SOLDER

REPORT

GEOTECHNICAL INVESTIGATION

Holland Cross Expansion Ottawa

Submitted to:

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1.0 INTRODUCTION

This report presents the results of a geotechnical and environmental assessment carried out at the Site of a proposed apartment building to be located at 1560 Scott Street in Ottawa, Ontario.

The purpose of this geo-environmental investigation was to assess the general subsurface conditions at the site by means of a limited number of boreholes. Based on an interpretation of the factual information obtained, a general description of the subsurface conditions is presented. These interpreted subsurface conditions and available project details were used to provide 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 PROJECT AND SITE

The site of the proposed development is located at 1560 Scott Street in Ottawa, Ontario. A site plan is shown in Figure 1.

The Site has been previously developed and is currently occupied by a number of structures:

- The overall site measures about 140 m by 140 m in plan view and contains two 7 storey office buildings, one along the northern perimeter and one on the western perimeter, and 2 storey building in the southern part of the site. A single storey building covers most of the remainder of the site footprint.
- The proposed apartment building will be located in the southeast corner bordered to the north by Scott Street, to the west by Holland Avenue, to the south by multi-storey residential buildings and to the east by Hamilton Avenue.
- It is understood that a portion of the existing building will be demolished to allow for construction of the construction of the new building. The existing building to be demolished currently has two floors of underground parking.

It is understood that the proposed apartment building will consist of the following:

- The proposed building footprint is shown on the Site plan, Figure 1
- The proposed building will be approximately 25 storeys in height and encompass a plan area of about 34 m by 49 m.
- Similar to the existing structure at the site, the proposed structure will have two basement/below-grade levels. These basement levels will be completely contained within the footprint of the current building and will be at the same elevations. They will, therefore, not require any additional basement excavation.
- Additional details on finished floor slab levels were not available at the time of preparation of this report. It is, however, assumed that there would be no significant regrading of the site (given that the building will occupy one corner of an already extensively developed property).

2.1 Available Subsurface Information

Previous subsurface investigations at or near the site were carried out by Golder, and also by McRostie Genest Middlemiss and Associates (McRostie) who have since joined Golder. The locations of those previous boreholes/test pits are shown on the attached Site Plan (Figure 1). The following reports were reviewed in the assessment of site conditions for this study, which include the investigations for the existing development:

- 1) Report to Pomerleau by Golder titled "*Geotechnical Investigation Design Input, Holland Cross Expansion, 1560 Scott Street, Ottawa, Ontario*" dated May 2020 (Report No. 20141578).
- Report to J.L. Richards & Associates Ltd. by Golder titled "Geotechnical Investigation, Proposed Watermain and Sanitary Sewer Replacement, Holland Avenue, Scott Street to Tyndall Street, Ottawa, Ontario" dated June 2012 (Report No. 11-1121-0281).
- 3) Letter to Laurnic Investments by McRostie titled "*Holland and Spencer Avenues, Beech Foundry Site, Rock Elevations*" dated June 6, 1984 (Report No. SF-2481).
- Report to Citicom Inc., Brisbin Brooke Beynon, Architects and Carwood Leclair Inc. Consulting Engineers by McRostie titled "*Holland Cross Project, Holland Ave., Spencer St. & Scott St., Ottawa*" dated July 3, 1986 (Report No. SF-2687).

Based on the available information, the subsurface conditions are anticipated to consist surficial fill material overlying a thin veneer of glacial till, over bedrock. In general, the bedrock surface at the Site is expected to vary from about 0.5 to 2.8 m below the existing ground surface.

Published bedrock geology mapping indicates that the site is underlain by dolomite and limestone of the Bobcaygeon Formation.

3.0 PROCEDURE

The field work for the current geotechnical and environmental investigation was carried out between August 29 and August 30, 2022. During that time, three boreholes (numbered 22-01 to 21-03) were advanced at the approximate locations shown in the site plan in Figure 1.

The boreholes were advanced with a truck-mounted hollow stem auger drill rig supplied and operated by Marathon Underground. The boreholes were advanced to depths ranging from approximately 6 to 9 m below the existing ground surface.

All boreholes were advanced to refusal on the bedrock surface at depths ranging from 1.5 to 2.0 m. Upon encountering refusal, all three boreholes were advanced into the bedrock using rotary diamond drilling techniques while retrieving NQ sized core up to a depth of approximately 9 m at BH22-02B and BH 22-03; and approximately 6 m below ground surface at BH22-01.

Standard Penetration Tests (SPTs) were carried out within the overburden at various intervals of depth in general conformance with ASTM D 1586. Soil samples were recovered using split-spoon sampling equipment.

Monitoring wells were sealed into all boreholes to allow for subsequent measurements of stabilized groundwater levels. The monitoring wells consist of 32 mm inside diameter rigid PVC pipe with 3 m long slotted screen sections, installed within silica sand backfill, and sealed by a section of bentonite hole plug. Measurement of the groundwater levels was completed on October 3, 2022.

The fieldwork was supervised by Golder staff who logged the boreholes, directed the in-situ testing, and collected the soil and rock samples retrieved in the boreholes. The samples obtained during the fieldwork were brought to our laboratory for further examination and laboratory testing.

The laboratory testing included determination of natural water content and grain size distribution on selected soil samples, as well as Uniaxial Compressive Strength (UCS) testing on selected bedrock samples.

Shear wave velocity profiling at the site was completed using the Multichannel Analysis of Surface Waves (MASW) technique and was carried out on September 15, 2022 by Golder personnel. For the MASW line, a series of 24 low frequency (4.5 Hz) geophones were laid out at about 1 m intervals. An 8-kg sledgehammer and a 40-kg weight drop were used as the seismic sources. The source locations were offset at distances of about 5 and 10 m from and collinear with the geophone array.

The borehole locations were marked in the field and surveyed by Golder. The positions and ground surface elevations at the borehole locations were determined 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 Universal Transverse Mercator (UTM Zone 09) coordinate system. The elevations are referenced to the Geodetic datum (CGVD28).

4.0 SUBSURFACE CONDITIONS

4.1 General

The approximate locations of the boreholes and test pits previously advanced at the site are identified on Figure 1. Relevant borehole and test pit records from the previous investigations in the immediate vicinity of the proposed building are provided in Appendix B.

The following sections provide an overview of the subsurface conditions encountered. It should be noted that the previous investigations pre-dated development of the site and, as such, the near surface conditions are likely to have been altered by the existing development (e.g., removal of materials to permit construction of the existing below-grade structures, changes to the site grading) including significant bedrock excavations at the building locations.

In general, the subsurface conditions within the footprint of the proposed building consist of surficial thin fill layer, over a thin deposit of Glacial Till overlying limestone with thin shale interbeds. It should be noted that the current building is understood to have two floors of basement. Upon demolition, there will therefore also be an area in which the bedrock has been removed two storeys and the bedrock surface below the existing building will therefore be lower than encountered in the boreholes (which were drilled around the perimeter of the building).

4.2 Pavement Structure

A layer of asphaltic concrete, ranging from 100 to 150 mm thick, was encountered at BH22-02B and BH22-01 during the current investigation.

A 60 mm concrete block surface was present at BH22-03. The concrete block surface was overlying granular base/subbase material.

4.3 Surficial Fill Materials

A thin layer of fill material was present underlying the concrete slab, and the asphaltic concrete, within the proposed building footprint; the fill extended to depths of up to 1.37 m below the original ground surface within the footprint of the new development).

The previous geotechnical investigations carried out on this site indicate that the fill and/or organic materials were underlain by glacial till at or near the proposed building footprint. The glacial till consists of a heterogeneous mixture of gravel, cobbles, and boulders in a silty sand matrix.

As the proposed building footprint currently contains two below grade levels, it is anticipated that the above noted materials were removed (within the footprint of the building) during construction of the existing building.

4.4 Glacial Till

A layer of Glacial Till (Silty Sand to Sandy Gravel) was encountered in all boreholes ranging from about 0.76 m to 1.52 m.

The Till layer is brown to grey in colour, with measured SPT "N" values ranging from 27 to greater than 55 blows per 0.3 m of penetration, indicating a compact to very dense state of compaction.

4.5 Bedrock / refusal

Refusal to augering was encountered in all boreholes during the current investigation at depths ranging from 1.5 to 2.0 m below the existing ground surface. The bedrock was cored in all of the current boreholes to a maximum depth of 9 m below the existing ground surface. The following table summarizes the ground surface, bedrock or auger refusal depths and elevations, and core lengths as encountered at the borehole locations within (or near to) the footprint of the proposed building:

Borehole/ Test Pit Number	Ground Surface Elevation (m)	Depth to Bedrock Surface or Auger Refusal (m)	Core Length (m)	Bedrock or Auger Refusal Elevation (m)
22-01 (Golder, 2022)	61.60	1.60	4.67	60.00
22-02B (Golder 2022)	61.72	1.85	7.15	59.87
22-03 (Golder 2022)	60.16	1.52	7.00	59.68
TP11 (Mcrostie,1984)	62.48	1.34	-	60.77
N150 E120 (Mcrostie,1986)	61.50	1.60	-	59.95
N120 E120 (Mcrostie,1986)	62.24	2.45	-	59.79
N180 E110 (Mcrostie, 1986)	62.06	2.30	-	59.76

The bedrock encountered in the cored boreholes typically consists of limestone with interbedded shale.

Rock Quality Designation (RQD) values measured in the boreholes ranges from 46 to 96%, indicating a poor to excellent quality rock.

The results of laboratory testing carried out on two samples of the cored bedrock from 22-02B and 22-03 measured Uniaxial Compressive Strengths (UCS) of about 169 and 118 MPa, respectively, indicating the samples of the rock tested are strong to very strong. Photographs of the recovered bedrock cores and results of the UCS testing are presented in Appendix G.

4.6 Groundwater conditions

Monitoring wells were sealed in three boreholes (22-01, 22-02B and 22-03) to allow for groundwater level measurements and hydraulic conductivity testing. The groundwater levels were measured on October 3, 2022. Hydraulic conductivity testing was completed on October 3, 2022. The results of the hydraulic conductivity analyses are provided in Appendix F. The measured groundwater levels and hydraulic conductivity testing results are presented in the table below.

Doucholo/Too4 Di4 Nourshou	Geological unit of	Ground Surface	Ground Water Depth urface evation (m) Depth (m) Elevation (m)		Ground Water Depth (m)		Measurement	Hydraulic
Borenole/Test Pit Number	screened Interval	Elevation (m)			Dates	(cm/s)		
22-01 (Golder, 2022)	Bedrock	61.60	3.83	57.77	Oct. 3, 2022	2x10 ⁻⁶		
22-02B (Golder 2022)	Bedrock	61.72	4.88	56.84	Oct. 3, 2022	3x10 ⁻⁴		
22-03 (Golder 2022)	Bedrock	61.68	5.43	56.25	Oct. 3, 2022	5x10 ⁻³		

Groundwater levels are expected to fluctuate seasonally and over shorter periods of time. Higher groundwater levels are expected during wet periods of the year, such as spring after the snowmelt or during periods of heavy rain. The water table elevation at the site may decrease in localized areas after development depending on the elevation of the building drains and linear infrastructure.

5.0 GEOPHYSICAL INVESTIGATION

Golder completed a Multichannel Analysis of Surface Waves (MASW) survey to estimate the shear velocity at the proposed development. The result of this investigation is presented in appendix D.

6.0 DISCUSSION AND GEOTECHNICAL RECOMMANDATION

6.1 General

This section of the report provides engineering guidelines on the geotechnical design aspects of the project based on our interpretation of the available information described herein and project requirements.

The information in this portion of the report is provided for planning and design purposes for the guidance of the design engineers and architects. The recommendations provided herein are consistent with the Ontario Building Code of 2012 (OBC 2012). Where comments are made on construction, they are provided only to highlight aspects of construction which could affect the design of the project. Contractors bidding on or undertaking the works should examine the factual results of the investigation, satisfy themselves as to the adequacy of the factual information for construction, and make their own interpretation of the factual data as it affects their proposed construction techniques, schedule, safety, and equipment capabilities, costs, sequencing and the like.

6.2 Foundation Design

It is understood that the proposed building will have two basement levels (at similar elevations to the existing basement levels). The proposed development will therefore not require bulk excavation. Minor, localized excavation may be required to accommodate footing construction.

The bedrock surface is at about 1.34 to 2.45 metres depth below the existing ground surface (i.e., elevations ranging from 61.1 to 59.7 metres). The proposed structure is planned to have two underground parking levels. As such, the excavation for the building tower is expected to extend to depths of about 7 to 9 metres below existing site grades. At these levels, new building foundations are expected to be founded within limestone bedrock (provided they are at or below the elevation of the existing basement excavation).

It is expected the tower could be supported on pad, strip or raft foundations placed on the bedrock at the base of the basement excavation. Foundations supported directly on the bedrock may be designed using a factored Ultimate Limit States bearing resistance of 6 MPa. Provided the bedrock surface is properly cleaned of soil and loose rock at the time of construction, the settlement of footings sized using this factored bearing resistance should be less than the 25 mm which is typically accepted and therefore Serviceability Limit States (SLS) typically do not govern the design of shallow foundations on rock.

Foundations should be entirely supported on rock. If the existing rock surface is below the planned footing level at the time of construction (for example where a previous excavation was present), mass concrete should be placed to bring the surface up to the planned underside of footing. Mass concrete, if used, should extend beyond the edge of the footing a distance equal to the depth of the mass concrete.

6.3 Seismic Design

Based on the results of the Multichannel Analysis of Surface Waves (MASW) testing carried out at this site, this site can be assigned a Site Class of **B** for seismic design purposes in accordance with the 2012 OBC for all structures founded on rock.

6.4 Excavations

Details on the finished floor elevations for the proposed building were not available at the time of preparation of this report. However, it is understood that the proposed building will be constructed within a portion of the existing building footprint which contains two below-grade levels, and which will be demolished prior to construction of the new building. The proposed building will incorporate two below-grade levels. As the proposed and existing buildings both have two underground levels, it is anticipated that excavations will be limited primarily to small, localized excavations in new footing areas, utility trenches, etc. These localized foundation excavations are therefore expected to be within limestone bedrock. Shallow excavations may also be required outside the building for utility trenches and other buried works.

In general, the subsurface conditions on this site consisted of topsoil and fill overlying glacial till, with the bedrock surface located at depths varying from about 1.6 to 2.5 m below the ground surface at the time of the previous investigations. In accordance with the Occupational Health and Safety Act (OHSA) of Ontario, the soils above the water table at this site would generally be classified as Type 3 soils and side slopes in the overburden <u>above the water table</u> may therefore be sloped at a minimum of 1H:1V. However, in accordance with the OHSA of Ontario, the soils below the water table would generally be classified as Type 4 soils, and excavation side slopes must be sloped at a minimum of 3H:1V if dewatering of these materials is not carried out. This condition is not, however, anticipated to exist based on the current information.

Depending on the final excavation geometry (i.e., if sloped excavations cannot be accommodated), some shoring/temporary support may be needed for the excavation in overburden adjacent to the loading dock facility located immediately north of the proposed building and/or adjacent to Hamilton Avenue to prevent undermining of the roadways.

It is expected that near vertical walls may be developed in the bedrock for the shallow excavations needed for new footing construction in the floor of the existing basement. Similarly, if/where the existing foundation walls are removed; leaving the existing vertical bedrock excavation walls in place is anticipated to be feasible.

However, the exposed bedrock should be inspected by qualified geotechnical personnel at the time of excavation to confirm this assessment. It is also possible that previous blasting has damaged/loosened the existing rock faces and localized rock stabilization (such as rock bolting, shotcreting, installation of rock fall mesh, etc.) may be required if areas of poor rock are exposed in the excavation.

Shallow depths of bedrock removal for this project, such as those required for localized excavations for footings, could be accomplished using mechanical methods (such as hoe ramming in conjunction with line drilling). Care will need to be taken to protect the adjacent structures/foundations from damage during bedrock excavation. It is expected/assumed that blasting will not be required.

It is assumed that there is an existing drainage system below the existing building floor slab which has lowered the groundwater level to below the base of the existing building. Provided that the bulk excavation for the new building does not extend substantially below the current below-grade building levels, groundwater inflow into the foundation excavations can probably be handled by pumping from properly constructed and filtered sumps located within the excavations.

6.4.1 Bedrock Excavation

It is likely that the localized bedrock removal will be carried out using drill and blast techniques or mechanical methods (such as hoe ramming or hydraulic jacks) in conjunction with line drilling. Small, shallow excavations in bedrock are typically carried out mechanically, while larger, deeper excavations are typically more economical using blasting.

If blasting is considered, blast induced damage to the bedrock must be avoided in the vicinity of existing structures (including buried structures such as the utilities), otherwise additional rock reinforcement could be required. At the final rock line, the bedrock should be line drilled at a close spacing in advance of blasting so that a clean bedrock face can be formed. It is considered that 75 mm diameter holes at a spacing of 200 mm or less would be appropriate for this purpose.

Based on the quality of the bedrock encountered in the boreholes, it is expected that existing near vertical bedrock walls around the existing basement can likely be maintained for the construction period provided that any loose pieces of the bedrock are scaled off the faces for worker safety. Where the localized new excavations extends deeper than 1.8 m into the bedrock, the near vertical walls should be reviewed by a geotechnical engineer for any sign of unstable pillars or slabs that should be removed or stabilized. Stabilization options could consist of rock anchors, mesh, shotcrete, sloping the side slopes or a combination thereof. The appropriate stabilization methodology, if required, will depend on the actual site conditions during construction, and further guidance can be provided at that time.

Where excavations are immediately adjacent to (and below) existing foundations the excavation designer must consider the potential for movement of the excavation walls and potential impacts to existing structures.

Vibration monitoring should be carried out as outlined in Section 6.4.2.

6.4.2 Vibration Monitoring

Due to the close proximity of the existing surrounding structures to the proposed development, construction vibration, (particularly when blasting, breaking rock, driving piles or carrying out other similar vibration intensive works) should be controlled to limit the peak particle velocities at all adjacent structures or services such that vibration induced damage will be avoided.

A pre-construction survey is recommended to be carried out on all nearby structures and services. Any area of concerns should be identified during the pre-construction survey and should be monitored for movements during construction.

If blasting is required, the contractor should be required to submit a complete and detailed blasting design, as well as a monitoring plan prepared by a blasting/vibration specialist before starting blasting. This should be reviewed and accepted in relation to the requirements of the blasting specifications. The contractor should be limited to only small, controlled moves. Peak vibration limits dependent on the following frequencies to the nearest structures and services are suggested.

The following frequency dependent peak vibration limits at the nearest structures and services are typical, but it is suggested they be confirmed by the structural engineer for the particular structure.

Frequency Range (Hz)	Vibration Limits (mm/s)
< 10	5
10 to 40	5 to 50 (sliding scale)
> 40	50

These limits should be practical and achievable on this project. Blasting will likely generate vibrations greater than 40 Hz at the nearest structures. The majority of structures and their components have natural frequencies in the range of 4 to 24 Hz.

These limits are based on reducing the risk of structural damage. These vibration limits will need to be adjusted if there is vibration-sensitive equipment in the vicinity of the new building. Guidelines can be provided; however, it is preferable for equipment manufacturers to provide these limits.

It is recommended that the monitoring of ground vibration intensities (peak ground vibrations and accelerations) from the construction activities (e.g., blasting) be carried out both in the ground adjacent to the closest structures and within or at the structures themselves.

6.5 Groundwater Control

It is understood that two levels of underground garage parking are being considered, which will be located within the footprint of the existing basement. These two levels are assumed to extend about 6.0 m below the existing ground surface (i.e., base elevation of 56.6 m). Accordingly, excavation to these depths will be through surficial fill and sand, into the underlying bedrock in areas outside the footprint of the existing building and parking garage (to be demolished). Based on the groundwater conditions observed in the monitoring wells, excavations will extend below the groundwater level. The rate of groundwater inflow to the excavation will depend on many factors, including: the details of the existing excavation, the exact size of the excavation, and the time of year at

which the excavation is made. Also, there may be instances where precipitation collects in an open excavation and must be rapidly pumped out.

According to O.Reg. 63/16 and O.Reg. 387/04, if the volume of water to be pumped from excavations for the purpose of construction dewatering is greater than 50,000 L/day and less than 400,000 L/day, the water taking will need to be registered as a prescribed activity in the Environmental Activity and Sector Registry (EASR) and has several requirements including the completion of a "Water Taking Plan". Alternatively, a Permit to Take Water (PTTW) is required from the Ministry of the Environment Conservation and Parks (MECP) if a volume of water greater than 400,000 L/day is to be pumped from an excavation.

It is possible that groundwater elevations encountered during construction may be higher than those observed in October 2022, if, for example, construction occurs during the spring. Therefore, groundwater inflow estimates were completed using a groundwater elevation that is 0.5 m higher than the measured groundwater elevations. Incident precipitation could add approximately 132,000 L/day to the underground parking excavation, assuming a footprint of 1,666 m², and assuming a 79.2 mm precipitation event (a 10-year event as observed at the Ottawa Airport weather station).

The Dupuit-Forcheimer analytical solution was used to estimate the potential groundwater inflow into the underground parking excavation using the average hydraulic conductivity measured in the wells. The initial head elevation of the analytical model was assigned a value of 58.3 m (i.e., 0.5 m above the value recorded at monitoring well 22-01). It is assumed that construction dewatering activities would lower the groundwater level to an elevation of 56.1 m (i.e., 0.5 m below the bottom of the excavation). The average bedrock hydraulic conductivity estimated at the monitoring wells was approximately 2x10⁻³ cm/s. The amount of dewatering needed for the excavation is estimated to be between 118,000 (steady-state inflow) and 804,000 (initial inflow) litres per day (L/day). The radius of influence for the excavation is estimated to be approximately 30 m from the edge of the excavation. Groundwater inflow and dewatering radius of influence calculations are included in Appendix E.

Based on the groundwater conditions observed at the site and depending on how the excavation proceeds, water taking exceeding 400,000 L/day may be initially required to dewater groundwater from the excavation. However, with careful management of groundwater pumping rates during the initial stages of opening excavations, it may be possible to keep water taking rates below 400,000 L/day. As a result, the proposed work could be carried out under an EASR registration.

Information regarding the discharge of pumped groundwater is provided in the Phase II ESA report for this project, which is provided under separate cover.

6.6 Frost Protection

All perimeter and exterior foundation elements 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.

It is expected that these requirements will be satisfied for all of the structure footings due to the deep founding levels required to accommodate the below-grade parking.

6.7 Basement Floor/Raft Slab

In preparation for the construction of the basement floor slab, all loose, wet, and disturbed material should be removed from beneath the floor slab. The feasibility of reusing existing underslab granular fill materials can also be evaluated.

Provision should be made for at least 300 mm of 16 mm clear crushed stone to form the base of the floor slab. To prevent hydrostatic pressure build up beneath the floor slab, it is suggested that the granular base for the floor slab be drained. This should be achieved by installing geotextile-wrapped, rigid 100 mm diameter perforated pipes in the floor slab bedding at 6 m centres. The perforated pipes should discharge to a positive outlet such as a storm sewer or a sump from which the water is pumped.

If an asphalt surface will be provided for the basement level, a thickness of at least 150 mm of OPSS Granular A base materials should be provided above the clear stone. The Granular A should be compacted to at least 100% of the material's Standard Proctor Maximum Dry Density (SPMDD).

6.8 Basement Walls

The backfill and drainage requirements for basement walls, as well as the lateral earth pressures will depend on the exact details of the existing excavation and the new basement structure.

The following sections assume that water-tight construction will not be required. If it is determined that water-tight construction is needed, additional design guidelines will be required.

6.8.1 Open Cut Excavations

The soils at this site are frost susceptible and should not be used as backfill against exterior, unheated, or well insulated foundation elements within the depth of potential frost penetration (1.5 m) to avoid problems with frost adhesion and heaving. Free draining backfill materials are also required if hydrostatic water pressure against the basement walls (and potential leakage) is to be avoided. The foundation and basement walls therefore should be backfilled with non-frost susceptible sand or sand and gravel conforming to the requirements for OPSS Granular B Type I.

To avoid ground settlements around the basement walls which could affect site grading and drainage, all of the backfill materials should be placed in 0.3 m thick lifts and compacted to at least 95% of the material's SPMDD.

The basement wall backfill should be drained by means of a perforated pipe subdrain in a surround of 19 mm clear stone, fully wrapped in a geotextile, which leads by positive drainage to a storm sewer or to a sump from which the water is pumped.

6.9 Lateral Earth Pressure for Design

It is considered that two possible design conditions could exist with regards to the lateral earth pressures that will be exerted on the basement walls:

- 1) Walls cast directly against the bedrock face or walls cast against formwork with a narrow, backfilled gallery provided between the basement wall and the adjacent excavation bedrock face.
- 2) Walls cast against formwork with a wide backfilled gallery provided between the basement wall and the adjacent excavation face.

For Case 1, the magnitude of the lateral earth pressure depends on the magnitude of the arching which can develop in the backfill and therefore depends on the width of the backfill, its angle of internal friction, as well as the interface friction angles between the backfill and both the rock face and the basement wall. The magnitude of the lateral earth pressure can be calculated as:

$$\sigma_h(z) = \frac{\gamma B}{2\tan\delta} \left(1 - e^{-2K\frac{Z}{B}\tan\delta} \right) + K q$$

Where: $\sigma_h(z)$ = Lateral earth pressure on the basement wall at depth z, in kPa;

K = Earth pressure coefficient, use 0.6;

- γ = Unit weight of retained soil, use 20 kN/m³ for clear stone chip;
- B = Width of backfill (between basement wall and bedrock face), m;
- δ = Average interface friction angle at backfill-basement wall and backfill-rock face interfaces, use 15°;
- z = Depth below top of formwork, m; and,
- q = Uniform surcharge at ground surface to account for traffic, equipment, or stock piled materials (use 15 kPa).

For Case 2, the basement walls should be designed to resist lateral earth pressures calculated as:

 $\sigma_h(z) = K_o (\gamma z + q)$

Where: $\sigma_h(z)$ = Lateral earth pressure on the wall at depth z, in kPa;

 K_0 = At-rest earth pressure coefficient, use 0.5;

 γ = Unit weight of retained soil, use 22 kN/m³;

z = Depth below top of wall, m; and,

Conventional damp proofing of the basement walls is appropriate with the above design approach. For concrete walls poured against shoring or bedrock, damp proofing using a crystalline barrier such as Crystal Lok, Xypex or equivalent could be used. The use of a concrete additive that provides reduced permeability could also be considered.

For all cases, hydrostatic groundwater pressures would also need to be considered if the structure is designed to be water-tight.

The lateral earth pressures acting on the below-grade walls as a result of seismic events will be highly dependent on the backfill types and methods. For Case 2, the lateral earth pressures noted above would increase under seismic loading conditions. The earthquake-induced dynamic pressure distribution, which is to be added to the static earth pressure distribution, is a linear distribution with maximum pressure at the top of the wall and minimum pressure at its toe (i.e., an inverted triangular pressure distribution).

The combined pressure distribution (static plus seismic) may be determined as follows:

 $\sigma_h(z) = K_o \gamma z + (K_{AE} - K_A) \gamma (H-z);$ non-yielding walls

Where: K_{AE} = The seismic earth pressure coefficient, use 0.42;

K_a = The static active earth pressure coefficient

H = The total depth to the bottom of the foundation wall (m).

For the other backfill design conditions, design lateral pressures resulting from seismic loading should be assessed during the next design stage once further details on building and backfill configuration are available.

Hydrodynamic groundwater pressures would also need to be considered if the structure is designed to be water-tight. However, more sophisticated analyses may need to be carried out at the detailed design stage.

All of the lateral earth pressure equations are given in an unfactored format and will need to be factored for Limit States Design purposes.

It has been assumed that the underground parking levels will be maintained at minimum temperatures but will not be permitted to freeze. If these areas are to be unheated, additional guidelines for the design of the basement walls and foundations will be required.

In areas where pavement or other hard surfacing will abut the building, differential frost heaving could occur between the granular fill immediately adjacent to the building and the more frost susceptible backfill placed beyond the wall backfill. To reduce the severity of this differential heaving, the backfill adjacent to the wall should be placed to form a frost taper. The frost taper should be brought up to pavement subgrade level from 1.5 m below finished exterior grade at a slope of 3 horizontal to 1 vertical, or flatter, away from the wall. The granular fill should be placed in maximum 300 mm thick lifts and should be compacted to at least 95 percent of the material's SPMDD using suitable vibratory compaction equipment.

6.10 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% of the material's SPMDD.

It should generally be possible to re-use the existing inorganic fill, weathered silty clay, sands 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% of the material's SPMDD using suitable vibratory compaction equipment.

7.0 PAVEMENT DESIGN

It is understood new parking lots and access roadway will be constructed as part of the development.

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).

Sections requiring grade raising to the proposed subgrade level should be filled using acceptable (compactable and inorganic) earth borrow (OPSS.MUNI 206/212), Select Subgrade Material (OPSS.MUNI 1010) or additional granular base if grade changes are minor. These materials should be placed in maximum 300 mm thick lifts and should be compacted to at least 98% of the materials SPMDD using suitable compaction equipment.

The surface of the subgrade or fill should be crowned or sloped to promote drainage of the roadway granular structure. Perforated pipe subdrains should be provided along the low sides of the roadway along the entire length. The subdrains should be installed in accordance with OPSS.MUNI 405. The subdrains should be connected to the catch basins such that the pavement structure will be positively drained and will intercept flows within the subbase.

Below the pavement structure, frost compatibility must be maintained across any new service trenches. Due to the variability of the soils within the project limits, the subsoil should be inspected by qualified geotechnical personnel to make sure that there is no potential for differential frost heaving. Frost tapers from the bottom of granular subbase to 1.8 m depth should be constructed at 10H:1V and should be provided where necessary.

The pavement recommendations have been split up into two categories of light duty and heavy-duty pavements. It has been assumed the light duty areas will consist of parking areas and lighter vehicles (i.e., no truck or bus traffic), and the heavy-duty pavements will consist of occasional truck traffic. The pavement in each area should be constructed as follows:

	Motorial	Thickness of Pavement Elements (mm)			
	material	Light Duty	Heavy Duty		
Asphaltic Concrete	Superpave 12.5 mm	40	50		
OPSS.MUNI 1151	Superpave 19.0 mm	50	70		
Granular Material	Granular A Base	150	150		
OPSS.MUNI 1010	Granular B, Type II Subbase	400	500		

The above pavement design is based on the assumption that the pavement subgrade has been acceptably prepared (i.e., where the bottom of the excavation has been adequately compacted to the required density and the subgrade surface is 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. Additionally, a Class II woven geotextile conforming to OPSS 1860 should be provided under pavement areas to prevent pumping of the subgrade into the Granular B Type II subbase.

8.0 IMPACT ON ADJACENTS DEVELOPMENTS

Possible impacts on adjacent developments could result from:

- Ground movement around the perimeter of new excavations.
- Ground settlements due to the planned temporary and permanent groundwater level lowering, if sensitive and compressible clay soils exist within the expected zone of influence of the groundwater level lowering (which, as discussed below, it not the case for this development).

A preconstruction survey of all structures located within close proximity to this site should be carried out prior to commencement of the excavation.

The structures that are mostly at risk of being impacted by ground movements associated with construction of the new building are the portions of the existing structure that are located immediately adjacent to the new structure (e.g., the parkade structure ramps to the south and the single storey building located in the central portion of the site). It is understood that these structures also contain two below-grade levels and are anticipated to be supported on spread footings on bedrock.

As a general guideline for excavation planning, unsupported excavations for the new structure should not come within 0.5 m of the edge of the footings of the existing buildings. To avoid undermining of the rock and/or disturbance of the rock, careful line drilling of the excavation limits in this area must be undertaken.

Given the relatively shallow depth of additional bedrock excavation, no rock reinforcement is anticipated to be required for this excavation. However, the exposed bedrock should be inspected by qualified geotechnical personnel at the time of excavation to confirm that assessment particularly in areas where excavations will be developed in close proximity to existing foundations.

9.0 ADDITIONAL CONSIDERATIONS

At the time of writing this report, only conceptual details related to the proposed building were available. 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.

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.

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.

10.0 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

Golder Associates Ltd.

Lukehht

Arthur Kuitchoua Petke, ing. *Geotechnical Engineer*

AKP/CH/ljv

C. G. HENDRY Allah, 100011328 Nov. 3, 2022 BOUNCE OF ON ARIO Chris Hendry, P.Eng

Senior Geotechnical Engineer

https://golderassociates.sharepoint.com/sites/164073/project files/6 deliverables/geotech/final nov 2022/22530229 rpt rev0 2022'11'03 - holland cross expansion.docx

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IMPORTANT INFORMATION AND LIMITATIONS OF THIS REPORT

Standard of Care: Golder Associates Ltd. (Golder) has prepared this report in a manner consistent with that level of care and skill ordinarily exercised by members of the engineering and science professions currently practicing under similar conditions in the jurisdiction in which the services are provided, subject to the time limits and physical constraints applicable to this report. No other warranty, expressed or implied is made.

Basis and Use of the Report: This report has been prepared for the specific site, design objective, development and purpose described to Golder by the Client, <u>Stantec</u>. The factual data, interpretations and recommendations pertain to a specific project as described in this report and are not applicable to any other project or site location. Any change of site conditions, purpose, development plans or if the project is not initiated within eighteen months of the date of the report may alter the validity of the report. Golder cannot be responsible for use of this report, or portions thereof, unless Golder is requested to review and, if necessary, revise the report.

The information, recommendations and opinions expressed in this report are for the sole benefit of the Client. No other party may use or rely on this report or any portion thereof without Golder's express written consent. If the report was prepared to be included for a specific permit application process, then the client may authorize the use of this report for such purpose by the regulatory agency as an Approved User for the specific and identified purpose of the applicable permit review process, provided this report is not noted to be a draft or preliminary report, and is specifically relevant to the project for which the application is being made. Any other use of this report by others is prohibited and is without responsibility to Golder. The report, all plans, data, drawings and other documents as well as all electronic media prepared by Golder are considered its professional work product and shall remain the copyright property of Golder, who authorizes only the Client and Approved Users to make copies of the report, but only in such quantities as are reasonably necessary for the use of the report or any portion thereof to any other party without the express written permission of Golder. The Client acknowledges that electronic media is susceptible to unauthorized modification, deterioration and incompatibility and therefore the Client cannot rely upon the electronic media versions of Golder's report or other work products.

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.



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APPENDIX A

Borehole Records – Current Investigation (GOLDER/WSP)

Organic or Inorganic	Soil Group	Туре	of Soil	Gradation or Plasticity	Cu	$a = \frac{D_{60}}{D_{10}}$		$Cc = \frac{(D)}{D_{10}}$	$\frac{(30)^2}{xD_{60}}$	Organic Content	USCS Group Symbol	Group Name			
		of is nm)	Gravels with	Poorly Graded		<4		≤1 or ≩	≥3		GP	GRAVEL			
ass)	S 75 mm)	S 75 mm	s 75 mm)	5 75 mm)	VELS y mass raction 1 4.75 r	fines (by mass)	Well Graded		≥4		1 to 3	3		GW	GRAVEL
by me	SOILS an 0.07	GRA 50% by oarse f	GRAV GRAV 50% by aarse fr min than 13%	Below A Line			n/a				GM	SILTY GRAVEL			
GANIC nt ≤30%	AINED arger th		fines (by mass)	Above A Line			n/a			≤30%	GC	CLAYEY GRAVEL			
INOR	SE-GR ass is la	tSE-GR ass is la	SE-GR iss is la	s of mm)	Sands with ≤12%	Poorly Graded		<6		≤1 or 2	≥3		SP	SAND	
Irganic	COAR 6 by m∈	NDS y mass fractior an 4.75	fines (by mass)	Well Graded		≥6		1 to 3	3	4	SW	SAND			
Ŋ	(>50%	SA ≥50% b coarse ⊺	Sands with >12%	Below A Line			n/a			4	SM	SILTY SAND			
		s (i	fines (by mass)	Above A Line			n/a				SC	CLAYEY SAND			
Organic	Soil	Turne	- 6 0 - 11	Laboratory			Field Indica	ators	Toughness	Organic	USCS Group	Primary			
or Inorganic	Group	туре	of Soli	Tests	Dilatancy	Dry Strength	Shine Test	Thread Diameter	(of 3 mm thread)	Content	Symbol	Name			
		not Pot	5	Liquid Limit	Rapid	None	None	>6 mm	N/A (can't roll 3 mm thread)	<5%	ML	SILT			
(ss)	75 mm	and L	Line icity ilow)	<50	Slow	None to Low	Dull	3mm to 6 mm	None to low	<5%	ML	CLAYEY SILT			
by ma	OILS an 0.0	SILTS tic or P	n Plasti nart be		Slow to very slow	Low to medium	Dull to slight	3mm to 6 mm	Low	5% to 30%	OL	ORGANIC SILT			
3ANIC It ≤30%	NED S	-Plas	ig o p	Liquid Limit	Slow to very slow	Low to medium	Slight	3mm to 6 mm	Low to medium	<5%	МН	CLAYEY SILT			
INORC	:-GRAII	Ň	2	≥50	None	Medium to high	Dull to slight	1 mm to 3 mm	Medium to high	5% to 30%	ОН	ORGANIC SILT			
ganic (FINE by mas		e on Jart	Liquid Limit <30	None	Low to medium	Slight to shiny	~ 3 mm	Low to medium	0%	CL	SILTY CLAY			
Ō	(≥50%	CLAYS	below)	Liquid Limit 30 to 50	None	Medium to high	Slight to shiny	1 mm to 3 mm	Medium	30%	CI	SILTY CLAY			
			Plast Plast	Liquid Limit ≥50	None	High	Shiny	<1 mm	High	(see Note 2)	СН	CLAY			
≻ S s f	11C *30% šs)	Peat and mix	mineral soil dures							30% to 75%		SILTY PEAT, SANDY PEAT			
HIGHL SOIL	(Organ intent > by mas	Predomin may con	nantly peat, ntain some							75%	PT	DEAT			
10	රි	mineral so amorph	vil, fibrous or 1ous peat							100%		PEAI			
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1.5						ALL TON		a hypnen,	for example,	GP-GIVI, a	SW-SC and CL				
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					/			gravel.				,			
(Id) x				SILTY CLAY	CLAYEY S	ILT MH		For cohes	ive soils, the	dual symb	ool must be us	ed when the			
pul 20				/	A Barrenser			liquid limit	and plasticity	y index val	ues plot in the	CL-ML area			
Plastici				Pine				of the plas	ticity chart (s	ee Plastici	ty Chart at left	t).			
		SILTY CL	LAY	/				Borderlin	e Symbol —	· A borderl	ine symbol is	two symbols			
10			0	LAYEY SILT ML				separated	by a slash, fo	or example	≥, CL/CI, GM/S	3M, CL/ML.			
5	ILTY CLAY-CLAY	IEY SILT, CL-ML	0	IGANIC SILT OL				has been	identified as	s having r	properties that	t are on the			
	SILT ML	See Note 1)						transition I	between simi	lar materia	ls. In addition	, a borderline			
0	10	ξά	25.5 30	40 S	03 04	70	80	symbol ma	ay be used to	indicate a	a range of simi	lar soil types			

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

Liquid Limit (LL) 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.

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>i.e.</i> , 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.) r equired 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); Nd:

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

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			

NON-COHESIVE (COHESIONLESS) SOILS

- 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.

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
ТО	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∟	liquid limit
С	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
Μ	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.

	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	

 SPT 'N' in accordance with ASTM D1586, uncorrected for overburden pressure effects; approximate only.

 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.

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

π 3.1416w or LLliquid limitIn xnatural logarithm of xw or LLliquid limitlogacceleration due to gravityw or LLliquid limitgacceleration due to gravityplastic limitplastic limitgacceleration due to gravityw or LLliquid limitttimew or LLliquid limitgacceleration due to gravitynon-plasticttimew or LLliquid limitliquid ty device(w - w_p) / lpgshear strain(m or liquid limit Δ change in, e.g. in stress: $\Delta \sigma$ hhhydraulic lossest statelemwlinear strainqrate of flow α volumetric strainqncoefficient of viscosityihhydraulic propertiesncoefficient of viscosityinhydraulic gradient α colarity explanation α volumetric strain α coefficient of volume α strainatin	I.	GENERAL	(a)	Index Properties (continued)
In X log no (bg x, log x,	π	3.1416	w _i or LL	liquid limit
	ln x	natural logarithm of x	w₀ or PL	plastic limit
$ \begin{array}{cccc} g & \operatorname{acceleration due to gravity} & NP & \operatorname{non-plastic} & \operatorname{init} kge[\operatorname{init} & _{h} & _{$	log ₁₀	x or log x, logarithm of x to base 10	Ip or PI	plasticity index = (wլ – wբ)
t time was shrinkage limit liquidiy index = $(w - w_0) / l_0$ consistency index = $(w - w_0) / l_0$ (conset state u index = $(w - w_0) / l_0$ consistency index = $(w - w_0) / l_0$ consistency index = $(w - w_0) / l_0$ (conset state w void ratio in closes state u index = $(w - w_0) / l_0$ (constitution index = $(w - w_0) / l_0$ (constitution index index = $(w - w_0) / l_0$ (constitution index index = $(w - w_0) / l_0$ (constitution index index = $(w - w_0) / l_0$ (constitution index index = $(w - w_0) / l_0$ (constitution index index = $(w - w_0) / l_0$ (constitution index index = $(w - w_0) / l_0$ (constitution index index = $(w - w_0) / l_0$ (constitution index index = $(w - w_0) / l_0$ (constitution index index = $(w - w_0) / l_0$ (constitution index index = $(w - w_0) / l_0$ (constitution index index = $(w - w_0) / l_0$ (constitution index index = $(w - w_0) / l_0$ (constitution index index = $(w - w_0) / l_0$ (constitution index index = $(w - w_0) / l_0$ (constitution index index = $(w - w_0) / l_0$ (constitution index = $(w - w_0) / l_0$ (constit	g	acceleration due to gravity	NP	non-plastic
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τSite a stress(b) (b) (b) (b) (b) (b) (b) (b) (b) (b)		$= (\sigma_1 + \sigma_2 + \sigma_3)/3$	Cr	recompression index
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$ \begin{array}{c} \textbf{G} & \textbf{inductor of deformation} \\ \textbf{K} & \textbf{bulk modulus of compressibility} \\ \textbf{K} & \textbf{coefficient of consolidation (horizontal direction) \\ \textbf{M} & \textbf{coefficient of consolidation preconsolidation stress} \\ \textbf{CR} & \textbf{correscolidation ratio = \sigma'_p / \sigma'_{vo} \\ \textbf{CR} & \textbf{correscolidation ratio = \sigma'_p / \sigma'_{vo} \\ \textbf{CR} & \textbf{correscolidation ratio = \sigma'_p / \sigma'_{vo} \\ \textbf{CR} & \textbf{correscolidation ratio = \sigma'_p / \sigma'_{vo} \\ \textbf{CR} & \textbf{correscolidation ratio = \sigma'_p / \sigma'_{vo} \\ \textbf{CR} & \textbf{correscolidation ratio = \sigma'_p / \sigma'_{vo} \\ \textbf{CR} & \textbf{correscolidation ratio = \sigma'_p / \sigma'_{vo} \\ \textbf{CR} & \textbf{correscolidation ratio = \sigma'_p / \sigma'_{vo} \\ \textbf{CR} & \textbf{correscolidation ratio = \sigma'_p / \sigma'_{vo} \\ \textbf{CR} & \textbf{correscolidation ratio = \sigma'_p / \sigma'_{vo} \\ \textbf{CR} & \textbf{correscolidation ratio = \sigma'_p / \sigma'_{vo} \\ \textbf{CR} & \textbf{correscolidation ratio = an \delta} \\ \textbf{CR} & \textbf{correscolidation ratio = tan \delta} \\ \textbf{CR} & \textbf{correscolidation ratio = tan \delta} \\ \textbf{CR} & \textbf{correscolidation ratio = tan \delta} \\ \textbf{CR} & \textbf{correscolidation ratio stress} \\ \textbf{CR} & \textbf{correscolidation ratio = an \delta} \\ \textbf{CR} & \textbf{correscolidation ratio = an \delta} \\ \textbf{CR} & \textbf{correscolidation ratio stress} \\ \textbf{CR} & correscolidation ratio str$	u F	modulus of deformation	Cs Ca	secondary compression index
Kbulk modulus of compressibilitycvcoefficientof consolidation(vertical direction)III.SOIL PROPERTIEScoefficientof consolidation(horizontal direction)III.SOIL PROPERTIESUdegree of consolidation(horizontal direction)III.SOIL PROPERTIESUdegree of consolidation(horizontal direction)III.SOIL PROPERTIESUdegree of consolidation stress(vertical direction)(a)Index PropertiesOCRover-consolidation ratio = σ'_p / σ'_{vo} $\rho(\gamma)$ bulk density (bulk unit weight)*(d)Shear Strength $\rho_{m}(\gamma_w)$ density (unit weight) of solid particles ϕ' effective angle of internal friction γ' unit weight of submerged soil δ angle of interface friction $(\gamma' = \gamma - \gamma_w)$ μ coefficient of friction = tan δ DRrelative density (specific gravity) of solidc'effective cohesion $particles (DR = \rho_s / \rho_w) (formerly G_s)cu, suundrained shear strength (\phi = 0 analysis)evoid ratiopmean total stress (\sigma_1 + \sigma_3)/2nporosityp'mean effective stress (\sigma_1 + \sigma_3)/2sdegree of saturationq(\sigma_1 - \sigma_3)/2 or (\sigma_1 - \sigma_3)/2*Density symbol is \rho. Unit weight symbol is \gammaNotes: 1\tau = c' + \sigma' \tan \phi'*shear strength = (compressive strength)/2shear strength = (compressive strength)/2$	G	shear modulus of deformation	m _v	coefficient of volume change
III.SOIL PROPERTIESdirection) coefficient of consolidation (horizontal direction)III.SOIL PROPERTIESTvtime factor (vertical direction)(a)Index PropertiesOCRover-consolidation ratio = σ'_p / σ'_{vo} $p(\gamma)$ bulk density (bulk unit weight)* $pd(yd)$ (d)Shear Strength effective angle of internal friction σ'_p $p(\gamma)$ bulk density (bulk unit weight)(d)Shear Strength σ'_p $p(\gamma)$ bulk density (unit weight) of solid particles 	ĸ	bulk modulus of compressibility	Cv	coefficient of consolidation (vertical
III.SOIL PROPERTIESChcoefficient of consolidation (horizontal direction)III.SOIL PROPERTIESTvtime factor (vertical direction)(a)Index PropertiesUdegree of consolidation stress $\rho(\gamma)$ bulk density (bulk unit weight)*OCRover-consolidation ratio = σ'_p / σ'_{vo} $\rho(\gamma)$ bulk density (unit weight) of water τ_p, τ_r peak and residual shear strength $\rho(\gamma)$ density (unit weight) of solid particles ϕ' effective angle of internal friction γ' unit weight of submerged soil δ angle of internae friction $(\gamma' = \gamma - \gamma_w)$ μ coefficient of friction = tan δ DRrelative density (specific gravity) of solid c' effective cohesion $\rho(\tau)$ porosity p' mean total stress $(\sigma_1 + \sigma_3)/2$ nporosity p' mean effective stress $(\sigma'_1 + \sigma'_3)/2$ q' unit weight is ρ . Unit weight symbol is γ Notes: 1*Density symbol is ρ . Unit weight symbol is γ $\tau = c' + \sigma' \tan \phi'$ *Density symbol is ρ . Unit weight symbol is γ verters: 1* $\tau = c' + \sigma' \tan \phi'$ shear strength = (compressive strength)/2				direction)
III.SOIL PROPERTIESTvtime factor (vertical direction)(a)Index PropertiesOCRover-consolidation stress $\rho(\gamma)$ bulk density (bulk unit weight)*OCRover-consolidation ratio = σ'_p / σ'_{vo} $\rho(\gamma)$ bulk density (unit weight)(d)Shear Strength $\rho(\gamma_w)$ density (unit weight) of water τ_p, τ_r peak and residual shear strength $\rho(\gamma)$ density (unit weight) of solid particles ϕ' effective angle of internal friction γ' unit weight of submerged soil δ angle of internal friction = tan δ γ' unit weight (specific gravity) of solid c' effective cohesion $(\gamma' = \gamma - \gamma_w)$ μ coefficient of friction = tan δ D_R relative density (specific gravity) of solid c' effective cohesion e void ratio p' mean total stress ($\sigma_1 + \sigma_3$)/2 n porosity p' mean effective stress ($\sigma'_1 + \sigma'_3$)/2 S degree of saturation q $(\sigma_1 - \sigma_3)/2$ or $(\sigma'_1 - \sigma'_3)/2$ χ where $\gamma = \rho g$ (i.e. mass density multiplied by acceleration due to acravity) γ acceleration ϕ' *Density symbol is ρ . Unit weight symbol is γ where $\gamma = \rho g$ (i.e. mass density multiplied by acceleration due to acravity) $\tau = c' + \sigma' \tan \phi'$ *Density symbol is ρ . Unit weight symbol is γ where $\gamma = \rho g$ (i.e. mass density multiplied by acceleration due to acravity) $\tau = c' + \sigma' \tan \phi'$ *Density symbol is ρ . Unit weight symbol is γ where $\gamma = \rho g$ (i.e. mass density			Ch	coefficient of consolidation (horizontal
III.SOIL PROPERTIESUdegree of consolidation(a)Index PropertiesOCRover-consolidation stress $\rho(\gamma)$ bulk density (bulk unit weight)*OCRover-consolidation ratio = σ'_p / σ'_{vo} $\rho(\gamma)$ bulk density (dry unit weight)(d)Shear Strength $\rho(\gamma_w)$ density (unit weight) of vater τ_p, τ_r peak and residual shear strength $\rho(\gamma)$ density (unit weight) of solid particles ϕ' effective angle of internal friction γ' unit weight of submerged soil δ angle of interface friction $(\gamma' = \gamma - \gamma_w)$ μ coefficient of friction = tan δ D_R relative density (specific gravity) of solid c' effective cohesion $particles (D_R = \rho_s / \rho_w)$ (formerly G_s) c_u, s_u undrained shear strength ($\phi = 0$ analysis) e void ratio p' mean total stress ($\sigma_1 + \sigma_3$)/2 n porosity p' mean effective stress ($\sigma'_1 + \sigma'_3$)/2 S degree of saturation q $(\sigma_1 - \sigma_3)/2$ q_u compressive strength ($\sigma_1 - \sigma_3$) s_t sensitivity			т.,	time factor (vertical direction)
Index PropertiesDescriptionDesc	Ш.	SOIL PROPERTIES	Ŭ	degree of consolidation
(a)Index PropertiesOCRover-consolidation ratio = σ'_p / σ'_{vo} $\rho(\gamma)$ bulk density (bulk unit weight)*over-consolidation ratio = σ'_p / σ'_{vo} $\rho(\gamma)$ dry density (dry unit weight)(d)Shear Strength $\rho_w(\gamma_w)$ density (unit weight) of water τ_p, τ_r peak and residual shear strength $\rho_s(\gamma_s)$ density (unit weight) of solid particles ϕ' effective angle of internal friction γ' unit weight of submerged soil δ angle of interface friction $(\gamma' = \gamma - \gamma_w)$ μ coefficient of friction = tan δ D_R relative density (specific gravity) of solid c' effective cohesionparticles ($D_R = \rho_s / \rho_w$) (formerly G_s) c_u, s_u undrained shear strength ($\phi = 0$ analysis)evoid ratiopmean total stress ($\sigma_1 + \sigma_3$)/2nporosityp'mean effective stress ($\sigma'_1 + \sigma'_3$)/2Sdegree of saturationq($\sigma_1 - \sigma_3$)/2 or ($\sigma'_1 - \sigma'_3$)/2*Density symbol is ρ . Unit weight symbol is γ Notes: 1 $\tau = c' + \sigma' \tan \phi'$ *where $\gamma = \rho g$ (i.e. mass density multiplied by2shear strength = (compressive strength)/2			σ'ρ	pre-consolidation stress
$ \begin{array}{cccc} \rho(\gamma) & \text{bulk density (bulk unit weight)}^{*} \\ \rho_{d}(\gamma_{d}) & dry density (dry unit weight) & \textbf{(d)} \\ \rho_{w}(\gamma_{w}) & \text{density (unit weight) of water} \\ \rho_{s}(\gamma_{s}) & \text{density (unit weight) of solid particles} \\ \gamma' & \text{unit weight of submerged soil} \\ \gamma' & \text{unit weight of submerged soil} \\ \gamma' & \text{unit weight of submerged soil} \\ \rho_{r}(\gamma = \gamma - \gamma_{w}) \\ \rho_{R} & \text{relative density (specific gravity) of solid} \\ particles (D_{R} = \rho_{s} / \rho_{w}) (formerly G_{s}) \\ e & \text{void ratio} \\ n & \text{porosity} \\ S & \text{degree of saturation} \\ \gamma' & \text{Density symbol is } \rho. Unit weight symbol is } \gamma \\ \gamma' & \text{where } \gamma = \rhog (i.e. mass density multiplied by \\ \gamma = \rho $	(a)	Index Properties	OCR	over-consolidation ratio = σ'_p / σ'_{vo}
$\begin{array}{cccc} \rho_{d}(\gamma_{d}) & dry density (dry unit weight) & (d) & Shear Strength \\ \rho_{w}(\gamma_{w}) & density (unit weight) of water & \tau_{p}, \tau_{r} & peak and residual shear strength \\ \rho_{s}(\gamma_{s}) & density (unit weight) of solid particles & \phi' & effective angle of internal friction \\ \gamma' & unit weight of submerged soil & \delta & angle of interface friction \\ (\gamma' = \gamma - \gamma_{w}) & \mu & coefficient of friction = tan \delta \\ particles (D_{R} = \rho_{s} / \rho_{w}) (formerly G_{s}) & c_{u}, s_{u} & undrained shear strength (\phi = 0 analysis) \\ e & void ratio & p & mean total stress (\sigma_{1} + \sigma_{3})/2 \\ n & porosity & p' & mean effective stress (\sigma'_{1} + \sigma'_{3})/2 \\ S & degree of saturation & q & (\sigma_{1} - \sigma_{3})/2 or (\sigma'_{1} - \sigma'_{3})/2 \\ q_{u} & compressive strength (\sigma_{1} - \sigma_{3}) \\ sensitivity \\ * & Density symbol is \rho. Unit weight symbol is \gamma \\ where \gamma = \rho g (i.e. mass density multiplied by \\ acceleration due to dravity) \end{pmatrix} $	ρ(γ)	bulk density (bulk unit weight)*		
$p_w(\gamma w)$ density (unit weight) of water τ_{p}, τ_r peak and residual shear strength $\rho_s(\gamma_s)$ density (unit weight) of solid particles ϕ' effective angle of internal friction γ' unit weight of submerged soil δ angle of interface friction $(\gamma' = \gamma - \gamma w)$ μ coefficient of friction = tan δ D_R relative density (specific gravity) of solid particles ($D_R = \rho_s / \rho_w$) (formerly G_s) c_u, s_u undrained shear strength ($\phi = 0$ analysis)evoid ratiopmean total stress ($\sigma_1 + \sigma_3$)/2mean effective stress ($\sigma'_1 + \sigma'_3$)/2nporosityp'mean effective stress ($\sigma'_1 - \sigma'_3$)/2Sdegree of saturationq($\sigma_1 - \sigma_3$)/2 or ($\sigma'_1 - \sigma'_3$)/2*Density symbol is ρ . Unit weight symbol is γ Notes: 1 $\tau = c' + \sigma' \tan \phi'$ *Density symbol is ρ . Unit weight multiplied by2*Density of use to gravity)specified by2	ρα(γα)	dry density (dry unit weight)	(d)	Shear Strength
$\begin{array}{cccc} \rho_{s}(\gamma_{s}) & \text{density (unit weight) of solid particles} & \phi' & \text{effective angle of internal friction} \\ \gamma' & \text{unit weight of submerged soil} & \delta & \text{angle of internal friction} \\ (\gamma' = \gamma - \gamma_{w}) & \mu & \text{coefficient of friction} = \tan \delta \\ D_{R} & \text{relative density (specific gravity) of solid} & c' & \text{effective cohesion} \\ particles (D_{R} = \rho_{s} / \rho_{w}) (formerly G_{s}) & C_{u}, s_{u} & \text{undrained shear strength } (\phi = 0 \text{ analysis}) \\ e & \text{void ratio} & p & \text{mean total stress } (\sigma_{1} + \sigma_{3})/2 \\ n & \text{porosity} & p' & \text{mean effective stress } (\sigma_{1} + \sigma_{3})/2 \\ S & \text{degree of saturation} & q & (\sigma_{1} - \sigma_{3})/2 \text{ or } (\sigma_{1} - \sigma_{3})/2 \\ q_{u} & \text{compressive strength } (\sigma_{1} - \sigma_{3}) \\ s_{t} & \text{sensitivity} \end{array}$ $\begin{array}{c} \star & \text{Density symbol is } \rho. \text{ Unit weight symbol is } \gamma \\ \text{where } \gamma = \rho g & (i.e. \text{ mass density multiplied by} & 2 \end{array}$	ρω(γω)	density (unit weight) of water	τp, τr	peak and residual shear strength
γ' unit weight of submerged soil δ angle of interface friction $(\gamma' = \gamma - \gamma_w)$ μ coefficient of friction = tan δ D_R relative density (specific gravity) of solid c' effective cohesionparticles $(D_R = \rho_s / \rho_w)$ (formerly G_s) c_u , s_u undrained shear strength ($\phi = 0$ analysis)evoid ratio p mean total stress ($\sigma_1 + \sigma_3$)/2nporosity p' mean effective stress ($\sigma'_1 + \sigma'_3$)/2Sdegree of saturation q ($\sigma_1 - \sigma_3$)/2 or ($\sigma'_1 - \sigma'_3$)/2*Density symbol is ρ . Unit weight symbol is γ Notes: 1*Density symbol is ρ . Unit weight symbol is γ $r = c' + \sigma' \tan \phi'$ *shear strength (ϕ compressive strength)/2	ρs(γs)	density (unit weight) of solid particles	<u>ه</u> ′	effective angle of internal friction
$\begin{array}{cccc} (\gamma' = \gamma - \gamma_w) & \mu & \text{coefficient of friction} = \tan \delta \\ particles (D_R = \rho_s / \rho_w) (formerly G_s) & c_u, s_u & \text{undrained shear strength } (\phi = 0 \text{ analysis}) \\ e & \text{void ratio} & p & \text{mean total stress } (\sigma_1 + \sigma_3)/2 \\ n & \text{porosity} & p' & \text{mean effective stress } (\sigma'_1 + \sigma'_3)/2 \\ S & \text{degree of saturation} & q & (\sigma_1 - \sigma_3)/2 \text{ or } (\sigma'_1 - \sigma'_3)/2 \\ q_u & \text{compressive strength } (\sigma_1 - \sigma_3) \\ S_t & \text{sensitivity} \end{array}$ * Density symbol is ρ . Unit weight symbol is γ Notes: 1 $\tau = c' + \sigma' \tan \phi'$ shear strength = (compressive strength)/2	γ′	unit weight of submerged soil	δ	angle of interface friction
DRrelative density (specific gravity) of solid particles (DR = ρ_s / ρ_w) (formerly Gs)c'effective conesion undrained shear strength ($\phi = 0$ analysis)evoid ratiopmean total stress ($\sigma_1 + \sigma_3$)/2nporosityp'mean effective stress ($\sigma_1 + \sigma_3$)/2Sdegree of saturationq($\sigma_1 - \sigma_3$)/2 or ($\sigma_1 - \sigma_3$)/2*Density symbol is ρ . Unit weight symbol is γ Notes: 1 $\tau = c' + \sigma' \tan \phi'$ *the mass density multiplied by2shear strength = (compressive strength)/2	-	$(\gamma' = \gamma - \gamma_w)$	μ	coefficient of friction = tan δ
$\begin{array}{c c} & \text{particles } (D_{R} = \rho_{s} / \rho_{w}) \text{ (formerly G_{s})} & C_{u}, s_{u} & \text{undrained shear strength } (\phi = 0 \text{ analysis}) \\ e & \text{void ratio} & p & \text{mean total stress } (\sigma_{1} + \sigma_{3})/2 \\ n & \text{porosity} & p' & \text{mean effective stress } (\sigma_{1} + \sigma_{3})/2 \\ S & \text{degree of saturation} & q & (\sigma_{1} - \sigma_{3})/2 \text{ or } (\sigma_{1} - \sigma_{3})/2 \\ q_{u} & \text{compressive strength } (\sigma_{1} - \sigma_{3}) \\ S_{t} & \text{sensitivity} \end{array}$ * Density symbol is ρ . Unit weight symbol is γ Notes: 1 $\tau = c' + \sigma' \tan \phi'$ shear strength = (compressive strength)/2	DR	relative density (specific gravity) of solid	C'	effective conesion
evoid failspmean total stress $(\sigma_1 + \sigma_3)/2$ nporosityp'mean effective stress $(\sigma_1 + \sigma_3)/2$ Sdegree of saturationq $(\sigma_1 - \sigma_3)/2$ or $(\sigma_1 - \sigma_3)/2$ Qucompressive strength $(\sigma_1 - \sigma_3)$ Stsensitivity*Density symbol is ρ . Unit weight symbol is γ Notes: 1where $\gamma = \rho g$ (i.e. mass density multiplied by2seceleration due to gravity)2		particles ($D_R = \rho_s / \rho_w$) (formerly G_s)	Cu, Su	undrained shear strength ($\phi = 0$ analysis)
* Density symbol is ρ . Unit weight symbol is γ where $\gamma = \rho g$ (i.e. mass density multiplied by 2 $compressive strength (\sigma_1 - \sigma_3)/2$ $\tau = c' + \sigma' tan \phi' shear strength = (compressive strength)/2$	e n	norosity	Р Р	mean iolar stress $(\sigma_1 + \sigma_3)/Z$
* Density symbol is ρ . Unit weight symbol is γ Notes: 1 where $\gamma = \rho g$ (i.e. mass density multiplied by 2 second strength = (compressive strength)/2 shear strength = (compressive strength)/2	S	degree of saturation	h.	$\frac{1}{(\sigma_1 - \sigma_2)/2} \text{ or } (\sigma_1 + \sigma_2)/2$
* Density symbol is ρ . Unit weight symbol is γ Notes: 1 where $\gamma = \rho g$ (i.e. mass density multiplied by 2 shear strength = (compressive strength)/2	0		Ч С.,	(01 - 03)/2 or $(0.1 - 0.3)/2$
* Density symbol is ρ . Unit weight symbol is γ Notes: 1 $\tau = c' + \sigma' \tan \phi'$ where $\gamma = \rho g$ (i.e. mass density multiplied by 2 shear strength = (compressive strength)/2			St St	sensitivity
* Density symbol is ρ . Unit weight symbol is γ Notes: 1 $\tau = c' + \sigma' \tan \phi'$ where $\gamma = \rho g$ (i.e. mass density multiplied by 2 shear strength = (compressive strength)/2			•• •	-
where $\gamma = \rho g$ (i.e. mass density multiplied by 2 shear strength = (compressive strength)/2	* Densi	ity symbol is ρ . Unit weight symbol is γ	Notes: 1	$\tau = c' + \sigma' \tan \phi'$
	accel	$rac{1}{r} \gamma - \rho g$ (i.e. mass density multiplied by eration due to gravity)	2	snear strength – (compressive strength)/2

PROJECT:	22530229 (3000)
LOCATION:	N 442688.50; E 5028045.00

S:CLIENTS!LASALLE_INVESTMENT_MANAGEMENT1560_SCOTT_ST_OTTAWa\02_DATA\GINT\1560_SCOTT_ST_OTTAWA.GPJ_GAL-MIS.GDT 11/2/22

GTA-BHS 001

BORING DATE: August 29, 2022

SHEET 1 OF 2

DATUM: Geodetic

HAMMER TYPE: AUTOMATIC

SPT/DCPT HAMMER: MASS, 64kg; DROP, 760mm

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m HYDRAULIC CONDUCTIVITY, k, cm/s SAMPLES SOIL PROFILE DEPTH SCALE METRES BORING METHOD ADDITIONAL LAB. TESTING PIEZOMETER STRATA PLOT 40 60 80 10⁻⁶ 10⁻⁵ 10-4 10⁻³ OR BLOWS/0.3m 20 NUMBER STANDPIPE ELEV. TYPE SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - O WATER CONTENT PERCENT DESCRIPTION INSTALLATION DEPTH OW - WI Wp (m) 10 20 40 60 80 20 30 40 GROUND SURFACE 61.60 0 ASPHALTIC CONCRETE (100 mm) 0.00 Concrete 4 Þ FILL - (SM) SILTY SAND, some gravel, contain rock fragments; dark brown; non-cohesive, moist GS 1 60.84 (GP) GRAVEL, some sand; grey (TILL); non-cohesive, dry, compact to very 0.76 Bentonite 2 SS 27 dense 60.23 Weathered bedrock, possible cobbles 1.37 60.00 SS 55 and boulders 3 1.60 END OF BOREHOLE/DRILLHOLE Notes: 2 1. Auger refusal. 3 4 5 6 7 8 9 10 **\\\)** GOLDER DEPTH SCALE LOGGED: PK 1:50 CHECKED: AKP

	PR	OJEC	T: 22530229 (3000)	I	REC	OR	DC)F	D	RI	LL	H	OL	.E:		В	H	22-	01								SH	EET 2	OF 2	
	INC	CATIC	dn: N 442688.5 ;E 5028045.0 Tion: -90° Azimuth:								NG I RIG:				∩R∙	Ma	rath	ion Dri	lling								DA	TUM:	Geodetic	
1.000.000	METRES	ING RECORD	DESCRIPTION	ABOLIC LOG	ELEV. DEPTH	RUN No.	4 COLOUR & RETURN	JN FLT SHI VN CJ RE	- Join - Fau R- She - Vei - Con	nt ult ear njuga ERY	ite R.(BD FO CO OR CL 2.D.	- Bed - Folia - Con - Orth - Clea FRAC	ding ation tact iogon avage	al	PL CL UN ST IR DI	- Pla J- CL N- Ur F - Sta SCC	anar urved ndulating epped egular	P K S R M TY DA1	D- Poli - Slic M- Sm D- Rou B- Med A	shed kensio ooth igh chanic	al Bre	ak s	BR - NOTE: abbrevi of abbr symbol ULIC TIVITY	Brok For ac ations eviations s. Diarr Point	en Ro dditiona refer t ns & netral Load	ock al to list RMC			
Ĺ	DE	DRILI		SYN	(11)		FLUSH	COR 883	AL : 8% C 8% &	SOLIE ORE 8848	2 8 8 8 8 8 8 8 8 8 8	% 898	PEF 0.25 	₹ B m 22 0	Angle	CC A) 0,00		TYPE AN DESI	ID SURF	ACE J	r Ja Ji	10 [°]	, cm/:		Inc (MI	dex Pa) t o	-Q' AVG.			
	2		Fresh fine-medium grained, slightly porous to non-porous, grey LIMESTONE bedrock with thin Shale interbeds		60.00	1																						Sand		e, ve, ve, ve
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	DE 1::	PTH S	SCALE								G () 		F	5									LC	GGED:	PK AKP	

PROJECT:	22530229 (3000)
LOCATION:	N 442665.40; E 5028039.00

GTA-BHS 001

RECORD OF BOREHOLE: BH22-02B

SHEET 1 OF 2 DATUM: Geodetic

BORING DATE: August 29, 2022

HAMMER TYPE: AUTOMATIC

SPT/DCPT HAMMER: MASS, 64kg; DROP, 760mm DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m HYDRAULIC CONDUCTIVITY, k, cm/s SAMPLES SOIL PROFILE DEPTH SCALE METRES BORING METHOD ADDITIONAL LAB. TESTING PIEZOMETER STRATA PLOT 40 60 80 10⁻⁶ 10⁻⁵ 10-4 10⁻³ OR BLOWS/0.3m 20 NUMBER STANDPIPE ELEV. TYPE SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - O WATER CONTENT PERCENT DESCRIPTION INSTALLATION DEPTH OW - WI WpH (m) 10 40 60 80 20 30 40 20 GROUND SURFACE 61.72 0 ASPHALTIC CONCRETE (150 mm) 0.00 Concrete 4 Þ FILL - (SM) SILTY SAND, (Granular B); 0.15 dark brown; non-cohesive, very dense, moist 0 1 GS 60.96 (SM) SILTY SAND, some gravel; brown to grey (TILL); dry, very dense 0.76 Bentonite 2 SS 55 0 ٩¢ 60.20 Weathered bedrock, possible cobbles and boulders 1.52 59.87 3 SS 50 b END OF BOREHOLE/DRILLHOLE 1.85 2 Notes: 1. Auger refusal. S:CLIENTS!LASALLE_INVESTMENT_MANAGEMENT1560_SCOTT_ST_OTTAWA\02_DATA\GINT\1560_SCOTT_ST_OTTAWA.GPJ_GAL-MIS.GDT 11/2/22 3 4 5 6 7 8 9 10 **NSD** GOLDER DEPTH SCALE LOGGED: PK 1:50 CHECKED: AKP

PR LO	OJ CA	ECT	 22530229 (3000) N: N 442665.4 :E 5028039.0 	R	ECC	DRI	DO)F	D				HC DAT		E:		В	вH	2	2-02	2B									s		'2(1:G	DF 2 Geode	tic		
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EPTH SCALE METRES		ררואפ אברטאט	DESCRIPTION	MBOLIC LOG	ELEV. DEPTH (m)	RUN No.	COLOUR	T 0 0 0 T 0	IN - ELT - SHR- /N - CJ - REC	Join Fau She Veir Con	it ar ijugat ERY iOLID	R.G	BE FC OF CL	D - Bec D - Foli D - Cor R - Orti - Cle FRAC INDE	Iding ation ntact nogo avag CT.	nal e	In D	PL - CU- UN- ST - IR - DIS	Plai Cur Und Ste Irre	nar rved dulating pped gular NTINUITY	PC K SN Ro ME DAT)- Po - Sli 1- Sn - Ro 3- Me A	lishe cken nooth ough echa	ed iside n nical	d Bre HYE CONE	ak solution	BR - NOTE: abbrev of abbr symbol JLIC TIVIT sec	- Bro : For a reviati ls. Poir	ken l additio s refe ions & metra nt Loa ndex	Rock onal r to list						
			Continued from Record of Borehole 22-02B	S	50.07		ELUS		0RE 9	% C0 3 8	DRE %	6 1 88	11	0.25 ₅ 24	m 22 23	- <u>85</u>	220	AXIS	66	DESCRI	PTION	ACE	Jr Ja	i Jn	100	2 2 2 2 2	9	4) ~	MPa)	AVG						
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DE	PT	нs	CALE								 G	 ; (0		 . [D			R	2										L	OGGI	ED:	PK			

	PR	ROJEC	CT: 22530229 (3000)	ł	REC	OR	DO	of B	ORE	HO	LE:	B⊦	122-()3				SI	HEET 1 OF 2
	LO	CATI	JN: N 4427 16.10; E 50280 19.00				BOF	ring da	TE: Au	gust 30,	2022							D	ATUM: Geodetic
	SP	T/DC	PT HAMMER: MASS, 64kg; DROP, 760mm	I				-				<u> </u>					HAM	MER T	YPE: AUTOMATIC
	SALE	тнор	SOIL PROFILE	⊢		SAM	PLES	RESIS	TANCE,	BLOWS/	0.3m	ζ,	HYDR/	AULIC CO k, cm/s	,	IVITY,	Ī	ING	PIEZOMETER
	DEPTH SC METRE	BORING ME	DESCRIPTION	STRATA PLO	ELEV. DEPTH (m)	NUMBER	BLOWS/0.3n	SHEA Cu, kF	20 4 R STREM 'a	10 6 NGTH n r 10 6	0 8 atV. + emV.⊕ 0 8	Q - • U - O	10 W W 1	0° 10 ATER CO	0 ⁷³ 10 DNTENT <u>OW</u> 0 3	PERCEI	0'3 NT WI 0	ADDITION LAB. TEST	OR STANDPIPE INSTALLATION
	- 0				61.68														our sector SI SI
AIGINT(1560 SCOTT ST OTTAWA.GPJ GAL-MIS.GDT 11/2/22	- 0 - 1 - 2 - 3 - 4		GROUND SURFACE CONCRETE BLOCK FILL - (GP) GRAVEL, poorly graded, trace sand; dry (GP) sandy GRAVEL, trace silt; grey (TILL); dry, compact to very dense END OF BOREHOLE/DRILLHOLE Notes: 1. Auger refusal.		61.68 0.00 60.92 0.76 60.31 1.37 1.52	2 S	S 16						0						Concrete
ANAGEMENT/1560 SCOTT ST OTTAWA\02 DAT	- 6 - 7 - 8																		
001 S:\CLIENTS\LASALLE_INVESTMENT_M.	- 9 - 10																		
GTA-BHS	DE 1 :	PTH 50	SCALE		,	//	5	P	G	OL	. D	ΕI	R					L(CH	DGGED: PK ECKED: AKP

PR LO INC	OJEC CATIC	T: 22530229 (3000) DN: N 442716.1 ;E 5028019.0 FION: -90° AZIMUTH:	I	REC	ORI	DC)F			NG E	HC DATI		E:	B	H2	2 2-()3								SH DA	IEET 2 OF 2 TUM: Geodetic	
DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	FLUSH <u>COLOUR</u>	JN FLT SHF VN CJ RE TOT CORI	- Join - Fau - Fau - Vei - Con ECOVI	nt ult ear n jugat ERY SOLID SORE %	e - R.Q		- Bed - Folia - Con - Orth - Clea FRAC INDE PEF 0.25	ding ation act ogona vage T. X B n R		- Plar - Cun - Und - Step - Irreg SCON	nar ved lulating pped gular ITINUIT YPE AND	PC K SM Ra ME TY DAT	D- Poli - Slic M- Sm D - Rou 3- Med ACE	shed kensio ooth igh chanio	ded cal Bro CON r 0	eak /DRA IDUC (, cm/	BR - NOTE: abbrevi of abbr symbol ULIC TIVITY sec 20	Broke For ad ations i eviation s. Diam Point Ind (MF	en Ro Iditiona refer to 1s & Loade Loade Pa) o t co	RMC -Q'		
2		Continued from Record of Borehole 22-03 Fresh fine-medium grained, slightly porous to non-porous, grey LIMESTONE bedrock with thin Shale interbeds		60.16	2																						
	HQ-Coring				4																					WL=5.20 Oct 3, 2022 Sand	
				52.68	5																						
- 11 - 11 - 11 - 11	PTH \$	CALE							G					E	R	2									LO	IGGED: PK ECKED: AKP	-

APPENDIX B

Borehole and Test Pit Records – Previous Investigation (McRostie Genest Middlemiss and Associates)

	McROSTI	E GENE	EST	MIDDLEMISS		\$(DIL PRO	FILE & TEST	SUMMA	RIES	······
	B ASSOCIATES	ENGINEERS	680C	IÉS LTÉE	*	RUFIL	5001EF	RAIN ET RE	SUME DI	ES ESS/	112
		OTTAW	A CA	NADA	H	Iollan	d and s	Spencer	SF	2687	
EL	LEVATION OF GROUN	D SURFACE () Fondeur Zei So	ROI ROI	62.03 m	DAT	May 2	26 & J1	une 2,86	HOLE	ε	No.
N	OTES		<u> </u>						& Test	E 30	86-4
		T O	No.			100 E	F	-Probing or- Vane Test	3e	niaga-au-	
N NET	H Scale Nomater Nomater Nomater Nomater Nomater Nomater	Standar tration mps/3	NOT	DESCRIPTION DU SOL		R - MET	AU m	Martea	4	Hemmer	
EPTH L		Errel. Páná	MPLE			EPTH IN	ELEVA	He Cau Barre	ing - Sans Yu	la, Rad	
A DI		ē	s A E C	Ground Surface 7 Niveau du Soi		0	62.03	-Biene /	10 emor Shea 1 Résistence at	Strength (h Cissilione)	Paj # (kPaj
	2.0 m			FILL - sand gravel meta wood concrete & brick	21	1.00	61.03				
	water a	El. 60.	73→					∝-water El. 60	1evel 5 .83	June 4	,86
2-	Bott	m of pit		LIMESTONE		2.20	- 59.83				
-3-				core recovery 98%		2.95	59.08				
				LIMESTONE							
4-			1	LIMESTONE core recovery 100%		3.94	58.09				
				LIMESTONE			57.07				
				core recovery 100%		5.64-	56.39				
6				LIMESTONE							
7				Core recovery 100%		6.91-	55.12	0 29	80	73	
R	* RENOULDED - RE CORE RECOVERY CAROTTE RECUPE R * NO RECOVERY-	MANIÉ RÉE NON RÉCUPÉR	, ié	Core recovery 100%		7.20	54.93	WATER CONTES % TENEUR EN NATURAL NATURELLE LIQUID LIMIT LIMITE DE LIQUIDI PLASTIC LIMIT LIMITE DE PLASTI	NT EAU 	PLATE PLAQUE No.	

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	MCROSTI	ST	MIDDLEMISS	SOIL PROFILE & TEST SUMMARIES												
BASSOCIATES LTD. BASSOCIES LTEE					PROFIL SOUTERRAIN ET RÉSUMÉ DES ESSAIS											
	CONSULTING E	NGINEERS	- IN CA	IGENIEURS CONSEILS	Hol	land	and S	pencer								
	VATION OF GROUND	SURFACE (2	ERO	рертн) 62 21 т		Maxz	27 6 7	SF2687								
NIVERU DU SOL (PROFONDEUR ZERO) 02.21 m See Plate No. 2						_ DATE May 27 & June 3,00 FORAGE No.										
							<u></u>	N. 94 E38								
Contraction and parts	And the second s	NOCE / Edino - Issai Property - Issai Property - Issai	SAMPLE ÉCHANTRLON NO.	DESCRIPTION OF SOL DU SOL Ground Surface - Niveau du Sol	DEPTH H METRES	TRES ETRES	ε	Vano Test Escol du Scissomètr								
							VATION	Chuta LibreDrop No Cosine - Sans Tubaca								
						DE FTH OFONDI		BarroDie, Red								
2						<u> </u>		Coups/30 cm on Resistance un Cisciliement (k)								
	Sec. 1	Re. St.				0	62.21									
	Sec. Con			FILL - sand gravel												
=				ashes brick wood and boulders up to 0.60 m dia.	1.0											
1.			di			.00	- 61.23									
								- overnight water leve								
	100							Е1. 60.73								
2																
	Bott	om of pit			2	.40	59.81									
								┣╋┿┿╪╋┥┥╪┿╋┥┥╴┥╴╸								
5-1-1		- 1		LIMESTONE												
19																
		u .		CORE RECOVERY 979												
2	· 4 51	2		COLC RECOVERY 578	3.	80 -	58.41									
	一半副	ETT.														
		Canc	-	LIMESTONE												
	- Cindital	Jana														
	HTS.						}									
	Sec. 24	ALS: N	- F	core recovery 98%	5.	32	56.89									
		and a														
	a U 12.4			LIMESTONE												
	Carl Law					ŀ										
		3		Some			-									
				recovery 100%	6.8	32-	55.39									
				THESTONE			<u>o</u>	WATER CONTENT								
	ADDRESS AND ADDRESS	EN ANAL		recovery 100%	- 7.3	35	54.86	% TENEUR EN EAU PLAQUE								
		the story	né.	Bottom of hole												
				And and a second s				PLASTIC LIMIT								
MCROSTI	E GENE	EST	MIDDLEMISS	PRO	S(FIL	DIL PROF	RAI	a 1 V E	TES T R	ÉS		MA È DI	RIE ES I	s Ess	AIS	-
--	---	--------------------------	---	------------------	---------------------	-------------------------	-----------	------------	---	---	------------------------	------------	--	-------------------------	-------------------------	---
B ASSOCIATES CONSULTING	ENGINEERS	550C 5 - 11 4 C/	NGÉNIEURS CONSEILS	Holl	lan	d and Sj	pen	ce	r							
ELEVATION OF GROUN NIVEAU DU SOL (PRO NOTES	D SURFACE () FONDEUR ZE	ZERO RO) See	DEPTH)62.41 m Plate No. 2	DATE Ma	iy :	27 & Jur	<u>ne</u>	4,	36		HO FO & T N 9	LE RAG	E t F E	No. Pit 86-7 E 97		
Real Range Real Range Provincial Range Rang Rang	Garty-Standera Garty-Standera Pênd rajjan Bine - Geupe / 30 th	sample échanthlon No.	DESCRIPTION OF SOIL DU SOL	DE PTH IN METRES	PROFONDEUR - METRES	ELEVATION m NIVEAU m		P+4	bing- Nar Chu No (Bar Bar	1901) 1901) 1901) 190 L1 20 sing 190		-See	idaga ai au H baga la. P Stra Cisa	emmi Drop ed	9779-1 17 14P-0-1	
	E1. 54.5	9	core recovery 99% LIMESTONE core recovery 94% Bottom of hole	7.	00 37 83	55.41										



F		E CENE	CT	MIDDLEMISS		SC	IL PRO	FIL	E 8 1	TEST	su	мма	RIE	5	
	McROSI	CEINE	SOC	IÉS LTÉE	P	ROFIL	SOUTE	RRA	IN E	T RÉ	SUN	IÉ D	ES E	SSA	IS
	CONSULTING	ENGINEERS	5 - 11	SENIEURS CONSEILS	Чо	lland	and C	'no.							
		OTTAW	A CA	NADA		Tanu		per	icer			SF	ac	, 8 7	
E	LEVATION OF BROUN	D SURFACE (ZERO I	61.67 m	DATE	May :	27 & J	une	= 3,	86	H	DLE		<u>N</u>	
N	OTES	Se	e Pl	Late No. 2							FC &	ORAC Tes	SE st P	it 8	36-8
1		1					1	_	Bas	bine as	N	107	E	139	
02		U. U.	S.	DESCRIPTION OF SOIL		TRES	E			- Teet			nddge i	Pelepemi	
ALC: NO.	11010	The second	TON	DU SOL			AU m			Morte	Libre.		He	rop	
TN IN	11.4.		APLE IANTIL			PTH II	ELEVI	Ĺ		No Co Barre	ing = :	Bana Tu 1	ibayo Dia, fto	d	
00			E CI	Ground Surface ₇ Niveau du Sol		PROF	61.67		 Ceupe/	Blaws./ 30-am-s	30 em a Réele		r Stree	gth (bP	e)
E	HTS JA	1		FILL - crushed stone FILL - fine sand & ashe	es	0.15	61.52			Π	Π				TE
H	Not .	h				0.40	61.27	H	-dv	erni	ght	wa	ter	lev	el
E	10 mark	om of nit		ORGANIC material		0 72	-60 05	Π	B1	. 61	. 27				
H	BOEL	ou or pro		LIMESTONE <u>core_recovery_91%</u>		0 07	-60 70								H
				LIMESTONE		0.57	50.70	Π							TE
		5	1÷	TTUED LONE											
				core recovery 100%		1 73	-59 Q <i>1</i>	Π	IT				\prod		ΤH
-2-				LIMESTONE		1.75	JJ. J4								
			ŀ	core recovery 100%	-	2.14	59.53								TB
H				LIMESTONE <u>core recovery</u> 94%		2 52	50 15								Ħ
						2.52	JJ. 1J					ĪT	\prod		TB
-3-															
				LIMESTONE											H
E														<u> </u>	
															H
4				CORP RECOVERY 100%											
		-	F	COLE TECOVELY TOUS		4.14	57.53								
													4		
	The second second			LIMESTONE											
	Carl Same	ALC: NO							┼┼╏	-+					
	sean at	E1.56.38													
	2								┼┼╏	+++	+		+		
-6-	The second		-	core recovery 99%		5.78-	55.89								
	The second			Bottom of hole					┼┼┠	+			┼╂╎		
	Maria -				Î										B
	24 1 1				1			+		┼┼┤	┼╊		┼╂┥		
24															H
H	1000								ER C		80 NT	•			100
	R. HEMOULDED		_					% T	ENEU	REN	EAU		PLA	QUE	4
	CR. CARCTY RECOVERY	EREE				1		NAT	IRELLE			0	No	•	
	and the state	MON RECUPÉR	É					PLAS	TE DE I	LIQUIDI MIT PLAST	TE			1	
	I G Parts				1	<u>I.</u>				- and -					





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MCROSTIE GENE	ST MIDDLEMISS	PROF	SOIL PI	ROFILE & TEST	SUMMARIES
& ASSOCIATES LTD. & ASS	OCIÉS LTÉE		12 3001		SOME DES ESSAIS
CONSULTING ENGINEERS	CANADA	Holl	and and	Spencer	SEAGRA
ELEVATION OF GROUND SURFACE (ZE	RO DEPTH) 61.73 m	DATE Ma	y 28 &	June 3,86	HOLE No.
NOTES	See Plate No. 2				& Test Pit 86-13
				Problem	N 195 E 9
Contraction of the second seco	Z DESCRIPTION OF SOIL	83	STEL E	-Vens-Test	
the second	T DU SOL		TION	E Marte	Hibre Dree
	ANTIL	Ē	ULEVA	No Ca	ting - Sono Tubago
	3 Ground Surface ₇ Niveau du Sol			-Bisse/	30 em er Elter Strength (LPe)
ASPHALT	FILL - crushed stone	ŏ. c	18 - 61.	65 11 11	
	FILL - sand & ashes wi	th 0.2	20 [61.	53	
	some metal wood & piece	es			
	of electric wire				
Bottom of pit] , ,	5 60		
		1		58	
all water lost at	LIMESTONE		1	╞┽┶┾╇┽┥	
E1.60.08					
2				A- Dvern	ight water level
				11 E E I	. 59.69
E A I	core recovery 95%	2.6	6 59.0	07	
	LIMESTONE			┠┿┾┼╏┾┽	
MITS.					
	core recovery 100%	4.1	6- 57.5	5 7	
				┠┼┿┽┼╉┾┿╸	
	LIMESTONE				
and the second second second				┢╅┼┼┿┨╎┼╸	
			1		
	core recovery 93%	5 6			
	LIMESTONE	1 3.0	50.0		
E CANEDO	core recovery 100%	6.2			
	Potter of hits		J- 55.5	S	
	- BOLLOM OI NOLE	-]	┟┿┾┼┽╏┼┼┽	
		1		WATER CONTEN	150 1 75 1 100
B & RDtoulors				* TENEUR EN	EAU PLAQUE
RENANIE RECUPERT				NATURELLE	O [No.
NON RECUPERE				PLASTIC LIMIT	
			<u>(</u>	I	

	McROSTI	E GENE	ST	MIDDLEMISS		SC	IL PROF	ILE & TEST	SUMM/	RIES
	B ASSOCIATES	ENGINEERS	SOC - II	IÉS LTÉE NGÉNIEURS CONSEILS NADA	Н	olland	and Sp	pencer	SUME D	ES ESSAIS
ELE NIV	VATION OF GROUN EAU CU SOL (PRO	D SURFACE (2 FONDEUR ZEF	ERO SO Se	бертні <u>бl.67 m</u> e Plate No. 2	. DATI	<u>May</u>	30 & Ji	ne 2,86	SF HOLE FORAC & Tes	3687 SE No. t Pit 86-
Cast in in the	Parking Parkin	ensil- Attractics Pensistration 4 - Calapt / 30 am	PLE No.	DESCRIPTION OF SOIL DU SOL		PTH M METRES NOEUR - METRES	LEVATION M	- Drabling an - Vano Test- Marten Chuta No Casi Borre -		E; 4 / Hdags au mai au Bolecomòireo Brep ubage Dia, Red
-HOLL		ka 👸	8 A M	Bround Surface ₇ Niveau du Sol		PROF	<u></u> u z	-Biswe /1	i O em er Ster	er Strongth (hPo)-
	Hoti	am of pit		FILL sand & crushed stone with a trace of metal & ashes		0.52	61.67 -61.15			
	11-			core recovery 54%		1.02	60.65			
	ACTS:	4 1		LIMESTONE						
				core recovery 60%	-	2.52	59.15			
				LIMESTONE				Water 1 E158_	evel J 86	hune 4,86
a a	11 water los	nt at E1.57.44	-	core recovery 91%		4.02	57.65			
	84	81.57.48	1	LIMESTONE						
	The series									
				Lettom of hole		5.57	56.10			
							0.		80	73 100
112	Contraction and a second secon	Emanié Tom steurés	м.				NA NA LI LI	TURAL TURAL TURAL SUID LIMIT MITE DE LIQUIDIT ASTIC LIMIT	Ο έ Ξ	No.

McROSTI	E GENES	ST MIDDLEMISS	PROF	SOIL PR	OFILE & TEST	SUMMA	RIES ES ESSAIS
B ASSOCIATES	ENGINEERS	INGÉNIEURS CONSEILS	Holla	ind and	Spencer		
ELEVATION OF CHOUND NIVERU DU SOL (PRO) NOTES	SURFACE (ZEI Fondeur Zero	See Plate No. 2	DATEMAY	7 <u>30</u> & J	une 4,86	HOLE FORAG	E No. E Pit 86- 55 - 13 86-13
In the second se	Zest-Standard Faistration Blave - Caupe / 300m SAMPLE	DESCRIPTION OF SOIL DU SOL	DETTH IM METRES	ELEVATION In NUVEAU	-Problem of Vene-Foot Mertes Chuta No Cas Barro. 	Libre	5 E 82 ndoge-se- sei-se Seissemètre — "Hemmer — Drop rbage Dia. Red r Strosgib (kPa)-
		FILL - topsoil		61.	73		Cidelflement (bPe)
		FILL - sand & gravel with a trace of ashes & meta	ith al	20- 61.1	53		
Bott	om of pit -		0.1	80- 60.9	93 	evel J	une 9,86
		LIMESTONE			E1.60.	73	
2							
NT5	1	core recovery 86%	2.3	32- 59.4	1		
0.5-	1/2	LIMESTONE					
				•			
		core recovery 100%	3.82	- 57.91			
		LIMESTONE					
			4.72	- 57.01			
		LIMESTONE					
		Core recovery 94%	5.80	- 55.93			
					0 20	80	75 184
A BARRIER					WATER CONTEN	IT EAU	PLATE PLAQUE
CR. CORE RELEASE	EMANIÉ ENÉR EDN RÉCUPÉRÉ				NATURAL NATURELLE	⊙ -é ⊡	№ . 16

McROST & ASSOCIATES	E GENES	T MIDDLEMISS DCIÉS LTÉE INGÉNIEURS CONSEILS	S PROFIL Hollan	OIL PROF SOUTER	FILE & TEST S RAIN ET RÉSU pencer	UMMARIES JMÉ DES ESSA	AIS
ELEVATION OF SEDUN HIVEAU DU SOL IPAC	OTTAWA	CANADA O DEPTH) 61.43 m See Plate No. 2	DATE May	30, 198	86=	SE 2687 HOLE FORAGE Test Pit	No.
Form on exception interction, out there for the former of the former of the former of the former of the former of the former of the former of the former of the former of the former of the former of the former of	Erri - Senera Pintu - Senera Senti - Senera / Gruco AMPLE	DESCRIPTION OF SOIL DU SOL	DEPTH IN METRES DFONDEUR - METRES	ELEVATION M NIVEAU m		N 60 E 00 	mètre
AT REMOULDED - RE	AANÉ	Ground Surface, Niveau du Soi FILL - topsoil FILL - sand & gravel wi some topsoil brick & concrete blocks with a little metal ashes & gl Bottom of pit on ro	0.30 th ass 2.00 ck	61.43			



MCROSTIE GENES	TMIDDLEMISS	sc	L PROFI	LE & TEST	SUMMARIES
BASSOCIATES LTD. BASSO CONSULTING ENGINEERS - OTTAWA	CIÉS LTÉE INGÉNIEURS CONSEILS ANADA	Hollan	d and S	oencer	SUME DES ESSAIS
ELEVATION OF GROUND SURFACE (ZERO NIVEAU OU SOL (PROFONDEUR ZERO) NOTES	Gee Plate No. 2	DATE May	26, 1986	5	HOLE No. FORAGE No. Test Pit
Etest-Standard Freedoman	DESCRIPTION OF SOIL DU SOL Ground Surface 7 Niveau du Sol	DEPTH W METRES PROFONDEUR - METRES	ELÉVATION m NIVEAU m	Probleg at Veno Test Martea Chata No Cas Barra,	IN 90 E 30
	FILL - sand & organic material with some ashe brick broken rock & boulders Bottom of pit on roc	0 es 2.05- k	61.91		

MCROST	E GENES	TMIDDLEMISS	S(DIL PROFI	LE & TEST	SUMMARIES
B ASSOCIATE	ENGINEERS -	INGENIEURS CONSEILS	Hollan	d and g	AIN ET RE	SUME DES ESSAIS
	OTTAWA	CANADA			pencer	SE2 687
RELEVATION OF GROUN NIVEAU OU BOL (PRO NOTES	D SURFACE (ZEF DFONDEUR ZERO)	See Plate No. 2	DATE May	29, 198	6	HOLE No. FORAGE Test Pit
-		<u>o</u>			-Probing ar-	N 120 E 00
	tendard ration sps / 30	DESCRIPTION OF SOIL	ME TREA METRE		Mortes	-Consi an Esissemetre
11-11-11-	Etaul-9 Finit-9 MPLE		PTH IN ONDEUR	ELEVAT I I V E A	Chute No Cas Berre	Libre Drop mg - Sanz Tubege
		Ground Surface, Niveau du Soi	0	61.25		Gemer Skour Strongth (kPat-
		FILL - crushed stone	0.10	61.15	ППП	
		topsoil with some bric	k			
		& a little cloth & glas	ss			
				┝		
	La soste					
water	Seepage at -	1 9	1.95	59.30		
12.24		Bottom of pit on roo	ck			
				ŀ		
	ġ.	6.25 Con				
	Real Property in					
	- 18 () - A	μ.		Ì		
	and the second	N.T. 5.		ļ		
Starting						
		and the same in the				
	1.	121111111111				
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MANUTA IN A		No. 12 States		WA	TER CONTEN	00 73 00 T PLATE
are a construction of the	CHANGE .			NAT NAT	URAL URAL	O No.
	The second second	Le Martin The			JID LIMIT JTE DE LIQUIDITI STIC LIMIT	i 🖸 27



McROST	E GENES		SO PROFIL		LE & TEST	SUMMARIES
CONSULTING	ENGINEERS - OTTAWA	INGÉNIEURS CONSEILS	Holland	and Sp	encer	SEDI &T
ELEVATION OF GROUN HIVEAU DU SOL (PRO MOTES	D SURFACE (ZERC Fondeur Zero)	See Plate No. 2	DATE May	26, 19	86	HOLE No. FORME No. Test Pit
		DESCRIPTION OF SOIL		E	-Probing or -Vene Test-	N 120 E 120 Condago da- Estat da Scientanitra-
	istal - Bhandar Póinótrathan an - Coupe / J APLE	DU SOL	TH IN METR NOEUR - MET	LEVATION	Morrao Chuta No Casj Barra	Lbre Prep Mi - Sana Tubege
		Ground Surface , Niveau du Soi	P N D C	₩ z 62.24	-Blove /3	Comer Shaar Strength (LPa) Résistance an Clastille mont (LPa)
	epage at El. 60.54	FILL - sand & clay with some wood brick & concrete ORGANIC material Bottom of pit on rock	- 1.70-	60.54 59.79		

McROSTI	E GENE	ST	MIDDLEMISS -	SO	IL PROF	ILE & TEST	SUMMAI	RIES
B ASSOCIATES	LTD. 8 AS	550CI 5 - 11 4 CA	IÉS LTÉE	Holland	and S	pencer	SUME DE	SESSAIS
ELEVATION OF GROUND NIVEAU DU SOL (PRO NOTES	SURFACE () FONDEUR ZE	ZERO (RO) Se	DEPTH)61.74 mDA De Plate No. 2	re <u>May</u> 3	30, 198	6	SF HOLE FORAG	≥687 E ^{No.} Pit
Line at the second seco	1- Standard nátration Caupe / 30em	E No.	DESCRIPTION OF SOIL DU SOL	1 IN METRES KUR - NETRES	EAU m	-Probing or -Vens Test- Martes Chute No Cat	N 1.50 - Sen - Sen - Sen Libre - Sana Tul	dege ov- ni ov Selspamètra- Hammer Hammer hage
41 2	Cana Pri Dina -	SAMPL ÉCHAN	Ground Surface ₂ Niveau du Sol	DEPTH	61.74	Barre Biewer -Couper/20em a	D 20 sm er Shaar 4 Résistance au	la. Rød Strength (hPa) Clastilamant (hPa
			FILL - sand gravel & topsoil with some brick & ashes & a little metal & glass	- 0.10	- 61.64 - 60.49			
			Bottom of pit on rock					
			<u>н.т.з.</u>					
				e:				
					<u>.</u>	23	50	73 100
W + STROUGERS CREATER AND	NEMADIĘ Tres Tres Stres	Éné				WATER CONTE % TENEUR EN NATURAL NATURELLE	NT EAU ○ ITÉ ○	PLATE PLAQUE No. 30

-	HOOST		FST	MIDDLEMICO	S					
	B ASSOCIATE	S LTD. 8 A	SSOC		PROFIL	SOUTERI	AIN ET RE	SUMM SUMÉ I	ARIES	-
	CONSULTING	ENGINEER: OTTAW	S - 11 A CA	NGÉNIEURS CONSEILS	Holland	and Sp	encer			
EL	EVATION OF BROUN	ND SURFACE () OFONDEUR ZE	ZERO (RO)	61.96 m	May	28, 19	86	SE	- <u>a687</u>	
NO	TES	1	i	See Plate No. 2				Test	Pit E 30	l
CTNR1	18 8	Mard Inn / 30om	No.	DESCRIPTION OF SOIL	ETRES	E	-Probing or-		iondage au- ional au Seconomà las	-
The second	The second secon	Fini-Sher Pini-Sher	PLE		th in Me	EVATION VEAU n	Martaa Chuta Na Casi	Libre	Drep Tubaye	
REP			3 A M ÉCH	Ground Surface , Niveau du Soi	D BEPT	ี่มี 2 61.96	Barro_	0 em er 84e	Die. Red er Streegth (kRe)	
				FILL - crushed stone	0.10	61.86		TTT	Clealing mant (1) Pa	F
				some wood ashes metal & brick	h					Ę
日				rock removed by shovel	1.00	60.96				Ę
I				Bottom of pit on rock	1.14	60.82				Ē
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				THE REAL PROPERTY AND A DECEMBER OF A DECEMBER OFOA DECEMBER OFOA DECEMBER OFOA DECEMBER OFOA DECEMBER OFOA DECEMBER OFOA DECEMBER OFOA DECEMBER OFOA DECEMB						
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11 × 11	CONT RECOVERY	MANIE	-			% 1	TENEUR EN EA	.υ 	PLATE PLAQUE No.	
	Allovery.	noù nécoren	٤			LIQI LIM PLA LIM	UID LIMIT ITE DE LIQUIDITÉ STIC LIMIT ITE DE PLASTICH	🖸	31	

F	McROST	IE GENE	SOC	MIDDLEMISS	PR	SO OFIL S		LE B TEST	SUMM/	RIES	
	CONSULTING	ENGINEERS OTTAWA		IGÉNIEURS CONSEILS	Ho	lland	and Sp	encer	SUME	ES ESSAIS	
ELS NIX NO	EVATION OF EROUI VEAU DU SOL (PR	ND SURFACE (2 OFONDEUR ZEI	See	Plate No. 2	DATE	May	26, 198	6	HOLE FORM Tes N 15	No. t Pit 0 E 120	
THE IN ALL TACK	Part and	teti - Standord Pédősestia a - Coope / 300m	PLE ANTILLON No.	DESCRIPTION OF SOIL DU SOL		TH IN METRES DEUR - MÉTRES	EVATION III VEAU III	- Probleg as - Vano Taol- Martea Chute No Cau		ndays-m_ osl an Baissanòire Hammer Brep ubage	
498 A			R AM	Ground Surface Niveau du Sol		PROFON	N N	Barra . Blama / J -Coups/JDam as	O em er Sher	Dia Red Ir Strength (hPa) Ir Clasiliament (hPi	 #}
				FILL - topsoil sand gravel bricks & pieces of wood	5	0	61.50				
.2.	ater scepag	e at El. 60.20		ORGANIC material L Bottom of pit on r	ock	1.35- 1.60-	60.25 59.95				
3				N N N N N N N N N N N N N N N N N N N							
				<u>N.T.S.</u>							
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7			1000				0	TER CONTEN	<u>80</u>		
e : : : : :	CORE ARCOVERY	Ener Rufe nécurén	É				NAT NAT LIQ LIM PLA	TENEUR EN E	AU O ś C	PLAQUE No. 32	

-	McROSTI	E GEN	EST	MIDDLEMISS	P	SO ROFIL S	IL PROFI	LE 8 TEST AIN ET RÉ	SUMMA	RIES ES ESSAIS
	B ASSOCIATES	OTTAW	s - II A CA	NGÉNIEURS CONSEILS	Ho	lland	and Sp	encer	SF	2607
ELI NIV NO	TES	SURFACE (Fondeur Ze	ZERO (RO) Se	e Plate No. 2	DATE	May	29, 19	86	HOLE FORM Test	No. Pit
Cristo ACTINITA			No.	DESCRIPTION OF SOIL		ETRES MÉTRES	E			U E 150 miles en est es Salassanà Ira-
DEPTH IN MI	A REAL	Easai - Etu Pánájran Okna - Caupi	BAMPLE ÉCHANTALOI			DEPTH IN M IOFONDEUR -	ELEVATK	Chute No Car Barre	Libre ing - Sans Tr	Drop whage Dig. Red
- E				FILL - Crushed stone		0	61.67	-Coupe/30cm a	a Résistance e	v-Cleaillement (kPa
				FILL - topsoil & sand with some broken rock ashes metal & glass Bottom of pit on roc <u>N.T.S.</u> N.T.S.	k	0.10	61.57			
en en	ADMOULDES - AD COME RECOVERT CANOTYE RECOVERT	CHANIÉ ERÉE MON RÉCUPÉ	RE				0 	ATER CONTES TENEUR EN TURAL TURELLE DUID LIMIT MITE DE LIQUIDI ASTIC LIMIT MITE DE PLAST	100 HT EAU ré — ⊡ cité — △	PLATE PLAQUE No. 33

F	MCROSTI B ASSOCIATES CONSULTING E	E GENI	EST ssoc s - II	MIDDLEMISS	SC PROFIL Hollan	DIL PROFI	LE & TEST	SUMM SUMÉ	ARIES DES ESSAIS
ELI HIV NO	EVATION OF BROUND VEAU OU SOL (PROP ITES	SURFACE (ZERO RO) Sec	DEPTH)62.37 mD	DATE May	28, 198	36	SF HOLE FORM Test	2687 No. Pit
SEPTA IN WEITHER][:]:	Estal - Branderd Pénétrgilan Blave - Ceups / 30cm	SAMPLE ÉCHANTILLON NO.	DESCRIPTION OF SOIL DU SOL	DEPTH IN METRES Nofondeur - Metres	ELEVATION m NIVEAU m	Hrobing or Vone Tost Marton Chato No Casi Barro	N I	80 E 30 Indege an Indege Salassemblike
				topsoil with some ashes brick broken rock metal & wood Bottom of pit on rock	1.80	60.57			
		Kamjé Én akcurén		N. I. J.		C WA % % NAT NAT LIOU LIMA	TER CONTENT TEREUR EN EX URAL URELLE URELLE TE DE LIQUIDITÉ STIC LIMIT	AU	PLATE PLAQUE No. 34

E	MCROSTI B ASSOCIATES CONSULTING E	E GENI LTD. 8 AS ENGINEERS OTTAW	EST ssoc 5 - II A CA	MIDDLEMISS Iés ltée NGÉNIEURS CONSEILS NADA	S PROFiL Holland	OIL PROF SOUTER	FILE & TEST RAIN ET RÉ pencer	SUMMA SUME D	RIES ES ESSAIS	
ELE NIVE NOT	ELU DU SOL (PROI	ONDEUR ZE	RO)	61.73 m ee Plate No. 2	DATE May	<u>30, 198</u>	36	FORA	No. Pit	
1 and		ulard ion 1/30em	No.	DESCRIPTION OF SOIL	TRES	EZE	-Probing or -Vano Tout Marter	<u> N 18(</u>) E 82	
IN NUMBER	Panelina Arte Arte Fuite RPG	Essel - Star Péréisa Blaus - Cauge	AMPLE CHANTILLON		DEPTH IN ME	ELEVATIO	Chute No Car Barre	Libre ing - Sans Ti	Drop uhays Dic. Rud	
1				Ground Surface, Niveau du Sai	0	61.73	-Blows/ -Goups/30 em -	30 om er Skoe u Rösletense a	ir Strangth (kPa) u Ciscillement(kPa)	H
				FILL - fine sand with a little metal & brick	0.20	- 61.53				
			=	Bottom of pit on roo	0.70	- 61.03				
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				<u> М.Т.5.</u>						
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	CHOREDER . AI Des RECOVENT SARITZ RECUP RE RECOVENT.	IMANIÉ ENTE NON RÉQUEÉ	ne			0 9 9 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1	ATTER CONTER VATER CONTER ATURAL ATURELLE INUTE DE LIQUIDI LASTIC LIMIT INITE DE PLASTI	150 IT EAU ré © cité &	PLATE PLAQUE No. 35	

MCROSTIE GENEST MIDDLEMISS

BASSOCIATES LTD. BASSOCIÉS LTÉE CONSULTING ENGINEERS - INGÉNIEURS CONSEILS SOIL PROFILE & TEST SUMMARIES

PROFIL SOUTERRAIN ET RÉSUMÉ DES ESSAIS

Holland ans Spencer

ELEV	ATION OF GROUN	OTTAWA D SURFACE () FONDEUR ZE	ZERO ROJ	DEPTH)62.06 m D	ATE May	29, 1986		ST HOLE	12687 N	0.
Statute of Late	and the second s	el - Standerd énéfration - Coupe / 30cm	TILLON No.	DESCRIPTION OF SOL	i th METRES EUR - MÉTRES	VATION III	-Probing or -Vano Tool Martea Chuta No Cau	Test <u>N 180</u> 	Pit <u>E 110</u> marge un set au Saloopmèt 	Hr-e-
		5	SAMP. ÉCHAI	Ground Surface , Niveau du Sol	D DEPTI	a a 62.06	Barra. Biasa/J -Conpo/20am an		Dia, Red I r Strength (LPa) IV Clastilement (1	 }
				FILL - topsoil FILL - fine sand TOPSOIL loose coarse SAND & GRAVEL	0.25 0.50 0.58 0.90	-61.81 61.56 61.48 -61.16				
	Water	eepage a		medium dense sandy TILL with a few boulders up to 0.45 m dia.						
		E., 33.3		Bottom of pit on rock	_ 2.30	-59.76				
				40 4 1.2 W						
				<u>N.T.S.</u>						
						0	23		73	
	AND AX-LINES	tain arcuri				WJ 76 NA NA Lit PL	TER CONTEN TENEUR EN E TURAL TURELLE UNID LIMIT NITE DE LIQUEDIT ASTIC LIMIT ITTE DE DI ACTOR	r AU Ο έΩ	PLATE PLAQUE No. 36	

F	MCROSTIE		EST	MIDDLEMISS	PI	SO ROFIL S	IL PROF	ILE 8 TEST	SUMMA	RIES ES ESSAIS
	CONSULTING E	NGINEERS	6 - IN A CA	IGÉNIEURS CONSEILS	Hc	lland	and Sp	encer	SF	3687
ELS HIV NO	TES	SURFACE () Ondeur 2e	ZERO (RO)	61.14 m ee Plate No. 2	DATE	May	29, 198	6	Test	Pit
1		Ę				. 5		-Probing ar-	<u> </u>	ndess-ou- Idi an Goissemètre-
PTH IN NEYNE	Barat Base Barat Sauta Aba Aba Barat Prastramolra APa	Essel - Standard Pásátration bws - Coups / 30	MPLE HANTELON	DESCRIPTION DU SOL		PTN 60 METRE Ondeur - Metre	ELEVATION III NIVEAU III	Marte Chate No Co Berre	Libre Libre Ling - Sans Tu	Hammer Orip bage Ha. Red
1014			5 -ŭ	Bround Surface , Niveau du Sol			61.14	-Blows/	30 em er Shee a Résistence w	- Streegth (bPa)- - Cissillement (kPa)-
				FILL - <u>crushed stone</u> FILL - sand & gravel w some ashes broken rock brick & metal	ith	0.10	60.14			
			-	Bottom of pit on roc	k					
2		5		L J M						
I COLORNAL DI				<u>N. T. S.</u>						
Distantistant										
THE REAL	A PARTY OF	No.				-				
HAIL							•	25		
11111	CONC ALCOVERY CONC ALCOVERY CAMPLES ALCOVERY	CMARTE BREE HEEVA	ini				W NA LI LI LI	AILN CONTE STENEUR EN ATURAL ATURELLE Guid Limit Mite de Liquidi Astic Limit Mite de Plast	EAU 	PLATE PLAQUE No. 37

	MCROSTIL B ASSOCIATES CONSULTING E	E GENI	EST ssoci s - IN A CA	MIDDLEMISS és ltée igénieurs conseils INADA	S PROFIL Hollar	OIL PROF SOUTERF	ILE & TEST RAIN ET RÉS pencer	SUMMARIES SUME DES ESSAIS
ELI NIV NO	NATION OF BROUND EAU OU SOL (PROF TES	SURFACE (ZERO I ROJ	e Plate No. 2	DATE May	28, 198	6	Test Pit
-	Provide State	E	ó			Ť	-Probing or-	<u>N 210 E 30</u>
N IN METRES	March Lands March Lands MAR March Lands March Lands Ma	tel-Standord Pánátration e - Coupe / 30e	PLE NITHLAN N	DESCRIPTION DU SOL	TH IN METRES	LEVATION m	Martea Chute Ne Cau Barre	Hammer LibraDrop ing - Sone Tubage
LUCION		City C	5 AMI	Ground Surface ₂ Nivedu du Sol	PROFO	u ≠ 62.38	-Blews/	LO am ar Chear Strength (bPa: Résistance en Cleatiliement (i
				FILL - topsoil FILL - sand & gravel with some broken rock brick metal wood glass & topsoil	- 0.30	- 62.08		
				Bottom of pit on roc	1.80	- 60.58		
					-1			
				יי <u>N.T.S.</u>				
						0	WATER CONTE	DO TO
a () a	CONE ALCONANY CONTRACTORNEY CAROLINE ALCONERY	ENANIÉ ANE SON RÉCUI	ini				% TENEUR EN NATURAL NATURELLE LIGUID LIMIT LIMITE DE LIGUID PLASTIC LIMIT LIMITE DE PLAST	EAU PLAQUE

EL	MCROSTI B ASSOCIATES CONSULTING	E GEN S LTD. & A ENGINEER OTTAW D SURFACE (FONDEUR ZE	EST SSOC S - I A C	Plate No 2	S PROFIL Hollan	OIL PROF SOUTERF d and Sp y 30, 19	ILE ATEST AIN ET RÉ Dencer 186	SUMMARIES SUMÉ DES ESSAIS SF2687
		East - Caupt / 20 cm	See sample femalica No.	Plate No. 2 DESCRIPTION OF SOIL DU SOL Ground Surface, Niveau du Soi FILL - Crushed stone FILL - topsoil FILL - till with a trace of brick & metal Bottom of pit on rock <u>N.T.S.</u>	мате <u>Ма</u> Salking - ипоинојон 0.25 0.47 0.86	E NOLLANDIN 61,73 -61.48 -61.26 -60.87	Presing or Vene Test Mortes Barre 	HOLE- Ne. Test Pit N 210 E 60
R = HED CR = COA DR = R	AGULDED . REMA NE ARTOVERY	NIÉ RÉCUPÉNÉ	A ALLANT CAL			WATE % TEI NATURI NATURI LIQUID LIMITE PLASTI LIMITE	R CONTENT NEUR EN EAU AL ELLE	- 0 No. - 33

MCROS & ASSOCIA CONSULTIN	TIE GEN TES LTD. & A IG ENGINEER OTTAW	EST ssoc s - 1 A CA	MIDDLEMISS	So PROFIL Hollan	DIL PROFI SOUTERR d and Sp	LE & TEST S AIN ET RÉSU pencer	UMMARIES IMÉ DES ESSAI SF2687	15
NOTES	PROFONDEUR ZE	S	ee Plate No. 2	TE May	30, 198		HOLE N FORAGE N Cest Pit	le.
And the second s	Estel - Standord Pánálrattan Bibut - Caupe / 30 cm	SAMPLE ÉCHANTILLON NO.	DESCRIPTION OF SOIL DU SOL Ground Surface J Niveau du Soi	O DEPTH IN ALETRES PROFONDEUR - METRES	ELEVATION M NIVEAU M	- Problem an- - Vana Tast- - Vana Tast- - Mortaus Chuto Libr Mo Casing Barro - Bierro - Bierro		
			FILL - topsoil FILL - medium sand with a piece of concrete pipe & a trace of metal Bottom of pit on rock <u> <u> o</u>.9n <u> t</u>. <u> N.T.S.</u></u>	0.20	60.72			

Г	McROST	IE GENI	EST	MIDDLEMISS	F	SC		LE & TEST	SUMM	ARIES	· <u></u>
	B ASSOCIATE CONSULTING	ENGINEERS OTTAW	5 - 11 A C4	NGÉNIEURS CONSEILS	H	olland	and Sp	encer	SUME	Facet	5
EL NU	EVATION OF JROU VEAU OU SCL (PR	ND SURFACE () OFONDEUR ZE	Se	e Plate No. 2	DATE	May	30, 198	6	HOL	t Pit	.
PLAN MEXAGO	National States	ti-Stendera strictiona - Coupe/30om	TILLON No.	DESCRIPTION OF SOIL DU SOL		I IN METRES FUR - METRES	VATION m EAU m	-Probing-or- -Vene-Foot- Martum Chute No Cast		Zendage-m- Sendage-m- Standag	ire-
PR070H		311	SAMP ÉCHAI	Ground Surface ₇ Niveau du Soi		O DEPTI	∄ ≩ 61.71	Burra_ Biors/3 -Coups/30.cm.us	it om er Eh	Dia Red	
				FILL - topsoil & sand with a trace of metal brick & ashes Dottom of pit on rock MT.S.	k	0.75	60.96				

Г	McROST	TE GEN	EST	MIDDLEMISS -	SC	DIL PROF	ILE & TEST	SUMMARIES	
	CONSULTING	ENGINEER	S - II A CA	NGÉNIEURS CONSEILS	Holland	l anđ Sp	encer	SE DES ESSAI	
ELI NIV NO	EVATION OF GROUI TEAU DU SOL (PR	ND SURFACE (See	<u>62.17 m</u> D. Plate No. 2	ATE May	30, 198	6	HOLE N FORAGE N Test Pit	10.
THE IN METHICS NOTINE METHICS	Pérférens La ser	181 - Bhundurg Pinistration a - Canpa / 30 cm	LE NO.	DESCRIPTION OF SOIL DU SOL	H IN METRES DEUR - MÈTRES	EVATION m VEAU m	- Probing or - Vone Tech- Marton Chute No Cast	Un 233 E 84 	
REP		- 1	S AMP É CHA	Ground Surface, Nivecu du Sol FILL - topsoil	O DEFT	⊒ ≨ _62.17	Barre_ Blone./-1 Coupe/30.om-ou	Dia. Red	
				FILL - sand gravel & topsoil with a trace of brick & metal Bottom of pit on rock	- 1 - 0 -	61.07			
		N. C.							
T ALL ROOM	CAUGULOKD - MED MRS MECONENT NO RECUVERT - 0	Namié née na nicuréné				O WAT % TI NATU LIMIT PLAS	ER CONTENT ENEUR EN EA RAL RELLE D LIMIT E DE LIQUIDITÉ- TIC LIMIT E DE LIMIT	U PLATE PLAQUE PLAQUE No. 42	

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	McROST	IE GENE	EST	MIDDLEMISS		ROFI	SOUTER	ILE & TEST	SUMM	ARIES
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60	CONSULTING	OTTAW	A C4	NADA	H	lolland	d and S	pencer		
-		D SURFACE (ZERO	DEPTH)					SF	2687
NIV	EAU OU GOL (PRO	FONDEUR ZE	ro) See	62.11 m Plate No. 2	DATE	May	30, 19	86	FORA	GE No.
NO	TES								Test	Pit Pit
						- 8		-Probing or-		endege og -
1111	11.1	minut tion ar/ 30	Z	DESCRIPTION DU SOL		ETHE	E Z E	Martea	u	Hammer
N IN N	152 5	- Cruz	LE			H HK W	EVATI	Chute No Cas	Libre Ing - Sans T	Drap 'ubayo
1454	-1 2	5 I	SAMP.	Realized Fundamental Million of the		DEPT	NI) ELL	Batro.		Dia. Red
-				ETTL - topsoil	\dashv	0	62.11	-Coupe/JCom e	Résistance :	Be Ciselillement (kPe)
				FILL = sand & gravel with		0.20	61.91			
				some ashes brick wood						
				metal asphalt & glass						
1		7.5				0.95	61.16	┝┿┿┿┿╋╌┿╋	┽╎┨╷╽	
		1.81 14-		boulders up to 0.6 m dia	a.					
				In dense sandy TILL					┿┽┠┼┝	╺╴╴╴╴╴╴
-				\$		1.80	60 31			
2-		C.S.		Bottom of pit on rock	c			┽┼┼┼┨┼┼	┟┥╏┝╽	┼┼╏┽╷╷╷┝┣
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Holland and Spencer Avenues, Beech Foundry Site, Rock Elevations

McRostie Genest Middlemiss

June 6, 1984

(Report No. SF-2481)

McRO:	STIE GEI	VEST	M	IDDLEMISS		SOIL	PROFILE & T	EST SUMA	ARIES
& ASSOC	ATES LTO). &	ASSO	CIÉS LTÉE	PR	OFIL SO	JTERRAIN ET	RÉSUMÉ	DES ESSAI
CONSUL	TING ENGI	NEER	S – 1	NGÉNIEURS CONSEILS			SPENCER	57	
	OTT	AWA	C	ANADA	-		07 2 H C C H	0 /.	SF 2481
ELEVATI NIVEAU C	ON OF GROU	JND SU		CE (ZERO DEPTH)	05.2		DATE MAY	6/84	HOLE
NOTES B	M (EL. 206.6	S)GO	DDET .	IC CITY OF ATTAWA PLATE OF HUSEST CORNER OF H	N NORTH	EAST CO	RNER OF OFS	PERRY	TEST PIT
- H (D 5		bd/s		DUG BY TRACK MOUNTED SH	wite's	0	PROBING O	2	SONDACE OU
K.S. K.S. ance ressi	Scale Scale F.	Coup	5	DESCRIPTION DU SOL	E C	non du	MARTEAUH	AMMER	NO CASING SANS TUBAGE
ompr ength Comp K/F	Rept. K.S K.S Fe	sai- Penet	ple		pth is	levo Nive	CHUTE LIBRE-	OR SHEAR S	RREDIA.
0545	<u> </u>	E B lov	Sam Ech	Ground Surface 7 Niveau du S	iol 0		GOUPS/Pico C	- CISAILL	MCE AU. K/P
					0	205.2	-		
			-	- FILL-		-			
				SAND GRAVE	2				
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				E ASPES, WITT	7				
			-	SOME METAL		-			
				a cias	-				
				WOOD, GLASS	-				
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SLIGH	T WATER	2							
SEEPA	65 AT	-			6	- 199.2		,	
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S. Contraction							NATURAL NATURELLE		- O
CORE REC	ED RE MANIE						LIQUID LIMIT LIMITE DE LIQUI PLASTIC LIMIT	DITÉ	

McRO	MCROSTIE GENES	TM	IDDLEMISS	SOIL PROFILE & TEST SUMMARIES									
& ASSO	& ASSOCIATES LTD. &	CIÉS LTEE	PROFIL SOUTERRAIN ET RÉSUMÉ DES ESSAIS										
CONSU	CONSULTING ENGINEE	CONSULTING ENGINEERS - INGÉNIEURS CONSEILS					SPENCER ST.						
TANSIS		C.	ANADA	SF 2481									
NIVEAU NOTES &	NIVEAU DU SOL (PROFON	DEUR	ZERO) 2002 SEE PLATE NO.2	DATE MAY 16/84 FORASE NO.									
197.097.9		2 .				-PROBING OR-	TEST PIT 2						
avias R.2.7 K.3.5	K.S.F K.S.F F Scale Scale F F F and artendar	sai - Standar Penetration 's/ttCoups, ble N	DESCRIPTION OF SOIL	pth in Feet ondeur-Pie	pth in Feet ondeur-Pie levation Niveau	MARTEAUHAMME	R SANS TUBAGE						
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APPENDIX C

Laboratory Test Results





APPENDIX D

Results of Geophysical Testing

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TECHNICAL MEMORANDUM

DATE September 21 2022

TO Arthur Kuitchoua Petke Golder Associates Ltd.

FROM Geophysics Group

EMAIL abilsondarko@golder.com; cphillips@golder.com

Project No.22530229

MASW SURVEY RESULTS - 1560 SCOTT STREET, OTTAWA, ONTARIO

This technical memorandum presents the processing and results of the Multichannel Analysis of Surface-Waves (MASW) test performed for the purpose of Seismic Site Classification for a site on 1560 Scott Street, located in Ottawa, Ontario. The geophysical testing was performed by Golder Associates Ltd. (Golder) personnel on September 15th, 2022, along the survey line shown in Plate 1, below.



Plate 1: MASW Survey Line Location in red.

Methodology

The MASW method measures variations in surface-wave velocity with increasing distance and wavelength and can be used to infer the rock/soil types, stratigraphy and soil conditions.

A typical MASW survey requires a seismic source, to generate surface-waves, and a minimum of two geophone receivers, to measure the ground response at some distance from the source. Surface-waves are a special type of seismic wave whose propagation is confined to the near surface medium.

The depth of penetration of a surface-wave into a medium is directly proportional to its wavelength. In a non-homogeneous medium surface-waves are dispersive, i.e., each wavelength has a characteristic velocity owing to the subsurface heterogeneities within the depth interval that wavelength of surface-wave propagates through. The relationship between surface-wave velocity and wavelength is used to obtain the shear-wave velocity and attenuation profile of the medium with increasing depth.

The seismic source used can be either active or passive, depending on the application and location of the survey. Examples of active sources include explosives, weight-drops, sledgehammer and vibrating pads. Examples of passive sources are road traffic, micro-tremors and water-wave action (in near-shore environments).

The geophone receivers measure the wave-train associated with the surface-wave travelling from a seismic source at different distances from the source.

The participation of surface-waves with different wavelengths can be determined from the wave-train by transforming the wave-train results into the frequency domain. The surface-wave velocity profile with respect to wavelength (called the 'dispersion curve') is determined by the delay in wave propagation measured between the geophone receivers. The dispersion curve is then matched to a theoretical dispersion curve using an iterative forward-modelling procedure. The result is a shear-wave velocity profile of the tested medium with depth, which can be used to estimate the dynamic shear modulus of the medium as a function of depth.

Field Work

The MASW field work was conducted on September 15th, 2022, by personnel from the Golder Mississauga office. For the MASW line, a series of 24 low frequency (4.5 Hz) geophones were laid out at 1 metre intervals. An 8-kilogram (kg) sledgehammer and 40 kg seismic weight drop were used as seismic sources for this investigation. Seismic records were collected with seismic sources located 5 and 10 metres from and collinear to the geophone array. An example of an active seismic record collected at the site is shown in Figure 1.



Figure 1: Typical seismic record collected for the MASW Line.

Data Processing

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

- 1) Transformation of the time domain data into the frequency domain using a Fast-Fourier Transform (FFT) for each source location;
- 2) Calculation of the phase for each frequency component;
- 3) Linear regression to calculate phase velocity for each frequency component;
- 4) Filtering of the calculated phase velocities based on the Pearson correlation coefficient (r²) between the data and the linear regression best fit line used to calculate phase velocity;
- 5) Generation of the dispersion curve by combining calculated phase velocities for each shot location of a single MASW test; and
- 6) Generation of the stiffness profile, through forward iterative modelling and matching of model data to the field collected dispersion curve.

Processing of the MASW data was completed using the SeisImager/SW software package (Geometrics Inc.). The calculated phase velocities for a seismic shot point were combined and the dispersion curve generated by choosing the minimum phase velocity calculated for each frequency component as shown on Figure 2. Shearwave velocity (V_s) profiles were generated through inverse modelling to best fit the calculated fundamental mode dispersion curves.



Figure 2: MASW Dispersion Curve Picks (red dots) for the MASW Line.

The minimum measured surface-wave frequency with sufficient signal-to-noise ratio to accurately measure phase velocity was approximately 25 Hz for the MASW Line.

Results

The MASW test results are presented on Figures 3, which present the calculated shear-wave velocity profile measured from the MASW Line. There is good correlation between the field collected and model calculated dispersion curves, with a root mean squared error of less than 5%.



Figure 3: MASW Modelled Shear-Wave Velocity Depth profile for the MASW Line.

Model Layer (mbgs)		Layer Thickness (m)	Shear Wave Velocity (m/s)	Shear Wave Travel Time Through Layer		
Тор	Bottom			(s)		
0	1.1	1.1	380	0.002893		
1.1	2.3	1.2	432	0.002780		
2.3	3.7	1.4	623	0.002247		
3.7	5.3	1.6	861	0.001857		
5.3	7.0	1.7	928	0.001832		
7.0	8.9	1.9	907	0.002095		
8.9	11.0	2.1	937	0.002242		
11.0	13.2	2.2	1012	0.002173		
13.2	15.6	2.4	1073	0.002237		
15.6	18.1	2.5	1153	0.002168		
18.1	20.9	2.8	1231	0.002274		
20.9	23.7	2.8	1294	0.002164		
23.7	26.8	3.1	1338	0.002317		
26.8	30.0	3.2	1368	0.002339		
	949					

Table 1: Shear-Wave Velocity Profile MASW Line

To calculate the average shear-wave velocity as required by Seismic Site Classification, the results were modelled to 30 metres below ground surface (mbgs).

The time-averaged shear-wave velocity (Vs30) for the MASW Line was found to be 949 m/s (Table 1).

Closure

We trust that this technical memorandum meets your needs at the present time. If you have any questions or require clarification, please contact the undersigned at your convenience.

Golder Associates Ltd.

Alex Bilson Darko, MSc. *Geophysics Group*

ABD/CRP

Christopher Phillips, MSc., PGeo Senior Geophysicist, Principal

APPENDIX E

Hydraulic Conductivity Testing Results

BOUWER AND RICE SLUG TEST ANALYSIS FALLING HEAD TEST 22-01

INTERVAL (metres be	low ground surface)
Top of Interval =	3.05
Bottom of Interval =	6.27
$r_c^2 ln\left(\frac{R_e}{r}\right)_1$	N.
$K = \frac{(r_w)}{2L_e} \frac{1}{t} ln$	$\frac{y_0}{y_t}$ where K=m/sec

where:

- r_c = casing radius (metres);
- R_e = effective radius (metres);
- L_e = length of screened interval (metres);

 r_w = radial distance to undisturbed aquifer (metres)

 y_0 = initial drawdown (metres)

 y_t = drawdown (metres) at time t (seconds)





Project Name: Holland Cross Project No.: 22530229 Test Date: 03-Oct-22 Analysis By: SPS Checked By: CAMC Analysis Date: 05-Oct-22

HVORSLEV SLUG TEST ANALYSIS RISING HEAD TEST 22-02B

INTERVAL (metres belo	ow ground surface)
Top of Interval =	5.95
Bottom of Interval =	9.00

$$K = \frac{r_c^2}{2L_e} ln \left[\frac{L_e}{2R_e} + \sqrt{1 + \left(\frac{L_e}{2R_e}\right)^2} \right] \left[\frac{ln \left(\frac{h_1}{h_2}\right)}{(t_2 - t_1)} \right]$$
 where K = (m/sec)

where:

 r_c = casing radius (metres) R_e = filter pack radius (metres)

 L_e = length of screened interval (metres)

t = time (seconds)

 h_t = head at time t (metres)





Project Name: Holland Cross Project No.: 22530229 Test Date: 2022-10-03 Analysis By: SPS Checked By: CAMC Analysis Date: 2022-10-05

Golder Associates Ltd.

BOUWER AND RICE SLUG TEST ANALYSIS RISING HEAD TEST 22-03



where:

- r_c = casing radius (metres);
- R_e = effective radius (metres);
- L_e = length of screened interval (metres);

 r_w = radial distance to undisturbed aquifer (metres)

 y_0 = initial drawdown (metres)

 y_t = drawdown (metres) at time t (seconds)





Project Name: Holland Cross Project No.: 22530229 Test Date: 03-Oct-22 Analysis By: SPS Checked By: CAMC Analysis Date: 05-Oct-22

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Dupuit-Forchheimer Equation: $Q=\pi K((h_o^2-h_p^2)/ln(R/r))$

Groundwater Infl	ow		-	Equivalent radius of excav	ation	
K (m/sec)	2E-05		Γ	AxB=πr ²		
h ₀ (m)	5.6	r - Equivale	nt Radius	width of excavation A =	34	m
հ _թ (m)	3.4	R - Radius	of Influence	length of excavation B =	49	m
r (m)	23.0			Area =	1,666	m²
				r=	23.0	m
	Q (m3/s)	R	Rad. of Inf. from edge	m³/day	L/day	
	1.4E-02	25.0	2	1,182	1,181,719	
Initial	9.3E-03	26.0	3	804	803,667	
	5.8E-03	28.0	5	501	500,875	
	3.2E-03	33.0	10	273	272,896	
	2.3E-03	38.0	15	196	196,204	
	1.8E-03	43.0	20	157	157,434	
	1.5E-03	48.0	25	134	133,889	
Steady State	1.4E-03	53.0	30	118	117,992	
	1.2E-03	58.0	35	106	106,488	
	1.1E-03	63.0	40	98	97,747	
	1.1E-03	68.0	45	91	90,858	
	9.9E-04	73.0	50	85	85,275	
	8.9E-04	83.0	60	77	76,741	
	7.9E-04	98.0	75	68	67,943	
	6.8E-04	123.0	100	59	58,732	





Sichart and Kyrieleis Equation: R=3000Δh(K^{1/2}) Excavation Radius of Influence (m) = 2 28

- Static groundwater elevation (m) 58.3 Elevation of bottom of excavation (m) 56.6
 - Dewatered elevation (masl) 56.1
 - Top of confining unit (masl) 52.7 assumed at bottom of BH

highest water level (BH22-01)

Rainfall Amount - Based on a 79.2 mm precipitation event in 24 hours with a return of 10 years (litres): Precipitation (litres) = 131,947

APPENDIX F

Rock Photos and Results of UCS Testing



https://golderassociates.sharepoint.com/sites/35409g/Shared Documents/Active/2022/22530229/













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