FINAL REPORT

Geotechnical Assessment Proposed Residential Development

3930 and 3960 Riverside Drive, Ottawa, Ontario

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1.0 INTRODUCTION

Golder Associates Ltd. (now WSP) was retained by Taggart Realty Management (Taggart) to carry out a geotechnical assessment of the proposed residential development site located at 3930 and 3960 Riverside Drive in Ottawa, Ontario.

The geotechnical assessment includes a desktop review of the geotechnical studies previously completed for this site and well as additional intrusive site investigation (by advancing boreholes and CPT holes) undertaken in January 2023 to support detailed design and address potential geotechnical concerns.

The purpose of this report is to assess the general subsurface and groundwater conditions within the study area, provide a general description of these interpreted subsurface conditions, and prepare engineering guidelines on the geotechnical design aspects of the project, including construction considerations which could influence design decisions.

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

2.0 PROJECT AND SITE DESCRIPTION

2.1 Site Background

The site is located immediately northwest of the intersection of Riverside Drive and Hunt Club Road, in the City of Ottawa, Ontario (see Key Plan Inset, Figure 1). The site is located between Riverside Drive and the Rideau River, extending north from Hunt Club Road and south from Kimberwick Crescent.

The site was previously used for granular material extraction (i.e., 'sand pit') activities that lasted at least until the 1970's. Over the subsequent years, the site has been sequentially filled to reclaim the land for development purposes and up to about 20 m of fill material has been placed at the site in some locations.

The property area between Riverside Drive and the Rideau River includes both an upland area and a lowland area. The upland area consists of higher elevation table land and is the area currently proposed for the development. The ground surface elevation varies across the upland area, ranging from about 90 to 98 m in the southern area and about 88 to 98 m in the northern portion of the site. Previous filling of these areas has resulted in an uneven ground surface across these areas.

The lowland area consists of a relatively narrow strip of land separating the table land from the Rideau River. The upland area is separated from the lowland area by moderate slopes. The lowland area is separated from the Rideau River by additional slopes. The slopes along the Rideau River are relatively steep and about 8 to 12 m in height within the southern portion of the site; however, within the northern portion of the site, the riverbank slopes along the river (beneath the 'lowlands') are only about 2 m high.

The high riverbank slope within the southern portion of the site is bisected by a major drainage gully, which drains the upland area runoff into the Rideau River. Several minor gullies (rills) also exist throughout the riverbank slopes.

The upland area is primarily vegetated with tall grass and occasional trees. The lowland and slope areas are vegetated with dense vegetation including young and mature trees, shrubs, and tall grass.

A privately-owned pump station is located within the lowlands on the north part of the site. It is understood that the pump station provides irrigation water for the Hunt Club golf course.

Based on the results of the current and previous geotechnical investigations carried out at this site as well as the published geologic mapping, the subsurface conditions consist of variable thicknesses (up to about 20 m) of miscellaneous fill underlain by native soils consisting primarily of sand with varying amounts of clay, silt, and gravel deposits, which are in turn underlain by very dense glacial till or sand and gravel. The underlying bedrock is mapped as sandstone of the March Formation or dolostone of the Oxford Formation. Bedrock was only proven in one previous borehole (advanced in 1983) at an elevation of about 65 m which is about 30 m below the general table land level. The bedrock encountered in that borehole was identified as limestone with shale interbeds.

2.2 Proposed Residential Development

It is understood that the residential development proposed at this site will eventually include townhomes, single family homes, and four high-rise residential apartment buildings. Additionally, supporting site services and features such as sanitary and storm sewers, watermains, access road, and a multi-use pathway (MUP) have also been proposed in the preliminary design. Based on the most recent information provided by Taggart, the following is understood about the currently proposed services and features at this site:

- A sanitary sewer is proposed which will connect to an existing manhole at the north side of the site, extending southward through the site, to a new manhole at the southwest corner of the site (adjacent to the Rideau River). The total length of the proposed sanitary sewer is about 400 m, and the diameter of the sewer is about 450 mm. The proposed invert depth of the sanitary sewer across the site ranges from 3 to 7 m below the existing grade (elevations of about 84 to 87 m).

- An access road is proposed which will extend from the northeastern limit of the site, running southward and parallel to Riverside Drive for about 170 m, then turning westward for about 90 m, and going northward again for another 90 m before ending. Two watermains of 250 mm diameter are also being proposed within the access road. The total length of the access road is about 350 m.

- A storm sewer is proposed along the base of the embankment leading from the site (near the manhole MH 100 at the north boundary of the site) to the stormwater management pond located outside the site (further north). This storm sewer will collect the discharge from all local storm sewers proposed within the site. The total length of the proposed storm sewer is about 300 m, and the diameter of the sewer varies from about 2400 to 1800 mm. The proposed invert level of the storm sewer varies between elevations of about 76 and 79 m.

- A multi-use path (MUP) is also proposed at the site. A section of that MUP will be built atop the proposed storm pipe. This is proposed to be done by building the base of the embankment (along which the storm pipe is running) while ensuring proper slope drainage. This will allow the MUP to gradually gain elevation as it approaches the top of the embankment. The MUP is proposed to continue along the top of the embankment around the western perimeter of the development.

- An engineered fill 2.5H:1V buttress slope is proposed along the northern edge of the site against the existing ‘upper slope’ to adjust the alignment of the slope crest in the North area (see Figures 1,1A, and 1B). The adjusted slope crest along the upper North Area slopes is considered technically feasible as these slopes do not abut against an active or perennial watercourse, provide material improvement to the site development potential, and do not have material impacts to existing sensitive habitats or species.

- Widening of Riverside Drive is proposed along the east boundary of the Site. Riverside Drive will be widened (by over 3 m) to facilitate the installation of additional traffic infrastructure. The proposed embankment slopes range from 2H:1V to 4H:1V and will be refined through the detailed design of the proposed roadway modifications by other design consultants who have been retained for this work. The widening of Riverside Drive is considered feasible from a geotechnical perspective.

3.0 PROCEDURE

3.1 Review of Previous Investigations

Subsurface information for the site was collected from several previous geotechnical investigations carried out by Golder Associates (now WSP). The results of these previous investigations are presented in the following reports:

- Report to the Ottawa Hunt and Golf Club titled "*Report on Geotechnical Investigation at Pumphouse Rebuilding Project, Ottawa Hunt and Golf Club*" dated September 2005 (report no. SF-4927).
- Report to the City of Ottawa titled "*Geotechnical Study, Uplands-River Road Study Area, Ottawa, Ontario*", dated October 1981 (report No. 811-2269).
- Report to the Regional Municipality of Ottawa-Carleton titled "*Soil Investigation, Drummond Pit, Ottawa, Ontario*", dated November 1983 (report No. 831-2386).
- Report to the Regional Municipality of Ottawa-Carleton titled "*Additional Soil Investigation, Drummond Pit, Ottawa, Ontario*", dated April 1984 (report No. 841-2088).
- Report to Delcan titled "*Geotechnical Considerations Proposed Widening and Realignment, Hunt Club Road and Riverside Drive, Ottawa, Ontario*", dated December 1984 (report No. 841-2470).
- Report to Perez Bramalea Ltd. titled "*Preliminary Subsurface Investigation, Proposed Commercial Development, St. Mary's Site, Ottawa, Ontario*", dated July 1991 (report No. 911-2151).
- Report to Cumming Cockburn Ltd. titled "*Phase I and Partial Phase II Environmental Site Assessment, Riverwalk Park and St. Mary's Sites, Riverside Drive, Ottawa, Ontario*", dated June 1994 (report No. 941-2735).
- Report to Perez Bramalea Ltd. titled "*Additional Geotechnical Investigation, Feasibility of Dynamic Compaction, St. Mary's Site, Riverside Drive, Ottawa, Ontario*", dated July 1994 (report No. 941-2135).
- Report to Taggart Realty Management titled "*Phase II Environmental Site Assessment, Riverside Drive and Hunt Club Road, Ottawa, Ontario*", dated September 2001 (reports No. 011-2898-5000 and 5500).
- Report to Taggart Corporation titled "*Preliminary Geotechnical Assessment, St. Mary's Site, Ottawa, Ontario*", dated September 2009 (report number 09-1121-0101).
- Technical Memorandum to The Taggart Group titled "*Site Conditions Report, Proposed PSAC Headquarters, Riverside Drive, Ottawa, Ontario*", dated May 2, 2011 (report No. 11-1121-0050).
- Report to Revera Inc. titled "*Phase Two Environmental Site Assessment, Part of 3930 Riverside Drive, Ottawa, Ontario*" dated September 2017 (report No. 1670692-5000).
- Report to St. Mary's Land Corporation titled "*Phase Two Environmental Site Assessment, Proposed Development at Riverside Drive and Hunt Club Road, Ottawa, Ontario*" dated January 2018 (report No. 1670692-3000).
- Report to The Taggart Group titled, "*Preliminary Geotechnical Assessment, Proposed Development, Hunt Club Road and Riverside Drive Ottawa, Ontario, Report No. 1670692-1000*" dated March 2018.

- Report to The Taggart Group titled, "*Golder's updated report (Rev 1) titled "Geotechnical Investigation, Proposed Sanitary Sewer, School and Retail Development, St. Mary's Site, Hunt Club Road and Riverside Drive, Ottawa, Ontario, Report number 1670692-2000*" dated March 2018.
- Technical Memorandum to The Taggart Group titled, "*Additional Slope Stability Guidelines – Rev 2, Proposed Development, Hunt Club Road and Riverside Drive, Ottawa, Ontario, Project No. 1670692*" dated October 17, 2018.
- Technical Memorandum to The Taggart Group titled, "*Geotechnical Treatments and Ground Improvement Options, Proposed Sanitary Sewer and Access Road, Hunt Club Road and Riverside Drive, Ottawa, Ontario, Project No. 1670692-TM2*" dated November 8, 2019.
- Technical Memorandum to The Taggart Group titled, "*Updated Limit of Hazard Lands Assessment along the Northern Section of the Site, St Mary's Lands, Hunt Club Road and Riverside Drive, Ottawa, Ontario, Project No. 21482114*" dated March 18, 2022.

The approximate locations of relevant boreholes from these previous subsurface investigations are shown on Figures 1, 1A, and 1B.

In addition to reviewing the previous borehole information, the thickness of fill material placed across the site has been assessed using available site topographic maps from the previous investigation reports. In particular, the topographic data given in the 1983 and 1984 investigation reports show the approximate site conditions prior to the placement of significant fill (only relatively minor filling had been carried out by that time). The borehole data was then compared with collected topographic data in about 2007 and again in 2017 for the site, and the resulting assessment of the fill thicknesses across the site is shown on Figure 1.

The site has been divided into two areas based on topographical characteristics at the site. These two areas, hereafter called the North Area and South Area, are shown on Figure 1. The two areas have then been subdivided into sub-areas based on the estimated amount of filling present at the site, as shown on Figure 1. It is noted that the boundary lines are approximate only and may not be representative of the actual fill thicknesses throughout the entire development site.

An overview of the subsurface conditions within each area, based on the previous boreholes data (combined with the findings of the current investigation) and available topographic elevation contours, is provided in Section 4.0.

3.2 Current Investigation

The fieldwork for this geotechnical investigation was carried out between January 17 and March 2, 2023. During that time, a total of 4 boreholes (numbered 23-01 to 23-04) were advanced at the approximate locations shown on the attached Site Plans (Figures 1, 1A, and 1B). The boreholes were advanced using a track-mounted drill rig supplied and operated by CCC Group of Ottawa, Ontario.

The boreholes were advanced (using mud rotary technique) to depths of about 22 m below the existing ground surface, i.e., elevations ranging from about 68 to 73 m. Standard penetration tests were carried out in the boreholes within the overburden at regular intervals of depth where possible. Samples of the soils encountered were recovered using split-spoon sampling equipment in general accordance with ASTM D1586-18.

A total of seven monitoring wells were sealed into the four boreholes to allow for measurement of the stabilised groundwater levels (in both, shallow and deep strata). The groundwater level measurements in these wells were carried out by WSP personnel on 28 February 2023.

Additionally, Cone Penetration Tests (CPTu's) with pore pressure dissipation were carried out at five locations, i.e., CPT 23-01 to CPT 23-04 (see Figures 1, 1A, and 1B). The testing was carried out using a 30-ton track mounted drill rig supplied and operated by Conetec Investigations Ltd. (Conetec) of Toronto, Ontario.

The CPT holes were advanced to depths ranging from about 0.6 to 22.3 m below the existing ground surface. Where cone refusal was encountered before reaching the target depth of 22 m, alternate CPT holes 23-01B, 23-02B/C, 23-03B, and 23-04B were attempted adjacent to the original CPT hole locations, except for alternate CPT holes 23-01C/D which were attempted about 40 m away from CPT 23-01/B (see Figures 1, 1A, and 1B). Despite the additional efforts, the target depth could not be achieved at any CPT location except CPT 22-04B due to the presence of very dense soils or cobbles/boulders encountered in the subsurface.

Shear wave velocity (Vs) testing was carried out as part of the seismic cone penetration testing performed in CPT 23-01/B/C/D and CPT 23-03/B. A built-in geophone within the cone penetration probe recorded seismic wave traces from a surface source as the CPTs were advanced. To supplement the shear wave velocity data generated from the seismic cone penetration testing and to produce time weighted average Vs in the top 30 m of subsurface, 1 Multichannel Analysis of Surface Waves (1D - MASW) test and 2 Horizontal to Vertical Spectral Ratio (HVSR) tests were also carried out at the site by Conetec.

The details and results of the CPT investigation and the geophysical testing are recorded in the two reports provided by Conetec, which are shown in Appendices E and G of this report, respectively. The results of the CPT investigation were assessed for the purposes of this report, and they were found to be in general conformance with the findings of the borehole investigation. The CPT results were mainly used for the assessment of liquefaction potential at the site.

The fieldwork was supervised by personnel from our engineering staff who located the boreholes and CPT holes, directed the drilling and insitu testing operations, logged the boreholes and samples, and took custody of the soil samples retrieved. On completion of the drilling operations, the soil samples were transported to our laboratory for further examination by the project engineer and for laboratory testing, which included natural water content, grain size distribution, Atterberg limits, etc. on selected soil samples. Four samples of soil (from boreholes 23-01, 23-02, 23-03, and 23-04) were submitted to Eurofins Environment Testing for basic chemical analyses related to potential sulphate attack on buried concrete elements and potential corrosion of buried ferrous elements.

The borehole and CPT hole locations were selected in consultation with Taggart, marked in the field, and subsequently surveyed by WSP personnel. The borehole and CPT hole coordinates and existing ground surface elevations were measured using a Trimble R8 GPS survey unit. The geodetic reference system used for the survey is the North American datum of 1983 (CSRS: CBNV6-2010.0 NAD83). The borehole coordinates are based on the Universal Transverse Mercator (UTM Zone 18) coordinate system. The elevations are referenced to Geodetic datum (CGVD28).

3.3 Slope Mapping

Seven slope cross sections were surveyed on July 9, 2009, at the relevant slope locations along the Rideau Riverbank as part of a previous study listed in Section 3.1.

At that time, the topography along each slope cross section was surveyed (both for horizontal and vertical positions) using a Trimble R8 GPS survey instrument, with a vertical and horizontal accuracy of less than 0.1 m. A hand clinometer was also used to confirm the slope inclination at selected locations. The data was then used to develop approximate cross sections of the slope geometry at each location. The approximate locations of the slope cross sections are shown on the Site Plans (Figures 1, 1A, and 1B). The slope cross sections were updated based on the topographic plans from 2017. The cross-sections of the surveyed slopes are shown on Figures 2 to 8.

Observations were also made on the state of erosion at the slope toe/riverbank in July 2009 and June 2018. Locations of minor to moderate to severe erosion observed at that time are also shown on Figures 1, 1A, and 1B.

In 2022, a detailed fluvial geomorphic assessment was also carried out by WSP which studied the 100-year erosion limit as well as toe erosion in detail using historical air photography analysis and field reconnaissance. The results of that assessment were provided in the below document.

- Technical Memorandum to Taggart Realty Management titled, "*Fluvial Geomorphic Assessment at Subject Area of the Rideau River to Support the Proposed Development at 3930 and 3960 Riverside Drive, Project No. 21482114*" dated December 20, 2022.

4.0 SUBSURFACE CONDITIONS

4.1 General

Based on the findings of the previous and current investigations, the subsurface conditions generally consist of variable thicknesses of random fill material (very loose to very dense granular soils and firm to very stiff cohesive soils with variable amounts of miscellaneous material) overlying loose to very dense native soil (generally sand to sand and gravel), overlying glacial till and then bedrock. The fill thickness ranges between about 5 and 20 m within the South Area (table land) and between about 3 and 9 m within the North Area (table land). The bedrock was encountered only in one borehole at an elevation of about 65 m.

The groundwater level in the North Area was generally measured between about 5 and 9 m depths (i.e., between elevations of about 83 to 88 m), but as deep as about 15 m (i.e., elevation of about 77 m). In the South Area, the groundwater level was generally found to be between 12 to 16 m depth (i.e., between elevations of about 75 to 83 m), but as shallow as about 6 m (i.e., elevation of about 88 m).

Since the time of completion of some of the previous geotechnical investigations, the ground surface at the site was further raised using miscellaneous fill. As such, some of the historical borehole records may not reflect the full thickness and composition of the fill material.

The following sections present a more detailed overview of the interpreted subsurface conditions on this property.

4.2 South Area

The South Area includes an upland (table land) area and a significant slope down to the Rideau River. The ground surface elevation in the table land decreases from about 100 m at Riverside Drive to about 92 to 94 m at the north and west boundaries of the table land. The slope down to the Rideau River is about 16 to 20 m high.

From the current investigation, boreholes 23-01 and 23-02 along with CPTs 23-01/B and 23-01C/D define the subsurface conditions within the table land. These are supplemented by boreholes 101, 102, 104, 105, 4, 01-5, 01-6, 11-3, 11-4, 17-204, 17-205, 17-206, 17-01, and 17-03 and test pit 11-103, from the previous investigations completed at this site. Previous borehole 103 defines the subsurface conditions within the slope area.

Records of current borehole and CPT logs are shown in Appendices A and E, respectively. Records of previous borehole and test pit logs are shown in Appendix C.

Significant infilling of the former sand pits was carried out throughout this area. From the available borehole information and topographic mapping, it appears that essentially the whole area (except the slope) is underlain by a layer of fill of variable composition and thickness. The fill generally consists of sandy silt, silty sand, clayey silt, and silty clay with variable amounts of one or more of the following materials: gravel, cobbles, boulders, topsoil, wood, concrete, bricks, plastic, metal, glass, and organic matter. A layer of concrete rubble, about 0.6 m thick, was encountered in previous borehole 17-01 at a depth of about 18.8 m below the existing ground surface.

The surface of the natural/original ground (beneath the fill) is indicated to vary between about elevations 75 and 92 m. The existing ground elevations within the table land area, based on the recent topographic mapping, vary between about 90 and 98 m. This aligns with the findings of the current and previous borehole investigations which indicate that the fill thickness is expected to vary between about 5 and 20 m within the table land area, with the fill being thickest in the central portion of South Area. The fill is indicated to range from very loose to very dense state of packing but is generally in a loose to compact state. Based on the borehole information and a review of previous and current topographic elevation contours, it appears that the deepest portion of the sand pit was essentially contained within this south part of the overall site. The fill thickness therefore tapers:

- To the east, adjacent to Riverside Drive.
- To the south, adjacent to Hunt Club Road and its approach to the bridge over the Rideau River.
- To the north, along the boundary with the North Area of the site.

These locations coincide with the slopes which formed the perimeter of the former pit. It also appears that a ridge of sand was left in-place (i.e., un-excavated) between the pit and the Rideau River, so that at least the lower part of the existing slope is the natural slope which pre-existed the sand pit. Small quantities of fill material appear to have been sporadically dumped over that slope, but otherwise there is minimal fill on the lower part of this slope. The overall site has however been filled up above the original ridge level, such that the upper part of the existing slope is composed of fill.

A thin layer of very stiff weathered silty clay crust (about 0.8 m thick) was encountered below the fill at boreholes 103, 104, and 105, located along the south and west edges of the site. Otherwise, the fill in South Area is generally underlain by a sand deposit (with varying amounts of silt and gravel) that transitions with depth, into a very dense sand and gravel deposit in some of the boreholes. The sand ranges from loose to very dense while the sand and gravel ranges from compact to very dense, however both materials would more typically be characterized as compact to dense.

The sand and gravel deposit was fully penetrated only in borehole 101 where it was proven to extend to an elevation of about 65 m (about 30 m beneath the current ground level), i.e., the bedrock surface.

Layers of very stiff clayey silt were found to exist at varying elevations within the native sand deposit in the South Area. For e.g., a deposit of very stiff clayey silt exists below the sand deposit in previous borehole 102 at a depth of about 23.5 m below the existing ground surface (elevation 75.3 m). This deposit was not fully penetrated but was proven to be at least 2.6 m thick. Similarly, in the current borehole 23-02, interbedded sand and clayey silt exist between depths of about 19.2 and 20.6 m below the ground surface (i.e., between elevations of about 69.2 and 70.5 m).

Similar deposits of relatively thin and very stiff cohesive material exist across the site at random elevations within the thicker native sand deposit, especially in the North Area (see Section 4.3).

The underlying bedrock surface appears to dip down to the north or northwest. Borehole 101, as well as previous boreholes (not shown on Figure 1) advanced by Golder Associates (now WSP) at the east abutment of the existing Hunt Club Road bridge (for its design) indicate that the bedrock surface beneath the south part of the site is at about elevation 60 to 65 m, which is about 30 m below the general table land level.

The groundwater level in the native deposits was recorded generally between about elevations 75 and 78 m but was as high as 88 m at the boundary of South and North Area. Also, the water level was higher near Riverside Drive, reflecting a downward gradient from east to west across the site, towards the river. An artesian water level was also recorded for the bedrock, at about elevation 82 m, in borehole 101 on November of 1983 (i.e., artesian relative to the ground level at that time).

The general groundwater level of about elevation 75 to 78 m approximately corresponds to the bottom of the fill material and likely controlled the lowest level to which the pit was apparently excavated. The groundwater levels measured most recently in this area of the site were measured on May 3, 2017 (in borehole 17-03), May 4, 2017 (in boreholes 17-01), and February 28, 2023 (in boreholes 23-01 and 23-02), and are summarized in the table below.

<table>
<tr><th>Borehole Number</th><th>Screen Interval (m)</th><th>Geological Unit</th><th>Ground Surface Elevation (m)</th><th>Water Level Depth (m)</th><th>Water Level Elevation (m)</th><th>Date of Measurement</th></tr>
<tr><td>17-01</td><td>19.7-21.2</td><td>Sandy Silt</td><td>94.8</td><td>16.4</td><td>78.4</td><td>May 4, 2017</td></tr>
<tr><td>17-03 (at the boundary of South and North Area)</td><td>5.2-8.2</td><td>Sand</td><td>94.3</td><td>6.6</td><td>87.6</td><td>May 3, 2017</td></tr>
<tr><td rowspan="2">23-01</td><td>10.1-13.1</td><td>Fill</td><td rowspan="2">94.7</td><td>12.2</td><td>82.5</td><td rowspan="2">February 28, 2023</td></tr>
<tr><td>18.5-20.1</td><td>Fill</td><td>16.0</td><td>78.7</td></tr>
<tr><td rowspan="2">23-02 (at the boundary of South and North Area)</td><td>12.0-15.2</td><td>Sand</td><td rowspan="2">89.7</td><td>12.6</td><td>77.1</td><td rowspan="2">February 28, 2023</td></tr>
<tr><td>18.4-20.0</td><td>Glacial Till</td><td>14.6</td><td>75.1</td></tr>
</table>

Groundwater levels are expected to fluctuate seasonally. Higher groundwater levels are expected during wet periods of the year, such as spring. Groundwater levels are also likely to be higher during periods of high water in the Rideau River.

4.3 North Area

The North Area includes two relatively flat areas, discussed as ‘upland’ and ‘lowland’ areas, which are separated by a slope. The lowland area abuts the Rideau River on its western boundary. The upland area, which is the area proposed for development, slopes from about elevation 99 to 102 m at Riverside Drive to about 88 m at the northwestern site boundary. The upland (or table land) area is higher than the lowland area by about 8 m (due to the placement of fill material within the upland area).

From the current investigation, boreholes 23-03 and 23-04 along with CPT 23-02/B/C, 23-03/B, and 23-04/B define the subsurface conditions within the table land. These are supplemented by boreholes 01-1, 01-2, 01-3, 91-1, 91-3, 91-4, 11-1, 11-2, 17-201, 17-202, 17-203, 17-207, and 17-07 along with test pits 94-8, 94-9, 94-15, 94-17, 94 18, 01 1, 01-2, 01-5, 01-6, 01-7, 01-8, 11-101, and 11-102, from the previous investigations completed at this site. Previous borehole 81-6 and test pit 01-9 define the conditions within the lowland area (along with the MGS geotechnical data for the pump station adjacent to the Rideau River).

Records of current borehole and CPT logs are shown in Appendices A and E, respectively. Records of previous borehole and test pit logs are shown in Appendix C.

From the available boreholes and topographic maps, it appears that eastern part of the North Area has also been filled though not as extensively as the South Area. The fill in North Area is of variable composition and thickness, consisting of silty sand, sand, silty clay, and clayey silt with variable amounts of one or more than one of the following materials: organic matter, gravel, cobbles, bricks, wood fragments, asphalt, metal etc.

The natural/original ground surface level beneath the fill is indicated to vary between elevations 86 and 90 m. The existing ground elevation within the upland area, based on the recent topographic mapping, varies between about 90 and 95 m, except within the east end where the ground level rises up to Riverside Drive. This aligns with the findings of the current and previous borehole investigations which indicate that the fill thickness is expected to vary between about 3 and 9 m within the table land but could be potentially thicker near Riverside Drive where the ground surface level rises. The fill generally ranges from a very loose to compact state of packing.

The fill is underlain by a thick sand deposit which generally contains variable amounts of silt, gravel, and clayey silty seams. This native sand deposit is also understood to consist of randomly distributed layers of very stiff cohesive material with varying amounts of sand. For e.g., a 0.6 m thick layer of sand and very stiff silty clay exists within the sand deposit in borehole 17-203 at a depth of about 7.6 m; a 1.1 m thick layer of very stiff clayey silt and silty clay exists within the sand and gravel deposit in borehole 17-207 at a depth of about 11.4 m; and a 4 to 5 m thick layer of very stiff silty clay exists within the sand deposit at the north end of the site in borehole 91-1.

Also, in boreholes 17-201 and 17-202, layers of very stiff sandy silty clay and stratified silty sand, silty clay and clayey silt were encountered below (but assumed within) the sand deposit. These deposits were not fully penetrated in the boreholes but were proven to depths of about 9.8 m below the existing ground surface (elevations of about 81.5 and 82.3 m in boreholes 17-201 and 17-202, respectively). Even though these layers were not fully penetrated, it is assumed that these are relatively thin layers of very stiff cohesive material within the thicker native sand deposit underlying the fill, like the layers encountered in boreholes 17-203, 17-207, and 91-1.

In the upland area, the sand deposit was fully penetrated only in boreholes 17-207 and 91-1 at depths of about 28.5 m and 26.2 m below the ground surface (elevations 66.1 m and 63.7 m, respectively), where the dense sand transitions into a very dense sand and gravel deposit. In the lowland area, borehole 81-6 indicates that the sand may be very thin and overlies very dense glacial till (silty sand with some gravel) at a depth of about 3 m below the ground surface (elevation 79 m). The thick native sand deposit in the North Area ranges from loose to very dense but would more typically be described as compact to dense.

The very dense sand and gravel deposit encountered below the thick sand deposit in boreholes 17-207 and 91-1 was not fully penetrated but was proven to extend to depths of about 31.7 m and 29.1 m (elevations 62.9 and 60.8 m) below the existing ground surface, respectively. The very dense glacial till deposit that underlies the sand in the lowland area in borehole 81-6 was also not fully penetrated but was proven to extend to a depth of about 3.7 m (elevation 78.2 m) below the ground surface.

Beneath the upland area, borehole 91-1 encountered auger refusal at about elevation 60.8 m, which could indicate potential bedrock surface (at a depth of about 30 m beneath the current ground level).

The groundwater level was generally recorded between elevations 85 and 89 m, but potentially as low as about elevation 77 m in the area closer to the river, likely reflecting a downward gradient in that direction. The groundwater levels measured most recently in this area of the site were measured on May 2, 2017 (in borehole 17-07),on January 19, 2018 (in boreholes 17-201 and 17-203), and on February 28, 2023 (in boreholes 23-03 and 23-04) and are summarized in the following table.

<table>
<tr><th>Borehole Number</th><th>Screen Interval (m)</th><th>Geological Unit</th><th>Ground Surface Elevation (m)</th><th>Water Level Depth (m)</th><th>Water Level Elevation (m)</th><th>Date of Measurement</th></tr>
<tr><td>17-201</td><td>6.3-9.3</td><td>Sand to sandy silty clay</td><td>91.2</td><td>5.2</td><td>86.0</td><td>Jan. 19, 2018</td></tr>
<tr><td>17-203</td><td>6.1-9.2</td><td>Sand and silty clay</td><td>93.6</td><td>5.8</td><td>87.8</td><td>Jan. 19, 2018</td></tr>
<tr><td>17-07</td><td>6.1-7.6</td><td>Sand</td><td>93.8</td><td>5.6</td><td>88.2</td><td>May 2, 2017</td></tr>
<tr><td>23-03</td><td>18.5-20.0</td><td>Sand</td><td>91.8</td><td>15.1</td><td>76.7</td><td>February 28, 2023</td></tr>
<tr><td rowspan="2">23-04</td><td>6.4-7.9</td><td>Sand</td><td rowspan="2">91.6</td><td>4.8</td><td>86.8</td><td rowspan="2">February 28, 2023</td></tr>
<tr><td>18.3-19.8</td><td>Sand</td><td>9.0</td><td>82.6</td></tr>
</table>

Groundwater levels are expected to fluctuate seasonally. Higher groundwater levels are expected during wet periods of the year, such as spring. Groundwater levels are also likely to be higher during periods of high water in the Rideau River.

4.4 Sanitary Sewer Alignment

Boreholes 17-201 to 17-205, inclusive, were advanced along the proposed sanitary sewer alignment during a previous investigation carried out in 2017. Based on these boreholes, the fill materials extend to depths ranging from about 4 to 5.6 m below the existing ground surface (elevations 85.6 to 88 m) along the northern section of the alignment (at boreholes 17-201 to 17-204) and become thicker towards the southern end of the alignment (near borehole 17-205) to a depth of about 14.9 m (elevation 78.2 m).

The fill materials encountered at the borehole locations consist of a heterogeneous mixture of sand, silty sand, clayey silt to silty clay, with variable amounts of gravel, cobbles, and organic matter. Construction debris (e.g., concrete, asphaltic concrete, wood, wire, plastic and brick fragments etc.) was also noted within the fill. The fill at boreholes 17-202 and 17-203 has a high clay content throughout its entire thickness.

4.5 Access Road and Watermain Alignment

Boreholes 4, 17-204, 17-206 and test pits 94-18, 11-103, 19-05 were advanced along the alignment of the proposed access road/watermain. Based on these boreholes, the fill materials extend to depths ranging from about 4 to 5 m below the ground surface (elevations 86.4 to 92.3 m) in the northern portion of the alignment and become thicker towards the southwest (near borehole 17-206) to a depth of about 15.9 m (elevation 76.4 m).

The fill materials encountered at the above borehole and test pit locations consist of a heterogeneous mixture of sand, silty sand, clayey silt to silty clay, with variable amounts of gravel, cobbles and organic matter. Construction debris (e.g., concrete, asphaltic concrete, wood, wire, plastic and brick fragments etc.) was also noted within the fill.

4.6 Storm Sewer Alignment

The proposed storm sewer will extend northwards from the north boundary of the site, across land owned by the City of Ottawa, and into a stormwater management pond located outside the site. No subsurface information is available along the proposed alignment of the storm sewer.

4.7 Laboratory Testing

4.7.1 Fill

Atterberg Limits testing carried out as part of current investigation on three samples of the clayey fill materials gave liquid limit values ranging from about 25 to 73 % and plasticity index values ranging from about 11 to 46 %. The results of the Atterberg limit testing indicates a soil of low to high plasticity. The Atterberg Limits are summarized on Figure B1 in Appendix B. The measured water content of 20 samples of the fill ranged from about 6 to 85 %. The results of grain size distribution testing carried out on five samples of the granular fill material are provided on Figures B2 and B3 in Appendix B.

Atterberg Limits testing carried out as part of previous investigations on five samples of the clayey fill materials gave liquid limit values ranging from about 28 to 55 % and plasticity index values ranging from about 10 to 37 %, indicating a soil of low to high plasticity. The Atterberg Limits are summarized on Figure D1 in Appendix D. The measured water content of 11 samples of the fill ranged from 5 to 51 %. The results of grain size distribution testing carried out on six samples of the granular fill material are provided on Figures D2 and D3 in Appendix D.

4.7.2 Sand with silt and gravel

Based on the current investigation, the measured water contents of 21 samples from these native granular deposits range from about 2 to 37 %. The results of grain size distribution testing carried out on three samples of sand are provided on Figures B4 and B5 in Appendix B.

Based on the previous investigations, the measured water contents of five samples from these native granular deposits range from about 10 to 30 %. The results of grain size distribution testing carried out on three samples of sand are provided on Figures D4 and D5 in Appendix D.

As noted previously, relatively thin layers of very stiff cohesive soils exist within the thicker native sand deposit at some locations. Atterberg Limits testing carried out on two samples from these layers gave liquid limit values of about 47 % and 23 % and plasticity index values of about 30 % and 12%, respectively, indicating a silty clay of low plasticity. The measured natural water content of these two samples was about 50% and 30 %, respectively. The results of the Atterberg limit testing are summarized on Figures D6 and D7.

4.7.3 Glacial Till

Based on the current investigation, the measured water contents of 5 samples from the glacial till range from about 7 to 12 %.

5.0 SLOPE MAPPING

5.1 South Area

The slopes within this portion of the site are composed of an ‘upper’ slope formed by the filling and a ‘lower’ slope composed of the native sand which extends down to the bank of the Rideau River. The approximate height and slope angle of the upper (between upland and lowland areas) and lower (Rideau Riverbank) slopes are as follows:

Slope Section	Upper Slope		Rideau River Slope	
	Slope Height (m)	Slope Angle (degrees)	Slope Height (m)	Slope Angle (degrees)
A-A’	7	19	12	36
B-B’	7	28	11	49
C-C’	9	14	8	47
D-D’	9	18	9	41

Based on the slope reconnaissance carried out in July 2009 and again in June 2018, the Rideau River slopes are generally covered with mature and dense vegetation (tall grass, shrubs and trees), while the upper slopes are grass covered. The vegetation along the Rideau Riverbank appears to be responsible for maintaining the surficial stability of these slopes. A major drainage gully (about 2 m wide by 2 m deep) has been cut through the riverbank slope by surface erosion.

No erosion protection is present along the Rideau Riverbank bordering the site. Areas of active erosion were noted at several locations along the Rideau Riverbank, which have resulted in over-steepened slope toes along the Riverbank. The results of the erosion mapping (from the 2009 and 2018 slope reconnaissance) along the Rideau Riverbank are provided on the Site Plan (Figures 1, 1A, and 1B). Above the zone of active erosion at the riverbank toe, the remaining portion of the slope appeared to be quite dry and stable (surficially), with the exception of the slope at section AA’. At a height of about 6 to 7 m above the riverbank (i.e., slope toe), the slope at section AA’ exhibits some evidence of soil softening and minor seepage. The soil within this area was observed to be bare of vegetation, indicating active erosion due to surface and seepage water runoff. However, this localized zone does not appear to be experiencing any deep-seated instability.

5.2 North Area

The slopes within this portion of the site are divided into table land slopes and Rideau Riverbank slopes. The approximate height and slope angle of the table land and Rideau River slopes are as follows:

Slope Section	Table Land Slope		Rideau River Slope	
	Slope Height (m)	Slope Angle (degrees – current/proposed fill)	Slope Height (m)	Slope Angle (degrees)
E-E’	8	14	2	54
F-F’	8	15/22	1.2	60
G-G’	6	7/22	2	45
H-H’	7	9/22	n/a	n/a
I-I’	8	20-22/22	n/a	n/a

An engineered fill 2.5H:1V buttress slope is proposed along the northern edge. This is discussed further in Section 6.4.

Both the Rideau River and table land slopes are generally covered with thick vegetation (tall grass, shrubs and trees). A broken drainage pipe was encountered at some distance (about 50 m) to the east of the river at the location of slope section EE'. A relatively deep gully has been formed between the pipe outlet and the Rideau River. Some sporadic rip rap erosion protection is present along the Rideau Riverbank at the locations of slope sections EE' and FF'.

Some moderate to severe active erosion of the Rideau Riverbank (over its 1 to 2 m height) was observed at the locations of cross sections EE' and FF'. Several small drainage gullies also exist which discharge into the Rideau River (i.e., cut into the bank). It appears that large trees and shrubs present along the Rideau Riverbank are responsible for maintaining the stability of the bank. No erosion was observed at the toe of the tableland slopes.

In addition to the observations made in 2009 and 2018, a detailed fluvial geomorphic assessment was carried out by WSP in 2022 which studied the 100-year erosion limit as well as toe erosion in detail, through historical air photography analysis and field reconnaissance completed in September 2022. The findings of that assessment are recorded in the following document and will supersede any relevant observations recorded in this report from 2009.

- Technical Memorandum to Taggart Realty Management titled, "*Fluvial Geomorphic Assessment at Subject Area of the Rideau River to Support the Proposed Development at 3930 and 3960 Riverside Drive, Project No. 21482114*" dated December 20, 2022.

6.0 DESIGN AND CONSTRUCTION CONSIDERATIONS

6.1 General

This section of the report provides engineering guidelines on the geotechnical design aspects of developing this site based on our interpretation of the current borehole and CPT records, available borehole records from previous investigations, and from a previous site slope survey carried out in 2009.

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.

6.2 Overview

The subsurface conditions on the site, based on the current and previous investigations, consist of variable thicknesses of very loose to dense fill material (generally silty sand, sandy silt, or silty clay with variable amount of miscellaneous material) overlying generally compact to dense native granular soils (sand overlying sand and gravel) extending to about 30 m or more below the current site ground level. Discontinuous deposits of very stiff silty clay (up to 5 m thick) also exist within the native granular soils at the site.

The fill thickness is greatest on the south part of the site (South Area), where the deepest part of the former sand pit was located. The fill material in the deep parts of this area ranges between about 10 and 20 m in thickness. Over the north part of the site (table land area), the fill thickness appears to generally range from about 3 to 9 m but may be thicker adjacent to Riverside Drive.

The groundwater level was generally reported to be at elevations 75 and 78 m within the South Area and between elevations 85 and 89 m within the North Area.

The ground surface elevation across the upland area in the South and North ranges from about 88 to 98 m, respectively, except where the ground level rises to Riverside Drive (about elevations 99 to 102 m), along the east side of the site.

The soil conditions encountered in the boreholes coupled with the slope conditions along the west side of the site present the following key issues associated with development of this property. Detailed geotechnical guidelines on each issue are provided in the subsequent sections of the report.

- The slopes along the west side of the South Area are only marginally stable under static conditions and are unstable under seismic loading conditions. Furthermore, the riverbank is being actively eroded. The lands adjacent to the slope are therefore considered to be ‘Hazard Lands’ and the development will need to be set-back from the slope. Based on the current development plan, it appears that the proposed development plans will not be impacted by the slope hazard.
- The surficial fill material is unsuitable for the support of foundations, floor slabs, or pavement in its current condition. The fill material (and anything relying on the fill for support) can be expected to settle even with modest loading.
- The proposed structures in the South Area (four high-rise apartment buildings) would need to be supported on deep foundations, which derive their support from below the fill layer (potentially bedrock). The floor slab would need to be structurally supported on the deep foundations.
- The proposed residential homes in the North Area, where the fill is expected to be somewhat thinner, can be founded on spread footings placed on engineered fill following the removal of the existing fill, and replacement with properly placed and compacted engineered fill, below the foundation footprint.
- After discussions with a ground improvement consultant, a ground improvement program using rammed aggregate piers (GeoPier or Controlled Modulus Columns) may be considered for this site to densify the soil to support residential homes.
- A ground improvement program (such as rapid impact compaction) should also be considered to improve the subgrade for the support of services and pavements. Otherwise, sub-excavation of the fill materials beneath service pipes could be required to avoid settlements that would otherwise be damaging to the operation and integrity of sewers and watermains. Pavements could also experience unacceptable settlement and distortion if a ground improvement program is not carried out.
- Complete sub-excavation of the fill beneath the services and pavements can also be considered as a viable option where the thicknesses are such that it is financially feasible. In this case, the subgrade preparation for the development should include removal of the fill material and proof-rolling (compaction) of the surface of the native soil layers with a heavy smooth-drum vibratory roller.
- Where the fill is relatively thicker (in south and northeast), a surcharge preloading method can be used to compensate for or minimize the post-construction differential settlements.

6.3 Seismic Considerations

The site falls within the Western Quebec Seismic Zone (WQSZ), as defined by the Geological Survey of Canada. The WQSZ constitutes a large area that extends from Montreal 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 Montreal-Maniwaki axis. Historical seismicity within the

WQSZ from 1900 to 2000 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.

A seismic Site Class also needs to be assigned (see Section 6.3.2), in accordance with Section 4.1.8.4 of the 2012 Ontario Building Code (OBC), to be used by the structural designer in determining the seismic forces to be considered in the design of the structures.

6.3.1 Liquefaction Assessment

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

- Instability of slopes, and even gently sloping ground can experience large lateral movements, which is referred to as "lateral spreading".
- Reduced shear resistance (i.e., bearing capacity) of soils which support foundations, as well as reduced resistance to sliding; and.
- Reduced shaft resistance for deep foundations as well as reduced resistance to lateral loading.

In addition, 'seismic settlement' 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 (which can be highly differential).

The following conditions are more prone to experiencing seismic liquefaction:

- Coarse grained soils (i.e., more probable for sands than for silts).
- Soils having a loose state of packing; and.
- Soils located below the groundwater level.

Previously, in 2017 and 2022, a preliminary assessment of the liquefaction potential of the existing fill materials and natural granular soil deposits (i.e., the sand plus the deeper sand and gravel deposits) was carried out using the Idriss and Boulanger (2008) simplified procedure based on SPT N_{60}-values from the previous boreholes advanced in 2017. The SPT N-values reported on the borehole records were corrected for overburden stress, rod length during sampling, and hammer energy efficiencies. The results of this assessment suggested that the existing fill and native submerged sands at the site would generally be classified as potentially liquefiable under an earthquake with a magnitude of 6.5 (Ottawa specified design value) and a peak ground acceleration of 0.302g. There were, however, concerns that some of the SPT N values were impacted by soil disturbance during drilling and may not be reliable indicators of liquefaction potential.

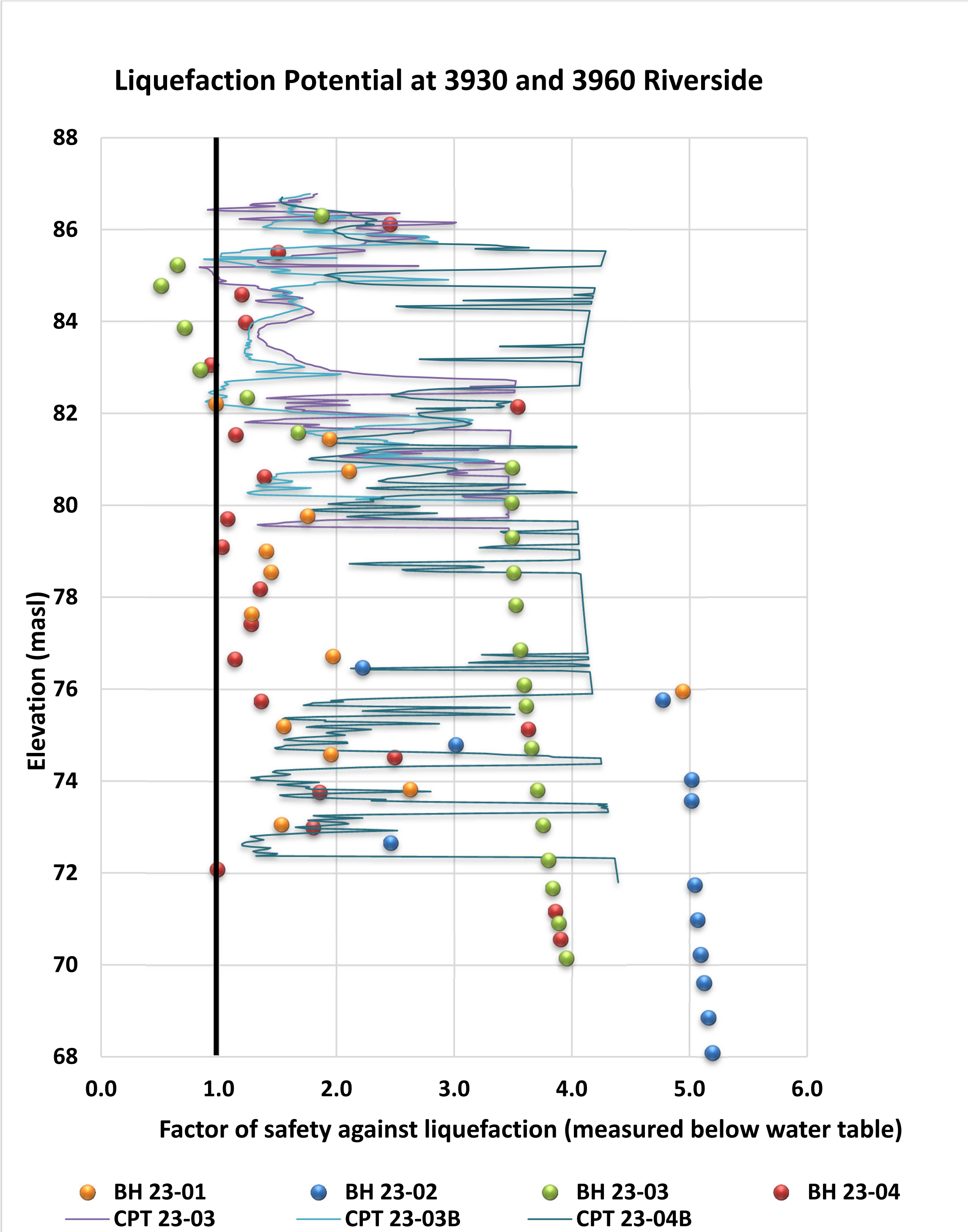
The current geotechnical investigations carried out at the site, i.e., CPTs and mud rotary based boreholes, were used to reassess the potential for liquefaction at this site. Three CPT results were used to assess the liquefaction potential, i.e., CPT 23-03, 03B, 04B, since these CPTs recorded data at deeper depths. Since liquefaction can only occur in saturated granular soils, only the data obtained below the inferred water table was used.

Additionally, SPT N-values obtained from boreholes advanced using the mud rotary technique were used for assessment. These values are considered relatively more reliable than the SPT N-values obtained during previous borehole investigations which were carried out without mud rotary technique or water in the augers. Furthermore, CPT data is considered even more reliable because it offers high resolution in situ results versus SPT, which often utilizes variable intervals, limited grab samples from various depths, and has a large uncertainty associated with measurement.

The following ground water levels were assumed for the assessment.

Borehole or CPT Number	Ground Surface Elevation (m)	Assumed Water Level Depth (m)	Assumed Water Level Elevation (m)	Basis of Assumption
BH 23-01	94.7	12.0	82.7	Two wells installed in BH 23-01 recorded water levels at 15.9 and 12.2 m depths. Additionally, water table was inferred at about 11 m in the nearby CPT 23-01C/D.
BH 23-02	89.7	12.5	77.2	Two wells installed in BH 23-02 recorded water levels at 14.6 and 12.6 m depths.
BH 23-03	91.8	5.0	86.8	One well installed in BH 23-03 recorded water level at 15.1 m depth. Additionally, water table was inferred at about 5 m in the nearby CPT 23-03/B. Also, water table was recorded at 5.8 m depth in the nearby previous borehole 17-203.
BH 23-04	91.6	5.0	86.6	Two wells installed in BH 23-04 recorded water levels at 9.0 and 4.8 m depths.
CPT 23-03/B	90.9	5.0	85.9	CPT 23-03 and 23-03B inferred water table at 4.5 and 4.8 m depths, respectively.
CPT 23-04B	94.1	7.4	86.7	CPT 23-04B inferred water table at 7.4m depth.

The (N1)60 I_c values which are essentially SPT N60 values corrected for overburden pressure were obtained using the I_c paramter from the CPT results and published correlations such as Lunne et al. (1997) , Robertson (2009), and Robertson (2012). These values were used to reassess liquefaction potential at the site through the Idriss and Boulanger (2008) simplified procedure based on SPT N_{60}-values. A plot is shown below which showcases the variation in factor of safety against liquefaction with elevation for each of the assessed boreholes and CPT holes.



Graph: Factor of safety against liquefaction vs geodetic elevation (m)

It can be observed in the above plot that while some small pockets of native or fill material may have an inadequate factor of safety against liquefaction, these pockets are highly localised and at varying elevations and therefore do not indicate a general liquefaction problem at the site. Furthermore, the more reliable CPT based results indicate relatively higher factors of safety for the same areas, compared to the less reliable SPT based results. As such, based on this reassessment of the liquefaction potential, this site in not considered to be at risk of large-scale liquefaction.

6.3.2 Seismic Site Classification

The results of the current and previous geophysical testing carried out at the site in the form of MASW (multi-channel analyses of surface waves) are presented in Appendices G and H, respectively.

The seismic design provisions of the 2012 Ontario Building Code depend, in part, on the shear wave velocity of the upper 30 m of soil and/or rock below founding level. Based on the measured shear wave velocity data, this site can be assigned a Site Class D for seismic design of structures founded in the overburden.

6.4 Slope Stability Assessment

6.4.1 General

The evaluation of the stability of a slope depends on several parameters, including:

1) The geometry of the slope
2) The ground conditions which form the slope (i.e., the thickness and orientation of the soil/bedrock strata)
3) The shear strength parameters of the soils which form the slope
4) The unit weight (i.e., density) of the soils which form the slope
5) The groundwater levels and flow gradients within the slope.

The stability of slope cross sections was assessed using the measured slope geometry and available information on the subsurface and overburden thickness conditions. The slope geometry used in the analyses was established from the topographical plans from June 24, 2009, and updated plans dated February 2, 2017, provided by Annis O'Sullivan of Vollebekk Ltd. The slope stability analyses output for all cross sections is shown in Appendix F.

The slope stability analysis was carried out to address both the "lower" and secondary "upper" slopes for each cross section analysed. Further, the stability analyses included the addition of a 2.5H:1V fill slope against the existing slope to adjust the alignment of the slope crest in the North area. The adjusted slope crest along the upper North Area slopes is considered technically feasible as these slopes do not abut against an active or perennial watercourse, provide material improvement to the site development potential, and do not have material impacts to existing sensitive habitats or species. Therefore, the limits of hazard lands provided based on this assessment are the cumulative hazard lands from the "lower" and "modified upper" slopes.

The ground conditions within the slope were based on the available borehole records as well as observations of the exposed soils made during the slope reconnaissance in 2009. For the slopes within the South Area, the lower portion of the slope was modelled as being composed of the native sand while the upper slope was modelled as being composed of fill material. The geometry of the former sand 'ridge' which separated the pit from the Rideau River was inferred from previous topographic records.

The slopes within the North Area were modelled as being composed of the native sand soils, but with a layer of fill material existing across the table land.

The soil parameters used in the analyses were based on experience with similar soils in the Ottawa area as well as published correlations with the results of the in-situ and laboratory testing. The soil parameters used in the analyses are:

Soil Type/ Material	Material Thickness (m)	Material Model	Bulk Unit Weight (kN/m3)	Drained Parameters		Undrained Parameters	
				Effective Angle of Internal Friction (°)	Effective Cohesion (kPa)	Angle of Internal Friction (°)	Cohesion (kPa)
Fill	2.1 – 11.8	Mohr-Coulomb	19	28	0	28	0
Engineered Fill: Sand/ Silty Sand	2.1 – 11.8	Mohr-Coulomb	20	34	0	34	0
Sand/ Silty Sand	4.0 – 12.5	Mohr-Coulomb	19 - 20	31	0	31	0
Sand and Gravel	13.0 – 21.0	Mohr-Coulomb	20 – 20.5	34	0	34	0
Silty Clay	Variable	Mohr-Coulomb	16.5	35	5	0	75
Bedrock	-	Bedrock (impenetrable)	-	-	-	-	-

For the South Area, the groundwater level was modelled as being at the level of the bottom of the fill material within the former sand pit (as indicated by the boreholes), with a slight gradient towards the river. The 'ridge' of sand between the former pit and the river was therefore modelled as being unsaturated. For the North Area, the groundwater level was modelled as being about 2 to 3 m below the slope surface, with flow generally parallel to the slope.

The stability of each slope cross section was evaluated for under both 'static' and seismic loading conditions. Effective stress soil parameters (as given above) were used under both the static and seismic loading conditions for cohesionless soils. The undrained parameters for silty clay were used for seismic loading conditions. The drained loading conditions may represent the long-term conditions of slope while the undrained loading conditions may represent the short-term during/immediately after the construction of the engineered slopes/proposed development.

The stability of the slopes was evaluated using the SLOPE/W software. The Morgenstern-Price method was used to compute a factor of safety. The factor of safety is defined as the ratio of the magnitude of the forces/moments tending to resist failure to the magnitude of the forces/moments tending to cause failure. Theoretically, a slope with a factor of safety of less than 1.0 will fail and one with a factor of safety of 1.0 or greater will stand. However, because the modelling is not exact and natural variations exist for all of the parameters affecting slope stability, a factor of safety of 1.5 is used to define a stable slope (for static loading conditions), or alternatively to define the acceptable set-back distance for permanent structures or valuable infrastructure from an unstable slope (i.e., the Limit of Hazard Lands). Under seismic loading conditions, a minimum factor of safety of 1.1 is used in a pseudo-static analysis along with a 10 % increase in mobilized shear strength to account for "strain-rate" effects.

6.4.2 Static Conditions

The results of the stability analyses carried out under static conditions for the sandy slopes indicate that the factor of safety against global instability of the existing Rideau Riverbank slopes (cross sections A-A' to D-D') within the South Area is generally less than 1.0 (i.e., potentially unstable).

For the shallower and flatter sand slopes within the North Area, which includes cross sections E-E’ to I-I’, the calculated factors of safety were greater than 1.5 (stable).

Based on these analyses, it is considered that the tall and steep existing Rideau River slopes within the South Area are not stable and could fail given appropriately high groundwater conditions, such as those that could be experienced during the spring thaw, or due to continuing erosion.

For the North Area, although the overall slopes are considered to be stable, continuing erosion at the creek bank could result in localized sloughing.

6.4.3 Seismic Conditions (Earthquake)

The potential instability under seismic (earthquake) loading was also evaluated at each of the selected cross section locations. These analyses were carried out using a simple “pseudo-static” model where a horizontal force equal to 50% of the peak ground acceleration for the 2% exceedance in 50-year earthquake hazard is applied to the failure mass. This horizontal force is proportional to the weight of the failure mass and is determined using a “seismic coefficient”.

For the South Area, the factors of safety against instability under seismic loading are less than 1.1. The slopes could therefore fail under the design seismic loading event.

For the North Area, the slopes are considered to be stable under seismic loading conditions.

6.4.4 Limit of Hazard Lands

In view of the low factors of safety against slope instability obtained for the slopes in the South Area, a setback from the slope crest for development was assessed at the cross-section locations. This setback was developed by carrying out further stability analyses to assess the limit beyond which there is an acceptable factor of safety (i.e., greater than about 1.5 static or 1.1 seismic) against slope failure. This setback is shown on Figures 1, 1A, and 1B as the “Limit of Hazard Lands.”

The land between the slope and the Limits of Hazard Lands, plus the slope area itself, would be defined as Hazard Lands in accordance with Ministry of Natural Resources and Forestry (MNRF) guidelines and provincial planning policies, as well as City of Ottawa guidelines. Hazard Lands are unsuitable for development with either publicly owned infrastructure or private development. No permanent structures or infrastructure (i.e., buildings, walkways, bridges, roadways, parking, etc.) should be constructed within the Hazard Lands.

In accordance with the MNRF guidelines, the setback distance from the crest of an unstable slope to the Limit of Hazard Lands includes three components, as appropriate, namely:

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

The magnitude of the *Erosion Allowance* is described in the MNRF guidelines and is a function of the soil type, state of erosion, and water course characteristics. The reconnaissance survey assessment carried out on July 9, 2009, identified active erosion along the Rideau Riverbank, adjoining to the site. As such, an *Erosion Allowance* of 15 m has been included in the determination of the Limit of Hazard Lands for slopes adjoining the Rideau River while no erosion allowance was provided for the North Area upper slopes based on the site reconnaissance observations. The fluvial geomorphic assessment report referred earlier in this report also suggests that the proposed development should include a minimum geomorphic (erosion) setback of 15 m to accommodate the potential for long-term channel migration/movement. It should be noted that the *Erosion Allowance* need not be considered if erosion protection were installed along the Rideau Riverbank.

For all the slopes, North and South Areas, a 6 m wide access allowance has been considered.

For the South Area (sections A-A' through D-D') where some slope sections have factors of safety lower than 1.5 for static or 1.1 for seismic, a stable slope allowance has been provided for.

For the North Area (sections E-E' through I-I'), all the slopes have adequate factors of safety under both static and seismic loading, with consideration of the compacted engineered buttress fill slope of 2.5H:1V used to adjust the slope crest location along sections F-F', G-G', H-H' and I-I'.

The resulting Limit of Hazard Lands based on the stable Slope Allowance, Access Allowance, and Erosion Allowance is shown on Figures 1, 1A, and 1B. Based on the current development plans and this assessment, the proposed development plans do not appear to conflict with the Limit of Hazard Lands.

The location of the Limit of Hazard Lands is based on the current slope geometry and site grading (including the fill slope modified site grading in the North Area). The results of the stability analyses were also confirmed (i.e., same limit of hazard lands) with a table land elevation that could be 1 m higher than currently proposed for the North Area slopes, to allow for some flexibility with the future development of the site grading plan. The subgrade and slope conditions at the vicinity of sections F-F', G-G', H-H' and I-I' allow for this 1 m higher table land elevation with an acceptable factor of safety.

Within the South Area, it has been confirmed with Taggart that the grade will not be raised higher than the ground elevations modelled in the attached slope stability assessment (Appendix F). If the ground level within the South Area (i.e., within that area adjacent to the highest and least stable slopes) is raised beyond what has been assessed, the location of the Limit of Hazard Lands may need to be re-assessed depending on the grade raise. Significant increase in the site grade could shift the Limit of Hazard Lands further from the slope and reduce the amount of developable land.

Conversely, the completion of a ground improvement program (see Section 6.5 of this report) could have a beneficial impact on the stability of the slope (by increasing the shear strength of the fill materials), which could shift the Limit of Hazard Lands closer to the slope and allow for more developable land.

For the North Area, although the overall slope is considered to be stable, the approximately 2 m high riverbank could be subject to erosion and sloughing. A modest set-back from the bank is therefore proposed, however there is no planned development for this part of the site.

6.4.5 Surface Drainage and Erosion Protection

Although the Limit of Hazard Lands indicated on Figures 1, 1A, and 1B do not apparently impact on (i.e., restrict) the current development plans, the line could be shifted towards the slope, and more table land defined as useable/developable land, if erosion protection were installed at the slope toe. With the installation of erosion protection, the 'Erosion Allowance' need not be considered in the evaluation of the Limit of hazard Lands.

Ongoing erosion of the slope toe is also one of the most likely potential triggers for a slope movement which, even if those movements did not impact on the development (since the development would be located outside of the

Limit of Hazard Lands), might have negative impacts on river navigation and aquatic habitat, and also be a cause of concern to the public.

The installation of erosion protection along the Rideau Riverbank could therefore have the following possible benefits:

- More developable land might be identified for the table land, by defining a Limit of Hazard Lands closer to the Rideau Riverbank slope;
- The risk of a future slope failure occurring and having to be repaired may be reduced; and,
- Fish habitat and riparian habitat might be improved.

The erosion protection measures could conceivably be of several forms, including riprap, gabion basket walls, or biotechnical measures such as live crib walls.

The decision as to whether to implement such measures (and which measures to implement) would however require consultation with the Rideau Valley Conservation Authority (RVCA) which regulates this waterway. An assessment of the regulatory or biological/ecological impacts would also be required and might preclude such measures being implemented. The RVCA has previously expressed a preference to not have erosion protection installed along the slope toe adjacent to this site.

As a general guideline, grading of the site should direct surface runoff away from the slopes into drainage channels (built along the rear yards of homes) designed specifically for this purpose and leading to a designed outlet. It is acknowledged that some drainage features may have to be accommodated in the “Access Allowance” area to achieve proper drainage design. Uncontrolled surface water runoff over the existing slopes can reduce the factor of safety against instability and should not be allowed.

6.4.6 Fill Slopes

The assessment provided in this report focuses on the ‘global’ stability of the slopes adjacent to the Rideau River, and on determination of the Limit of Hazard Lands associated with deep-seated failure of those slopes. There are however localized fill slopes on this site that, having been created by end-dumping, are overly steep.

Surficial instability of these slopes could be expected. Therefore, where these slopes exist within the development area, it should be planned to re-grade them to a flatter geometry. The required slope angle depends on the height of each filled slope but, as a preliminary guideline, it should be planned to flatten all slopes within the development area to no steeper than 2.5H: 1V (horizontal: vertical).

6.5 Site Grading and Ground Improvement

As described previously, the fill materials on this site were apparently placed under uncontrolled conditions and are therefore highly variable in composition and state of packing. These fill materials cannot be relied upon to support foundations, floor slabs, or grade-sensitive services. The fill materials are likely still consolidating under their own self weight and could settle significantly if stressed by additional load. The magnitude of the potential settlements cannot be predicted with any accuracy but would be significant. Even without the addition of further load, it could be expected that the fill materials would continue to settle over many years.

Typically, unsuitable fill materials (e.g., those fill materials containing organic matter and debris, such as on this site) should be excavated and replaced from below the founding level of structures, invert level of the services, and pavement areas. However, fill materials at the site were found to be up to 20 m thick in the south area at the location of the proposed residential homes. Fill materials over some sections of the proposed sewers and access road/watermains (e.g., near boreholes 4, 01-5, 17-205 and 17-206) are also up to about 15 to 16 m thick. As such, removing this material and replacing with an engineered fill material would be impractical in some locations.

It is therefore proposed that consideration be given to carrying out a ground improvement program for this site. Some options for geotechnical treatment and ground improvement options are provided below. These ground improvement techniques will result in densification of the variable fill present at the site and would likely allow for the densified fill to have adequate capacity to support the proposed structural loads. These ground improvement programs would also permit slab on grade floor slabs, site services, and pavements to be supported within the fill material.

6.5.1 Sanitary Sewer North Section (i.e., Fill about 6 m or less)

Along the northern end of the sanitary sewer alignment (i.e., north of borehole 17-204), where the fill thicknesses are relatively thin (about 3 to 6 m thick), the existing fill could be sub-excavated below invert level of the sanitary sewer (with the invert between about 3 to 7 m below existing grades) and replaced with properly placed and compacted engineered fill.

Based on the nearby boreholes (17-201 and 17-203), the groundwater level was measured at about 5 to 6 m below the existing ground surface (i.e., about elevations 86 to 87.8 m), which is at or just above the interface of the fill and native soils. Minor groundwater inflow should be expected during the sub-excavations of the fill materials.

However, depending on the final proposed invert elevations, the excavations for the construction of the sewer itself will be through the fill materials, and likely into the underlying native sandy and gravelly deposits (i.e., slightly below the measured groundwater level). Geotechnical recommendations related to excavation, groundwater inflow and control, pipe bedding, cover and trench backfill are provided in the subsequent sections.

Prior to placing the engineered fill, the exposed subgrade at the sewer invert should be inspected by qualified geotechnical personnel to confirm that the exposed soils are native and undisturbed. In the event localized areas of significantly thicker fill are encountered, geotechnical treatments described in section 6.5.2 can be considered. Remedial work (i.e., further sub-excavation and replacement) should be carried out as directed by geotechnical personnel.

6.5.2 Access Road/Watermain – Northeast Segment (i.e., Fill about 5 to 9 m)

At the northeastern portion of the site, where an access road and two 250 mm diameter watermain are being proposed, the fill materials are thicker (i.e., about 5 to 9 m thick) and sub-excavation of the fill may not feasible. The fill materials have limited capacity to accept additional stresses from the weight of compacted backfill or engineered fill without undergoing compression. That compression could lead to ground settlements and settlement of the services and roadway.

Consideration could be given to preloading (and possibly surcharging) to compress the fill materials (i.e., forcing the settlement of the fill materials to occur) prior to construction of the services as outlined in Section 6.5.2.1 below. Alternatively, a ground improvement program could be carried out as outlined in Section 6.5.2.2 below.

6.5.2.1 Pre-loading and Surcharging

To avoid excessive post-construction settlements of the proposed services/roadway, the site could be preloaded, the settlements allowed to occur (and monitored), and then the services/roadway constructed once the settlements have been completed (or sufficient settlement had occurred so that functionality of services/roadway would not be negatively impacted). A temporary surcharge above the proposed services/roadway alignment would need to be placed for the preload period, to apply a stress equivalent to the future weight of the grade raise, compacted pipe bedding, cover, and the service itself. It is envisioned that a 2m high surcharge would be placed above the final grade elevations.

The subgrade settlements would need to be monitored to establish when sufficient settlements have occurred such that construction of the services could proceed. The settlement monitoring should be carried out by measuring the movement of settlement plates placed at selected locations within the preload area. Once the monitoring of the settlement plates indicates that sufficient settlements have occurred, the surcharge could be removed, and the services/roadway be constructed. As a preliminary estimate, most of the settlements should occur within about 4 to 6 months upon completion placement of the preload and surcharge, although this should be verified by settlement monitoring.

Further details on the monitoring program, including the settlement plate locations, construction details, and the frequency and accuracy of the survey, can be provided if required. The approximate boundaries between areas of different thicknesses of fill materials are shown on the attached Figure 1. The lines are drawn based on the available test hole information and may not be representative of the actual fill thicknesses throughout the entire development site. At the time of carrying out the preloading and surcharge program, additional test pits may need to be advanced to confirm the thicknesses of the fill so that the program can be optimized.

6.5.2.2 Ground Improvement

Alternatively, a ground improvement program to densify the fill by either Dynamic Compaction (DC) or Rapid Impact Compaction (RIC) is considered feasible in this area where fill materials are less than about 9 m thick.

Conceptually, the following construction sequence is envisioned:

- Sub-excavate the existing fill materials within the full width of the proposed access road to the roadway subgrade
- Carry out ground improvement by means of either DC or RIC on the exposed subgrade to densify the underlying fill materials
- Following the ground improvement program, sub-excavate the service trench (about 2 m wide) to about 0.5 m below the proposed invert of the watermain and backfill with compacted engineered fill
- Install the watermain, then cover and backfill the watermain to the underside of the roadway subbase with compacted engineered fill

For both options, there will be some potential for post-construction settlement due to long term consolidation of the deeper fill materials. However, those settlements should not be excessive, and should probably not be noticeable or impact on the performance of the roadway or watermain.

To help reduce the impact of possible differential settlement, the thickness of the subbase material should be increased (see Section 6.15 on pavement structures). A geogrid placed at the pavement subgrade level will also be needed to reduce the differential settlement.

6.5.3 South Area (i.e., Fill about 10 to 20 m)

In the southern portion of the site, the fill materials are the thickest (up to about 20 m). Residential homes, apartment buildings, a deep sanitary sewer (which is grade sensitive), and access road/watermain are being proposed in this area. A more extensive ground improvement program such as the Geopier Rammed Aggregate Pier Impact System (RAP) or equivalent alternate by other specialists, to densify the fill to a deeper depth is therefore recommended in this area.

RAP is a ground improvement method whereby the soils are densified by installing closely spaced columns of compacted granular material (clear stone). RAP soil reinforcing elements using the Geopier installation methodology are installed by drilling 0.76 m diameter cavity and ramming thin lifts of well graded aggregates within the cavity to form very stiff high density aggregate piers. The drilled holes are typically placed at 2 m spacing and can extend to depths of up to about 15 to 20 m.

Conceptually, the following construction sequence is envisioned:

- Sub-excavate the existing fill materials within the full width of the access road to the invert of the proposed watermain and/or the shallower sanitary sewer pipes (e.g., MH 104, MH 105 and MH 106), whichever is deeper, expected to be about 3 m below the existing ground surface.
- Install RAP from the exposed subgrade to the native ground surface (about elevation 77 m on average).
- Following the ground improvement program, excavate to the proposed invert of the deep sanitary sewer (e.g., between MH104A and MH106A). Shoring may be required for this excavation.
- Install the sanitary sewer, then cover and backfill the sewer to the underside of the roadway subbase with compacted engineered fill.

For this option, there will be a low potential for post-construction settlement due to long term consolidation of the deeper fill materials. The densified fill will allow adequate capacity to support lighter building loads such as residential homes. The slabs, roadways, and services could be constructed using typical construction methodology without the need of thickening the roadway subbase and/or use of woven geogrid. It should be noted that since the apartment buildings are proposed to be founded on deep foundations with a structural slab on grade, ground improvement will not be required on the footprint of these buildings.

6.5.4 Site Grading

In regard to the site grading, although the placement of additional fill materials could add further load and increase the magnitude of potential long-term settlements, it is expected that this effect could be mitigated by the ground improvement program. From that perspective, there is not considered to be a restrictive limit on the permissible grade raise for this site (although significant grade raises could negatively impact on the stability of the slopes and on the location of the Limit of Hazard Lands). It should also be noted that in designing the ground improvement program, the proposed grade raise will need to be considered. WSP should review the final grade raise specifications for this project prior to tendering to confirm that our guidelines and recommendations have been adequately interpreted.

6.6 Site Servicing

Significant thicknesses of fill material exist on this site. The fill materials extend to depths varying from about 3 to 20 m below the existing ground surface, generally increasing in thickness to the south. Due to the potential for long term settlement, and the effects of this settlement on grade sensitive services, the existing random fill materials, in their current state, are not considered suitable for the support of the site services; even modest loading on the fill materials could result in compression of the fill materials.

Where fill material is encountered below invert level of the services, the fill material should be removed, where feasible, from below the services, and the services should be supported on engineered fill consisting of OPSS Granular A and B Type I or II. Prior to placing the engineered fill, the exposed subgrade should be inspected by qualified geotechnical personnel to confirm that the exposed soils are native and undisturbed. Remedial work (i.e., further sub-excavation and replacement) should be carried out as directed by geotechnical personnel. The engineered fill should be placed in maximum 300 mm thick loose lifts and should be compacted to at least 95% of the materials standard Proctor maximum dry density (SPMDD) using suitable vibratory compaction equipment.

The placement of engineered fill must be monitored by qualified geotechnical personnel on a full-time basis. The top surface of the engineered fill should be protected as necessary from construction traffic and should be sloped to provide positive drainage for surface water during the construction period. The engineered fill should be placed to occupy the full width of the service trench and the full zone of influence/support for the services. That zone is considered to extend down and out from the outside edge of the services at a slope of 1 horizontal to 1 vertical.

The fill material appears to be thinnest on the northern part of the site (i.e., north of boreholes 17-204 and 23-02). This being the case, site services should (from a geotechnical perspective) enter the development site from the north (if possible) to minimize the amount of sub-excavation. Where the fill is the thickest (i.e., south of borehole 17-204 and 23-02), consideration will need to be given to carrying out ground improvements in the area of the services. Consideration could also be given to preloading (and possibly surcharging) the areas of thickest fills to compress the fill materials (i.e., forcing the settlement of the fill materials to occur) prior to construction of the services. Guidelines for a preloading and surcharging program as well as ground improvement options are provided in Section 6.5.

6.6.1 Pipe Bedding and Cover

At least 150 mm of OPSS Granular A should be used as pipe bedding for the sewers. Where unavoidable disturbance to the subgrade surface does occur, it may be necessary to place a sub-bedding layer consisting of compacted OPSS Granular B Type II beneath the Granular A or to thicken the Granular A bedding.

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 standard Proctor maximum dry density. The use of clear crushed stone as a bedding layer should not be permitted anywhere on this project since fine particles from the sandy backfill materials and native granular soils could potentially migrate into the voids in the clear crushed stone and cause loss of lateral pipe support.

Cover material, from bedding level 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 standard Proctor maximum dry density using suitable vibratory compaction equipment.

Where the pipe invert level extends into silty clay to clayey silt (fine material) or falls below the anticipated water table, the bedding material should be placed on a layer of non-woven geotextile to act as a separation membrane to minimize the loss of the bedding material into the subgrade. This geotextile should extend across the trench bottom and up the sides of the pipe invert level.

6.6.2 Trench Backfill

It should generally be possible to re-use the granular inorganic soil from above the water table as trench backfill. Material from below the water table may be re-used provided that they can be adequately handled, including stockpiled, placed, and compacted. Some of the fill materials and siltier overburden below the water table may be too wet to compact. Where that is the case, these materials should be wasted (and drier materials imported) or these materials should be placed only in the lower portions of the trench, recognizing that some future ground settlement over the trenches may occur. This could be problematic in areas which will be covered with roadways. In that case, it would also be prudent to delay final paving for as long as practical and significant padding of the roadways may be required in these areas prior to final paving.

Boulders larger than 300 mm in diameter will also interfere with the backfill compaction and should be removed from the excavated material prior to re-use as backfill.

Where the trench will be covered with hard surfaced areas in the future, 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 backfills should be placed in maximum 300 mm thick lifts and should be compacted to at least 95 % of the material’s standard Proctor maximum dry density. As discussed above, some of the excavated materials will be quite wet and difficult to compact and may need to be wasted and replaced with drier materials.

6.7 Excavation

The groundwater level at the site was generally reported to be between about 5 to 7 m deep, i.e., between elevations of about 75 and 78 m within the South Area and between elevations of about 85 and 89 m within the North Area.

Excavations for the construction of the residential homes and the apartment buildings would likely be carried out within the fill materials above the groundwater level; however, the invert for the sanitary sewer is proposed at depths ranging from about 6.5 to 6.6 m depth below the existing ground surface (i.e., elevation about 84.6 to 86.5 m).

Based on the proposed invert depths, excavations for the construction of the sewers will be through fill, and along the north end of the alignment, between boreholes 17-201 to 17-204, possibly into the native sand and gravel deposits. The excavations will generally extend about 1 to 2 m below the measured groundwater level.

No unusual problems are anticipated in excavating (or trenching) in the overburden using conventional hydraulic excavating equipment, recognizing that significant cobble and boulder removal should be expected within the fill materials. Boulders larger than 0.3 m in diameter should be removed from the excavation side slopes for worker safety. In accordance with the Occupational Health and Safety Act (OHSA) of Ontario, the soils above the water table at this site would generally be classified as Type 3 soils. Unsupported side slopes in the overburden above the water table may therefore be sloped at a minimum of 1 horizontal to 1 vertical. However, in accordance with the OHSA of Ontario, the soils below the water table would generally be classified as Type 4 soils, and excavation side slopes must be sloped at a minimum of 3 horizontal to 1 vertical or be carried out within protective trench boxes.

6.8 Groundwater Inflow Control

6.8.1 Site Services

As noted in Section 6.7, the excavation for the site services may extend slightly below the existing groundwater level at the site. The fill and native sand and gravel deposits at the site are considered to have a relatively high hydraulic conductivity (although a hydrogeology assessment was not part of the current scope of work). Therefore, where excavations below the groundwater level are required, it may be necessary to lower the groundwater level in advance of excavation by first pumping from sumps excavated around the excavation. For deeper excavations, an active dewatering program could be needed such as pumping from wells or well points around the excavation.

Under the new regulations, a Permit-To-Take-Water (PTTW) is required from the Ministry of the Environment, Conservation and Parks (MECP) if a volume of water greater than 400,000 litres per day is pumped from the excavations. 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. The groundwater level and hydrogeological conditions in this area should be confirmed to assess the need for a Permit-To-Take-Water. Based on the soil descriptions, the potential groundwater inflow could be significant, and a Permit-to-Take-Water would likely be required for excavations below the groundwater level.

6.8.2 Residential Houses and Apartment Buildings

Based on the groundwater level data, the excavations for the proposed residential homes and apartment buildings would be carried out above the groundwater level, and hence no significant issue with respect to groundwater control is generally anticipated.

If excavation needs to be carried out below groundwater level, then an active groundwater management program, such as pumping from wells or well points around the excavation, would be required. The rate of pumping could be very high. As discussed above, a Permit-To-Take-Water would need to be obtained from the MECP. An evaluation of the impacts of the groundwater level lowering on the settlement of surrounding structures would be required as part of that permit application. The disposal options for the pumped groundwater would also need to be evaluated. Given the permeable ground conditions and related issues, it is recommended that excavations below the groundwater level on this site, for both foundations and services, be avoided.

6.9 Foundation Options

Preliminary development plans indicate residential homes (single family and townhomes) proposed over North Area as well as some portion of the South Area. Four residential apartment buildings are also proposed along the southern boundary of the site (in the South Area). All of these buildings would be constructed within the table land.

As discussed earlier in this report, the random fill materials that cover most of this site are not suitable for the support of foundations. These materials are variable in composition and state of packing, and were placed under unknown and likely uncontrolled conditions. Foundations supported on these materials could be expected to undergo unpredictable, highly differential, and potentially large settlements. In general, it should be planned to:

1) Provide ground densification to the fill materials as described in Section 6.5; or,
2) Remove these materials from beneath structures and replace them with compacted engineered fills; or,
3) Extend the foundations through these materials to the more competent native soils/bedrock, i.e., use deep foundations.

The first option of ground improvement is likely the most feasible in the South Area where the fill material is the thickest. This will allow for the residential homes to be founded on conventional shallow footings at typical depths within the densified fill. WSP previously had preliminary discussions with a ground improvement subcontractor to assess the feasibility of undertaking Geopier Rammed Aggregate Pier Impact System or Geopier GeoConcrete Columns systems for the fill material at the site. Since the fill thickness is greater than about 10 m in a major portion of the South Area, it is expected that densification of the full thickness of the fill by either Dynamic Compaction or Rapid Impact Compaction may not be feasible.

The second option may be more feasible/applicable to the North Area where the fill materials are thinner. Depending on the design site grading and the design founding level for site services and residential homes, the founding levels at some locations may already be below the fill materials.

The third option will likely be required at the location of the apartment buildings in the South Area. For the apartment buildings proposed in the South Area, consideration should be given to supporting the buildings on the following deep foundation options:

- Driven steel piles (either pipe piles or H-piles) end-bearing on the bedrock (which is expected at about 30 m depth). It should be noted that the piles may however have difficulty penetrating the sand and gravel deposits to reach the bedrock surface at depth and may hang-up in the very dense portions of these deposits.
- Cast-in-place concrete caissons, socketed into the bedrock at depth. However, this system is unlikely to be economical considering the significant depth to bedrock at this site.

The choice of foundation type will likely depend on the particular subsurface conditions at each building location and the required capacities. It is understood that a subsurface investigation (to bedrock surface) will be carried out in future (after the construction of Phase 1, i.e., residential homes) at the site of the proposed apartment buildings based on which the detailed foundation design will be provided for these buildings. However, some preliminary guidance has been provided in the subsection below.

6.9.1 Shallow Foundations on Engineered Fill

In the North Area where the residential homes are proposed, the fill thickness generally ranges from approximately 3 to 6 m. Consideration could be given to sub-excavating the fill and replacing with compacted engineered fill. The surface of the native subgrade should be proof rolled prior to placement of engineered fill to identify soft areas that will require sub-excavation and replacement with engineered fill. The engineered fill should consist of OPSS Granular A or B Type II, should be placed in maximum of 300 mm thick lifts, and should be compacted to at least 95 % of the material's standard Proctor maximum dry density using suitable vibratory compaction equipment. The engineered fill must be placed within the zone of influence of the foundations. The zone of influence is considered to extend out and down from the edge of the footings at a slope of 1 horizontal to 1 vertical.

The single family and townhomes can then be supported on shallow footings founded on the compacted engineered fill. For the preliminary design of typical residential houses, strip footing foundations, up to 1 m in width, can be designed using a maximum allowable bearing pressure of 100 kPa, consistent with design in accordance with Part 9 of the Ontario Building Code. However, this value should be reassessed at the stage of detailed design after the ground improvement program is completed and when a grading plan for founding and finished elevations for each residential block is available.

The post-construction total and differential settlements of footings supported on soil and sized using the above maximum allowable bearing pressure should be less than 25 and 15 mm, respectively, provided that the soil at or below founding level is not disturbed before or during construction.

6.9.2 Shallow Foundations with Ground improvement

If ground improvement methods are used on the site, to densify the fill materials and to reduce the total and differential settlements to levels which might feasibly be tolerated, the proposed single-family homes and townhomes may be able to be supported on shallow footings placed on or within the improved fill materials.

The use of Rammed Aggregate Pier (RAP) or GeoConcrete Columns (GCC) would be a feasible ground improvement method for this site. RAP and GCC are propriety systems developed by Geopier Foundation Company Inc. RAP soil reinforcing elements using the Geopier installation methodology are installed by drilling 0.76 m diametre cavity and ramming thin lifts of well graded aggregates within the cavity to form very stiff high density aggregate piers. The drilled holes are typically placed at 2 m spacing and can extend to depths of up to about 15 to 20 m. Geopier GCC involves the installation of concrete columns within the soil by pumping ready-mix concrete into the soil under pressure.

The result of Geopier RAP or GCC installation is a significant strengthening and stiffening of subsurface soils that then support shallow foundations and floor slabs.

If Geopier RAP or GCC are used to treat the soils at the site, an engineered fill granular pad will be required to “bridge” the foundation loads to these foundation elements. The thickness of the granular pad will depend on the foundation loads and spacing between these foundation elements.

Based on a preliminary discussion with Geopier Foundation Company Inc., if Geopier RAPs are installed, the net bearing resistance at Serviceability Limit States (SLS) for spread footing foundations founded on the piers may be taken as 150 kPa. The factored bearing resistance at Ultimate Limit States (ULS) may be taken as 250 kPa.

6.9.3 Piled Foundations

At the proposed apartment towers, where the fill materials are thicker, a piled foundation system could be used to transfer the foundation loads through the fill to more competent bearing at depth (i.e., to the dense to very dense sand and gravel or down to the bedrock surface). The use of a piled foundation would avoid the structure experiencing any significant total or differential settlement (for both static and seismic loading conditions).

A suitable pile type would be concrete filled steel pipe piles (driven closed-ended) or H-piles. For this site, the piles would be driven to practical refusal on the bedrock surface which is expected to be at an elevation of about 60 to 65 m.

The sand and gravel that overlie the bedrock is very dense. Pipe piles should be equipped with a base plate having a thickness of at least 20 mm to limit damage to the pile tip during driving. If H-piles are used, the piles should be provided with Titus-type bearing points or equivalent to protect the pile tips during driving. It is expected that some of the piles may have difficulty penetrating to the bedrock at depth and may ‘hang up’ at shallower depth in the very dense sand and gravel; diamond drilling techniques were required to penetrate through the sand and gravel in some of the boreholes. These piles (which hang up in the overburden material) might therefore have a lesser geotechnical capacity. Alternatively, pre-drilling of the overburden could be considered, wherever the piles do not initially reach the bedrock surface.

6.9.3.1 Axial Resistance

As one possible design example, the Ultimate Limit States (ULS) factored *structural* resistance of a 245 mm diameter steel pipe pile with a wall thickness of 12 mm may be taken as 1,500 kN. The ULS factored *geotechnical* resistance of the pile should equal or exceed the structural resistance if the piles are driven to the bedrock and are installed using an appropriate set criterion and using a hammer of sufficient energy. The pile capacity/size to be used in the design may also be controlled by the dynamic testing program (see later discussion in this section).

H-piles, although typically more expensive, could also be considered due to their possible greater likelihood of penetrating the dense soils at depth and reaching bedrock. The ULS factored *structural* resistance of an HP 310 x 110 pile may be taken as 2,000 kN. The ULS factored *geotechnical* resistance of the pile should equal or exceed the structural resistance if the piles are driven to the bedrock and are installed using an appropriate set criterion and using a hammer of sufficient energy.

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

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

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

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

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

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

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

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

For example, for the PDA testing to be able to record/confirm a factored *geotechnical* resistance of 1,500 kN (per the previously indicated pipe pile design example), it will be necessary to successfully proof load the tested piles to 3,000 kN during the PDA testing (per the resistance factor of 0.5 to be applied to PDA test results, as specified in Commentary K of the National Building Code of Canada). However, that proof load may exceed the actual structural capacity of the piles. If the piles fail (structurally) at a lower load, then the full geotechnical capacity cannot be confirmed (and piles will have been damaged and will need to be wasted).

The following options could therefore be considered:

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

2) A reduced ULS factored geotechnical resistance could be used for the design (e.g., 1,000 kN instead of 1,500 kN), such that the piles would have sufficient structural capacity to be loaded to twice the design geotechnical resistance. This option would again increase the cost for the piled foundations, by increasing the number of piles that would be required.

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

As discussed previously, the piles may not fully penetrate the very dense sandy deposits to reach the bedrock surface; some of the piles may ‘hang up’ at a shallower depth in these layers. In that case, pre-drilling of these layers, where the piles do not initially reach the bedrock surface, could be considered. However, this option would likely be costly.

Alternatively, the piles may need to be designed for a reduced capacity. The ULS factored axial resistance of these piles will depend on the depth to which they penetrate and the set that is achieved. The capacities of these piles will have to be confirmed in the field by carrying out load testing. As a preliminary guideline, for a single HP 310 x 110 pile, or a 245 mm diameter steel pipe pile with a wall thickness of 12 mm, founded within the very dense sandy deposit or sand and gravel, a ULS factored geotechnical resistance of 1,400 kN may be used. The axial resistance at SLS for 25 mm of settlement would likely be in the order of 1,100 kN.

Consideration could also be given to using this lower capacity for general design purposes, and thereby limit the potential need for additional piles should refusal in the overburden materials occur.

Friction piles could also be considered, which would need to penetrate only the upper portions of the dense sandy deposit and would therefore have less difficulty penetrating to the required depth. However, these piles would have a much lower capacity and this option is not considered to be cost effective.

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.

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

6.10 Floor Slab Construction

Floor slabs should not be constructed on the unimproved fill materials. Excessive settlement could occur for floor slabs constructed on the fill materials. The fill materials could alternatively be densified (per the ground improvement program described in Sections 6.5 and 6.9 of this report) or, where feasible, subexcavated and replaced with compacted engineered fill.

For predictable performance of the floor slabs for the single-family homes and townhouses, the existing topsoil and fill materials containing deleterious materials (i.e., organic matter) should be removed from within the proposed building areas. Provision should be made for at least 200 mm of OPSS Granular A to form the base for the floor slabs. Any bulk fill required to raise the grade to the underside of the Granular A should consist of OPSS Granular B Type II. The underslab fill should be placed in maximum 300 mm thick lifts and should be compacted to at least 95 % of the material's standard Proctor maximum dry density using suitable vibratory compaction equipment.

Within the North Area, if the residential homes are provided with basement levels, it may be feasible to construct the slabs as slabs-on-grade on the native competent sand. However, in the South Area, where there exists up to about 20 m of fill, construction of slabs-on-grade would require densification of under-slab fill, or structural slabs could be used.

If the foundations are supported on piles, the structure should be provided with a structural floor slab, which derives its support from the pile foundations. Consideration should be given to placing a granular working pad over the footprint area upon which the structural floor slab will be constructed. For example, a 150 mm thickness of OPSS Granular A might be suitable.

6.11 Frost Protection

The soils on this site are considered to be frost susceptible. Therefore, all exterior perimeter foundation elements or foundation elements in unheated areas should be provided with a minimum of 1.5 m of earth cover for frost protection purposes. Isolated, unheated exterior footings adjacent to surfaces which are cleared of snow cover during winter months should be provided with a minimum of 1.8 m of earth cover.

Insulation of the bearing surface with high density insulation could also be considered as an alternative to earth cover for frost protection. Where that option would be considered, further geotechnical input would need to be provided.

6.12 Foundation Wall Backfill

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

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

The granular backfill against the foundation walls should be drained by means of a perforated pipe subdrain in a surround of 19 mm clear stone, fully wrapped in geotextile, which leads by gravity drainage to a positive outlet.

6.13 Material Reuse

The fill materials as well as the native silts, sands, and gravel are not considered to be generally suitable for reuse as structural/engineered fill. Within all building areas (including pavements and services), imported engineered fill should be used for construction as recommended in other sections of this report. However, the existing soils at the site (native or fill) may be used for rough grading of the site prior to (and in preparation of) the ground improvement program.

Reference should be made to the Phase II Environmental Site Assessment for guidelines on the reuse of materials on site. The recommendations can be found in the following report:

- Report to St. Mary's Land Corporation titled "*Phase Two Environmental Site Assessment, Proposed Development at Riverside Drive and Hunt Club Road, Ottawa, Ontario*" dated January 2018 (Report No. 1670692-3000).
- Report to Taggart Realty titled "*Phase Two Environmental Site Assessment, Proposed Development at Riverside Drive and Hunt Club Road, Ottawa, Ontario*" dated December 2022 (Report No. 21482114).

6.14 Corrosion and Cement Type Testing

Samples of soil from current boreholes 23-01 to 23-04 and previous boreholes 17-202, 17-204, and 17-207 were submitted to Eurofins Environmental Testing for basic chemical analysis related to potential corrosion of buried steel elements and potential sulphate attack on buried concrete elements. The results of this testing completed on soil samples from current and previous investigation boreholes are provided in Appendices I and J, respectively and are also summarized below.

Borehole/ Sample Number	Geological Unit	Depth (m)	pH	Sulphates (%)	Chlorides (%)	Resistivity (Ohm-cm)
23-01 / 4	Fill	2.3 – 2.9	7.77	0.03	0.002	2041
23-02 / 5	Fill	3.1 – 3.7	7.96	<0.01	<0.002	5882
23-03 / 5	Fill	3.1 – 3.7	7.67	0.01	0.004	4000
23-04 / 3	Fill	1.5 – 2.1	7.73	<0.01	0.002	6250
17-202 / 4	Fill and Sand	4.6 – 5.2	8.1	<0.01	<0.002	10,000
17-204 / 3	Fill	3.1 – 3.7	8.3	<0.01	<0.002	8,330
17-207 / 3	Fill	3.1 – 3.7	7.6	0.04	<0.002	2,630

The concentration of soluble 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 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 this site. Accordingly, Type GU Portland cement should be acceptable for buried concrete substructures. The pH, resistivity and chloride concentration provide an indication of the degree of corrosiveness of the sub-surface environment. Generally, the test results indicate an elevated potential for corrosion of exposed ferrous metal at the site which should be considered in the design of substructures.

6.15 Pavement Design

In preparation for pavement construction, the topsoil should be excavated from all pavement areas. Typically, unsuitable fill material (e.g., those fill materials containing organic matter and debris, such as on this site) should also be excavated from the pavement areas. However, given the extensive thickness of fill over some areas of the site, removing this material and replacing with an engineered fill material would be impractical. As such, ground improvement is recommended.

Sections requiring grade raising to proposed subgrade level should be filled using acceptable (compactable and inorganic) earth borrow or OPSS Select Subgrade Material meeting the requirements of OPSS.MUNI 212 and 1010, respectively. The fill should be compacted to at least 95 % of standard Proctor maximum dry density up to 450 mm below subgrade. The upper 450 mm of the fill must be compacted to 100 % of SPMDD. The placement of the fill should be monitored by geotechnical personnel on a regular basis. Placement of the upper 450 mm should be monitored on a full-time basis.

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

Below paved areas, backfill for service trenches must consist of frost compatible material (native or fill) between the roadway subgrade level and the depth of seasonal frost penetration (i.e., 1.8 meters below finished grade). The backfill materials within this zone must match the materials exposed on the trench walls. The subsoil should be inspected by qualified geotechnical personnel to make sure that there is no potential for differential frost heaving.

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

The granular base and subbase materials should be uniformly compacted to at least 100 % of the material's standard Proctor maximum dry density using suitable vibratory compaction equipment. The asphaltic concrete should be compacted in accordance with Table 10 of OPSS.MUNI 310.

The below pavement design is based on the assumption that the pavement subgrade has been acceptably prepared (i.e., where the bottom of the excavation has been adequately compacted to the required density and the subgrade surface is not disturbed by construction operations or precipitation). Depending on the actual conditions of the pavement subgrade at the time of construction, it could be necessary to increase the thickness of the subbase. Additionally, a Class II woven geotextile conforming to OPSS 1860 should be provided under pavement areas to prevent pumping of the subgrade into the Granular B Type II subbase.

Table: Thickness of Pavement Elements

Material		Thickness of Pavement Elements (mm)	
		Light Duty	Heavy Duty
Asphaltic Concrete OPSS.MUNI 1151	Superpave 12.5 or HL 3 Surface Course	40	50
	Superpave 19.0 or HL 8 Binder Course	50	70
Granular Material OPSS.MUNI 1010 or City of Ottawa specification F3147	Granular A Base	150	150
	Granular B, Type II Subbase	600	750

7.0 ADDITIONAL CONSIDERATIONS

The soils at this site are sensitive to disturbance from ponded water, construction traffic and frost. Cobbles and boulders may be present in the native sand deposit and overlying fill.

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 filing or concreting to ensure that soil having adequate bearing capacity has been reached and that the bearing surfaces have been properly prepared. The placing and compaction of any engineered fill as well as sewer bedding and backfill should be inspected to ensure that the materials used conform to the specifications from both a grading and compaction viewpoint. Asphalt and concrete testing should be carried out in CCIL and CSA certified laboratories, respectively.

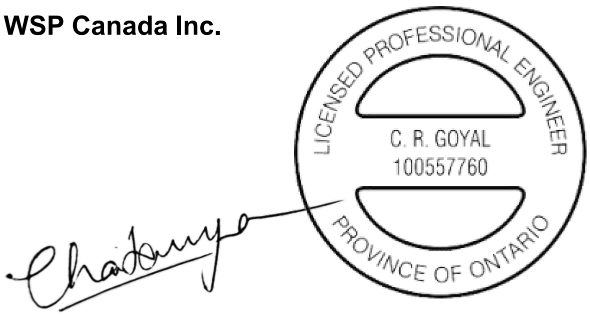
Piling operations should be inspected on a full-time basis by geotechnical personnel to monitor the pile locations and plumbness, initial sets, penetrations on restrike, and to check the integrity of the piles following installation.

The standpipe piezometers and wells installed at the site will ultimately require decommissioning in accordance with Ontario Regulation 128/03. However, the devices may be useful during construction, and it is expected that most of the wells will either be destroyed during construction or can be more economically abandoned as part of the construction contract.

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

Signature Page

WSP Canada Inc.



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Senior Geotechnical Engineer

CRG/CH/ljv/al

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IMPORTANT INFORMATION AND LIMITATIONS OF THIS REPORT

Standard of Care: WSP Canada Inc. (WSP) 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 practising 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 WSP by the Client. 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. WSP cannot be responsible for use of this report, or portions thereof, unless WSP 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 upon the reasonable request of the client, WSP may authorize in writing the use of this report by the regulatory agency as an Approved User for the specific and identified purpose of the applicable permit review process. Any other use of this report by others is prohibited and is without responsibility to WSP. The report, all plans, data, drawings and other documents as well as all electronic media prepared by WSP are considered its professional work product and shall remain the copyright property of WSP, 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 WSP. The Client acknowledges that electronic media is susceptible to unauthorized modification, deterioration and incompatibility and therefore the Client can not 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 WSP by the Client, communications between WSP and the Client, and to any other reports prepared by WSP 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. WSP can not 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 Ground Water 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, WSP 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 WSP 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.

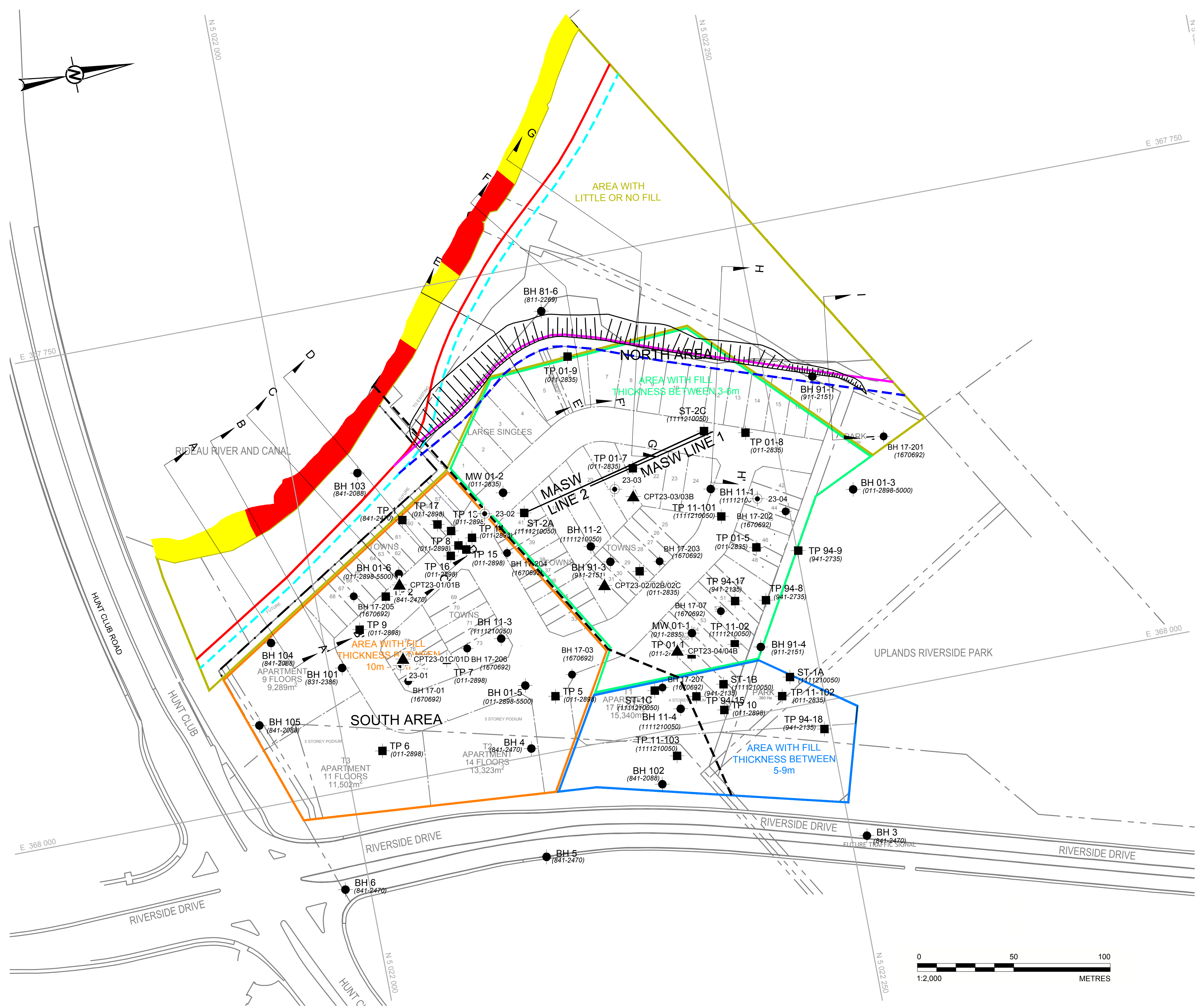
Sample Disposal: WSP will dispose of all uncontaminated soil and/or rock samples 90 days following issue of this report or, upon written request of the Client, will store uncontaminated samples and materials at the Client's expense. In the event that actual contaminated soils, fills or groundwater are 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. WSP 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, WSP 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 WSP 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 WSP 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 WSP 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. WSP takes no responsibility for the effects of drainage unless specifically involved in the detailed design and construction monitoring of the system.



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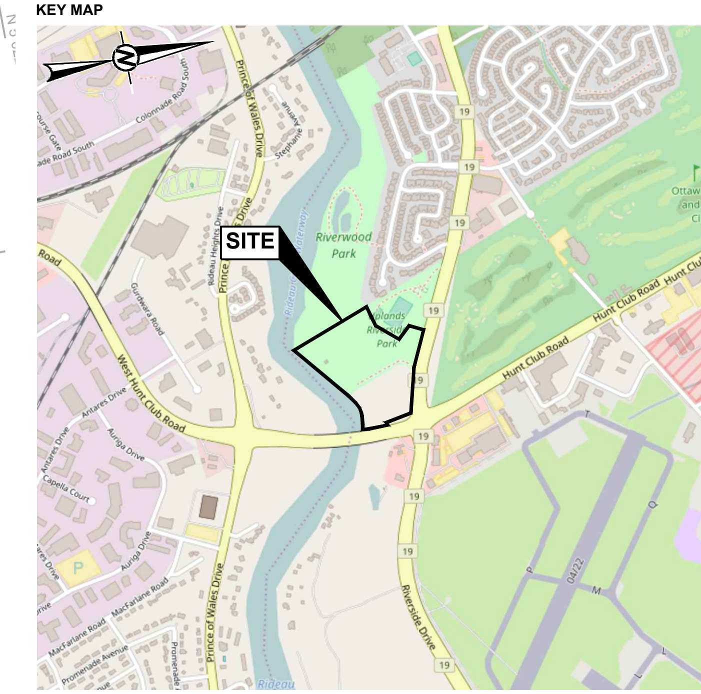

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LEGEND

- APPROXIMATE BOREHOLE LOCATION, CURRENT INVESTIGATION BY WSP
- APPROXIMATE CPT LOCATION, CURRENT INVESTIGATION BY CONETEC
- APPROXIMATE BOREHOLE LOCATION, PREVIOUS INVESTIGATION BY GOLDER
- APPROXIMATE TEST PIT LOCATION, PREVIOUS INVESTIGATION BY GOLDER
- A' APPROXIMATE SLOPE CROSS-SECTION LOCATION IN PLAN
- LIMIT OF HAZARD LANDS (STABLE SLOPE AND EROSION ALLOWANCE) LOWER SLOPE
- LIMIT OF HAZARD LANDS (ACCESS ALLOWANCE) LOWER SLOPE
- LIMIT OF HAZARD LANDS (STABLE SLOPE AND EROSION ALLOWANCE) UPPER SLOPE
- LIMIT OF HAZARD LANDS (ACCESS ALLOWANCE) UPPER SLOPE
- APPROXIMATE BOUNDARY BETWEEN ASSESSMENT AREAS
- APPROXIMATE SITE BOUNDARY
- SLOPE MODIFICATION AREA - 2.5H:1V
- AREA WITH MODERATE TO SEVERE EROSION
- AREA WITH MODERATE EROSION

REFERENCE(S)

1. BASE PLAN PROVIDED IN ELECTRONIC FORMAT BY HOBIN ARCHITECTURE INCORPORATED ON SEPTEMBER 25, 2023, DRAWING NO. Riverside_Site Plan_230920.dwg
2. PROJECTION: TRANSVERSE MERCATOR DATUM: NAD 83,COORDINATE SYSTEM: MTM ZONE 9, VERTICAL DATUM: CGVD28T

CLIENT
ST. MARY'S LANDS CORPORATION

PROJECT
GEOTECHNICAL ASSESSMENT
PROPOSED RESIDENTIAL DEVELOPMENT
3930 AND 3960 RIVERSIDE DRIVE, OTTAWA, ONTARIO

TITLE
SITE PLAN

CONSULTANT

wsp

YYYY-MM-DD	2023-09-26
DESIGNED	CRG
PREPARED	ABD/JM
REVIEWED	CRG
APPROVED	CH

PROJECT NO.	CONTROL	REV.	FIGURE
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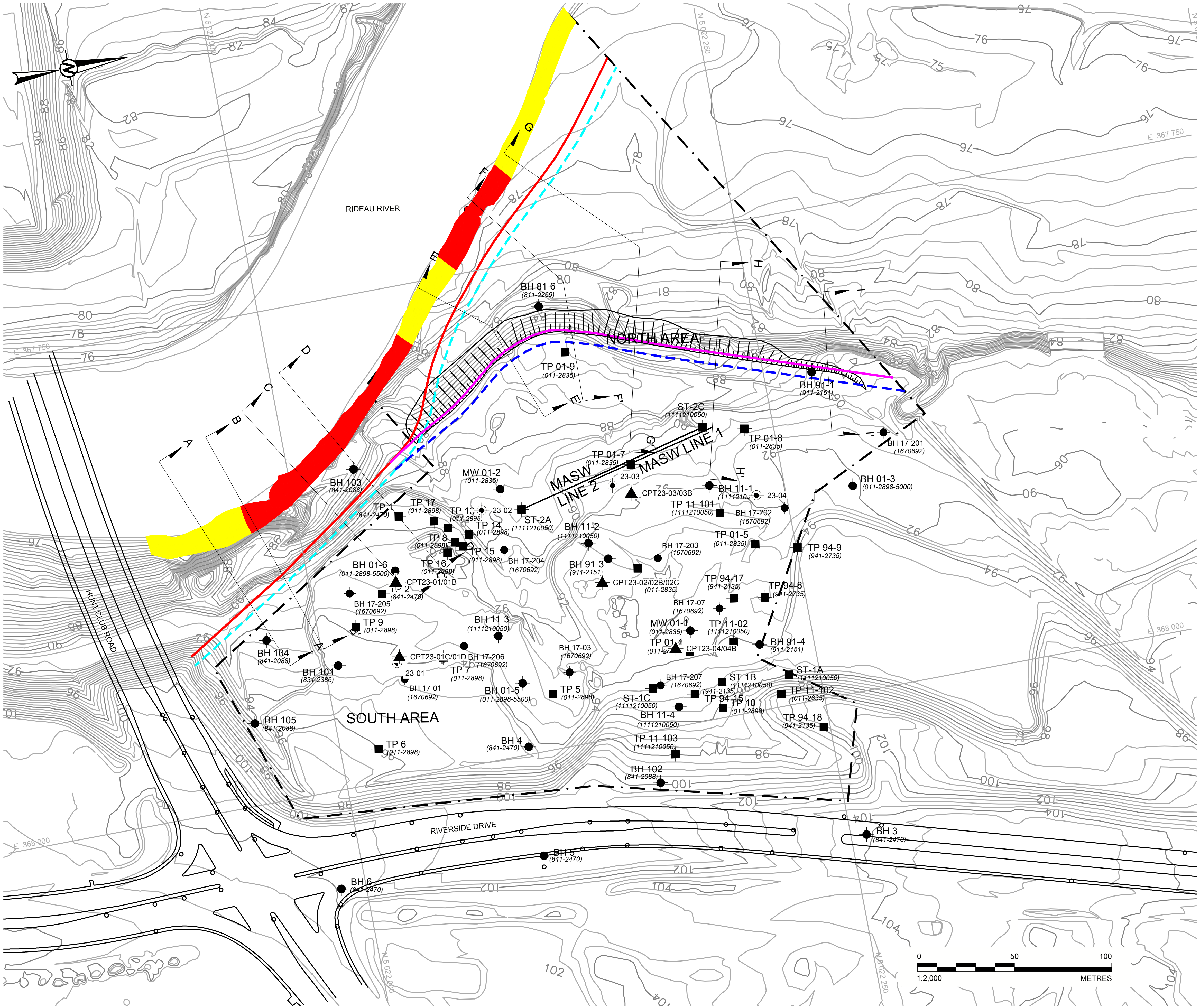

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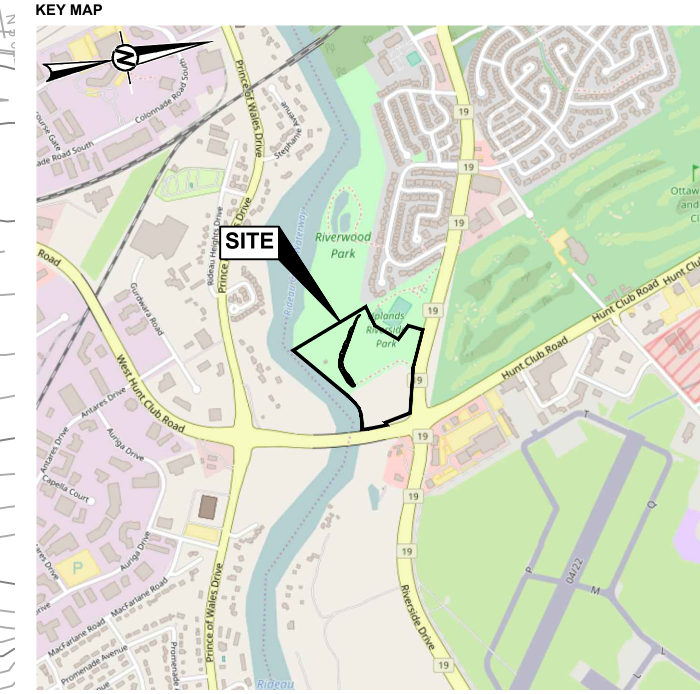

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LEGEND

- APPROXIMATE BOREHOLE LOCATION, CURRENT INVESTIGATION BY WSP
- APPROXIMATE CPT LOCATION, CURRENT INVESTIGATION BY CONETEC
- APPROXIMATE BOREHOLE LOCATION, PREVIOUS INVESTIGATION BY GOLDER
- APPROXIMATE TEST PIT LOCATION, PREVIOUS INVESTIGATION BY GOLDER
- APPROXIMATE SLOPE CROSS-SECTION LOCATION IN PLAN
- LIMIT OF HAZARD LANDS (STABLE SLOPE AND EROSION ALLOWANCE) LOWER SLOPE
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- APPROXIMATE SITE BOUNDARY
- SLOPE MODIFICATION AREA - 2.5H:1V
- AREA WITH MODERATE TO SEVERE EROSION
- AREA WITH MODERATE EROSION

REFERENCE(S)

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

CLIENT
ST. MARY'S LANDS CORPORATION

PROJECT
GEOTECHNICAL ASSESSMENT
PROPOSED RESIDENTIAL DEVELOPMENT
3930 AND 3960 RIVERSIDE DRIVE, OTTAWA, ONTARIO

TITLE
SITE PLAN

CONSULTANT

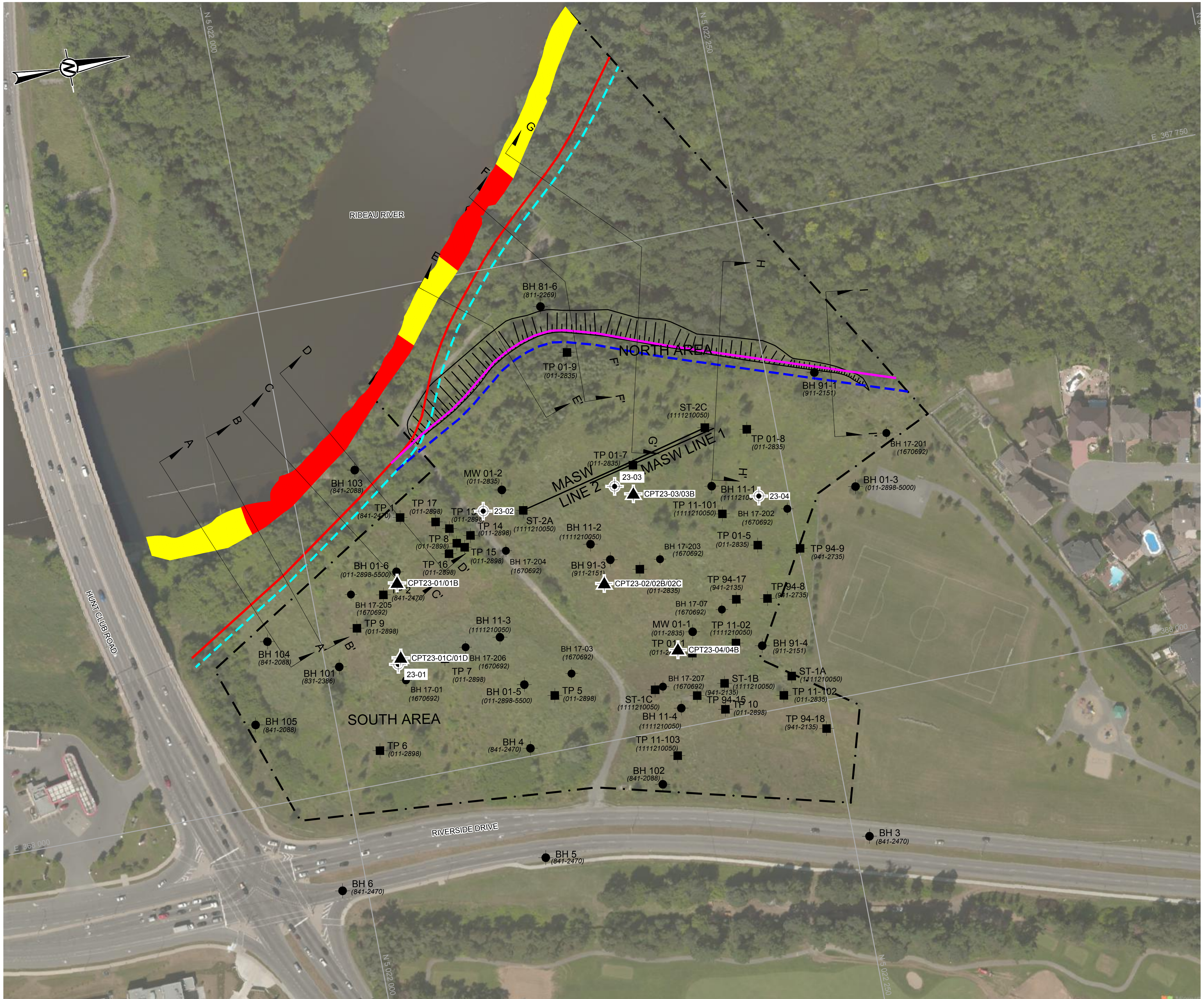
WSP

YYYY-MM-DD	2023-09-26
DESIGNED	CRG
PREPARED	ABD/JM
REVIEWED	CRG
APPROVED	CH

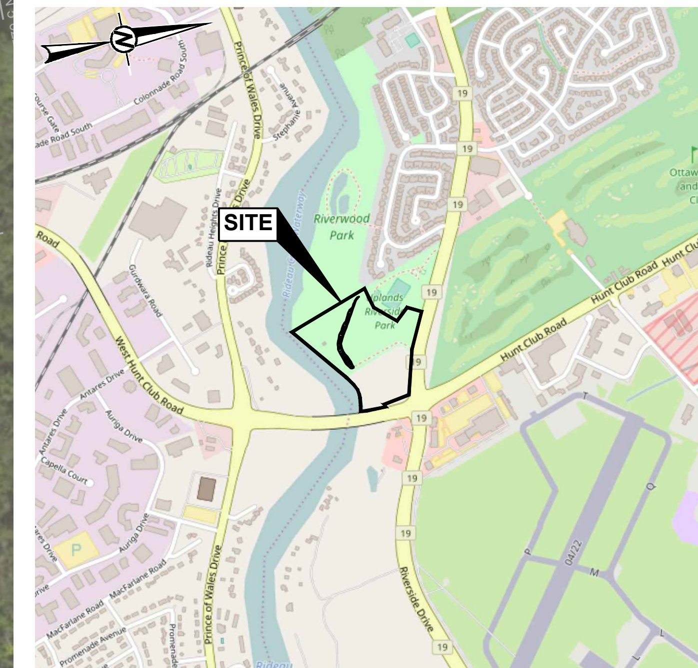
PROJECT NO.	CONTROL	REV.	FIGURE
21482114	0005	0	1A

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

25 mm



KEY MAP



SCALE 1:25,000

LEGEND

- APPROXIMATE BOREHOLE LOCATION, CURRENT INVESTIGATION BY WSP
- APPROXIMATE CPT LOCATION, CURRENT INVESTIGATION BY CONETEC
- APPROXIMATE BOREHOLE LOCATION, PREVIOUS INVESTIGATION BY GOLDER
- APPROXIMATE TEST PIT LOCATION, PREVIOUS INVESTIGATION BY GOLDER
- A' APPROXIMATE SLOPE CROSS-SECTION LOCATION IN PLAN
- LIMIT OF HAZARD LANDS (STABLE SLOPE AND EROSION ALLOWANCE) LOWER SLOPE
- LIMIT OF HAZARD LANDS (ACCESS ALLOWANCE) LOWER SLOPE
- LIMIT OF HAZARD LANDS (STABLE SLOPE AND EROSION ALLOWANCE) UPPER SLOPE
- LIMIT OF HAZARD LANDS (ACCESS ALLOWANCE) UPPER SLOPE
- APPROXIMATE SITE BOUNDARY
- SLOPE MODIFICATION AREA - 2.5H:1V
- AREA WITH MODERATE TO SEVERE EROSION
- AREA WITH MODERATE EROSION

REFERENCE(S)

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

0 50 100

1:2,000 METRES

CLIENT
ST. MARY'S LANDS CORPORATION

PROJECT
GEOTECHNICAL ASSESSMENT
PROPOSED RESIDENTIAL DEVELOPMENT 3930
AND 3960 RIVERSIDE DRIVE, OTTAWA, ONTARIO

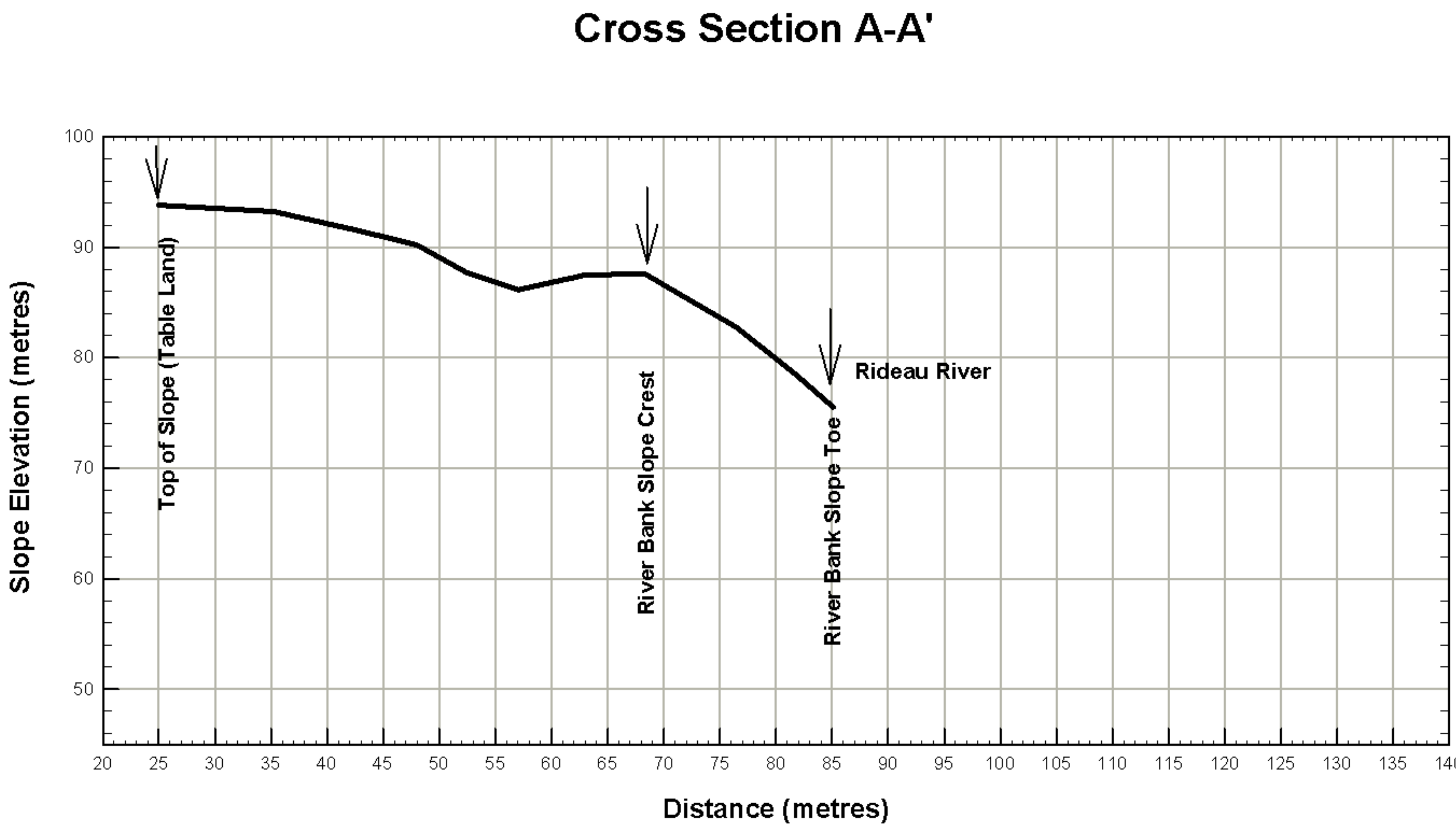
TITLE
SITE PLAN

CONSULTANT
wsp

YYYY-MM-DD	2023-09-26
DESIGNED	CRG
PREPARED	ABD/JM
REVIEWED	CRG
APPROVED	CH

PROJECT NO.	CONTROL	REV.	FIGURE
21482114	0005	0	1B

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



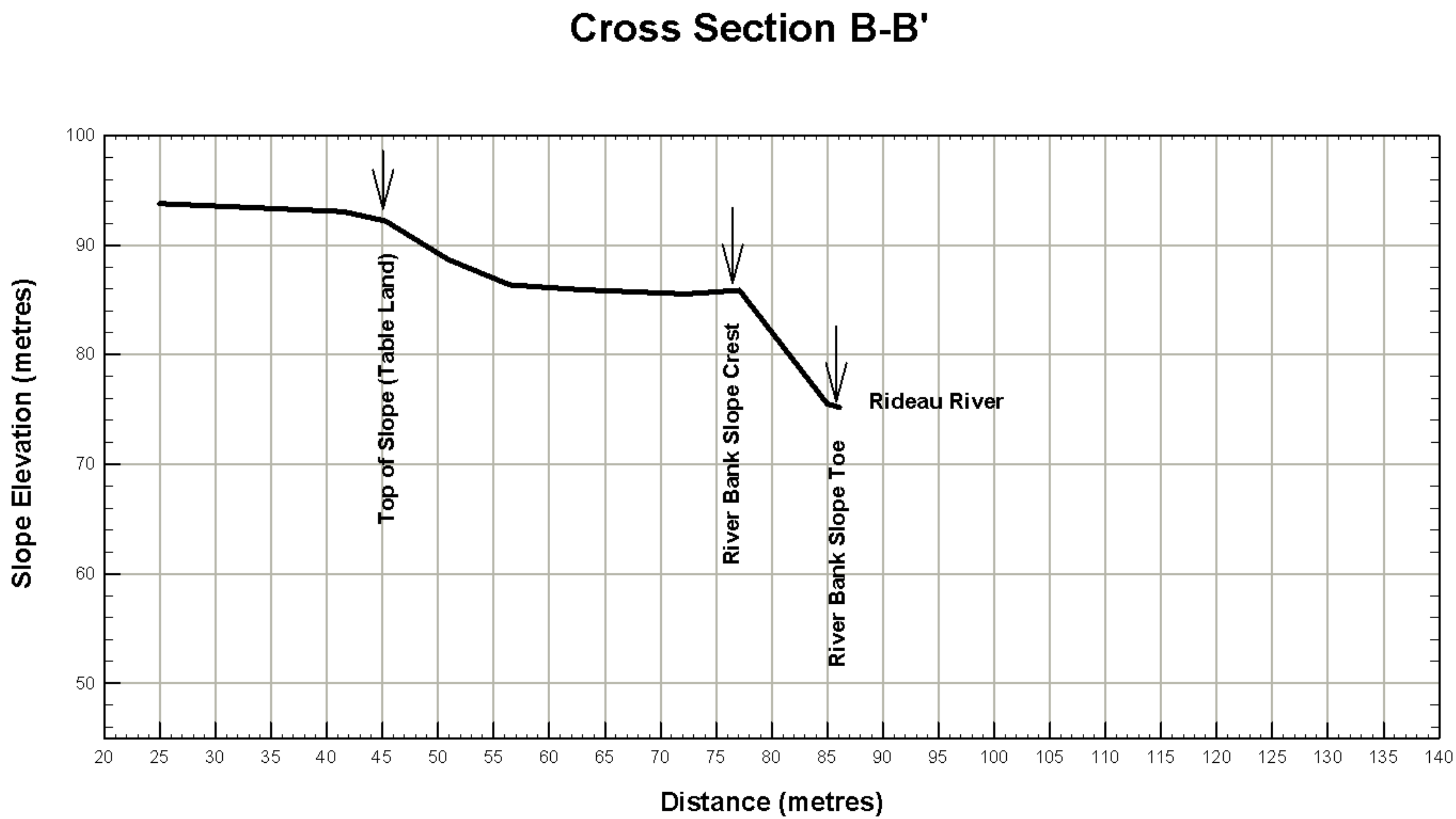
Golder Associates

Date: May 8, 2017 Drawn: WAM

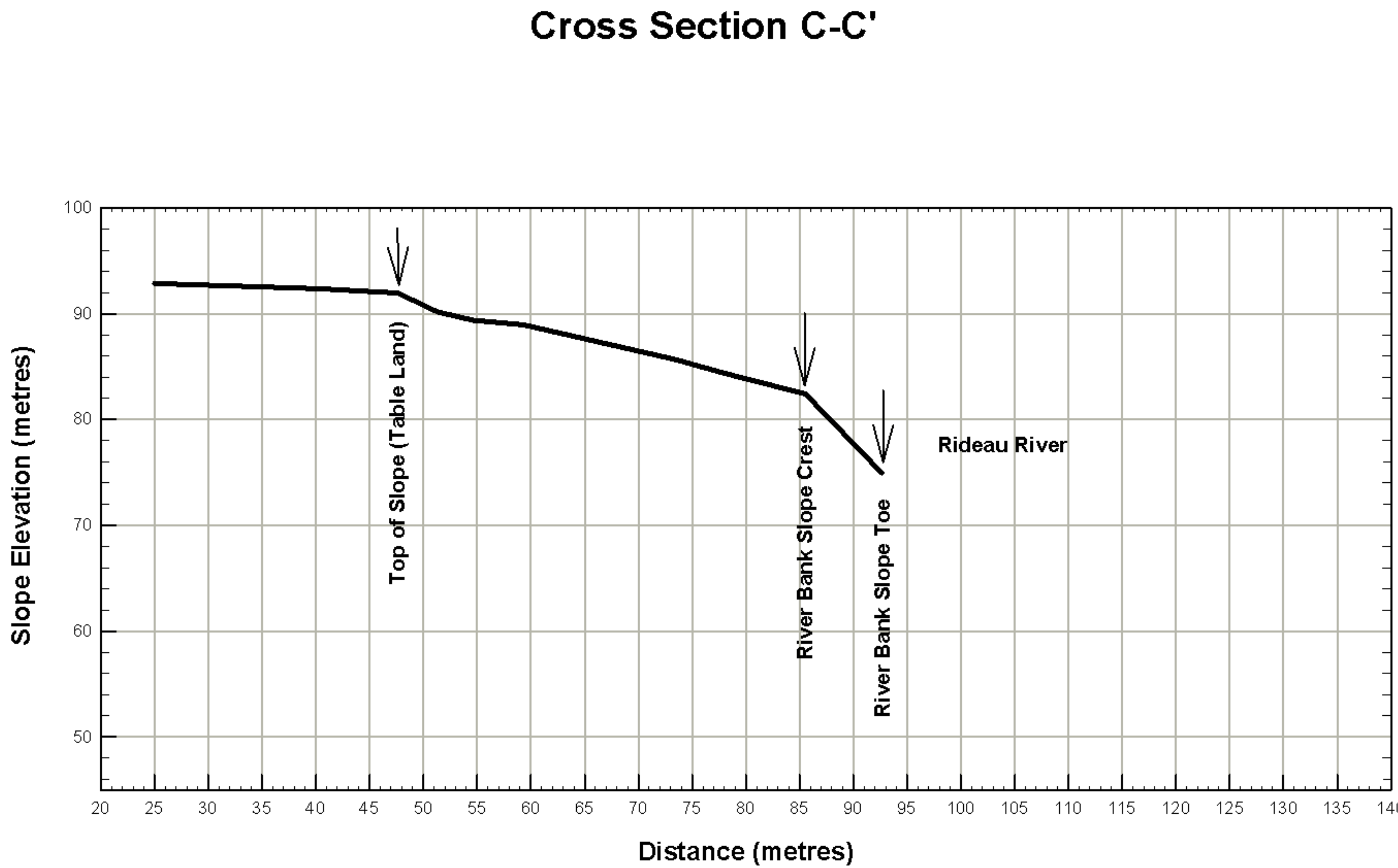
Project: 1670692 Chkd:

SLOPE CROSS SECTION

FIGURE 2



Golder Associates	Date: May 8, 2017 Drawn: WAM Project: 1670692 Chkd:	**SLOPE CROSS SECTION**	**FIGURE 3**



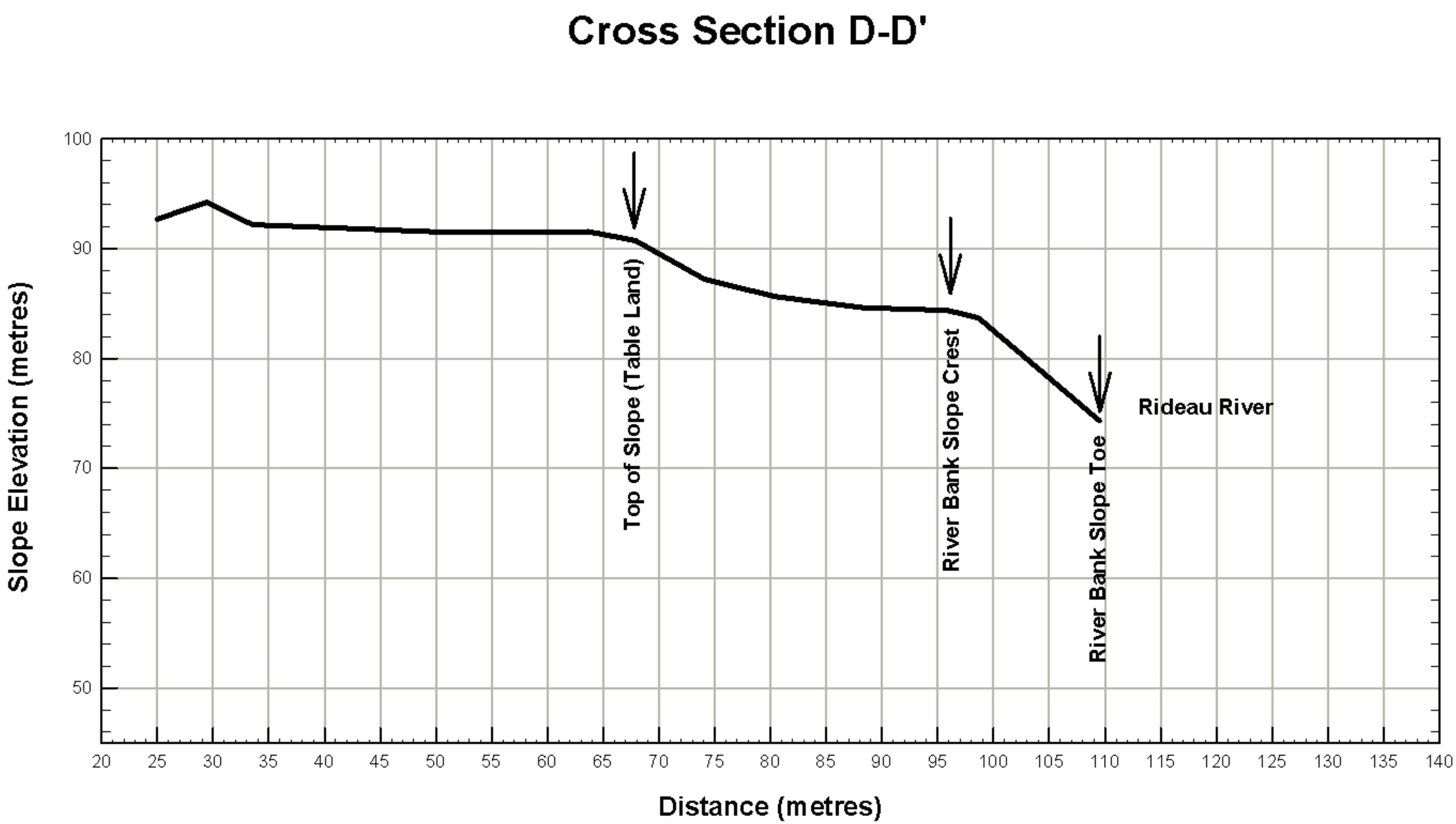
Golder Associates

Date: May 8, 2017 Drawn: WAM

Project: 1670692 Chkd:

SLOPE CROSS SECTION

FIGURE 4



Golder Associates

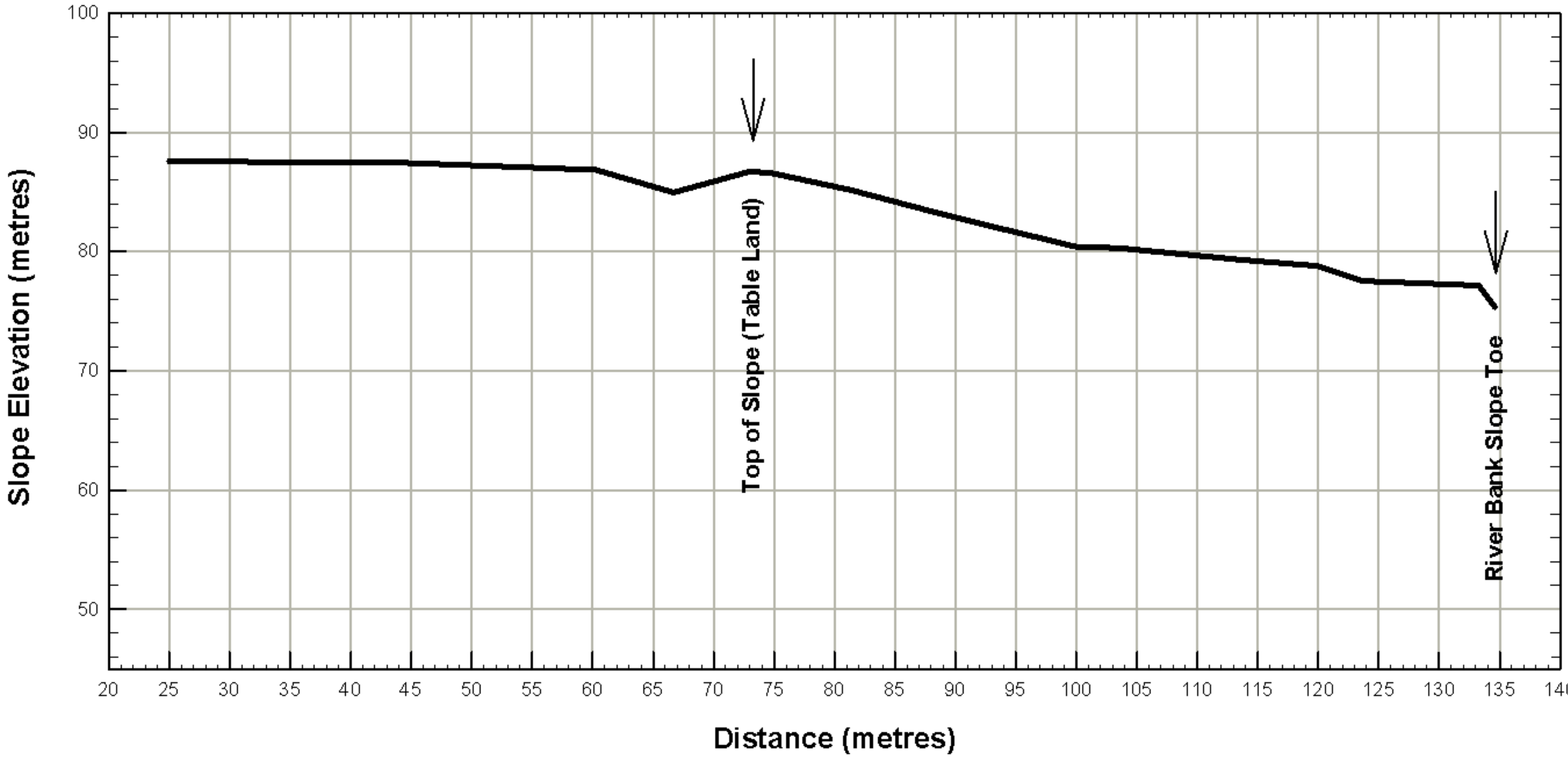
Date: May 8, 2017 Drawn: WAM

Project: 1670692 Chkd:

SLOPE CROSS SECTION

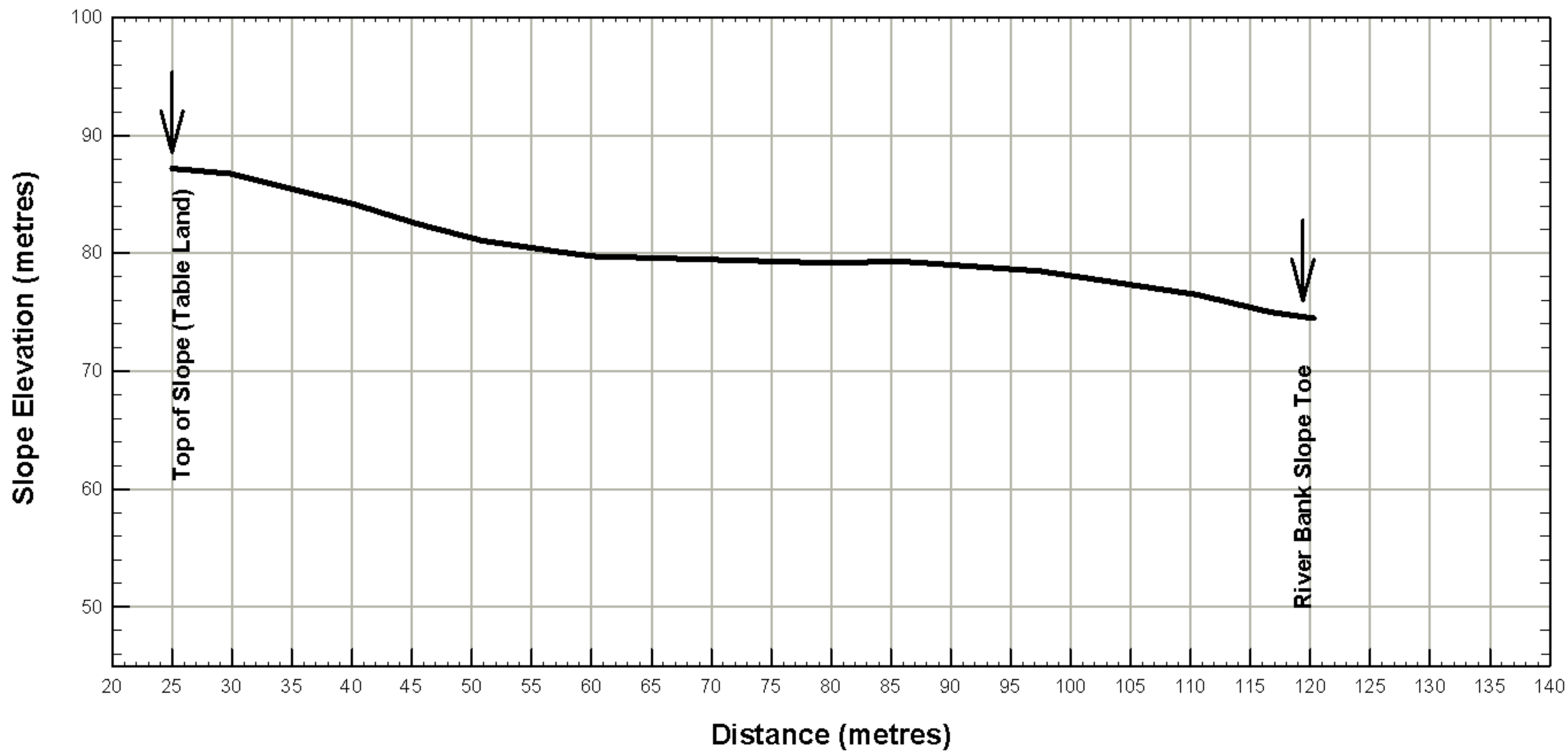
FIGURE 5

Cross Section E-E'

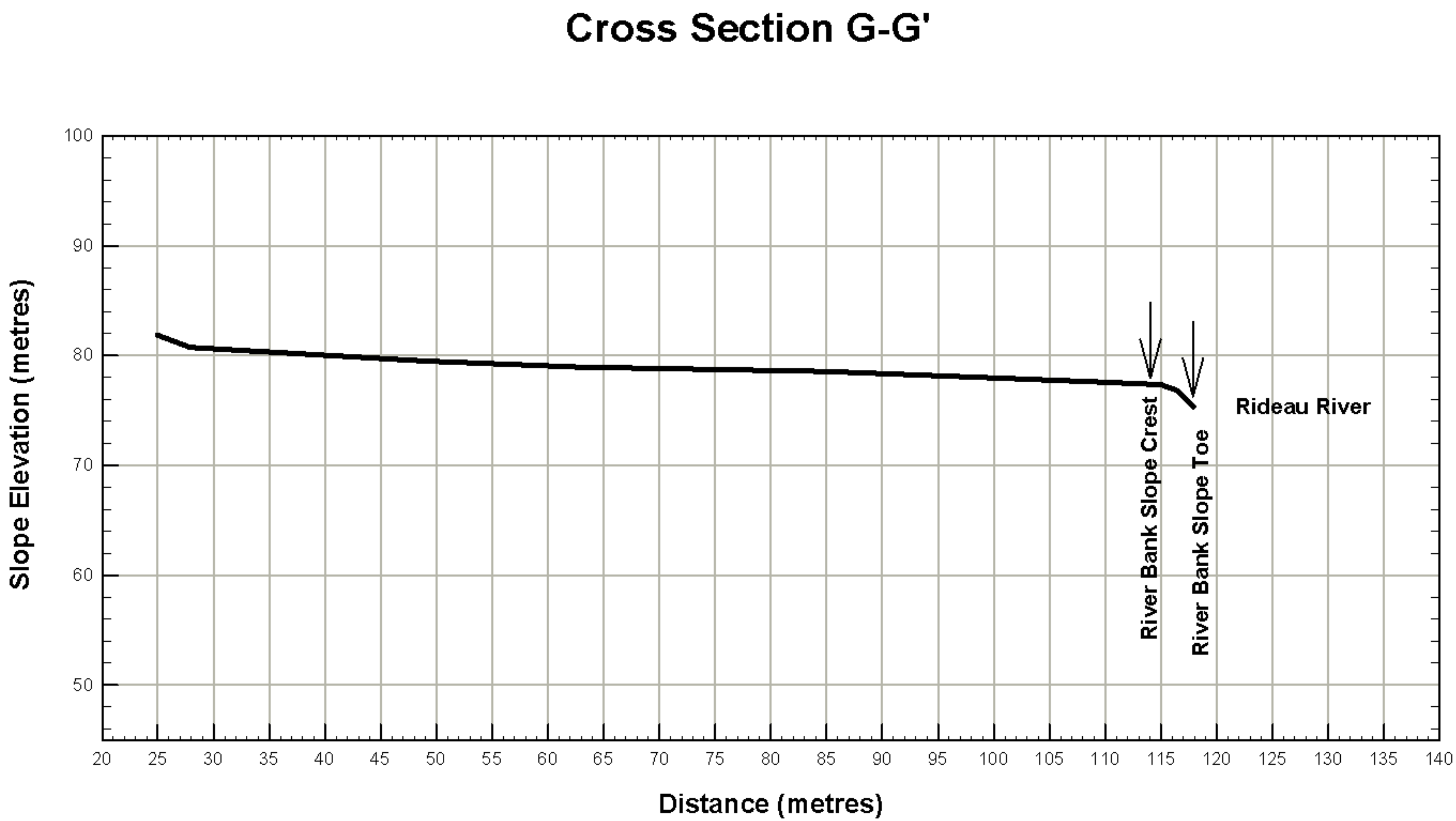


Golder Associates	Date: May 8, 2017 Drawn: WAM Project: 1670692 Chkd:	SLOPE CROSS SECTION	FIGURE 6

Cross Section F-F'



Golder Associates	Date: May 8, 2017 Drawn: WAM Project: 1670692 Chkd:	**SLOPE CROSS SECTION**	**FIGURE 7**



Golder Associates

Date: May 8, 2017 Drawn: WAM

Project: 1670692 Chkd:

SLOPE CROSS SECTION

FIGURE 8

APPENDIX A

Records of Current Borehole Logs

METHOD OF SOIL CLASSIFICATION

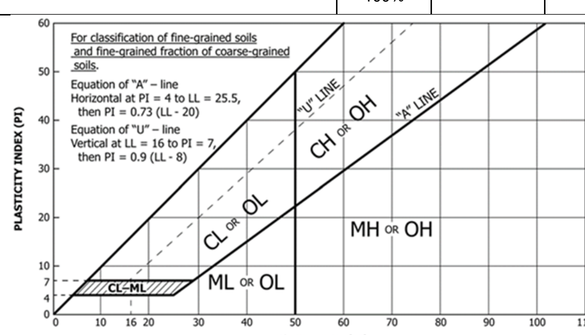
The WSP Canada Soil Classification[1] System is based on the Unified Soil Classification System (USCS) (after ASTM D2487)

Organic or Inorganic	Soil Group	Type of Soil		Gradation or Plasticity	$Cu = \frac{D_{60}}{D_{10}}$	$Cc = \frac{(D_{30})^2}{D_{10}xD_{60}}$	Organic Content [6,9]	USCS Group Symbol [3,5,7]	Primary Group Name [2]
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)	Clean Gravels with <5% fines [3] (by mass)	Well Graded	≥4 (and)	≥1 to ≤3	≤30%	GW	Well-graded GRAVEL [4,6]
				Poorly Graded	<4 (and/or)	<1 or >3		GP	Poorly graded GRAVEL [4,6]
			Gravels with >12% fines [3] (by mass)	Below A Line	n/a			GM	SILTY GRAVEL [4,6]
				Above A Line	n/a			GC	CLAYEY GRAVEL [4,5,6]
		SANDS (≥50% by mass of coarse fraction is smaller than 4.75 mm)	Clean Sands with <5% fines [7] (by mass)	Well Graded	≥6 (and)	≥1 to ≤3		SW	Well-graded SAND [6,8]
				Poorly Graded	<6 (and/or)	<1 or >3		SP	Poorly graded SAND [6,8]
			Sands with >12% fines [7] (by mass)	Below A Line	n/a			SM	SILTY SAND [6,8]
				Above A Line	n/a			SC	CLAYEY SAND [5,6,8]

Organic or Inorganic	Soil Group	Type of Soil	Laboratory Tests	Field Indicators: Dilatancy	Field Indicators: Dry Strength	Field Indicators: Shine Test	Field Indicators: Thread Diameter (mm)	Field Indicators: Toughness (of 3 mm thread)	Organic Content [B,H]	USCS Group Symbol [A]	Primary Group Name [A]
INORGANIC (Organic Content <30% by mass)	**FINE-GRAINED SOILS** (≥50% by mass is smaller than 0.075 mm)	SILTS (Nonplastic or PI and LL plot **below A-Line** on Plasticity Chart below) [C]	Liquid Limit <50 [D]	Rapid	None to Low	Dull to None	3 to >6	Low/can't roll 3 mm	<15%	ML	SILT [H]
				None to Slow	Low to Medium	Dull to Slight	3 to 6	Low	15% to 30%	OL	ORGANIC SILT
			Liquid Limit ≥50 [D]	None to V.Slow	Low to Medium	Slight	3 to 6	Low to Medium	<15%	MH	ELASTIC SILT [H]
				None	Medium to High	Dull to Slight	1 to 3	Low to Medium	15% to <30%	OH	ORGANIC SILT
		CLAYS (PI and LL plot **above A-Line** on Plasticity Chart below) [A]	Liquid Limit <50 [D]	None to Medium Slow	Medium to High	Slight to Shiny	1 to 3	Medium	<15%	CL	LEAN CLAY [A,E,F,G,H]
				None to V.Slow	Medium to High	Slight to Shiny	1 to 3	Medium	15% to <30%	OL	ORGANIC CLAY [E,F,G]
			Liquid Limit ≥50 [D]	None	High to V.High	Shiny	<1	High	<15%	CH	FAT CLAY [E,F,G,H]
				None	High	Shiny	<1 to 1	High	15% to <30%	OH	ORGANIC CLAY [E,F,G]
HIGHLY ORGANIC SOILS (Organic Content >30% by mass)		Peat and mineral soil mixtures	Relatively lightweight, possibly spongy. Some water may squeeze from sample. Some shrinkage may occur on air drying. Sand fraction may be visible. Low to high dilatancy. Thread weak near plastic limit. Low to medium dry strength.						30% to <75%	PT	SILTY PEAT, SANDY PEAT
		Predominantly peat, may contain some mineral soil, fibrous or amorphous peat	Lightweight, spongy. Much water squeezes from sample. Shrinks considerably on air drying (i.e., very high water content). Plant structure identiable to altered.						75% to 100%		PEAT

Coarse-Grained Soil Note(s):

1. Based on the material passing the 75 mm sieve.
2. If field sample contains or drilling observations indicate cobbles or boulders or both, add, "with cobbles" or "with cobbles and boulders". Include notes on the depth(s) encountered, and sizes if possible.
3. Gravels with 5% to 12% fines require dual symbols:
   (GW-GM) Well-graded GRAVEL with silt,
   (GW-GC) Well-graded GRAVEL with clay,
   (GP-GM) Poorly graded GRAVEL with silt,
   (GP-GC) Poorly graded GRAVEL with clay.
4. If soil contains ≥15% sand, add "with sand" to Group Name.
5. If fines classify as CL-ML, use dual symbol (GC-GM) or (SC-SM) for Group Symbol.
6. If the soil has an organic content (OC) 15%≤OC<30% the prefix "Organic" should be added before the Group Name. If the soil has an organic content 3%≤OC<15% add "with organic fines" to Group Name. If the soil contains >0% to ≤3% organics, the descriptor "trace organics" may be added.
7. Sands with 5% to 12% fines require dual symbols:
   (SW-SM) Well-graded SAND with silt,
   (SW-SC) Well-graded SAND with clay,
   (SP-SM) Poorly graded SAND with silt,
   (SP-SC) Poorly graded SAND with clay.
8. If soil contains ≥15% gravel, add "with gravel" to Group Name.



Fine-Grained Soil Note(s):

A. If Atterberg limits plot above the A-line but in the 'hatched' area on the plasticity chart, soil is a (CL-ML) SILTY CLAY.
B. If the soil contains >0% to ≤3% organics, the descriptor "trace organics" may be added.
C. If fine-grained materials are nonplastic (i.e., a plastic limit (PL) cannot be measured), soil is a (ML) SILT.
D. If soil has a liquid limit (LL) >30% to <50%, the term 'medium plasticity' may be included in the description, but the Group Name/Symbol is not changed.
E. If soil contains 15% to <30% +No.200, add "with sand" or "with gravel".
F. If soil contains ≥30% +No.200 mainly sand, add "Sandy" to Group Name.
G. If soil contains ≥30% +No.200 mainly gravel, add "Gravelly" to Group Name.
H. If the soil has an organic content (OC) 3%≤OC<15% add "with organic fines" to Group Name.

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)

GRADATIONAL COMPONENT TERMS

% (by mass)	Term
≤ 5	Use "trace"
> 5 to ≤ 12	Use "few"
> 12 to <30	Use "little"
≥ 30 to <50	Use "some"
≥ 50	Use "mostly"

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

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

SAMPLES

AS	Auger sample
BS	Block sample
CS	Chunk sample
DD	Diamond Drilling
DO or DP	Seamless open ended, driven, pushed tube sampler, or geoprobe macro-core – note size
DS	Denison type sample
FS	Foil Sample
GS	Grab Sample
MC	Modified California Samples – note sample diameter and hammer weight
MS	Modified Shelby (for frozen soil)
RC	Rock core
SC	Soil core
SS	Split-spoon sampler (50 mm OD); larger sizes use MC
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 general 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 general 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	nonplastic
w_s	shrinkage limit
I_L	liquidity index = $(w - w_p) / I_p$
I_C	consistency index = $(w_l - w) / I_p$
e_{max}	void ratio in loosest state
e_{min}	void ratio in densest state
I_D	density index = $(e_{max} - e) / (e_{max} - e_{min})$ (formerly relative density)

(b) Hydraulic Properties

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

(c) Consolidation (one-dimensional)

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

(d) Shear Strength

τ_p, τ_r	peak and residual shear strength
ϕ'	effective angle of internal friction
δ	angle of interface friction
μ	coefficient of friction = tan δ
c'	effective cohesion
c_u, s_u	undrained shear strength (ϕ = 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

RECORD OF BOREHOLE: BH23-01

PROJECT: 21482114
LOCATION: N 5020449.06; E 445590.99
SPT/DCPT HAMMER: MASS, 64kg; DROP, 760mm

BORING DATE: January 17 to 19, 2023
DRILL RIG: CME 850 Track Mount

SHEET 1 OF 3
DATUM: Geodetic
HAMMER TYPE: AUTOMATIC

DEPTH SCALE METRES	BORING METHOD	DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	NUMBER	TYPE	BLOWS/0.3m	WATER CONTENT (%)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		GROUND SURFACE		94.69						
0	200 mm Power Auger; 108 mm I.D. Hollow Stem	TOPSOIL - (CL) SILTY CLAY, some sand; dark brown; w>PL		0.00						
		FILL - (CL) SILTY CLAY, trace sand, trace gravel, contains organic matter and rootlets; grey-brown; cohesive, w>PL, very stiff		0.15	1	SS	7			Bentonite Seal
1		FILL - (SM) SILTY SAND, some gravel to gravelly, some low plasticity fines, contains organic matter; dark brown to brown; non-cohesive, moist to wet, loose to dense		93.78 / 0.91	2	SS	22			
					3	SS	7	~20	M (4 72 12 12)	
2–3					4	SS	33			
		FILL - (CL) SILTY CLAY, some sand, trace gravel, contains organic matter and cobbles; grey-brown; cohesive, w>PL, very stiff		91.34 / 3.35	5	SS	5			
4					6	SS	16			
5					7	SS	10	~22		Native Backfill
		FILL - (GM/SM) gravelly SILTY SAND to silty sandy GRAVEL, some low plasticity fines, contains brick and shale fragments, and organic matter; grey-brown to brown; non-cohesive, moist to wet, compact		89.36 / 5.33	8	SS	20			
6	Mud Rotary Drilling; HW Casing				9	SS	10			
7					10	SS	17	~17		
8					11	SS	14			
		FILL - (CL) SILTY CLAY, trace to some sand, trace gravel, contains seams of silty sand; grey-brown to grey; cohesive, w>PL, very stiff		86.16 / 8.53	12	SS	7			
9					13	SS	5	~32		Bentonite Seal
10					14	SS	9			Silica Sand

CONTINUED NEXT PAGE

DEPTH SCALE 1 : 50

wsp

LOGGED: RI
CHECKED: CRG

PROJECT: 21482114

RECORD OF BOREHOLE: BH23-01

LOCATION: N 5020449.06; E 445590.99

BORING DATE: January 17 to 19, 2023

DATUM: Geodetic

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

DRILL RIG: CME 850 Track Mount

HAMMER TYPE: AUTOMATIC

DEPTH SCALE METRES | BORING METHOD | SOIL PROFILE | SAMPLES | 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: Mud Rotary Drilling; HW Casing

--- CONTINUED FROM PREVIOUS PAGE ---

DESCRIPTION	ELEV. / DEPTH (m)	NUMBER	TYPE	BLOWS/0.3m	ADDITIONAL LAB. TESTING
FILL - (CL) SILTY CLAY, trace to some sand, trace gravel, contains seams of silty sand; grey-brown to grey; cohesive, w>PL, very stiff		14	SS	9	
FILL - (CL-SM) interbedded silty clay and silty sand, trace gravel to gravelly, contains organic matter, asphalt fragments, and wood pieces; grey-brown to grey to brown; cohesive and non-cohesive, w>PL (cohesive) and wet (non-cohesive), very stiff (cohesive) and compact	84.02 / 10.67	15	SS	10	
		16	SS	16	
		17	SS	7	M (12 24 32 32)
		18	SS	24	
		19	SS	26	
		20	SS	23	
		21	SS	18	
		22	SS	19	
FILL - (ML) sandy SILT, some gravel, some low plasticity fines, contains organic matter, wood pieces, brick fragments, and cobbles; dark brown to light brown to black; non-cohesive, wet, compact to very dense	77.93 / 16.76	23	SS	16	
		24	SS	27	
		25	SS	56	
		26	SS	22	
	74.88 / 19.81	27A			

Piezometer / standpipe installation: Screen; Feb 28, 2023 (water level); Silica Sand; Bentonite Seal; Feb 28, 2023 (water level); Bentonite Seal; Silica Sand; Screen

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

CONTINUED NEXT PAGE

DEPTH SCALE

1 : 50

LOGGED: RI

CHECKED: CRG

PROJECT: 21482114

LOCATION: N 5020449.06; E 445590.99

RECORD OF BOREHOLE: BH23-01

BORING DATE: January 17 to 19, 2023

DATUM: Geodetic

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

DRILL RIG: CME 850 Track Mount

HAMMER TYPE: AUTOMATIC

DEPTH SCALE METRES	BORING METHOD	DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	NUMBER	TYPE	BLOWS/0.3m	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
		--- CONTINUED FROM PREVIOUS PAGE ---									
20	Mud Rotary Drilling, HW Casing	FILL - (ML) CLAYEY SILT, trace gravel, some sand, contains wood pieces and organic matter; dark brown; cohesive, w>PI, very stiff		74.57 / 20.12	27A	SS	37				
					27B				○		
		(SW/GW) gravelly SAND to sandy GRAVEL, trace to some silt, possible cobbles; brown; non-cohesive, wet, dense									
21					28	SS	43				Silica Sand
		(GW) sandy GRAVEL, some silt; brown (TILL); non-cohesive, wet, dense		73.35 / 21.34	29	SS	32		○		
22		END OF BOREHOLE		72.74 / 21.95							

Notes:

1. Water level measured in the piezometer as follows:

Date	Depth (mbgs)
28-Feb-23	15.92 (Deep Well)
28-Feb-23	12.18 (Shallow Well)

Depth scale: 23, 24, 25, 26, 27, 28, 29, 30

GTA-BHS 001 S:\CLIENTS\TAGGART_REALTY_MGMT\RIVERSIDE-HUNTCLUB\02_DATA\GINT\RIVERSIDE-HUNTCLUB.GPJ GAL-MIS.GDT 9/14/23

DEPTH SCALE

1 : 50

LOGGED: RI

CHECKED: CRG

RECORD OF BOREHOLE: BH23-02

PROJECT: 21482114
LOCATION: N 5020507.80; E 445522.94
BORING DATE: January 19 to 24, 2023
DATUM: Geodetic
SPT/DCPT HAMMER: MASS, 64kg; DROP, 760mm
DRILL RIG: CME 850 Track Mount
HAMMER TYPE: AUTOMATIC

BORING METHOD: 200 mm Power Auger; 108 mm I.D. Hollow Stem

SOIL PROFILE

DEPTH SCALE METRES	DESCRIPTION	ELEV. DEPTH (m)
	GROUND SURFACE	89.72
0	TOPSOIL - (SM) gravelly SILTY SAND; dark brown; moist	0.00
	FILL - (SM/GM) silty sandy GRAVEL to silty gravelly SAND, contains shale fragments and silty clay seams; dark brown; non-cohesive, moist, compact to loose	0.10
	FILL - (SM) SILTY SAND, trace gravel, rootlets; brown; non-cohesive, moist, loose	88.20 / 1.52
	FILL - (SP/SW) SAND, trace to some silt and gravel, with silty clay seams; brown to light brown; non-cohesive, moist, very loose to compact	87.89 / 1.83
	(SW/SP) SAND, trace to some gravel, contains cobbles; brown to light brown to grey-brown; non-cohesive, moist, dense to very dense	84.39 / 5.33
10	CONTINUED NEXT PAGE	

SAMPLES

NUMBER	TYPE	BLOWS/0.3m	ADDITIONAL LAB. TESTING	Gradation
1	SS	21		
2	SS	9		
3	SS	6		
4	SS	3	M	1 84 7 8
5	SS	3		
6	SS	2	M	0 93 3 4
7	SS	27		
8	SS	36		
9	SS	37		
10	SS	38		
11	SS	49		
12	SS	60		
13	SS	37		
14	SS	35		

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

PIEZOMETER OR STANDPIPE INSTALLATION

Bentonite Seal (0 to approx. 1.52 m)
Native Backfill (approx. 1.52 to 10 m)

DEPTH SCALE 1 : 50
LOGGED: RI
CHECKED: CRG

--- CONTINUED FROM PREVIOUS PAGE ---

DEPTH SCALE METRES	BORING METHOD	DESCRIPTION	ELEV. DEPTH (m)	SAMPLE NUMBER	TYPE	BLOWS/0.3m	PIEZOMETER OR STANDPIPE INSTALLATION
10	200 mm Power Auger, 108 mm I.D. Hollow Stem	(SW/SP) SAND, trace to some gravel, contains cobbles; brown to light brown to grey-brown; non-cohesive, moist, dense to very dense		14	SS	35	Native Backfill
11				15	SS	37	Bentonite Seal
				16	SS	64	Silica Sand
12	Mud Rotary Drilling, HW Casing			17	SS	24	Feb 28, 2023
13				18	SS	35	Screen
		(SP) SAND, fine to medium; brown; non-cohesive, wet, dense	76.00 13.72				
14				19	SS	45	
				20	SS	41	Feb 28, 2023
15		(SW) gravelly SAND, fine to coarse, trace to some silt; grey-brown to brown; non-cohesive, wet, very dense	74.48 15.24	21	SS	65	Silica Sand, Bentonite Seal
16		(SW/GW) gravelly SAND, fine to coarse to sandy GRAVEL, some silt, contains cobbles; brown (TILL); non-cohesive, wet, dense to very dense	73.72 16.00	22	SS	50	Bentonite Seal
17				23	SS	40	
				24	SS	60	Silica Sand
18				25	SS	72	
19		(SP-ML) interbedded SAND and CLAYEY SILT; brown (TILL); non-cohesive, wet, very dense	70.52 19.20	26	SS	64	Screen
20				27	SS	70	

CONTINUED NEXT PAGE

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

RECORD OF BOREHOLE: BH23-02

PROJECT: 21482114
LOCATION: N 5020507.80; E 445522.94
SPT/DCPT HAMMER: MASS, 64kg; DROP, 760mm

BORING DATE: January 19 to 24, 2023
DRILL RIG: CME 850 Track Mount

DATUM: Geodetic
HAMMER TYPE: AUTOMATIC

DEPTH SCALE METRES	BORING METHOD	DESCRIPTION	ELEV. DEPTH (m)	NUMBER	TYPE	BLOWS/0.3m	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
		--- CONTINUED FROM PREVIOUS PAGE ---								
20	Mud Rotary Drilling, HW Casing	(SP-ML) interbedded SAND and CLAYEY SILT; brown (TILL); non-cohesive, wet, very dense		27	SS	70				
		(SW) SAND, trace silt, some gravel to gravelly; brown (TILL); non-cohesive, wet, very dense	69.15 / 20.57							
21				28	SS	101		W		Silica Sand
				29	SS	66				
22		END OF BOREHOLE	67.77 / 21.95							

Notes:

1. Water level measured in the piezometer as follows:

Date	Depth (mbgs)
28-Feb-23	14.60 (Deep Well)
28-Feb-23	12.60 (Shallow Well)

Depth scale: 23, 24, 25, 26, 27, 28, 29, 30

DEPTH SCALE
1 : 50

LOGGED: RI
CHECKED: CRG

PROJECT: 21482114

LOCATION: N 5020576.36; E 445524.31

RECORD OF BOREHOLE: BH23-03

DATUM: Geodetic

BORING DATE: January 24 to 25, 2023

DRILL RIG: CME 850 Track Mount

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

HAMMER TYPE: AUTOMATIC

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE – DESCRIPTION	ELEV. / DEPTH (m)
		GROUND SURFACE	91.78
0	200 mm Power Auger, 108 mm I.D. Hollow Stem	TOPSOIL - (CL) sandy SILTY CLAY, trace gravel; dark brown; w~PL	0.00
		FILL - (CL) SILTY CLAY, trace sand to sandy, trace gravel, with thick seams to thin layers of silty sand, contains organic matter and asphalt fragments; grey to grey-brown; cohesive, w>PL, firm to very stiff	91.48 / 0.30
		FILL - (SM) SILTY SAND, fine to medium, contains organic matter and rootlets; grey; non-cohesive, wet, very loose	85.38 / 6.40
7	Mud Rotary Drilling, HW Casing	(SP) SAND, fine to medium, trace to some silt, trace gravel, thin seams to very thin layers of silty clay to clayey silt; grey to grey-brown; non-cohesive, wet, very loose to compact	84.92 / 6.86
10		*CONTINUED NEXT PAGE*	

SAMPLES

NUMBER	TYPE	BLOWS/0.3m	ADDITIONAL LAB. TESTING
1	SS	6	
2	SS	6	
3	SS	4	
4	SS	1	
5	SS	WH	
6	SS	WH	
7	SS	WH	
8	SS	1	
9A	SS	2	
9B			
10	SS	4	M 0 95 2 3
11	SS	10	
12	SS	14	
13	SS	22	
14	SS	26	

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

PIEZOMETER OR STANDPIPE INSTALLATION: Bentonite Seal; Native Backfill

DEPTH SCALE 1 : 50

LOGGED: RI

CHECKED: CRG

BORING DATE: January 24 to 25, 2023
DRILL RIG: CME 850 Track Mount
SPT/DCPT HAMMER: MASS, 64kg; DROP, 760mm
HAMMER TYPE: AUTOMATIC

--- CONTINUED FROM PREVIOUS PAGE ---

BORING METHOD: Mud Rotary Drilling; HW Casing

DEPTH SCALE METRES	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLE NUMBER	TYPE	BLOWS/0.3m	PIEZOMETER OR STANDPIPE INSTALLATION
10	(SP) SAND, fine to medium, trace to some silt, trace gravel, thin seams to very thin layers of silty clay to clayey silt; grey to grey-brown; non-cohesive, wet, very loose to compact		14	SS	26	
	(SW/SP) SAND, trace to some gravel, trace silt, contains cobbles; grey-brown to brown; non-cohesive, wet, very dense	81.11 / 10.67	15	SS	104	
			16	SS	108	
			17	SS	112	
			18	SS	113	Native Backfill
			19	SS	99	
			20	SS	85	Feb 28, 2023
			21	SS	56	
			22	SS	47	
			23	SS	56	Bentonite Seal
			24	SS	65	Silica Sand
			25	SS	80	
			26	SS	68	Screen
20			27	SS	61	

Graph 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

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PROJECT: 21482114

LOCATION: N 5020576.36; E 445524.31

RECORD OF BOREHOLE: BH23-03

BORING DATE: January 24 to 25, 2023

DATUM: Geodetic

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

DRILL RIG: CME 850 Track Mount

HAMMER TYPE: AUTOMATIC

DEPTH SCALE METRES	BORING METHOD	DESCRIPTION	ELEV. / DEPTH (m)	NUMBER	TYPE	BLOWS/0.3m	HYDRAULIC CONDUCTIVITY / WATER CONTENT PERCENT	PIEZOMETER OR STANDPIPE INSTALLATION
		--- CONTINUED FROM PREVIOUS PAGE ---						
20	Mud Rotary Drilling; HW Casing	(SW/SP) SAND, trace to some gravel, trace silt, contains cobbles; grey-brown to brown; non-cohesive, wet, very dense		27	SS	61	W ≈ 20	Silica Sand
21		(SW) gravelly SAND, fine to coarse, trace to some silt; brown; non-cohesive, wet, very dense	70.90 / 20.88	28	SS	84		Bentonite Seal
				29	SS	94		
22		END OF BOREHOLE	69.83 / 21.95					

Notes:

1. Water level measured in the piezometer as follows:

Date	Depth (mbgs)
28-Feb-23	15.09

DEPTH SCALE

1 : 50

LOGGED: RI

CHECKED: CRG

RECORD OF BOREHOLE: BH23-04

PROJECT: 21482114
LOCATION: N 5020647.77; E 445544.27
SPT/DCPT HAMMER: MASS, 64kg; DROP, 760mm

BORING DATE: January 27 to 31, 2023
DRILL RIG: CME 850 Track Mount

SHEET 1 OF 3
DATUM: Geodetic
HAMMER TYPE: AUTOMATIC

Column headings: DEPTH SCALE METRES | BORING METHOD | SOIL PROFILE (DESCRIPTION, STRATA PLOT, ELEV. DEPTH (m)) | SAMPLES (NUMBER, TYPE, BLOWS/0.3m) | DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20, 40, 60, 80); SHEAR STRENGTH Cu, kPa (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: 200 mm Power Auger, 108 mm I.D. Hollow Stem (0 to ~6.1 m); Mud Rotary Drilling, HW Casing (~6.1 to 10 m)

Description	Elev. / Depth (m)
GROUND SURFACE	91.58
TOPSOIL - (SM) SILTY SAND, some low plasticity fines; dark brown; moist	0.00
FILL - (SP) SAND, fine to medium, trace to some silt; brown; non-cohesive, moist, very loose	0.08
FILL - (CL-SM) interbedded sandy silty clay and gravelly silty sand; grey-brown to grey to brown; cohesive and non-cohesive, w>PL (cohesive) and moist (non-cohesive), very loose (non-cohesive) and stiff to very stiff (cohesive)	0.18
FILL - (SP) SAND, fine to medium, contains organic matter and rootlets; grey; non-cohesive, moist to wet	86.55 / 5.03
(SP) SAND, fine to medium, trace to some silt, contains very thin layers of silty clay to clayey silt; grey to brown; non-cohesive, wet, compact to dense	86.25 / 5.33
	81.83 / 9.75

Sample No.	Type	Blows/0.3m	Additional Lab. Testing
1	SS	4	
2	SS	3	
3	SS	4	
4	SS	2	
5	SS	2	
6	SS	2	M — 1 25 37 37
7A	SS	1	
7B			
8	SS	25	
9	SS	18	
10	SS	16	
11	SS	14	
12	SS	10	M — 0 67 16 17
13	SS	34	
14	SS	18	

Piezometer / standpipe installation: Bentonite Seal; water level Feb 28, 2023 (~4.8 m); Silica Sand; Screen; Silica Sand; Bentonite Seal; water level Feb 28, 2023 (~9.0 m); Native Backfill

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DEPTH SCALE
1 : 50

wsp

LOGGED: RI
CHECKED: CRG

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RECORD OF BOREHOLE: BH23-04

PROJECT: 21482114
LOCATION: N 5020647.77; E 445544.27
BORING DATE: January 27 to 31, 2023
DRILL RIG: CME 850 Track Mount

DATUM: Geodetic
SPT/DCPT HAMMER: MASS, 64kg; DROP, 760mm
HAMMER TYPE: AUTOMATIC

--- CONTINUED FROM PREVIOUS PAGE ---

DEPTH SCALE METRES	BORING METHOD	DESCRIPTION	NUMBER	TYPE	BLOWS/0.3m	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
10	Mud Rotary Drilling; HW Casing	(SP/SW) SAND, some low plasticity fines, contains clay seams; brown; non-cohesive, wet, compact to very dense	14	SS	18		
11			15	SS	22		
			16	SS	15		
12		- Pockets of silty sand and clayey silt					
			17	SS	3	M	0 36 32 32
13							Native Backfill
			18	SS	20		
14			19	SS	20		
15			20	SS	21		
			21	SS	25		
16			22	SS	43		Bentonite Seal
17			23	SS	34		Silica Sand
18			24	SS	31		
			25	SS	31		
19							Screen
			26	SS	19		
20			27	SS	51		Silica Sand

CONTINUED NEXT PAGE

DEPTH SCALE
1 : 50

LOGGED: RI
CHECKED: CRG

RECORD OF BOREHOLE: BH23-04

PROJECT: 21482114
LOCATION: N 5020647.77; E 445544.27

BORING DATE: January 27 to 31, 2023
DATUM: Geodetic
SPT/DCPT HAMMER: MASS, 64kg; DROP, 760mm
DRILL RIG: CME 850 Track Mount
HAMMER TYPE: AUTOMATIC

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES: NUMBER	TYPE	BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20 40 60 80); SHEAR STRENGTH Cu, kPa 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	Mud Rotary Drilling; HW Casing	(SP/SW) SAND, some low plasticity fines, contains clay seams; brown; non-cohesive, wet, compact to very dense		71.16 20.42	27	SS	51				
21		(SP) SAND, fine, trace to some silt; brown; non-cohesive, wet, very dense			28	SS	61		○		Silica Sand
				69.63 21.95	29	SS	51				
22		END OF BOREHOLE									

Notes:

1. Water level measured in the piezometer as follows:

Date	Depth (mbgs)
28-Feb-23	9.00 (Deep Well)
28-Feb-23	4.75 (Shallow Well)

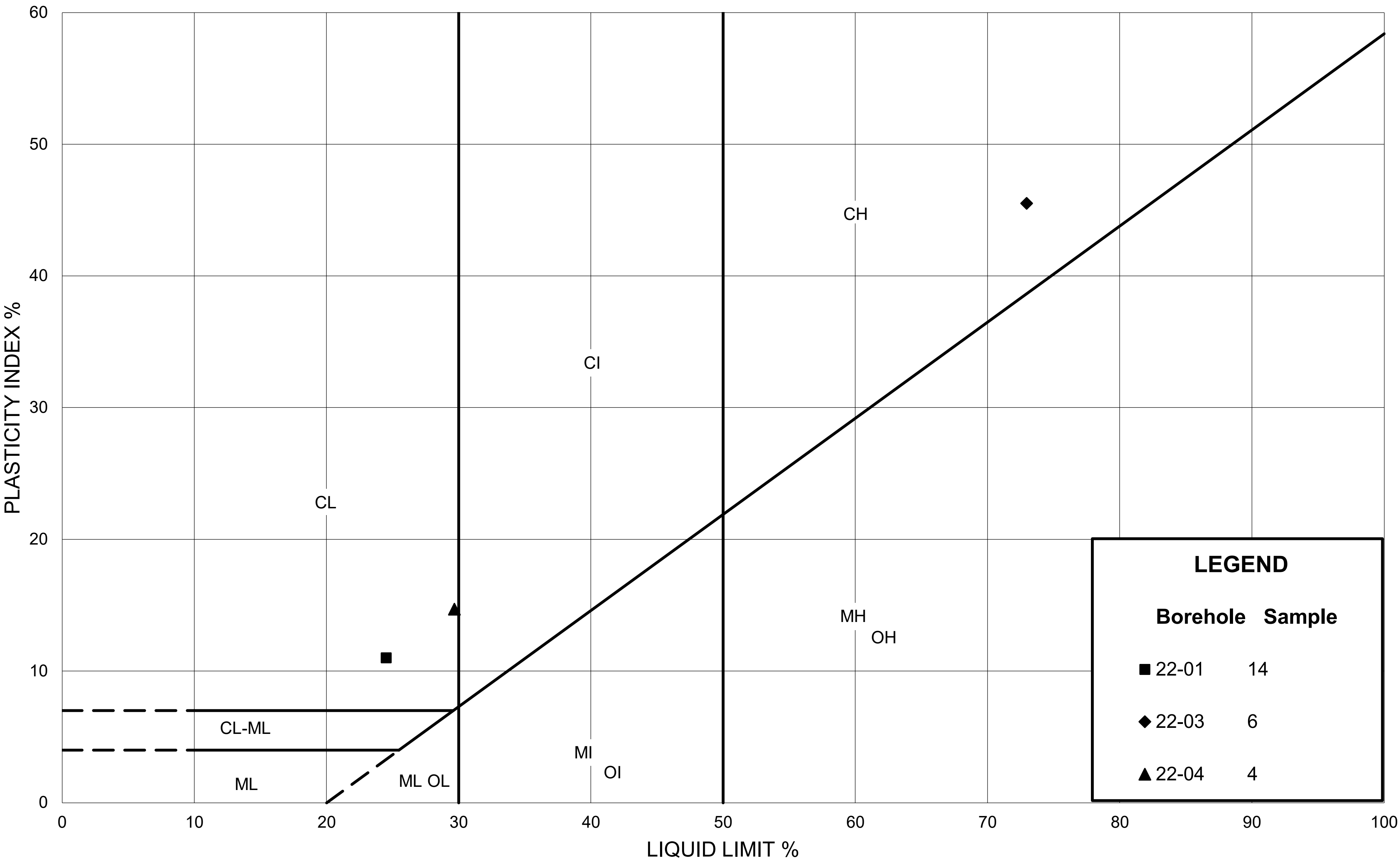
Depth scale: 23, 24, 25, 26, 27, 28, 29, 30

DEPTH SCALE
1 : 50

LOGGED: RI
CHECKED: CRG

APPENDIX B

Results of Laboratory Testing (2023)



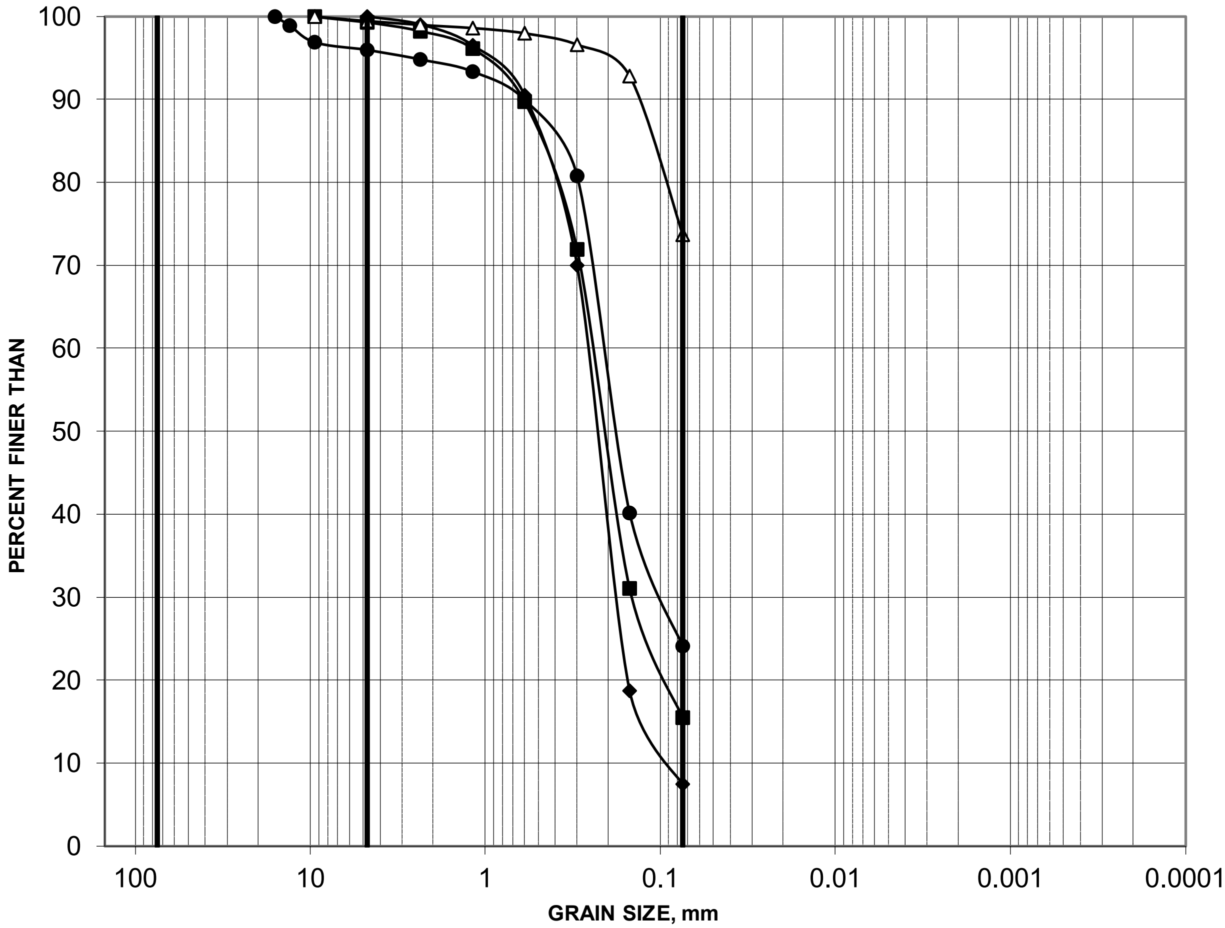
PLASTICITY CHART
FILL

Figure:	B1
Project:	21482114
Created By: MI	Checked By: JB

GRAIN SIZE DISTRIBUTION

FIGURE B2

FILL



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

			Constituents (%)			
Borehole	Sample	Depth (m)	Gravel	Sand	Silt	Clay
● 22-01	3	1.52-2.13	4	72	24	
■ 22-02	4	2.29-2.90	1	84	15	
◆ 22-02	6	3.81-4.42	0	93	7	
△ 22-04	6	3.81-4.42	1	25	74	

Project: 21482114

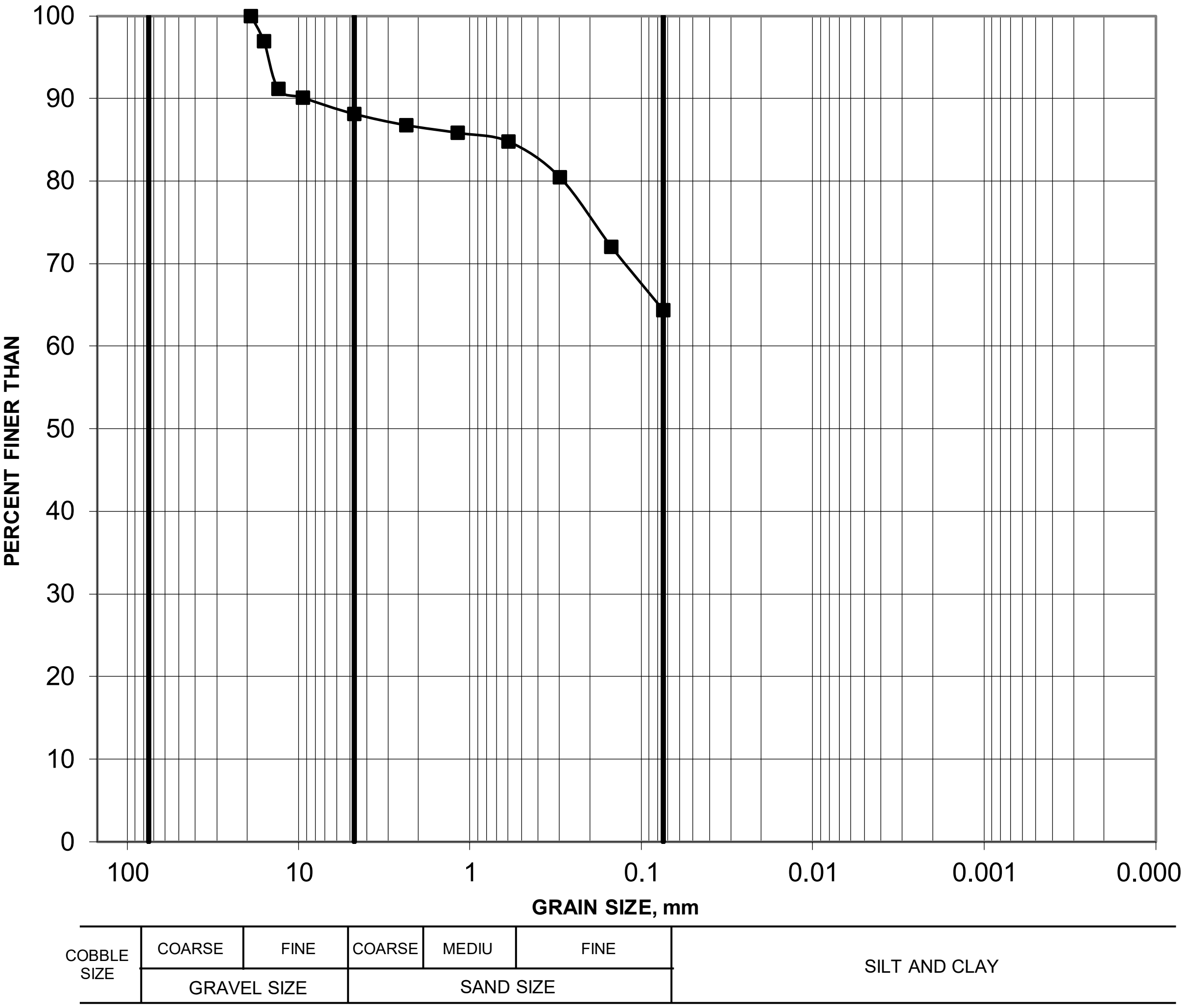
Created by: MI
Checked by: JB

https://wsponlinecan.sharepoint.com/Sites/Global-OttawaLab/Shared Documents/Active/2021/21482114/Figures/

GRAIN SIZE DISTRIBUTION

FIGURE B3

FILL



			Constituents (%)			
Borehole	Sample	Depth (m)	Gravel	Sand	Silt	Clay
22-01	17	12.20-12.80	12	24	64	

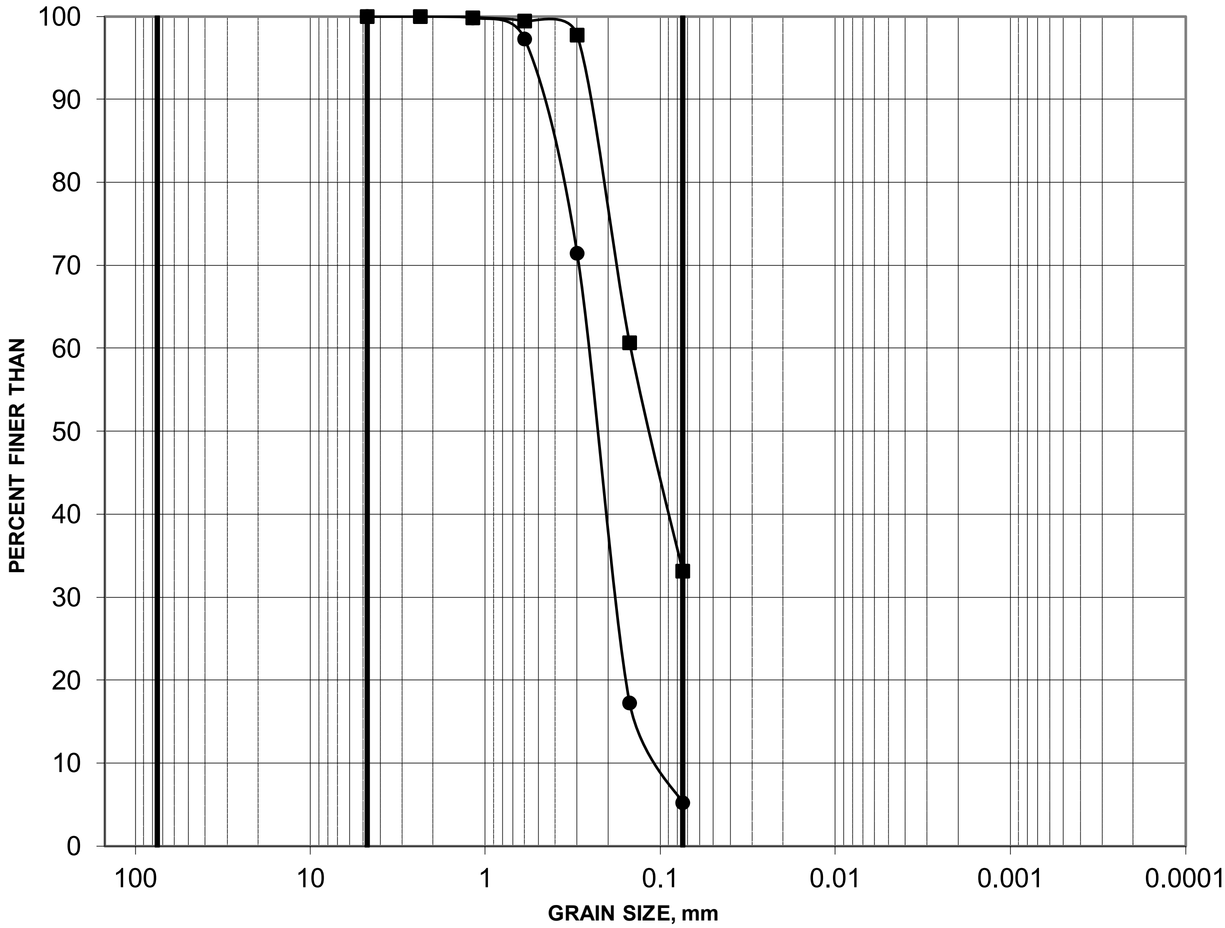
Project: 21482114

Created by: MI

Checked by: JB

GRAIN SIZE DISTRIBUTION	FIGURE B4

SAND



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

Borehole	Sample	Depth (m)	Gravel	Sand	Silt	Clay
			Constituents (%)			
● 22-03	10	6.86-7.47	0	95	5	
■ 22-04	12	8.38-8.99	0	67	33	

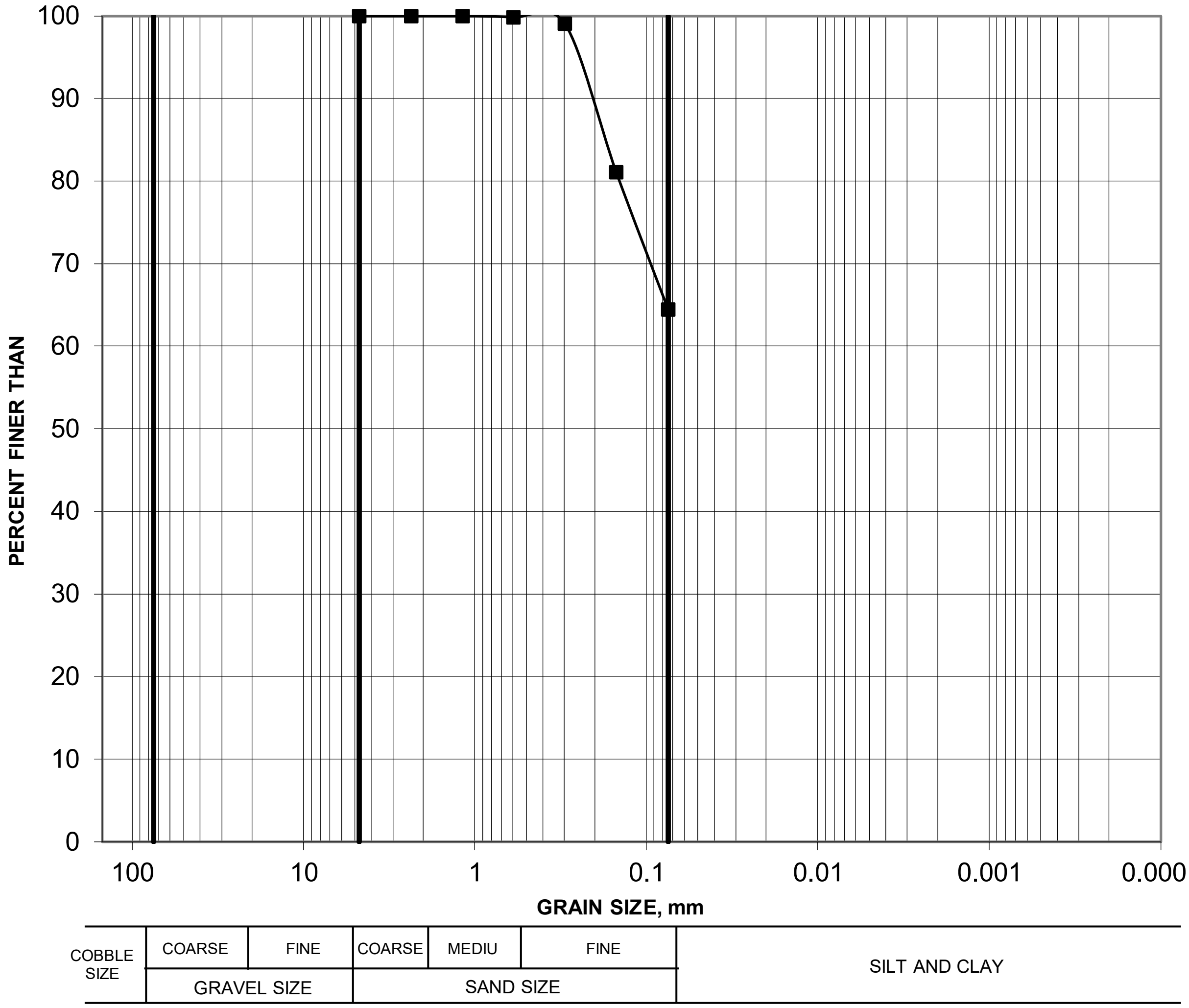
Project: 21482114

Created by:	MI
Checked by:	JB

https://wsponlinecan.sharepoint.com/Sites/Global-OttawaLab/Shared Documents/Active/2021/21482114/Figures/

GRAIN SIZE DISTRIBUTION	FIGURE B5

SILTY SAND TO CLAYEY SILT



				Constituents (%)			
	Borehole	Sample	Depth (m)	Gravel	Sand	Silt	Clay
■	22-04	17	12.20-12.80	0	36	64	

Project: 21482114

Created by: MI

Checked by: JB

APPENDIX C

Records of Previous Borehole Logs

RECORD OF BOREHOLE: 17-201

PROJECT: 1670692 — SHEET 1 OF 1

LOCATION: N 5022305.1 ;E 367871.0 — BORING DATE: December 21, 2017 — DATUM: CGVD28

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

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

DEPTH SCALE METRES	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLE NUMBER	TYPE	BLOWS/0.30m	WATER CONTENT / HYDRAULIC CONDUCTIVITY	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
	GROUND SURFACE	91.24						
0	FILL - (SM) SILTY SAND, some gravel; dark brown, contains organic matter and cobbles; non-cohesive, moist, loose	0.00	1	SS	8			Flush Mount Casing; Silica Sand; Bentonite Seal
1.37	FILL - (SP) SAND, trace non-plastic fines; brown, contains silty clay lumps; non-cohesive, moist, very loose	89.87 / 1.37	2	SS	3			
			3	SS	2	○	MH	Native Backfill
			4	SS	2			Bentonite Seal
5.64	(SP) SAND; grey brown, contains clayey silt seams; non-cohesive, wet, compact	85.60 / 5.64	5	SS	11	○	M	Silica Sand
			6	SS	14			31 mm Diam. PVC #10 Slot Screen
			7	SS	20			
			8	SS	17			
8.84	(CI) sandy SILTY CLAY; grey, contains fine sand seams; cohesive, w>PL, very stiff	82.40 / 8.84	9	SS	8	Wp ├──┤○		Silica Sand
9.75	End of Borehole	81.49 / 9.75						WL in Screen at Elev. 86.02 m on Jan. 19, 2018

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

DEPTH SCALE 1 : 50

Golder Associates

LOGGED: PAH

CHECKED:

PROJECT: 1670692

RECORD OF BOREHOLE: 17-202

SHEET 1 OF 1

LOCATION: N 5022248.0 ;E 367899.7

BORING DATE: December 18, 2017

DATUM: CGVD28

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	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
		GROUND SURFACE	92.01							
0	Power Auger 200 mm Diam. (Hollow Stem)	TOPSOIL - (SM) SILTY SAND; brown; non-cohesive, moist	0.00							
		FILL - (CI/CH) SILTY CLAY to CLAY, some sand; dark grey brown; cohesive, w>PL	0.10	1	SS	6				
2				2	SS	3				
3		FILL - (CI/ML) CLAYEY SILT to SILTY CLAY, trace gravel; grey; cohesive, w>PL	88.96 3.05	3	SS	1				
5		(SP) SAND, medium to fine, trace non-plastic fines; grey; non-cohesive, wet, compact	87.21 4.80	4	SS	16				
				5	SS	21				
				6	SS	19				
7				7	SS	20				
		(SM, CL and ML) Stratified SILTY SAND/SILTY CLAY and CLAYEY SILT; grey; non-cohesive, wet, compact	84.69 7.32	8	SS	20				
9				9	SS	23				
				10	SS	18				
		End of Borehole	82.26 9.75							
10										

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: PAH

CHECKED:

MIS-BHS 001 1670692.GPJ GAL-MIS.GDT 2/12/18 JEM

PROJECT: 1670692

RECORD OF BOREHOLE: 17-203

SHEET 1 OF 1

LOCATION: N 5022179.0 ;E 367913.0

BORING DATE: December 15, 2017

DATUM: CGVD28

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

Boring method: Power Auger, 200 mm Diam. (Hollow Stem)

DEPTH SCALE METRES	SOIL PROFILE DESCRIPTION	ELEV. / DEPTH (m)	SAMPLE NUMBER	TYPE	BLOWS/0.30m	WATER CONTENT PERCENT	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
	GROUND SURFACE	93.63						
0	FILL - (ML/CL) SILTY CLAY and CLAYEY SILT, some sand; brown and grey brown, contains organic matter and sandy silt layers; cohesive, w>PL	0.00	1	SS	7			Flush Mount Casing
2			2	SS	3			Native Backfill
3			3	SS	1	Wp–W plotted		
5			4	SS	3			Bentonite Seal
	(SP/SM) SAND, medium to fine, some non-plastic fines; grey brown; non-cohesive, wet, compact	87.99 / 5.64						Silica Sand
6			5	SS	21	W plotted	MH	
7			6	SS	19			31 mm Diam. PVC #10 Slow Screen
	(SM/CL) SAND and SILTY CLAY, fine, layered; grey brown; non-cohesive, wet, compact	86.01 / 7.62	7	SS	20			
	(SP) SAND, fine to medium, trace non-plastic fines; brown; non-cohesive, wet, compact	85.40 / 8.23	8	SS	21			
9			9	SS	19			Cave
	End of Borehole	83.88 / 9.75						WL in Screen at Elev. 87.82 m on Jan. 19, 2018

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

DEPTH SCALE 1 : 50

Golder Associates

LOGGED: PAH

CHECKED:

PROJECT: 1670692

RECORD OF BOREHOLE: 17-204

SHEET 1 OF 1

LOCATION: N 5022102.1 ;E 367894.0

BORING DATE: December 14, 2017

DATUM: CGVD28

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

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

DEPTH SCALE METRES	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. / DEPTH (m)	SAMPLE 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
	GROUND SURFACE		90.34							
0	FILL - (SM/GM) SILTY SAND and GRAVEL; grey brown; non-cohesive, moist, loose		0.00	1	SS	6				
1	FILL - (SP/SM) SAND to SILTY SAND, fine; brown, contains clayey silt layers; non-cohesive, moist to wet, compact to loose		88.97 / 1.37	2	SS	11		○	MH	
3				3	SS	9				
4	(SP/SM) SAND, fine to coarse, some non-plastic fines, trace gravel; brown; non-cohesive, moist, dense to very dense		86.38 / 3.96	4	SS	51				
5				5	SS	54		○	M	
6				6	SS	54				
7				7	SS	42				
8				8	SS	34				
9				9	SS	38				
				10	SS	35				
10	End of Borehole		80.59 / 9.75							

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: PAH

CHECKED:

PROJECT: 1670692

RECORD OF BOREHOLE: 17-205

SHEET 1 OF 2

LOCATION: N 5022019.8 ;E 367901.3

BORING DATE: December 14-15, 2017

DATUM: CGVD28

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 - O) (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 (0–10 m): Power Auger, 200 mm Diam. (Hollow Stem)

Depth Scale (m)	Description	Elev. / Depth (m)	Sample Number	Type	Blows/0.30m	Additional Lab. Testing
	GROUND SURFACE	93.16				
0	FILL - (SM) SILTY CLAY and SILTY SAND; grey brown, contains brick fragments and organic matter (rootlets); non-cohesive, moist, loose	0.00	1	SS	7	
1	FILL - (SM) gravelly SILTY SAND; grey and dark grey, contains clayey silt pockets; non-cohesive, very moist, loose to compact	92.25 / 0.91	2	SS	6	M
3			3	SS	11	
4	FILL - (ML/SM) sandy SILT to SILTY SAND, some shaley gravel; dark grey, contains organic matter (peat and wood); wet, loose	88.89 / 4.27	4	SS	7	
6			5	SS	5	
8			6	SS	8	
9	FILL - (CI/ML) SILTY CLAY and CLAYEY SILT, trace gravel; dark grey and grey brown, contains organic matter; non-cohesive, moist, loose to compact	84.02 / 9.14	7	SS	7	
10						

CONTINUED NEXT PAGE

DEPTH SCALE
1 : 50

Golder Associates

LOGGED: PAH
CHECKED:

PROJECT: 1670692

RECORD OF BOREHOLE: 17-205

SHEET 2 OF 2

LOCATION: N 5022019.8 ;E 367901.3

BORING DATE: December 14-15, 2017

DATUM: CGVD28

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	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 ─O^W─ Wl (20 40 60 80)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		--- CONTINUED FROM PREVIOUS PAGE ---									
10	Power Auger 200 mm Diam. (Hollow Stem)	FILL - (CI/ML) SILTY CLAY and CLAYEY SILT, trace gravel; dark grey and grey brown, contains organic matter; non-cohesive, moist, loose to compact									
11					8	SS	13				
12					9	SS	7				
13											
14					10	SS	8				
		FILL - (SP/SM) SAND, some non-plastic fines, trace gravel, angular; brown; non-cohesive, moist, compact		78.68 14.48	11	SS	22				
15		(GP) sandy GRAVEL, trace non-plastic fines; grey brown, contains cobbles; non-cohesive, wet, very dense		78.22 14.94	12	SS	141				
16					13	SS	120				
17					14	SS	57				
		End of Borehole		75.79 17.37							
18											
19											
20											

MIS-BHS 001 1670692.GPJ GAL-MIS.GDT 2/12/18 JEM

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: PAH

CHECKED:

PROJECT: 1670692

RECORD OF BOREHOLE: 17-206

SHEET 1 OF 3

LOCATION: N 5022072.5 ;E 367938.8

BORING DATE: December 21-22, 2017

DATUM: CGVD28

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE - DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	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
		GROUND SURFACE		92.26							
0	Power Auger 200 mm Diam. (Hollow Stem)	FILL - (CI) sandy SILTY CLAY; brown, contains organic matter (topsoil); cohesive, w>PL, stiff		0.00	1	SS	3				
1											
		FILL - (SM-GP/GM) gravelly SILTY SAND to sandy GRAVEL, some low plasticity fines; grey brown, contains cobbles; non-cohesive, moist, compact to loose		90.74 1.52	2	SS	24		○	M	
3					3	SS	9				
4											
5					4	SS	13				
6					5	SS	7				
7											
		FILL - (SM) gravelly SILTY SAND, grey brown and black, contains asphaltic concrete fragments and cinder; non-cohesive, moist, compact to very dense		84.64 7.62	6	SS	22				
8					7	SS	>50				
9					8	SS	10				
10											

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DEPTH SCALE

1 : 50

Golder Associates

LOGGED: PAH

CHECKED:

RECORD OF BOREHOLE: 17-206

LOCATION: N 5022072.5 ;E 367938.8

BORING DATE: December 21-22, 2017

DATUM: CGVD28

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

--- CONTINUED FROM PREVIOUS PAGE ---

DEPTH SCALE METRES	BORING METHOD	DESCRIPTION	ELEV. DEPTH (m)	NUMBER	TYPE	BLOWS/0.30m	WATER CONTENT PERCENT	ADDITIONAL LAB. TESTING
10	Power Auger 200 mm Diam. (Hollow Stem)	FILL - (SM) gravelly SILTY SAND, grey brown and black, contains asphaltic concrete fragments and cinder; non-cohesive, moist, compact to very dense						
11				9	SS	22		
12	Wash Boring NW Casing			10	SS	19		
13		FILL - (SM) SILTY SAND, some gravel; brown, contains organic matter (wood fibres) and white shells; non-cohesive, moist to wet, compact to dense	79.31 12.95	11	SS	30		
14				12	SS	11		
				13	SS	18		
15				14	SS	38		
16		(SP) SAND, medium to fine, trace non-plastic fines; brown; non-cohesive, wet, loose to compact	76.41 15.85	15	SS	8	o (≈20)	M
17				16	SS	26		
		(SP/GP) gravelly SAND to SAND and GRAVEL, medium to coarse, trace non-plastic fines; grey brown; non-cohesive, wet, very dense to dense	75.04 17.22					
18				17	SS	73		
19				18	SS	42		
20								

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LOCATION: N 5022072.5 ;E 367938.8

BORING DATE: December 21-22, 2017

DATUM: CGVD28

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES: NUMBER	TYPE	BLOWS/0.30m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20 40 60 80); SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - O (20 40 60 80)	HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-6} 10^{-5} 10^{-4} 10^{-3}); WATER CONTENT PERCENT Wp ⊢—O W—⊣ Wl (20 40 60 80)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		--- CONTINUED FROM PREVIOUS PAGE ---									
20	Wash Boring / NW Casing	(SP/GP) gravelly SAND to SAND and GRAVEL, medium to coarse, trace non-plastic fines; grey brown; non-cohesive, wet, very dense to dense									
21											
22					19	SS	44				
23											
24											
25				66.81 / 25.45	20	SS	104				
		(SM) gravelly SILTY SAND; grey brown; non-cohesive, wet, very dense									
26					21	SS	171				
27					22	SS	>100				
28		End of Borehole		64.45 / 27.81							
29											
30											

PROJECT: 1670692

RECORD OF BOREHOLE: 17-207

SHEET 1 OF 4

LOCATION: N 5022168.2 ;E 367977.6

BORING DATE: December 18-20, 2017

DATUM: CGVD28

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	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
		GROUND SURFACE	94.58							
0	Power Auger, 200 mm Diam. (Hollow Stem)	FILL - (SM/GM) SILTY SAND and GRAVEL; grey and brown, contains cobbles and boulders; non-cohesive, moist, dense	0.00	1	SS	33				
				2	SS	33				
3		FILL - (CI/SM) SILTY CLAY and SILTY SAND, some gravel; dark grey and brown, contains alluvium and organic matter; non-cohesive, moist, compact	91.53 / 3.05	3	SS	24				
	Wash Boring, NW Casing	FILL - (SP/SM) SAND to SILTY SAND, some gravel; contains organic matter, plastic and metal fragments; non-cohesive, wet, very loose to compact	90.01 / 4.57	4	SS	4		O	M	
				5	SS	29				
6	Wash Boring with Polymer Drill Mud, NW Casing	(SP) SAND, medium to fine, trace non-plastic fines; grey; non-cohesive, wet, compact to dense	88.48 / 6.10	6	SS	31				
				7	SS	39				
				8	SS	23				
		(SP/SM) SAND to SILTY SAND, medium to fine; brown, contains clayey silt layers; non-cohesive, wet, compact	86.20 / 8.38	9	SS	22				
				10	SS	18				
10			84.67 / 9.91	11	SS	25				

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DEPTH SCALE 1 : 50

Golder Associates

LOGGED: PAH

CHECKED:

RECORD OF BOREHOLE: 17-207

LOCATION: N 5022168.2 ;E 367977.6

BORING DATE: December 18-20, 2017

DATUM: CGVD28

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	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 with Polymer Drill Mud; NW Casing	(SP) SAND, medium to fine, trace non-plastic fines; brown; non-cohesive, wet, compact			11	SS	25				
11					12	SS	29				
		(ML, CL) CLAYEY SILT and SILTY CLAY; grey brown, contains sand seams; cohesive, w>PL, very stiff		83.15 11.43	13	SS	17				
12		(SP/SM) SAND to SILTY SAND; brown, contains clayey silt layers; non-cohesive, wet, compact		82.08 12.50	14	SS	13		Wp ⊢⊣ ○		
13					15	SS	20				
		(SM) SILTY SAND; brown; non-cohesive, wet, compact to dense		80.86 13.72							
14					16	SS	31				
					17	SS	25				
15		(SP/SM) SAND to SILTY SAND; brown, contains clayey silty layers; non-cohesive, wet, compact		79.34 15.24	18	SS	22				
16											
17					19	SS	21				
18											
19		(SM) SILTY SAND; grey, contains clayey silt pockets; non-cohesive, wet, dense		75.38 19.20	20	SS	33				
20											

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DEPTH SCALE
1 : 50

Golder Associates

LOGGED: PAH
CHECKED:

PROJECT: 1670692

RECORD OF BOREHOLE: 17-207

LOCATION: N 5022168.2 ;E 367977.6

BORING DATE: December 18-20, 2017

DATUM: CGVD28

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES: NUMBER	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 with Polymer Drill Mud; NW Casing	(SM) SILTY SAND; grey, contains clayey silt pockets; non-cohesive, wet, dense		73.85 20.73							
21		(SP) SAND, fine, trace non-plastic fines; grey; non-cohesive, wet, very dense									
22					21	SS	46				
23											
24											
25					22	SS	49				
26											
27											
28		(SM) SILTY SAND; grey; non-cohesive, wet, very dense		66.39 28.19							
		(SP/GP) gravelly SAND to sandy GRAVEL; grey, contains cobbles; non-cohesive, wet, very dense		66.08 28.50	23	SS	77				
29											
					24	SS	172				
30											

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DEPTH SCALE

1 : 50

LOGGED: PAH

CHECKED:

PROJECT: 1670692

RECORD OF BOREHOLE: 17-207

LOCATION: N 5022168.2 ;E 367977.6

BORING DATE: December 18-20, 2017

DATUM: CGVD28

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	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 with Polymer Drill Mud; NW Casing	(SP/GP) gravelly SAND to sandy GRAVEL; grey, contains cobbles; non-cohesive, wet, very dense			25	RC	DD				
31				62.88 31.70	26	SS	185				
		End of Borehole									
32											
33											
34											
35											
36											
37											
38											
39											
40											

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: PAH

CHECKED:

PROJECT: 1670692

RECORD OF BOREHOLE: 17-01

LOCATION: N 5020451.2 ;E 445599.8

BORING DATE: May 2 & 3, 2017

DATUM: CGVD28

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE - DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.30m	HEADSPACE ORGANIC VAPOUR CONCENTRATIONS [PPM] ⊕ ND = Not Detected (20 40 60 80); HEADSPACE COMBUSTIBLE VAPOUR CONCENTRATIONS [%LEL] ND = Not Detected □ (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
		GROUND SURFACE		94.82							
0	Power Auger 200 mm Diam. (Hollow Stem)	FILL - (CL-ML-SM) SILTY CLAY, sandy SILT and SILTY SAND, trace to some gravel; brown and grey, contains sand and gravel layers, trace brick and organics; moist to wet, loose to compact		0.00	1	GRAB	-	⊕			
1					2	SS	13	⊕			
					3	SS	38				
2					4	SS	16	⊕			
3					5	SS	8				
4					6	SS		⊕			
5					7	SS	12	⊕			Native Backfill and Bentonite Mix
					8	SS	9	⊕			
6					9	SS	10	⊕			
7					10	SS	3	⊕			
8					11	SS	15				
					12	SS	11	⊕			
9		FILL - (ML-CL) sandy SILT, some gravel and silty clay; grey brown and grey, trace asphaltic concrete and organics; very moist to wet, loose		85.68 9.14	13	SS	8	⊕			
10					14	SS	9				

CONTINUED NEXT PAGE

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: PAH

CHECKED:

RECORD OF BOREHOLE: 17-01

PROJECT: 1670692

LOCATION: N 5020451.2 ;E 445599.8

BORING DATE: May 2 & 3, 2017

DATUM: CGVD28

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) | HEADSPACE ORGANIC VAPOUR CONCENTRATIONS [PPM] ⊕ ND = Not Detected (20, 40, 60, 80); HEADSPACE COMBUSTIBLE VAPOUR CONCENTRATIONS □ [%LEL] ND = Not Detected (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)

--- CONTINUED FROM PREVIOUS PAGE ---

Depth scale (m)	Description	Elev. / Depth (m)	Sample No.	Type	Blows/0.30m	Piezometer/Standpipe
10	FILL - (ML-CL) sandy SILT, some gravel and silty clay; grey brown and grey, trace asphaltic concrete and organics; very moist to wet, loose		14	SS	9	
			15	SS	5	
11			16	SS	7	
12			17	SS	20	
13			18	SS	4	Native Backfill and Bentonite Mix
	FILL - (ML) sandy SILT with gravel; dark grey and grey brown, contains silty clay layers, trace organics	81.10 / 13.72				
14			19	SS	15	
15			20	SS	13	
			21	SS	9	
16			22	SS	19	∇
17			23	SS	21	
18						Bentonite Seal
			24	SS	21	
	CONCRETE RUBBLE	76.07 / 18.75				
19						Silica Sand
	(ML) sandy SILT with gravel; dark grey and grey brown, contains silty clay layers, trace organics	75.47 / 19.35				
20			25	SS	49	51 mm Diam. PVC #10 Slot Screen

CONTINUED NEXT PAGE

PROJECT: 1670692

RECORD OF BOREHOLE: 17-01

LOCATION: N 5020451.2 ;E 445599.8

BORING DATE: May 2 & 3, 2017

DATUM: CGVD28

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES: NUMBER	TYPE	BLOWS/0.30m	HEADSPACE ORGANIC VAPOUR CONCENTRATIONS [PPM] ⊕ ND = Not Detected (20 40 60 80); HEADSPACE COMBUSTIBLE VAPOUR CONCENTRATIONS [%LEL] ND = Not Detected □ (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	Power Auger 200 mm Diam. (Hollow Stem)	(ML) sandy SILT with gravel; dark grey and grey brown, contains silty clay layers, trace organics			25	SS	49	⊕			51 mm Diam. PVC #10 Slot Screen
21					26	SS	42	⊕			Silica Sand
22		End of Borehole		72.87 21.95							W.L. in Screen at Elev. 78.41 m on May 4, 2017
23											
24											
25											
26											
27											
28											
29											
30											

MIS-BHS 001 1670692.GPJ GAL-MIS.GDT 05/11/17 JEM

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: PAH

CHECKED:

RECORD OF BOREHOLE: 17-03

PROJECT: 1670692

SHEET 1 OF 1

LOCATION: N 5020535.1 ;E 445613.8

BORING DATE: May 2, 2017

DATUM: CGVD28

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

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

Elev. (m) / Depth (m)	Soil Profile Description
94.26	GROUND SURFACE
0.00	FILL - (CL-ML) SILTY CLAY and sandy SILT, some gravel; grey brown, contains organics; very moist, loose to compact
92.13 / 2.13	FILL - (SM-GW) SILTY SAND and SAND and GRAVEL; grey brown; non-cohesive, moist, compact
91.11 / 3.15	(SP) SAND, fine to medium; brown; non-cohesive, moist, very dense
90.60 / 3.66	(SP) SAND, fine to medium; brown; non-cohesive, moist to wet with depth, compact
86.03 / 8.23	End of Borehole

Sample Number	Type	Blows/0.30m	Headspace Organic Vapour Concentration [PPM] (ND = Not Detected)
1	GRAB	-	155.7
2	SS	7	177
3	SS	15	136
4	SS	23	141.8
5	SS	91	plotted (~75)
6	SS	19	plotted (~10)
7	SS	16	plotted (~40)
8	SS	24	plotted (~45)
9	SS	20	plotted (~85)
10	SS	16	plotted (~95)
11	SS	15	plotted (~25)

Column headings: DEPTH SCALE METRES; BORING METHOD; SOIL PROFILE; SAMPLES; HEADSPACE ORGANIC VAPOUR CONCENTRATIONS [PPM] ND = Not Detected (20, 40, 60, 80); HEADSPACE COMBUSTIBLE VAPOUR CONCENTRATIONS [%LEL] ND = Not Detected (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

Piezometer / Standpipe Installation:
- Native Backfill
- Bentonite Seal
- Silica Sand
- 51 mm Diam. PVC #10 Slot Screen
- W.L. in Screen at Elev. 87.63 m on May 3, 2017

DEPTH SCALE 1 : 50

Golder Associates

LOGGED: PAH

CHECKED:

RECORD OF BOREHOLE: 17-07

PROJECT: 1670692

SHEET 1 OF 1

LOCATION: N 5020617.3 ;E 445597.4

BORING DATE: April 26, 2017

DATUM: CGVD28

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

Boring method: Power Auger, 200 mm Diam. (Hollow Stem)

DEPTH SCALE METRES	SOIL PROFILE – DESCRIPTION	ELEV. / DEPTH (m)	SAMPLE NUMBER	TYPE	BLOWS/0.30m	PIEZOMETER OR STANDPIPE INSTALLATION
	GROUND SURFACE	93.80				
0	FILL - (CL-ML) SILTY CLAY and sandy SILT, some gravel; grey brown, contains cobbles; non-cohesive, very moist, loose to compact	0.00	1	GRAB	-	
1			2	SS	11	
			3	SS	5	
2	FILL - (SM) SILTY SAND, fine; grey with clayey silt lumps; non-cohesive, very moist, compact	91.67 / 2.13	4	SS	16	Native Backfill
3			5	SS	12	
	FILL - (CL/ML) SILTY CLAY and CLAYEY SILT, trace gravel; grey; cohesive, w~PL, loose	90.14 / 3.66	6	SS	4	
5			7	SS	4	
			8	SS	3	Bentonite Seal
6						Silica Sand
	(SP) SAND, fine; brown with occasional thin clayey silt seams; non-cohesive, wet, loose to compact	87.40 / 6.40	9	SS	6	51 mm Diam. PVC #10 Slot Screen
7			10	SS	6	
8			11	SS	11	Native Backfill
	End of Borehole	85.57 / 8.23				W.L. in Screen at Elev. 88.20 m on May 2, 2017
9						
10						

HEADSPACE ORGANIC VAPOUR CONCENTRATIONS [PPM] ⊕ *ND = Not Detected* 20 40 60 80

HEADSPACE COMBUSTIBLE VAPOUR CONCENTRATIONS [%LEL] *ND = Not Detected* □ 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 Associates

LOGGED: PAH

CHECKED:

PROJECT: 11-1121-0050

RECORD OF BOREHOLE: 11-01

SHEET 1 OF 1

LOCATION: See Site Plan

BORING DATE: Apr. 11, 2011

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20 40 60 80); SHEAR STRENGTH Cu, kPa 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
		GROUND SURFACE		92.03							
0	Power Auger 200mm Diam. (Hollow Stem)	Grey and grey brown silty clay, some brown silty sand and sand and gravel, with occasional wood fragments (FILL)		0.00							
1					1	50 DO	4				
2					2	50 DO	1				
					3	50 DO	2				
3					4	50 DO	2				
4					5	50 DO	5				
5					6	50 DO	1				
					7	50 DO	2				
6		Compact grey fine SAND, trace silt		85.63 6.40	8	50 DO	24				
				85.32							
7		End of Borehole		6.71							
8											
9											
10											

DRAFT

MIS-BHS 001 11-1121-0050.GPJ GAL-MIS.GDT 4/18/11 JM

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: D.G.

CHECKED: ______

RECORD OF BOREHOLE: 11-02

PROJECT: 11-1121-0050
LOCATION: See Site Plan
SAMPLER HAMMER, 64kg; DROP, 760mm
BORING DATE: Apr. 11, 2011
SHEET 1 OF 1
DATUM: Geodetic
PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.3m	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
0	Power Auger 200mm Diam. (Hollow Stem)	GROUND SURFACE		92.84					
		Grey brown silty clay, some brown silty sand and fine sand, trace organic matter (FILL)		0.00					
1					1	50 DO	4		
2					2	50 DO	2		
					3	50 DO	5		
3					4	50 DO	4		
4					5	50 DO	1		
5					6	50 DO	1		
					7	50 DO	2		
6		Compact grey fine SAND, trace silt		86.74 6.10					∇
					8	50 DO	22		
		End of Borehole		86.13 6.71					
7									W.L. in open borehole at 6.10m depth below ground surface upon completion of drilling
8									
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

DEPTH SCALE
1 : 50

Golder Associates

LOGGED: D.G.
CHECKED: ________

PROJECT: 11-1121-0050

RECORD OF BOREHOLE: 11-03

SHEET 1 OF 1

LOCATION: See Site Plan

BORING DATE: Apr. 11, 2011

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE – DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20 40 60 80); SHEAR STRENGTH Cu, kPa 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
		GROUND SURFACE		94.27							
0	Power Auger 200mm Diam. (Hollow Stem)	Layered brown silty sand and grey brown silty clay, with occasional cobbles, boulders, brick and asphalt fragments (FILL)		0.00							
1					1	50 DO	14		○		
					2	50 DO	5		○		
2											
					3	50 DO	12		○		
3					4	50 DO	>100		○		
4					5	50 DO	>100		○		
5					6	50 DO	20		○		
				89.09							
		End of Borehole		5.18							
6											
7											
8											
9											
10											

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: D.G.

CHECKED: ________

PROJECT: 11-1121-0050

RECORD OF BOREHOLE: 11-04

SHEET 1 OF 1

LOCATION: See Site Plan

BORING DATE: Apr. 11, 2011

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/0.3m	WATER CONTENT PERCENT (W)	PIEZOMETER OR STANDPIPE INSTALLATION
		GROUND SURFACE	95.41					
0	Power Auger 200mm Diam. (Hollow Stem)	Layered grey silty sand/sandy silt, fine sand and grey brown silty clay, trace gravel, with occasional wood fragments (FILL)	0.00					Native Backfill; Bentonite Seal; Native Backfill
~2				1	50 DO	2	O	
~3.4				2	50 DO	18	O	
~5				3	50 DO	12	O	
6.10		Loose light brown fine SAND, trace silt	89.31 / 6.10					
				4	50 DO	8		Bentonite Seal
								Silica Sand
~8				5	50 DO	8		Standpipe
8.23		End of Borehole	87.18 / 8.23					W.L. in Standpipe at Elev. 88.28m on Apr. 15, 2011

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 - O (20, 40, 60, 80); HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-6}, 10^{-5}, 10^{-4}, 10^{-3}); WATER CONTENT PERCENT Wp ⊢—O W—⊣ Wl (20, 40, 60, 80); ADDITIONAL LAB. TESTING.

DRAFT

DEPTH SCALE
1 : 50

Golder Associates

LOGGED: D.G.

CHECKED: ________

RECORD OF TEST PITS

Test Pit Number (Elevation - metres)	Depth (metres)	Description
11-101 (92.36)	0.0 – 4.5 4.5 – 4.8 4.8	Grey brown sandy silt and silty clay, some gravel and shale fragments (FILL) Grey brown SILTY CLAY (Weathered Crust) End of Test Pit Note: Test pit dry upon completion. Sample — Depth (m) 1 — 3.0 2 — 4.2
11-102 (94.29)	0.0 – 5.0 5.0 – 5.2 5.2	Grey brown sandy silt and silty clay, with numerous rock slabs, wood, roots, organic matter, and brick fragments (FILL) Grey SILTY SAND, with gravel End of Test Pit Note: Test pit dry upon completion. Sample — Depth (m) 1 — 1.5 2 — 3.2
11-103 (97.95)	0.0 – 5.0 5.0 – 5.3 5.3	Grey brown sandy silt, trace clay, with cobbles, wood, plastic, and brick fragments (FILL) Grey SILTY SAND, with gravel End of Test Pit Note: Test pit dry upon completion. Sample — Depth (m) 1 — 3.5

Test Pit (Date Excavated)	Depth (m)	Soil description	Sample #	Depth (m)	CSV (ppm)	Remarks
TP01-1 (April 26, 2001)	0.0-3.0	Wet, dark brown to grey, sandy silt to silty sand & gravel(Fill). No odour or sheen.	1	1.0	0	Minor seepage observed at 3.5m
			2	2.0	0	
			3	3.0	0	
	3.0-3.5	Wet, dark brown sand with some silt and native organic (Fill) No odour or sheen.	4	4.0	2-5	
	3.5-4.5	Wet,grey,stratified Sand with trace to some gravel. No odour or sheen.				
TP01-2 (April 26, 2001)	0.0-3.8	Wet,light to dark grey,silty fine sand with trace to some gravel, cobbles and native organic material (Fill). No odour or sheen.	1	1.0	0	
			2	2.0	0	
			3	3.0	0	
	3.8-4.5	Moist to wet, reddish brown, silty sand & gravel with native organics (Fill). No odour or sheen.	4	4.0	0	
TP01-3 (April 26, 2001)	0.0-5.0	Wet, grey, silty sand & gravel with wood, tile, brick, glass, asphalt and concrete (Fill). No odour or sheen.	1	1.0	0	
			2	2.0	0	debris
			3	3.0	0	
	5.0-5.2	Wet,grey,sandy Silt with trace clay and native organics. No odour or sheen.	4	4.0	0	
			5	5.0	0	
TP01-4 (April 26, 2001)	0.0-1.3	Moist to wet, light brown, silty sand & gravel with asphalt, wood,rubber and native organics (Fill). No odour or sheen.	1	1.0	0	
			2	2.0	0	debris
			3	3.0	0	
	1.3-2.5	Moist to wet,grey,sandy Silt with trace to some gravel(Fill). No odour or sheen.				
	2.5-3.0	Compact to dense,wet,light brown,stratified Sand trace gravel. No odour or sheen.				

Test Pit (Date Excavated)	Depth (m)	Soil description	Sample #	Depth (m)	CSV (ppm)	Remarks
TP01-5 (April 26, 2001)	0.0-1.8	Dark brown,moist to wet,silty sand & gravel with asphalt,rubber and native organics(Fill). No odour or sheen.	1	1.0	0	debris
			2	2.0	0	
			3	3.0	5-6	
	1.8-2.0	Wet,dark brown,silty Sand & Gravel with rootlets (Fill). No odour or sheen.	4	4.0	2-4	Minor Seepage at 4.0-4.2 m
	2.0-4.2	Wet,light brown, stratified Sand trace gravel. No odour or sheen.				
TP01-6 (April 26, 2001)	0.0-2.0	Wet, grey, sandy silt with trace gravel and native organics (Fill). No odour or sheen.	1	1.0	0	
			2	2.0	0	
	2.0-3.5	Moist to wet,grey to brown, silty Sand & Gravel with rootlets (Fill). No odour or sheen.	3	3.0	0	
			4	4.0	0	
	3.5-4.5	Wet, light brown, Sand trace gravel (Stratified). No odour or sheen.				
TP01-7 (April 26, 2001)	0.0-5.2	Wet, grey to dark grey, sandy silt with native organics (Fill) No odour or sheen.	1	1.0	0	
			2	2.0	0	
			3	3.0	0	
			4	5.0	0	
TP01-8 (April 26, 2001)	0.0-2.0	Wet, sandy silt with native organics (Fill). No odour or sheen.	1	1.0	0	Minor seepage at 2.0m
			2	2.0	0	
	2.0-4.5	Moist to wet, sand with trace to some gravel (Fill) No odour or sheen.	3	3.0	0	
			4	4.0	0	
TP01-9 (April 26, 2001)	0.0-4.8	Wet, dark grey to grey, sandy silt with some gravel, cobbles and wood (Fill). No odour or sheen.	1	1.0	0	Most material frozen to partially frozen.
			2	2.0	0	
			3	3.0	0	debris
			4	4.0	0	Minor seepage at 4.0m.

PROJECT: 011-2835 3000

RECORD OF BOREHOLE: MW 01-1

SHEET 1 OF 1

LOCATION: See Site Plan

BORING DATE: April 26, 2001

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20 40 60 80); SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - O (20 40 60 80)	HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-6} 10^{-5} 10^{-4} 10^{-3}); WATER CONTENT PERCENT Wp —O W— Wl (10 20 30 40)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		GROUND SURFACE		93.24							
0	Power Auger 200mm DIA. (Hollow Stem)	Brown and grey silty clay and sandy silt, with gravel and organic matter, trace brick (FILL)		0.00							Bentonite Seal w/ Steal Casing
2					1	50 DO	18				Native Backfill
		Loose brown fine sand, trace wood and metal (FILL)		90.50 / 2.74	2	50 DO	7				Bentonite Seal
4		Compact brown stratified fine SAND		89.28 / 3.96	3	50 DO	17				Native Backfill
		Loose brown fine to medium SAND		88.09 / 5.15							
6					4	50 DO	2				50mm PVC Slot Screen
		END OF BOREHOLE		85.77 / 7.47							
8											W.L. in screen at Elev. 88.50 m April 27, 2001
10											
12											
14											
16											
18											
20											

DEPTH SCALE

1 : 100

Golder Associates

LOGGED: D.J.S.

CHECKED: DHP

PROJECT: 011-2835 3000

RECORD OF BOREHOLE: MW 01-2

SHEET 1 OF 1

LOCATION: See Site Plan

BORING DATE: April 27, 2001

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

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

DEPTH SCALE METRES	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. / DEPTH (m)	SAMPLE NUMBER	TYPE	BLOWS/0.3m	PIEZOMETER OR STANDPIPE INSTALLATION
0	GROUND SURFACE		90.69				
	Grey brown silty clay, some gravel and concrete (FILL)		0.00				Bentonite Seal w/ Steal Casing
2				1	50 DO	8	
				2	50 DO	3	Native Backfill
4	Very loose brown fine sand, trace organic matter (FILL)		86.42 / 4.27	3	50 DO	2	
	Brown fine to medium SAND, trace gravel		85.66 / 5.03				Bentonite Seal
6	Compact brown fine to coarse SAND, trace gravel		84.44 / 6.25	4	50 DO	27	
8				5	50 DO	29	Native Backfill
	Dense brown fine to medium SAND		81.45 / 9.24	6	50 DO	33	
10	Compact brown fine to coarse SAND, trace gravel		79.87 / 10.82	7	50 DO	46	
	Dense brown fine to coarse SAND, some gravel		79.11 / 11.58				
12	Brown fine SAND		78.04 / 12.65	8	50 DO	44	50mm PVC Slot Screen
14	END OF BOREHOLE		75.75 / 14.94				W.L in screen at Elev. 78.11 m April 30, 2001

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 10 20 30 40

ADDITIONAL LAB. TESTING

DEPTH SCALE 1 : 100

Golder Associates

LOGGED: D.J.S.

CHECKED: DHP

PROJECT: 011-2898

RECORD OF BOREHOLE: 01-3

LOCATION: 3930 Riverside Drive

BORING DATE: August 8, 2001

DATUM: Local

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/0.3m | 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; 10, 20, 30, 40) | ADDITIONAL LAB. TESTING | PIEZOMETER OR STANDPIPE INSTALLATION

Boring method: POWER AUGER, 200 mm Diam. (Hollow Stem)

Elev. / Depth (m)	Description	Sample Number	Type	Blows/0.3m
92.04	Ground Surface			
92.04 / 0.00	Grey silty clay, trace gravel (FILL)			
90.21 / 1.83	Brown fine SAND	1	50 DO	4
		2	50 DO	5
87.77 / 4.27	Grey brown SILTY SAND, occasional thin silty clay seam			
87.24 / 4.80	Brown fine SAND, with occasional 0.3 to 0.6 m silty sand layer	3	50 DO	25
		4	50 DO	15
		5	50 DO	8
83.81 / 8.23	END OF BOREHOLE			

Piezometer or standpipe installation (top to bottom): Bentonite Seal; Native Backfill; Bentonite Seal; Sand Backfill; 50 mm PVC #10 Slot Screen; Caved Material

W.L. in Screen at Elev. 87.18m Aug. 23, 2001

Depth scale: 0, 2, 4, 6, 8, 10, 12, 14, 16, 18, 20

BOREHOLE 011-2898.GPJ HYDROGEO.GDT 9 25 01

DEPTH SCALE 1 : 100

Golder Associates

LOGGED: D.J.S.

CHECKED: KC

PROJECT: 011-2898

LOCATION:

SAMPLER HAMMER, 64kg; DROP, 760mm

RECORD OF BOREHOLE: 01-5

BORING DATE: 08/08/2001

SHEET 1 OF 1

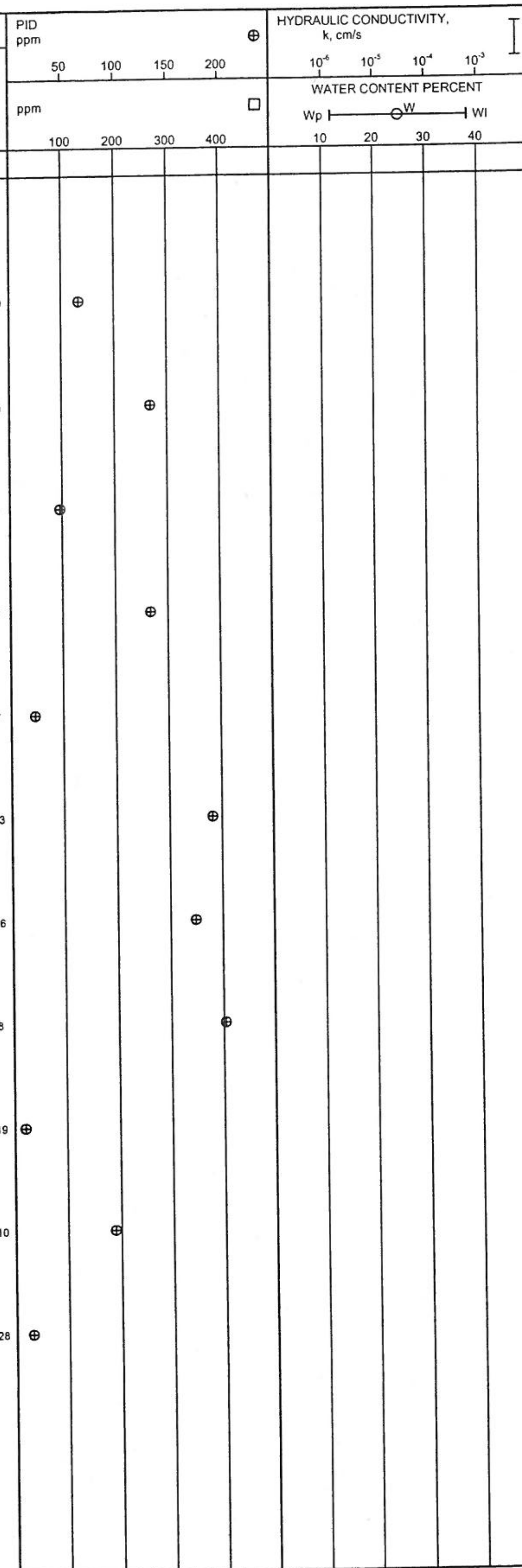
DATUM: Local

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

BORING METHOD: POWER AUGER, 200 mm Diam. (Hollow Stem)

SOIL PROFILE – DESCRIPTION	STRATA PLOT	ELEV. / DEPTH (m)
GROUND SURFACE		93.64 / 0.00
Brown silty clay, trace sand and gravel (FILL)	Fill	93.03 / 0.61
Dark brown sandy silt, some gravel, trace clay, cinders, ashes and wood (FILL)	Fill	90.07 / 3.57
Dark grey silty clay, trace topsoil and brick (FILL)	Fill	88.31 / 5.33
Brown sandy silt, some gravel, trace asphalt and silty sand (FILL)	Fill	85.11 / 8.53
Probably mainly concrete rubble (FILL)	Fill	84.34 / 9.30
Brown sandy silt, some gravel and asphalt (FILL)	Fill	82.06 / 11.58
Dark brown sandy silt, some gravel, trace asphalt and ashes (FILL)	Fill	80.23 / 13.41
Concrete rubble (FILL)	Fill	79.47 / 14.17
Brown sandy silt, some gravel, occasional cobble (FILL)	Fill	78.70 / 14.94
Brown fine to medium SAND, occasional thin sandy silt seam	Sand	78.10 / 15.54
Brown fine to coarse SAND	Sand	15.85
Brown fine to coarse SAND and GRAVEL, occasional cobble	Sand and gravel	76.27 / 17.37
END OF BOREHOLE		

SAMPLES NUMBER	TYPE	BLOWS/0.3m
1	50 DO	10
2	50 DO	31
3	50 DO	4
4	50 DO	7
5	50 DO	7
6	50 DO	13
7	50 DO	16
8	50 DO	8
9	50 DO	19
10	50 DO	10
11	50 DO	28



ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION:

- Bentonite Seal
- Native Backfill
- Bentonite Seal
- Sand Backfill
- 50 mm PVC #10 Slot Screen
- Caved Material

W.L. in Screen at Elev. 78.33m Aug. 23, 2001

DEPTH SCALE 1 : 100

Golder Associates

LOGGED: D.J.S.

CHECKED: [signature]

PROJECT: 011-2898

RECORD OF BOREHOLE: 01-6

SHEET 1 OF 1

LOCATION:

BORING DATE: 09/08/2001

DATUM: Local

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

BORING METHOD: POWER AUGER, 200 mm Diam. (Hollow Stem)

Description	Elev. / Depth (m)	Sample Number	Type	Blows/0.3m
GROUND SURFACE	92.97			
Brown sandy silt, some gravel and cobbles, trace wood (FILL)	0.00			
Brown and dark grey silty sand, trace gravel and wood (FILL)	92.21 / 0.76	1	50 DO	4
Grey brown silty clay, trace to some gravel and concrete (FILL)	90.23 / 2.74	2	50 DO	14
Brown and dark brown silty sand, some gravel and wood (FILL)	89.01 / 3.96	3	50 DO	4
Grey brown silty clay, some dark brown sand, trace organic matter and concrete (FILL)	87.18 / 5.79	4	50 DO	4
		5	50 DO	37
Probably rubble concrete (FILL)	84.74 / 8.23			
Grey brown silty clay, trace organic matter, wood and gravel (FILL)	84.13 / 8.84	6	50 DO	11
Concrete rubble with occasional void (FILL)	83.12	7	50 DO	>50
Grey brown silty clay, some gravel, trace silty sand (FILL)	81.39 / 11.58	8	50 DO	15
Brown and grey silty sand, occasional cobble, trace wood (FILL)	79.86 / 13.11			
Brown SILTY SAND	79.10 / 13.87	9	50 DO	10
Brown SAND and GRAVEL, occasional cobble and boulder	78.19 / 14.78	10	50 DO	>100
		11	50 DO	50/ 50mm
END OF BOREHOLE	75.41 / 17.56			

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 10 20 30 40

ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION:

- Bentonite Seal
- Native Backfill
- Bentonite Seal
- Sand Backfill
- 50 mm PVC #10 Slot Screen

W.L. in Screen at Elev. 76.80m Aug. 23, 2001

DEPTH SCALE 1 : 100

Golder Associates

LOGGED: D.J.S.

CHECKED: TK

BOREHOLE 011-2898.GPJ GLDR_CAN.GDT 9/27/01

RECORD OF TEST PIT 94-8

PROJECT: 941-2735
LOCATION: See Plan
DATE: June 3, 1994
SHEET 1 OF 1
DATUM: Geodetic

Depth (m)	Method	Description	Strata Plot	Elev. / Depth (m)	Sample Number	Type	USCS	Groundwater Conditions
	Backhoe	Ground Surface		92.32				
0	Backhoe	Grey brown silty sand, some gravel (FILL)	Fill	0.00	1	CS	-	
	Backhoe	Dark brown silty sand, some gravel, trace organics, brick (FILL)	Fill	92.12 / 0.20				
	Backhoe	Grey silty sand, some gravel and clay, occasional cobble and boulder, trace brick (FILL)	Fill	0.38	2	CS	-	
					3	CS	-	
					4	CS	-	
					5	CS	-	
	Backhoe	Grey sand, trace silt, gravel, metal tank, wood, occasional cobble and boulder (FILL)	Fill	89.42 / 2.90	6	CS	-	
					7	CS	-	
	Backhoe	Medium brown SAND	Sand	88.12 / 4.20	8	CS	-	
5		End of Test Pit		87.32 / 5.00	9	CS	-	Test Pit wet at 4.2m on completion of excavation

Column headings: HYDRAULIC CONDUCTIVITY, k, cm/s; COMBUSTIBLE VAPOUR (ppm); WATER CONTENT, PERCENT Wp Wn Wl; ADDITIONAL LAB. TESTING (no data entered)

5 0 10 5 PERCENT AXIAL STRAIN AT FAILURE

DATA INPUT: Disk 18, S.Leighton

DEPTH SCALE 1 to 50

Golder Associates

LOGGED: B.S
CHECKED:

PROJECT: 941-2735

LOCATION: See Plan

RECORD OF TEST PIT 94-9

DATE: June 3, 1994

SHEET 1 OF 1

DATUM: Geodetic

DEPTH SCALE METRES	METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	USCS	COMBUSTIBLE VAPOUR (ppm)	HYDRAULIC CONDUCTIVITY, k, cm/s / WATER CONTENT, PERCENT Wp Wn Wl	ADDITIONAL LAB. TESTING	GROUNDWATER CONDITIONS
		Ground Surface		91.31							
0	Backhoe	Grey brown silty sand, trace to some clay, some gravel, trace asphalt, occasional cobble (FILL)		0.00							
					1	CS	-				
1					2	CS	-				
					3	CS	-				
2					4	CS	-				
					5	CS	-				
3				88.31							
		Medium brown SAND		3.00							
				88.11	6	CS	-				Test Pit Dry on completion of excavation
		End of Hole		3.20							
4											
5											
6											
7											
8											
9											
10											

0 5 5 PERCENT AXIAL STRAIN AT FAILURE 10

DATA INPUT: Disk 18, S Leighton

DEPTH SCALE

1 to 50

Golder Associates

LOGGED: B.S

CHECKED: MAYV

PROJECT: 941-2135

LOCATION: See Plan

RECORD OF TEST PIT 94-17

DATE: June 21, 1994

SHEET 1 OF 1

DATUM: Geodetic

Method: Backhoe

DEPTH SCALE METRES	DESCRIPTION	ELEV. / DEPTH (m)	SAMPLE NUMBER	TYPE	USCS	WATER CONTENT, PERCENT	ADDITIONAL LAB. TESTING	GROUNDWATER CONDITIONS
	Ground Surface	91.80						
0	Grey brown sandy silt, some gravel and brick, occasional silty clay pocket (FILL)	0.00	1	CS				
	Brown silty sand, trace organic material, occasional silty clay pocket (FILL)	91.10 / 0.70						
	Grey brown silty clay, some gravel and sand, trace brick (FILL)	88.10 / 3.70	2	CS		Wn ≈ 20	MH	
	Brown SAND	87.50 / 4.30						
5	End of Hole	86.80 / 5.00						W.L in Open Test Pit at 4.8 metres below ground surface

Column headings: SOIL PROFILE; STRATA PLOT; SAMPLES; COMBUSTIBLE VAPOUR (ppm); HYDRAULIC CONDUCTIVITY, k, cm/s; WATER CONTENT, PERCENT Wp — Wn — Wl, 20 40 60 80

DEPTH SCALE 1 to 50

LOGGED: A.F.C

CHECKED:

Golder Associates

DATA INPUT: Disk 18, S.Leighton

PROJECT: 911-2151

LOCATION: See Figure 2

SAMPLER HAMMER, 63.5kg; DROP, 760mm

RECORD OF BOREHOLE 91-1

BORING DATE: June 3&4, 1991

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

DATUM:

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	ELEV. / DEPTH (m)	SAMPLE NUMBER	TYPE	BLOWS/0.3m	PIEZOMETER OR STANDPIPE INSTALLATION
0	Power Auger 200mm Diam (Hollow Stem)	Ground Surface	89.88 / 0.00				Native Backfill
1		Loose brown fine to medium SAND, trace silt, occasional sandy silt seam at depth		1	50 DO	4	
2				2	50 DO	5	Bentonite Seal
3				3	50 DO	9	
		Loose to compact grey brown fine to medium SAND, trace to some silt, some silty clay layers at 6.7 metres depth	86.83 / 3.05	4	50 DO	15	Native Backfill
4				5	50 DO	11	
5				6	50 DO	8	
6				7	50 DO	13	
7				8	50 DO	15	
				9	50 DO	12	Standpipe
8		Stiff to very stiff grey SILTY CLAY, some sand seams and layers	82.26 / 7.62	10	50 DO	3	W.L in Standpipe at Elev.84.75 June 25, 1991
9				11	50 DO	1	NOTE : Standpipe installed in an adjacent borehole
10				12	50 DO	4	
11				13	50 DO	17	
				14	50 DO	10	
12		Loose grey brown fine to coarse SAND, trace gravel and silt	77.88 / 12.20	15	50 DO	7	
13		Possibly Sand	77.08 / 12.80				
14							
15							

Column headings also printed: STRATA PLOT; 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 20 40 60 80; ADDITIONAL LAB. TESTING

CONTINUED ON NEXT PAGE

15 -◆- 5 PERCENT AXIAL STRAIN AT FAILURE (0, 10)

DEPTH SCALE 1 to 75

LOGGED: S.Leighton

CHECKED:

Golder Associates

DATA INPUT: Disk 8, Stever

RECORD OF BOREHOLE 91-1

PROJECT: 911-2151
LOCATION: See Figure 2
SAMPLER HAMMER, 63.5kg; DROP, 760mm

BORING DATE: June 3&4, 1991

SHEET 2 OF 2
DATUM:
PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m / SHEAR STRENGTH Cu, kPa (nat.V - +, rem.V - ⊕, Q - ●, U - O) 20 40 60 80	HYDRAULIC CONDUCTIVITY, k, cm/s / WATER CONTENT, PERCENT Wp —O W— Wl 20 40 60 80	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
15	Power Auger 200mm Diam (Hollow Stem)	CONTINUED FROM PREVIOUS PAGE									
20		Possibly Sand									
26		Possibly Sand and Gravel and Sand		63.67 26.21							
29		End of Hole Auger Refusal		60.77 29.11							

0
15 ◆ 5 PERCENT AXIAL STRAIN AT FAILURE
10

DEPTH SCALE
1 to 75

LOGGED: S.Leighton
CHECKED:

DATA INPUT: Disk 8, Stever

PROJECT: 911-2151

LOCATION: See Figure 2

SAMPLER HAMMER, 63.5kg; DROP, 760mm

RECORD OF BOREHOLE 91-3

BORING DATE: June 20,1991

SHEET 1 OF 1

DATUM:

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	DESCRIPTION	ELEV. / DEPTH (m)	SAMPLE NUMBER	TYPE	BLOWS/0.3m	PIEZOMETER OR STANDPIPE INSTALLATION
0	Power Auger 200mm Diam (Hollow Stem)	Ground Surface	88.05 / 0.00				Native Backfill
1		Very loose to loose brown to grey brown, fine to medium and fine to coarse sand, trace silt (Fill)	86.22 / 1.83	1	50 DO	2	
2		Loose grey SAND, trace silt, trace organic matter and roots	85.92 / 2.13	2	50 DO	8	Standpipe B
3		Loose to compact brown to grey brown fine to coarse SAND, trace silt, occasional clayey silt seam	84.39 / 3.66	3	50 DO	13	Bentonite Seal
				4	50 DO	8	
4		Compact grey layered SAND, some silt and gravel and CLAYEY SILT	83.18 / 4.87	5	50 DO	11	
5		Dense to very dense grey brown SAND, some gravel, trace to some silt	81.34 / 6.71	6	50 DO	30	Caved Material
6				7	50 DO	84	
				8	50 DO	50	
7–9		Dense grey brown fine to medium SAND, trace silt	78.30 / 9.75	9	50 DO	49	Standpipe A
				10	50 DO	39	
10		End of Hole					

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 — 20 40 60 80

ADDITIONAL LAB. TESTING

W.L in Standpipe A at Elev. 86.74 Standpie B dry, June 25, 1991

Tower D

15 - 5 PERCENT AXIAL STRAIN AT FAILURE
0 / 10

DEPTH SCALE 1 to 75

DATA INPUT: Disk 8, Stever

Golder Associates

LOGGED: S.Leighton

CHECKED:

PROJECT: 911-2151

LOCATION: See Figure 2

SAMPLER HAMMER, 63.5kg; DROP, 760mm

RECORD OF BOREHOLE 91-4

BORING DATE: June 21,1991

SHEET 1 OF 1

DATUM:

PENETRATION TEST HAMMER, 63.5kg; 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 - ○) 20 40 60 80	HYDRAULIC CONDUCTIVITY, k, cm/s; WATER CONTENT, PERCENT Wp W Wl 20 40 60 80	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
0	Power Auger 200mm Diam (Hollow Stem)	Ground Surface		92.40							
		Grey silty clay, some dark brown silt with organic matter, some grey brown silty sand and sand, some gravel, wood fragments, asphalt, and organic matter (FILL)		0.00							
1					1	50 DO	10				
2					2	50 DO	16				
3					3	50 DO	9				
					4	50 DO	7				
4		Compact brown fine to medium and fine to coarse SAND, trace to some silt, occasional clayey silt seam at depth		88.54 3.86	5	50 DO	29				
5					6	50 DO	11				▽
6					7	50 DO	16				
					8	50 DO	21				
7		End of Hole		85.54 6.86							W.L in Open Hole at Elev.87.78 on completion of drilling

Depth scale: 0–15 m

Tow

15-◆-5 PERCENT AXIAL STRAIN AT FAILURE (0, 10)

DATA INPUT: Disk 8, Stever

DEPTH SCALE

1 to 75

Golder Associates

LOGGED: S.Leighton

CHECKED:

RECORD OF BOREHOLE 4

841-2470

LOCATION See Figure

BORING DATE NOV. 13, 1984

DATUM

SAMPLER HAMMER, 63.5 kg.; DROP, 760 mm

PENETRATION TEST HAMMER, 63.5 kg.; DROP, 760 mm

BORING METHOD: CME 55 POWER AUGER BORING, 200 mm DIA. HOLLOW STEM AUGER

ELEV'N. / DEPTH (m)	DESCRIPTION	SAMPLE NUMBER	TYPE	BLOWS/0.3 m	ELEVATION SCALE
90.88	GROUND SURFACE				91
0.00	GREY BROWN SILTY CLAY, TRACE GRAVEL, ASPHALT (FILL)				
90.33 / 0.55	LOOSE TO COMPACT BROWN SANDY SILT, SOME GRAVEL, ASPHALT BRICK, TRACE ORGANIC MATTER (FILL)	1	50 mm D.O.	4/150 mm	90
		2	"	5	89
		3	"	19	88
		4	"	4	
87.22 / 3.66	LOOSE BROWN SANDY SILT SOME DARK BROWN ORGANIC MATTER AND WOOD, TRACE GRAVEL AND BRICK (FILL)	5	"	6	87
		6	"	6	86
		7	"	6	
85.09 / 5.79	LOOSE BROWN FINE TO MEDIUM (SAND)	8	50 mm D.O.	4	85
84.17 / 6.71	END OF HOLE				84

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m

SHEAR STRENGTH Cu, kPa NAT. V. - + Q. - ● REM. V. - ⊕ U. - O

HYDRAULIC CONDUCTIVITY, k, cm/sec. 1x10 1x10 1x10 1x10

WATER CONTENT, PERCENT Wp W WL

ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION

BOREHOLE DRY NOVEMBER 1984

15 0 5 10 Percent axial strain at failure

VERTICAL SCALE 1:50 (METRIC)

Golder Associates

DRAWN RWR

CHECKED AC

RECORD OF BOREHOLE 101

LOCATION See Figure 2 — BORING DATE NOV. 8 & 10, 1983 — DATUM GEODETIC

SAMPLER HAMMER, 63.5 kg.; DROP, 760 mm — PENETRATION TEST HAMMER, 63.5 kg.; DROP, 760 mm

BORING METHOD	ELEV'N. / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLE NUMBER	TYPE	BLOWS/0.3m	ADDITIONAL LAB. TESTING
	77.92	GROUND SURFACE				
ROTARY DRILLING, BW CASING	0.00	DENSE BROWN SAND, SOME GRAVEL AND SILT **(FILL)**	1	50 mm D.O.	28	M
	77.22 / 0.70	COMPACT BROWN **SANDY SILT**				
	76.91 / 1.01	COMPACT BROWN FINE TO MEDIUM **SAND** TRACE GRAVEL, SOME SILT	2	"	10	M
	76.09 / 1.83	DENSE BROWN **SAND** AND **GRAVEL**, TRACE TO SOME SILT, OCCASIONAL COBBLE	3	"	27	
			4	"	27	M
			5	"	36	M
	74.26 / 3.66	COMPACT TO DENSE BROWN **FINE** TO **COARSE SAND**, TRACE TO SOME GRAVEL AND SILT	6	"	23	M
			7	"	31	M
	72.74 / 5.18	COMPACT TO DENSE BROWN MEDIUM TO COARSE **SAND** TRACE TO SOME SILT	8	"	22	M
			9	"	32	M
	70.76 / 7.16	VERY DENSE BROWN **SAND** AND **GRAVEL**, TRACE TO SOME SILT, SOME COBBLES	10	"	49	M
			11	"	>100	M
			12	BX R.C.	—	
	69.69 / 8.23	DENSE BROWN FINE TO COARSE **SAND** AND **GRAVEL** TO SAND SOME GRAVEL, TRACE TO SOME SILT	13	50 mm D.O.	34	M
			14	"	42	M
	68.23 / 9.69	VERY DENSE BROWN **SAND** AND **GRAVEL**, SOME COBBLE, TRACE SILT	15	BX R.C.	—	
			16	50 mm D.O.	>100	M
			17	BX R.C.	—	
			18	50 mm D.O.	>100	
	66.95 / 10.97	VERY DENSE BROWN **COARSE GRAVEL** AND COBBLES IN TILL LIKE SILTY SAND MATRIX	19	BX R.C.	—	
			20	50 mm D.O.	>100	
			21	BX R.C.	—	
BXL CORE	64.86 / 13.06	FAIRLY SOUND **LIMESTONE BEDROCK** OCCASIONAL SHALEY INTERBED, FRACTURED SEAM AT 15 m DEPTH	22		—	
			23		—	
	62.31 / 15.61	END OF HOLE				

ELEVATION	CORE RECOVERY (%)	R.Q.D. (%)
65.5–65	41	
64.5	97	25
63.2	94	45

ELEVATION SCALE: 78, 77, 76, 75, 74, 73, 72, 71, 70, 69, 68, 67, 66, 65, 64, 63, 62

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m

SHEAR STRENGTH Cu, kPa — NAT. V. - + Q. - ● REM. V. - ⊕ U. - O

PERMEABILITY, Km cm/sec. 1x10⁻¹ 1x10⁻² 1x10⁻³ 1x10⁻⁴

WATER CONTENT, PERCENT Wp W WL

— CALCULATED

● IN SITU TEST

15 0 5 10 Percent axial strain at failure

PIEZOMETER OR STANDPIPE INSTALLATION

W.L. IN OPEN ... AT ELEV. 7... NOV. 9, 19...

NOTE: ARTI... W.L. FROM ... BEDROCK ... ABOVE GR... SURFACE (ELEV. 81.... NOV. 10, 198...

VERTICAL SCALE

Golder Associates

DRAWN J.C. CHECKED

RECORD OF BOREHOLE 102

LOCATION See Figure 2 BORING DATE MAR. 7, 1984 DATUM GEODETIC

SAMPLER HAMMER, 63.5 kg.; DROP, 760 mm PENETRATION TEST HAMMER, 63.5 kg.; DROP, 760 mm

BORING METHOD: POWER AUGER, 200 mm DIAM. (HOLLOW STEM)

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m

HYDRAULIC CONDUCTIVITY, k, cm/sec. 1x10 1x10 1x10 1x10

SHEAR STRENGTH Cu, kPa NAT. V. - + Q. - ● REM. V. - ⊕ U. - O

WATER CONTENT, PERCENT Wp W WL 20 40 60 80

ELEVATION SCALE: 100, 99, 98, 97, 96, 95, 94, 93, 92, 91, 90, 89, 88, 87, 86, 85, 84, 83

ELEV'N. / DEPTH	SOIL PROFILE DESCRIPTION
99.39	GROUND SURFACE
0.00	VERY LOOSE TO LOOSE BROWN **SAND AND GRAVEL**, TRACE SILT (STOCKPILE)
92.38 / 7.01	VERY DENSE BROWN **SAND AND GRAVEL**, SOME SILT, OCCASIONAL COBBLES
91.16 / 8.23	DENSE TO VERY DENSE BROWN MEDIUM TO COARSE **SAND**, TRACE GRAVEL AND SILT
89.58 / 9.81	COMPACT TO DENSE BROWN FINE TO MEDIUM **SAND**, TRACE SILT
85.83 / 13.56	COMPACT TO DENSE BROWN MEDIUM **SAND**, TRACE SILT
84.00 / 15.39	VERY DENSE BROWN FINE **SAND**, SOME SILT AND SILTY FINE SAND, OCCASIONAL THIN

SAMPLE NUMBER	TYPE	BLOWS/0.3m	ADDITIONAL LAB. TESTING
1	50 mm D.O.	7	
2	"	3	M
3	"	4	
4	"	3	M
5	"	5	
6	"	10	
7	"	8	
8	"	11	M
9	"	>100	
10	"	>100	MH
11	"	96	
12	"	33	M
13	"	36	
14	"	28	MH
15	"	41	
16	"	21	MH
17	"	27	
18	"	6	
19	"	40	M
20	"	47	
21	"	62	MH

PIEZOMETER OR STANDPIPE INSTALLATION: GROUND SURFACE; SURFACE SEAL; NATIVE BACKFILL; PLASTIC TUBING; STANDPIPE

ROTARY DRILLING
BW CASING WITH DRILL MUD

Elev. / Depth	Stratigraphy description
	COMPACT TO DENSE BROWN FINE TO MEDIUM **SAND**, TRACE SILT
[illegible] / 13.56	COMPACT TO DENSE BROWN MEDIUM **SAND**, TRACE SILT
[illegible] / 15.39	VERY DENSE BROWN FINE **SAND**, SOME SILT AND SILTY FINE SAND, OCCASIONAL THIN FINE SAND LAYER
[illegible] / 17.53	VERY DENSE BROWN MEDIUM **SAND**, TRACE SILT
75.85 / 19.51	VERY DENSE BROWN BECOMING GREY **SILTY** FINE **SAND**, OCCASIONAL THIN FINE SAND LAYER, TRACE SHELLS
[illegible] / 23.47	VERY STIFF GREY **CLAYEY SILT** WITH FINE SAND AND SILTY FINE SAND LAYERS, TRACE SHELLS
74.09 / 25.30	COMPACT GREY **SILTY FINE SAND** WITH CLAYEY SILT SEAMS
73.63 / 25.76	VERY STIFF GREY **CLAYEY SILT**, SOME SILTY FINE SAND SEAMS, TRACE SHELLS
72.72 / 26.67	END OF HOLE

Sample No.	Type	Blows
13	"	36
14	"	[illegible]
15	"	41
16	"	21
17	"	27
18	"	6
19	"	40
20	"	47
21	"	62
22	"	70
23	"	68
24	"	[illegible]
25	"	51
26	"	[illegible]
27	"	62
28	"	44
29	"	36
30	"	37
31	"	16
32	"	8
33	"	24
34	"	10

Elevation scale: 89, 88, 87, 86, 85, 84, 83, 82, 81, 80, 79, 78, 77, 76, 75, 74, 73, 72

MH
MH
PLASTIC TUBING
M
MH
STANDPIPE
M
$\pm 1.0-1.5 \times 10^{-7}$
MH
MH
H ⊙
$\pm 1.0-1.5 \times 10^{-7}$
MH
$\pm 1.0-1.5 \times 10^{-7}$
H ⊙
MH

W.L. IN STANDP[IPE]
AT ELEV. 87.[illegible]
APRIL 5, 19[illegible]

15 0 5 10 Percent axial strain at failure

VERTICAL SCALE
1:50

Golder Associates

DRAWN J.C.
CHECKED [illegible]

Form [illegible] Project No. [illegible]-20

RECORD OF BOREHOLE 103

LOCATION See Figure 2

BORING DATE MARCH. 10, 1984

DATUM GEODETIC

SAMPLER HAMMER, 63.5 kg.; DROP, 760 mm

PENETRATION TEST HAMMER, 63.5 kg.; DROP, 760 mm

BORING METHOD	ELEV'N. / DEPTH	DESCRIPTION	NUMBER	TYPE	BLOWS/0.3m	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
POWER AUGER 200 mm DIAM. (HOLLOW STEM)	83.52 / 0.00	GROUND SURFACE					GROUND SURFACE; SURFACE SEAL
		VERY LOOSE TO VERY DENSE BROWN SILTY FINE SAND AND FINE SAND, TRACE WOOD AND GRAVEL (FILL)	1	50 mm D.O.	62		
	81.97 / 1.55	SNOW AND ICE					
	1.68	VERY STIFF GREY BROWN SILTY CLAY, SOME SANDY SEAMS (WEATHERED CRUST)	2	"	4		
			3	"	>100		
	80.93 / 2.59	VERY DENSE BROWN FINE TO COARSE SAND, SOME GRAVEL, TRACE SILT	4	"	>100		
			5	"	90	M	
			6	"	37	M	PLASTIC TUBING
			7	"	44		
			8	"	60	M	
			9	"	55		
	76.20 / 7.32	DENSE BROWN FINE SAND, TRACE SILT					
	75.96 / 7.56	VERY DENSE BROWN SILTY SAND AND GRAVEL, SOME COBBLES	10	"	>100		
			11	BX RC	—		
			12	50 mm D.O.	90	MH	
ROTARY DRILLING WITH DRILLING MUD			13	"	64		STANDPIPE
	73.92 / 9.60	VERY DENSE BROWN MEDIUM TO COARSE SAND, TRACE SILT	14	"	83	M	
	72.94 / 10.58	VERY DENSE BROWN SAND AND GRAVEL, SOME COBBLES	15	"	>100		
			16	BX RC	—		
			17	50 mm D.O.	>100	M	
			18	BX RC	—		
			19	50 mm D.O.	96	M	NATIVE BACKFILL
			20	BX RC	—	M	
			21	50 mm D.O.	55		
			22	BX RC	—		
			23	50 mm D.O.	>100		
			24	BX RC	—		
			25	50 mm D.O.	80	M	
			26	BX RC	—	M	
			27	50 mm D.O.	51		
			28	BX RC	—		
			29	50 mm D.O.	44		

ELEVATION SCALE: 84, 83, 82, 81, 80, 79, 78, 77, 76, 75, 74, 73, 72, 71, 70, 69, 68, 67

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m

SHEAR STRENGTH Cu, kPa — NAT. V. - + Q. - ● REM. V. - ⊕ U. - O

HYDRAULIC CONDUCTIVITY, k, cm/sec. — 1×10^{-4} 1×10^{-3} 1×10^{-2} 1×10^{-1}

WATER CONTENT, PERCENT — Wp W WL — 20 40 60 80

ROTARY DRILLING
BW CASING WITH DRILLING MUD

Elev. / Depth	Stratigraphy description
73.92 / 9.60	VERY DENSE BROWN MEDIUM TO COARSE **SAND**, TRACE SILT
72.94 / 10.58	VERY DENSE BROWN **SAND AND GRAVEL**, SOME COBBLES
59.78 / 23.74	VERY DENSE BROWN **SAND AND GRAVEL**, SOME COBBLES AND BOULDERS, TRACE SILT
58.07 / 25.45	VERY DENSE GREY **FINE TO COARSE SAND**, TRACE GRAVEL AND SILT
57.46 / 26.06	VERY DENSE GREY SILTY SAND TO SANDY SILT, SOME GRAVEL, TRACE CLAY, OCCASIONAL COBBLES AND BOULDERS **(GLACIAL TILL)**
56.51 / 27.01	END OF HOLE

Sample No.	Type	Blows
13	〃	64
14	〃	83
15	〃	>100
16	BX RC	—
17	50 mm DO	>100
18	BX RC	—
19	50 mm DO	96
20	BX RC	—
21	50 mm DO	55
22	BX RC	—
23	50 mm DO	>100
24	BX RC	—
25	50 mm DO	80
26	BX RC	—
27	50 mm DO	51
28	BX RC	—
29	50 mm DO	44
30	〃	48
31	〃	48
32	BX RC	—
33	50 mm DO	62
34	BX RC	—
35	50 mm DO	>100
36	BX RC	—
37	〃	—
38	〃	—
39	50 mm DO	84
40	BX RC	—
41	50 mm DO	>100
42	BX RC	—
43	50 mm DO	78
44	BXL RC	—
45	BXL RC	—
46	50 mm DO	66
47	BXL RC	—
48	50 mm DO	>100

Elevation scale: 74, 73, 72, 71, 70, 69, 68, 67, 66, 65, 64, 63, 62, 61, 60, 59, 58, 57, 56

M M M M M M M M M

STANDPIPE
NATIVE BACKFILL

— CALCULATED
• IN SITU

WL IN STA... AT ELEV... APRIL 5...

15 0 5 10 Percent axial strain at failure

VERTICAL SCALE 1:50

Golder Associates

DRAWN J.C.
CHECKED ...

F... G.A. ...l (m) ... Project No. ...4L...

RECORD OF BOREHOLE 104

LOCATION See Figure 2 | BORING DATE MARCH 17, 1984 | DATUM GEODETIC

SAMPLER HAMMER, 63.5 kg.; DROP, 760 mm | PENETRATION TEST HAMMER, 63.5 kg.; DROP, 760 mm

BORING METHOD: POWER AUGER, 200 mm DIAM. (HOLLOW STEM)

ELEV'N. / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	BLOWS/0.3 m	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
83.32	GROUND SURFACE					GROUND SURFACE
0.00	FROZEN BROWN FINE SAND, TRACE GRAVEL AND COBBLES **(FILL)**	1	50 mm D.O.	>100		SURFACE SEAL
82.04 / 1.28	VERY STIFF GREY BROWN **SILTY CLAY**, OCCASIONAL SAND SEAM (WEATHERED CRUST)	2	"	6		PLASTIC TUBING
81.25 / 2.07	DENSE BROWN FINE TO MEDIUM **SAND**, TRACE SILT	3	"	37		
		4	"	36	M	NATIVE BACKFILL
79.21 / 4.11	DENSE BROWN **SILTY FINE SAND**	5	"	42		
78.44 / 4.88	VERY DENSE BROWN FINE TO COARSE **SAND**, SOME SILT	6	"	67	MH	
77.93 / 5.39	VERY DENSE BROWN **SAND AND GRAVEL**, OCCASIONAL COBBLE, TRACE SILT	7	"	>100		STANDPIPE
		8	"	>100		
77.07 / 6.25	END OF HOLE AUGER REFUSAL PROBABLY BOULDERS					STANDPIPE DESTROYED MAR. ...

ELEVATION SCALE: 84, 83, 82, 81, 80, 79, 78, 77, 76

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m

SHEAR STRENGTH Cu, kPa — NAT. V. - + Q. - ● ; REM. V. - ⊕ U. - O

HYDRAULIC CONDUCTIVITY, k, cm/sec. — 1x10 1x10 1x10 1x10

WATER CONTENT, PERCENT — Wp W WL — 20 40 60 80

15 ⟡ 5 (0, 10) Percent axial strain at failure

VERTICAL SCALE 1:50

Golder Associates

DRAWN J.C.

CHECKED ...

F[illegible] G. A. [illegible] I (m) — Project No. [illegible] 41-[illegible]

RECORD OF BOREHOLE 105

LOCATION See Figure 2 — BORING DATE MAR. 22, 1984 — DATUM GEODETIC

SAMPLER HAMMER, 63.5 kg.; DROP, 760 mm — PENETRATION TEST HAMMER, 63.5 kg.; DROP, 760 mm

BORING METHOD	ELEV'N. / DEPTH	DESCRIPTION	STRATA PLOT	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES BLOWS/0.3m	ELEVATION SCALE	ADDITIONAL LAB. TESTING
							90	
	89.4±	GROUND SURFACE						
POWER AUGER 200 mm DIAM. (HOLLOW STEM)	0.00	BROWN FINE TO COARSE SAND, TRACE GRAVEL (FILL)					89	
	88.64 / 0.76	FROZEN FINE SAND, TRACE SILT (FILL)					88	
							87	
							86	
				1	50 mm D.O.	34	85	
				2	"	18	84	
	83.76 / 5.64	VERY LOOSE BROWN FINE SAND, TRACE SILT (FORMER PIT BOTTOM FILL)		3	"	7		
				4	"	W.R.	83	MH
	82.24 / 7.16	VERY STIFF GREY BROWN SILTY CLAY, OCCASIONAL SAND SEAM (WEATHERED CRUST)		5	"	"	82	
	81.6[illegible] / 7.80	VERY DENSE BROWN FINE TO MEDIUM SAND, TRACE SILT		6	"	>100		
				7	"	"	81	
	79.95 / 9.45	END OF HOLE		8	"	"	80	
							79	

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m

SHEAR STRENGTH Cu, kPa — NAT. V. - + Q. - ● — REM. V. - ⊕ U. - O

HYDRAULIC CONDUCTIVITY, k, cm/sec. — 1x10 1x10 1x10 1x10

WATER CONTENT, PERCENT — Wp W WL — 20 40 60 80

PIEZOMETER OR STANDPIPE INSTALLATION

15 0 5 10 Percent axial strain at failure

VERTICAL SCALE 1:50

Golder Associates

DRAWN J.C.

CHECKED [illegible]

RECORD OF BOREHOLES 81-6 & 81-7

LOCATION See Figure 2 BORING DATE SEPT 9 & 11, 1981 DATUM GEODETIC

SAMPLER HAMMER WEIGHT 63.5 Kg., DROP 0.76 m PENETRATION TEST HAMMER WEIGHT 63.5 Kg., DROP 0.76 m

BORING METHOD: POWER AUGER 0.20 m DIAM. (HOLLOW STEM)

Column headings: SOIL PROFILE (ELEV'N. / DEPTH, DESCRIPTION, STRAT. PLOT); SAMPLES (NUMBER, TYPE, BLOWS/0.3 m); ELEVATION SCALE; DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3 m; SHEAR STRENGTH Cu., K Pa. NAT. V. - + Q. - ● REM.V. - ⊕ U. - O; COEFFICIENT OF PERMEABILITY, k., CM./SEC. Ix10 Ix10 Ix10 Ix10; WATER CONTENT, PERCENT Wp W WL; ADDITIONAL LAB. TESTING; PIEZOMETER OR STANDPIPE INSTALLATION

BH. 81-6

ELEV'N. / DEPTH	DESCRIPTION	NUMBER	TYPE	BLOWS/0.3 m	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
81.9±	GROUND SURFACE					
0.00	LOOSE TO COMPACT LIGHT BROWN FINE TO MEDIUM SAND & SILTY SAND, SOME GRAVEL	1	50 mm D.O.	13	MH	
		2	"	8		
		3	"	17		
78.94 / 2.96	DENSE, GREY BROWN SILTY SAND, SOME GRAVEL (GLACIAL TILL)	4	"	78		
78.24 / 3.66	END OF HOLE					BOREHOLE DRY @ COMPLETION OF DRILLING SEPT. 9, 1981

BH. 81-7

ELEV'N. / DEPTH	DESCRIPTION	NUMBER	TYPE	BLOWS/0.3 m	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
105.2±	GROUND SURFACE					
0.00	BROWN SILTY TOPSOIL					
0.21	COMPACT TO DENSE LIGHT BROWN FINE TO MEDIUM SAND, SOME GRAVEL	1	50 mm D.O.	6		
		2	"	5		
		3	"	29		
		4	"	36		
		5	"	25		
100.23 / 4.97	COMPACT LIGHT BROWN FINE SAND	6	"	27		
5.18	END OF HOLE					BOREHOLE DRY @ COMPLETION OF DRILLING SEPT. 11, 1981

Elevation scale: 82, 81, 80, 79, 78 (BH. 81-6); 105, 104, 103, 102, 101, 100, 99 (BH. 81-7)

15 — 5 Percent axial strain at failure (0, 10)

VERTICAL SCALE 1:50

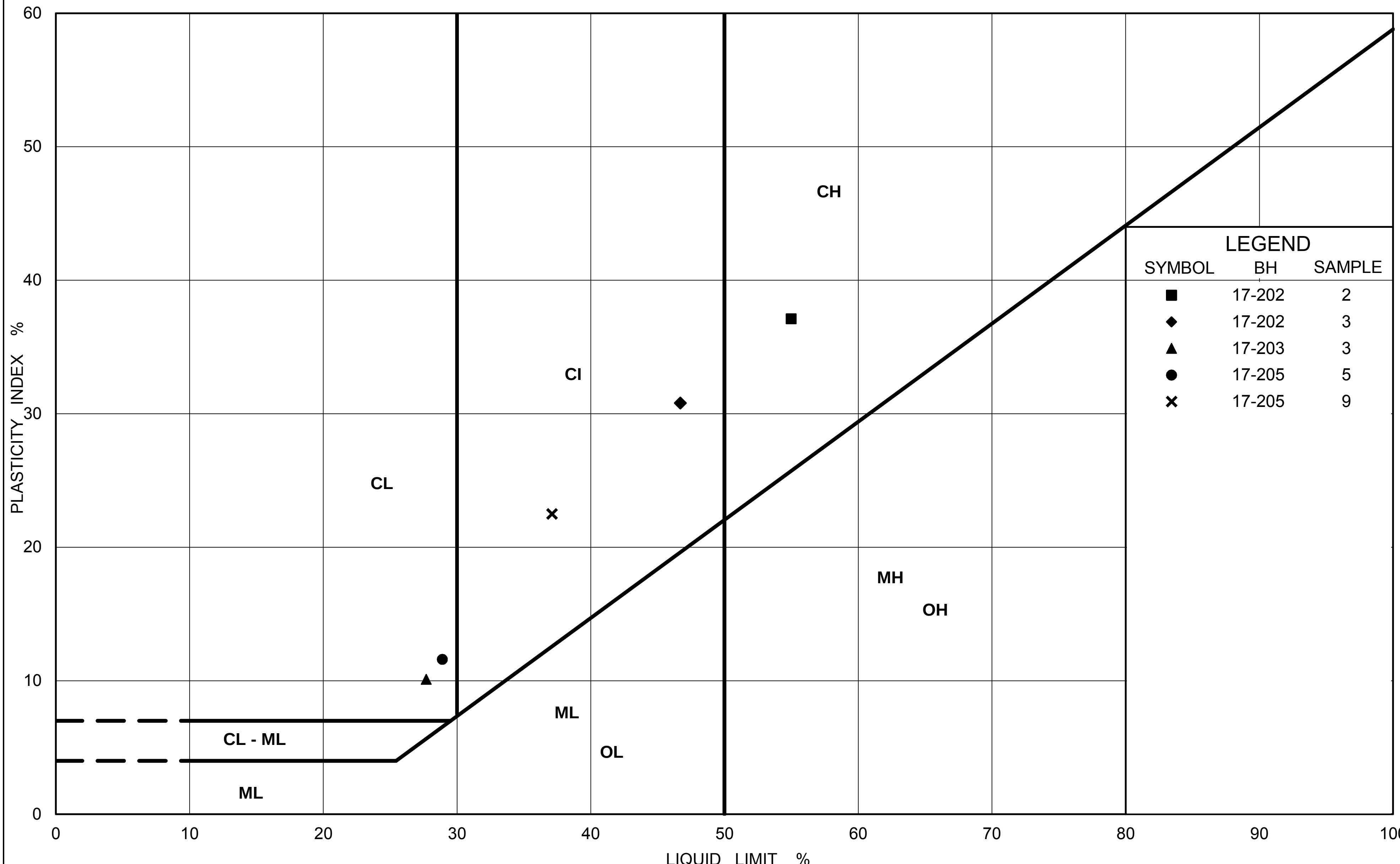
Golder Associates

DRAWN RKB

CHECKED [signature]

APPENDIX D

Results of Laboratory Testing (2017)

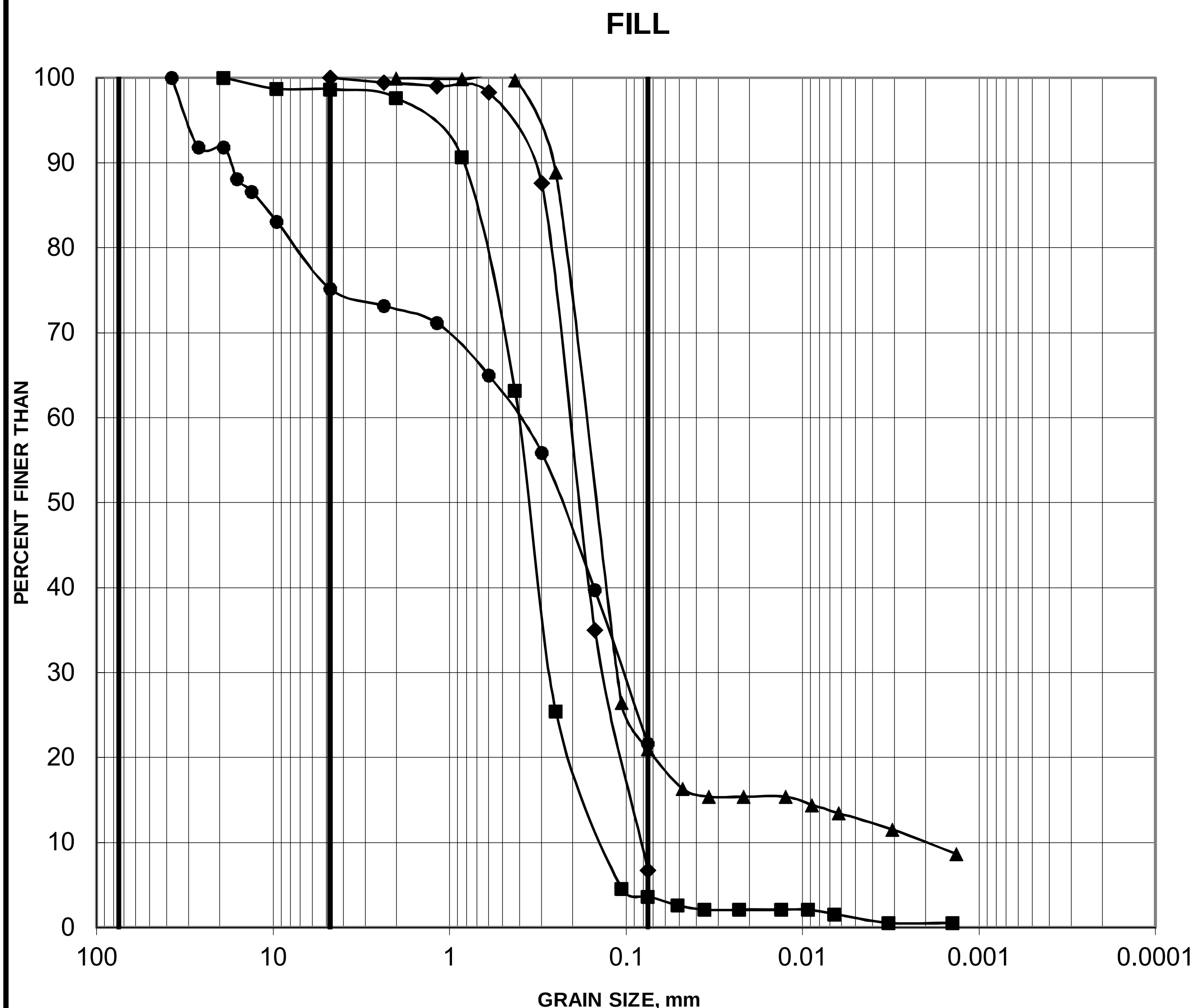


Golder Associates

PLASTICITY CHART
FILL

Figure : D1	
Project No.: 1670692 /4000	
Compiled By : MI	Checked By : CNM

GRAIN SIZE DISTRIBUTION	FIGURE D2



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

Borehole	Sample	Depth (m)
17-201 (■)	3	3.05-3.66
17-201 (◆)	5	6.10-6.71
17-204 (▲)	2	1.52-2.13
17-205 (●)	2	1.52-2.13

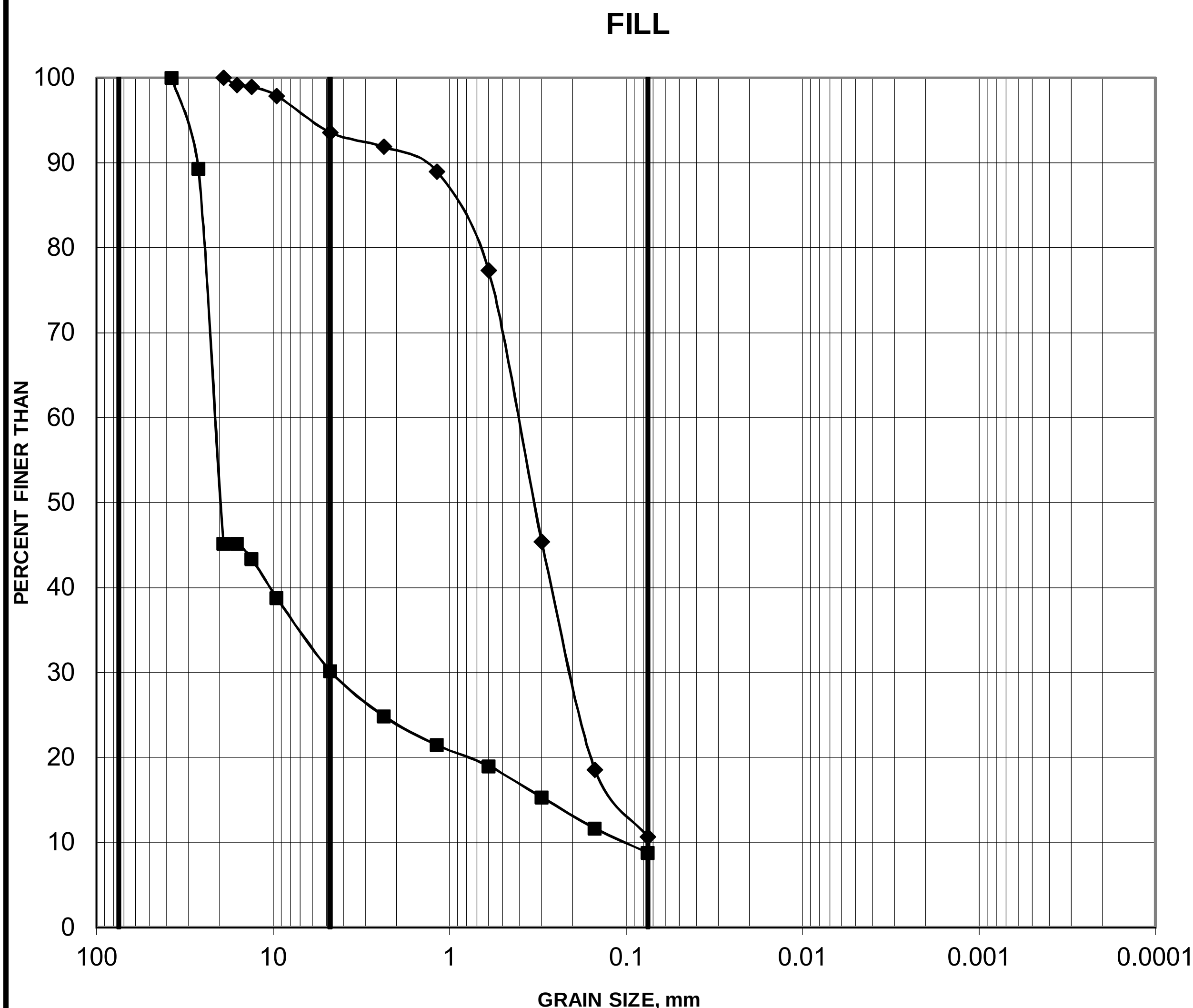
Project: 1670692 /4000

Golder Associates

Created by: MI

Checked by: CNM

GRAIN SIZE DISTRIBUTION	FIGURE D3



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

Borehole	Sample	Depth (m)
17-206	2	1.52-2.13
17-207	4	4.57-5.18

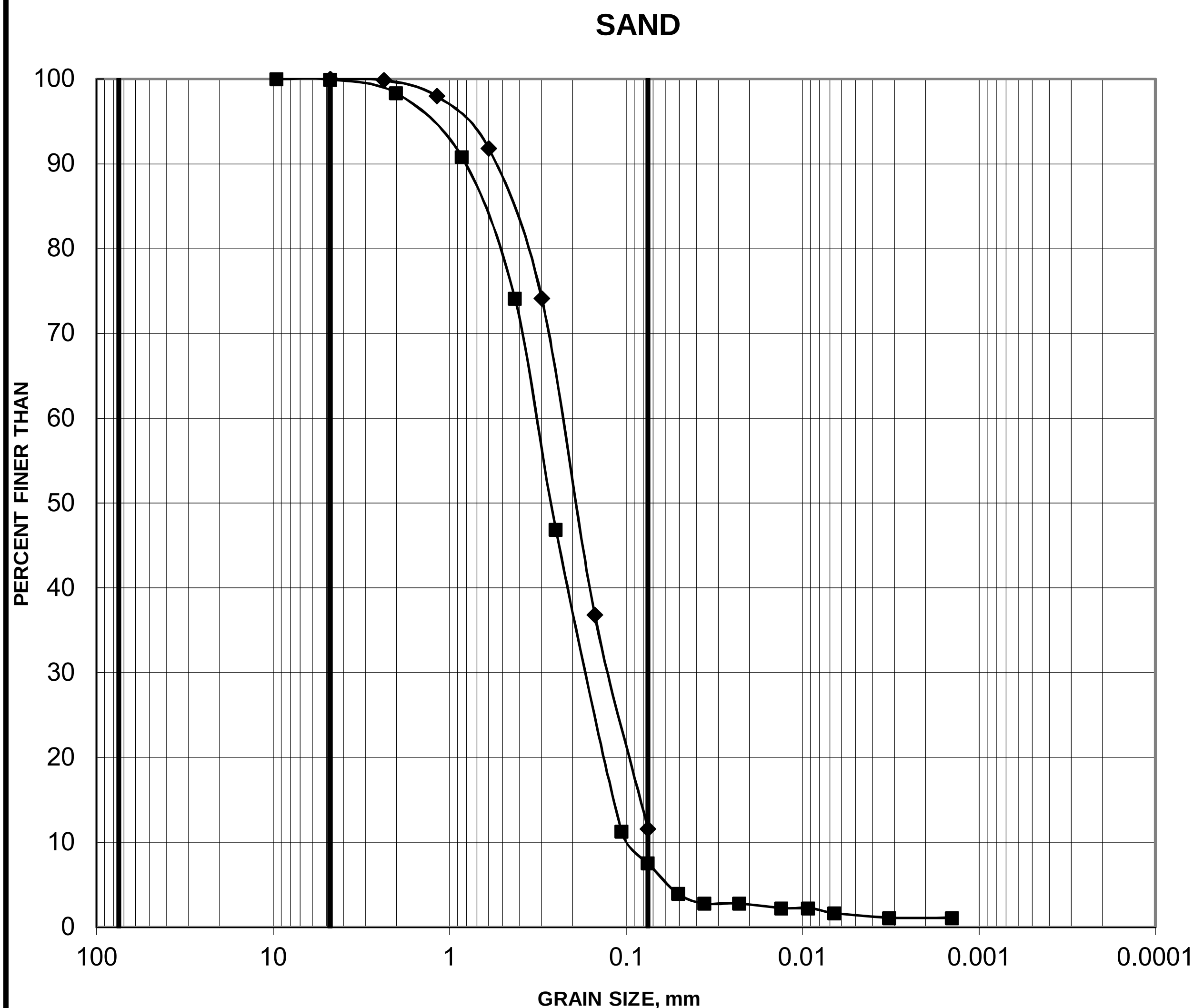
Project: 1670692 /4000

Golder Associates

Created by: MI
Checked by: CNM

GRAIN SIZE DISTRIBUTION

FIGURE D4



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

Borehole	Sample	Depth (m)
■ 17-203	5	6.10-6.71
◆ 17-204	5	5.18-5.79

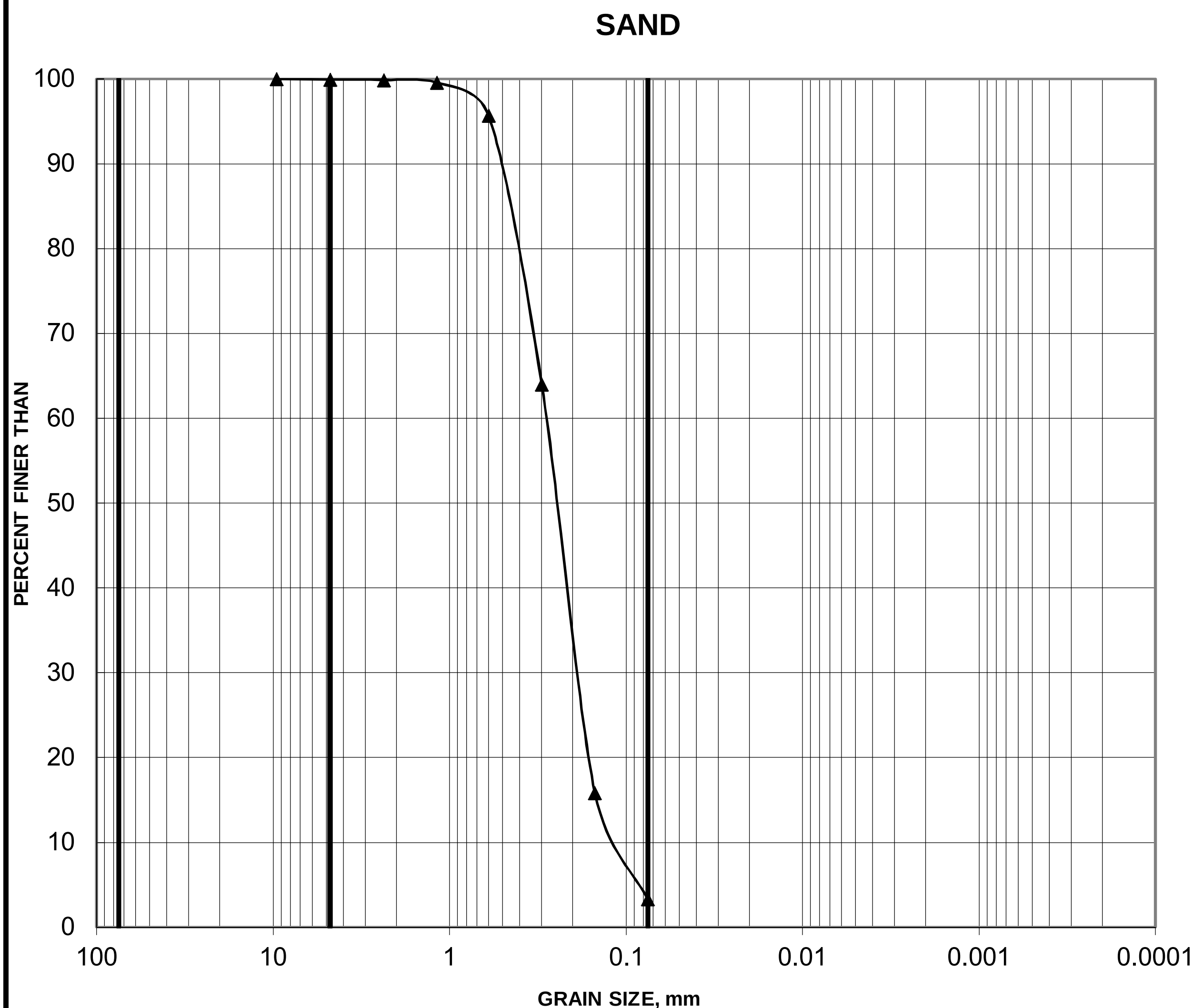
Project: 1670692

Golder Associates

Created by: MI
Checked by: CNM

GRAIN SIZE DISTRIBUTION

FIGURE D5



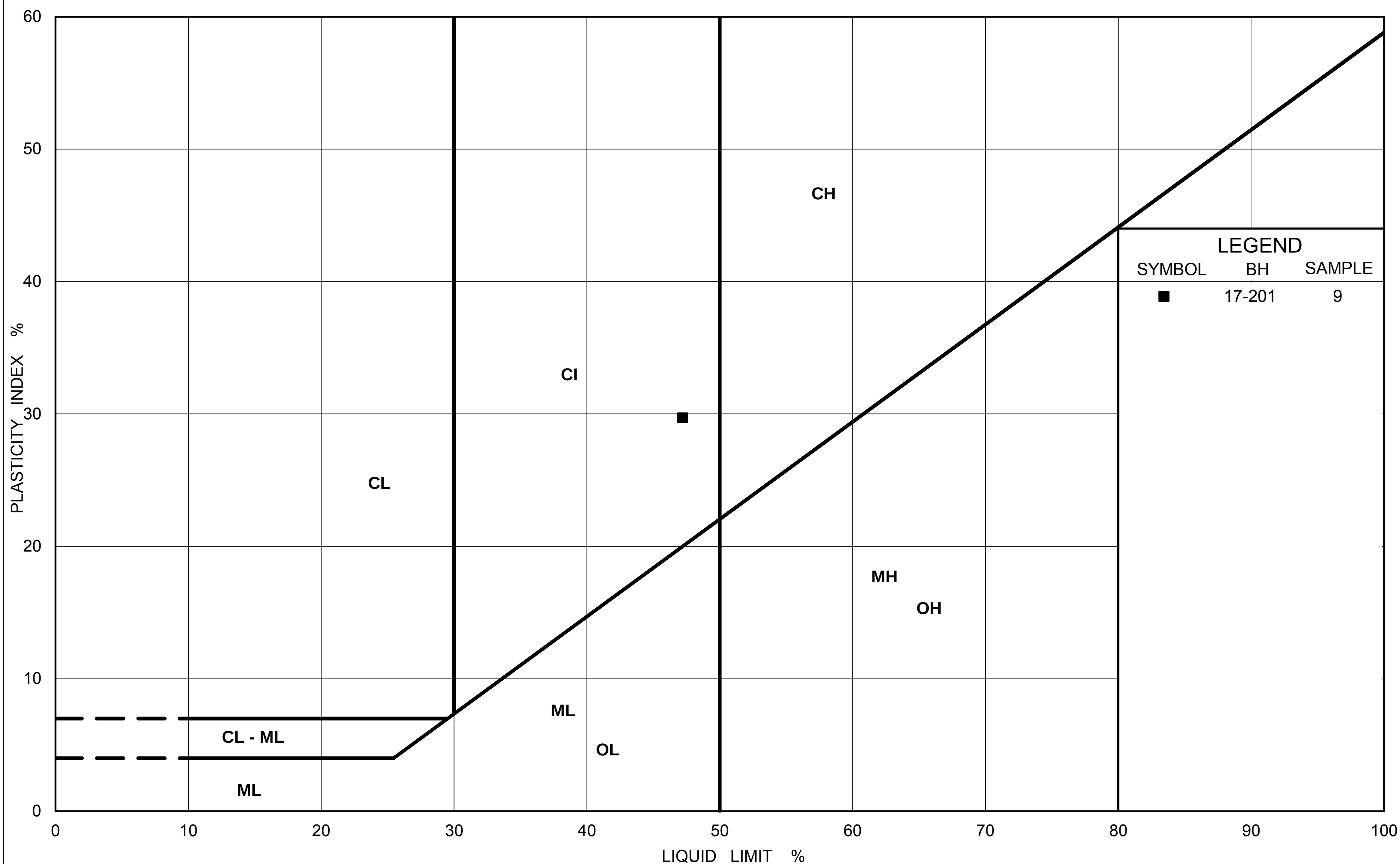
Cobble	coarse	fine	coarse	medium	fine	SILT AND CLAY
Size	GRAVEL SIZE		SAND SIZE			

Borehole	Sample	Depth (m)
17-206	15	16.00-16.61

Project: 1670692 /4000

Golder Associates

Created by: MI
Checked by: CNM



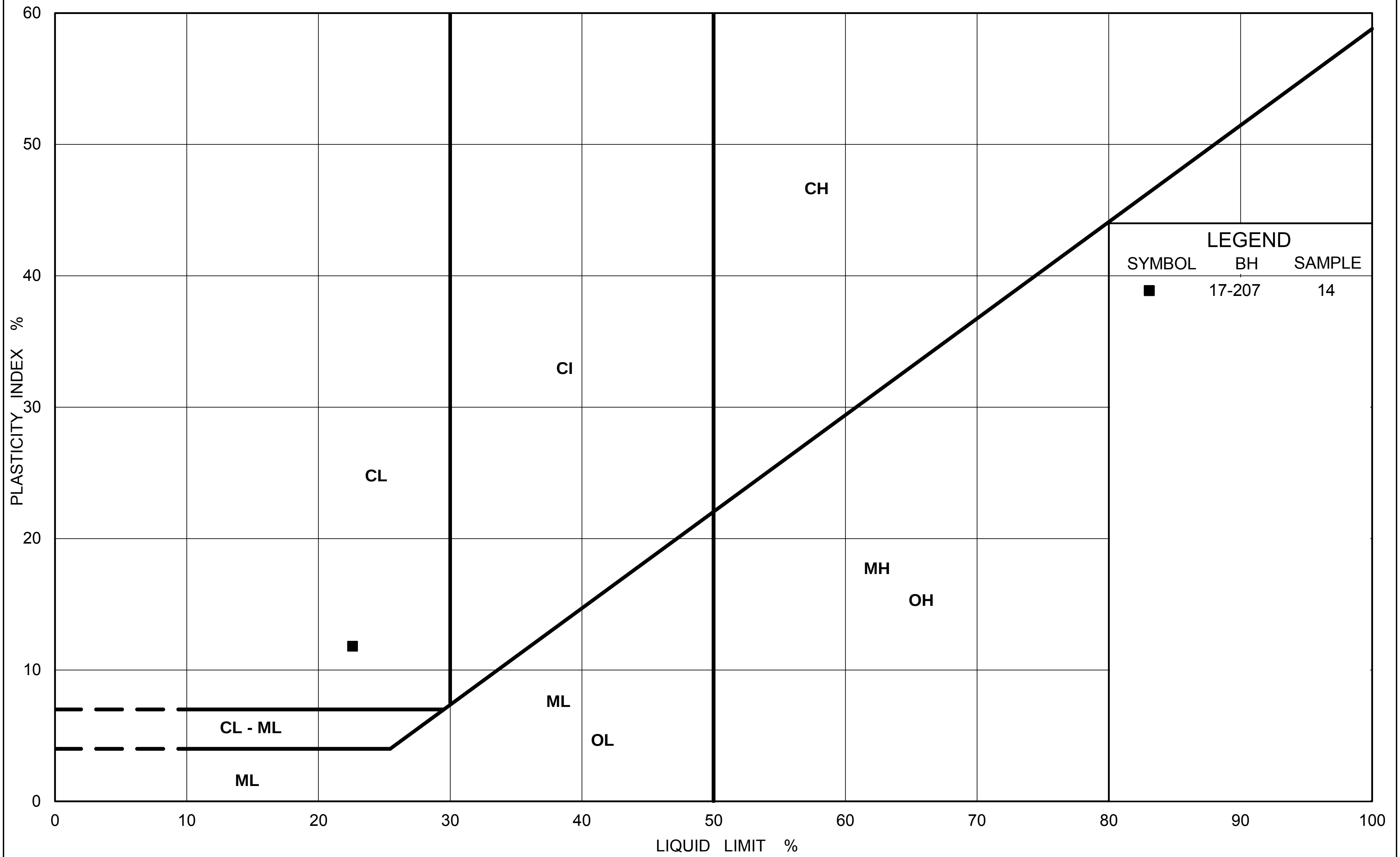
Golder Associates

PLASTICITY CHART
Sandy SILTY CLAY

Figure : D6

Project No.: 1670692 /4000

Compiled By : MI Checked By : CNM



Golder Associates

PLASTICITY CHART
CLAYEY SILT and SILTY CLAY

Figure : D7

Project No.: 1670692 /4000

Compiled By : MI Checked By : CNM

APPENDIX E

Results of CPT Investigation (2023)

PRESENTATION OF SITE INVESTIGATION RESULTS

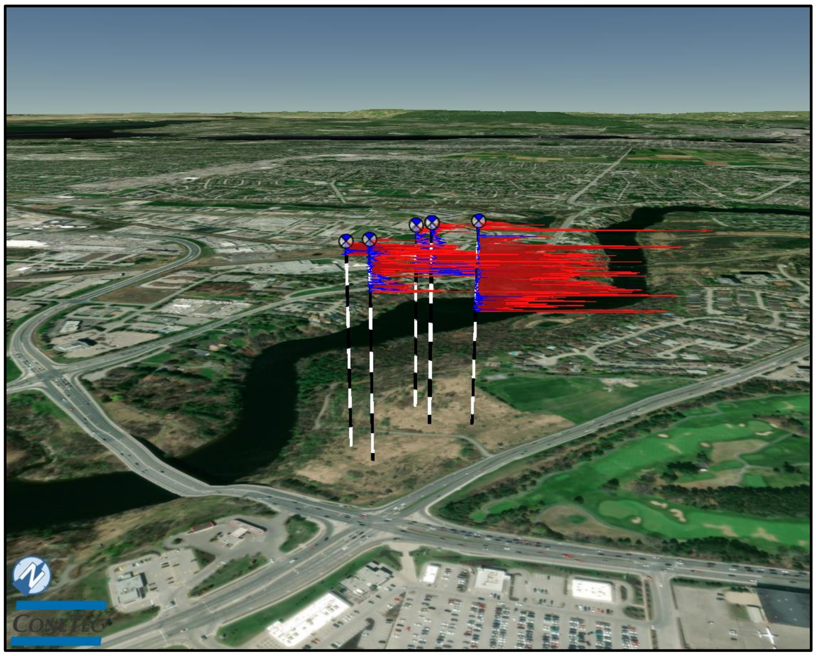
Riverside Dr and Hunt Club Rd

Prepared for:

WSP Canada Inc.

ConeTec Job No: 23-05-25254

Project Start Date: 27-Feb-2023
Project End Date: 02-Mar-2023
Report Date: 10-Mar-2023



Prepared by:

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www.conetecdataservices.com

Introduction

The enclosed report presents the results of the site investigation program conducted by ConeTec Investigations Ltd. for WSP Canada Inc. near Riverside Dr and Hunt Club Rd in Ottawa, ON. The program consisted of 6 cone penetration tests (CPTu) and 5 seismic cone penetration tests (SCPTu). Please note that this report, which also includes all accompanying data, are subject to the 3rd Party Disclaimer and Client Disclaimer that follow in the 'Limitations' section of this report.

Project Information

Project	
Client	WSP Canada Inc.
Project	Riverside Dr and Hunt Club Rd
ConeTec project number	23-05-25254

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

Rig Description	Deployment System	Test Type
CPT track rig (TC23)	30 ton rig cylinder	CPTu and SCPTu

Coordinates		
Test Type	Collection Method	EPSG Number
CPTu and SCPTu	Consumer grade GPS	32618

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 (bar)
947:T1000F10U35	947	15	225	1000	10	35
Cone 947 was used for all CPTu soundings.						

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 plots with expanded range • Advanced plots with Ic, Su, phi and N1(60) • Shear wave velocity plots • Soil Behaviour Type (SBT) scatter plots

Calculated Geotechnical Parameter Tables	
Additional information	The Normalized Soil Behaviour Type Chart based on Q_{tn} (SBT Q_{tn}) (Robertson, 2009) was used to classify the soil for this project. A detailed set of calculated CPTu parameters have been generated and are provided in Excel format files in the release folder. The CPTu parameter calculations are based on values of corrected tip resistance (q_t) sleeve friction (f_s) and pore pressure (u_2). Effective stresses are calculated based on unit weights that have been assigned to the individual soil behaviour 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 and sand mixtures (zones 4 and 5). Equilibrium pore pressure profiles generated from the pore pressure dissipation data and assumed equilibrium points were used for the calculated parameters. Based on the dynamic pore pressure response, hydrostatic conditions were assumed after the last equilibrium pore pressure point. The equilibrium pore pressure profile points and profile line, as well as the hydrostatic line are plotted on the dynamic pore pressure for comparison.

Limitations

3rd Party Disclaimer

This report titled “Riverside Dr and Hunt Club Rd”, referred to as the (“Report”), was prepared by ConeTec for WSP Canada Inc.. The Report is confidential and may not be distributed to or relied upon by any third parties without the express written consent of ConeTec. Any third parties gaining access to the Report do not acquire any rights as a result of such access. Any use which a third party makes of the Report, or any reliance on or decisions made based on it, are the responsibility of such third parties. ConeTec accepts no responsibility for loss, damage and/or expense, if any, suffered by any third parties as a result of decisions made, or actions taken or not taken, which are in any way based on, or related to, the Report or any portion(s) thereof.

Client Disclaimer

ConeTec was retained by WSP Canada Inc. to collect and provide the raw data (“Data”) which is included in this report titled “Riverside Dr and Hunt Club Rd”, which is referred to as the (“Report”). ConeTec has collected and reported the Data in accordance with current industry standards. No other warranty, express or implied, with respect to the Data is made by ConeTec. In order to properly understand the Data included in the Report, reference must be made to the documents accompanying and other sources referenced in the Report in their entirety. Any analysis, interpretation, judgment, calculations and/or geotechnical parameters (collectively “Interpretations”) included in the Report, including those based on the Data, are outside the scope of ConeTec’s retainer and are included in the Report as a courtesy only. Other than the Data, the contents of the Report (including any Interpretations) should not be relied upon in any fashion without independent verification and ConeTec is in no way responsible for any loss, damage or expense resulting from the use of, and/or reliance on, such material by any party.

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 two geophone sensors for recording seismic signals. All signals are amplified and measured with minimum sixteen-bit resolution down hole within the cone body, and the signals are sent to the surface using a high bandwidth, error corrected digital interface through a shielded cable.

ConeTec penetrometers are manufactured with various tip, friction and pore pressure capacities in both 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 millimeters diameter over a length of 32 millimeters with tapered leading and trailing edges) located at a distance of 585 millimeters 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 six millimeters 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.

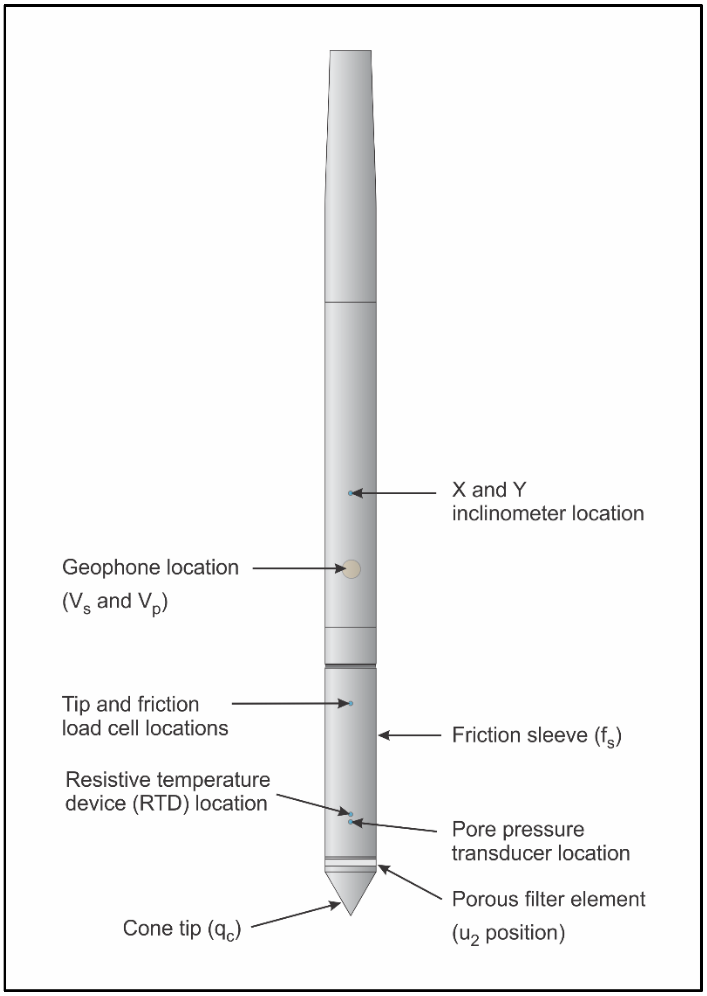


Figure CPTu. Piezocone Penetrometer (15 cm^2)

The ConeTec data acquisition systems consist of a Windows based computer and a signal interface box and power supply. The signal interface combines depth increment signals, seismic trigger signals and the downhole digital data. This combined data is then sent to the Windows based computer for collection and presentation. 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 centimeters; 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 CPTu 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 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 two centimeters per second, within acceptable tolerances. Typically one meter length rods with an outer diameter of 38.1 millimeters 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
- 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):

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

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).

References

ASTM D5778-20, 2020, "Standard Test Method for Performing Electronic Friction Cone and Piezocone Penetration Testing of Soils", ASTM International, West Conshohocken, PA. DOI: 10.1520/D5778-20.

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. DOI: 10.1061/9780784412770.027.

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.

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

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. DOI: 10.1139/T90-014.

Robertson, P.K., 2009, "Interpretation of cone penetration tests – a unified approach", Canadian Geotechnical Journal, Volume 46: 1337-1355. DOI: 10.1139/T09-065.

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 15 cm^2 piezocone penetrometers are manufactured with one horizontally active geophone (28 hertz) and one vertically active geophone (28 hertz). Both geophones are rigidly mounted in the body of the cone penetrometer, 0.2 meters behind the cone tip. The vertically mounted geophone is more sensitive to compression waves; however, it is often affected by the compression wave travelling through the cone rods.

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 may be 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 in the memory of the cone using a fast analog to digital converter. The seismic trace is then transmitted digitally uphole to a Windows based computer through a signal interface box for recording and analysis. An illustration of the shear wave testing configuration is presented in Figure SCPTu-1.

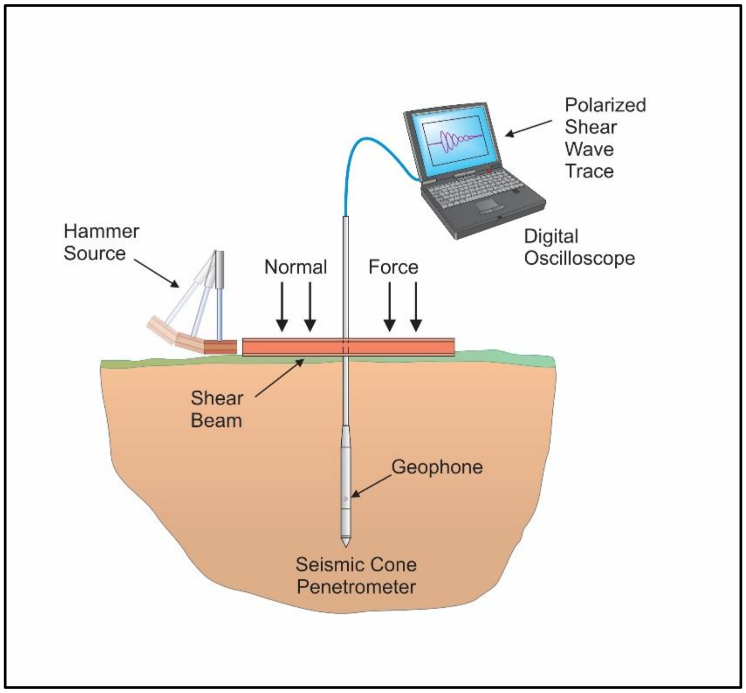


Figure SCPTu-1. Illustration of the SCPTu system

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).

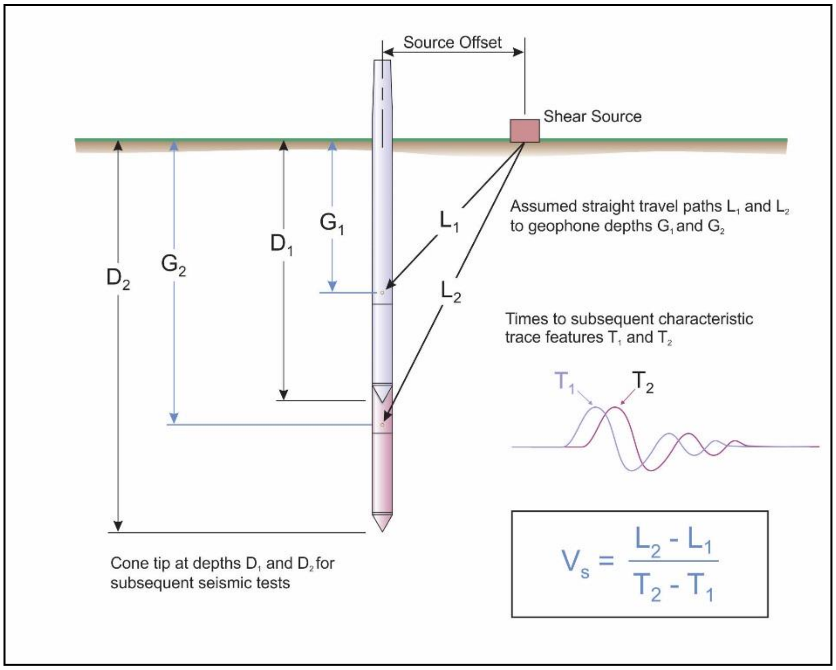


Figure SCPTu-2. Illustration of a seismic cone penetration test

For the determination of interval travel times the wave traces from all depths are displayed in analysis software. The results of the interval picks are supplied in the relevant appendix of this report. Standard practice for ConeTec is to record five wave traces for each source direction at each test depth. Outlier impacts are identified in the field and the impacts are repeated. For the final wave trace profile, the traces are stacked in the time domain to display a single average trace.

Determination of the shear wave interval velocities are performed by visually picking a common feature (e.g. the first characteristic peak, trough, or crossover) on all of the trace depths and taking the difference in ray path divided by the time difference between features at subsequent depths. The same process is used for compression waves, however the first break is most commonly used for selecting an arrival time. For velocity calculation, the 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.

In some cases, usually for shear wave velocity testing, more than one characteristic marker may be used. If there is an overlap between different sets of characteristic markers, then the average time value for those sets of interval times is applied to the determination of velocity.

Ideally, all depths are used for the determination of the velocity profile. However, an interval may be skipped if there is some ambiguity or quality concern with a particular depth, resulting in a larger interval.

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

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

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

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

References

ASTM D5778-20, 2020, "Standard Test Method for Performing Electronic Friction Cone and Piezocone Penetration Testing of Soils", ASTM International, West Conshohocken, PA. DOI: 10.1520/D5778-20.

ASTM D7400/D7400M-19, 2019, "Standard Test Methods for Downhole Seismic Testing", ASTM International, West Conshohocken, PA. DOI: 10.1520/D7400_D7400M-19.

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.

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. DOI: 10.1061/(ASCE)0733-9410(1986)112:8(791).

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).

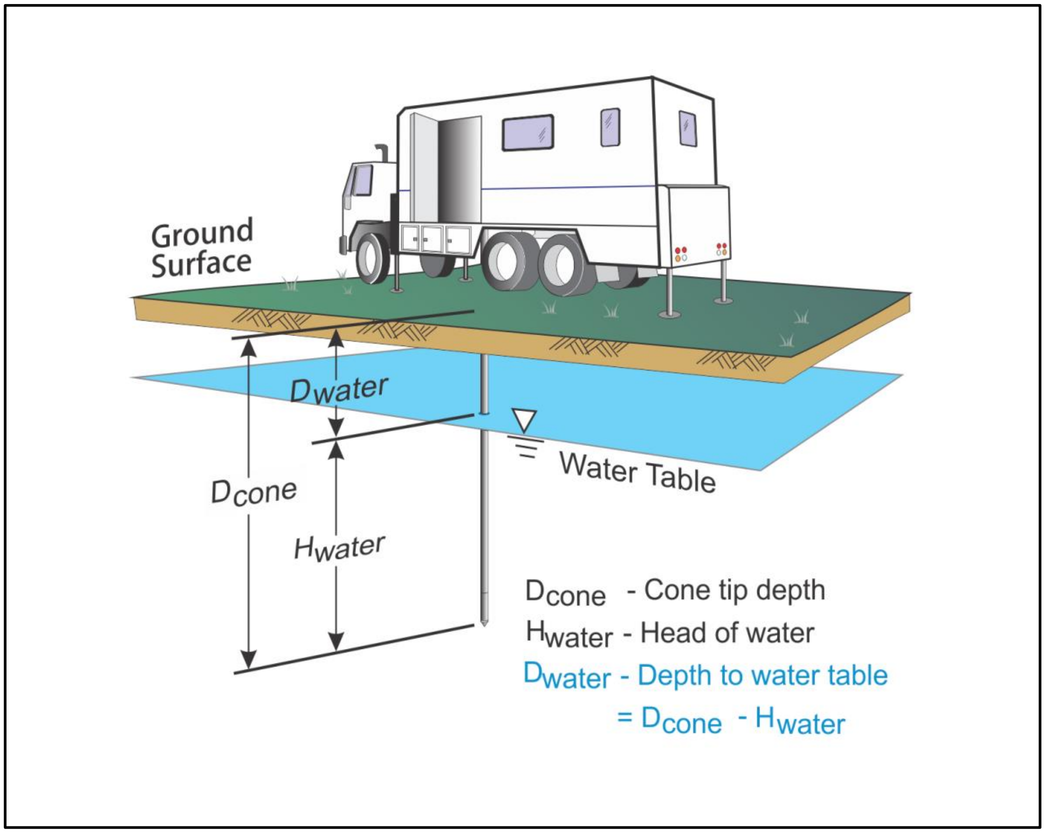


Figure PPD-1. Pore pressure dissipation test setup

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.

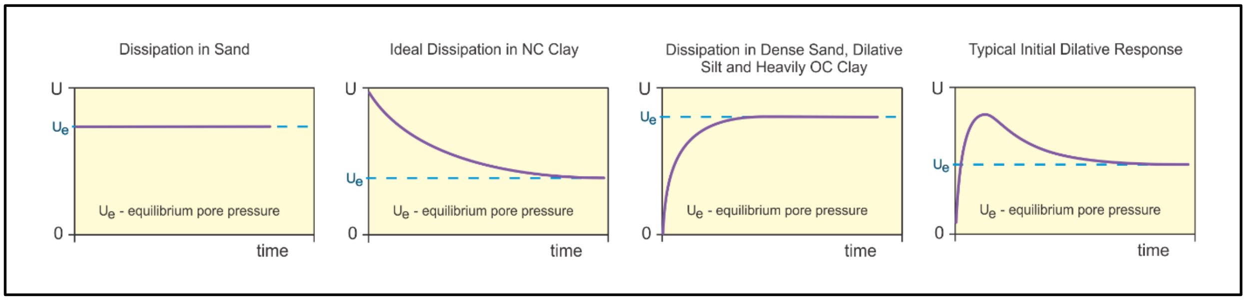


Figure PPD-2. Pore pressure dissipation curve examples

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.

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

References

Teh, C.I., and Houlsby, G.T., 1991, "An analytical study of the cone penetration test in clay", Geotechnique, 41(1): 17-34. DOI: 10.1680/geot.1991.41.1.17.

The appendices listed below are included in the report:

- Cone Penetration Test Summary and Standard Cone Penetration Test Plots
- Standard Cone Penetration Test Plots with Expanded Range
- Advanced Cone Penetration Test Plots with Ic, Su(Nkt), Phi and N1(60)Ic
- Seismic Cone Penetration Test Plots
- Seismic Cone Penetration Test Shear Wave (Vs) Tabular Results
- Seismic Cone Penetration Test Shear Wave (Vs) Traces
- Soil Behaviour Type (SBT) Scatter Plots
- Pore Pressure Dissipation Summary and Pore Pressure Dissipation Plots
- Description of Methods for Calculated CPT Geotechnical Parameters

Cone Penetration Test Summary and Standard Cone Penetration Test Plots

Job No: 23-05-25254
Client: WSP Canada Inc.
Project: Riverside Dr and Hunt Club Rd
Start Date: 27-Feb-2023
End Date: 02-Mar-2023

CONE PENETRATION TEST SUMMARY

Sounding ID	File Name	Date	Cone	Cone Area (cm^2)	Assumed Phreatic Surface[1] (m)	Final Depth (m)	Northing[2] (m)	Easting[2] (m)	Refer to Notation Number
SCPT23-01	23-05-25254_SP01	01-Mar-2023	947:T1000F10U35	15		2.650	5020457	445550	3, 4
SCPT23-01B	23-05-25254_SP01B	01-Mar-2023	947:T1000F10U35	15		2.925	5020456	445550	3
SCPT23-01C	23-05-25254_SP01C	02-Mar-2023	947:T1000F10U35	15	10.9	11.325	5020451	445588	
SCPT23-01D	23-05-25254_SP01D	02-Mar-2023	947:T1000F10U35	15	10.9	11.425	5020451	445589	
CPT23-02	23-05-25254_CP02	28-Feb-2023	947:T1000F10U35	15		0.575	5020561	445572	3
CPT23-02B	23-05-25254_CP02B	28-Feb-2023	947:T1000F10U35	15	11.8	12.225	5020561	445573	
CPT23-02C	23-05-25254_CP02C	01-Mar-2023	947:T1000F10U35	15	11.8	12.900	5020561	445574	
SCPT23-03	23-05-25254_SP03	01-Mar-2023	947:T1000F10U35	15	4.5	12.350	5020585	445530	
SCPT23-03B	23-05-25254_SP03B	01-Mar-2023	947:T1000F10U35	15	4.8	11.800	5020584	445530	
CPT23-04	23-05-25254_CP04	27-Feb-2023	947:T1000F10U35	15		2.225	5020591	445613	3
CPT23-04B	23-05-25254_CP04B	28-Feb-2023	947:T1000F10U35	15	7.4	22.325	5020590	445613	

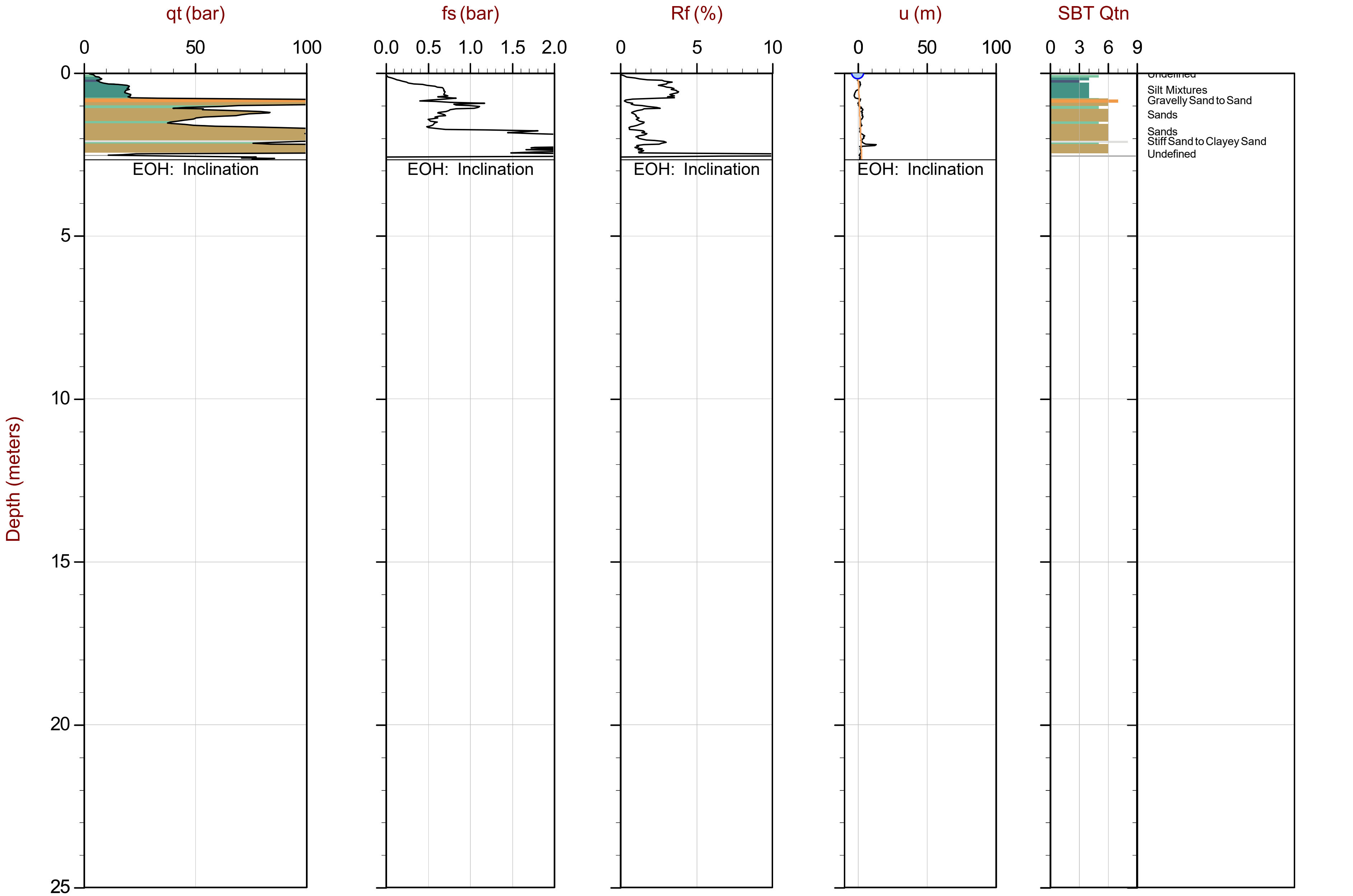
1. The assumed phreatic surface was based on pore pressure dissipation tests unless otherwise noted. Equilibrium pore pressure profiles were assumed for the calculated parameters.
2. Coordinates were collected with a consumer grade GPS device with datum WGS84/UTM Zone 18 North.
3. No clear phreatic surface detected.
4. No shear wave data due to the source offset exceeding the test depth.

CONETEC

WSP Canada

Job No: 23-05-25254
Date: 2023-03-01 15:36
Site: 3630 and 3690 Riverside Drive

Sounding: SCPT23-01
Cone: 947:T1000F10U35



Max Depth: 2.650 m / 8.69 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-05-25254_SP01.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 18N N: 5020457m E: 445550m
Sheet No: 1 of 1

Overplot Item: Ueq Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Dissipation, Ueq assumed Ueq Line Hydrostatic Line

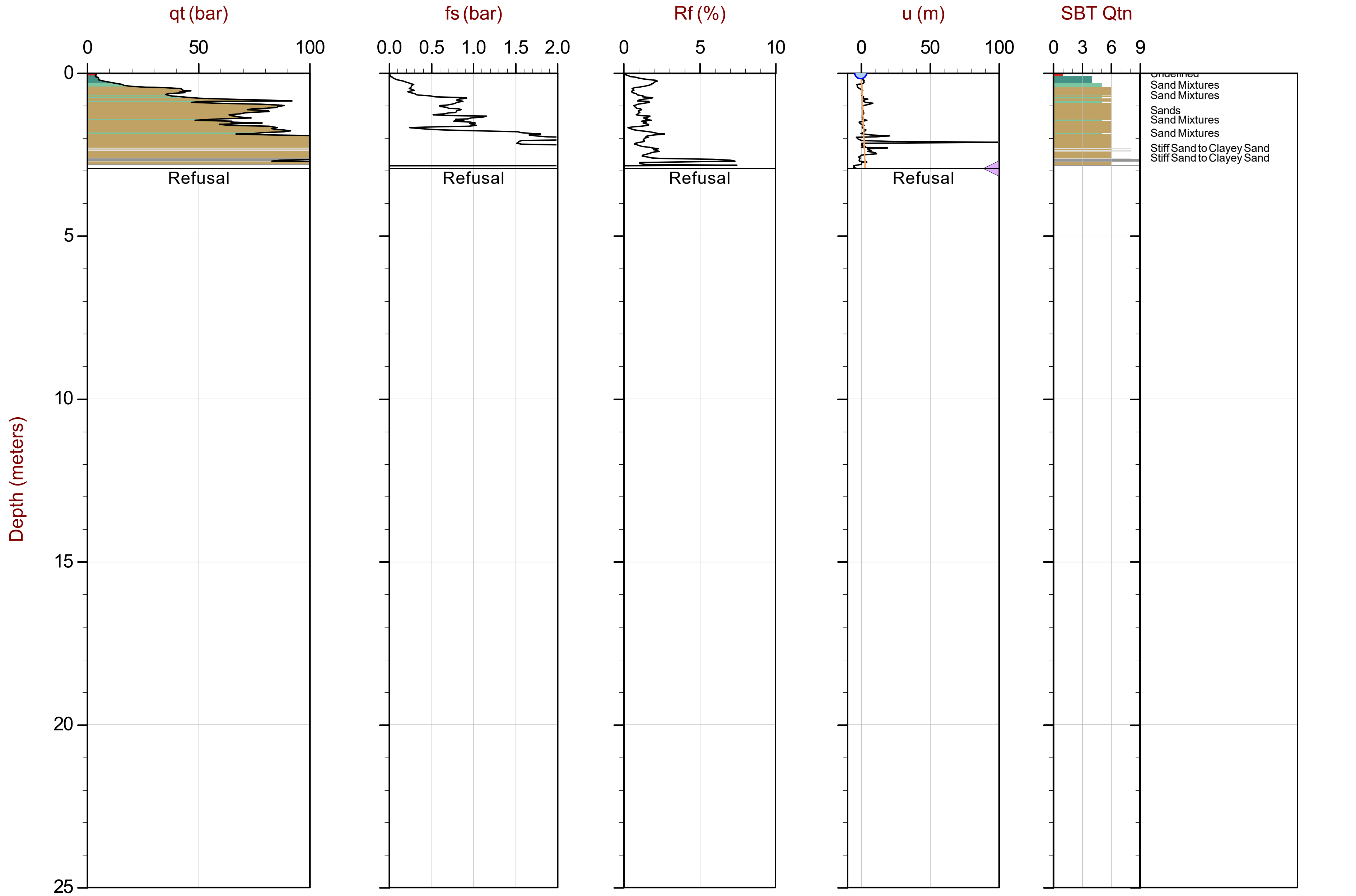
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

WSP Canada

Job No: 23-05-25254
Date: 2023-03-01 16:44
Site: 3630 and 3690 Riverside Drive

Sounding: SCPT23-01B
Cone: 947:T1000F10U35



Max Depth: 2.925 m / 9.60 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-05-25254_SP01B.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 18N N: 5020456m E: 445550m
Sheet No: 1 of 1

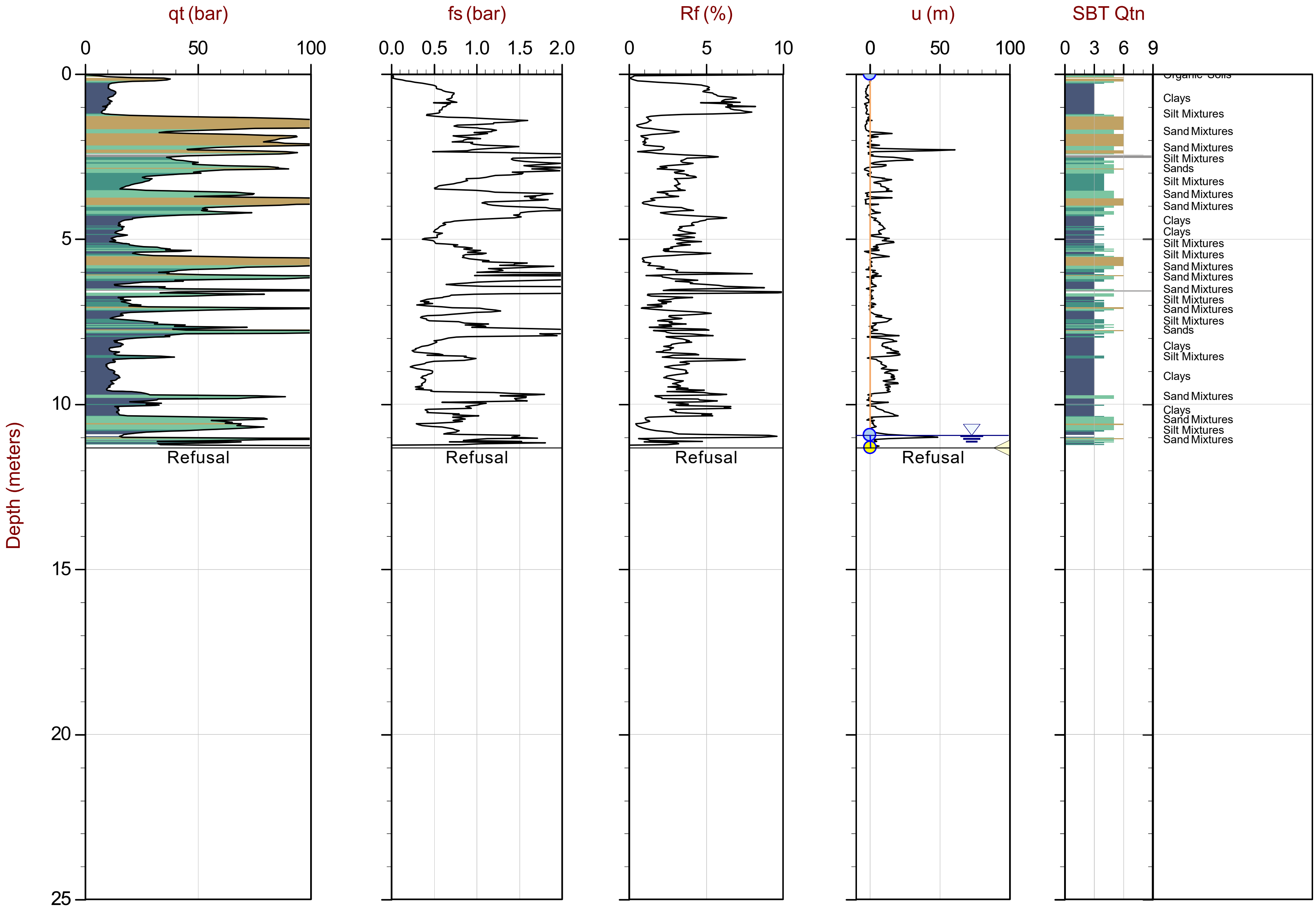
Overplot Item: Ueq Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Dissipation, Ueq assumed Ueq Line Hydrostatic Line

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

WSP Canada

Job No: 23-05-25254
Date: 2023-03-02 07:48
Site: 3630 and 3690 Riverside Drive

Sounding: SCPT23-01C
Cone: 947:T1000F10U35



Max Depth: 11.325 m / 37.16 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-05-25254_SP01C.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 18N N: 5020451m E: 445588m
Sheet No: 1 of 1

Overplot Item: Ueq Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Dissipation, Ueq assumed Ueq Line Hydrostatic Line

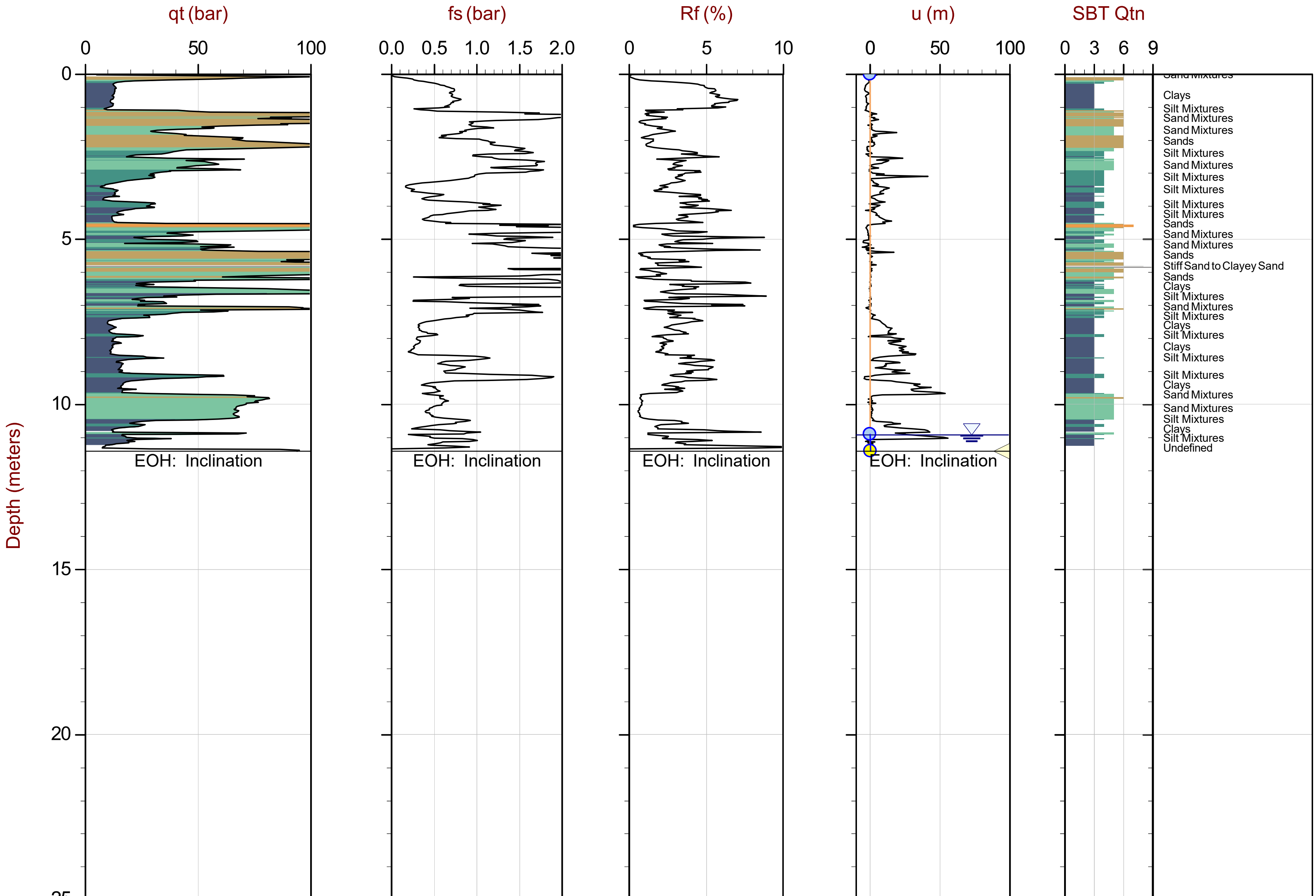
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

WSP Canada

Job No: 23-05-25254
Date: 2023-03-02 09:43
Site: 3630 and 3690 Riverside Drive

Sounding: SCPT23-01D
Cone: 947:T1000F10U35



Max Depth: 11.425 m / 37.48 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-05-25254_SP01D.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 18N N: 5020451m E: 445589m
Sheet No: 1 of 1

Overplot Item: Ueq Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Dissipation, Ueq assumed Ueq Line Hydrostatic Line

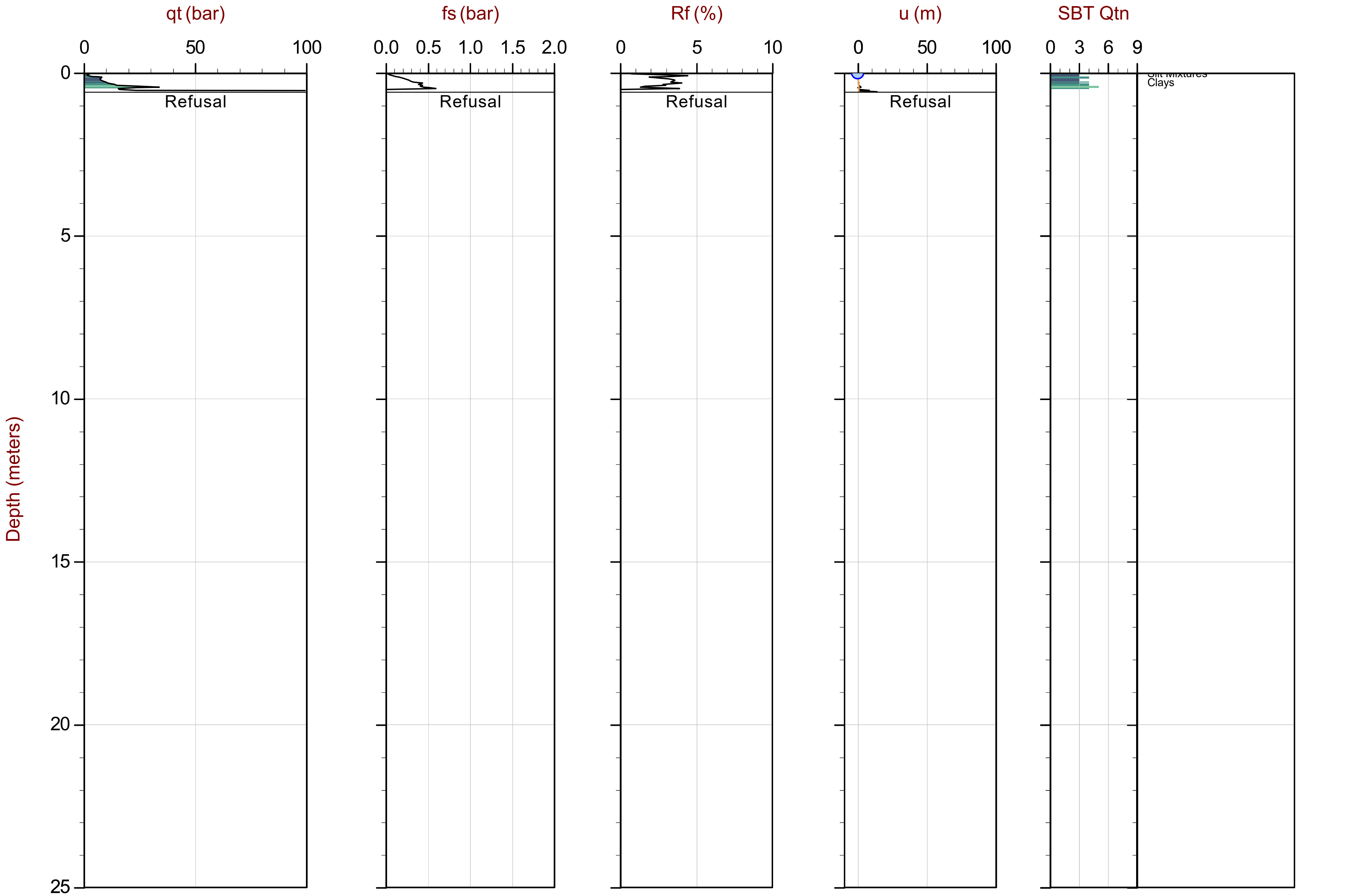
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

WSP Canada

Job No: 23-05-25254
Date: 2023-02-28 14:39
Site: 3630 and 3690 Riverside Drive

Sounding: CPT23-02
Cone: 947:T1000F10U35



Max Depth: 0.575 m / 1.89 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-05-25254_CP02.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 18N N: 5020561m E: 445572m
Sheet No: 1 of 1

Overplot Item: Ueq   Assumed Ueq   Dissipation, Ueq achieved   Dissipation, Ueq not achieved   Dissipation, Ueq assumed   Ueq Line   Hydrostatic Line

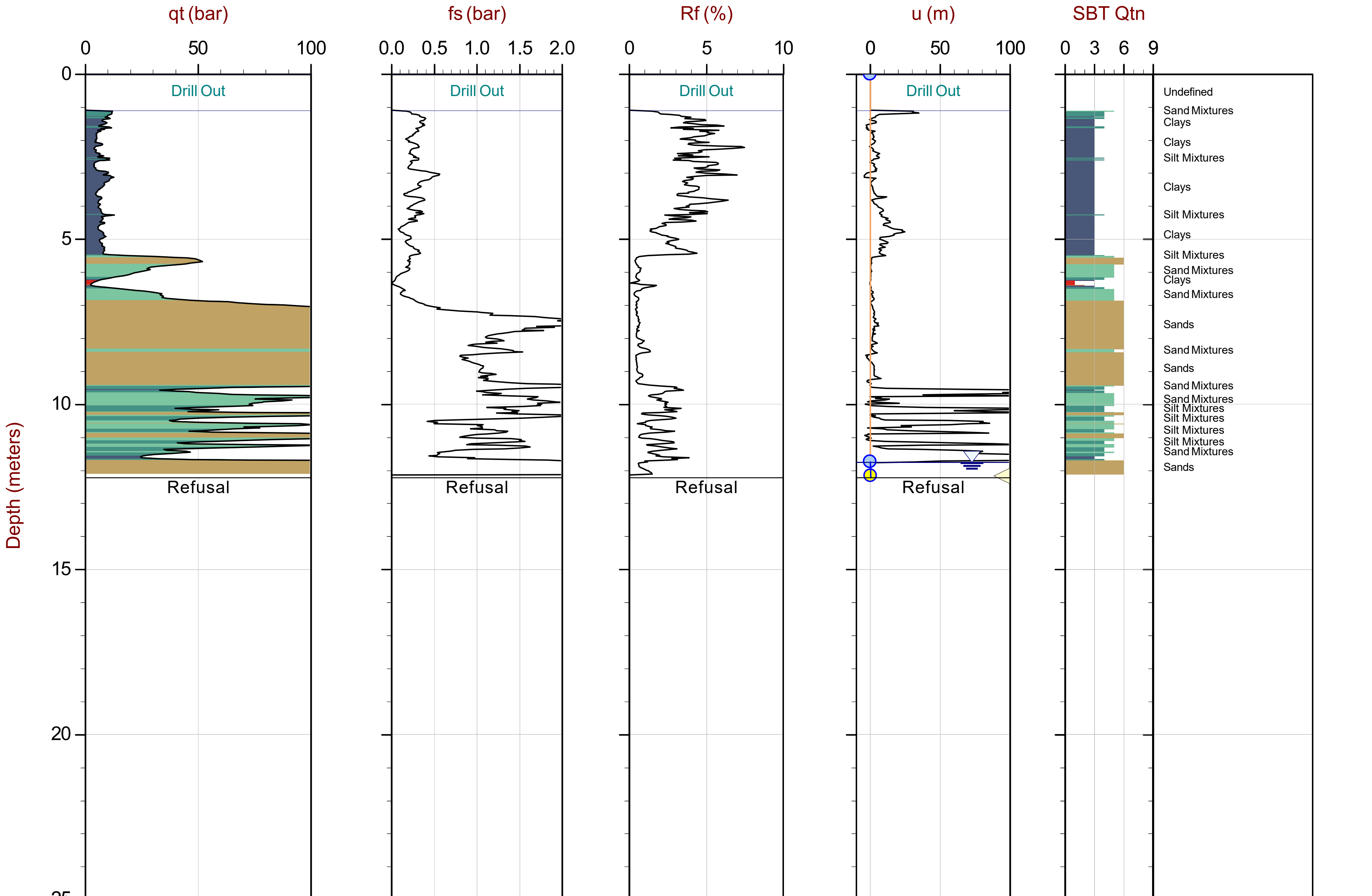
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

WSP Canada

Job No: 23-05-25254
Date: 2023-02-28 15:47
Site: 3630 and 3690 Riverside Drive

Sounding: CPT23-02B
Cone: 947:T1000F10U35



Max Depth: 12.225 m / 40.11 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-05-25254_CP02B.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 18N N: 5020561m E: 445573m
Sheet No: 1 of 1

Overplot Item: Ueq, Assumed Ueq, Dissipation, Ueq achieved, Dissipation, Ueq not achieved, Dissipation, Ueq assumed, Ueq Line, Hydrostatic Line

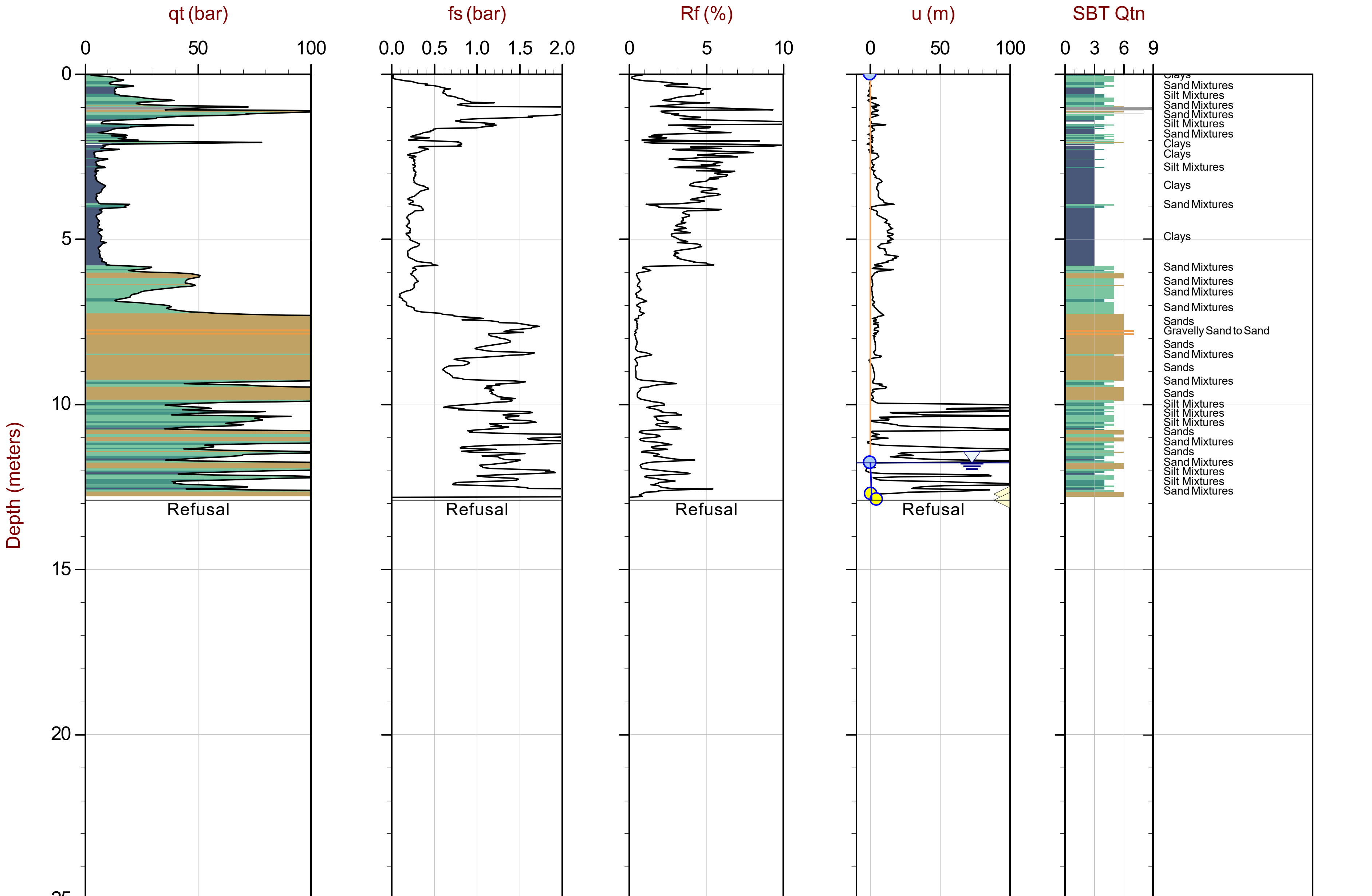
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

WSP Canada

Job No: 23-05-25254
Date: 2023-03-01 06:59
Site: 3630 and 3690 Riverside Drive

Sounding: CPT23-02C
Cone: 947:T1000F10U35



Max Depth: 12.900 m / 42.32 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-05-25254_CP02C.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 18N N: 5020561m E: 445574m
Sheet No: 1 of 1

Overplot Item: Ueq Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Dissipation, Ueq assumed Ueq Line Hydrostatic Line

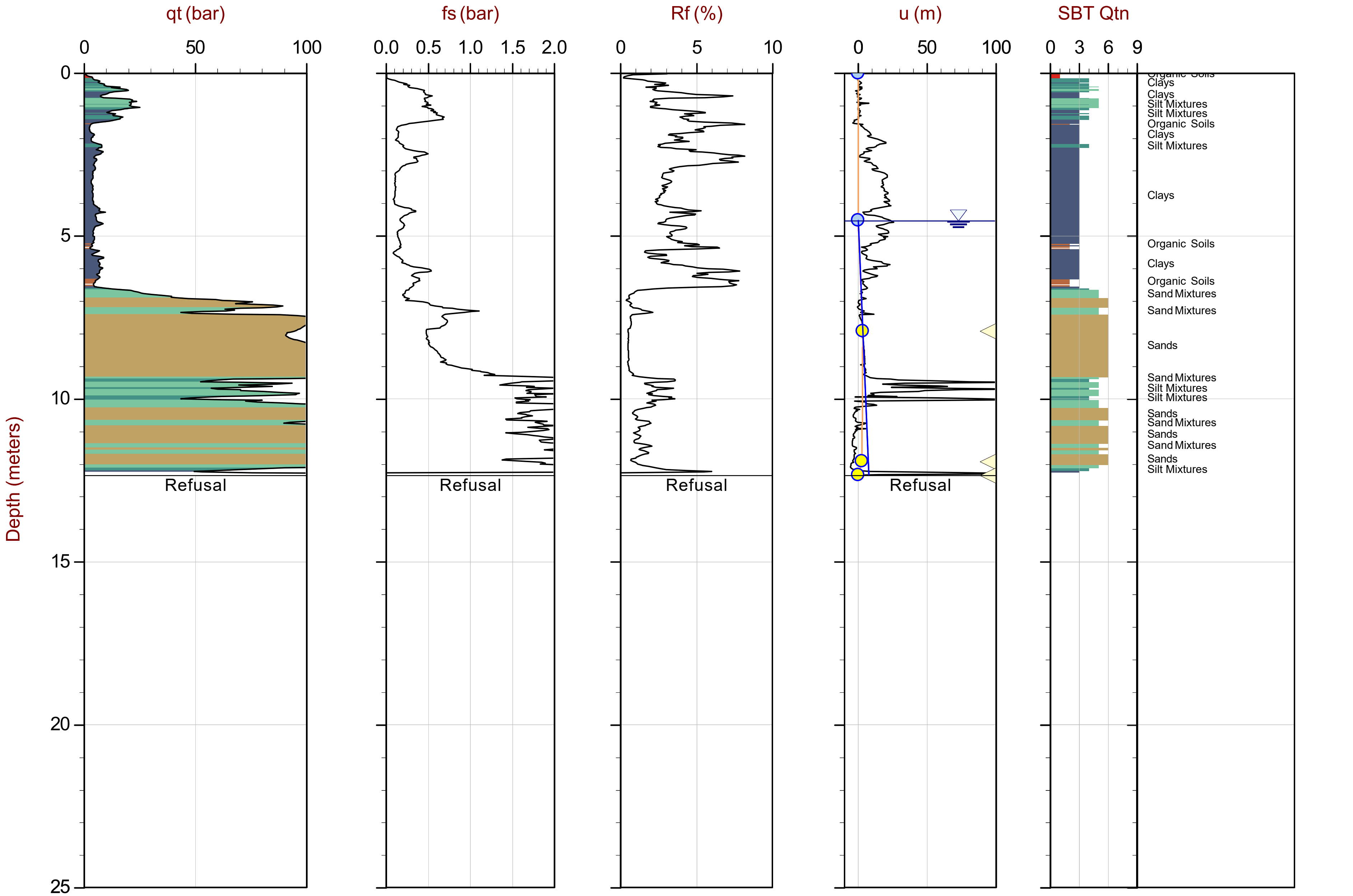
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

ConeTec

WSP Canada

Job No: 23-05-25254
Date: 2023-03-01 09:49
Site: 3630 and 3690 Riverside Drive

Sounding: SCPT23-03
Cone: 947:T1000F10U35



Max Depth: 12.350 m / 40.52 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-05-25254_SP03.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 18N N: 5020585m E: 445530m
Sheet No: 1 of 1

Overplot Item: Ueq Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Dissipation, Ueq assumed Ueq Line Hydrostatic Line

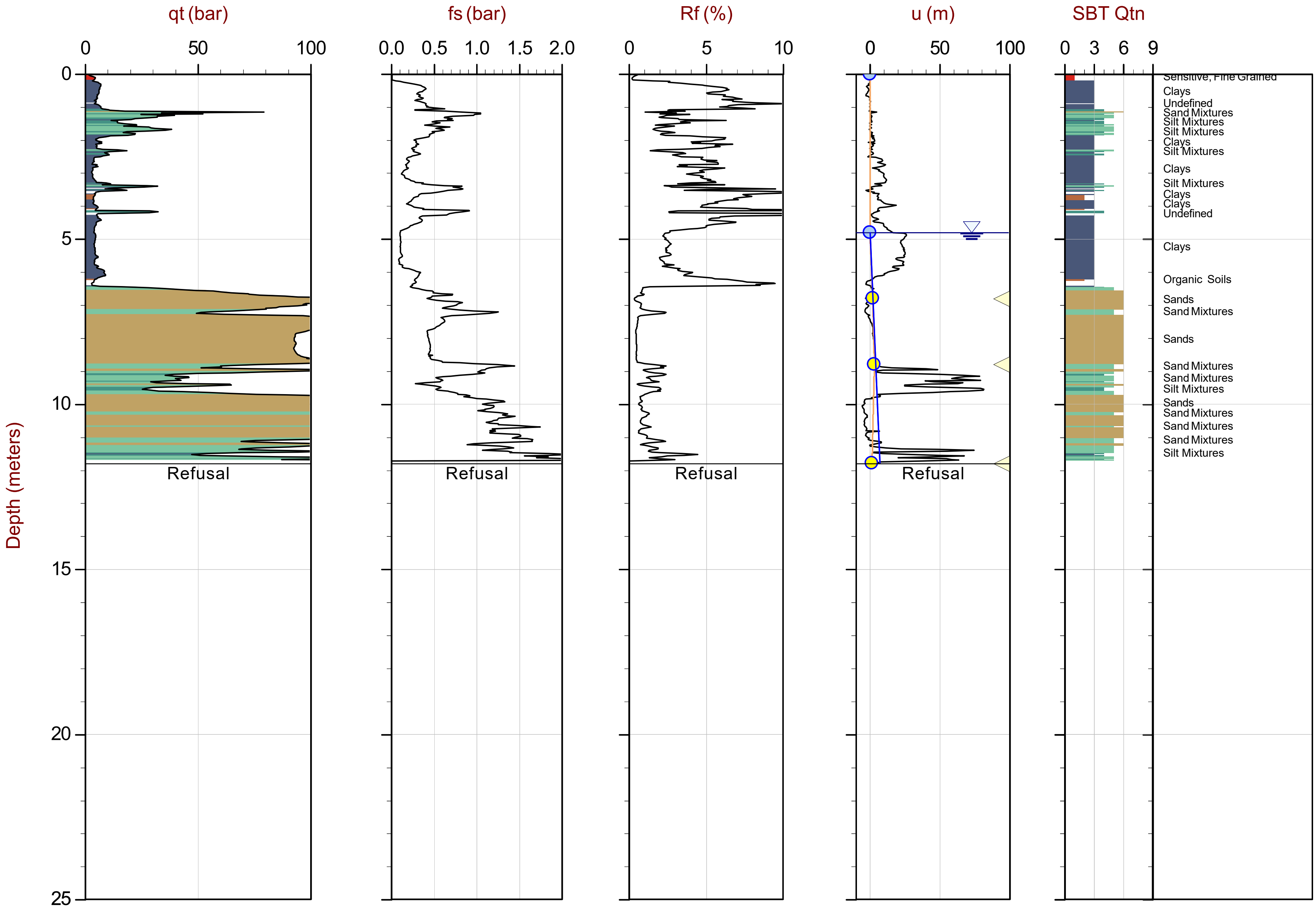
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

ConeTec

WSP Canada

Job No: 23-05-25254
Date: 2023-03-01 12:55
Site: 3630 and 3690 Riverside Drive

Sounding: SCPT23-03B
Cone: 947:T1000F10U35



Max Depth: 11.800 m / 38.71 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-05-25254_SP03B.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 18N N: 5020584m E: 445530m
Sheet No: 1 of 1

Overplot Item: Ueq   Assumed Ueq   Dissipation, Ueq achieved   Dissipation, Ueq not achieved   Dissipation, Ueq assumed   Ueq Line   Hydrostatic Line

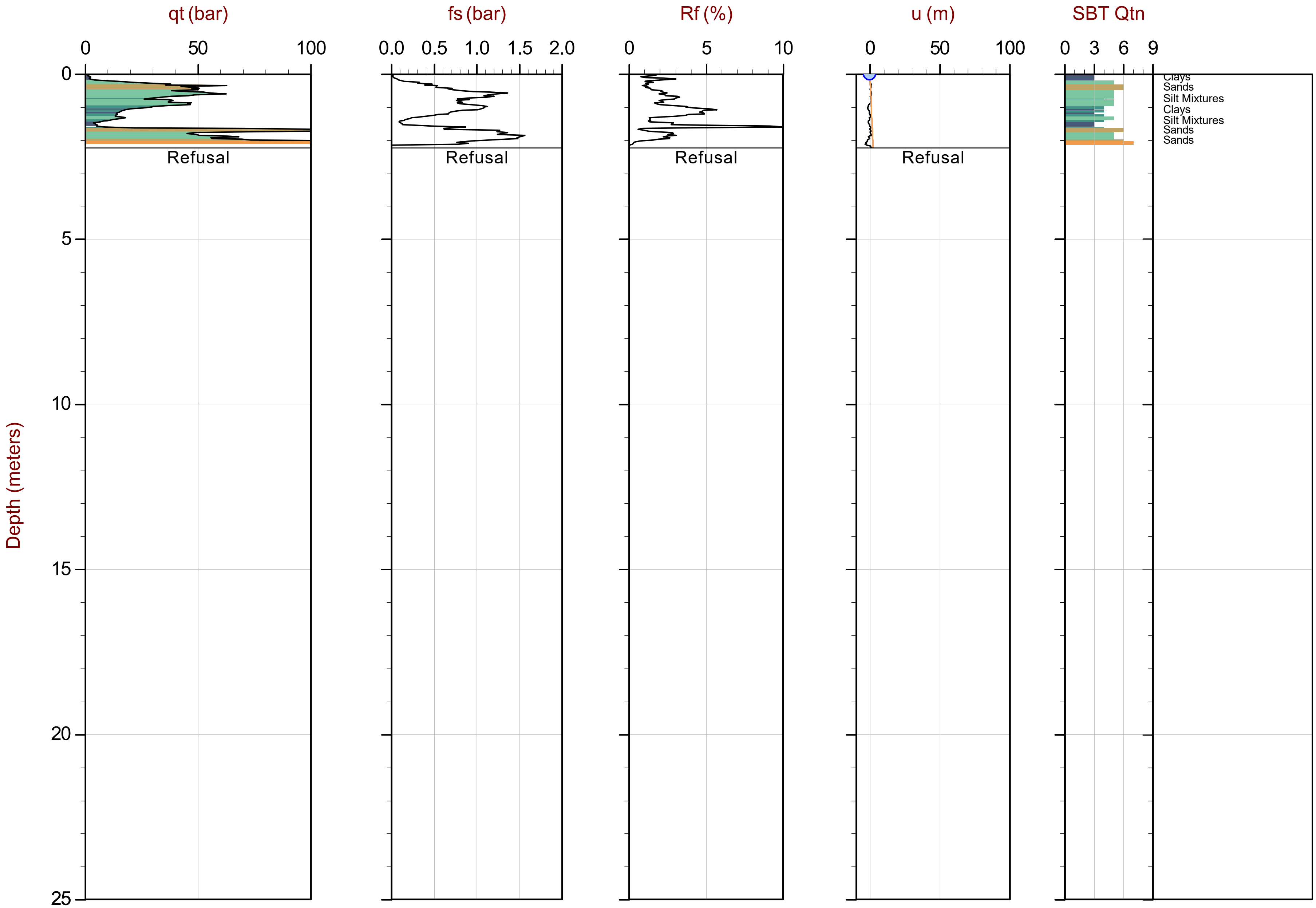
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

WSP Canada

Job No: 23-05-25254
Date: 2023-02-27 16:03
Site: 3630 and 3690 Riverside Drive

Sounding: CPT23-04
Cone: 947:T1000F10U35



Max Depth: 2.225 m / 7.30 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-05-25254_CP04.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 18N N: 5020591m E: 445613m
Sheet No: 1 of 1

Overplot Item: Ueq Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Dissipation, Ueq assumed Ueq Line Hydrostatic Line

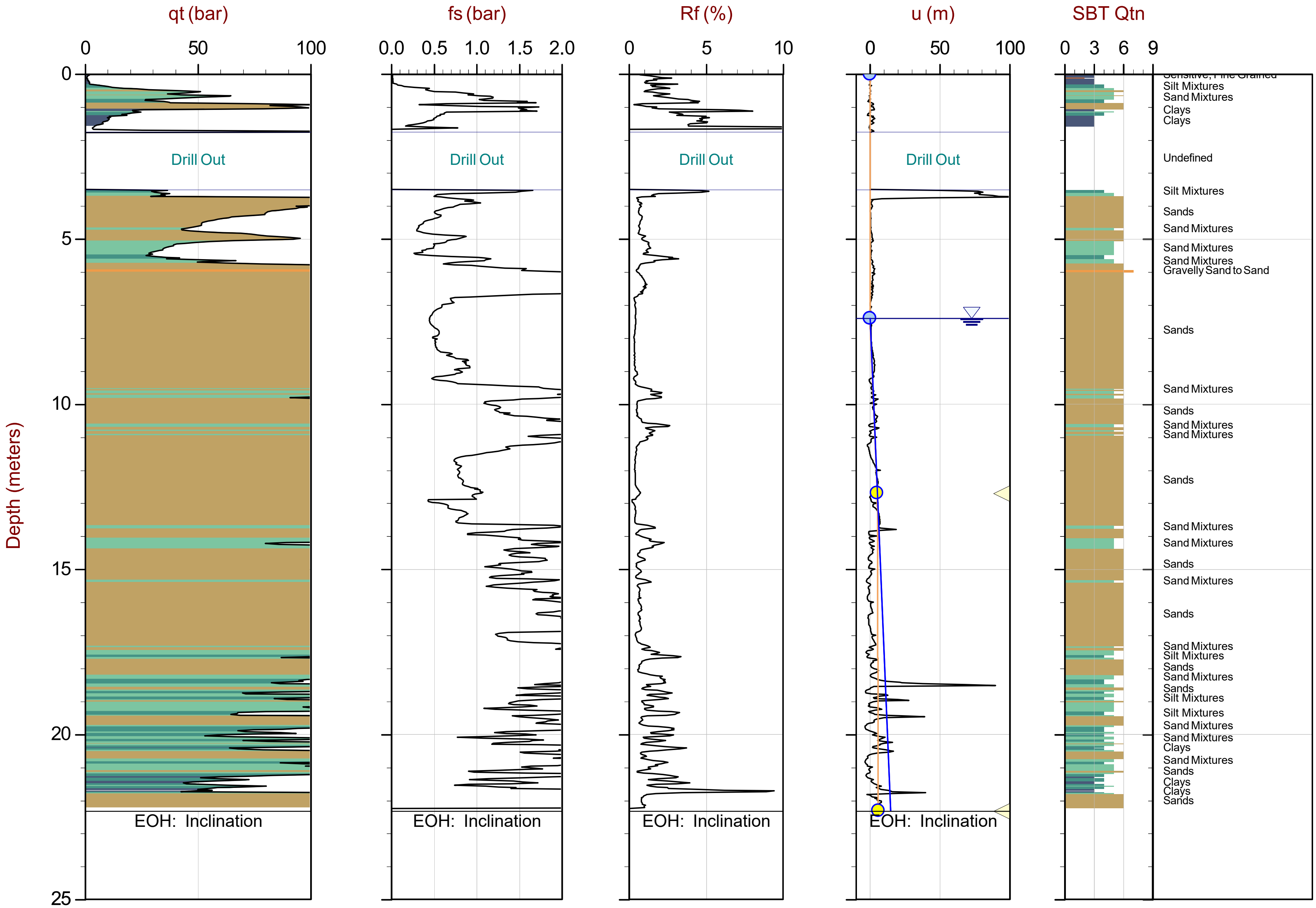
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

WSP Canada

Job No: 23-05-25254
Date: 2023-02-28 10:33
Site: 3630 and 3690 Riverside Drive

Sounding: CPT23-04B
Cone: 947:T1000F10U35



Max Depth: 22.325 m / 73.24 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-05-25254_CP04B.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 18N N: 5020590m E: 445613m
Sheet No: 1 of 1

Overplot Item: Ueq Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Dissipation, Ueq assumed Ueq Line Hydrostatic Line

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

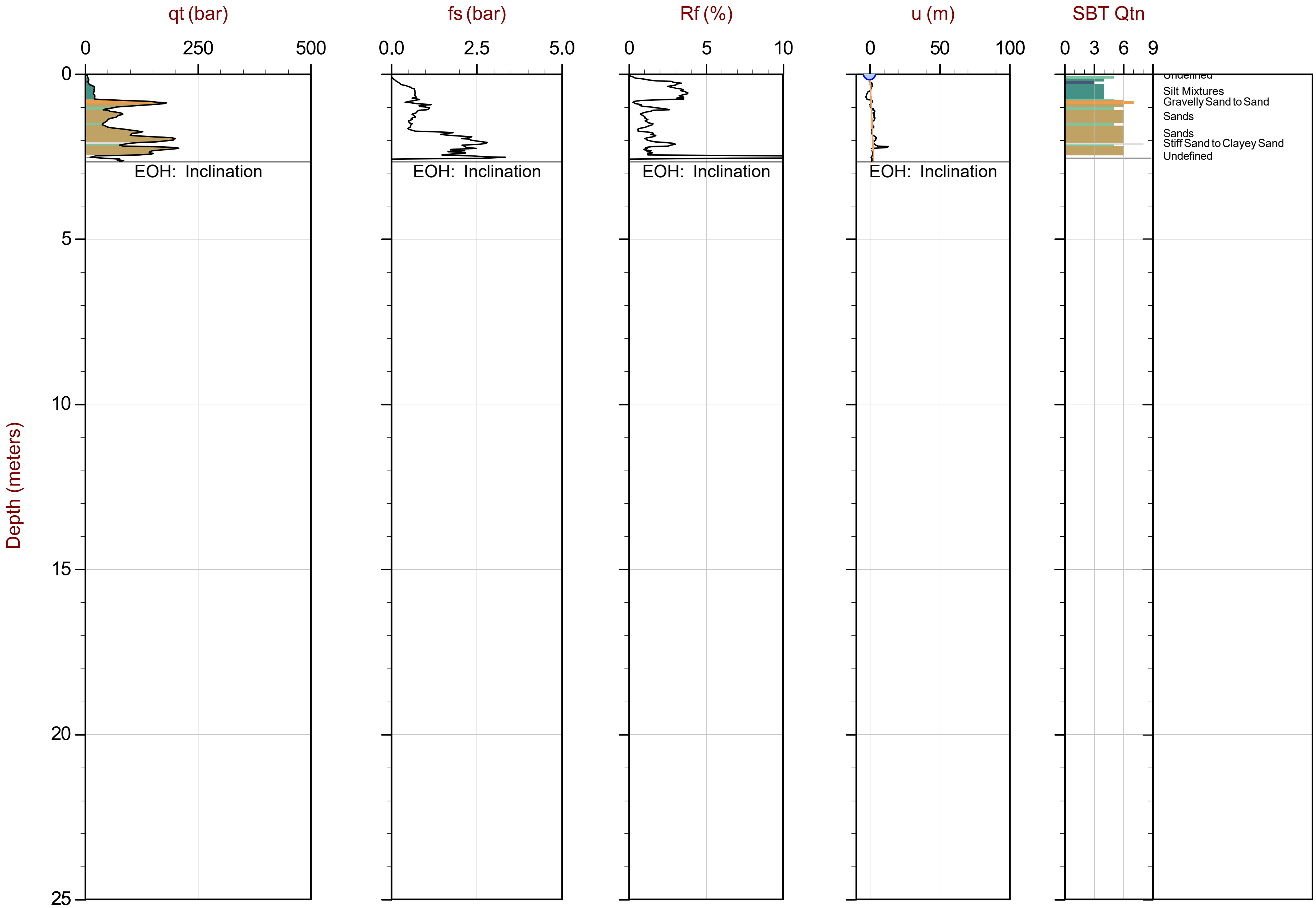
Standard Cone Penetration Test Plots with Expanded Range

CONETEC

WSP Canada

Job No: 23-05-25254
Date: 2023-03-01 15:36
Site: 3630 and 3690 Riverside Drive

Sounding: SCPT23-01
Cone: 947:T1000F10U35



Max Depth: 2.650 m / 8.69 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-05-25254_SP01.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 18N N: 5020457m E: 445550m
Sheet No: 1 of 1

Overplot Item: Ueq Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Dissipation, Ueq assumed Ueq Line Hydrostatic Line

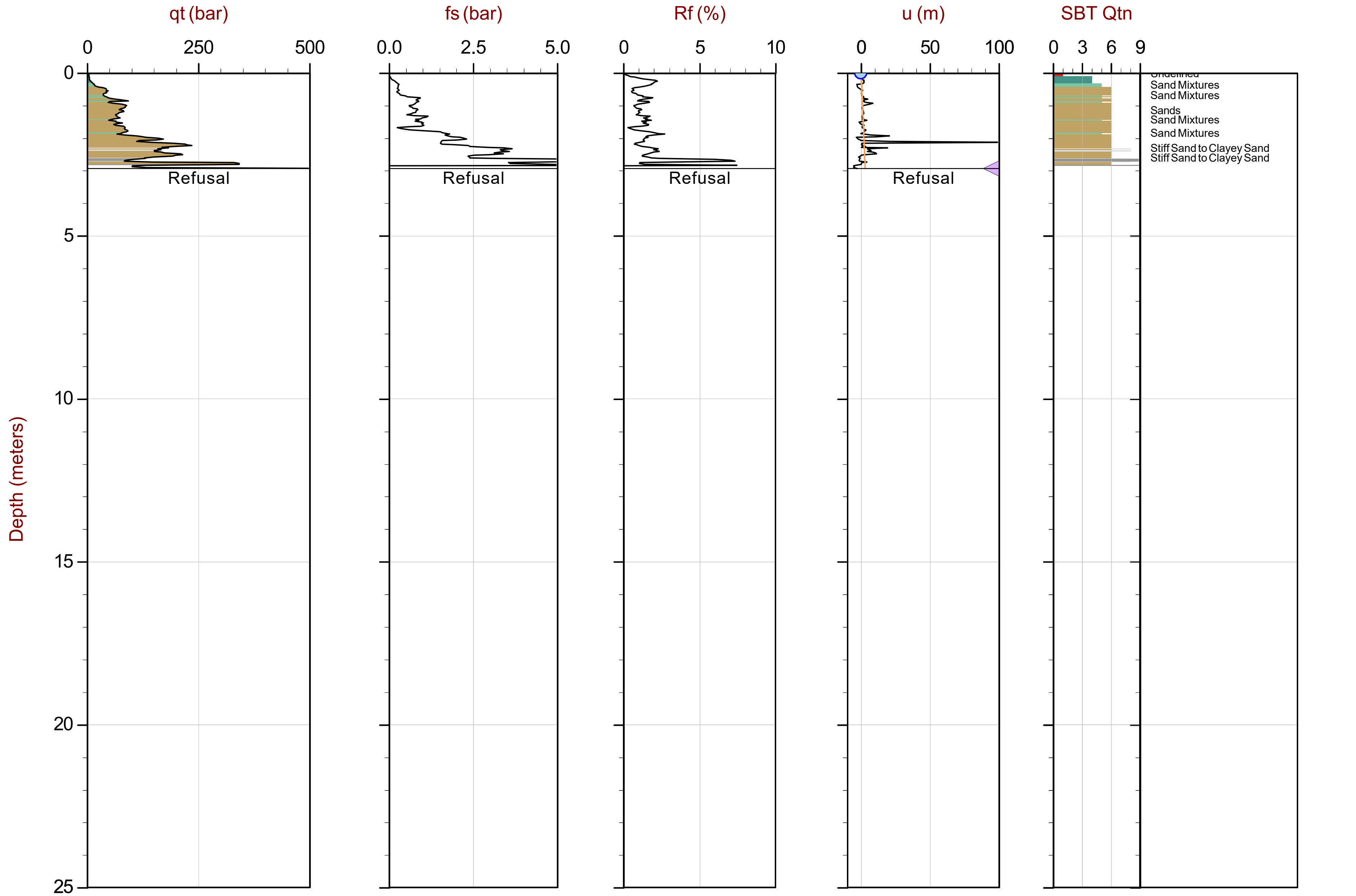
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

ConeTec

WSP Canada

Job No: 23-05-25254
Date: 2023-03-01 16:44
Site: 3630 and 3690 Riverside Drive

Sounding: SCPT23-01B
Cone: 947:T1000F10U35



Max Depth: 2.925 m / 9.60 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-05-25254_SP01B.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 18N N: 5020456m E: 445550m
Sheet No: 1 of 1

Overplot Item: Ueq Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Dissipation, Ueq assumed Ueq Line Hydrostatic Line

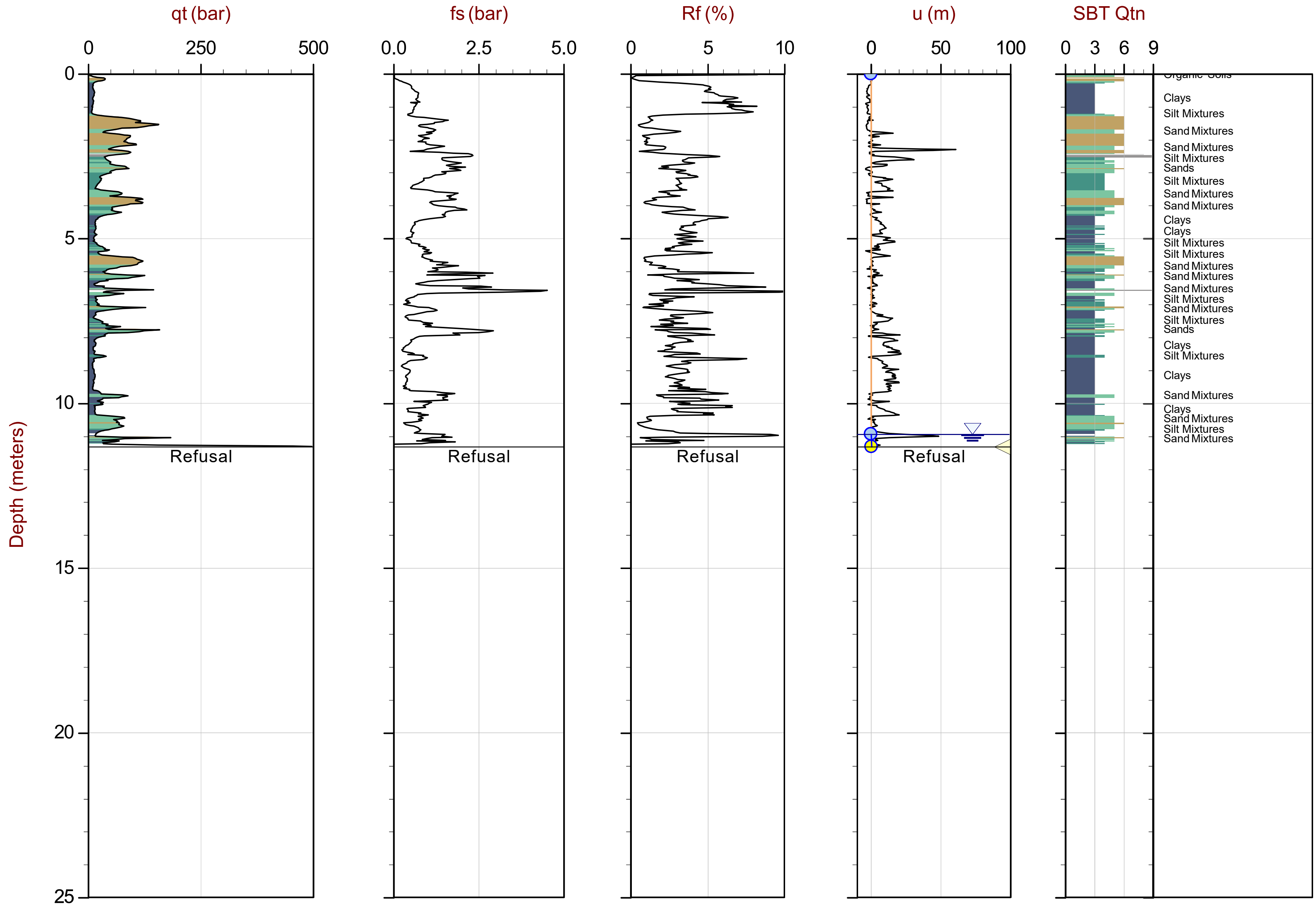
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

ConeTec

WSP Canada

Job No: 23-05-25254
Date: 2023-03-02 07:48
Site: 3630 and 3690 Riverside Drive

Sounding: SCPT23-01C
Cone: 947:T1000F10U35



Max Depth: 11.325 m / 37.16 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-05-25254_SP01C.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 18N N: 5020451m E: 445588m
Sheet No: 1 of 1

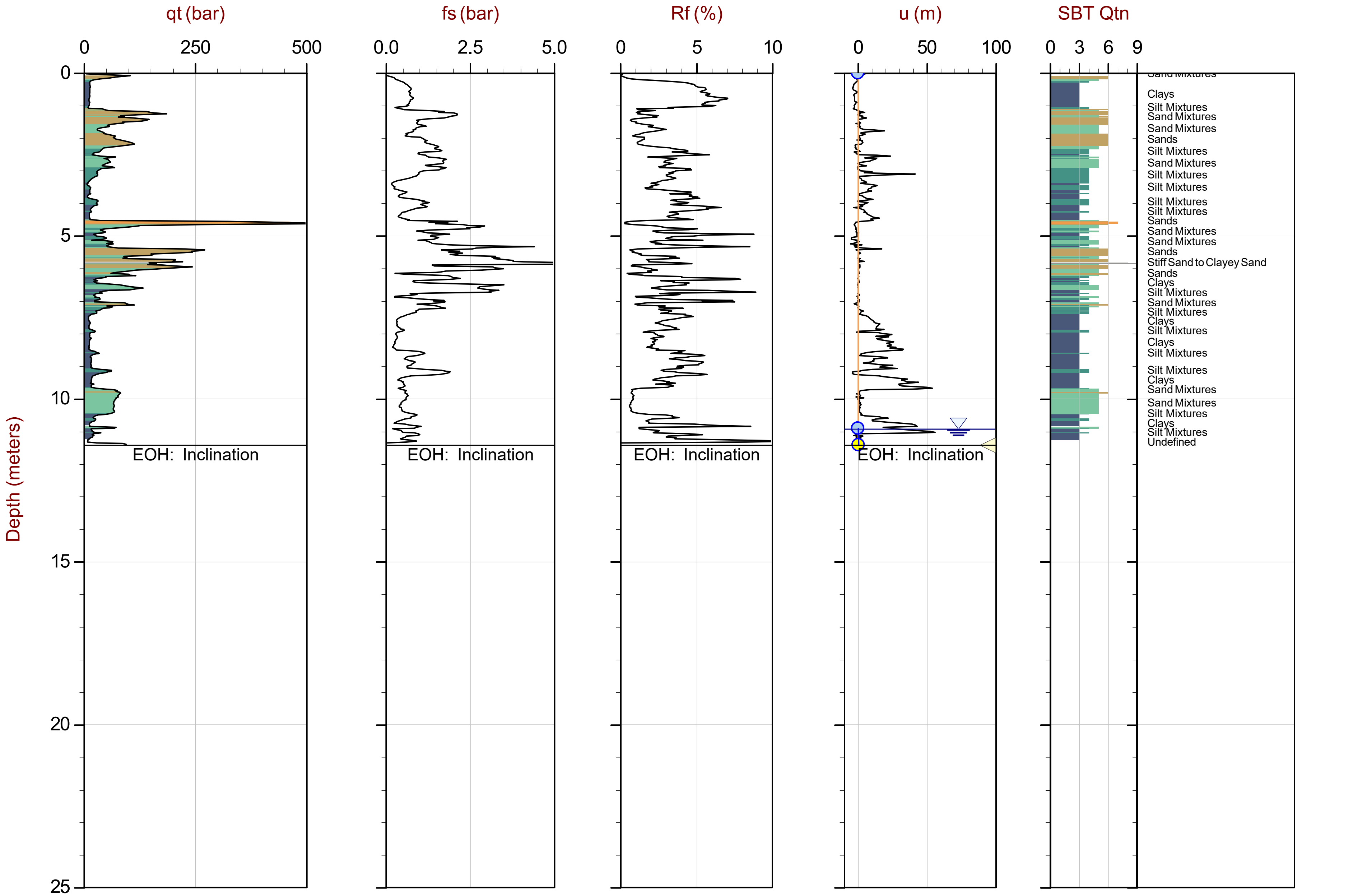
Overplot Item: Ueq · Assumed Ueq · Dissipation, Ueq achieved · Dissipation, Ueq not achieved · Dissipation, Ueq assumed · Ueq Line · Hydrostatic Line

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

WSP Canada

Job No: 23-05-25254
Date: 2023-03-02 09:43
Site: 3630 and 3690 Riverside Drive

Sounding: SCPT23-01D
Cone: 947:T1000F10U35



Max Depth: 11.425 m / 37.48 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-05-25254_SP01D.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 18N N: 5020451m E: 445589m
Sheet No: 1 of 1

Overplot Item: Ueq   Assumed Ueq   Dissipation, Ueq achieved   Dissipation, Ueq not achieved   Dissipation, Ueq assumed   Ueq Line   Hydrostatic Line

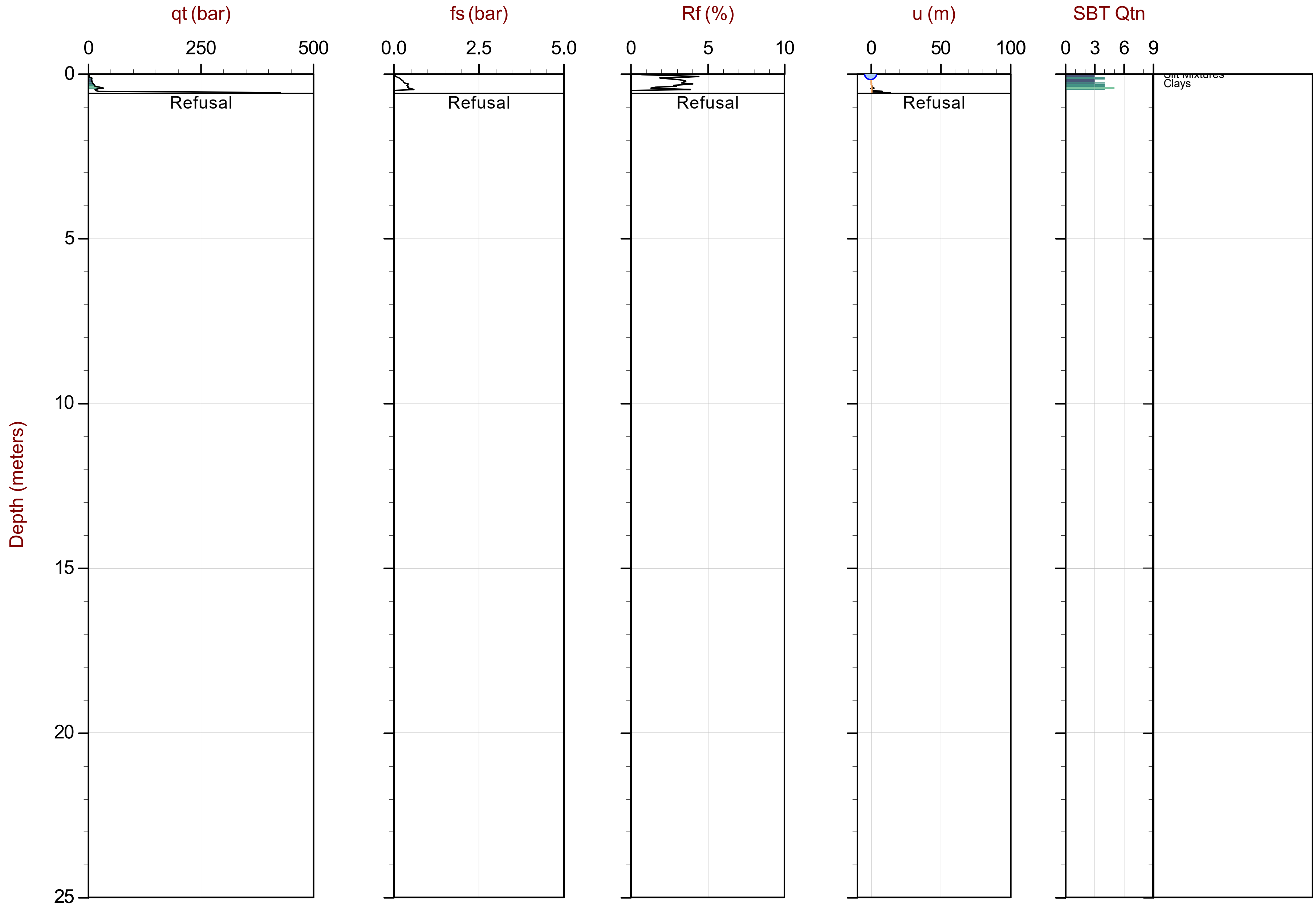
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

WSP Canada

Job No: 23-05-25254
Date: 2023-02-28 14:39
Site: 3630 and 3690 Riverside Drive

Sounding: CPT23-02
Cone: 947:T1000F10U35



Max Depth: 0.575 m / 1.89 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-05-25254_CP02.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 18N N: 5020561m E: 445572m
Sheet No: 1 of 1

Overplot Item: Ueq, Assumed Ueq, Dissipation, Ueq achieved, Dissipation, Ueq not achieved, Dissipation, Ueq assumed, Ueq Line, Hydrostatic Line

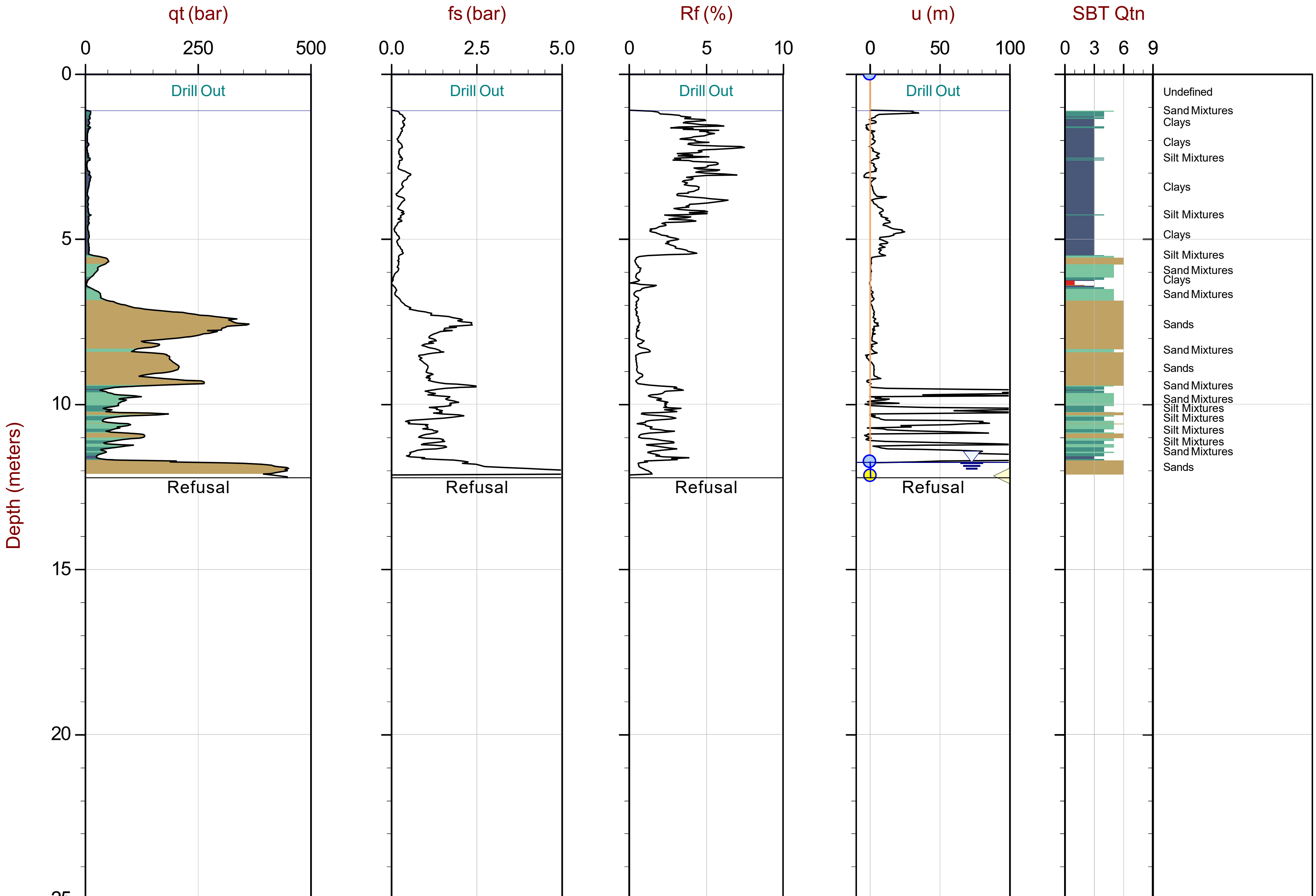
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

WSP Canada

Job No: 23-05-25254
Date: 2023-02-28 15:47
Site: 3630 and 3690 Riverside Drive

Sounding: CPT23-02B
Cone: 947:T1000F10U35



Max Depth: 12.225 m / 40.11 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-05-25254_CP02B.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 18N N: 5020561m E: 445573m
Sheet No: 1 of 1

Overplot Item: Ueq Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Dissipation, Ueq assumed Ueq Line Hydrostatic Line

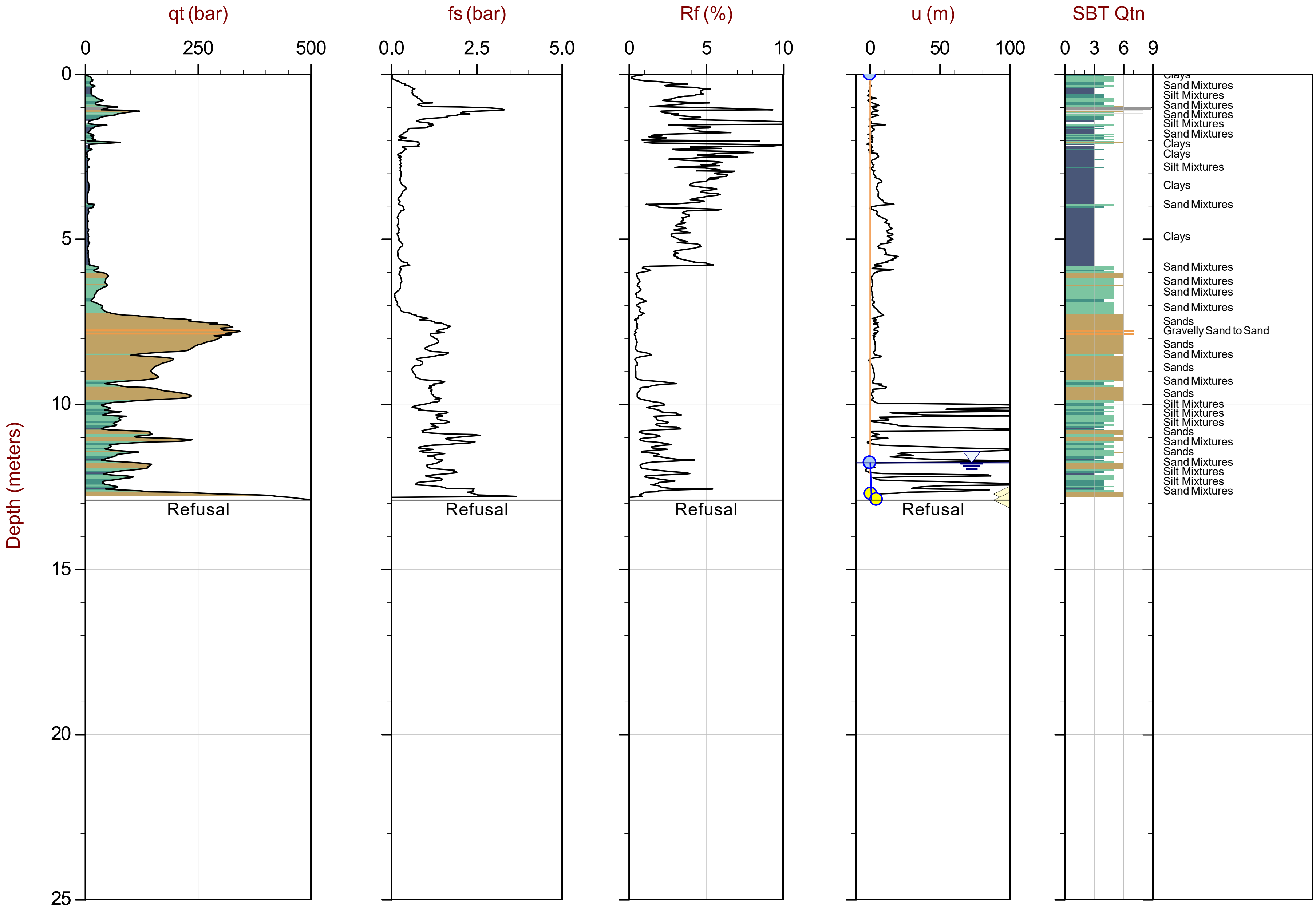
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

WSP Canada

Job No: 23-05-25254
Date: 2023-03-01 06:59
Site: 3630 and 3690 Riverside Drive

Sounding: CPT23-02C
Cone: 947:T1000F10U35



Max Depth: 12.900 m / 42.32 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-05-25254_CP02C.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 18N N: 5020561m E: 445574m
Sheet No: 1 of 1

Overplot Item: Ueq Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Dissipation, Ueq assumed Ueq Line Hydrostatic Line

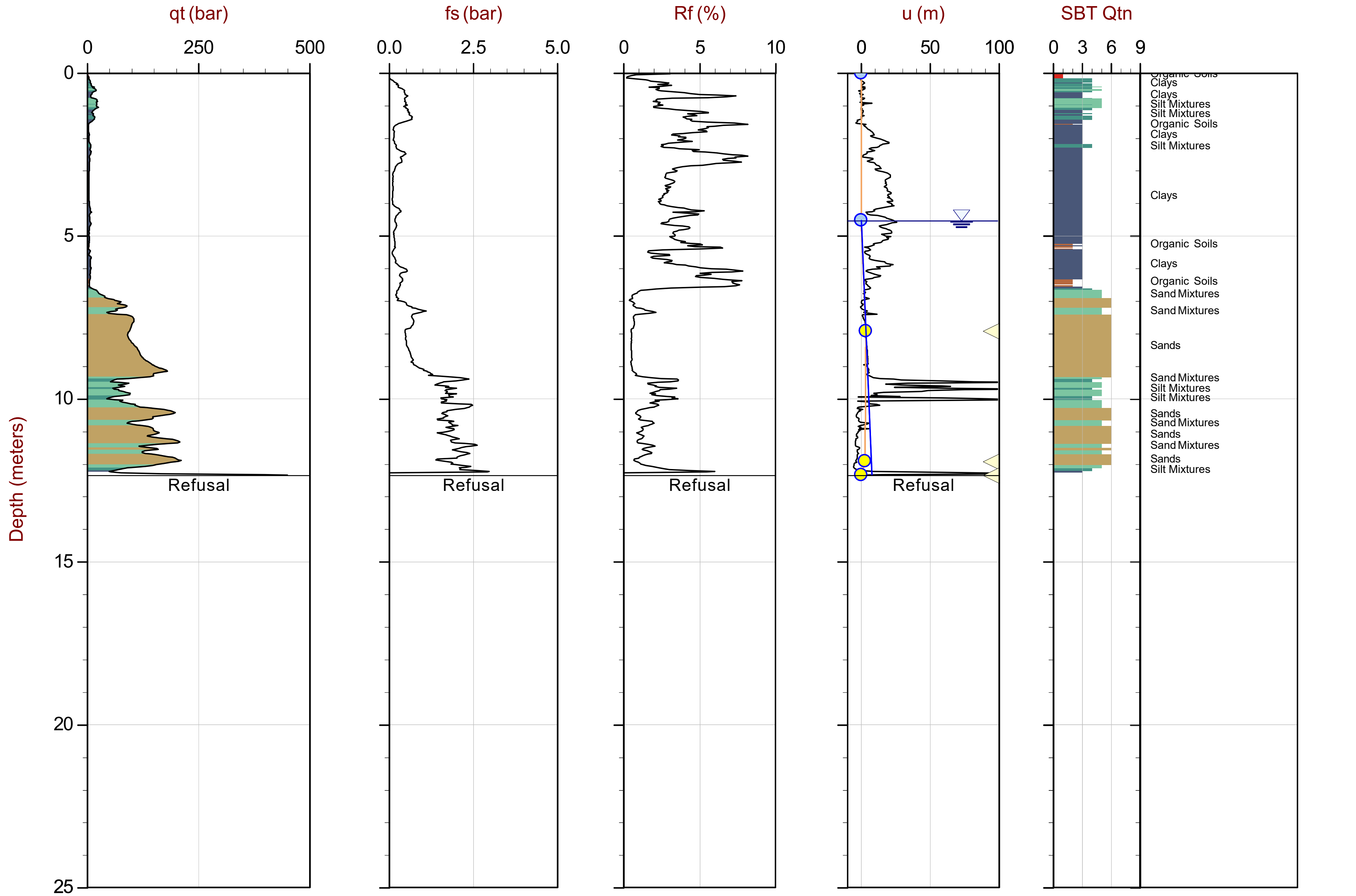
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

WSP Canada

Job No: 23-05-25254
Date: 2023-03-01 09:49
Site: 3630 and 3690 Riverside Drive

Sounding: SCPT23-03
Cone: 947:T1000F10U35



Max Depth: 12.350 m / 40.52 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-05-25254_SP03.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 18N N: 5020585m E: 445530m
Sheet No: 1 of 1

Overplot Item: Ueq | Assumed Ueq | Dissipation, Ueq achieved | Dissipation, Ueq not achieved | Dissipation, Ueq assumed | Ueq Line | Hydrostatic Line

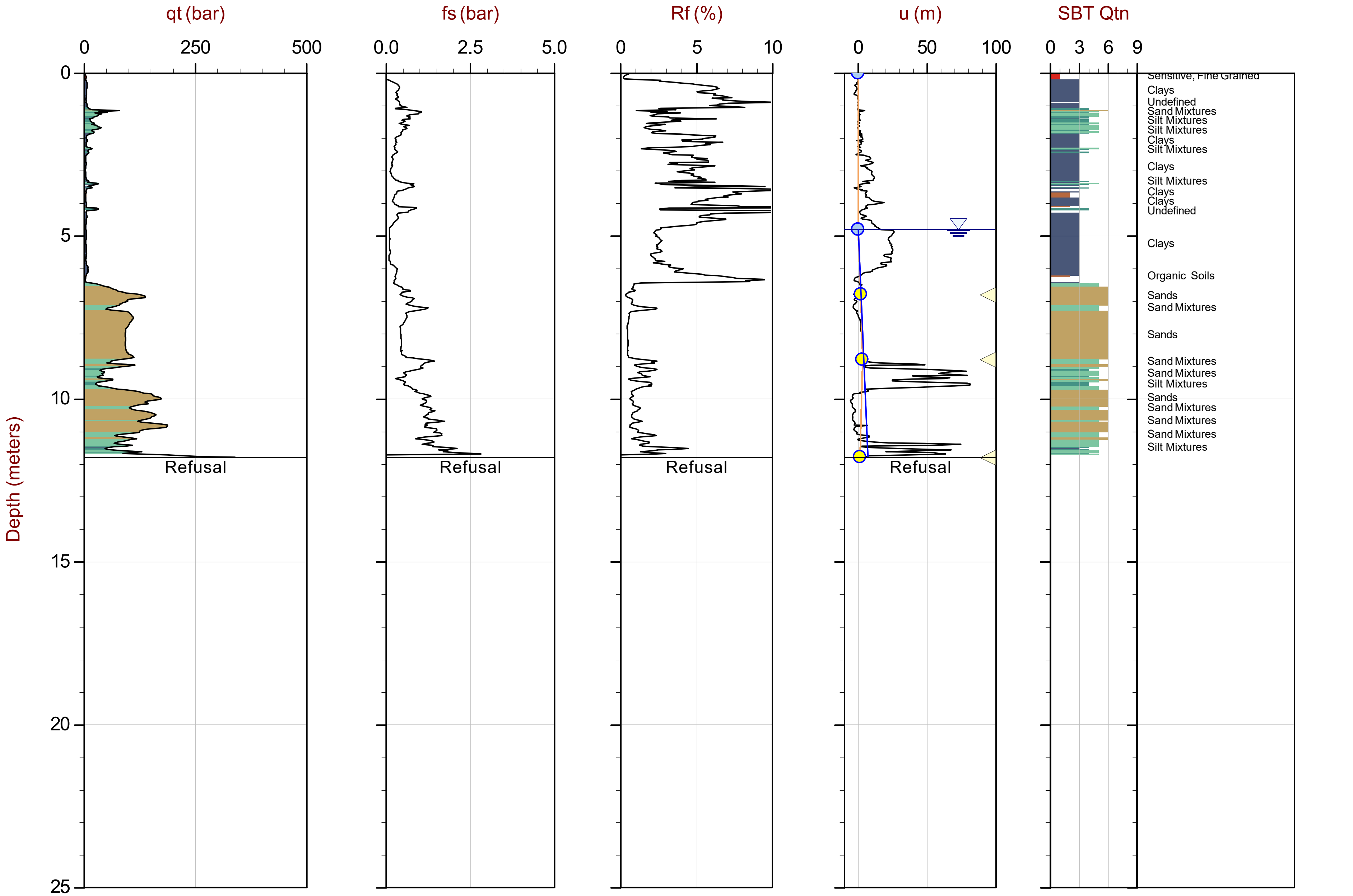
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

WSP Canada

Job No: 23-05-25254
Date: 2023-03-01 12:55
Site: 3630 and 3690 Riverside Drive

Sounding: SCPT23-03B
Cone: 947:T1000F10U35



Max Depth: 11.800 m / 38.71 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-05-25254_SP03B.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 18N N: 5020584m E: 445530m
Sheet No: 1 of 1

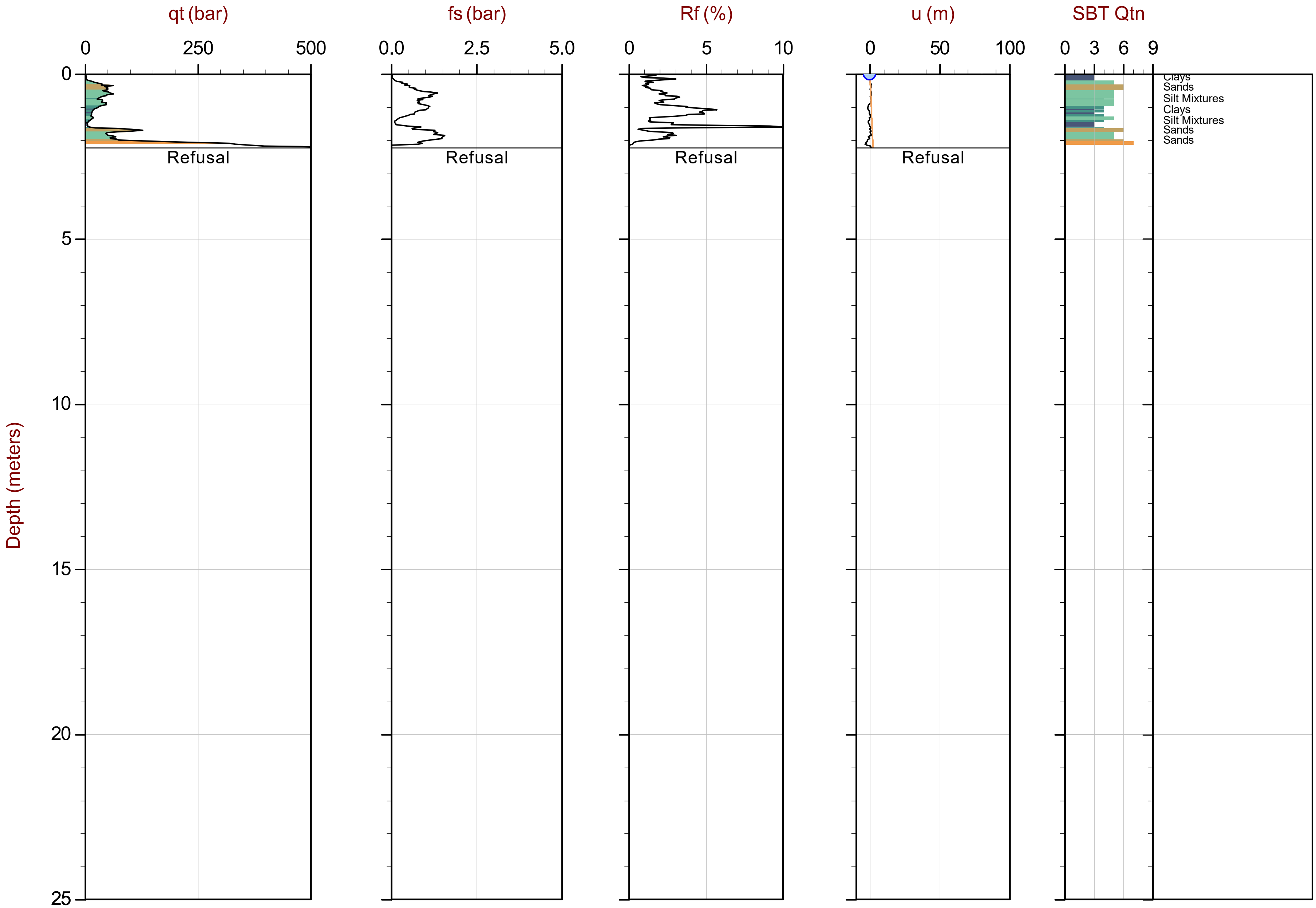
Overplot Item: Ueq Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Dissipation, Ueq assumed Ueq Line Hydrostatic Line

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

WSP Canada

Job No: 23-05-25254
Date: 2023-02-27 16:03
Site: 3630 and 3690 Riverside Drive
Sounding: CPT23-04
Cone: 947:T1000F10U35



Max Depth: 2.225 m / 7.30 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-05-25254_CP04.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 18N N: 5020591m E: 445613m
Sheet No: 1 of 1

Overplot Item: Ueq Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Dissipation, Ueq assumed Ueq Line Hydrostatic Line

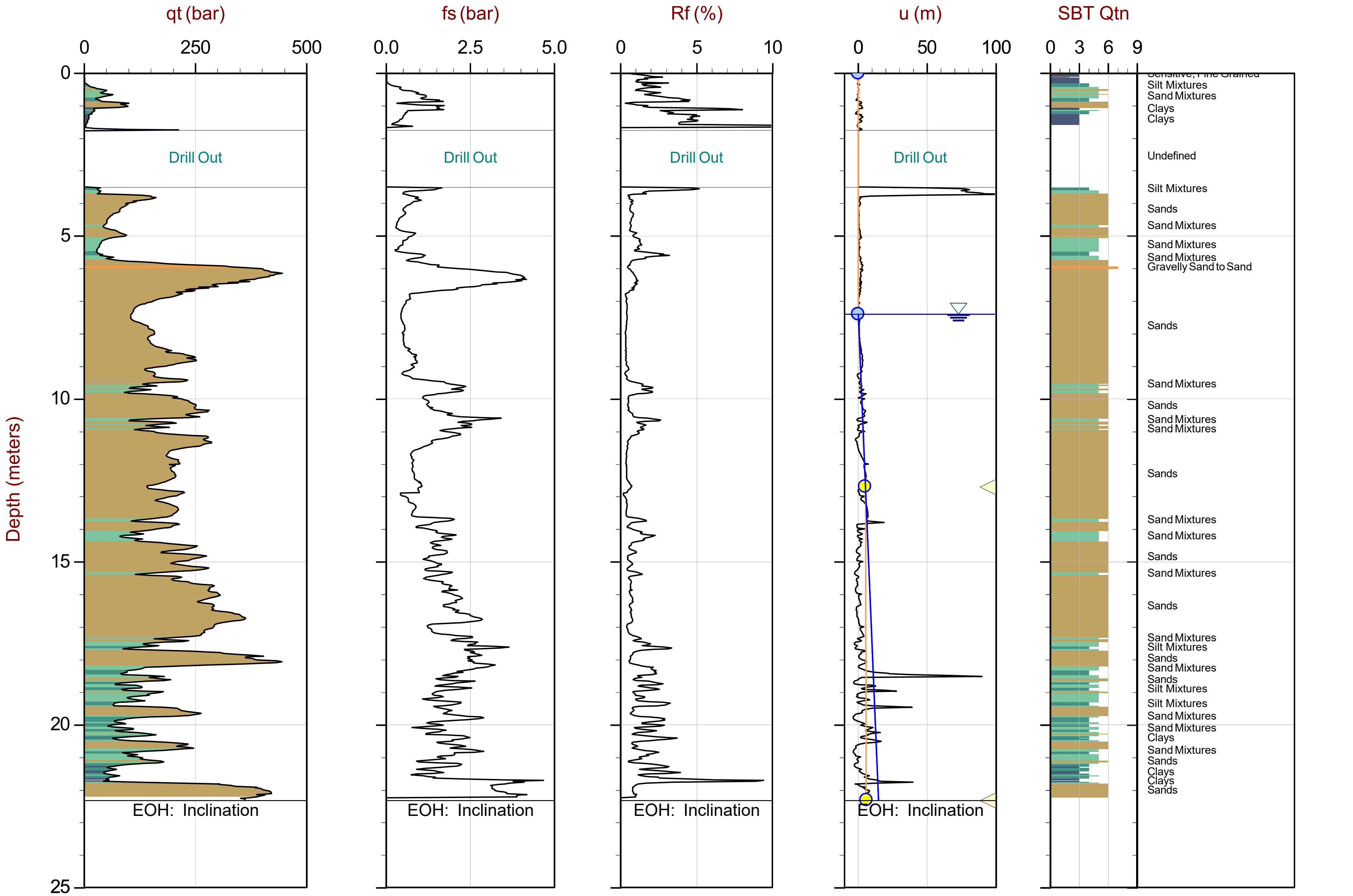
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

WSP Canada

Job No: 23-05-25254
Date: 2023-02-28 10:33
Site: 3630 and 3690 Riverside Drive

Sounding: CPT23-04B
Cone: 947:T1000F10U35



Max Depth: 22.325 m / 73.24 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-05-25254_CP04B.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 18N N: 5020590m E: 445613m
Sheet No: 1 of 1

Overplot Item: Ueq Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Dissipation, Ueq assumed Ueq Line Hydrostatic Line

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

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

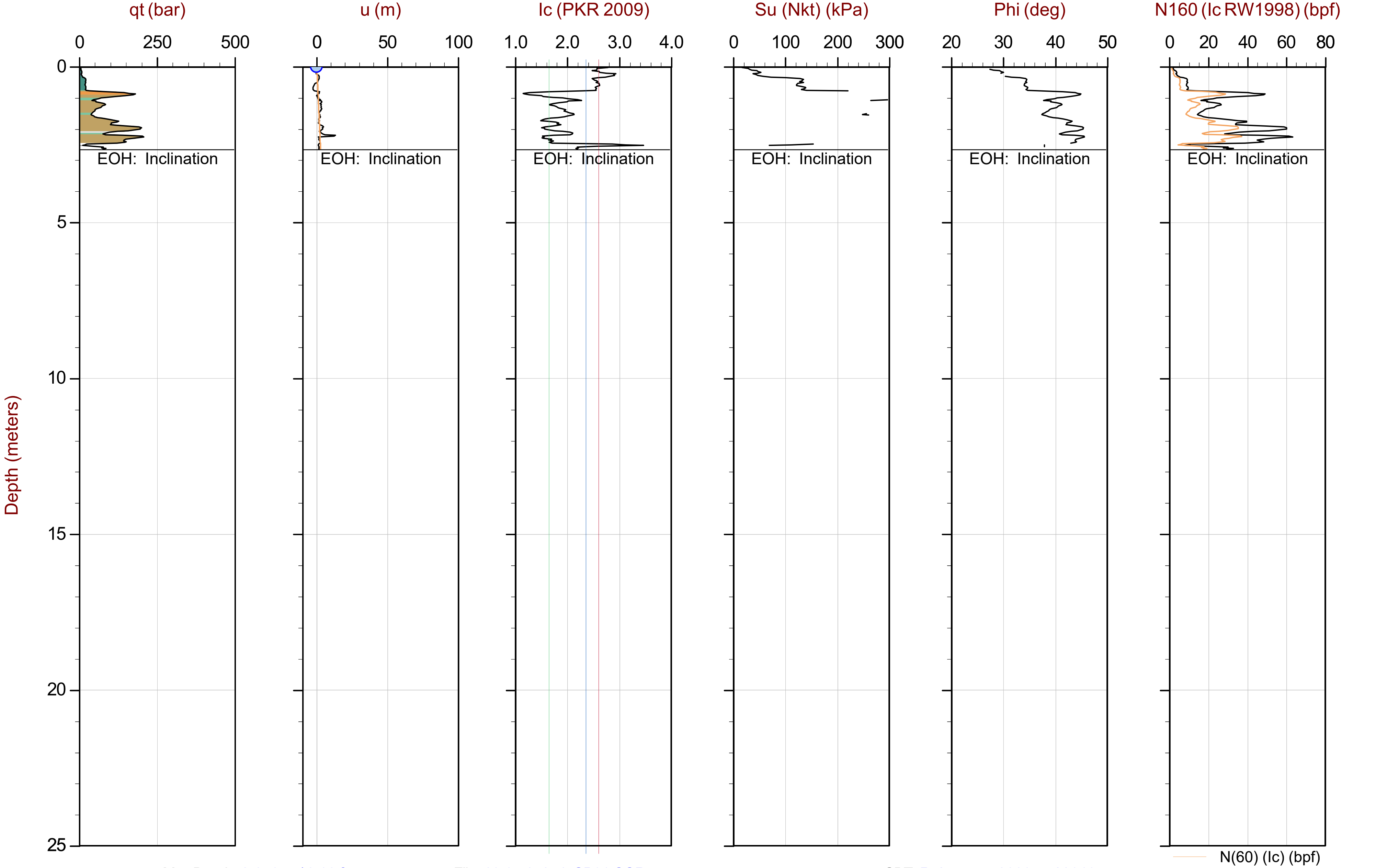
CONETEC

CONETEC

WSP Canada

Job No: 23-05-25254
Date: 2023-03-01 15:36
Site: 3630 and 3690 Riverside Drive

Sounding: SCPT23-01
Cone: 947:T1000F10U35



Max Depth: 2.650 m / 8.69 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-05-25254_SP01.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0

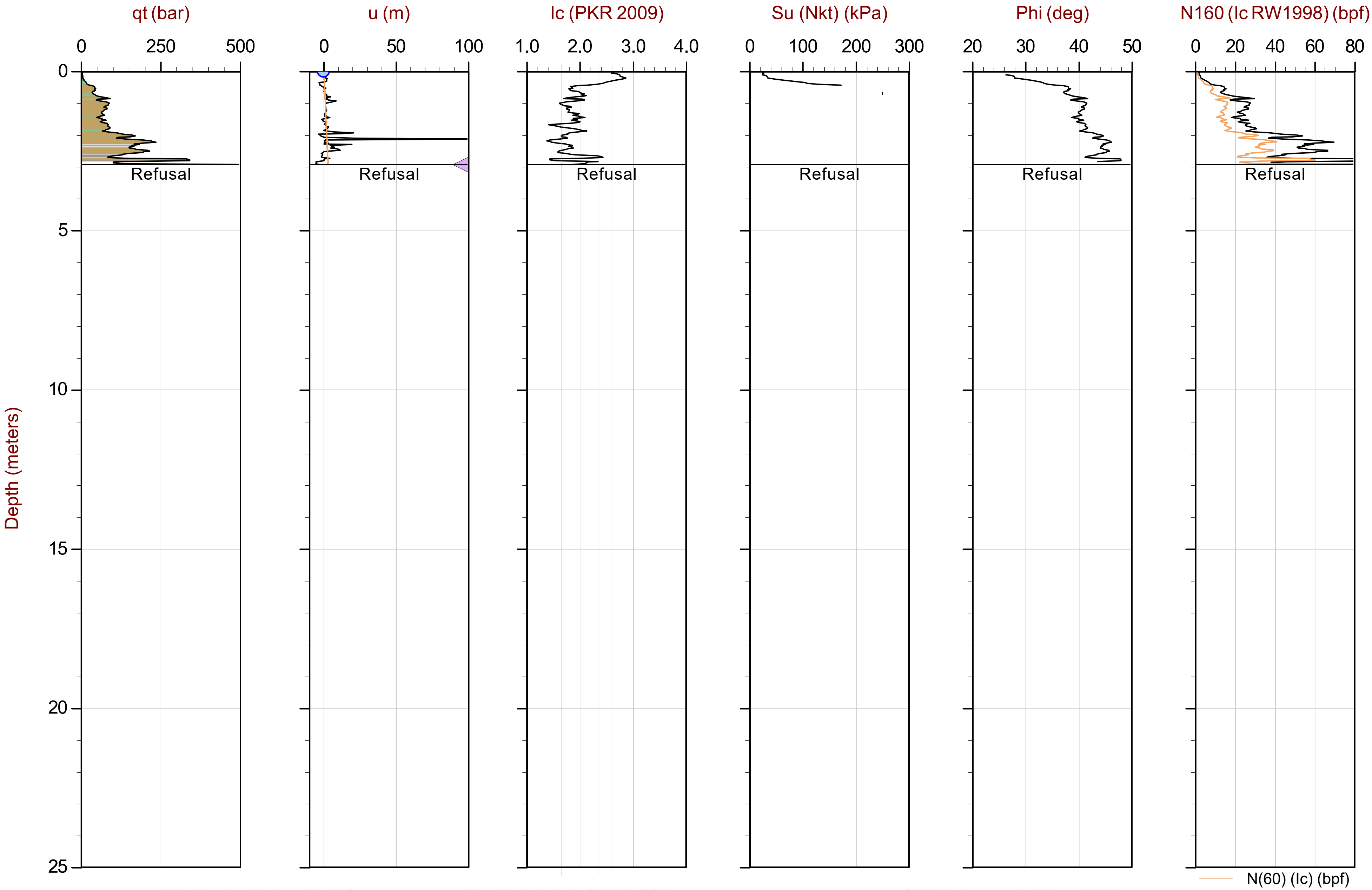
SBT: Robertson, 2009 and 2010
Coords: UTM 18N N: 5020457m E: 445550m
Sheet No: 1 of 1

Overplot Item: Ueq Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Dissipation, Ueq assumed Ueq Line Hydrostatic Line

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

WSP Canada

Job No: 23-05-25254
Date: 2023-03-01 16:44
Site: 3630 and 3690 Riverside Drive
Sounding: SCPT23-01B
Cone: 947:T1000F10U35



Max Depth: 2.925 m / 9.60 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-05-25254_SP01B.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0

SBT: Robertson, 2009 and 2010
Coords: UTM 18N N: 5020456m E: 445550m
Sheet No: 1 of 1

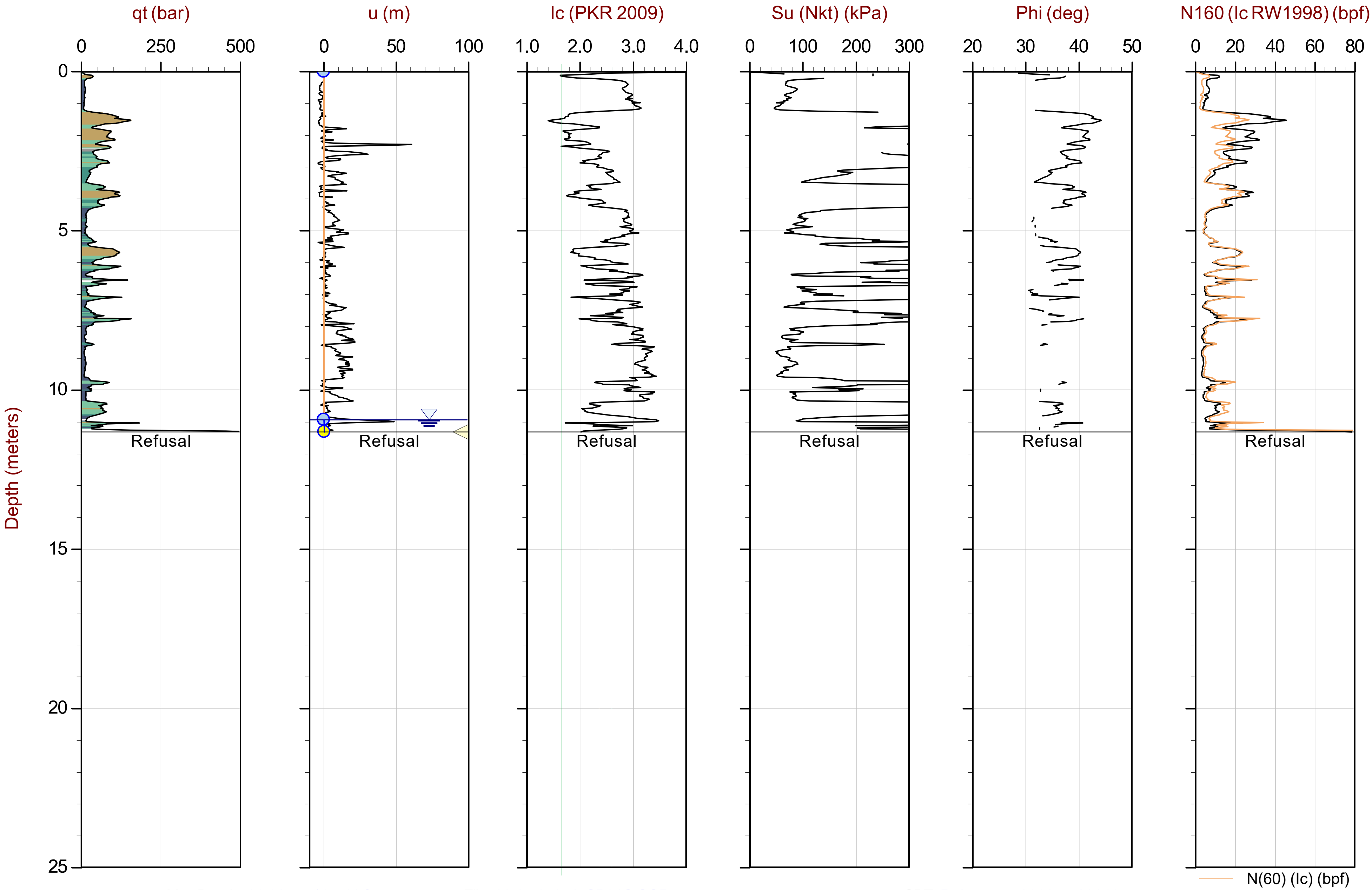
Overplot Item: Ueq Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Dissipation, Ueq assumed Ueq Line Hydrostatic Line

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

WSP Canada

Job No: 23-05-25254
Date: 2023-03-02 07:48
Site: 3630 and 3690 Riverside Drive

Sounding: SCPT23-01C
Cone: 947:T1000F10U35



Max Depth: 11.325 m / 37.16 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-05-25254_SP01C.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0

SBT: Robertson, 2009 and 2010
Coords: UTM 18N N: 5020451m E: 445588m
Sheet No: 1 of 1

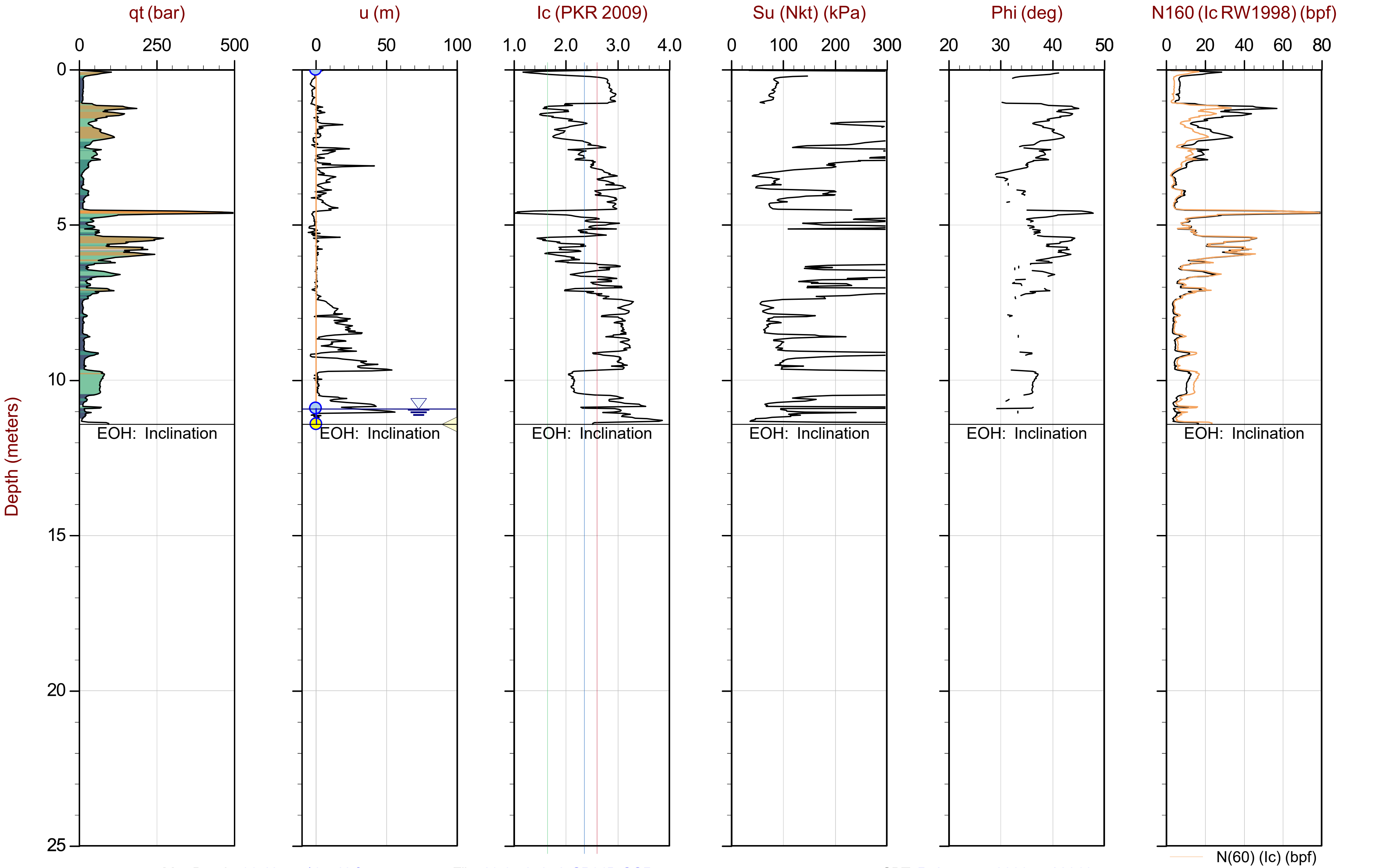
Overplot Item: Ueq | Assumed Ueq | Dissipation, Ueq achieved | Dissipation, Ueq not achieved | Dissipation, Ueq assumed | Ueq Line | Hydrostatic Line

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

WSP Canada

Job No: 23-05-25254
Date: 2023-03-02 09:43
Site: 3630 and 3690 Riverside Drive

Sounding: SCPT23-01D
Cone: 947:T1000F10U35



Max Depth: 11.425 m / 37.48 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-05-25254_SP01D.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0

SBT: Robertson, 2009 and 2010
Coords: UTM 18N N: 5020451m E: 445589m
Sheet No: 1 of 1

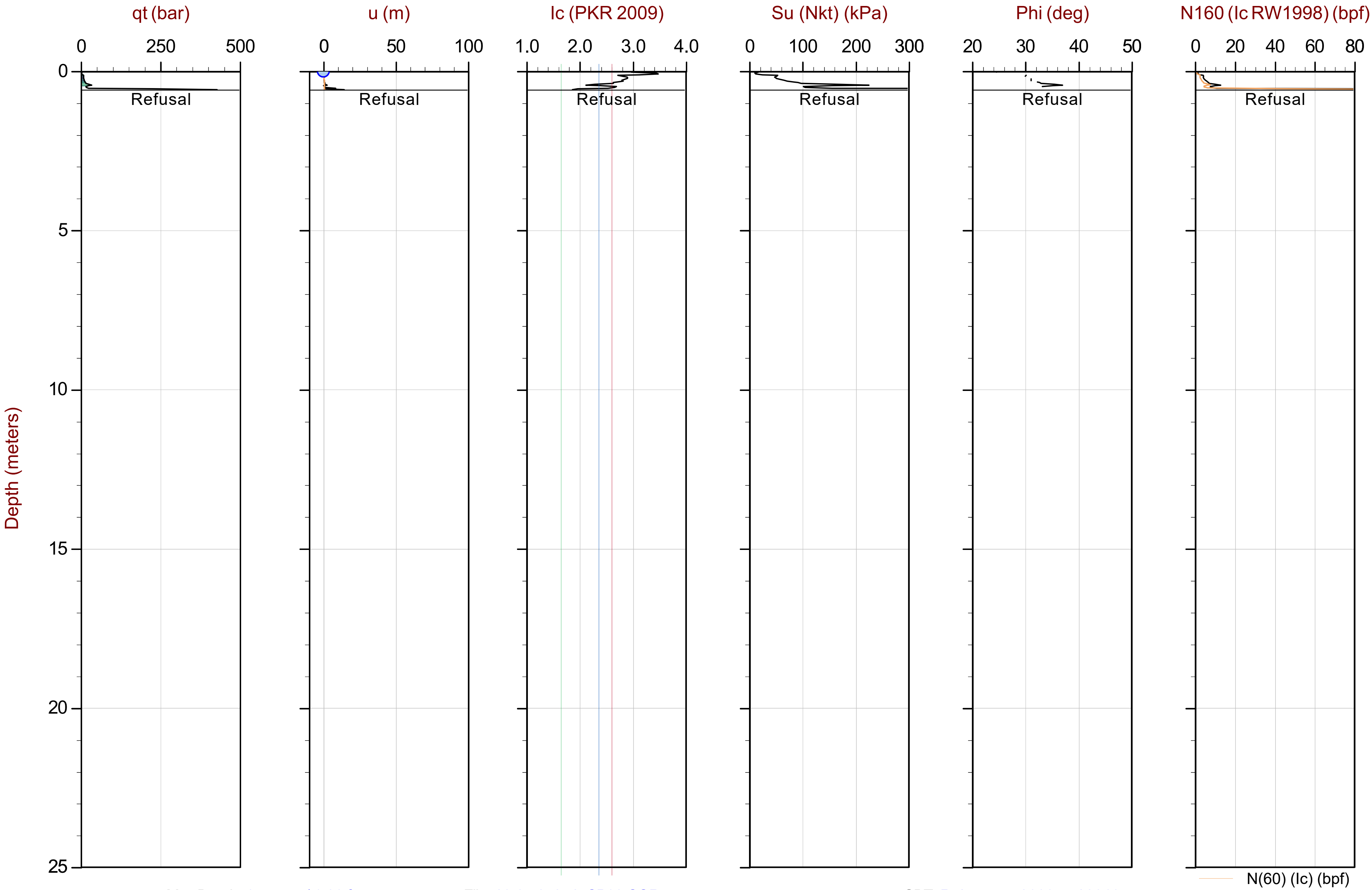
Overplot Item: Ueq · Assumed Ueq · Dissipation, Ueq achieved · Dissipation, Ueq not achieved · Dissipation, Ueq assumed · Ueq Line · Hydrostatic Line

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

WSP Canada

Job No: 23-05-25254
Date: 2023-02-28 14:39
Site: 3630 and 3690 Riverside Drive
Sounding: CPT23-02
Cone: 947:T1000F10U35



Max Depth: 0.575 m / 1.89 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-05-25254_CP02.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0

SBT: Robertson, 2009 and 2010
Coords: UTM 18N N: 5020561m E: 445572m
Sheet No: 1 of 1

Overplot Item: Ueq Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Dissipation, Ueq assumed Ueq Line Hydrostatic Line

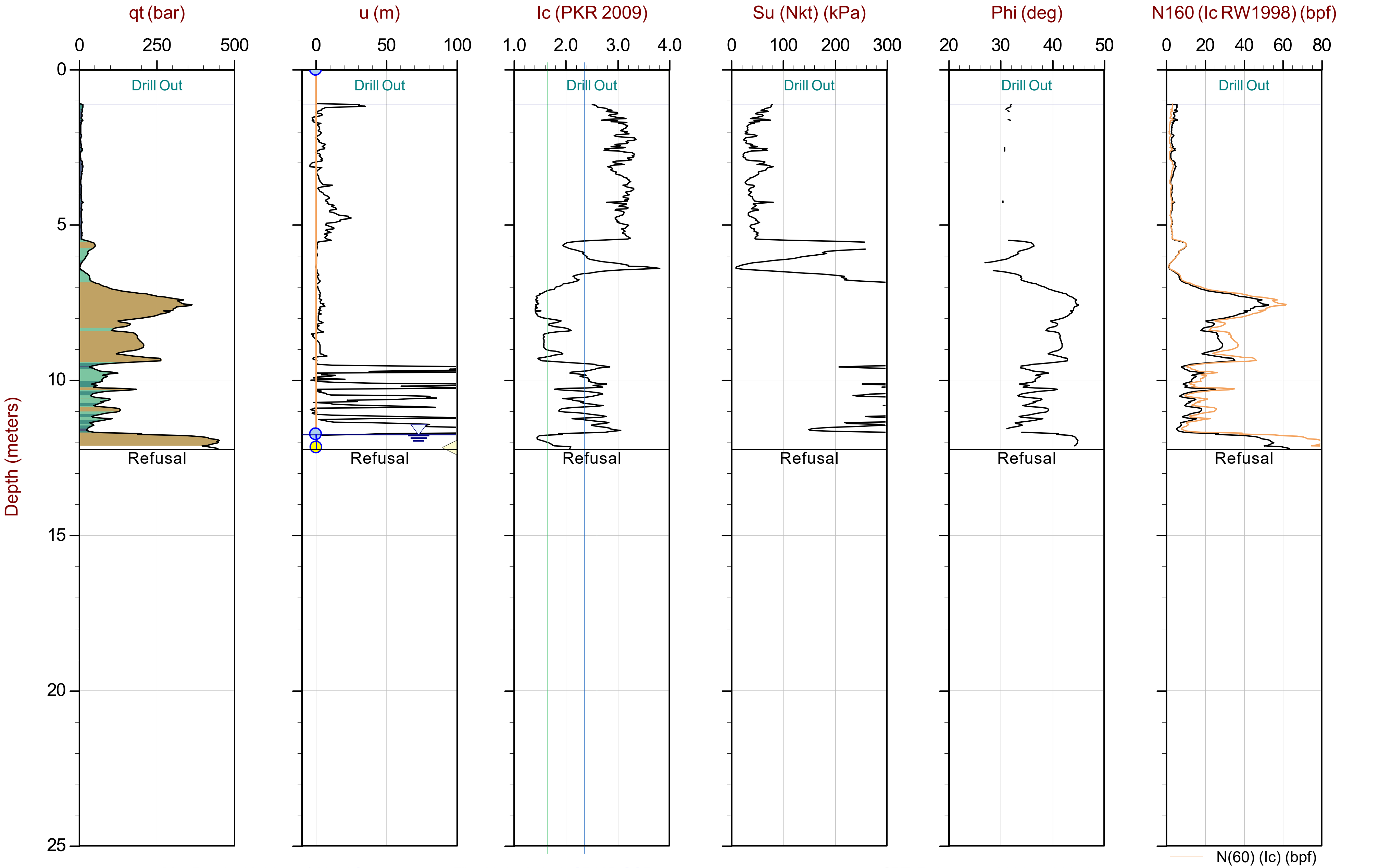
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

ConeTec

WSP Canada

Job No: 23-05-25254
Date: 2023-02-28 15:47
Site: 3630 and 3690 Riverside Drive

Sounding: CPT23-02B
Cone: 947:T1000F10U35



Max Depth: 12.225 m / 40.11 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-05-25254_CP02B.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0

SBT: Robertson, 2009 and 2010
Coords: UTM 18N N: 5020561m E: 445573m
Sheet No: 1 of 1

Overplot Item: Ueq Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Dissipation, Ueq assumed Ueq Line Hydrostatic Line

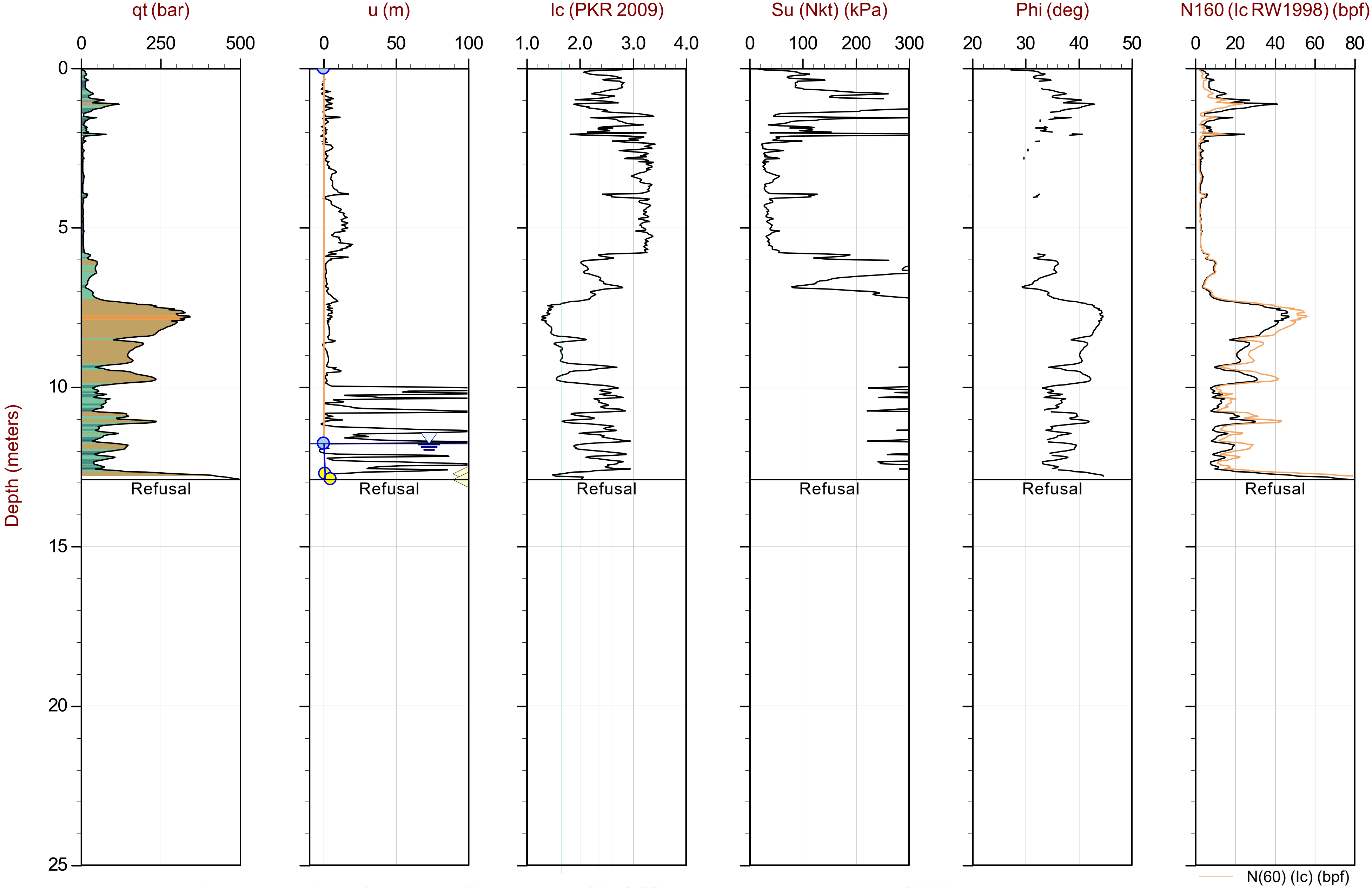
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

WSP Canada

Job No: 23-05-25254
Date: 2023-03-01 06:59
Site: 3630 and 3690 Riverside Drive

Sounding: CPT23-02C
Cone: 947:T1000F10U35



Max Depth: 12.900 m / 42.32 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-05-25254_CP02C.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0

SBT: Robertson, 2009 and 2010
Coords: UTM 18N N: 5020561m E: 445574m
Sheet No: 1 of 1

Overplot Item: Ueq Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Dissipation, Ueq assumed Ueq Line Hydrostatic Line

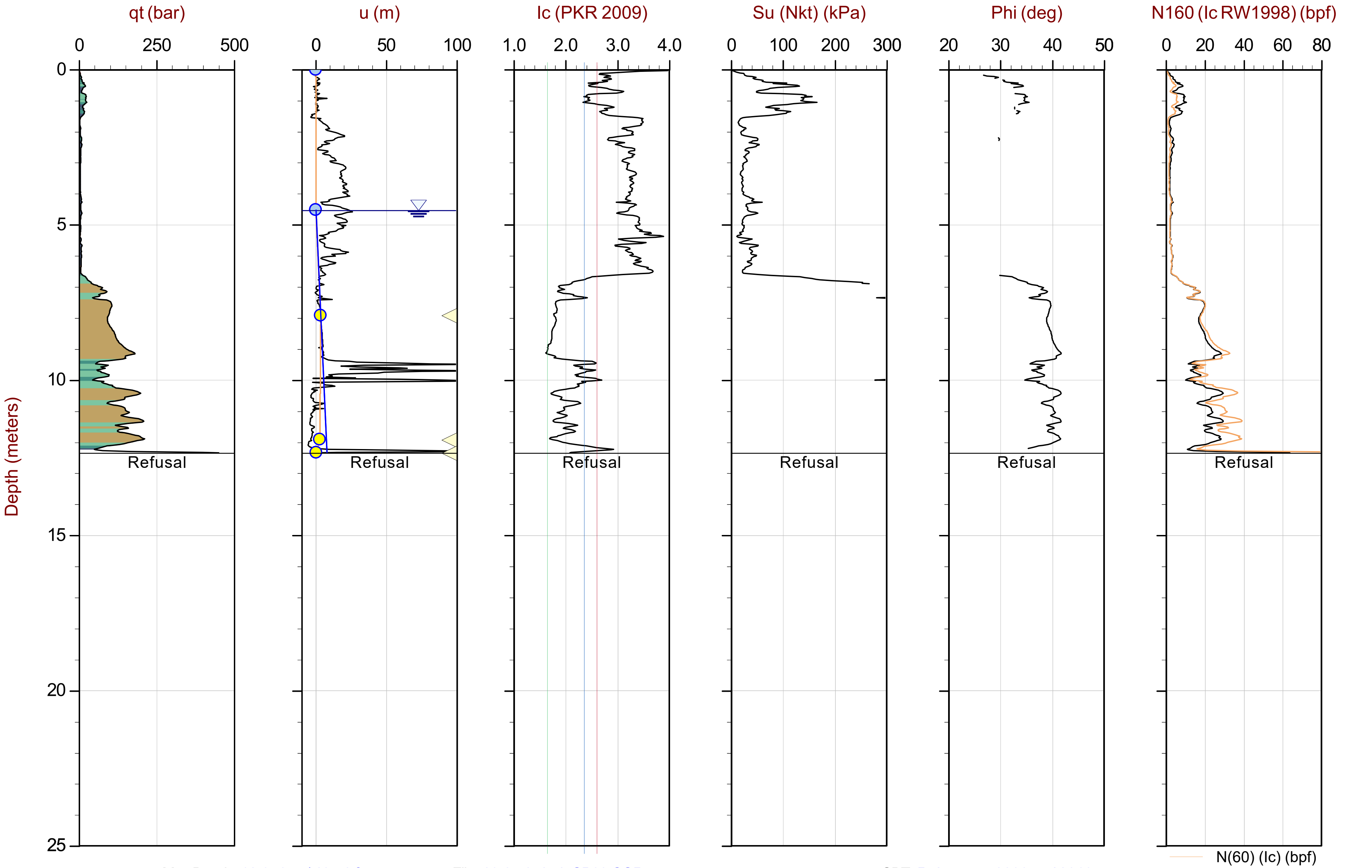
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

WSP Canada

Job No: 23-05-25254
Date: 2023-03-01 09:49
Site: 3630 and 3690 Riverside Drive

Sounding: SCPT23-03
Cone: 947:T1000F10U35



Max Depth: 12.350 m / 40.52 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-05-25254_SP03.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0

SBT: Robertson, 2009 and 2010
Coords: UTM 18N N: 5020585m E: 445530m
Sheet No: 1 of 1

Overplot Item: Ueq Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Dissipation, Ueq assumed Ueq Line Hydrostatic Line

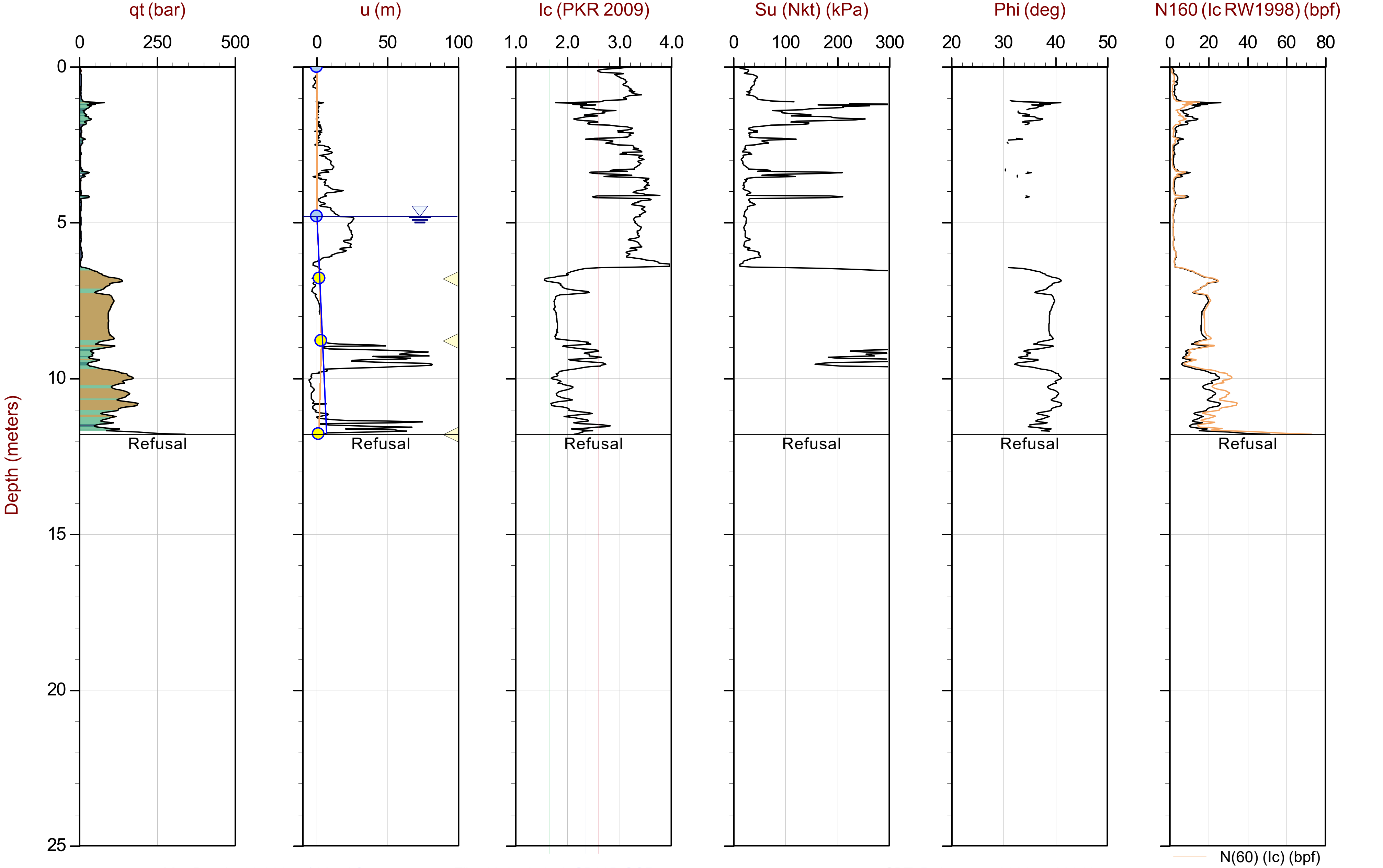
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

WSP Canada

Job No: 23-05-25254
Date: 2023-03-01 12:55
Site: 3630 and 3690 Riverside Drive

Sounding: SCPT23-03B
Cone: 947:T1000F10U35



Max Depth: 11.800 m / 38.71 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-05-25254_SP03B.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0

SBT: Robertson, 2009 and 2010
Coords: UTM 18N N: 5020584m E: 445530m
Sheet No: 1 of 1

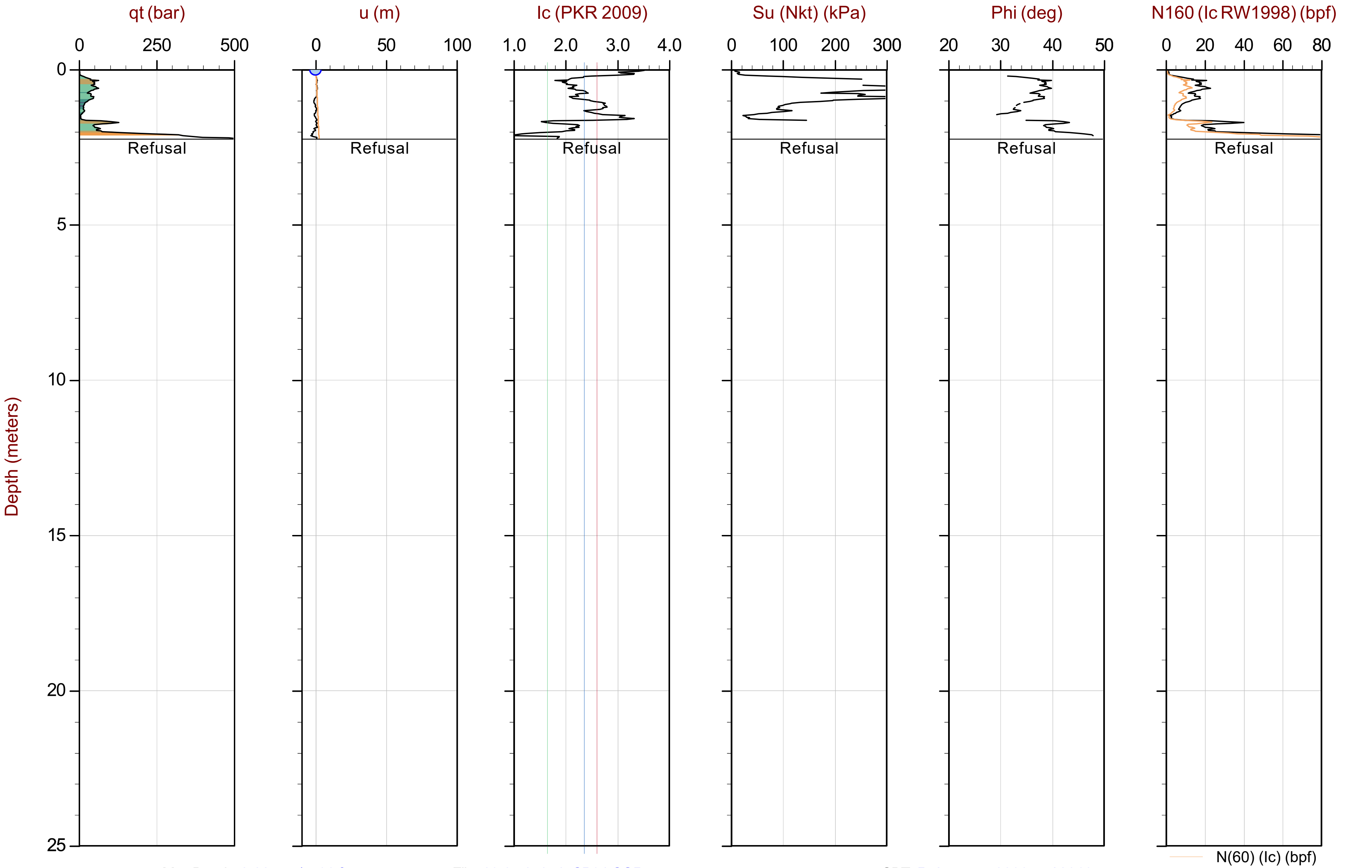
Overplot Item: Ueq Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Dissipation, Ueq assumed Ueq Line Hydrostatic Line

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

WSP Canada

Job No: 23-05-25254
Date: 2023-02-27 16:03
Site: 3630 and 3690 Riverside Drive

Sounding: CPT23-04
Cone: 947:T1000F10U35



Max Depth: 2.225 m / 7.30 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-05-25254_CP04.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0

SBT: Robertson, 2009 and 2010
Coords: UTM 18N N: 5020591m E: 445613m
Sheet No: 1 of 1

Overplot Item: Ueq Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Dissipation, Ueq assumed Ueq Line Hydrostatic Line

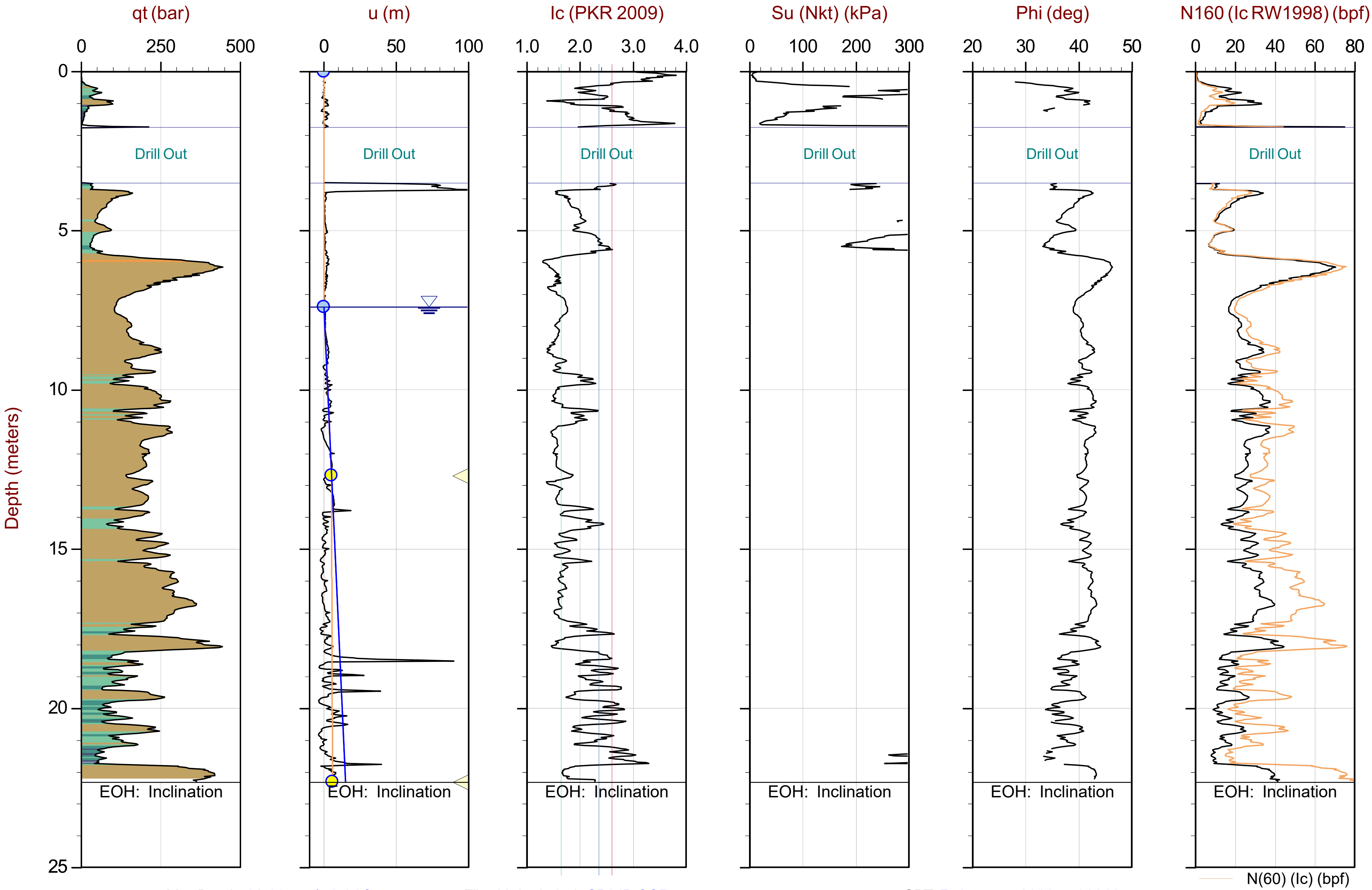
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

WSP Canada

Job No: 23-05-25254
Date: 2023-02-28 10:33
Site: 3630 and 3690 Riverside Drive

Sounding: CPT23-04B
Cone: 947:T1000F10U35



Max Depth: 22.325 m / 73.24 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-05-25254_CP04B.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0

SBT: Robertson, 2009 and 2010
Coords: UTM 18N N: 5020590m E: 445613m
Sheet No: 1 of 1

Overplot Item: Ueq   Assumed Ueq   Dissipation, Ueq achieved   Dissipation, Ueq not achieved   Dissipation, Ueq assumed   Ueq Line   Hydrostatic Line

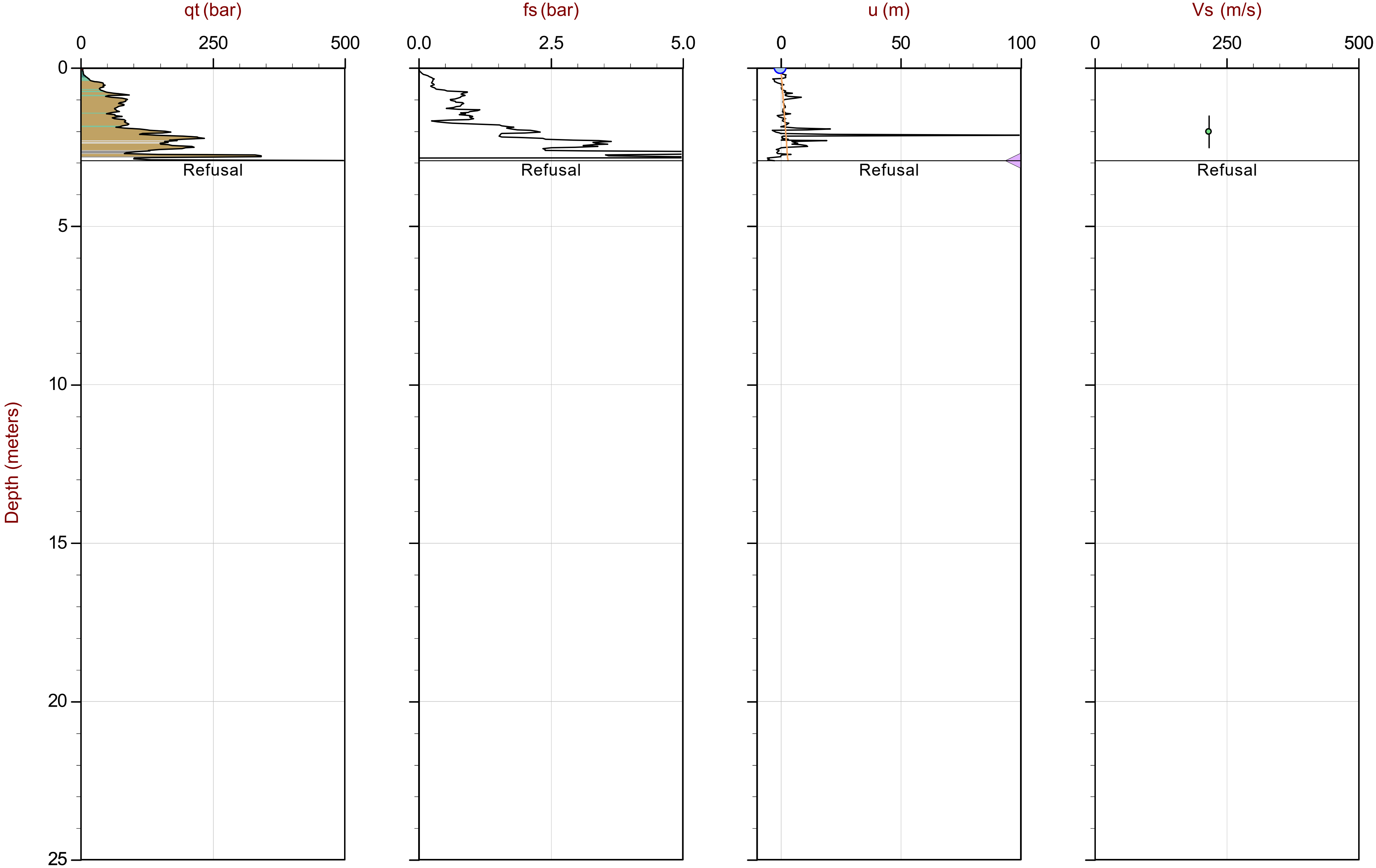
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

Seismic Cone Penetration Test Plots

WSP Canada

Job No: 23-05-25254
Date: 2023-03-01 16:44
Site: 3630 and 3690 Riverside Drive

Sounding: SCPT23-01B
Cone: 947:T1000F10U35



Max Depth: 2.925 m / 9.60 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-05-25254_SP01B.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 18N N: 5020456m E: 445550m
Sheet No: 1 of 1

Overplot Item: Ueq Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Dissipation, Ueq assumed Ueq Line Hydrostatic Line

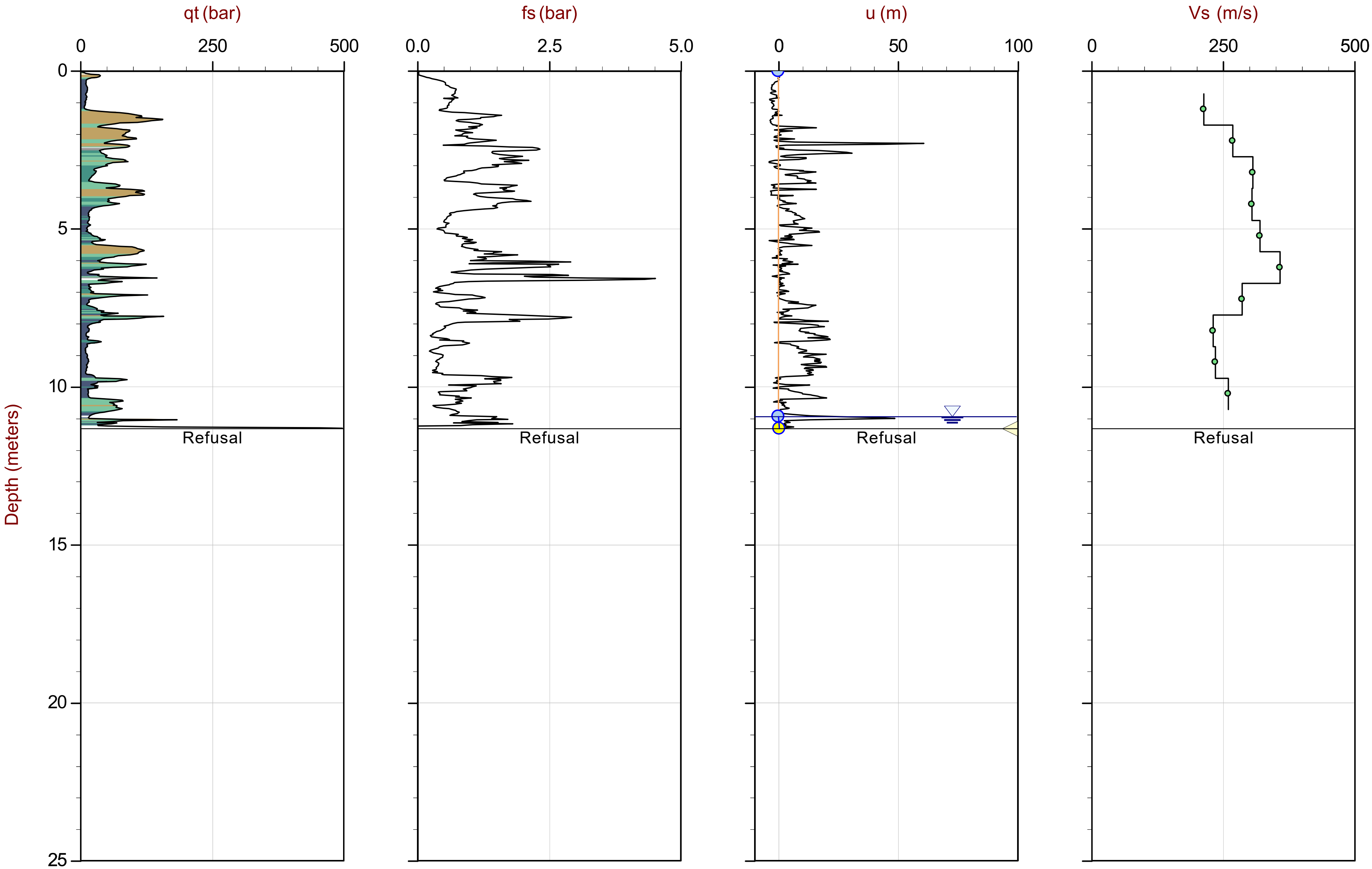
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

WSP Canada

Job No: 23-05-25254
Date: 2023-03-02 07:48
Site: 3630 and 3690 Riverside Drive

Sounding: SCPT23-01C
Cone: 947:T1000F10U35



Max Depth: 11.325 m / 37.16 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-05-25254_SP01C.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 18N N: 5020451m E: 445588m
Sheet No: 1 of 1

Overplot Item: Ueq   Assumed Ueq   Dissipation, Ueq achieved   Dissipation, Ueq not achieved   Dissipation, Ueq assumed   Ueq Line   Hydrostatic Line

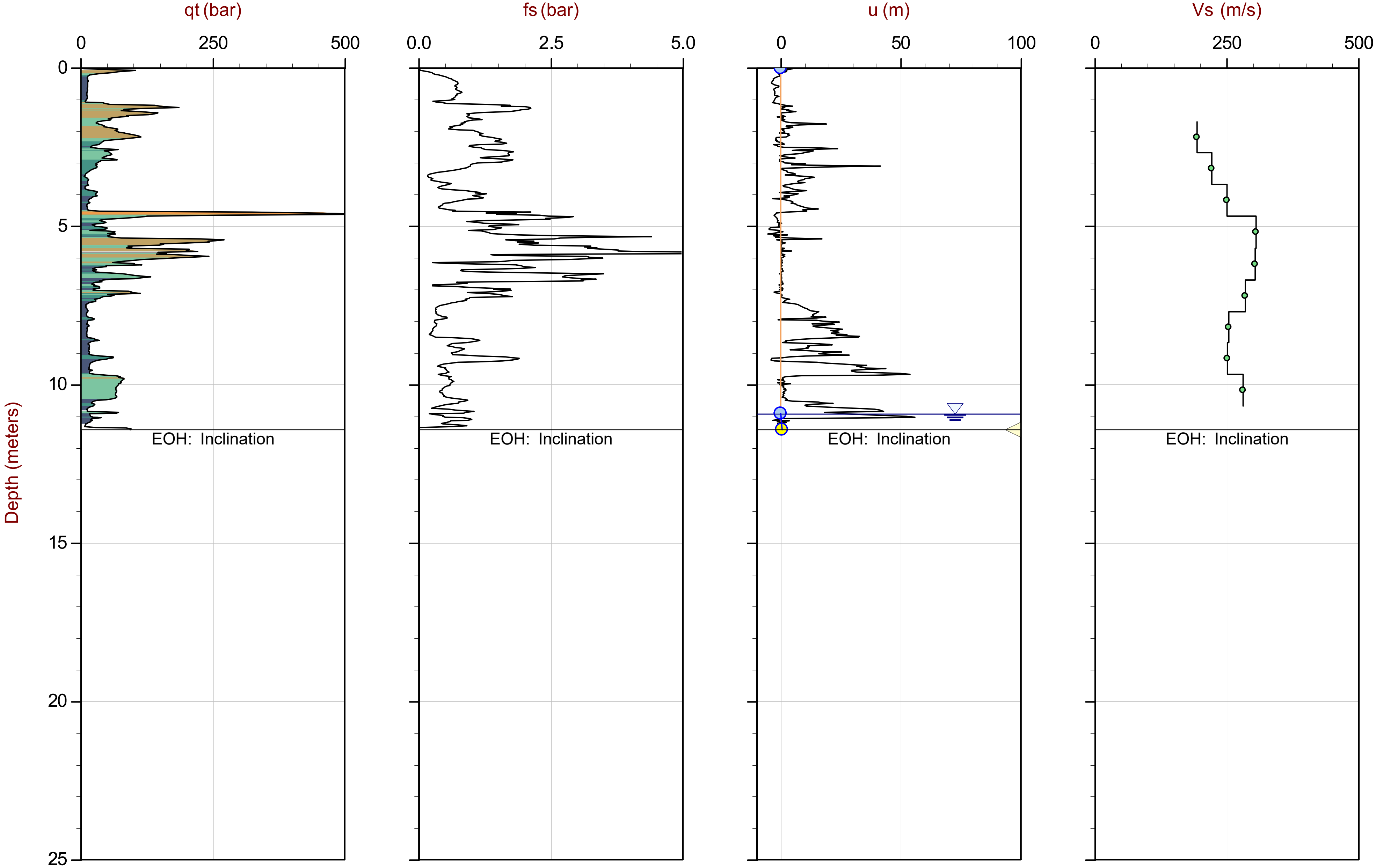
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

WSP Canada

Job No: 23-05-25254
Date: 2023-03-02 09:43
Site: 3630 and 3690 Riverside Drive

Sounding: SCPT23-01D
Cone: 947:T1000F10U35



Max Depth: 11.425 m / 37.48 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-05-25254_SP01D.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 18N N: 5020451m E: 445589m
Sheet No: 1 of 1

Overplot Item: Ueq Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Dissipation, Ueq assumed Ueq Line Hydrostatic Line

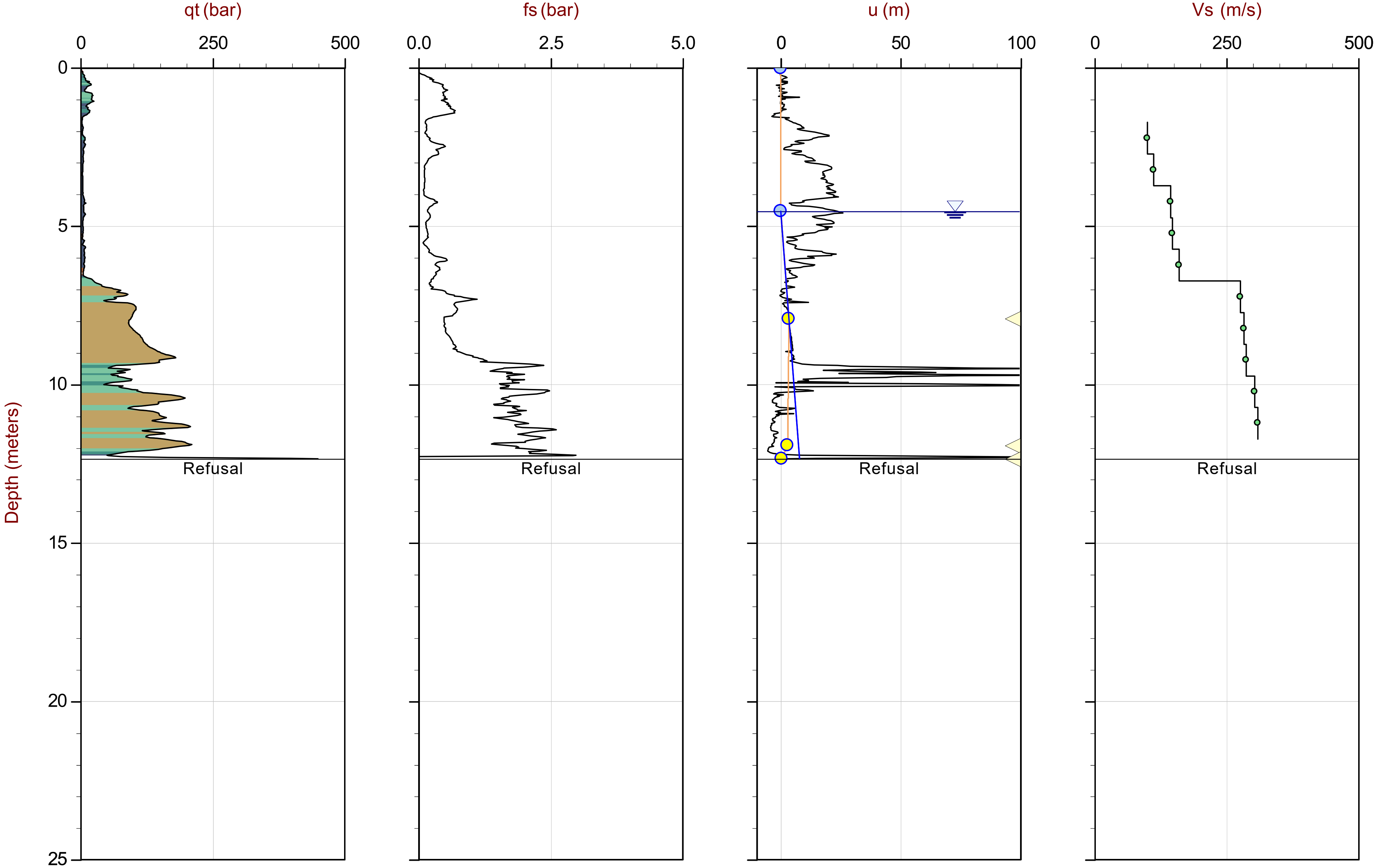
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

WSP Canada

Job No: 23-05-25254
Date: 2023-03-01 09:49
Site: 3630 and 3690 Riverside Drive

Sounding: SCPT23-03
Cone: 947:T1000F10U35



Max Depth: 12.350 m / 40.52 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-05-25254_SP03.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 18N N: 5020585m E: 445530m
Sheet No: 1 of 1

Overplot Item: Ueq Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Dissipation, Ueq assumed Ueq Line Hydrostatic Line

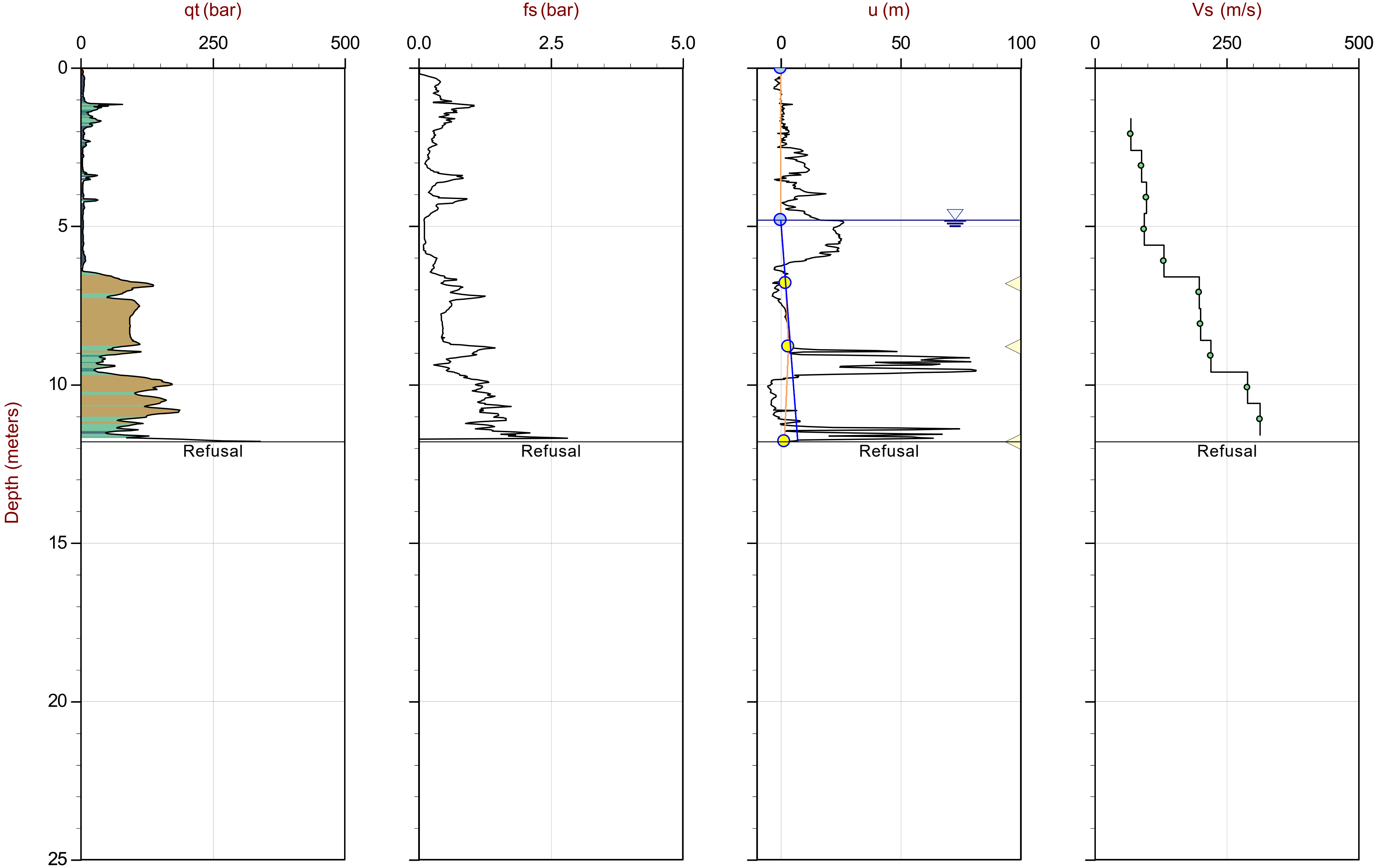
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

WSP Canada

Job No: 23-05-25254
Date: 2023-03-01 12:55
Site: 3630 and 3690 Riverside Drive

Sounding: SCPT23-03B
Cone: 947:T1000F10U35



Max Depth: 11.800 m / 38.71 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 23-05-25254_SP03B.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 18N N: 5020584m E: 445530m
Sheet No: 1 of 1

Overplot Item: Ueq Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Dissipation, Ueq assumed Ueq Line Hydrostatic Line

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

Seismic Cone Penetration Test Shear Wave (Vs) Tabular Results

Job No: 23-05-25254
Client: WSP Canada Inc.
Project: Riverside Dr and Hunt Club Rd
Sounding ID: SCPT23-01B
Date: 01-Mar-2023

Seismic Source: Beam
Seismic Offset (m): 3.50
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)
1.72	1.52	3.82			
2.72	2.52	4.31	0.50	2.30	216

Job No: 23-05-25254
Client: WSP Canada Inc.
Project: Riverside Dr and Hunt Club Rd
Sounding ID: SCPT23-01C
Date: 02-Mar-2023

Seismic Source: Beam
Seismic Offset (m): 3.50
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)
0.92	0.72	3.57			
1.92	1.72	3.90	0.33	1.53	213
2.92	2.72	4.43	0.53	1.98	269
3.92	3.72	5.11	0.68	2.20	307
4.93	4.73	5.88	0.78	2.55	305
5.93	5.73	6.71	0.83	2.60	320
6.93	6.73	7.59	0.87	2.44	358
7.93	7.73	8.49	0.90	3.15	286
8.93	8.73	9.41	0.92	3.98	231
9.93	9.73	10.34	0.94	3.98	235
10.92	10.72	11.28	0.94	3.61	260

Job No:	23-05-25254
Client:	WSP Canada Inc.
Project:	Riverside Dr and Hunt Club Rd
Sounding ID:	SCPT23-01D
Date:	02-Mar-2023
Seismic Source:	Beam
Seismic Offset (m):	3.50
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)
1.90	1.70	3.89			
2.88	2.68	4.41	0.52	2.67	194
3.88	3.68	5.08	0.67	3.03	222
4.88	4.68	5.84	0.77	3.06	250
5.90	5.70	6.69	0.85	2.77	306
6.90	6.70	7.56	0.87	2.86	304
7.90	7.70	8.46	0.90	3.15	285
8.88	8.68	9.36	0.90	3.55	254
9.88	9.68	10.29	0.93	3.71	252
10.88	10.68	11.24	0.95	3.36	281

Job No: 23-05-25254
Client: WSP Canada Inc.
Project: Riverside Dr and Hunt Club Rd
Sounding ID: SCPT23-03
Date: 01-Mar-2023

Seismic Source: Beam
Seismic Offset (m): 3.50
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)
1.92	1.72	3.90			
2.92	2.72	4.43	0.53	5.37	99
3.92	3.72	5.11	0.68	6.05	112
4.93	4.73	5.88	0.78	5.39	144
5.93	5.73	6.71	0.83	5.64	147
6.93	6.73	7.59	0.87	5.47	160
7.93	7.73	8.49	0.90	3.26	276
8.93	8.73	9.41	0.92	3.26	282
9.93	9.73	10.34	0.94	3.26	287
10.92	10.72	11.28	0.94	3.09	303
11.92	11.72	12.23	0.95	3.08	309

Job No:	23-05-25254
Client:	WSP Canada Inc.
Project:	Riverside Dr and Hunt Club Rd
Sounding ID:	SCPT23-03B
Date:	01-Mar-2023
Seismic Source:	Beam
Seismic Offset (m):	3.50
Source Depth (m):	0.00
Geophone Offset (m):	0.20

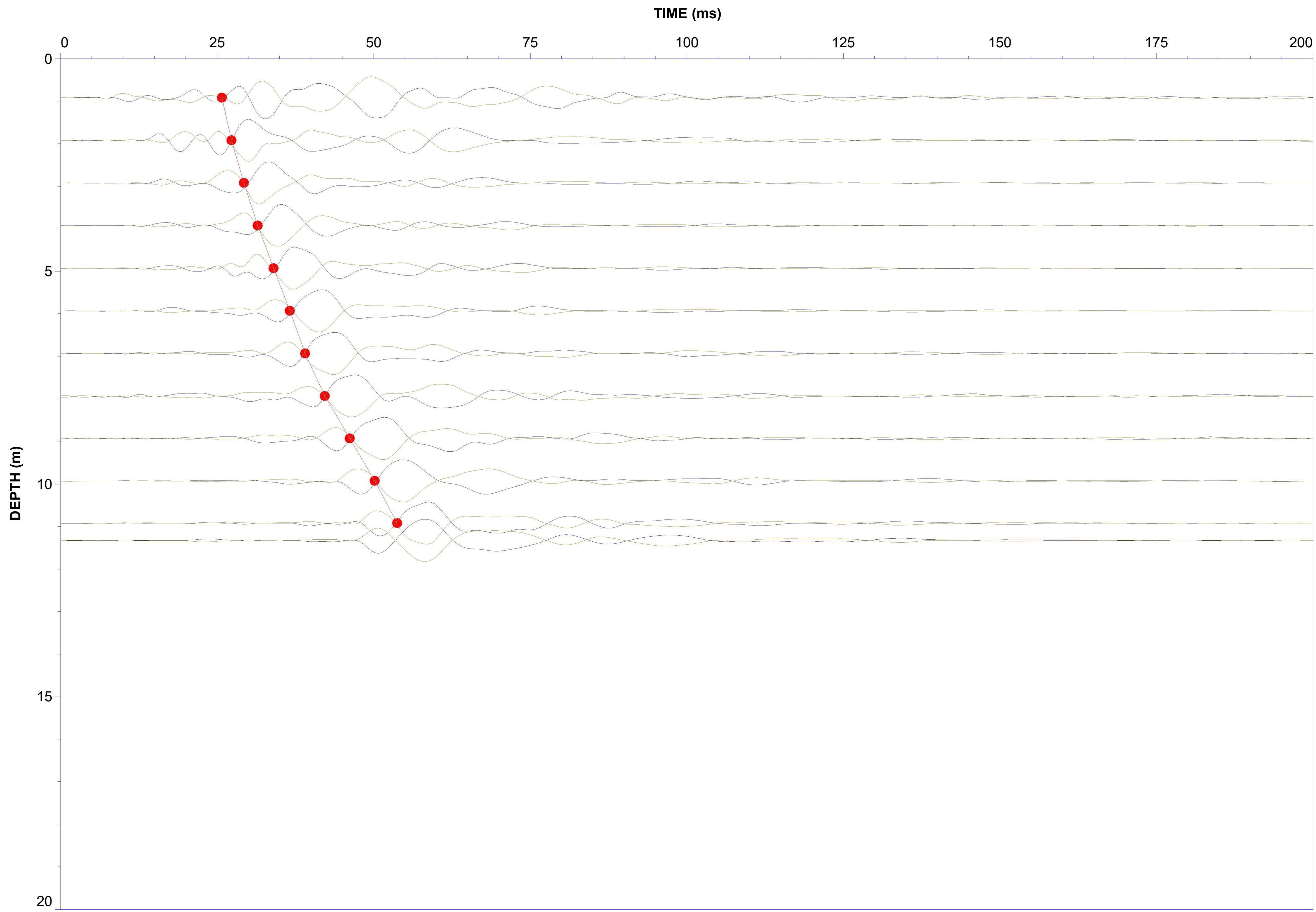
SCPTu SHEAR WAVE VELOCITY TEST RESULTS - Vs

Tip Depth (m)	Geophone Depth (m)	Ray Path (m)	Ray Path Difference (m)	Travel Time Interval (ms)	Interval Velocity (m/s)
1.80	1.60	3.85			
2.80	2.60	4.36	0.51	7.54	68
3.80	3.60	5.02	0.66	7.44	89
4.80	4.60	5.78	0.76	7.77	98
5.80	5.60	6.60	0.82	8.83	93
6.80	6.60	7.47	0.87	6.62	131
7.80	7.60	8.37	0.90	4.53	198
8.80	8.60	9.29	0.92	4.58	200
9.80	9.60	10.22	0.93	4.24	220
10.80	10.60	11.16	0.95	3.26	290
11.80	11.60	12.12	0.95	3.05	313

Seismic Cone Penetration Test Shear Wave (Vs) Traces

Job No: 23-05-25254	Client: WSP Canada	Project: Riverside Dr and Hunt Club Rd	Sounding: SCPT23-01B	Filter: 0-200 Hz
Date: 01-Mar-2023	Analysis: S Wave - Geo X			

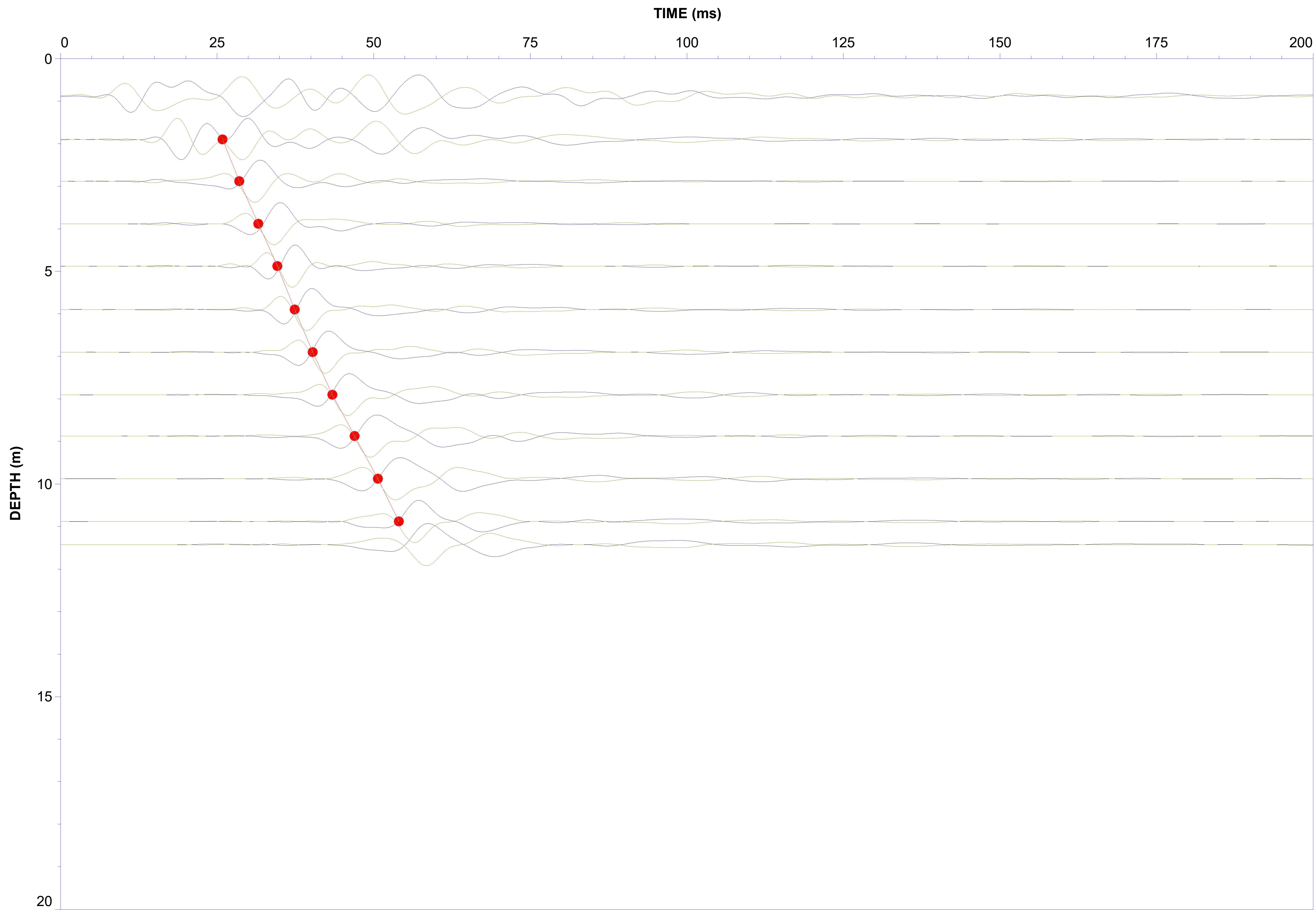
Job No: 23-05-25254 Client: WSP Canada Project: Riverside Dr and Hunt Club Rd Sounding: SCPT23-01C Filter: 0-300 Hz Date: 02-Mar-2023
Analysis: S Wave - Geo X



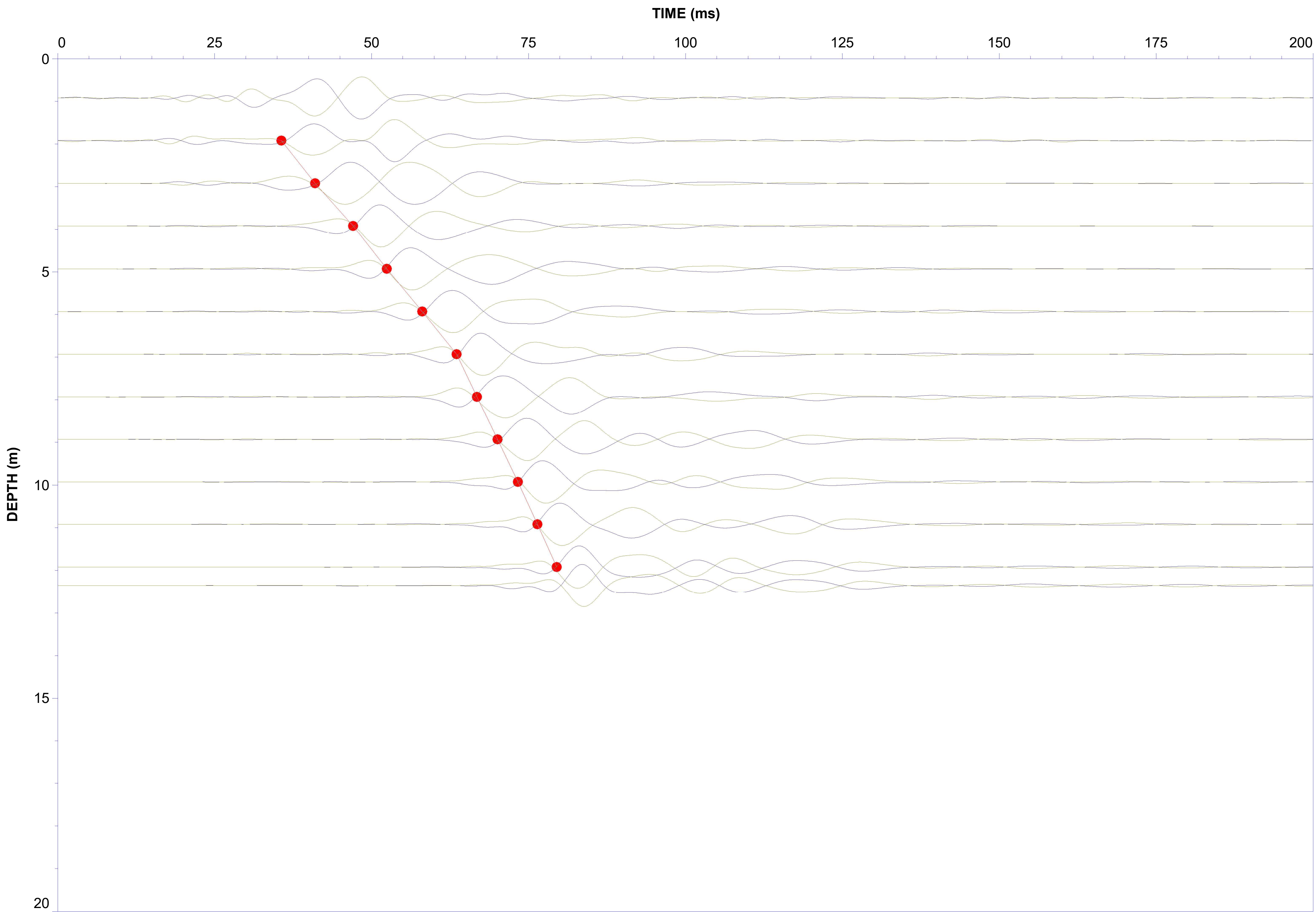
CONETEC

Job No: 23-05-25254 | Client: WSP Canada | Project: Riverside Dr and Hunt Club Rd | Sounding: SCPT23-01D | Filter: 0-200 Hz

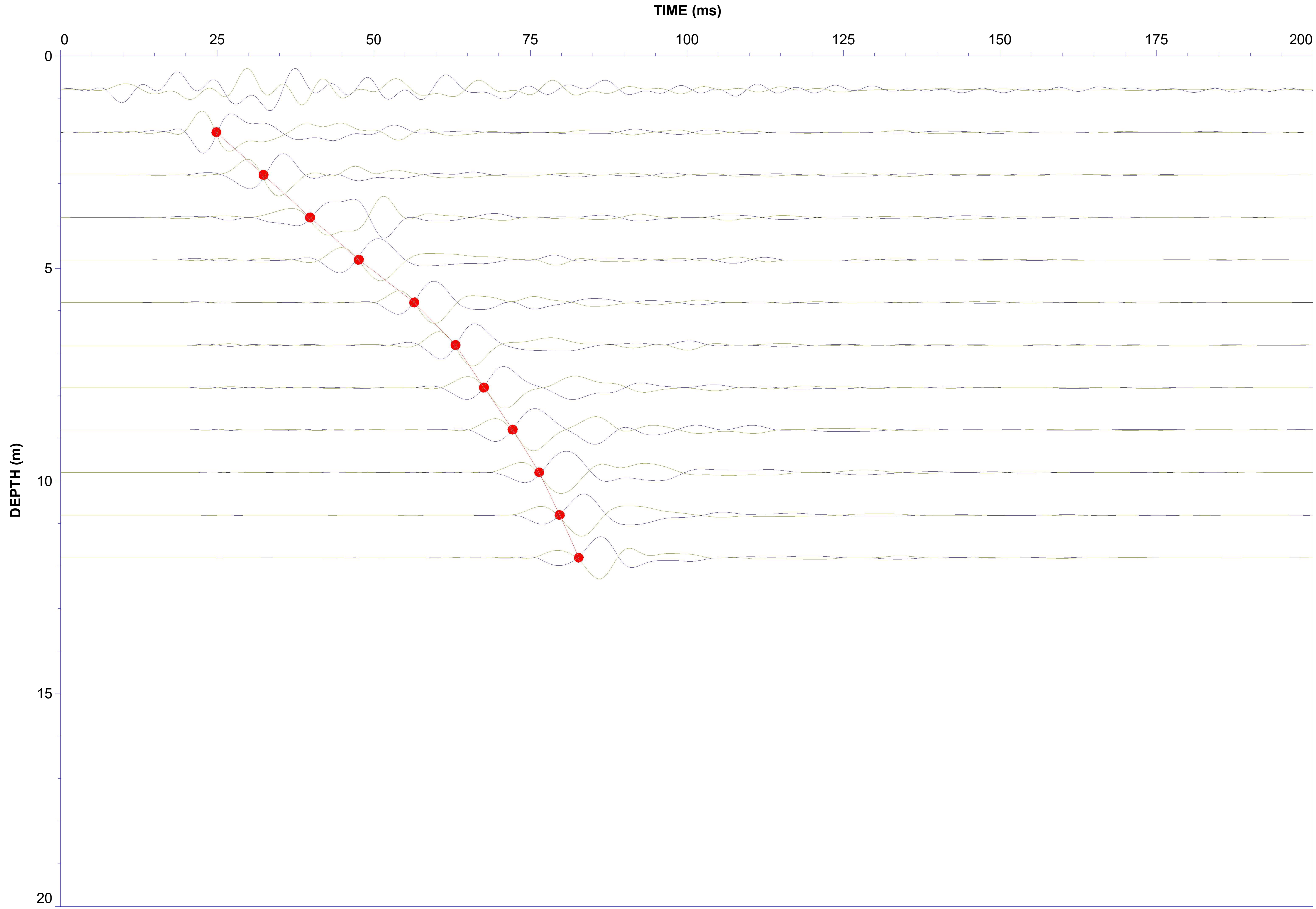
Date: 02-Mar-2023 | Analysis: S Wave - Geo X



Job No: 23-05-25254
Date: 01-Mar-2023
Client: WSP Canada
Analysis: S Wave - Geo X
Project: Riverside Dr and Hunt Club Rd
Sounding: SCPT23-03
Filter: 0-200 Hz



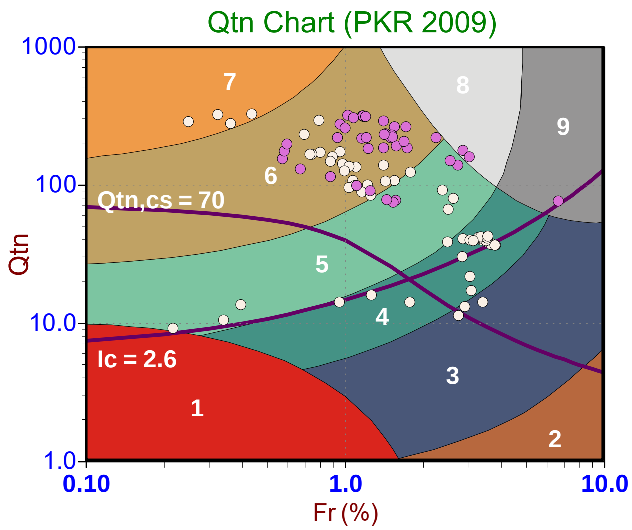
Job No: 23-05-25254 | Client: WSP Canada | Project: Riverside Dr and Hunt Club Rd | Sounding: SCPT23-03B | Filter: 0-200 Hz | Date: 01-Mar-2023
Analysis: S Wave - Geo X

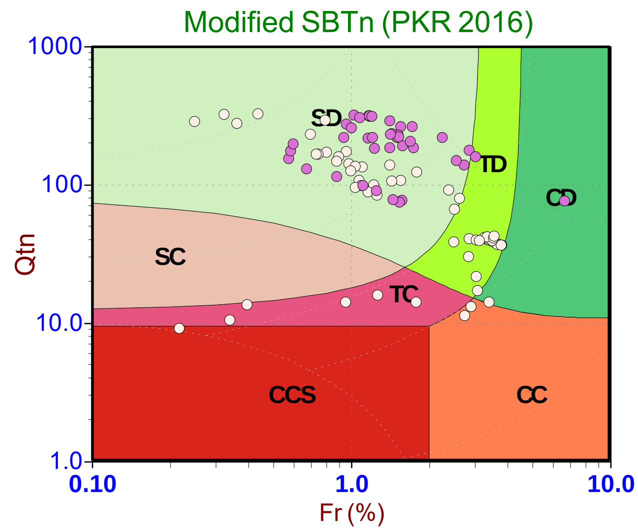


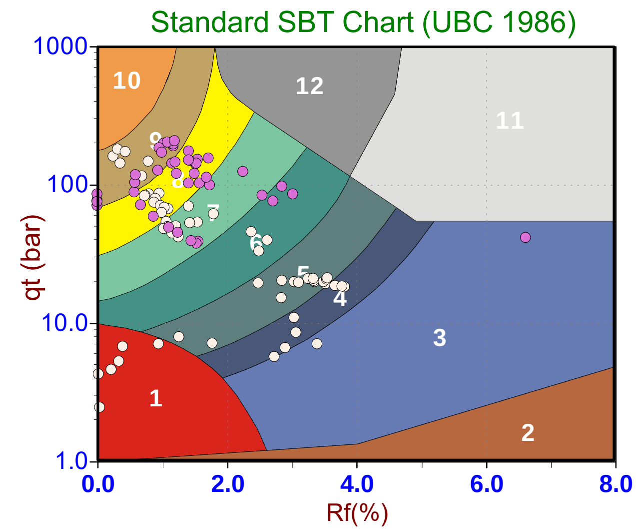
Soil Behaviour Type (SBT) Scatter Plots

ConeTec

WSP Canada	Job No: 23-05-25254	Sounding: SCPT23-01
	Date: 2023-03-01 15:36	Cone: 947:T1000F10U35
	Site: 3630 and 3690 Riverside Drive	







Depth Ranges
- >0.0 to 1.5 m
- >1.5 to 3.0 m
- >3.0 to 4.5 m
- >4.5 to 6.0 m
- >6.0 to 7.5 m
- >7.5 to 9.0 m
- >9.0 to 10.5 m
- >10.5 to 12.0 m
- >12.0 to 13.5 m
- >13.5 to 15.0 m
- >15.0 m

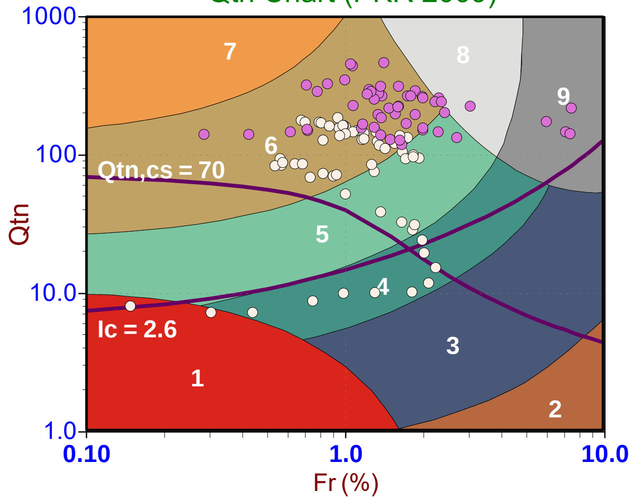
Legend
- Sensitive, Fine Grained
- Organic Soils
- Clays
- Silt Mixtures
- Sand Mixtures
- Sands
- Gravelly Sand to Sand
- Stiff Sand to Clayey Sand
- Very Stiff Fine Grained

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)

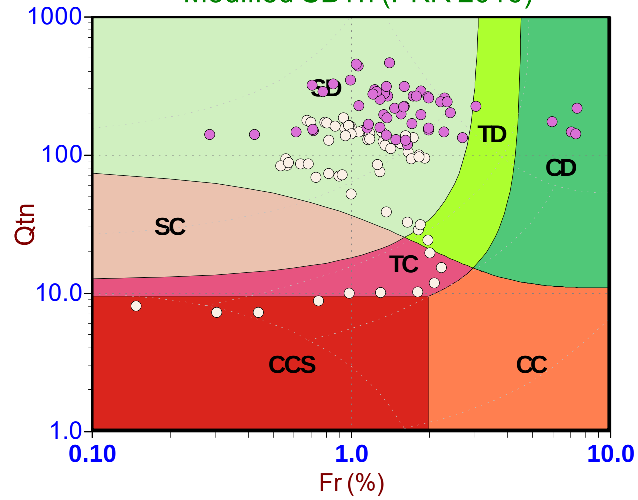
Legend
- Sensitive Fines
- Organic Soil
- Clay
- Silty Clay
- Clayey Silt
- Silt
- Sandy Silt
- Silty Sand/Sand
- Sand
- Gravelly Sand
- Stiff Fine Grained
- Cemented Sand

WSP Canada	Job No: 23-05-25254 Date: 2023-03-01 16:44 Site: 3630 and 3690 Riverside Drive	Sounding: SCPT23-01B Cone: 947:T1000F10U35

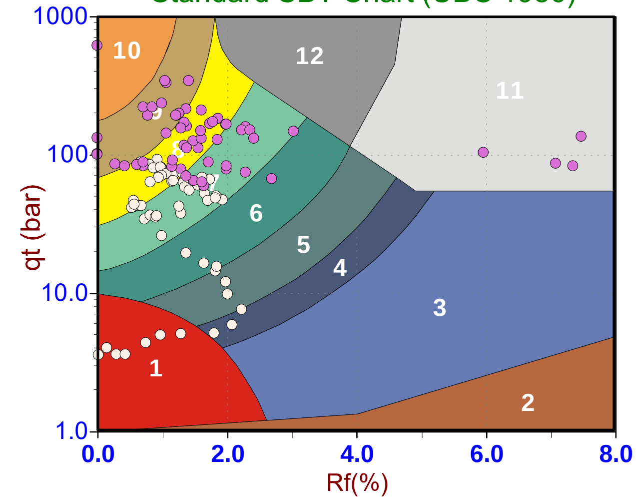
Qtn Chart (PKR 2009)



Modified SBTn (PKR 2016)



Standard SBT Chart (UBC 1986)



Depth Ranges
- >0.0 to 1.5 m
- >1.5 to 3.0 m
- >3.0 to 4.5 m
- >4.5 to 6.0 m
- >6.0 to 7.5 m
- >7.5 to 9.0 m
- >9.0 to 10.5 m
- >10.5 to 12.0 m
- >12.0 to 13.5 m
- >13.5 to 15.0 m
- >15.0 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

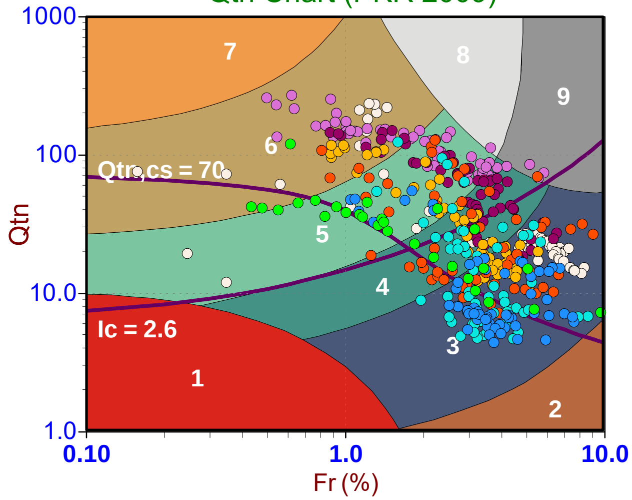
CONETEC

WSP Canada

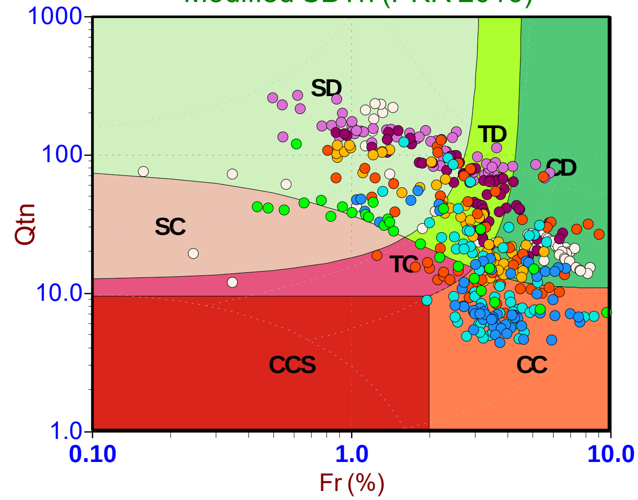
Job No: 23-05-25254
Date: 2023-03-02 07:48
Site: 3630 and 3690 Riverside Drive

Sounding: SCPT23-01C
Cone: 947:T1000F10U35

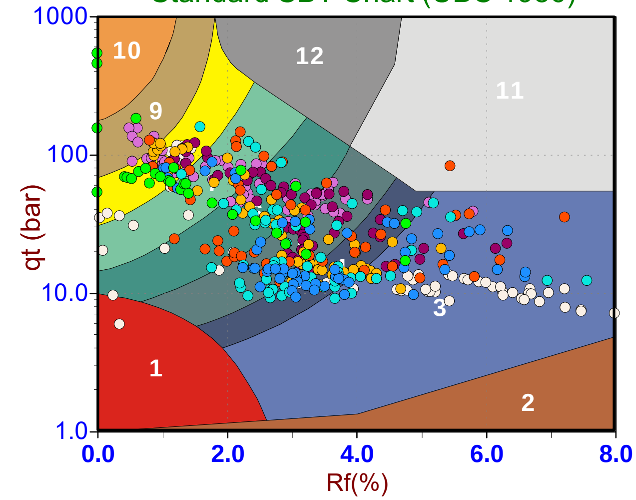
Qtn Chart (PKR 2009)



Modified SBTn (PKR 2016)



Standard SBT Chart (UBC 1986)



Depth Ranges
- >0.0 to 1.5 m
- >1.5 to 3.0 m
- >3.0 to 4.5 m
- >4.5 to 6.0 m
- >6.0 to 7.5 m
- >7.5 to 9.0 m
- >9.0 to 10.5 m
- >10.5 to 12.0 m
- >12.0 to 13.5 m
- >13.5 to 15.0 m
- >15.0 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

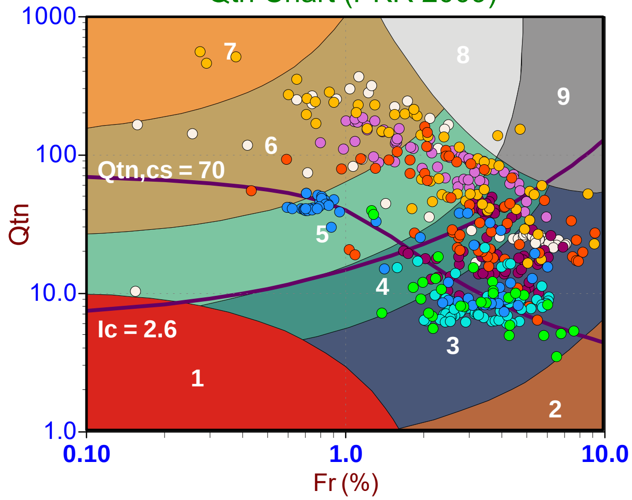
CONETEC

WSP Canada

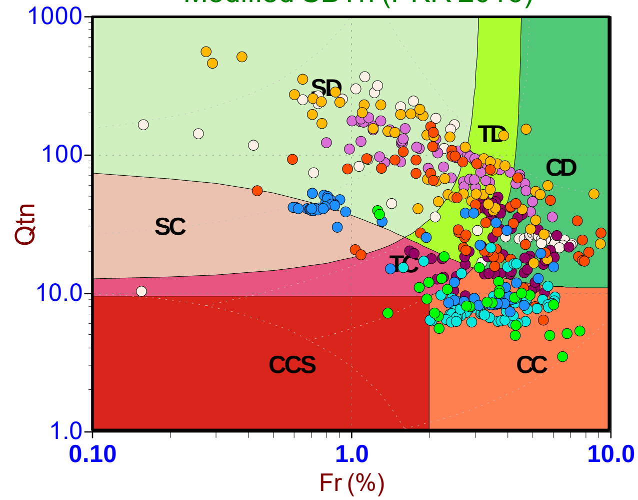
Job No: 23-05-25254
Date: 2023-03-02 09:43
Site: 3630 and 3690 Riverside Drive

Sounding: SCPT23-01D
Cone: 947:T1000F10U35

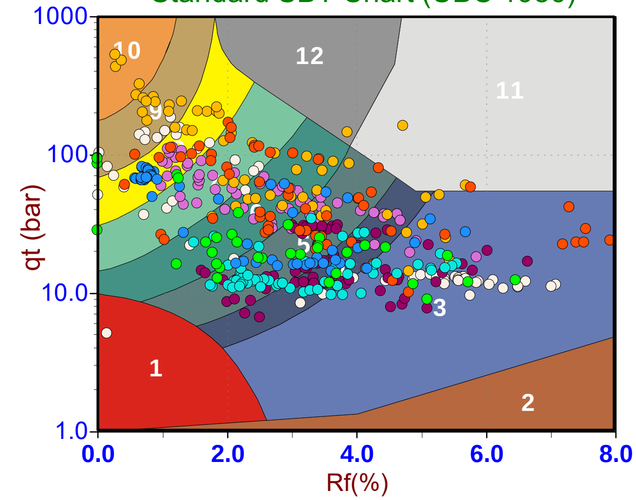
Qtn Chart (PKR 2009)



Modified SBTn (PKR 2016)



Standard SBT Chart (UBC 1986)



Depth Ranges

- >0.0 to 1.5 m
- >1.5 to 3.0 m
- >3.0 to 4.5 m
- >4.5 to 6.0 m
- >6.0 to 7.5 m
- >7.5 to 9.0 m
- >9.0 to 10.5 m
- >10.5 to 12.0 m
- >12.0 to 13.5 m
- >13.5 to 15.0 m
- >15.0 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

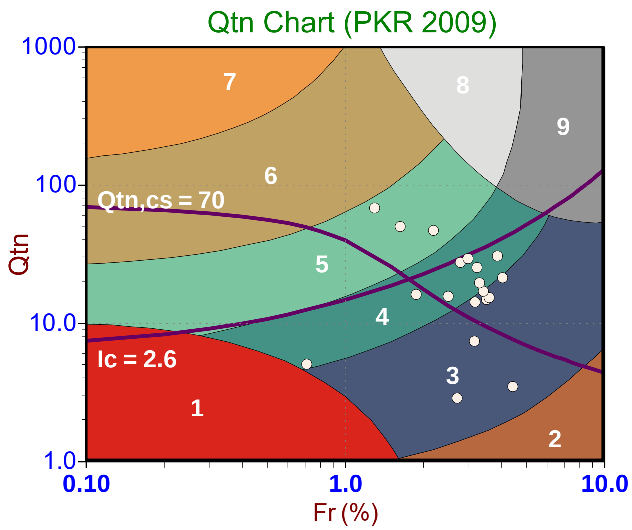
Legend

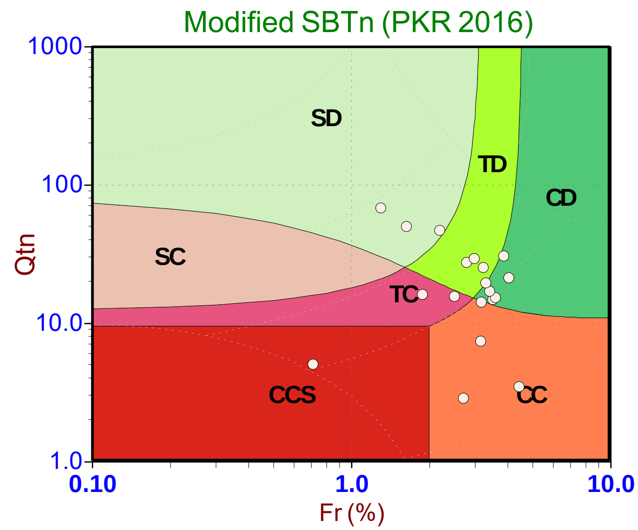
- CCS (Cont. sensitive clay like)
- CC (Cont. clay like)
- TC (Cont. transitional)
- SC (Cont. sand like)
- CD (Dil. clay like)
- TD (Dil. transitional)
- SD (Dil. sand like)

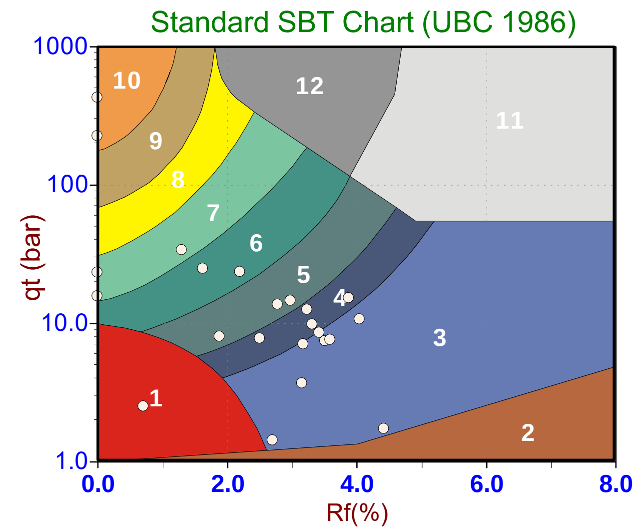
Legend

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

WSP Canada	Job No: 23-05-25254 Date: 2023-02-28 14:39 Site: 3630 and 3690 Riverside Drive	Sounding: CPT23-02 Cone: 947:T1000F10U35







Depth Ranges

- >0.0 to 1.5 m
- >1.5 to 3.0 m
- >3.0 to 4.5 m
- >4.5 to 6.0 m
- >6.0 to 7.5 m
- >7.5 to 9.0 m
- >9.0 to 10.5 m
- >10.5 to 12.0 m
- >12.0 to 13.5 m
- >13.5 to 15.0 m
- >15.0 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

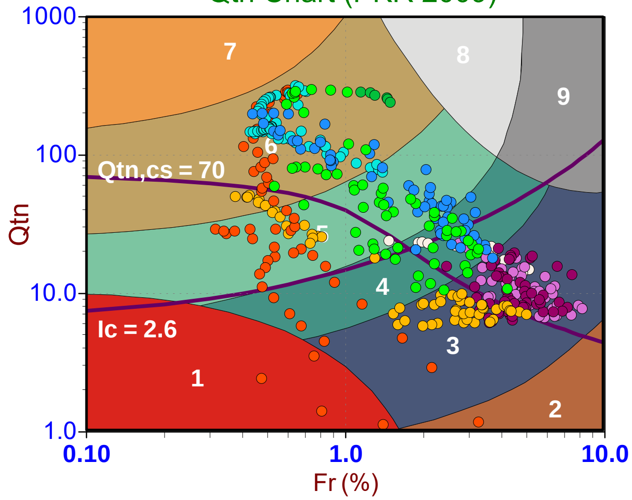
- Sensitive Fines
- Organic Soil
- Clay
- Silty Clay
- Clayey Silt
- Silt
- Sandy Silt
- Silty Sand/Sand
- Sand
- Gravelly Sand
- Stiff Fine Grained
- Cemented Sand

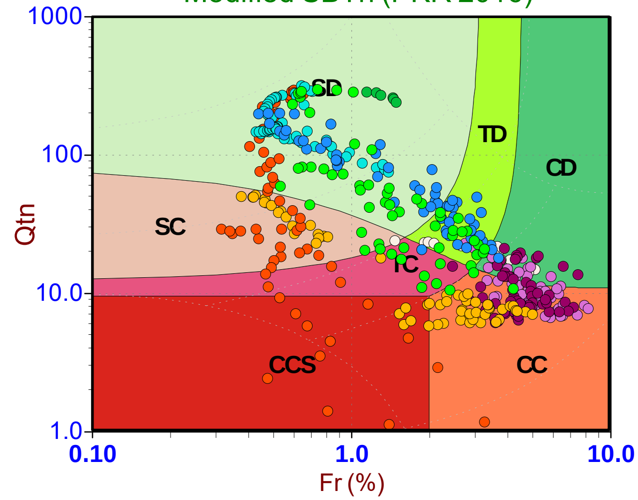
CONETEC

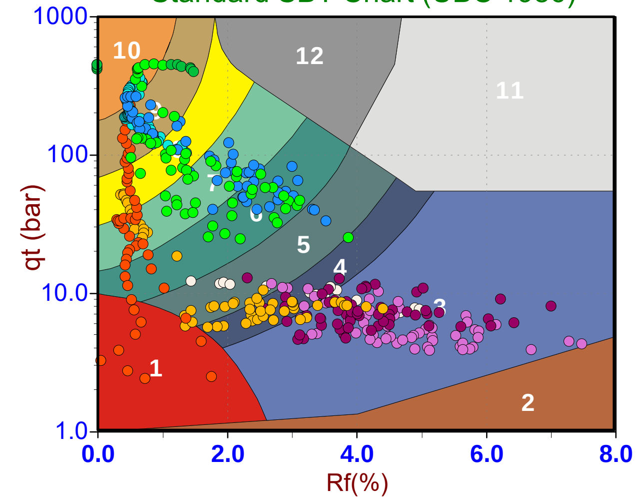
WSP Canada

Job No: 23-05-25254
Date: 2023-02-28 15:47
Site: 3630 and 3690 Riverside Drive

Sounding: CPT23-02B
Cone: 947:T1000F10U35







Depth Ranges
- >0.0 to 1.5 m
- >1.5 to 3.0 m
- >3.0 to 4.5 m
- >4.5 to 6.0 m
- >6.0 to 7.5 m
- >7.5 to 9.0 m
- >9.0 to 10.5 m
- >10.5 to 12.0 m
- >12.0 to 13.5 m
- >13.5 to 15.0 m
- >15.0 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

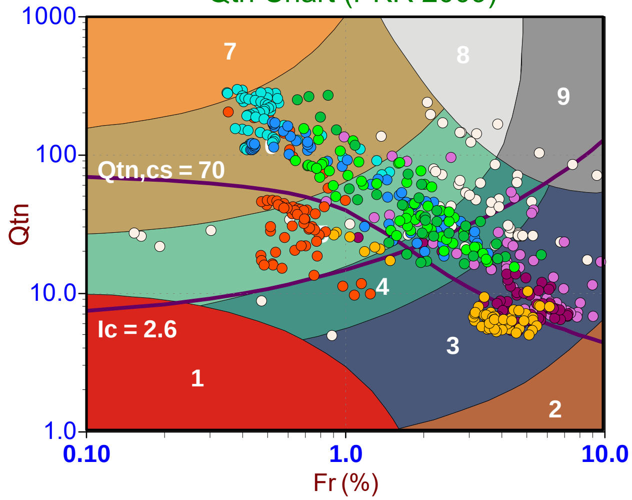
CONETEC

WSP Canada

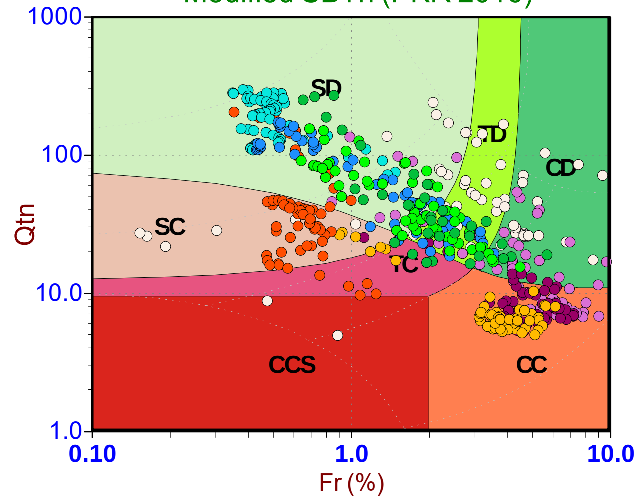
Job No: 23-05-25254
Date: 2023-03-01 06:59
Site: 3630 and 3690 Riverside Drive

Sounding: CPT23-02C
Cone: 947:T1000F10U35

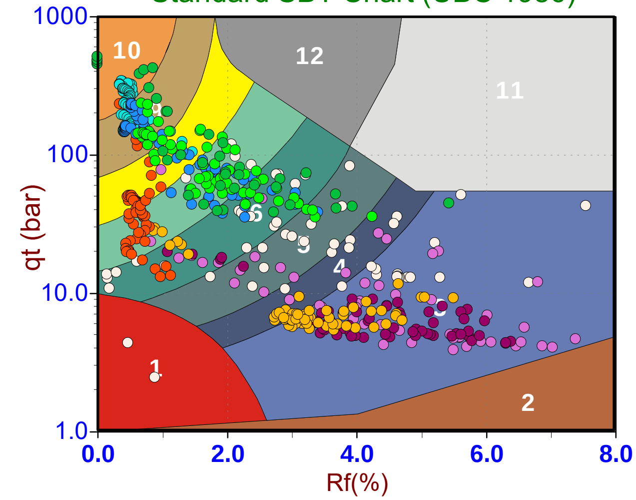
Qtn Chart (PKR 2009)



Modified SBTn (PKR 2016)



Standard SBT Chart (UBC 1986)



Depth Ranges

- >0.0 to 1.5 m
- >1.5 to 3.0 m
- >3.0 to 4.5 m
- >4.5 to 6.0 m
- >6.0 to 7.5 m
- >7.5 to 9.0 m
- >9.0 to 10.5 m
- >10.5 to 12.0 m
- >12.0 to 13.5 m
- >13.5 to 15.0 m
- >15.0 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)

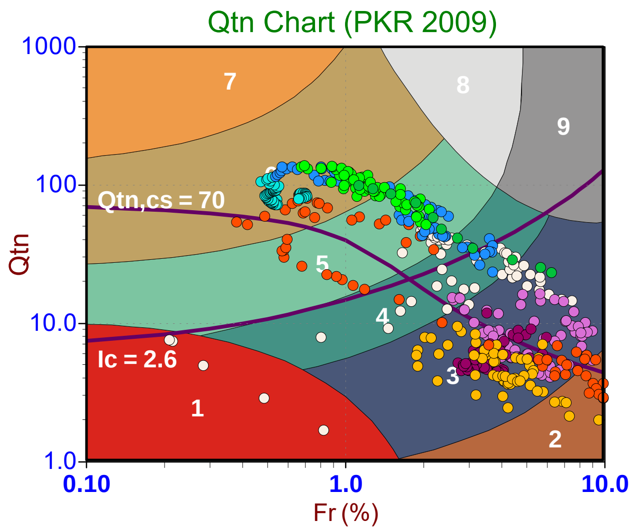
Legend

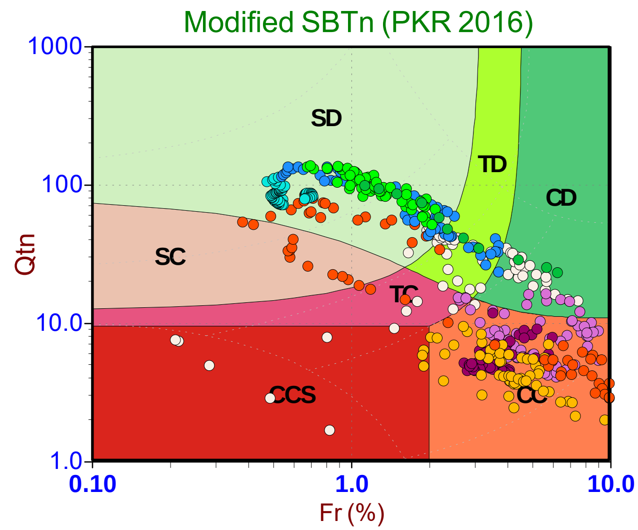
- Sensitive Fines
- Organic Soil
- Clay
- Silty Clay
- Clayey Silt
- Silt
- Sandy Silt
- Silty Sand/Sand
- Sand
- Gravelly Sand
- Stiff Fine Grained
- Cemented Sand

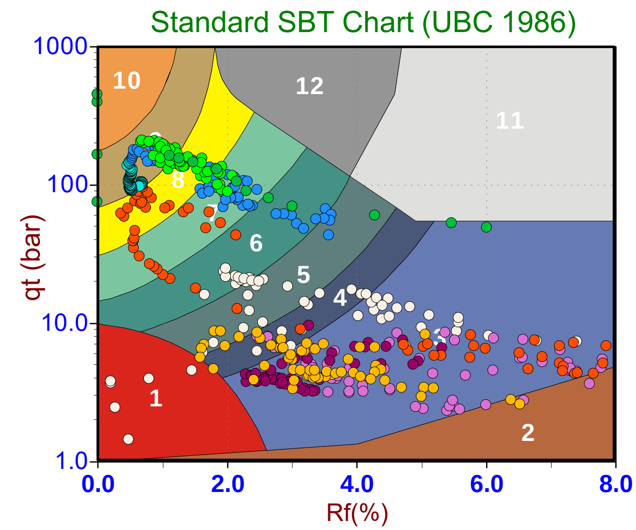
WSP Canada

Job No: 23-05-25254
Date: 2023-03-01 09:49
Site: 3630 and 3690 Riverside Drive

Sounding: SCPT23-03
Cone: 947:T1000F10U35







Depth Ranges
- >0.0 to 1.5 m
- >1.5 to 3.0 m
- >3.0 to 4.5 m
- >4.5 to 6.0 m
- >6.0 to 7.5 m
- >7.5 to 9.0 m
- >9.0 to 10.5 m
- >10.5 to 12.0 m
- >12.0 to 13.5 m
- >13.5 to 15.0 m
- >15.0 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

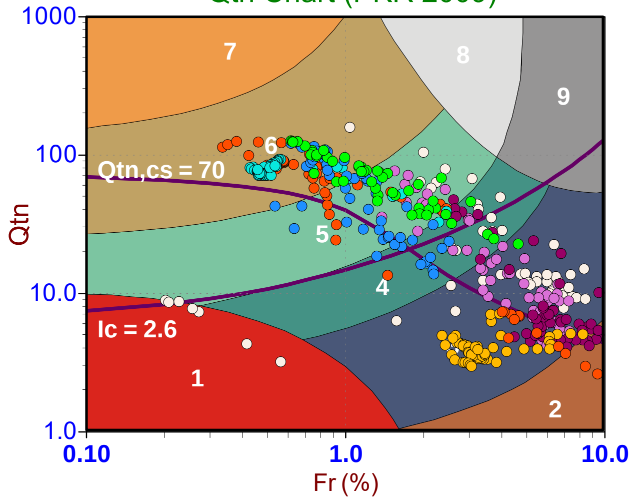
CONETEC

WSP Canada

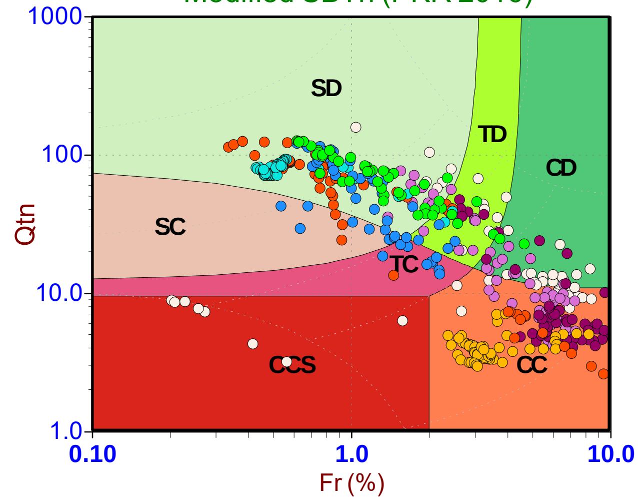
Job No: 23-05-25254
Date: 2023-03-01 12:55
Site: 3630 and 3690 Riverside Drive

Sounding: SCPT23-03B
Cone: 947:T1000F10U35

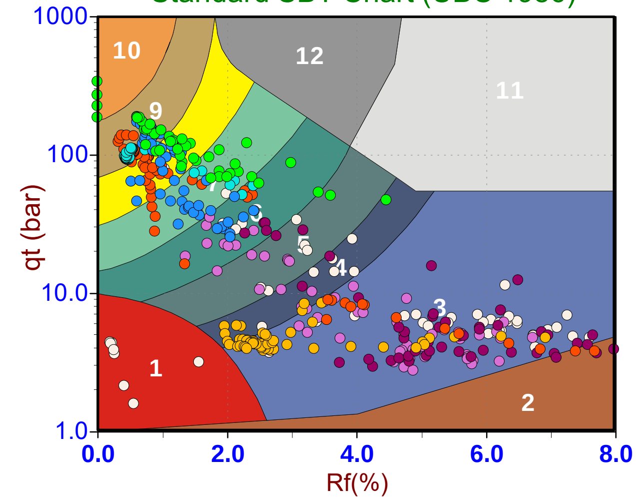
Qtn Chart (PKR 2009)



Modified SBTn (PKR 2016)



Standard SBT Chart (UBC 1986)



Depth Ranges
- >0.0 to 1.5 m
- >1.5 to 3.0 m
- >3.0 to 4.5 m
- >4.5 to 6.0 m
- >6.0 to 7.5 m
- >7.5 to 9.0 m
- >9.0 to 10.5 m
- >10.5 to 12.0 m
- >12.0 to 13.5 m
- >13.5 to 15.0 m
- >15.0 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)

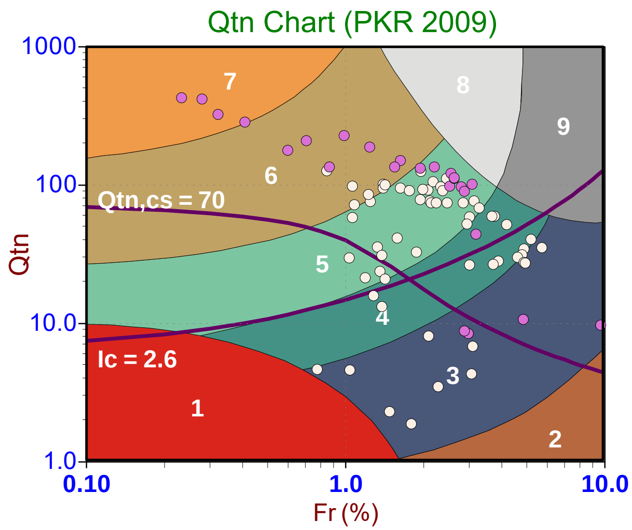
Legend
- Sensitive Fines
- Organic Soil
- Clay
- Silty Clay
- Clayey Silt
- Silt
- Sandy Silt
- Silty Sand/Sand
- Sand
- Gravelly Sand
- Stiff Fine Grained
- Cemented Sand

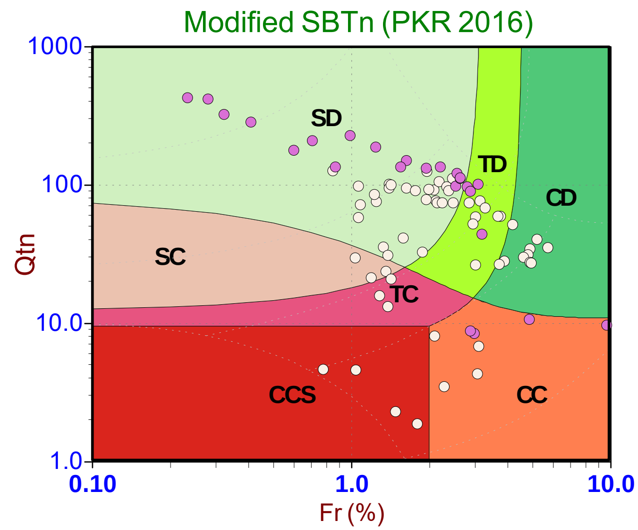
CONETEC

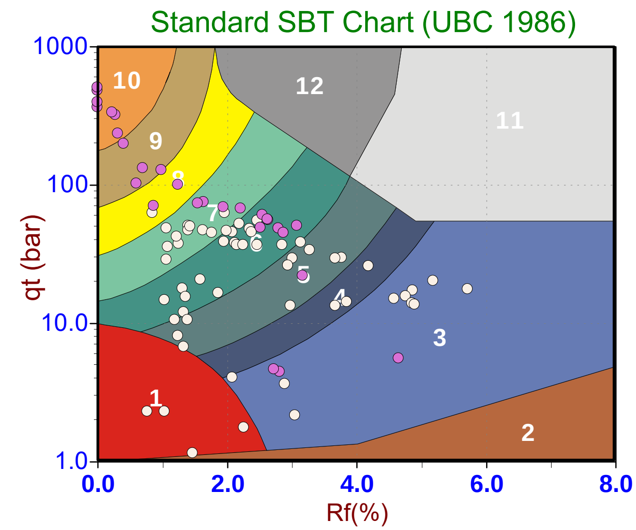
WSP Canada

Job No: 23-05-25254
Date: 2023-02-27 16:03
Site: 3630 and 3690 Riverside Drive

Sounding: CPT23-04
Cone: 947:T1000F10U35







Depth Ranges
- >0.0 to 1.5 m
- >1.5 to 3.0 m
- >3.0 to 4.5 m
- >4.5 to 6.0 m
- >6.0 to 7.5 m
- >7.5 to 9.0 m
- >9.0 to 10.5 m
- >10.5 to 12.0 m
- >12.0 to 13.5 m
- >13.5 to 15.0 m
- >15.0 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)
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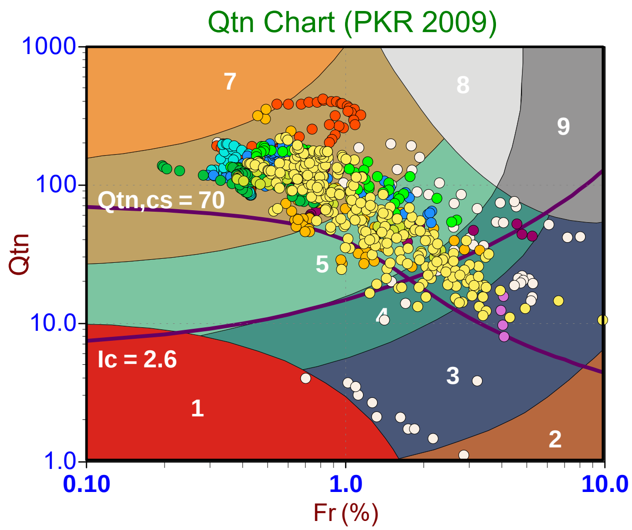
Legend
- Sensitive Fines
- Organic Soil
- Clay
- Silty Clay
- Clayey Silt
- Silt
- Sandy Silt
- Silty Sand/Sand
- Sand
- Gravelly Sand
- Stiff Fine Grained
- Cemented Sand

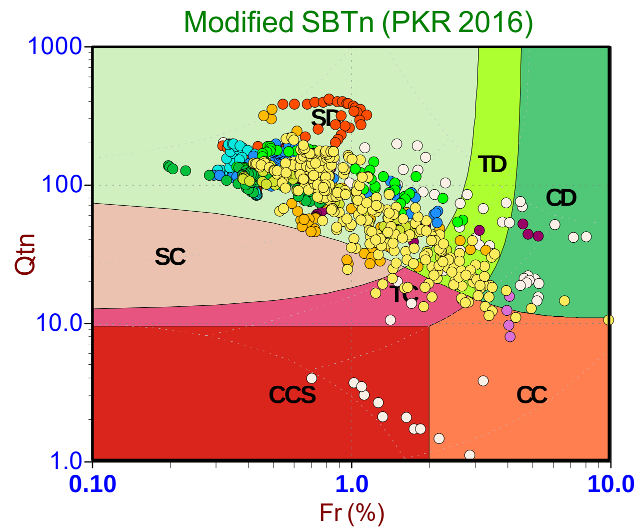
CONETEC

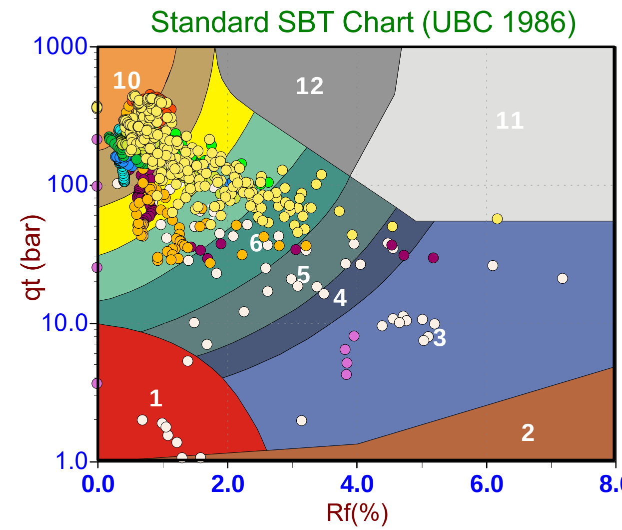
WSP Canada

Job No: 23-05-25254
Date: 2023-02-28 10:33
Site: 3630 and 3690 Riverside Drive

Sounding: CPT23-04B
Cone: 947:T1000F10U35







Depth Ranges
- >0.0 to 1.5 m
- >1.5 to 3.0 m
- >3.0 to 4.5 m
- >4.5 to 6.0 m
- >6.0 to 7.5 m
- >7.5 to 9.0 m
- >9.0 to 10.5 m
- >10.5 to 12.0 m
- >12.0 to 13.5 m
- >13.5 to 15.0 m
- >15.0 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: 23-05-25254
Client: WSP Canada Inc.
Project: Riverside Dr and Hunt Club Rd
Start Date: 27-Feb-2023
End Date: 02-Mar-2023

CPTu PORE PRESSURE DISSIPATION SUMMARY

Sounding ID	File Name	Cone Area (cm^2)	Duration (s)	Test Depth (m)	$U_{initial}$ (m)	U_{max} (m)	U_{min} (m)	U_{final} (m)	Equilibrium Pore Pressure U_{eq} (m)	Estimated Equilibrium Pore Pressure U_{eq} (m)	Calculated Phreatic Surface (m)	Percent Dissipation (%)	t_{50} (s)[1]	Assumed Rigidity Index (I_r)	c_h (cm^2/min)[2]	Refer to Notation Number
SCPT23-01B	23-05-25254_SP01B	15	1520	2.925	-3.6	22.5	-7.0	1.3								
SCPT23-01C	23-05-25254_SP01C	15	1830	11.325	2.2	2.2	-4.1	0.4	0.4		10.9					
SCPT23-01D	23-05-25254_SP01D	15	1950	11.425	0.9	0.9	-2.3	0.5	0.5		10.9					
CPT23-02B	23-05-25254_CP02B	15	1360	12.175	1.4	32.6	0.4	0.4	0.4		11.8	100				
CPT23-02C	23-05-25254_CP02C	15	1800	12.725	3.8	7.9	0.9	0.9	0.9		11.8	100	22	100	32.0	
CPT23-02C	23-05-25254_CP02C	15	1800	12.900	5.8	20.2	1.2	5.0	4.9		8.0	99	4	100	184.6	
SCPT23-03	23-05-25254_SP03	15	80	7.925	3.5	5.0	3.1	3.6	3.4		4.5					
SCPT23-03	23-05-25254_SP03	15	460	11.925	-4.0	3.1	-5.9	2.8	2.8		9.1					
SCPT23-03	23-05-25254_SP03	15	1800	12.350	1.2	2.3	-5.8	0.3	0.3		12.0	100	82	100	8.6	
SCPT23-03B	23-05-25254_SP03B	15	56	6.800	-3.8	1.9	-4.1	1.9	2.0		4.8					
SCPT23-03B	23-05-25254_SP03B	15	56	8.800	4.7	5.0	3.0	3.3	3.2		5.6	97	6	100	115.5	
SCPT23-03B	23-05-25254_SP03B	15	1790	11.800	3.5	8.7	1.4	1.5	1.5		10.3	100	34	100	20.9	
CPT23-04B	23-05-25254_CP04B	15	1800	12.700	2.6	5.4	1.3	5.3	5.3		7.4					
CPT23-04B	23-05-25254_CP04B	15	1800	22.325	6.7	6.7	-1.3	6.0	6.0		16.3					

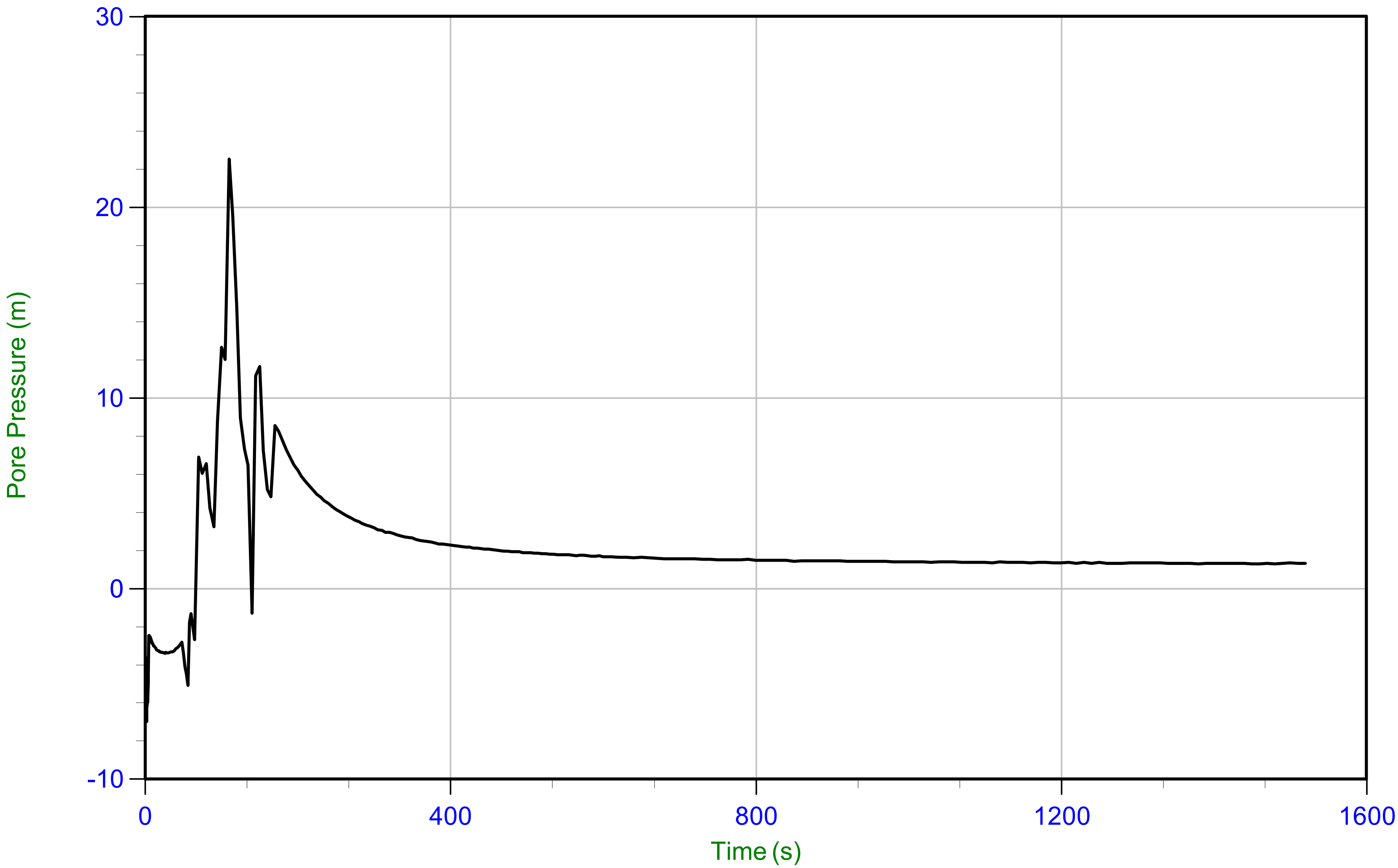
1. Time for 50 percent dissipation based on U_{max}, U_{min}, and the applied U_{eq}. Note the time is relative to where U_{max} occurred.
2. Houlsby and Teh, 1991.

ConeTec

WSP Canada

Job No: 23-05-25254
Date: 03/01/2023 16:44
Site: 3630 and 3690 Riverside Drive

Sounding: SCPT23-01B
Cone: 947:T1000F10U35 Area=15 cm²



Trace Summary:

Filename: 23-05-25254_SP01B.PPF2
Depth: 2.925 m / 9.596 ft
Duration: 1520.0 s

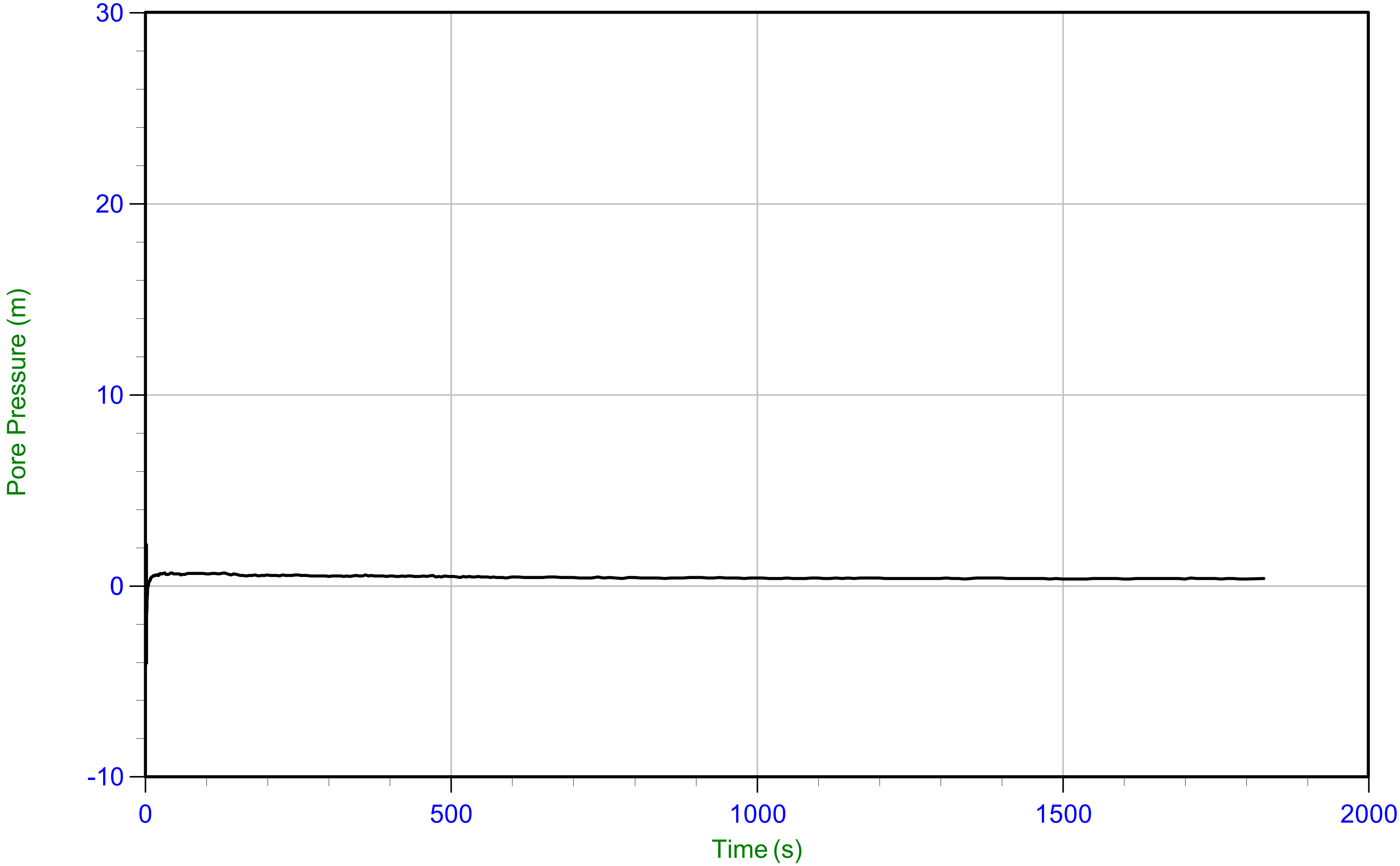
u Min: -7.0 m
u Max: 22.5 m
u Final: 1.3 m

ConeTec

WSP Canada

Job No: 23-05-25254
Date: 03/02/2023 07:48
Site: 3630 and 3690 Riverside Drive

Sounding: SCPT23-01C
Cone: 947:T1000F10U35 Area=15 cm²



Trace Summary:

Filename: 23-05-25254_SP01C.PPF2
Depth: 11.325 m / 37.155 ft
Duration: 1830.0 s

u Min: -4.1 m
u Max: 2.2 m
u Final: 0.4 m

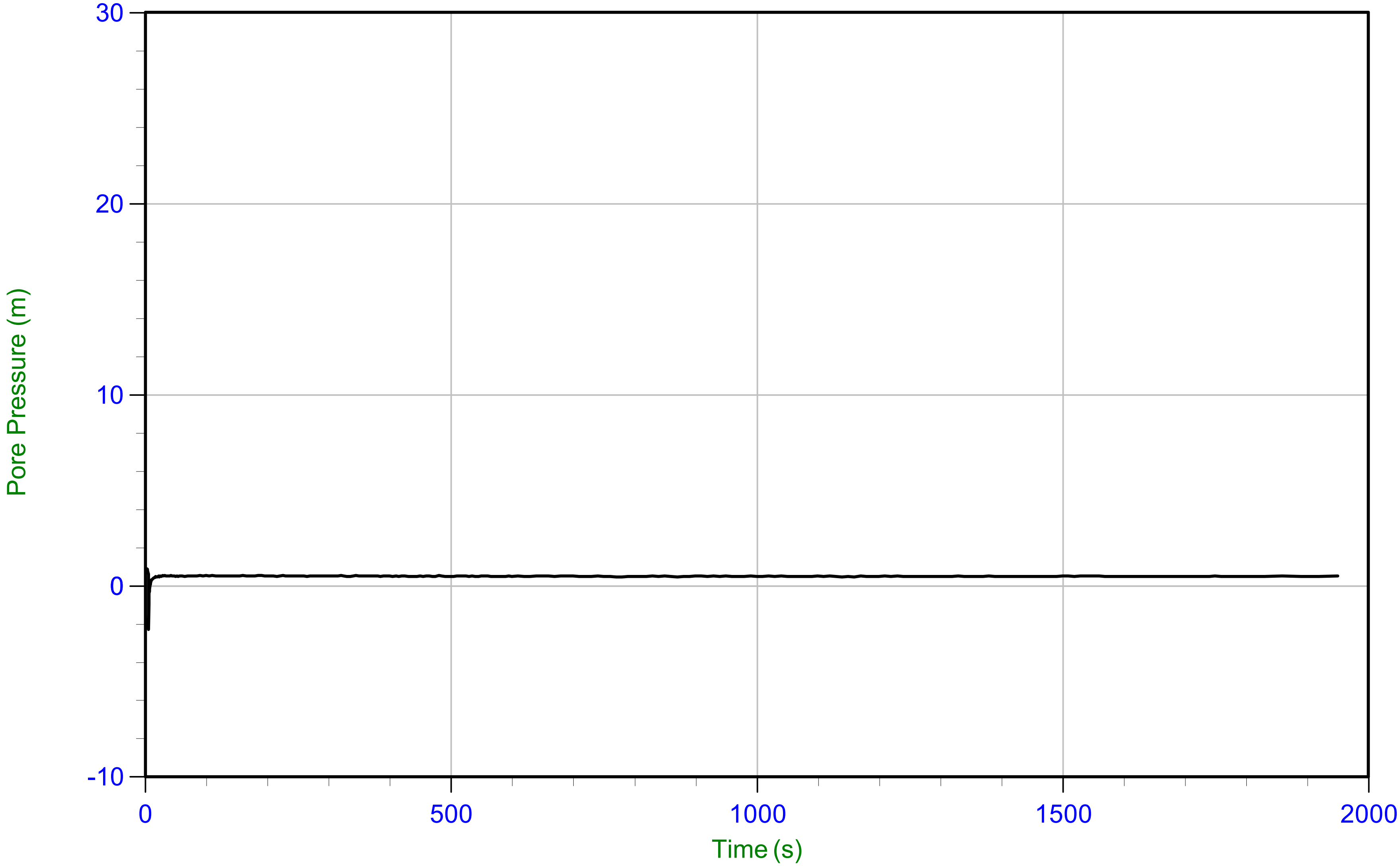
WT: 10.943 m / 35.902 ft
Ueq: 0.4 m

CONETEC

WSP Canada

Job No: 23-05-25254
Date: 03/02/2023 09:43
Site: 3630 and 3690 Riverside Drive

Sounding: SCPT23-01D
Cone: 947:T1000F10U35 Area=15 cm²



Trace Summary:

Filename: 23-05-25254_SP01D.PPF2
Depth: 11.425 m / 37.483 ft
Duration: 1950.0 s

u Min: -2.3 m
u Max: 0.9 m
u Final: 0.5 m

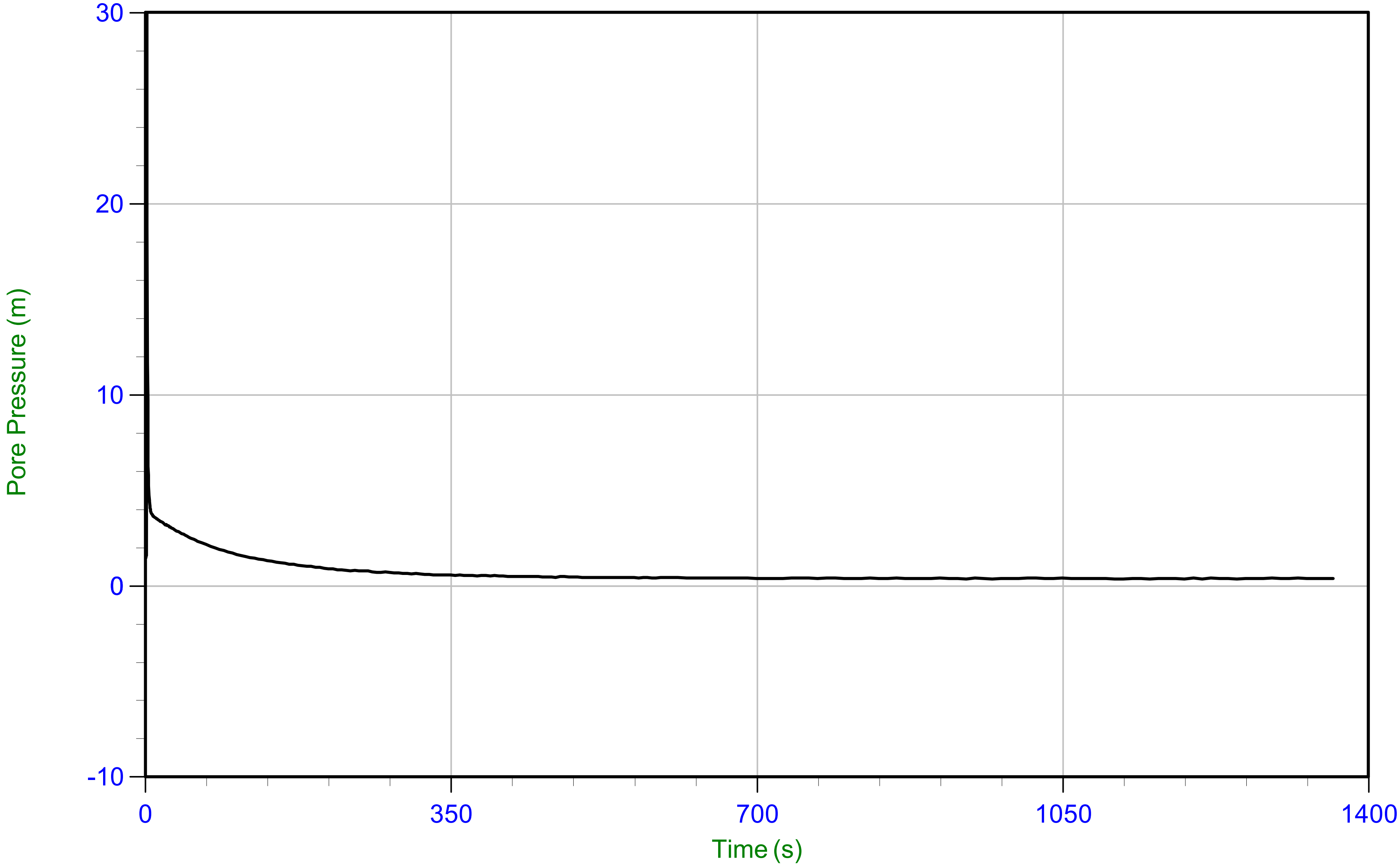
WT: 10.925 m / 35.843 ft
Ueq: 0.5 m

CONETEC

WSP Canada

Job No: 23-05-25254
Date: 02/28/2023 15:47
Site: 3630 and 3690 Riverside Drive

Sounding: CPT23-02B
Cone: 947:T1000F10U35 Area=15 cm²



Trace Summary:

Filename: 23-05-25254_CP02B.PPF2
Depth: 12.175 m / 39.944 ft
Duration: 1360.0 s

u Min: 0.4 m
u Max: 32.6 m
u Final: 0.4 m

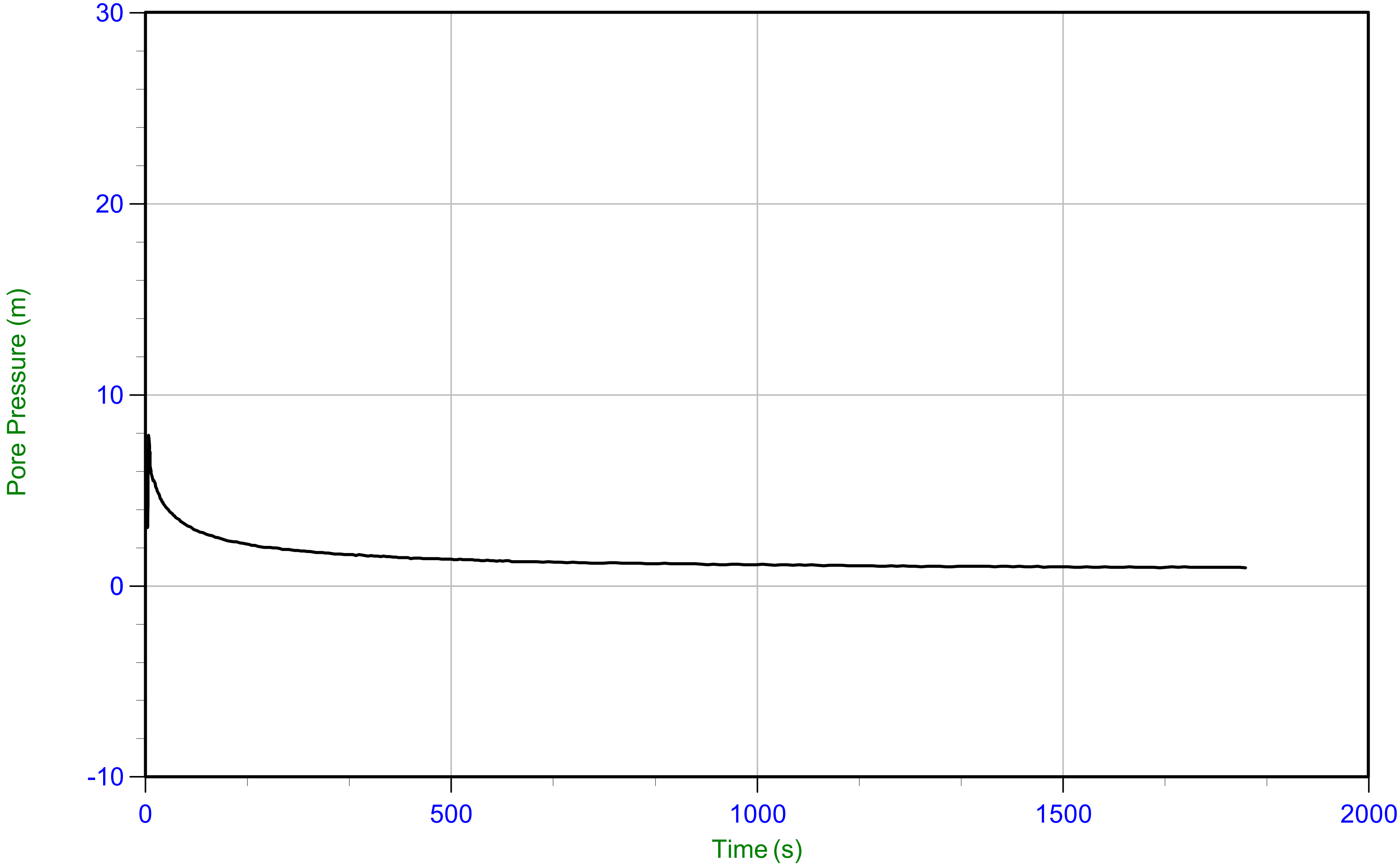
WT: 11.754 m / 38.563 ft
Ueq: 0.4 m

ConeTec

WSP Canada

Job No: 23-05-25254
Date: 03/01/2023 06:59
Site: 3630 and 3690 Riverside Drive

Sounding: CPT23-02C
Cone: 947:T1000F10U35 Area=15 cm²



Trace Summary:

Filename: 23-05-25254_CP02C.PPF2
Depth: 12.725 m / 41.748 ft
Duration: 1800.0 s

u Min: 0.9 m
u Max: 7.9 m
u Final: 0.9 m

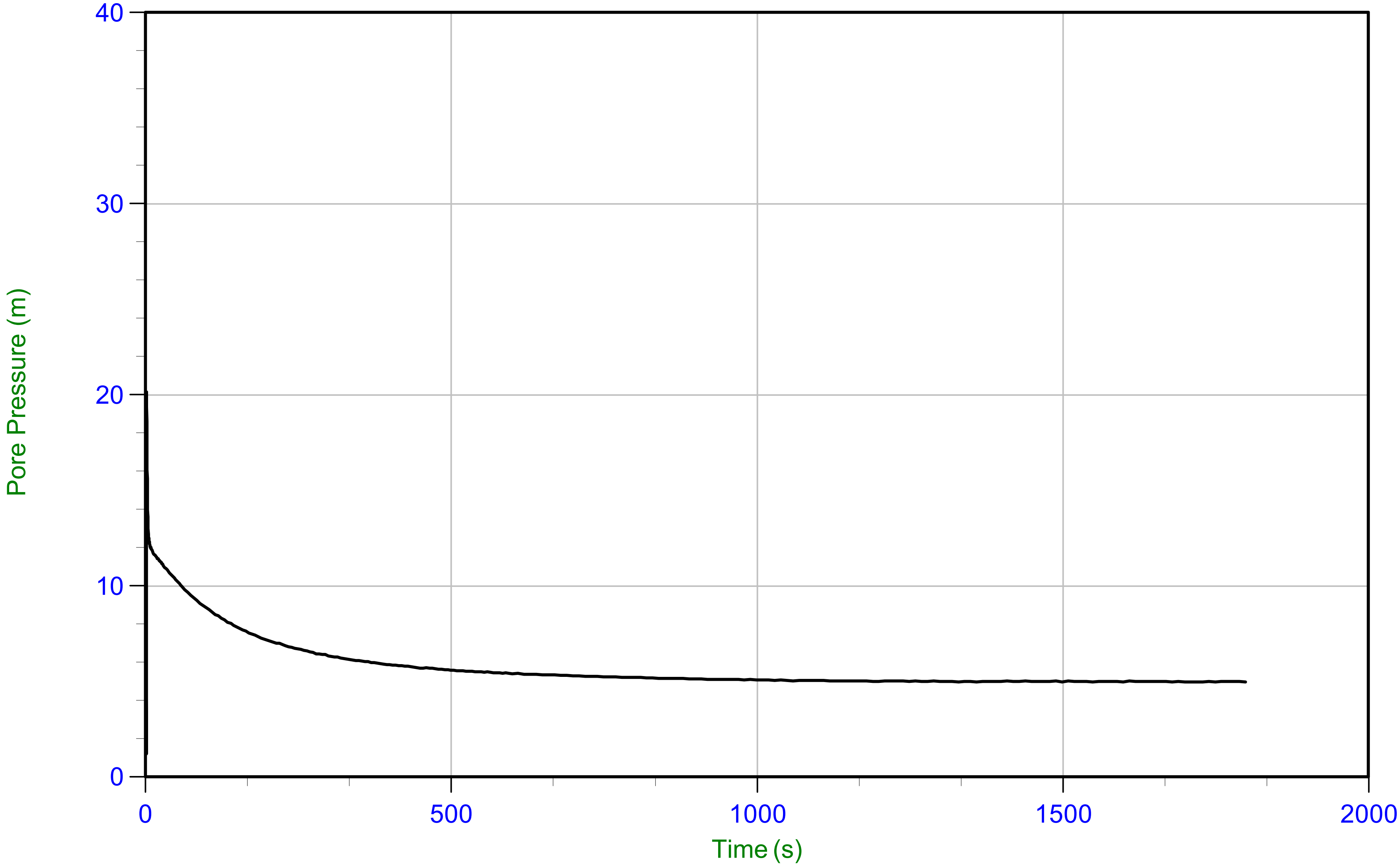
WT: 11.778 m / 38.641 ft
Ueq: 0.9 m
U(50): 4.41 m

T(50): 21.9 s
Ir: 100
Ch: 32.0 cm²/min

WSP Canada

Job No: 23-05-25254
Date: 03/01/2023 06:59
Site: 3630 and 3690 Riverside Drive

Sounding: CPT23-02C
Cone: 947:T1000F10U35 Area=15 cm²



Trace Summary:

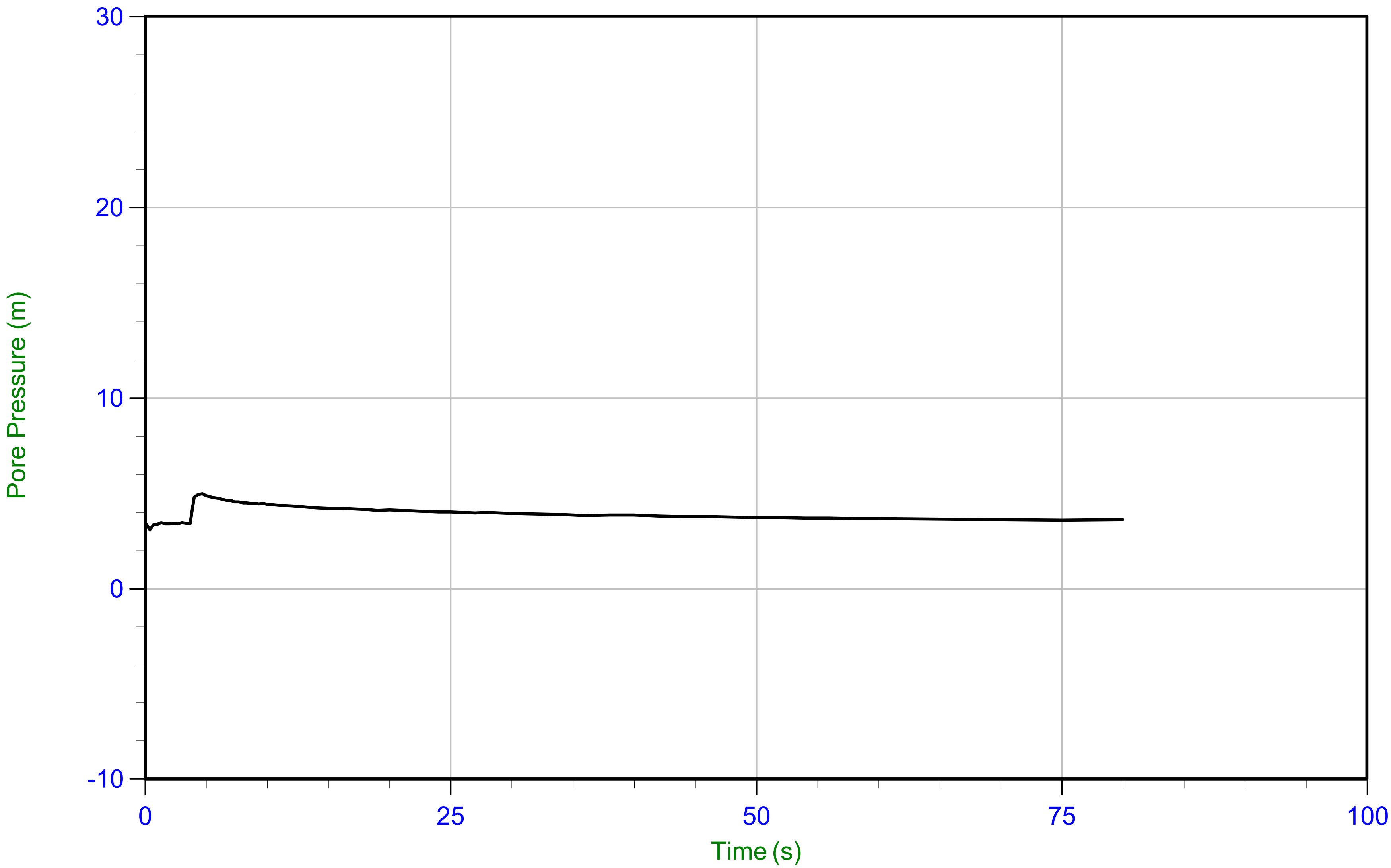
Filename: 23-05-25254_CP02C.PPF2
Depth: 12.900 m / 42.322 ft
Duration: 1800.0 s

u Min: 1.2 m
u Max: 20.2 m
u Final: 5.0 m

WT: 8.000 m / 26.246 ft
Ueq: 4.9 m
U(50): 12.53 m

T(50): 3.8 s
Ir: 100
Ch: 184.6 cm²/min

WSP Canada	Job No: 23-05-25254 Date: 03/01/2023 09:49 Site: 3630 and 3690 Riverside Drive	Sounding: SCPT23-03 Cone: 947:T1000F10U35 Area=15 cm²



Trace Summary:

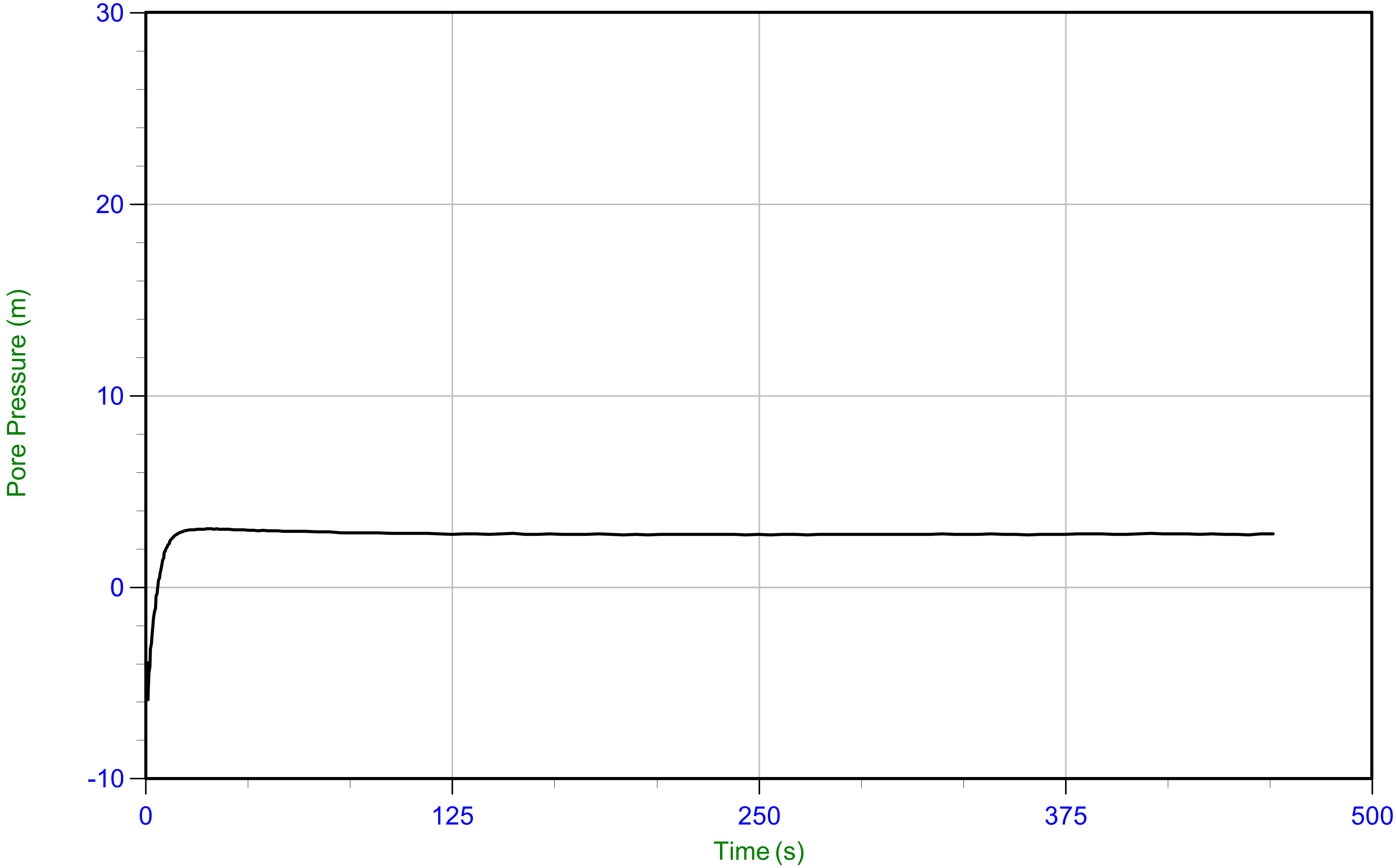
Filename: 23-05-25254_SP03.PPF2	u Min: 3.1 m	WT: 4.525 m / 14.846 ft
Depth: 7.925 m / 26.000 ft	u Max: 5.0 m	Ueq: 3.4 m
Duration: 80.0 s	u Final: 3.6 m	

ConeTec

WSP Canada

Job No: 23-05-25254
Date: 03/01/2023 09:49
Site: 3630 and 3690 Riverside Drive

Sounding: SCPT23-03
Cone: 947:T1000F10U35 Area=15 cm²



Trace Summary:

Filename: 23-05-25254_SP03.PPF2
Depth: 11.925 m / 39.124 ft
Duration: 460.0 s

u Min: -5.9 m
u Max: 3.1 m
u Final: 2.8 m

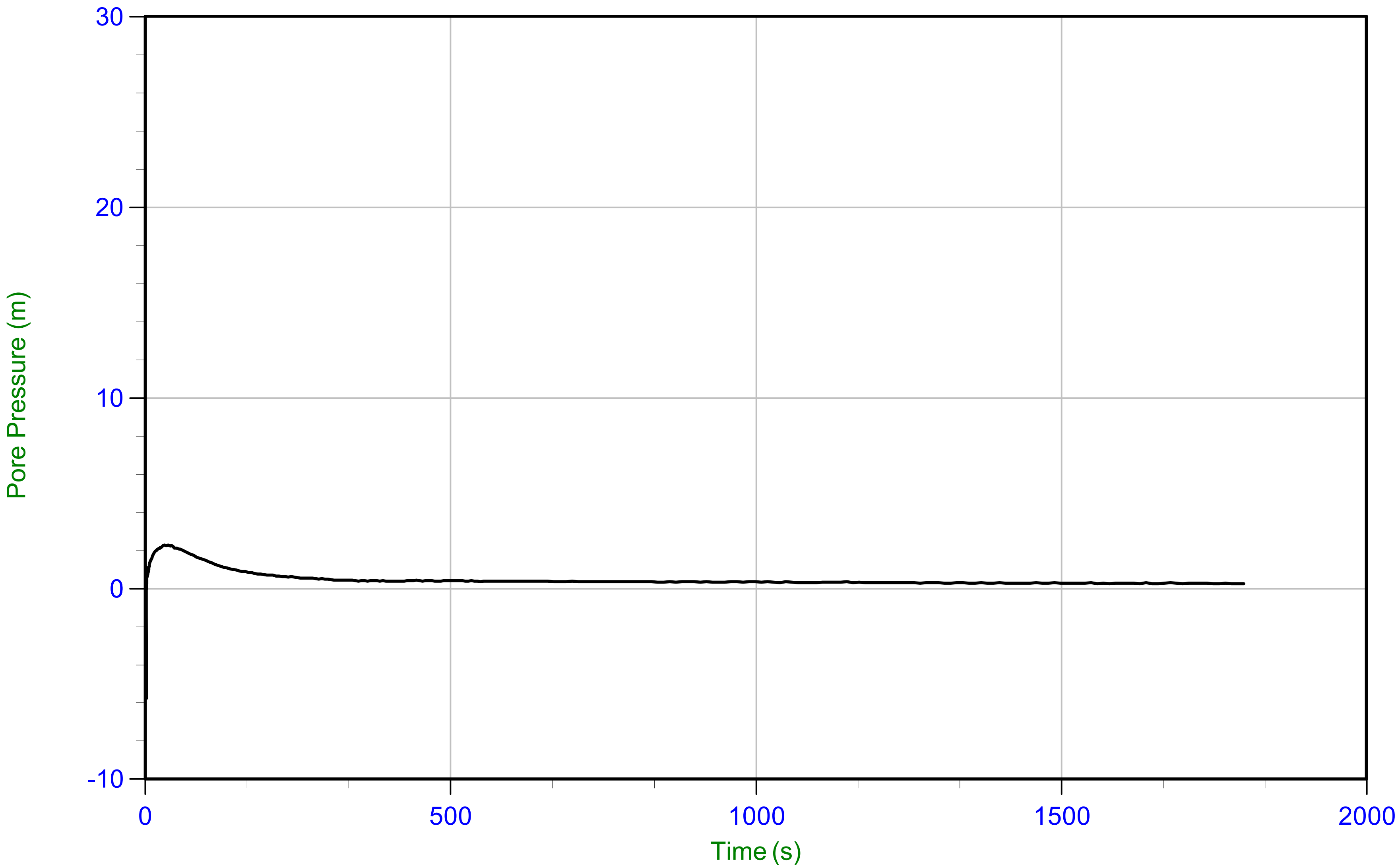
WT: 9.083 m / 29.800 ft
Ueq: 2.8 m

ConeTec

WSP Canada

Job No: 23-05-25254
Date: 03/01/2023 09:49
Site: 3630 and 3690 Riverside Drive

Sounding: SCPT23-03
Cone: 947:T1000F10U35 Area=15 cm²



Trace Summary:

Filename: 23-05-25254_SP03.PPF2
Depth: 12.350 m / 40.518 ft
Duration: 1800.0 s

u Min: -5.8 m
u Max: 2.3 m
u Final: 0.3 m

WT: 12.034 m / 39.481 ft
Ueq: 0.3 m
U(50): 1.29 m

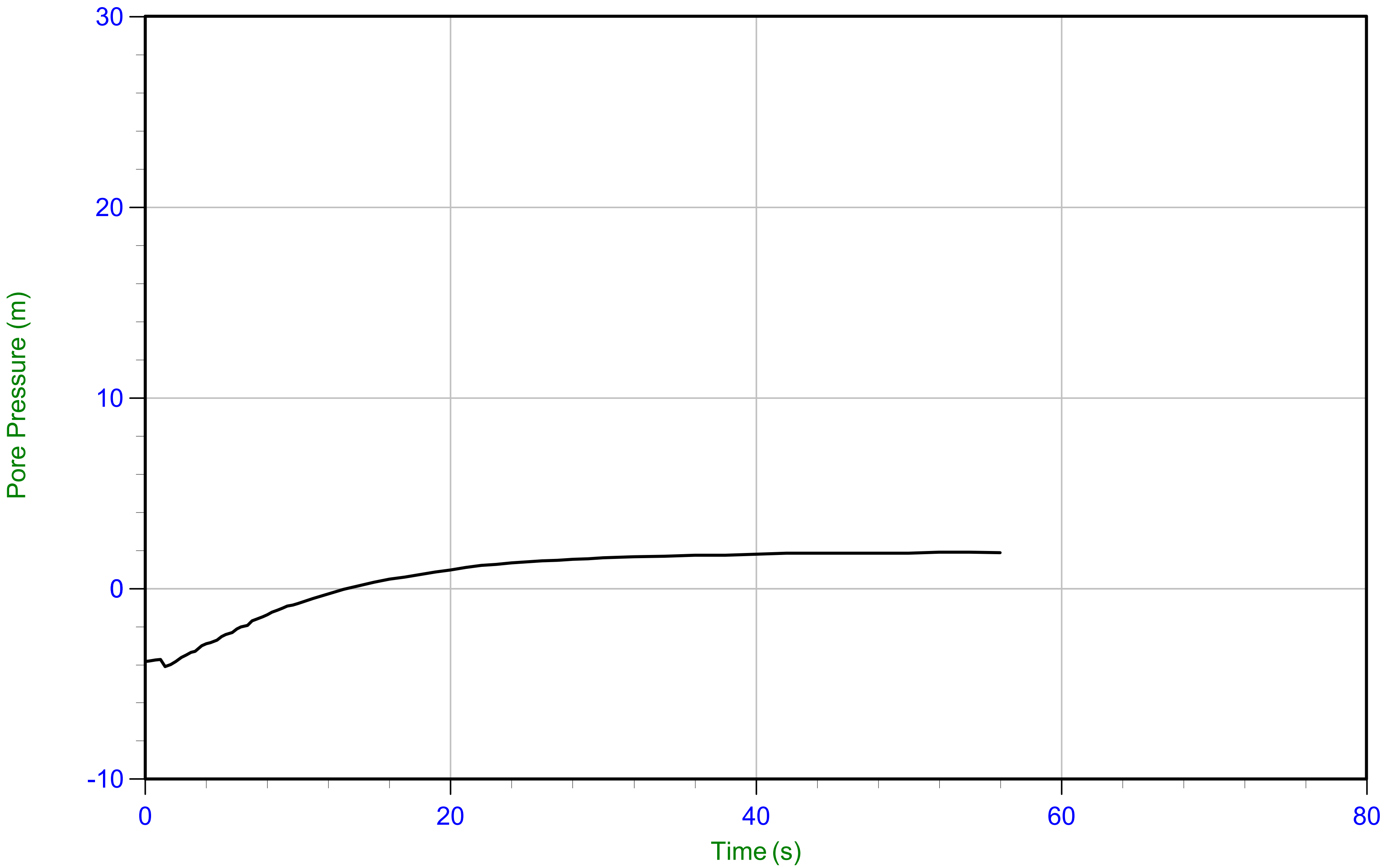
T(50): 81.6 s
Ir: 100
Ch: 8.6 cm²/min

CONETEC

WSP Canada

Job No: 23-05-25254
Date: 03/01/2023 12:55
Site: 3630 and 3690 Riverside Drive

Sounding: SCPT23-03B
Cone: 947:T1000F10U35 Area=15 cm²



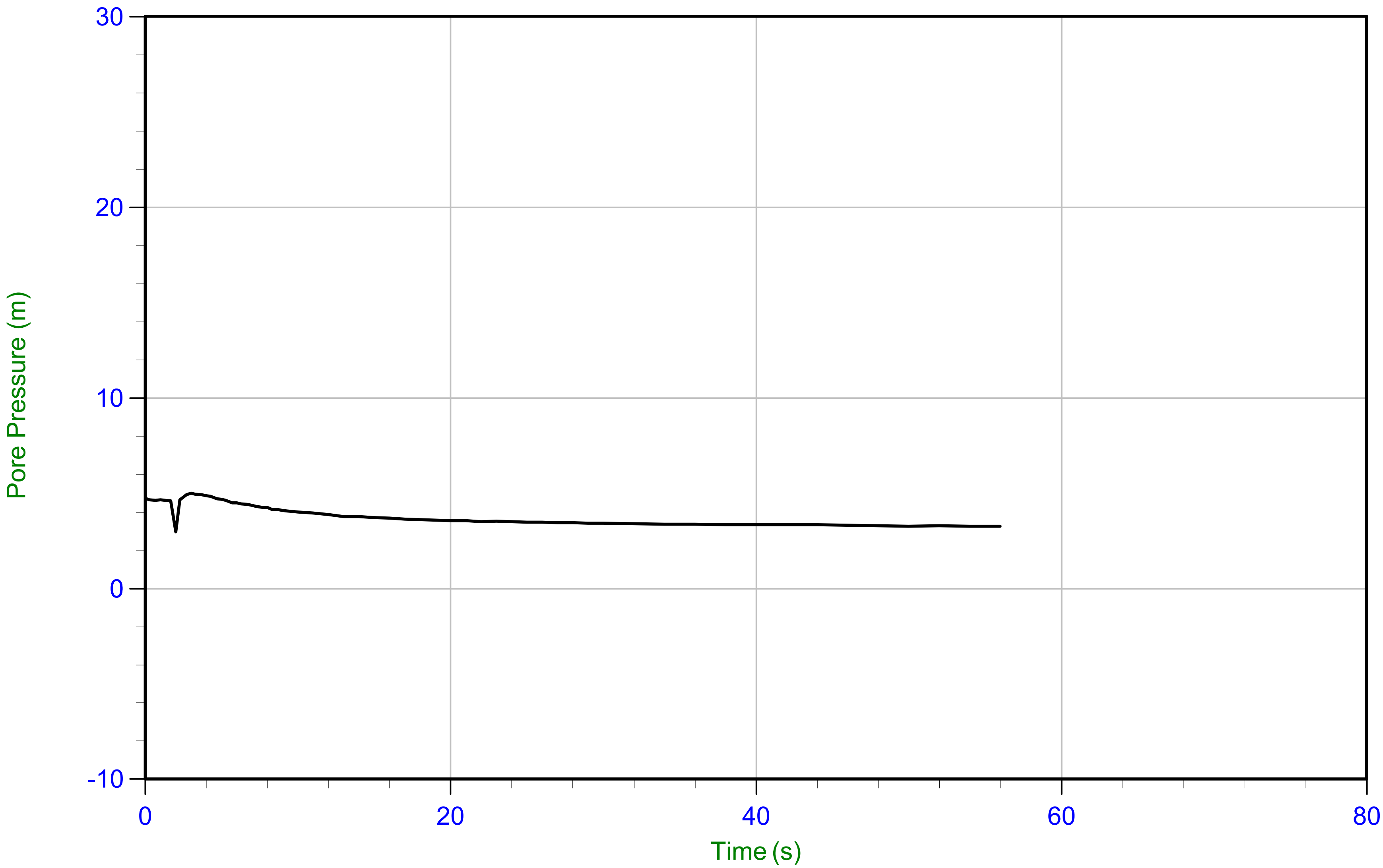
Trace Summary:

Filename: 23-05-25254_SP03B.PPF2
Depth: 6.800 m / 22.309 ft
Duration: 56.0 s

u Min: -4.1 m
u Max: 1.9 m
u Final: 1.9 m

WT: 4.800 m / 15.748 ft
Ueq: 2.0 m

WSP Canada	Job No: 23-05-25254 Date: 03/01/2023 12:55 Site: 3630 and 3690 Riverside Drive	Sounding: SCPT23-03B Cone: 947:T1000F10U35 Area=15 cm²



Trace Summary:

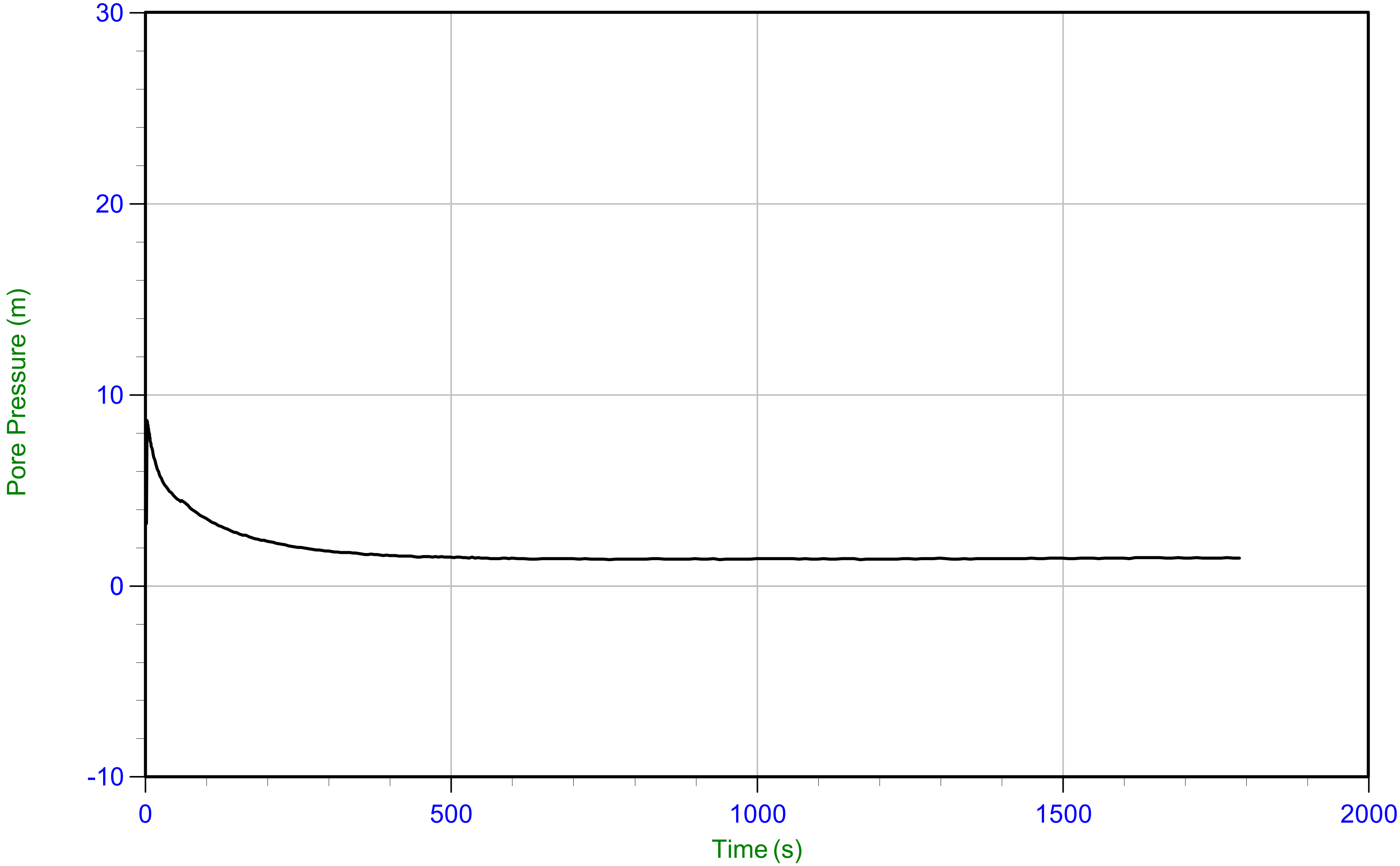
Filename: 23-05-25254_SP03B.PPF2	u Min: 3.0 m	WT: 5.600 m / 18.372 ft	T(50): 6.1 s
Depth: 8.800 m / 28.871 ft	u Max: 5.0 m	Ueq: 3.2 m	Ir: 100
Duration: 56.0 s	u Final: 3.3 m	U(50): 4.09 m	Ch: 115.5 cm²/min

ConeTec

WSP Canada

Job No: 23-05-25254
Date: 03/01/2023 12:55
Site: 3630 and 3690 Riverside Drive

Sounding: SCPT23-03B
Cone: 947:T1000F10U35 Area=15 cm²



Trace Summary:

Filename: 23-05-25254_SP03B.PPF2
Depth: 11.800 m / 38.713 ft
Duration: 1790.0 s

u Min: 1.4 m
u Max: 8.7 m
u Final: 1.5 m

WT: 10.313 m / 33.835 ft
Ueq: 1.5 m
U(50): 5.08 m

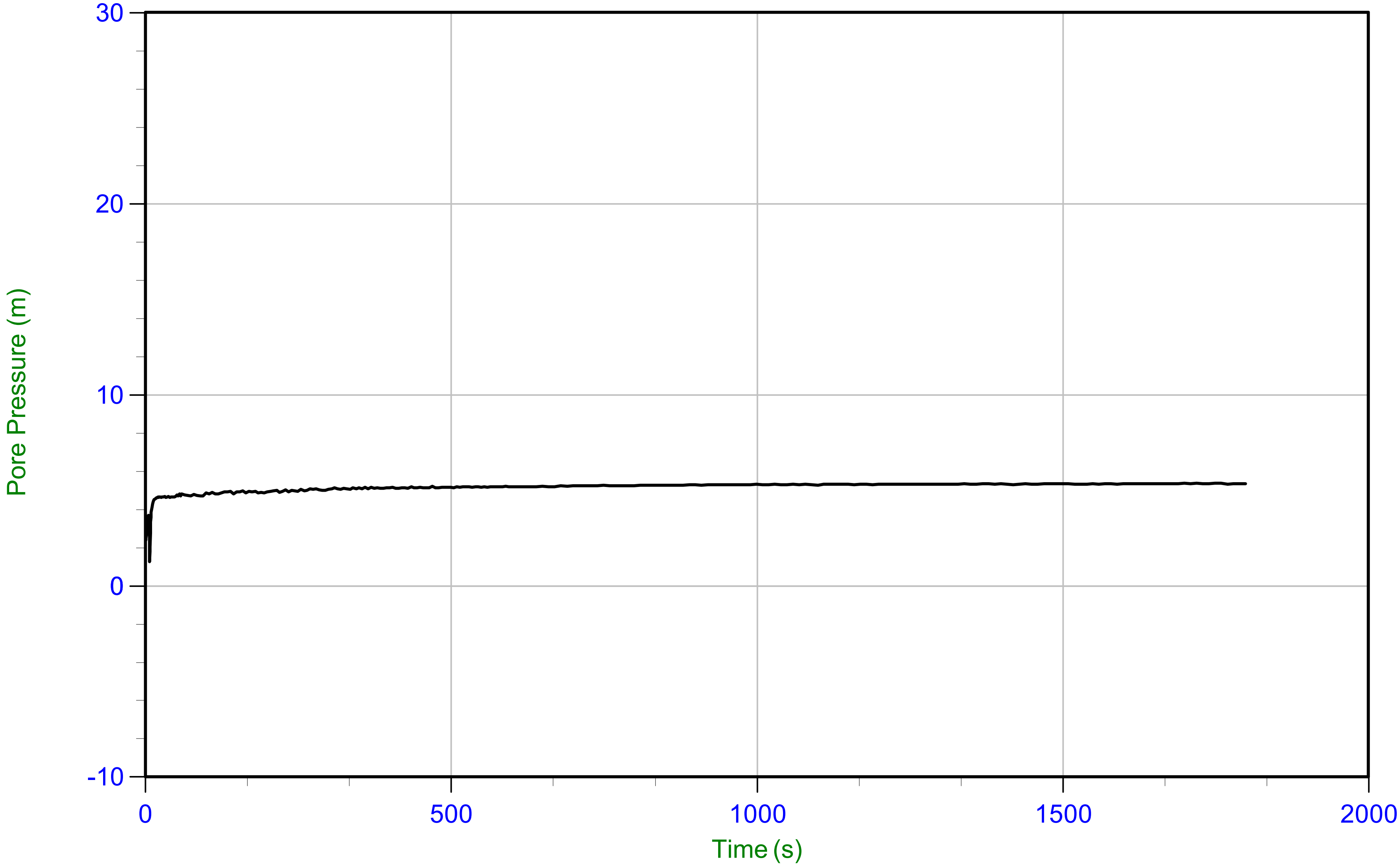
T(50): 33.6 s
Ir: 100
Ch: 20.9 cm²/min

ConeTec

WSP Canada

Job No: 23-05-25254
Date: 02/28/2023 10:33
Site: 3630 and 3690 Riverside Drive

Sounding: CPT23-04B
Cone: 947:T1000F10U35 Area=15 cm²



Trace Summary:

Filename: 23-05-25254_CP04B.PPF2
Depth: 12.700 m / 41.666 ft
Duration: 1800.0 s

u Min: 1.3 m
u Max: 5.4 m
u Final: 5.3 m

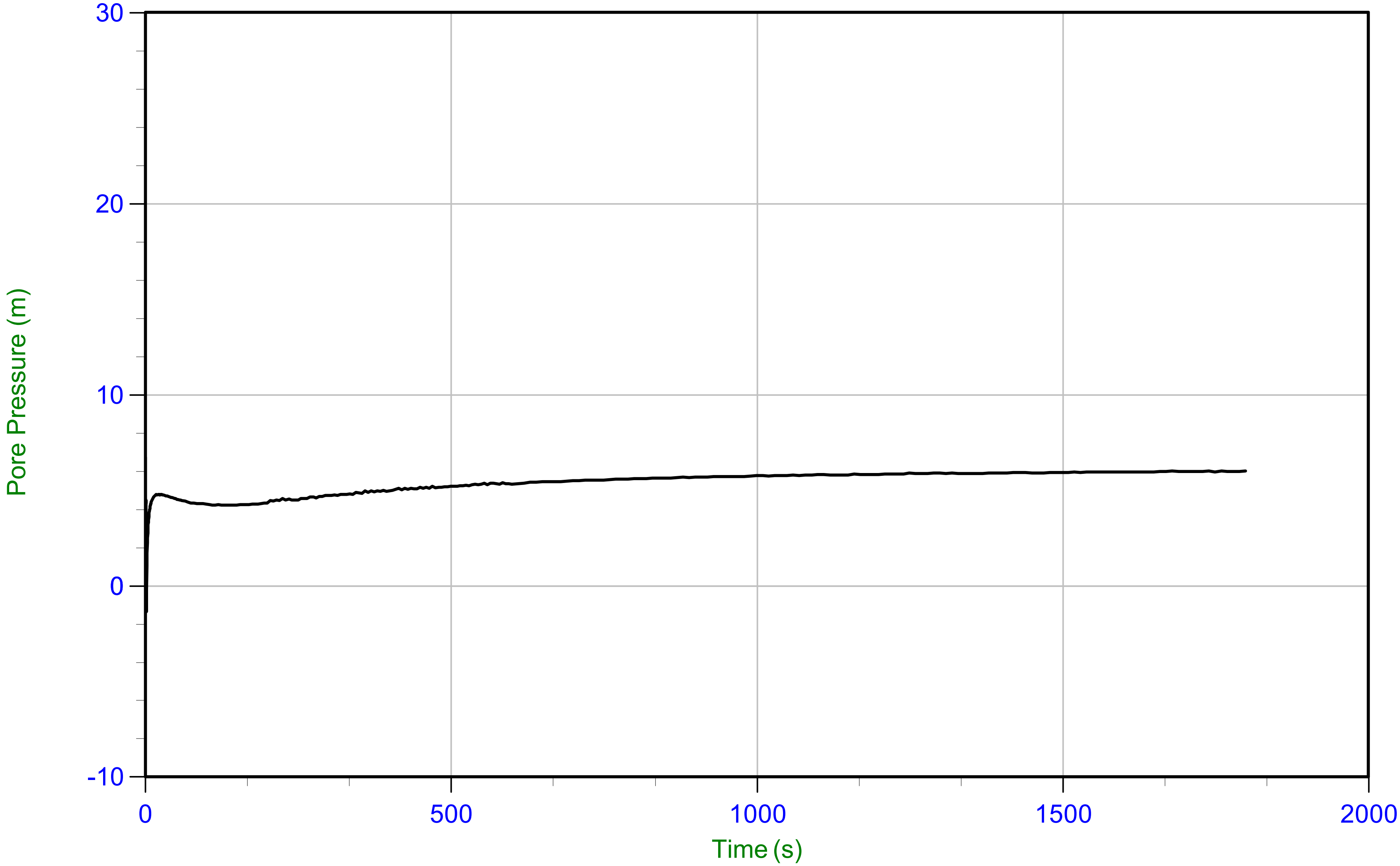
WT: 7.400 m / 24.278 ft
Ueq: 5.3 m

CONETEC

WSP Canada

Job No: 23-05-25254
Date: 02/28/2023 10:33
Site: 3630 and 3690 Riverside Drive

Sounding: CPT23-04B
Cone: 947:T1000F10U35 Area=15 cm²



Trace Summary:

Filename: 23-05-25254_CP04B.PPF2
Depth: 22.325 m / 73.244 ft
Duration: 1800.0 s

u Min: -1.3 m
u Max: 6.7 m
u Final: 6.0 m

WT: 16.325 m / 53.559 ft
Ueq: 6.0 m

Description of Methods for Calculated CPT Geotechnical Parameters

CALCULATED CPT GEOTECHNICAL PARAMETERS

A Detailed Description of the Methods Used in ConeTec's CPT Geotechnical Parameter Calculation and Plotting Software

Revision SZW-Rev 14

Revised November 26, 2019

Prepared by Jim Greig, M.A.Sc, P.Eng (BC)

ConeTec

Limitations

The geotechnical parameter output was prepared specifically for the site and project named in the accompanying report subject to objectives, site conditions and criteria provided to ConeTec by the client. The output may not be relied upon by any other party or for any other site without the express written permission of ConeTec Group (ConeTec) or any of its affiliates. For this project, ConeTec has provided site investigation services, prepared factual data reporting and produced geotechnical parameter calculations consistent with current best practices. No other warranty, expressed or implied, is made.

To understand the calculations that have been performed and to be able to reproduce the calculated parameters the user is directed to the basic descriptions for the methods in this document and the detailed descriptions and their associated limitations and appropriateness in the technical references cited for each parameter.

ConeTec's Calculated CPT Geotechnical Parameters as of November 26, 2019

ConeTec's CPT parameter calculation and plotting routine provides a tabular output of geotechnical parameters based on current published CPT correlations and is subject to change to reflect the current state of practice. Due to drainage conditions and the basic assumptions and limitations of the correlations, not all geotechnical parameters provided are considered applicable for all soil types. The results are presented only as a guide for geotechnical use and should be carefully examined for consideration in any geotechnical design. Reference to current literature is strongly recommended. ConeTec does not warranty the correctness or the applicability of any of the geotechnical parameters calculated by the program and does not assume liability for any use of the results in any design or review. For verification purposes we recommend that representative hand calculations be done for any parameter that is critical for design purposes. The end user of the parameter output should also be fully aware of the techniques and the limitations of any method used by the program. The purpose of this document is to inform the user as to which methods were used and to direct the end user to the appropriate technical papers and/or publications for further reference.

The geotechnical parameter output was prepared specifically for the site and project named in the accompanying report subject to objectives, site conditions and criteria provided to ConeTec by the client. The output may not be relied upon by any other party or for any other site without the express written permission of ConeTec Group (ConeTec) or any of its affiliates.

The CPT calculations are based on values of tip resistance, sleeve friction and pore pressures considered at each data point or averaged over a user specified layer thickness (e.g. 0.20 m). Note that q_t is the tip resistance corrected for pore pressure effects and q_c is the recorded tip resistance. The corrected tip resistance (corrected using u_2 pore pressure values) is used for all of the calculations. Since all ConeTec cones have equal end area friction sleeves pore pressure corrections to sleeve friction, f_s, are not required.

The tip correction is: $q_t = q_c + (1\text{-}a) \bullet u_2$ (consistent units are implied)

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 cone (typically 0.80 for ConeTec cones)

The total stress calculations are based on soil unit weight values that have been assigned to the Soil Behavior Type (SBT) zones, from a user defined unit weight profile, by using a single uniform value throughout the profile, through unit weight estimation techniques described in various technical papers or from a combination of these methods. The parameter output files indicate the method(s) used.

Effective vertical overburden stresses are calculated based on a hydrostatic distribution of equilibrium pore pressures below the water table or from a user defined equilibrium pore pressure profile (typically obtained from CPT dissipation tests) or a combination of the two. For over water projects the stress effects of the column of water above the mudline have been taken into account as has the appropriate unit weight of water. How this is done depends on where the instruments were zeroed (i.e. on deck or at the mudline). The parameter output files indicate the method(s) used.

A majority of parameter calculations are derived or driven by results based on material types as determined by the various soil behavior type charts depicted in Figures 1 through 5. The parameter output files indicate the method(s) used.

The Soil Behavior Type classification chart shown in Figure 1 is the classic non-normalized SBT Chart developed at the University of British Columbia and reported in Robertson, Campanella, Gillespie and Greig (1986). Figure 2 shows the original normalized (linear method) SBT chart developed by Robertson (1990). The Bq classification charts shown in Figures 3a and 3b incorporate pore pressures into the SBT classification and are based on the methods described in Robertson (1990). Many of these charts have been summarized in Lunne, Robertson and Powell (1997). The

Jefferies and Davies SBT chart shown in Figure 3c is based on the techniques discussed in Jefferies and Davies (1993) which introduced the concept of the Soil Behavior Type Index parameter, I_c. Please note that the I_c parameter developed by Robertson and Fear (1995) and Robertson and Wride (1998) is similar in concept but uses a slightly different calculation method than that used by Jefferies and Davies (1993) as the latter incorporates pore pressure in their technique through the use of the Bq parameter. The normalized Qtn SBT chart shown in Figure 4 is based on the work by Robertson (2009) utilizing a variable stress ratio exponent, n, for normalization based on a slightly modified redefinition and iterative approach for I_c. The boundary curves drawn on the chart are based on the work described in Robertson (2010).

Figure 5 shows a revised behavior based chart by Robertson (2016) depicting contractive-dilative zones. As the zones represent material behavior rather than soil gradation ConeTec has chosen a set of zone colors that are less likely to be confused with material type colors from previous SBT charts. These colors differ from those used by Dr. Robertson.

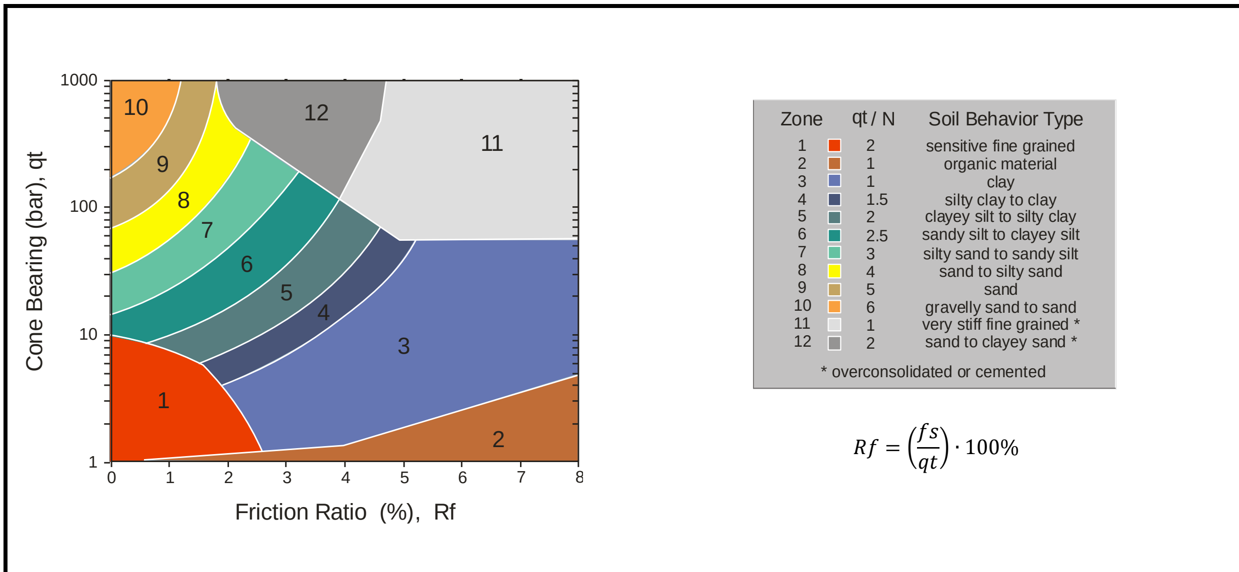


Figure 1. Non-Normalized Soil Behavior Type Classification Chart (SBT)

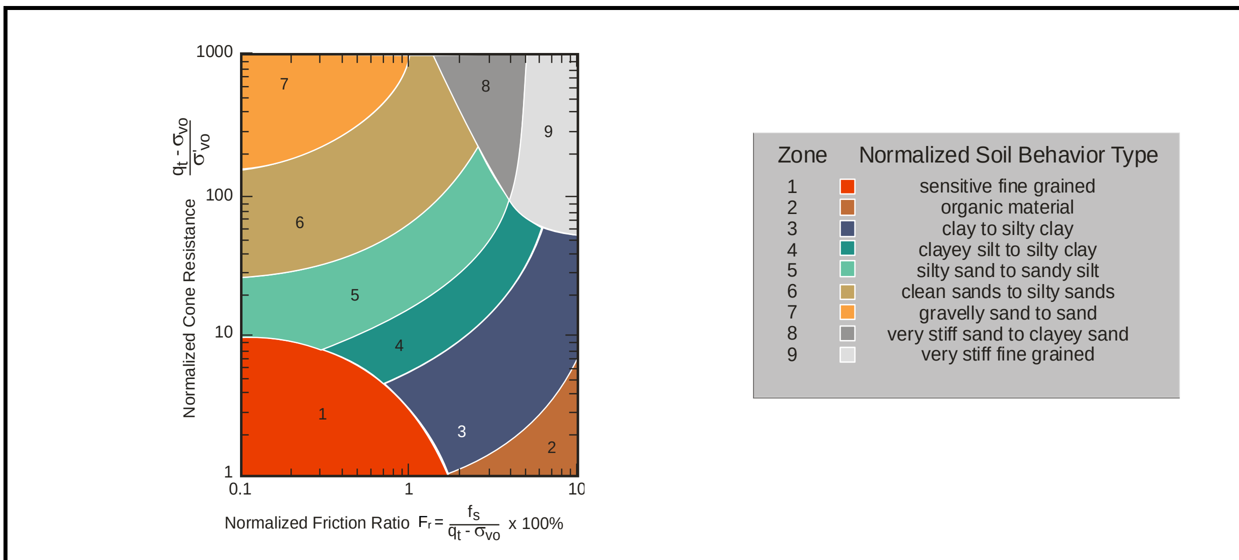


Figure 2. Normalized Soil Behavior Type Classification Chart (SBTn)

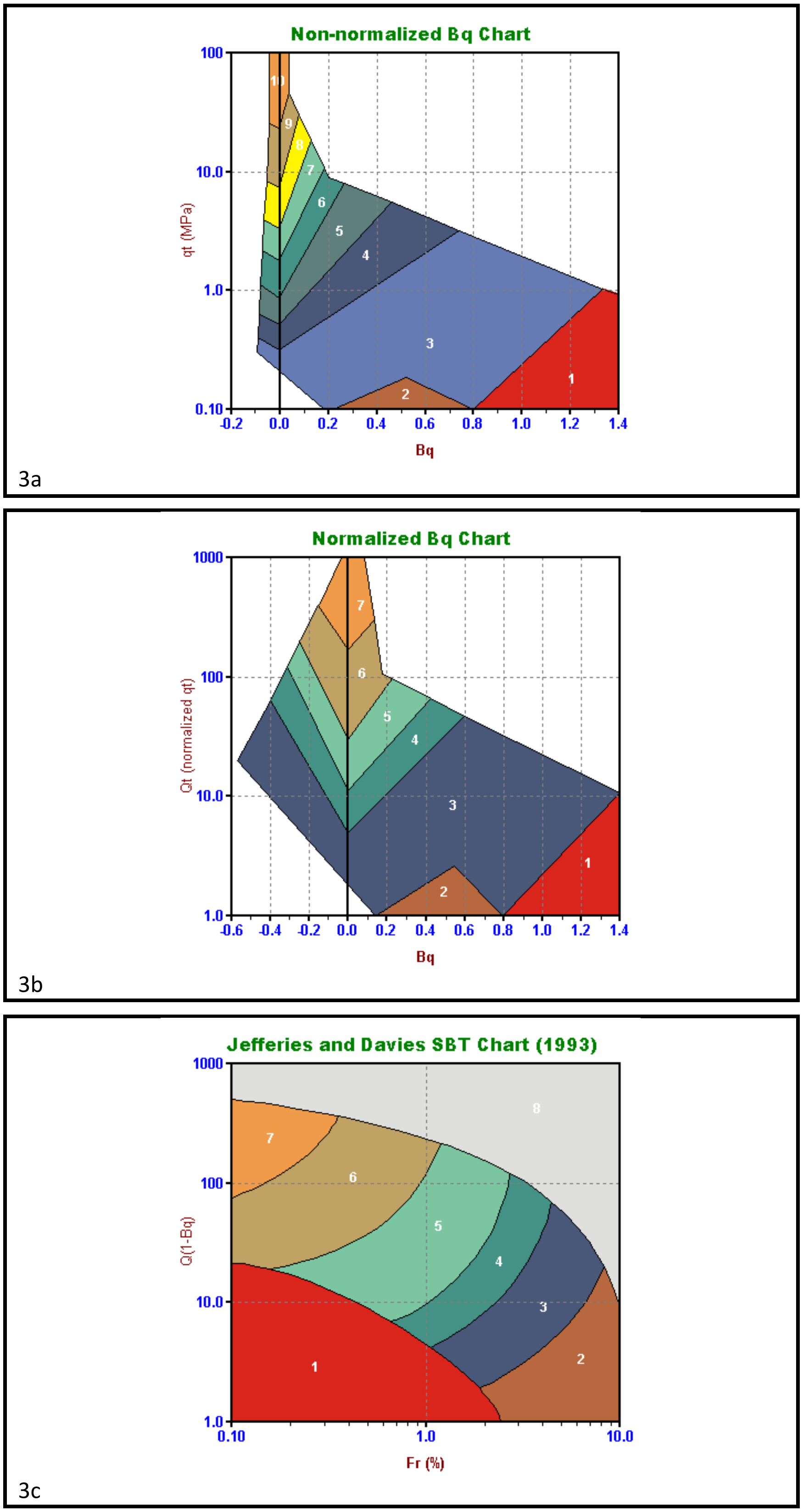


Figure 3. Alternate Soil Behavior Type Charts

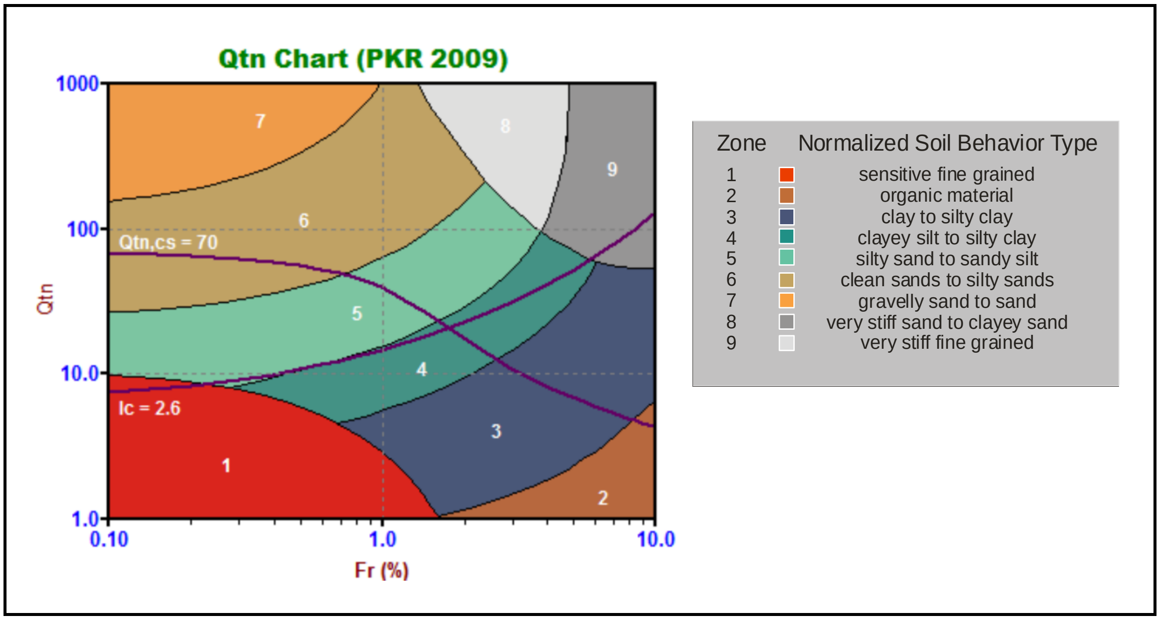


Figure 4. Normalized Soil Behavior Type Chart using Q_{tn} (SBT Qtn)

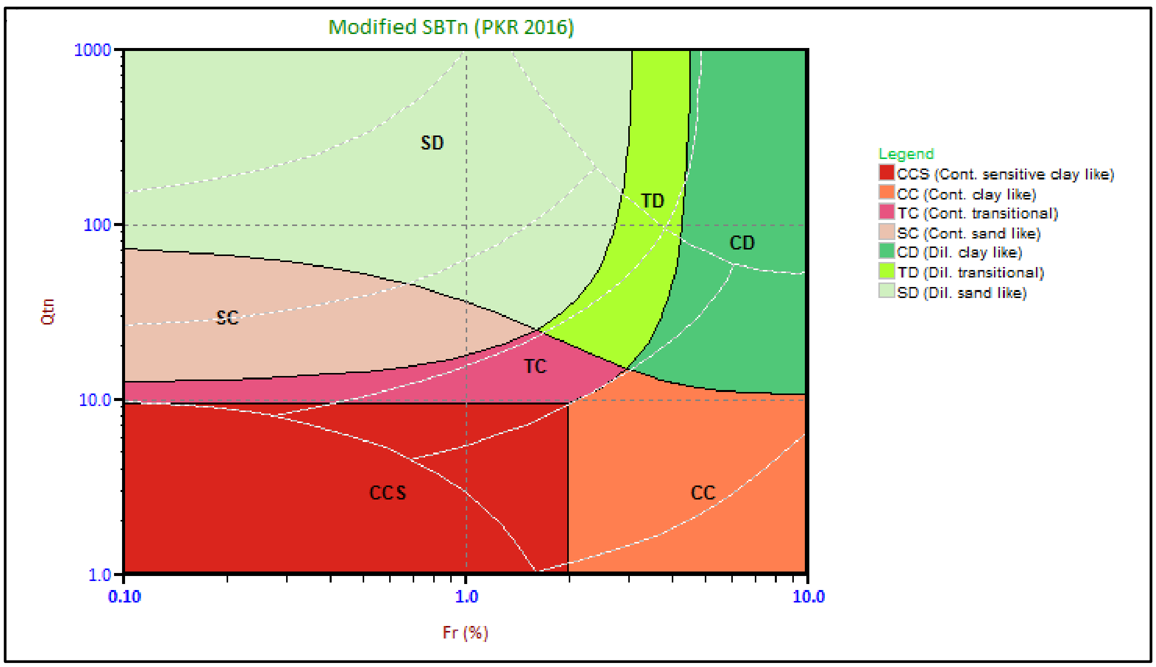


Figure 5. Modified SBTn Behavior Based Chart

Details regarding the geotechnical parameter calculations are provided in Tables 1a and 1b. The appropriate references cited are listed in Table 2. Non-liquefaction specific parameters are detailed in Table 1a and liquefaction specific parameters are detailed in Table 1b.

Where methods are based on charts or techniques that are too complex to describe in this summary the user should refer to the cited material. Specific limitations for each method are described in the cited material.

Where the results of a calculation/correlation are deemed *'invalid'* the value will be represented by the text strings "-9999", "-9999.0", the value 0.0 (Zero) or an empty cell. Invalid results will occur because of (and not limited to) one or a combination of:

1. Invalid or undefined CPT data (e.g. drilled out section or data gap).
2. Where the calculation method is inappropriate, for example, drained parameters in a material behaving as an undrained material (and vice versa).
3. Where input values are beyond the range of the referenced charts or specified limitations of the correlation method.
4. Where pre-requisite or intermediate parameter calculations are invalid.

The parameters selected for output from the program are often specific to a particular project. As such, not all of the calculated parameters listed in Table 1 may be included in the output files delivered with this report.

The output files are typically provided in Microsoft Excel XLS or XLSX format. The ConeTec software has several options for output depending on the number or types of calculated parameters desired or requested by the client. Each output file is named using the original COR file base name followed by a three or four letter indicator of the output set selected (e.g. BSC, TBL, NLI, NL2, IFI, IFI2) and possibly followed by an operator selected suffix identifying the characteristics of the particular calculation run.

Table 1a. CPT Parameter Calculation Methods – Non liquefaction Parameters

Calculated Parameter	Description	Equation	Ref
Depth	Mid Layer Depth *(where calculations are done at each point then Mid Layer Depth = Recorded Depth)*	*[Depth (Layer Top) + Depth (Layer Bottom)]/ 2.0*	CK*
Elevation	Elevation of Mid Layer based on sounding collar elevation supplied by client or through site survey	Elevation = Collar Elevation - Depth	CK*
Avg qc	Averaged recorded tip value (q_c)	$Avgqc = \frac{1}{n}\sum_{i=1}^{n} q_c$ *n=1 when calculations are done at each point*	CK*
Avg qt	Averaged corrected tip (q_t) where: $q_t = q_c + (1-a) \bullet u_2$	$Avgqt = \frac{1}{n}\sum_{i=1}^{n} q_t$ *n=1 when calculations are done at each point*	1
Avg fs	Averaged sleeve friction (f_s)	$Avgfs = \frac{1}{n}\sum_{i=1}^{n} fs$ *n=1 when calculations are done at each point*	CK*
Avg Rf	Averaged friction ratio (R_f) where friction ratio is defined as: $Rf = 100\% \bullet \frac{fs}{q_t}$	$AvgRf = 100\% \cdot \frac{Avgfs}{Avgqt}$ *n=1 when calculations are done at each point*	CK*
Avg u	Averaged dynamic pore pressure *(u)*	$Avgu = \frac{1}{n}\sum_{i=1}^{n} u_i$ *n=1 when calculations are done at each point*	CK*

Calculated Parameter	Description	Equation	Ref
Avg Res	Averaged Resistivity (this data is not always available since it is a specialized test requiring an additional module)	$AvgRes = \frac{1}{n}\sum_{i=1}^{n} Resistivity_i$ *n=1 when calculations are done at each point*	CK*
Avg UVIF	Averaged UVIF ultra-violet induced fluorescence (this data is not always available since it is a specialized test requiring an additional module)	$AvgUVIF = \frac{1}{n}\sum_{i=1}^{n} UVIF_i$ *n=1 when calculations are done at each point*	CK*
Avg Temp	Averaged Temperature (this data is not always available since it requires specialized calibrations)	$AvgTemp = \frac{1}{n}\sum_{i=1}^{n} Temperature_i$ *n=1 when calculations are done at each point*	CK*
Avg Gamma	Averaged Gamma Counts (this data is not always available since it is a specialized test requiring an additional module)	$AvgGamma = \frac{1}{n}\sum_{i=1}^{n} Gamma_i$ *n=1 when calculations are done at each point*	CK*
SBT	Soil Behavior Type as defined by Robertson et al 1986 (often referred to as Robertson and Campanella, 1986)	See Figure 1	1, 5
SBTn	Normalized Soil Behavior Type as defined by Robertson 1990 (linear normalization)	See Figure 2	2, 5
SBT-Bq	Non-normalized Soil Behavior type based on the Bq parameter	See Figure 3	1, 2, 5
SBT-Bqn	Normalized Soil Behavior based on the Bq parameter	See Figure 3	2, 5
SBT-JandD	Soil Behavior Type as defined by Jeffries and Davies	See Figure 3	7
SBT Qtn	Soil Behavior Type as defined by Robertson (2009) using a variable stress ratio exponent for normalization based on I_c	See Figure 4	15
Modified SBTn (contractive /dilative)	Modified SBTn chart as defined by Robertson (2016) indicating zones of contractive/dilative behavior.	See Figure 5	30
Unit Wt.	Unit Weight of soil determined from one of the following user selectable options: 1) uniform value 2) value assigned to each SBT zone 3) value assigned to each SBTn zone 4) value assigned to SBTn zone as determined from Robertson and Wride (1998) based on q_{c1n} 5) values assigned to SBT Qtn zones 6) Mayne fs (sleeve friction) method 7) Robertson 2010 method 8) user supplied unit weight profile The last option may co-exist with any of the other options	See references	3, 5, 15, 21, 24, 29

Calculated Parameter	Description	Equation	Ref
TStress σ_v	Total vertical overburden stress at Mid Layer Depth *A layer is defined as the averaging interval specified by the user where depths are reported at their respective mid-layer depth.* *For data calculated at each point layers are defined using the recorded depth as the mid-point of the layer. Thus, a layer starts half-way between the previous depth and the current depth unless this is the first point in which case the layer start is at zero depth. The layer bottom is half-way from the current depth to the next depth unless it is the last data point.* *Defining layers affects how stresses are calculated since the unit weight attributed to a data point is used throughout the entire layer. This means that to calculate the stresses the total stress at the top and bottom of a layer are required. The stress at mid layer is determined by adding the incremental stress from the layer top to the mid-layer depth. The stress at the layer bottom becomes the stress at the top of the subsequent layer. Stresses are NOT calculated from mid-point to mid-point.* *For over-water work the total stress due to the column of water above the mud line is taken into account where appropriate.*	$TStress = \sum_{i=1}^{n} \gamma_i h_i$ *where* γ_i is layer unit weight h_i is layer thickness	CK*
EStress σ_v'	Effective vertical overburden stress at mid-layer depth	$\sigma_v' = \sigma_v - u_{eq}$	CK*
Equil u u_{eq} or u_0	Equilibrium pore pressure determined from one of the following user selectable options: 1) hydrostatic below water table 2) user supplied profile 3) combination of those above When a user supplied profile is used/provided a linear interpolation is performed between equilibrium pore pressures defined at specific depths. If the profile values start below the water table then a linear transition from zero pressure at the water table to the first defined pointed is used. Equilibrium pore pressures may come from dissipation tests, adjacent piezometers or other sources. Occasionally, an extra equilibrium point ("assumed value") will be provided in the profile that does not come from a recorded value to smooth out any abrupt changes or to deal with material interfaces. These "assumed" values will be indicated on our plots and in tabular summaries.	For hydrostatic option: $u_{eq} = \gamma_w \cdot (D - D_{wt})$ where u_{eq} is equilibrium pore pressure γ_w is unit weight of water D is the current depth D_{wt} is the depth to the water table	CK*
K_0	Coefficient of earth pressure at rest, K_0	$K_0 = (1 - \sin\Phi') \, OCR^{\sin\Phi'}$	17
C_n	Overburden stress correction factor used for $(N_1)_{60}$ and older CPT parameters	$C_n = (P_a/\sigma_v')^{0.5}$ where *$0.0 < C_n < 2.0$ (user adjustable, typically 1.7)* *P_a is atmospheric pressure (100 kPa)*	12
C_q	Overburden stress normalizing factor	$C_q = 1.8 / (0.8 + (\sigma_v'/P_a))$ where *$0.0 < C_q < 2.0$ (user adjustable)* *P_a is atmospheric pressure (100 kPa)*	3, 12

Calculated Parameter	Description	Equation	Ref
N_{60}	SPT N value at 60% energy calculated from q_t/N ratios assigned to each SBT zone. This method has abrupt N value changes at zone boundaries.	*See Figure 1*	5
(N1)60	SPT N_{60} value corrected for overburden pressure	$(N_1)_{60} = C_n \bullet N_{60}$	4
N60Ic	SPT N_{60} values based on the I_c parameter [as defined by Roberston and Wride 1998 (5), or by Robertson 2009 (15)].	$(q_t/P_a)/\ N_{60} = 8.5\ (1 - I_c/4.6)$ $(q_t/P_a)/\ N_{60} = 10^{\,(1.1268 - 0.2817Ic)}$ Pa being atmospheric pressure	5 15, 31
(N1)60Ic	SPT N_{60} value corrected for overburden pressure (using N_{60} I_c). User has 3 options.	1) $(N_1)_{60}Ic = C_n \bullet (N_{60}\, I_c)$ 2) $q_{c1n}/\ (N_1)_{60}I_c = 8.5\ (1 - I_c/4.6)$ 3) $(Q_{tn})/\ (N_1)_{60}I_c = 10^{\,(1.1268 - 0.2817Ic)}$	4 5 15, 31
Su or Su (Nkt)	Undrained shear strength based on q_t S_u factor N_{kt} is user selectable	$Su = \frac{qt - \sigma_v}{N_{kt}}$	1, 5
Su or Su (Ndu)	Undrained shear strength based on pore pressure S_u factor $N_{\Delta u}$ is user selectable	$Su = \frac{u_2 - u_{eq}}{N_{\Delta u}}$	1, 5
Dr	Relative Density determined from one of the following user selectable options: a) Ticino Sand b) Hokksund Sand c) Schmertmann (1978) d) Jamiolkowski (1985) - All Sands e) Jamiolkowski et al (2003) (various compressibilities, K_o)	See reference (methods a through d) Jamiolkowski et al (2003) reference	5 14
PHI ϕ	Friction Angle determined from one of the following user selectable options (methods a through d are for sands and method e is for silts and clays): a) Campanella and Robertson b) Durgunoglu and Mitchel c) Janbu d) Kulhawy and Mayne e) NTH method (clays and silts)	See appropriate reference	5 5 5 11 23
Delta U/qt	Differential pore pressure ratio (older parameter used before B_q was established)	$= \frac{\Delta u}{qt}$ *where:* $\Delta u = u - u_{eq}$ *and* *u = dynamic pore pressure* u_{eq} *= equilibrium pore pressure*	CK*
Bq	Pore pressure parameter	$Bq = \frac{\Delta u}{qt - \sigma_v}$ where : $\Delta u = u - u_{eq}$ *and* *u = dynamic pore pressure* u_{eq} *= equilibrium pore pressure*	1, 2, 5
Net qt or qtNet	Net tip resistance (used in many subsequent correlations)	$qt - \sigma_v$	CK*
qe	Effective tip resistance (using the dynamic pore pressure u_2 and not equilibrium pore pressure)	$qt - u_2$	CK*

Calculated Parameter	Description	Equation	Ref
qeNorm	Normalized effective tip resistance	$\frac{qt-u_2}{\sigma'_v}$	CK*
Q_t or Norm: Qt	Normalized q_t for Soil Behavior Type classification as defined by Robertson (1990) using a linear stress normalization. Note this is different from Q_{tn}.	$Qt=\frac{qt-\sigma_v}{\sigma'_v}$	2, 5
F_r or Norm: Fr	Normalized Friction Ratio for Soil Behavior Type classification as defined by Robertson (1990)	$Fr=100\%\cdot\frac{fs}{qt-\sigma_v}$	2, 5
Q(1-Bq)	Q(1-Bq) grouping as suggested by Jefferies and Davies for their classification chart and the establishment of their I_c parameter	$Q\cdot(1-Bq)$ *where Bq is defined as above and Q is the same as the normalized tip resistance, Q_t, defined above*	6, 7
qc1	Normalized tip resistance, q_{c1}, using a fixed stress ratio exponent, n (this method has stress units)	$q_{c1} = q_t \cdot (Pa/\sigma_v')^{0.5}$ where: Pa = atmospheric pressure	21
qc1 (0.5)	Normalized tip resistance, q_{c1}, using a fixed stress ratio exponent, n (this method is unit-less)	$q_{c1}\,(0.5)= (q_t/P_a) \cdot (Pa/\sigma_v')^{0.5}$ where: Pa = atmospheric pressure	5
qc1 (Cn)	Normalized tip resistance, q_{c1}, based on C_n (this method has stress units)	$q_{c1}(Cn) = C_n * q_t$	5, 12
qc1 (Cq)	Normalized tip resistance, q_{c1}, based on C_q (this method has stress units)	$q_{c1}\,(Cq)= C_q * q_t$ (some papers use q_c)	5, 12
qc1n	normalized tip resistance, q_{c1n}, using a variable stress ratio exponent, n (where n=0.0, 0.70, 1.0) (this method is unit-less)	$q_{c1n} = (q_t / P_a)(P_a/\sigma_v')^n$ where: P_a = atm. Pressure and n varies as described below	3, 5
I_c or Ic (RW1998)	Soil Behavior Type Index as defined by Robertson and Fear (1995) and Robertson and Wride (1998) for estimating grain size characteristics and providing smooth gradational changes across the SBTn chart	$I_c = [(3.47 - \log_{10}Q)^2 + (\log_{10} Fr + 1.22)^2\,]^{0.5}$ *Where:* $Q=\left(\frac{qt-\sigma_v}{P_a}\right)\left(\frac{P_a}{\sigma'_v}\right)^n$ *Or* $Q=q_{c1n}=\left(\frac{qt}{P_a}\right)\left(\frac{P_a}{\sigma'_v}\right)^n$ *depending on the iteration in determining I_c* *And Fr is in percent* *P_a = atmospheric pressure* *n varies between 0.5, 0.70 and 1.0 and is selected in an iterative manner based on the resulting I_c*	3, 5, 21
Ic (PKR 2009)	Soil Behavior Type Index, I_c (PKR 2009) based on a variable stress ratio exponent n, which itself is based on I_c (PKR 2009). An iterative calculation is required to determine Ic (PKR 2009) and its corresponding n (PKR 2009).	I_c (PKR 2009) = $[(3.47 - \log_{10}Q_{tn})^2 + (1.22 + \log_{10}F_r)^2]^{0.5}$	15

Calculated Parameter	Description	Equation	Ref
n (PKR 2009)	Stress ratio exponent n, based on I_c (PKR 2009). An iterative calculation is required to determine n (PKR 2009) and its corresponding Ic (PKR 2009).	*n (PKR 2009) = 0.381 (I_c) + 0.05 (σ_v'/P_a) – 0.15*	15
Qtn (PKR 2009)	Normalized tip resistance using a variable stress ratio exponent based on I_c (PKR 2009) and n (PKR 2009). An iterative calculation is required to determine Qtn (PKR 2009).	$Q_{tn} = [(qt - \sigma_v)/P_a](P_a/\sigma_v')^n$ *where P_a = atmospheric pressure (100 kPa)* *n = stress ratio exponent described above*	15
FC	Apparent fines content (%)	$FC=1.75(Ic^{3.25}) - 3.7$ $FC=100$ for $I_c > 3.5$ $FC=0$ for $I_c < 1.26$ $FC = 5\%$ if $1.64 < I_c < 2.6$ AND $F_r<0.5$	3
I_c Zone	This parameter is the Soil Behavior Type zone based on the I_c parameter (valid for zones 2 through 7 on SBTn or SBT Qtn charts)	$I_c < 1.31$ Zone = 7 $1.31 < I_c < 2.05$ Zone = 6 $2.05 < I_c < 2.60$ Zone = 5 $2.60 < I_c < 2.95$ Zone = 4 $2.95 < I_c < 3.60$ Zone = 3 $I_c > 3.60$ Zone = 2	3
State Param or State Parameter or ψ	The state parameter index, ψ, is defined as the difference between the current void ratio, e, and the critical void ratio, e_c. Positive ψ - contractive soil Negative ψ - dilative soil This is based on the work by Been and Jefferies (1985) and Plewes, Davies and Jefferies (1992) - vertical effective stress is used rather than a mean normal stress	See reference	6, 8
Yield Stress σ_p'	Yield stress is calculated using the following methods a) General method b) 1st order approximation using q_tNet (clays) c) 1st order approximation using Δu_2 (clays) d) 1st order approximation using q_e (clays)	All stresses in kPa a) $\sigma_p' = 0.33\cdot(q_t - \sigma_v)^{m'} (\sigma_{atm}/100)^{1-m'}$ where $m' = 1 - \dfrac{0.28}{1+(I_c/2.65)^{25}}$ b) $\sigma_p' = 0.33\cdot(q_t - \sigma_v)$ c) $\sigma_p' = 0.54\cdot(\Delta u_2)$ $\Delta u_2 = u_2 - u_0$ d) $\sigma_p' = 0.60\cdot(q_t - u_2)$	19 20 20 20
OCR OCR(JS1978) OCR(Mayne2014) OCR (qtNet) OCR (deltaU) OCR (qe) OCR (Vs) OCR (PKR2015)	Over Consolidation Ratio based on a) Schmertmann (1978) method involving a plot plot of S_u/σ_v' /($S_u/\sigma_v')_{NC}$ and OCR b) based on Yield stresses described above c) approximate version based on qtNet d) approximate version based on Δu e) approximate version based on effective tip, q_e f) approximate version based on shear wave velocity, V_s g) based on Qt	a) requires a user defined value for NC S_u/P_c' ratio b through f) *based on yield stresses* g) $OCR = 0.25\cdot(Qt)^{1.25}$	9 19 20 20 20 18 32

Calculated Parameter	Description	Equation	Ref
Es/qt	Intermediate parameter for calculating Young's Modulus, E, in sands. It is the Y axis of the reference chart.	Based on Figure 5.59 in the reference	5
Es Young's Modulus E	Young's Modulus based on the work done in Italy. There are three types of sands considered in this technique. The user selects the appropriate type for the site from: a) OC Sands b) Aged NC Sands c) Recent NC Sands Each sand type has a family of curves that depend on mean normal stress. The program calculates mean normal stress and linearly interpolates between the two extremes provided in the E_s/q_t chart. Es is evaluated for an axial strain of 0.1%.	Mean normal stress is evaluated from: $\sigma'_m = \frac{1}{3}(\sigma'_v + \sigma'_h + \sigma'_h)^3$ where σ_v'= vertical effective stress σ_h'= horizontal effective stress *and* $\sigma_h = K_o \bullet \sigma_v'$ with K_o assumed to be 0.5	5
Delta U/TStress	Differential pore pressure ratio with respect to total stress	$= \frac{\Delta u}{\sigma_v}$ *where:* $\Delta u = u - u_{eq}$	CK*
Delta U/Estress, P Value, Excess Pore Pressure Ratio	Differential pore pressure ratio with respect to effective stress. Key parameter (P, Normalized Pore Pressure Parameter, Excess Pore Pressure Ratio) in the Winckler et. al. static liquefaction method.	$= \frac{\Delta u}{\sigma'_v}$ *where:* $\Delta u = u - u_{eq}$	25, 25a, CK*
Su/EStress	Undrained shear strength ratio with respect to vertical effective overburden stress using the S_u (N_{kt}) method	$= Su\ (N_{kt}) / \sigma_v'$	CK*
Gmax	G_{max} determined from SCPT shear wave velocities (not estimated values)	$G_{max} = \rho V_s^2$ where ρ is the mass density of the soil determined from the estimated unit weights at each test depth	27
qtNet/Gmax	Net tip resistance ratio with respect to the small strain modulus G_{max} determined from SCPT shear wave velocities (not estimated values)	$= (qt - \sigma_v) / G_{max}$ where $G_{max} = \rho V_s^2$ and ρ is the mass density of the soil determined from the estimated unit weights at each test depth	15, 28, 30

*CK – common knowledge

Table 1b. CPT Parameter Calculation Methods – Liquefaction Parameters

Calculated Parameter	Description	Equation	Ref
K_{SPT}	Equivalent clean sand factor for $(N_1)60$	$K_{SPT} = 1 + ((0.75/30) \bullet (FC - 5))$	10
K_{CPT} or K_C (RW1998)	Equivalent clean sand correction for q_{c1N}	$K_{cpt} = 1.0$ for $I_c \leq 1.64$ $K_{cpt} = f(I_c)$ for $I_c > 1.64$ (see reference) $K_c = -0.403\ I_c^4 + 5.581\ I_c^3 - 21.63 I_c^2 + 33.75\ I_c - 17.88$	3, 10
Kc (PKR 2010)	Clean sand equivalent factor to be applied to Q_{tn}	$K_c = 1.0$ for $I_c \leq 1.64$ $K_c = -0.403\ I_c^4 + 5.581\ I_c^3 - 21.63 I_c^2 + 33.75\ I_c - 17.88$ for $I_c > 1.64$	16
$(N_1)_{60cs}Ic$	Clean sand equivalent SPT $(N_1)_{60}I_c$. User has 3 options.	1) $(N_1)_{60cs}Ic = \alpha + \beta((N_1)_{60}I_c)$ 2) $(N_1)_{60cs}Ic = K_{SPT} * ((N_1)_{60}I_c)$ 3) $(q_{c1ncs})/ (N_1)_{60cs}I_c = 8.5\ (1 - I_c/4.6)$ FC ≤ 5%: $\alpha = 0$, $\beta=1.0$ FC ≥ 35% $\alpha = 5.0$, $\beta=1.2$ 5% < FC < 35% $\alpha = exp[1.76 - (190/FC^2)]$ $\beta = [0.99 + (FC^{1.5}/1000)]$	10 10 5
q_{c1ncs}	Clean sand equivalent q_{c1n}	$q_{c1ncs} = q_{c1n} \bullet K_{cpt}$	3
Qtn,cs (PKR 2010)	Clean sand equivalent for Q_{tn} described above - Q_{tn} being the normalized tip resistance based on a variable stress exponent as defined by Robertson (2009)	$Q_{tn,cs} = Q_{tn} \cdot K_c$ (PKR 2016)	16
Su(Liq)/ESv	Liquefied shear strength ratio as defined by Olson and Stark	$\frac{Su(Liq)}{\sigma_v'} = 0.03 + 0.0143(q_{c1})$ Note: σ_v' and s_v' are synonymous	13
Su(Liq)/ESv (PKR 2010)	Liquefied shear strength ratio as defined by Robertson (2010)	$\frac{Su(Liq)}{\sigma_v'}$ Based on a function involving $Q_{tn,cs}$	16
Su (Liq) (PKR 2010)	Liquefied shear strength derived from the liquefied shear strength ratio and effective overburden stress		16
Cont/Dilat Tip	Contractive / Dilative qc1 Boundary based on $(N_1)_{60}$	$(\sigma_v')_{boundary} = 9.58 \times 10^{-4}\ [(N_1)_{60}]^{4.79}$ *qc1* is calculated from specified qt(MPa)/N ratio	13
CRR	Cyclic Resistance Ratio (for Magnitude 7.5)	$q_{c1ncs} < 50$: $CRR_{7.5} = 0.833\ [q_{c1ncs}/1000] + 0.05$ $50 \leq q_{c1ncs} < 160$: $CRR_{7.5} = 93\ [q_{c1ncs}/1000]^3 + 0.08$	10
Kg	Small strain Stiffness Ratio Factor, Kg	$[Gmax/qt]/[qc1n^{-m}]$ m = empirical exponent, typically 0.75	26

Calculated Parameter	Description	Equation	Ref
SP Distance	State Parameter Distance, Winckler static liquefaction method	Perpendicular distance on Qtn chart from plotted point to state parameter Ψ = -0.05 curve	25
URS NP Fr	Normalized friction ratio point on Ψ = -0.05 curve used in SP Distance calculation		25
URS NP Qtn	Normalized tip resistance (Qtn) point on Ψ = -0.05 curve used in SP Distance calculation		25

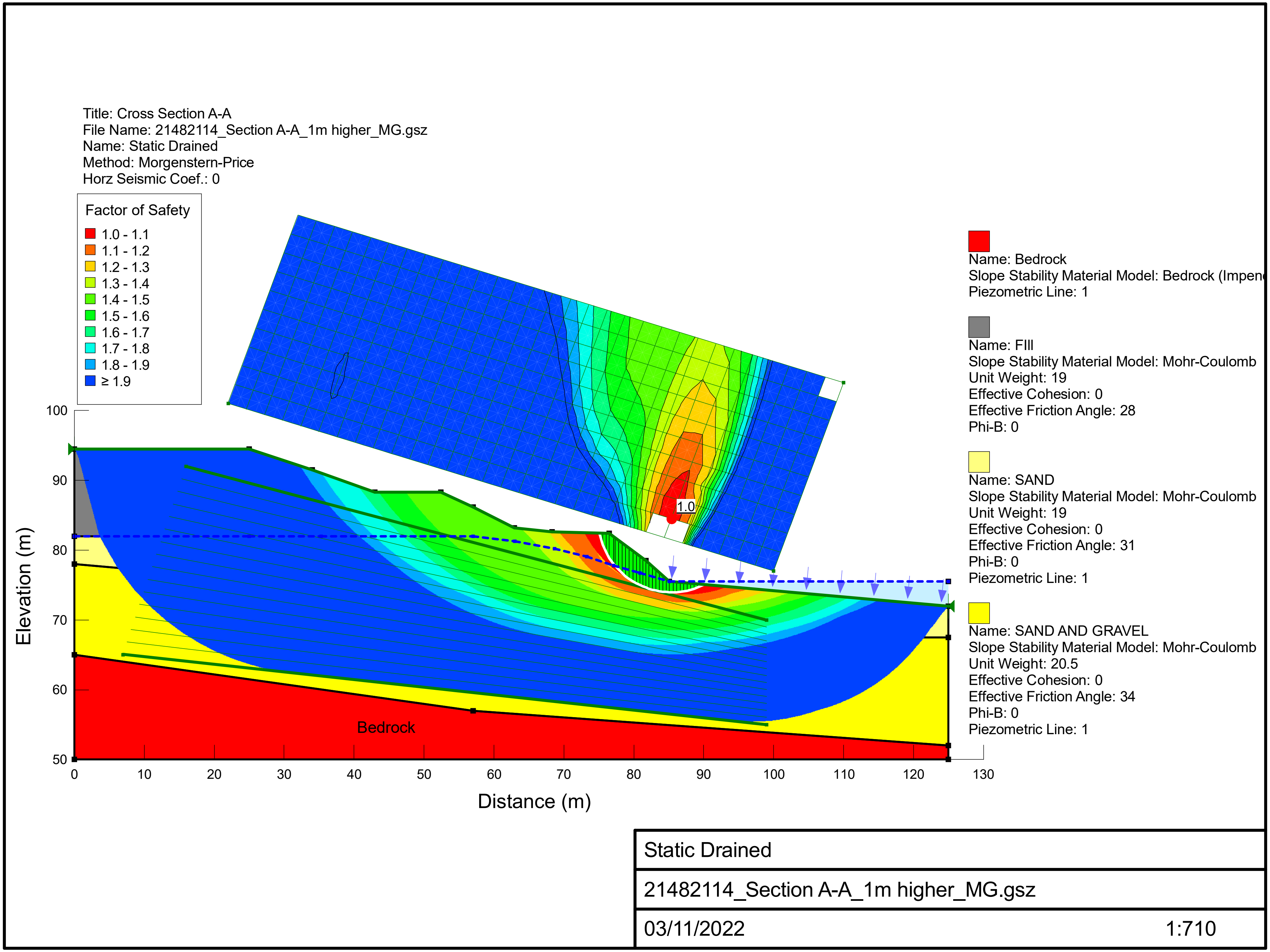
Table 2. References

No.	Reference
1	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.
2	Robertson, P.K., 1990, "Soil Classification Using the Cone Penetration Test", Canadian Geotechnical Journal, Volume 27. This includes the discussions and replies.
3	Robertson, P.K. and Wride (Fear), C.E., 1998, "Evaluating cyclic liquefaction potential using the cone penetration test", Canadian Geotechnical Journal, 35: 442-459.
4	Robertson, P.K. and Wride, C.E., 1997, "Cyclic Liquefaction and its Evaluation Based on SPT and CPT", NCEER Workshop Paper, January 22, 1997.
5	Lunne, T., Robertson, P.K. and Powell, J. J. M., 1997, " Cone Penetration Testing in Geotechnical Practice," Blackie Academic and Professional.
6	Plewes, H.D., Davies, M.P. and Jefferies, M.G., 1992, "CPT Based Screening Procedure for Evaluating Liquefaction Susceptibility", 45th Canadian Geotechnical Conference, Toronto, Ontario, October 1992.
7	Jefferies, M.G. and Davies, M.P., 1993, "Use of CPTu to Estimate equivalent N_{60}", Geotechnical Testing Journal, 16(4): 458-467.
8	Been, K. and Jefferies, M.P., 1985, "A state parameter for sands", Geotechnique, 35(2), 99-112.
9	Schmertmann, 1978, "Guidelines for Cone Penetration Test Performance and Design", Federal Highway Administration Report FHWA-TS-78-209, U.S. Department of Transportation.
10	Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils, Salt Lake City, 1996, chaired by Leslie Youd.
11	Kulhawy, F.H. and Mayne, P.W., 1990, "Manual on Estimating Soil Properties for Foundation Design, Report No. EL-6800", Electric Power Research Institute, Palo Alto, CA, August 1990, 306 p.
12	Olson, S.M. and Stark, T.D., 2002, "Liquefied strength ratio from liquefied flow failure case histories", Canadian Geotechnical Journal, 39: 951-966.
13	Olson, Scott M. and Stark, Timothy D., 2003, "Yield Strength Ratio and Liquefaction Analysis of Slopes and Embankments", Journal of Geotechnical and Geoenvironmental Engineering, ASCE, August 2003.
14	Jamiolkowski, M.B., Lo Presti, D.C.F. and Manassero, M., 2003, "Evaluation of Relative Density and Shear Strength of Sands from CPT and DMT", Soil Behaviour and Soft Ground Construction, ASCE, GSP N0. 119, 201-238.
15	Robertson, P.K., 2009, "Interpretation of cone penetration tests – a unified approach", Canadian Geotechnical Journal, 46: 1337-1355.
16	Robertson, P.K., 2010, "Evaluation of Flow Liquefaction and Liquefied Strength Using the Cone Penetration Test", Journal of Geotechnical and Geoenvironmental Engineering, ASCE, June 2010.
17	Mayne, P.W. and Kulhawy, F.H., 1982, "Ko-OCR Relationships in Soil", Journal of the Geotechnical Engineering Division, ASCE, Vol. 108, GT6, pp. 851-872.
18	Mayne, P.W., Robertson P.K. and Lunne T., 1998, "Clay stress history evaluated from seismic piezocone tests", Proceedings of the First International Conference on Site Characterization – ISC '98, Atlanta Georgia, Volume 2, 1113-1118.

No.	Reference
19	Mayne, P.W., 2014, “Generalized CPT Method for Evaluating Yield Stress in Soils”, Geocharacterization for Modeling and Sustainability (GSP 235: Proc. GeoCongress 2014, Atlanta, GA), ASCE, Reston, Virginia: 1336-1346.
20	Mayne, P.W., 2015, “Geocharacterization by In-Situ Testing”, Continuing Education Course, Vancouver, BC, January 6-8, 2015.
21	Robertson, P.K. and Fear, C.E., 1995, “Liquefaction of sands and its evaluation”, Proceedings of the First International Conference on Earthquake Engineering, Keynote Lecture IS Tokyo ‘95, Tokyo Japan, 1995.
22	Mayne, P.W., Peuchen, J. and Boumeester, D., 2010, “Soil unit weight estimation from CPTs”, Proceeding of the 2nd International Symposium on Cone Penetration Testing (CPT ’10), Vol 2, Huntington Beach, California; Omnipress: 169-176.
23	Mayne, P.W., 2007, “NCHRP Synthesis 368 on Cone Penetration Test”, Transportation Research Board, National Academies Press, Washington, D.C., 118 pages.
24	Mayne, P.W., 2014, “Interpretation of geotechnical parameters from seismic piezocone tests.”, Key note address #2, proceedings, 3rd International Symposium on Cone Penetration Testing (CPT’14, Las Vegas), ISSMGE Technical Committee TC102.
25	Winckler, Christina, Davidson, Richard, Yenne, Lisa, Pilz, Jorgen, 2014, “CPTu-Based State Characterization of Tailings Liquefaction Susceptibility”, Tailings and Mine Waste, 2014.
25a	Winckler, Christina, Davidson, Richard, Yenne, Lisa, Pilz, Jorgen, 2014, “CPTu-Based State Characterization of Tailings Liquefaction Susceptibility”, Powerpoint presentation, Tailings and Mine Waste, 2014.
26	Schneider, J.A. and Moss, R.E.S., 2011, “Linking cyclic stress and cyclic strain based methods for assessment of cyclic liquefaction triggering in sands”, Geotechnique Letters 1, 31-36.
27	Rice, A., 1984, “The Seismic Cone Penetrometer”, M.A.Sc. thesis submitted to the University of British Columbia, Dept. of Civil Engineering, Vancouver, BC, Canada.
28	Gillespie, D.G., 1990, “Evaluating Shear Wave Velocity and Pore Pressure Data from the Seismic Cone Penetration Test”, Ph.D. thesis submitted to the University of British Columbia, Dept. of Civil Engineering, Vancouver, BC, Canada.
29	Robertson, P.K and Cabal, K.L., 2010, “Estimating soil unit weight from CPT”, Proceedings of the 2nd International Symposium on Cone Penetration Testing (CPT ’10), Huntington Beach, California.
30	Robertson, P.K., 2016, “Cone penetration test (CPT)-based soil behaviour type (SBT) classification system – an update”, Canadian Geotechnical Journal, July 2016.
31	Robertson, P.K., 2012, “Interpretation of in-situ tests – some insights”, Mitchell Lecture, ISC’4, Recife, Brazil.
32	Robertson, P.K., Cabal, K.L. 2015, “Guide to Cone Penetration Testing for Geotechnical Engineering”, 6th Edition.

APPENDIX F

Slope/W Results – Sections A-A’ to I-I’



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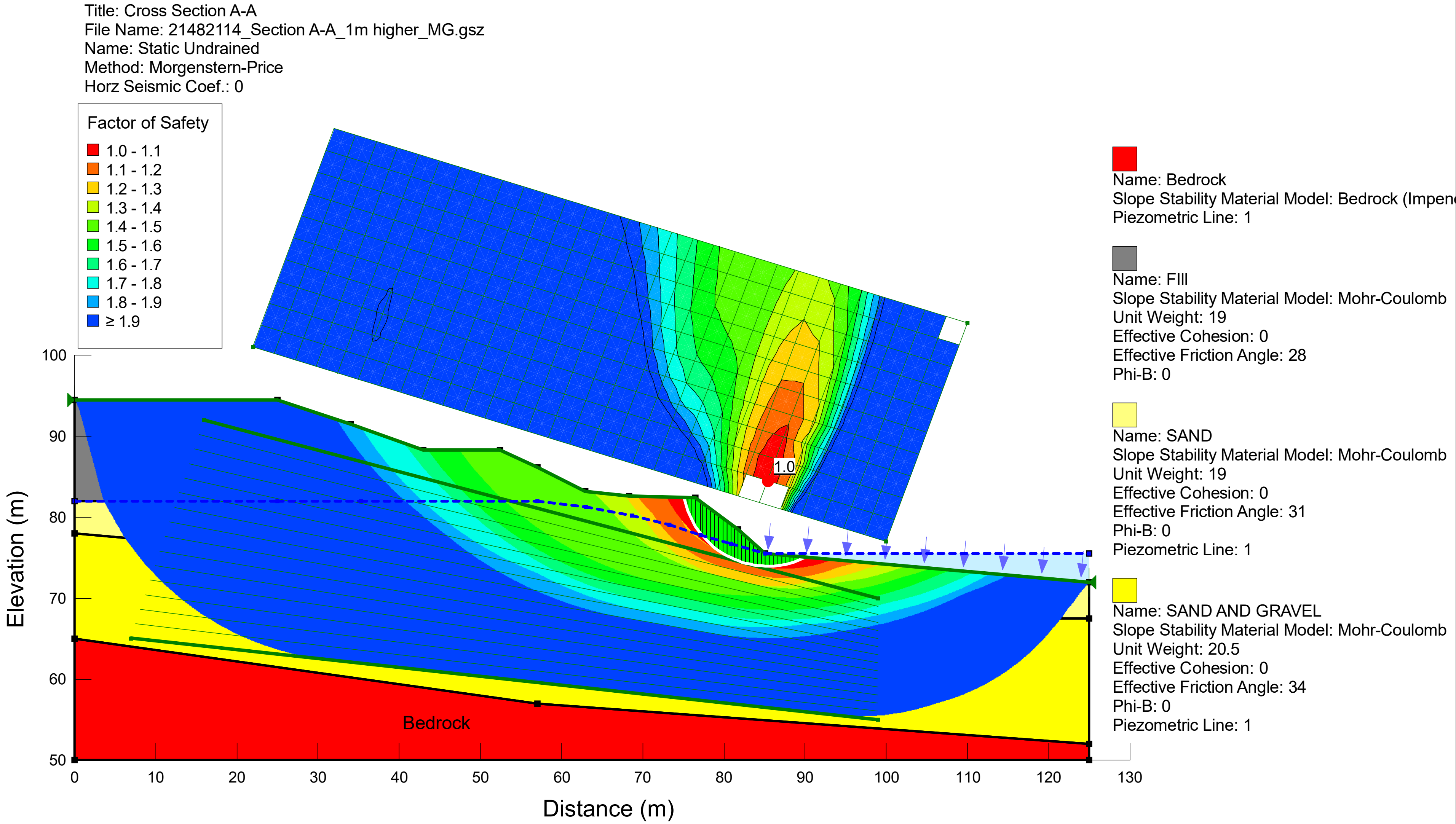
Method: Morgenstern-Price
Horz Seismic Coef.: 0
Factor of Safety
1.0 - 1.1
1.1 - 1.2
1.2 - 1.3
1.3 - 1.4
1.4 - 1.5
1.5 - 1.6
1.6 - 1.7
1.7 - 1.8
1.8 - 1.9
≥ 1.9
1.0
Bedrock
Elevation (m)
Distance (m)
100
90
80
70
60
50
0
10
20
30
40
50
60
70
80
90
100
110
120
130
Name: Bedrock
Slope Stability Material Model: Bedrock (Impen
Piezometric Line: 1
Name: FIll
Slope Stability Material Model: Mohr-Coulomb
Unit Weight: 19
Effective Cohesion: 0
Effective Friction Angle: 28
Phi-B: 0
Name: SAND
Slope Stability Material Model: Mohr-Coulomb
Unit Weight: 19
Effective Cohesion: 0
Effective Friction Angle: 31
Phi-B: 0
Piezometric Line: 1
Name: SAND AND GRAVEL
Slope Stability Material Model: Mohr-Coulomb
Unit Weight: 20.5
Effective Cohesion: 0
Effective Friction Angle: 34
Phi-B: 0
Piezometric Line: 1
Static Drained
21482114_Section A-A_1m higher_MG.gsz
03/11/2022
1:710



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03/11/2022	1:710

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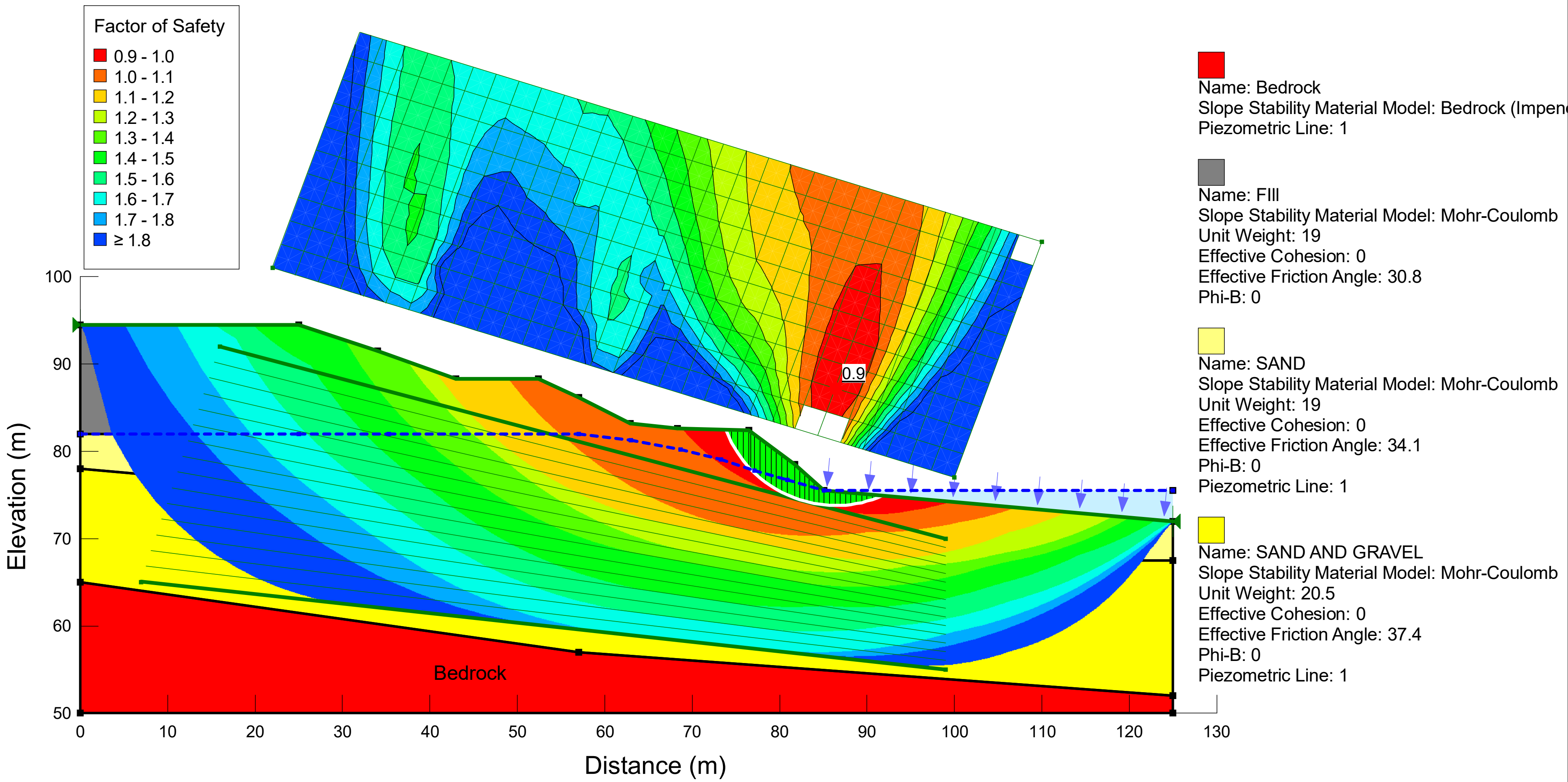
Method: Morgenstern-Price
Horz Seismic Coef.: 0.155



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03/11/2022	1:710

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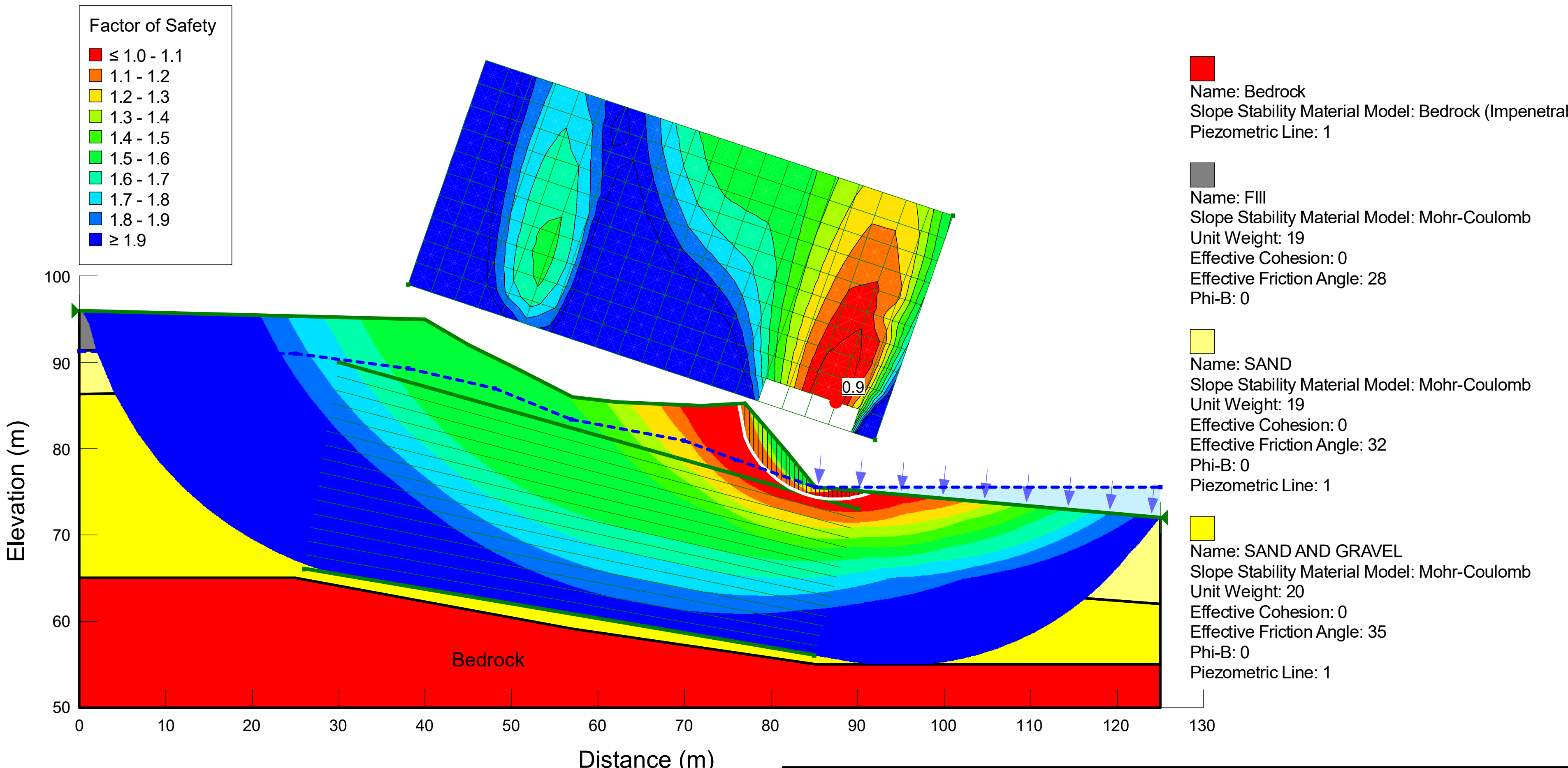
Name: Static Drained
Method: Morgenstern-Price
Horz Seismic Coef.: 0



Static Drained	
21482114_Section B-B_1m higher_MG.gsz	
03/17/2022	1:720

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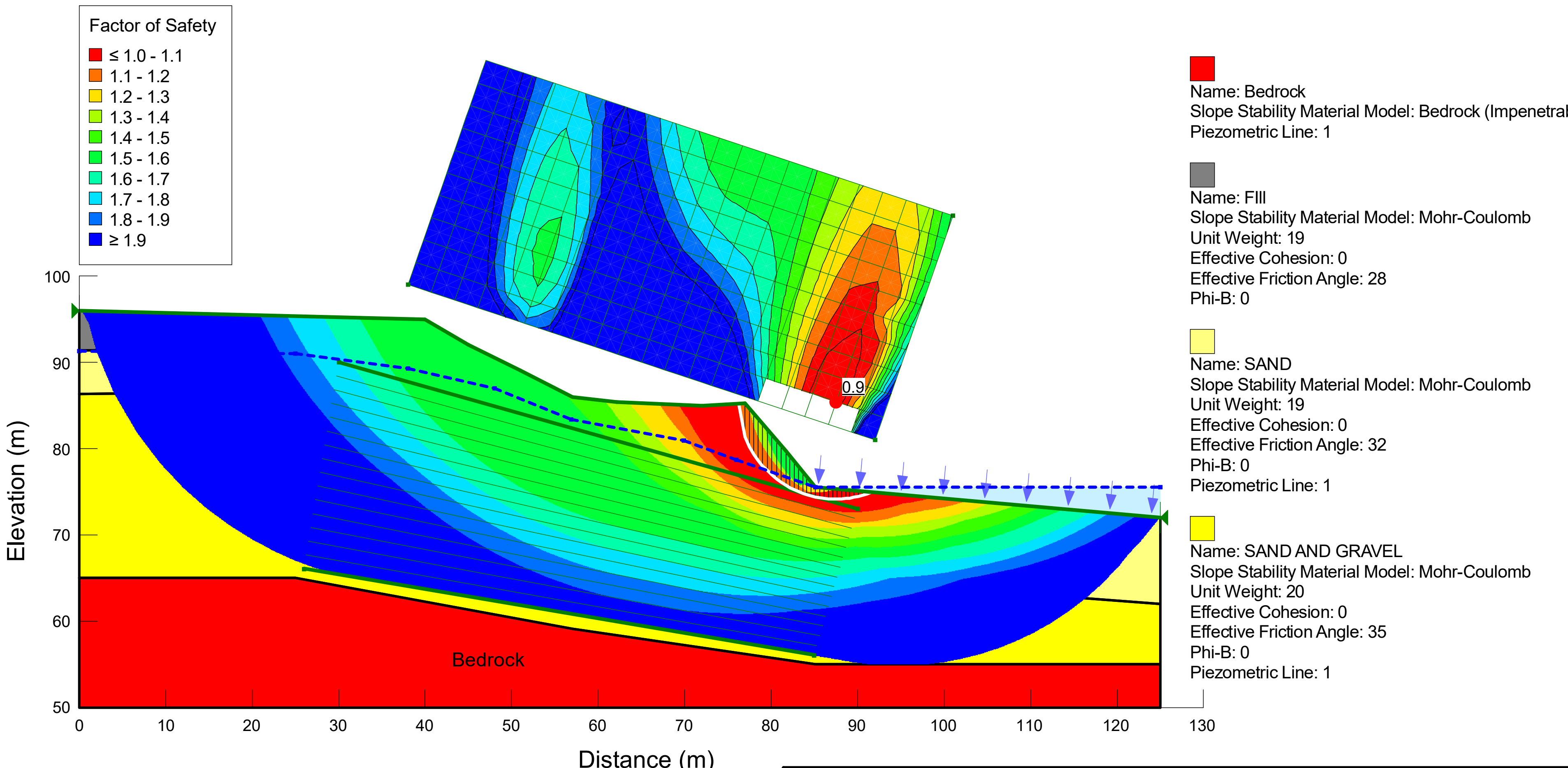
Method: Morgenstern-Price
Horz Seismic Coef.: 0



Static Undrained	
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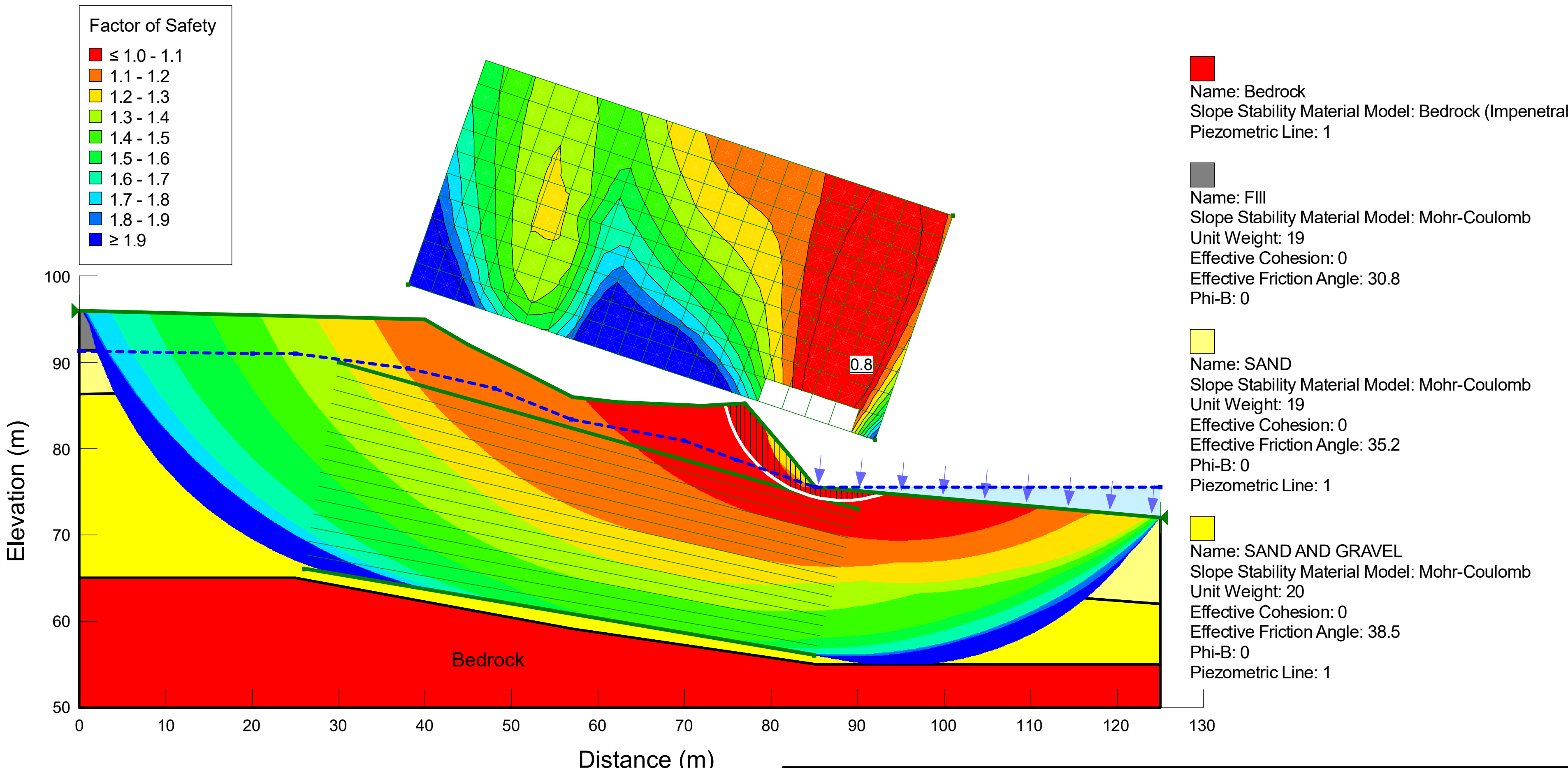
Method: Morgenstern-Price
Horz Seismic Coef.: 0.155



Seismic Undrained	
21482114_Section B-B_1m higher_10%_MG.gsz	
03/17/2022	1:720

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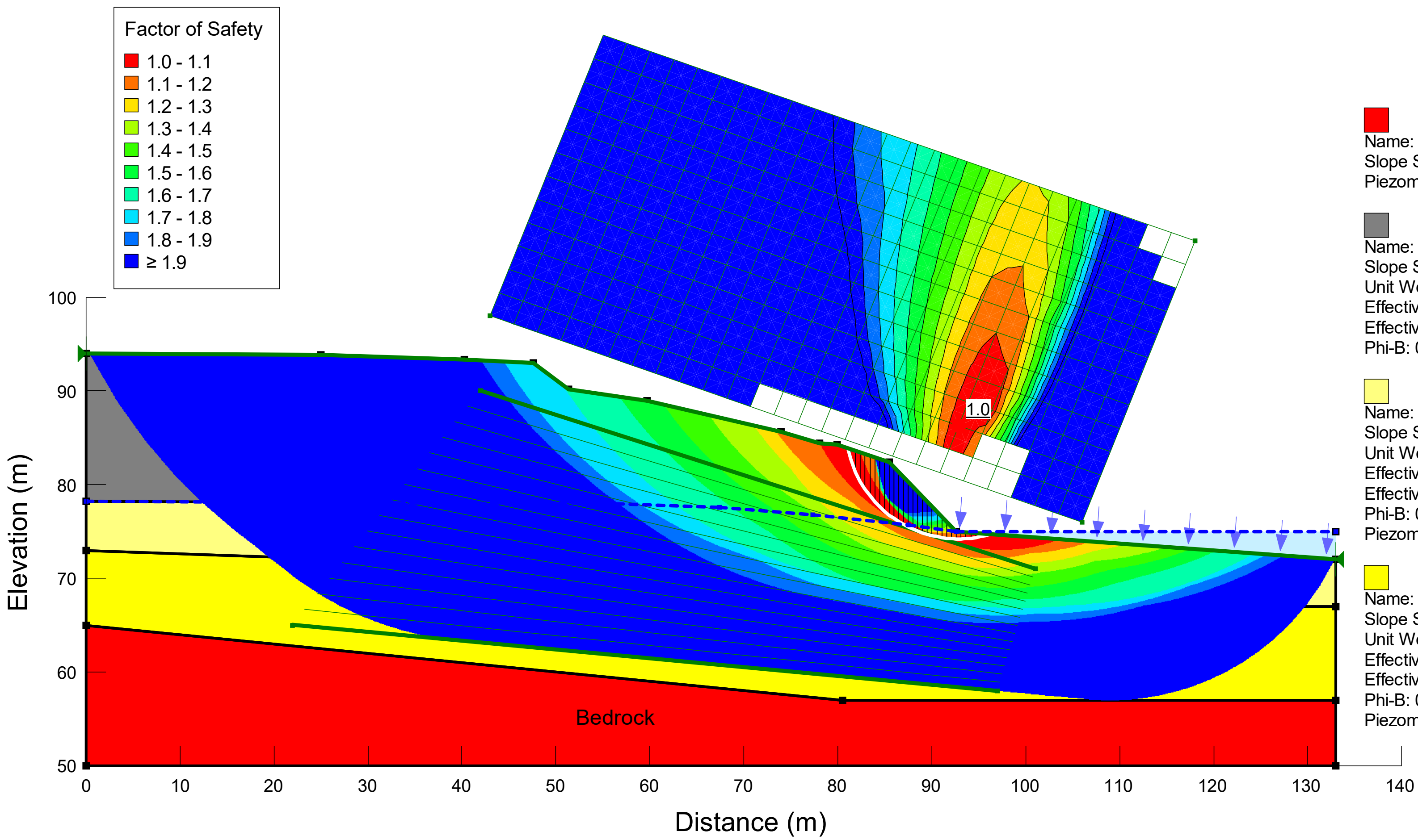
Name: Static Drained
Method: Morgenstern-Price
Horz Seismic Coef.: 0



Name: Bedrock
Slope Stability Material Model: Bedrock (Imper
Piezometric Line: 1

Name: FIll
Slope Stability Material Model: Mohr-Coulomb
Unit Weight: 19
Effective Cohesion: 0
Effective Friction Angle: 28
Phi-B: 0

Name: SAND
Slope Stability Material Model: Mohr-Coulomb
Unit Weight: 19
Effective Cohesion: 0
Effective Friction Angle: 31
Phi-B: 0
Piezometric Line: 1

Name: SAND AND GRAVEL
Slope Stability Material Model: Mohr-Coulomb
Unit Weight: 20.5
Effective Cohesion: 0
Effective Friction Angle: 34
Phi-B: 0
Piezometric Line: 1

Static Drained	
21482114_Section C-C_1m higher_MG.gsz	
03/11/2022	1:720

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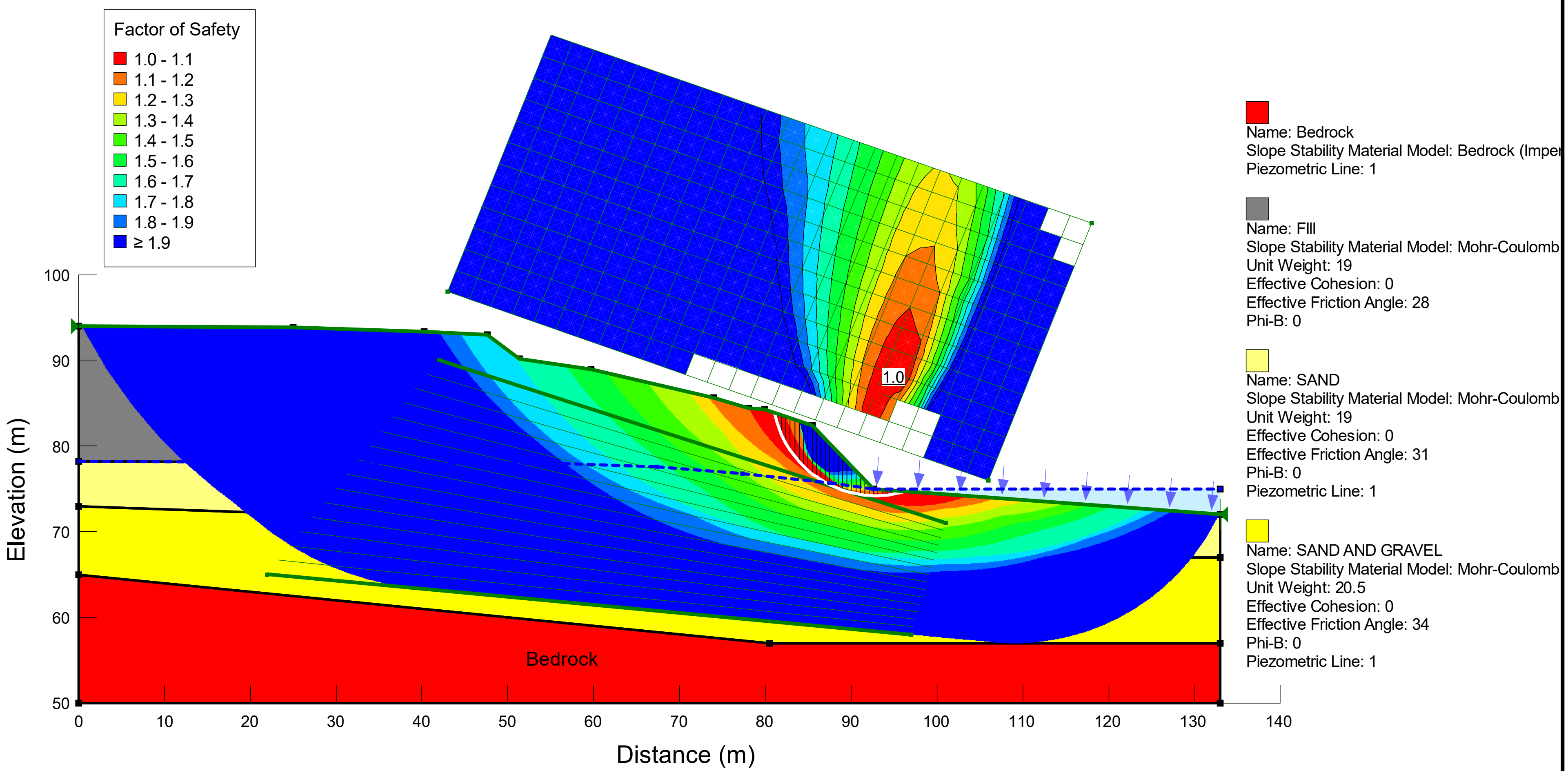
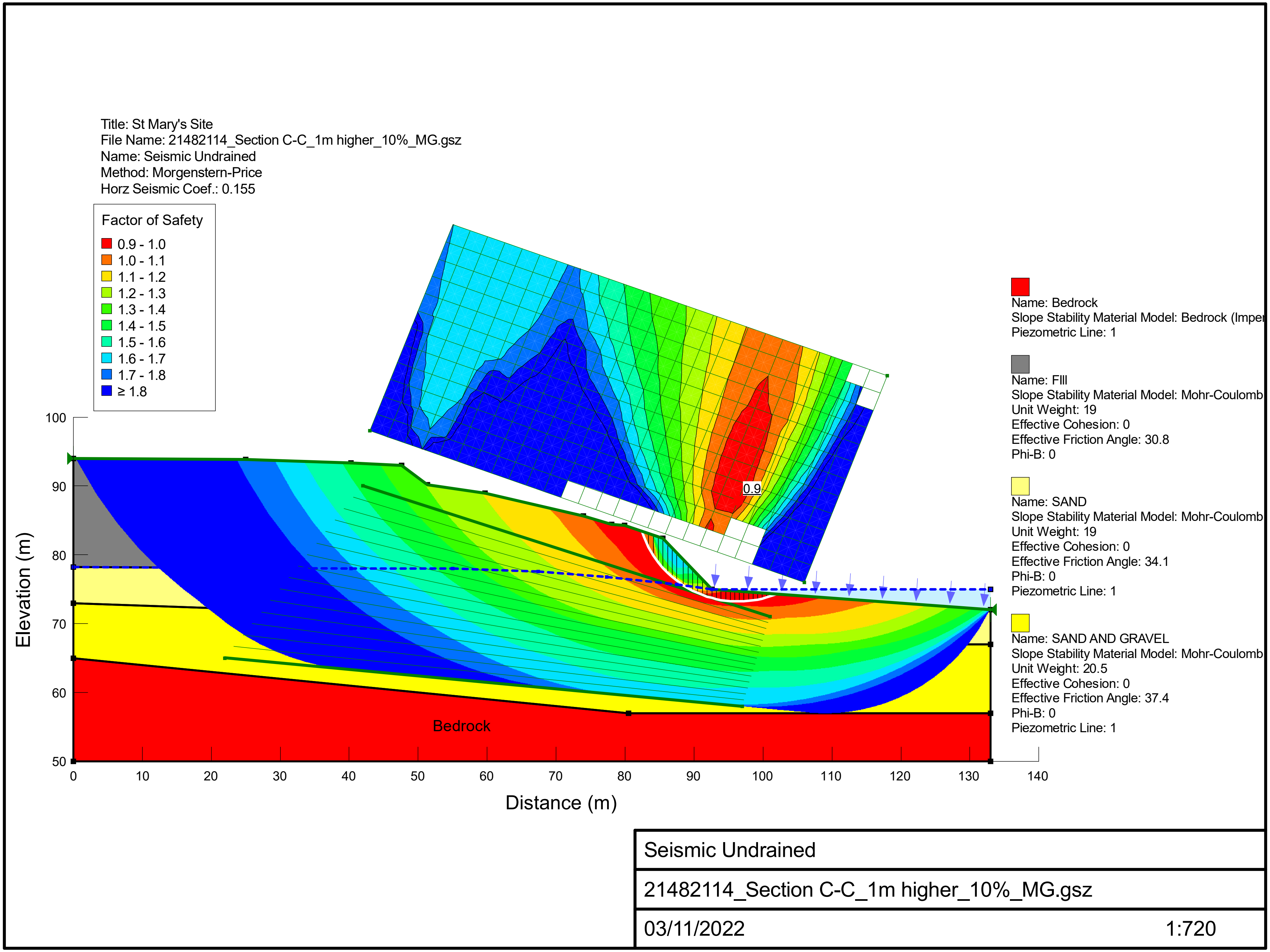
Method: Morgenstern-Price
Horz Seismic Coef.: 0



Static Undrained	
21482114_Section C-C_1m higher_MG.gsz	
03/11/2022	1:720


Title: St Mary's Site
File Name: 21482114_Section C-C_1m higher_10%_MG.gsz
Name: Seismic Undrained
Method: Morgenstern-Price
Horz Seismic Coef.: 0.155
Factor of Safety
0.9 - 1.0
1.0 - 1.1
1.1 - 1.2
1.2 - 1.3
1.3 - 1.4
1.4 - 1.5
1.5 - 1.6
1.6 - 1.7
1.7 - 1.8
≥ 1.8
0.9
Bedrock
Elevation (m)
Distance (m)
100
90
80
70
60
50
0
10
20
30
40
50
60
70
80
90
100
110
120
130
140
Name: Bedrock
Slope Stability Material Model: Bedrock (Imper
Piezometric Line: 1
Name: FIll
Slope Stability Material Model: Mohr-Coulomb
Unit Weight: 19
Effective Cohesion: 0
Effective Friction Angle: 30.8
Phi-B: 0
Name: SAND
Slope Stability Material Model: Mohr-Coulomb
Unit Weight: 19
Effective Cohesion: 0
Effective Friction Angle: 34.1
Phi-B: 0
Piezometric Line: 1
Name: SAND AND GRAVEL
Slope Stability Material Model: Mohr-Coulomb
Unit Weight: 20.5
Effective Cohesion: 0
Effective Friction Angle: 37.4
Phi-B: 0
Piezometric Line: 1
Seismic Undrained
21482114_Section C-C_1m higher_10%_MG.gsz
03/11/2022
1:720

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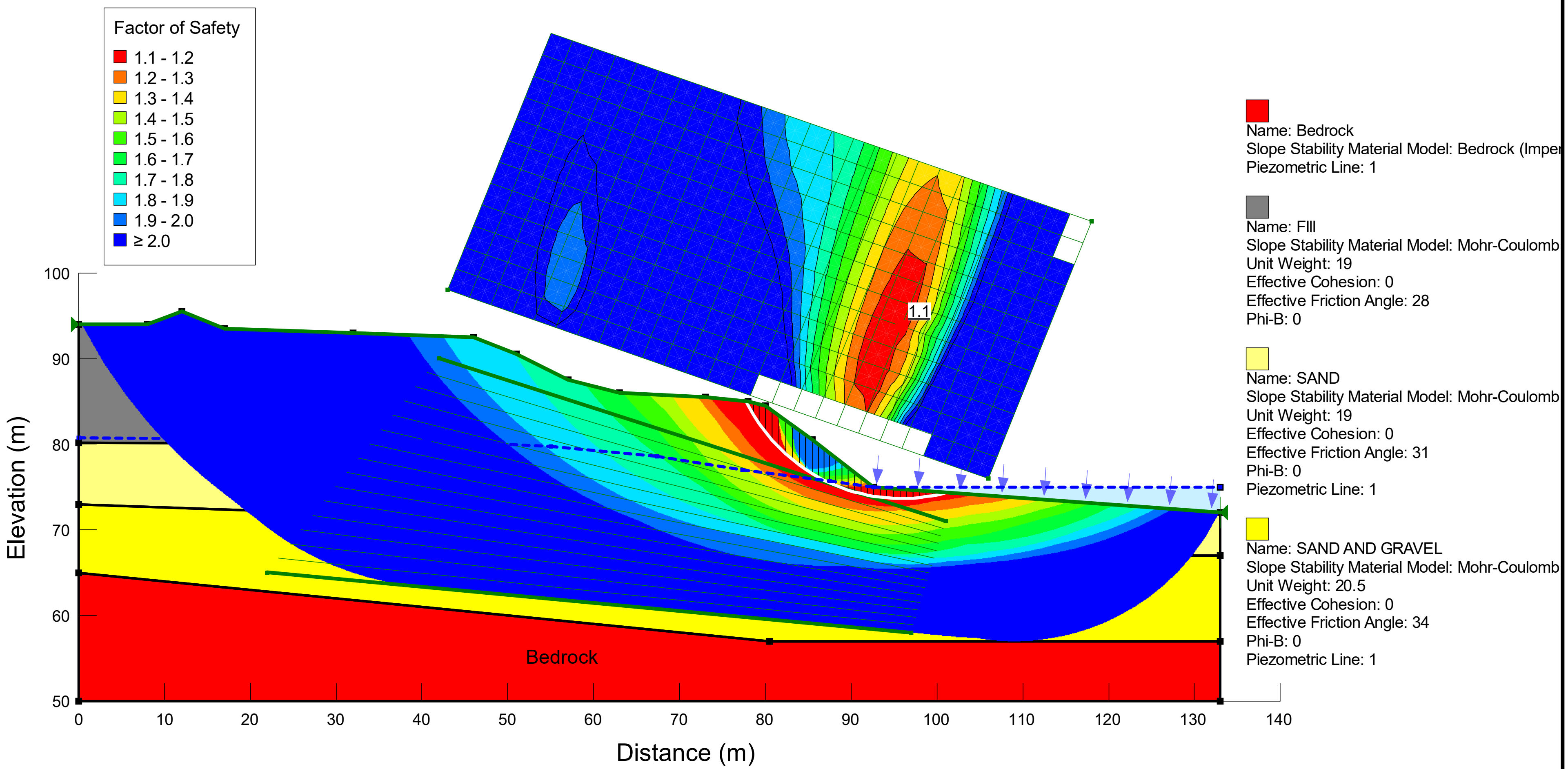
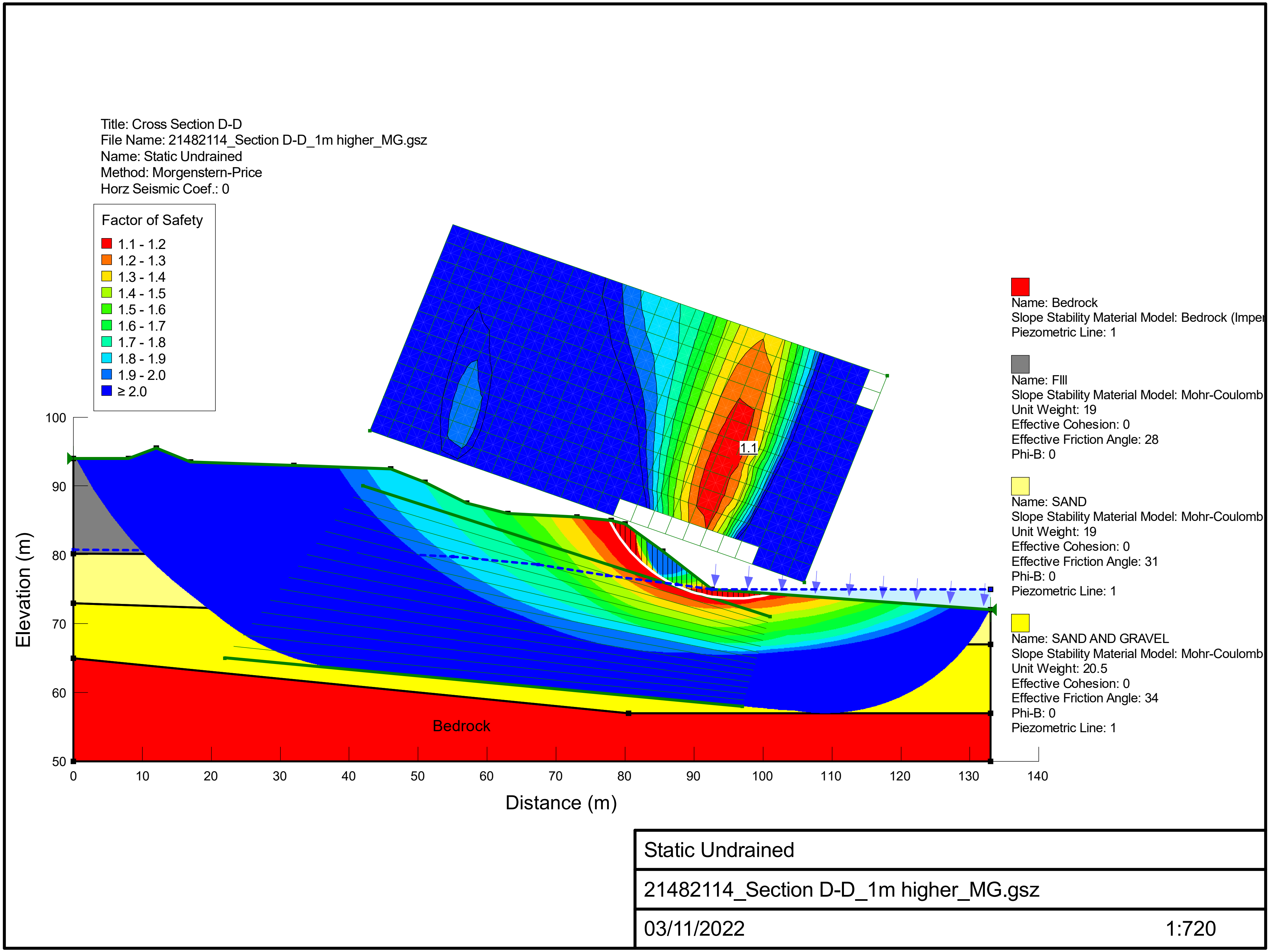
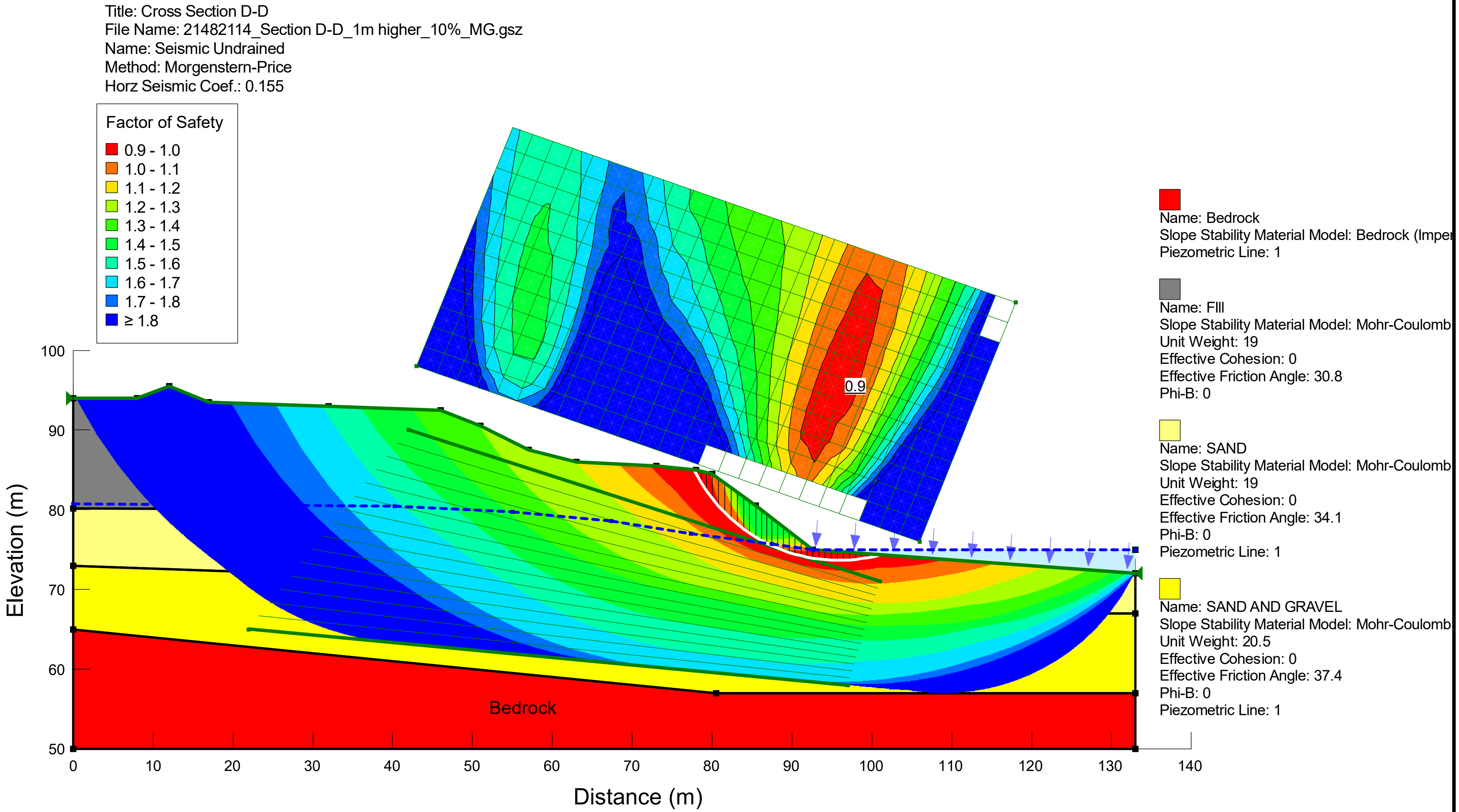
Name: Static Drained
Method: Morgenstern-Price
Horz Seismic Coef.: 0



Static Drained	
21482114_Section D-D_1m higher_MG.gsz	
03/11/2022	1:720


Title: Cross Section D-D
File Name: 21482114_Section D-D_1m higher_MG.gsz
Name: Static Undrained
Method: Morgenstern-Price
Horz Seismic Coef.: 0
Factor of Safety
1.1 - 1.2
1.2 - 1.3
1.3 - 1.4
1.4 - 1.5
1.5 - 1.6
1.6 - 1.7
1.7 - 1.8
1.8 - 1.9
1.9 - 2.0
≥ 2.0
1.1
Name: Bedrock
Slope Stability Material Model: Bedrock (Imper
Piezometric Line: 1
Name: FIll
Slope Stability Material Model: Mohr-Coulomb
Unit Weight: 19
Effective Cohesion: 0
Effective Friction Angle: 28
Phi-B: 0
Name: SAND
Slope Stability Material Model: Mohr-Coulomb
Unit Weight: 19
Effective Cohesion: 0
Effective Friction Angle: 31
Phi-B: 0
Piezometric Line: 1
Name: SAND AND GRAVEL
Slope Stability Material Model: Mohr-Coulomb
Unit Weight: 20.5
Effective Cohesion: 0
Effective Friction Angle: 34
Phi-B: 0
Piezometric Line: 1
Bedrock
Elevation (m)
100
90
80
70
60
50
0
10
20
30
40
50
60
70
80
90
100
110
120
130
140
Distance (m)
Static Undrained
21482114_Section D-D_1m higher_MG.gsz
03/11/2022
1:720

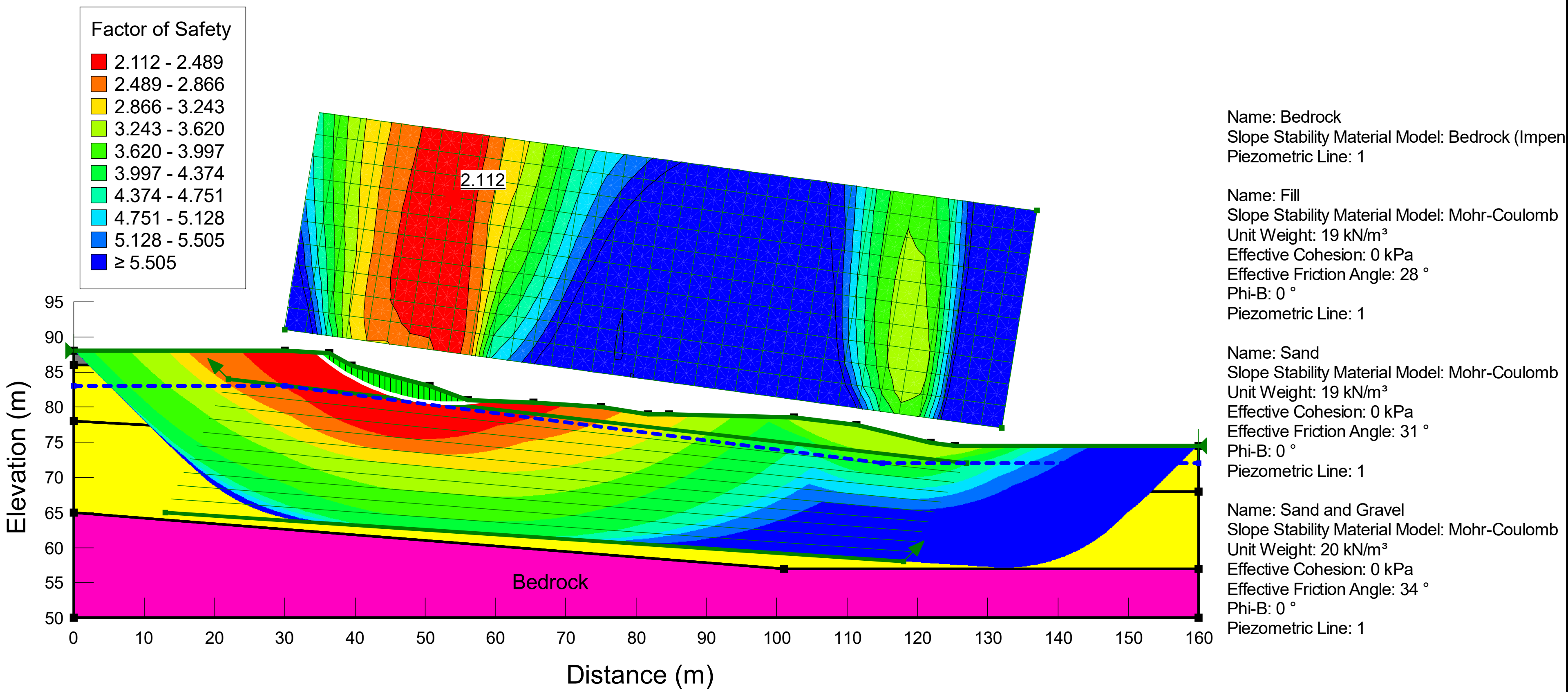


Seismic Undrained

21482114_Section D-D_1m higher_10%_MG.gsz

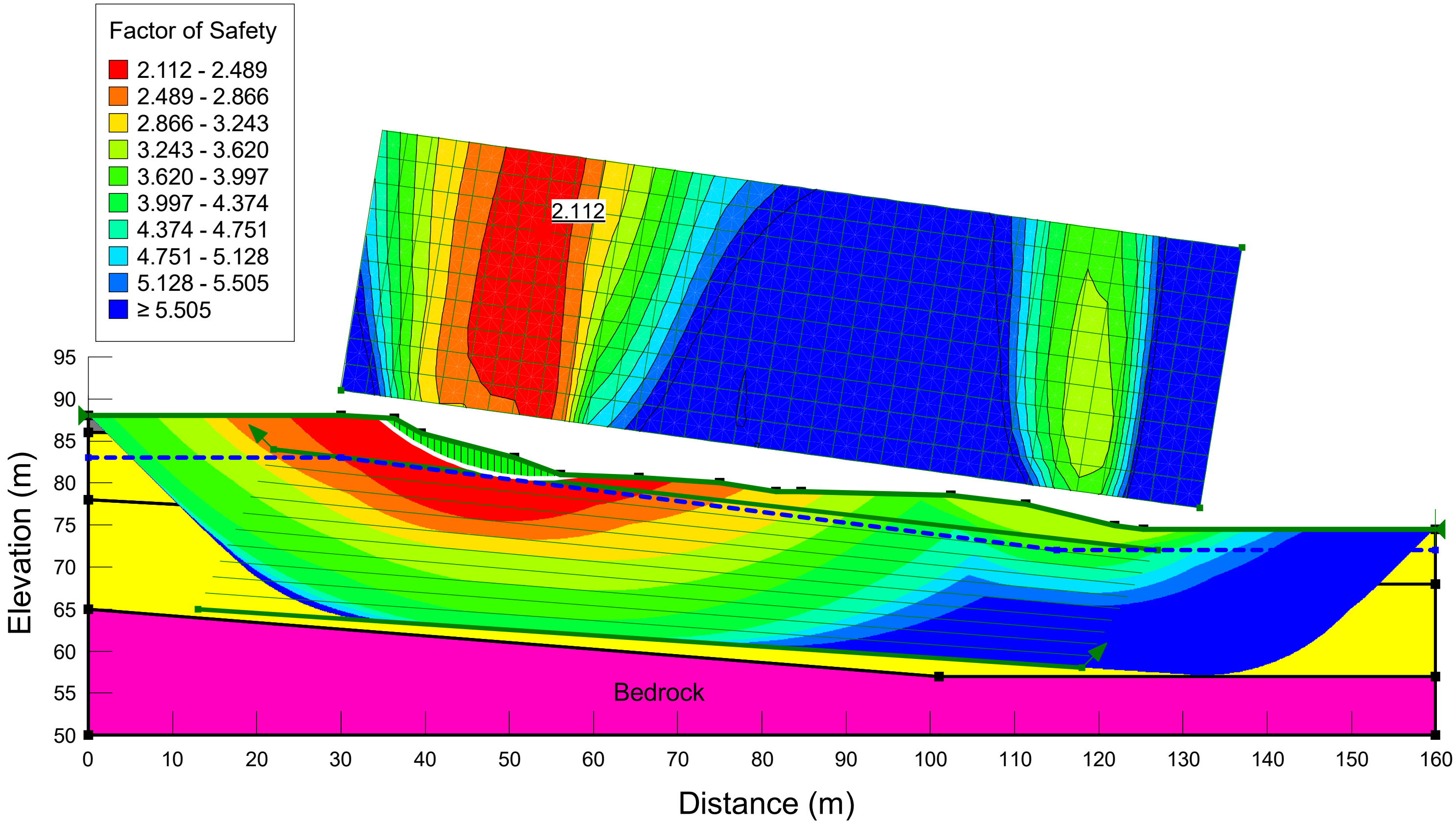
03/11/2022 1:720

Title: Taggart/St Mary Site/Ottawa
File Name: 21482114_Section E-E_1m higher_MG.gsz
Name: 4.1 Static Drained
Method: Morgenstern-Price
Horz Seismic Coef.: 0



4.1 Static Drained	
21482114_Section E-E_1m higher_MG.gsz	
03/11/2022	1:880

Title: Taggart/St Mary Site/Ottawa
File Name: 21482114_Section E-E_1m higher_MG.gsz
Name: 4.2 Static Undrained
Method: Morgenstern-Price
Horz Seismic Coef.: 0



Name: Bedrock
Slope Stability Material Model: Bedrock (Impen
Piezometric Line: 1

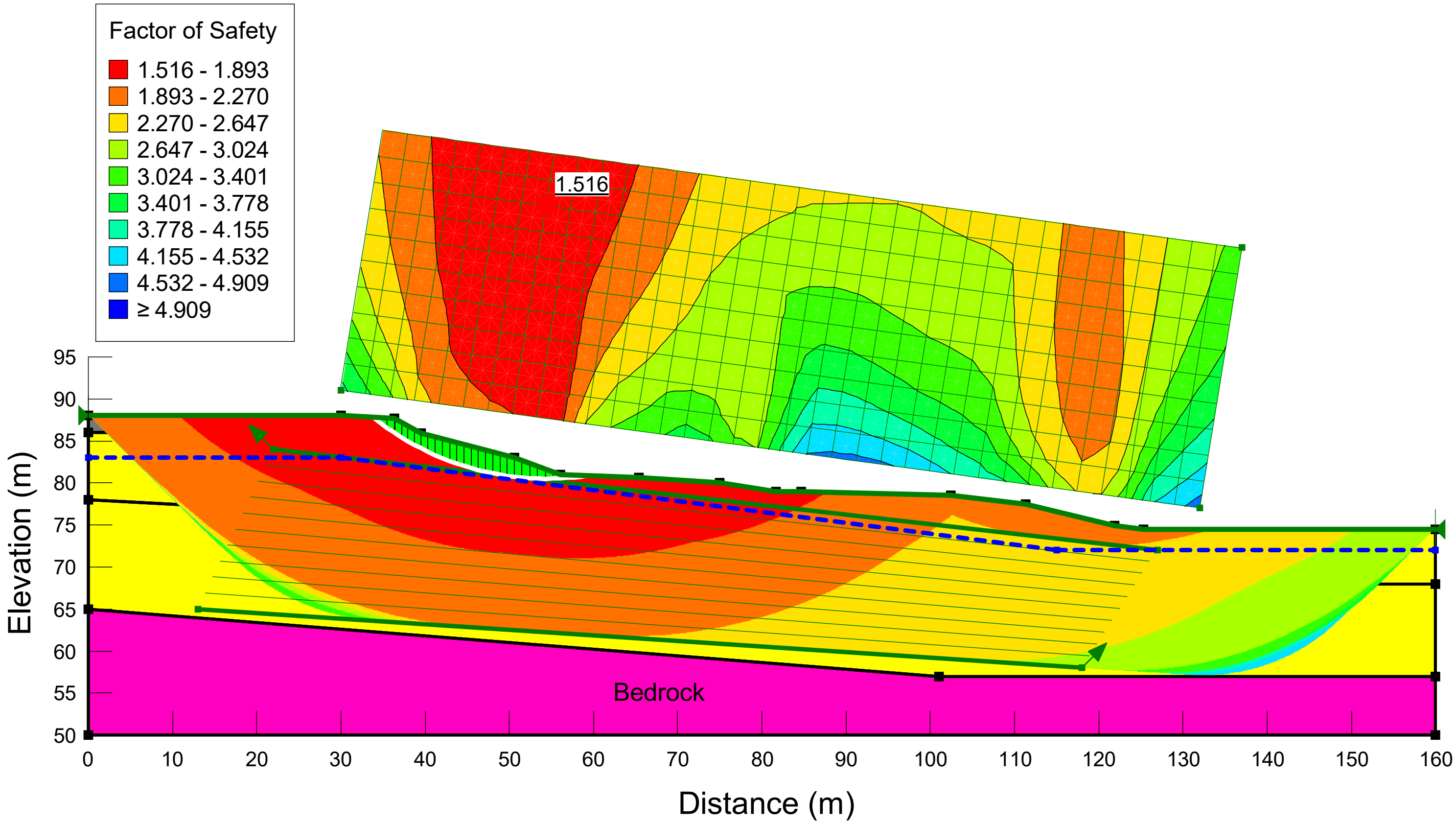
Name: Fill
Slope Stability Material Model: Mohr-Coulomb
Unit Weight: 19 kN/m³
Effective Cohesion: 0 kPa
Effective Friction Angle: 28 °
Phi-B: 0 °
Piezometric Line: 1

Name: Sand
Slope Stability Material Model: Mohr-Coulomb
Unit Weight: 19 kN/m³
Effective Cohesion: 0 kPa
Effective Friction Angle: 31 °
Phi-B: 0 °
Piezometric Line: 1

Name: Sand and Gravel
Slope Stability Material Model: Mohr-Coulomb
Unit Weight: 20 kN/m³
Effective Cohesion: 0 kPa
Effective Friction Angle: 34 °
Phi-B: 0 °
Piezometric Line: 1

4.2 Static Undrained	
21482114_Section E-E_1m higher_MG.gsz	
03/11/2022	1:880

Title: Taggart/St Mary Site/Ottawa
File Name: 21482114_Section E-E_1m higher_10%_MG.gsz
Name: 4.3 Seismic Undrained
Method: Morgenstern-Price
Horz Seismic Coef.: 0.155



Name: Bedrock
Slope Stability Material Model: Bedrock (Impen
Piezometric Line: 1

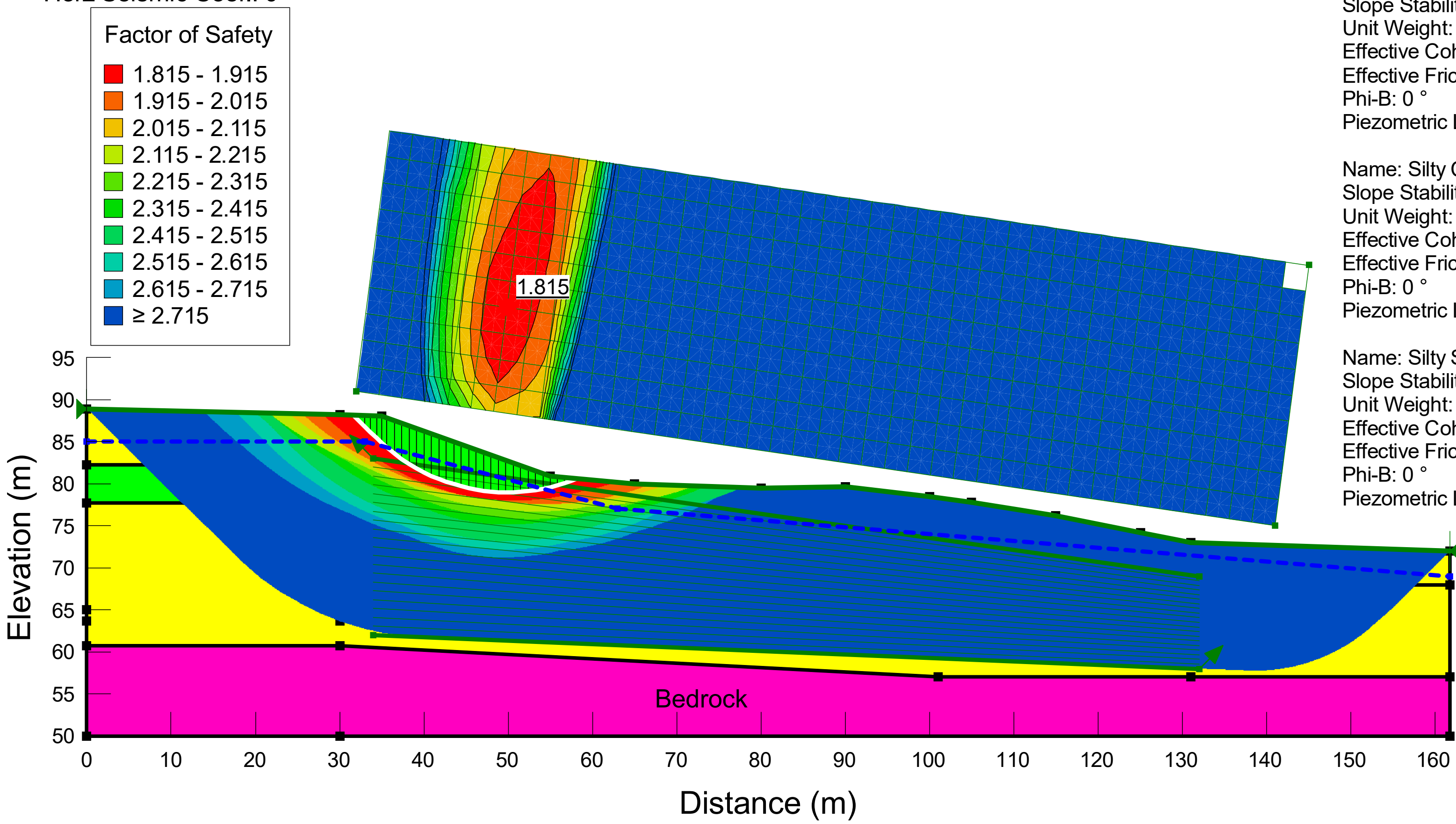
Name: Fill
Slope Stability Material Model: Mohr-Coulomb
Unit Weight: 19 kN/m³
Effective Cohesion: 0 kPa
Effective Friction Angle: 30.8 °
Phi-B: 0 °
Piezometric Line: 1

Name: Sand
Slope Stability Material Model: Mohr-Coulomb
Unit Weight: 19 kN/m³
Effective Cohesion: 0 kPa
Effective Friction Angle: 34.1 °
Phi-B: 0 °
Piezometric Line: 1

Name: Sand and Gravel
Slope Stability Material Model: Mohr-Coulomb
Unit Weight: 20 kN/m³
Effective Cohesion: 0 kPa
Effective Friction Angle: 37.4 °
Phi-B: 0 °
Piezometric Line: 1

4.3 Seismic Undrained	
21482114_Section E-E_1m higher_10%_MG.gsz	
03/11/2022	1:880

Title: Taggart/St Mary Site/Ottawa
File Name: 21482114_Section F-F_1m higher_MG.gsz
Name: 1.1 Static Drained
Method: Morgenstern-Price
Horz Seismic Coef.: 0



Name: Bedrock
Slope Stability Material Model: Bedrock (Impenetra
Piezometric Line: 1

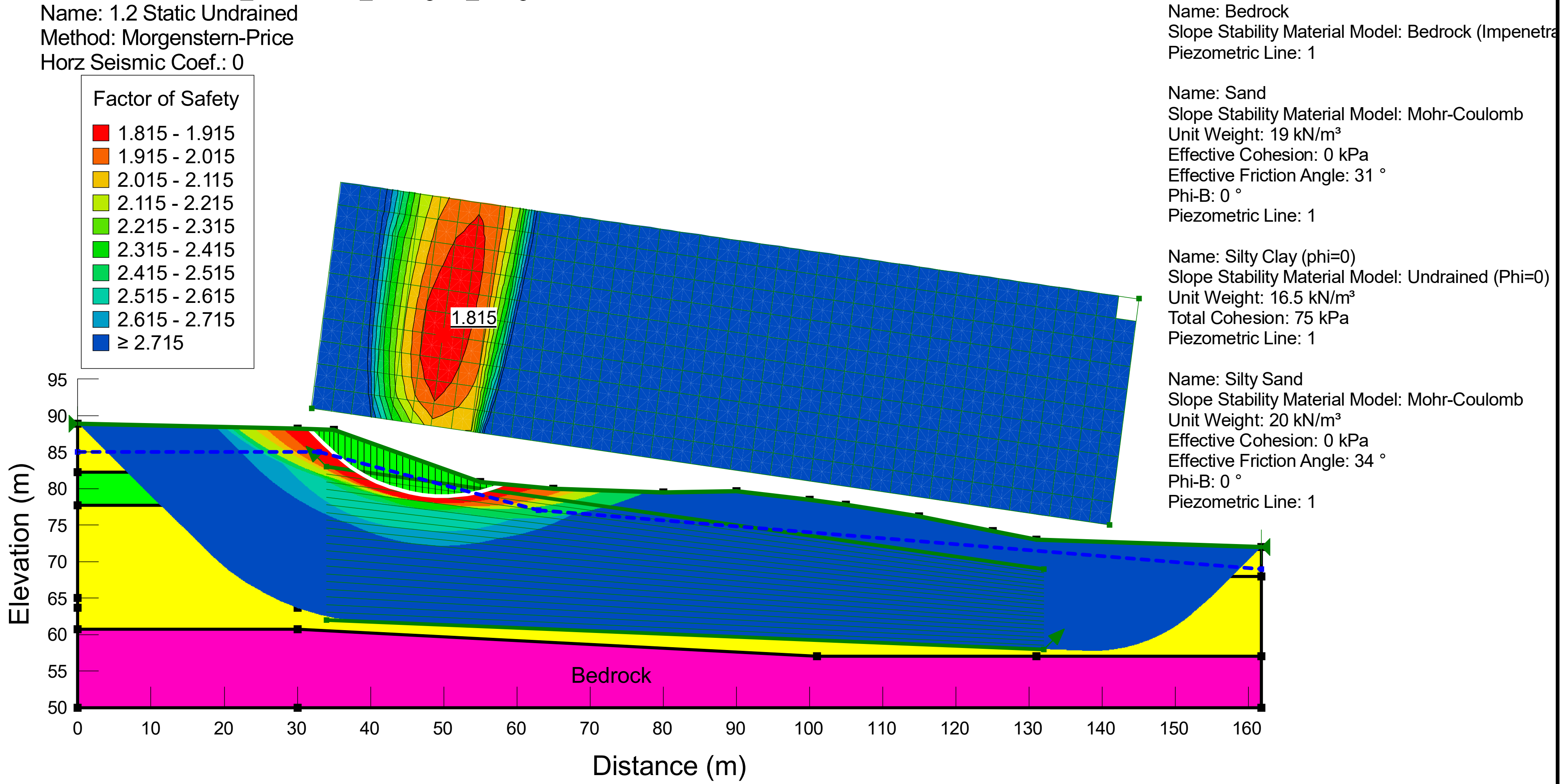
Name: Sand
Slope Stability Material Model: Mohr-Coulomb
Unit Weight: 19 kN/m³
Effective Cohesion: 0 kPa
Effective Friction Angle: 31 °
Phi-B: 0 °
Piezometric Line: 1

Name: Silty Clay
Slope Stability Material Model: Mohr-Coulomb
Unit Weight: 16.5 kN/m³
Effective Cohesion: 5 kPa
Effective Friction Angle: 35 °
Phi-B: 0 °
Piezometric Line: 1

Name: Silty Sand
Slope Stability Material Model: Mohr-Coulomb
Unit Weight: 20 kN/m³
Effective Cohesion: 0 kPa
Effective Friction Angle: 34 °
Phi-B: 0 °
Piezometric Line: 1

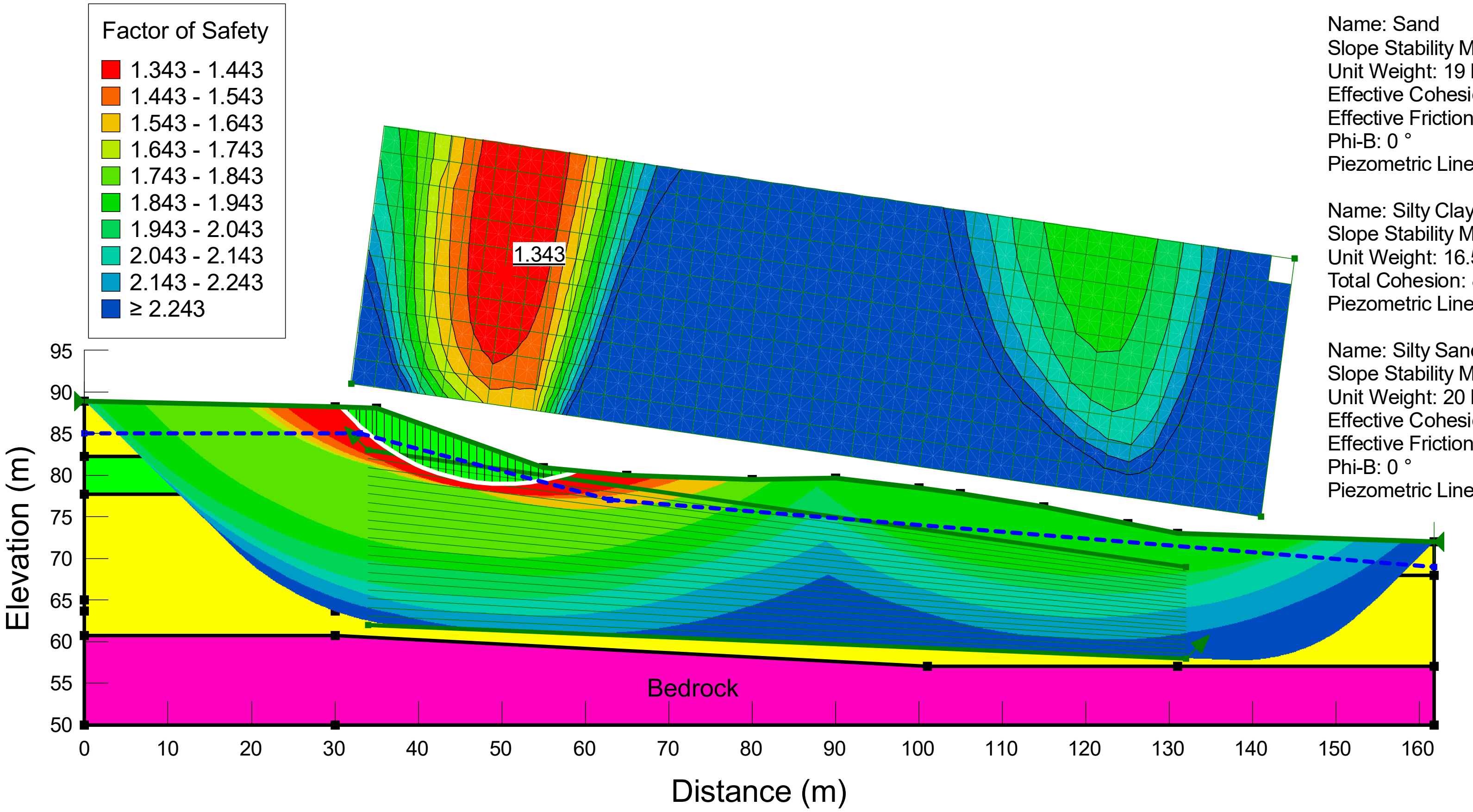
1.1 Static Drained	
21482114_Section F-F_1m higher_MG.gsz	
03/11/2022	1:840

Title: Taggart/St Mary Site/Ottawa
File Name: 21482114_Section F-F_1m higher_MG.gsz
Name: 1.2 Static Undrained
Method: Morgenstern-Price
Horz Seismic Coef.: 0



1.2 Static Undrained	
21482114_Section F-F_1m higher_MG.gsz	
03/11/2022	1:840

Title: Taggart/St Mary Site/Ottawa
File Name: 21482114_Section F-F_1m higher_10%_MG.gsz
Name: 1.3 Seismic Undrained
Method: Morgenstern-Price
Horz Seismic Coef.: 0.155



Name: Bedrock
Slope Stability Material Model: Bedrock (Impenetra
Piezometric Line: 1

Name: Sand
Slope Stability Material Model: Mohr-Coulomb
Unit Weight: 19 kN/m³
Effective Cohesion: 0 kPa
Effective Friction Angle: 34.1 °
Phi-B: 0 °
Piezometric Line: 1

Name: Silty Clay (phi=0)
Slope Stability Material Model: Undrained (Phi=0)
Unit Weight: 16.5 kN/m³
Total Cohesion: 82.5 kPa
Piezometric Line: 1

Name: Silty Sand
Slope Stability Material Model: Mohr-Coulomb
Unit Weight: 20 kN/m³
Effective Cohesion: 0 kPa
Effective Friction Angle: 37.4 °
Phi-B: 0 °
Piezometric Line: 1

1.3 Seismic Undrained	
21482114_Section F-F_1m higher_10%_MG.gsz	
03/11/2022	1:840

Title: Taggart/St Mary Site/Ottawa
File Name: 21482114_Section G-G_1m higher_MG.gsz
Name: 1.1 Static Drained
Method: Morgenstern-Price
Horz Seismic Coef.: 0

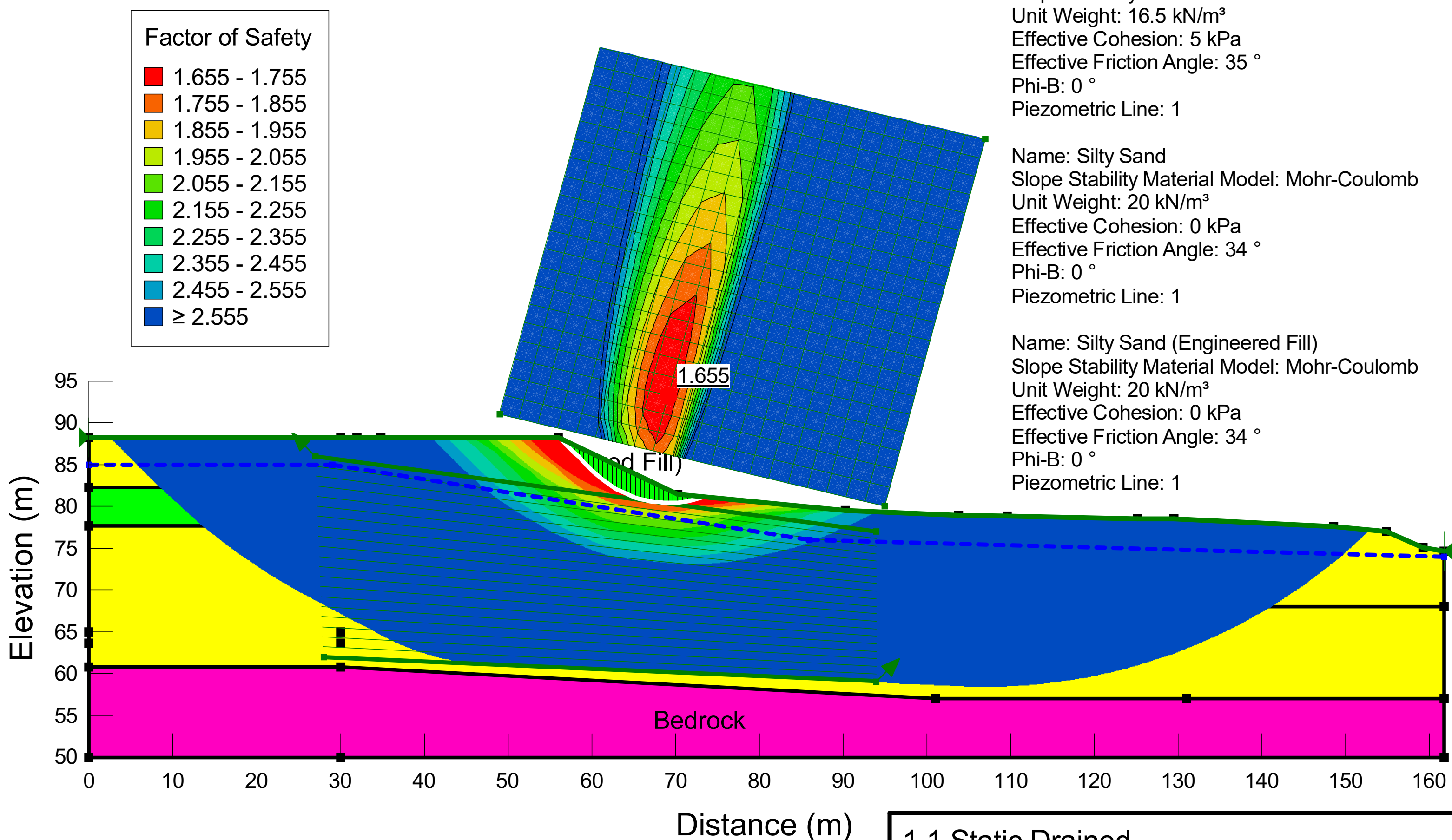
Name: Bedrock
Slope Stability Material Model: Bedrock (Impenetrable)
Piezometric Line: 1

Name: Sand
Slope Stability Material Model: Mohr-Coulomb
Unit Weight: 19 kN/m³
Effective Cohesion: 0 kPa
Effective Friction Angle: 31 °
Phi-B: 0 °
Piezometric Line: 1

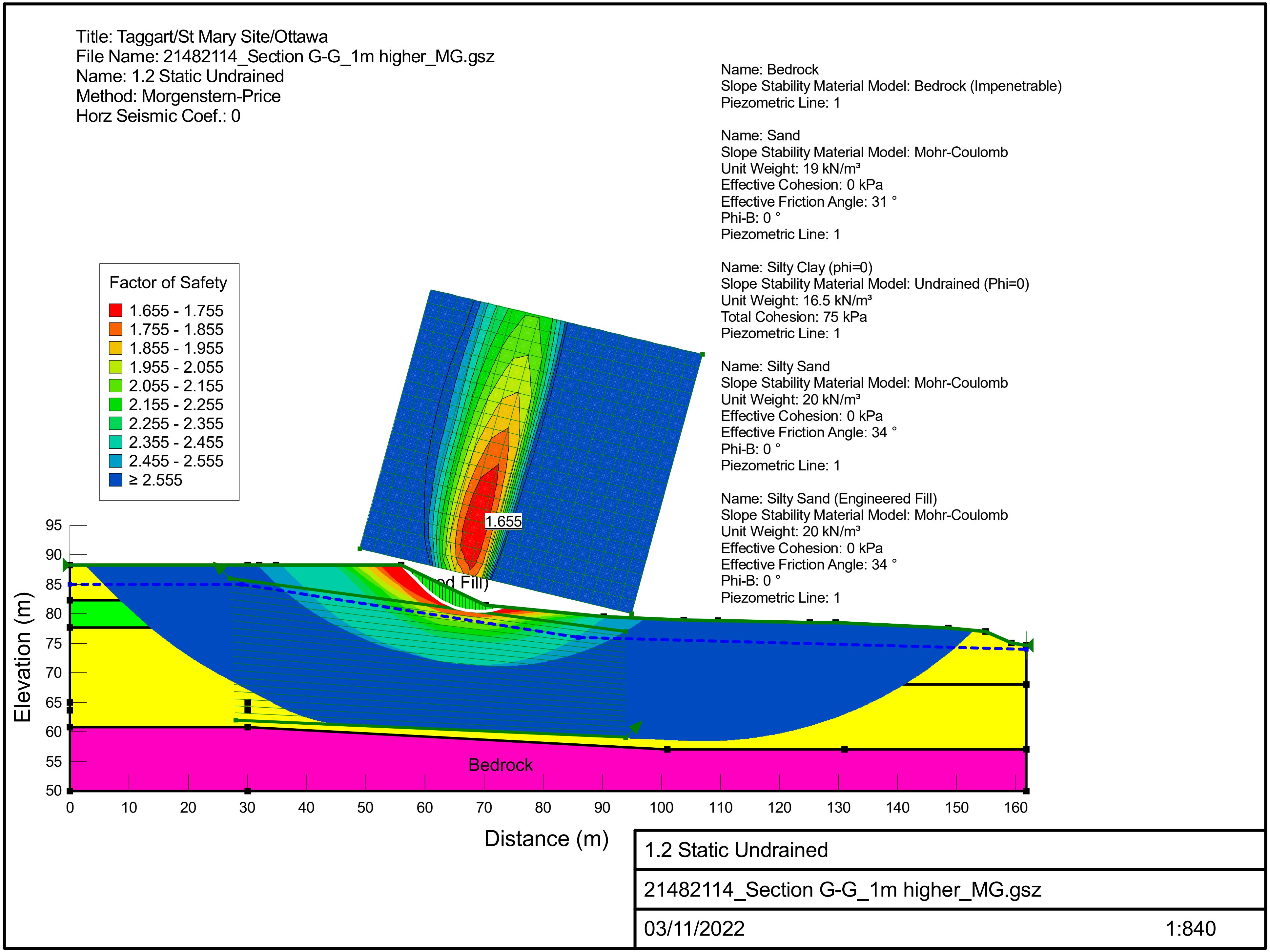
Name: Silty Clay
Slope Stability Material Model: Mohr-Coulomb
Unit Weight: 16.5 kN/m³
Effective Cohesion: 5 kPa
Effective Friction Angle: 35 °
Phi-B: 0 °
Piezometric Line: 1

Name: Silty Sand
Slope Stability Material Model: Mohr-Coulomb
Unit Weight: 20 kN/m³
Effective Cohesion: 0 kPa
Effective Friction Angle: 34 °
Phi-B: 0 °
Piezometric Line: 1

Name: Silty Sand (Engineered Fill)
Slope Stability Material Model: Mohr-Coulomb
Unit Weight: 20 kN/m³
Effective Cohesion: 0 kPa
Effective Friction Angle: 34 °
Phi-B: 0 °
Piezometric Line: 1



1.1 Static Drained	
21482114_Section G-G_1m higher_MG.gsz	
03/11/2022	1:840


Title: Taggart/St Mary Site/Ottawa
File Name: 21482114_Section G-G_1m higher_MG.gsz
Name: 1.2 Static Undrained
Method: Morgenstern-Price
Horz Seismic Coef.: 0
Name: Bedrock
Slope Stability Material Model: Bedrock (Impenetrable)
Piezometric Line: 1
Name: Sand
Slope Stability Material Model: Mohr-Coulomb
Unit Weight: 19 kN/m³
Effective Cohesion: 0 kPa
Effective Friction Angle: 31 °
Phi-B: 0 °
Piezometric Line: 1
Name: Silty Clay (phi=0)
Slope Stability Material Model: Undrained (Phi=0)
Unit Weight: 16.5 kN/m³
Total Cohesion: 75 kPa
Piezometric Line: 1
Name: Silty Sand
Slope Stability Material Model: Mohr-Coulomb
Unit Weight: 20 kN/m³
Effective Cohesion: 0 kPa
Effective Friction Angle: 34 °
Phi-B: 0 °
Piezometric Line: 1
Name: Silty Sand (Engineered Fill)
Slope Stability Material Model: Mohr-Coulomb
Unit Weight: 20 kN/m³
Effective Cohesion: 0 kPa
Effective Friction Angle: 34 °
Phi-B: 0 °
Piezometric Line: 1
Factor of Safety
1.655 - 1.755
1.755 - 1.855
1.855 - 1.955
1.955 - 2.055
2.055 - 2.155
2.155 - 2.255
2.255 - 2.355
2.355 - 2.455
2.455 - 2.555
≥ 2.555
1.655
Bedrock
Elevation (m)
Distance (m)
95
90
85
80
75
70
65
60
55
50
0
10
20
30
40
50
60
70
80
90
100
110
120
130
140
150
160
1.2 Static Undrained
21482114_Section G-G_1m higher_MG.gsz
03/11/2022
1:840

Title: Taggart/St Mary Site/Ottawa
File Name: 21482114_Section G-G_1m higher_10%_MG.gsz
Name: 1.3 Seismic Undrained
Method: Morgenstern-Price
Horz Seismic Coef.: 0.155

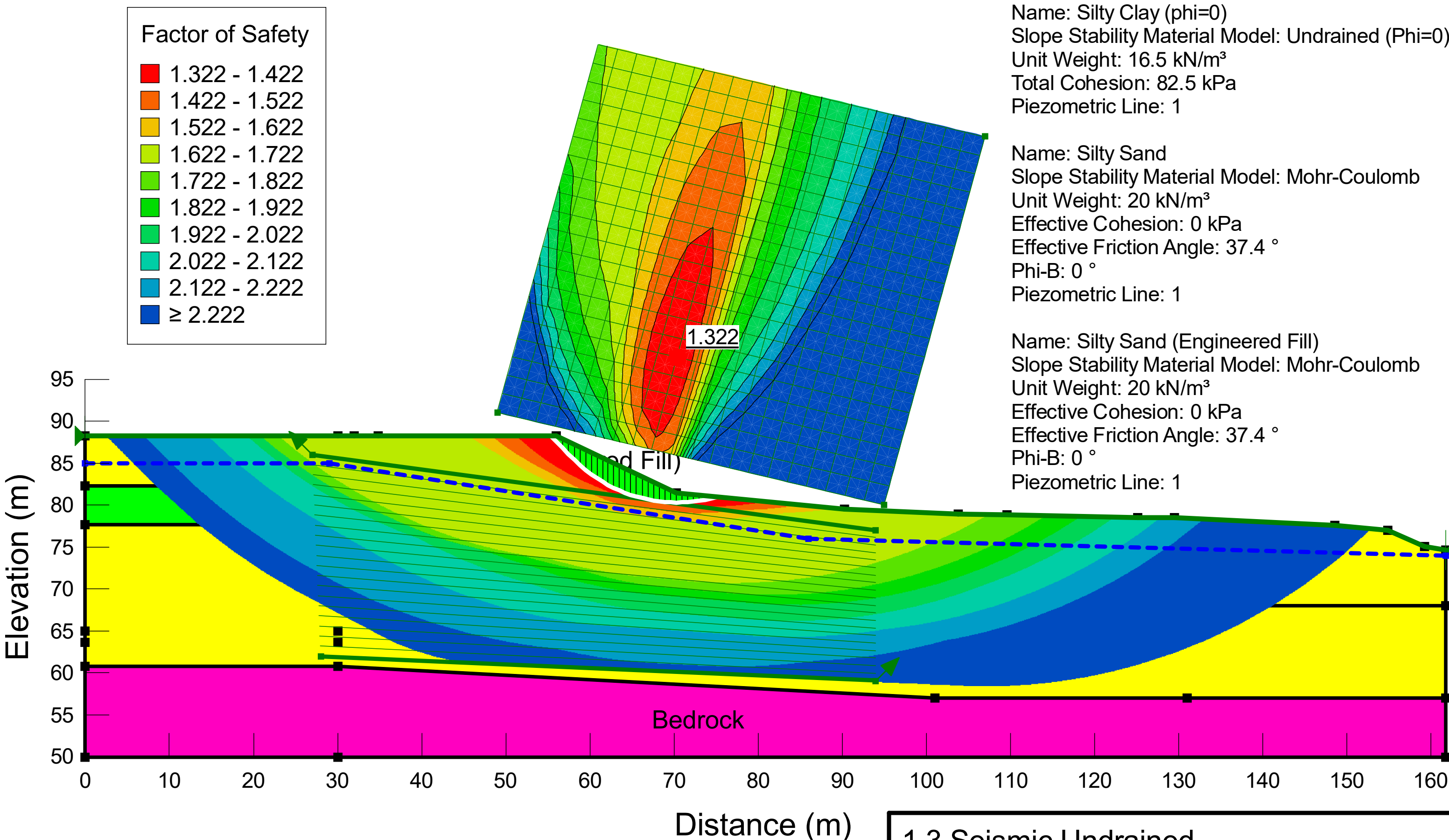
Name: Bedrock
Slope Stability Material Model: Bedrock (Impenetrable)
Piezometric Line: 1

Name: Sand
Slope Stability Material Model: Mohr-Coulomb
Unit Weight: 19 kN/m³
Effective Cohesion: 0 kPa
Effective Friction Angle: 34.1 °
Phi-B: 0 °
Piezometric Line: 1

Name: Silty Clay (phi=0)
Slope Stability Material Model: Undrained (Phi=0)
Unit Weight: 16.5 kN/m³
Total Cohesion: 82.5 kPa
Piezometric Line: 1

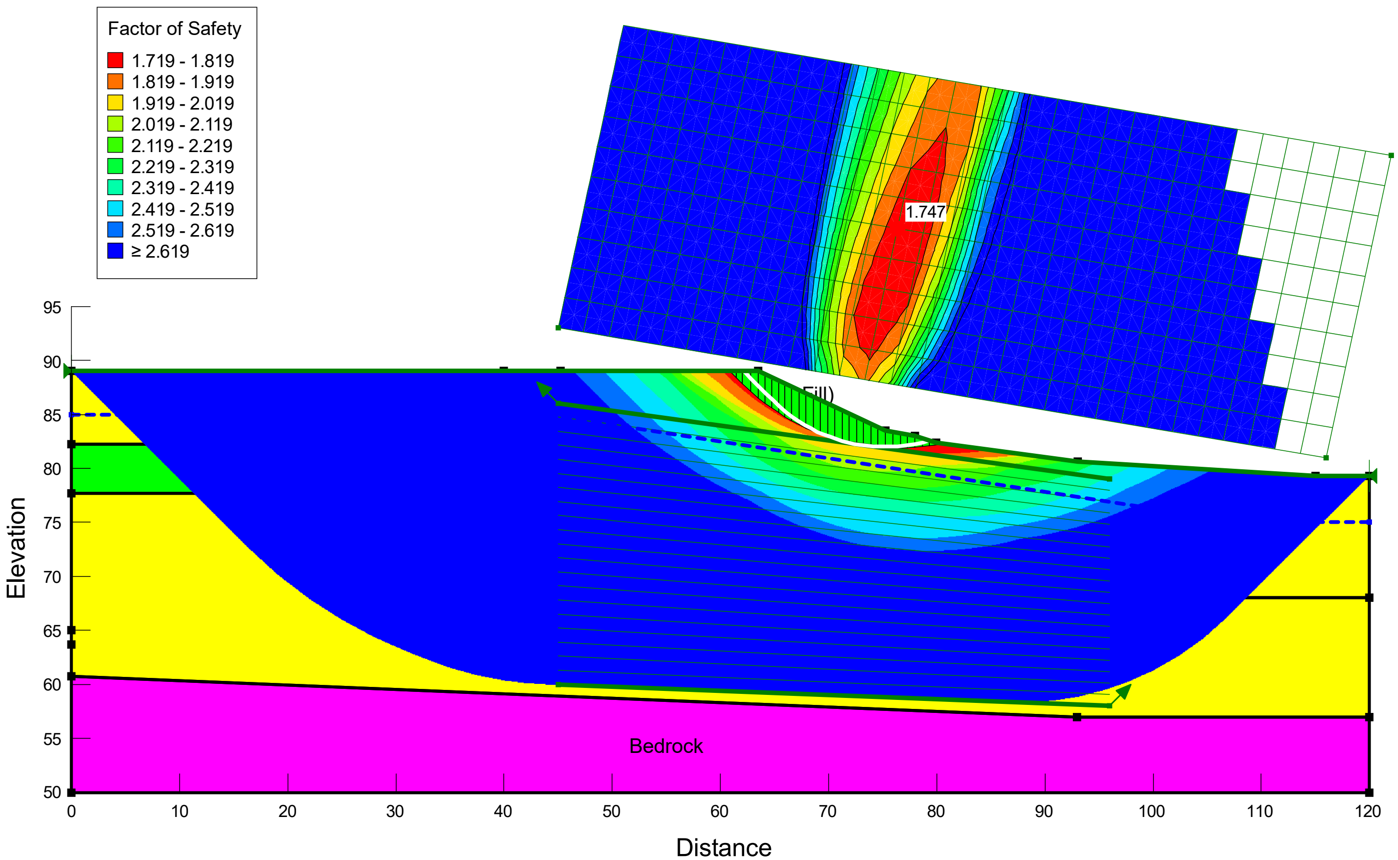
Name: Silty Sand
Slope Stability Material Model: Mohr-Coulomb
Unit Weight: 20 kN/m³
Effective Cohesion: 0 kPa
Effective Friction Angle: 37.4 °
Phi-B: 0 °
Piezometric Line: 1

Name: Silty Sand (Engineered Fill)
Slope Stability Material Model: Mohr-Coulomb
Unit Weight: 20 kN/m³
Effective Cohesion: 0 kPa
Effective Friction Angle: 37.4 °
Phi-B: 0 °
Piezometric Line: 1



1.3 Seismic Undrained	
21482114_Section G-G_1m higher_10%_MG.gsz	
03/11/2022	1:840

Title: Slope Cross Section H-H
File Name: 21482114_Section H-H_1m higher_MG.gsz
Name: 2.1 Static Drained
Method: Morgenstern-Price
Horz Seismic Coef.: 0
Scale: 1:700



Name: Bedrock
Slope Stability Material Model: Bedrock (Impenetrabl
Piezometric Line: 1

Name: Sand
Slope Stability Material Model: Mohr-Coulomb
Unit Weight: 19 kN/m³
Effective Cohesion: 0 kPa
Effective Friction Angle: 31 °
Phi-B: 0 °
Piezometric Line: 1

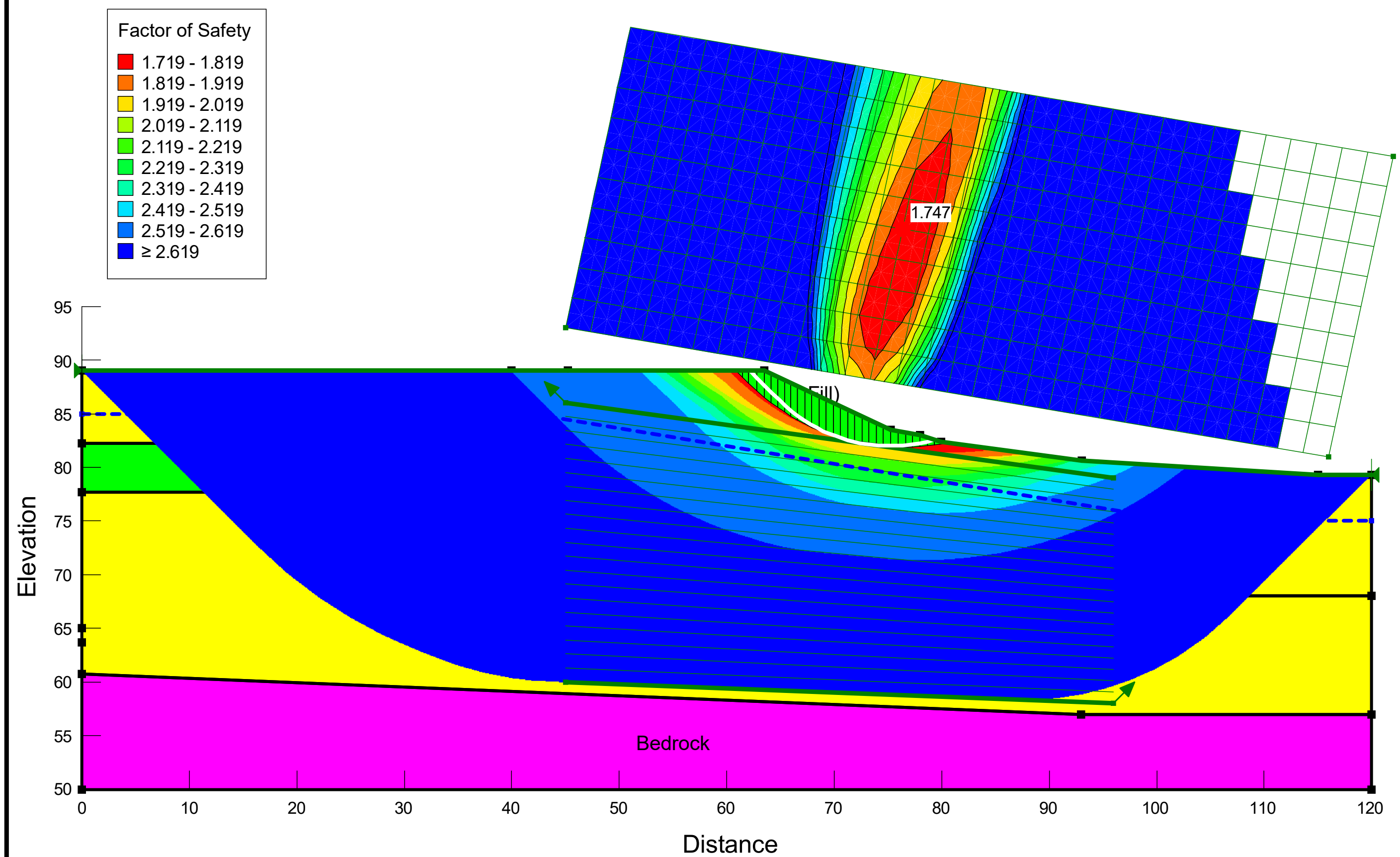
Name: Silty Clay
Slope Stability Material Model: Mohr-Coulomb
Unit Weight: 16.5 kN/m³
Effective Cohesion: 5 kPa
Effective Friction Angle: 35 °
Phi-B: 0 °
Piezometric Line: 1

Name: Silty Sand
Slope Stability Material Model: Mohr-Coulomb
Unit Weight: 20 kN/m³
Effective Cohesion: 0 kPa
Effective Friction Angle: 34 °
Phi-B: 0 °
Piezometric Line: 1

Name: Silty Sand (Engineered Fill)
Slope Stability Material Model: Mohr-Coulomb
Unit Weight: 20 kN/m³
Effective Cohesion: 0 kPa
Effective Friction Angle: 34 °
Phi-B: 0 °
Piezometric Line: 1

2.1 Static Drained	
21482114_Section H-H_1m higher_MG.gsz	
03/11/2022	1:700

Title: Slope Cross Section H-H
File Name: 21482114_Section H-H_1m higher_MG.gsz
Name: 2.2 Static Undrained
Method: Morgenstern-Price
Horz Seismic Coef.: 0
Scale: 1:700



Name: Bedrock
Slope Stability Material Model: Bedrock (Impenetrabl
Piezometric Line: 1

Name: Sand
Slope Stability Material Model: Mohr-Coulomb
Unit Weight: 19 kN/m³
Effective Cohesion: 0 kPa
Effective Friction Angle: 31 °
Phi-B: 0 °
Piezometric Line: 1

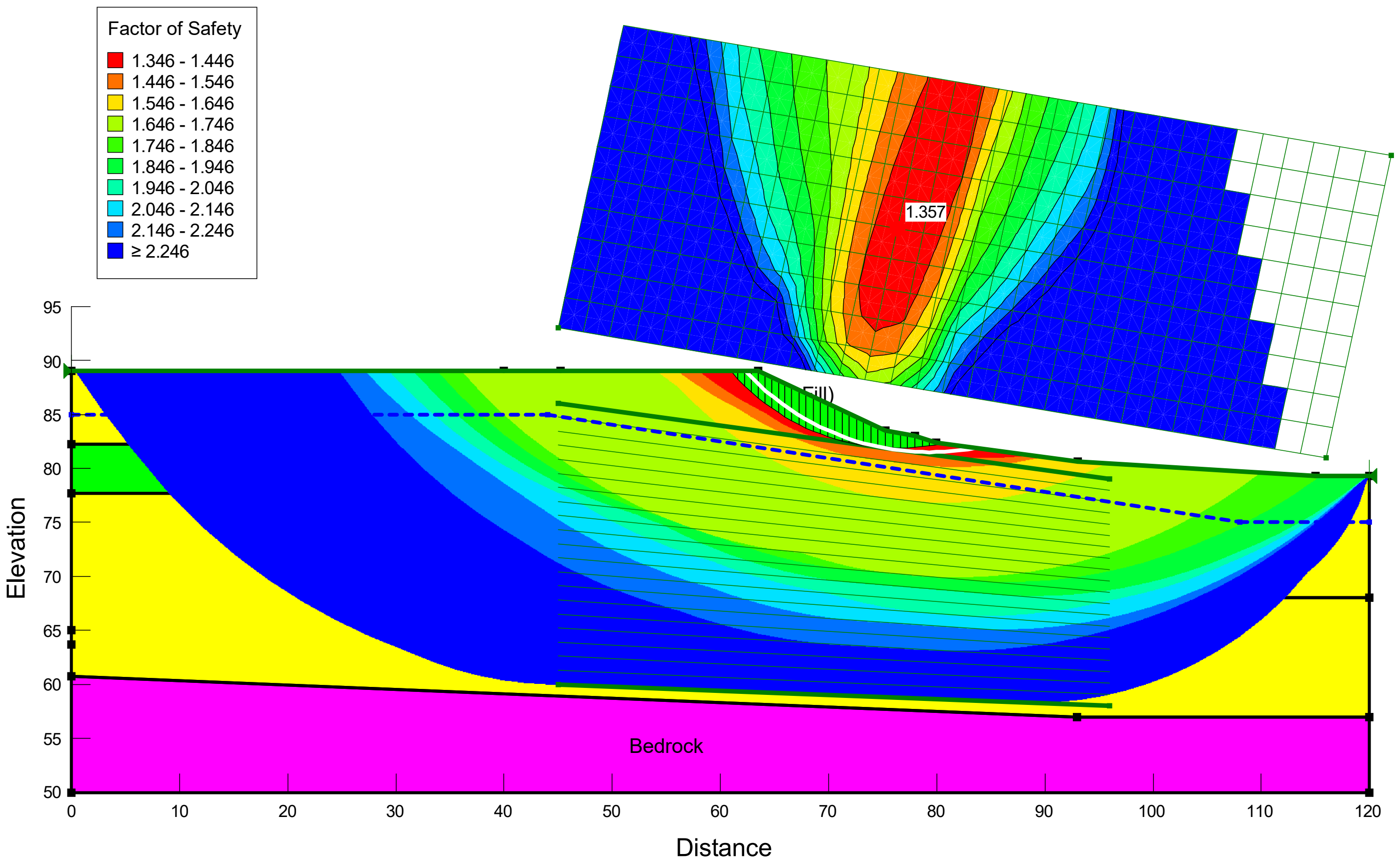
Name: Silty Clay (phi=0)
Slope Stability Material Model: Undrained (Phi=0)
Unit Weight: 16.5 kN/m³
Total Cohesion: 75 kPa
Piezometric Line: 1

Name: Silty Sand
Slope Stability Material Model: Mohr-Coulomb
Unit Weight: 20 kN/m³
Effective Cohesion: 0 kPa
Effective Friction Angle: 34 °
Phi-B: 0 °
Piezometric Line: 1

Name: Silty Sand (Engineered Fill)
Slope Stability Material Model: Mohr-Coulomb
Unit Weight: 20 kN/m³
Effective Cohesion: 0 kPa
Effective Friction Angle: 34 °
Phi-B: 0 °
Piezometric Line: 1

2.2 Static Undrained	
21482114_Section H-H_1m higher_MG.gsz	
03/11/2022	1:700

Title: Slope Cross Section H-H
File Name: 21482114_Section H-H_1m higher_10%_MG.gsz
Name: 2.3 Seismic Undrained
Method: Morgenstern-Price
Horz Seismic Coef.: 0.155
Scale: 1:700



Name: Bedrock
Slope Stability Material Model: Bedrock (Impenetrabl
Piezometric Line: 1

Name: Sand
Slope Stability Material Model: Mohr-Coulomb
Unit Weight: 19 kN/m³
Effective Cohesion: 0 kPa
Effective Friction Angle: 34.1 °
Phi-B: 0 °
Piezometric Line: 1

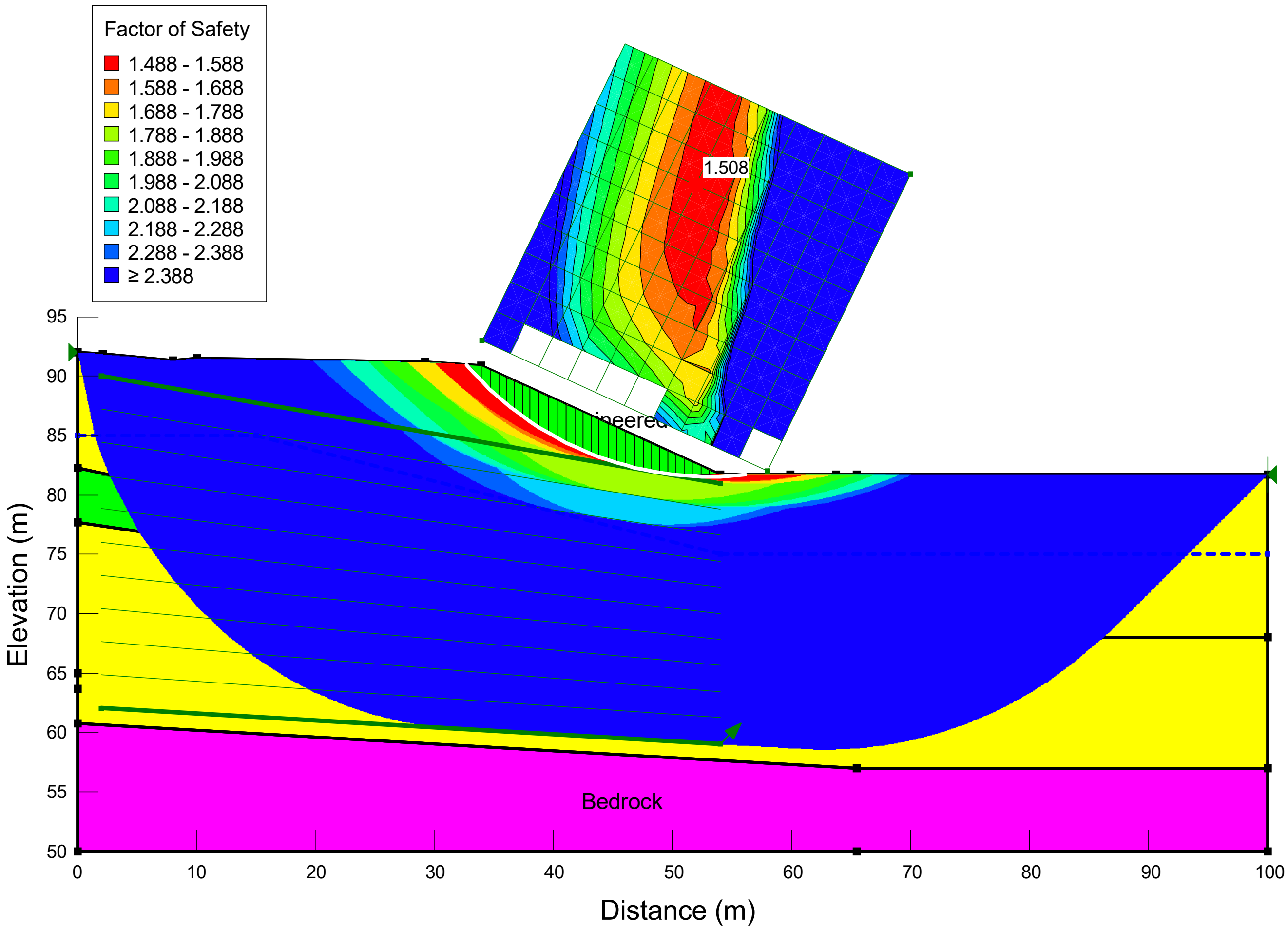
Name: Silty Clay (phi=0)
Slope Stability Material Model: Undrained (Phi=0)
Unit Weight: 16.5 kN/m³
Total Cohesion: 82.5 kPa
Piezometric Line: 1

Name: Silty Sand
Slope Stability Material Model: Mohr-Coulomb
Unit Weight: 20 kN/m³
Effective Cohesion: 0 kPa
Effective Friction Angle: 37.4 °
Phi-B: 0 °
Piezometric Line: 1

Name: Silty Sand (Engineered Fill)
Slope Stability Material Model: Mohr-Coulomb
Unit Weight: 20 kN/m³
Effective Cohesion: 0 kPa
Effective Friction Angle: 37.4 °
Phi-B: 0 °
Piezometric Line: 1

2.3 Seismic Undrained	
21482114_Section H-H_1m higher_10%_MG.gsz	
03/11/2022	1:700

Title: Cross Section I-I
File Name: 21482114_Section I-I_1m higher_MG.gsz
Name: 3.1 Static Drained
Method: Morgenstern-Price
Scale: 1:620
Horz Seismic Coef.: 0



Name: Bedrock
Slope Stability Material Model: Bedrock (Impenetrabl
Piezometric Line: 1

Name: Sand
Slope Stability Material Model: Mohr-Coulomb
Unit Weight: 19 kN/m³
Effective Cohesion: 0 kPa
Effective Friction Angle: 31 °
Phi-B: 0 °
Piezometric Line: 1

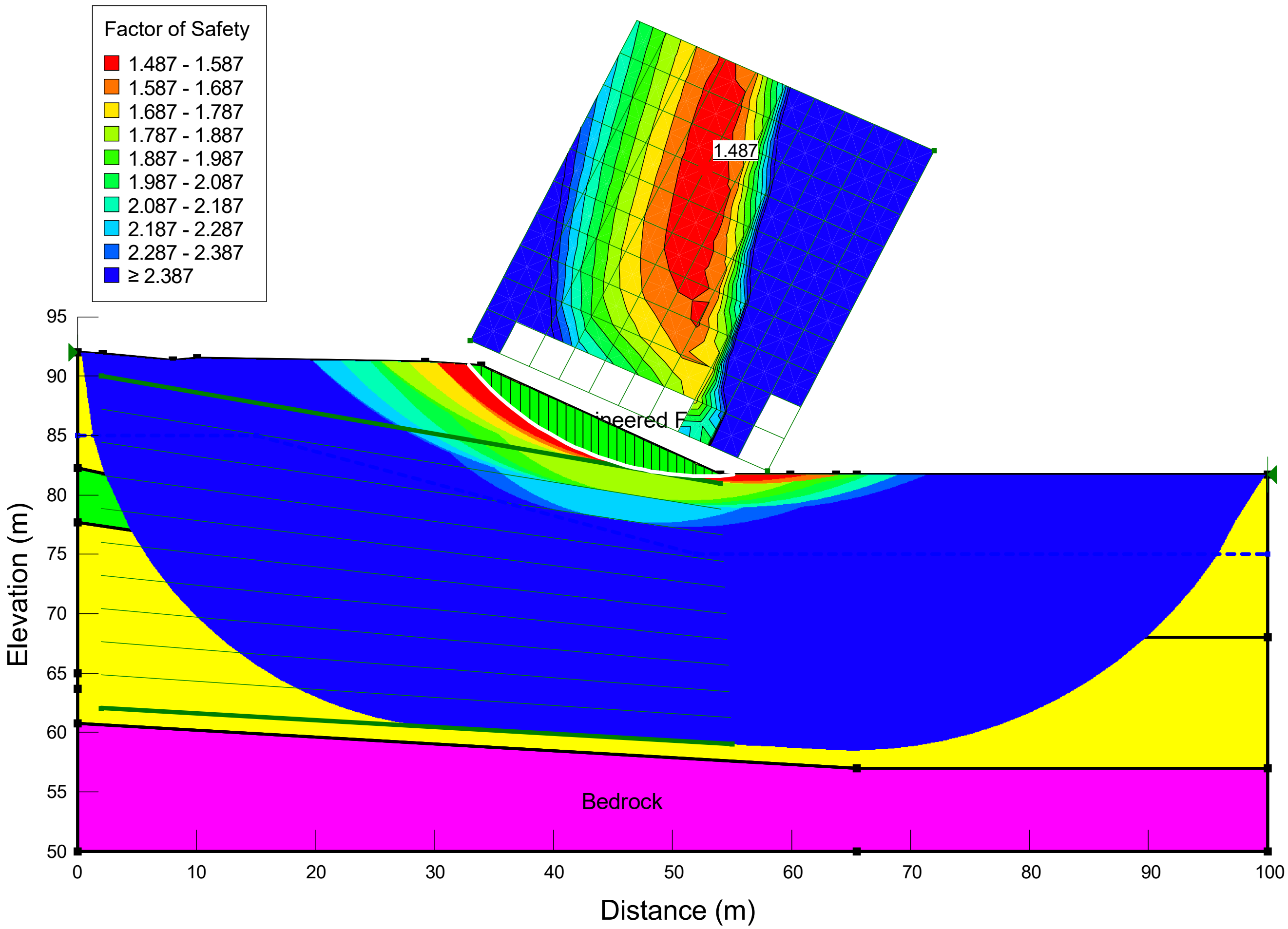
Name: Silty Clay
Slope Stability Material Model: Mohr-Coulomb
Unit Weight: 16.5 kN/m³
Effective Cohesion: 5 kPa
Effective Friction Angle: 35 °
Phi-B: 0 °
Piezometric Line: 1

Name: Silty Sand
Slope Stability Material Model: Mohr-Coulomb
Unit Weight: 20 kN/m³
Effective Cohesion: 0 kPa
Effective Friction Angle: 34 °
Phi-B: 0 °
Piezometric Line: 1

Name: Silty Sand (Engineered Fill)
Slope Stability Material Model: Mohr-Coulomb
Unit Weight: 20 kN/m³
Effective Cohesion: 0 kPa
Effective Friction Angle: 34 °
Phi-B: 0 °
Piezometric Line: 1

3.1 Static Drained	
21482114_Section I-I_1m higher_MG.gsz	
03/11/2022	1:620

Title: Cross Section I-I
File Name: 21482114_Section I-I_1m higher_MG.gsz
Name: 3.2 Static Undrained
Method: Morgenstern-Price
Scale: 1:620
Horz Seismic Coef.: 0



Name: Bedrock
Slope Stability Material Model: Bedrock (Impenetrabl
Piezometric Line: 1

Name: Sand
Slope Stability Material Model: Mohr-Coulomb
Unit Weight: 19 kN/m³
Effective Cohesion: 0 kPa
Effective Friction Angle: 31 °
Phi-B: 0 °
Piezometric Line: 1

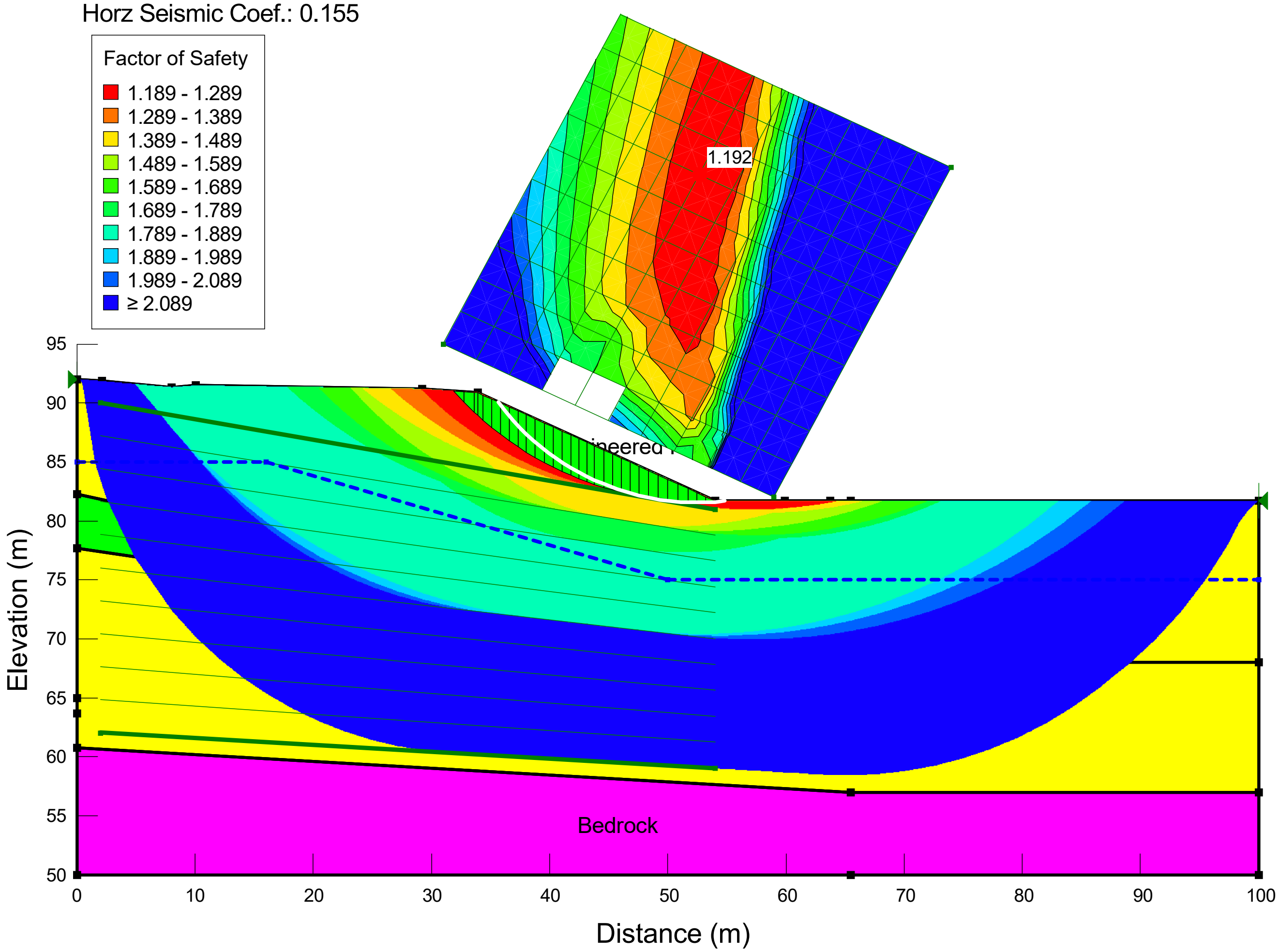
Name: Silty Clay (phi=0)
Slope Stability Material Model: Undrained (Phi=0)
Unit Weight: 16.5 kN/m³
Total Cohesion: 75 kPa
Piezometric Line: 1

Name: Silty Sand
Slope Stability Material Model: Mohr-Coulomb
Unit Weight: 20 kN/m³
Effective Cohesion: 0 kPa
Effective Friction Angle: 34 °
Phi-B: 0 °
Piezometric Line: 1

Name: Silty Sand (Engineered Fill)
Slope Stability Material Model: Mohr-Coulomb
Unit Weight: 20 kN/m³
Effective Cohesion: 0 kPa
Effective Friction Angle: 34 °
Phi-B: 0 °
Piezometric Line: 1

3.2 Static Undrained	
21482114_Section I-I_1m higher_MG.gsz	
03/11/2022	1:620

Title: Cross Section I-I
File Name: 21482114_Section I-I_1m higher_10%_MG.gsz
Name: 3.3 Seismic Undrained
Method: Morgenstern-Price
Scale: 1:620
Horz Seismic Coef.: 0.155



Name: Bedrock
Slope Stability Material Model: Bedrock (Impenetrabl
Piezometric Line: 1

Name: Sand
Slope Stability Material Model: Mohr-Coulomb
Unit Weight: 19 kN/m³
Effective Cohesion: 0 kPa
Effective Friction Angle: 34.1 °
Phi-B: 0 °
Piezometric Line: 1

Name: Silty Clay (phi=0)
Slope Stability Material Model: Undrained (Phi=0)
Unit Weight: 16.5 kN/m³
Total Cohesion: 82.5 kPa
Piezometric Line: 1

Name: Silty Sand
Slope Stability Material Model: Mohr-Coulomb
Unit Weight: 20 kN/m³
Effective Cohesion: 0 kPa
Effective Friction Angle: 37.4 °
Phi-B: 0 °
Piezometric Line: 1

Name: Silty Sand (Engineered Fill)
Slope Stability Material Model: Mohr-Coulomb
Unit Weight: 20 kN/m³
Effective Cohesion: 0 kPa
Effective Friction Angle: 37.4 °
Phi-B: 0 °
Piezometric Line: 1

3.3 Seismic Undrained
21482114_Section I-I_1m higher_10%_MG.gsz
03/11/2022 1:620

APPENDIX G

Technical Memorandum – Geophysics (2023)

PRESENTATION OF SITE INVESTIGATION RESULTS

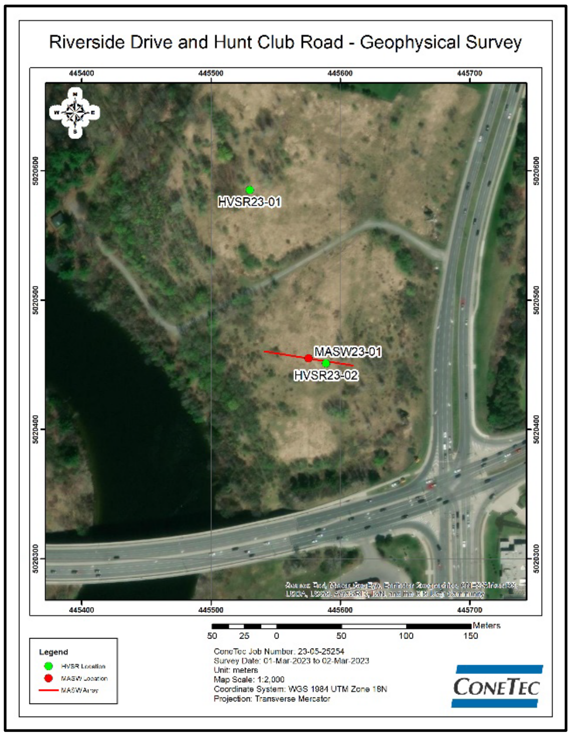
Riverside Dr and Hunt Club Rd.

Prepared for:

Golder Associates

ConeTec Job No: 23-05-25254.02

Project Start Date: 01-Mar-2023
Project End Date: 02-Mar-2023
Report Date: 09-Mar-2023



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Introduction

The enclosed report presents the results of the geophysical site investigation program conducted by ConeTec Investigations Ltd. for Golder Associates at the Riverside Dr and Hunt Club Rd in Ottawa, Ontario. The program consisted of 1 one-dimensional (1D) Multichannel Analysis of Surface Waves (MASW) test and 2 Horizontal to Vertical Spectral Ratio (HVSR) tests. The purpose was to provide shear wave velocity (Vs) of the subsurface and to produce time weighted average Vs in the top 30 m of sediment and to measure resonant frequency. SCPT data collected at the same project are reported on job number 23-05-25254.

Project Information

Project	
Client	Golder Associates
Project	Riverside Dr and Hunt Club Rd
ConeTec project number	23-05-25254.02

Coordinates			
Test Type	Collection Method	EPSG Number	Comments
MASW, HVSR	Site measurements	32618	

MASW Acquisition Procedures

MASW datasets were acquired using the equipment outlined in the table below. A static receiver array was used for each profile. Each array consisted of 24 channels, where geophones were placed every 3 m along the line. A sledgehammer was used as a seismic source. Readings were gathered at shot spacings of 3, 6 and 12 m off either end of the array. Each source location had a minimum of 3 shots collected and stacked to produce the seismic record. In addition, at least 2 passive seismic records were collected with each array.

HVSR data was collected by placing the 3-component seismograph flat on the ground with good coupling. Readings were taken near SCPT locations. Each reading was 60 minutes long. Equipment outlined in the table below.

Equipment Used for MASW Testing on this Project				
Seismograph(s)	Geophones	Coupling Mechanism	Trigger Style	Seismic Sources
1 x Geometrics Geode 24	24 x Geospace 4.5 Hz vertical	Steel pucks	Piezoelectric trigger	10 lb sledgehammer with aluminium plate

Equipment Used for HVSR Testing on this Project			
Seismograph(s)	Coupling Mechanism	Trigger Style	Seismic Sources
Tromino 3-Component Seismograph	Spikes	Timed Trigger	Passive Sources

Data Analysis and Quality

MASW data quality at this site was good. Traffic passing by on nearby roads were a considerable source of seismic noise. Timing the shots to coincide with slow moving vehicles mitigated this effect. Additionally, the signal to noise ratio was increased by taking multiple stacks at each shot location. Example time domain traces and overtone images are included in the appendices of this report.

The HVSR data quality collected on this project was good. Traffic and other work on site were significant sources of noise, but this was mitigated by taking 60-minute readings, allowing noisy sections to be removed from the processed data set. A clear H/V peak was seen on each reading and were considered reliable and clear.

Results

Good quality overtone images showed coherent dispersion curves from 6-28 Hz. Shear wave velocity ranged from 200 - 800 m/s and the depth of investigation was about 40 m. Inverted shear wave velocity (Vs) test results and Vs30 calculations are included in the appendices of this report. MASW pdf profiles and csv files with coordinates, depth, and Vs data are included in the release of this report.

The H/V peaks were clear and reliable and showed resonant frequencies of 3.38 Hz and 3.08 Hz for HVSR23-01 and HVSR23-02 respectively. The HVSR summary and quality reports are included in the appendices of this report.

Closure

Thank you for the opportunity to work on this project. The equipment used and the field procedures followed complied with current accepted practice standards. This report has been prepared under my supervision and I have reviewed and approved the content.

ConeTec Investigations Ltd.

Matvei Kootchin, P. Geo.

Limitations

3rd Party Disclaimer

This report titled “Riverside Dr and Hunt Club Rd”, referred to as the (“Report”), was prepared by ConeTec for Golder Associates. The Report is confidential and may not be distributed to or relied upon by any third parties without the express written consent of ConeTec. Any third parties gaining access to the Report do not acquire any rights as a result of such access. Any use which a third party makes of the Report, or any reliance on or decisions made based on it, are the responsibility of such third parties. ConeTec accepts no responsibility for loss, damage and/or expense, if any, suffered by any third parties as a result of decisions made, or actions taken or not taken, which are in any way based on, or related to, the Report or any portion(s) thereof.

Client Disclaimer

ConeTec was retained by Golder Associates to collect MASW data to provide Vs measurements from which Vs30 was calculated and HVSR data to measure resonant frequency (“Data”). The Data is included in this report titled “Riverside Dr and Hunt Club Rd”, which is referred to as the (“Report”). ConeTec has collected and reported the Data in accordance with current industry standards. No other warranty, express or implied, with respect to the Data is made by ConeTec. In order to properly understand the Data included in the Report, reference must be made to the documents accompanying and other sources referenced in the Report in their entirety. Other than the Data, the contents of the Report should not be relied upon in any fashion without independent verification and ConeTec is in no way responsible for any loss, damage or expense resulting from the use of, and/or reliance on, such material by any party.

Multichannel analysis of surface waves (MASW) is a non-intrusive in-situ test that uses the principles of elasticity and surface wave dispersion to determine the variation of shear wave velocity with depth at a site. The observation that surface waves (Rayleigh waves) of different wavelengths propagate at different phase velocities in non-ideal media, is called dispersion. This is a direct result of the fact that surface waves of different wavelengths propagate along the surface to varying depths, and hence, if material stiffness changes with depth (as is the case with most non-ideal materials), then an appropriately selected wavelength band will reflect such changes in the velocity of propagation.

The field methods for surface wave testing are very similar to other surface seismic data collection methods. Surface geophones are placed in a linear array along a survey line at a known separation (typically one metre). A series of recordings (shots) are collected with a known in-line source offset from the array. Each shot gather is represented in the time-offset domain and shows the amplitude of wave propagation through the array (refer to Figure MASW-1). For detailed frequency analysis, multiple records with different shot offset distances are collected to help better define the broad spectrum frequency-phase velocity response of the medium. Two-dimensional cross sections can be collected by moving the geophone array a small distance (typically two meters) along the line and repeating the shots at set offsets.

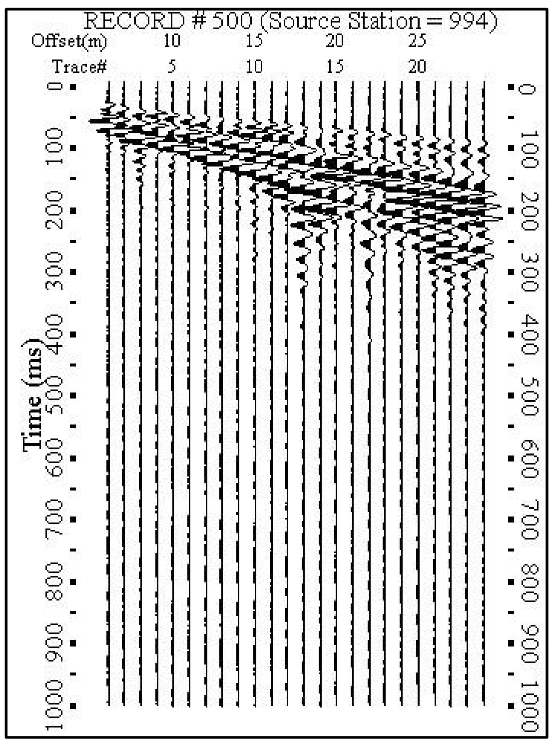


Figure MASW-1. Typical MASW time domain record (shot gather)

Given that surface wave velocity is closely related to the shear wave velocity and the wavelength related to depth, the surface wave results can be used to develop a profile of shear wave velocity versus depth through a process referred to as inversion. The program used to perform the inversion is SurfSeis 4.0, developed by the Kansas Geological Survey. In SurfSeis, the raw time domain traces are transformed to the frequency domain to create what is referred to as an overtone image as shown in Figure MASW-2. The overtone image displays the amplitude of the primary surface wave mode and any potential higher modes. A dispersion curve is fitted to the overtone image, and the inversion process is then used to determine the most appropriate shear wave velocity profile. The parameters used for the inversion of the dispersion data are provided in the data release folder in an Excel table.

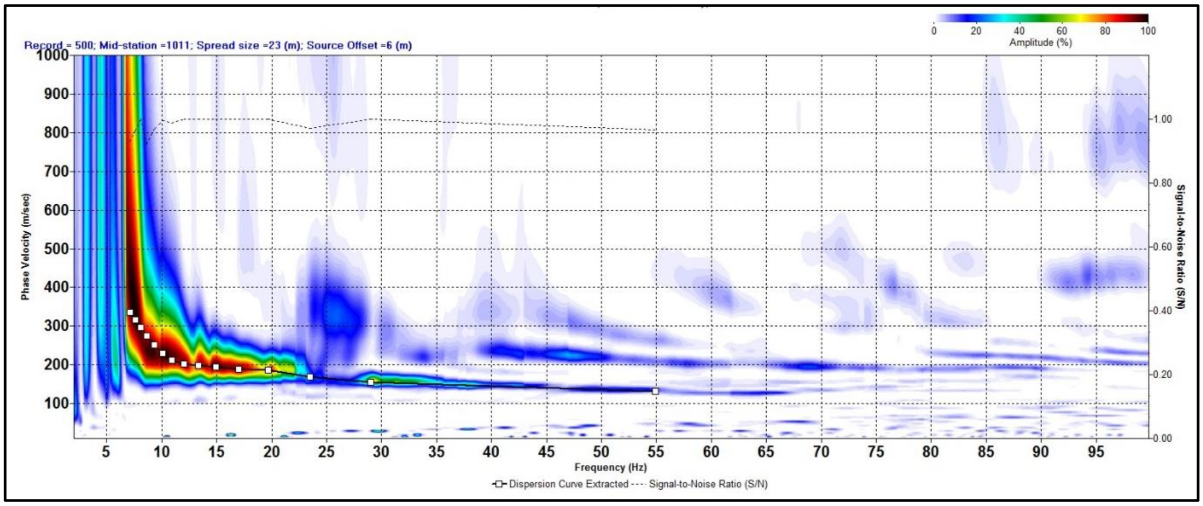


Figure MASW-2. Overtone image and a picked dispersion curve

For each test location, a 1D shear wave velocity profile comprising of a number of velocity layers of variable thickness (refer to Figure MASW-3) is provided . For 2D testing a series of 1D tests are combined to produce a shear wave velocity cross section.

The depth of investigation is related to the ground conditions and the amount of energy delivered by the surface wave source. The surface wave method uses Rayleigh waves that travel horizontally along the ground surface to a depth of about one wavelength. The actual depth of sampling of the ground is considered to be one-half to one-third of the Rayleigh (surface) wave wavelength. The wavelengths measured by the equipment will be a function of the frequency of the source and the velocity of the surface waves through the ground. As the depth of investigation increases, there will be less certainty in terms of layer boundaries and velocity values.

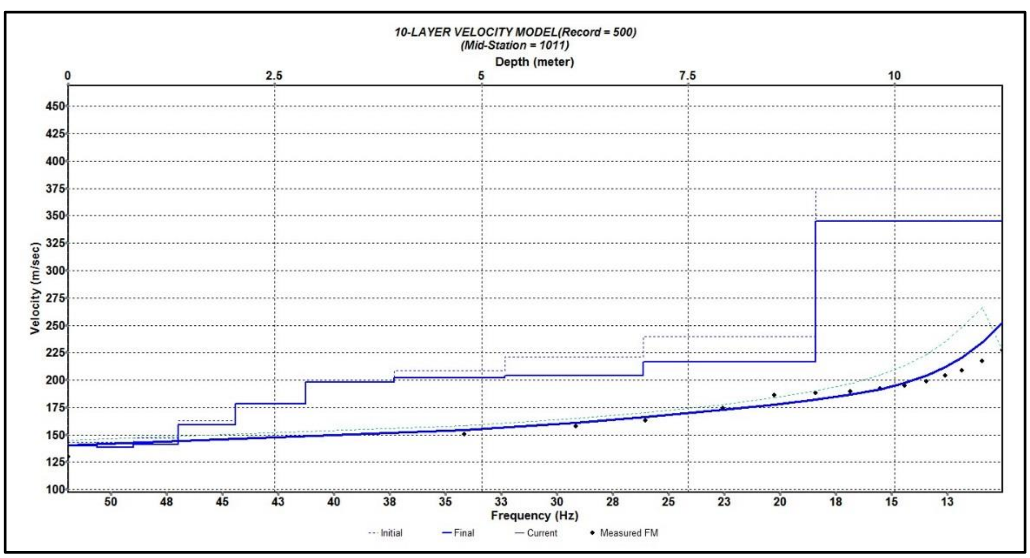


Figure MASW-3. 1D inversion result with fitted dispersion curve

The equipment, field procedures, and analysis software used by ConeTec all conform to the currently accepted best practices for MASW testing. The results of geophysical testing are always interpretative to a certain extent and should be confirmed by drilling or other intrusive testing.

References

Miller, R.D., Xia, J., Park, C.B., and Ivanov, J.M., 1999, Multichannel analysis of surface waves to map bedrock, Kansas Geological Survey, The Leading Edge, December, p. 1392-1396.

Park, C.B., Miller, R.D., and Xia, J., 1998b, Ground roll as a tool to image near-surface anomaly: 68th Ann. Internat. Mtg. Soc. Expl.Geophys., Expanded Abstracts, p. 874-877.

Park, C.B., Miller, R.D., and Xia, J., 1999, Multichannel analysis of surface waves: Geophysics, v. 64, n. 3, pp. 800-808.

Park, C.B., Miller, R.D., Xia, J., and Ivanov, J., 2007, Multichannel analysis of surface waves (MASW)-active and passive methods: The Leading Edge, January.

SurfSeis website: http://www.kgs.ku.edu/software/surfseis/index.html

Xia, J., R.D. Miller, and C.B. Park, 2000a, Advantages of calculating shear-wave velocity from surface waves with higher modes: [Exp. Abs.]: Soc. Expl. Geophys., p. 1295-1298.

Xia, J., Miller, R.D., Park, C.B., and Ivanov, J., 2000b, Construction of 2-D vertical shear-wave velocity field by the Multichannel Analysis of Surface Wave technique, Proceedings of the Symposium on the Application of Geophysics to Engineering and Environmental Problems (SAGEEP 2000), Washington D.C, February 20-24, p. 1197-1206.

The horizontal to vertical spectral ratio (HVSR) method is a passive seismic technique that can be used to measure site period, estimate sediment thickness or the depth to bedrock. HVSR uses the ratio of the average horizontal and vertical component amplitude spectrums to generate a spectral ratio curve with a peak at the fundamental resonance frequency (f_0). A low frequency multi-component seismograph is used to measure the horizontal and vertical components of ambient seismic noise. These elastic waves are created naturally by sea and wind action but can also be produced at higher frequencies through anthropogenic sources such as vehicle traffic or industrial activity. The HVSR method is best suited to sites with a sharp contrast in acoustic impedance at the sediment-bedrock interface.

A measure of the site period or resonant frequency can be important in determining how a site will respond during an earthquake. In instances where multiple peaks are measured in the spectral response, the peak with the highest amplitude is considered the site period. If multiple peaks are near the same amplitude, then multiple site periods will be reported. If a velocity profile is known, the HVSR data can be used to estimate depth to bedrock. This information is often used to produce depth to bedrock plan view maps over large areas. The reverse is also true where if the depth to bedrock is known then an average velocity of the overburden can be estimated.

Prior to collecting data, a measurement location is selected to avoid heavy traffic, industrial noise and artificial ground surfaces such as asphalt, pavement, or cement or as directed by the client. If many readings will be taken across a site, then the seismometer will be placed in a common orientation for each reading. Typically, spikes attached to the bottom of the sensor are used to couple it to the ground. Good coupling, where the ground tightly holds on to the spikes, is essential for high quality data. After the equipment is coupled, it is levelled using the built-in level. Data is typically recorded with a sampling frequency of 128 Hz. In general, longer readings can detect lower frequencies, down to a limit of 0.1 Hz. Lower frequency data typically correspond to deeper investigation depths. A 15 to 20-minute reading will be able to measure frequencies down to the 0.5 Hz range with a depth of investigation of 100 meters or more. Once the reading has started the user walks away and waits for the reading to complete.

The passive seismic data is analyzed using software developed by MoHo s.r.l. An example HVSR time domain record is shown in Figure HVSR-1.

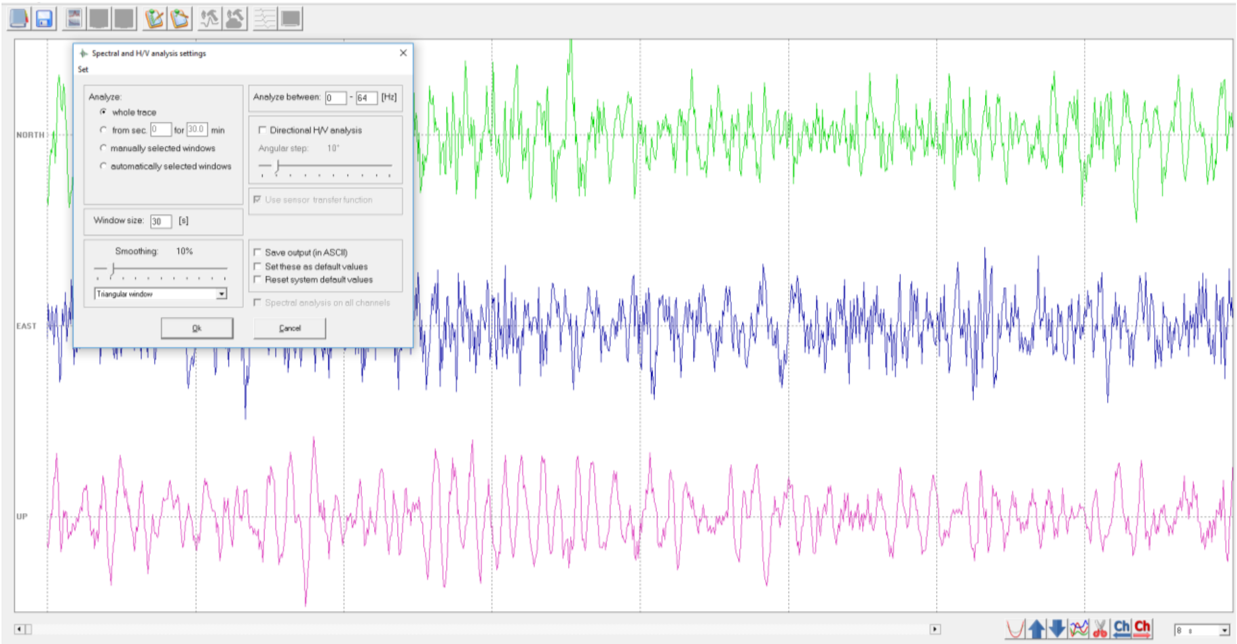


Figure HVSR-1. Typical HVSR time domain record

Post processing typically includes band-pass filtering to remove high frequency noise and spectral smoothing. The software computes the average spectrums of the horizontal and vertical components over a user-specified time window. After selecting processing parameters, a series of windows are used to analyse the data (Figure HVSR-2). These windows include the H/V stability, the amplitude spectra, and the H/V curve. The H/V stability window shows the signal response of the sensor over time and is used to edit out noise. Amplitude spectra show the amplitude of certain frequency bands for each orthogonal sensor and are used to compare the response of each component and to help differentiate peaks that are stratigraphic in origin from anthropic. The H/V curve shows the ratio between the horizontal readings and the vertical readings and the standard deviations. It is used to determine the site period for the reading by locating the H/V peak.

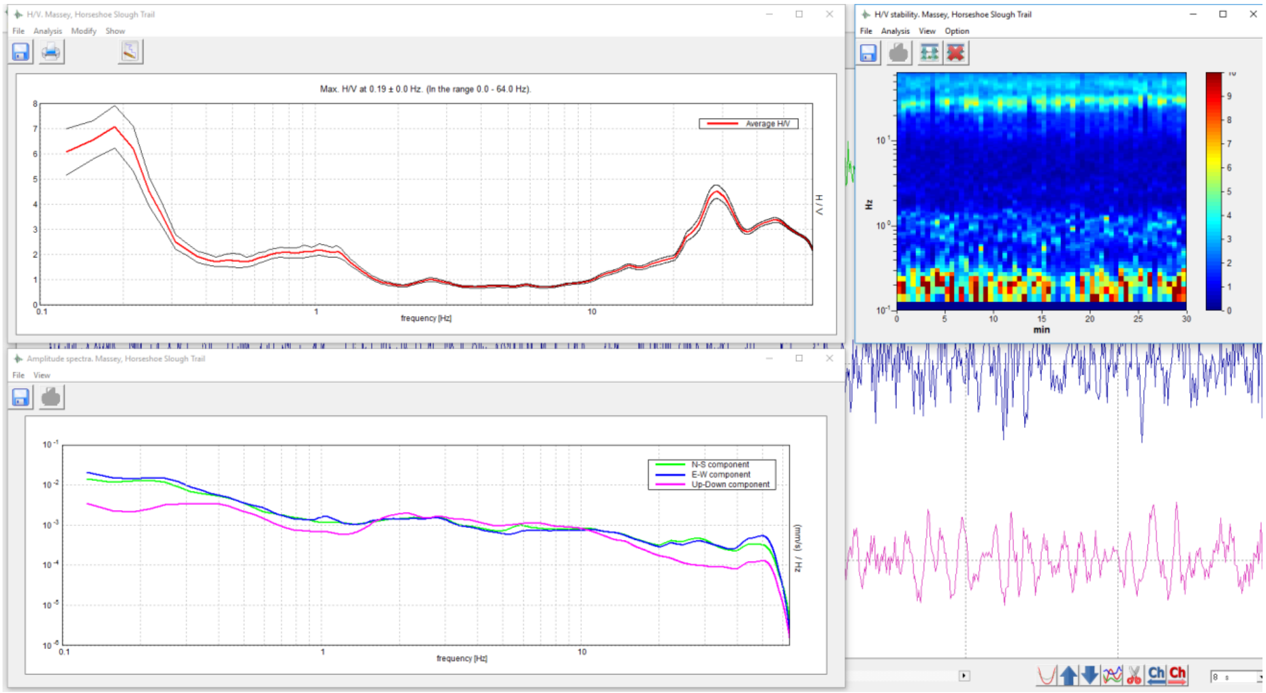


Figure HVSR-2. H/V processing windows

If the purpose of the test is to determine shear wave velocity or layer thickness, then the next step is to fit a model to the H/V curve. The model is built in a table containing Vs, Vp, layer thickness, Poisson's ratio and density (Figure HVSR-3). If a layer thickness or Vs is known, the remainder of the table can be populated to create a model that fits the H/V curve.

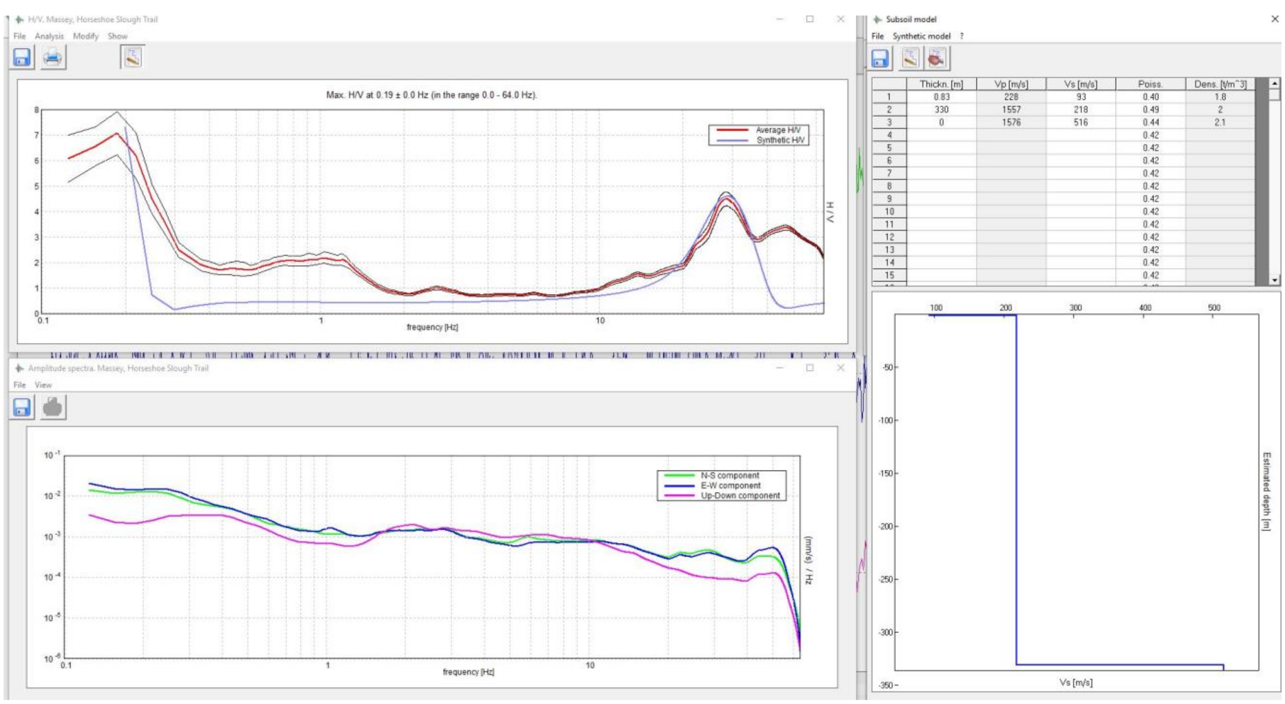


Figure HVSR-3. H/V processing windows with modeled data

For each test location, a site period value is provided. Lower frequencies require longer reading times to collect statistically significant values. Ground conditions, reflector topography, velocity contrast between layers and site noise can all have significant impacts on the reading quality. As lower frequencies are recorded there will be less certainty in terms of period. Likewise, if the site period is translated to shear wave velocity or layer thickness, there will be less certainty in those values as the depth of investigation increases.

The equipment, field procedures, and analysis software used by ConeTec are in general accordance with the SESAME (2004) guidelines. The results of geophysical testing are always interpretative to a certain extent and should be used as a part of a larger site investigation.

References

SESAME European Research Project WP12 – Deliverable D23.12 (2004), Guidelines for the Implementation of the H/V Spectral Ratio Technique on Ambient Vibrations Measurements, Processing and interpretation: European Commission – Research General Directorate
Project No. EVG1-CT-2000-00026 SESAME

The following appendices listed below are included in the report:

- 1D MASW Summary and Map
- 1D MASW Results
- Vs30 Calculation Tables
- MASW Time Domain Traces and Overtone Images
- HVSR Summary and Results

1D MASW Summary and Map

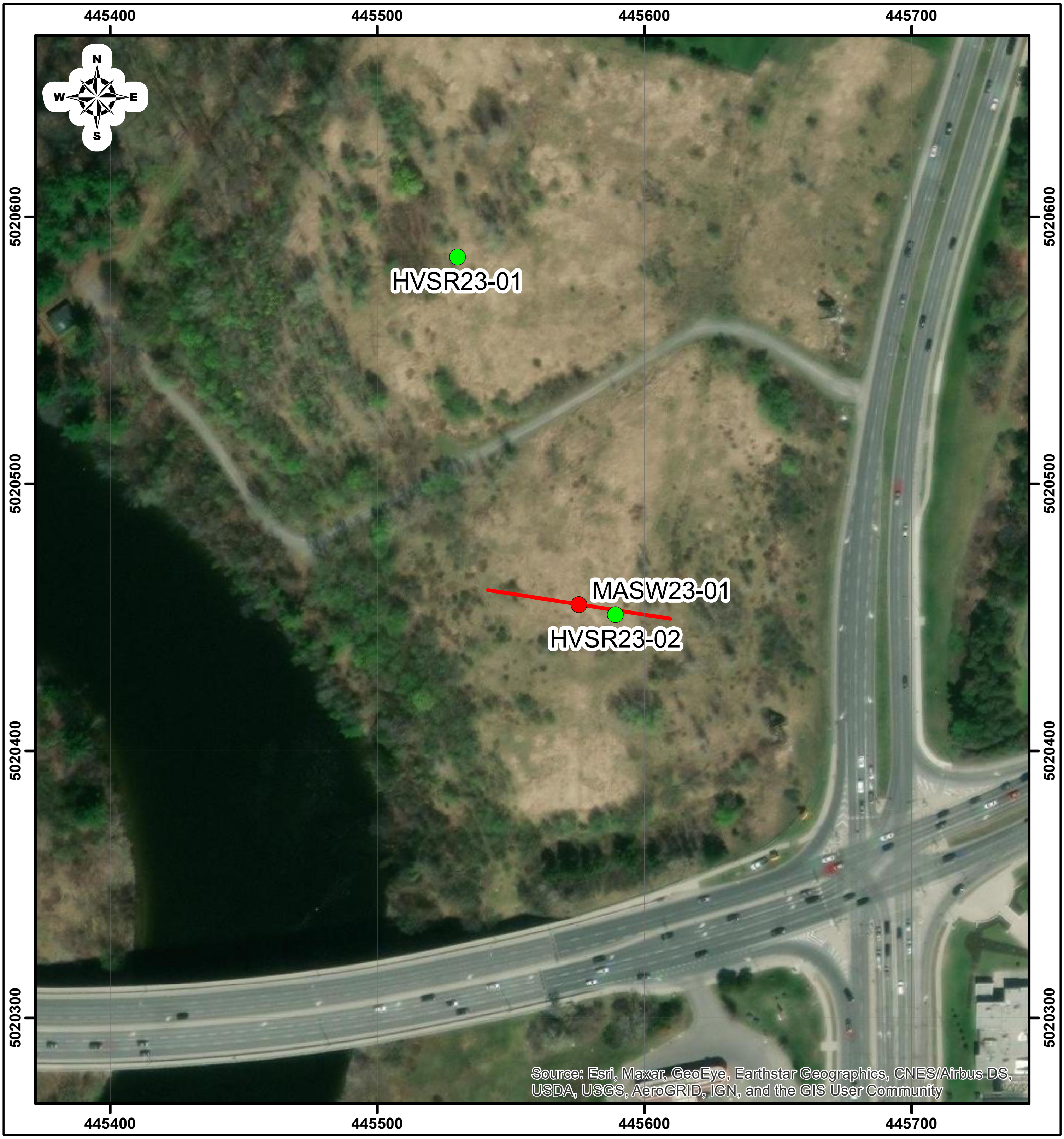
Job No: 23-05-25254
Client: Golder Associates
Project: Riverside Dr and Hunt Club Rd
Start Date: 02-Mar-2023
End Date: 02-Mar-2023

1D MASW TEST SUMMARY

Sounding ID	Date	Source Type	Geophone Spacing (m)	Array Length (m)	Start of Section Northing[1] (m)	Start of Section Easting (m)	Refer to Notation Number
MASW23-01	2-Mar-23	Sledgehammer	3.0	69	5020455	445575	1

1. Coordinates were determined using site measurments. WGS84 / UTM Zone 18 North.

Riverside Drive and Hunt Club Road - Geophysical Survey



Meters
50 25 0 50 100 150

Legend

- HVSR Location
- MASW Location
- MASW Array

ConeTec Job Number: 23-05-25254
Survey Date: 01-Mar-2023 to 02-Mar-2023
Unit: meters
Map Scale: 1:2,000
Coordinate System: WGS 1984 UTM Zone 18N
Projection: Transverse Mercator

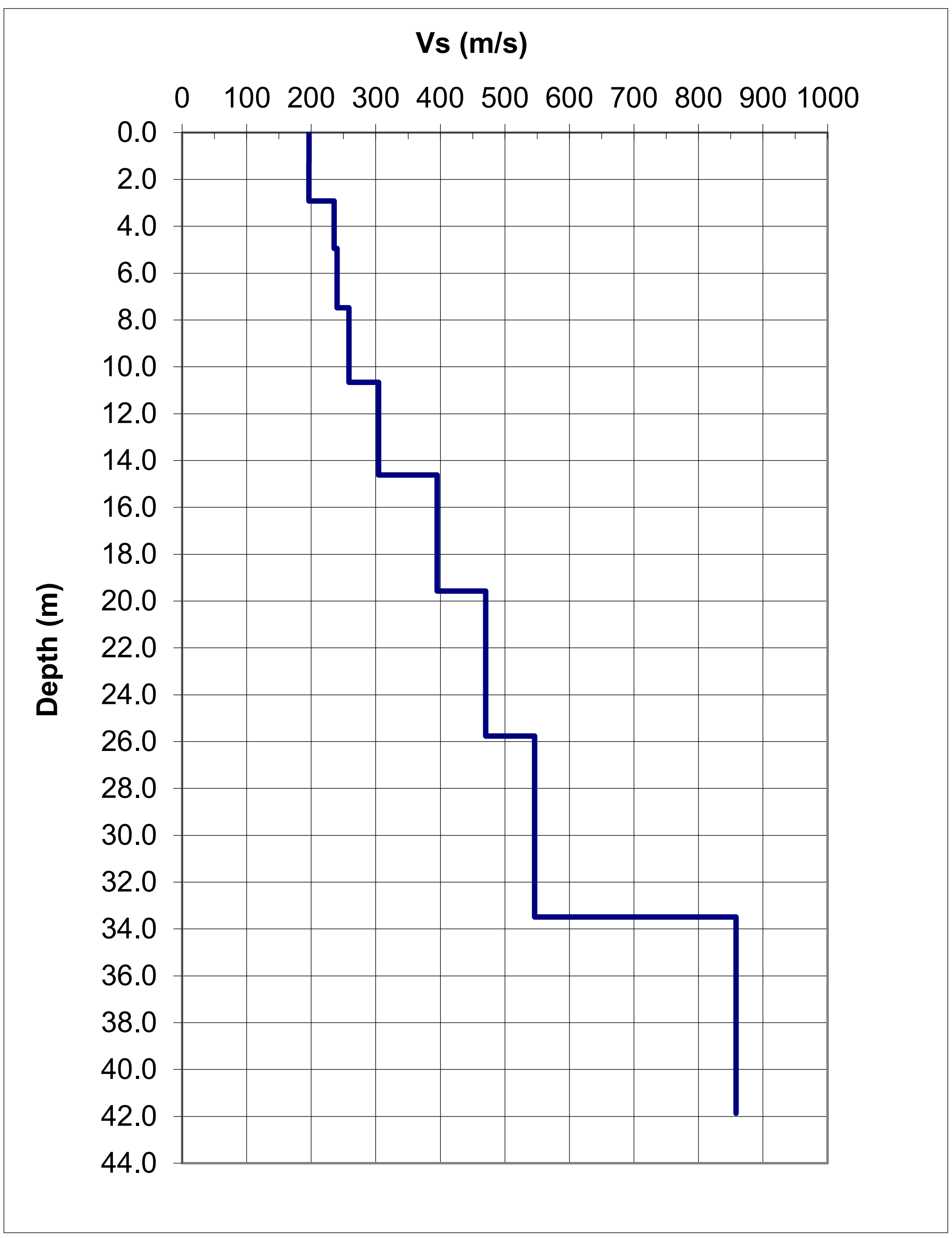
CONETEC

1D MASW Results

Job No:	23-05-25254
Client:	Golder Associates
Project:	Riverside Dr and Hunt Club Rd
Sounding ID:	MASW23-01
Date:	02-Mar-2023

1D MASW SHEAR WAVE VELOCITY TEST RESULTS

Layer	Layer Thickness (m)	Depth of Bottom of Layer (m)	Vs (m/s)
1	1.30	1.30	197
2	1.62	2.92	197
3	2.03	4.95	235
4	2.54	7.49	240
5	3.17	10.65	259
6	3.96	14.62	304
7	4.95	19.57	395
8	6.19	25.76	470
9	7.74	33.50	546
10	8.37	41.87	858



Vs30 Calculation Tables

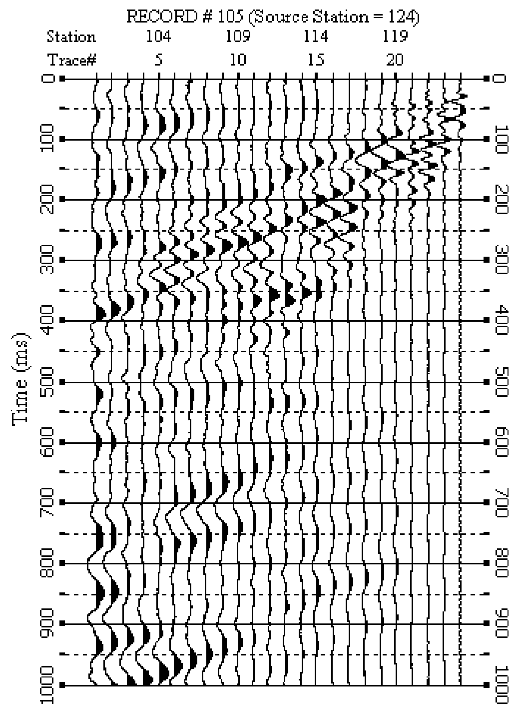
Job No:	23-05-25254
Client:	Golder Associates
Project:	Riverside Dr and Hunt Club Rd
Sounding:	MASW23-01
Date:	02-Mar-2023

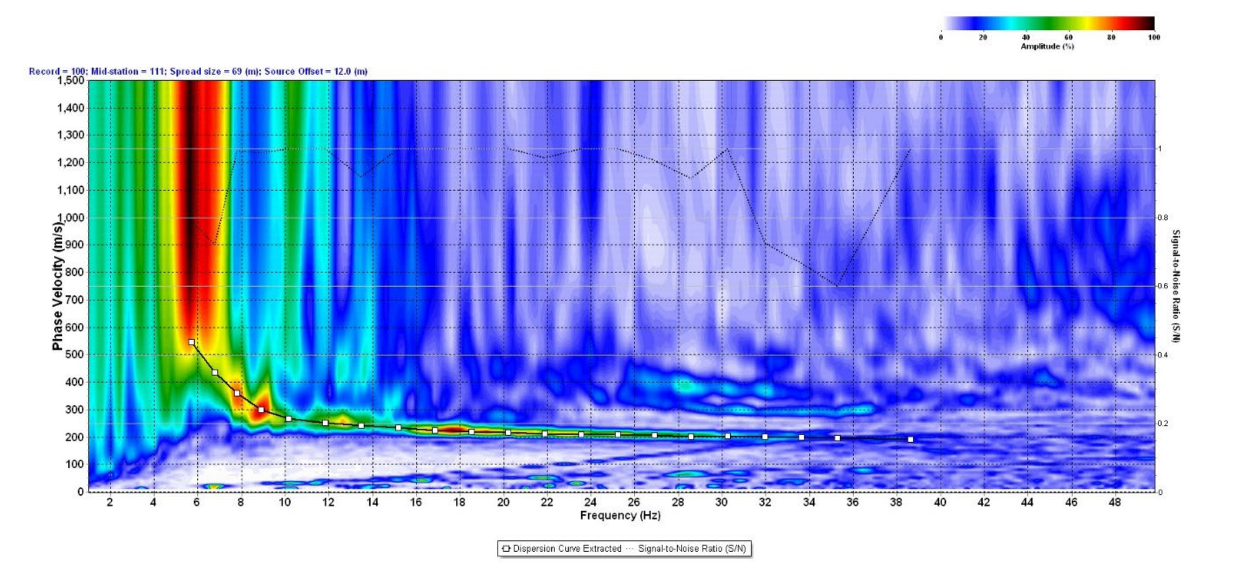
VS30 CALCULATION

Layer Number	Layer Thickness (m)	Layer Bottom (m)	Vs (m/s)	Equivalent Vertical Travel Time (s)
1	1.30	1.30	197	0.00660
2	1.62	2.92	197	0.00826
3	2.03	4.95	235	0.00862
4	2.54	7.49	240	0.01056
5	3.17	10.65	259	0.01226
6	3.96	14.62	304	0.01302
7	4.95	19.57	395	0.01254
8	6.19	25.76	470	0.01316
9	4.24	30.00	546	0.00776
Total Vertical Travel Time for 30m (s)				0.09278
Average Travel Time Weighted Shear Wave Velocity (m/s)				323

Notes:

MASW Time Domain Traces and Overtone Images





MASW23-01: Example time domain trace for active source (top) with resulting overtone image with picked dispersion curve (bottom).

HVSR Summary and Results

Job No:	23-05-25254
Client:	Golder Associates
Project:	Riverside Dr and Hunt Club Rd
Start Date:	01-Mar-2023
End Date:	02-Mar-2023

HVSR TEST SUMMARY

Sounding ID	Date	Location	Reading Length (min)	Resonant Frequency (Hz)	H/V Reliable Curve	H/V Clear Peak	Northing (m)	Easting (m)	Refer to Notation Number
HVSR23-01	1/Mar/23	Near SCPT23-03	60	3.38 ± 0.03	Y	Y	5020585	445530	1
HVSR23-02	2/Mar/23	Near SCPT23-01D	60	3.06 ± 0.05	Y	Y	5020451	445589	1

1. Coordinates collected using a consumer handheld GPS device. Datum: WGS84 UTM Zone 18N

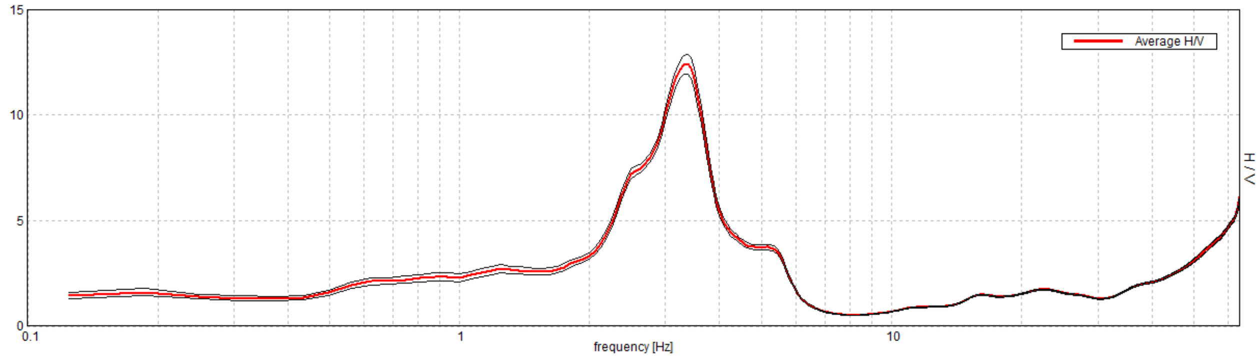
Riverside Dr and Hunt Club Rd, HVSR22-01

Instrument: TZ3-0084/02-19
Data format: 32 byte
Full scale [mV]: 51
Start recording: 01/03/23 13:25:18 End recording: 01/03/23 14:25:18
Channel labels: NORTH SOUTH; EAST WEST ; UP DOWN
GPS data not available

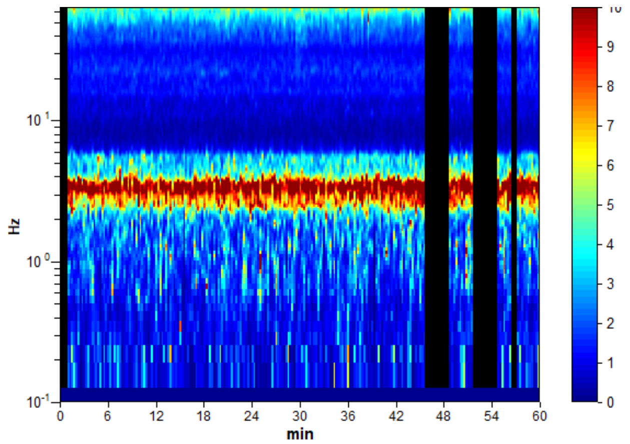
Trace length: 1h00'00''. Analyzed 87% trace (manual window selection)
Sampling rate: 128 Hz
Window size: 15 s
Smoothing type: Triangular window
Smoothing: 10%

HORIZONTAL TO VERTICAL SPECTRAL RATIO

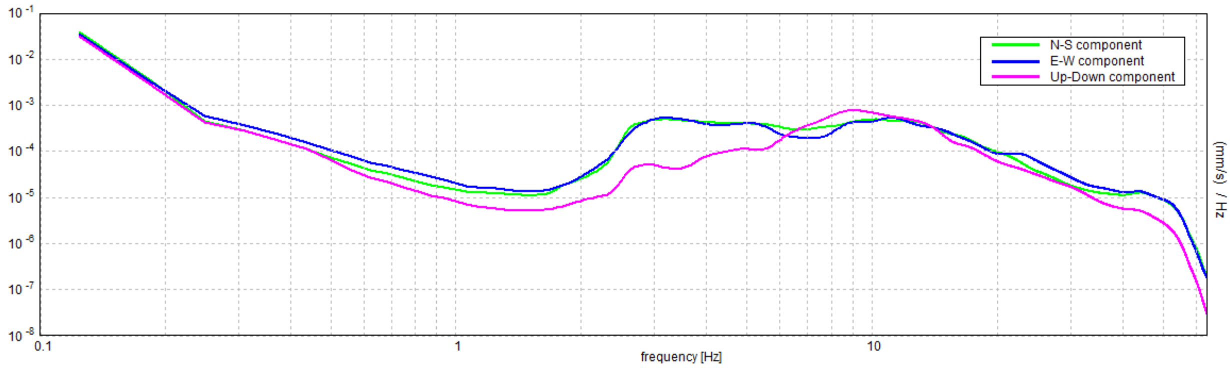
Max. H/V at 3.38 ± 0.02 Hz. (In the range 0.0 - 64.0 Hz).



H/V TIME HISTORY



SINGLE COMPONENT SPECTRA



Max. H/V at 3.38 ± 0.02 Hz (in the range 0.0 - 64.0 Hz).

Criteria for a reliable H/V curve [All 3 should be fulfilled]			
$f_0 > 10 / L_w$	3.38 > 0.67	OK	
$n_c(f_0) > 200$	10580.6 > 200	OK	
$\sigma_A(f) < 2$ for $0.5f_0 < f < 2f_0$ if $f_0 > 0.5Hz$ $\sigma_A(f) < 3$ for $0.5f_0 < f < 2f_0$ if $f_0 < 0.5Hz$	Exceeded 0 out of 82 times	OK	
Criteria for a clear H/V peak [At least 5 out of 6 should be fulfilled]			
Exists f^- in $[f_0/4, f_0]$ | $A_{H/V}(f^-) < A_0 / 2$	2.375 Hz	OK	
Exists f^+ in $[f_0, 4f_0]$ | $A_{H/V}(f^+) < A_0 / 2$	3.938 Hz	OK	
$A_0 > 2$	12.40 > 2	OK	
$f_{peak}[A_{H/V}(f) \pm \sigma_A(f)] = f_0 \pm 5\%$	|0.00628| < 0.05	OK	
$\sigma_f < \varepsilon(f_0)$	0.02118 < 0.16875	OK	
$\sigma_A(f_0) < \theta(f_0)$	0.4661 < 1.58	OK	

L_w	window length
n_w	number of windows used in the analysis
$n_c = L_w n_w f_0$	number of significant cycles
f	current frequency
f_0	H/V peak frequency
σ_f	standard deviation of H/V peak frequency
$\varepsilon(f_0)$	threshold value for the stability condition $\sigma_f < \varepsilon(f_0)$
A_0	H/V peak amplitude at frequency f_0
$A_{H/V}(f)$	H/V curve amplitude at frequency f
f^-	frequency between $f_0/4$ and f_0 for which $A_{H/V}(f^-) < A_0/2$
f^+	frequency between f_0 and $4f_0$ for which $A_{H/V}(f^+) < A_0/2$
$\sigma_A(f)$	standard deviation of $A_{H/V}(f)$, $\sigma_A(f)$ is the factor by which the mean $A_{H/V}(f)$ curve should be multiplied or divided
$\sigma_{logH/V}(f)$	standard deviation of log $A_{H/V}(f)$ curve
$\theta(f_0)$	threshold value for the stability condition $\sigma_A(f) < \theta(f_0)$

Threshold values for σ_f and $\sigma_A(f_0)$					
Freq. range [Hz]	< 0.2	0.2 – 0.5	0.5 – 1.0	1.0 – 2.0	> 2.0
$\varepsilon(f_0)$ [Hz]	0.25 f_0	0.2 f_0	0.15 f_0	0.10 f_0	0.05 f_0
$\theta(f_0)$ for $\sigma_A(f_0)$	3.0	2.5	2.0	1.78	1.58
log $\theta(f_0)$ for $\sigma_{logH/V}(f_0)$	0.48	0.40	0.30	0.25	0.20

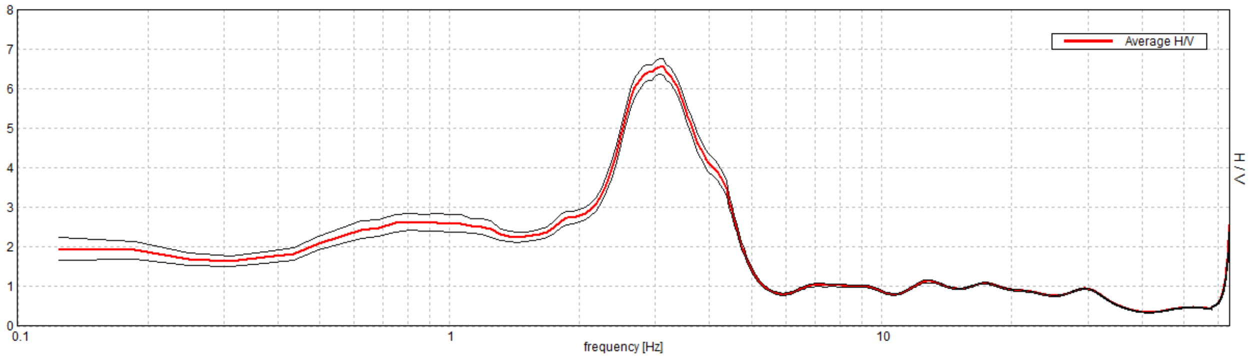
Riverside Dr and Hunt Club Rd, HVSR23-02

Instrument: TZ3-0084/02-19
Data format: 32 byte
Full scale [mV]: 51
Start recording: 02/03/23 10:05:06 End recording: 02/03/23 11:05:06
Channel labels: NORTH SOUTH; EAST WEST ; UP DOWN
GPS data not available

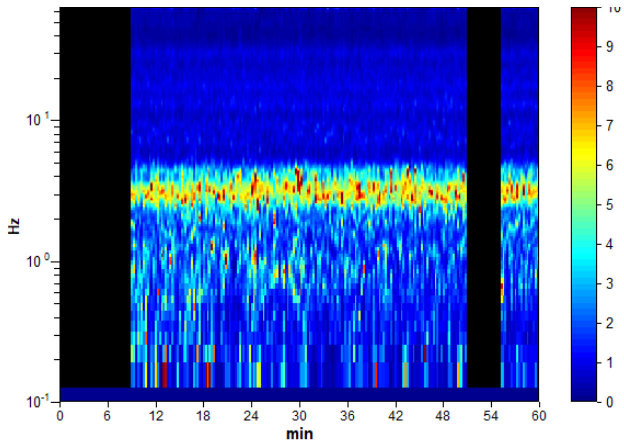
Trace length: 1h00'00''. Analyzed 78% trace (manual window selection)
Sampling rate: 128 Hz
Window size: 15 s
Smoothing type: Triangular window
Smoothing: 10%

HORIZONTAL TO VERTICAL SPECTRAL RATIO

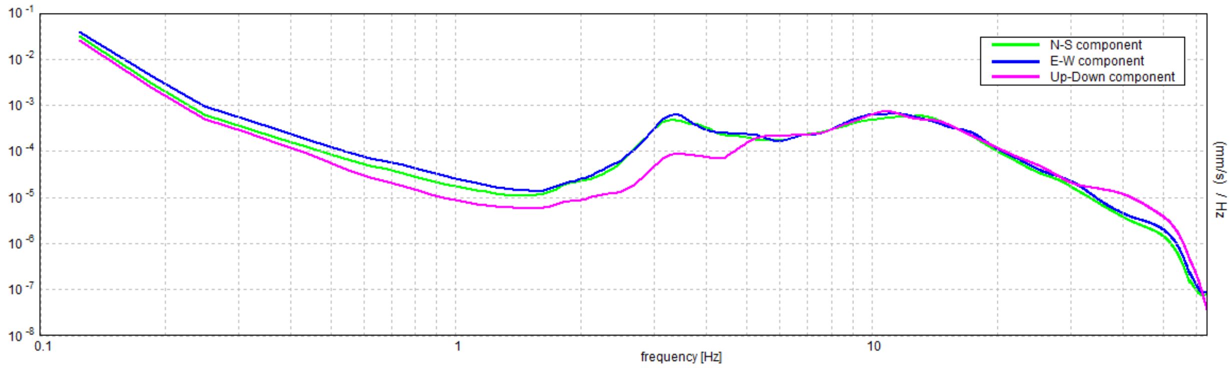
Max. H/V at 3.06 ± 0.05 Hz. (In the range 0.0 - 64.0 Hz).



H/V TIME HISTORY



SINGLE COMPONENT SPECTRA



Max. H/V at 3.06 ± 0.05 Hz (in the range 0.0 - 64.0 Hz).

Criteria for a reliable H/V curve [All 3 should be fulfilled]			
$f_0 > 10 / L_w$	3.06 > 0.67	OK	
$n_c(f_0) > 200$	8590.3 > 200	OK	
$\sigma_A(f) < 2$ for $0.5f_0 < f < 2f_0$ if $f_0 > 0.5Hz$ **$\sigma_A(f) < 3$ for $0.5f_0 < f < 2f_0$ if $f_0 < 0.5Hz$**	Exceeded 0 out of 74 times	OK	
Criteria for a clear H/V peak [At least 5 out of 6 should be fulfilled]			
Exists f^- in $[f_0/4, f_0]$ | $A_{H/V}(f^-) < A_0 / 2$	2.188 Hz	OK	
Exists f^+ in $[f_0, 4f_0]$ | $A_{H/V}(f^+) < A_0 / 2$	4.438 Hz	OK	
$A_0 > 2$	6.56 > 2	OK	
$f_{peak}[A_{H/V}(f) \pm \sigma_A(f)] = f_0 \pm 5\%$	|0.01655| < 0.05	OK	
$\sigma_f < \varepsilon(f_0)$	0.05069 < 0.15313	OK	
$\sigma_A(f_0) < \theta(f_0)$	0.2002 < 1.58	OK	

L_w	window length
n_w	number of windows used in the analysis
$n_c = L_w n_w f_0$	number of significant cycles
f	current frequency
f_0	H/V peak frequency
σ_f	standard deviation of H/V peak frequency
$\varepsilon(f_0)$	threshold value for the stability condition $\sigma_f < \varepsilon(f_0)$
A_0	H/V peak amplitude at frequency f_0
$A_{H/V}(f)$	H/V curve amplitude at frequency f
f^-	frequency between $f_0/4$ and f_0 for which $A_{H/V}(f^-) < A_0/2$
f^+	frequency between f_0 and $4f_0$ for which $A_{H/V}(f^+) < A_0/2$
$\sigma_A(f)$	standard deviation of $A_{H/V}(f)$, $\sigma_A(f)$ is the factor by which the mean $A_{H/V}(f)$ curve should be multiplied or divided
$\sigma_{logH/V}(f)$	standard deviation of log $A_{H/V}(f)$ curve
$\theta(f_0)$	threshold value for the stability condition $\sigma_A(f) < \theta(f_0)$

Threshold values for σ_f and $\sigma_A(f_0)$					
Freq. range [Hz]	< 0.2	0.2 – 0.5	0.5 – 1.0	1.0 – 2.0	> 2.0
$\varepsilon(f_0)$ [Hz]	0.25 f_0	0.2 f_0	0.15 f_0	0.10 f_0	0.05 f_0
$\theta(f_0)$ for $\sigma_A(f_0)$	3.0	2.5	2.0	1.78	1.58
log $\theta(f_0)$ for $\sigma_{logH/V}(f_0)$	0.48	0.40	0.30	0.25	0.20

APPENDIX H

Technical Memorandum – Geophysics (2011)

TECHNICAL MEMORANDUM

DATE April 5, 2011 **PROJECT No.** 11-1121-0050

TO Mike Cunningham
Golder Associates Ltd.

CC

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

NBCC SEISMIC SITE CLASS TESTING RESULTS – ST. MARY'S SITE, OTTAWA, ONTARIO

This technical memorandum presents the processing and results of two Multichannel Analysis of Surface Waves (MASW) tests performed for the purpose of National Building Code of Canada Seismic Site Classification for a site located Northwest of the intersection of Hunt Club Road and Riverside Drive in Ottawa, Ontario. The geophysical testing was performed by Golder personnel on April 1, 2011.

Methodology

The Multichannel Analysis of Surface Waves (MASW) method measures variations in surface wave velocity with increasing distance and wavelength and can be used to infer the rock/soil types, stratigraphy and soil conditions.

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

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

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

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

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

Field Work

The MASW field work was conducted on April 1, 2011, by personnel from the Golder Mississauga and Ottawa offices. The two MASW lines were oriented nearly parallel to Riverside Road. The location of the lines is provided in Table 1. At each line, a shallow trench was dug to remove the frozen layer, which would affect testing results. For both MASW lines, a series of 24 low frequency (4.5 Hz) geophones were laid out at 3 m intervals. A seismic weight drop of 45 kg and a 5.5 kg sledge hammer were used as seismic sources for this investigation. Seismic records were collected with seismic sources located 5, 10 and 20 m from and collinear to the geophone array. An example of an active seismic record collected at MASW Lines 1 and 2 is shown in Figures 1 and 2, respectively (below).

Table 1: Surveyed MASW Lines

MASW LINES	Easting (m)	Northing (m)
Line 1 - Start	445,630E	5,020,577N
Line 1 - End	445,638E	5,020,649N
Line 2 - Start	445,520E	5,020,565N
Line 2 - End	445,506E	5,020,631N

Datum: UTM NAD 83, Zone 18

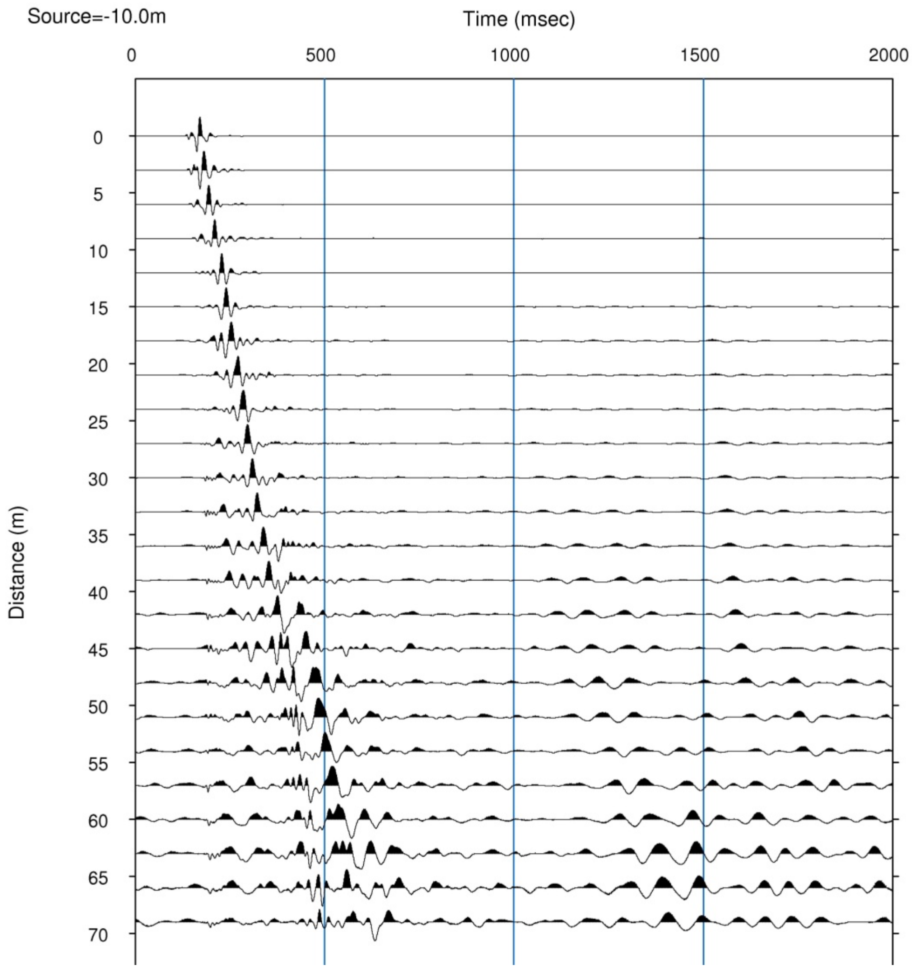


Figure 1: Typical seismic record collected along MASW Line1.

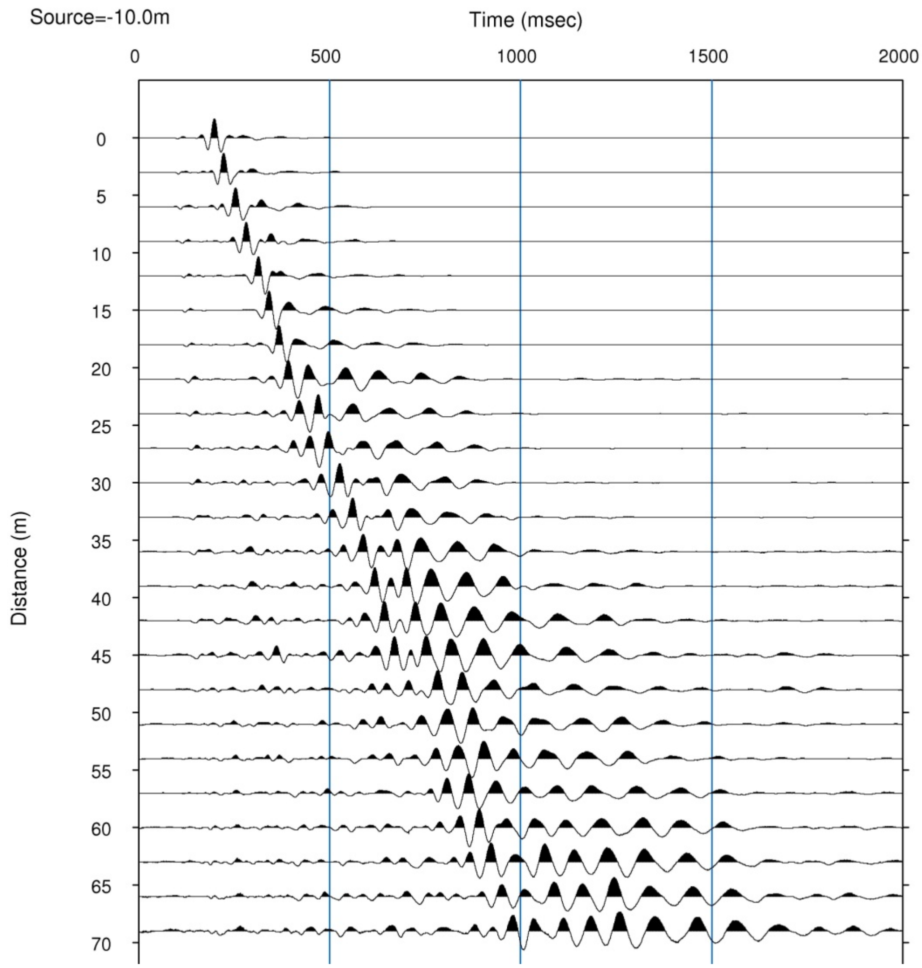


Figure 2: Typical seismic record collected along MASW Line 2.

Data Processing

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

1) Transformation of the time domain data into the frequency domain using a Fast-Fourier Transform (FFT) for each source location;

2) Calculation of the phase for each frequency component;

3) Linear regression to calculate phase velocity for each frequency component;

4) Filtering of the calculated phase velocities based on the Pearson correlation coefficient (r2) between the data and the linear regression best fit line used to calculate phase velocity;

5) Generation of the dispersion curve by combining calculated phase velocities for each shot location of a single MASW test; and

6) Generation of the stiffness profile, through forward iterative modelling and matching of model data to the field collected dispersion curve.

Processing of the MASW data was completed using the SeisImager/SW software package (Geometrics Inc.). The calculated phase velocities for a seismic shot point were combined and the dispersion curve generated by choosing the minimum phase velocity calculated for each frequency component as shown on Figures 3 and 4. Shear wave velocity profiles were generated through inverse modelling to best fit the calculated dispersion curves.

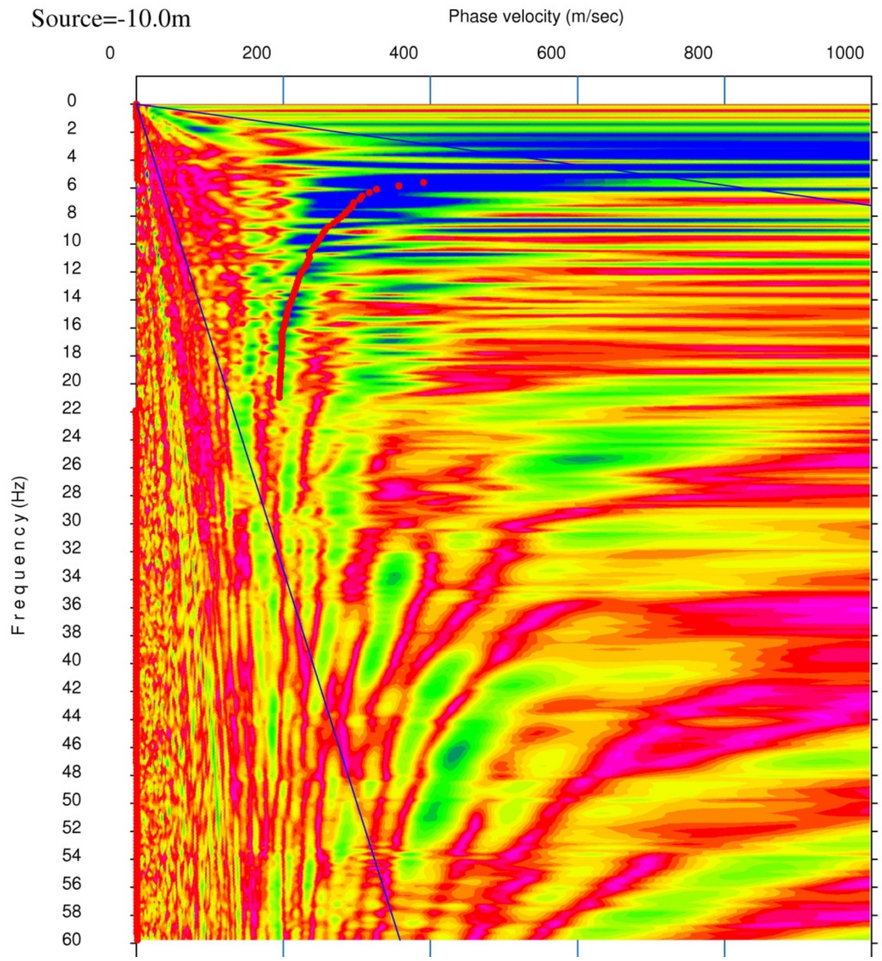


Figure 3: MASW Dispersion Curve Picks for Line 1(red dots).

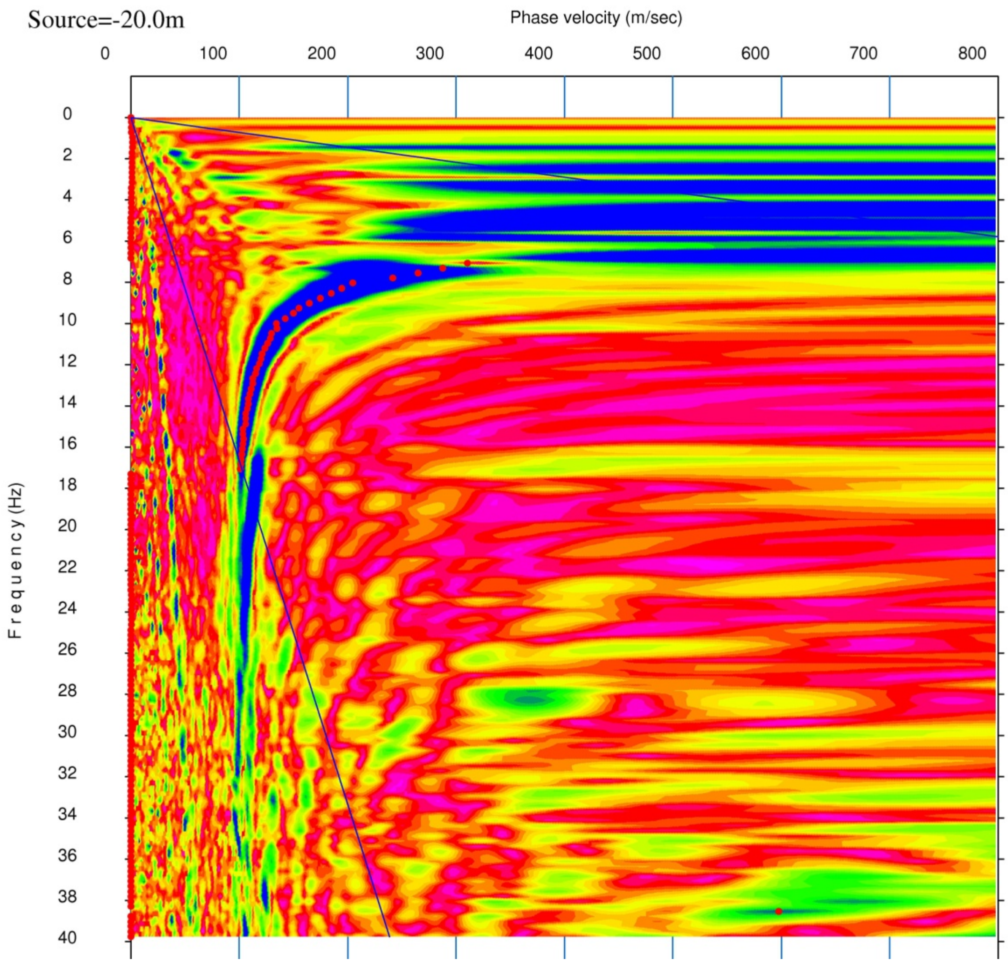


Figure 4: MASW Dispersion Curve Picks for Line 2(red dots).

The minimum measured surface wave frequency with sufficient signal-to-noise ratio to accurately measure phase velocity was approximately 6 Hz and 7 Hz for MASW Lines 1 and 2, respectively.

Results

The MASW test results are presented in Figures 5 and 6, which present the calculated shear wave velocity profiles measured from the field testing at the two locations. The results at each line have been inferred using a weight drop located at 10 m from the first geophone. The field collected dispersion curves are compared with the model generated dispersion curves on Figures 7 and 8. At MASW Line 1 there is a good correlation between the field collected and model calculated dispersion curves, with a root mean squared error of 3.5%. At MASW Line 2 there is an excellent correlation between the field collected and model calculated dispersion curves, with a root mean squared error of 0.8%.

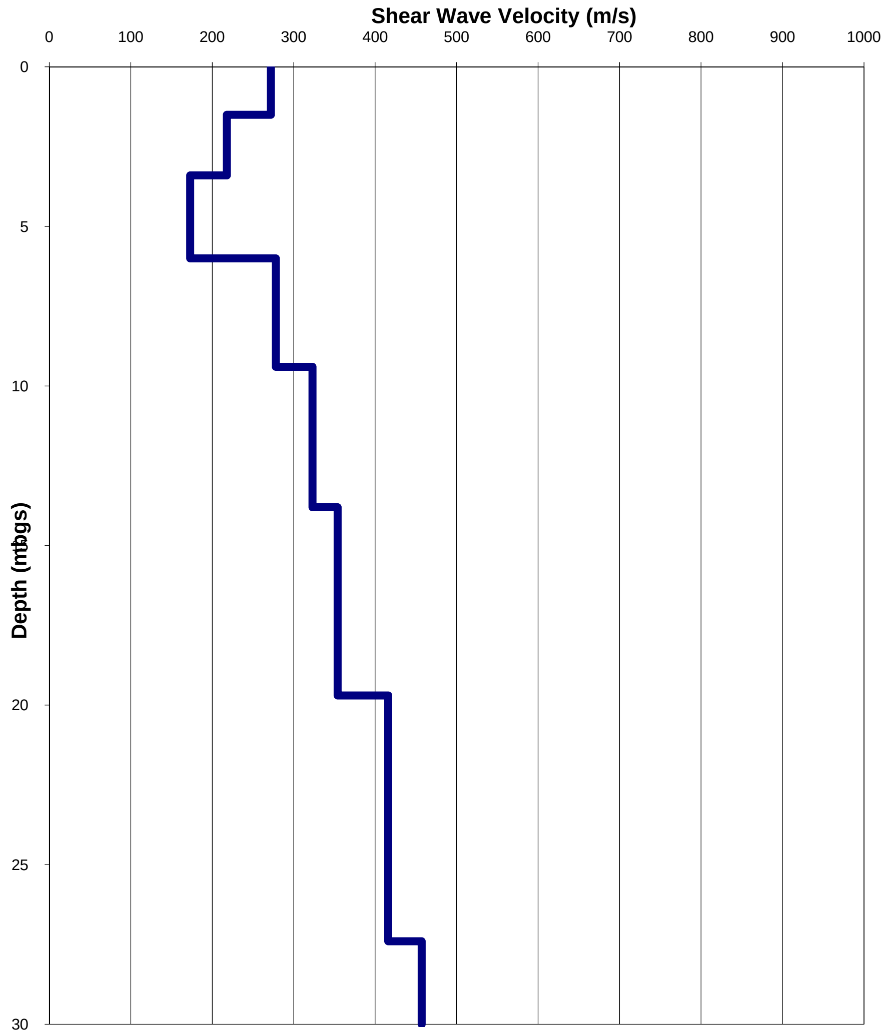


Figure 5: MASW Modelled Shear Wave Velocity Depth profile for MASW Line 1.

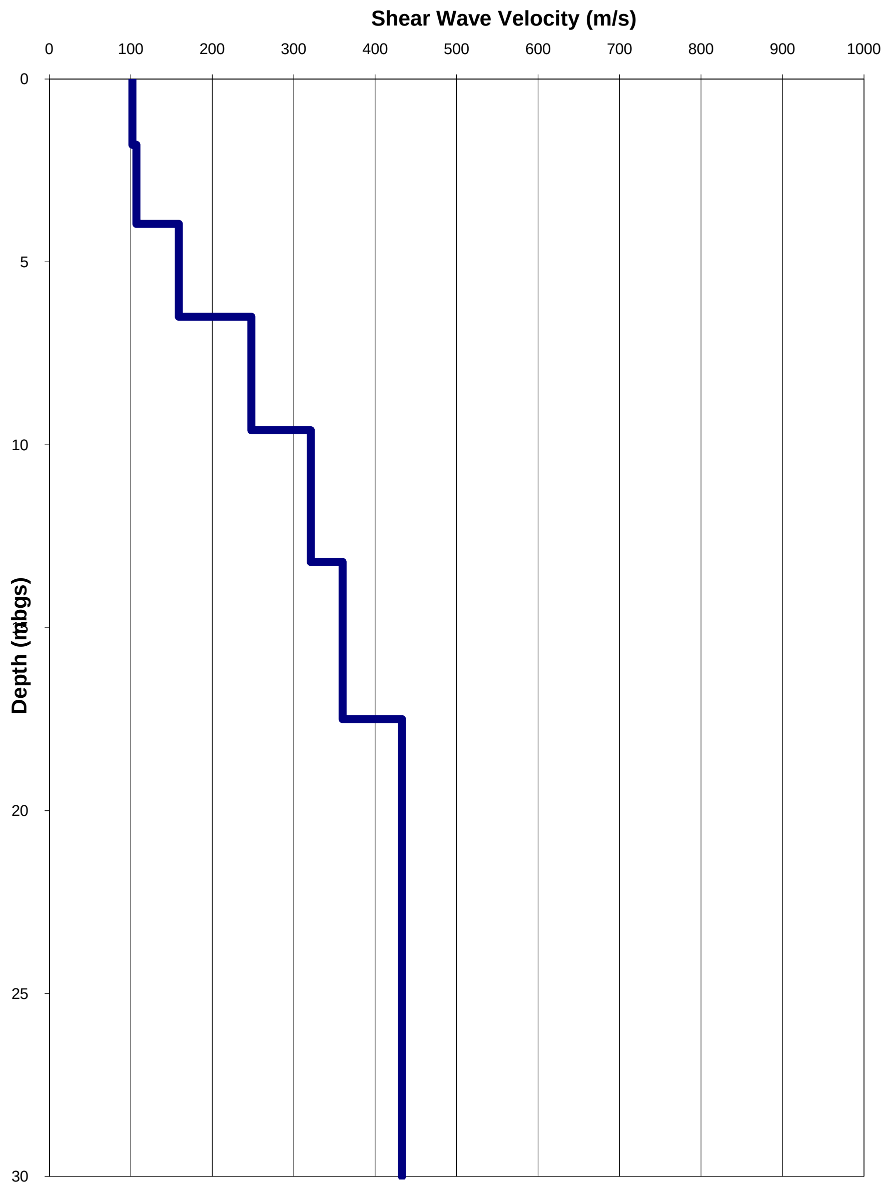


Figure 6: MASW Modelled Shear Wave Velocity Depth profile for MASW Line 2.

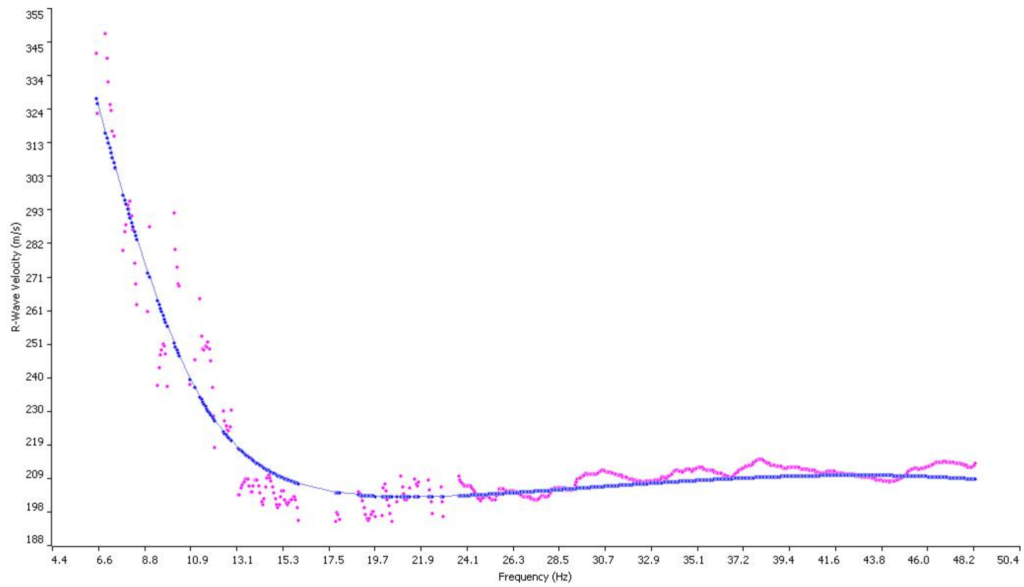


Figure 7: Comparison of Field (pink dots) vs. Modelled Data (blue dots) for the MASW Line 1.

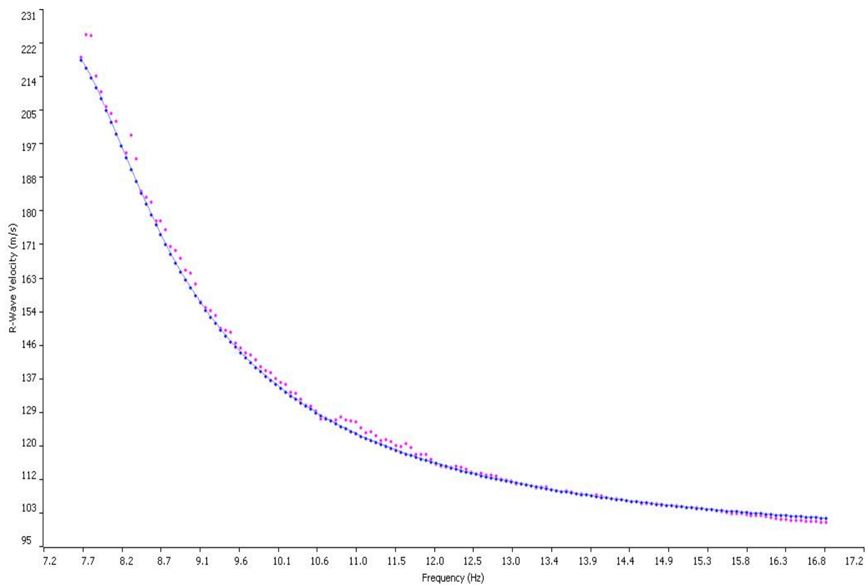


Figure 8: Comparison of Field (pink dots) vs. Modelled Data (blue dots) for the MASW Line 2.

To calculate the average shear wave velocity as required by the National Building Code of Canada, 2005 (NBCC2005), the results were modelled to 30 metres below ground surface.

At MASW Line 1, the limited low frequency content of the dispersion curve did not allow us to sufficiently resolve shear-wave velocities at depth below 27 m. Therefore the average velocity was calculated assuming that the velocity from the maximum resolved depth to a depth of 30 m was constant and equal to the velocity of the maximum resolved depth layer. The average shear-wave velocity was found to be 313 m/s (Table 2).

At MASW Line 2, the limited low frequency content of the dispersion curve did not allow us to sufficiently resolve shear-wave velocities at depth below 17.5 m. Therefore the average velocity was calculated assuming that the velocity from the maximum resolved depth to a depth of 30 m was constant and equal to the velocity of the maximum resolved depth layer. The average shear-wave velocity was found to be 254 m/s (Table 3).

Table 2: Shear Wave Velocity Profile MASW Line 1

Model Layer (mbgs)		Layer Thickness (m)	Shear Wave Velocity (m/s)	Shear Wave Travel Time Through Layer (s)
Top	Bottom			
0.00	1.50	1.50	272	0.005515
1.50	3.40	1.90	218	0.008716
3.40	6.00	2.60	173	0.015029
6.00	9.40	3.40	278	0.012230
9.40	13.80	4.40	323	0.013622
13.80	19.70	5.90	354	0.016667
19.70	27.40	7.70	416	0.018510
27.40	30.00	2.60	457	0.005689
Vs Average to 30 mbgs (m/s)				**313**

Table 3: Shear Wave Velocity Profile MASW Line 2

Model Layer (mbgs)		Layer Thickness (m)	Shear Wave Velocity (m/s)	Shear Wave Travel Time Through Layer (s)
Top	Bottom			
0.00	1.80	1.80	102	0.017647
1.80	3.96	2.16	107	0.020187
3.96	6.50	2.54	159	0.015975
6.50	9.60	3.10	248	0.012500
9.60	13.20	3.60	321	0.011215
13.20	17.50	4.30	360	0.011944
17.50	30.00	12.50	433	0.028868
Vs Average to 30 mbgs (m/s)				**254**

Closure

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

Stephane Sol, Ph.D.
Geophysics Group

Christopher Phillips, M.Sc.
Senior Geophysicist, Associate

SS/CRP/wlm

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APPENDIX I

Results of Chemical Analyses (2023)

Certificate of Analysis

eurofins | Environment Testing

Client:	Golder Associates Ltd. (Ottawa) 1931 Robertson Road Ottawa, ON K2H 5B7	Report Number:	1995130
		Date Submitted:	2023-03-24
		Date Reported:	2023-03-31
		Project:	21482114
		COC #:	906541
Attention:	Chaitanya Raj Goyal		
PO#:			
Invoice to:	WSP Canada Inc.		

Dear Chaitanya Raj Goyal:

Please find attached the analytical results for your samples. If you have any questions regarding this report, please do not hesitate to call (613-727-5692).

Report Comments:

APPROVAL: ______________________________

Raheleh Zafari, Environmental Chemist

All analysis is completed at Eurofins Environment Testing Canada Inc. (Ottawa, Ontario) unless otherwise indicated.

Eurofins Environment Testing Canada Inc. (Ottawa, Ontario) is accredited by CALA, Canadian Association for Laboratory Accreditation to ISO/IEC 17025 for tests which appear on the scope of accreditation. The scope is available at: https://directory.cala.ca/.

Eurofins Environment Testing Canada Inc. (Ottawa, Ontario) is licensed by the Ontario Ministry of the Environment, Conservation, and Parks (MECP) for specific tests in drinking water (license #2318). A copy of the license is available upon request.

Eurofins Environment Testing Canada Inc. (Ottawa, Ontario) is accredited by the Ontario Ministry of Agriculture, Food, and Rural Affairs for specific tests in agricultural soils.

Please note: Field data, where presented on the report, has been provided by the client and is presented for informational purposes only. Guideline values listed on this report are provided for ease of use (informational purposes) only. Eurofins recommends consulting the official provincial or federal guideline as required. Unless otherwise stated, measurement uncertainty is not taken into account when determining guideline or regulatory exceedances.

Certificate of Analysis

eurofins Environment Testing

Client:	Golder Associates Ltd. (Ottawa) 1931 Robertson Road Ottawa, ON K2H 5B7	Report Number:	1995130
Attention:	Chaitanya Raj Goyal	Date Submitted:	2023-03-24
PO#:		Date Reported:	2023-03-31
Invoice to:	WSP Canada Inc.	Project:	21482114
		COC #:	906541

Group	Analyte	MRL	Units	Guideline	1679159	1679160	1679161	1679162
				Lab I.D.	1679159	1679160	1679161	1679162
				Sample Matrix	Soil	Soil	Soil	Soil
				Sample Type				
				Sampling Date	2023-01-17	2023-01-19	2023-01-24	2023-01-27
				Sample I.D.	22-01 Sa4 / 7.5-9.5'	22-02 Sa5 / 10-12'	22-03 Sa5 / 10-12'	22-04 Sa3 / 5-7'
Anions	Cl	0.002	%		0.002	<0.002	0.004	0.002
	SO4	0.01	%		0.03	<0.01	0.01	<0.01
General Chemistry	Electrical Conductivity	0.05	mS/cm		0.49	0.17	0.25	0.16
	pH	2.00			7.77	7.96	7.67	7.73
	Resistivity	1	ohm-cm		2041	5882	4000	6250

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

Certificate of Analysis

Eurofins Environment Testing

Client:	Golder Associates Ltd. (Ottawa) 1931 Robertson Road Ottawa, ON K2H 5B7	Report Number:	1995130
		Date Submitted:	2023-03-24
		Date Reported:	2023-03-31
		Project:	21482114
Attention:	Chaitanya Raj Goyal	COC #:	906541
PO#:			
Invoice to:	WSP Canada Inc.		

QC Summary

Analyte	Blank	QC % Rec	QC Limits
Run No 439384 **Analysis/Extraction Date** 2023-03-30 **Analyst** AsA **Method** C CSA A23.2-4B			
Chloride	<0.002 %		90-110
Run No 439452 **Analysis/Extraction Date** 2023-03-31 **Analyst** IP **Method** Cond-Soil			
Electrical Conductivity	<0.05 mS/cm	100	90-110
pH	7.12	99	90-110
Resistivity			
Run No 439455 **Analysis/Extraction Date** 2023-03-31 **Analyst** IP **Method** AG SOIL			
SO4	<0.01 %	97	70-130

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 J

Results of Chemical Analyses (2017)

Certificate of Analysis

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

Report Number: 1801130
Date Submitted: 2018-01-23
Date Reported: 2018-01-30
Project: 1670692
COC #: 827674

Group	Analyte	MRL	Units	Guideline	1342189	1342190	1342191
				Lab I.D.	1342189	1342190	1342191
				Sample Matrix	Soil	Soil	Soil
				Sample Type			
				Sampling Date	2018-01-23	2018-01-23	2018-01-23
				Sample I.D.	17-202 SA4/15-17	17-204 SA3/10-12	17-207 SA3/10-12
Agri. - Soil	pH	2.00			8.08	8.25	7.63
	SO4	0.01	%		<0.01	<0.01	0.04
General Chemistry	Cl	0.002	%		<0.002	<0.002	<0.002
	Electrical Conductivity	0.05	mS/cm		0.10	0.12	0.38
	Resistivity	1	ohm-cm		10000	8330	2630

Guideline = *** = Guideline Exceedence**

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

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