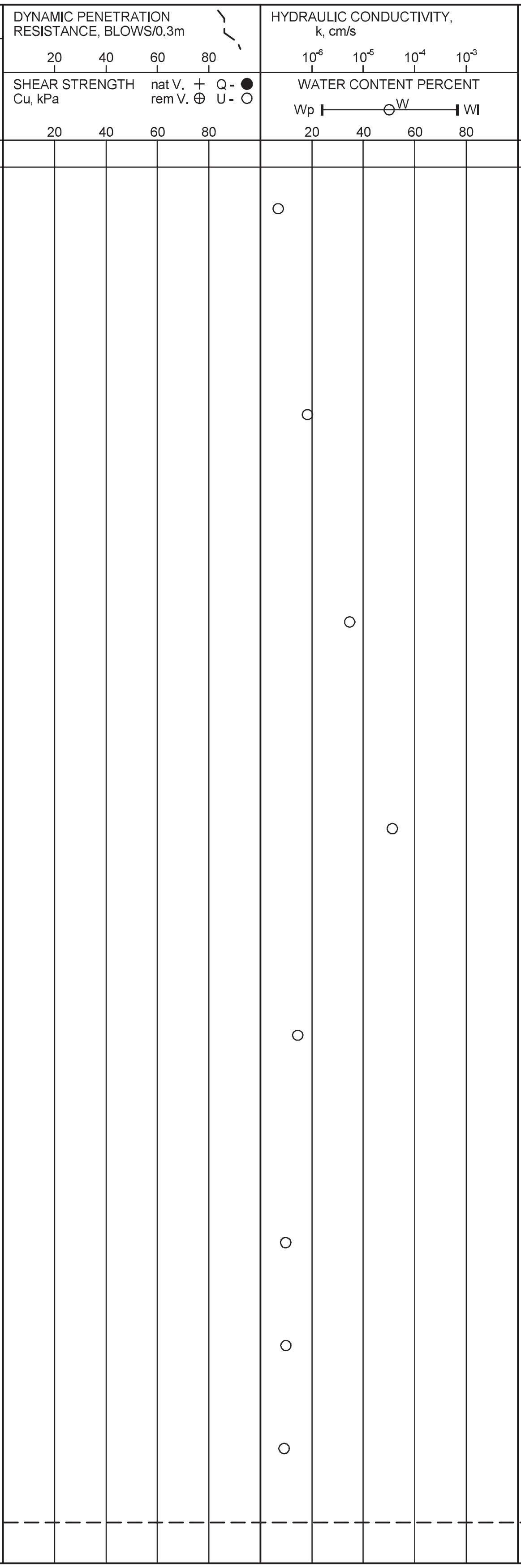
SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

BORING METHOD: Power Auger, 200 mm Diam. (Hollow Stem)

DESCRIPTION	ELEV. / DEPTH (m)
GROUND SURFACE	62.34
FILL/TOPSOIL - (ML) sandy SILT; brown, contains wood; non-cohesive	0.00
FILL - (ML) sandy SILT, trace gravel; brown to dark brown, contains rootlets, ash and cinders; non-cohesive, moist, compact to very loose	0.08
FILL - (ML) gravelly sandy SILT, some plastic fines; dark grey, contains cinders and ash; non-cohesive, moist, loose to very loose	59.29 3.05
FILL - (SM) gravelly SILTY SAND, some plastic fines; grey, contains cinders and ash; non-cohesive, moist, very, loose	57.77 4.57
(SM) gravelly SILTY SAND, some clay (GLACIAL TILL); wet, very loose to compact	57.15 5.19

NUMBER	TYPE	BLOWS/0.30m	ADDITIONAL LAB. TESTING
1	SS	14	
2	SS	3	
3	SS	2	
4	SS	4	
5	SS	4	
6	SS	2	
7	SS	2	
8	SS	6	
9	SS	15	
10	SS	9	
11	SS	10	MH
12	SS	2	
13	SS	12	
14	SS	WH	



PIEZOMETER OR STANDPIPE INSTALLATION: Flush Mount Casing; Bentonite Seal

CONTINUED NEXT PAGE

DEPTH SCALE 1 : 50

LOGGED: RA

CHECKED: BB

PROJECT: 20144766

RECORD OF BOREHOLE: 20-01

SHEET 2 OF 3

LOCATION: N 5031000.8 ;E 369909.0

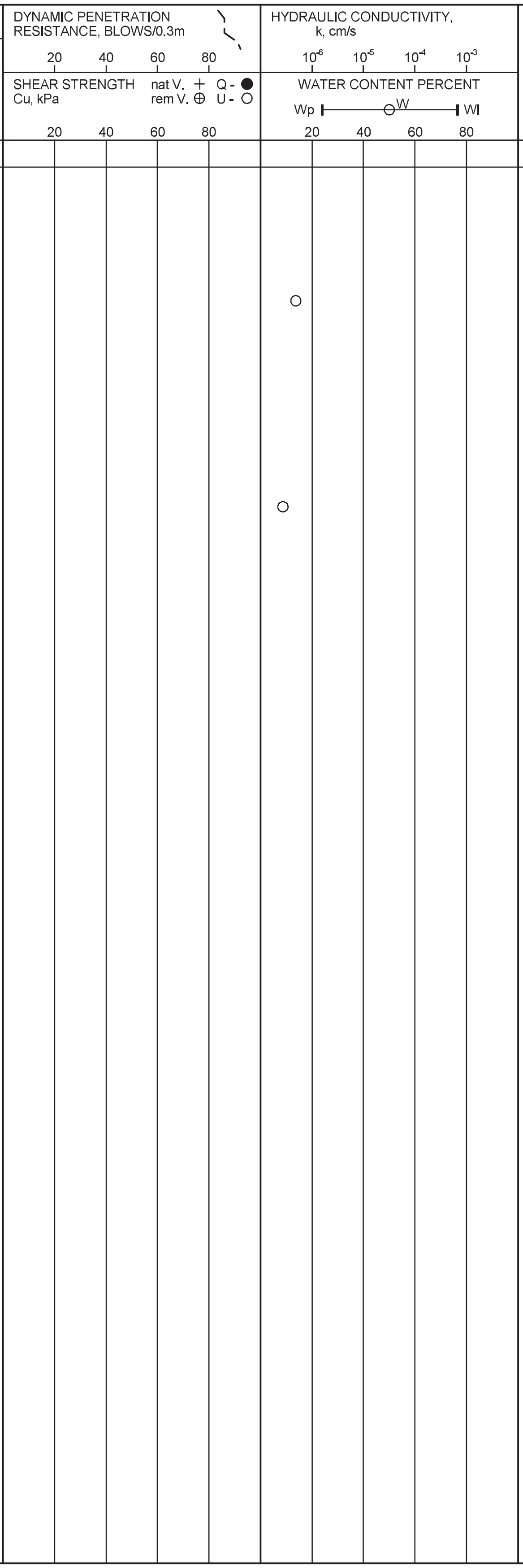
BORING DATE: July 6, 2020

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.30m	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		--- *CONTINUED FROM PREVIOUS PAGE* ---						
10	Power Auger 200 mm Diam. (Hollow Stem)	(SM) gravelly SILTY SAND, some clay (GLACIAL TILL); wet, very loose to compact		14	SS	WH		Bentonite Seal
11				15	SS	2		
12				16	SS	3		
				17	SS	>50		
			49.48 12.86					
13		Borehole continued on RECORD OF DRILLHOLE 20-01						
14								
15								
16								
17								
18								
19								
20								



DEPTH SCALE

1 : 50

LOGGED: RA

CHECKED: BB

PROJECT: 20144766

RECORD OF DRILLHOLE: 20-01

SHEET 3 OF 3

LOCATION: N 5031000.8 ;E 369909.0

DRILLING DATE: July 6, 2020

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

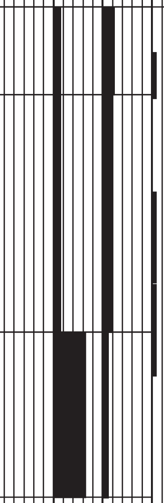
DRILL RIG: CME-75

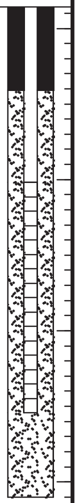
DRILLING CONTRACTOR: Grenville Drilling

JN - Joint
FLT - Fault
SHR - Shear
VN - Vein
CJ - Conjugate
BD - Bedding
FO - Foliation
CO - Contact
OR - Orthogonal
CL - Cleavage
PL - Planar
CU - Curved
UN - Undulating
ST - Stepped
IR - Irregular
PO - Polished
K - Slickensided
SM - Smooth
Ro - Rough
MB - Mechanical Break
BR - Broken Rock

NOTE: For additional abbreviations refer to list of abbreviations & symbols.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	FLUSH COLOUR % RETURN	RECOVERY TOTAL CORE % 80 60 40 20	RECOVERY SOLID CORE % 80 60 40 20	R.Q.D. % 80 60 40 20	FRACT. INDEX PER 0.25 m 5 10 15 20	DIP w.r.t. CORE AXIS 0 30 60 90	TYPE AND SURFACE DESCRIPTION	Jcon	Jr	Ja	HYDRAULIC CONDUCTIVITY K, cm/sec 10^{-6} 10^{-5} 10^{-4} 10^{-3}	Diametral Point Load Index (MPa) 2 4 6	RMC -Q' AVG.	
		GROUND SURFACE		49.48															
13	Rotary Drill HQ Core	Slightly weathered to fresh, black, fine grained SHALE		12.86	1														Bentonite Seal
14					2														Silica Sand 51 mm Diam. PVC #10 Slot Screen
15					3														
16				46.24 16.10															
		End of Drillhole																	WL in Screen at Elev. 54.94 m on July 21, 2020
17																			
18																			
19																			
20																			
21																			
22																			





MIS-RCK 004 20144766.GPJ GAL-MISS.GDT 9/4/20 JEM

DEPTH SCALE
1 : 50

GOLDER

LOGGED: RA
CHECKED: BB

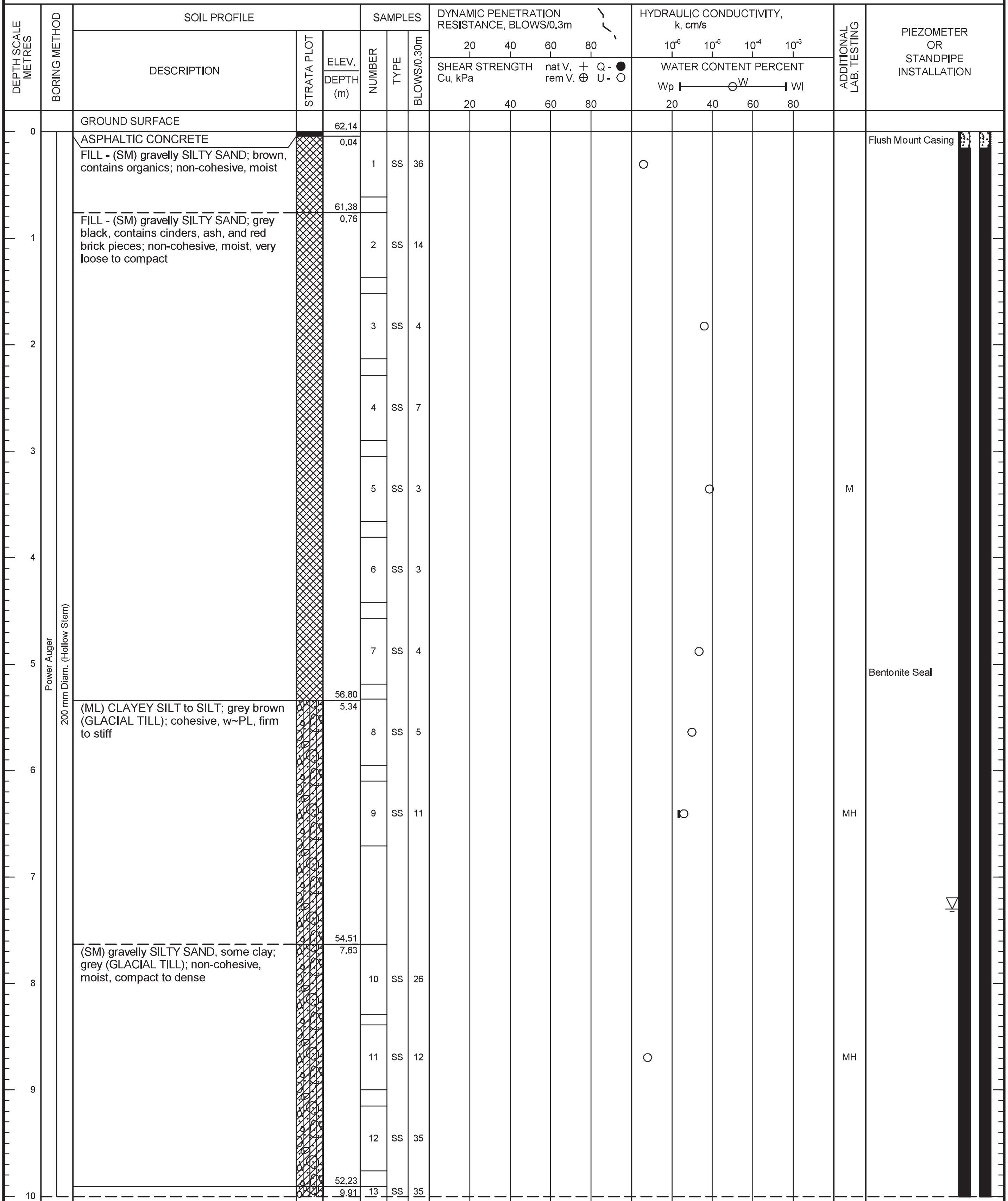
RECORD OF BOREHOLE: 20-02

PROJECT: 20144766
SHEET 1 OF 3
LOCATION: N 5030948.6 ;E 369860.9
BORING DATE: July 9, 2020
DATUM: Geodetic
SAMPLER HAMMER, 64kg; DROP, 760mm
PENETRATION TEST HAMMER, 64kg; DROP, 760mm

BORING METHOD: Power Auger, 200 mm Diam. (Hollow Stem)

SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)
GROUND SURFACE	62.14
ASPHALTIC CONCRETE	0.04
FILL - (SM) gravelly SILTY SAND; brown, contains organics; non-cohesive, moist	
FILL - (SM) gravelly SILTY SAND; grey black, contains cinders, ash, and red brick pieces; non-cohesive, moist, very loose to compact	61.38 0.76
(ML) CLAYEY SILT to SILT; grey brown (GLACIAL TILL); cohesive, w~PL, firm to stiff	56.80 5.34
(SM) gravelly SILTY SAND, some clay; grey (GLACIAL TILL); non-cohesive, moist, compact to dense	54.51 7.63
	52.23 9.91

SAMPLES NUMBER	TYPE	BLOWS/0.30m	ADDITIONAL LAB. TESTING
1	SS	36	
2	SS	14	
3	SS	4	
4	SS	7	
5	SS	3	M
6	SS	3	
7	SS	4	
8	SS	5	
9	SS	11	MH
10	SS	26	
11	SS	12	MH
12	SS	35	
13	SS	35	



CONTINUED NEXT PAGE

DEPTH SCALE
1 : 50

GOLDER

LOGGED: RA
CHECKED: BB

PROJECT: 20144766

RECORD OF BOREHOLE: 20-02

SHEET 2 OF 3

LOCATION: N 5030948.6 ;E 369860.9

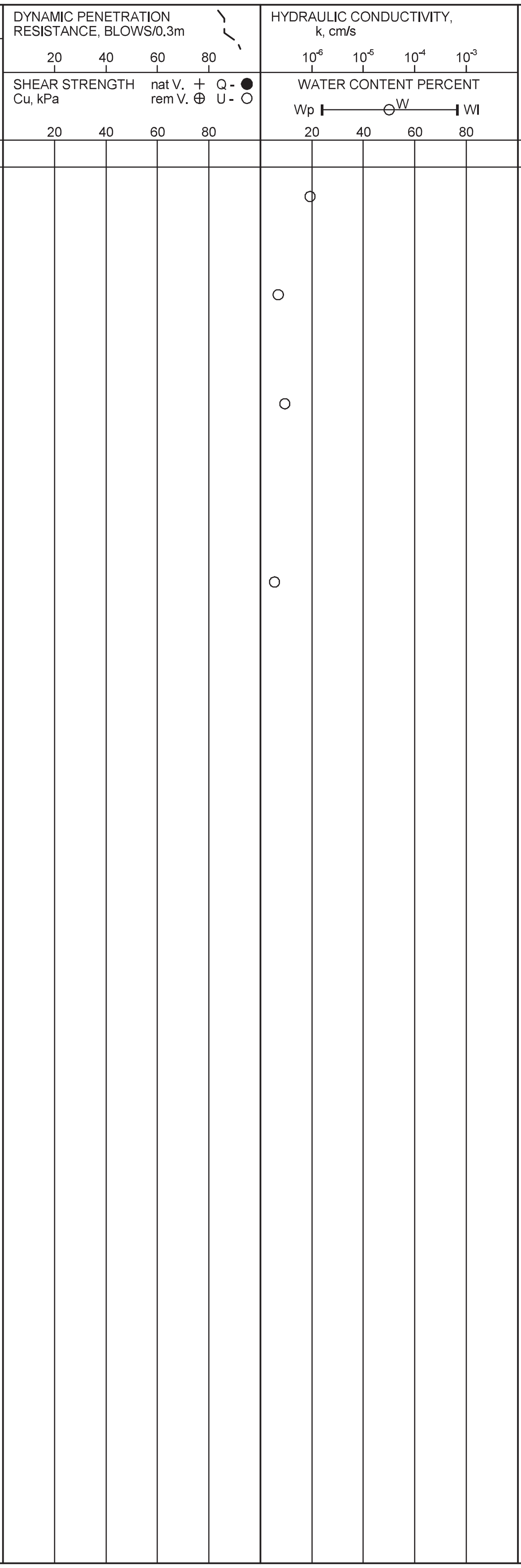
BORING DATE: July 9, 2020

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	SAMPLES TYPE	BLOWS/0.30m	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		--- CONTINUED FROM PREVIOUS PAGE ---							
10	Power Auger 200 mm Diam. (Hollow Stem)	(ML) sandy SILT to SILT, some gravel; grey, contains shale fragments (GLACIAL TILL); non-cohesive, wet, dense to very dense			13	SS	35		
11					14	SS	39		
					15	SS	78		
12					16	SS	>50		Bentonite Seal
13					17	SS	>50		
				48.28 13.86	18	SS	>50		
14		Borehole continued on RECORD OF DRILLHOLE 20-02							
15									
16									
17									
18									
19									
20									



MIS-BHS 001 20144766.GPJ GAL-MIS.GDT 9/4/20 JEM

DEPTH SCALE

1 : 50

GOLDER

LOGGED: RA

CHECKED: BB

PROJECT: 20144766

RECORD OF DRILLHOLE: 20-02

SHEET 3 OF 3

LOCATION: N 5030948.6 ;E 369860.9

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: July 9, 2020

DRILL RIG: CME-75

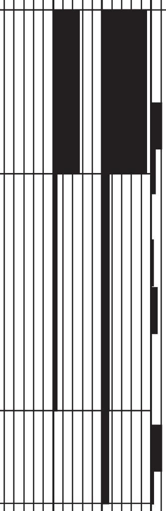
DRILLING CONTRACTOR: Grenville Drilling

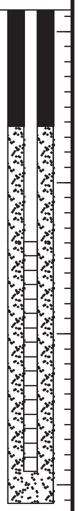
DATUM: Geodetic

JN - Joint
FLT - Fault
SHR - Shear
VN - Vein
CJ - Conjugate
BD - Bedding
FO - Foliation
CO - Contact
OR - Orthogonal
CL - Cleavage
PL - Planar
CU - Curved
UN - Undulating
ST - Stepped
IR - Irregular
PO - Polished
K - Slickensided
SM - Smooth
Ro - Rough
MB - Mechanical Break
BR - Broken Rock

NOTE: For additional abbreviations refer to list of abbreviations & symbols.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	COLOUR % RETURN FLUSH	RECOVERY TOTAL CORE % 80 60 40 20	RECOVERY SOLID CORE % 80 60 40 20	R.Q.D. % 80 60 40 20	FRACT. INDEX PER 0.25 m 5 10 15 20	DIP w.r.t. CORE AXIS 0 30 60 90	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	Jcon	Jr	Ja	HYDRAULIC CONDUCTIVITY K, cm/sec 10^{-6} 10^{-5} 10^{-4} 10^{-3}	Diametral Point Load Index (MPa) 2 4 6	RMC -Q' AVG.	
		GROUND SURFACE		48.28															
14 15 16 17	Rotary Drill HQ Core	Highly weathered to fresh, black, fine grained SHALE		13.86 45.02 17.12	1 2 3														Bentonite Seal Silica Sand 51 mm Diam. PVC #10 Slot Screen
18 19 20 21 22 23		End of Drillhole																	WL in Screen at Elev. 54.84 m on July 21, 2020





DEPTH SCALE

1 : 50

GOLDER

LOGGED: RA

CHECKED: BB

PROJECT: 20144766

RECORD OF BOREHOLE: 20-03

SHEET 1 OF 2

LOCATION: N 5030932.9 ;E 369899.2

BORING DATE: July 13, 2020

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

BORING METHOD: Power Auger 200 mm Diam. (Hollow Stem)

DEPTH SCALE METRES	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.30m	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
	GROUND SURFACE	62.47					
0	CONCRETE	0.00					Flush Mount Casing
	FILL - (SM) gravelly SILTY SAND; brown; non-cohesive, moist, compact to dense	0.13	1	SS	11		
1			2	SS	36		
	FILL - (SM) gravelly SILTY SAND; brown black, contains cinders, ash, and red brick pieces; non-cohesive, wet, loose	60.94 1.53	3	SS	5		
2			4	SS	4		
3			5	SS	6	CHEM	
4			6	SS	4		
5			7	SS	4		Bentonite Seal
	(CI) SILTY CLAY; grey brown; cohesive, w>PL, moist, firm to stiff	57.13 5.34	8	SS	12		
6			9	SS	7		
7	(ML) CLAYEY SILT to SILT; grey (GLACIAL TILL); cohesive, w~PL, wet, firm	55.61 6.86	10	SS	5		∇
8	(SM) gravelly SILTY SAND, some clay; dark grey (GLACIAL TILL); cohesive, wet, loose to compact	54.85 7.62	11	SS	17		
9			12	SS	24		
			13	SS	9		Silica Sand
10		52.56 9.91	14	SS	15		

CONTINUED NEXT PAGE

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m: 20 40 60 80

SHEAR STRENGTH Cu, kPa: nat V. + Q - ● rem V. ⊕ U - ○ 20 40 60 80

HYDRAULIC CONDUCTIVITY, k, cm/s: 10^{-6} 10^{-5} 10^{-4} 10^{-3}

WATER CONTENT PERCENT: Wp ⊢—○W—⊣ Wl 20 40 60 80

DEPTH SCALE

1 : 50

GOLDER

LOGGED: RA

CHECKED: BB

PROJECT: 20144766

RECORD OF BOREHOLE: 20-03

SHEET 2 OF 2

LOCATION: N 5030932.9 ;E 369899.2

BORING DATE: July 13, 2020

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m: 20 40 60 80

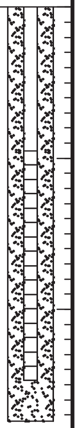
SHEAR STRENGTH Cu, kPa: nat V. + Q - ● rem V. ⊕ U - ○ 20 40 60 80

HYDRAULIC CONDUCTIVITY, k, cm/s: 10^{-6} 10^{-5} 10^{-4} 10^{-3}

WATER CONTENT PERCENT: Wp ⊢—○W—⊣ Wl 20 40 60 80

ADDITIONAL LAB. TESTING

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.30m	PIEZOMETER OR STANDPIPE INSTALLATION
		--- *CONTINUED FROM PREVIOUS PAGE* ---					
10	Power Auger 200 mm Diam. (Hollow Stem)	(ML) sandy SILT to SILT; grey (GLACIAL TILL); non-cohesive, compact to very dense		14	SS	15	Silica Sand
11				15	SS	64	
		(SM) gravelly SILTY SAND, some clay; grey (GLACIAL TILL); non-cohesive, wet, very dense	51.03 11.44	16	SS	55	51 mm Diam. PVC #10 Slot Screen
12				17	SS	>50	
13				18	SS	>50	WL in Screen at Elev. 55.19 m on July 21, 2020
				19	SS	>50	
14	Rotary Drill HW Casing	Mechanically broken SHALE	48.35 14.12				
		End of Borehole	47.67 14.80				
15							
16							
17							
18							
19							
20							



DEPTH SCALE

1 : 50

GOLDER

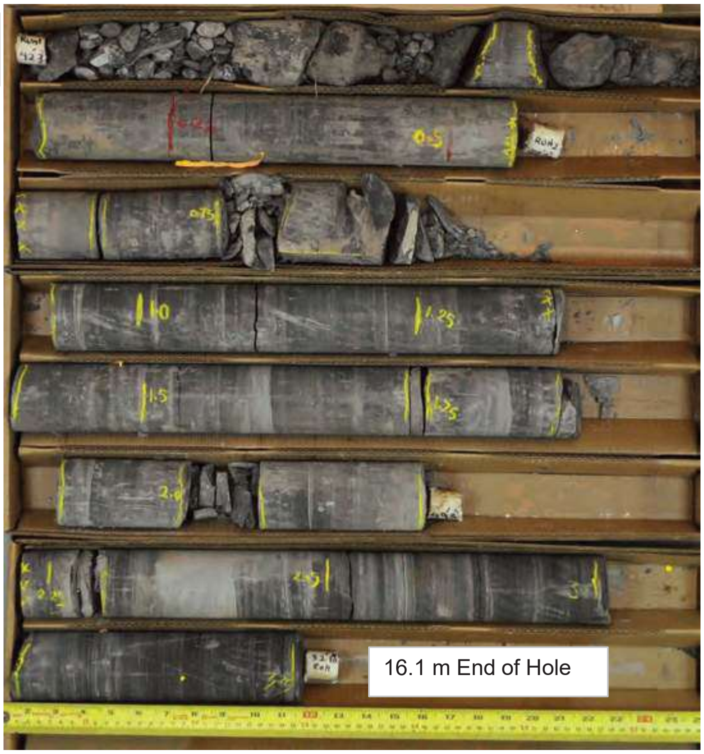
LOGGED: RA

CHECKED: BB

Path: https://golderassociates.sharepoint.com/sites/127981/Project Files/5 Technical Work/Bedrock Core Photos/20144766 Core Photos/Core Plates | File: Core Plates

BH 20-01 (Dry)
Cored Length of 12.90 to 16.10 metres
Core Box 1 to 3 of 3

12.9 m Top of Bedrock



16.1 m End of Hole

CLIENT
The Univeristy of Ottawa

PROJECT
200 Lees Ave Redevelopment

CONSULTANT
GOLDER

DD/MM/YYYY	2020-08-07
PREPARED	BB
DESIGN	
REVIEW	CH
APPROVED	CH

TITLE
BOREHOLE 20-01 (DRY)
CORE PHOTOGRAPHS

PROJECT No.	PHASE	Rev.	FIGURE
20144766	2000	A	A1

IF THIS MEASUREMENT DOES NOT MATCH WHAT IS SHOWN, THE SHEET SIZE HAS BEEN MODIFIED FROM: ANSI A

Path: https://golderassociates.sharepoint.com/sites/127981/Project Files/5 Technical Work/Bedrock Core Photos/20144766 Core Photos/Core Plates | File: Core Plates

BH 20-01 (Wet)
Cored Length of 12.90 to 16.10 metres
Core Box 1 to 3 of 3

12.9 m Top of Bedrock

16.1 m - End of Hole

CLIENT
The University of Ottawa

PROJECT
200 Lees Ave Redevelopment

CONSULTANT
GOLDER

DD/MM/YYYY	2020-08-07
PREPARED	BB
DESIGN	
REVIEW	CH
APPROVED	CH

TITLE
BOREHOLE 20-01 (WET)
CORE PHOTOGRAPHS

PROJECT No.	PHASE	Rev.	FIGURE
20144766	2000	A	A2

IF THIS MEASUREMENT DOES NOT MATCH WHAT IS SHOWN, THE SHEET SIZE HAS BEEN MODIFIED FROM: ANSI A

1 in

0

BH 20-02 (Dry)
Cored Length of 13.90 to 17.10 metres
Core Box 1 to 3 of 3

13.9 m Top of Bedrock

17.1 m - End of Hole

CLIENT The University of Ottawa	PROJECT 200 Lees Ave Redevelopment

CONSULTANT

GOLDER

DD/MM/YYYY	2020-08-07
PREPARED	BB
DESIGN	
REVIEW	CH
APPROVED	CH

TITLE
BOREHOLE 20-02 (DRY)
CORE PHOTOGRAPHS

PROJECT No.	PHASE	Rev.	FIGURE
20144766	2000	A	A3

BH 20-02 (Wet)
Cored Length of 13.90 to 17.10 metres
Core Box 1 to 3 of 3

13.9 m Top of Bedrock

17.1 m - End of Hole

CLIENT The Univeristy of Ottawa	PROJECT 200 Lees Ave Redevelopment

CONSULTANT

GOLDER

DD/MM/YYYY	2020-08-07
PREPARED	BB
DESIGN	
REVIEW	CH
APPROVED	CH

TITLE
BOREHOLE 20-02 (WET)
CORE PHOTOGRAPHS

PROJECT No.	PHASE	Rev.	FIGURE
20144766	2000	A	A4

BH 20-03 (Dry)
Cored Length of 14.10 to 14.80 metres
Core Box 1 to 1 of 1

14.1 m Top of Bedrock

14.8 m - End of Hole

CLIENT
The Univeristy of Ottawa

PROJECT
200 Lees Ave Redevelopment

CONSULTANT
GOLDER

DD/MM/YYYY	2020-08-07
PREPARED	BB
DESIGN	
REVIEW	CH
APPROVED	CH

TITLE
BOREHOLE 20-03 (DRY)
CORE PHOTOGRAPHS

PROJECT No.	PHASE	Rev.	FIGURE
20144766	2000	A	A5

BH 20-03 (Wet)
Cored Length of 14.10 to 14.80 metres
Core Box 1 to 1 of 1

14.1 m Top of Bedrock

14.8 m - End of Hole

CLIENT	PROJECT
The Univeristy of Ottawa	200 Lees Ave Redevelopment

CONSULTANT

GOLDER

DD/MM/YYYY	2020-08-07
PREPARED	BB
DESIGN	
REVIEW	CH
APPROVED	CH

TITLE
BOREHOLE 20-03 (WET)
CORE PHOTOGRAPHS

PROJECT No.	PHASE	Rev.	FIGURE
20144766	2000	A	A6

PROJECT: 11-1121-0057

RECORD OF BOREHOLE: 11-1

SHEET 1 OF 3

LOCATION: See Site Plan

BORING DATE: May 12, 2011

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DYNAMIC PENETRATION RESISTANCE, BLOWS/0 3m: 20 40 60 80

SHEAR STRENGTH Cu, kPa: nat V ┼ Q - ● rem V ⊕ U - ○ : 20 40 60 80

HYDRAULIC CONDUCTIVITY, k, cm/s: 10^{-6} 10^{-5} 10^{-4} 10^{-3}

WATER CONTENT PERCENT: Wp ├── ○W ──┤ Wl : 20 40 60 80

ADDITIONAL LAB TESTING

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0 3m	PIEZOMETER OR STANDPIPE INSTALLATION
		GROUND SURFACE	62.23				
0	Power Auger 200mm Diam (Hollow Stem)	ASPHALTIC CONCRETE	0.09				
		Grey crushed stone (FILL)	0.26				
		Brown sand, some gravel, trace silt (FILL)	61.41 0.82				
1		Very loose brown and black cinders and ash in sand and sandy silt matrix, trace to some glass, brick and mortar (FILL)		1	50 DO	5	
2				2	50 DO	2	
				3	50 DO	2	
3				4	50 DO	1	
4				5	50 DO	1	
5				6	50 DO	2	
		Dark grey organic clayey silt (ALLUVIUM)	57.05 5.18				
		Very stiff grey brown SILTY CLAY	56.59 5.64	7	50 DO	1	
6				8	50 DO	4	
7		Very loose to compact grey SANDY SILT, with some gravel (GLACIAL TILL)	55.37 6.86	9	50 DO	1	63.5mm Diam VSP Pipe
8				10	50 DO	>100	
9				11	50 DO	25	
				12	50 DO	19	
10				13	50 DO	30	
11	Rotary Drill HQ Core	Fresh black SHALE BEDROCK (Carlsbad Formation)	51.29 10.94	C1	HQ RC	DD	
12							
13				C2	HQ RC	DD	
14				C3	HQ RC	DD	
15			47.29				

CONTINUED NEXT PAGE

DEPTH SCALE

1 : 75

Golder Associates

LOGGED: P.A.H.

CHECKED: N.R.L.

PROJECT: 11-1121-0057

RECORD OF BOREHOLE: 11-1

SHEET 2 OF 3

LOCATION: See Site Plan

BORING DATE: May 12, 2011

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES: NUMBER	TYPE	BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m 20 40 60 80; SHEAR STRENGTH Cu, kPa nal V. + Q - ● rem V. ⊕ U - ○ 20 40 60 80	HYDRAULIC CONDUCTIVITY, k, cm/s 10^{-6} 10^{-5} 10^{-4} 10^{-3}; WATER CONTENT PERCENT Wp ├──○W──┤ Wl 20 40 60 80	ADDITIONAL LAB TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		— CONTINUED FROM PREVIOUS PAGE ---									
15	Rotary Drill HQ Core	Fresh black SHALE BEDROCK some grey limestone interbedding (Carlsbad Formation)									
16					C4	HQ RC	DD				
17					C5	HQ RC	DD				63.5mm Diam VSP Pipe
18					C6	HQ RC	DD				
19		End of Borehole		42.56 19.67							
20											
21											
22											
23											
24											
25											
26											
27											
28											
29											
30											

DEPTH SCALE

1 : 75

Golder Associates

LOGGED: P.A.H.

CHECKED: N.R.L.

PROJECT: 11-1121-0057

RECORD OF DRILLHOLE: 11-1

SHEET 3 OF 3

LOCATION: See Site Plan

DRILLING DATE: May 12, 2011

DATUM: Geodetic

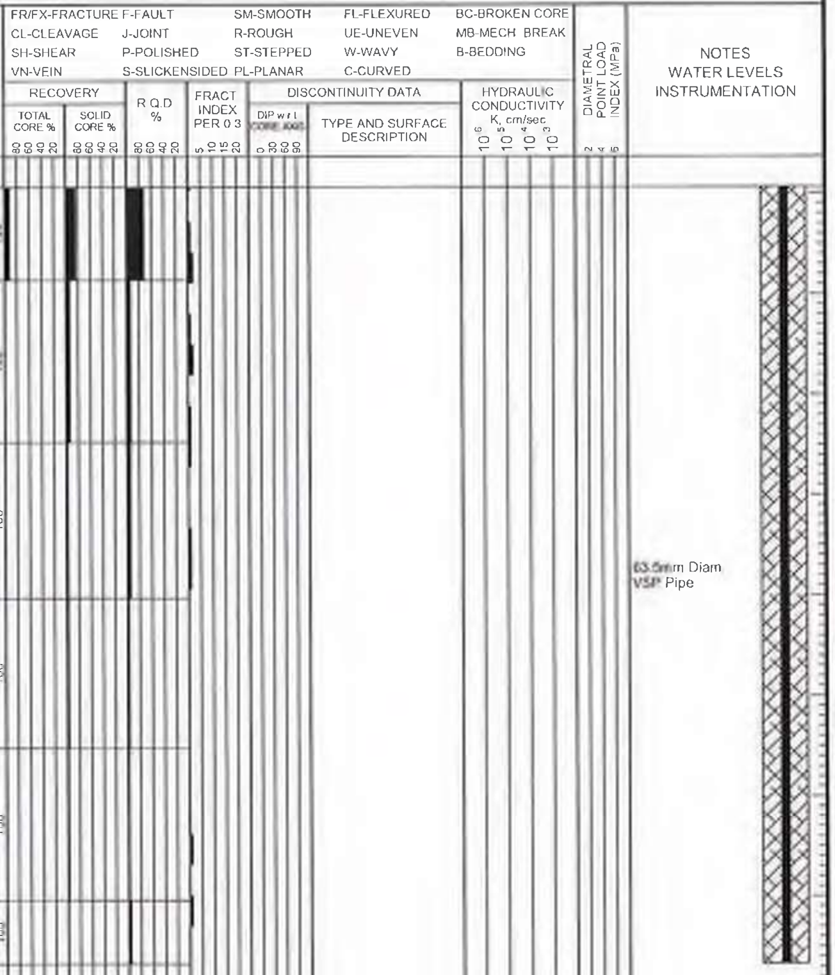
DRILL RIG: CME 75

INCLINATION: -90° AZIMUTH: ---

DRILLING CONTRACTOR: Marathon Drilling

FR/FX-FRACTURE F-FAULT SM-SMOOTH FL-FLEXURED BC-BROKEN CORE
CL-CLEAVAGE J-JOINT R-ROUGH UE-UNEVEN MB-MECH BREAK
SH-SHEAR P-POLISHED ST-STEPPED W-WAVY B-BEDDING
VN-VEIN S-SLICKENSIDED PL-PLANAR C-CURVED

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No	PENETRATION RATE (m/min)	COLOUR % RETURN FLUSH	NOTES WATER LEVELS INSTRUMENTATION
		BEDROCK SURFACE		51.29				
11	Rotary Drill HQ Core	Fresh black SHALE BEDROCK, some grey limestone interbedding (Carlsbad Formation)		10.94	1		100	63.5mm Diam VSP Pipe
12					2		100	
14					3		100	
16					4		100	
17					5		100	
18				43.56	6		100	
		End of Borehole		18.67				



DEPTH SCALE
1 : 75

Golder Associates

LOGGED: P.A.H.
CHECKED: J.M.R.

Golder Associates Ltd

32 Steacie Drive
Kanata, Ontario K2K 2A9
Tel: (613) 592-9600
Fax: (613) 592-9601

TEST PIT RECORD

TEST PIT # 11-7

DATE: May 13, 2011

PROJECT: U of O - 200 Lees Avenue Block A Redevelopment
PROJECT No.: 11-1121-0057

EQUIPMENT: Test pits excavated with rubber-tired backhoe.

Depth (m)	Elevation (m)	Description		Remarks
0.00	62.20	Grade Beam Foundation	TOPSOIL	
0.55	61.65		Brown cinders and ashes in a matrix of silty sand, some gravel and cobbles, with pieces of glass, steel, wire and other miscellaneous debris (FILL)	
2.00	60.20	Franki Pile		Franki Pile width about 300 to 400 mm
2.55	59.65	Bottom of test pit -- No groundwater infiltration.		

Logged by : NRL
Compiled by : NRL
Checked by : TJN

PROJECT: 001-2721

RECORD OF BOREHOLE: 00-2

SHEET 1 OF 1

LOCATION: See Site Plan

BORING DATE: March 23, 2000

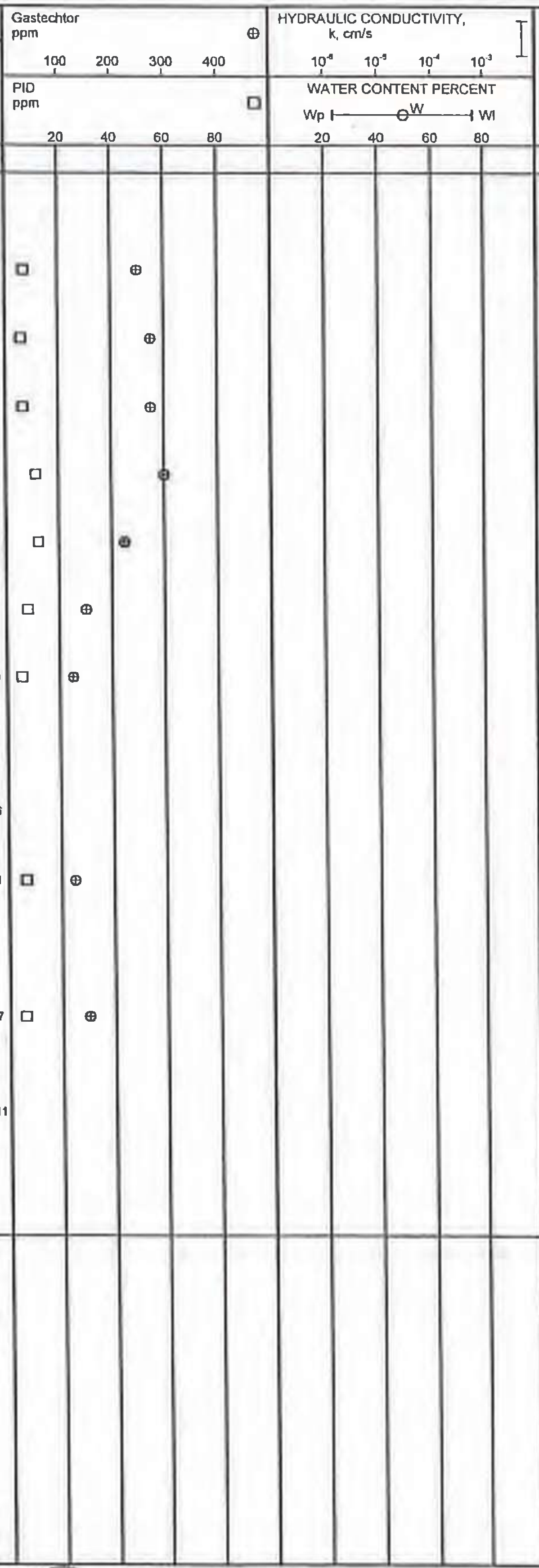
DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

BORING METHOD: Power Auger, 200mm Diam (Hollow Stem)

SOIL PROFILE DESCRIPTION	ELEV. / DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.3m
Ground Surface	62.08			
TOPSOIL	0.15			
Very loose grey brown and red brown to black cinders and ash, sand, trace brick, glass, ceramics, organics and coal fragments (Cinder & Ash FILL)		1	50 DO	3
		2	76 DO	3
		3	76 DO	2
		4	76 DO	2
Compact to loose grey brown to dark grey sandy silt with gravel (GLACIAL TILL)	58.37 3.81	5	76 DO	17
		6	50 DO	16
		7	50 DO	10
		8	50 DO	7
Dense dark grey sandy silt with gravel, occasional medium to coarse sand seam/layer (GLACIAL TILL)	55.37 6.71	9	50 DO	36
		10	50 DO	41
		11	50 DO	37
Very dense dark grey SILTY SAND and GRAVEL	51.72 10.36	12	50 DO	111
END OF BOREHOLE REFUSAL TO AUGER	50.19 11.89			



PIEZOMETER OR STANDPIPE INSTALLATION

Cement Seal

Pea Stone Backfill

Native Backfill

Bentonite Seal

Granular Filter

50mm PVC #10 Slot Screen

Bentonite Seal

Sand Backfill

Native and Caved Material

Screen Dry Mar. 31, 2000

DEPTH SCALE 1 : 75

Golder Associates

LOGGED: P.A.H.

CHECKED: PLE

PROJECT: 001-2721

RECORD OF BOREHOLE: 00-3

SHEET 1 OF 1

LOCATION: See Site Plan

BORING DATE: March 21, 2000

DATUM: Geodetic

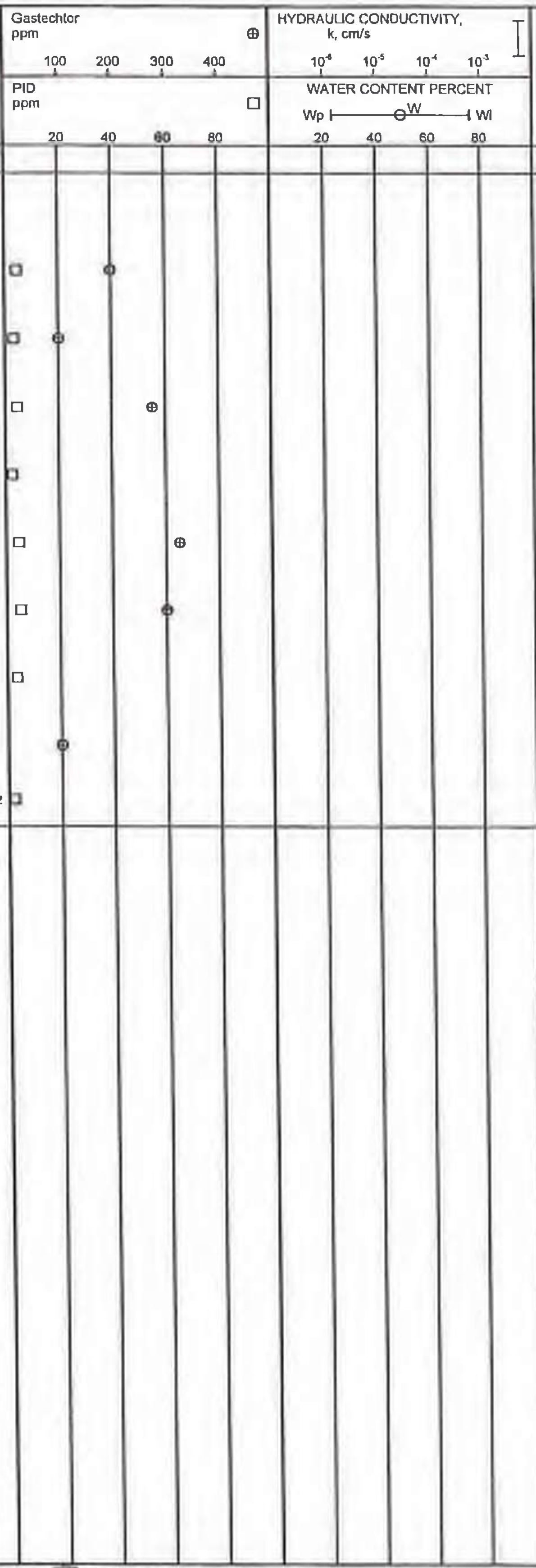
SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

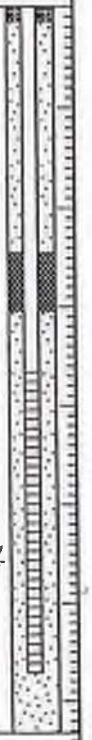
BORING METHOD: Power Auger — 200mm Diam (Hollow Stem)

SOIL PROFILE — DESCRIPTION	ELEV. / DEPTH (m)
Ground Surface	62.10
SAND, some organics	0.15
Very loose to loose grey brown, grey, **red-brown** to black **cinders** and ash, some sand, trace glass, organics, coal fragments (Cinder & Ash FILL)	56.92 5.18
Loose to compact, layered, grey **fine** sand and silts, trace **organics** (ALLUVIUM)	55.55 6.55
Compact grey brown sandy silt, some gravel (GLACIAL TILL)	54.78 7.32
END OF BOREHOLE	

SAMPLES NUMBER	TYPE	BLOWS/0.3m
1	50 DO	3
2	50 DO	2
3	76 DO	5
4	50 DO	7
5	76 DO	5
6	76 DO	3
7	50 DO	6
8	50 DO	11
9	50 DO	12



PIEZOMETER OR STANDPIPE INSTALLATION



- **Cement Seal**
- **Pea Stone Backfill**
- Bentonite Seal
- Granular Filter
- **50mm** PVC **#10** Slot Screen
- ∇

W.L in Screen at Elev.56.52m Mar. 31, 2000

DEPTH SCALE 1 : 75

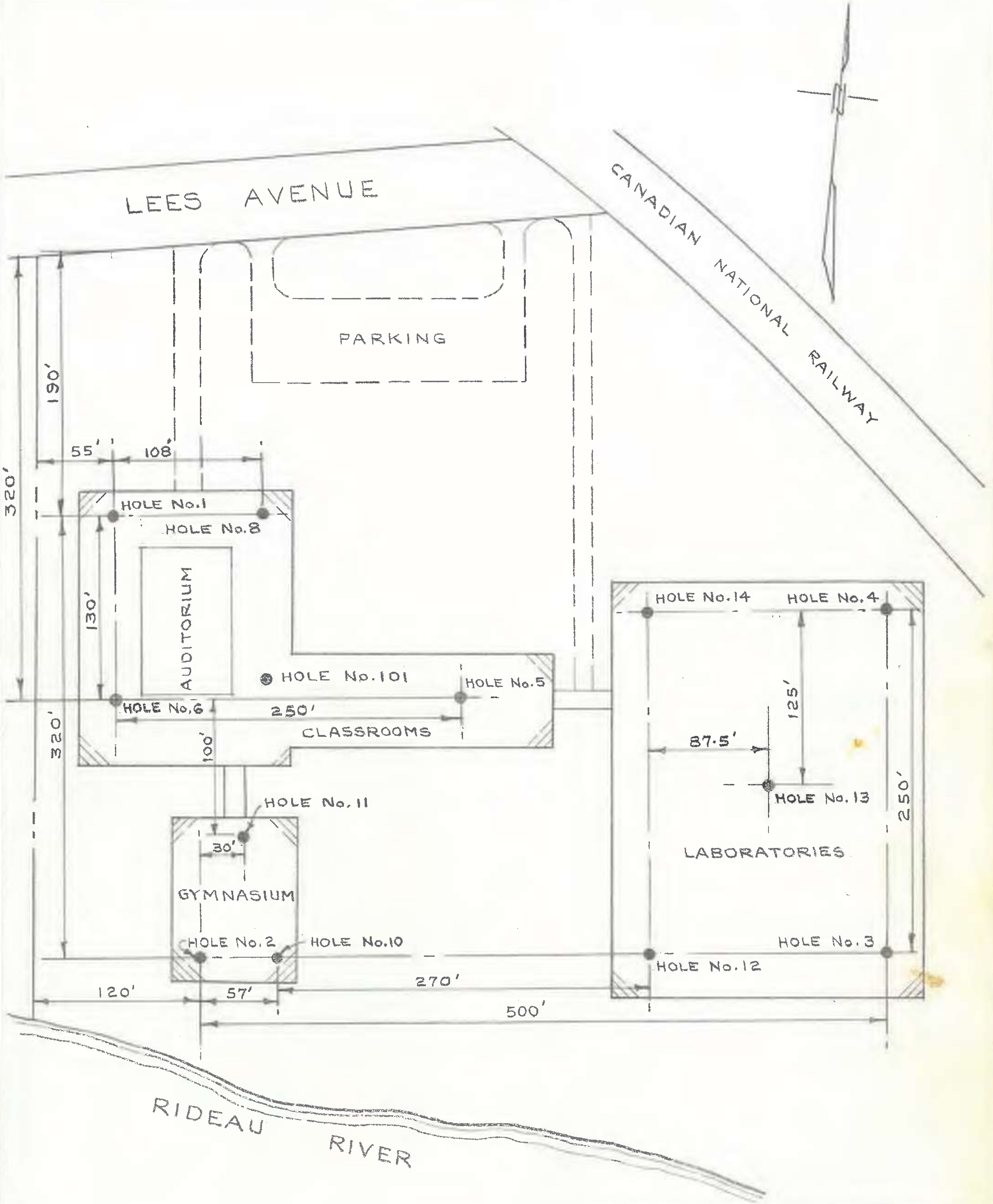
Golder Associates

LOGGED: P.A.H.

CHECKED: PE

RECORD OF TEST PITS

Test Pit *Number*	**Depth** **(metres)**	*Description*
TP00-1 Ground Surface Elev. 61.68 m	0.00 – 0.09	Asphalt
	0.09 – 0.90	Grey stone, sand and gravel (FILL)
	0.90 – 1.50	Red brown and grey brown cinders and ash, some sand, trace boulders, glass and wood (CINDER AND ASH FILL)
	1.50 – 1.80	Grey silty clay (FILL)
	1.70 – 1.80	North side of test pit only – Light brown medium sand (FILL)
	1.80 – 2.10	Dark brown to black fine to medium sand, some gravel (FILL)
	2.10 – 2.40	Dark brown fine to medium sand, some gravel, trace ash (FILL)
	2.40 – 4.00	Brown fine to medium sand, some gravel, trace boulders (GLACIAL TILL)
	4.00	End of Test Pit
		Note: Test pit dry upon completion
TP00-2 Ground Surface Elev. 62.16 m	0.00 – 0.13	Organic TOPSOIL
	0.13 – 4.00	Red brown and grey brown cinders and ash, some sand, glass and metal, trace coal, wood, gravel, brick and concrete (metal content increases with depth) (CINDER AND ASH FILL)
	4.00	End of Test Pit
		Note: Test pit dry upon completion
TP00-3 Ground Surface Elev. 62.21 m	0.00– 0.10	Organic TOPSOIL
	0.10 – 0.50	Dark brown to black fine sand (FILL)
	0.50 – 2.10	Brown sand, some cinders and ash, glass, traces of coal, metal and organics (CINDER AND ASH FILL)
	2.10 – 2.50	Black sand, trace coal fragments and gravel, strong odour (FILL)
	2.50	End of Test Pit
		Note: Test pit dry upon completion
TP00-4 Ground Surface Elev. 62.05 m	0.00 – 0.13	Organic TOPSOIL
	0.13 – 2.50	Red brown and grey brown cinders and ash, some sand, glass, trace coal, wood, gravel and brick (CINDER AND ASH FILL)
	2.50 – 3.00	Black sand, trace coal fragments and gravel (FILL)
	3.00 – 4.00	Grey brown cinders and ash, some sand, trace coal (CINDER AND ASH FILL)
	4.00	End of Test Pit
		Note: Test pit dry upon completion



McROSTIE & ASSOCIATES LTD.
CONSULTING ENGINEERS

BOREHOLE LOCATIONS
LEES AVENUE

SCALE 1" = 100'

PLATE 1

McROSTIE & ASSOCIATES LTD.
CONSULTING ENGINEERS
OTTAWA CANADA

SOIL PROFILE AND SUMMARY OF FIELD AND LABORATORY TESTS

LEES AVENUE

ELEVATION OF GROUND SURFACE (ZERO DEPTH) 202.5' DATE MARCH 22 1962

REMARKS B.M. (EL. 195.47') GEODETIC CITY B.M. NORTH SIDE OF LEES AVENUE ON OTTAWA GAS OFFICE.

HOLE No. 1

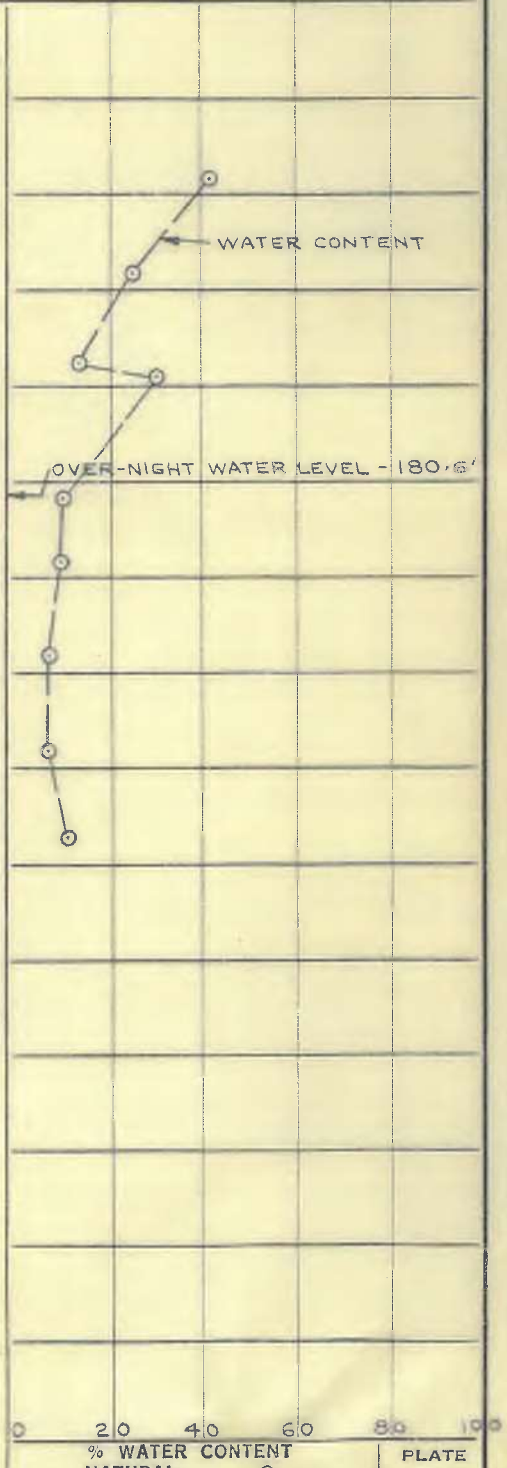
UNCONFINED COMPRESSIVE STRENGTH Kips/FT.^{2}	SMALL SCALE PENETROMETER KIPS/FT.^{2}	STANDARD PENETRATION BLOWS/FT.	SAMPLE NUMBER	DESCRIPTION OF SOIL	DEPTH IN FEET	ELEVATION
				GROUND SURFACE	0'	202.5'
		7	1-1	FILL (SAND, ASHES, CINDERS, GLASS, WOOD & A NAIL)		
		12	1-2			
				LOOSE SILTY FINE SAND WITH A LITTLE CLAY & A FEW PEBBLES	15'	187.5'
		3 for 6"	1-3A			
		8	1-3B	LOOSE SILT WITH A FEW 1/8" CLAY LAYERS	15.5'	
					20'	182.5'
		26	1-4	MEDIUM DENSE SANDY TILL		
		20	1-5			
		28	1-6			
					35'	167.5'
		123	1-7	DENSE TILL		
					38'	
		23 for 6"	1-8	DENSE SILT WITH A FEW PEBBLES & SHALE PARTICLES		
					43'	
				SHALEY TILL	44.6	157.9'
				WEATHERED OR FRACTURED ROCK CORE RECOVERY 46%	49.6'	152.9
				ROCK CORE RECOVERY 98%	55'	147.5'
				BOTTOM OF HOLE		

~~PROBING OR VANE TEST~~

..................LB. HAMMER ....INCH DROP

NO CASING ....INCH DIA. ROD

~~BLOWS PER FOOT OR~~ ~~SHEAR STRENGTH IN KIPS PER FT.^{2}~~



% WATER CONTENT
NATURAL ⊙
LIQUID LIMIT □
PLASTIC LIMIT △

PLATE 2

R - REMOULDED

FORM S-10-4

McROSTIE & ASSOCIATES LTD.
CONSULTING ENGINEERS
OTTAWA CANADA

SOIL PROFILE AND SUMMARY OF FIELD AND LABORATORY TESTS

LEES AVENUE

ELEVATION OF GROUND SURFACE (ZERO DEPTH) 203.7' DATE MARCH 26 1962

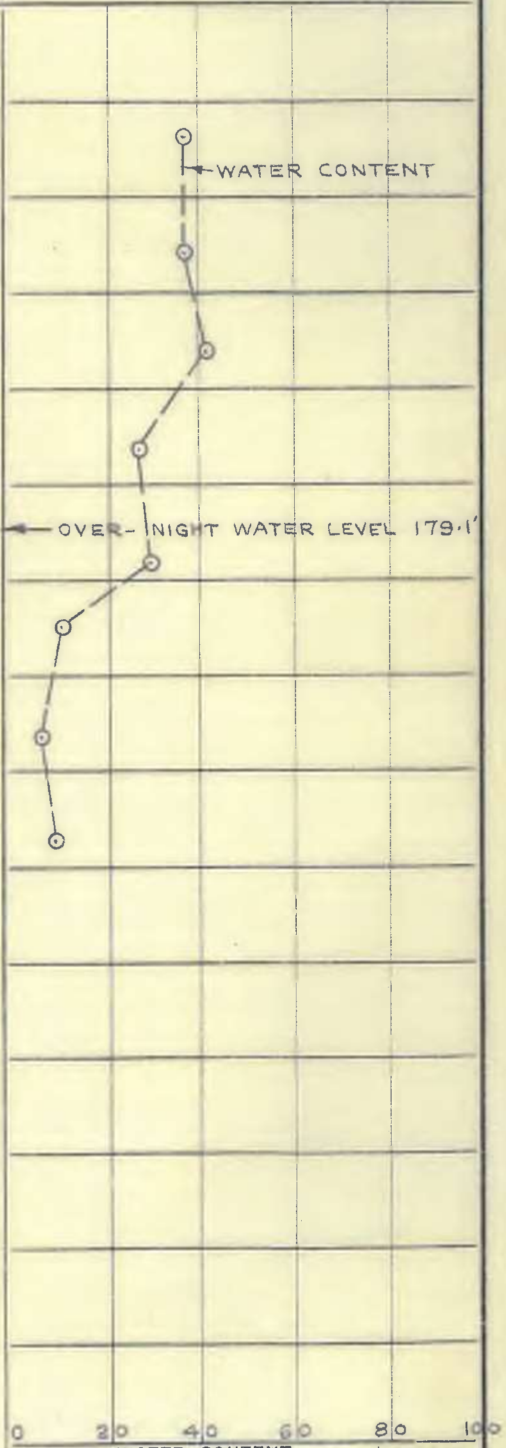
REMARKS SEE PLATE No. 2

HOLE No. 2

UNCONFINED COMPRESSIVE STRENGTH KIPS/FT.^{2}	SMALL SCALE PENETROMETER KIPS/FT.^{2}	STANDARD PENETRATION BLOWS/FT.	SAMPLE NUMBER	DESCRIPTION OF SOIL	DEPTH IN FEET	ELEVATION
				GROUND SURFACE	0'	203.7'
		3	2-1	FILL (ASHES WITH SILT WOOD, CINDERS & TILE)		
		5	2-2			
		5	2-3			
					20'	183.7'
		6 6 for 6"	2-4	LOOSE TO MEDIUM DENSE SILT WITH A FEW 1/8" CLAY LAYERS		
		15 2 for 6"	2-5			
					28'	175.7'
		58	2-6	BOULDERS IN DENSE SANDY TILL		
		138	2-7			
					40'	
		48	2-8	DENSE SILTY FINE SAND WITH SOME PEBBLES & SHALE PARTICLES		
					43.1'	160.6'
				ROCK CORE RECOVERY 94%		
					49'	
				ROCK CORE RECOVERY 97%		
				BOTTOM OF HOLE	54.5'	149.2'

~~PROBING OR VANE TEST~~

........LB. HAMMER
........INCH DROP
NO CASING
........INCH DIA. ROD
~~BLOWS PER FOOT OR SHEAR STRENGTH IN KIPS PER FT.^{2}~~



% WATER CONTENT
NATURAL ⊙
LIQUID LIMIT ☐
PLASTIC LIMIT △

R - REMOULDED

PLATE 3

FORM S-10-4

McROSTIE & ASSOCIATES LTD.
CONSULTING ENGINEERS
OTTAWA CANADA

SOIL PROFILE AND SUMMARY OF FIELD AND LABORATORY TESTS

LEES AVENUE

ELEVATION OF GROUND SURFACE (ZERO DEPTH) 205.9' DATE MARCH 28, 1962

REMARKS SEE PLATE No. 2

HOLE No. 3

UNCONFINED COMPRESSIVE STRENGTH Kips/FT.^{2}	SMALL SCALE PENETROMETER KIPS/FT.^{2}	STANDARD PENETRATION BLOWS/FT.	SAMPLE NUMBER	DESCRIPTION OF SOIL	DEPTH IN FEET	ELEVATION
				GROUND SURFACE	0'	205.9'
		13	3-1			
		19	3-2	FILL (SAND, ASHES, CONCRETE GLASS, TIN, WOOD & BRICK)		
		46	3-3			
		39	3-4			
		58	3-5		25.5'	180.4'
		77	3-6	DENSE TILL		
		90 fo. 6"	3-7		35'	170.9'
				WEATHERED SHALE	38.3'	167.6'
				ROCK CORE RECOVERY 87%	42.9'	
				ROCK CORE RECOVERY 82%	48.3'	157.6'
				BOTTOM OF HOLE		

R - REMOULDED

~~PROBING OR VANE TEST~~

..................LB. HAMMER ....INCH DROP	NO CASING ....INCH DIA. ROD

~~BLOWS PER FOOT OR~~ ~~SHEAR STRENGTH IN KIPS PER FT.^{2}~~

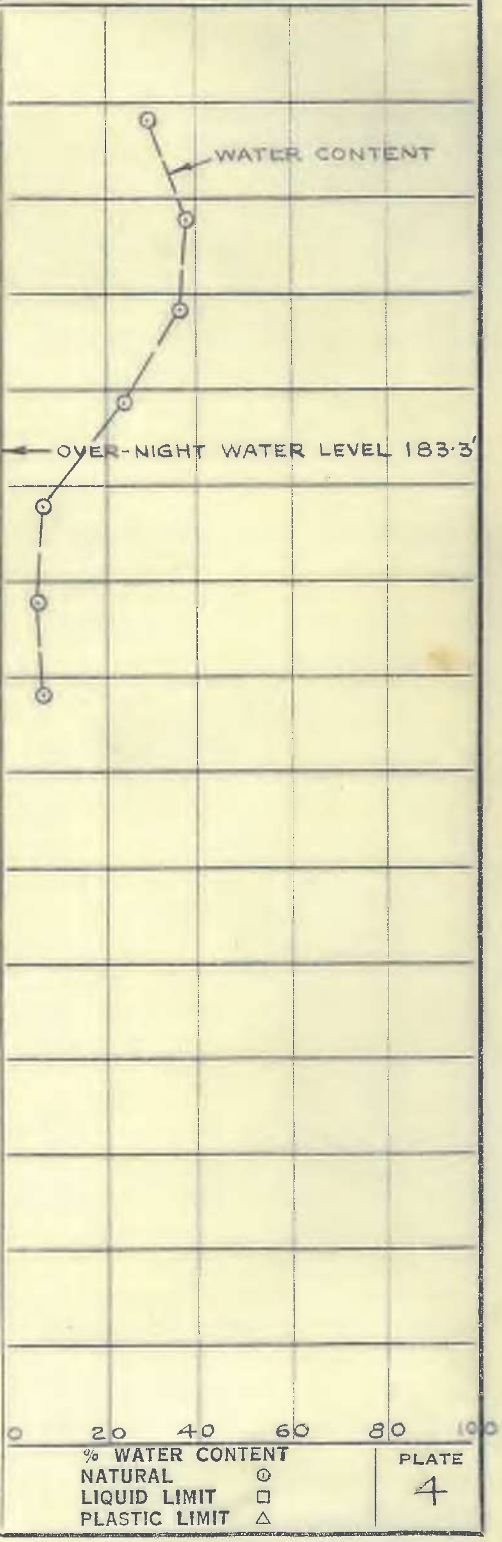


PLATE 4

LOWE-MARTIN CO.—A55512

FORM S-10-4

McROSTIE & ASSOCIATES LTD.
CONSULTING ENGINEERS
OTTAWA CANADA

SOIL PROFILE AND SUMMARY OF FIELD AND LABORATORY TESTS

LEES AVENUE

ELEVATION OF GROUND SURFACE (ZERO DEPTH) 204.0' DATE MAR. 30, 1962

REMARKS SEE PLATE No. 2

HOLE No. 4

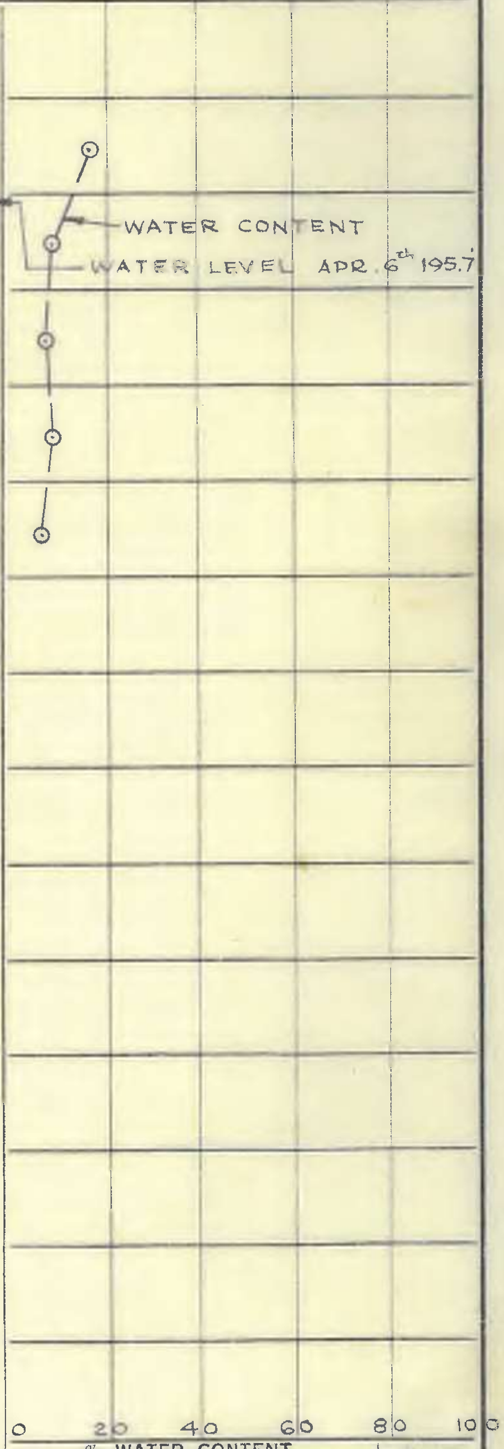
UNCONFINED COMPRESSIVE STRENGTH KIPS/FT.²	SMALL SCALE PENETROMETER KIPS/FT.²	STANDARD PENETRATION BLOWS/FT.	SAMPLE NUMBER	DESCRIPTION OF SOIL	DEPTH IN FEET	ELEVATION
				GROUND SURFACE	0'	204.0'
				FILL (ASHES WITH SOME TILL)		
		4	4-1		6.5'	197.5'
		52	4-2	DENSE TILL		
					15'	
		22	4-3	MEDIUM DENSE TILL		
					20'	184.0'
		34	4-4	DENSE TILL		
		105	4-5			
					28.4'	175.6'
		58 for 4"	4-6	BOULDERS IN TILL		
					32.5'	171.5'
				BOTTOM OF HOLE		

~~PROBING OR VANE TEST~~

..........LB. HAMMER ....INCH DROP

NO CASING ....INCH DIA. ROD

~~BLOWS PER FOOT OR~~ ~~SHEAR STRENGTH IN KIPS PER FT.²~~



% WATER CONTENT
NATURAL ⊙
LIQUID LIMIT □
PLASTIC LIMIT △

PLATE 5

R - REMOULDED

FORM S-10-4

McROSTIE & ASSOCIATES LTD.
CONSULTING ENGINEERS
OTTAWA CANADA

SOIL PROFILE AND SUMMARY OF FIELD AND LABORATORY TESTS

LEES AVENUE

ELEVATION OF GROUND SURFACE (ZERO DEPTH) 205·2′ DATE APRIL 27, 1962

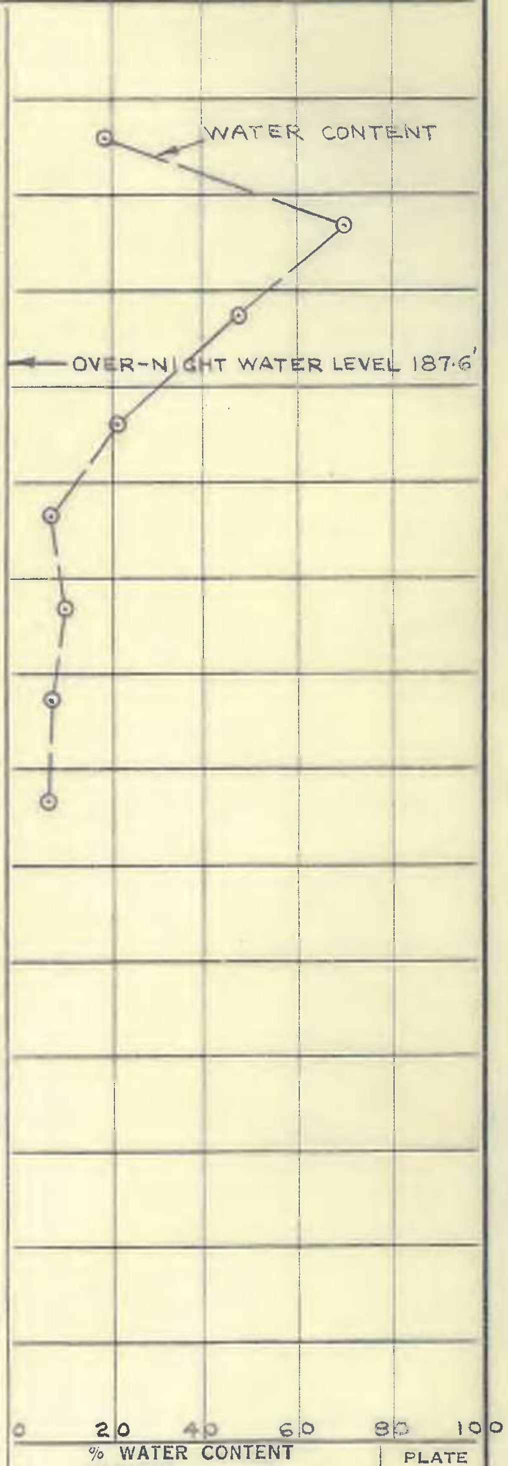
REMARKS SEE PLATE No. 2

HOLE No. 5

UNCONFINED COMPRESSIVE STRENGTH KIPS/FT.²	SMALL SCALE PENETROMETER KIPS/FT.²	STANDARD PENETRATION BLOWS/FT.	SAMPLE NUMBER	DESCRIPTION OF SOIL	DEPTH IN FEET	ELEVATION
				GROUND SURFACE	0′	205·2′
		2	5-1	FILL SAND, CINDERS GLASS, SILT & ASHES		
		6	5-2			
		8	5-3			
		34	5-4			
		21	5-5	MEDIUM DENSE TILL WITH A 1′ DENSE LAYER	25′	180·2′
		37″ 10 for 6″	5-6			
		70	5-7	DENSE SANDY TILL	35′	
		22	5-8	MEDIM DENSE SANDY TILL	40′ 41·2′	164·0′
				ROCK CORE RECOVERY 93%		
				BOTTOM OF HOLE	47·2′	158·0′

~~PROBING OR VANE TEST~~

.....................LB. HAMMER
.....................INCH DROP
NO CASING
...........INCH DIA. ROD
~~BLOWS PER FOOT OR~~ ~~SHEAR STRENGTH IN KIPS PER FT.²~~



% WATER CONTENT
NATURAL ⊙
LIQUID LIMIT □
PLASTIC LIMIT △

PLATE 6

R - REMOULDED

LOWE-MARTIN CO.—A55512

FORM 5-10-4

McROSTIE & ASSOCIATES LTD.
CONSULTING ENGINEERS
OTTAWA CANADA

SOIL PROFILE AND SUMMARY OF FIELD AND LABORATORY TESTS

LEES AVENUE

ELEVATION OF GROUND SURFACE (ZERO DEPTH) 204.9' DATE MAY 3, 1962

REMARKS SEE PLATE No. 2

HOLE No. 6

UNCONFINED COMPRESSIVE STRENGTH KIPS/FT.²	SMALL SCALE PENETROMETER KIPS/FT.²	STANDARD PENETRATION BLOWS/FT.	SAMPLE NUMBER	DESCRIPTION OF SOIL	DEPTH IN FEET	ELEVATION
				GROUND SURFACE	0'	204.9'
		5	6-1	FILL ASHES, SILT, CINDERS SAND, TOPSOIL CLAY, WOOD & ORGANIC MATERIAL		
		4	6-2			
		8	6-3			
		4	6-4		18'	186.9'
		6 6 for 6"	6-5	LOOSE SILT	21.5'	183.4
				DENSE SANDY TILL	26'	
		45 14 for 6"	6-6	MEDIUM DENSE SANDY TILL	30.5'	
		13 8	6-7	BOULDERS IN SANDY TILL	44.6'	160.3'
				ROCK CORE RECOVERY 83%	50.6	
				ROCK CORE RECOVERY 80%	55.3'	149.6'
				BOTTOM OF HOLE		

R - REMOULDED

PROBING OR VANE TEST

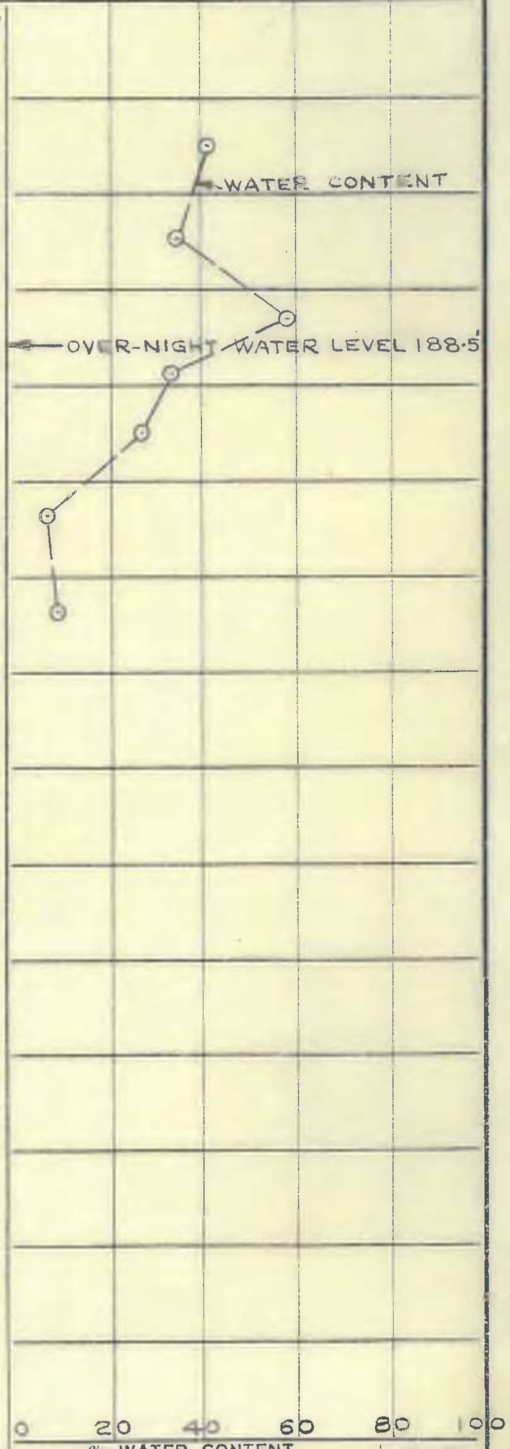
.......LB. HAMMER

.......INCH DROP

NO CASING

.......INCH DIA. ROD

BLOWS PER FOOT OR SHEAR STRENGTH IN KIPS PER FT.²



% WATER CONTENT

NATURAL ⊙

LIQUID LIMIT ☐

PLASTIC LIMIT △

PLATE 7

McROSTIE & ASSOCIATES LTD.
CONSULTING ENGINEERS
OTTAWA CANADA

SOIL PROFILE AND SUMMARY OF FIELD AND LABORATORY TESTS

LEES AVENUE

ELEVATION OF GROUND SURFACE (ZERO DEPTH) 204.6' DATE APRIL 18, 1960

REMARKS THIS BOREHOLE WAS MADE IN 1960 AS PART OF QUEENSWAY ROUTE STUDY - UNIFIED CLASSIFICATION SYSTEM USED.

HOLE No. 101

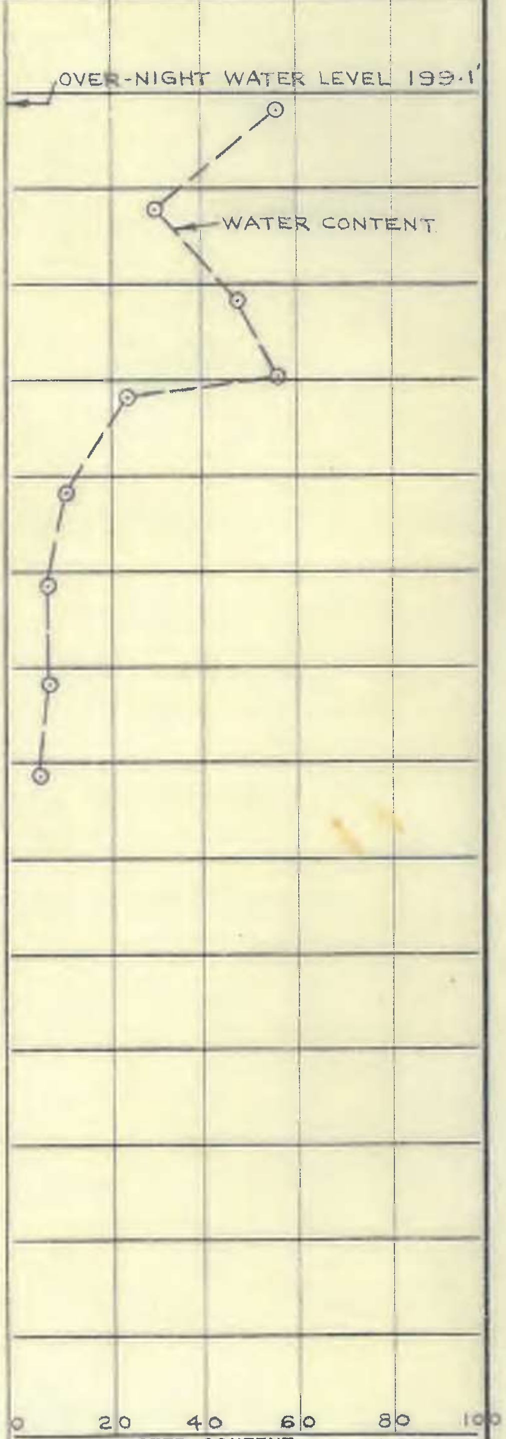
~~PROBING OR VANE TEST~~

……LB. HAMMER ……INCH DROP

NO CASING ……INCH DIA. ROD

~~BLOWS PER FOOT OR~~ ~~SHEAR STRENGTH IN KIPS PER FT.²~~

UNCONFINED COMPRESSIVE STRENGTH KIPS/FT.²	SMALL SCALE PENETROMETER KIPS/FT.²	STANDARD PENETRATION BLOWS/FT.	SAMPLE NUMBER	DESCRIPTION OF SOIL	DEPTH IN FEET	ELEVATION
				GROUND SURFACE	0'	204.6'
		5	1-1	FILL MOSTLY SAND & CINDERS		
		7	1-2			
		8	1-3			
		6 for 6" 24	1-4A 1-4B	SANDY SILT WITH A TRACE OF CLAY NON-PLASTIC, MEDIUM DENSE (ML)	20.5'	184.1'
				GRAVELLY SAND WITH SOME SILT & A TRACE OF CLAY (TILL) MEDIUM DENSE (SM)	24'	180.6'
		15	1-5		26.5'	
		56	1-6	GRAVELLY SAND WITH SOME SILT & A TRACE OF CLAY (TILL) DENSE (SM)	31.5'	173.1'
		14	1-7	SAND & GRAVEL MIXTURE WITH SOME SILT & A TRACE OF CLAY (TILL) MEDIUM DENSE (SM)		
		15	1-8		43.7'	160.9'
				SHALE CORE RECOVERY - 86%	48.6'	
				SHALE CORE RECOVERY - 97%	53.7'	150.9'
				BOTTOM OF HOLE		



% WATER CONTENT
NATURAL ⊙
LIQUID LIMIT □
PLASTIC LIMIT △

PLATE 8

R - REMOULDED

FORM S-10-4

McROSTIE & ASSOCIATES LTD.
CONSULTING ENGINEERS
OTTAWA CANADA

SOIL PROFILE AND SUMMARY OF FIELD AND LABORATORY TESTS

LEES AVENUE

ELEVATION OF GROUND SURFACE (ZERO DEPTH) 203·6'
DATE MAY 4, 1962
REMARKS SEE PLATE No. 2

HOLE No. 8

UNCONFINED COMPRESSIVE STRENGTH KIPS/FT.²	SMALL SCALE PENETROMETER KIPS/FT.²	STANDARD PENETRATION BLOWS/FT.	SAMPLE NUMBER	DESCRIPTION OF SOIL	DEPTH IN FEET	ELEVATION
				GROUND SURFACE	0'	203·6'
		4	8-1	FILL CINDERS, ASHES, CLAY, SAND, GRAVEL, COAL & ORGANIC MATERIAL		
		6	8-2			
		8	8-3		17'	186·6'
		87	8-4	BOULDERS IN DENSE SILTY TILL	25'	
		16	8-5	MEDIUM DENSE SILTY TILL	30'	
		12	8-6	MEDIUM DENSE SHALEY TILL	35'	
		52	8-7	BOULDERS IN DENSE SHALEY TILL		
		129	8-8		42·5'	161·1'
				WEATHERED OR FRACTURED ROCK CORE RECOVERY 62%	48·2'	155·4
				ROCK CORE RECOVERY 100%	53·3'	150·3'
				BOTTOM OF HOLE		

~~PROBING OR VANE TEST~~

......LB. HAMMER
......INCH DROP
NO CASING
......INCH DIA. ROD
~~BLOWS PER FOOT OR~~ ~~SHEAR STRENGTH IN KIPS PER FT.²~~

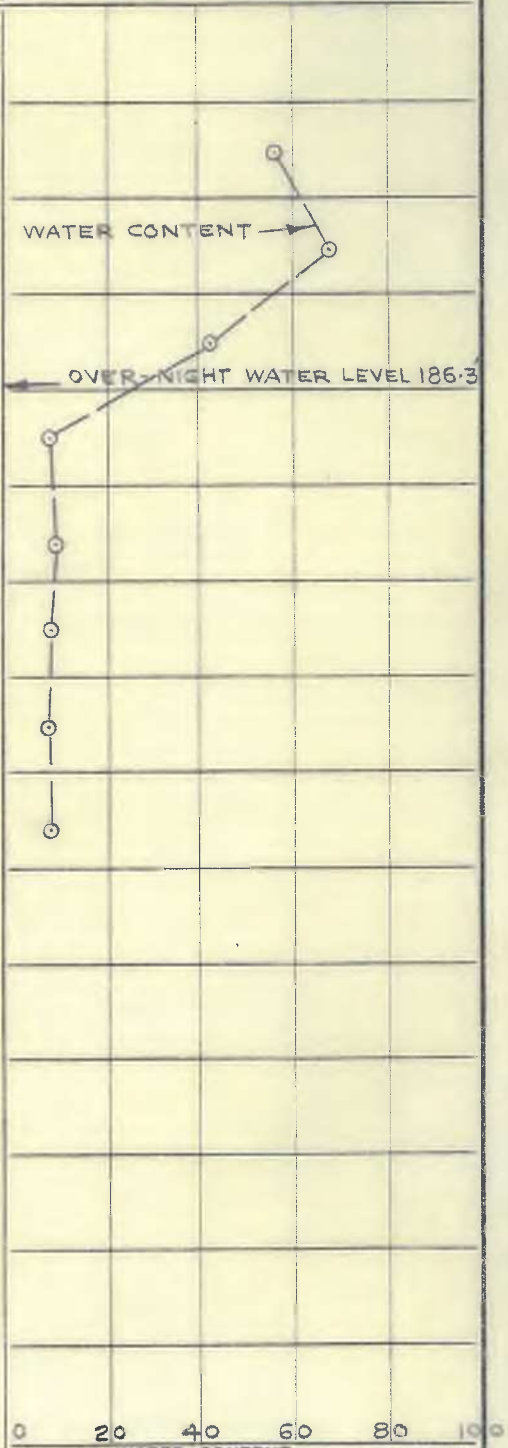


PLATE 9

R - REMOULDED

LOWE-MARTIN CO.—A55512

FORM S-10-4

McROSTIE & ASSOCIATES LTD.
CONSULTING ENGINEERS
OTTAWA CANADA

SOIL PROFILE AND SUMMARY OF FIELD AND LABORATORY TESTS

LEES AVENUE

ELEVATION OF GROUND SURFACE (ZERO DEPTH) 204.2' DATE MAY 8, 1962

REMARKS SEE PLATE No. 2

HOLE No. 10

UNCONFINED COMPRESSIVE STRENGTH KIPS/FT.²	SMALL SCALE PENETROMETER KIPS/FT.²	STANDARD PENETRATION BLOWS/FT.	SAMPLE NUMBER	DESCRIPTION OF SOIL	DEPTH IN FEET	ELEVATION
				GROUND SURFACE	0'	204.2'
		4	10-1	FILL CINDERS, SAND, BRICK TOPSOIL, COAL, ASHES & GLASS		
		4	10-2			
		4	10-3			
		12	10-4A		20.8'	183.4'
		5 for 6"	10-4B	STIFF GRAY CLAY WITH SOME SAND	21.5'	
		12	10-5	MEDIUM DENSE SILT		
					28.5'	175.7'
		25	10-6	MEDIUM DENSE SILTY VERY FINE SAND WITH A TRACE OF GRAVEL & A FEW SHALE PARTICLES		
					35'	169.2'
		65	10-7	DENSE SHALEY TILL		
					40'	
		33 for 6"	10-8	DENSE SILT & VERY FINE SAND WITH SOME SHALE PARTICLES	42.2'	162.0'
				ROCK CORE RECOVERY - 96%		
					48'	156.2'
				BOTTOM OF HOLE		

R - REMOULDED

~~PROBING OR VANE TEST~~

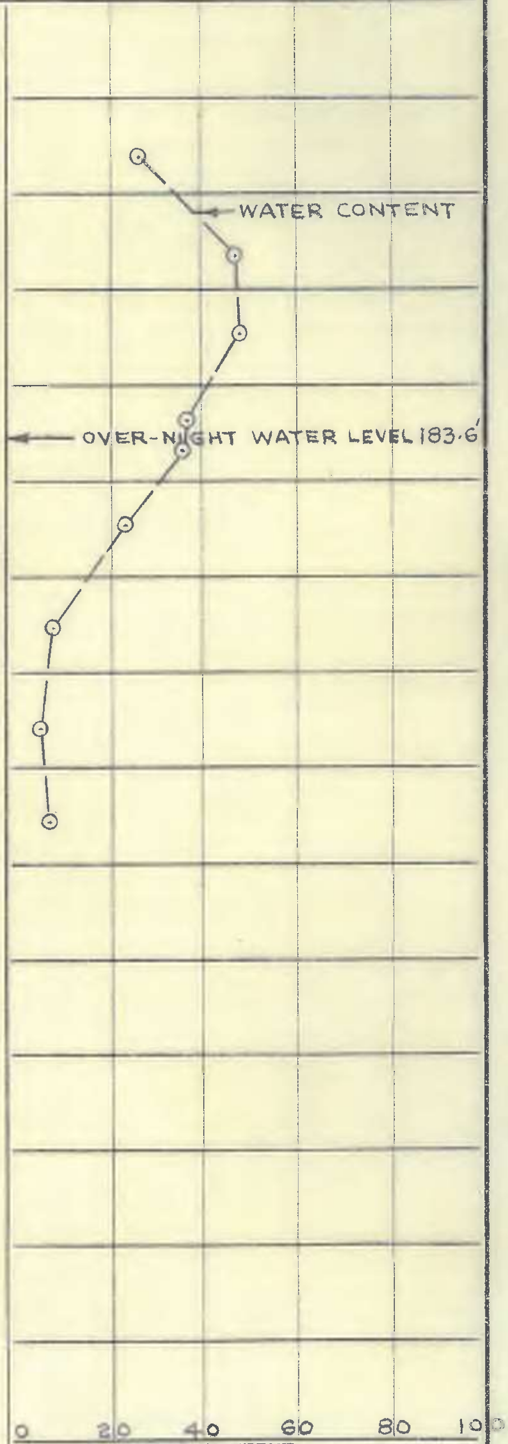
.....................LB. HAMMER

.....................INCH DROP

NO CASING

...........INCH DIA. ROD

~~BLOWS PER FOOT OR~~ ~~SHEAR STRENGTH IN KIPS PER FT.²~~



% WATER CONTENT

NATURAL ⊙

LIQUID LIMIT ☐

PLASTIC LIMIT △

PLATE 10

LOWE-MARTIN CO.—A55512

McROSTIE & ASSOCIATES LTD.
CONSULTING ENGINEERS
OTTAWA CANADA

SOIL PROFILE AND SUMMARY OF FIELD AND LABORATORY TESTS

LEES AVENUE

ELEVATION OF GROUND SURFACE (ZERO DEPTH) 205.2' DATE MAY 10, 1962

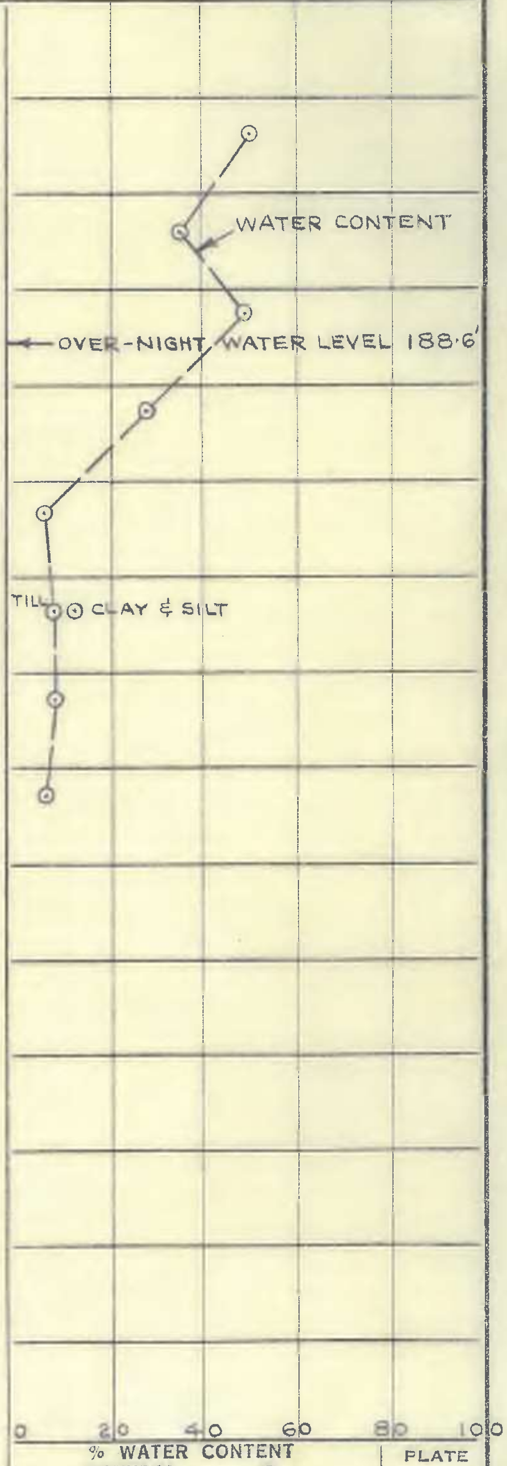
REMARKS SEE PLATE No. 2

HOLE No. 11

UNCONFINED COMPRESSIVE STRENGTH KIPS/FT.²	SMALL SCALE PENETROMETER KIPS/FT.²	STANDARD PENETRATION BLOWS/FT.	SAMPLE NUMBER	DESCRIPTION OF SOIL	DEPTH IN FEET	ELEVATION
				GROUND SURFACE	0'	205.7'
		4	11-1	FILL SAND, ASHES, WOOD, TOPSOIL, GLASS & GRAVEL		
		4	11-2			
		5	11-3			
					20'	185.7'
		8	11-4	LOOSE CLAYEY SILT		
					24'	181.7'
		59	11-5	BOULDERS IN DENSE TILL WITH A 2" CLAY & SILT LAYER & A FEW MEDIUM DENSE LAYERS		
		26 24 for 6"	11-6			
					35'	170.7'
		13 for 6" 35	11-7	DENSE SILTY VERY FINE SAND WITH A LITTLE GRAVEL & A FEW SHALE PARTICLES & A MEDIUM DENSE LAYER		
					40'	
		67	11-8	DENSE SANDY TILL		
					43.3	162.4'
				ROCK CORE RECOVERY 83%		
					49.1'	
				ROCK CORE RECOVERY 95%		
					54.6	151.1'
				BOTTOM OF HOLE		

~~PROBING OR VANE TEST~~

......LB. HAMMER
......INCH DROP
NO CASING
......INCH DIA. ROD
~~BLOWS PER FOOT OR~~ ~~SHEAR STRENGTH IN KIPS PER FT.²~~



% WATER CONTENT
NATURAL ⊙
LIQUID LIMIT □
PLASTIC LIMIT △

PLATE 11

R - REMOULDED

McROSTIE & ASSOCIATES LTD.
CONSULTING ENGINEERS
OTTAWA CANADA

SOIL PROFILE AND SUMMARY OF FIELD AND LABORATORY TESTS

LEES AVENUE

ELEVATION OF GROUND SURFACE (ZERO DEPTH) 206.2' DATE APRIL 12, 1962

REMARKS SEE PLATE No. 2

HOLE No. 12

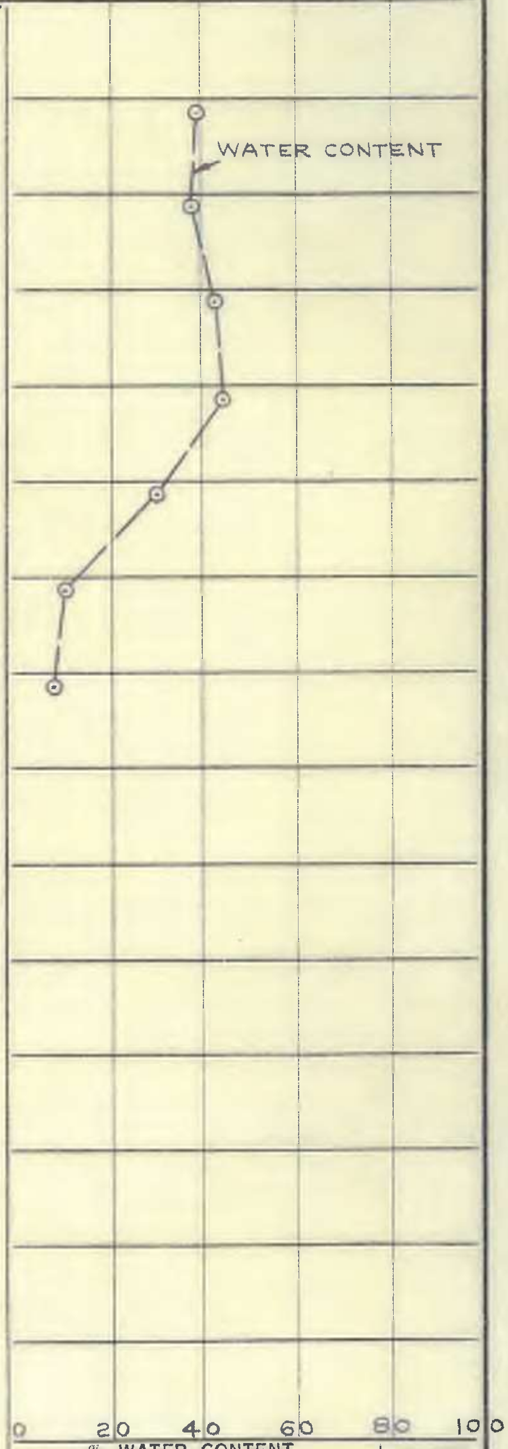
~~PROBING OR VANE TEST~~

......LB. HAMMER ......INCH DROP

NO CASING ......INCH DIA. ROD

~~BLOWS PER FOOT OR~~ ~~SHEAR STRENGTH IN KIPS PER FT.~~^{2}

UNCONFINED COMPRESSIVE STRENGTH KIPS/FT.^{2}	SMALL SCALE PENETROMETER KIPS/FT.^{2}	STANDARD PENETRATION BLOWS/FT.	SAMPLE NUMBER	DESCRIPTION OF SOIL	DEPTH IN FEET	ELEVATION
				GROUND SURFACE	0'	206.2'
		4	12-1	FILL (ASHES, CINDERS, SAND, SILT, WOOD & GLASS)		
		8	12.2			
		9	12.3			
		4	12.4			
					25'	
		74	12.5	FILL GRAY CLAY WITH SOME SAND & A FEW PEBBLES		
					28.5'	177.7'
		50	12.6	DENSE TILL		
		94	12.7			
					39'	167.2'
				WEATHERED OR FRACTURED ROCK CORE RECOVERY 59%	44.7'	161.5
				ROCK CORE RECOVERY 89%	47'	
				WEATHERED OR FRACTURED ROCK CORE RECOVERY 56%	49.3'	156.9'
				BOTTOM OF HOLE		



% WATER CONTENT
NATURAL ⊙
LIQUID LIMIT □
PLASTIC LIMIT △

PLATE 12

R - REMOULDED

FORM S-10-4

McROSTIE & ASSOCIATES LTD.
CONSULTING ENGINEERS
OTTAWA CANADA

SOIL PROFILE AND SUMMARY OF FIELD AND LABORATORY TESTS

LEES AVENUE

ELEVATION OF GROUND SURFACE (ZERO DEPTH) 204.4' DATE APRIL 25, 1962

REMARKS SEE PLATE No. 2

HOLE No. 13

UNCONFINED COMPRESSIVE STRENGTH Kips/FT.²	SMALL SCALE PENETROMETER KIPS/FT.²	STANDARD PENETRATION BLOWS/FT.	SAMPLE NUMBER	DESCRIPTION OF SOIL	DEPTH IN FEET	ELEVATION
				GROUND SURFACE	0'	204.4'
		7	13-1	FILL WOOD, ASHES, CLAY, GLASS, TOPSOIL, CINDERS & BRICK		
		7	13-2			
		11	13-3			
					18'	-186.4'
		37	13-4	DENSE TILL		
					25'	
		25	13-5	MEDIUM DENSE TILL		
		25 5 for 6"	13-6		31'	173.4'
					34.7'	169.7'
				WEATHERED OR FRACTURED ROCK CORE RECOVERY 60%		
					40.8'	163.6
				WEATHERED OR FRACTURED ROCK CORE RECOVERY INDEFINITE		
					46.6'	157.8'
				BOTTOM OF HOLE		

~~PROBING OR VANE TEST~~

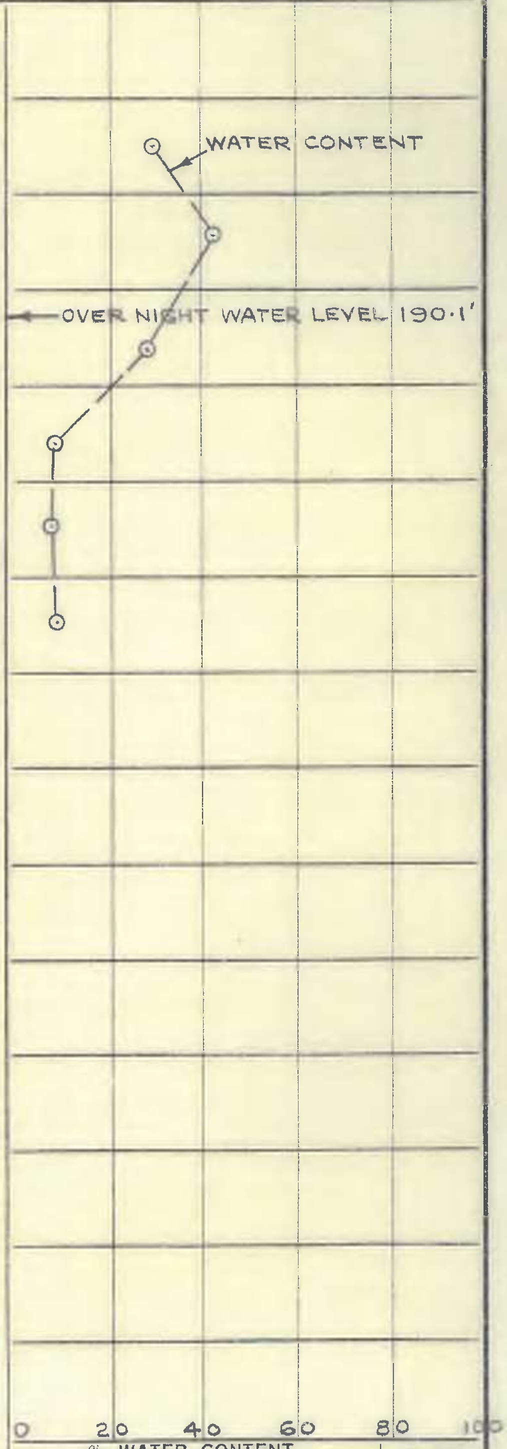
......LB. HAMMER

......INCH DROP

NO CASING

......INCH DIA. ROD

~~BLOWS PER FOOT OR~~ ~~SHEAR STRENGTH IN KIPS PER FT.²~~



% WATER CONTENT
NATURAL ⊙
LIQUID LIMIT □
PLASTIC LIMIT △

R - REMOULDED

PLATE 13

LOWE-MARTIN CO.—A55512

McROSTIE & ASSOCIATES LTD.
CONSULTING ENGINEERS
OTTAWA CANADA

SOIL PROFILE AND SUMMARY OF FIELD AND LABORATORY TESTS

LEES AVENUE

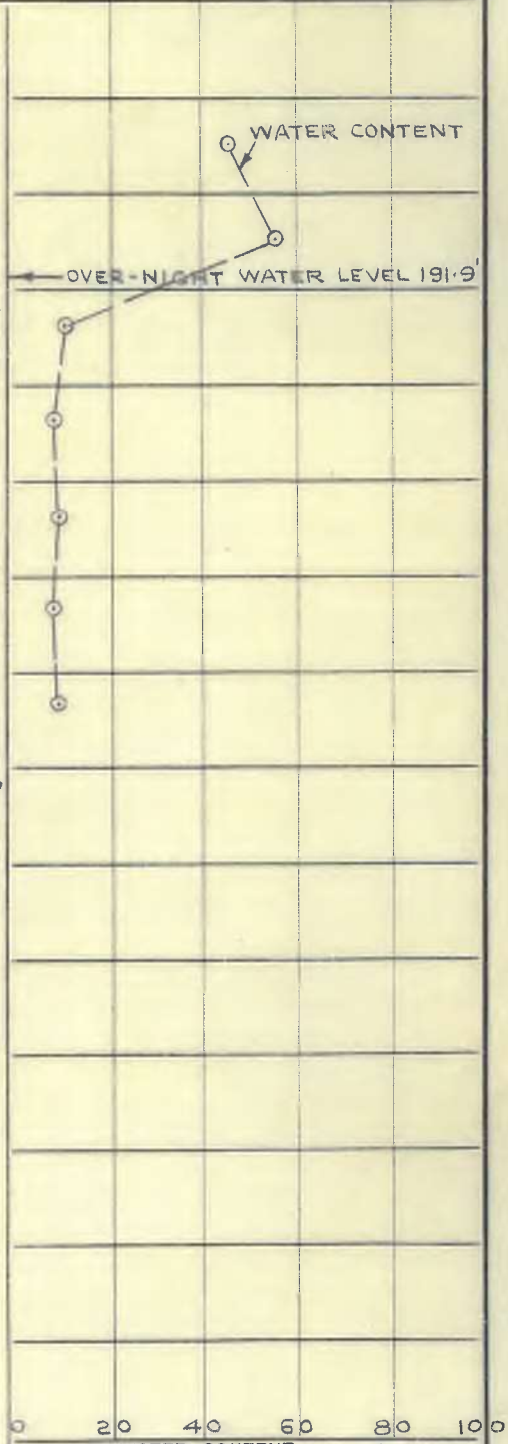
ELEVATION OF GROUND SURFACE (ZERO DEPTH) 204·5' DATE APRIL 11, 1962
REMARKS SEE PLATE No. 2

HOLE No. 14

UNCONFINED COMPRESSIVE STRENGTH Kips/FT.²	SMALL SCALE PENETROMETER KIPS/FT.²	STANDARD PENETRATION BLOWS/FT.	SAMPLE NUMBER	DESCRIPTION OF SOIL	DEPTH IN FEET	ELEVATION
				GROUND SURFACE	0'	204·5'
		5	14-1	FILL (ASHES, CINDERS & SAND)		
		4	14-2			
					15'	189·5'
		24 18 for 6"	14-3	MEDIUM DENSE TILL WITH SOME DENSE LAYERS		
		22 19 for 6"	14-4			
		21	14-5			
		20	14-6			
		24 25 for 6"	14-7			
					39·6'	164·9'
				ROCK CORE RECOVERY 100%	41·5'	
				WEATHERED OR FRACTURED ROCK C.R.36%	42·6'	
				WEATHERED OR FRACTURED ROCK C.R.38%	43·7'	
				WEATHERED OR FRACTURED ROCK CORE RECOVERY 74%	46·6'	
				ROCK CORE RECOVERY 100%	51·5'	153·0'
				ROCK CORE RECOVERY INDEFINITE	55'	149·5'
				BOTTOM OF HOLE		

~~PROBING OR VANE TEST~~

..................LB. HAMMER
..................INCH DROP
NO CASING
...........INCH DIA. ROD
~~BLOWS PER FOOT OR~~ ~~SHEAR STRENGTH IN KIPS PER FT.²~~



% WATER CONTENT
NATURAL ⊙
LIQUID LIMIT ☐
PLASTIC LIMIT △

PLATE 14

R - REMOULDED

LOWE-MARTIN CO.—A55512

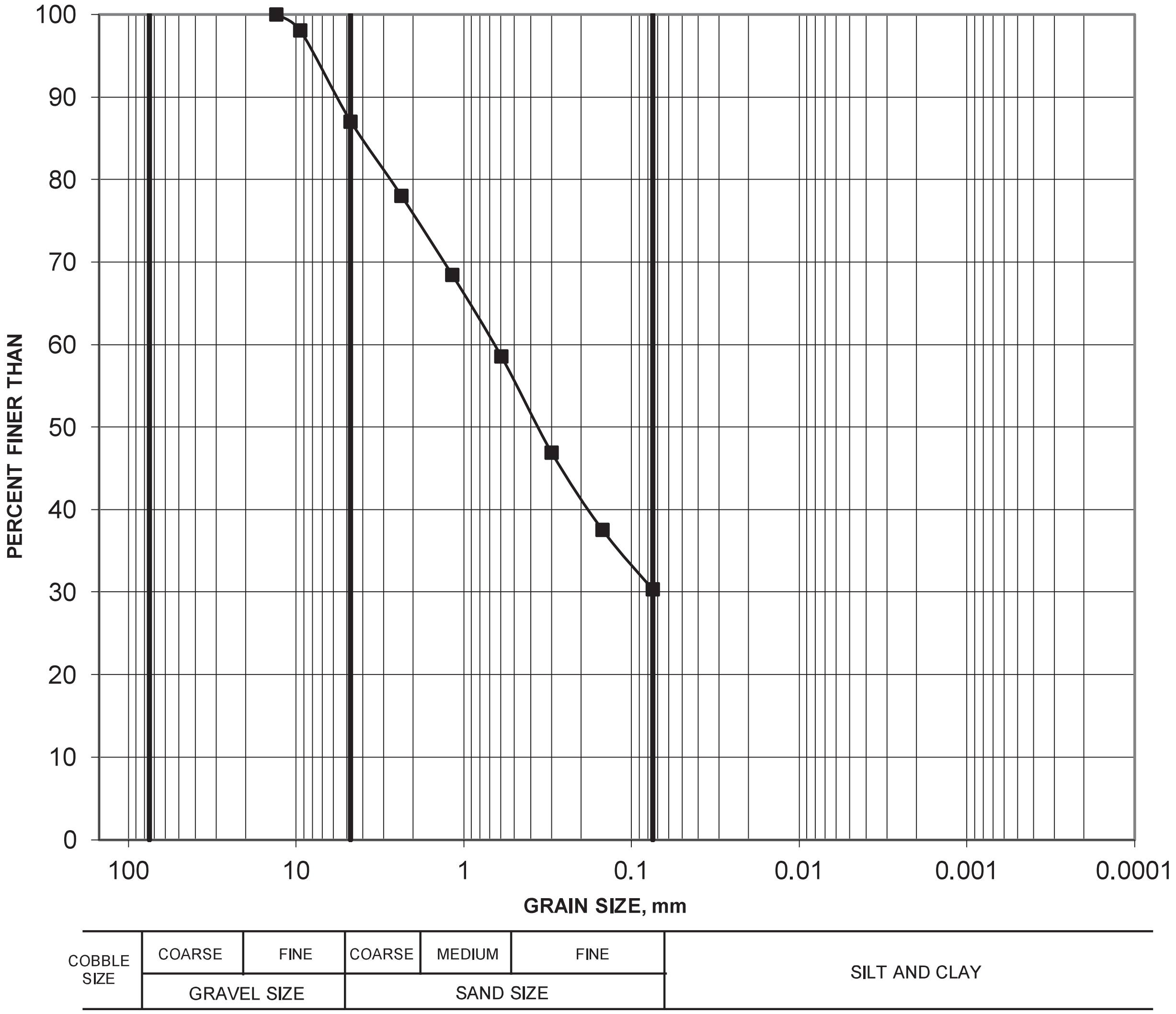
APPENDIX B

- Laboratory Test Results

GRAIN SIZE DISTRIBUTION

FIGURE B1

FILL - (SM) GRAVELLY SILTY SAND



			Constituents (%)			
Borehole	Sample	Depth (m)	Gravel	Sand	Silt	Clay
■ 20-02	5	3.10-3.70	13	57	30	

Project: 20144766

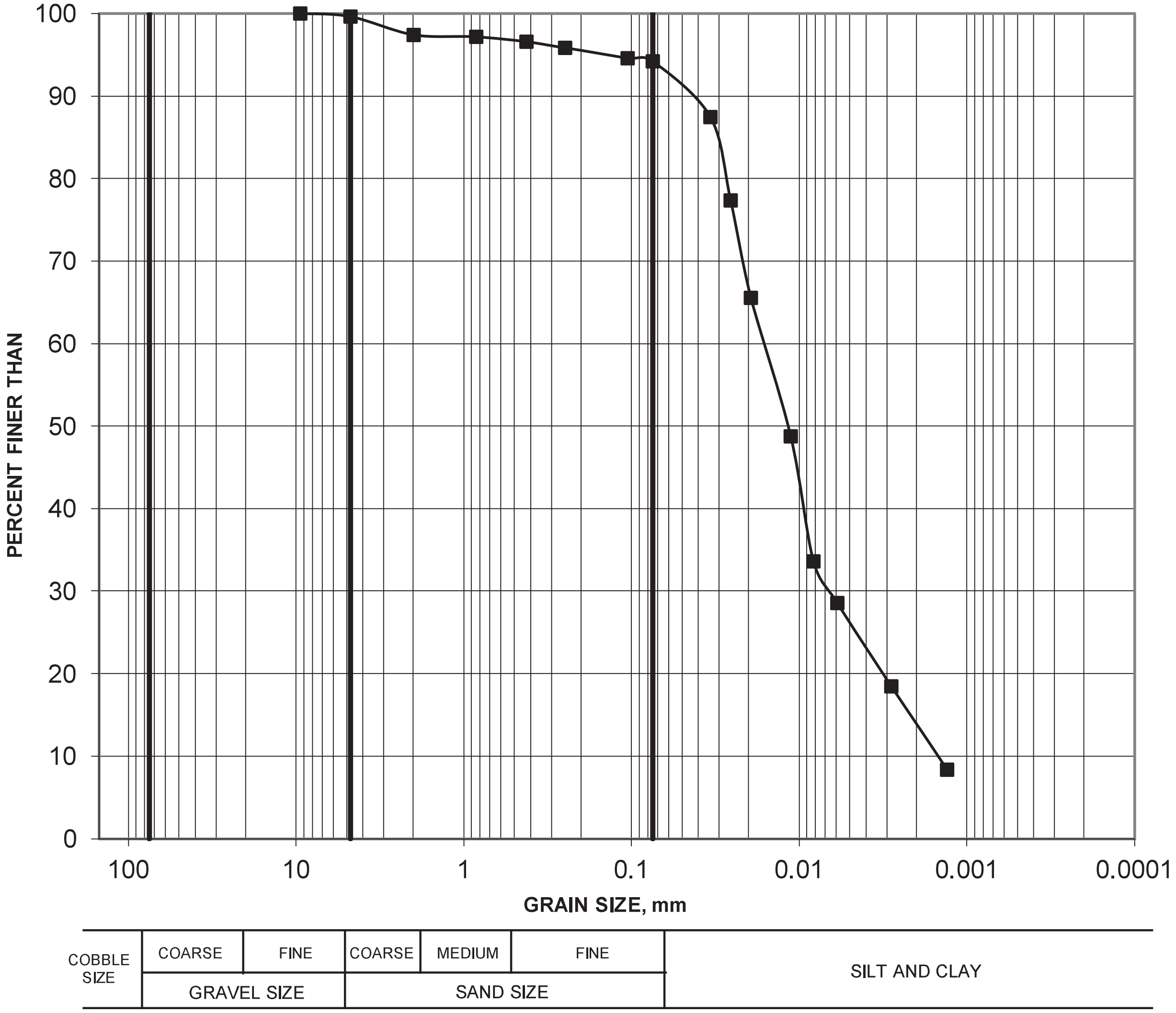
Created by:	CW
Checked by:	MI

https://golderassociates.sharepoint.com/sites/35409g/Shared Documents/Active/2020/20144766/Figures/

GRAIN SIZE DISTRIBUTION

FIGURE B2

(ML) CLAYEY SILT TO SILT (GLACIAL TILL)



			Constituents (%)			
Borehole	Sample	Depth (m)	Gravel	Sand	Silt	Clay
■ 20-02	9	6.10-6.70	0	6	80	14

Project: 20144766

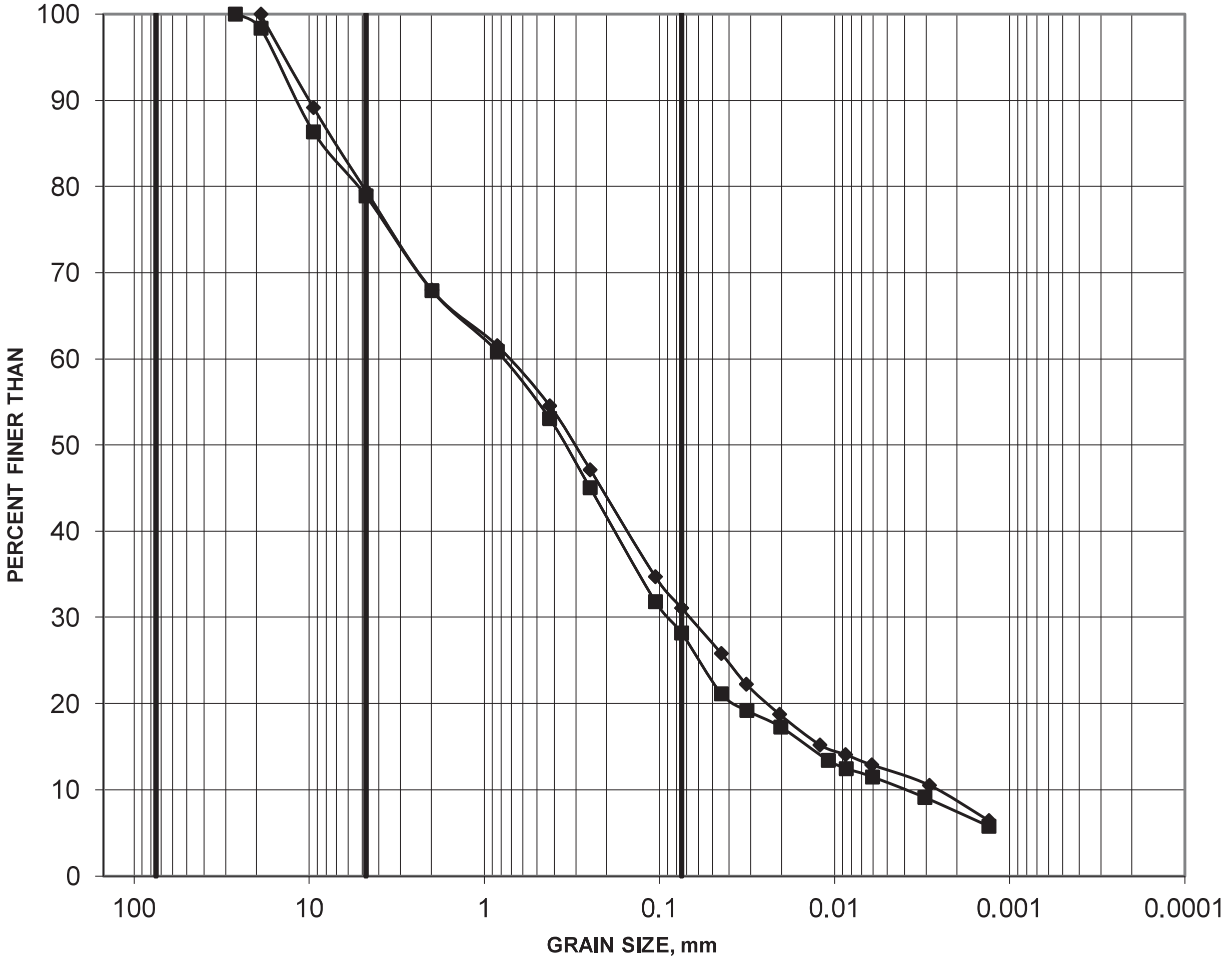
Created by: CW
Checked by: MI

https://golderassociates.sharepoint.com/sites/35409g/Shared Documents/Active/2020/20144766/Figures/

GRAIN SIZE DISTRIBUTION

FIGURE B3

(SM) GRAVELLY SILTY SAND (GLACIAL TILL)



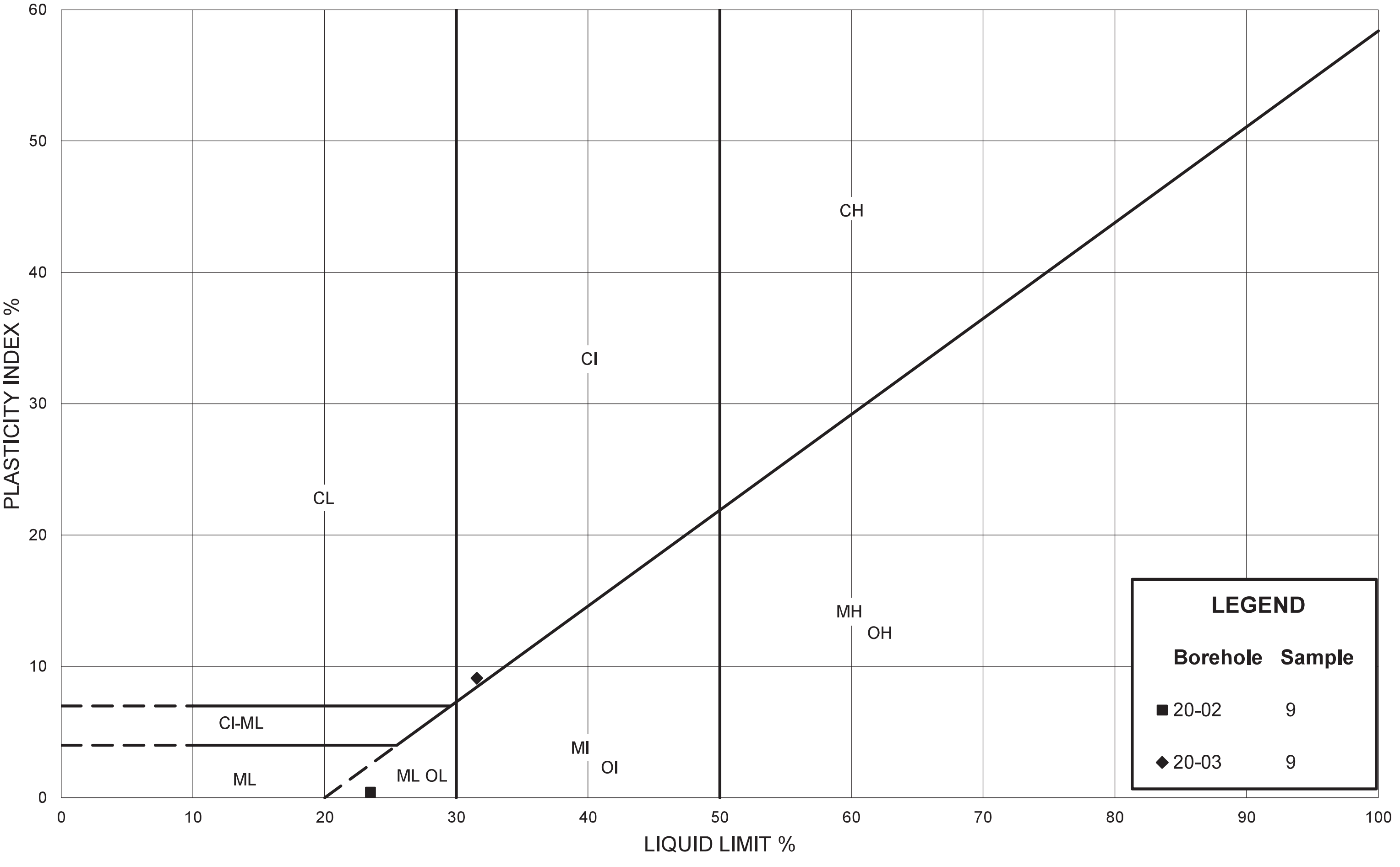
COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND CLAY
	GRAVEL SIZE		SAND SIZE			

			Constituents (%)			
Borehole	Sample	Depth (m)	Gravel	Sand	Silt	Clay
■ 20-01	11	7.60-8.20	21	51	21	7
◆ 20-02	11	8.40-9.00	20	49	22	9

Created by: CW
Checked by: MI

Project: 20144766

https://golderassociates.sharepoint.com/sites/35409g/Shared Documents/Active/2020/20144766/Figures/



PLASTICITY CHART

Figure:	B4
Project:	20144766
Created By: CW	Checked By: MI

APPENDIX C

- Chemical Testing Results

Certificate of Analysis

Client: Golder Associates Ltd. (Ottawa)
1931 Robertson Road
Ottawa, ON
K2H 5B7
Attention: Ms Bridgit Bocage
PO#:
Invoice to: Golder Associates Ltd. (Ottawa)

Report Number: 1935625
Date Submitted: 2020-07-30
Date Reported: 2020-08-07
Project: 20144766
COC #: 860833

				Lab I.D.	1507482
				Sample Matrix	Soil
				Sample Type	
				Sampling Date	2020-07-10
				Sample I.D.	20-03 sa5 / 10-12'
Group	**Analyte**	**MRL**	**Units**	**Guideline**	
Anions	SO4	0.01	%		1.04
Cl in Concrete	Cl	0.002	%		0.093
General Chemistry	Electrical Conductivity	0.05	mS/cm		2.30
	pH	2.00			6.15
	Resistivity	1	ohm-cm		435

Guideline = *** = Guideline Exceedence**

Results relate only to the parameters tested on the samples submitted.
Methods references and/or additional QA/QC information available on request.

MRL = Method Reporting Limit, AO = Aesthetic Objective, OG = Operational Guideline, MAC = Maximum Acceptable Concentration, IMAC = Interim Maximum Acceptable Concentration, STD = Standard, PWQO = Provincial Water Quality Guideline, IPWQO = Interim Provincial Water Quality Objective, TDR = Typical Desired Range

APPENDIX D

- 2011 Vertical Seismic Profiling Memo

Golder Associates

TECHNICAL MEMORANDUM

DATE June 7, 2011 **PROJECT No.** 11-1121-0057

TO Nicolas LeBlanc
Golder Associates Ltd.

CC Michel St-Louis Golder Associates Ltd.

FROM Stephane Sol, Christopher Phillips **EMAIL** ssol@golder.com; cphillips@golder.com

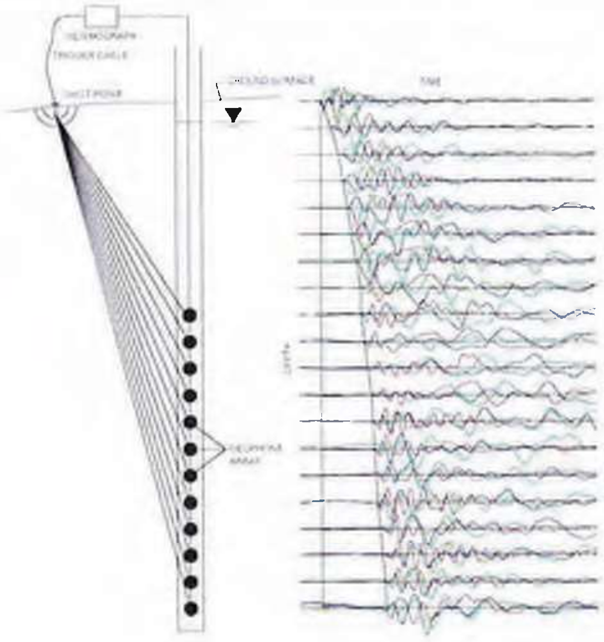
VSP TEST RESULTS – 200 LEES AVENUE, UNIVERSITY OF OTTAWA, OTTAWA, ONTARIO

This memorandum presents the results of the vertical seismic profile (VSP) testing performed at 200 Lees Avenue in Ottawa, Ontario. VSP testing was completed in Borehole BH11-1 on May18, 2011. Borehole BH11-1 is flush mounted and cased with a PVC pipe grouted in place. Borehole logs for BH11-1 indicate approximately 5 metres of fill overlying approximately 6 metres of clayey silt and sandy silt. The clayey silt layer is underlaid by shale bedrock to the bottom of the borehole (approximately 18 metres).

Methodology

For the VSP (Vertical Seismic Profiling) method, seismic energy is generated at the ground surface by an active seismic source and recorded by a geophone located in a nearby borehole at a known depth. The active seismic source can be either compression or shear wave. The time required for the energy to travel from the source to the receiver (geophone) provides a measurement of the average compression or shear-wave seismic velocity of the medium between the source and the receiver. Data obtained from different geophone depths are used to calculate a detailed vertical seismic velocity profile of the subsurface in the immediate vicinity of the test borehole.

The high resolution results of a VSP survey are often used for earthquake engineering site classification, as per the National Building Code of Canada, 2005.



Example 1: Layout and resulting time traces from a VSP survey

Field Work

The field work was completed on May 18, 2011, by personnel from the Golder Mississauga and Ottawa offices.

Both compression and shear-wave seismic sources were used and both were located in close vicinity to the borehole. The seismic source for the compression wave test consisted of a 5.5 kilogram sledge hammer vertically impacted on a metal plate. The plate was located 2 metres from the borehole on asphalt. The seismic source for the shear-wave test consisted of a 2.4 metres long, 150 millimetres by 150 millimetres wooden beam, weighted by a vehicle and horizontally struck with a 5.5 kilogram sledge hammer on alternate ends of the beam to induce polarized shear waves. The shear source was also located 2 metres from the borehole BH11-1, and coupled to the ground surface by parking a vehicle on top of it. Test measurements started at 1-metre below the surface. Data were recorded in the borehole with a 3-component receiver spaced sequentially at 1-metre intervals below the ground surface, to a maximum depth of the borehole (18 metres).

The seismic records collected for each source location were stacked a minimum of ten times to minimize the effects of ambient background seismic noise on the collected data. The data was sampled at 0.020833 millisecond intervals and a total time window of 0.341 seconds was collected for each seismic shot.

Data Processing

Processing of the VSP test results consisted of the following main steps:

1) Combination of seismic records to present seismic traces for all depth intervals on a single plot for each seismic source and for each component;
2) Low Pass Filtering of data to remove spurious high frequency noise;

3) First break picking of the compression and shear-wave arrivals; and

4) Calculation of the average compression and shear-wave velocity to each tested depth interval.

Processing of the VSP data was completed using the SeisImager/SW software package (Geometrics Inc.). The seismic records are presented on the following two plots and show the first break picks of the compression-wave and shear wave arrivals overlaid on the seismic waveform traces recorded at the different geophone depths (Figures 1 and 2). The arrivals were picked on the vertical component for the compression source and on the two horizontal components for the shear source.

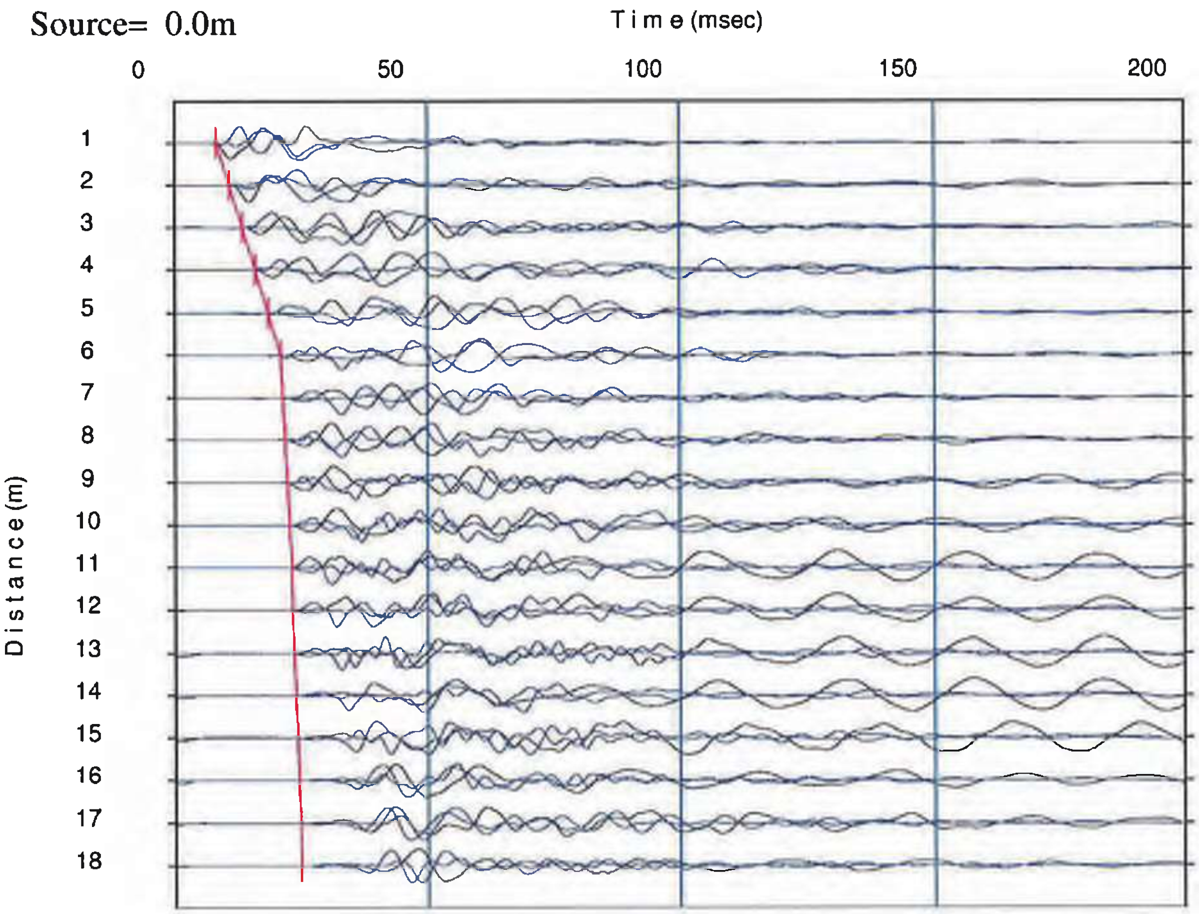


Figure 1: First break picking of compression wave arrivals (red) along the seismic traces recorded at each receiver depth

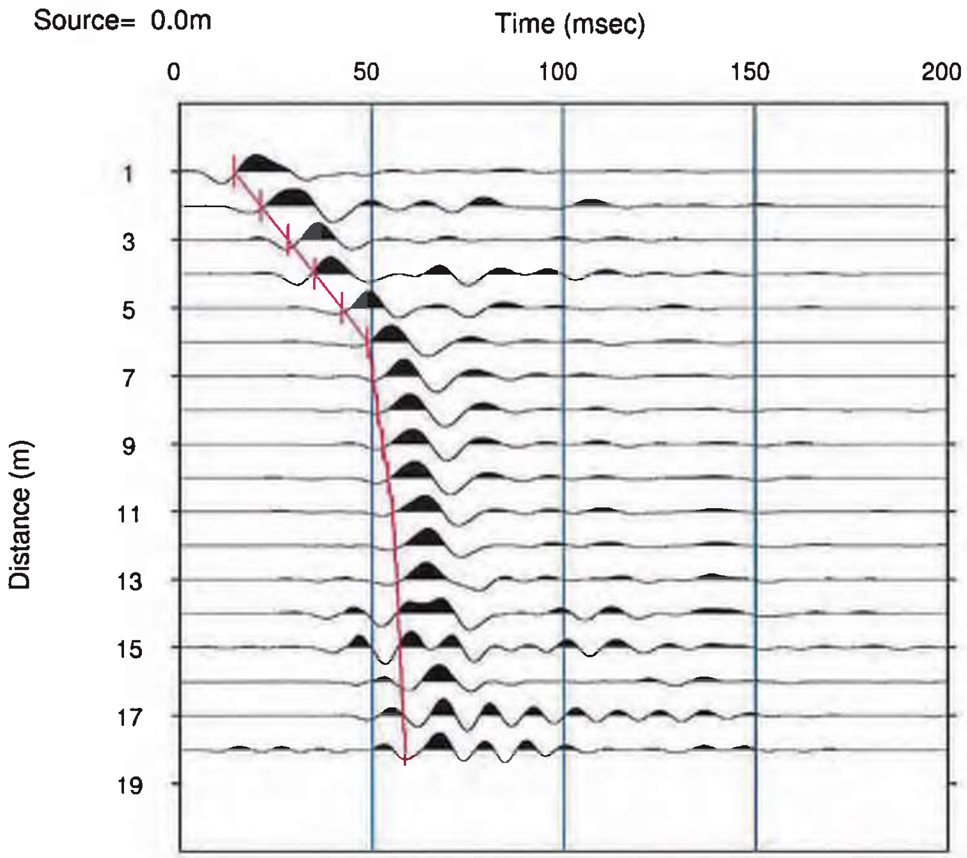


Figure 2: First break picking of shear wave arrivals (red) along the seismic traces recorded at each receiver depth

Results

The VSP results are summarized in Table 1. The shear-wave and compression-wave layer velocities, at one metre intervals, were calculated by best fitting a theoretical travel time model to the field data collected at one metre intervals. The depths presented on the tables are relative to ground surface.

The estimated dynamic engineering moduli, based on the calculated wave velocities, are also presented on Table 1. The engineering moduli were calculated using an estimated bulk density, based on the borehole log, but a more detailed geotechnical investigation would be necessary to determine a more exact density for each layer. For the top soil down to a depth of approximately 11 metres, a bulk density of 1,750 kg/m^3 was estimated. Further down to a depth of 30 metres, the bulk density for the shale bedrock clay was estimated at 2,500 kg/m^3.

The average velocity was calculated assuming that the velocity from 18 metres to a depth of 30 metres was constant with an average shear-wave velocity value of 2,000 m/s which is equal to the velocity of the bedrock at the bottom of the borehole.

The average shear-wave velocity from ground surface to a depth of 30 metres was measured to be 467 m/s.

Closure

We trust that these results meet your current needs. If you have any questions or require clarification, please contact the undersigned at your convenience.

GOLDER ASSOCIATES LTD.

PROFESSIONAL GEOSCIENTISTS ONTARIO
CHRISTOPHER R. PHILLIPS
PRACTISING MEMBER
1503

Stephane Sol, Ph.D.
Geophysics Group

Christopher Phillips, M.Sc., P.Geo.
Senior Geophysicist, Associate

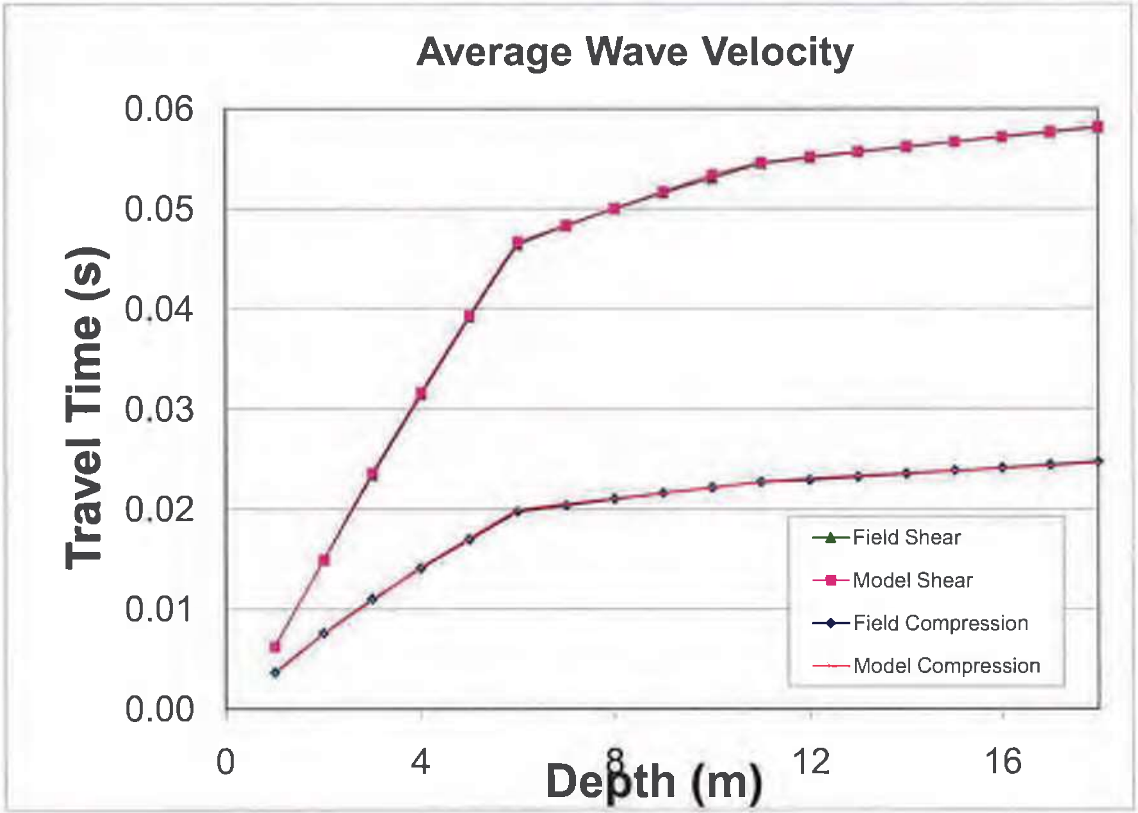
SS/CRP/wlm

Attachments: Table 1

n:\active\2011\other offices\11-1121-0057 - u of o vsp\reporting\11-1121-0057 tm geophysical vsp survey ottawa lees av 07jun11 .docx

TABLE 1
VSP SURVEY RESULTS - BOREHOLE BH11-1
LEES AVENUE
OTTAWA, ONTARIO

Layer Depth (m)				Estimated Bulk Density (kg/m^3)	Dynamic Engineering Properties			
Top	Bottom	Compression Wave	Shear Wave		Poissons Ratio	Shear Modulus (MPa)	Deformation Modulus (MPa)	Bulk Modulus (MPa)
0	1	280	160	1750	0.26	45	113	77
1	2	250	115	1750	0.37	23	63	79
2	3	300	115	1750	0.41	23	65	127
3	4	300	125	1750	0.39	27	76	121
4	5	350	130	1750	0.42	30	84	175
5	6	350	135	1750	0.41	32	90	172
6	7	1800	600	1750	0.44	630	1811	4830
7	8	1800	600	1750	0.44	630	1811	4830
8	9	1800	600	1750	0.44	630	1811	4830
9	10	1800	600	1750	0.44	630	1811	4830
10	11	1800	800	1750	0.38	1120	3084	4177
11	12	3000	1700	2500	0.26	7225	18258	12867
12	13	3800	2000	2500	0.31	10000	26169	22767
13	14	3800	2000	2500	0.31	10000	26169	22767
14	15	3800	2000	2500	0.31	10000	26169	22767
15	16	3800	2000	2500	0.31	10000	26169	22767
16	17	3800	2000	2500	0.31	10000	26169	22767
17	18	3800	2000	2500	0.31	10000	26169	22767



Notes
1. Depth presented relative to ground surface.
2. This Table to be analyzed in conjunction with the accompanying report.

APPENDIX E

- Slope Stability Analysis Output

- Shoreline Photographs

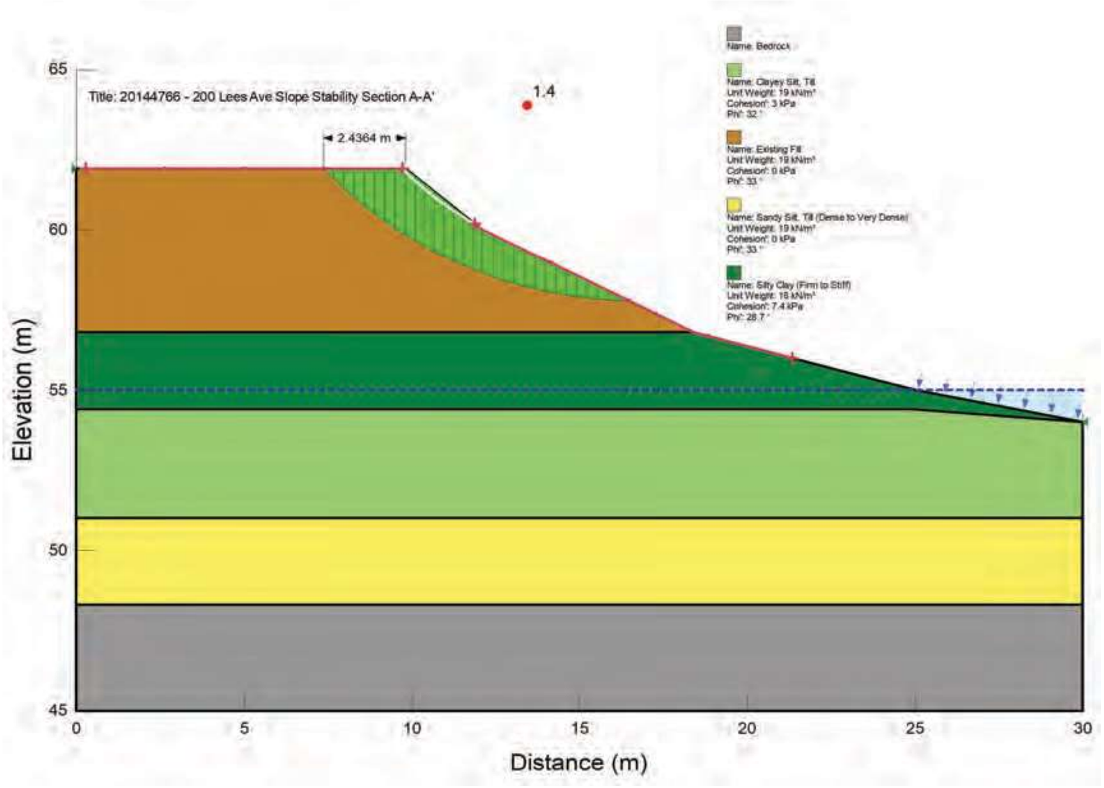


FIGURE E1

REVIEW		
CHECK	CH	2020-08-07
CADD		
DESIGN	BB	2020-08-07
REV. 0	SCALE AS SHOWN	
FILE No.	20144766	
PROJECT No.	20144766	

TITLE

SECTION A-A'
STABILITY ANALYSES
STATIC DRAINED CONDITION

PROJECT

GEOTECHNICAL INVESTIGATION
AND SLOPE STABILITY ASSESSMENT
200 LEES REDEVELOPMENT
OTTAWA, ONTARIO

GOLDER

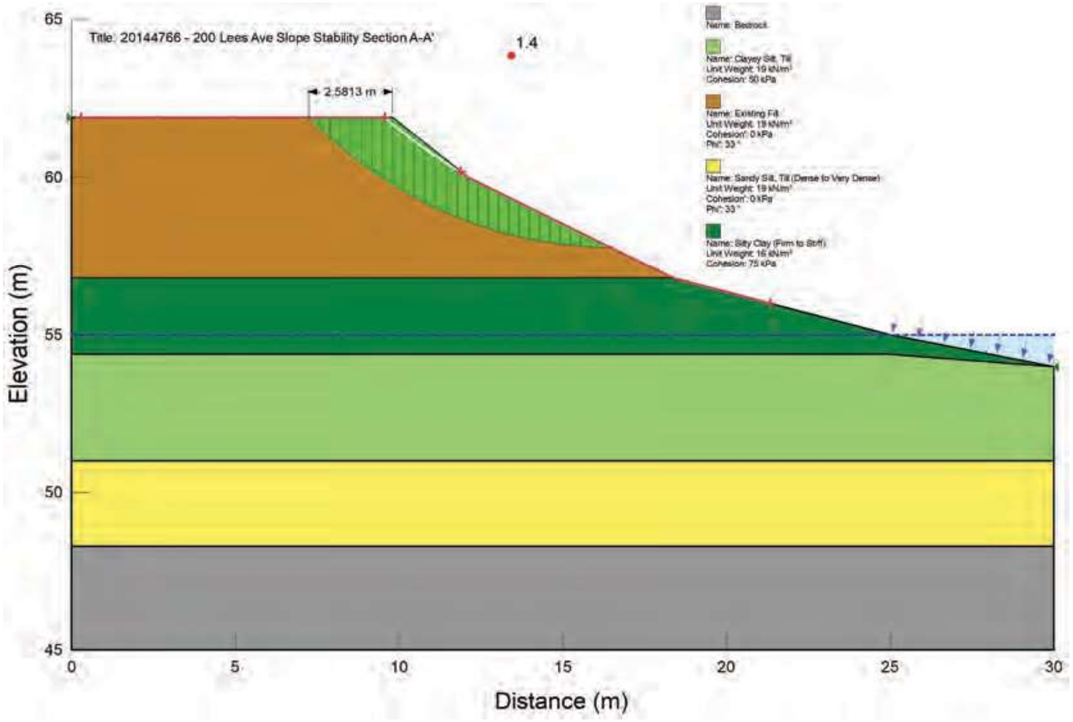


FIGURE E2

REVIEW
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CADD
DESIGN BB 2020-08-07
REV. 0 SCALE AS SHOWN
FILE No. 20144766
PROJECT No. 20144766

TITLE

SECTION A-A'
STABILITY ANALYSES
STATIC UNDRAINED CONDITION

PROJECT

GEOTECHNICAL INVESTIGATION
AND SLOPE STABILITY ASSESSMENT
200 LEES REDEVELOPMENT
OTTAWA, ONTARIO

GOLDER

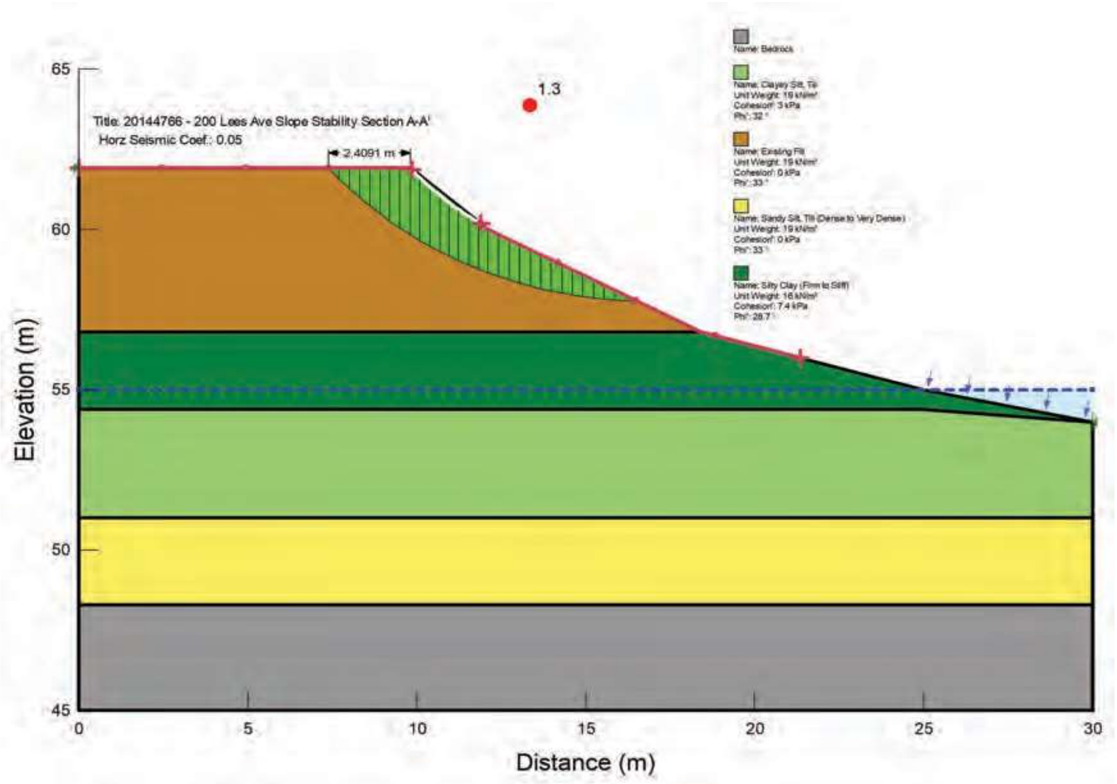

FIGURE E3

REVIEW

CHECK CH 2020-08-07

CADD

DESIGN BB 2020-08-07

REV. 0 SCALE AS SHOWN

FILE No. 20144766

PROJECT No. 20144766

TITLE

SECTION A-A'
STABILITY ANALYSES
SEISMIC CONDITION

PROJECT

GEOTECHNICAL INVESTIGATION
AND SLOPE STABILITY ASSESSMENT
200 LEES REDEVELOPMENT
OTTAWA, ONTARIO

GOLDER

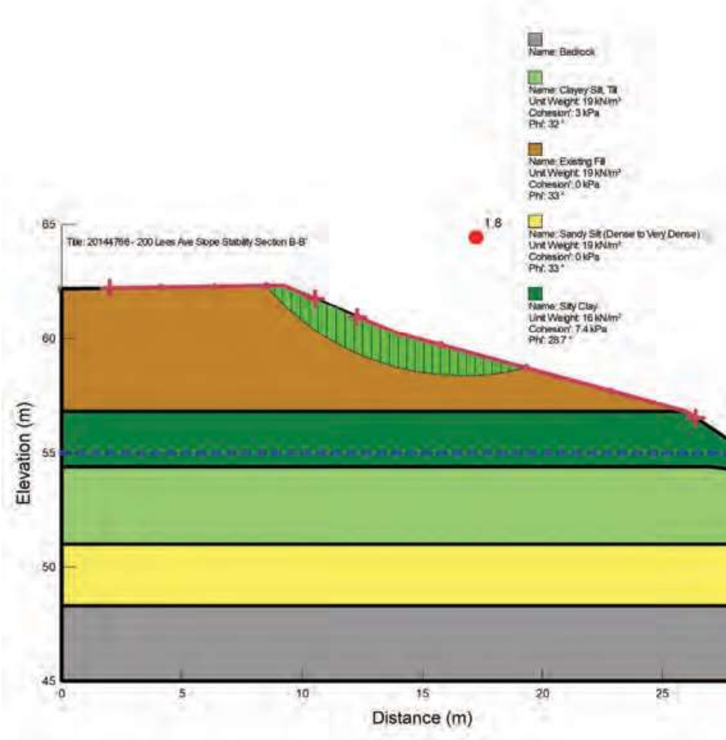

FIGURE E4

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CADD		
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REV. 0	SCALE AS SHOWN	
FILE No.	20144766	
PROJECT No.	20144766	

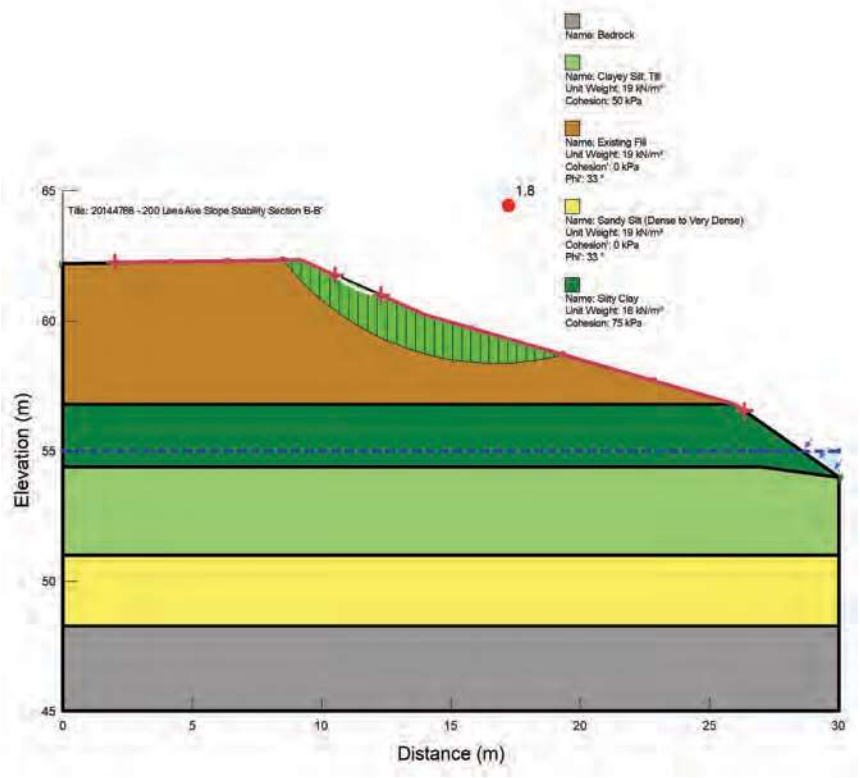
TITLE

SECTION B-B'
STABILITY ANALYSES
STATIC DRAINED CONDITION

PROJECT

GEOTECHNICAL INVESTIGATION
AND SLOPE STABILITY ASSESSMENT
200 LEES REDEVELOPMENT
OTTAWA, ONTARIO

GOLDER



TITLE

SECTION B-B'
STABILITY ANALYSES
STATIC UNDRAINED CONDITION

PROJECT

GEOTECHNICAL INVESTIGATION
AND SLOPE STABILITY ASSESSMENT
200 LEES REDEVELOPMENT
OTTAWA, ONTARIO

PROJECT No. 20144766
FILE No. 20144766
REV. 0 SCALE AS SHOWN
DESIGN BB 2020-08-07
CADD
CHECK CH 2020-08-07
REVIEW

FIGURE E5

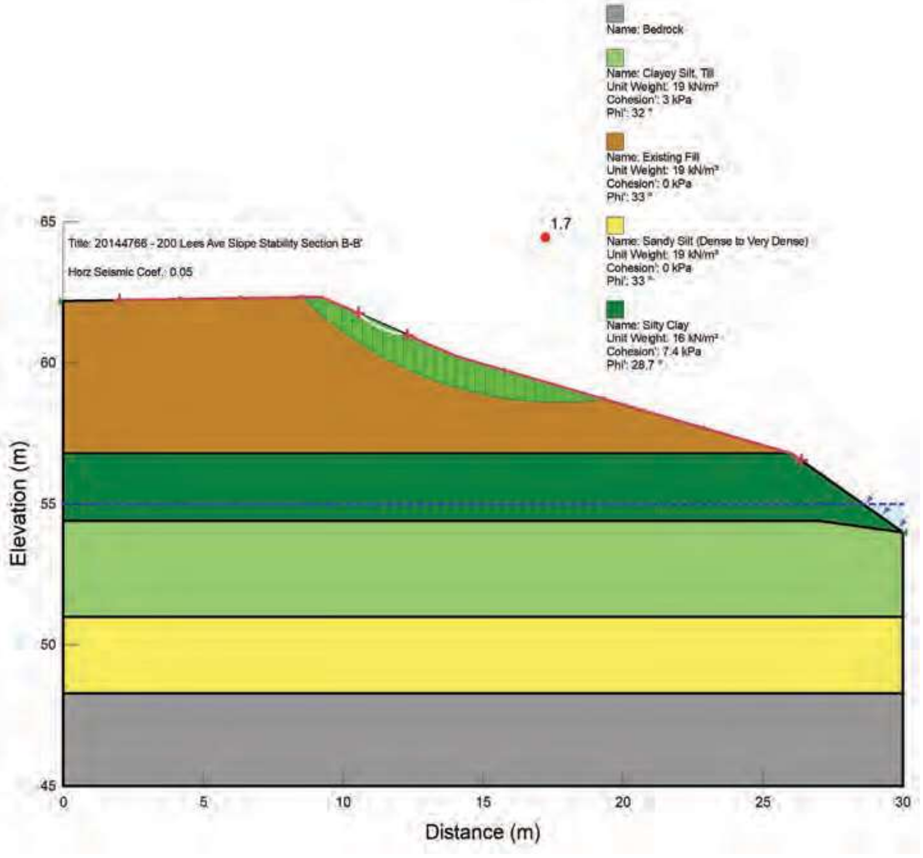


FIGURE E6

REVIEW		
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REV. 0	SCALE AS SHOWN	
FILE No.	20144766	
PROJECT No.	20144766	

TITLE

SECTION B-B'
STABILITY ANALYSES
SEISMIC CONDITION

PROJECT

GEOTECHNICAL INVESTIGATION
AND SLOPE STABILITY ASSESSMENT
200 LEES REDEVELOPMENT
OTTAWA, ONTARIO

GOLDER

PHOTO 1

PROJECT No. 20144766

FILE No. 20144766

REV. 0

SCALE AS SHOWN

DESIGN	BB	2020-08-07
CADD		
CHECK	CH	2020-08-07
REVIEW		

TITLE

SHORELINE PHOTO
FACING EAST

PROJECT

GEOTECHNICAL INVESTIGATION
AND SLOPE STABILITY ASSESSMENT
200 LEES REDEVELOPMENT
OTTAWA, ONTARIO

GOLDER

PHOTO 2

PROJECT No.		20144766
FILE No.		20144766
REV. 0		SCALE AS SHOWN
DESIGN	BB	2020-08-07
CADD		
CHECK	CH	2020-08-07
REVIEW		

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PHOTO 3

PROJECT No.	20144766		
FILE No.	20144766		
REV. 0	SCALE AS SHOWN		
DESIGN	BB	2020-08-07	
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