FOUNDATION INVESTIGATION AND DESIGN REPORT

REPLACEMENT OF GLEN ROAD PEDESTRIAN BRIDGE

ENVIRONMENTAL ASSESSMENT STUDY

TORONTO, ONTARIO

November 7, 2017

WSP

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1 INTRODUCTION

WSP (Environment) Toronto (“WSP”) was retained by WSP/MMM Group (“MMM”) to carry out foundation investigations for an EA Study. The geotechnical investigation was to provide necessary geotechnical information and make recommendations for the replacement and widening of the existing Glen Road Pedestrian Bridge over Rosedale Valley Road, connecting Bloor Street on the South with Dale Avenue on the North, and replacement and widening of the existing tunnel, under Bloor Street, as part of the EA study.

The purpose of the Geotechnical Investigation was to determine the sub-surface conditions at the site by means of boreholes, field and laboratory tests. Based on the information obtained, the engineering characteristics of the subsurface soils have been assessed and site conditions described. Geotechnical information obtained has been used to develop preliminary geotechnical recommendations regarding the design and re-construction/widening of the structures mentioned above and alternative foundation options discussed.

The present EA Study addresses environmental issues. Geotechnical field investigations included field components planned and directed by the MMM environmental group and the findings will be reported by them separately.

Minor horizontal and no vertical alignment changes to the existing pedestrian bridge or approaches are proposed as per the preliminary GA.

Initially the report presents factual information concerning the subsurface conditions based on all of the subsurface information at hand and is followed by engineering discussion and recommendations for the design and re-construction/widening of the structures mentioned above.

2 BACKGROUND INFORMATION

2.1 GEOLOGICAL SETTING

According to surficial geology of the Greater Toronto Area Map -3062 (Scale: 1:200 000), regionally, the project site lies within glacial lake deposits (silt and clay).

According to bedrock geology of Ontario Map MNDM-2544 (Scale: 1:1 000 000), the bedrock underlying the site comprises Georgian Bay Shale, limestone, dolostone and siltstone of the Upper Ordovician.

2.2 PREVIOUS GROUND INVESTIGATIONS

The general geology of the area was further explored using regional water well data collected (MOE water well records (WWR)).

The wells were reportedly used for local groundwater level measurements and were installed in the Silty

Clay/Clayey Silt deposits.

Well records indicate that the silty clay/clayey silt deposits were predominately encountered up to drilled depths varying from 6 m to 18 m in the vicinity of Sherbourne TTC station.

2.3 SITE AND STRUCTURE DESCRIPTION

A plan view of the site location, which is in the City of Toronto, Ontario is shown on **Drawing 1**. The existing Glen Road Pedestrian Bridge, constructed in 1973, is a structure that spans from Bloor Street East in the south to Glen Road in the north, crossing over Rosedale Valley Road. South of the bridge, a pedestrian tunnel with stairs at each end, crosses under Bloor Street, and connects the bridge to Glen Road at the south end.

The bridge is a three span rigid frame steel bridge with inclined legs and timber deck, originally built in 1973. The overall span and width of the bridge are approximately 107.0 m and 3.7 m respectively. The original structure drawings, dated June 1973, indicate that the abutments and pier legs are supported on spread footings. The surrounding area of the bridge site is generally well-vegetated with treed/grassy landscape in the valley slope. No signs of significant gully erosion (in the locality of pier footings) and undermining of the pier footings were noted during our site investigation. The approximate gradients of south valley slope and north valley slope were 1 (vertical) : 2.4 (horizontal) and 1 (vertical) : 2.0 (horizontal) respectively. Photographs 1 to 6 (all photographs in **Appendix C**) give a general impression of the site.

3 FIELD AND LABORATORY INVESTIGATIONS

3.1 FIELD INVESTIGATIONS

The fieldwork undertaken by WSP during May 2017 consisted of carrying out six (6) boreholes (BH16-1 to BH16-6) to investigate the subsurface conditions.

Prior to WSP drilling programme, a subsurface utility engineering (SUE) - Level A investigation was done at the planned borehole locations at Bloor Street and Dale Avenue to resolve various utility conflicts in the vicinity of the borehole locations. The SUE investigation was carried out by Planview Utility Services Limited, Markham, Ontario. Table 3-1 presents the borehole details of the WSP investigation program. The borehole locations are shown on **Drawing 1** following the text of the report.

Table 3-1: Summary of Borehole Details

BH No:	*Co-ordinates (m)	Ground Elevation (m)	Drilled Depth (m)	Remarks*/ Drilling Methodology
BH16-1	E 314873 N 4836859	110.2	5.2	In the vicinity of north abutment (Dale Ave.); Hollow Stem Auger; terminated within native silty clay; split spoon sampling and monitoring well installed

BH No:	*Co-ordinates (m)	Ground Elevation (m)	Drilled Depth (m)	Remarks*/ Drilling Methodology
BH16-2	E 314863 N 4836725	114.9	12.8	Adjacent to west side of the proposed tunnel alignment (Bloor Street); Hollow stem auger; terminated within native silty clay till; split spoon sampling and monitoring well installed.
BH16-3	E 314887 N 4836787	89.2	11.1	South side sidewalk of Rosedale Valley Road; Hollow stem auger; terminated within silty clay-till shale complex; split spoon sampling and monitoring well installed.
BH16-4	E 314866 N 4836819	92.6	3.5	Drilled on slope; North side of Rosedale Valley Road along the existing bridge alignment; Pionjar Method without SPT test; continuous spoon sampling by percussion drill; terminated within native silty clay till.
BH16-5	E 314871 N 4836754	99.4	3.1	Drilled on slope; South side of Rosedale Valley Road along the bridge alignment; Pionjar Method without SPT test; continuous spoon sampling by percussion drill; terminated within native silty clay till.
BH16-6	E 314883 N 4836722	114.9	20.4	In the vicinity of south abutment; adjacent to east side of the existing pedestrian tunnel alignment; Hollow stem auger; terminated within native silty clay till; split spoon sampling and monitoring well installed.

Notes*:

1. Co-ordinates: based on MTM10 NAD27 coordinates; terminology of directions, e.g. north, are project defined and do not relate to geographic directions.
2. Name of Drilling Company: Geo-Environmental Drilling Inc. Acton, Ontario.
3. Type of Drilling rig Used: Track mounted - CME 55 rig and Portable Pionjar in close proximity to the existing pier locations.
4. Drilling Supervision by: WSP staff from Toronto office.
5. Borehole Survey: Coordinates and Elevations for BH16-1, BH16-2 and BH16-6 are based on Planview SUE report; Coordinates and Elevations for BH16-3, BH16-4 and BH16-5 are based on hand-held GPS equipment and dumpy level surveying respectively. For dumpy level surveying, local bench mark was

referenced to elevation of top surface (El.99.25 m) of existing south side inclined leg footing (from old construction drawing-drawing no. 4 of Albery Pullerits, Dickson & Associates Ltd.)

The soil stratigraphy was recorded by observing the quality and changes of augered materials, which were withdrawn from the boreholes, and by sampling the soils at regular intervals of depth using a 50mm O.D. split spoon sampler, in accordance with the Standard Penetration Test (ASTM D 1586) method. This sampling method recovers samples from the soil strata, and the number of blows required to drive the sampler 300 mm depth into the undisturbed soil (SPT 'N'-values) gives an indication of the compactness condition or consistency of the sampled soil material. The SPT 'N' values are indicated on the Record of Borehole Sheets (Refer to **Appendix A**). An asphalt core sample was collected at each of the three locations (BH16-1, BH16-2 and BH16-3) to perform asbestos content testing.

The WSP borehole drilling was carried out under full-time supervision of WSP technical staff who directed the drilling and sampling operation, logged borehole data in accordance with MTO Soils Classification System (as per RFP-Appendix A.3.3) and took custody of soil samples retrieved for subsequent laboratory testing and identification. Soil samples were visually classified in the field and later re-evaluated by an engineer. The recovered soil samples were placed in labelled moisture-proof bags and returned to WSP's Galaxy Boulevard laboratory for further assessment.

Geo-Environmental Investigation

In accordance with MOECC sampling protocols, soil and water samples for potential chemical analysis of parameters were placed directly into laboratory supplied glass jars at the time of sampling and packed with minimal headspace to reduce the volatilization of organic compounds. Headspace combustible vapour measurements were taken inside plastic bags containing soil using Phocheck (Serial no.10-01535) equipment. The detailed reporting for geo-environmental investigation (including asbestos test results) is issued as a separate report. Hence, further comment on geo-environmental investigations will not be discussed in this report.

3.2 LABORATORY INVESTIGATIONS

Visual examination and classification were undertaken on the soil samples returned to the WSP laboratory. A laboratory testing program consisting of natural water content tests, grain size analyses, including hydrometer testing and Atterberg limits, was carried out on selected representative soil samples. The results of the laboratory tests are summarized on the appropriate Record of Borehole Sheets in **Appendix A**, and the details presented in **Appendix B**.

3.3 GROUNDWATER INVESTIGATIONS

Groundwater conditions in the boreholes were observed during and on completion of drilling in the open boreholes. Monitoring wells were installed in Boreholes BH16-1, BH16-2, BH16-3 and BH16-6 upon their completion to enable long term groundwater level monitoring. The rest of the boreholes were grouted (decommissioned) using a cement/bentonite mixture as per MOE procedures. As part of the construction, the wells need to be decommissioned in accordance with Ontario Regulation 903 (amended by Ontario Regulation 372/07).

Table 3.2 below provides information about the wells installed for this investigation, including ground surface elevations, depths, and the approximate elevations of the screened intervals.

Table 3-2: Well Installation Details

BH ID	Ground Surface Elevation (m)	Borehole Bottom		Well Screen Interval Depth, m		Well Screen Interval Elevation, m	
		Depth (m)	Elevation (m)	From	To	From	To
BH 16-1	110.2	5.2	105.0	3.1	4.6	107.1	105.6
BH 16-2	114.9	12.8	102.1	2.1	5.2	112.8	109.7
BH 16-3	89.2	11.1	78.1	4.6	7.6	84.6	81.6
BH 16-6	114.9	20.4	94.5	16.5	19.5	98.4	95.4

4 SUBSURFACE CONDITIONS

4.1 GENERAL

The subsurface conditions encountered at the bridge location are described in the following sections. For purposes of soil description, the MTO soil classification manual was generally followed.

A borehole location plan and a subsurface profile (stick logs) are shown on **Drawing 1** and **Drawing 2** at the end of the text.

The soil descriptions are based on visual and tactile observations, complemented by the results of field and laboratory soil test results. It should be noted that the subsurface conditions and the topsoil thicknesses encountered may vary in between and beyond the borehole locations.

An overview of subsurface conditions is described below. All depths quoted are below existing ground surface. It is to be noted that based on the borehole data, the elevations (El.) reported for strata boundaries are from the shallowest occurrence to the deepest occurrence.

4.2 OVERVIEW

In general terms, the stratigraphic sequence encountered can be described as topsoil/pavement structure underlain by fill materials (sandy silt to silty sand / silty clay to clayey silt). The underlying native deposits generally consist of upper silty clay followed by silty clay till, whereas in BH 16-4 and BH 16-5, the fill was underlain by silty clay till. BH16-3 advanced from the Rosedale Valley Road had a ground elevation below the bottom elevations of the other boreholes. A silt/sandy silt to silty sand deposit was encountered beneath the fill material in BH16-3 overlying the lower silty clay layer and the borehole was terminated in the underlying till-shale complex.

Boreholes BH16-1, BH16-2, BH16-3 and BH 16-6 were each installed with a monitoring well. The measured groundwater levels in the wells were 1.7 m and 17.3 m in BH16-3 and BH16-6 respectively below existing ground surface (i.e., elevations of El. 87.5 m and 97.6 m) when measured approximately five (5) weeks (in BH16-3) and one week (in BH16-6) following their installation. Wells in BH16-1 and BH16-2 recorded dry conditions during this monitoring period.

Note that the topsoil/pavement structure thicknesses reported may vary beyond the borehole locations. Further, this information will not be sufficient for quantity estimation. Although not intercepted, the fill can contain cobbles/boulders/debris.

It must be noted that the factual data presented on the Record of Borehole Sheets would govern any interpretation of the site conditions provided in the text of this report.

The glacial deposits, due to their mode of deposition, can be expected to contain cobbles and boulders; auger grinding was observed in BH16-2 suggesting that they are indeed present.

The following paragraphs are intended to give more detailed descriptions of the data documented on the Record of Borehole Sheets (**Appendix A**).

4.3 SUBSOIL CONDITIONS

4.3.1 TOPSOIL

Boreholes BH16-4 and BH16-5 encountered topsoil at the ground surface, which ranged between 75 mm and 100 mm in thickness. However, it is to be noted, based on our experience, the thickness of topsoil frequently varies in between and beyond borehole locations, especially in depressed areas.

4.3.2 PAVEMENT STRUCTURE

A pavement structure consisting of 100 mm to 170 mm thick surficial asphalt layer overlying 190 mm to 200 mm of granular base/subbase material (sand/sand and gravel) was encountered at the boreholes BH16-1 and BH 16-2, BH16-3 and BH16-6. A 150 mm thick concrete base was only encountered at borehole BH16-2.

Measured moisture contents of the pavement granular base materials were between 8% (BH16-3) to 12% (BH 16-2), indicative of a generally moist to wet condition (based on three (3) SPT samples).

Standard Penetration Tests performed on the pavement granular fill materials yielded N-values ranging from 4 blows/300 mm (BH16-1/SS1) to 7 blows/300 mm (BH16-2/SS1 & BH16-3/SS1), which indicate a loose relative density condition (based on three (3) SPT results).

4.3.3 FILL MATERIALS

4.3.3.1 UPPER COHESIONLESS FILL (SILTY SAND/SAND TO SANDY SILT)

A silty sand/sand to sandy silt fill was contacted in boreholes BH16-2, BH16-3, BH16-5 and BH16-6 underlying the granular fill / topsoil. The fill contained trace to some gravel and trace to some clay. A hydrocarbon odour was noted within this fill material only in the BH16-3.

The thicknesses of this material ranged between 0.1 m and 3.4 m and the elevation of the base of the unit varied between El. 99.2 m (BH16-5) and El. 85.5 m (BH16-3).

The grain size distributions of two (2) samples from the fill were determined in the laboratory and gave the grain size information shown in Table 4-1.

Table 4-1: Grain Size Distribution Summary-Silty Sand to Sand (Fill)

Sample Tested	Size Fraction	% Passing by weight	Remarks
BH16-3/SS2 **BH16-6/SS3**	Gravel	1 to 3%	Shown as Fig.1a, in Appendix B; Summarized on the relevant Record of Borehole Sheets
	Sand	63 to 78%	
	Silt	16 to 31%	
	Clay	3 to 5%	

The grading result shown above indicates this fill can be classified generally as cohesionless (SM/SP).

The moisture content based on ten (10) samples recovered from this fill material ranged from 7% to 21% indicative of a moist to wet condition.

SPT testing carried out in the boreholes, gave SPT ‘N’ values ranging from 2 blows/300 mm to 28 blows/300 mm (based on 10 SPT results) which indicate generally a very loose to compact relative density condition.

4.3.3.2 COHESIVE FILL (SILTY CLAY TO CLAYEY SILT)

A silty clay to clayey silt fill was contacted in the boreholes BH16-1, BH16-2, BH16-4, BH16-5 and BH16-6 underlying topsoil/granular fill/silty sand to sandy silt fill materials. The fill contained traces of gravel and sand. Traces of rootlets and asphalt fragments were also encountered within this fill material.

The thicknesses of this material ranged between 1.7 m and 3.7 m and the elevation of the base of the unit varied between El. 110.4 m (BH16-2) and El. 90.8 m (BH16-4).

The grain size distributions of two (2) samples from the fill were determined in the laboratory and gave the grain size information shown in Table 4-2

Table 4-2: Grain Size Distribution Summary-Silty Clay to Clayey Silt (Fill)

Sample Tested	Size Fraction	% Passing by weight	Remarks

BH16-1/SS2 **BH16-2/SS4**	Gravel	0 to 3%	Shown as Fig.1b, in Appendix B; Summarized on the relevant Record of Borehole Sheets
	Sand	10 to 32%	
	Silt	46 to 58%	
	Clay	19 to 32%	

Atterberg Limits test was performed on one (1) sample from this fill material. This test indicated the following index values as shown in Table 4-3.

Table 4-3: Atterberg Limits Test Results-Silty Clay (Fill)

Sample Tested	**Atterberg Limits**	**Index Values**	**Remarks**
BH16-2/SS4	Liquid Limit	16%	Shown as Fig.1c, in Appendix B; Summarized on the relevant Record of Borehole Sheet
	Plastic Limit	25%	
	Plasticity Index	9%	

The above values are characteristic of a cohesive soil of low plasticity (CL).

The moisture contents based on twenty one (21) samples recovered from this fill material ranged from 7% to 27% indicative of a moist to wet condition.

SPT testing carried out in the boreholes, gave SPT ‘N’ values ranging from 4 blows/300 mm to 20 blows/300 mm (based on 10 SPT results) which indicate generally a firm to very stiff consistency. A high blow count of 69 blows/225 mm was encountered only in the BH16-6 due to presence of asphalt fragments at a depth of 4.8 m.

4.3.3.3 LOWER COHESIONLESS FILL (SAND AND GRAVEL)

A sand and gravel fill was contacted only in the borehole BH16-2 underlying the cohesive fill (silty clay). The fill contained traces of silt.

The thickness of this material was 1.3 m and the elevation of the base of the unit was El.109.1 m.

The moisture content based on one (1) sample recovered from this fill material was 12% indicative of a moist condition.

SPT testing carried out in the borehole, gave a SPT ‘N’ value of 7 blows/300 mm (based on one (1) SPT result) which indicates generally a loose relative density condition.

4.3.4 UPPER SILTY CLAY

A native brown to grey upper silty clay was contacted in boreholes BH16-1, BH16-2 and BH16-6 underlying the fill materials. This deposit contained traces of sand and gravel.

The thicknesses of this material ranged between 2.9 m and 4.1 m and the elevation of the base of the unit varied between El. 104.4 m (BH16-6) and El. 106.2 m (BH16-2).

The grain size distributions of two (2) samples from the deposit were determined in the laboratory and gave the information shown in Table 4-4.

Table 4-4: Grain Size Distribution Summary-Upper Silty Clay

Sample Tested	Size Fraction	% Passing by weight	Remarks
BH16-1/SS7 **BH16-2/SS8**	Gravel	0%	Shown as Fig.2a, in Appendix B; Summarized on the relevant Record of Borehole Sheet
	Sand	2 to 3%	
	Silt	40%	
	Clay	57 to 58%	

Atterberg Limits test was performed on two (2) samples from this deposit. This test indicated the following index values as shown in Table 4-5.

Table 4-5: Atterberg Limits Test Results-Upper Silty Clay

Sample Tested	Atterberg Limits	Index Values	Remarks
BH16-1/SS7 **BH16-2/SS8**	Liquid Limit	38 to 49%	Shown as Fig.2b, in Appendix B; Summarized on the relevant Record of Borehole Sheet
	Plastic Limit	17 to 21%	
	Plasticity Index	20 to 28%	

The above values are characteristic of a cohesive soil of intermediate plasticity (CI).

The moisture content based on nine (9) samples recovered from this deposit varied from 15% to 40% indicative of a moist to wet condition.

SPT testing carried out in the boreholes, gave SPT ‘N’ values ranging from 7 blows/300 mm to 22 blows/300 mm (based on 9 SPT results) which indicate generally a firm to very stiff consistency.

4.3.5 SILTY CLAY TILL

A native grey to brown silty clay till was contacted in boreholes BH16-2 BH16-4, BH16-5 and BH16-6 underlying the fill materials/native silty clay deposit. This deposit contained trace to some sand and traces of gravel.

Occasional sand seams were also noted within this deposit at BH16-5.

The explored thickness of this deposit varied from 0.6 m to 9.9 m and the elevation of the explored base of the unit varied from El. 89.1 m (BH16-4) to El. 102.1 m (BH16-2). These boreholes were terminated within this deposit.

The grain size distributions of four (4) samples from the deposit were determined in the laboratory and gave the grain size information shown in Table 4-6.

Table 4-6: Grain Size Distribution Summary-Silty Clay Till

Sample Tested	Size Fraction	% Passing by weight	Remarks
BH 16-2/SS12 **BH 16-4/SS5** **BH 16-5/SS5** **BH 16-6/SS14**	Gravel	1 to 2%	Shown as Fig.3a, in Appendix B; Summarized on the relevant Record of Borehole Sheets
	Sand	18 to 30%	
	Silt	46 to 58%	
	Clay	21 to 31%	

Atterberg Limits tests were performed on four (4) samples from this deposit. These tests indicated the following index values as shown in Table 4-7.

Table 4-7: Atterberg Limits Test Results-Silty Clay Till

Sample Tested	Atterberg Limits	Index Values	Remarks
BH 16-2/SS12 **BH 16-4/SS5** **BH 16-5/SS5** **BH 16-6/SS14**	Liquid Limit	24 to 26%	Shown as Fig.3b, in Appendix B; Summarized on the relevant Record of Borehole Sheets
	Plastic Limit	12 to 15%	
	Plasticity Index	10 to 12%	

The above values are characteristic of a cohesive soil of low plasticity (CL).

The moisture content based on fourteen (14) samples recovered from the layer ranged from 13% to 28% indicative of a moist to wet condition.

SPT testing carried out in the boreholes, gave SPT ‘N’ values ranging from 15 blows/300 mm to 29 blows/300 mm (based on 10 SPT results) which indicate generally a stiff to very stiff consistency.

4.3.6 SILT TO SANDY SILT/SILTY SAND

A grey silt to sandy silt/silty sand native deposit was contacted only in borehole BH16-3. It contained some clay and a trace of gravel.

The thickness of this deposit was 4.2 m and the elevation of the base of the unit was El. 81.3 m

The grain size distributions of one (1) sample from the deposit was determined in the laboratory and gave the grain size information shown in Table 4-8.

Table 4-8: Grain Size Distribution Summary-Sandy Silt

Sample Tested	Size Fraction	% Passing by weight	Remarks
BH16-3/SS7	Gravel	1%	Shown as Fig.4, in Appendix B; Summarized on the relevant Record of Borehole Sheet
	Sand	30%	
	Silt	56%	
	Clay	12	

The moisture content based on four (4) samples recovered from the layer ranged from 6% to 16% indicative of a moist condition.

SPT testing carried out in the boreholes gave SPT ‘N’ values ranging from 55 blows/300 mm to greater than 100 blows/300 mm (based on 4 SPT results) which indicate a generally very dense relative density.

4.3.7 LOWER SILTY CLAY

A native grey lower silty clay was contacted the boreholes BH16-3 underlying the native silt to sandy silt/silty sand deposit. This deposit contained traces of sand.

The thickness of this deposit was 2.8 m and the elevation of the base of the unit was El. 78.5 m.

The grain size distribution of one (1) sample from the deposit was determined in the laboratory and gave the information shown in Table 4-9.

Table 4-9: Grain Size Distribution Summary-Lower Silty Clay

Sample Tested	Size Fraction	% Passing by weight	Remarks
BH16-3/SS10	Gravel	0%	Shown as Fig.2, in Appendix B; Summarized on the relevant Record of
	Sand	7%	

	Silt	62%	Borehole Sheet
	Clay	31%	

An Atterberg Limits test was performed from the same spoon sample from this deposit. This test indicated the following index values as shown in Table 4-10.

Table 4-10: Atterberg Limits Test Results-Lower Silty Clay

Sample Tested	Atterberg Limits	Index Values	Remarks
BH16-3/SS10	Liquid Limit	34%	Shown as Fig.5, in Appendix B; Summarized on the relevant Record of Borehole Sheet
	Plastic Limit	17%	
	Plasticity Index	17%	

The above values are characteristic of a cohesive soil of low plasticity (CL).

The moisture content values based on two (2) samples recovered from this deposit were 10% and 16% indicative of a moist condition.

SPT testing carried out in the borehole, gave a SPT ‘N’ value of 86 blows/300 mm which indicates a hard consistency.

4.3.8 SILTY CLAY TILL/SHALE COMPLEX

A grey silty clay till/shale complex deposit was encountered only in borehole BH16-3. Shale fragments were generally identified within this deposit.

The explored thickness of this deposit was 0.4 m and the elevation of the explored base of the unit was El. 78.1 m. The borehole was terminated within this deposit.

SPT testing carried out in the borehole, gave SPT ‘N’ values of greater than 100 blows/300 mm (based on 3 SPT results) which indicate generally a hard consistency. Split spoon bouncing was experienced in the borehole suggesting the presence of shale fragments.

4.4 GROUNDWATER OBSERVATIONS

Groundwater conditions in the open boreholes were observed during drilling and at the completion of each borehole. A monitoring well was installed in each of the Boreholes BH16-1, BH16-2, BH16-3 and BH16-6. The groundwater observations are shown on the individual Record of Borehole Sheets in **Appendix A**.

The observed water levels in the open boreholes on completion ranged from 1.5 m/El.97.9m (BH16-5) to 3.1 m/El.86.1 m (BH16-3) m below grade level. The boreholes BH16-1, BH16-2 and BH16-6 showed dry condition upon completion. It should be noted that these water levels had not stabilized. In the wells, the water levels were measured at depths of 1.7 m (approx. 5 weeks after installation in BH16-3) and 17.3 m (1 week after installation in BH16-6) below ground surface or at El. 87.5 m and 97.6 m respectively.

Table 4-11 summarizes the ground water level measurements.

Table 4-11: Summary of Groundwater Observations

BH No.	**Ground Elevation (m)**	**Top of Screen Depth/Elevation (m)**	**Water Level Measurements**		**Remarks**
			Depth (m)	Elevation (m)	
BH16-1	110.2	3.1/107.1	dry		
BH16-2	114.9	2.1/112.8	dry		
BH16-3	89.2	4.6/84.6	3.1*(May03,2017)	86.1	
			1.5 (May17, 2017)	87.7	
			1.7 (June13, 2017)	87.47	
BH16-4	92.6	N/A	2.4* (May3, 2017)	90.2	Caved-in at 2.7 m
BH16-5	99.4	N/A	1.5*(May3, 2017)	97.9	Caved-in at 2.7 m
BH16-6	114.9	16.5/98.4	dry*	N/A	
			17.3 (May11, 2017)	97.6	

*water level measurement upon completion of borehole

It should be pointed out that groundwater levels would be subject to seasonal fluctuations in response to major weather events.

5 DISCUSSION AND RECOMMENDATIONS

5.1 GENERAL

This section of the report provides recommendations for the foundation aspects for the proposed Reconstruction of Glen Road Pedestrian Bridge over Rosedale Valley Road, connecting Bloor Street East on the South with Dale Avenue on the North and widening of the existing tunnel under Bloor Street, as part of the EA study. The pedestrian bridge reconstruction, according to the preliminary GA, almost coincides with the existing on plan but with a wider bridge cross-section. On plan, there is a slight shift of the alignment to the west at the southern abutment end and similar shift to the east at the north abutment end. The proposed pier locations are downslope from the existing pier locations on both valley slopes.

The recommendations are based on our understanding of the project and on the interpretation of factual data compiled from both field and laboratory investigations carried out by WSP for this project. The discussions and recommendations presented in this report are intended to assist the designers with sufficient information that would enable them to proceed with the design of the structure foundations/abutments, approach embankments and address the proposed tunnel widening.

Construction comments made herein are based on geotechnical considerations only and should not be relied upon without further independent assessment and qualification in the selection of means and methods for construction.

Based on the Preliminary General Arrangement Drawing (dated September, 2016), the proposed pedestrian bridge structure will be similar to the existing, a three (3) span steel bridge, however with concrete decking with a total span length of 100 m (central span of 40 m and equal 30 m end spans). The existing bridge has timber decking. The existing bridge approaches at the abutments are approximately 2.8 m high based on the preliminary GA.

5.2 GEOTECHNICAL MODEL

5.2.1 OVERVIEW OF SUB-SURFACE CONDITIONS

Drawing 2 shows a stick diagram of the borehole logs called a fence diagram which shows an preliminary view of a geotechnical profile based on the borelogs. The main findings are as follows:

- A pavement structure was intercepted in boreholes BH16-1, BH16-2, BH16-3 and BH16-6.
- Fill material to varying thicknesses (with an upper sandy fill followed by a lower clayey fill) was observed at all borehole locations up to depths varying from 1.8 m (BH16-4, i.e. on Rosedale Valley North slope) to 6.4 m (BH16-6, i.e. in close proximity to the southern abutment of the pedestrian bridge).
- Underlying the fill, a silty clay deposit was contacted in all boreholes except in BH 16-3, BH16-4 and BH16-5. The upper 3 m or so of this deposit shows high moisture levels as seen in BH 16-1, BH16-2 and BH16-6. The upper horizon of this deposit is generally firm and becomes stiff with depth. The

confirmed thicknesses (those that were penetrated to different lithologies below) of this deposit were 2.9 m (BH16-2) and 4.1 m (BH16-6).

- Underlying this silty clay deposit, a glacial silty clay till deposit was contacted in BH 16-2 and BH16-6 (both on Bloor St.) and on the slopes in boreholes BH 16-4 and BH16-5 and these boreholes were terminated within this deposit. Based on the SPT N values recorded in boreholes BH16-2 and BH16-6, this deposit is generally of very stiff consistency. Cobbles and boulders can be expected within the till due to the mode of deposition. In fact, auger grinding was experienced in BH16-2.
- BH16-3 on Rosedale Valley Road that had a ground elevation below the terminal elevations of the rest of the boreholes, had a different subsurface profile. Following fill material, a 4.2 m thick, very dense cohesionless deposit was intercepted. This was underlain by a hard silty clay deposit of 4.2 m thickness.
- A till shale complex, generally a precursor to bedrock, was intercepted in BH 16-3 as the terminal deposit, following the silty clay deposit.
- Based on the four (4) monitoring wells installed for long-term monitoring of ground water levels, BH16-1 and BH16-2 were found to be dry, whilst BH16-6 recorded a water table with time and was monitored at 17.3 m below ground level. BH 16-3 where the piezometer screen was predominantly in the dense cohesionless deposit recorded a maximum groundwater level of 1.5 m below ground surface and thus showed sub-artesian conditions. Borehole cave-in was observed in boreholes BH16-4 and BH16-5 at 2.7 m depth on completion of drilling. These depths are within the silty clay till layer.

Table 5-1: Preliminary Sub-Surface Geotechnical Model

Soil Type	**Consistency or Compactness Condition**	**Unit Weight (kN/m^3)**	**Effective Stress Parameters (c' = 0), Φ' (degrees)**	**Total Stress Parameters, S_u (kPa)**	**Drained Stiffness Parameters* E'(MPa); ν'**
Cohesionless Fill	Loose	18	30	NA	10;0.2
Cohesive Fill	Firm	18	28	25	8;0.4
Upper Silty Clay	Firm	19	29	40	10;0.35
Upper Silty Clay	Very stiff	20	31	120	20;0.3
Silty Clay Till	Very stiff	21	32	150	25;0.3
Silty Sand	Dense	22	36	NA	125;0.25

Soil Type	Consistency or Compactness Condition	Unit Weight (kN/m^3)	Effective Stress Parameters (c' = 0), Φ' (degrees)	Total Stress Parameters, S_u (kPa)	Drained Stiffness Parameters* E'(MPa); ν'
Silty Clay	Hard	22	33	150	78;0.22
Till shale Complex	Hard	23	38	200	100;0.2

Partly based on : Schnaid, F., *In-situ Testing in Geomechanics*, Taylor & Francis (2009)

5.2.2 GROUND MOTION PARAMETERS

Based on the borehole information and our review of the general subsurface conditions in the area, the subject site for the proposed structures can be classified as 'Class D' for seismic site response according to Table 4.1 of CHBDC S6-14.

The peak ground acceleration for the City of Toronto, is 0.099g (NBC 2015 Table 3C). Accordingly, for pseudo-static analysis, the site adjusted horizontal peak acceleration coefficient, k_{h0} is 0.13 for non-yielding walls.

- a k_h value of 0.065 (k_h = 0.5*k_{h0}; CSA S6-14) for lateral earth pressure calculations with the M-O formulation for walls that can move laterally

5.2.3 FROST DEPTH/SUSCEPTIBILITY

The frost depth for the project site is 1.2 m. The soils at the proposed culvert site have low to moderate frost susceptibility based on the MTO Frost Classification.

5.3 PRELIMINARY FOUNDATION RECOMMENDATIONS

5.3.1 PROPOSED PEDESTRIAN BRIDGE FOUNDATIONS: ABUTMENTS (BH16-1 AND BH16-6 REFER)

Due to significant fill thicknesses, and/or inadequate integrity of the upper horizon of the underlying native silty clay deposit, shallow foundations are not recommended for the bridge abutments.

Driven steel H piles, caissons, CFA piling and helical piles founded within the silty clay till can be considered as alternative foundation types at the abutment locations (assuming the subsurface profile at Dale Ave follows a similar stratigraphy pattern to that found on Bloor Street. This needs to be confirmed during detailed design). In view of the expected lighter loads for a pedestrian bridge several alternative foundation types including ones not conventional for mainstream bridges are considered as geotechnical foundations subject to meeting structural requirements. For example, a multi-helix helical pile (with 30 cm, 25 cm and 20 cm helices) embedded about 4 m to 5 m into the silty clay till (say El. 100 m or below) will have an SLS capacity of about 130 kN and a factored ULS of 150 kN in axial compression. For a CFA pile of 0.5 m diameter, based on BH16-6, a SLS capacity of 250 kN (factored ULS of 300 kN) would be available at El. 100 m or below. As another alternative, with a 900 mm caisson at a tip elevation of El. 100 m or below in similar deposit to BH16-6, a SLS capacity of 560 kN (factored ULS of 670 kN) can be mobilized. At the same elevation, for a steel HP-310x110 driven pile, a SLS capacity of 225 kN (factored ULS of 275 kN) can be mobilized. For higher pile resistances, much deeper piling would be required. For example, if the native sub-surface geology of Borehole BH16-3 can be established below the penetrated depths in BH 16-6 during detailed design investigations, then the very dense cohesionless deposit below El. 85.5 would be a competent end-bearing layer for driven piles. A HP-310x110 driven to El. 83 m or below can carry a SLS of 800 kN with a factored ULS of 110 kN. Pile installation should be in accordance with OPSS 903. Pile driving should be monitored using the Hiley Formula (Standard Structural Drawing SS-103-11). The piles should be reinforced with flange plates as per OPSD 3000.100 or driving shoes such as Titus Standard “H” Bearing Pile Point design for protection during heavy driving. The ground vibration and noise issues may preclude the use of driven H-piles under the current noise by-laws in the City but down Rosedale Valley Road this may not pose an issue. A micro-pile option can also be considered. A drilled and grouted micro-pile option is discussed in Section 5.3.2.

5.3.2 PROPOSED PEDESTRIAN BRIDGE FOUNDATIONS: PIERS (BOREHOLES BH16-4, BH16-5 AND BH16-3 REFER)

The pier foundations would be subjected to greater loads compared to the abutments and by virtue of the inclination of the pier legs would likely be subjected to, in addition to vertical and horizontal loading, to bending moments. In addition, the proposed bridge cross-section is wider and the deck will be of concrete. These considerations would imply that the imposed loads would likely be greater than the loads on the existing pier foundations.

At the pier locations, based on the limited geotechnical information obtained from BHs 16-4 and 16-5 due to the constraints on the method of drilling, and assuming the subsurface profile to follow a trend similar to BH 16-6, the underlying silty clay till beneath the fill, can be considered as a founding stratum for spread footings as an option. For example, a horizontal footing, founded on horizontal ground, can resist a SLS applied pressure of 225 kPa with a total settlement not exceeding 25 mm and a differential settlement not exceeding 19 mm, under vertical concentric loading. However, this allowable pressure needs to be reduced to account for sloping ground and loading direction/inclination and any moment loading from the bridge pier. In addition, the impacts of all these loads (which are expected to be greater than the existing loading) need to be considered on the stability of the valley slope itself.

SPT N “100 Blow” material was intercepted in BH16-3 in the silty clay till/shale complex below El. 78.5 m. Subject to logistics of driving, steel HP 310x110 piles can be driven to refusal within this till/shale complex. Under conditions of refusal, a factored ULS of 1600 kN and SLS of 1400 kN in axial compression can be assumed for the HP 310x110 piles. Owing to the dense silt/sandy silt/silty sand deposit (4.2 m thick) and the underlying hard clay as intercepted in BH16-3, pile refusal can be expected in these deposits before reaching the till/shale complex. As such, conservative pile capacity estimates should be adopted assuming pile refusal in the hard clay at El. 81.0 or below. Based on this line of reasoning, it is recommended the following pile capacities be adopted for preliminary design, a factored ULS of 700 kN and a SLS of 575 kN. However, a deeper penetration of the H-Piles into the slope would be conducive to minimise potential loading effects on slope stability issues. In this respect, in order to facilitate deeper driving, it is recommended that the H-Piles be reinforced with driving shoes, should this option be the adopted.

It can be expected to intercept shale bedrock within a couple of metres following the till/shale complex. Subject to proving such conditions, grouted micro-piles socketed a minimum of 3m into sound shale can be designed for an allowable socket bond resistance of 750 kPa in axial compression. From a constructability point of view, this micro-pile option poses lesser limitations for installation from a slope face and can be installed deeper into the slope as well.

5.3.3 RECOMMENDED FOUNDATION OPTION FOR PEDESTRIAN BRIDGE

The recommended foundation option is based on a number of considerations:

-Geological (unknown nature of possible fill inclusions, potential for cobbles and boulders in glacial tills, groundwater issues)

-Constructability (accessibility for rigs and plant on slopes, construction footprint required including staging requirements)

-Environmental (noise, vibration, excavation spoil management)

-Contractual (minimize different technologies being used)

Based on the above criteria, and given that the proposed structure is a pedestrian bridge (lighter structure compared to a bridge for vehicles, the recommended foundation option is drilled and grouted micro-piles socketed into sound shale. In the detailed design, depths to sound shale should be established. Drilled and grouted micro-piles socketed a minimum of 3m into sound shale can be designed for an allowable socket bond resistance of 750 kPa in axial compression. At the abutment locations, the micro-piles should be load tested during construction as discussed in Section 5.3.5.

5.3.4 PROPOSED WIDENING OF PEDESTRIAN TUNNEL: (BH16-2 REFERS)

The vicinity of the existing pedestrian tunnel is heavily congested with services and utilities and the proposed widening is expected to take place to the west of existing centreline of the tunnel under partial road closure. The widened excavation will need temporary excavation support and this is likely to be provided with sheet piling. The ground conditions have been explored down to 12.8 m below the road grade at Bloor St in BH16-2. Based on the intercepted subsurface conditions, the native silty clay and the underlying silty clay till can give adequate passive toe support for an excavation that is not expected to be more than 5 m deep. Due to access constraints, BH16-2 was drilled further west of the proposed western widened limit of the pedestrian tunnel. Depending on the type of backfill that was used for the existing tunnel, e.g. drainage gravel or due to presence of construction debris, it may pose some limitations on the driveability of steel sheet piles. This could warrant, for example, pre-drilling staggered holes (say 100 mm dia) to facilitate sheet pile penetration. Perhaps, other temporary excavation support alternatives, such as, bored soldier piles and timber lagging or contiguous caisson wall with soldier piles and filler piles or excavation support/permanent lateral support such as secant pile wall could be options that may need to be considered at the detailed design stage. The use of construction material such as gravel as drainage backfill in connection with existing TTC subway tunnel construction and associated perched groundwater issues should also be considered as likely impacts during construction for the proposed tunnel widening.

A box type of culvert is recommended. Assuming the depth to the underside of the box culvert is about 5 m, based on BH16-2 findings, a minimum of 1 m sub-excavation would be required below the underside of the box footing elevation to remove fill material. Engineered OPSS Granular 'A" is recommended for replacement. Subject to this ground improvement, a SLS of 200 kPa and a factored ULS of 300 kPa can be used for preliminary design. Any sub-excavation depth should be factored into the lateral support design considerations for temporary excavation support.

5.3.5 PROOF LOAD TESTING

Any helical pile or micro-pile option adopted needs to be load tested in the field to at least 1.5 times the corresponding SLS load.

5.3.6 GEOTECHNICAL CONSIDERATIONS – LATERAL LOADING

Geotechnical considerations (pertaining to lateral capacity and lateral deformations) are addressed in the following with respect to lateral loading of piles:

- Lateral capacity of piles: the estimated undrained strength S_u (for cohesive soils) and passive earth pressure coefficient, K_p and the effective unit weight of soils (for cohesionless soils) address pile lateral capacity issues

- Lateral deformation of piles: the coefficient of horizontal subgrade reaction, k_s (via S_u for cohesive soils and via n_h for cohesionless soils) provide the input for lateral deformation analyses

Vertical piles can provide resistance to lateral loading and this geotechnical resistance can be enhanced by the use of batter piles. The geotechnical lateral resistance is greatly affected by the soil properties close to the ground level (about 10 pile diameters, Ref: *Piling Engineering*, Fleming, et al).

In cohesionless soils, the coefficient of horizontal subgrade reaction can be estimated from:

$k_s = n_h z / d$

where k_s = coefficient of horizontal subgrade reaction
n_h = coefficient related to soil density as given in **Table 5.2**
z = depth
d = pile width

Where the soil is primarily cohesive, the coefficient of horizontal subgrade reaction can be estimated from:

$k_s = 67 S_u / d$

where S_u = undrained shear strength shown as "S_u" in **Table 5.2**

Table 5-2 Geotechnical Parameters – Lateral Pile Resistance* (Based on BH16-6)

Location	**Elevation (m)**	**Soil**	**n_h (MN/m^3)**	**Su (kPa)**	**Kp**	**Buoyant Unit Weight (kN/m^3)**
South Abutment						
	114.9 to 112.2	Cohesionless Fill	1.0	na	2.5	8
	112.2 to 108.5	Cohesive Fill	na	30	na	9
	108.5 to 105.8	Firm Silty Clay	na	40	na	20
	105.8 to 104.4	Very Stiff Silty Clay	na	125	na	11
	104.4 to 94.5	Very Stiff Silty Clay Till	na	150	na	12
		Sound Shale	na	2500	na	26

*Note: The design water level should be assumed at the ground surface for lateral pile resistance considerations:

** na = not applicable

For preliminary design purposes, for ground conditions pertaining to BH16-6, the recommended horizontal resistances for HP 310x110 driven steel H-piles are as follows:

Horizontal Resistance at ULS = 110 kN/pile

Horizontal Resistance at SLS* = 50 kN/pile

* for a lateral displacement of 10 mm at the pile head.

Pile interaction effects should be considered in the design.

5.4 ABUTMENTS AND ASSOCIATED SUB-STRUCTURES

5.4.1 GENERAL

Lateral earth pressures that mobilize behind abutment walls and any associated wing walls/retaining walls depend on many factors. They are such as, the type of backfill material, the method of placement, the stiffness and the freedom of the walls to move, nature of drainage behind walls, type of soil behind backfill, slope geometry behind the walls and the magnitude of imposed surcharge including those during construction. Seismic loading must also be considered.

5.4.2 BEARING RESISTANCE

As discussed in Section 5.3.3, the abutments will be carried on piles. Based on BH16-6, the top 6.4 m consists of low SPT variable fill and unsuitable to support any sub-structure foundation. Since the depth of replacement is too deep, the sub-structure foundations can be carried, for example, on helical piles. Helical piles installed at or below El. 104 m within the silty clay till can resist a SLS of 70 kN and a factored ULS of 90 kN in axial compression.

5.4.3 LATERAL EARTH PRESSURE – STATIC LOADING

Backfill behind structures and retaining walls should consist of non-frost susceptible, free-draining granular materials in accordance with OPSD 3101.150. Free-draining backfill (Granular 'A' or Granular 'B' Type I or Type II, with less than 5% fines, i.e. 200 sieve). The provision of drain pipes and weep holes should prevent hydrostatic pressure build-up. For design purposes, the following unfactored static earth pressure parameters can be used (assuming wall friction is neglected, the back wall is vertical and the ground surface is not sloping up behind the wall):

Table 5-3: Unfactored Rankine Static Earth Pressure Coefficients

Wall Movement Condition	**Compacted Granular 'A' and Granular 'B' Type II** **Angle of Internal Friction, ϕ = 35°** **Unit Weight = 22 kN/m^3** **(Wall friction neglected)**		**Compacted Granular 'B' Type I** **Angle of Internal Friction, ϕ = 32°** **Unit Weight = 21 kN/m3** **(Wall friction neglected)**	
	Top Ground Surface Angle		Top Ground Surface Angle	
	Horizontal	2H:1V	Horizontal	2H:1V
Active Earth Pressure (K_A)	0.27	0.38	0.31	0.46
At-Rest Earth Pressure (K_O)	0.43	0.62	0.47	0.68
Passive Earth Pressure (K_P)	NA	-NA	NA	-NA

.Note: Passive earth pressures in front of the walls, in view of the sloping valley in front of the wall, should be disregarded.

CHBDC should be consulted to assess the minimum movements required before the adoption of active lateral pressures and if movements are found to be inadequate due to the restraints imposed by the superstructure and sub-structure elements, then at-rest earth pressures should be considered for design. A compaction surcharge of 12 kPa should be considered in the design (12 kPa decreasing to 0 kPa at a depth of 1.7 m) and other surcharge loadings, if relevant, should also be accounted for in the design.

5.4.4 LATERAL EARTH PRESSURE – SEISMIC LOADING

Yielding Walls:

Seismic (earthquake) loading should be taken into account in the design. These estimates are based on the Monobe-Okabe (M-O) pseudo-static method of analysis. The M-O method produces seismic loads that are more critical than the static loads that act prior to an earthquake.

The horizontal seismic coefficient, k_h, used in the calculation of the seismic active pressure coefficient, can be taken as, k_h=0.065. The seismic active earth pressure coefficient is also dependent on the vertical component of the earthquake acceleration coefficient, k_v, although the influence of k_v is estimated to be less than 10% on the seismic active earth pressure (Kramer, 1996) and can be neglected.

It should be noted that in the computation of seismic earth pressure coefficients, the wall back-face geometry, backfill slope and wall friction effects need to be addressed.

For design purposes, the following unfactored seismic lateral earth pressure parameters can be used (assuming wall friction is neglected, the back wall is vertical and the ground surface is horizontal in front of the toe):

Table 5-4: Unfactored Seismic Earth Pressure Coefficients

Wall Movement Condition	**Compacted Granular 'A' and Granular 'B' Type II** **Angle of Internal Friction, ϕ = 35°** **Unit Weight = 22 kN/m^3** **(Wall friction neglected)**		**Compacted Granular 'B' Type I** **Angle of Internal Friction, ϕ = 32°** **Unit Weight = 21 kN/m3** **(Wall friction neglected)**	
	Top Ground Surface Angle		Top Ground Surface Angle	
	Horizontal	2H:1V	Horizontal	2H:1V
Seismic Active Earth Pressure (K_{AE})	0.29	0.43	0.32	0.52
Seismic Passive Earth Pressure (K_{PE})	NA	NA	NA	NA

Non-yielding Walls:

When the wall movements are insufficient to mobilize the shear strength of the backfill soil, the limiting conditions of minimum active or maximum passive conditions cannot develop. The horizontal seismic coefficient, k_h, used in the calculation of the seismic pressure coefficient, should be taken as, k_h=0.19.

5.4.5 BACKFILL AND DRAINAGE

Positive drainage of the granular backfill should be provided with transverse drains and weep holes whilst OPSD 3101.150 and OPSD 3121.150 requirements should be met with respect to backfill, sub-drains and frost taper. Selection of compaction equipment should be compliant with OPSS 501. Minimum backfill placement requirements should conform to CHBDC. Erosion protection should be provided in front of the retaining walls in view of the valley slope.

Backfill and frost taper to the abutments should consist of Granular A or Granular B and placement should be in accordance with OPSS 902. Drainage should be provided as per OPSD 3102.100.

5.5 APPROACH EMBANKMENTS

According to the preliminary GA, no grade raise is indicated. However, a widened bridge cross-section is shown for the proposed reconstruction. Hence the abutment approaches need to be widened to match the existing, with benching provided with the existing back-slope. Any extension of the existing shallow retainment holding the existing approach fill, should be founded on compacted (minimum 98% of the material’s SPMDD) granular (OPSS 1010 Granular ‘A’) pad of minimum 1.0 m thickness by sub-excavating the existing loose fill. A minimum preload period of 1 month should be maintained post-construction before any asphalt paving or concreting is put over the extension. This minimum preload period is routine as part of any sound construction control. In the case of a concrete cover, the joint between any reinstatement of the existing paving and the extension should be reinforced with dowels to accommodate any differential movement, to mitigate any continuing settlement, should that occur with time.

Under the height of the widened approach fill (about 1.5 m width on each side) estimated to be less than 3 m in height (this fill loading will be much less than the pre-consolidation pressure of the native deposits) the resulting settlements would be negligible. A minimum 1 m thick granular pad mentioned above as the approach fill subgrade for the widening should not pose any stability issues. Further, a similar height approach fill exists at present and the additional construction activity is to widen the approach width without any grade raise. Ground conditions at the approaches should be inspected and approved by an engineer at the time of construction.

For the recommended foundation option, i.e. rock socketed micro-piles, and in the geological setting of the project, downdrag should not be an issue.

5.6 CONSTRUCTION CONSIDERATIONS

5.6.1 GENERAL

During construction, the contract Administrator should employ experienced geotechnical staff to observe construction activities to ensure geotechnical recommendations are carried out and includes earthworks, pile installations and testing.

5.6.2 EXCAVATIONS

All excavations, shoring and backfilling should be carried out in accordance with the Occupational Health and Safety Act (OHSA), as well as the following specifications.

OPSS 539 – Construction Specification for Temporary Protection Systems

OPSS 902 – Construction Specification for Excavating and Backfilling Structures.

In accordance with OHSA, the sub-soils intercepted in the boreholes drilled can be classified as follows:

Fill	Not steeper than 2H:1V
Upper Silty Clay	1.5H: 1V or shallower
Clayey Till	Not steeper than 1H: 1V above water or 2H: 1V below water table
Lower silty Clay	1.5H:1V or shallower
Silt/Sandy Silt/Silty Sand	3H:1V or shallower, without groundwater control

The above slopes are for short-term open excavations in initially horizontally levelled ground only and must be visually monitored especially when people are working inside. If the open cut excavations are carried out on the valley slope, then a site specific excavation stability assessment should be undertaken by a geotechnical engineer.

Excavations in the native soils should be possible using heavy equipment such as a hydraulic excavator and cobbles and boulders within the native till deposits and including debris within the fill cannot be ruled out.

Should shoring be undertaken, such system should be designed by a Professional Engineer, experienced in this type of work and such work should conform to OPSS 539 and performance level for protection system shall be three (3).

For shoring design, assume groundwater to be at the original ground surface.

Table 5-5: Recommended Unfactored Parameters for Temporary Shoring Design

Soil Type	**Unfactored Parameters for Temporary Shoring Design**			
	K_a	K_o	K_p	Bulk Unit Weight (kN/m^3)
Cohesionless Fill	0.33	0.5	NA	18
Cohesive Fill	0.36	0.53	NA	18
Upper Silty Clay Firm	0.33	0.5	3.0	19

Upper Silty Clay-Very stiff	0.31	0.47	3.25	20
Silty Clay Till- Very stiff	0.29	0.45	3.43	21
Sandy Silt -Dense	0.28	0.44	3.54	22
Lower Silty Clay-Hard	0.28	0.44	3.54	22
Till Shale Complex-Hard	0.26	0.41	3.87	23.0

5.6.3 GROUNDWATER

Significant groundwater ingress into excavations is not expected. The seepage should be able to be handled by gravity drainage and pumping from open sumps and on occasions more aggressive dewatering may be required especially for excavations into the silt/sandy silt/silty sand that showed some sub-artesian nature. Open cut excavations into these materials, subject to a geotechnical assessment if carried out on the valley slope, could require positive dewatering measures (e.g. vacuum well points) to stiffen the cut slopes during construction. These aspects should be 'red flagged' in the contract documents. Intensity of seepage depends on the weather (i.e. precipitation), time of construction (i.e. snow melt) and construction methodology employed by the Contractor.

5.6.4 SLOPE PROTECTION/EROSION CONTROL

This can be achieved by prompt seed and cover (OPSS 804) or sodding (OPSS 803). Rip-rap placed at 1H: 1V without an underlying geotextile will be stable.

Rip-rap protection should be provided in front of the abutments and outside the toe area of any retaining wall and should generally follow OPSD 810.010 and any specific recommendations in the drainage report.

5.6.5 RE-USE/DISPOSAL OF EXCAVATED MATERIAL

The excavated soil will not be suitable for any engineered construction. The investigation findings relating to disposal requirements of excavated soil is addressed in a separate report.

Management of excavated material should conform to OPSS 180.

CLOSURE

The “Limitations of Report” as presented in Appendix G are an integral part of this report.

SIGNATURES

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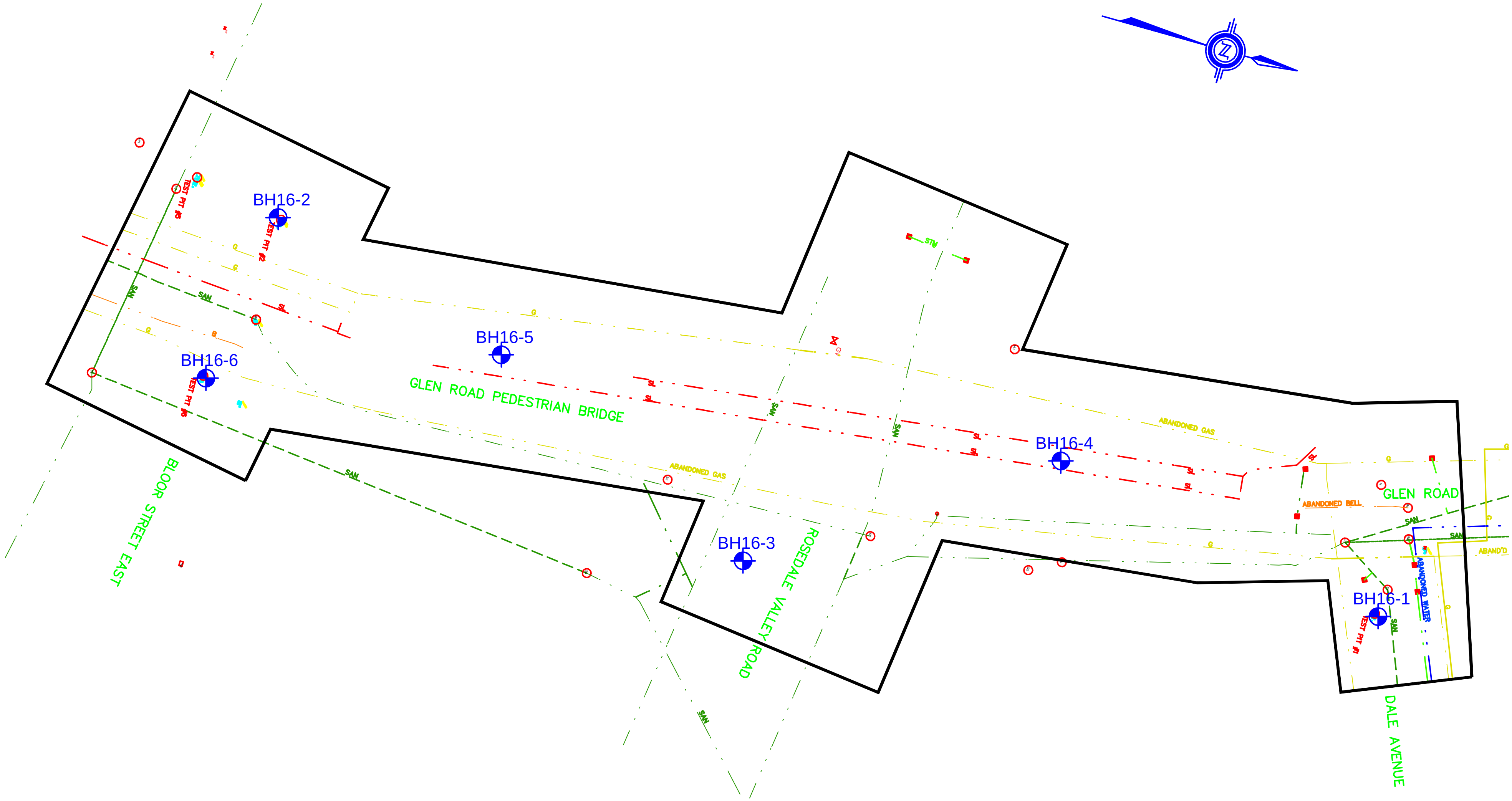
DRAWINGS: Borehole Location Plan
Elevational Profile of Stick Logs
Preliminary GA Drawing



LEGEND

⊕ Borehole Location

WSP

51 CONSTELLATION COURT
TORONTO, ONTARIO CANADA M9W 1K4
TEL.: 416-798-0065 | FAX: 416-798-0518 | WWW.WSP.COM

CLIENT: WSP-MMM GROUP LIMITED		PROJECT NO: 16M-01410-01	DRAWING NO: 1
TITLE: BOREHOLE LOCATION PLAN		DRAWN BY: ZMO	CHECKED BY: MP
PROJECT: GEOTECHNICAL INVESTIGATION - GLENN ROAD PEDESTRIAN BRIDGE EA STUDY		DATE: JUNE 28, 2017	SCALE: AS SHOWN
		ORIGINAL SIZE: LETTER	REV. # N/A

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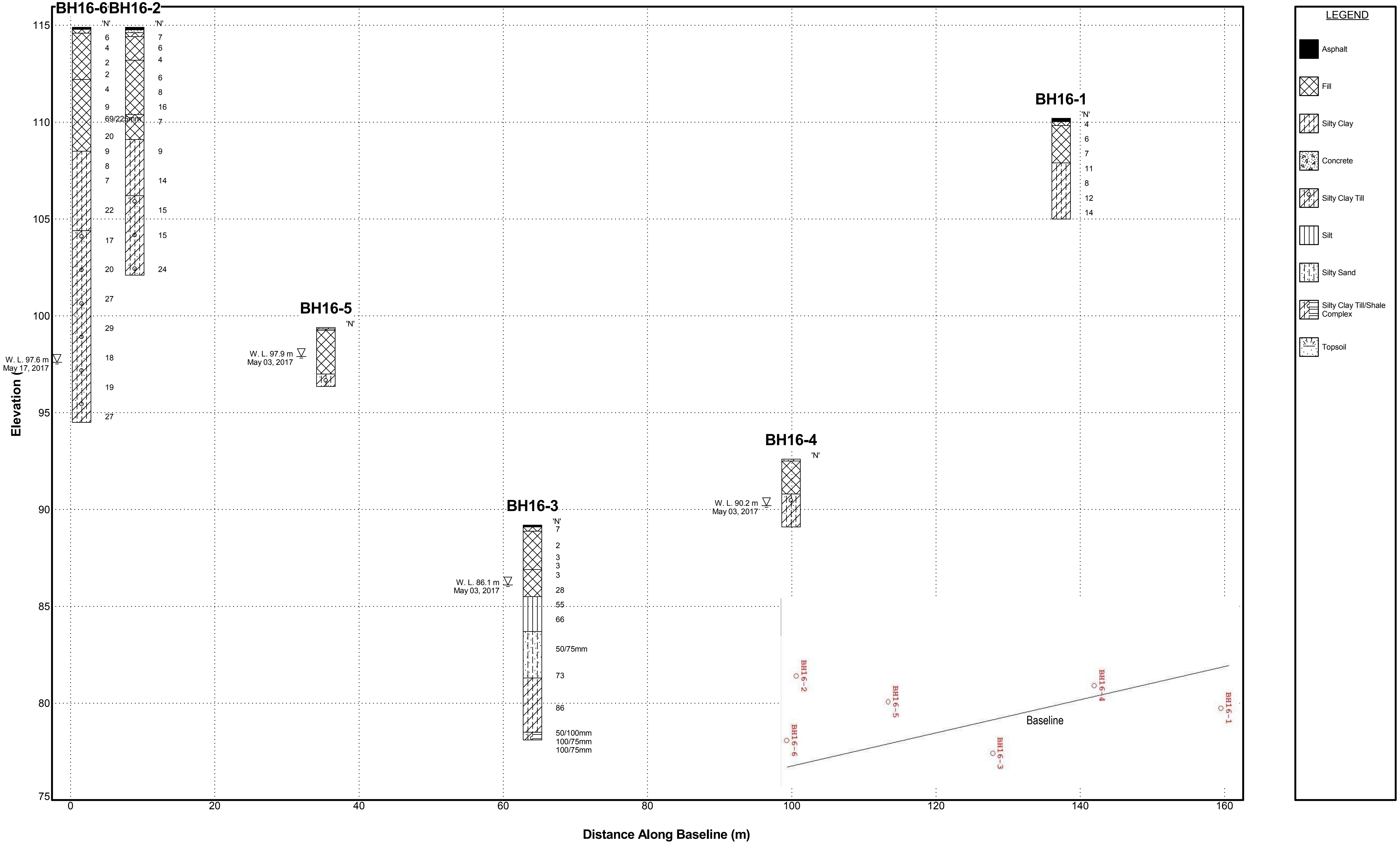


wsp

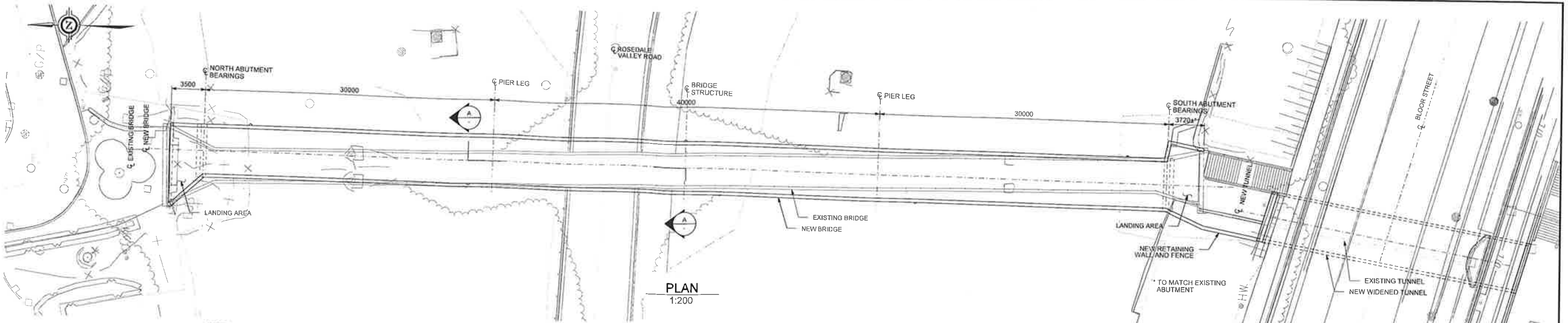
51 Constellation Court
Toronto, Ontario
6M9W 1K4

Elevational Profile of Stick Logs

DRAWING NO. 2

Job No.16M-01410-01

Date: July 6, 2017



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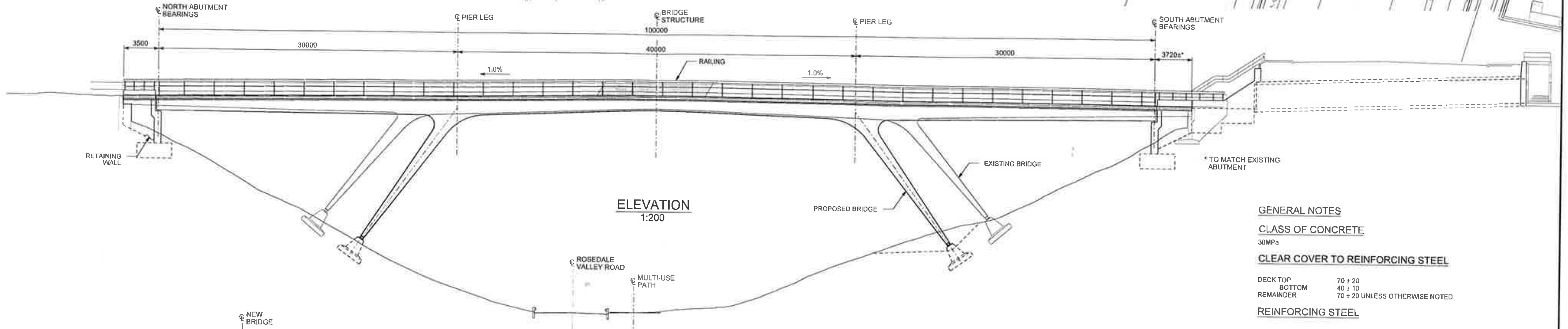
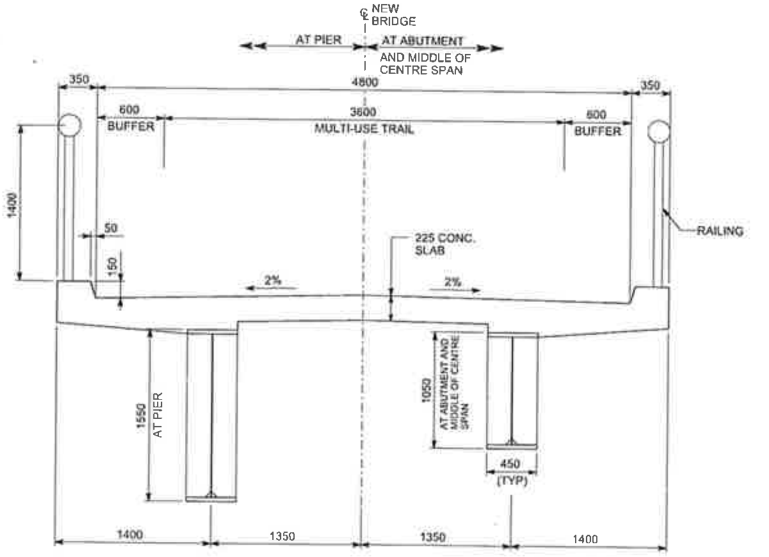
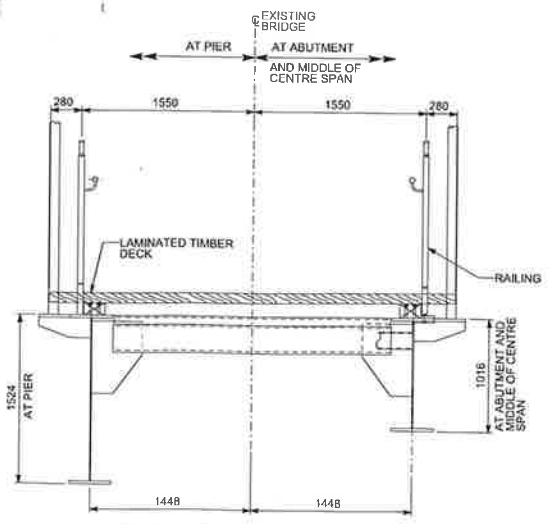


ELEVATION
1:200



PROPOSED BRIDGE SECTION (A)
1:30



EXISTING BRIDGE SECTION (A)
1:30

GENERAL NOTES

CLASS OF CONCRETE

30MPa

CLEAR COVER TO REINFORCING STEEL

DECK TOP	70 ± 20
BOTTOM	40 ± 10
REMAINDER	70 ± 20 UNLESS OTHERWISE NOTED

REINFORCING STEEL

1. REINFORCING STEEL SHALL BE GRADE 400W.
2. UNLESS SHOWN OTHERWISE, TENSION LAP SPLICES FOR PLAIN REINFORCING STEEL BARS SHALL BE CLASS B.
3. BAR HOOKS SHALL HAVE STANDARD HOOK DIMENSIONS USING MINIMUM BEND DIAMETERS, WHILE STIRRUPS AND TIES SHALL HAVE MINIMUM HOOK DIMENSIONS. ALL HOOKS SHALL BE IN ACCORDANCE WITH THE STRUCTURAL STANDARD DRAWING SS12-1 UNLESS INDICATED OTHERWISE.

CONSTRUCTION NOTES

THE CONTRACTOR SHALL VERIFY ALL DIMENSIONS, DETAILS AND ELEVATIONS OF EXISTING STRUCTURE THAT ARE RELEVANT TO THE WORK SHOWN ON THE DRAWINGS PRIOR TO COMMENCEMENT OF WORK. ANY DISCREPANCIES SHALL BE REPORTED TO THE CONTRACT ADMINISTRATOR AND THE PROPOSED ADJUSTMENT OF THE WORK REQUIRED TO MATCH THE EXISTING STRUCTURE SHALL BE SUBMITTED FOR APPROVAL.

DRAFT

DESIGN y2016det.dgn y2016pd.dgn y2016pd1.dgn

DIGITAL INFORMATION	No.	DATE	REVISIONS	INITIAL	SIGNED

TORONTO Engineering & Construction Services

MICHAEL D'ANOREA, P.ENG
EXECUTIVE DIRECTOR
ENGINEERING &
CONSTRUCTION SERVICES

FRANK CLARIZIO, P.ENG DIRECTOR,
ENGINEERING & CONSTRUCTION SERVICES
CAPITAL WORKS DELIVERY A

EASTON GORDEN, P.ENG.
ACTING MANAGER,
ENGINEERING & CONSTRUCTION SERVICES
LINEAR OF STRUCTURES

MMM GROUP
CONSULTANTS

GLEN ROAD SOUTH PEDESTRIAN BRIDGE

ALTERNATIVE 1: STEEL GIRDER WITH TWO INCLINED STEEL LEGS

PRELIMINARY GENERAL ARRANGEMENT

DESIGN	K. YUSEK	DRAWN	G. LI	CHECKED	M. NIE	CONTRACT No.
SCALE:	AS NOTED			DRAWING NUMBER		SHEET
DATE	SEPTEMBER, 2016					S1

S:\2016\32\3216026\f\projects\STRUCTURES_EXPRESSWAYS\249_GLEN_R_PED\MASTER\249-S2019-01.dgn

Appendix A: Record of Borehole Sheets

LOG OF BOREHOLE BH16-1

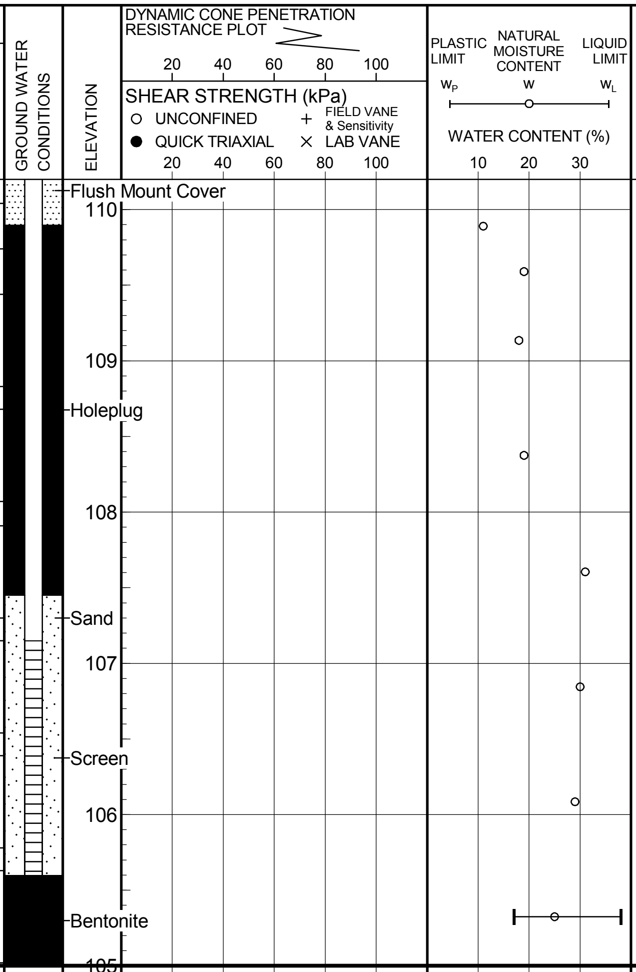
PROJECT: Glenn Road Pedestrian Bridge EA Study
REF. NO.: 16M-01410-01
CLIENT: WSP-MMM Group Limited
Method: Hollow Stem Auger
ENCL NO.: 1
PROJECT LOCATION: Toronto, Ontario
Diameter: 203 mm
ORIGINATED BY EY
DATUM: Geodetic
Date: May/10/2017 to May/10/2017
COMPILED BY MP
BH LOCATION: N 4836859 E 314873
CHECKED BY VW

SOIL PROFILE

(m) ELEV DEPTH	DESCRIPTION
110.2	Ground Surface
110.0 0.0	**ASPHALT:** 170mm
109.8 0.2	**GRANULAR FILL:** 200mm sand, trace gravel, brown, moist, loose.
0.4	**FILL:** silty clay to clayey silt, trace gravel, trace to some sand, brown to dark brown, moist, firm. ------------------------- sandy from 1.1m to 1.4m ------------------------ occasional sand seams
107.9 2.3	**SILTY CLAY:** trace sand, brown to grey, moist, stiff.
105.0 5.2	**END OF THE BOREHOLE**

SAMPLES

NUMBER	TYPE	"N" BLOWS 0.3 m	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1T	SS	4	
1B	SS		
2	SS	6	0 10 58 32
3	SS	7	
4	SS	11	
5	SS	8	
6	SS	12	
7	SS	14	0 3 40 57



POCKET PEN. (Cu) (kPa)

NATURAL UNIT WT (kN/m³)

Note:
1) 50mm dia. monitoring well was installed upon completion.

Water Level Readings:

Date	Depth (m)	Elevation (m)
May10-17	dry	
May17-17	dry	

GROUNDWATER ELEVATIONS
Measurement 1st ▽ 2nd ▼ 3rd ▼ 4th ▼

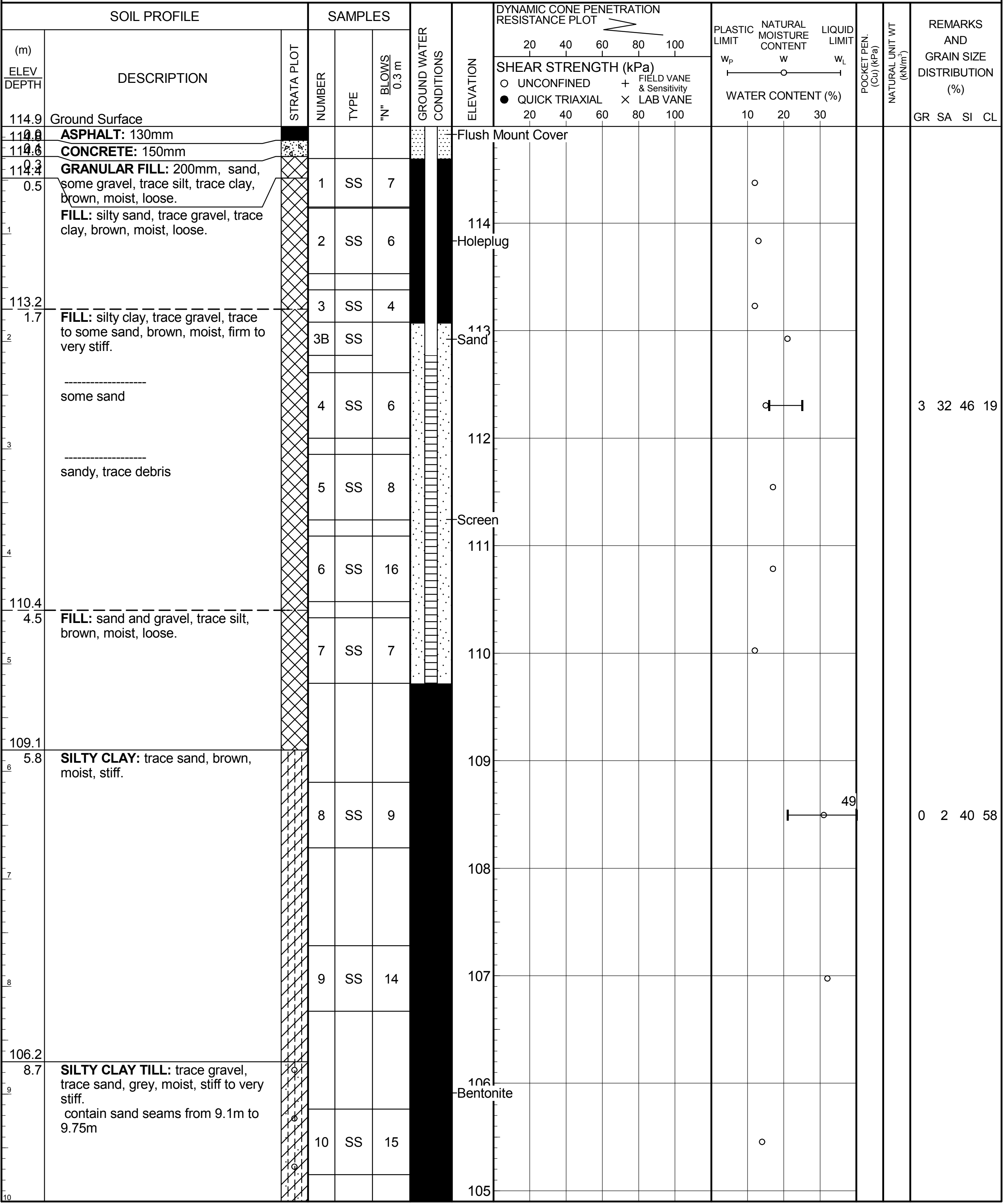
GRAPH NOTES
+ 3, × 3: Numbers refer to Sensitivity
○ ε=3% Strain at Failure

LOG OF BOREHOLE BH16-2

PROJECT: Glenn Road Pedestrian Bridge EA Study
CLIENT: WSP-MMM Group Limited
PROJECT LOCATION: Toronto, Ontario
DATUM: Geodetic
BH LOCATION: N 4836725 E 314863

Method: Hollow Stem Auger
Diameter: 203 mm
Date: May/10/2017 to May/10/2017

REF. NO.: 16M-01410-01
ENCL NO.: 2
ORIGINATED BY EY
COMPILED BY MP
CHECKED BY VW



SOIL PROFILE

(m) ELEV / DEPTH	DESCRIPTION
114.9	Ground Surface
114.8 / 0.0	**ASPHALT:** 130mm
114.6 / 0.1	**CONCRETE:** 150mm
114.4 / 0.3	**GRANULAR FILL:** 200mm, sand, some gravel, trace silt, trace clay, brown, moist, loose.
0.5	**FILL:** silty sand, trace gravel, trace clay, brown, moist, loose.
113.2 / 1.7	**FILL:** silty clay, trace gravel, trace to some sand, brown, moist, firm to very stiff. ------------------ some sand ------------------ sandy, trace debris
110.4 / 4.5	**FILL:** sand and gravel, trace silt, brown, moist, loose.
109.1 / 5.8	**SILTY CLAY:** trace sand, brown, moist, stiff.
106.2 / 8.7	**SILTY CLAY TILL:** trace gravel, trace sand, grey, moist, stiff to very stiff. contain sand seams from 9.1m to 9.75m

SAMPLES

NUMBER	TYPE	"N" BLOWS / 0.3 m	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	7	
2	SS	6	
3	SS	4	
3B	SS		
4	SS	6	3 32 46 19
5	SS	8	
6	SS	16	
7	SS	7	
8	SS	9	0 2 40 58
9	SS	14	
10	SS	15	

Continued Next Page

GROUNDWATER ELEVATIONS
Measurement 1st ▽ 2nd ▼ 3rd ▼ 4th ▼

GRAPH NOTES + 3, × 3: Numbers refer to Sensitivity ○ ε=3% Strain at Failure

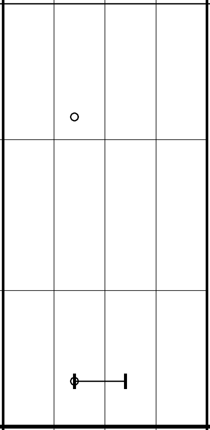
LOG OF BOREHOLE BH16-2

PROJECT: Glenn Road Pedestrian Bridge EA Study
CLIENT: WSP-MMM Group Limited
PROJECT LOCATION: Toronto, Ontario
DATUM: Geodetic
BH LOCATION: N 4836725 E 314863

Method: Hollow Stem Auger
Diameter: 203 mm
Date: May/10/2017 to May/10/2017

REF. NO.: 16M-01410-01
ENCL NO.: 2
ORIGINATED BY EY
COMPILED BY MP
CHECKED BY VW

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH (kPa) ○ UNCONFINED + FIELD VANE & Sensitivity ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT w LIQUID LIMIT W_L WATER CONTENT (%) 10 20 30	POCKET PEN. (Cu) (kPa)	NATURAL UNIT WT (kN/m^3)	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
(m) ELEV DEPTH	DESCRIPTION Continued	NUMBER	TYPE	"N" BLOWS/0.3 m							
	SILTY CLAY TILL: trace gravel, trace sand, grey, moist, stiff to very stiff.(Continued)										Auger grinding
		11	SS	15		104					
11											
12						103					
		12	SS	24							2 30 46 22
102.1 12.8	**END OF THE BOREHOLE** Note: 1) 50mm dia. monitoring well was installed upon completion. Water Level Readings: Date Depth (m) Elevation (m) May10-17 dry May 17-17 dry										



GROUNDWATER ELEVATIONS
Measurement 1st ▽ 2nd ▼ 3rd ▼ 4th ▼

GRAPH NOTES + 3, × 3: Numbers refer to Sensitivity

○ ɛ=3% Strain at Failure

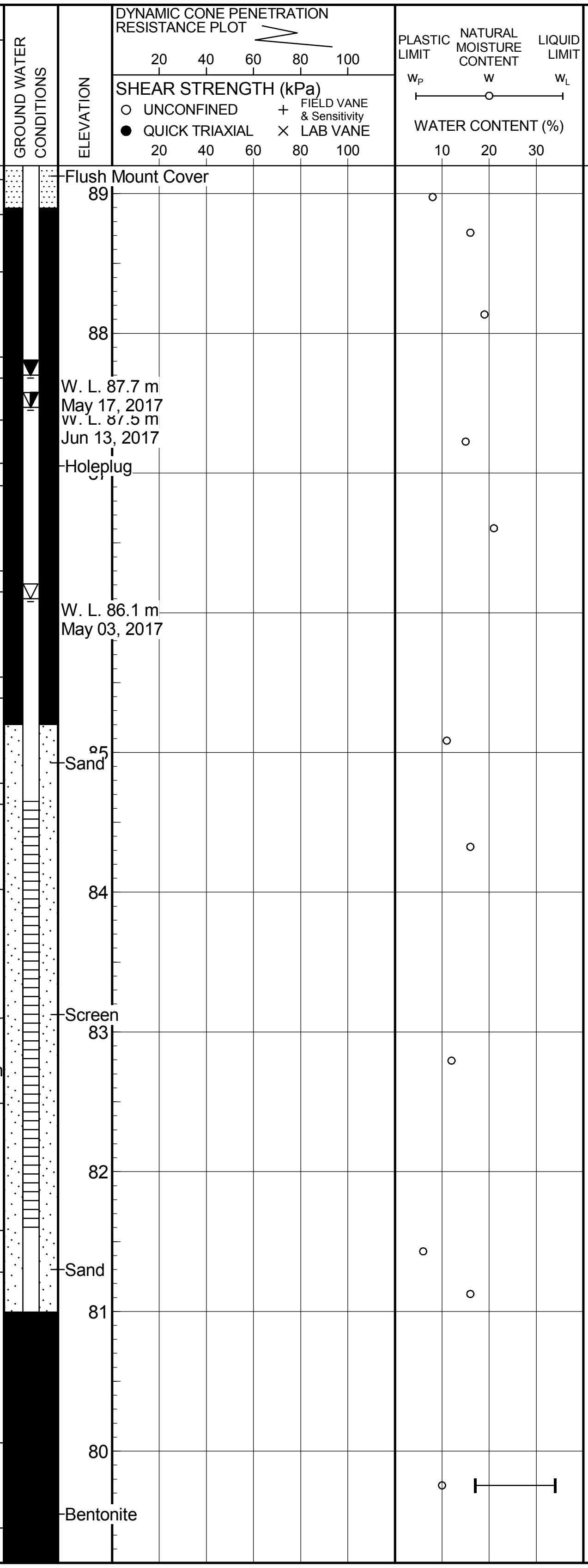
LOG OF BOREHOLE BH16-3

PROJECT: Glenn Road Pedestrian Bridge EA Study
CLIENT: WSP-MMM Group Limited
PROJECT LOCATION: Toronto, Ontario
DATUM: Geodetic
BH LOCATION: N 4836787 E 314887

Method: Hollow Stem Auger
Diameter: 203 mm
Date: May/03/2017 to May/03/2017

REF. NO.: 16M-01410-01
ENCL NO.: 3
ORIGINATED BY EY
COMPILED BY MP
CHECKED BY VW

(m) ELEV / DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	"N" BLOWS / 0.3 m	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
89.2	Ground Surface							
89.1 / 0.1	**ASPHALT:** 100mm							
	GRANULAR FILL: 200mm sand and gravel, brown, moist, loose	1T	SS	7				
88.9 / 0.3	**FILL:** silty sand, trace gravel, trace clay, brown to dark brown, moist, very loose.	1B	SS					
		2	SS	2	1	63	31	5
		3T	SS	3				
		3B	SS	3				
86.9 / 2.3	**FILL:** sandy silt, trace gravel, some clay, gas odor, brown, moist, very loose to compact.	4	SS	3				
		5	SS	28	No sample recovery			
85.5 / 3.7	**SILT TO SANDY SILT:** trace gravel, some clay, grey, moist, very dense.	6	SS	55				
		7	SS	66	1	30	56	12
83.7 / 5.5	**SILTY SAND:** trace gravel, grey, moist, very dense.	8	SS	50/ 75mm				
		9T	SS	73				
81.3 / 7.9	**SILTY CLAY :** trace sand, occasional silt seams, grey, moist, hard.	9B	SS					
		10	SS	86	0	7	62	31



POCKET PEN. (Cu) (kPa)

NATURAL UNIT WT (kN/m^3)

Continued Next Page

GROUNDWATER ELEVATIONS
Measurement 1st ▽ 2nd ▼ 3rd ▼ 4th ▼

GRAPH NOTES
$+^3$, $\times^3$: Numbers refer to Sensitivity
○ $^{\varepsilon}$=3% Strain at Failure

LOG OF BOREHOLE BH16-3

PROJECT: Glenn Road Pedestrian Bridge EA Study		REF. NO.: 16M-01410-01
CLIENT: WSP-MMM Group Limited	Method: Hollow Stem Auger	ENCL NO.: 3
PROJECT LOCATION: Toronto, Ontario	Diameter: 203 mm	ORIGINATED BY EY
DATUM: Geodetic	Date: May/03/2017 to May/03/2017	COMPILED BY MP
BH LOCATION: N 4836787 E 314887		CHECKED BY VW

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH (kPa) ○ UNCONFINED + FIELD VANE & Sensitivity ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L WATER CONTENT (%) 10 20 30	POCKET PEN. (Cu) (kPa)	NATURAL UNIT WT (kN/m³)	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
(m) ELEV DEPTH	DESCRIPTION Continued	STRATA PLOT	NUMBER	TYPE	"N" BLOWS / 0.3 m							
	SILTY CLAY : trace sand, occasional silt seams, grey, moist, hard.(Continued)						79					
78.5 10.7	**SILTY CLAY TILL /SHALE COMPLEX:** some shale fragments, grey, moist, hard.		11	SS	50/ 100mm							
			12	SS	100/ 5mm							
78.1 11.1			13	SS	100/ 75mm							spoon bouncing
	END OF THE BOREHOLE											

11

Notes:
1) Borehole was open upon completion.
2) Borehole water level was at 3.1m upon completion of drilling.
3) 50mm dia. monitoring well was installed upon completion.

Water Level Readings:

Date	Depth (m)	Elevation (m)
May3-17	3.1	86.1
May17-17	1.5	87.7
June13-17	1.7	87.47

WSP-SOIL-ROCK-MAY-29-2017.GLB
WSP SOIL LOG 1000016M-01410-01 LOG.GPJ 7/14/17

GROUNDWATER ELEVATIONS

Measurement 1st ▽ 2nd ▼ 3rd ▼ 4th ▼

GRAPH NOTES $+^3, \times^3$: Numbers refer to Sensitivity ○ ε=3% Strain at Failure

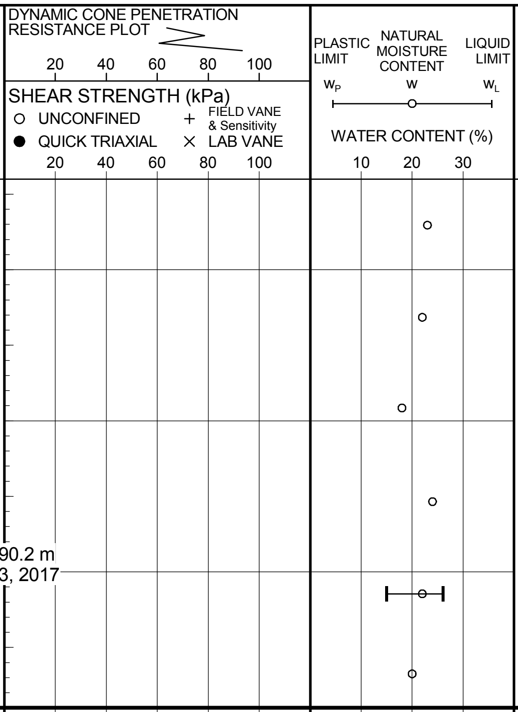
LOG OF BOREHOLE BH16-4

PROJECT: Glenn Road Pedestrian Bridge EA Study
CLIENT: WSP-MMM Group Limited
PROJECT LOCATION: Toronto, Ontario
DATUM: Geodetic
BH LOCATION: N 4836819 E 314866

Method: Pionjar
Diameter: 51 mm
Date: May/03/2017 to May/03/2017

REF. NO.: 16M-01410-01
ENCL NO.: 4
ORIGINATED BY EY
COMPILED BY MP
CHECKED BY VW

SOIL PROFILE			SAMPLES							
(m) ELEV DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" BLOWS/0.3 m	GROUND WATER CONDITIONS	ELEVATION	POCKET PEN. (Cu) (kPa)	NATURAL UNIT WT (kN/m³)	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
92.6	Ground Surface									
92.5 0.1	**TOPSOIL:** 100mm									
	FILL: silty clay, some topsoil, trace gravel, trace sand, trace rootlets, brown to darkish brown, moist. ------------------- stiff to very stiff		1	SS			92			
			2	SS				100		
90.8			3	SS			91	100		
1.8	**SILTY CLAY TILL:** trace gravel, some sand, brown, moist. stiff to very stiff		4	SS		W. L. 90.2 m May 03, 2017		113		
			5	SS						2 18 58 22
89.1	------------------- occasional sand seams, occasional oxidized, moist to wet		6	SS						
3.5	**END OF THE BOREHOLE** Notes: 1) Borehole caved-in at 2.7m and water level was at 2.4 m upon completion of drilling. 2) SPT blow counts were not possible due to use of pionjar drilling method.									



GROUNDWATER ELEVATIONS

Measurement 1st ▽ 2nd ▼ 3rd ▼ 4th ▼

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

○ ε=3% Strain at Failure

LOG OF BOREHOLE BH16-5

PROJECT: Glenn Road Pedestrian Bridge EA Study
CLIENT: WSP-MMM Group Limited
PROJECT LOCATION: Toronto, Ontario
DATUM: Geodetic
BH LOCATION: N 4836754 E 314871

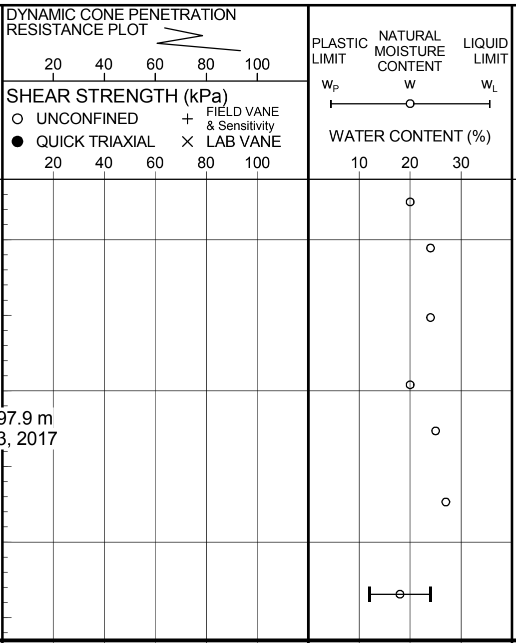
Method: Pionjar
Diameter: 51 mm
Date: May/03/2017 to May/03/2017

REF. NO.: 16M-01410-01
ENCL NO.: 5
ORIGINATED BY EY
COMPILED BY MP
CHECKED BY VW

(m) ELEV / DEPTH	DESCRIPTION	SAMPLES NUMBER	TYPE	"N" BLOWS / 0.3 m	GROUND WATER CONDITIONS	ELEVATION	POCKET PEN. (Cu) (kPa)	NATURAL UNIT WT (kN/m³)	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
99.4	Ground Surface								
99.3 / 0.1	**TOPSOIL:** 75mm	1T	SS						
99.3 / 0.2	**FILL:** sand, some silt, brown, moist.								
	FILL: silty clay, trace gravel, trace sand, trace rootlets, brown, moist, stiff. ------------------- occasional oxidized from 0.6m to 1.2m	1B	SS			99			
		2	SS				75		
		3T	SS		∇ W. L. 97.9 m May 03, 2017	98			
	------------------------ ocassional sandy silt seams,	3B	SS						
		4	SS						
97.0 / 2.4	**SILTY CLAY TILL:** trace gravel, occasional sand seams, grey, moist, stiff to very stiff.	5	SS			97	100		1 20 48 31
96.4 / 3.1	**END OF THE BOREHOLE**								

Notes:
1) Borehole caved-in at 2.7m and water level was at 1.5m upon completion of drilling.
2) SPT blow counts were not possible due to use of pionjar drilling method.

NP



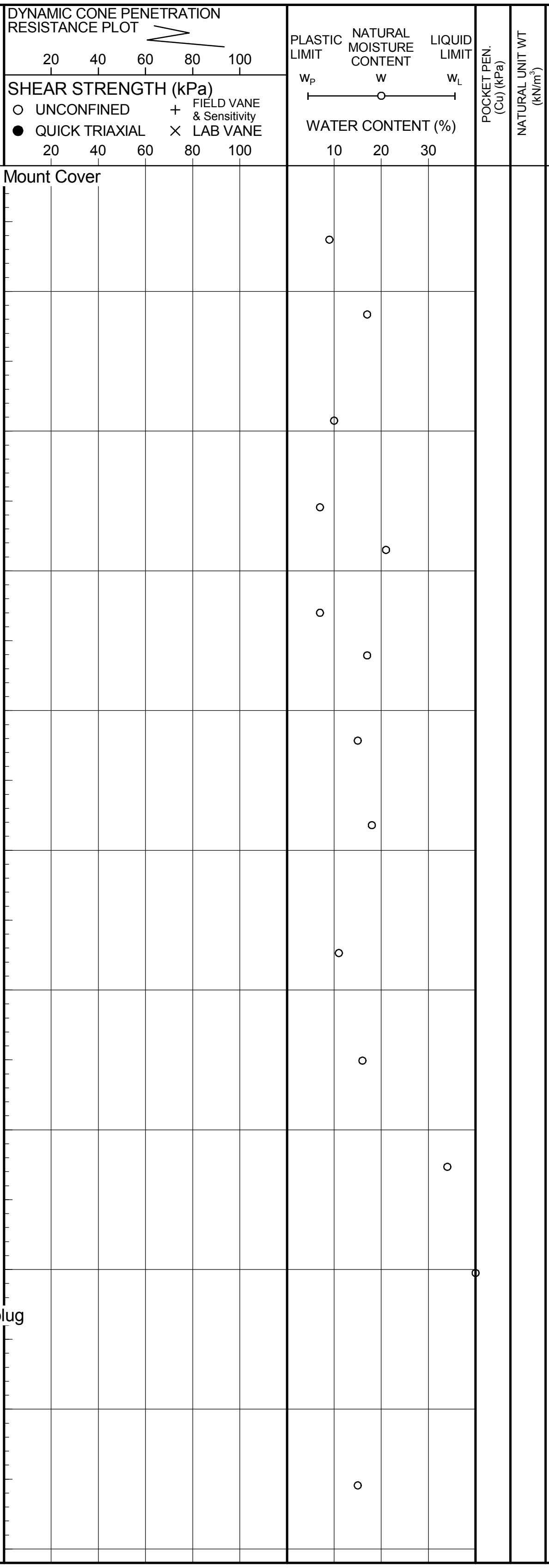
GROUNDWATER ELEVATIONS
Measurement 1st ∇ 2nd ▼ 3rd ∇ 4th ∇

GRAPH NOTES + 3, × 3: Numbers refer to Sensitivity
○ ε=3% Strain at Failure

LOG OF BOREHOLE BH16-6

PROJECT: Glenn Road Pedestrian Bridge EA Study
REF. NO.: 16M-01410-01
CLIENT: WSP-MMM Group Limited
Method: Hollow Stem Auger
ENCL NO.: 6
PROJECT LOCATION: Toronto, Ontario
Diameter: 203 mm
ORIGINATED BY EY
DATUM: Geodetic
Date: May/11/2017 to May/11/2017
COMPILED BY MP
BH LOCATION: N 4836722 E 314883
CHECKED BY VW

SOIL PROFILE (m) ELEV DEPTH	DESCRIPTION	SAMPLES NUMBER	TYPE	"N" BLOWS 0.3 m	GROUND WATER CONDITIONS	ELEVATION	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
114.9	Ground Surface						
114.8 0.0	**ASPHALT:** 110mm				Flush Mount Cover		
114.6 0.1	**GRANULAR FILL:** 190mm sand, trace gravel, brown, moist, loose.						
0.3	**FILL:** sand, trace gravel, trace to some silt, trace clay, brown, moist, very loose to loose.	1	SS	6			
1	------------------------- silty clay, trace gravel from 0.6m to 0.75m -------------------------	2	SS	4		114	
2		3	SS	2		113	3 78 16 3
		4T	SS	2			
112.2 2.7	**FILL:** silty clay, trace gravel, trace sand, brown, moist, firm to very stiff.	4B	SS			112	
3		5T	SS	4			
		5B	SS				
4	------------------------- occasional sand seams	6	SS	9		111	
	------------------------- asphalt fragments, trace sand	7T	SS	69/ 225mm			
5		7B	SS			110	
	------------------------- occasional sand seams	8	SS	20			
6	------------------------- contain rootlets					109	
108.5 6.4	**SILTY CLAY:** trace gravel, trace sand, brown, moist, firm to very stiff.	9	SS	9			
7		10	SS	8		108	
8		11	SS	7		107	
					Holeplug		
9						106	
		12	SS	22			
10						105	



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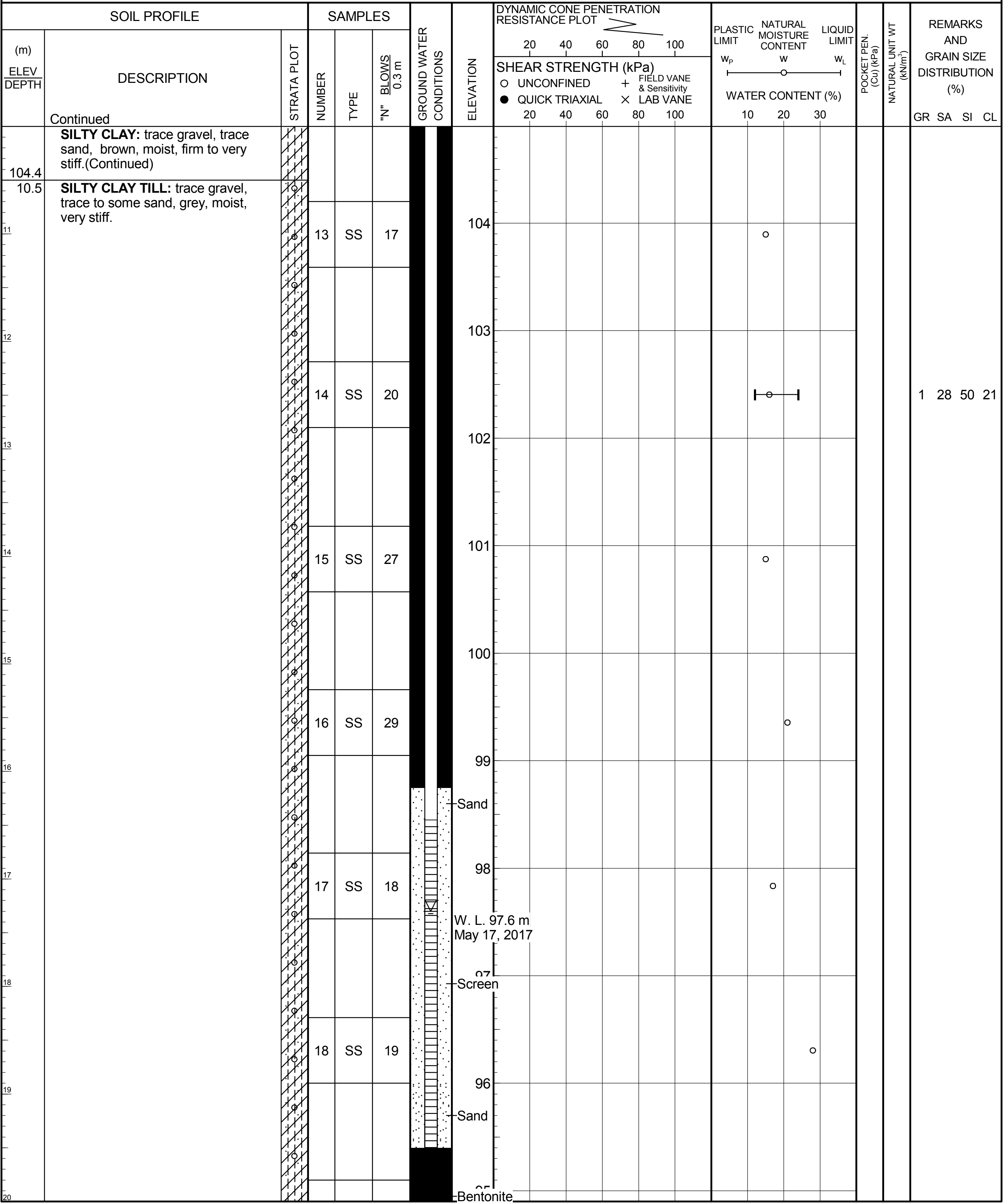
GROUNDWATER ELEVATIONS

Measurement 1st ▽ 2nd ▼ 3rd ▼ 4th ▼

GRAPH NOTES $+^3, \times^3$: Numbers refer to Sensitivity ○ ε=3% Strain at Failure

LOG OF BOREHOLE BH16-6

PROJECT: Glenn Road Pedestrian Bridge EA Study		REF. NO.: 16M-01410-01
CLIENT: WSP-MMM Group Limited	Method: Hollow Stem Auger	ENCL NO.: 6
PROJECT LOCATION: Toronto, Ontario	Diameter: 203 mm	ORIGINATED BY EY
DATUM: Geodetic	Date: May/11/2017 to May/11/2017	COMPILED BY MP
BH LOCATION: N 4836722 E 314883		CHECKED BY VW



ELEV DEPTH (m)	DESCRIPTION	NUMBER	TYPE	"N" BLOWS/0.3 m	GR SA SI CL
	SILTY CLAY: trace gravel, trace sand, brown, moist, firm to very stiff.(Continued)				
104.4 10.5	**SILTY CLAY TILL:** trace gravel, trace to some sand, grey, moist, very stiff.	13	SS	17	
		14	SS	20	1 28 50 21
		15	SS	27	
		16	SS	29	
		17	SS	18	
		18	SS	19	

Continued Next Page

GROUNDWATER ELEVATIONS

Measurement 1st ▽ 2nd ▼ 3rd ▼ 4th ▼

GRAPH NOTES + 3, × 3: Numbers refer to Sensitivity ○ ε=3% Strain at Failure

LOG OF BOREHOLE BH16-6

PROJECT: Glenn Road Pedestrian Bridge EA Study		REF. NO.: 16M-01410-01
CLIENT: WSP-MMM Group Limited	Method: Hollow Stem Auger	ENCL NO.: 6
PROJECT LOCATION: Toronto, Ontario	Diameter: 203 mm	ORIGINATED BY EY
DATUM: Geodetic	Date: May/11/2017 to May/11/2017	COMPILED BY MP
BH LOCATION: N 4836722 E 314883		CHECKED BY VW

SOIL PROFILE			SAMPLES					DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH (kPa) ○ UNCONFINED + FIELD VANE & Sensitivity ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L WATER CONTENT (%) 10 20 30			REMARKS AND GRAIN SIZE DISTRIBUTION (%)
(m) ELEV DEPTH	DESCRIPTION Continued	STRATA PLOT	NUMBER	TYPE	"N" BLOWS/0.3 m	GROUND WATER CONDITIONS	ELEVATION			POCKET PEN. (Cu) (kPa)	NATURAL UNIT WT (kN/m³)	GR SA SI CL
94.5			19	SS	27				○			
20.4	**END OF THE BOREHOLE** Note: 1) 50mm dia. monitoring well was installation upon completion. Water Level Readings: Date Depth (m) Elevation (m) May11-17 dry May17-17 17.3 97.6											

GROUNDWATER ELEVATIONS

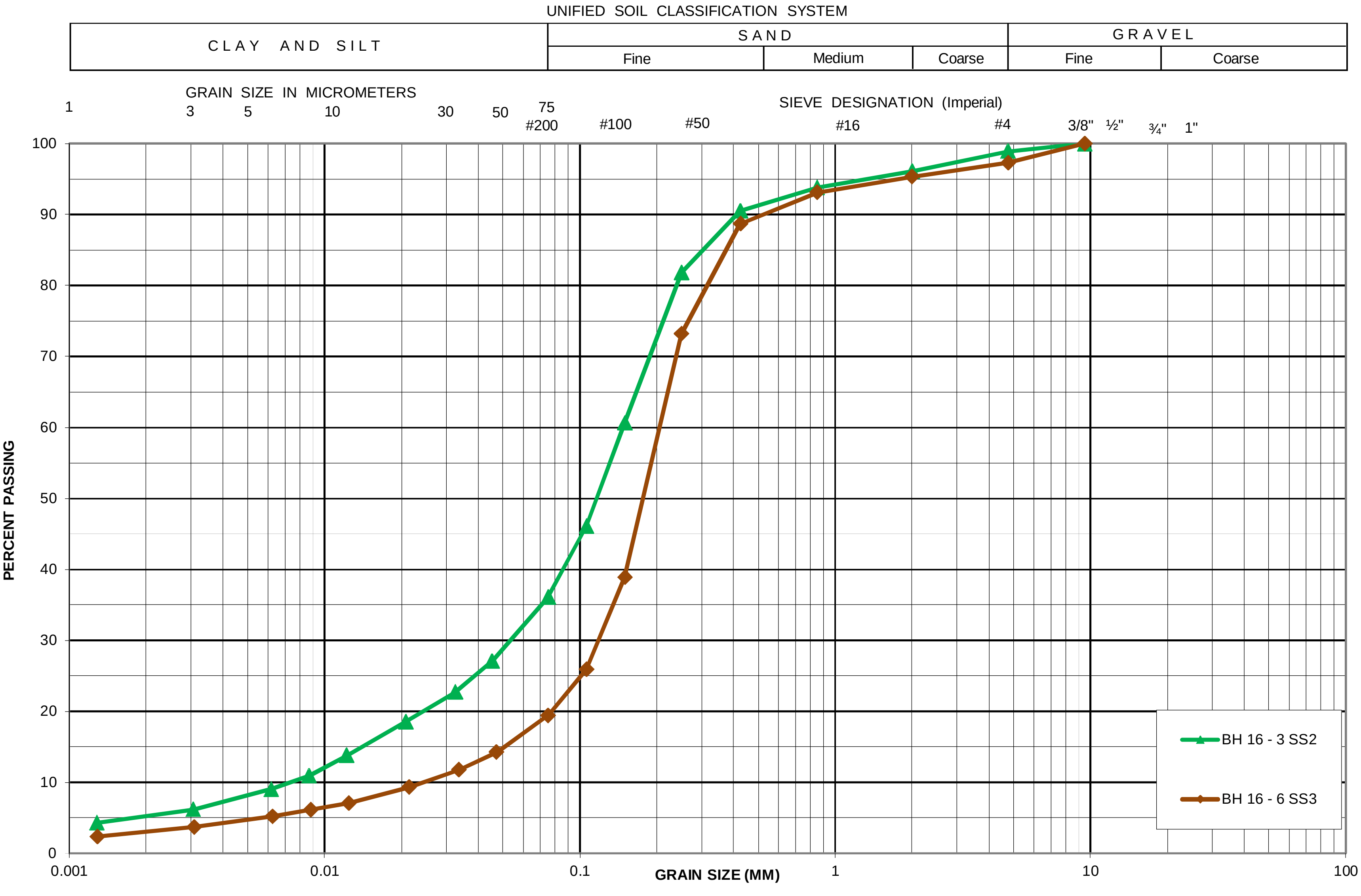
Measurement 1st ▽ 2nd ▼ 3rd ▼ 4th ▼

GRAPH NOTES

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

○ ε=3% Strain at Failure

Appendix B: Laboratory Test Results



WSP	GRAIN SIZE DISTRIBUTION **Silty Sand to Sand (Fill)**	Figure No. 1a Project No. 16M-01410-01 Date : June-04-2017

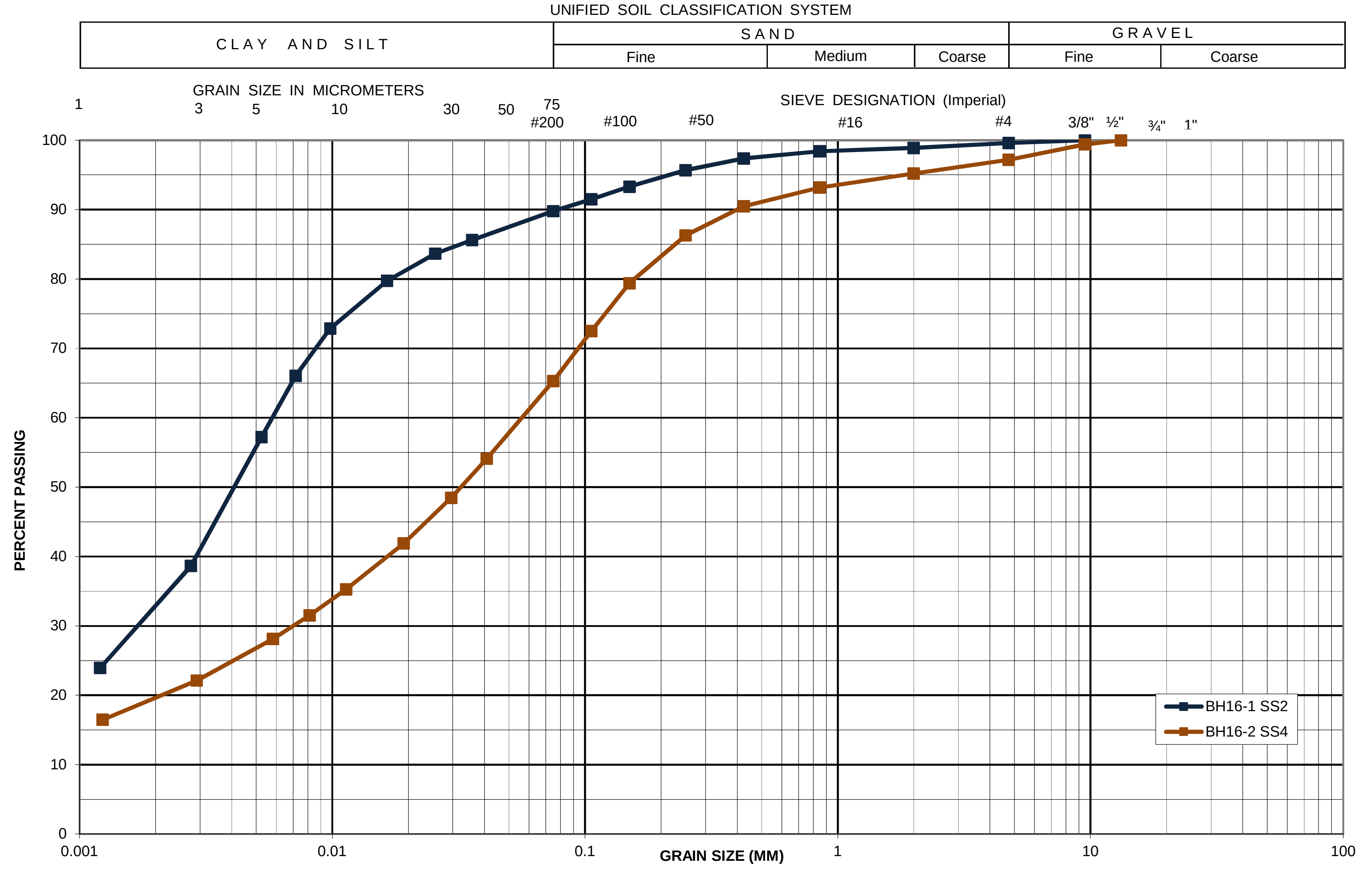
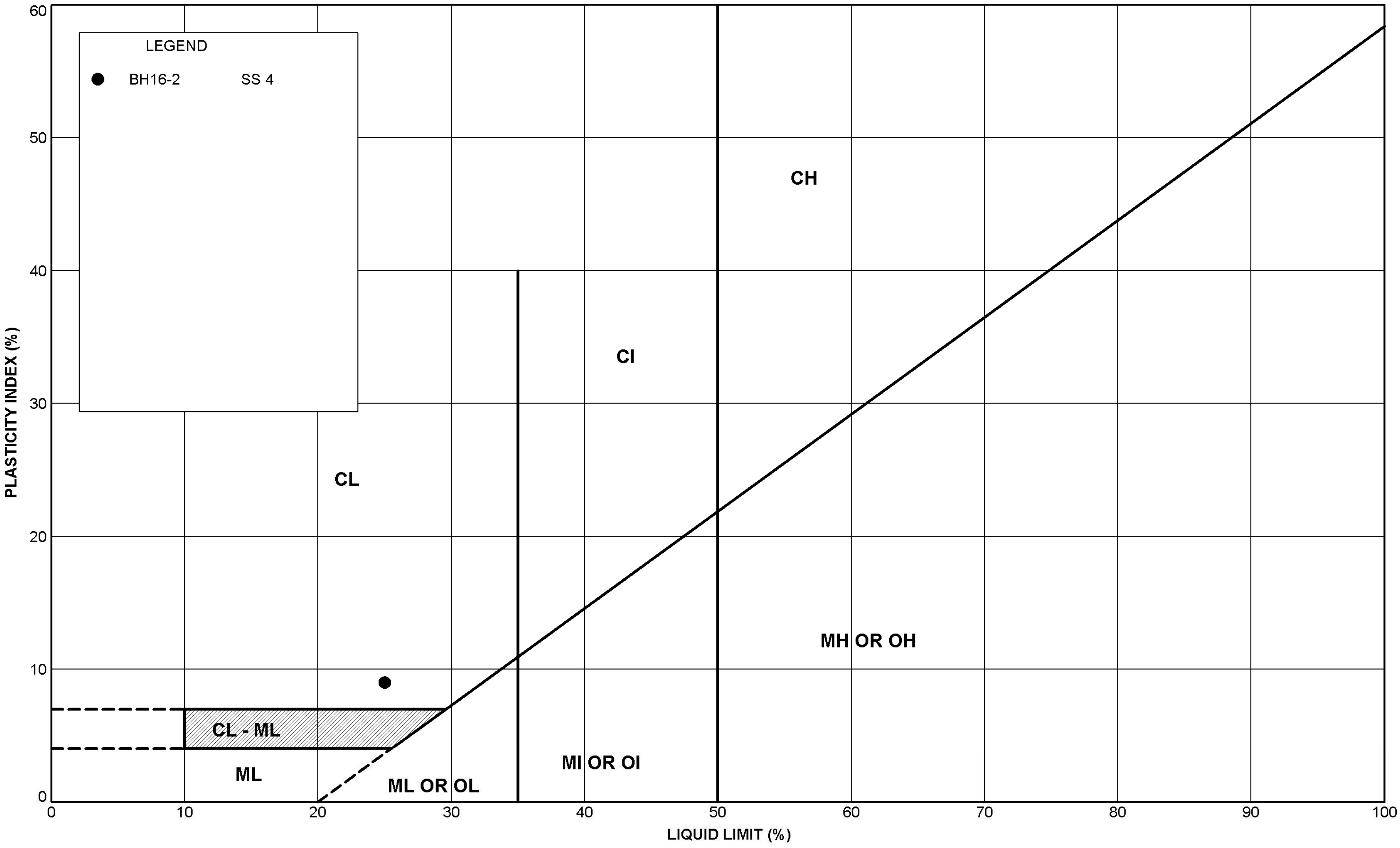


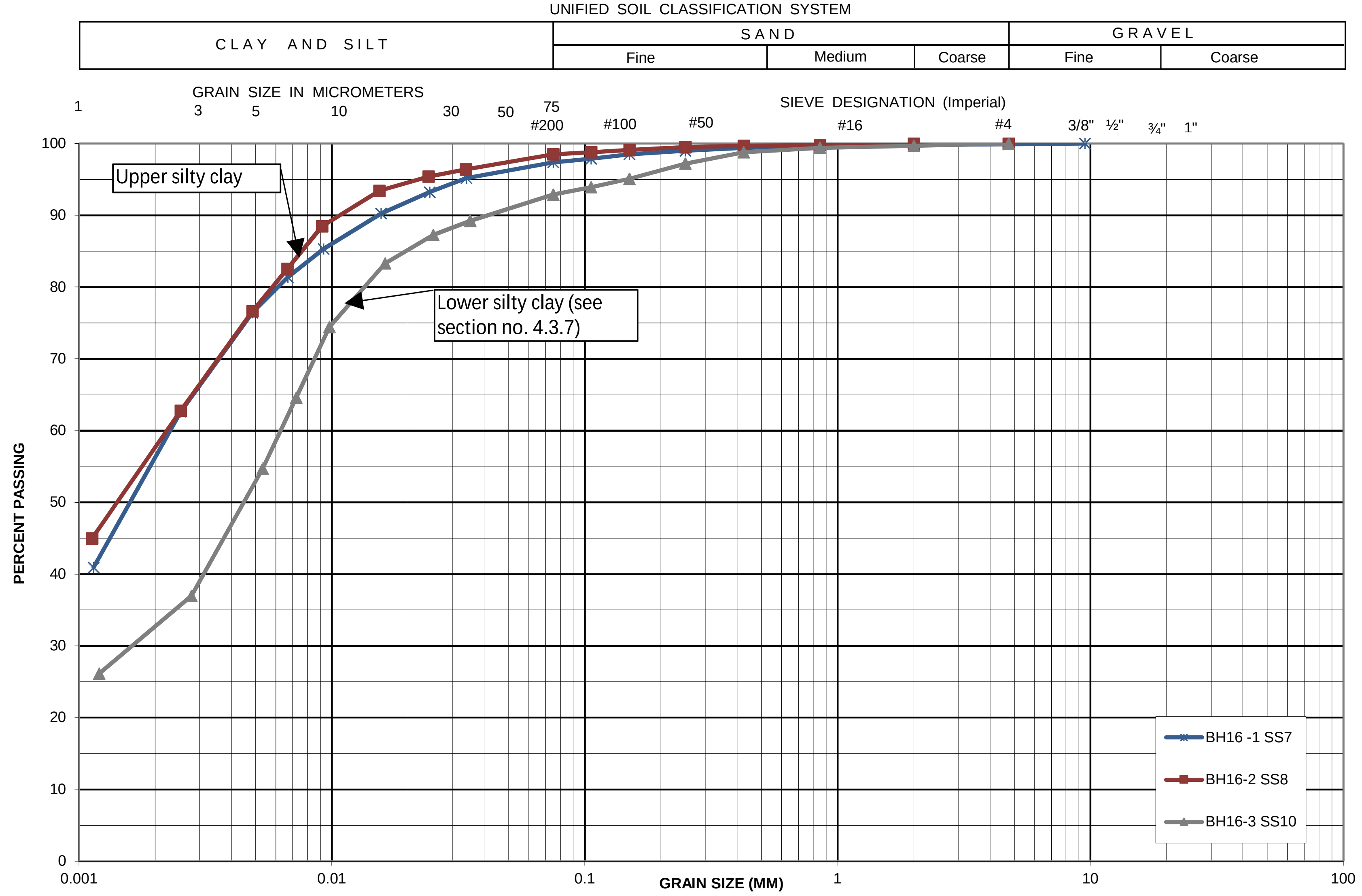
Figure No.	1b
Project No.	16M-01410-01
Date :	June-04-2017

GRAIN SIZE DISTRIBUTION

Silty Clay to Clayey Silt (Fill)



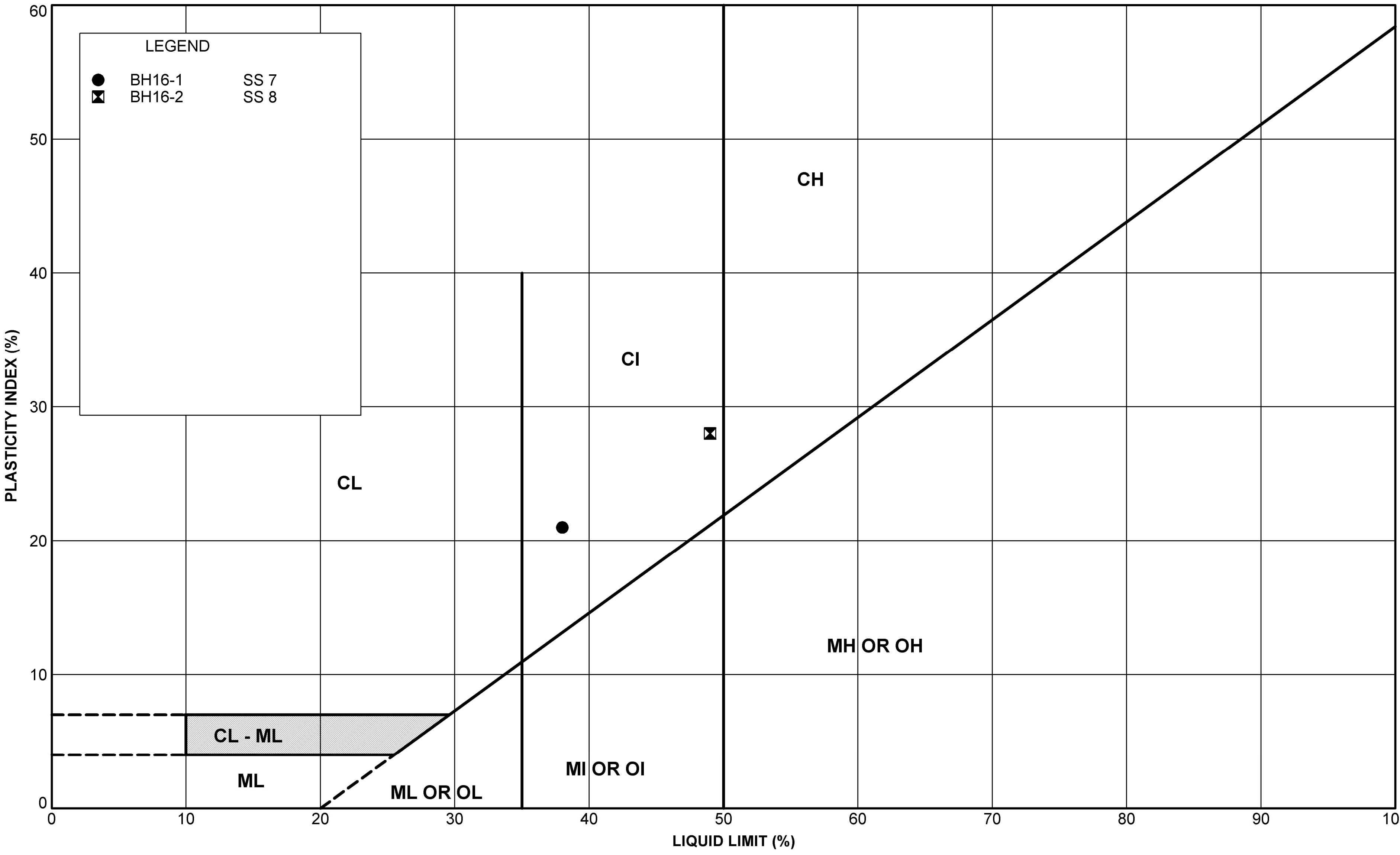
wsp	PLASTICITY CHART	FIGURE NO.	1c
		JOB NO.	16M-01410-01
		DATE	July 13, 2017



GRAIN SIZE DISTRIBUTION

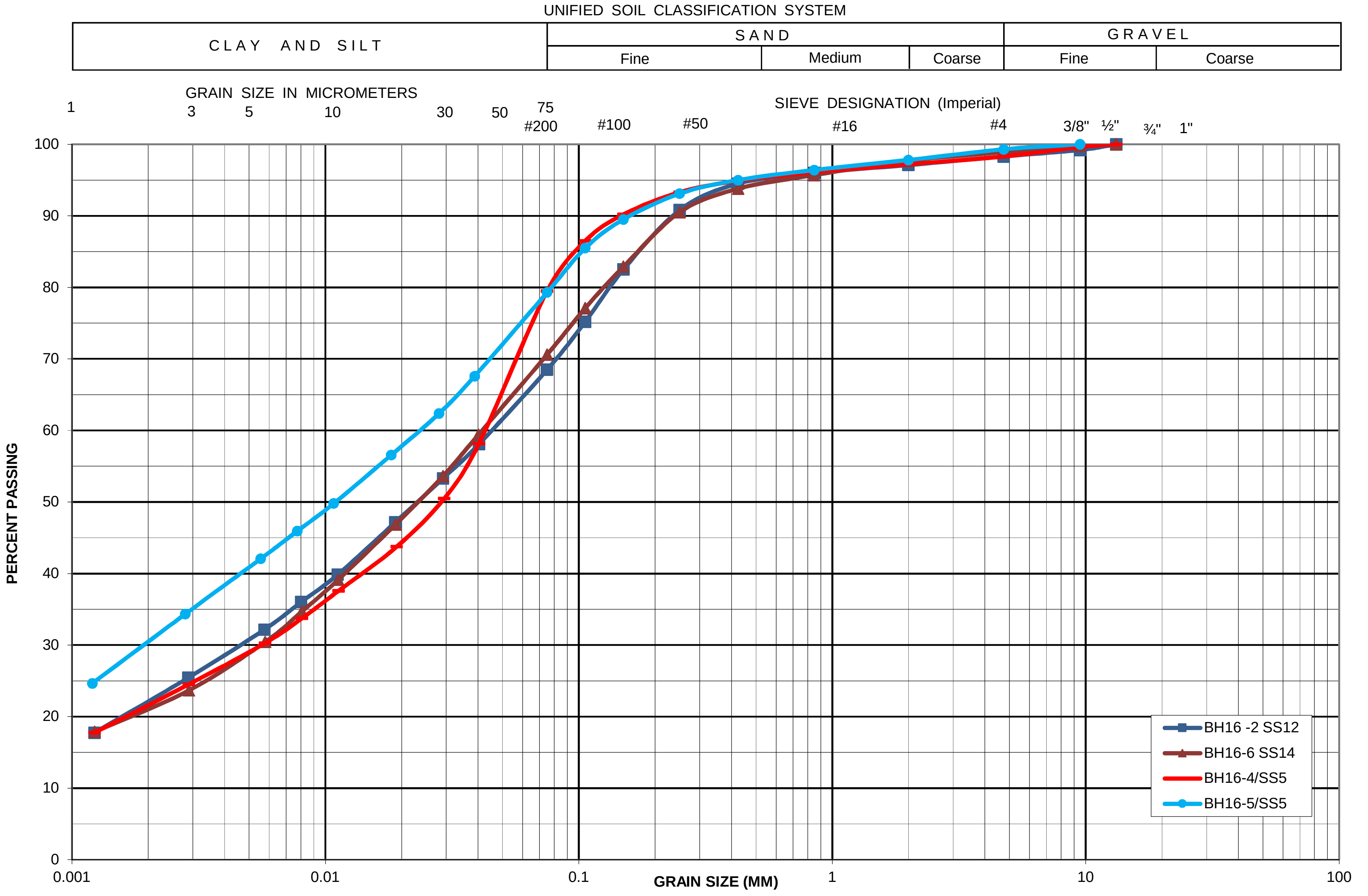
Silty Clay

Figure No.	2a
Project No.	16M-01410-01
Date :	June-04-2017



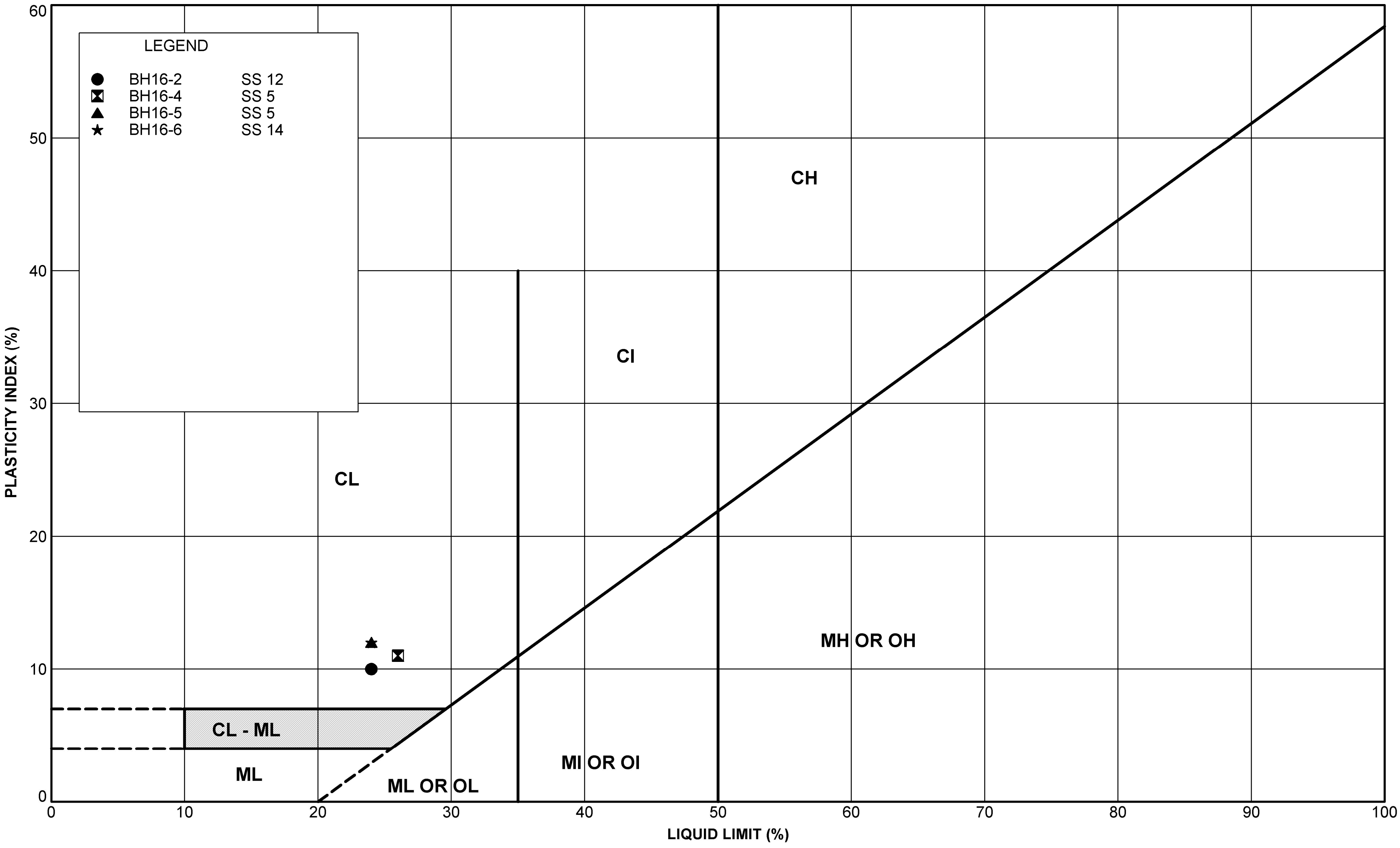
PLASTICITY CHART

FIGURE NO. 2b
JOB NO. 16M-01410-01
DATE July14,2017



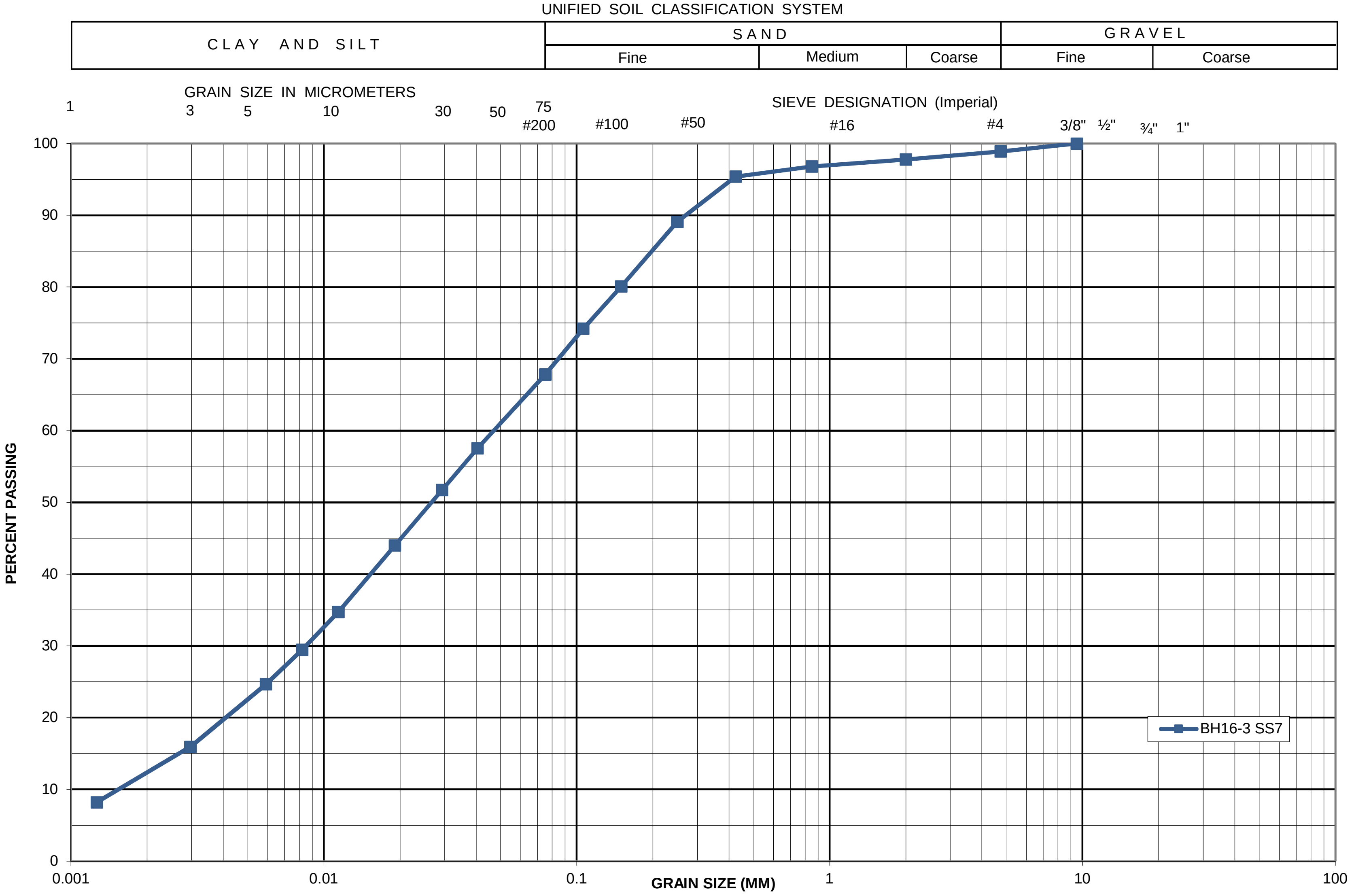
GRAIN SIZE DISTRIBUTION
Silty Clay Till

Figure No.	3a
Project No.	16M-01410-01
Date :	July-04-2017



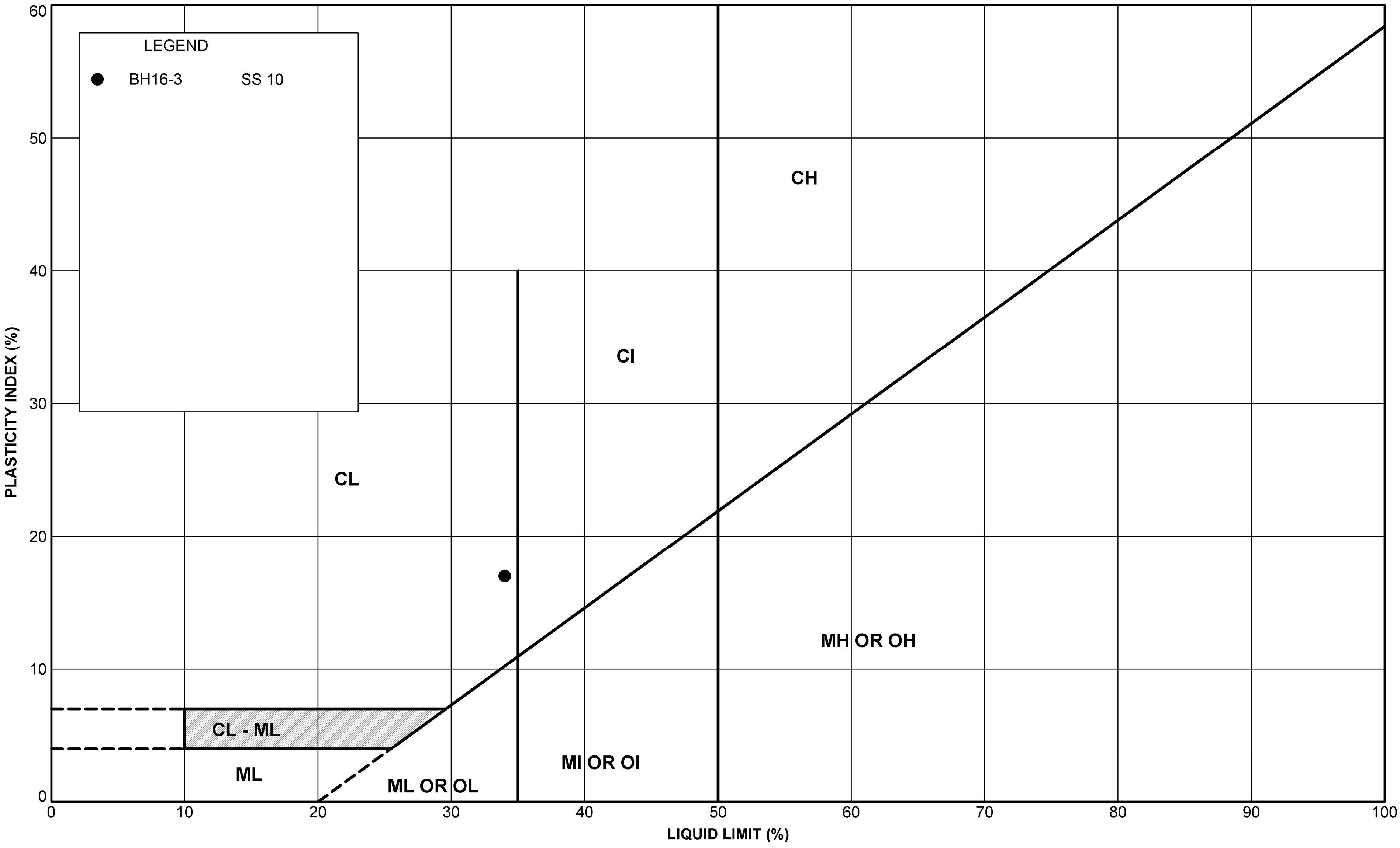
PLASTICITY CHART

FIGURE NO. 3b
JOB NO. 16M-01410-01
DATE July 14, 2017



GRAIN SIZE DISTRIBUTION
Sandy Silt

Figure No.	4
Project No.	16M-01410-01
Date :	June-04-2017



PLASTICITY CHART

FIGURE NO. 5
JOB NO. 16M-01410-01
DATE July 14, 2017

Appendix C: Site Photographs

Photo 1: Looking toward east: Borehole BH 16-1(Dale Ave.) in the vicinity of north abutment

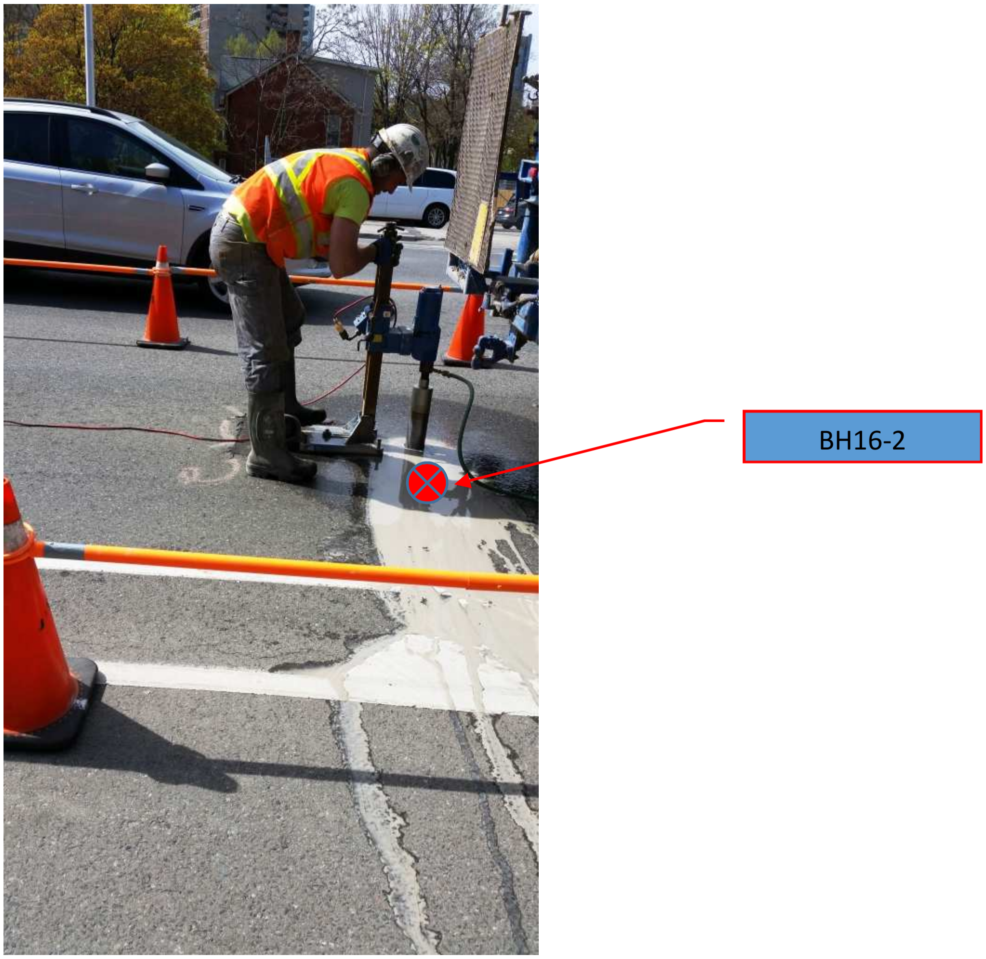


Photo 2: Looking toward south: Borehole BH16-2 (on Bloor St.); adjacent to west side of the tunnel alignment

Photo 3: Looking toward west: Borehole BH16-3 (Rosedale Valley Rd.)

Photo 4: Looking toward north: Borehole BH16-4 on the north side slope of Rosedale Valley Road

Photo 5: Looking toward north: Borehole BH16-5 on the south side slope of Rosedale Valley Road

Photo 6: Looking towards east: Borehole BH16-6 in the vicinity of south abutment

APPENDIX G: LIMITATIONS

LIMITATIONS OF REPORT

This report is intended solely for the Client named. The material in it reflects our best judgment in light of the information available to WSP Canada Inc. at the time of preparation. Unless otherwise agreed in writing by WSP Canada Inc, it shall not be used to express or imply warranty as to the fitness of the property for a particular purpose. No portion of this report may be used as a separate entity, it is written to be read in its entirety.

The conclusions and recommendations given in this report are based on information determined at the test hole locations. The information contained herein in no way reflects on the environment aspects of the project, unless otherwise stated. Subsurface and groundwater conditions between and beyond the test holes may differ from those encountered at the test hole locations, and conditions may become apparent during construction, which could not be detected or anticipated at the time of the site investigation. The benchmark and elevations used in this report are primarily to establish relative elevation differences between the test hole locations and should not be used for other purposes, such as grading, excavating, planning, development, etc.

The design recommendations given in this report are applicable only to the project described in the text and then only if constructed substantially in accordance with the details stated in this report.

The comments made in this report on potential construction problems and possible methods are intended only for the guidance of the designer. The number of test holes may not be sufficient to determine all the factors that may affect construction methods and costs. For example, the thickness of surficial topsoil or fill layers may vary markedly and unpredictably. The contractors bidding on this project or undertaking the construction should, therefore, make their own interpretation of the factual information presented and draw their own conclusions as to how the subsurface conditions may affect their work. This work has been undertaken in accordance with normally accepted geotechnical engineering practices.

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

We accept no responsibility for any decisions made or actions taken as a result of this report unless we are specifically advised of and participate in such action, in which case our responsibility will be as agreed to at that time.