REPORT

Geotechnical Investigation

Proposed Commercial Development, 2707 Solandt Road, Ottawa, Ontario

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Distribution List

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1.0 INTRODUCTION

This report presents the results of a geotechnical investigation carried out for a proposed commercial development to be located at 2707 Solandt Road in Ottawa, Ontario.

The geotechnical investigation included an assessment of the general subsurface conditions in the area of the proposed development by means of six boreholes and laboratory testing. 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 prepare engineering guidelines on the geotechnical design aspects of the project, including construction considerations which could influence design decisions.

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

2.0 DESCRIPTION OF PROJECT AND SITE

Plans are being prepared for the construction of a commercial development to be located at 2707 Solandt Road in Ottawa, Ontario. The approximate location of the site is shown on the Key Map inset on the attached Site Plan (Figure 1).

The following information is known about the site:

- The site is located along the north-west side of Solandt Road and is bounded to the north-west by a golf course, to the north-east by an existing at grade parking lot, and to the south-west by existing commercial developments.
- The site is roughly rectangular in shape and measures about 180 metres by 150 metres in plan area.
- The south-western half of the site is undeveloped, with dense bush and tree cover.
- It is understood that the development will consist of an eight-storey building of slab on grade construction (i.e., no basement level) to be located at the south east corner of the site with at grade parking.

Based on a review of the published geological mapping, and previous investigations carried out adjacent to the site, the subsurface conditions at this site are expected to consist of about 4 to 6 metres of sand, silty clay and glacial till overlying bedrock. The bedrock is mapped to be dolomitic limestone of the Oxford Formation.

3.0 PROCEDURE

The fieldwork for the geotechnical investigation was carried out between November 5 and 7, 2018. During that time, six boreholes (numbered 18-101 to 18-106, inclusive) were advanced using a rubber-tired ATV-mounted hollow-stem auger drill rig supplied and operated by CCC Geotechnical and Environmental Drilling of Ottawa, Ontario. The approximate locations of the boreholes are shown on the attached Site Plan (Figure 1).

The boreholes were advanced to refusal to auger advancement, which was encountered at depths ranging from about 3.7 to 7.5 metres below the existing ground surface. Upon reaching refusal to auger advancement in boreholes 18-102, 18-103, and 18-104, the boreholes were advanced into the bedrock for a length ranging from about 1.5 to 1.6 metres into the bedrock surface using rotary diamond drilling techniques while retrieving NQ sized bedrock core.

Standard penetration tests were carried out within the boreholes at regular intervals of depth and soil samples were recovered using split-spoon sampling equipment. In situ vane testing was carried out, where possible, in the silty clay to determine the undrained shear strength of this soil unit.

Standpipe piezometers were sealed into boreholes 18-102, 18-105, and 18-106 to allow for subsequent measurement of the groundwater level. The groundwater levels were measured in the monitoring wells on November 16, 2018.

The fieldwork was supervised by experienced personnel from our staff who directed the drilling and in situ testing operations, logged the boreholes and samples, and took custody of the samples retrieved. On completion of the drilling operations, soil samples obtained from the boreholes were transported to our laboratory for further examination by the project engineer and for laboratory testing, including natural water content, Atterberg limits, and grain size distribution tests.

One sample of soil from borehole 18-103 was submitted to Eurofins Environment Testing for basic chemical analyses related to potential sulphate attack on buried concrete elements and potential corrosion of buried ferrous elements.

The borehole locations were selected by KRP Properties and marked in the field and subsequently surveyed by Golder Associates personnel. The location and ground surface elevation at each borehole location were determined using a precision Trimble R8 GPS survey unit. The geo-reference coordinates are based on NAD 83 Coordinate system, UTM Zone 18. The elevations are referenced to Geodetic datum (CGVD 1928).

4.0 SUBSURFACE CONDITIONS

4.1 General

Information on the subsurface conditions is presented as follows:

- Record of Borehole and Drillhole Sheets and core photographs are provided in Appendix A.
- Results of the basic chemical analyses are provided in Appendix B.
- Results of the grain size distribution testing are provided on Figures 2 and 4.
- Results of Atterberg limit testing are provided on Figure 3.
- Photographs of the bedrock core are provided on Figures 5 to 7.
- Results of unconfined compressive strength testing on samples of the bedrock core are provided in Appendix C.
- Results of the laboratory natural water content and Atterberg limits testing are provided on the Record of Borehole sheets.

In general, the subsurface conditions on this site consist of topsoil and silty sand overlying a deposit of silty clay to clay over a discontinuous deposit of glacial till over sandstone bedrock. The following sections present a more detailed overview of the subsurface conditions encountered in the boreholes advanced during the investigation.

4.2 Topsoil

A layer of topsoil exists at the ground surface at the borehole locations. The topsoil ranges in thickness from about 150 to 300 millimetres.

4.3 Silty Sand to Sand

A deposit of silty sand to sand exists below the topsoil at all the borehole locations. The silty sand to sand extends to depths ranging from about 0.9 to 1.7 metres below the existing ground surface.

Standard penetration tests carried out within the silty sand to sand gave SPT ‘N’ values ranging from 2 to 15 blows per 0.3 metres of penetration, indicating a very loose to compact state of packing.

The measured water content of two samples of the silty sand to sand are about 4 and 8 percent.

The results of grain size distribution testing carried out on two samples of the silty sand are provided on Figure 2.

4.4 Silty Clay to Clay

A deposit of sensitive silty clay to clay (hereafter collectively referred to as silty clay) exists beneath the topsoil and silty sand to sand at the borehole locations. The silty clay deposit extents to depths ranging from about 3.4 to 6.1 metres below the existing round surface.

The upper portion of the silty clay deposit has been weathered to a grey brown crust, which extends to depths ranging from about 1.5 to 2.9 metres below the existing ground surface. Standard penetration tests carried out within the weathered crust gave SPT ‘N’ values ranging from 3 to 9 blows per 0.3 metres of penetration, indicating a very stiff to stiff consistency. The results of the Atterberg limit testing are provided on Figure 3. The measured water content of three samples of the weathered crust ranged from about 32 to 43 percent.

The silty clay beneath the depth of weathering is grey in colour. The unweathered silty clay extends to depths ranging from about 3.4 to 6.1 metres below the existing ground surface. The results of in situ vane testing in the grey silty clay gave undrained shear strengths ranging from about 54 to 73 kilopascals, indicating a stiff consistency.

The results of Atterberg limit testing carried out on two samples of the grey silty clay gave plasticity index values of about 35 and 38 percent and liquid limit values of about 55 and 58 percent, indicating high plasticity soil. The measured water content of seven samples of the grey silty clay ranged from about 37 to 58 percent.

4.5 Glacial Till

A deposit of glacial till exists below the silty clay in the boreholes, except at borehole 18-104. The glacial till generally consists of a heterogeneous mixture of gravel, cobbles, and boulders in a soil matrix of silty sand to sandy silt. The glacial till was not fully penetrated in all the boreholes but was proven to depths ranging from about 3.7 to 7.5 metres below the existing ground surface.

Standard penetration tests carried out within the glacial till gave SPT ‘N’ values ranging from 2 to greater than 50 blows per 0.3 metres of penetration, but more generally between 2 and 13 blows per 0.3 metres of penetration, indicating a very loose to compact state of packing. The higher blow counts could possible reflect the surface of the bedrock rather than the state of packing of the soil matrix.

The measured water content of one sample of the glacial till is about 17 percent.

The results of grain size distribution testing carried out on one sample of the glacial till are provided on Figure 4.

4.6 Auger Refusal Bedrock

Refusal to auger advancement was encountered in boreholes 18-101, 18-105, and 18-106 at depths ranging from about 3.7 to 7.5 metres below the existing ground surface.

The bedrock was encountered below the overburden in boreholes 18-102, 18-103, and 18-104 at depths ranging from about 4.9 to 7.5 metres below the existing ground surface. These boreholes were then advanced into the bedrock to a total depth ranging from about 1.5 to 1.6 metres using rotary diamond drilling techniques.

The inferred depth to bedrock and elevation of the bedrock surface is summarized in the table below:

Borehole No.	Ground Surface Elevation (m)	Depth to Refusal/Bedrock (m)	Refusal/Bedrock Elevation (m)
18-101	77.35	4.87(1)	72.48(1)
18-102	77.23	6.20	71.03
18-103	77.18	7.53	69.65
18-104	77.36	4.90	72.46
18-105	77.03	3.70(1)	73.33(1)
18-106	77.16	5.76(1)	71.40(1)

Note: (1) = Depth and elevation to bedrock inferred from refusal to auger advancement.

The bedrock encountered at this site typically consists of fresh, thinly to medium bedded, fine grained, non-porous, grey sandstone bedrock. The Rock Quality Designation (RQD) values measured on recovered bedrock core samples ranged from about 97 to 100 percent, indicating an excellent quality rock.

Photos of the bedrock core are provided in Figures 5 to 7.

The result of unconfined compressive strength testing carried out on one bedrock core sample is about 182.9 megapascals indicating a very strong bedrock and is provided in Appendix C.

4.7 Groundwater

Standpipe piezometers were sealed into boreholes 18-102, 18-105, and 18-106 to allow for subsequent measurement of the groundwater level. The groundwater levels were measured on November 16, 2018 and are summarized in the following table:

Borehole	Ground Surface Elevation (m)	Soil Strata Screened	Groundwater Depth (m)	Groundwater Elevation (m)	Date of Measurement
18-102	77.23	Sandstone Bedrock	1.56	75.67	November 16, 2018
18-105	77.03	Till/Silty Clay	1.78	75.25	November 16, 2018
18-106	77.16	Till/Silty Clay	2.20	74.96	November 16, 2018

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

5.0 DISCUSSION

5.1 General

This section of the report provides engineering guidelines on the geotechnical design aspects of the proposed development based on our interpretation of the borehole information and project requirements. Reference should be made to the “Important Information and Limitations of This Report”, which follows the text but forms part of this document.

The following guidelines are provided on the basis that the multi-storey building will be designed in accordance with Part 4 of the 2012 Ontario Building Code (OBC).

5.2 Site Grading

The subsurface conditions on this site consist of topsoil and silty sand overlying sensitive silty clay over glacial till and bedrock.

The compressibility of the silty clay deposit negatively impacts the permissible filling of this site. The silty clay deposit has limited capacity to support the combined loading from grade raise filling, foundation loads, groundwater level lowering, floor loads, etc. Overstressing of the silty clay will lead to excessive foundation settlements. For the purposes of this assessment, it has been assumed that the proposed grading will be at about the same elevation as the existing grades on site (at about the same grade as the adjacent Solandt Road). This final grade will need to be maintained for the bearing resistance values given in Section 5.4 to be applicable. Additional filling will require additional geotechnical analysis.

The topsoil and any fill containing organic matter are not suitable as engineered fill and should be removed from the site or stockpiled separately for re-use in landscaping applications only. It is important that stockpiles, if located on site, not be adjacent to excavations but rather should be located within the future landscaping areas.

In addition to the material that will be excavated within the footprint of the building for construction of the foundations, the topsoil should also be removed from beneath pavement areas.

5.3 Excavations

It is understood that the proposed building will not have a basement level founded on either shallow foundations, a raft slab foundation or deep foundations. For preliminary design purposes, it has been assumed that the base of the excavation would be at about elevation 75.7 metres.

The excavations for the raft slab foundation will be through the surficial topsoil and silty sand and into the very stiff to stiff weathered silty clay crust. Based on the borehole information, the sensitive grey silty clay is present below about elevation 75.1 metres at the building location. If the founding level is lowered below this elevation, the excavations will potentially extend into this sensitive layer, but the following guidelines would remain unchanged.

No unusual problems are anticipated with excavating the overburden using conventional hydraulic excavating equipment. If the excavations are carried out in the sensitive silty clay, it is suggested that the excavation equipment be fitted with a smooth bladed bucket (i.e., no teeth), to limit disturbance of the subgrade.

The existing silty sand and silty clay deposits in the area of the proposed building would generally be classified as Type 3 soils in accordance with the Occupational Health and Safety Act (OHSA) and therefore open cut side slopes would need to be cut back at an inclination no steeper than 1 horizontal to 1 vertical (1H:1V). For slopes which are unsupported in the longer term, and might experience freeze-thaw cycles, flatter side slope inclinations could be required.

For the 1.8 metre deep excavations required at this site, it is anticipated that open-cut excavations will generally be feasible.

Based on present groundwater levels, excavations deeper than about 1.5 metres will extend below the groundwater level. Groundwater inflow into the excavations should feasibly be handled by pumping from sumps within the excavations. Groundwater inflow from the weathered silty clay crust is expected to be low to moderate; however, the actual rate of groundwater inflow will depend on many factors including the contractor's schedule and rate of excavation, the size of the excavation, the number of working areas being excavated at one time, and the time of year at which the excavation is made. Also, there may be instances where significant volumes of precipitation, surface runoff and/or groundwater collects in an open excavation and must be pumped out.

Under the new regulations, a Permit-To-Take-Water (PTTW) is required from the Ministry of the Environment and Climate Change (MOECC) if a volume of water greater than 400,000 litres per day is pumped from the excavation. If the volume of water to be pumped will be less than 400,000 litres per day, but more than 50,000 litres per day, the water taking will not require a PTTW, but will need to be registered in the Environmental Activity and Sector Registry (EASR) as a prescribed activity. Based on the groundwater information collected during the investigation, it is considered unlikely that an EASR or PTTW would be required during construction for this project. However, the requirement for registration in the EASR is possible if inflows are greater than expected. The requirement for registration (i.e., if more than 50,000 litres per day is being pumped) can be assessed at the time of construction. Registration is a quick process that will not significantly disrupt the construction schedule.

5.4 Foundations

As discussed previously, the subsurface conditions at this site consist of topsoil and silty sand underlain by a deposit of sensitive silty clay over glacial till and bedrock. The bedrock surface is at depths ranging from about 3.7 to 7.5 metres below the existing ground surface.

Shallow spread footing foundations or a raft slab foundation could be considered provided that the bearing resistance values provided in the subsequent sections are adequate to support the loads imposed by the structure. If the loading from the structure prohibits the use of these options, deep foundations will need to be considered. The most feasible and practical deep foundation system for this building will likely be driven end-bearing steel piles.

5.4.1 Shallow Spread Footings

Shallow spread footings can be considered provided that they can be designed using the bearing resistance values provided below. For this assessment, an underside of footing elevation of about 75.7 metres has been assumed. At this elevation, the spread footings would bear on the very stiff to stiff weathered silty clay crust.

The Serviceability Limit States (SLS) bearing resistance value for spread footing foundations is based on limiting the stress increases on the firm grey silty clay to an acceptable level, so that foundation settlements do not become excessive. Important parameters in calculating the stress increase on the grey silty clay are:

- The thickness of the weathered crust below the underside of the footings
- The size (dimensions) of the footings
- The amount of surcharge in the vicinity of the foundation due to landscape fill, underslab fill, floor loads, etc.
- The amount of unloading due to the soil removed for basement construction
- The effects of groundwater lowering caused by this or other construction

The floor load for the basement floor slab has been assumed to be 4.8 kilopascals.

The maximum bearing resistance for strip footing foundations up to 2.0 metres in width may be taken as 175 kilopascals for Serviceability Limit State (SLS) and 190 kPa for the factored Ultimate Limit State (ULS). The maximum bearing resistance for pad footings up to 5 metres in width may be taken as 155 kilopascals for SLS and 210 kilopascals for the factored ULS.

The post construction total and differential settlements of footings sized using the above SLS net bearing resistance values should be less than about 25 and 15 millimetres, respectively, provided that the soil at or below founding level is not disturbed during construction. Further, these bearing resistances correspond to a settlement resulting from consolidation of the silty clay. Consolidation of the silty clay is a process which takes months or longer and, as such, results from sustained loading. Therefore, the foundation loads to be used in conjunction with the SLS resistance values given above should be the full dead load plus sustained live load. The factored dead plus full factored live load should be used in conjunction with the ULS factored bearing resistance.

The silty clay subgrade is sensitive to disturbance (such as from construction traffic) and a mud slab of lean concrete, at least 50 millimetres in thickness, should be provided on the bearing surfaces after excavation to founding level following review/approval of the bearing surface by geotechnical personnel. Excavations to expose the silty clay subgrade should be carried out using a smooth-edged excavator bucket (i.e., no teeth) to minimize disturbance.

5.4.2 Raft Slab

A foundation alternative for the proposed building at this site would be to use a ‘raft’ foundation. A raft foundation would need to be sufficiently rigid so that the building loads would be relatively uniformly distributed over the entire building footprint.

The available bearing resistance for support of the raft foundation depends on the founding level, since it impacts on both the bearing stratum and on the compensating effect of the weight of the excavated soil. For preliminary design purposes, the founding level has been assumed at about elevation 75.7 metres, which is within the weathered silty clay crust. The founding level should ideally be uniform across the footprint to limit differential settlements.

For the assumed founding level, it is considered that the raft foundation can be designed using an SLS gross contact stress of 80 kilopascals. This bearing resistance is based on maintaining the stress level in the clay deposit at a reasonable margin below the preconsolidation pressure of the clay deposit below founding level; i.e., such that the stress level in the clay will not approach or surpass its ‘yield’ stress.

The ULS factored bearing resistance that may be used for the design of the raft foundation is 150 kilopascals.

The post-construction total and differential settlements of the raft will depend, in part, upon the duration of time from when the excavation is made to when the building load is applied, since the clay will “rebound” (i.e., swell) following removal of the weight of the overlying soil. This rebound will be recovered as settlement once the structure loads are imposed on the raft. The post-construction settlements will be larger for corresponding longer lengths of time between excavating and re-loading. In addition, the clay will also undergo heave and subsequent settlement as a result of undrained distortion of the deposit.

If the bearing stress under the raft were to reach the SLS bearing resistance provided above (i.e., if the full structure weight were to equal the full available SLS bearing resistance), the *calculated total* settlement of the raft foundation is expected to be in the order of 25 to 50 millimetres (accounting for the recovered rebound and distortion settlement of the clay), depending in part upon that duration of unloading/construction and noting that the larger settlement estimate would correspond to a period of several months of full unloading.

For design purposes, it is recommended that a differential settlement of up to 70 percent of the total settlement be expected/accommodated. However, this differential settlement will also depend greatly on the stiffness of the raft; even for uniform ground conditions, the settlement of the edge of the raft would typically be less than that of the centre. If variations in the raft level are needed, such as to accommodate sloping parking levels or deeper foundation areas for elevator pits, there would be an increased potential for differential settlements.

It should also be noted that the localized differential settlements (i.e., raft slab deflections) within/beneath individual bays (such as directly beneath a column versus the mid-span of the bay) will depend upon the relative stiffness between the raft slab and the underlying subgrade. The deflections and the resulting forces and bending moments in the slab to be used in its structural design could be determined by structural analysis using a modulus of subgrade reaction, k_s, for the subgrade.

It should be noted, however, that the modulus of subgrade reaction is not a fundamental soil property and its value depends, in part, on the size and shape of the loaded area. For the analysis of the contact stress distribution beneath a raft foundation, its value would depend on the size of the areas over which increased/concentrated contact stresses are anticipated (analogous to equivalent footings beneath the columns); the size of these areas is in turn related to the value the modulus of subgrade reaction, i.e., they are inter-related.

Accordingly, the analysis of the raft slab should ideally involve an iterative analysis between the determination of the contact stress distribution by the structural engineer and the geotechnical determination of the modulus of subgrade reaction value, until the two are consistent with each other. For initial analyses, the modulus of subgrade reaction may be assumed to be about 3 megapascals per metre.

The silty clay subgrade is sensitive to disturbance (such as from construction traffic) and a mud slab of lean concrete, at least 50 millimetres in thickness, should be provided on the bearing surfaces after excavation to founding level following review/approval of the bearing surface by geotechnical personnel. Excavations to expose the silty clay subgrade should be carried out using smooth-edged excavator bucket (i.e., no teeth) to minimize disturbance.

5.4.3 Pile Foundations

A piled foundation system could be used to transfer the foundation loads through the silty clay and glacial till to more competent bearing at depth (i.e., down to the bedrock surface).

A suitable pile type would be concrete filled steel pipe piles (driven closed-ended) or H-piles, with the piles end bearing on bedrock. For this site, the piles would be driven to practical refusal on the bedrock surface at depth ranging from about 4.9 to 7.5 metres below the existing ground surface at the proposed building location (i.e., about 3.4 to 6.0 metres from the underside of the foundations).

A minimum 0.6 metre thick granular working mat should be provided for pile driving equipment to protect the silty clay subgrade.

5.4.3.1 Axial Resistance

As one possible design example, the ULS factored *structural* resistance of a 245-millimetre diameter steel pipe pile with a wall thickness of 13 millimetres may be taken as 1,400 kilonewtons. The ULS factored *geotechnical* resistance of the pile should equal or exceed the structural resistance if the piles are driven to the bedrock and are installed using an appropriate set criteria and using a hammer of sufficient energy. Note: The pile capacity/size to be used in the design may also be controlled by the dynamic testing program (see later discussion in this section).

For piles end-bearing on or within the bedrock, SLS conditions generally do not govern the design since the stresses required to induce 25 millimetres of movement (i.e., the typical SLS criteria) exceed those at ULS. Accordingly, the post-construction settlement of structural elements which derive their support from piles bearing on bedrock should be negligible.

Pipe piles should be equipped with a base plate having a thickness of at least 20 millimetres to limit damage to the pile tip during driving.

The piles should be driven no closer than three pile widths/diameters centre to centre.

The pile termination or set criteria will be dependent on the pile driving hammer type, helmet, selected pile, and length of pile; the criteria must therefore be established at the time of construction and after the piling equipment is known. All of these factors must be taken into consideration in establishing the driving criteria to ensure that the piles will have adequate capacity but are also not overdriven and damaged. In this regard, it is a generally accepted practice to reduce the hammer energy after abrupt peaking is met on the bedrock surface, and then to gradually increase the energy over a series of blows to seat the pile.

Relaxation of the piles following the initial set could result from several processes, including:

- The dissipation of negative excess pore water pressures in the overburden material above the bedrock surface
- The driving of adjacent piles

Provision should therefore be made for restriking all of the piles at least once to confirm the design set and/or the permanence of the set and to check for upward displacement due to driving adjacent piles. Piles that do not meet the design set criteria on the first restrike should receive additional restriking until the design set is met. All restriking should be performed after 48 hours of the previous set.

Some of the piles may not fully penetrate the bouldery glacial till to reach the bedrock surface; some of the piles may instead “hang up” at a shallower depth in the glacial till. In that case, pre-drilling of the glacial till could be considered, which would be costly. Alternatively, these particular piles may need to be designed for a reduced capacity. The ULS factored axial resistance of these piles will depend on the depth to which they penetrate and the set that is achieved. The capacities of these piles may have to be confirmed in the field by carrying out load testing.

Due to their smaller cross section, H-piles might have more success in penetrating the glacial till and reaching the bedrock surface. However, the integrity of pipe piles following driving may be more readily inspected (by visual examination of the pile interiors) than for H-piles, and therefore damaged piles can be more easily identified. As well, H-piles are typically more expensive. The option of using H-piles could however be discussed with the piling contractor.

It is recommended that dynamic monitoring and capacity testing (known as PDA testing) be carried out (by the contractor) at an early stage in the piling operation to verify both the transferred energy from the pile driving equipment and the load carrying capacity of the piles. As a preliminary guideline, the specification should require that at least 10 percent of the piles be included in the dynamic testing program. CASE method estimates of the capacities should be provided for all piles tested. These estimates should be provided by means of a field report on the day of testing. As well, CAPWAP analyses should be carried out for at least one third of the piles tested, with the results provided no later than one week following testing. The final report should be stamped by a professional engineer licensed in the province of Ontario.

The purpose of the PDA testing will be to confirm that the contractor's proposed set criteria is appropriate and that the required pile geotechnical capacity is being achieved. It will therefore be necessary for the pile to have sufficient structural capacity to survive that testing, which could require a stronger pile section than would otherwise be required by the design loading.

For example, for the PDA testing to be able to record/confirm a factored geotechnical resistance of 1,400 kilonewtons (per the previously indicated design example), it will be necessary to successfully proof load the tested piles to 2,800 kilonewtons during the PDA testing (per the resistance factor of 0.5 to be applied to PDA test results, as specified in Commentary K of the National Building Code of Canada). However, that proof load may exceed the actual structural capacity of the piles. If the piles fail (structurally) at a lower load, then the full geotechnical capacity cannot be confirmed (and piles will have been damaged and will need to be wasted).

The following options could therefore be considered:

1) Piles with a higher *structural* capacity could be specified (i.e., piles with a ULS factored structural resistance higher than the factored geotechnical resistance, and higher than required by the design loading), so that the piles can be successfully tested to the required loading, so that the geotechnical capacity can then be confirmed by the PDA testing. This option could significantly increase the cost of the piled foundations (due, for example, to the increased wall thickness or diameter of pile that would be used). It might be feasible to use these stronger piles only for those that will be tested, however this option would not permit random testing of the 'production' piles, as is typically part of a PDA testing program.

2) A reduced ULS factored geotechnical resistance could be used for the design (e.g., 1,000 kilonewtons instead of 1,400 kilonewtons), such that the piles would have sufficient structural capacity to be loaded to twice the design geotechnical resistance. This option would again increase the cost for the piled foundations, by increasing the number of piles that would be required.

3) Static load testing could be carried out, rather than PDA testing, to confirm the ULS geotechnical resistance of the piles, since the OBC/NBCC specify a resistance factor of 0.6 for static load tests (instead of 0.5). However, it may still not be feasible to prove the full geotechnical resistance.

The foundation and piling specifications should be reviewed by Golder Associates prior to tender and the contractor's submission (i.e., shop drawings, equipment, procedures, and set criteria) should be reviewed by the geotechnical consultant prior to the start of piling. That submission should include a WEAP (Wave Equation Analysis of Piles) analysis of the driveability of the pile, to the design depth, using the contractor's selected hammer.

Vibration monitoring should be carried out during pile installation to ensure that the vibration levels at nearby existing structures are maintained below tolerable levels. A maximum peak particle velocity of 50 millimetres per second is recommended for structures.

Piling operations should be inspected on a full time basis by geotechnical personnel to monitor the pile locations and plumbness, initial sets, penetrations on restrike, and to check the integrity of the piles following installation.

5.4.3.2 Resistance to Lateral Loading

Lateral loading could be resisted fully or partially by the use of battered piles.

Alternatively, the resistance to lateral loading could be derived from the soil resistance in front of the piles, and it may be assumed that this resistance will be nearly the same for vertical and inclined piles.

The SLS geotechnical response of the soil in front of the piles under lateral loading may be calculated using subgrade reaction theory where the coefficient of horizontal subgrade reaction, k_h, is based on the equation given below, as described by Terzaghi (1955) and the Canadian Foundation Engineering Manual (3rd Edition).

For cohesionless soils:

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

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

z = the depth (m); and,

B = the pile diameter/width (m).

For cohesive soils:

$$k_h = \frac{67 s_u}{B}$$

Where: s_u = the undrained shear strength of the soil (kPa); and,

B = the pile diameter/width (m).

The constant of horizontal subgrade reaction depends on the soil type and soil density/consistency around the pile shaft. For the design of resistance to lateral loads, the values indicated in the table below may be used. The values provided are unfactored geotechnical parameters.

Elevation (m)	Soil Type	n_h (kPa/m)	s_u (kPa)
Pile cap to 75.5	Weathered silty clay crust	-	100
75.5 to 72.0	Stiff grey silty clay	-	60
72.0 to 71.0	Compact glacial till	4,400	-

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

Pile Spacing in Direction of Loading (d = Pile Diameter)	Reduction Factor
8d	1.0
6d	0.7
4d	0.4
3d	0.25

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

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

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

For individual piles in cohesive soils (i.e., silty clay) the ULS lateral resistance is assumed to vary linearly with a magnitude of $2S_u$ at the surface of the deposit (i.e., the underside of pile cap level) and a magnitude of $9S_u$ at a depth equal to three pile diameters below the underside of the pile cap (where S_u is the previously provided undrained shear strength). Below a depth equal to 3 pile diameters, and to the bottom of the deposit, the lateral resistance is assumed to be constant at $9S_u$.

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

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

If uplift resistance is required, the piles would have some capacity which could be relied upon. Rock anchors could also be used, but the significant depth to the bedrock surface could make that an expensive option. Further geotechnical input on both issues can be provided, if required.

5.4.4 Frost Protection

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

Insulating the bearing surface with high density insulation could be considered as an alternative to earth cover for frost protection.

5.4.5 Seismic Design

The seismic design provisions of the 2012 OBC depend, in part, on the shear wave velocity of the upper 30 metres of soil and/or rock below founding level. The OBC permits the Site Class to be specified based solely on the stratigraphy and in situ testing data (i.e., shear strengths and standard penetration test results), rather than from direct measurements of the shear wave velocity.

Based on the in situ testing data, this site can be assigned a Site Class of C for seismic design purposes.

5.5 Slab on Grade

If spread footings or a piled foundation system are used, then a slab on grade concrete floor slab can be provided.

To prevent hydrostatic pressure build up beneath the slab on grade and potential groundwater infiltration, it is suggested that the granular base for the slab on grade be drained. Provision should be made for at least 300 millimetres of 6 millimetre clear crushed stone to underlie the floor. Where a concrete floor slab will be provided, the clear stone chip can form the base layer for that floor.

Any bulk fill required to raise the grade to the underside of the clear stone should consist of OPSS Granular ‘B’ Type I or II.

The underslab fill should be placed in maximum 300 millimetre thick lifts and should be compacted to at least 95 percent of the material’s standard Proctor maximum dry density using suitable vibratory compaction equipment.

Rigid 100 millimetre diameter perforated pipes should be installed within the clear stone at 6 metre centres. The perforated pipes should discharge by gravity to a positive outlet.

5.6 Foundation Wall Backfill

The soils at this site are frost susceptible and should not be used as backfill against exterior or unheated foundation elements. To avoid problems with frost adhesion and heaving, these foundation elements should be backfilled with non-frost susceptible sand or sand and gravel conforming to the requirements of OPSS Granular B Type I.

In areas where pavement or other hard surfacing will abut the proposed building, differential frost heaving could occur between the granular fill and the adjacent areas. To reduce 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 metres 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 millimetre thick lifts and should be compacted to at least 95 percent of the material’s standard Proctor maximum dry density using suitable vibratory compaction equipment.

The pavement or hard surfacing could be expected to perform better in the long term if the granular backfill against the foundation walls is drained by means of a perforated pipe subdrain in a surround of 19 millimetre clear stone, fully wrapped in a geotextile, which leads by gravity drainage to a positive outlet.

5.7 Site Servicing

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

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

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

5.8 Pavement Design

In preparation for pavement construction, all topsoil and any unsuitable fill (i.e., fill containing organic matter) should be excavated from the pavement areas for predictable pavement performance.

Those portions of the fill not containing organic matter may be left in place provided that some long term settlement of the pavement surface can be tolerated. However, the surface of the fill material at subgrade level should be proof rolled with a heavy smooth drum roller under the supervision of qualified geotechnical personnel to compact the surface of the existing fill and to identify soft areas requiring sub-excavation and replacement with more suitable fill.

Areas requiring grade raising to proposed subgrade level should be filled using acceptable (compactable and inorganic) earth borrow or OPSS Select Subgrade Material. The existing inorganic fill on site may be suitable for this purpose, but would need to be confirmed by the geotechnical engineer at the time of construction. Grade raise fill should be placed in maximum 300 millimetre thick lifts and should be compacted to at least 95 percent of the material's standard Proctor maximum dry density using suitable compaction equipment.

The surface of the subgrade or fill should be crowned to promote drainage of the pavement granular structure. Perforated pipe subdrains should be provided at subgrade level extending from the catch basins for a distance of at least 3 metres in four orthogonal directions, or longitudinally where parallel to a curb.

The pavement structure for car parking areas should consist of:

Pavement Component	Thickness (mm)
Asphaltic Concrete	50
OPSS Granular A Base	150
OPSS Granular B Type II Subbase	300

The pavement structure for access roadways and truck traffic areas should consist of:

Pavement Component	Thickness (mm)
Asphaltic Concrete	90
OPSS Granular A Base	150
OPSS Granular B Type II Subbase	450

The granular base and subbase materials should be uniformly compacted to at least 100 percent of the material's standard Proctor maximum dry density using suitable vibratory compaction equipment. The asphaltic concrete should be compacted in accordance with OPSS 310.

The composition of the asphaltic concrete pavement in car parking areas should be as follows:

- Superpave 12.5 Surface Course – 50 millimetres

The composition of the asphaltic concrete pavement in access roadways and truck traffic areas should be as follows:

- Superpave 12.5 Surface Course – 40 millimetres
- Superpave 19.0 Binder Course – 50 millimetres

The pavement design should be based on a Traffic Category of Level B. The asphalt cement used on this project should be made with PG 58-34 asphalt cement on all lifts.

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

5.9 Corrosion and Cement Type

One soil sample from test pit 18-103 was submitted to Eurofins Environment Ontario for basic chemical analysis related to potential sulphate attack on buried concrete elements and corrosion of buried ferrous elements. The results of this testing are provided in Appendix C and are summarized below.

Borehole Number / Sample Number	Sample Depth (m)	Chloride (%)	SO_4 (%)	pH	Resistivity (Ohm-cm)
18-103 / Sa 3	1.5 – 2.1	0.014	<0.01	7.21	2630

The results indicate that concrete made with Type GU Portland cement should be acceptable for substructures. The results also indicate an elevated potential for corrosion of exposed ferrous metal, which should be considered in the design of substructures.

5.10 Trees

In general, silty clay soil has the potential to be sensitive to water depletion by trees of high water demand during periods of dry weather. When trees draw water from the clayey soil, the clay undergoes shrinkage which can result in settlement of adjacent structures.

Based on the new guidelines from the City of Ottawa for tree planting within residential developments, the shrinkage potential of the silty clay is dependant, in part, on the plasticity index. The results of the Atterberg limit testing indicate that the plasticity index values for the unweathered silty clay are generally less than 40 percent (see Figure 3). Since the plasticity index is less than 40 percent, the shrinkage potential is considered low and, in that case, the tree to foundation setback distance can be reduced to 4.5 metres for small (mature tree height up to 7.5 metres) and medium sized trees (mature tree height of 7.5 to 14 metres), provided that the tree is of low to moderate water demand. Large trees (mature height greater than 14 metres) can also be considered provided that the setback distance is equal to or greater than the full mature height of the tree.

Table 1 provides a list of the common trees in decreasing order of water demand and, accordingly, decreasing risk of potential effects on structures.

6.0 ADDITIONAL CONSIDERATIONS

The soils at this site are sensitive to disturbance from ponded water, construction traffic, and frost.

All footing and subgrade areas should be inspected by experienced geotechnical personnel prior to filling or concreting to ensure that soil having adequate bearing capacity has been reached and that the bearing surfaces have been properly prepared. The placing and compaction of any engineered fill should be inspected to ensure that the materials used conform to the specifications from both a grading and compaction view point.

Ontario Regulation 903 would ultimately require abandonment of the monitoring wells installed for this investigation. However, these devices may be useful during construction. It is therefore proposed that decommissioning of these devices be made part of the construction contract.

At the time of the writing of this report, only conceptual details for the proposed development were available. Golder Associates should be retained to review the final drawings and specifications for this project prior to tendering to ensure that the guidelines in this report have been adequately interpreted.

Signature Page

Golder Associates Ltd.

LICENSED PROFESSIONAL ENGINEER
W. CAVERS
100077934
Sep 9/2019
PROVINCE OF ONTARIO

William Cavers, P.Eng.
Associate, Senior Geotechnical Engineer

WC/hdw
https://golderassociates.sharepoint.com/sites/34377g/deliverables/18111016-001-r-rev1-solandt road-sep2019.docx

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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.

TABLE 1

SOME COMMON TREES IN DECREASING ORDER OF WATER DEMAND

BROAD LEAVED DECIDUOUS

Poplar

Alder

Aspen

Willow

Elm

Maple

Birch

Ash

Beech

Oak

DECIDUOUS CONIFER

Larch

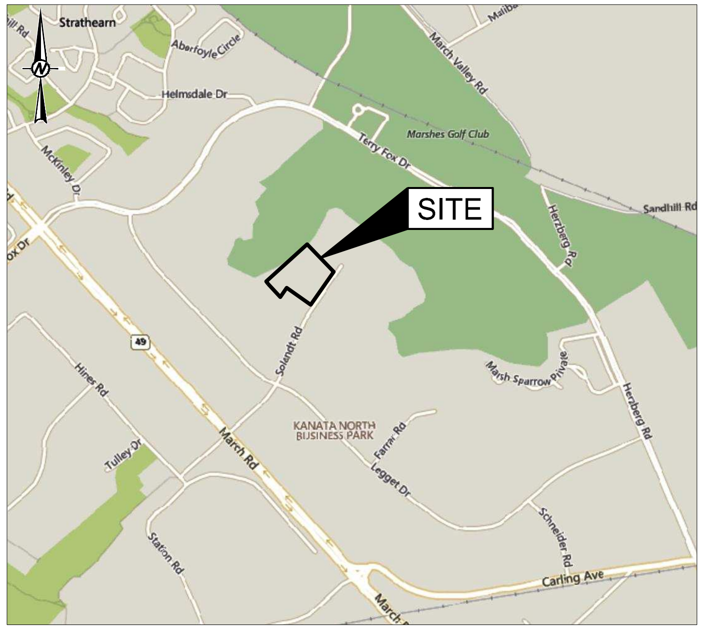
EVERGREEN CONIFERS

Spruce

Fir

Pine

KEY MAP



SCALE 1:25,000

LEGEND

⊕ APPROXIMATE BOREHOLE LOCATION

REFERENCE(S)

1. PROJECTION: TRANSVERSE MERCATOR, DATUM: NAD 83, COORDINATE SYSTEM: MTM ZONE 9, VERTICAL DATUM: CGVD28

0 25 50
1:1,250 METRES

CLIENT
KRP PROPERTIES

PROJECT
GEOTECHNICAL INVESTIGATION
PROPOSED DEVELOPMENT
2707 SOLANDT ROAD, OTTAWA, ONTARIO

TITLE
SITE PLAN

CONSULTANT

GOLDER

YYYY-MM-DD	2018-11-20
DESIGNED	----
PREPARED	JM/ZS
REVIEWED	WAM
APPROVED	WC

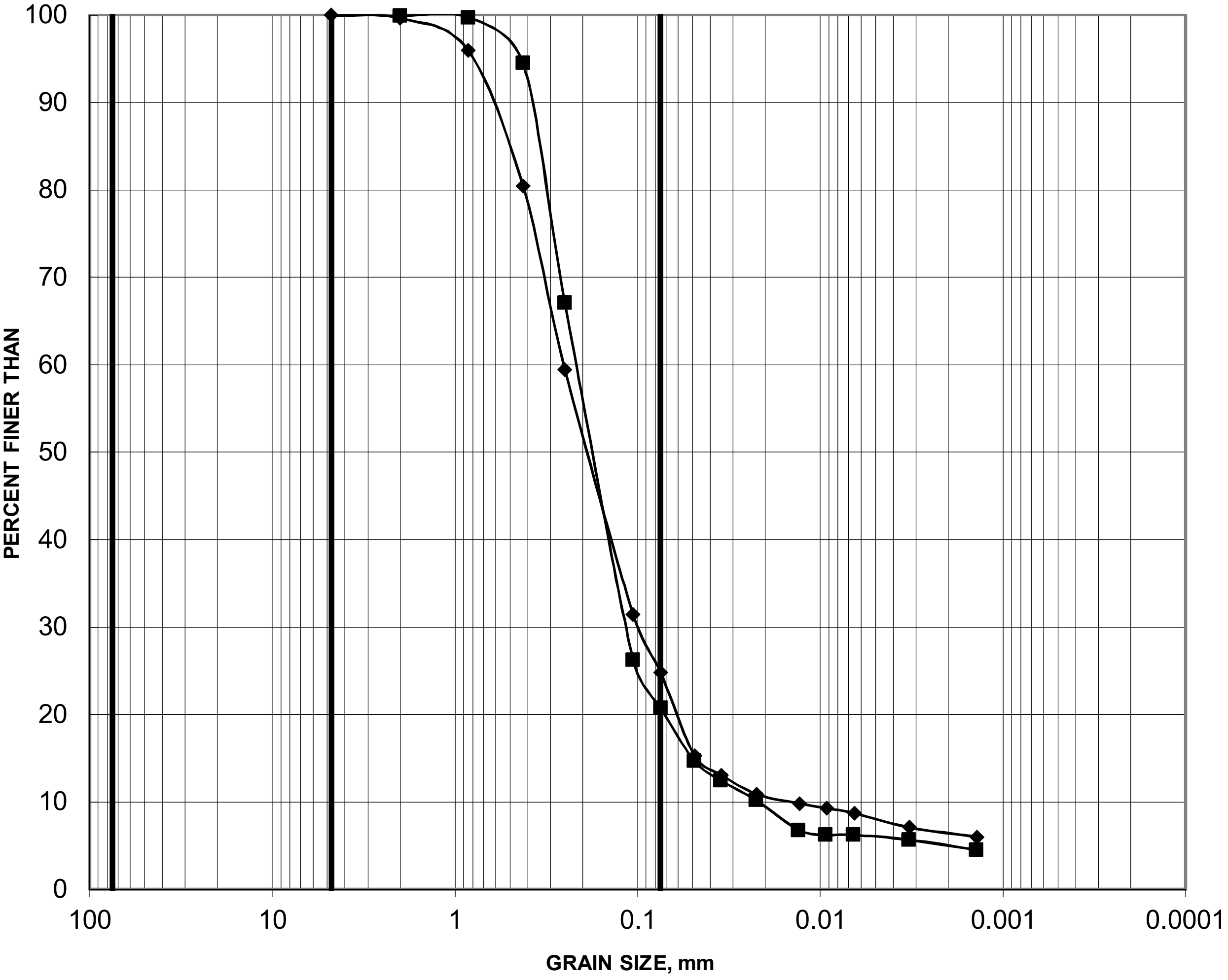
PROJECT NO.	CONTROL	REV.	FIGURE
18111016	0001	A	1

Path: \\golder.gds\gal\ottawa\Active\Spatial_IM\KRPC\2707_SolandtRd\99_PROJ\18111016_KRP_PropDev\40_PROD\0001_GeotechInvestigation\ | File Name: 18111016-0001-BG-0001.dwg | Last Edited By: zsauve Date: 2019-01-14 Time:1:13:26 PM | Printed By: ZSauve Date: 2019-01-14 Time:1:14:20 PM

0 25 mm IF THIS MEASUREMENT DOES NOT MATCH WHAT IS SHOWN, THE SHEET SIZE HAS BEEN MODIFIED FROM: ANSI B

GRAIN SIZE DISTRIBUTION	FIGURE 2

(SM) SILTY SAND



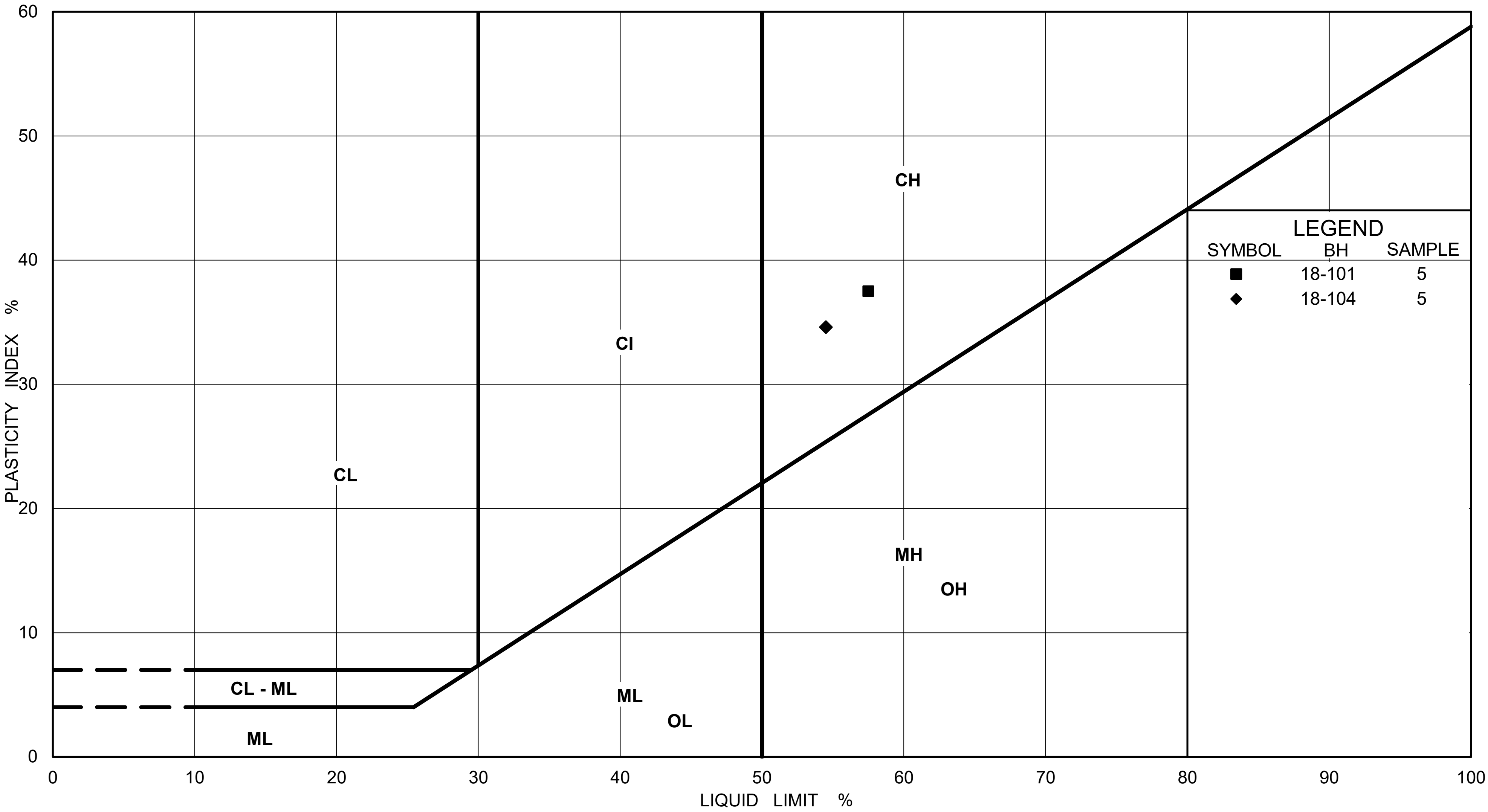
Cobble	coarse	fine	coarse	medium	fine	SILT AND CLAY
Size	GRAVEL SIZE		SAND SIZE			

Borehole	Sample	Depth (m)
■ 18-101	2	0.76-1.37
◆ 18-104	2	0.76-1.37

Project: 18111016

Golder Associates

Created by: MI
Checked by: CW



GOLDER

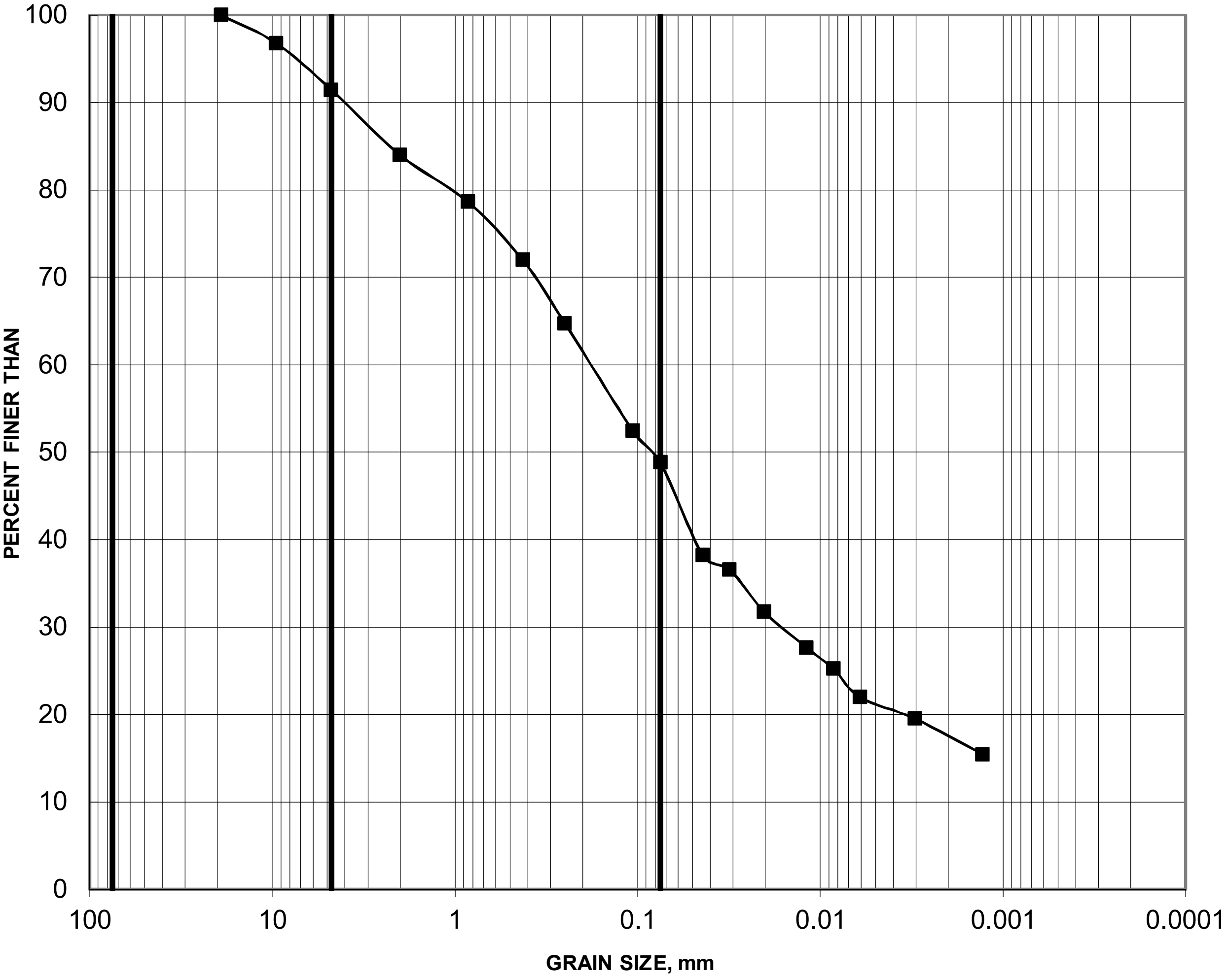
PLASTICITY CHART
SILTY CLAY TO CLAY

Figure : 3
Project No.: 18111016
Compiled By : MI Checked By : CW

GRAIN SIZE DISTRIBUTION

FIGURE 4

GLACIAL TILL



Cobble	coarse	fine	coarse	medium	fine	SILT AND CLAY
Size	GRAVEL SIZE		SAND SIZE			

Borehole	Sample	Depth (m)
—■— 18-103	6	6.10-6.71

Project: 18111016

Golder Associates

Created by: MI

Checked by: CW

BH 18-102 (Wet)
Cored Length of 6.20 to 7.67 metres
Core Box 1 of 1

6.20 m Top of Bedrock

7.67 m EOH

GOLDER

Geotechnical Investigation

2707 Solandt Road

Ottawa, Ontario

Project No.	18111016
Drawn:	WAM
Date:	06/12/2018
Checked:	
Review:	

Figure 5

BH 18-103 (Wet)
Cored Length of 7.53 to 9.05 metres
Core Box 1 of 1

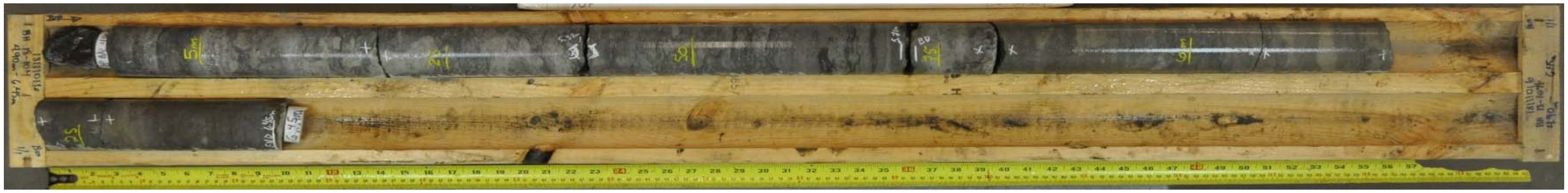
7.53 m Top of Bedrock

9.05 m EOH

GOLDER	**Geotechnical Investigation** **2707 Solandt Road** **Ottawa, Ontario**	Project No. 18111016 Drawn: WAM Date: 06/12/2018 Checked: Review:	**Figure 6**

BH 18-104 (Wet)
Cored Length of 4.90 to 6.45 metres
Core Box 1 of 1

4.90 m Top of Bedrock



6.45 m EOH

Geotechnical Investigation

2707 Solandt Road

Ottawa, Ontario

GOLDER

Project No.	18111016
Drawn:	WAM
Date:	06/12/2018
Checked:	
Review:	

Figure 7

APPENDIX A

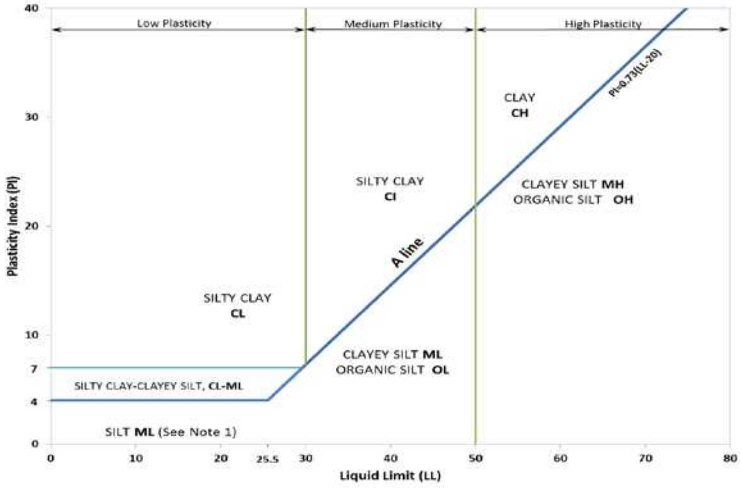
List of Abbreviations and Symbols
Record of Borehole and Drillhole Logs

METHOD OF SOIL CLASSIFICATION

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

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

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



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

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

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

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

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

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

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

ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES AND TEST PITS

PARTICLE SIZES OF CONSTITUENTS

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

MODIFIERS FOR SECONDARY AND MINOR CONSTITUENTS

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

PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

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

Cone Penetration Test (CPT)

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

Dynamic Cone Penetration Resistance (DCPT); N_d:

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

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

SAMPLES

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

SOIL TESTS

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

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

NON-COHESIVE (COHESIONLESS) SOILS

Compactness[2]

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

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

Field Moisture Condition

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

COHESIVE SOILS

Consistency

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

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

Water Content

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

LIST OF SYMBOLS

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

I. GENERAL

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

II. STRESS AND STRAIN

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

III. SOIL PROPERTIES

(a) Index Properties

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

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

(a) Index Properties (continued)

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

(b) Hydraulic Properties

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

(c) Consolidation (one-dimensional)

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

(d) Shear Strength

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

Notes: 1 $\tau = c' + \sigma' \tan \phi'$
2 shear strength = (compressive strength)/2

PROJECT: 18111016-1000

RECORD OF BOREHOLE: 18-101

SHEET 1 OF 1

LOCATION: N 5023240.0 ;E 350737.9

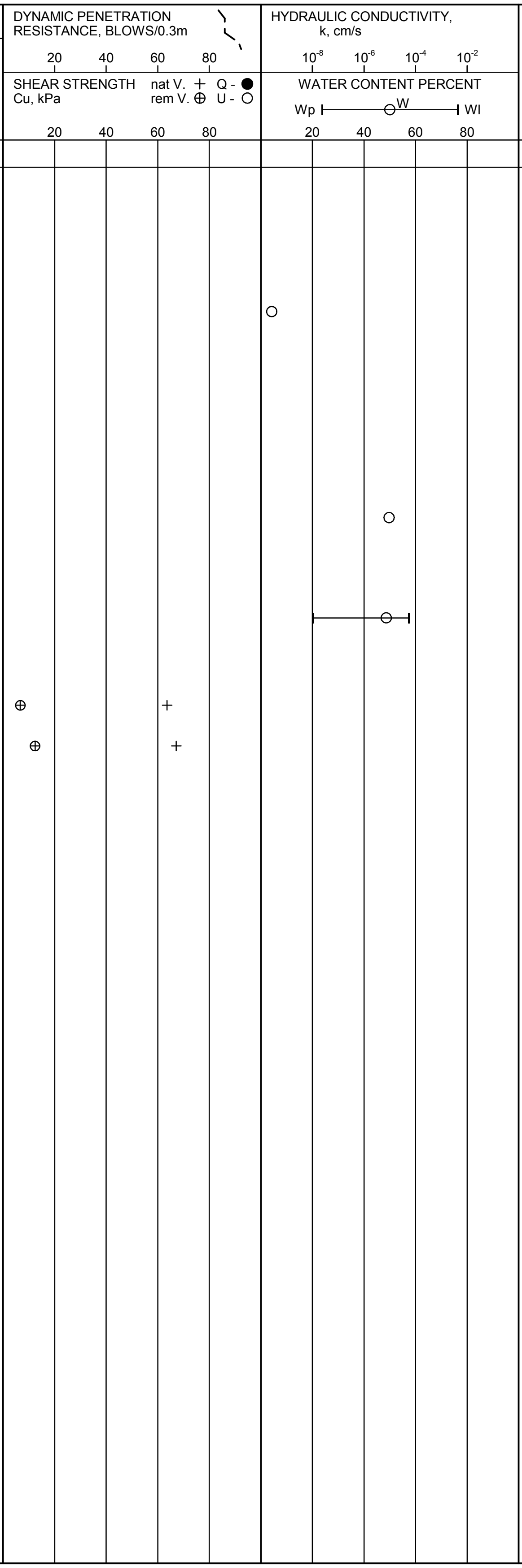
BORING DATE: November 5, 2018

DATUM: CGVD28

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.30m	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		GROUND SURFACE	77.35					
0	Power Auger 200 mm Diam. (Hollow Stem)	TOPSOIL - (SM) SILTY SAND; brown	0.00					
		(SM) SILTY SAND; red brown; non-cohesive, moist, very loose to compact	77.10 0.25	1	SS	3		
1				2	SS	13	MH	
		(CI/CH) SILTY CLAY to CLAY; grey brown (WEATHERED CRUST); cohesive, w>PL, very stiff to stiff	75.83 1.52	3	SS	6		
2		(CI/CH) SILTY CLAY to CLAY; grey; cohesive, w>PL, stiff	75.07 2.28	4	SS	2		
3				5	SS	WH		
4								
		(SM/ML) SILTY SAND to sandy SILT, some gravel; grey, contains clayey silt seams (GLACIAL TILL); non-cohesive, wet, very dense	72.78 4.57	6	SS	>50		
5		End of Borehole Auger Refusal	72.48 4.87					



DEPTH SCALE

1 : 50

LOGGED: DJG/RK

CHECKED: WAM

PROJECT: 18111016-1000

RECORD OF BOREHOLE: 18-102

SHEET 1 OF 2

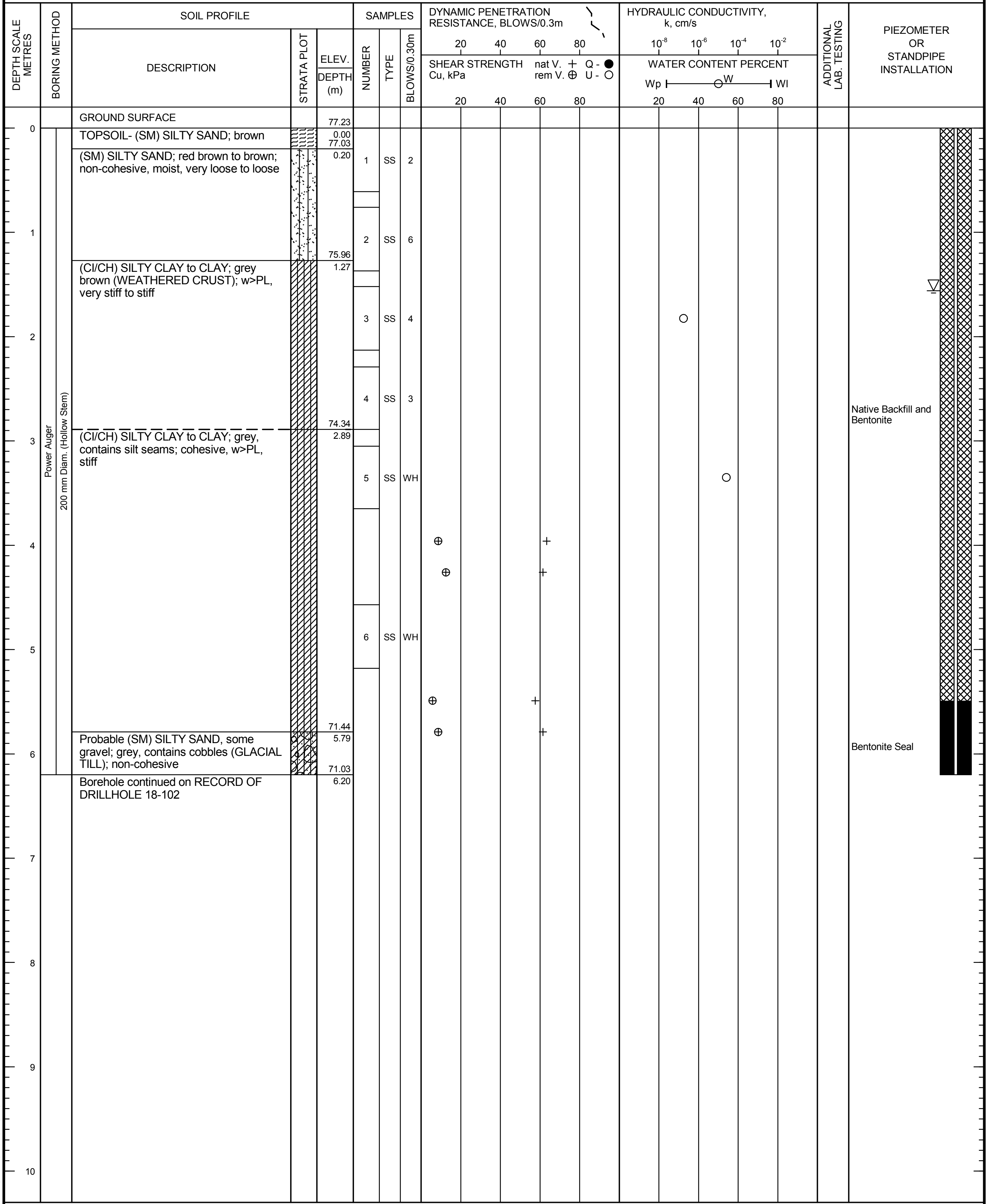
LOCATION: N 5023196.9 ;E 350745.2

BORING DATE: November 7, 2018

DATUM: CGVD28

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm



DESCRIPTION	ELEV. / DEPTH (m)
GROUND SURFACE	77.23
TOPSOIL- (SM) SILTY SAND; brown	0.00
(SM) SILTY SAND; red brown to brown; non-cohesive, moist, very loose to loose	77.03 0.20
(CI/CH) SILTY CLAY to CLAY; grey brown (WEATHERED CRUST); w>PL, very stiff to stiff	75.96 1.27
(CI/CH) SILTY CLAY to CLAY; grey, contains silt seams; cohesive, w>PL, stiff	74.34 2.89
Probable (SM) SILTY SAND, some gravel; grey, contains cobbles (GLACIAL TILL); non-cohesive	71.44 5.79
Borehole continued on RECORD OF DRILLHOLE 18-102	71.03 6.20

NUMBER	TYPE	BLOWS/0.30m
1	SS	2
2	SS	6
3	SS	4
4	SS	3
5	SS	WH
6	SS	WH

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DEPTH SCALE

1 : 50

GOLDER

LOGGED: DJG/RK

CHECKED: WAM

PROJECT: 18111016-1000

RECORD OF DRILLHOLE: 18-102

SHEET 2 OF 2

LOCATION: N 5023196.9 ;E 350745.2

DRILLING DATE: November 7, 2018

DATUM: CGVD28

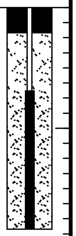
INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 75

DRILLING CONTRACTOR: CCC Drilling

JN - Joint FLT - Fault SHR- Shear VN - Vein CJ - Conjugate	BD- Bedding FO- Foliation CO- Contact OR- Orthogonal CL - Cleavage	PL - Planar CU- Curved UN- Undulating ST - Stepped IR - Irregular	PO- Polished K - Slickensided SM- Smooth Ro - Rough MB- Mechanical Break	BR - Broken Rock **NOTE:** For additional abbreviations refer to list of abbreviations & symbols.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	FLUSH COLOUR % RETURN	RECOVERY TOTAL CORE % 80 60 40 20	RECOVERY SOLID CORE % 80 60 40 20	R.Q.D. % 80 60 40 20	FRACT. INDEX PER 0.25 m 5 10 15 20	DISCONTINUITY DATA DIP w.r.t. CORE AXIS 0 30 60 90	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	Jcon	Jr	Ja	HYDRAULIC CONDUCTIVITY K, cm/sec 10^{-8} 10^{-6} 10^{-4} 10^{-2}	Diametral Point Load Index (MPa) 2 4 6	RMC -Q' AVG.	
		BEDROCK SURFACE		71.03 6.20															
7	Rotary Drill HQ3 Core	Fresh, thinly to medium bedded, grey, fine to medium grained, non-porous, very strong SANDSTONE			1														Bentonite Seal Silica Sand Standpipe
		End of Drillhole		69.56 7.67															
8																			WL in Standpipe at Elev. 75.67 m on Nov. 16, 2018
9																			
10																			
11																			
12																			
13																			
14																			
15																			
16																			



DEPTH SCALE

1 : 50

GOLDER

LOGGED: DJG/RK

CHECKED: WAM

PROJECT: 18111016-1000

RECORD OF BOREHOLE: 18-103

SHEET 1 OF 2

LOCATION: N 5023202.5 ;E 350791.8

BORING DATE: November 7, 2018

DATUM: CGVD28

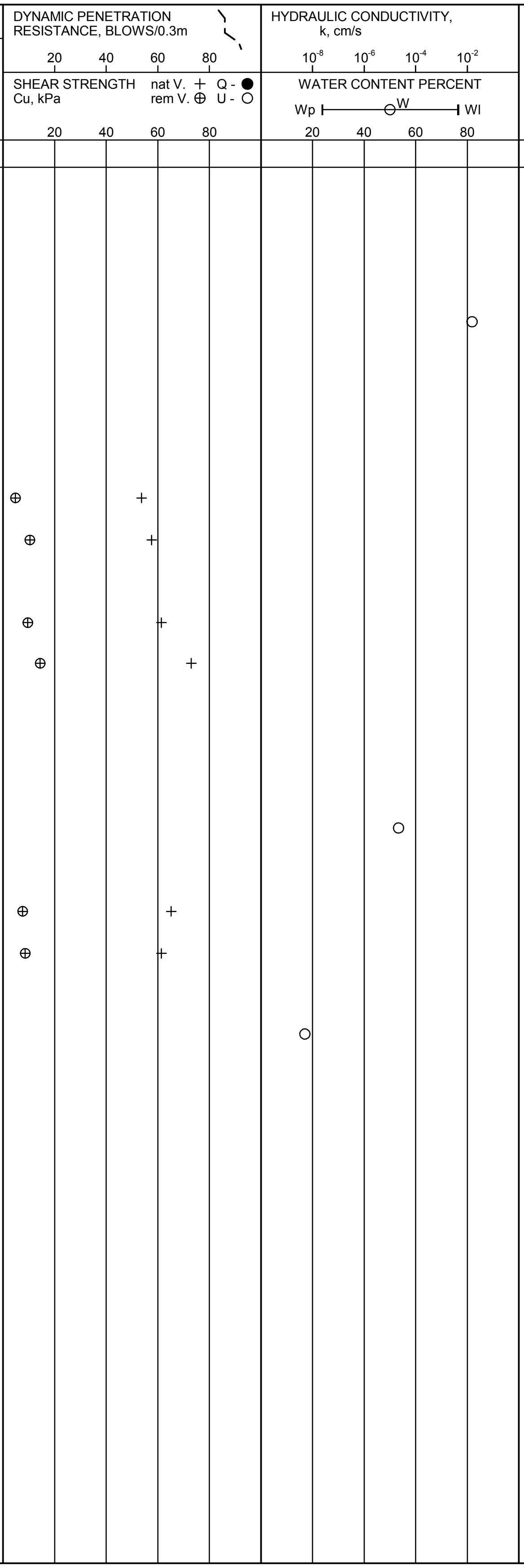
SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

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

DESCRIPTION	ELEV. DEPTH (m)
GROUND SURFACE	77.18
TOPSOIL - (SM) SILTY SAND; brown	0.00
(SM) SILTY SAND; brown; non-cohesive, moist, loose	0.15
(CI/CH) SILTY CLAY to CLAY; grey brown (WEATHERED CRUST); cohesive, w>PL, very stiff to stiff	76.27 0.91
(CI/CH) SILTY CLAY to CLAY; grey; cohesive, w>PL, stiff	75.05 2.13
(SM/ML) SILTY SAND to sandy SILT, some gravel to gravelly; grey, contains cobbles and boulders (GLACIAL TILL); non-cohesive, very loose to very dense	71.09 6.09
Borehole continued on RECORD OF DRILLHOLE 18-103	69.65 7.53

SAMPLES NUMBER	TYPE	BLOWS/0.30m	ADDITIONAL LAB. TESTING
1	SS	7	
2	SS	7	
3	SS	3	
4	SS	WH	
5	SS	WH	
6	SS	2	MH
7	SS	>50	



PIEZOMETER OR STANDPIPE INSTALLATION

DEPTH SCALE METRES: 0 1 2 3 4 5 6 7 8 9 10

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DEPTH SCALE
1 : 50

GOLDER

LOGGED: DJG/RK
CHECKED: WAM

PROJECT: 18111016-1000

RECORD OF DRILLHOLE: 18-103

SHEET 2 OF 2

LOCATION: N 5023202.5 ;E 350791.8

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: November 7, 2018

DRILL RIG: CME 75

DRILLING CONTRACTOR: CCC Drilling

DATUM: CGVD28

JN - Joint	BD - Bedding	PL - Planar	PO - Polished	BR - Broken Rock
FLT - Fault	FO - Foliation	CU - Curved	K - Slickensided	**NOTE:** For additional abbreviations refer to list of abbreviations & symbols.
SHR - Shear	CO - Contact	UN - Undulating	SM - Smooth	
VN - Vein	OR - Orthogonal	ST - Stepped	Ro - Rough	
CJ - Conjugate	CL - Cleavage	IR - Irregular	MB - Mechanical Break	

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	COLOUR % RETURN FLUSH	RECOVERY TOTAL CORE % 80 60 40 20	RECOVERY SOLID CORE % 80 60 40 20	R.Q.D. % 80 60 40 20	FRACT. INDEX PER 0.25 m 5 10 15 20	DISCONTINUITY DATA DIP w.r.t. CORE AXIS 0 30 60 90	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	Jcon	Jr	Ja	HYDRAULIC CONDUCTIVITY K, cm/sec 10^{-8} 10^{-6} 10^{-4} 10^{-2}	Diametral Point Load Index (MPa) 2 4 6	RMC -Q' AVG.
		BEDROCK SURFACE		69.65														
8 9	Rotary Drill HQ3 Core	Fresh, thinly to medium bedded, grey, fine to medium grained, non-porous, very strong SANDSTONE		7.53 68.13	1	100												
		End of Drillhole		9.05														
10																		
11																		
12																		
13																		
14																		
15																		
16																		
17																		

DEPTH SCALE

1 : 50

LOGGED: DJG/RK

CHECKED: WAM

PROJECT: 18111016-1000

RECORD OF BOREHOLE: 18-104

SHEET 1 OF 2

LOCATION: N 5023266.5 ;E 350723.5

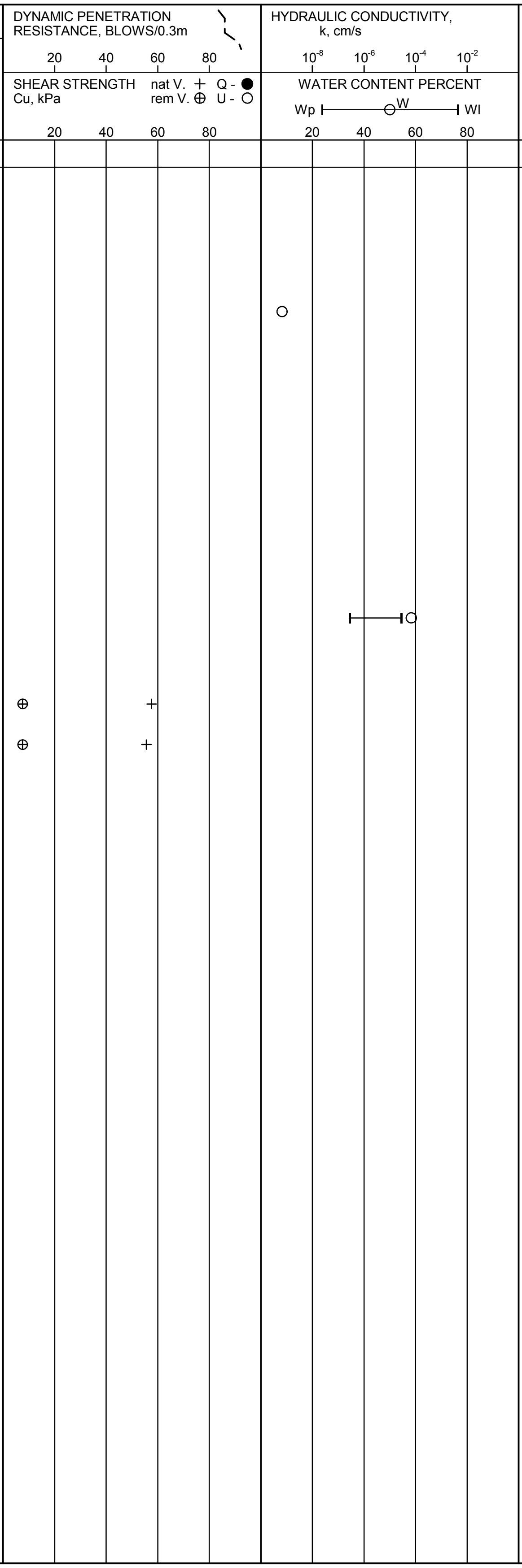
BORING DATE: November 6, 2018

DATUM: CGVD28

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES BLOWS/0.30m	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		GROUND SURFACE		77.36					
0	Power Auger 200 mm Diam. (Hollow Stem)	TOPSOIL - (SM) SILTY SAND; dark brown		0.00					
		(SM) SILTY SAND; grey brown; non-cohesive, moist, loose		77.11 0.25	1	SS	7		
1					2	SS	9	MH	
		(CI/CH) SILTY CLAY to CLAY; grey brown (WEATHERED CRUST); cohesive, w>PL, very stiff to stiff		75.69 1.67	3	SS	7		
2					4	SS	3		
3		(CI/CH) SILTY CLAY to CLAY; grey, contains silt seams; cohesive, w>PL, stiff		74.47 2.89	5	SS	2		
4					6	SS	WH		
5		Borehole continued on RECORD OF DRILLHOLE 18-104		72.46 4.90					
6									
7									
8									
9									
10									



DEPTH SCALE

1 : 50

LOGGED: DJG/RK

CHECKED: WAM

PROJECT: 18111016-1000

RECORD OF DRILLHOLE: 18-104

SHEET 2 OF 2

LOCATION: N 5023266.5 ;E 350723.5

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: November 6, 2018

DRILL RIG: CME 75

DRILLING CONTRACTOR: CCC Drilling

DATUM: CGVD28

JN - Joint
FLT - Fault
SHR- Shear
VN - Vein
CJ - Conjugate

BD- Bedding
FO- Foliation
CO- Contact
OR- Orthogonal
CL - Cleavage

PL - Planar
CU- Curved
UN- Undulating
ST - Stepped
IR - Irregular

PO- Polished
K - Slickensided
SM- Smooth
Ro - Rough
MB- Mechanical Break

BR - Broken Rock

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

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	COLOUR % RETURN FLUSH	RECOVERY TOTAL CORE % 80 60 40 20	RECOVERY SOLID CORE % 80 60 40 20	R.Q.D. % 80 60 40 20	FRACT. INDEX PER 0.25 m 5 10 15 20	DISCONTINUITY DATA DIP w.r.t. CORE AXIS 0 30 60 90	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	Jcon	Jr	Ja	HYDRAULIC CONDUCTIVITY K, cm/sec 10^{-8} 10^{-6} 10^{-4} 10^{-2}	Diametral Point Load Index (MPa) 2 4 6	RMC -Q' AVG.
		BEDROCK SURFACE		72.46														
5	Rotary Drill HQ3 Core	Fresh, thinly to medium bedded, grey fine to medium grained, non-porous, very strong SANDSTONE, with thinly interbedded shale		4.90	1	100												
6				70.91														
		End of Drillhole		6.45														
7																		
8																		
9																		
10																		
11																		
12																		
13																		
14																		

DEPTH SCALE
1 : 50

GOLDER

LOGGED: DJG/RK
CHECKED: WAM

PROJECT: 18111016-1000

RECORD OF BOREHOLE: 18-105

SHEET 1 OF 1

LOCATION: N 5023242.5 ;E 350674.9

BORING DATE: November 5, 2018

DATUM: CGVD28

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

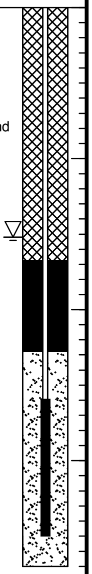
DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m: 20 40 60 80

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

HYDRAULIC CONDUCTIVITY, k, cm/s: 10^{-8} 10^{-6} 10^{-4} 10^{-2}

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

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.30m	WATER CONTENT PERCENT	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		GROUND SURFACE	77.03						
0	Power Auger 200 mm Diam. (Hollow Stem)	TOPSOIL - (SM) SILTY SAND; dark brown	0.00						
		(SM-SP) SILTY SAND to SAND, some low-plasticity fines; grey brown; non-cohesive, moist, very loose to compact	76.73 0.30	1	SS	3			
1				2	SS	15			Native Backfill and Bentonite
		(CI/CH) SILTY CLAY to CLAY; grey brown (WEATHERED CRUST); cohesive, w>PL, very stiff to stiff	75.81 1.22						∇
		(CI/CH) SILTY CLAY to CLAY; grey, contains silt seams; cohesive, w>PL, stiff	75.51 1.52	3	SS	6	○		
2									Bentonite Seal
				4	SS	4	○		Silica Sand
3									Standpipe
		(SM/ML) SILTY SAND to sandy SILT, some gravel to gravelly; grey, contains clayey silt seams and cobbles (GLACIAL TILL); non-cohesive, wet, compact	73.68 3.35	5	SS	13			Silica Sand
		End of Borehole Auger Refusal	73.33 3.70						
4									WL in Standpipe at Elev. 75.25 m on Nov. 16, 2018
5									
6									
7									
8									
9									
10									



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DEPTH SCALE

1 : 50

GOLDER

LOGGED: DJG/RK

CHECKED: WAM

PROJECT: 18111016-1000

RECORD OF BOREHOLE: 18-106

SHEET 1 OF 1

LOCATION: N 5023250.2 ;E 350784.3

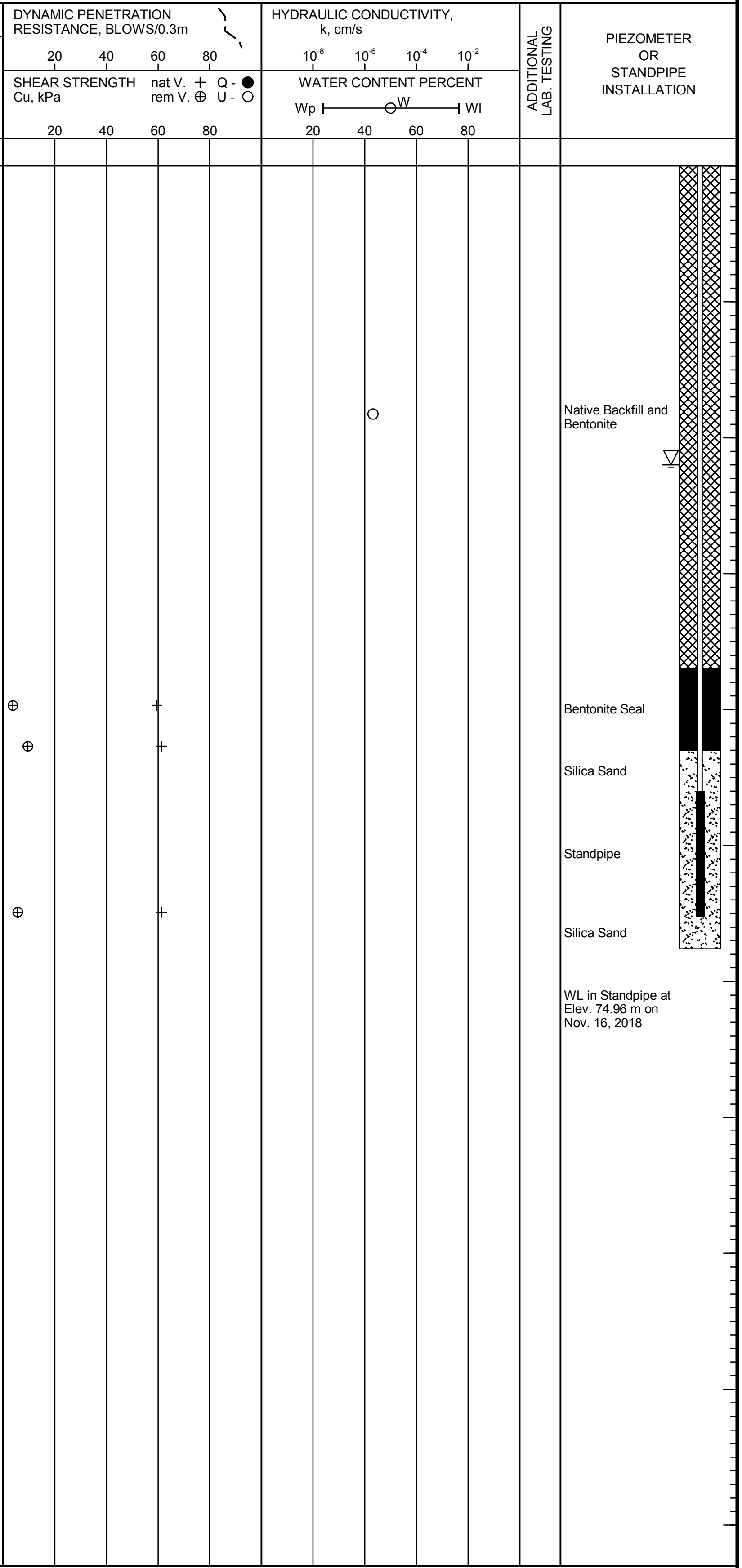
BORING DATE: November 5, 2018

DATUM: CGVD28

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.30m
		GROUND SURFACE	77.16			
0	Power Auger 200 mm Diam. (Hollow Stem)	TOPSOIL - (SM) SILTY SAND; dark brown	0.00			
		(SM) SILTY SAND; brown, contains silt seams; non-cohesive, moist, loose	76.96 0.20	1	SS	9
1		(CI/CH) SILTY CLAY to clay; grey brown (WEATHERED CRUST); cohesive, w>PL, very stiff to stiff	76.20 0.96	2	SS	9
				3	SS	5
2				4	SS	3
3		(CI/CH) SILTY CLAY to CLAY; grey; cohesive, w>PL, stiff	74.27 2.89	5	SS	2
4						
5				6	SS	WH
		(SM/ML) gravelly SILTY SAND to sandy SILT; grey, contains clayey silt seams (GLACIAL TILL); non-cohesive, wet, very dense	71.53 5.63	7	SS	>50
6		End of Borehole Auger Refusal	5.76			
7						
8						
9						
10						



DEPTH SCALE
1 : 50

GOLDER

LOGGED: DJG/RK
CHECKED: WAM

APPENDIX B

Basic Chemical Analysis
Eurofins Environment Report Number 1821309

Certificate of Analysis

Client: Golder Associates Ltd. (Ottawa)
1931 Robertson Road
Ottawa, ON
K2H 5B7
Attention: Mr. Alex Meacoe
PO#:
Invoice to: Golder Associates Ltd. (Ottawa)

Report Number: 1821309
Date Submitted: 2018-11-23
Date Reported: 2018-11-29
Project: 18111016 ph.1000
COC #: 838275

				Lab I.D. Sample Matrix Sample Type Sampling Date Sample I.D.	1401021 Soil 2018-11-07 18103 SA3/5-7
Group	**Analyte**	**MRL**	**Units**	**Guideline**	
Anions	Cl	0.002	%		0.014
	SO4	0.01	%		<0.01
General Chemistry	Electrical Conductivity	0.05	mS/cm		0.38
	pH	2.00			7.21
	Resistivity	1	ohm-cm		2630

Guideline = *** = Guideline Exceedence**

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

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

APPENDIX C

Results of Unconfined Compressive Strength Testing

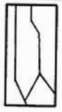
Golder Associates Ltd.
1931 Robertson Road
Ottawa, Ontario
K2H 5B7

UNCONFINED COMPRESSIVE STRENGTH OF ROCK CORE

Project: KRP Properties Kinaxis Building **Project No.:** 18111016

Date: December 4, 2018

Location(s): See Table Below

Bore Hole No.	Depth (m)	Date Tested	Core Size	Diameter (mm)	Density (kg/m³)	Compressive Strength (MPa)	Failure Mode
18-103	8.29-8.43	Nov 29/18	HQ	60.5	2667	182.9	

REMARKS :
- Cores tested in vertical direction.
- Cores tested in air-dry condition.
- Specimen ends prepared with high-strength plaster, but un-restrained.
- L/D ratio's between 2.0:1 and 2.5:1
- Time to failure > 2 and < 15 minutes.
- This report constitutes a testing service only. Interpretation of results will be provided on request only.

TESTING WAS CARRIED OUT IN GENERAL ACCORDANCE WITH ASTM D7012 - Method C

SIGNED: Melanie Ireland

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golder.com