



Terraprobe

Consulting Geotechnical & Environmental Engineering

Construction Materials Inspection & Testing

**FOUNDATION INVESTIGATION AND DESIGN REPORT
TEE, LYONS AND BLACK CREEKS BRIDGE STRUCTURES
BLACK CREEK BRIDGE REPLACEMENTS
QUEEN ELIZABETH WAY (QEW)
MINISTRY OF TRANSPORTATION, ONTARIO
DB-2014-2036, SITES 34-128/1 & 34-128/2
GEOCRES NO. 30L14-59**

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File No. 1-15-0689
July 08, 2016

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PART A – FOUNDATION INVESTIGATION REPORT

**BLACK CREEK BRIDGE REPLACEMENTS, SITES 34-128/1 & 34-128/2
QUEEN ELIZABETH WAY
REGIONAL MUNICIPALITY OF NIAGARA, ONTARIO
CONTRACT NUMBER DB-2014-2036**



1.0 INTRODUCTION

Terraprobe Inc. (Terraprobe) has been retained by MMM Group Limited (MMM) on behalf of Dufferin Construction to provide foundation engineering services in support of detailed designs for the replacement of the Black Creek North-Bound and South-Bound bridges.

This project is based on the Ministry of Transportation, Ontario (MTO) Design Build Minor Request for Proposals titled *"Tee Creek, Lyons and Black Creeks Bridge Structures, Central Region"*, Contract Number. DB-2014-2036. The terms of reference and scope of work for the foundation engineering services are outlined in MTO's RFP.

This report presents the factual data on the subsurface conditions at the Black Creek Bridges, Sites 34-128/1 and 34-128/2 on the Queen Elizabeth Way (QEW), Welland County, Regional Municipality of Niagara, Ontario.

2.0 SITE DESCRIPTION

The site (with MTM coordinates of N 4,758,300; E 343,800) is located on the QEW, at Townline Road in Welland County, Ontario. The key plan on the Borehole Locations and Soil Strata Drawing, (Drawing 1) provides an overview of the site location.

The existing north-bound and south-bound bridges are cast-in-place concrete T-beam structures that are approximately 30.3 m long and 14.5 m wide. Both bridges consist of a 19.7 m centre span and two 5.3 m long cantilevered end spans. Construction records indicate that the existing bridge foundations were constructed by driving interlocking sheet piles arranged in a cruciform-shaped cross-section, excavating the soil within the sheet pile and filling the excavated area with concrete.

The terrain at the bridge site and surrounding area is generally flat and both bridges span Black Creek which flows from west to east. Based on the existing topography both bridges were constructed to span Black Creek with approximately 4 m high approach embankments adjacent to the east bank of Black Creek. The approaches adjacent to the west bank of Black Creek were constructed at existing grade.

3.0 INVESTIGATION PROCEDURES

3.1.1 Current Investigation

The field work for this project was carried out on November 30, 2016 and consisted of drilling and sampling one borehole to a depth of 11.7 m below ground surface. The approximate borehole location is shown on Drawing 1.

Based on drawings provided by Terraprobe, MMM's surveyors staked out the borehole location and supplied the borehole coordinates and geodetic elevation to Terraprobe. The actual borehole location drilled by Terraprobe was referenced to MMM's original staked location.

The borehole was drilled with a truck-mounted drill rig supplied and operated by a specialist drilling contractor. Samples of the overburden soils were generally obtained at intervals of 0.75 m and 1.5 m depth using a 50 mm outer diameter (O.D.) split-spoon sampler in conjunction with the Standard Penetration

Testing (SPT) procedures as specified in ASTM Method D1586¹. In the clay deposit an MTO 'N' vane was used to perform in-situ field vane tests, in order to determine the undrained shear strength of the soil. The bedrock was also cored to a depth of 3.7 m by NQ-size diamond coring techniques. The field work was monitored on a full-time basis by a member of Terraprobe's staff who observed the drilling, sampling and in situ testing operations and logged the borehole and rock core.

Ground water conditions in the open borehole were observed during the drilling operations and prior to bedrock coring. The borehole was backfilled in accordance with current MTO procedures and Ontario Regulation 903 (as amended).

The recovered soil and rock samples were subjected to Visual Identification (VI) and select soil samples were also subjected to a laboratory testing programme consisting of natural moisture content, grain size distribution analyses and Atterberg limits determinations in accordance with MTO and/or ASTM Standards as appropriate. Point load index testing was also carried out on the bedrock core samples.

3.1.2 Previous Investigation

In June 2013 and May 2015, subsurface investigations were carried out by Golder Associates Limited (Golder) of Mississauga, Ontario. Five boreholes (Boreholes 13-11, 13-12, 13-13, 13-14 and 15-01) were drilled and the data from these investigations were used to supplement the current investigation. These boreholes were advanced to depths ranging from 6.6 m to 12.2 m below ground surface and the Record of Borehole sheets and associated laboratory test results are provided in Appendix A and B respectively. The approximate locations of these boreholes are shown on Drawing 1.

The Golder boreholes were drilled using continuous flight hollow stem auger drilling techniques. The overburden soil samples were obtained at selected intervals using a split-spoon sampler in conjunction with the Standard Penetration Test (SPT) method, and the bedrock was cored in Borehole 15-01 to a depth of 2.9 m below ground surface by NQ-size diamond coring techniques.

3.1.3 Borehole Locations

The borehole locations in MTM NAD83 northing and easting coordinates, the ground surface elevations referenced to geodetic datum and depths drilled are summarized in the following table.

Borehole Data

Borehole No.	MTM Coordinate System		Ground Surface Elevation (m)	Borehole Depth (m)
	Northing (m)	Easting (m)		
BH1	4 758 258.4	343 819.9	175.6	11.7
BH 13-11	4 758 285.1	343 768.6	175.7	8.5
BH 13-12	4 758 255.1	343 799.6	175.6	6.7
BH 13-13	4 758 274.2	343 818.4	175.8	6.6
BH 13-14	4 758 302.1	343 785.7	175.8	8.6
BH 15-01	4 758 299.2	343 782.6	175.8	12.2

1 ASTM D1586 – Standard Test Method for Standard Penetration Tests and Split Barrel Sampling of Soils.

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

The site is located between the Niagara Escarpment and Lake Erie in the physiographic region of Southern Ontario referred to as the Haldimand Clay Plain. The Haldimand Clay Plain is best described as falling into a series of parallel belts with the highest ground adjacent to the Escarpment. Generally, this region is flat and poorly drained although it includes several distinctive landforms such as dunes, cobble, clay and sand beaches, limestone pavements and back-shore wetland basins².

The Niagara Region is underlain by a sequence of very gently south-dipping dolostones, limestones, shales and sandstones overlying Precambrian basement rock. The key elements in the bedrock geology of the region are the multiple layers of softer sedimentary limestones, shale, sandstone and dolostone.

The bedrock unit at this site is the Salina and Guelph Formation of Upper Silurian Age. This unit consists essentially of easily weathered, grey, very finely crystalline, laminated argillaceous dolostone with grey, calcareous shale partings and gypsum veins and lenses of varying thicknesses.

4.2 Subsurface Conditions

Reference is made to the Record of Borehole Sheets in Appendix A. Details of the encountered soil and rock stratigraphy are presented in this appendix and on the "Borehole Locations and Soil Strata" drawings. An overall description of the stratigraphy is given in the following paragraphs.

The stratigraphic boundaries shown on the Record of Boreholes and on the interpreted stratigraphic sections are inferred from non-continuous soil sampling and therefore represent transitions between soil types rather than exact planes of geological change. The subsurface conditions will vary between and beyond the borehole locations.

In summary, the ground surface is underlain by a flexible pavement and clay fill. The pavement and fill material are underlain by deposits of soft to stiff clayey organic silt, soft to very stiff silty clay to clay, soft clayey silt and sand, very stiff to hard clayey silt till and loose to very dense sand and gravel and sand till. The overburden soils are further underlain by dolomitic shale bedrock of the Salina Formation. A more detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

4.2.1 Flexible Pavement

A flexible pavement consisting of 200 mm to 300 mm thick asphalt concrete underlain by sand and gravel fill was encountered. The locations, thicknesses and base elevations of the sand and gravel fill are summarized in the following table.

Pavement Granular Borehole Data

Borehole No.	Fill Thickness (mm)	Fill Base Elevation (m)
BH1	400	175.0
BH 13-11	550	174.9

² Chapman and Putnam, "The Physiography of South Ontario", 3rd Edition, 1984.

Borehole No.	Fill Thickness (mm)	Fill Base Elevation (m)
BH 13-12	1150	174.2
BH 13-13	1150	174.4
BH 13-14	500	175.0

Standard Penetration tests carried out in the gravelly sand and sand fill gave SPT N-values that range from of 5 blows to 30 blows for 0.3 m of penetration indicating a loose to compact relative density.

The grain size distribution curve of a sample of the sand and gravel fill is depicted on Golder Figure B1 in Appendix B2. The results show a grain size distribution consisting of 43% gravel, 45% sand, 8% silt and 4% clay size particles. The natural water content of samples of the granular fill range from 1% to 4% by weight.

4.2.2 Fill - Clay

Borehole 1 encountered a layer of clay fill below the flexible pavement. The clay fill is approximately 1.5 m thick and extends to elevation 173.5 m. Standard Penetration tests performed in the clay fill measure SPT N-values that range from 11 to 14 blows for 0.3 m of penetration indicating a stiff consistency. The natural water content of a sample of the clay fill is 23% by weight.

The grain size distribution curve of a sample of the clay fill is depicted on Figure B1 in Appendix B1. The results show a grain size distribution consisting of 11% gravel, 13% sand, 20% silt and 56% clay size particles.

The clay fill was also subjected to an Atterberg Limits test and the results are presented on Figure B2 in Appendix B1. These results indicate that the fill is a high plasticity (CH) cohesive soil. The results from the Atterberg limits test are summarized below:

Liquid Limit:	51 %
Plastic Limit:	23 %
Plasticity Index:	28 %
Natural Moisture Content:	23 %

4.2.3 Clayey Organic Silt

Clayey organic silt was encountered in three of the Golder boreholes. Summarized below are the locations, thicknesses, depths and base elevations of the clayey organic silt deposit.

Clayey Organic Silt Borehole Data

Borehole No.	Clayey Organic Silt Thickness (m)	Clayey Organic Silt Depth (m)	Clayey Organic Silt Base Elevation (m)
BH 13-11	2.1	2.9	172.8
BH 13-12	4.4	5.8	169.8
BH 13-13	3.1	4.5	171.3



Standard Penetration tests performed in the clayey organic silt measure SPT N-values that range from 3 to 10 blows for 0.3 m of penetration, described on the Golder logs as soft to stiff consistency. The natural water content of the clayey organic silt varies from 26% to 38% by weight.

Three samples of the clayey organic silt were subjected to Atterberg Limits test and the results are presented on Golder Figure B2 in Appendix B2. These results indicate a clayey organic silt (MH-OH). The results from the Atterberg limits tests are summarized below:

Liquid Limit:	59% to 60%
Plastic Limit:	30% to 34%
Plasticity Index:	26% to 29%
Natural Moisture Content:	29% to 38%

4.2.4 Silty Clay to Clay

Silty clay to clay soils were encountered at this site. Summarized below are the locations, thicknesses, depths and base elevations of these deposits.

Silty Clay to Clay Borehole Data

Borehole No.	Silty Clay to Clay Thickness (m)	Silty Clay to Clay Depth (m)	Silty Clay to Clay Base Elevation (m)
BH1	3.8	5.9	169.7
BH 13-11	1.7	4.6	171.1
BH 13-13	1.1	5.6	170.2
BH 13-14	4.8	5.6	170.2

Standard Penetration tests in the silty clay to clay measured SPT N-values of 2 to 16 blows per 0.3 m of penetration and, field vane tests measured in-situ undrained shear strengths ranging from 44 kPa to more than 96 kPa. Based on these tests the silty clay to clay is described as having a soft to very stiff consistency. The sensitivity of the silty clay to clay varies from 1.4 to 2.0, indicating a low sensitivity soil class (Canadian Foundation Engineering Manual [CFEM], 2006). The moisture content of samples of the silty clay to clay varies from 16% to 36% by weight.

The Terraprobe grain size distribution plots of two samples of the clay are depicted in Figure B3 in Appendix B1. The grain size distribution plots of three samples of the silty clay to clay from the Golder study are depicted in Figure B3 in Appendix B2. These results show a grain size distribution consisting of 0% gravel, 0% to 20% sand, 24% to 39% silt and, 35% to 76% clay sized particles.

A sample of the clay deposit from the Terraprobe study was also subjected to an Atterberg limits test and the results are presented in Figure B4 in Appendix B1. The Atterberg limits tests of three samples of the silty clay to clay from the Golder study are plotted on the Plasticity Chart Figure B4, Appendix B2. These values indicate that the silty clay to clay deposit is an intermediate to high plasticity (CI-CH) cohesive soil. The Atterberg limits test results are summarized below.

Liquid Limit:	40% to 58 %
Plastic Limit:	20% to 25%
Plasticity Index:	20% to 33%
Natural Moisture Content:	24% to 34%

4.2.5 Clayey Silt and Sand

Terraprobe Borehole 1 encountered a 0.8 m thick layer of clayey silt and sand that extends to a depth of 6.7 m (elevation 168.9 m) below ground surface. A Standard Penetration test in the clayey silt and sand measured an SPT N-value of 1 blow per 0.3 m of penetration. Based on this N-value the clayey silt and sand is described as having a soft consistency. The moisture content of a sample of the clayey silt and sand is 10% by weight.

The grain size distribution plot of a sample of the clayey silt and sand is depicted in Figure B5 in Appendix B1. These results show a grain size distribution consisting of 5% gravel, 40% sand, 37% silt and, 18% clay sized particles.

A sample of the clayey silt and sand deposit was also subjected to an Atterberg limits test and the results are presented in Figure B6 in Appendix B1. These values indicate that the clayey silt and sand deposit is low plasticity (CL-ML) cohesive soil. The Atterberg limits test results are summarized below.

Liquid Limit:	18%
Plastic Limit:	11%
Plasticity Index:	7%
Natural Moisture Content:	10%

4.2.6 Clayey Silt Till

Three of the Golder boreholes encountered a clayey silt till deposit. The locations, thicknesses, depths and base elevations of the clayey silt till deposit are summarized in the following table.

Clayey Silt Till Borehole Data

Borehole No.	Clayey Silt Till Thickness (m)	Clayey Silt Till Depth (m)	Clayey Silt Till Base Elevation (m)
BH 13-11	2.4	7.0	168.7
BH 13-14	1.6	7.2	168.6
BH 15-01	-	7.2	168.6

The N-values of Standard Penetration tests carried out in the silty clay till deposit range from 23 to 48 blows per 0.3 m of penetration, suggesting a very stiff to hard consistency, and the moisture content of samples of this deposit range from 8% to 10% by weight.

In the Golder study a sample of the clayey silt till was subjected to a grain size distribution test and the grain size distribution curve is illustrated on Figure B5 in Appendix B2. The test results show a grain size distribution consisting of 19% gravel, 29% sand, 34% silt and 18% clay sized particles.

An Atterberg limits test was also carried out on a sample of the clayey silt till in the Golder study and the results are plotted on the plasticity chart, Figure B6 in Appendix B2. The results indicate that the till matrix generally consists of a low plasticity (CL) silty clay soil. The Atterberg limits test results are summarized below.

Liquid Limit:	20%
Plastic Limit:	11%
Plasticity Index:	9%
Natural Moisture Content:	8%

4.2.7 Sand and Gravel and Sand Till

Till soils with a soil matrix that ranges in composition from sand and gravel to sand were encountered in the boreholes. Summarized in the following table are the locations, explored depths and base elevations of these deposits.

Sand and Gravel and Sand Till Borehole Data

Borehole No.	Sand and Gravel and Sand Till Thickness (m)	Sand and Gravel and Sand Till Depth of Deposit (m)	Sand and Gravel and Sand Till Base Elevation (m)
BH1	1.3	8.0	167.6
BH 13-11	-	8.5*	167.2
BH 13-12	-	6.7*	168.9
BH 13-13	-	6.6*	169.2
BH 13-14	-	8.6*	167.2
BH 15-01	2.1	9.3	166.6

* Borehole termination depth.

Standard Penetration tests carried out in these deposits gave N-values that generally range from 7 to more than 100 blows per 0.3 m of penetration indicating a loose to very dense relative density. In Borehole 1 an SPT N-value of 2 blows per 0.3 m of penetration was recorded in the upper portion of this deposit indicating a very loose relative density. The moisture content of samples from this stratum range from 8% to 24% by weight.

A sample of the sand till deposit from the Terraprobe study was subjected to a grain size distribution test and the results are presented in Figure B7 in Appendix B1. Three samples from the Golder study were subjected to grain size distribution tests and the results are presented in Figure B7, Appendix B2. These results show a grain size distribution consisting of 3% to 58% gravel, 32% to 75% sand, 5% to 19% silt and; 5% to 14% clay sized particles.

In the Golder study Atterberg limits tests were also carried out two samples of the sand and gravel till and the results are plotted on the plasticity chart, Figure B8 in Appendix B2. The results indicate that the fines fraction of the till matrix generally consists of low plasticity (CL-ML) clayey silt and cohesionless (ML) silts. The Atterberg limits test results are summarized below.

Liquid Limit:	18% and 20%
Plastic Limit:	14% and 15%
Plasticity Index:	4% and 5%
Natural Moisture Content:	9% and 13%

4.2.8 Bedrock

In Boreholes 1 and 15-01 the overburden soils are underlain by dolomitic shale bedrock of the Salina Formation. The bedrock was encountered at depths of 8.0 m and 9.3 m below ground surface or at elevations 167.6 m and 166.6 m respectively. Photographs of the bedrock core samples are provided in Figure B8 in Appendix B1 and Figure 1 in Appendix B2.

The bedrock is described as slightly weathered to unweathered, laminated to thickly bedded and its colour is grey with white gypsum beds. The Rock Quality Designation (RQD) generally varies from 15% to 47%

and in Borehole 15-01 an RQD value of 0% was recorded. Based on this data the rock mass can be described as generally poor quality with occasional poor quality interbeds. The Total Core Recovery (TCR) of the core samples varies from 89% to 100% and the Solid Core Recovery (SCR) of the core samples ranged from 38% to 91%.

Point Load Index Tests were carried out on the bedrock core samples obtained from Borehole 1 and the interpreted unconfined compressive strength (UCS) results range from 24 MPa to 215 MPa. In the Golder study a UCS test was carried out on a bedrock core sample obtained from Borehole 15-01 and a value of 15.7 MPa was obtained. The Point Load Index Test results and the UCS result classify the tested portions of the bedrock as weak (R2 grade, 5 MPa to 25 MPa) to very strong (R5 grade, 100 MPa to 250 MPa) according to the rock strength classification in Table 3.5 of the *Canadian Foundation Engineering Manual 2006*.

4.3 Ground Water Levels

The ground water conditions were observed in the boreholes during and upon completion of drilling. Based on the ground water observations, the soil moisture contents and the creek water level, the ground water level at this site is estimated to be at an approximate elevation of 171.7± m. The ground water level is expected to fluctuate seasonally and is expected to rise during wet periods of the year.

5.0 MISCELLANEOUS

The investigation was carried out using equipment supplied and operated by Elite Drilling Services of St. Catharines, Ontario. The field operations were observed and monitored by Mr. Anthony Felice and the routine laboratory testing was carried out at Terraprobe's Brampton laboratory. Point Load Index testing on the rock cores was carried out at Thurber's Oakville laboratory.

This report was prepared by Mr. Rehman Abdul, P.Eng., a Senior Geotechnical Engineer and Associate with Terraprobe. Mr. Michael Tanos, P.Eng., Terraprobe's Designated MTO Contact conducted an independent quality control review.

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PART B – FOUNDATION DESIGN REPORT

**BLACK CREEK BRIDGE REPLACEMENTS, SITES 34-128/1 & 34-128/2
QUEEN ELIZABETH WAY
REGIONAL MUNICIPALITY OF NIAGARA, ONTARIO
CONTRACT NUMBER DB-2014-2036**



6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

6.1 General

This report presents interpretation of the geotechnical data in the factual report and presents geotechnical design recommendations to assist the design team to select a preferred foundation alternative for the Black Creek North-Bound and South-Bound Bridge replacements.

This report was prepared in the context of a design-build contract. The discussion and recommendations presented in this report are based on our understanding of the project and our interpretation of the factual data obtained from the subsurface investigations. If conditions are encountered during construction that are different than what is understood at the time this report was prepared, based on the subsurface conditions and testing described herein; Terraprobe must be consulted to update, supplement or otherwise revise these recommendations as appropriate.

The existing north-bound and south-bound bridges are cast-in-place concrete T-beam structures that are approximately 30.3 m long and 14.5 m wide. Both bridges have a 19.7 m centre span and two 5.3 m long cantilevered end spans. The bridges carry the QEW north bound and south bound traffic over Black Creek.

The replacement structures being considered are single span integral abutment bridges each with a span length of 22.8 m and a deck width of 14.3 m. The bridges will be constructed on the same alignment as the existing structures and, the QEW NBL and SBL profiles will generally be maintained with minor paving adjustments to eliminate bridge deck drains.

6.2 Foundation Alternatives

The advantages, disadvantages, risks and consequences of practical foundation options for supporting the new bridges are presented in Table 1. These foundation alternatives are summarized below.

- Augered Caissons (drilled shafts); and
- Pre-bored H-piles;

Spread footings are not considered to be a feasible and practical solution because:

- Relatively deep excavations extending up to 8 m below the QEW are required to found spread footings on competent strata, or to construct an engineered fill pad to support a spread footing at higher elevations; and
- Excavations will be made adjacent to Black Creek and will extend below the free water level of the creek. Therefore, a cofferdam arrangement and extensive unwatering requirements will be necessary, both of which are costly from a construction perspective.

Driven piles were also considered but were excluded from further consideration because:

- The Townline Road bridge is directly above the south abutment of the new southbound bridge and relatively close the south abutment of the northbound bridge. It is therefore impractical to operate conventional pile driving equipment in this area because of the limited headroom;
- Pile driving may impart unacceptably high vibrations to the subsurface soils and bedrock that could compromise the integrity of the Townline Road pier footings; and

- The upper portion of the bedrock contains softer water soluble gypsum interbeds. End bearing piles founded on bedrock above the gypsum interbeds can fail over time if the gypsum dissolves to form cavities below the pile tip.

6.2.1 Caissons (Drilled Shafts)

Augered caissons socketed into sound bedrock are a practical foundation scheme that merits consideration. The caissons will have to be extended through relatively impermeable clay soils as well as more permeable sand to sand and gravel till deposits below the ground water table. Therefore, a steel casing will be required to support each caisson sidewall and provide seepage cut-off.

When extending the caissons, random cobble and boulder inclusions may be encountered in the till soils. Therefore, provisions should be made to have on site equipment to remove these obstructions if encountered.

6.2.1.1 Axial Resistance (Drilled Shafts)

Softer and weaker gypsum interbeds exist within the bedrock. Therefore, caissons should be extended below the gypsum layers and shall be founded on sound bedrock. It is therefore recommended that the caissons be socketed into the bedrock at the specified base elevations tabulated below.

Estimated Caisson Base Elevations

Location	Reference Borehole	Estimated Base Elevation (m)	Founding Stratum
North Abutments	13-11, 15-01 and 13-14	165.5±	Bedrock
South Abutments	13-12, 1, and 13-13	165.0±	Bedrock

Caissons socketed into the bedrock can be designed as end-bearing units using a factored axial geotechnical resistance at ULS of 5 MPa. For a 1 m diameter caisson founded on sound bedrock the factored axial resistance at ULS will be 4,000 kN and the SLS condition will not govern. These values are for vertical, concentric loads only.

6.2.2 Prebored H-Piles

It is recognised that an integral abutment bridge (which requires pile foundations) offers significant long term advantages. The site in its present condition is not suitable for driven piles because of the low headroom imposed by the Townline Road overpass and the relatively shallow bedrock depth. Therefore, a prebored H-pile arrangement foundation alternative was considered. An advantage of this design concept is that the foundations can be designed to bear on sound bedrock below the gypsum interbeds.

An integral abutment structure may be designed if the abutment foundations are prepared as follows.

- Excavate the original ground to the underside of the bridge abutment. Pre-auger through the overburden soils to the top of bedrock and use a temporary 350 mm diameter steel liner to preclude water ingress into the excavation, provide sidewall support and to maintain an open excavation.

The estimated top of bedrock elevations at the north and south bridge abutments are 166.6 m and 167.6 m respectively.

- Extend the excavation into the bedrock by churn drilling or other similar techniques. The base elevation of the bedrock excavation shall be 164.8 m at the north and south abutments which allows for founding the piles below the gypsum layers and also provides for a 750 mm rock socket formed in the sound bedrock;
- Clean the augered hole and remove all debris including soil and rock cuttings at the excavation base. Carry out a camera inspection of the rock socket with a special borehole camera to verify that the excavation is clean and the rock socket is at least 750 mm below any gypsum interbeds within the rock. A specialist operator experienced in downhole photography and inspection should be employed for this aspect of the work and the data should be provided to Terraprobe for review;
- Place the steel H-pile in the augered hole, centre and tap the pile to seat its tip on the bedrock;
- Backfill around the H-pile from the base of the rock socket to the top of bedrock with concrete and allow the concrete to set before proceeding with integral abutment construction. Concrete must be placed by pumping; and
- After the concrete in the rock socket has set, proceed with normal integral abutment construction by withdrawing the liner while simultaneously filling the annular space between the H-pile and liner with sand meeting MTO's grading requirements.

6.2.2.1 Axial Resistance

It is recommended that the base of the prebored holes for the H-piles be founded on bedrock at the specified pile tip elevations tabulated below.

Estimated Pile Tip Elevations

Location	Reference Borehole	Estimated Pile Tip Elevation (m)	Founding Stratum
North Abutments	15-01	164.8±	Sound Bedrock
South Abutments	1	164.8±	Sound Bedrock

A HP 200x54 steel pile installed in a 0.75 m long and 350 mm diameter rock socket can be designed for a factored axial geotechnical resistance of 790 kN and the SLS condition will not govern. This value is for vertical, concentric loads only. The structural resistance of the prebored H-pile arrangement combination should be checked by the structural designer.

6.2.2.2 Downdrag

Excavations to the underside of the bridge abutments will extend to elevation 171± m and the borehole data indicates that the underlying silty clay to clay overburden below the excavation will not be more than about 1.5± m thick. Therefore downdrag forces on the steel liner of the prebored H-pile foundation scheme will be negligible.

6.2.2.3 Integral Abutment Considerations

The ground conditions at this site are considered suitable for an integral abutment design. The integral abutment design requires that the piles possess flexibility in the upper 3 m of the pile length. To provide the required flexibility in the piles, the upper 3 m minimum of the prebored H-piles will be backfilled with sand as the temporary 350 mm diameter steel liner is withdrawn, which will satisfy the requirements for MTO's integral abutment design procedures.

A Non Standard Special Provision (NSSP) will be required specifying the gradation of the sand according to the data tabulated below.

Integral Abutment Sand Grading

MTO Sieve Designation		Percentage Passing
2 mm	#10	100%
600 µm	#30	80%-100%
425 µm	#40	40%-80%
250 µm	#60	5%-25%
150 µm	#100	0%-6%

6.2.2.4 Lateral Resistance

The lateral resistance of the piles may be calculated using a value for the coefficient of horizontal subgrade reaction (k_s) and the ultimate lateral resistance (p_{ult}) as outlined in the equations below:

$$\begin{array}{llll}
 k_s & = & n_h \ z / D \text{ [cohesionless soils]} & (\text{kN/m}^3) \\
 k_s & = & 67 \ S_u / D \text{ [cohesive soils]} & (\text{kN/m}^3) \\
 p_{ult} & = & 3 \ \gamma \ z \ K_p \text{ [cohesionless soils]} & (\text{kPa}) \\
 p_{ult} & = & 9 \ S_u \text{ [cohesive soils]} & (\text{kPa}) \\
 \text{where } z & = & \text{depth of pile embedment} & (\text{m}) \\
 D & = & \text{pile width} & (\text{m}) \\
 S_u & = & \text{undrained shear strength} & (\text{kPa}) \\
 n_h & = & \text{coefficient of horizontal subgrade reaction} & (\text{kN/m}^3) \\
 \gamma & = & \text{unit weight} & (\text{kN/m}^3) \\
 K_p & = & \text{passive earth pressure coefficient} & (\text{dimensionless})
 \end{array}$$

The spring constant K , for analysis of a pile segment or element of length L metres, can be obtained from the expression, $K = k_s \times L \times D$ (kN/m). The ultimate lateral resistance P_{ult} , of a pile segment or element of length L metres, can be obtained from the expression, $P_{ult} = p_{ult} \times L \times D$.

The equations provided above and the soil parameters provided in the following table, may be used to analyze the interaction between a pile and the surrounding soil. The lateral pressures obtained from the analysis must not exceed the ultimate lateral resistance or the factored structural flexural resistance of the pile. A maximum horizontal passive resistance of 80 kN (ULS) is recommended for design.

Recommended Soil Parameters

Area Reference Borehole No	Applicable Elevation**	Soil Type	Bulk Unit Weight (kN/m ³)	Angle of Internal Friction (ϕ) Degrees	Undrained Shear Strength (S_u) (kPa)	Recommended n_h Value (kN/m ³)*
Black Creek North Bound Bridge						
North Abutment BH 13-14 and 15-01	171.1 – 170.2	Silty Clay to Clay	20	0	25	–
	170.2 – 168.6	Clayey Silt Till	20	0	175	–
	168.6 – 166.6	Sand and Gravel Till	21	30	–	11000
South Abutment BH 13-13 and BH-1	171.1 – 170.2	Silty Clay	20	0	40	–
	170.2 – 167.6	Sand and Gravel Till	21	30	–	7700
Black Creek South Bound Bridge						
North Abutment BH 13-11 and 15-01	171.1 – 168.7	Clayey Silt Till	20	0	175	–
	168.7 – 166.6	Sand and Gravel Till	21	30	–	11000
South Abutment BH 13-12 BH-1	171.1 – 169.8	Clayey Org. Silt	18.5	28	–	1300
	169.8 – 167.6	Sand and Gravel Till	21	30	–	7700

* Values estimated based on Table 20.3 data, Canadian Foundation Engineering Manual, 3rd edition, 1992

** Based on an underside abutment elevation of 171.1 m.

Since the piles are end bearing, their vertical resistance will not be significantly affected by the pile spacing. Pile interaction should be considered with reference to CHBDC Clause 6.8.9.2.

For lateral soil/pile group interaction analysis, the equation for k_s quoted in this section may be used in conjunction with appropriate reduction factors. Intermediate values of the horizontal subgrade reaction reduction factor R may be obtained by interpolation. Where a pile group is oriented perpendicular to the direction of loading, group action may be considered by reducing values for k_s by a reduction factor R as follows:

Pile Spacing Perpendicular to Direction of Loading	Horizontal Subgrade Reaction Reduction Factor, R
4 D*	1.00
1 D*	0.50

* D is the width of the pile, and spacing is measured centre to centre.

Where a pile group is oriented parallel to the direction of loading, group action may be considered by reducing values for k_s by a reduction factor R as follows:

Pile Spacing Parallel to Direction of Loading	Horizontal Subgrade Reaction Reduction Factor, R
8 D*	1.00
6 D*	0.70
4 D*	0.40
3 D*	0.25

* D is the width of the pile, and spacing is measured centre to centre.

6.2.3 Recommended Foundation Scheme

From a geotechnical point of view, it is recommended that the new bridges be supported on pre-bored steel H-pile foundations socketed into the sound bedrock. Based on the advantages, disadvantages, risks and consequences, a pre-bored steel H-pile foundation scheme is reliable, allows for the design of integral abutment bridges, has the lowest construction risk of disturbing the existing Townline Road underpass footings and has the highest probability of acceptable structural performance.

6.2.4 Design Frost Depth

Pile caps and footings should be founded at a minimum depth of 1.2 m of earth cover below the lowest surrounding grade to provide adequate protection against frost penetration, as per OPSD 3090.101. In addition, the footings should extend below any existing fill and surficial organic materials, where present. Rock protection provides frost protection equivalent to 50% of the layer thickness and this aspect should be considered when designing frost depths.

6.3 Lateral Earth Pressure

Earth pressures are generally calculated using the following expression:

$$P_h = K(\gamma h + q)$$

where P_h = horizontal pressure on the wall (kPa);

K = lateral earth pressure coefficient;

γ = unit weight of retained soil (kN/m³);

h = depth below top of fill where pressure is computed (m); and

q = value of any surcharge (kPa).

Earth pressures acting on the structure should be computed in accordance with Clause 6.9 of the CHBDC 2006 and according to Clause 6.9.3 of the CHBDC 2006; a compaction surcharge should also be added. For soils with an angle of internal friction ranging from 30° to 35° the magnitude should be 12 kPa at the top of the fill decreasing linearly to 0 kPa at a depth of 1.7 m; or decreasing linearly to 0 kPa at a depth of 2.0 m for soils with an angle of internal friction that exceeds 35°.

The lateral earth pressure coefficients are dependent on the material used as backfill and typical values are provided in the following table.



Lateral Earth Pressure Coefficients

Wall Condition	Lateral Earth Pressure Coefficient (K)			
	OPSS Granular A or OPSS Granular B Type II $\phi = 35^\circ; \gamma = 22.8 \text{ kN/m}^3$		OPSS Granular B Type I $\phi = 32^\circ; \gamma = 21.2 \text{ kN/m}^3$	
	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)
Active (Unrestrained Wall)	0.27	0.38*	0.30	0.46*
At rest (Restrained Wall)	0.43	-	0.47	-
Passive (Movement Towards Soil Mass)	3.70	-	3.30	-

* For wing walls.

The lateral earth pressure coefficients provided in the table above are “ultimate” values that require certain structural movements for the respective conditions to be mobilized. The values to use in design can be estimated from Figure C6.16 in the Commentary to the CHBDC, 2006.

6.4 Abutment Backfill

The backfill to the abutment walls should be carried out in accordance with OPSS 902. Granular backfill should be placed to the extents shown in OPSD 3101.150 and shall comply with the OPSS.PROV 1010 specifications. The design of the abutment should also incorporate a subdrain as shown in OPSD 3101.150.

All granular fill should be placed in loose lifts not exceeding 150 mm thick and should be compacted to at least 95 % of the materials Standard Proctor Maximum Dry Density (SPMDD). Equal heights of backfill should be maintained on both sides of the structure during all stages of backfill placement, and backfilling operations should be carried out in accordance with OPSS 902. Compaction equipment including hand operated vibratory equipment shall comply with OPSS.PROV 501.

6.5 Erosion Protection

The November 2015 water level in Black Creek is at elevation 171.7 m and this water will partially submerge and erode the forward and side slopes at the bridge abutments if the slopes are not protected. Design of an erosion protection scheme will depend on hydrologic, hydraulic and/or other concerns. We recommend using rip-rap to armour the embankment slopes with which creek water is likely to be in contact. The rip-rap should be installed in accordance with OPSS 511.

Surface water can also cause erosion beneath the rip-rap and loss of fines through the rip-rap. Therefore, a properly designed filter should be installed between the rip-rap and the embankment material.

We recommend that a qualified Hydraulics Engineer be consulted to provide inputs on the design thickness and lateral extent of rip-rap protection and to estimate the scour depth. Footings must be placed below the scour depth.

6.6 Excavations

All excavations must be carried out in accordance with the guidelines outlined in the *Occupational Health and Safety Act (OHSA) and Regulations for Construction Projects*. Where workers must enter excavations extending deeper than 1.2 m, the trench walls must be suitably sloped and/or braced in accordance with the OHSA. Within the envisaged depths of temporary excavations (i.e. up to elevation 171± m), the OHSA soil classifications are:

- Embankment fill – Type 3 soil;
- Clayey organic silt – Type 3 soil;
- Clayey silt till – Type 3 soil; and
- Silty clay – Type 3 soil.

The side slopes of temporary excavations may be formed no steeper than 1H:1V for Type 3 soils and excavations should be carried out in accordance with OPSS 902.

6.7 Ground Water Control

Surface water and ground water control will be necessary to enable construction below the ground water table. We recommend temporarily diverting the flow of creek water away from the construction area. Around the perimeter of the excavation, a cofferdam and an interceptor perimeter trench should also be installed to prevent surface water from entering the excavation.

The design, installation, operation and maintenance of the dewatering system is the Contractor's responsibility. Excavations will extend through the existing embankment fill, clayey organic silt, silty clay to clay and the clayey silt till. A suitable dewatering system that can be employed is gravity drainage and pumping from strategically placed filtered sumps.

6.8 Approach Embankments

6.8.1 Settlement

The embankment settlement analysis was carried out using elastic deformation moduli established from predictions/empirical correlations using undrained shear strengths, Atterberg limits and SPT N-values, tempered with engineering judgement from our experience with similar soils in this region of Ontario.

The General Arrangement drawing shows that the existing QEW profile will be generally maintained with minor paving adjustments to eliminate bridge deck drains. However, abutment backfill for the new bridges will be placed in the open forward slope area below the existing bridges. The new abutments construction will require excavations that extend approximately to the underside abutment elevations of the new bridges i.e. elevation 171± m. It is estimated that the abutment backfill will induce approximately 20± mm of total settlement in the footprint area of the new fill. This settlement will be essentially complete in one month.

Embankments constructed with local earth fill or Granular A material will also settle during construction (fill compression) and, the magnitude of this settlement is expected to be about 1% and 0.5% of the fill height respectively. This settlement should be immediate in nature and essentially be complete shortly after construction is complete.

6.8.2 Stability

The global, internal and surficial stability of the embankment side slopes and forward slopes will depend on the slope geometry and also to a large degree on the material used to construct the embankment. For the purpose of embankment stability analyses, the commercially available slope stability program Slide 6.0 developed by Rocscience Inc. was used.

The Morgenstern-Price and Spencer methods for stability analysis were employed and a minimum target factor of safety of 1.3 was established. The soil parameters used for the slope stability analyses and the factors of safety that were obtained are provided in the following table. The slope stability models depicting the corresponding factors of safety are provided in Figure C1 and C2 in Appendix C. The analyses indicate that the factors of safety will be greater than the target factor of safety of 1.3, provided that the embankment is constructed at a minimum side slope and forward slope geometry of 2 Horizontal to 1 Vertical (2H:1V) or flatter.

Slope Stability Design Parameters and Results

Material Type	Total Stress Analysis		Effective Stress Analysis		Unit Weight
	ϕ (degrees)	c (kPa)	ϕ' (degrees)	c' (kPa)	γ (kN/m ³)
New Embankment Fill	35	0	35	0	22.8
Clayey Organic Silt	0	30	30	0	18.5
Clayey Silt and Sand	0	10	29	0	18
Silty Clay to Clay	0	40	28	0	20
Clayey Silt Till	0	175	30	0	20
Sand to Sand and Gravel Till	30	0	30	0	21
Design Factors of Safety	1.4 to 1.6		1.3		-

6.8.3 Embankment Construction

Materials used for embankment construction should be placed in lifts not exceeding 300 mm (before compaction), and each lift should be uniformly compacted to at least 95% of the material's SPMD. Embankment construction should be carried out in accordance with OPSS.PROV 209, OPSS.PROV 501 and OPSS.PROV 206. Borrow material must meet the requirements of OPSS.PROV 212 and bonding between existing fill and new fill should be carried out by benching in accordance with OPSS 208.010.

Proper erosion control measures should be implemented both during construction and permanently. Temporary erosion and sediment control must be provided in accordance with OPSS 805 and embankment slopes must be reinstated with permanent erosion protection in accordance with OPSS 803 and OPSS.PROV 804.

6.9 Temporary Protection Systems

Temporary protection systems should be designed in accordance with OPSS.PROV 539 by a licensed Professional Engineer experienced in shoring design. The shape of the soil pressure distribution diagram behind a temporary protection system depends upon the type of soil to be supported and the amount of movement that can be permitted. The sequence of work will alter the shape of the pressure diagram during the various construction phases.

Earth pressure computations must also take into account the ground water level. Above the ground water level, earth pressure is computed using the bulk unit weight of the retained soil. Below the ground water level, the earth pressures are computed using the submerged unit weight of the soil. A hydrostatic pressure is also applied if the retained soil is not fully drained.

Flexible shoring should be designed on the basis of the active earth pressure coefficient (K_a). In this case, the performance level should be Level 2 – Angular Distortion 1:200 but shall not be more than 25 mm. Where limited shoring movement (Performance Level 1A or 1B) is required the design should be based on the at rest earth pressure coefficient (K_o). For “kick out” design the lateral resistance should be computed on the basis of the passive earth pressure coefficient (K_p). It should be noted that the lateral earth pressure coefficients chosen for design require certain movements for the active and passive conditions to be mobilized.

The appropriate lateral earth pressure parameters for use in the design of structures subject to unbalanced earth pressures are provided in the following table. The active earth pressure coefficients are based on the assumption that the ground surface behind the temporary protection system is horizontal. Where the retained ground is sloping, the lateral earth pressure coefficients must be adjusted to account for the slope and, these earth pressure coefficients can be estimated from the equations provided on Figures C6.17 and C6.18 of the CHBDC 2006.

Temporary Protection System Design Parameters

Stratigraphic Unit	Friction Angle ϕ (degrees)	Unit Weight γ (kN/m)	Active Earth Pressure Coefficient	At - Rest Earth Pressure Coefficient	Passive Earth Pressure Coefficient
			K_a	K_o	K_p
Granular A Fill	35	22.8	0.27	0.43	3.69
Existing Fill Soils	30	19	0.33	0.50	3.00
Clayey Organic Silt	30	18.5	0.33	0.50	3.00
Clayey Silt and Sand	29	18	0.35	0.52	2.88
Silty Clay to Clay	28	20	0.36	0.53	2.77
Clayey Silt Till	30	20	0.33	0.50	3.00
Sand to Sand & Gravel Till	30	21	0.33	0.50	3.00

6.10 Seismic Requirements

The site is treated as lying in Seismic Zone 0. Reference to Annex A3.1 of the CHBDC 2006 indicates that the following seismic parameters (Fort Erie) should be used for design:

- Velocity Related Seismic Zone 0
- Zonal Velocity Ratio 0.05
- Acceleration Related Seismic Zone 2
- Zonal Acceleration Ratio 0.10
- Peak Horizontal Ground Acceleration 0.08 g (10% in 50 years)

The soil profile type at this site has been classified as Type I and the Site Coefficient “S (ground motion amplification factor) that should be used in seismic design as per Clause 4.4.6.1, Table 4.4 of the CHBDC is 1.0.

6.11 Construction Concerns

During construction, the Design Builder should employ experienced geotechnical staff to observe construction activities related to foundation construction. Potential construction concerns include, but are not necessarily limited to:

- the possibility of encountering cobbles and boulders when carrying out pre-boring operations to install the H-piles;
- removing all debris including soil and rock cuttings in the rock socket;
- monitoring the existing Townline Road underpass for vibrations and any signs of detrimental movement while constructing the two bridges; and
- pollution, siltation or disruption of environmentally sensitive areas.

7.0 CLOSURE

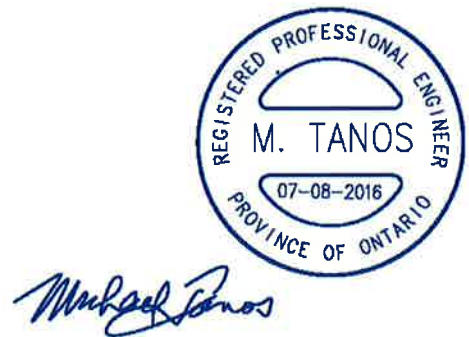
This report was prepared by Mr. Rehman Abdul, P.Eng., a Senior Geotechnical Engineer and Associate with Terraprobe. Mr. Michael Tanos, P.Eng., Terraprobe's Designated MTO Contact conducted an independent quality control review.

Terraprobe Inc.



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R. Abdul, P.Eng.
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Michael Tanos, P.Eng.
Designated MTO Contact



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- Canadian Geotechnical Society, 2006. *Canadian Foundation Engineering Manual*, 4th Edition. The Canadian Geotechnical Society c/o BiTech Publisher Ltd, British Columbia.
- Canadian Highway Bridge Design Code (CHBDC) and Commentary on CAN/CSA S6 06. 2006.* CSA Special Publication, S6.1 06. Canadian Standard Association.
- Chapman and Putnam, "The Physiography of South Ontario", 3rd Edition, 1984.
- Ontario Division of Mines, "Quaternary Geology of The Welland Area", Preliminary Map P.796, 1972.

Ontario Provincial Standard Specifications (OPSS)

OPSS.PROV 206	Construction Specification For Grading.
OPSS.PROV 209	Construction Specification For Embankments Over Swamps And Compressible Soils.
OPSS.PROV 212	Construction Specification For Earth Borrow.
OPSS.PROV 501	Construction Specification For Compacting.
OPSS 511	Construction Specification For Rip-Rap, Rock Protection and Granular Sheeting.
OPSS.PROV 539	Construction Specification For Temporary Protection Systems.
OPSS 803	Construction Specification For Sodding.
OPSS.PROV 804	Construction Specification For Seed and Cover.
OPSS 805	Construction Specification For Temporary Erosion And Sediment Control Measures.
OPSS 902	Construction Specification For Excavating and Backfilling – Structures.
OPSS.PROV 1010	Material Specification For Aggregates – Base, Subbase, Select Subgrade and Backfill Material.

Ontario Provincial Standard Drawings (OPSD)

OPSD 208.010	Benching Of Earth Slopes.
OPSD 3090.101	Foundation, Frost Penetration Depths For Southern Ontario
OPSD 3101.150	Walls Abutment Backfill, Minimum Granular Requirement

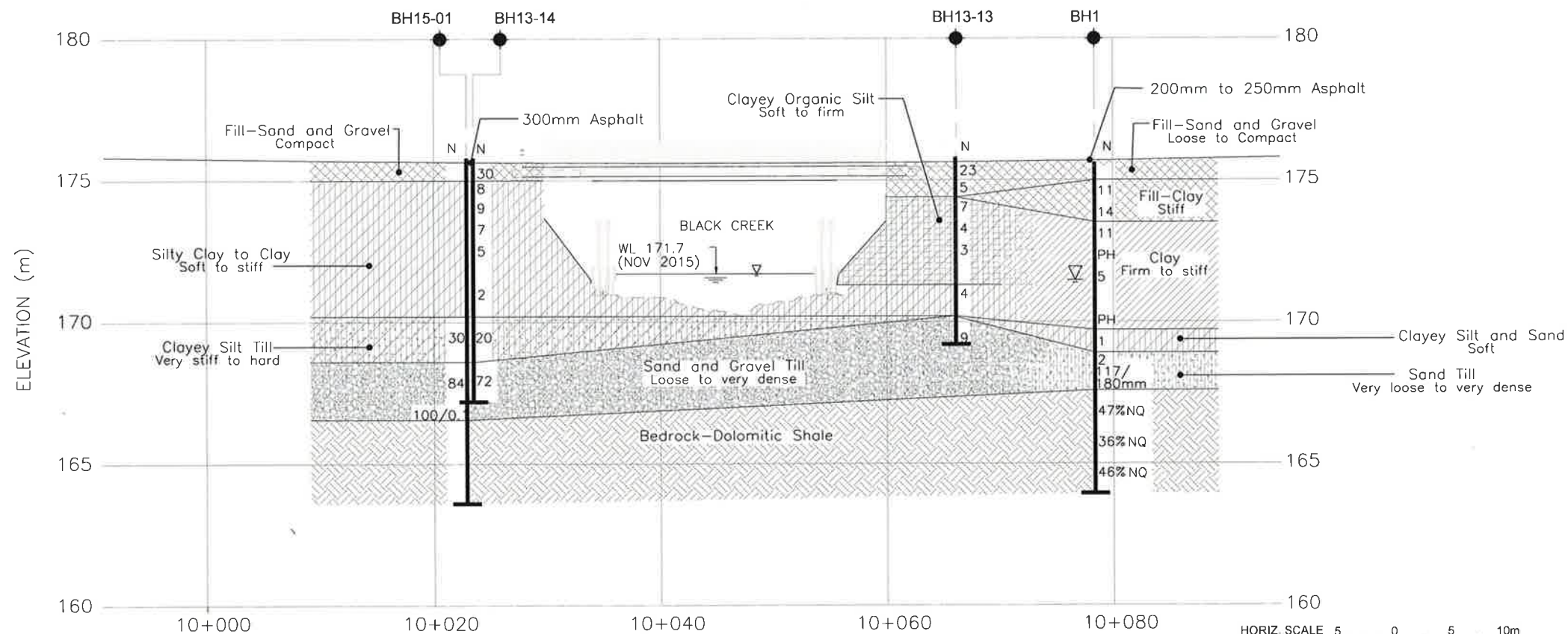
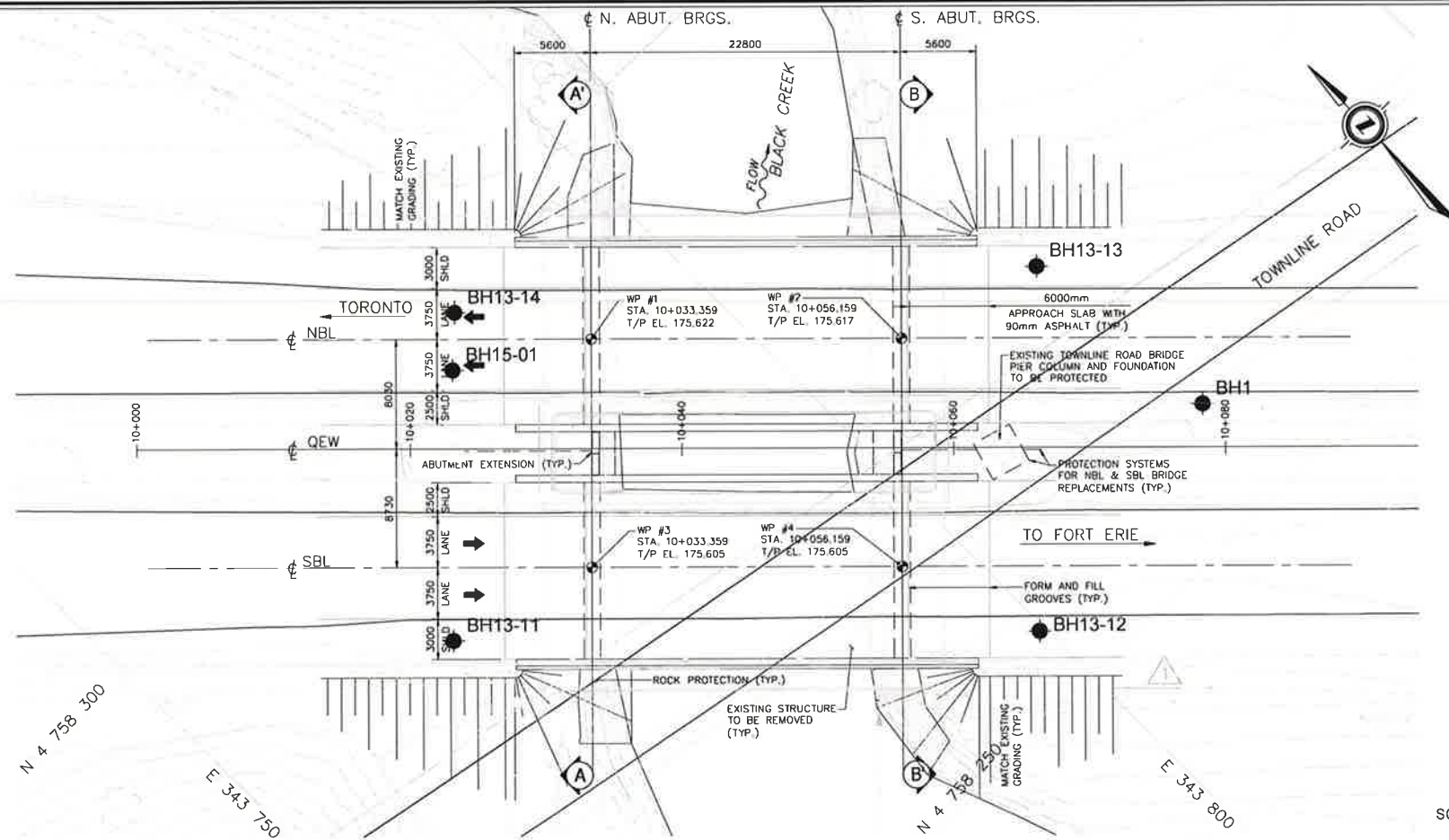


TABLE 1
COMPARISON OF FOUNDATION ALTERNATIVES

Pre-bored H-Piles	Caissons (Drilled Shafts)
<p>Advantages:</p> <ul style="list-style-type: none">Reliable performance expected.Easy to construct in the limited headroom area imposed by Townline Road Underpass.Allows for the design of an integral or semi integral abutment bridge.Shorter abutment stems can be provided compared to constructing spread footings on native soils.Minimal vibrations transferred to subsurface soils and rock which reduces the risk of vibrations imparted on the existing Townline Road bridge footings.Relatively large and deep excavations are not required. <p>Disadvantages:</p> <ul style="list-style-type: none">Requires good construction techniques and proper cleaning and inspection of the rock socket.Augering through cobbles and boulders may be problematic.	<p>Advantages:</p> <ul style="list-style-type: none">Reliable performance expected.Higher geotechnical resistance available compared to a Pre-bored H-Pile arrangement.Relatively large and deep excavations are not required.Shorter abutment stems can be provided compared to constructing spread footings on native soils.Minimal vibrations transferred to subsurface soils and rock which reduces the risk of vibrations on the existing Townline Road bridge footings. <p>Disadvantages:</p> <ul style="list-style-type: none">Does not allow for the design of an integral abutment.Augering through cobbles and boulders may be problematic.
<p>Risks/Consequences</p> <ul style="list-style-type: none">Very Low risk of bearing capacity failure.Very low risk that total settlement will exceed 25 mm.Very low risk of disturbing the existing Townline Road underpass footings.	<p>Risks/Consequences</p> <ul style="list-style-type: none">Very low risk of bearing capacity failure.Very low risk that total settlement will exceed 25 mm.Very low risk of disturbing the existing Townline Road underpass footings.

DRAWINGS





METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETERS UNLESS
OTHERWISE SHOWN



CONT DB 2014-2036
GWP No 2177-08-00

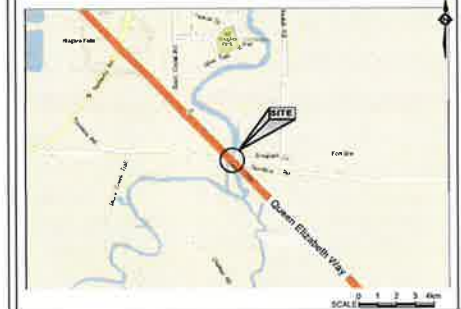


BLACK CREEK BRIDGE
NORTH BOUND LANE
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET



Terraprobe Inc.
Consulting Geotechnical & Environmental Engineering
Construction Materials Engineering, Inspection & Testing
11 Indell Lane - Brampton Ontario L6T 3Y3 (905) 796-2650



KEY PLAN

LEGEND	
	Bore Hole
	Dynamic Cone Penetration Test
	Bore Hole And Cone
	Blows/0.3m (Std Pen Test, 475 J/blow)
	Blows/0.3m (60' Cone, 475 J/blow)
	WL at Time of Investigation
	WL in Piezometer
	Piezometer
	90% Rock Quality Designation
	A/R Auger Refusal

No	ELEV.	COORDINATES	
		NORTHING	EASTING
1	175.6	4 758 258.4	343 819.9
13-11	175.7	4 758 285.1	343 788.6
13-12	175.6	4 758 255.1	343 799.6
13-13	175.8	4 758 274.2	343 818.4
13-14	175.8	4 758 302.1	343 785.7
15-01	175.8	4 758 299.2	343 782.6

NOTE

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

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

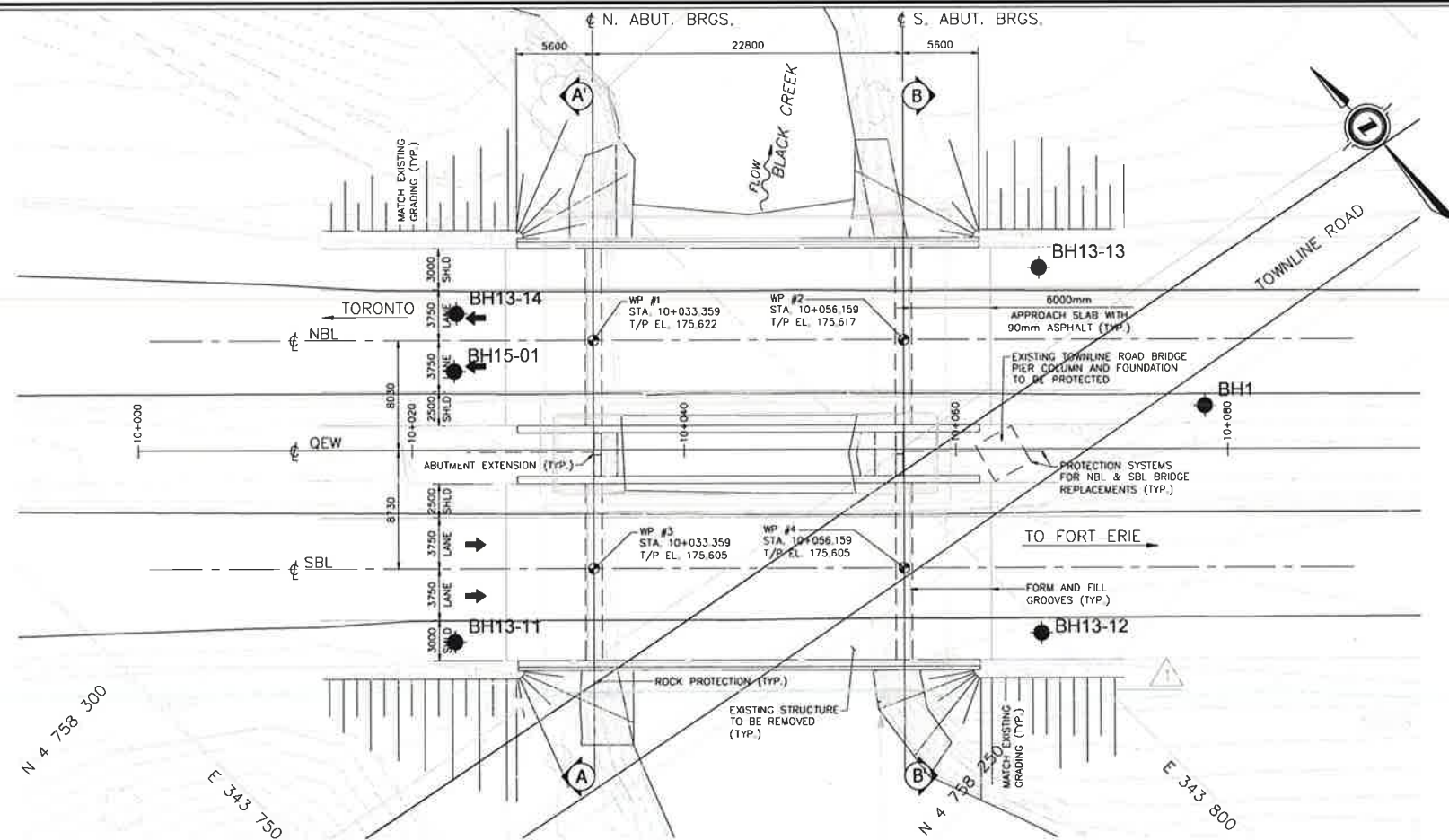
The complete foundation investigation and design report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in this report and related documents are specifically excluded in accordance with Section GC 2.01 of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

REFERENCE

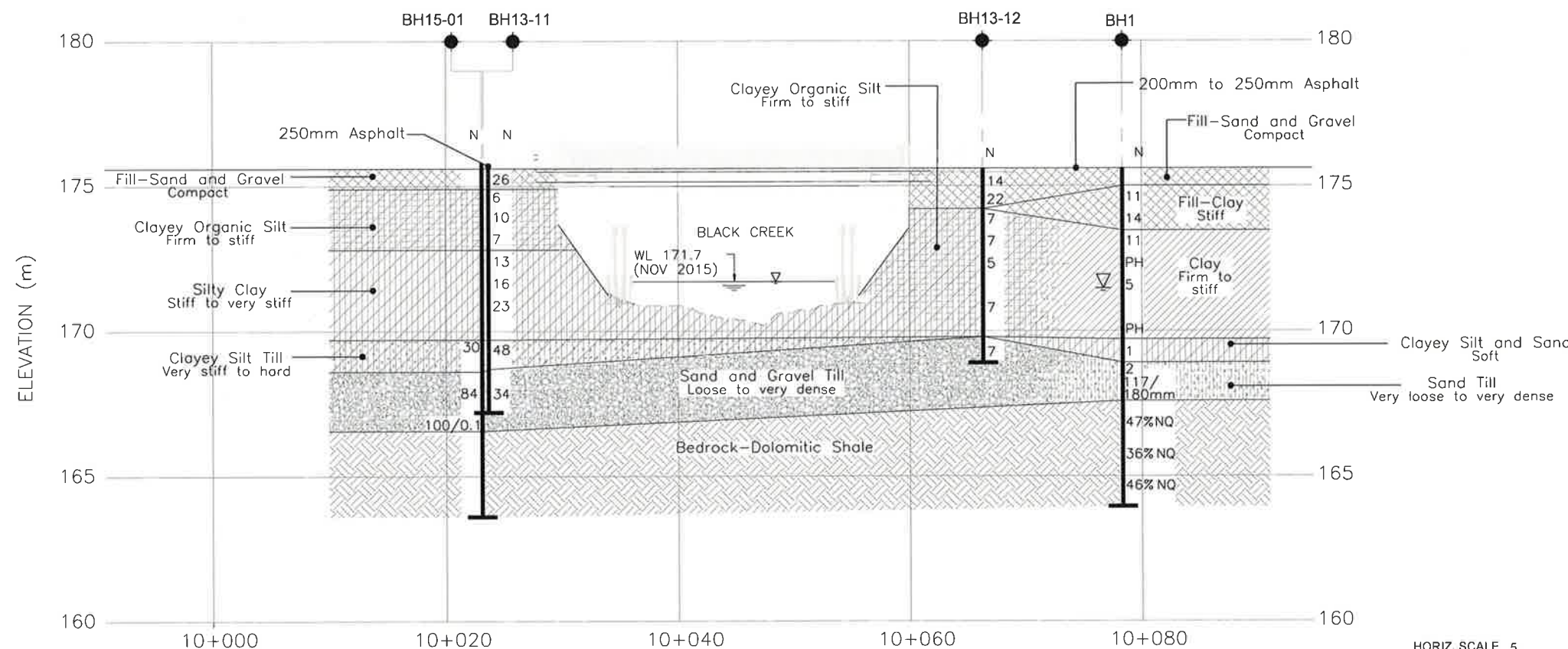
Drawings provided in digital format by MMM Group Limited, by email drawing file x3215095-320-001GA.dwg, 3215095-320-001XG.dwg, H3215095XB01.dwg, H3215095XB02.dwg received January 12, 2016.

REVISIONS	DATE	BY	DESCRIPTION

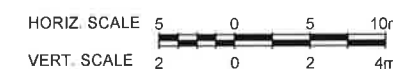
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SUBM'D.RA	CHKD. RA	DATE: FEB. 2016
DRAWN: KC	CHKD. RA	APPD: MT
		DWG. 1



PLAN



PROFILE OF QEW SBL



METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETERS UNLESS
OTHERWISE SHOWN

CONT DB 2014-2036

GWP No 2177-08-00



BLACK CREEK BRIDGE
SOUTH BOUND LANE
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET



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KEY PLAN

LEGEND	
	Bore Hole
	Dynamic Cone Penetration Test
	Bore Hole And Cone
	Blows/0.3m (Std Pen Test, 475 J/blow)
	Blows/0.3m (60' Cone, 475 J/blow)
	WL at Time of Investigation
	WL in Piezometer
	90% A/R
	Rock Quality Designation
	Auger Refusal

No	ELEV.	COORDINATES	
		NORTHING	EASTING
1	175.6	4 758 258.4	343 819.9
13-11	175.7	4 758 285.1	343 788.8
13-12	175.6	4 758 255.1	343 799.6
13-13	175.8	4 758 274.2	343 818.4
13-14	175.8	4 758 302.1	343 785.7
15-01	175.8	4 758 299.2	343 782.6

NOTE
This drawing is for subsurface information only. The proposed structure details/works if shown are for illustration purposes only and may not be consistent with final design configuration as shown elsewhere in the contract 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 are specifically excluded in accordance with Section GC 2.01 of OPS General Conditions

REFERENCE
Drawings provided in digital format by MMM Group Limited, by email drawing file x3215095-320-001GA.dwg, 3215095-320-001XG.dwg, H3215095XB01.dwg, H3215095XB02.dwg received January 12, 2016.

REVISIONS	DATE	BY	DESCRIPTION

HWY: QEW	PROJECT No. 1-15-0689	Geocres No. 30L14-59
SUBM'D: RA	CHKD: RA	DATE: FEB. 2016 SITE: 34-128/2
DRAWN: KC	CHKD: RA	APPD: MT DWG. 2

APPENDIX A1
Record of Borehole Sheets
Terraprobe Inc.



LIMITATIONS AND RISK

Procedures

The soil conditions were confirmed at the borehole locations only and conditions may vary between and beyond the boreholes. The boundaries between the various strata as shown on the logs are based on non-continuous sampling. These boundaries represent an inferred transition between the various strata, rather than a precise plane of stratigraphic change.

This investigation has been carried out using investigation techniques and engineering analysis methods consistent with those ordinarily exercised by Terraprobe and other engineering practitioners, working under similar conditions and subject to the time, financial and physical constraints applicable to this project. The discussions and recommendations that have been presented are based on the factual data obtained.

It must be recognized that there are special risks whenever engineering or related disciplines are applied to identify subsurface conditions. Even a comprehensive sampling and testing programme implemented in accordance with the most stringent level of care may fail to detect certain conditions. Terraprobe has assumed for the purposes of providing design parameters and advice, that the conditions that exist between sampling points are similar to those found at the sample locations. The conditions that Terraprobe has interpreted to exist between sampling points can differ from those that actually exist.

It may not be possible to drill a sufficient number of boreholes or sample and report them in a way that would provide all the subsurface information that could affect construction costs, techniques, equipment and scheduling. Contractors bidding on or undertaking work on the project should be directed to draw their own conclusions as to how the subsurface conditions may affect them, based on their own investigations and their own interpretations of the factual investigation results, cognizant of the risks implicit in the subsurface investigation activities.

Changes In Site And Scope

It must be recognized that the passage of time, natural occurrences, and direct or indirect human intervention at or near the site have the potential to alter subsurface conditions. Groundwater levels are particularly susceptible to seasonal fluctuations.

The design advice is based on the factual data obtained from this investigation made at the site by Terraprobe and are intended for use by the owner and its retained designers in the design phase of the project. If there are changes to the project scope and development features, or there is any additional information relevant to the interpretations made of the subsurface information, the geotechnical design parameters and comments relating to constructibility issues and quality control may not be relevant or complete for the revised project. Terraprobe should be retained to review the implications of such changes with respect to the contents of this report.

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EXPLANATION OF TERMS USED IN REPORT

N VALUE: THE STANDARD PENETRATION TEST (SPT) N VALUE IS THE NUMBER OF BLOWS REQUIRED TO CAUSE A STANDARD 51mm O.D. SPLIT BARREL SAMPLER TO PENETRATE 0.3m INTO UNDISTURBED GROUND IN A BOREHOLE WHEN DRIVEN BY A HAMMER WITH A MASS OF 63.5kg. FALLING FREELY A DISTANCE OF 0.76m. FOR PENETRATIONS OF LESS THAN 0.3m N VALUES ARE INDICATED AS THE NUMBER OF BLOWS FOR THE PENETRATION ACHIEVED. AVERAGE N VALUE IS DENOTED THUS \bar{u} .

DYNAMIC CONE PENETRATION TEST: CONTINUOUS PENETRATION OF A CONICAL STEEL POINT (51mm O.D. 60° CONE ANGLE) DRIVEN BY 475J IMPACT ENERGY ON 'A' SIZE DRILL RODS. THE RESISTANCE TO CONE PENETRATION IS MEASURED AS THE NUMBER OF BLOWS FOR EACH 0.3m ADVANCE OF THE CONICAL POINT INTO THE UNDISTURBED GROUND.

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

CONSISTENCY: COHESIVE SOILS ARE DESCRIBED ON THE BASIS OF THEIR UNDRAINED SHEAR STRENGTH (c_u) AS FOLLOWS:

c_u (kPa)	0 – 12	12 – 25	25 – 50	50 – 100	100 – 200	>200
	VERY SOFT	SOFT	FIRM	STIFF	VERY STIFF	HARD

DENSENESS: COHESIONLESS SOILS ARE DESCRIBED ON THE BASIS OF DENSENESS AS INDICATED BY SPT N VALUES AS FOLLOWS:

N (BLOWS/0.3m)	0 – 5	5 – 10	10 – 30	30 – 50	>50
	VERY LOOSE	LOOSE	COMPACT	DENSE	VERY DENSE

ROCKS ARE DESCRIBED BY THEIR COMPOSITION AND STRUCTURAL FEATURES AND/OR STRENGTH.

RECOVERY: SUM OF ALL RECOVERED ROCK CORE PIECES FROM A CORING RUN EXPRESSED AS A PERCENT OF THE TOTAL LENGTH OF THE CORING RUN.

MODIFIED RECOVERY: SUM OF THOSE INTACT CORE PIECES, 100mm+ IN LENGTH EXPRESSED AS A PERCENT OF THE LENGTH OF THE CORING RUN. THE ROCK QUALITY DESIGNATION (RQD), FOR MODIFIED RECOVERY IS:

RQD (%)	0 – 25	25 – 50	50 – 75	75 – 90	90 – 100
	VERY POOR	POOR	FAIR	GOOD	EXCELLENT

JOINTING AND BEDDING:

SPACING	50mm	50 – 300mm	0.3m – 1m	1m – 3m	>3m
JOINTING	VERY CLOSE	CLOSE	MOD. CLOSE	WIDE	VERY WIDE
BEDDING	VERY THIN	THIN	MEDIUM	THICK	VERY THICK

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

SS	SPLIT SPOON	TP	THINWALL PISTON
WS	WASH SAMPLE	OS	OSTERBERG SAMPLE
ST	SLOTTED TUBE SAMPLE	RC	ROCK CORE
BS	BLOCK SAMPLE	PH	TW ADVANCED HYDRAULICALLY
CS	CHUNK SAMPLE	PM	TW ADVANCED MANUALLY
TW	THINWALL OPEN	FS	FOIL SAMPLE

STRESS AND STRAIN

u_w	kPa	PORE WATER PRESSURE
r_u	1	PORE PRESSURE RATIO
σ	kPa	TOTAL NORMAL STRESS
σ'	kPa	EFFECTIVE NORMAL STRESS
τ	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
ϵ	%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
μ	1	COEFFICIENT OF FRICTION

MECHANICAL PROPERTIES OF SOIL

m_v	kPa ⁻¹	COEFFICIENT OF VOLUME CHANGE
C_c	1	COMPRESSION INDEX
C_s	1	SWELLING INDEX
C_{α}	1	RATE OF SECONDARY CONSOLIDATION
C_v	m ² /s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
T_v	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
σ'_{vo}	kPa	EFFECTIVE OVERBURDEN PRESSURE
σ'_p	kPa	PRECONSOLIDATION PRESSURE
τ_f	kPa	SHEAR STRENGTH
c'	kPa	EFFECTIVE COHESION INTERCEPT
ϕ'	- °	EFFECTIVE ANGLE OF INTERNAL FRICTION
c_u	kPa	APPARENT COHESION INTERCEPT
ϕ_u	- °	APPARENT ANGLE OF INTERNAL FRICTION
τ_R	kPa	RESIDUAL SHEAR STRENGTH
τ_r	kPa	REMOULDED SHEAR STRENGTH
S_r	1	SENSITIVITY = c_u / τ_r

PHYSICAL PROPERTIES OF SOIL

ρ_s	kg/m ³	DENSITY OF SOLID PARTICLES	e	1.0%	VOID RATIO	e_{min}	1.0%	VOID RATIO IN DENSEST STATE
γ_s	kN/m ³	UNIT WEIGHT OF SOLID PARTICLES	n	1.0%	POROSITY	I_D	1	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
ρ_w	kg/m ³	DENSITY OF WATER	w	1.0%	WATER CONTENT	D	mm	GRAIN DIAMETER
γ_w	kN/m ³	UNIT WEIGHT OF WATER	S_r	%	DEGREE OF SATURATION	D_n	mm	n PERCENT - DIAMETER
ρ	kg/m ³	DENSITY OF SOIL	w_L	%	LIQUID LIMIT	C_u	1	UNIFORMITY COEFFICIENT
γ	kN/m ³	UNIT WEIGHT OF SOIL	w_p	%	PLASTIC LIMIT	h	m	HYDRAULIC HEAD OR POTENTIAL
ρ_d	kg/m ³	DENSITY OF DRY SOIL	w_S	%	SHRINKAGE LIMIT	q	m ² /s	RATE OF DISCHARGE
γ_d	kN/m ³	UNIT WEIGHT OF DRY SOIL	I_p	%	PLASTICITY INDEX = $(w_L - w_p)$	v	m/s	DISCHARGE VELOCITY
ρ_{sat}	kg/m ³	DENSITY OF SATURATED SOIL	I_L	1	LIQUIDITY INDEX = $(w - w_p)/I_p$	i	1	HYDRAULIC GRADIENT
γ_{sat}	kN/m ³	UNIT WEIGHT OF SATURATED SOIL	I_C	1	CONSISTENCY INDEX = $(w_L - w)/I_p$	k	m/s	HYDRAULIC CONDUCTIVITY
ρ'	kg/m ³	DENSITY OF SUBMERGED SOIL	e_{max}	1.0%	VOID RATIO IN LOOSEST STATE	j	kN/m ³	SEEPAGE FORCE
γ'	kN/m ³	UNIT WEIGHT OF SUBMERGED SOIL						

RECORD OF BOREHOLE No 1

1 of 1

METRIC

G.W.P. _____ LOCATION _____ Coords: E:343808.9 N:4758270.3 ORIGINATED BY AF
 DIST _____ HWY QEW _____ BOREHOLE TYPE HOLLOW STEM AUGERS, NQ ROCK CORING COMPILED BY HA
 DATUM GEODETIC DATE 2015-11-30 CHECKED BY RA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH (m)	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	SPT 'N' VALUE			SHEAR STRENGTH (kPa)										WATER CONTENT (%)		
								20	40	60	80	100						20	40	60
175.6	GROUND SURFACE																GR SA SI CL			
175.4	200mm ASPHALTIC CONCRETE																			
0.2	400mm Fill, Sand and Gravel, brown																			
175.0	FILL, clay, some sand, some gravel, stiff, brown, moist																			
0.6			1	SS	11												11 13 20 56			
			2	SS	14															
173.5	CLAY, trace sand, trace organics, firm to stiff, grey, moist																			
2.1			3	SS	11												0 0 24 76			
			4	TW	PH															
			5	SS	5												0 1 27 72			
169.7	CLAYEY SILT and SAND, trace gravel, soft, brown, moist to wet																			
5.9			6	TW	PH															
168.9	SAND, some silt, trace clay, trace gravel, very loose to very dense, grey, wet (GLACIAL TILL)																			
6.7			7	SS	1												5 40 37 18			
			8	SS	2												3 75 16 6			
167.6	Dolomitic Shale,slightly weathered to 9.7m, unweathered below, fine grained, laminated to thickly bedded, grey, very strong, with white weak to medium strong gypsum beds above 10.0m; (SALINA FORMATION)																			
8.0			9	SS	117 / 180mm															
			1	RUN	NQ												Run #1 TCR: 96% SCR: 88% RQD: 47% UCS* = 24-110 MPa			
			2	RUN	NQ												Run #2 TCR: 89% SCR: 86% RQD: 36% UCS* = 170 MPa			
			3	RUN	NQ												Run #3 TCR: 98% SCR: 91% RQD: 46% UCS* = 140-215 MPa			
163.9																				

END OF BOREHOLE

Unstabilized water level measured at
4.3m below ground surface prior to
rock coring.

Borehole filled with drill water upon
completion of drilling.

*Uniaxial Compressive Strength
determined from Point Load Strength
Index values.

RECORD OF ROCK CORE No 1

1 of 1

METRIC

G.W.P. _____ LOCATION _____ Coords: E:343808.9 N:4758270.3 ORIGINATED BY AF
DIST _____ HWY QEW BOREHOLE TYPE HOLLOW STEM AUGERS, NQ ROCK CORING COMPILED BY HA
DATUM GEODETIC DATE 2015-11-30 CHECKED BY RA

Depth (m)	Graphic Log	GENERAL DESCRIPTION	Run Elev Depth (m)	Recovery	Elevation (m)	ISRM Weathering Zones	UCS (MPa) ● 5 25 50 100 250 Estimated Strength	Natural Fractures		Laboratory Testing	Comments	Elevation (m)
								Frequency	Spacing			
		Rock coring started at 8.0m below grade	167.6									
9		SALINA FORMATION Dolomitic Shale, slightly weathered to 9.7m, unweathered below, fine grained, laminated to thickly bedded, grey, very strong, with white weak to medium strong gypsum beds above 10.0m	8.0	R1 TCR = 96% SCR = 88% RQD = 47%	167			3			8.0m: close to very close bedding joints, smooth and rough, planar, coatings/filling of gypsum and carbonaceous material	167
								5				
								6				
			166.1	R2 9.5 TCR = 89% SCR = 86% RQD = 36%	166			4				
10								2				166
			165.4	R3 10.2 TCR = 98% SCR = 91% RQD = 46%	165			5				165
								6				
								5				
11								2				
								4				
			163.9		164			4				164

END OF COREHOLE

11.7m

APPENDIX A2

Record of Borehole Sheets

Golder





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
FoS	factor of safety

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
ν	Poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma - u$)
σ'_{vo}	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stress (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

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

(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 = $\tan \delta$
c'	effective cohesion
c_u, s_u	undrained shear strength ($\phi = 0$ 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

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

Notes: 1
2

$$\tau = c' + \sigma' \tan \phi'$$

$$\text{shear strength} = (\text{compressive strength})/2$$



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
DS	Denison type sample
FS	Foil sample
RC	Rock core
SC	Soil core
SS	Split-spoon
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, 60° cone attached to "A" size drill rods for a distance of 300 mm (12 in.).

PH: Sampler advanced by hydraulic pressure

PM: Sampler advanced by manual pressure

WH: Sampler advanced by static weight of hammer

WR: Sampler advanced by weight of sampler and rod

Piezo-Cone Penetration Test (CPT)

A electronic cone penetrometer with a 60° conical tip and a project end area of 10 cm² 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) Non-Cohesive (Cohesionless) Soils

Density Index	N
Relative Density	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	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

Per cent 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 (non-cohesive (cohesionless)) or With (cohesive)	Sand and Gravel Silty Clay with sand / Clayey Silt with sand




PROJECT 12-1111-0088		RECORD OF BOREHOLE No 13-11		SHEET 1 OF 1		METRIC											
G.W.P. 2177-08-00		LOCATION N 4758285.1 ; E 343768.6		ORIGINATED BY SB													
DIST Central HWY QEW		BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers		COMPILED BY AV													
DATUM Geodetic		DATE June 25, 2013		CHECKED BY MM													
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa					WATER CONTENT (%)			γ	GR SA SI CL
								20 40 60 80 100					20 40 60				
175.7	GROUND SURFACE							○ UNCONFINED + FIELD VANE					W _p W W _L				
0.0	ASPHALT (250 mm)							● QUICK TRIAXIAL × REMOULDED					20 40 60				
0.2	Sand and gravel, trace silt, trace clay (FILL)		1	SS	26		175										
174.9	Compact Grey Moist		2	SS	6		174										
0.8	CLAYEY ORGANIC SILT, some silt, trace sand		3	SS	10		173										
	Firm to stiff		4	SS	7		172										
	Dark grey to black Wet		5	SS	13		171										
172.8	SILTY CLAY, trace sand		6	SS	16		170										
2.9	Stiff to very stiff		7	SS	23		169										
	Brown and grey Wet		8	SS	48		168										
171.1	Sandy CLAYEY SILT, some gravel (TILL)		9	SS	34												
4.6	Very stiff to hard																
	Brown Wet																
168.7	SAND and GRAVEL, some silt, trace clay (TILL)																
7.0	Dense Grey Wet																
167.2	END OF BOREHOLE AUGER REFUSAL																
8.5	NOTE: 1. Water level in open borehole at a depth of 4.1 m below ground surface (Elev. 171.6 m) upon completion of drilling.																

PROJECT 12-1111-0088		RECORD OF BOREHOLE No 13-12		SHEET 1 OF 1		METRIC										
G.W.P. 2177-08-00		LOCATION N 4758255.1 ; E 343799.6		ORIGINATED BY SB												
DIST Central HWY QEW		BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers		COMPILED BY AV												
DATUM Geodetic		DATE June 25, 2013		CHECKED BY MM												
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								
175.6	GROUND SURFACE															
0.0	ASPHALT (250 mm)															
0.2	Sand and gravel, trace to some silt, trace clay (FILL) Compact Grey Moist		1	SS	14											43 45 8 4
			2	SS	22											
174.2																
1.4	CLAYEY ORGANIC SILT, some silt, trace sand, containing sand pockets Firm to stiff Dark grey to black Wet		3	SS	7											
			4	SS	7											
			5	SS	5											
			6	SS	7											
169.8																
5.8	SAND and GRAVEL, some silt, some clay (TILL) Loose Grey Wet		7	SS	7											32 35 19 14
168.9																
6.7	END OF BOREHOLE AUGER REFUSAL															
	NOTE: 1. Water level in open borehole at a depth of 3.9 m below ground surface (Elev. 171.7 m) upon completion of drilling.															

PROJECT 12-1111-0088		RECORD OF BOREHOLE No 13-13		SHEET 1 OF 1		METRIC											
G.W.P. 2177-08-00		LOCATION N 4758274.2; E 343818.4		ORIGINATED BY SB													
DIST Central HWY QEW		BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers		COMPILED BY AV													
DATUM Geodetic		DATE June 25, 2013		CHECKED BY MM													
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					W _p W W _L WATER CONTENT (%)			γ kN/m ³	GR SA SI CL
							20 40 60 80 100	20 40 60 80 100	20 40 60								
175.8	GROUND SURFACE																
0.0	ASPHALT (250 mm)																
0.2	Sand and gravel, trace silt, trace clay (FILL) Loose to compact Grey Moist		1	SS	23		175										
			2	SS	5												
174.4																	
1.4	CLAYEY ORGANIC SILT, some silt, trace sand, containing rootlets Soft to firm Dark grey to black Wet		3	SS	7		174										
			4	SS	4		173										
			5	SS	3		172										
171.3	SILTY CLAY, some sand, trace to some gravel Firm Grey Moist to wet		6	SS	4		171										6 20 39 35
170.2																	
5.6	SAND and GRAVEL, trace silt, trace clay (TILL) Loose Grey Wet		7	SS	9		170										58 32 5 5
169.2																	
6.6	END OF BOREHOLE AUGER REFUSAL NOTE: 1. Water level inside auger at a depth of 4.2 m below ground surface (Elev. 171.6 m) upon completion of drilling. 2. Split spoon refusal at a depth of 6.6 m below ground surface (Elev. 169.2 m).																

GTA-MTO 001 T:\PROJECTS\2012\12-1111-0088 [URS, VARIOUS STRUCTURE REPLACEMENT, QEW]\LOG\12-1111-0088.GPJ GAL-GTA.GDT 12/24/14

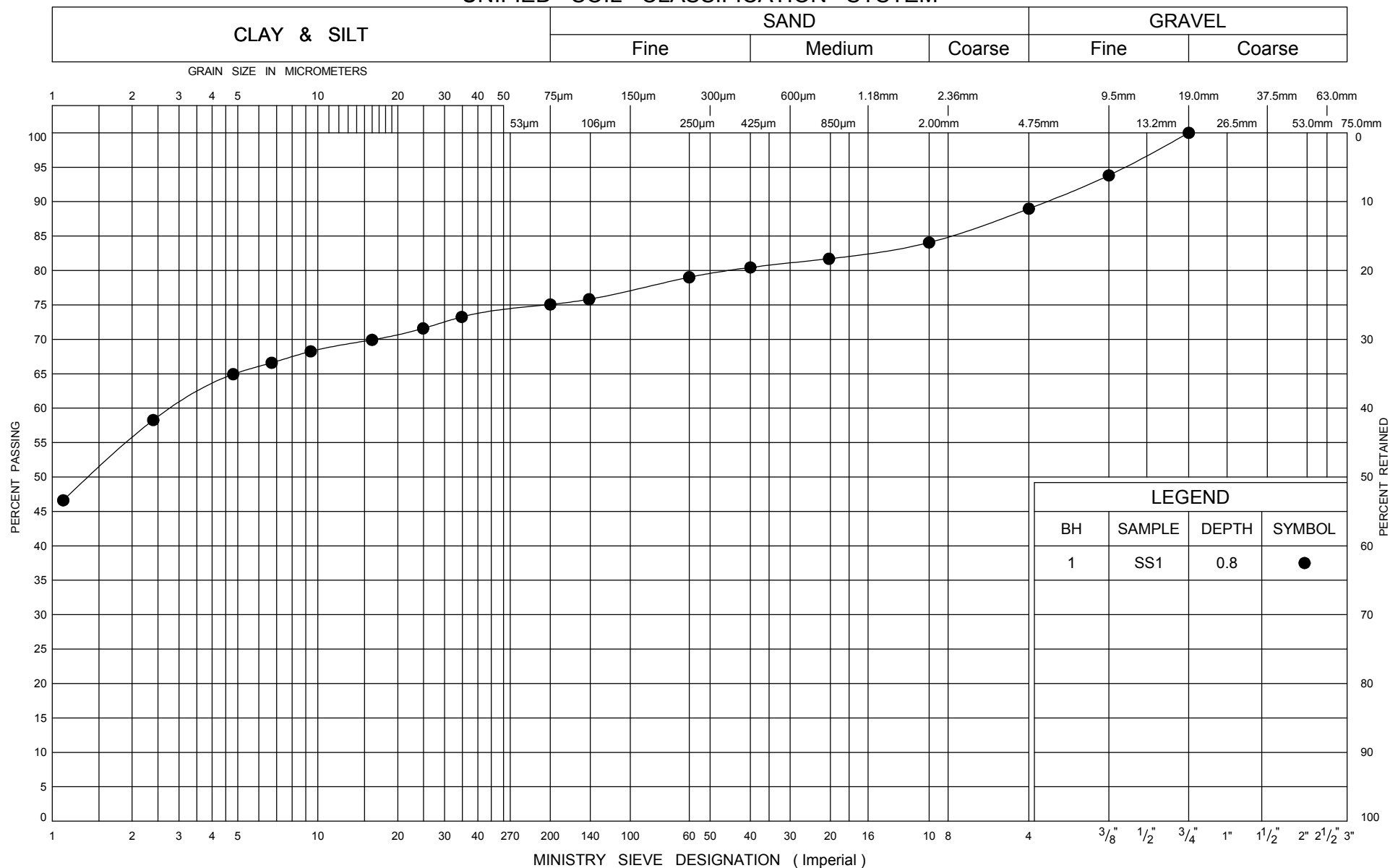
PROJECT 12-1111-0088		RECORD OF BOREHOLE No 13-14		SHEET 1 OF 1		METRIC											
G.W.P. 2177-08-00		LOCATION N 4758302.1 ; E 343785.7		ORIGINATED BY SB													
DIST Central HWY QEW		BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers		COMPILED BY AV													
DATUM Geodetic		DATE June 25, 2013		CHECKED BY MM													
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					WATER CONTENT (%) W _p — W — W _L			γ	GR SA SI CL
175.8	GROUND SURFACE							20 40 60 80 100									
0.0	ASPHALT (300 mm)							20 40 60 80 100									
0.3	Sand and gravel, trace silt, trace clay (FILL)		1	SS	30		175										
175.0	Compact Brown Moist		2	SS	8												
0.8	SILTY CLAY to CLAY, trace sand, trace organics		3	SS	9		174										
	Soft to stiff		4	SS	7												
	Brown becoming grey below a depth of 1.4 m		5	SS	5		173										0 3 39 58
	Wet																
			6	SS	2		172										
							171										OC = 3.9%
			7	SS	20		170										
170.2	Sandy CLAYEY SILT, some gravel (TILL)																
5.6	Very stiff Brown Wet						169										
168.6	SAND and GRAVEL, some silt, trace to some clay (TILL)		8	SS	72		168										36 42 12 10
7.2	Very dense Grey Wet																
167.2	END OF BOREHOLE AUGER REFUSAL																
8.6	NOTE: 1. Water level in open borehole at a depth of 4.3 m below ground surface (Elev. 171.5 m) upon completion of drilling.																

PROJECT 12-1111-0088		RECORD OF BOREHOLE No 15-01		SHEET 1 OF 1		METRIC										
W.P. 2177-08-00		LOCATION N 4758299.2 E 343782.6		ORIGINATED BY OS												
DIST Central HWY QEW		BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers and NQ Core Barrel		COMPILED BY JFC												
DATUM Geodetic		DATE May 12 and 13, 2015		CHECKED BY MM												
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa							WATER CONTENT (%)	
175.8	GROUND SURFACE						20	40	60	80	100					
0.0	Augered to a depth of 6.1 m															
169.7																
6.1	Sandy CLAYEY SILT, some gravel (TILL) Very stiff to hard Brown Wet		1	SS	30											
168.6																
7.2	SILTY SAND and GRAVEL, trace clay (TILL)		2	SS	84											
166.6																
9.3	DOLOMITIC SHALE (BEDROCK) Bedrock cored between depths of 9.25 m and 12.2 m. Refer to Record of Drillhole 15-01 for bedrock coring details.		3	SS	100/0.1											
			1	RC	REC 100%											RQD = 26%
			2	RC	REC 100%											RQD = 0%
			3	RC	REC 96%											RQD = 15%
163.6																
12.2	END OF BOREHOLE NOTES: 1. Water was noted inside hollow stem auger at a depth of 5.1 m below ground surface (Elev. 170.7 m) completion of drilling. 2. Borehole backfilled with portland cement grout upon completion.															

APPENDIX B1
Laboratory Test Results
Terraprobe Inc.



UNIFIED SOIL CLASSIFICATION SYSTEM



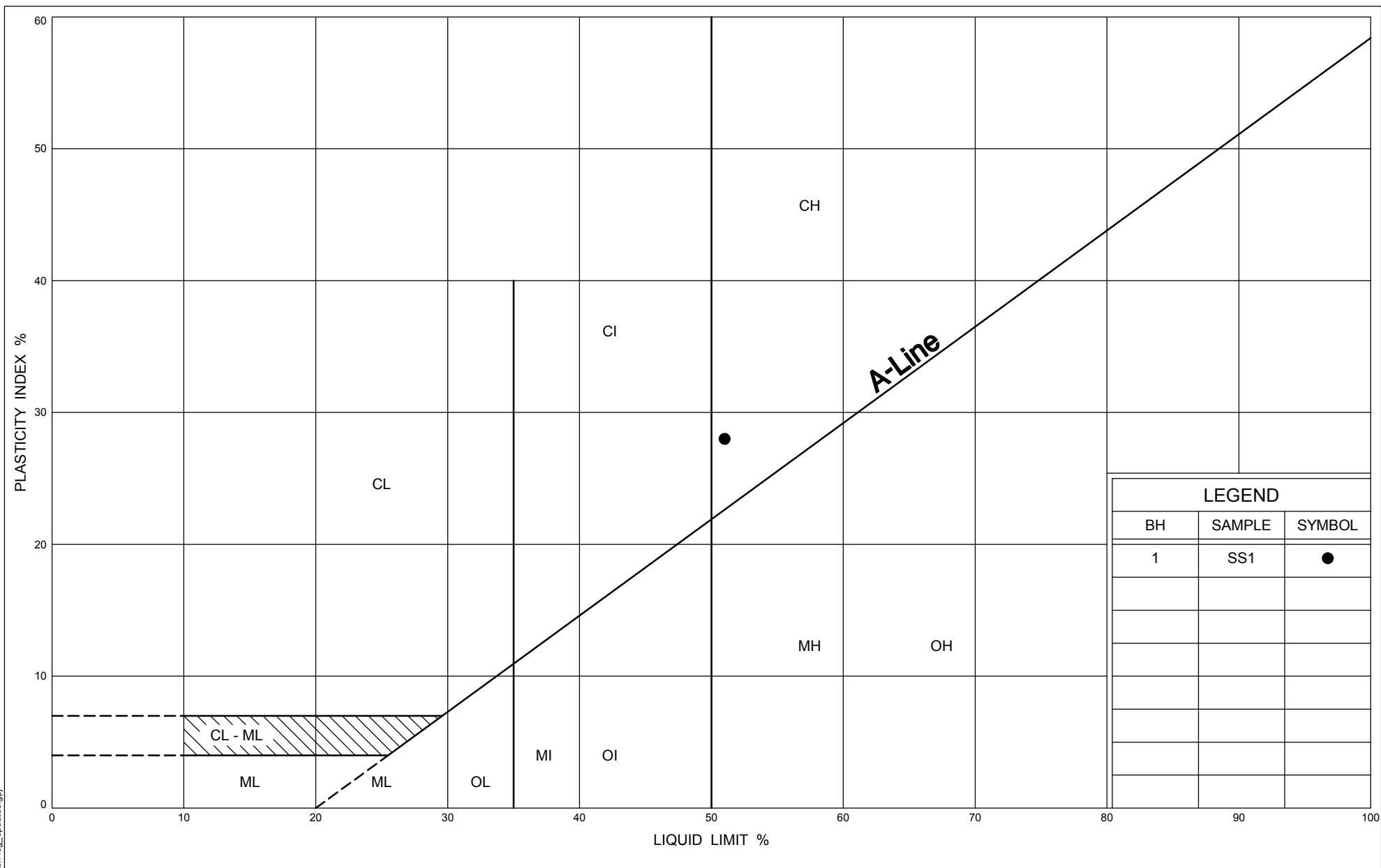
Ministry of
Transportation

GRAIN SIZE DISTRIBUTION FILL - CLAY

FIG No B1

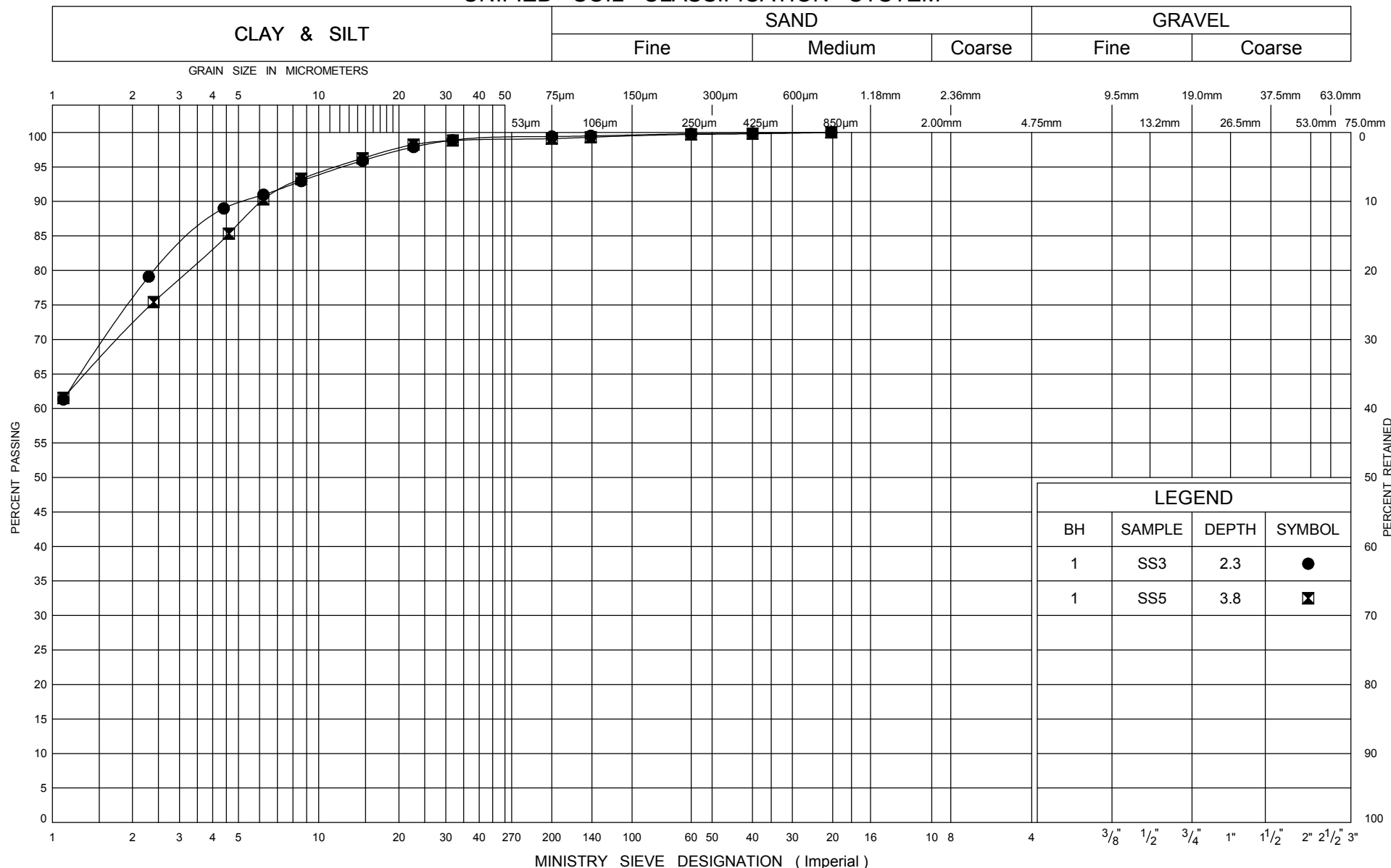
G W P

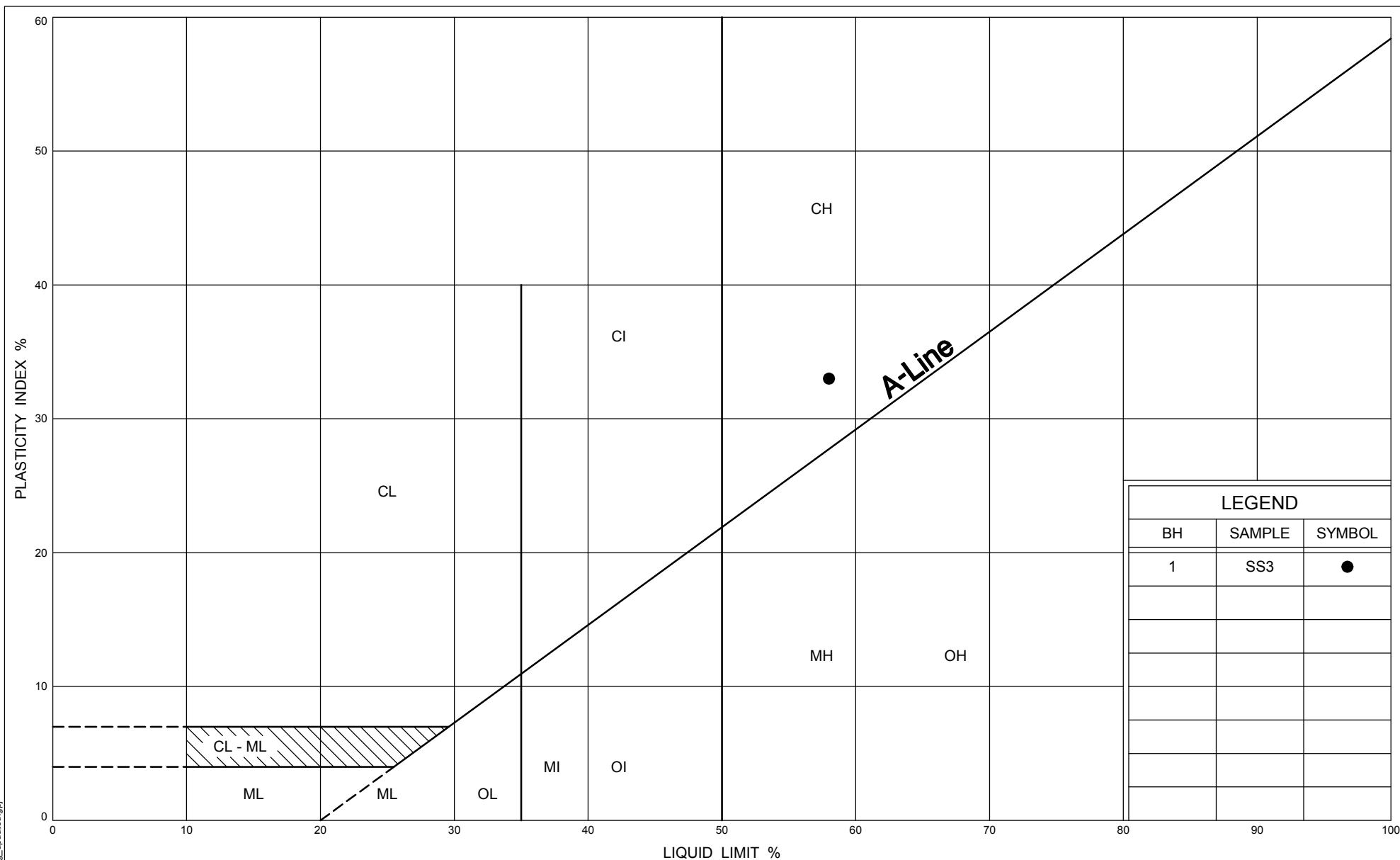
Black Creek Bridge Structures



file: 1-15-0689 black creek bh log_updated.gpj

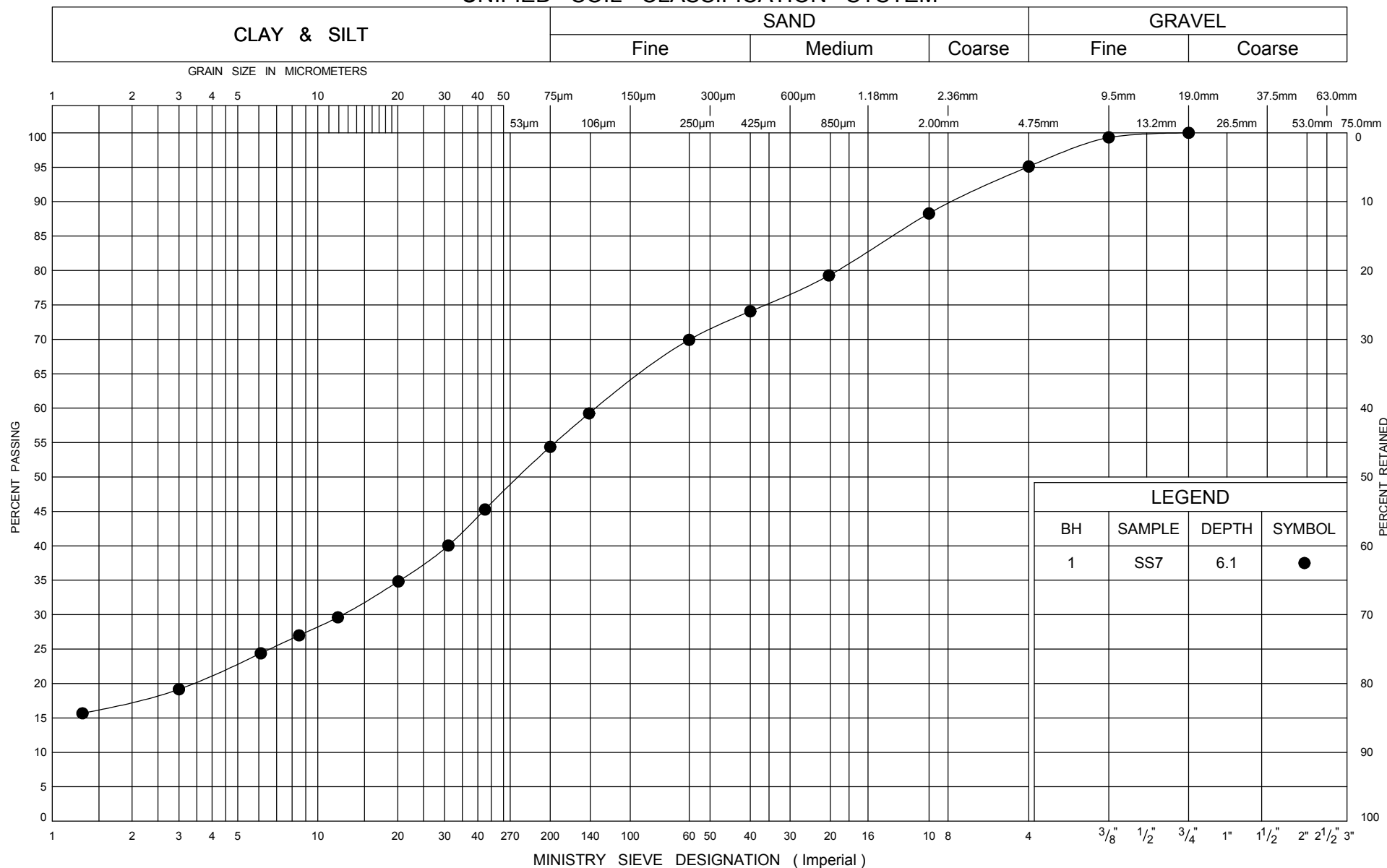
UNIFIED SOIL CLASSIFICATION SYSTEM





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UNIFIED SOIL CLASSIFICATION SYSTEM



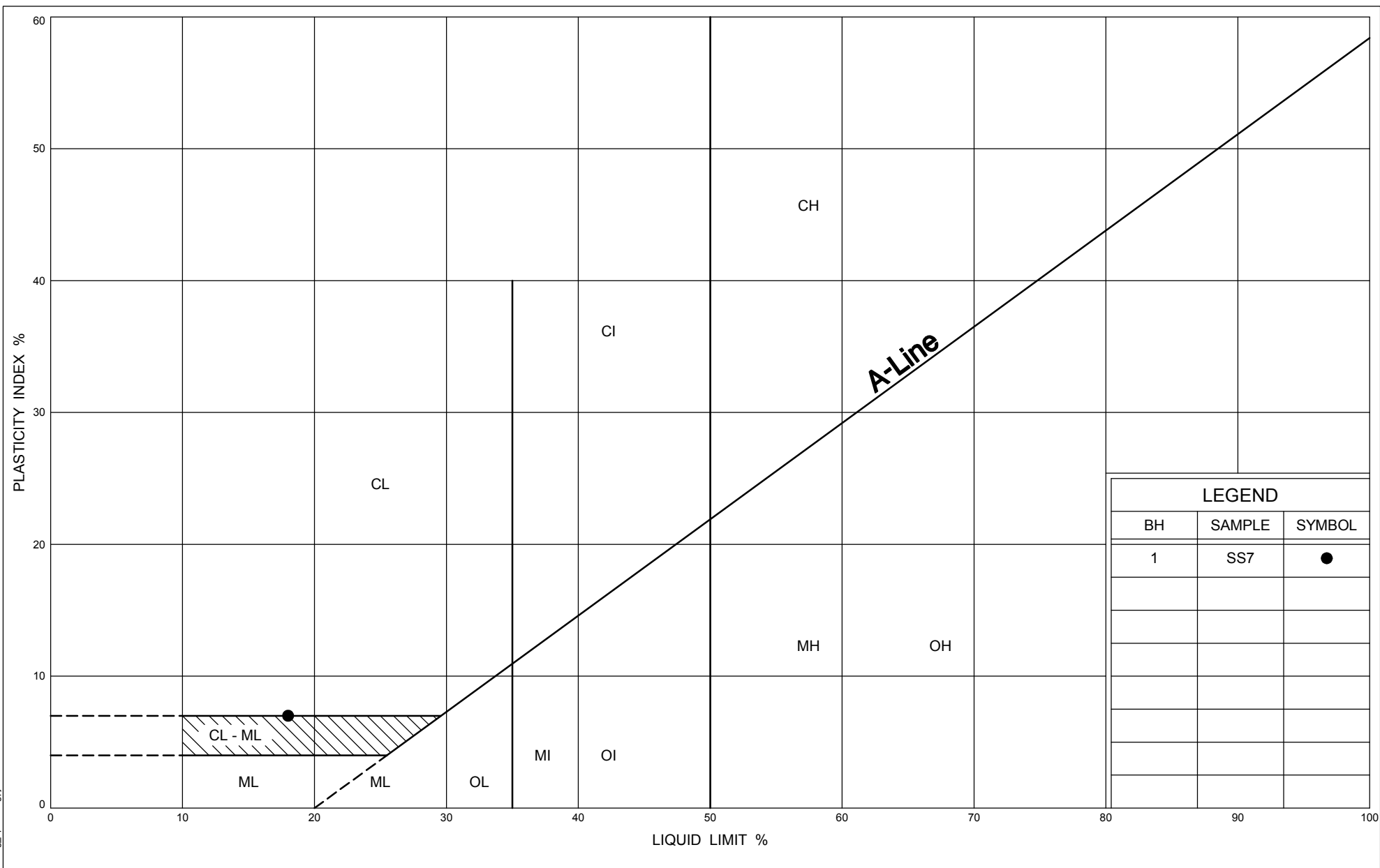
Ministry of
Transportation

GRAIN SIZE DISTRIBUTION CLAYEY SILT AND SAND

FIG No B5

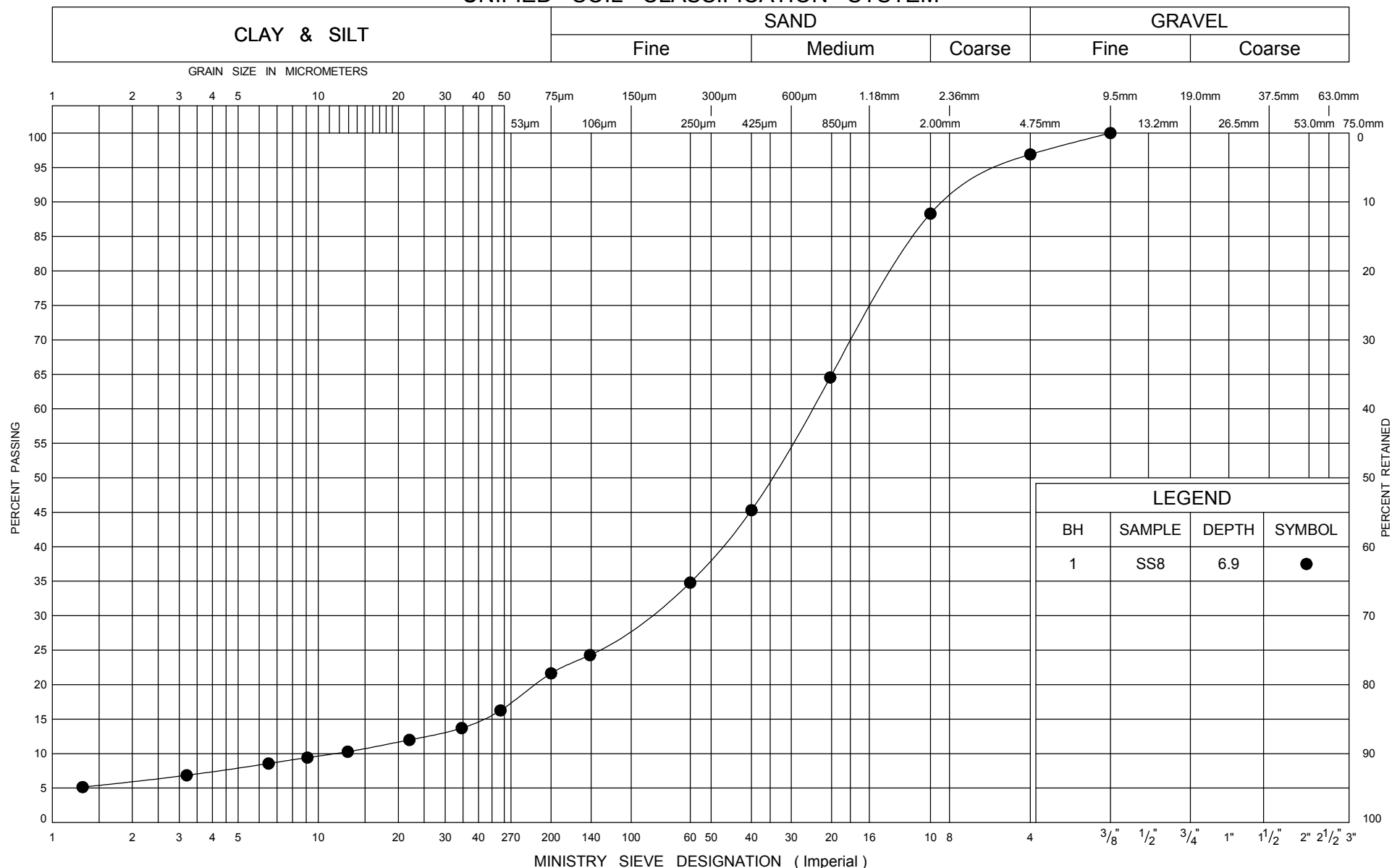
G W P

Black Creek Bridge Structures



file: 1-15-0689 black creek bh log_updated.gpj

UNIFIED SOIL CLASSIFICATION SYSTEM



PHOTOGRAPHS OF BEDROCK CORE SAMPLES

FIGURE B8

Black Creek Bridge Structures
Borehole No. 1



Project No. : 1-15-0689
Date : February, 2016



Prepared by : SD
Checked by : RA

APPENDIX B2

Laboratory Test Results

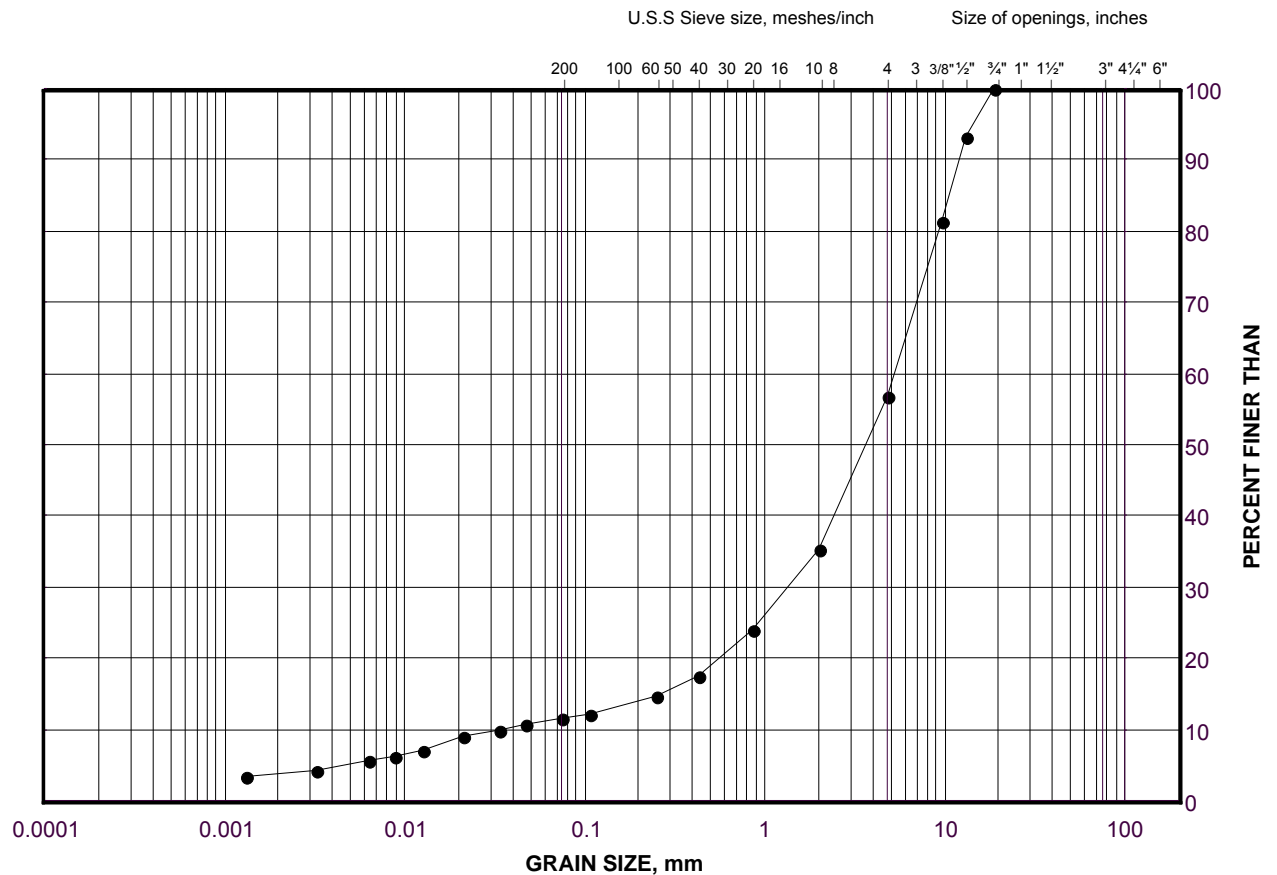
Golder



GRAIN SIZE DISTRIBUTION

Sand and Gravel Fill

FIGURE B1



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

LEGEND

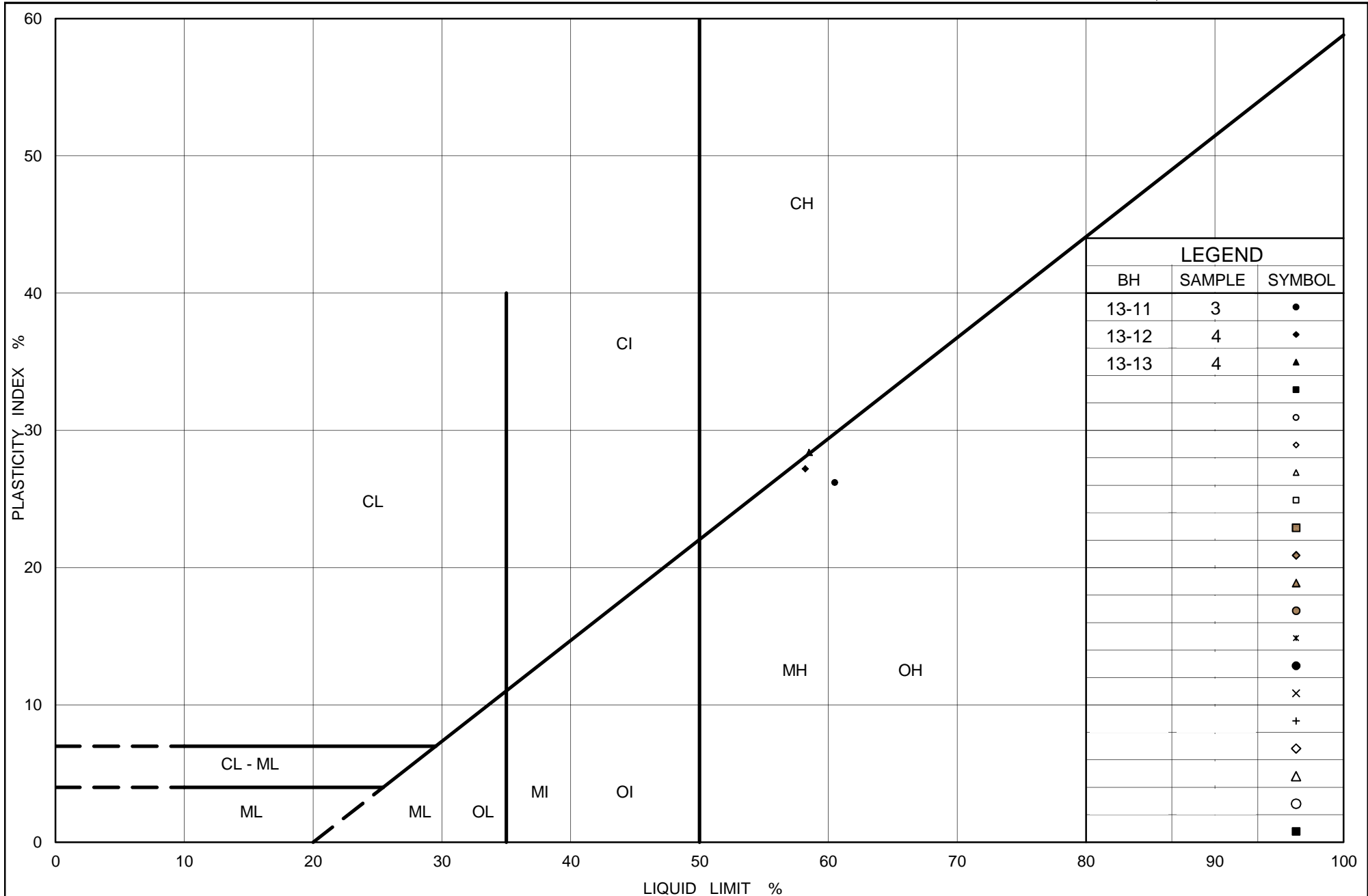
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
•	13-12	1	175.1

Project Number: 12-1111-0088

Checked By: MM

Golder Associates

Date: 08-Oct-13



Ministry of
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PLASTICITY CHART CLAYEY ORGANIC SILT

Figure No. B2

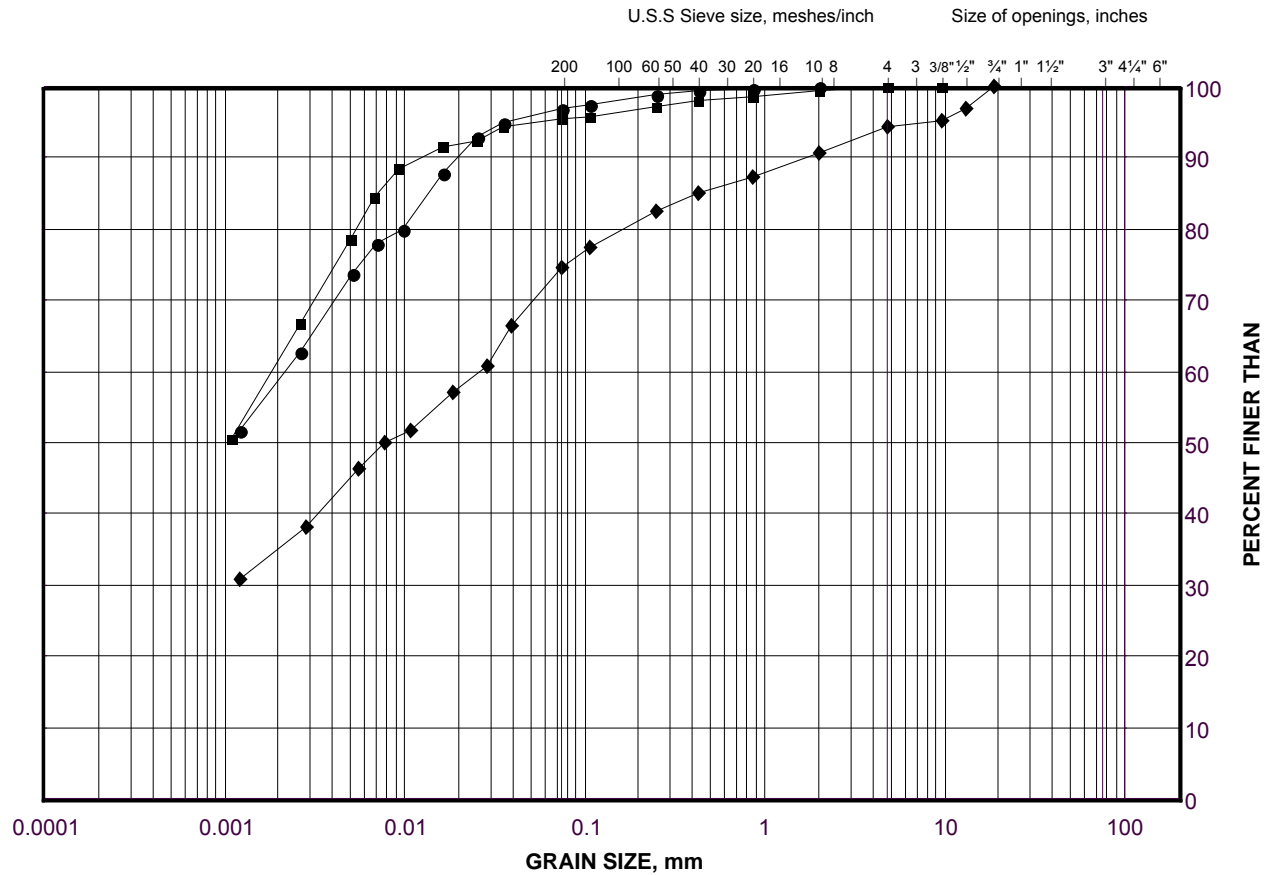
Project No. 12-1111-0088

Checked By: MM

GRAIN SIZE DISTRIBUTION

SILTY CLAY to CLAY

FIGURE B3



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

LEGEND

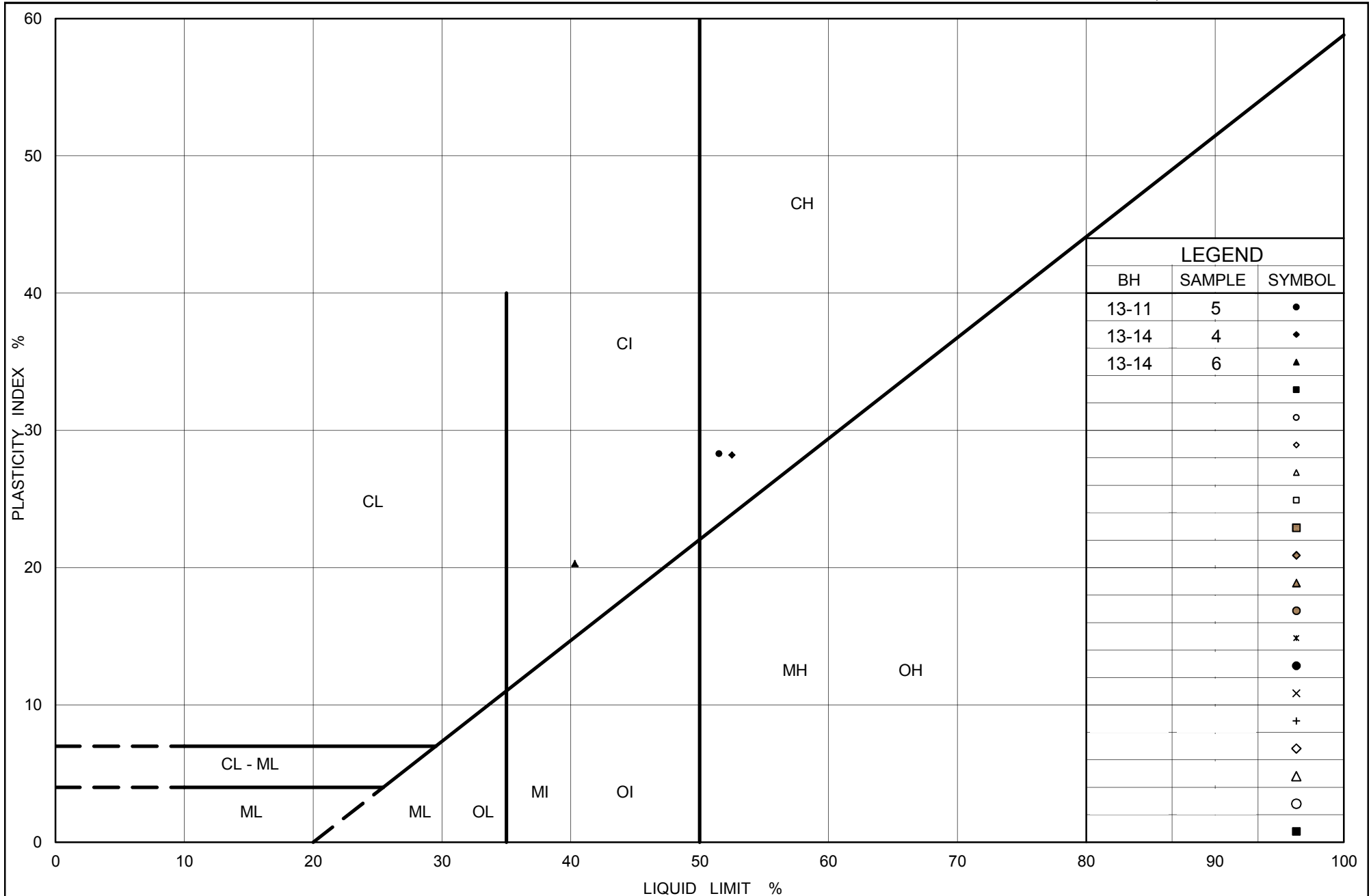
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	13-14	4	173.2
■	13-11	5	172.3
◆	13-13	6	170.9

Project Number: 12-1111-0088

Checked By: MM

Golder Associates

Date: 08-Oct-13



Ministry of
Transportation

Ontario

PLASTICITY CHART SILTY CLAY to CLAY

Figure No. B4

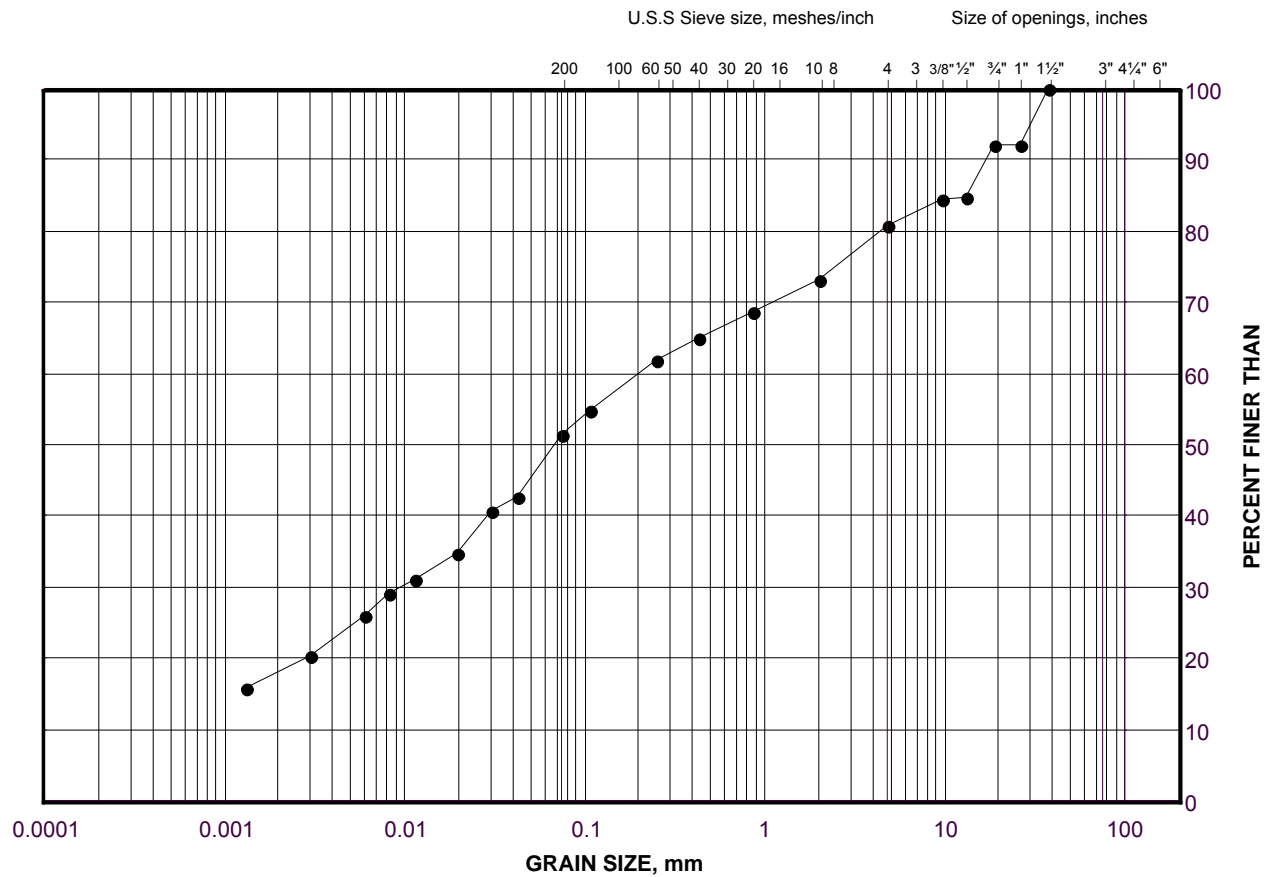
Project No. 12-1111-0088

Checked By: MM

GRAIN SIZE DISTRIBUTION

Sandy CLAYEY SILT Till

FIGURE B5



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

LEGEND

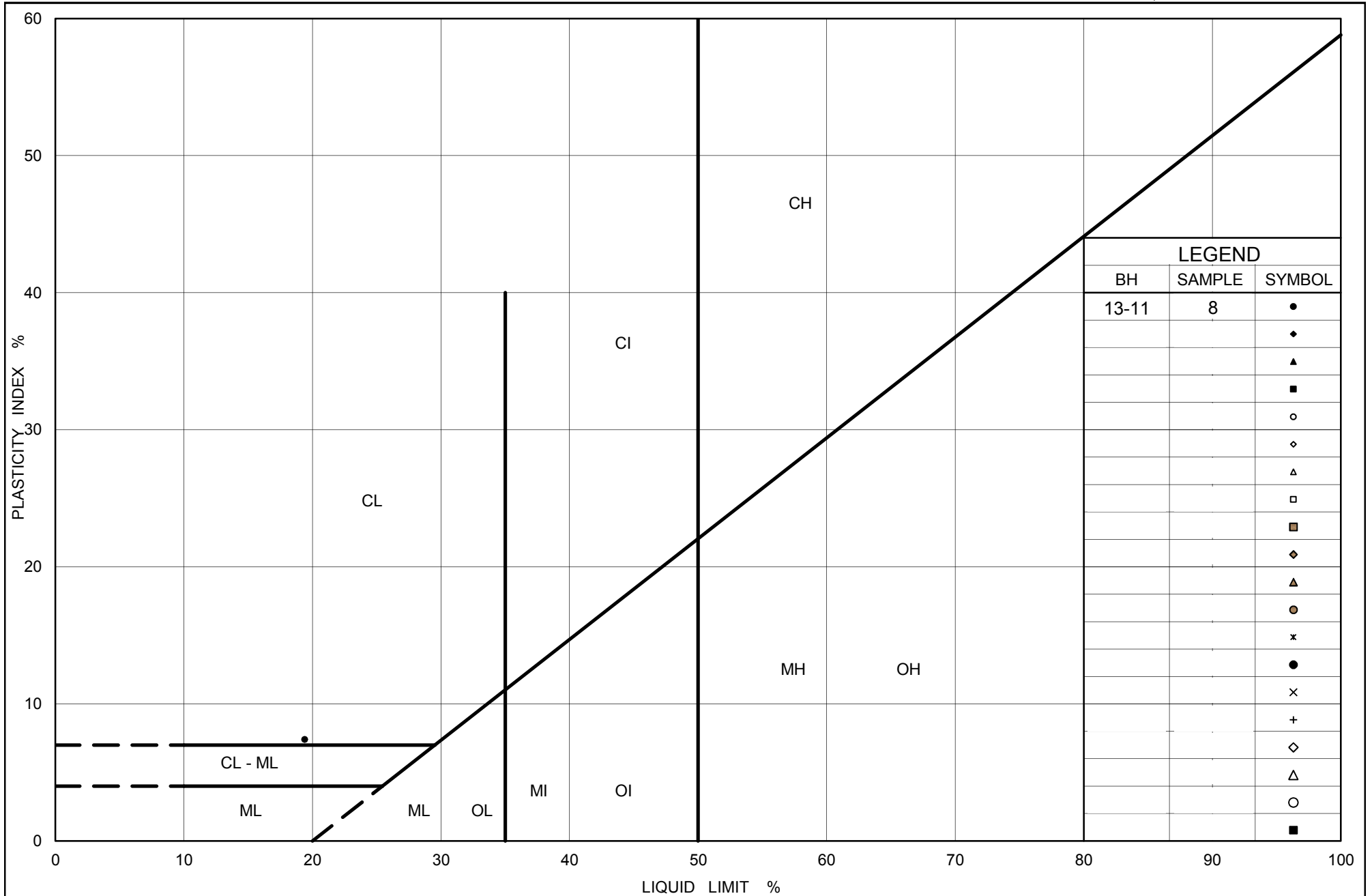
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
•	13-11	8	169.3

Project Number: 12-1111-0088

Checked By: MM

Golder Associates

Date: 08-Oct-13



Ministry of
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PLASTICITY CHART CLAYEY SILT TILL

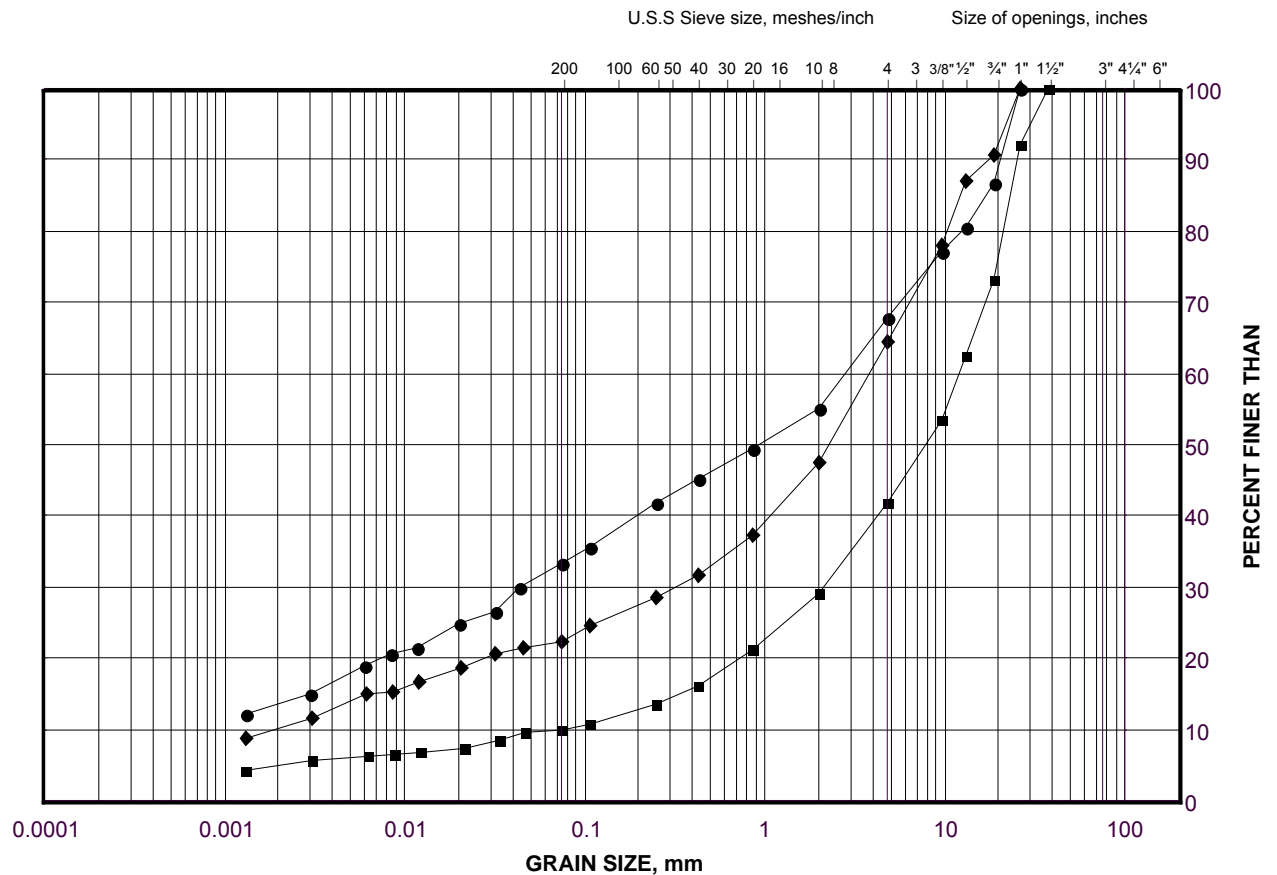
Figure No. B6

Project No. 12-1111-0088

Checked By: MM

SÁND and GRAVEL Till

FIGURE B7



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

LEGEND

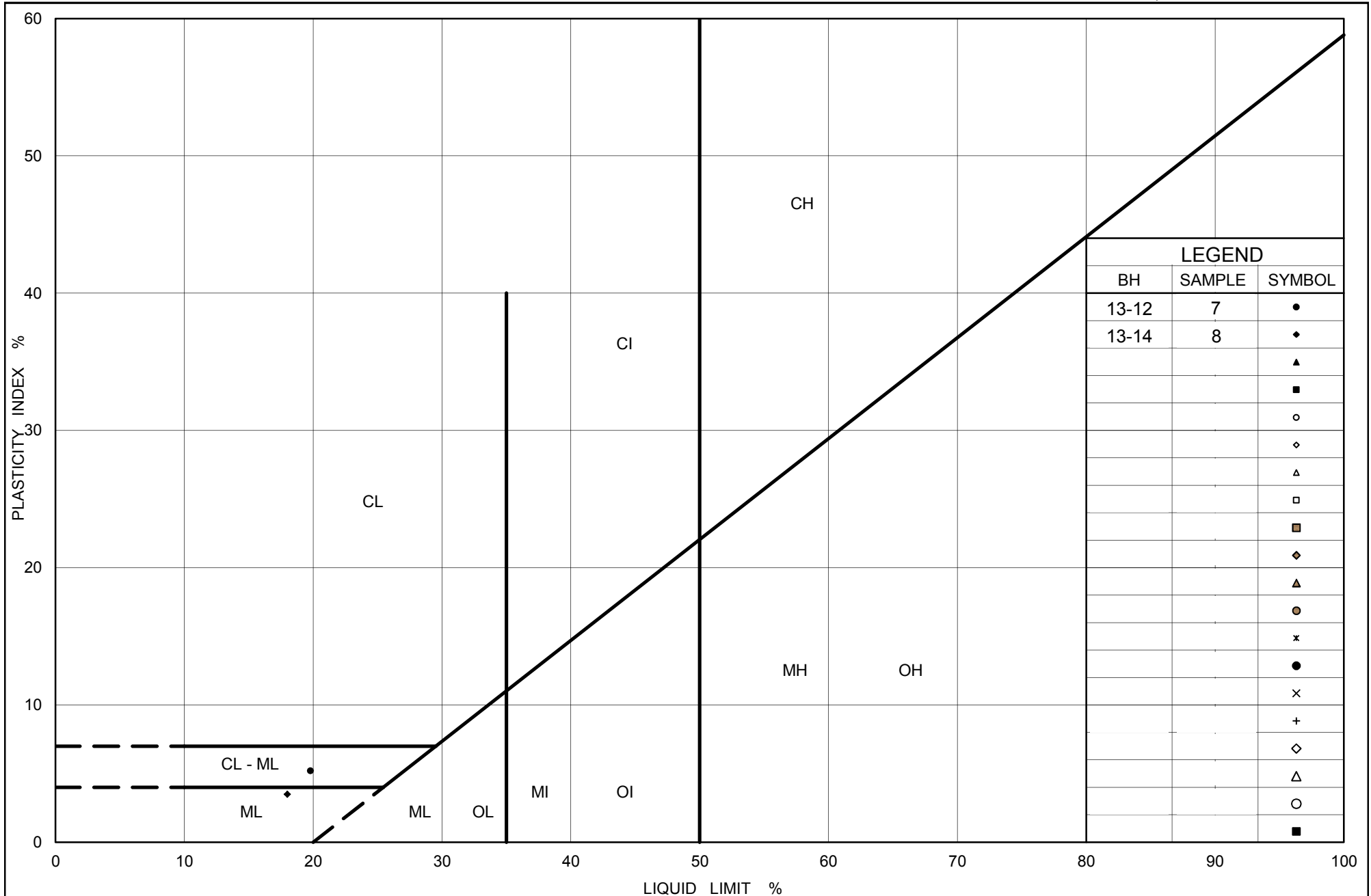
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	13-12	7	169.2
■	13-13	7	169.4
◆	13-14	8	167.9

Project Number: 12-1111-0088

Checked By: MM

Golder Associates

Date: 08-Oct-13



Ministry of
Transportation

Ontario

PLASTICITY CHART

Sand and Gravel Till

Figure No. B8

Project No. 12-1111-0088

Checked By: MM

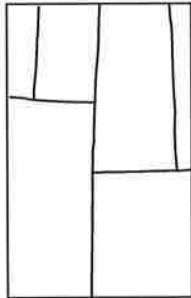


BEFORE COMPRESSION



AFTER COMPRESSION

TABLE 1 - UNCONFINED COMPRESSION TEST (UC) OF INTACT ROCK CORE SPECIMENS**ASTM D7012**

SAMPLE IDENTIFICATION			
PROJECT NUMBER	12-1111-0088	SAMPLE NUMBER	-
PROJECT NAME	Various / 5 Structure Replacement / QEW	SAMPLE DEPTH, m	9.53-9.68
BOREHOLE NUMBER	-	DATE:	05/14/15
TEST CONDITIONS			
MACHINE SPEED, mm/min	N/A	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST,min	>2 <15	L/D	2.23
SPECIMEN INFORMATION			
SAMPLE HEIGHT, cm	14.04	WATER CONTENT, (specimen) %	12.11
SAMPLE DIAMETER, cm	6.30	UNIT WEIGHT, kN/m ³	22.98
SAMPLE AREA, cm ²	31.14	DRY UNIT WT., kN/m ³	20.50
SAMPLE VOLUME, cm ³	437.34	SPECIFIC GRAVITY	-
WET WEIGHT, g	1025.20	VOID RATIO	-
DRY WEIGHT, g	914.46		
VISUAL INSPECTION		FAILURE SKETCH	
			
TEST RESULTS			
STRAIN AT FAILURE, %	N/A	COMPRESSIVE STRENGTH, MPa	15.7

REMARKS:

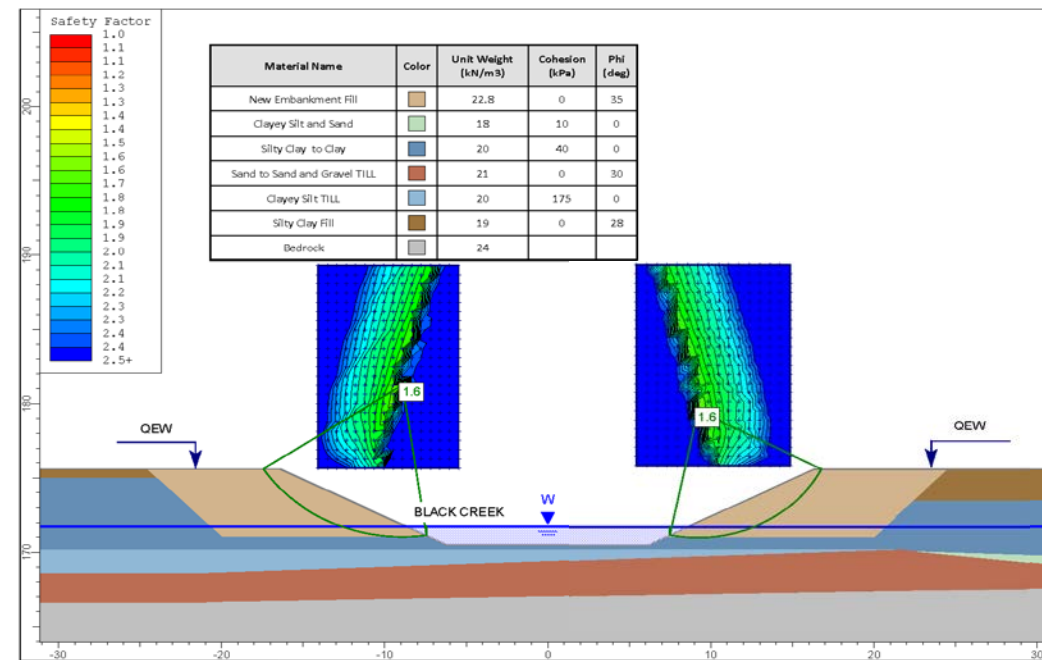
Checked By: MM

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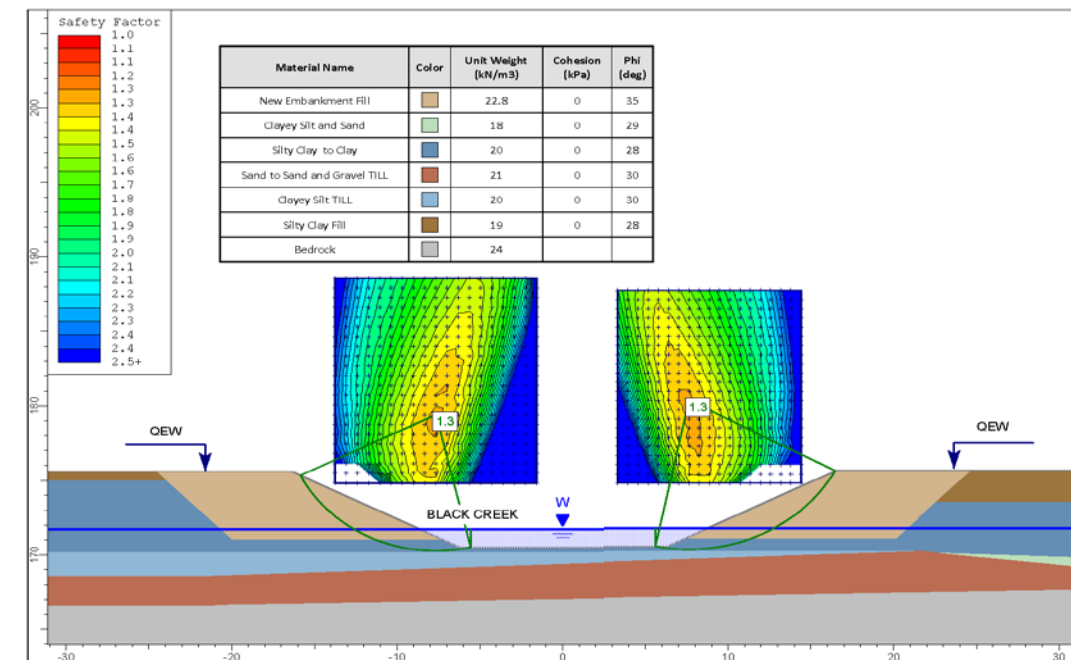
APPENDIX C

Slope Stability Analysis

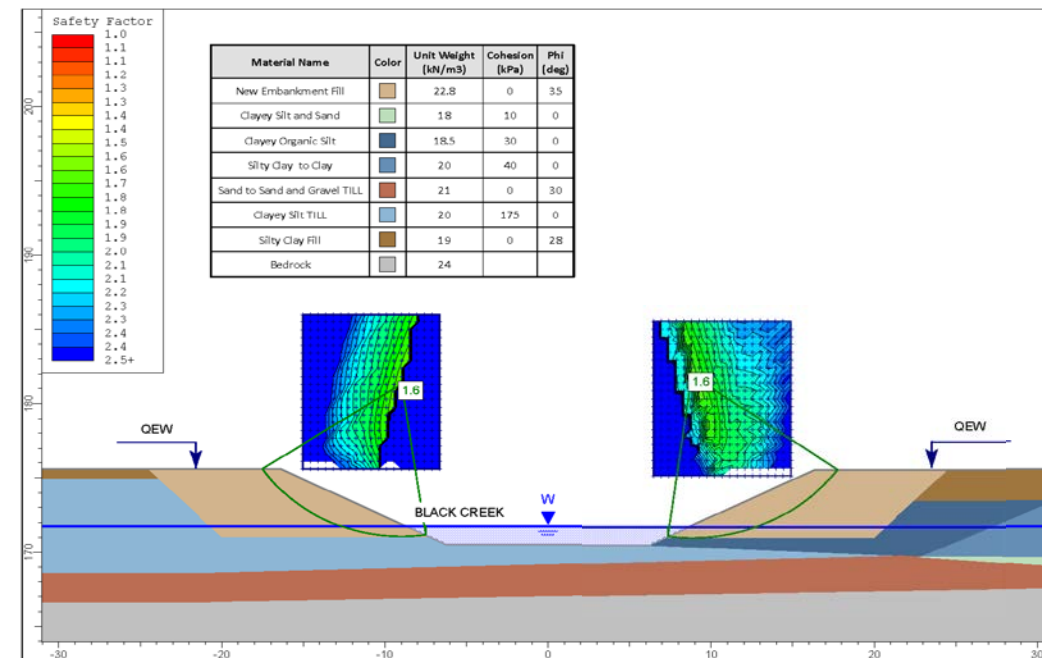




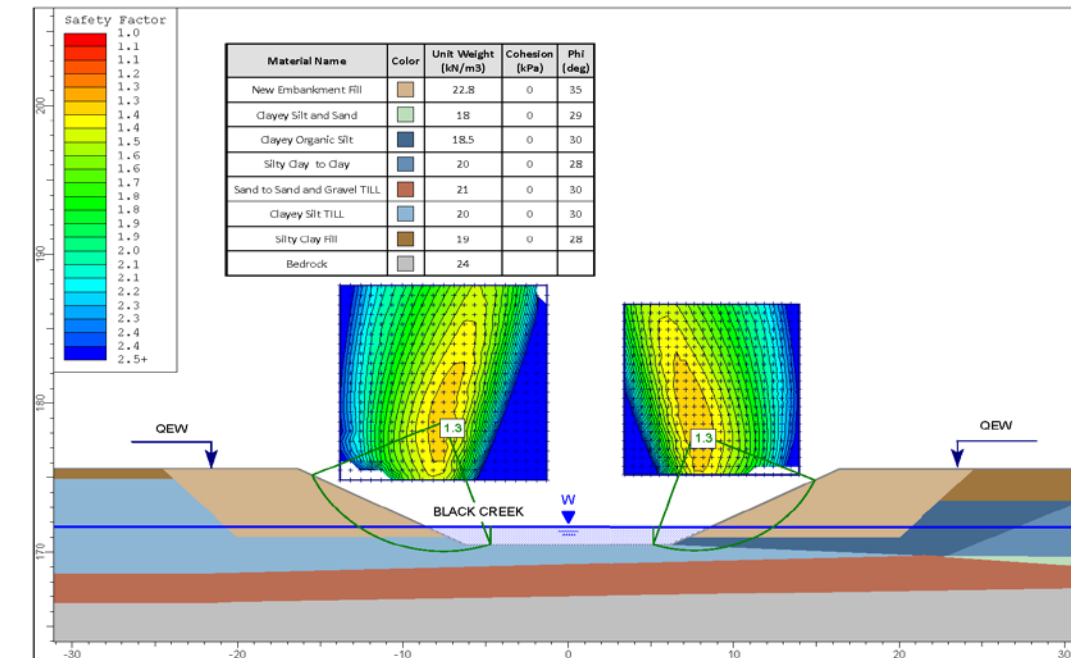
Black Creek North-Bound Structure
Forward Slope Stability (Total Stress Analysis)



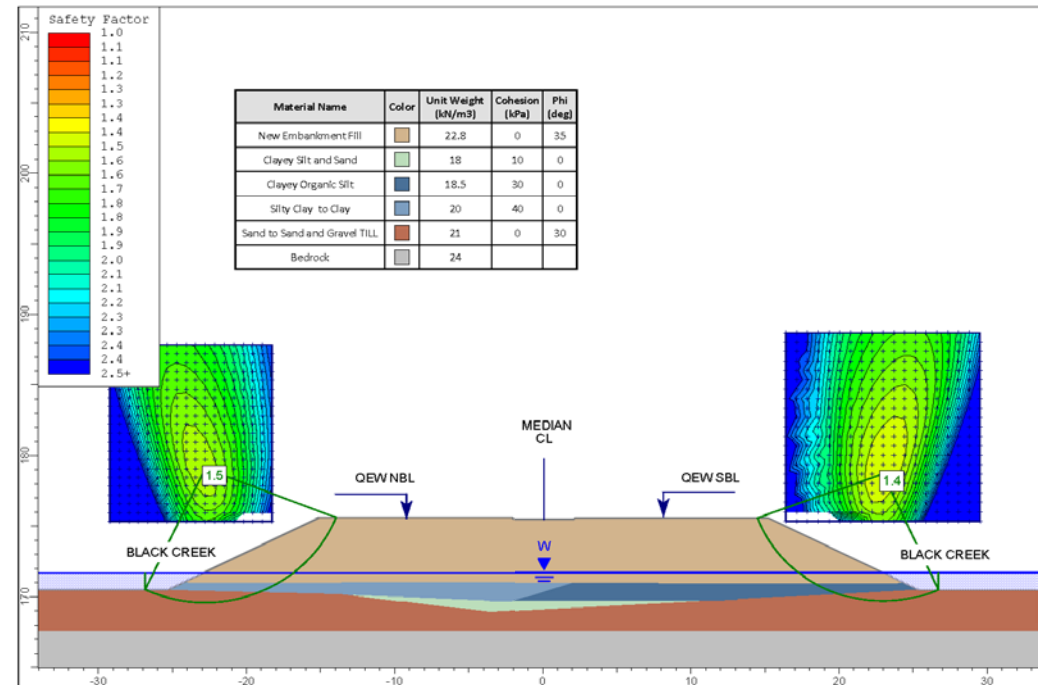
Black Creek North-Bound Structure
Forward Slope Stability (Effective Stress Analysis)



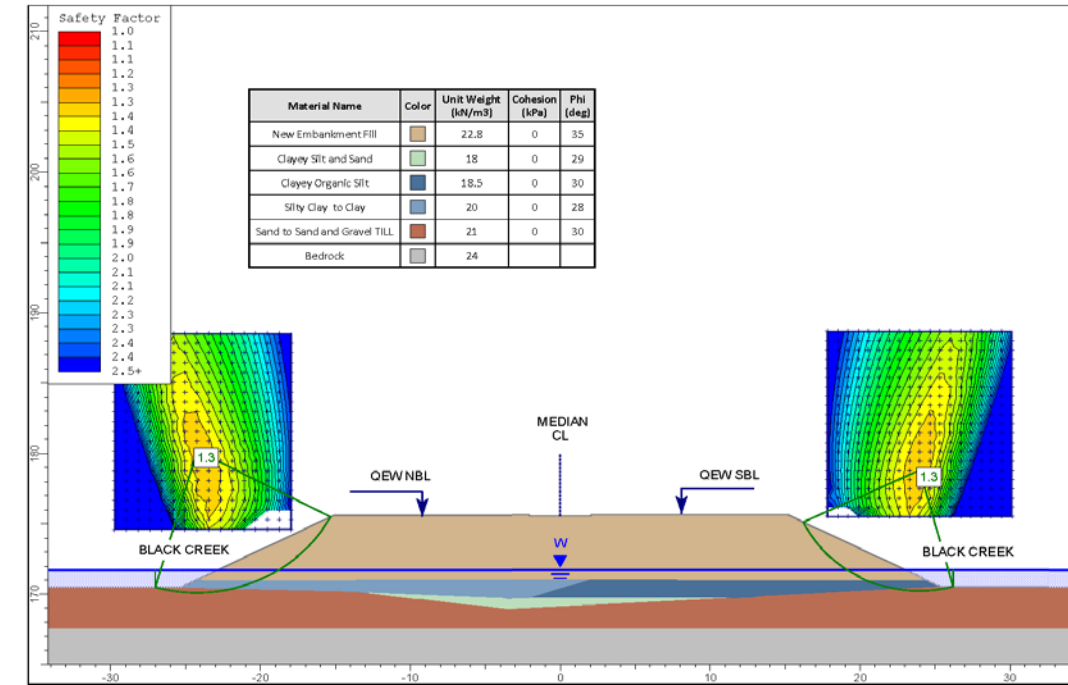
Black Creek South-Bound Structure
Forward Slope Stability (Total Stress Analysis)



Black Creek South-Bound Structure
Forward Slope Stability (Effective Stress Analysis)



Black Creek North-Bound and South-Bound Structures
South Approach Abutment (Total Stress Analysis)



Black Creek North-Bound and South-Bound Structures
South Approach Abutment (Effective Stress Analysis)