July 2012

PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT

HIGHWAY 17 (NEW) INTERCHANGE AND FLYOVER STRUCTURES
ACCESS REVIEW FROM 5.5 KM SOUTH OF HIGHWAY 638 TO 2 KM NORTH OF HIGHWAY 638, ECHO BAY, ONTARIO
MINISTRY OF TRANSPORTATION, ONTARIO
G.W.P. 5022-07-00, P.O. 5007-E-0021

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FROM 5.5 KM SOUTH OF HWY 638 TO 2.0 KM NORTH OF HWY 638
G.W.P.5022-07-00

1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by GENIVAR on behalf of the Ministry of Transportation, Ontario (MTO) to provide foundation engineering services for the preliminary design of a proposed new interchange at Highway 17 and Highway 638 and for a proposed flyover at Highway 17/Bar River Road.

This report addresses the foundation investigation carried out at the interchange and at two of the flyover alternatives (Flyover East and Flyover West) proposed at the time of the subsurface investigation. In addition, the subsurface conditions anticipated at a third alternative flyover location (Flyover West Alternative 5) proposed following the completion of the foundationinvestigation is included based on a review of available subsurface information from MTO's Geocres system. The locations of the preferred interchange and flyovers are shown on Drawing 1.

The terms of reference for the scope of work are outlined in MTO's Request for Proposal (Purchase Order No. 5007-E-0021) dated December 9, 2008, Addendum #1 dated December 17, 2008, Addendum #2 dated December 24, 2008 and Golder's Scope Change letter dated February 3, 2012. Golder's proposal P81-1728 dated January 2009 is contained in Section 5.8 of GENIVAR's Technical Proposal for this assignment. The work was carried out in accordance with Golder's Project Supplemental Specialty Quality Control Plan for Foundation Engineering Services for this project, dated April 15, 2009.

The work carried out for this study should be considered preliminary in nature and is intended only to provide the designers with sufficient information for use in comparing foundation design alternatives. Detail foundation investigations will be required at the final interchange and flyover locations in order to obtain additional information to assess the subsurface conditions and to provide recommendations for detail foundation design. The base plan showing the location of the proposed structures and road re-alignments was provided to Golder by GENIVAR in April 2011 and updated on May 15, 2012 to show the location and alignment of the additional Flyover West Alternative 5.

2.0 SITE DESCRIPTION

The project area is located approximately 25 km east of Sault Ste. Marie, Ontario and generally encompasses the land surrounding the new Highway 17, southeast of the Town of Echo Bay, as shown in the key plan on Drawing 1. Highway 17 is oriented north-south in this area and all references and directions in this report are relative to the Highway 17 orientation.

The proposed new Highway 17/Highway 638 interchange (shown on Drawing 1) is located approximately 800 m south of the existing Highway 17/Highway 638 at-grade intersection. The proposed interchange consists of a diamond ramp configuration, with a realigned Highway 638 connecting to the existing Highway 17B approximately 1 km south of the existing Highway 638/Church Street intersection on the west side of Highway 17, and to Pioneer Road/existing Highway 638 on the east side of Highway 17 about 200 m south of Findlay Hill Road.

The three proposed flyover structure alternatives are located approximately 100 m north (called Flyover East), 950 m north (called Flyover West) and 1150 m north (called Flyover West Alternative 5) of the existing Highway 17/Bar River Road intersection. The locations of the flyover alternatives are shown in plan on Drawing 1.

Within the footprint of the proposed interchange the topography is relatively flat and the ground is generally low-lying, well drained with sandy soils near the ground surface, and covered in dense brush. East of Highway 17 and west of Pioneer Road, the ground within the proposed interchange footprint is low-lying, poorly drained with shrub-like vegetation to open areas and has areas of standing water.

The topography in the area of the flyover alternatives is similar, being relatively flat, low-lying and well drained farm fields or open grassy areas. Ground cover in the area of the proposed flyovers is generally comprised of crops or open grassy fields with scattered brush and wood lots.

3.0 INVESTIGATION PROCEDURES

Following the identification of the preferred interchange and two preferred flyover locations and configurations by GENIVAR, Golder met with MTO Foundations on September 10, 2010 to discuss the results of the foundations component of the access review study, the anticipated subsurface conditions at the preferred alternative location(s) and to agree on a scope of work for the Preliminary Foundation Investigation. It was concurred that a subsurface investigation be carried out at the preferred interchange location and at both ‘Flyover East’ and ‘Flyover West’ locations to confirm the anticipated subsurface conditions at each potential structure and obtain additional, site specific information for use in the evaluation and preliminary design of the flyover alternatives. Following completion of the subsurface investigation, four new alternatives for the Flyover West location were proposed by GENIVAR for consideration in selecting a preferred flyover location. The new Flyover West alternatives were evaluated based on social, economic, natural environment and technical considerations and Flyover West Alternative 5 was selected as the preferred alternative to be carried forward to preliminary design. The foundation input to the evaluation of the proposed new Flyover West alternatives was presented in Golder’s Technical Memorandum dated October 28, 2011.

The foundation investigation at the proposed Flyover East and Flyover West and Highway 638 Interchange locations was carried out between January 6 and 15, 2011 and March 5 and 10, 2011, during which time a total of eight (8) boreholes and eight (8) Cone Penetration Tests (CPTs) were advanced at approximately the locations of the proposed structure abutments and on the proposed interchange ramp alignments. Boreholes were not advanced at the Flyover West Alternative 5 location during this study as it was proposed following completion of the subsurface investigation.

The locations of the boreholes and CPTs advanced at the Flyover East and Flyover West are shown on Drawings A1 and B1 in Appendices A and B. The subsurface information provided for Flyover West Alternative 5 is based on existing borehole information, obtained through the MTO Geocres system, at the locations shown on Drawing C1 in Appendix C. The locations of the boreholes and CPTs advanced at the Highway 638 Interchange and associated ramps are shown on Drawings D1 and D2 in Appendix D. The approximate depths of the boreholes and CPTs are as follows:

- Flyover East
  - 2 boreholes at the abutments (53 m deep and 11 m deep)
  - 2 CPTs at the abutments (42 m deep and 48 m deep)
- Flyover West
  - 2 boreholes at the abutments (29 m deep including 3 m of rock core and 11 m deep)
  - 2 CPTs at the abutments (13 m deep and 14 m deep)

- Highway 17/Highway 638 Interchange
  - 2 boreholes at the abutments (53 m deep and 12 m deep)
  - 4 CPTs at the abutments (46 m deep and 19 m to 44 m deep)
  - 2 boreholes along the ramps (12 m deep and 11 m deep)

The field investigation was carried out using a D-120 track-mounted drill rig supplied and operated by Walker Drilling Ltd. of Utopia, Ontario. The boreholes were advanced through the overburden using 108 mm inside diameter (I.D.) hollow-stem augers, NW casing with water flush or a tri-cone bit with water flush. Soil samples were taken at varying depths and depth intervals, depending on the depth to and thickness of the cohesive deposits, using a 50 mm outer diameter (O.D.) split-spoon sampler operated by an automatic hammer, performed in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586 Standard Test Method for Standard Penetration Test). Field vane shear tests were conducted in cohesive soils for measurement of undrained shear strengths (ASTM D2573 Standard Test Method for Field Vane Shear Test) using an MTO standard 'N' size vane. All boreholes were backfilled with bentonite or a bentonite grout upon completion in accordance with Ontario Regulation 903 (as amended.

The boreholes were advanced to depths of up to about 55.3 m below existing ground surface including the depths of dynamic cone penetration tests (DCPTs) advanced through the bottom of the boreholes to refusal to further penetration at two of the investigation locations. The depths to refusal do not confirm bedrock, but may be inferred to indicate potential proximity to the bedrock surface. Bedrock coring was carried out at one borehole location (Borehole 10-3) using an 'NQ' sized core barrel to a depth of about 29.3 m below ground surface (i.e. 3.3 m of bedrock core).

The CPTs were advanced to refusal (or as deep as practical) using Golder's CPT equipment, to depths ranging from about 13.8 m to 48.0 m below ground surface. The CPT consists of a special probe equipped with electronic sensing elements to continuously measure tip resistance, local side friction on a sleeve and porewater pressure. It is pushed into the ground at a constant rate (ASTM D5778 Standard Test Method for Piezocone Penetration) to obtain an in situ nearly continuous profile of data. Stratigraphy and engineering properties such as shear strength and stress history can be inferred from the results.

At this site, the CPT equipment was advanced using the hydraulic system on the drill rig. Cone Penetration Test sheets are included in Appendices A, B and D for the three structure locations that were investigated. Profiles of tip resistance, sleeve friction and porewater pressure are presented together with interpreted profiles of undrained shear strength and classification index that is used to infer the soil type (i.e. soil stratigraphy). A CD containing the CPT data files is included in Appendix E.

The groundwater conditions and water levels in the open boreholes were observed during the drilling operations and are described on the Record of Borehole sheets in Appendices A, B and D. It should be noted that groundwater elevations as encountered during the subsurface investigation may not be representative of static groundwater levels since the groundwater levels in the boreholes may not have stabilized on completion of drilling. Furthermore, groundwater elevations will vary and fluctuate depending on seasonal precipitation and local soil permeability. At the CPT locations, the groundwater level can be inferred from the porewater pressure (PWP) measurements.

The field work was carried out under the overall supervision of members of our engineering and technical staff, who located the boreholes and CPTs, cleared these locations for buried utilities, supervised the drilling, sampling and in situ testing operations, logged the boreholes, and examined and cared for the soil and rock core samples. The samples were identified in the field, placed in appropriate containers, labelled and transported to our Mississauga geotechnical laboratory where the samples underwent further visual examination and laboratory testing. All of the laboratory tests were carried out to relevant MTO and/or ASTM Standards. Index testing such as water content, grain size distribution, organic content, and Atterberg limits were carried out on selected soil samples. Point load and Unconfined Compressive Strength (UCS) tests were carried out on selected samples of the bedrock core.

The results of the laboratory testing are included in Appendices A, B and D and are shown on the Record of Borehole and Drillhole sheets.

The borehole locations were staked in the field by a member of Golder's engineering staff using a hand-held Global Positioning System (GPS) unit, based on the preliminary structure location drawings provided by GENIVAR on November 18, 2010. The as-drilled borehole locations and elevations were surveyed by D.S. Urso Surveying Ltd., a registered Ontario land surveyor. The CPT locations and elevations were measured in the field by a member of our technical staff relative to the as-drilled borehole locations. The borehole and CPT locations presented in the Record of Borehole and Cone Penetration Test sheets and shown on Drawings A1, B1, D1 and D2 are positioned relative to MTM NAD 83 northing and easting coordinates and the ground surface elevations are referenced to Geodetic datum. The borehole locations, ground surface elevations and as-drilled and as-pushed CPT depths are as follows:

Structure	Borehole-DCPT / CPT	Location		Ground Surface Elevation (m)	Borehole-DCPT / CPT Depth (m)
		Northing	Easting		
Flyover East	BH 10-1	5 144 739.6	299 616.8	180.9	11.1
	BH 10-2	5 144 738.5	299 538.2	180.3	52.7
	CPT 10-1	5 144 742.6	299 617.0	180.9	42.0
	CPT 10-2	5 144 738.5	299 535.2	180.3	48.0
Flyover West	BH 10-3	5 145 477.2	299 811.1	186.0	29.3
	BH 10-4	5 145 545.8	299 769.2	183.8	11.1
	CPT 10-3	5 145 474.2	299 811.1	186.0	13.8
	CPT 10-4	5 145 545.8	299 772.2	183.8	13.9
Highway 17/ Highway 638 Interchange	BH 10-5	5 148 463.4	300 792.0	184.4	11.6
	BH 10-6	5 148 637.3	300 775.4	183.3	11.6
	BH 10-7	5 148 662.8	300 675.7	183.8	55.3
	BH 10-8	5 148 810.1	300 643.5	183.3	11.1
	CPT 10-6	5 148 638.3	300 757.4	184.4	45.7
	CPT 10-7	5 148 662.8	300 676.7	183.8	18.5
	CPT 10-7B/C	5 148 663.8	300 676.7	183.8	20.0 44.2

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

Based on geologic information published by the Ontario Geologic Society (OGS, 1991), the interchange and flyover sites are located in the physiographic region known as the Canadian Shield. The bedrock in the vicinity of the site is complex with considerable folding, intrusive activity and faulting. Pleistocene lacustrine/fluvial deposits and recent swamp sediments have been laid down in the bedrock depressions and are associated with Glacial Lake Algonquin. During periodic oscillations of the ice levels, lacustrine sediments, typically varved/stratified clays, were deposited within/under the lacustrine/fluvial deposits.

The present day topography is typically a flat plain interrupted by bedrock protrusions and dissected by fault controlled bedrock valleys. The predominant soil type deposited in these bedrock valleys is a lacustrine silty clay to clay underlying a thin veneer of locally deposited sands and silts. In general the thickness of the clay deposit increases as the distance from the nearby bedrock protrusions increases and the distance from local Lake George decreases.

4.2 Subsurface Conditions

The detailed subsurface soil and groundwater conditions as encountered in the boreholes and CPTs advanced during this investigation at Flyover East, Flyover West and the Highway 17/638 Interchange, together with the results of the laboratory tests carried out on selected soil and bedrock core samples, are given on the Record of Borehole and Cone Penetration Test sheets in Appendices A, B and D. More detailed results from the laboratory testing are included in Appendices A, B and D. The subsurface soil and groundwater conditions as presented on the Record of Boreholes obtained from MTO's Geocres system for the area surrounding Flyover West Alternative 5 are included in Appendix C.

The stratigraphic boundaries shown on the Record of Borehole/Drillhole sheets and on the stratigraphic profiles shown on Drawings A1, B1, C1 and D2 are inferred from non-continuous sampling, observations of drilling progress and the results of Standard Penetration Tests (SPTs) and in situ testing (CPTs and DCPTs). These boundaries, therefore, represent transitions between soil types rather than exact planes of geological change. Further, subsurface conditions will vary between and beyond the borehole, CPT and DCPT locations.

A detailed description of the subsurface conditions encountered in the boreholes and CPTs at each of the three flyover alternative locations and at the interchange location, is provided in the following sections.

4.3 Flyover East

The plan and profile along the centreline of the proposed structure at the Flyover East location showing the borehole and CPT locations and interpreted stratigraphy within the extent of the structure foundation area are shown on Drawing A1 in Appendix A. The proposed approach embankments will be up to about 8.6 m high above the existing ground surface. A total of two (2) boreholes (Boreholes 10-1 and 10-2) and two (2) cone penetration tests (CPTs 10-1 and 10-2) were completed to investigate the subsurface conditions at this site.

In general, the subsurface conditions at the site of the proposed structure consist of a thin layer of topsoil underlain by an approximately 47.3 m thick clay stratum. Borehole 10-1 was advanced through the clayey stratum into a sand and gravel to gravely sand deposit to a depth of about 50.8 m (Elevation 129.5 m), and a dynamic cone penetration test (DCPT) was advanced through the bottom of the borehole to practical refusal at a depth of about 52.7 m (Elevation 127.6 m).

4.3.1 Topsoil

Approximately 0.4 m and 0.3 m of silty topsoil was encountered at ground surface at the location of Boreholes 10-1 and 10-2, respectively.

SPT 'N'-values of 2 blows and 12 blows per 0.3 m of penetration were measured within the topsoil and into the upper portion of the underlying clayey silt deposit, suggesting a loose to compact relative density.

4.3.2 Clayey Silt and Silt

A cohesive deposit was encountered below the topsoil in Boreholes 10-1 and 10-2 intersected by a silt layer in Borehole 10-2. The clayey silt stratum is about 2.1 m and 0.4 m thick at the respective boreholes, and the underlying silt layer is about 0.7 m thick, with the bottom of this deposit extending to about Elevations 178.4 m to 178.9 m.

The SPT 'N'-values recorded within the clayey silt range between 1 blow and 2 blows per 0.3 m of penetration. An in situ field vane test carried out near the bottom of the cohesive deposit measured an undrained shear strength of about 10 kPa. The sensitivity is calculated to be about 2. The field vane test result together with the SPT 'N'-values indicate that the clayey silt deposit has a very soft consistency.

An SPT 'N'-value of 4 blows per 0.3 m of penetration was recorded in the silt layer, indicating a loose relative density.

Atterberg limits testing was carried out on one sample of the cohesive soil, and measured a plastic limit of 15 percent, a liquid limit of 34 percent, and corresponding plasticity index of 19 percent. These results, which are plotted on a plasticity chart on Figure A.FE.1 in Appendix A, indicate that the cohesive deposit consists of clayey silt of low plasticity.

The measured water content of one sample of the clayey silt was about 41 percent.

An organic content determination was performed on one sample of the silt layer from Borehole 10-2 and measured an organic content of 1.4 percent. The measured water content of a sample of the silt layer was about 29 percent.

4.3.3 Clay

A clay stratum was encountered underlying the clayey silt deposit and silt layer in Boreholes 10-1 and 10-2, respectively. The top of the clay stratum is at about Elevations 178.4 m and 178.9 m and the thickness of the stratum is about 46.2 m in Borehole 10-2 where it was fully penetrated at about Elevation 132.7 m. Borehole 10-1 was terminated within this deposit at a depth of 11.1 m (Elevation 169.8 m).

The SPT 'N'-values measured within the clay stratum range between 0 blows (weight of rods) and 6 blows per 0.3 m of penetration. In situ field vane tests carried out within this deposit measured undrained shear strengths ranging from about 20 kPa to 80 kPa in the upper 2.5 m of the deposit (to Elevation 176 m) and 10 kPa to greater than 120 kPa but typically less than about 40 kPa between about Elevations 176 m and 155 m. The sensitivity is calculated to range between about 1.8 and 5.3. The field vane test results together with the SPT

'N'-values indicate that the clay stratum has consistency ranging from very soft to very stiff, but typically soft to firm.

The upper about 2.5 m of the stratum in Borehole 10-2 varies in composition from clay, some silt to silty clay, trace sand, and contains silt lenses and organic silt interlayers. However the main portion of the stratum is comprised of clay, and below about Elevation 154.5 m (25.9 m depth) contains silt lenses. The results of grain size distribution tests completed on three selected samples of the clay stratum are shown on Figure A.FE.2, contained in Appendix A. Atterberg limits testing was carried out on thirteen samples of the clay stratum, and measured plastic limits between 16 percent and 24 percent, liquid limits between 47 percent and 75 percent, and corresponding plasticity indices between 29 percent and 55 percent. These results, which are plotted on a plasticity chart on Figure A.FE.3 in Appendix A, confirm that the deposit is comprised predominantly of clay of high plasticity.

The measured water content of eighteen samples from this deposit ranges between about 38 percent and 128 percent, with an average of about 59 percent.

A total of two (2) cone penetration tests (CPTs 10-1 and 10-2) were pushed through this stratum for determination of the tip resistance, sleeve friction and pore water pressure. In addition, two (2) pore pressure dissipation tests were carried out with the CPT at specific horizons within the stratum. The results of the pore water pressure dissipation tests carried out at about Elevations 169.6 m and 160.0 m (corresponding to about 11.3 m and 20.3 m below ground surface) are shown on Figure A.FE.4 in Appendix A.

4.3.4 Sand and Gravel to Gravelly Sand

A deposit of sand and gravel to gravelly sand was encountered underlying the clay stratum in Borehole 10-2 at a depth of about 47.6 m below ground surface (Elevation 132.7 m) and was not fully penetrated to a depth of 50.8 m (Elevation 129.5 m) at which depth the borehole was terminated. A DCPT was extended through the bottom of the borehole to a depth of about 52.7 m below ground surface (Elevation 127.6 m) where practical refusal to further penetration was encountered.

The measured SPT 'N' values in the sand and gravel to gravelly sand deposit are 11 and 16 blows per 0.3 m of penetration, indicating that this deposit has a compact relative density.

The deposit varies in composition from sand and gravel to gravelly sand, containing some silt and trace clay. The results of grain size distribution tests completed on two selected samples of the deposit are shown on Figure A.FE.5 in Appendix A.

The measured water content of two samples from this deposit ranges between about 10 percent and 12 percent.

4.3.5 Groundwater Conditions

Groundwater levels were observed during the drilling process and are recorded on the Record of Borehole sheets in Appendix A. The groundwater levels were measured at a depth of 5.5 m and 4.1 m below ground surface (Elevations 175.4 m and 176.2 m) in Boreholes 10-1 and 10-2, respectively, upon completion of drilling. It is noted that the groundwater levels recorded during drilling may not be representative of the natural or static groundwater level at the site. It is anticipated that the groundwater table within the area of the Flyover East structure is at or within about 1 m of the ground surface as reflected by the pore water pressure measurements in CPTs 10-1 and 10-2. The groundwater level in the area will be subject to seasonal fluctuations and precipitation events, and should be expected to be higher during wet periods of the year.

4.4 Flyover West

The plan and profile along the centreline of the proposed structure at the Flyover West location showing the borehole and CPT locations and interpreted stratigraphy within the extent of the structure foundation area are shown on Drawing B1 in Appendix B. The proposed approach embankments are to be up to about 9.0 m high above existing grade. A total of two (2) boreholes (Boreholes 10-3 and 10-4) and two (2) cone penetration tests (CPTs 10-3 and 10-4) were completed to investigate the subsurface conditions at this site.

In general, the subsurface conditions at the site of the proposed structure consist of a thin layer of topsoil underlain by an approximately 12 m thick clayey stratum, underlain by granular deposits of silt, sand and gravel containing cobbles and boulders, which are in turn underlain by granite bedrock at a depth of about 26 m below ground surface.

4.4.1 Topsoil

Approximately 0.1 m and 0.3 m of silty topsoil was encountered at ground surface in Boreholes 10-3 and 10-4, respectively.

4.4.2 Silty Sand

An approximately 0.5 m thick deposit of silty sand, trace clay was encountered below the topsoil in Borehole 10-4, at Elevation 183.5 m.

An SPT 'N'-value of 9 blows per 0.3 m of penetration was recorded through the interface with the overlapping topsoil, indicating a loose relative density.

4.4.3 Clay to Silty Clay

A clay to silty clay stratum was encountered underlying the topsoil in Borehole 10-3 and underlying the silty sand in Borehole 10-4. The top of this stratum was encountered at depths of about 0.1 m and 0.8 m below ground surface, corresponding to Elevations 185.9 m and 183.0 m at the respective boreholes. The thickness of the stratum is about 11.9 m in Borehole 10-3 where it was fully penetrated. Borehole 10-4 was terminated within this deposit at a depth of 11.1 m below ground surface (Elevation 172.7 m).

The SPT 'N'-values recorded within the clay to silty clay stratum range between 0 blows (weight of rods) and 3 blows per 0.3 m of penetration. In situ field vane tests carried out within this stratum measured undrained shear strengths ranging from about 19 kPa to 48 kPa, but typically less than about 30 kPa. The sensitivity is calculated to range between about 2.1 and 5.6. The field vane test results together with the SPT 'N'-values indicate that the clay to silty clay stratum has a generally soft to firm consistency.

The stratum varies in composition from clay, some silt to silty clay, contains organics in the upper 0.5 m in Borehole 10-3, and silt interlayers below about Elevation 177.8. The results of grain size distribution tests completed on two selected samples of the silty clay to clay stratum are shown on Figure B.FW.1 in Appendix B. Atterberg limits testing was carried out on nine samples of the stratum, and measured plastic limits between about 18 percent and 23 percent, liquid limits between about 41 percent and 65 percent, and corresponding plasticity indices between about 20 percent and 43 percent. These results, which are plotted on a plasticity chart on Figure B.FW.2 contained in Appendix B, confirm that the stratum is comprised predominantly of a clay of high plasticity.

An organic content determination was performed on one sample of the upper portion of this deposit and recorded an organic content of 2.4 per cent. The measured water content of eleven samples from this deposit ranges between about 38 percent and 74 percent with an average of about 46 percent.

A total of two (2) cone penetration tests (CPTs 10-3 and 10-4) were pushed through this stratum for determination of the tip resistance, sleeve friction and pore water pressure. In addition, two (2) pore pressure dissipation tests were carried out with the CPT at specific horizons within the stratum. The results of the pore water pressure dissipation tests carried out at about Elevations 174.4 m and 179.7 m (corresponding to about 6.3 m and 9.4 m below ground surface) are shown on Figure B.FW.3 in Appendix B.

4.4.4 Silt

A 0.8 m thick layer of silt was encountered underlying the clay stratum in Borehole 10-3 at a depth of about 12.0 m (Elevation 174.0 m).

A measured SPT 'N' value within the silt deposit is 0 blows (weight of hammer) per 0.3 m of penetration, indicating that this deposit has a very loose relative density.

The deposit consists of silt, trace to some sand, trace clay. The results of a grain size distribution test completed on a sample of the silt are shown on Figure B.FW.4 in Appendix B.

The measured water content of one sample from this deposit is about 28 percent.

4.4.5 Sand to Sand and Gravel to Gravelly Silty Sand

Interlayered cohesionless deposits of sand, sand and gravel and gravelly silty sand were encountered underlying the silt in Borehole 10-3 at a depth of about 12.8 m below ground surface (Elevation 173.2 m). The overall thickness of the deposit is about 13.2 m thick, comprised of layers varying in composition from sand, some silt, some gravel to sand and gravel, some silt, trace clay to gravelly silty sand, containing cobbles and boulders up to about 0.5 m in diameter. The results of grain size distribution tests completed on four selected samples of the cohesionless deposit are shown on Figure B.FW.5 in Appendix B.

The measured SPT 'N' values in the cohesionless deposit range from 8 to 91 blows per 0.3 m of penetration, indicating that this deposit has a loose to very dense relative density.

The measured water content of four samples from this deposit ranges between about 10 percent and 21 percent.

4.4.6 Bedrock

Bedrock was encountered and core samples were recovered from Borehole 10-3 at a depth of about 26.0 m (Elevation 160.0 m).

Based on the cored bedrock samples, the bedrock at this location consists of granitic gneiss. In general, the bedrock samples are described as slightly weathered to fresh, black and pink granitic gneiss. The Rock Quality Designation (RQD) measured on the core samples is about 98 percent to 100 percent, indicating a rock mass of excellent quality as per Table 3.10 of CFEM (2006). The Total Core Recovery (TCR) and Solid Core Recovery (SCR) of samples recovered is between 98 percent and 100 percent.

An Unconfined Compressive Strength (UCS) test carried out on a sample of the granitic gneiss bedrock measured a compressive strength of about 82 MPa. The test result which is plotted on the Record of Drillhole sheet and summarised on Table B.FW.2 in Appendix B, indicates that the bedrock is Strong (R4) as per Table 3.5 of CFEM (2006) reproduced here in Table B.FW.3 in Appendix B of the report.

Point load index tests were performed on four selected samples of the rock core. Axial point load strength index values are shown on the Record of Drillhole Sheets and on Table B.FW.1 in Appendix B. The point load index (Is_{50}) results from the axial laboratory tests carried out on four samples of the granitic gneiss bedrock range from approximately 2.1 MPa to 4.7 MPa. These index values correspond to UCS values ranging between 46 MPa and 101 MPa, based on a relationship between Is_{50} and UCS which is given by a correlation factor (k), estimated to be equal to 21.5 for this site, and calculated as the ratio of the laboratory UCS and average point load test index value(s). These values have been given for comparison only and should be interpreted together with the result of the UCS test.

Based on the laboratory UCS test and point load testing results (refer to Table B.FW.3 in Appendix B for details on the field estimation of rock hardness and R0, R1, etc. values outlined below), the estimated intact strength of the granitic gneiss bedrock ranges from medium strong (R3, 25 MPa < UCS < 50 MPa) to very strong (R5, 100 MPa < UCS < 250 MPa); (CFEM, 2006).

4.4.7 Groundwater Conditions

Groundwater levels were observed during the drilling process and are recorded on the Record of Borehole sheets in Appendix B. The groundwater levels were measured at a depth of 0.0 m and 5.2 m below ground surface (Elevations 186 m and 178.6 m) in Boreholes 10-3 and 10-4, respectively upon completion of drilling. It is noted that the groundwater levels recorded during drilling may not be representative of the natural or static groundwater level at the site. It is anticipated that the groundwater table within the area of the Flyover West structure is at or within about 1 m of the ground surface as reflected by the pore water pressure measurements in CPTs 10-3 and 10-4. The groundwater level in the area will be subject to seasonal fluctuations and precipitation events, and should be expected to be higher during wet periods of the year.

4.5 Flyover West Alternative 5

No boreholes, CPTs or DCPTs were advanced at the location of the proposed Flyover West Alternative 5 as part of the current investigation. The interpreted stratigraphy discussed in the following sections is based on available existing borehole information from MTO’s Geocres system for the nearest locations to the proposed structure.

The alignment of the proposed structure at the Flyover West Alternative 5 location together with the available existing boreholes and DCPTs and the interpreted stratigraphy along the centreline of Highway 17 in this area, are shown on Drawing C1 in Appendix C. The Record of Borehole sheets for the existing boreholes shown on Drawing C1 are also provided in Appendix C. The proposed approach embankments are to be up to about 9.0 m high above existing grade at this location.

The closest existing boreholes are located approximately 25 m east and 170 m west of the proposed structure. Borehole 17+000 19 m Lt, advanced approximately 25 m east of the proposed location, encountered greater than 7 m of soft to firm clay. Borehole 16+800 19 m Rt, advanced approximately 170 m west of the proposed structure location, encountered a thin layer of stiff clay at the ground surface underlain by soft to firm clay to a depth greater than 7 m below ground surface. A DCPT was advanced from the bottom of Borehole 16+800 19 m Rt to a depth of about 17 m below ground surface, at which depth refusal to further penetration (greater than 100 blows per 0.3 m of penetration) was encountered. Approximately 225 m east and 425 m west of the proposed Flyover West Alternative 5 location at Golder's CPT 10-4 and at existing Borehole 16+560 19 m Lt, respectively, the soft to firm clay stratum was fully penetrated at depths of about 13 m and 11 m below ground surface, respectively. A sand deposit is typically encountered underlying the clay stratum at this site, extending to depths between about 12 m and 26 m below ground surface where it is underlain by bedrock.

4.5.1 Topsoil

Approximately 0.2 m of topsoil was encountered at ground surface in Boreholes 17+000 19 m Lt and 16+800 19 m Rt.

4.5.2 Sand to Silty Sand

An approximately 0.5 m thick deposit of fine sand to silty sand was encountered below the topsoil at Elevations 186.1 m and 187.2 m in Boreholes 17+000 19 m Lt and 16+800 19 m Rt, respectively.

SPT 'N'-values of 8 blows and 14 blows per 0.3 m of penetration were recorded through the interface with the overlapping topsoil, indicating a loose to compact relative density.

The measured water content of one sample of the sand deposit from Borehole 16+800 19 m Rt is about 24 percent.

4.5.3 Clay

A clay stratum was encountered underlying the sand to silty sand in Boreholes 17+000 19 m Lt and 16+800 19 m Rt. The top of this stratum was encountered at a depth of about 0.7 m below ground surface, corresponding to Elevations 185.6 m and 186.7 m at the respective boreholes. The stratum was not fully penetrated in either of these boreholes. The depth to the bottom of the clay stratum as shown on Drawing C1 is estimated to be about 13.5 m below ground surface based on a linear interpolation between Golder's CPT 10-4 and the existing Borehole 16+560 19 m Lt. (approximately 225 m east and 425 m west of the proposed Flyover West Alternative 5 location, respectively). At these locations, the clay stratum was penetrated at depths of about 13 m and 11 m below ground surface, respectively, corresponding to about Elevations 171 m and 176.1 m.

The SPT 'N'-values recorded within the clay stratum range between 0 blows (weight of hammer) and 10 blows per 0.3 m of penetration. In situ field vane tests carried out within this stratum measured undrained shear strengths ranging from about 18 kPa to 52 kPa, but typically less than about 30 kPa. The sensitivity is reported to range between about 3 and 11. The field vane test results indicate that the clay stratum has a predominantly soft to firm consistency.

Atterberg limits testing was carried out on two samples of the clay in Boreholes 17+000 19 m Lt and 16+800 19 m Rt and measured plastic limits of about 24 percent and 26 percent, liquid limits of about 58 percent and 55 percent, and corresponding plasticity indices of about 34 percent and 29 percent. These results, which are show on the Record of Borehole sheets in Appendix C, confirm that the stratum is comprised predominantly of clay of high plasticity.

The measured water content of eight samples from this deposit ranges between about 32 percent and 75 percent.

4.5.4 Sand to Sand and Gravel to Gravelly Silty Sand

The sandy deposits typically present underlying the clay stratum at this site were not encountered in the existing boreholes located closest to Flyover West Alternative 5, which were terminated within the overlying clay stratum. The depth to the top of the sand deposit at the Flyover West Alternative 5 location, as shown on Drawing C1, has been estimated to be about 13.5 m below ground surface based on a linear interpolation between Golder's CPT 10-4 and the existing Borehole 16+560 19 m Lt (approximately 225 m east and 425 m west of the proposed Flyover West Alternative 5 location, respectively). At these locations, the top of the deposit was encountered at depths of about 13 m and 11 m below ground surface, respectively, corresponding to Elevations 171 m and 176.1 m. At the other boreholes in the vicinity of the Flyover West Alternative 5 alignment, interlayered cohesionless deposits of sand, sand and gravel and gravelly silty sand were encountered underlying the clay stratum. The SPT 'N'-values measured within this stratum in the nearest existing Borehole 16+560 19 m Lt and Golder's Borehole 10-3, range from 8 blows to 91 blows per 0.3 m of penetration, suggesting that this deposit may have a loose to very dense relative density. The bottom of this deposit is estimated to be at about Elevation 163.5 m based on a linear interpolation between the existing Borehole No. 7 and Golder's Borehole 10-3.

4.5.5 Bedrock

Bedrock was encountered and core samples were recovered from Boreholes No. 7 and 10-3 (approximately 225 m east and 830 m west of the proposed structure, respectively) underlying the sand deposits at depths of about 12 m and 26 m below ground surface (Elevation 176.2 m and 160.0 m). Based on a linear interpolation between these boreholes, it is estimated that the bedrock surface may be at about 21 m below ground surface (approximate Elevation 163.3m) at the proposed Flyover West Alternative 5 alignment. The actual depth to bedrock will require confirmation at the detail investigation and design stage.

4.5.6 Groundwater Conditions

Groundwater level observations recorded on the Record of Borehole sheets indicate that upon completion of the drilling process, the groundwater was at about 5.2 m below ground surface in Borehole 16+800 19 m Rt. Borehole 17+000 19 m Lt caved at about 5.5 m below ground surface suggesting the presence of groundwater near this depth. However these groundwater levels were likely not stabilized and may not be representative of the natural or static groundwater level at the site. It is anticipated that the groundwater table within the area of the West Flyover Alternative 5 structure is at or within about 1 m of the ground surface, similar to that indicated by the pore water pressure measurements in CPTs 10-3 and 10-4 advanced at the proposed Flyover West location. The groundwater level in the area will be subject to seasonal fluctuations and precipitation events, and should be expected to be higher during wet periods of the year.

4.6 Highway 17/Highway 638 Interchange

A plan view of the proposed interchange configuration showing the borehole and CPT locations and interpreted stratigraphy along the centreline of the bridge structure are shown on Drawings D1 and D2 in Appendix D. The structure is to be located approximately 800 m south of the existing Highway 17/Highway 638 intersection. The proposed approach embankments are to be up to approximately 9.4 m high above existing grade and the proposed interchange ramps embankments will be up to about 7.5 m high above existing grade. A total of two boreholes (Borehole 10-7 and 10-8) and four cone penetration tests (CPTs 10-6 and 10-7, 10-7b/c) were completed at the structure abutments and two (2) boreholes (Boreholes 10-5 and 10-8) were completed along the ramp alignments, to investigate the subsurface conditions at the interchange location.

The subsurface conditions at the site of the proposed interchange generally consist of approximately 1.5 m to 2.8 m of fill and/or topsoil, underlain by between about 3.3 m and 7.3 m of sand and silt deposits, with a clayey silt interlayer at one location, underlain by a stratum of clayey silt to clay to a depth of about 53 m below ground surface (about Elevation 130.8 m). The clay stratum is intersected by a layer of sand approximately 3.4 m thick at a depth of about 22.5 m below ground surface (about Elevation 161 m).

4.6.1 Topsoil

An approximately 0.3 m to 0.5 m thick layer of sandy topsoil was encountered at ground surface in all of the boreholes advanced within the interchange area.

The SPT ‘N’-values measured immediately above the interface with the underlying native soil or fill layer area range from 6 to 48 blows per 0.3 m of penetration indicating a compact to dense relative density.

The measured water content of one sample of the sandy topsoil from Borehole 10-6 is about 22 percent.

4.6.2 Sand to Silty Sand Fill

Fill was encountered below the topsoil at Boreholes 10-6 and 10-7 advanced in the proposed abutment areas of the interchange structure, extending from a depth of 0.4 m and 0.3 m below ground surface (Elevation 182.9 m and 183.5 m) for a thickness 1.1 m and 2.5 m, respectively.

The measured SPT ’N’- values within the fill range from 1 to 8 blows per 0.3 m of penetration, indicating that the fill has a very loose to loose relative density.

The fill varies in composition from sand, trace to some silt to silty sand, trace clay and contains wood fragments at one location. The results of grain size distribution tests on two samples of the sand fill are shown on Figure D.IC.1, in Appendix D. One organic content determination carried out on a sample of the sand fill from Borehole 10-7 measured an organic content of 2.6 per cent.

The measured water content of four samples from this deposit ranges between about 27 percent and 39 percent.

4.6.3 Sand to Silt

A sequence of granular, cohesionless soil layers was encountered underlying the topsoil in Boreholes 10-5 and 10-8 and underlying the fill in Boreholes 10-6 and 10-7 at depths ranging from about 0.3 m to 2.8 m below ground surface (Elevation 183.9 m to 181.1 m) and the overall deposit is between 3.3 m and 7.3 m thick.

The measured SPT ‘N’ values in the granular deposits range between 0 blows (weight of hammer) and 13 blows per 0.3 m of penetration, indicating that this deposit has a very loose to compact relative density.

The granular deposit varies in composition from sand, trace to some silt, to silt, some sand to sandy silt/silty sand trace to some clay. The results of grain size distribution tests completed on ten selected samples of the granular layers are shown on Figure D.IC.3A and 3B, in Appendix D. An Atterberg limits test was carried out on the fines portion of a sample of the silty sand deposit from Borehole 10-5 and indicates that the fines have slight plasticity (see plasticity chart on Figure D.IC.4).

A 1 m thick clayey silt stratum was encountered within the sand to sand and silt portion of the deposit at a depth of about 5.6 m below ground surface (about Elevation 178.2 m) in Borehole 10-7. One Atterberg limits test carried out on a sample of this layer measured a plastic limit of about 15 percent, a liquid limit of about 25 percent and a plasticity index of about 10 percent, and a water content of about 36 percent. The result of the Atterberg Limits test is plotted on a plasticity chart on Figure D.IC.2 and indicates that the layer is a clayey silt of low plasticity.

Organics and wood fragments were noted in the sand and silt layer in Borehole 10-7 and an organic content determination carried out on one sample of the soil from this layer measured an organic content of 0.9 percent. The water content measured on twelve samples from the various layers of this deposit range from about 21 percent to 34 percent.

4.6.4 Clay to Clayey Silt

A clayey silt to clay stratum was encountered underlying the granular deposits in all of the boreholes advanced within the interchange footprint. The stratum was encountered at depths between about 3.8 m and 8.7 m below ground surface (about Elevations 180.6 m and 175.1 m) and was not fully penetrated in any of the boreholes to the depths drilled (up to 53.0 m below ground surface (Elevation 130.8 m) in Borehole 10-7). A dynamic cone penetration test (DCPT) was advanced through the bottom of Borehole 10-7 to a depth of 55.3 m (Elevation 128.5 m) where practical refusal to further penetration was encountered.

The SPT 'N'-values measured within the clayey silt to clay stratum range between 0 blows (weight of rods) and 16 blows per 0.3 m of penetration. In situ field vane tests carried out within this deposit measured undrained shear strengths ranging from about 25 kPa to about 75 kPa to about Elevation 161.3 m (22.5 m below ground surface) and from about 60 kPa to greater than 120 kPa below Elevation 161.3 m. The sensitivity is calculated to range between about 2.1 and 5.6. The field vane test results together with the SPT 'N'-values indicate that the clayey silt to clay stratum is generally firm to very stiff in consistency.

The stratum varies in composition from clayey silt to clay, and contains silt seams and interlayers at varying depths. The results of grain size distribution tests completed on four selected samples of the clayey stratum are shown on Figure D.IC.5, in Appendix D. Atterberg limits testing was carried out on fifteen samples of the stratum and measured plastic limits between about 14 percent and 28 percent, liquid limits between about 23 percent and 68 percent, and corresponding plasticity indices between about 9 percent and 43 percent. These results, which are plotted on a plasticity chart on Figure D.IC.6 in Appendix D, confirm that the composition of the stratum ranges from clayey silt of low plasticity to clay of high plasticity.

The measured water content of fifteen samples from this deposit ranges between about 29 percent and 69 percent with an average of about 42 percent.

An approximately 3.4 m thick layer of sand, some silt was encountered in Borehole 10-7 within the clayey silt to clay stratum at a depth of about 22.5 m below ground surface (about Elevation 161.3 m). An SPT 'N'-value measured within this layer is 0 blows (weight of rods) per 0.3 m of penetration indicating a very loose relative density. However, based on the results of the CPT testing, in particular CPT 10-6 that was pushed through this layer, the relatively high q_t (tip stress) and f_s (sleeve friction) values over this elevation suggest the sand has a loose to compact relative density. The results of a grain size distribution test completed on one sample of the sand layer are shown on Figure D.IC.8, contained in Appendix D.

A total of four (4) cone penetration tests (CPTs 10-6, 10-7 and 10-7B/C) were pushed through the clayey silt to clay stratum at the interchange structure location to measure the tip resistance, sleeve friction and pore water pressure. In addition, three (3) pore pressure dissipation tests were carried out with the CPT at specific horizons within the stratum. The results of the pore water pressure dissipation tests carried out at about Elevation 162.6 m, 163.8 m and 147.5 m (corresponding to about 20.7 m, 20.0 m and 36.3 m below ground surface) respectively and shown on Figure D.IC.7 in Appendix D.

4.6.5 Groundwater Conditions

Groundwater levels were observed during the drilling process and are recorded on the Record of Borehole sheets in Appendix D. The groundwater levels were measured in Boreholes 10-5 to 10-8 at depths between about 1.6 m and 4.6 m below ground surface, corresponding to between about Elevation 182.2 m and 178.7 m. It is noted that the groundwater levels recorded during drilling may not be representative of the natural or static groundwater level at the site. It is anticipated that the groundwater table within the area of the interchange is at or within about 1 m of the ground surface, as reflected by the pore water pressure measurements in CPTs 10-6 and 10-7. The groundwater level in the area will be subject to seasonal fluctuations and precipitation events, and should be expected to be higher during wet periods of the year.

5.0 CLOSURE

This Foundation Investigation Report was prepared by Mr. Matthew Kelly, P.Eng., and reviewed by Mr. J. Paul Dittrich, Ph.D., P.Eng., a Senior Geotechnical Engineer and Principal with Golder. Mr. Jorge M. A. Costa, P.Eng., Golder's Designated MTO Contact for this project, carried out an independent quality control review of the report.

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PART B

FOUNDATION DESIGN REPORT
HIGHWAY 17 (NEW) INTERCHANGE AND FLYOVER STRUCTURES
ACCESS REVIEW
FROM 5.5 KM SOUTH OF HWY 638 TO 2.0 KM NORTH OF HWY 638
G.W.P. 5022-07-00

6.0 FOUNDATION ENGINEERING RECOMMENDATIONS

This section of the report provides preliminary recommendations on the foundation aspects of design for the Highway 17/Highway 638 interchange and three alternative flyover locations (Flyover East, Flyover West and Flyover West Alternative 5), as shown on Drawing 1. The recommendations are based on interpretation of the factual geotechnical data obtained from the boreholes and Cone Penetration Tests (CPTs) advanced during the subsurface investigation at the sites and from the existing geotechnical information available through MTO's Geocres system. It is important to note that the subsurface conditions used in the analyses for Flyover West Alternative 5 have been modeled based on interpolation of the soil and groundwater conditions in existing boreholes located between about 25 m and 800 m from the proposed structure location.

The interpretation and preliminary recommendations provided in the following sections are intended only to provide the designers with sufficient information to assess the feasible foundation alternatives and to perform a preliminary design of the proposed structure foundations. Further detailed investigation will be required to confirm the soil and bedrock conditions at the detail design stage. As such, where comments are made on construction they are provided only in order to highlight those aspects which could affect the design of the project. Those requiring information on aspects of construction should make their own interpretation of the factual information provided as it may affect equipment selection, proposed construction methods, scheduling and the like.

6.1 Structure Foundations

6.1.1 Foundation Options

Given the low strength and highly compressible nature of the thick, clayey strata and the significant depth to competent foundation strata at each of the sites, spread footings founded at shallow depth are not considered to be a feasible foundation alternative for support of the bridge structures. Instead, it is recommended that the interchange and flyover structure(s) be supported on deep foundations comprised of either end-bearing piles, driven to bedrock/refusal, or friction piles driven to/terminated within the clayey silt to clay stratum encountered at the proposed bridge sites. Given the significant thickness of soft clayey strata at the site and the presence of the underlying saturated sandy deposits (some containing cobbles and boulders), caissons are not considered to be a practical alternative due to the anticipated construction problems associated with soil squeeze, base heave, and need for long temporary or permanent liners. Supporting the structures on steel H-Piles driven to refusal in the lower granular deposits or on bedrock is considered to be the preferred alternative from a foundations perspective, for the structures at this site, subject to confirmation at the detail design stage of depth to bedrock/refusal at each of the foundation elements.

The design recommendations for the driven pile options for the structure foundations are presented in the following sections. A summary of the advantages, disadvantages, relative costs and risks/consequences for the structure foundation alternatives for each structure location (i.e. Flyover East, Flyover West, Flyover West Alternative 5 and the Highway 638 Interchange) is presented in Tables A2, B2, C2 and D2 in Appendices A through D, respectively. A comparison between Flyover East, Flyover West and Flyover West Alternative 5 sites, for the preferred alternative from a structure foundation and approach embankment foundation perspective, is presented in Tables 1 and 2 following the text of this report.

Significant negative skin friction or downdrag loads, due to the consolidation of the clayey soils deposit present at each of the sites, will develop along the portion of the pile shaft that is embedded within the clayey silt deposit.

The actual downdrag loads that develop will depend on the type of pile (friction or end bearing), pile dimensions, pile loading and construction sequence, and will require further evaluation at the detail design stage. It is recommended that embankment construction and preloading be carried out prior to the installation of the piles to reduce the downdrag loads that develop on the pile shaft.

6.1.1.1 Friction Piles

A system of driven piles developing resistance primarily from shaft friction could be considered to support the structure foundations. Steel H-piles or concrete filled, steel tube piles could be employed. Given the soft/loose nature of the soil deposits directly below the ground surface, friction piles would have to be driven through the upper sand and silt layers (where present) and the upper very soft to firm clayey stratum, approximately to mid-depth into the lower stiff clayey stratum (at Flyover East and the Highway 638 Interchange) or into the lower compact to dense sandy stratum (at Flyover West and Flyover West Alternative 5). Further discussion and design recommendations of the suitability of friction piles for support of the structure foundations, for each structure site, is provided below.

6.1.1.2 End-bearing piles

A system of piles driven to refusal within the lower granular deposits underlying the thick clay strata (at Flyover East and the Highway 638 Interchange) or on bedrock underlying the sand strata (at Flyover West and Flyover West Alternative 5) and developing resistance primarily from end bearing could be considered to support the structure foundations. Steel H-piles or concrete filled steel tube piles could be employed. In this case the ends of the piles should be reinforced with flange plates and web stiffeners to further mitigate against possible damage during seating of the piles, in particular at Flyover West and Flyover West Alternative 5 where cobbles and boulders are expected to be encountered overlying the bedrock.

Refusal conditions were encountered in the boreholes advanced on the west side of the Flyover East structure (as resistance to further dynamic cone penetration) and on the east side of the Flyover West structure (on bedrock confirmed by coring). As such, it will be critical to carry out additional investigation within the footprint of each foundation element at the detail design stage to confirm the depths to refusal on bedrock and hence establish the required pile lengths. Further discussion and design recommendations of the suitability of end-bearing piles for support of the structure foundations, for each structure site, is presented below.

6.1.2 Resistance to Lateral Loads

Lateral loading could be resisted fully or partially by the use of battered piles. If vertical piles are used, the resistance to lateral loading will have to be derived from the soil in front of the piles.

The evaluation of the piles subjected to lateral loads should take into account such factors as the relative rigidity of the pile to the surrounding soil, the fixity condition at the head of the pile (pile cap level), the structural capacity of the pile to withstand bending moment, the soil resistance that can be mobilized, the tolerable lateral deflection at the head of the pile and pile group effects.

The resistance to lateral loading in front of a vertical pile may be calculated using subgrade reaction theory where the coefficient of horizontal subgrade reaction, k_h (in kPa/m), is determined in accordance with Section C6.8.7 in the Commentary to the Canadian Highway Bridge Design Code (CHDBC, 2006) based on the equations given below (as per CFEM, 1992):

For cohesionless soils:

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

where n_h is the constant of horizontal subgrade reaction (kPa/m)
z is the depth (m)
B is the pile diameter/width (m)

and for cohesive soils:

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

where s_u is the undrained shear strength of the soil (kPa)
B is the pile diameter/width (m)

The values of n_h and s_u to be assumed for preliminary structural analysis are as given below. For preliminary design, an interpolation of the elevations of the applicable soil units from the nearest boreholes will have to be carried out. The elevation of the applicable soil units will then have to be confirmed at each foundation element at detail design. Both the structural and geotechnical resistances of the piles should be evaluated to establish the governing case.

Soil Unit	**s_u (kPa)**	**n_h* (kPa/m)**
Soft to firm clayey silt to clay	20	-
Stiff to very stiff clayey silt to clay	75	
Very loose to loose sand and silt	-	1300
Compact sand and silt	-	4400
Dense to very dense sand and silt	-	11000

Note: *values provided for cohesionless deposits below the ground water table.

The near surface zone of soil (down to a depth below the pile cap equal to about 1.5 x B, where B = pile diameter, after Broms (1964)) should be neglected in the calculation of lateral resistance of the pile to account for disturbance effects during installation.

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

Pile Spacing in Direction of Loading d = Pile Diameter	Subgrade Reaction Reduction Factor
8d	1.00
6d	0.70
4d	0.40
3d	0.25

Reference: NAVFAC (1986)

The subgrade reaction reduction factor should be interpolated for pile spacing in between those listed in the table above.

6.1.3 Frost Protection

All pile caps should be provided with a minimum of 1.8 m of soil cover for frost protection.

6.1.4 Site Coefficient

For seismic design purposes, the Site Coefficient, S, for this site, based on experience and considering the guidelines in Section 4.4.6 of the CHBDC (2006) may be taken as:

- S = 2.0 for Flyover East, consistent with Soil Profile Type IV
- S = 1.5 for Flyover West, Flyover West Alternative 5 and Highway 638 Interchange, consistent with Soil Profile Type III.

6.1.5 Lateral Earth Pressures

The lateral earth pressures acting on the abutment stems and any associated wing walls/retaining walls 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. Seismic (earthquake) loading must also be taken into account in the design.

The following recommendations are made concerning the design of the walls. 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.

- Select, free-draining granular fill meeting the specifications of Special Provision (SP) 110S13 (Aggregates) Granular ‘A’ or Granular ‘B’ Type II but with less than 5 percent passing the 200 sieve should be used as backfill behind the walls. 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) and OPSD 3121.150 (Walls, Retaining, Backfill).
- For structures that are not comprised of integral or semi-integral abutments, rock fill may be used as backfill behind the walls and the material should meet the specification as outlined in the Northeastern Region Directive for backfill to structures adjacent to rock embankments. Other aspects of rock backfill requirements should be in accordance with OPSD 3101.200 (Walls, Abutment, Backfill, Rock).

- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the wall stem, in accordance with *CHBDC* Section 6.9.3 and Figure 6.6. Compaction equipment should be used in accordance with OPSS 501 (Compacting). Other surcharge loadings should be accounted for in the design as required.

- For the abutment/wing walls, fill should be placed within the wedge shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical (1.5H:1V) extending up and back from the rear face of the footing (in accordance with Figure C6.20(b) of the *Commentary* to the *CHBDC*). The pressures are based on the fill as placed and the following parameters (unfactored) may be assumed:

Fill Type	**Fill Unit Weight (kN/m^3)**	**Coefficients of Static Lateral Earth Pressure**	
		At-Rest, Ko	Active, Ka
Granular 'A'	22	0.43	0.27
Granular 'B' Type II	21	0.43	0.27

- Where lightweight fill (EPS) is installed behind the abutment wall, the pressure acting over the depth of the EPS may be calculated using the following parameters (unfactored):

Fill Type	**Fill Unit Weight (kN/m^3)**	**Coefficients of Static Lateral Earth Pressure**	
		At-Rest, Ko	Active, Ka
EPS	0.5	0.11	0.11

If the wall support and superstructure allow lateral yielding of the stem, active earth pressures may be used in the geotechnical design of the structure. If the abutment support does not allow lateral yielding, at-rest earth pressures should be assumed for geotechnical design. The movement required to allow active pressures to develop within the backfill, and thereby assume an unrestrained structure for design, should be calculated in accordance with Section C6.9.1 and Table C6.6 of the *Commentary* to the *CHBDC*.

Restrained structures are typically concrete box culverts or rigid frame bridge structures where the rotational and/or horizontal movement is not sufficient to mobilize the active pressure condition. For this condition, an at-rest pressure plus any compaction surcharge should be included in the design of the structure.

Seismic (earthquake) loading must also be taken into account in the design in accordance with Section 4.6 of the *CHBDC*. In this regard, the following should be included in the assessment of lateral earth pressures:

- Seismic loading will result in increased lateral earth pressures acting on the abutment stem and/or retaining walls. The walls should be designed to withstand the combined lateral loading for the appropriate static pressure conditions given above, plus the earthquake-induced dynamic earth pressure. According to the National Building Code of Canada (1995) seismic hazard values (as referenced in the CHBDC and its *Commentary*), the site specific peak horizontal ground acceleration (PHA) for the Sault Ste. Marie area is 0.03 (for a probability of exceedance of 10 percent in 50 years). For the thicknesses and type of overburden soils at this site, an amplification factor of 2.0 (i.e. S = 2.0 for the Flyover East location) and an amplification factor of 1.5 (i.e. S = 1.5 for the Flyover West, Flyover West Alternative 5 and Highway 638 Interchange locations), of the ground motion is recommended for design. As such, the ground surface acceleration would be 0.06 at Flyover East and 0.045 at Flyover West, Flyover West Alternative 5 and Highway 638 Interchange.

- Based on the above, according to Table C4.2 of the *Commentary* to the *CHBDC*, this site would be located in Seismic Performance Zone 1 and the corresponding site specific Zonal Acceleration Ratio, A, would be 0.05. The seismic lateral earth pressure coefficients given below have been derived based on a design zonal acceleration ratio of A = 0.05.

- In accordance with Sections 4.6.4 and C.4.6.4 of the *CHBDC* and its *Commentary*, for structures which allow lateral yielding, the horizontal seismic coefficient, k_h, used in the calculation of the seismic active pressure coefficient, is taken as 0.5 times the zonal acceleration ratio (i.e. k_h = 0.5(A) = 0.025 at this site). For structures that do not allow lateral yielding, k_h is taken as 1.5 times the zonal acceleration ratio (i.e. k_h = 1.5(A) = 0.075 at this site). The seismic active earth pressure coefficient is also dependent on the vertical component of the earthquake acceleration, k_v. Three discrete values of vertical acceleration are typically selected for analysis, corresponding to $k_v = +2/3\ k_h$, $k_v = 0$, and $k_v = 2/3\ k_h$.

- The following seismic active pressure coefficients (K_{AE}) for unrestrained walls (in accordance with Figure C6.20(b) of the *Commentary* to the CHBDC) may be used in design; these coefficients reflect the maximum K_{AE} obtained using the k_h and three values of k_v as described above. It should be noted that these seismic earth pressure coefficients assume that the back of the wall is vertical and the ground surface behind the wall is level.

Seismic Active Pressure Coefficients, K_{AE}

Wall Type	Fill Type	
	Granular 'A'	Granular 'B' Type II
Yielding Wall	0.26	0.26
Non-Yielding Wall	0.29	0.29

- Where lightweight fill (EPS) is installed behind the abutment wall(s), the following seismic active pressure coefficients (K_{AE}) may be used for design.

Seismic Active Pressure Coefficients, K_{AE}

Wall Type	Fill Type
	Lightweight Fill (EPS)
Yielding Wall	0.07
Non-Yielding Wall	0.08

- The above K_{AE} values for yielding walls are applicable provided that the wall can move up to 250A (mm), where A is the site specific Zonal Acceleration Ratio of 0.05. This corresponds to displacements of up to 13 mm at this site.

- The earthquake-induced dynamic pressure distribution, which is to be added to the static earth pressure distribution, is a linear distribution with maximum pressure at the top of the wall and minimum pressure at its toe (i.e. an inverted triangular pressure distribution). The total pressure distribution (static plus seismic) may be determined as follows:

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

Where: $\sigma_{h(z)}$ is the lateral earth pressure at depth 'z' (kPa);

K is either the static active earth pressure coefficient (K_a) or the static at-rest earth pressure coefficient (K_o);

K_{AE} is the seismic earth pressure coefficient;

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

z is the depth below the top of the wall (m); and,

H is the total height of the wall (m).

6.1.6 Flyover East

6.1.6.1 General

It is understood that the current preliminary design for the proposed Bar River Road Flyover East structure alternative, located immediately north of the existing Bar River Road alignment, consists of a two-span structure supported on pile foundations with a central pier located near the Highway 17 centreline and abutments located to the east and west of the current Highway 17 alignment.

6.1.6.2 Foundation Options

The recommendations for the deep foundation options for the Flyover East structure are presented in the following sections. A summary of the advantages, disadvantages, relative costs and risks/consequences for the structure foundation alternatives is given in Table A2 in Appendix A. The preferred foundation system will be governed by structural design considerations, however supporting the structure on steel H-Piles driven to refusal within the sand and gravel to gravelly sand deposit underlying the very thick clay deposit, is considered feasible and the preferred alternative for this site. If at the Detail Design stage the depth to bedrock at each of the foundation elements is confirmed essentially at or near the depth at which refusal condition was encountered during the Preliminary Design than the piles could be founded on bedrock.

6.1.6.2.1 Friction Piles Axial Geotechnical Resistance

Given the relatively low shear strength of the cohesive deposits underlying the site, friction piles would have to be driven through the upper very soft to stiff clayey strata and into the lower stiff to very stiff clayey stratum, resulting in piles about 35 m to 40 m long.

For steel HP310x110 piles or HP310x79 piles or standard MTO 324 mm diameter 6.4 mm thick wall (12 ¾ in x ¼ in) concrete filled, steel tube piles driven approximately to mid-depth into the lower stiff to very stiff clayey stratum (approximately 35 m to 40 m below ground surface), based on calculation of resistance using Meyerhoff (1976), and CFEM (2006), a factored geotechnical axial resistance at ULS of 650 kN may be assumed for preliminary design. The geotechnical resistance at SLS for 10 mm of settlement (for this length of pile) is estimated to be about 900 kN.

It should be noted that given the variable depth to the stiff clayey stratum at the borehole and two CPTs advanced at the flyover location, a structure specific foundation investigation and pile design will be required at the detail design stage to confirm the required pile lengths and available pile capacities for friction piles at each foundation unit.

If friction piles are adopted for support of the structure at this location, it is recommended that at least one pile load test be performed to verify the design pile capacity.

6.1.6.2.2 End-bearing Piles Axial Geotechnical Resistance

Steel H-Piles driven to refusal within the lower granular layer (below the thick clayey stratum) or on bedrock are considered to be the preferred foundation alternative, from a foundations perspective, for support of the Flyover East structure location. Based on the boreholes completed to date, this could result in piles about 50 m to 55 m long.

For HP 310 x 110 piles, end reinforced with flange plates and web stiffeners, driven to practical refusal within the granular layer, a factored axial resistance at ULS of 1,500 kN may be assumed for design. If at the detail design stage, the bedrock surface is confirmed at or close to the DCPT refusal depth, then the piles could be founded in bedrock. In this case, a factored geotechnical axial resistance at ULS of 2,000 kN may be used for design. The geotechnical resistance at SLS for 25 mm of settlement (for the length of piles required at this site) will be greater than the factored axial resistance at ULS; as such, ULS conditions will govern for this foundation type.

6.1.7 Flyover West

6.1.7.1 General

It is understood that the current preliminary design for the proposed Bar River Road Flyover West structure located approximately 800 m north of the existing Bar River Road alignment consists of a two-span structure supported on pile foundations with a central pier located near the Highway 17 centreline and abutments located to the east and west of the current Highway 17 alignment.

6.1.7.2 Foundation Options

The recommendations for the deep foundation options for the Flyover West structure alternative are presented in the following sections. A summary of the advantages, disadvantages, relative costs and risks/consequences for the structure foundation alternatives is given in Table B2 in Appendix B. The preferred foundation system will be governed by structural design considerations, however supporting the structure on steel H-Piles driven to refusal on bedrock, is considered feasible and the preferred alternative for this site, subject to confirmation at the detail design stage of the depth to bedrock at each foundation element.

6.1.7.2.1 Friction Piles Axial Geotechnical Resistance

Given the relatively low shear strength of the cohesive deposits underlying the site, friction piles would have to be driven through the upper soft to firm clayey strata and at least midway into the underlying compact to very dense sand to gravelly silty sand deposit. Based on the boreholes completed to date, this could result in piles at least about 20 m long.

For steel HP310x110 piles or standard MTO 324 mm diameter 6.4 mm thick wall (12 ¾ in x ¼ in) concrete filled, steel tube piles driven approximately to mid-depth into the compact to very dense sand to gravelly silty sand deposit (approximately 20 m below ground surface), based on calculation of resistance using Meyerhoff (1976) and CFEM (2006), tempered by the results of a pile load test for Site #40 (MTO Foundation Section, 1993), a factored geotechnical axial resistance at ULS of 900 kN may be assumed for preliminary design. The geotechnical resistance at SLS for 10 mm of settlement (for this length of pile) is estimated to be about 800 kN.

If friction piles are adopted for support of the structure at this location, it is recommended that at least one pile load test be performed to verify the design pile capacity.

6.1.7.2.2 End-bearing Piles Axial Geotechnical Resistance

Steel H-Piles driven to refusal on bedrock, underlying the sand to gravelly silty sand deposits, are considered to be the preferred foundation alternative, from a foundations perspective, for support of the Flyover West structure. Based on the boreholes completed to date, this could result in piles about 25 m to 30 m long. It should be noted that boulders up to about 0.5 m in size were encountered within the sand and gravelly silty sand soil deposits. Given the presence of boulders, it is recommended that a heavy pile section (HP310x132) be utilized to minimize damage during driving. In addition, it is recommended that the ends of the piles be reinforced with flange plates and web stiffeners to further mitigate against damage during driving.

For HP 310x110 piles or HP 310x132 piles, end reinforced with flange plates and web stiffeners, driven to refusal on bedrock, a factored axial resistance at ULS of 2,000 kN or 2,400 kN, respectively may be assumed for design. The geotechnical resistance at SLS for 25 mm of settlement (for the length of piles required at this site) will be greater than the factored axial resistance at ULS; as such, ULS conditions will govern for this foundation type.

6.1.8 Flyover West Alternative 5

6.1.8.1 General

It is understood that the current preliminary design for the proposed Bar River Road Flyover West Alternative 5 structure located approximately 1,150 m north of the existing Bar River Road alignment consists of a two-span structure supported on pile foundations with a central pier located near the Highway 17 centreline and abutments located to the east and west of the current Highway 17 alignment.

6.1.8.2 Foundation Options

The recommendations for the deep foundation options for the Flyover West Alternative 5 structure alternative are presented in the following sections. A summary of the advantages, disadvantages, relative costs and risks/consequences for the structure foundation alternatives is given in Table C2 in Appendix C. The preferred foundation system will be governed by structural design considerations, however supporting the structure on steel H-Piles driven to refusal on bedrock is considered feasible and the preferred alternative for this site, subject to confirmation at the detail design stage of the depth to bedrock at each foundation element.

6.1.8.2.1 Friction Piles Axial Geotechnical Resistance

Given the relatively low shear strength of the cohesive deposits underlying the site, friction piles would have to be driven through the upper soft to firm clayey strata and at least midway into the underlying compact to very dense sand to sand and gravel to gravelly silty sand deposit. Based on the nearest existing boreholes, this could result in piles at least about 17 m long.

For steel HP310x110 piles or standard MTO 324 mm diameter 6.4 mm thick wall (12 ¾ in x ¼ in) concrete filled, steel tube piles driven approximately to mid-depth into the compact to very dense sand to sand and gravel to gravelly silty sand deposit (approximately 17 m below ground surface), based on calculation of resistance using Meyerhoff (1976) and CFEM (2006), tempered by the results of a pile load test for Site #40 (MTO Foundation Design Section, 1993), a factored geotechnical axial resistance at ULS of 800 kN may be assumed for preliminary design. The geotechnical resistance at SLS for 10 mm of settlement (for this length of pile) is estimated to be about 700 kN.

If friction piles are adopted for support of the structure at this location, it is recommended that at least one pile load test be performed to verify the design pile capacity.

6.1.8.2.2 End-bearing Piles Axial Geotechnical Resistance

Steel H-Piles driven to refusal on bedrock, underlying the sand to sand and gravel to gravelly silty sand deposits, are considered to be the preferred foundation alternative, from a foundations perspective, for support of the Flyover West Alternative 5 structure. Based on interpolation between the available adjacent existing information, this could result in piles about 20 m to 25 m long. Given the expected presence of cobbles and boulders within the cohesionless deposits overlying the bedrock, it is recommended that a heavier pile section (HP310x132) be utilized to minimize damage during driving. In addition, it is recommended that the ends of the piles be reinforced with flange plates and web stiffeners to further mitigate against damage during driving.

For HP 310x110 piles or HP 310x132 piles, end reinforced with flange plates and web stiffeners, driven to refusal on bedrock, a factored axial resistance at ULS of 2,000 kN or 2,400 kN, respectively, may be assumed for design. The geotechnical resistance at SLS for 25 mm of settlement (for the length of piles required at this site) will be greater than the factored axial resistance at ULS; as such ULS conditions will govern for this foundation type.

6.1.9 Highway 17/Highway 638 Interchange

6.1.9.1 General

It is understood that the current preliminary design for the new Highway 638 Interchange structure consists of a two-span structure supported on pile foundations with a central pier located near the Highway 17 centreline and abutments located to the east and west of the current Highway 17 alignment.

6.1.9.2 Foundation Options

The recommendations for the deep foundation options for the interchange structure are presented in the following sections. A summary of the advantages, disadvantages, relative costs and risks/consequences for the structure foundation alternatives is given in Table D2 in Appendix D. The preferred foundation system will be governed by structural design considerations, however supporting the structure on steel H-Piles driven to refusal in the anticipated granular deposits below the cohesive soil strata or on bedrock is considered feasible and preferred alternative for this site, subject to confirmation at the detail design stage of the depth to bedrock/refusal at each of the foundation elements.

6.1.9.2.1 Friction Piles Axial Geotechnical Resistance

Given the loose to compact relative density of the near surface cohesionless deposits and firm to stiff consistency of the upper cohesive deposits underlying the site, friction piles would have to be driven through these strata and approximately to mid-depth into the lower stiff to very stiff clay stratum. Based on the boreholes completed to date, this could result in piles about 40 m to 45 m long.

For steel HP310x110 piles or HP310x79 piles or standard MTO 324 mm diameter 6.4 mm thick wall (12 ¾ in x ¼ in) concrete filled, steel tube piles driven approximately to mid-depth into the lower stiff to very stiff clay stratum, a factored axial geotechnical resistance at ULS of 900 kN may be assumed for preliminary design.

The geotechnical resistance at SLS for 10 mm of settlement (for this length of pile) is estimated to be about 900 kN.

It should be noted that given the variable depth to the stiff to very stiff clay stratum at the boreholes and CPTs advanced at the interchange location, a structure specific foundation investigation and pile design will be required at the detail design stage to check the required pile lengths and available pile capacities for friction piles at each foundation unit.

If friction piles are adopted for support of the structure at this location, it is recommended that at least one pile load test be performed to verify the design pile capacity.

6.1.9.2.2 End-bearing piles Axial Geotechnical Resistance

Steel H-Piles driven to refusal in the anticipated granular deposits below the clay stratum or on bedrock are considered to be the preferred foundation alternative, from a foundations perspective, for support of the interchange structure. Based on the boreholes completed to date, this could result in piles about 55 m to 60 m long. Refusal was only encountered in one borehole advanced at the interchange location and, as such, it will be critical to carry out additional investigation within the footprint of each foundation element at the detail design stage to confirm the required pile lengths.

For HP 310 x 110 piles, end reinforced with flange plates and web stiffeners, driven to practical refusal into the anticipated granular deposits below the clay stratum, a factored geotechnical axial resistance at ULS of 1,500 kN may be assumed for design. If at the detail design stage, the bedrock surface is confirmed at or close to the DCPT refusal depth, then the piles could be founded in bedrock. In this case, a factored geotechnical axial resistance at ULS of 2,000 kN may be used for design. The geotechnical resistance at SLS for 25 mm of settlement (for the length of piles required at this site) will be greater than the factored axial resistance at ULS; as such, ULS conditions will govern for this foundation type.

6.2 Approach Embankment Design

Based on the preliminary vertical alignment profiles provided by GENIVAR, the proposed approach embankments for the Flyover East, Flyover West, or Flyover West Alternative 5 and for the Highway 638 Interchange will be up to about 9.4 m high above the existing grade.

Sections 6.2.2 and 6.2.3 of this report summarize the methods used for the analysis of stability and settlement for critical sections of the approach embankments for the new structures. Section 6.3 provides a general discussion and recommendations related to potential alternatives for mitigating stability and settlement-related design and construction issues. The results of the analyses and recommendations on mitigating stability and time-dependent settlements of the approach embankments are presented for each structure in Section 6.4.

6.2.1 Embankment Fill Types and Berm Requirements

Different embankment fill alternatives (i.e. rock fill and granular fill) provide relative advantages and disadvantages in terms of availability, weight (i.e. driving force and applied load to founding subsoils), construction cost and time, ease of construction and post-construction performance.

It is understood that granular fill is the preferred embankment fill material for this project and as such, the stability and settlement analyses discussed in Section 6.2.2 and 6.2.3 have been carried out on the basis that the majority of the roadway embankments will be constructed of granular fill. For granular fill embankments, 2 m wide berms should be incorporated into the side slope profiles so that no uninterrupted slope is greater than 8 m high (N.R. Geotechnical Section, 2005).

6.2.2 Stability

Analyses were carried out on the critical (i.e. highest or thickest fill) sections of the proposed new approach embankments to assess the stability for the proposed heights and geometries. Critical sections include those through the side slopes and front slopes of the new approaches. The following sections outline the methodology used to evaluate embankment stability at the critical locations. In addition, the parameters used in the analyses for each of the critical section(s) are also presented. The results of the analyses are presented in Section 6.4.1.1.

6.2.2.1 Methodology

Limit equilibrium slope stability analyses were performed using the commercially available program Slide (version 6.00), produced by Rocscience Inc., employing the Morgenstern-Price method of analysis. For all analyses, the factors of safety of numerous potential failure surfaces were computed in order to establish the minimum factor of safety. The stability analyses were performed to check that a target minimum factor of safety of 1.3 was achieved for the proposed embankment heights and geometries. In general, circular slip surfaces were analysed in the design.

The stability analyses assume that all organic soils will be removed prior to construction of the new embankments and that granular fill (i.e. Granular 'B' Type I) will be used for replacement of sub-excavated material (as discussed in Sections 6.3.1 and 6.5.3). The piezometric conditions required in the analyses were based on observations during drilling and CPT testing, which generally indicate that the groundwater is located at about the level of the natural ground surface. The stability analysis was carried out assuming a 2H:1V side slope profile for the earth fill embankments.

Both total stress (undrained) and effective stress (long-term and short-term with excess pore pressure development estimated using the B-bar method) analyses were performed to assess the maximum height of embankment that could be constructed while still maintaining a Factory of Safety of 1.3, and to double-check the minimum size of stability berms, if required. The results of the effective stress analysis were compared to and used as an indicator of the suitability of the strength parameters selected for and the results of the total stress analysis.

6.2.2.2 Parameter Selection

The simplified stratigraphies together with the associated strength and unit weights employed for the different soil types at the critical sections for each structure location are summarized for all soil layers in Tables A1, B1, C1 and D1 and are plotted for the cohesive deposits on Figures A1, B1, C1 and D1 in Appendices A to D for the four structure locations. The earth fill modeled in the analyses is assumed to have a unit weight of 21 kN/m^3 and an effective friction angle of 32^o with the embankments constructed with 2H:1V side slopes.

The subsoils encountered in the various areas are composed of a combination of cohesive deposits (clayey silt, silty clay and clay) and granular deposits (silt, sand and gravel). For granular soils, effective stress parameters were employed in the analyses assuming drained conditions. The effective stress parameters (effective friction angle) for the granular soils were estimated from empirical correlations using the results of in-situ Standard Penetration Tests (SPT) as suggested by US Navy (1986) and Kulhawy and Mayne (1990), in conjunction with engineering judgement based on experience in similar soil conditions.

For cohesive deposits, total stress parameters were employed in the undrained analyses while effective stress parameters (with excess pore pressure development estimated using a B-bar equal to 1.0) were employed in the short-term effective stress analyses.

The total stress parameters (i.e. average mobilized undrained shear strength – s_u) for the cohesive soils were assessed based on the results of in situ field vane shear tests and inferred from estimates of preconsolidation stress (σ_p', kPa) from the in situ CPT tests (as described in Section 6.2.3.2), by employing the following correlation proposed by Mesri (1975):

$$s_u = 0.22\sigma_p'$$

where:
s_u = average mobilized undrained shear strength (kPa)
σ_p' = preconsolidation stress (kPa)

Where appropriate, Bjerrum's correction factor was employed to estimate the average mobilized undrained shear strength from the results of the in situ field vane tests as follows:

$$s_{u(mob)} = \mu s_{u(FV)} \quad \text{(after Bjerrum, 1973)}$$

where:
$s_{u(mob)}$ = average mobilized undrained shear strength (kPa)
$s_{u(FV)}$ = undrained shear strength from field vane test (kPa)
μ = Bjerrum's correction factor based on Plasticity Index

The effective stress parameters (i.e. c'=0 kPa and ϕ') for the cohesive soils were estimated from empirical correlations using the results of the laboratory index testing as suggested by Ladd (1977) and Mitchell (1993) in conjunction with engineering judgement based on experience in similar soil conditions.

6.2.3 Settlement

The following sections outline the methods used to assess parameters and carry out settlement analyses for each of the critical section(s) of the various approach embankments. The results of the analyses are presented in Section 6.4.1.2, 6.4.2.2 and 6.4.3.2.

6.2.3.1 Methodology

To estimate the magnitude of the expected settlements, analyses were carried out at the critical sections of the proposed approach embankments using the commercially available program Settle^{3D} (Version 2.0) produced by Rocscience Inc. combined with hand/spreadsheet calculations, where appropriate. Critical sections correspond to the greatest new embankment height. The rate of settlement/consolidation of the cohesive foundation soils was assessed using Terzaghi's one-dimensional consolidation theory.

The sources of settlement were considered to include:

- primary time-dependent consolidation of the cohesive deposits;

- secondary time-dependent (creep) consolidation of the cohesive deposits (long-term); and
- immediate settlement of the granular foundation soils.

The height of the approach embankments vary along the alignment at each structure location. Given that the analyses were carried out at the critical sections of each approach embankment, the settlements estimated will generally represent the maximum value at each structure, however, it is noted that analysis was also carried out at several points along the embankments in order to quantify the details and extents of the various settlement mitigation measures evaluated.

The settlement analyses assume that any surficial or near surface organic soils of significant thickness (greater than about 0.1 m), as well as any existing fill and near surface very soft clayey soils (as discussed in Sections 6.3.1, 6.5.2 and 6.5.4) will be removed prior to construction of the new embankments and that earth fill (SP 110S13 Granular B Type 1) will be used for replacement of sub-excavated material. The piezometric conditions required in the analyses were based on the groundwater level indicated in Section 4.5.5 which was essentially located at about the level of the natural ground surface at most locations.

6.2.3.2 Parameter Selection

The simplified stratigraphies together with the associated deformation and time-rate consolidation parameters employed for the different native soil types for the critical sections at each structure location are given in Tables A1, B1, C1 and D1 for the Flyover East, Flyover West, Flyover West Alternative 5 and Interchange, respectively.

The immediate compression of the very loose to very dense sand, silt and gravel layers was modeled by estimating an elastic modulus of deformation based on the SPT 'N'-values and using correlations proposed by Bowles (1984) and Kulhawy and Mayne (1990). These estimated values were compared with the typical range of expected values for similar soil types, as outlined in CHBDC (2006) and adjusted, as necessary.

The consolidation settlement of the cohesive deposits was assessed using the results of in situ field vane and CPT test data and the results of the laboratory index testing using empirical correlations proposed in literature by Koppula (1986), Terzaghi and Peck (1967), Kulhawy and Mayne (1990) and Azzouz et al. (1976).

The following correlation relating in situ undrained shear strength to preconsolidation stress (Mesri, 1975) was employed:

$$\sigma_p' = \frac{s_{u(mob)}}{0.22}$$

where :

$s_{u(mob)} = \mu s_{u(FV)}$
σ_p' = preconsolidation stress (kPa)
$s_{u(mob)}$ = average mobilized undrained shear strength (kPa)
$s_{u(FV)}$ = undrained shear strength from field vane test (kPa)
μ = Bjerrum's correction factor based on Plasticity Index

The preconsolidation stress was also estimated from the results of the CPTs (Demer and Leroueil, 2002):

$$\sigma_p' = \frac{q_t - \sigma_{vo}}{3.4}$$

where	q_t	=	$q_c - u_2(1 - A_n)$ (kPa)
	q_c	=	tip stress measured by the CPT (kPa)
	u_2	=	pore pressure measured at cone 'shoulder' (kPa)
	A_n	=	cone constant
	σ_{vo}	=	total vertical stress (kPa)

The recompression index (C_r) and compression index (C_c) for the cohesive deposits was evaluated based on the results of the laboratory consolidation tests. The results from the consolidation tests were supplemented with estimates of C_c based on the Atterberg limits and water content testing using the following empirical correlations:

$$C_c = 0.009w_n + 0.005w_L \qquad \text{(Koppula, 1986)}$$

where	w_n	=	natural water content (percent, %)
	w_L	=	liquid limit (percent, %)

$$C_c = 0.009(w_L - 10) \qquad \text{(Terzaghi and Peck, 1967)}$$

where	w_L	=	liquid limit (percent, %)

$$C_c = \frac{PI}{74} \qquad \text{(Kulhawy and Mayne, 1990)}$$

where	PI	=	plasticity index (percent, %)

Based on previous experience with the clayey soils in the area, an approximate ratio between the compression index and the recompression index of 10 (i.e. $C_r = 0.1\ C_c$) was utilized to estimate recompression index (c_r) from the above correlations.

The values of the coefficient of consolidation in the horizontal direction (c_h) were assessed from the results of the pore pressure dissipation tests carried out as part of the CPT testing at each structure location. A total of seven (7) pore pressure dissipation tests were carried out and the results are shown on Figures A2, B2 and D2 in Appendices A, B and D for the three structure locations where the CPTs were advanced. Based on these data, c_h was estimated using the following method proposed by Robertson et al. (1992):

$$C_h = \left(\frac{m}{M}\right)^2 \sqrt{I_r r^2}$$

where: m = gradient of the initial linear portion of the CPT pore pressure dissipation curve

M = 1.15 (for CPT pore pressure sensor at position u_2)

I_r = rigidity index, $\frac{G}{s_u}$ (ranges from about 90 to 100 for these sites)

r = radius of CPT probe (17.8 mm)

The gradient of the initial portion of the dissipation curves (m) selected to represent the estimated average horizontal coefficient of consolidation was based on the average value of the gradient of the initial linear portion of the CPT pore pressure dissipation from all the CPT dissipation tests carried out in the cohesive deposit at a particular structure location. The gradient of the initial linear portion used to calculate the estimated average horizontal coefficient of consolidation is shown as the 'mean' line on Figures A2, B2 and D2. The estimated range(s) and where applicable, average values of the coefficient of consolidation in the horizontal direction (c_h) obtained from the CPT pore pressure dissipation tests are summarized below.

Location	**Estimated C_h from In Situ CPT Dissipation Tests (cm^2/s)**		
	Lowerbound	**Upperbound**	**Average**
Flyover East	1.3×10^{-3}	2.2×10^{-3}	-
Flyover West	-	-	4.3×10^{-3}
Highway 17/638 Interchange	2.1×10^{-3}	4.4×10^{-3}	-

In addition to primary consolidation within clays, secondary compression may also occur. Secondary compression is referred to as creep settlement and occurs over a long period of time, after substantial dissipation of excess pore pressure under a constant stress. The following relationships have been employed for estimating the magnitude of creep settlement for each log cycle of time following completion of primary settlement at each location.

$$S_c = HC_{\alpha\varepsilon}\log\left(\frac{t}{t_{EoP}}\right)$$

$$C_{\alpha\varepsilon} \approx \frac{w_n}{10,000}$$ (after Mesri, 1973)

where :
- S_c/log cycle = secondary consolidation (creep) settlement (mm)
- $C_{\alpha\varepsilon}$ = modified secondary compression index
- H = initial thickness of compressible clay deposit (mm)
- w_n = average natural water content (%)
- t = time period of interest (or design life of structure) (years)
- t_{EoP} = time to reach 90% primary consolidation (years)

6.2.3.3 *Settlement of New Granular Embankment Fill*

For the granular material proposed to be employed for the construction of the new approach embankments at this site, very little additional settlement due to compression of the embankment fill itself is expected to occur over and above the estimated settlement of the foundation soils. In this case, the additional settlement from properly compacted granular fills is expected to be less than about 25 mm and will occur during construction.

6.3 Stability and Settlement Mitigation Options

At the approach embankments for each structure alternative, stability and settlement have been assessed based on existing subsurface conditions and proposed embankment fill heights. The presence of the weak and thick compressible soils underlying the proposed embankments results in the potential for instability and unacceptably large settlements due to the placement of fills. There are a number of options for mitigating the potential for settlements and instability. A brief discussion on these alternatives is given below.

Details of the foundation options for each of the structure locations to mitigate embankment stability and settlement issues are provided in Section 6.4.1.3, and the advantages, disadvantages, relative costs and risks/consequences are summarized in Tables A3, B3, C3 and D3 in Appendices A to D, for the Flyover East, Flyover West, Flyover West Alternative 5 and Interchange, respectively.

6.3.1 Partial Sub-Excavation and Replacement

A partial subexcavation of the near surface weak and compressible soils underlying the footprint of the proposed embankments in advance of the placement of fill is a viable option for improving the stability of the proposed embankments at these sites. A partial replacement of the near surface very soft, compressible cohesive soils, with granular fill would result in improved stability within the areas underlain by deep cohesive deposits, in particular at the Flyover East site. This option has the advantage that construction of the above-grade embankment could proceed with a greater initial fill lift height, with a lower risk of instability. In cases where wick drains are combined with sub-excavation and replacement, it is important that the type of fill placed below grade will not prevent or obstruct the subsequent installation of the wick drains (i.e. rock fill or fill with cobbles and boulders should not be used).

Due to the great depths of soft clayey silt to clay deposits encountered at these sites (ranging from about 13.5 m to up to about 50 m below ground surface), full sub-excavation and replacement of the soft material at the embankment locations is not considered to be a practical or feasible alternative, however partial sub-excavation and replacement may be used, as recommended at specific locations in Sections 6.4.1.3, 6.4.2.3 6.4.3.3 and 6.4.4.3, to improve the embankment stability.

The advantages of this option are:

- Improved stability;
- Reduced delay in construction or reduced number of fill stages (staged construction may still be required); and,
- Reduction in the size of stabilizing toe berms.

The disadvantages of this option are:

- Generation of large volume of excavation spoil requiring disposal/management;
- Greater quantities of fill required.

6.3.2 Preloading With Stabilizing Toe Berms

For areas where cohesive deposits are thick and/or soft, as is the case at these sites, and given that these conditions coincide with the proposed high embankment fills, stability berms constructed along the embankment toes will improve the stability of the embankments. The height of toe berms is typically on the order of about one third to one half of the height of the final embankment. The lateral extent (width) of the toe berms will vary depending on the actual stratigraphy and shear strength of the foundation soils at each site and the results of the stability analyses.

At these sites, this option should be combined with partial sub-excavation and replacement of the soft soils, and/or staged construction which will result in smaller toe berms required to maintain the embankment stability.

The advantages of this option are:

- Higher initial fill stages are achievable or (depending on subsurface conditions), potential to construct to full embankment height in one stage.

The disadvantages of this option are:

- Increased quantity of fill;
- Increased primary settlement due to additional volume of fill and wider loading area; and
- Potentially increased Right-of-Way (ROW) requirements.

It is noted that surcharging with stabilizing toe berms is not considered a feasible or practical alternative a these sites given the weak subsurface conditions and the large width(s) of toe berms required to maintain embankment stability under preload fill heights. If embankment surcharging was to be considered, even larger (and likely impractical) toe berms would be required.

6.3.3 Wick Drains (with Staged Construction)

Where full sub-excavation of the soft, compressible deposits is not practical (i.e. due to the thickness of or depth to the bottom of the compressible soil deposits), consideration can be given to installing wick drains (in conjunction with staged construction and/or partial sub-excavation and/or stabilizing berms) to accelerate the rate of primary consolidation settlement and to facilitate shear strength gain in the clayey strata. Wick drains are pre-fabricated geotextile drains installed vertically from ground surface into or through the soft, compressible foundation soils in order to increase the rate of excess pore pressure dissipation and consolidation. Typically, wick drains are installed on a 1 m to 3 m triangular grid spacing over the footprint of the embankment and toe berm(s).

Use of wick drains are most suited to areas with thick (i.e. greater than about 5 m) deposits of soft, compressible foundation soils and/or high proposed embankment fills and where staged construction is required and where primary consolidation times (without wick drains) are expected to be large.

At these sites, it will still be beneficial to sub-excavate and remove a portion of the near surface soft, weak foundation soils followed by placement of a granular drainage blanket at the ground surface level prior to the installation of the wick drains.

The advantages of this option are:

- Substantially decreased time for primary consolidation; and,
- Increased rate of staged construction (to maintain stability) during construction.

The disadvantages of this option are:

- Additional time and expense to install wick drains prior to embankment construction;
- At these sites, a specialist contractor may be required to install the very deep/long wick drains (i.e. greater than about 30 m and up to as much as about 48 m) and possibly to penetrate the thick surficial cohesionless deposits that are present above the cohesive layers (in particular at the interchange);
- Additional long-term settlements due to secondary consolidation (i.e. creep settlement) of the cohesive layer (if not compensated for by top-up with lightweight expanded polystyrene (EPS) fill); and,
- Additional expense associated with implementation of an instrumentation and monitoring program to assess when sufficient excess pore pressure has dissipated and required strength increase has been achieved to allow construction to proceed.

6.3.4 Lightweight Fill

Another alternative for reducing the magnitude of long-term settlement and improving stability in areas of soft, compressible foundation soils is to use lightweight fill, such as expanded polystyrene (EPS), for embankment construction.

The use of lightweight fill reduces the load applied to the foundation soils due to the low density of the fill materials. This in turn reduces the magnitude of post-construction settlement and reduces the potential for instability. Lightweight fill can be used in place of part of the embankment fill to increase the stability of the embankment, or in place of all of the embankment fill to increase the embankment stability and eliminate most of the primary and long-term settlements.

The advantages of this option are:

- Improved stability;
- Reduced post-construction settlements;
- No significant delay in construction; and,
- Reduced width or possible elimination of the need for stabilizing toe berms.

The disadvantages of this option are:

- Significant additional expense of lightweight fill (depending on the volume required).

6.3.5 Instrumentation and Monitoring

For areas where the preloading with staged construction and/or wick drains foundation options are adopted, the magnitude and time rate of settlement as well as dissipation of pore pressures during and after construction of embankments should be assessed with monitoring instrumentation. Such monitoring would consist of installing settlement pins/stakes (Ss), settlement plates (SPs) and vibrating wire piezometers (VWPs) below the embankment and taking regular measurements/readings at given intervals of time during and after construction of the embankment for the duration of the preloading period. In addition, standpipe piezometers (SPPs) may be required and are usually installed to provide background pore pressure readings for the vibrating wire piezometers. This monitoring instrumentation is particularly important where it is necessary to carefully monitor the stability of the subsoils during staged placement of fill.

The extent of instrumentation and the frequency of monitoring required will depend on the foundation treatment alternative chosen for a given site and the height of the proposed embankment fill. Specifications for the type, number and layout of the instrumentation, together with the supply, installation, protection and monitoring should be included as Special Provisions in the Contract Documents.

6.4 Results of Analysis

The results of the stability and settlement analyses for the approach embankments at each of the structure locations are provided in the following sections. In addition, the options and recommendations for achieving the target factor of safety for the required embankment geometry and for minimizing/mitigating the time dependent, post-construction settlements are also discussed. The advantages, disadvantages, relative costs, and risks / consequences for the various alternatives for approach embankment construction for each of the sites are summarized and ranked in Tables A3, B3, C3 and D3 in Appendices A to D.

Given that the foundation soils at each of the structure locations are comprised of thick and soft cohesive subsoils, time-dependent settlements of the new embankments are expected. In addition, the presence of the weak/soft cohesive deposits constitute zones of potential instability of the proposed embankments. As such, consideration must be given to an enhanced design and/or to follow a construction sequence that will achieve the minimum target Factor of Safety of 1.3 for the proposed new embankment heights and geometries and limit the post-construction settlements and subsequent maintenance requirements on the new roadways.

6.4.1 Flyover East

The preliminary design for the proposed Flyover East location requires new embankments up to about 8.6 m high to achieve the required vertical roadway profile. The natural topography of this area of the site is flat and low-lying with ground surface at about Elevation 182 m.

In general, the subsurface conditions at the proposed Flyover East location consist of approximately 47 m of very soft to very stiff clayey strata, underlain by a sand and gravel to gravelly sand deposit. Borehole 10-2 was advanced through the cohesive strata and into the underlying granular deposits to a depth of 50.8 m, and a dynamic cone penetration test (DCPT) was advanced through the bottom of the borehole to practical refusal at a depth of about 52.7 m below ground surface.

Details of the subsurface conditions at this structure location are presented in Section 4.3 and shown on Drawing A1 in Appendix A.

The stability and settlement analyses of the new embankments have been carried-out assuming a granular fill composition and 2H:1V side slopes and removal of organic soils and existing fill prior to construction of the new embankments (in accordance with OPSD 203.010 Embankments Over Swamps). The simplified stratigraphy and associated unit weight, strength, deformation and time rate consolidation parameters for the different soil types used in the analysis are summarized in Table A1 and detailed for the clayey strata on Figure A1 in

Appendix A. A piezometric condition where the water table is at ground surface, based on the groundwater levels noted during the investigation, has been assumed.

6.4.1.1 Stability

The stability analysis carried out on the critical section(s) indicates that for rapid construction to full height (i.e. 8.6 m), the embankment would have a Factor of Safety (FoS) of less than 1.0 (i.e. failure) for a deep-seated, global failure surface that would impact the operation of the roadway, as shown on Figure A3-1 in Appendix A. The stability analysis also indicates that for a FoS of 1.3, the maximum allowable initial height of embankment fill at this location is 2.4 m. As such, to be able to construct the proposed embankment to the full design height while maintaining a FoS of 1.3 or greater, staged construction methods together with partial sub-excavation and replacement and the use of either toe berms and/or lightweight fill, will be required.

6.4.1.2 Settlement

The settlement of the foundation soils in the critical section is estimated to be about 2.4 m under the loading imposed by an 8.6 m high embankment constructed with conventional (granular) fill. This settlement is due to primary consolidation within the cohesive deposit.

Assuming an average coefficient of consolidation (c_v) of about 1.75×10^{-3} cm^2/s estimated for the cohesive deposit, the imposed loading conditions and assuming two-way drainage of the approximately 48 m thick cohesive deposit (less the 2 m subexcavate and replace noted above and discussed below), it is estimated that about 90 percent of the primary consolidation settlement of the cohesive deposit will be completed in about 80 years, if no special construction/foundation mitigation options are implemented.

The rate of secondary consolidation (creep) settlement for the cohesive deposit is expected to be up to about 230 mm per log-cycle of time for this area. The magnitude of creep settlement following construction will depend on the method of construction/foundation mitigation adopted and the actual time required to achieve 90 percent of primary consolidation. If measures can be implemented to achieve 90 percent of primary consolidation within about one to two years after embankment filling, it is estimated that up to about 180 mm to 200 mm of creep settlement could occur over a 10-year period following completion of construction, which may require long-term maintenance at this location.

6.4.1.3 Mitigation of Stability Issues and/or Time Dependent Settlements

The presence of the up to about 48 m deep cohesive deposit influences both the stability and the settlement of the proposed up to 8.6 m high embankments at this location. In order to construct the embankments to achieve a FoS equal to or greater than 1.3, and to minimize post-construction settlements, the alternatives presented below have been considered. The individual alternatives described below have been evaluated and ranked on the basis of the advantages, disadvantages, relative costs and risk/consequences and are summarised in Table A3 in Appendix A. Given the relatively thick clay deposit and the relatively high and long embankments requiring stability and settlement mitigation measures at this location, and the need to minimize the size of the front slope in the abutment area due to the existing, adjacent Highway 17, the following combination of mitigation measures is ranked as the preferred option for this site:

- 2 m subexcavation and replacement of near surface organic and soft silty clay to clay foundation soils;
- wick drains;

- staged construction;
- 10 m wide by 2 m high toe berms;
- EPS to top-up for settlements during construction; and,
- full height EPS behind abutments to mitigate front slope instability.

Sub-Excavation and Replacement

Considering the depth to the bottom of the clay deposit (i.e. up to about 48 m below the existing ground surface), full sub-excavation of the cohesive deposit is not a practical alternative at this location.

Sub-excavation of the upper 2 m of topsoil/organics and very soft clayey silt and loose silt and replacement with granular (Granular B Type 1) fill is recommended and will be required to be combined with the other stability mitigation measure(s) adopted for this site. It is noted that with 2 m of subexcavation and replacement, the maximum initial embankment height increases from 2.4 m to 3.0 m while maintaining a factor of safety equal to 1.3. The subexcavation and replacement is required beneath all stability toe berms (wherever present) as well as below the embankment footprint wherever the embankment height is 2.4 m or greater.

Wick Drains with Staged Construction

Preliminary analysis indicates that wick drains installed to full depth through the cohesive deposit (up to about 48 m deep) over the portions of both of the approaches and adjacent embankments that are greater than 3 m high (a total length of approximately 350 m) and at a spacing of 1.5 m in a triangular pattern, would reduce the estimated time to reach 90 percent of primary consolidation to about 1.3 years (i.e. compared to 80 years without wick drains). The use of wick drains, and subsequent reduction in the estimated time to reach 90 percent of primary consolidation, makes staged construction a practical alternative for embankment construction at this location.

Preliminary analysis indicates that, because of stability considerations, construction of the embankments to the design height of 8.6 m would require filling in up to 7 stages with delays of about 1.3 years between each stage to allow for pore pressure dissipation, consolidation and corresponding shear strength gain to occur (i.e. for a total construction duration of up to about 9 years). A 10 m wide by 2 m high toe berm would also be required to maintain embankment stability during the staged construction beyond a height of 3 m. The toe berms would need to taper from a maximum width of about 10 m at the highest points of the embankment (i.e. within the approach area) and gradually reduce in width to 0 m at the point where the approach embankments are reduced to a height of about 3 m. This design alternative would also require the use of lightweight (EPS) fill to top-up the embankments following completion of primary consolidation in order to reduce long-term settlements, as well as behind the abutment walls to mitigate stability issues at the front slope (as discussed below).

Toe Berms

To achieve a FoS equal to or greater than 1.3 for the proposed 8.6 m high embankments, without implementing any other stability mitigation measures (other than the 2 m partial subexcavation and replacement), it would be necessary to construct very large earth fill berms along the toes of the embankments. Stability analysis indicates that toe berms 3 m high by up to about 47 m wide would be required along both sides of the embankments, as shown on Figure A3-2 in Appendix A. The toe berms would need to extend from the abutment walls gradually reducing in width from about 47 m to 0 m at the point where the approach embankments are reduced to a height of about 3 m. In addition it would take up to about 80 years to reach 90 per cent consolidation of the underlying cohesive deposit.

Given the very large size of the toe berms that would be required to maintain embankment stability at this site, and the lack of available space at the front slope areas, and the very long period of time required to mitigate settlement issues, toe berms on their own are not considered to be a practical mitigation option. Therefore, other stability and settlement mitigation options and/or a combination of options need to be considered, including the use of staged construction and/or lightweight fill and smaller toe berms (as described above).

Lightweight Fill

In order to reduce the magnitude of the load imposed by the 8.6 m high embankment on the compressible foundation soils, the use of lightweight fill (i.e. expanded polystyrene (EPS)) could be considered for construction of the approach embankments. The use of this material for embankment fill would eliminate the need for stabilizing toe berms and would result in very little long-term, time-dependent (consolidation) settlement of the foundation soils. However, considering the volume of EPS fill required to construct the up to 8.6 m high by 350 m (total) long embankments in this area, the cost would be significantly higher for this alternative than the other mitigation options.

Partial lightweight fill (EPS) construction will, however, be required in conjunction with the other stability and settlement mitigation options given the great thickness of the cohesive deposits at this site. The full embankments are expected to settle by up to about 2.4 m if constructed entirely of conventional (i.e. granular) fill. Placing a volume of EPS that is about 2.4 m thick over an 11 m wide strip to top-up the embankment to the design elevation following completion of primary consolidation would reduce the post-construction settlements by reducing the additional loading on the embankment that would result from the additional 2.4 m of conventional fill otherwise required to compensate for the primary settlement. In addition, lightweight (EPS) fill will also be required behind the abutment walls to maintain stability of the front slopes of the approaches. As a result of the structure geometry and insufficient space for a 2H:1V slope and/or stability berm infront of the abutment walls, it is estimated that lightweight fill over an area of about 55 m long by the road width and 6.4 m thick will be required behind each abutment wall to mitigate front slope stability issues and achieve a factor of safety greater than 1.3, as shown on Figure A3-3 in Appendix A.

6.4.2 Flyover West

The preliminary design for the proposed Flyover West location requires new embankments up to about 9.0 m high to achieve the required vertical roadway profile. The natural topography of this area of the site is relatively flat and low-lying with ground surface between about Elevations 184 m and 185 m.

In general, the subsurface conditions at the proposed Flyover West location consist of a 0.3 m thick layer of organic soils underlain by an approximately 13 m thick stratum of soft to firm silty clay to clay, underlain by granular deposits of sand to gravelly silty sand, in turn underlain by granite bedrock at a depth of about 26 m below ground surface (Elevation 160.0 m).

Details of the subsurface conditions at this structure location are presented in Section 4.4 and shown on Drawing B1 in Appendix B.

The stability and settlement analyses of the new embankments have been carried out assuming a granular fill composition and 2H:1V side slopes and removal of organic soils and existing fill prior to construction of the new embankments (in accordance with OPSD 203.010 Embankments over Swamps). The simplified stratigraphy and associated unit weight, strength, deformation and time rate consolidation parameters for the different soil types used in the analyses are summarized in Table B1 and detailed for the clayey strata on Figure B1 in Appendix B. A piezometric condition where the water table is at ground surface, based on the groundwater levels noted during the investigation, has been assumed.

6.4.2.1 Stability

The stability analysis carried out on the critical section(s) indicates that for rapid construction to full height (i.e. 9.0 m), the embankment would have a Factor of Safety (FoS) of less than 1.0 (i.e. failure) for a deep-seated, global failure surface that would impact the operation of the roadway, as shown on Figure B3-1 in Appendix B. The stability analysis also indicates that for a FoS of 1.3, the maximum allowable initial height of embankment fill at this location is 4.0 m. As such, to be able to construct the proposed embankment to the full design height while maintaining a FoS of 1.3 or greater, staged construction methods together with partial sub-excavation and replacement or lightweight and the use of either toe berms and/or lightweight fill, will be required.

6.4.2.2 Settlement

The settlement of the foundation soils in the critical section is estimated to be about 1.6 m under the loading imposed by a 9.0 m high embankment constructed with conventional (granular) fill. This settlement is due to primary consolidation within the cohesive deposit.

Assuming an average coefficient of consolidation (c_v) of about 4.3×10^{-3} cm^2/s estimated for the cohesive deposit, the imposed loading conditions and assuming two-way drainage of the approximately 12 m thick cohesive deposit (less the 2 m subexcavate and replace noted above and discussed below), it is estimated that about 90 percent of the primary consolidation settlement of the cohesive deposit will be completed in about 2 years, if no special construction/foundation mitigation options are implemented.

The rate of secondary consolidation (creep) settlement for the cohesive deposit is expected to be about 50 mm per log-cycle of time for this area. The magnitude of creep settlement following construction will depend on the method of construction/foundation mitigation adopted and the actual time required to achieve 90 percent of primary consolidation. If measures can be implemented to achieve 90 percent of the primary within about one to two years after embankment filling, it is estimated that about 40 mm to 50 mm of creep settlement could occur over a 10-year period following completion of construction.

6.4.2.3 Mitigation of Stability Issues and/or Time Dependent Settlements

The presence of the up to about 12 m thick cohesive deposit influences both the stability and the settlement of the proposed up to 9.0 m high embankments at this location. In order to construct the embankments to achieve a FoS equal to or greater than 1.3, and to minimize post-construction settlements, the alternatives presented below have been considered. The individual alternatives described below have been evaluated and ranked on the basis of the advantages, disadvantages, relative costs and risk/consequences and are summarised in Table B3 in Appendix B. Given the thickness of the clay deposit and the relatively high and long embankments requiring stability and settlement mitigation measures at this location, and the need to minimize the size of the front slope in the abutment area due to the existing, adjacent Highway 17, the following combination of mitigation measures is ranked as the preferred option for this site:

- 2 m subexcavation and replacement of near surface foundation soils;
- wick drains;
- staged construction;
- 10 m wide by 2 m high toe berms;
- EPS to top-up for settlements during construction; and,
- full height EPS behind abutments to mitigate front slope instability.

Sub-Excavation and Replacement

Considering the depth to the bottom of the clay deposit (i.e. up to about 12 m below the existing ground surface), full sub-excavation of the cohesive deposit is not a practical alternative at this location.

Sub-excavation of the upper 2 m of topsoil/organics and soft silty clay to clay and replacement with granular (Granular B Type 1) fill is recommended and will be required to be combined with the other stability mitigation measures adopted for this site. It is noted that with 2 m of subexcavation and replacement, the maximum initial embankment height increases from 4.0 m to 4.5 m while maintaining a factor of safety equal to 1.3. The subexcavation and replacement is required beneath all stability toe berms (wherever present) as well as below the embankment footprint wherever the embankment height is 4.0 m or greater.

Wick Drains with Staged Construction

Preliminary analysis indicates that wick drains installed to full depth through the cohesive deposit (up to about 12 m deep) over the portions of the approach and adjacent embankments that are greater than 4.5 m high (a total length of approximately 495 m) and at a spacing of 1.5 m in a triangular pattern, would reduce the estimated time to reach 90 per cent of primary consolidation to about 180 days (i.e. compared to 2 years without wick drains). The use of wick drains, and subsequent reduction in the estimated time to reach 90 per cent of primary consolidation, makes staged construction a practical alternative for embankment construction at this location.

Preliminary analysis indicates that because of stability considerations, construction of the embankments to the design height of 9.0 m would require filling in up to 6 stages with delays of about 180 days (6 months) between each stage to allow for pore pressure dissipation, consolidation and corresponding shear strength gain to occur (i.e. for a total construction duration of up to about 3 years). A 10 m wide by 2 m high toe berm would also be required to maintain embankment stability during staged construction beyond a height of 4.5 m. The toe berms would need to taper from a maximum width of about 10 m at the highest points of the embankment (i.e. within the approach area) and gradually reduce in width to 0 m at the point where the approach embankments are reduced to a height of about 4.5 m. This design alternative would also require the use of lightweight (EPS) fill to top-up the embankments following completion of primary consolidation in order to reduce long-term settlements, as well as behind the abutment walls to mitigate stability issues at the front slope (as discussed below).

Toe Berms

To achieve a FoS equal to or greater than 1.3 for the proposed 9.0 m high embankments without implementing any other stability mitigation measures (other than the 2 m partial subexcavation and replacement), it would be necessary to construct large earth fill berms along the toes of the embankments. Stability analysis indicates that toe berms 3 m high by up to 25 m wide would be required along both sides of the embankments, as shown on Figure B3-2 in Appendix B. The toe berms would need to extend from the abutment walls gradually reducing in width from about 25 m to 0 m at the point where the approach embankments are reduced to a height of about 4.5 m. In addition it would take up to about 2 years to reach 90 per cent consolidation of the underlying cohesive deposit.

Given the large size of the toe berms that would be required to maintain embankment stability at this site, and the lack of space at the front slope areas, and the 2 year period of time required to mitigate settlement issues, toe berms on their own are not considered to be a practical mitigation option. Therefore, other stability and settlement mitigation options and/or a combination of options need to be considered, including the use of staged construction and/or lightweight fill and smaller toe berms (as described above).

Lightweight Fill

In order to reduce the magnitude of the load imposed by the 9.0 m high embankment on the compressible foundation soils, the use of lightweight fill (i.e. expanded polystyrene (EPS)) could be considered for construction of the approach embankments. The use of this material for embankment fill would eliminate the need for stabilizing toe berms and would result in very little long-term, time-dependent (consolidation) settlement of the foundation soils. However, considering the volume of EPS fill required to construct the up to 9.0 m high by 495 m long (total) embankments in this area, the cost would be significantly higher for this alternative than the other mitigation options.

Partial lightweight fill (EPS) construction will, however, be required in conjunction with the other stability and settlement mitigation options given the thickness of the cohesive deposits at this site. The full embankments are expected to settle by up to about 1.6 m if constructed entirely of conventional (i.e. granular) fill. Placing a volume of EPS that is about 1.6 m thick over an 11 m wide strip to top-up the embankment height to the design elevation following completion of primary consolidation would reduce the post-construction settlements by reducing the additional loading on the embankment that would result from the additional 1.6 m of conventional fill otherwise required placed to compensate for the primary settlement. In addition, lightweight (EPS) fill will also be required behind the abutment walls to maintain stability of the front slopes of the approaches. As a result of the structure geometry and the insufficient space for a 2H:1V slope and/or stabilizing berm in front of the abutment walls, it is estimated that lightweight fill over an area of about 41 m long by the road width and 7.2 m thick, will be required behind each abutment wall to mitigate front slope stability issues and achieve a factor of safety greater than 1.3, as shown in Figure B3-3 in Appendix B.

6.4.3 Flyover West Alternative 5

The preliminary design for the proposed Flyover West Alternative 5 location requires new embankments up to about 9.0 m high (in the abutment areas) to achieve the required vertical roadway profile. The natural topography of this area of the site is relatively flat and low-lying with a natural ground surface between about Elevations 184 m and 189 m.

In general, the subsurface conditions anticipated to be present at the proposed Flyover West Alternative 5 location may consist of a 0.7 m thick layer of topsoil and loose to compact sand to silty sand underlain by an approximately 13 m thick stratum of soft to stiff clay, underlain by granular deposits of sand to sand and gravel to gravelly silty sand, in turn underlain by bedrock at a depth of about 20 m to 25 m below ground surface (Elevation 163.5 m).

Details of the subsurface conditions at this structure location are presented in Section 4.5 and shown on Drawing C1 in Appendix C.

The stability and settlement analyses of the new embankments have been carried out assuming a granular fill composition and 2H:1V side slopes and removal of any surficial organic soils and existing fill prior to construction of the new embankments (in accordance with OPSD 203.010 Embankments over Swamps). The simplified stratigraphy and associated unit weight, strength, deformation and time rate consolidation parameters for the different soil types used in the analyses are summarized in Table C1 and detailed for the clayey strata on Figure C1 in Appendix C. A piezometric condition where the water table is at ground surface, based on the groundwater levels noted in Section 4.5.6, has been assumed.

6.4.3.1 Stability

The stability analysis carried out on the critical section(s) indicates that for rapid construction to full height (i.e. 9.0 m), the embankment would have a Factor of Safety (FoS) of less than 1.0 (i.e. failure) for a deep-seated, global failure surface that would impact the operation of the roadway, as shown on Figure C2-1 in Appendix C. The stability analysis also indicates that for a FoS of 1.3, the maximum allowable initial height of embankment fill at this location is 4.0 m. As such, to be able to construct the proposed embankment to the full design height while maintaining a FoS of 1.3 or greater, staged construction methods together with partial sub-excavation and replacement or lightweight and the use of either toe berms and/or lightweight fill, will be required.

6.4.3.2 Settlement

The settlement of the foundation soils in the critical section is estimated to be up to about 1.3 m under the loading imposed by an up to 9.0 m high (at the abutments) embankment constructed with conventional (granular) fill. Beyond the approach areas, the settlement is estimated to be less than about 1.0 m. This settlement is due to primary consolidation within the cohesive deposit.

Assuming an average coefficient of consolidation (c_v) of about 4.3×10^{-3} cm^2/s estimated for the cohesive deposit (based on the results of the CPT dissipation testing carried out at the West Flyover location), the imposed loading conditions and assuming two-way drainage of an approximately 13.5 m thick cohesive deposit (less the 2 m subexcavate and replace noted above and discussed below), it is estimated that about 90 percent of the primary consolidation settlement of the cohesive deposit will be completed in about 2 years, if no special construction/foundation mitigation options are implemented.

The rate of secondary consolidation (creep) settlement for the cohesive deposit is expected to be about 50 mm per log-cycle of time for this area. The magnitude of creep settlement following construction will depend on the method of construction/foundation mitigation adopted and the actual time required to achieve 90 percent of primary consolidation. If measures can be implemented to achieve 90 percent of the primary within about one to two years after embankment filling, it is estimated that about 40 mm to 50 mm of creep settlement could occur over a 10-year period following completion of construction.

6.4.3.3 Mitigation of Stability Issues and/or Time Dependent Settlements

The presence of an up to about 13.5 m thick cohesive deposit influences both the stability and the settlement of the proposed up to 9.0 m high embankments at this location. In order to construct the embankments to achieve a FoS equal to or greater than 1.3, and to minimize post-construction settlements, the alternatives presented below have been considered. The individual alternatives described below have been evaluated and ranked on the basis of the advantages, disadvantages, relative costs and risk/consequences and are summarised in Table C3 in Appendix C. Given the thickness of the clay deposit and the relatively high embankments requiring stability and settlement mitigation measures at this location, and the need to minimize the size of the front slope in the abutment area due to the existing, adjacent Highway 17, the following combination of mitigation measures is ranked as the preferred option for this site:

- 2 m subexcavation and replacement of near surface foundation soils;
- wick drains;
- staged construction;
- 10 m wide by 2 m high toe berms;
- EPS to top-up for settlements during construction; and,
- full height EPS behind abutments to mitigate front slope instability.

Sub-Excavation and Replacement

Considering the depth to the bottom of the clay deposit (i.e. up to at least 13.5 m below the existing ground surface), full sub-excavation of the cohesive deposit is not a practical alternative at this location.

Sub-excavation of the upper 2 m of topsoil/organics, loose sand to silty sand and soft clay and replacement with granular (Granular B Type 1) fill is recommended and will be required to be combined with the other stability mitigation measures adopted for this site. Depending on the subsurface conditions encountered away from the abutment areas during the detail investigation stage, the required depth of subexcavation and replacement may be reduced. It is noted that with 2 m of subexcavation and replacement, the maximum initial embankment height increases from 4.0 m to 4.5 m while maintaining a factor of safety equal to 1.3. The subexcavation and replacement is required beneath all stability toe berms (wherever present) as well as below the embankment footprint wherever the embankment height is 4.0 m or greater.

Wick Drains with Staged Construction

Preliminary analysis indicates that wick drains installed to full depth through the cohesive deposit (up to about 13.5 m deep) over the portions of the approach and adjacent embankments that are greater than 4.5 m high (a total length of approximately 110 m) and at a spacing of 1.5 m in a triangular pattern, would reduce the estimated time to reach 90 per cent of primary consolidation to about 180 days (i.e. compared to 2 years without wick drains). The use of wick drains, and subsequent reduction in the estimated time to reach 90 per cent of primary consolidation, makes staged construction a practical alternative for embankment construction at this location.

Preliminary analysis indicates that because of stability considerations, construction of the embankments to the design height of 9.0 m (at the abutments) would require filling in up to 6 stages with delays of about 180 days (6 months) between each stage to allow for pore pressure dissipation, consolidation and corresponding shear strength gain to occur (i.e. for a total construction duration of up to about 3 years). Beyond the abutments, where the required embankment fill heights are less, a smaller number of fill stages and overall shorter construction period may be possible. A 10 m wide by 2 m high toe berm would also be required to maintain embankment stability during staged construction beyond a height of 4.5 m. The toe berms would need to taper from a maximum width of about 10 m at the highest points of the embankment (i.e. within the approach area) and reduce in width to 0 m at the point where the approach embankments are reduced to a height of about 4.5 m. This design alternative would also require the use of lightweight (EPS) fill to top-up the embankment following completion of primary consolidation in order to reduce long-term settlements, as well as behind the abutment walls to mitigate stability issues at the front slope (as discussed below).

Toe Berms

To achieve a FoS equal to or greater than 1.3 for the proposed 9.0 m high embankments without implementing any other stability mitigation measures (other than the 2 m partial subexcavation and replacement), it would be necessary to construct large earth fill berms along the toes of the embankments. Stability analysis indicates that toe berms 3 m high by up to 31 m wide would be required along both sides of the embankments, as shown on Figure C2-2 in Appendix C. The toe berms would need to extend from the abutment walls reducing in width from about 27 m to 0 m at the point where the approach embankments are reduced to a height of about 4.5 m. In addition it would take up to about 2 years to reach 90 per cent consolidation of the underlying cohesive deposit.

Given the large size of the toe berms that would be required to maintain embankment stability at this site, and the lack of space at the front slope areas, and the 2 year period of time required to mitigate settlement issues, toe berms on their own are not considered to be a practical mitigation option. Therefore, other stability and settlement mitigation options and/or a combination of options need to be considered, including the use of staged construction and/or lightweight fill and smaller toe berms (as described above).

Lightweight Fill

In order to reduce the magnitude of the load imposed by the 9.0 m high embankment on the compressible foundation soils, the use of lightweight fill (i.e. expanded polystyrene (EPS)) could be considered for construction of the approach embankments. The use of this material for embankment fill would eliminate the need for stabilizing toe berms and would result in very little long-term, time-dependent (consolidation) settlement of the foundation soils. However, considering the volume of EPS fill required to construct the up to 9.0 m high by 110 m long (total) embankments in this area, the cost would be significantly higher for this alternative than the other mitigation options.

Partial lightweight fill (EPS) construction will, however, be required in conjunction with the other stability and settlement mitigation options given the thickness of the cohesive deposits at this site. The full embankments are expected to settle by up to about 1.3 m if constructed entirely of conventional (i.e. granular) fill. Placing a volume of EPS that is about 1.3 m thick over an 11 m wide strip to top-up the embankment height to the design elevation following completion of primary consolidation would reduce the post-construction settlements by reducing the additional loading on the embankment that would result from the additional 1.3 m of conventional fill otherwise required to be placed to compensate for the primary settlement. In addition, lightweight (EPS) fill will also be required behind the abutment walls to maintain stability of the front slopes of the approaches. As a result of the structure geometry and the insufficient space for a 2H:1V slope and/or stabilizing berm in front of the abutment walls, it is estimated that lightweight fill over an area of about 36 m long by the road width and average 7.4 m thick, will be required behind each abutment wall to mitigate front slope stability issues and achieve a factor of safety greater than 1.3, as shown in Figure C2-3 in Appendix C.

6.4.4 Interchange

The preliminary design for the proposed Highway 638 Interchange location requires new embankments up to about 9.4 m high to achieve the required vertical roadway profile. The natural topography of this area of the site is relatively flat and low-lying with ground surface between about Elevations 181 m and 183 m.

The subsurface conditions at the proposed Interchange location generally consist of an approximately 3.8 m to 7.6 m thick deposit of sand and silt, underlain by a stratum of firm to very stiff clayey silt to clay, to a depth of about 53 m below ground surface (Elevation 130.8 m). The clay stratum contains a layer of sand approximately 3.4 m thick at a depth of about 22.5 m below ground surface (Elevation 161.3). Refusal to further penetration of a DCPT was encountered about 2.3 m below the clay stratum that is at a depth of about 55.3 m below ground surface (Elevation 128.5 m).

Details of the subsurface conditions at this structure location are presented in Section 4.5 and shown on Drawing D2 in Appendix D.

The stability and settlement analyses of the new embankments have been carried out assuming a granular fill composition and 2H:1V side slopes and removal of organic soils and existing fill prior to construction of the new embankments (in accordance with OPSD 203.010 Embankments over Swamps). The simplified stratigraphy and associated unit weight, strength, deformation and time rate consolidation parameters for the different soil types used in the analysis are summarized in Table D1 and detailed for the clayey strata on Figure D1 in Appendix D. A piezometric condition where the water table is at ground surface, based on the groundwater levels noted during the investigation, has been assumed.

6.4.4.1 Stability

The stability analysis performed on the critical section(s) indicates that for rapid construction to full height (i.e. 9.4 m), the embankment would have a Factor of Safety (FoS) of less than 1.3 for a deep-seated, global failure surface that would impact the operation of the roadway, as shown on Figure D3-1 in Appendix D. The stability analysis also indicates that for a FoS of 1.3, the maximum allowable initial height of embankment fill at this location is 7.3 m. As such, to be able to construct the proposed embankment to the full design height while maintaining a FoS of 1.3 or greater, staged construction methods together with the use of toe berms and/or lightweight fill will be required.

6.4.4.2 Settlement

The settlement of the foundation soils in the critical section is estimated to be about 1.5 m under the loading imposed by a 9.4 m high embankment constructed with conventional (granular) fill. This settlement is due to primary consolidation within the cohesive deposit (about 1.4 m) and elastic settlement of the surficial cohesionless soils (about 0.1 m).

Based on an average coefficient of consolidation (c_v) of about 3.25×10^{-3} cm^2/s estimated for the cohesive deposit, the imposed loading conditions and assuming two-way drainage of the approximately 14 m thick upper clay and 27 m thick lower clay deposits, it is estimated that about 90 percent of the primary consolidation settlement of the cohesive deposit will be completed in about 15 years, if no special construction/foundation mitigation options are implemented.

The rate of secondary consolidation (creep) settlement for the cohesive deposit is expected to be up to about 200 mm per log-cycle of time for this area. The magnitude of creep settlement following construction will depend on the method of construction/foundation mitigation adopted and the actual time required to achieve 90 percent of primary consolidation. If measures can be implemented to achieve 90 percent of the primary within about one to two years after embankment filling, it is estimated that up to about 150 mm to 200 mm of creep settlement could occur over a 10-year period following completion of construction, which may require some long term maintenance at this location.

6.4.4.3 Mitigation of Stability Issues and/or Time Dependent Settlements

The presence of the up to about 53 m deep cohesive deposit influences both the stability and the settlement of the proposed up to 9.4 m high embankments at this location. In order to construct the embankments to achieve a FoS equal to or greater than 1.3, and to minimize post-construction settlements, the alternatives presented below have been considered. The individual alternatives described below have been evaluated and ranked on the basis of the advantages, disadvantages, relative costs and risk/consequences and are summarised in Table D3 in Appendix D. Given the thickness of the clayey silt to clay deposit and the relatively high and long embankments requiring stability and settlement mitigation measures at this location, and the need to minimize the size of the front slope in the abutment area due to the existing, adjacent Highway 17, the following combination of mitigation measures is ranked as the preferred option for this site:

- wick drains;
- staged construction;
- EPS to top-up for settlements during construction; and,
- full height EPS behind abutments to mitigate front slope instability.

Sub-Excavation and Replacement

Considering the depth to the bottom of the clay deposit (i.e. up to about 53 m below the existing ground surface), full sub-excavation of the cohesive deposit is not a practical alternative at this location.

Sub-excavation of a portion of the upper foundation soils and replacement with granular fill (as recommended at the other two sites) is not required at this structure location due to the presence of the 3.8 m to 7.6 m thick deposit of cohesionless soils at the existing ground surface. Sub-excavation of any organic materials found within the embankment footprint should however be carried out and replaced with granular fill concurrent with embankment construction.

Wick Drains with Staged Construction

Preliminary analysis indicates that wick drains installed to full depth through the cohesive deposit (up to about 53 m deep) over the portions of the approaches and adjacent embankments that are greater than 7.3 m high (a total length of approximately 210 m) and at a spacing of 1.5 m in a triangular pattern, would reduce the estimated time to reach 90 percent of primary consolidation to about 250 days (i.e. compared to about 15 years without wick drains). The use of wick drains, and subsequent reduction in the estimated time to reach 90 percent of primary consolidation, makes staged construction a practical alternative for embankment construction at this location.

Preliminary analysis indicates that because of stability considerations, construction of the embankment to the design height of 9.4 m would require filling in 2 stages with delays of about 250 days (8 months) between each stage to allow for pore pressure dissipation, consolidation and corresponding shear strength gain to occur (i.e.

for a total construction duration of about 1.5 years). This design alternative would also require the use of lightweight (EPS) fill to top-up the embankment following completion of primary consolidation in order to reduce long-term settlements, as well as behind the abutment walls to mitigate stability issues at the front slope (as discussed below).

Toe Berms

To achieve a FoS equal to or greater than 1.3 for the proposed 9.4 m high embankments without implementing any other stability mitigation measures it would be necessary to construct large earth fill berms along the toes of the embankments. Stability analysis indicates that toe berms 3 m high by up to about 20 m wide would be required along both sides of the embankments, as shown on Figure D3-2 in Appendix D. The toe berms would need to extend from the abutment walls gradually reducing in width from about 20 m to 0 m at the point where the approach embankments are reduced to a height of about 7.3 m. In addition it would take up to about 15 years to reach 90 per cent consolidation of the underlying cohesive deposit.

Due to the large size of the toe berms that would be required to maintain the embankment stability at the critical section(s) and the long period of time required to mitigate settlement issues, toe berms on their own are not considered to be practical due to the large long-term settlements that this mitigation option does not address and as such, other stability and settlement mitigation options and/or a combination of options need to be considered, including the use of staged construction and/or lightweight fill.

Lightweight Fill

In order to reduce the magnitude of the load imposed by the 9.4 m high embankment on the compressible foundation soils, the use of lightweight fill (i.e. expanded polystyrene (EPS)) could be considered for construction of the approach embankments. The use of this material for embankment fill would eliminate the need for stabilizing toe berms and would result in very little long-term, time-dependent (consolidation) settlement of the foundation soils. However, considering the volume of EPS fill required to construct the up to 9.4 m high by 210 m (total) long embankments in this area, the cost would be significantly higher for this alternative than the other mitigation options.

Partial lightweight fill (EPS) will, however, likely be required in conjunction with other stability and settlement mitigation options given the thickness of the silty clay to clay deposits at this site. The full embankments are expected to settle due to consolidation by about 1.4 m if built entirely out of conventional (i.e. granular) fill. Placing a volume of EPS that is about 1.4 m thick over an 11 m wide strip to top-up the embankment height to the design elevation following completion of primary consolidation would reduce the post-construction settlements by reducing the additional loading on the embankment that would result from the additional 1.4 m of conventional fill otherwise required to compensate for the primary settlement. In addition, lightweight (EPS) fill will also be required behind the abutment walls to maintain stability of the front slopes of the approaches. As a result of the structure geometry and the insufficient space for a 2H:1V slope and stabilizing berm in front of the abutment walls, it is estimated that lightweight fill over an area of about 9 m long by the road width and 7.0 m thick, will be required behind each abutment wall to mitigate front slope stability issues and achieve a factor of safety greater than 1.3, (as shown in Figure D3-3 in Appendix D).

6.5 Construction Considerations

6.5.1 Subgrade Preparation and Embankment Construction

The existing fills and any topsoil/organic deposits encountered within the footprint of the embankments should be stripped from the plan limits of the proposed works and the subgrade soils should be proof-rolled. The following sections provide recommendations for subgrade preparation and embankment construction.

6.5.2 Removal of Organic Materials

Based on the information from the boreholes obtained during the field investigation, layers of topsoil up to about 0.5 m thick and organic soils up to about 1.5 m thick can be expected in some areas of the new approach embankments. Deposits of existing fill up to 2.8 m thick were encountered within the plan limits of the new approach embankments at the interchange site. These surficial organics/fill layers, where encountered, should be stripped from the plan limits of the approach embankment areas prior to fill placement for the new embankment(s).

6.5.3 Excavation and Replacement of Soft Subsoils

In areas where stability and/or settlement require mitigation measures to enhance the long-term performance of the embankments and roadway, partial subexcavation and replacement of soft subsoils is recommended. Excavation up to about 2 m below existing ground surface (and replacement with granular fill) is required in the areas of the approaches and where the embankment heights beyond the approaches exceed 2.4 m at the East Flyover, and 4 m at the West Flyover and West Flyover Alternative 5. At the Interchange, subexcavation and replacement is not required given the presence of the near surface sandy soils. Conventional excavators should be suitable for all of the excavating operations. The soft subsoils should be removed using construction procedures in accordance with OPSS 209 (Embankments Over Swamps).

Based on discussions with MTO Foundations regarding their previous experience during construction of the Highway 17 (New) embankments in the area of Bar River Road and westerly, it is anticipated that subexcavation and replacement of the upper 2 m of the near surface soils may be required in some areas of the site regardless of embankment height. Given the very soft nature of the upper clayey soils at some locations, carrying out the subexcavation and replacement may be required in order to provide a working platform to support equipment and facilitate embankment construction. The plan limits of where this constructability requirement should be implemented will have to be defined at the detail design stage.

All excavations must be carried out in accordance with Ontario Regulation 213 Ontario Occupational Health and Safety Act for Construction Projects (as amended by Ontario Regulation 443).

6.5.4 Embankment Fill Placement

Placement of granular fill material for embankment construction should be carried out in accordance with the requirements as outlined in OPSS 206 (Grading). Side slopes for earth fill embankments should be no steeper than 2 horizontal to 1 vertical (2H:1V).

In areas where embankment heights are greater than 8 m, a 2 m wide mid-height berm must be incorporated into the slope such that the uninterrupted slope height is not greater than 8 m.

6.5.5 Control of Groundwater and Surface Water

Groundwater levels observed during the field investigation are recorded on the Record of Borehole sheets in Appendices A to D. It is noted that the groundwater levels recorded during drilling may not be representative of the natural or static groundwater level at the site. Although the groundwater table is expected to be located at or near ground surface, given the clayey nature of the soils it is anticipated that the majority of the sub-excavation will likely be 'in-the-dry'. Unwatering is not anticipated to be required for the excavation at the approach embankment locations, however, surface water should be directed away from the excavations at all times.

6.5.6 Obstructions During Pile Driving

It is anticipated that cobbles and/or boulders will be encountered within the lower granular deposits (eg. as encountered in Borehole 10-3 at the Flyover West site) during driving of the piles, and may affect pile installation. It is recommended that flange plate reinforcement or driving shoes be used on all piles to facilitate driving into/through the very dense granular soils or for seating the piles on bedrock. In addition, as part of the detail design and contract preparation, it is recommended that consideration be given to including a Non-Standard Special Provision in the contract documents to warn the contractor of the possible presence of cobbles and/or boulders within the overburden soils.

6.5.7 Embankment Platform Widening

In accordance with the requirements of MTO Northern Region Engineering Directive NRE 98-200, Northern Region Embankment Design Guidelines, the construction of the embankments should include an allowance for platform widening (in 0.5 m increments) to accommodate settlement during construction as well as post-construction settlements so that the minimum standard shoulder widths are maintained if future grade raises on the embankments are required. According to NRE 98-200, the need for future raises in road grade could occur due to settlement/compression of the embankment fill, settlement of the foundation soils and to accommodate future pavement overlays up to 200 mm thick. It is understood that this directive applies to all rock fill embankments as well as for granular fill embankments where widening restrictions are present (i.e. due to space/property issues, presence of a sensitive body of water and so on). It is further understood that the minimum required platform widening on major highways over soft compressible subsoils is 2 m per side, unless the preferred mitigation option eliminates uncertainty regarding embankment settlement/performance (i.e. full sub-excavation to bedrock and backfilling with granular material). For non-major highways and roadways (i.e. ramps and side roads) over swamp crossings, the minimum required platform widening is 1 m per side.

The minimum required embankment platform widening (per embankment side) is calculated based on the estimated consolidation settlement of the foundation soils (including creep) and long-term settlement/compression of the embankment fill plus an additional 200 mm for the future pavement overlay, multiplied by the horizontal component of the side slope of the pavement structure (4H:1V), but cannot be less than the minimum platform widening requirements as described above.

For the proposed approach embankments in these areas, the minimum platform widening is summarized below.

Embankment Location	Minimum Embankment Platform Widening Per Side (m)
Flyover East	5.5
Flyover West	4.0
Flyover West Alternative 5	4.0
Highway 17 / Highway 638 Interchange	3.5

6.5.8 Post-Construction Maintenance

As noted in Sections 6.4.1.2 and 6.4.3.2, given the very thick clayey deposits present at the Flyover East and Interchange sites, some long-term post-construction creep settlements are likely to occur regardless of the mitigation measures adopted. As such, provisions would have to be made in future maintenance contracts in the project area to address the settlements in these areas.

6.6 Construction Considerations for Detail Design

6.6.1 Additional Investigation Requirements

As noted previously, additional borehole investigation, laboratory testing and analysis will be required during detail design, once the preferred flyover and interchange structure locations and ramp/approach embankment configurations have been selected, to confirm the preliminary foundation recommendations presented herein, including founding elevations, subexcavation and replacement requirements, geotechnical resistances, settlement/stability issues and mitigation measures.

In particular, it is recommended that further investigation be completed to confirm:

- The extent and strength of the cohesive deposits at the structure locations, below the approach embankments and below the ramps/high embankments adjacent to the approaches wherever the proposed embankments are greater than 2 m in height to assess for stability/settlement mitigation measures and time period for preloading conditions;
- The depth to bedrock or refusal at each foundation element of the structures to assess whether end bearing pile foundations are appropriate and defin pile lengths; and,
- The extent and thickness of the granular soils present at the ground surface in the area of the interchange approaches and ramps and within and beyond the approaches to the Flyover West Alternative 5 for consideration of the use of friction piles terminating in this deposit.

Additional in-situ and laboratory testing is also recommended to:

- Further characterize the undrained shear strength profile of the cohesive deposits.
- Confirm the compressibility properties of the clayey soil deposits by complex (oedometer) laboratory testing.
- Confirm the effective stress strength properties of the clayey soil deposits by complex (triaxial) laboratory testing.

7.0 CLOSURE

This report was prepared by Mr. Matthew Kelly, P.Eng., and reviewed by Mr. J. Paul Dittrich, Ph.D., P.Eng., a senior geotechnical engineer and Principal with Golder. Mr. Jorge Costa, P.Eng., Golder's Designated MTO Contact for this project, carried out an independent quality control review of the report.

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STANDARDS:

ASTM International:

ASTM D1586	Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils
ASTM D2573	Standard Test Method for Field Vane Shear Test in Cohesive Soil
ASTM D5778	Standard Test Method for Electronic Friction Cone and Piezocone Penetration Testing of Soils

Ministry of Transportation Ontario:

Northern Region Engineering Directive NRE 98-200. Northern Region Embankment Design Guidelines. October 1998.

Northeastern Region Geotechnical Section Memorandum. “Use of Mid-Slope Berms for Rockfill Embankments, Northeastern Region” dated February 8, 2005.

Ontario Occupational Health and Safety Act:

Ontario Regulation 213/91	Construction Projects
Ontario Regulation 443/09	Amendment to Ontario Regulation 213

Ontario Provincial Standard Drawing:

OPSD 203.010	Embankments Over Swamps, New Construction
OPSD 3101.150	Walls, Abutment, Backfill, Minimum Granular Requirement
OPSD 3121.150	Walls, Retaining, Backfill, Mininmum Granular Requirement
OPSD 3101.200	Walls, Abutment, Backfill, Rock

Ontario Provincial Standard Specification:

OPSS 206	Construction Specification for Grading.
OPSS 209	Construction Specification for Embankments Over Swamps and Compressible Soils.
OPSS 501	Construction Specification for Compacting

Ontario Water Resources Act:

Ontario Regulation 468/10	Amendment to Ontario Regulation 903
Ontario Regulation 903/90	Wells

LIST OF ABBREVIATIONS

The abbreviations commonly employed on Records of Boreholes, on figures and in the text of the report are as follows:

I. SAMPLE TYPE

AS	Auger sample
BS	Block sample
CS	Chunk sample
SS	Split-spoon
DS	Denison type sample
FS	Foil sample
RC	Rock core
SC	Soil core
ST	Slotted tube
TO	Thin-walled, open
TP	Thin-walled, piston
WS	Wash sample

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:
The number of blows by a 63.5 kg. (140 lb.) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) drive open sampler for a distance of 300 mm (12 in.)

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

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

Piezo-Cone Penetration Test (CPT)
A electronic cone penetrometer with a 60° conical tip and a project end area of 10 cm^2 pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (Q_t), porewater pressure (PWP) and friction along a sleeve are recorded electronically at 25 mm penetration intervals.

III. SOIL DESCRIPTION

(a) Cohesionless Soils

Density Index / Relative Density	N Blows/300 mm or Blows/ft
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	over 50

(b) Cohesive Soils
Consistency

	c_u, s_u kPa	c_u, s_u psf
Very soft	0 to 12	0 to 250
Soft	12 to 25	250 to 500
Firm	25 to 50	500 to 1,000
Stiff	50 to 100	1,000 to 2,000
Very stiff	100 to 200	2,000 to 4,000
Hard	over 200	over 4,000

IV. SOIL TESTS

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

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

V. MINOR SOIL CONSTITUENTS

Percent by Weight	Modifier	Example
0 to 5	Trace	Trace sand
5 to 12	Trace to Some (or Little)	Trace to some sand
12 to 20	Some	Some sand
20 to 30	(ey) or (y)	Sandy
over 30	And (cohesionless) or With (cohesive)	Sand and Gravel / Silty Clay with sand / Clayey Silt with sand

LIST OF SYMBOLS

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

I. GENERAL

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

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. in stress: $\Delta\,\sigma$
ε	linear strain
ε_v	volumetric strain
η	coefficient of viscosity
υ	Poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma$ - u)
σ'_{vo}	initial effective overburden stress
σ_1, σ_2, σ_3	principal stress (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

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

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

(a) Index Properties (continued)

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

(b) Hydraulic Properties

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

(c) Consolidation (one-dimensional)

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

(d) Shear Strength

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

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

LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

WEATHERINGS STATE

Fresh: no visible sign of weathering

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

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

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

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

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

BEDDING THICKNESS

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

JOINT OR FOLIATION SPACING

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

GRAIN SIZE

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

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

CORE CONDITION

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

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

Rock Quality Designation (RQD)
The percentage of solid drill core, greater than 100 mm length, recovered at full diameter, measured relative to the length of the total core run. RQD varied from 0% for completely broken core to 100% for core in solid sticks.

DISCONTINUITY DATA

Fracture Index
A count of the number of discontinuities (physical separations) in the rock core, including both naturally occurring fractures and mechanically induced breaks caused by drilling.

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

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

Abbreviations

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

TABLE 1 – COMPARISON BETWEEN FLYOVER LOCATIONS FOR THE PREFERRED STRUCTURE FOUNDATION ALTERNATIVE[1]

Alternative	*Rank*	*Advantages*	*Disadvantages*	*Relative Costs*	*Risks / Consequences*
Flyover West Alternative 5 (Steel piles driven to bedrock) (approx. 20 m to 25 m long piles)	1	• Sub-excavation is not required, except for pile cap construction. • Negligible post-construction settlement. • Higher axial capacity of piles than at Flyover East. • Shorter piles to bedrock than at Flyover East or Flyover West alternatives.	• Heavier pile sections will be required to penetrate cobbles and boulders at depth and seat piles on bedrock.	• Higher cost associated with heavier pile sections. • Lower cost associated with shorter pile lengths and lesser number of piles.	• Potential risk of damage to piles and piles driven out of alignment due to boulders which could require removal and replacement with new piles. • The abutment/pier design should be flexible enough to allow for installation of extra piles within the foundation unit/pile cap.
Flyover West (Steel piles driven to bedrock) (approx. 25 m to 30 m long piles)	2	• Sub-excavation is not required, except for pile cap construction. • Negligible post-construction settlement. • Higher axial capacity of piles than at Flyover East. • Shorter piles to bedrock than at Flyover East alternative.	• Heavier pile sections will be required to penetrate cobbles and boulders at depth and seat piles on bedrock.	• Higher cost associated with heavier pile sections. • Lower cost associated with shorter pile lengths and lesser number of piles.	• Potential risk of damage to piles and piles driven out of alignment due to boulders which could require removal and replacement with new piles. • The abutment/pier design should be flexible enough to allow for installation of extra piles within the foundation unit/pile cap.

TABLE 1 – COMPARISON BETWEEN FLYOVER LOCATIONS FOR THE PREFERRED STRUCTURE FOUNDATION ALTERNATIVE[1]

Alternative	*Rank*	*Advantages*	*Disadvantages*	*Relative Costs*	*Risks / Consequences*
Flyover East (Steel piles driven to refusal/bedrock) (approx. 55 m to 60 m long piles)	3	• Sub-excavation is not required, except for pile cap construction.	• Lower pile capacity than at Flyover West Alternative 5 and Flyover West if piles not founded on bedrock and due to higher downdrag loads. • Significant depth to refusal and to bedrock, the depth of which still has to be confirmed; will require very long piles which could result in installation difficulties.	• Higher cost associated with greater pile lengths. • Higher cost associated with additional piles due to lower axial capacity.	• Greater risk of piles to be driven out of alignment which may require removal and replaced with new piles. • The abutment/pier design should be flexible enough to allow for installation of extra piles in the footing area.

[1] Table compares advantages, disadvantages and relative costs of the recommended foundation design alternative (steel piles driven to bedrock/refusal). For a comparison of the various foundation alternatives for each structure refer to Tables A2, B2, C2 and D2 in Appendices A through D, respectively.

TABLE 2 – COMPARISON BETWEEN FLYOVER LOCATIONS FOR THE PREFERRED APPROACH EMBANKMENT FOUNDATION ALTERNATIVE[1]

Alternative	*Rank*	*Advantages*	*Disadvantages*	*Relative Costs*[2]	*Risks / Consequences*
Flyover West Alternative 5 (Staged construction with wick drains, 10 m wide by 2 m high toe berms and 2 m subexcavate and replace)	1	• Relatively short staged construction time (approx. 3 years). • Smaller downdrag loads on piles. • Thinner/stronger clay stratum - less onerous settlement mitigation measures required, including significantly shorter wick drain lengths than Flyover East. • Smaller area required for wick drain treatment than at Flyover East and Flyover West due to shorter length of high embankment profile. • Lesser volume of EPS required to maintain front slope stability as compared with Flyover East. • Lesser volume of EPS required to top-up to mitigate long-term settlements as compared with Flyover East and Flyover West. • Lesser total volume of fill required for toe berms due to shorter length of high embankment profile. • Lesser total volumes of subexcavation and replacement fill required due to shorter length of high embankment profile.	• Large post construction settlement.	• $420,000 (13.5 m deep Wick drains at 1.5 m spacing) + • $57,500 (berms) + • $374,500 (subexcavate / replace) + • cost of EPS to mitigate long term settlements $1,637,200 • Estimated Total = $2,489,200	• Staged construction sequence required with potential for additional delays during construction (depending on monitoring). • Post construction settlements may require long-term maintenance. • Some secondary consolidation (creep) will occur.

TABLE 2 – COMPARISON BETWEEN FLYOVER LOCATIONS FOR THE PREFERRED APPROACH EMBANKMENT FOUNDATION ALTERNATIVE[1]

Alternative	*Rank*	*Advantages*	*Disadvantages*	*Relative Costs*[2]	*Risks / Consequences*
Flyover West (Staged construction with wick drains, 10 m wide by 2 m high toe berms and 2 m subexcavate and replace)	2	• Relatively short staged construction time (Approx. 3 years). • Smaller downdrag loads on piles. • Thinner/stronger clay stratum - less onerous settlement mitigation measures required, including significantly shorter wick drain lengths than at Flyover East. • Lesser volume of EPS required to maintain front slope stability and to top-up to mitigate long-term settlements as compared with Flyover East.	• Large post construction settlement. • Slightly larger total volume of fill required for toe berms due to longer length of high embankment profile. • Slightly larger total volumes of subexcavation and replacement fill required due to longer length of high embankment profile.	• $891,000 (12 m deep Wick drains at 1.5 m spacing) + • $198,000 (berms) + • $445,000 (subexcavate / replace) + • cost of EPS to mitigate long term settlements $2,835,200 • Estimated Total = $4,369,200	• Staged construction sequence required with potential for additional delays during construction (depending on monitoring). • Post construction settlements may require long-term maintenance. • Some secondary consolidation (creep) will occur.

TABLE 2 – COMPARISON BETWEEN FLYOVER LOCATIONS FOR THE PREFERRED APPROACH EMBANKMENT FOUNDATION ALTERNATIVE[1]

Alternative	*Rank*	*Advantages*	*Disadvantages*	*Relative Costs*[2]	*Risks / Consequences*
Flyover East (Staged construction with wick drains, 10 m wide by 2 m high toe berms and 2 m subexcavate and replace)	3	• Smaller area required for wick drain treatment due to lower embankment profile. • Slightly smaller total volumes of subexcavation and replacement fill required due to lower embankment profile. • Slightly smaller total volume of fill required for toe berms due to lower embankment profile.	• Longer staged construction time (approximately 9 years). • Large post construction settlement. • Large downdrag loads reduce pile capacity. • Greater volume of EPS required to maintain front slope stability and to top-up to mitigate long-term settlements. • Thicker/weaker clay stratum - more onerous settlement mitigation measures required including significantly longer wick drain lengths (up to 3 times longer than for Flyover West and for Flyover West Alternative 5).	• $2,621,250 (wick drains at 1.5 m spacing) + • $140,000 (berms) + • $315,000 (subexcavate / replace) + • cost of EPS to mitigate long term settlements $4,171,600 Estimated Total = $7,247,850	• Staged construction sequence required with potential for additional delays during construction (depending on monitoring). • Large post-construction settlements will require long-term maintenance. • Potentially larger secondary consolidation (creep) will occur. • Significantly longer wick drains may require specialty construction techniques/equipment and/or a specialised contractor for installation which may increase cost.

[1] Table compares advantages, disadvantages and relative costs of the recommended approach embankment foundation design alternative (i.e 2 m subexcavation and replacement, wick drains with staged construction and EPS top-up for long term settlement and front slope stability mitigation). For a comparison of the various foundation alternatives for each structure location see Tables A3, B3, C3 and D3 in Appendicies A through D, respectively.

[2] Estimated costs are for stability/settlement mitigation measures only and do not include costs which would be incurred for typical embankment construction (i.e. embankment filling and embankment platform widening).

MINISTRY OF TRANSPORTATION, ONTARIO

METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.
GWP No. 5022-07-00

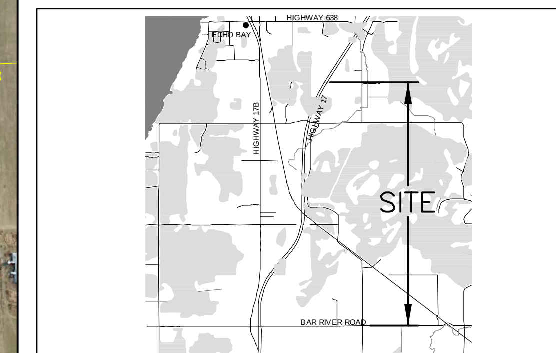
HIGHWAY 17
FROM 5.5 km SOUTH OF HWY 638 TO 2.0 km NORTH OF HWY 638 ACCESS REVIEW
SITE LOCATON PLAN

SHEET

Golder Associates
Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA

PLAN

SCALE
150 0 150 300 m



KEY PLAN

SCALE
1.2 0 1.2 2.4 km

NOTES

This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contracts Documents.

The boundaries between soil strata have been established only at borehole locations. Between boreholes the boundaries are assumed from geological evidence.

The complete Foundation Investigation and Design Report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with Section GC 2.01 of OPS General Conditions.

REFERENCE

Base plans provided in digital format by Genivar, drawing file nos. 09-079-Location 4IDiamond.dwg and background file 0937SSMA0001_10cm_2009.sid. Received July. 2010.

NO.	DATE	BY	REVISION

Geocres No. 41K-90

HWY. 17	PROJECT NO. 09-1111-0016		DIST.
SUBM'D. MWK	CHKD. JPD	DATE: 7/12/2012	SITE:
DRAWN: JFC	CHKD. MWK	APPD. JMAC	DWG. 1

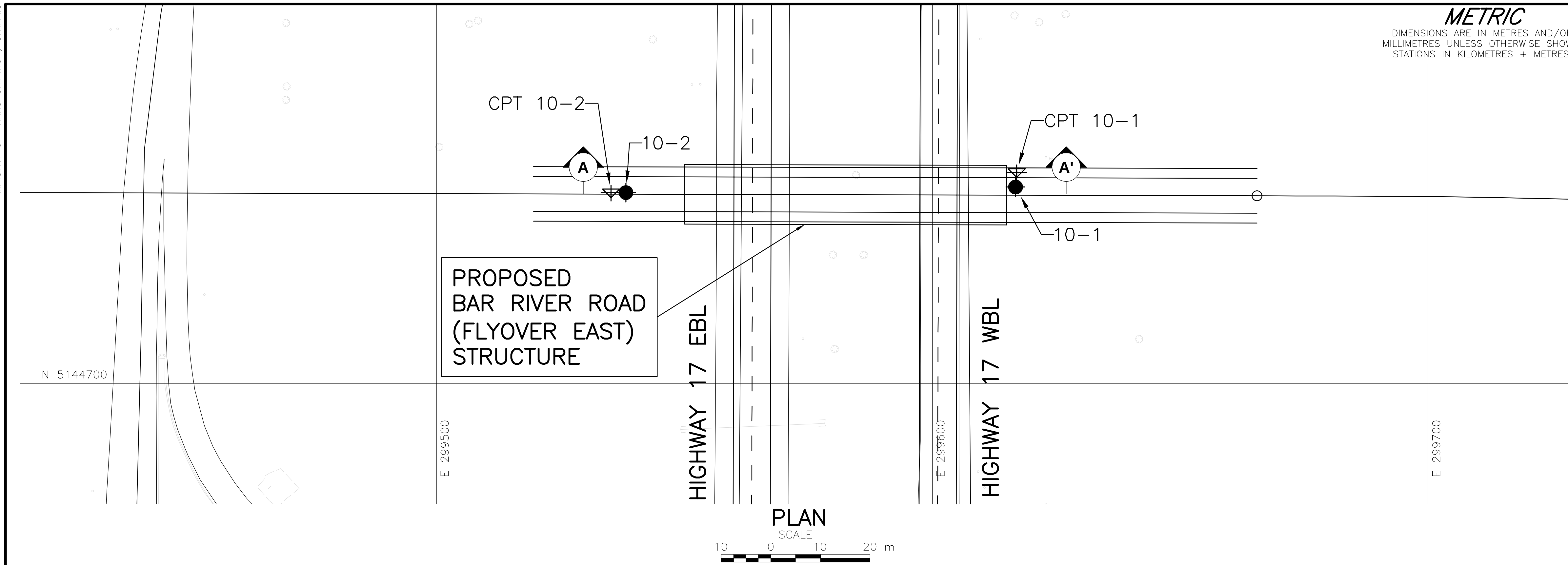
PLOT DATE: July 12, 2012

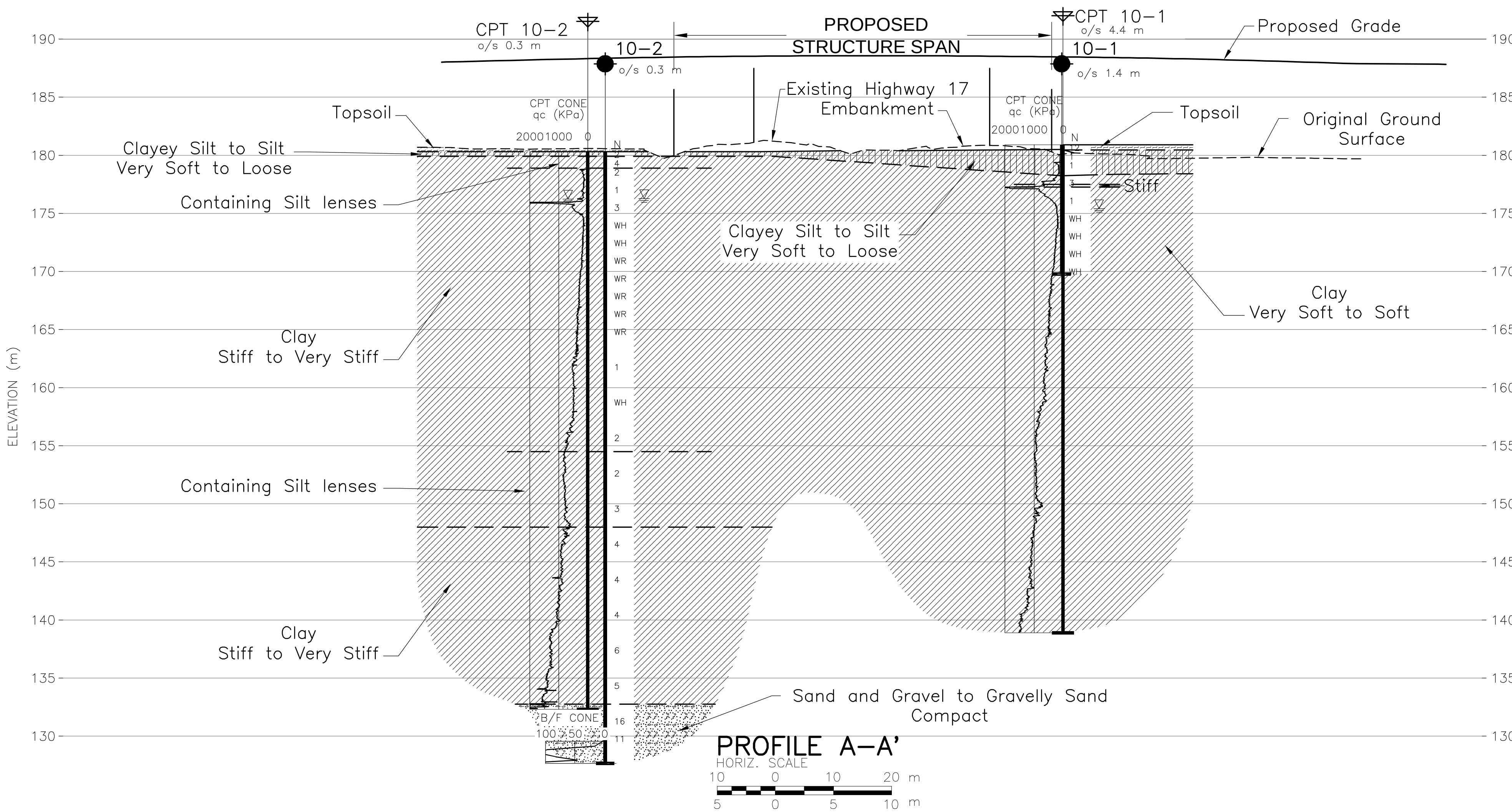
APPENDIX A

Bar River Road Flyover (Flyover East)

MINISTRY OF TRANSPORTATION, ONTARIO

METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.



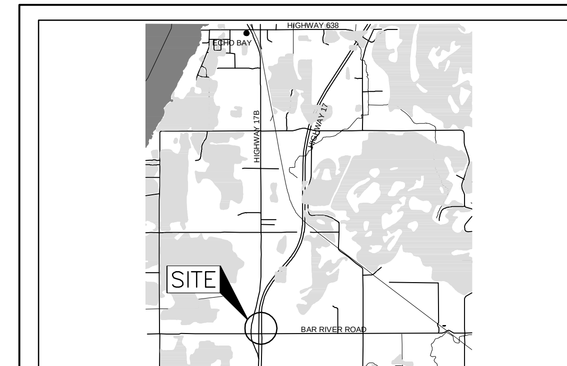


CONT No.
GWP No. 5022-07-00

SHEET

HIGHWAY 17
FLYOVER EAST STRUCTURE
BOREHOLE LOCATIONS AND SOIL STRATA

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



KEY PLAN
SCALE
1.2 0 1.2 2.4 km

LEGEND

- Borehole – Current Investigation
- CPT – Current Investigation
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- WL upon completion of drilling
- CPT tip resistance qc (kPa)

BOREHOLE CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
10-1	180.9	5144739.6	299616.8
10-2	180.3	5144738.5	299538.2

CPT CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
CPT 10-1	180.9	5144742.6	299617.0
CPT 10-2	180.3	5144738.5	299535.2

NOTES

This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contracts Documents.

The boundaries between soil strata have been established only at borehole locations. Between boreholes the boundaries are assumed from geological evidence.

The complete Foundation Investigation and Design Report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with Section GC 2.01 of OPS General Conditions.

REFERENCE

Base plans provided in digital format by URS, drawing file 09-079 – Base Plan.dwg, received July 15, 2010 and for the General Arrangement drawing file 09-079 Recommended Plan for Geotech-A.dwg, received April 4, 2011.

NO.	DATE	BY	REVISION

Geocres No. 41K-90

HWY. 17		PROJECT NO. 09-1111-0016	DIST.
SUBM'D. MWK	CHKD. JPD	DATE: 7/12/2012	SITE:
DRAWN: JFC	CHKD. MWK	APPD. JMAC	DWG. A1

Golder Associates

RECORD OF BOREHOLE No 10-1

SHEET 1 OF 1 **METRIC**

PROJECT 09-1111-0016			
G.W.P. 5022-07-00	LOCATION N 5144739.6 ;E 299616.8		ORIGINATED BY MWK
DIST	HWY 17	BOREHOLE TYPE Power Auger, 100 mm I.D. Continous Flight Hollow Stem Augers	COMPILED BY MWK
DATUM Geodetic		DATE March 10, 2011	CHECKED BY JPD

SOIL PROFILE			SAMPLES					
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
180.9	GROUND SURFACE							
0.0	TOPSOIL							
180.5			1	SS	12			
0.4	CLAYEY SILT Very soft Brown Wet		2	SS	1		180	
			3	SS	1		179	
178.4							178	
2.5	CLAY, some silt Very soft to stiff Brown Wet		4	SS	3		177	
			5	SS	1		176	
						∇	175	
			6	SS	WH		174	0 0 16 84
			7	SS	WH		173	
							172	
			8	SS	WH		171	
169.8			9	SS	WH		170	
11.1	END OF BOREHOLE NOTE 1. Water level in borehole measured at a depth of 5.5 m below ground surface (Elev. 175.4 m) on completion of drilling.							

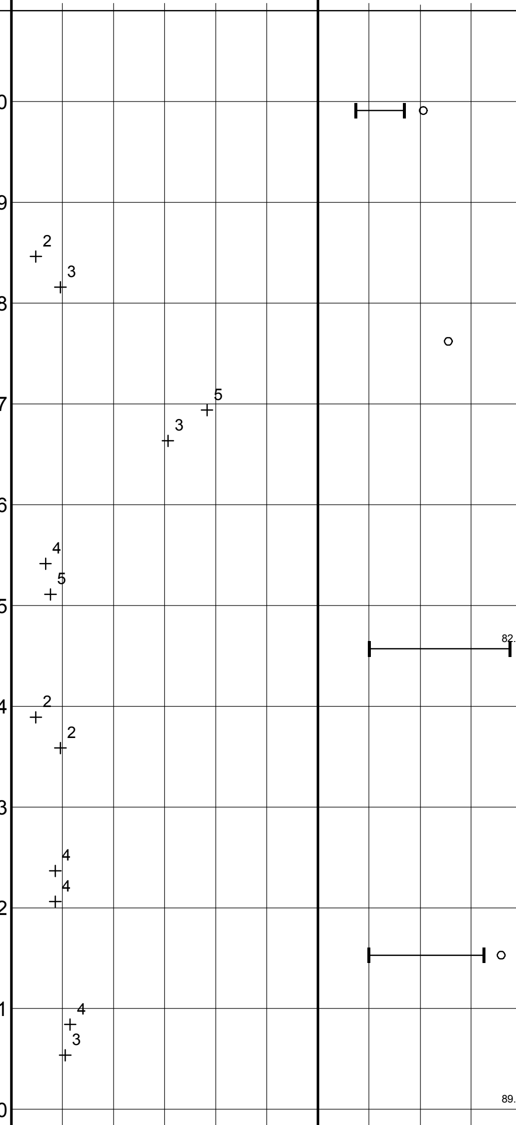
DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

SHEAR STRENGTH kPa
○ UNCONFINED + FIELD VANE
● QUICK TRIAXIAL × REMOULDED
20 40 60 80 100

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L

WATER CONTENT (%) 20 40 60

UNIT WEIGHT γ kN/m³



+ 3, × 3: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

Golder Associates

RECORD OF BOREHOLE No 10-2

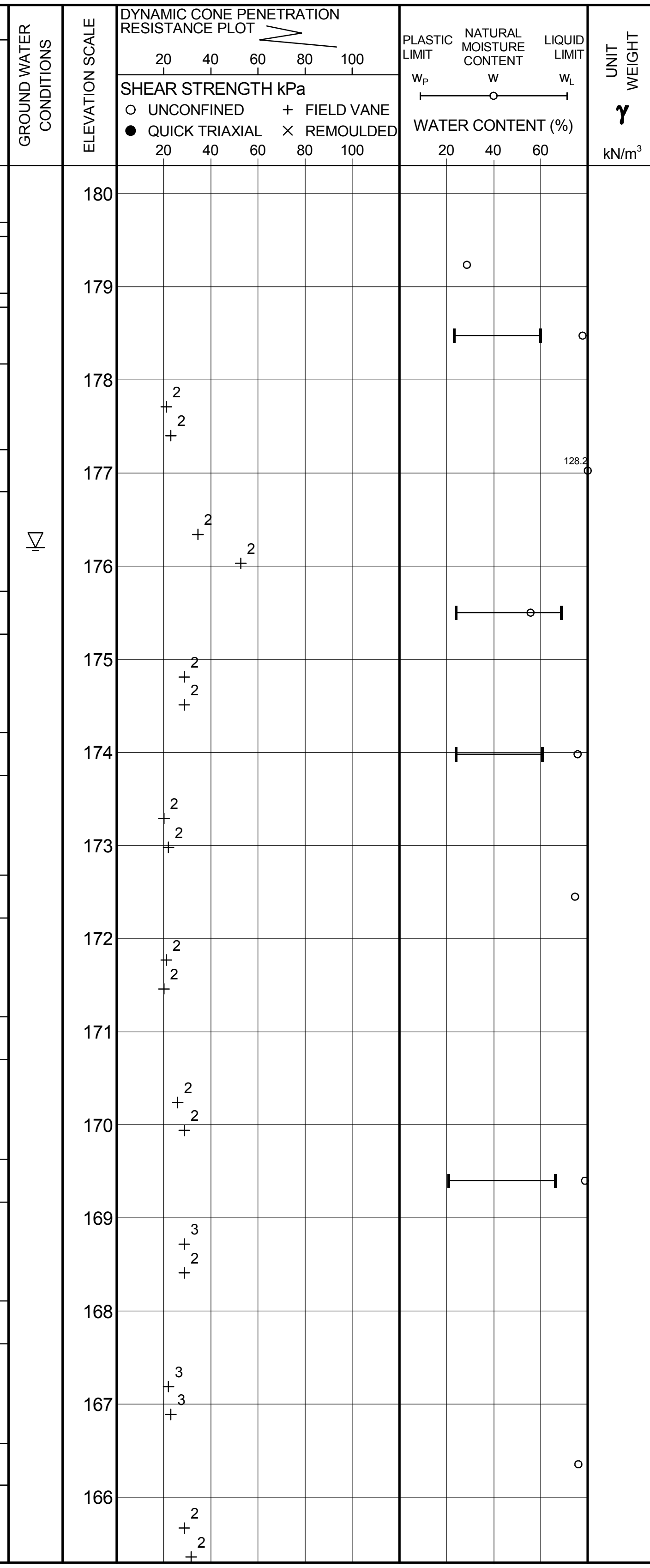
SHEET 1 OF 4 **METRIC**

PROJECT 09-1111-0016
G.W.P. 5022-07-00
DIST HWY 17
DATUM Geodetic

LOCATION N 5144738.5 ;E 299538.2
BOREHOLE TYPE 108 mm I.D. Continous Flight Hollow Stem Augers and NW Casing, Water Flush
DATE January 7, 2011

ORIGINATED BY MR
COMPILED BY MWK
CHECKED BY JPD

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
180.3	GROUND SURFACE				
180.0 / 0.0	TOPSOIL, silty Very loose Dark brown Moist	1	SS	2	
179.6 / 0.7	CLAYEY SILT, some sand, containing organics and roots Very soft Brown Moist	2	SS	4	OC=1.4%
178.9 / 1.4	SILT, some sand, some clay with fine sand layers, slightly organic Loose Brownish grey Moist	3	SS	2	
177.7 / 2.6	CLAY, some silt, with sandy silt lenses Soft Brownish grey Moist	4	SS	1	OC=11.1%
	SILTY CLAY with organic silt layers Soft to firm Brown Moist				
176.2 / 4.1	CLAY, some silt, with sandy silt lenses Firm to stiff Brown Wet	5	SS	3	
174.8 / 5.5	CLAY, some silt to SILTY CLAY, trace sand Soft to stiff Brown Wet	6	SS	WH	0 1 18 81
		7	SS	WH	
		8	SS	WR	
		9	SS	WR	
		10	SS	WR	
		11	SS	WR	



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$+^3, \times^3$: Numbers refer to Sensitivity O $^{3\%}$ STRAIN AT FAILURE

GTA-MTO 001 09-1111-0016.GPJ GAL-MISS.GDT 7/16/12 JFC

RECORD OF BOREHOLE No 10-2

SHEET 2 OF 4

METRIC

PROJECT 09-1111-0016

G.W.P. 5022-07-00

LOCATION N 5144738.5 ;E 299538.2

ORIGINATED BY MR

DIST HWY 17

BOREHOLE TYPE 108 mm I.D. Continous Flight Hollow Stem Augers and NW Casing, Water Flush

COMPILED BY MWK

DATUM Geodetic

DATE January 7, 2011

CHECKED BY JPD

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	SHEAR STRENGTH kPa (field vane, sensitivity)	WATER CONTENT	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	CLAY, some silt to SILTY CLAY, trace sand Soft to stiff Brown Wet	12	SS	WR	165			0 0 22 78
					164	+ 3, + 3		
					163			
		13	SS	1	162			
					161	+ 3, + 3		
					160			
		14	SS	WH	159			
					158	+ 2, + 3		
	Becoming grey at a depth of 23.5 m				157			
		15	SS	2	156			
					155	+ 3, + 3		
	Becoming stiff and containing grey silt lenses below a depth of 25.9 m				154			
		16	SS	2	153			
					152	+ 3, + 3		
					151			

Continued Next Page

$+^3, \times^3$: Numbers refer to Sensitivity

$\bigcirc^{3\%}$ STRAIN AT FAILURE

RECORD OF BOREHOLE No 10-2

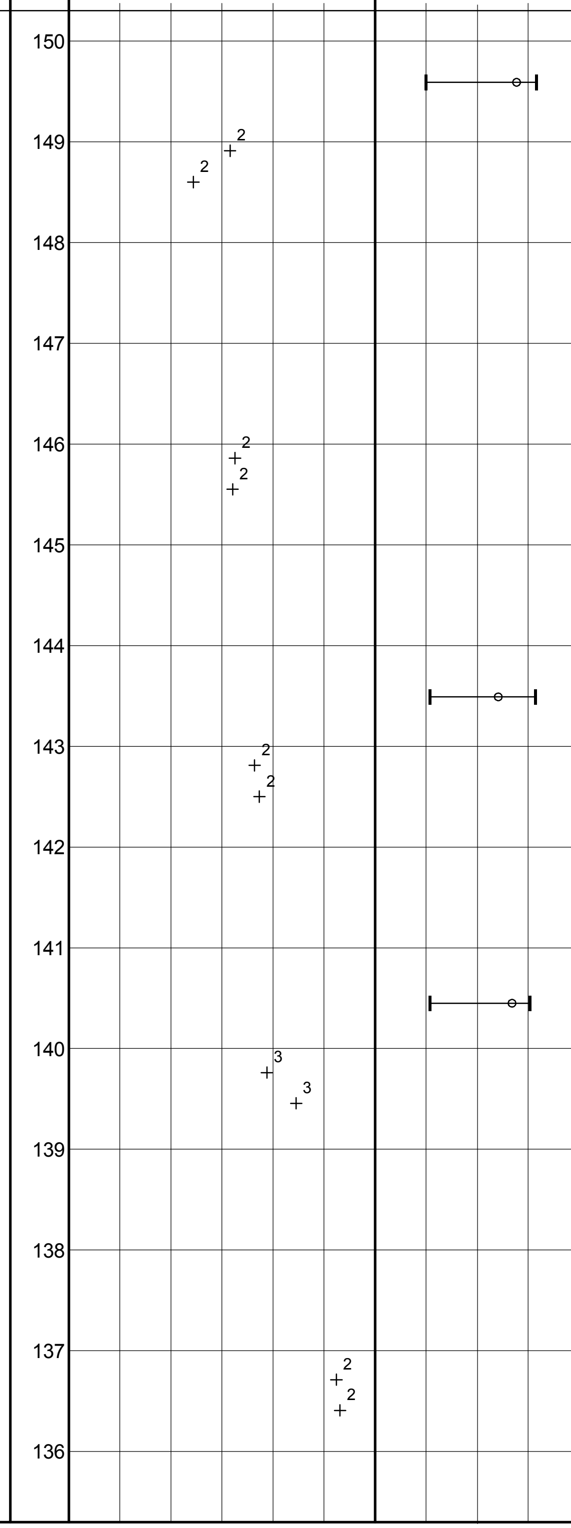
SHEET 3 OF 4 **METRIC**

PROJECT 09-1111-0016			
G.W.P. 5022-07-00	LOCATION N 5144738.5 ;E 299538.2		ORIGINATED BY MR
DIST HWY 17	BOREHOLE TYPE 108 mm I.D. Continous Flight Hollow Stem Augers and NW Casing, Water Flush		COMPILED BY MWK
DATUM Geodetic	DATE January 7, 2011		CHECKED BY JPD

SOIL PROFILE — --- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES
	CLAY, some silt to SILTY CLAY, trace sand Soft to stiff Brown Wet		17	SS	3
148.0 32.3	CLAY, some silt Stiff to very stiff Brown Moist		18	SS	4
			19	SS	4
	Containing grey silt lenses below a depth of 38.1 m		20	SS	4
			21	SS	6

Columns: GROUND WATER CONDITIONS; ELEVATION SCALE (150 to 136); DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100); SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED; 20, 40, 60, 80, 100); PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L — WATER CONTENT (%) (20, 40, 60); UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL



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+ 3, × 3: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

Golder Associates

RECORD OF BOREHOLE No 10-2

SHEET 4 OF 4 **METRIC**

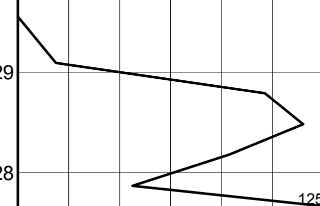
PROJECT 09-1111-0016			
G.W.P. 5022-07-00	LOCATION N 5144738.5 ;E 299538.2		ORIGINATED BY MR
DIST	HWY 17	BOREHOLE TYPE 108 mm I.D. Continous Flight Hollow Stem Augers and NW Casing, Water Flush	COMPILED BY MWK
DATUM Geodetic	DATE January 7, 2011		CHECKED BY JPD

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV / DEPTH	DESCRIPTION (SOIL PROFILE)	SAMPLE NUMBER	TYPE	"N" VALUES	SHEAR STRENGTH kPa	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	CLAY, some silt Stiff to very stiff Brown Moist	22	SS	5	+ 2 >150 +	wP – w – wL	
132.7 / 47.6	SAND and GRAVEL, some silt, trace clay Compact Brown Wet	23	SS	16		o	34 43 19 4
131.1 / 49.2	Medium to coarse, gravelly, SAND, some silt, trace clay Compact Brown Wet	24	SS	11		o	23 56 19 2
129.5 / 50.8	END OF BOREHOLE Start of Dynamic Cone Penetration Test (DCPT)						
127.6 / 52.7	END OF DCPT Refusal to further penetration (125 blows/0.23 m) NOTE: 1. Water level in borehole measured at a depth of 4.1 m below ground surface (Elev. 176.2 m) on completion of drilling.						

Elevation scale: 135, 134, 133, 132, 131, 130, 129, 128

DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100)



SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED (20 40 60 80 100)

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60

UNIT WEIGHT γ kN/m³

+ 3, × 3: Numbers refer to Sensitivity   O 3% STRAIN AT FAILURE

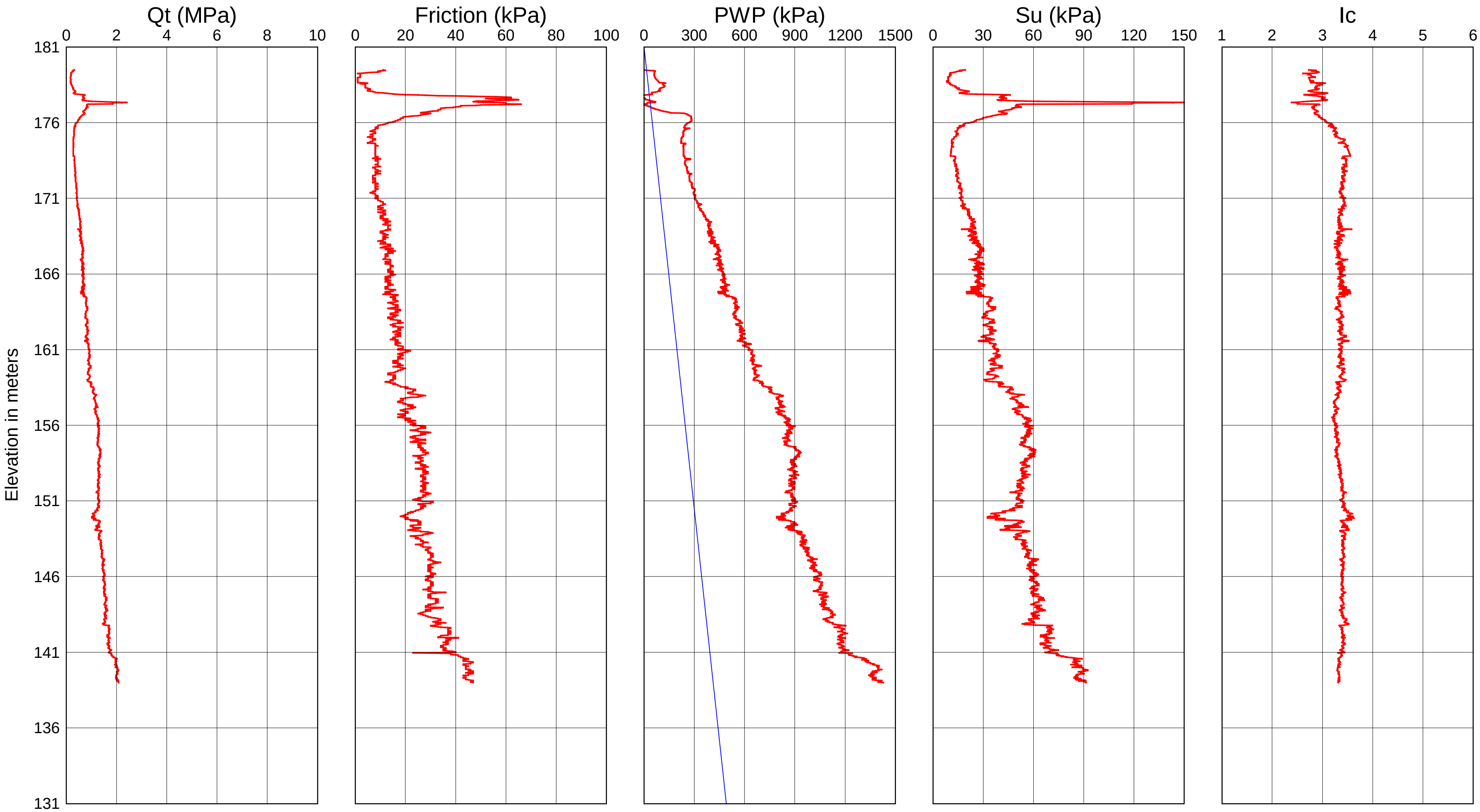
GTA-MTO 001 09-1111-0016.GPJ GAL-MISS.GDT 7/16/12 JFC

Cone Penetration Test - CPT 10-1

Test Date : 3/7/11
Location : N5144742.6 E299617.0

Operator : Golder Associates

Ground Surf. Elev. : 180.90
Water Table Depth : 0.00



Qt normalized for
unequal end area effects

Su = (Qt - SigmaV) / Nk
Nk = 15
Gamma = 17 kN/m³

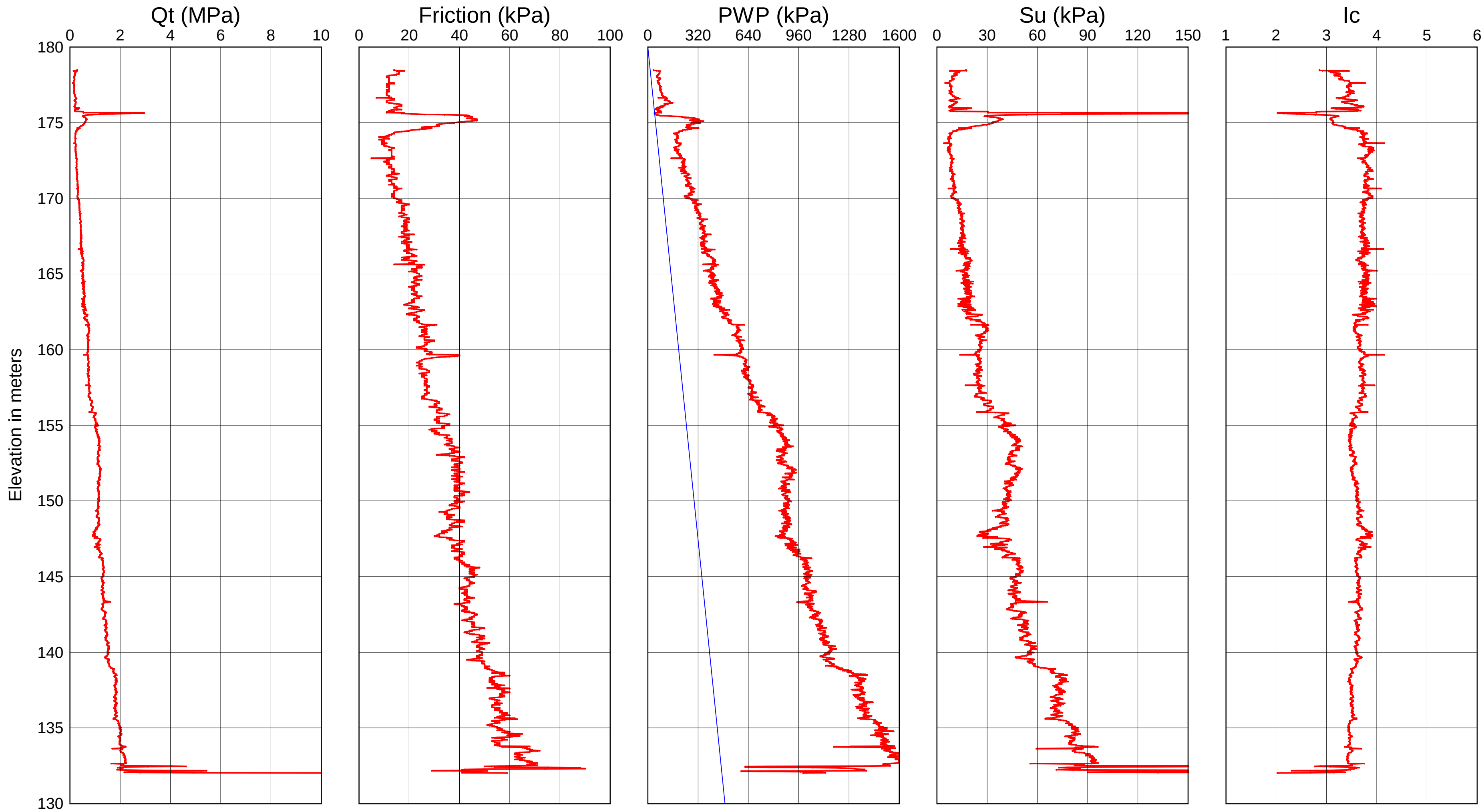
After Robertson and (Fear) Wride (1998)
Ic < 1.31 - Gravelly sands
1.31 <Ic < 2.05 - Clean to silty sand
2.05 <Ic < 2.60 - Silty sand to sandy silt
2.60 <Ic < 2.95 - Clayey silt to silty clay
2.95 <Ic < 3.60 - Clays

Cone Penetration Test - CPT 10-2

Test Date : 3/7/11
Location : N5144738.5 E299535.2

Operator : Golder Associates

Ground Surf. Elev. : 180.30
Water Table Depth : 0.00



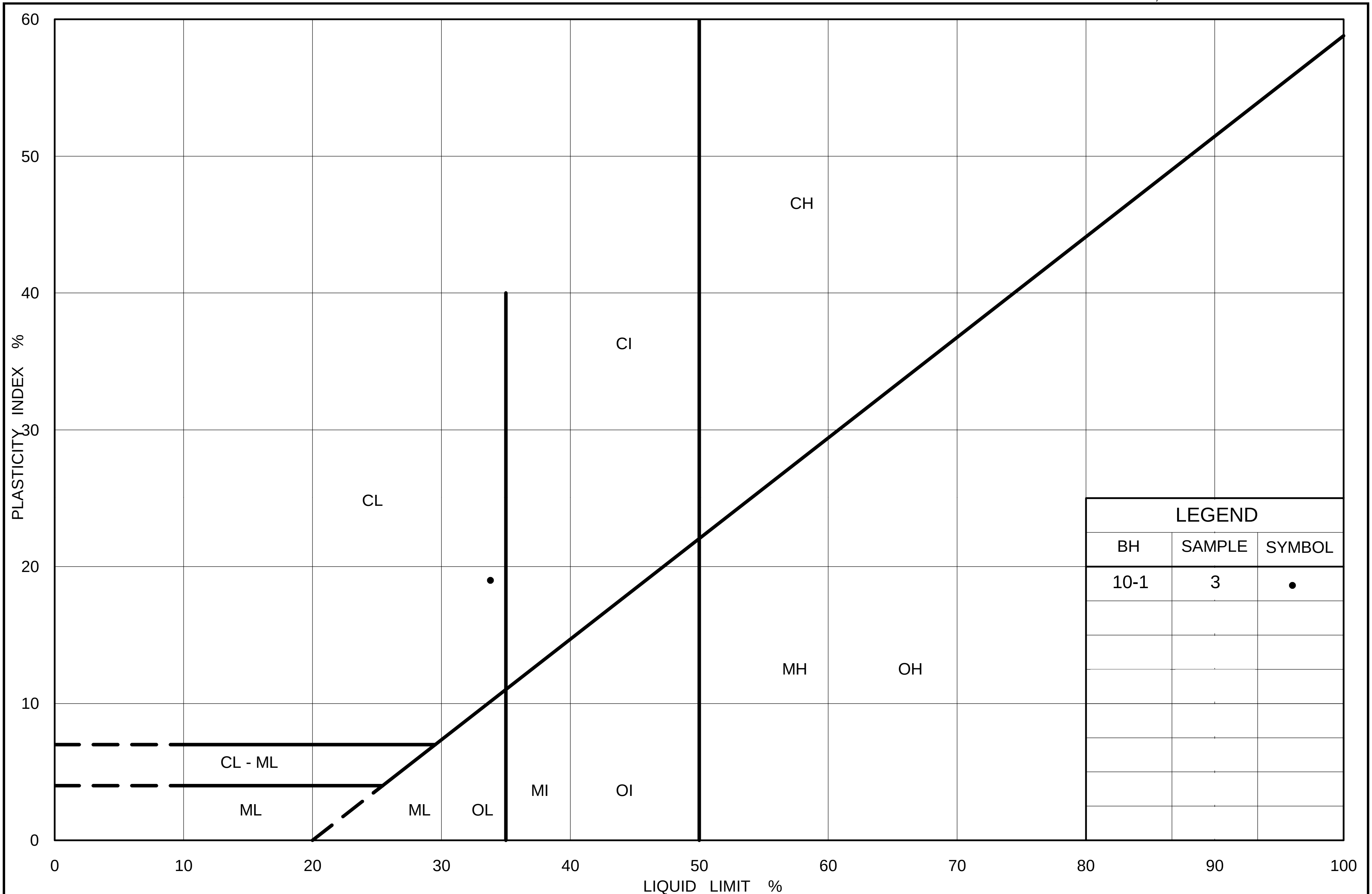
Qt normalized for
unequal end area effects

Su = (Qt - SigmaV) / Nk
Nk = 15
Gamma = 17 kN/m³

After Robertson and (Fear) Wride (1998)
Ic < 1.31 - Gravelly sands
1.31 <Ic < 2.05 - Clean to silty sand
2.05 <Ic < 2.60 - Silty sand to sandy silt
2.60 <Ic < 2.95 - Clayey silt to silty clay
2.95 <Ic < 3.60 - Clays

PROJECT NO. 09-1111-0016 DATE: 6/7/2011 DRAWN BY: MWK

Oct 75, FF-S-21



Ministry of Transportation

Ontario

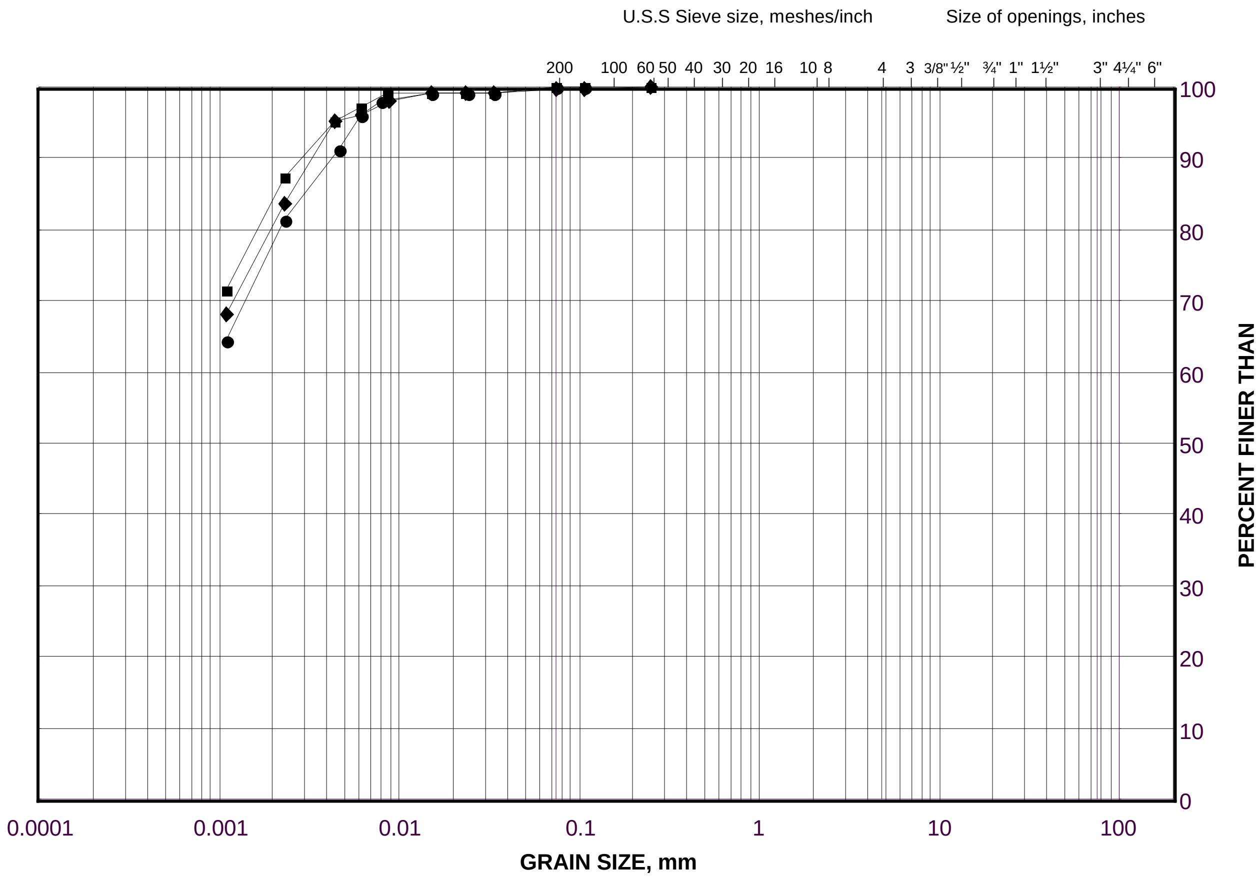
PLASTICITY CHART
Clayey Silt

Figure No. A.FE.1

Project No. 09-1111-0016

Checked By:

GRAIN SIZE DISTRIBUTION Clay	FIGURE A.FE.2



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

LEGEND

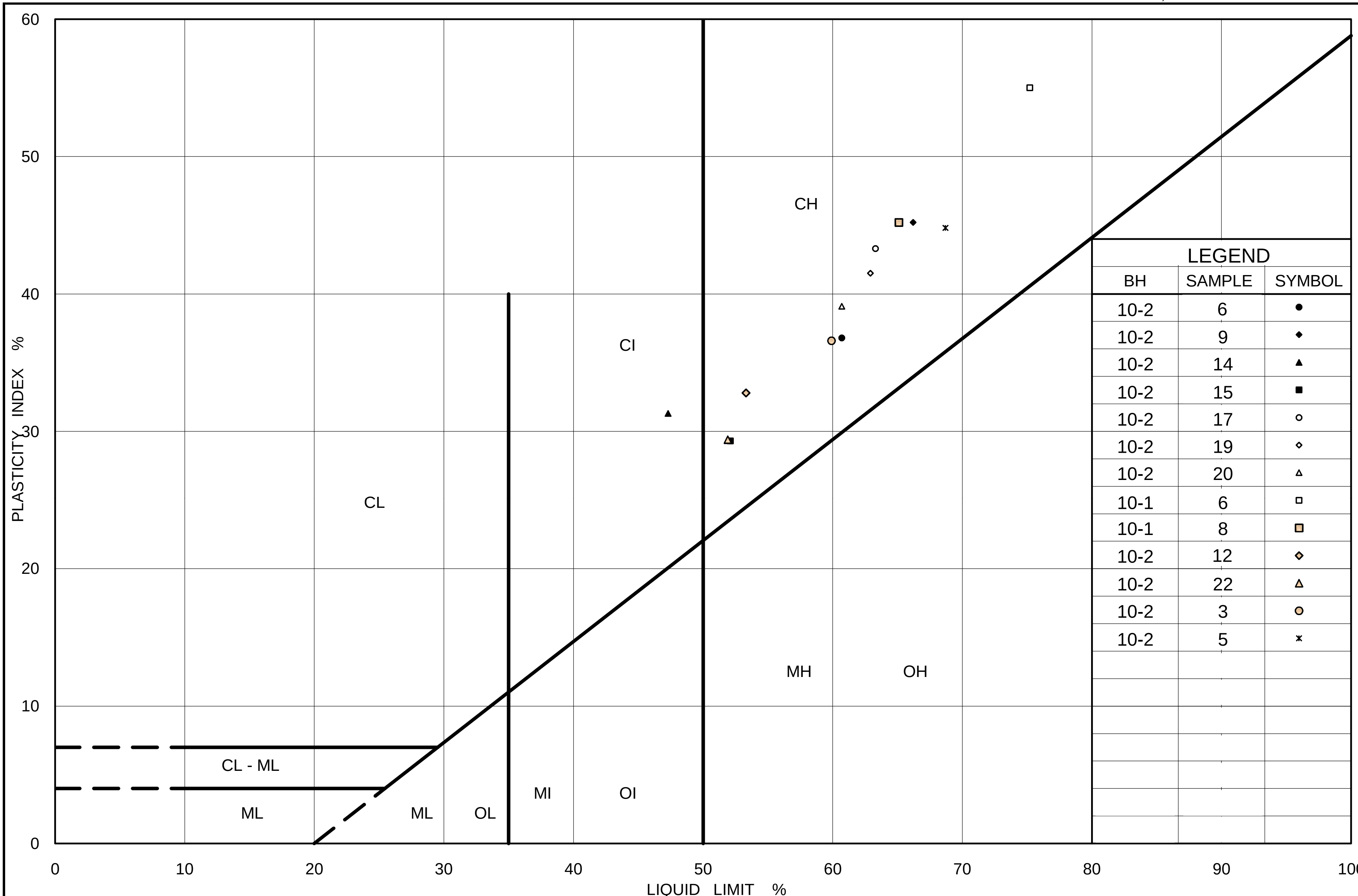
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	10-2	12	164.8
■	10-1	6	174.6
◆	10-2	6	173.9

Project Number: 09-1111-0016

Checked By: ____________

Golder Associates

Date: 01-Jun-11



LEGEND

BH	SAMPLE	SYMBOL
10-2	6	●
10-2	9	◆
10-2	14	▲
10-2	15	■
10-2	17	○
10-2	19	◇
10-2	20	△
10-1	6	□
10-1	8	□
10-2	12	◇
10-2	22	△
10-2	3	○
10-2	5	×

Ministry of Transportation
Ontario

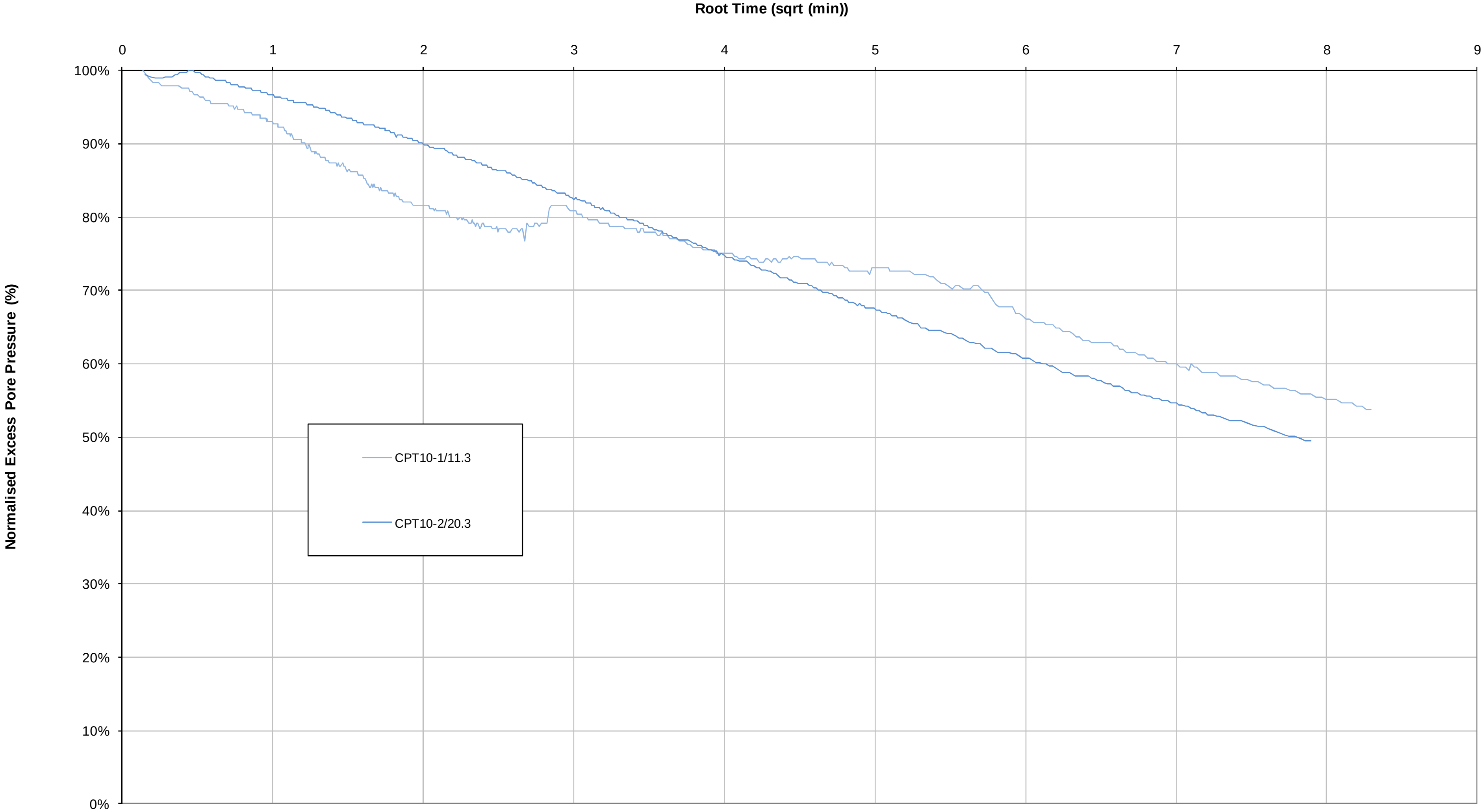
PLASTICITY CHART
Silty Clay to Clay

Figure No. A.FE.3

Project No. 09-1111-0016

Checked By:

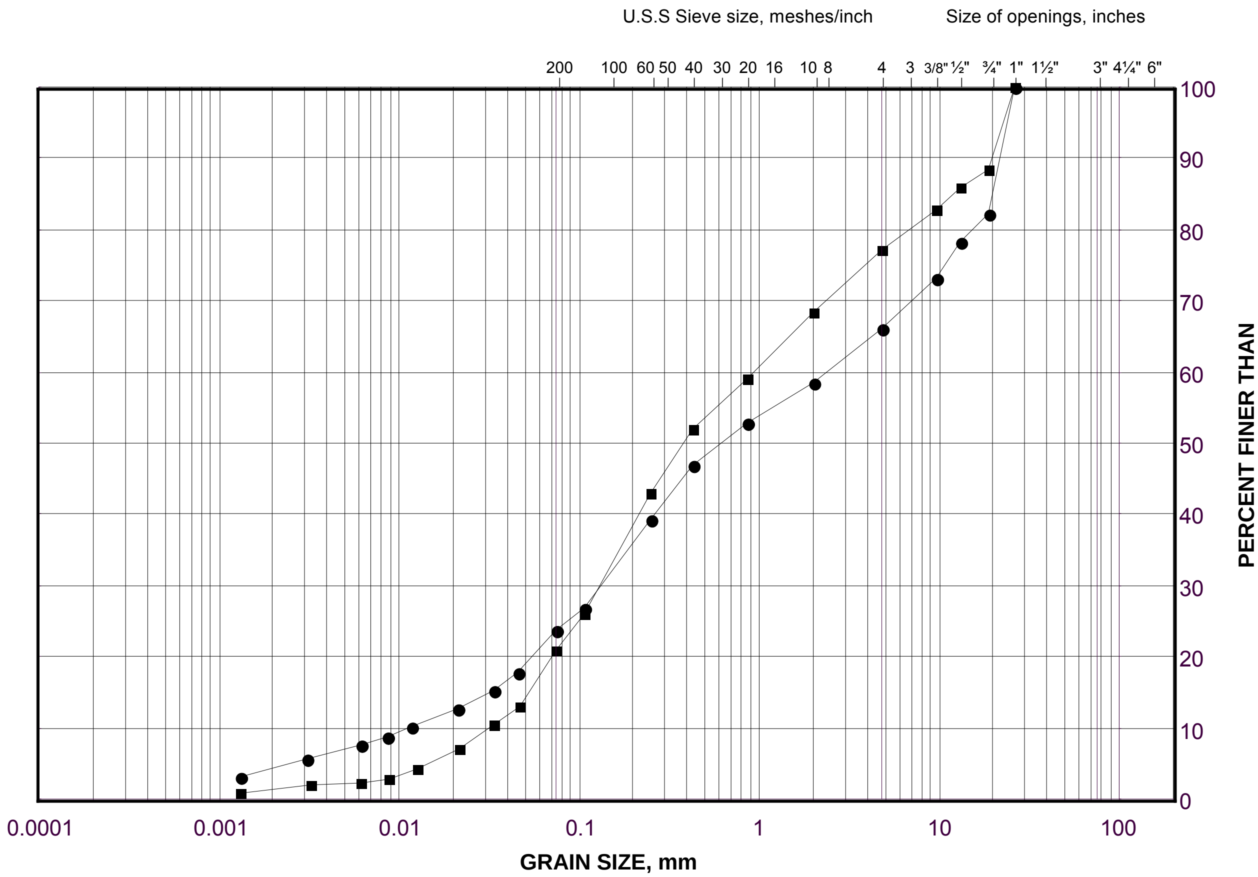
CPT Pore Water Pressure Dissipation Tests
Flyover East



Date: July 2011
Project No: 09-1111-0016

Analysis By: MWK **Reviewed By:** JPD

GRAIN SIZE DISTRIBUTION Sand and Gravel to Gravelly Sand	FIGURE A.FE.5



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	10-2	23	131.2
■	10-2	24	129.7

Project Number: 09-1111-0016

Checked By: ______________________

Golder Associates

Date: 01-Jun-11

TABLE A1 – SUMMARY OF FOUNDATION ENGINEERING PARAMETERS - FLYOVER EAST

Stratigraphic Unit	***Average Top Elevation (m)****	***Thickness** (m)***	***γ' (kN/m^3)***	***ϕ' (°)***	***c' (kPa)***	***s_u (kPa)***	***σ_p' (kPa)***	***e_o***	***C_c***	***C_r***	***E' (MPa)***	***$C_{\alpha(\varepsilon)}$ (%)***		***c_h (cm^2/s)***
												N/C	***O/C***	
Granular Fill (sub-excavate and replace near subsurface topsoil and soft clay soil)	180.6	2.0	21	32	0	--	--	--	--	--	15	--	--	--
Clayey Silt to Clay	178.6	4.5	17	21	0	12	55	2.0	1.0	0.1	--	0.5	0.05	1.58 x 10^{-3}
Clay (soft to firm)	174.1	9.3	17	21	0	12 - 28	55 - 129	2.0	1.0	0.1	--	0.5	0.05	1.58 x 10^{-3}
Clay (firm to stiff)	164.8	32.1	17	21	0	28 - 85	129 - 386	1.5	0.8	0.08	--	0.5	0.05	1.58 x 10^{-3}
Sand and Gravel	132.7	3.2	20	34	0	--	--	--	--	--	25	--	--	--

*Interpreted average Elevation of top of stratigraphic unit at Borehole and CPT locations (refer to Drawing A1)

**Interpreted average Thickness of stratigraphic unit at Borehole and CPT locations (refer to Drawing A1)

Prepared By: MWK

Reviewed By: JPD/JMAC

TABLE A2 – EVALUATION OF BRIDGE STRUCTURE FOUNDATION ALTERNATIVES - FLYOVER EAST

Foundation Option	Rank	Advantages	Disadvantages	Relative Costs	Risks / Consequences
Spread Footings on Overburden	Not feasible	• Relative ease of construction.	• Groundwater control required for excavation and during footing construction. • Large post-construction settlements. • Low geotechnical resistance at ULS and SLS of native soils and hence very large footings required.	• Lower relative cost than piled foundations.	• Footing size required to accommodate very low geotechnical resistances is not practical. • Very large post-construction settlements could not be tolerated by bridge structure.
Piles driven to bedrock or refusal in granular soils (50 m to 55 m long piles)	1	• Limited sub-excavation required for pile cap construction. • Negligible post-construction settlement. • Higher axial resistance than friction piles. • Fewer piles required than for friction piles option.	• Significant depth to refusal and/or bedrock will require very long piles which could result in installation difficulties.	• Higher cost associated with greater pile lengths. • Higher cost associated with provisions for re-driving piles for piles driven out of alignment.	• Piles driven out of alignment may require removal and replacement with new piles. • The abutment/pier design should be flexible enough to allow for installation of extra piles in the footing area, if deemed necessary during construction.
Friction Piles (35 m to 40 m long piles)	2	• Limited sub-excavation required for pile cap construction. • Minor post-construction settlement. • Shorter piles required than for piles driven to refusal option.	• Lower pile capacity than piles driven to refusal.	• Lower cost associated with shorter pile lengths and lighter pile section. • Higher cost associated with additional piles due to lower axial capacity. • Additional cost for pile load tests.	• Lower pile capacity will require more piles at each foundation unit. • May require pile load tests to verify pile capacity.

Prepared By: MWK

Reviewed By: JPD/JMAC

TABLE A3 – EVALUATION OF APPROACH EMBANKMENT FOUNDATION STABILITY/SETTLEMENT MITIGATION ALTERNATIVES - FLYOVER EAST

Stability / Settlement Mitigation Option	*Rank*	*Advantages*	*Disadvantages*	*Relative Costs*	*Risks / Consequences*
Staged construction (with wick drains, 10 m wide by 2 m high toe berms and 2 m subexcavate and replace) (7 stages) (approximately 9 years of construction delays for staging)	1	• Smaller embankment footprint and less land acquisition requirements as compared with toe berms only option.	• Somewhat greater quantities of fill required for replacement in subexcavated area due to berms. • Delay of approximately 9 years during staged construction and preloading. • Large post construction settlement. • Large downdrag loads reduce pile capacity. • EPS required to maintain front slope stability and to top-up to mitigate long-term settlements.	• $2,621,250 (wick drains at 1.5 m spacing) + • $140,000 (berms) + • $315,000 (subexcavate / replace) + • $4,171,600 cost of EPS to mitigate long-term settlements.	• Staged construction sequence required with potential for additional delays during construction depending on monitoring. • Large post-construction settlements will require long-term maintenance. • Nominal size toe berms are required for stability, increasing footprint. • Some secondary consolidation (creep) will occur. • Potential need to acquire some additional lands for right-of-way.

TABLE A3 – EVALUATION OF APPROACH EMBANKMENT FOUNDATION STABILITY/SETTLEMENT MITIGATION ALTERNATIVES - FLYOVER EAST

Stability / Settlement Mitigation Option	*Rank*	*Advantages*	*Disadvantages*	*Relative Costs*	*Risks / Consequences*
Toe berms - up to 47 m wide (with 2 m subexcavate and replace) (with up to 80 year preload)	4	• Standard construction operation. • No construction delays associated with staging.	• Generation of larger volume of excess excavation spoil due to very large toe berm footprint. • Greater quantities of fill required for very large berms and for subexcavate and replace area. • Large embankment footprint. • Large post-construction settlement. • Large downdrag loads reduce pile capacity. • EPS required to maintain front slope stability and top-up to mitigate long-term settlements. • Very long preload period required to mitigate settlements	• $1,102,520 (subexcavate and replace/berm construction) + • land acquisition costs + • $4,171,600 cost of EPS to mitigate long-term settlements and front slope stability.	• Risk of instability (low). • Secondary consolidation (creep) will occur. • Large post-construction settlements will require long-term maintenance. • Likely need to acquire additional right-of-way due to very large berm size.

TABLE A3 – EVALUATION OF APPROACH EMBANKMENT FOUNDATION STABILITY/SETTLEMENT MITIGATION ALTERNATIVES - FLYOVER EAST

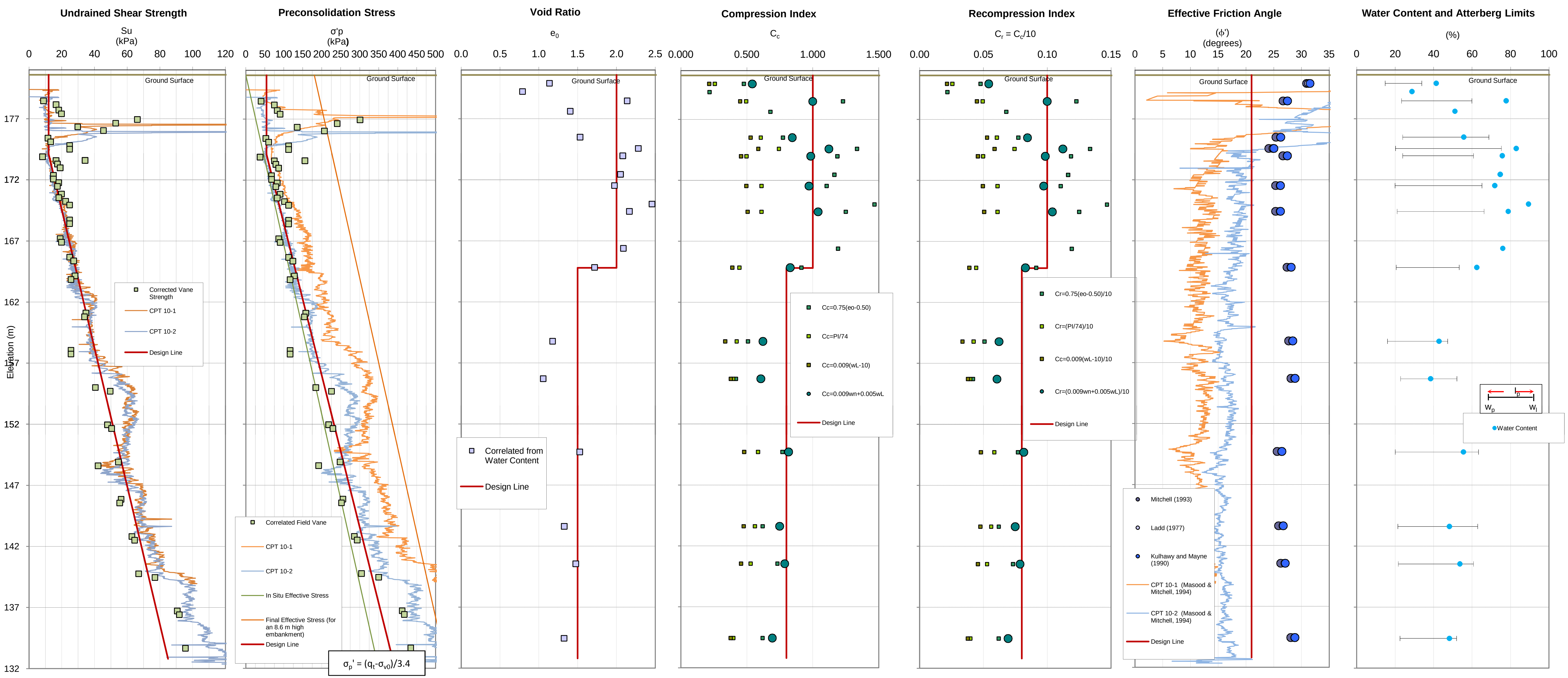
Stability / Settlement Mitigation Option	*Rank*	*Advantages*	*Disadvantages*	*Relative Costs*	*Risks / Consequences*
Partial Lightweight Fill (EPS) (with 2 m subexcavate and replace) (with up to 80 year preload)	3	• Standard construction operation. • No construction delays associated with staging or preloading. • Reduced secondary (creep) consolidation settlement. • Generation of smaller volume of excess excavation spoil since no toe berms. • Smaller quantities of fill required for subexcavate and replace since no toe berms. • Smaller embankment footprint.	• Higher cost for specialized materials. • EPS required to maintain front slope stability and top-up to mitigate long-term settlements. • Some post-construction settlements will occur. • Very long preload period required to mitigate settlements	• $315,000 (sub-excavate/replace)+ • $5,781,200 (partial EPS and EPS to mitigate long-term settlements and front slope stability).	• Risk of instability (low). • Secondary consolidation (creep) will occur. • Post-construction settlement will require long-term maintenance • Potential for smaller property acquisition needs.
Full Lightweight Fill (EPS) (with 2 m subexcavate and replace)	2	• Standard construction operation. • No construction delays associated with staging or preloading. • Minimized post-construction settlement. • Smallest embankment footprint.	• Higher cost for specialized materials • Restricted use of EPS within the embankment cross-section to above water table.	• $315,000 (sub-excavate/replace)+ • $9,007,600 (full EPS and EPS to mitigate long-term settlements and front slope stability).	• Risk of instability (low). • Risk of long term settlement of foundation soils (low).

Prepared By: MWK

Reviewed By: JPD/JMAC

Summary of Engineering Parameters for Cohesive Deposits
Flyover East

Figure A1



NOTES:

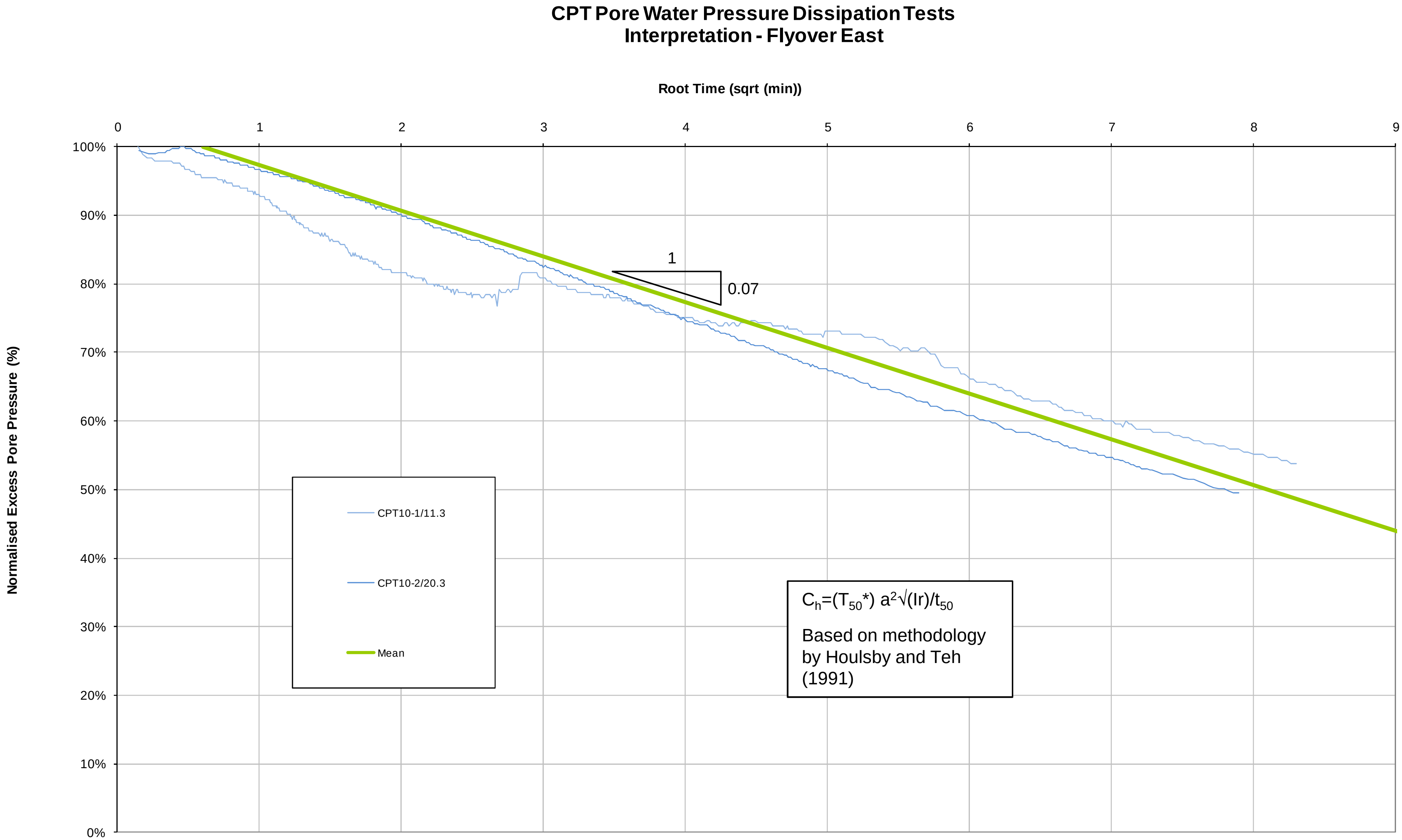
Average ground surface at the borehole locations is about Elevation 180.6 m

Average elevation of bottom of cohesive deposit at the borehole locations is about 132.8 m

Date:	Jun-11	**DB:**	MWK
Project No:	09-1111-0016	**CHK:**	JPD

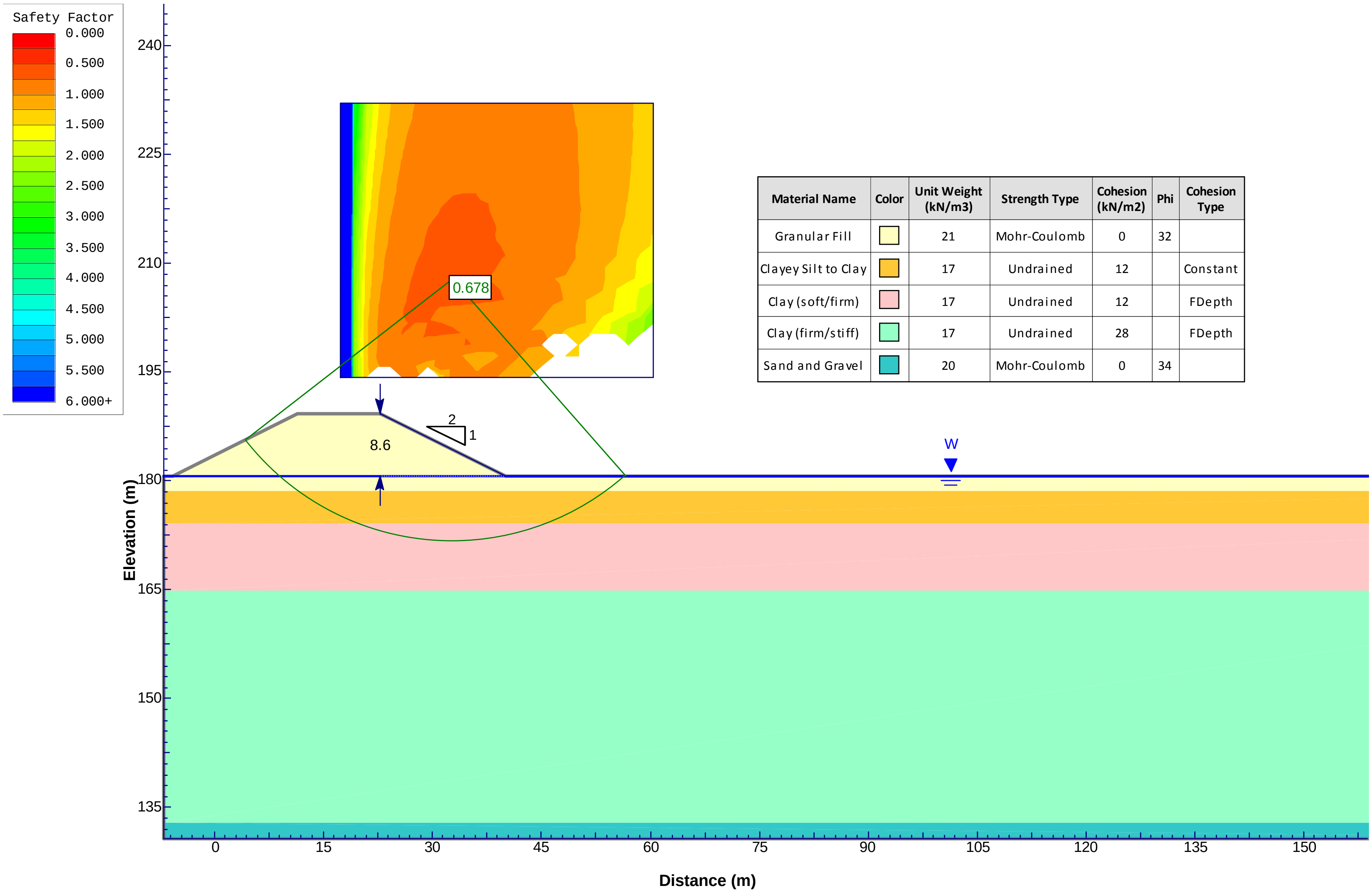
Golder Associates

#N/A


CPT Pore Water Pressure Dissipation Tests
Interpretation - Flyover East
Root Time (sqrt (min))
0
1
2
3
4
5
6
7
8
9
100%
90%
80%
70%
60%
50%
40%
30%
20%
10%
0%
Normalised Excess Pore Pressure (%)
1
0.07
CPT10-1/11.3
CPT10-2/20.3
Mean
$C_h=(T_{50}^*)\ a^2\sqrt{(Ir)}/t_{50}$
Based on methodology by Houlsby and Teh (1991)

Slope Stability – Total Stress Analysis – 2.0 m Subexcavate and Replace Only

Figure A3-1



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kN/m2)	Phi	Cohesion Type
Granular Fill		21	Mohr-Coulomb	0	32	
Clayey Silt to Clay		17	Undrained	12		Constant
Clay (soft/firm)		17	Undrained	12		FDepth
Clay (firm/stiff)		17	Undrained	28		FDepth
Sand and Gravel		20	Mohr-Coulomb	0	34	

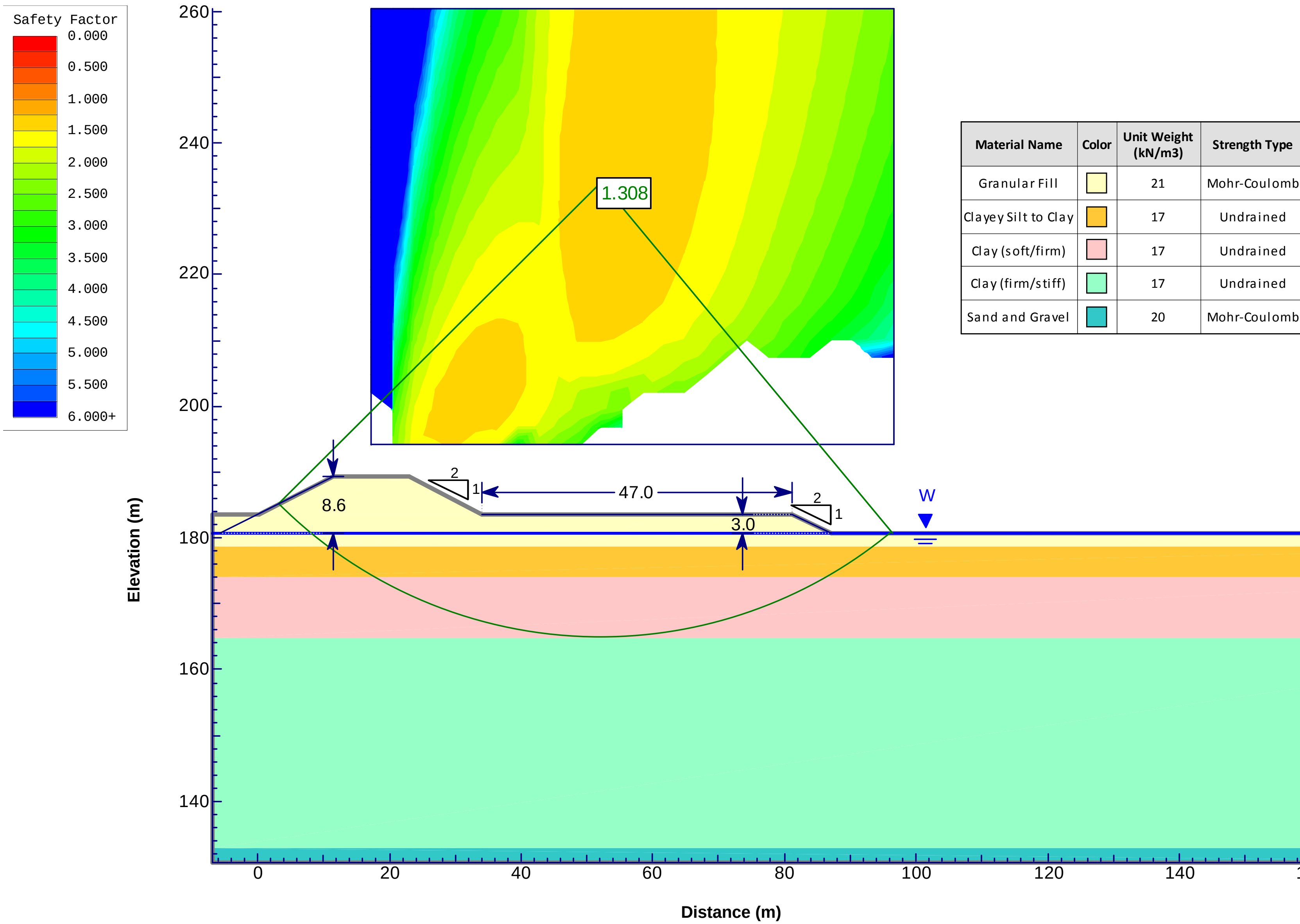
Date: September 2011
Project No: 09-1111-0016

Analysis By: MWK **Reviewed By:** JPD

Golder Associates

Slope Stability – Total Stress Analysis – Stabilizing Berms

Figure A3-2



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kN/m2)	Phi	Cohesion Type
Granular Fill		21	Mohr-Coulomb	0	32	
Clayey Silt to Clay		17	Undrained	12		Constant
Clay (soft/firm)		17	Undrained	12		FDepth
Clay (firm/stiff)		17	Undrained	28		FDepth
Sand and Gravel		20	Mohr-Coulomb	0	34	

Date: September 2011

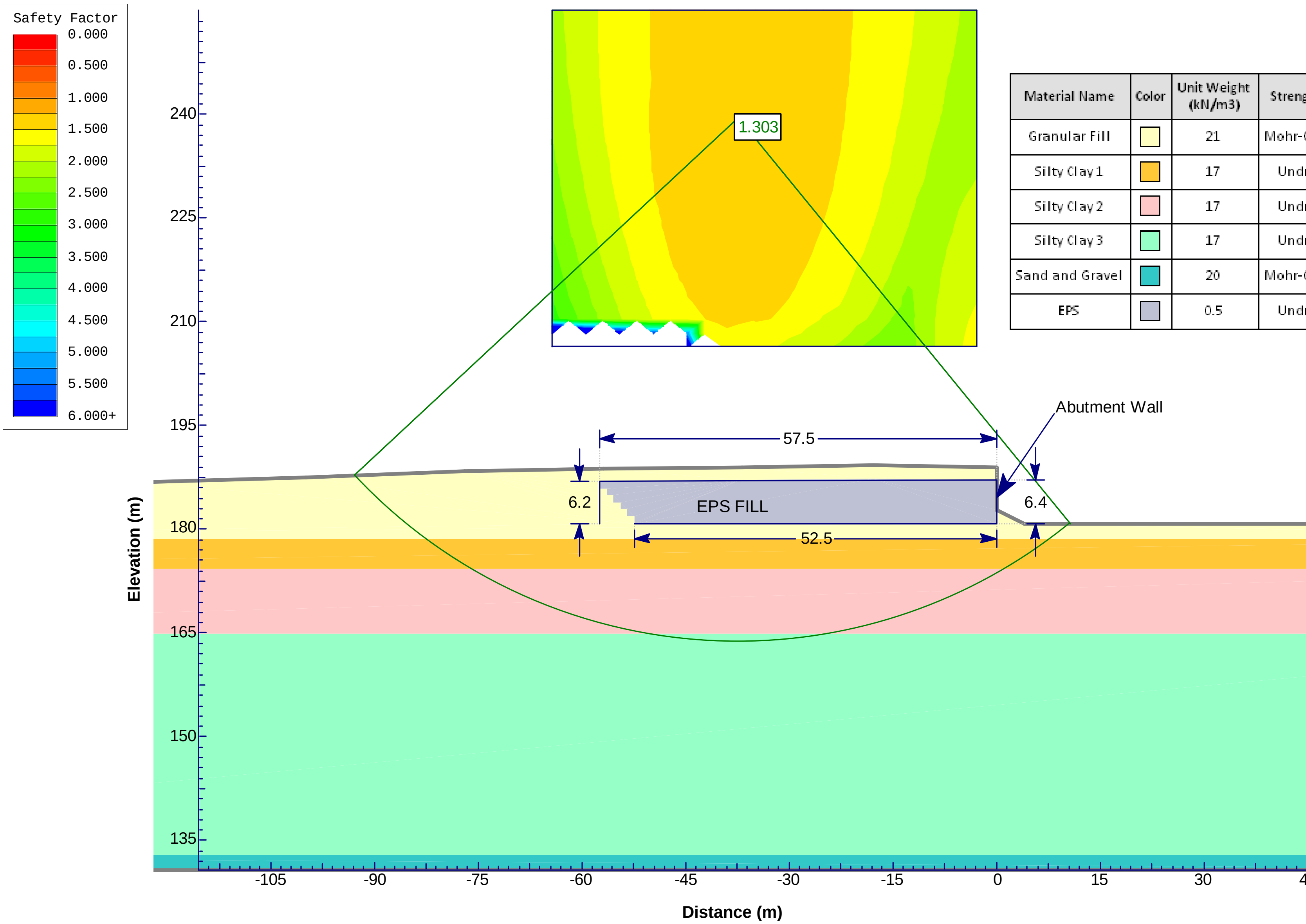
Project No: 09-1111-0016

Analysis By: MWK **Reviewed By:** JPD

Golder Associates

Slope Stability – Total Stress Analysis – Front Slope Stability (with EPS)

Figure A3-3



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kN/m2)	Phi	Cohesion Type
Granular Fill		21	Mohr-Coulomb	0	32	
Silty Clay 1		17	Undrained	12		Constant
Silty Clay 2		17	Undrained	12		FDepth
Silty Clay 3		17	Undrained	28		FDepth
Sand and Gravel		20	Mohr-Coulomb	0	34	
EPS		0.5	Undrained	15		Constant

Date: September 2011
Project No: 09-1111-0016

Analysis By: MWK **Reviewed By:** JPD

Golder Associates

APPENDIX B

Realigned Bar River Road Flyover (Flyover West)

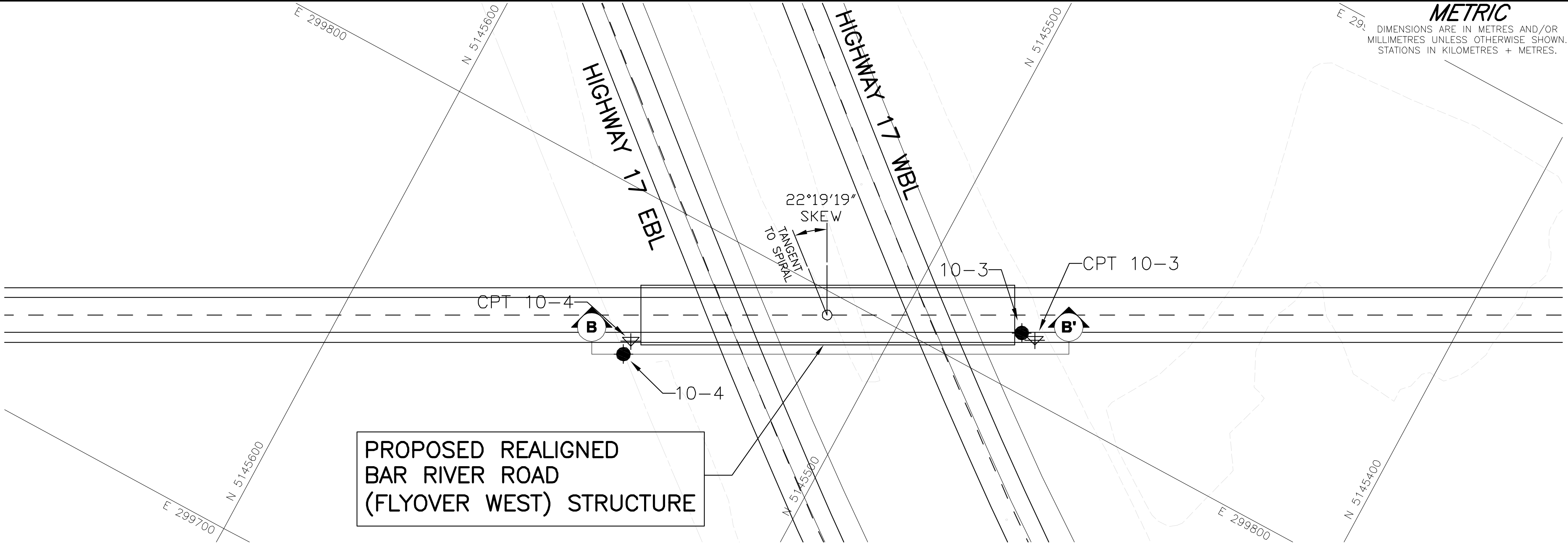
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.
GWP No. 5022–07–00

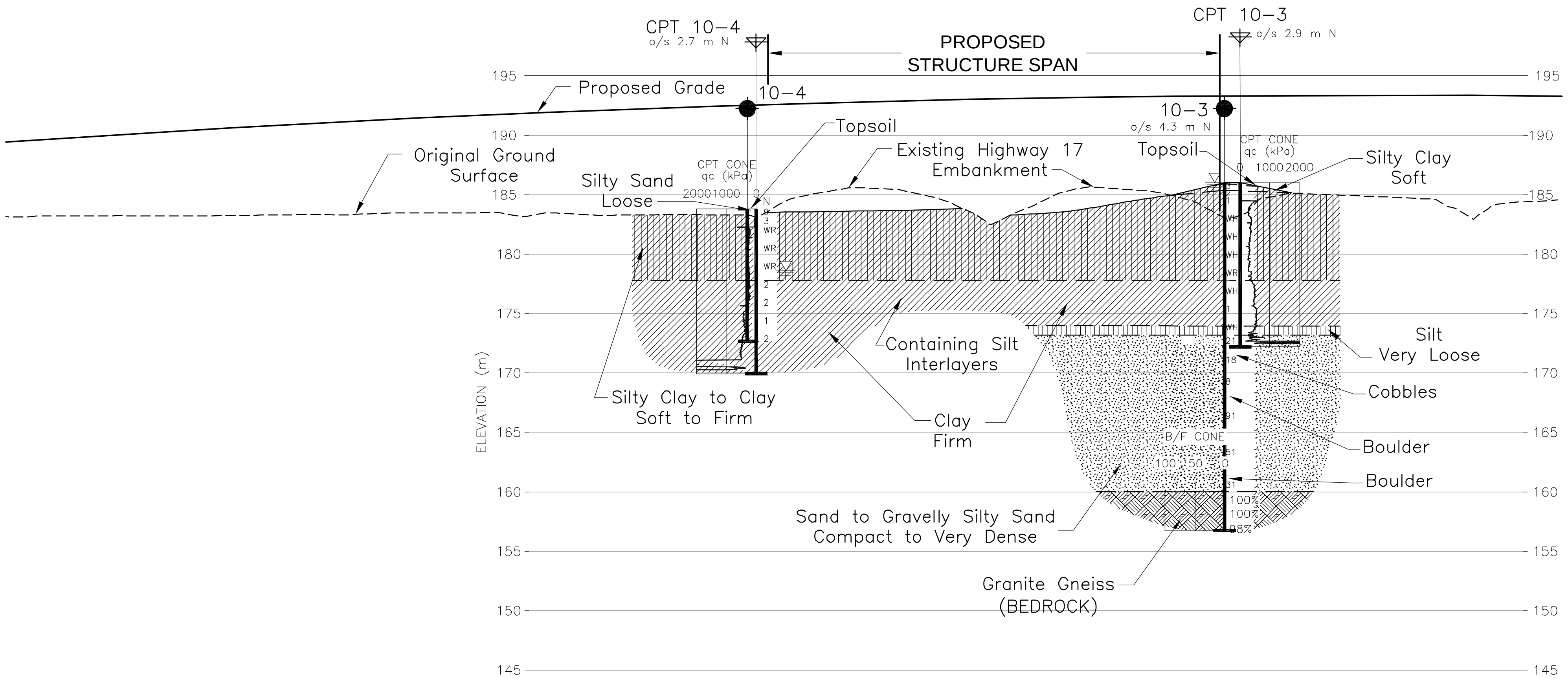
SHEET

HIGHWAY 17
FLYOVER WEST STRUCTURE
BOREHOLE LOCATIONS AND SOIL STRATA

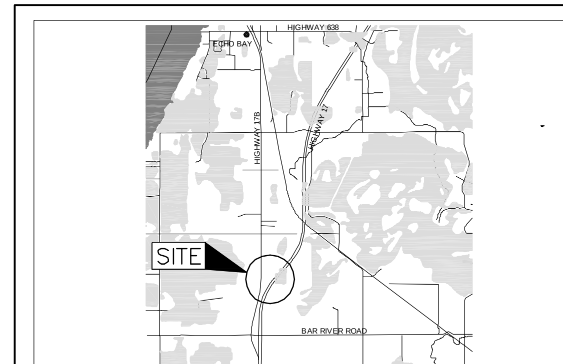
Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



PLAN
SCALE
10 0 10 20 m



PROFILE B–B'
HORIZ. SCALE
10 0 10 20 m
5 0 5 10 m
VERT. SCALE



KEY PLAN
SCALE
700 0 700 1400m

LEGEND

- ● Borehole – Current Investigation
- CPT – Current Investigation
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- 100% Rock Quality Designation (RQD)
- WL upon completion of drilling
- CPT tip resistance qc (kPa)

BOREHOLE CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
10–3	186.0	5145477.2	299811.1
10–4	183.8	5145545.8	299769.2

CPT CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
CPT 10–3	186.0	5145474.2	299811.1
CPT 10–4	183.8	5145545.8	299772.2

NOTES

This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contracts Documents.

The boundaries between soil strata have been established only at borehole locations. Between boreholes the boundaries are assumed from geological evidence.

The complete Foundation Investigation and Design Report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with Section GC 2.01 of OPS General Conditions.

REFERENCE

Base plans provided in digital format by URS, drawing file 09–079 – Base Plan.dwg, received July 15, 2010 and for the General Arrangement drawing file 09–079 Recommended Plan for Geotech–A.dwg, received April 4, 2011.

NO.	DATE	BY	REVISION

Geocres No. 41K–90

HWY. 17		PROJECT NO. 09–1111–0016	DIST.
SUBM'D. MWK	CHKD. JPD	DATE: 7/12/2012	SITE:
DRAWN: JFC	CHKD. MWK	APPD. JMAC	DWG. B1

Golder Associates

RECORD OF BOREHOLE No 10-3

 METRIC

PROJECT 09-1111-0016

G.W.P. 5022-07-00 LOCATION N 5145477.2 ;E 299811.1 ORIGINATED BY MR

DIST ____ HWY 17 BOREHOLE TYPE 108 mm I.D. Continous Flight Hollow Stem Augers and NW Casing, Water Flush COMPILED BY MWK

DATUM Geodetic DATE January 12, 2011 CHECKED BY JPD

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
186.0	GROUND SURFACE
186.0 / 0.1	TOPSOIL, silty Dark brown Moist
185.4 / 0.6	SILTY CLAY, trace roots, slightly organic Soft Grey Moist
	CLAY, some silt to SILTY CLAY Soft to firm Brownish grey to brown Moist
177.8 / 8.2	CLAY, some silt, with grey silt interlayers Firm Brown Wet
174.0 / 12.0	SILT, trace to some fine sand, trace clay Very loose Grey Wet
173.2 / 12.8	SAND, some gravel, some silt, trace clay Compact Brown Wet
171.7	COBBLES
171.4 / 14.6	

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	3	OC=2.4%
2	SS	2	
3	SS	1	
4	SS	WH	
5	SS	WH	0 0 20 80
6	SS	WH	
7	SS	WR	
8	SS	WH	
9	SS	1	
10A 10B	SS	WH	0 10 87 3
11	SS	21	17 64 18 1
12	SS	18	

Column headings: SOIL PROFILE (ELEV DEPTH, DESCRIPTION, STRAT PLOT); SAMPLES (NUMBER, TYPE, "N" VALUES); GROUND WATER CONDITIONS; ELEVATION SCALE (185 to 172); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED, 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL.

Field vane sensitivity values (by elevation, top to bottom): 2, 3, 3, 3, 3, 3, 3, 3, 3, 4, 4, 4, 4, 4.

Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No 10-3

SHEET 2 OF 3 **METRIC**

PROJECT 09-1111-0016
G.W.P. 5022-07-00 LOCATION N 5145477.2 ;E 299811.1 ORIGINATED BY MR
DIST HWY 17 BOREHOLE TYPE 108 mm I.D. Continous Flight Hollow Stem Augers and NW Casing, Water Flush COMPILED BY MWK
DATUM Geodetic DATE January 12, 2011 CHECKED BY JPD

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	SAND and GRAVEL, some silt, trace clay, Boulder at 17.6 m depth Compact to loose Brown Wet				170		
		13	SS	8	169		48 36 13 3
168.4 17.6	BOULDER (Granite)	14	RC	REC 89%			RQD = 61%
168.0 18.0	Gravelly Silty SAND, trace clay, Boulder at a 24.6 m depth Very dense Brown Wet				168		
					167		
		15	SS	91	166	o	26 48 23 3
					165		
					164		
		16	SS	51	163		
					162		
161.4 24.6	BOULDER (Granite)	17	RC	REC 94%	161		RQD = 22%
160.9 25.1	SAND, some silt, trace gravel Dense Brown Wet	18	SS	31		o	1 86 13 0
160.0 26.0	BEDROCK Refer to Record of Drillhole Log 10-3 for details	1	RC	REC 100%	160		RQD = 100%
		2	RC	REC 100%	159, 158		RQD = 100%
		3	RC	REC 98%	157		RQD = 98%
156.7 29.3							

Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity   ○ 3% STRAIN AT FAILURE

Golder Associates

RECORD OF BOREHOLE No 10-3

SHEET 3 OF 3 **METRIC**

PROJECT 09-1111-0016

G.W.P. 5022-07-00

LOCATION N 5145477.2 ;E 299811.1

ORIGINATED BY MR

DIST HWY 17

BOREHOLE TYPE 108 mm I.D. Continous Flight Hollow Stem Augers and NW Casing, Water Flush

COMPILED BY MWK

DATUM Geodetic

DATE January 12, 2011

CHECKED BY JPD

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED (20 40 60 80 100)	PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) (20 40 60)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	--- CONTINUED FROM PREVIOUS PAGE ---										
	END OF BOREHOLE NOTE: 1. Water level in borehole measured at ground surface (Elev. 186.0 m) on completion of drilling.										

$+^3, \times^3$: Numbers refer to Sensitivity

O $^{3\%}$ STRAIN AT FAILURE

RECORD OF DRILLHOLE: 10-3

PROJECT: 09-1111-0016

SHEET 1 OF 1

LOCATION: N 5145477.2 ;E 299811.1

DRILLING DATE: January 12, 2011

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: D-120 Track

DRILLING CONTRACTOR: Walker Drilling

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

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

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. DEPTH (m)	RUN No.	Diametral Point Load Index (MPa)	NOTES WATER LEVELS INSTRUMENTATION
		BEDROCK SURFACE	160.03			
26	NQ ROCK CORE January 12, 2011	Granitic Gneiss Slightly weatered to fresh, strong, pink and black	25.97	1	•	(Axial)
27				2	•	(Axial)
					82	(UCS = 82 MPa)
28					•	(Axial)
29			156.71	3	•	(Axial)
		END OF DRILLHOLE	29.29			

Column headings also printed: SYMBOLIC LOG; PENETRATION RATE min/(m); COLOUR % RETURN; FLUSH; RECOVERY TOTAL CORE % 80 60 40 20; SOLID CORE % 80 60 40 20; R.Q.D. % 80 60 40 20; FRACT. INDEX PER 0.3 m 5 10 15 20; DISCONTINUITY DATA: B Angle 0 90 180 270; DIP w.r.t. CORE AXIS 0 30 60 90; TYPE AND SURFACE DESCRIPTION; Jr; Ja; Jn; HYDRAULIC CONDUCTIVITY K, cm/sec 10^{-6} 10^{-5} 10^{-4} 10^{-3}; Diametral Point Load Index (MPa) 2 4 6; RMC -Q' AVG.

Depth scale: 26 27 28 29 30 31 32 33 34 35

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: MR

CHECKED: JPD

GTA-RCK 004 09-1111-0016.GPJ GAL-MISS.GDT 7/16/12 JFC

Golder Associates

RECORD OF BOREHOLE No 10-4

SHEET 1 OF 1 **METRIC**

PROJECT 09-1111-0016
G.W.P. 5022-07-00
LOCATION N 5145545.8 ;E 299769.2
ORIGINATED BY MWK
DIST HWY 17
BOREHOLE TYPE Power Auger, 100 mm I.D. Continous Flight Hollow Stem Augers
COMPILED BY MWK
DATUM Geodetic
DATE March 9, 2011
CHECKED BY JPD

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	FIELD VANE (sensitivity)	PLASTIC LIMIT / NATURAL MOISTURE / LIQUID LIMIT	UNIT WEIGHT	GR SA SI CL
183.8	GROUND SURFACE									
0.0	TOPSOIL									
183.5 / 0.3	Silty SAND, trace clay, Loose, Brown, Moist	1	SS	9		183				
183.0 / 0.8	CLAY, some silt, Soft to firm, Brown to grey, Wet	2	SS	3						
		3	SS	WR		182	4, 5			
		4	SS	WR		181	2, 6			
		5	SS	WR	∇	180, 179	3, 5			0 0 28 72
		6	SS	2		178, 177	3, 4			
		7	SS	2		176, 175	2, 3			
		8	SS	1		174	3, 3			
172.7		9	SS	2		173				
11.1	END OF BOREHOLE									

NOTE:

1. Water level in borehole measured at a depth of 5.2 m below ground surface (Elev. 178.6 m) on completion of drilling.

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED — 20 40 60 80 100
WATER CONTENT (%): 20 40 60
UNIT WEIGHT γ kN/m³
REMARKS & GRAIN SIZE DISTRIBUTION (%)

$+^3, \times^3$: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

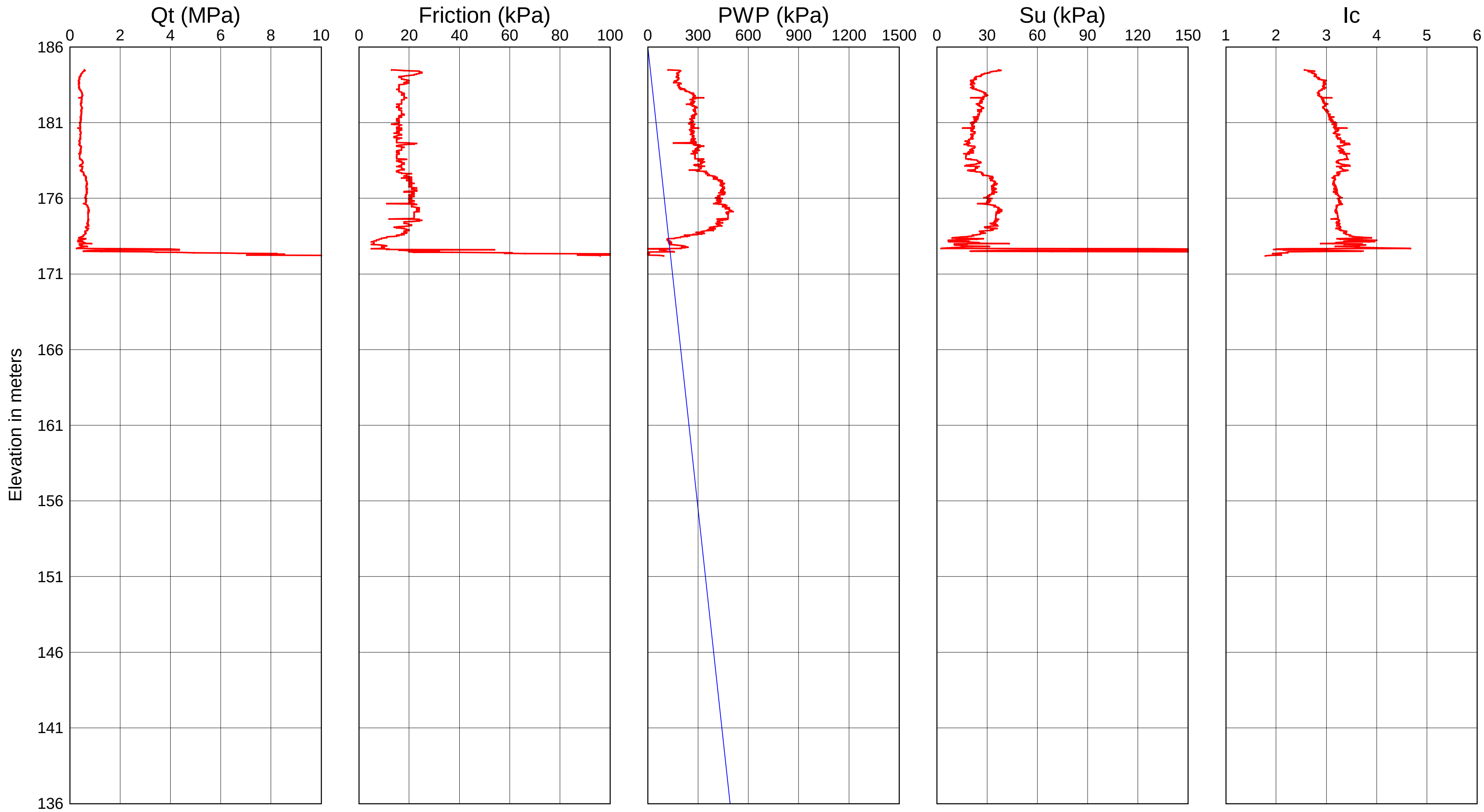
GTA-MTO 001 09-1111-0016.GPJ GAL-MISS.GDT 7/16/12 JFC

Cone Penetration Test - CPT 10-3

Test Date : 3/8/11
Location : N5145474.2 E299811.1

Operator : MWK

Ground Surf. Elev. : 186.00
Water Table Depth : 0.00



Qt normalized for
unequal end area effects

Su = (Qt - SigmaV) / Nk
Nk = 15
Gamma = 17 kN/m³

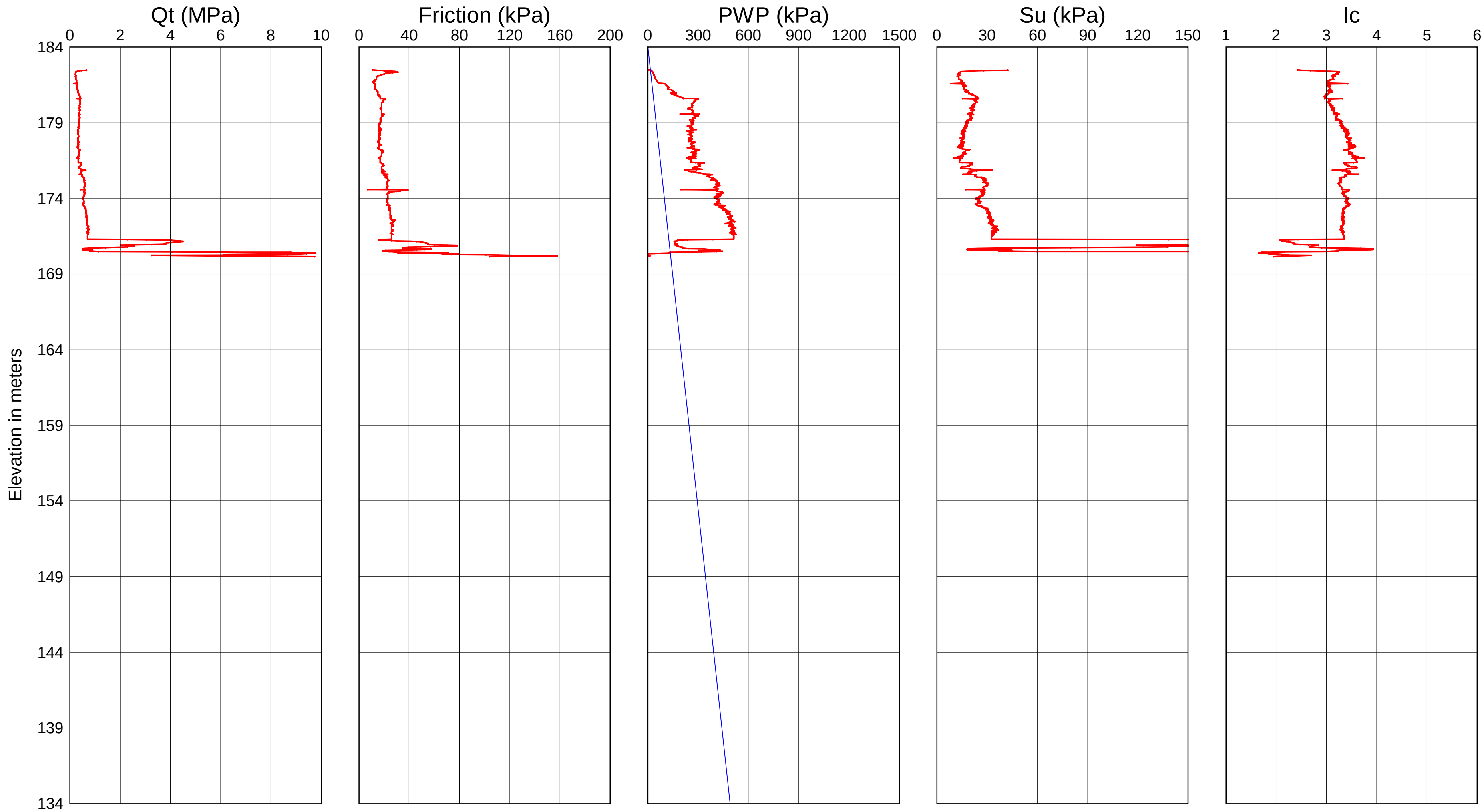
After Robertson and (Fear) Wride (1998)
Ic < 1.31 - Gravelly sands
1.31 <Ic < 2.05 - Clean to silty sand
2.05 <Ic < 2.60 - Silty sand to sandy silt
2.60 <Ic < 2.95 - Clayey silt to silty clay
2.95 <Ic < 3.60 - Clays

Cone Penetration Test - CPT 10-4

Test Date : 3/7/11
Location : N5145545.8 E299772.2

Operator : MWK

Ground Surf. Elev. : 183.80
Water Table Depth : 0.00



Qt normalized for
unequal end area effects

Su = (Qt - SigmaV) / Nk
Nk = 15
Gamma = 17 kN/m³

After Robertson and (Fear) Wride (1998)
Ic < 1.31 - Gravelly sands
1.31 <Ic < 2.05 - Clean to silty sand
2.05 <Ic < 2.60 - Silty sand to sandy silt
2.60 <Ic < 2.95 - Clayey silt to silty clay
2.95 <Ic < 3.60 - Clays

PROJECT NO. 09-1111-0016 DATE: 6/7/2011 DRAWN BY: MWK

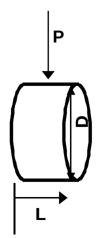
TABLE B.FW.1

POINT LOAD TESTS ON ROCK SAMPLES

Borehole Number	Run Number	Sample Depth (m)	Sample Elevation (m)	Bedrock Description	Test Type	**Is (50mm)** (MPa)	**Approx.[1] UCS** (MPa)
10-3	1	26.4	159.6	Granitic Gneiss	Axial	4.293	92
10-3	2	27.1	158.9	Granitic Gneiss	Axial	2.143	46
10-3	2	28.1	157.9	Granitic Gneiss	Axial	4.168	90
10-3	3	28.8	157.2	Granitic Gneiss	Axial	4.712	101

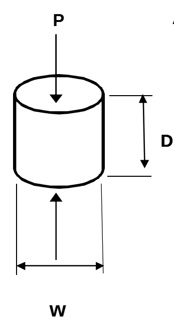
[1] Is_{50} x K (the value of K was estimated based on the average Is(50) point load test result and one UCS test), from ISRM. The estimated K value = 21.5
("Suggested Methods for Determining Point Load Strength", International Society for Rock Mechanics Commission
on Testing Methods, Int. J. Rock. Mech. Min. Sci. and Geomechanical Abstr., Vol 22, No. 2 1985, pp. 51-60.

DIAMETRAL SPECIMEN SHAPE REQUIREMENTS
note: Diametral tests are perpendicular to core axis
(planes of weakness)



L>0.5 D

AXIAL SPECIMEN SHAPE REQUIREMENTS
note: Axial tests are parallel to core axis
(planes of weakness)



0.3W<D<W

TABLE B.FW.2 - UNCONFINED COMPRESSION TEST (UC)

ASTM D 7012-07

SAMPLE IDENTIFICATION

PROJECT NUMBER	09-1111-0016	SAMPLE NUMBER	UCS
BOREHOLE NUMBER	10-3	SAMPLE DEPTH, m	27.6-27.7

TEST CONDITIONS

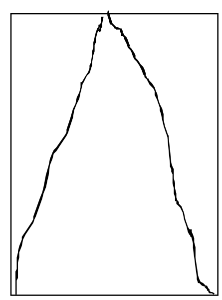
MACHINE SPEED, mm/min	-	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST,min	>2 <15	L/D	2.23

SPECIMEN INFORMATION

SAMPLE HEIGHT, cm	10.50	WATER CONTENT, (specimen) %	1.20
SAMPLE DIAMETER, cm	4.70	UNIT WEIGHT, kN/m^3	24.55
SAMPLE AREA, cm^2	17.35	DRY UNIT WT., kN/m^3	24.26
SAMPLE VOLUME, cm^3	182.17	SPECIFIC GRAVITY, assumed	2.70
WET WEIGHT, g	456.15	VOID RATIO	0.09
DRY WEIGHT, g	450.74		

VISUAL INSPECTION

FAILURE SKETCH



TEST RESULTS

STRAIN AT FAILURE, %	-	COMPRESSIVE STRESS, MPa	82.0

REMARKS:

DATE: 2/18/2011

Golder Associates

TABLE B.FW.3
Field Estimation of Rock Hardness
(Representation of Intact Rock Strength)

Grade	Description	Field Identification	Approx. Range of UCS (MPa)
R0	Extremely weak rock	Indented by thumbnail.	0.25 – 1
R1	Very weak rock	Material can be shaped with a pocket knife or can be peeled by a pocket knife. Crumbles under firm blows of pick (or point) of geological hammer.	1.0 – 5.0
R2	Weak rock	Knife cuts material but too hard to shape into triaxial specimens or material can be peeled by a pocket knife with difficulty. Shallow indentations (< 5 mm) made by firm blow with pick (or point) of geological hammer.	5.0 – 25
R3	Medium strong rock	Cannot be scraped or peeled with a pocket knife. Hand held specimens can be fractured with *single* firm blow of geological hammer.	25 – 50
R4	Strong rock	Hand held a specimen requires *more than one* blow of geological hammer to fracture it.	50 – 100
R5	Very strong rock	Specimen requires many blows of geological hammer to break intact rock specimens (or to fracture it).	100 – 250
R6	Extremely strong rock	Specimen can only be chipped under repeated hammer blows, rings when hit.	> 250

NOTES:

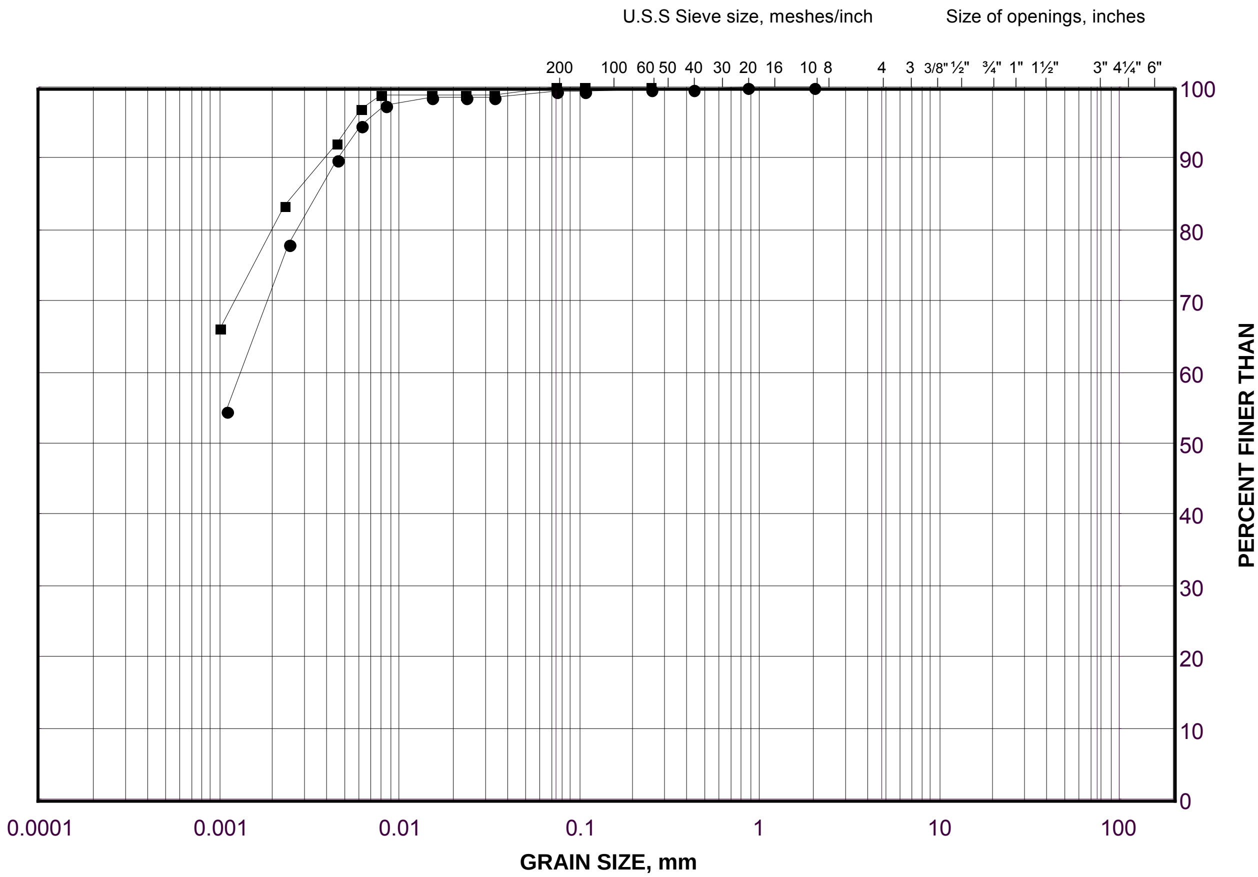
1. Hand held specimens should have height ≅ 2 times the diameter.
2. Materials having a uniaxial compressive strength (UCS) of less than about 0.5 MPa and cohesionless materials should be classified using soil classification systems.
3. Rocks with a uniaxial compressive strength below 25 MPa (i.e., below R2) are likely to yield highly ambiguous results under point load testing.

REFERENCES:

1. Brown (1981). “Suggested Methods for Rock Characterization Testing and Monitoring”, International Society for Rock Mechanics.

2. Hoek, E., Kaiser, P.K., Bawden, W.F. (1995). “Support of Underground Excavations in Hard Rock”, Balkema, Rotterdam.

GRAIN SIZE DISTRIBUTION Silty Clay to Clay	FIGURE B.FW.1



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	10-4	5	179.0
■	10-3	5	181.5

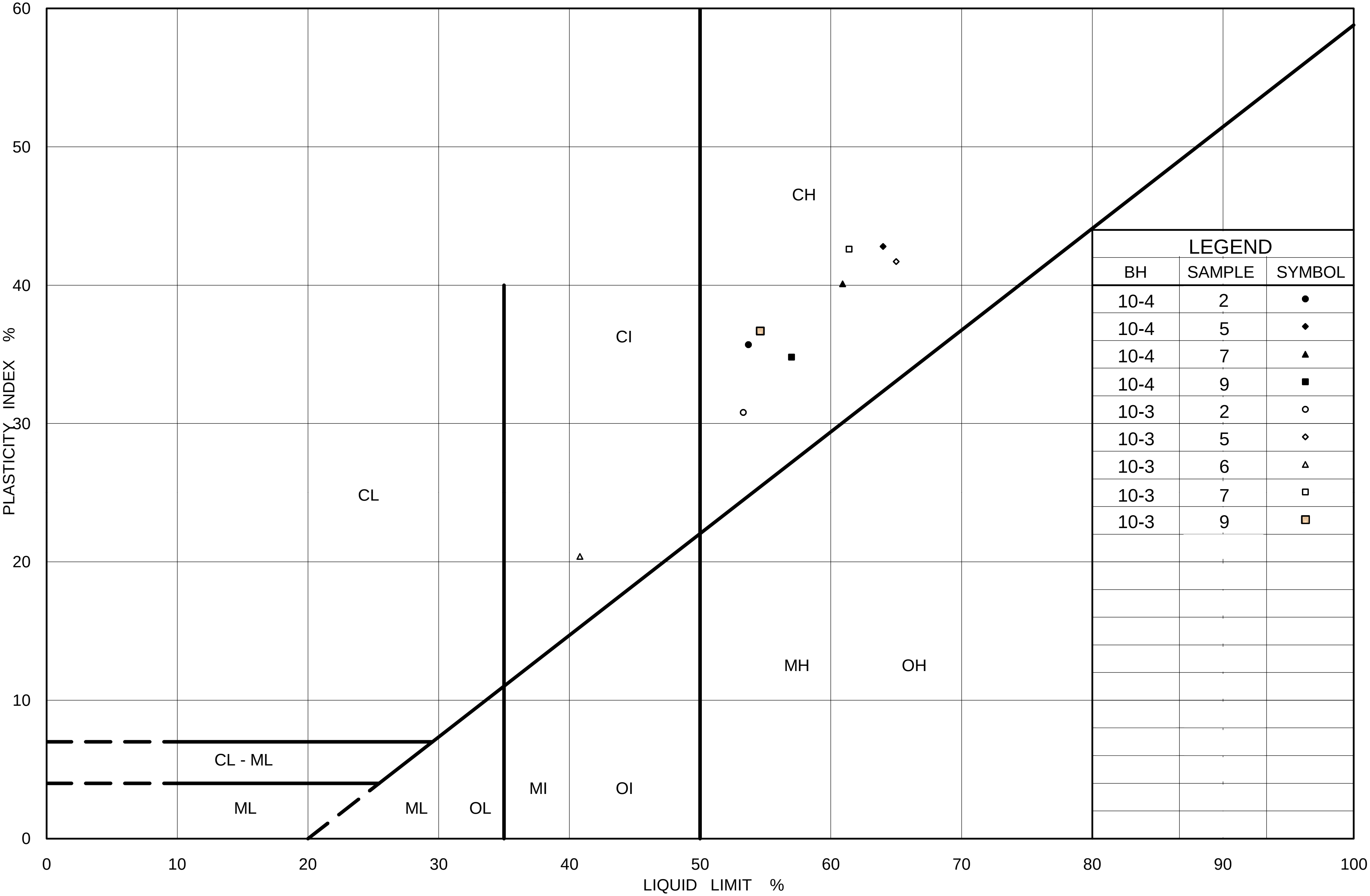
Project Number: 09-1111-0016

Checked By: ______________________

Golder Associates

Date: 01-Jun-11

Oct 75, FF-S-21



LEGEND

BH	SAMPLE	SYMBOL
10-4	2	●
10-4	5	◆
10-4	7	▲
10-4	9	■
10-3	2	○
10-3	5	◇
10-3	6	△
10-3	7	□
10-3	9	□

Ministry of Transportation

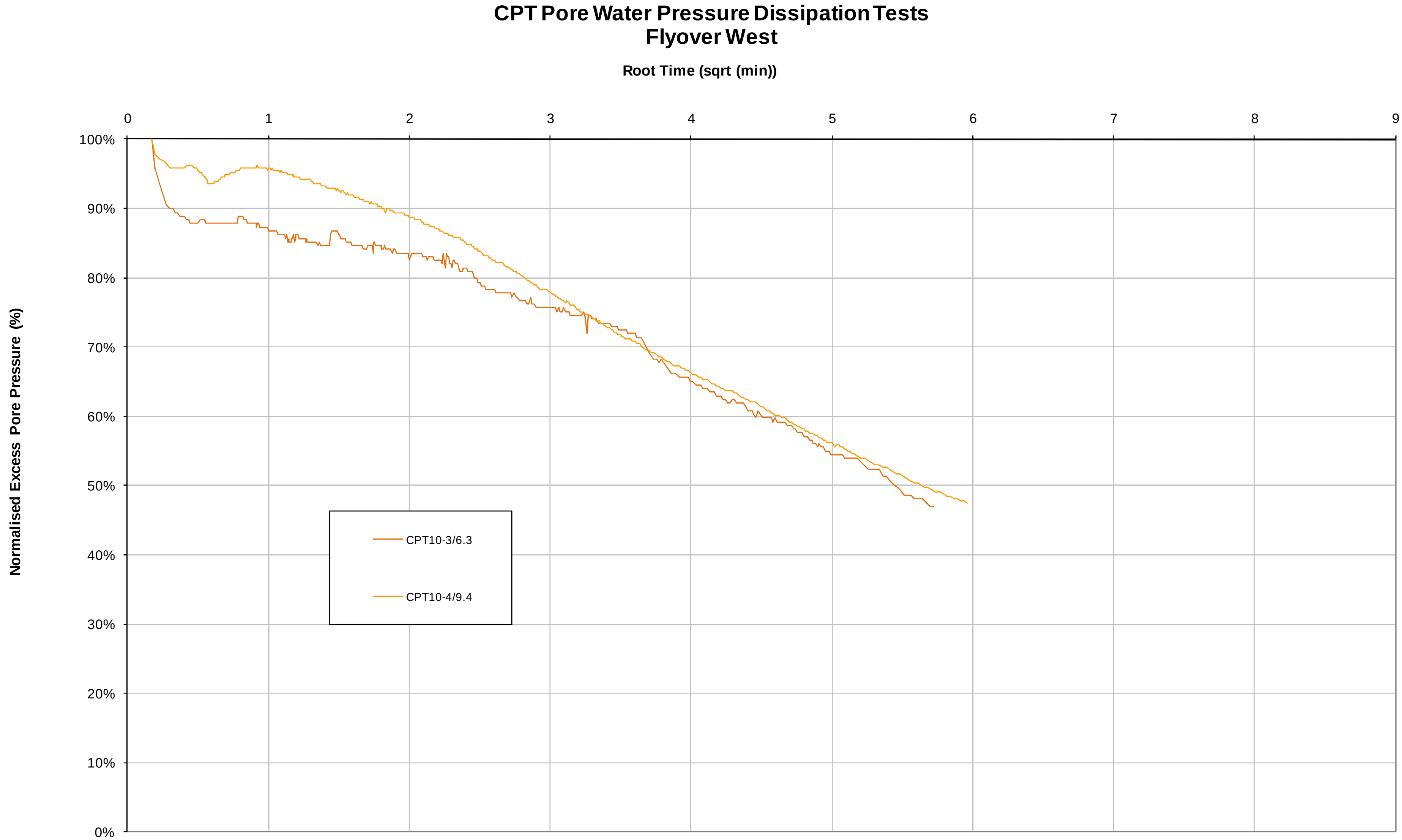
Ontario

PLASTICITY CHART
Clay to Silty Clay

Figure No. B.FW.2

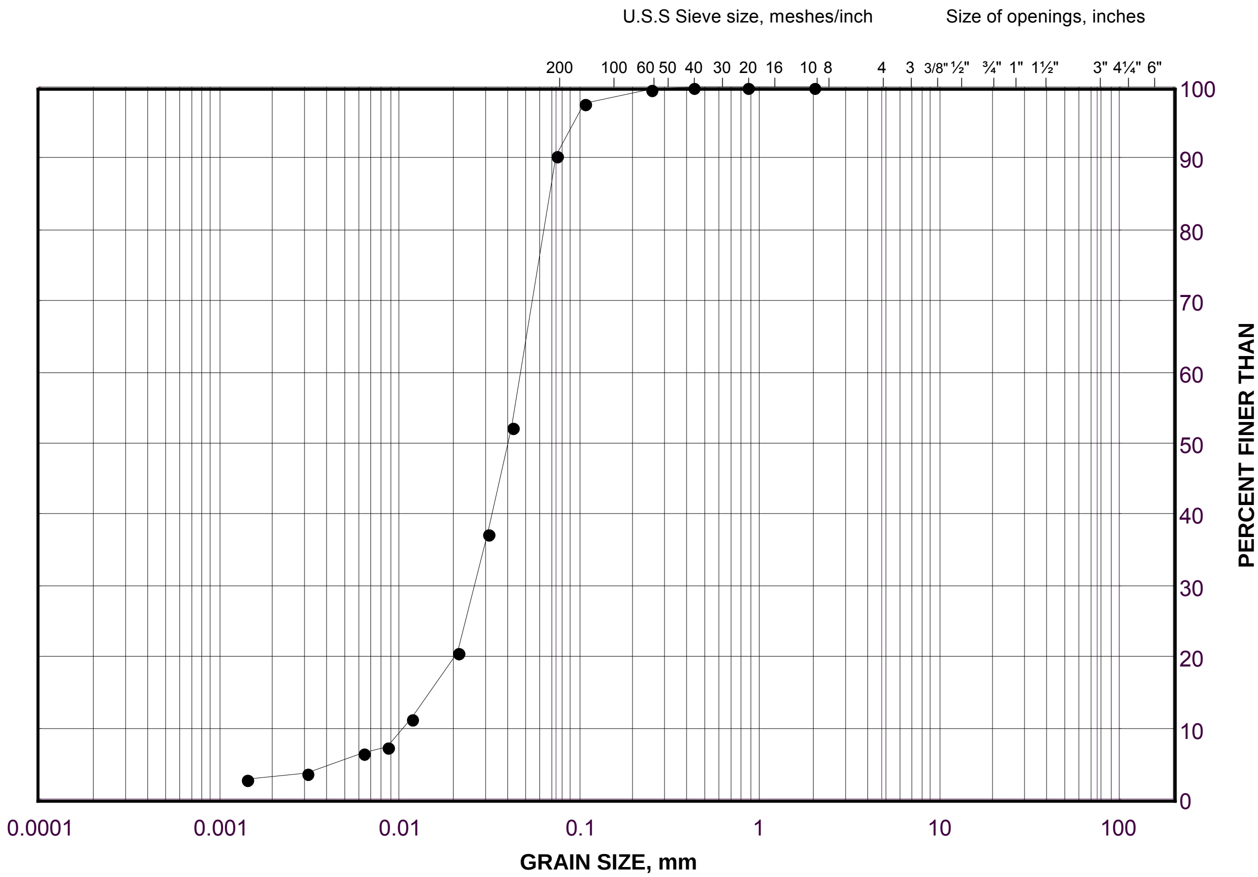
Project No. 09-1111-0016

Checked By:


CPT Pore Water Pressure Dissipation Tests
Flyover West
Root Time (sqrt (min))
0
1
2
3
4
5
6
7
8
9
100%
90%
80%
70%
60%
50%
40%
30%
20%
10%
0%
Normalised Excess Pore Pressure (%)
CPT10-3/6.3
CPT10-4/9.4

GRAIN SIZE DISTRIBUTION
Silt

FIGURE B.FW.4



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	10-3	10B	173.9

Project Number: 09-1111-0016

Checked By: ____________

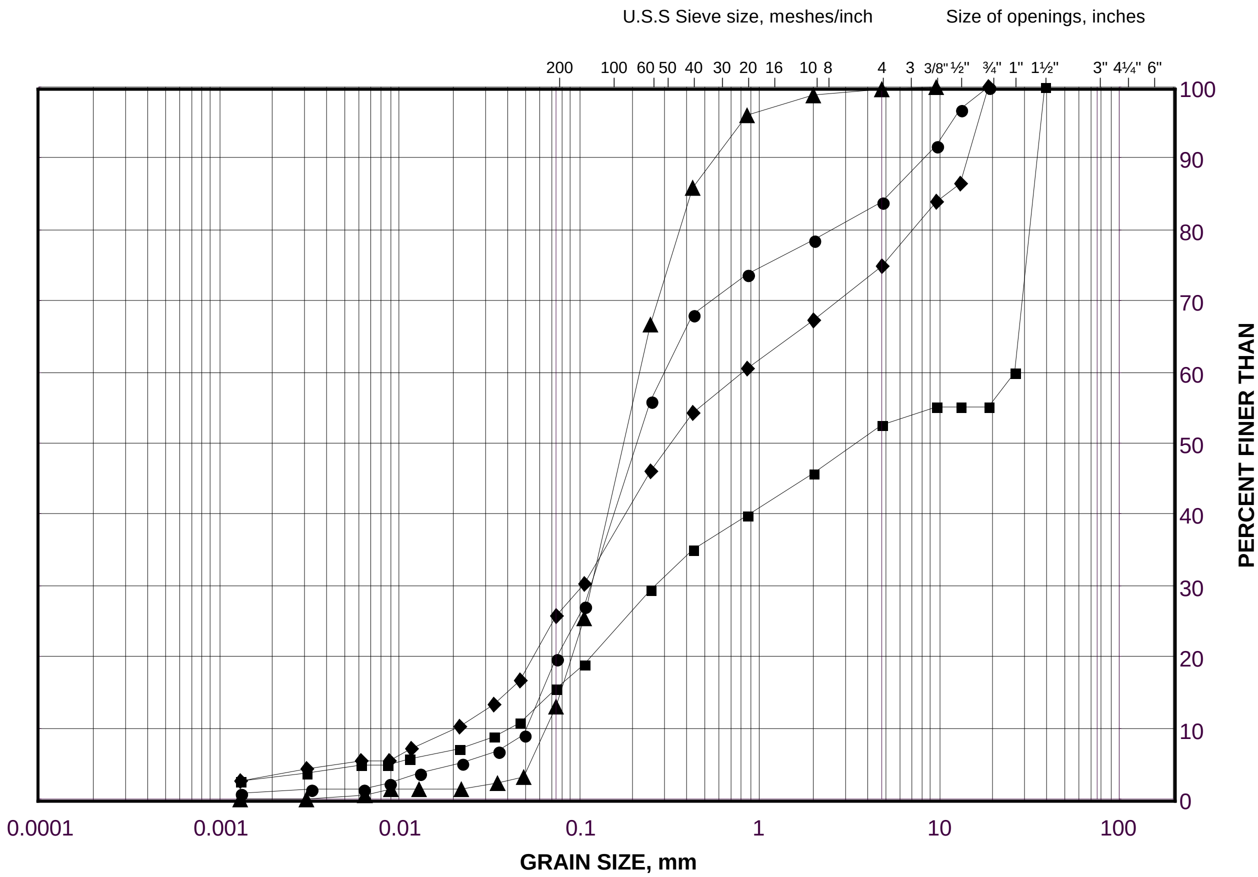
Golder Associates

Date: 01-Jun-11

GRAIN SIZE DISTRIBUTION

Sand to Sand and Gravel to Gravelly Silty Sand

FIGURE B.FW.5



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	10-3	11	172.4
■	10-3	13	169.3
◆	10-3	15	166.3
▲	10-3	18	159.8

Project Number: 09-1111-0016

Checked By: ______________________

Golder Associates

Date: 01-Sep-11

TABLE B1 – SUMMARY OF FOUNDATION ENGINEERING PARAMETERS - FLYOVER WEST

Stratigraphic Unit	***Average Top Elevation (m)****	***Thickness** (m)***	***γ' (kN/m^3)***	***ϕ' (°)***	***c' (kPa)***	***s_u (kPa)***	***σ_p' (kPa)***	***e_o***	***C_c***	***C_r***	***E' (MPa)***	***$C_{\alpha(\varepsilon)}$ (%)***		***c_h (cm^2/s)***
												N/C	***O/C***	
Granular Fill (sub-excavate and replace near surface topsoil/soft clay soil)	184.9	2.0	21	32	0	--	--	--	--	--	15	--	--	--
Silty Clay to Clay (soft)	182.9	3.6	17	21	0	20	91	1.7	0.9	0.09	--	0.5	0.05	3.5×10^{-3}
Silty Clay to Clay (soft to firm)	179.3	7.4	17	21	0	20 - 45	91 - 205	1.7	0.9	0.09	--	0.5	0.05	3.5×10^{-3}
Sand / Silt / Gravel	171.9	11.9	20	30	0	--	--	--	--	--	30	--	--	--

*Average Elevation of top of stratigraphic unit at Borehole and CPT locations (refer to Drawing B1)

**Average Thickness of stratigraphic unit at Borehole and CPT locations (refer to Drawing B1)

Prepared By: MWK

Reviewed By: JPD/JMAC

TABLE B2 – EVALUATION OF BRIDGE STRUCTURE FOUNDATION ALTERNATIVES - FLYOVER WEST

Foundation Option	***Rank***	***Advantages***	***Disadvantages***	***Relative Costs***	***Risks / Consequences***
Spread Footings on Overburden	Not feasible	• Relative ease of construction.	• Groundwater control required for excavation and during footing construction. • Large post-construction settlements. • Low geotechnical resistance at ULS and SLS of native soils and hence very large footings required.	• Lower relative cost than piled foundations.	• Footing size required to accommodate very low geotechnical resistances is not practical. • Very large post-construction settlements could not be tolerated by bridge structure.
Piles driven to bedrock (25 m to 30 m long piles)	1	• Limited sub-excavation required for pile cap construction. • Negligible post-construction settlement. • Higher axial resistance than for friction piles. • Fewer piles required than for friction piles option	• Heavier pile sections will be required to penetrate cobbles and boulders and seat piles on bedrock.	• Higher cost associated with heavier pile sections and somewhat greater pile lengths. • Higher cost associated with provisions for re-driving piles for piles driven out of alignment.	• Damaged piles and piles driven out of alignment may require removal and replacement with new piles. • The abutment/pier design should be flexible enough to allow for installation of extra piles in the footing area, if deemed necessary during construction.
Friction Piles (20 m long piles)	2	• Limited sub-excavation required for pile cap construction. • Minor post-construction settlement. • Shorter piles required than for piles driven to bedrock option.	• Lower pile capacity than piles driven to refusal.	• Lower cost associated with shorter pile lengths and lighter pile section. • Higher cost associated with additional piles due to lower axial capacity. • Additional cost for pile load tests.	• Lower pile capacity will require more piles at each foundation unit. • May require pile load tests to verify pile capacity.

Prepared By: MWK

Reviewed By: JPD/JMAC

TABLE B3 – EVALUATION OF APPROACH EMBANKMENT FOUNDATION STABILITY/SETTLEMENT MITIGATION ALTERNATIVES - FLYOVER WEST

Stability / Settlement Mitigation Option	*Rank*	*Advantages*	*Disadvantages*	*Relative Costs*	*Risks / Consequences*
Staged construction (with wick drains,10 m wide by 2 m high toe berms and 2 m subexcavate and replace) (6 stages) (approximately 3 years of construction delays for staging)	1	• Smaller embankment footprint and less land acquisition requirements as compared with toe berms only option.	• Somewhat larger volume of excess excavation spoil due to berms. • Somewhat greater quantities of fill required for replacement in subexcavated area due to berms. • Delay of approximately 3 years during staged construction and preloading. • Large post-construction settlement. • Large downdrag loads reduce pile capacity. • EPS required to maintain front slope stability and top-up to mitigate long-term settlements.	• $891,000 to $1,188,000 (wick drains at 1.5 m spacing) + • $198,000 (berms) + • $445,000 (subexcavate / replace) + • $2,835,200 cost of EPS to mitigate long-term settlements.	• Staged construction sequence required with potential for additional delays during construction depending on monitoring. • Post-construction settlements may require long-term maintenance. • Nominal size Toe berms are trequired for stability, increasing footprint. • Some secondary consolidation (creep) will occur. • Potential need to acquire some additional lands for right-of-way.

TABLE B3 – EVALUATION OF APPROACH EMBANKMENT FOUNDATION STABILITY/SETTLEMENT MITIGATION ALTERNATIVES - FLYOVER WEST

Stability / Settlement Mitigation Option	*Rank*	*Advantages*	*Disadvantages*	*Relative Costs*	*Risks / Consequences*
Toe berms up to 25 m wide (with 2 m subexcavate and replace) (with up to 2 year preload)	4	• Standard construction operation. • No construction delays associated with staging.	• Generation of larger volume of excess excavation spoil due to large toe berm footprint. • Greater quantities of fill required for large berms and for subexcavate and replace area. • Large embankment footprint. • Large post-construction settlement. • Large downdrag loads reduce pile capacity. • EPS required to maintain front slope stability and top-up to mitigate long-term settlements. • Preload period required to mitigate settlements	• $1,064,250 (subexcavate/replace and toe berms)+ • land acquisition costs + • $2,835,200 cost of EPS to mitigate long-term settlements and front slope stability.	• Risk of instability (low). • Secondary consolidation (creep) will occur. • Large post-construction settlements will require long-term maintenance. • Likely need to acquire additional right-of-way due to large berm size.

TABLE B3 – EVALUATION OF APPROACH EMBANKMENT FOUNDATION STABILITY/SETTLEMENT MITIGATION ALTERNATIVES - FLYOVER WEST

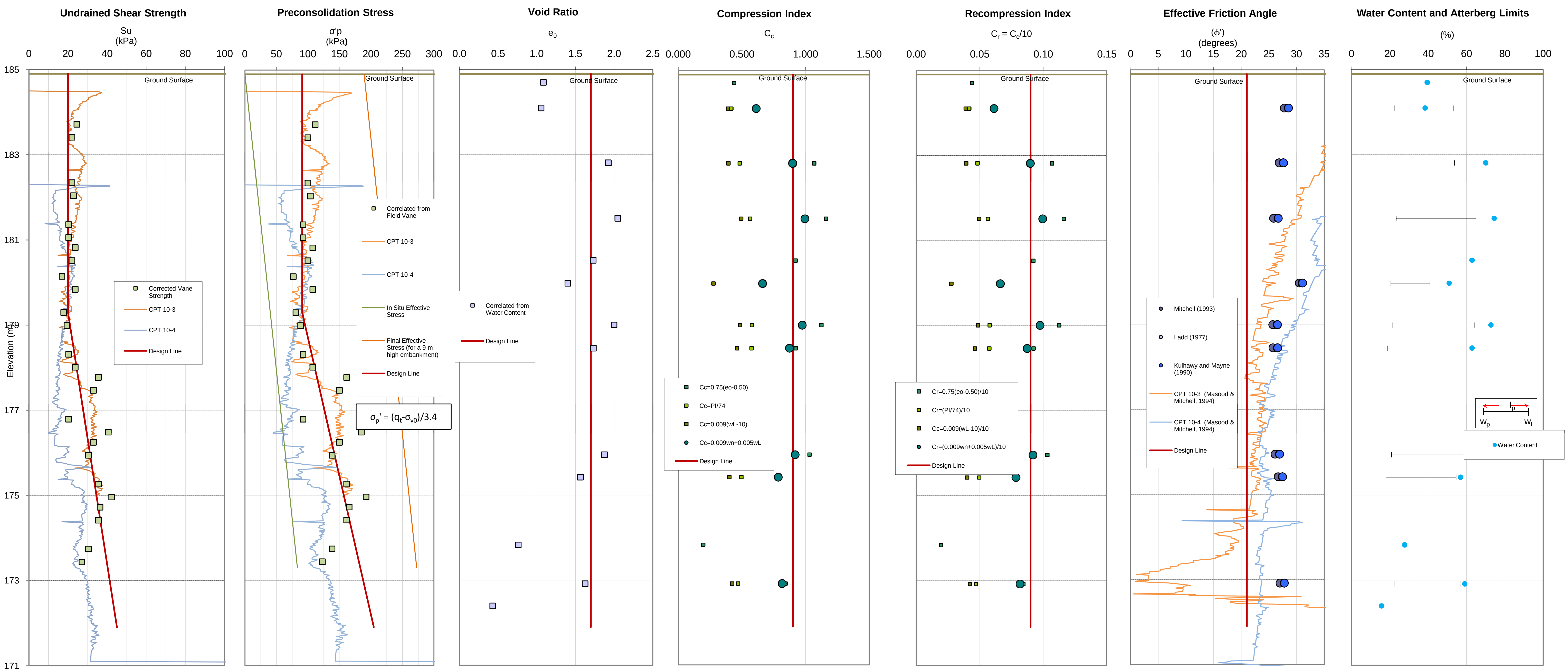
Stability / Settlement Mitigation Option	*Rank*	*Advantages*	*Disadvantages*	*Relative Costs*	*Risks / Consequences*
Partial Lightweight Fill (EPS) (with 2 m subexcavate and replace) (with up to 2 year preload)	3	• Standard construction operation. • No construction delay • Reduced secondary (creep) consolidation settlement. • Generation of smaller volume of excess excavation spoil since no toe berms. • Smaller quantities of fill required for subexcavate and replace since no toe berms. • Smaller embankment footprint.	• Higher cost for specialized materials • Additional effort required for sub-excavation and replacement. • EPS required to maintain front slope stability and to-up to mitigate long term settlements. • Some post construction settlements. • Preload period required to mitigate settlements	• $445,500 (subexcavate/replace)+ • $6,120,400 cost of EPS to mitigate long-term settlements and front slope stability.	• Risk of instability (low). • Secondary consolidation (creep) will occur. • Post-construction settlement may require long-term maintenance. • Potential for smaller property acquisition needs.
Full Lightweight Fill (EPS) (with 2 m subexcavate and replace)	2	• Standard construction operation. • No construction delays associated with staging or preloading. • Minimized post-construction settlement. • Smallest embankment footprint.	• Higher cost for specialized materials • Restricted use of EPS within the embankment cross-section to above water table.	• $445,500 (subexcavate/replace)+ • $13,860,000 cost of EPS to mitigate long-term settlements and front slope stability.	• Low risk of instability. • Low risk of long-term settlement of foundation soils.

Prepared By: MWK

Reviewed By: JPD/JMAC

Summary of Engineering Parameters for Cohesive Deposits
Flyover West

Figure B1



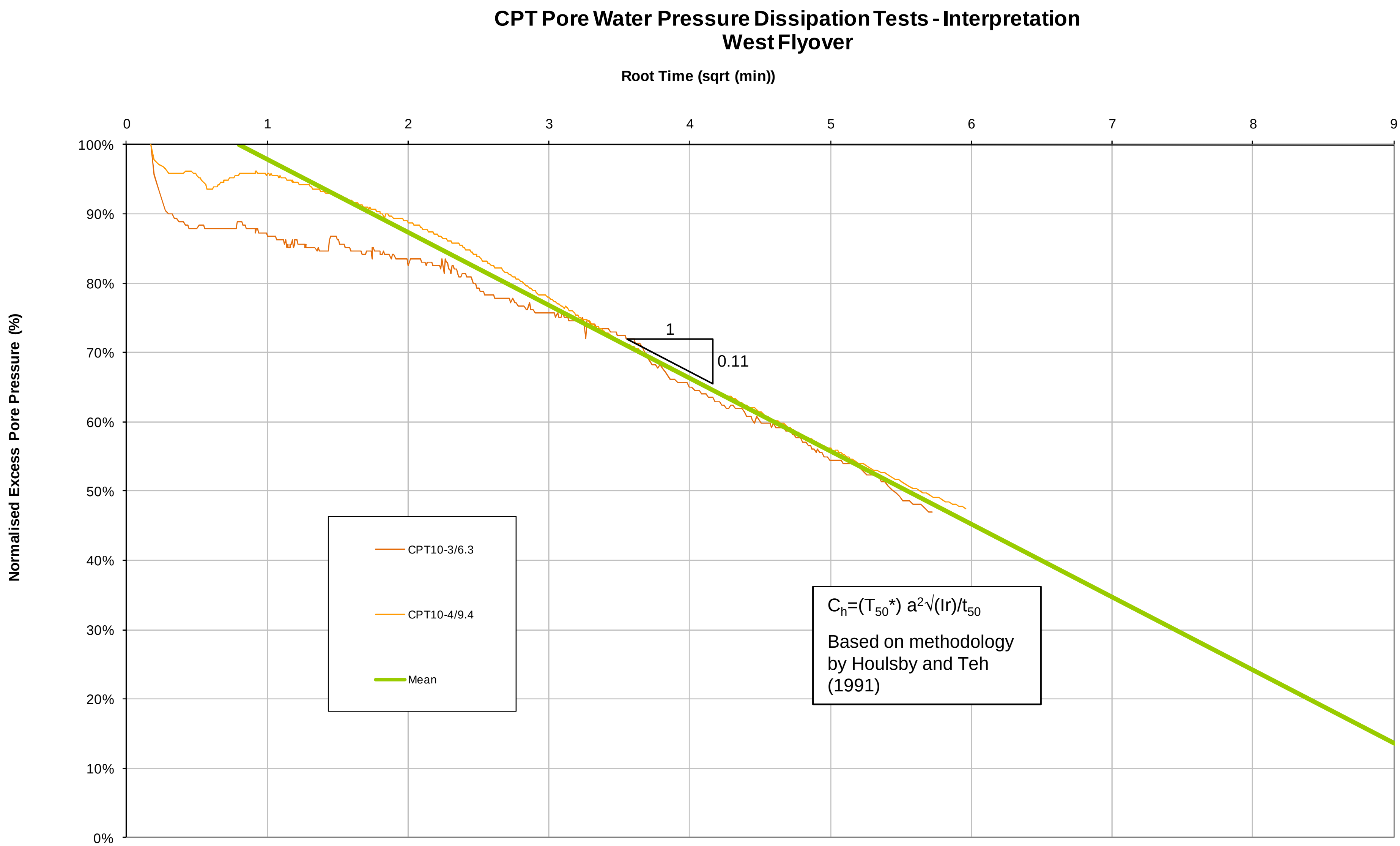
NOTES:

Average ground surface at the borehole locations is about Elevation 184.9 m
Average elevation of bottom of cohesive deposit at the borehole locations is about 171.9 m

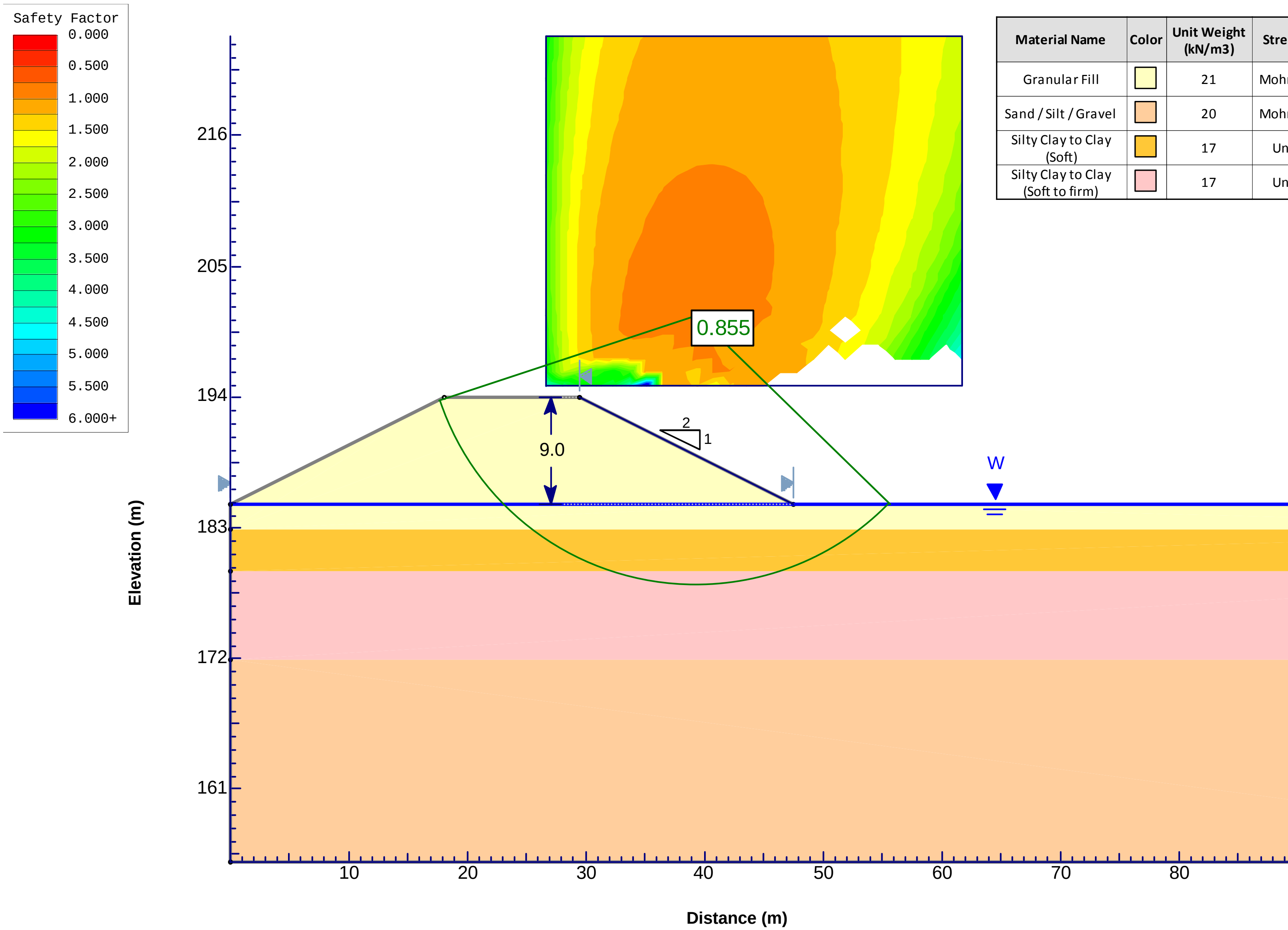
Date:	Jun-11	**DB:**	MWK
Project No:	09-1111-0016	**CHK:**	JPD

Golder Associates

#N/A

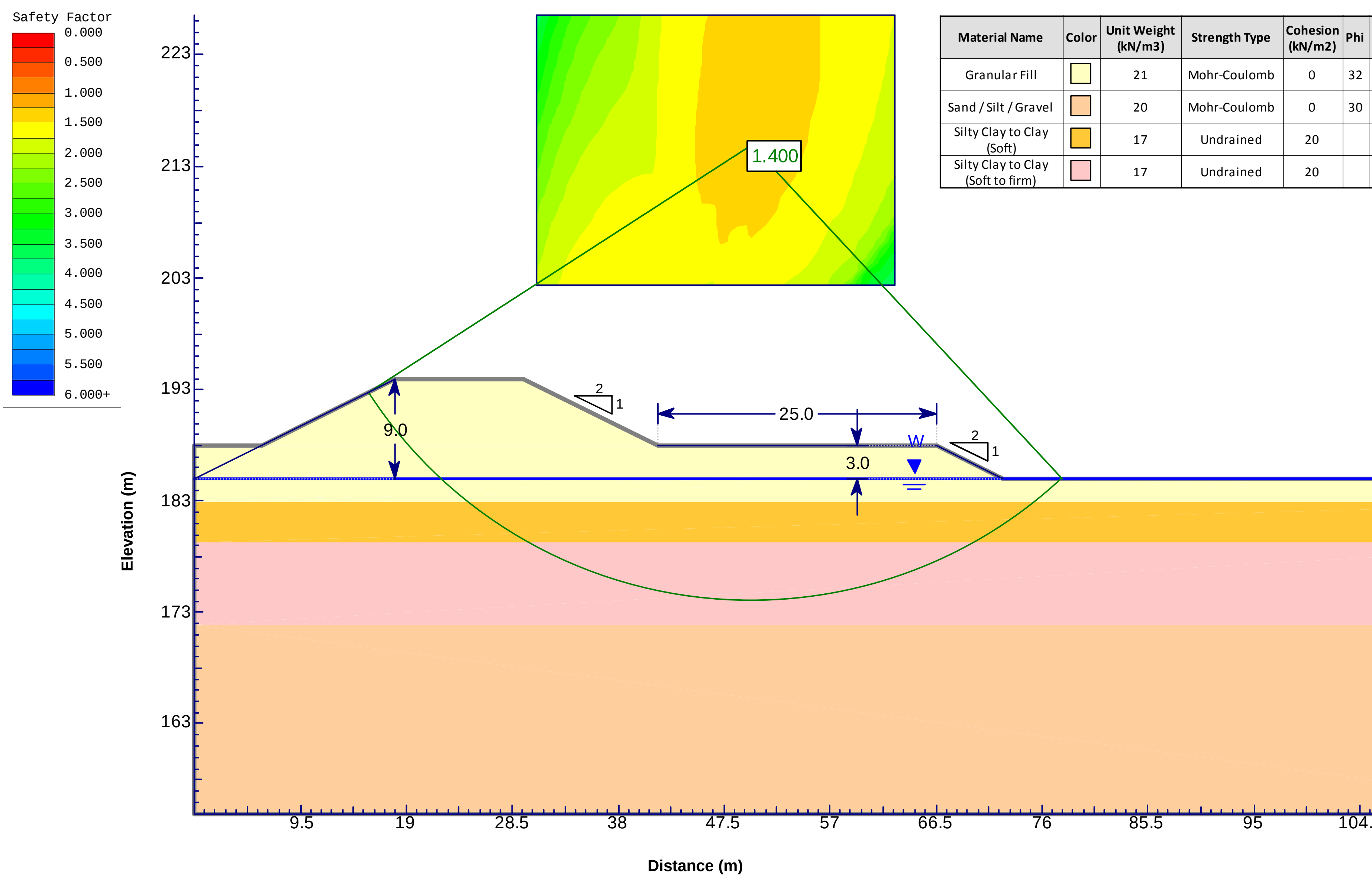

CPT Pore Water Pressure Dissipation Tests - Interpretation
West Flyover
Root Time (sqrt (min))
0
1
2
3
4
5
6
7
8
9
100%
90%
80%
70%
60%
50%
40%
30%
20%
10%
0%
Normalised Excess Pore Pressure (%)
1
0.11
CPT10-3/6.3
CPT10-4/9.4
Mean
Ch=(T50*) a2√(Ir)/t50
Based on methodology by Houlsby and Teh (1991)

Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kN/m2)	Phi	Cohesion Type
Granular Fill		21	Mohr-Coulomb	0	32	
Sand / Silt / Gravel		20	Mohr-Coulomb	0	30	
Silty Clay to Clay (Soft)		17	Undrained	20		Constant
Silty Clay to Clay (Soft to firm)		17	Undrained	20		FDepth



Slope Stability – Total Stress Analysis – Stabilizing Berms

Figure B3-2



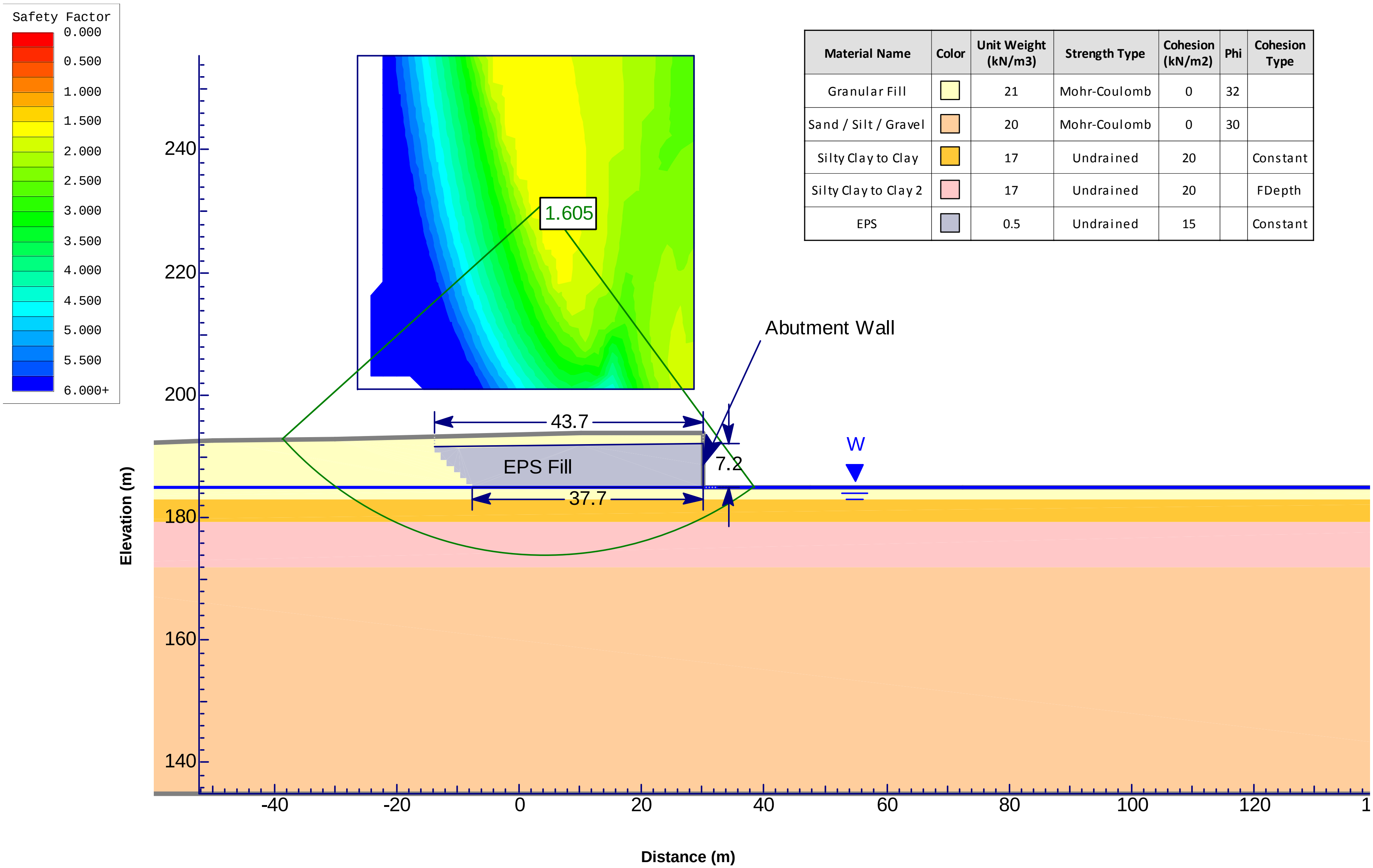
Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kN/m2)	Phi	Cohesion Type
Granular Fill		21	Mohr-Coulomb	0	32	
Sand / Silt / Gravel		20	Mohr-Coulomb	0	30	
Silty Clay to Clay (Soft)		17	Undrained	20		Constant
Silty Clay to Clay (Soft to firm)		17	Undrained	20		FDepth

Date: July 2011
Project No: 09-1111-0016

Analysis By: MWK **Reviewed By:** JPD

Slope Stability – Total Stress Analysis – Front Slope Stability (with EPS)

Figure B3-3



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kN/m2)	Phi	Cohesion Type
Granular Fill		21	Mohr-Coulomb	0	32	
Sand / Silt / Gravel		20	Mohr-Coulomb	0	30	
Silty Clay to Clay		17	Undrained	20		Constant
Silty Clay to Clay 2		17	Undrained	20		FDepth
EPS		0.5	Undrained	15		Constant

Date: July 2011
Project No: 09-1111-0016

Analysis By: MWK **Reviewed By:** JPD

Golder Associates

APPENDIX C

Realigned Bar River Road Flyover (Flyover West Alternative 5)

MINISTRY OF TRANSPORTATION, ONTARIO

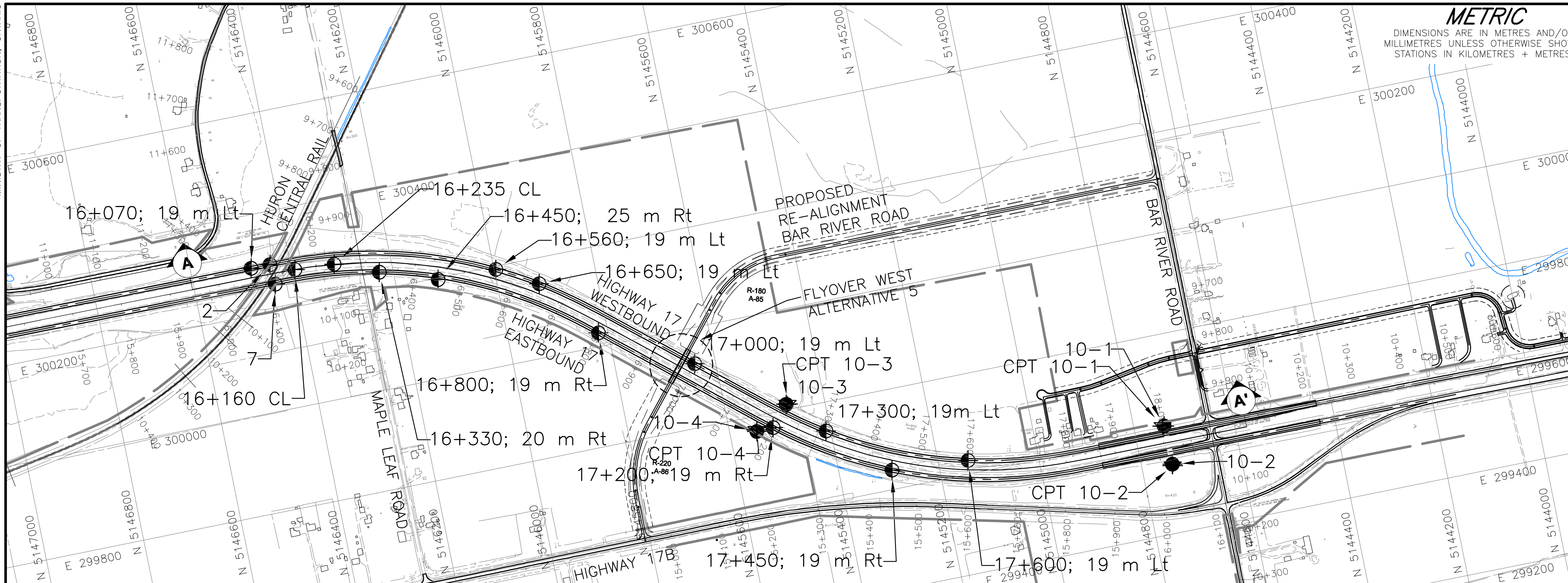
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.
WP No. 5022-07-00

SHEET

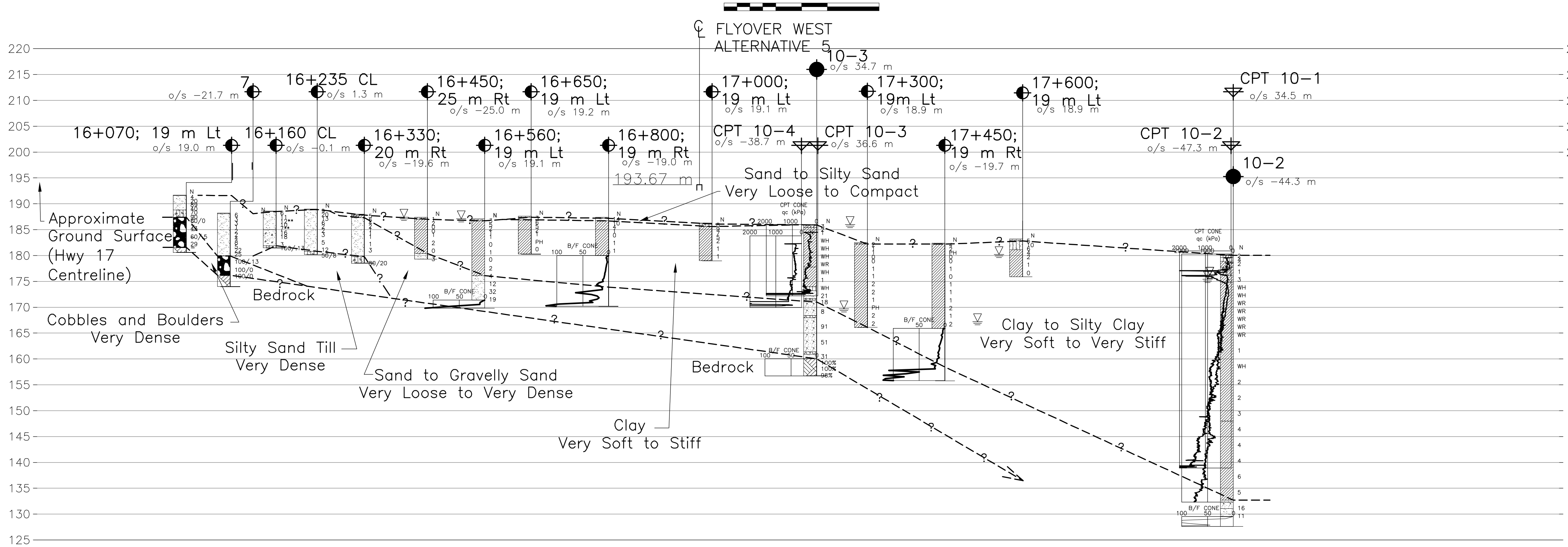
HIGHWAY 17 FLYOVER WEST ALTERNATIVE 5 BOREHOLE LOCATIONS AND SOIL STRATA

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



PLAN

SCALE
100 0 100 200 m

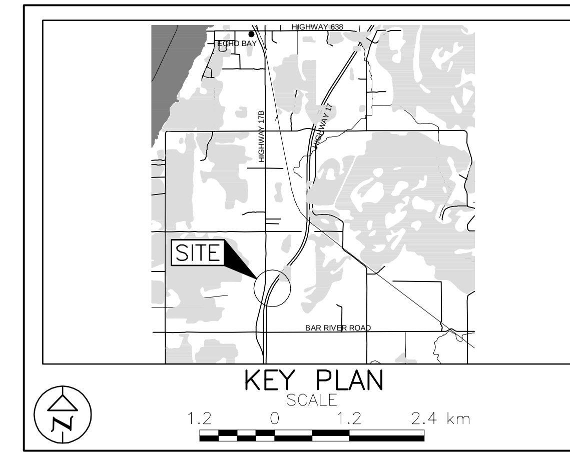


CENTRELINE PROFILE

A-A'

HORIZONTAL SCALE
100 0 100 200 m
10 0 10 20 m
VERTICAL SCALE

NOTE:
Ground Surface was taken from Highway 17 (New) Construction Drawings Contract No. 2004-5120.



KEY PLAN
SCALE
1.2 0 1.2 2.4 km

LEGEND

Symbol	Description
	Borehole – Current Investigation
	CPT – Current Investigation
	Borehole – Previous Investigation (MTO, 2003)
N	Standard Penetration Test Value
16	Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
--?--	Interpreted Subsurface Stratigraphy based on existing information, not confirmed by borehole drilling
100%	Rock Quality Designation (RQD)
	WL upon completion of drilling
7	CPT tip resistance qc (kPa)

BOREHOLE CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
10-1	180.9	5144739.6	299616.8
10-2	180.3	5144738.5	299538.2
10-3	186.0	5145477.2	299811.1
10-4	183.8	5145545.8	299769.2

CPT CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
CPT 10-1	180.9	5144742.6	299617.0
CPT 10-2	180.3	5144738.5	299535.2
CPT 10-3	186.0	5145474.2	299811.1
CPT 10-4	183.8	5145545.8	299772.2

NOTES

This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contracts Documents.

The boundaries between soil strata have been established only at borehole locations. Between boreholes the boundaries are assumed from geological evidence.

The complete Foundation Investigation and Design Report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with Section GC 2.01 of OPS General Conditions.

REFERENCE

Base plans provided in digital format by Genvair, drawing file 09-079 – Base Plan.dwg and the General Arrangement drawing file 09-079 Recommended Plan Feb 2012.dwg received May 15, 2012.

NO.	DATE	BY	REVISION

Geocres No. 41K-90			
HWY. 17		PROJECT NO. 09-1111-0016	DIST.
SUBM'D. MWK	CHKD. JPD	DATE: 7/12/2012	SITE:
DRAWN: JFC	CHKD. MWK	APPD. JMAC	DWG. C1

RECORD OF BOREHOLE No 2

METRIC

GWP	354-94-00	LOCATION	Echo River to Bar River Road, Sta. 16+109, 19m Lt. - Coords: N 5 146 440.4; E: 300 293.6
DIST	62 HWY 17 (New)	BOREHOLE TYPE	Solid Stem Augers, D.C.P.T., Casing & Washboring & NQ Rock Coring
DATUM	Geodetic	DATE	11/15/2002

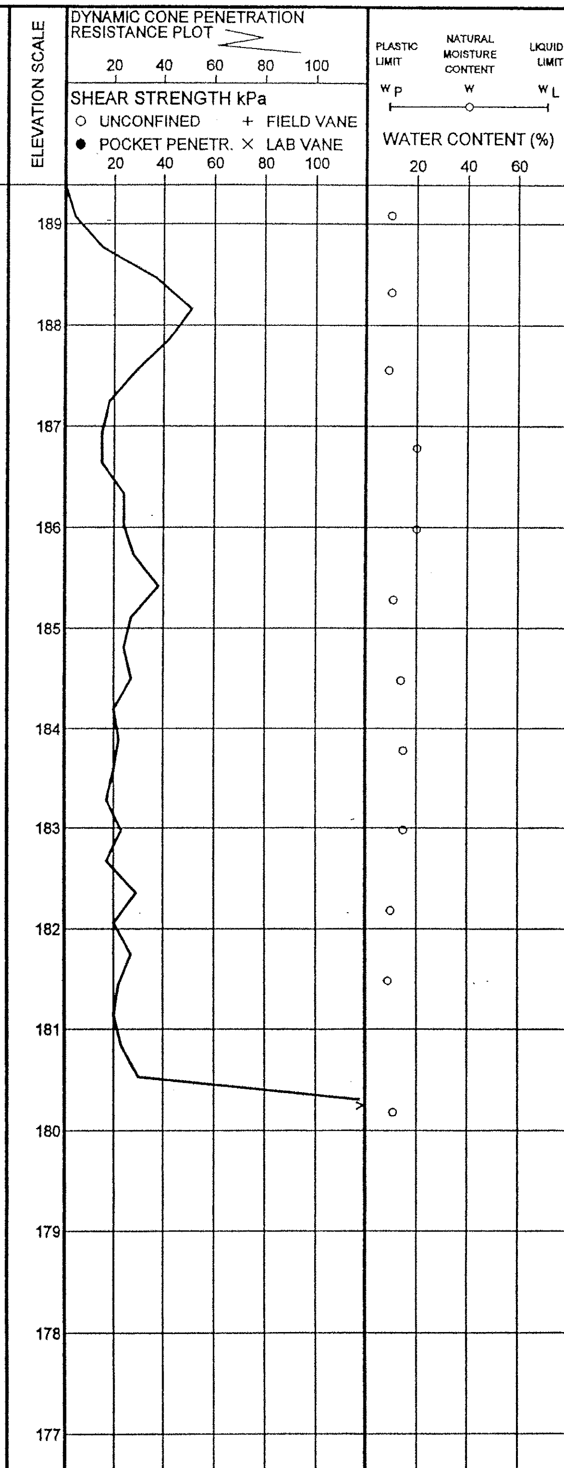
ORIGINATED BY G.I.
COMPILED BY G.T.
CHECKED BY R.A.

Soil Profile

ELEV DEPTH	DESCRIPTION
189.4	Ground Surface
0.0	0.15 m Topsoil; SAND, some silt and gravel, occasional cobbles, brown; trace organics; moist / wet; very loose to compact; very dense; loose, wet; compact to dense; loose
180.6 / 8.8	COBBLES AND BOULDERS and fractured rock, very dense (inferred)
179.6 / 9.8	SANDSTONE BEDROCK, fractured, reddish brown
176.6 / 12.8	End of borehole

Samples

NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	4	*	
2	SS	17		
3	SS	58		
4	SS	7		1 94 (5)
5	SS	28		
6	SS	44		
7	SS	7		
8	SS	8		18 72 (10)
9	SS	10	**	** Commence casing and washboring.
10	SS	10		
11	SS	8		
12	SS	60/10		
13	NQ RC	Rec. 94%		RQD=58%
14	NQ RC	Rec. 98%		RQD=85%
15	NQ RC	Rec. 96%		RQD=96%



UNIT WEIGHT γ kN/m³

* Water used to facilitate washboring and rock coring, water level not stabilized and hole open to 3.4 m on completion.

Dynamic Cone Penetration Test (DCPT) performed from 0 to 9 m.

+³, ×³: Numbers refer to Sensitivity

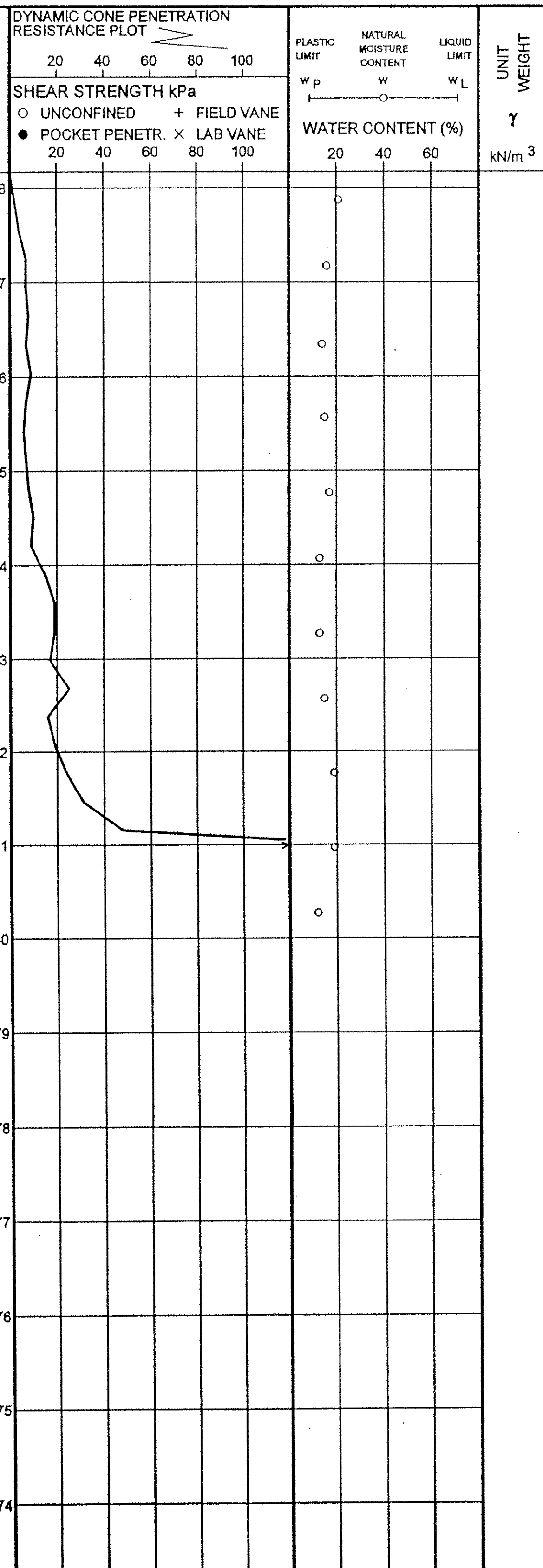
15-⊕-5 (20 above, 10 below) (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 7

METRIC

GWP	354-94-00	LOCATION	Echo River to Bar River Road, Sta. 16+112, 22m Rt. - Coords: N: 5 146 437.7; E: 300253.1	ORIGINATED BY	Y.L.
DIST 62	HWY 17 (New)	BOREHOLE TYPE	Hollow Stem Augers, D.C.P.T., Casing & Washboring & NQ Rock Coring	COMPILED BY	M.L.
DATUM	Geodetic	DATE	11/25/2002	CHECKED BY	R.A.

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
188.2	Ground Surface					
0.0	0.1 m Topsoil; SAND, Fine, trace to some silt, brownish grey, loose to very loose; moist	1	SS	6	* ▼	
	wet	2	SS	3		
		3	SS	3		0 95 (5)
		4	SS	1		
		5	SS	2		
		6	SS	2		
	occasional Cobbles	7	SS	4		
		8	SS	8	**	
		9	SS	5		0 80 (20)
	with silt, occasional cobbles, compact	10	SS	22		Possible cobble at 7.1 m.
		11	SS	25		5 57 25 13
179.9 / 8.3	COBBLES and BOULDERS with, sand and gravel, very dense (inferred); boulders	12	RC	-		
		13	SS	100/13		
		14	RC	-		
	boulders	15	RC	-		
		16	SS	100/0		
		17	RC	-		
	boulder	18	RC	-		
		19	SS	100/0		
176.2 / 12.0	SANDSTONE BEDROCK reddish brown	20	NQ RC	Rec. 96%		RQD=93%
		21	NQ RC	Rec. 100%		RQD=92%



Continued Next Page

$+^3, \times^3$: Numbers refer to Sensitivity

20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 7

METRIC

GWP 354-94-00 LOCATION Echo River to Bar River Road, Sta. 16+112, 22m Rt. - Coords: N: 5 146 437.7; E: 300253.1 ORIGINATED BY Y.L.

DIST 62 HWY 17 (New) BOREHOLE TYPE Hollow Stem Augers, D.C.P.T., Casing & Washboring & NQ Rock Coring COMPILED BY M.L.

DATUM Geodetic DATE 11/25/2002 CHECKED BY R.A.

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● POCKET PENETR., × LAB VANE) 20 40 60 80 100	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
173.1											
15.1	End of borehole						173				
	* Water used to facilitate washboring and rock coring, water level at 0.6 m (not stabilized) and hole open to 15.1 m on completion.										
	**No recovery, 1.2 m sand back-up inside auger. Commence casing and washboring.										
	Dynamic Cone Penetration Test (DCPT) performed from 0 to 7.1 m. Refusal at 7.1 m probably on a cobble or boulder										

$+^3, \times^3$: Numbers refer to Sensitivity   15-⌖-5 (20 above, 10 below) (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 16+ 070; 19 m Lt

METRIC

GWP 354-94-00 LOCATION Echo River to Bar River Road, Sault Ste. Marie, ON - Coords: N 5 146 479.4; E 300 294.0 ORIGINATED BY G.I.

DIST 62 HWY 17 (New) BOREHOLE TYPE Hollow Stem Augers, casing & washboring & NQ Rock Coring COMPILED BY G.T.

DATUM Geodetic DATE 11/13/2002 CHECKED BY R.A.

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
191.7	Ground Surface					
0.0	0.1 m Topsoil; very loose	1	SS	4	191	
	compact	2	SS	20		24 57 17 2
	dense to very dense	3	SS	89	190	
188.8 / 2.9	**SAND** Silty, trace to some gravel brown, damp	4	SS	40	189	4 60 32 4
	Heterogeneous mixture of silty sand, some gravel some cobbles and boulders **SILTY SAND TILL** reddish grey, moist, very dense	5	SS	70		
		6	RC	-	188	
187.4 / 4.3		7	SS	90		
	COBBLES and BOULDERS with sand and gravel brown, wet compact to very dense inferred	8	RC	-		
		9	SS	60/0	187	
		10	RC	-		
		11	RC	-	186	
		12	SS	43	185	
		13	RC	-		
		~~14~~	~~SS~~	~~60/5~~	184	
		15	RC	-	183	
		16	SS	29		
		17	RC	-	182	
181.6 / 10.1		18	RC	-		
180.6 / 11.1	**SANDSTONE BEDROCK** reddish brown	19	NQ RC	Rec. 100%	181	RQD = 89%

End of borehole.

* Water used to facilitate coring and wash boring, water level not stabilized.

Columns also include: STRAT PLOT; GROUND WATER CONDITIONS; DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● POCKET PENETR., × LAB VANE; 20 40 60 80 100); PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m^3.

+^{3}, ×^{3}: Numbers refer to Sensitivity

20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

SPT 1055

RECORD OF BOREHOLE No 16+160 CL

METRIC

GWP 354-94-00	LOCATION Echo River to Bar River Road, ON - Coords: N 5 146 393.1; E 300 273.8		ORIGINATED BY Y.L.
DIST 62	HWY 17 (New)	BOREHOLE TYPE Hollow Stem Augers	COMPILED BY M.L.
DATUM Geodetic	DATE 4/19/2002		CHECKED BY Z.O.

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
188.6	Ground Surface					
0.0	0.1 m Topsoil some organics to 0.5 m	1	SS	6	188	
	SAND Fine, silty, loose to compact, wet	2	SS	11		
	brown ------ reddish grey	3	SS	12 **	187	
		4	SS	10	186	0 60 39 1
185.0 / 3.6		5	SS	7 **	185	
	GRAVELLY SAND reddish grey, wet	6	SS	16		
	compact -------- very loose	7	SS	18	184	
					183	
181.9 / 6.7		8	SS	3	182	
181.5 / 7.1	**SANDY SILT TILL** reddish grey, very dense, wet	9	SS	100/13		4 36 57 3

End of borehole

* Water level at 0.05 m upon completion.

Piezometer installed to 5.2 m.
Water level on:
04/21/2002 - 1.4 m (El. 188.2m)

**Slight hydrocarbon odour.

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● POCKET PENETR. × LAB VANE; 20 40 60 80 100
PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60
UNIT WEIGHT γ kN/m^3

$+^3, \times^3$: Numbers refer to Sensitivity — 20 / 15 ◇ 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 16+235 CL

METRIC

GWP 354-94-00 LOCATION Echo River to Bar River Road, ON - Coords: N 5 146 313.6; E 300 268.8 ORIGINATED BY Y.L.

DIST 62 HWY 17 (New) BOREHOLE TYPE Hollow Stem Augers COMPILED BY M.L.

DATUM Geodetic DATE 4/17/2002 CHECKED BY Z.O.

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	GR	SA	SI	CL
189.0	Ground Surface					189				
0.0	0.1 m Topsoil; some organics to 0.3 m; sandy silt seams to 0.7 m	1	SS	9	*					
	brown, moist	2	SS	20		188	0	88	(12)	
	grey, wet	3	SS	13		187				
	loose to compact	4	SS	6		186				
	very loose	5	SS	2			0	81	(19)	
	SAND Fine, some silt	6	SS	2		185				
		7	SS	3		184				
	loose, occasional gravel reddish grey	8	SS	5		183				
	compact, some gravel	9	SS	12		182, 181				
180.8 / 8.2	**SILTY SAND TILL** reddish grey, very dense, wet	10	SS	50/8						
180.2 / 8.8	End of borehole									

* Wet cave at 1.7 m on completion.

Chart columns: DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100); SHEAR STRENGTH kPa — ○ UNCONFINED, + FIELD VANE, ● POCKET PENETR., × LAB VANE (20, 40, 60, 80, 100); PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) (20, 40, 60); UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%).

$+^3, \times^3$: Numbers refer to Sensitivity

20 / 15 – 5 / 10 (%) STRAIN AT FAILURE

101

SPT 1055

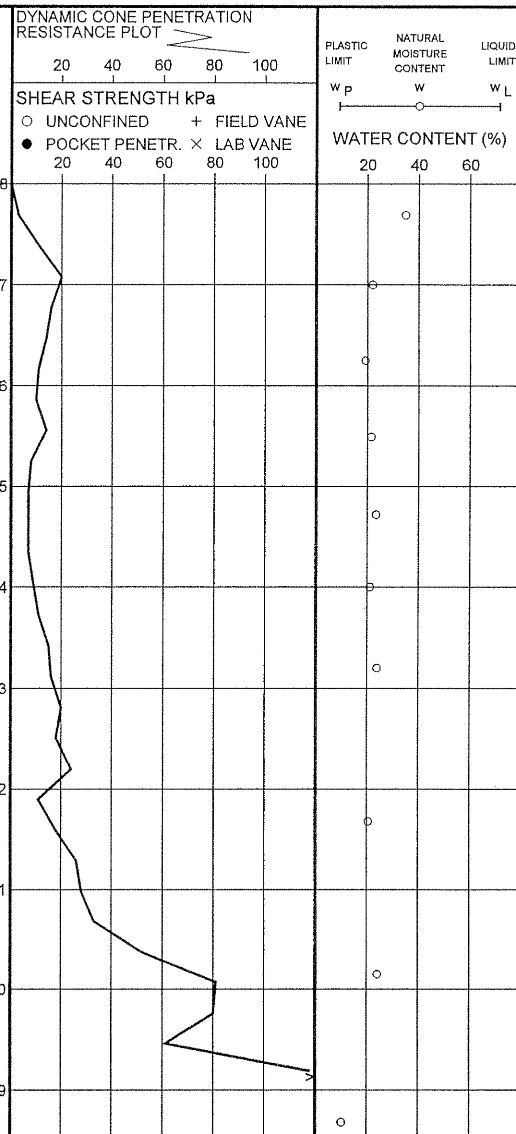
RECORD OF BOREHOLE No 16+330; 20 m Rt 1 OF 1 METRIC

GWP 354-94-00 LOCATION Echo River to Bar River Road, ON - Coords: N 5 146 227.5; E 300 234.9 ORIGINATED BY Y.L.

DIST 62 HWY 17 (New) BOREHOLE TYPE Hollow Stem Augers & DCPT COMPILED BY M.L.

DATUM Geodetic DATE 4/16/2002 CHECKED BY Z.O.

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
188.0	Ground Surface				188		
0.0	0.2 m **TOPSOIL** **CLAY** reddish brown, soft	1	SS	5			
187.3 / 0.7	traces of organics to 2m; loose	2	SS	6	187		
		3	SS	4	186		
	SAND Fine, silty very loose, wet	4	SS	2	185		
		5	SS	1			0 74 24 2
		6	SS	1	184		
	grey; very thin reddish clay grey seams	7	SS	1	183		
					182		
		8	SS	1	181		0 58 42 0
		9	SS	3	180		
179.8 / 8.2	Heterogeneous mixture of Silt and Sand with traces of gravel **(SILTY SAND TILL)** reddish grey, very dense, wet				179		
178.5		10	SS	80/20			13 52 32 3
9.5	End of borehole						



Dynamic Cone Penetration Test (DCPT) performed from 0 m to 8.8 m.

* Wet cave at 1.5 m on completion.

**Piezometer installed to 6.1 m.
Water level on:
10/19/2002 - 0.9 m (El.187.1)

+ ³, × ³: Numbers refer to Sensitivity

20 / 15 5 / 10 (%) STRAIN AT FAILURE

10 6

RECORD OF BOREHOLE No 16+450; 25 m Rt

METRIC

GWP 354-94-00 LOCATION Echo River to Bar River Road, Sault Ste. Marie - Coords: N 5 146 114.3; E 300 196.9 ORIGINATED BY G.I.

DIST 62 HWY 17 (New) BOREHOLE TYPE Hollow Stem Augers COMPILED BY Y.L.

DATUM Geodetic DATE 5/31/2003 CHECKED BY R.M.

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
187.4	Ground Surface							
0.0	0.4 m Peaty Topsoil		1	SS	1	▼*	187	
	very soft / soft to firm		2	SS	2		186	
	CLAY reddish grey, wet		3	SS	0		185	
			4	SS	0		184	
			5	SS	1		183	
			6	SS	2		182	
			7	SS	0		181	
180.4 / 6.9	**SAND** Fine, trace to some silt grey, wet, very loose					**	180	
			8	SS	3			0 90 (10)
179.3 / 8.1	End of Borehole.							

* Water level at ground surface and hole open to 5.2 m on completion.

** Field vane test attempted, unable to push vane beyond 6.9 m

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● POCKET PENETR. × LAB VANE: 20 40 60 80 100

Field vane sensitivities plotted: 10, 6, 6, 6, 7, 6, 7

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L

WATER CONTENT (%): 20 40 60

UNIT WEIGHT γ kN/m³

+³, ×³: Numbers refer to Sensitivity

20 / 15 5 / 10 (%) STRAIN AT FAILURE

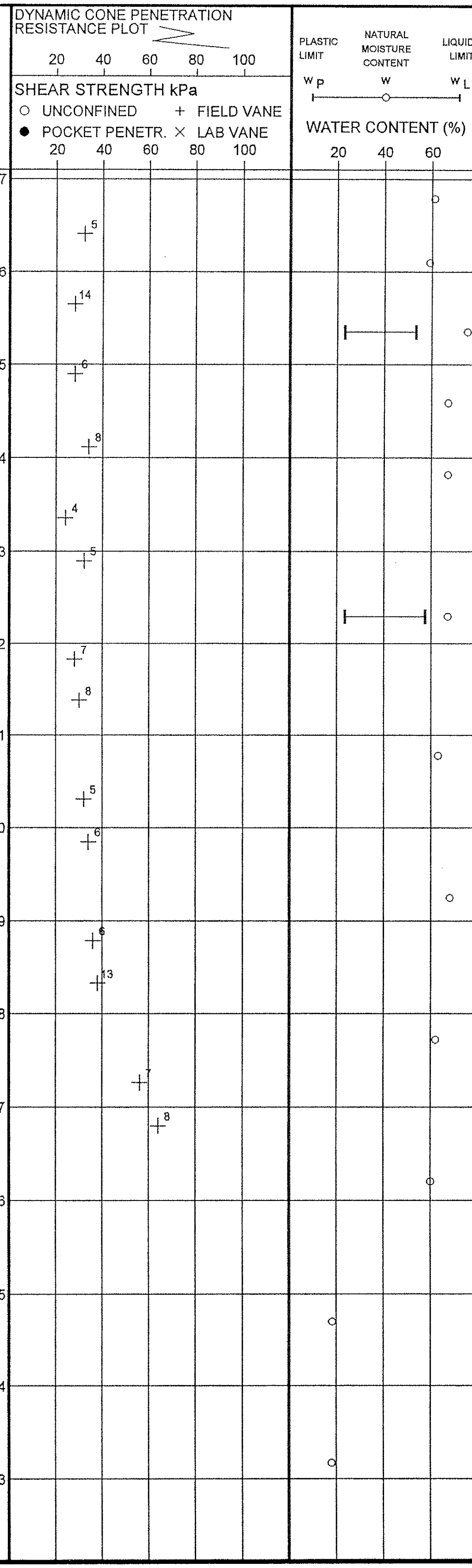
110

RECORD OF BOREHOLE No 16+560; 19 m Lt

METRIC

GWP 354-94-00	LOCATION Echo River to Bar River Road, Sault Ste. Marie - Coords: N 5 145 996.3; E 300 193.7	ORIGINATED BY G.I.
DIST 62 HWY 17 (New)	BOREHOLE TYPE Hollow Stem Augers & D.C.P.T.	COMPILED BY Y.L.
DATUM Geodetic	DATE 5/31/2003	CHECKED BY R.M.

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
187.1	Ground Surface							
0.0	**PEAT** dark brown to black, wet, soft	1	SS	3		187		0.2 m water above ground surface
186.7 / 0.4	**CLAY** reddish grey, wet							
		2	SS	5		186		
		3	SS	2		185		
		4	SS	1		184		
		5	SS	1			15.4	
						183		
		6	SS	0		182		
						181		
		7	SS	1			14.7	
						180		
		8	SS	0		179		
						178		
	soft to firm ------- stiff	9	SS	2		177		
176.1 / 11.0		10	SS	4		176		
	SAND Fine, some silt trace gravel below 14.6 m. grey, wet				**	175		
		11	SS	12		174		
	compact ------- dense							
		12	SS	32		173		



Continued Next Page

+³, ×³: Numbers refer to Sensitivity

20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 16+560; 19 m Lt

METRIC

GWP 354-94-00 LOCATION Echo River to Bar River Road, Sault Ste. Marie - Coords: N 5 145 996.3; E 300 193.7 ORIGINATED BY G.I.

DIST 62 HWY 17 (New) BOREHOLE TYPE Hollow Stem Augers & D.C.P.T. COMPILED BY Y.L.

DATUM Geodetic DATE 5/31/2003 CHECKED BY R.M.

ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	SAND some silt trace gravel, grey, wet, compact		13	SS	19		172				
171.4 15.7	End of Borehole.						171				
169.8 17.3	End of D.C.P.T.						170				

Dynamic Cone Penertration Test (D.C.P.T) performed from 15.5 m to 17.3 m.

* Water level at ground surface and hole open to 10.7 m on completion.

** Field vane test attempted, unable to push vane beyond 11.4 m.

Legend: SHEAR STRENGTH kPa: ○ UNCONFINED; + FIELD VANE; ● POCKET PENETR.; × LAB VANE. PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L.

$+^3, \times^3$: Numbers refer to Sensitivity

20 / 15 5 / 10 (%) STRAIN AT FAILURE

116

Ministry of Transportation Ontario

Foundation Design

SPT 1055

RECORD OF BOREHOLE No 16+650; 19 m Lt

METRIC

GWP 354-94-00 LOCATION Echo River to Bar River Road, Sault Ste. Marie - Coords: N 5 145 916.5; E 300 148.1 ORIGINATED BY G.I.

DIST 62 HWY 17 (New) BOREHOLE TYPE Hollow Stem Augers COMPILED BY Y.L.

DATUM Geodetic DATE 5/30/2003 CHECKED BY R.M.

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	UNIT WEIGHT γ kN/m³
187.6	Ground Surface					
0.0	0.2 m Topsoil **FINE SAND** trace to some silt, brown, moist, very loose	1	SS	3	0 93 (7)	
186.9 0.7	**CLAY** reddish grey to grey frequent silt seams/partings	2	SS	6		17.7
	stiff ------- soft to firm	3	SS	5		16.1
		4	SS	2		
		5	SS	1		
		6	TW	PH		
		7	SS	0		
180.3 7.3	End of Borehole. * Wet cave at 4.9 m on completion					

Elevation scale: 187, 186, 185, 184, 183, 182, 181

Field vane shear strength (sensitivity): +7, +4, +6, +7, +6, +4, +4, +5, +6

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● POCKET PENETR. × LAB VANE 20 40 60 80 100

PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L

WATER CONTENT (%) 20 40 60 (81)

+³, ×³: Numbers refer to Sensitivity

20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

90

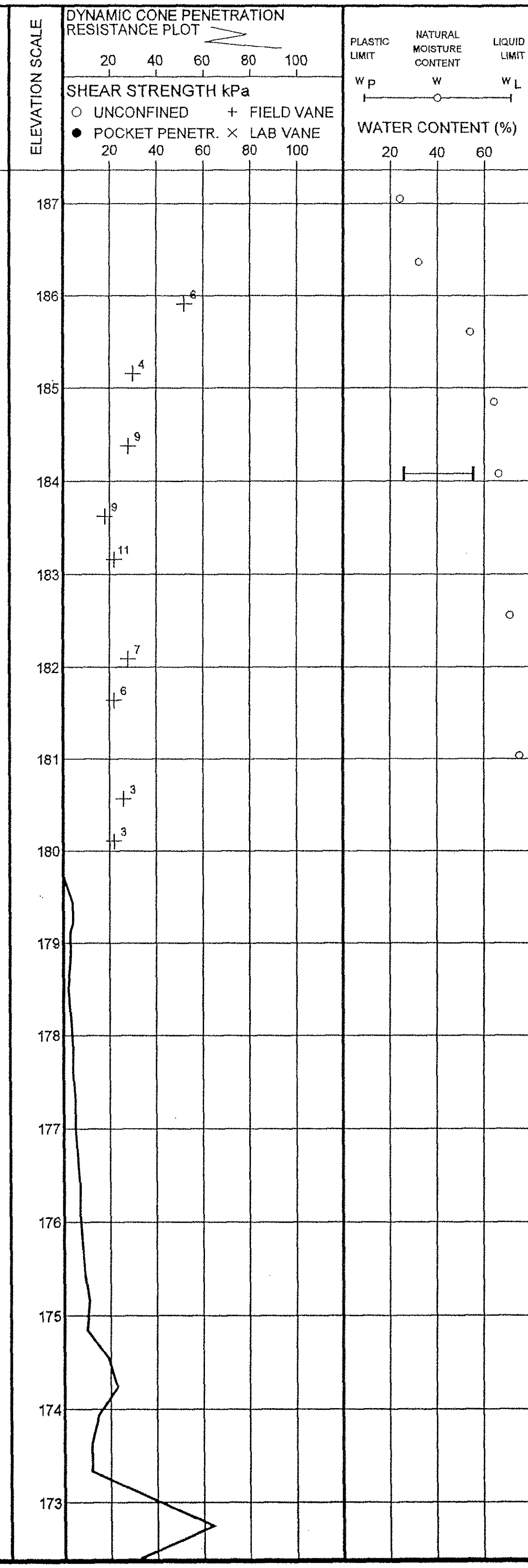
SPT 1055

RECORD OF BOREHOLE No 16+800; 19 m Rt

METRIC

GWP 354-94-00	LOCATION Echo River to Bar River Road, Sault Ste. Marie - Coords: N 5 145 819.2; E 300 027.4		ORIGINATED BY G.I.
DIST 62	HWY 17 (New)	BOREHOLE TYPE Hollow Stem Augers & D.C.P.T.	COMPILED BY Y.L.
DATUM Geodetic	DATE 5/30/2003		CHECKED BY R.M.

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	UNIT WEIGHT γ kN/m³	GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
187.4	Ground Surface						
0.0	0.2 m Topsoil **FINE SAND** some silt, brown, moist, compact	1	SS	14			2 85 (13)
186.7 0.7	**CLAY** grey to reddish grey stiff occasional silt seams/pockets firm soft to firm	2	SS	10		17.8	
		3	SS	4			
		4	SS	1			
		5	SS	0			
		6	SS	1	▼*		
		7	SS	1			
180.0 7.3	End of Borehole.						



Continued Next Page

$+^3, \times^3$: Numbers refer to Sensitivity

20 / 15 5 / 10 (%) STRAIN AT FAILURE

91

SPT 1055

RECORD OF BOREHOLE No 16+800; 19 m Rt

METRIC

GWP 354-94-00	LOCATION Echo River to Bar River Road, Sault Ste. Marie - Coords: N 5 145 819.2; E 300 027.4		ORIGINATED BY G.I.
DIST 62	HWY 17 (New)	BOREHOLE TYPE Hollow Stem Augers & D.C.P.T.	COMPILED BY Y.L.
DATUM Geodetic		DATE 5/30/2003	CHECKED BY R.M.

SOIL PROFILE			SAMPLES									
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● POCKET PENETR. × LAB VANE (20 40 60 80 100)	PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) (20 40 60)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
							172				
							171				
170.2 / 17.2	End of D.C.P.T. Dynamic Cone Penertration Test (D.C.P.T) performed from 7.3 m to 17.2 m. * Water level at 5.2 m (not stabilized), and hole open to 10.7 m on completion.										

$+^3, \times^3$: Numbers refer to Sensitivity

20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

92

RECORD OF BOREHOLE No 17+000; 19 m Lt

METRIC

GWP 354-94-00 LOCATION Echo River to Bar River Road, Sault Ste. Marie - Coords: N 5 145 641.6; E 299 927.9 ORIGINATED BY G.I.
DIST 62 HWY 17 (New) BOREHOLE TYPE Hollow Stem Augers COMPILED BY Y.L.
DATUM Geodetic DATE 5/30/2003 CHECKED BY R.M.

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS
186.3	Ground Surface				
0.0	0.2 m Topsoil **SILTY SAND** brown, moist, loose	1	SS	8	*
185.6 / 0.7	**CLAY** reddish grey to grey; firm / soft to firm	2	SS	9	
		3	SS	7	
		4	SS	5	
		5	SS	2	
		6	SS	1	
		7	SS	1	
179.0 / 7.3	End of Borehole.				

* Wet cave at 5.5 m on completion

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; ELEVATION SCALE (186, 185, 184, 183, 182, 181, 180, 179); DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● POCKET PENETR., × LAB VANE; 20 40 60 80 100); PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

Field vane sensitivity values plotted: 4, 8, 7, 6, 4, 4, 8, 9

$+^3, \times^3$: Numbers refer to Sensitivity 15–20–5–10 (%) STRAIN AT FAILURE

93

Ministry of Transportation
Ontario

SPT 1055

RECORD OF BOREHOLE No 17+300; 19 m Lt 1 OF 2 METRIC

GWP 354-94-00 LOCATION Echo River to Bar River Road, Sault Ste. Marie - Coords: N 5 145 409.1; E 299 742.5 ORIGINATED BY G.I.

DIST 62 HWY 17 (New) BOREHOLE TYPE Hollow Stem Augers COMPILED BY Y.L.

DATUM Geodetic DATE 5/29/2003 CHECKED BY R.M.

SOIL PROFILE

ELEV DEPTH	DESCRIPTION
182.5	Ground Surface
0.0	0.3 m Topsoil
	CLAY trace rootlets to 1.5 m
	reddish grey
	grey
	reddish grey
	soft to firm
	firm to stiff

SAMPLES

NUMBER	TYPE	"N" VALUES
1	SS	2
2	SS	1
3	SS	1
4	SS	0
5	SS	0
6	SS	1
7	SS	1
8	SS	2
9	SS	2
10	SS	1
11	TW	PH
12	SS	2

GROUND WATER CONDITIONS

ELEVATION SCALE: 182, 181, 180, 179, 178, 177, 176, 175, 174, 173, 172, 171, 170, 169, 168

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

SHEAR STRENGTH kPa
○ UNCONFINED + FIELD VANE
● POCKET PENETR. × LAB VANE
20 40 60 80 100

PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L

WATER CONTENT (%) 20 40 60

UNIT WEIGHT γ kN/m³

REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

Continued Next Page

+³, ×³: Numbers refer to Sensitivity

20 15 5 10 (%) STRAIN AT FAILURE

96

Ministry of Transportation
Ontario

SPT 1055

RECORD OF BOREHOLE No 17+300; 19 m Lt

METRIC

GWP 354-94-00 | LOCATION Echo River to Bar River Road, Sault Ste. Marie - Coords: N 5 145 409.1; E 299 742.5 | ORIGINATED BY G.I.

DIST 62 HWY 17 (New) | BOREHOLE TYPE Hollow Stem Augers | COMPILED BY Y.L.

DATUM Geodetic | DATE 5/29/2003 | CHECKED BY R.M.

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● POCKET PENETR., × LAB VANE) 20 40 60 80 100	PLASTIC LIMIT w_P – NATURAL MOISTURE CONTENT w – LIQUID LIMIT w_L / WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	CLAY reddish grey, stiff		13	SS	2		167	+5, +4 (field vane, ~60 kPa)	○ (~62)		
166.1 / 16.5	End of Borehole. * Water level at 12.8 m (not stabilized), and hole open to 13.1 m on completion.										

+ ³, × ³: Numbers refer to Sensitivity

15-⊕-5 (20 above, 10 below) (%) STRAIN AT FAILURE

97

Ministry of Transportation Ontario

Foundation Design

SPT 1055

RECORD OF BOREHOLE No 17+450; 19 m Rt

METRIC

GWP 354-94-00 LOCATION Echo River to Bar River Road, Sault Ste. Marie - Coords: N 5 145 294.5; E 299 638.8 ORIGINATED BY G.I.

DIST 62 HWY 17 (New) BOREHOLE TYPE Hollow Stem Augers & D.C.P.T. COMPILED BY Y.L.

DATUM Geodetic DATE 5/28/2003 CHECKED BY R.M.

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	UNIT WEIGHT γ kN/m³
182.4	Ground Surface					
0.0	0.2 m Topsoil	1	SS	4	182	
	very stiff					
	stiff to firm	2	SS	2	181	
		3	TW	PH		15.4
		4	SS	0	180	
		5	SS	0	179	
					178	
	CLAY trace rootlets to 0.7 m	6	SS	1	177	
		7	SS	0	176	
	reddish grey	8	SS	1	175, 174	
		9	SS	1	173	
	grey	10	SS	1	172, 171	
	reddish grey	11	SS	2	170, 169	15.1
		12	SS	1	168	15.4
	stiff					

Continued Next Page

$+^3, \times^3$: Numbers refer to Sensitivity

15, 20, 5, 10 (%) STRAIN AT FAILURE

98

Ministry of Transportation
Ontario

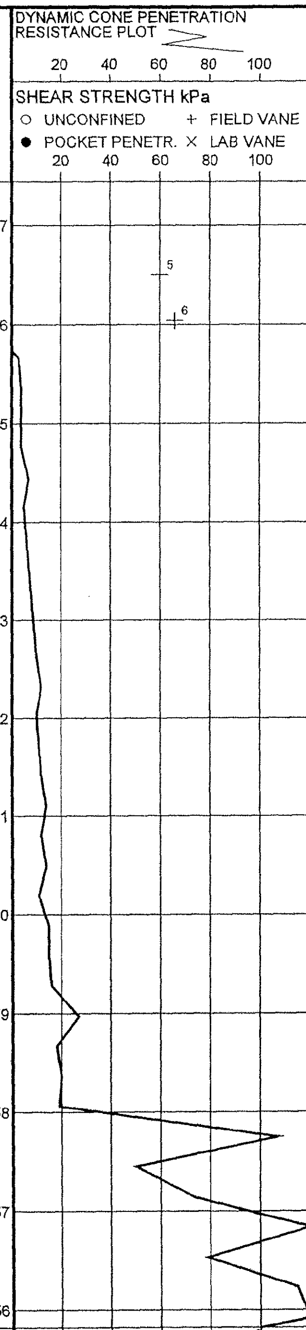
SPT 1055

RECORD OF BOREHOLE No 17+450; 19 m Rt

METRIC

GWP 354-94-00	LOCATION Echo River to Bar River Road, Sault Ste. Marie - Coords: N 5 145 294.5; E 299 638.8	ORIGINATED BY G.I.
DIST 62 HWY 17 (New)	BOREHOLE TYPE Hollow Stem Augers & D.C.P.T.	COMPILED BY Y.L.
DATUM Geodetic	DATE 5/28/2003	CHECKED BY R.M.

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	CLAY reddish grey, stiff		13	SS	2	▼*	167		15.5	
166.0 16.5	End of Borehole.						166			
155.8 26.6	End of D.C.P.T. Dynamic Cone Penertration Test (D.C.P.T) performed from 15.7 m to 19 m. * Water level at 15.2 m (not stabilized), and hole open to 15.8 m on completion.									



PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60

$+^3, \times^3$: Numbers refer to Sensitivity; 15-5 (20/10) (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 17+600; 19 m Lt   1 OF 1   METRIC

GWP 354-94-00   LOCATION Echo River to Bar River Road, Sault Ste. Marie - Coords: N 5 145 140.4; E 299 627.4   ORIGINATED BY G.I.
DIST 62   HWY 17 (New)   BOREHOLE TYPE Hollow Stem Augers   COMPILED BY Y.L.
DATUM Geodetic   DATE 5/28/2003   CHECKED BY R.M.

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa (Field Vane, sensitivity)	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
183.2	Ground Surface												
0.0	0.4 m Topsoil					183							
	SILTY SAND moist to wet, loose to compact	1	SS	5									
	brown / grey	2	SS	10		182				0	63	29	8
		3	SS	6									
181.1 / 2.1	**CLAY** reddish grey, wet					181							
	stiff to very stiff	4	SS	4									
	soft to firm				▼*								
		5	SS	2		180			16.3				
							18						
						179	4						
		6	SS	1				98					
						178	8						
							6						
		7	SS	0		177		87					
							5						
175.9 / 7.3	End of Borehole.					176	6						

* Water level at 3.0 m (not stabilized), and hole open to 5.2 m on completion.

Legend: DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100); SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● POCKET PENETR., × LAB VANE (20, 40, 60, 80, 100); PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) (20, 40, 60); REMARKS & GRAIN SIZE DISTRIBUTION (%)

$+^3$, $\times^3$: Numbers refer to Sensitivity   15-⊕-5 (20 above, 10 below) (%) STRAIN AT FAILURE

100

TABLE C1 – SUMMARY OF FOUNDATION ENGINEERING PARAMETERS - FLYOVER WEST ALTERNATIVE 5

Stratigraphic Unit	***Average Top Elevation (m)****	***Thickness** (m)***	***γ' (kN/m^3)***	***ϕ' (^{o})***	***c' (kPa)***	***s_u (kPa)***	***σ_p' (kPa)***	***e_o***	***C_c***	***C_r***	***E' (MPa)***	***$C_{\alpha(\varepsilon)}$ (%)***		***c_h (cm^2/s)***
												N/C	***O/C***	
Granular Fill (sub-excavate and replace near surface topsoil/soft clay soil)	184.3	2.0	21	32	0	--	--	--	--	--	15	--	--	--
Silty Clay to Clay (soft)	182.3	4.3	17	21	0	20	91	1.85	0.9	0.09	--	0.5	0.05	3.5×10^{-3}
Silty Clay to Clay (soft to firm)	178	7.2	17	21	0	20 - 32	91 - 145	1.85	0.9	0.09	--	0.5	0.05	3.5×10^{-3}
Sand / Silt / Gravel	170.8	7.5	20	30	0	--	--	--	--	--	30	--	--	--

*Average Elevation of top of stratigraphic unit at Borehole and CPT locations (refer to Drawing C1)

**Average Thickness of stratigraphic unit at Borehole and CPT locations (refer to Drawing C1)

Prepared By: MWK

Reviewed By: JPD

TABLE C2 – EVALUATION OF BRIDGE STRUCTURE FOUNDATION ALTERNATIVES - FLYOVER WEST ALTERNATIVE 5

Foundation Option	*Rank*	*Advantages*	*Disadvantages*	*Relative Costs*	*Risks / Consequences*
Spread Footings on Overburden	Not feasible	• Relative ease of construction.	• Groundwater control required for excavation and during footing construction. • Large post-construction settlements. • Low geotechnical resistance at ULS and SLS of native soils and hence very large footings required.	• Lower relative cost than piled foundations.	• Footing size required to accommodate very low geotechnical resistances is not practical. • Very large post-construction settlements could not be tolerated by bridge structure.
Piles driven to bedrock (approx. 20 m to 25 m long piles)	1	• Limited sub-excavation required for pile cap construction. • Negligible post-construction settlement. • Higher axial resistance than for friction piles. • Fewer piles required than for friction piles option	• Heavier pile sections will be required to penetrate cobbles and boulders and seat piles on bedrock.	• Higher cost associated with heavier pile sections and somewhat greater pile lengths. • Higher cost associated with provisions for re-driving piles for piles driven out of alignment.	• Damaged piles and piles driven out of alignment may require removal and replacement with new piles. • The abutment/pier design should be flexible enough to allow for installation of extra piles in the footing area, if deemed necessary during construction.
Friction Piles (17 m long piles)	2	• Limited sub-excavation required for pile cap construction. • Minor post-construction settlement. • Shorter piles required than for piles driven to bedrock option.	• Lower pile capacity than piles driven to refusal.	• Lower cost associated with shorter pile lengths and lighter pile section. • Higher cost associated with additional piles due to lower axial capacity. • Additional cost for pile load tests.	• Lower pile capacity will require more piles at each foundation unit. • May require pile load tests to verify pile capacity.

Prepared By: MWK

Reviewed By: JPD/JMAC

TABLE C3 – EVALUATION OF APPROACH EMBANKMENT FOUNDATION STABILITY/SETTLEMENT MITIGATION ALTERNATIVES - FLYOVER WEST ALTERNATIVE 5

Stability / Settlement Mitigation Option	*Rank*	*Advantages*	*Disadvantages*	*Relative Costs*	*Risks / Consequences*
Staged construction (with wick drains,10 m wide by 2 m high toe berms and 2 m subexcavate and replace) (6 stages) (approximately 3 years of construction delays for staging)	1	• Smaller embankment footprint and less land acquisition requirements as compared with toe berms only option.	• Somewhat larger volume of excess excavation spoil due to berms. • Somewhat greater quantities of fill required for replacement in subexcavated area due to berms. • Delay of approximately 3 years during staged construction and preloading. • Large post-construction settlement. • Large downdrag loads reduce pile capacity. • EPS required to maintain front slope stability and top-up to mitigate long-term settlements.	• $280,000 to $420,000 (wick drains at 1.5 m spacing) + • $57,500 (berms) + • $374,500 (subexcavate / replace) + • $1,637,200 cost of EPS to mitigate long-term settlements and front slope stability.	• Staged construction sequence required with potential for additional delays during construction depending on monitoring. • Post-construction settlements may require long-term maintenance. • Nominal size toe berms are required for stability, increasing footprint. • Some secondary consolidation (creep) will occur. • Potential need to acquire some additional lands for right-of-way.

TABLE C3 – EVALUATION OF APPROACH EMBANKMENT FOUNDATION STABILITY/SETTLEMENT MITIGATION ALTERNATIVES - FLYOVER WEST ALTERNATIVE 5

Stability / Settlement Mitigation Option	***Rank***	***Advantages***	***Disadvantages***	***Relative Costs***	***Risks / Consequences***
Toe berms up to 31 m wide (with 2 m subexcavate and replace) (with up to 2 year preload)	4	• Standard construction operation. • No construction delays associated with staging.	• Generation of larger volume of excess excavation spoil due to large toe berm footprint. • Greater quantities of fill required for large berms and for subexcavate and replace area. • Large embankment footprint. • Large post-construction settlement. • Large downdrag loads reduce pile capacity. • EPS required to maintain front slope stability and top-up to mitigate long-term settlements. • Preload period required to mitigate settlements	• $640,483 (subexcavate/replace and toe berms) + • land acquisition costs + • $1,637,200 cost of EPS to mitigate long-term settlements and front slope stability.	• Risk of instability (low). • Secondary consolidation (creep) will occur. • Large post-construction settlements will require long-term maintenance. • Likely need to acquire additional right-of-way due to large berm size.

TABLE C3 – EVALUATION OF APPROACH EMBANKMENT FOUNDATION STABILITY/SETTLEMENT MITIGATION ALTERNATIVES - FLYOVER WEST ALTERNATIVE 5

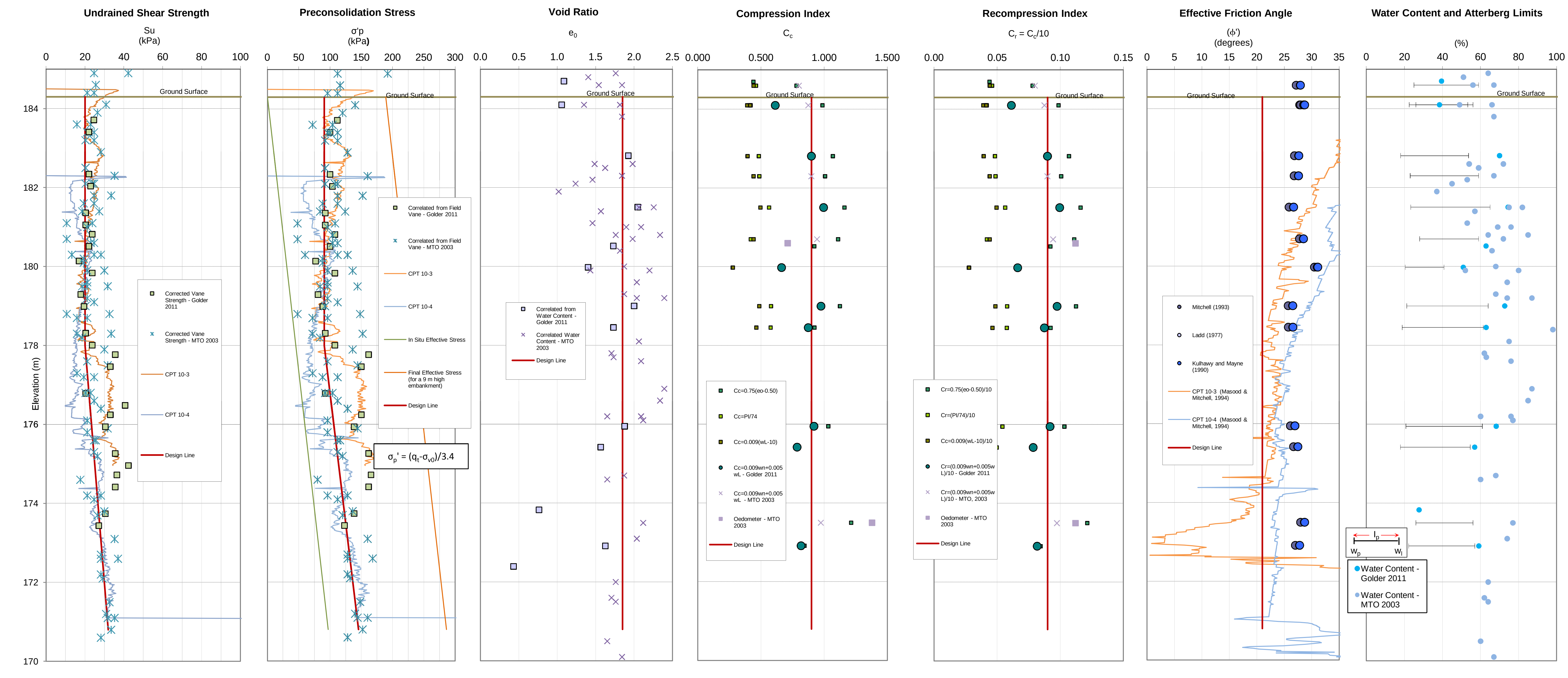
Stability / Settlement Mitigation Option	*Rank*	*Advantages*	*Disadvantages*	*Relative Costs*	*Risks / Consequences*
Partial Lightweight Fill (EPS) (with 2 m subexcavate and replace) (with up to 2 year preload)	3	• Standard construction operation. • No construction delay • Reduced secondary (creep) consolidation settlement. • Generation of smaller volume of excess excavation spoil since no toe berms. • Smaller quantities of fill required for subexcavate and replace since no toe berms. • Smaller embankment footprint.	• Higher cost for specialized materials • Additional effort required for sub-excavation and replacement. • EPS required to maintain front slope stability and to-up to mitigate long term settlements. • Some post construction settlements. • Preload period required to mitigate settlements	• $374,500 (subexcavate/replace)+ • $2,511,600 cost of EPS to mitigate long-term settlements and front slope stability.	• Risk of instability (low). • Secondary consolidation (creep) will occur. • Post-construction settlement may require long-term maintenance. • Potential for smaller property acquisition needs.
Full Lightweight Fill (EPS) (with 2 m subexcavate and replace)	2	• Standard construction operation. • No construction delays associated with staging or preloading. • Minimized post-construction settlement. • Smallest embankment footprint.	• Higher cost for specialized materials • Restricted use of EPS within the embankment cross-section to above water table.	• $374,500 (subexcavate/replace)+ • $4,368,000 cost of EPS to mitigate long-term settlements and front slope stability.	• Low risk of instability. • Low risk of long-term settlement of foundation soils.

Prepared By: MWK

Reviewed By: JPD/JMAC

Summary of Engineering Parameters for Cohesive Deposits
Flyover West Alternative 5

Figure C1



NOTES:

Average ground surface at proposed abutments is at about 184.3 m
Elevation of bottom of cohesive deposit at flyover location is about 170.8 m

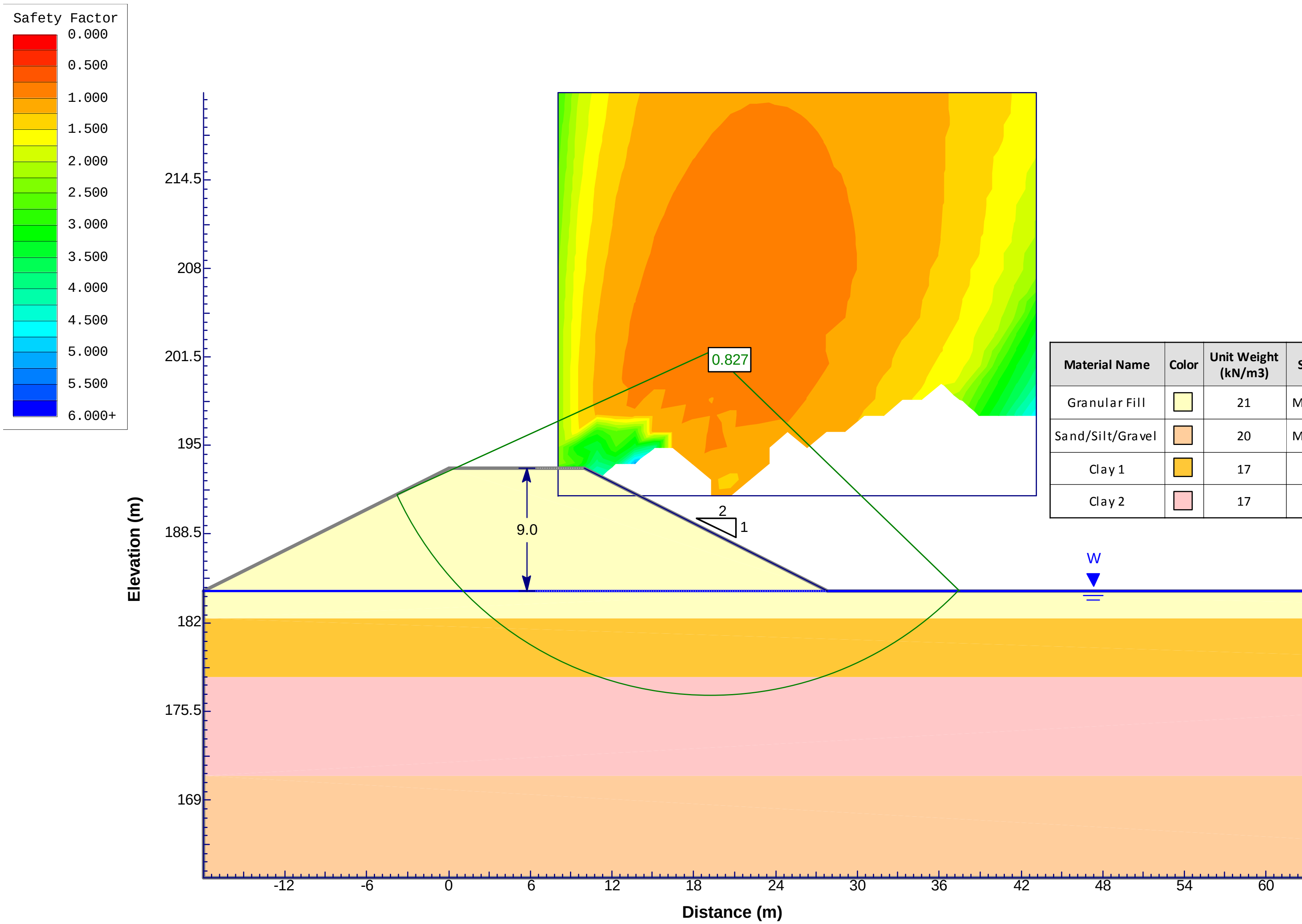
Date: Jun-12 **DB:** MWK
Project No: 09-1111-0016 **CHK:** JPD

Golder Associates

N:\Active\2009\1111\09-1111-0016 Genivar - Hwy 17 Interchange - Echo Bay\Reporting\Prelim Design\FINAL\Appendix C - West Flyover Alternative 5\working\[09-1111-0016 Highway 69 - Echo Bay soil properties - new flyover west.xls]Su

Slope Stability – Total Stress Analysis – 2.0 m Subexcavate and Replace Only

Figure C2-1



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kN/m2)	Phi	Cohesion Type
Granular Fill		21	Mohr-Coulomb	0	32	
Sand/Silt/Gravel		20	Mohr-Coulomb	0	30	
Clay 1		17	Undrained	20		Constant
Clay 2		17	Undrained	20		FDepth

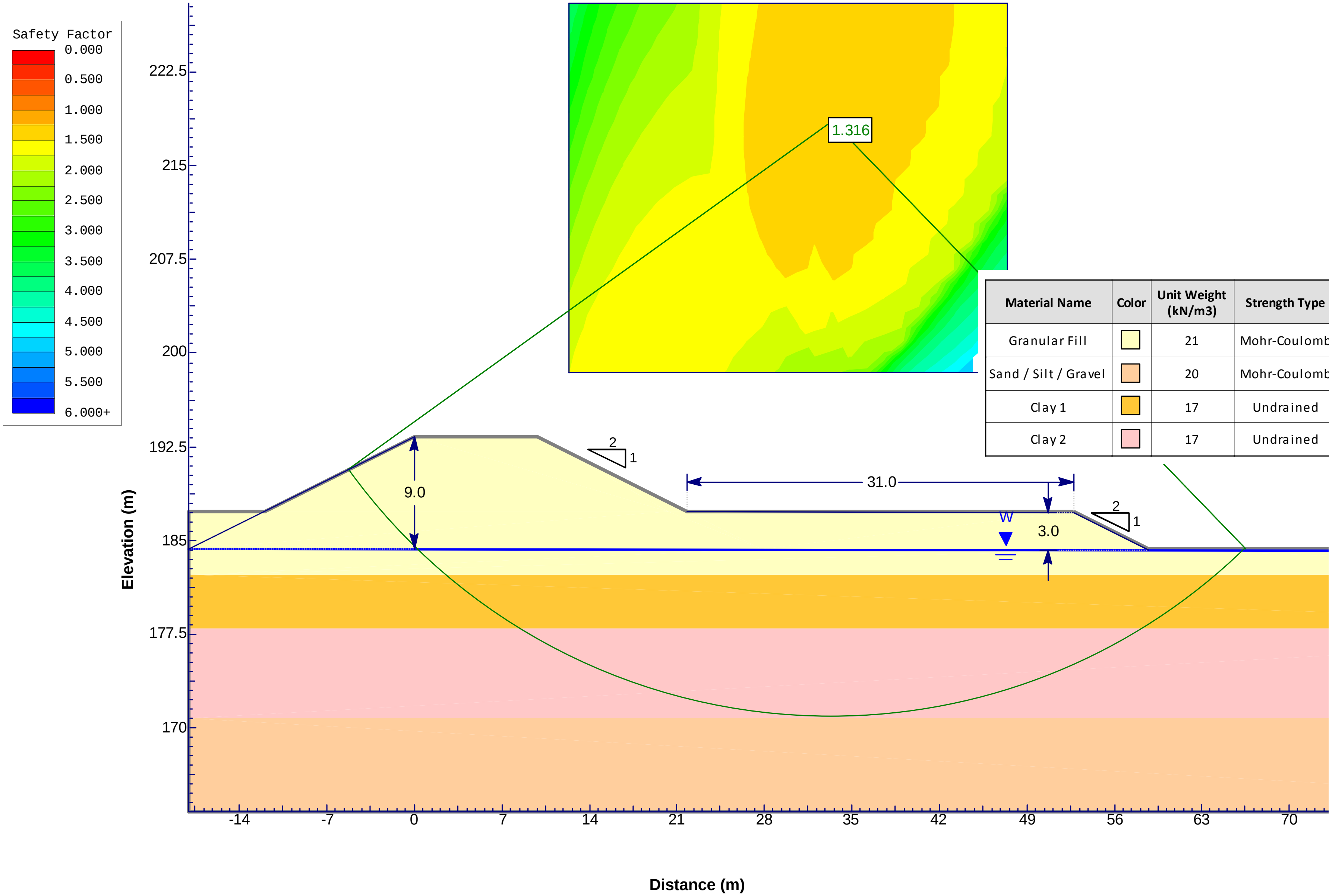
Date: June 2012
Project No: 09-1111-0016

Analysis By: MWK **Reviewed By:** JPD

Golder Associates

Slope Stability – Total Stress Analysis – Stabilizing Berms

Figure C2-2



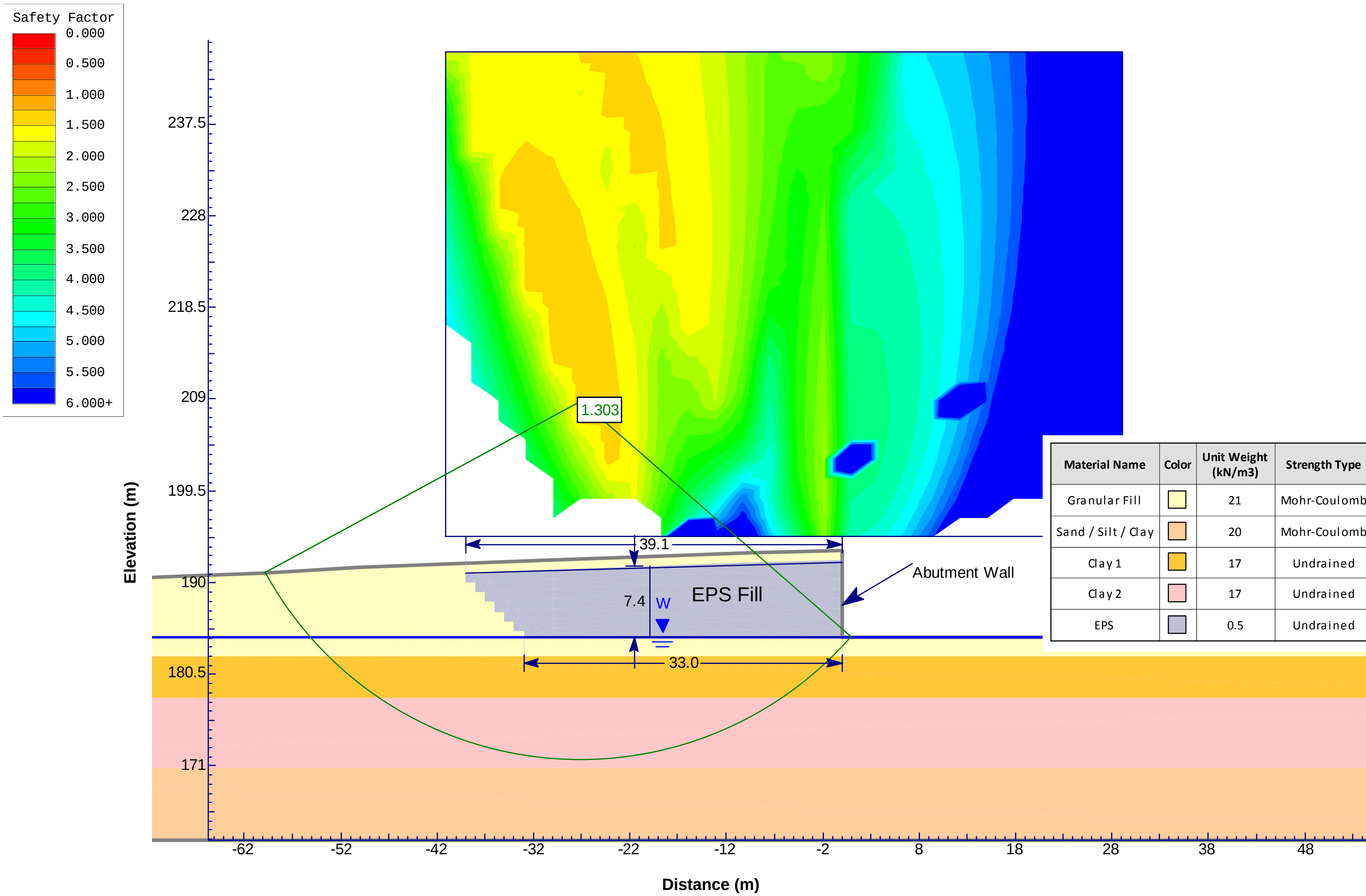
Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kN/m2)	Phi	Cohesion Type
Granular Fill		21	Mohr-Coulomb	0	32	
Sand / Silt / Gravel		20	Mohr-Coulomb	0	30	
Clay 1		17	Undrained	20		Constant
Clay 2		17	Undrained	20		FDepth

Date: June 2012
Project No: 09-1111-0016

Analysis By: MWK **Reviewed By:** JPD

Golder Associates

Slope Stability – Total Stress Analysis – Front Slope Stability (with EPS)



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kN/m2)	Phi	Cohesion Type
Granular Fill		21	Mohr-Coulomb	0	32	
Sand / Silt / Clay		20	Mohr-Coulomb	0	30	
Clay 1		17	Undrained	20		Constant
Clay 2		17	Undrained	20		FDepth
EPS		0.5	Undrained	15		Constant

Date: June 2012
Project No: 09-1111-0016

Analysis By: MWK **Reviewed By:** JPD

Golder Associates

APPENDIX D

Highway 17 / Highway 638 Interchange

MINISTRY OF TRANSPORTATION, ONTARIO

METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

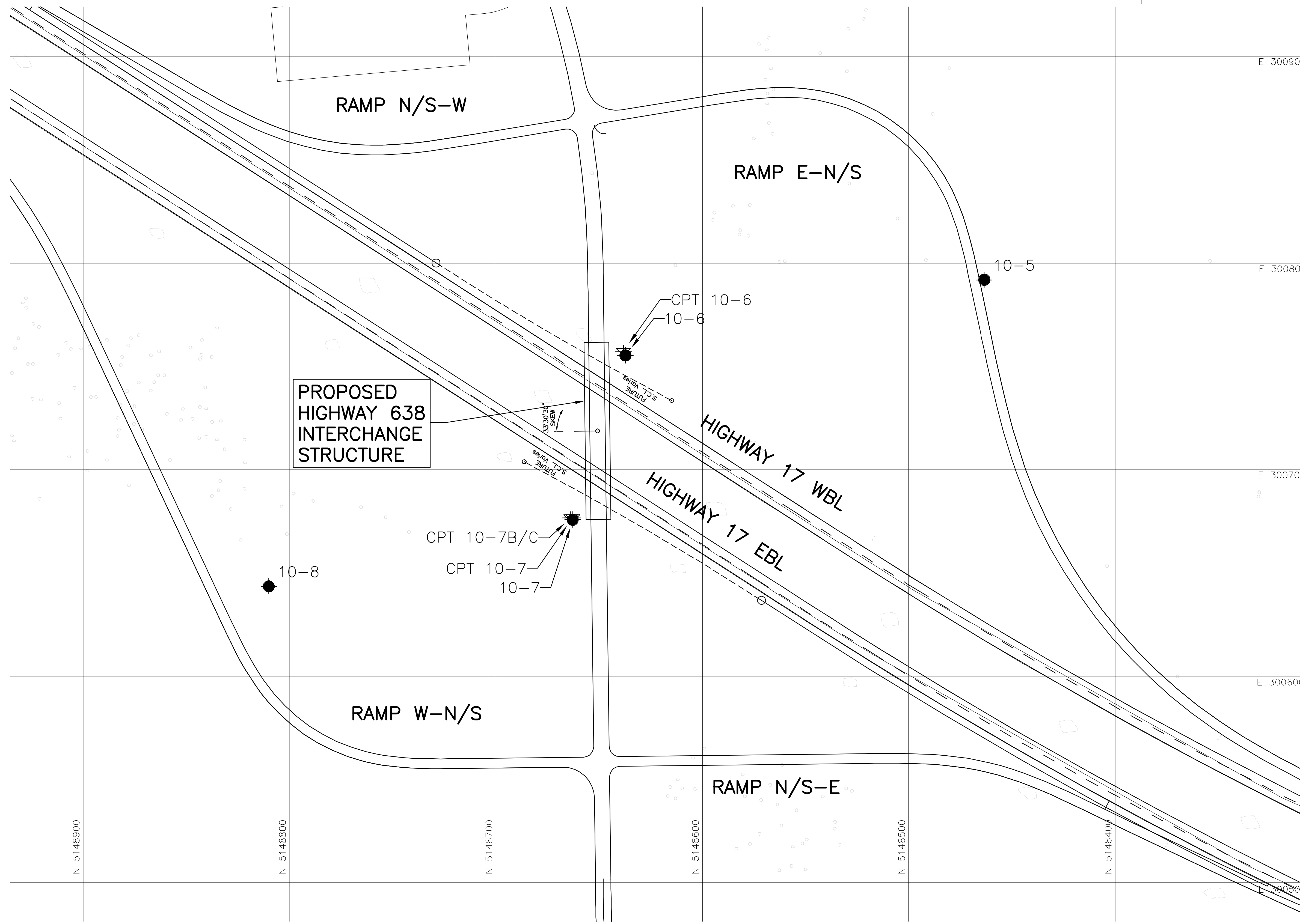
CONT No.
GWP No. 5022-07-00

SHEET

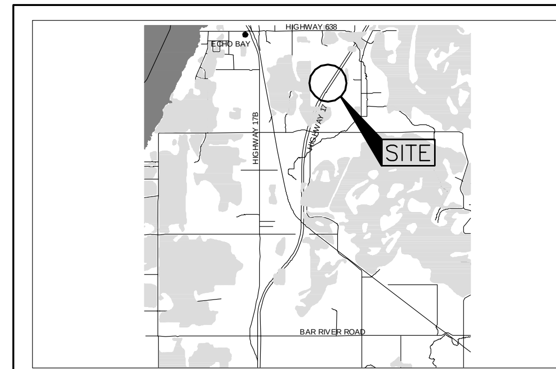
HIGHWAY 17
HIGHWAY 638 INTERCHANGE STRUCTURE AND RAMPS
BOREHOLE LOCATIONS

Golder Associates

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



PLAN
SCALE
20 0 20 40 m



KEY PLAN
SCALE
1.2 0 1.2 2.4 km

LEGEND

Borehole – Current Investigation

CPT – Current Investigation

BOREHOLE CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
10-5	184.4	5148463.4	300792.0
10-6	183.3	5148637.3	300755.4
10-7	183.8	5148662.8	300675.7
10-8	183.3	5148810.1	300643.5

CPT CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
CPT 10-6	184.4	5148638.3	300757.4
CPT 10-7	183.8	5148662.8	300676.7
CPT 10-7B/C	183.8	5148663.8	300676.7

NOTES

This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contracts Documents.

The boundaries between soil strata have been established only at borehole locations. Between boreholes the boundaries are assumed from geological evidence.

The complete Foundation Investigation and Design Report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with Section GC 2.01 of OPS General Conditions.

REFERENCE

Base plans provided in digital format by URS, drawing file 09-079 – Base Plan.dwg, received July 15, 2010 and for the General Arrangement drawing file 09-079 Recommended Plan for Geotech-A.dwg, received April 4, 2011.

NO.	DATE	BY	REVISION

Geocres No. 41K-90			
HWY. 17	PROJECT NO. 09-1111-0016		DIST.
SUBM'D. MWK	CHKD. JPD	DATE: 7/12/2012	SITE:
DRAWN: JFC	CHKD. MWK	APPD. JPD	DWG. D1

PLOT DATE: July 12, 2012
FILENAME: T:\Projects\2009\09-1111-0016 (Genivar, Hwy 17, Echo Bay)\-BA- (Highway 17 & Bar River Road)\0911110016BA0D1.dwg

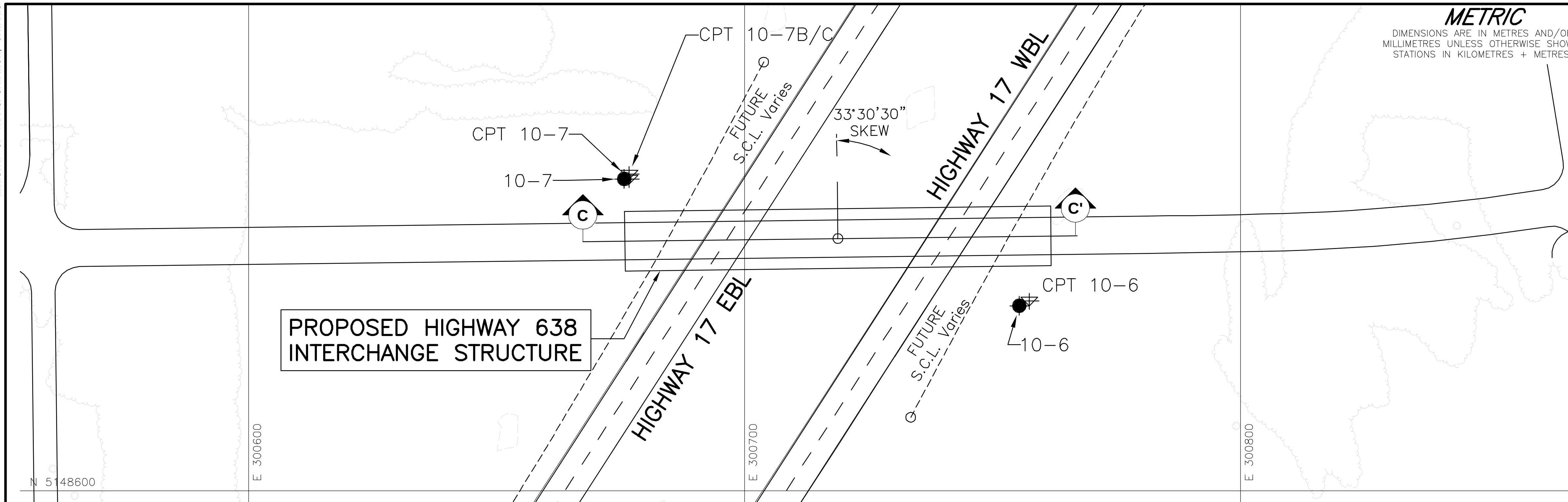
CONT No.
WP No. 5022-07-00

HIGHWAY 17
HIGHWAY 638 INTERCHANGE STRUCTURE
BOREHOLE LOCATIONS AND SOIL STRATA

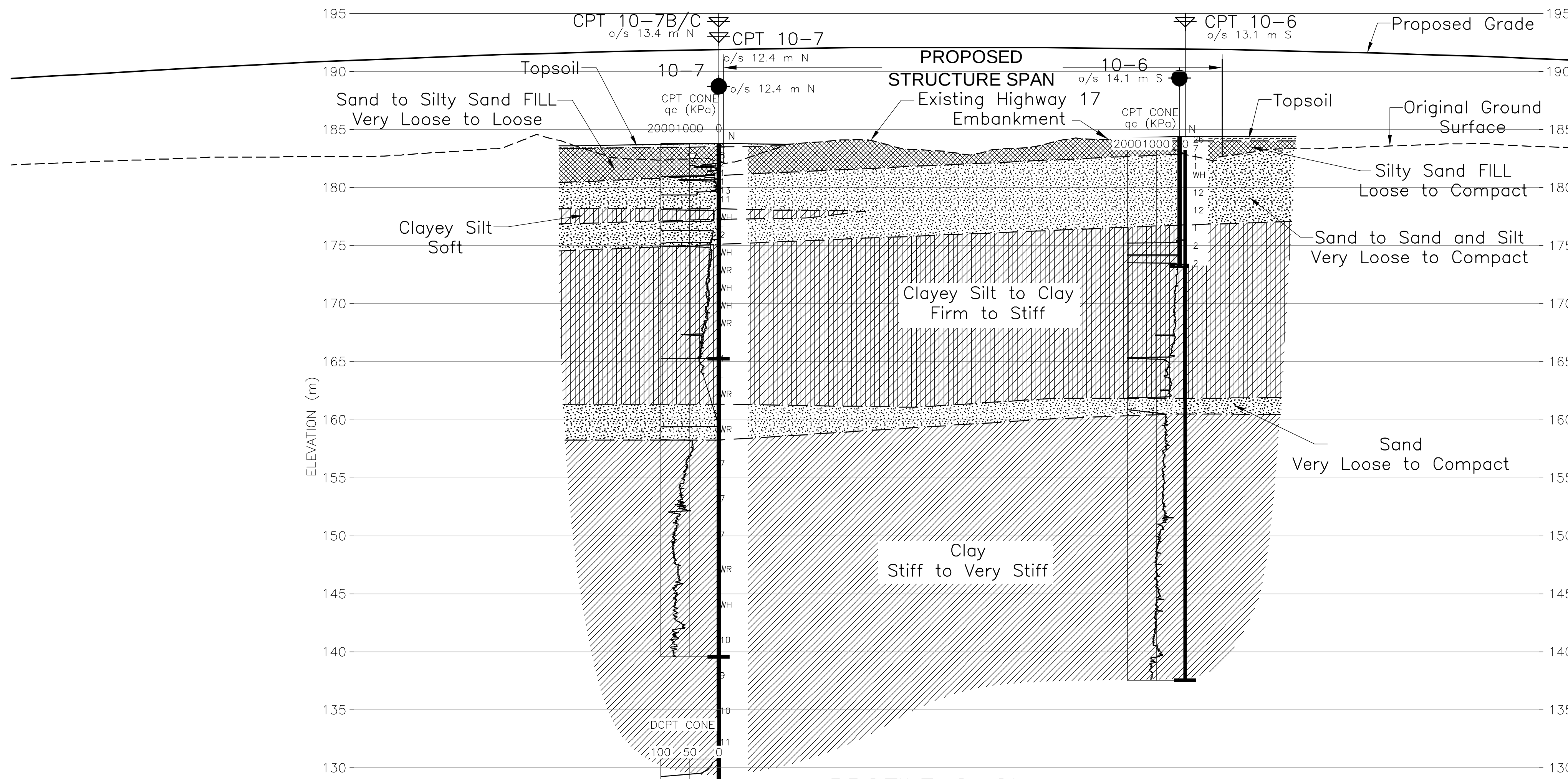
SHEET

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA

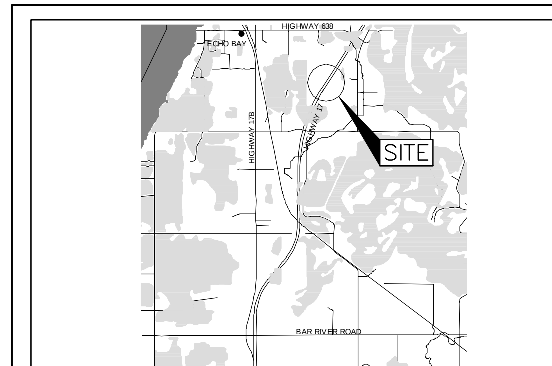
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.



PLAN
SCALE
10 0 10 20 m



PROFILE C–C'
HORIZ. SCALE
10 0 10 20 m
5 0 5 10 m
VERT. SCALE



KEY PLAN
SCALE
1.2 0 1.2 2.4 km

LEGEND

●	Borehole – Current Investigation
CPT symbol	CPT – Current Investigation
N	Standard Penetration Test Value
16	Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
100%	Rock Quality Designation (RQD)
7	CPT tip resistance qc (kPa)

BOREHOLE CO-ORDINATES			
No.	ELEVATION	NORTHING	EASTING
10-6	183.3	5148637.3	300755.4
10-7	183.8	5148662.8	300675.7

CPT CO-ORDINATES			
No.	ELEVATION	NORTHING	EASTING
CPT 10-6	184.4	5148638.3	300757.4
CPT 10-7	183.8	5148662.8	300676.7
CPT 10-7B/C	183.8	5148663.8	300676.7

NOTES

This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contracts Documents.

The boundaries between soil strata have been established only at borehole locations. Between boreholes the boundaries are assumed from geological evidence.

The complete Foundation Investigation and Design Report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with Section GC 2.01 of OPS General Conditions.

REFERENCE

Base plans provided in digital format by URS, drawing file 09-079 – Base Plan.dwg, received July 15, 2010 and for the General Arrangement drawing file 09-079 Recommended Plan for Geotech-A.dwg, received April 4, 2011.

NO.	DATE	BY	REVISION

Geocres No. 41K-90

HWY. 17		PROJECT NO. 09-1111-0016	DIST.
SUBM'D. MWK	CHKD. JPD	DATE: 7/12/2012	SITE:
DRAWN: JFC	CHKD. MWK	APPD. JMAC	DWG. D2

![Golder Associates]

Foundation Design

RECORD OF BOREHOLE No 10-5

SHEET 1 OF 1 **METRIC**

PROJECT 09-1111-0016
G.W.P. 5022-07-00 LOCATION N 5148463.4 ;E 300792.0 ORIGINATED BY MWK
DIST ___ HWY 17 BOREHOLE TYPE Power Auger, 100 mm I.D. Continous Flight Hollow Stem Augers COMPILED BY MWK
DATUM Geodetic DATE March 9, 2011 CHECKED BY JPD

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
184.4	GROUND SURFACE							
0.0 / 183.9	TOPSOIL, sandy, Dense	1	SS	48		184		
0.5	SAND, some silt, trace clay, Very loose to compact, Brown, Wet	2	SS	10		183		
		3	SS	2			○	0 81 14 5
182.1 / 2.3	Silty SAND, trace to some clay with occasional clayey silt to silt interlayers, Very loose, Brown, Wet	4	SS	1		182		
		5	SS	WH		181	H ○	0 64 24 12
180.6 / 3.8	CLAYEY SILT, some sand, Firm to stiff, Grey, Wet	6	SS	WH	▽	180		
		7	SS	WH		179	├─┤ ○	0 16 59 25
		8	SS	14		178		
		9	SS	16		177 / 176	○	
		10A / 10B	SS	1		175	├┤ ○	
						174	Field vane + (sensitivity 3)	
		11	SS	WH		173	○; Field vane + (sensitivity 5), (sensitivity 4)	
172.8 / 11.6	END OF BOREHOLE							

NOTE:

1. Water level in borehole measured at a depth of 4.3 m below ground surface (Elev. 180.1 m) on completion of drilling.

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED; 20 40 60 80 100
PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60
UNIT WEIGHT γ kN/m^3

$+^3, \times^3$: Numbers refer to Sensitivity ○$^{3\%}$ STRAIN AT FAILURE

Golder Associates

RECORD OF BOREHOLE No 10-6

SHEET 1 OF 1 **METRIC**

PROJECT 09-1111-0016

G.W.P. 5022-07-00 LOCATION N 5148637.3 ;E 300755.4 ORIGINATED BY MWK

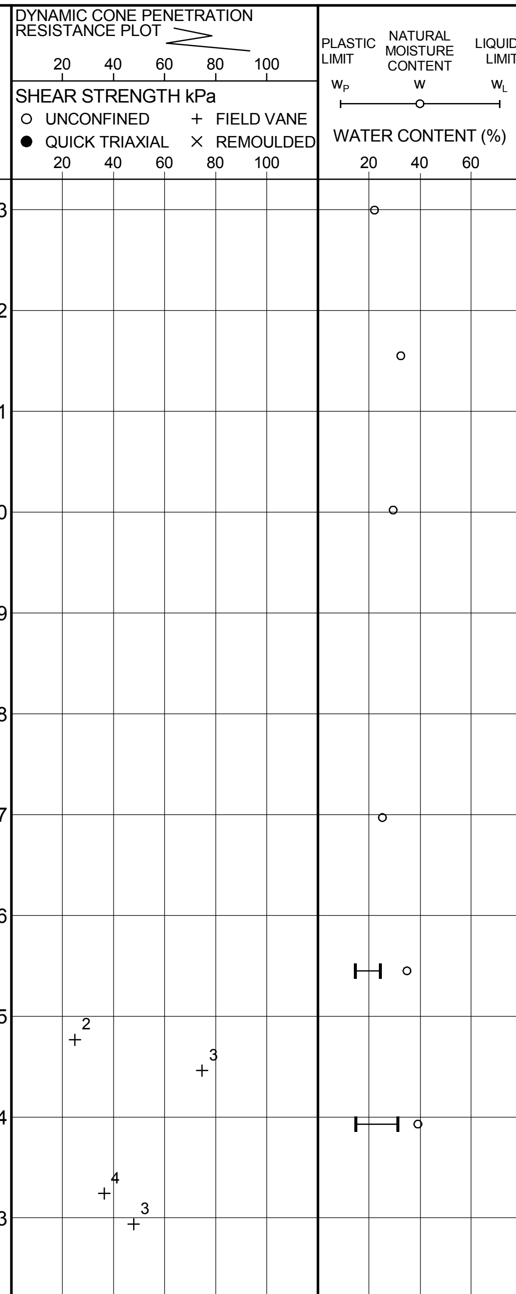
DIST HWY 17 BOREHOLE TYPE Power Auger, 100 mm I.D. Continous Flight Hollow Stem Augers COMPILED BY MWK

DATUM Geodetic DATE March 8, 2011 CHECKED BY JPD

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
183.3	GROUND SURFACE					
0.0 / 182.9	TOPSOIL, sandy Compact	1	SS	26	183	
0.4	Silty sand (FILL) Loose to compact Brown Moist to wet	2	SS	7	182	
181.8 / 1.5	SAND and SILT, some clay Very loose Grey Wet	3	SS	1	181	
		4	SS	1		
		5	SS	WH	180	0 51 36 13 NP
178.7 / 4.6	Sandy SILT, trace clay Compact Grey Wet	6	SS	12	179, 178	
		7	SS	12	177	0 22 75 3
175.7 / 7.6	CLAYEY SILT, trace sand Firm to stiff Brown Wet	8	SS	1	176, 175	
		9	SS	2	174	0 4 54 42
		10	SS	2	173	
172.2 / 11.1	END OF BOREHOLE					

NOTE:

1. Water level in borehole measured at a depth of 3.9m below ground surface (Elev. 179.4 m) on completion of drilling.



+3, ×3: Numbers refer to Sensitivity ○3% STRAIN AT FAILURE

Foundation Design

Golder Associates

RECORD OF BOREHOLE No 10-7

SHEET 1 OF 4 **METRIC**

PROJECT 09-1111-0016

G.W.P. 5022-07-00 LOCATION N 5148662.8 ;E 300675.7 ORIGINATED BY MR

DIST HWY 17 BOREHOLE TYPE 108 mm I.D. Continous Flight Hollow Stem Augers and Tri-Cone, Wash Boring COMPILED BY MWK

DATUM Geodetic DATE January 13, 2011 CHECKED BY JPD

SOIL PROFILE

ELEV DEPTH	DESCRIPTION
183.8	GROUND SURFACE
0.0 / 183.5 / 0.3	TOPSOIL, sandy; Loose; Dark brown; Moist
	Sand, trace to some silt, trace clay (FILL); Loose; Brown; Wet
182.4 / 1.4	Silty Sand, trace to some clay, containing wood fragments (FILL); Very loose; Dark brown; Wet
181.1 / 2.8	Sandy SILT, containing wood fragments; Very Loose; Grey; Wet
180.1 / 3.7	SAND and SILT, trace clay, containing wood fragments; Compact; Grey; Wet
178.2 / 5.6	CLAYEY SILT, trace sand, occasional brown silty clay interlayers; Very soft; Grey; Wet
177.3 / 6.6	SAND, trace to some silt; Very loose; Grey; Wet
176.6 / 7.2	SAND and SILT, trace clay; Very loose; Grey; Moist
175.1 / 8.7	CLAY, some silt, trace sand; Firm to stiff; Brown; Moist

Continued Next Page

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1A / 1B	SS	4	
2	SS	8	0 93 6 1
3A / 3B	SS	2	OC=2.6%
4	SS	1	0 62 31 7
5	SS	1	
6	SS	13	OC=0.9%
7	SS	11	0 33 65 2
8A / 8B	SS	WH	0 4 64 32
9	SS	2	0 57 37 6
10	SS	WH	
11	SS	WR	
12	SS	WH	0 1 12 87
13	SS	WH	

GROUND WATER CONDITIONS ∇ (approx. elev. 182.4)

ELEVATION SCALE: 183, 182, 181, 180, 179, 178, 177, 176, 175, 174, 173, 172, 171, 170, 169

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED — 20 40 60 80 100

Field vane sensitivities: 4, 3, 4, 3, 3, 3, 3, 3

PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L — WATER CONTENT (%) 20 40 60

UNIT WEIGHT γ kN/m^3

+^{3}, ×^{3}: Numbers refer to Sensitivity ○$^{3\%}$ STRAIN AT FAILURE

GTA-MTO 001 09-1111-0016.GPJ GAL-MISS.GDT 7/16/12 JFC

Foundation Design

Golder Associates

RECORD OF BOREHOLE No 10-7

SHEET 2 OF 4 **METRIC**

PROJECT 09-1111-0016
G.W.P. 5022-07-00
LOCATION N 5148662.8 ;E 300675.7
ORIGINATED BY MR
DIST HWY 17
BOREHOLE TYPE 108 mm I.D. Continous Flight Hollow Stem Augers and Tri-Cone, Wash Boring
COMPILED BY MWK
DATUM Geodetic
DATE January 13, 2011
CHECKED BY JPD

--- CONTINUED FROM PREVIOUS PAGE ---

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
	CLAY, some silt, trace sand; Firm to stiff; Brown; Moist
163.7 / 20.1	CLAYEY SILT, containing occasional grey silt seams; Stiff; Brown; Moist
161.3 / 22.5	SAND, some silt, trace clay; Very loose; Grey; wet
157.9 / 25.9	CLAY, some silt, trace sand; Stiff to very stiff; Brown to grey; Moist

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
14	SS	WR	
15	SS	1	
16	SS	WR	
17	SS	WR	0 80 19 1
18	SS	7	

ELEVATION SCALE: 168, 167, 166, 165, 164, 163, 162, 161, 160, 159, 158, 157, 156, 155, 154

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED: 20 40 60 80 100

PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L

WATER CONTENT (%): 20 40 60

UNIT WEIGHT γ kN/m³

Field vane sensitivity values plotted: 3, 3 (near el. 167.5); 3, 3 (near el. 164.5); 2 (near el. 161.5); 3, 3 (near el. 155.5)

Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

GTA-MTO 001 09-1111-0016.GPJ GAL-MISS.GDT 7/16/12 JFC

RECORD OF BOREHOLE No 10-7

SHEET 3 OF 4 **METRIC**

PROJECT 09-1111-0016			
G.W.P. 5022-07-00	LOCATION N 5148662.8 ;E 300675.7		ORIGINATED BY MR
DIST	HWY 17	BOREHOLE TYPE 108 mm I.D. Continous Flight Hollow Stem Augers and Tri-Cone, Wash Boring	COMPILED BY MWK
DATUM Geodetic	DATE January 13, 2011		CHECKED BY JPD

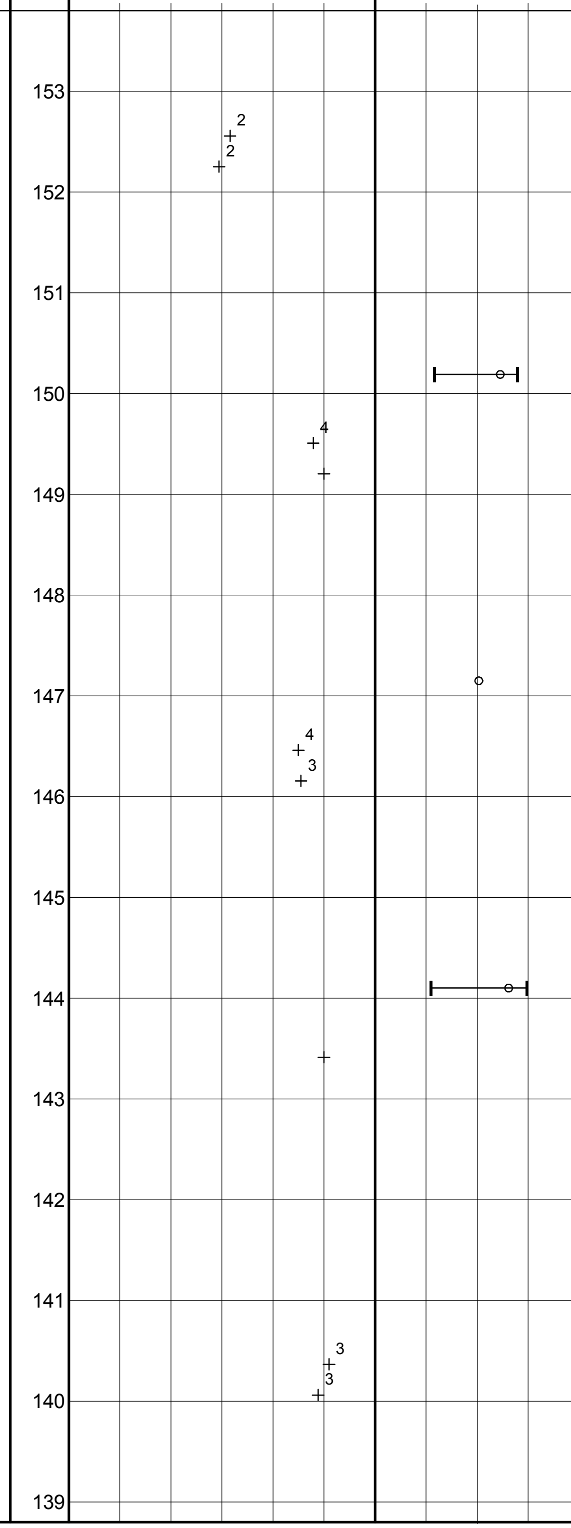
SOIL PROFILE — ELEV DEPTH / DESCRIPTION / STRAT PLOT; SAMPLES — NUMBER / TYPE / "N" VALUES; GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED; 20 40 60 80 100); PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

--- CONTINUED FROM PREVIOUS PAGE ---

CLAY, some silt, trace sand
Stiff to very stiff
Brown to grey
Moist

Containing grey silt interlayers
below a depth of 38.1 m

NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
19	SS	7				
20	SS	7				
21	SS	WR	0	2	42	56
22	SS	WH				
23	SS	10				



Continued Next Page

+ ³, × ³: Numbers refer to Sensitivity   ○ ³% STRAIN AT FAILURE

Golder Associates

RECORD OF BOREHOLE No 10-7

SHEET 4 OF 4 **METRIC**

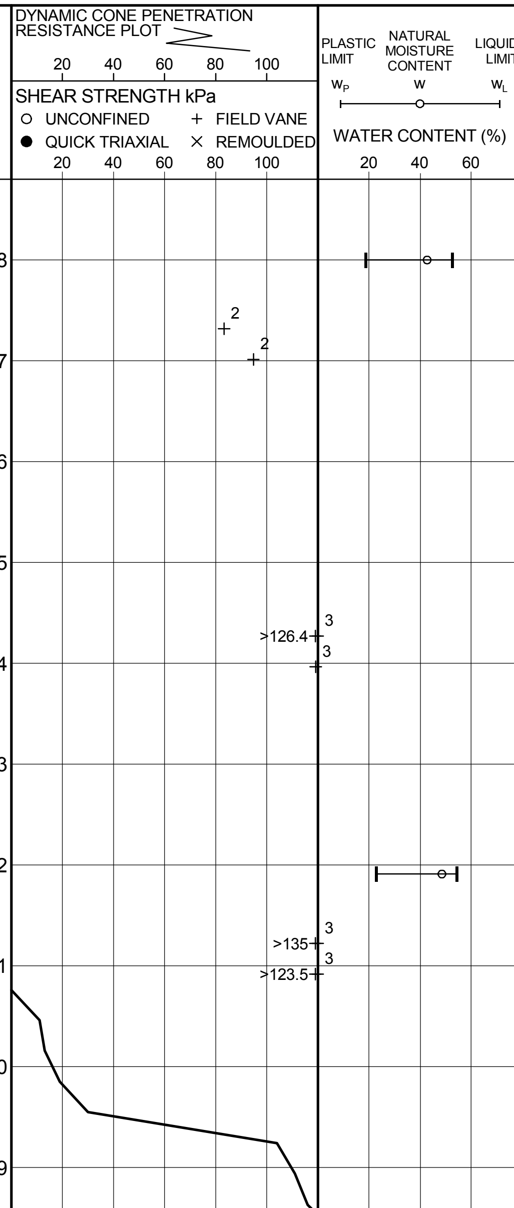
PROJECT 09-1111-0016

G.W.P. 5022-07-00 LOCATION N 5148662.8 ;E 300675.7 ORIGINATED BY MR

DIST HWY 17 BOREHOLE TYPE 108 mm I.D. Continous Flight Hollow Stem Augers and Tri-Cone, Wash Boring COMPILED BY MWK

DATUM Geodetic DATE January 13, 2011 CHECKED BY JPD

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	--- CONTINUED FROM PREVIOUS PAGE ---								
	CLAY, some silt, trace sand Stiff to very stiff Brown to grey Moist								
			24	SS	9		138		
							137		
							136		
			25	SS	10		135		
							134		
							133		
			26	SS	11		132		
							131		
130.8 53.0	END OF BOREHOLE Start of Dynamic Cone Penetration Test (DCPT)						130		
							129		
128.5 55.3	END OF DCPT Refusal to further penetration (125 blows /.10 m) NOTE: 1. Water level in borehole measured at a depth of 1.6 m below ground surface (Elev. 182.2 m) on completion of drilling.								



$+^3, \times^3$: Numbers refer to Sensitivity O $^{3\%}$ STRAIN AT FAILURE

RECORD OF BOREHOLE No 10-8

SHEET 1 OF 1 **METRIC**

PROJECT 09-1111-0016
G.W.P. 5022-07-00 LOCATION N 5148810.1 ;E 300643.5 ORIGINATED BY MWK
DIST HWY 17 BOREHOLE TYPE Power Auger, 100 mm I.D. Continous Flight Hollow Stem Augers COMPILED BY MWK
DATUM Geodetic DATE March 9, 2011 CHECKED BY JPD

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	SHEAR STRENGTH kPa (field vane, sensitivity)	WATER CONTENT (%)	GR	SA	SI	CL
183.3	GROUND SURFACE										
183.0 / 0.0	TOPSOIL	1	SS	5							
0.3	SAND, some silt, containing organics; Very loos to loose; Brown; Wet	2	SS	3							
181.8 / 1.5	SILT, some sand, some clay; Very loose; Grey; Wet	3	SS	1							
181.0 / 2.3	SAND, some silt, trace to some clay; Very loose; Grey; Wet	4	SS	1				0	77	15	8
		5	SS	2				0	76	16	8
179.5 / 3.8	SAND and SILT, trace clay; Very loose to compact; Grey; Wet	6	SS	10	∇ (El. 178.7)						
		7	SS	13				0	31	65	4
		8	SS	3							
175.7 / 7.6	CLAYEY SILT, occasional sandy silt interlayers; Firm; Grey; Wet	9	SS	1		+ 3; + 2					
		10	SS	6							
173.7 / 9.6	CLAY; Stiff; Grey; Wet					+ 6; + 4					
		11	SS	1							
172.2 / 11.1	END OF BOREHOLE										

NOTE:

1. Water level in borehole measured at a depth of 4.6 m below ground surface (Elev. 178.7 m) on completion of drilling.

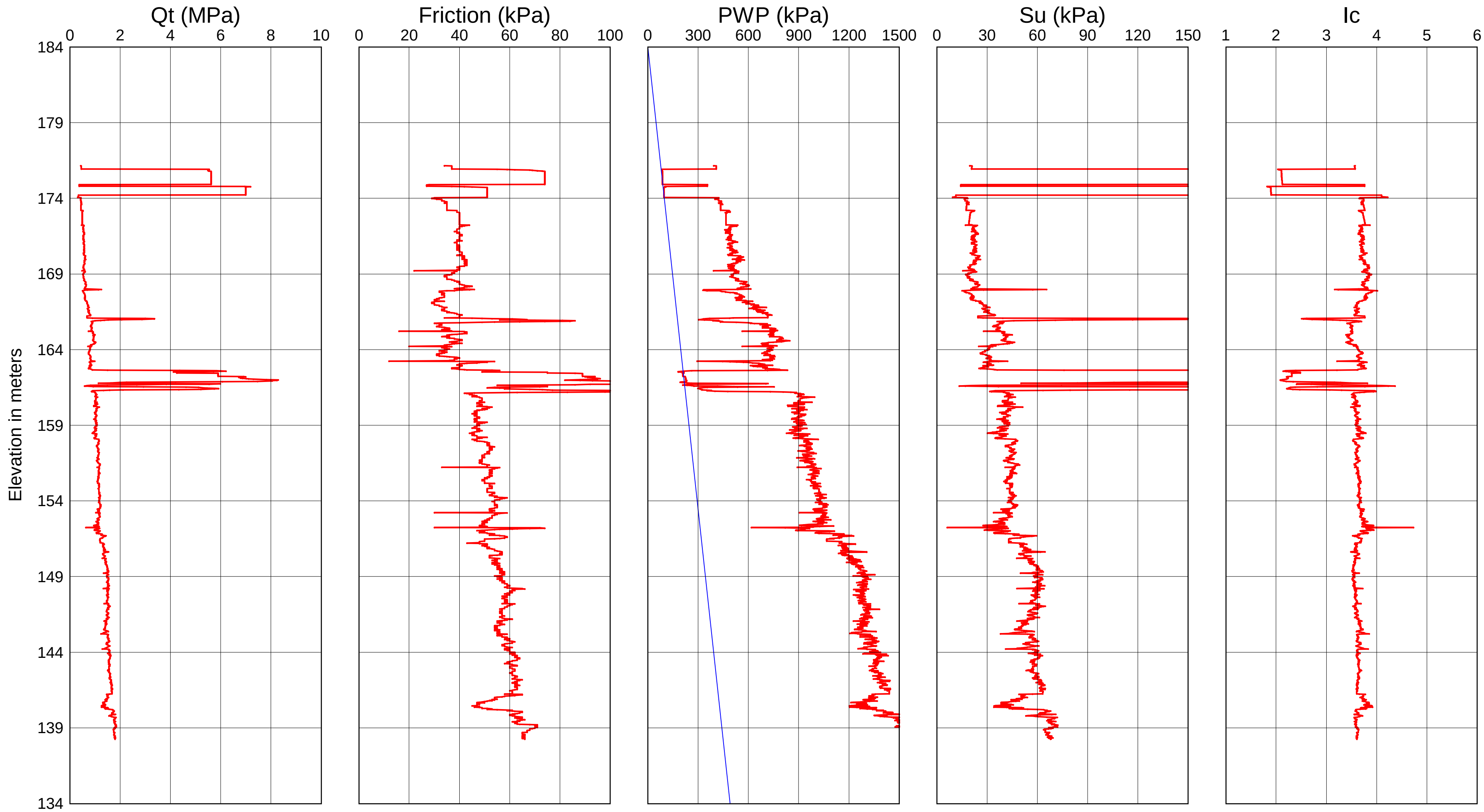
+ 3, × 3: Numbers refer to Sensitivity   ○ 3% STRAIN AT FAILURE

Cone Penetration Test - CPT 10-6

Test Date : 3/8/11
Location : N5148638.3 E300757.4

Operator : Golder Associates

Ground Surf. Elev. : 184.40
Water Table Depth : 0.00



Qt normalized for
unequal end area effects

Su = (Qt - SigmaV) / Nk
Nk = 15
Gamma = 17 kN/m³

After Robertson and (Fear) Wride (1998)
Ic < 1.31 - Gravelly sands
1.31 <Ic < 2.05 - Clean to silty sand
2.05 <Ic < 2.60 - Silty sand to sandy silt
2.60 <Ic < 2.95 - Clayey silt to silty clay
2.95 <Ic < 3.60 - Clays

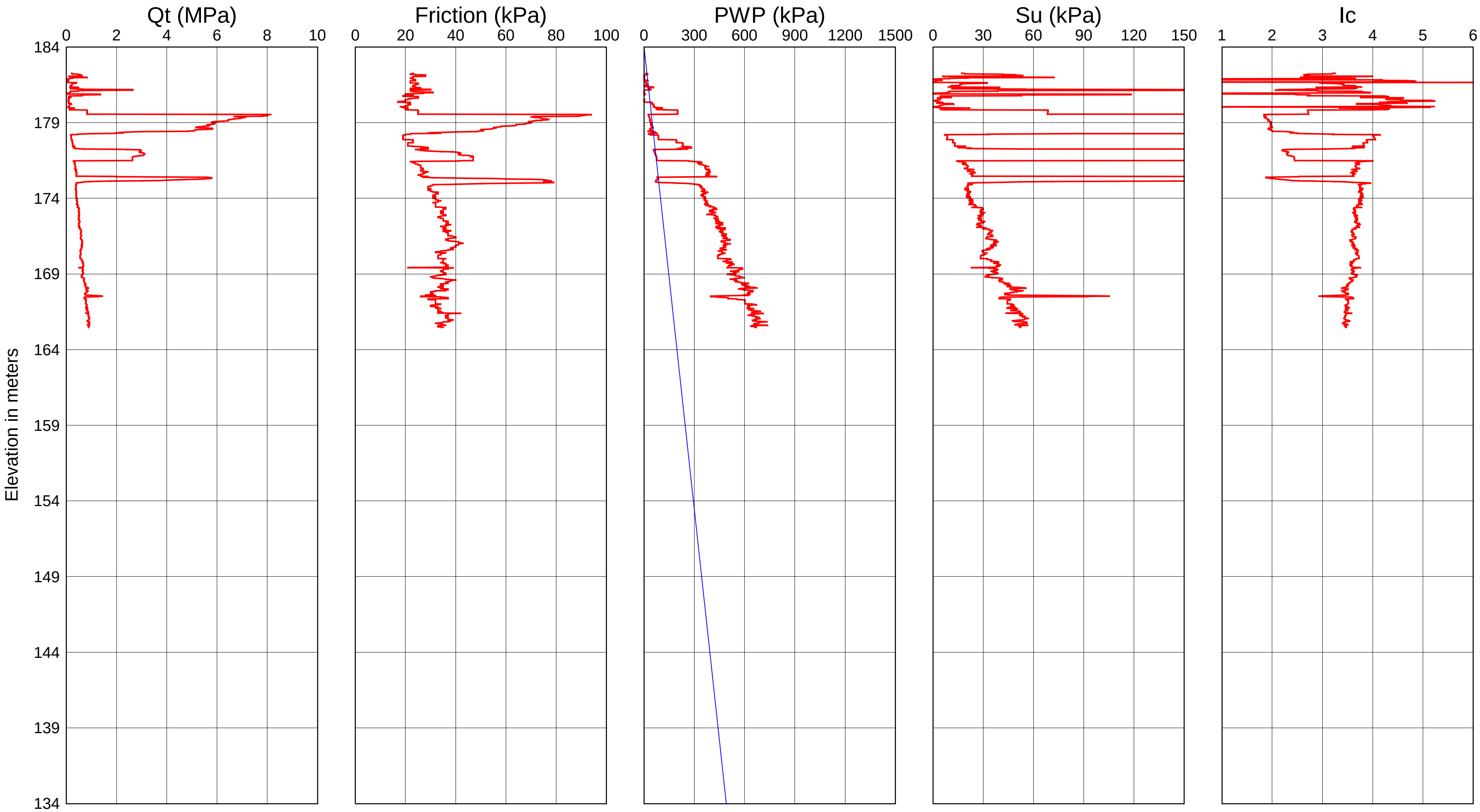
PROJECT NO. 09-1111-0016 DATE: 6/1/2011 DRAWN BY: MWK

Cone Penetration Test - CPT 10-7

Test Date : 3/6/11
Location : N5148662.8 E300676.7

Operator : Golder Associates

Ground Surf. Elev. : 183.80
Water Table Depth : 0.00



Qt normalized for
unequal end area effects

Su = (Qt - SigmaV) / Nk
Nk = 11
Gamma = 17 kN/m³

After Robertson and (Fear) Wride (1998)
Ic < 1.31 - Gravelly sands
1.31 <Ic < 2.05 - Clean to silty sand
2.05 <Ic < 2.60 - Silty sand to sandy silt
2.60 <Ic < 2.95 - Clayey silt to silty clay
2.95 <Ic < 3.60 - Clays

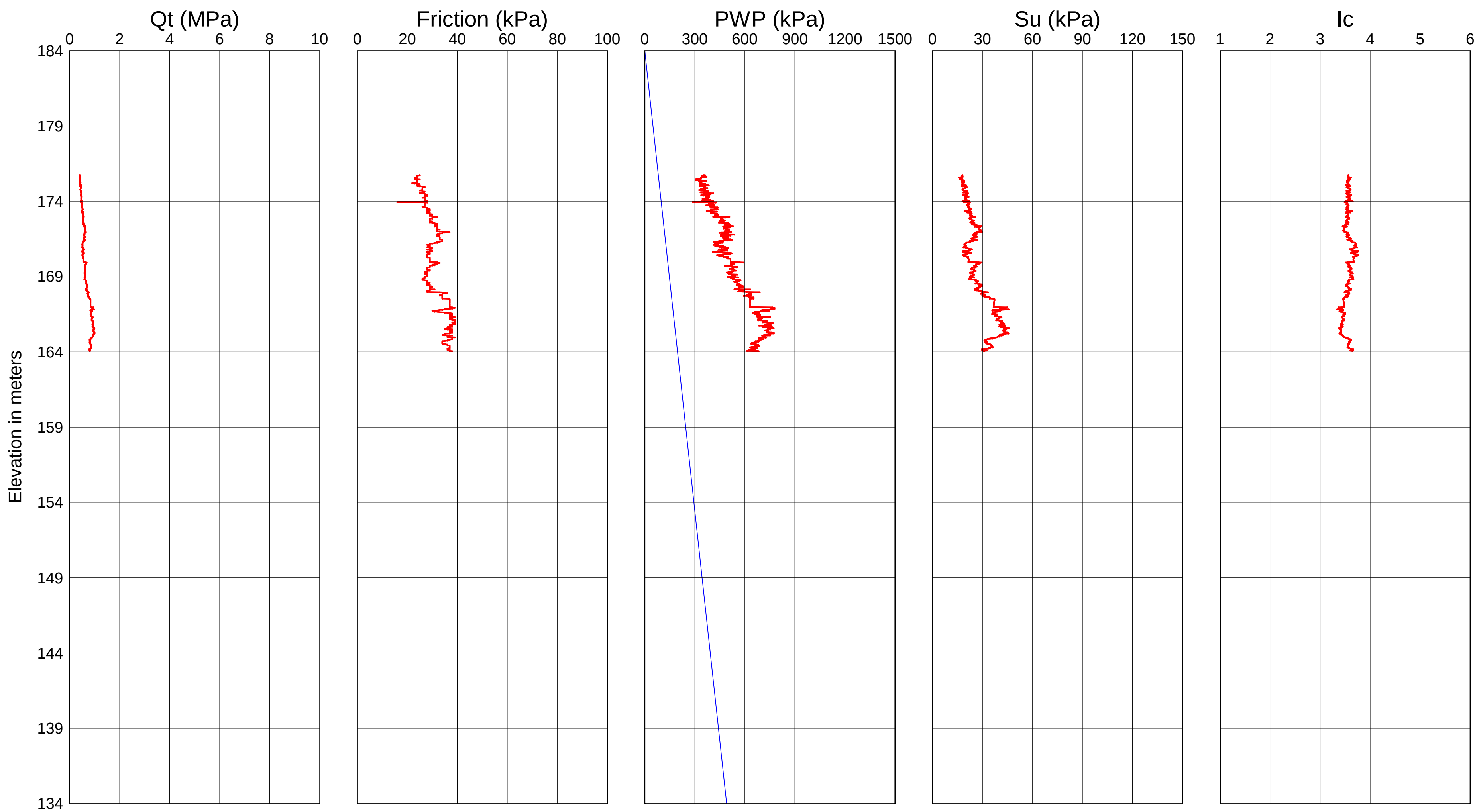
PROJECT NO. 09-1111-0016 DATE: 6/1/2011 DRAWN BY: MWK

Cone Penetration Test - CPT 10-7B

Test Date : 3/6/11
Location : N5148663.8 E300676.7

Operator : MWK

Ground Surf. Elev. : 183.80
Water Table Depth : 0.00



Qt normalized for
unequal end area effects

Su = (Qt - SigmaV) / Nk
Nk = 15
Gamma = 17 kN/m³

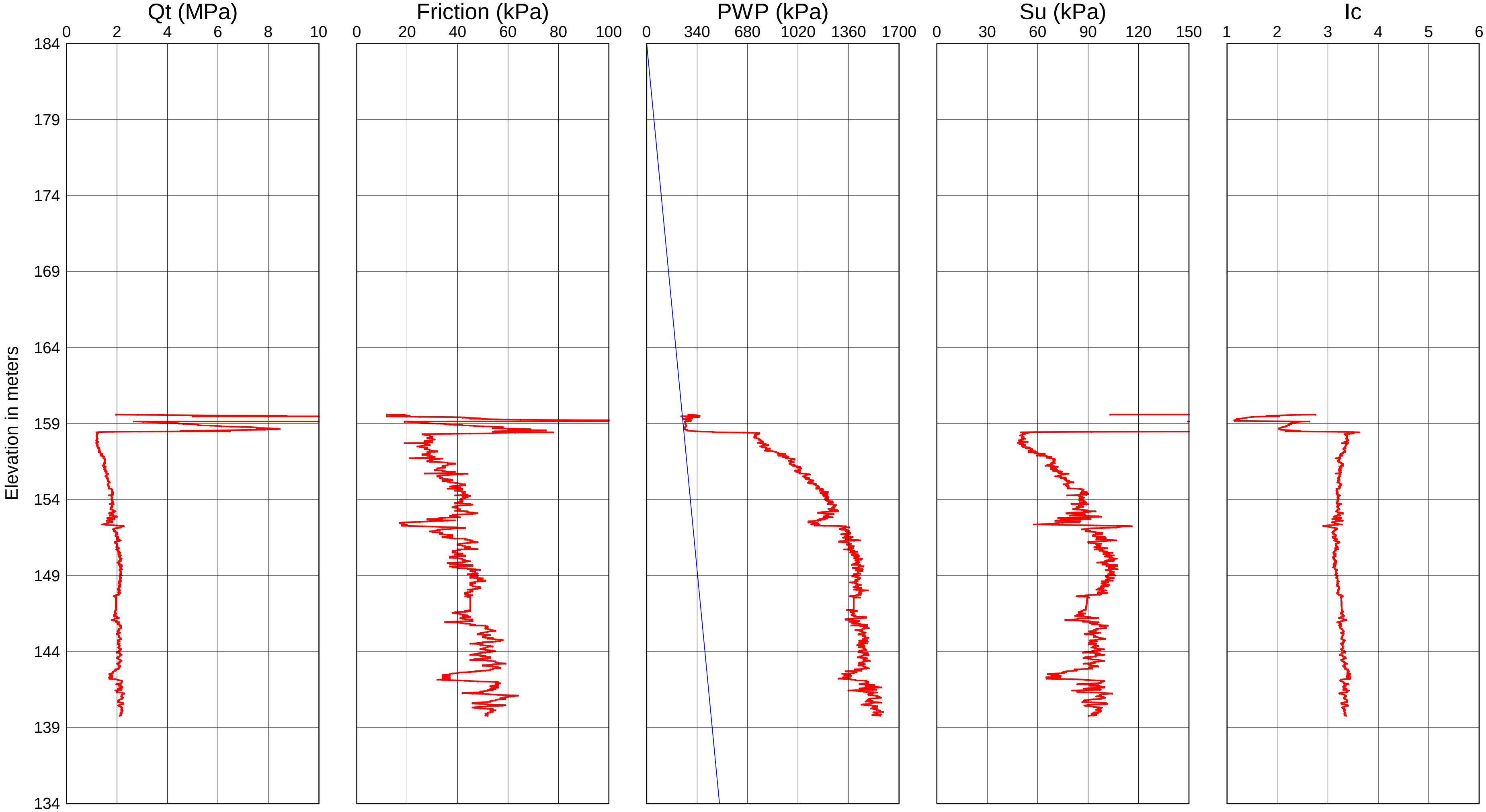
After Robertson and (Fear) Wride (1998)
Ic < 1.31 - Gravelly sands
1.31 <Ic < 2.05 - Clean to silty sand
2.05 <Ic < 2.60 - Silty sand to sandy silt
2.60 <Ic < 2.95 - Clayey silt to silty clay
2.95 <Ic < 3.60 - Clays

Cone Penetration Test - CPT 10-7C

Test Date : 3/6/11
Location : N5148663.8 E300676.7

Operator : MWK

Ground Surf. Elev. : 183.80
Water Table Depth : 0.00



Qt normalized for
unequal end area effects

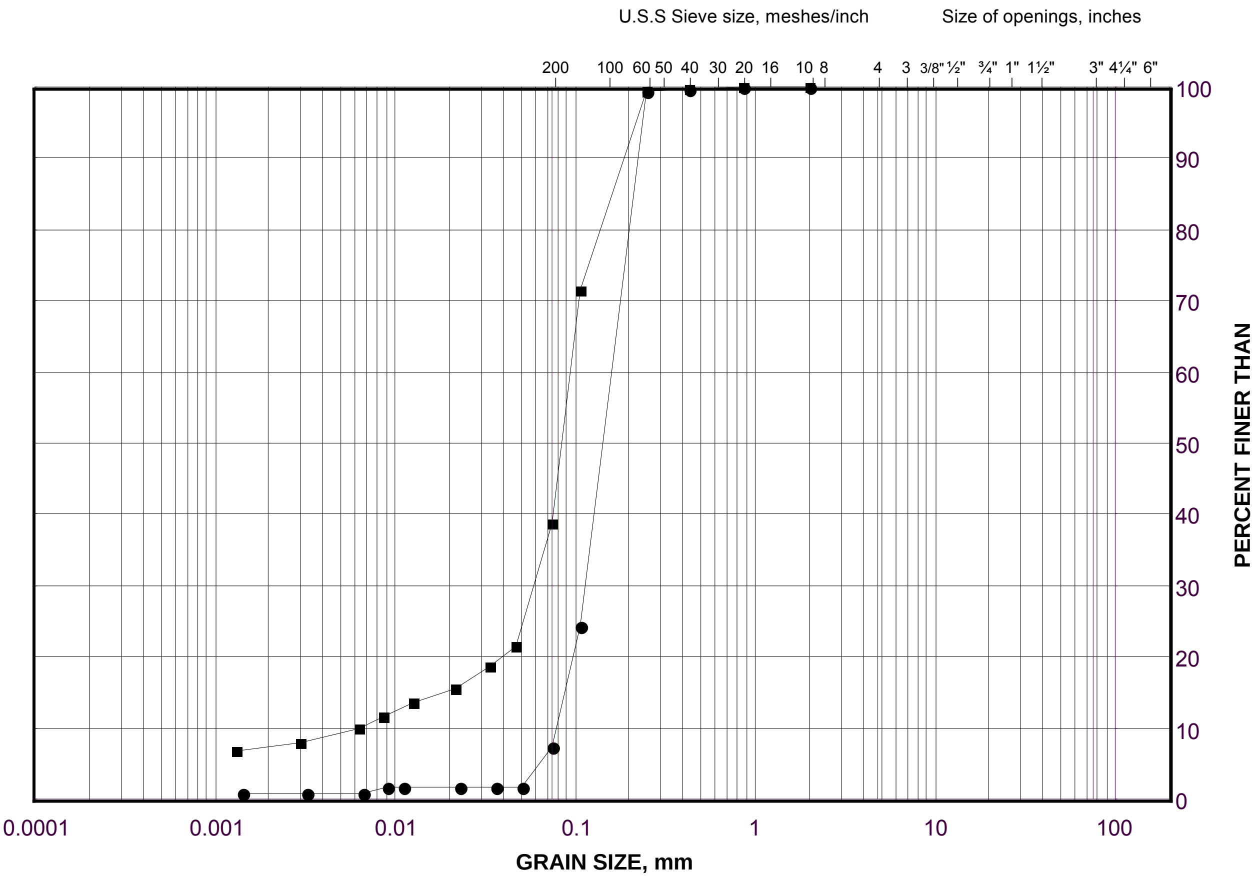
Su = (Qt - SigmaV) / Nk
Nk = 15
Gamma = 17 kN/m³

After Robertson and (Fear) Wride (1998)
Ic < 1.31 - Gravelly sands
1.31 <Ic < 2.05 - Clean to silty sand
2.05 <Ic < 2.60 - Silty sand to sandy silt
2.60 <Ic < 2.95 - Clayey silt to silty clay
2.95 <Ic < 3.60 - Clays

GRAIN SIZE DISTRIBUTION

Sand to Silty Sand Fill

FIGURE D.IC.1



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	10-7	2	182.6
■	10-7	4	181.2

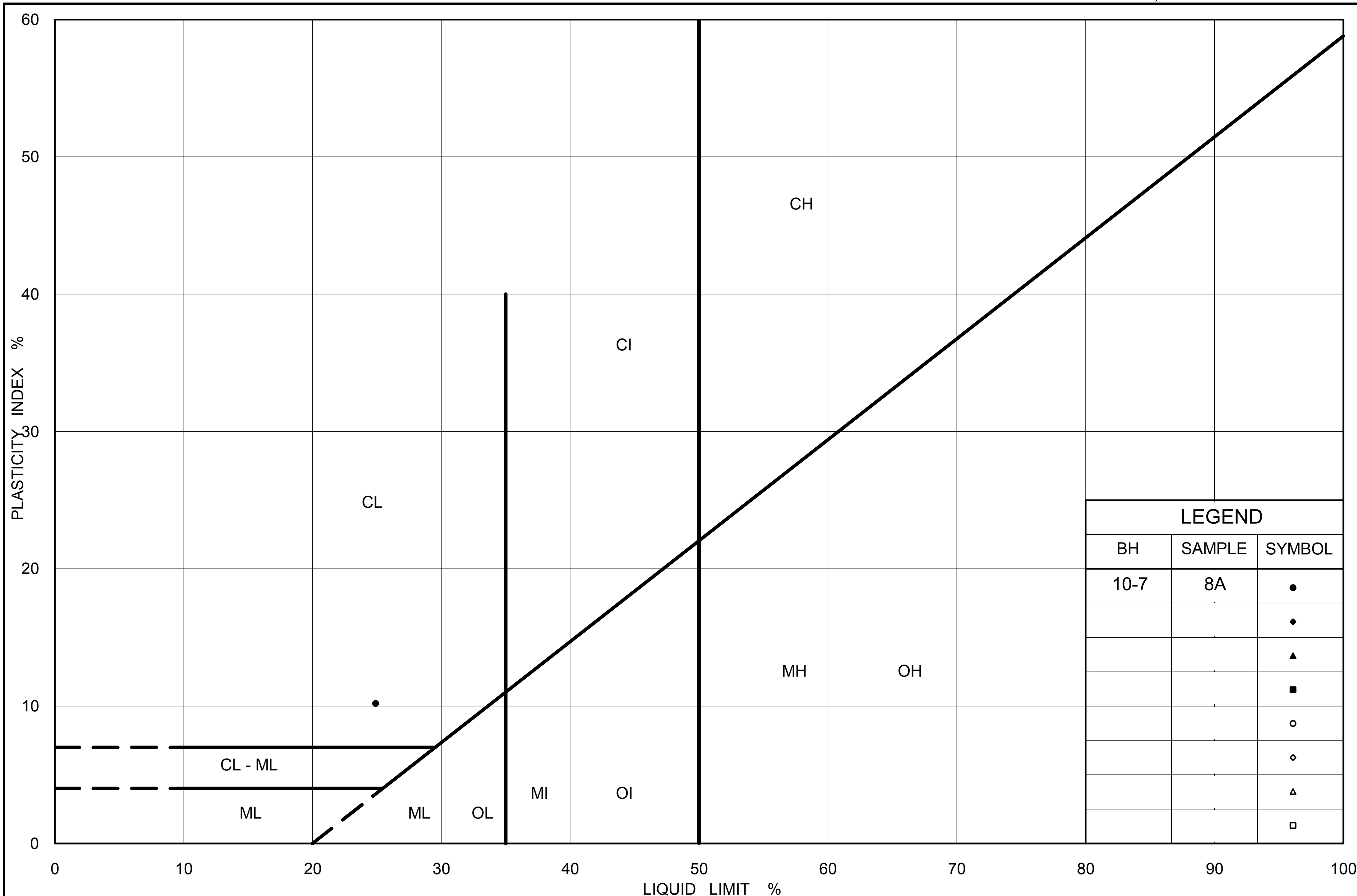
Project Number: 09-1111-0016

Checked By: ____________________

Golder Associates

Date: 01-Jun-11

Oct 75, FF-S-21



Ministry of Transportation

Ontario

PLASTICITY CHART
Clayey Silt

Figure No. D.IC.2

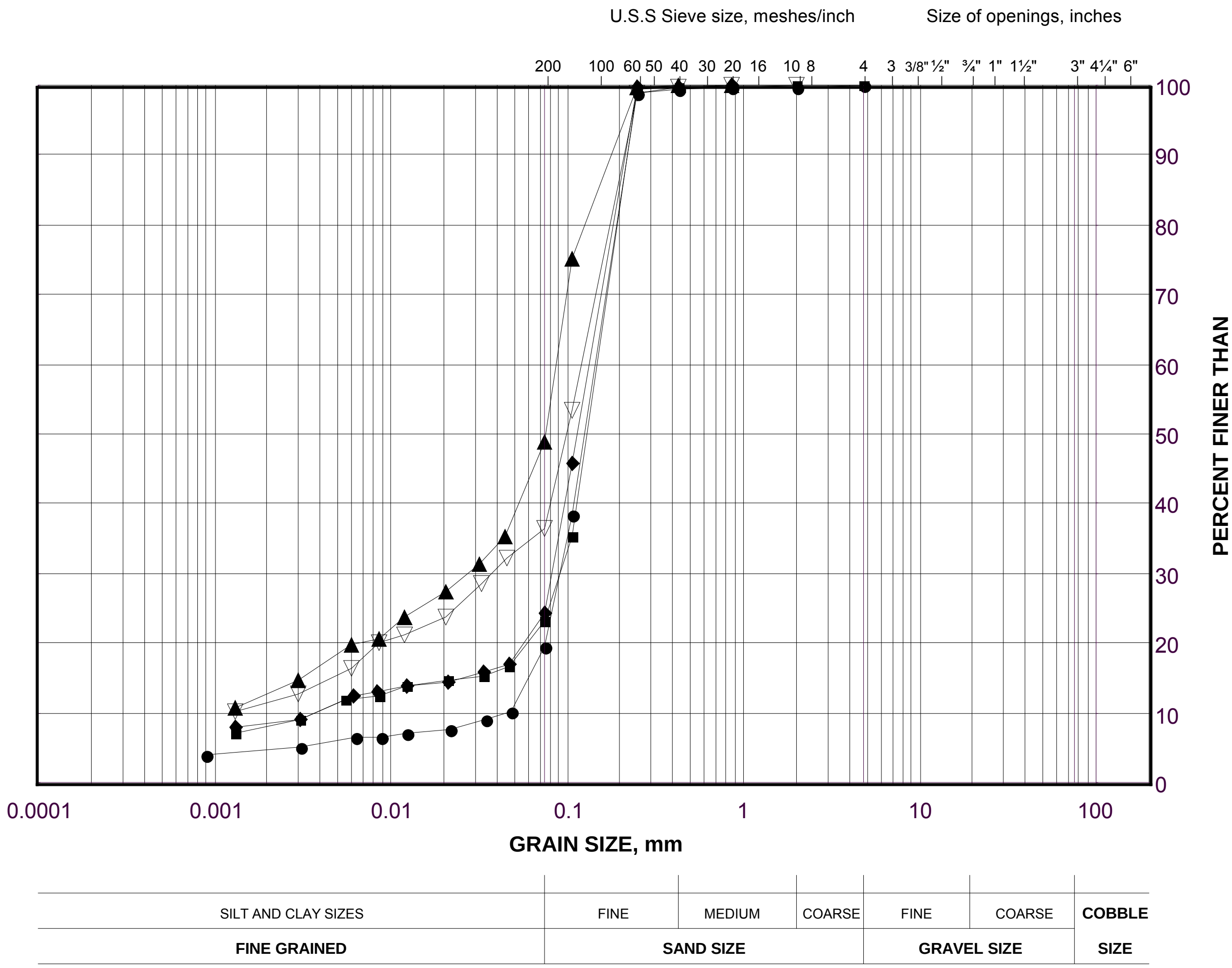
Project No. 09-1111-0016

Checked By:

GRAIN SIZE DISTRIBUTION

Sand to Silty Sand to Sand and Silt

FIGURE D.IC.3A



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	10-5	3	182.7
■	10-8	4	180.8
◆	10-8	5	180.1
▲	10-6	5	180.0
▽	10-5	5	181.2

Project Number: 09-1111-0016

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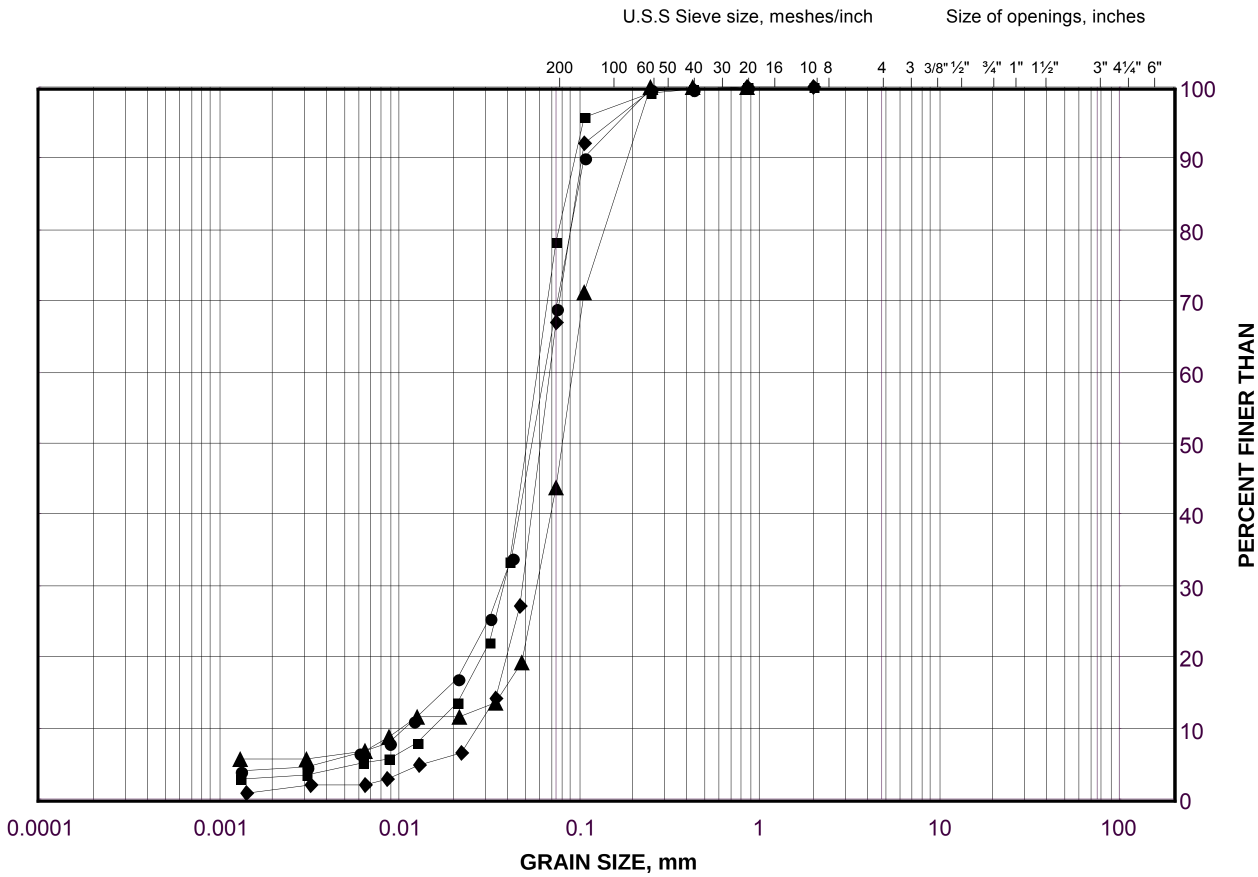
Date: 01-Sep-11

GRAIN SIZE DISTRIBUTION

Sand and Silt to Sandy Silt

FIGURE D.IC.3B



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	10-8	7	178.5
■	10-6	7	176.9
◆	10-7	7	178.9
▲	10-7	9	175.9

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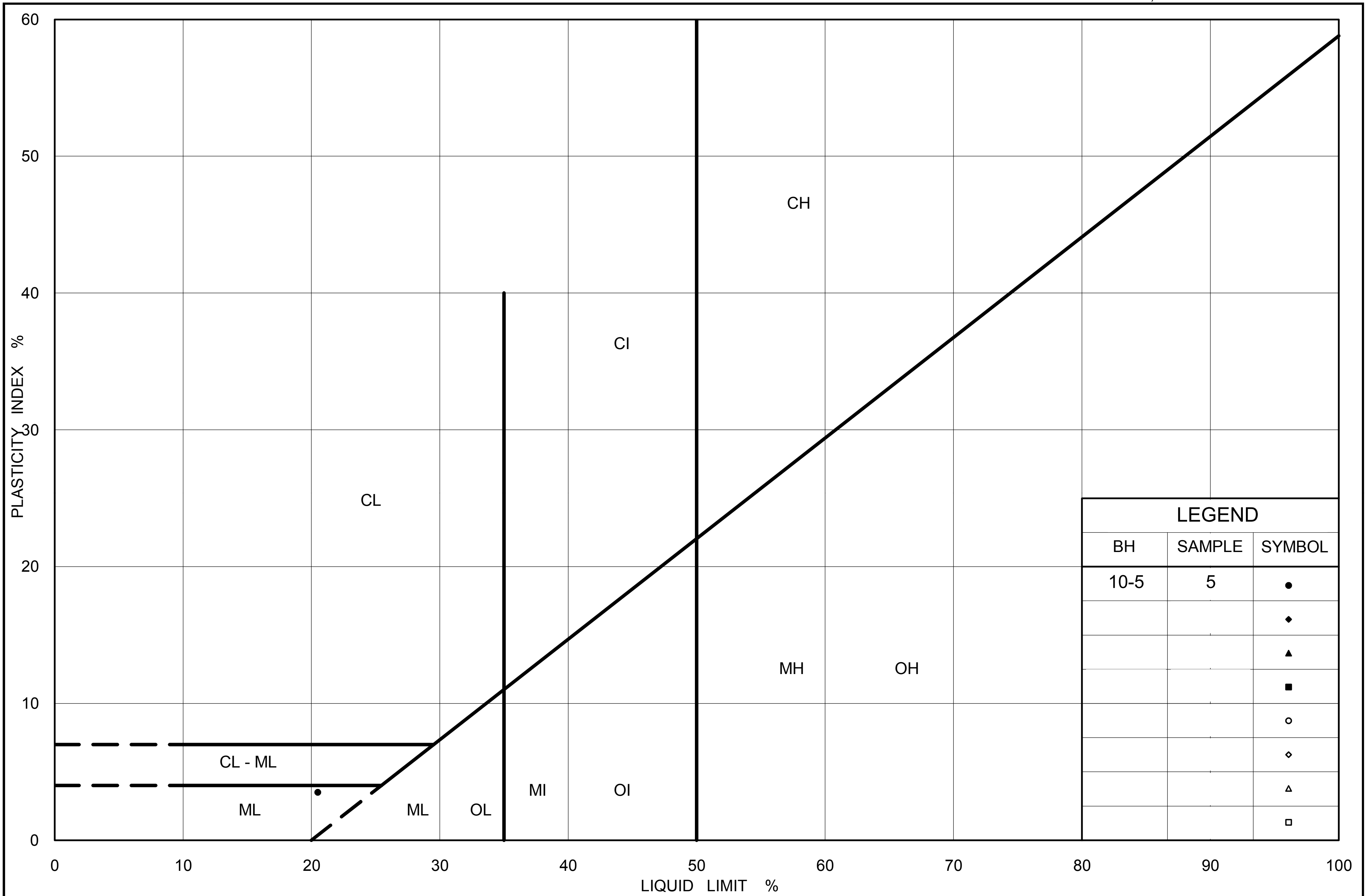
Date: 01-Sep-11

Oct 75, FF-S-21



Ministry of Transportation

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PLASTICITY CHART
Silt (Slight Plasticity)

Figure No. D.IC.4

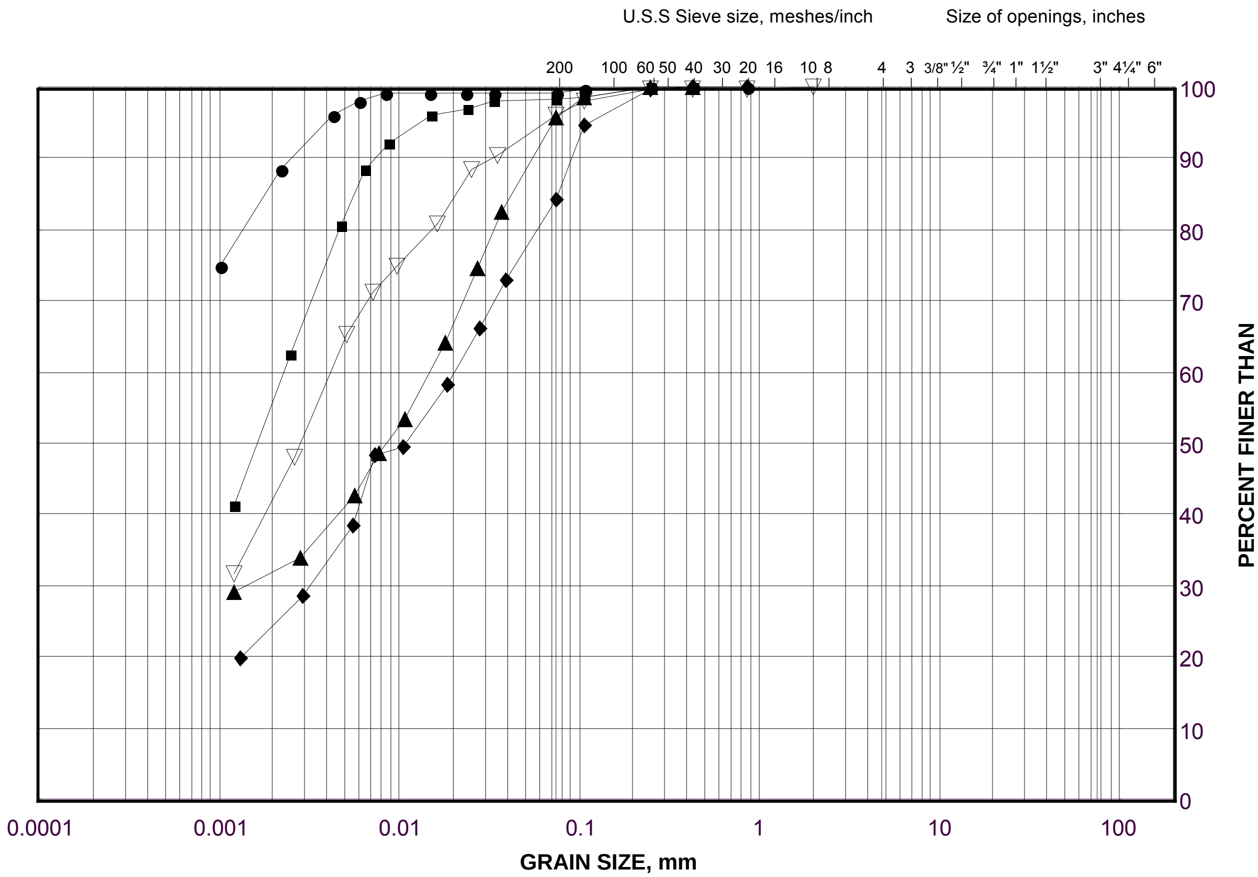
Project No. 09-1111-0016

Checked By:

GRAIN SIZE DISTRIBUTION

Clay to Clayey Silt

FIGURE D.IC.5



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	10-7	12	171.4
■	10-7	21	147.1
◆	10-5	7	178.5
▲	10-7	8A	177.4
▽	10-6	9	175.1

Project Number: 09-1111-0016

Checked By: ______________________

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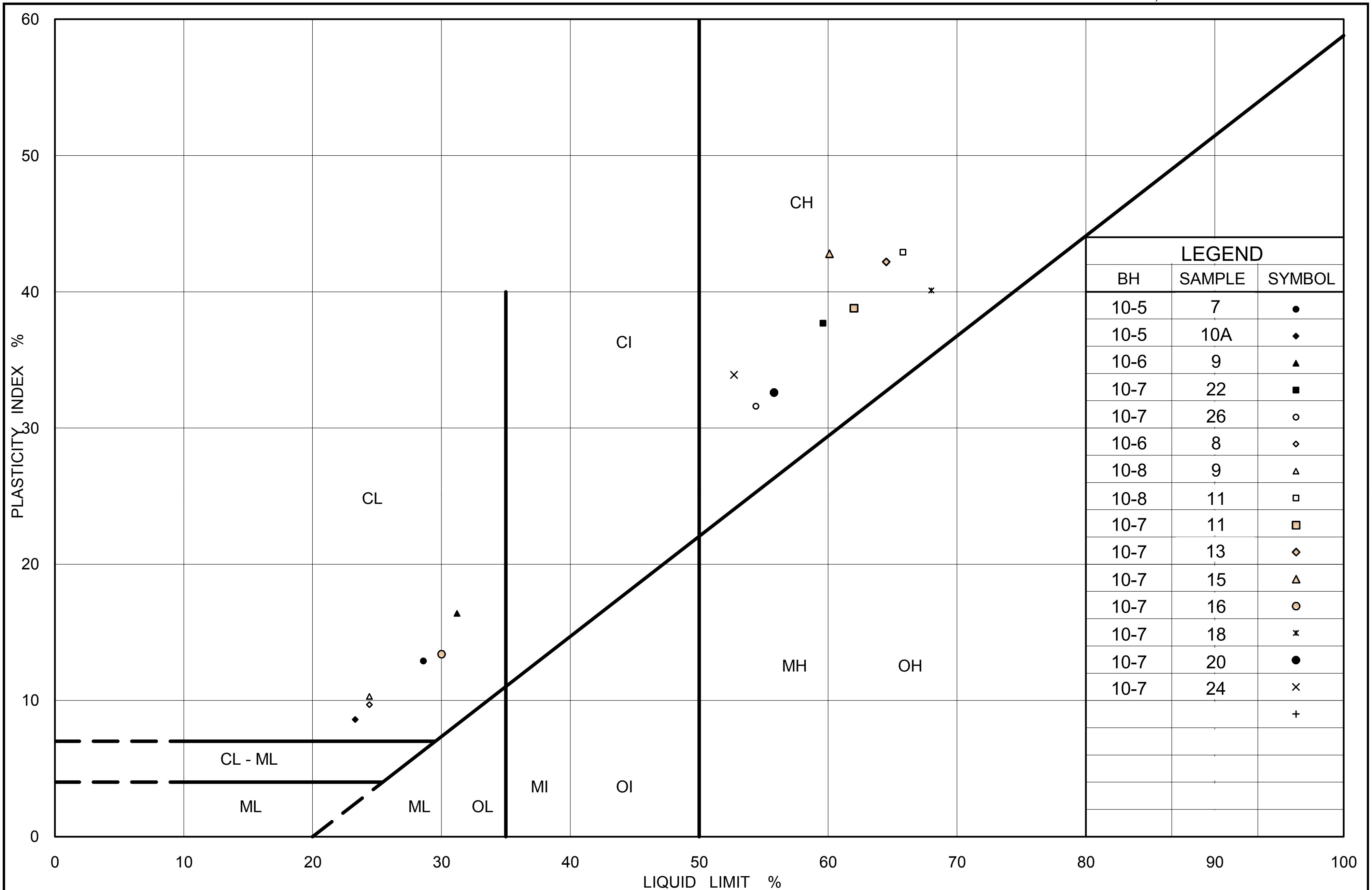
Date: 01-Sep-11

Oct 75, FF-S-21



Ministry of Transportation

Ontario

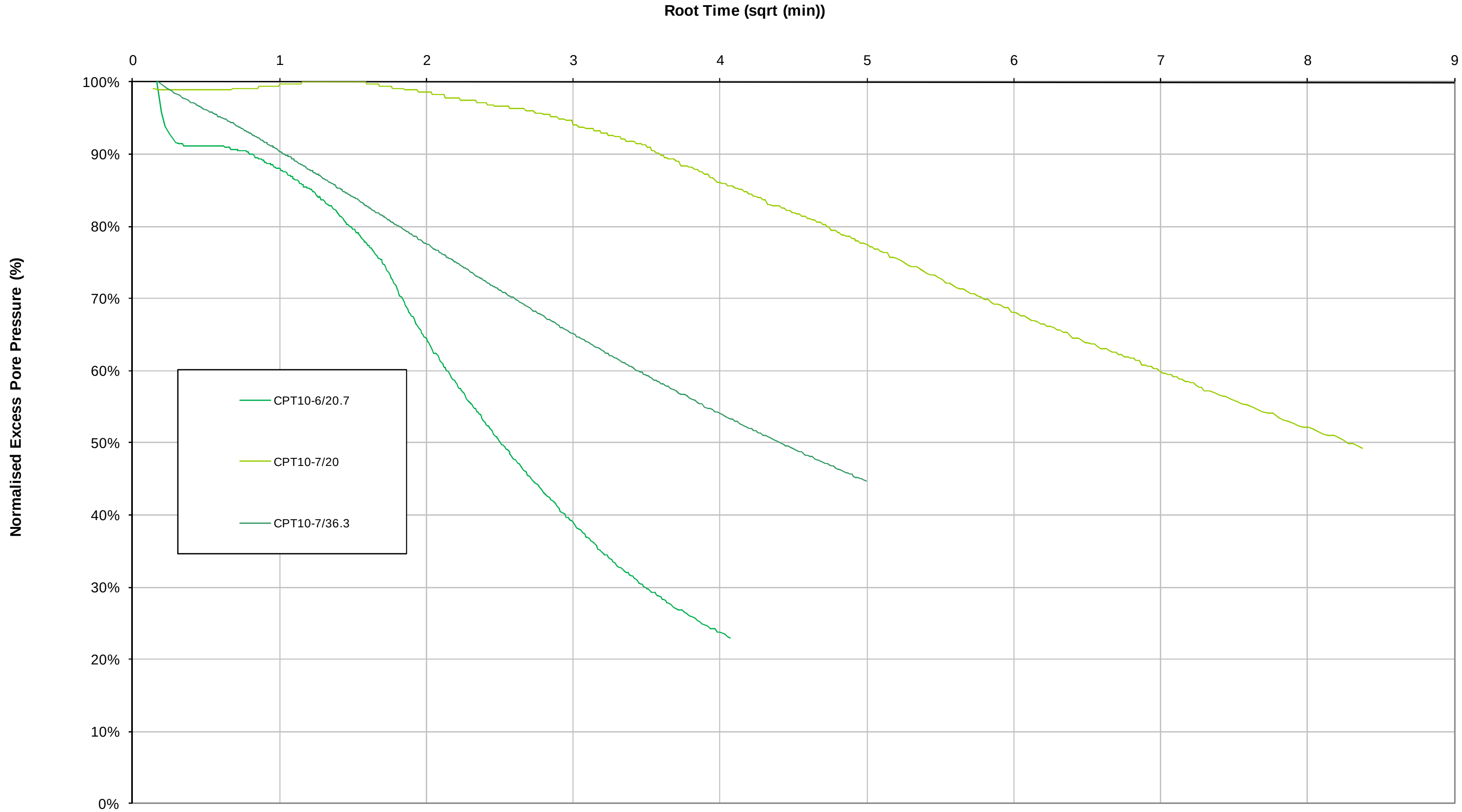
PLASTICITY CHART
Clayey Silt to Clay

Figure No. D.IC.6

Project No. 09-1111-0016

Checked By:

CPT Pore Water Pressure Dissipation Tests
Highway 638 Interchange

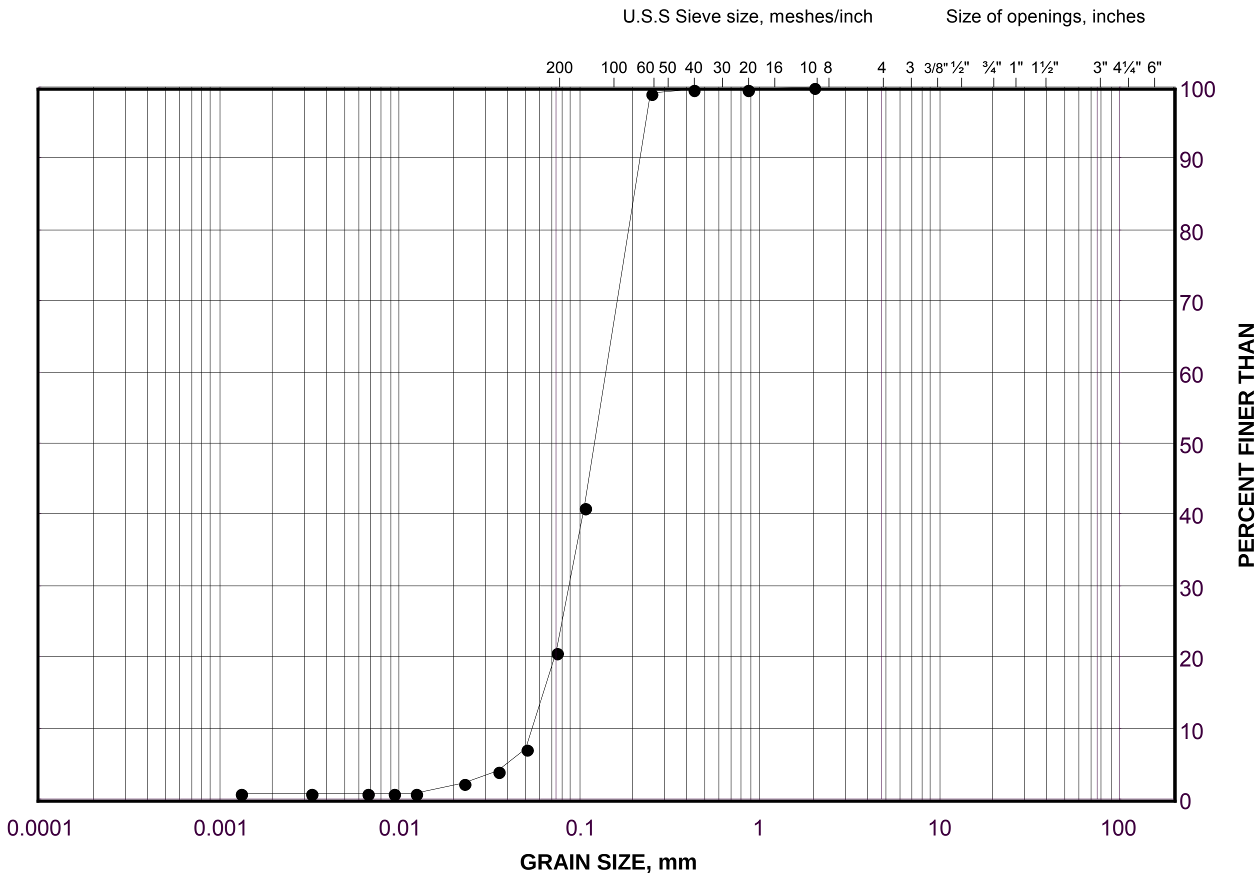


Date: September 2011
Project No: 09-1111-0016

Analysis By: MWK **Reviewed By:** JPD

GRAIN SIZE DISTRIBUTION
Sand

FIGURE D.IC.8



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	10-7	17	159.2

Project Number: 09-1111-0016

Checked By: ______________________

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Date: 15-Jun-11

TABLE D1 – SUMMARY OF FOUNDATION ENGINEERING PARAMETERS - HIGHWAY 638 INTERCHANGE

Stratigraphic Unit	*Average Top Elevation (m)**	*Thickness** (m)*	γ' (kN/m^3)	ϕ' (°)	c' (kPa)	s_u (kPa)	σ_p' (kPa)	e_o	C_c	C_r	E' (MPa)	$C_{\alpha(\varepsilon)}$ (%)		c_h (cm^2/s)
												N/C	*O/C*	
Sand to Sand and Silt	183.6	8.2	19	28	0	--	--	--	--	--	30	--	--	--
Clayey Silt to Clay	175.4	1.4	17	21	0	22	100	1.4	0.8	0.08	--	0.5	0.05	3.68×10^{-3}
Clayey Silt to Clay	174	12.7	17	21	0	22 - 45	100 - 200	1.4	0.8	0.08	--	0.5	0.05	3.68×10^{-3}
Sand	161.3	3.4	20	30	0	--	--	--	--	--	30	--	--	--
Clay	157.9	27.1	17	21	0	50 - 100	230 - 454	1.4	0.8	0.08	--	0.5	0.05	3.68×10^{-3}

*Average Elevation of top of stratigraphic unit at borehole and CPT locations (refer to Drawings C1 and C2)

**Average Thickness of stratigraphic unit at borehole and CPT locations (refer to Drawings C1 and C2)

Prepared By: MWK

Reviewed By: JPD/JMAC

TABLE D2 – EVALUATION OF BRIDGE STRUCTURE FOUNDATION ALTERNATIVES - HIGHWAY 638 INTERCHANGE

Foundation Option	*Rank*	*Advantages*	*Disadvantages*	*Relative Costs*	*Risks / Consequences*
Spread Footings on Overburden	Not feasible	• Relative ease of construction.	• Groundwater control required for excavation and during footing construction. • Large post-construction settlements. • Low geotechnical resistance at SLS of native soils and hence very large footings required.	• Lower relative cost than piled foundations.	• Footing size required to accommodate low geotechnical resistance is not practical. • Very large post-construction settlements could not be tolerated by bridge structure.
Piles driven to bedrock or refusal (55 m to 60 m long piles)	1	• Limited sub-excavation required for pile cap construction. • Negligible post-construction settlement. • Higher axial resistance than for friction piles. • Fewer piles required than for friction piles option.	• Significant depth to refusal and/or bedrock will required very long piles which could result in installation difficulties.	• Higher cost associated with greater pile lengths. • Higher cost associated with provisions for re-driving piles for damaged piles or piles driven out of alignment.	• Damaged piles and piles driven out of alignment may require removal and replacement with new piles. • The abutment/pier design should be flexible enough to allow for installation of extra piles in the footing area, if deemed necessary during construction.
Friction Piles (40 m to 45 m long piles)	2	• Limited sub-excavation required for pile cap construction. • Minor post-construction settlement. • Shorter piles required than for piles driven to refusal option.	• Lower pile capacity than piles driven to refusal.	• Lower cost associated with shorter pile lengths. • Higher cost associated with additional piles due to lower axial capacity. • Additional cost for pile load tests.	• Lower pile capacity will require more piles at each foundation unit. • May require pile load tests to verify pile capacity.

Prepared By: MWK

Reviewed By: JPD/JMAC

TABLE D3 – EVALUATION OF APPROACH EMBANKMENT FOUNDATION STABILITY/SETTLEMENT MITIGATION ALTERNATIVES - HIGHWAY 638 INTERCHANGE

Stability / Settlement Mitigation Option	*Rank*	*Advantages*	*Disadvantages*	*Relative Costs*	*Risks / Consequences*
Staged construction (with wick drains) (2 stages) (approximately 1.5 years of construction delays for staging)	1	• Smaller embankment footprint and less land acquisition requirements as compared with toe berms option. • Shortest period of construction delays for staging.	• Delay of approximately 1.5 years during staged construction and preloading. • Large post-construction settlement. • Large downdrag loads reduce pile capacity. • EPS required to maintain front slope stability and top-up to mitigate long-term settlements.	• $1,576,125 (wick drains at 1.5 m spacing) + • $1,878,200 cost of EPS to mitigate long-term settlements	• Staged construction sequence required with potential for additional delays during construction depending on monitoring. • Post-construction settlements may require long-term maintenance. • Some secondary consolidation (creep) will occur.
Toe berms (with up to 15 year preload)	4	• Standard construction operation. • No construction delays associated with staging.	• Greater quantities of fill required for berms. • Larger embankment footprint due to berms. • Large post-construction settlement. • Large downdrag loads reduce pile capacity. • EPS required to maintain front slope stability and top-up to mitigate long-term settlements. • Long preload period required to mitigate settlements	• $105,000 (berms) + • land acquisition costs + • $1,878,200 cost of EPS to mitigate long-term settlements and front slope stability.	• Risk of instability (low). • Secondary consolidation (creep) will occur. • Large post-construction settlements will require long-term maintenance. • Likely need to acquire additional right-of-way due to large berm size.

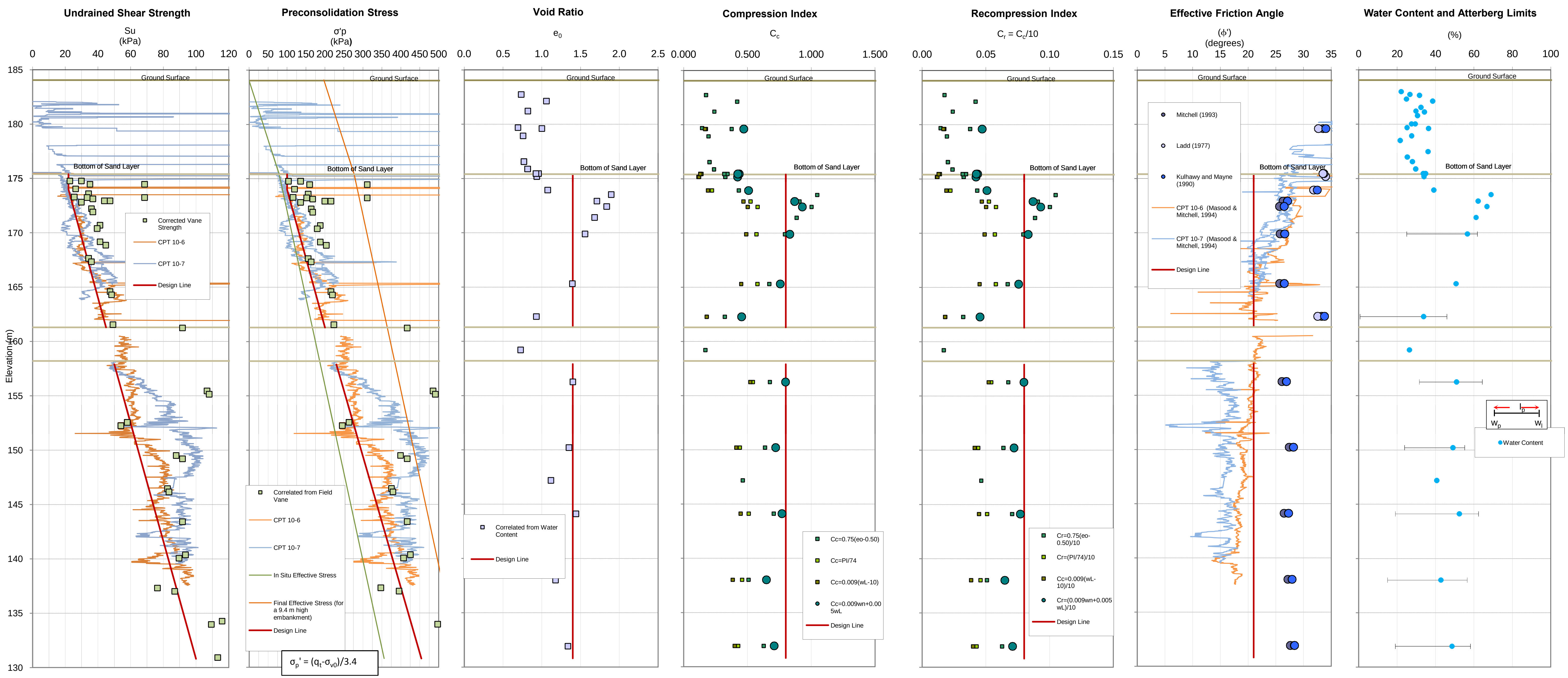
TABLE D3 – EVALUATION OF APPROACH EMBANKMENT FOUNDATION STABILITY/SETTLEMENT MITIGATION ALTERNATIVES - HIGHWAY 638 INTERCHANGE

Stability / Settlement Mitigation Option	*Rank*	*Advantages*	*Disadvantages*	*Relative Costs*	*Risks / Consequences*
Partial Lightweight Fill (EPS) (with up to 15 year preload)	3	• Standard construction operation. • No construction delays associated with staging or preloading. • Reduced secondary (creep) consolidation settlement. • Generation of smaller volume of excess excavation spoil since no toe berms. • Smaller quantities of fill required for subexcavate and replace since no toe berms. • Smaller embankment footprint.	• Higher cost for specialized materials. • EPS required to maintain front slope stability and top-up to mitigate long-term settlements. • Some post-construction settlements. • Long preload period required to mitigate settlements	• $1,223,200 cost of EPS to mitigate long-term settlements and front slope stability.	• Risk of instability (low). • Secondary consolidation (creep) will occur. • Post-construction settlement may require long-term maintenance.
Full Lightweight Fill (EPS)	2	• Standard construction operation. • No construction delays • Minimized post-construction settlement. • Smaller property acquisition required than with toe berms.	• Higher cost for specialized materials • Restricted use of EPS within the embankment cross-section to above water table.	• $5,922,400 cost of EPS to mitigate long-term settlements and front slope stability.	• Low risk of instability. • Low risk of long-term settlement of foundation soils.

Prepared By: MWK

Reviewed By: JPD/JMAC

Summary of Engineering Parameters for Cohesive Deposits Highway 638 Interchange	Figure D1



NOTES:

Average ground surface at the borehole locations is about Elevation 184.1 m

Average elevation of bottom of cohesive deposit at the borehole locations is about 130.8 m

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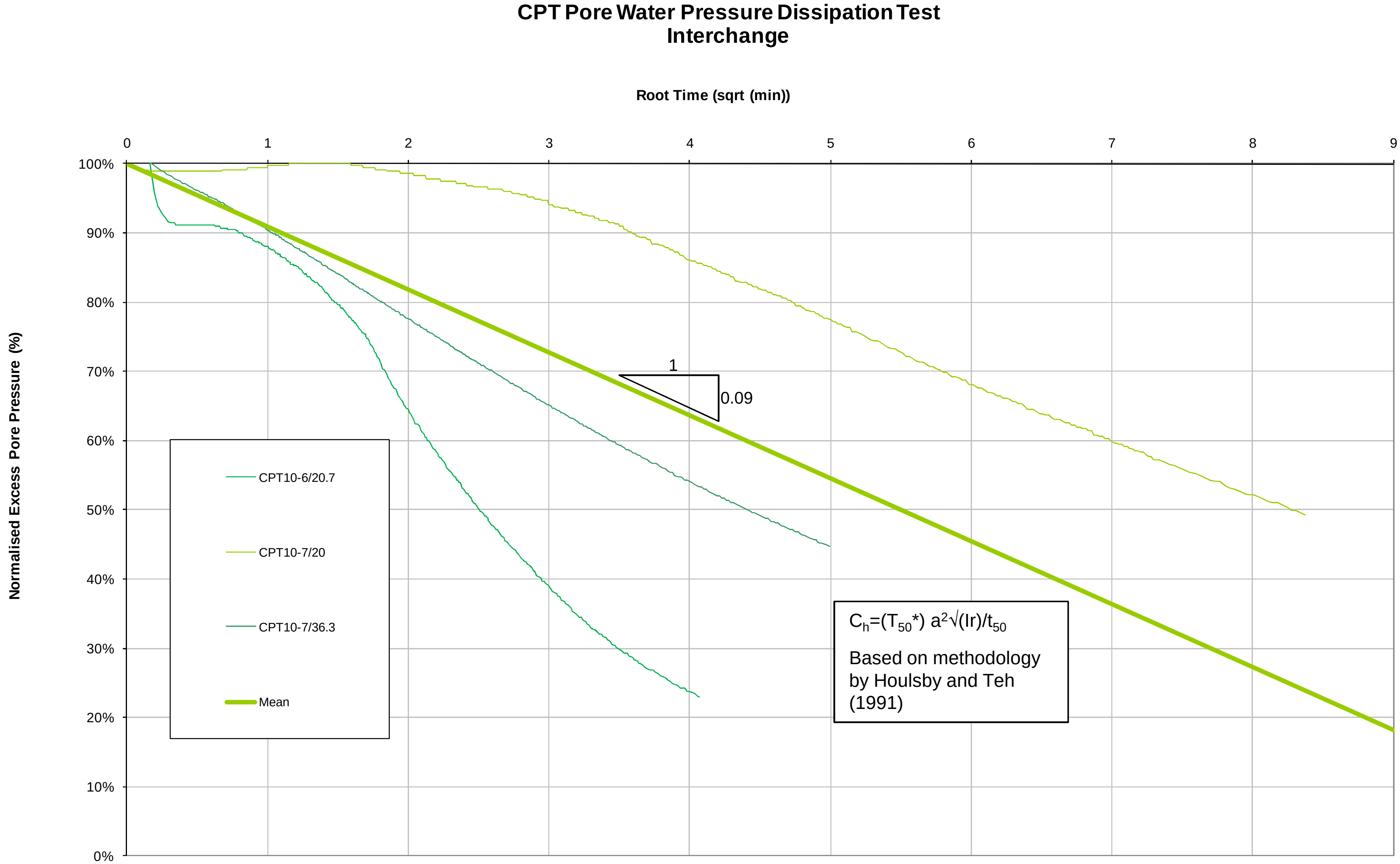
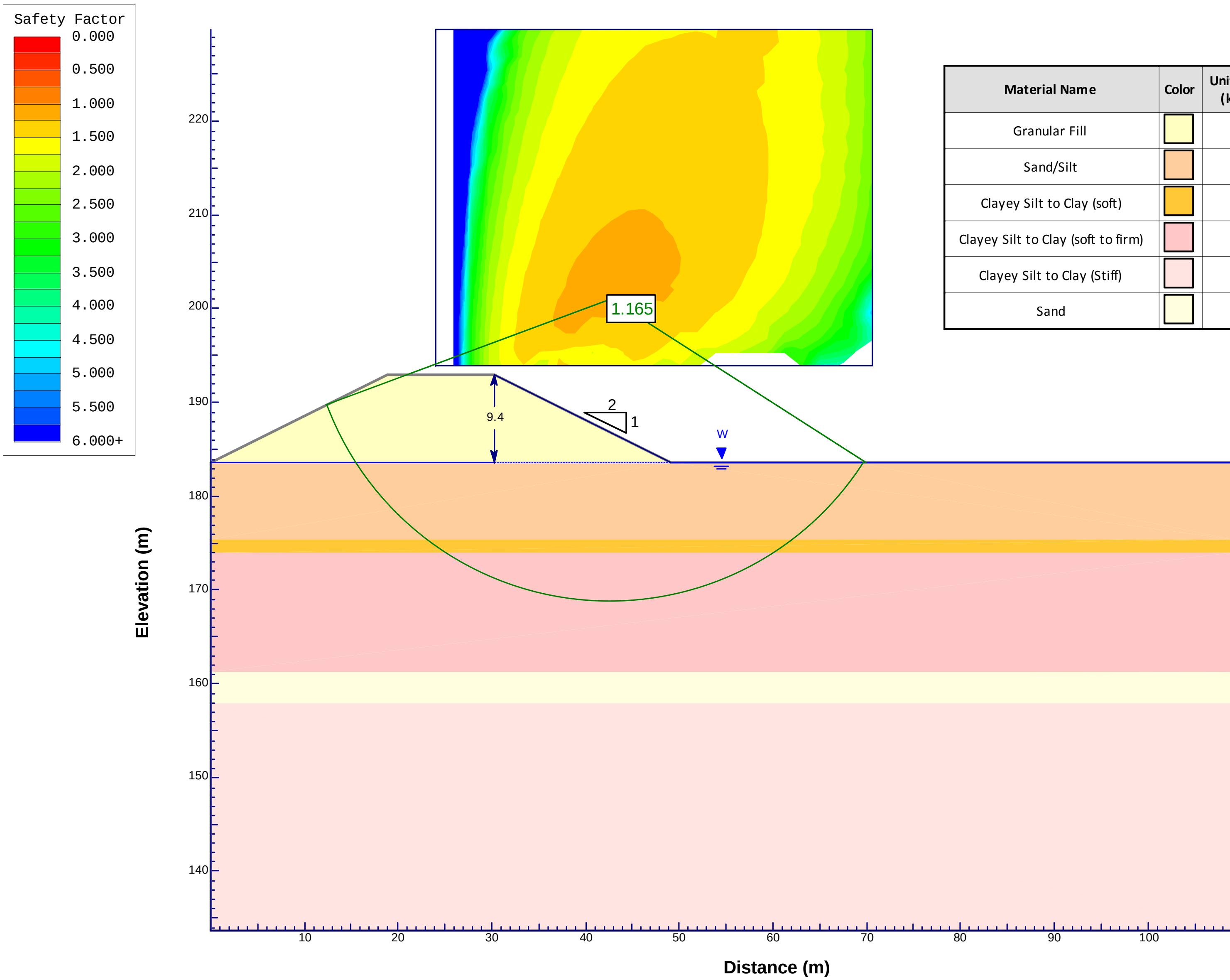
Date:	Jun-11	DB:	MWK
Project No:	09-1111-0016	CHK:	JPD

N:\Active\2009\1111\09-1111-0016 Genivar - Hwy 17 Interchange - Echo Bay\Analysis\Pile Analysis\[09-1111-0016 -West Flyover -Pile Capacity-MWK.xlsx]Pile Capacity


CPT Pore Water Pressure Dissipation Test
Interchange
Root Time (sqrt (min))
0
1
2
3
4
5
6
7
8
9
100%
90%
80%
70%
60%
50%
40%
30%
20%
10%
0%
Normalised Excess Pore Pressure (%)
1
0.09
CPT10-6/20.7
CPT10-7/20
CPT10-7/36.3
Mean
$C_h=(T_{50}^*)\ a^2\sqrt{(Ir)}/t_{50}$
Based on methodology by Houlsby and Teh (1991)

Slope Stability – Total Stress Analysis – No Stability Mitigation

Figure D3-1



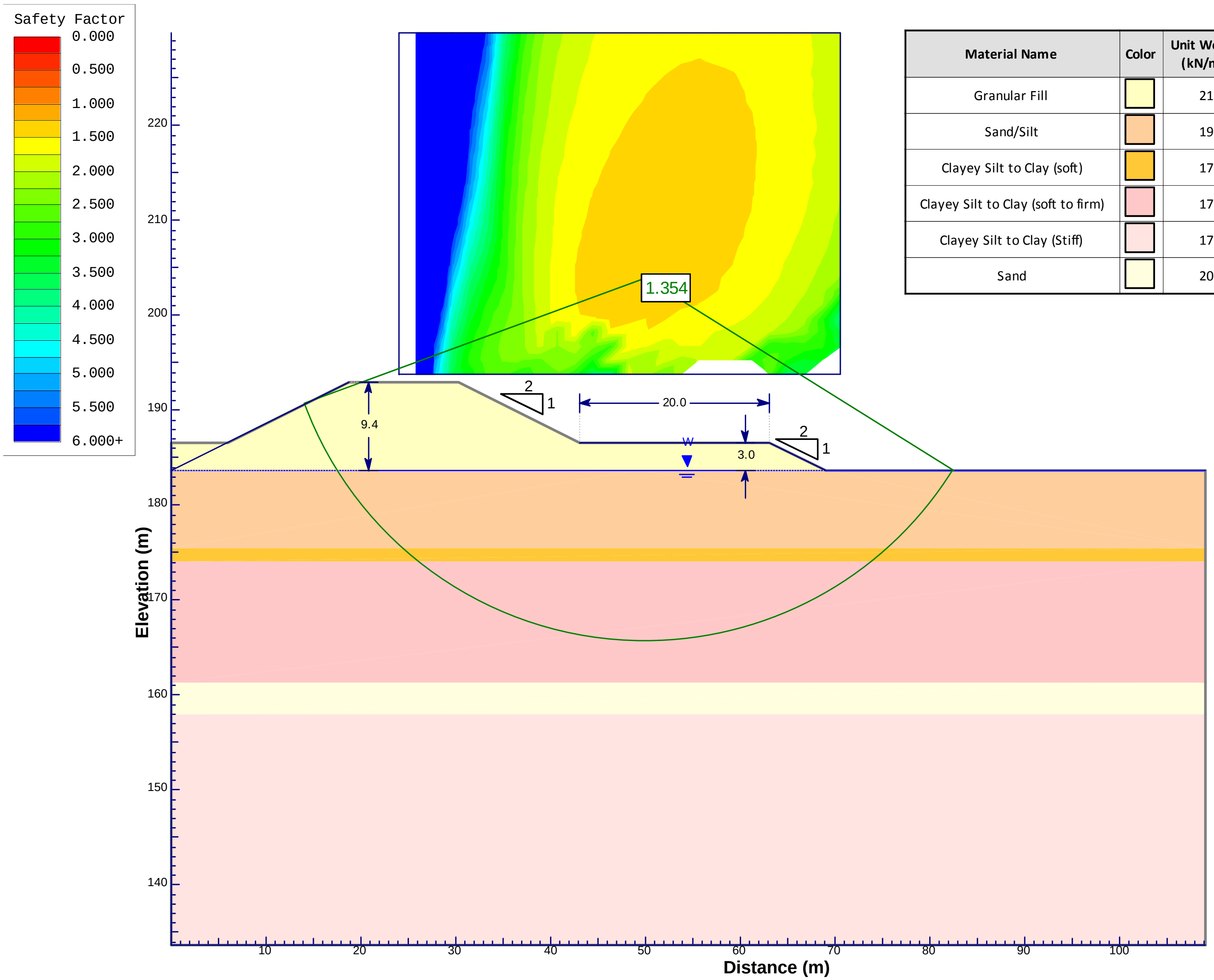
Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kN/m2)	Phi	Cohesion Type
Granular Fill		21	Mohr-Coulomb	0	32	
Sand/Silt		19	Mohr-Coulomb	0	28	
Clayey Silt to Clay (soft)		17	Undrained	22		Constant
Clayey Silt to Clay (soft to firm)		17	Undrained	22		FDepth
Clayey Silt to Clay (Stiff)		17	Undrained	50		FDepth
Sand		20	Mohr-Coulomb	0	30	

Date: June 2011
Project No: 09-1111-0016

Analysis By: MWK **Reviewed By:** JPD

Slope Stability – Total Stress Analysis – Stabilizing Berms

Figure D3-2



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kN/m2)	Phi	Cohesion Type
Granular Fill		21	Mohr-Coulomb	0	32	
Sand/Silt		19	Mohr-Coulomb	0	28	
Clayey Silt to Clay (soft)		17	Undrained	22		Constant
Clayey Silt to Clay (soft to firm)		17	Undrained	22		FDepth
Clayey Silt to Clay (Stiff)		17	Undrained	50		FDepth
Sand		20	Mohr-Coulomb	0	30	

Date: June 2011
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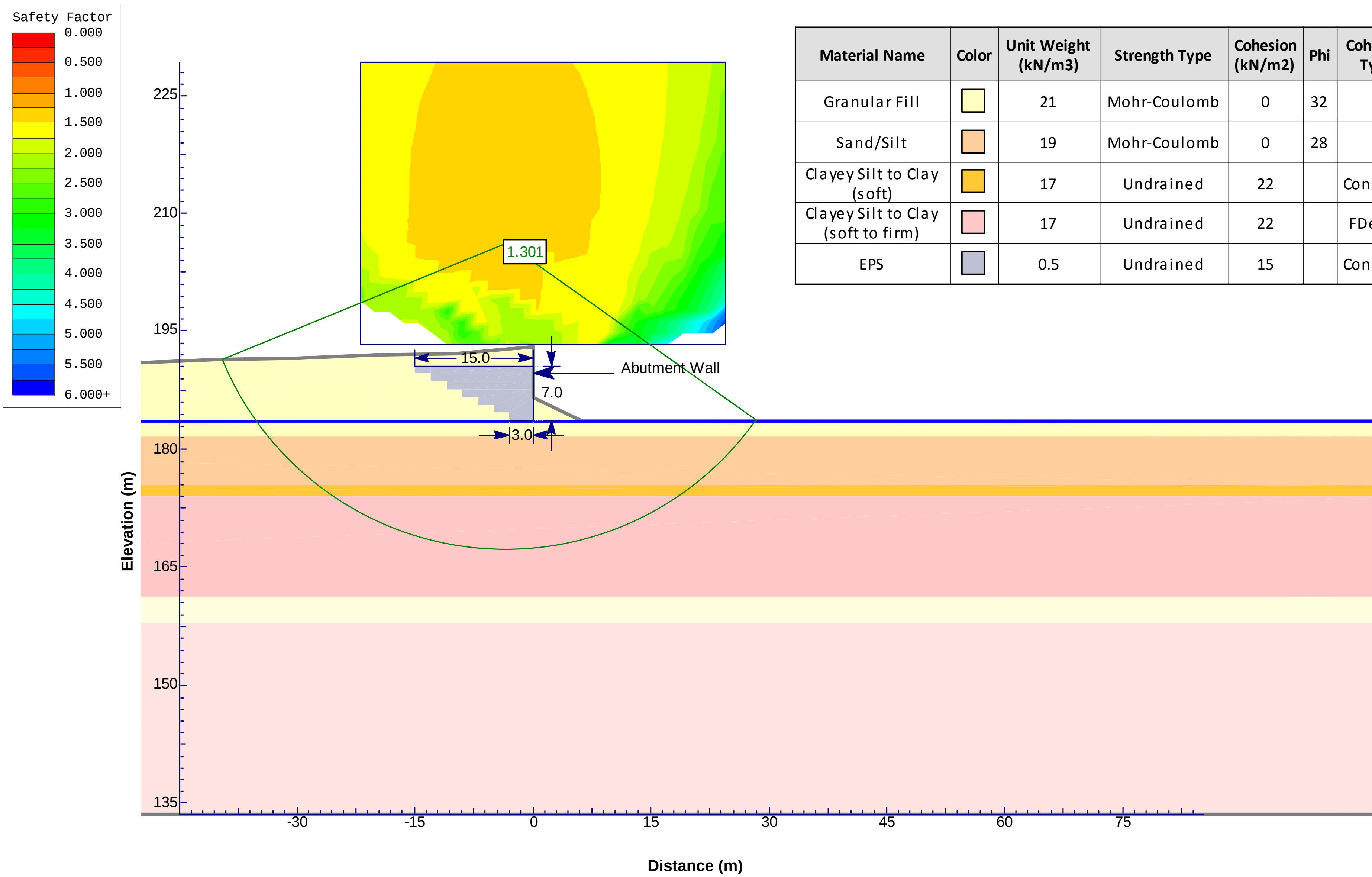

Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kN/m2)	Phi	Cohesion Type
Granular Fill		21	Mohr-Coulomb	0	32	
Sand/Silt		19	Mohr-Coulomb	0	28	
Clayey Silt to Clay (soft)		17	Undrained	22		Constant
Clayey Silt to Clay (soft to firm)		17	Undrained	22		FDepth
EPS		0.5	Undrained	15		Constant

Date: June 2011
Project No: 09-1111-0016

Analysis By: MWK **Reviewed By:** JPD

APPENDIX E

Cone Penetration Test Data Files (on CD)

At Golder Associates we strive to be the most respected global company providing consulting, design, and construction services in earth, environment, and related areas of energy. Employee owned since our formation in 1960, our focus, unique culture and operating environment offer opportunities and the freedom to excel, which attracts the leading specialists in our fields. Golder professionals take the time to build an understanding of client needs and of the specific environments in which they operate. We continue to expand our technical capabilities and have experienced steady growth with employees who operate from offices located throughout Africa, Asia, Australasia, Europe, North America, and South America.

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