FINAL REPORT

Preliminary Foundation Report
IEA for Teston Road Area Between Highway 400 and Bathurst Street, Regional Municipality of York, Ontario

Submitted to:

Morrison Hershfield
125 Commerce Valley Drive West
Suite 300
Thornhill, Ontario
L3T 7W4

Submitted by:

WSP Canada Inc.
100 Scotia Court
Whitby, Ontario
L1N 8Y6

+1 905 723 2727

21496759

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Distribution List

1 e-Copy - Morrison Hershfield

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1 INTRODUCTION

WSP Canada Inc. (WSP), formerly Golder Associates Ltd., is pleased to submit this preliminary foundation report to Morrison Hershfield (MH) as part of the pavement and foundation engineering services for the Individual Environmental Assessment (IEA) for Teston Road Area project, located between Highway 400 and Bathurst Street in York Region, Ontario.

York Region (Region) is undertaking an IEA to address transportation problems and opportunities which will include the design of a new Teston Road alignment between Keele Street and Dufferin Street, in the City of Vaughan. At the time of this preliminary report preparation, MH has identified the previously named Alternative 4-E, as described in our desktop study entitled, “Pavement and Foundation Desktop Review, IEA for Teston Road Area Between Highway 400 and Bathurst Street, York Region, Ontario”, by Golder Associated Ltd., dated November 30, 2021, as the preferred alignment for the new Teston Road alignment to be further evaluated by the various project disciplines. The approximate alignment of the proposed new Teston Road is shown on Figures 1 and 2.

This draft geotechnical report should be read in conjunction with the “Important Information and Limitations of this Report” in Appendix A. The reader’s attention is specifically drawn to this information, as it is essential for the proper use and interpretation of this report. This report is not intended for use by any other party and WSP accepts no responsibility or liability for damages resulting from decisions or actions made by any other party/parties based on the contents of this report.

2 SITE AND PROJECT DESCRIPTION

An existing roadway easement extends from the current eastern limit of Teston Road (about 300 m east of Keele Street) in an east-west direction through to Dufferin Street. The project site is defined as the area between Keele Street and Bathurst Street, along the existing Teston Road easement, as shown on Figures 1 and 2.

Currently, there is an access road in the vicinity of the easement which is used by the surrounding landfills. The Disposal Services Landfill is located north of the existing portion of Teston Road. The Vaughan Landfill is located north of the existing easement and the City of Toronto Keele Valley Landfill is located south of the existing easement. It is understood that all three landfills are closed.

The Avondale Access Road runs in an approximately north-south direction along the eastern boundary of the Vaughan Landfill.

The topography at the site varies greatly. The ground surface at the existing Teston Road easement, between the landfills, slopes downward from the south to the north, ranging from Elevation 330 m to 276 m. The Avondale Access Road slopes from the south to the north, ranging from about Elevation 290 m (at intersection with the existing Teston Road easement) to Elevation 265 m (about 250 m north of the intersection with the existing Teston Road easement).

The East Don River tributary is located east of the landfill sites, within a deep valley. The west and east valley slopes range from about Elevation 290 m (at the Avondale Access Road) and Elevation 276 m (at Dufferin Street), respectively, to about Elevation 253 m at the river level.

A bridge is proposed to carry Teston Road across the East Don River tributary, at approximately Sta. 03+040, with approach embankments up to 12.0 m and 13.5 m at the west approach and east approaches, respectively. The existing culvert crossing Teston Road west of Saul Court, at approximately Sta. 04+600, is proposed to be replaced / extended during the widening of Teston Road.

3 INVESTIGATION PROCEDURES

The geotechnical field work for this investigation was carried out between October 7 and 24, 2022, during which time five boreholes (i.e. Boreholes 22-1 to 22-4 and C1) were advanced at the approximate locations shown on the Borehole Location Plans (Figures 1 and 2). The boreholes were drilled using either a conventional track-mounted drill rig (Boreholes 22-3 and 22-4) with hollow stem augers or portable drilling equipment (Boreholes 22-1, 22-2 and C1) with wash boring techniques, operated by specialist drilling contractors, subcontracted to WSP. Soil samples were generally obtained from the boreholes at 0.75 m and 1.5 m intervals of depth using a 50 mm outer diameter split-spoon sampler driven by an automatic hammer or a rope and cathead operated donut hammer in accordance with American Society for Testing and Materials (ASTM) procedure D1586 18. The split-spoon samplers used in the investigation limit the maximum particle size that can be sampled and tested to about 35 mm. Therefore, particles or objects that may exist within the subsurface soils that are larger than this dimension are not sampled or represented in the grain size distributions. The results of the in situ field tests, the Standard Penetration Test (SPT) ”N”-values, as presented on the Record of Boreholes in Appendix B and in Section 5 of this report are unfactored.

The shallow groundwater conditions were noted in the boreholes during drilling, except in Borehole 22-1,22 2 and C 1 where water was introduced for wash boring. Three 50 mm diameter monitoring wells were installed in Boreholes 22 2, 22-3 and C1 to further monitor the groundwater level. The remaining boreholes were backfilled upon completion of drilling in accordance with Ontario Regulation 903 (as amended).

The field work for this investigation was monitored by a member of our engineering staff who determined the approximate borehole locations in the field, cleared the borehole locations of underground services, logged the boreholes and took custody of the recovered samples. All the soil samples obtained during this investigation were brought to our Whitby or Mississauga laboratories for further examination and selective classification testing (natural water content testing in accordance with ASTM D2216-10, grain size distribution analyses in accordance with Ministry of Transportation, Ontario (MTO) Laboratory Standard (LS) LS-702, and Atterberg limits testing in accordance with ASTM D4318).

The approximate locations of the boreholes were determined in the field relative to existing site features, and as such should be considered approximate.

4 REGIONAL GEOLOGY

According to surficial geology mapping of (Surficial Geology of Southern Ontario, Geological Survey and Ministry of North Development), the surficial geology the site generally consists of ice-contact stratified deposits boarding on glaciolacustrine-derived silty to clayey till deposits. Further the tributary of the East Don River is classified as a Provincially Significant Wetland.

According to physiography mapping (Physiography of Southern Ontario, Third Edition, by Chapman and Putnam 1984) the site is primarily within the physiographic region known as the Oak Ridges Moraine, boarding on the South Slope physiographic region. The Oak Ridges Moraine physiographic region is mapped as a kame moraine. The surface generally consists of glaciofluvial sands and gravels. The sandy soil is commonly under pressure. In the site area, the surface sands and gravels are commonly underlain by an extensive lacustrine clay and silt deposit. The South Slope physiographic region generally consists of a surficial till sheet, which follows the surface topography. The till is typically unsorted consisting of a mixture of any or all of clay, silt, sand, gravel, cobble, and boulders.

According to bedrock mapping (Karst of Southern Ontario and Manitoulin Island, Ontario Geological Survey), the bedrock within the western portion of the site consists of shale of the Blue Mountain Formation and the bedrock within the eastern portion of the site consists of shale and limestone of the Georgian Bay Formation.

5 SUBSURFACE CONDITIONS

The subsurface soil and groundwater conditions encountered in the boreholes are shown in detail on the Record of Borehole sheets (i.e., borehole records) in Appendix B. “Method of Soil Classification, Abbreviations and Terms Used on Records of Boreholes and Test Pits” and “List of Symbols” sheets are also provided to assist in the interpretation of the Record of Boreholes. The geotechnical laboratory results are presented in Appendix C.

The boundaries between the strata on the borehole records have been inferred from drilling observations and non continuous sampling. Therefore, these boundaries typically represent transitions between soil types rather than exact planes of geological change. Further, the subsurface conditions will vary between and beyond the borehole locations and across the site and caution should be used when extrapolating subsurface conditions between the boreholes. A more detailed description of the subsurface conditions encountered in the sampled boreholes is provided in the following sections.

5.1 Proposed Bridge and High Fill Embankments

Boreholes 22-1 to 22-4 were advanced along the proposed Teston Road alignment within the East Don River tributary valley as shown on the Borehole Location Plan, Figure 1. In summary, the subsoil conditions encountered along the alignment generally consist of fill material underlain by silty sand to sandy silt. Localized deposits of glacial till were encountered on the east slopes of the valley. Groundwater levels were measured within monitoring wells installed along the alignment at depths of 0.9 m below ground surface (mbgs) and 3.6 mbgs. A more detailed description of the soil deposits encountered along the alignment is provided below.

5.1.1 Asphalt/Fill

Asphalt, 110 mm in thickness, was encountered at Borehole 22 3 at ground surface.

A fill layer, ranging in thickness from 0.6 m to 2.1 m, consisting of non cohesive silty sand or sandy gravel, was encountered at ground surface in Boreholes 22 1, 22-2 and 22 4 and beneath the asphalt in Borehole 22 3. Trace organics were observed within the fill in Boreholes 22 1 and 22 2.

The SPT “N”-values measured within the non cohesive fill ranges from 1 blow to 68 blows per 0.3 m of penetration, indicating a very loose to very dense state of compactness. Typically, the fill is compact to very dense. The in situ water content measured on two samples of the fill are 4% and 6%.

5.1.2 Silty Sand to Silt

An approximately 7.6 m to 13.7 m thick non-cohesive silty sand to silt and sand to sandy silt to silt deposit was encountered beneath the fill or beneath the glacial till in all boreholes advanced on site. The silty sand to silt deposit was encountered at depths ranging from 0.6 m to 7.1 mbgs and extends to depths ranging from 8.2 m to 15.9 mbgs. Boreholes 22 1, 22 3 and 22 4 were terminated within this deposit. Seams of gravel and sand, 0.7 m in thickness, were observed within the silty sand to silt in Borehole 22-2 between depths of 8.7 m and 10.4 mbgs. Borehole 22-2 was terminated within the gravel and sand seam.

The SPT “N”-values measured within the silty sand to silt range from 10 blows per 0.3 m of penetration to greater than 50 blows per 0.07 m of penetration, indicating a compact to very dense state of compactness.

Grain size distribution testing was carried out on nine samples of the silty sand to silt deposit and the results are presented on Figures C1-A and C1-B in Appendix C. Atterberg limit testing was completed on three samples of silty sand to silt. The results of one limits testing indicate the fines are non plastic and the results of the two other tests indicate plastic limits of approximately 19% and 20%, liquid limits of approximately 20% and 21% and plastic indices of approximately 1% and 2% indicating the fines have slight plasticity. The natural water content measured on samples of the silty sand to silt deposit ranges from about 4% to 23%.

5.1.3 Glacial Till

An approximately 5.6 m thick deposit of glacial till was encountered beneath the fill in Borehole 22 4. The glacial till ranged in composition from non cohesive gravelly sandy silt to cohesive sandy silty clay. The glacial till deposit was encountered at a depth of 1.5 mbgs and extends to a depth of 7.1 mbgs. Auger grinding was observed within the deposit in Borehole 22 4, indicating the presence of cobbles and / or boulders.

The SPT “N”-values measured within the non cohesive till are 19 blows and 27 blows per 0.3 m of penetration, indicating a compact state of compactness. The SPT “N”-values measured within the cohesive till range from 28 blows per 0.3 m of penetration to greater than 50 blows per 0.13 m of penetration, indicating a very stiff to hard consistency.

Grain size distribution testing was carried out on one sample of the glacial till deposit and the results are presented on Figure C3 in Appendix C. Atterberg limit testing was completed on one sample of the glacial till. The results indicate a plastic limit of about 15%, a liquid limit of about 17% and a plastic index about 2% indicating the fines have slight plasticity. The natural water content measured on samples of the till deposit ranges from about 11% to 18%.

5.1.4 Groundwater Conditions

Details of the groundwater conditions encountered in the boreholes during and on completion of drilling are shown on the Record of Borehole Sheets in Appendix B. The groundwater levels measured in the monitoring well installed in Boreholes 22 2 and 22 3 are summarized in the table below:

Borehole No.	Depth to Groundwater (m)	Measurement Date	Soil Conditions at Screen Interval
22-2	0.9	November 07, 2022	Silt
22-3	3.6	November 07, 2022	Silty Sand to Sandy Silt

The groundwater level at the site is expected to fluctuate seasonally in response to changes in precipitation and snow melt and is expected to be higher during the spring and periods of precipitation.

5.2 Subsurface Conditions at Proposed Culvert

A total of one borehole, identified as Borehole C1, was advanced at the northern inlet of the existing culvert located at approximately Sta. 04+600 as shown on the Borehole Location Plan, Figure 2. In summary, the subsoil conditions encountered at this location consist of fill underlain by silty sand. The groundwater levels was measured at ground surface. A more detailed description of the soil deposits encountered at the proposed culvert is provided below.

5.2.1 Fill

An approximately 1.4 m thick layer of fill, consisting of non cohesive silty sand with trace organics and containing wood fragments, was encountered at ground surface in Borehole C1.

The SPT “N”-values measured within the non cohesive fill are 6 blows and 26 blows per 0.3 m of penetration, indicating a loose to compact state of compactness.

5.2.2 Silty Sand

An approximately 5.7 m thick non-cohesive silty sand deposit was encountered beneath the fill in Borehole C1. The silty sand deposit was encountered at a depth of 1.4 mbgs and extends to a depth of 7.1 mbgs.

The SPT “N”-values measured within the silty sand range from 18 blows per 0.3 m of penetration to greater than 98 blows per 0.25 m of penetration, indicating a compact to very dense state of compactness.

Grain size distribution testing was carried out on two samples of the silty sand deposit and the results are presented on Figures C1-A and C1-B in Appendix C. The natural water content measured on samples of the silty sand deposit ranges from about 14% to 23%.

5.2.3 Groundwater Conditions

Details of the groundwater conditions encountered in the boreholes during and on completion of drilling are shown on the Record of Borehole Sheet in Appendix B. The groundwater levels measured in the monitoring well installed in Borehole C1 is summarized in the table below.

Borehole No.	Depth to Groundwater (m)	Measurement Date	Soil Conditions at Screen Interval
C1	0.0	November 07, 2022	Silty Sand

The groundwater level at the site is expected to fluctuate seasonally in response to changes in precipitation and snow melt and is expected to be higher during the spring and periods of precipitation.

6 DECOMMISSIONING OF MONITORING WELLS

Procedures for borehole and well drilling are legislated under Ontario Regulation (O. Reg.) 903 amended by O. Reg. 128/03 of the Ontario Water Resources Act. This regulation outlines responsibilities for the drilling/well contractor and the owners of boreholes/wells and stipulates that all boreholes/wells be properly decommissioned in accordance with the regulation. The contract documents should require the contractor to decommission the piezometers/wells in accordance with the regulation prior to construction. Dependent on the location and depth of the wells, decommissioning may be carried out at various times throughout the project. The wells installed on the alignment and/or within construction zones/easements will need to be decommissioned prior to construction. All wells located off the alignment may be left in place for further monitoring during construction provided that they are located at least 5 m from the construction works and will not be disrupted by construction activities. These monitoring wells should be decommissioned at the end of the post-construction monitoring period and provision should be made in future contract documents for this purpose.

7 PRELIMINARY GEOTECHNICAL DISCUSSION AND RECOMMENDATIONS

This section of the report provides preliminary geotechnical information based on our interpretation of the available borehole information and on our understanding of the project requirements and is subject to the limitations given following the text of this report.

The recommendations are based on interpretation of the factual data obtained from the boreholes advanced to date during the subsurface investigations. The interpretation, recommendations and discussions presented are intended to provide the designers with sufficient information for design.

The information in this portion of the report is provided for the guidance of the design engineers and professionals. Where comments are made on construction, they are provided only in order to highlight aspects of construction which could affect the design of the project, and for which special provisions or operational constraints may be required in the Contract Documents. Contractors bidding on or undertaking any work at the site should examine the factual results of the investigation, satisfy themselves as to the adequacy of the information for construction and make their own interpretation of the factual data as it affects their proposed construction techniques, schedule, equipment capabilities, costs, sequencing, safety and the like. If the project is modified in concept, location or elevation, WSP should be given the opportunity to confirm that the recommendations in this report are still valid.

7.1 Proposed Bridge and Approaches

It is understood that a 40 m single span bridge structure is proposed to be constructed approximately between Sta. 03+016 and Sta. 03+056, where the proposed Teston Road crosses the East Don River tributary and valley. The East Don River tributary flows from the northwest to the southeast. The proposed approach embankments require grade raises of about 12.0 m and 13.5 m for the west and east approaches, respectively. For the purposes of this report, it is assumed that the embankment will have side and front slopes of no steeper than 2 horizontal to 1 vertical (2H:1V). Due to this preliminary stage, details about the bridge (i.e., footing type, foundation elevation, footing location) are not available.

7.1.1 Embankment Fill Type

For embankments of this height and considering that the abutments will likely be perched within the embankments, it is recommended that the embankment fill consist of OPSS.PROV 1010 Granular B Type II or Granular “A” as any settlement of the embankment fill will occur during construction prior to abutment construction. The area of fill that should be constructed out of granular fill should extend 3 m beyond the limits of the abutment footprint and then outward and downward at 1 horizontal to 1 vertical (1H:1V). Beyond these limits, OPSS.PROV 1010 Select Subgrade Material or OPSS.PROV 212 Earth Borrow could be utilized, although embankment settlement outside the abutment areas could still be a concern if materials with a high percentage of fines are utilized. If Earth Fill is being considered, an evaluation of the potential source material, including its plasticity and water content, should be made to confirm the suitability in terms of settlement and stability. Details of embankment construction are discussed in Section 7.1.7.

7.1.2 Consequences and Site Understanding Classification

In accordance with Section 6.5 of the 2019 Canadian Highway Bridge Design Code (CHBDC, 2019) and its Commentary, the proposed bridge and approach embankments are expected to carry moderate traffic volumes and its performance may have potential impacts on other transportation corridors, hence having a “typical consequence level” associated with exceeding limits state design. In addition, given the typical project specific geotechnical/foundation investigation carried out within the project limits (as presented in Section 5.0 of this report), in comparison to the degree of site understanding in Section 6.5 of CHBDC, the level of confidence for design is considered to be a “low” degree of site and prediction model understanding. Accordingly, the appropriate corresponding Ultimate Limit States (ULS) and

Serviceability Limits States (SLS) consequence factor, Ψ of 1.0, and geotechnical resistance factor φgu of 0.45 and φgs of 0.7 (for shallow foundations), from Tables 6.1 and 6.2 of the CHBDC have been used for design. As the design progresses to the detail design stage, it may be possible to adjust these factors when more subsurface investigation and testing is carried out.

7.1.3 Frost Depth

Spread/strip footings should be provided with a minimum of 1.2 m of conventional soil cover for frost protection, in accordance with OPSD 3090.101 (Foundation Frost Penetration Depths for Southern Ontario), measured perpendicular to the face of the abutment slope or surface in front of the abutments to the edge of the underside of the footing.

If adequate soil cover cannot be provided for the footing, rigid Styrofoam insulation could be installed to compensate for the lack of soil cover and provide protection from frost penetration.

7.1.4 Preliminary Geotechnical Resistances

7.1.4.1 Strip Foundations

Strip footings, founded on compacted Granular “A” pad perched within the embankment constructed out of granular fill, are feasible for support of the bridge abutments.

The abutment founding depths presented in the table below represent founding depths based on frost depth relative to the lowest point of the proposed final grade on Teston Road. At and below this depth, both abutments will be founded on a minimum 3 m thick compacted Granular “A” pad. The compacted Granular “A” pad should extend not less than 1 m beyond the limits of the footing footprint and then outward and downward at a slope no steeper than 1H:1V.

Footings should be designed based on the factored ultimate geotechnical resistance and the factored serviceability geotechnical resistance (for 25 mm of settlement) outlined in the table below.

Foundation Element	Founding Stratum	Foundation Width (m)	Factored Ultimate Geotechnical Resistance (kPa)	Factored Serviceability Geotechnical Resistance (kPa)
West Abutment	3 m thick compacted Granular “A” pad over granular embankment fill over compact to very dense silty sand to silt	2.0	600	150
		3.0	750	200
		4.0	850	250
East Abutment	3 m thick compacted Granular “A” pad over granular embankment fill over compact to very dense silty sand to sandy silt	2.0	600	150
		3.0	750	200
		4.0	850	250

The factored ultimate and serviceability geotechnical resistances are dependent on the footing width and founding depth and as such, the geotechnical resistances should be reviewed once founding elevation and footing width are known. The factored ultimate geotechnical resistances provided are based on a load applied concentrically to the centreline/centroid of the footing, as shown on Figure 6.4 of the CHBDC. Where a load is applied eccentrically from the centreline/centroid of the footing, the pressure distribution at ULS and SLS and the eccentricity limit of the footing should be taken into consideration in accordance with Section 6.10.5 of the CHDBC (2019) and its Commentary.

The compacted Granular “A” pad may be susceptible to disturbance and degradation on exposure to water and construction traffic and therefore it is recommended that a concrete working slab be placed over the subgrade to protect the integrity of the foundation soils.

7.1.4.2 Driven H-Pile Foundations

Based on the proposed structure configuration and the subsurface conditions encountered at this site, H-pile foundations have been considered for support of the new abutments.

Driven steel H-piles are considered feasible for support of the new abutments, provided they extend within the dense to very dense gravel and sand at the west abutment and the compact to dense silty sand to sandy silt at the east abutment. Due to the preliminary nature of the report, details regarding the pile cap elevation, construction sequencing, etc., are not known. Downdrag has not been considered within this report and should not be an issue provided the piles are driven after the embankment is constructed. The geotechnical resistance and reaction values provided below need to be reviewed during detailed design.

For steel 310 x 110 H-piles driven to the dense to very dense gravel and sand at the west abutment and the compact to dense silty sand to sandy silt at the east abutment using piles lengths of 10 m, the factored axial resistance at Ultimate Limit States (ULS) and the geotechnical reaction at Serviceability Limit States (SLS) (for 25 mm of total settlement) may be taken as outlined in the table below.

Foundation Element	Pile Type and Size	Factored ULS Resistance (kN)	SLS Geotechnical Reaction (kN)
West/East Abutment	HP 310 x 110	550	550

Pile installation should be in accordance with OPSS.MUNI 903 (Deep Foundations). The pile termination or set criteria will be dependent on the pile driving hammer type, helmet, pile size and length of pile. The set criteria must therefore be established at the time of construction once the piling equipment is confirmed. The pile capacity should be verified in the field during the final stages of driving by the use of high strain dynamic testing (more commonly known as pile dynamic analyzer (PDA) testing) on a minimum of 10% of the piles at each foundation element.

Given the variability of the subsurface conditions at this site, it is recommended that any test results below the acceptance criteria be assessed by a foundation engineer in conjunction with the owner, including consideration of the measured results from PDA testing for nearby piles.

7.1.5 Resistance to Lateral Loads

7.1.5.1 Strip Footings

Resistance to lateral forces / sliding between the concrete footings and the subgrade should be calculated in accordance with Section 6.10.4 of the CHBDC. The coefficient of friction, tan ϕ, between the cast-in-place concrete footings, or concrete working slab, and the properly prepared subgrade is provided below.

Subgrade Material	Coefficient of Friction, $tan\ \varphi'$
Cast-in-place footing or working slab to compacted Granular “A” pad	0.62

7.1.5.2 Driven H-Piles

The design of piles subjected to lateral loads should take into account such factors as the batter of the pile (if any), the relative rigidity of the pile to the surrounding soil, the fixity condition at the head of the pile (i.e., at the pile cap level), the structural capacity of the pile to withstand bending moments, the soil resistance that can be mobilized, the tolerable lateral deflections at the head of the pile and group effects. For a longer, more flexible pile, the maximum yield moment of the pile may be reached prior to mobilization of the lateral geotechnical resistance. For design purposes, both the structural and geotechnical resistances should be evaluated to establish the governing case. Lateral loading could be resisted fully or partially using battered piles, where possible.

The resistance to lateral loading in front of a single pile may be calculated using subgrade reaction theory where the coefficient of horizontal subgrade reaction, k_h (kPa/m), is based on the equations presented below. However, the response of a pile to lateral loads is highly non linear and methods that assume linear behaviour (such as subgrade reaction theory) are only appropriate where the maximum deflections are less than about 1% of the pile width, where the loading is static (no cycling) and where the pile material is linear (CFEM, 2023). If one or more of these conditions are not satisfied, lateral pile analysis should be carried out using p-y curves.

For non-cohesive soils:

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

where: n_h = coefficient related to soil density (kPa/m)

z = depth below pile cap for semi-integral abutment and bottom of CSP for integral abutments (m), and,

B = width of pile (m)

The values of n_h (Terzaghi, 1955 and Reese, 1975) and s_u to be incorporated into the calculations of the coefficient of horizontal subgrade reaction (k_h) within the native overburden, to be used for the structural analysis of the piles at this site for the west and east abutments, respectively, are summarized below. The ranges in values reflect the variability in the subsurface conditions, the soil properties, the approximate nature of the analysis and the non linear nature of the soil behaviour (such that k_h is a function of deflection). In developing these recommendations, the design groundwater level has been taken at approximately 1 mbgs. Any portions of the pile length within undocumented fill or above the depth of frost penetration (1.2 m) should be excluded in the analysis of lateral capacity.

West Abutment

Soil Unit (Elevation m)	Location Relative to Groundwater	n_h (kN/m^2/m)	s_u (kPa)
Compact to very dense silt	Below Groundwater	16,000	N/A
Dense to very dense gravel and sand	Below Groundwater	22,800	N/A

East Abutment

Soil Unit (Elevation m)	Location Relative to Groundwater	n_h (kN/m^2/m)	s_u (kPa)
Compact to dense silty sand to sandy silt	Below Groundwater	16,000	N/A

Both the structural and geotechnical resistances of the piles should be evaluated to establish the governing case at ULS. At SLS, the horizontal reaction of the piles will be controlled by deflections and the horizontal resistance of the piles should be calculated based on the coefficient of horizontal subgrade reaction ( ) of the soil as discussed above. The SLS reaction should be taken as that corresponding to a horizontal deflection of 10 mm at the underside of the abutment wall for units supporting the abutments (CHBDC (2019) Commentary Section 6.11.2.2).

Group action for lateral loading should also be evaluated by reducing the coefficient of horizontal subgrade reaction either in the direction of loading or perpendicular to the direction of loading by relevant group pile efficiency factors as outlined in Section C6.11.3.4 of the Commentary to the CHBDC (2019).

7.1.6 Lateral Earth Pressures

The lateral earth pressures acting on the abutment walls, any associated wingwalls or culverts will depend on the type and method of placement of the backfill materials, the nature of the soils behind the backfill, the magnitude of surcharge including construction loadings, the freedom of lateral movement of the structure, and the drainage conditions behind the walls / structures.

The following recommendations are made concerning the design of the bridge. It should be noted that these design recommendations and parameters assume level backfill and ground surface behind the walls. Where there is sloping ground behind the walls, the coefficient of lateral earth pressure must be adjusted to account for the slope.

- Free draining granular material meeting the specifications of OPSS.MUNI 1010 (Aggregates) Granular A or Granular B Type II, should be used as backfill behind the walls. This material should be compacted in accordance with OPSS.MUNI 501 (Compacting). Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the granular backfill requirements with respect to sub drains and frost taper should be in accordance with OPSD 3101.150 (Walls, Abutment, Backfill, Minimum Granular Requirement), OPSD 3121.150 (Walls, Retaining, Backfill, Minimum Granular Requirement), and 3190.100 (Walls, Retaining and Abutment, Wall Drain).
- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the walls. Care must be taken during the compaction operation not to overstress the wall, with limitations required on heavy construction equipment and requirements for the use of hand operated compaction equipment per OPSS.MUNI 501 (Compacting). Other surcharge loadings should be accounted for in the design, as required.
- Granular fill (where utilized) should be placed in a zone with the width equal to at least 1.2 m behind the back of the culvert structure. The pressures are based on the proposed granular backfill (placed behind the culvert, wingwalls, etc.).

For the cases where the pressures are based on the proposed embankment fill materials, the parameters in the table below may be used assuming the use of approved earth fill for embankment construction or granular fill placed behind the wall:

Static Lateral Earth Pressures

Fill Type	Soil Unit Weight	Coefficients of Static Lateral Earth Pressure	
		At-Rest, K_o	Active, K_a
Engineered Fill (Silty Sand)	20 kN/m^3	0.50	0.33
SSM	20 kN/m^3	0.50	0.33
Granular A	22 kN/m^3	0.43	0.27
Granular B Type II	21 kN/m^3	0.43	0.27

If the wall supports and structures allow for lateral yielding, active earth pressures may be used in the foundation design of the structures. If the structures do not allow lateral yielding (i.e., restrained structure where the rotational or horizontal movement is not sufficient to mobilize an active earth pressure condition), at rest earth pressures (plus any compaction surcharge) should be assumed for foundation design of the structures.

7.1.7 Approach Embankments

As outlined in Section 7.1, the proposed Teston Road bridge requires placement of up to about 12 m and 13.5 m of fill to construct the west and east approach embankments, respectively, with embankment side slopes of 2H:1V. As discussed in Section 7.1, we recommend that the embankment be constructed out of granular fill below the perched abutments. Elsewhere, Select Subgrade Material or potentially Earth Fill could be utilized.

All existing unsuitable fill on the valley slope that is loose or contains organics must be stripped prior to embankment construction. The new compacted fill should be keyed into the existing slope by benching as per OPSD 208.010 (Benching of Earth Slopes).

The parameters of the non-cohesive silty sand to silt deposit and glacial till deposits employed for the settlement and stability analyses were determined based on empirical correlations with the field (SPT ‘N’ values) and laboratory index testing from the boreholes and tempered with engineering judgment from experience with similar soils.

7.1.7.1 Settlement

Settlement analyses were carried out to estimate the magnitude of expected settlement of the native soils under the high fill embankment as we well as the settlement of the fill itself, using the commercially available program Settle3 (Version 5.0), developed by Rocscience Inc. The analyses assume new fill in the abutment areas consists of Granular ”A” or Granular ‘B’ Type II.

In general, embankments approaching structural elements such as bridge abutments are to be designed such that total settlements and differential settlement do not exceed 25 mm, over a 20-year period following completion of construction.

Based on the results of the settlement analyses, the total settlement of the existing site soils under the loading imposed by the approach embankments, as well as settlement of the fill itself is presented below. The total settlement is expected to exceed 25 mm, however, as both the fill and native materials are non-cohesive and generally compact to dense, the settlement is expected to occur during construction and therefore settlement mitigation measures are not required.

Approach Embankment	Anticipated Grade Raise (m)	Estimated Settlement of Embankment Fill (Immediate)	Estimated Settlement of Subsoils (Immediate)	Estimated Settlement (Long-Term)
West Approach	12.0	~120 mm	100 to 150 mm	<25 mm
East Approach	13.5	~135 mm	100 to 150 mm	<25 mm

If Earth Fill is proposed to be used, an evaluation of the potential source material, including its plasticity and water content, should be made and additional settlement analyses should be carried out to assess the total and differential settlement along the approach embankments, particularly in any transition zone between where granular fill and earth fill are used.

7.1.7.2 Global Stability

Limit equilibrium global stability analyses have been carried out for conceptual RSS walls using the commercially available program Slide2 (Version 9.0), developed by Rocscience Inc., employing the Morgenstern-Price method of analysis. The Factors of Safety of numerous potential failure surfaces were computed to establish the minimum Factor of Safety. The Factor of Safety is defined as the ratio of the forces tending to resist failure to the driving forces tending to cause failure. The Factor of Safety is equal to the inverse of the product of the consequence factor, Ψ, and the geotechnical resistance factor, ϕ_{gu} (i.e., $FoS=1/(\Psi\ \phi_{gu})$.

The minimum Factors of Safety listed below have been established as the target for the design of the approach embankments at this site, as per Table 6.2 of CHBDC.

- 1.43 for temporary (undrained) conditions; and
- 1.67 for long-term (drained) conditions.

The stability analyses for the proposed approach embankments in long term (drained) conditions for the west and east approach are shown in Figures D1 and D2, respectively, in Appendix D. The deep-seated failure surfaces meet the target global Factor of Safety values noted above , provided that the ground surface at the bottom and top of the embankment is level. There are more surficial surfaces, between a FOS of 1.5 and 1.7, however when detailed investigation is undertaken the long-term factor of safety will be reduced from 1.67 to 1.5, as per Table 6.2 of the CHBDC, and therefore it is likely the global stability conditions will be met. The global stability will need to be re-evaluated once the final geometry of the site is known.

7.2 Proposed Culvert

It is understood that a precast concrete box culvert is proposed to be installed at approximately Sta. 04+600, where the existing culvert crosses Teston Road. It is not currently known if the existing culvert will be replaced or if culvert extensions will be utilized to accommodate the widening of Teston Road. The inlet of the existing culvert is located north of Teston Road and the outlet of is located south of Teston Road.

7.2.1 Founding Level

Borehole C1 was advanced to a depth of 7.1 mbgs at the culvert inlet. The native soils below the fill consist of compact to very dense silty sand. The groundwater level within the monitoring well was measured at ground surface. The native silty sand below 1.4 mbgs are considered suitable for support of the proposed box culvert. If the founding level for the proposed culvert is located above the native subgrade level, the existing fill (loose and containing organics and wood fragments), should be completely sub-excavated and replaced with properly compacted engineered fill to the founding level. The limits of the engineered fill will need to extend not less than 1 m beyond the limits of the culvert footprint and then outward and downward at a slope no steeper than 1H:1V.

7.2.2 Preliminary Geotechnical Resistances

Precast box closed bottom culverts placed on the properly prepared native subgrade or engineered fill as discussed above , should be designed based on a factored ultimate geotechnical resistance at ULS of 400 kPa and a factored serviceability geotechnical resistance at SLS (for 25 mm of settlement) of 100 kPa. These recommendations are based on assumed culvert widths up to 6 m and assume a minimum 0.3 m thick compacted granular pad below the concrete box.

Precast box open footing culverts placed on the properly prepared native subgrade or engineered fill with a minimum of 1.2 m of soil cover for frost protection, with footing widths up to 2 m, should be designed based on a factored ultimate geotechnical resistance at ULS of 350 kPa and a factored serviceability geotechnical resistance at SLS (for 25 mm of settlement) of 300 kPa. These recommendations are based on assumed minimum compacted granular pad thickness 0.3 m.

The factored ultimate geotechnical resistance at ULS and the factored serviceability geotechnical resistance at SLS for 25 mm of settlement are dependent on the culvert/footing widths and founding elevation and as such, the geotechnical resistances should be reviewed when culvert details are known. The factored ultimate geotechnical resistances are based on loading applied perpendicular to the top surface slab of the culvert. Where the load is not applied perpendicular to the top surface slab of the culvert, inclination of the load should be taken into account.

Culvert construction, including placement of bedding, cover and backfill should be placed in accordance with OPSD 803.010 and OPSS 422 (Precast Reinforced Concrete Box Culverts). Inspection and field density testing should be carried out by a qualified engineer during all engineered fill placement operations to ensure that appropriate materials are used, and that adequate levels of compaction have been achieved.

It is recommended that at least 300 mm of Granular “A” or Granular ’B’ Type II material be used for bedding purposes. In addition, a minimum 75 mm thick uncompacted levelling pad consisting of Granular “A” or concrete fine aggregate meeting the gradation requirements specified in OPSS.PROV 1002 (Aggregates – Concrete) should be provided as shown on OPSD 803.010 (Backfill and Cover for Concrete Culverts) for culvert construction. Backfill should be placed concurrently on both sides of the culvert walls, ensuring that the backfill depth on one side does not exceed the other side by more than 400 mm as per OPSS 422 (Precast Reinforced Concrete Box Culverts).

7.2.3 Settlement and Stability

The proposed concrete box culvert includes widening of the existing roadway embankment due to the addition of lanes, the details of which are not known. For the purposes of this report, it is assumed that the road way embankment will be widened up to about 3 m on each side, which will result in filling in above the existing side slopes/culvert extension of about 3 m.

Based on the subsurface conditions in Borehole C1, the magnitude of settlement resulting from filling up to 3 m is estimated to be less than 10 mm. This settlement will occur differentially longitudinally along the length of the culvert being highest at the locations of the existing culvert's inlet and outlet and tapering to a minimum at the locations of the proposed culvert's inlet and outlet. Due to the non-cohesive nature of the soils at site, this settlement is expected to occur immediately during fill placement. If the culvert is extended on each side as opposed to being replaced, then the widening will also result in settlement if the existing culvert. Once details of the proposed culvert and embankments are known, a more robust analysis of the settlement caused by widening can be completed, but in general, settlement is not expected to be a major issue.

For widened embankment side slopes up to 3 m in height constructed of engineered fill and formed at no steeper than 2H:1V, a Factor of Safety of greater than 1.5 is achieved for the long-term drained case. Undrained conditions were not considered at the location of the proposed culvert due to the non cohesive soils present. The results of the global stability analyses are shown on Figure D3 in Appendix D.

7.2.4 Resistance to Lateral Loads and Lateral Earth Pressures

Resistance to lateral forces/sliding resistance between the base of the pre-cast concrete box culvert and the levelling pad/bedding material should be based on the coefficient of friction (tan δ) provided below:

Coefficient of Friction

Founding Material	Coefficient of Friction (tan d)
Precast concrete footing on compacted Granular "A" pad	0.3

The recommendations for lateral earth pressures acting on the culvert walls are provided in Section 7.1.6.

7.2.5 Frost Protection

Closed bottom concrete box culverts are typically not provided with the standard depth for frost protection as they are tolerant to small magnitudes of movement related to freeze-thaw cycles should these occur. Rigid frame open box culvert footings require a minimum of 1.2 m of cover for frost protection.

However, consideration could be given to the use of rigid insulation placed under the box culvert along its entire length to prevent differential frost heaving. Insulation details can be provided if required. The concrete box culvert should, however, be founded below any existing fill and organic soils as discussed above.

8 CONSTRUCTION CONSIDERATIONS

8.1 Subgrade Preparation

Prior to construction of the embankments, all soils containing organics, as well as any loosened or soft deleterious materials should be stripped from within the embankment footprints.

8.2 Erosion Protection

To reduce erosion of the embankment side slopes (bridge and culvert) due to surface water runoff, placement of topsoil and seeding or pegged sod should be carried out on the embankment slopes as soon as practicable after construction of the embankments. In the short term, if placement of cover material cannot be carried out soon after the construction of the embankments, erosion control blankets should be installed to minimize erosion of the embankment slopes and to prevent surface runoff water from infiltrating into the backfill. The erosion protection should be in accordance with OPSS.MUNI 804 (Seed and Cover). Maintenance may be required over the first several years until the vegetative mat is fully established.

8.3 Scour Protection

Provision should be made for scour and erosion protection (suitable non-woven geotextiles and / or rip rap) at the culvert inlet / outlet locations and at the toe of the bridge embankment slopes adjacent to the East Don River tributary.

In order to prevent surface water from flowing beneath or around the culvert creating seepage through the embankment fill, and potentially causing erosion and loss of fine soil particles, a barrier such as a clay seal or concrete cut off headwall should be provided at the upstream and downstream end of the culvert. If a clay seal is adopted, the clay material should meet the requirements of OPSS.MUNI 1205 (Clay Seal), and the seal should be a minimum thickness of 1 m. The clay seal should extend from a depth of 1 m below the scour level to a minimum vertical height equivalent to the high water level. The seal should also extend from the open footing to a minimum horizontal distance of 2 m on either side of the culvert inlet opening. Alternatively, a 0.6 m thick clay blanket may be constructed (assuming a headwall is not constructed), extending upstream three times the structure height and along the adjacent slopes to a height of two times the structure height or the high water level, whichever is greater.

The requirements for and design of erosion protection measures (i.e., size, thickness, and extents) for the inlet and outlet of the culvert and at the toe of the bridge embankment slopes should be assessed by the hydraulics design engineer. As a minimum, rip rap treatment for the inlet / outlet of the culvert and toe of the bridge embankment slopes should be consistent with the standard Treatment Type A presented in OPSD 810.010 (Rip Rap Treatment).

8.4 Excavations

All excavations for the proposed Teston Road embankments, bridge and culvert including for stripping or footing construction should be carried out in accordance with O.Reg. 213 Ontario Occupational Health and Safety Act for Construction Projects (as amended). The site soils to be excavated can be classified as follows:

- Existing fill above/below the groundwater level and silty sand to silt below the groundwater level – Type 4; and
- Silty sand to silt deposits above the groundwater level, and glacial till above or below the water table – Type 3.

Temporary excavations (i.e., those open for a relatively short time period) should be made with side slopes no steeper than 3H:1V in Type 4 soils and no steeper than 1H:1V in Type 3 soils, assuming dewatering is provided, where required. However, depending upon the construction procedures adopted by the contractor, groundwater seepage conditions and weather conditions at the time of construction, some local flattening and/or blanketing of the slopes may be required, especially where localized seepage is encountered. Care should be taken to direct surface runoff away from the open excavations.

Considering that the embankment fill and native soils encountered immediately below the fill are most likely erodible, the following requirements must be taken into consideration to protect the workers:

- Exposed soils along the slope should be protected from surface erosion and from drying out by using waterproof tarps or plastic sheeting. If ravelling/sloughing of the slope face becomes an issue, the exposed cut should be treated with a heavy-duty soil binder / tackifier;
- Construction activities should be scheduled so that the length of time the unsupported temporary cut slopes are left open is reduced to the extent possible;
- Erosion control measures should be implemented as appropriate such that runoff from the site is reduced to the extent practical;
- Surface water should be diverted away from the excavation and from the top of the slope;
- The general condition of the slope should be inspected weekly by a qualified geotechnical engineer and the Contractor should perform daily inspections before the start of work and as needed throughout each shift; and,
- Excavated materials should not be stored above the crest of the slope, for a minimum distance of at least 3 m from the crest of the slope.

8.5 Construction Groundwater Control

Construction of the bridge footings, using open cut excavations, will take place within the engineered fill above the groundwater level and as such, dewatering is not expected to be required. However, dewatering will be necessary to be able to construct the culvert footings and/or carry out sub-excavation/replacement in dry conditions. Initially, higher inflow rates will occur as groundwater is removed from storage within the zone of influence. With time, rates will decrease toward a steady-state condition. Incident precipitation into excavations will also need to be managed with the groundwater contributions and factored into the total pumping rate estimates.

Surficial water seepage into the excavations should be expected and will be heavier during periods of sustained precipitation. Surface water runoff should be directed away from the excavations at all times.

For the culvert, depending on construction staging and final footing or sub-excavation depths, consideration should be given to diverting creek flow away from the excavation areas. Further, a temporary cut off wall may be required to prevent any dewatering measures from impacting the creek water levels.

9 CLOSURE

We trust that this draft report provides sufficient preliminary foundation information to proceed with the design of this project. If you have any questions, please do not hesitate to contact this office.

Signature Page

WSP Canada Inc.

LICENSED PROFESSIONAL ENGINEER
M Bentley
M. J. BENTLEY
100547109
01Aug2024
PROVINCE OF ONTARIO

Michael Bentley, P.Eng.
Geotechnical Engineer

Sarah E. M. Poot, P.Eng.
Principal Geotechnical Engineer

MJB/SEMP/kj

https://wsponline-my.sharepoint.com/personal/karen_jenkins_wsp_com/documents/desktop/teston road aoda reports_michael bentley/geotech/21496759_r-rev0_2024'08'01_foundation and pavement preliminary report.docx

LEGEND

●	APPROXIMATE BOREHOLE

REFERENCE(S)

1. DESING PLAN PROVIDED IN DIGITAL FORMAT BY MORRISON HERSHFIELD, FILE NO.Teston Design.dwg, RECEIVED.
2. BASE IMAGERY - © 2022 MICROSOFT CORPORATION © 2022 MAXAR ©CNES (2022) DISTRIBUTION AIRBUS DS.

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1:2,000 METRES

CLIENT
MORRISON HERSHFIELD

PROJECT
PRELIMINARY GEOTECHNICAL REPORT

TITLE
BRIDGE BOREHOLE LOCATION PLAN

CONSULTANT
wsp

YYYY-MM-DD	2023-01-06
DESIGNED	
PREPARED	SA
REVIEWED	MJB
APPROVED	SEMP

PROJECT NO.	PHASE	REV.	FIGURE
21496759		----	1

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LEGEND

●	APPROXIMATE BOREHOLE LOCATION

REFERENCE(S)

1. DESING PLAN PROVIDED IN DIGITAL FORMAT BY MORRISON HERSHFIELD, FILE NO.Teston Design N1.dwg, RECEIVED, JANUARY 17, 2023.
2. BASE IMAGERY - © 2022 MICROSOFT CORPORATION © 2022 MAXAR ©CNES (2022) DISTRIBUTION AIRBUS DS.

0 50 100
1:2,000 METRES

CLIENT
MORRISON HERSHFIELD

PROJECT
PRELIMINARY GEOTECHNICAL REPORT

TITLE
CULVERT BOREHOLE LOCATION PLAN

CONSULTANT
wsp

YYYY-MM-DD	2023-01-06
DESIGNED	
PREPARED	SA
REVIEWED	MJB
APPROVED	SEMP

PROJECT NO.	PHASE	REV.	FIGURE
21496759		----	2

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APPENDIX A

Important Information and Limitations of this Report

IMPORTANT INFORMATION AND LIMITATIONS OF THIS REPORT

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

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

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

The report is of a summary nature and is not intended to stand alone without reference to the instructions given to WSP by the Client, communications between WSP and the Client, and to any other reports prepared by WSP for the Client relative to the specific site described in the report. In order to properly understand the suggestions, recommendations and opinions expressed in this report, reference must be made to the whole of the report. WSP can not be responsible for use of portions of the report without reference to the entire report.

Unless otherwise stated, the suggestions, recommendations and opinions given in this report are intended only for the guidance of the Client in the design of the specific project. The extent and detail of investigations, including the number of test holes, necessary to determine all of the relevant conditions which may affect construction costs would normally be greater than has been carried out for design purposes. Contractors bidding on, or undertaking the work, should rely on their own investigations, as well as their own interpretations of the factual data presented in the report, as to how subsurface conditions may affect their work, including but not limited to proposed construction techniques, schedule, safety and equipment capabilities.

Soil, Rock and Ground Water Conditions: Classification and identification of soils, rocks, and geologic units have been based on commonly accepted methods employed in the practice of geotechnical engineering and related disciplines. Classification and identification of the type and condition of these materials or units involves judgment, and boundaries between different soil, rock or geologic types or units may be transitional rather than abrupt. Accordingly, WSP does not warrant or guarantee the exactness of the descriptions.

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

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

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

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

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

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

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

APPENDIX B

Record of Boreholes

PROJECT: 21496759
LOCATION: See Figure 1

RECORD OF BOREHOLE: 22-1

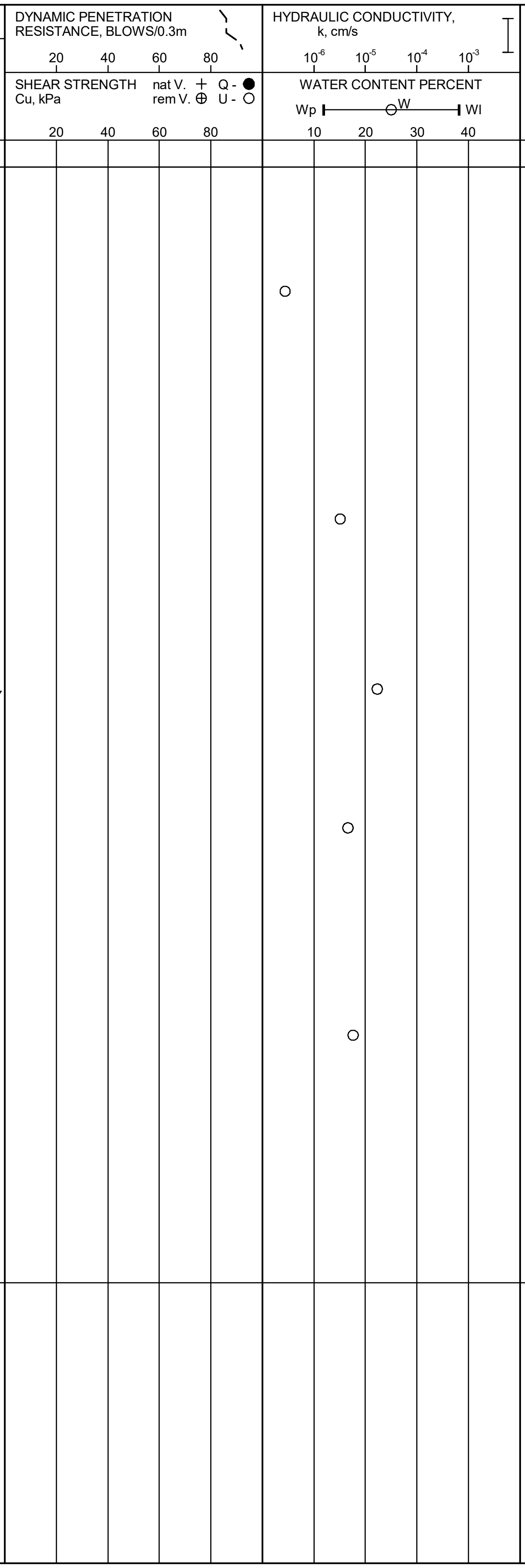
SHEET 1 OF 1

BORING DATE: October 11, 2022
DATUM: Geodetic

SPT/DCPT HAMMER: MASS, 64kg; DROP, 760mm
DRILL RIG: Portable Tripod
HAMMER TYPE: CATHEAD

BORING METHOD: Tripod, B-Casing

SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.3m	ADDITIONAL LAB. TESTING	GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
GROUND SURFACE						
FILL - (SM) SILTY SAND, trace organics; brown; non-cohesive, moist, compact	0.00	1	SS	10		
(SM/ML) SILTY SAND to SILT and SAND; brown; non-cohesive, moist, compact to very dense	0.61	2	SS	22	MH	0 67 30 3
		3	SS	61		
		4	SS	48		
		5	SS	68		
		6	SS	50/ 0.07		
		7	SS	59	MH	0 45 52 3
		8	SS	65		
		9	SS	73		
END OF BOREHOLE	8.23					



PIEZOMETER OR STANDPIPE INSTALLATION

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

DEPTH SCALE
1 : 50

LOGGED: KC
CHECKED: MJB

PROJECT: 21496759

RECORD OF BOREHOLE: 22-2

SHEET 1 OF 2

LOCATION: See Figure 1

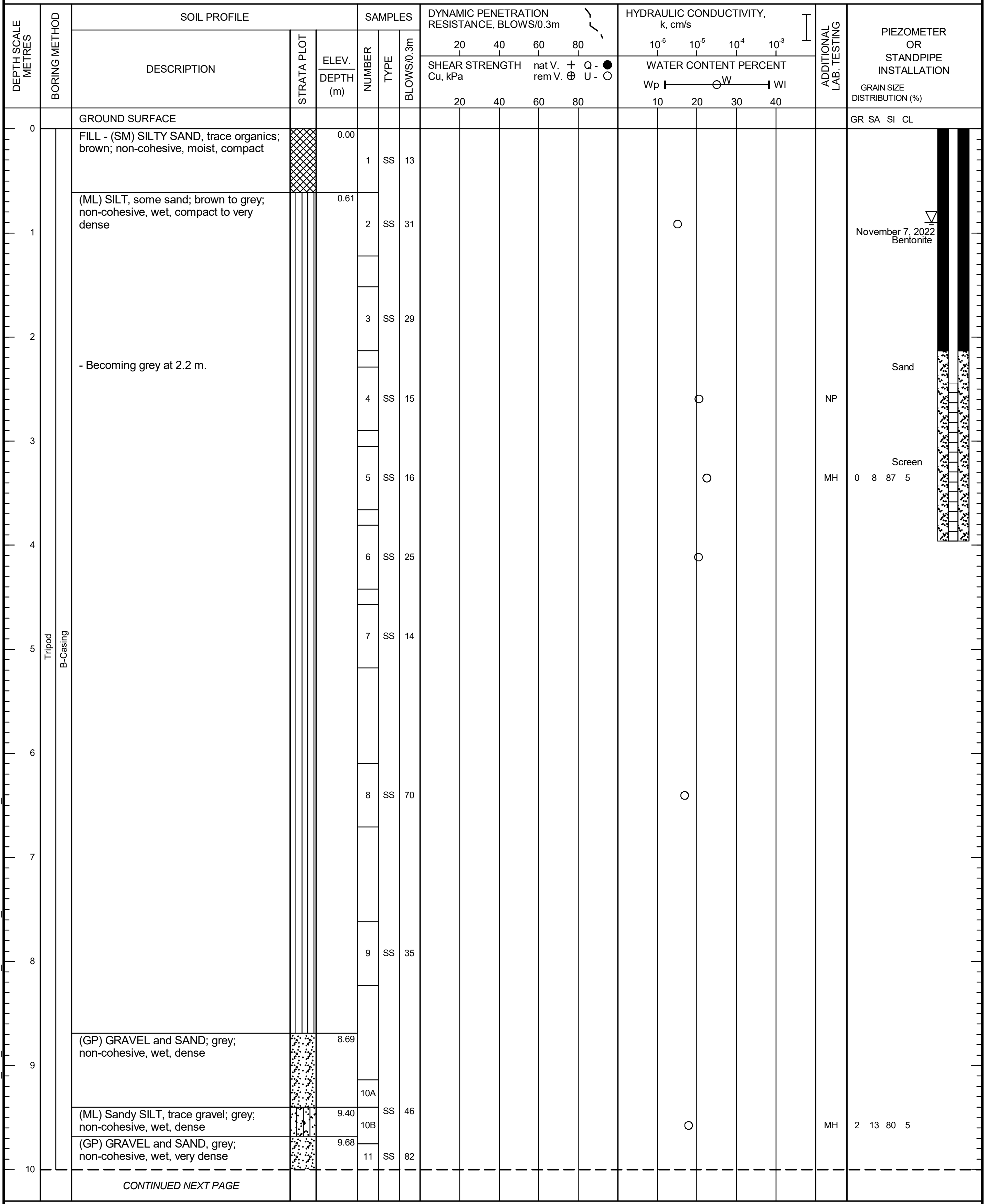
BORING DATE: October 11, 2022

DATUM: Geodetic

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

DRILL RIG: Portable Tripod

HAMMER TYPE: CATHEAD



GROUND SURFACE

DESCRIPTION	ELEV. DEPTH (m)
FILL - (SM) SILTY SAND, trace organics; brown; non-cohesive, moist, compact	0.00
(ML) SILT, some sand; brown to grey; non-cohesive, wet, compact to very dense - Becoming grey at 2.2 m.	0.61
(GP) GRAVEL and SAND; grey; non-cohesive, wet, dense	8.69
(ML) Sandy SILT, trace gravel; grey; non-cohesive, wet, dense	9.40
(GP) GRAVEL and SAND, grey; non-cohesive, wet, very dense	9.68

CONTINUED NEXT PAGE

NUMBER	TYPE	BLOWS/0.3m	ADDITIONAL LAB. TESTING	GR	SA	SI	CL
1	SS	13					
2	SS	31					
3	SS	29					
4	SS	15	NP				
5	SS	16	MH	0	8	87	5
6	SS	25					
7	SS	14					
8	SS	70					
9	SS	35					
10A	SS	46					
10B			MH	2	13	80	5
11	SS	82					

DEPTH SCALE

1 : 50

LOGGED: KC

CHECKED: MJB

PROJECT: 21496759

LOCATION: See Figure 1

RECORD OF BOREHOLE: 22-2

SHEET 2 OF 2

BORING DATE: October 11, 2022

DATUM: Geodetic

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

DRILL RIG: Portable Tripod

HAMMER TYPE: CATHEAD

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	SOIL PROFILE: STRATA PLOT	SOIL PROFILE: ELEV. DEPTH (m)	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: 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 GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
		--- CONTINUED FROM PREVIOUS PAGE ---									
10	Tripod B-Casing	(GP) GRAVEL and SAND, grey; non-cohesive, wet, very dense	▨		11	SS	82				
		END OF BOREHOLE		10.36							
11		NOTES: 1. Borehole caved to a depth of 4.0 m upon completion of drilling. 2. Water level measured in monitoring well at a depth of 0.9 m below ground surface on November 7, 2022.									
12											
13											
14											
15											
16											
17											
18											
19											
20											

DEPTH SCALE

1 : 50

LOGGED: KC

CHECKED: MJB

PROJECT: 21496759
LOCATION: See Figure 1

RECORD OF BOREHOLE: 22-3

SHEET 1 OF 2

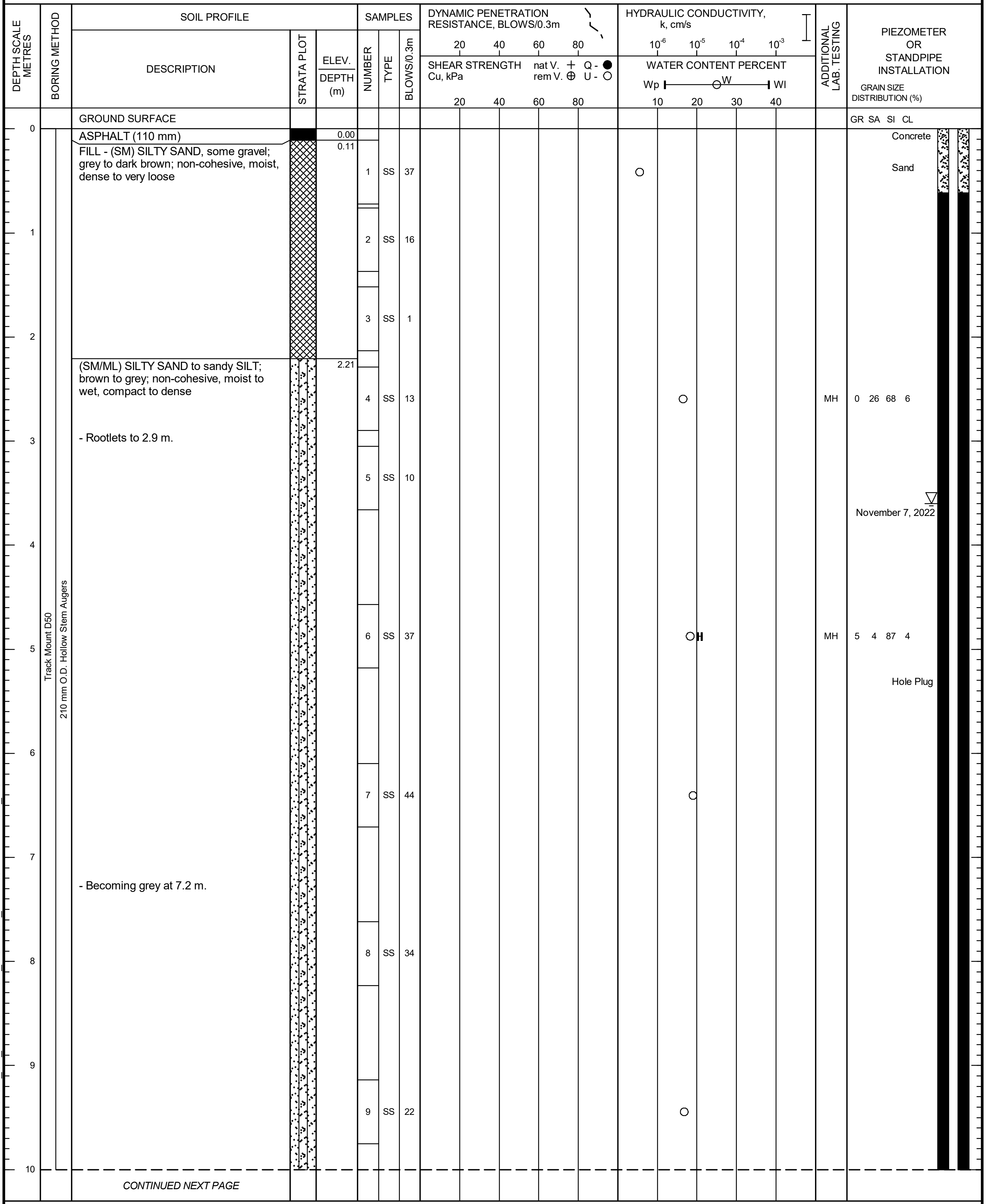
BORING DATE: October 24, 2022

DATUM: Geodetic

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

DRILL RIG: D50

HAMMER TYPE: AUTOMATIC



DESCRIPTION	ELEV. DEPTH (m)
GROUND SURFACE	
ASPHALT (110 mm)	0.00
FILL - (SM) SILTY SAND, some gravel; grey to dark brown; non-cohesive, moist, dense to very loose	0.11
(SM/ML) SILTY SAND to sandy SILT; brown to grey; non-cohesive, moist to wet, compact to dense - Rootlets to 2.9 m. - Becoming grey at 7.2 m.	2.21

NUMBER	TYPE	BLOWS/0.3m	ADDITIONAL LAB. TESTING	GR	SA	SI	CL
1	SS	37					
2	SS	16					
3	SS	1					
4	SS	13	MH	0	26	68	6
5	SS	10					
6	SS	37	MH	5	4	87	4
7	SS	44					
8	SS	34					
9	SS	22					

CONTINUED NEXT PAGE

DEPTH SCALE
1 : 50

LOGGED: ML
CHECKED: MJB

RECORD OF BOREHOLE: 22-3

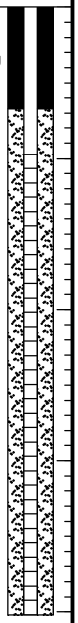
PROJECT: 21496759
LOCATION: See Figure 1
SHEET 2 OF 2
BORING DATE: October 24, 2022
DATUM: Geodetic
SPT/DCPT HAMMER: MASS, 64kg; DROP, 760mm
DRILL RIG: D50
HAMMER TYPE: AUTOMATIC

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: Track Mount D50, 210 mm O.D. Hollow Stem Augers

--- CONTINUED FROM PREVIOUS PAGE ---

DEPTH SCALE METRES	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.3m	WATER CONTENT PERCENT	ADDITIONAL LAB. TESTING	GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	PIEZOMETER OR STANDPIPE INSTALLATION
10	(SM/ML) SILTY SAND to sandy SILT; brown to grey; non-cohesive, moist to wet, compact to dense								Hole Plug
11			10	SS	46				Sand
12			11	SS	20	○	MH	0 63 36 1	
13									Screen
14			12	SS	10				
15			13	SS	16	○			
16	END OF BOREHOLE	15.86							



END OF BOREHOLE

NOTES:

1. Water encountered at a depth of 9.1 m during drilling.

2. Water level measured in monitoring well at a depth of 3.6 m below ground surface on November 7, 2022.

DEPTH SCALE
1 : 50

LOGGED: ML
CHECKED: MJB

PROJECT: 21496759

LOCATION: See Figure 1

RECORD OF BOREHOLE: 22-4

SHEET 1 OF 2

BORING DATE: October 21, 2022

DATUM: Geodetic

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

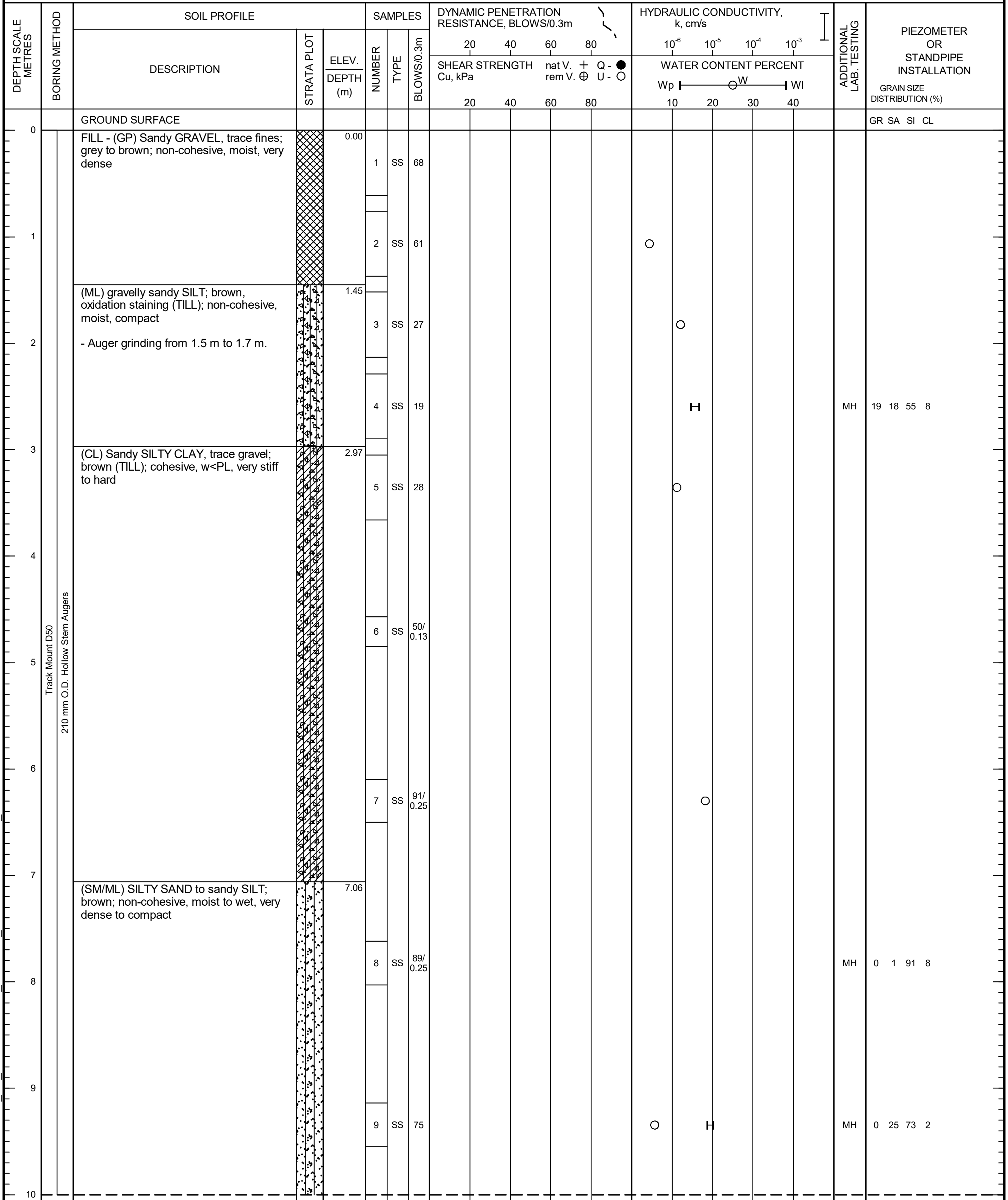
DRILL RIG: D50

HAMMER TYPE: AUTOMATIC

BORING METHOD: Track Mount D50
210 mm O.D. Hollow Stem Augers

DESCRIPTION	ELEV. DEPTH (m)
GROUND SURFACE	
FILL - (GP) Sandy GRAVEL, trace fines; grey to brown; non-cohesive, moist, very dense	0.00
(ML) gravelly sandy SILT; brown, oxidation staining (TILL); non-cohesive, moist, compact - Auger grinding from 1.5 m to 1.7 m.	1.45
(CL) Sandy SILTY CLAY, trace gravel; brown (TILL); cohesive, w<PL, very stiff to hard	2.97
(SM/ML) SILTY SAND to sandy SILT; brown; non-cohesive, moist to wet, very dense to compact	7.06

NUMBER	TYPE	BLOWS/0.3m	ADDITIONAL LAB. TESTING	GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	68		
2	SS	61		
3	SS	27		
4	SS	19	MH	19 18 55 8
5	SS	28		
6	SS	50/ 0.13		
7	SS	91/ 0.25		
8	SS	89/ 0.25	MH	0 1 91 8
9	SS	75	MH	0 25 73 2



CONTINUED NEXT PAGE

DEPTH SCALE

1 : 50

LOGGED: ML

CHECKED: MJB

PROJECT: 21496759

LOCATION: See Figure 1

RECORD OF BOREHOLE: 22-4

SHEET 2 OF 2

BORING DATE: October 21, 2022

DATUM: Geodetic

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

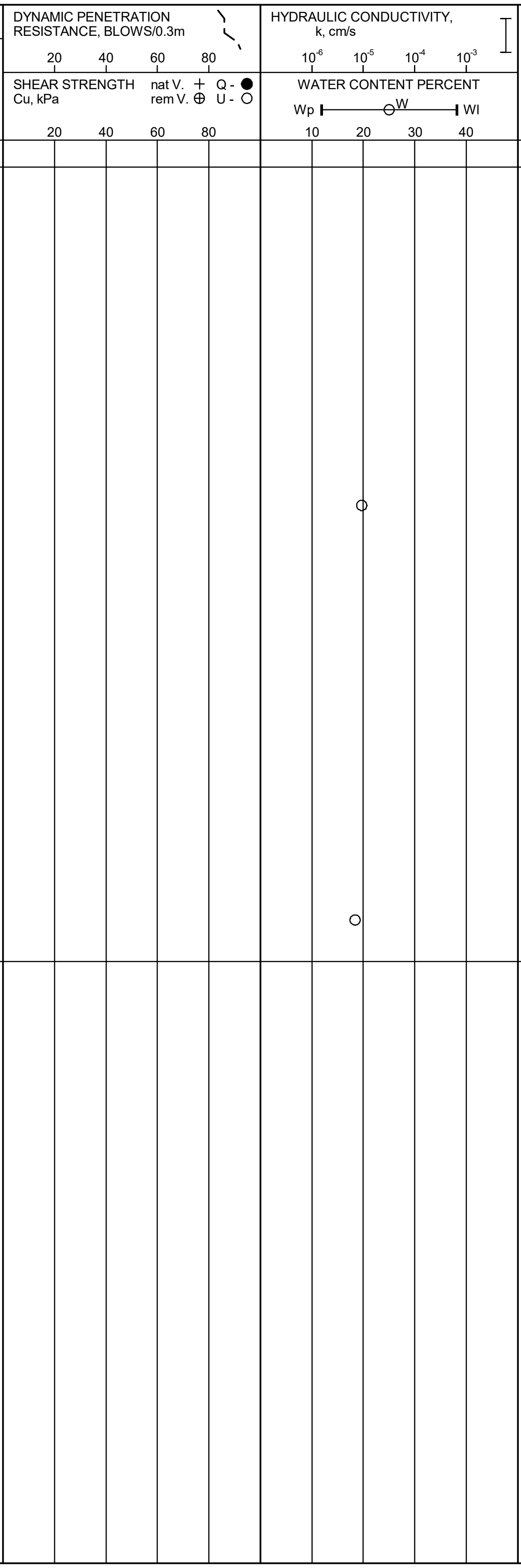
DRILL RIG: D50

HAMMER TYPE: AUTOMATIC

BORING METHOD: Track Mount D50 / 210 mm O.D. Hollow Stem Augers

--- CONTINUED FROM PREVIOUS PAGE ---

DEPTH SCALE METRES	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLE NUMBER	TYPE	BLOWS/0.3m
10	(SM/ML) SILTY SAND to sandy SILT; brown; non-cohesive, moist to wet, very dense to compact				
11			10	SS	50
12			11	SS	58
14			12	SS	36
15			13	SS	21
16	END OF BOREHOLE	15.86			



ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION

GRAIN SIZE DISTRIBUTION (%)

GR SA SI CL

NOTES:

1. Borehole caved to a depth of 12.2 m upon completion of drilling.

2. Open borehole dry upon completion of drilling.

DEPTH SCALE

1 : 50

LOGGED: ML

CHECKED: MJB

PROJECT: 21496759
LOCATION: See Figure 1

RECORD OF BOREHOLE: C1

SHEET 1 OF 1

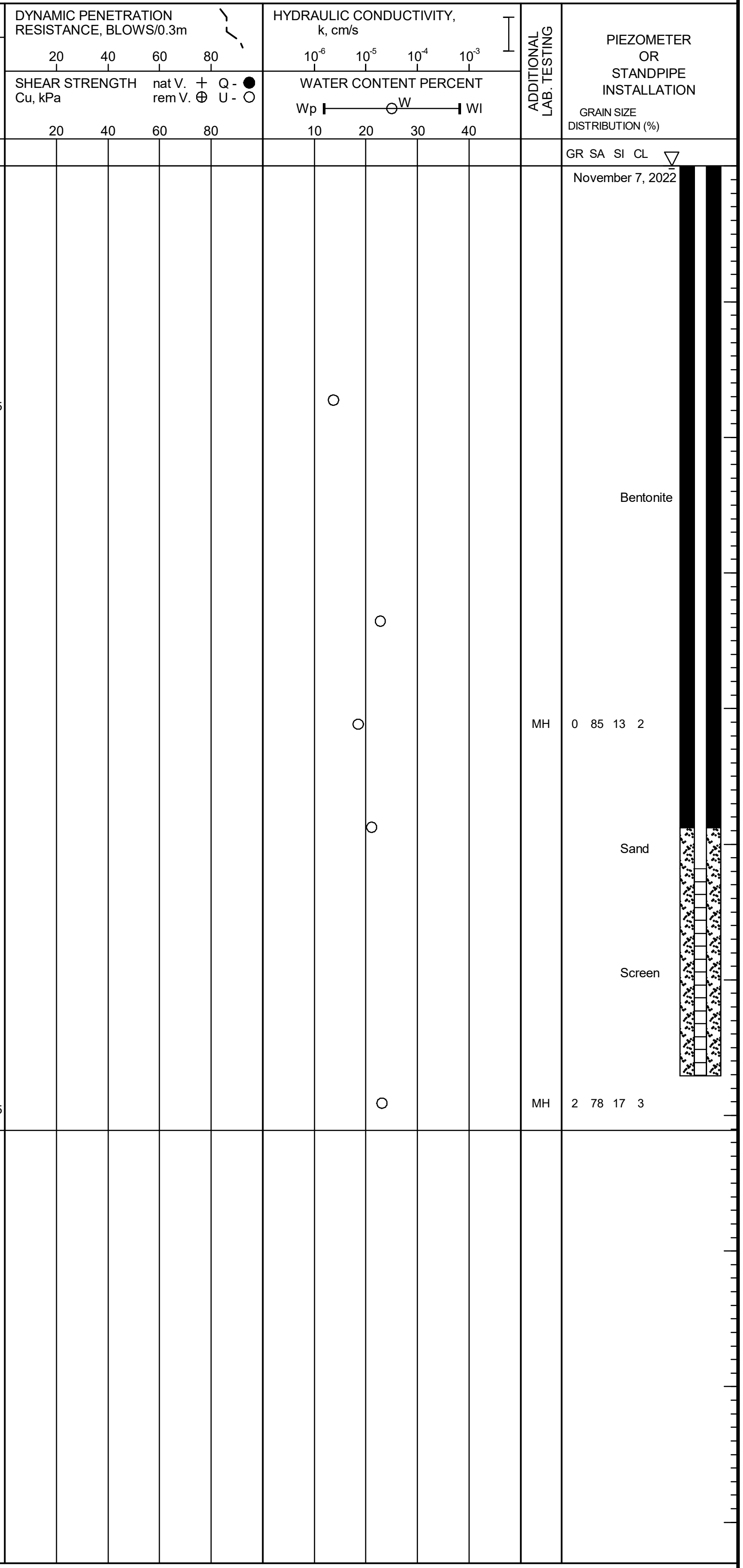
BORING DATE: October 11, 2022
DATUM: Geodetic

SPT/DCPT HAMMER: MASS, 64kg; DROP, 760mm
DRILL RIG: Portable Tripod
HAMMER TYPE: CATHEAD

BORING METHOD: Tripod, B-Casing

DESCRIPTION	ELEV. DEPTH (m)
GROUND SURFACE	
FILL - (SM) SILTY SAND, trace organics; brown and dark brown, wood fragments; non-cohesive, moist, loose to compact	0.00
(SM) SILTY SAND; brown; non-cohesive, wet to moist, compact to very dense	1.37
END OF BOREHOLE	7.11

SAMPLES NUMBER	TYPE	BLOWS/0.3m	ADDITIONAL LAB. TESTING	GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	6		
2	SS	26		
3	SS	66/ 0.25		
4	SS	18		
5	SS	37		
6	SS	36	MH	0 85 13 2
7	SS	50		
8	SS	26		
9	SS	98/ 0.25	MH	2 78 17 3



NOTE:

1. Water level measured in monitoring well at ground surface on November 7, 2022.

DEPTH SCALE
1 : 50

LOGGED: KC
CHECKED: MJB

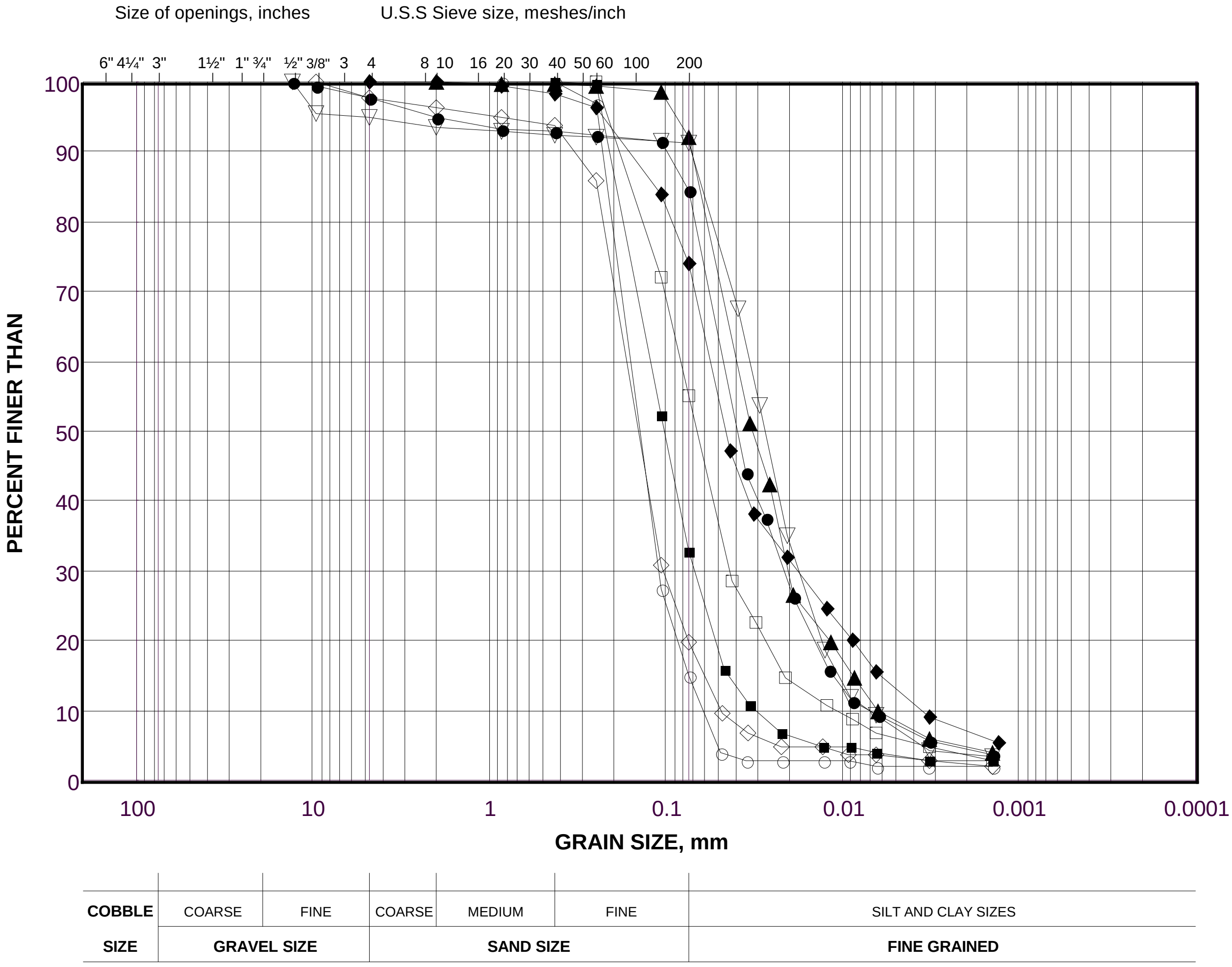
APPENDIX C

Geotechnical Laboratory Test Results

GRAIN SIZE DISTRIBUTION

(SM/ML) SILTY SAND to SILT

FIGURE C1-A



LEGEND

SYMBOL	BOREHOLE	SAMPLE	DEPTH(m)
●	22-2	10B	9.4 - 9.8
■	22-1	2	0.6 - 1.2
◆	22-3	4	2.3 - 2.9
▲	22-2	5	3.1 - 3.7
▽	22-3	6	4.6 - 5.2
❍	C1	6	3.8 - 4.4
□	22-1	7	4.6 - 5.2
△	C1	9	6.7 - 7.1

Project Number: 21496759

Checked By: MJB

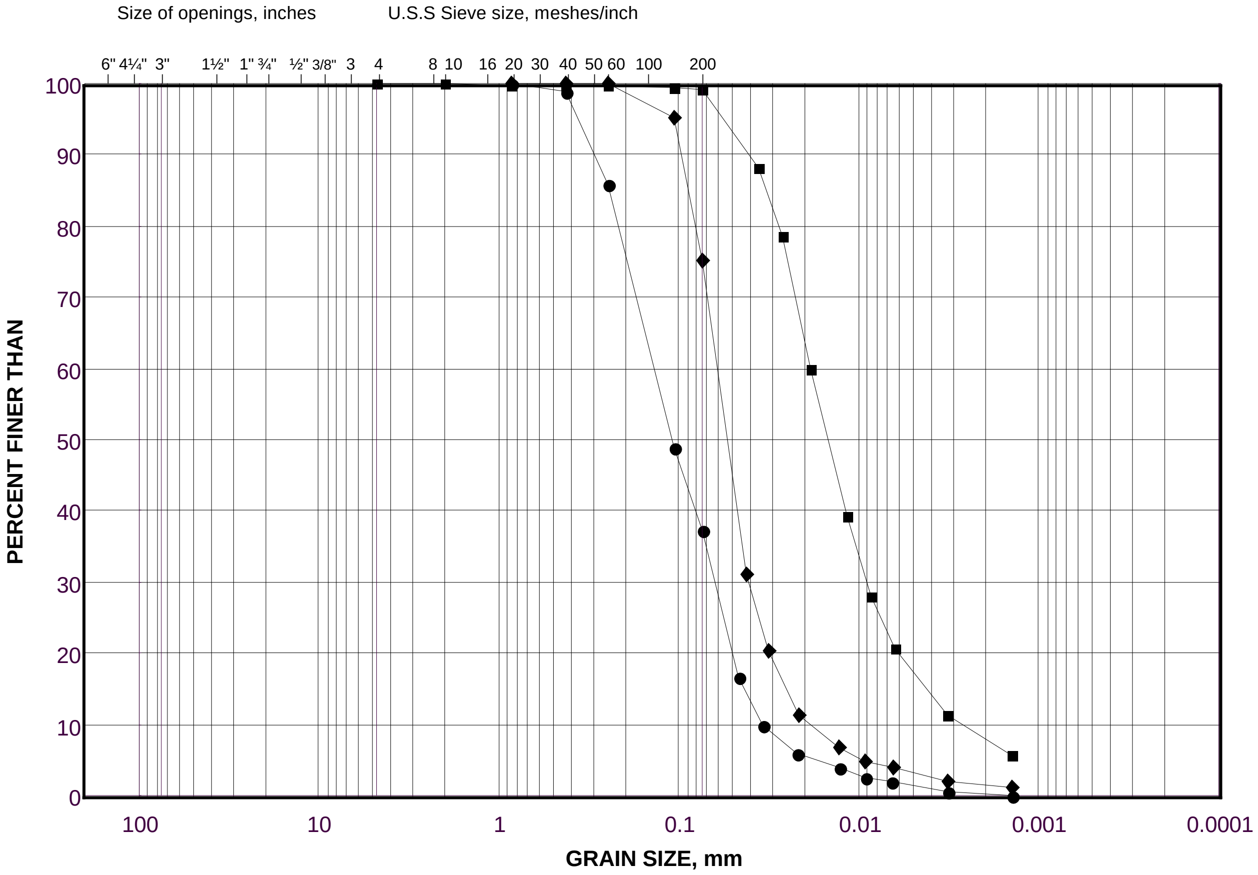
Golder Associates

Date: 16-Jan-23

GRAIN SIZE DISTRIBUTION

(SM/ML) SILTY SAND to SILT

FIGURE C1-B



COBBLE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND CLAY SIZES
SIZE	**GRAVEL SIZE**		**SAND SIZE**			**FINE GRAINED**

LEGEND

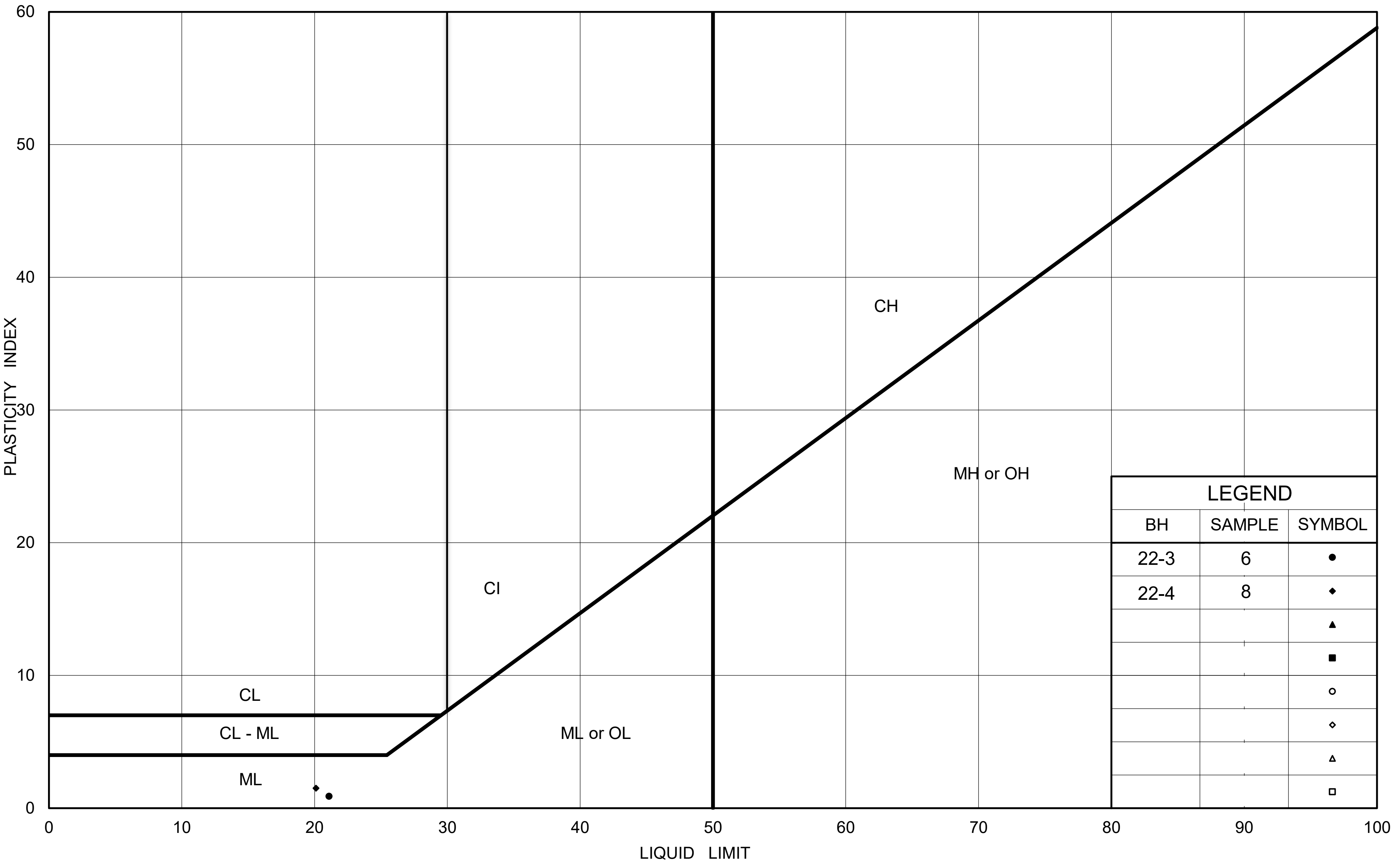
SYMBOL	BOREHOLE	SAMPLE	DEPTH(m)
●	22-3	11	12.2 - 12.8
■	22-4	8	7.6 - 8.2
◆	22-4	9	9.1 - 9.8

Project Number: 21496759

Checked By: MJB

Golder Associates

Date: 16-Jan-23



PLASTICITY CHART
(SM/ML) SILTY SAND to SILT

Figure No. C2

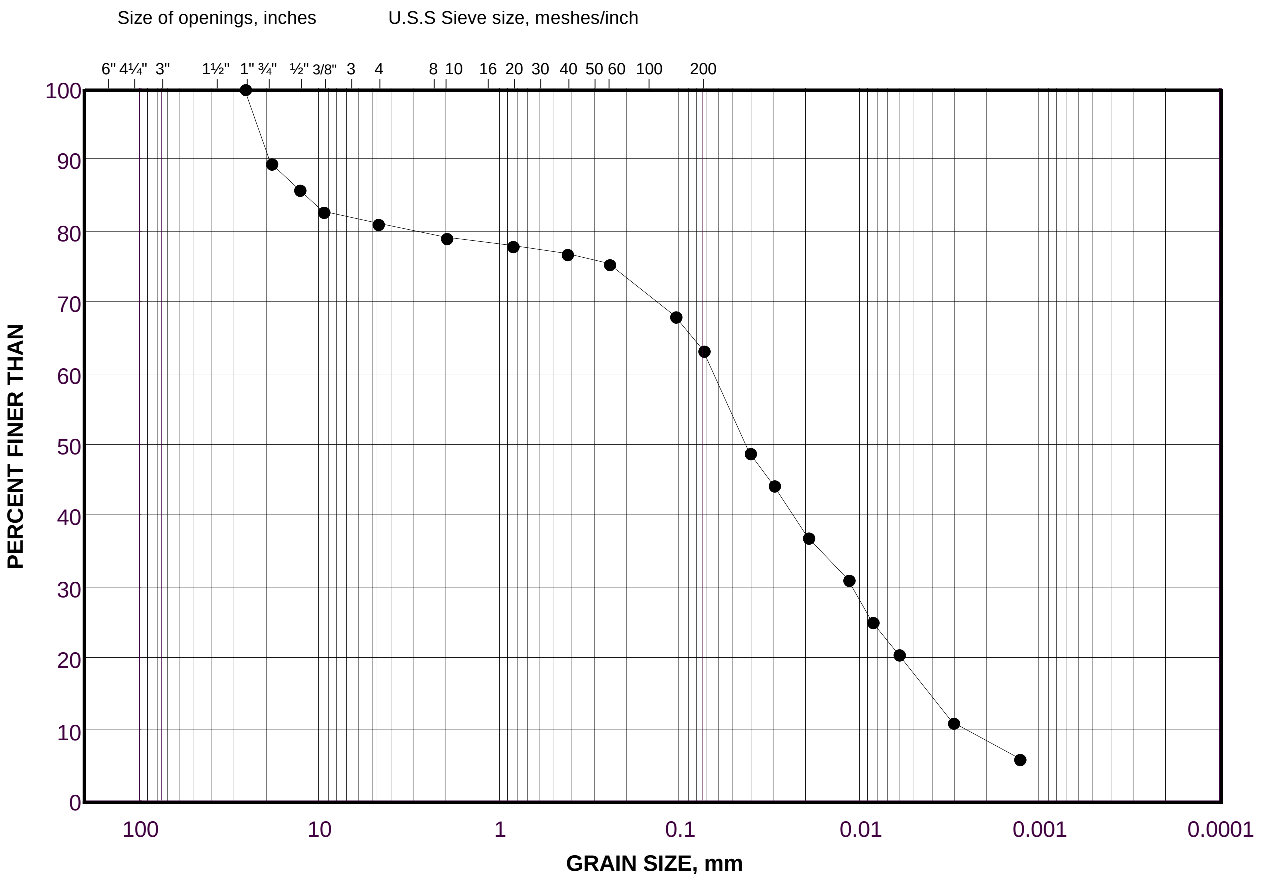
Project No. 21496579 (2000)

Checked By:MJB

GRAIN SIZE DISTRIBUTION

(ML) SILT (TILL)

FIGURE C3



COBBLE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND CLAY SIZES
SIZE	**GRAVEL SIZE**		**SAND SIZE**			**FINE GRAINED**

LEGEND

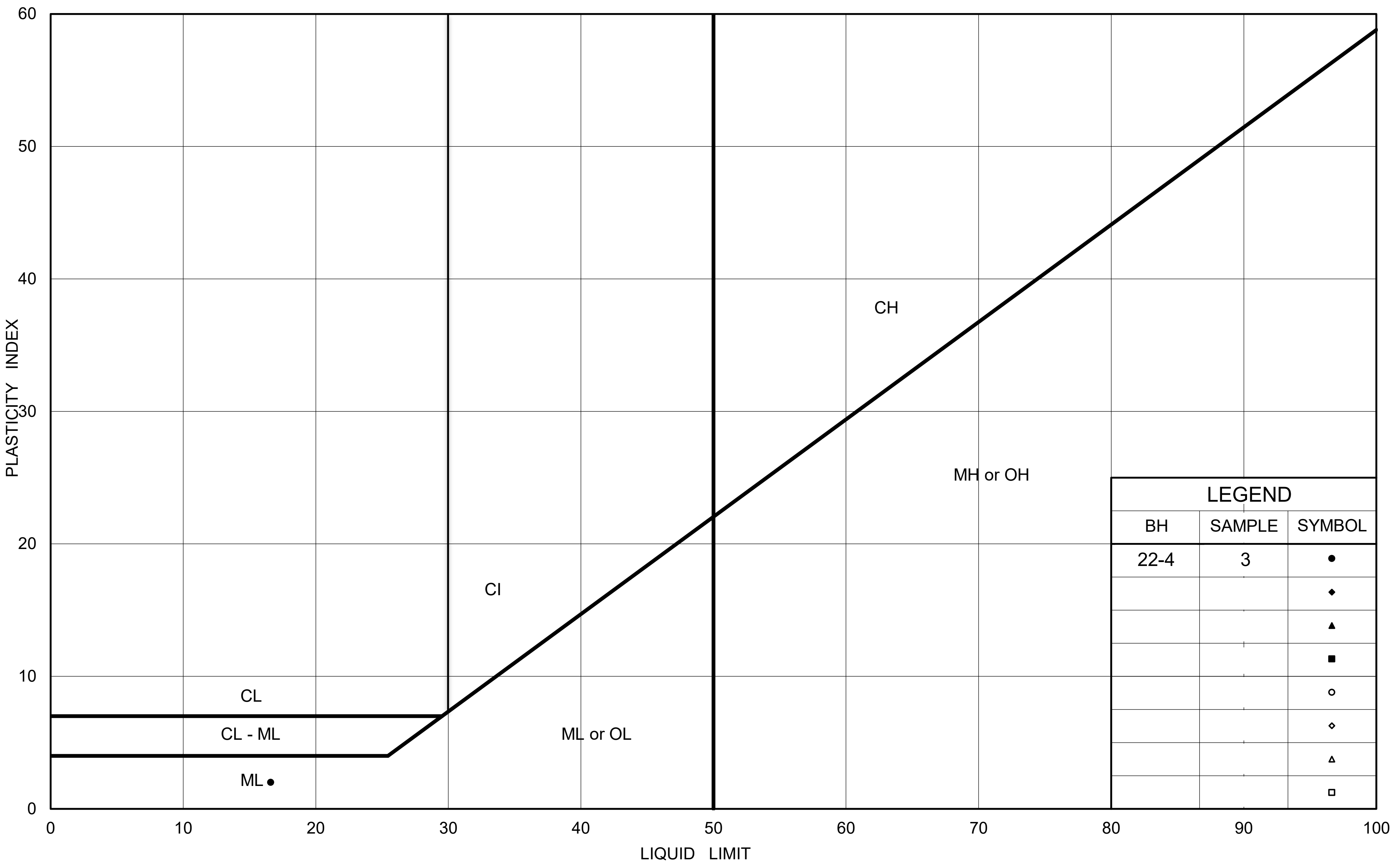
SYMBOL	BOREHOLE	SAMPLE	DEPTH(m)
●	22-4	3	1.5 - 2.1

Project Number: 21496759

Checked By: MJB

Golder Associates

Date: 16-Jan-23



PLASTICITY CHART
(ML) SILT (TILL)

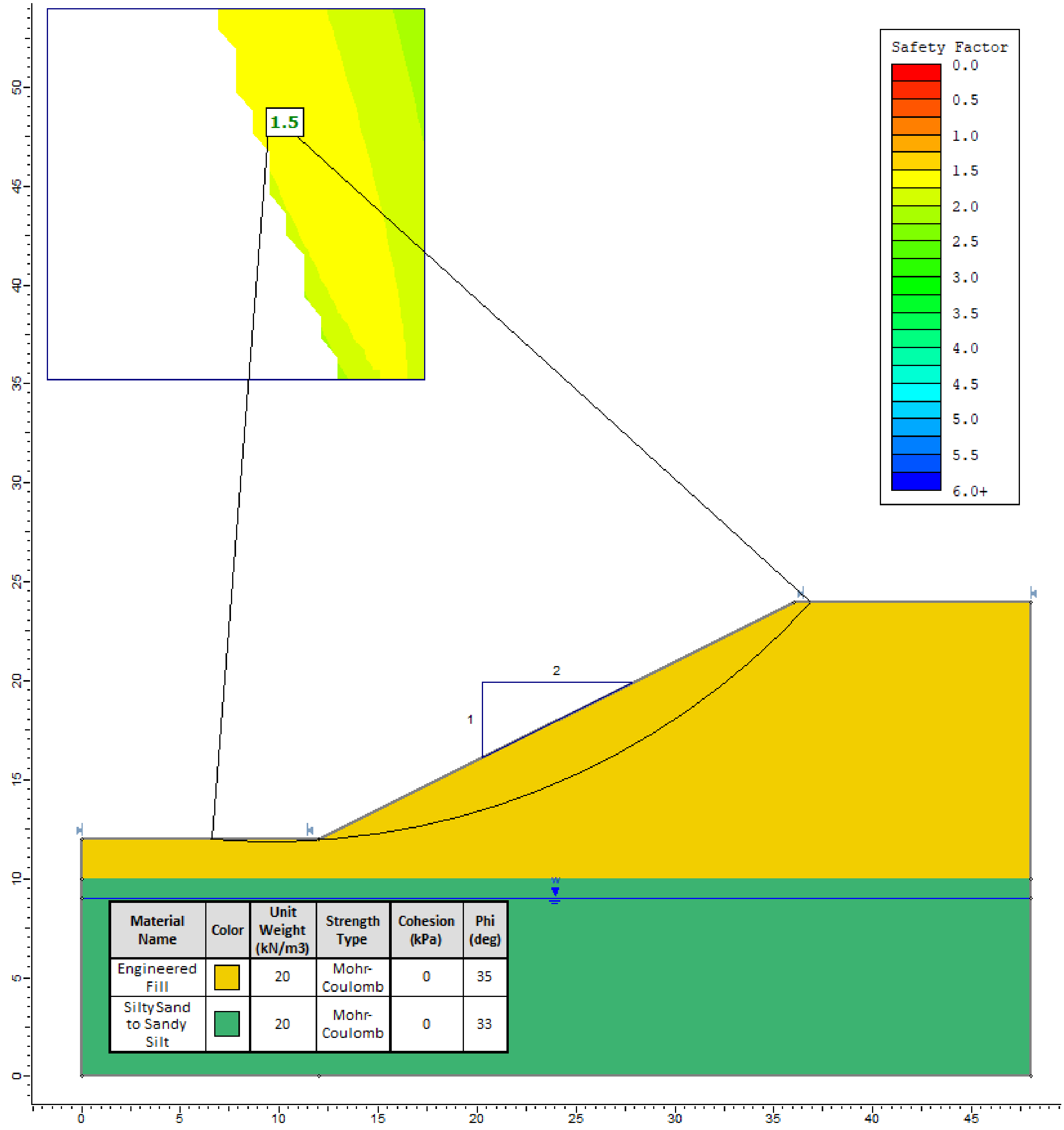
Figure No. C4

Project No. 21496579 (2000)

Checked By:MJB

APPENDIX D

Slope Stability Analysis



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)
Engineered Fill	🟨	20	Mohr-Coulomb	0	35
Silty Sand to Sandy Silt	🟩	20	Mohr-Coulomb	0	33

CLIENT

Morrison Hershfield

PROJECT

IEA for Teston Road Area Between Highway 400 and Bathurst Street

CONSULTANT

2023-02-15

PREPARED	MJB
DESIGN	MJB
REVIEW	MJB
APPROVED	SEMP

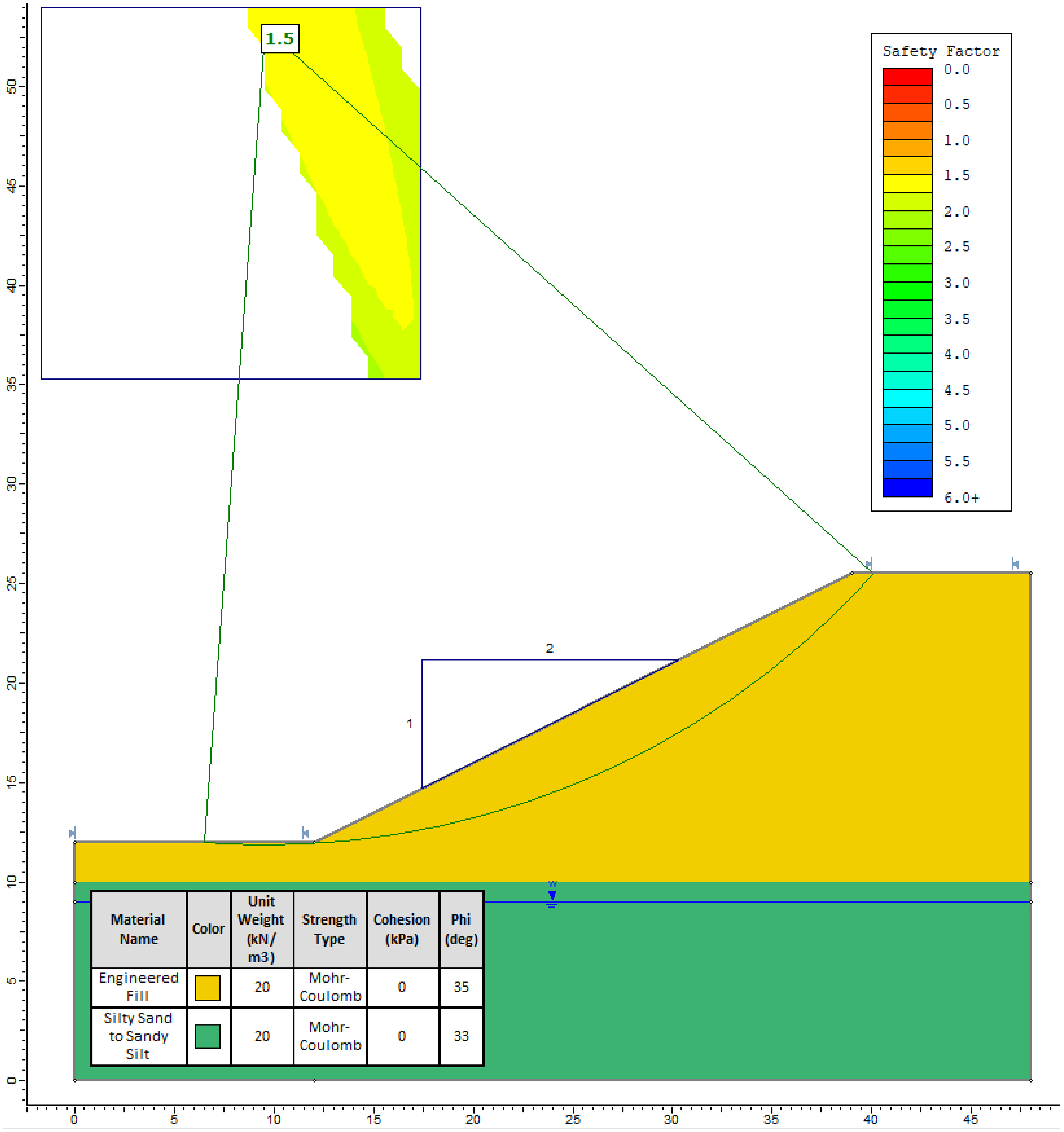
TITLE

West Approach Embankment Global Stability

PROJECT No
21496759

FIGURE
D1

Path: XXX



Material Name	Color	Unit Weight (kN/ m3)	Strength Type	Cohesion (kPa)	Phi (deg)
Engineered Fill	🟨	20	Mohr-Coulomb	0	35
Silty Sand to Sandy Silt	🟩	20	Mohr-Coulomb	0	33

CLIENT

Morrison Hershfield

PROJECT

IEA for Teston Road Area Between Highway 400 and Bathurst Street

CONSULTANT

2023-02-15	
PREPARED	MJB
DESIGN	MJB
REVIEW	MJB
APPROVED	SEMP

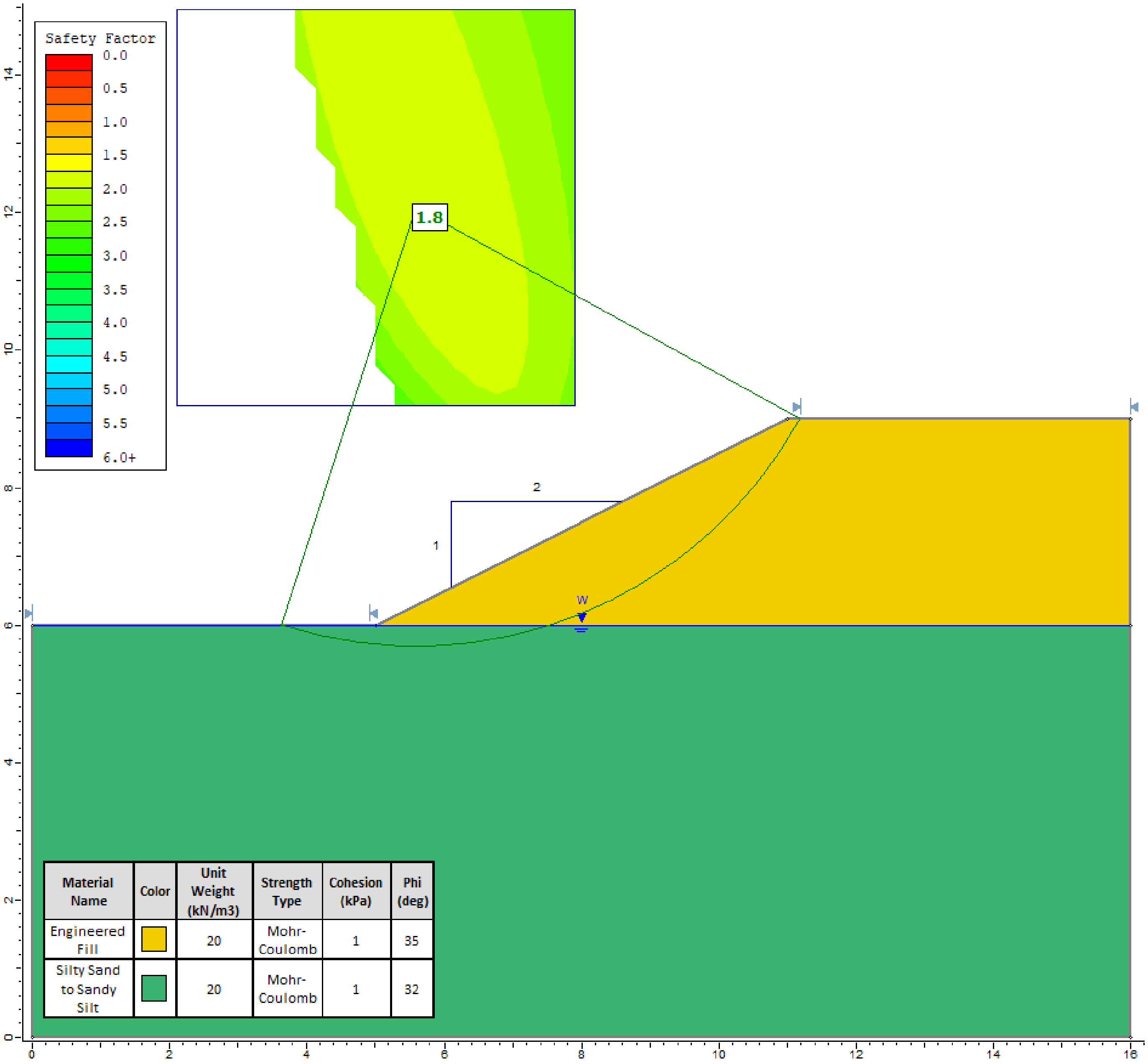
TITLE

East Approach Embankment Global Stability

PROJECT No
21496759

FIGURE
D2

Path: XXX



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)
Engineered Fill	🟨	20	Mohr-Coulomb	1	35
Silty Sand to Sandy Silt	🟩	20	Mohr-Coulomb	1	32

CLIENT

Morrison Hershfield

PROJECT

IEA for Teston Road Area Between Highway 400 and Bathurst Street

CONSULTANT

2023-02-15

PREPARED	MJB
DESIGN	MJB
REVIEW	MJB
APPROVED	SEMP

TITLE

Culvert Widening Global Stability

PROJECT No
21496759

FIGURE
D3

Path: XXX

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