GOLDER

REPORT

Geotechnical Investigation

Proposed Residential Buildings
100 Bayshore Drive
Ottawa, Ontario

Submitted to:

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1.0 INTRODUCTION

This report presents the results of a geotechnical investigation carried out at the site of a proposed high-rise development at 100 Bayshore Drive in Ottawa, Ontario

The purpose of this geotechnical 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 soil, bedrock, and groundwater 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

Plans are being prepared for a high-rise residential development to be located at 100 Bayshore Drive in Ottawa, Ontario. The project limits and the location of the proposed development are shown on Figure 1.

The site of the proposed development is bordered to the north by Woodridge Crescent, to the south by a bus station and Transitway, to the west by vacant land, and to the east by Bayshore Mall. The site is currently vacant and is roughly rectangular in shape; measuring about 70 m x 125 m in maximum dimension.

The proposed residential development consists of 30-storey (called hereafter "East or Phase 1 Tower") and 27-storey ("West or Phase 2 Tower") buildings which are to be connected by a three-storey parking podium with a rooftop amenity space. One level of underground parking will be located underneath each of the Towers and the podium structure. The development will also include landscaped areas, a small above-ground parking area, roadways, and a fire route with a total developed area of about 6,700 m^2.

Golder Associates (Golder) carried out a geotechnical investigation for the adjacent multi-storey parking garage (i.e., West Parking) of the Bayshore Shopping Mall. The results of that investigation were provided in the following report:

- Report to Ivanhoé Cambridge titled "*Geotechnical Investigation, Proposed West Parking Structure Reconstruction, Bayshore Shopping Centre, 100 Bayshore Drive, Ottawa, Ontario*" dated November 2015 (Report Number 1536735).

Based on a review of the published geological mapping and results of the previous investigation, the subsurface conditions at this site are expected to consist of fill overlying native layered deposits of silty clay to clayey silt, sand, silty sand/sandy silt and sand and gravel, with the bedrock surface at about 29 to 35 m depth. The bedrock is mapped to be dolostone with shale and limestone interbeds of the Upper Rockcliffe formation. This interpretation is generally consistent with the results of the current investigation.

3.0 PROCEDURE

The field work for the current geotechnical investigation was carried out between June 10 and July 13, 2020. During that time, seven boreholes (numbered 20-01 to 20-08) were advanced at the approximate locations shown on Figure 1.

The boreholes were advanced using a track-mounted drill rig supplied and operated by CCC Geotechnical & Environmental Drilling of Ottawa, Ontario. The top portion of the boreholes (i.e., about 5 m depth) were drilled using hollow stem augers for Phase 2 ESA assessment/soil sampling, followed by wash boring, with bentonite slurry as needed, for the deeper portions of the boreholes. Boreholes 20-03, 20-04 and 20-05 were advanced within/near the footprint of East Tower, while boreholes 20-01, 20-02 and 20-07 were advanced within the footprint of West Tower. The “deep” boreholes, (20-01 to 20-05, inclusive, and 20-07) were advanced to depths varying from 35.8 to 44.0 m below the existing ground surface. Two “short” boreholes (20-06 and 20-08) were advanced to depths of about 5.2 and 4.4 m, respectively. Refusal to wash boring was encountered at depths of 33.5 to 35.9 m in all of the “deep” boreholes. Upon encountering refusal, all “deep” boreholes were advanced an additional 1.8 to 8.0 m into the bedrock using rotary diamond drilling techniques while retrieving HQ3 sized core.

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 35 mm inside diameter split-spoon sampling equipment. In-situ vane testing was carried out, where possible, in the clayey silt to silty clay deposit to measure the undrained shear strength of this soil unit.

Standpipe piezometers (single or multi-level) were installed in boreholes 20-01, 20-02, 20-04, 2-06 and 20-08 to allow for subsequent monitoring of groundwater levels at the site. The standpipes consist of 32 mm inside diameter rigid PVC pipe with up to a 3.0 m long slotted screen section, installed within silica sand backfill and sealed by a section of bentonite pellet backfill. Groundwater levels were measured on August 10, 2020.

At borehole 20-05, a 60 mm inside diameter rigid PVC casing was grouted for the full advancement depth (i.e., through the overburden and into the bedrock) to allow for Vertical Seismic Profile (VSP) testing to support the selection of a seismic Site Class for the site and the assessment of liquefaction potential.

Seismic Cone Penetration Testing (SCPTu) with pore pressure dissipation and shear wave velocity measurements were carried out at two additional test hole locations (CPT20-01, CPT20-01B and CPT20-02) within the footprint of the East and West Towers, respectively. The testing was carried out by Conetec Investigations Ltd. of Richmond Hill, Ontario. Test hole 20-01 was advanced to a refusal depth of about 14.4 m and subsequently the SCPTu testing was repeated (20-01B) at an alternative location (i.e., 5 m to the south of the initial location) to a refusal depth of about 11.7 m. Test hole 20-02 was advanced to a refusal depth of about 24.5 m. Shear wave velocity testing was carried out as part of seismic cone penetration testing. A built-in geophone within the cone penetration probe recorded seismic wave traces from a surface source as the CPTs were advanced. Measurements were recorded at roughly one-meter intervals for the full depth. A more detailed description of the test methodology is provided in the Conetec report in Appendix G.

The fieldwork was supervised by technicians from our 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 measurement, grain size distribution, fine content, and Uniaxial Compressive Strength (UCS) testing of selected bedrock samples.

Two samples of soil from boreholes 20-04 and 20-07 were submitted to Eurofins Environment Testing for basic chemical analysis related to potential sulphate attack on buried concrete elements and corrosion of buried ferrous elements.

The borehole and SCPTu locations were marked in the field and surveyed by Golder. The positions and ground surface elevations at the borehole and test hole 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 18) coordinate system. The elevations are referenced to Geodetic datum (CGVD28).

4.0 SUBSURFACE CONDITIONS

4.1 General

The following information on the subsurface conditions is provided in this report:

- Record of Borehole Sheets from the current investigation is provided in Appendix A.
- Record of Borehole Sheets from the previous investigation is provided in Appendix B.
- Laboratory test results for the current investigation are provided in Appendix C, and on the relevant borehole records. In addition, the variation of SPT "N" values and of the UCS values with depth are presented on Figures C-18 and C-19, respectively, in Appendix C.
- Rock core photographs are provided in Appendix D.
- Results of the basic chemical analyses are provided in Appendix E.
- Results of geophysical testing are provided in Appendix F.
- The results of SCPTu testing are provided in Appendix G.

In general, the subsurface conditions at this site consist of fill underlain by a deposit of clayey silt to silty clay, overlaying a layered deposit of silt underlain by a thick and compact to dense deposit of sands which is in turn underlain by a dense to very dense sand and gravel deposit over dolomite bedrock.

The following sections present a more detailed overview of the subsurface conditions encountered during the field investigation.

4.2 Topsoil

Topsoil exists at the ground surface at all of the borehole locations, with the exceptions of 20-04 and 20-06. The thickness of the topsoil ranges from about 150 to 250 mm. The topsoil generally consists of dark brown silty sand with organic matter.

4.3 Fill

Fill was encountered below the topsoil at all of the borehole locations, with the exceptions of boreholes 20-04 and 20-06 where the fill was encountered at the ground surface. The fill consists of gravelly sand to gravelly silty sand, silty clay to clayey silt, and sand and gravel. The fill extends to depths ranging between about 0.8 and 2.4 m below the existing ground surface.

The results of SPT tests carried out within the fill gave 'N' values ranging from 10 to 46 blows per 0.3 m of penetration, indicating a loose to very dense state of packing.

4.4 Clayey Silty to Silty Clay

Clayey silt to silty clay was encountered below the fill at all of the borehole locations. This deposit was fully penetrated in all of the boreholes to depths of 3.8 to 7.6 m, except at borehole 20-06 where the borehole was terminated at a depth of about 5.2 m below existing ground surface within the clayey silt to silty clay deposit.

The upper portion of the clayey silt to silty clay deposit has been weathered to form a grey to brown crust, which is typically stiffer than the unweathered clayey silt to silty clay below the upper layer. The weathered crust extends to depths ranging from 3.0 to 4.6 m at all borehole locations and is about 1.5 to 3.3 m in thickness. SPT tests carried out within the weathered crust gave “N” values of 2 to 22 blows per 0.3 m of penetration indicating a stiff to very stiff consistency.

Below the depth of weathering, an unweathered grey clayey silt to silty clay layer, with a thickness ranging from about 0.8 to 4.6 m, was encountered in all of the boreholes. SPT “N” values within the grey clayey silt to silty clay layer ranged from weight of hammer to 17 blows per 0.3 m of penetration, indicating a stiff to very stiff consistency. A “N” value of 17 per 0.3 m of penetration was measured at depth of about 6.4 m in borehole 20-01, which could indicate the presence of a sand seam within the unweathered clayey silty to silty clay, as also indicated during the SCPTu testing at test hole CPT20-02.

In-situ field vane shear testing was carried out within the grey clayey silt to silty clay in boreholes 20-01 to 20-03, 20-05 and 20-07and the measured undrained shear strength (S_u) ranged from 57 to greater than 96 kPa, indicating a stiff to very stiff consistency.

The measured natural water contents of 30 samples of the clayey silt to silty clay ranged from 10% to 54%.

The results of grain size distribution testing carried out on four samples of this deposit are provided on Figures C-1 to C-4 in Appendix C.

4.5 Layered Clayey Silt, Silt, Sandy Silt and Silty Sand

The clayey silt to silty clay is underlain by layered deposits of clayey silt, silt, sandy silt, and silty sand (called hereafter “silt”). These deposits were fully penetrated in all of the boreholes, with the exceptions of 20-06 and 20-08 and extend to depths varying between about 10.7 to 16.8 m below the existing ground surface.

The layered silt deposits are grey to grey brown in colour, with measured SPT ‘N’ values typically ranging from weight of rods to 14 blows per 0.3 m of penetration indicating a very loose to compact state of packing; but more typically very loose to loose. The deposits contain trace to some gravel and occasional cobbles.

The measured natural water contents of 16 samples of the silts ranged from 8% to 33%.

The results of grain size distribution testing on three samples of the layered silts deposit are provided on Figures C-6 to C-8 and in Appendix C.

4.6 Sand

A deposit of sand to gravelly sand (called hereafter “sand”) was encountered below layered deposits of clayey silt, silt, sandy silt, and silty sand in all of the deep boreholes and forms a transitional zone between the layered silts deposit above and the sand and gravel deposit below. The sand deposit contains trace to some silt and gravel and extends to depths ranging from 18.5 to 29.9 m below the existing ground surface, with thicknesses ranging between about 6.1 to 16.8 m.

SPT testing carried out within the sand deposit provided SPT ‘N’ values varying between 6 and to greater than 50 (> 50) blows per 0.3 m of penetration, but typically ranging between 30 and 50 blows per 0.3 m of penetration indicating a compact to dense state of packing. SPT “N” values generally increase with depth within this deposit.

The measured natural water contents of 12 samples of this deposit ranged from about 7% to 30%.

The results of grain size distribution testing carried out on nine samples of sands deposit are provided on Figures C-9 to C-17 in Appendix C.

4.7 Sand and Gravel

A deposit of sand and gravel exists below the sand deposit in all of the deep boreholes. The sand and gravel deposit generally consists of a heterogeneous mixture of gravel, cobbles, and boulders in a matrix of silty sand to sandy silt. The sand and gravel deposit was penetrated to depths ranging between 33.5 and 35.9 m beneath the existing ground surface prior to encountering refusal on bedrock.

SPT “N” values within the sand and gravel layer ranged from 12 to greater than 50 blows per 0.3 m of penetration, indicating a compact to very dense state of packing, but were more typically greater than 40, indicating a dense to very dense state of packing.

4.8 Bedrock

Refusal to wash boring was encountered in all of the deep boreholes at depths ranging from 33.5 to 35.9 m below the existing ground surface (i.e., elevations ranging from about 32.8 to 31.7 m). The bedrock was cored in all deep boreholes to depths ranging between about 35.8 and 43.9 m below the existing ground surface.

The following table summarizes the ground surface, depth to bedrock, bedrock elevations and core lengths as encountered at the borehole locations.

Borehole Number	Ground Surface Elevation (m)	Bedrock Depth (m)	Core Length (m)	Bedrock Elevation (m)
20-01	66.3	33.5	2.3	32.8
20-02	66.8	34.4	2.1	32.4
20-03	66.8	34.6	1.8	32.2
20-04	66.9	34.9	3.6	32.0
20-05	67.7	35.9	8.0	31.
20-07	66.6	34.4	3.4	32.2
97-11[1]	67.1	34.9	2.9	32.2

Note 1: borehole 97-11 from previous investigation.

The bedrock encountered in the cored boreholes typically consists of dark grey dolostone with interbeds of shale and limestone. The top portion of the bedrock is weathered, and the bedrock becomes fresh below the weathered zone.

Rock Quality Designation (RQD) values measured in all “deep” boreholes range from about zero to 95%, but typically varying between 60 to 95% indicating a fair to excellent quality rock. In general, the RQD values increase with depth.

Seven UCS tests were carried out on core specimens of the dolostone bedrock, and measured UCS values ranging from 99 to 147 MPa, indicating a very strong bedrock. The results of the UCS tests are included in Appendix C. The variation of UCS values with depth is presented in Figure C-19 in Appendix C and in general, the UCS values increase with depth.

Photographs of the recovered bedrock core are presented in Appendix D.

4.9 Groundwater

Monitoring wells were sealed into boreholes 20-01, 20-02, 20-04, 20-06 and 20-08 with single or double screens at various depths, to allow for subsequent groundwater level measurements. The measured groundwater levels are provided in the table below. The groundwater elevations were measured on August 10, 2020.

Borehole Number/Well*	Ground Surface Elevation (m)	Groundwater Depth (m)	Groundwater Elevation (m)	Depth of Screen (m)	Geological Unit
20-01	66.3	2.7	63.6	2 to 5	Clayey Silt to Silty Clay
20-02/B	66.8	4.4	62.4	2 to 5	Clayey Silt to Silty Clay
20-02/A	66.8	5.6	61.2	12 to 15	Sands
20-04	66.9	5.6	61.3	15 to 18	Sands
20-06/B	66.3	3.1	63.2	3 to 6	Clayey Silt to Silty Clay
20-06/A	66.3	4.9	61.4	7 to 10	Layered “Silt” Deposit
20-08/B	66.4	3.4	63.0	2 to 5	Clayey Silt to Silty Clay
20-08/A	66.4	3.1	63.3	12 to 15	Sands

*‘A’ wells are deep and ‘B’ wells are shallow in boreholes with two wells.

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.

4.10 Corrosion Testing

Two sample of soils from boreholes 20-04 and 20-07 were submitted to Eurofins Environment Testing 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 E and are summarized below.

Borehole Number	Sample Number	Depth Intervals (m)	Chlorides (%)	Sulphates (%)	pH	Resistivity (Ohm-cm)
20-04	5	3.1 – 3.7	0.014	0.02	7.7	1,960
20-07	5	3.1 – 3.7	0.024	0.01	7.4	5,000

5.0 DESIGN AND CONSTRUCTION CONSIDERATIONS

5.1 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. Where comments are made on construction, they are provided only in order 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. The reader is referred to the "*Important Information and Limitations of This Report*" which follows the text of this report but forms an integral part of this document.

The guidelines provided in this section of the report are based on design in accordance with Part 4 of the 2012 Ontario Building Code (OBC) for limit states design.

5.2 Seismic Considerations

5.2.1 Seismic Zone

The site falls within the Western Québec Seismic Zone (WQSZ) according to the Geological Survey of Canada. The WQSZ constitutes a large area that extends from Montréal to Témiscaming, and which encompasses the Ottawa area. Within the WQSZ, recent seismic activity has been concentrated in two subzones; one along the Ottawa River and another more active subzone along the Montréal-Maniwaki axis. Historical seismicity within the WQSZ includes the 1935 Témiscaming event which had a magnitude (i.e., a measure of the intensity of the earthquake) of 6.2 and the 1944 Cornwall-Massena event which had a magnitude of 5.6. In comparison to other seismically active areas in the world (e.g., California, Japan, New Zealand), the frequency of earthquake activity within the WQSZ is significantly lower but there still exists the potential for significant earthquake events to be generated.

5.2.2 Liquefaction Hazard Assessment

Based on the results of this investigation, the ground conditions consist of topsoil or fill at ground surface, underlain by a clayey silt to silty clay layer, followed by deposits of interlayered silts and sand, which is in turn underlain by dense to very dense sand and gravel. The bedrock surface is at depths of about 33.5 to 36 m below the existing ground surface.

Seismic liquefaction occurs when earthquake vibrations cause an increase in pore water pressures within the soil. The presence of increased/excess pore water pressures reduces the effective stress between the soil particles, and the soil's frictional resistance to shearing. This phenomenon, which leads to a temporary reduction in the shear strength of the soil, may cause:

- Large lateral movements of even gently sloping ground, referred to as 'lateral spreading'.
- Reduced shear resistance (i.e., bearing capacity) of soils which support foundations, as well as reduced resistance to sliding.
- Reduced shaft resistance for deep foundations as well as reduced resistance to lateral loading.
- Buoyant uplift of buried structures (such as tanks or sewer pipes).

In addition, seismic settlements may occur once the vibrations and shear stresses have ceased. Seismic settlement is the process whereby the soils stabilize into a denser arrangement after an earthquake, causing potentially large surface settlements.

The following conditions are more prone to experiencing seismic liquefaction:

- Granular soils, rather than cohesive soils (i.e., more probable for sands and silts than for clays).
- Soils in a loose (i.e., dilative) state.
- Soils located below the groundwater level.

The very loose to loose silt deposit encountered beneath the clayey silt to silty clay would satisfy these criteria. As such, an assessment has been carried out of the liquefaction potential of this deposit.

The liquefaction susceptibility of granular soils was evaluated by comparing the shear stresses generated by a design earthquake to the available cyclic shear strength of the soil. Liquefaction is predicted to occur when the cyclic shear stresses exceed the cyclic shear strength required to trigger liquefaction.

The methodology used involves comparing the cyclic shear stresses applied to the soil by the design earthquake, represented as the cyclic stress ratio (CSR), to the cyclic shear strength, represented as the cyclic resistance ratio (CRR) provided by the soil.

An assessment of the liquefaction potential of the silt deposit was carried out using the approach outlined in the Canadian Highway Bridge Design Code (CHBDC-2014) and by Idriss and Boulanger (2204 & 2008). The CRR with depth was calculated at each SCPTu test hole location using the parameters obtained during CPT testing including tip resistance, skin friction and dynamic pore pressure.

The assessment was based on a 2,475-return period design earthquake with a modal magnitude of 6.5 and a peak ground acceleration of 0.32g for a Site Class E condition (see Section 5.2.4).

Other factors were also considered to refine the liquefaction assessment including the increased shear strength that comes from soil cementation/bonding and aging (Leon, 2006), and modified ground response relationships for Eastern Canada (Perret et al., 2019).

The results of shear wave velocity (Vs) based on the VSP and seismic CPT testing were used to estimate the average normalized shear wave velocity within the liquefiable zones at test holes CPT20-01, CPT20-02 and borehole 20-05 (VSP testing). Average Vs values were found to vary between 180 to 210 m/s at these test hole locations. This information was also used to assess/validate the liquefaction potential at the site.

The results of this assessment indicate that the very loose to loose silt deposit (see Section 4.5) which is encountered from about 5.2 to 10.4 m (or elevations 56.2 to 61.4 m) at test hole CPT20-01, between about 7.0 and 11.8 m (elevations 54.8 to 59.6 m) at test hole CPT20-02, and from about 6.1 to 11.6 m (elevations 56.1 to 61.6 m) at borehole 20-05 (VSP testing location), are potentially liquefiable under the design ground motions corresponding to the 2,475 return period earthquake (i.e., 2 % probability of exceedance in 50 years). CPT 20-01, 20-02 and 20-05 represent the ground conditions (in terms of liquefaction potential) for the East Tower, West Tower and podium structure, respectively.

5.2.3 Consequence of Liquefaction

In summary, the potential consequences of liquefaction of the silt deposit are as follows:

- Following an earthquake, total vertical settlements of up to about 200 mm may occur across each of the proposed structures.
- Following an earthquake, differential settlements of up to about 100 mm may occur across each of the proposed structures. The induced differential settlement post liquefaction may be assumed to be up to about 50% of the total estimated settlement.
- Accuracy of the estimated seismic settlements is typically considered to be within 25% to 50% of the actual value.

The structure would need to be designed to accommodate these horizontal and vertical movements without experiencing collapse. Guarding against collapse (i.e., allowing for 'safe exit') is considered to be the objective of design for earthquake conditions (recognizing that the 'design' earthquake has a return period of 2,475 years), though the structure may be damaged and rendered unserviceable.

The issue of seismic liquefaction and the associated ground settlements are consistent with what was encountered at the nearby LRT station as well as for the expansion of the Bayshore Mall parking garage in 2015.

These estimated total and differential vertical settlements would only be experienced if the structure was not supported directly on bedrock (i.e., not on deep foundations).

While shallow foundations may have been an option for the podium structure, the founding elevation of such foundations with one level of basement would place them too close to the liquefiable layers and the bearing capacity in such a case would be materially lower with a corresponding risk of potential failure during the earthquake. In this case, the top of the liquefiable zones would be located within2 to 3 m of the underside of the raft foundation, which is within the zone of highest stress imposed by the foundations. This situation typically creates a punching failure with excessive total and differential settlements.

Furthermore, deep foundations supported on bedrock would be subject to significant down drag loads, following a seismic event and the structural capacity of the deep foundations to support those loads would need to be evaluated. The deep foundation elements would also be subjected to lateral loading and deformation, due to both the inertial forces on the structure as well as due to lateral spreading. Guidelines on this issue are provided in Section 5.3.

5.2.4 Site Class

The Ontario Building Code requires the use of the time-averaged (harmonic) shear wave velocity (V_s) for the upper 30 m of soil or rock below the structure for determining the appropriate Site Class for seismic design purposes. Table 4.1.8.4A of the OBC also specifies circumstances for which a Site Class of "F" is applicable and a site-specific evaluation must be carried out; the presence of liquefiable soils is one of those conditions. That requirement is therefore applicable to the sites of the East and West Towers, where the fundamental period of vibration will likely be greater than 0.5 seconds for these buildings. As such, the OBC outlines that a site-specific ground response analysis is required to be completed for this site in order to develop a site-specific design response spectra. Golder can assist the structural engineer to carry out such analyses once required.

With regards to the podium structure (which is a 3-storey building with one level of underground parking), the structures may have a fundamental period of vibration of less than or equal to 0.5 seconds. In such a case, the liquefaction can be considered to have very minimal negative impact on the performance of the structure. Therefore, Site Class “E” (i.e., non-liquefied Site Class) can be considered for design of the podium structure, as long as its fundamental period of vibration is less than or equal to 0.5 seconds.

5.3 Foundations

5.3.1 General and Review of Options

The proposed development will consist of two high-rise buildings including 27-storey and 30-storey Towers, and a three-storey parking podium with one level of underground parking.

Different foundation options including shallow foundations (i.e., raft), deep foundations (i.e., driven steel piles and drilled concrete caissons) along with ground improvement techniques, such as stone/concrete columns (i.e., Geopiers), have been assessed as part of the foundation feasibility studies carried out for this site. These foundation options and ground improvement techniques along with their feasibilities are further disused in the following sections.

A summary of the advantages, disadvantages and constructability associated with each “feasible” foundation option is also provided in the section.

- **Raft foundation**: The feasibility of raft foundation was assessed for the proposed development and it is our opinion that raft foundations would not be feasible for the Towers and the podium structure, as the raft foundations would not provide sufficient bearing resistances or acceptable settlement performance for the proposed structures due excessive total and differential settlements (i.e., under both static and seismic loading conditions).
- **Ground improvement (Geopier columns)**: An evaluation of the ground improvement option including application of Geopiers (i.e., stone and concrete columns) for the podium structure was assessed in consultation with a specialized geo-contractor (GeoSolv). Based on the assessments carried out for this site and, given the estimated magnitudes of the post earthquake settlements and the extent (depth and thickness) of the liquefiable zone, ground improvement is not considered to be feasible for this site.
- **Driven steel H-piles**: Steel H-piles driven to refusal on the dolostone bedrock could be feasible for support of the proposed Towers and podium structure. This option would provide high geotechnical resistances and minimal post-construction settlements. The driven piles are more flexible (in comparison to concrete caissons) and have smaller reaction surfaces to resist the lateral loads imposed by seismic or wind loads.
- **Driven steel pipe (tube) piles**: Closed-ended steel tube (pipe) piles could also be considered as a deep foundation option for support of the proposed Towers and podium structure. This foundation option would have similar advantages to steel H-piles in terms of high geotechnical resistances and minimal settlements. Since cobbles and boulders were encountered below depths of 20.1 to 29.0 m (within the dense to very dense sand and gravel deposit), pipe piles are considered to have a higher risk than H-piles to meet refusal above the bedrock surface (which would result in a reduced capacity and/or the need for additional piles) or to be damaged or deflected away from their vertical or battered orientation (potentially also requiring driving replacement piles), if cobbles and/or boulders are encountered within the this deposit during driving.

- **Rock socketed steel pipe (tube) piles**: Socketed steel pipe piles installed using the down-the-hole hammer drilling method could also be considered as a deep foundation option for support of the Towers and podium structure. This foundation option would also have similar advantages to those above of high geotechnical resistances and minimal settlements. This foundation type would penetrate through any cobbles or boulders encountered within the sand and gravel deposit during installation (reducing construction risk) and can provide high uplift capacity if socketed adequately into the bedrock.
- **Rock Socketed Drilled Concrete Caissons**: Caissons deriving their support from bearing within the bedrock are also feasible for this site. Caissons would require the use of temporary (or permanent) liners to mitigate the potential risks of ground loss from the water-bearing cohesionless layers within much of the overburden materials (particularly the upper portion of the ground) that are wet and loose, and which would not stand un-supported. The caissons would have to be socketed at least nominally into the bedrock to permit cleaning of the caisson bases, and such sockets would have to be advanced by rock coring and/or chisel drilling into the strong dolostone bedrock. This foundation option is considered feasible for both Towers and podium structure. Rock socketed caissons would also have the required stiffness to resist the seismic lateral loads as well as provide 'fixity' at the interface with the bedrock surface during a seismic event.

Based on the above considerations, the preferred options from a geotechnical/foundations perspective are to support the Towers and podium structure on concrete caissons socketed into the bedrock (where significant uplift capacity is required), or steel H-piles driven to refusal on the dolostone bedrock. The latter option can provide less uplift capacity as the uplift resistance is mobilized based on skin friction between the piles and lower compact to very dense cohesionless deposits.

In addition, the deep foundation options would be subject to the following considerations:

- The use of deep foundations would avoid the structure experiencing any significant total or differential vertical settlements. However, any structures or utilities connected to the Towers or podium which are supported at shallow depths will require flexible connections to the buildings to allow for the potential differential movements that are likely during a seismic event.
- The seismic settlement due to an earthquake event will no longer be a concern for the structures supported on deep foundations but the foundation elements would be subjected to significant down drag loads following a seismic event and the structural capacity of the deep foundations to support those loads would need to be evaluated.
- The piles would be subjected to lateral loading and deformation, due to both the inertial forces on the structure and the loss of lateral support due to an earthquake event. Site-specific lateral load-displacement relationships (i.e., p-y curves) for the preferred deep foundation elements can be developed once the size and type of the foundation elements are determined.

5.3.2 Driven Steel Piles

The proposed structures may be supported on steel H-piles or closed-ended steel pipe piles driven to refusal either on or within the underlying dolostone bedrock.

5.3.2.1 Founding Elevations

Based on the geotechnical investigations carried out at the site, the bedrock surface is considered to be relatively flat and was encountered between about Elevations 31.7 m and 32.8 m at the borehole locations. Based on the borehole results, and assuming about 0.1 m of penetration into the bedrock to allow for some weathering in the upper portion of the rock, the following pile tip elevations are recommended for design of steel H-piles or pipe piles:

Steel Driven Piles Founding Elevations

Structure	Referenced Boreholes	Bedrock Surface Elevation (m)	Design Pile Tip Elevation (m)
Parking Podium	20-02, 20-03 and 20-05	32.1	32.0
East Tower	20-03, 20-04 & 20-05	31.9	31.8
West Tower	20-01, 20-02 & 20-07	32.5	32.4

The pile caps should be constructed at a minimum depth of 1.8 m for frost protection purposes, per OPSD 3090.101 (Foundation Frost Penetration Depths for Southern Ontario).

5.3.2.2 Compressive Resistance

Piles driven to rock typically generate high ultimate geotechnical capacities, generally equal to or in excess of the structural capacity of the steel section. For preliminary design purposes, the factored ultimate geotechnical resistance may be assumed to be equal to the ultimate structural resistance of the steel section.

A resistance factor of 0.4 should be applied to this value to obtain the factored geotechnical resistance of the pile. This factor may be increased to 0.5 if a program of dynamic (PDA) testing is implemented during construction, or 0.6 if a static load test is completed at the site.

Pipe piles should be driven closed-end and then concrete-filled, with a minimum wall thickness of not less than 9.5 mm.

Settlements for piles driven to sound rock are generally negligible, and the geotechnical resistance mobilized at 25 mm of settlement (a typical SLS criteria) would normally exceed the factored axial resistance at ULS. Geotechnical SLS considerations therefore do not generally govern the design of piles driven to sound rock.

5.3.2.3 Downdrag

Downdrag forces (or negative skin friction) will also be applied to the piles as a result of seismic settlement of the layered “silts” deposit which apply additional negative skin friction to the pile shaft (in addition to the static loads applied on piles). The negative skin friction can be assumed to be equal to the shaft friction as calculated in Section 5.3.2.4 for uplift resistance in soil (the resistance factor of 0.3 should not be applied). Downdrag is typically considered in conjunction with dead and sustained live loads (not transient loads such as wind, earthquake and transient live loads) in evaluating the structural capacity of the piles.

For a preliminary design purposes, the resulting unfactored downdrag loads may be taken as 360 kN for a 406 mm diameter pipe pile, and 620 kN for a HP 360 x 132 pile. Downdrag forces applied on other pile sizes can be provided, if required.

5.3.2.4 Uplift Resistance

The uplift resistance of a pile is a result of skin friction acting along the surface area of the embedded pile. The unfactored shaft resistance for the cohesive soil (i.e., clayey silt to silty clay) may be assumed to be equal to:

$$q_s = \alpha S_u$$

Where:

q_s = the unfactored shaft resistance (in kPa)

α = a shaft resistance factor based on soil type and strength (see table below)

S_u = the undrained shear strength of the soil (see table below)

The unfactored shaft resistance for the cohesionless soil (i.e., granular deposits below the clayey silt and silty clay) may be assumed to be equal to:

$$q_s = \beta\sigma_v'$$

β = a shaft resistance factor based on soil type and strength (see table below)

σ_v' = vertical effective stress adjacent to the pile at depth z

Shaft Uplift Resistance Parameters

Elevation (m)	Soil Unit	α or β	S_u or σ_v' (kPa)	$q_{s\,(Ave.)}$ (kPa)
64 to 59	Clayey silt to silty clay	$\alpha = 0.6$	$S_u = 70$	40
59 to 53	Layered silt deposit	$\beta = 0.37$	$\sigma'_{v(ave.)} = 90$ to 140	60
53 to 42	Sand deposits	$\beta = 0.57$	$\sigma'_{v(ave.)} = 140$ to 240	150
42 to 32	Sand and gravel	$\beta = 0.85$	$\sigma'_{v(ave.)} = 240$ to 330	250

A resistance factor of 0.3 should be applied to the resistance calculated using these values, to obtain the factored geotechnical uplift resistance. The dead weight of the pile itself, with an appropriate resistance factor for dead weight, may also be added to the geotechnical resistance in calculating the total uplift resistance.

The total uplift resistance of a pile group is the lesser of the sum of the individual pile resistances as described above, or the resistance of a single “block” of soil with a perimeter equal to the perimeter of the pile group (the mass of the soil inside the “block” may be included in the calculation; use a soil weight of 18 kN/m^3).

It should be noted that the uplift resistance of piles is highly dependent upon the installation of the piles as well as the layout of the pile groups. If the piles are relied upon to resist significant uplift loads, and uplift governs the design, consideration should be given to carrying out a tension test to confirm the uplift capacity.

5.3.2.5 Construction Considerations

Steel piles should be driven to bedrock with the pile tip elevations (or founding elevations) as indicated above. It should, however, be noted that the upper portion of the bedrock is weathered and fractured, and therefore piles may penetrate slightly into the bedrock prior to refusal to advancement is encountered. Pile lengths will need to be adjusted as required during construction.

Cobbles or boulders were encountered within the sand deposit and sand and gravel deposit at the borehole locations. For the installation of steel H-piles or steel pipe piles, consideration must be given to the potential presence of cobbles and boulders within these deposits. H-piles should be reinforced at the tip with rock point driving shoes (e.g., Titus HD Rock Injector) to improve seating of the piles on the bedrock and to reduce the potential for damage to the piles during driving through the overlying cobbles and boulders, in accordance with Ontario Provincial Standard Specification (OPSS) 903 *(Construction Specification for Deep Foundations)*. If steel pipe piles are used, driving shoes should be in accordance with OPSD 3001.100 Type II (*Steel Tube Pile Driving Shoe*).

Piles can deflect or become damaged if they encounter boulders in the sand and gravel deposit. It should therefore be expected that replacement piles will be required for some of the damaged piles.

Provision should be made for restriking at least 10% of the piles 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 a minimum of 48 hours after the previous set.

The piling specifications should be reviewed by Golder prior to tender, as should the contractor’s submission (shop drawings, equipment, procedures and preliminary set criteria) prior to construction. Preliminary pile driving criteria should be established prior to construction using wave equation analysis (WEAP or similar) or other approved means and confirmed through a program of dynamic (PDA) testing carried out at an early stage in the piling program. Additional PDA testing should be used to confirm the pile capacities at regular intervals as the project progresses. As a preliminary guideline, the specification should require that at least 10% 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 three days following testing. The final report should be stamped by an engineer licensed in the province of Ontario.

For piles driven to refusal on bedrock, and as described in OPSS 903, it is a generally accepted practice to reduce the hammer energy after abrupt peaking is met on the bedrock surface, and to then gradually increase the energy over a series of blows to seat the pile.

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.

Vibration monitoring should be carried out during pile installation to ensure that the vibration levels at the adjacent existing structures and services are maintained below tolerable levels, as further outlined in Section 5.10.

5.3.3 Socketed Steel Pipe Pile Foundations

5.3.3.1 Founding Elevations

Alternatively, the Towers and podium structure may be supported on steel pipe piles may be socketed into the dolostone bedrock using a down-the-hole hammer. Based on the geotechnical investigations carried out at the site, the bedrock surface is considered to be relatively flat and was encountered between about Elevation 31.7 m and 32.8 m at the borehole locations. It is recommended that the steel pipe piles be socketed 2.0 m into the bedrock for axial resistance considerations. Therefore, the following socket founding elevations are recommended for design.

Socketed Steel Driven Piles Founding Elevations

Structure	Referenced Boreholes	Bedrock Surface Elevation (m)	Design Pile Tip Elevation (m)
Parking Podium	20-02, 20-03 and 20-05	32.1	30.1
East Tower	20-03, 20-04 & 20-05	31.9	29.9
West Tower	20-01, 20-02 & 20-07	32.5	30.5

The pile caps should be constructed at a minimum depth of 1.8 m for frost protection purposes, per Ontario Provincial Standard Drawing (OPSD) 3090.101 (*Foundation Frost Penetration Depths for Southern Ontario*). Cobbles and boulders were encountered within the sand and gravel deposits but the down hole hammer should be able to penetrate those obstructions. Vibration monitoring should be carried out during pile installation as described in Section 5.10.

5.3.3.2 Axial Geotechnical Resistance

The following foundation design recommendations have been based on the side-wall (shaft) resistance of the rock socket and a factored geotechnical resistance at Ultimate Limit States (ULS) of 1,800 kPa. As an example, for a 406 mm diameter concrete-filled pipe pile socketed 2.0 m into the dolostone bedrock this would equate to a factored geotechnical resistance at ULS of 4,500 kN (geotechnical resistance factor of 0.4).

The ULS resistance considers the RQD values recorded for the bedrock as well as the compressive strength data for the rock core. This value is applicable provided that the socket is within competent bedrock and that the side wall of the socket is cleaned of any smeared material. Pile installation should be in accordance with OPSS.PROV 903 (*Deep Foundations*).

5.3.3.3 Downdrag

Downdrag forces will also be applied to the piles as a result of seismic settlement of the layered silts deposit which apply additional negative skin friction to the pile shaft (in addition to the static loads applied on piles). The resulting unfactored downdrag loads may be taken as 360 kN for a 406 mm diameter pipe pile.

5.3.3.4 Uplift Resistance

The uplift resistance of a pile is a result of skin friction acting along the surface area of the rock socket. For a 406 mm diameter concrete-filled pipe pile socketed 2.0 m into the dolostone bedrock, a factored geotechnical resistance at ULS of 3,300 kN may be considered for design using a geotechnical resistance factor of 0.3.

5.3.4 Caisson Foundations

As an alternative to driven pile foundations, the proposed Tower and podium structure can be supported on caisson foundations socketed into the shale bedrock.

5.3.4.1 Founding Elevations and Construction Considerations

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

For design purposes, it is recommended that the caissons be founded at the following elevations (i.e., a rock socket of approximately 2 m).

Concrete Caissons Founding Elevations

Structure	Referenced Boreholes	Bedrock Surface Elevation (m)	Design Pile Tip Elevation (m)
Parking Podium	20-02, 20-03 and 20-05	32.1	30.1
East Tower	20-03, 20-04 & 20-05	31.9	29.9
West Tower	20-01, 20-02 & 20-07	32.5	30.5

The use of a temporary (or permanent) liner or casing will be required to advance the caissons through the potential water-bearing cohesionless layers while minimizing loss of ground. The casing should be extended so that it is "seated" adequately into the bedrock. Casing installation through the dense "sand and gravel" deposit containing cobbles and boulders may be difficult. Churn drilling and possibly rock coring techniques may be required to advance the caissons through the dense layers and upper weathered portion of the bedrock.

If caisson caps are to be included as part of the design, they should be constructed at a minimum depth of 1.8 m for frost protection purposes, per OPSD 3090.101 (*Foundation Frost Penetration Depths for Southern Ontario*). Similar to pile installation, vibration monitoring should be carried out during caisson installation to ensure that the vibration levels at the existing structure are maintained below tolerable level, as described in Section 5.10.

The concrete for each caisson must be poured continuously to avoid formation of cold joints within the caissons.

To reduce damage to the rock between two adjacent caissons during construction, it is recommended to maintain a minimum distance of 2 times diameters edge to edge, or minimum 2 metres, whichever is greater, between the caissons.

Post-construction inspection including Cross Sonic Logging (CSL) should be carried out on all installed caissons in accordance with ASTM D6760. The testing should be carried out no sooner than 3 calendar days subsequent to concrete placement, but within 45 days after concrete placement.

Caissons construction must be monitored by a qualified geotechnical engineer or his/her representative at all times.

5.3.4.2 Axial Geotechnical Resistance

The factored geotechnical side wall (shaft) resistance at ULS can be taken as 1,800 kPa provided that the caisson socket is formed within competent bedrock. This value assumes that the side wall of the socket will be cleaned of any cuttings or smeared material. To provide full fixity, the caissons should be provided with a minimum socket length equal to 2 times the caisson diameter.

For a 1.0 m diameter caisson socketed 2 m into the competent bedrock, this would equate to a factored axial compressive geotechnical resistance at ULS of about 11,300 kN (geotechnical resistance factor of 0.4). For socket depths and diameters that differ from above, the ULS resistance can be pro-rated based on the resulting socket side-wall circumference for sockets up to 5 m deep and diameter up to 1.5 m. SLS resistances do not apply to caissons founded within the dolostone bedrock, since the SLS resistance for 25 mm of settlement is typically greater than the factored axial geotechnical resistance at ULS.

The ULS caisson capacities are based on static analyses and should incorporate a geotechnical resistance factor of 0.4 for compressive loads. A resistance value of 0.6 may be used if a static load test is carried out.

The uplift capacity of a rock-socketed caisson should be estimated as indicated above for compressive resistance but a resistance factor of 0.3 should be applied to estimate the factored ULS uplift capacity of the caissons.

The structural engineer should check that the shear strength of the concrete is adequate to support these loads.

5.3.4.3 Downdrag

Downdrag forces will also be applied to the caissons as a result of seismic settlement of the layered silts deposit which will apply additional negative skin friction to the pile shaft. The resulting unfactored downdrag loads may be taken as 900 kN for a 1.0 mm diameter concrete caisson. Negative friction is typically only considered in conjunction with dead and sustained live loads (not transient loads such as wind, earthquake and transient live loads) in evaluating the structural capacity of the piles. Negative shaft friction does not impact the geotechnical resistance of the piles.

5.3.5 Lateral Resistance

5.3.5.1 Lateral Resistance at SLS

The resistance to lateral loading could be derived from the soil resistance in front of the piles or caissons.

The preliminary SLS geotechnical response of the soil in front of the piles or caissons 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:

$$k_h = n_h \, z / d$$

Where:

k_h = the modulus of subgrade reaction (kN/m^3);

n_h = the constant of horizontal subgrade reaction (see table below);

z = the depth (m); and,

d = the pile diameter or width (m).

For cohesive soils:

$$k_h = 67 \, S_u / d$$

Where:

k_h = the modulus of subgrade reaction (kN/m^3);

S_u = the undrained shear strength of the soil; and,

d = the pile diameter or 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.

Lateral Resistance Parameters

Elevation (m)*	Soil Unit	n_h (MPa/m)	s_u (kPa)
Pile cap to 59	Clayey silt to silty clay	–	70
59 to 53	Loose layered silt deposit	1.3	–
53 to 42	Comact to dense sand deposit	4.5	–
42 to 32	Dense to very dense sand and gravel	11.0	–

Note: *Average elevations are considered.

The value above is for a single pile group. Group interaction must be considered when piles or caissons are spaced closely together. Group effects may be accounted for by reducing the coefficient of horizontal reaction (k_h) by an appropriate factor as follows:

Reduction Factors for Pile Group Effect

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

Values for other spacings may be interpolated from the values above. No reduction is required for the first row of piles (i.e., the row which bears against undisturbed soil with no piles in front).

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

It should be noted that the method of applying a linear “spring” to represent the soil reaction to loading is a significant simplification. If lateral load resistance governs the pile design, more rigorous, non-linear methods of analysing resistance exist, one common one being the method of p-y curves. These methods, however, require knowledge of the pile or caisson size, location, loading, pile cap construction, etc. and are therefore typically more suited to the detailed design phase when these items are known. Additional assistance can be provided during detailed design, if required.

5.3.5.2 Lateral Resistance at ULS

For establishing the ULS factored *structural* resistance, the shear force and bending moment distribution in the piles or caissons under factored loading can be established using the 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., clayey silt to 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 or caisson diameters below the underside of the pile cap (where S_u = 70 kPa). Below a depth equal to 3 pile or caisson diameters, and to the bottom of the deposit, the lateral resistance is assumed to be constant at $9S_u$.

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

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

Where:

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

γ = is average unit weight of overlying soil, use 18 kN/m^3;

K_p = is the coefficient of passive earth pressure, use 3.0;

D_w = is the depth to groundwater table below the ground surface(m), assume at underside of pile cap level; use 3.0 m below the ground surface;

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

D = is the pile diameter or width (m).

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

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

5.3.6 Siding Resistance

The values given in the table below may be used to calculate the lateral resistance to sliding/shearing at the foundation-soil or pile cap-subgrade interface. The coefficient of friction, tan φ', may be taken as follows:

Sliding Resistance at the Foundation Interface

Interface Condition	Coefficient of Friction (tan φ')
Concrete to Compacted Granular B Type II	0.6
Concrete to Native Clayey Silt to Silty Clay	0.4

The sliding resistance values given herein are provided in unfactored format, and a resistance factor of 0.8 would need to be applied to the sliding resistance in accordance with limit states design.

5.4 Rock Anchors

Rock anchors could potentially be used to resist uplift and/or overturning of the foundation. Rock anchors should consist of grouted anchors installed into the bedrock at depth. The resistance provided by rock anchors would be additional to the uplift values (as indicated in the sections above for foundation elements founded on the bedrock surface) for the foundation element in soil and therefore only the resistance provided by the rock (and not the overlying overburden) is considered as described below.

The design of the rock anchors is often the responsibility of the contractor and supplier, since there are several proprietary products/systems. However, the rock anchors would likely be installed in a borehole that is drilled with air-percussion equipment or with rotary diamond drilling equipment with water circulation; those drilling methods can fairly readily penetrate through boulder/cobblery ground such as exists on this site. A cased hole would be drilled through the overburden, a socket drilled into the bedrock, the steel anchor inserted, and then a portion of the annular space around the bar filled with grout.

Because the rock anchors would be permanent elements of the foundations, a 'double corrosion protection' system should be provided.

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

i) failure of the steel tendon or top anchorage

ii) failure of the grout/tendon bond

iii) failure of the rock/grout bond

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

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

For potential failure mode iii), the factored bond stress at the concrete/rock interface may be taken as 1,800 kPa for ULS design purposes. This value should be used in calculating the resistance under ULS conditions. If the response of the anchor under SLS conditions needs to be evaluated, for a preliminary assessment it may be taken as the elastic elongation of the unbonded portion of the anchor under the design loading.

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

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

Where:

Qr = Factored uplift resistance of the anchor (KN);

φ = Resistance factor (use 0.4);

γ/ = Effective unit weight of rock (use 16 KN/m^3 below the groundwater level);

D = Anchor length in metres; and,

θ = one-half of the apex angle of the rock failure cone (use 30°).

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

$$Q_r{}' = Q_r \cos(\alpha)$$

Where:

Qr' = Factored uplift resistance of the anchor subject to inclined load in kN;

Qr = Factored uplift resistance of the anchor (KN); and,

α = Angle between the load direction and the vertical.

For a group of anchors or for a line of closely spaced anchors, the resistance must consider the potential overlap between the rock masses mobilized by individual anchors. In the case of group effects for a series of rock anchors in a rectangle with width "a" and length "b" installed to a depth "D", the equation for the volume of the truncated trapezoid failure zone would be as follows:

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

Where:

V = Volume of the truncated trapezoid failure zone (m^3);

D = Depth of anchor group (m);

a = Width of anchor group (m);

b = Length of the anchor group (m); and,

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

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

$$Q_r = \varphi\gamma' V$$

Where:

Qr = Factored uplift resistance of the anchor (KN);

φ = Resistance factor, use 0.4;

$\gamma/$ = Effective unit weight of rock, use 16 kN/m^3; and,

V = Volume of truncated trapezoid (m^3).

The method described above does not explicitly consider the tensile strength of the rock that must be overcome prior to mobilization of the weight of the rock mass. If required, the tensile strength of the rock mass can be assessed based on the unconfined compressive strength, recovery, and quality of bedrock core obtained.

It is recommended that proof load tests be carried out on the anchors to confirm their resistance. The proof load tests should be carried out in accordance with OPSS 942 *(Prestressed Soil and Rock Anchors).*

A geotechnical professional should be present during the installation and testing of the anchors. Care must be taken during grouting to ensure that the grouting pressure is sufficient to bond the entire length of the grouted area with minimum voids. Confirmation of sufficient embedment into the rock beneath the foundations should be carried out to make sure that the anchors are being installed in rock of adequate quality. The anchor holes must be thoroughly flushed with water to remove all debris and rock flour. It is essential that rock flour be completely removed from the holes to be grouted to promote an adequate bond between the grout and the rock. Prestressing of the anchors prior to loading will minimize anchor movement due to service loads.

5.5 Frost Protection

The presence of frost-susceptible soils within the frost penetration depth will require that isolated, unheated exterior footings/pile caps adjacent to surfaces which are cleared of snow cover during winter months be provided with a minimum of 1.8 m of earth cover (or equivalent insulation). Exterior foundations of heated structures should be provided with a minimum of 1.5 m of earth cover (or equivalent insulation).

If sufficient earth cover cannot be provided, foundation insulation details can be provided during detailed design.

5.6 Lateral Earth Pressures

Lateral earth pressures acting on the foundation walls and retaining walls, if considered, will depend on the type and method of placement of the backfill materials, on the nature of the soils behind the backfill, on the magnitude of surcharge including construction loadings, on the freedom of lateral movement of the structure, and on the drainage conditions behind the walls. Seismic (earthquake) loading may also need to be taken into account in the design.

5.6.1 Static Lateral Earth Pressures for Design

It is assumed that the basement walls will be non-yielding, and therefore at-rest conditions will apply for those walls. The appropriate earth pressure coefficients for retaining walls will depend on the type of retaining wall.

It is assumed that the basement walls will be drained but if the structures will not be drained, the earth pressure equation below the groundwater level should be used for the depth of the soil below groundwater level. The groundwater level was measured to be between about 2.4 and 3.2 m at this site.

As a first, but likely conservative approximation, the static lateral earth pressure can be calculated as:

$$\sigma_h(z) = K(\gamma \cdot z + q) \text{ (Above the groundwater level)}$$

$$\sigma_{h(z)} = K[\gamma d_w + (\gamma - \gamma_w)(z - d_w) + q] + (z - d_w)\gamma_w \text{ (Below the groundwater level)}$$

Where:

$\sigma_{h(z)}$ = Lateral earth pressure on the wall at depth z, kPa;

K = Earth pressure coefficient, K_o for restrained structures or K_a for unrestrained structures

γ = Unit weight of retained soil,

γ_w = Unit weight of water, 9.81 kN/m^3;

z = Depth below the top of wall, m;

d_w = Depth to groundwater level (see discussion below); and,

q = Uniform surcharge at ground surface behind the wall to account for traffic,equipment, or stockpiled soil (use 12 kPa).

The pressures are based on the existing fill and native materials behind the wall and the following parameters (unfactored) should be used to estimate the lateral earth pressures:

Material	Fill	Clayey Silt to Silty Clay	Granular B Type II	Clear Stone
Soil Unit Weight:	20 kN/m^3	17.5 kN/m^3	21 kN/m^3	18 kN/m^3
Coefficients of static lateral earth pressure: Active, K_a At rest, K_o Passive, K_P	 0.33 0.50 3.00	 0.36 0.53 2.77	 0.27 0.43 3.70	 0.26 0.41 3.85

The above lateral earth pressures have not been factored; factoring of these loads will be required if the shoring is being designed in accordance with Limit States Design.

A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the structure. Care must be taken during the compaction operation not to overstress the structure. Heavy construction equipment should be maintained at a distance of at least 1 m away from the structure while the backfill soils are being placed and the backfill should be uniformly raised around the structure. Hand-operated compaction equipment should be used to compact the backfill soils within a 1 m wide zone adjacent to the walls. Other surcharge loadings should be accounted for in the design, as required.

5.6.2 Seismic Lateral Earth Pressures for Design

Seismic loading will result in increased lateral earth pressures acting on the retaining walls. The walls should be designed to withstand the combined lateral loading for the appropriate static pressure conditions given above, plus the earthquake-induced dynamic earth pressure. The earthquake-induced dynamic 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 total pressure distribution (static plus seismic) may be determined as follows:

$$\sigma_h(z) = K\,\gamma\,z + (K_{AE} - K_a)\,\gamma\,(H\text{-}z)$$

Where:

$\sigma_{h(z)}$ = is the (static plus seismic) lateral earth pressure at depth, d, (kPa);

K_a = the static active earth pressure coefficient, (see table above);

K_o = the static at-rest earth pressure coefficient, (see table above);

K = Earth pressure coefficient, K_o for restrained structures or K_a for unrestrained structures;

K_{AE} = the seismic earth pressure coefficient;

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

K_{AE} = is the seismic active earth pressure coefficient (see table below);

γ = is the unit weight of the backfill soil (kN/m^3), as given previously; and,

z = is the depth below the top of the wall (m).

Seismic (earthquake) loading must be taken into account in the assessment of lateral earth pressures:

- The horizontal seismic coefficient (k_h) used in the calculation of the seismic active pressure coefficient is taken as 1.0 times the PGA. For structures which allow lateral yielding, (k_h) is taken as 0.5 times the PGA.
- The following seismic active pressure coefficients (K_{AE}) are for the clayey silt to silty clay materials and granular backfills:

Seismic Active Pressure Coefficients, K_{AE}

Wall Behavior	Design Earthquake	Site PGA	Clayey Silt to Silty Clay	Granular B Type II	Clear Stone
Yielding wall	2,475 Yr	0.32 g	0.41	0.30	0.28
Non-yielding wall	2,475 Yr	0.32 g	0.64	0.48	0.45

The above K_{AE} values for yielding walls are applicable provided that the wall can move up to 250A mm, where A is the design peak horizontal ground acceleration (0.32g). This corresponds to displacements of up to approximately 80 mm at this site.

It should be noted that the above seismic earth pressure coefficients assume that the back of the wall is vertical and that the ground surface behind the wall is flat. Where sloping backfill is present above the top of the wall, the lateral earth pressures under seismic loading conditions should be calculated by treating the weight of the backfill located above the top of the wall as a surcharge.

5.7 Excavations and Groundwater Control

5.7.1 Excavations

Excavations for the underground parking area and pile caps construction will be made through the fill and underlaying clayey silt to silty clay deposit.

It is expected that the excavation will extend up to approximately 3.0 m below the existing ground surface to accommodate the foundations and underground space construction. Excavations may also be required for site services.

No unusual problems are anticipated with excavating the overburden materials using large hydraulic excavating equipment.

In accordance with the Occupational Health and Safety Act (OHSA) of Ontario, the fill materials above the groundwater table and the clayey silt and silty clay would generally be classified as a Type 3 soils and therefore, the side slopes should be stable in the short term at 1 horizontal to 1 vertical.

The clayey silt to silty clay deposit encountered at this site is sensitive to disturbance. Therefore, consideration should be given to protecting the subgrade in foundation areas with a mud slab of lean concrete or a layer of compacted granular fill materials such as a 0.3 m thick pad of OPSS Granular A or B Type II, possibly underlain by a geotextile. The thickness of the mud slab and compacted granular fill working mat will depend on the size and weight of the equipment to be used at the bottom of the excavation. Any disturbed soil will need to be removed prior to placing the protective layer.

The mud slab and granular fill materials should be placed immediately following inspection and approval of the subgrade. The period of time between exposure of the subgrade and covering with the protective layer should be limited to as brief as possible and, in the interim, no construction traffic should be permitted on the subgrade.

5.7.2 Groundwater Control

Excavations deeper than about 2.5 m below the existing ground surface may extend below the groundwater level. The floor of excavations below the groundwater would be within the clayey silt to silty clay. Groundwater inflow into the excavations should however be feasibly handled by pumping from sumps within the excavations. 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.

A detailed inflow analysis has not been completed, but based on the soil and groundwater conditions as well as possible size of the foundation excavations, it is considered likely that an EASR would be required during construction of the project.

The requirement for either an EASR or PTTW should be confirmed during detailed design when the actual excavation depths and extents are known.

5.8 Floor Slab

For predictable performance of the floor slab, the existing topsoil, fill, and any wet or disturbed material should be removed from underneath the underground parking floor slab and grade beams. Any soft or weak areas should be excavated and replaced with engineered fill.

Provision should be made for at least 150 mm of OPSS.MUNI 1010 Granular A compacted to 100% of the material's Standard Proctor Maximum Dry Density (SPMDD) to form the base for the floor slab. Any engineered fill required to raise the grade to the underside of the Granular A, including the repair of weak or soft areas, should consist of OPSS.MUNI 1010 Granular B Type II. The underslab fill should be placed in maximum 300 mm thick lifts and compacted to at least 95% of the material's SPMDD using suitable vibratory compaction equipment.

Provision should be made for a drainage layer, consisting of a layer of clear stone, immediately below the slab and drainage piping leading to a positive outlet.

5.9 Ground Movements

During the excavation of the one-level underground parking space (where they are near the existing structures such as the bus station), lateral deformation and vertical settlement of the adjacent ground may occur as a result of installation and deflection of the excavation activities. The ground movements induced could affect the stability or performance of structures and buildings or underground utilities adjacent to the excavation. Therefore, the magnitude and extent of ground movement and potential impacts on surrounding infrastructure should be assessed prior to construction to confirm movements will be in tolerable limits and monitored during construction.

Protective measures such as temporally shoring for the underground space excavation should be adopted where the excavations interfere with the zone of influence of the adjacent building foundations. Additional input can be provided once the extent and depth of required excavations are known.

Since the majority of the excavations will be carried above the groundwater level, no significant groundwater level reduction induced settlement, as a result of dewatering is anticipated during the construction.

5.10 Vibration Monitoring

Due to the close proximity of the existing surrounding structures to the proposed development (i.e., West parking of Bayshore Shopping Mall, bus station and residential buildings located to the north of the site), construction vibration, particularly if driven piles are adopted, 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.

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

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

5.11 Foundation Backfill

The surficial fill at this site is potentially frost susceptible and should not be used as backfill against the foundation elements (e.g., grade beams/pile caps, foundation walls, etc.). 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 for Ontario Provincial Standard Specification (OPSS) Granular B Type I, Granular B Type II, or Granular A.

To avoid ground settlements around the foundations, which could affect site grading and drainage, all of the backfill materials should be placed in maximum 300 mm lifts and be compacted to at least 95% SPMDD.

If hard surfacing will be provided in the area over the edge of the foundation, differential frost heaving could occur between the granular fill and other areas. To reduce this differential heaving, the granular backfill adjacent to the foundations may be placed to form a frost taper. The frost taper should be brought up to pavement subgrade level from 1.5 m below the finished exterior grade at a slope of 3 horizontal to 1 vertical, or flatter, away from the wall.

The foundation wall backfill for the underground parking area 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 pit from which the water is pumped.

5.12 Site Servicing

At least 150 mm of OPSS Granular A should be used as pipe bedding for sewer and water pipes. Where unavoidable disturbance to the subgrade surface occurs, or if fill material is located below the invert of the pipe, it will be necessary to remove the disturbed material or fill, and place a sub-bedding layer consisting 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% of the material’s SPMDD. 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 or surrounding soil could potentially migrate into the voids in the clear crushed stone and cause loss of lateral pipe support.

Cover material, from spring line of the pipe to at least 300 mm above the top of pipe, should consist of OPSS Granular A or Granular B Type I with a maximum particle size of 25 mm. 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 and clayey silt to 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 m depth) should match the soil exposed on the trench walls for frost heave compatibility. Trench backfill should be placed in maximum 300 mm thick lifts and should be compacted to at least 95% of the material's SPMDD using suitable vibratory compaction equipment.

Since the site of the proposed development is considered liquefiable with potential post-earthquake differential and total settlements, it is recommended to use flexible couplings that can accommodate large displacements where the services connect into the structures.

5.13 Pavement Design

In preparation for pavement construction, all topsoil, disturbed, or otherwise deleterious materials (i.e., those materials containing organic material) should be removed from the roadway areas. To minimize potential for disturbance, the general grade should not be cut to final subgrade level until all services have been installed.

Sections requiring grade raising to the proposed subgrade level should be filled using acceptable (compactable and inorganic) earth borrow or OPSS Select Subgrade Material (SSM). These materials should be placed in maximum 300 mm thick lifts and should be compacted to at least 95% of the material's SPMDD using suitable compaction equipment.

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 10 horizontal to 1 vertical and should be provided where necessary.

5.13.1 Pavement Drainage

The surface of the pavement subgrade should be crowned to promote drainage of the roadway granular structure. The subgrade surface 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 the City of Ottawa Specification F-4050 "Pipe Subdrain" and as per City of Ottawa Drawing No. R1. The geotextile should consist of a Class I nonwoven geotextile to OPSS 1860. The geotextile should have a maximum Apparent Opening Size A.O.S. of 212 µm. 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.

Backfilling of catch basin laterals located below subgrade level should be completed using acceptable native soils or fill which match the material types exposed on the lateral trench walls. This will reduce potential problems associated with differential frost heaving.

5.13.2 Granular Pavement Materials

Good drainage significantly improves the freeze-thaw resistance of the asphaltic concrete and decreases the frequency of transverse cracking, thereby extending the life of the pavement. The granular base and subbase for new construction should consist of Granular O and Granular B Type II (City of Ottawa F-3147), respectfully. Granular O provides superior drainage of the pavement and will decrease the amount of pavement transverse cracking. This will decrease maintenance and extend the life of the pavement. However, if the exposed granular base is to remain open to traffic over extended periods of time, consideration may be given to the use of Granular A in lieu of Granular O.

5.13.3 Pavement Design

The pavement structure for local roads or parking lots, which will not experience bus or truck traffic (other than school bus and garbage collection), should consist of:

Pavement Component	Thickness (mm)
Asphaltic Concrete OPSS Granular A Base OPSS Granular B Type II Subbase	90 150 400

The pavement structure for roadways which will experience bus and/or truck traffic as well as fire routes should consist of:

Pavement Component	Thickness (mm)
Asphaltic Concrete OPSS Granular A Base OPSS Granular B Type II Subbase	120 150 450

The granular base and subbase materials should be uniformly compacted to at least 100% of material's SPMDD using suitable vibratory compaction equipment. The asphaltic concrete should be compacted in accordance with Table 10 of OPSS 310. T

The composition of the asphaltic concrete pavement with 90 mm thickness should be as follows:

- Superpave 12.5 mm Surface Course – 40 mm
- Superpave 19.0 mm Base Course – 50 mm

The composition of the asphaltic concrete pavement with 120 mm thickness should be as follows:

- Superpave 12.5 mm Surface Course – 50 mm
- Superpave 19.0 mm Base Course – 70 mm

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

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

The above pavement design is 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 density and the subgrade surface not disturbed by construction operations or precipitation). Depending on the actual conditions of the pavement subgrade at the time of construction, it could be necessary to increase the thickness of the subbase and/or to place a woven geotextile beneath the granular materials.

5.13.4 Pavement Structure Compaction

Adequate compaction of the granular materials will be essential to the continued acceptable performance of the roadway and parking areas. Compaction should be carried out in conformance with procedures outlined in OPSS 501 “Construction Specification for Compacting” with compacted densities of the various materials being in accordance with Subsection 501.08.02 Method A. The granular base and subbase material should be uniformly compacted to at least 100% of the material’s SPMDD using suitable vibratory compaction equipment. Compaction of the asphaltic concrete should be carried out in accordance with OPSS 310, Table 10.

The placement and compaction of any engineered fill, as well as sewer and watermain bedding and backfill, should be inspected to ensure that the materials used conform to the specifications from both a grading and compaction view point. In addition, compaction testing and sampling of the asphaltic concrete used on site should be carried out to make sure that the materials used and level of compaction achieved during construction meet the project requirements.

5.13.5 Joints, Tie-ins with Existing Pavements, Pavement Resurfacing

At intersections with roadways at the project extents, the new pavement structure should be continued at least to the limits of construction or the end of the curb “return” (i.e., the start of the constant width portion of the side road). At these streets, the pavement should be milled back beyond the curb return an additional 300 mm to a depth of 40 mm to accept the new surface course asphaltic concrete.

The granular courses and subbase level should be tapered between the new and existing pavements by using 10 horizontal to 1 vertical tapers up or down as required, starting from behind the curb return. At driveways and commercial entrances, butt joints may be used.

A tack coat should be provided on all and vertical and milled horizontal surfaces. The tack coat should consist of SS-1 emulsified asphalt diluted with an equal amount of water. The undiluted and emulsified asphalt shall be in conformance with OPSS 1103.

5.14 Site Grading

The subsurface conditions at this site generally consist of fill underlain by a deposit of clayey silt to silty clay overlying layered deposits of silts and sands, in turn underlain by sand and gravel over bedrock. As the proposed Tower and podium structure will be supported on a deep foundation system, no practical restrictions would apply to the thickness of grade raise fill which may be placed on the site from a foundation design perspective.

As a more general guideline, preparation of the site for development/construction should include removal of the fill from building areas and stripping of the topsoil from building and pavement areas. The topsoil is not suitable as engineered fill and should be stockpiled separately for re-use in landscaping applications. In areas with no proposed structures, services, or roadways, the topsoil could be left in place provided some settlement of the ground surface following filling can be tolerated.

5.15 Material Reuse

The fill and native clayey silt to silty clay and fill materials encountered at this site are not considered to be generally suitable for reuse as structural/engineered fill. Within foundation areas, imported engineered fill such as OPSS Granular B Type II should be used (if required). The fill and native clayey silt to silty clay could however be reused in non-structural areas (i.e., landscaping).

5.16 Trees

The silty clay soils in Ottawa are 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.

This site is underlain by a deposit of sensitive silty clay and clayey silt. However, given that the structures at this site will be supported on deep foundations, no restrictions on tree planting are considered to be necessary.

5.17 Corrosion and Cement Type

The concentration of sulphate provides an indication of the degree of sulphate attack that is expected for concrete in contact with soil and groundwater at the site. The sulphate results (see Section 4.10) were compared with Table 3 of Canadian Standards Association Standards A23.1-14 (CSA A23.1) and generally indicate a low degree of sulphate attack potential on concrete structures at the locations of all tested samples. Therefore, concrete made with Type GU Portland cement is considered acceptable for all substructures.

The pH, resistivity and chloride concentration provide an indication of the degree of corrosiveness of the sub-surface environment. Generally, the results indicate an elevated potential for corrosion of exposed ferrous metal within the study area, which should be taken into consideration in the design of substructures.

6.0 ADDITIONAL CONSIDERATIONS

The guidelines in this report have been developed on the basis of the structures on this site being designed in accordance with Part 4 of the OBC.

The soils at this site are sensitive to disturbance from ponded water, construction traffic and frost. If construction is carried out during periods of sustained below freezing temperatures, all subgrade areas should be protected from freezing (e.g., by using insulated tarps and/or heating).

All footing and subgrade areas should be inspected by experienced geotechnical personnel prior to filling or concreting to document that the correct/expected strata exist and that the bearing surfaces have been properly prepared. The placing and compaction of any engineered fill, pipe bedding, and pavement base and subbase materials should be inspected to ensure that the materials used conform to the specifications from both a grading and compaction point of view.

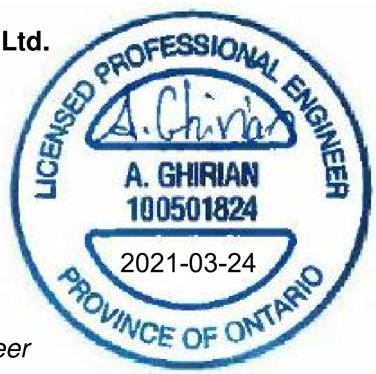
At the time of the writing of this report, only conceptual details for the proposed development were available. Golder Associates should review the final drawings and specifications for this project prior to tendering to confirm that the guidelines in this report have been adequately interpreted and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

7.0 CLOSURE

We trust that this report contains sufficient information for your present purposes. If you have any questions regarding this report or if we can be of further service to you on this project, please call us.

Yours truly,

Golder Associates Ltd.



Ali Ghirian, P.Eng.
Geotechnical Engineer

Bill Cavers, P.Eng.
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AG/WC/MSS/hdw

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https://golderassociates.sharepoint.com/sites/119469/project files/6 deliverables/geotechnical report/final/19134931-3000-r-rev0-geotech investigation-bayshore residential-march 24, 2021.docx

GOLDER

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, **Ivanhoé Cambridge.** 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 by those parties. The Client and Approved Users may not give, lend, sell, or otherwise make available 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.

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.

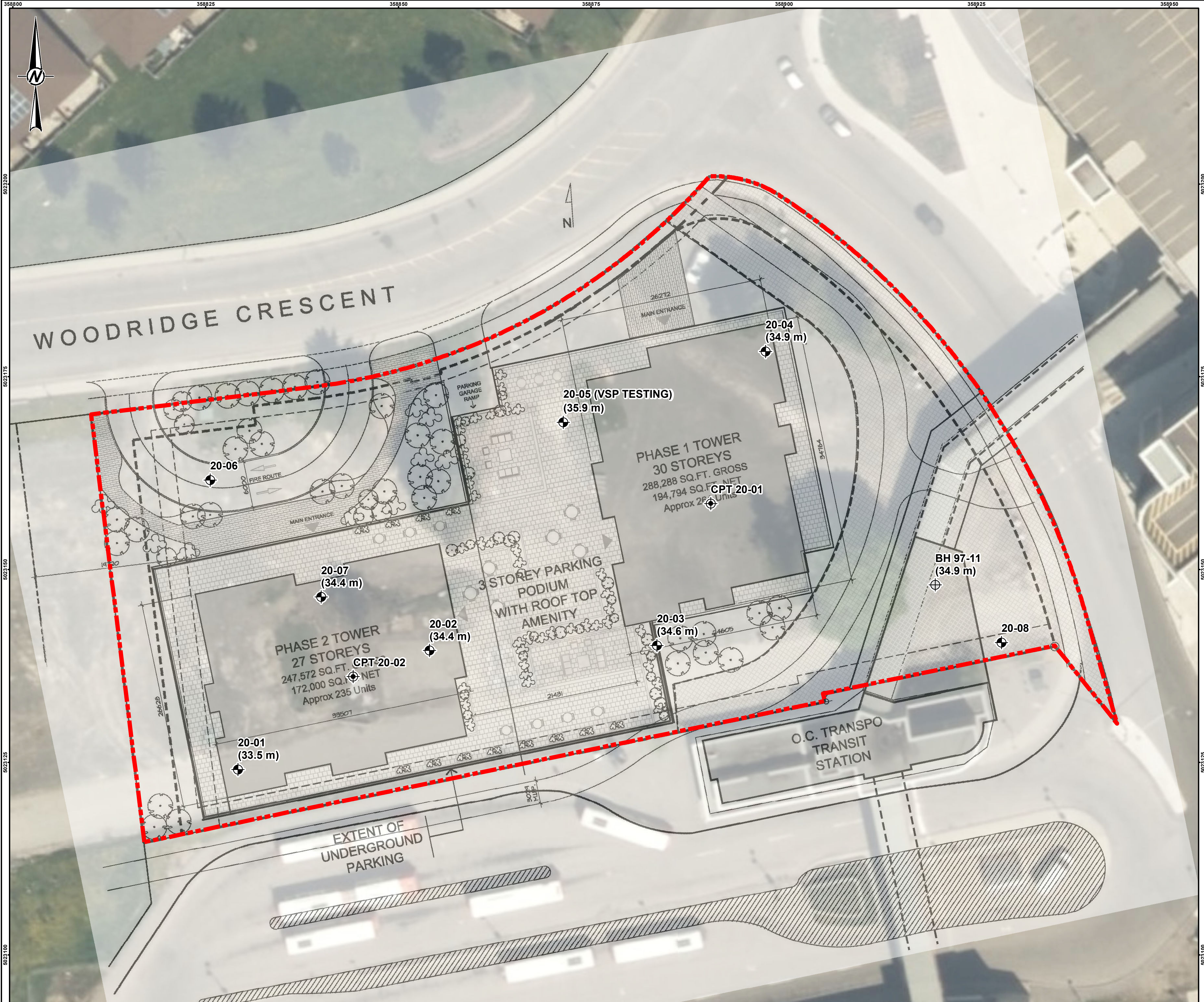
Sample Disposal: Golder will dispose of all uncontaminated soil and/or rock samples 90 days following issue of this report or, upon written request of the Client, will store uncontaminated samples and materials at the Client's expense. In the event that actual contaminated soils, fills or groundwater 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.

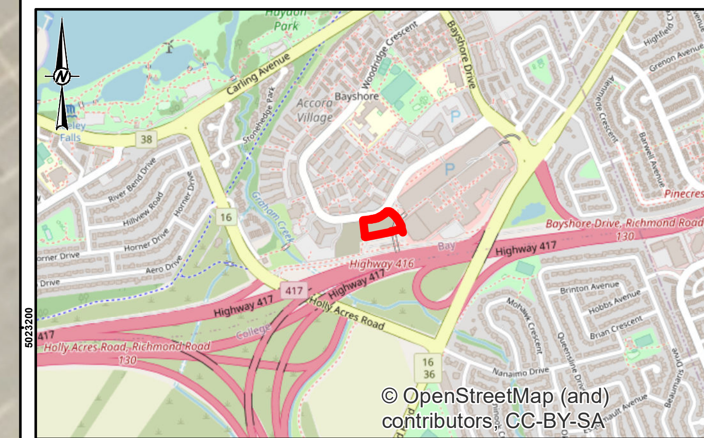
During construction, Golder should be retained to perform sufficient and timely observations of encountered conditions to confirm and document that the subsurface conditions do not materially differ from those interpreted conditions considered in the preparation of Golder's report and to confirm and 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.



KEY MAP



SCALE 1:25,000

LEGEND

- APPROXIMATE BOREHOLE LOCATION, CURRENT INVESTIGATION
- APPROXIMATE CPT LOCATION, CURRENT INVESTIGATION
- APPROXIMATE BOREHOLE LOCATION, PREVIOUS INVESTIGATION
- (34.6 m) DEPTH TO BEDROCK
- APPROXIMATE SITE BOUNDARY

NOTE(S)

1. ALL LOCATIONS ARE APPROXIMATE

REFERENCE(S)

1. LAND INFORMATION ONTARIO (LIO) DATA PRODUCED BY GOLDER ASSOCIATES LTD. UNDER LICENCE FROM ONTARIO MINISTRY OF NATURAL RESOURCES, © QUEENS PRINTER 2014
2. PROJECTION: TRANSVERSE MERCATOR, DATUM: NAD 83, COORDINATE SYSTEM: MTM ZONE 9, VERTICAL DATUM: CGVD28

0 5 10 20

1:500 METRES

CLIENT

IVANHOÉ CAMBRIDGE

PROJECT

GEOTECHNICAL INVESTIGATION
100 BAYSHORE DRIVE, OTTAWA, ONTARIO

TITLE

SITE PLAN

CONSULTANT

GOLDER

YYYY-MM-DD	2020-08-05
DESIGNED	----
PREPARED	JEM
REVIEWED	AG
APPROVED	WC

PROJECT NO.	CONTROL	REV.	FIGURE
19134931	0003	0	1

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

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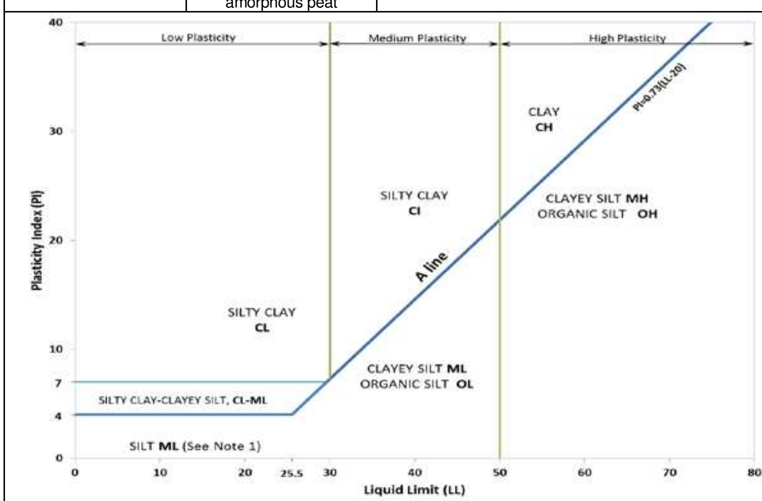
APPENDIX A

Record of Borehole Sheets – Current Investigation

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	$Cu = \frac{D_{60}}{D_{10}}$	$Cc = \frac{(D_{30})^2}{D_{10} x D_{60}}$	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: Dilatancy	Dry Strength	Shine Test	Thread Diameter	Toughness (of 3 mm thread)	Organic Content	USCS Group Symbol	Primary Name
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 $= \sigma'_p / \sigma'_{vo}$

(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 δ
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

LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

WEATHERINGS STATE

Fresh: no visible sign of rock material weathering.

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

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

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

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

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

BEDDING THICKNESS

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

JOINT OR FOLIATION SPACING

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

GRAIN SIZE

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

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

CORE CONDITION

Total Core Recovery (TCR)
The percentage of solid drill core recovered regardless of quality or length, measured relative to the length of the total core run.

Solid Core Recovery (SCR)
The percentage of solid drill core, regardless of length, recovered at full diameter, measured relative to the length of the total core run.

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

DISCONTINUITY DATA

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

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

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

Abbreviations

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

RECORD OF BOREHOLE: 20-01

PROJECT: 19134931 — SHEET 1 OF 5

LOCATION: N 5021705.3 ;E 436503.7 — BORING DATE: June 29, 2020 — DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm — PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	DESCRIPTION	ELEV. / DEPTH (m)	NUMBER	TYPE	BLOWS/0.30m	SHEAR STRENGTH Cu, kPa	WATER CONTENT PERCENT / HYDRAULIC CONDUCTIVITY	PIEZOMETER OR STANDPIPE INSTALLATION
		GROUND SURFACE	66.31						
0	Power Auger 200 mm Diam. (Hollow Stem)	TOPSOIL - (SM) SILTY SAND; grey brown, contains organics; non-cohesive, dry, loose	0.00						Flush Mount Casing
		FILL - (SP) gravelly SAND, fine to coarse, angular; grey, contains rootlets; non-cohesive, dry, compact	66.11 / 0.20	1	SS	17			Bentonite and Cuttings
1		FILL - (CL/ML) SILTY CLAY to CLAYEY SILT, trace sand; grey brown, slightly fissured; w<PL, very stiff	65.70 / 0.61	2	SS	25			
2		(ML/CL) CLAYEY SILT to SILTY CLAY; grey, fissured (WEATHERED CRUST); cohesive, w<PL, stiff to very stiff	64.94 / 1.37	3	SS	22			Bentonite Seal; Silica Sand
				4	SS	11			
3		(CL/ML) SILTY CLAY to CLAYEY SILT; grey; cohesive, w>PL, soft to very soft	63.26 / 3.05	5	SS	4			32 mm Diam. PVC #10 Slot Screen
4				6	SS	4			
5	Wash Boring HW Casing			7	SS	2			
							⊕ / +		WL in Screen at Elev. 63.596 m on August 10, 2020
6				8	SS	17			
7		(ML) CLAYEY SILT to sandy SILT; grey; non-cohesive, w>PL, very loose	58.69 / 7.62	9	SS	WH			
8									
9									
10									

CONTINUED NEXT PAGE

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m: 20 40 60 80
SHEAR STRENGTH Cu, kPa: nat V. + Q - ● rem V. ⊕ U - ○ ; 20 40 60 80
HYDRAULIC CONDUCTIVITY, k, cm/s: 10^{-6} 10^{-5} 10^{-4} 10^{-3}
WATER CONTENT PERCENT: Wp —○W— Wl ; 20 40 60 80
ADDITIONAL LAB. TESTING

DEPTH SCALE 1 : 50 — GOLDER — LOGGED: AK — CHECKED: AG

PROJECT: 19134931

RECORD OF BOREHOLE: 20-01

LOCATION: N 5021705.3 ;E 436503.7

BORING DATE: June 29, 2020

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	ELEV. / DEPTH (m)	SAMPLES: NUMBER	TYPE	BLOWS/0.30m	WATER CONTENT PERCENT
10	Wash Boring, HW Casing	--- CONTINUED FROM PREVIOUS PAGE --- (ML) CLAYEY SILT to sandy SILT; grey; non-cohesive, w>PL, very loose					
		(ML/SM) sandy SILT to SILTY SAND; grey, contains clay seams; non-cohesive, wet, loose	55.65 / 10.66				
11				10	SS	WH	~25 (W)
14		(SP) SAND, some gravel, fine to coarse, angular, trace non-plastic fines; grey brown; non-cohesive, wet, loose to dense	52.60 / 13.71	11	SS	40	~21 (W)
17				12	SS	6	~24 (W)
20		- contains cobbles		13	SS	12	

Chart column headings: DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20, 40, 60, 80); SHEAR STRENGTH Cu, kPa, nat V. + Q - ●, rem V. ⊕ U - ○ (20, 40, 60, 80); HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-6}, 10^{-5}, 10^{-4}, 10^{-3}); WATER CONTENT PERCENT, Wp ⊢—○W—⊣ Wl (20, 40, 60, 80); ADDITIONAL LAB. TESTING; PIEZOMETER OR STANDPIPE INSTALLATION; STRATA PLOT

CONTINUED NEXT PAGE

DEPTH SCALE 1 : 50

GOLDER

LOGGED: AK

CHECKED: AG

LOCATION: N 5021705.3 ;E 436503.7

BORING DATE: June 29, 2020

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES: NUMBER	TYPE	BLOWS/0.30m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20 40 60 80); SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - ○ (20 40 60 80)	HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-6} 10^{-5} 10^{-4} 10^{-3}); WATER CONTENT PERCENT Wp ⊢—○W—⊣ Wl (20 40 60 80)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		--- *CONTINUED FROM PREVIOUS PAGE* ---									
20	Wash Boring, HW Casing	(SP) SAND, some gravel, fine to coarse, angular, trace non-plastic fines; grey brown; non-cohesive, wet, loose to dense			13	SS	12				
21					14	SS	25				
22											
23											
24					15	SS	40				
25											
26											
27				38.88 / 27.43							
		(SW/GW) SAND and GRAVEL, angular to sub-rounded, trace non-plastic fines; grey, contains cobbles and boulders; non-cohesive, wet, dense to very dense			16	SS	100				
28											
29											
30											

CONTINUED NEXT PAGE

PROJECT: 19134931

RECORD OF BOREHOLE: 20-01

LOCATION: N 5021705.3 ;E 436503.7

BORING DATE: June 29, 2020

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.30m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20 40 60 80); SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - ○ (20 40 60 80)	HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-6} 10^{-5} 10^{-4} 10^{-3}); WATER CONTENT PERCENT Wp W Wl (20 40 60 80)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		--- CONTINUED FROM PREVIOUS PAGE ---									
30	Wash Boring, HW Casing	(SW/GW) SAND and GRAVEL, angular to sub-rounded, trace non-plastic fines; grey, contains cobbles and boulders; non-cohesive, wet, dense to very dense									
					17	SS	52				
31											
32											
33				32.79 / 33.52							
		Borehole continued on RECORD OF DRILLHOLE 20-01									
34											
35											
36											
37											
38											
39											
40											

DEPTH SCALE

1 : 50

GOLDER

LOGGED: AK

CHECKED: AG

PROJECT: 19134931

RECORD OF DRILLHOLE: 20-01

LOCATION: N 5021705.3 ;E 436503.7

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: June 29, 2020

DRILL RIG: CME-850

DRILLING CONTRACTOR: CCC Drilling

DATUM: Geodetic

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

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

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	FLUSH COLOUR % RETURN	RECOVERY TOTAL CORE %	RECOVERY SOLID CORE %	R.Q.D. %	FRACT. INDEX PER 0.25 m	DIP w.r.t. CORE AXIS	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	Jcon	Jr	Ja	HYDRAULIC CONDUCTIVITY K, cm/sec	Diametral Point Load Index (MPa)	RMC -Q' AVG.
		GROUND SURFACE		32.79														
34	Rotary Drill, HQ Core	Fresh, thinly to medium bedded, medium grey, fine grained, non-porous, very strong DOLOSTONE, with thin laminations to very thin beds of dark grey to black, non-porous, medium strong to weak shale and limestone		33.52	1													
					2													
35					3													
				30.56														
		End of Drillhole		35.75														
36																		
37																		
38																		
39																		
40																		
41																		
42																		
43																		

DEPTH SCALE
1 : 50

GOLDER

LOGGED: AK
CHECKED: AG

RECORD OF BOREHOLE: 20-02

PROJECT: 19134931

LOCATION: N 5021720.3 ;E 436528.9

BORING DATE: July 2, 2020

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE – DESCRIPTION	ELEV. / DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.30m	PIEZOMETER OR STANDPIPE INSTALLATION
		GROUND SURFACE	66.82				
0	Power Auger, 200 mm Diam. (Hollow Stem)	TOPSOIL - (SP) SAND, fine to medium; brown, contains rootlets, organics; non-cohesive, very dense	0.00				Flush Mount Casing
		FILL - (SM) gravelly SILTY SAND, fine to coarse; brown; non-cohesive, dry to moist, dense	66.57 / 0.25	1	SS	61	
1				2	SS	37	
2				3	SS	35	Bentonite Seal; Silica Sand
		(CL/ML) CLAYEY SILT to SILTY CLAY; grey with black mottling, highly fissured (WEATHERED CRUST); cohesive, w<PL to w~PL, stiff to soft	64.39 / 2.43	4	SS	9	
3				5	SS	4	32 mm Diam. PVC #10 Slot Screen 'B'
4				6	SS	2	
	Wash Boring, HW Casing	(CL/ML) CLAYEY SILT to SILTY CLAY; grey; cohesive, w>PL, very soft	62.25 / 4.57	7	SS	WH	
5–6							
6		- sand and gravel seam from 6.1 to 6.25 m depth					
		(ML/SM) CLAYEY SILT to SILTY SAND grey; non-cohesive, wet, loose	60.57 / 6.25	8	SS	33	
7–8							Bentonite and Cuttings
9				9	SS	WH	
10		CONTINUED NEXT PAGE					

Plot columns: DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20, 40, 60, 80); SHEAR STRENGTH Cu, kPa (nat V. + Q - ●; rem V. ⊕ U - ○) (20, 40, 60, 80); HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-6}, 10^{-5}, 10^{-4}, 10^{-3}); WATER CONTENT PERCENT (Wp ⊢—○W—⊣ Wl) (20, 40, 60, 80); ADDITIONAL LAB. TESTING.

Shear strength at ~5.9 m: >96+

DEPTH SCALE 1 : 50

GOLDER

LOGGED: AK

CHECKED: AG

LOCATION: N 5021720.3 ;E 436528.9 BORING DATE: July 2, 2020 DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm PENETRATION TEST HAMMER, 64kg; DROP, 760mm

--- CONTINUED FROM PREVIOUS PAGE ---

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLE NUMBER	TYPE	BLOWS/0.30m	WATER CONTENT (w)	PIEZOMETER OR STANDPIPE INSTALLATION
10	Wash Boring, HW Casing	(ML/SM) CLAYEY SILT to SILTY SAND grey; non-cohesive, wet, loose						Bentonite and Cuttings
		(SW) gravelly SAND, trace non-plastic fines; grey, contains cobbles; non-cohesive, dense to very dense	56.15 / 10.67					Bentonite Seal
12								Silica Sand
				10	SS	35	○	
14								32 mm Diam. PVC #10 Slot Screen 'A'
15				11	SS	53	○	WL in Screen 'B' at Elev. 62.386 m on August 10, 2020; WL in Screen 'A' at Elev. 61.196 m on August 10, 2020
18		(ML) CLAYEY SILT to SILT; grey, contains clay seams; non-cohesive, w>PL, stiff	48.33 / 18.49	12	SS	39	○	
20								

CONTINUED NEXT PAGE

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m: 20 40 60 80
SHEAR STRENGTH Cu, kPa: nat V. + Q - ● rem V. ⊕ U - ○
HYDRAULIC CONDUCTIVITY, k, cm/s: 10^{-6} 10^{-5} 10^{-4} 10^{-3}
WATER CONTENT PERCENT: Wp ⊢—○W—⊣ Wl; 20 40 60 80
ADDITIONAL LAB. TESTING

PROJECT: 19134931

RECORD OF BOREHOLE: 20-02

LOCATION: N 5021720.3 ;E 436528.9

BORING DATE: July 2, 2020

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES: NUMBER	TYPE	BLOWS/0.30m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20 40 60 80); SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - ○ (20 40 60 80)	HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-6} 10^{-5} 10^{-4} 10^{-3}); WATER CONTENT PERCENT Wp ⊢ ○W ⊣ Wl (20 40 60 80)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		--- CONTINUED FROM PREVIOUS PAGE ---									
20	Wash Boring HW Casing										
		(SW/GW) SAND and GRAVEL, fine to coarse; grey brown, contains cobbles and boulders; non-cohesive, wet, compact to very dense		46.65 20.17							
21											
					13	SS	17				
22											
23											
24											
					14	SS	52				
25											
26											
27											
					15	SS	15				
28											
29											
30											

CONTINUED NEXT PAGE

DEPTH SCALE
1 : 50

GOLDER

LOGGED: AK
CHECKED: AG

PROJECT: 19134931

RECORD OF BOREHOLE: 20-02

LOCATION: N 5021720.3 ;E 436528.9

BORING DATE: July 2, 2020

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.30m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20 40 60 80); SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - ○ (20 40 60 80)	HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-6} 10^{-5} 10^{-4} 10^{-3}); WATER CONTENT PERCENT Wp —○W— Wl (20 40 60 80)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		--- CONTINUED FROM PREVIOUS PAGE ---									
30	Wash Boring, HW Casing	(SW/GW) SAND and GRAVEL, fine to coarse; grey brown, contains cobbles and boulders; non-cohesive, wet, compact to very dense									
					16	SS	37				
31											
32											
33											
34				32.42 34.4							
		Borehole continued on RECORD OF DRILLHOLE 20-02									
35											
36											
37											
38											
39											
40											

DEPTH SCALE
1 : 50

GOLDER

LOGGED: AK
CHECKED: AG

PROJECT: 19134931

RECORD OF DRILLHOLE: 20-02

SHEET 5 OF 5

LOCATION: N 5021720.3 ;E 436528.9

DRILLING DATE: July 2, 2020

DATUM: Geodetic

DRILL RIG: CME-850

INCLINATION: -90° AZIMUTH: ---

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 %	RECOVERY SOLID CORE %	R.Q.D. %	FRACT. INDEX PER 0.25 m	DIP w.r.t. CORE AXIS	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	Jcon	Jr	Ja	HYDRAULIC CONDUCTIVITY K, cm/sec	Diametral Point Load Index (MPa)	RMC -Q' AVG.
		GROUND SURFACE		32.42														
35	Rotary Drill, HQ Core	Fresh, thinly to medium bedded, medium grey, fine grained, non-porous, very strong DOLOSTONE, with thin laminations to very thin beds of dark grey to black, non-porous, medium strong to weak shale and limestone		34.40	1													
36				30.32 / 36.50	2													
		End of Drillhole																

Depth scale: 35, 36, 37, 38, 39, 40, 41, 42, 43, 44

DEPTH SCALE

1 : 50

GOLDER

LOGGED: AK

CHECKED: AG

MIS-RCK 004 19134931.GPJ GAL-MISS.GDT 3-19-21 JEM

PROJECT: 19134931

RECORD OF BOREHOLE: 20-03

SHEET 1 OF 5

LOCATION: N 5021720.4 ;E 436558.3

BORING DATE: July 7, 2020

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

Column headings: DEPTH SCALE METRES | BORING METHOD | SOIL PROFILE (DESCRIPTION, STRATA PLOT, ELEV. DEPTH (m)) | SAMPLES (NUMBER, TYPE, BLOWS/0.30m) | DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20 40 60 80); SHEAR STRENGTH Cu, kPa, nat V. + Q - ● rem V. ⊕ U - ○ (20 40 60 80) | HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-6} 10^{-5} 10^{-4} 10^{-3}); WATER CONTENT PERCENT Wp ⊢—○W—⊣ Wl (20 40 60 80) | ADDITIONAL LAB. TESTING | PIEZOMETER OR STANDPIPE INSTALLATION

Depth (m)	Boring Method	Description	Elev. / Depth (m)	Sample No.	Type	Blows/0.30m
		GROUND SURFACE	66.83			
0	Power Auger, 200 mm Diam. (Hollow Stem)	TOPSOIL - mixture of SAND and ORGANICS, fine to medium, some gravel; brown, contains rootlets; non-cohesive, dry, dense	0.00			
		FILL - (SW) gravelly SAND, fine to coarse, contains rootlets; brown; non-cohesive, dry, dense	0.17	1	SS	47
		FILL - (CL/ML) SILTY CLAY to CLAYEY SILT; dark grey to grey with black mottling; cohesive, moist to dry, very stiff	66.22 / 0.61	2	SS	24
		(CL/ML) CLAYEY SILT to SILTY CLAY; grey, highly fissured (WEATHERED CRUST); cohesive, w<PL to w~PL, very stiff to firm	65.46 / 1.37	3	SS	16
				4	SS	16
				5	SS	7
		(ML/CL) CLAYEY SILT to SILTY CLAY; grey; cohesive, w>PL, soft to very soft	63.02 / 3.81	6	SS	3
				7	SS	2
	Wash Boring, HW Casing	(ML) CLAYEY SILT to sandy SILT, some fines; grey; non-cohesive, wet, very loose	60.73 / 6.10	8	SS	WH
				9	SS	WH
10						

CONTINUED NEXT PAGE

DEPTH SCALE

1 : 50

GOLDER

LOGGED: AK

CHECKED: AG

PROJECT: 19134931

RECORD OF BOREHOLE: 20-03

LOCATION: N 5021720.4 ;E 436558.3

BORING DATE: July 7, 2020

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES: NUMBER	TYPE	BLOWS/0.30m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20 40 60 80); SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - ○ (20 40 60 80)	HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-6} 10^{-5} 10^{-4} 10^{-3}); WATER CONTENT PERCENT Wp ⊢ ○W ⊣ Wl (20 40 60 80)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		--- *CONTINUED FROM PREVIOUS PAGE* ---									
10	Wash Boring / HW Casing	(ML) CLAYEY SILT to sandy SILT, some fines; grey; non-cohesive, wet, very loose									
12		(SP) SAND, some gravel, fine to coarse; grey, contains cobbles and boulders; non-cohesive, wet, compact		54.64 12.19	10	SS	45				
16					11	SS	29		○		
18					12	SS	36		○		
20		*CONTINUED NEXT PAGE*									

DEPTH SCALE

1 : 50

GOLDER

LOGGED: AK

CHECKED: AG

RECORD OF BOREHOLE: 20-03

LOCATION: N 5021720.4 ;E 436558.3 | BORING DATE: July 7, 2020 | DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm | PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.30m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20 40 60 80); SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - ○ (20 40 60 80)	HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-6} 10^{-5} 10^{-4} 10^{-3}); WATER CONTENT PERCENT Wp ⊢ ○W ⊣ Wl (20 40 60 80)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		--- CONTINUED FROM PREVIOUS PAGE ---									
20	Wash Boring, HW Casing	(SP) SAND, some gravel, fine to coarse; grey, contains cobbles and boulders; non-cohesive, wet, compact									
21											
		(ML) CLAYEY SILT to SILT; grey; non-cohesive, wet, dense		45.49 21.34	13	SS	69				
		(SW) SAND, fine to medium, some angular gravel; grey brown; non-cohesive, wet, dense to very dense		45.10 21.73							
22											
23											
24											
		(SW/GW) SAND and GRAVEL, some non-plastic fines; grey, contains cobbles and boulders; non-cohesive, dense to very dense		42.45 24.38	14	SS	26				
25											
26											
27											
					15	SS	53				
28											
29											
30											

CONTINUED NEXT PAGE

PROJECT: 19134931

RECORD OF BOREHOLE: 20-03

LOCATION: N 5021720.4 ;E 436558.3

BORING DATE: July 7, 2020

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.30m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20 40 60 80); SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - ○ (20 40 60 80)	HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-6} 10^{-5} 10^{-4} 10^{-3}); WATER CONTENT PERCENT Wp —○W— Wl (20 40 60 80)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		--- CONTINUED FROM PREVIOUS PAGE ---									
30	Wash Boring, HW Casing	(SW/GW) SAND and GRAVEL, some non-plastic fines; grey, contains cobbles and boulders; non-cohesive, dense to very dense									
					16	SS	76				
31											
32											
33											
					17	SS	60				
34											
				32.27							
		Borehole continued on RECORD OF DRILLHOLE 20-03		34.56							
35											
36											
37											
38											
39											
40											

DEPTH SCALE

1 : 50

GOLDER

LOGGED: AK

CHECKED: AG

RECORD OF DRILLHOLE: 20-03

PROJECT: 19134931

SHEET 5 OF 5

LOCATION: N 5021720.4 ;E 436558.3

DRILLING DATE: July 7, 2020

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME-850

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 %	RECOVERY SOLID CORE %	R.Q.D. %	FRACT. INDEX PER 0.25 m	DIP w.r.t. CORE AXIS	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	Jcon	Jr	Ja	HYDRAULIC CONDUCTIVITY K, cm/sec	Diametral Point Load Index (MPa)	RMC -Q' AVG.
		GROUND SURFACE		32.27														
35 36	Rotary Drill HQ Core	Fresh, thinly to medium bedded, medium grey, fine grained, non-porous, very strong DOLOSTONE, with thin laminations to very thin beds of dark grey to black, non-porous, medium strong to weak shale and limestone		34.56 30.41	1 2													
		End of Drillhole		36.42														
37 38 39 40 41 42 43 44																		

DEPTH SCALE

1 : 50

GOLDER

LOGGED: AK

CHECKED: AG

MIS-RCK 004 19134931.GPJ GAL-MISS.GDT 3-19-21 JEM

RECORD OF BOREHOLE: 20-04

PROJECT: 19134931

LOCATION: N 5021758.2 ;E 436573.1

BORING DATE: July 9, 2020

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

Column headings: DEPTH SCALE METRES | BORING METHOD | SOIL PROFILE (DESCRIPTION, STRATA PLOT, ELEV. DEPTH (m)) | SAMPLES (NUMBER, TYPE, BLOWS/0.30m) | DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20, 40, 60, 80); SHEAR STRENGTH Cu, kPa (nat V. + Q - ●; rem V. ⊕ U - ○) (20, 40, 60, 80) | HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-6}, 10^{-5}, 10^{-4}, 10^{-3}); WATER CONTENT PERCENT (Wp ⊢—○W—⊣ Wl) (20, 40, 60, 80) | ADDITIONAL LAB. TESTING | PIEZOMETER OR STANDPIPE INSTALLATION

Boring method: Power Auger, 200 mm Diam. (Hollow Stem) (0 to 4.57 m); Wash Boring, HW Casing (4.57 to 10 m)

Depth (m)	Description	Elev. / Depth (m)	Sample No.	Type	Blows/0.30m	Water content (%)	Piezometer
	GROUND SURFACE	66.93					
0	FILL - (SM) gravelly SAND, fine to coarse, angular gravel; brown, contains rootlets and organics; non-cohesive, dry, compact	0.00	1	SS	26		Flush Mount Casing
			2	SS	18		
	FILL - (CL/CI) SILTY CLAY, trace sand; grey; cohesive, w<PL, very stiff	65.71 / 1.22	3	SS	20		
	(CL/ML) CLAYEY SILT to SILTY CLAY, trace sand; grey, fissured (WEATHERED CRUST); cohesive, w<PL to w~PL, stiff to firm	64.64 / 2.29	4	SS	14	○	
			5	SS	5	○	
			6	SS	3	○	
	(CL/ML) SILTY CLAY to CLAYEY SILT, trace fines; grey; cohesive, w>PL, soft	62.36 / 4.57	7	SS	2	○	Bentonite Seal
							▽
	(ML) CLAYEY SILT to SILT; grey to grey brown, contains clay seams; non-cohesive, wet, very loose to compact	59.31 / 7.62	8	SS	WH	○	
10	*CONTINUED NEXT PAGE*						

DEPTH SCALE

1 : 50

GOLDER

LOGGED: AK

CHECKED: AG

PROJECT: 19134931

RECORD OF BOREHOLE: 20-04

SHEET 2 OF 5

LOCATION: N 5021758.2 ;E 436573.1

BORING DATE: July 9, 2020

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES | BORING METHOD | SOIL PROFILE (DESCRIPTION, STRATA PLOT, ELEV. DEPTH (m)) | SAMPLES (NUMBER, TYPE, BLOWS/0.30m) | DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20, 40, 60, 80) | SHEAR STRENGTH Cu, kPa (nat V. + Q - ●; rem V. ⊕ U - ○) (20, 40, 60, 80) | HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-6}, 10^{-5}, 10^{-4}, 10^{-3}) | WATER CONTENT PERCENT (Wp —○W— Wl) (20, 40, 60, 80) | ADDITIONAL LAB. TESTING | PIEZOMETER OR STANDPIPE INSTALLATION

--- CONTINUED FROM PREVIOUS PAGE ---

Boring method: Wash Boring, HW Casing

Depth (m)	Description	Elev. / Depth (m)
10	(ML) CLAYEY SILT to SILT; grey to grey brown, contains clay seams; non-cohesive, wet, very loose to compact	
	(ML/SM) sandy SILT to SILTY SAND; grey; non-cohesive, moist, dense	56.26 / 10.67
	(SP) SAND, fine to medium, some gravel; grey brown; non-cohesive, wet, dense	50.17 / 16.76
20	- becoming well graded	

Sample Number	Type	Blows/0.30m	Approx. Depth (m)
9	SS	18	11
10	SS	33	14
11	SS	33	17
12	SS	11	20

Piezometer installation notes:

Bentonite Seal

Silica Sand

32 mm Diam. PVC #10 Slot Screen

WL in Screen at Elev. 61.279 m on August 10, 2020

CONTINUED NEXT PAGE

DEPTH SCALE 1 : 50

GOLDER

LOGGED: AK

CHECKED: AG

PROJECT: 19134931

RECORD OF BOREHOLE: 20-04

LOCATION: N 5021758.2 ;E 436573.1

BORING DATE: July 9, 2020

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

--- CONTINUED FROM PREVIOUS PAGE ---

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	SAMPLES TYPE	BLOWS/0.30m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20 40 60 80); SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - ○ (20 40 60 80)	HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-6} 10^{-5} 10^{-4} 10^{-3}); WATER CONTENT PERCENT Wp ⊢—○W—⊣ Wl (20 40 60 80)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
20	Wash Boring / HW Casing	(SP) SAND, fine to medium, some gravel; grey brown; non-cohesive, wet, dense			12	SS	11		○		
21											
22											
		(SW/GW) SAND and GRAVEL; grey, contains cobbles and boulders; non-cohesive, wet, dense to very dense		44.07 / 22.86							
23					13	SS	39				
24											
25											
26					14	SS	22				
27											
28											
29					15	SS	12				
30											

CONTINUED NEXT PAGE

DEPTH SCALE

1 : 50

GOLDER

LOGGED: AK

CHECKED: AG

RECORD OF BOREHOLE: 20-04

PROJECT: 19134931

LOCATION: N 5021758.2 ;E 436573.1

BORING DATE: July 9, 2020

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE – DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.30m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20 40 60 80); SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - ○ (20 40 60 80)	HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-6} 10^{-5} 10^{-4} 10^{-3}); WATER CONTENT PERCENT Wp ⊢—○W—⊣ Wl (20 40 60 80)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		--- CONTINUED FROM PREVIOUS PAGE ---									
30	Wash Boring / HW Casing	(SW/GW) SAND and GRAVEL; grey, contains cobbles and boulders; non-cohesive, wet, dense to very dense									
32					16	SS	17				
35				32.00 / 34.93							
		Borehole continued on RECORD OF DRILLHOLE 20-04									

DEPTH SCALE
1 : 50

GOLDER

LOGGED: AK
CHECKED: AG

RECORD OF DRILLHOLE: 20-04

PROJECT: 19134931

LOCATION: N 5021758.2 ;E 436573.1

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: July 9, 2020

DRILL RIG: CME-850

DRILLING CONTRACTOR: CCC Drilling

DATUM: Geodetic

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

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

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.
		GROUND SURFACE		32.00	
35	Rotary Drill HQ Core	Fresh, thinly to medium bedded, medium grey, fine grained, non-porous, very strong DOLOSTONE, with thin laminations to very thin beds of dark grey to black, non-porous, medium strong to weak shale and limestone		34.93	1
36					
37					2
38				28.48 38.45	3
		End of Drillhole			
39					
40					
41					
42					
43					
44					

Other column headings: COLOUR % RETURN; FLUSH; RECOVERY (TOTAL CORE %, 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, TYPE AND SURFACE DESCRIPTION, Jcon, Jr, Ja; HYDRAULIC CONDUCTIVITY K, cm/sec 10^{-6} 10^{-5} 10^{-4} 10^{-3}; Diametral Point Load Index (MPa) 2 4 6; RMC -Q' AVG.

DEPTH SCALE

1 : 50

GOLDER

LOGGED: AK

CHECKED: AG

PROJECT: 19134931

RECORD OF BOREHOLE: 20-05

SHEET 1 OF 5

LOCATION: N 5021749.5 ;E 436546.8

BORING DATE: June 10, 2020

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE – DESCRIPTION	ELEV. DEPTH (m)	SAMPLE NUMBER	TYPE	BLOWS/0.30m
		GROUND SURFACE	67.67			
0	Power Auger, 200 mm Diam. (Hollow Stem)	TOPSOIL - (SM) SILTY SAND, some gravel; brown, contains organics; non-cohesive, dry, loose	0.00	1	SS	48
		FILL - (SW) gravelly SAND, non-plastic fines; brown to grey; non-cohesive, moist, compact	0.15			
1				2	SS	10
		(ML/CL) CLAYEY SILT to SILTY CLAY, some gravel and sand; grey with mottling and fissuring (WEATHERED CRUST); cohesive, w<PL, stiff to very stiff	66.15 / 1.52	3	SS	13
2				4	SS	14
3				5	SS	13
4				6	SS	6
		(ML/CL) CLAYEY SILT to SILTY CLAY; brown grey, contains layers of sandy silt; cohesive, w>PL, firm to soft	63.10 / 4.57	7	SS	2
5	Wash Boring, HW Casing	(ML/CL) CLAYEY SILT to SILTY CLAY; grey, contains sandy silt layers; cohesive, w>PL, stiff or loose	62.49 / 5.18	8	SS	WH
6		(ML) CLAYEY SILT to sandy SILT; grey; non-cohesive, wet, loose	61.57 / 6.10	9	SS	3
7						
8				10	SS	1
		- layers of stiff silty clay		11	SS	WH
9				12	SS	2
10				13	SS	3

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

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

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

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

ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION

CONTINUED NEXT PAGE

DEPTH SCALE 1 : 50

GOLDER

LOGGED: JS

CHECKED: AG

MIS-BHS 001 19134931.GPJ GAL-MIS.GDT 3-19-21 JEM

RECORD OF BOREHOLE: 20-05

PROJECT: 19134931

LOCATION: N 5021749.5 ;E 436546.8

BORING DATE: June 10, 2020

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

Column headings: DEPTH SCALE METRES | BORING METHOD | SOIL PROFILE (DESCRIPTION, STRATA PLOT, ELEV. DEPTH (m)) | SAMPLES (NUMBER, TYPE, BLOWS/0.30m) | DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20, 40, 60, 80); SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - ○ (20, 40, 60, 80) | HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-6}, 10^{-5}, 10^{-4}, 10^{-3}); WATER CONTENT PERCENT Wp ⊢—○W—⊣ Wl (20, 40, 60, 80) | ADDITIONAL LAB. TESTING | PIEZOMETER OR STANDPIPE INSTALLATION

Boring method: Wash Boring, HW Casing

--- CONTINUED FROM PREVIOUS PAGE ---

Description	Elev. / Depth (m)
(ML) CLAYEY SILT to sandy SILT; grey; non-cohesive, wet, loose	57.00 / 10.67
(ML) sandy SILT, some plastic fines; grey; non-cohesive, wet, loose - layers of clayey silt; grey; cohesive, w>PL, firm to stiff present	56.09 / 11.58
(SW) SAND, fine to coarse, some gravel and non-plastic fines; grey; non-cohesive, moist, dense	54.72 / 12.95
(SM/ML) SILTY SAND to CLAYEY SILT; grey; non-cohesive, moist, dense	53.80 / 13.87
(GW) sandy GRAVEL, fine to coarse, trace non-plastic fines; grey; non-cohesive, wet, compact - cobbles and boulders based on resistance	51.82 / 15.85
(SW) gravelly SAND, fine to coarse, some non-plastic fines; grey; non-cohesive, wet, compact to dense - lense of sandy silt	47.93 / 19.74
(SP) SAND, fine, some non-plastic fines; grey; non-cohesive, wet, dense	

Sample Number	Type	Blows/0.30m
13	SS	3
14	SS	WH
15	SS	22
16	SS	34
17	SS	37
18	SS	28
19	SS	27
20	SS	35
21	SS	25
22	SS	29
23	SS	36
24	SS	29
25	SS	29
26	SS	41

Depth scale: 10, 11, 12, 13, 14, 15, 16, 17, 18, 19, 20

CONTINUED NEXT PAGE

PROJECT: 19134931

RECORD OF BOREHOLE: 20-05

LOCATION: N 5021749.5 ;E 436546.8

BORING DATE: June 10, 2020

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

--- CONTINUED FROM PREVIOUS PAGE ---

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	ELEV. / DEPTH (m)	SAMPLE NUMBER	TYPE	BLOWS/0.30m
20	Wash Boring, HW Casing	(SP) SAND, fine, some non-plastic fines; grey; non-cohesive, wet, dense		26	SS	41
		- lense of clayey silt				
21				27	SS	38
		(SP) SAND, fine to coarse, some gravel and non-plastic fines; grey; non-cohesive, wet, dense to very dense	46.33 / 21.34	28	SS	41
22				29	SS	45
23				30	SS	67
24		(SW/GW) SAND and GRAVEL, sub-angular to sub-rounded to compact, contains cobbles and boulders, trace to some fines; grey; non-cohesive, wet, very dense	43.29 / 24.38	31	SS	48
25						
26				32	SS	46
27		(SW/GW) SAND and GRAVEL, some fines; grey, contains cobbles and boulders; non-cohesive, wet, very dense	40.24 / 27.43	33	SS	58
28						
29				34	SS	71
30						

Column headings also include: STRATA PLOT; DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20, 40, 60, 80); SHEAR STRENGTH Cu, kPa (nat V. + Q - ●, rem V. ⊕ U - ○) (20, 40, 60, 80); HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-6}, 10^{-5}, 10^{-4}, 10^{-3}); WATER CONTENT PERCENT (Wp —○W— Wl) (20, 40, 60, 80); ADDITIONAL LAB. TESTING; PIEZOMETER OR STANDPIPE INSTALLATION

CONTINUED NEXT PAGE

DEPTH SCALE 1 : 50

GOLDER

LOGGED: JS

CHECKED: AG

PROJECT: 19134931

RECORD OF BOREHOLE: 20-05

LOCATION: N 5021749.5 ;E 436546.8

BORING DATE: June 10, 2020

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.30m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20 40 60 80); SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - ○ (20 40 60 80)	HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-6} 10^{-5} 10^{-4} 10^{-3}); WATER CONTENT PERCENT Wp ⊢—○W—⊣ Wl (20 40 60 80)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		--- CONTINUED FROM PREVIOUS PAGE ---									
30	Wash Boring / HW Casing	(SW/GW) SAND and GRAVEL, some fines; grey, contains cobbles and boulders; non-cohesive, wet, very dense									
					35	SS	93				
31											
32					36	SS	64				
33				34.17 / 33.50							
		(SW/GW) SAND and GRAVEL, sub-rounded to sub-angular; grey, contains cobbles and boulders; non-cohesive, wet, very dense			37	SS	74				
34											
35				31.73 / 35.94							
36		Borehole continued on RECORD OF DRILLHOLE 20-05									
37											
38											
39											
40											

Borehole continued on RECORD OF DRILLHOLE 20-05

DEPTH SCALE
1 : 50

GOLDER

LOGGED: JS
CHECKED: AG

RECORD OF DRILLHOLE: 20-05

LOCATION: N 5021749.5 ;E 436546.8

DRILLING DATE: June 10, 2020

DRILL RIG: CME-850

DRILLING CONTRACTOR: CCC Drilling

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

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 %	RECOVERY SOLID CORE %	R.Q.D. %	FRACT. INDEX PER 0.25 m	DIP w.r.t. CORE AXIS	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	Jcon	Jr	Ja	HYDRAULIC CONDUCTIVITY K, cm/sec	Diametral Point Load Index (MPa)	RMC -Q' AVG.
		GROUND SURFACE		31.73														
36	Rotary Drill, HQ Core	Fresh, thinly to medium bedded, medium grey, fine grained, non-porous, very strong DOLOSTONE, with thin laminations to very thin beds of dark grey to black, non-porous, medium strong to weak shale and limestone		35.94	1													
37					2													
38		- mud seam from 38.37 to 38.40 m depth			3													
39		- slightly porous, cavities																
40					4													
41					5													
42																		
43		- slightly porous			6													
44				23.70 43.97														
		End of Borehole																
45																		

DEPTH SCALE

1 : 50

GOLDER

LOGGED: JS

CHECKED: AG

PROJECT: 19134931

RECORD OF BOREHOLE: 20-06

SHEET 1 OF 2

LOCATION: N 5021742.9 ;E 436500.8

BORING DATE: June 22, 2020

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE - DESCRIPTION	ELEV. / DEPTH (m)	SAMPLE NUMBER	TYPE	BLOWS/0.30m	PIEZOMETER OR STANDPIPE INSTALLATION
		GROUND SURFACE	66.28				
0	Power Auger 200 mm Diam. (Hollow Stem)	FILL - (SP) SAND, coarse, some silt and gravel; grey (STONE DUST); non-cohesive, dry, loose	0.00				Flush Mount Casing
		FILL - (SW) gravelly SAND; brown, mottled; non-cohesive, moist, compact	66.08 / 0.20	1	SS	41	
		FILL - (CL/ML) SILTY CLAY to CLAYEY SILT, some to trace fine sand; brown grey, mottled and fissured; cohesive, w<PL, very stiff to stiff	65.82 / 0.46	2	SS	27	Bentonite and Cuttings
2		(CL/ML) SILTY CLAY to CLAYEY SILT; brown grey, mottled, fissured (WEATHERED CRUST); cohesive, w<PL to w~PL, stiff	64.76 / 1.52	3	SS	20	Bentonite Seal
3				4	SS	10	Silica Sand
				5	SS	5	
4		(CL/ML) CLAYEY SILT to SILTY CLAY; grey; cohesive, w>PL, stiff	62.62 / 3.66	6	SS	2	32 mm Diam. PVC #10 Slot Screen 'B'
5				7	SS	2	
		End of Sampling	61.10 / 5.18				
6							Bentonite and Cuttings
							Bentonite Seal
7							Silica Sand
8							
9							32 mm Diam. PVC #10 Slot Screen 'A'
10							

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

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

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

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

ADDITIONAL LAB. TESTING

CONTINUED NEXT PAGE

DEPTH SCALE 1 : 50

GOLDER

LOGGED: JS

CHECKED: AG

MIS-BHS 001 19134931.GPJ GAL-MIS.GDT 3-19-21 JEM

LOCATION: N 5021742.9 ;E 436500.8

BORING DATE: June 22, 2020

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES: NUMBER	TYPE	BLOWS/0.30m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20 40 60 80); SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - ○ (20 40 60 80)	HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-6} 10^{-5} 10^{-4} 10^{-3}); WATER CONTENT PERCENT Wp —○W— Wl (20 40 60 80)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		--- CONTINUED FROM PREVIOUS PAGE ---									
10	Power Auger										32 mm Diam. PVC #10 Slot Screen 'A'
		End of Borehole		55.61 10.67							
11											WL in Screen 'B' at Elev. 63.209 m on August 10, 2020 WL in Screen 'A' at Elev. 61.419 m on August 10, 2020
12											
13											
14											
15											
16											
17											
18											
19											
20											

DEPTH SCALE
1 : 50

GOLDER

LOGGED: JS
CHECKED: AG

RECORD OF BOREHOLE: 20-07

PROJECT: 19134931

SHEET 1 OF 5

LOCATION: N 5021727.5 ;E 436514.9

BORING DATE: June 22, 2020

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES | BORING METHOD | SOIL PROFILE: DESCRIPTION, STRATA PLOT, ELEV. DEPTH (m) | SAMPLES: NUMBER, TYPE, BLOWS/0.30m | DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20, 40, 60, 80) | SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - ○ (20, 40, 60, 80) | HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-6}, 10^{-5}, 10^{-4}, 10^{-3}) | WATER CONTENT PERCENT Wp ├──○W──┤ Wl (20, 40, 60, 80) | ADDITIONAL LAB. TESTING | PIEZOMETER OR STANDPIPE INSTALLATION

Boring method: Power Auger, 200 mm Diam. (Hollow Stem) (0 to approx. 5.2 m); Wash Boring, HW Casing (below approx. 5.2 m)

DESCRIPTION	ELEV. (m)	DEPTH (m)
GROUND SURFACE	66.57	
TOPSOIL - (SM) SILTY SAND; dark brown, contains organics; non-cohesive, dry, loose		0.00
FILL - (SP) gravelly SAND, fine to coarse, angular gravel; grey, contains wood debris; non-cohesive, dry, compact		0.15
FILL - (CL/ML) SILTY CLAY to CLAYEY SILT, trace fine sand and organics; brown to grey, highly fissured; cohesive, w<PL, stiff	66.12	0.45
(ML/CL) CLAYEY SILT to SILTY CLAY, trace sand; grey, highly fissured (WEATHERED CRUST); cohesive, w<PL to w>PL, stiff to very stiff	65.05	1.52
(CL/ML) SILTY CLAY to CLAYEY SILT; grey, contains sandy silt layers; cohesive, w~PL to w>PL, very soft to soft	63.52	3.05
(ML/SM) CLAYEY SILT to SILTY SAND; grey, contains clayey seams; non-cohesive, wet, loose to very loose	60.47	6.10

NUMBER	TYPE	BLOWS/0.30m
1	SS	40
2	SS	46
3	SS	20
4	SS	9
5	SS	5
6	SS	2
7	SS	2
8	SS	14
9	SS	WH
10	SS	8

Shear strength: >96+

CONTINUED NEXT PAGE

DEPTH SCALE

1 : 50

GOLDER

LOGGED: AK

CHECKED: AG

MIS-BHS 001 19134931.GPJ GAL-MIS.GDT 3-19-21 JEM

RECORD OF BOREHOLE: 20-07

PROJECT: 19134931

LOCATION: N 5021727.5 ;E 436514.9

BORING DATE: June 22, 2020

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

Boring method: Wash Boring, HW Casing

DEPTH SCALE METRES	SOIL PROFILE – DESCRIPTION	ELEV. / DEPTH (m)	SAMPLE NUMBER	TYPE	BLOWS/0.30m	WATER CONTENT PERCENT (W)
	--- CONTINUED FROM PREVIOUS PAGE ---					
10	(ML/SM) CLAYEY SILT to SILTY SAND; grey, contains clayey seams; non-cohesive, wet, loose to very loose					
11			11	SS	WH	○
12	(SW) SAND, some plasticity fines and gravel; grey to brown grey, contains clay and silt seams; non-cohesive, wet, compact	54.38 / 12.19	12	SS	26	○
13.72	(SW) gravelly SAND, fine to coarse; brown grey to grey, contains clay/silt seams; non-cohesive, wet, very dense	52.85 / 13.72	13	SS	87	○
15			14	SS	58	○
17			15	SS	35	
18.29	(SM/ML) SAND to sandy SILT; brown grey to grey; non-cohesive, moist, very dense	48.28 / 18.29	16	SS	73	
19.81		46.76 / 19.81	17	SS	59	
20	CONTINUED NEXT PAGE					

Column headings: DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20, 40, 60, 80); SHEAR STRENGTH Cu, kPa (nat V. + Q - ●; rem V. ⊕ U - ○) (20, 40, 60, 80); HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-6}, 10^{-5}, 10^{-4}, 10^{-3}); WATER CONTENT PERCENT Wp —○W— Wl (20, 40, 60, 80); ADDITIONAL LAB. TESTING; PIEZOMETER OR STANDPIPE INSTALLATION

DEPTH SCALE
1 : 50

GOLDER

LOGGED: AK
CHECKED: AG

RECORD OF BOREHOLE: 20-07

PROJECT: 19134931

LOCATION: N 5021727.5 ;E 436514.9

BORING DATE: June 22, 2020

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

--- CONTINUED FROM PREVIOUS PAGE ---

DEPTH SCALE METRES	BORING METHOD	DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	NUMBER	TYPE	BLOWS/0.30m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m / SHEAR STRENGTH Cu, kPa	HYDRAULIC CONDUCTIVITY, k, cm/s / WATER CONTENT PERCENT	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
20	Wash Boring, HW Casing	(SM) SAND, some gravel, fine to coarse; brown grey, contains pockets of clay; non-cohesive, wet, dense to very dense			17	SS	59				
23		- cobbles and boulders			18	SS	45				
26					19	SS	41				
29		(SW/GW) SAND and GRAVEL, some non-plastic fines; grey, contains cobbles and boulders; non-cohesive, wet, very dense		37.61	20	SS	77				
30											

CONTINUED NEXT PAGE

DEPTH SCALE 1 : 50

GOLDER

LOGGED: AK

CHECKED: AG

LOCATION: N 5021727.5 ;E 436514.9

BORING DATE: June 22, 2020

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES: NUMBER	TYPE	BLOWS/0.30m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20 40 60 80); SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - ○ (20 40 60 80)	HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-6} 10^{-5} 10^{-4} 10^{-3}); WATER CONTENT PERCENT Wp ⊢ ○W ⊣ Wl (20 40 60 80)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		--- *CONTINUED FROM PREVIOUS PAGE* ---									
30	Wash Boring / HW Casing	(SW/GW) SAND and GRAVEL, some non-plastic fines; grey, contains cobbles and boulders; non-cohesive, wet, very dense									
32					21	SS	86				
		Borehole continued on RECORD OF DRILLHOLE 20-07		32.13 / 34.44							

Depth scale marks: 30, 31, 32, 33, 34, 35, 36, 37, 38, 39, 40

PROJECT: 19134931

RECORD OF DRILLHOLE: 20-07

SHEET 5 OF 5

LOCATION: N 5021727.5 ;E 436514.9

DRILLING DATE: June 22, 2020

DATUM: Geodetic

DRILL RIG: CME-850

INCLINATION: -90° AZIMUTH: ---

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	ELEV. / DEPTH (m)	RUN No.
		GROUND SURFACE		
	Rotary Drill, HQ Core	Fresh, thinly to medium bedded, medium grey, fine grained, non-porous, very strong DOLOSTONE, with thin laminations to very thin beds of dark grey to black, non-porous, medium strong to weak shale and limestone	32.13 / 34.44	1
35				2
36				
37				3
			28.73 / 37.84	
38		End of Borehole		
39				
40				
41				
42				
43				
44				

Column headings: COLOUR % RETURN; FLUSH; RECOVERY (TOTAL CORE %, 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; TYPE AND SURFACE DESCRIPTION; Jcon; Jr; Ja); HYDRAULIC CONDUCTIVITY K, cm/sec (10^{-6} 10^{-5} 10^{-4} 10^{-3}); Diametral Point Load Index (MPa) (2 4 6); RMC -Q' AVG.

RECORD OF BOREHOLE: 20-08

PROJECT: 19134931

LOCATION: N 5021719.9 ;E 436603.0

BORING DATE: June 19, 2020

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE – DESCRIPTION	ELEV. / DEPTH (m)	SAMPLE NUMBER	TYPE	BLOWS/0.30m	PIEZOMETER OR STANDPIPE INSTALLATION
		GROUND SURFACE	66.36				
0	Power Auger, 200 mm Diam. (Hollow Stem)	TOPSOIL - (SM) SILTY SAND, some gravel; brown, contains organics; non-cohesive, dry, compact	0.00				
		FILL - (SW/GW) SAND and GRAVEL, some non-plastic fines; grey, angular; non-cohesive, dry, compact to dense	66.16 / 0.20	1	SS	78	
1		(CL/ML) CLAYEY SILT to SILTY CLAY, trace fine sand; brown, mottling and fissured (WEATHERED CRUST); cohesive, w<PL to w~PL, hard	65.60 / 0.76	2	SS	17	Bentonite Seal
2				3	SS	16	
				4	SS	7	Silica Sand
3		(CL/ML) SILTY CLAY to CLAYEY SILT, trace fine sand; brown to grey brown; cohesive, w~PL to w>PL, firm	63.31 / 3.05	5	SS	3	
4		(ML) CLAYEY SILT to fine sandy SILT; grey; cohesive, w>PL, firm	62.55 / 3.81	6	SS	3	32 mm Diam. PVC #10 Slot Screen 'B'
		End of sampling	61.94 / 4.42				
5							
6							Silica Sand
							Bentonite Seal
7							
8							Bentonite and Cuttings
9							
10							

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

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

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

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

ADDITIONAL LAB. TESTING

CONTINUED NEXT PAGE

MIS-BHS 001 19134931.GPJ GAL-MIS.GDT 3-19-21 JEM

DEPTH SCALE 1 : 50

GOLDER

LOGGED: JS

CHECKED: AG

RECORD OF BOREHOLE: 20-08

LOCATION: N 5021719.9 ;E 436603.0

BORING DATE: June 19, 2020

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. / DEPTH (m)	SAMPLES: NUMBER	TYPE	BLOWS/0.30m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20 40 60 80); SHEAR STRENGTH Cu, kPa (nat V. + Q - ●; rem V. ⊕ U - ○) (20 40 60 80)	HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-6} 10^{-5} 10^{-4} 10^{-3}); WATER CONTENT PERCENT (Wp — W — Wl) (20 40 60 80)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		--- CONTINUED FROM PREVIOUS PAGE ---									
10		End of sampling									Bentonite and Cuttings
11											Bentonite Seal
12											Silica Sand
13											
14											32 mm Diam. PVC #10 Slot Screen 'A'
15		End of Borehole		51.12 15.24							WL in Screen 'B' at Elev. 62.963 m on August 10, 2020 WL in Screen 'A' at Elev. 63.355 m on August 10, 2020
16											
17											
18											
19											
20											

DEPTH SCALE

1 : 50

GOLDER

LOGGED: JS

CHECKED: AG

APPENDIX B

Record of Borehole Sheets – Previous Investigation

PROJECT: 971-2086

LOCATION: See Plan

SAMPLER HAMMER, 63.6kg; DROP, 760mm

RECORD OF BOREHOLE 97-11

BORING DATE: Aug.7-14, 1997

DATUM: Geodetic

PENETRATION TEST HAMMER, 63.6kg; DROP, 760mm

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

DEPTH SCALE METRES	SOIL PROFILE DESCRIPTION	ELEV. / DEPTH (m)	SAMPLE NUMBER	TYPE	BLOWS/0.3m
0	Ground Surface	67.07			
	Dark brown silty TOPSOIL	0.00			
	Brown silty clay, trace gravel and roots (FILL)	0.15			
1	Dark brown silty clay TOPSOIL	66.13 / 0.94			
	Very stiff to stiff grey brown SILTY CLAY, occasional sand seam (Weathered Crust)	1.07	1	50 DO	14
2			2	50 DO	8
3			3	50 DO	4
4	Stiff grey SILTY CLAY	62.80 / 4.27			
5			4	50 DO	WH
6	Compact grey SILTY fine SAND	61.13 / 5.94			
	Stiff grey SILTY CLAY with some silty sand seams and layers	60.76 / 6.31	5	50 DO	5
7					
8			6	50 DO	WH
9	Very loose grey SILTY fine SAND, with occasional 0.15-0.60m silty clay seam	58.23 / 8.84	7	50 DO	2
10					
11	Compact grey SILTY fine to coarse SAND, trace to some gravel	56.10 / 10.97	8	50 DO	WR
12	Compact grey SILTY fine to coarse SAND, some gravel, occasional cobble	55.49 / 11.58	9	50 DO	17
13	Compact grey SANDY SILT, some clay	53.81 / 13.26			
14			10	50 DO	12
15	Compact grey fine to coarse SAND, trace to some gravel	52.29 / 14.78	11	50 DO	17
16					
17			12	50 DO	22
18			13	50 DO	25
19					
	Dense grey SILTY fine SAND	47.56 / 19.51	14	50 DO	33
20					

CONTINUED ON NEXT PAGE

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m

SHEAR STRENGTH Cu, kPa: nat.V - + Q - ● rem.V - ⊕ U - O

20 40 60 80

HYDRAULIC CONDUCTIVITY, k, cm/s

WATER CONTENT, PERCENT Wp W Wl

ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION

96 96

DATA INPUT: O:9711-086.drf/S.L

DEPTH SCALE 1 to 100

Golder Associates

LOGGED: D.J.S

CHECKED:

PROJECT: 971-2086

RECORD OF BOREHOLE 97-11

SHEET 2 OF 2

LOCATION: See Plan
BORING DATE: Aug.7-14, 1997
DATUM: Geodetic

SAMPLER HAMMER, 63.6kg; DROP, 760mm
PENETRATION TEST HAMMER, 63.6kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. / DEPTH (m)	SAMPLES: NUMBER	TYPE	BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m; SHEAR STRENGTH Cu, kPa (nat.V - + Q - ● rem.V - ⊕ U - O) 20 40 60 80	HYDRAULIC CONDUCTIVITY, k, cm/s; WATER CONTENT, PERCENT Wp —O W— Wl	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		CONTINUED FROM PREVIOUS PAGE									
20	Power Auger, 200mm Diam (Hollow Stem)	Dense grey SILTY fine SAND		46.34 / 20.73	14	50 DO	33				
21–22	Power Auger, 200mm Diam (Hollow Stem)	Compact grey fine to coarse SAND, trace gravel		44.51 / 22.56	15	50 DO	26				
23–28	Power Auger, 200mm Diam (Hollow Stem) / Rotary Drill, NW Casing	Dense to very dense grey fine to coarse SAND, trace gravel		38.72 / 28.35	16	50 DO	35				
					17	50 DO	89				
					18	50 DO	35				
					19	50 DO	31				
29	Rotary Drill, NW Casing	Dense grey fine SAND, some silt		37.20 / 29.87	20	50 DO	37				
30–34	Rotary Drill, NW Casing	Compact to dense grey fine to coarse SAND and GRAVEL, some cobbles with depth		32.93 / 34.14	21	50 DO	27				
					22	50 DO	46				
					23	NQ RC	--				
					24	NQ RC	--				
34–35	Rotary Drill, NW Casing	Dense grey fine to coarse SAND and GRAVEL, some cobbles		32.17 / 34.90	25	NQ RC	--				
35–38	Rotary Drill, NQ Core	Fairly sound grey LIMESTONE BEDROCK, occasional near vertical fracture and thin fractured zone		29.27 / 37.80	26	NQ RC	--	T.C.R. 100% S.C.R. 75% R.Q.D. 71%			
					27	NQ RC	--	T.C.R. 100% S.C.R. 88% R.Q.D. 80%			
38		End of Hole									
39											
40											

DATA INPUT: O:9711-086.drf/S.L

DEPTH SCALE
1 to 100

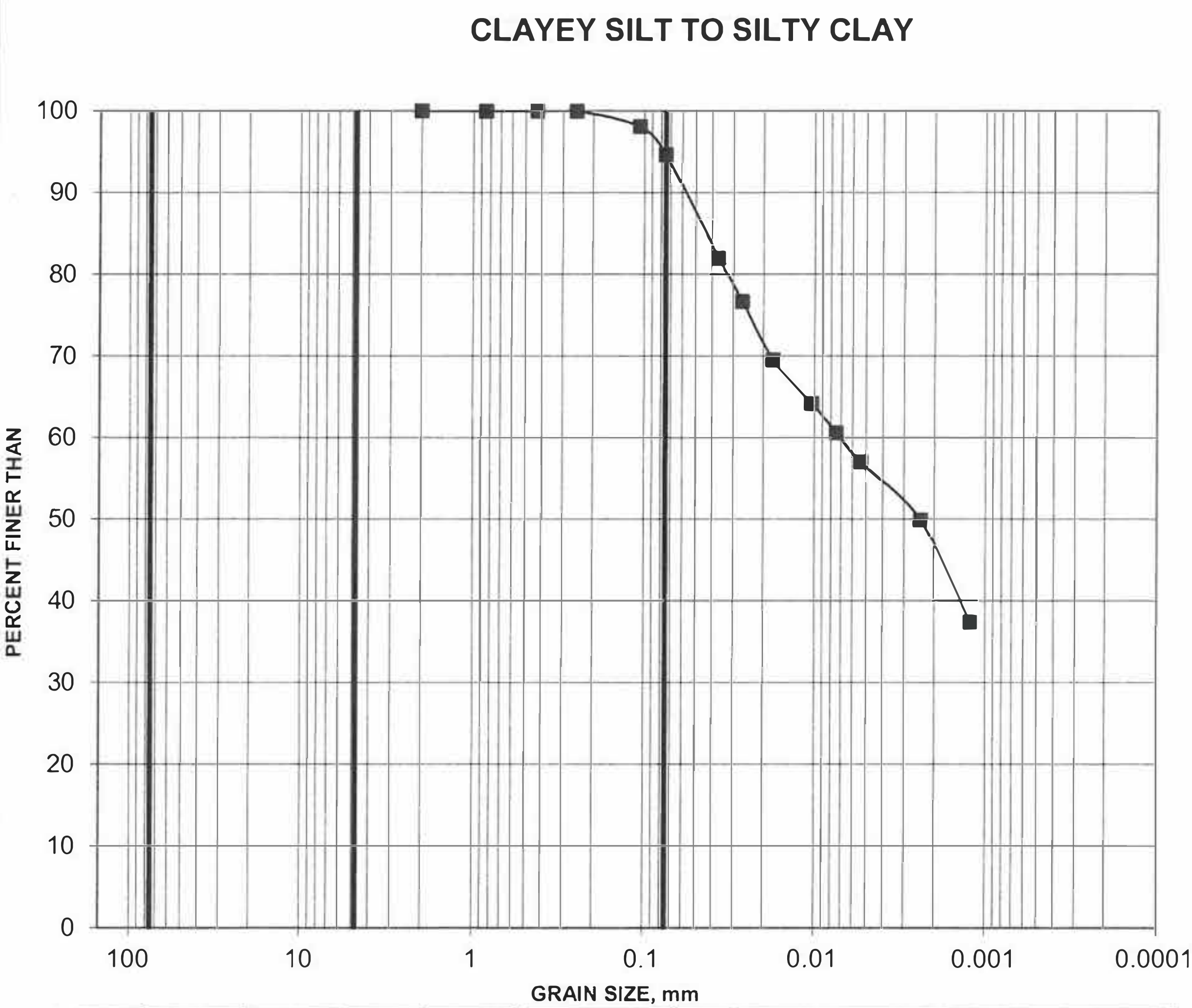
Golder Associates

LOGGED: D.J.S
CHECKED:

APPENDIX C

Laboratory Test Results – Current Investigation

GRAIN SIZE DISTRIBUTION	C-1



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

			Constituents (%)			
Borehole	Sample	Depth (m)	Gravel	Sand	Silt	Clay
20-01	6	3.81-4.42	0	5	48	47

GOLDER

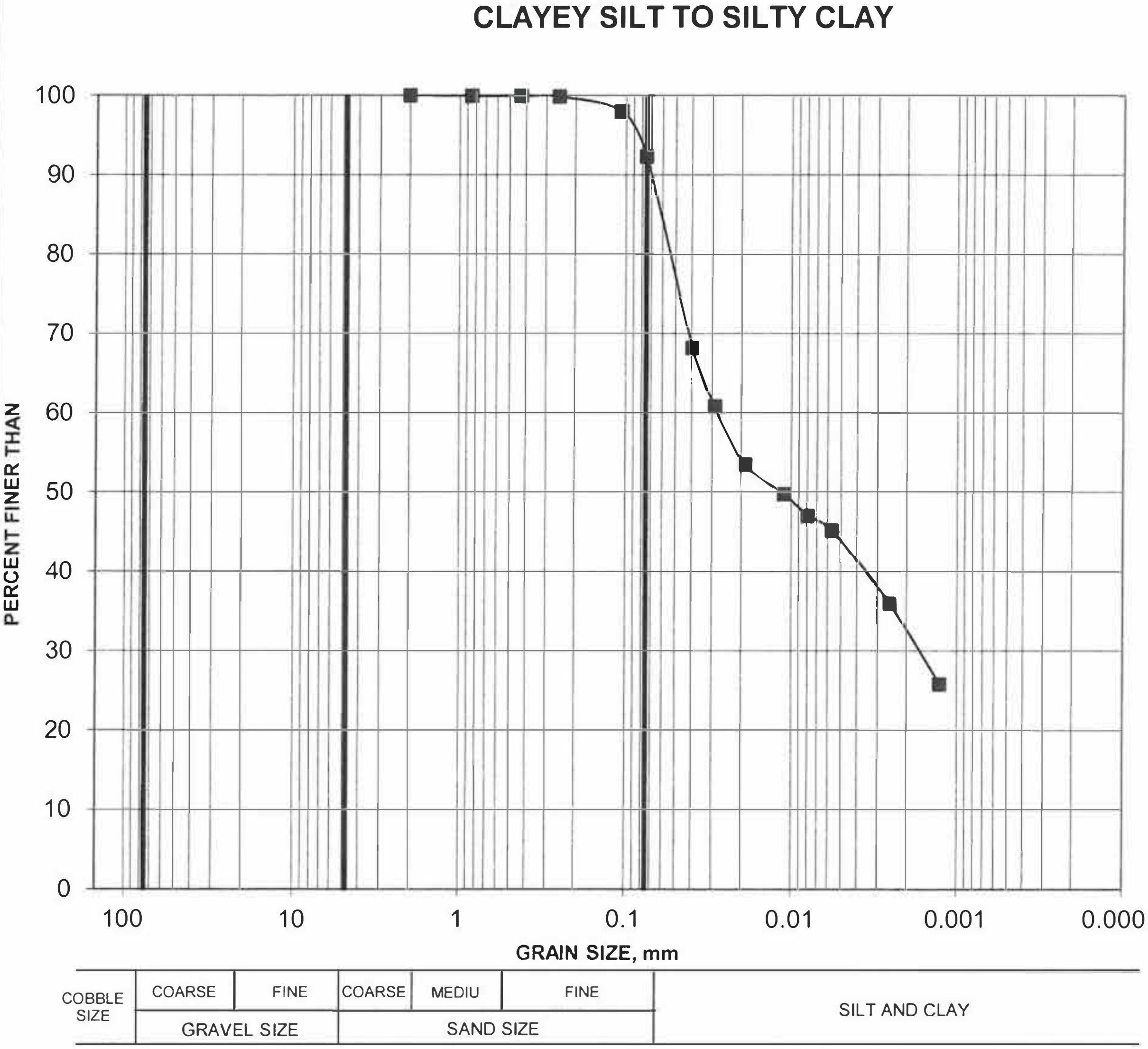
Project: 19134931/3000

Created by: LW

Checked by: [illegible]

GRAIN SIZE DISTRIBUTION

C-2



			Constituents (%)			
Borehole	Sample	Depth (m)	Gravel	Sand	Silt	Clay
20-02	7	4.57-5.18	0	8	59	33

GOLDER

Project: 19134931/3000

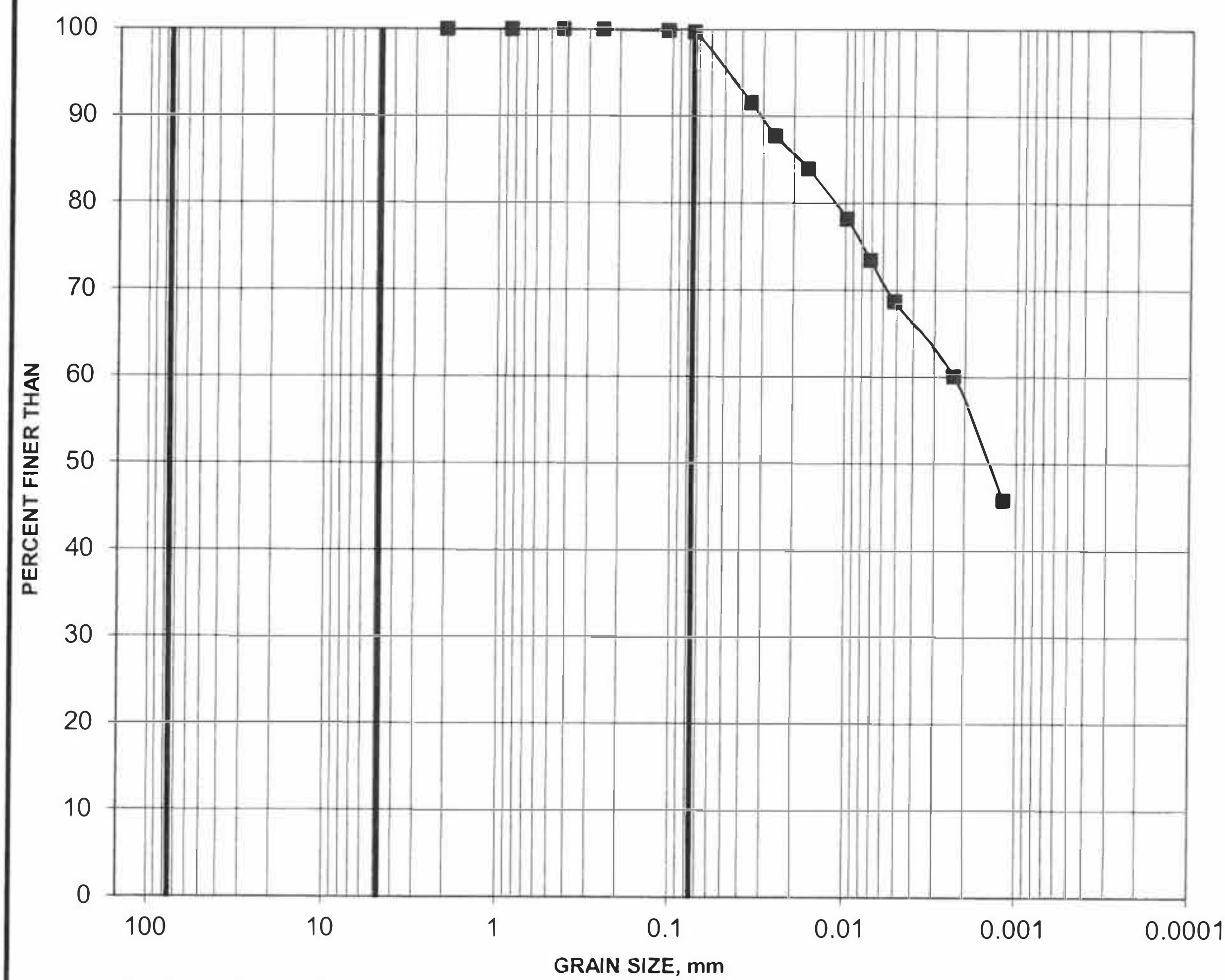
Created by: [signature]

Checked by: [signature]

GRAIN SIZE DISTRIBUTION

C-3

CLAYEY SILT TO SILTY CLAY



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

			Constituents (%)			
Borehole	Sample	Depth (m)	Gravel	Sand	Silt	Clay
20-03	5	3.05-3.66	0	0	43	57

GOLDER

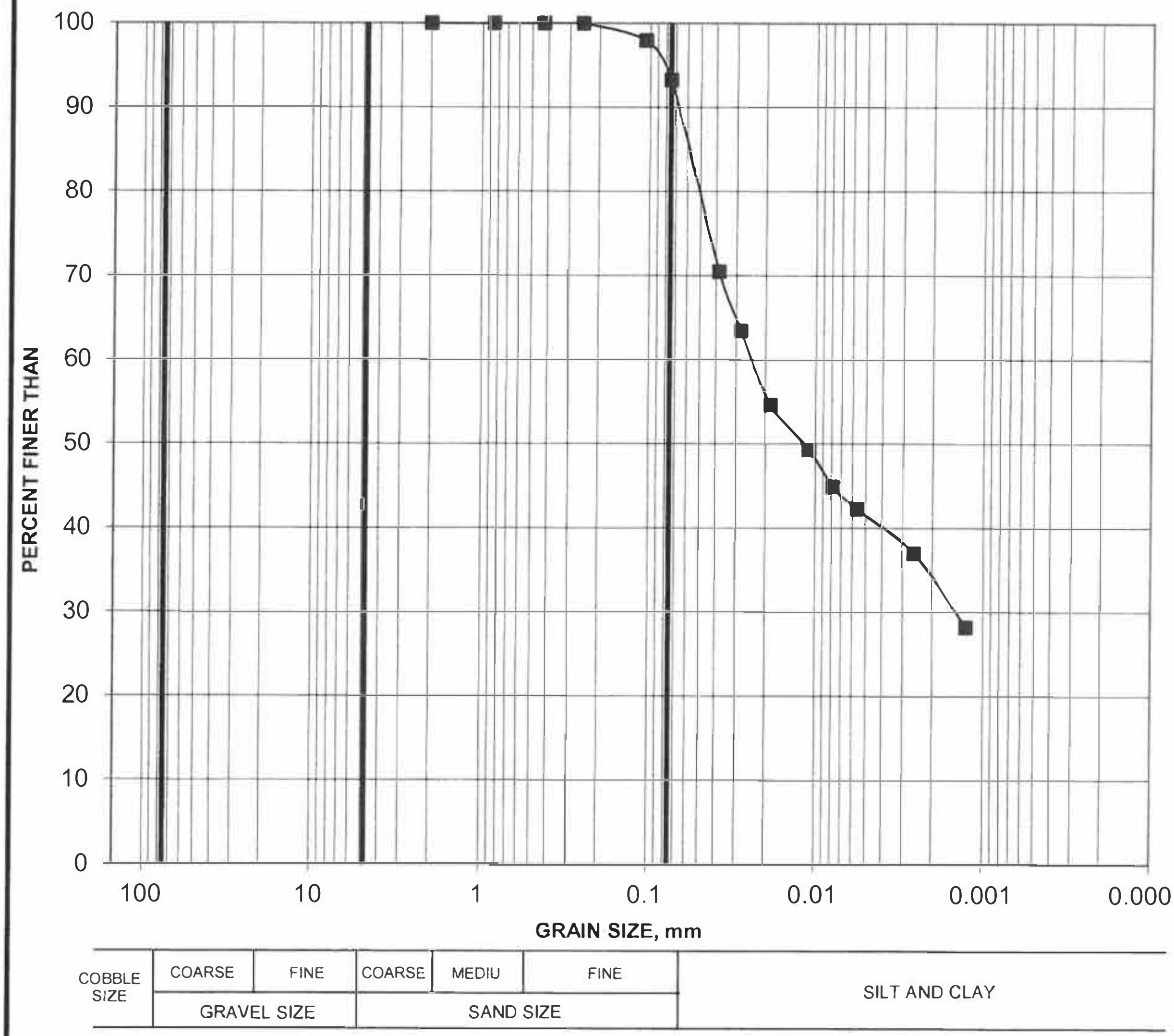
Project: 19134931/3000

Created by: CW

Checked by: [signature]

GRAIN SIZE DISTRIBUTION	C-4

CLAYEY SILT TO SILTY CLAY



			Constituents (%)			
Borehole	Sample	Depth (m)	Gravel	Sand	Silt	Clay
20-04	7	4.57-5.18	0	7	59	34

GOLDER

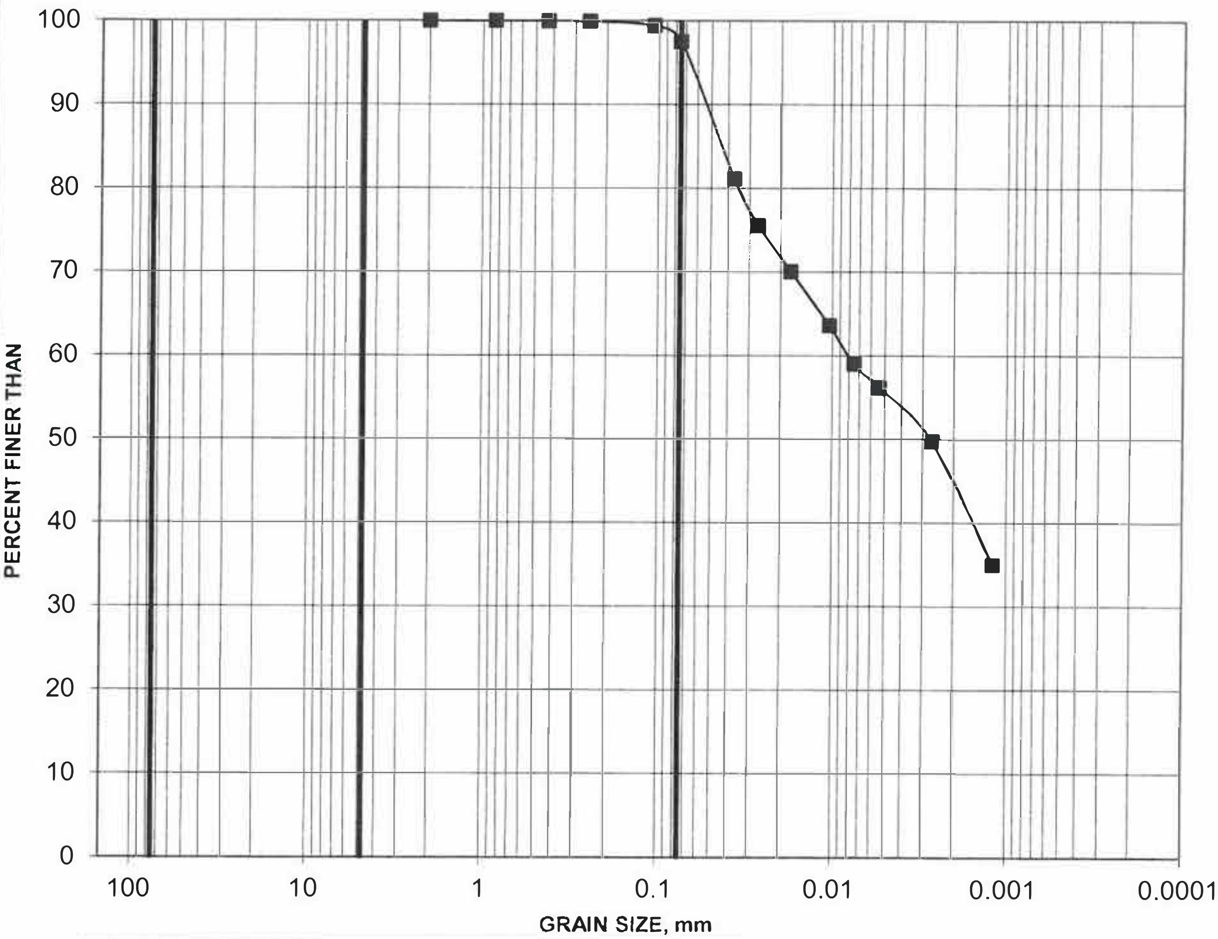
Project: 19134931/3000

Created by: CW

Checked by: VCA

GRAIN SIZE DISTRIBUTION	C-5

CLAYEY SILT TO SILTY CLAY



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

	Borehole	Sample	Depth (m)	Constituents (%) Gravel	Sand	Silt	Clay
—■—	20-07	6	3.78-4.42	0	3	52	45

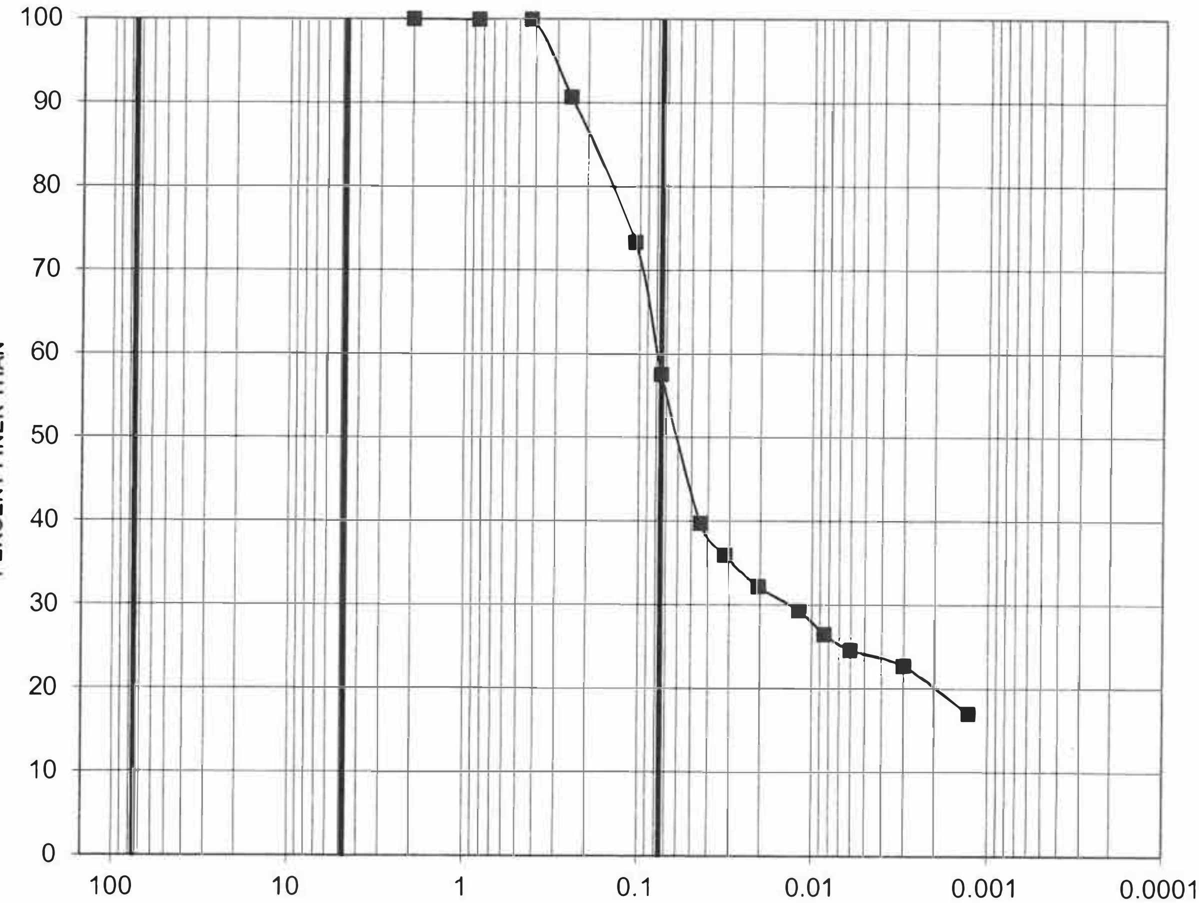
Project: 19134931/3000

GOLDER

Created by:
Checked by:

GRAIN SIZE DISTRIBUTION	C-6

SANDY SILT



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

			Constituents (%)			
Borehole	Sample	Depth (m)	Gravel	Sand	Silt	Clay
20-03	8	6.10-6.71	0	42	38	20

Project: 19134931/3000

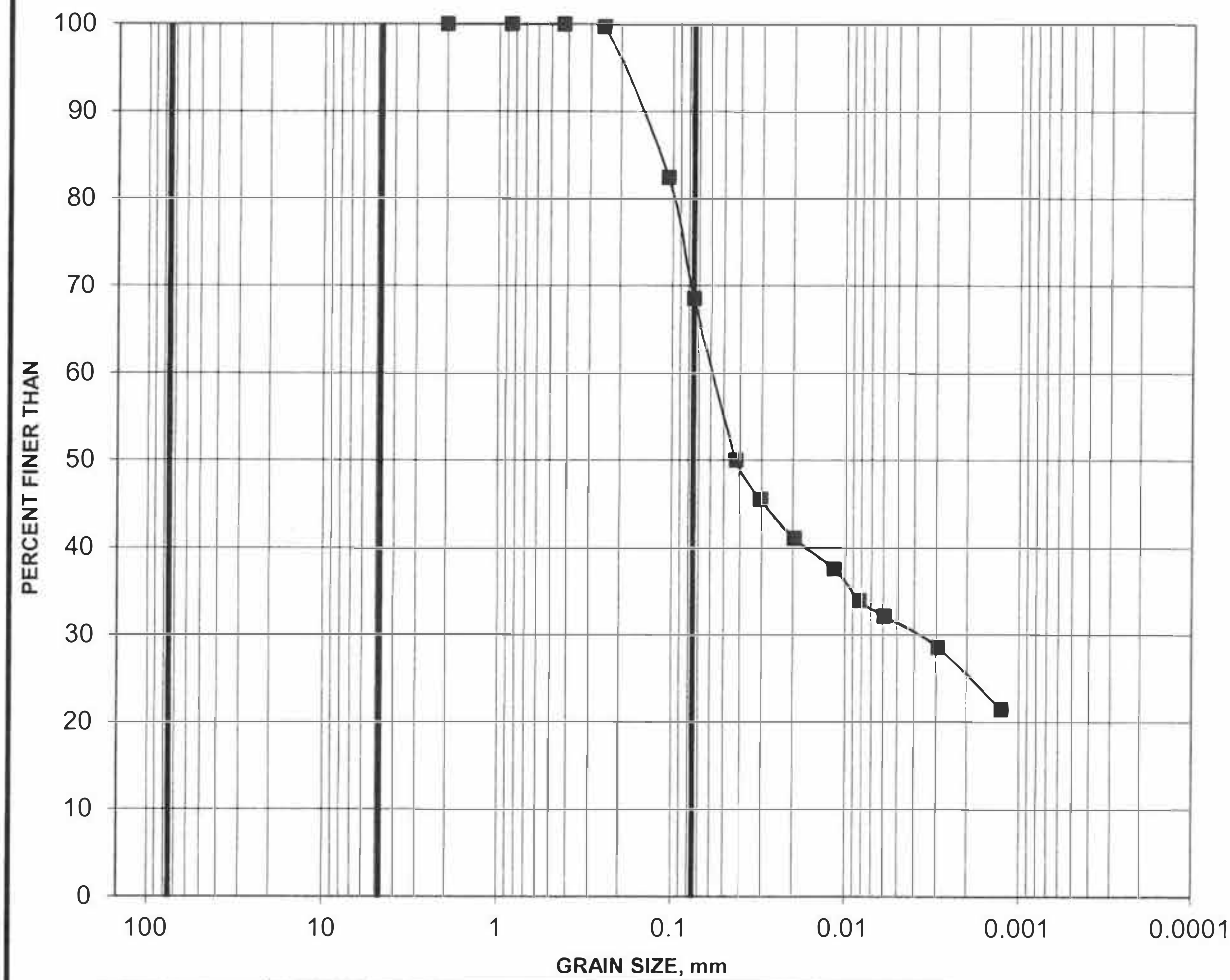
GOLDER

Created by: CW

Checked by: WA

GRAIN SIZE DISTRIBUTION	C-7

SANDY SILT



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

	Borehole	Sample	Depth (m)	Constituents (%) Gravel	Sand	Silt	Clay
—■—	20-05	11	7.62-8.23	0	31	44	25

GOLDER

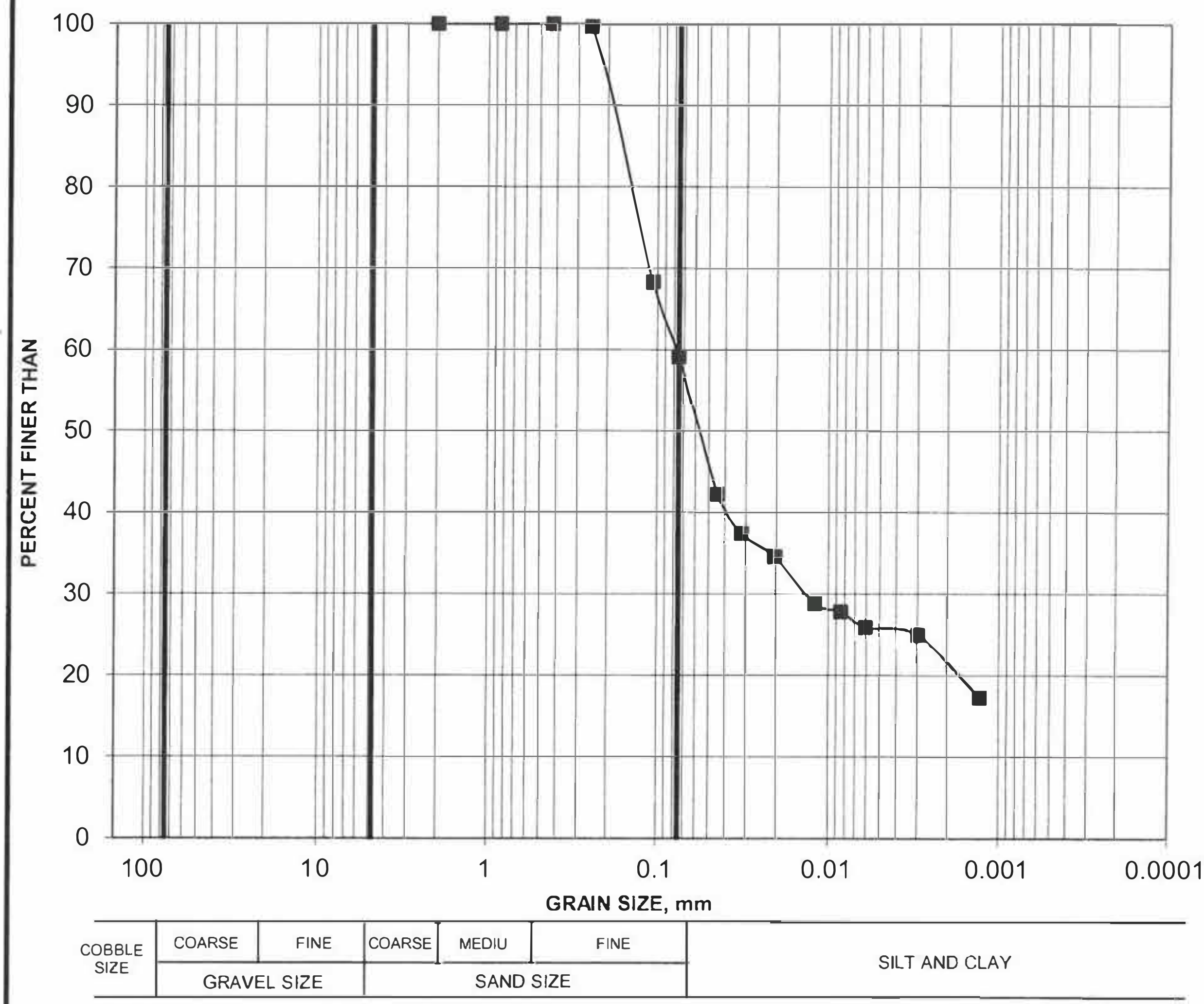
Project: 19134931/3000

Created by: CW

Checked by: [signature]

GRAIN SIZE DISTRIBUTION	C-8

SANDY SILT



			Constituents (%)			
Borehole	Sample	Depth (m)	Gravel	Sand	Silt	Clay
20-07	9	7.62-8.23	0	41	38	21

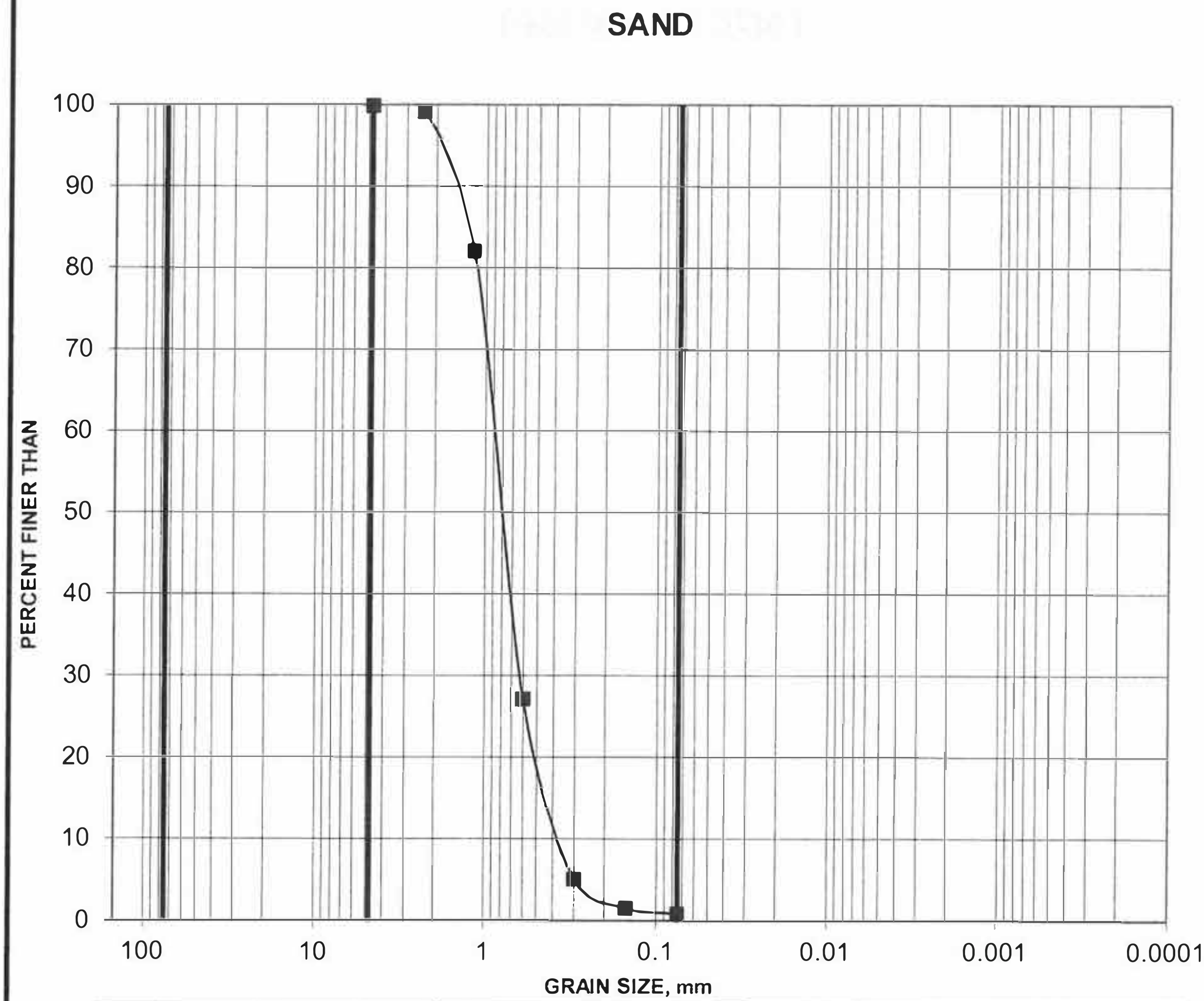
GOLDER

Project: 19134931/3000

Created by: [signature]

Checked by: [signature]

GRAIN SIZE DISTRIBUTION	C-9



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

			Constituents (%)			
Borehole	Sample	Depth (m)	Gravel	Sand	Silt	Clay
20-01	12	16.76-17.37	0	99	1	

GOLDER

Project: 19134931/3000

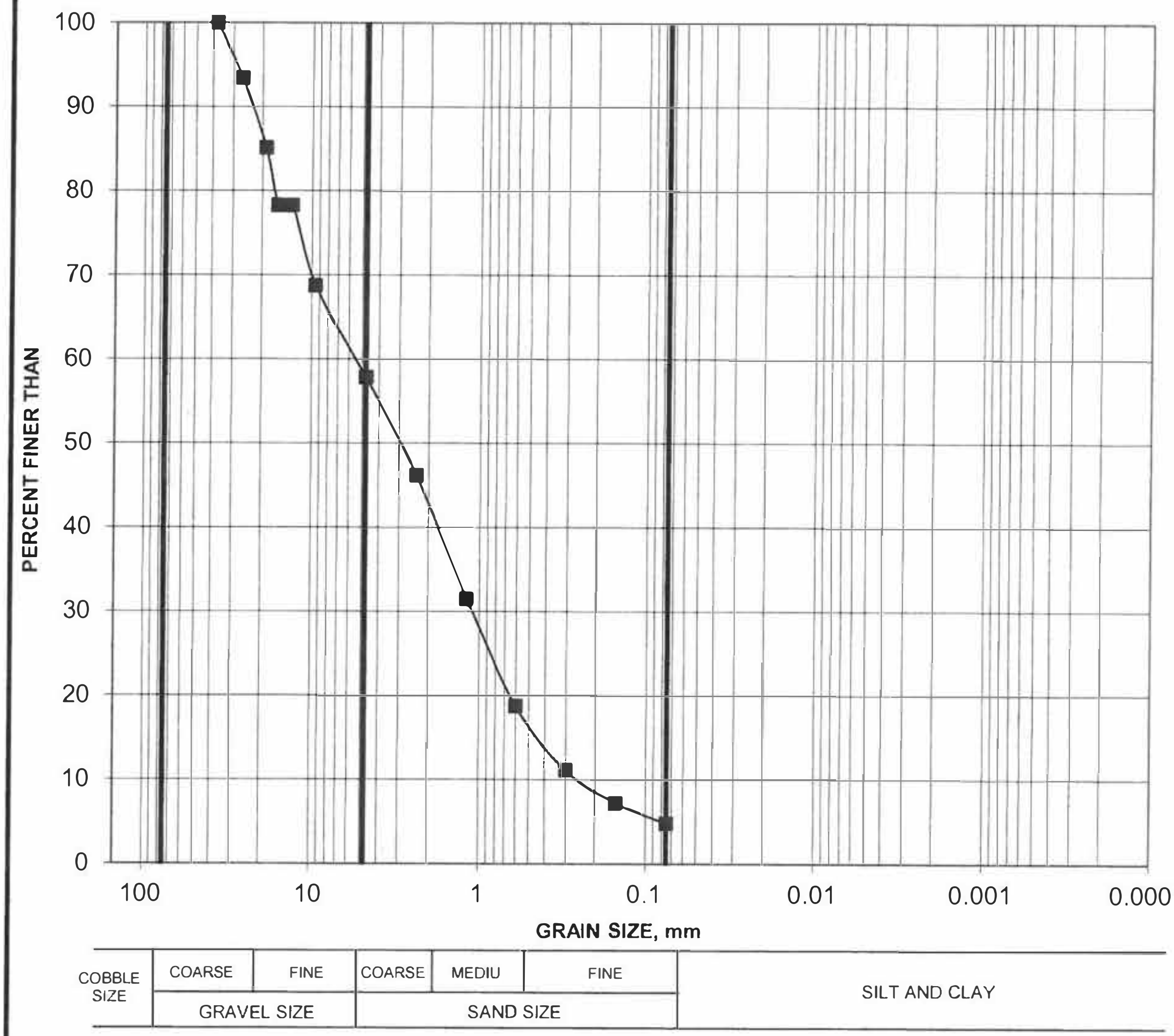
Created by:

Checked by:

GRAIN SIZE DISTRIBUTION

C-10

GRAVELLY SAND



	Borehole	Sample	Depth (m)	Constituents (%) Gravel	Sand	Silt	Clay
■	20-02	11	15.24-15.85	42	53	5	

GOLDER

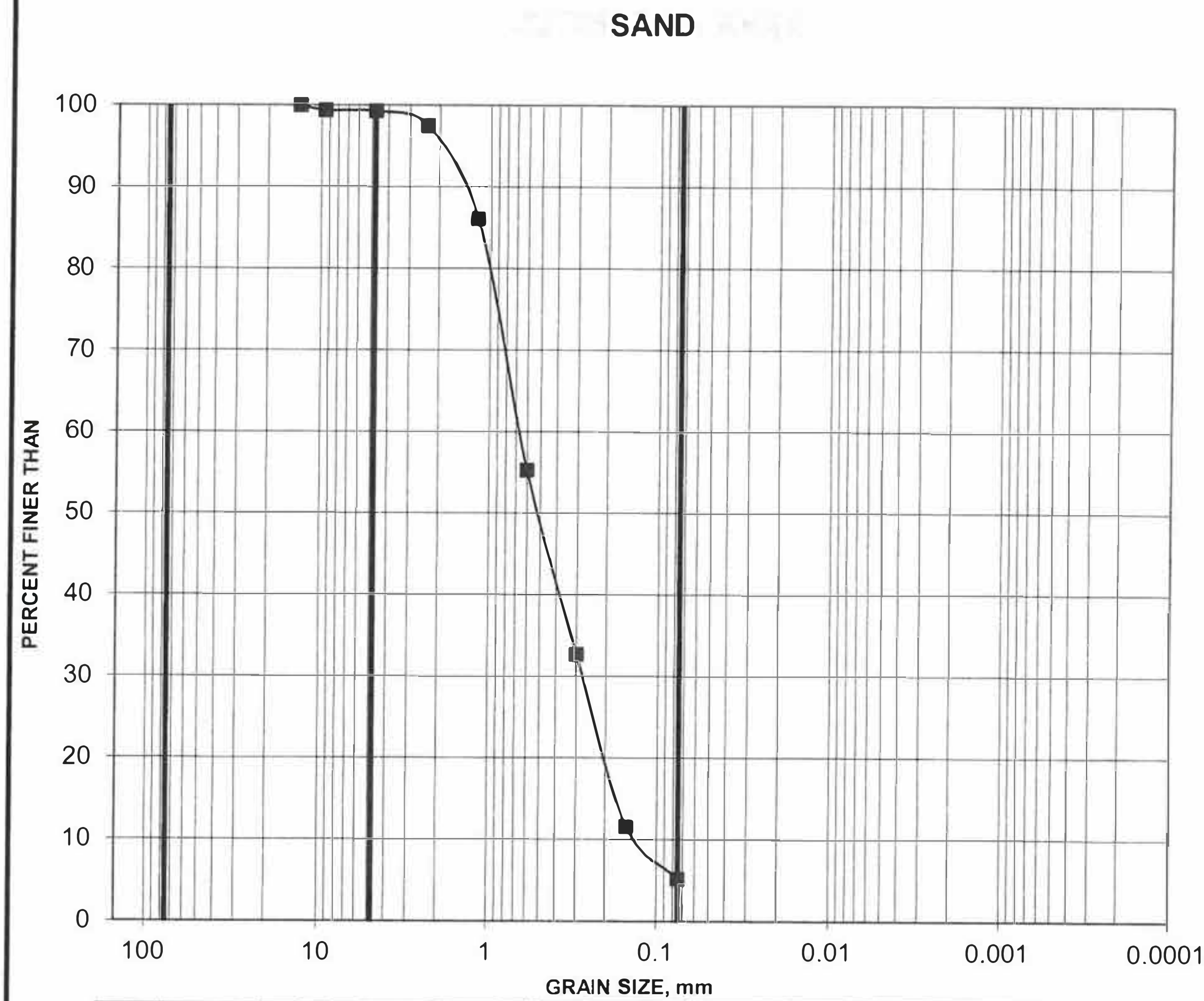
Project: 19134931/3000

Created by: CW

Checked by: MI

GRAIN SIZE DISTRIBUTION

C-11



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

Borehole	Sample	Depth (m)	Constituents (%) Gravel	Sand	Silt	Clay
20-03	12	18.29-18.90	1	94	5	

GOLDER

Project: 19134931/3000

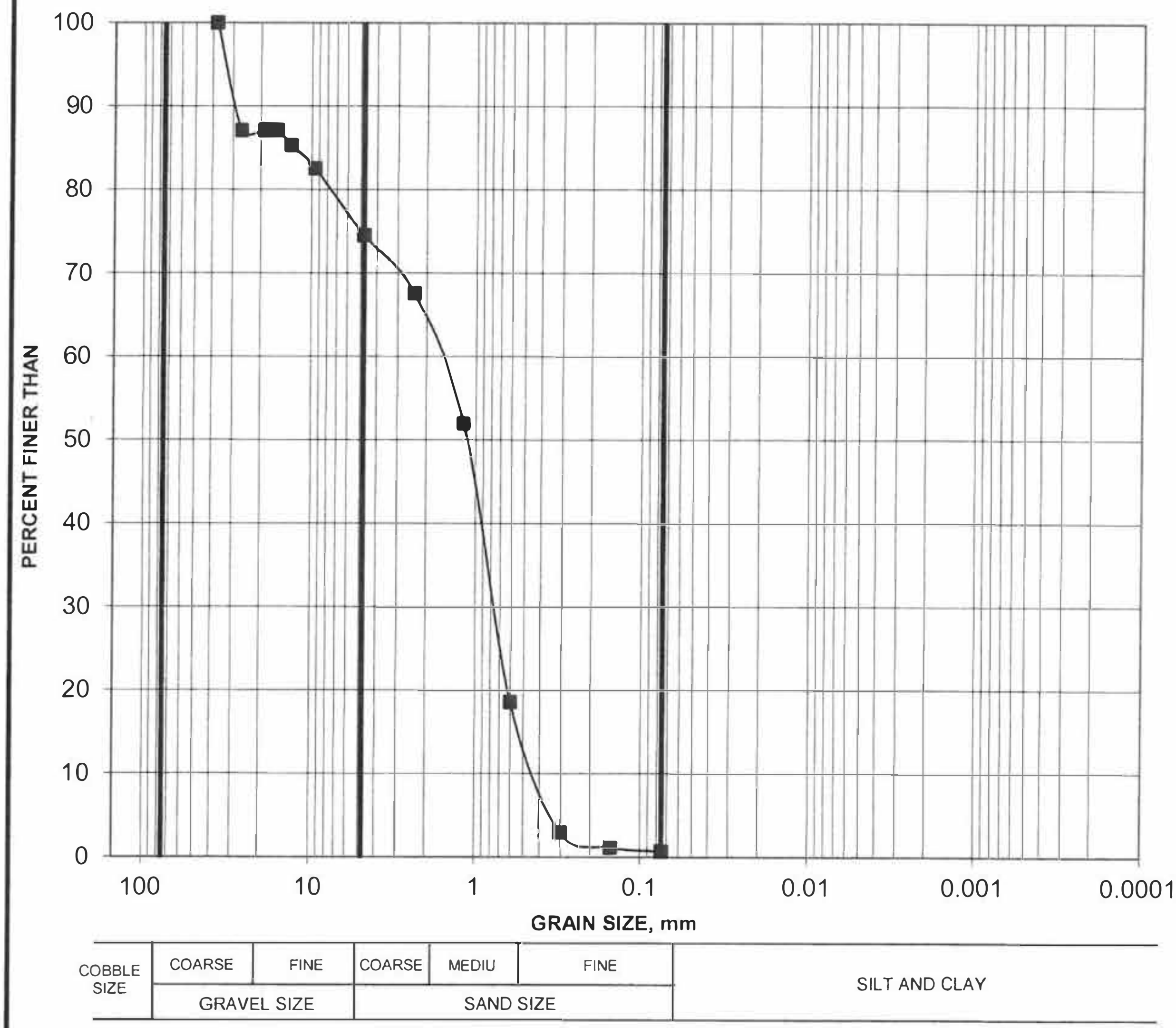
Created by:

Checked by:

GRAIN SIZE DISTRIBUTION

C-12

GRAVELLY SAND



			Constituents (%)			
Borehole	Sample	Depth (m)	Gravel	Sand	Silt	Clay
20-04	12	19.81-20.42	25	74	1	

GOLDER

Project: 19134931/3000

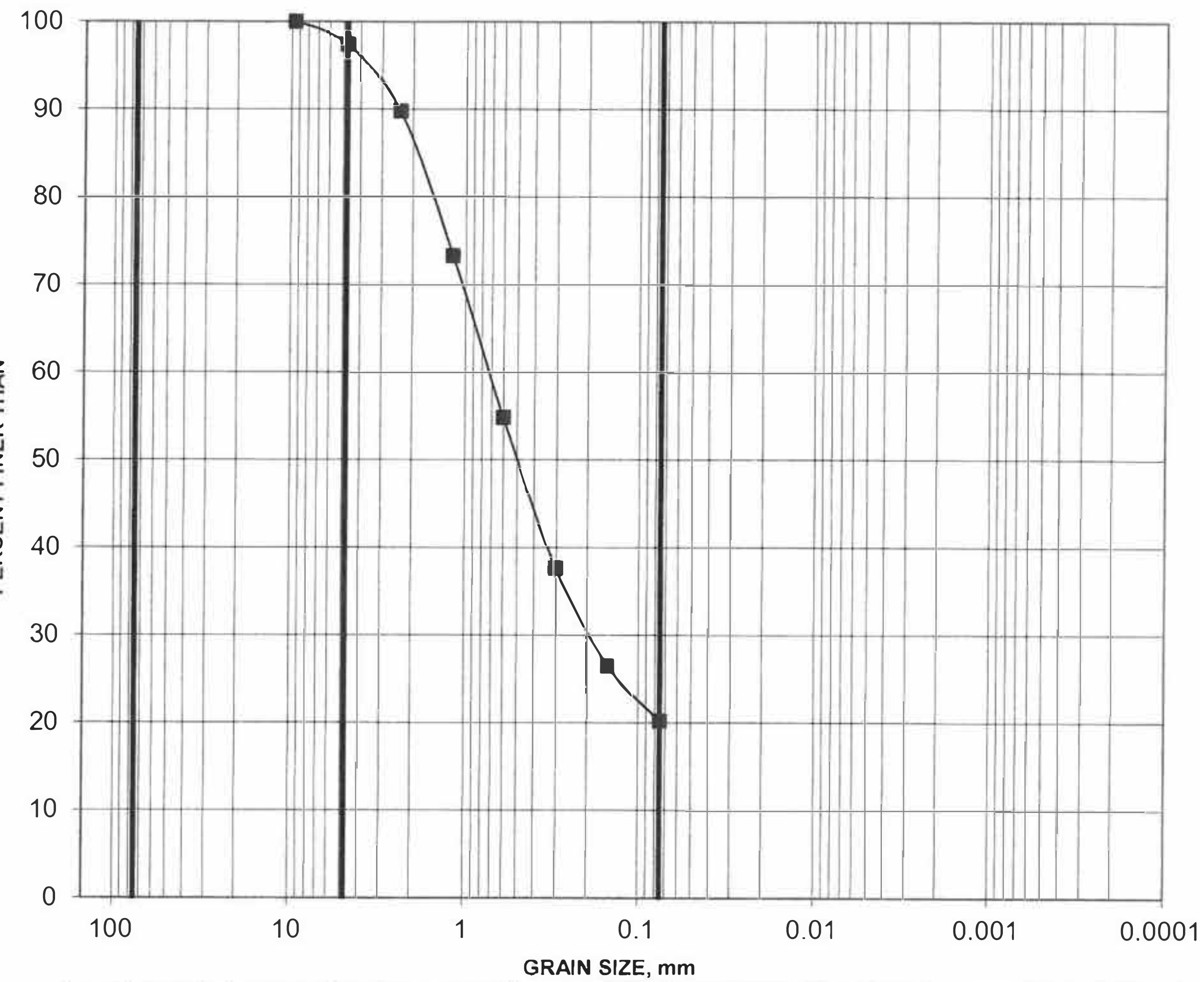
Created by: CW

Checked by: [illegible]

GRAIN SIZE DISTRIBUTION

C-13

SAND



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

Borehole	Sample	Depth (m)	Gravel	Sand	Silt	Clay
			Constituents (%)			
20-05	16	12.19-12.81	3	77	20	

GOLDER

Project: 19134931/3000

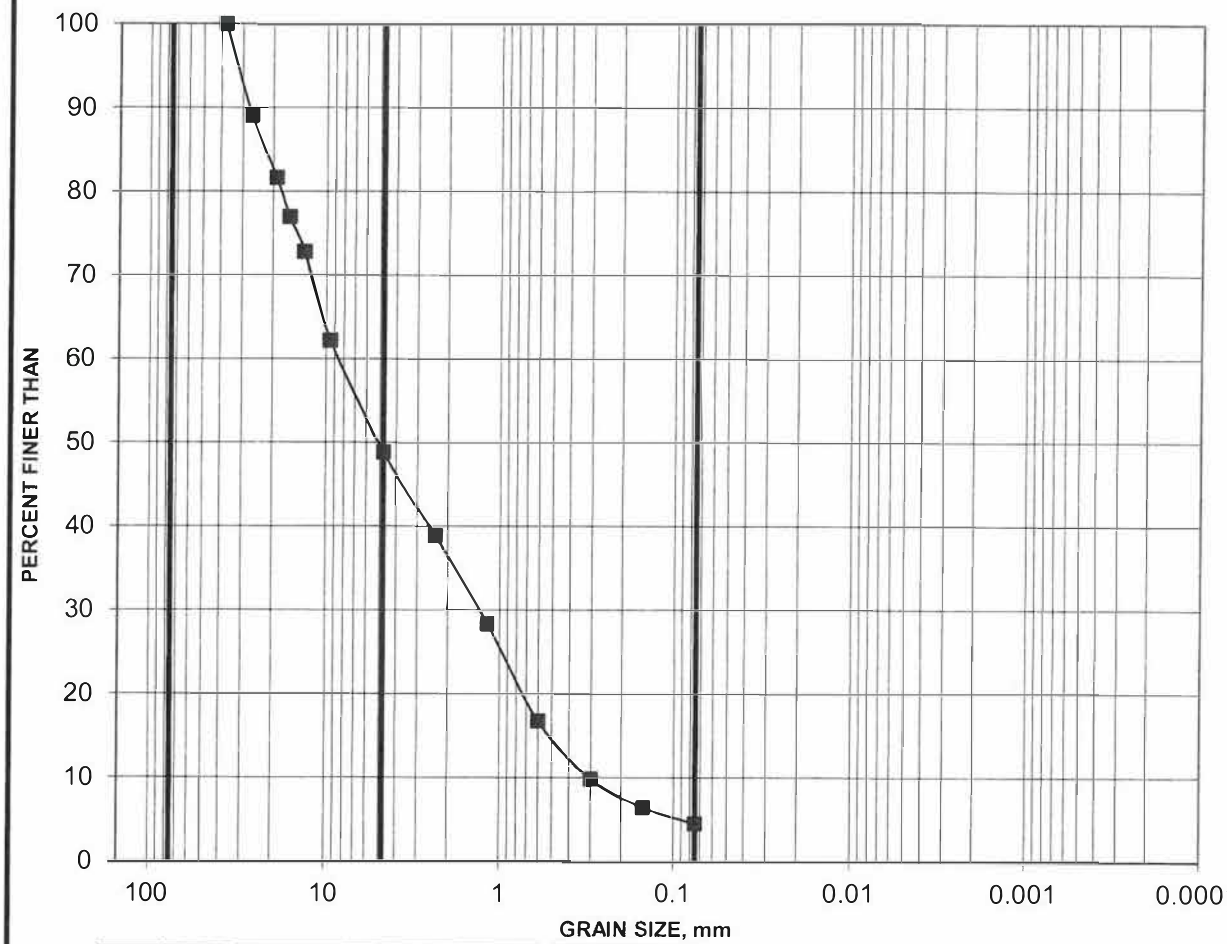
Created by: CW

Checked by: [signature]

GRAIN SIZE DISTRIBUTION

C-14

SANDY GRAVEL



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

	Borehole	Sample	Depth (m)	Constituents (%) Gravel	Sand	Silt	Clay
—■—	20-05	20	15.24-15.85	51	44	5	

GOLDER

Project: 19134931/3000

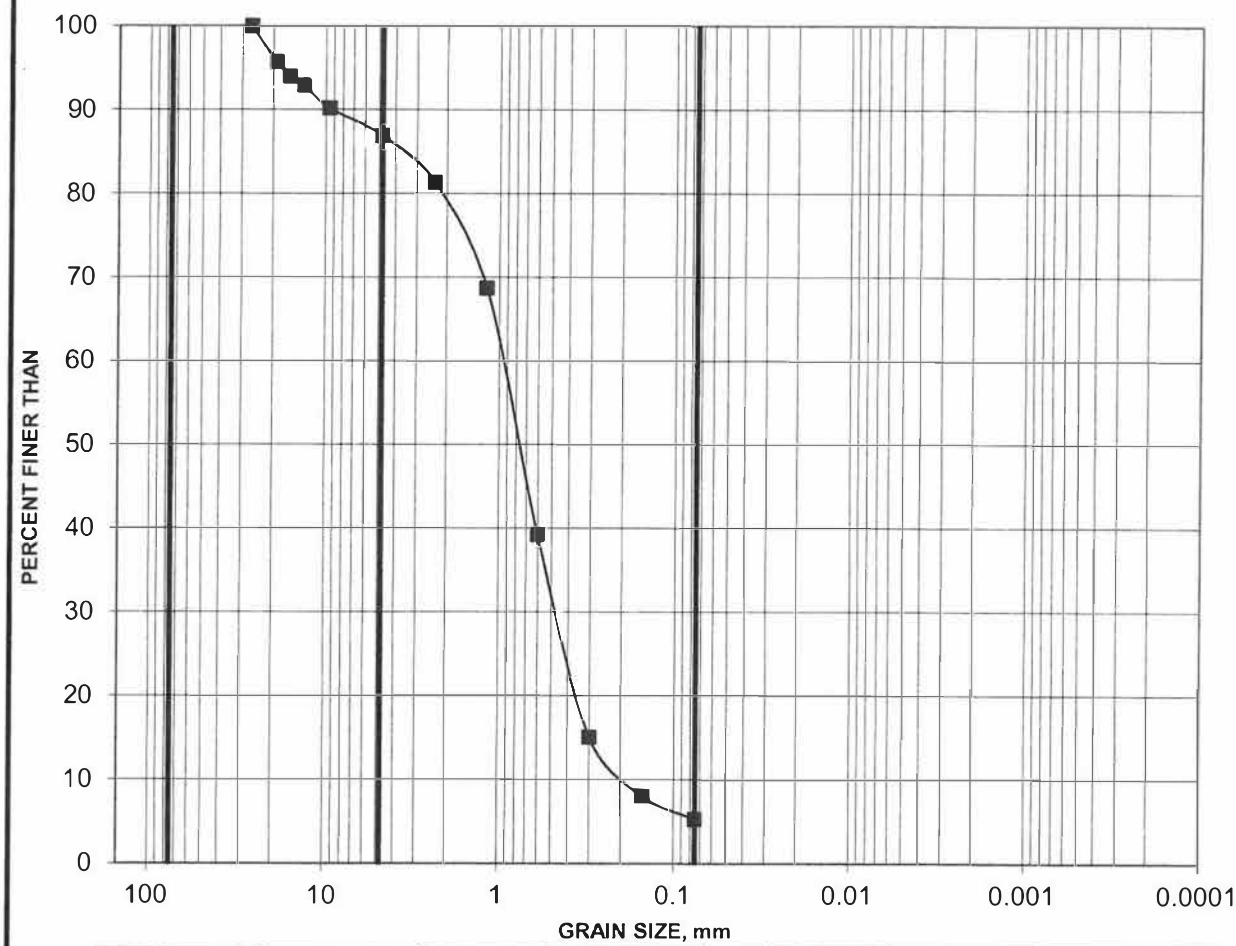
Created by: CW

Checked by: MJ

GRAIN SIZE DISTRIBUTION

C-15

GRAVELLY SAND



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

			Constituents (%)			
Borehole	Sample	Depth (m)	Gravel	Sand	Silt	Clay
20-05	23	17.53-18.14	13	82	5	

GOLDER

Project: 19134931/3000

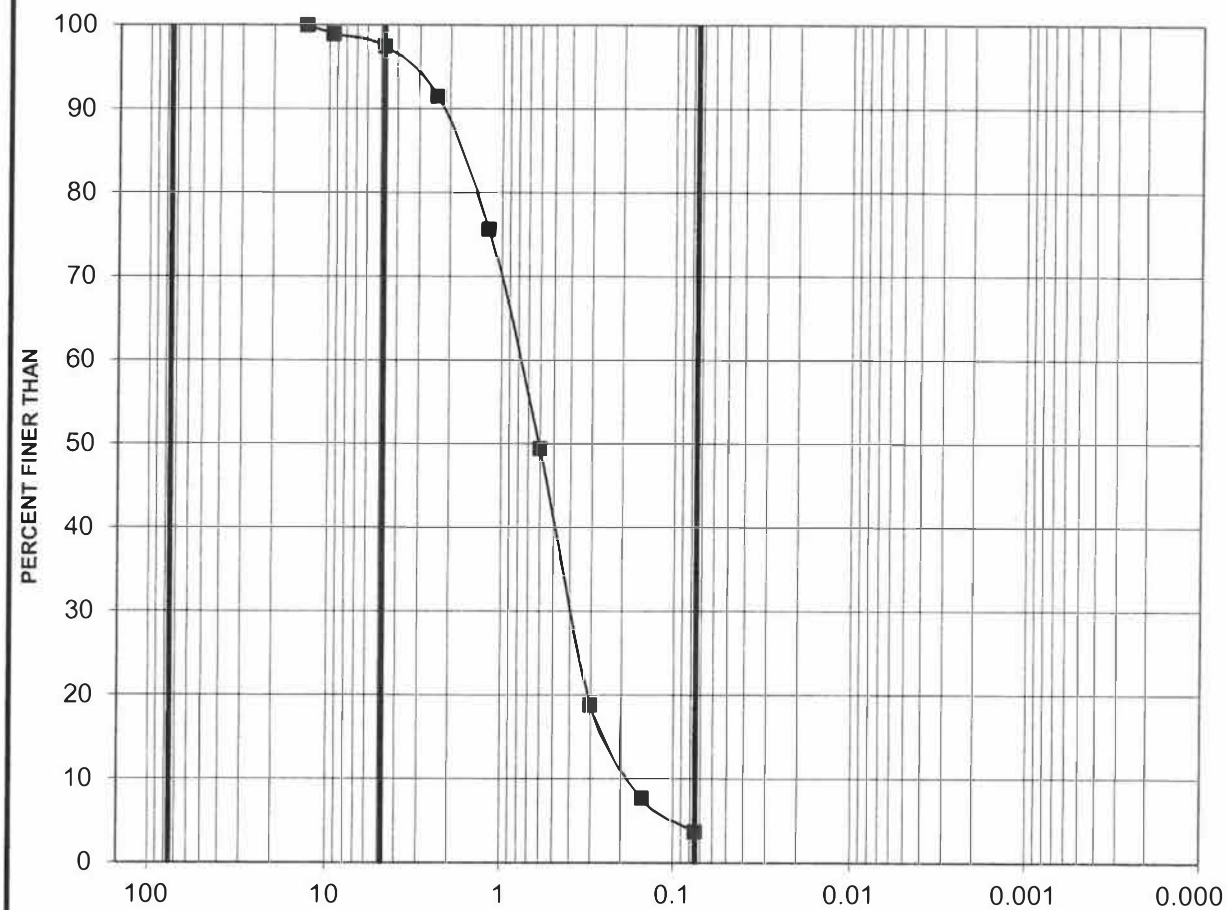
Created by: CW

Checked by: [illegible]

GRAIN SIZE DISTRIBUTION

C-16

SAND



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

	Borehole	Sample	Depth (m)	Constituents (%) Gravel	Sand	Silt	Clay
—■—	20-05	30	22.68-23.47	3	93	4	

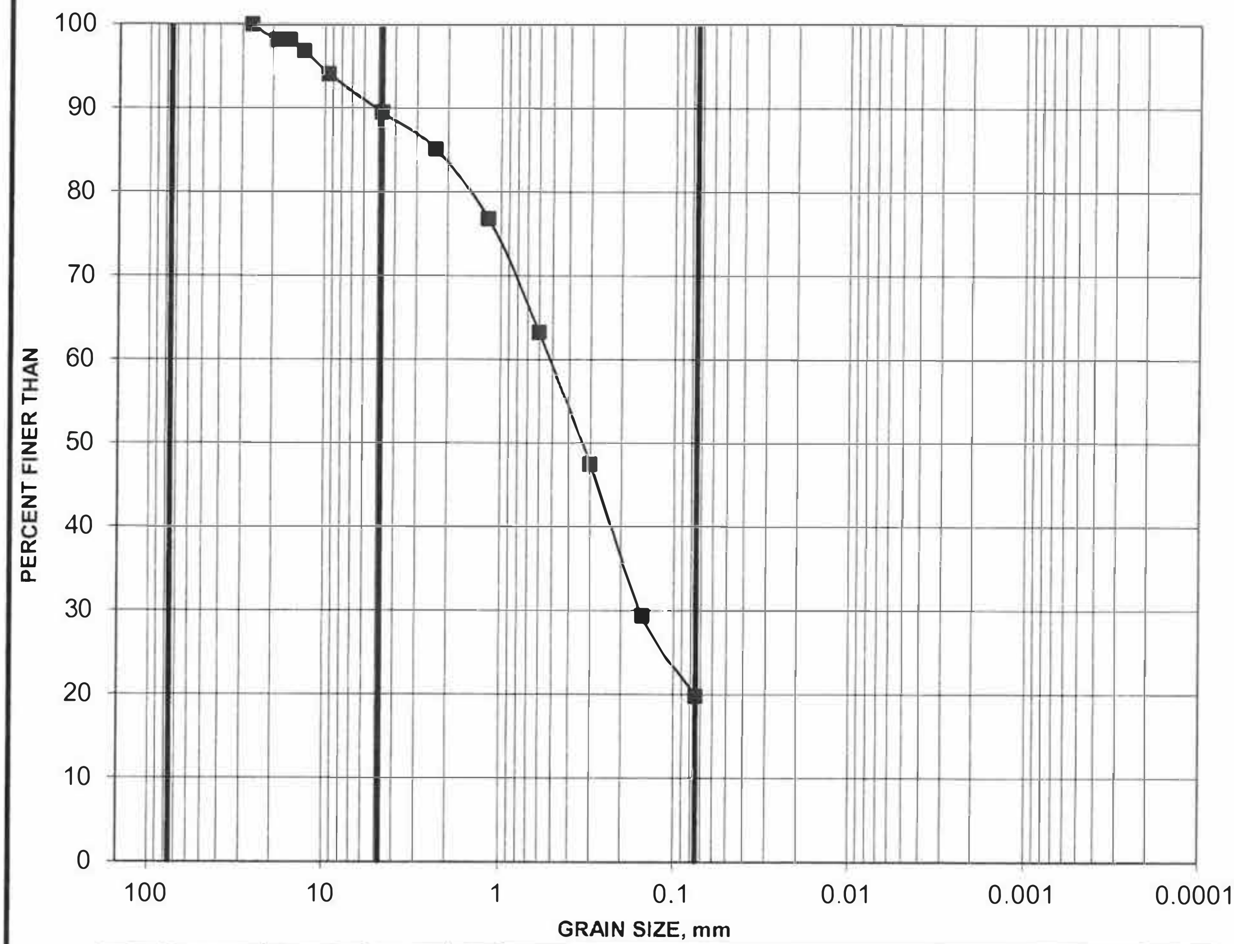
GOLDER

Project: 19134931/3000

Created by: [signature]
Checked by: [signature]

GRAIN SIZE DISTRIBUTION	C-17

SAND



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

				Constituents (%)			
	Borehole	Sample	Depth (m)	Gravel	Sand	Silt	Clay
—■—	20-07	12	12.19-12.80	11	69	20	

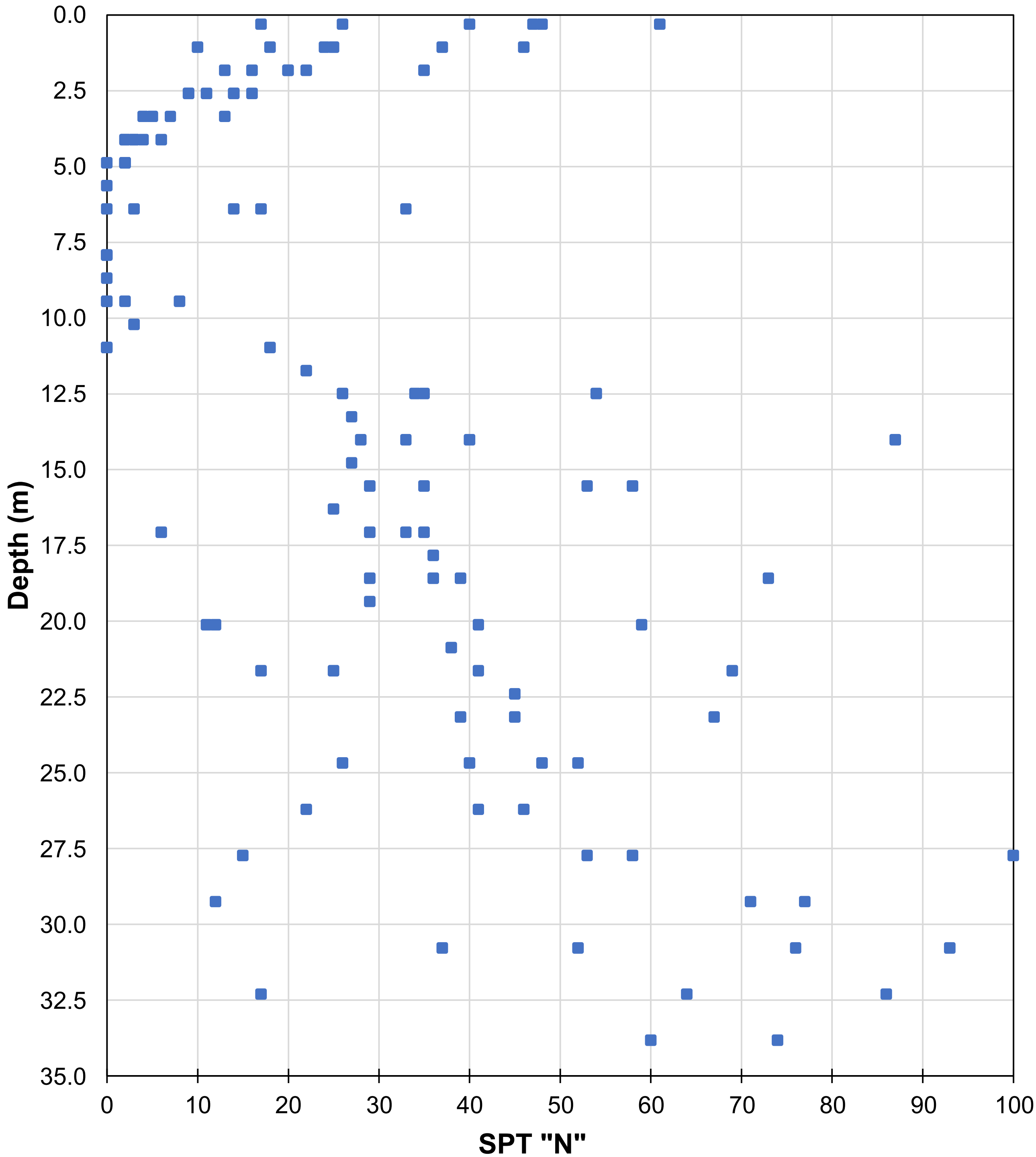
Project: 19134931/3000

GOLDER

Created by: CW

Checked by: MT

SPT "N" vs. Depth 100 Bayshore Drive, Ottawa, ON	FIGURE C-18

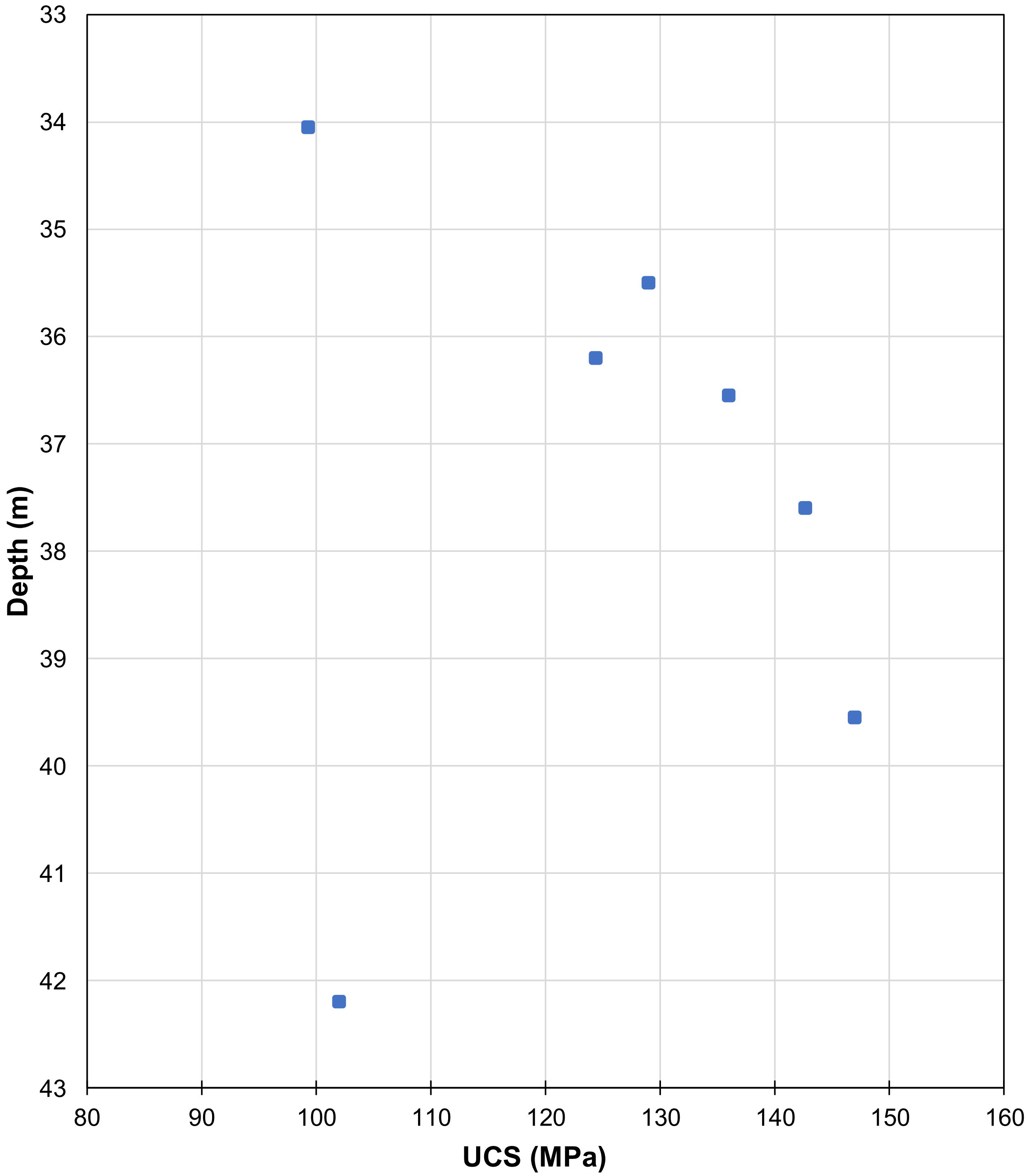


GOLDER

Project	19134931-3000	Drawn AG
Date	September 01, 2020	Chkd WC

UCS vs. Depth
100 Bayshore Drive, Ottawa, ON

FIGURE C-19



GOLDER

Project 19134931-3000
Date September 01, 2020

Drawn AG
Chkd WC

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

GOLDER

UNCONFINED COMPRESSIVE STRENGTH OF ROCK CORE

Project: 100 Bayshore Drive/ Ottawa

Project No.: 19134931

Date: August 13, 2020

Location(s): See Table Below

Bore Hole No.	Depth (m)	Date Tested	Core Size	Diameter (mm)	Density (kg/m³)	Compressive Strength (MPa)	Failure Mode
20-01	33.96-34.11	Aug 11/20	HQ	62.6	2812	99.3	
20-02	36.04-36.15	Aug 11/20	NQ	44.8	2788	124.4	
20-03	35.44-35.58	Aug 11/20	HQ	62.7	2719	129.0	
20-04	36.48-36.63	Aug 11/20	HQ	62.8	2786	136.0	

REMARKS :
- Cores tested in vertical direction.
- Cores tested in air-dry condition.
- 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: Chelsea Ward

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

GOLDER

UNCONFINED COMPRESSIVE STRENGTH OF ROCK CORE

Project: 100 Bayshore Drive/ Ottawa

Project No.: 19134931

Date: August 13, 2020

Location(s): See Table Below

Bore Hole No.	Depth (m)	Date Tested	Core Size	Diameter (mm)	Density (kg/m³)	Compressive Strength (MPa)	Failure Mode
20-05	39.50-39.65	Aug 11/20	HQ	63.0	2670	147.0	
20-05	42.17-42.32	Aug 11/20	HQ	62.9	2795	102.0	
20-07	37.53-37.67	Aug 11/20	HQ	62.8	2774	142.7	

REMARKS : - Cores tested in vertical direction.

- Cores tested in air-dry condition.
- 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: Chelsea Ward

TABLE 1

SUMMARY OF FINES CONTENT DETERMINATIONS

PROJECT NUMBER	19134931/3000
PROJECT NAME	100 Bayshore Drive/Ottawa
DATE TESTED	July 20. 2020

Borehole No.	Sample No.	Depth (m)	Wash Fines (%)
20-01	4	2.29-2.90	98.3%
20-01	9	7.62-8.23	46.2%
20-01	10	10.67-11.28	37.8%
20-02	5	3.05-3.66	94.9%
20-02	9	9.14-10.67	39.1%
20-03	7	4.57-5.18	64.7%
20-03	9	9.14-9.75	53.0%
20-04	8	7.62-8.23	58.1%
20-04	9	10.67-11.28	15.9%
20-05	7	4.57-5.18	89.3%
20-05	9	6.10-6.71	43.7%
20-05	13	9.14-9.75	52.4%
20-05	14	9.91-10.52	63.9%
20-07	11	10.67-11.28	44.1%

NOTE : "Wash Fines" = percent mass lost on washing over #200 (75μm) sieve.

APPENDIX D

Rock Core Photographs

BH 20-01 (Dry)
Rock core from a depth of 33.5 m to 35.8 m
Core Box 1 of 1

Boulder

33.5 m

35.8 m

GOLDER	**Geotechnical Investigation** **Ivanhoé Cambridge - Bayshore Residential Towers** **100 Bayshore Drive, Ottawa, ON**	Project No. 19134931-3000 Drawn: AG Date: 8/1/2020 Checked: AG Review: WC	**BH 20-01** **1 of 2**

BH 20-01 (Wet)
Rock core from a depth of 33.5 m to 35.8 m
Core Box 1 of 1

Boulder

33.5 m

35.8 m

GOLDER	**Geotechnical Investigation** **Ivanhoé Cambridge - Bayshore Residential Towers** **100 Bayshore Drive, Ottawa, ON**	Project No. 19134931-3000 Drawn: AG Date: 8/1/2020 Checked: AG Review: WC	**BH 20-01** **2 of 2**

BH 20-02 (Dry)
Rock core from a depth of 34.4 m to 36.5 m
Core Box 1 of 1

Cobbles & Boulders

33.5 m

34.4 m

36.5 m

GOLDER	**Geotechnical Investigation** **Ivanhoé Cambridge - Bayshore Residential Towers** **100 Bayshore Drive, Ottawa, ON**	Project No. 19134931-3000 Drawn: AG Date: 8/1/2020 Checked: AG Review: WC	**BH 20-02** **1 of 2**

BH 20-02 (Wet)
Rock core from a depth of 34.4 m to 36.5 m
Core Box 1 of 1

Cobbles & Boulders

33.5 m

34.4 m

36.5 m

GOLDER	**Geotechnical Investigation** **Ivanhoé Cambridge - Bayshore Residential Towers** **100 Bayshore Drive, Ottawa, ON**	Project No. 19134931-3000 Drawn: AG Date: 8/1/2020 Checked: AG Review: WC	**BH 20-02** **2 of 2**

BH 20-03 (Dry)
Rock core from a depth of 34.6 m to 36.4 m
Core Box 1 of 1

34.6 m

36.4 m

GOLDER	**Geotechnical Investigation** **Ivanhoé Cambridge - Bayshore Residential Towers** **100 Bayshore Drive, Ottawa, ON**	Project No. 19134931-3000 Drawn: AG Date: 8/1/2020 Checked: AG Review: WC	**BH 20-03** **1 of 2**

BH 20-03 (Wet)
Rock core from a depth of 34.6 m to 36.4 m
Core Box 1 of 1

34.6 m

36.4 m

Geotechnical Investigation

Ivanhoé Cambridge - Bayshore Residential Towers

100 Bayshore Drive, Ottawa, ON

GOLDER

Project No.	19134931-3000
Drawn:	AG
Date:	8/1/2020
Checked:	AG
Review:	WC

BH 20-04 (Dry)
Rock core from a depth of 34.9 m to 37.8 m
Core Box 1 of 2

34.9 m

37.8 m

GOLDER	**Geotechnical Investigation** **Ivanhoé Cambridge - Bayshore Residential Towers** **100 Bayshore Drive, Ottawa, ON**	Project No. 19134931-3000 Drawn: AG Date: 8/1/2020 Checked: AG Review: WC	**BH 20-04** **1 of 4**

BH 20-04 (Dry)
Rock core from a depth of 37.8 m to 38.5
Core Box 2 of 2

37.8 m

38.5 m

GOLDER	**Geotechnical Investigation** **Ivanhoé Cambridge - Bayshore Residential Towers** **100 Bayshore Drive, Ottawa, ON**	Project No. 19134931-3000 Drawn: AG Date: 8/1/2020 Checked: AG Review: WC	**BH 20-04** **2 of 4**

BH 20-04 (Dry)
Rock core from a depth of 34.9 m to 37.8 m
Core Box 1 of 2

34.9 m

37.8 m

GOLDER	**Geotechnical Investigation** **Ivanhoé Cambridge - Bayshore Residential Towers** **100 Bayshore Drive, Ottawa, ON**	Project No. 19134931-3000 Drawn: AG Date: 8/1/2020 Checked: AG Review: WC	**BH 20-04** **3 of 4**

BH 20-04 (Wet)
Rock core from a depth of 37.8 m to 38.5
Core Box 2 of 2

37.8 m

38.5 m

GOLDER	**Geotechnical Investigation** **Ivanhoé Cambridge - Bayshore Residential Towers** **100 Bayshore Drive, Ottawa, ON**	Project No. 19134931-3000 Drawn: AG Date: 8/1/2020 Checked: AG Review: WC	**BH 20-04** **4 of 4**

BH 20-05 (Dry)
Rock core from a depth of 36.0 m to 38.6 m
Core Box 1 of 3

36.0 m

38.6 m

GOLDER

Geotechnical Investigation

Ivanhoé Cambridge - Bayshore Residential Towers

100 Bayshore Drive, Ottawa, ON

Project No.	19134931-3000
Drawn:	AG
Date:	8/1/2020
Checked:	AG
Review:	WC

BH 20-05 (Dry)
Rock core from a depth of 38.6 m to 41.3 m
Core Box 2 of 3

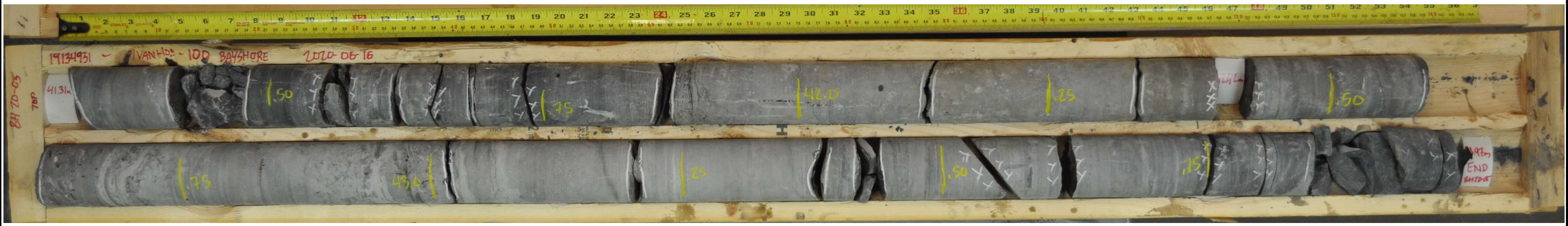
38.6 m

41.3 m

GOLDER	**Geotechnical Investigation** **Ivanhoé Cambridge - Bayshore Residential Towers** **100 Bayshore Drive, Ottawa, ON**	Project No. 19134931-3000 Drawn: AG Date: 8/1/2020 Checked: AG Review: WC	**BH 20-05** **2 of 6**

BH 20-05 (Dry)
Rock core from a depth of 41.3 m to 44.0 m
Core Box 3 of 3

41.3 m



44.0 m

GOLDER	**Geotechnical Investigation** **Ivanhoé Cambridge - Bayshore Residential Towers** **100 Bayshore Drive, Ottawa, ON**	Project No. 19134931-3000 Drawn: AG Date: 8/1/2020 Checked: AG Review: WC	**BH 20-05** **3 of 6**

BH 20-05 (Wet)
Rock core from a depth of 36.0 m to 38.6 m
Core Box 1 of 3

36.0 m

38.6 m

GOLDER	**Geotechnical Investigation** **Ivanhoé Cambridge - Bayshore Residential Towers** **100 Bayshore Drive, Ottawa, ON**	Project No. 19134931-3000 Drawn: AG Date: 8/1/2020 Checked: AG Review: WC	**BH 20-05** **4 of 6**

BH 20-05 (Wet)
Rock core from a depth of 38.6 m to 41.3 m
Core Box 2 of 3

38.6 m

41.3 m

GOLDER	**Geotechnical Investigation** **Ivanhoé Cambridge - Bayshore Residential Towers** **100 Bayshore Drive, Ottawa, ON**	Project No. 19134931-3000 Drawn: AG Date: 8/1/2020 Checked: AG Review: WC	**BH 20-05** **5 of 6**

BH 20-05 (Wet)
Rock core from a depth of 41.3 m to 44.0 m
Core Box 3 of 3

41.3 m

44.0 m

GOLDER	**Geotechnical Investigation** **Ivanhoé Cambridge - Bayshore Residential Towers** **100 Bayshore Drive, Ottawa, ON**	Project No. 19134931-3000 Drawn: AG Date: 8/1/2020 Checked: AG Review: WC	**BH 20-05** **6 of 6**

BH 20-07 (Dry)
Rock core from a depth of 34.4 m to 36.9 m
Core Box 1 of 2

Boulder

34.4 m

36.9 m

GOLDER	**Geotechnical Investigation** **Ivanhoé Cambridge - Bayshore Residential Towers** **100 Bayshore Drive, Ottawa, ON**	Project No. 19134931-3000 Drawn: AG Date: 8/1/2020 Checked: AG Review: WC	**BH 20-07** **1 of 4**

BH 20-07 (Dry)
Rock core from a depth of 36.9 m to 37.8 m
Core Box 2 of 2

36.9 m

37.8 m

GOLDER

Geotechnical Investigation

Ivanhoé Cambridge - Bayshore Residential Towers

100 Bayshore Drive, Ottawa, ON

Project No.	19134931-3000
Drawn:	AG
Date:	8/1/2020
Checked:	AG
Review:	WC

BH 20-07
2 of 4

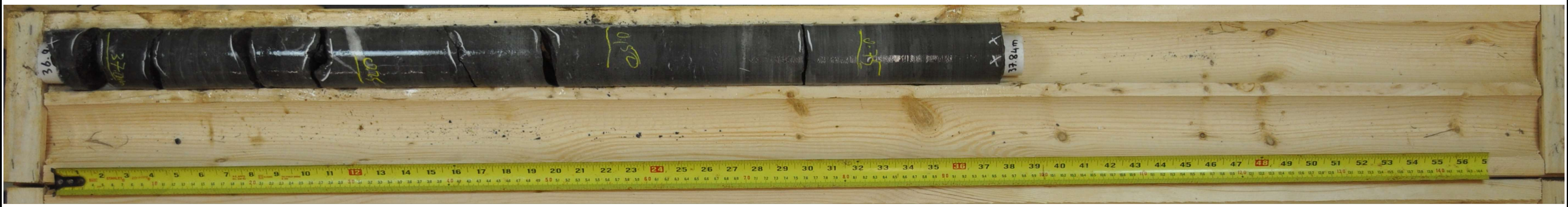
BH 20-07 (Wet)
Rock core from a depth of 34.4 m to 36.9 m
Core Box 1 of 2

GOLDER	**Geotechnical Investigation** **Ivanhoé Cambridge - Bayshore Residential Towers** **100 Bayshore Drive, Ottawa, ON**	Project No. 19134931-3000 Drawn: AG Date: 8/1/2020 Checked: AG Review: WC	**BH 20-07** **3 of 4**

BH 20-07 (Wet)
Rock core from a depth of 36.9 m to 37.8 m
Core Box 2 of 2

36.9 m

37.8 m



GOLDER

Geotechnical Investigation

Ivanhoé Cambridge - Bayshore Residential Towers

100 Bayshore Drive, Ottawa, ON

Project No.	19134931-3000
Drawn:	AG
Date:	8/1/2020
Checked:	AG
Review:	WC

BH 20-07
4 of 4

APPENDIX E

Results of Basic Chemical Analyses

Certificate of Analysis

Environment Testing

Client:	Golder Associates Ltd. (Ottawa) 1931 Robertson Road Ottawa, ON K2H 5B7	Report Number:	1934929
Attention:	Ms. Ali Ghirian	Date Submitted:	2020-07-21
PO#:		Date Reported:	2020-07-29
Invoice to:	Golder Associates Ltd. (Ottawa)	Project:	19134931/3000
		COC #:	860475

Group	Analyte	MRL	Units	Guideline	1505695	1505696
				Lab I.D.	1505695	1505696
				Sample Matrix	Soil	Soil
				Sample Type		
				Sampling Date	2020-06-22	2020-07-09
				Sample I.D.	BH20-04 sa5	BH20-07 sa5
Anions	Cl	0.002	%		0.014	0.024
	SO4	0.01	%		0.02	0.01
General Chemistry	Electrical Conductivity	0.05	mS/cm		0.51	0.20
	pH	2.00			7.67	7.37
	Resistivity	1	ohm-cm		1960	5000

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 F

Results of Geophysical Testing (VSP Testing)

GOLDER

TECHNICAL MEMORANDUM

DATE July 16, 2020 **Project No.** 19134931-3000

TO Ali Ghirian, P.Eng., Golder Associates Ltd

FROM Peter Leith, Christopher Phillips **EMAIL** pleith@golder.com, cphillips@golder.com

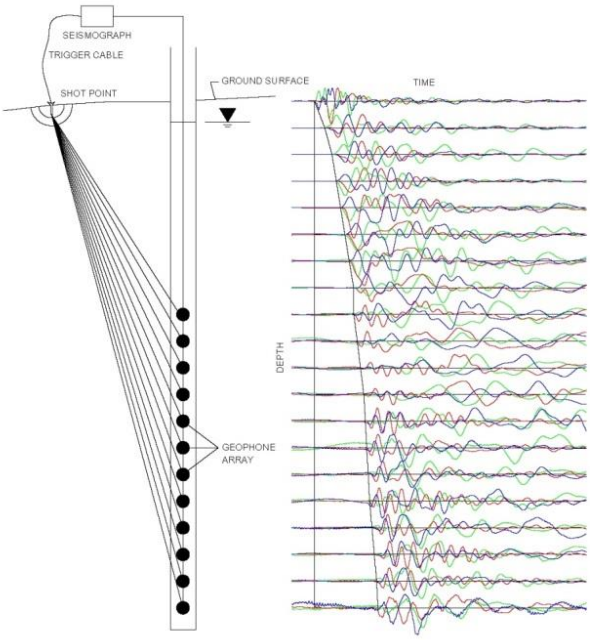
VERTICAL SEISMIC PROFILING TEST RESULTS
100 BAYSHORE DRIVE, OTTAWA, ONTARIO

This memorandum presents the results of a Vertical Seismic Profiling (VSP) test carried out for the Bayshore Residential development to be located at 100 Bayshore Drive in Ottawa, Ontario. VSP testing was carried out on June 26, 2020. Borehole 20-05 was drilled to an approximate depth of 44 m below the existing ground surface and then cased with a 2.5 inch PVC pipe grouted in place. Golder was retained by Ivanhoé Cambridge to carry out this work.

Methodology

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

The high resolution results of a VSP survey are often used for earthquake engineering site classification, as per the National Building of Canada (2015).



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

Field Work

The field work was carried out on June 26, 2020 by personnel from the Golder Mississauga office.

At BH 20-05, the compression and shear-wave seismic sources were used, and they were located 2.75 m from the borehole. The seismic source for the compression wave test consisted of a 9.9 kilogram sledge hammer vertically impacted on a metal plate. The seismic source for the shear-wave test consisted of a 2.4-metre-long, 150 millimetre by 150 millimetre wooden beam, weighted by a vehicle and horizontally struck with a 9.9 kilogram sledge hammer on alternate ends of the beam to induce polarized shear waves. Test measurements started at ground surface and were recorded in the borehole with a 3-component receiver spaced at 1-metre intervals below the ground surface to the maximum depth of the casing (44 m).

The seismic records collected for each source location were stacked a minimum of five times to minimize the effects of ambient background seismic noise on the collected data. The data was sampled at 0.020833 millisecond intervals and a total time window of 0.128 seconds was collected for each seismic shot.

Data Processing

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

1) Combination of seismic records to present seismic traces for all depth intervals on a single plot for each seismic source and for each component;
2) Low Pass Filtering of data to remove spurious high frequency noise;
3) First break picking of the compression and shear-wave arrivals; and,
4) Calculation of the average compression and shear-wave velocity to each tested depth interval.

Processing of the VSP data was completed using the SeisImager/SW software package (Geometrics Inc.). The seismic records at BH20-05 are presented on the following two plots and show the first break picks of the compression wave (Figure 1) and shear wave arrivals (Figure 2) overlaid on the seismic waveform traces recorded at the different geophone depths. The arrivals were picked on the vertical component for the compression source and on the two horizontal components for the shear source.

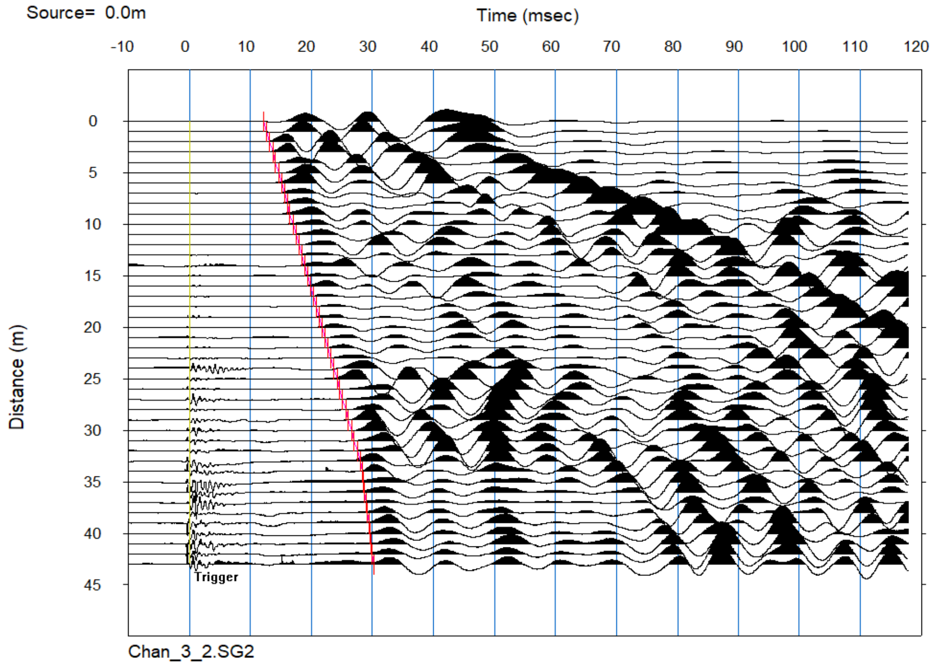


Figure 1: First break picking of compression wave arrivals (red) along the seismic traces recorded at each receiver depth of Borehole 20-05.

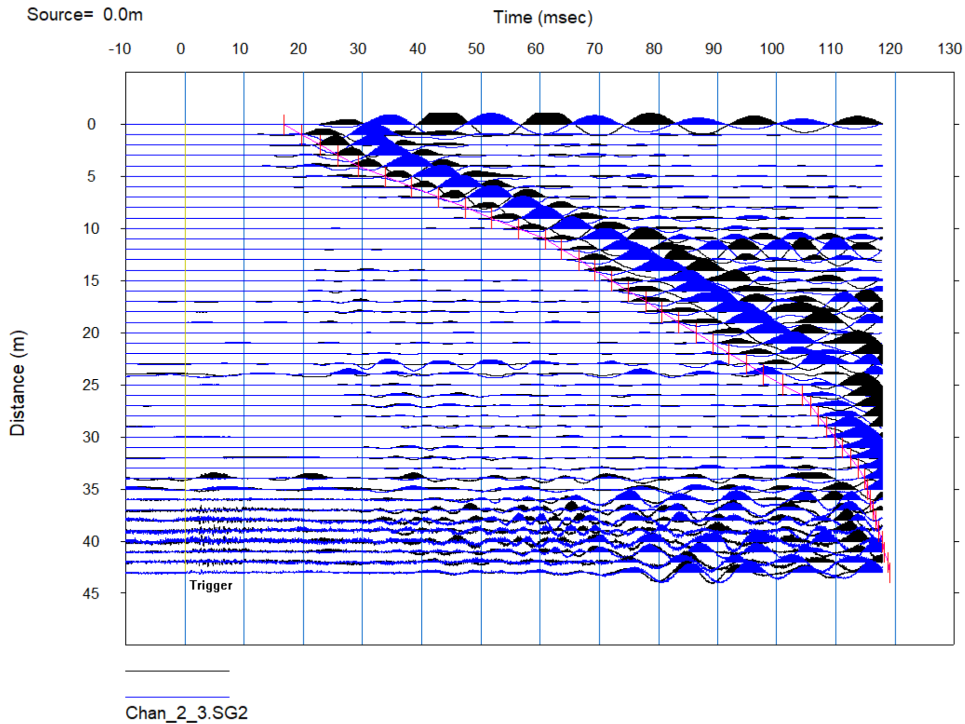


Figure 2: First break picking of compression wave arrivals (red) along the seismic traces recorded at each receiver depth of Borehole 20-05.

Results

The VSP results at borehole 20-05 are summarized in Table 1. The shear wave and compression wave layer velocities were calculated by best fitting a theoretical travel time model to the field data. The depths presented on the table are relative to ground surface.

The estimated dynamic engineering moduli, based on the calculated wave velocities, are also presented in Table 1. The engineering moduli were calculated using an estimated bulk density, based on the borehole log. An estimated bulk density of 1,650 kg/m^3 was used for the sandy silt layers which comprised the first 10 m of the borehole, changing to 2,000 kg/m^3 at the onset of gravelly sand while a bulk density of 2,830 kg/m^3 better corresponded to the dolostone seen from 36 m to the end of the borehole.

At borehole 20-05 the average shear wave velocity from ground surface to a depth of 30 metres was measured to be 308 metres per second.

Limitations

This technical memorandum, which specifically includes all tables, figures and attachments, is based on data and information collected by Golder Associates Ltd. and is based solely on the conditions of the properties at the time of the work, supplemented by historical information and data obtained by Golder Associates Ltd. as described in this memo.

Golder Associates Ltd. has relied in good faith on all information provided and does not accept responsibility for any deficiency, misstatements, or inaccuracies contained in the reports as a result of omissions, misinterpretation, or fraudulent acts of the persons contacted or errors or omissions in the reviewed documentation.

The services performed, as described in this memo, were conducted in a manner consistent with that level of care and skill normally exercised by other members of the engineering and science professions currently practicing under similar conditions, subject to the time limits and financial and physical constraints applicable to the services.

Any use which a third party makes of this memo, or any reliance on, or decisions to be made based on it, are the responsibilities of such third parties. Golder Associates Ltd. accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this memo.

The findings and conclusions of this memo are valid only as of the date of this memo. If new information is discovered in future work, including excavations, borings, or other studies, Golder Associates Ltd. should be requested to re-evaluate the conclusions of this memo, and to provide amendments as required.

Closure

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

GOLDER ASSOCIATES LTD.

Peter Leith, B.Sc.
Junior Geophysicist

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

PL/CRP/jl

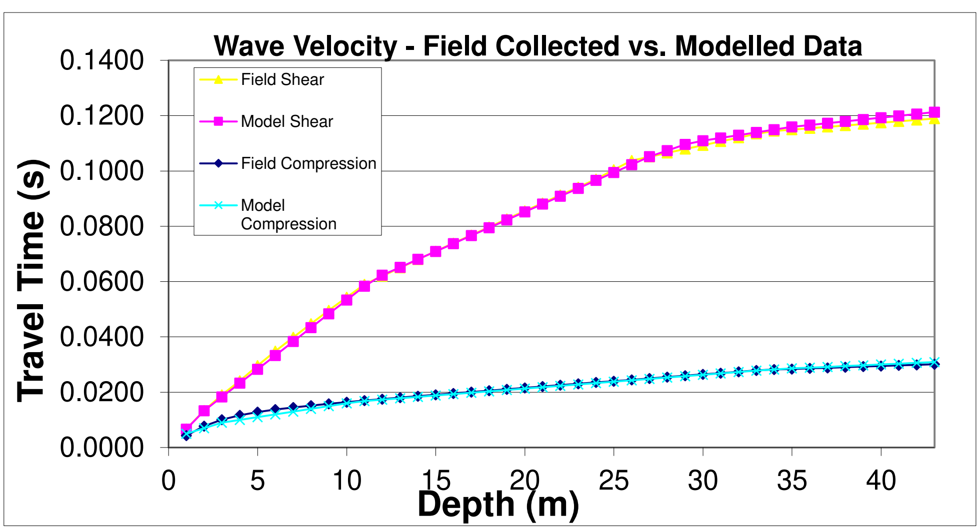
Attach: Table 1

c:\users\jrlee\desktop\projects\temp files -1\19134931\19134931 tech memo 100 bayshore vsp 16 july 2020.docx

TABLE 1
SHEAR WAVE VELOCITY PROFILE AT BOREHOLE 20-05

Layer Depth (m)				Estimated Bulk Density	Dynamic Engineering Properties			
Top	Bottom	Compressional Wave (m/s)	Shear Wave (m/s)	(kg/m³)	Poissons Ratio	Shear Modulus (MPa)	Deformation Modulus (MPa)	Bulk Modulus (MPa)
0.0	1	200	150	1650	-0.14	37	64	17
1.0	2	500	150	1650	0.45	37	108	363
2.0	3	500	200	1650	0.40	66	185	325
3.0	4	1000	200	1650	0.48	66	195	1562
4.0	5	1000	200	1650	0.48	66	195	1562
5.0	6	1000	200	1650	0.48	66	195	1562
6.0	7	1000	200	1650	0.48	66	195	1562
7.0	8	1000	200	1650	0.48	66	195	1562
8.0	9	1000	200	1650	0.48	66	195	1562
9.0	10	1000	200	1650	0.48	66	195	1562
10.0	11	1250	200	2000	0.49	80	238	3018
11.0	12	2000	250	2000	0.49	125	373	7833
12.0	13	2000	350	2000	0.48	245	727	7673
13.0	14	2000	350	2000	0.48	245	727	7673
14.0	15	2000	350	2000	0.48	245	727	7673
15.0	16	2000	350	2000	0.48	245	727	7673
16.0	17	2000	350	2000	0.48	245	727	7673
17.0	18	2000	350	2000	0.48	245	727	7673
18.0	19	2000	350	2000	0.48	245	727	7673
19.0	20	2000	350	2000	0.48	245	727	7673
20.0	21	2000	350	2000	0.48	245	727	7673
21.0	22	2000	350	2000	0.48	245	727	7673
22.0	23	2000	350	2000	0.48	245	727	7673
23.0	24	2000	350	2000	0.48	245	727	7673
24.0	25	2000	350	2150	0.48	263	782	8249
25.0	26	2000	350	2150	0.48	263	782	8249
26.0	27	2000	350	2150	0.48	263	782	8249
27.0	28	2000	450	2150	0.47	435	1283	8020
28.0	29	2000	450	2150	0.47	435	1283	8020
29.0	30	2000	750	2150	0.42	1209	3430	6988
30.0	31	2000	1000	2150	0.33	2150	5733	5733
31.0	32	2000	1000	2150	0.33	2150	5733	5733
32.0	33	2000	1000	2150	0.33	2150	5733	5733
33.0	34	2500	1000	2150	0.40	2150	6040	10571
34.0	35	2500	1000	2150	0.40	2150	6040	10571
35.0	36	3500	1500	2830	0.39	6368	17670	26178
36.0	37	3500	1500	2830	0.39	6368	17670	26178
37.0	38	3500	1500	2830	0.39	6368	17670	26178
38.0	39	3500	1500	2830	0.39	6368	17670	26178
39.0	40	3500	1500	2830	0.39	6368	17670	26178
40.0	41	3500	1500	2830	0.39	6368	17670	26178
41.0	42	3500	1500	2830	0.39	6368	17670	26178
42.0	43	3500	1500	2830	0.39	6368	17670	26178

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



APPENDIX G

Results of SCPTu Testing

PRESENTATION OF SITE INVESTIGATION RESULTS

100 Bayshore Drive

Prepared for:

Golder Associates

ConeTec Job No: 20-05-21040

Project Start Date: 03-Jul-2020
Project End Date: 03-Jul-2020
Report Date: 08-Jul-2020
Revision Date: 10-Jul-2020

Prepared by:

ConeTec Investigations Ltd.
9033 Leslie Street, Unit 15
Richmond Hill, ON L4B 4K3

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Toll Free: (800) 504-1116

ConeTecON@conetec.com
www.conetec.com
www.conetecdataservices.com

CONETEC

Introduction

The enclosed report presents the results of the site investigation program conducted by ConeTec Investigations Ltd. for Golder Associates at 100 Bayshore Drive, Ottawa, ON. The program consisted of three seismic cone penetration tests (SCPTu).

Project Information

Project	
Client	Golder Associates
Project	100 Bayshore Drive
ConeTec project number	20-05-21040

An aerial overview from Google Earth including the SCPTu test locations is presented below.

Rig Description	Deployment System	Test Type
CPT truck rig (C3)	30 ton rig cylinder	SCPTu

Coordinates		
Test Type	Collection Method	EPSG Number
SCPTu	Client-provided	32168

Cone Penetrometers Used for this Project						
Cone Description	Cone Number	Cross Sectional Area (cm^2)	Sleeve Area (cm^2)	Tip Capacity (bar)	Sleeve Capacity (bar)	Pore Pressure Capacity (psi)
531:T1500F15U500	531	15	225	1500	15	500
545:T1500F15U500	531	15	225	1500	15	500
The CPT summary indicates which cone was used for each sounding.						

Cone Penetration Test (CPTu)	
Depth reference	Depths are referenced to the existing ground surface at the time of each test.
Tip and sleeve data offset	0.1 meter This has been accounted for in the CPT data files.
Additional plots	Standard-expanded range, Seismic-Vs and Advanced CPT plots with I_c, Su(N_{kt}), Phi and $N_{1(60)}$(IcRW1998) as well as SBT Scatter plots are provided in the release package.

Calculated Geotechnical Parameter Tables	
Additional information	The Normalized Soil Behavior Type Chart based on Q_{tn} (SBT Qtn) (Robertson, 2009) was used to classify the soil for this project. A detailed set of calculated CPT parameters have been generated and are provided in Excel format files in the release folder. The CPT parameter calculations are based on values of corrected tip resistance (q_t) sleeve friction (f_s), and pore pressure (u_2). Equilibrium pore pressure profiles were used for the calculated parameters. Effective stresses are calculated based on unit weights that have been assigned to the individual soil behavior type zones and the assumed equilibrium pore pressure profile. Soils were classified as either drained or undrained based on the Q_{tn} Normalized Soil Behaviour Type Chart (Robertson, 2009). Calculations for both drained and undrained parameters were included for materials that classified as silt mixtures (zone 4).

Limitations

This report has been prepared for the exclusive use of Golder Associates (Client) for the project titled "100 Bayshore Drive". The report's contents may not be relied upon by any other party without the express written permission of ConeTec Investigations Ltd. (ConeTec). ConeTec has provided site investigation services, prepared the factual data reporting and provided geotechnical parameter calculations consistent with current best practices. No other warranty, expressed or implied, is made.

The information presented in the report document and the accompanying data set pertain to the specific project, site conditions and objectives described to ConeTec by the Client. In order to properly understand the factual data, assumptions and calculations, reference must be made to the documents provided and their accompanying data sets, in their entirety.

Cone penetration tests (CPTu) are conducted using an integrated electronic piezocone penetrometer and data acquisition system manufactured by Adara Systems Ltd., a subsidiary of ConeTec.

ConeTec's piezocone penetrometers are compression type designs in which the tip and friction sleeve load cells are independent and have separate load capacities. The piezocones use strain gauged load cells for tip and sleeve friction and a strain gauged diaphragm type transducer for recording pore pressure. The piezocones also have a platinum resistive temperature device (RTD) for monitoring the temperature of the sensors, an accelerometer type dual axis inclinometer and a geophone sensor for recording seismic signals. All signals are amplified down hole within the cone body and the analog signals are sent to the surface through a shielded cable.

ConeTec penetrometers are manufactured with various tip, friction and pore pressure capacities in 5 cm^2, 10 cm^2 and 15 cm^2 tip base area configurations in order to maximize signal resolution for various soil conditions. The specific piezocone used for each test is described in the CPT summary table presented in the first appendix. The 15 cm^2 penetrometers do not require friction reducers as they have a diameter larger than the deployment rods. The 10 cm^2 piezocones use a friction reducer consisting of a rod adapter extension behind the main cone body with an enlarged cross sectional area (typically 44 mm diameter over a length of 32 mm with tapered leading and trailing edges) located at a distance of 585 mm above the cone tip.

The penetrometers are designed with equal end area friction sleeves, a net end area ratio of 0.8 and cone tips with a 60 degree apex angle.

All ConeTec piezocones can record pore pressure at various locations. Unless otherwise noted, the pore pressure filter is located directly behind the cone tip in the "u_2" position (ASTM Type 2). The filter is 6 mm thick, made of porous plastic (polyethylene) having an average pore size of 125 microns (90-160 microns). The function of the filter is to allow rapid movements of extremely small volumes of water needed to activate the pressure transducer while preventing soil ingress or blockage.

The piezocone penetrometers are manufactured with dimensions, tolerances and sensor characteristics that are in general accordance with the current ASTM D5778 standard. ConeTec's calibration criteria also meets or exceeds those of the current ASTM D5778 standard. An illustration of the piezocone penetrometer is presented in Figure CPTu.

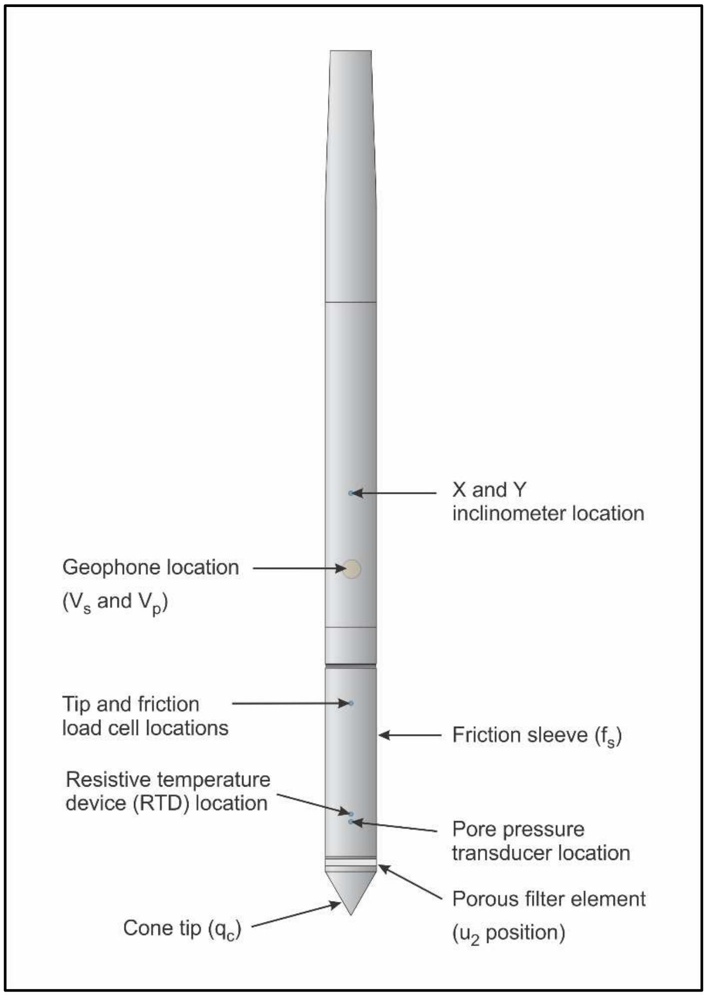


Figure CPTu. Piezocone Penetrometer (15 cm^2)

The ConeTec data acquisition systems consist of a Windows based computer and a signal conditioner and power supply interface box with a 16 bit (or greater) analog to digital (A/D) converter. The data is recorded at fixed depth increments using a depth wheel attached to the push cylinders or by using a spring loaded rubber depth wheel that is held against the cone rods. The typical recording interval is 2.5 cm; custom recording intervals are possible.

The system displays the CPTu data in real time and records the following parameters to a storage media during penetration:

- Depth
- Uncorrected tip resistance (q_c)
- Sleeve friction (f_s)
- Dynamic pore pressure (u)
- Additional sensors such as resistivity, passive gamma, ultra violet induced fluorescence, if applicable

All testing is performed in accordance to ConeTec's CPT operating procedures which are in general accordance with the current ASTM D5778 standard.

Prior to the start of a CPTu sounding a suitable cone is selected, the cone and data acquisition system are powered on, the pore pressure system is saturated with either glycerine or silicone oil and the baseline readings are recorded with the cone hanging freely in a vertical position.

The CPTu is conducted at a steady rate of 2 cm/s, within acceptable tolerances. Typically one meter length rods with an outer diameter of 38.1 mm are added to advance the cone to the sounding termination depth. After cone retraction final baselines are recorded.

Additional information pertaining to ConeTec's cone penetration testing procedures:

- Each filter is saturated in silicone oil under vacuum pressure prior to use
- Recorded baselines are checked with an independent multi-meter
- Baseline readings are compared to previous readings
- Soundings are terminated at the client's target depth or at a depth where an obstruction is encountered, excessive rod flex occurs, excessive inclination occurs, equipment damage is likely to take place, or a dangerous working environment arises
- Differences between initial and final baselines are calculated to ensure zero load offsets have not occurred and to ensure compliance with ASTM standards

The interpretation of piezocone data for this report is based on the corrected tip resistance (q_t), sleeve friction (f_s) and pore water pressure (u). The interpretation of soil type is based on the correlations developed by Robertson et al. (1986) and Robertson (1990, 2009). It should be noted that it is not always possible to accurately identify a soil behaviour type based on these parameters. In these situations, experience, judgment and an assessment of other parameters may be used to infer soil behaviour type.

The recorded tip resistance (q_c) is the total force acting on the piezocone tip divided by its base area. The tip resistance is corrected for pore pressure effects and termed corrected tip resistance (q_t) according to the following expression presented in Robertson et al. (1986):

$$q_t = q_c + (1-a) \bullet u_2$$

where: q_t is the corrected tip resistance
q_c is the recorded tip resistance
u_2 is the recorded dynamic pore pressure behind the tip (u_2 position)
a is the Net Area Ratio for the piezocone (0.8 for ConeTec probes)

The sleeve friction (f_s) is the frictional force on the sleeve divided by its surface area. As all ConeTec piezocones have equal end area friction sleeves, pore pressure corrections to the sleeve data are not required.

The dynamic pore pressure (u) is a measure of the pore pressures generated during cone penetration. To record equilibrium pore pressure, the penetration must be stopped to allow the dynamic pore pressures to stabilize. The rate at which this occurs is predominantly a function of the permeability of the soil and the diameter of the cone.

The friction ratio (Rf) is a calculated parameter. It is defined as the ratio of sleeve friction to the tip resistance expressed as a percentage. Generally, saturated cohesive soils have low tip resistance, high friction ratios and generate large excess pore water pressures. Cohesionless soils have higher tip resistances, lower friction ratios and do not generate significant excess pore water pressure.

A summary of the CPTu soundings along with test details and individual plots are provided in the appendices. A set of files with calculated geotechnical parameters were generated for each sounding based on published correlations and are provided in Excel format in the data release folder. Information regarding the methods used is also included in the data release folder.

For additional information on CPTu interpretations and calculated geotechnical parameters, refer to Robertson et al. (1986), Lunne et al. (1997), Robertson (2009), Mayne (2013, 2014) and Mayne and Peuchen (2012).

Shear wave velocity (Vs) testing is performed in conjunction with the piezocone penetration test (SCPTu) in order to collect interval velocities. For some projects seismic compression wave velocity (Vp) testing is also performed.

ConeTec's piezocone penetrometers are manufactured with a horizontally active geophone (28 hertz) that is rigidly mounted in the body of the cone penetrometer, 0.2 meters behind the cone tip.

Shear waves are typically generated by using an impact hammer horizontally striking a beam that is held in place by a normal load. In some instances an auger source or an imbedded impulsive source maybe used for both shear waves and compression waves. The hammer and beam act as a contact trigger that initiates the recording of the seismic wave traces. For impulsive devices an accelerometer trigger may be used. The traces are recorded using an up-hole integrated digital oscilloscope which is part of the SCPTu data acquisition system. An illustration of the shear wave testing configuration is presented in Figure SCPTu-1.

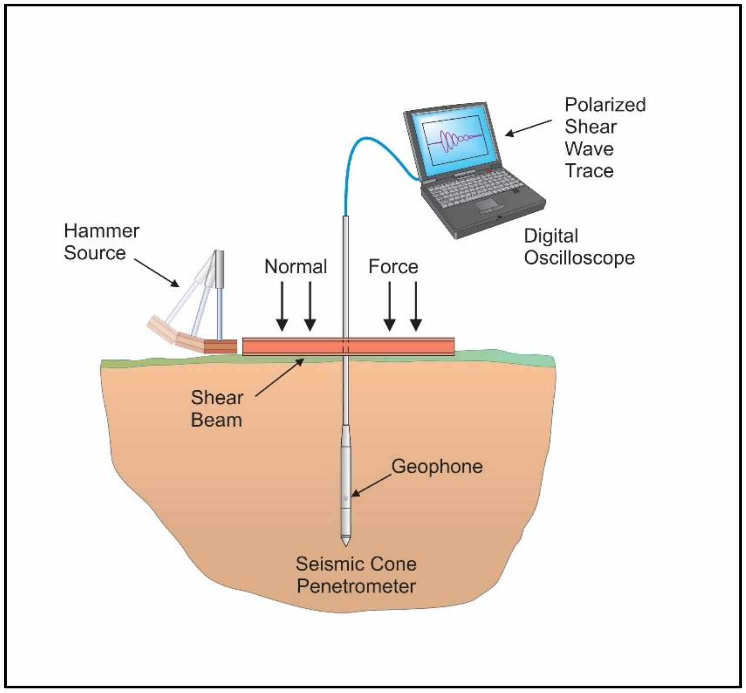


Figure SCPTu-1. Illustration of the SCPTu system

All testing is performed in accordance to ConeTec's SCPTu operating procedures which are in general accordance with the current ASTM D5778 and ASTM D7400 standards.

Prior to the start of a SCPTu sounding, the procedures described in the Cone Penetration Test section are followed. In addition, the active axis of the geophone is aligned parallel to the beam (or source) and the horizontal offset between the cone and the source is measured and recorded.

Prior to recording seismic waves at each test depth, cone penetration is stopped and the rods are decoupled from the rig to avoid transmission of rig energy down the rods. Typically, five wave traces for each orientation are recorded for quality control and uncertainty analysis purposes. After reviewing wave traces for consistency the cone is pushed to the next test depth (typically one meter intervals or as requested by the client). Figure SCPTu-2 presents an illustration of a SCPTu test.

For additional information on seismic cone penetration testing refer to Robertson et. al. (1986).

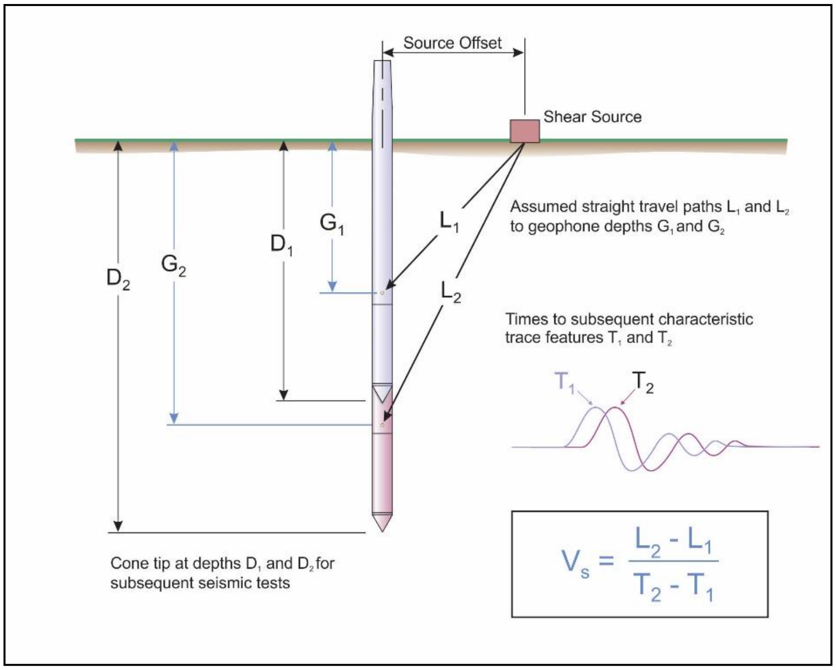


Figure SCPTu-2. Illustration of a seismic cone penetration test

Calculation of the interval velocities are performed by visually picking a common feature (e.g. the first characteristic peak, trough, or crossover) on all of the recorded wave sets and taking the difference in ray path divided by the time difference between subsequent features. Ray path is defined as the straight line distance from the seismic source to the geophone, accounting for beam offset, source depth and geophone offset from the cone tip.

The average shear wave velocity to a depth of 30 meters (V_{s30}) has been calculated and provided for all applicable soundings using an equation presented in Crow et al. (2012).

$$V_{s30} = \frac{total\ thickness\ of\ all\ layers\ (30m)}{\sum(layer\ traveltimes)}$$

The layer travel times refers to the travel times propagating in the vertical direction, not the measured travel times from an offset source.

Tabular results and SCPTu plots are presented in the relevant appendix.

The cone penetration test is halted at specific depths to carry out pore pressure dissipation (PPD) tests, shown in Figure PPD-1. For each dissipation test the cone and rods are decoupled from the rig and the data acquisition system measures and records the variation of the pore pressure (u) with time (t).

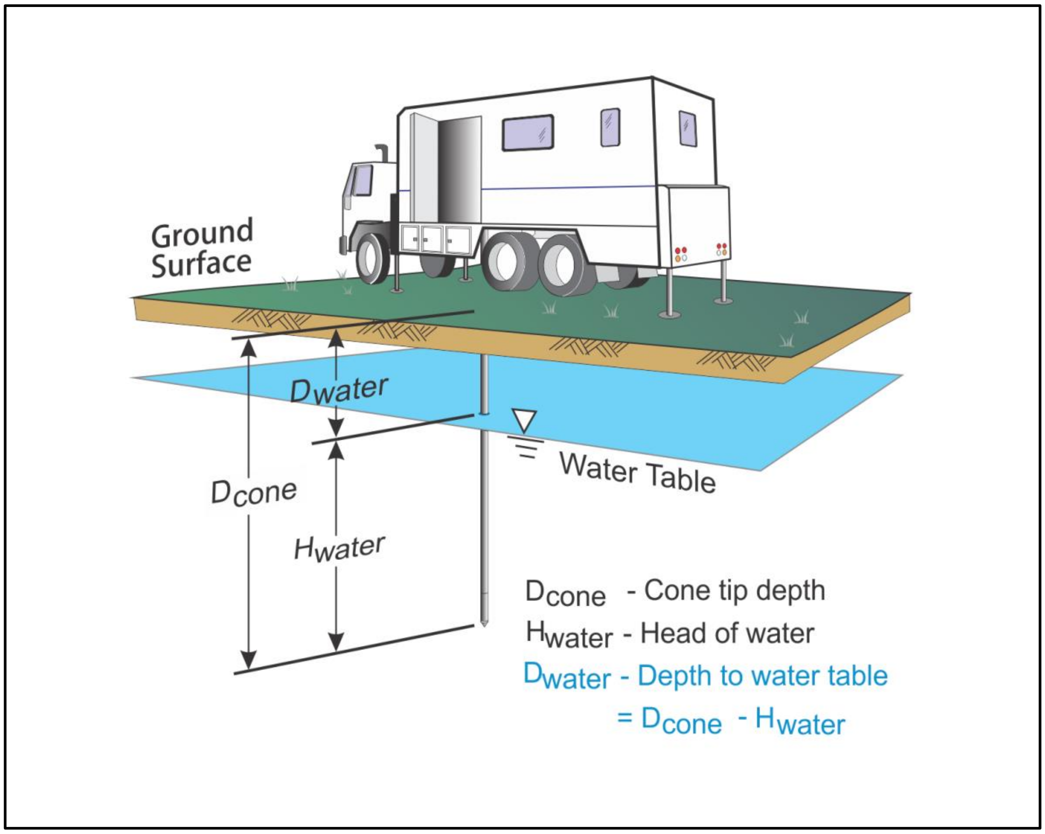


Figure PPD-1. Pore pressure dissipation test setup

Pore pressure dissipation data can be interpreted to provide estimates of ground water conditions, permeability, consolidation characteristics and soil behaviour.

The typical shapes of dissipation curves shown in Figure PPD-2 are very useful in assessing soil type, drainage, in situ pore pressure and soil properties. A flat curve that stabilizes quickly is typical of a freely draining sand. Undrained soils such as clays will typically show positive excess pore pressure and have long dissipation times. Dilative soils will often exhibit dynamic pore pressures below equilibrium that then rise over time. Overconsolidated fine-grained soils will often exhibit an initial dilatory response where there is an initial rise in pore pressure before reaching a peak and dissipating.

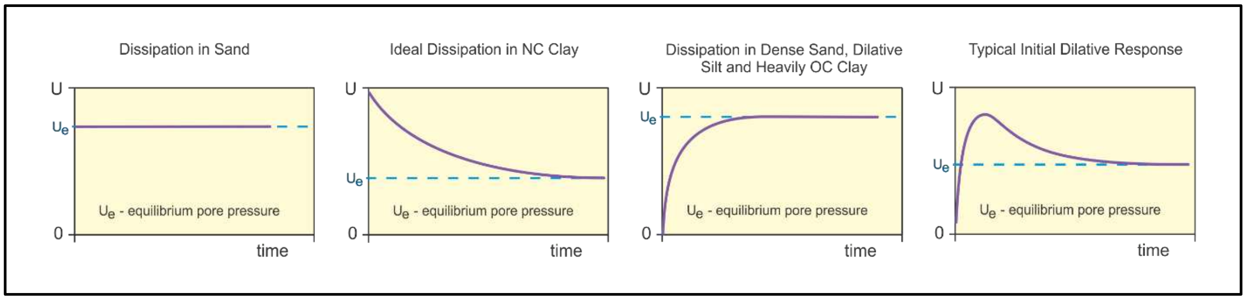


Figure PPD-2. Pore pressure dissipation curve examples

In order to interpret the equilibrium pore pressure (u_{eq}) and the apparent phreatic surface, the pore pressure should be monitored until such time as there is no variation in pore pressure with time as shown for each curve in Figure PPD-2.

In fine grained deposits the point at which 100% of the excess pore pressure has dissipated is known as t_{100}. In some cases this can take an excessive amount of time and it may be impractical to take the dissipation to t_{100}. A theoretical analysis of pore pressure dissipations by Teh and Houlsby (1991) showed that a single curve relating degree of dissipation versus theoretical time factor (T*) may be used to calculate the coefficient of consolidation (c_h) at various degrees of dissipation resulting in the expression for c_h shown below.

$$c_h = \frac{T^* \cdot a^2 \cdot \sqrt{I_r}}{t}$$

Where:

T* is the dimensionless time factor (Table Time Factor)
a is the radius of the cone
I_r is the rigidity index
t is the time at the degree of consolidation

Table Time Factor. T* versus degree of dissipation (Teh and Houlsby (1991))

Degree of Dissipation (%)	20	30	40	50	60	70	80
T* (u_2)	0.038	0.078	0.142	0.245	0.439	0.804	1.60

The coefficient of consolidation is typically analyzed using the time (t_{50}) corresponding to a degree of dissipation of 50% (u_{50}). In order to determine t_{50}, dissipation tests must be taken to a pressure less than u_{50}. The u_{50} value is half way between the initial maximum pore pressure and the equilibrium pore pressure value, known as u_{100}. To estimate u_{50}, both the initial maximum pore pressure and u_{100} must be known or estimated. Other degrees of dissipations may be considered, particularly for extremely long dissipations.

At any specific degree of dissipation the equilibrium pore pressure (u at t_{100}) must be estimated at the depth of interest. The equilibrium value may be determined from one or more sources such as measuring the value directly (u_{100}), estimating it from other dissipations in the same profile, estimating the phreatic surface and assuming hydrostatic conditions, from nearby soundings, from client provided information, from site observations and/or past experience, or from other site instrumentation.

For calculations of c_h (Teh and Houlsby (1991)), t_{50} values are estimated from the corresponding pore pressure dissipation curve and a rigidity index (I_r) is assumed. For curves having an initial dilatory response in which an initial rise in pore pressure occurs before reaching a peak, the relative time from the peak value is used in determining t_{50}. In cases where the time to peak is excessive, t_{50} values are not calculated.

Due to possible inherent uncertainties in estimating I_r, the equilibrium pore pressure and the effect of an initial dilatory response on calculating t_{50}, other methods should be applied to confirm the results for c_h.

Additional published methods for estimating the coefficient of consolidation from a piezocone test are described in Burns and Mayne (1998, 2002), Jones and Van Zyl (1981), Robertson et al. (1992) and Sully et al. (1999).

A summary of the pore pressure dissipation tests and dissipation plots are presented in the relevant appendix.

ASTM D5778-12, 2012, "Standard Test Method for Performing Electronic Friction Cone and Piezocone Penetration Testing of Soils", ASTM, West Conshohocken, US.

ASTM D7400-14, 2014, "Standard Test Methods for Downhole Seismic Testing", ASTM, West Conshohocken, US.

Burns, S.E. and Mayne, P.W., 1998, "Monotonic and dilatory pore pressure decay during piezocone tests", Canadian Geotechnical Journal 26 (4): 1063-1073.

Burns, S.E. and Mayne, P.W., 2002, "Analytical cavity expansion-critical state model cone dissipation in fine-grained soils", Soils & Foundations, Vol. 42(2): 131-137.

Crow, H.L., Hunter, J.A., Bobrowsky, P.T., 2012, "National shear wave measurement guidelines for Canadian seismic site assessment", GeoManitoba 2012, Sept 30 to Oct 2, Winnipeg, Manitoba.

Jones, G.A. and Van Zyl, D.J.A., 1981, "The piezometer probe: a useful investigation tool", Proceedings, 10th International Conference on Soil Mechanics and Foundation Engineering, Vol. 3, Stockholm: 489-495.

Lunne, T., Robertson, P.K. and Powell, J. J. M., 1997, "Cone Penetration Testing in Geotechnical Practice", Blackie Academic and Professional.

Mayne, P.W., 2013, "Evaluating yield stress of soils from laboratory consolidation and in-situ cone penetration tests", Sound Geotechnical Research to Practice (Holtz Volume) GSP 230, ASCE, Reston/VA: 406-420.

Mayne, P.W., 2014, "Interpretation of geotechnical parameters from seismic piezocone tests", CPT'14 Keynote Address, Las Vegas, NV, May 2014.

Mayne, P.W. and Peuchen, J., 2012, "Unit weight trends with cone resistance in soft to firm clays", Geotechnical and Geophysical Site Characterization *4,* Vol. 1 (Proc. ISC-4, Pernambuco), CRC Press, London: 903-910.

Robertson, P.K., Campanella, R.G., Gillespie, D. and Greig, J., 1986, "Use of Piezometer Cone Data", Proceedings of InSitu 86, ASCE Specialty Conference, Blacksburg, Virginia.

Robertson, P.K., 1990, "Soil Classification Using the Cone Penetration Test", Canadian Geotechnical Journal, Volume 27: 151-158.

Robertson, P.K., 2009, "Interpretation of cone penetration tests – a unified approach", Canadian Geotechnical Journal, Volume 46: 1337-1355.

Robertson, P.K., Campanella, R.G., Gillespie D and Rice, A., 1986, "Seismic CPT to Measure In-Situ Shear Wave Velocity", Journal of Geotechnical Engineering ASCE, Vol. 112, No. 8: 791-803.

Robertson, P.K., Sully, J.P., Woeller, D.J., Lunne, T., Powell, J.J.M. and Gillespie, D.G., 1992, "Estimating coefficient of consolidation from piezocone tests", Canadian Geotechnical Journal, 29(4): 551-557.

Sully, J.P., Robertson, P.K., Campanella, R.G. and Woeller, D.J., 1999, "An approach to evaluation of field CPTU dissipation data in overconsolidated fine-grained soils", Canadian Geotechnical Journal, 36(2): 369-381.

Teh, C.I., and Houlsby, G.T., 1991, "An analytical study of the cone penetration test in clay", Geotechnique, 41(1): 17-34.

The appendices listed below are included in the report:

- Cone Penetration Test Summary and Standard Cone Penetration Test Plots
- Standard Cone Penetration Test – Expanded Range Plots
- Advanced Cone Penetration Test Plots with Ic, Su(Nkt), Phi and $N_{1(60)}$(Ic RW1998)
- Seismic Cone Penetration Test (Vs) Tabular Results
- Seismic Cone Penetration Test (Vs) Plots
- Seismic Cone Penetration Test Time Domain Traces (Vs)
- Soil Behaviour Type (SBT) Scatter Plots
- Pore Pressure Dissipation Summary and Pore Pressure Dissipation Plots

Cone Penetration Test Summary and Standard Cone Penetration Test Plots

Job No:	20-05-21040
Client:	Golder Associates
Project:	100 Bayshore Drive
Start Date:	03-Jul-2020
End Date:	03-Jul-2020

CONE PENETRATION TEST SUMMARY

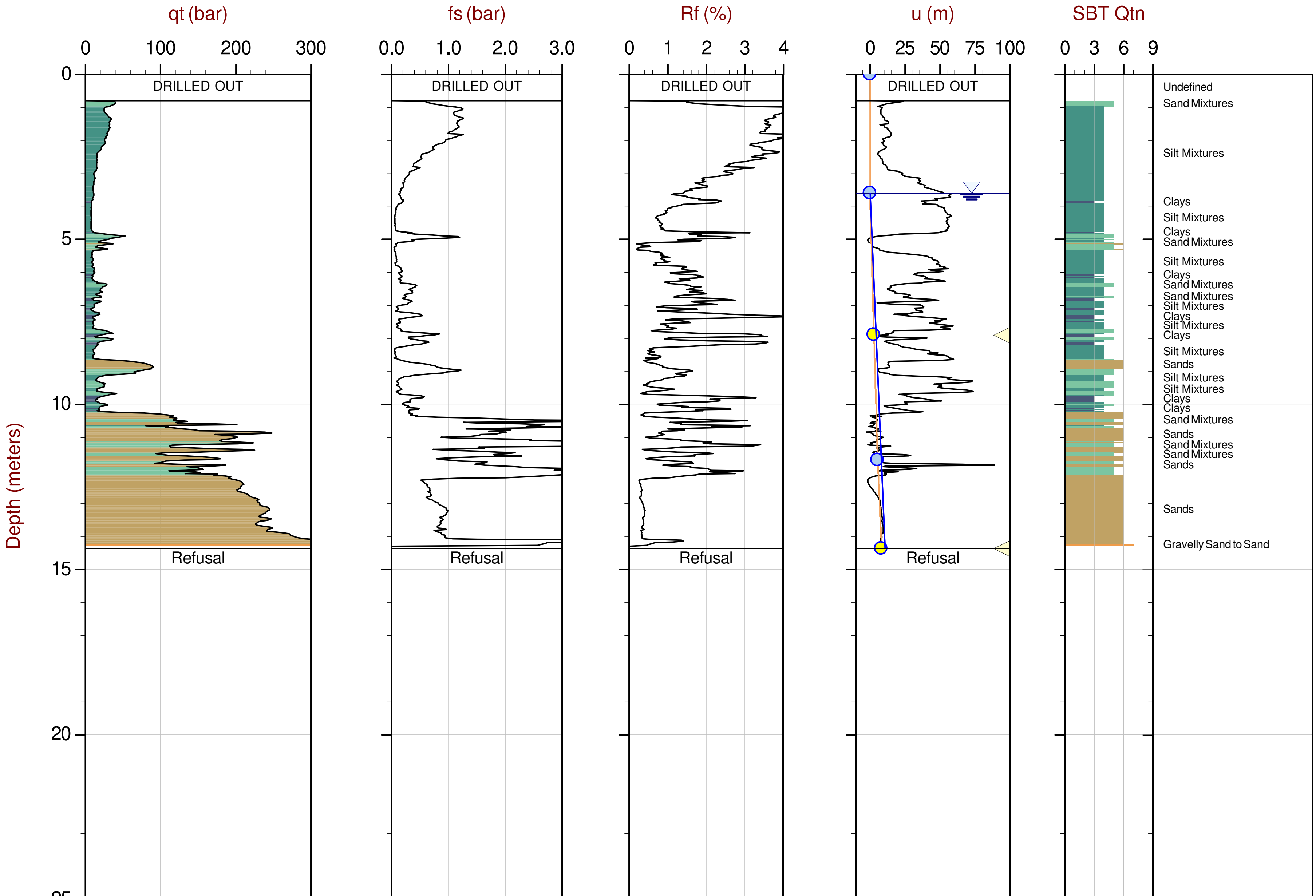
Sounding ID	File Name	Date	Cone	Assumed Phreatic Surface[1] (m)	Final Depth (m)	Northing[2] (m)	Easting[2] (m)	Refer to Notation Number
SCPT20-01	20-05-21040_SP01	3-Jul-2020	531:T1500F15U500	3.6	14.375	5021742.4	436565.1	3
SCPT20-01B	20-05-21040_SP01B	3-Jul-2020	531:T1500F15U500	3.6	11.700	5021737.0	436567.0	3, 4
SCPT20-02	20-05-21040_SP02	3-Jul-2020	545:T1500F15U500	4.4	24.525	5021717.6	436516.0	3

1. The assumed phreatic surface was based on pore pressure dissipations unless otherwise noted.
   Equilibrium pore pressure profiles were used for the calculated parameters.
2. Coordinates were provided by the client with datum: WGS84/UTM 18 North.
3. The assumed phreatic surface was based on client-provided monitoring well readings.
4. No seismic data collected as the test was refused at a shallower depth than SCPT20-01

Job No: 20-05-21040
Date: 2020-07-03 12:47
Site: Bayshore Mall Ottawa

Sounding: SCPT20-01
Cone: 531:T1500F15U500

Golder Associates



Max Depth: 14.375 m / 47.16 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 20-05-21040_SP01.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM Zone 18 N: 5021742.4m E: 436565.1m
Sheet No: 1 of 1

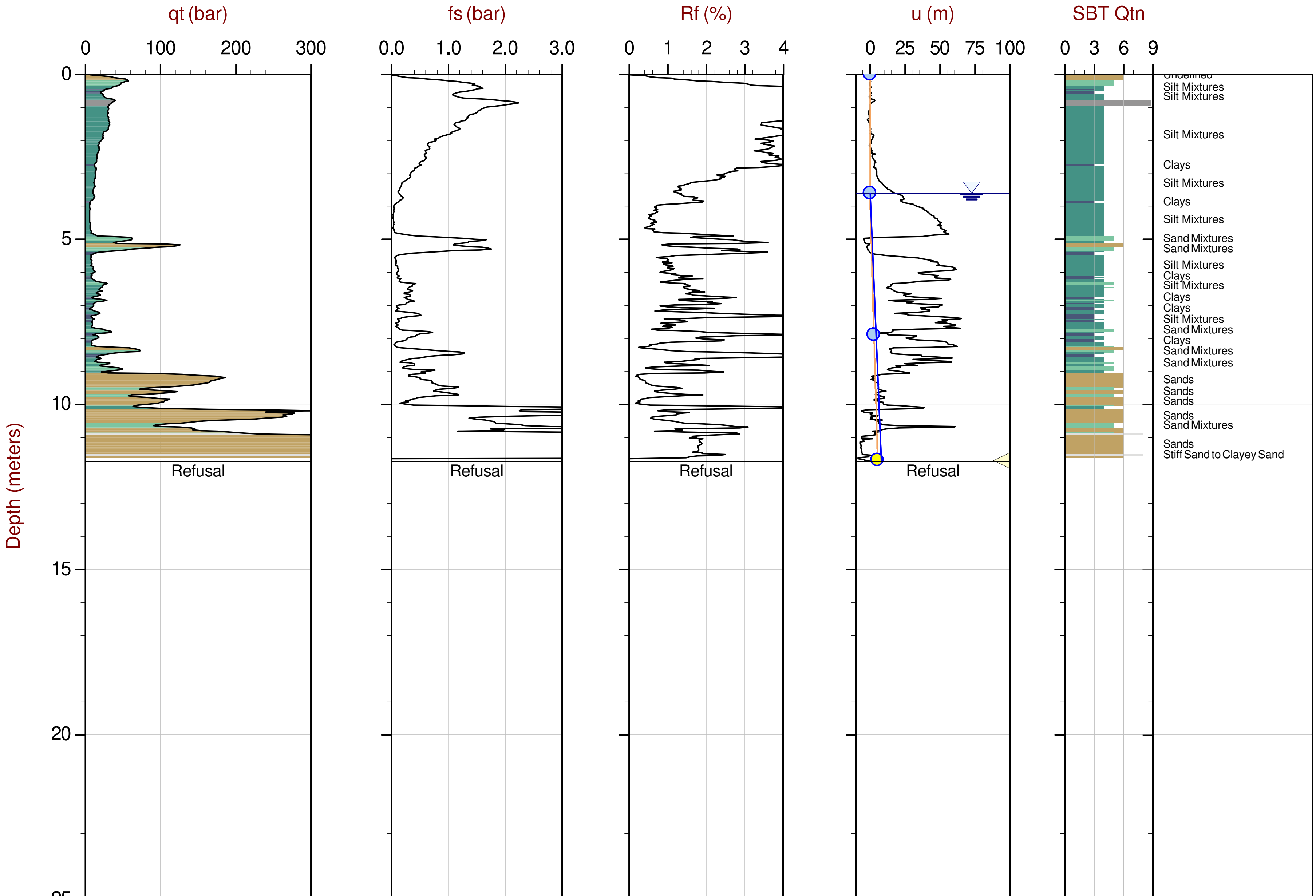
Overplot Item: Assumed Ueq; Ueq; Dissipation, equilibrium achieved; Dissipation, equilibrium not achieved; Hydrostatic Line; Equilibrium Profile

CONETEC

Golder Associates

Job No: 20-05-21040
Date: 2020-07-03 13:45
Site: Bayshore Mall Ottawa

Sounding: SCPT20-01B
Cone: 531:T1500F15U500



Max Depth: 11.725 m / 38.47 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 20-05-21040_SP01B.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM Zone 18 N: 5021737.0m E: 436567.3m
Sheet No: 1 of 1

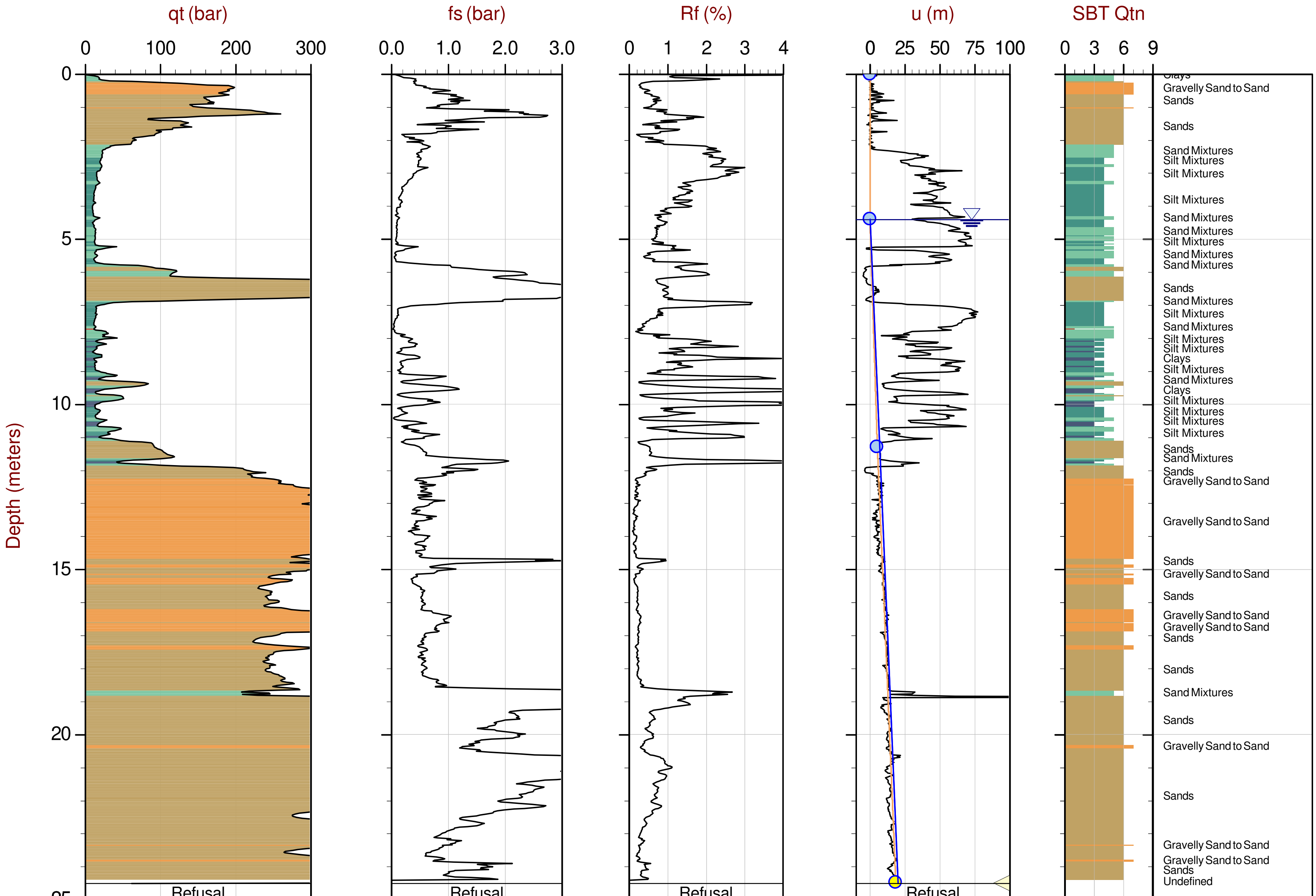
Overplot Item: Assumed Ueq; Ueq; Dissipation, equilibrium achieved; Dissipation, equilibrium not achieved; Hydrostatic Line; Equilibrium Profile

CONETEC

Golder Associates

Job No: 20-05-21040
Date: 2020-07-03 14:32
Site: Bayshore Mall Ottawa

Sounding: SCPT20-02
Cone: 545:T1500F15U500



Max Depth: 24.525 m / 80.46 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 20-05-21040_SP02.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM Zone 18 N: 5021717.6m E: 436516.0m
Sheet No: 1 of 1

Overplot Item: Assumed Ueq; Ueq; Dissipation, equilibrium achieved; Dissipation, equilibrium not achieved; Hydrostatic Line; Equilibrium Profile

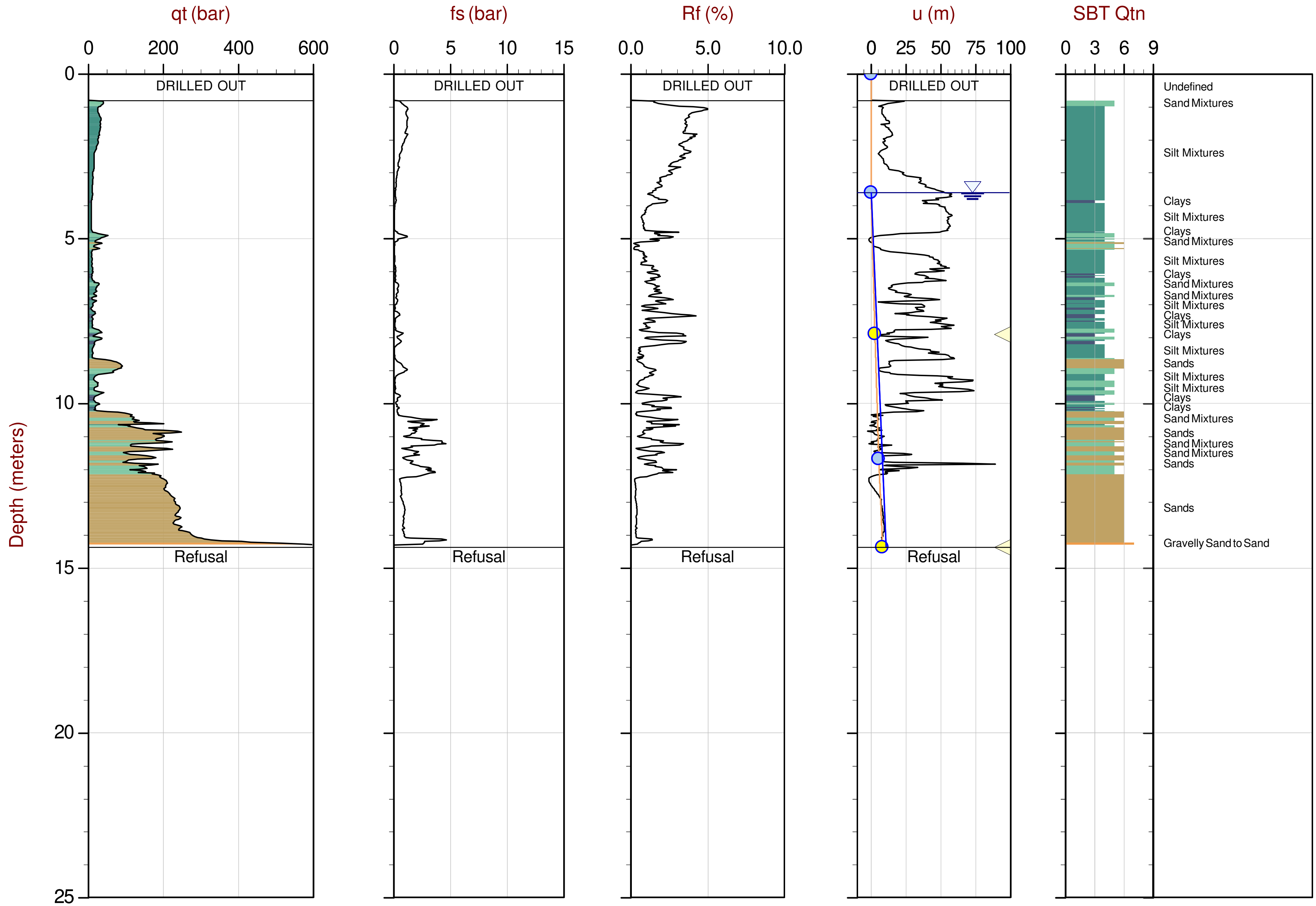
Standard Cone Penetration Test Plots - Expanded Range

CONETEC

Golder Associates

Job No: 20-05-21040
Date: 2020-07-03 12:47
Site: Bayshore Mall Ottawa

Sounding: SCPT20-01
Cone: 531:T1500F15U500



Max Depth: 14.375 m / 47.16 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 20-05-21040_SP01.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM Zone 18 N: 5021742.4m E: 436565.1m
Sheet No: 1 of 1

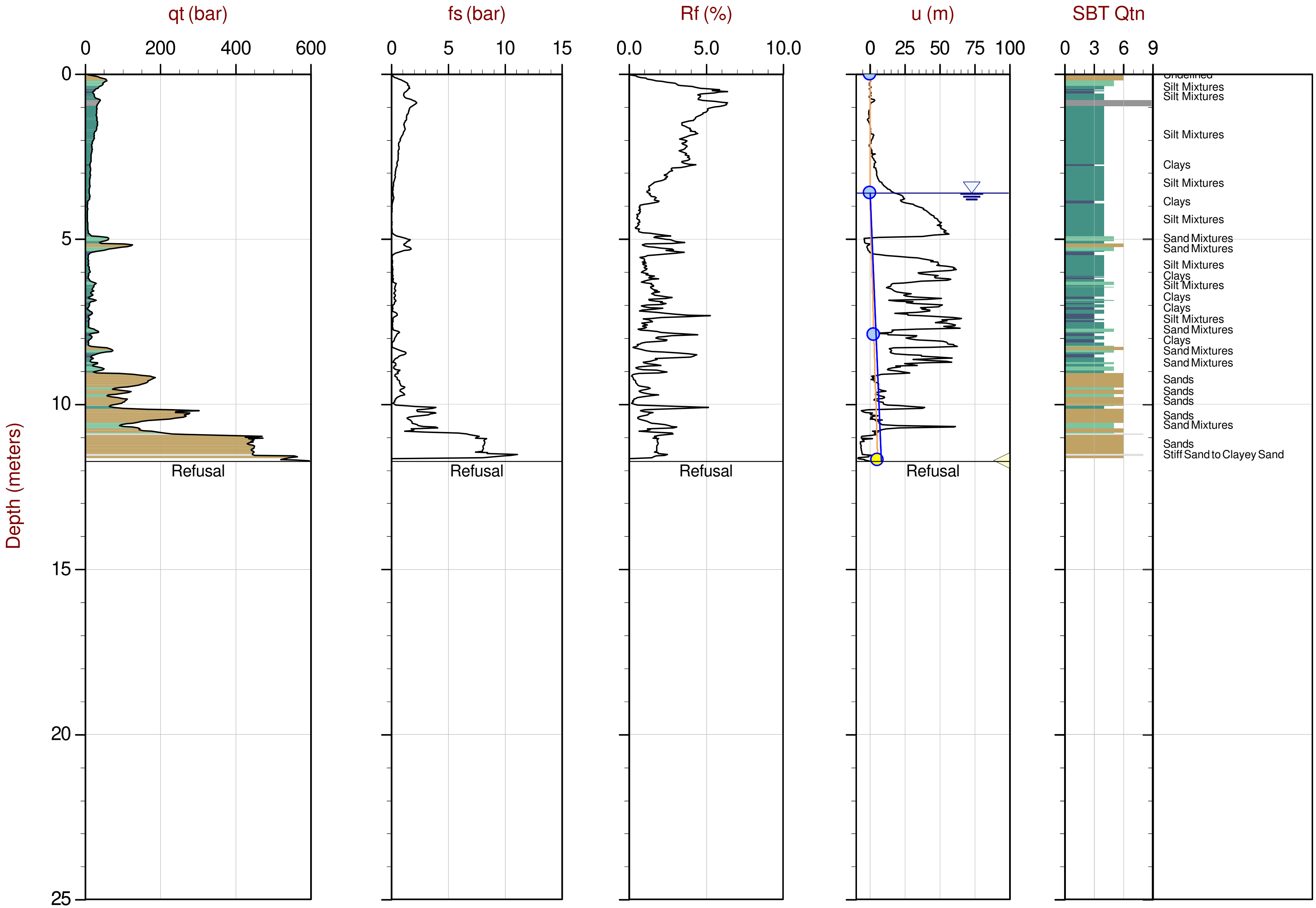
Overplot Item: Assumed Ueq; Ueq; Dissipation, equilibrium achieved; Dissipation, equilibrium not achieved; Hydrostatic Line; Equilibrium Profile

ConeTec

Golder Associates

Job No: 20-05-21040
Date: 2020-07-03 13:45
Site: Bayshore Mall Ottawa

Sounding: SCPT20-01B
Cone: 531:T1500F15U500



Max Depth: 11.725 m / 38.47 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 20-05-21040_SP01B.COR
Unit Wt: SBTQtn (PKR2009)

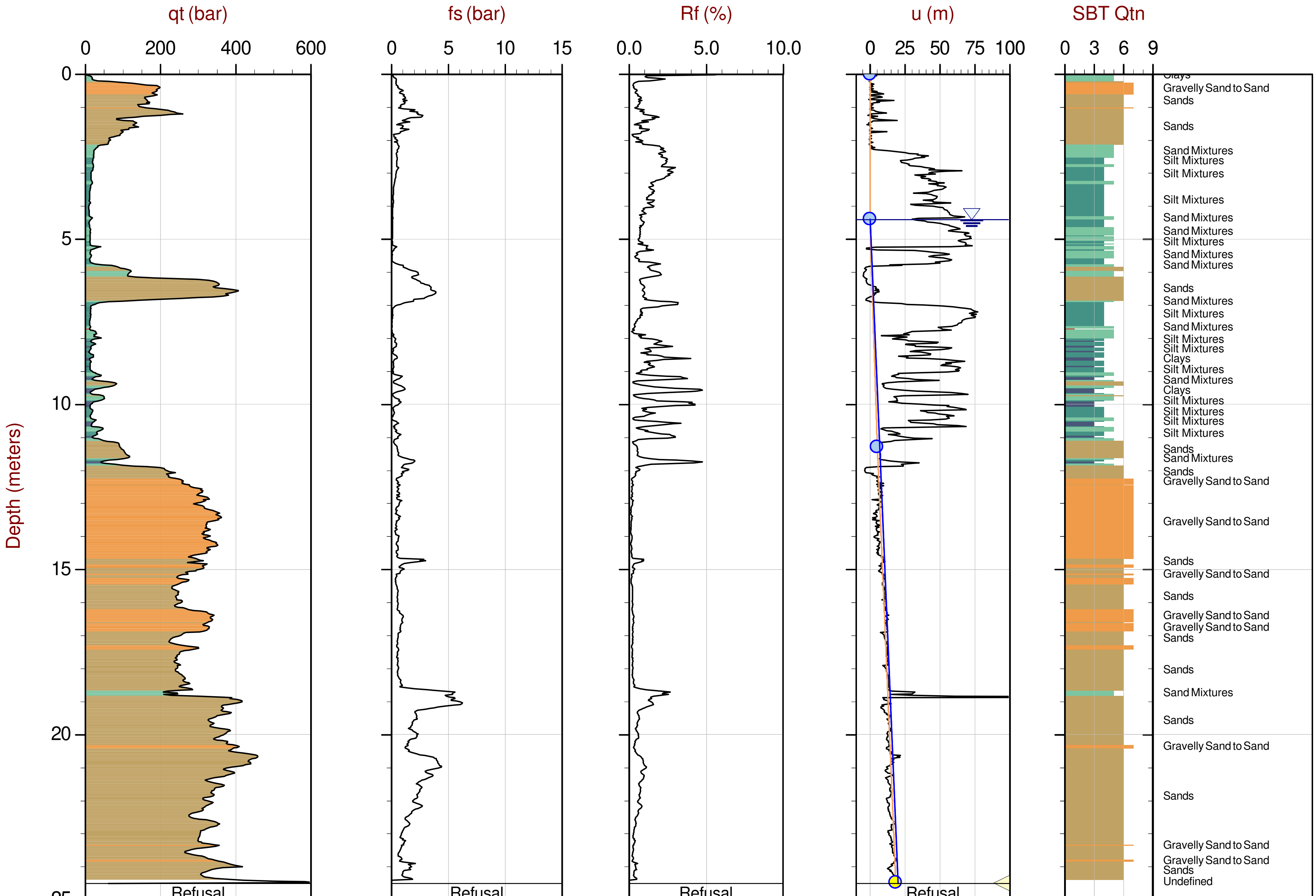
SBT: Robertson, 2009 and 2010
Coords: UTM Zone 18 N: 5021737.0m E: 436567.3m
Sheet No: 1 of 1

Overplot Item: Assumed Ueq; Ueq; Dissipation, equilibrium achieved; Dissipation, equilibrium not achieved; Hydrostatic Line; Equilibrium Profile

Golder Associates

Job No: 20-05-21040
Date: 2020-07-03 14:32
Site: Bayshore Mall Ottawa

Sounding: SCPT20-02
Cone: 545:T1500F15U500



Max Depth: 24.525 m / 80.46 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 20-05-21040_SP02.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM Zone 18 N: 5021717.6m E: 436516.0m
Sheet No: 1 of 1

Overplot Item: Assumed Ueq; Ueq; Dissipation, equilibrium achieved; Dissipation, equilibrium not achieved; Hydrostatic Line; Equilibrium Profile

Advanced Cone Penetration Plots with Ic, Su(Nkt), Phi and N1(60)Ic

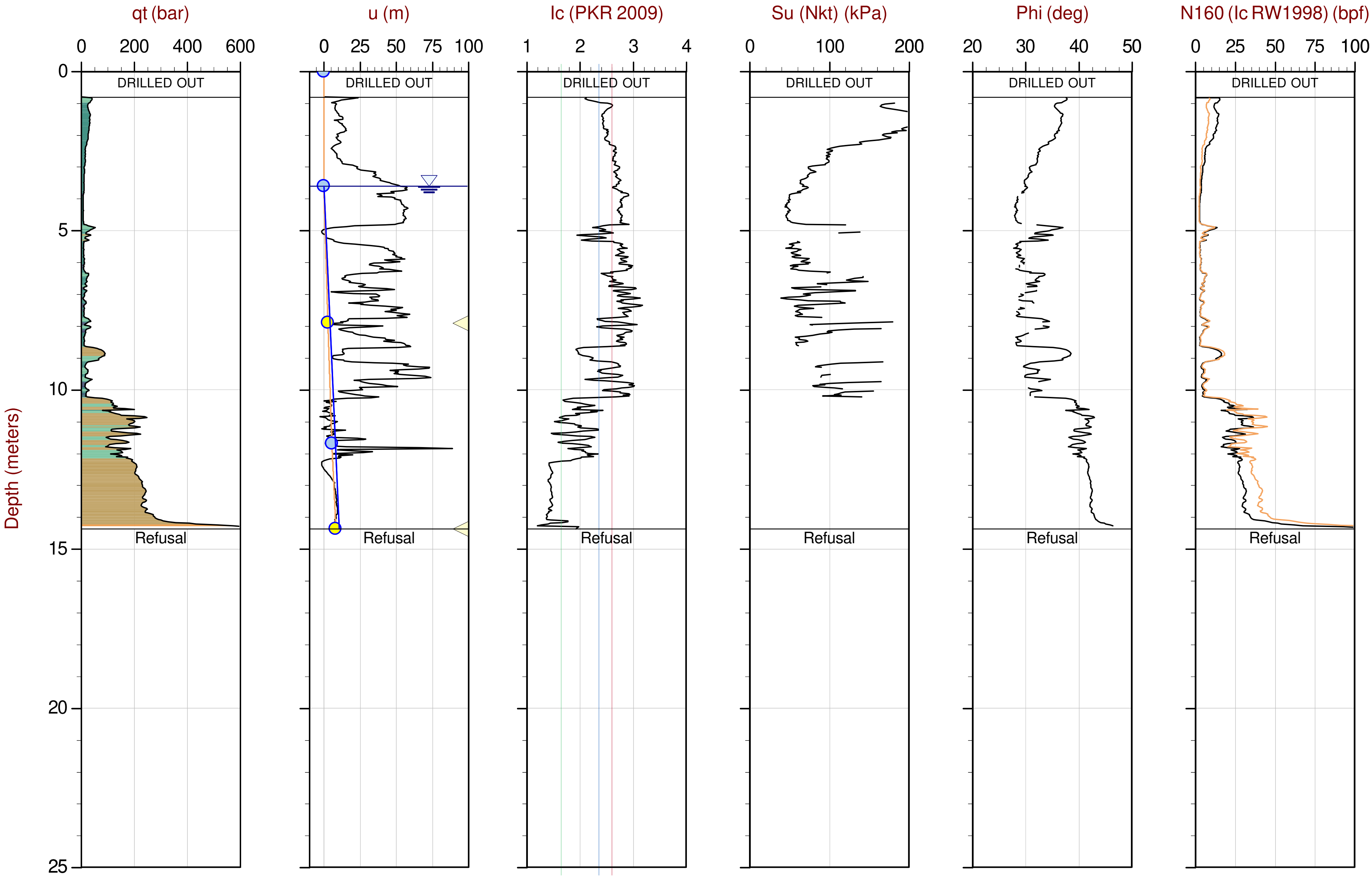
ConeTec

CONETEC

Golder Associates

Job No: 20-05-21040
Date: 2020-07-03 12:47
Site: Bayshore Mall Ottawa

Sounding: SCPT20-01
Cone: 531:T1500F15U500



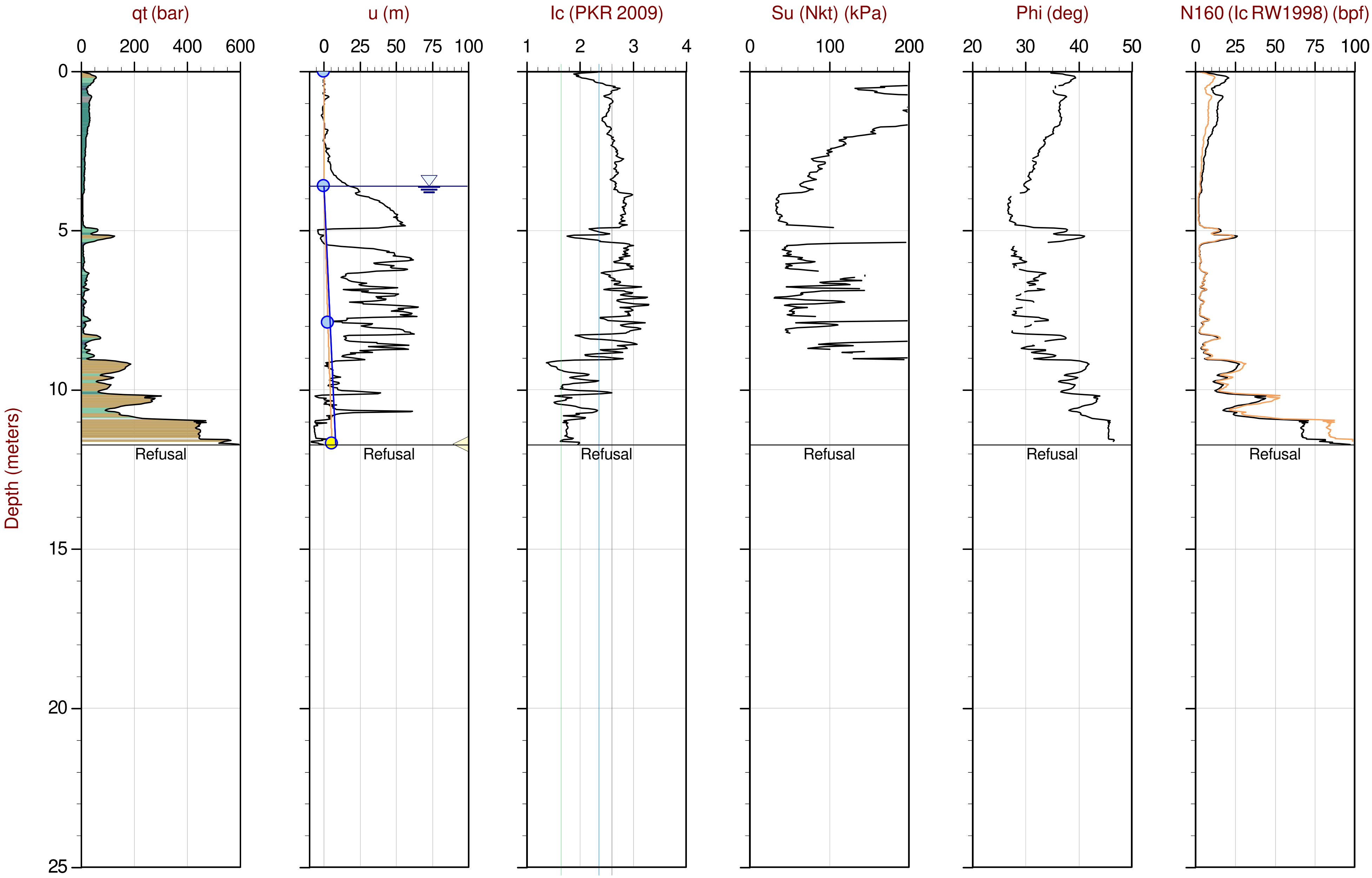
Max Depth: 14.375 m / 47.16 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 20-05-21040_SP01.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0

SBT: Robertson, 2009 and 2010
Coords: UTM Zone 18 N: 5021742.4m E: 436565.1m
Sheet No: 1 of 1

N60

Overplot Item: Assumed Ueq; Ueq; Dissipation, equilibrium achieved; Dissipation, equilibrium not achieved; Hydrostatic Line; Equilibrium Profile



Max Depth: 11.725 m / 38.47 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point
Overplot Item: Assumed Ueq; Ueq; Dissipation, equilibrium achieved; Dissipation, equilibrium not achieved; Hydrostatic Line; Equilibrium Profile

File: 20-05-21040_SP01B.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0

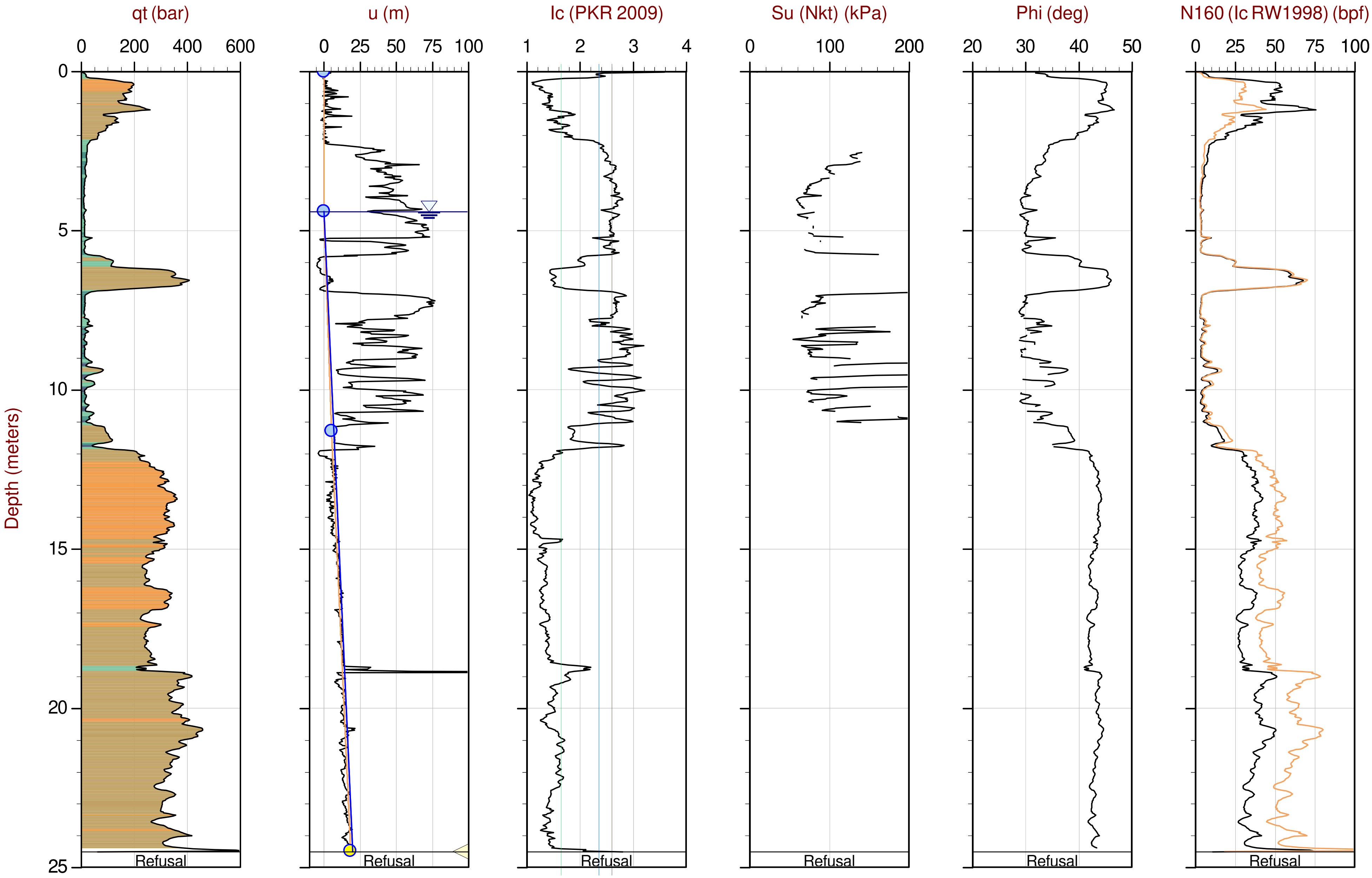
SBT: Robertson, 2009 and 2010
Coords: UTM Zone 18 N: 5021737.0m E: 436567.3m
Sheet No: 1 of 1

CONETEC

Golder Associates

Job No: 20-05-21040
Date: 2020-07-03 14:32
Site: Bayshore Mall Ottawa

Sounding: SCPT20-02
Cone: 545:T1500F15U500



Max Depth: 24.525 m / 80.46 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 20-05-21040_SP02.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0

SBT: Robertson, 2009 and 2010
Coords: UTM Zone 18 N: 5021717.6m E: 436516.0m
Sheet No: 1 of 1

Overplot Item: Assumed Ueq; Ueq; Dissipation, equilibrium achieved; Dissipation, equilibrium not achieved; Hydrostatic Line; Equilibrium Profile

Seismic Cone Penetration Test Shear Wave (Vs) Tabular Results

Job No:	20-05-21040
Client:	Golder Associates
Project:	100 Bayshore Drive
Sounding ID:	SCPT20-01
Date:	07:03:20 12:47
Seismic Source:	Beam
Seismic Offset (m):	0.55
Source Depth (m):	0.00
Geophone Offset (m):	0.20

SCPTu SHEAR WAVE VELOCITY TEST RESULTS - Vs

Tip Depth (m)	Geophone Depth (m)	Ray Path (m)	Ray Path Difference (m)	Travel Time Interval (ms)	Interval Velocity (m/s)
0.90	0.70	0.89			
1.90	1.70	1.79	0.90	5.84	154
2.90	2.70	2.76	0.97	5.15	188
3.90	3.70	3.74	0.99	5.12	193
4.90	4.70	4.73	0.99	5.60	177
5.90	5.70	5.73	0.99	5.66	176
6.90	6.70	6.72	1.00	5.28	189
7.90	7.70	7.72	1.00	4.72	211
8.90	8.70	8.72	1.00	4.71	212
9.90	9.70	9.72	1.00	4.51	221
10.90	10.70	10.71	1.00	4.02	249
11.90	11.70	11.71	1.00	2.76	362
12.90	12.70	12.71	1.00	3.08	324
14.38	14.18	14.19	1.48	4.77	310

Job No:	20-05-21040
Client:	Golder Associates
Project:	100 Bayshore Drive
Sounding ID:	SCPT20-02
Date:	07:03:20 14:32

Seismic Source:	Beam
Seismic Offset (m):	0.55
Source Depth (m):	0.00
Geophone Offset (m):	0.20

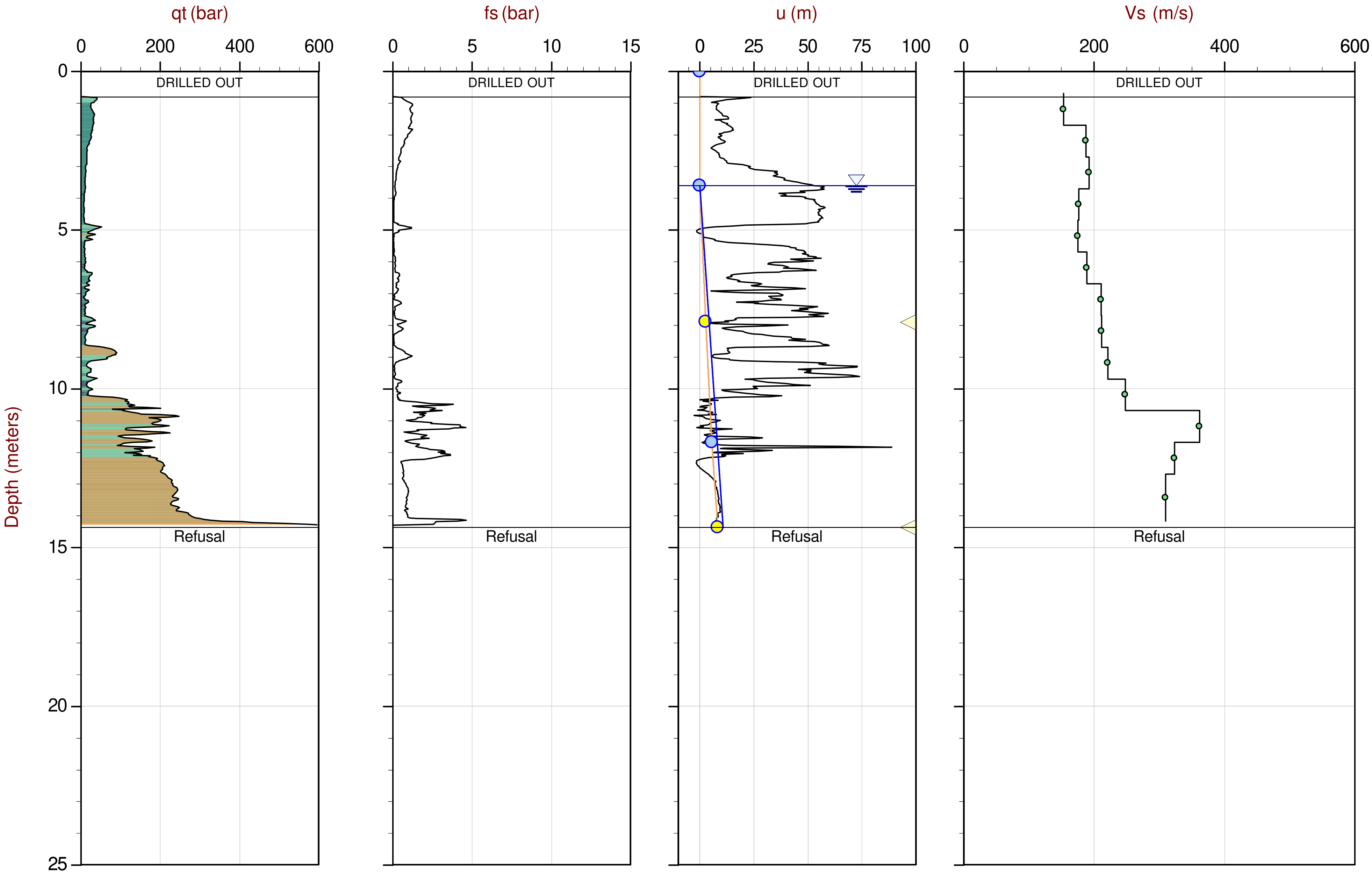
SCPTu SHEAR WAVE VELOCITY TEST RESULTS - Vs

Tip Depth (m)	Geophone Depth (m)	Ray Path (m)	Ray Path Difference (m)	Travel Time Interval (ms)	Interval Velocity (m/s)
0.90	0.70	0.89			
1.90	1.70	1.79	0.90	3.87	232
2.90	2.70	2.76	0.97	4.93	197
3.90	3.70	3.74	0.99	5.45	181
4.90	4.70	4.73	0.99	6.30	157
5.90	5.70	5.73	0.99	5.95	167
6.90	6.70	6.72	1.00	4.63	215
7.90	7.70	7.72	1.00	5.24	190
8.90	8.70	8.72	1.00	4.52	221
9.90	9.70	9.72	1.00	4.55	220
10.90	10.70	10.71	1.00	4.48	223
11.90	11.70	11.71	1.00	4.00	250
12.90	12.70	12.71	1.00	3.44	290
13.90	13.70	13.71	1.00	3.12	320
14.90	14.70	14.71	1.00	3.04	329
15.90	15.70	15.71	1.00	3.60	278
16.90	16.70	16.71	1.00	3.44	291
17.90	17.70	17.71	1.00	3.36	297
18.90	18.70	18.71	1.00	3.44	291
19.90	19.70	19.71	1.00	3.03	330
20.90	20.70	20.71	1.00	3.46	288
21.90	21.70	21.71	1.00	3.09	324
22.90	22.70	22.71	1.00	3.30	303
23.90	23.70	23.71	1.00	2.72	368
24.52	24.32	24.33	0.62	1.52	408

Seismic Cone Penetration Test Plots

Golder Associates

Job No: 20-05-21040
Date: 2020-07-03 12:47
Site: Bayshore Mall Ottawa
Sounding: SCPT20-01
Cone: 531:T1500F15U500



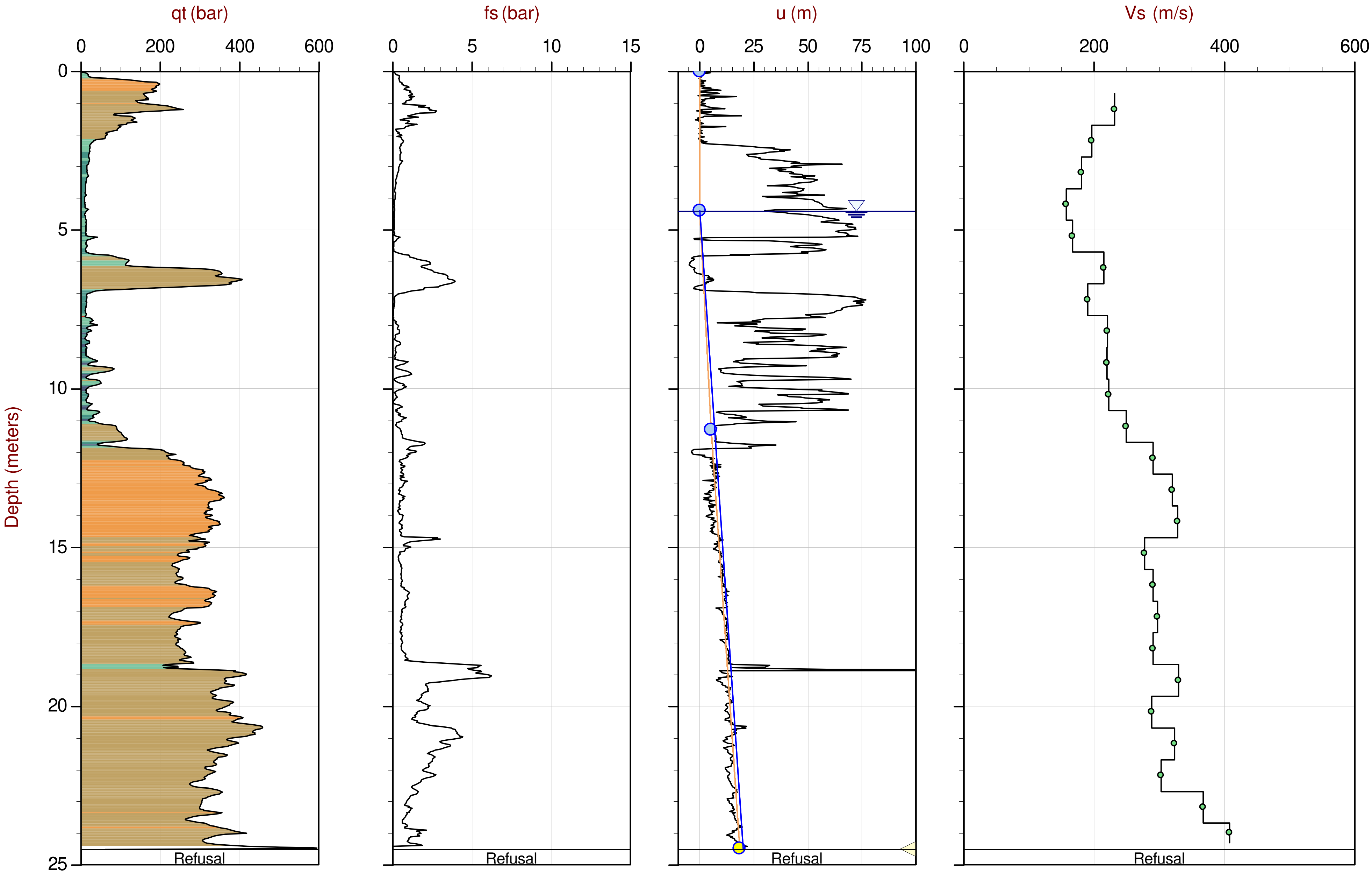
Max Depth: 14.375 m / 47.16 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 20-05-21040_SP01.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM Zone 18 N: 5021742.4m E: 436565.1m
Sheet No: 1 of 1

Overplot Item: Assumed Ueq; Ueq; Dissipation, equilibrium achieved; Dissipation, equilibrium not achieved; Hydrostatic Line; Equilibrium Profile

Golder Associates	Job No: 20-05-21040	Sounding: SCPT20-02
	Date: 2020-07-03 14:32	Cone: 545:T1500F15U500
	Site: Bayshore Mall Ottawa	



Max Depth: 24.525 m / 80.46 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 20-05-21040_SP02.COR
Unit Wt: SBTQtn (PKR2009)

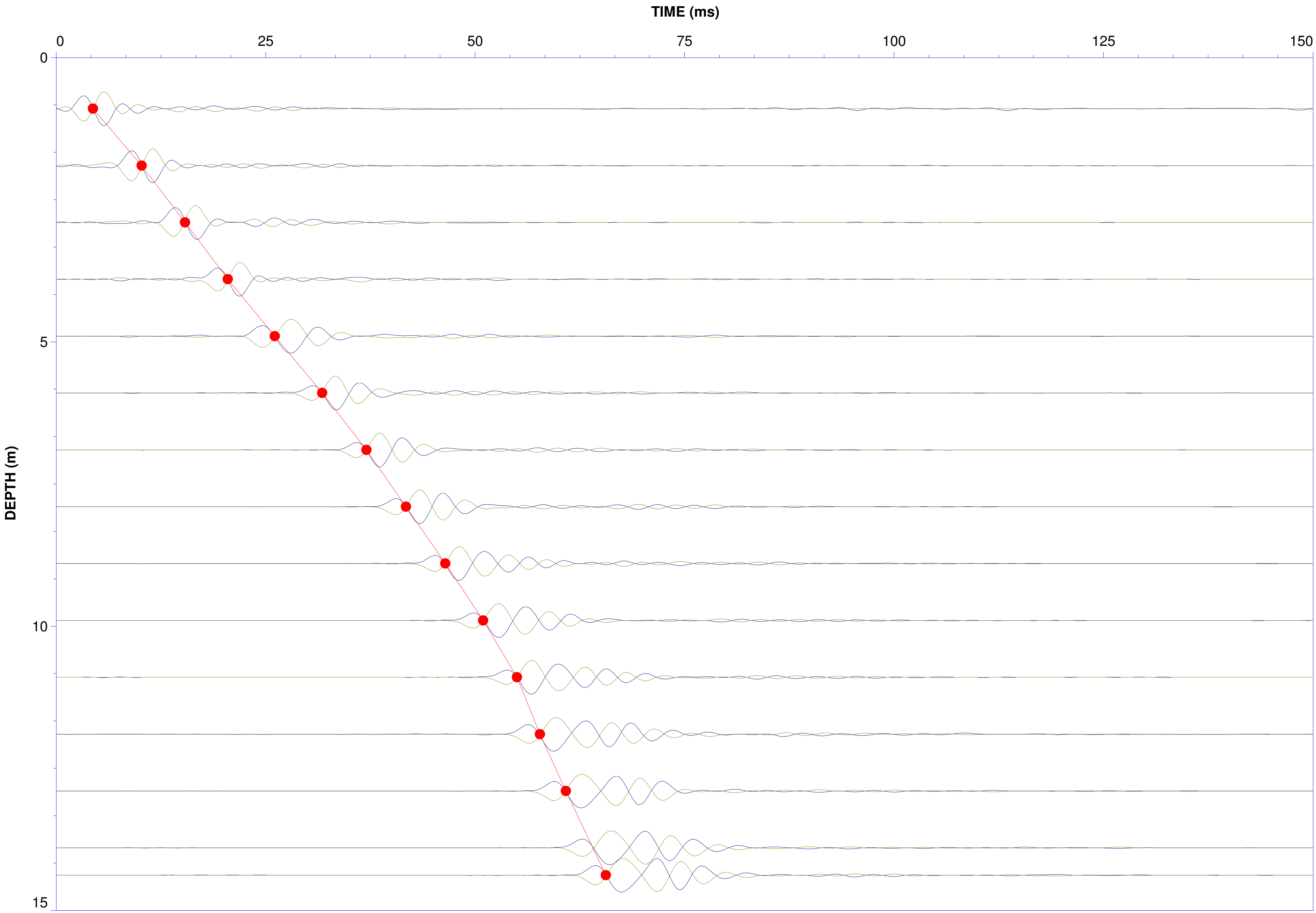
SBT: Robertson, 2009 and 2010
Coords: UTM Zone 18 N: 5021717.6m E: 436516.0m
Sheet No: 1 of 1

Overplot Item: Assumed Ueq; Ueq; Dissipation, equilibrium achieved; Dissipation, equilibrium not achieved; Hydrostatic Line; Equilibrium Profile

Seismic Cone Penetration Test Time DomainTraces (Vs)

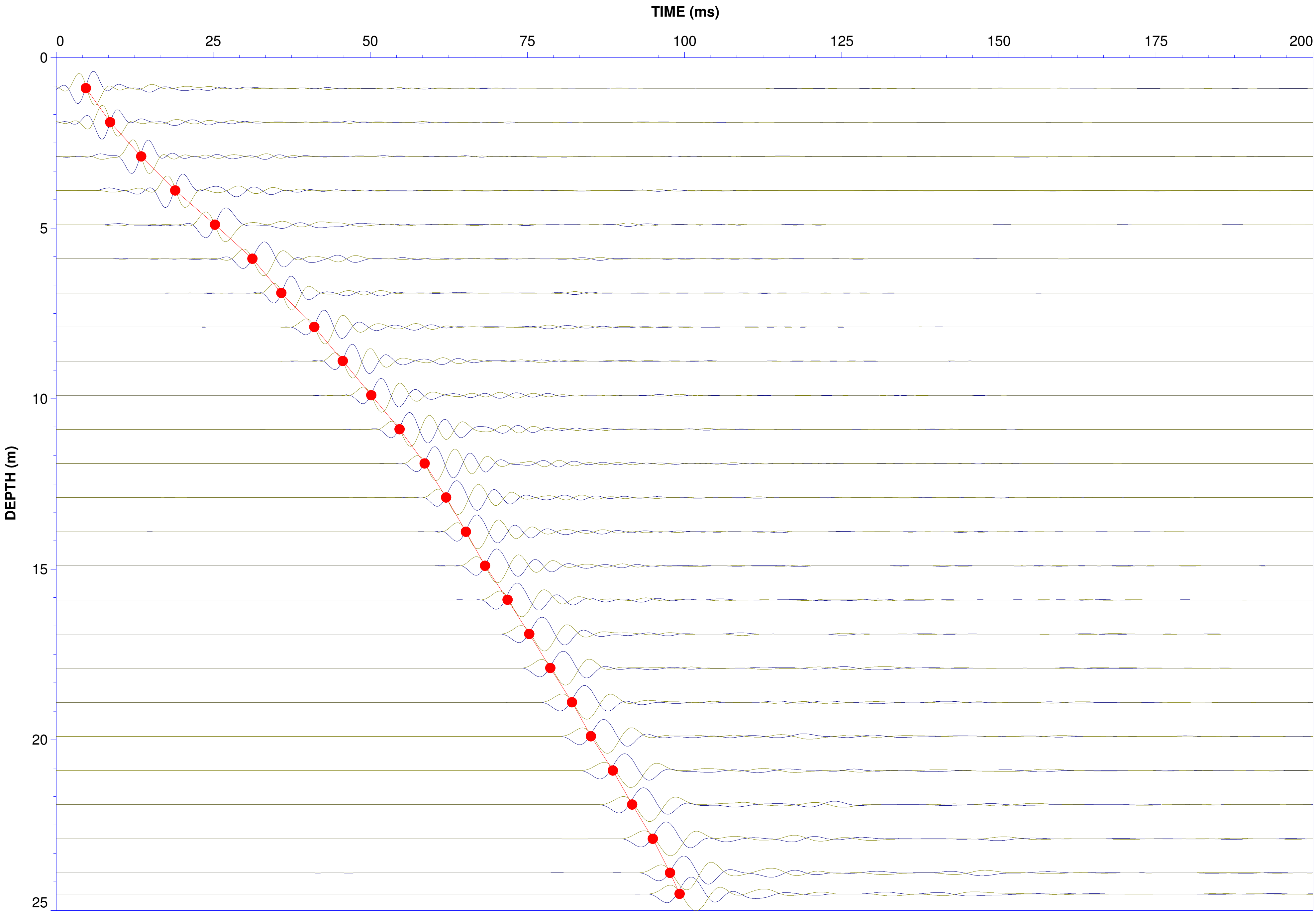
CONETEC

Job No: 20-05-21040 | Client: Golder Associates | Project: 100 Bayshore Drive | Filter: 0-300 Hz | Sounding: SCPT20-01 | Date: 07:03:20 12:47



ConeTec

Job No: 20-05-21040	Client: Golder Associates	Project: 100 Bayshore Drive	Filter: 0-300 Hz	Sounding: SCPT20-02	Date: 07:03:20 14:32



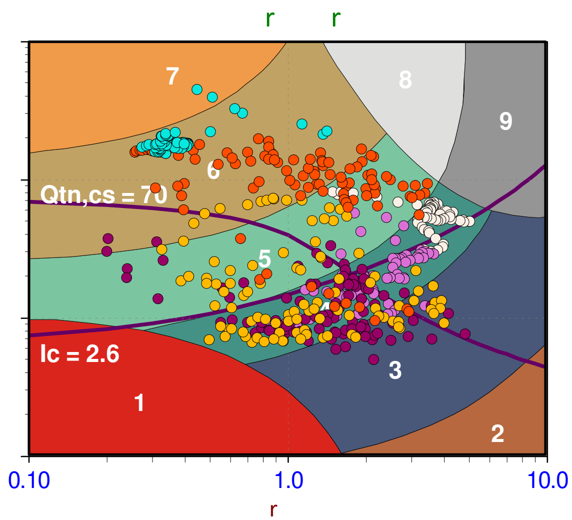
Soil Behaviour Type (SBT) Scatter Plots

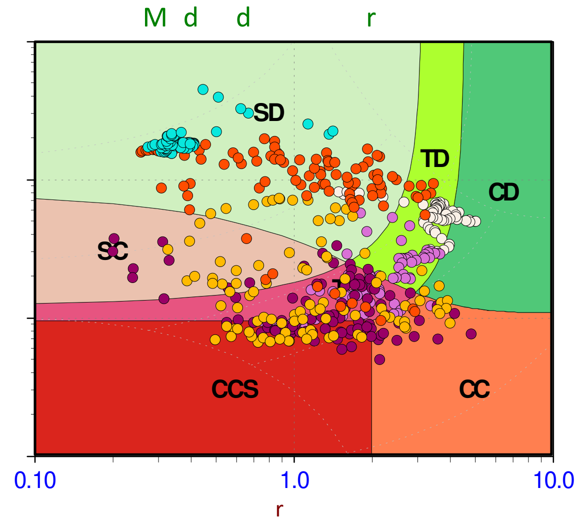
CONETEC

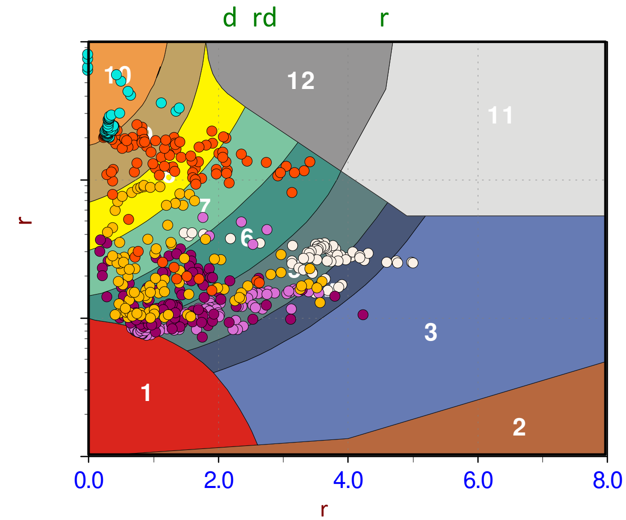
Golder Associates

Job No: 20-05-21040
Date: 2020-07-03 12:47
Site: Bayshore Mall Ottawa

Sounding: SCPT20-01
Cone: 531:T1500F15U500







Depth Ranges

- >0.0 to 2.5 m
- >2.5 to 5.0 m
- >5.0 to 7.5 m
- >7.5 to 10.0 m
- >10.0 to 12.5 m
- >12.5 to 15.0 m
- >15.0 to 17.5 m
- >17.5 to 20.0 m
- >20.0 to 22.5 m
- >22.5 to 25.0 m
- >25.0 m

Legend

- Sensitive, Fine Grained
- Organic Soils
- Clays
- Silt Mixtures
- Sand Mixtures
- Sands
- Gravelly Sand to Sand
- Stiff Sand to Clayey Sand
- Very Stiff Fine Grained

Legend

- CCS (Cont. sensitive clay like)
- CC (Cont. clay like)
- TC (Cont. transitional)
- SC (Cont. sand like)
- CD (Dil. clay like)
- TD (Dil. transitional)
- SD (Dil. sand like)

Legend

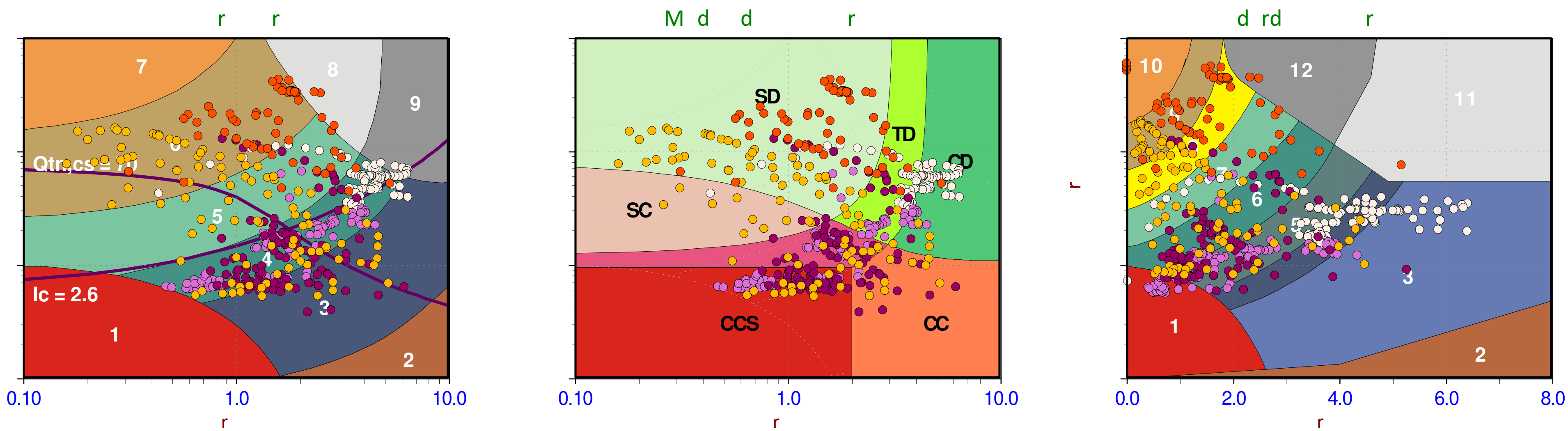
- Sensitive Fines
- Organic Soil
- Clay
- Silty Clay
- Clayey Silt
- Silt
- Sandy Silt
- Silty Sand/Sand
- Sand
- Gravelly Sand
- Stiff Fine Grained
- Cemented Sand

CONETEC

Golder Associates

Job No: 20-05-21040
Date: 2020-07-03 13:45
Site: Bayshore Mall Ottawa

Sounding: SCPT20-01B
Cone: 531:T1500F15U500



Depth Ranges
- >0.0 to 2.5 m
- >2.5 to 5.0 m
- >5.0 to 7.5 m
- >7.5 to 10.0 m
- >10.0 to 12.5 m
- >12.5 to 15.0 m
- >15.0 to 17.5 m
- >17.5 to 20.0 m
- >20.0 to 22.5 m
- >22.5 to 25.0 m
- >25.0 m

Legend
- Sensitive, Fine Grained
- Organic Soils
- Clays
- Silt Mixtures
- Sand Mixtures
- Sands
- Gravelly Sand to Sand
- Stiff Sand to Clayey Sand
- Very Stiff Fine Grained

Legend
- CCS (Cont. sensitive clay like)
- CC (Cont. clay like)
- TC (Cont. transitional)
- SC (Cont. sand like)
- CD (Dil. clay like)
- TD (Dil. transitional)
- SD (Dil. sand like)

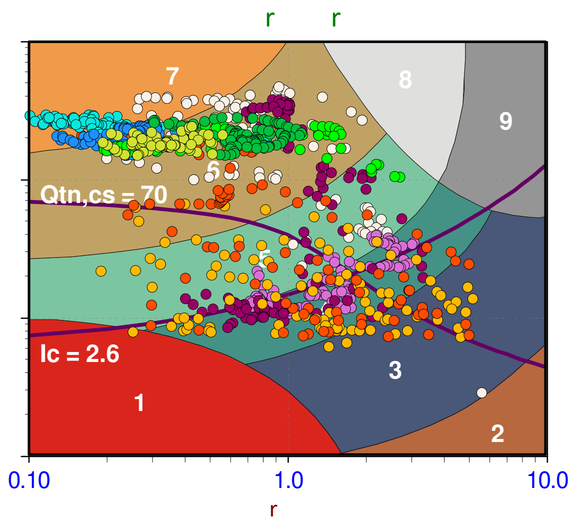
Legend
- Sensitive Fines
- Organic Soil
- Clay
- Silty Clay
- Clayey Silt
- Silt
- Sandy Silt
- Silty Sand/Sand
- Sand
- Gravelly Sand
- Stiff Fine Grained
- Cemented Sand

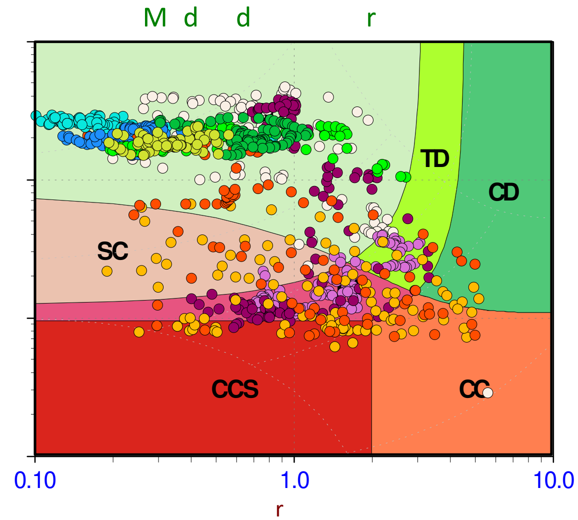
CONETEC

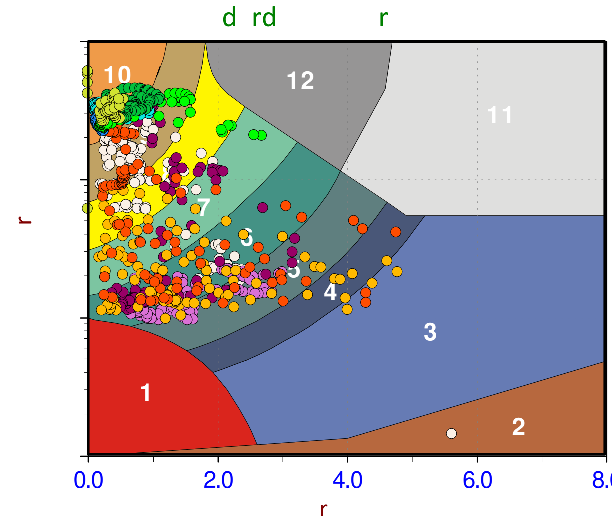
Golder Associates

Job No: 20-05-21040
Date: 2020-07-03 14:32
Site: Bayshore Mall Ottawa

Sounding: SCPT20-02
Cone: 545:T1500F15U500







Depth Ranges
- >0.0 to 2.5 m
- >2.5 to 5.0 m
- >5.0 to 7.5 m
- >7.5 to 10.0 m
- >10.0 to 12.5 m
- >12.5 to 15.0 m
- >15.0 to 17.5 m
- >17.5 to 20.0 m
- >20.0 to 22.5 m
- >22.5 to 25.0 m
- >25.0 m

Legend
- Sensitive, Fine Grained
- Organic Soils
- Clays
- Silt Mixtures
- Sand Mixtures
- Sands
- Gravelly Sand to Sand
- Stiff Sand to Clayey Sand
- Very Stiff Fine Grained

Legend
- CCS (Cont. sensitive clay like)
- CC (Cont. clay like)
- TC (Cont. transitional)
- SC (Cont. sand like)
- CD (Dil. clay like)
- TD (Dil. transitional)
- SD (Dil. sand like)

Legend
- Sensitive Fines
- Organic Soil
- Clay
- Silty Clay
- Clayey Silt
- Silt
- Sandy Silt
- Silty Sand/Sand
- Sand
- Gravelly Sand
- Stiff Fine Grained
- Cemented Sand

Pore Pressure Dissipation Summary and Pore Pressure Dissipation Plots

Job No:	20-05-21040
Client:	Golder Associates
Project:	100 Bayshore Drive
Start Date:	03-Jul-2020
End Date:	03-Jul-2020

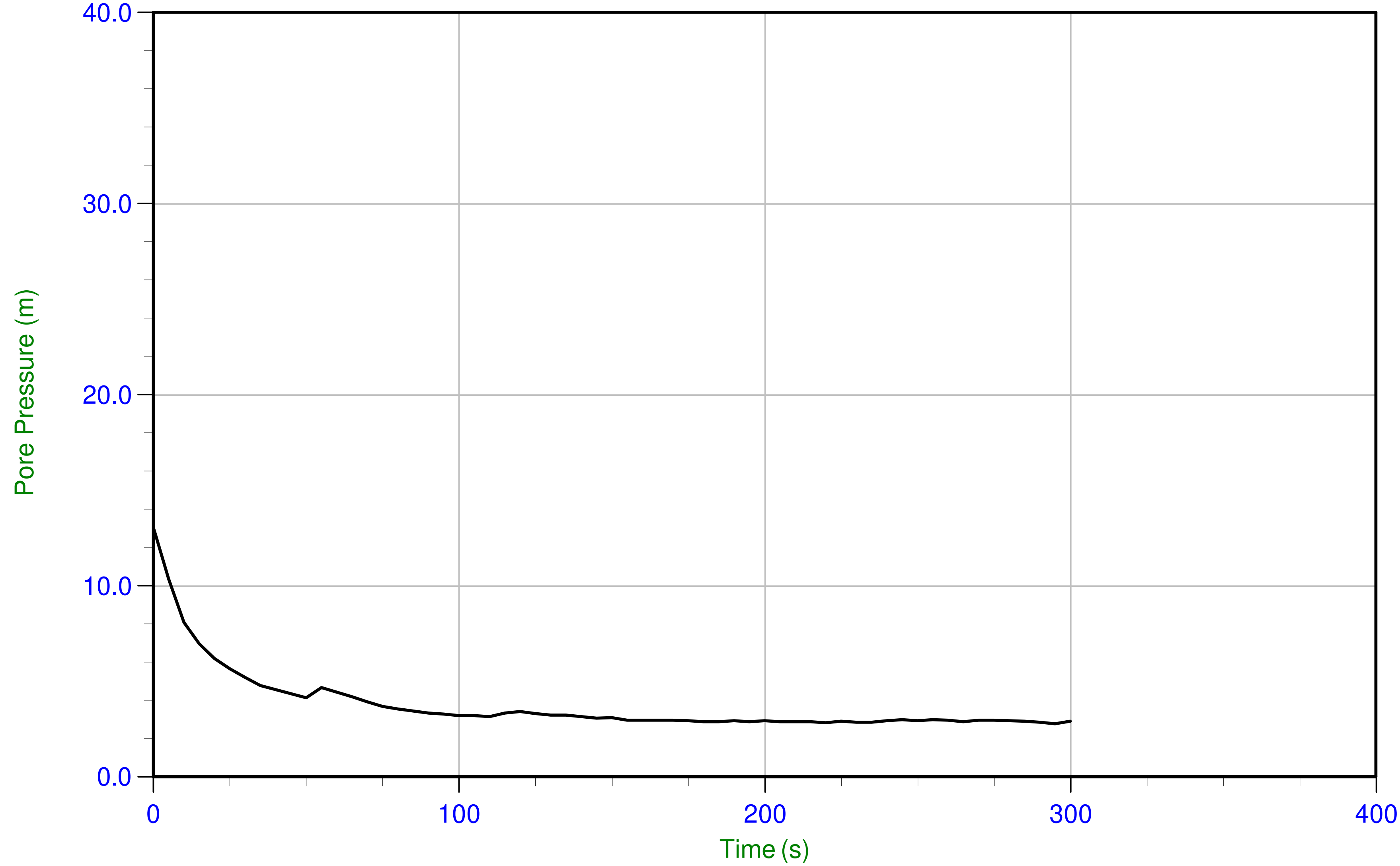
CPTu PORE PRESSURE DISSIPATION SUMMARY

Sounding ID	File Name	Cone Area (cm^2)	Duration (s)	Test Depth (m)	Equilibrium Pore Pressure U_{eq} (m)	Calculated Phreatic Surface (m)
SCPT20-01	20-05-21040_SP01	15	300	7.900	2.8	5.1
SCPT20-01	20-05-21040_SP01	15	300	14.375	8.3	6.0
SCPT20-01B	20-05-21040_SP01B	15	300	11.700	5.6	6.1
SCPT20-02	20-05-21040_SP02	15	305	24.500	18.5	6.0

ConeTec | Golder Associates

Job No: 20-05-21040
Date: 07/03/2020 12:47
Site: Bayshore Mall, Ottawa

Sounding: SCPT20-01
Cone: 531:T1500F15U500 Area=15 cm²



Trace Summary:

Filename: 20-05-21040_SP01.PPF
Depth: 7.900 m / 25.918 ft
Duration: 300.0 s

u Min: 2.8 m
u Max: 13.1 m
u Final: 2.9 m

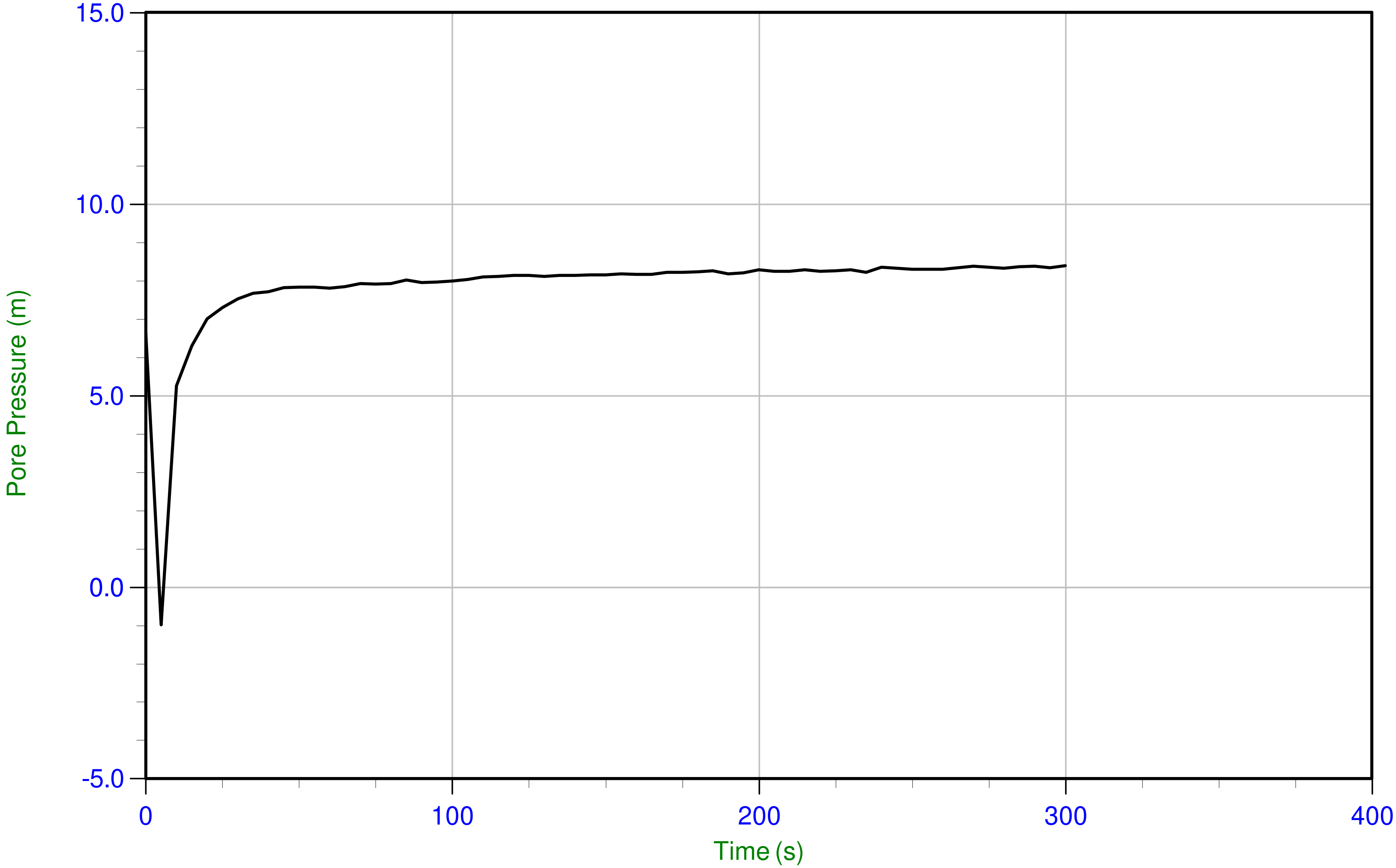
WT: 5.097 m / 16.724 ft
Ueq: 2.8 m

CONETEC

Golder Associates

Job No: 20-05-21040
Date: 07/03/2020 12:47
Site: Bayshore Mall, Ottawa

Sounding: SCPT20-01
Cone: 531:T1500F15U500 Area=15 cm²



Trace Summary:

Filename: 20-05-21040_SP01.PPF
Depth: 14.375 m / 47.162 ft
Duration: 300.0 s

u Min: -1.0 m
u Max: 8.4 m
u Final: 8.4 m

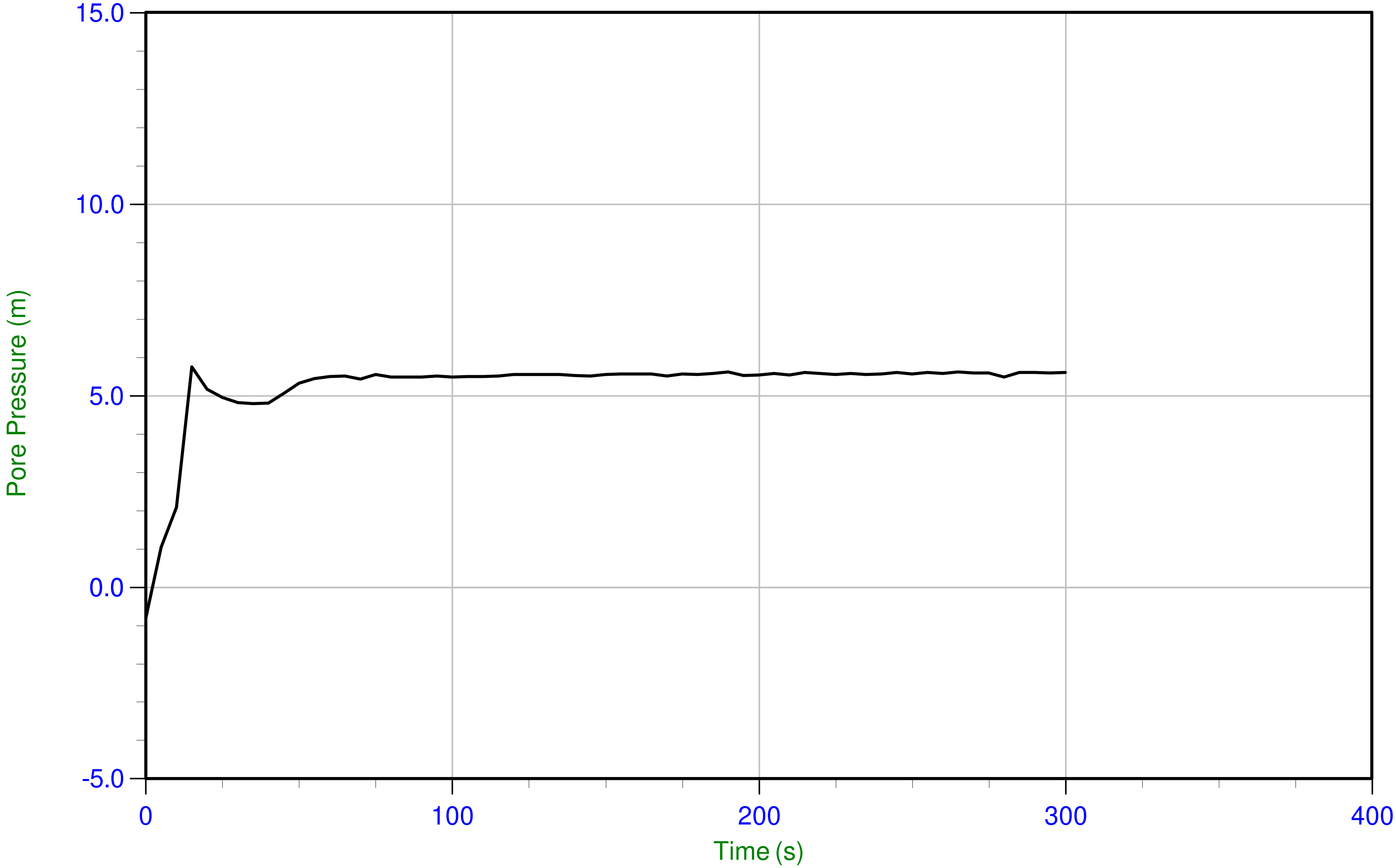
WT: 6.042 m / 19.822 ft
Ueq: 8.3 m

ConeTec

Golder Associates

Job No: 20-05-21040
Date: 07/03/2020 13:45
Site: Bayshore Mall, Ottawa

Sounding: SCPT20-01B
Cone: 531:T1500F15U500 Area=15 cm²



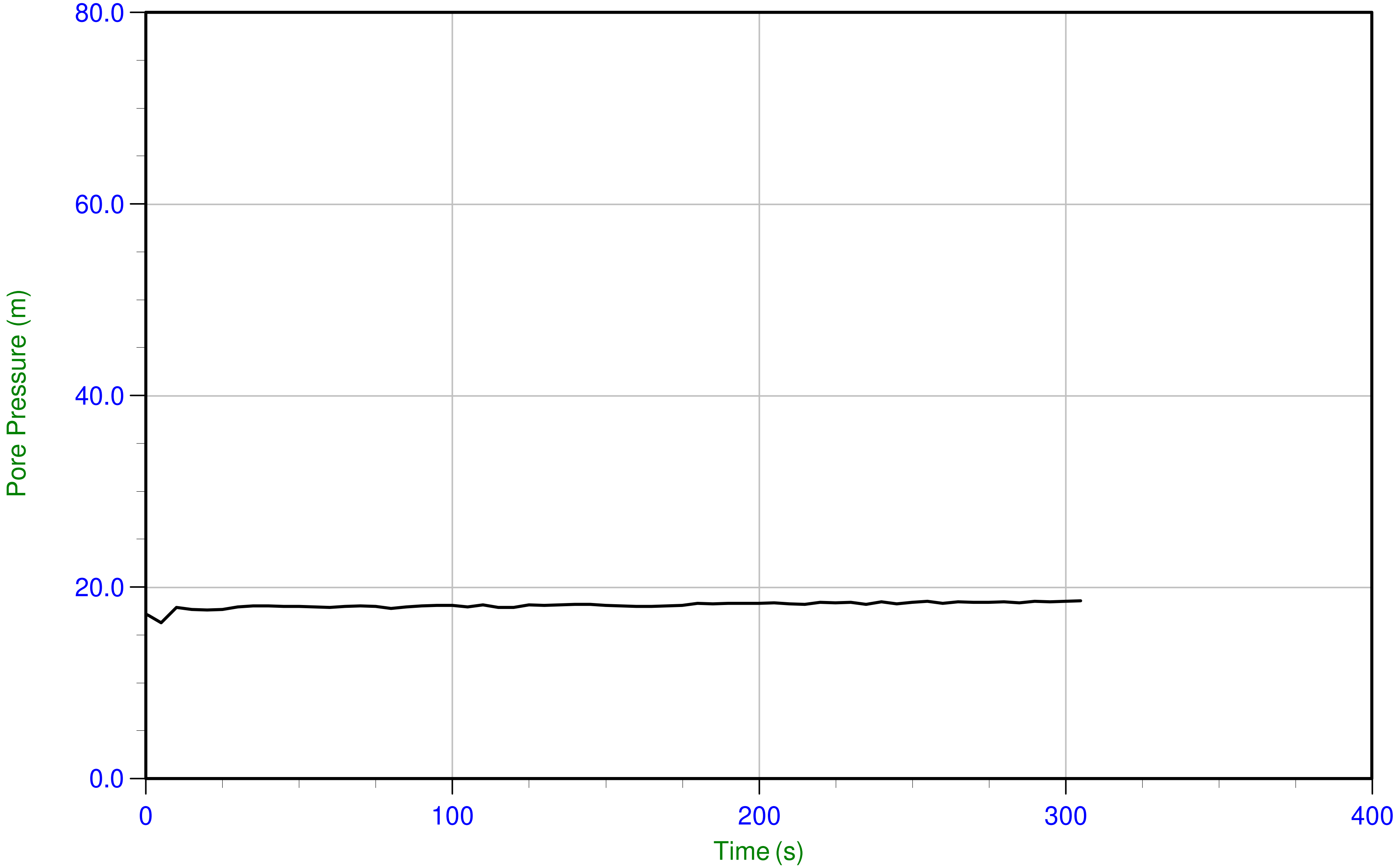
Trace Summary:

Filename: 20-05-21040_SP01B.PPF
Depth: 11.700 m / 38.385 ft
Duration: 300.0 s

u Min: -0.8 m
u Max: 5.8 m
u Final: 5.6 m

WT: 6.127 m / 20.101 ft
Ueq: 5.6 m

ConeTec	Golder Associates	Job No: 20-05-21040	Sounding: SCPT20-02
	Date: 07/03/2020 14:32	Cone: 545:T1500F15U500 Area=15 cm²
	Site: Bayshore Mall, Ottawa



Trace Summary:

Filename: 20-05-21040_SP02.PPF
Depth: 24.500 m / 80.380 ft
Duration: 305.0 s

u Min: 16.3 m
u Max: 18.6 m
u Final: 18.6 m

WT: 5.986 m / 19.639 ft
Ueq: 18.5 m

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