DRAFT REPORT

Geotechnical Assessment Proposed Residential Development

3930 and 3960 Riverside Drive, Ottawa, Ontario

Submitted to:

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Distribution List

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ATTACHMENTS

APPENDICES

1.0 INTRODUCTION

Golder Associates Ltd. (Golder) 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. Additional intrusive site investigation (by advancing boreholes and CPT holes) will also be undertaken in January 2023 to support detailed design and address potential geotechnical concerns.

The purpose of this report is to assess (based on previous site investigations) 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 19 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 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 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 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 19 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.

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.

2.2 Proposed Residential Development

It is understood that the residential development proposed at this site will 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 the 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 diametre of the sewer is about 450 mm. The proposed invert depth of the sanitary sewer across the site ranges from about 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 diametre 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 diametre 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 and 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. 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.

3.0 PROCEDURE

3.1 Review of Previous Investigations

For the present assessment, subsurface information for the site was collected from several previous geotechnical investigations carried out by Golder Associates. No intrusive investigation works such as boreholes, test pits and the like were carried out for this study.

The results of the previous investigations are presented in the following Golder Associates reports:

- 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.

Geotechnical information for the lowland area on the north part of the site is also available from the report prepared by McRostie Genest St-Louis and Associates (MGS) for 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).

In addition to reviewing the 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 a total of six 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 and available topographic elevation contours, is provided in Section 4.0.

3.2 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 Plan, 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 Golder 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

In general, the subsurface conditions consist of variable thicknesses of random fill material (generally loose to compact silty sand, sand, silty clay, or clayey silt 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 19 m within the South Area (table land). Within the North Area (table land), the fill thickness ranges from about 3 to 9 m. The groundwater level was generally measured between about 5 and 7 m depths (i.e., about elevations 86 to 88 m) in the North Area but was found to be as deep as about 16 m (i.e., about elevations 77 to 78 m) in the South Area. The groundwater level was typically observed at or slightly above the interface between the fill and native soils. The bedrock was encountered only in one borehole at an elevation of about 65 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 available 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 table land ground surface elevation 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.

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 previous investigations define subsurface conditions within the table land, while borehole 103 defines the subsurface conditions with the slope area. Records of previous borehole and test pit logs are shown in Appendix A.

Significant infilling of the former sand pits was carried out through 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 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. Based on this information, the fill thickness is expected to vary between about 5 and 19 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 dense in state of packing but is generally in a loose to compact state. Based on the borehole information and a review of topographic elevation contours from previous investigation reports, 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 crust silty clay (about 0.8 m thick) exists below the fill at boreholes 103, 104, and 105, located along the south and west edges of the site. The fill is otherwise underlain by a sand deposit with varying amounts of silt and gravel, that transitions into a very dense sand and gravel deposit with depth 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.

In borehole 17-205, the sand deposit doesn’t exist, and the fill is directly underlain by very dense sandy gravel at a depth of about 15 m below the ground surface (elevation 78.2 m). 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 (at a depth of about 30 m beneath the current ground level).

A deposit of very stiff clayey silt exists below the sand deposit in 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 extend to a depth of about 26.7 m below the ground surface (elevation 72.7 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 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 76 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 76 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 and 4, 2017 in boreholes 17-03 and 17-01, respectively, and are summarized in the table below.

Borehole Number	Geological Unit	Ground Surface Elevation (m)	Water Level Depth (m)	Water Level Elevation (m)	Date of Measurement
17-01	Sandy Silt	94.8	16.4	78.4	May 4, 2017
17-03 (at the boundary of South and North Area)	Sand	94.3	6.6	87.6	May 3, 2017

The groundwater level in 2017 was observed slightly above the interface between the fill and the native sand in borehole 17-01. The groundwater level was about 3.5 m below the interface between the fill and the native sand in borehole 17-03 which is at the boundary of the South and North Area. 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 of 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 area is higher than the lowland area by about 8 m (due to the placement of fill material within the upland area).

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 define the subsurface conditions within the table land, while 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 previous borehole and test pit logs are shown in Appendix A.

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 original/native 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 extreme east part where the ground level rises up to Riverside Drive. Based on this information, the fill thickness is expected to generally 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 very loose to compact.

The fill is underlain by a thick sand deposit which generally contains variable amounts of silt, gravel, and clayey silty seams.

This 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 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 sand deposit 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 78 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) and on January 19, 2018 (in boreholes 17-201 and 17-203) and are summarized in the following table.

Borehole Number	Geological Unit	Ground Surface Elevation (m)	Water Level Depth (m)	Water Level Elevation (m)	Date of Measurement
17-201	Sand to sandy silty clay	91.2	5.2	86.0	Jan. 19, 2018
17-203	Sand and silty clay	93.6	5.8	87.8	Jan. 19, 2018
17-07	Sand	93.8	5.6	88.2	May 2, 2017

The groundwater levels measured in 2017 and 2018 were generally observed slightly above the interface between the fill and the native sand. 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 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 %. 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 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 B2 and B3 in Appendix B.

4.7.2 Sand with silt and gravel

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 B4 and B5 in Appendix B.

As noted previously, relatively thin layers of very stiff cohesive soils exist within the thicker 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 B6 and B7.

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 Golder 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 preliminary engineering guidelines on the geotechnical design aspects of developing this site based on our interpretation of the 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 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 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 this area ranges between about 10 and 19 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 76 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 up to Riverside Drive, (about elevations 99 - 100 m), along the east side of the site.

The soil conditions encountered in the previous 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.

- The fill materials and the native coarse-grained soils below the water table (at some locations) are potentially liquefiable under seismic events.
- 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 is located in an area where there exists a history of earthquake activity and saturated granular soils. The potential for seismic liquefaction of the overburden therefore needs to be assessed.

A seismic Site Class also needs to be assigned, 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.

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 boreholes. 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 suggest 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 0.302 g. Ground surface settlements of up to 100 mm could be generated following a seismic event.

Note that the liquefaction assessment carried out for this study is preliminary in nature and a more detailed analysis will be carried out after the drilling program is completed at the site in January 2023. At that time the potential for lateral slope movements will also be assessed. If the soils are found to be liquefiable even after a detailed assessment of the newly obtained data (in 2023), then ground improvement techniques may be used to densify these deeper liquefiable soils to reduce or eliminate their liquefaction potential.

6.3.2 Site Classification for Seismic Site Response

The results of the previous geophysical testing carried out at the site in the form of MASW (multi-channel analyses of surface waves) are presented in Appendix D. For sites where potential liquefaction is a concern (following additional investigation and analysis), as identified above for this site, the 2012 OBC requires a Site Class of F (i.e., special soils) designation. The 2012 OBC allows the use of a “non-liquefied” seismic site class where the fundamental period of the proposed structure is less than 0.5 seconds (i.e., typically 3 stories or less). Thus, for preliminary planning purposes, a seismic site class designation of Site Class D, based on the MASW results, appears appropriate for buildings with a fundamental period of less than 0.5 seconds. For Structures with a fundamental period greater than 0.5 seconds, the development of a site-specific response spectra will be required unless a ground improvement program to mitigate potential liquefaction is undertaken.

6.4 Slope Stability Assessment

6.4.1 General

The evaluation of the stability of a slope depends on several param, 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 param 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 C.

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 param 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 param 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 param (as given above) were used under both the static and seismic loading conditions for cohesionless soils. The undrained param 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 param 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.

If the preliminarily identified liquefaction potential at the site is confirmed during the upcoming detailed assessment, a “post-earthquake” stability analysis with liquefied soil strengths will also need to be undertaken in consideration of any ground improvement works, with such works also likely leading to improved slope stability conditions in some areas.

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”.

As discussed earlier, these analyses were carried out using soil param consistent with the soil not being vulnerable to liquefaction during an earthquake.

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, provided there is no seismic liquefaction at the site (which must be confirmed through additional testing and analysis) but should be re-assessed during final design to address any potentially liquefiable areas.

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.

It is assumed that the ground level within the South Area (i.e., within that area adjacent to the highest and least stable slopes) is unlikely to be raised significantly. However, the location of the Limit of Hazard Lands will need to be re-assessed once the final site grading has been confirmed. Increases 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 does 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 more general guideline, grading of the site should direct surface runoff away from the slopes into drainage channels designed specifically for this purpose. 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 3H: 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 19 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 diametre 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 19 m)

In the southern portion of the site, the fill materials are the thickest (up to about 19 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 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.

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. Golder Associates 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 19 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 17 203). 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), 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.

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 diametre 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 76 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 diametre 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 and Climate Change (MOECC) 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 MOECC. 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;

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

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 19 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 perim 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 pavement could be expected to perform better in the long term if the granular backfill against the foundation walls is drained by means of a perforated pipe subdrain in a surround of 19 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 building areas, imported engineered fill should be used.

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

As part of a previous investigation, samples of soil from 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 are provided in Appendix E and are summarized below.

Borehole/ Sample Number	**Geological Unit**	**Depth (m)**	**pH**	Sulphates (%)	Chlorides (%)	Resistivity (Ohm-cm)
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. The fill material could therefore be left in place provided some post-construction settlement of the pavement surface could be accepted. To help minimize the settlement, the thickness of the subbase material should be significantly increased (see pavement structures below). In addition, the surface of the fill material at subgrade level should be proof rolled with a heavy smooth drum roller under the supervision of qualified geotechnical personnel to compact the existing fill and to identify soft areas requiring sub-excavation and replacement with more suitable fill. In addition, a woven geotextile may have to be provided. Ground improvement could be carried out to reduce the amount of settlement.

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 controlled 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 controlled fill must be compacted to 100 % of SPMDD. The placement of the controlled 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 the pavement structure, frost compatibility must be maintained across any new service trenches. The subsoil should be inspected by qualified geotechnical personnel to make sure that there is no potential for differential frost heaving. Frost tapers from the bottom of granular subbase to 1.8 m depth should be constructed at 10H:1V and should be provided where necessary.

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

Table 1: 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

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

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. Golder Associates should be retained to review the final drawings and specifications for this project prior to tendering to ensure that the guidelines in this report have been adequately interpreted.

Additional geotechnical investigations are proposed for early 2023. In particular these investigations are intended to help to further define the liquefaction potential at the site, and the need and extent of any required ground improvements. The recommendations of this report (in particular related to foundations, slope stability, utilities and road construction) will be reviewed and updated based on the additional investigation and testing.

Signature Page

Golder Associates Ltd.

DRAFT ***DRAFT***

Chaitanya Goyal, P.Eng.
Geotechnical Engineer

Chris Hendry, P.Eng.
Senior Geotechnical Engineer

CRG/CH/ljv/ml

https://golderassociates.sharepoint.com/sites/150381/project files/6 deliverables/geotechnical report/21482114-3000-001 rpt draft 2022'12'22 geotech report.docx

IMPORTANT INFORMATION AND LIMITATIONS OF THIS REPORT

Standard of Care: Golder Associates Ltd. (Golder) has prepared this report in a manner consistent with that level of care and skill ordinarily exercised by members of the engineering and science professions currently 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 Golder 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. Golder cannot be responsible for use of this report, or portions thereof, unless Golder is requested to review and, if necessary, revise the report.

The information, recommendations and opinions expressed in this report are for the sole benefit of the Client. No other party may use or rely on this report or any portion thereof without Golder's express written consent. If the report was prepared to be included for a specific permit application process, then upon the reasonable request of the client, Golder 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 Golder. The report, all plans, data, drawings and other documents as well as all electronic media prepared by Golder are considered its professional work product and shall remain the copyright property of Golder, who authorizes only the Client and Approved Users to make copies of the report, but only in such quantities as are reasonably necessary for the use of the report by those parties. The Client and Approved Users may not give, lend, sell, or otherwise make available the report or any portion thereof to any other party without the express written permission of Golder. The Client acknowledges that electronic media is susceptible to unauthorized modification, deterioration and incompatibility and therefore the Client 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 Golder by the Client, communications between Golder and the Client, and to any other reports prepared by Golder for the Client relative to the specific site described in the report. In order to properly understand the suggestions, recommendations and opinions expressed in this report, reference must be made to the whole of the report. Golder 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, Golder does not warrant or guarantee the exactness of the descriptions.

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

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

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

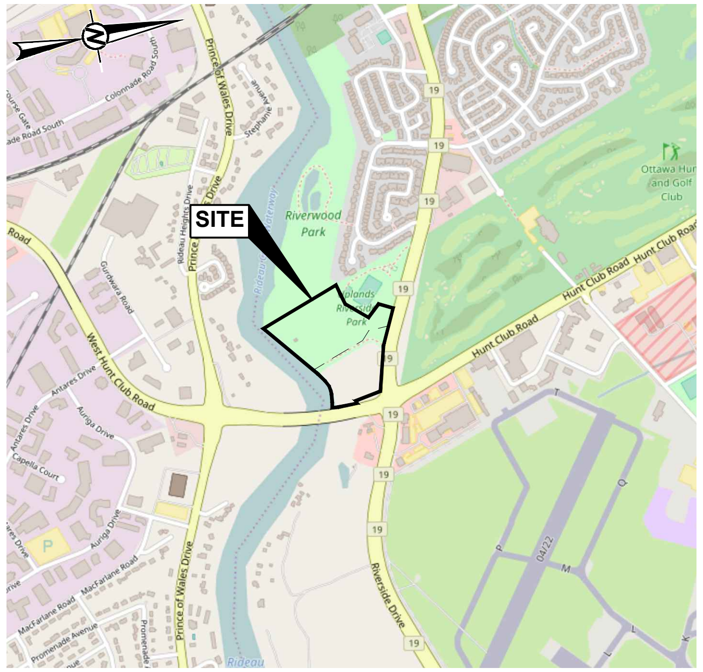
Follow-Up and Construction Services: All details of the design were not known at the time of submission of Golder’s report. Golder should be retained to review the final design, project plans and documents prior to construction, to confirm that they are consistent with the intent of Golder’s report.

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

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

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

KEY MAP



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LEGEND

● 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

🟥 AREA WITH MODERATE TO SEVERE EROSION

🟨 AREA WITH MODERATE EROSION

REFERENCE(S)

1. 1. PROJECTION: TRANSVERSE MERCATOR DATUM: NAD 83,COORDINATE SYSTEM: MTM ZONE 9, VERTICAL DATUM: CGVD28T
2. BASE PLAN PROVIDED IN ELECTRONIC FORMAT BY HOBIN ARCHITECTURE INCORPORATED ON JANUARY 11, 2023, DRAWING NO. Riverside_Residential_georeferenced_230111.dwg

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CLIENT

ST. MARY'S LANDS CORPORATION

PROJECT

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

TITLE

SITE PLAN

CONSULTANT

WSP GOLDER

YYYY-MM-DD	2023-01-11
DESIGNED	CRG
PREPARED	ABD/JM
REVIEWED	CRG
APPROVED	CH

PROJECT NO.	CONTROL	REV.	FIGURE
21482114	0005	A	1

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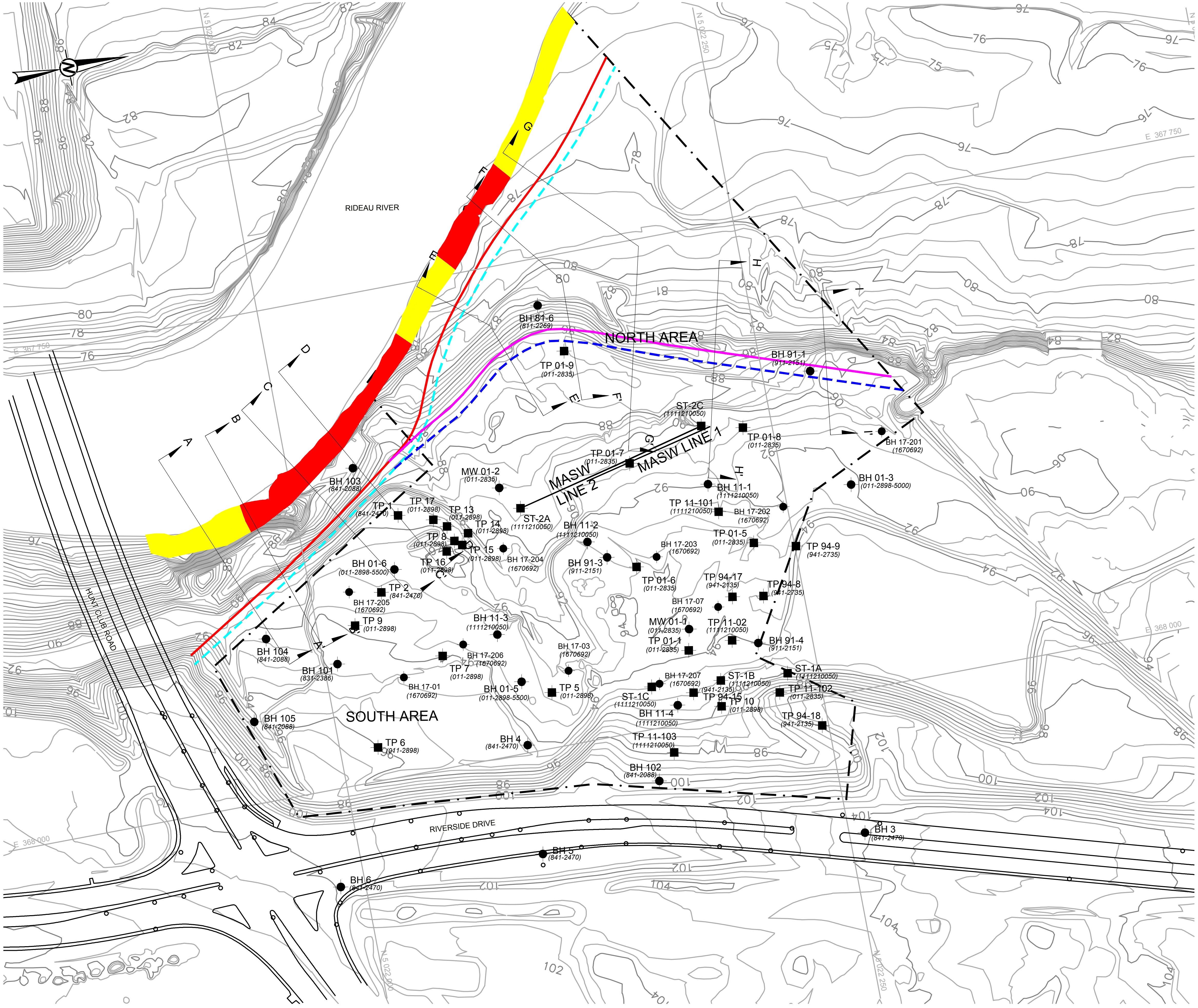

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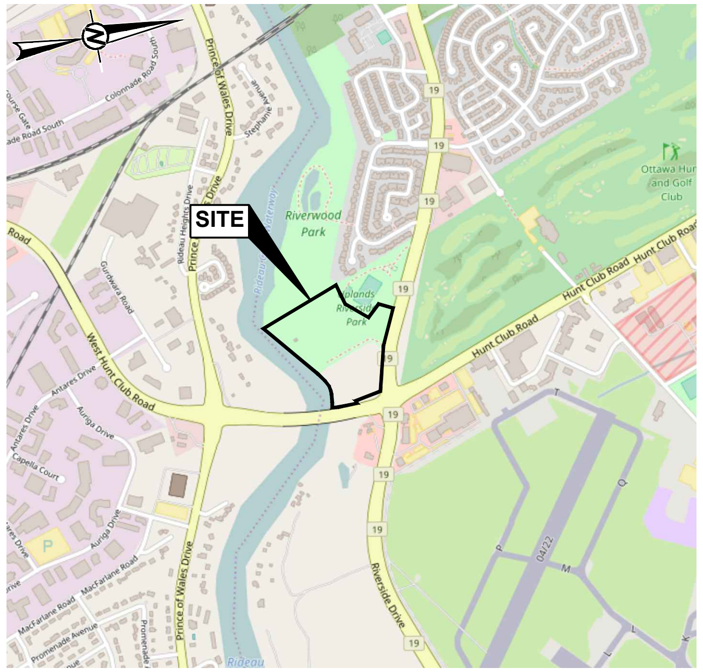

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LEGEND

●	APPROXIMATE BOREHOLE LOCATION, PREVIOUS INVESTIGATION BY GOLDER
■	APPROXIMATE TEST PIT LOCATION, PREVIOUS INVESTIGATION BY GOLDER
A'	APPROXIMATE SLOPE CROSS-SECTION LOCATION IN PLAN
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- - -	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
🟥	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

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0 50 100
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CLIENT
ST. MARY'S LANDS CORPORATION

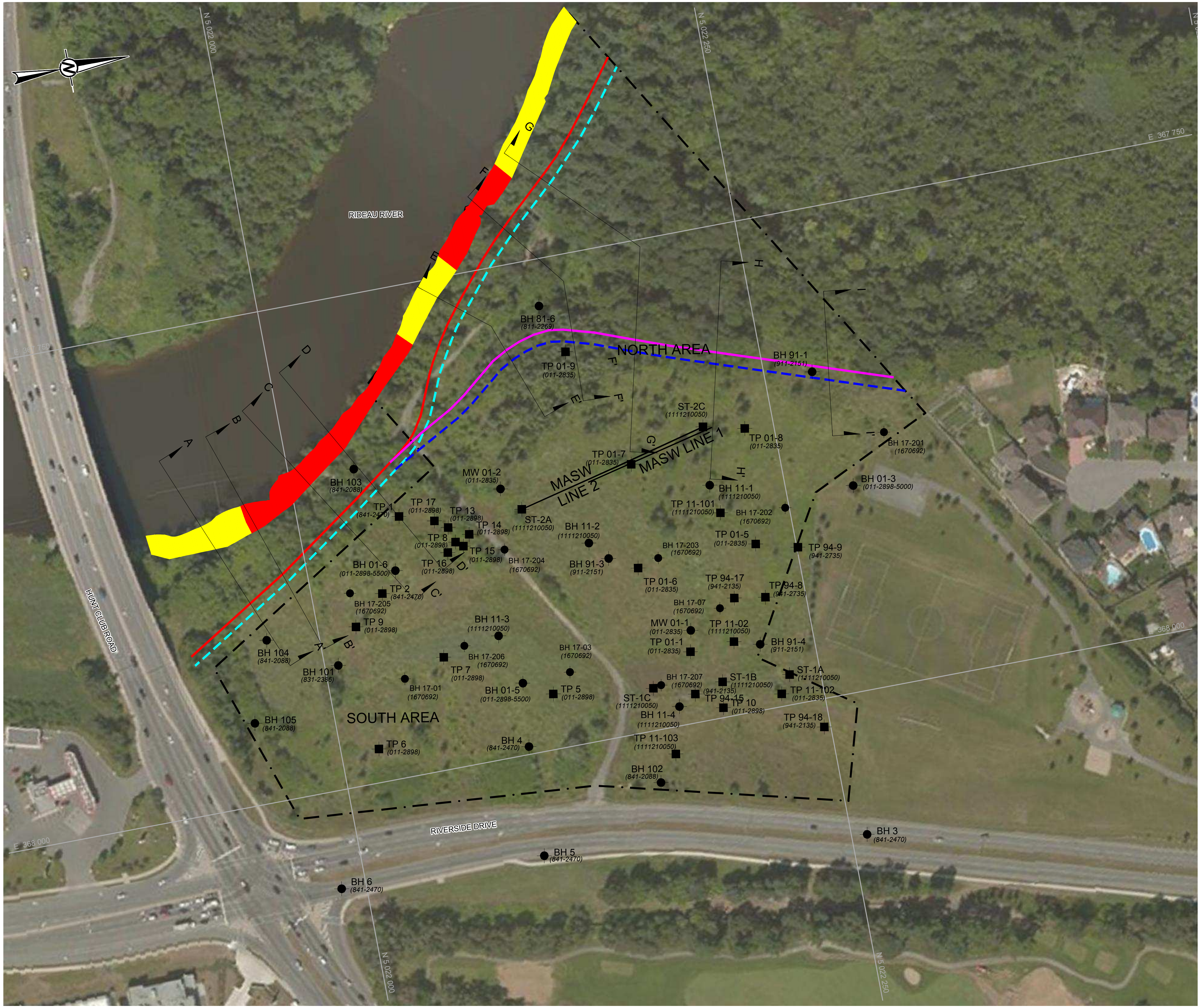
PROJECT
GEOTECHNICAL ASSESSMENT
PROPOSED RESIDENTIAL DEVELOPMENT
3930 AND 3960 RIVERSIDE DRIVE, OTTAWA, ONTARIO

TITLE
SITE PLAN

CONSULTANT
WSP GOLDER

YYYY-MM-DD	2023-01-11
DESIGNED	CRG
PREPARED	ABD/JM
REVIEWED	CRG
APPROVED	CH

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



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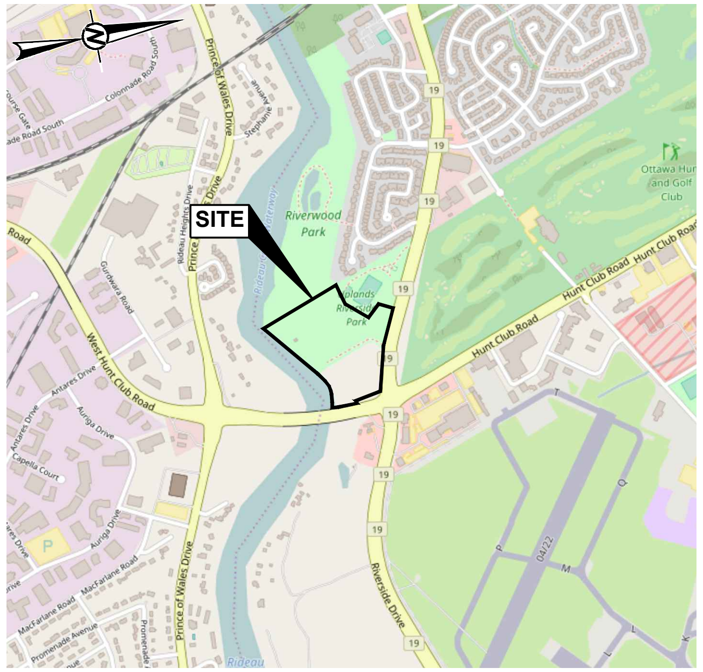

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LEGEND

●	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
🟥	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

DRAFT

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 GOLDER

YYYY-MM-DD	2023-01-11
DESIGNED	CRG
PREPARED	ABD/JM
REVIEWED	CRG
APPROVED	CH

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

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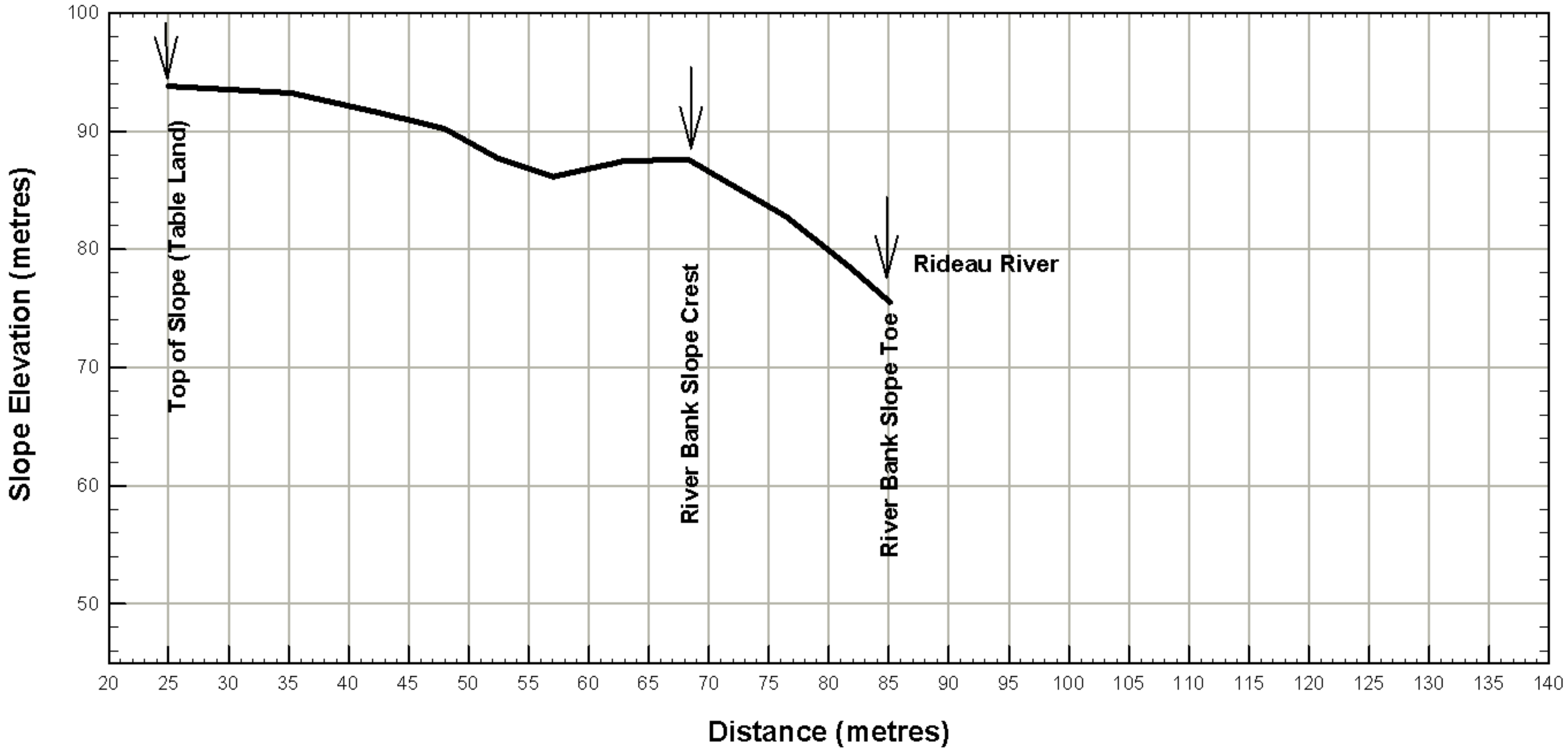
Cross Section A-A'



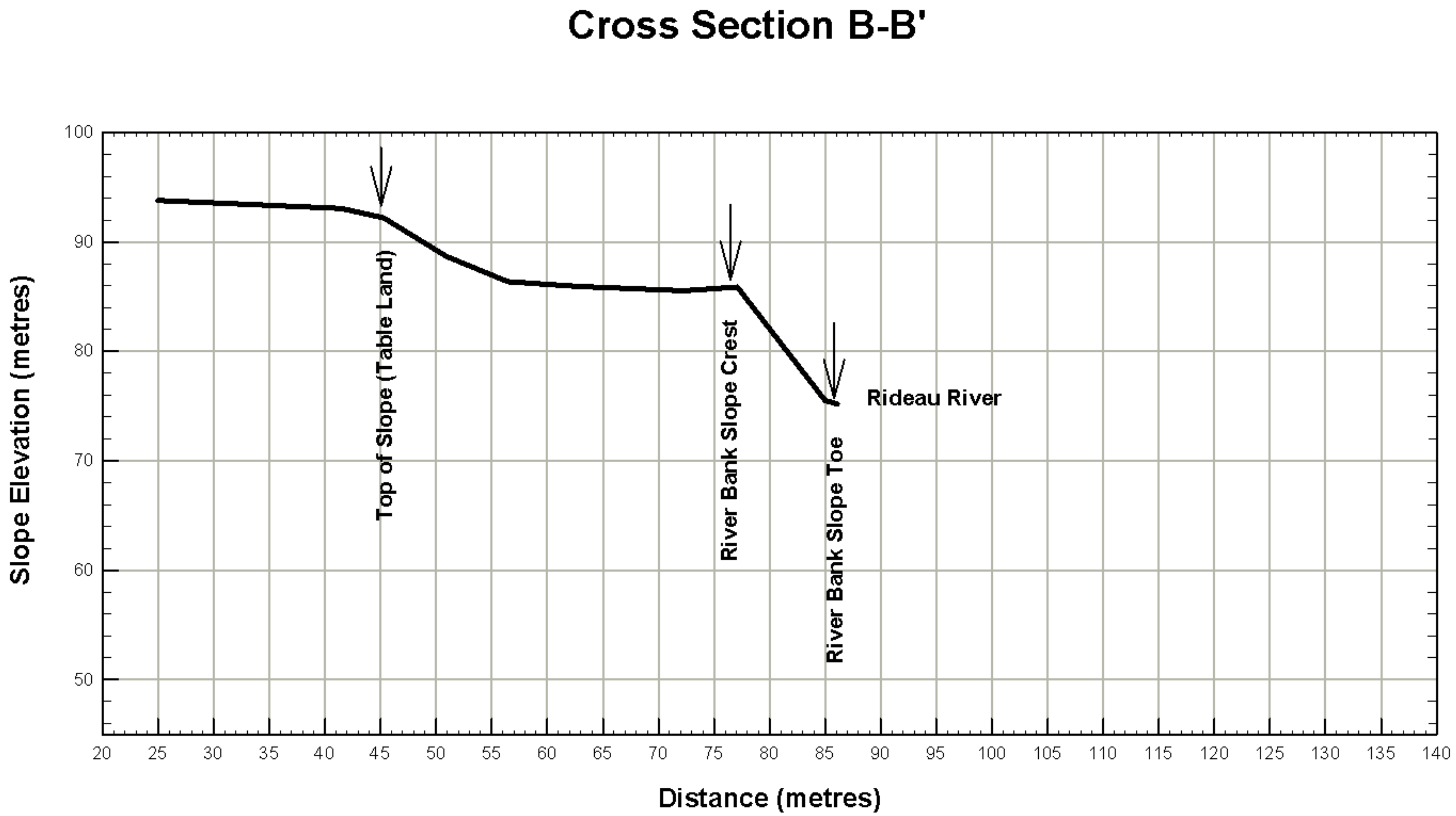
Golder Associates

Date: May 8, 2017 Drawn: WAM

Project: 1670692 Chkd:

SLOPE CROSS SECTION

FIGURE 2



Golder Associates

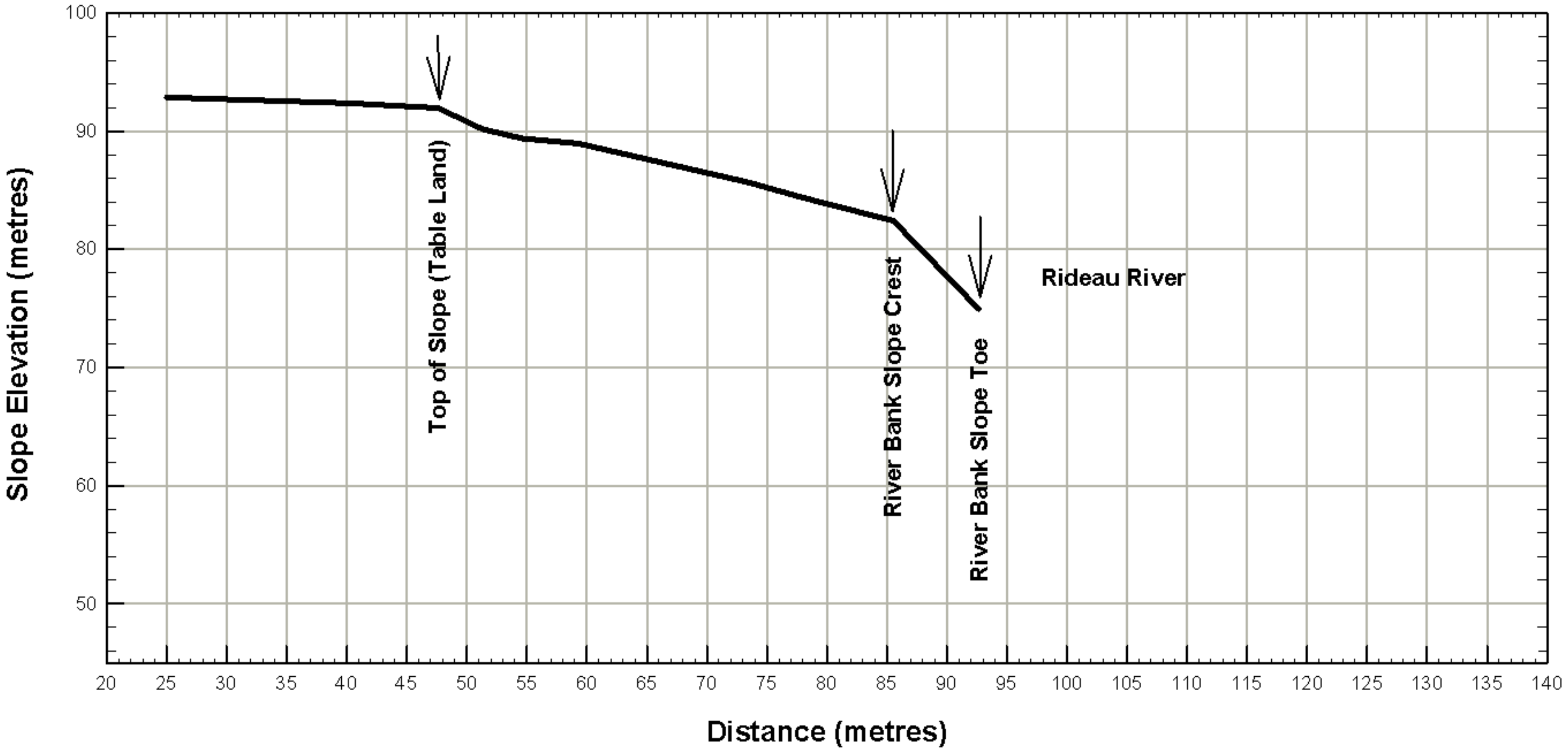
Date: May 8, 2017 Drawn: WAM

Project: 1670692 Chkd:

SLOPE CROSS SECTION

FIGURE 3

Cross Section C-C'



Golder Associates

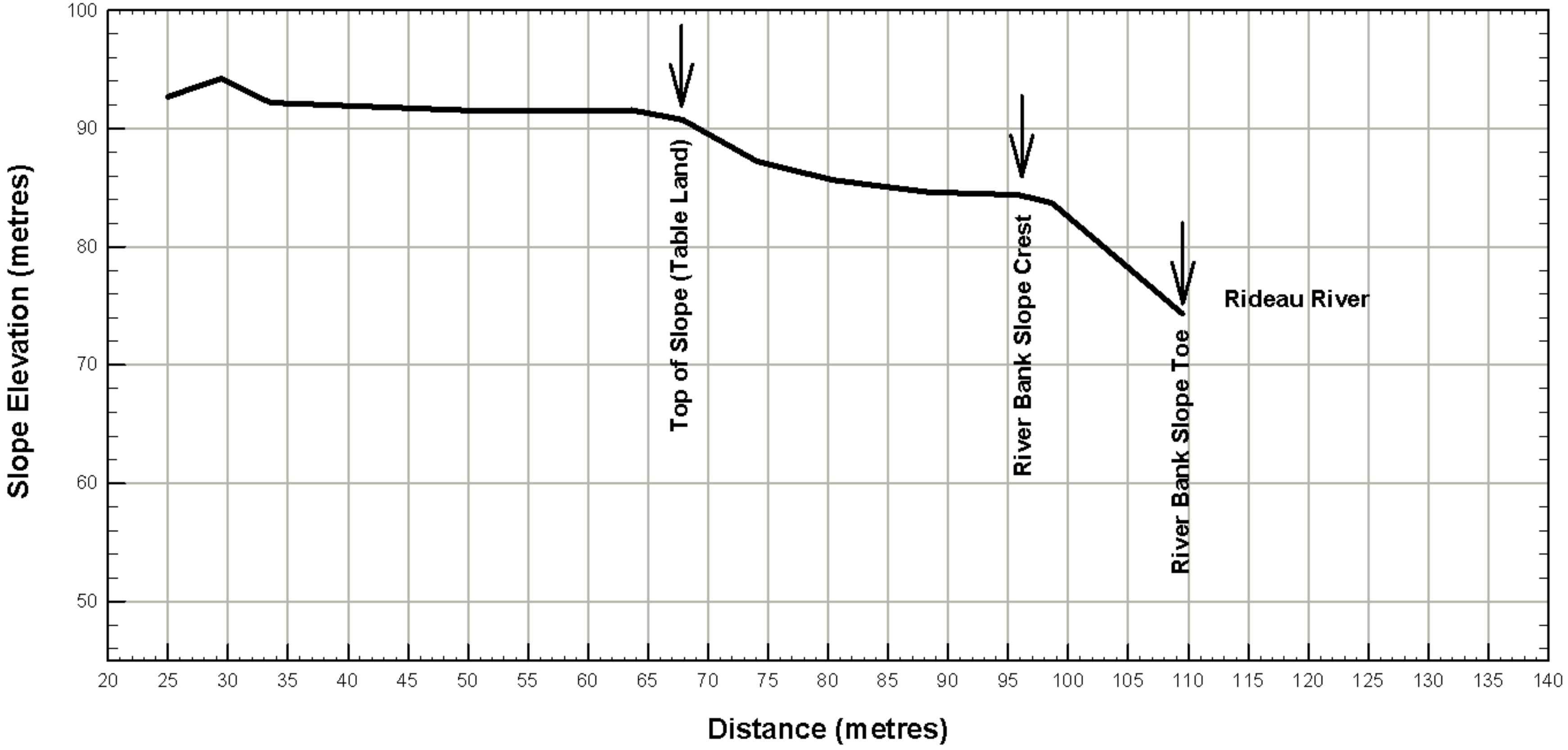
Date: May 8, 2017 Drawn: WAM

Project: 1670692 Chkd:

SLOPE CROSS SECTION

FIGURE 4

Cross Section D-D'



Golder Associates

Date: May 8, 2017

Drawn: WAM

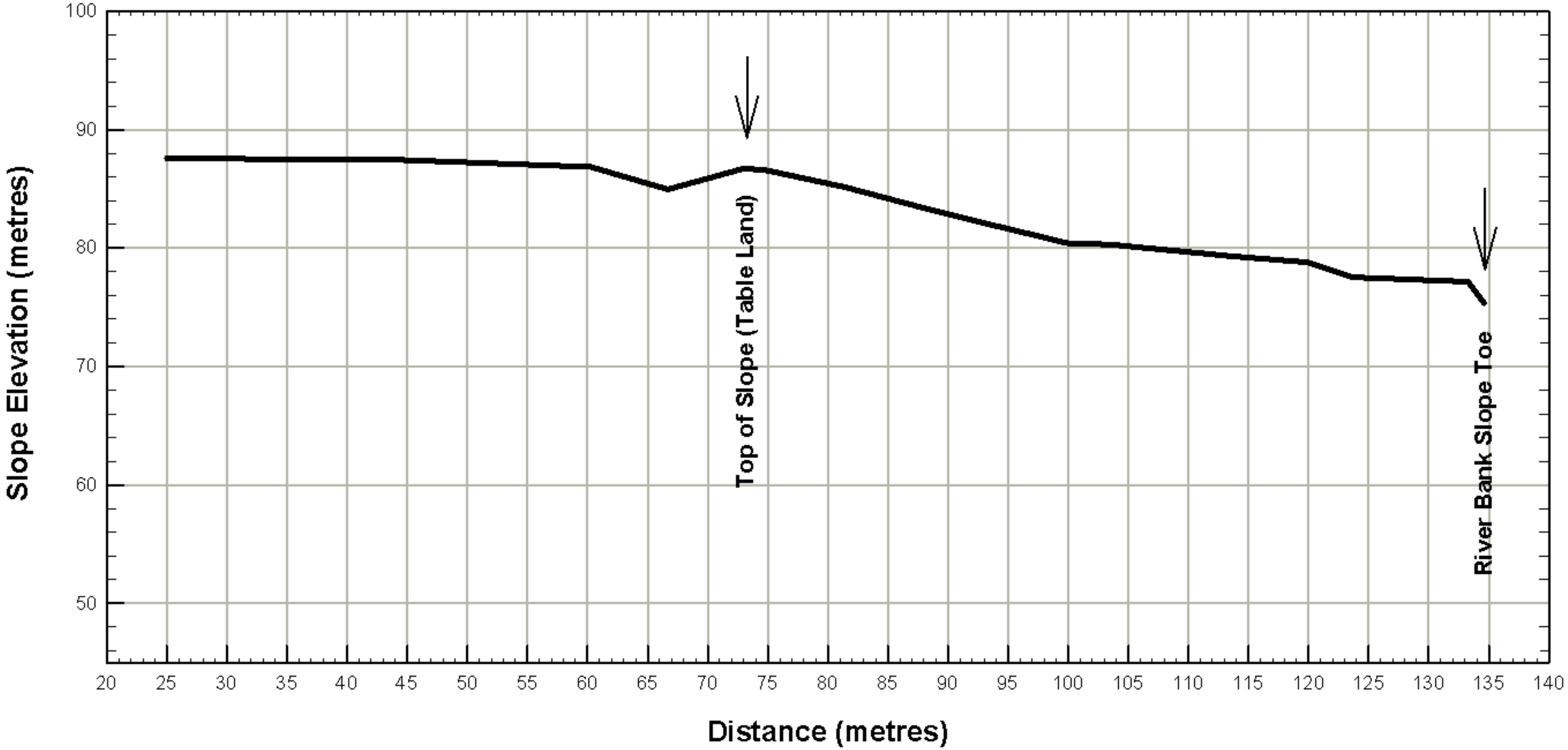
Project: 1670692

Chkd:

SLOPE CROSS SECTION

FIGURE 5

Cross Section E-E'



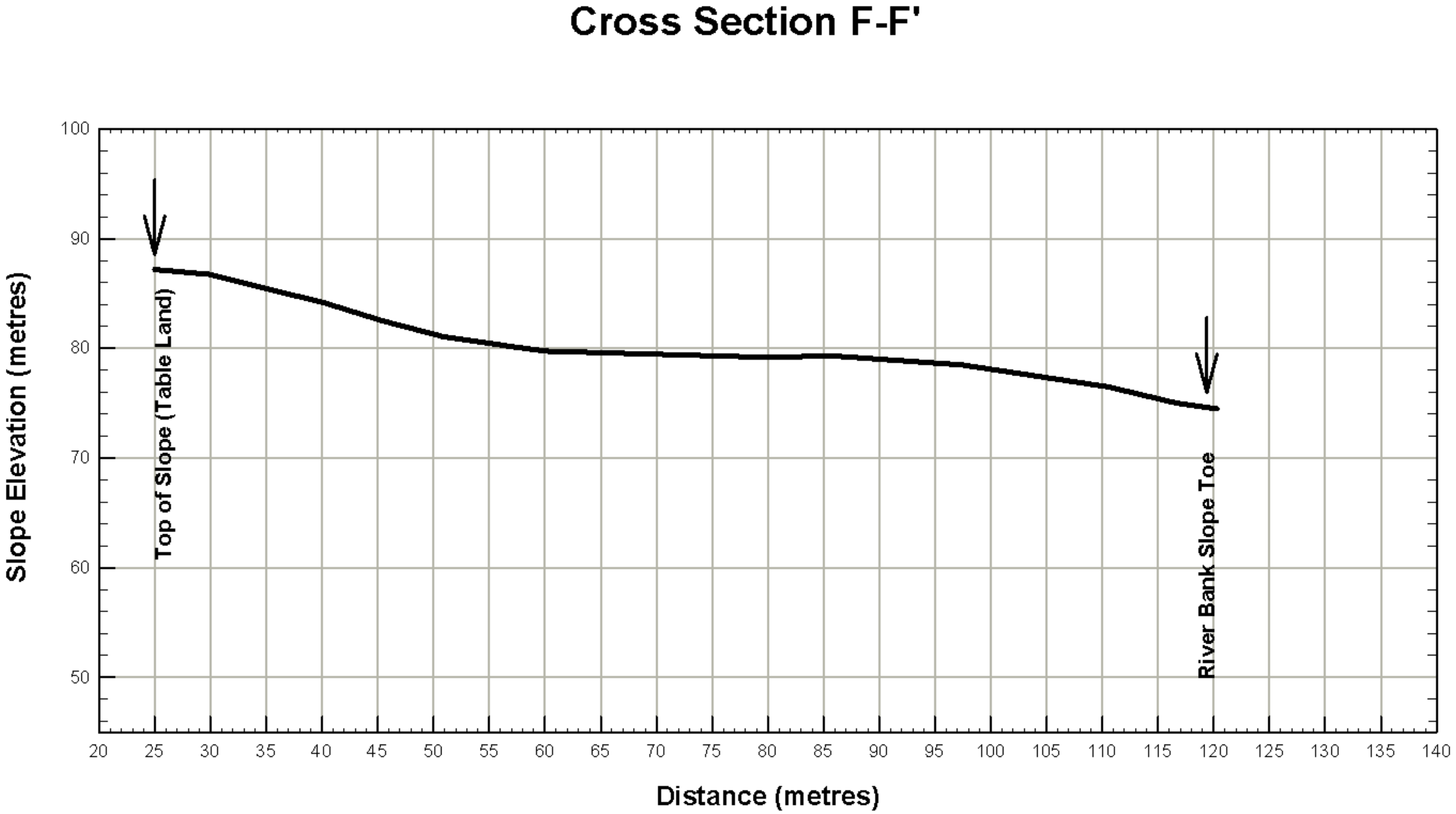
Golder Associates

Date: May 8, 2017 Drawn: WAM

Project: 1670692 Chkd:

SLOPE CROSS SECTION

FIGURE 6



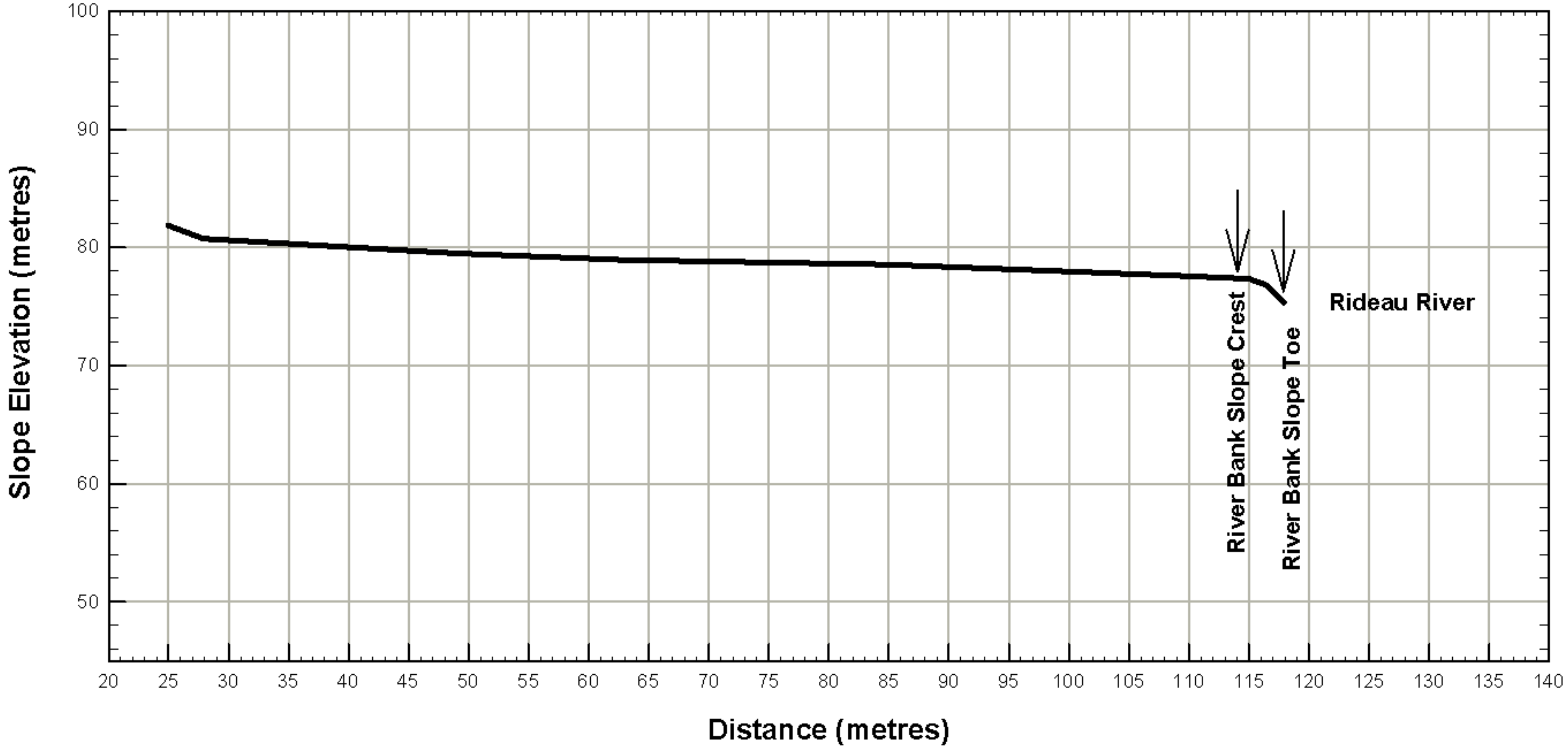
Golder Associates

Date: May 8, 2017
Drawn: WAM
Project: 1670692
Chkd:

SLOPE CROSS SECTION

FIGURE 7

Cross Section G-G'



Date: May 8, 2017 Drawn: WAM

Project: 1670692 Chkd:

SLOPE CROSS SECTION

FIGURE 8

APPENDIX A

Records of Previous Borehole Logs

PROJECT: 1670692

RECORD OF BOREHOLE: 17-201

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

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLES NUMBER	SAMPLES TYPE	BLOWS/0.30m	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		GROUND SURFACE	91.24					
0	Power Auger 200 mm Diam. (Hollow Stem)	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
		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
		(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		
				7	SS	20		31 mm Diam. PVC #10 Slot Screen
				8	SS	17		
		(CI) sandy SILTY CLAY; grey, contains fine sand seams; cohesive, w>PL, very stiff	82.40 8.84	9	SS	8		Silica Sand WL in Screen at Elev. 86.02 m on Jan. 19, 2018
		End of Borehole	81.49 9.75					

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

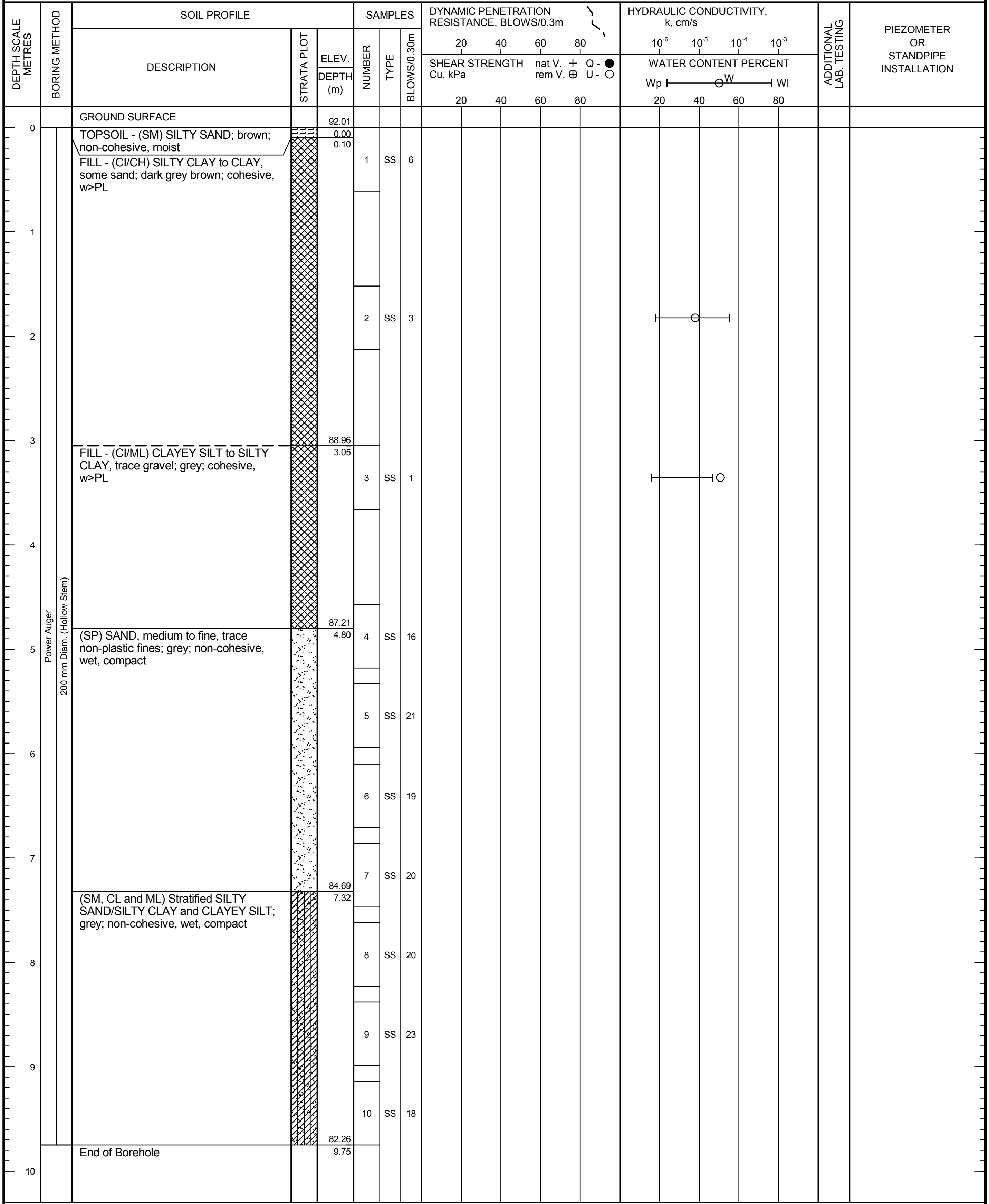
SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

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

SOIL PROFILE DESCRIPTION	ELEV.	DEPTH (m)
GROUND SURFACE	92.01	
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
FILL - (CI/ML) CLAYEY SILT to SILTY CLAY, trace gravel; grey; cohesive, w>PL	88.96	3.05
(SP) SAND, medium to fine, trace non-plastic fines; grey; non-cohesive, wet, compact	87.21	4.80
(SM, CL and ML) Stratified SILTY SAND/SILTY CLAY and CLAYEY SILT; grey; non-cohesive, wet, compact	84.69	7.32
End of Borehole	82.26	9.75

SAMPLES NUMBER	TYPE	BLOWS/0.30m
1	SS	6
2	SS	3
3	SS	1
4	SS	16
5	SS	21
6	SS	19
7	SS	20
8	SS	20
9	SS	23
10	SS	18



DEPTH SCALE

1 : 50

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)

SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	SAMPLES TYPE	BLOWS/0.30m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m 20 40 60 80; SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - ○ 20 40 60 80	HYDRAULIC CONDUCTIVITY, k, cm/s 10^{-6} 10^{-5} 10^{-4} 10^{-3}; WATER CONTENT PERCENT Wp ⊢ ○W ⊣ Wl 20 40 60 80	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
GROUND SURFACE		93.63							
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	SS	3				
			3	SS	1				Native Backfill
			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
			5	SS	21			MH	
			6	SS	19				
(SM/CL) SAND and SILTY CLAY, fine, layered; grey brown; non-cohesive, wet, compact		86.01 7.62	7	SS	20				31 mm Diam. PVC #10 Slow Screen
(SP) SAND, fine to medium, trace non-plastic fines; brown; non-cohesive, wet, compact		85.40 8.23	8	SS	21				
			9	SS	19				Cave
End of Borehole		83.88 9.75							WL in Screen at Elev. 87.82 m on Jan. 19, 2018

DEPTH SCALE

1 : 50

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

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

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

SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)
GROUND SURFACE	90.34
FILL - (SM/GM) SILTY SAND and GRAVEL; grey brown; non-cohesive, moist, loose	0.00
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
(SP/SM) SAND, fine to coarse, some non-plastic fines, trace gravel; brown; non-cohesive, moist, dense to very dense	86.38 3.96
End of Borehole	80.59 9.75

NUMBER	TYPE	BLOWS/0.30m	ADDITIONAL LAB. TESTING
1	SS	6	
2	SS	11	MH
3	SS	9	
4	SS	51	
5	SS	54	M
6	SS	54	
7	SS	42	
8	SS	34	
9	SS	38	
10	SS	35	

PIEZOMETER OR STANDPIPE INSTALLATION

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: 17-205

SHEET 1 OF 2

LOCATION: N 5022019.8 ;E 367901.3

BORING DATE: December 14-15, 2017

DATUM: CGVD28

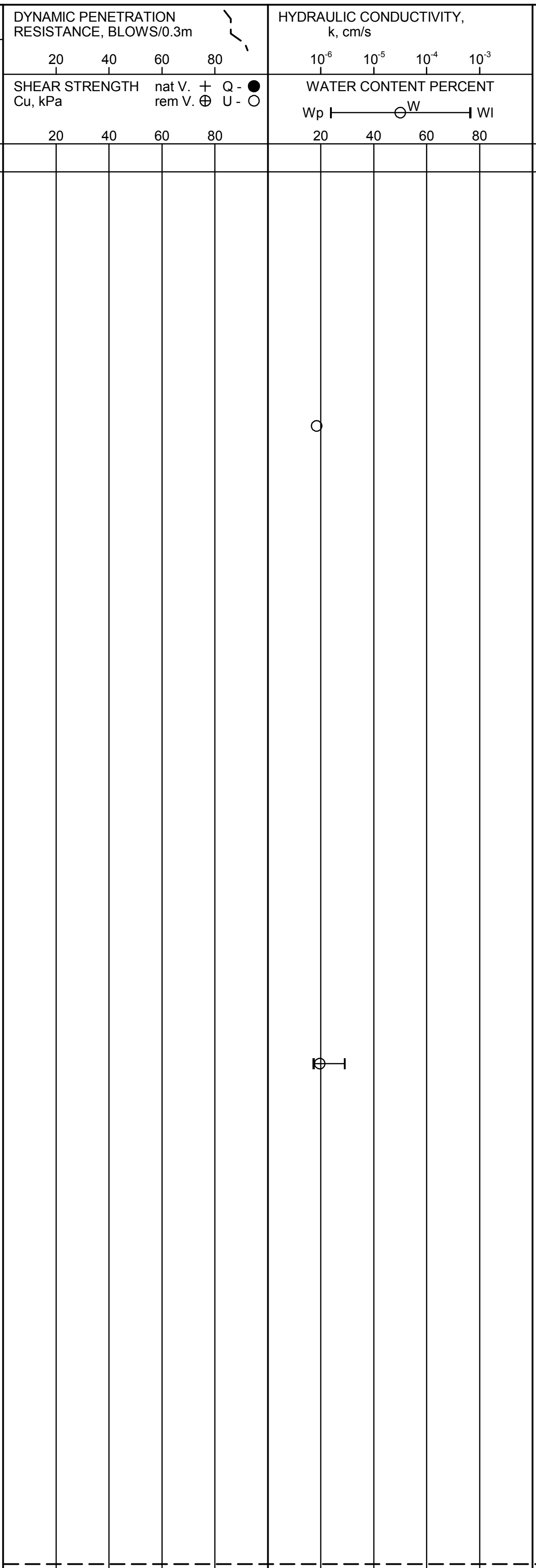
SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

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

SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)
GROUND SURFACE		93.16
FILL - (SM) SILTY CLAY and SILTY SAND; grey brown, contains brick fragments and organic matter (rootlets); non-cohesive, moist, loose	▨	0.00
FILL - (SM) gravelly SILTY SAND; grey and dark grey, contains clayey silt pockets; non-cohesive, very moist, loose to compact	▨	92.25 0.91
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
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
CONTINUED NEXT PAGE		

SAMPLES NUMBER	TYPE	BLOWS/0.30m	ADDITIONAL LAB. TESTING
1	SS	7	
2	SS	6	M
3	SS	11	
4	SS	7	
5	SS	5	
6	SS	8	
7	SS	7	



PIEZOMETER OR STANDPIPE INSTALLATION

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

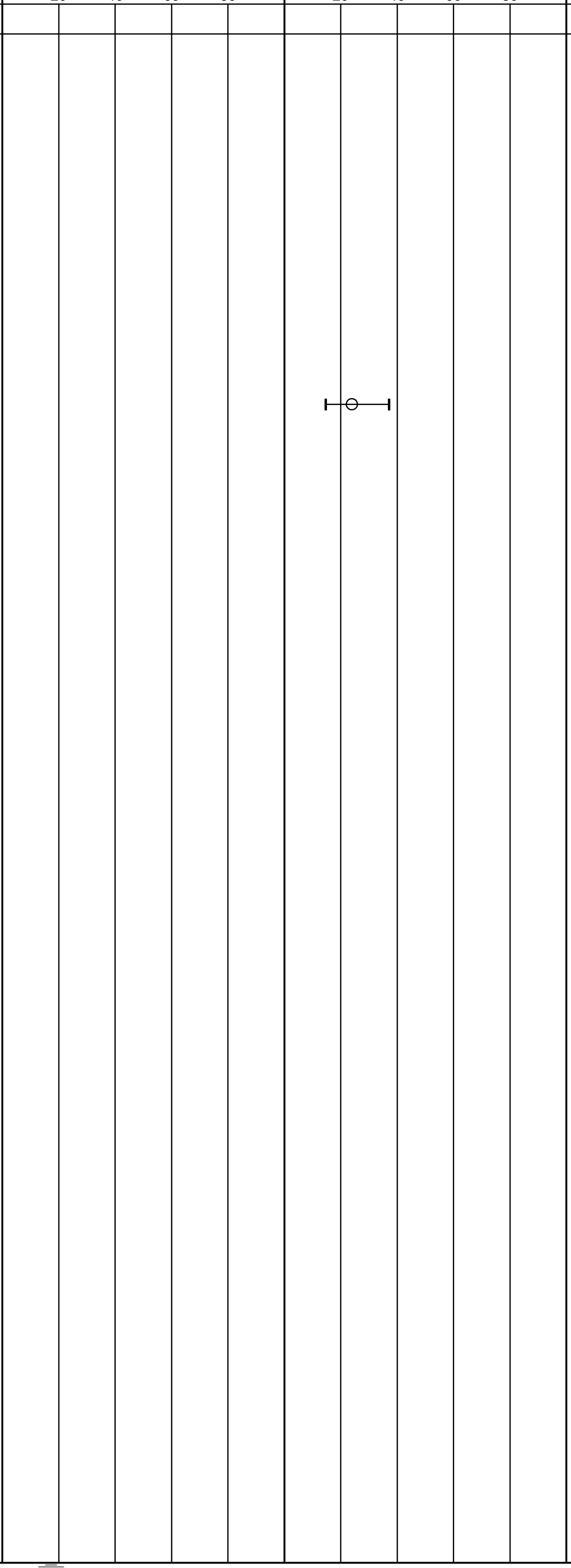
BORING DATE: December 14-15, 2017

DATUM: CGVD28

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

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



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: 17-206

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

DESCRIPTION	ELEV. / DEPTH (m)
GROUND SURFACE	92.26
FILL - (CI) sandy SILTY CLAY; brown, contains organic matter (topsoil); cohesive, w>PL, stiff	0.00
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
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

NUMBER	TYPE	BLOWS/0.30m	WATER CONTENT	ADDITIONAL LAB. TESTING
1	SS	3		
2	SS	24	○	M
3	SS	9		
4	SS	13		
5	SS	7		
6	SS	22		
7	SS	>50		
8	SS	10		

CONTINUED NEXT PAGE

DEPTH SCALE
1 : 50

Golder Associates

LOGGED: PAH
CHECKED:

PROJECT: 1670692

RECORD OF BOREHOLE: 17-206

SHEET 2 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	SAMPLES TYPE	SAMPLES BLOWS/0.30m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m 20 40 60 80 SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - ○ 20 40 60 80	HYDRAULIC CONDUCTIVITY, k, cm/s 10^{-6} 10^{-5} 10^{-4} 10^{-3} WATER CONTENT PERCENT Wp ⊢—○W—⊣ Wl 20 40 60 80	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		--- CONTINUED FROM PREVIOUS PAGE ---									
10	Power Auger 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				
15					13	SS	18				
					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		○ 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		*CONTINUED NEXT PAGE*									

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: PAH

CHECKED:

PROJECT: 1670692

RECORD OF BOREHOLE: 17-206

SHEET 3 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

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

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

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

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

ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	ELEV. DEPTH (m)	SAMPLES: NUMBER	TYPE	BLOWS/0.30m
		--- 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
			64.45 27.81			
28		End of Borehole				
29						
30						

DEPTH SCALE

1 : 50

LOGGED: PAH

CHECKED:

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)	SAMPLES NUMBER	SAMPLES TYPE	BLOWS/0.30m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m 20 40 60 80; SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - ○ 20 40 60 80	HYDRAULIC CONDUCTIVITY, k, cm/s 10^{-6} 10^{-5} 10^{-4} 10^{-3}; WATER CONTENT PERCENT Wp ⊢ ○W ⊣ Wl 20 40 60 80	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		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				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				
5	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		○	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				7	SS	39				
8				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				
9				10	SS	18				
10			84.67 9.91	11	SS	25				

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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: 17-207

SHEET 2 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

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

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

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

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

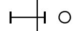
ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION

BORING METHOD: Wash Boring with Polymer Drill Mud; NW Casing

--- CONTINUED FROM PREVIOUS PAGE ---

DEPTH SCALE METRES	DESCRIPTION	ELEV. DEPTH (m)	NUMBER	TYPE	BLOWS/0.30m
10	(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
13			15	SS	20
	(SM) SILTY SAND; brown; non-cohesive, wet, compact to dense	80.86 13.72			
14			16	SS	31
15			17	SS	25
	(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

SHEET 3 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

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

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

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

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

ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	ELEV. DEPTH (m)	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: BLOWS/0.30m
		--- 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				
		(SP) SAND, fine, trace non-plastic fines; grey; non-cohesive, wet, very dense	73.85 20.73			
21						
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						

CONTINUED NEXT PAGE

DEPTH SCALE
1 : 50

Golder Associates

LOGGED: PAH
CHECKED:

PROJECT: 1670692

RECORD OF BOREHOLE: 17-207

SHEET 4 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	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: 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			▨		26	SS	185				
				62.88 31.70							
		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

SHEET 1 OF 3

LOCATION: N 5020451.2 ;E 445599.8

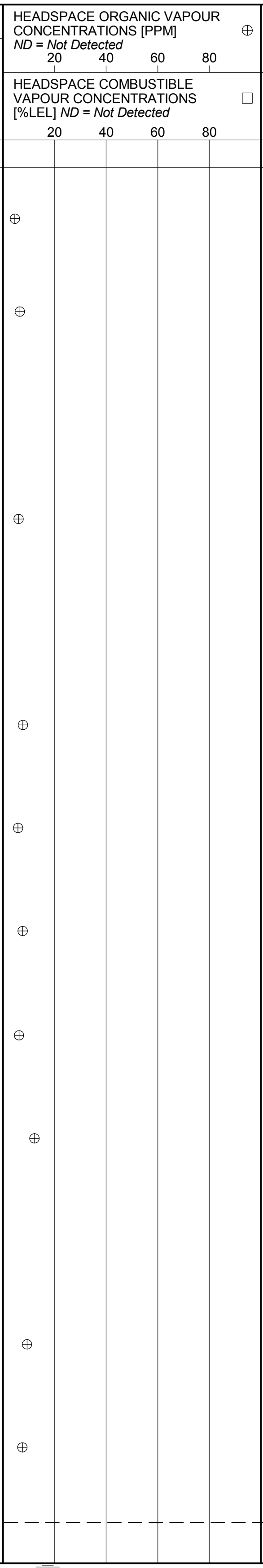
BORING DATE: May 2 & 3, 2017

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	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			
					7	SS	12		
5					8	SS	9		Native Backfill and Bentonite Mix
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		



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

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

CONTINUED NEXT PAGE

DEPTH SCALE

1 : 50

LOGGED: PAH

CHECKED:

PROJECT: 1670692

RECORD OF BOREHOLE: 17-01

SHEET 2 OF 3

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

--- CONTINUED FROM PREVIOUS PAGE ---

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

DESCRIPTION	ELEV. DEPTH (m)
FILL - (ML-CL) sandy SILT, some gravel and silty clay; grey brown and grey, trace asphaltic concrete and organics; very moist to wet, loose	
FILL - (ML) sandy SILT with gravel; dark grey and grey brown, contains silty clay layers, trace organics	81.10 13.72
CONCRETE RUBBLE	76.07 18.75
(ML) sandy SILT with gravel; dark grey and grey brown, contains silty clay layers, trace organics	75.47 19.35

NUMBER	TYPE	BLOWS/0.30m
14	SS	9
15	SS	5
16	SS	7
17	SS	20
18	SS	4
19	SS	15
20	SS	13
21	SS	9
22	SS	19
23	SS	21
24	SS	21
25	SS	49

Native Backfill and Bentonite Mix

Bentonite Seal

Silica Sand

51 mm Diam. PVC #10 Slot Screen

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

DEPTH SCALE
1 : 50

LOGGED: PAH
CHECKED:

PROJECT: 1670692

RECORD OF BOREHOLE: 17-01

SHEET 3 OF 3

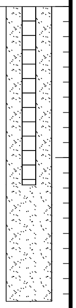
LOCATION: N 5020451.2 ;E 445599.8

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			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
		DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	NUMBER	TYPE	BLOWS/0.30m				
		--- *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
				72.87							
22		End of Borehole		21.95							W.L. in Screen at Elev. 78.41 m on May 4, 2017
23											
24											
25											
26											
27											
28											
29											
30											

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)

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})

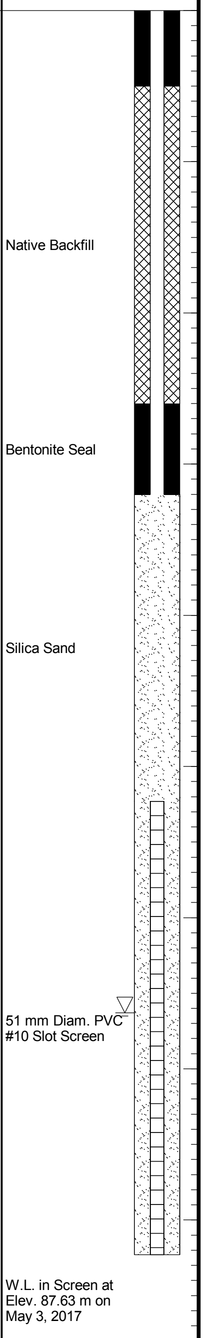
WATER CONTENT PERCENT Wp ⊢—○W—⊣ Wl (20, 40, 60, 80)

ADDITIONAL LAB. TESTING

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

NUMBER	TYPE	BLOWS/0.30m	HEADSPACE ORGANIC VAPOUR [PPM]
1	GRAB	-	155.7
2	SS	7	177
3	SS	15	136
4	SS	23	141.8
5	SS	91	
6	SS	19	
7	SS	16	
8	SS	24	
9	SS	20	
10	SS	16	
11	SS	15	

PIEZOMETER OR STANDPIPE INSTALLATION



W.L. in Screen at Elev. 87.63 m on May 3, 2017

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: PAH

CHECKED:

PROJECT: 1670692

RECORD OF BOREHOLE: 17-07

SHEET 1 OF 1

LOCATION: N 5020617.3 ;E 445597.4

BORING DATE: April 26, 2017

DATUM: CGVD28

SAMPLER HAMMER, 64kg; DROP, 760mm

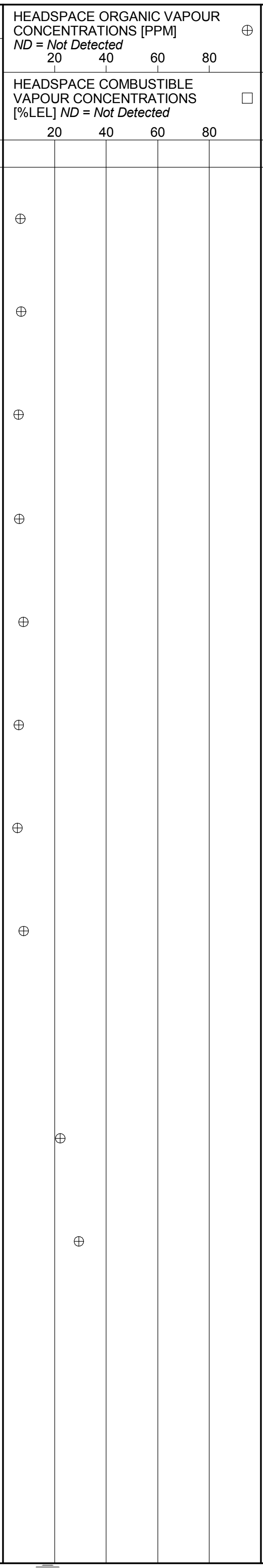
PENETRATION TEST HAMMER, 64kg; DROP, 760mm

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

DEPTH SCALE METRES: 0 to 10

ELEV. DEPTH (m)	SOIL PROFILE DESCRIPTION	STRATA PLOT
93.80	GROUND SURFACE	
0.00	FILL - (CL-ML) SILTY CLAY and sandy SILT, some gravel; grey brown, contains cobbles; non-cohesive, very moist, loose to compact	▨
91.67 2.13	FILL - (SM) SILTY SAND, fine; grey with clayey silt lumps; non-cohesive, very moist, compact	▨
90.14 3.66	FILL - (CL/ML) SILTY CLAY and CLAYEY SILT, trace gravel; grey; cohesive, w~PL, loose	▨
87.40 6.40	(SP) SAND, fine; brown with occasional thin clayey silt seams; non-cohesive, wet, loose to compact	
85.57 8.23	End of Borehole	

SAMPLES NUMBER	TYPE	BLOWS/0.30m
1	GRAB	-
2	SS	11
3	SS	5
4	SS	16
5	SS	12
6	SS	4
7	SS	4
8	SS	3
9	SS	6
10	SS	6
11	SS	11

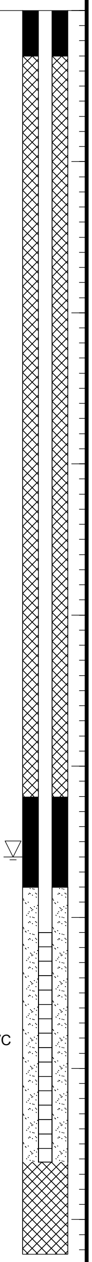


HYDRAULIC CONDUCTIVITY, k, cm/s: 10^{-6} 10^{-5} 10^{-4} 10^{-3}

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

ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION



W.L. in Screen at Elev. 88.20 m on May 2, 2017

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

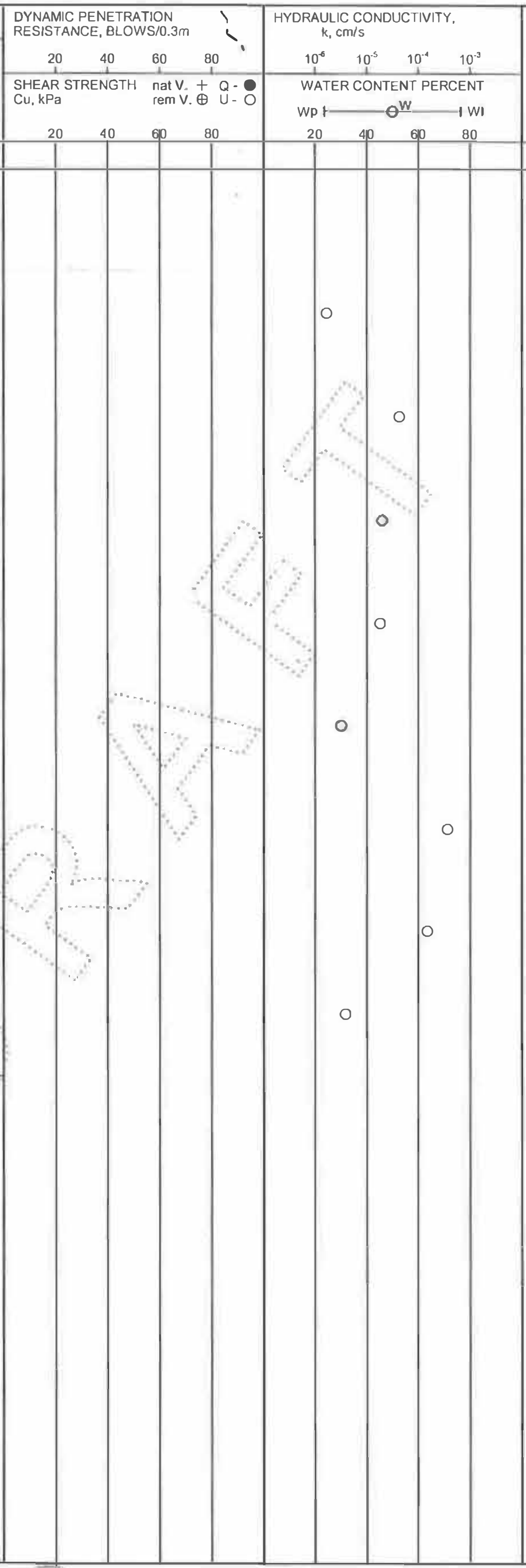
SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)
		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
6.40		Compact grey fine SAND, trace silt	⁘	85.63 6.40
6.71		End of Borehole		85.32 6.71

SAMPLES

NUMBER	TYPE	BLOWS/0.3m
1	50 DO	4
2	50 DO	1
3	50 DO	2
4	50 DO	2
5	50 DO	5
6	50 DO	1
7	50 DO	2
8	50 DO	24



ADDITIONAL LAB TESTING

PIEZOMETER OR STANDPIPE INSTALLATION

DEPTH SCALE 1 : 50

Golder Associates

LOGGED: D.G.

CHECKED:

PROJECT: 11-1121-0050

RECORD OF BOREHOLE: 11-02

SHEET 1 OF 1

LOCATION: See Site Plan

BORING DATE: Apr. 11, 2011

DATUM: Geodetic

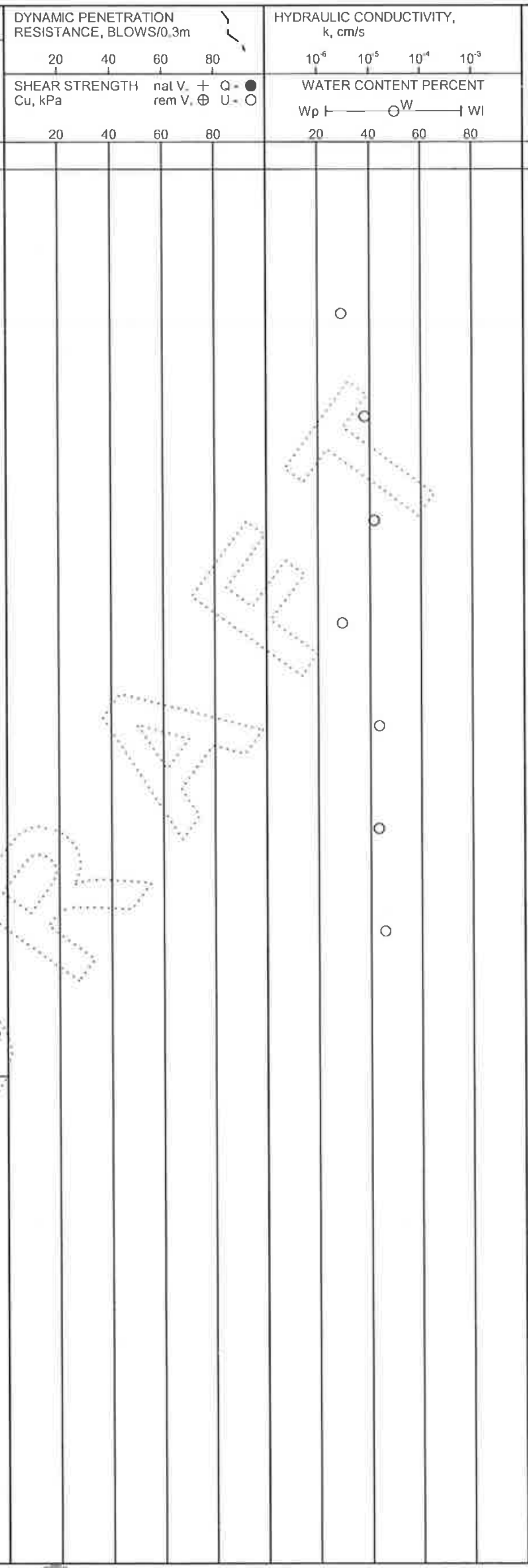
SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

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

SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV.	DEPTH (m)
GROUND SURFACE		92.84	0.00
Grey brown silty clay, some brown silty sand and fine sand, trace organic matter (FILL)	▩		
Compact grey fine SAND, trace silt	⁘	86.74	6.10
End of Borehole		86.13	6.71

SAMPLES NUMBER	TYPE	BLOWS/0.3m
1	50 DO	4
2	50 DO	2
3	50 DO	5
4	50 DO	4
5	50 DO	1
6	50 DO	1
7	50 DO	2
8	50 DO	22



ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION

∇

W.L. in open borehole at 6.10m depth below ground surface upon completion of drilling

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

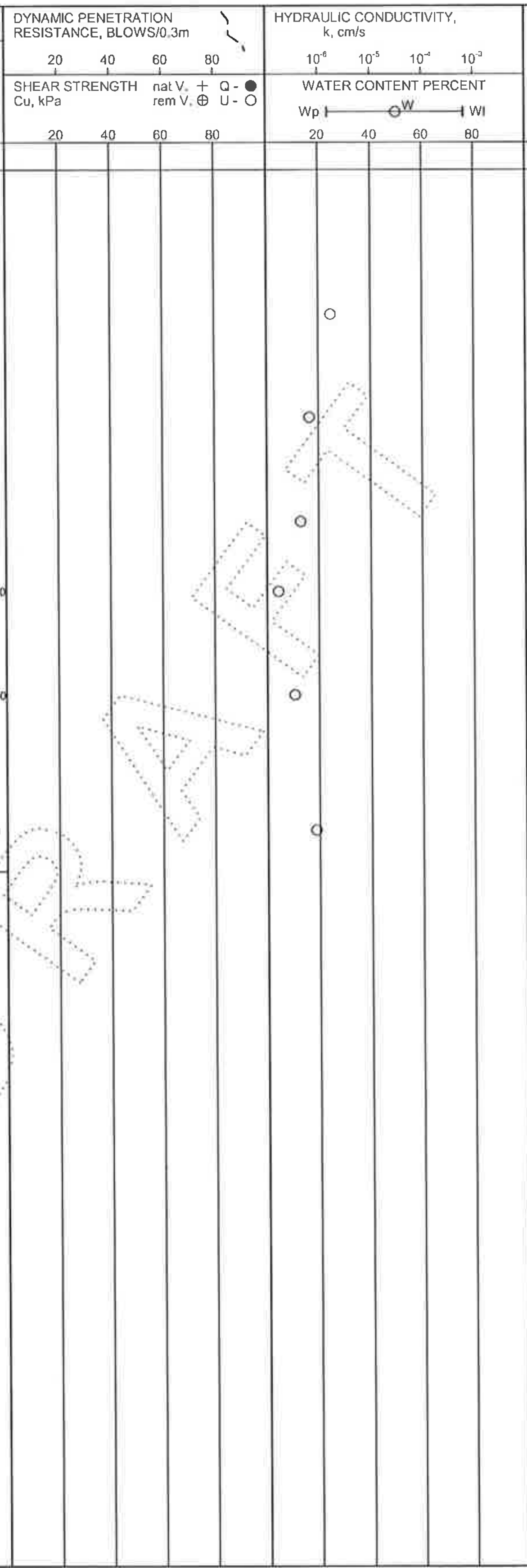
BORING DATE: Apr. 11, 2011

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	SAMPLES: TYPE	SAMPLES: BLOWS/0.3m
		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					2	50 DO	5
					3	50 DO	12
3					4	50 DO	>100
4					5	50 DO	>100
5					6	50 DO	20
				89.09 5.18			
		End of Borehole					
6							
7							
8							
9							
10							



ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION

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

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

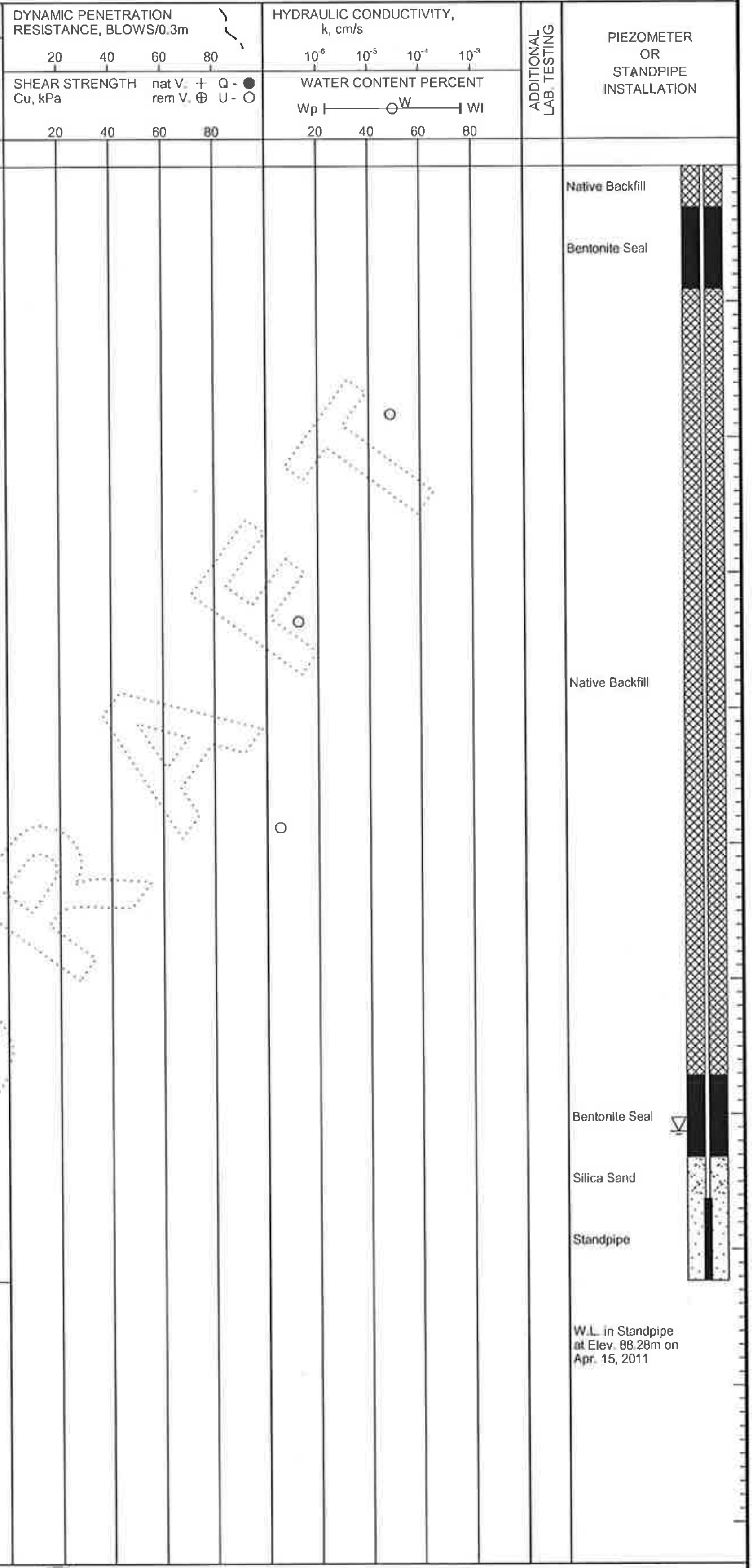
BORING DATE: Apr. 11, 2011

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. / DEPTH (m)	SAMPLES: NUMBER	TYPE	BLOWS/0.3m
		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			
2					1	50 DO	2
3					2	50 DO	18
5					3	50 DO	12
6		Loose light brown fine SAND, trace silt	░	89.31 6.10	4	50 DO	8
8					5	50 DO	8
		End of Borehole		87.18 8.23			



W.L. in Standpipe at Elev. 88.28m on Apr. 15, 2011

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	Grey brown sandy silt and silty clay, some gravel and shale fragments (FILL)
	4.5 – 4.8	Grey brown SILTY CLAY (Weathered Crust)
	4.8	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	Grey brown sandy silt and silty clay, with numerous rock slabs, wood, roots, organic matter, and brick fragments (FILL)
	5.0 – 5.2	Grey SILTY SAND, with gravel
	5.2	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	Grey brown sandy silt, trace clay, with cobbles, wood, plastic, and brick fragments (FILL)
	5.0 – 5.3	Grey SILTY SAND, with gravel
	5.3	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 2 3	1.0 2.0 3.0	0 0 0	Minor seepage observed at 3.5m
	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 2 3	1.0 2.0 3.0	0 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 2 3	1.0 2.0 3.0	0 0 0	debris
	5.0-5.2	Wet,grey,sandy Silt with trace clay and native organics. No odour or sheen.	4 5	4.0 5.0	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 2 3	1.0 2.0 3.0	0 0 0	debris
	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 Summary

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 - ○ 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
		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							Native Backfill
					3	50 DO	17				∇
		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

BOREHOLE 011-2835.GPJ GLDR_CAN.GDT 9/5/01 Ken Taylor

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

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

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

DESCRIPTION	ELEV. / DEPTH (m)	NUMBER	TYPE	BLOWS/0.3m	PIEZOMETER OR STANDPIPE INSTALLATION
GROUND SURFACE	90.69				
Grey brown silty clay, some gravel and concrete (FILL)	0.00				Bentonite Seal w/ Steal Casing
		1	50 DO	8	
		2	50 DO	3	Native Backfill
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
Compact brown fine to coarse SAND, trace gravel	84.44 6.25	4	50 DO	27	
		5	50 DO	29	Native Backfill
Dense brown fine to medium SAND	81.45 9.24	6	50 DO	33	
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				
Brown fine SAND	78.04 12.65	8	50 DO	44	∇ 50mm PVC Slot Screen
END OF BOREHOLE	75.75 14.94				W.L in screen at Elev. 78.11 m April 30, 2001

BOREHOLE 011-2835.GPJ GLDR_CAN.GDT 9/5/01 Ken Taylor

DEPTH SCALE
1 : 100

Golder Associates

LOGGED: D.J.S.

CHECKED: DHP

PROJECT: 011-2898

RECORD OF BOREHOLE: 01-3

SHEET 1 OF 1

LOCATION: 3930 Riverside Drive

BORING DATE: August 8, 2001

DATUM: Local

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

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

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

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

WATER CONTENT PERCENT: Wp ⊢—○W—⊣ Wl: 10 20 30 40

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES BLOWS/0.3m	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		Ground Surface	92.04					
0	POWER AUGER 200 mm Diam. (Hollow Stem)	Grey silty clay, trace gravel (FILL)	0.00					Bentonite Seal
2		Brown fine SAND	90.21 1.83	1	50 DO	4		Native Backfill
				2	50 DO	5		Bentonite Seal Sand Backfill
4		Grey brown SILTY SAND, occasional thin silty clay seam	87.77 4.27					
		Brown fine SAND, with occasional 0.3 to 0.6 m silty sand layer	87.24 4.80	3	50 DO	25		∇ 50 mm PVC #10 Slot Screen
6				4	50 DO	15		
8				5	50 DO	8		Caved Material
		END OF BOREHOLE	83.81 8.23					W.L. in Screen at Elev. 87.18m Aug. 23, 2001
10								
12								
14								
16								
18								
20								

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

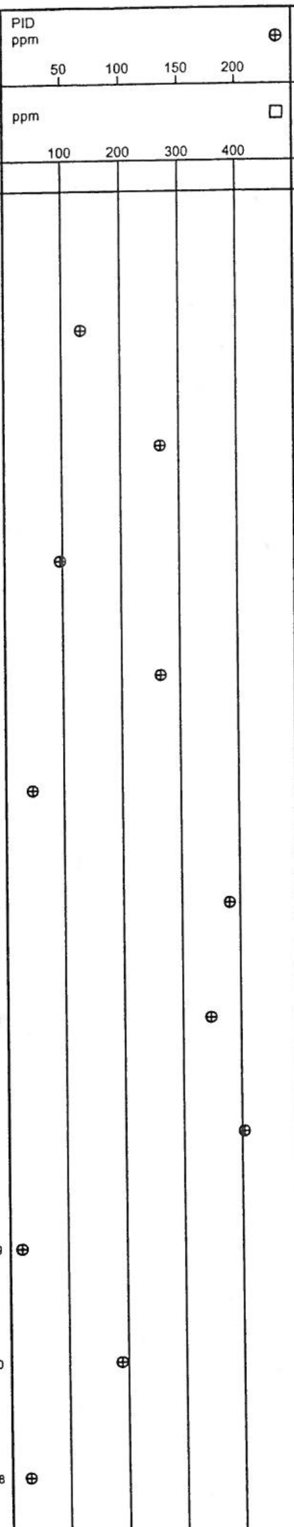
SHEET 1 OF 1

DATUM: Local

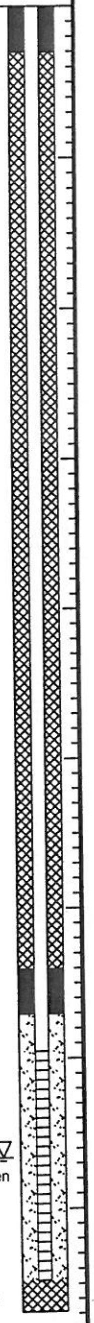
PENETRATION TEST HAMMER, 64kg; DROP, 760mm

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

DESCRIPTION	ELEV. / DEPTH (m)	NUMBER	TYPE	BLOWS/0.3m
GROUND SURFACE	93.64			
Brown silty clay, trace sand and gravel (FILL)	0.00			
Dark brown sandy silt, some gravel, trace clay, cinders, ashes and wood (FILL)	93.03 0.61			
		1	50 DO	10
		2	50 DO	31
Dark grey silty clay, trace topsoil and brick (FILL)	90.07 3.57			
		3	50 DO	4
Brown sandy silt, some gravel, trace asphalt and silty sand (FILL)	88.31 5.33			
		4	50 DO	7
		5	50 DO	7
Probably mainly concrete rubble (FILL)	85.11 8.53			
Brown sandy silt, some gravel and asphalt (FILL)	84.34 9.30	6	50 DO	13
		7	50 DO	16
Dark brown sandy silt, some gravel, trace asphalt and ashes (FILL)	82.06 11.58			
		8	50 DO	8
Concrete rubble (FILL)	80.23 13.41	9	50 DO	19
Brown sandy silt, some gravel, occasional cobble (FILL)	79.47 14.17			
Brown fine to medium SAND, occasional thin sandy silt seam	78.70 14.94	10	50 DO	10
Brown fine to coarse SAND	78.10 15.54			
Brown fine to coarse SAND and GRAVEL, occasional cobble	15.85			
		11	50 DO	28
END OF BOREHOLE	76.27 17.37			



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: JR

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

DEPTH SCALE METRES: 0 – 20

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

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

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

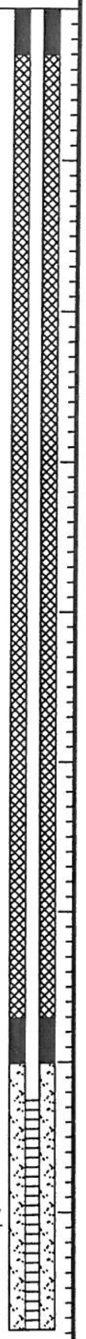
HYDRAULIC CONDUCTIVITY, k, cm/s: 10^{-6} 10^{-5} 10^{-4} 10^{-3}

WATER CONTENT PERCENT: Wp ⊢—○W—⊣ Wl 10 20 30 40

ADDITIONAL LAB. TESTING

SOIL PROFILE DESCRIPTION	ELEV. / DEPTH (m)	SAMPLES 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 5	50 DO 50 DO	4 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 11	50 DO 50 DO	>100 50/ 50mm
END OF BOREHOLE	75.41 17.56			

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

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

DEPTH SCALE
1 : 100

Golder Associates

LOGGED: D.J.S.
CHECKED: TK

PROJECT: 941-2735

RECORD OF TEST PIT 94-8

SHEET 1 OF 1

LOCATION: See Plan

DATE: June 3, 1994

DATUM: Geodetic

DEPTH SCALE METRES	METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES USCS	HYDRAULIC CONDUCTIVITY, k, cm/s ⊕ COMBUSTIBLE VAPOUR (ppm) ☐	WATER CONTENT, PERCENT Wp ├──○Wn──┤ Wl	ADDITIONAL LAB. TESTING	GROUNDWATER CONDITIONS
		Ground Surface		92.32							
0	Backhoe	Grey brown silty sand, some gravel (FILL)	▨	0.00 92.12 0.20	1	CS	-				
		Dark brown silty sand, some gravel, trace organics, brick (FILL)	▨	0.38							
1			▨		2	CS	-				
		Grey silty sand, some gravel and clay, occasional cobble and boulder, trace brick (FILL)	▨		3	CS	-				
2			▨		4	CS	-				
			▨		5	CS	-				
3		Grey sand, trace silt, gravel, metal tank, wood, occasional cobble and boulder (FILL)	▨	89.42 2.90	6	CS	-				
4			▨		7	CS	-				
		Medium brown SAND	⁘	88.12 4.20	8	CS	-				
5			⁘	87.32 5.00	9	CS	-				
		End of Test Pit									Test Pit wet at 4.2m on completion of excavation
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:

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	-				
				88.31	5	CS	-				
3		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

RECORD OF TEST PIT 94-17

SHEET 1 OF 1

LOCATION: See Plan

DATE: June 21, 1994

DATUM: Geodetic

DEPTH SCALE METRES	METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES USCS	HYDRAULIC CONDUCTIVITY, k, cm/s ⊕ / COMBUSTIBLE VAPOUR (ppm) ☐	WATER CONTENT, PERCENT Wp ⊢—○Wn—⊣ Wl 20 40 60 80	ADDITIONAL LAB. TESTING	GROUNDWATER CONDITIONS
		Ground Surface		91.80							
0	Backhoe	Grey brown sandy silt, some gravel and brick, occasional silty clay pocket (FILL)	▨	0.00	1	CS					
1		Brown silty sand, trace organic material, occasional silty clay pocket (FILL)	▨	91.10 0.70							
2											
3											
		Grey brown silty clay, some gravel and sand, trace brick (FILL)	▨	88.10 3.70	2	CS			○	MH	
4		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
6											
7											
8											
9											
10											

DATA INPUT: Disk 18, S.Leighton

DEPTH SCALE
1 to 50

Golder Associates

LOGGED: A.F.C
CHECKED:

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

RECORD OF BOREHOLE 91-1

BORING DATE: June 3&4, 1991

SHEET 1 OF 2
DATUM:
PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

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

DEPTH SCALE METRES	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)
0	Ground Surface	89.88 0.00
0–3	Loose brown fine to medium SAND, trace silt, occasional sandy silt seam at depth	86.83 3.05
3–7.6	Loose to compact grey brown fine to medium SAND, trace to some silt, some silty clay layers at 6.7 metres depth	82.26 7.62
7.6–12.2	Stiff to very stiff grey SILTY CLAY, some sand seams and layers	77.68 12.20
12.2–12.8	Loose grey brown fine to coarse SAND, trace gravel and silt	77.08 12.80
12.8–15	Possibly Sand	
15	CONTINUED ON NEXT PAGE	

SAMPLES NUMBER	TYPE	BLOWS/0.3m
1	50 DO	4
2	50 DO	5
3	50 DO	9
4	50 DO	15
5	50 DO	11
6	50 DO	8
7	50 DO	13
8	50 DO	15
9	50 DO	12
10	50 DO	3
11	50 DO	1
12	50 DO	4
13	50 DO	17
14	50 DO	10
15	50 DO	7

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

PIEZOMETER OR STANDPIPE INSTALLATION

Native Backfill

Bentonite Seal

Native Backfill

▽

Standpipe

W.L in Standpipe at Elev.84.75 June 25, 1991

NOTE : Standpipe installed in an adjacent borehole

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

DATA INPUT: Disk 8, Stever

DEPTH SCALE
1 to 75

Golder Associates

LOGGED: S.Leighton
CHECKED:

PROJECT: 911-2151

RECORD OF BOREHOLE 91-1

SHEET 2 OF 2

LOCATION: See Figure 2

BORING DATE: June 3&4, 1991

DATUM:

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m SHEAR STRENGTH Cu, kPa nat.V - + Q - ● rem.V - ⊕ U - O 20 40 60 80	HYDRAULIC CONDUCTIVITY, k, cm/s WATER CONTENT, PERCENT Wp ⊢—O W—⊣ Wl 20 40 60 80	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
15		CONTINUED FROM PREVIOUS PAGE								
16										
17										
18										
19										
20		Possibly Sand								
21										
22	Power Auger 200mm Diam (Hollow Stem)									
23										
24										
25										
26			63.67 26.21							
27										
28		Possibly Sand and Gravel and Sand								
29		End of Hole Auger Refusal	60.77 29.11							
30										

0
15 ◆ 5 PERCENT AXIAL STRAIN AT FAILURE
10

DATA INPUT: Disk 8, Stever

DEPTH SCALE
1 to 75

Golder Associates

LOGGED: S.Leighton

CHECKED:

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

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

SOIL PROFILE

DESCRIPTION	ELEV. / DEPTH (m)
Ground Surface	88.05 0.00
Very loose to loose brown to grey brown, fine to medium and fine to coarse sand, trace silt (Fill)	86.22 1.83
Loose grey SAND, trace silt, trace organic matter and roots	85.92 2.13
Loose to compact brown to grey brown fine to coarse SAND, trace silt, occasional clayey silt seam	84.39 3.66
Compact grey layered SAND, some silt and gravel and CLAYEY SILT	83.18 4.87
Dense to very dense grey brown SAND, some gravel, trace to some silt	81.34 6.71
Dense grey brown fine to medium SAND, trace silt	78.30 9.75
End of Hole	

SAMPLES

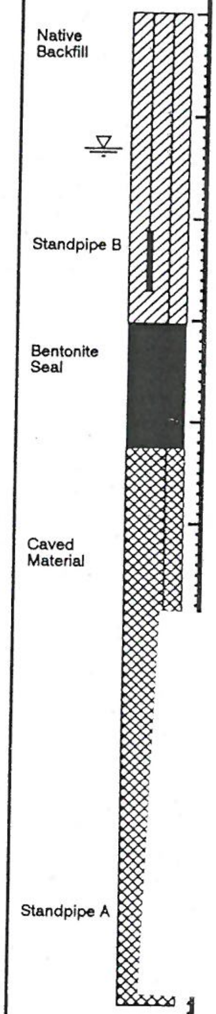
NUMBER	TYPE	BLOWS/0.3m
1	50 DO	2
2	50 DO	8
3	50 DO	13
4	50 DO	8
5	50 DO	11
6	50 DO	30
7	50 DO	84
8	50 DO	50
9	50 DO	49
10	50 DO	39

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m
SHEAR STRENGTH Cu, kPa 20 40 60 80 nat.V - + Q - ● rem.V - ⊕ U - O

HYDRAULIC CONDUCTIVITY, k, cm/s
WATER CONTENT, PERCENT Wp W Wl 20 40 60 80

ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION



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

DATA INPUT: Disk 8, Stever

DEPTH SCALE
1 to 75

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			SAMPLES			DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m	HYDRAULIC CONDUCTIVITY, k, cm/s	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	NUMBER	TYPE	BLOWS/0.3m	SHEAR STRENGTH Cu, kPa nat.V - + Q - ● rem.V - ⊕ U - ○ 20 40 60 80	WATER CONTENT, PERCENT Wp ⊢—○W—⊣ Wl 20 40 60 80		
0		Ground Surface		92.40 0.00							
1	Power Auger 200mm Diam (Hollow Stem)	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)	▩		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
8											
9											
10											
11											Tow
12											
13											
14											
15											

DATA INPUT: Disk 8, Stever

0
15-◆-5 PERCENT AXIAL STRAIN AT FAILURE
10

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	SOIL PROFILE ELEV'N. DEPTH (m)	SOIL PROFILE DESCRIPTION	STRATA PLOT	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES BLOWS/0.3 m	ELEVATION SCALE	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m; SHEAR STRENGTH Cu, kPa NAT. V. - + Q. - ● REM. V. - ⊕ U. - ○	HYDRAULIC CONDUCTIVITY, k, cm/sec. 1x10 1x10 1x10 1x10; WATER CONTENT, PERCENT Wp W WL	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
	90.88	GROUND SURFACE					91				
CME 55 POWER AUGER BORING 200 mm DIA. HOLLOW STEM AUGER	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				BOREHOLE D NOVEMBER 1984
				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				

15 0 5 10 Percent axial strain at failure

VERTICAL SCALE
1:50 (METRIC)

Golder Associates

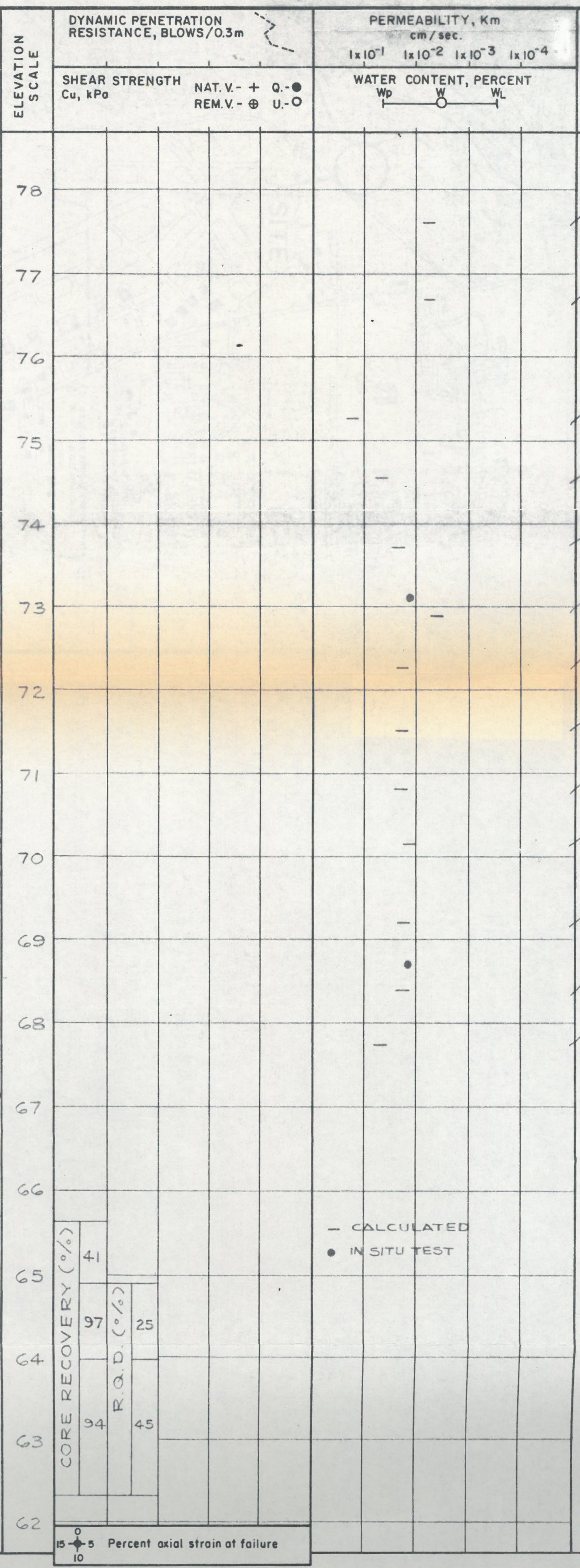
DRAWN RWR

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	DESCRIPTION	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES BLOWS/0.3 m	CORE RECOVERY (%)	R.Q.D. (%)	ADDITIONAL LAB. TESTING
ROTARY DRILLING BW CASING	77.92	**GROUND SURFACE**						
	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.	—	41		
BXL CORE	64.86 13.06	FAIRLY SOUND **LIMESTONE BEDROCK** OCCASIONAL SHALEY INTERBED, FRACTURED SEAM AT 15 m DEPTH	22		—	97	25	
			23		—	94	45	
	62.31 15.61	**END OF HOLE**						



PIEZOMET OR STANDPIP INSTALLAT

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

NOTE: ART
W.L. FROM
BEDROCK
ABOVE GR
SURFACE
(ELEV. 81.
NOV. 10, 19

VERTICAL SCALE

Golder Associates

DRAWN J.C.

CHECKED

P[redacted]t No. [redacted]1-2[redacted]

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

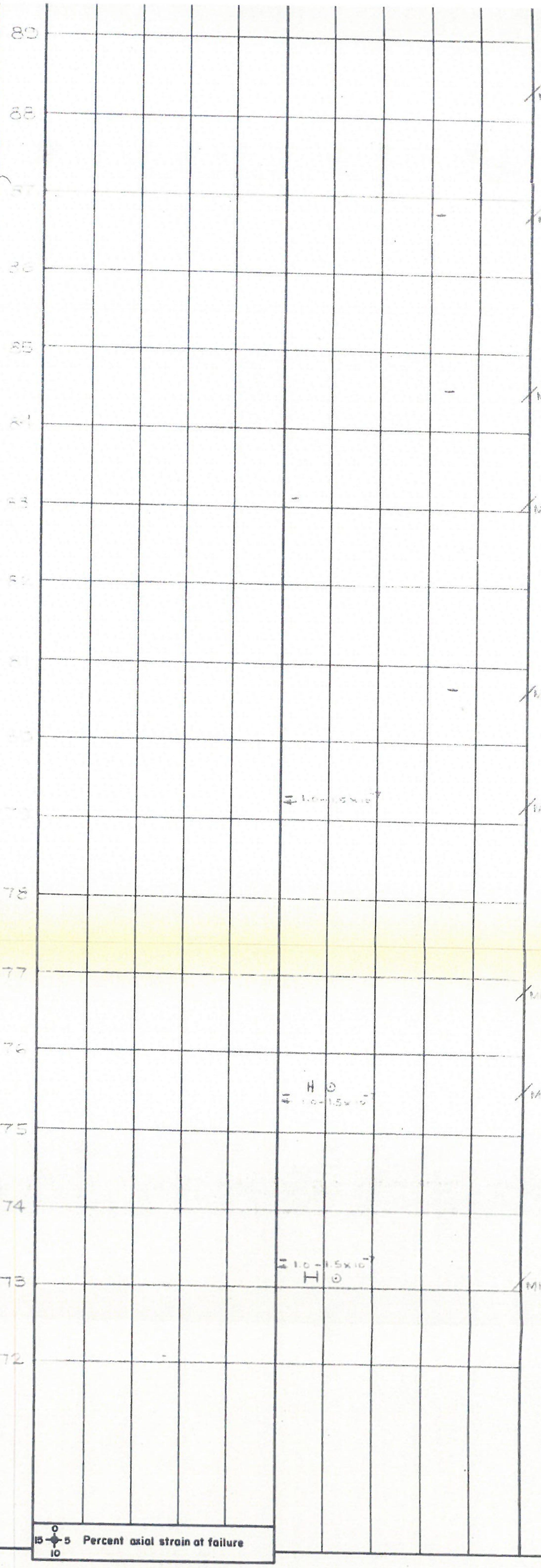
BORING METHOD	SOIL PROFILE			SAMPLES			ELEVATION SCALE	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m SHEAR STRENGTH Cu, kPa NAT. V. - + Q. - ● REM. V. - ⊕ U. - O	HYDRAULIC CONDUCTIVITY, k, cm/sec. 1x10 1x10 1x10 1x10 WATER CONTENT, PERCENT Wp W WL 20 40 60 80	ADDITIONAL LAB. TESTING	PIEZOMETE OR STANDPIPE INSTALLATI
	ELEV'N. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m					
							100				
	99.39	GROUND SURFACE									GROUND SURFACE
POWER AUGER 200 mm DIAM. (HOLLOW STEM)	0.00	VERY LOOSE TO LOOSE BROWN **SAND AND GRAVEL**, TRACE SILT (STOCKPILE)									SURFACE SEAL
							99				
				1	50 mm D.O.	7					
							98				
				2	"	3				M	
							97				
				3	"	4					
				4	"	3	96			M	
				5	"	5					
							95				
				6	"	10					
							94				NATIVE BACKFILL
				7	"	8					
				8	"	11	93			M	
	92.38 7.01	VERY DENSE BROWN **SAND AND GRAVEL**, SOME SILT, OCCASIONAL COBBLES		9	"	>100					
							92				
				10	"	>100				MH	
	91.16 8.23	DENSE TO VERY DENSE BROWN MEDIUM TO COARSE **SAND**, TRACE GRAVEL AND SILT					91				
				11	"	96					
				12	"	33	90				
	89.58 9.81	COMPACT TO DENSE BROWN FINE TO MEDIUM **SAND**, TRACE SILT								M	
				13	"	36	89				
				14	"	28				MH	
							88				
				15	"	41					
							87				
				16	"	21				MH	PLASTIC TUBING
				17	"	27	86				
	85.83 13.56	COMPACT TO DENSE BROWN MEDIUM **SAND**, TRACE SILT									
				18	"	6					
							85				
				19	"	40				M	
	84.00 15.39	VERY DENSE BROWN FINE **SAND**, SOME SILT AND SILTY FINE SAND, OCCASIONAL THIN					84				
				20	"	47					
				21	"	62	83			MH	STANDPIPE

ROTARY DRILLING

BW CASING WITH DRILL MUD

	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] 17.63	VERY DENSE BROWN MEDIUM **SAND**, TRACE SILT
[illegible] 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

13	"	36
14	"	33
15	"	41
16	"	21
17	"	27
18	"	6
19	"	40
20	"	47
21	"	62
22	"	70
23	"	68
24	"	[illegible]
25	"	51
26	"	52
27	"	62
28	"	44
29	"	36
30	"	37
31	"	16
32	"	8
33	"	24
34	"	10



PLASTIC TUBING —

STANDPIPE —

W.L. IN STANDP
AT ELEV 87.
APRIL 5,19

VERTICAL SCALE
1:50

Golder Associates

DRAWN J.C.
CHECKED

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	SOIL PROFILE			SAMPLES		
	ELEV'N. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3 m
	83.52	GROUND SURFACE				
POWER AUGER 200 mm DIAM. (HOLLOW STEM)	0.00	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
				6	"	37
				7	"	44
				8	"	60
				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
				13	"	64
	73.92 9.60	VERY DENSE BROWN MEDIUM TO COARSE **SAND**, TRACE SILT		14	"	83
	72.94 10.58	VERY DENSE BROWN **SAND AND GRAVEL**, SOME COBBLES		15	"	>100
				16	BX RC	—
				17	50 mm D.O.	>100
				18	BX RC	—
				19	50 mm D.O.	96
				20	BX RC	—
				21	50 mm D.O.	55
				22	BX RC	—
				23	50 mm D.O.	>100
RY DRILLING ING WITH DRILLING MUD				24	BX RC	—
				25	50 mm D.O.	80
				26	BX RC	—
				27	50 mm D.O.	51
				28	BX RC	—
				29	50 mm D.O.	44

ELEVATION SCALE

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m

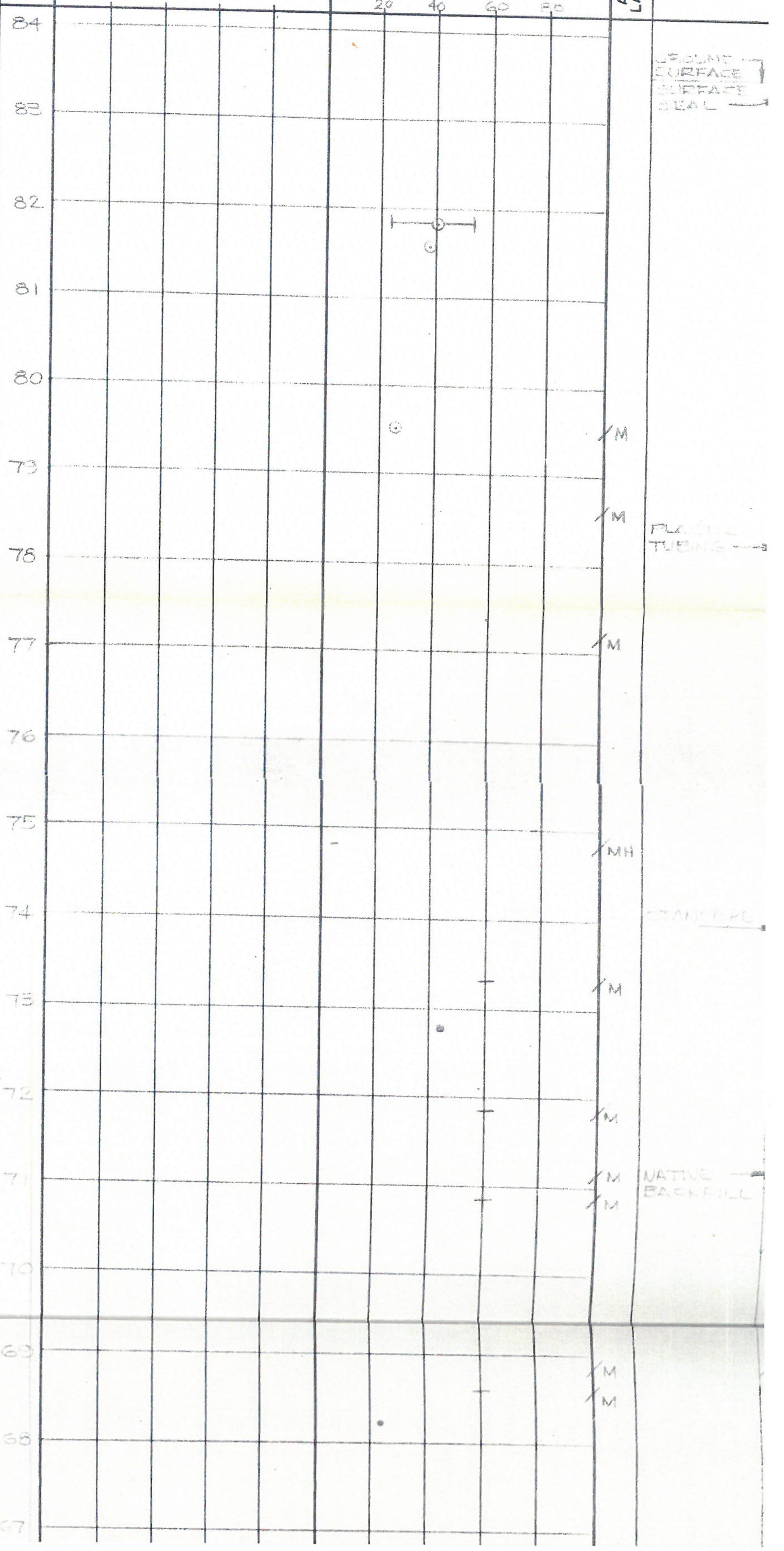
SHEAR STRENGTH Cu, kPa NAT. V. - + Q. - ● REM. V. - ⊕ U. - O

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

WATER CONTENT, PERCENT Wp W WL

ADDITIONAL LAB. TESTING

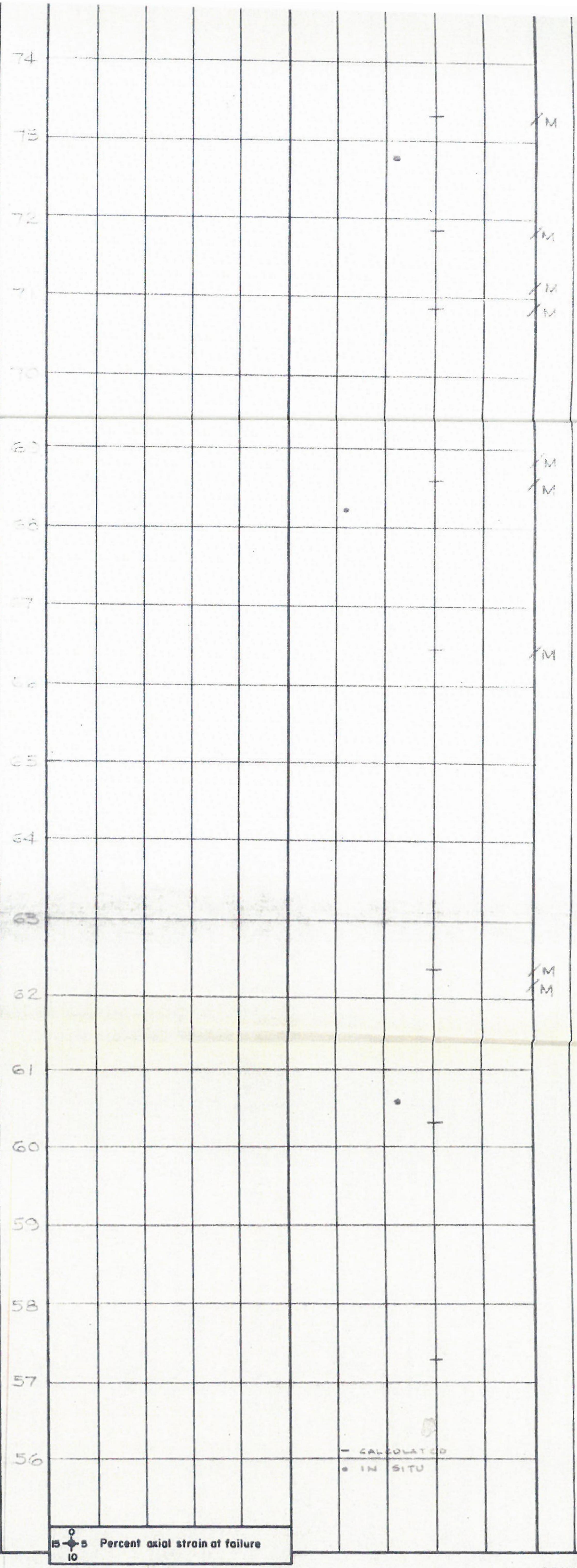
PIEZOMETER OR STANDPIPE INSTALLATIO



ROTARY DRILLING

BW CASING WITH DRILLING MUD

73.92 9.60	VERY DENSE BROWN MEDIUM TO COARSE **SAND**, TRACE SILT	13	"	64
		14	"	83
72.94 10.58	VERY DENSE BROWN **SAND AND GRAVEL**, SOME COBBLES	15	"	>100
		16	BX RC	—
		17	50 mm D.O.	>100
		18	BX RC	—
		19	50 mm D.O.	96
		20	BX RC	—
		21	50 mm D.O.	55
		22	BX RC	—
		23	50 mm D.O.	>100
		24	BX RC	—
		25	50 mm D.O.	80
		26	BX RC	—
		27	50 mm D.O.	51
		28	BX RC	—
		29	50 mm D.O.	44
		30	"	48
		31	"	48
		32	BX RC	—
		33	50 mm D.O.	62
		34	BX RC	—
		35	50 mm D.O.	>100
		36	BX RC	—
		37	"	—
		38	"	—
		39	50 mm D.O.	84
		40	BX RC	
		41	50 mm D.O.	>100
		42	BX RC	—
		43	50 mm D.O.	78
59.78 23.74	VERY DENSE BROWN **SAND AND GRAVEL**, SOME COBBLES AND BOULDERS, TRACE SILT	44	BXL RC	—
58.07 25.45	VERY DENSE GREY **FINE TO COARSE SAND**, TRACE GRAVEL AND SILT	45	BXL RC	—
57.46 26.06	VERY DENSE GREY SILTY SAND TO SANDY SILT, SOME GRAVEL, TRACE CLAY, OCCASIONAL COBBLES AND BOULDERS **(GLACIAL TILL)**	46	50 mm D.O.	66
		47	BXL RC	—
56.51		48	50 mm D.O.	>100
27.01	END OF HOLE			



STANDPIPE

NATIVE BACKFILL

WL IN STA
AT ELEV
APRIL 5

VERTICAL SCALE
1:50

Golder Associates

DRAWN J.C.
CHECKED

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	SOIL PROFILE ELEV'N. DEPTH	DESCRIPTION	STRATA PLOT	SAMPLES NUMBER	TYPE	BLOWS/0.3 m	ELEVATION SCALE	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
							84		
	83.32	GROUND SURFACE							GROUND SURFACE
POWER AUGER 200 mm DIAM. (HOLLOW STEM)	0.00 82.04	FROZEN BROWN FINE SAND, TRACE GRAVEL AND COBBLES **(FILL)**		1	50 mm D.O.	>100	83		SURFACE SEAL
	1.28 81.25	VERY STIFF GREY BROWN **SILTY CLAY**, OCCASIONAL SAND SEAM (WEATHERED CRUST)		2	"	6	82		PLASTIC - TUBING
	2.07 79.21	DENSE BROWN FINE TO MEDIUM **SAND**, TRACE SILT		3 4 5	" " "	37 36 42	81 80	M	NATIVE - BACKFILL
	4.11 78.44	DENSE BROWN **SILTY FINE SAND**					79		
	4.88 77.93	VERY DENSE BROWN FINE TO COARSE **SAND**, SOME SILT		6	"	67	78	MH	
	5.39 77.07	VERY DENSE BROWN **SAND AND GRAVEL**, OCCASIONAL COBBLE, TRACE SILT		7 8	" "	>100 >100			STANDPIF
	6.25	END OF HOLE AUGER REFUSAL PROBABLY BOULDERS					77 76		STANDPIF DESTROYE MAR.

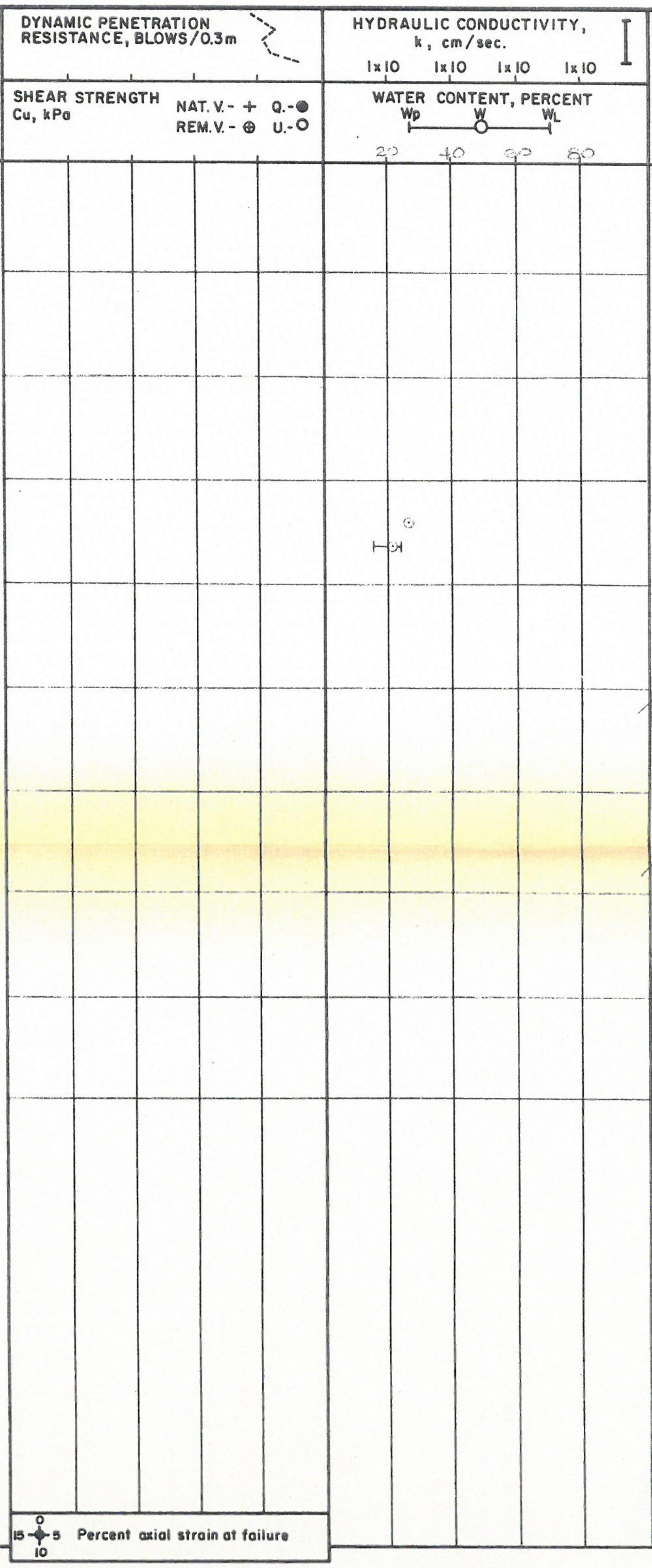
DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m

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

HYDRAULIC CONDUCTIVITY, k, cm/sec.

1x10 1x10 1x10 1x10

WATER CONTENT, PERCENT Wp W WL



VERTICAL SCALE
1:50

Golder Associates

DRAWN J.C.
CHECKED

G.A. l (m) ect No 41-2

RECORD OF BOREHOLE 105

LOCATION See Figure 2

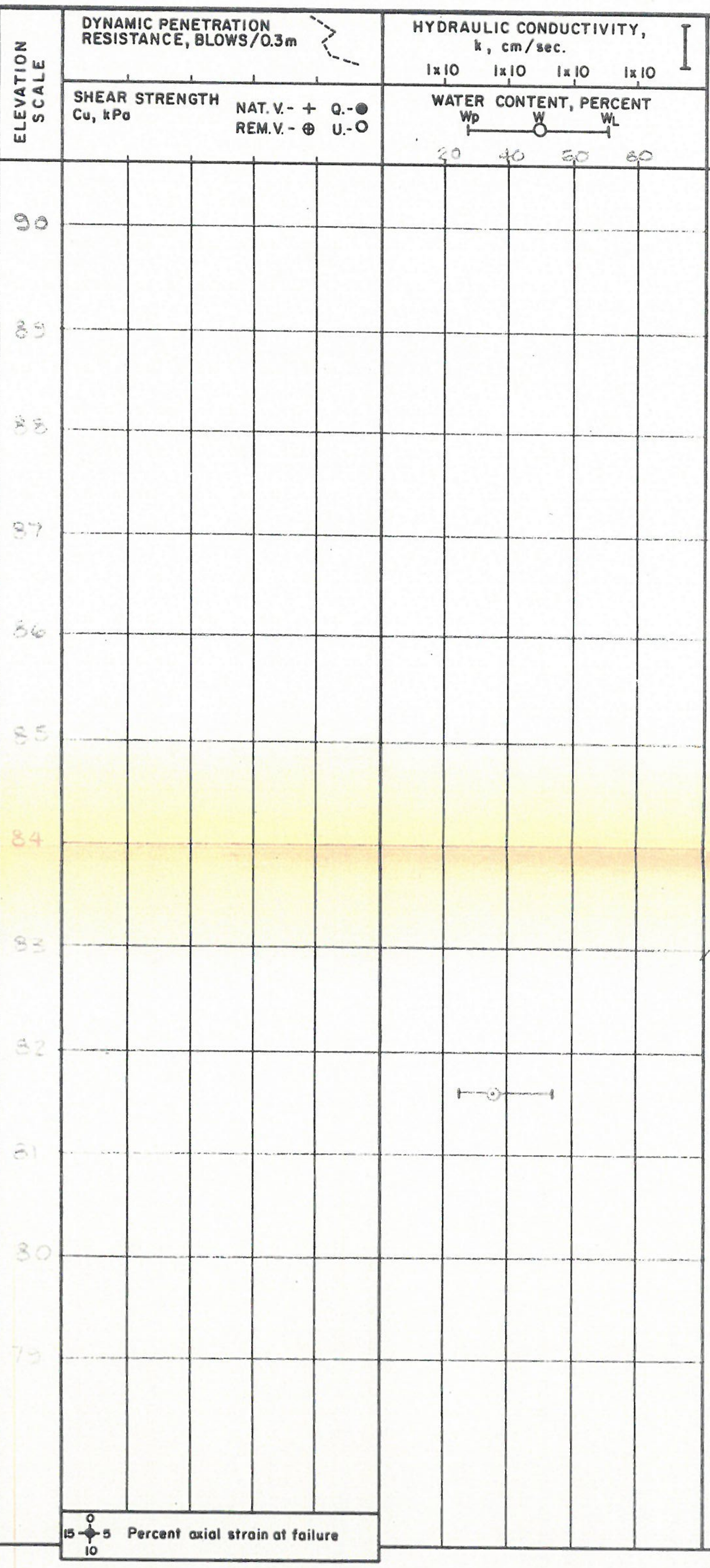
BORING DATE MAR. 22 ,1984

DATUM GEODETIC

SAMPLER HAMMER, 63.5 kg.; DROP, 760 mm

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

BORING METHOD	SOIL PROFILE			SAMPLES			ADDITIONAL LAB. TESTING	PIEZOMET OR STANDPIF INSTALLA
	ELEV'N. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m		
	89.4±	GROUND SURFACE						
POWER AUGER 200mm DIAM. (HOLLOW STEM)	0.00	BROWN FINE TO COARSE SAND, TRACE GRAVEL (FILL)	▨					
	88.64 0.76	FROZEN FINE SAND, TRACE SILT (FILL)	▨	1	50 mm D.O.	34		
				2	"	18		
				3	"	7		
	83.76 5.64	VERY LOOSE BROWN FINE **SAND**, TRACE SILT (FORMER PIT BOTTOM FILL)	▨	4	"	W.R.	MH	
				5	"	"		
	82.24 7.16	VERY STIFF GREY BROWN **SILTY CLAY**, OCCASIONAL SAND SEAM (WEATHERED CRUST)	🟩					
	81.60 7.80	VERY DENSE BROWN FINE TO MEDIUM **SAND**, TRACE SILT	🟨	6	"	>100		
				7	"	"		
				8	"	"		
	79.95 9.45	END OF HOLE						



VERTICAL SCALE
1:50

Golder Associates

DRAWN J.C.
CHECKED

Form G.A.-D-I.

Project No. 811-2269

811-2269

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

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3 m

COEFFICIENT OF PERMEABILITY, k., CM./SEC. 1x10 1x10 1x10 1x10

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

WATER CONTENT, PERCENT Wp W WL

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

BH. 81-6

ELEV'N. DEPTH	DESCRIPTION	SAMPLES NUMBER	TYPE	BLOWS/0.3 m	ELEVATION SCALE	ADDITIONAL LAB. TESTING	PIL... OR STANDPIPE INSTALLATION
81.9±	GROUND SURFACE				82		
0.00	LOOSE TO COMPACT LIGHT BROWN FINE TO MEDIUM SAND & SILTY SAND, SOME GRAVEL	1	50 mm D.O.	13	81	MH	
		2	"	8	80		
		3	"	17			
78.94 2.96	DENSE, GREY BROWN SILTY SAND, SOME GRAVEL (GLACIAL TILL)	4	"	78	79		
78.24 3.66	END OF HOLE				78		BOREHOLE DRY @ COMPLETION OF DRILLING SEPT. 9, 1981

BH. 81-7

ELEV'N. DEPTH	DESCRIPTION	SAMPLES NUMBER	TYPE	BLOWS/0.3 m	ELEVATION SCALE	ADDITIONAL LAB. TESTING	PIL... OR STANDPIPE INSTALLATION
	BROWN SILTY TOPSOIL						
105.2±	GROUND SURFACE						
0.00 0.21					105		
	COMPACT TO DENSE LIGHT BROWN FINE TO MEDIUM SAND, SOME GRAVEL	1	50 mm D.O.	6	104		
		2	"	5	103		
		3	"	29			
		4	"	36	102		
		5	"	25	101		
100.23 4.97	COMPACT LIGHT BROWN FINE SAND	6	"	27			
5.18	END OF HOLE				100		BOREHOLE DRY @ COMPLETION OF DRILLING SEPT. 11, 1981
					99		

15 -◆- 5 Percent axial strain at failure (0 above, 10 below)

VERTICAL SCALE 1:50

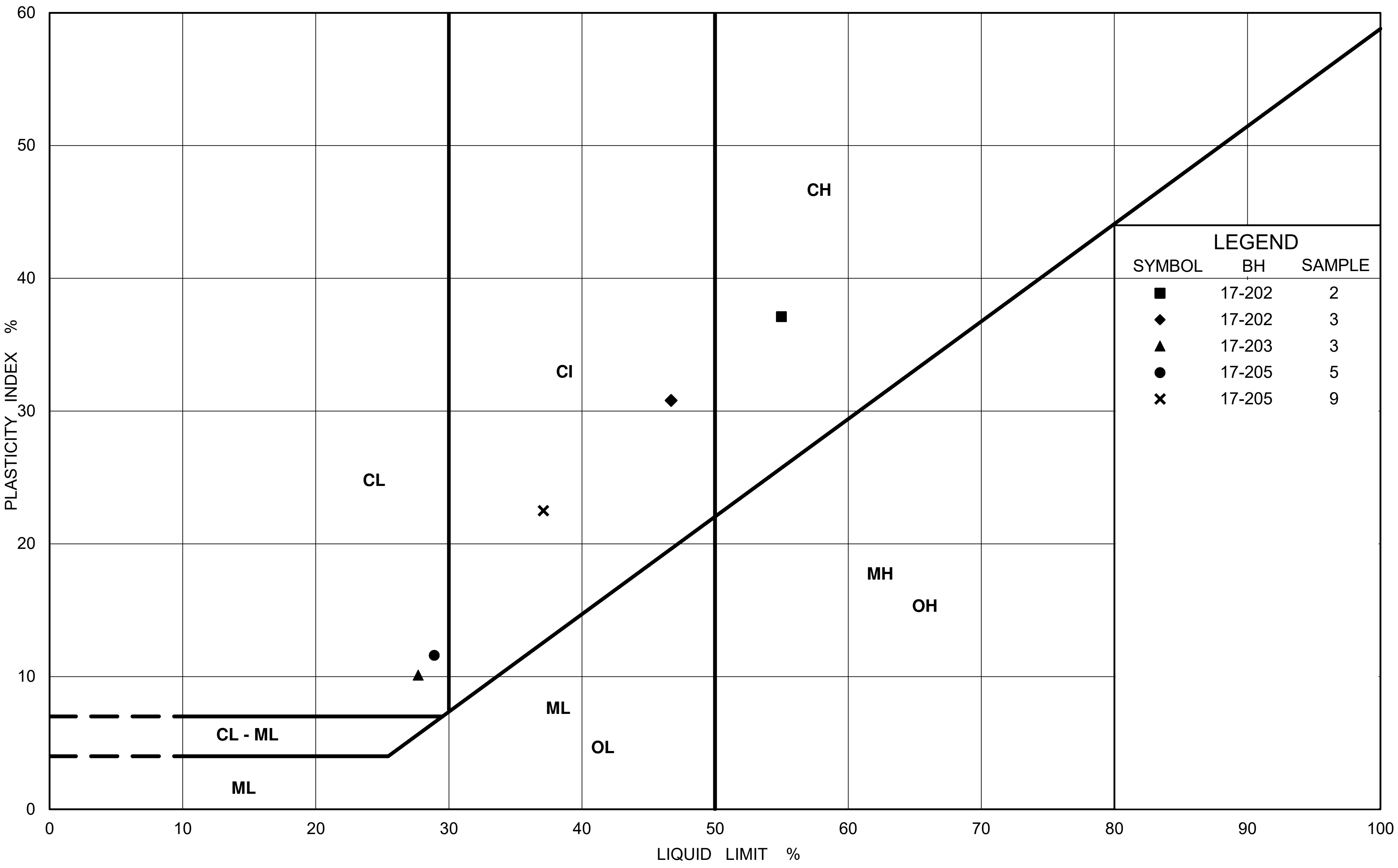
Golder Associates

DRAWN RKB

CHECKED

APPENDIX B

Results of Laboratory Testing (2017)



Golder Associates

PLASTICITY CHART
FILL

Figure : B1

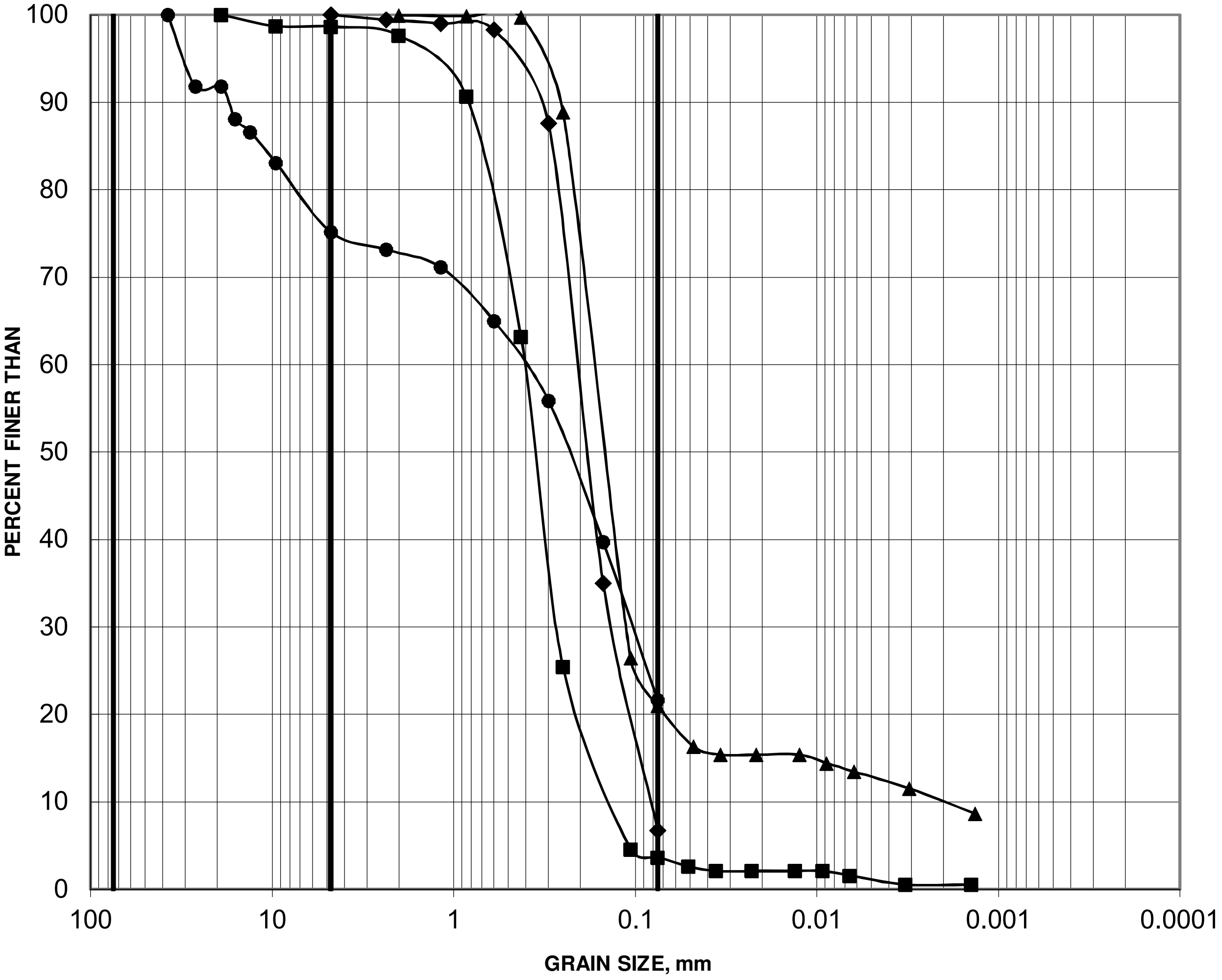
Project No.: 1670692 /4000

Compiled By : MI Checked By : CNM

GRAIN SIZE DISTRIBUTION

FIGURE B2

FILL



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

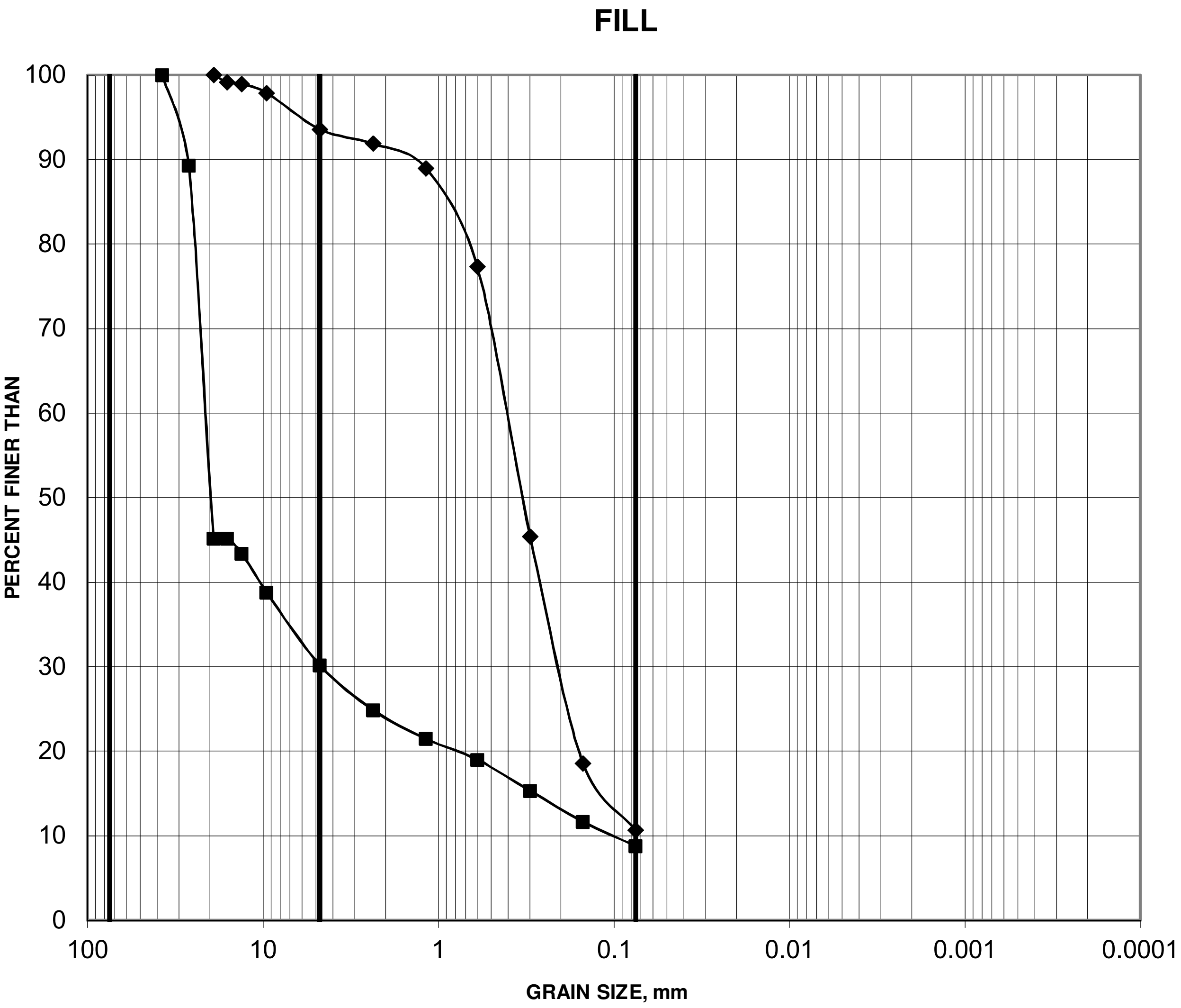
Created by: MI
Checked by: CNM

Project: 1670692 /4000

Golder Associates

GRAIN SIZE DISTRIBUTION

FIGURE B3



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

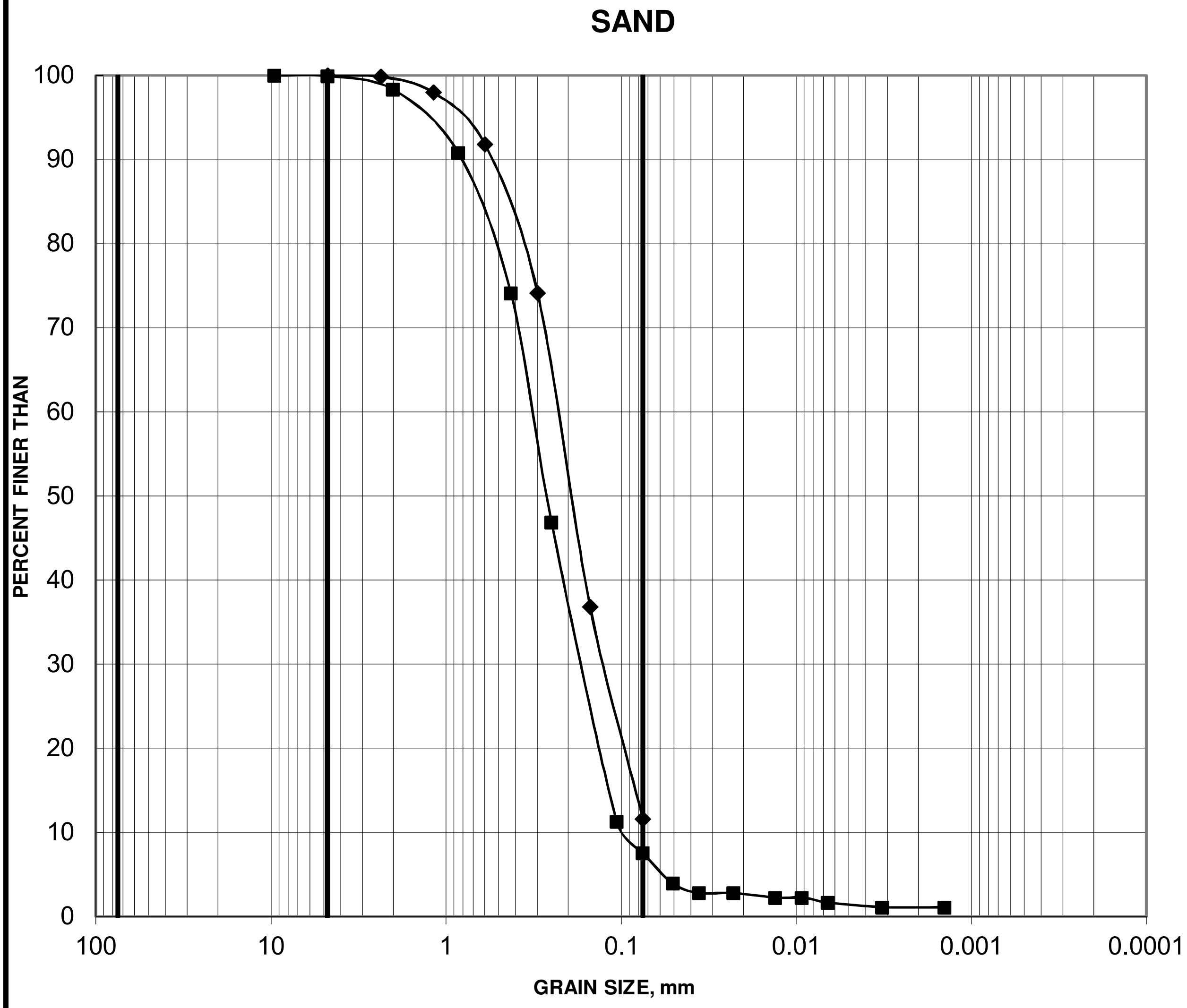
Project: 1670692 /4000

Golder Associates

Created by: MI
Checked by: CNM

GRAIN SIZE DISTRIBUTION

FIGURE B4



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

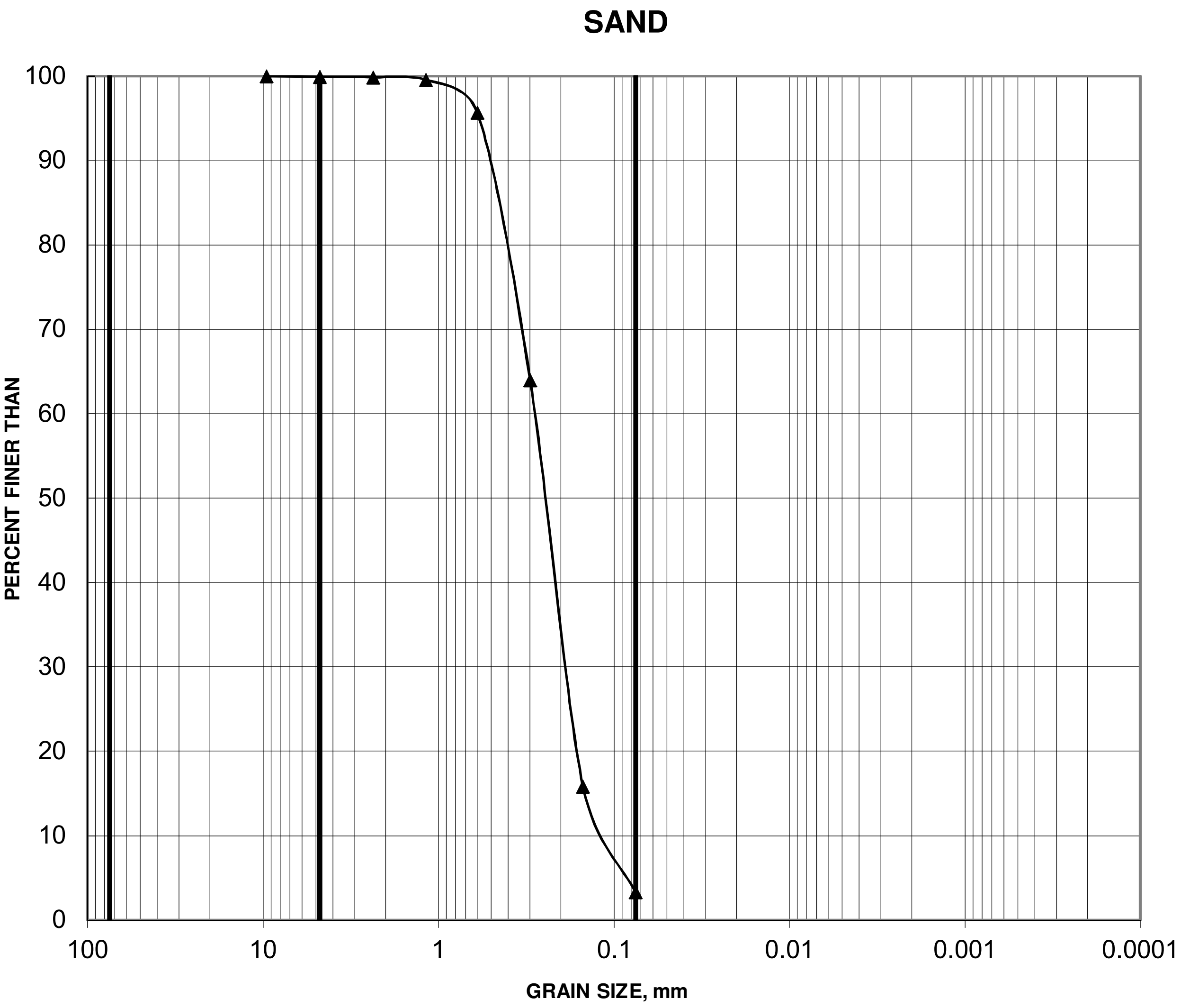
Project: 1670692

Golder Associates

Created by: MI
Checked by: CNM

GRAIN SIZE DISTRIBUTION

FIGURE B5



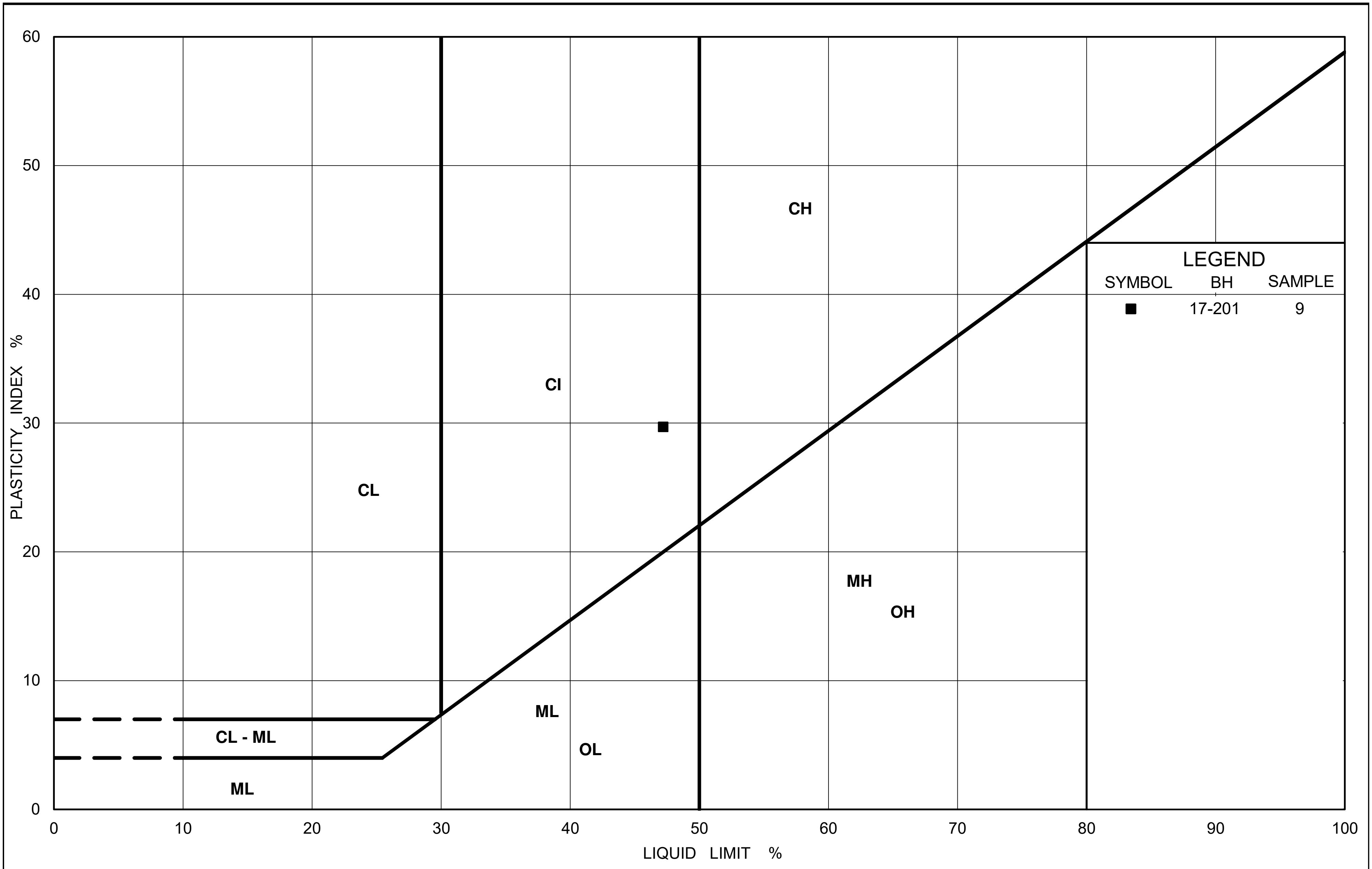
Cobble	coarse	fine	coarse	medium	fine	SILT AND CLAY
Size	GRAVEL SIZE		SAND SIZE			

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

Project: 1670692 /4000

Golder Associates

Created by: MI
Checked by: CNM



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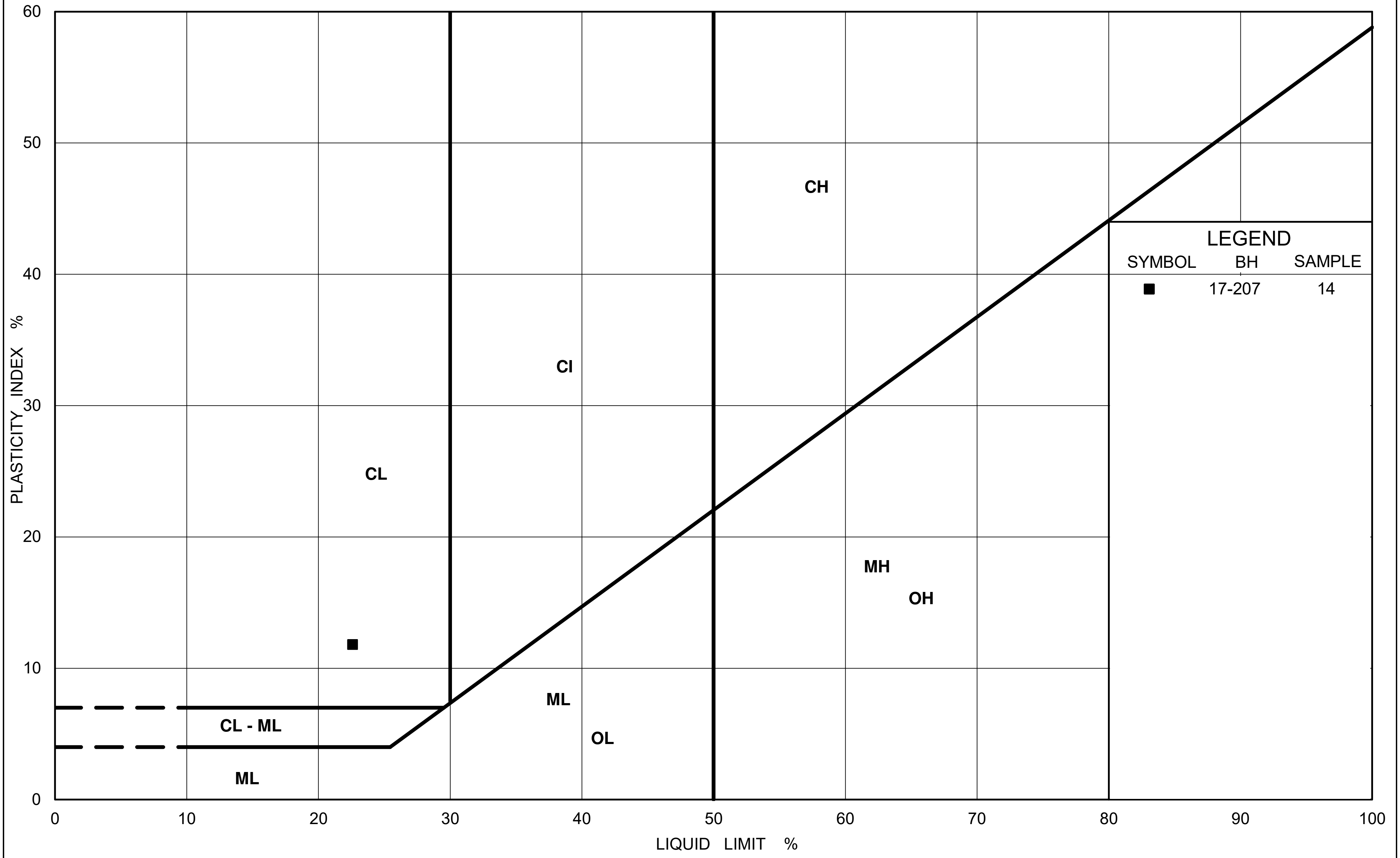
PLASTICITY CHART
Sandy SILTY CLAY

Figure : B6

Project No.: 1670692 /4000

Compiled By : MI Checked By : CNM



Golder Associates

PLASTICITY CHART
CLAYEY SILT and SILTY CLAY

Figure : B7

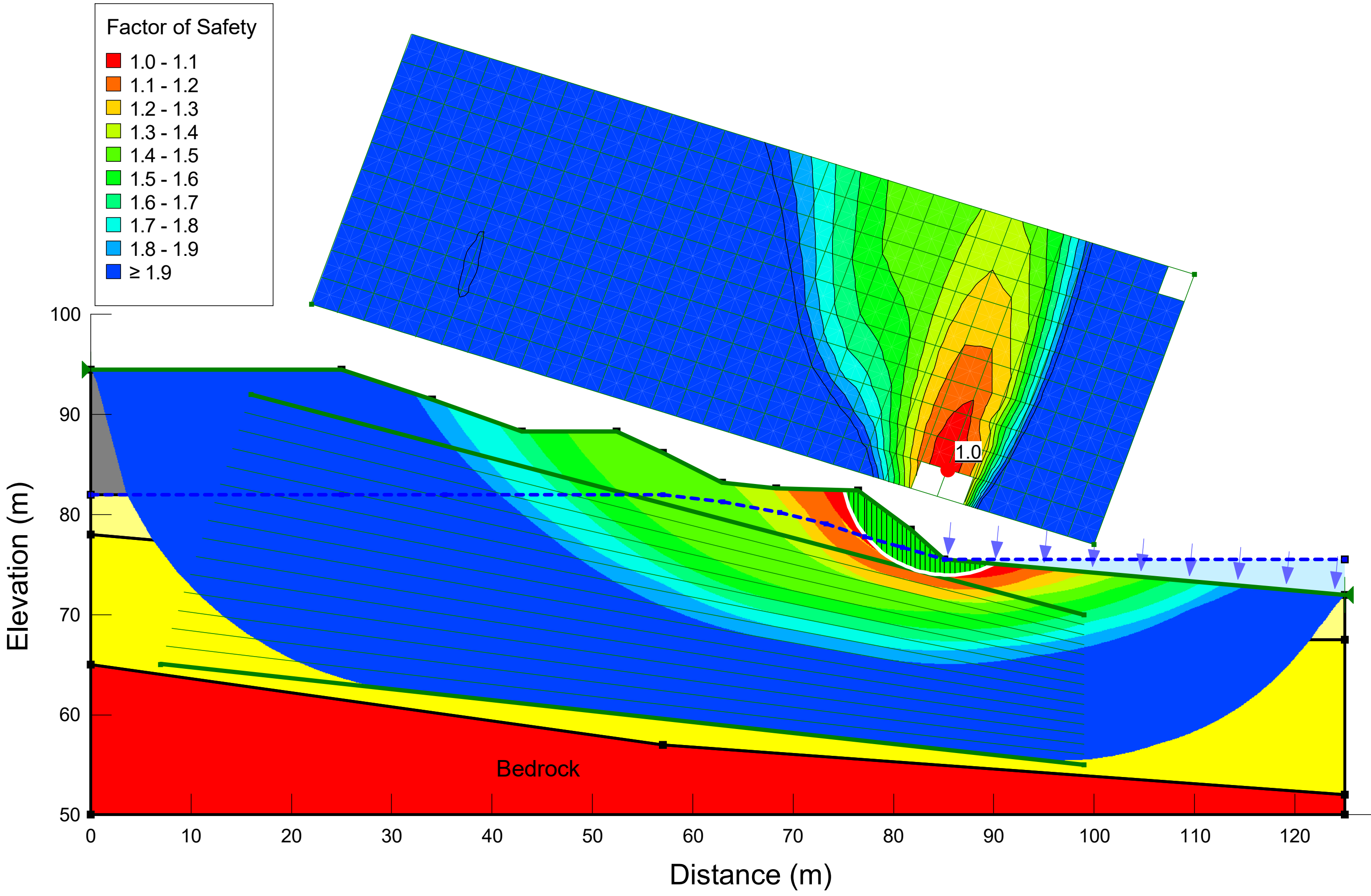
Project No.: 1670692 /4000

Compiled By : MI Checked By : CNM

APPENDIX C

Slope/W Results – Sections A-A’ to I-I’

Title: Cross Section A-A
File Name: 21482114_Section A-A_1m higher_MG.gsz
Name: Static Drained
Method: Morgenstern-Price
Horz Seismic Coef.: 0



🟥
Name: Bedrock
Slope Stability Material Model: Bedrock (Impen
Piezometric Line: 1

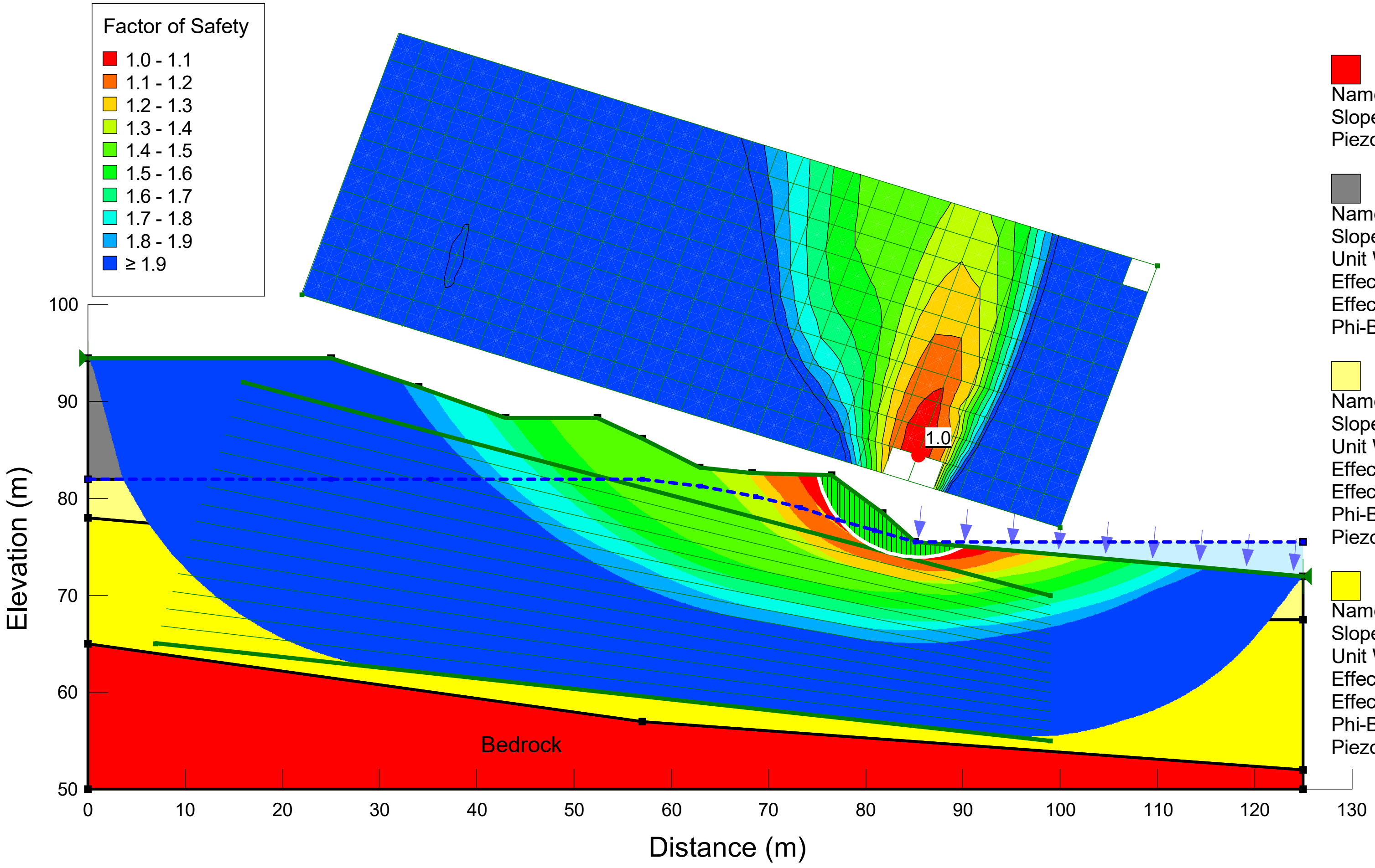
⬛
Name: FIll
Slope Stability Material Model: Mohr-Coulomb
Unit Weight: 19
Effective Cohesion: 0
Effective Friction Angle: 28
Phi-B: 0

🟨
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

Title: Cross Section A-A
File Name: 21482114_Section A-A_1m higher_MG.gsz
Name: Static Undrained
Method: Morgenstern-Price
Horz Seismic Coef.: 0



🟥
Name: Bedrock
Slope Stability Material Model: Bedrock (Impen
Piezometric Line: 1

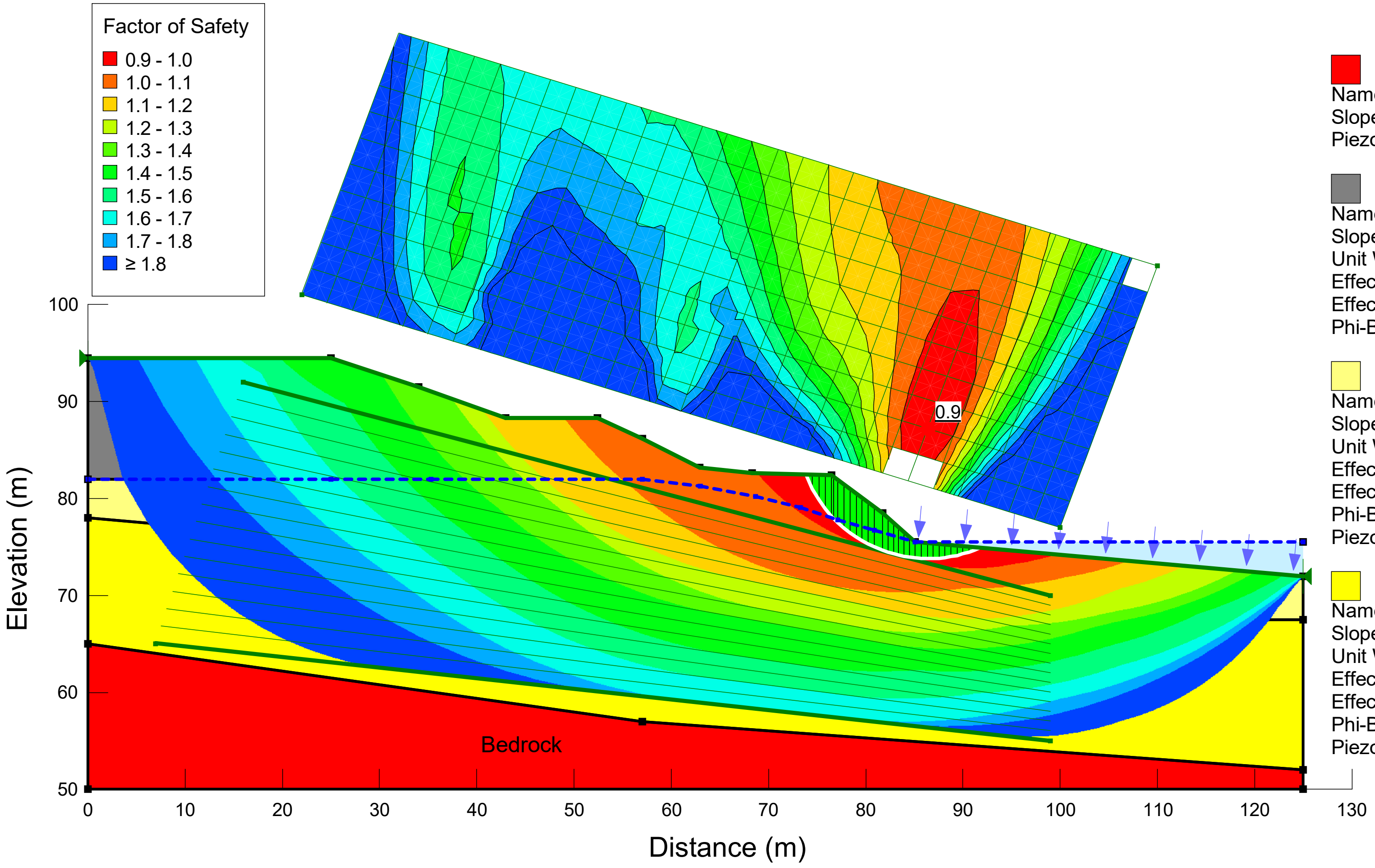
▨
Name: FIll
Slope Stability Material Model: Mohr-Coulomb
Unit Weight: 19
Effective Cohesion: 0
Effective Friction Angle: 28
Phi-B: 0

🟨
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 Undrained	
21482114_Section A-A_1m higher_MG.gsz	
03/11/2022	1:710

Title: Cross Section A-A
File Name: 21482114_Section A-A_1m higher_10%_MG.gsz
Name: Seismic Undrained
Method: Morgenstern-Price
Horz Seismic Coef.: 0.155



🟥
Name: Bedrock
Slope Stability Material Model: Bedrock (Impene
Piezometric Line: 1

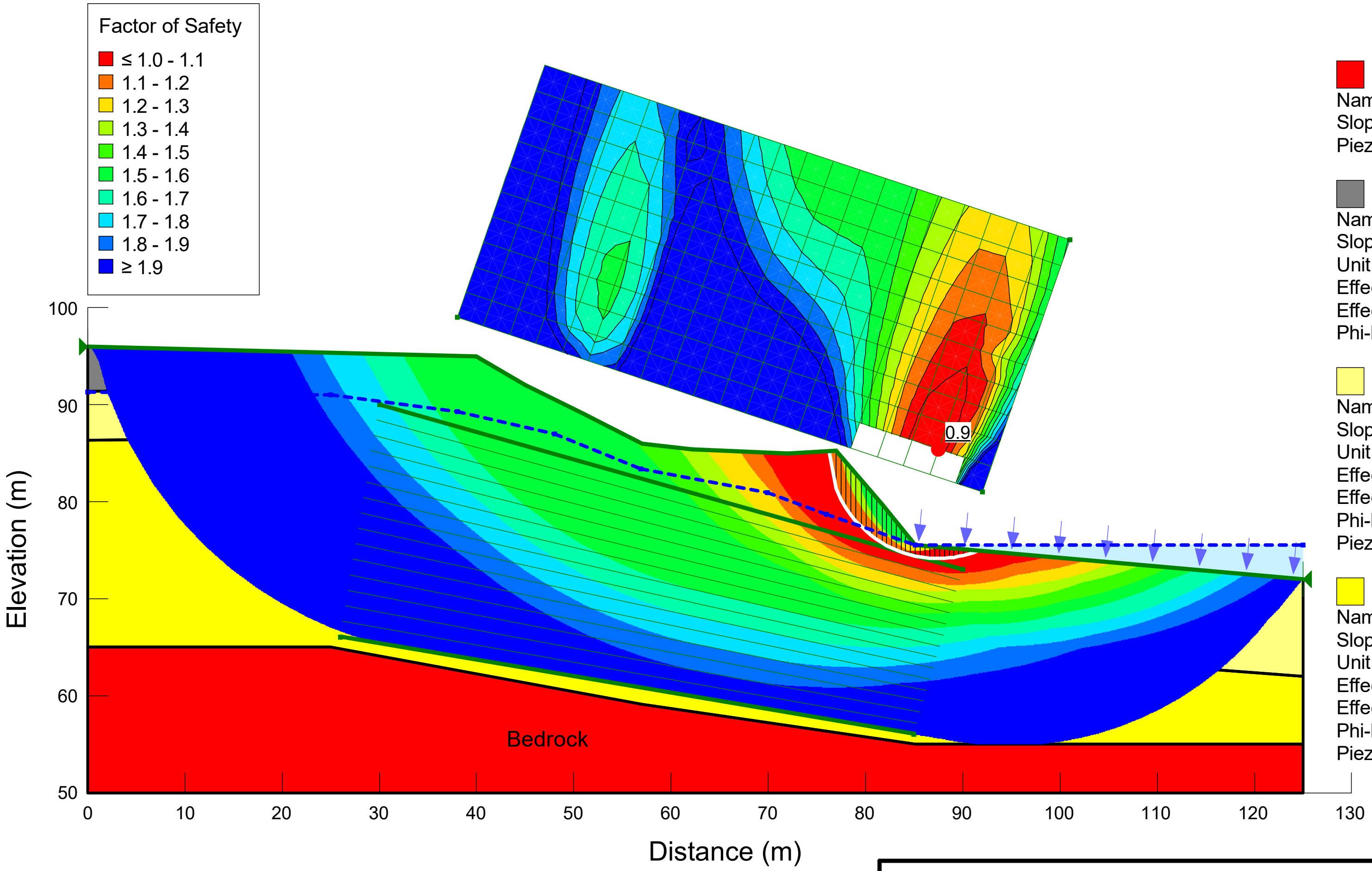
⬛
Name: FIll
Slope Stability Material Model: Mohr-Coulomb
Unit Weight: 19
Effective Cohesion: 0
Effective Friction Angle: 30.8
Phi-B: 0

🟨
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 A-A_1m higher_10%_MG.gsz	
03/11/2022	1:710

Title: Cross Section B-B
File Name: 21482114_Section B-B_1m higher_MG.gsz
Name: Static Drained
Method: Morgenstern-Price
Horz Seismic Coef.: 0



🟥
Name: Bedrock
Slope Stability Material Model: Bedrock (Impenetrab
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: 32
Phi-B: 0
Piezometric Line: 1

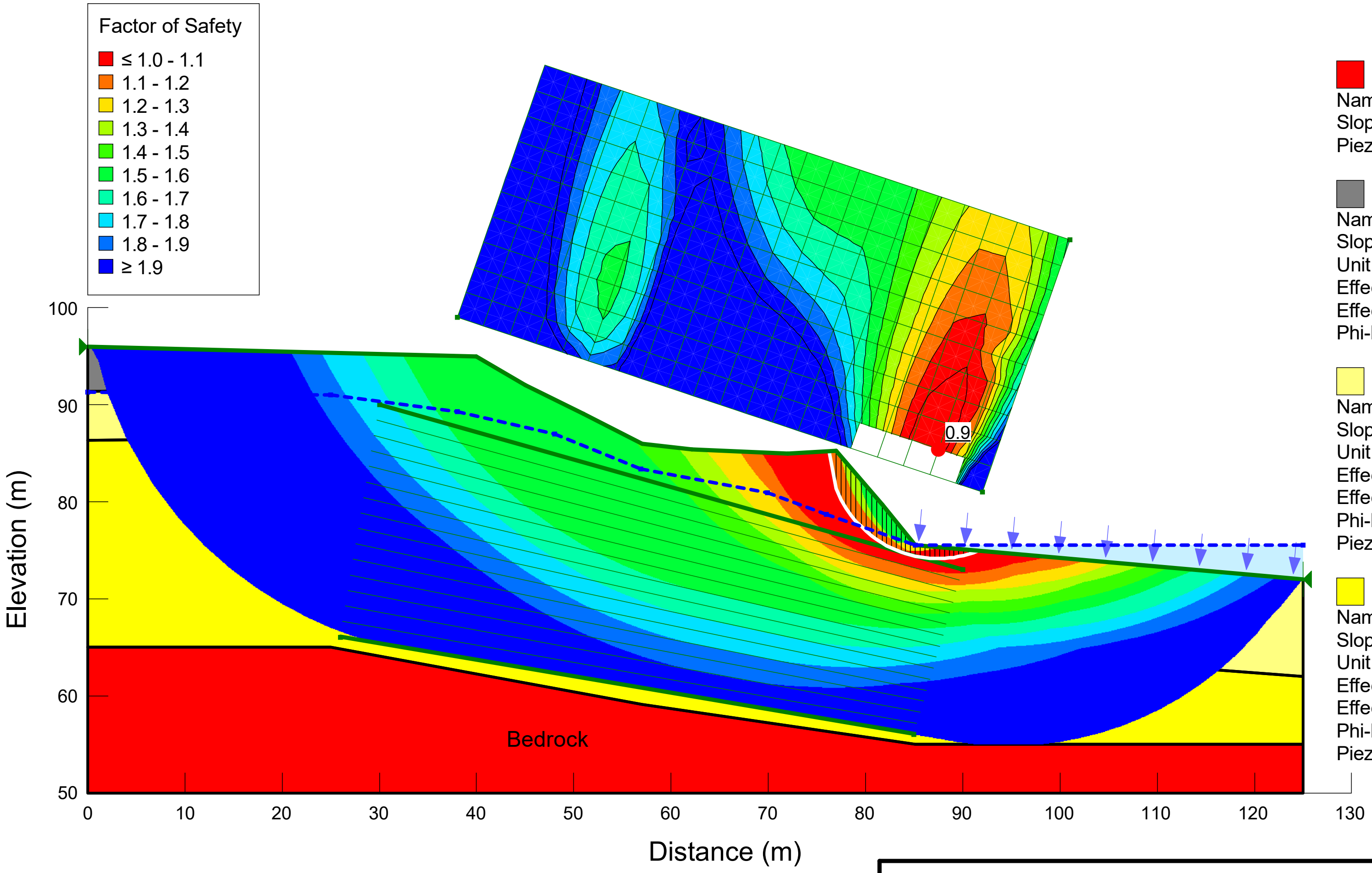
🟨
Name: SAND AND GRAVEL
Slope Stability Material Model: Mohr-Coulomb
Unit Weight: 20
Effective Cohesion: 0
Effective Friction Angle: 35
Phi-B: 0
Piezometric Line: 1

Static Drained

21482114_Section B-B_1m higher_MG.gsz

03/17/2022 1:720

Title: Cross Section B-B
File Name: 21482114_Section B-B_1m higher_MG.gsz
Name: Static Undrained
Method: Morgenstern-Price
Horz Seismic Coef.: 0



🟥
Name: Bedrock
Slope Stability Material Model: Bedrock (Impenetrab
Piezometric Line: 1

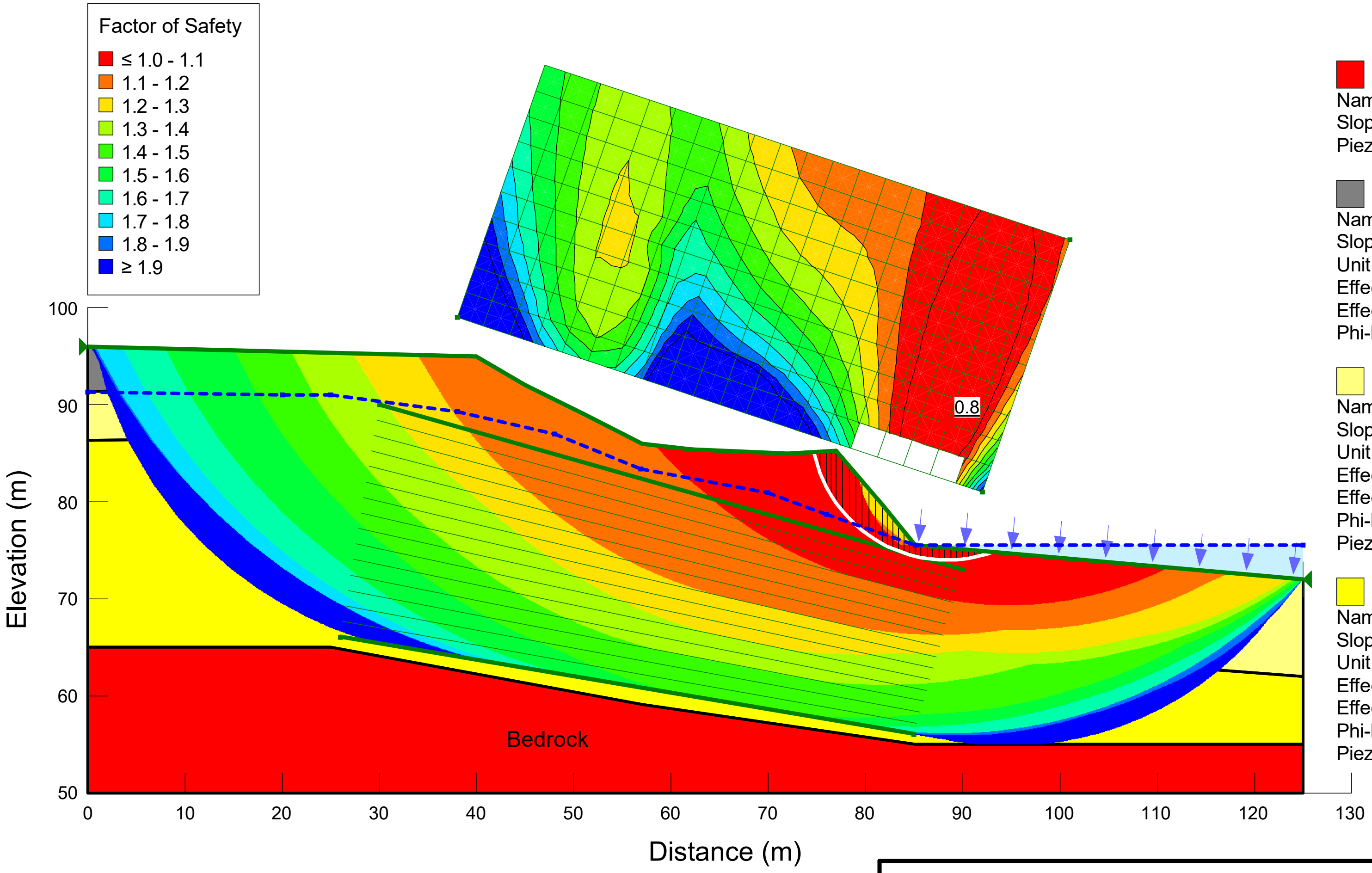
⬛
Name: FIll
Slope Stability Material Model: Mohr-Coulomb
Unit Weight: 19
Effective Cohesion: 0
Effective Friction Angle: 28
Phi-B: 0

🟨
Name: SAND
Slope Stability Material Model: Mohr-Coulomb
Unit Weight: 19
Effective Cohesion: 0
Effective Friction Angle: 32
Phi-B: 0
Piezometric Line: 1

🟨
Name: SAND AND GRAVEL
Slope Stability Material Model: Mohr-Coulomb
Unit Weight: 20
Effective Cohesion: 0
Effective Friction Angle: 35
Phi-B: 0
Piezometric Line: 1

Static Undrained
21482114_Section B-B_1m higher_MG.gsz
03/17/2022 1:720

Title: Cross Section B-B
File Name: 21482114_Section B-B_1m higher_10%_MG.gsz
Name: Seismic Undrained
Method: Morgenstern-Price
Horz Seismic Coef.: 0.155



🟥
Name: Bedrock
Slope Stability Material Model: Bedrock (Impenetrab
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: 35.2
Phi-B: 0
Piezometric Line: 1

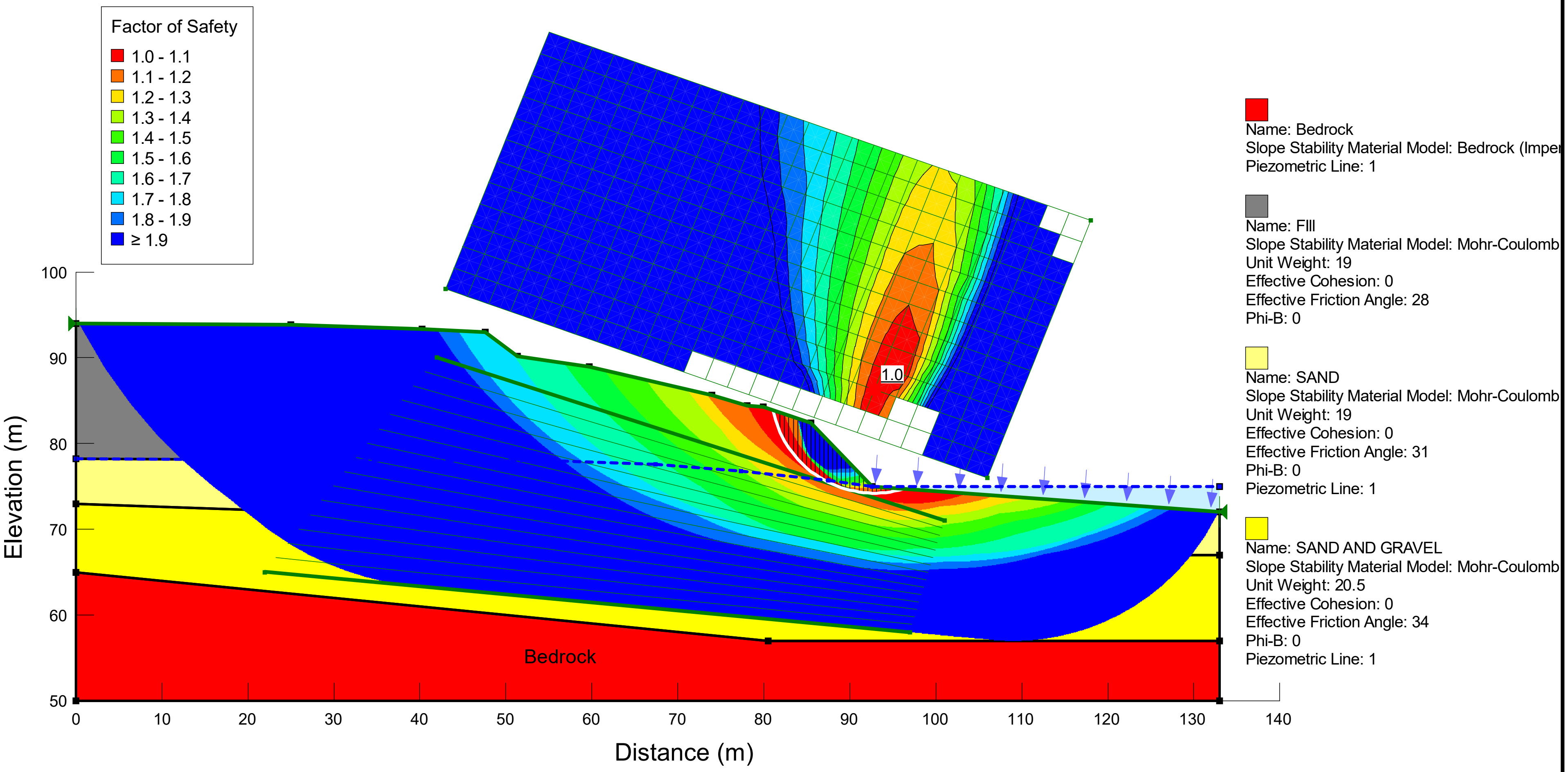
🟨
Name: SAND AND GRAVEL
Slope Stability Material Model: Mohr-Coulomb
Unit Weight: 20
Effective Cohesion: 0
Effective Friction Angle: 38.5
Phi-B: 0
Piezometric Line: 1

Seismic Undrained

21482114_Section B-B_1m higher_10%_MG.gsz

03/17/2022 1:720

Title: St Mary's Site
File Name: 21482114_Section C-C_1m higher_MG.gsz
Name: Static Drained
Method: Morgenstern-Price
Horz Seismic Coef.: 0

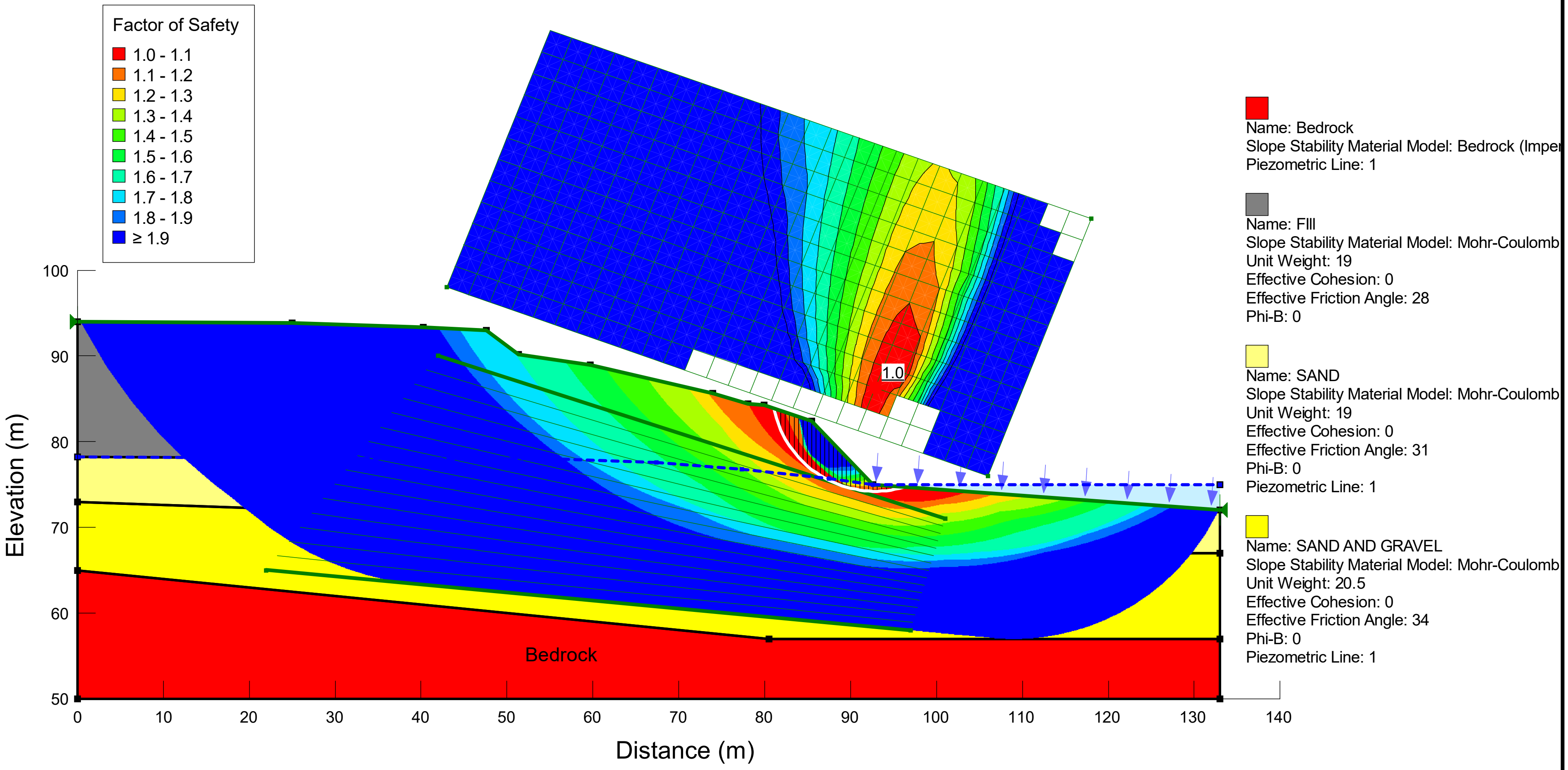


Static Drained

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_MG.gsz
Name: Static Undrained
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

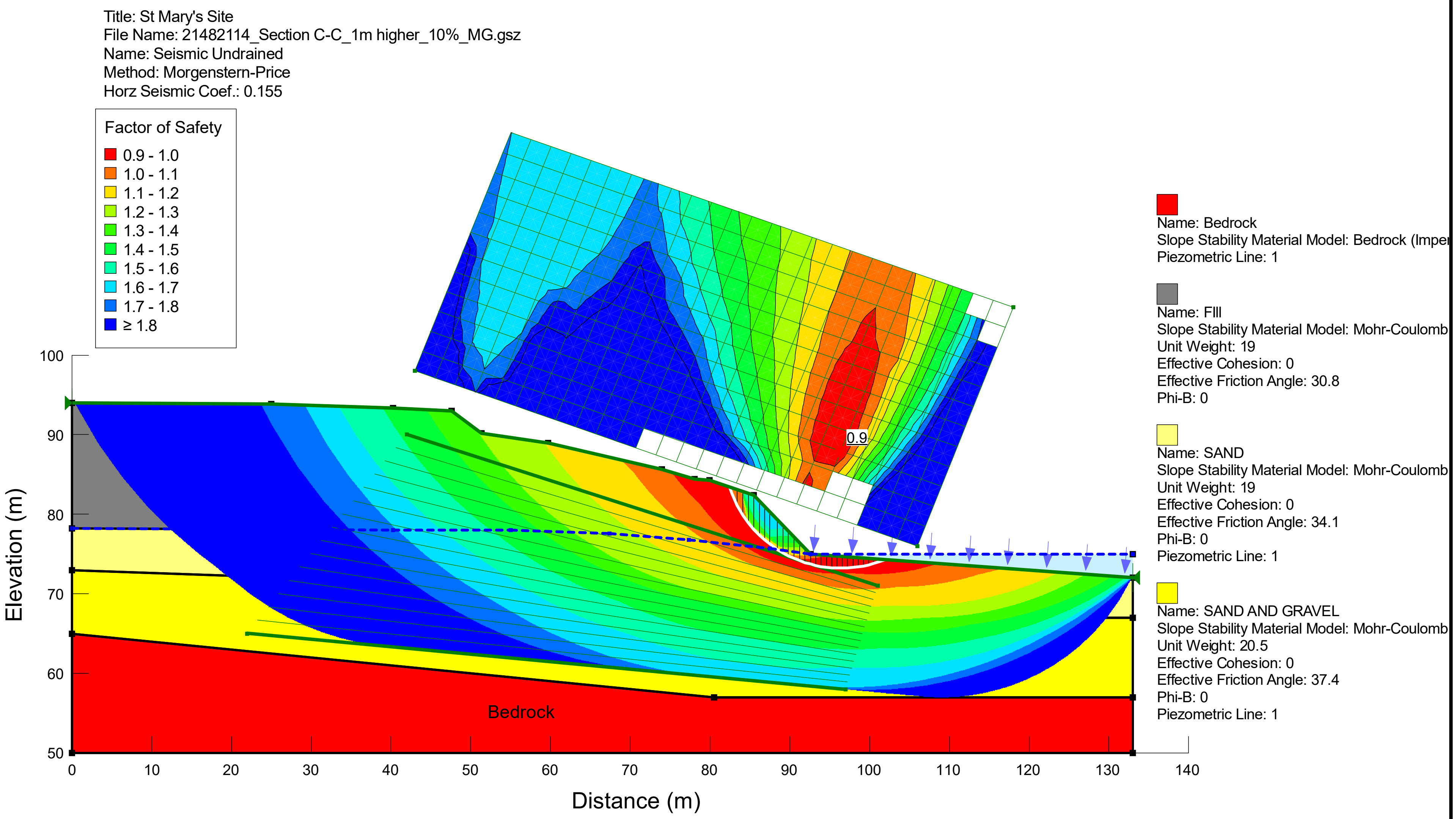
🟥
Name: Bedrock
Slope Stability Material Model: Bedrock (Imper
Piezometric Line: 1

⬛
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 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

🟥
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

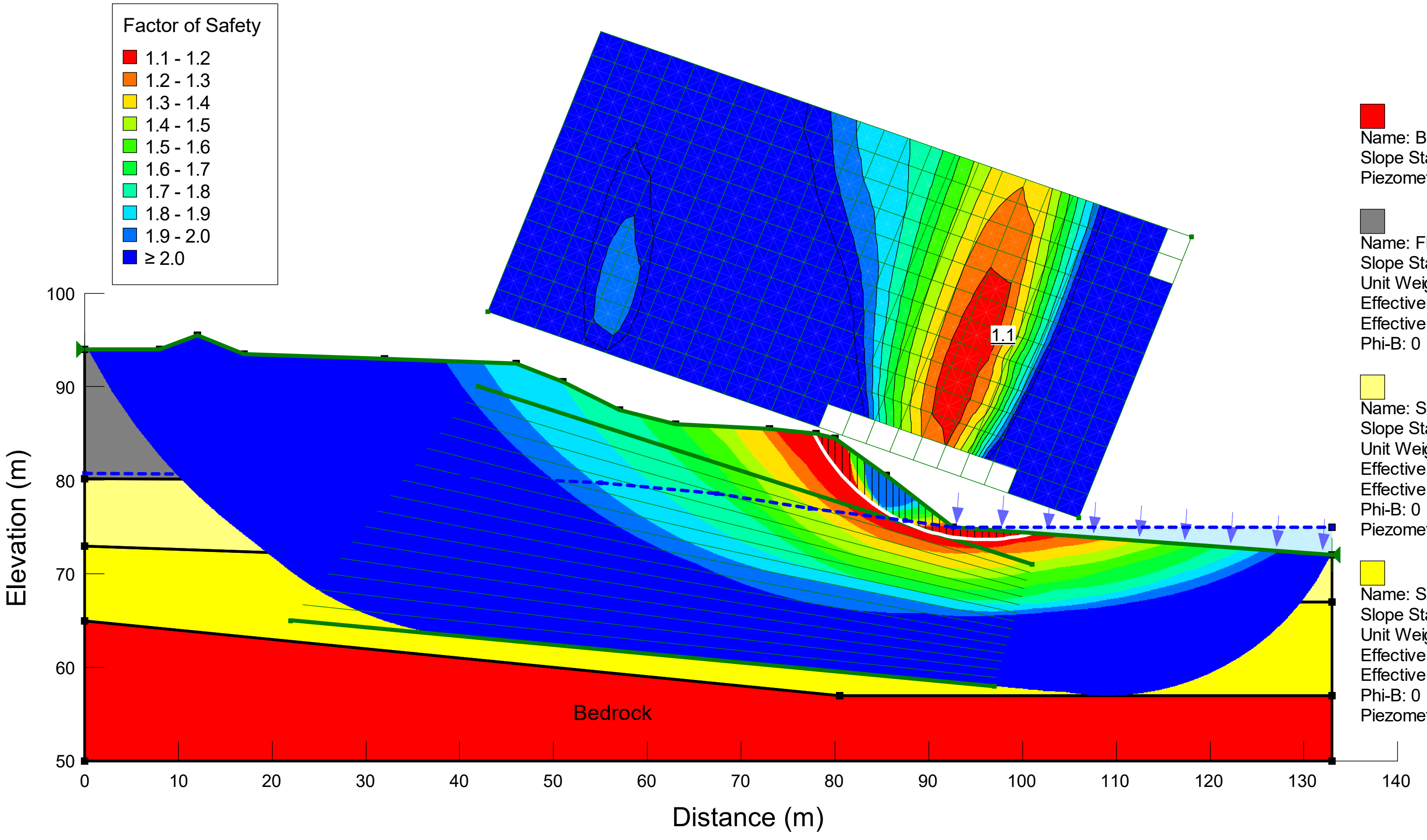
Name: SAND AND GRAVEL
Slope Stability Material Model: Mohr-Coulomb
Unit Weight: 20.5
Effective Cohesion: 0
Effective Friction Angle: 37.4
Phi-B: 0
Piezometric Line: 1

Seismic Undrained

21482114_Section C-C_1m higher_10%_MG.gsz

03/11/2022 1:720

Title: Cross Section D-D
File Name: 21482114_Section D-D_1m higher_MG.gsz
Name: Static Drained
Method: Morgenstern-Price
Horz Seismic Coef.: 0



🟥
Name: Bedrock
Slope Stability Material Model: Bedrock (Imper
Piezometric Line: 1

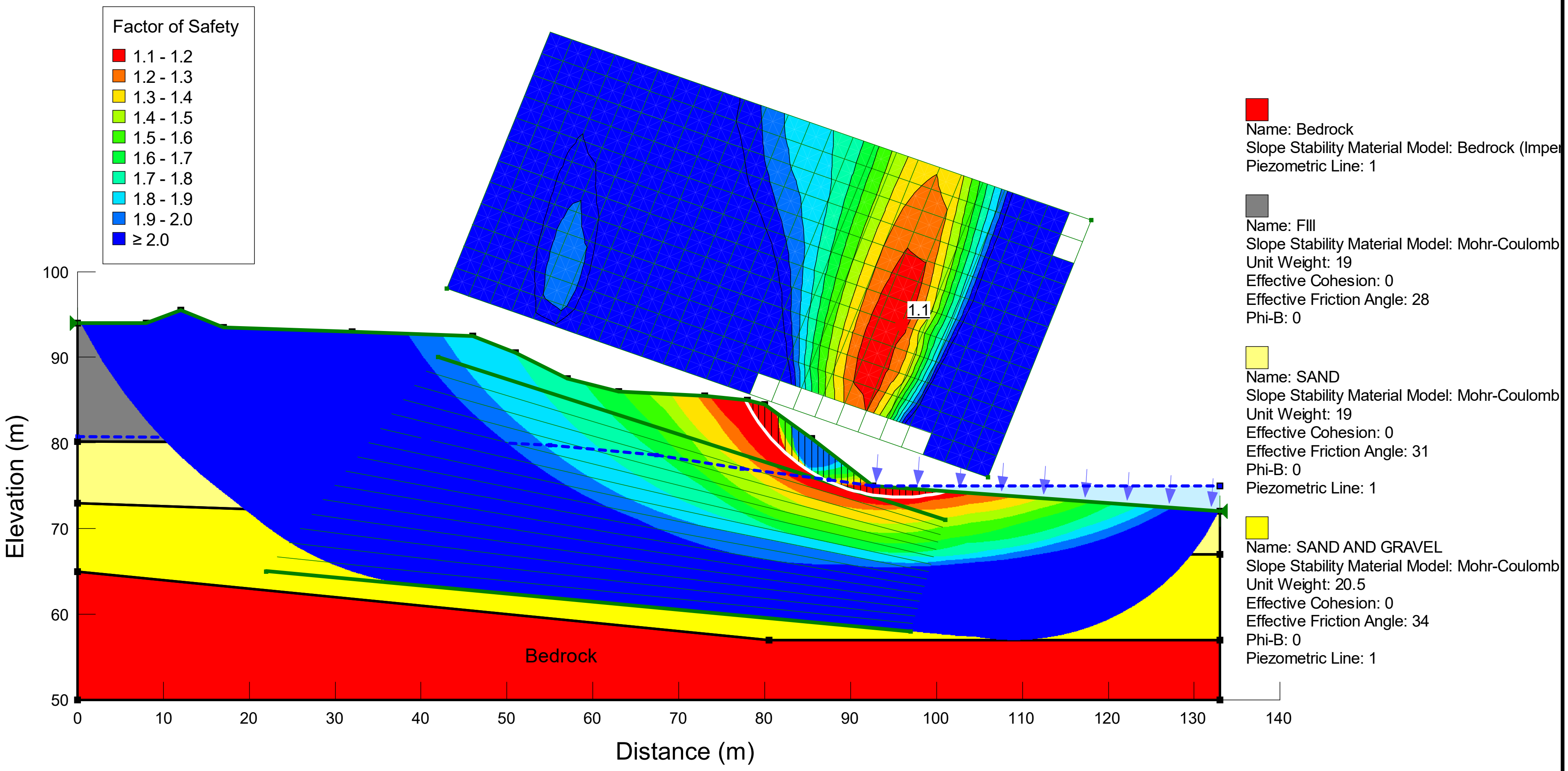
Name: FIll
Slope Stability Material Model: Mohr-Coulomb
Unit Weight: 19
Effective Cohesion: 0
Effective Friction Angle: 28
Phi-B: 0

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 D-D_1m higher_MG.gsz	
03/11/2022	1:720

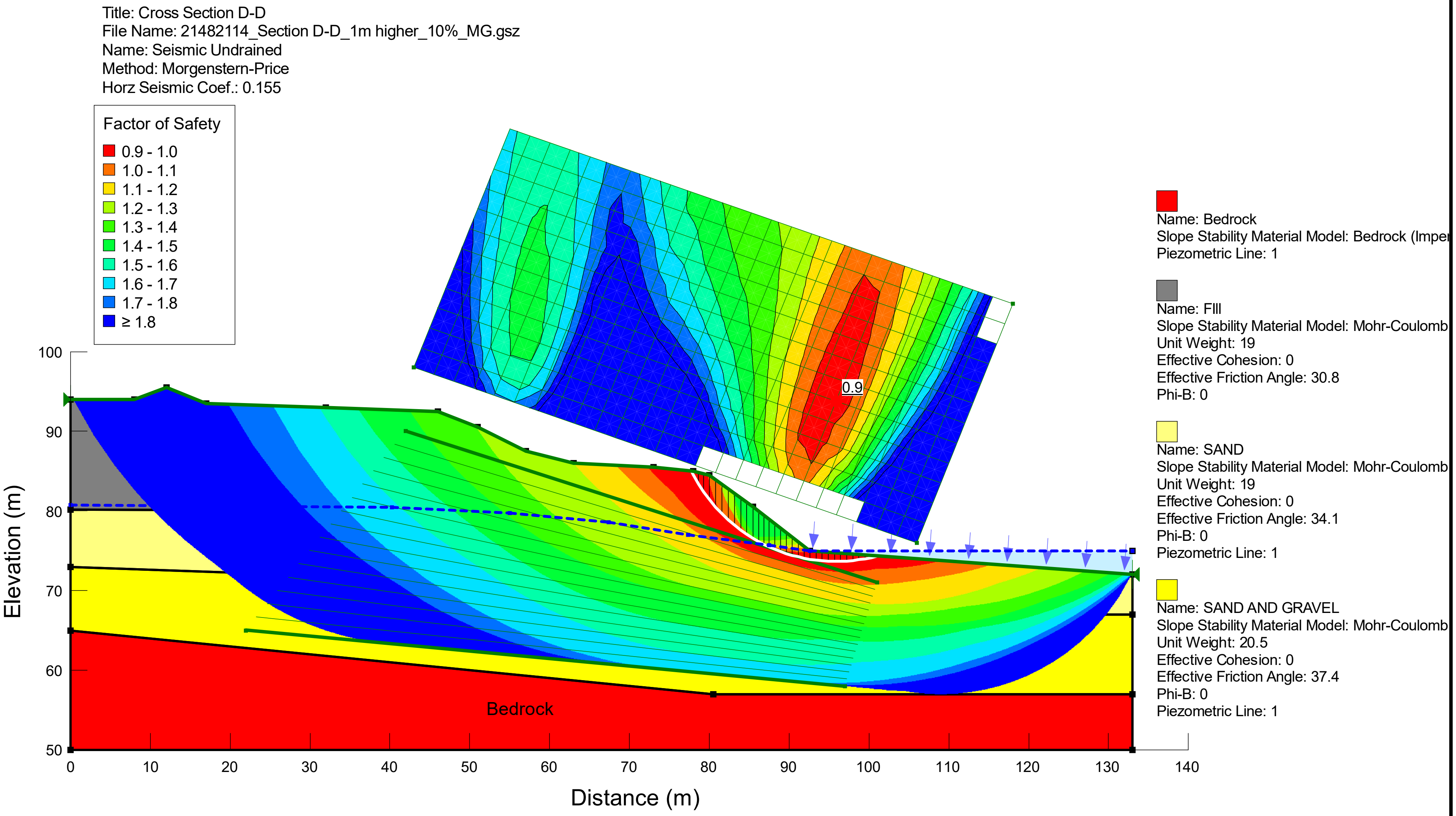
Title: Cross Section D-D
File Name: 21482114_Section D-D_1m higher_MG.gsz
Name: Static Undrained
Method: Morgenstern-Price
Horz Seismic Coef.: 0



Static Undrained

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_10%_MG.gsz
Name: Seismic Undrained
Method: Morgenstern-Price
Horz Seismic Coef.: 0.155

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

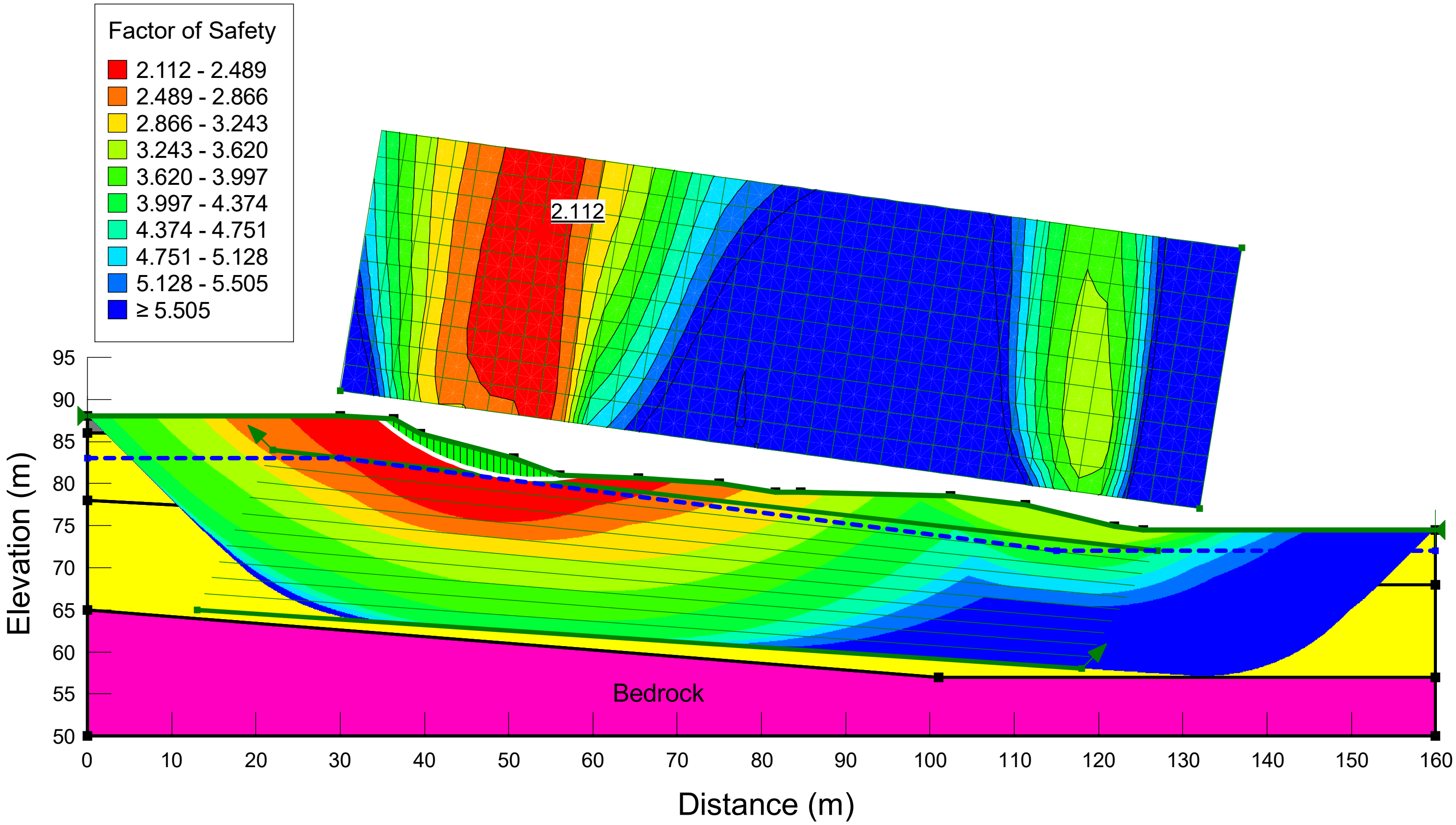
Name: SAND AND GRAVEL
Slope Stability Material Model: Mohr-Coulomb
Unit Weight: 20.5
Effective Cohesion: 0
Effective Friction Angle: 37.4
Phi-B: 0
Piezometric Line: 1

Seismic Undrained

21482114_Section 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



Name: Bedrock
Slope Stability Material Model: Bedrock (Impen
Piezometric Line: 1

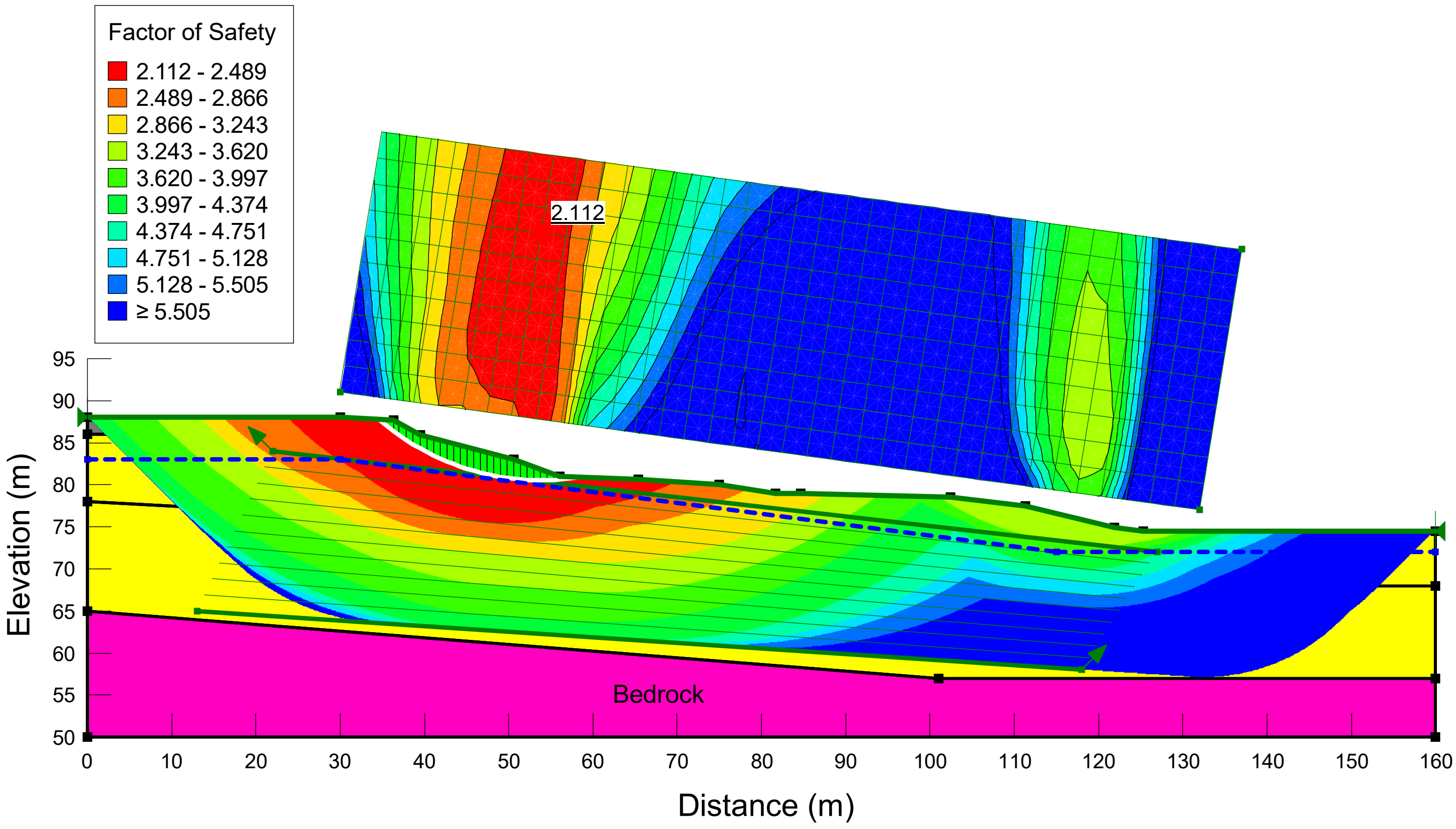
Name: Fill
Slope Stability Material Model: Mohr-Coulomb
Unit Weight: 19 kN/m³
Effective Cohesion: 0 kPa
Effective Friction Angle: 28 °
Phi-B: 0 °
Piezometric Line: 1

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

4.1 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

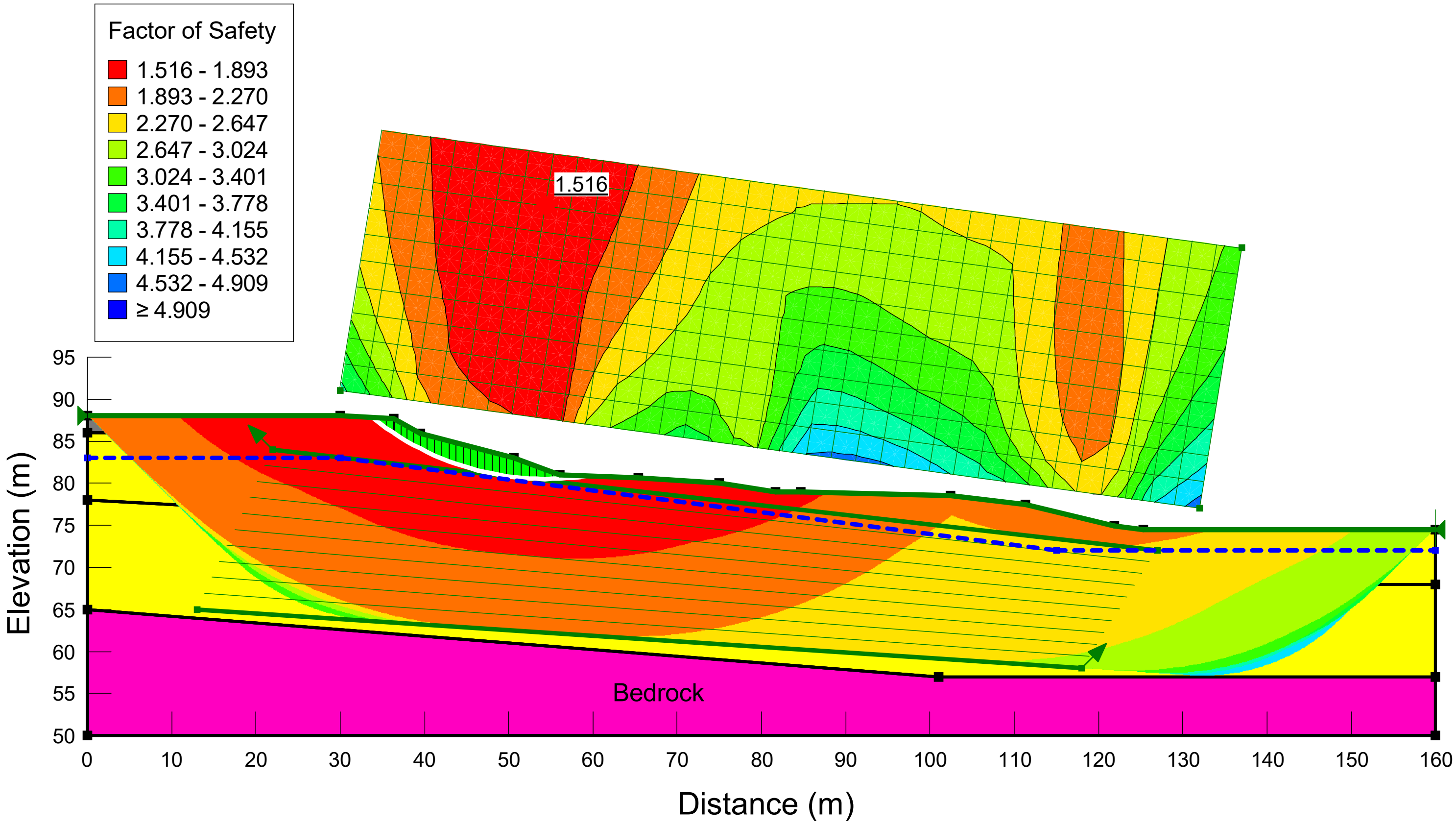
Name: Sand and Gravel
Slope Stability Material Model: Mohr-Coulomb
Unit Weight: 20 kN/m³
Effective Cohesion: 0 kPa
Effective Friction Angle: 34 °
Phi-B: 0 °
Piezometric Line: 1

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

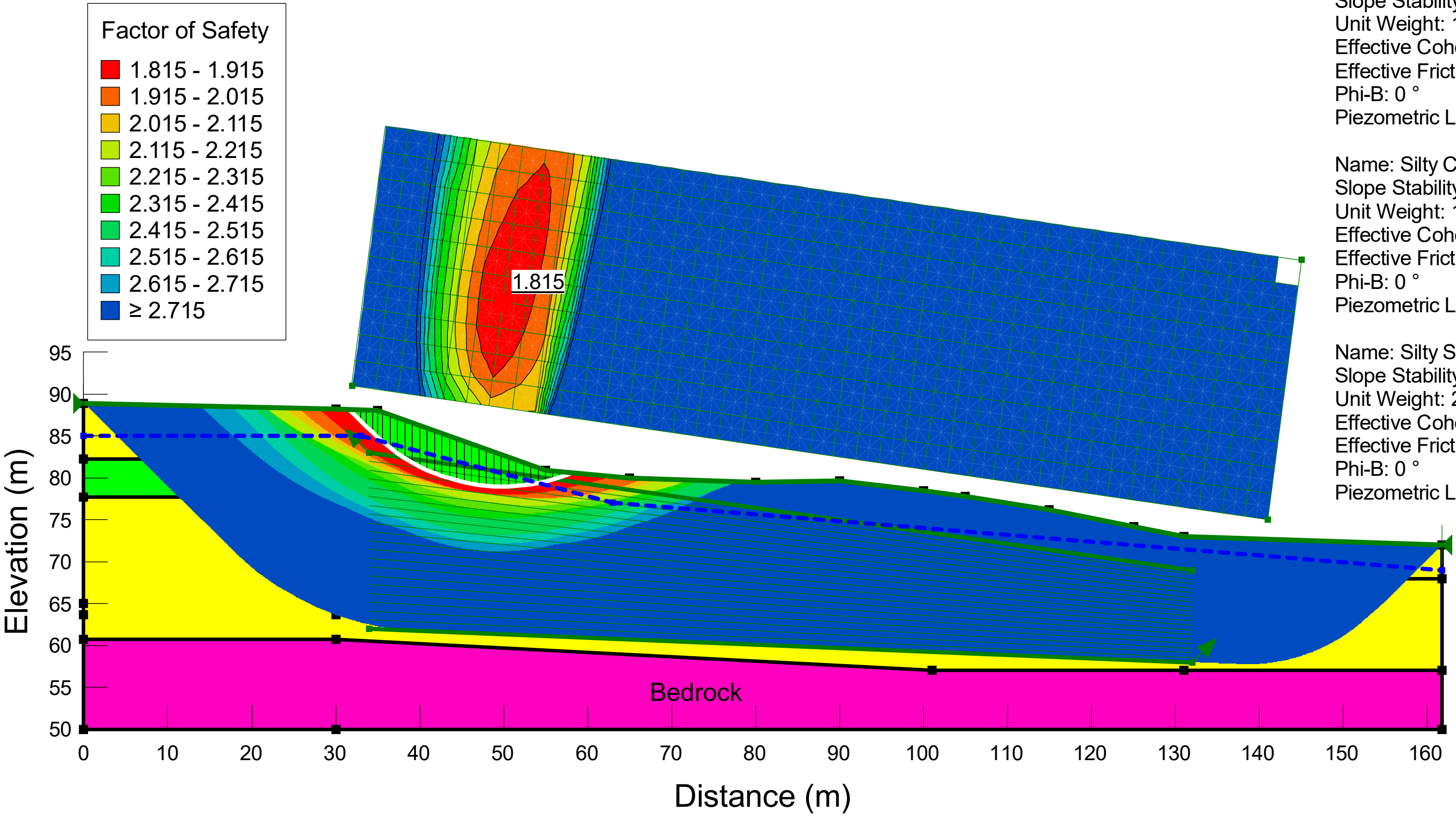
Name: Fill
Slope Stability Material Model: Mohr-Coulomb
Unit Weight: 19 kN/m³
Effective Cohesion: 0 kPa
Effective Friction Angle: 30.8 °
Phi-B: 0 °
Piezometric Line: 1

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

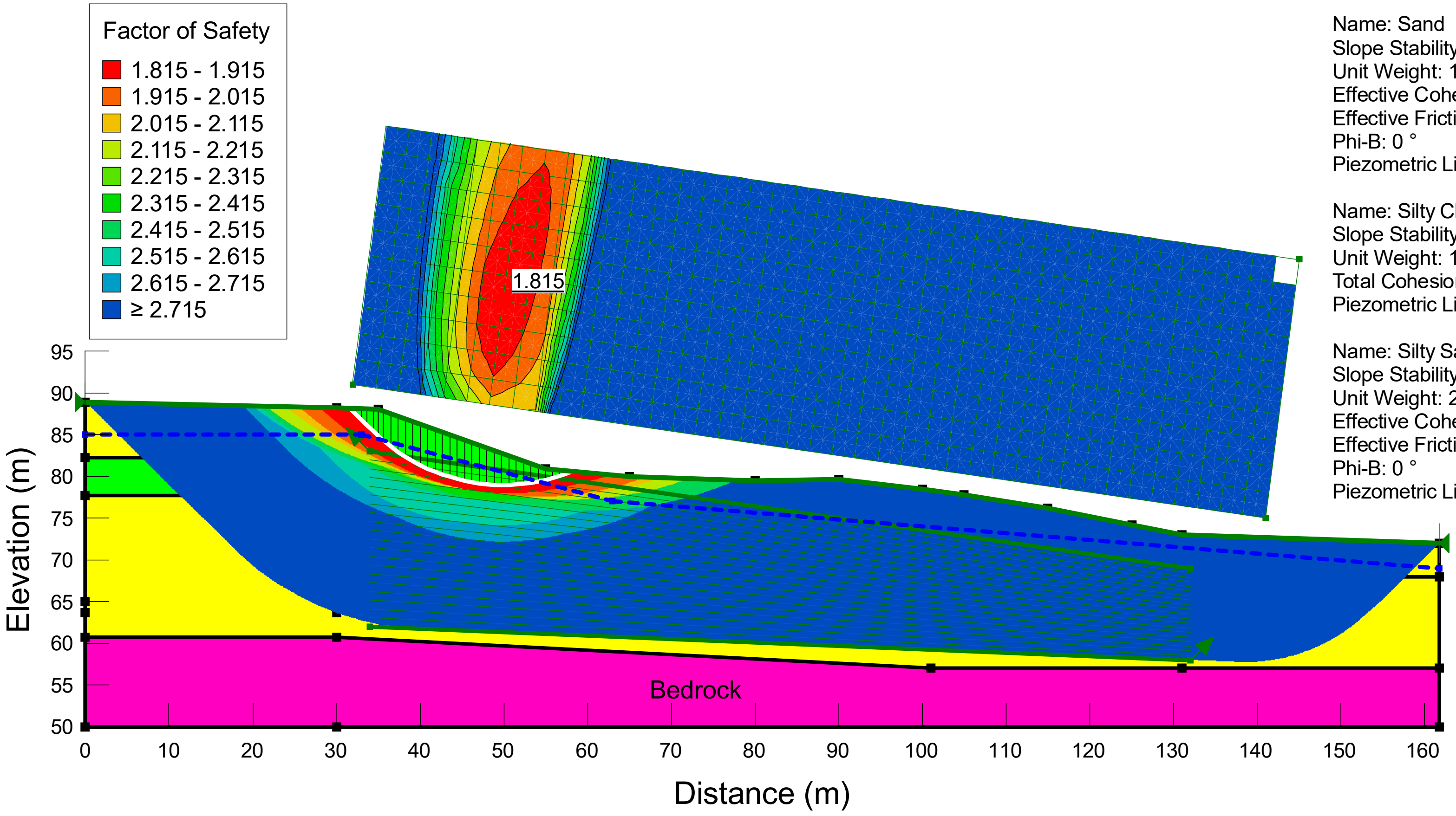
Name: Sand
Slope Stability Material Model: Mohr-Coulomb
Unit Weight: 19 kN/m³
Effective Cohesion: 0 kPa
Effective Friction Angle: 31 °
Phi-B: 0 °
Piezometric Line: 1

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



Name: Bedrock
Slope Stability Material Model: Bedrock (Impenetra
Piezometric Line: 1

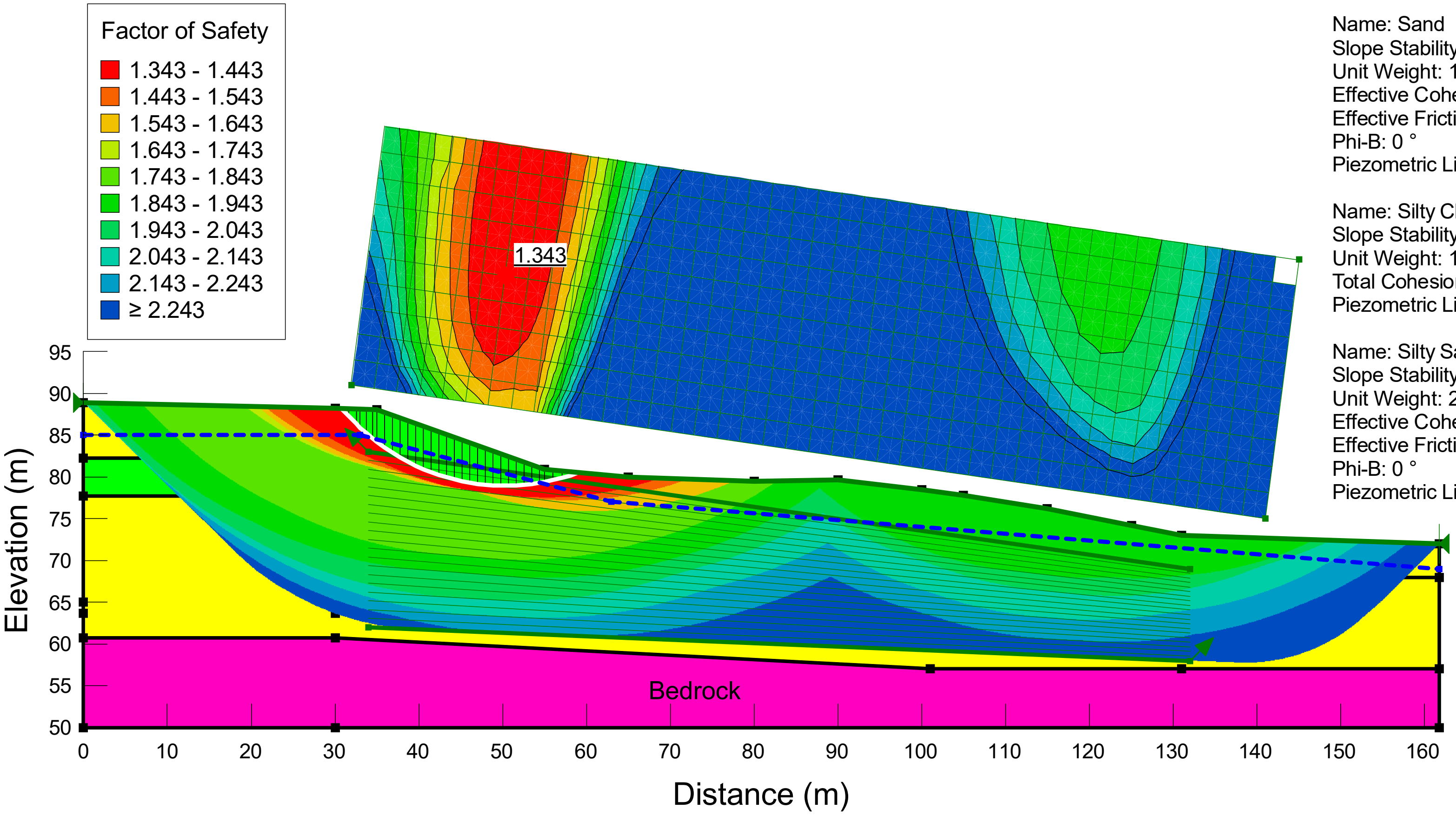
Name: Sand
Slope Stability Material Model: Mohr-Coulomb
Unit Weight: 19 kN/m³
Effective Cohesion: 0 kPa
Effective Friction Angle: 31 °
Phi-B: 0 °
Piezometric Line: 1

Name: Silty Clay (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

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

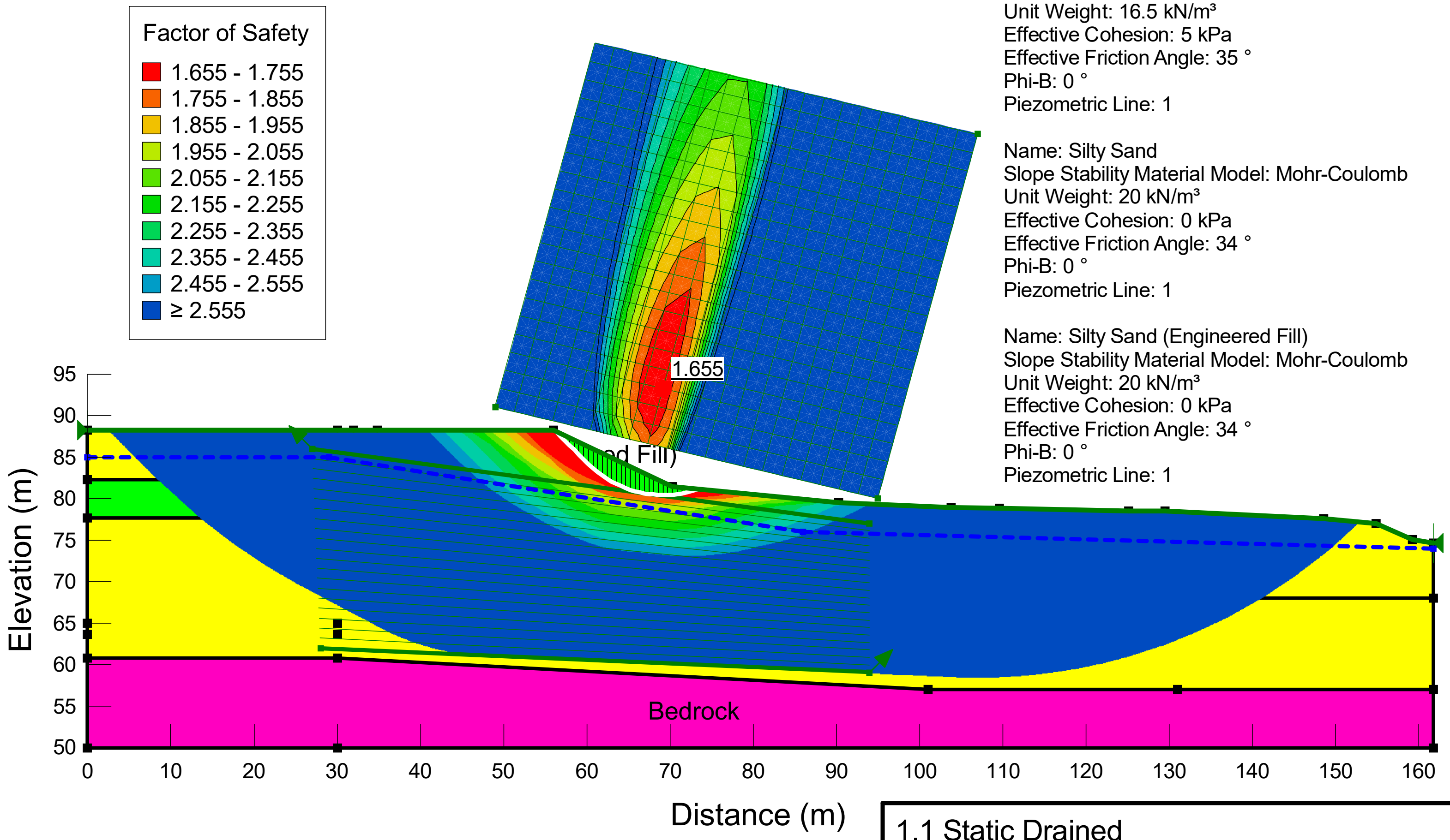
Name: Bedrock
Slope Stability Material Model: Bedrock (Impenetrable)
Piezometric Line: 1

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

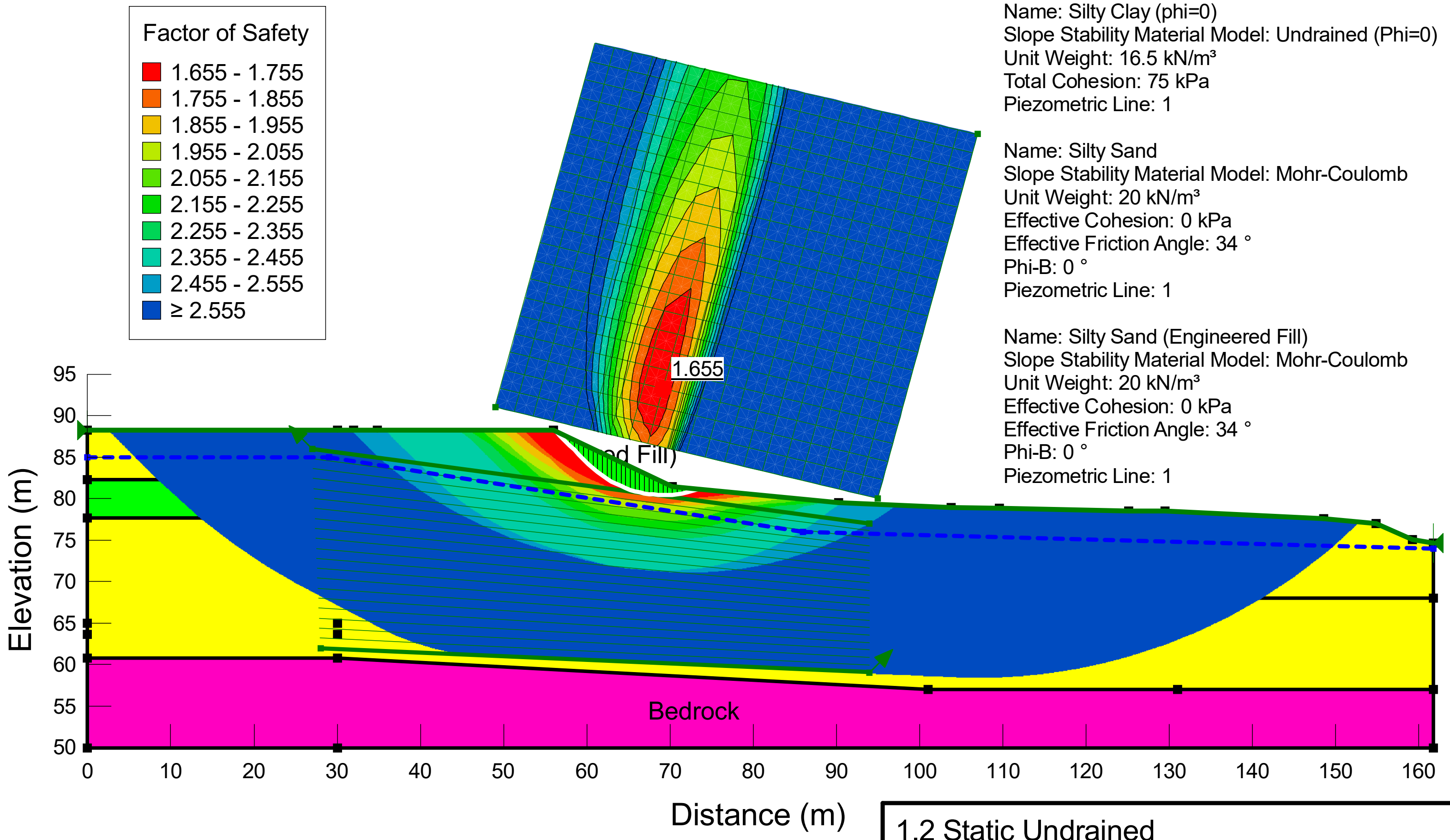
Name: Bedrock
Slope Stability Material Model: Bedrock (Impenetrable)
Piezometric Line: 1

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



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

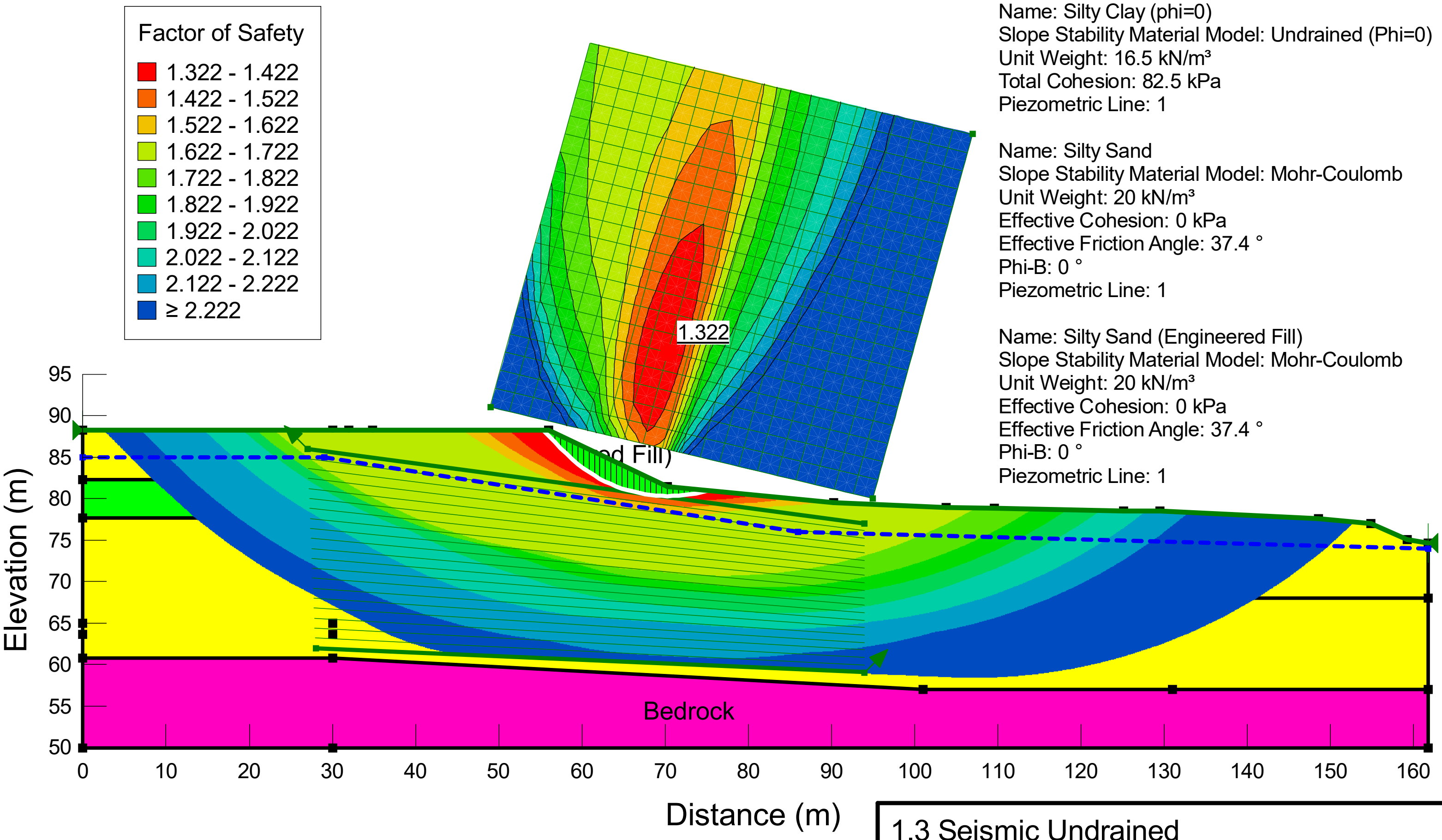
Name: Bedrock
Slope Stability Material Model: Bedrock (Impenetrable)
Piezometric Line: 1

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

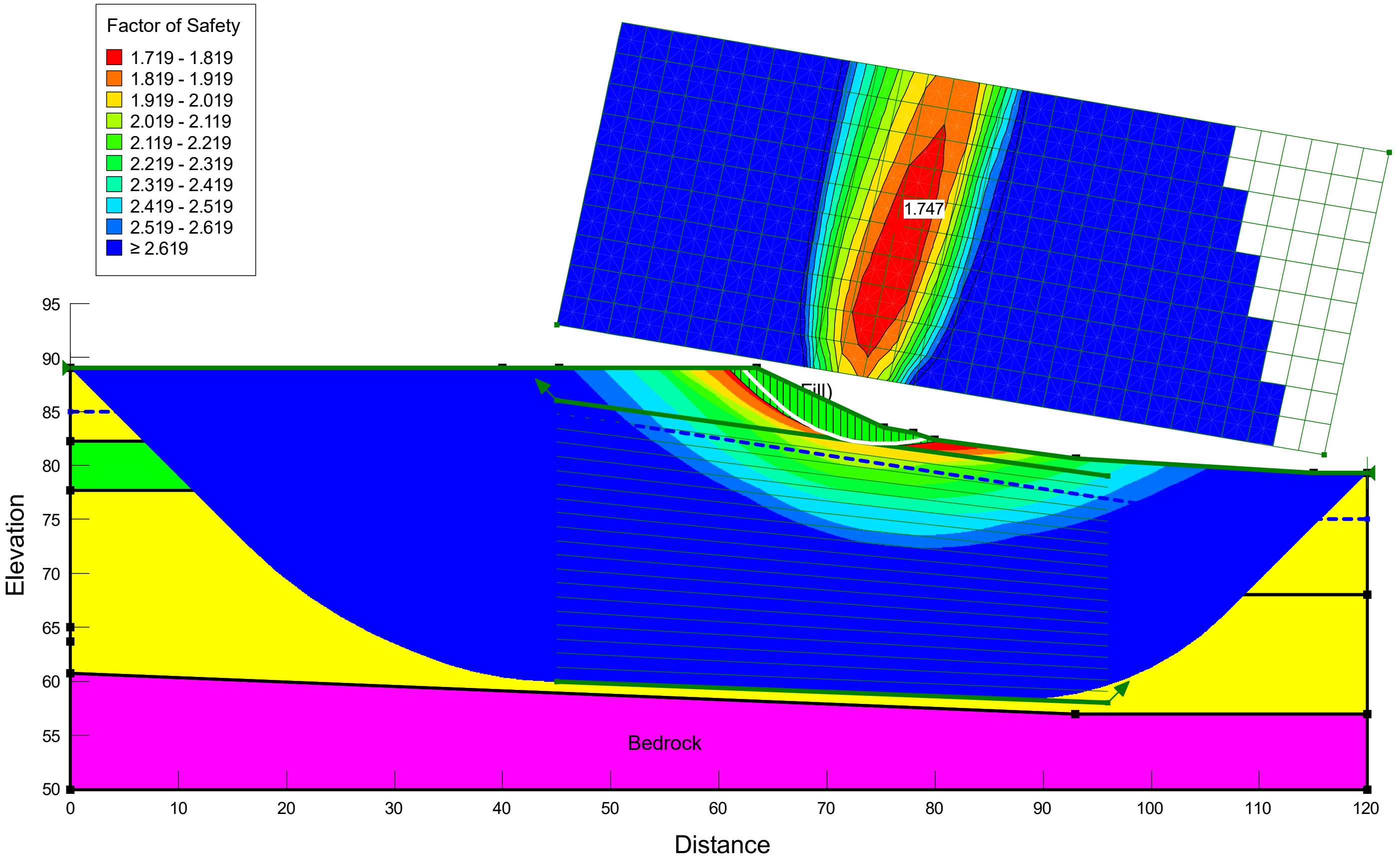
Name: Silty Sand
Slope Stability Material Model: Mohr-Coulomb
Unit Weight: 20 kN/m³
Effective Cohesion: 0 kPa
Effective Friction Angle: 37.4 °
Phi-B: 0 °
Piezometric Line: 1

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

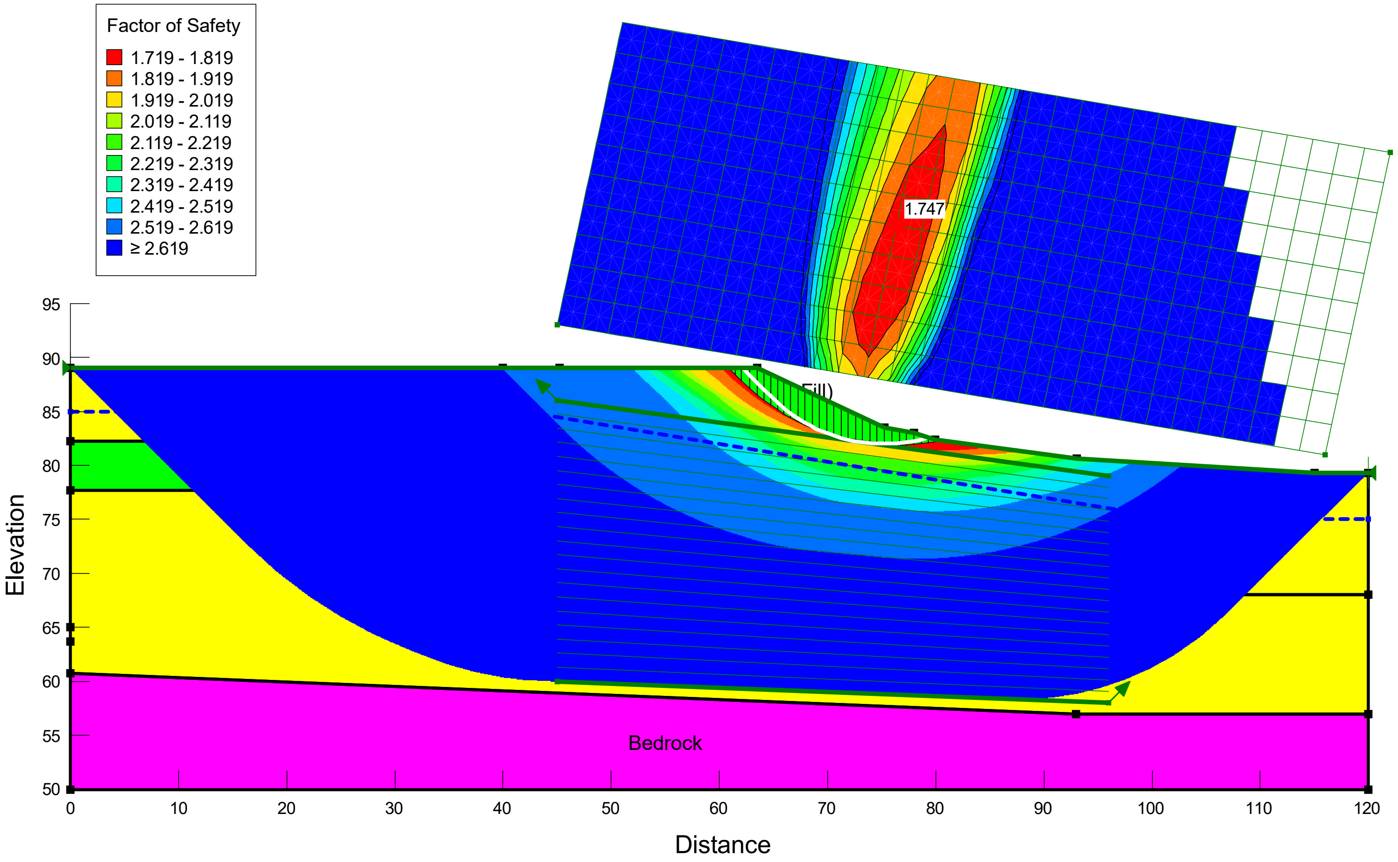
Name: Silty Sand (Engineered Fill)
Slope Stability Material Model: Mohr-Coulomb
Unit Weight: 20 kN/m³
Effective Cohesion: 0 kPa
Effective Friction Angle: 34 °
Phi-B: 0 °
Piezometric Line: 1

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

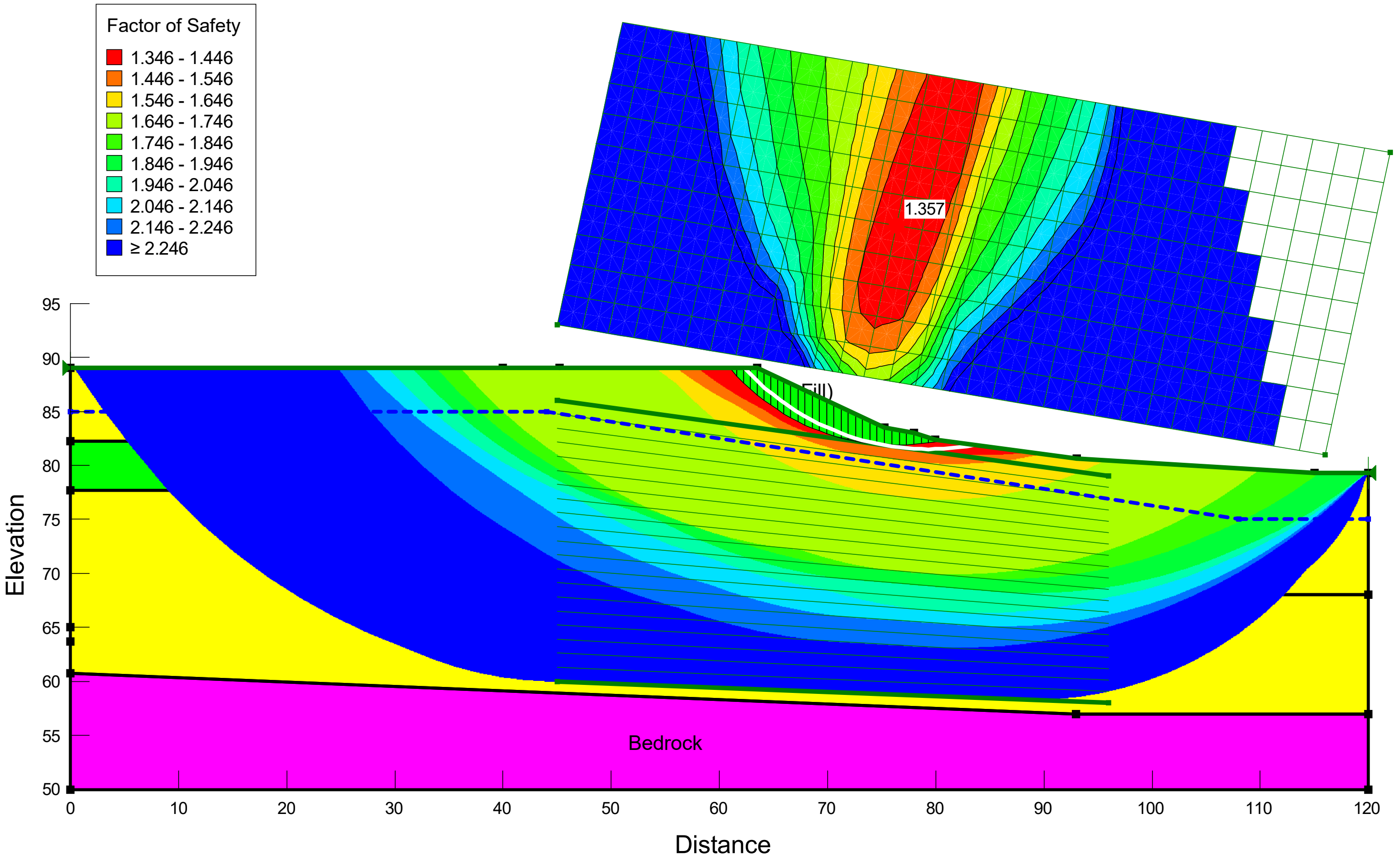
Name: Silty Sand (Engineered Fill)
Slope Stability Material Model: Mohr-Coulomb
Unit Weight: 20 kN/m³
Effective Cohesion: 0 kPa
Effective Friction Angle: 34 °
Phi-B: 0 °
Piezometric Line: 1

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

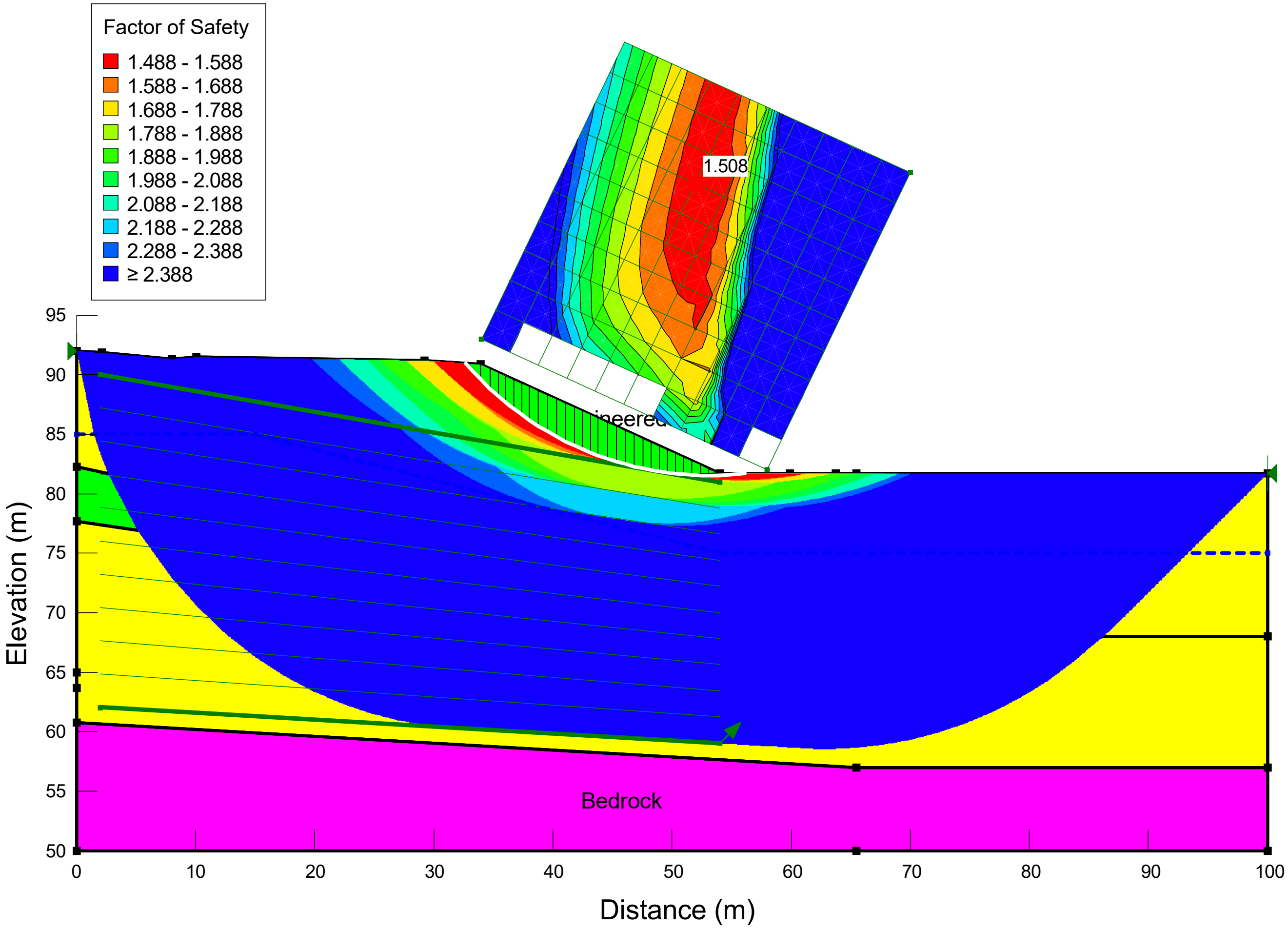
Name: Silty Sand (Engineered Fill)
Slope Stability Material Model: Mohr-Coulomb
Unit Weight: 20 kN/m³
Effective Cohesion: 0 kPa
Effective Friction Angle: 37.4 °
Phi-B: 0 °
Piezometric Line: 1

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

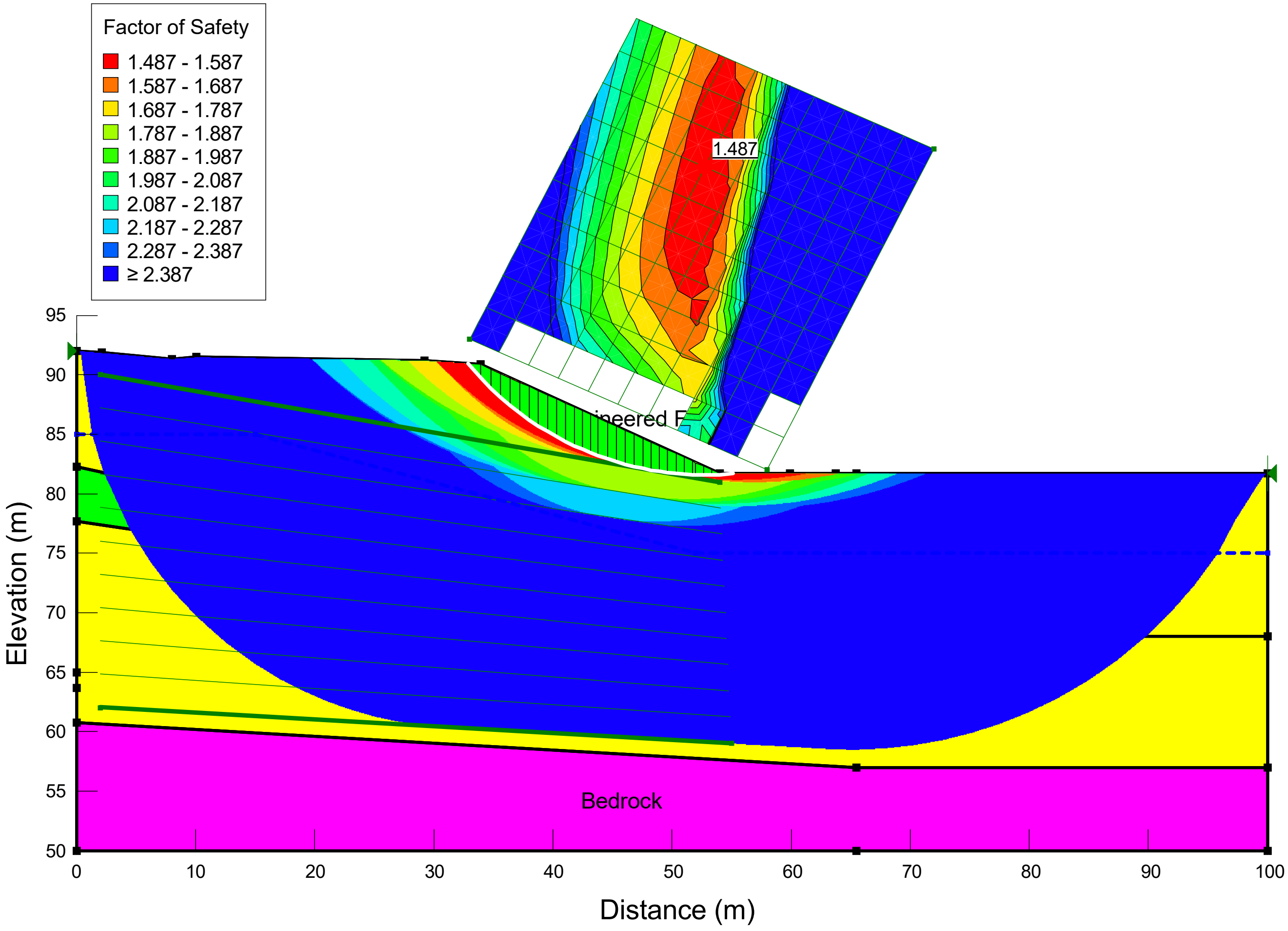
Name: Silty Clay
Slope Stability Material Model: Mohr-Coulomb
Unit Weight: 16.5 kN/m³
Effective Cohesion: 5 kPa
Effective Friction Angle: 35 °
Phi-B: 0 °
Piezometric Line: 1

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

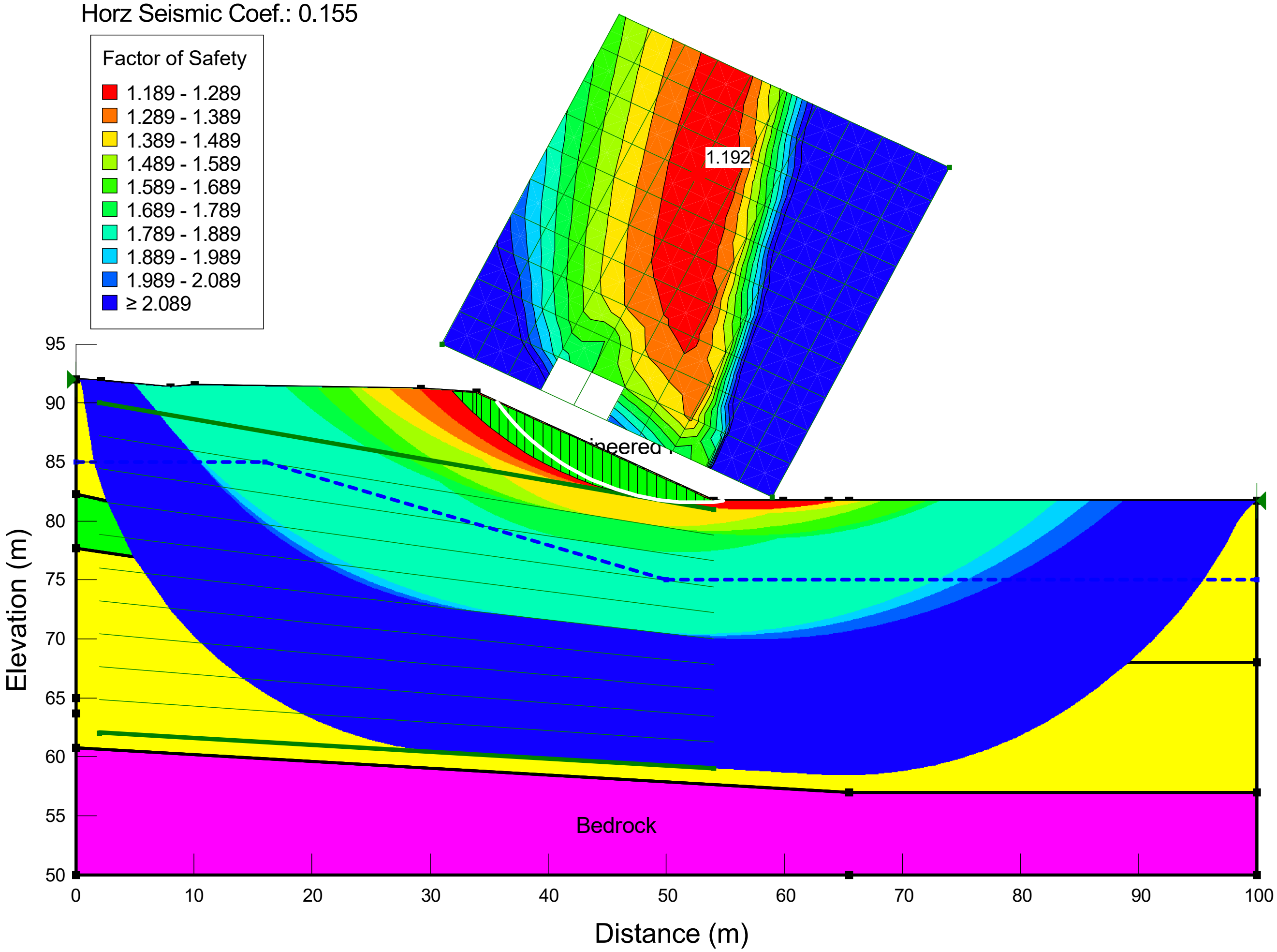
Name: Silty Sand (Engineered Fill)
Slope Stability Material Model: Mohr-Coulomb
Unit Weight: 20 kN/m³
Effective Cohesion: 0 kPa
Effective Friction Angle: 34 °
Phi-B: 0 °
Piezometric Line: 1

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 D

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

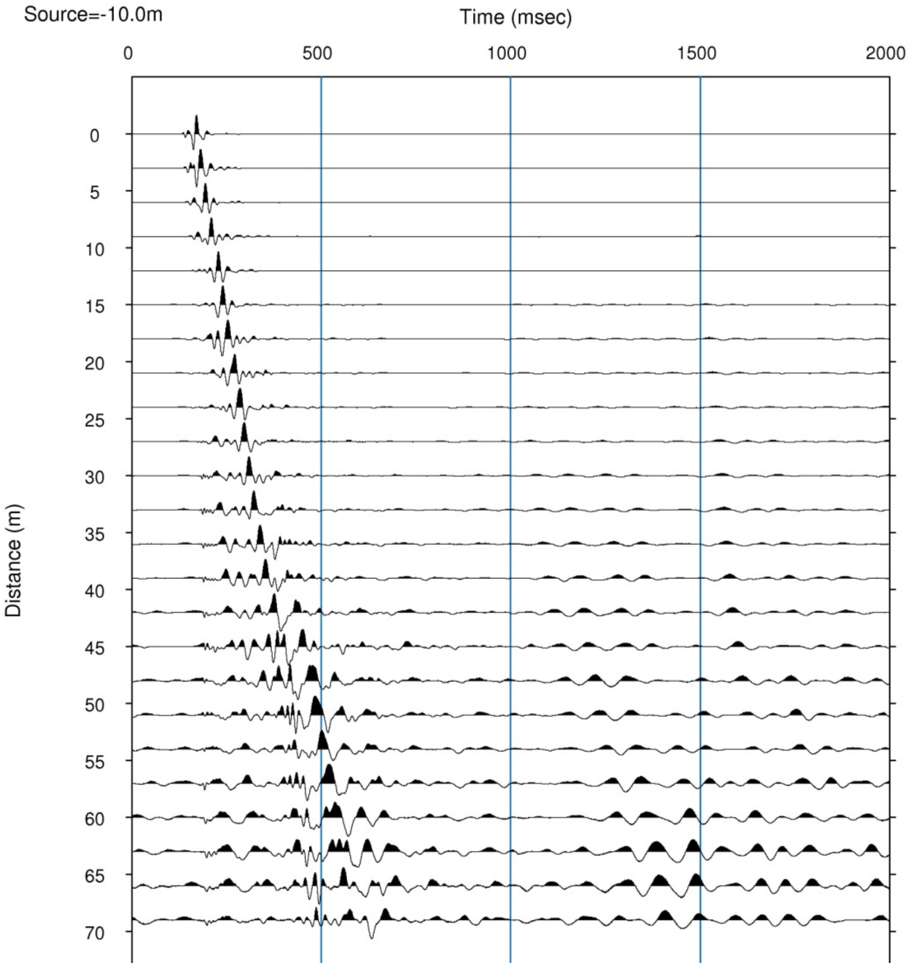


Figure 1: Typical seismic record collected along MASW Line1.

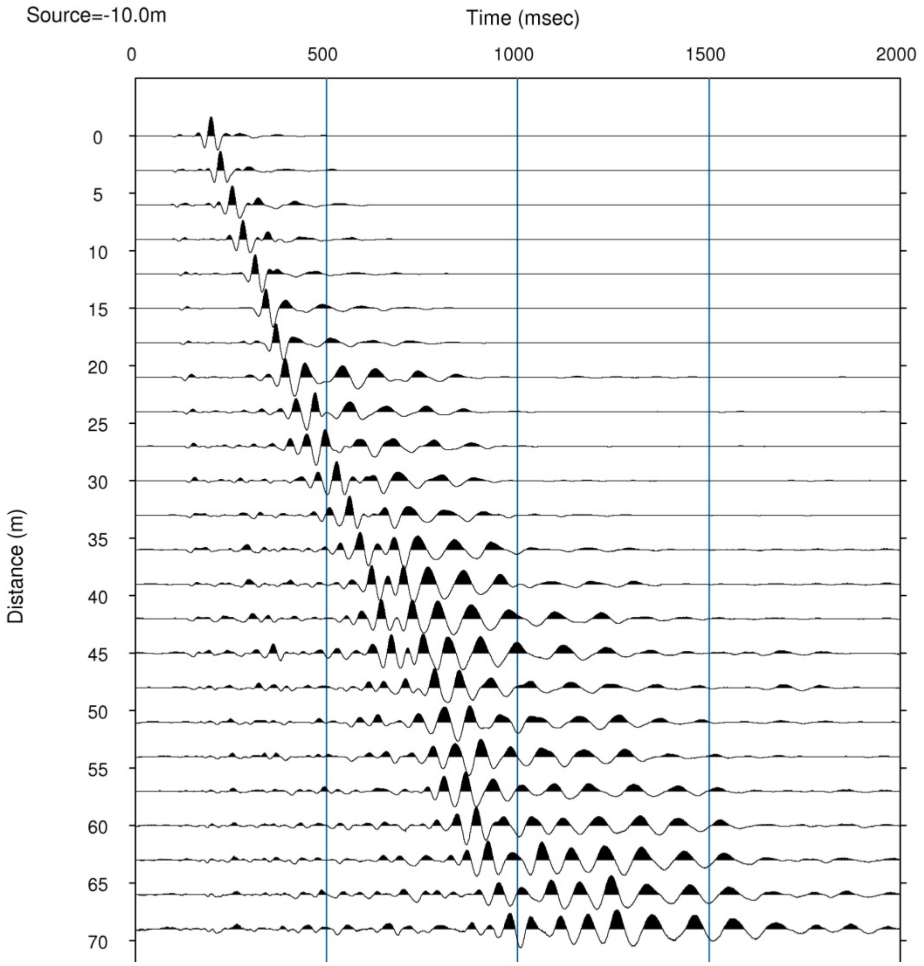


Figure 2: Typical seismic record collected along MASW Line 2.

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.

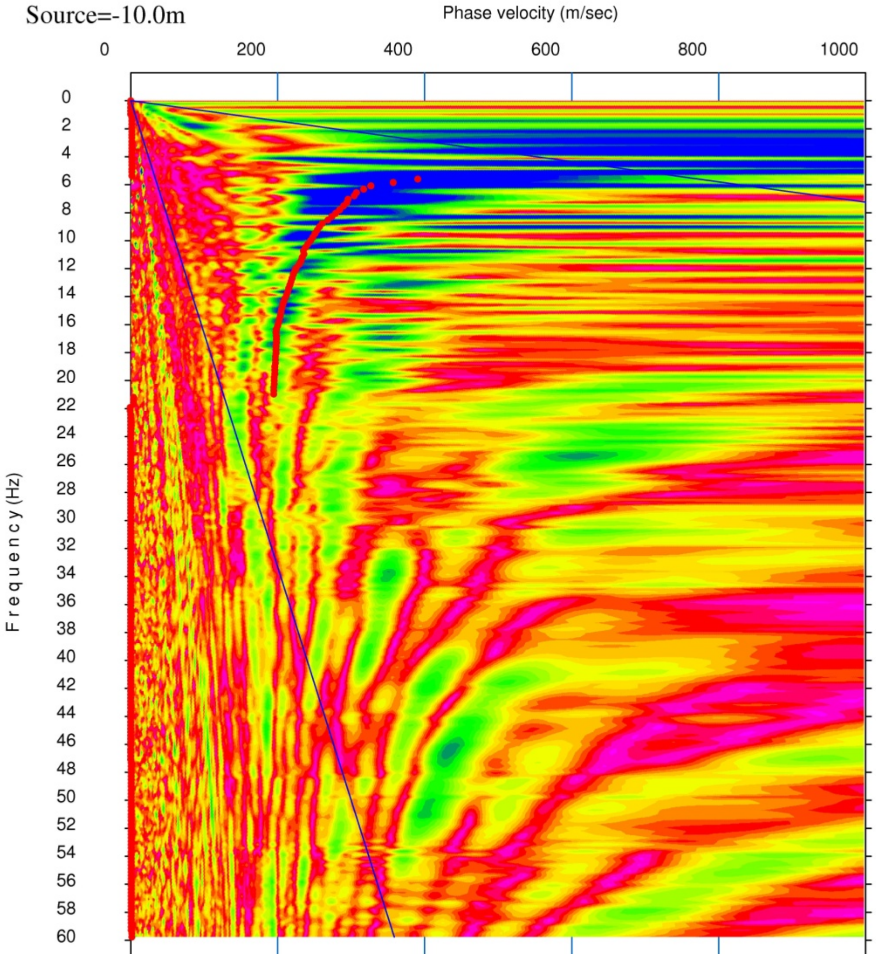


Figure 3: MASW Dispersion Curve Picks for Line 1(red dots).

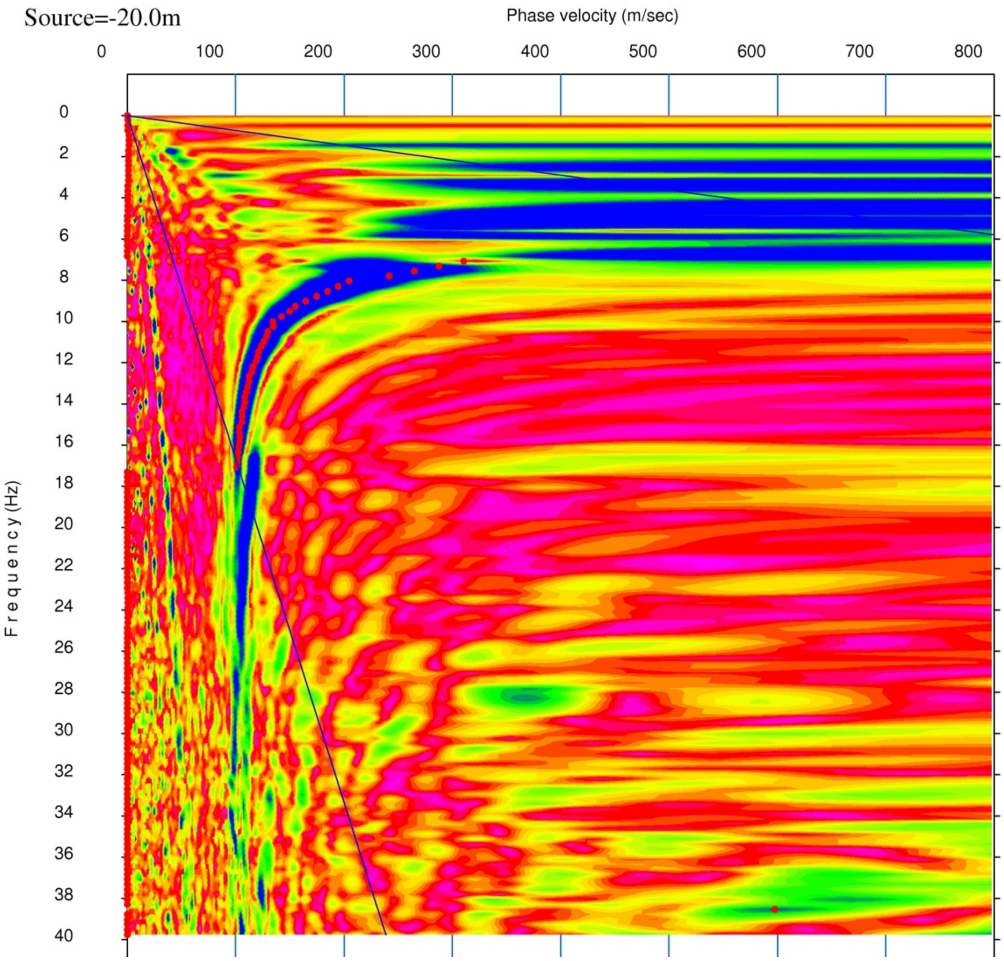


Figure 4: MASW Dispersion Curve Picks for Line 2(red dots).

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%.

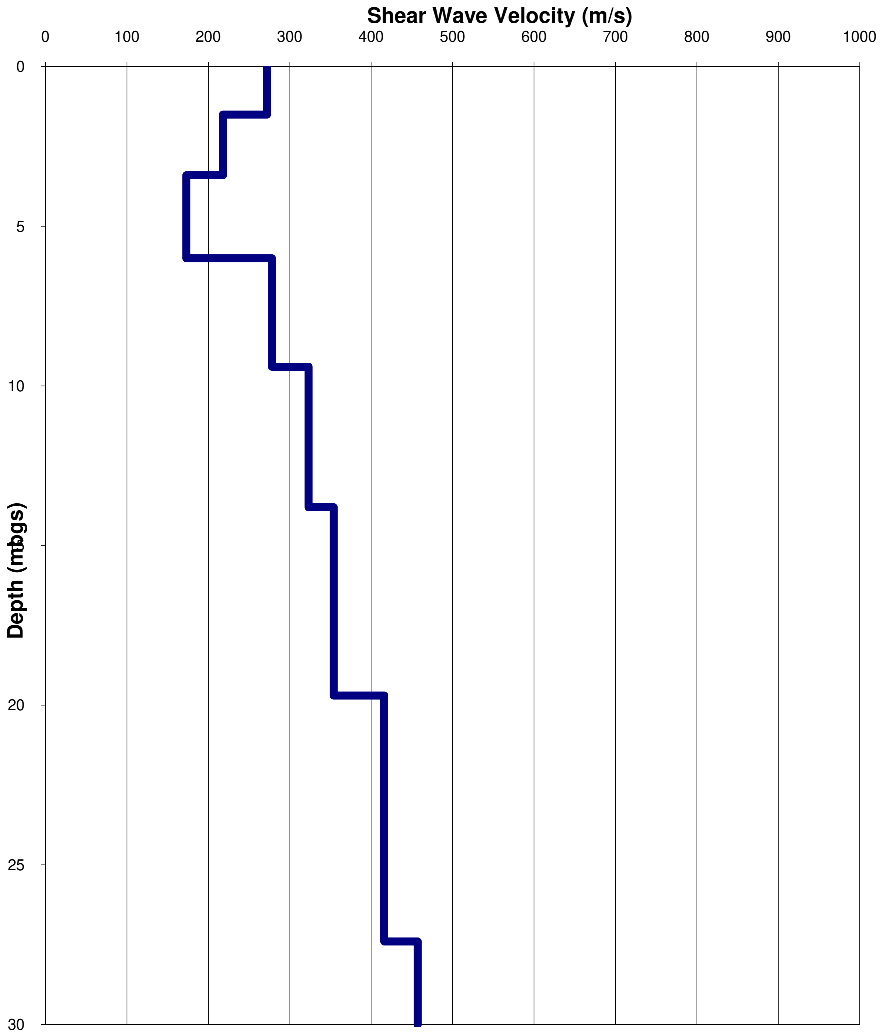


Figure 5: MASW Modelled Shear Wave Velocity Depth profile for MASW Line 1.

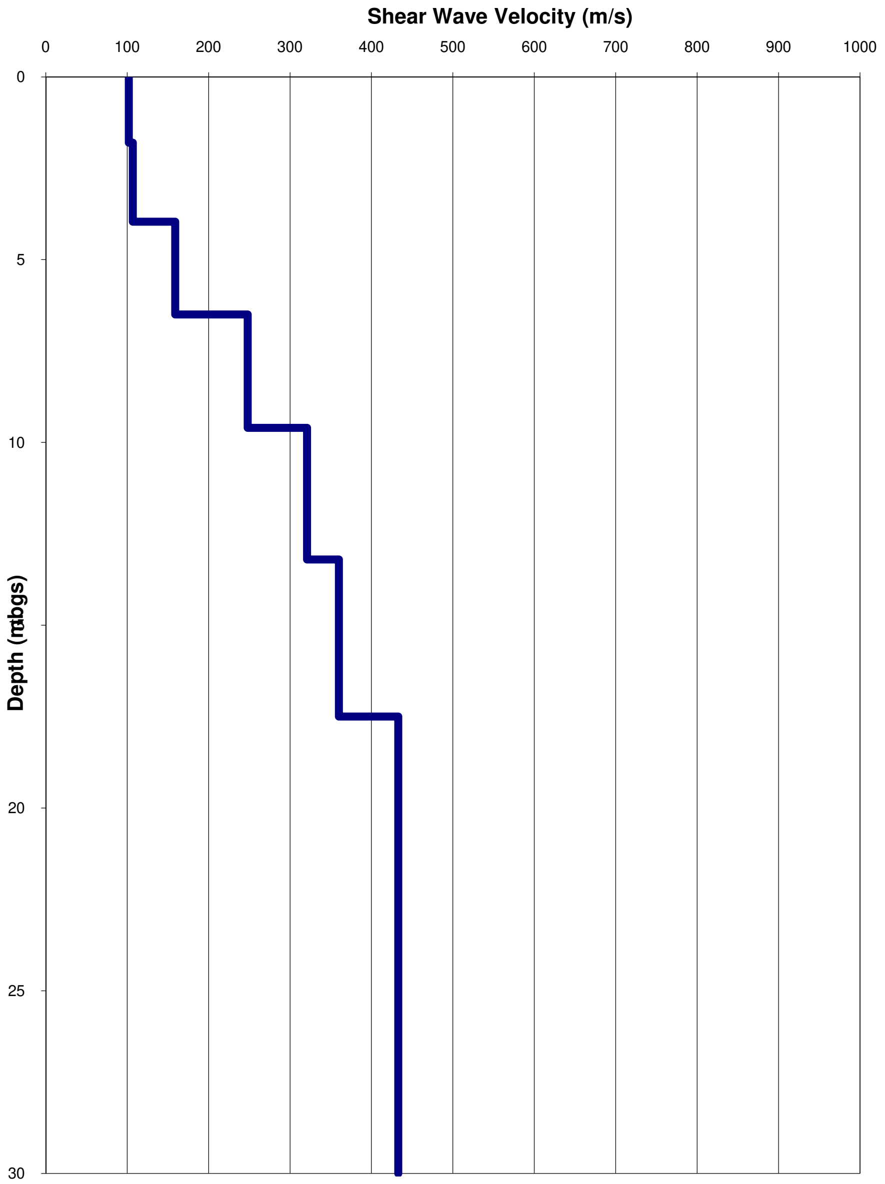


Figure 6: MASW Modelled Shear Wave Velocity Depth profile for MASW Line 2.

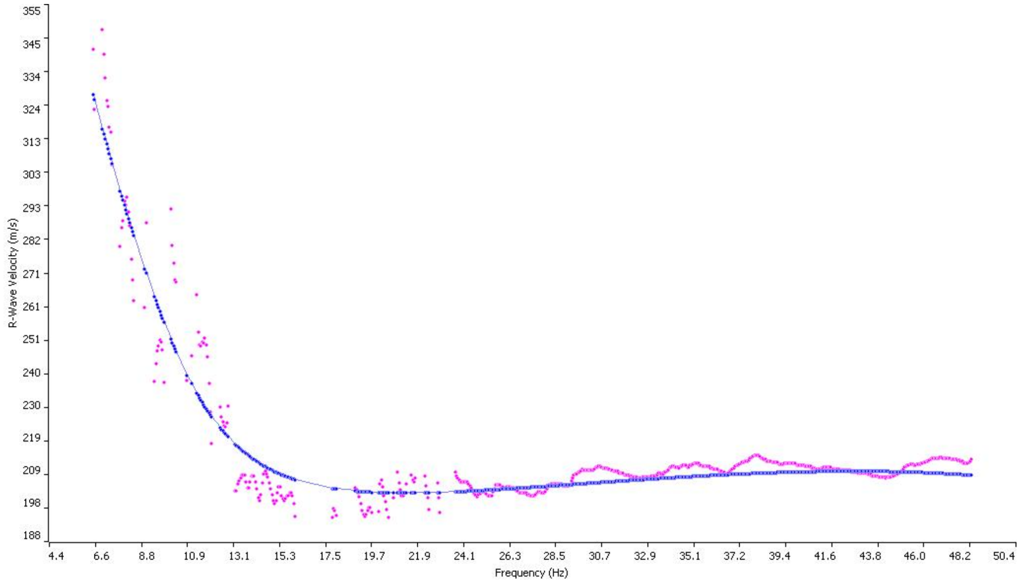


Figure 7: Comparison of Field (pink dots) vs. Modelled Data (blue dots) for the MASW Line 1.

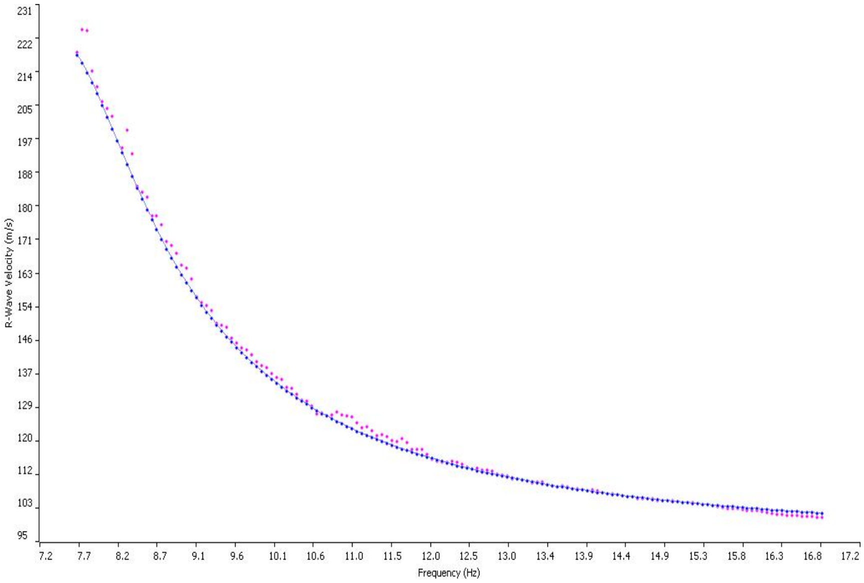


Figure 8: Comparison of Field (pink dots) vs. Modelled Data (blue dots) for the MASW Line 2.

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

PROFESSIONAL GEOSCIENTIST
CHRISTOPHER R. PHILLIPS
PRACTISING MEMBER
1503
ONTARIO

Stephane Sol, Ph.D.
Geophysics Group

Christopher Phillips, M.Sc.
Senior Geophysicist, Associate

SS/CRP/wlm

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APPENDIX E

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

				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
Group	**Analyte**	**MRL**	**Units**	**Guideline**			
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

146 Colonnade Rd. Unit 8, Ottawa, ON K2E 7Y1

GOLDER

golder.com