



April 25, 2017

FOUNDATION INVESTIGATION AND DESIGN REPORT

**STRAIGHT LAKE NBL BRIDGE STRUCTURE, SITE NO. 44-461/1
HIGHWAY 69 FOUR-LANING FROM 1.7 KM NORTH OF HIGHWAY 529
NORTHERLY TO 3.9 KM NORTH OF HIGHWAY 522
MINISTRY OF TRANSPORTATION, ONTARIO
GWP 5347-08-00; WP 5145-08-01**

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REPORT





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OC	Obstructions
OC	Artesian Conditions
NSSP	Drilled Steel Casings – Drilling and Cleaning/Flushing of Rock Sockets
NSSP	Laboratory Testing of Granular Materials for Pad Construction (to Support Spread Footings)



PART A

FOUNDATION INVESTIGATION REPORT

STRAIGHT LAKE NBL BRIDGE, SITE NO. 44-461/1

HIGHWAY 69 FOUR-LANING FROM 1.7 KM NORTH OF HIGHWAY 529

NORTHERLY TO 3.9 KM NORTH OF HIGHWAY 522

MINISTRY OF TRANSPORTATION, ONTARIO

GWP 5347-08-00; WP 5145-08-01



1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by URS Canada Inc. (URS) on behalf of the Ministry of Transportation, Ontario (MTO) to provide foundation engineering services for the proposed Highway 69 northbound lane (NBL) bridge over Straight Lake (Site No. 44-461/1) which is within the Contract 5 limits of the new Highway 69 alignment. The proposed work in Contract 5 is part of the overall four-laning of Highway 69 from 1.7 km north of Highway 529 northerly to 3.9 km north of Highway 522, for a total distance of 19.7 km, which includes: high fill embankments and embankments over swamps; the Canadian National Railway (CNR) re-alignment; the Bekanon Road and Highway 522 interchanges and structures; the Still River, Straight Lake and Key River structures; the Canadian Pacific Railway and CNR overpass structures; as well as culvert crossings. The general location of this proposed bridge along the new Highway 69 four-laning alignment is shown on the Index Plan on Drawing 1.

The Terms of Reference and the Scope of Work for the foundation investigation are outlined in MTO's Request for Proposal, dated December 2008. Golder's proposal (Scope of Work) for foundation engineering services associated with the Contract 5 Straight Lake bridge is contained in Section 6.8 of URS's Technical Proposal for this assignment. The work has been carried out in accordance with Golder's Supplementary Specialty Plan for foundation engineering services for this project, dated April 19, 2010.

This report addresses the investigation carried out for the Straight Lake NBL bridge only. Separate reports address the foundation investigations for the related swamp crossings and high fill areas, culverts and other bridge structures for the project.

The purpose of this investigation is to establish the subsurface conditions at the proposed bridge location, by borehole drilling, rock coring, in situ testing and laboratory testing on selected soil and rock core samples. The foundation units/limits for this investigation were located in the field by Callon Dietz Inc. (Callon Dietz), a professional surveying company retained by URS. The investigation area is shown in plan on Drawing 2.

Golder completed the investigation for the NBL bridge in the following two phases:

- Phase 1 in 2013: Two boreholes were advanced within Straight Lake along the proposed NBL centreline, at two potential pier locations, following which the investigation was carried out for the high fill area immediately north of Straight Lake (High Fill Swamp 502, reported under separate cover).
- Phase 2 in 2014: The remaining boreholes for the approach embankments and for the bridge were advanced after the span arrangement and location of piers/abutments were established by URS based on preliminary input by Golder from the Phase 1 investigation.

Subsequent to completion of the Phase 2 field investigation in 2014 and a detailed assessment of the subsurface conditions, embankment stability and settlement at the north abutment and approach locations, the layout of the proposed bridge piers was revised and the length of the structure extended to the north. This change to the General Arrangement was agreed upon by MTO, URS and Golder and finalized in February 2016 after detailed comparison of several options to take advantage of more suitable foundation conditions and minimize potential foundation mitigation measures that would otherwise be required for the north abutment and approach embankment. As such, select pertinent boreholes advanced along the proposed NBL centreline within High Fill Swamp 502 north of Straight Lake are also referenced in this report.

Preliminary subsurface information for this project is available and was supplied by the MTO, specifically:



- Preliminary Foundation Investigation and Design Report for Structural Areas (Foundation Investigation 2), Highway 69 Four Laning, From 3.5 km North of Highway 559 to 3.8 km North of Highway 522, GWP 5377-02-00, GEOCRE No. 41H-57, dated July 2006, by Amec Earth and Environmental.

2.0 SITE DESCRIPTION

The proposed Highway 69 alignment is oriented generally in a south-north direction spanning the Township of Wallbridge to the south, the Township of Henvey and the Henvey Inlet First Nation Reserve No. 2 and the Township of Mowat to the north. The Contract 5 section of the new four-lane Highway 69 alignment is also oriented generally in a south-north direction within the overall project limits, for a total distance of about 1.6 km in the Henvey Inlet First Nation Reserve No. 2. The proposed Straight Lake bridge is located approximately 1.1 km east of the existing Highway 69 alignment and 1.5 km from the northern limit of Contract 5, corresponding to approximately 9.0 km north of the junction of the existing Highway 69 and Highway 526.

In general, the topography of this section of the overall project limits consists of rolling terrain, including sparsely or densely treed areas and numerous bedrock outcrops separated by valleys and swamps containing areas of standing water and various types of vegetation and organic soils. At the Straight Lake bridge site, at the south abutment and along the south approach, a bedrock outcrop protrudes from the lake surface (at about Elevation 178 m and extends up to about Elevation 190 m, resulting in an outcrop up to about 12 m high above the lake level. At the north abutment and north approach, the ground surface in the high fill swamp area gradually increases in elevation from the north shore of the lake to the north abutment at Elevation 190.3 m, about 12 m high above the lake level.

3.0 INVESTIGATION PROCEDURES

3.1 Foundation Investigation

The fieldwork for the investigation for High Fill 502 north of Straight Lake and for the Phase 1 bridge investigation, (Boreholes B502-01 and B502-02), was carried out in February, 2013. The fieldwork for the Phase 2 bridge investigation, which includes the remaining B502-series boreholes, was carried out in February and March, 2014. A total of 18 boreholes were advanced for the Phases 1 and 2 bridge investigation, supplemented with a total of 11 boreholes and three DCPTs advanced along the proposed NBL centreline and near each proposed embankment toe in High Fill 502 north of Straight Lake. A summary of the boreholes and their respective locations relative to each foundation element and approach area is presented below. The locations of the boreholes are shown in plan on Drawing 2 and the Record of Borehole/Drillhole sheets are presented in Appendix A.

Foundation Element/Approach Area	Borehole No.
South Approach	B502-03
	B502-04
South Abutment	B502-05
	B502-06
	B502-07
	B502-08



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Foundation Element/Approach Area	Borehole No.
Between South Abutment and Pier 1	B502-09
	B502-15
	B502-16
	B502-17
	B502-18
Pier 1 Area	B502-01
Pier 2 Area	B502-10
	B502-11
Pier 3 Area	B502-02
Between Piers 3 and 4	B502-12
	S502-18
Pier 4 Area	B502-13
North Abutment Area	S502-24
North Approach, including Front Slope	B502-13, B502-14, S502-08, S502-12, S502-18 to S502-20, S502-20A, S502-20B and 502-21 to S502-24, S502-DC02, S502-DC08 and S502-DC09

The field investigation was carried out using a variety of drilling equipment as a result of the varying nature of the terrain and accessibility within the Contract 5 project limits. The details of the drilling equipment and suppliers are listed below.

Drilling Equipment	Supplied and Operated By
Skid Mounted Diedrich D-25	Landcore Drilling of Sudbury, Ontario
Skid Mounted Diedrich D-25	Walker Drilling Ltd. of Utopia, Ontario
Portable Drilling Equipment	OGS Inc. of Almonte, Ontario

The boreholes on land were advanced to depths between 0.1 m (on a bedrock outcrop) and 19.7 m and the boreholes in the lake were advanced to depths of up to 53.4 m below the ice surface, through an ice/water column between 1.2 m and 4.0 m deep.

The boreholes were advanced through the overburden using 150 mm outer diameter (O.D.) solid-stem augers, and 'BW' and/or 'NW' casing with wash boring techniques. In general, soil samples were taken at intervals of depth of about 0.75 m and 1.5 m, using a 50 mm O.D. split-spoon sampler using a manual hammer (rope and cat head) on both the portable and skid-mounted equipment, and carried out in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586). Samples of cohesive soils were obtained at selected locations/depths using 76 mm O.D. thin-walled 'Shelby' tubes (ASTM D1587) for relatively undisturbed samples. Field vane shear tests were carried out in cohesive soils for assessment of undrained shear strengths (ASTM D2573) using MTO Standard 'N' size vanes. Bedrock coring was carried out using an 'NQ' core barrel. Photographs of the recovered rock core samples are provided in Appendix B.



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The groundwater conditions were observed during the drilling operations and all boreholes were backfilled upon completion in accordance with Ontario Regulation 903, Wells (as amended). A standpipe piezometer was installed in each of Boreholes B502-07 and B502-13 to monitor the groundwater level at these locations. The piezometer consists of 32 mm diameter PVC pipe, with slotted screen sealed within the sand to sand and gravel and the sand strata in the respective boreholes. The borehole and annulus surrounding the piezometer pipes above the screen (and sand pack) was backfilled to near the ground surface with bentonite pellets and soil cuttings, topped with a bentonite seal at the ground surface. Piezometer installation details and water level readings are described on the Record of Borehole sheets presented in Appendix A. The water level in the piezometers was measured on March 19, 2014.

The field work was observed by members of our engineering and technical staff, who located the boreholes, arranged for the clearance of underground services, observed the drilling, sampling and in situ testing operations, logged the boreholes, and examined and cared for the soil and rock samples. The samples were identified in the field, placed in appropriate containers, labelled and transported to our Mississauga geotechnical laboratory where the samples underwent further visual examination and laboratory testing. All of the laboratory tests were carried out to MTO and/or ASTM Standards, as appropriate. Classification testing (water content, organic content, grain size distribution and Atterberg limits) was carried out on selected samples. Strength testing, consisting of uniaxial (unconfined) compression and point load index, was carried out on selected specimens of the rock core. The results of the laboratory testing are included in Appendix B.

For the Phase 1 investigation, the proposed centreline of the new highway alignment was staked in the field by Callon Dietz prior to drilling and the as-drilled borehole locations and ground/ice surface elevations were measured/surveyed by a member of our technical staff in reference to the centreline stakes. For the Phase 2 investigation, the boreholes were located in the field and the ground/ice surface elevations were surveyed by Callon Dietz prior to drilling. The locations given on the Record of Borehole/Drillhole sheets and shown on Drawing 2 are positioned relative to MTM NAD 83 northing and easting coordinates and the ground surface elevations are referenced to Geodetic datum. The borehole locations, ground surface elevations and drilled depths are summarized below.

Borehole No.	Location (MTM NAD 83)		Water/Ground Surface Elevation (Ice/Water Column)* (m)	Borehole Depth (m)
	Northing	Easting		
B502-01	5082947.9	223072.2	178.6*	15.7**
B502-02	5083086.2	223014.2	178.6*	31.8**
B502-03	5082876.8	223101.9	190.2	0.1
B502-04	5082891.5	223090.4	185.7	8.6
B502-05	5082895.1	223088.8	185.7	7.9**
B502-06	5082895.2	223094.2	186.0	9.4**
B502-07	5082895.3	223099.6	186.4	9.8**
B502-08	5082899.0	223098.0	186.3	3.2
B502-09	5082941.3	223074.9	178.6*	6.6**
B502-10	5083001.3	223049.8	178.5*	53.4**
B502-11	5083061.3	223024.7	178.5*	40.3**
B502-12	5083121.2	222999.6	179.3	18.0**
B502-13	5083167.3	222980.3	183.5	18.0**



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Borehole No.	Location (MTM NAD 83)		Water/Ground Surface Elevation (Ice/Water Column)* (m)	Borehole Depth (m)
	Northing	Easting		
B502-14	5083185.8	222972.5	186.2	14.8
B502-15	5082935.7	223069.1	178.6*	5.6**
B502-16	5082941.2	223066.8	178.6*	6.9
B502-17	5082941.5	223083.0	178.6*	2.0
B502-18	5082947.0	223080.6	178.6*	10.7**
S502-08	5083197.7	222942.6	190.6	19.7
S502-12	5083243.4	222922.1	192.1	12.5
S502-18	5083149.0	222988.0	181.3	14.7
S502-19	5083170.6	223007.3	180.9	9.8
S502-20	5083172.0	222978.4	184.1	13.5
S502-20A	5083189.5	222993.4	184.2	12.4
S502-20B	5083189.5	222993.9	184.2	6.8
S502-21	5083176.3	222956.2	187.3	16.5
S502-22	5083195.1	222968.7	188.0	18.7
S502-23	5083212.8	222978.6	188.7	16.3
S502-24	5083218.2	222959.0	190.3	17.7
S502-DC02	5083151.2	222966.7	183.4	12.7
S502-DC08	5083189.8	222994.3	184.2	13.0
S502-DC09	5083199.0	222951.4	190.0	20.3

*Ice/Water surface; Borehole Depth includes water column.

**Borehole Depth includes bedrock core length between 1.5 m and 3.6 m.

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

As delineated in The Physiography of Southern Ontario¹, this section of the new Highway 69 lies within the physiographic region known as the Georgian Bay Fringe, which extends along the east side of Georgian Bay through the Parry Sound and Muskoka areas, then eastward from Muskoka in patches into the area north of the Kawartha Lakes.

This part of the Georgian Bay Fringe physiographic region was never submerged during periods of glacial recession. As a result, the surficial soils in this area consist of relatively shallow deposits of sand, silt and clay underlain by metamorphic bedrock and numerous bare knobs and ridges of bedrock are present throughout the area. Localized low-lying swampy areas, containing peat and/or organic soils overlying soft/loose native soils, sometimes to significant depth, are present in valleys between the bedrock knobs and ridges.

The bedrock in the area consists typically of crystalline gneisses of the Britt Domain of the Central Gneiss Belt, a subdivision of the Grenville Structural Province, as described in Geology of Ontario (OGS Special Volume 4)².

¹ Chapman, L.J. and Putnam, D.F., 1984. The Physiography of Southern Ontario, Ontario Geological Survey, Special Volume 2, Third Edition. Accompanied by Map P.2715, Scale 1:600,000.

² Ontario Geological Society, 1991. Geology of Ontario, Society Special Volume 4, Part 2. Ministry of Northern Development and Mines, Ontario.



Deposition of Paleozoic strata initially covered the bedrock and later erosion during glaciation exposed these Precambrian rocks.

4.2 Subsurface Conditions

The detailed subsurface soil, bedrock and groundwater conditions as encountered in the boreholes advanced during this investigation, together with the results of the laboratory tests carried out on selected soil and bedrock core samples, are presented on the Record of Borehole and Drillhole sheets and on the laboratory test figures provided in Appendix A and Appendix B, respectively. The results of the in situ field tests (i.e., SPT 'N'-values and the undrained shear strengths obtained from the field vanes) as presented on the Record of Borehole sheets and in Sections 4.3 to 4.9 are uncorrected. The stratigraphic boundaries shown on the Record of Borehole sheets and on the stratigraphic profile and cross-sections are inferred from non-continuous sampling, observations of drilling progress and the results of SPTs and in situ testing. These boundaries, therefore, represent transitions between soil types rather than exact planes of geological change. The bedrock surface has been inferred from observations made during drilling and coring and generally represents a transition from overburden to the bedrock surface and should not be inferred to represent the exact surface elevation of the bedrock. Furthermore, subsurface conditions will vary between and beyond the borehole locations. It should be noted that the interpreted stratigraphy shown on Drawings 2 to 5 is a simplification of the subsurface conditions.

The subsurface conditions encountered at the site are characterized essentially by:

- At the south abutment/approach: deposits of silt to sand and/or sand and gravel, over granitic gneiss bedrock.
- Within the lake and on the north side of the lake: thick deposits of organic silt, clayey silt to clay, silt to sand, underlain by granitic gneiss bedrock.

The results of the unconfined compressive strength tests on the rock core samples (ASTM D7102 - Standard Test Method for Compressive Strength and Elastic Moduli of Intact Rock Core Specimens under Varying States of Stress and Temperatures) are presented in Tables B1 and B2 and the results of the laboratory testing on the soil samples are presented on Figures B1 to B9, in Appendix B. Photographs of the bedrock core samples are presented on Figures B10 to B13, inclusive, in Appendix B.

The quality of the bedrock with respect to the Rock Quality Designation (RQD) value is classified in accordance with Table 3.10 in the Canadian Foundation Engineering Manual (2006)³.

The degree of weathering of the bedrock samples (e.g. slightly weathered – W2) is based on field identification, and the strength classification of the intact rock mass (e.g. strong – R4) is based on laboratory testing of bedrock core samples and is described in accordance with the International Society for Rock Mechanics (ISRM⁴) standard classification system, as presented in Table 3.5 of CFEM (2006).

Axial and diametral point load strength index tests (ASTM D5731 – Standard Test Method for Determination of the Point Load Strength Index of Rock and Application to Rock Strength Classification) were carried out on selected samples of the bedrock core. The axial and diametral point load strength index values are shown on the Record of Drillhole sheets and are presented in Table B1 in Appendix B.

³ Canadian Geotechnical Society. 2006. Canadian Foundation Engineering Manual, 4th Edition. The Canadian Geotechnical Society c/o BiTech Publisher Ltd, British Columbia.

⁴ International Society for Rock Mechanics Commission on Test Methods, 1985. Int. J. Rock Mech.Min. Sci. & Geomech. Abstr. Vol 22, No. 2, pp. 51-60.



The relationship between Is_{50} and uniaxial compressive strength (UCS) values, given by correlation factor (K), varies depending on the size of the core sample and the strength of the rock. For the NBL (as well as for the SBL) bridge, using the consolidated rock strength data from both sites, an average correlation factor (K) was calculated by matching UCS test values and point load test values at similar depths from the same boreholes. The average correlation factor (K) of 14 was estimated.

A detailed description of the subsurface conditions encountered in the boreholes in the area of each approach/abutment and at/in the area of each pier is provided in the following sections. Borehole B502-09 and Boreholes B502-15 to B502-18 were advanced between the south abutment and Pier 1 and Borehole B502-12 was advanced between proposed Piers 3 and 4 and as these boreholes were not advanced in proximity of the foundation elements, the soil stratigraphy and laboratory test results are presented in this report but are not discussed further in the following sections of the report.

Groundwater and lake water levels are subject to seasonal fluctuations and precipitation events, and should be expected to be higher during wet periods of the year. For the lake boreholes, because the boreholes were advanced in the water, and water was introduced into the boreholes during the drilling process, the water level noted in the boreholes is not considered representative of the stabilized groundwater conditions.

4.3 South Abutment/Approach

A total of six boreholes (B502-03 to B502-08) were advanced in the vicinity of the proposed south abutment/approach. The interpreted stratigraphy at the south approach/abutment is shown in profile on Drawing 2 and in cross-section on Drawing 3.

4.3.1 Topsoil

A 0.1 m to 0.2 m thick layer of topsoil was encountered from ground surface at all the boreholes advanced at the south approach/abutment with the ground surface at Elevation 190.2 m at the south approach (Borehole B502-03) and between Elevations 186.4 m and 185.7 m at the south abutment (Boreholes B502-04 to B502-08).

4.3.2 Silt and Sand to Sand

A 2.3 m to 2.8 m thick deposit of brown to grey silt and sand, silty sand or sand was encountered below the topsoil in Boreholes B502-04 to B502-08 between Elevations 186.2 m and 185.5 m.

The SPT 'N'-values measured within the silt and sand to sand deposit range between 4 blows and 13 blows per 0.3 m of penetration indicating a loose to compact relative density.

The natural water content measured on samples of the deposit range between 8 per cent and 20 per cent.

The results of grain size distribution tests completed on five samples of the silt and sand to sand deposit are shown on Figure B1-A and B1-B in Appendix B.



4.3.3 Organic Silt to Organic Silt and Sand

A 0.2 m to 2.8 m thick deposit of dark brown to black organic silt or organic silt and sand was encountered below the silt and sand to sand in Boreholes B502-04 to B502-08 between Elevations 183.9 m and 182.7 m. Trace roots/wood fragments were encountered within the deposit in Boreholes B502-04 to B502-07.

The SPT 'N'-values measured within the deposit range between 3 blows and 21 blows per 0.3 m of penetration indicating a soft to very stiff consistency.

The natural water content measured on samples of the deposit range between 30 per cent and 65 per cent. The organic content measured on three samples of the deposit is about 4 per cent.

The results of grain size distribution tests completed on four samples of the deposit are shown on Figure B1-J in Appendix B.

Atterberg limits testing carried out on four samples of the organic silt to organic silt and sand yielded liquid limits between about 34 per cent and 44 per cent, plastic limits ranging between about 26 per cent and 31 per cent, and plasticity indices ranging between about 8 per cent and 13 per cent, indicating that the material has slight plasticity attributed to the organic material within the deposit. The results of the Atterberg limits tests are shown on the plasticity chart on Figure B2-A.

4.3.4 Silty Sand to Sand and Gravel

A deposit of brown to grey silty sand to sand and gravel was encountered below the organic silt or organic silt and sand deposit in Boreholes B502-04 and B502-06 to B502-08. A 0.8 m thick interlayer of silt was encountered between the upper layer of sand and gravel and the lower layer of silty sand in Borehole B502-04. The silty sand to sand and gravel deposit was encountered between Elevation 183.4 m and 179.9 m, and the thicknesses of the deposit ranges between 0.3 m and 3.6 m. Cobbles were inferred within the deposit at a depth of 6.0 m in Borehole B502-06 and at depths of 3.3 m, 3.9 m and 4.7 m in Borehole B502-07.

The SPT 'N'-values measured within this deposit range between 11 blows and 43 blows per 0.3 m of penetration, and is 10 blows per 0.3 m of penetration within the silt interlayer, indicating a compact to dense relative density. Several SPTs did not penetrate the full test depth, inferred as a result of contact with gravel/cobbles within the deposit.

The natural water content measured on samples of the silty sand to sand and gravel deposit range between 10 per cent and 14 per cent and measured on one sample of the silt interlayer in Borehole B502-04 is 19 per cent.

The result of two grain size distribution tests completed on samples of this deposit is shown on Figure B3.

4.3.5 Bedrock/Refusal

Bedrock core samples were recovered from Boreholes B502-05 to B502-07. Refusal to split-spoon advancement and/or casing refusal was encountered in Boreholes B502-04 and B502-08, while bedrock was exposed underlying the topsoil in Borehole B502-03. The corresponding bedrock surface elevations and refusal are summarized below.



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Foundation Element	Borehole	Bedrock Surface Elevation (m)	Comments
South Approach/Abutment	B502-03	190.1	Exposed Bedrock
	B502-04	177.1	Split-Spoon and Casing Refusal
	B502-05	181.2	Bedrock Cored
	B502-06	178.1	Bedrock Cored
	B502-07	180.1	Bedrock Cored
	B502-08	183.1	Split-Spoon Refusal

In general, the bedrock surface along the south approach and in the area of the proposed south abutment slopes downward from south to north with the bedrock surface elevation changing by as much as 6.0 m at the abutment borehole locations and up to 13.0 m relative to the approach borehole about 20 m south of the abutment.

Based on a review of the bedrock core samples recovered from Boreholes B502-05 to B502-07, the bedrock consists of granitic gneiss. In general the bedrock samples are described as fresh to moderately weathered, grey to dark grey to green to red, foliated, medium to coarse grained, weak to very strong, as presented in the Record of Drillhole sheets in Appendix A, and shown on the photographs of the recovered core samples on Figures B7 and B8 in Appendix B.

The Rock Quality Designation (RQD) measured on the core samples ranges from 30 per cent to 100 per cent, generally indicating a rock mass of poor to excellent quality as per Table 3.10 of CFEM (2006). The Total Core Recovery (TCR) of the samples obtained are between 82 per cent and 100 per cent.

The axial test carried out on six samples of the granitic gneiss bedrock core measured Is_{50} values ranging from 1.4 MPa to 12.7 MPa and the diametral tests carried out on nine samples of the granitic gneiss bedrock core measured Is_{50} values ranging from 2.9 MPa to 10.5 MPa, as presented in Table B1.

Based on the point load tests results at the south abutment, in accordance with Table 3.5 in CFEM (2006), the granitic gneiss bedrock is classified as weak to very strong (R2 to R5, 5 MPa < UCS < 250 MPa).

4.3.6 Groundwater Conditions

Borehole B502-03 was dry upon completion of drilling. The groundwater level in Boreholes B502-04 to B502-06 and B502-08, upon completion of drilling was measured between Elevation 184.1 m and 183.4 m, at depths between 2.2 m and 2.3 m below ground surface.

A standpipe piezometer was installed in Borehole B502-07 to allow monitoring of the groundwater level at the borehole location. Details of the piezometer installation are shown on the Record of Borehole sheet in Appendix A and the groundwater level measured in the piezometer is summarized below.

Foundation Element	Borehole	Ground Surface Elevation (m)	Groundwater Elevation (m)	Date of Measurement
South Abutment	B502-07	186.4	182.7	March 19, 2014



4.4 Pier 1 Area

A total of six boreholes (B502-01, B502-09 and B502-15 to B502-18) were advanced south of the proposed Pier 1. The interpreted stratigraphy at Borehole B502-01 located about 8 m south of Pier 1 is shown in cross-section on Drawing 3 and the interpreted stratigraphy perpendicular to the centreline of the foundation area south of Pier 1 is also shown on Drawing 3. The boreholes encountered variable depth of water, variable overburden thickness and bedrock surface steeply sloping to the north. The following section presents the results of Borehole BH502-01, which is the closest borehole to Pier 1 relative to the remaining boreholes.

4.4.1 Ice/Water

The ice surface at Borehole B502-01 in Straight Lake in February 2013 was at Elevation 178.6 m and the depth of ice/water was 4.0 m.

4.4.2 Organic Silt

A deposit of organic silt was encountered below the water in Borehole B502-01 with the top of the deposit encountered at Elevation 174.6 m and the thickness of the deposit is 4.8 m.

The SPT 'N'-values measured within the organic deposit are 0 blows (weight of rod) per 0.3 m of penetration suggesting a very soft consistency; and two in situ field vane tests carried out near the bottom of the organic silt deposit measured undrained shear strengths of 9 kPa and 14 kPa, with corresponding sensitivities of 1 and 2, indicating a very soft consistency.

The natural water content measured on a sample of the deposit is 135 per cent. The organic content measured on two samples of the deposit are about 9 per cent and 12 per cent.

An Atterberg limits test carried out on one sample of the organic deposit yielded a liquid limit of 62 per cent, a plastic limit of 45 per cent, corresponding to a plasticity index of 17 per cent, indicating that the material is classified as an organic silt of high plasticity. The result of the Atterberg limits test is shown on the plasticity chart on Figure B4.

4.4.3 Upper Silt

A 1.1 m thick deposit of grey silt was encountered below the organic silt at Elevation 169.8 m in Borehole B502-01.

One SPT 'N'-value measured within the deposit is 1 blows per 0.3 m of penetration indicating a very loose relative density.

The natural water content measured a sample of the deposit is about 24 per cent.

4.4.4 Clayey Silt

A 1.9 m thick deposit of grey clayey silt was encountered below the upper silt deposit in at Elevation 168.7 m in Borehole B502-01.



One SPT 'N'-value measured within the clayey silt deposit is 1 blow per 0.3 m of penetration. Two in situ field vane tests carried out within the clayey silt measured undrained shear strengths of 20 kPa and 35 kPa, indicating a soft to firm consistency and corresponding to sensitivities of 2 and 4.

The natural water content measured on a sample of the deposit is 46 per cent.

Atterberg limits tests carried out on a sample of the clayey silt yielded a liquid limit of 31 per cent, a plastic limit of 18 per cent, corresponding to a plasticity index of 13 per cent, indicating that the material is classified as clayey silt of low plasticity. The results of the Atterberg limits test are shown on the plasticity chart on Figure B5-A.

4.4.5 Lower Silt

A 0.9 m thick deposit of grey silt was encountered below the clayey silt deposit at Elevation 166.8 m in Borehole B502-01.

One SPT 'N'-value measured within the deposit is 6 blows per 0.3 m of penetration indicating a loose relative density.

The natural water content measured a sample of the deposit is about 29 per cent.

The result of one grain size distribution test completed on a sample of the deposit is shown on Figure B1-A.

4.4.6 Bedrock/Refusal

Bedrock was encountered and core samples were recovered from Borehole B502-01 at Elevation 165.9 m.

In general, the bedrock surface in the area of the proposed Pier 1 slopes down from south to north based on Boreholes B502-09 and B502-15 to B502-18 further to the south and Borehole B502-10 to the north of the pier.

Based on a review of the bedrock core samples recovered from the boreholes, the bedrock consists of granitic gneiss. In general the bedrock samples are described as fresh to slightly weathered, medium to coarse grained, strong, grey, as presented in the Record of Drillhole sheets in Appendix A, and shown on the photograph of the recovered core samples on Figures B7.

The RQD measured on two core samples is 94 per cent and 96 per cent, indicating the rock is of excellent quality, according to Table 3.10 in CFEM (2006). The TCR for the core samples ranges from 94 per cent to 100 per cent.

The point load strength index axial tests carried out on two samples of the granitic gneiss bedrock core measured Is_{50} values of 8.0 MPa and 6.9 MPa and the diametral tests carried out on three samples of the granitic gneiss bedrock core measured Is_{50} values ranging from 9.9 MPa to 14.3 MPa, as presented in Table B1.

One UCS test (ASTM D7012) carried out on a selected sample of the granitic gneiss bedrock measured a uniaxial compressive strength of about 68 MPa as summarized in Table B2-1 and detailed in Tables B2-2, respectively, in Appendix B.

Based on the UCS and point load test results on bedrock core samples from the Pier 1 area, in accordance with Table 3.5 in CFEM (2006), the granitic gneiss bedrock is classified as strong to very strong (R4 to R5, 50 MPa < UCS < 250 MPa).



4.5 Pier 2 Area

Boreholes B502-10 and B502-11 were advanced about 25 m south and 40 m north of the proposed Pier 2. The stratigraphy in Borehole B502-10 is shown in cross-section on Drawing 4.

4.5.1 Ice/Water

The ice surface at Boreholes B502-10 and B502-11 in Straight Lake in February 2014 is Elevation 178.5 m, and the depth of ice/water is 3.9 m and 1.8 m, respectively.

4.5.2 Organic Silt to Clayey Organic Silt

A 4.1 m thick deposit of dark grey organic silt was encountered below the water in Borehole B502-10 with the top of the deposit at Elevations 174.6 m. A 4.4 m thick deposit of dark grey clayey organic silt deposit was encountered below the water in Borehole B502-11 at Elevation 176.7 m.

The SPT 'N'-values measured within the organic silt to clayey organic silt are 0 blows (weight of rod) per 0.3 m of penetration, suggesting a very soft consistency.

The natural water content measured on samples of the deposit range between 114 per cent and 189 per cent. The organic content measured on two samples of the clayey organic silt is about 6 per cent.

4.5.3 Clayey Silt to Clay with Silt and Sand to Silty Sand Interlayers

A 13.0 m and 15.1 m thick deposit of grey to brown clayey silt to clay, with interlayers of sandy silt, silt and sand and silty sand, was encountered below the organic silt deposit in Boreholes B502-10 and B502-11 at Elevation 170.5 m and 172.3 m, respectively. In Borehole B502-10, the two interlayers of silt and sand, and silty sand were encountered at Elevations 167.1 m and 164.0 m, and are 1.7 m and 1.5 m thick, respectively. In Borehole B502-11, the two interlayers of sandy silt and silty sand were encountered at Elevations 167.8 m and 163.3 m, and the thicknesses of the layers is 1.5 m and 1.0 m, respectively. The lower portion of the clay deposit contains silt seams between the depths of 16.0 m and 21.3 m. Reportedly, Amec's Borehole ST-37 encountered occasional cobbles and a possible boulder at about approximate Elevation 166 m.

The SPT 'N'-values measured within the clayey silt to clay deposit range between 1 blow and 6 blows per 0.3 m of penetration. In situ field vane tests carried out within the deposit measured undrained shear strengths ranging between 15 kPa and 67 kPa, suggesting a soft to stiff consistency with corresponding sensitivities between 3 and 13. One vane test within the upper clayey silt zone in Borehole B502-10 measured an undrained shear strength greater than 96 kPa, likely as a result of the interface with the underlying silt and sand interlayer.

The SPT 'N'-values measured within the sandy silt, silt and sand, and silty sand interlayers are between 5 blows and 18 blows per 0.3 m of penetration indicating a loose to compact relative density.

The natural water content measured on samples of the clayey silt to clay range from 25 per cent to 54 per cent. The natural water content measured on two samples of the silt and sand and sandy silt interlayers is 16 per cent and 19 per cent.



The results of two grain size distribution test completed on samples of the silt and sand and sandy silt interlayers are shown on Figure B1-B.

Atterberg limits testing carried out on five samples of the clayey silt to clay deposit yielded liquid limits between 20 per cent and 59 per cent, plastic limits between 14 per cent and 19 per cent, with plasticity indices between 7 per cent and 40 per cent, indicating that the material is classified as a clayey silt of low plasticity (upper zone) to a clay of high plasticity (lower zone). The results of the Atterberg limits tests are shown on the plasticity chart on Figure B5-A.

4.5.4 Silt

An 8.0 m and 9.2 m thick deposit of grey silt was encountered below the clayey silt to clay deposit in Boreholes B502-10 and B502-11 at Elevation 157.5 m and 157.2 m, respectively. In Borehole B502-11, a cobble was inferred present within the deposit from auger grinding at a depth of 29.0 m, corresponding to Elevation 149.5 m.

The SPT 'N'-values measured within the silt range between 6 blows and 26 blows per 0.3 m of penetration indicating a loose to compact relative density.

The natural water content measured on samples of the deposit range between 22 per cent and 32 per cent.

The results of three grain size distribution tests completed on samples of the silt deposit are shown on Figures B1-B and B1-C.

Atterberg limits testing carried out on three samples of the silt deposit indicates that the material is non-plastic.

4.5.5 Silty Sand to Sand

A 16.6 m thick deposit of grey sand was encountered below the silt in Borehole B502-10 at Elevation 149.5 m. In Borehole B502-11, a 6.3 m thick deposit of grey silty sand was encountered below the silt at Elevation 148.0 m. In Borehole B502-10, gravel and cobbles were encountered within the deposit at a depth of 36.3 m, corresponding to Elevation 142.2 m. In Borehole B502-11, a cobble was encountered at a depth of 31.7 m, corresponding to Elevation 146.8 m and a sandy silt pocket was encountered at a depth of 36.7 m, corresponding to Elevation 141.8 m.

The SPT 'N'-values measured within the silty sand to sand deposit range between 13 blows and 24 blows per 0.3 m of penetration indicating a compact relative density.

The natural water content measured on two samples of the deposit are 19 per cent and 23 per cent.

The results of one grain size distribution test completed on a sample of the silty sand is shown on Figure B1-D and that of a sample of the sandy silt pocket is shown on Figure B1-C. The results of two grain size distribution tests completed on a sample of the sand are shown on Figure B3.



4.5.6 Cobbles and Boulders

A 1.2 m thick deposit of cobbles and boulders was encountered in Borehole B502-10 at Elevation 132.9 m. Reportedly, Amec's Borehole ST-37 encountered occasional cobbles and a possible boulder at about approximate Elevation 166 m.

4.5.7 Sandy Gravel

A 3.4 m thick deposit of grey sandy gravel was encountered below the cobbles and boulders in Borehole B502-10 at Elevation 131.7 m. Cobbles were encountered within the deposit at a depth of 49.8 m, corresponding to Elevation 128.7 m.

One SPT 'N'-value measured within the deposit is 19 blows per 0.3 m of penetration indicating a compact relative density.

The natural water content measured on one sample of the deposit is 6 per cent.

4.5.8 Bedrock

In Boreholes B502-10 and B502-11, bedrock was encountered at Elevation 128.3 m and 141.7 m, respectively, and core samples were recovered from both boreholes.

Based on a review of the bedrock core samples recovered from Boreholes B502-10 and B502-11, the bedrock consists of granitic gneiss. In general the bedrock samples, as presented in the Record of Drillhole sheets in Appendix A and as shown on the photograph of the recovered core samples on Figure B12 in Appendix B, are described as slightly weathered, foliated, medium grained, weak to very strong and grey with pink bands.

The RQD measured on the core samples ranges between 67 per cent and 100 per cent, indicating the rock is of fair to excellent quality, according to Table 3.10 in CFEM (2006). The TCR for the core samples ranges from 93 per cent to 100 per cent.

The point load strength index axial tests carried out on seven samples of the granitic gneiss bedrock core measured Is_{50} values ranging from 1.7 MPa to 9.4 MPa and the diametral tests carried out on five samples of the granitic gneiss bedrock core measured Is_{50} values ranging from 0.3 MPa to 10.1 MPa, as presented in Table B1.

One UCS test carried out on a selected core sample of the granitic gneiss bedrock obtained in Borehole B502-11 measured a compressive strength of about 23 MPa as summarized in Table B2-1 and detailed in Table B2-4, in Appendix B.

Based on the UCS and point load test results of the bedrock core samples from the vicinity of Pier 2, in accordance with Table 3.5 in CFEM (2006), the granitic gneiss bedrock is classified as weak rock to very strong (R2 to R5, 5 MPa < UCS < 250 MPa).

4.6 Pier 3 Area

Borehole B502-02 was advanced about 8 m south of the proposed Pier 3. The stratigraphy in Borehole B502-02 is shown in cross-section on Drawing 4.



4.6.1 Ice / Water

The ice surface at Borehole B502-02 in Straight Lake in February 2013 was measured at Elevation 178.6 m, and the depth of ice/water was 1.4 m.

4.6.2 Peat

A 0.7 m thick deposit of dark brown peat was encountered below the ice/water at Elevation 177.2 m in Borehole B502-02.

One SPT 'N'-value measured within the peat is 0 blows (i.e., weight of rod) per 0.3 m of penetration indicating a very soft relative density.

The natural water content measured on a sample of the deposit is 598 per cent. The organic content measured on one sample of the peat is about 30 per cent.

4.6.3 Organic Silt

A 2.3 m thick deposit of dark brown to dark grey organic silt was encountered below the peat in Borehole B502-02 at Elevation 176.5 m.

The SPT 'N'-values measured within the organic silt deposit are 0 blows (weight of rod) per 0.3 m of penetration, suggesting a very soft consistency. Two in situ field vane tests carried out within the organic silt deposit measured undrained shear strengths of 8 kPa and 10 kPa, suggesting a very soft consistency, with corresponding sensitivities of 2 and 3.

The organic content measured on one sample of the organic silt is about 13 per cent.

4.6.4 Clayey Silt to Clay

A 16.4 m thick deposit of grey clayey silt to silt clay was encountered in Borehole B502-02 below the organic silt at Elevation 174.2 m. An interlayer of silt and sand were encountered within the cohesive deposit at Elevation 168.7 m, and the thicknesses of the layer is 1.4 m. The clayey silt to silty clay deposit contains silt seams throughout.

The SPT 'N'-values measured within the clayey silt to silty clay deposit range between 1 blow and 15 blows per 0.3 m of penetration. In situ field vane tests carried out within the clayey silt to silty clay deposit measured undrained shear strengths ranging between 45 kPa and greater than 96 kPa, suggesting a soft to stiff consistency, with corresponding sensitivities between 2 and 10.

One SPT 'N'-value measured within the silt and sand interlayer is 1 blow per 0.3 m of penetration, indicating a very loose relative density.

The natural water content measured on samples of the clayey silt to silty clay deposit ranges from 30 per cent to 68 per cent. The natural water content measured on one sample of the silt and sand interlayer is 21 per cent.



The result of the grain size distribution tests completed on two samples of the clayey silt portion of the deposit are shown on Figure B6. The result of one grain size distribution test completed on a sample of the silt and sand interlayer is shown on Figure B1-D.

Atterberg limits testing carried out on five samples of the clayey silt to silty clay deposit yielded liquid limits ranging between 18 per cent and 41 per cent, plastic limits ranging between 13 per cent and 20 per cent, and plasticity indices of ranging between 5 per cent and 23 per cent, indicating that the material is classified as a clayey silt of low plasticity to a silty clay of intermediate plasticity. The results of the Atterberg limits tests are shown on the plasticity chart on Figure B5-D.

4.6.5 Silt

A 5.6 m thick deposit of grey silt was encountered at Elevation 157.8 m in Borehole B502-02.

Two SPT 'N'-values measured within the silt deposit are 10 blows and 17 blows per 0.3 m of penetration, indicating a compact relative density.

The natural water content measured on two samples of the deposit is 23 per cent.

The result of one grain size distribution test completed on a sample of the silt is shown on Figure B1-D.

Atterberg limits testing carried out on one sample of the silt deposit indicates that the material is non-plastic.

4.6.6 Gravelly Silt and Sand

A 1.6 m thick deposit of grey gravelly silt and sand was encountered at Elevation 152.2 m in Borehole B502-02.

An SPT 'N'-value measured within the gravelly silt and sand deposit is 36 blows per 0.3 m of penetration, indicating a dense relative density.

The natural water content measured on a sample of the deposit is 18 per cent.

The results of one grain size distribution test completed on a sample of the gravelly silty sand deposit is shown on Figure B1-E.

4.6.7 Cobbles and Boulders

A 0.5 m thick layer of cobbles and boulders was encountered at Elevation 150.6 m.

4.6.8 Bedrock

Bedrock was encountered at Elevation 150.1 m and core samples were recovered in Borehole B502-02.

Based on a review of the bedrock core samples recovered from the borehole, the bedrock consists of granitic gneiss. In general the bedrock samples, as presented in the Record of Drillhole sheets in Appendix A, and shown on the photograph of the recovered core samples on Figure B10 in Appendix B, are described as slightly weathered to fresh, medium to coarse grained, very strong and grey with light pink bands.



The RQD measured on the core samples ranges between 71 per cent and 96 per cent, indicating the rock is of fair to excellent quality, according to Table 3.10 in CFEM (2006). The TCR for the core samples ranges from 97 per cent to 100 per cent.

The point load strength index axial tests carried out on three samples of the granitic gneiss bedrock core measured Is_{50} values ranging from 9.4 MPa and 10.8 MPa and the diametral tests carried out on three samples of the granitic gneiss bedrock core measured Is_{50} values of 5.6 MPa and 8.9 MPa, as presented in Table B1.

Based on the point load test results of the bedrock core samples from Pier 3, in accordance with Table 3.5 in CFEM (2006), the granitic gneiss bedrock is classified as strong to very strong (R4 to R5, 50 MPa < Pt. Load Index < 250 MPa).

4.7 Pier 4 Area

Borehole B502-13 was advanced about 5 m north of the proposed Pier 4. The stratigraphy at Borehole B502-13 is shown in cross-section on Drawing 5.

4.7.1 Topsoil

A 0.2 m thick deposit of topsoil was encountered from ground surface at Elevation 183.5 m in Borehole B502-13.

4.7.2 Silt

A 0.3 m thick deposit of grey silt was encountered below the topsoil in Borehole B502-13 at Elevation 183.3 m.

4.7.3 Sand

A 5.0 m thick deposit of grey sand was encountered below the silt layer in Borehole B502-13 at Elevation 183.0 m.

SPT 'N'-values measured within the deposit range between 2 blows and 12 blows per 0.3 m of penetration indicating a very loose to compact relative density.

The natural water content measured on three samples of the sand deposit ranges from about 22 per cent to 26 per cent.

The results of a grain size distribution test carried out on a sample of the sand deposit is shown on Figure B1-C.

4.7.4 Clayey Silt to Silty Clay

A 5.0 m thick deposit of grey clayey silt to silty clay was encountered below the sand deposit at Elevation 178.0 m in Borehole B502-13.

Two SPT 'N'-values measured within the clayey silt to silty clay deposit are 1 blow and 4 blows per 0.3 m of penetration. In situ field vane tests carried out within the clayey silt to silty clay deposit measured undrained shear



strengths ranging between 35 kPa and 42 kPa, and the sensitivity ranges between 6 and 9. The results of the field vane tests indicate that the clayey silt to silty clay deposit has a firm consistency.

The natural water content measured on samples of the clayey silt to silty clay deposit ranges between 32 per cent and 41 per cent.

Atterberg limits tests carried out on three samples of the clayey silt to silty clay deposit yielded liquid limits ranging between about 25 per cent and 42 per cent, plastic limits ranging between about 13 per cent and 18 per cent and plasticity indices ranging between about 13 per cent and 25 per cent. The results of the Atterberg limits tests are shown on the plasticity chart on Figure B5-B, and indicate that the material is classified as a clayey silt of low plasticity to a silty clay of intermediate plasticity.

A laboratory consolidation test was completed on a sample of the cohesive deposit from Borehole B502-13 and the results are discussed in Section 4.9.5 (North Approach) as the test results are relevant to the approach embankment front slope rather than the pier.

4.7.5 Silt

A 4.0 m thick deposit of grey silt was encountered at Elevation 173.0 m underlying the clayey silt to silty clay deposit in Borehole B502-13.

SPT 'N'-values measured within the silt deposit range between 4 blows and 10 blows per 0.3 m of penetration indicating a loose to compact relative density.

The natural water content measured on two samples of the deposit is 22 per cent.

The result of a grain size distribution test carried out on a sample of the silt deposit is shown on Figure B1-C.

4.7.6 Boulder

A 0.3 m thick boulder was encountered between the silt deposit and the underlying bedrock at Elevation 169.0 m in Borehole B502-13.

4.7.7 Bedrock

Bedrock was encountered at Elevation 168.7 m and core samples were recovered from Borehole B502-13.

Based on a review of the bedrock core samples recovered from Borehole B502-13, the bedrock consists of granitic gneiss. In general the bedrock samples, as presented in the Record of Drillhole sheets in Appendix A, and shown on the photograph of the recovered core samples on Figure B13 in Appendix B are described as slightly weathered, fine to medium grained, very strong and dark grey with pink bands.

The RQD measured on the two core samples is between 100 per cent and 87 per cent, indicating rock of good to excellent quality, according to Table 3.10 in CFEM (2006). The TCR for the core samples was 100 per cent.

The point load strength index diametral tests carried out on two samples of the granitic gneiss bedrock core measured Is_{50} values of 7.2 MPa and 7.5 MPa, as presented in Table B1.

One UCS test (ASTM D7012) carried out on a selected sample of the granitic gneiss bedrock obtained in Borehole B502-13 measured a UCS of about 111 MPa as summarized in Table B2-1 and detailed in Table B2-5, in Appendix B.



Based on the point load strength index and the UCS test results of the bedrock core sample from the Pier 4 area, in accordance with Table 3.5 in CFEM (2006), the granitic gneiss bedrock is classified as very strong (R4 to R5, 50MPa < UCS < 250 MPa).

4.7.8 Groundwater Conditions

A standpipe piezometer was installed in Borehole B502-13 sealed within the sand deposit to allow monitoring of the groundwater level at this location. Details of the piezometer installation are shown on the Record of Borehole sheet in Appendix A and the groundwater level measured in the piezometer is summarized below.

Foundation Element	Borehole	Ground Surface Elevation (m)	Groundwater Elevation (m)	Date of Measurement
Pier 4	B502-13	183.5	182.1	March 19, 2014

4.8 North Abutment

Borehole S502-24 was advanced approximately 4.9 m south of the proposed north abutment. The interpreted stratigraphy near the north abutment is shown in cross-section on Drawing 5.

4.8.1 Topsoil

A 0.8 m thick layer of topsoil was encountered at ground surface at Elevation 190.3 m in Borehole S502-24.

One SPT 'N'-value measured with the topsoil is 3 blows per 0.3 m of penetration, suggesting a soft consistency.

The natural water content measured on a sample of the topsoil is 67 per cent and the organic content measured on the same sample of the topsoil is about 6 per cent.

4.8.2 Silt to Sand with Clay Interlayer

A 16.9 m thick deposit comprised of layers of brown to grey sand, silt and sand, and silt was encountered below the topsoil at Elevation 189.5 m in Borehole S502-24. A 1.8 m thick clay interlayer was encountered between an overlying sand layer and an underlying silt layer at Elevation 178.6 m.

The SPT 'N'-values measured within the overall silt to sand deposit range between 0 blows (weight of hammer) and 17 blows per 0.3 m of penetration indicating a very loose to compact relative density.

One SPT 'N'-value measured with the clay interlayer was 1 blow per 0.3 m of penetration. One in situ field vane test carried out within the clay interlayer measured an undrained shear strength of about 25 kPa and a sensitivity of 5. The result of the field vane test indicates that the clay interlayer has a firm consistency.

The natural water content measured on samples of the silt to sand deposit ranges between 8 per cent and 29 per cent. The natural water content measured on one sample of the clay interlayer is 50 per cent.

The results of four grain size distribution tests completed on samples of the various layers of the silt to sand deposit from Borehole S502-24 are shown on Figure B1-F.

The results of Atterberg limits tests carried out on one sample of the silt and the silt and sand layers of the deposit indicate that the materials are non-plastic.



4.8.3 Refusal

Refusal to further split spoon and auger advancement was encountered at Elevation 172.6 m in Borehole S502-24, at a depth of 17.7 m below ground surface.

4.8.4 Groundwater Conditions

In Borehole S502-24, the water level upon completion of drilling was measured at a depth of 6.0 m below ground surface (Elevation 184.3 m).

4.9 North Approach

A total of 13 boreholes (Boreholes B502-13, B502-14, S502-08, S502-12, S502-18 to S502-24, S502-20A and S502-20B) three DCPTs (S502-DC02, S502-DC08 and S502-DC09) were advanced in the area of the north approach embankment, including the front slope area of the north abutment. Borehole S502-20B was advanced adjacent to Borehole B502-20A to obtain a Shelby Tube sample of the cohesive deposit. The interpreted stratigraphy at the north approach is shown in profile on Drawing 2.

4.9.1 Peat/Topsoil

A 0.3 m and 0.4 m thick layer of fibrous peat was encountered at the ground surface in Boreholes S502-18 and S502-19 and an approximately 0.2 m to 0.8 m thick layer of topsoil was encountered at the ground surface in Boreholes B502-13, B502-14, S502-08, S502-12, S502-18 to S502-24 and S502-20A. The surface of the peat/topsoil layers ranges between Elevations 192.1 m and 180.9 m.

An SPT 'N'-value of 3 blows and 2 blows per 0.3 m of penetration was measured within the topsoil deposit and peat deposit in Boreholes S502-24 and S502-19, respectively, suggesting a soft consistency.

The natural water content measured on a two samples of the topsoil is about 67 per cent and 68 per cent.

The organic content measured on a sample of the topsoil is about 6 per cent.

4.9.2 Organic Silt

An approximately 0.8 m thick deposit of organic silt was encountered below the peat deposit in Boreholes S502-19 at Elevation 180.5 m.

An SPT 'N'-value of 2 blows per 0.3 m of penetration was measured at the interface with the underlying deposit of the clayey silt in Borehole S502-19, suggesting that the organic silt has a soft consistency.

4.9.3 Clayey Silt to Silty Clay (Pockets/Interlayers)

Boreholes B502-14, S502-08, S502-12, S502-18, S502-19, S502-20A, S502-21 and S502-22 penetrated various layers/pockets of cohesive soils comprised of clayey silt to silty clay ranging in thickness between about 0.5 m and 1.5 m below the peat/topsoil/organic silt layers and/or within the non-cohesive silt and sand to sand deposit. The pockets/layers encountered below the topsoil/peat generally contain trace organics. The top of the cohesive pockets/layers ranges between Elevations 191.9 m and 179.8 m.



SPT 'N'-values ranging from 0 blows (weight of hammer) to 7 blows per 0.3 m of penetration were measured within the cohesive soil pockets/layers. In situ field vane tests carried out within the cohesive pocket in Boreholes B502-14 and S502-08 measured undrained shear strengths of about 39 kPa and 62 kPa and sensitivity values of 5. The SPT 'N'-values and field vane test results suggest that the cohesive soil pockets/layers have a very soft to stiff consistency.

The natural water content measured on seven samples of the cohesive soil pockets ranges from about 18 per cent to 38 per cent.

A grain size distribution test result for a sample from the cohesive soil pocket from Borehole S502-20A is shown on Figure B6 in Appendix B.

Atterberg limits tests were carried out on six samples from the cohesive soil pockets and measured liquid limits ranging from about 24 per cent to 42 per cent, plastic limits ranging from about 15 per cent to 24 per cent and plasticity indices ranges from about 8 per cent to 19 per cent. The results of the Atterberg limits tests are shown on the plasticity chart on Figures B5-B to B5-F in Appendix B and indicate that the material is classified as clayey silt of low plasticity to silty clay of intermediate plasticity.

4.9.4 Silt to Sand (Upper Deposit)

An upper deposit of brown to grey silt to silt and sand to silty sand to sand, trace to some silt was encountered below the topsoil layer in Boreholes B502-13, B502-14, S502-20, S502-20A, S502-21 and S502-24 and underlying the cohesive soil pockets in Boreholes S502-08, S502-18, S502-12, S502-22 and S502-23. The non-cohesive deposit generally contains trace to some clay and trace to some gravel and occasional pockets/layers of clayey silt up to about 1.5 m thick. The top of this deposit is at Elevation 191.0 m at the north limit of the site decreasing to Elevation 180.5 m near the south limit of the site. The overall thickness of the deposit ranges from 2.9 m at the south limit (Borehole S502-18) to 10.9 m at the north limit of the from slope area (Borehole S502-24), respectively, and is about 3.0 m thick in the north approach area (Borehole S502-12).

The SPT 'N'-values measured within the non-cohesive deposit range from 0 blows (weight of hammer) to 20 blows per 0.3 m of penetration, indicating a very loose to compact relative density.

The natural water content measured on samples of this deposit ranges from about 7 per cent to 31 per cent.

The results of grain size distribution tests completed on 19 samples from this deposit are shown on Figures B1-C to B1-I in Appendix B.

Atterberg limits tests were carried out on two (2) samples of the fines portion from the silt to silt and sand of this deposit and measured liquid limits of about 16 per cent and 15 per cent, plastic limits of about 14 per cent and corresponding plasticity indices of about 2 per cent and 1 per cent. The results of the Atterberg limits tests are shown on the plasticity chart on Figure B2-B in Appendix B and indicate that the fines portion of the deposit is a silt of slight plasticity.

4.9.5 Clayey Silt to Clay

A deposit of brown to grey clayey silt to silty clay to clay was encountered below the organic silt or below the silt to sand deposit in Boreholes B502-13, B502-14, S502-08, S502-12, S502-20A and S502-18 to S502-24. The deposit contains silt to silty sand layers/pockets about 0.2 m and 0.5 m thick in Boreholes S502-19 and S502-22, respectively. The top of the cohesive deposit ranges between Elevations 187.8 m and 174.2 m and the overall



thickness of the deposit varies between 0.3 m at the north limit (Borehole S502-24) and 7.4 m at the south limit (Borehole S502-18), including the thickness of the non-cohesive layers/pockets.

The SPT 'N'-values measured within this deposit range from 0 blows (weight of hammer) to 7 blows per 0.3 m of penetration. In situ field vane tests carried out within this deposit measured undrained shear strengths ranging from 23 kPa to greater than 62 kPa but generally greater than 25 kPa, indicating a soft to stiff consistency. The sensitivity of the cohesive deposit ranges from 2 to 10.

The natural water content measured on samples of this deposit ranges from about 23 per cent to 65 per cent.

Atterberg limits tests carried out on 22 samples of the cohesive deposit measured liquid limits ranging from about 22 per cent to 55 per cent, plastic limits ranging from about 13 per cent to 22 per cent and plasticity indices ranging from about 4 per cent to 36 per cent. The results of the Atterberg limits tests are shown on the plasticity charts on Figures B5-B to B5-F in Appendix B and indicate that the material is classified as a clayey silt of low plasticity to a clay of high plasticity.

Laboratory consolidation tests were carried out on three specimens of the silty clay portion of the cohesive deposit obtained from Shelby tube samples in Boreholes B502-13, S502-20 and S502-20B. Preconsolidation stresses ranging from about 150 kPa and 195 kPa were estimated from the void ratio versus logarithmic pressure plot and from the total work versus pressure plot. Bulk unit weights ranging from about 16.9 kN/m³ to 17.6 kN/m³ and specific gravities of about 2.75 and 2.76 were measured on the consolidation test specimens. Details of the test results are shown on Figures B7 to B9 in Appendix B, and the test results are summarized below.

Borehole Sample No.	Sample Depth / Elevation	σ_{vo}' (kPa)	σ_p' (kPa)	$\sigma_p' - \sigma_{vo}'$ (kPa)	OCR	C_c	C_r	e_o	c_v^* (cm ² /s)
Borehole B502-13 Sample 10	7.7 m / 175.8 m	75	160	85	2.1	0.62	0.06	1.24	4.6×10^{-4}
Borehole S502-20 Sample 10	9.4 m / 174.7 m	100	150	50	1.5	0.79	0.02	1.40	7.2×10^{-4}
Borehole S502-20B Sample 1	5.7 m / 178.5 m	55	195	140	3.5	0.80	0.06	1.43	1.0×10^{-3}

Note: * For stress range between effective overburden stress and final stress due to a 4.4 m high embankment, that is $40 \text{ kPa} \leq \sigma_v' \leq 185 \text{ kPa}$.

where: σ_{vo}' is the in situ vertical effective overburden stress in kPa
 σ_p' is the preconsolidation stress in kPa
 OCR is the overconsolidation ratio
 e_o is the initial void ratio
 C_c is the compression index
 C_r is the recompression index
 c_v is the coefficient of consolidation in cm²/s

4.9.6 Silt to Sand (Lower Deposit)

A lower deposit of grey silt to sand was encountered underlying the cohesive deposit in Boreholes B502-13, B502-14, S502-08, S502-12, S502-18 to S502-24 and S502-20A. The deposit generally contains trace to some clay and trace gravel. A 0.3 m boulder was encountered at the bottom of the deposit in Boreholes B502-13 at Elevation 169.0 m. The top of this deposit ranges from about Elevation 176.8 m at the north limit of the front slope (Borehole S502-24) to about Elevation 172.5 m towards the south limit or toe of the front slope (Borehole B502-14)



and the thickness of the deposit ranges from 0.9 m to 5.3 m. At Borehole S502-12, at the toe of the North Approach Embankment slope the surface of this deposit is at Elevation 187.5 m and the thickness of the deposit is 7.9 m. The bottom of this deposit is defined by refusal to further casing and/or split-spoon advancement in Boreholes B502-14, S502-08, S502-12, S502-18 to S502-24 and S502-20A.

The SPT 'N'-values measured within this deposit range from 0 blows (weight of rod) per 0.3 m of penetration to 18 blows per 0.15 m of penetration, indicating a very loose to compact relative density, with one SPT 'N'-value measured in the sand in Borehole S502-20A at 50 blows per 0.15 m of penetration, indicating a very dense relative density.

The natural water content measured on samples of this deposit ranges from about 19 per cent to 25 per cent.

The results of grain size distribution tests completed on 15 samples of the deposit are shown on Figure B1-C to B1-I in Appendix B.

Atterberg limits tests were carried out on eight samples of the silt to silt and sand deposit. The results of seven of the Atterberg limits tests indicate that the fines portions of the silt to sand deposit are non-plastic and the other Atterberg limits test measured a liquid limit of about 17 per cent, a plastic limit of about 16 per cent and a plasticity index of about 1 per cent. The result of the Atterberg limits test is shown on the plasticity chart on Figure B2-B in Appendix B indicating that the silt layer of the overall deposit exhibits slight plasticity.

4.9.7 Bedrock / Refusal

In Boreholes B502-14, S502-08, S502-12, S502-18 to S502-24 and in DCPTs S502-DC01, S502-DC02, S502-DC04, S502-DC08 and S502-DC09 the bedrock surface is inferred by refusal to further split-spoon and/or auger advancement or dynamic cone penetration at depths between about 9.8 m and 20.7 m below the ground surface, between Elevations 179.6 m and 160.8 m. Refusal was encountered at shallower depths near the north limit of the front slope/approach embankment area and the greatest depths to refusal were encountered near the south limit of the high fill area adjacent to the north shore of Straight Lake.

The bedrock encountered in Borehole B502-13 advanced near the toe of the front slope area in the vicinity of Pier 4 is described in Section 4.7.7.

4.9.8 Groundwater Conditions

The depth to the groundwater level measured in the open boreholes, upon completion of drilling range from 0.3 m to 7.6 m below ground surface or between Elevations 184.5 m and 178.9 m, with the groundwater table generally located closer to the ground surface in the southern portion of the north approach (front slope area).

A standpipe piezometer was installed in Borehole B502-13 to allow monitoring of the groundwater level at the borehole location. Details of the piezometer installation are shown on the Record of Borehole sheet in Appendix A and the groundwater level measured in the piezometer is summarized below.

Foundation Element	Borehole	Ground Surface Elevation (m)	Groundwater Elevation (m)	Date of Measurement
North Approach (Front Slope/Pier Area)	B502-13	183.5	182.1	March 19, 2014



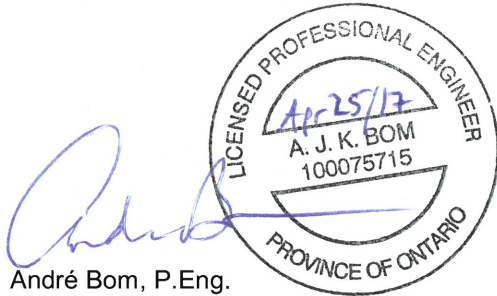
5.0 CLOSURE

The drilling program was directed by Mike Arthur, Indulis Dumpis, Trevor Moxam and Mat Riopelle. This report was prepared by Mr. Matt Thibeault, P.Eng., and reviewed by Mr. André Bom, P.Eng., a senior geotechnical engineer and Associate of Golder. Mr. Jorge M. A. Costa, P.Eng., Golder's Designated MTO Contact for this project and a Senior Consultant with Golder, conducted an independent quality control review of the report.



Report Signature Page

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PART B

FOUNDATION DESIGN REPORT

STRAIGHT LAKE NBL BRIDGE, SITE NO. 44-461/1

HIGHWAY 69 FOUR-LANING FROM 1.7 KM NORTH OF HIGHWAY 529

NORTHERLY TO 3.9 KM NORTH OF HIGHWAY 522

MINISTRY OF TRANSPORTATION, ONTARIO

GWP 5347-08-00; WP 5145-08-01



6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides engineering design recommendations for the proposed Straight Lake NBL structure. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the subsurface investigation. The discussion and recommendations presented are intended to provide the designers with sufficient information to assess the feasible foundation alternatives and to carry out the design of the structure foundations and approaches. The foundation investigation report and the discussion and recommendations are intended for the use of the Ministry of Transportation, and shall not be used or relied upon for any other purposes or by any other parties including the construction or design-build contractor. The design-build contractor must make their own interpretation based on the factual data in Part A of the report. Where comments are made on construction, they are provided only to highlight those aspects which could affect the design of the project. Those requiring information on the aspects of construction must make their own interpretation of the factual information provided as such interpretation may affect equipment selection, proposed construction methods, scheduling and the like.

6.1 General

Golder has been retained by URS on behalf of the Ministry of Transportation, Ontario (MTO) to provide recommendations on foundation aspects for the detail design of the Straight Lake NBL structure within Contract 5 along the proposed section of four-laning of Highway 69. We understand that the 355 m long NBL bridge will be a five span, variable depth steel girder structure consisting of two end-spans 65 m long and three 75 m long centre-spans. The abutments and the northerly pier (Pier 4) are located on the shores of Straight Lake and the three central/southerly piers (Piers 1 to 3) are located in the lake.

6.1.1 Background to General Arrangement / Structure Layout

As discussed in Section 1, the original Terms of Reference for the Foundation Investigation for the Straight Lake structures included a two phase process that consisted of:

- Phase 1 – a preliminary foundation investigation consisting of a total of four boreholes (two for the SBL structure and two for the NBL structure) at approximate locations of anticipated in-water piers.
- Phase 2 – detailed foundation investigation for all foundation elements after the bridge span arrangement and pier locations were finalized.

In January 2013, the preliminary design and General Arrangement for the Straight Lake bridges was based on twin 264 m long structures. Two structure layouts were being considered at that time, most likely comprised of either:

- Four-span bridges with spans of 57 m, 75 m, 75 m, 57 m.
- Five-span bridges with spans of 45 m, 58 m, 58 m, 58 m, 45 m.

Based on this information and the January 2013 GA drawings, the Phase 1 preliminary foundation investigation borehole locations were planned (with input from Golder, URS and MTO) and then carried out from February 20 to March 8, 2013, at which time a total of four boreholes and one dynamic cone penetration test (DCPT) were advanced at or near the locations of the footprints of the in-water piers of the proposed SBL and NBL structures.



During this investigation, two boreholes and one DCPT were advanced for the SBL bridge alignment and two boreholes were advanced for the NBL bridge alignment.

In May 2013, a summary of the subsurface conditions encountered in the Phase 1 investigation and a preliminary assessment of the foundation alternatives was provided in a Draft Letter to URS dated May 31, 2013 (see Appendix C). In this letter, the results of preliminary stability analysis and a summary of the anticipated challenges associated with construction of the proposed up to 15 m high north approach embankments on the thick (up to 15 m) firm silty clay stratum at the location based on the general arrangement of the structure at that time was identified. In this letter, it was recommended that consideration be given to lowering the grade of the highway in this area to reduce the height of the north approaches and make the design/construction more feasible.

Subsequent discussions with URS clarified that reducing the grade of the highway and height of the north approach embankments was not feasible at this location due to geometrical constraints associated with the CP Rail crossing to the south of Straight Lake. As such, a more detailed assessment of the subsurface conditions at the north approaches and adjacent Swamp 502 was carried out and provided in a letter to URS dated December 13, 2013 (see Appendix C). In this letter, the preliminary recommendations concluded that lengthening the bridge structures and moving the north abutments of both structures by approximately 30 m to the north was preferred from a foundations perspective, to reduce the height of the embankments and make the design and construction of the north approaches more feasible and reduce the risk associated with instability and post-construction settlement performance. It was noted however, that even with this change to the location of the north abutments, additional foundation mitigation measures including either full sub-excavation and replacement (up to 15 m deep), use of lightweight (EPS) fill, wick drains and staged construction, with toe berms, and/or installation of grouted aggregate piers and wick drains and staged construction, would still be required. The selection of the preferred foundation mitigation for the north abutment/approach area would be decided after completion of the detailed foundation investigation and analysis for the Straight Lake bridges as well as the analysis for the adjacent swamp/high fill areas (S502).

In January 2014, MTO agreed to the proposal to extend the structures and move the north abutments 30 m to the north. At this time, the design and General Arrangement for the Straight Lake bridges was based on twin 295 m long structures with the structure layouts considered to most likely be comprised of:

- Five-span bridges with spans of 50 m, 65 m, 65 m, 65 m, 50 m.

Based on this General Arrangement/structure layout, the Phase 2 foundation investigation was carried out between February 6 and March 18, 2014 at which time a total of 33 boreholes and one dynamic cone penetration test (DCPT) were advanced at the proposed locations of foundation units for the SBL and NBL structures. During this investigation, 17 boreholes and one DCPT were advanced for the SBL bridge alignment and 16 boreholes were advanced for the NBL bridge alignment.

Following completion of the Phase 2 field investigation and laboratory testing, a detailed assessment of the foundation stability and settlement mitigation options for the approaches/embankments located to the north of the Straight Lake structures was carried out. This assessment included the results of the investigation carried out for the swamp/high fill area (S502) located to the north of the north abutments of the Straight Lake bridges. In October 2014, the results of this assessment were summarized in the draft Tables B1 and B2 addressing the stability/settlement mitigation options for these embankments as they relate to the design/layout of the Straight Lake bridges (see Appendix C). Based on this assessment, the preferred foundation option to enhance the stability and mitigate settlement of these approaches/embankments was to shift the north abutments of the bridges further north by 35 m for the SBL and either 35 m or 60 m for the NBL. This recommendation was discussed in a meeting



with Golder, URS and MTO in November 2014. In November 2015, URS submitted a design memo to MTO outlining several options for the final design/layout of the Straight Lake bridges that summarized the foundation mitigation options to be considered along with the risks and costs associated with the various options.

On February 24, 2016, the General Arrangement for the Straight Lake bridges (dated February 2016) was provided to Golder for the structures comprised of:

- SBL – Five-span bridge with spans of 52.5 m, 75 m, 75 m, 75 m, 52.5 m (total length = 330 m).
- NBL – Five-span bridge with spans of 65 m, 75 m, 75 m, 75 m, 65 m (total length = 355 m).

Based on the 2016 GA, the grade of the proposed NBL bridge deck varies between about Elevation 199.3 m (south abutment) and about Elevation 195.8 m (north abutment). The proposed abutments are to be founded at about Elevation 193.6 m (south abutment) and about Elevation 190.1 m (north abutment). The ice level in Straight Lake was at Elevation 178.5 m and 178.6 m in February 2013 and February 2014, respectively, during the foundation investigations in the lake for both the proposed NBL and SBL bridges. Based on the GA drawing provided by URS, the Straight Lake water level was measured by others at Elevation 178.0 m (+/-) and the high water level is indicated at Elevation 180.1 m.

6.2 Foundation Options

At the south abutment area on the south shore of Straight Lake, in general, bedrock is either exposed/outcropping or is present below overburden up to about 8 m thick. The footprint of the proposed south abutment is located above a depression in the surface topography that has an average ground surface at Elevation 186.0 m with exposed bedrock at higher elevation immediately south and north of the area. With the proposed underside of abutment foundation at Elevation 193.6 m and the proposed approach roadway at Elevation 199.3 m, about 8 m and 13 m of embankment fill will be required to raise grades to the foundation level and roadway level, respectively. The south abutment could be supported on a shallow foundation perched on a high (up to 13 m high) compacted granular pad (as discussed in Section 6.3), however in order to ensure global stability of the front slope of the approach embankment and to maintain consistency with the other foundation units of the bridge, it is recommended that the south abutment be founded on deep (pile) foundations (as discussed in Section 6.4).

At the piers and north abutment, the bedrock is not exposed but is present at deeper depths below water/lakebed and/or thick, very soft/very loose overburden deposits that are unsuitable for support of spread footings due to low geotechnical resistances at ULS and/or SLS. In addition, at the north abutment, the factor of safety for the global stability of the front slope of the approach embankment is less than 1.3 when including the loading from a shallow foundation. As such, shallow foundations are not feasible at these locations and deep (pile) foundations (as discussed in Section 6.4) are required for support of the piers and north abutment foundations.

The following sections provide additional details and recommendations for the design of deep foundations (drilled steel casings at the piers and driven steel H-piles at the abutments) to support the proposed bridge foundation elements. Recommendations for the design of a shallow foundation (spread footing) at the south abutment are also provided, but this is not considered the preferred alternative at this location.

A comparison of the foundation options (shallow and deep) at the South Abutment, Pier 1, Piers 2 to 4 and the North Abutment, based on advantages/disadvantage, relative costs and risks/consequences for feasible foundation elements, is presented in Tables 1 to 4.



6.3 Shallow Foundations

6.3.1 Spread Footings (South Abutment)

Supporting the south abutment on a spread footing founded on a high compacted granular pad, constructed within the core of the approach embankment, is considered a feasible (but not preferred) design option for this foundation unit as summarized in Table 1. For this alternative, all loose/soft surficial soils and all organic soil deposits within the approach embankment footprint would have to be removed and replaced with Granular B Type II as part of the granular pad construction. In this regard, sub-excavation to between Elevation 179.9 m (Borehole B502-04) and 183.4 m (Borehole B502-08) would be required to remove the upper 2.9 m to 5.8 m of very loose to loose sandy silt to sand to organic silt deposits, with wood fragments and/or roots encountered within several of the deeper samples as noted on the borehole logs. The underlying native silts, sands, gravels, cobbles and/or boulders encountered below the surficial deposits may remain in place, however, the excavated subgrade would have to be reviewed and approved by a geotechnical engineer and heavily proof-rolled prior to backfilling with Granular B Type II and constructing the overlying granular pad. With the proposed underside of the south abutment at about Elevation 193.6 m, the compacted granular pad would be up to 13.7 m thick following sub-excavation of the organic soils to as deep as Elevation 179.9 m. The granular pad would have to extend a minimum of 1 m beyond the footing footprint in all directions (at the base of the footing) and be constructed with side slopes no steeper than 1H:1V using OPSS.PROV 1010 (Aggregates) Granular 'B' Type II as discussed further in Section 6.8.2.

6.3.2 Geotechnical Axial Resistances/Reactions

The following factored geotechnical axial resistance at Ultimate Limit States (ULS) and geotechnical reaction at Serviceability Limits States (SLS) for 25 mm of settlement may be used for design of a spread footing founded on the compacted granular pad.

Foundation Unit	Foundation Alternative	Factored Geotechnical Axial Resistance at ULS (kPa)	Geotechnical Reaction at SLS for 25 mm of Settlement (kPa)
South Abutment (Elevation 193.6 m)	Spread footing on granular pad	400 ¹	300 ²

- Notes:
1. The ULS value is controlled by the global stability considering the location of the footing relative to the adjacent high front slope face of the approach embankment (refer to Section 6.7.1.3)
 2. The geotechnical reaction at SLS is estimated for a 4.5 m wide spread footing.

In order to achieve the above factored geotechnical axial resistance at ULS (i.e., 400 kPa) while satisfying the global stability of the front slope of the high approach embankment under the influence of the footing load, it would be necessary to confirm that the fill materials utilized to construct the compacted granular pad (i.e., Granular B Type II and/or Granular A) have a minimum friction angle (ϕ') of 38 degrees. In this regard, an example NSSP outlining the minimum the laboratory testing requirements for the granular materials is included in Appendix D, but would only be required if the shallow foundation option was selected for supporting the south abutment.

The geotechnical resistance/reaction(s) provided above is given for loads applied perpendicular to the surface of the footing. Where the load is not applied perpendicular to the surface of the footings, inclination of the load should be taken into account in accordance with Sections 6.7.2 and 6.7.4 of the Canadian Highway Bridge Design Code (CHBDC, 2006) and its Commentary.



6.3.3 Resistance to Lateral Loads

The resistance to lateral forces/sliding resistance between the concrete footing and the compacted granular pad, should be calculated in accordance with Section 6.7.5 of CHBDC (2006). The following presents the coefficient of friction, $\tan \delta$, for the interface between the concrete footing and the granular pad.

Interface Material(s)	Coefficient of Friction
Concrete Footing on Compacted Granular Pad	$\tan \delta = 0.58$

The value presented above is an unfactored value.

6.3.4 Frost Protection

The required thickness of conventional soil cover for frost protection of the footing is 1.8 m as per OPSD 3090.010 (Frost Penetration Depths for Southern Ontario) as measured perpendicular from the face of the abutment slope to the edge of the underside of the footing.

If adequate soil cover cannot be provided for the footing, rigid extruded polystyrene insulation could be installed to compensate for the lack of soil cover and provide protection from frost penetration, however, the edge of the footing must be set-back at least 1.8 m (measured perpendicular) from the crest of the adjacent slope. Further details on the requirements for the position of the footing relative to the granular pad and the surrounding rock fill are provided in Section 6.8.2.

6.4 Deep Foundations

The variable subsurface conditions encountered along the alignment of the bridge (as shown on Drawing 2) present a range of design and construction challenges for deep foundations to support the in-water and on-land piers.

The difficult subsurface conditions include:

- At Pier 1 – deep water (about 4.0 m deep), underlain by variable thickness, very soft, highly organic overburden soils (up to about 4.8 m thick) underlain by very loose to loose silt with a clayey silt layer, overlying steeply sloping, very strong bedrock. The bedrock is estimated to be dipping at up to approximately 50° across the footprint of the pier/projected footprint of the piles at the bedrock surface.
- At Piers 2 and 3 – deep water (about 3.9 m at Pier 2; shallower to the north at depths of 1.1 m to 1.4 m at Pier 3), underlain by very thick, variable composition overburden soils (up to 46 m thick) including up to 4.4 m of very soft, highly organic soils immediately below the lakebed. In addition, cobbles and boulders were encountered within the silt, sand and sand and gravel stratum at depth above the bedrock at these locations. The very strong bedrock is estimated to be dipping at up to approximately 15° across the footprint of the pier/projected footprint of the piles at the bedrock surface.
- At Pier 4 – variable composition overburden soils, including up to 5.0 m of firm clay, overlying very strong bedrock estimated to be dipping at up to approximately 10° across the footprint of the pier/projected footprint of the piles at the bedrock surface.



The above noted water/subsurface conditions can present several challenges to pile installation/construction and have to be considered when selecting the preferred pile type for the various foundation units. These challenges, as well as pile installation methods more conducive to these conditions, are outlined as follows:

- Where water is deep and overburden is very soft, it is difficult to develop sufficient lateral resistance with small diameter pile foundation elements. As such, where lateral loads on foundation units are high, relatively larger diameter pile elements are required for a more efficient design.
- Where overburden is thin and soft and underlain by very strong, sloping bedrock, the proper seating of driven piles (even if fitted with rock points) will be difficult; driven piles will have a tendency to slide along the sloping bedrock surface. In these instances, a rock socketed pile will provide higher lateral and axial resistances.
- Where bedrock is very strong and sloping, the seating of liners for large diameter conventional caissons (drilled shafts), to allow for the excavation of rock sockets, can be problematic.
- Drilled steel casings advanced with ring bits, using rotary duplex and Down-the-Hole (DTH) hammer drilling methods offer the best chance of achieving proper sealing of the casing and creating bedrock sockets in steeply sloping, very strong bedrock, provided that careful and controlled drilling practices are followed.
- In general, the smaller the diameter of the drill casing, the easier the constructability during over-water construction and ability to achieve a proper seal in the bedrock. However, the diameter of the pile elements must also be adequately large enough to satisfy the structural loading requirements (in particular lateral loading) on the pier.

Considering the challenges noted above, a range of different pile foundation options has been considered for this site as outlined in the following section.

6.4.1 Pile Options

The different pile types considered for support of the piers and abutments include:

- Drilled steel casings socketed into bedrock and filled with concrete.
- Steel H-piles or pipe piles driven to refusal on bedrock.
- Steel H-piles fixed into bedrock sockets backfilled with concrete.
- Micropiles socketed into the bedrock.
- Caissons (drilled shafts) socketed into bedrock.
- Composite foundation elements comprised of a large diameter drilled shaft enclosing several micropiles socketed into the bedrock.

A brief discussion on the installation details for each of the above, including comments on the applicability to the conditions at this site, is provided in the following sections. A comparison of the pile foundation alternatives, including the advantages, disadvantages, relative costs and risks/consequences for the feasible alternatives at the foundation elements for the South Abutment, Pier 1, Piers 2 to 4, and the North Abutment, is presented in Tables 1 to 4, respectively.

The preferred deep foundation/pile type at each foundation element is summarized as follows:



South Abutment

Considering the large embankment fill height at the south approach which results in the south abutment foundation being located about 8 m high above the original ground surface, steel H-piles driven through the granular embankment fill mass and to refusal on bedrock are preferred to support the south abutment (see Table 1). It is understood that this alternative is also preferable from a structural design perspective in order that all of the bridge foundation units be supported on similar (i.e., deep) foundations. Supporting the south abutment on piles will also simplify the design of, and ensure the stability of, the front slope of the south approach embankment. It is noted however that the embankment fill/granular core placed in the area below the south abutment must be granular (i.e., Granular B Type II) and not rock fill so that the H-piles can be driven through the fill mass, and to refusal at depth, without hitting obstructions. Further details on the granular core construction are provided in Section 6.8.2.

Pier 1

Given the constraints and challenges at the Pier 1 location (as described in the previous section), in particular the depth of water, the weak and variable thickness of overburden (that affects the lateral resistance of the piles), as well as the steeply sloping bedrock surface and strong to very strong nature of the bedrock (that affects the potential for proper seating/sealing of pile foundations), 0.609 m diameter drilled steel casings socketed into bedrock are the preferred pile type at this location (see Table 2).

Piers 2 to 4

Considering the depth of water and presence of very soft/weak overburden below the lakebed which provides very little lateral resistance to the piles at Piers 2 and 3, as well as the sloping bedrock surface at Pier 3, 0.609 m diameter drilled steel casings socketed into the bedrock are the preferred pile type for support of Piers 2 and 3 (see Table 3). Driven steel H-piles have been considered for support of these piers, however, detailed analysis has shown that there is insufficient lateral resistance provided by H-piles at the locations of Piers 2 and 3 to enable an efficient design. Driven steel H-piles may provide sufficient lateral resistance at Pier 4 given the stronger near surface soils in this area, however, it is preferable to have a consistent pile type at the piers. As such, in order to maintain a consistent pile type and foundation design at all of the pier locations, it is recommended that 0.609 mm diameter drilled steel casings be installed at Piers 2 to 4.

North Abutment

At the North Abutment, steel H-piles driven to refusal on bedrock are the preferred pile type at this location (see Table 4). The use of H-piles at the north abutment will also result in a similar/consistent foundation design to that recommended for use at the South Abutment.

6.4.1.1 Drilled Steel Casings (0.609 m diameter)

The drilled steel casings are installed by rotary duplex drilling using a sacrificial ring bit on the bottom of a permanent steel casing. A drag bit using a water or polymer flush (discharging to a storage vessel/tank for off-site disposal) is employed to advance and clean out the casing within the overburden. A DTH hammer utilizing water/surfactant flush (discharging to a storage vessel/tank for off-site disposal) is employed to advance and seat



the casing within the rock and also to create an uncased rock socket within the bedrock below the bottom of the casing.

Information from product suppliers indicates that this type of drilling system allows accurate and straight penetration in steeply sloping bedrock surfaces and can also readily penetrate cobbles and boulders. In addition, based on discussions with local piling contractors, this type of system has been successfully used to drill rock sockets in very strong and very steeply sloping (60° to 70° and in some extreme cases up to 80°), granitic bedrock in northern Ontario.

Considering the above and given that this larger diameter pile element can provide sufficient lateral resistance in the weak overburden soils, the use of 0.609 m diameter drilled steel casings is the preferred foundation type at Piers 1 to 4.

In order to develop sufficient capacity in compression and tension, an uncased rock socket with a Length/Diameter (L/D) ratio of at least 3 is recommended. To achieve the axial capacities provided in Section 6.4.2, the uncased socket should have a minimum length of 2 m into the fair to good quality bedrock.

The permanent steel casing must be embedded at least 1 m below the lowest point of contact with the bedrock surface and a minimum of 1 m into fair quality bedrock, but additional casing embedment length may be required to satisfy the lateral loads on the piers, and also to achieve a proper seal in the bedrock prior to socket construction, if the upper bedrock at the pile location is of poor or very poor quality.

The pile is designed to develop the majority of its axial capacity based on the shear resistance along the rock socket wall (i.e., between the concrete and bedrock interface) rather than on end-bearing at the base of the socket. As such, the requirement to thoroughly clean and inspect the base of the socket will be lessened, however a thorough and proper flushing of the side walls of the rock socket is required. A reinforcing bar cage would have to be lowered through the casing and into the rock socket prior to placement of concrete by tremie methods.

6.4.1.2 HP 310x110 Steel H-Piles Driven to Refusal on Bedrock

At Pier 1, steel H-piles driven to bedrock are not considered feasible due to the steeply sloping, very strong bedrock underlying the thin, soft overburden. Although information from product suppliers suggests that special rock points (such as injector-type or Oslo-type) can be used where the bedrock surface is dipping up to 50°, based on discussions with local piling contractors, it is our understanding that proper seating of driven steel piles onto very strong granitic bedrock dipping at angles ranging from about 35° to 45° (or greater) can be problematic, especially where the overburden soils are thin and/or weak and where battered piles are employed. The presence of cobbles and boulders over the bedrock can further complicate the installation process. For these types of conditions (which exist at Pier 1), there is a risk of the piles deflecting off the face/surface of the sloping bedrock, potentially resulting in improper seating, in damage to the piles and/or much longer than anticipated pile elements.

At Piers 2 and 3 (and at Pier 1), for the water depth(s) and variable and very soft, upper subsurface conditions at these locations, detailed analysis indicates that driven steel H-piles will not provide sufficient lateral resistance to enable an efficient pile group design at these foundation units. Although installation and developing sufficient lateral resistance with a driven steel H-pile is considered feasible at Pier 4, it is not the preferred pile type at this location in order to achieve a consistent foundation design at all of the piers. As such, the use of driven steel H-piles is not recommended at the pier locations.



At the South Abutment and North Abutment however, supporting the foundation on steel H-piles driven to refusal on bedrock is feasible. In this regard, driven steel H-piles are the preferred foundation type at the North and South Abutments.

6.4.1.3 HP 310x110 Steel H-Piles Socketed into Bedrock

To avoid problems with the seating of driven steel piles onto sloping bedrock, consideration could be given to placing steel H-piles into 0.609 m diameter sockets drilled into the bedrock (a minimum of 1.5 m deep) by rotary duplex drilling methods and backfilled with concrete. The rock sockets would be constructed using a temporary casing with ring bit and the methodology described in Section 6.4.1.1. Upon completion of drilling of the rock socket, the H-pile would be lowered through the temporary casing and rest on the bedrock at the bottom of the socket and the socket would be backfilled with concrete placed by tremie methods prior to removing the casing. A disadvantage with this method of installation is the disturbance (softening and loosening) that will be created within the overburden upon removal of the temporary 0.609 m diameter drill casing. This disturbance will reduce the already low lateral resistance available within the overburden and would make the design for the high lateral loads on the piers difficult. Given this, the use of steel H-piles socketed into bedrock is not recommended at this site and is not discussed further in this report.

6.4.1.4 Small Diameter Drilled Steel Casings (0.406 m) or Micropiles (0.273 m diameter)

These piles would be installed in a similar manner to that described above for the larger diameter drilled steel casings using rotary duplex drilling, a DTH hammer and a sacrificial ring bit on the bottom of a permanent steel casing. An advantage of using the smaller diameter drilled steel casings (or micropiles) is that, in general, the smaller the diameter of the pile element, the easier to drill, seal and socket into steeply sloping bedrock.

A disadvantage of using the smaller diameter pile elements is that, individually, they provide relatively less lateral resistance. Given the high lateral design loads on the piers, a large number of small pile elements would likely be required. For Piers 1 to 3, given the depth of water and very soft overburden soils, consideration would likely have to be given to installing the small diameter rock socketed pile elements in groups of two or three within a larger diameter permanent outer steel casing. More details of this 'composite' foundation type are described in Section 6.4.1.6.

Based on the above, the small diameter drilled steel casings and micropiles are not considered practical for this site and are not discussed further in this report.

6.4.1.5 0.9 m Diameter Concrete Caissons (Drilled Shafts)

Caissons (large diameter drilled shafts) would be advanced into bedrock using permanent casings and conventional large caisson drilling equipment. It is anticipated that difficulties would arise when attempting to seal the large diameter casings into the very strong, sloping and fractured bedrock at some locations and, in general, the larger the caisson diameter the greater the difficulties in sealing the caisson and drilling the rock socket. Caissons with a diameter larger than 0.9 m are not recommended at this site due to the potential constructability issues associated with the sloping bedrock. The presence of cobbles and boulders overlying the bedrock will make advancing the large diameter casings and sealing them into bedrock more difficult. If a proper seal cannot be formed, there will be difficulties forming the rock socket below the casing. Given the risks associated with drilling the larger diameter caisson hole and rock socket, combined with the difficulties associated with the use of this



larger type of drilling equipment for over-water work, the use of caissons is not recommended at this site and is not discussed further in this report.

6.4.1.6 Composite Foundation Element (Drilled Shaft and Small Diameter Piles)

Composite foundation elements, comprised of a large diameter drilled shaft (or caisson) enclosing a number of smaller diameter, equally spaced piles, could be considered for support of some of the piers at this site. With this type of foundation element, a permanent steel casing would be initially installed through the overburden, terminating at a point above the bedrock surface. After cleaning out the outer casing, the smaller diameter pile elements would be installed through and below the casing and either terminated on or socketed into the underlying bedrock, following which the outer casing would be backfilled with concrete, embedding the small diameter pile elements into a composite section. The smaller diameter pile elements could be comprised of micropiles (each consisting of a permanent steel casing and central reinforcing bar advanced into bedrock from within the drilled shaft) or driven steel H-piles.

The composite foundation has the combined advantage of a large diameter along the upper portion of the pile to provide higher lateral resistance in weaker soils, as well as small diameter pile elements at the base of the pile that can either be more easily socketed into strong and/or sloping bedrock (micropiles) or driven to refusal on bedrock (driven steel piles) depending on the subsurface conditions.

Given the very thin overburden and very steeply sloping bedrock at Pier 1, the construction of composite foundation elements is likely not feasible at this location. The use of such a foundation at Piers 2 and 3 may be feasible, but it is not considered practical or cost effective to have multiple, different foundation types for a single bridge structure. In addition, detailed soil-structure interaction analysis would be required to optimize the relative lengths of the upper drilled shaft and lower micropiles and also to determine the minimum number of micropiles and cross-section composition (stiffness) required to transfer the high axial loads to the bedrock.

Based on the above considerations from a foundations perspective, this type of foundation is not recommended for this site.

6.4.2 Piles

The contribution of the overburden soils to the axial capacity of the piles will be relatively small and as such it is recommended that all drilled steel casing pile foundations at Piers 1 to 4 be advanced to and socketed into the bedrock. At the South and North Abutments, the recommended steel H-piles would be driven to refusal on bedrock.

Based on the subsurface information at the boreholes, the following summarizes the details of the river bed elevation, approximate bedrock surface elevation (interpreted where applicable), depth to bedrock below the design underside of the pile cap, minimum recommended casing embedment lengths and rock sockets lengths (for the drilled steel casing pile foundations), as well as approximate pile lengths for steel H-piles driven to refusal at the South Abutment, and the North Abutment.



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Foundation Element (Borehole)	Underside of Pile Cap Elevation (m)	Approx. Lake Bed Elevation (m)	Approx. Bedrock Surface Elevation (m)	Depth to Bedrock Below Underside of Pile Cap (m)	Recommended Pile Type	Minimum Casing Embedment Length below Top of Bedrock ¹ (m)	Minimum Uncased Rock Socket Length ² (m)	Estimated Length of Pile Below U/S Pile Cap ³ (m)
South Abutment (B502-04 to B502-08)	193.6	N/A	183 to 177	11 to 17	H-pile	N/A	N/A	11 to 17
Pier 1 (B502-01)	175.7	174.6	160 to 155	16 to 21	Drilled Steel Casing	1.0	2.0	19 to 24
Pier 2 (B502-10 and B502-11)	175.7	175.6	134 to 131	42 to 45	Drilled Steel Casing	1.0	2.0	45 to 48
Pier 3 (B502-02)	175.7	177.2	154 to 151	22 to 25	Drilled Steel Casing	1.0	2.0	25 to 28
Pier 4 (B502-13)	181.7 (based on 1.8 m of frost cover)	N/A	169 to 168	13 to 14	Drilled Steel Casing	1.0	2.0	16 to 17
North Abutment (S502-24)	190.1	N/A	174 to 172 ⁴	16 to 18	H-Pile	N/A	N/A	16 to 18

- Note: 1. Bedrock is sloping. Minimum casing embedment into bedrock to be determined relative to lowest elevation/point of contact of casing onto bedrock surface. Additional casing embedment length into bedrock may be required to satisfy the lateral loads on the pier; to be determined by structural engineer per Section 6.4.2.3.
2. Minimum uncased socket embedment length of pile into bedrock to satisfy the axial geotechnical resistance presented in Section 6.4.2. Additional embedment length into bedrock may be required to satisfy the lateral loads on the pier; to be determined by structural engineer per Section 6.4.5.
3. Bedrock surface is variable and the actual pile lengths will vary and will be determined during pile installation.
4. Bedrock surface inferred from split-spoon, casing and/or auger refusal; not confirmed by rock coring.

Due to the sloping surface of, and varying depths to, the bedrock at this site and given the distance separating the closest borehole from the proposed foundation element at Piers 1 to 4 and the North Abutment, the actual pile lengths at the foundation units will vary and the estimated pile lengths indicated above should be considered approximate only. The Contract should allow for the supply/installation of varying pile lengths.

It is noted that the minimum casing embedment length below the top of bedrock has been selected considering the variation in the RQD of the bedrock at the boreholes, where available, and estimated at the other locations. The RQD of the bedrock at the site is variable and deeper casing embedment into the rock may be required at some locations in order to achieve a proper seal prior to constructing the uncased rock socket. In addition, it is noted that the bedrock surface at the site is steeply sloping in some areas and the minimum casing embedment length into bedrock should be determined relative to lowest elevation/point of contact of casing onto the bedrock surface.

A preliminary assessment of the possible range in contact angles between the drilled steel casing and the bedrock surface has been carried out based on the proposed pier locations, pile layout and pile batters (as provided by URS). Based on this information, it is estimated that the piles will come in contact with the bedrock surface at angles (relative to the axis of the piles) ranging from as steep as approximately 80° (i.e., close to perpendicular



contact to bedrock for Pier 4 piles battered to the north) to as shallow as approximately 30° (for Pier 1 piles battered to the north).

6.4.2.1 Geotechnical Axial Resistances / Reactions

As noted in Sections 6.4 and 6.4.1, several deep (pile) foundation options have been considered for support of the south and north abutments and piers. However, after considering the challenges associated with constructability of the different pile types on the steeply sloping and strong to very strong bedrock at Pier 1, the presence of the very soft upper soils at Piers 2 and 3, the following foundation types are considered the most suitable for this site:

- 1) **South Abutment:** steel H-piles driven through the granular embankment fill and to refusal on bedrock.
- 2) **Piers 1 to 4:** drilled steel casings (0.609 m diameter) with casings embedded a minimum of 1 m into fair quality (i.e., rock mass with RQD > 50 per cent as per Table 3.10 of CFEM, 2006) bedrock and a minimum 2 m uncased rock socket below the bottom of the casing and into the fair to good quality rock, with steel reinforcement and filled with 30 MPa concrete placed by tremie methods.
- 3) **North Abutment:** steel H-piles driven to refusal on bedrock.

It is recommended that the axial capacity of the drilled steel casings be designed primarily based on side wall resistance within the rock socket along the concrete (or grout) and bedrock interface. The contribution from end-bearing resistance within the rock socket will depend on the level of cleanliness/removal of debris and sediment that can be achieved and given that the piles are long (between 16 m and 48 m), battered and will be filled with water, proper cleaning and inspection of the socket base may be difficult. An example NSSP for drilling, flushing/cleaning and inspecting the bottom of the rock sockets is included in Appendix D.

For drilled steel casings supported in the fresh to slightly weathered granitic gneiss bedrock, the strength of the concrete will be less than the bedrock strength and as such the concrete strength will govern. A factored side wall resistance at ULS of 1.3 MPa may be assumed for design assuming a minimum concrete strength of 30 MPa. The casing for each drilled pile should extend a minimum of 1.0 m below the lowest elevation/point of contact of the casing onto the bedrock surface.

The following summarizes the factored axial geotechnical resistance and reaction for the different foundation options at the pier and north abutment locations.

Pile Foundation Alternative	Factored Axial Geotechnical Resistance at ULS ¹ (kN) (Compression)	Axial Geotechnical Reaction at SLS ² (kN) (Compression)	Factored Axial Geotechnical Resistance at ULS (kN) (Tension)
0.609 m diameter Drilled Steel Casings (Piers 1 to 4)	5,000	N/A	3,000
HP310X110 (South Abutment and North Abutment)	2,000	N/A	N/A

Note: ¹ Uncased rock socket length = 2 m (minimum). Structural capacity of pile must be checked.

² For piles founded on bedrock, the SLS reaction for 25 mm of settlement is greater than the ULS resistance and therefore ULS governs.



The recommended embedment lengths into bedrock for the drilled steel casings are the minimum required to satisfy the axial loads provided above. Additional embedment length into bedrock may be required to satisfy lateral loads on the piers and is to be determined by the structural engineer (refer to Section 6.4.2.3 below).

6.4.2.2 *Downdrag Load (Negative Skin Friction)*

At Piers 1 to 4, soft organic silt and/or clayey silt strata was encountered in the boreholes advanced at the foundation units. However, given that no filling is proposed to be carried out within the vicinity of the piers, consolidation and settlement of the clayey silt stratum at the piers is not anticipated and as such, drag loads are not expected on the pier pile foundations. At Pier 3, downdrag may occur and drag loads will need to be incorporated into the design of the piles if consideration is given to constructing a temporary access platform with granular fill to facilitate equipment access for pier construction, as discussed further in Section 6.9.3.

At the North Abutment, if the H-piles are installed prior to the preload period discussed in Section 6.7.2.5, settlement of the foundation soils (as a result of embankment construction) will result in downdrag occurring. It is estimated that drag loads of approximately 1,000 kN/pile for the H-piles could occur. To avoid these drag loads, it is recommended that the piles be installed after completion of the three month north embankment preload period.

6.4.2.3 *Resistance to Lateral Loads*

The design of piles subjected to lateral loads should take into account such factors as the batter of the piles, the relative rigidity of the pile to the surrounding soil and bedrock, the fixity condition at the head of the pile (pile cap level) as well as at the base of the pile, the structural capacity of the pile to withstand bending moments, the soil and/or bedrock resistance that can be mobilized, the tolerable lateral deflection at the head of the pile and the pile group effects. For a longer, more flexible pile, the maximum yield moment of the pile may be reached prior to mobilisation of the full lateral geotechnical resistance. For design purposes, both the structural and geotechnical resistances should be evaluated to establish the governing case.

Where ground conditions are generally competent and the lateral loads on piles are relatively small such that the maximum lateral pile deflections will be relatively small, the resistance to lateral loading in front of a single pile can be estimated using subgrade reaction theory (as outlined below for the South Abutment, Pier 4 and the North Abutment). However, response of a pile to lateral loads is highly nonlinear and methods that assume linear behavior (such as subgrade reaction theory) are only appropriate where the maximum pile deflections are less than 1 per cent of the pile diameter, where the loading is static (no cycling) and where the pile material is linear (CFEM, 2006). Where these conditions are not met, the non-linear lateral behavior of the soil should be considered by the use of P-y curves.

Considering the challenging design conditions at some foundation elements at this site (i.e., at Piers 1 to 3 that have high pier columns, a long free-length or unsupported length of pile through relatively deep water in the lake, and very soft organic soils below the lake bed), it is recommended that a more rigorous soil-structure interaction analysis employing P-y curves (that better represent the non-linear lateral soil behaviour) be carried out, in particular for the pile groups at these critical foundation units.

It is our understanding that URS has carried out preliminary soil-structure interaction analysis using P-y curves to design the layout of the pile groups at the piers employing the commercially available software package FB-MultiPier (by BSI). The modelling requirements have been discussed with URS, and foundations input has



been provided on the selection of the soil and rock models for the different overburden layers and bedrock conditions. A summary of the recommended models for the lateral group pile analysis using P-y curves along with the key soil parameters for each is provided for South Abutment in Table 5, Piers 1 to 4 in Tables 6 to 8 and for the North Abutment in Table 9.

In the boreholes advanced in the vicinity of the piers, the bedrock generally slopes downwards towards the centre of the lake and as such the proposed piles that are battered towards the centre of the lake will have a greater length and will provide relatively lower lateral resistance compared to the piles that are battered towards the shore.

For a single 0.609 m diameter drilled steel casing with a 12 mm wall thickness (at Piers 1 to 4) and for a single HP310X110 driven steel H-pile (at the South and North Abutments) advanced to the design depths provided in Section 6.4.2 and battered at 1H:10V towards the centre of the river (i.e., perpendicular to the pier centreline), the estimated factored lateral resistance at ULS and the lateral reaction at SLS (for 10 mm of horizontal deflection at the pile cap) are presented below. These values are based on analysis carried out using the commercially available program LPILE Plus (Version 2013.7.07), developed by Ensoft Inc.

Foundation Location	Recommended Pile Type	Factored Geotechnical Lateral Resistance at Ultimate Limit States (ULS) (kN)	Geotechnical Lateral Reaction at Serviceability Limit States (SLS) for 10 mm of Deflection (kN)
South Abutment	HP310X110	140 ¹	250 ¹
Pier 1	Drilled Steel Casing	165	70
Pier 2 and 3	Drilled Steel Casing	150	80
Pier 4	Drilled Steel Casing	280	255
North Abutment	HP310X110	130	215

Note: ¹ Based on 13.7 m thick granular core constructed in accordance with the requirements described in Section 6.8.2 and extending a minimum of 3 m beyond the plan limits of the cap at the pile cap level.

The lateral resistances given above are based on an assumed drilled steel casing embedment length into bedrock of 3 m and HP310X110 piles driven to refusal on bedrock; an assumed fixed-head pile condition, and an axial load applied to the top of pile. No bending moment was applied to the top of the pile. The lateral resistances should be reviewed if greater vertical loads or a different loading condition is anticipated as additional embedment length into bedrock may be required to satisfy the lateral loads on the pier, which is to be determined by structural engineer.

As noted above, at the South Abutment, Pier 4 and the North Abutment, so long as the maximum pile deflections are less than 1 per cent of the pile diameter, the loading is static (no cycling) and the pile material is linear (CFEM, 2006), the resistance to lateral loading in front of a single pile could also be estimated using subgrade reaction theory and the coefficient of horizontal subgrade reaction, k_h (kPa/m), where the coefficient of horizontal subgrade reaction, k_h (kPa/m), is based on the following equations (CFEM, 1992 as referenced in the CHBDC Commentary, 2006).



For non-cohesive soils:

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

where: n_h = constant of subgrade reaction (kPa/m) (see Tables 5 to 9)
 z = depth (m)
 B = pile diameter or width (m)

And for cohesive soils:

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

where: s_u = undrained shear strength of the soil (kPa) (see Tables 6 to 9)
 B = pile diameter or width (m)

6.4.2.4 Group Effects

Group action for lateral loading should also be considered when the pile spacing in the direction of the loading is less than six to eight pile diameters. Group action can be evaluated by reducing the coefficient of horizontal subgrade reaction, or the lateral reaction defined by the P-y curve(s) (NAVFAC, 1982) in the direction of loading by a reduction factor, R, as follows:

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

Where a pile group is oriented perpendicular to the direction of loading, group action may be considered by reducing the coefficient of horizontal subgrade reaction, or the lateral reaction defined by the P-y curve(s) (NAVFAC, 1982) by a reduction factor R as follows:

Pile Spacing Perpendicular to Direction of Loading d = Pile Diameter	Horizontal Subgrade Reaction Reduction Factor, R
4 d	1.00
1 d	0.50

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



6.4.3 Frost Protection

As discussed in Section 6.3.4, the required thickness of conventional soil cover for frost protection for the foundations at this site is 1.8 m, as per OPSD 3090.010.

At Piers 1 to 3, the underside of the pile caps is proposed to be at Elevation 175.7 m according to the GA, which is about 2.8 m below the water level in Straight Lake as measured in February 2014. The proposed elevation of the underside of the pile cap is considered sufficient from a frost penetration perspective provided that ice does not extend below Elevation 175.7 m. If it is possible that the lake ice could extend below Elevation 175.7 m, the proposed underside of the pile cap should be lowered.

At Pier 4 and the south and north abutments, the underside of the pile caps should be founded 1.8 m below final ground surface for frost protection.

6.5 Seismic Site Coefficient

6.5.1 Site Coefficient

For seismic design purposes, at the south abutment, given that the bedrock is either exposed or the overburden/backfill below the footing will be comprised of stable deposits of sand and gravels, the Site Coefficient, S , may be taken as 1.0 considering the guidelines in Section 4.4.6 of the *CHBDC* (2006), consistent with Soil Profile Type I. At the piers and the north abutment, given the thickness and consistency/relative density of the overburden soils, the Site Coefficient, S , may be taken as 1.5 consistent with Soil Profile III.

6.5.2 Seismic Analysis Coefficient

According to the National Building Code of Canada (1995) seismic hazard values (as referenced in the *CHBDC* (2006) and its Commentary), the site specific peak horizontal ground acceleration for the Sudbury and Parry Sound area is 0.051 (for a probability of exceedance of 10 per cent in 50 years). According to Table 4.1 of the *CHBDC*, this site is located in Seismic Performance Zone 1 and the corresponding site-specific zonal acceleration ratio, A , is 0.05.

Given this assessment and the fact that the proposed bridge structure is not designated as a lifeline or truss bridge, and in accordance with Section 4.4.5.1, Table 4.2 of the *CHBDC*, no seismic analysis is required for structures located in Seismic Performance Zone 1.

6.6 Lateral Earth Pressures

The lateral earth pressures acting on the abutment stems and any associated wing walls/retaining walls will depend on the type and method of placement of the backfill material, the nature of the soils behind the backfill, the magnitude of surcharge including construction loadings, the freedom of lateral movement of the structure, and the drainage conditions behind the walls.

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



- Select, free draining granular fill meeting the specifications of OPSS.PROV 1010 Aggregates Granular 'A' or Granular 'B' Type II, should be used as backfill behind the walls. Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill in accordance with OPSD 3102.100 (Walls, Abutment, Backfill, Drain) and OPSD 3190.100 (Walls, Retaining and Abutment, Wall Drain). Compaction (including type of equipment, target densities, etc.) should be carried out in accordance with OPSS.PROV 501 (Compacting). Other aspects of the granular backfill requirements with respect to sub-drains and frost tapers should be in accordance with OPSD 3101.150 (Walls, Abutment, Backfill, Minimum Granular Requirement) and OPSD 3121.150 (Walls, Retaining, Backfill, Minimum Granular Requirement).
- For structures that are not comprised of integral or semi integral abutments, rock fill may be used as backfill behind the walls and the material should meet the specification as outlined in the Northeastern Region Engineering Directive (2002) for backfill of structures adjacent to rock embankments. Other aspects of rock backfill requirements should be in accordance with OPSD 3101.200 (*Walls, Abutment, Backfill, Rock*). The following parameters (unfactored) may be used for rock backfill:

Fill Type	Soil Unit Weight	Coefficients of Static Lateral Earth Pressure	
		At-Rest, K_o	Active, K_a
Rock Fill	19 kN/m ³	0.36	0.22

- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the wall stem, in accordance with CHBDC Section 6.9.3 and Figure 6.6. Other surcharge loadings should be accounted for in the design as required.
- For restrained walls, granular fill should be placed in a zone with the width equal to at least 1.8 m behind the back of the walls (in accordance with Figure C6.20(a) of the Commentary to the CHBDC). For unrestrained walls, granular fill should be placed within the wedge shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical (1.5H:1V) extending up and back from the rear face of the footing (in accordance with Figure C6.20(b) of the Commentary to the CHBDC). The pressures are based on the proposed embankment fill materials and the existing overburden soils and the following parameters (unfactored) may be used assuming the use of granular fill or rock fill:

Fill Type	Soil Unit Weight (kN/m ³)	Coefficients of Static Lateral Earth Pressure	
		At-Rest, K_o	Active, K_a
Granular 'A'	22	0.43	0.27
Granular 'B' Type II	21	0.43	0.27
Rock Fill	19	0.36	0.22

If the wall support and superstructure allow lateral yielding of the stem, active earth pressures may be used in the geotechnical design of the structure. If the abutment support does not allow lateral yielding, at-rest earth pressures should be assumed for geotechnical design. The movement required to allow active pressures to develop within



the backfill, and thereby assume an unrestrained structure for design, should be calculated in accordance with Section C6.9.1 and Table C6.6 of the *Commentary* to the *CHBDC*.

6.7 Approach Embankment Design

Based on the GA drawing and cross-sections provided by URS, the proposed road grade at the new south and north approaches will be at about Elevations 199.3 m and 195.8 m, respectively.

At the South Approach, the existing ground surface at the investigated location (about 20 m south of the abutment) is at Elevation 190.2 m, and at the South Abutment area, the ground surface varies from about Elevations 186.4 m to 185.7 m. At the south end of the South Approach, the embankment will be about 9.1 m high and constructed on bedrock, while in the area of the South Abutment, the approach embankment height will increase up to about 13.6 m. As discussed in Sections 6.3 and 6.8, prior to placing embankment fill in the south abutment area, sub-excavation to between Elevations 179.9 m (Borehole B502-04) and 183.4 m (Borehole B502-08) is recommended to remove the upper 2.9 m to 5.8 m of very loose to loose sandy silt to sand to organic silt deposits. It is noted that wood fragments and/or roots were encountered within several of the deeper samples at this location as noted on the Record of Boreholes.

At the North Approach, the existing ground surface at about 20 m north of the abutment is at Elevation 191.5 m (based on the contour elevations on the GA) and at the North Abutment area, the ground surface is about Elevation 190.3 m. At the North Abutment, the approach embankment will about 5.5 m high relative to existing ground surface with sub-excavation of the upper 0.8 m of organic material required. The foundation soils at the abutment area (Borehole S502-24) generally consist of very loose to compact silt to sand up to 16.9 m thick with an approximately 1.8 m thick layer of firm deposit of clay within the silt to sand strata at 11.7 m depth.

It is understood that rock fill is the preferred embankment fill material for this project. However, in order to facilitate pile installation to support the south and north abutments, all fill placed below the abutment areas must be granular fill and not rock fill. In this regard, the stability and settlement analyses have been carried out on the basis that the highway embankment will be constructed of rock fill and that granular fill will be constructed in the core of the embankment below the abutments. In accordance with MTO Northern Region Pavement Practices and Guideline (1997) as amended by MTO Memorandum "Use of Mid-Slope Berms for Rock Fill Embankments" (2005), 2 m wide berms should be incorporated into the rock fill embankment side slope profile for uninterrupted slopes greater than 10 m high. It should be noted that sections of the south approach embankment are expected to be in excess of 10 m high and as such, 2 m wide mid-slope berms will be required along the higher section(s) of the embankment.

The following sections address stability and settlement analysis for the new approach embankments.

6.7.1 Stability

At the South Approach, the embankment will be founded either directly on bedrock or on the compact to very dense non-cohesive native soils below the south abutment area. It is understood that the south abutment will be founded on piles and so the loading from the abutment foundation will not influence the stability of the high embankment fill. As such, global instability of the South Approach embankment is not expected, so long as embankment side slopes are constructed no steeper than 2H:1V for granular fill and no steeper than 1.25H:1V for rock fill, with a 2 m wide mid-height berm incorporated in the approach embankment. However, given that the



approach embankment in the south abutment area will be very high (up to 13 m high), a stability analysis has been carried out to check the stability of the front slope, as discussed below.

At the North Approach, the embankment fill will be up to about 5.5 m high and constructed primarily over native sands and silts. Although the north abutment will also be founded on piles so that the loading from the abutment foundation will not influence the embankment stability, the presence of a clayey stratum at depth between the approach and Straight Lake has the potential to affect the stability of the front slope of the approach. Given this, a stability analysis has been carried out to check the stability of the front slope, as discussed below.

The following sections outline the methodology used to evaluate embankment stability at the South and North Approaches. Parameters used in the analyses for each of the critical section(s) are also presented as well as the results of the stability analyses.

6.7.1.1 Methodology

Stability analyses were carried out for the critical sections of the proposed fill embankments at the approaches, which consist of the front slopes (for both the North and South Approaches) and the side slopes immediately behind the approach slab at STA 21+085 (for the North Approach). Critical sections correspond to the greatest new embankment height and/or the maximum thickness of soft, compressible cohesive soils. The stability of the proposed new embankment section(s) were analyzed using limit equilibrium methods. The stability analyses assume that all of the organic soils encountered at/below ground surface (including at depth below the south abutment) have been removed and replaced in accordance OPSD 203.010 (Embankments Over Swamp – New Construction) prior to construction of the new embankments.

All limit equilibrium slope stability analyses were carried out using the commercially available program Slide (Version 6.0), developed by Rocscience Inc., employing the Morgenstern-Price method of analysis. For all analyses, the Factor of Safety (FoS) of numerous potential failure surfaces was computed in order to establish the minimum FoS. The FoS is defined as the ratio of the forces tending to resist failure to the driving forces tending to cause failure. A target minimum FoS of 1.3 is normally adopted for the design of embankment slopes under static conditions for MTO embankments. This FoS is considered adequate for the approaches considering the design requirements and the field data available and is based on deep-seated, global failure surfaces that would affect the operation of the highway. The stability analyses were carried out to check that the target minimum FoS was achieved for the embankment heights and geometries considered.

6.7.1.2 Parameter Selection

The following is a summary of embankment slope geometries, unit weights and effective friction angles/cohesion for the rock fill and granular pad fill modelled in the analyses.

Fill Type	Recommended Slope Inclination	Unit Weight, γ (kN/m³)	Effective Friction Angle, ϕ' (°)	Cohesion, c' (kPa)
Rock Fill	1.25H:1V	19	40	0
Granular Fill (Granular 'B' Type II)	1H:1 (min)	22	35	0



At the South Approach, the bedrock is either outcropping or present at relatively shallow depth below overburden soils comprised primarily of granular soils (sands, silts and gravels). All organic soils (including those at depth), are to be removed prior to fill embankment construction.

At the North Approach, the overburden encountered is generally composed of granular soils (silts and sands) with thin layers/pockets of cohesive deposits (clayey silt, silty clay and/or clay) present at depth.

For granular soils, effective stress parameters were employed in the stability analyses assuming drained conditions. The effective stress parameters (effective friction angle and effective cohesion) for the granular soils were estimated from empirical correlations proposed by US Navy (1986) using the results of in situ SPTs, in conjunction with engineering judgement based on experience in similar soil conditions.

For cohesive deposits and for the clayey organics, or plastic silts (that will remain in place beyond the embankment footprint), total stress parameters were employed in the embankment assuming short-term, undrained conditions. The total stress parameters (i.e., average mobilized undrained shear strength – s_u) for the cohesive soils were assessed based on the results of in situ field vane shear tests, inferred from the laboratory consolidation tests results, and estimated from correlations with the SPT results and other laboratory test data (i.e., natural water content), where appropriate. From the consolidation tests, the following correlation proposed by Mesri (1975) was employed to estimate the undrained shear strength:

$$s_u = 0.22 \sigma'_p$$

where:

s_u	=	average mobilized undrained shear strength (kPa)
σ'_p	=	preconsolidation stress (kPa)

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

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

where:

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

The simplified stratigraphy together with the associated unit weight(s) and foundation engineering parameters employed for the different native soil types at the critical sections at the South and North Approach areas are summarized below.

Stratigraphic Unit	γ' (kN/m ³)	ϕ' (°)	c' (kPa)	S_u (kPa)
South Approach				
Sand and Gravel	22	34	0	-
Silt to Silty Sand	19	29	0	-



Stratigraphic Unit	γ' (kN/m ³)	ϕ' (°)	c' (kPa)	S_u (kPa)
North Approach				
Peat / Topsoil	12	28	1	-
Organic Silt	15	-	-	10
Clayey Silt to Silty Clay (Surficial)	17	-	-	30
Sand	18.5	29	0	-
Silt to Sandy Silt	19	28	0	-
Clayey Silt (Pocket)	17	-	-	30
Clayey Silt to Clay	17	-	-	20 to 45
Silt to Silt and Sand (Pockets)	19	28	0	-
Gravelly Silt and Sand	20	32	0	-

6.7.1.3 Results of Stability Analysis

The stability analyses performed on the front and side slopes at the North Approach indicate that after completion of construction, the embankment will have a FoS greater than 1.3 for a deep-seated, global failure surface as shown on Figures 1 and 2, for the front and side slopes, respectively.

The stability analysis performed on the front slope at the South Approach/Abutment area indicates that after construction of the embankment fill, including the granular core to facilitate pile installation and assuming the south abutment is supported on piles, the embankment will have a FoS of 1.3 for a deep seated, global failure surface as shown on Figure 3.

6.7.2 Settlement

The following sections outline the methods used to carry out the settlement analyses at the south and north approaches and also present the parameters used in the analyses for each of the embankment critical section(s). The results of the analyses are also presented along with recommendations to mitigate post-construction settlement as applicable.

6.7.2.1 Methodology

To estimate the magnitude of the expected settlements, analyses were carried out at the critical sections of the proposed fill embankments using the commercially available program Settle3D (Version 3.0), developed by Rocscience Inc., as well as hand/spreadsheet calculations. Critical sections correspond to the greatest new embankment height and/or the maximum thickness of compressible soils. The settlement analyses assume that



all of the organic soils encountered at/below ground surface have been removed and replaced in accordance with OPSD 203.010 (Embankments Over Swamp – New Construction) prior to construction of the new embankments.

The sources of settlement are considered to include:

- Immediate settlement of the native granular soils.
- Consolidation settlement of the thin native clay layer at the north approach.
- Self-weight compression of the backfill and embankment fill materials (short-term and long-term).

Below the footprint of the South Abutment, the compact to very dense non-cohesive soils that will remain in place following sub-excavation and removal of the organic layers (as discussed in Sections 6.7 and 6.8.1) are expected to settle less than 25 mm under the loading from the construction of the approach embankment and granular core. Further, the granular core will be compacted and the majority of the settlement associated with the placement of the backfill materials as well as the self-weight of the granular fill is expected to occur during the construction. As such, the critical section at the South Approach is the 9.1 m high rock fill embankment overlying bedrock about 20 m behind the abutment (near Borehole B502-03).

At the North Approach, the critical section is located at approximately STA 21+085 where the proposed approach embankment is up to about 4.4 m high overlying about 15 m of non-cohesive soils and an approximately 1 m thick cohesive layer (at depth).

6.7.2.2 *Parameter Selection*

At the North Approach, the immediate compression of the 15.1 m thick non-cohesive deposits (i.e., silt to sand) was modelled by estimating elastic moduli of deformation of 5 MPa to 10 MPa based on the SPT 'N'-values and using correlations proposed by Bowles (1984) and Kulhawy and Mayne (1990). These estimated values were compared with the typical range of expected values for similar soil types, as outlined in CHBDC (2006) and adjusted, as necessary. The settlement of the estimated approximately 1 m thick clay deposit anticipated to be present at a depth of about 11.7 m below the north abutment/approach area was assessed using the parameters summarized below.

Foundation Area	Consolidation Settlement Parameter(s)	
Straight Lake NBL - North Approach STA 21+085 Clay Approx. Elev. 178.6 m to 177.6 m	γ	17 kN/m ³
	σ_p'	110 kPa
	e_o	1.4
	C_c	1.2
	C_r	0.06
	C_v	1.4×10^{-3} cm ² /s

6.7.2.3 *Settlement of Rock Fill*

Where rock fill is used for the construction of the proposed embankments, there will be settlement due to compression of the rock fill itself under self-weight, in addition to the settlement of the underlying foundation soils as described above. The magnitude of settlement of the rock fill depends on the following factors:



- type of rock/strength of particles
- size and shape of rock particles
- gradation of rock fill
- total height/thickness of rock fill (stress level)
- method of construction and sequence of placement (including lift thickness, compactive effort and state of packing)

The settlement of rock fill occurs as a result of re-arrangement of rock particles under load and wetting and as a result of localized crushing of rock particles at point contacts. The magnitude of both the short-term and long-term post-construction settlement of the rock fill is a function of the height of fill as well as the method of fill placement (i.e., compacted versus dumped rock fill) as outlined in “MTO Guideline for Rock Fill Settlement and Rock Fill Quantity Estimates”, dated September 2010.

Rock fill should be placed, whenever possible, in a controlled manner (i.e., not end-dumped) in accordance with OPSS.PROV 206 (Grading). Blading, dozing and ‘chinking’ the rock fill to form a dense, compact mass is required to minimize voids and bridging and reduce settlements, and should be used to construct rock fill embankments above the existing groundwater table. Where rock fill cannot be placed in a controlled manner (i.e., below the groundwater table), the post-construction settlement of the rock fill is expected to be greater.

Short-Term Rock Fill Settlement

The magnitude of short-term post-construction settlement associated with compacted and end-dumped rock fill may be estimated in accordance with the MTO Guideline (September 2010), as follows:

Height of Rock Fill, H	Short-Term Rock Fill Settlement	
	Compacted Rock Fill	Dumped Rock Fill
Up to 5 m	0.5% H	1.0% H
>5 m to 10 m	0.75% H	1.5% H
>10 m to 15 m	1.0% H	2.0% H

Approximately 90 per cent of the short-term settlement may be expected to occur within the first six months following construction of the embankment to full height. The short-term settlement is expected to be fully completed within one (1) year following the completion of embankment construction to full height.

Long-Term Rock Fill Settlement

The magnitude of long-term post-construction settlement for compacted and end-dumped rock fill may be estimated in accordance with the MTO Guideline (September 2010), as follows:



Total Height of Rock Fill, H	Long-Term Rock Fill Settlement	
	Compacted Rock Fill	Dumped Rock Fill
Up to 15 m	0.1% H	0.2% H

The long-term rock fill settlement is expected to occur from one year following the completion of construction over the life of the embankment.

6.7.2.4 Settlement Performance Requirements

Based on MTO's *Embankment Settlement Criteria for Design* (MTO 2010) the following post-construction settlement and differential settlement criteria are considered acceptable to occur within 20 years post-paving for the bridge approach embankments at this site.

Location	Distance from Transition Point (i.e., Abutment)	Total Post-Construction Settlement (mm)
Transition/Taper to Bridge Abutments	0 m to 20 m	25
	20 m to 50 m	50
	50 m to 75 m	100

These criteria have been used for determining whether mitigation measures are required to limit post-construction settlement of the approach embankments.

6.7.2.5 Results of Settlement Analysis

At the South Approach, about 20 m behind the abutment, the total settlement of the rock fill embankment itself (based on a 9.1 m high embankment) is estimated to be about 80 mm, with about 60 mm expected to occur within six (6) months of construction of the embankment, about 10 mm occurring during the next six months and about 10 mm expected to occur over the remaining design life of the embankment. In order to satisfy the post-construction settlement criterion of 25 mm over a 20-year period following completion of construction, preloading of the rock fill for a duration of six months will be required.

At the North Approach, immediately behind the North Abutment, the embankment will be about 4.4 m high relative to the original ground surface with 0.8 m of sub-excavation required. The results of the settlement analysis indicate that with the surficial organic material and any near surface clayey silt to silty clay sub-excavated and replaced with rock fill, the settlement of the foundation soils at the critical section is estimated to be about 305 mm. The settlement is estimated to be comprised of about 235 mm of immediate settlement due to compression of the non-cohesive deposits and about 70 mm of primary consolidation within the relatively thin cohesive deposit.

Based on an average coefficient of consolidation (c_v) of about $1.4 \times 10^{-3} \text{ cm}^2/\text{s}$ estimated for the cohesive deposit at the North Approach, the imposed loading conditions for the approximately 4.4 m high embankment plus about 0.6 m backfill for replacement of the surficial organics and/or surficial clayey silt/silty clay layer, and assuming



two-way drainage of the 1 m thick cohesive deposit, it is estimated that about 90 per cent of the primary consolidation settlement will be completed in about 20 days.

The magnitude of total secondary consolidation (creep) settlement for the cohesive deposit at the north approach is estimated to be about 5 mm per log-cycle of time for this area corresponding to about 15 mm over a 20-year period following completion of construction.

In addition, the total settlement of the rock fill embankment itself (based on a 4.4 m high embankment plus about 0.6 m of additional rock fill required after removal of the surficial organic/silty clay deposits) is estimated to be about 30 mm, with about 20 mm expected to occur within six (6) months of construction of the embankment, 5 mm occurring during the next six (6) months and about 5 mm expected to occur over the remaining design life of the embankment.

In order to satisfy the long-term settlement performance criterion of the north approach embankment, preloading of the rock fill embankment for a duration of 90 days (3 months) will be required. The magnitude of the remaining primary and secondary consolidation settlement of the clay interlayer and the post-construction settlement due to compression of the rock fill embankment is estimated to be about 25 mm within the first 20 years following completion of construction, within the settlement tolerances outlined in Section 6.7.2.4. As discussed in Section 6.4.2.2, preloading in the north abutment area is also recommended prior to pile installation to avoid downdrag and drag loads on the piles.

6.8 Subgrade Preparation and Embankment Construction

The following sections provide recommendations for overburden excavations as well as for backfill, granular core construction and embankment fill placement in abutment areas.

6.8.1 Overburden Excavation for Approach Embankments

Prior to the placement of any fill, all organic materials (including peat, topsoil and organic silt near surface and at depth) and surficial clayey silt/silty clay layers as encountered at the site must be stripped from the plan limits of the proposed works, following which the exposed subgrade should be heavily proof-rolled, where applicable.

At the South Approach, about 20 m behind the abutment, excavations/stripping will consist of removal of the very thin layer of organics overlying the bedrock. However, in the abutment/granular core area, excavations will extend to between Elevation 183.3 (B502-08) and 179.9 m (B502-04) to remove the upper 2.9 m and 5.8 m of very loose to loose sandy silt to sand to organic silt deposits as discussed in Sections 6.3 and 6.4.1 prior to construction of the granular core. The lateral extent of the excavation will be dependent on the footprint of the granular core discussed in Section 6.8.2. The sub-excavation in this area will be below the groundwater level as discussed further in Section 6.9.1.

At the North Abutment and approach, the excavations will consist of the removal of surficial organic material to about 0.8 m below existing ground surface.

The overburden soils requiring sub-excavation are considered Type 4 soils according to Occupational Health and Safety Act and Regulation for Construction Projects (OHSA). Excavation through these overburden soils should be carried out with side slopes no steeper than 3 horizontal to 1 vertical (3H:1V).



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

6.8.2 Granular Core / Pad

The placement of granular fill will be required below the South and North Abutments in order to provide a level surface on which to install piles and construct the pile caps. In these areas, it is recommended that the granular used be OPSS.PROV 1010 Granular 'B' Type II with a maximum particle size of 75 mm to facilitate pile installation.

At the South Abutment, the granular core below the abutment will be about 13.7 m thick based on an underside of the pile cap or footing at Elevation 193.6 m and the bottom of sub-excavation at Elevation 179.9 m.

At the North Abutment, the granular pad would only be about 0.6 m thick based on the underside of the pile cap at Elevation 190.1 m and bottom of sub-excavation at Elevation 189.5 m. However, it is recommended that the granular pad have a minimum thickness of 1 m.

The top of the granular pad should extend at least 3 m beyond the plan limits of the proposed footing/pile cap in all directions with side slopes no steeper than one horizontal to one vertical (1H:1V). The granular pad should be constructed in accordance with OPSS.PROV 206 (Grading) and compacted to at least 100 per cent of the material's standard Proctor maximum dry density (SPMDD). The granular fill should be constructed concurrently with the adjacent rock fill embankment to reduce the potential of differential settlement occurring and for constructability reasons (including facilitating compaction, inspection and testing). The rock fill protection surrounding the granular core / pad should be at least 1.5 m thick (measured perpendicular to the slope of the granular pad) at the level of the base of the pile cap.

6.8.3 Embankment Fill Placement

Placement of rock fill material for approach embankment construction should be carried out in accordance with the requirements as outlined in the OPSS.PROV 206 (Grading). The rock fill should not be dumped in final position, but should be deposited on and pushed forward over the end of the layer being constructed. Blading, dozing and 'chinking' the rock fill to form a dense, compact mass will be required to minimize voids and bridging. Side slopes for rock fill embankments should be no steeper than 1.25 horizontal to 1 vertical (1.25H:1V).

6.8.4 Embankment Platform Widening

In accordance with the requirements of MTO's Northern Region Engineering Directive NRE 98-200 "Northern Region Embankment Design Guidelines" (1998), the minimum required embankment widening at this site to account for the estimated post-construction settlement and for future pavement overlays is 2 m per embankment side.

6.9 Design and Construction Considerations

The following sections provide recommendations for control of groundwater and surface water, cofferdam construction, access to in-water piers and a brief discussion on obstructions and artesian conditions.



6.9.1 Control of Groundwater and Surface Water

At the South Abutment, the sub-excavation for removal of the organic layer(s) will extend to as deep as about Elevation 179.9 m, about 2.8 m below the groundwater level measured at Elevation 182.7 m in the piezometer in Borehole B502-07 in March 2014. The bottom of the sub-excavation at the south abutment is about 1.3 m above the ice/water surface in the adjacent Straight Lake as measured in March 2014, as such it may be possible for the groundwater in the excavation to be directed overland (by trenching) to lower areas to promote infiltration and seepage of the groundwater locally away from the south abutment foundation area. If not, the sub-excavation of the organic soils and replacement with Granular 'B' Type II fill will have to be carried out in-the-wet.

At Piers 1 to 3, foundation/pile cap construction in the lake will need be to be completed within a cofferdam as discussed further in Section 6.9.2.

Pier 4 is located on land on the north shore of the lake with the recommended underside of the pile cap at 1.8 m below final ground surface. The water level in the piezometer installed in Borehole B502-13 in March 2014 was at Elevation 182.1 m, about 0.4 m above the assumed underside of pile cap at Elevation 181.7 m. Provided the water level is at or near the base of the excavation at the time of excavation, it may be possible to construct the pile cap within an open excavation with temporary groundwater control provided by pumping from sumps. However, if the water level is above the base of the excavation, some form of shoring and dewatering will likely be required.

At the North Abutment, the underside of the pile cap is proposed at Elevation 190.1 m, well above the groundwater level in this area as measured at the time of the investigation, and as such, dewatering is not anticipated to be required at this location.

Surface water should be directed away from the excavations at all times.

6.9.2 Cofferdam Construction

The proposed underside of the pile caps at Piers 1 and 2 is below the lake surface and above the lake bed, while at Pier 3 the underside of the pile cap is below the lake surface and about 1.5 m below the lake bed. Construction of the pile caps for Piers 1 to 3 will require some form of cofferdam, such as the use of interlocking sheet piles driven through the water and into the overburden to a sufficient depth to form a water tight structure and maintain base stability of the excavation. Conventional sheet pile cofferdam construction at Piers 1 and 3 will be difficult because of the following challenges:

- Thin overburden and steeply sloping bedrock at Pier 1; the sheet piling would need to be keyed into the bedrock or affixed with toe-pins in order to achieve sufficient lateral fixity at the toe.
- Depth of water at Piers 1 and 2 (up to 4.0 m deep).
- Thickness of upper weak, very soft overburden (up to 4.8 m deep), underlain by thick soft to firm clay.
- Excavation and pier foundation pile installations would have to be carried out in-the-wet until a sufficiently thick/heavy tremie-plug is constructed at the base of the cofferdams otherwise there is a high risk that base heavy failure will occur during unwatering.
- The upper weak, very soft and compressible overburden (up to about 4.8 m thick) will likely compress and consolidate under the weight of a heavy concrete tremie plug (during curing). This could lead to complications in maintaining an adequate water-tight seal within the cofferdam. It could also result in drag loads forming on the pier piles.



Given the above, the use of conventional cofferdams for pier construction at Piers 1 to 3 would likely carry high costs and as well as high risks to successful completion. As such, it is recommended that consideration be given to using prefabricated cofferdam(s) for these piers, constructed with pre-drilled holes and steel tube sleeves through the base large enough to accommodate the foundation pile elements. These types of cofferdams could be floated and then anchored into place (at Piers 1 and 2) or lifted into place (at Pier 3), act as a template during pile installation and, upon completion of piling, could be backfilled with concrete to form the pile cap(s).

6.9.3 Access to In-Water Piers

At Piers 1 and 2, it is envisaged that access for installation of the drilled steel casings will be from a floating platform, however, the contractor will need to confirm that the depth of water at the time of construction is adequate for a floating platform to be used. At Pier 3, access will likely be from one of the following three alternatives:

- Pile supported temporary access structural platform extending from the north shore.
- Dredging the lake bed for positioning a floating platform (similar to that proposed for access to Piers 1 and 2), which may or may not be favourable or permissible from an environmental perspective.
- Temporary access embankment constructed with rock fill, however, this option is not recommended as the factor of safety is estimated to be about unity for stability (including anticipated equipment loading) and the fill will cause settlement of the organic silt and underlying clay, which will cause downdrag and impose drag loads on the pier piles. If this option is to be considered, the contractor would have to retain a geotechnical engineer to review the stability of the temporary embankment fill (including the requirements for staging the fill placement and monitoring during construction).

The selection, design and obtaining approvals (if necessary) of the most appropriate temporary access to the pier locations for construction will be the responsibility of the contractor, but the above factors should be taken into consideration.

6.9.4 Obstructions

The presence of cobbles and/or boulders was inferred from drilling resistance and confirmed by coring within the silt and sand to sand deposit in several boreholes advanced in the vicinity of the piers and at the south abutment. Given this condition, it is recommended that an Operational Constraint be included in the Contract Document to warn the Contractor of these obstructions and to ensure that the Contractor is equipped to handle such obstructions; an example Operational Constraint is included in Appendix D.

6.9.5 Artesian Conditions

At the SBL structure, artesian conditions were encountered inside the casing during drilling within the silt and sand to sand deposit in Boreholes B501-10 and B501-12 near Piers 2 and 3, respectively. The Contractor should be alerted to these artesian conditions with respect to pile installation at this site as such conditions may also present at locations along the NBL structure foundations; an example Operational Constraint is included in Appendix D.



7.0 CLOSURE

This report was prepared by Mr. André Bom, P.Eng., a senior geotechnical engineer and Associate of Golder. The technical aspects were reviewed by Mr. J. Paul Dittrich, Ph.D., P.Eng., a senior geotechnical engineer and Principal of Golder. Mr. Jorge M. A. Costa, P.Eng., Golder's Designated MTO Contact for this project and a Senior Consultant with Golder, conducted an independent quality control review of the report.

Report Signature Page

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- | | |
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| ASTM D1586 | Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils |
| ASTM D1587 | Standard Practice for Thin-Walled Tube Sampling of Soils for Geotechnical Purposes |
| ASTM D2573 | Standard Test Method for Field Vane Shear Test in Cohesive Soil |
| ASTM D5731 | Standard Test Method for Determination of the Point Load Strength Index of Rock and Application of Rock Strength Classification |



Commercial Software:

LPILE (Version 2013.7.07) by Ensoft Inc.

Settle3D (Version 2.0) by Rocscience Inc.

Slide (Version 6.0) by Rocscience Inc.

Ministry of Transportation Ontario:

Embankment Settlement Criteria for Design. July 2, 2010.

MTO Guideline for Rock Fill Settlement and Rock Fill Quantity Estimates. September 2010.

MTO Northern Region Pavement Practices and Guideline (1997) as amended by MTO Memorandum "Use of Mid-Slope Berms for Rock Fill Embankments" (2005).

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

Northeastern Region Engineering Directive. Backfill to Structures Adjacent to Rock Embankment Approaches. November 2002.

Ontario Occupational Health and Safety Act:

Ontario Regulation 213 Construction Projects (as amended)

Ontario Provincial Standard Drawings:

OPSD 202.010	Slope Flattening Using Surplus Excavated Material on Earth or Rock Embankment
OPSD 203.010	Embankments Over Swamp – New Construction
OPSD 3090.010	Foundation Frost Penetration Depths for Southern Ontario
OPSD 3101.150	Walls – Abutment, Backfill, Minimum Granular Requirement
OPSD 3101.200	Walls – Abutment, Backfill, Rock
OPSD 3102.100	Walls – Abutment, Backfill, Drain
OPSD 3190.100	Walls – Retaining and Abutment, Wall Drain
OPSD 3121.150	Walls – Retaining, Backfill, Minimum Granular Requirement

Ontario Provincial Standard Specifications:

OPSS. PROV 206	Construction Specification for Grading
OPSS.PROV 501	Construction Specification for Compacting
OPSS.PROV 1010	Material Specification for Aggregates – Base, Subbase, Select Subgrade, and Backfill Material

Ontario Water Resources Act:

Ontario Regulation 903 Wells (as amended)



Table 1: Evaluation of Foundation Alternatives – South Abutment

Foundation Option	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Steel H-piles Driven through Compacted Granular Core to Refusal on Bedrock	1	<ul style="list-style-type: none">■ Negligible post-construction settlement.■ Loading from abutment foundation does not affect stability of high approach embankment.■ Fully integral abutment design achievable.	<ul style="list-style-type: none">■ Sub-excavation of up to about 6 m of upper very loose to loose sandy silt to sand to organic overburden soils recommended prior to construction of granular core to minimize settlements and avoid drag loads on piles.■ Minimum 14 m thick granular core required (below pile cap).	<ul style="list-style-type: none">■ Higher relative cost than spread footings due to additional costs for piling.	<ul style="list-style-type: none">■ Compacted granular pad must extend laterally at least 10 pile diameters (3 m) in all directions around the pile cap.■ Risk of rear (south) row of piles sliding on sloping bedrock.■ Potential for variable pile lengths due to variable depth to bedrock.
Spread Footing on Compacted Granular Pad over Bedrock	2	<ul style="list-style-type: none">■ Relative ease of construction.■ Negligible post-construction settlement.	<ul style="list-style-type: none">■ Sub-excavation of up to about 6 m of upper very loose to loose sandy silt to sand to organic overburden soils required prior to construction of granular pad.■ Fully integral abutment design not achievable.■ Lower factored geotechnical axial resistance at ULS for spread footing on thick compacted granular pad within high approach fill slope.■ Higher level of quality control required for placement and compaction of granular pad for support of spread footing compared with granular core placed for steel H-piles / pile cap support.■ Requires laboratory testing of proposed granular materials to be used for pad construction to verify/confirm that adequate angle of internal friction is achieved (for stability considerations).	<ul style="list-style-type: none">■ Lower relative cost than piled foundation option.	<ul style="list-style-type: none">■ Review of sub-excavation to be completed by geotechnical engineer to confirm adequate depth of removal of soft/loose soil to minimize risk of settlement of granular pad and footing.■ Granular pad should be comprised of Granular 'A' (upper pad to a depth of 1.5B below base of footing) and Granular 'B' Type II (lower pad).
Drilled Steel Casings (0.6 m Ø) socketed into bedrock using DTH hammer drilling	3	<ul style="list-style-type: none">■ Higher axial and lateral capacity than small diameter pile elements.■ Smaller number of pile elements required per foundation element as compared with driven steel H-.■ DTH hammer drilling method offers best chance of seating casings on very strong and sloping bedrock and creating rock socket; however, careful drilling practices required.■ Consistent foundation design and construction with Piers 1 to 4 results in single mobilization of specialty contractor.	<ul style="list-style-type: none">■ Permanent steel casings required during construction to control potential ground losses in overburden soils.■ Requires specialty contractor to install casings.■ Construction staging need to be considered in design.	<ul style="list-style-type: none">■ Higher relative cost than driven piles.■ Smaller number of pile elements required which may result in some cost savings.	<ul style="list-style-type: none">■ If casings not adequately sealed, and the base not properly cleaned, there is a potential of debris and materials impeding rock socket construction.■ Potential for variable pile lengths due to variable depth to bedrock.

Prepared By: AB
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Table 2: Evaluation of Foundation Alternatives – Pier 1

Foundation Option	Rank	Advantages	Disadvantages	Relative Costs	Risks / Consequences
Drilled Steel Casings (0.6 m Ø) socketed into bedrock using DTH hammer drilling	1	<ul style="list-style-type: none">Higher axial and lateral capacity than small diameter pile elements.Smaller number of pile elements required as compared with driven steel H-piles or tube piles.DTH hammer drilling method offers best chance of seating casings on very strong and sloping bedrock and creating rock socket; however, careful drilling practices required.Consistent foundation design and construction with piers for nearby Key River structures results in single mobilization of specialty contractor.	<ul style="list-style-type: none">Requirement for larger drill rig set-up on barge in lake may make over-water construction difficult.Permanent steel casings required during construction to control potential ground losses in overburden soils.Requires specialty contractor to install piles.	<ul style="list-style-type: none">Higher cost per pile than driven piles and smaller diameter drilled steel casings and/or micropiles due to more complex installation, but potentially fewer piles required as a result of higher capacity.Smaller number of pile elements may result in some cost savings.	<ul style="list-style-type: none">Challenging subsurface conditions resulting in more difficult construction and potential difficulties seating larger diameter steel casings on sloping, strong to very strong bedrock and drilling rock sockets which could increase costs and potentially affect schedule.If casings not adequately sealed, and the base not properly cleaned, there is a potential of debris and materials impeding rock socket construction.High potential for variable pile lengths due to variable depth to bedrock.
Steel H-piles (HP310x110) or concrete filled Steel Tube Piles (300 mm Ø) driven to refusal on bedrock	NF	<ul style="list-style-type: none">Relatively straight forward construction.	<ul style="list-style-type: none">Highly variable overburden thickness/depth to bedrock may result in some piles having insufficient length to support piers (since piles are not socketed into bedrock).May be difficult to seat driven piles on sloping (up to 50° or more), strong to very strong bedrock (even with rock points) considering the presence of the weak, thin overburden.Weak and thin overburden soils will result in insufficient lateral resistance (since piles are small diameter and not socketed into bedrock).	<ul style="list-style-type: none">Lower cost per pile than drilled steel casings or micropiles.	<ul style="list-style-type: none">Difficulties seating driven piles on sloping, strong to very strong bedrock which will raise costs and potentially affect schedule.High potential for variable pile lengths due to variable depth to bedrock.Insufficient lateral resistance offered by small diameter pile elements.
Steel H-piles (HP310x110) fixed into bedrock sockets backfilled with concrete	NF	<ul style="list-style-type: none">Fixing base of steel piles into bedrock sockets provides improved seating conditions over driven pile options.No concern over near shore piles having insufficient length since piles seated into bedrock (socket length can be increased if necessary).DTH hammer drilling method offers best chance of seating temporary casings on very strong and sloping bedrock and creating rock socket; however, careful drilling practices required.	<ul style="list-style-type: none">Installation of battered piles somewhat more difficult.Requirement for larger drill rig and equipment set-up on barge in lake for socket construction may make over-water construction difficult.Requires specialty contractor to drill rock sockets prior to installation of steel piles.Weak and thin overburden soils (combined with disturbance after removal of temporary liner after construction of rock socket) will result in insufficient lateral resistance relying entirely on bedrock sockets for lateral resistance.Large pile driving rig required for installation makes over-water work difficult.	<ul style="list-style-type: none">High cost per pile given that drilling of rock socket (with temporary liner) required before installation of steel pile.Higher overall cost given that more pile elements likely required to satisfy lateral resistances due to weak/thin overburden and due to disturbances caused during installation.	<ul style="list-style-type: none">High potential for variable pile lengths due to variable depth to bedrock.Difficult construction and subsurface conditions and potential for difficulties seating larger diameter steel casings on sloping, strong to very strong bedrock and drilling rock sockets which could raise costs and potentially affect schedule.Insufficient lateral resistance offered by small diameter pile elements; larger number of pile elements likely required.
Small Diameter Drilled Steel Casings (0.406 m Ø) or Micropiles (0.273 m Ø) socketed into bedrock using DTH drilling;	NF	<ul style="list-style-type: none">Relatively straight forward construction.Smaller diameter casings may be installed with relatively smaller drilling equipment (making it easier for over-water work).Small diameter micropiles drilled using DTH offers best chance of seating and socketing piles in sloping, strong to very strong bedrock.	<ul style="list-style-type: none">Smaller axial capacity than large diameter drilled steel casings (limited by structural design); larger number of piles required.Weak and thin overburden soils in combination with smaller diameter of pile elements will result in insufficient lateral resistance; lateral resistance will be derived entirely from fixity in bedrock sockets resulting in smaller lateral capacity than for large diameter drilled steel casings.Requires specialty contractor to install piles.	<ul style="list-style-type: none">Higher relative cost than driven piles.Lower cost per pile than larger diameter drilled steel casings, but more piles may be required as a result of lower individual axial and lateral capacity.	<ul style="list-style-type: none">High potential for variable pile lengths due to variable depth to bedrock.Insufficient lateral resistance offered by small diameter pile elements.



Table 2: Evaluation of Foundation Alternatives – Pier 1

Foundation Option	Rank	Advantages	Disadvantages	Relative Costs	Risks / Consequences
Concrete Caissons (Drilled Shafts) with permanent steel liners (0.9 m Ø) socketed into bedrock	NF	<ul style="list-style-type: none">■ High axial capacity.■ High lateral capacity.■ Smaller number of pile elements required per pier.	<ul style="list-style-type: none">■ Presence of very strong to extremely strong sloping bedrock (up to about 50° or more) will make socketing large diameter steel liners into bedrock very difficult.■ Cannot be easily battered.■ Requirement for large drill rig set-up on barge in lake will make over-water construction difficult.■ The larger the caisson diameter the greater the likelihood of encountering difficulties to drill the caisson into the sloping surface and very strong bedrock.	<ul style="list-style-type: none">■ Difficult construction conditions (i.e., large caisson rig on barge in lake) may result in higher equipment / mobilization cost than drilled steel casing alternatives.■ Smaller number of pile elements may result in some cost savings, but likely offset by higher costs per single pile element.	<ul style="list-style-type: none">■ Potential for variable caisson lengths due to variable depth to bedrock.■ Potentially difficult construction conditions and difficulties seating steel liners on steeply sloping and very strong to extremely strong bedrock and drilling rock sockets which will raise costs and potentially affect schedule.■ If liners not adequately sealed, and the caisson base not properly cleaned, debris and materials will impede rock socket construction and concrete placed by tremie methods.■ Large diameter drilled shafts would require the use of large equipment installation. The use of large equipment would pose additional constructability challenges due to working over the water.
Composite Foundation Elements (Large diameter Drilled Shaft combined with smaller diameter pile elements socketed into bedrock)	NF	<ul style="list-style-type: none">■ Ease of socketing smaller diameter pile elements (i.e., micropiles) into the very strong and sloping bedrock.■ Large diameter drilled shaft provides stiffer section through water column and increased lateral resistance in upper very soft organic soil stratum.	<ul style="list-style-type: none">■ Higher lateral loads on fewer, large diameter drilled shafts; given that the overburden along the length of the drilled shafts offers low lateral resistance, lateral forces from pile cap expected to transfer along full depth of piles resulting in transfer of lateral loads across small diameter pile elements socketed into bedrock.■ Designing the small diameter pile elements (i.e., micropiles) to transfer the high lateral loads and/or bending moments from the drilled shaft to the bedrock would be challenging.■ Requirement for large drill rig set-up on barge in lake to advance outer liners/casings will make over-water construction difficult.	<ul style="list-style-type: none">■ Probably highest cost per pile element given size and considering that composite pile requires construction of two different pile types per element.■ Additional costs for mobilizing two different types of equipment (over water) to install different pile types.■ Although a potentially smaller number of pile elements may result in some cost savings, total costs are expected to be higher than drilled steel casings option.	<ul style="list-style-type: none">■ Large diameter drilled shafts (even as composite foundation elements) would require the use of large equipment for installation. The use of large equipment would pose additional constructability challenges due to working over the water.■ Design of small diameter pile elements to transfer high lateral loads and/or bending moments from upper large diameter pile will be challenging.
Shallow Spread Footings on overburden in lakebed	NF		<ul style="list-style-type: none">■ Presence of loose/soft overburden soils and significant depth to competent founding stratum will result in very low geotechnical resistance(s).■ Significant settlement expected to occur.		<ul style="list-style-type: none">■ Not feasible due to presence of very weak and compressible overburden soils.
Spread Footings on bedrock	NR	<ul style="list-style-type: none">■ High axial capacity for footings founded on granite gneiss bedrock.	<ul style="list-style-type: none">■ Presence of variable thickness overburden and bedrock at variable depth will make construction of cofferdam/cut-off for in-the-dry footing construction very difficult.	<ul style="list-style-type: none">■ Very high cost associated with construction of cofferdam in water and in challenging subsurface conditions.	<ul style="list-style-type: none">■ Not recommended due to variable thickness overburden and variable depth to bedrock making sealing of cofferdam for in-the-dry construction very difficult.

NF: Foundation option is not feasible.
NR: Foundation option is not recommended.
DTH: Down-The-Hole (hammer-type for pile installation).

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Reviewed By: JMAC



Table 3: Evaluation of Foundation Alternatives – Piers 2 to 4

Foundation Option	Rank	Advantages	Disadvantages	Relative Costs	Risks / Consequences
Drilled Steel Casings (0.6 m Ø) socketed into bedrock using DTH hammer drilling	1	<ul style="list-style-type: none">■ Higher axial and lateral capacity than small diameter pile elements.■ Smaller number of pile elements required per pier as compared with driven steel H-piles or tube piles.■ DTH hammer drilling method offers best chance of seating casings on very strong and sloping bedrock and creating rock socket; however, careful drilling practices required.■ Consistent foundation design and construction with Pier 1 results in single mobilization of specialty contractor.	<ul style="list-style-type: none">■ Requirement for larger drill rig set-up on barge in lake may make over-water construction difficult.■ Permanent steel casings required during construction to control potential ground losses in overburden soils.■ Requires specialty contractor to install piles.	<ul style="list-style-type: none">■ Higher cost per pile than driven piles due to more complex installation, but potentially fewer piles required as a result of higher capacity.■ Smaller number of pile elements may result in some cost savings.	<ul style="list-style-type: none">■ Some risk of difficulties drilling rock sockets on sloping and strong to very strong bedrock.■ If casings not adequately sealed, and the base not properly cleaned, there is a potential of debris and materials impeding rock socket construction■ Some potential for variable pile lengths due to variable depth to bedrock.
Steel H-piles (HP310x110 or larger) or concrete filled Steel Tube Piles (300 mm Ø) driven to refusal on bedrock	NF at Piers 2 and 3 and NR at Pier 4	<ul style="list-style-type: none">■ Relatively straightforward construction.	<ul style="list-style-type: none">■ Presence of very soft/loose overburden soils does not provide adequate lateral resistance per pile resulting in inefficient pile group design, especially at Piers 2 and 3.■ Potential for encountering obstructions (cobbles and boulders) during pile driving; may require heavier pile section and/or pile shoes/flange stiffeners; pile axial capacity reduced due to potential for seating on cobbles/boulders instead of bedrock.■ Potential for difficulties seating driven piles on sloping, strong to very strong bedrock; rock points will be required, especially at Pier 3.■ Long pile lengths (i.e., about 45 m at Pier 2); may require heavier pile section for greater stiffness.■ Although H-pile foundations are feasible at Pier 4, it would be inconsistent with drilled steel casing foundations recommended at Piers 1 to 3.	<ul style="list-style-type: none">■ Lower relative cost per pile than drilled steel casings.	<ul style="list-style-type: none">■ Difficulties seating piles on strong to very strong, sloping bedrock and/or on cobbles and boulders. Rock points likely required and reduced axial pile capacity.■ Insufficient lateral resistance of smaller diameter pile elements in weak/very soft overburden soils.
Composite Foundation Elements (Large diameter Drilled Shaft combined with smaller diameter pile elements socketed into bedrock)	NF	<ul style="list-style-type: none">■ Ease of socketing smaller diameter pile elements (i.e., micropiles) into the very strong and sloping bedrock.■ Large diameter drilled shaft provides stiffer section through water column and increased lateral resistance in upper very soft organic soil stratum.	<ul style="list-style-type: none">■ Drilling with mud/slurry required to balance pressures when advancing and cleaning out large diameter liners/casings otherwise base instability/heave may occur in particular where artesian groundwater conditions encountered.■ Cobbles and boulders within overburden may impede advance and clean-out of large diameter liners/casings.■ Detailed soil-structure analysis required to determine number of smaller diameter pile elements for proper transfer of loads.■ Requirement for large drill rig set-up on barge in lake to advance outer liners/casings will make over-water construction difficult.	<ul style="list-style-type: none">■ Probably highest cost per pile element given size and considering that composite pile requires construction of two different pile types per element.■ Additional costs for mobilizing two different types of equipment (over water) to install different pile types.■ Although a potentially smaller number of pile elements may result in some cost savings, total costs are expected to be higher than drilled steel casings option.	<ul style="list-style-type: none">■ Large diameter drilled shafts (even as composite foundation elements) would require the use of large equipment for installation. The use of large equipment would pose additional constructability challenges due to working over the water.■ Risk of base instability/heave when drilling/cleaning out large diameter liners/casings in overburden if pressures not balanced.■ Risk of obstructions (cobbles and boulders) impeding advance of large diameter liners/casings.



Table 3: Evaluation of Foundation Alternatives – Piers 2 to 4

Foundation Option	Rank	Advantages	Disadvantages	Relative Costs	Risks / Consequences
Concrete Caissons (Drilled Shafts) with permanent steel liners (0.9 m Ø) socketed into bedrock	NF	<ul style="list-style-type: none">■ High axial capacity.■ High lateral capacity.■ Smaller number of pile elements required per pier.	<ul style="list-style-type: none">■ Drilling with mud/slurry required to balance pressures when advancing and cleaning out large diameter liners/casings otherwise base instability/heave may occur in particular where artesian groundwater conditions encountered.■ Cobbles and boulders within overburden may impede advance and clean-out of large diameter liners/casings.■ Presence of very strong to extremely strong sloping bedrock will make socketing large diameter steel liners into bedrock very difficult.■ Cannot be easily battered.■ Requirement for large drill rig set-up on barge in lake will make over-water construction difficult.■ The larger the caisson diameter the greater the likelihood of encountering difficulties to drill the caisson into the sloping surface and very strong bedrock.	<ul style="list-style-type: none">■ Difficult construction conditions (i.e., large caisson rig on barge in lake) may result in higher equipment / mobilization cost than drilled steel casing alternatives.■ Smaller number of pile elements may result in some cost savings, but likely offset by higher costs per single pile element.	<ul style="list-style-type: none">■ Risk of obstructions (cobbles and boulders) impeding advance of large diameter liners/casings.■ Risk of base instability/heave when drilling/cleaning out large diameter liners/casings in overburden if pressures not balanced.■ Potentially difficult construction conditions and difficulties seating steel liners on steeply sloping and very strong to extremely strong bedrock and drilling rock sockets which will raise costs and potentially affect schedule.■ If liners not adequately sealed, and the caisson base not properly cleaned, debris and materials will impede rock socket construction and concrete placed by tremie methods.■ Large diameter drilled shafts would require the use of large equipment installation. The use of large equipment would pose additional constructability challenges due to working over the water.
Shallow Spread Footings founded on overburden in lakebed	NF		<ul style="list-style-type: none">■ Presence of loose/soft overburden soils and significant depth to competent founding stratum will result in very low geotechnical resistance(s).■ Significant settlement expected to occur.■ Cofferdam and dewatering required to allow for excavations to be taken to the required founding level.		<ul style="list-style-type: none">■ Not feasible due to presence of very weak and compressible overburden soils.

NF: Foundation option is not feasible.
NR: Foundation option is not recommended.
DTH: Down-The-Hole (hammer-type for pile installation).

Prepared By: AB
Checked By: JPD
Reviewed By: JMAC



Table 4: Evaluation of Foundation Alternatives –North Abutment

Foundation Option	Rank	Advantages	Disadvantages	Relative Costs	Risks / Consequences
Steel H-piles (HP310x110) or concrete filled Steel Tube Piles (300 mm Ø) driven to refusal on bedrock	1	<ul style="list-style-type: none">■ Relatively straight forward construction.■ Consistent foundation design with south abutment.	<ul style="list-style-type: none">■ Rock points may be required to ensure proper seating of piles on potentially sloping, strong to very strong bedrock.■ Drag loads and/or appropriate north approach embankment construction staging need to be considered in design.	<ul style="list-style-type: none">■ Lower relative cost per pile than drilled steel casings.	<ul style="list-style-type: none">■ Difficulties seating piles on strong to very strong, sloping bedrock; rock points may be required.■ Some potential for variable pile lengths due to variable depth to bedrock.
Drilled Steel Casings (0.6 m Ø) socketed into bedrock using DTH hammer drilling	2	<ul style="list-style-type: none">■ Higher axial and lateral capacity than small diameter pile elements.■ Smaller number of pile elements required per foundation element as compared with driven steel H-piles or tube piles.■ DTH hammer drilling method offers best chance of seating casings on very strong and sloping bedrock and creating rock socket; however, careful drilling practices required.■ Consistent foundation design and construction with Piers 1 to 4.	<ul style="list-style-type: none">■ Permanent steel casings recommended during construction to control potential ground losses in overburden soils.■ Requires specialty contractor to install piles.■ Dragloads and/or appropriate north approach embankment construction staging need to be considered in design.	<ul style="list-style-type: none">■ Higher relative cost than driven piles.■ Smaller number of pile elements may result in some cost savings.	<ul style="list-style-type: none">■ If casings not adequately sealed, and the base not properly cleaned, there is a potential of debris and materials impeding rock socket construction.■ Some potential for variable pile lengths due to variable depth to bedrock.
Shallow Spread Footings on engineered granular pad founded on overburden	NF	<ul style="list-style-type: none">■ Relatively straightforward footing construction.	<ul style="list-style-type: none">■ Presence of very loose to loose overburden soils of significant thickness results in very low geotechnical resistance(s), in particular for SLS.■ Settlement(s) greater than 25 mm expected for geotechnical resistance at SLS approaching the values required for abutment foundation design.■ Global stability check of front approach embankment slope (at North Abutment) combined with loading from north abutment foundation results in FoS<1.3 for geotechnical resistance at SLS approaching the values required for abutment foundation design.	<ul style="list-style-type: none">■ Lower relative cost than deep foundations.	<ul style="list-style-type: none">■ Not feasible due to low SLS resistance available at North Abutment as well as FoS<1.3 for global stability of front slope of approach embankment when considering loading from shallow foundation.

NF: Foundation option is not feasible
DTH: Down-The-Hole (hammer-type for pile installation)

Prepared By: AB
Checked By: JPD
Reviewed By: JMAC



Table 5: Geotechnical Parameters for Lateral Pile Analysis at South Abutment – Stratigraphy based on Boreholes B502-06

Overburden Stratum	Top/ Bottom of Stratum Elevations (m)	Employed Model for Lateral Analysis	Bulk* Unit Weight, γ_b (kN/m ³)	Undrained Shear Strength, s_u (kPa)	Effective Angle of Internal Friction, ϕ (Deg)	Strain at one-half the Maximum Difference in Principal Stresses, ϵ_{50}	Initial Modulus of Subgrade Reaction, k_{py} ($=n_1$) (kPa/m)	Elastic Modulus, E' (kPa)	Shear Modulus, G (kPa)	Poisson's Ratio, ν'
Granular Core/Pad	Underside of Abutment (193.6) – 181.7	API Sand (O'Neill)	21	-	35	-	22,500	50,000	19,200	0.3
Compact to Dense Sand and Gravel	181.7 – 178.1	API Sand (O'Neill)	22	-	34	-	19,200	50,000	19,200	0.3



Table 6: Geotechnical Parameters for Lateral Pile Analysis at Pier 1 – Stratigraphy based on Boreholes B502-01 and B502-10

Overburden Stratum	Top/ Bottom of Stratum Elevations (m)	Employed Model for Lateral Analysis	Bulk* Unit Weight, γ_b (kN/m ³)	Undrained Shear Strength, s_u (kPa)	Effective Angle of Internal Friction, ϕ (Deg)	Strain at one-half the Maximum Difference in Principal Stresses, ϵ_{50}	Initial Modulus of Subgrade Reaction, k_{py} ($=n_1$) (KPa/m)	Elastic Modulus, E' (kPa)	Shear Modulus, G (kPa)	Poisson's Ratio, ν'
Water	Underside of Pier (175.7) - 174.6	-	-	-	-	-	-	-	-	-
Very soft Organic Silt	174.6 – 170	API Soft Clay (Matlock)	14	10	-	0.04	-	1,000	330	0.5
Very Loose Silt	170 – 169	API Sand (O'Neill)	18	-	28	-	1,300	1,500	550	0.35
Soft to Firm Clayey Silt	169 – 164	API Soft Clay (Matlock)	17	20	-	0.02	-	2,000	660	0.5
Loose Silt	164 – 157	API Sand (O'Neill)	18	-	29	-	2,200	3,000	1,100	0.35
Rock Type	Elevation (m)	Employed Model for Lateral Analysis	Bulk Unit Weight, γ_b (kN/m ³)	Uniaxial Comp. Strength (MPa)	Rock Mass Modulus (GPa)	Modulus Ratio	RQD	-	-	Poisson's Ratio, ν'
Granitic Gneiss	Below 157	User Defined (after Glick)	26	100	31	0.7	90%	-	-	0.25



Table 7: Geotechnical Parameters for Lateral Pile Analysis at Piers 2 and 3 – Stratigraphy based on Boreholes B502-10 and B502-11

Overburden Stratum	Top/ Bottom of Stratum Elevations (m)	Employed Model for Lateral Analysis	Bulk* Unit Weight, γ_b (kN/m ³)	Undrained Shear Strength, s_u (kPa)	Effective Angle of Internal Friction, ϕ (Deg)	Strain at one-half the Maximum Difference in Principal Stresses, ϵ_{50}	Initial Modulus of Subgrade Reaction, k_{py} ($=n_h$) (KPa/m)	Elastic Modulus, E' (kPa)	Shear Modulus, G (kPa)	Poisson's Ratio, ν'
Water	Underside of Pier (175.7) – 175	-	-	-	-	-	-	-	-	-
Very Soft Organic Silt	175 – 171	API Soft Clay (Matlock)	14	10	-	0.04	-	1,000	330	0.5
Soft to Firm Silty Clay	171 – 167.5	API Soft Clay (Matlock)	17	20	-	0.02	-	2,000	660	0.5
Loose to Compact Silt and Sand to Sandy Silt	167.5 - 166	API Sand (O'Neill)	18	-	29	-	2,200	3,000	1,100	0.35
Soft to Firm Clayey Silt to Silty Clay	166 – 163.5	API Soft Clay (Matlock)	17	40	-	0.01	-	8,000	3,000	0.35
Loose to Compact Silty Sand	163.5 – 162.5	API Sand (O'Neill)	18	-	29	-	2,200	3,000	1,100	0.35
Firm to Stiff Clay	162.5 – 157.5	API Soft Clay (Matlock)	18	50	-	0.007	-	10,000	3,700	0.35
Loose to Compact Silt to Sand	157.5 – 135 (Pier 2) and 153 (Pier 3)	API Sand (O'Neil)	20	-	32	-	14,000	10,000	3,700	0.35
Rock Type	Elevation (m)	Employed Model for Lateral Analysis	Bulk Unit Weight, γ_b (kN/m ³)	Uniaxial Comp. Strength (MPa)	Rock Mass Modulus (GPa)	Modulus Ratio	RQD	-	-	Poisson's Ratio, ν'
Granitic Gneiss	Below 135 (Pier 2) 153 (Pier 3)	User Defined (after Glick)	26	100	26	0.7	75%	-	-	0.25



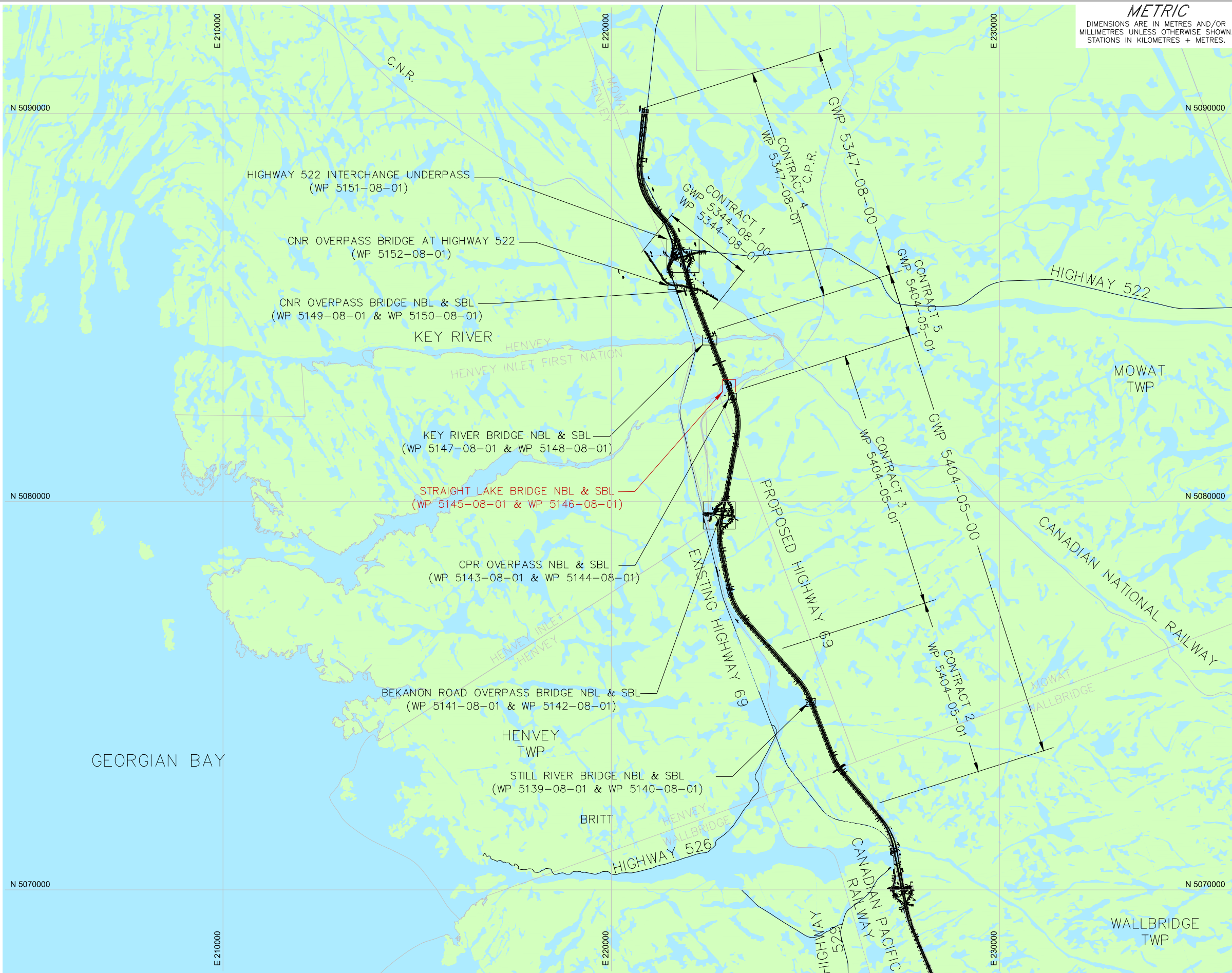
Table 8: Geotechnical Parameters for Lateral Pile Analysis at Pier 4 – Stratigraphy based on Borehole B502-13

Overburden Stratum	Top/ Bottom of Stratum Elevations (m)	Employed Model for Lateral Analysis	Bulk* Unit Weight, γ_b (kN/m ³)	Undrained Shear Strength, s_u (kPa)	Effective Angle of Internal Friction, ϕ (Deg)	Strain at one-half the Maximum Difference in Principal Stresses, ϵ_{50}	Initial Modulus of Subgrade Reaction, k_{py} (=n _h) (kPa/m)	Elastic Modulus, E' (kPa)	Shear Modulus, G (kPa)	Poisson's Ratio, ν'
Very Loose to Compact Silt to Sand	Underside of Pier (181.7) – 178.0	API Sand (O'Neill)	19	-	30	-	5,000	5,000	1,850	0.35
Firm Clayey Silt to Silty Clay	178.0 – 173.0	API Soft Clay (Matlock)	17	35	-	0.01	-	5,000	1,850	0.35
Loose to Compact Silt	173.0 – 168.7	API Sand (O'Neill)	18	-	29	-	2,200	3,000	1,100	0.35
Rock Type	Elevation (m)	Employed Model for Lateral Analysis	Bulk Unit Weight, γ_b (kN/m ³)	Uniaxial Comp. Strength (MPa)	Rock Mass Modulus (GPa)	Modulus Ratio	RQD	-	-	Poisson's Ratio, ν'
Granitic Gneiss	Below 168.7	User Defined (after Glick)	26	100	31	0.7	90%	-	-	0.25



Table 9: Geotechnical Parameters for Lateral Pile Analysis at North Abutment – Stratigraphy based on Borehole S502-24

Overburden Stratum	Top/ Bottom of Stratum Elevations (m)	Employed Model for Lateral Analysis	Bulk* Unit Weight, γ_b (kN/m ³)	Undrained Shear Strength, s_u (kPa)	Effective Angle of Internal Friction, ϕ (Deg)	Strain at one-half the Maximum Difference in Principal Stresses, ϵ_{50}	Initial Modulus of Subgrade Reaction, k_{py} ($=n_1$) (KPa/m)	Elastic Modulus, E' (kPa)	Shear Modulus, G (kPa)	Poisson's Ratio, ν'
Granular Core/Pad	Underside of Abutment (190.1) – 189.5	API Sand (O'Neill)	21	-	35	-	22,500	50,000	19,200	0.30
Very Loose to Compact Sand to Silt and Sand	189.5 – 178.6	API Sand (O'Neill)	19	-	30	-	12,000 (Above WL) 9,000 (Below WL)	7,500	2,900	0.3
Firm Clay	178.6 – 176.8	API Soft Clay (Matlock)	17	25	-	0.015	-	2,500	830	0.5
Very Loose to Loose Silt to Silt and Sand	176.8 – 172.6	API Sand (O'Neill)	18	-	28	-	1,300	1,500	550	0.35
Rock Type	Elevation (m)	Employed Model for Lateral Analysis	Bulk Unit Weight, γ_b (kN/m ³)	Uniaxial Comp. Strength (MPa)	Rock Mass Modulus (GPa)	Modulus Ratio	RQD	-	-	Poisson's Ratio, ν'
Granitic Gneiss (Assumed)	Below 172.6	User Defined (after Glick)	26	100	26	0.7	75%	-	-	0.25



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CONT No. .
WP No. 5145-08-01

HIGHWAY 69
STRAIGHT LAKE NBL BRIDGE
INDEX PLAN



KEY PLAN
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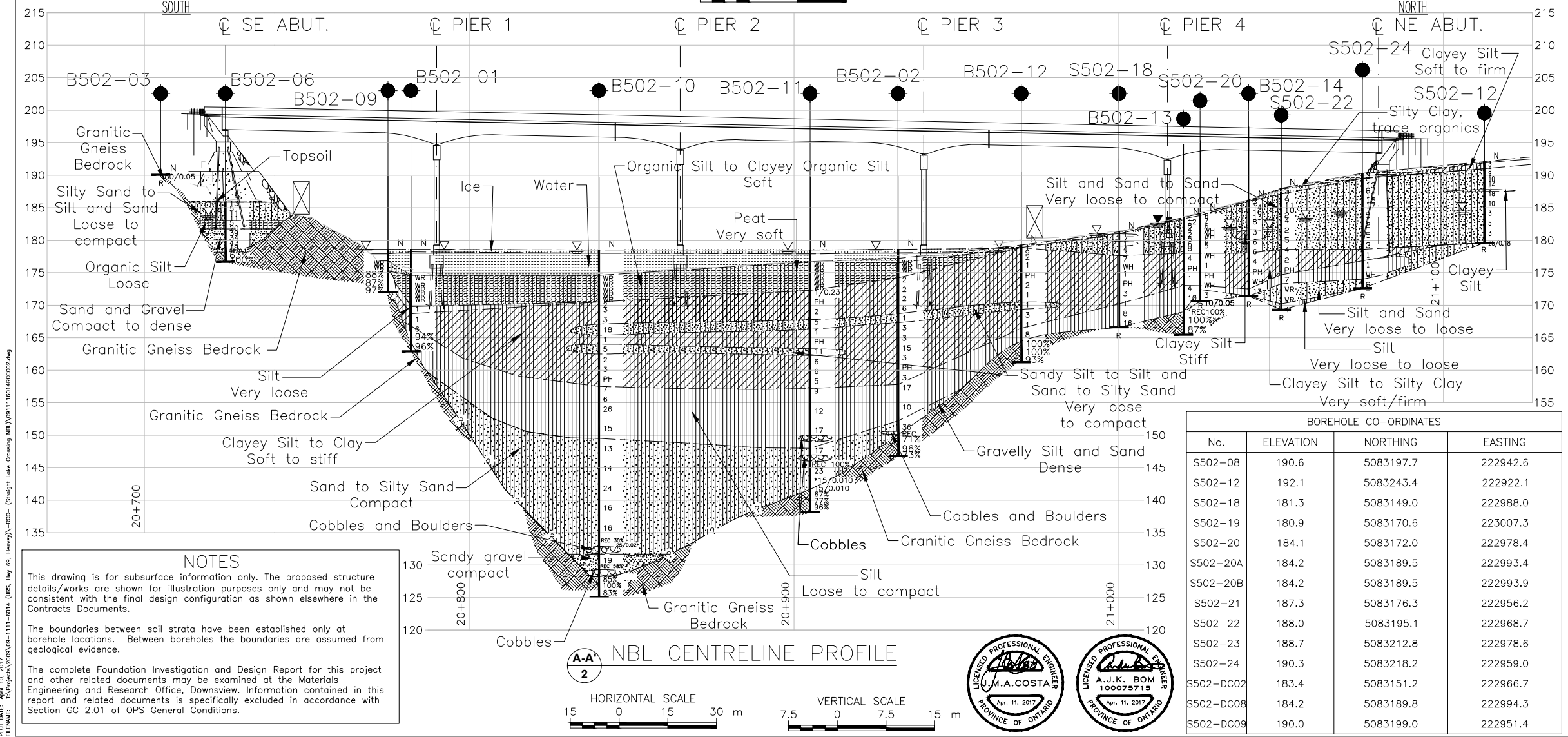
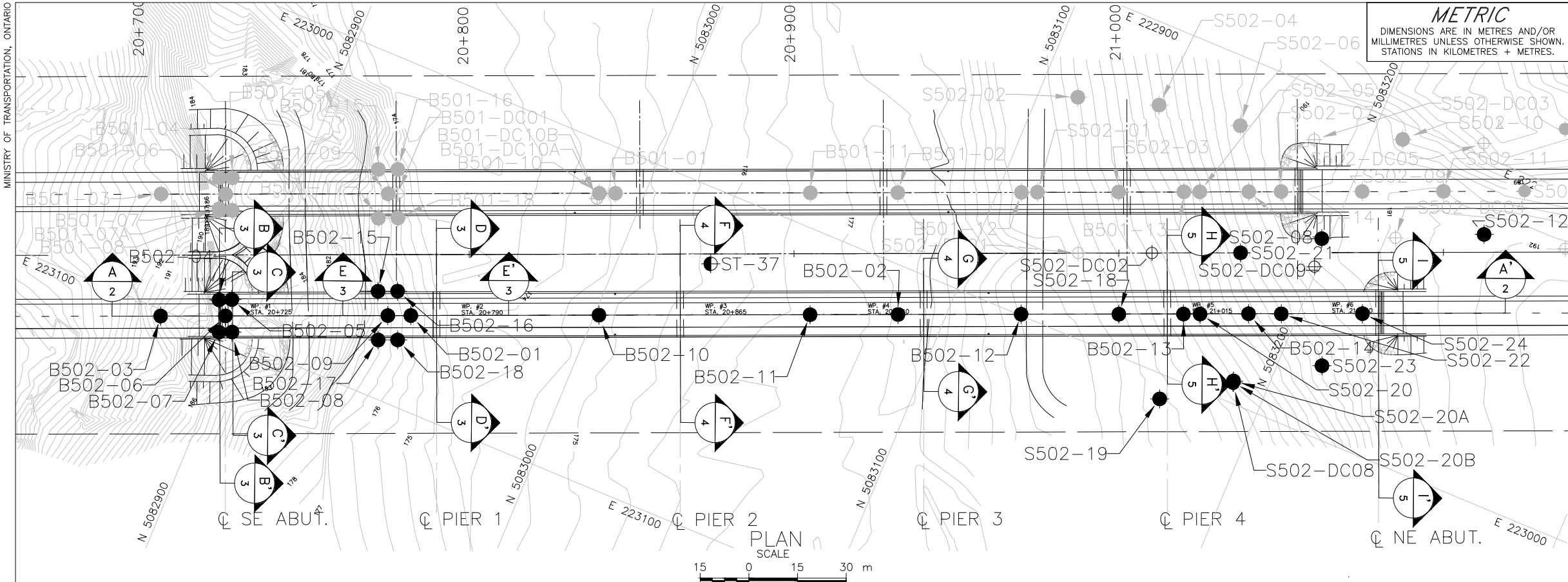
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Base Data - MNR NRVIS, obtained 2004, CANMAP v2008
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Datum : NAD 83 Projection : MTM Zone 10

NO.	DATE	BY	REVISION
Geocres No. 41H-165			
HWY. 69	PROJECT NO. 09-1111-6014		DIST. .
SUBM'D. MCK	CHKD. MCK	DATE: July 2015	SITE: 44-461/1
DRAWN: JFC	CHKD. AB	APPD. JMAC	DWG. 1



DATE: April 13, 2017
DRAWN: JFC/MR
CHECKED: AB
PROJECT NO. 09-1111-6014 (URS, Hwy 69, Henney) - RCD - (Straight Lake Crossing NBL) (091111601460202.dwg)

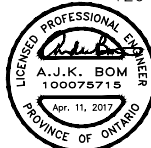
NOTES

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

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

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

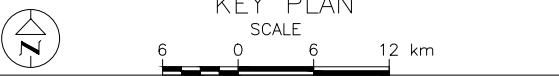
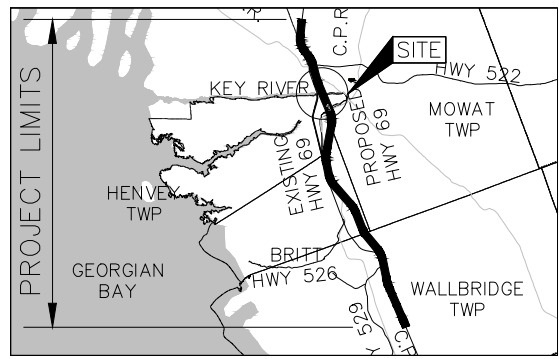
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CONT No. _____
WP No. 5145-08-01

HIGHWAY 69
STRAIGHT LAKE NBL BRIDGE
BOREHOLE LOCATIONS AND
SOIL STRATA



LEGEND

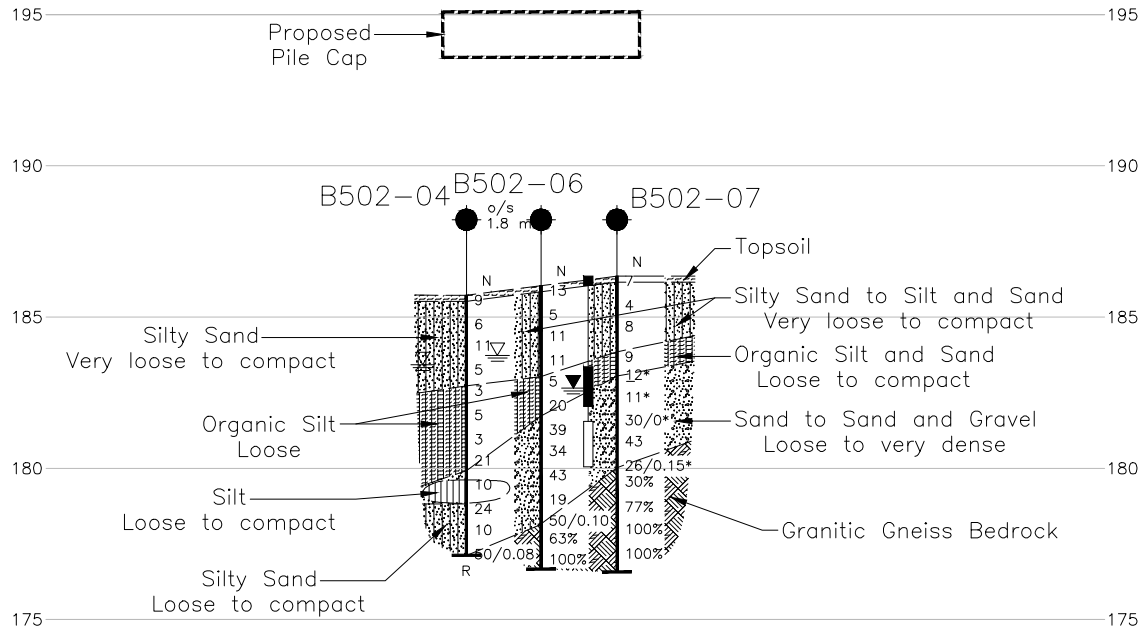
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- ⊕ Dynamic Cone Penetration Test
- Borehole - Previous Investigation (AMEC)
- N Standard Penetration Test Value
- R Refusal
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- 100% Rock Quality Designation (RQD)
- WL in piezometer (March 8/14)
- WL upon completion of drilling

BOREHOLE CO-ORDINATES			
No.	ELEVATION	NORTHING	EASTING
B502-01	178.6	5082947.9	223072.2
B502-02	178.6	5083086.2	223014.2
B502-03	190.2	5082876.8	223101.9
B502-04	185.7	5082891.5	223090.4
B502-05	185.7	5082895.1	223088.8
B502-06	186.0	5082895.2	223094.2
B502-07	186.4	5082895.3	223099.6
B502-08	186.3	5082899.0	223098.0
B502-09	178.6	5082941.3	223074.9
B502-10	178.5	5083001.3	223049.8
B502-11	178.5	5083061.3	223024.7
B502-12	179.3	5083121.2	222999.6
B502-13	183.5	5083167.3	222980.3
B502-14	186.2	5083185.8	222972.5
B502-15	178.6	5082935.7	223069.1
B502-16	178.6	5082941.2	223066.8
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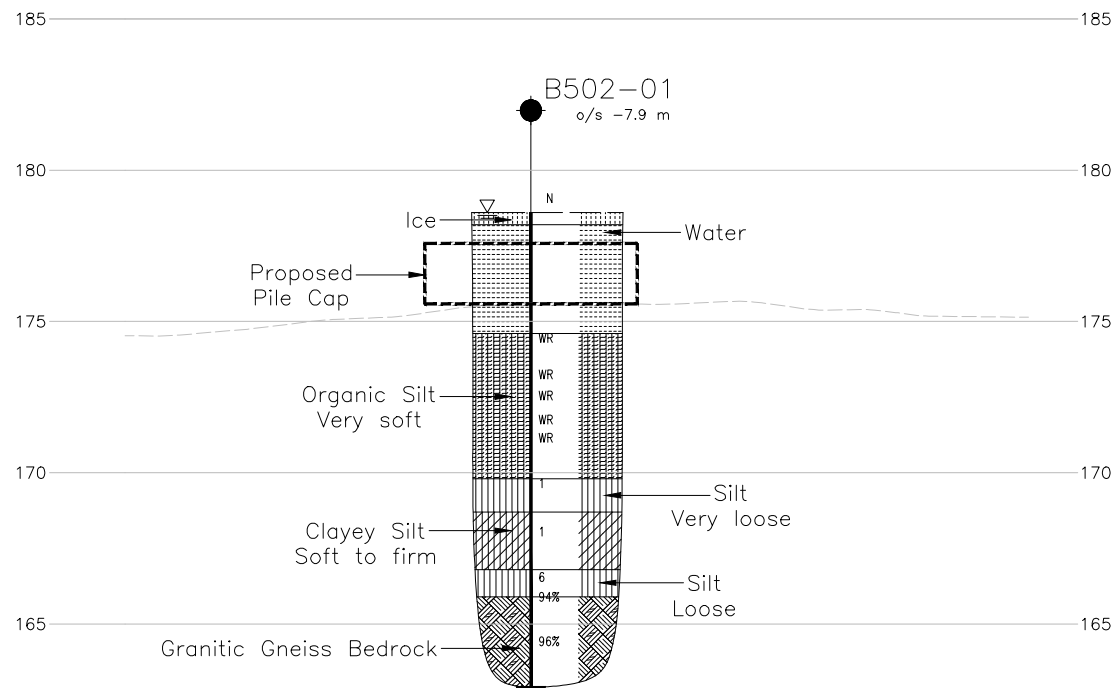
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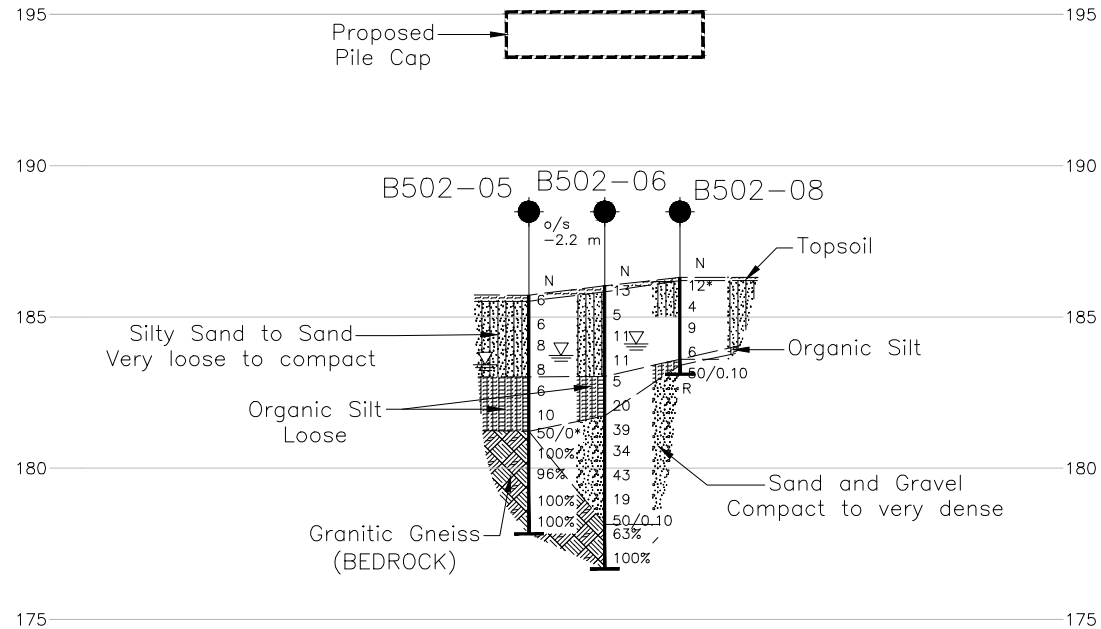
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DRAWN: JFC/MR	CHKD. AB	APPD. JMAC	DWG. 2		



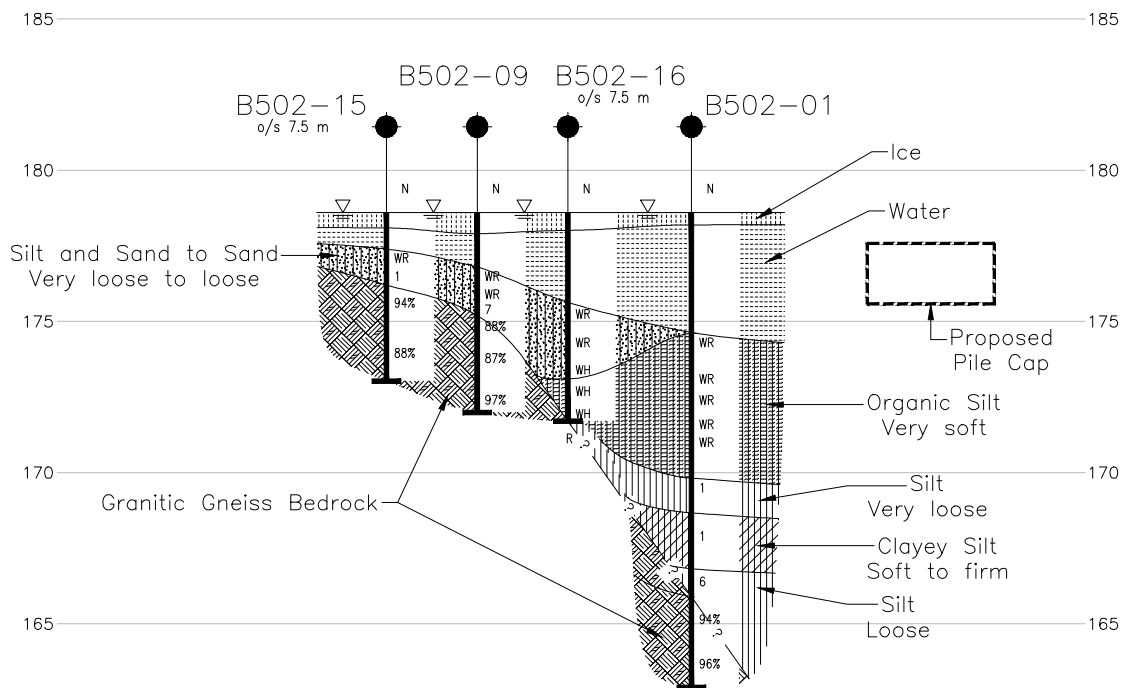
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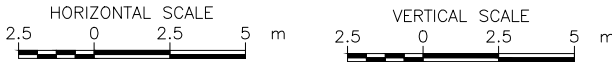
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1 NBL PIER 1 AREA CROSS-SECTION D-D'



C-C'
1 NBL SOUTH ABUTMENT CROSS-SECTION C-C'



E-E'
1 NBL PIER 1 AREA CROSS-SECTION E-E'

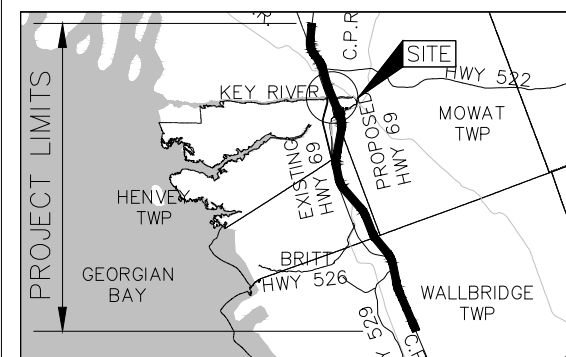


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STATIONS IN KILOMETRES + METRES.

CONT No. .
WP No. 5145-08-01

HIGHWAY 69
STRAIGHT LAKE NBL BRIDGE
SOIL STRATA

SHEET



KEY PLAN
SCALE
6 0 6 12 km

LEGEND

- Borehole - Current Investigation
- ⬮ Seal
- ⬮ Piezometer
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated
(Std. Pen. Test, 475 j/blow)
- 100% Rock Quality Designation (RQD)
- ▽ WL in piezometer, measured on March 19, 2014
- ▽ WL upon completion of drilling
- R Refusal

BOREHOLE CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
B502-01	178.6	5082947.9	223072.2
B502-04	185.7	5082891.5	223090.4
B502-05	185.7	5082895.1	223088.8
B502-06	186.0	5082895.2	223094.2
B502-07	186.4	5082895.3	223099.6
B502-08	186.3	5082899.0	223098.0
B502-09	178.6	5082941.3	223074.9
B502-15	178.6	5082935.7	223069.1
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NOTES

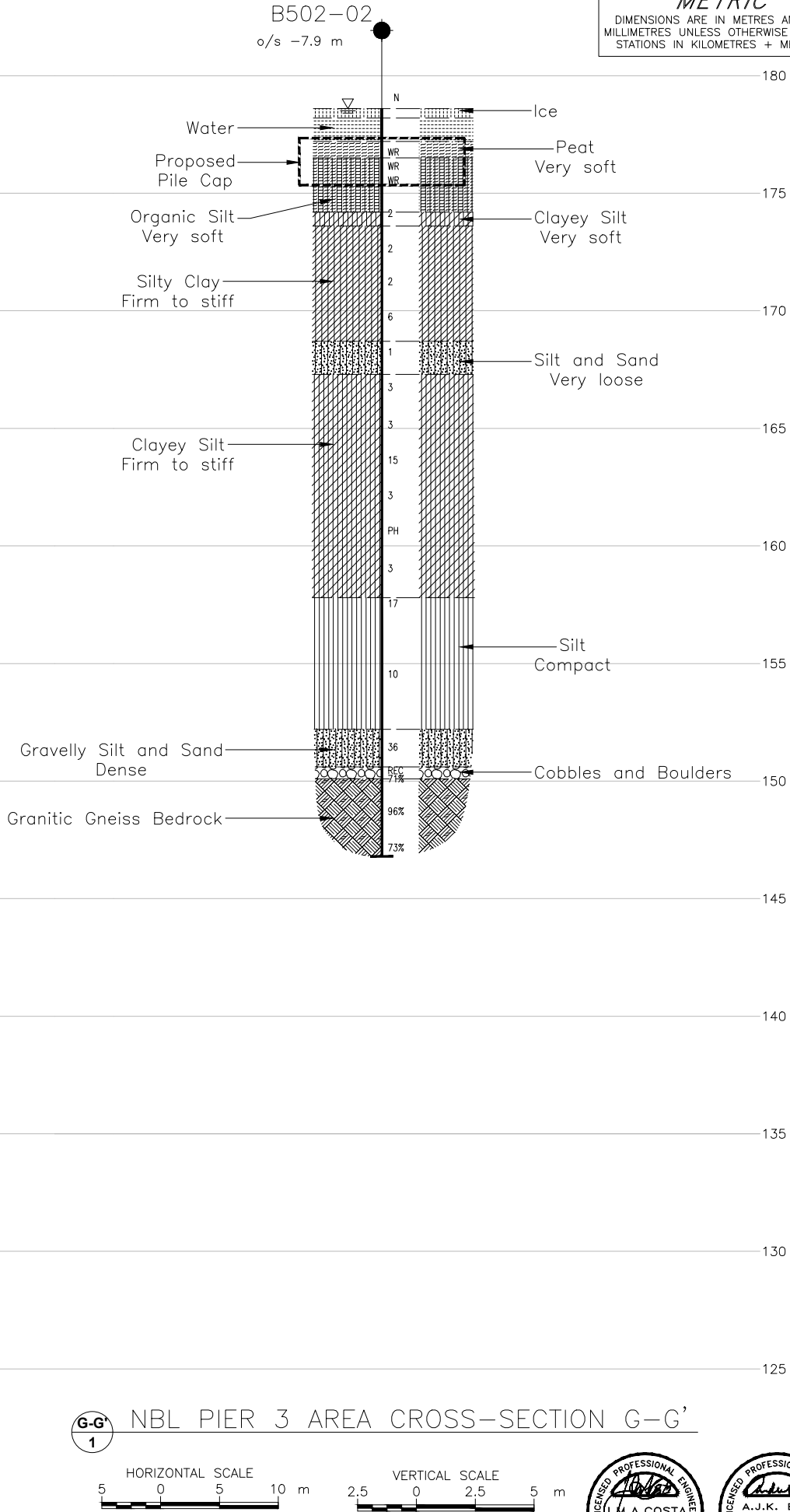
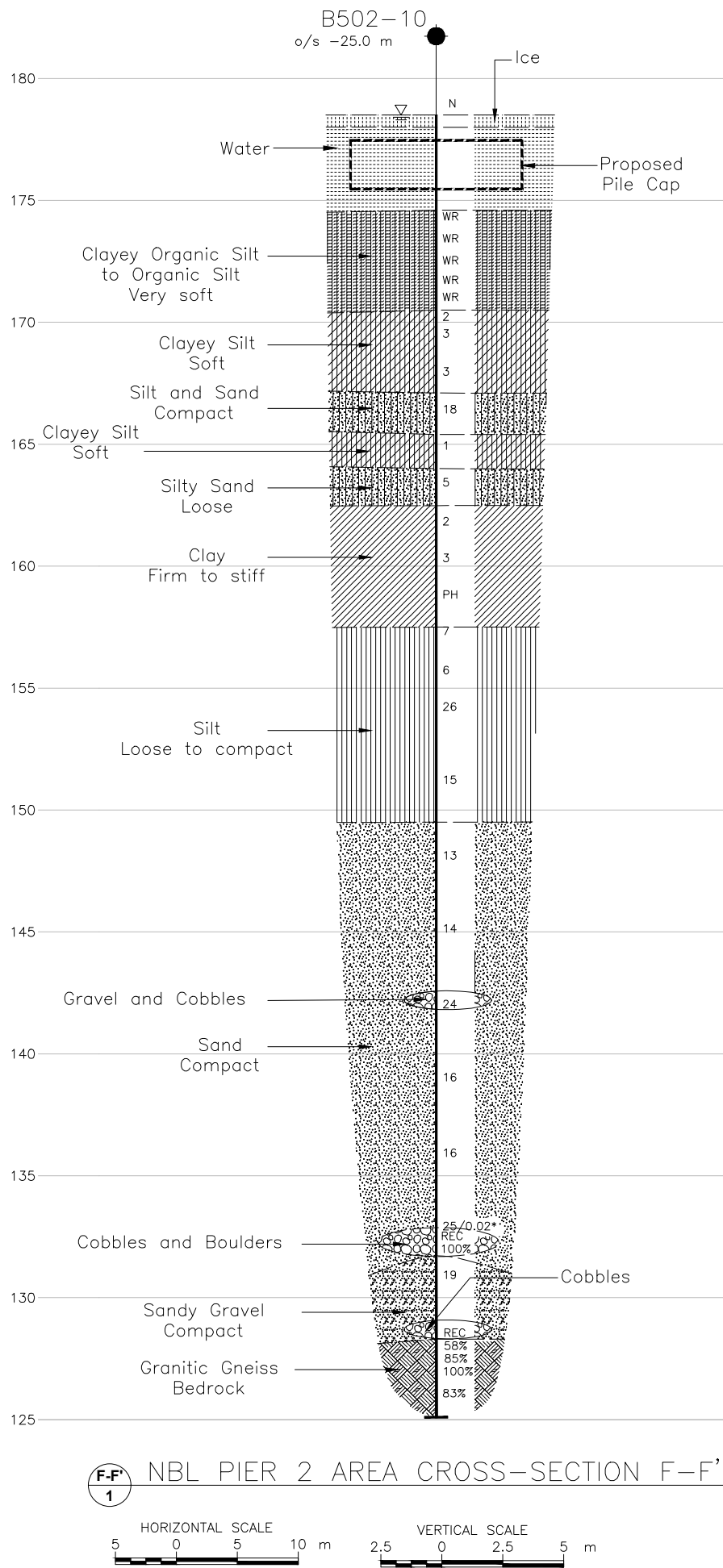
This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contracts Documents.

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

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

NO.	DATE	BY	REVISION
Geocres No. 41H-165			
HWY. 69			PROJECT NO. 09-1111-6014
SUBM'D. MCK	CHKD.	DATE: Nov 2015	SITE: 44-461/1
DRAWN: JFC/MR	CHKD. AB	APPD. JMAC	DWG. 3





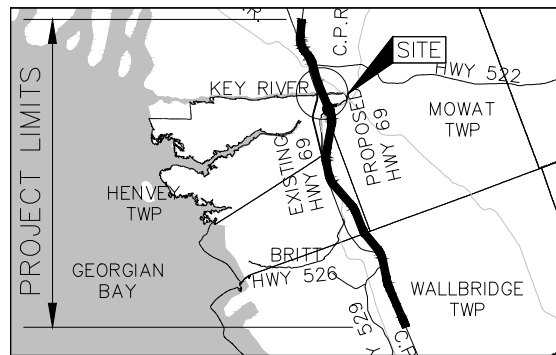
METRIC
DIMENSIONS ARE IN METRES AND/OR
MILLIMETRES UNLESS OTHERWISE SHOWN.
STATIONS IN KILOMETRES + METRES.

CONT No.
WP No. 5145-08-01

HIGHWAY 69
STRAIGHT LAKE NBL BRIDGE

SOIL STRATA

SHEET



LEGEND

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

BOREHOLE CO-ORDINATES			
No.	ELEVATION	NORTHING	EASTING
B502-02	178.6	5083086.2	223014.2
B502-10	178.5	5083001.3	223049.8

NOTES

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NO.	DATE	BY	REVISION
Geocres No. 41H-165			
HWY. 69		PROJECT NO. 09-1111-6014	DIST. .
SUBM'D. MCK	CHKD. .	DATE: Nov. 2015	SITE: 44-461/1
DRAWN: JFC	CHKD. AB	APPD. JMAC	DWG. 4



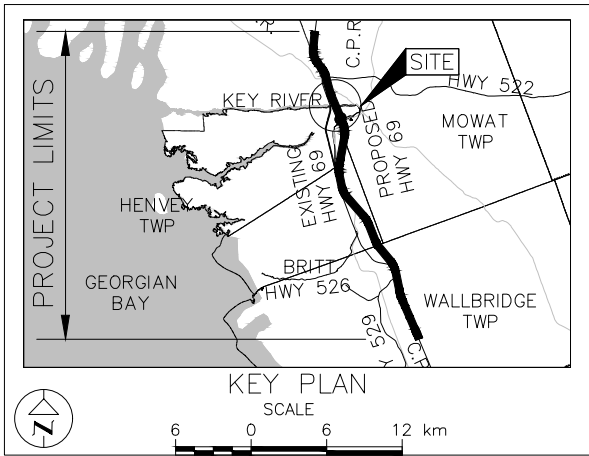
METRIC
DIMENSIONS ARE IN METRES AND/OR
MILLIMETRES UNLESS OTHERWISE SHOWN.
STATIONS IN KILOMETRES + METRES.

CONT No. .
WP No. 5145-08-01

HIGHWAY 69
STRAIGHT LAKE NBL BRIDGE

SOIL STRATA

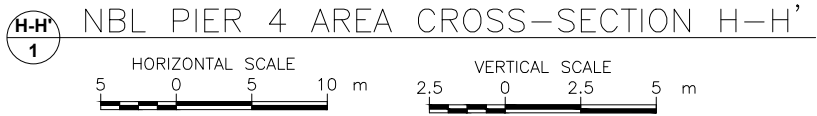
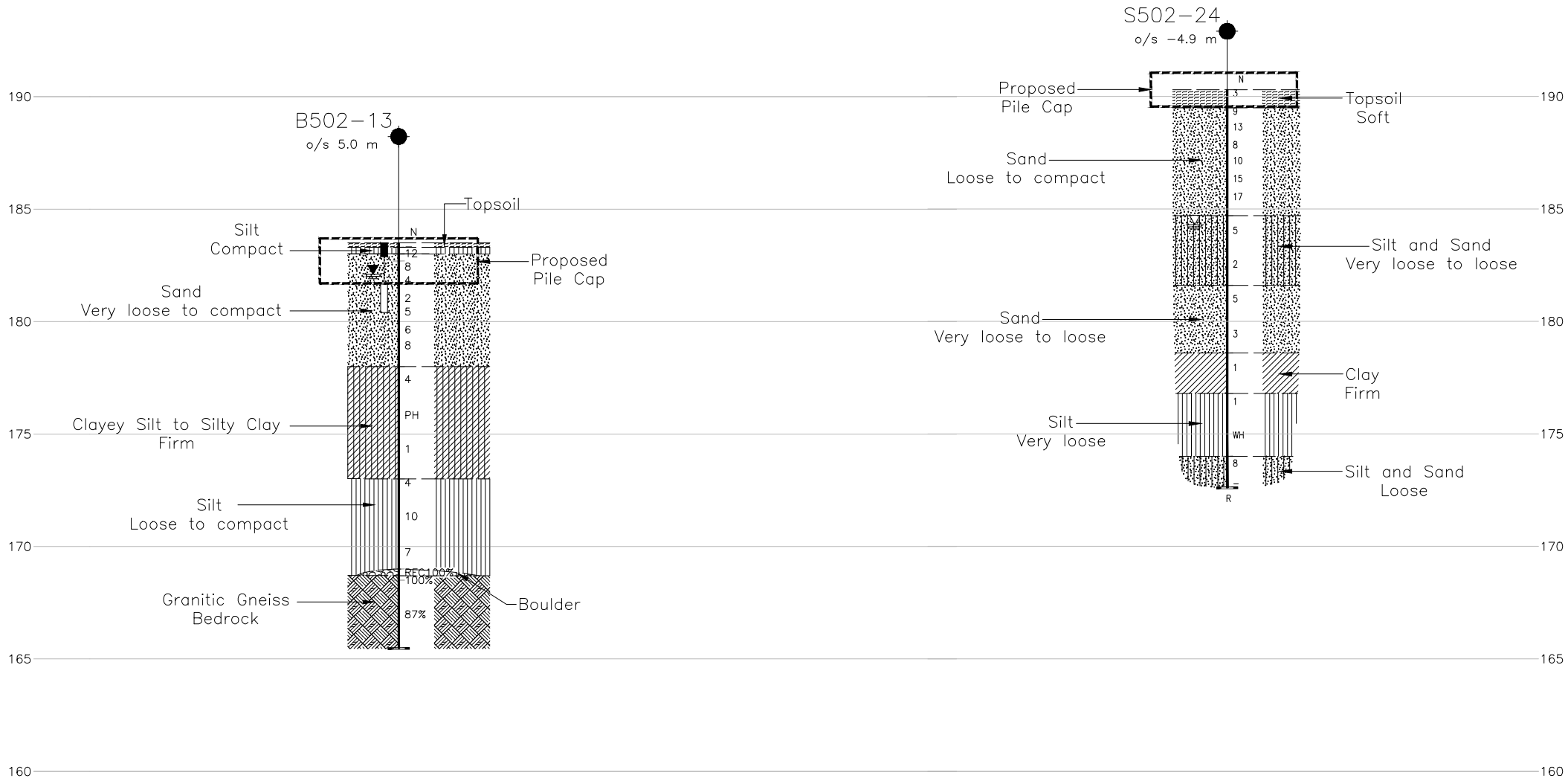
SHEET



LEGEND

- Borehole - Current Investigation
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- 100% Rock Quality Designation (RQD)
- WL in piezometer, measured on March 19, 2014
- WL upon completion of drilling
- REC Recovery

BOREHOLE CO-ORDINATES			
No.	ELEVATION	NORTHING	EASTING
B502-13	183.5	5083167.3	222980.3
S502-24	190.3	5083218.2	222959.0



NOTES

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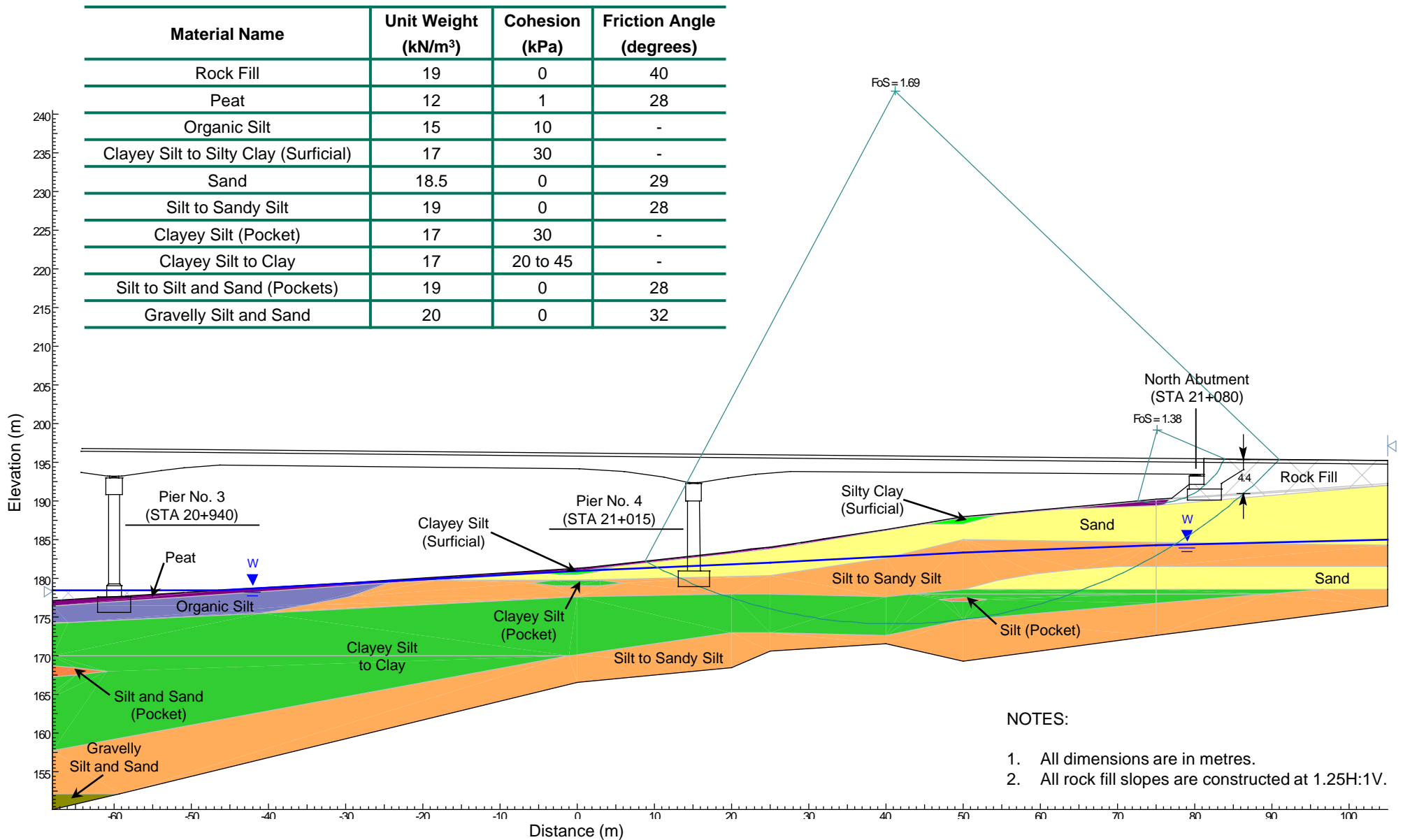
NO.	DATE	BY	REVISION
Geocres No. 41H-165			
HWY. 69	PROJECT NO. 09-1111-6014		DIST. .
SUBM'D. MCK	CHKD. .	DATE: Nov. 2015	SITE: 44-461/1
DRAWN: JFC	CHKD. AB	APPD. JMAC	DWG. 5





Highway 69 NBL – STA 21+080 to 21+100 (North Approach/High Fill 502), Front Slope Stability – Base Case

Figure 1





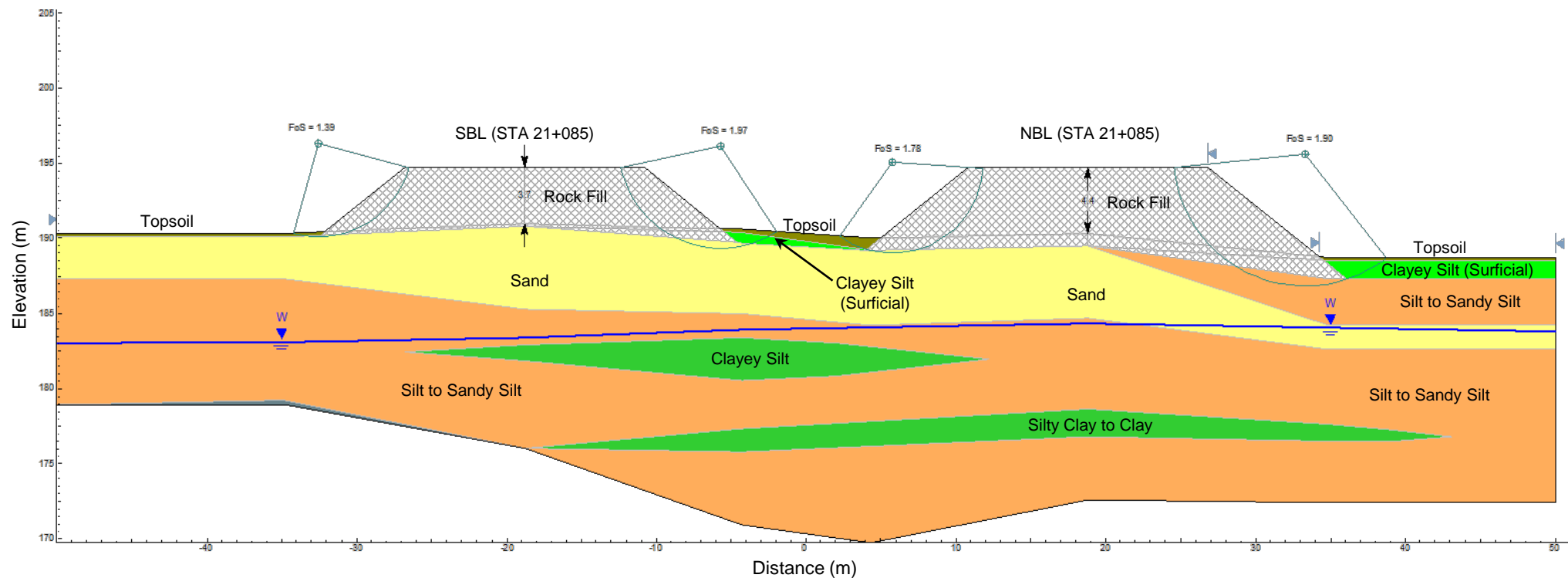
Highway 69 NBL – STA 21+085 (North Approach/High Fill 502), Side Slope Stability – Base Case

Figure 2

NOTES:

1. All dimensions are in metres.
2. All rock fill slopes are constructed at 1.25H:1V.
3. Borehole S502-24 was utilized to model the stratigraphy below the centerline of the Highway 69 NBL embankment.

Material Name	Unit Weight (kN/m ³)	Cohesion (kPa)	Friction Angle (degrees)
Rock Fill	19	0	40
Topsoil	15	1	28
Clayey Silt (Surficial)	17	30	-
Sand	18.5	0	29
Silt to Sandy Silt	19	0	28
Clayey Silt to Clay	17	20 to 45	-



Date: March 2016

Project No: 09-1111-6014-5520

Analysis By: TZ Reviewed By: JPD/JMAC

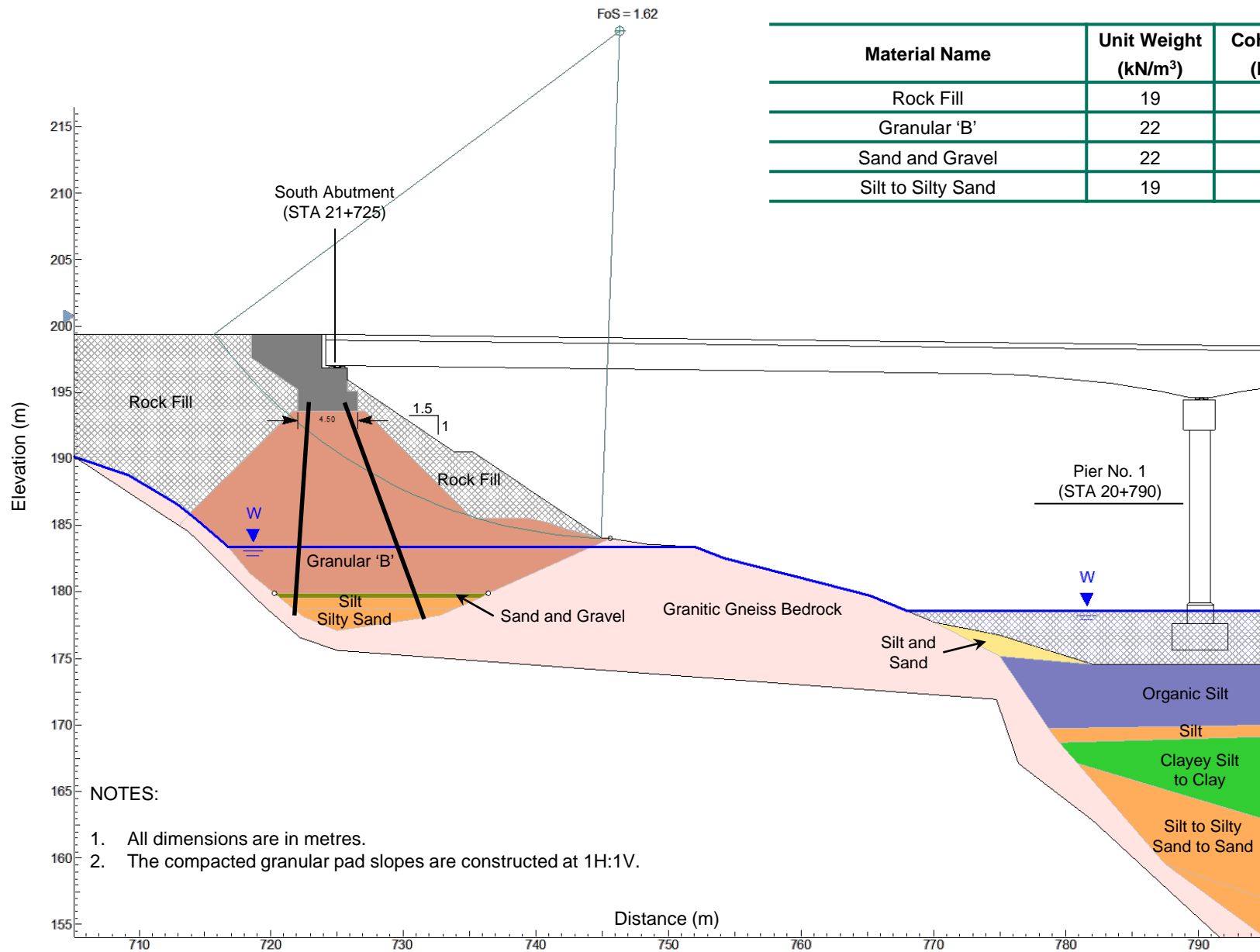




Highway 69 NBL – STA 20+705 to 20+725 (South Approach/High Fill 502), Front Slope Stability – Base Case

Figure 3

Material Name	Unit Weight (kN/m ³)	Cohesion (kPa)	Friction Angle (degrees)
Rock Fill	19	0	40
Granular 'B'	22	0	35
Sand and Gravel	22	0	34
Silt to Silty Sand	19	0	29





APPENDIX A

Record of Borehole and Drillhole Sheets



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

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

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.)

(b) Cohesive Soils Consistency

	kPa	Cu, Su	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

Dynamic Cone Penetration Resistance (DCPT); 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.

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

0 to 5	Trace
5 to 12	Trace to Some (or Little)
12 to 20	Some
20 to 30	(ey) or (y)
over 30	And (non-cohesive (cohesionless)) or With (cohesive)

Example

Trace sand
Trace to some sand
Some sand
Sandy
Sand and Gravel
Silty Clay with sand / Clayey Silt with sand



LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

WEATHERINGS STATE

Fresh: no visible sign of weathering

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

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

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

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

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

BEDDING THICKNESS

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

JOINT OR FOLIATION SPACING

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

GRAIN SIZE

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

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

CORE CONDITION

Total Core Recovery (TCR)

The percentage of solid drill core recovered regardless of quality or length, measured relative to the length of the total core run.

Solid Core Recovery (SCR)

The percentage of solid drill core, regardless of length, recovered at full diameter, measured relative to the length of the total core run.

Rock Quality Designation (RQD)

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

DISCONTINUITY DATA

Fracture Index

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

Dip with Respect to Core Axis

The angle of the discontinuity relative to the axis (length) of the core. In a vertical borehole a discontinuity with a 90° angle is horizontal.

Description and Notes

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

Abbreviations

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

PROJECT 09-1111-6014			RECORD OF BOREHOLE No B502-01			SHEET 1 OF 2			METRIC					
W.P. 5145-08-01			LOCATION N 5082947.9 ; E 223072.2			ORIGINATED BY MA								
DIST HWY 69			BOREHOLE TYPE NW Casing, Wash Boring, NQ Coring			COMPILED BY MCK								
DATUM Geodetic			DATE February 20 to 21, 2013			CHECKED BY AB								
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						
178.6	ICE SURFACE													
0.0	ICE													
178.2														
0.4	WATER													
174.6														
4.0	ORGANIC SILT Very soft Dark brown to dark grey Wet		1	SS	WR									
			2	SS	WR									
			3	SS	WR									
			4	SS	WR									
			5	SS	WR									
169.8	SILT, some sand, with silty clay seams Very loose Grey Wet		6	SS	1									
168.7	CLAYEY SILT Soft to firm Grey Wet		7	SS	1									
166.8	SILT, trace sand, trace clay Loose Grey Wet		8	SS	6									
165.9	Granitic Gneiss (BEDROCK) Bedrock cored from depths of 12.6 m to 15.7 m. For bedrock coring details refer to Record of Drillhole B502-01.		1	RC	REC 94%									
			2	RC	REC 100%									

Continued Next Page

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

GTA-MTO 001 T:\PROJECTS\2009\09-1111-6014 (URS, HWY 69, HENVEY)\LOG\09-1111-6014.GPJ GAL-GTA.GDT 10/13/16



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

GTA-MTO 001 T:\PROJECTS\2009\09-1111-6014 (URS, HWY 69, HENVEY)\LOG\09-1111-6014.GPJ GAL-GTA.GDT 10/13/16

PROJECT: 09-1111-6014

RECORD OF DRILLHOLE: B502-01

SHEET 1 OF 1

LOCATION: N 5082947.9 ;E 223072.2

DRILLING DATE: February 21, 2013

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: —

DRILL RIG: Diedrich D55

DRILLING CONTRACTOR: WALKER DRILLING

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	COLOUR % RETURN	FLUSH	JN - Joint FLT - Fault SH - Shear VN - Vein CJ - Conjugate	BD - Bedding FO - Foliation CO - Contact OR - Orthogonal CL - Cleavage	PL - Planar CU - Curved UN - Undulating ST - Stepped IR - Irregular	PO - Polished K - Slickensided SM - Smooth RO - Rough VR - Very Rough	MB - Mechanical Break BR - Broken Rock NOTE: For additional abbreviations refer to list of abbreviations & symbols.	NOTES
13	NW Casing	Continued from Record of Borehole B502-1		165.90									
13		Slightly weathered to fresh, thinly laminated to thinly bedded, dark grey with light pink bands, medium to coarse grained, strong, GRANITIC GNEISS		12.70	1								8.0 MPa (Axial) 9.9 MPa UCS = 68.1 MPa
14	NQRC February 21, 2013												
14													
15					2								6.9 MPa (Axial) 14.3 MPa
15													12.4 MPa
16		END OF DRILLHOLE		162.90									
16				15.70									
17													
18													
19													
20													
21													
22													

DEPTH SCALE

1 : 50

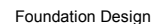


LOGGED: MA

CHECKED: AB

GTA-RCK 018 T:\PROJECTS\2009\09-1111-6014 (URS, HWY 69, HENVEY)\LOG\09-1111-6014.GPJ GAL-MISS.GDT 10/13/16

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




METRIC

CHECKED BY AB

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

GTA-MTO 001 T:\PROJECTS\2009\09-1111-6014 (URS, HWY 69, HENVEY)\LOG\09-1111-6014.GPJ GAL-GTA.GDT 10/13/16

PROJECT 09-1111-6014		RECORD OF BOREHOLE No B502-02				SHEET 3 OF 3		METRIC										
W.P. 5145-08-01		LOCATION N 5083086.2 ; E 223014.2				ORIGINATED BY MA												
DIST HWY 69		BOREHOLE TYPE NW Casing, Wash Boring, NQ Coring				COMPILED BY MCK												
DATUM Geodetic		DATE February 26 to 28, 2013				CHECKED BY AB												
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 (%) GR SA SI CL		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa										
	--- CONTINUED FROM PREVIOUS PAGE ---						20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					20 40 60 WATER CONTENT (%)						
146.8	Granitic Gneiss (BEDROCK)		3	RC	REC 100%	148											RQD = 96%	
31.8	Bedrock cored from depths of 28.5 m to 31.8 m. For bedrock coring details refer to Record of Drillhole B502-02.		4	RC	REC 97%	147											RQD = 73%	
	END OF BOREHOLE																	

PROJECT		RECORD OF BOREHOLE		No B502-03		SHEET 1 OF 1		METRIC								
W.P. 09-1111-6014		LOCATION		N 5082876.8 ; E 223101.9		ORIGINATED BY		TM								
DIST		HWY 69		BOREHOLE TYPE		Portable Equipment		COMPILED BY								
MCK		DATE		March 18, 2014		CHECKED BY		AB								
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								
190.2	GROUND SURFACE															
0.0	TOPSOIL		1	SS	50/0.05											
0.1	END OF BOREHOLE BEDROCK EXPOSED SPLIT-SPOON REFUSAL															

GTA-MTO 001 T:\PROJECTS\2009\09-1111-6014 (URS, HWY 69, HENVEY)\LOG\09-1111-6014.GPJ GAL-GTA.GDT 10/13/16

PROJECT		RECORD OF BOREHOLE		No B502-04		SHEET 1 OF 1		METRIC								
W.P. 09-1111-6014		LOCATION		N 5082891.5 ; E 223090.4		ORIGINATED BY		TM								
DIST		HWY 69		BOREHOLE TYPE		BW Casing, Wash Boring		COMPILED BY								
MCK		DATE		March 12, 2014		CHECKED BY		AB								
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								
185.7	GROUND SURFACE															
0.0	TOPSOIL															
0.2	SILT and SAND, trace to some clay Loose to compact Light brown Moist to wet		1	SS	9											
			2	SS	6											
			3	SS	11											
			4	SS	5											
182.7	ORGANIC SILT with wood fragments and roots and with silty sand layers (about 50 mm to 75 mm thick) between 4.7 m and 5.8 m Soft to very stiff Dark brown to black Wet		5	SS	3											
3.0			6	SS	5											
			7	SS	3											
179.9			8A	SS	21											
179.6	SAND and GRAVEL, trace silt Compact Grey Wet		9	SS	10											
6.1	SILT, some sand Loose to compact Grey Wet		10	SS	24											
178.8	Silty SAND, some gravel, trace clay Loose to compact Grey Wet		11	SS	10											
6.9			12	SS	50/0.08											
177.1	END OF BOREHOLE SPLIT-SPOON AND CASING REFUSAL															
8.6	NOTE: 1. Water level in open borehole at a depth of 2.3 m below ground surface (Elev. 183.4 m) upon completion of drilling.															

GTA-MTO 001 T:\PROJECTS\2009\09-1111-6014 (URS, HWY 69, HENVEY)\LOG\09-1111-6014.GPJ GAL-GTA.GDT 10/13/16

PROJECT		RECORD OF BOREHOLE		No B502-05		SHEET 1 OF 1		METRIC									
W.P. 09-1111-6014		LOCATION		N 5082895.1 ; E 223088.8		ORIGINATED BY		TM									
DIST		HWY 69		BOREHOLE TYPE		BW Casing, Wash Boring, Thin Wall NQ Coring		COMPILED BY									
DATUM		Geodetic		DATE		March 12, 13 and 14, 2014		CHECKED BY									
								AB									
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									
185.7	GROUND SURFACE							20	40	60	80	100					
0.0	TOPSOIL																
0.2	Silty SAND, trace clay Loose Light brown Moist		1	SS	6												
			2	SS	6												
			3	SS	8												
	Becoming wet below a depth of 2.3 m.		4A	SS	8												
183.0	ORGANIC SILT with wood fragments and roots Firm to stiff Dark brown Wet		4B	SS	6												
2.7			5	SS	6												
			6	SS	10												
181.2	Granitic Gneiss (BEDROCK)		7	SS	50 - 11												
4.5	Bedrock cored from depths of 4.5 m to 7.9 m. For bedrock coring details refer to Record of Drillhole B502-05.		1	RC	REC 100%												
			2	RC	REC 96%												
			3	RC	REC 100%												
			4	RC	REC 100%												
177.8	END OF BOREHOLE																
7.9	NOTE: 1. Water level in open borehole at a depth of 2.3 m below ground surface (Elev. 183.4 m) upon completion of drilling.																

GTA-MTO 001 T:\PROJECTS\2009\09-1111-6014 (URS, HWY 69, HENVEY)\LOG\09-1111-6014.GPJ GAL-GTA.GDT 10/13/16

SHEET 1 OF 1

DATUM: Geodetic

DRILLING CONTRACTOR: OGS DRILLING

CHECKED: AB

PROJECT 09-1111-6014		RECORD OF BOREHOLE No B502-06		SHEET 1 OF 1		METRIC																		
W.P. 5145-08-01		LOCATION N 5082895.2 ; E 223094.2		ORIGINATED BY TM																				
DIST HWY 69		BOREHOLE TYPE BW Casing, Wash Boring, Thin Wall NQ Coring		COMPILED BY MCK																				
DATUM Geodetic		DATE March 15 and 18, 2014		CHECKED BY AB																				
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS			ELEVATION SCALE			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																			
186.0	GROUND SURFACE																							
0.0	TOPSOIL																							
0.2	Silty SAND to SILT and SAND, trace clay Loose to compact Brown Moist to wet		1	SS	13																			
			2	SS	5																			
			3	SS	11																			
	Becoming wet below a depth of 2.3 m.		4	SS	11																			
183.0	ORGANIC SILT with wood fragments and roots Firm to very stiff Dark brown to black Wet		5	SS	5																			
3.0			6A	SS	20																			
181.7	SAND and GRAVEL, trace to some silt, trace clay Compact to dense Brown Wet		7	SS	39																			
4.3			8	SS	34																			
	Cobble inferred at a depth of 6.0 m.		9	SS	43																			
			10	SS	19																			
178.1	Granitic Gneiss (BEDROCK)		11	SS	50/0.10																			
7.9	Bedrock cored from depths of 7.9 m to 9.4 m. For bedrock coring details refer to Record of Drillhole B502-06.		1	RC	REC 97%																			
			2	RC	REC 100%																			
176.6	END OF BOREHOLE																							
9.4	NOTE: 1. Water level in open borehole at a depth of 2.3 m below ground surface (Elev. 183.7 m) upon completion of drilling.																							

PROJECT: 09-1111-6014

RECORD OF DRILLHOLE: B502-06

SHEET 1 OF 1

LOCATION: N 5082895.2 ;E 223094.2

DRILLING DATE: March 15 and 18, 2014

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: —

DRILL RIG: Portable Tripod

DRILLING CONTRACTOR: OGS DRILLING

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	COLOUR % RETURN	FLUSH	JN - Joint FLT - Fault SH - Shear VN - Vein CJ - Conjugate BD - Bedding FO - Foliation CO - Contact OR - Orthogonal CL - Cleavage PL - Planar CU - Curved UN - Undulating ST - Stepped IR - Irregular PO - Polished K - Slickensided SM - Smooth RO - Rough VR - Very Rough MB - Mechanical Break BR - Broken Rock NOTE: For additional abbreviations refer to list of abbreviations & symbols.										NOTES																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																											
								RECOVERY		R.Q.D. %	FRACT. INDEX PER 0.25	DISCONTINUITY DATA				HYDRAULIC CONDUCTIVITY K, cm/sec				Diametral Point Load Index (MPa)	RMC -Q AVG																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																								
								TOTAL CORE %	SOLID CORE %			B Angle	DIP w.r.t CORE AXIS	TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jn		10 ⁰			10 ¹	10 ²	10 ³																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																					
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8	NORC March 17 and 18, 2014	Continued from Record of Borehole B502-06		178.14 7.89	1																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																								</

DEPTH SCALE

1 : 50



LOGGED: TM

CHECKED: AB

PROJECT 09-1111-6014		RECORD OF BOREHOLE No B502-07				SHEET 1 OF 1		METRIC						
W.P. 5145-08-01		LOCATION N 5082895.3 ; E 223099.6				ORIGINATED BY ID								
DIST HWY 69		BOREHOLE TYPE BW Casing, Wash Boring, Thin Wall NQ Coring				COMPILED BY MCK								
DATUM Geodetic		DATE February 25 to 27, 2014				CHECKED BY AB								
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 (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
186.4	GROUND SURFACE													
0.0	TOPSOIL													
0.2	SILTY SAND to SILT and SAND, trace clay Very loose to compact Brown Moist		1	SS	7									
			2	SS	4									
			3	SS	8									
183.9	ORGANIC SILT and SAND with wood fragments and roots Stiff to compact Brown and grey Moist		4	SS	9									
183.1			5	SS	12*									
3.3	SAND to SAND and GRAVEL Compact to very dense Grey to brown Wet Cobbles inferred at 3.3 m to 3.9 m and 4.7 m depths		6	RC	-									
			7	SS	11*									
			8	SS	30/0*									
			9	SS	43									
180.1	Granite Gneiss (BEDROCK)		10	SS	26/0.15*									
6.3	Bedrock cored from depths of 6.3 m to 9.8 m For bedrock coring details Refer to Record of Drillhole B502-07.		1	RC	REC 100%									RQD = 30%
			2	RC	REC 82%									RQD = 77%
			3	RC	REC 100%									RQD = 100%
			4	RC	REC 100%									RQD = 100%
176.6	END OF BOREHOLE													
9.8	NOTE: 1. Water level in piezometer at a depth of 3.7 m below ground surface (Elev. 182.7 m) on March 19, 2014. * Split-Spoon Sampler bouncing.													

GTA-MTO 001 T:\PROJECTS\2009\09-1111-6014 (URS, HWY 69, HENVEY)\LOG\09-1111-6014.GPJ GAL-GTA.GDT 10/13/16

SHEET 1 OF 1

DATUM: Geodetic

DRILLING CONTRACTOR: OGS DRILLING

CHECKED: AB

PROJECT		RECORD OF BOREHOLE		No B502-08		SHEET 1 OF 1		METRIC								
W.P. 09-1111-6014		LOCATION		N 5082899.0 ; E 223098.0		ORIGINATED BY		TM								
DIST		HWY 69		BOREHOLE TYPE		NW Casing, Wash Boring		COMPILED BY								
MCK		DATE		March 18, 2014		CHECKED BY		AB								
Geodetic																
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								
186.3	GROUND SURFACE															
0.0	TOPSOIL															
	Silty SAND to SAND, some silt, trace clay Very loose to compact Brown to grey Moist to wet		1	SS	12*											
			2	SS	4											
			3	SS	9											
			4A	SS	6											
183.6	ORGANIC SILT, trace sand Brown and grey Wet		4B	SS	50/0.10											
183.1	SAND and GRAVEL, some silt Very dense Brown Wet		5	SS												
3.2	END OF BOREHOLE SPLIT-SPOON REFUSAL															
NOTE: 1. Water level in open borehole at a depth of 2.2 m below ground surface (Elev. 184.1 m) upon completion of drilling. * 'N' Value is impacted by frost.																

PROJECT 09-1111-6014		RECORD OF BOREHOLE No B502-09		SHEET 1 OF 1		METRIC												
W.P. 5145-08-01		LOCATION N 5082941.3 ; E 223074.9		ORIGINATED BY ID														
DIST HWY 69		BOREHOLE TYPE NW Casing, Wash Boring, NQ Coring		COMPILED BY MCK														
DATUM Geodetic		DATE February 19 and 20, 2014		CHECKED BY AB														
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC NATURAL 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 (%)					
178.6	ICE SURFACE							20 40 60 80 100	○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					W _p W W _L				
0.0	ICE							20 40 60 80 100						20 40 60				
177.9	0.7 WATER						178											
176.8	1.8 SILT and SAND, trace organics Very loose to loose Dark grey Wet		1	SS	WR		177											
			2	SS	WR		176											
175.2			3	SS	7													
3.4	Granitic Gneiss (BEDROCK) Bedrock cored from depths of 3.4 m to 6.6 m. For bedrock coring details refer to Record of Drillhole B502-09.		1	RC	REC 100%		175										RQD = 88%	
			2	RC	REC 98%		174										RQD = 87%	
			3	RC	REC 97%		173										RQD = 97%	
172.0	6.6 END OF BOREHOLE						172											

GTA-MTO 001 T:\PROJECTS\2009\09-1111-6014 (URS, HWY 69, HENVEY)\LOG09-1111-6014.GPJ GAL-GTA.GDT 10/13/16

PROJECT: 09-1111-6014

RECORD OF DRILLHOLE: B502-09

SHEET 1 OF 1

LOCATION: N 5082941.3 ;E 223074.9

DRILLING DATE: February 19 and 20, 2014

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: —

DRILL RIG: Diedrich D25

DRILLING CONTRACTOR: LANDCORE DRILLING

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	COLOUR % RETURN	JN - Joint FLT - Fault SH - Shear VN - Vein CJ - Conjugate												BD - Bedding FO - Foliation CO - Contact OR - Orthogonal CL - Cleavage												PL - Planar CU - Curved UN - Undulating ST - Stepped IR - Irregular												PO - Polished K - Slickensided SM - Smooth RO - Rough VR - Very Rough												MB - Mechanical Break BR - Broken Rock NOTE: For additional abbreviations refer to list of abbreviations & symbols.												NOTES																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																									
							FLUSH	RECOVERY		R.Q.D. %	FRACT. INDEX PER 0.25	B Angle	DIP w.r.t CORE AXIS	TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jn	HYDRAULIC CONDUCTIVITY K, cm/sec			Diametral Point Load Index (MPa)	RMC -Q AVG																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																						
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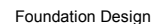
DEPTH SCALE

1 : 50



LOGGED: ID

CHECKED: AB



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

PROJECT		RECORD OF BOREHOLE		No B502-10		SHEET 2 OF 4		METRIC						
W.P. 09-1111-6014		LOCATION		N 5083001.3 ; E 223049.8		ORIGINATED BY		MR						
DIST		HWY 69		BOREHOLE TYPE		NW Casing, Wash Boring, NQ Coring		COMPILED BY						
MCK		DATE		February 13, 19 to 21, 2014		CHECKED BY		AB						
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						
	--- CONTINUED FROM PREVIOUS PAGE ---							20 40 60 80 100						
162.5	Silty SAND, trace clay Loose Grey Wet		11	SS	5		163							
16.0	CLAY, with silt seams to a depth of 17.1 m Fim to stiff Grey and brown Wet		12	SS	2		162							
							161							
			13	SS	3		160							
							159							
			14	SS	PH		158							
157.5	SILT, trace to some sand, trace to some clay Loose to compact Grey Wet		15	SS	7		157							
21.0							156							
			16	SS	6		155							
							154							
			17	SS	26		153							
							152							
			18	SS	15		151							
							150							
149.5	SAND, some silt, trace to some gravel Compact Grey Wet						149							
29.0														

Continued Next Page

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

GTA-MTO 001 T:\PROJECTS\2009\09-1111-6014 (URS, HWY 69, HENVEY)\LOG\09-1111-6014.GPJ GAL-GTA.GDT 10/13/16



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

GTA-MTO 001 T:\PROJECTS\2009\09-1111-6014 (URS, HWY 69, HENVEY)\LOG\09-1111-6014.GPJ GAL-GTA.GDT 10/13/16

PROJECT 09-1111-6014		RECORD OF BOREHOLE No B502-10		SHEET 4 OF 4		METRIC									
W.P. 5145-08-01		LOCATION N 5083001.3 ; E 223049.8		ORIGINATED BY MR											
DIST HWY 69		BOREHOLE TYPE NW Casing, Wash Boring, NQ Coring		COMPILED BY MCK											
DATUM Geodetic		DATE February 13, 19 to 21, 2014		CHECKED BY AB											
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 (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER			TYPE	"N" VALUES	SHEAR STRENGTH kPa							
	--- CONTINUED FROM PREVIOUS PAGE ---														
132.9			24	SS	25/0.02	133									
45.6	COBBLES and BOULDERS		25	RC	REC 30%	132									
131.7															
46.8	Sandy GRAVEL Compact Grey Wet		26	SS	19	131									
						130									
						129									
128.3	Cobbles encountered at a depth of 49.8 m.		27	RC	REC 58%	128									
50.2	Granitic Gneiss (BEDROCK)		1	RC	REC 95%	127									
	Rock cored from depths of 50.2 m to 53.4 m		2	RC	REC 100%	126									
	For rock coring details refer to Record of Drillhole B502-10.		3	RC	REC 100%										
125.1	END OF BOREHOLE														
53.4	NOTE: * Split-Spoon Sampler bouncing.														

GTA-MTO 001 T:\PROJECTS\2009\09-1111-6014 (URS, HWY 69, HENVEY)\LOG\09-1111-6014.GPJ GAL-GTA.GDT 10/13/16

SHEET 1 OF 1

DATUM: Geodetic

DRILLING CONTRACTOR: WALKER DRILLING

[illegible]

CHECKED: AB

GTARCK 018 T:\PROJECTS\2009\09-1111-6014 (URS, HWY 69, HENVEY)\LOG\09-1111-6014.GPJ GAL-MISS.GDT 10/13/16

PROJECT <u>09-1111-6014</u>		RECORD OF BOREHOLE No B502-11		SHEET 1 OF 3		METRIC	
W.P. <u>5145-08-01</u>		LOCATION <u>N 5083061.3 ; E 223024.7</u>		ORIGINATED BY <u>ID</u>			
DIST <u> </u> HWY <u>69</u>		BOREHOLE TYPE <u>NW Casing, Wash Boring, NQ Coring</u>		COMPILED BY <u>MCK</u>			
DATUM <u>Geodetic</u>		DATE <u>February 13 to 14 and 18 to 19, 2014</u>		CHECKED BY <u>AB</u>			

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 (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED						
178.5 0.0	ICE SURFACE ICE						20 40 60 80 100			20 40 60				
177.9 0.6	WATER						20 40 60 80 100			20 40 60				
176.7 1.8	Clayey ORGANIC SILT Very soft Dark grey Wet	1	SS	WR	178									151.3 OC = 6.4%
		2	SS	WR	177									
		3	SS	WR	176									149.2 OC = 6.4%
		4	SS	WR	175									
		5	SS	WR	174									
		6	SS	WR	173									189.5
172.3 6.2	CLAYEY SILT Soft to firm Grey Wet	7A	SS	1	172									113.6
		7B												
		8	TO	PH	171									
		9	SS	2	170									
167.8 10.7	Sandy SILT, trace to some clay Loose Grey Wet	10	SS	5	169						0 26 66 8			
						168								
166.3 12.2	SILTY CLAY, with silt lenses Firm Brown to grey Wet	11	SS	1	167									
						166								
			12	TO	PH	165								
					164									

Continued Next Page

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

GTA-MTO 001 T:\PROJECTS\2009\09-1111-6014 (URS, HWY 69, HENVEY)\LOG\09-1111-6014.GPJ GAL-GTA.GDT 10/13/16

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



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

SHEET 1 OF 1

DATUM: Geodetic

DRILLING CONTRACTOR: LANDCORE DRILLING

CHECKED: AB


PROJECT 09-1111-6014		RECORD OF BOREHOLE No B502-12		SHEET 1 OF 2		METRIC	
W.P. 5145-08-01		LOCATION N 5083121.2; E 222999.6		ORIGINATED BY MR			
DIST HWY 69		BOREHOLE TYPE 150 mm O.D. Continuous Flight Solid Stem Augers, NW Casing, Wash Boring		COMPILED BY MCK			
DATUM Geodetic		DATE March 4 and 5, 2014		CHECKED BY AB			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		W _p	W	W _L		
								○ UNCONFINED	+ FIELD VANE					
179.3	GROUND SURFACE													
0.0	PEAT		1A	SS	1									
178.9	Very loose Dark brown Moist		1B	SS										
0.4														
178.1	Silty SAND, trace organics Loose Grey Wet		2A	SS	9									
1.2			2B	SS										
	CLAYEY SILT to SILTY CLAY, trace to some sand Soft to firm Grey Wet		3	SS	2									
			4	SS	1									
			5	TO	PH									
			6	SS	2									
			7	SS	1									
			8	SS	1									
			9	SS	3									
			10	SS	1									
			11	SS	8									
			1	RC	REC 100%									

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

GTA-MTO 001 T:\PROJECTS\2009-1111-6014 (URS, HWY 69, HENVEY)\LOG09-1111-6014.GPJ GAL-GTA.GDT 10/13/16

PROJECT 09-1111-6014		RECORD OF BOREHOLE No B502-12				SHEET 2 OF 2		METRIC									
W.P. 5145-08-01		LOCATION N 5083121.2; E 222999.6				ORIGINATED BY MR											
DIST HWY 69		BOREHOLE TYPE 150 mm O.D. Continuous Flight Solid Stem Augers, NW Casing, Wash Boring				COMPILED BY MCK											
DATUM Geodetic		DATE March 4 and 5, 2014				CHECKED BY AB											
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 (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa									
	--- CONTINUED FROM PREVIOUS PAGE ---						<div style="display: flex; justify-content: space-between;"> 20 40 60 80 100 20 40 60 80 100 </div> <div style="display: flex; justify-content: space-between;"> ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED </div>										
161.3	Granitic Gneiss (BEDROCK)		1	RC	REC 100%		164										RQD = 100%
	Bedrock cored from depths of 14.7 m to 18.0 m.		2	RC	REC 100%		163										RQD = 100%
	For bedrock coring details refer to Record of Drillhole B502-12.		3	RC	REC 100%		162										RQD = 93%
18.0	END OF BOREHOLE																
	NOTE: 1. Water level in open borehole at a depth of 0.5 m below ground surface (Elev. 178.8 m) upon completion of drilling.																

PROJECT: 09-1111-6014

RECORD OF DRILLHOLE: B502-12

SHEET 1 OF 1

LOCATION: N 5083121.2 ;E 222999.6

DRILLING DATE: March 5, 2014

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: —

DRILL RIG: Diedrich D25

DRILLING CONTRACTOR: WALKER DRILLING

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	COLOUR % RETURN	FLUSH	RECOVERY				R.Q.D. %	FRACT. INDEX PER 0.25	B Angle	DIP w.r.t CORE AXIS	DISCONTINUITY DATA				HYDRAULIC CONDUCTIVITY K, cm ³ /sec				Diameter Point Load Index (MPa)	RMC -Q AVG	NOTES																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																	
								TOTAL CORE %	SOLID CORE %	JN	FLT					SH	VN	CJ	BD	FO	CO	OR	CL				PL	CU	UN	ST	IR	PO	K	SM	RO	VR	MB	BR																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																					
NOTE: For additional abbreviations refer to list of abbreviations & symbols.																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																											
15	NW Casing	Continued from Record of Borehole B502-12		164.54																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																							

DEPTH SCALE

1 : 50



LOGGED: MR

CHECKED: AB

GTA-RCK 018 T:\PROJECTS\2009\09-1111-6014 (URS, HWY 69, HENVEY)\LOG\09-1111-6014.GPJ GAL-MISS.GDT 10/13/16


PROJECT		RECORD OF BOREHOLE		No B502-13		SHEET 1 OF 2		METRIC	
W.P.		LOCATION		ORIGINATED BY		DIST		BOREHOLE TYPE	
DATE		DATE		COMPILED BY		CHECKED BY			
09-1111-6014		N 5083167.3 ; E 222980.3		MR		HWY 69		150 mm O.D. Continuous Flight Solid Stem Augers, NW Casing, Wash Boring	
Geodetic		March 8, 2014		MCK		AB			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	W _p W W _L	20 40 60			
183.5	GROUND SURFACE		1	AS	-									
0.0	TOPSOIL		2A	SS	12		183							
183.0	SILT, some clay, trace sand Compact Brown Moist		2B	SS	8		182							
0.5	SAND, trace silt, trace organics to a depth of 1.0 m Very loose to compact Brown to grey Wet		3	SS	4		181							
			4	SS	2		180							
			5	SS	5		179							
			6	SS	6		178							
			7	SS	8		177							
			8	SS	4		176							
178.0	CLAYEY SILT to SILTY CLAY, trace sand Firm Grey Wet		9	SS	1		175							
5.5			10	TO	PH		174							
			11	SS	4		173							
			12	SS	10		172							
			13	SS	7		171							
173.0	SILT, trace to some sand, trace to some clay, trace gravel Loose to compact Grey Wet		14	SS	100%		170							
10.5			15	RC REC	100%		169							
169.0	BOULDER													
168.7														

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

GTA-MTO 001 T:\PROJECTS\2009\09-1111-6014 (URS, HWY 69, HENVEY)\LOG09-1111-6014.GPJ GAL-GTA.GDT 10/13/16

PROJECT <u>09-1111-6014</u>		RECORD OF BOREHOLE No B502-13				SHEET 2 OF 2		METRIC									
W.P. <u>5145-08-01</u>		LOCATION <u>N 5083167.3 ; E 222980.3</u>				ORIGINATED BY <u>MR</u>											
DIST <u> </u> HWY <u>69</u>		BOREHOLE TYPE <u>150 mm O.D. Continuous Flight Solid Stem Augers, NW Casing, Wash Boring</u>				COMPILED BY <u>MCK</u>											
DATUM <u>Geodetic</u>		DATE <u>March 8, 2014</u>				CHECKED BY <u>AB</u>											
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa					W _p W W _L				
--- CONTINUED FROM PREVIOUS PAGE ---																	
15.0	Granitic Gneiss (BEDROCK)		1	RC	REC 100%	168											RQD = 100%
	Bedrock cored from depths of 15.0 m to 18.0 m. For bedrock coring details refer to Record of Drillhole B502-13.					167											
			2	RC	REC 100%	166											RQD = 87%
165.5 18.0	END OF BOREHOLE																
	NOTES: 1. Water level in piezometer at a depth of 1.4 m below ground surface (Elev. 182.1 m) on March 19, 2014. 2. Piezometer was installed 0.5 m West of Borehole B502-13.																

GTA-MTO 001 T:\PROJECTS\2009\09-1111-6014 (URS, HWY 69, HENVEY)\LOG\09-1111-6014.GPJ GAL-GTA.GDT 10/13/16

PROJECT 09-1111-6014		RECORD OF BOREHOLE No B502-14		SHEET 1 OF 2		METRIC														
W.P. 5145-08-01		LOCATION N 5083185.8 ; E 222972.5		ORIGINATED BY MAS																
DIST _____ HWY 69		BOREHOLE TYPE 150 mm O.D. Continuous Flight Solid Stem Augers, NW Casing, Wash Boring		COMPILED BY MCK																
DATUM Geodetic		DATE March 10, 2014		CHECKED BY AB																
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
186.2	GROUND SURFACE							20 40 60 80 100												
0.0	TOPSOIL		1A		7		186													
0.2	SILT and SAND, trace clay Loose to compact Brown Moist		1B	SS																
			2	SS	14		185													
			3	SS	16		184													
			4	SS	13		183													
			5	SS	8		182													
181.6	CLAYEY SILT Stiff Grey Wet		6	SS	3		181													
							180													
180.1	SILT and SAND, trace clay Loose Grey Wet		7	SS	6		179													
			8	SS	6		178													
178.0	CLAYEY SILT to SILTY CLAY Firm Grey Wet		9	SS	4		177													
			10	TO	PH		176													
			11	SS	WH		175													
172.5	SILT, some sand Compact Grey Wet		12	SS	13		174													
13.7							173													
171.4							172													
14.8																				

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

GTA-MTO 001 T:\PROJECTS\2009\09-1111-6014 (URS, HWY 69, HENVEY)\LOG\09-1111-6014.GPJ GAL-GTA.GDT 10/13/16

PROJECT <u>09-1111-6014</u>		RECORD OF BOREHOLE No B502-14				SHEET 2 OF 2		METRIC	
W.P. <u>5145-08-01</u>		LOCATION <u>N 5083185.8 ;E 222972.5</u>				ORIGINATED BY <u>MAS</u>			
DIST <u> </u> HWY <u>69</u>		BOREHOLE TYPE <u>150 mm O.D. Continuous Flight Solid Stem Augers, NW Casing, Wash Boring</u>				COMPILED BY <u>MCK</u>			
DATUM <u>Geodetic</u>		DATE <u>March 10, 2014</u>				CHECKED BY <u>AB</u>			

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	20	40	60	80	100	W _p	W		
	<p style="text-align: center;">--- CONTINUED FROM PREVIOUS PAGE ---</p> <p>END OF BOREHOLE SPLIT-SPOON REFUSAL</p> <p>NOTE:</p> <p>1. Water level in open borehole at a depth of 4.3 m below ground surface (Elev. 181.9 m) upon completion of drilling.</p>															

GTA-MTO 001 T:\PROJECTS\2009\09-1111-6014 (URS, HWY 69, HENVEY)\LOG\09-1111-6014.GPJ GAL-GTA.GDT 10/13/16

PROJECT		RECORD OF BOREHOLE		No B502-15		SHEET 1 OF 1		METRIC							
W.P. 09-1111-6014		LOCATION		N 5082935.7 ; E 223069.1		ORIGINATED BY		MR							
DIST		HWY 69		BOREHOLE TYPE		NW Casing, Wash Boring, NQ Coring		COMPILED BY							
DATUM		Geodetic		DATE		February 23, 2014		CHECKED BY							
								AB							
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							
178.6 0.0	ICE SURFACE ICE														
178.1 0.5	WATER														
177.4 1.2	SAND, some silt, trace organics Very loose Dark brown Wet		1	SS	WR										
176.2 2.4	Granitic Gneiss (BEDROCK) Bedrock cored from depths of 2.4 m to 5.6 m. For bedrock coring details refer to Record of Drillhole B502-15.		2	SS	1										
176.2 2.4	Granitic Gneiss (BEDROCK)		1	RC	REC 100%										RQD = 94%
175.0 5.6	END OF BOREHOLE		2	RC	REC 100%										RQD = 88%
173.0 5.6	END OF BOREHOLE														

GTA-MTO 001 T:\PROJECTS\2009\09-1111-6014 (URS, HWY 69, HENVEY)\LOG\09-1111-6014.GPJ GAL-GTA.GDT 10/13/16

PROJECT: 09-1111-6014

RECORD OF DRILLHOLE: B502-15

SHEET 1 OF 1

LOCATION: N 5082935.7 ;E 223069.1

DRILLING DATE: February 25, 2014

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: —

DRILL RIG: Diedrich D25

DRILLING CONTRACTOR: WALKER DRILLING

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	COLOUR % RETURN	FLUSH	RECOVERY		R.Q.D. %	FRACT. INDEX PER 0.25	DISCONTINUITY DATA				HYDRAULIC CONDUCTIVITY K, cm/sec				Diametral Point Load Index (MPa)	RMC -Q AVG	NOTES																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																															
								TOTAL CORE %	SOLID CORE %			TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jn	10 °	10 °	10 °	10 °																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																		
		Continued from Record of Borehole B502-15		176.25 2.35																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																	</

DEPTH SCALE

1 : 50



LOGGED: MR

CHECKED: AB

PROJECT		RECORD OF BOREHOLE		No B502-16		SHEET 1 OF 1		METRIC								
W.P. 5145-08-01		LOCATION		N 5082941.2 ; E 223066.8		ORIGINATED BY		MR								
DIST		HWY 69		BOREHOLE TYPE		NW Casing, Wash Boring		COMPILED BY								
DATUM		Geodetic		DATE		February 23, 2014		CHECKED BY								
								AB								
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								
178.6	ICE SURFACE															
0.0	ICE															
178.0																
0.6	WATER															
175.6																
3.0	Silty SAND, some clay, trace organics Very loose Dark grey Wet		1	SS	WR											
			2	SS	WR											
			3	SS	WH											
173.1																
5.5	ORGANIC SILT, trace to some clay Very soft Dark grey Wet		4	SS	WH											
			5	SS	WH											
171.7																
6.9	END OF BOREHOLE CASING REFUSAL															

GTA-MTO 001 T:\PROJECTS\2009\09-1111-6014 (URS, HWY 69, HENVEY)\LOG\09-1111-6014.GPJ GAL-GTA-GDT 10/13/16

PROJECT		RECORD OF BOREHOLE				No B502-17		SHEET 1 OF 1		METRIC							
W.P. 09-1111-6014		LOCATION		N 5082941.5 ; E 223083.0		ORIGINATED BY		MR									
DIST		HWY 69		BOREHOLE TYPE		NW Casing, Wash Boring		COMPILED BY		MCK							
DATUM		Geodetic		DATE		February 25, 2014		CHECKED BY		AB							
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									
178.6	ICE SURFACE							20	40	60	80	100					
0.0	ICE																
178.0							178										
0.6	WATER																
177.1																	
1.5	ORGANIC SILT						177										
176.6	Very soft		1	SS	WR												
2.0	Grey Wet																
	END OF BOREHOLE CASING REFUSAL																

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PROJECT		09-1111-6014		RECORD OF BOREHOLE No B502-18		SHEET 1 OF 1		METRIC								
W.P.		5145-08-01		LOCATION		N 5082947.0 ; E 223080.6		ORIGINATED BY								
DIST		HWY 69		BOREHOLE TYPE		NW Casing, Wash Boring, NQ Coring		COMPILED BY								
DATUM		Geodetic		DATE		February 22, 2014		CHECKED BY								
								AB								
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 (%)
178.6	WATER SURFACE							20	40	60	80	100				
0.0	ICE															
178.1																
0.5	WATER															
175.5																
3.1	ORGANIC SILT, trace to some sand, trace to some clay Very soft Dark grey Wet		1	SS	WR											
			2	SS	WR											
			3	SS	WH											
			4	SS	WR											
			5	SS	WR											
171.5																
7.1	Granitic Gneiss (BEDROCK) Rock cored from depths of 7.1 m to 10.7 m For rock coring details refer to Record of Drillhole B502-18.		1	RC	REC 100%											
			2	RC	REC 100%											
			3	RC	REC 100%											
167.9																
10.7	END OF BOREHOLE															

GTA-MTO 001 T:\PROJECTS\2009\09-1111-6014 (URS, HWY 69, HENVEY)\LOG\09-1111-6014.GPJ GAL-GTA.GDT 10/13/16

SHEET 1 OF 1

DATUM: Geodetic

DRILLING CONTRACTOR: WALKER DRILLING

[illegible]

DEPTH SCALE

1 : 50



LOGGED: MR

CHECKED: AB

PROJECT		RECORD OF BOREHOLE		No S502-08		SHEET 1 OF 2		METRIC														
W.P.		LOCATION		ORIGINATED BY		DIST		BOREHOLE TYPE														
5005-10-01		N 5083197.7 ; E 222942.6		SA		HWY 69		106 mm I.D. Continuous Flight Hollow Stem Augers														
DATUM		DATE		CHECKED BY		COMPILED BY		GRL														
Geodetic		March 6 and 7, 2013		CN																		
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		
190.6	0.0	GROUND SURFACE							20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					20 40 60 W _p W W _L			kN/m ³					
0.0	0.2	PEAT (Fibrous) Very soft Dark brown Wet		1A	SS	2		190														
189.7	0.9	CLAYEY SILT Very soft Brown Wet		2A	SS	17		189														
				2B	SS	17																
		SAND, trace to some silt, trace clay Loose to compact Brown to becoming grey below a depth of 3.8 m Moist to Wet		3	SS	15		188														
				4	SS	7																
				5	SS	6		187														
				6	SS	17																
				7	SS	20		186														
185.0	5.6	Sandy SILT, trace clay Loose Grey Wet		8	SS	7		185														
								184														
183.4	7.2	CLAYEY SILT Firm Grey Wet		9	SS	2		183														
								182														
				10	SS	WH		181														
180.5	10.1	SILT, trace to some sand, trace clay Very loose to loose Grey Wet						180														
				11	SS	3		179														
								178														
				12	SS	5																
177.3	13.3	CLAYEY SILT, containing silt seams Soft Grey Wet						177														
				13	SS	3																
175.8	14.8							176														

Continued Next Page

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

GTA-MTO 001 T:\PROJECTS\2009\09-1111-6014 (URS, HWY 69, HENVEY)\LOG\09-1111-6014.GPJ GAL-GTA.GDT 07/25/16

PROJECT		RECORD OF BOREHOLE		No S502-08		SHEET 2 OF 2		METRIC							
W.P. 09-1111-6014		LOCATION		N 5083197.7 ; E 222942.6		ORIGINATED BY		SA							
DIST		HWY 69		BOREHOLE TYPE		106 mm I.D. Continuous Flight Hollow Stem Augers		COMPILED BY							
GRL		DATE		March 6 and 7, 2013		CHECKED BY		CN							
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 (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							
	--- CONTINUED FROM PREVIOUS PAGE ---														
172.7	SILT, trace to some sand, trace clay Very loose to loose Grey Wet		14	SS	9		175								0 10 87 3
							174								
			15	SS	WR		173								
17.9	SILT and SAND, trace clay Very loose Grey Wet		16	SS	3		172								0 39 57 4
170.9	END OF BOREHOLE SPLIT-SPOON AND AUGER REFUSAL		17	SS			171								
19.7	NOTE: 1. Water level in open borehole at a depth of 6.7 m below ground surface (Elev. 183.9 m) upon completion of drilling.														

GTA-MTO 001 T:\PROJECTS\2009\09-1111-6014 (URS, HWY 69, HENVEY)\LOG\09-1111-6014.GPJ GAL-GTA.GDT 07/25/16

PROJECT 09-1111-6014		RECORD OF BOREHOLE No S502-12		SHEET 1 OF 1		METRIC																					
W.P. 5005-10-01		LOCATION N 5083243.4 ; E 222922.1		ORIGINATED BY SA																							
DIST _____ HWY 69		BOREHOLE TYPE 106 mm I.D. Continuous Flight Hollow Stem Augers		COMPILED BY GRL																							
DATUM Geodetic		DATE March 7, 2013		CHECKED BY CN																							
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			SHEAR STRENGTH kPa			WATER CONTENT (%)			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES		ELEVATION SCALE																				
192.1	GROUND SURFACE																										
0.0	TOPSOIL		1A	SS	3		192																				
0.2	CLAYEY SILT Soft to firm Grey Wet		1B																								
191.0			2A	SS	6		191																				
1.1	SAND, trace to some silt, trace to some clay, trace gravel Loose to compact Brown and grey Wet		2B																								
			3	SS	9																						
			4	SS	10		190																				
			5	SS	12		189																				
			6A	SS	5		188																				
187.8	CLAYEY SILT Brown Wet		6B																								
187.5	SILT and SAND, trace clay Very loose to compact Grey Wet		7	SS	18		187																				
4.6			8	SS	10		186																				
			9	SS	3		185																				
			10	SS	5		184																				
			11	SS	3		183																				
			12	SS	25/0.18		182																				
179.6	END OF BOREHOLE SPLIT-SPOON AND AUGER REFUSAL						181																				
12.5	NOTE: 1. Water level in open borehole at a depth of 7.6 m below ground surface (Elev. 184.5 m) upon completion of drilling.						180																				

GTA-MTO 001 T:\PROJECTS\2009\09-1111-6014 (URS, HWY 69, HENVEY)\LOG09-1111-6014.GPJ GAL-GTA.GDT 07/25/16

PROJECT		09-1111-6014		RECORD OF BOREHOLE No S502-18		SHEET 1 OF 2		METRIC														
W.P.		5005-10-01		LOCATION		N 5083149.0 ; E 222988.0		ORIGINATED BY ID														
DIST		HWY 69		BOREHOLE TYPE		106 mm I.D. Continuous Flight Hollow Stem Augers		COMPILED BY AV														
DATUM		Geodetic		DATE		March 13 and 14, 2013		CHECKED BY CN														
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS			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		ELEVATION SCALE	SHEAR STRENGTH kPa					WATER CONTENT (%)			γ	GR	SA	SI	CL	
								20	40	60	80	100	20	40	60							
181.3	0.0	GROUND SURFACE		1A	SS	1		181														
181.0	0.3	PEAT (Fibrous) Very soft Black Wet		1B	SS	1																
180.5	0.8	CLAYEY SILT, trace to some sand, trace gravel, trace organics Very soft Dark grey Wet		2	SS	7		180														
179.8	1.5	SAND, trace to some gravel Loose Brown Wet		3	SS	1																
179.1	2.2	CLAYEY SILT, with silt seams Very soft Grey Wet		4	SS	4		179														
		SILT and SAND, trace clay Very loose Grey Wet		5	SS	3		178														
177.6	3.7	SILTY CLAY, with silt seams Firm Grey Wet		6	SS	7		177														
				7	SS	WH		176														
175.5	5.8	CLAYEY SILT Firm to stiff Grey Wet		8	SS	1		175														
				9	TO	PH		174														
				10	SS	3		173														
				11A	SS	1		172														
170.2	11.1	SILT, trace to some sand, trace to some clay Very loose to compact Grey Wet		11B	SS	1		171														
				12	SS	8		170														
				13	SS	16		169														
								168														
166.6	14.7	END OF BOREHOLE						167														

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

GTA-MTO 001 T:\PROJECTS\2009\09-1111-6014 (URS, HWY 69, HENVEY)\LOG\09-1111-6014.GPJ GAL-GTA.GDT 07/25/16

PROJECT <u>09-1111-6014</u>		RECORD OF BOREHOLE No S502-18		SHEET 2 OF 2		METRIC																		
W.P. <u>5005-10-01</u>		LOCATION <u>N 5083149.0 ; E 222988.0</u>		ORIGINATED BY <u>ID</u>																				
DIST <u> </u> HWY <u>69</u>		BOREHOLE TYPE <u>106 mm I.D. Continuous Flight Hollow Stem Augers</u>		COMPILED BY <u>AV</u>																				
DATUM <u>Geodetic</u>		DATE <u>March 13 and 14, 2013</u>		CHECKED BY <u>CN</u>																				
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL								
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa					W _p	W			W _L							
	--- CONTINUED FROM PREVIOUS PAGE ---						<div style="display: flex; justify-content: space-between;"> 20 40 60 80 100 20 40 60 80 100 </div> <div style="display: flex; justify-content: space-between;"> ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED </div>																	
	AUGER REFUSAL NOTE: 1. Water level in open borehole at a depth of 0.3 m below ground surface (Elev. 181.0 m) upon completion of drilling.																							

GTA-MTO 001 T:\PROJECTS\2009\09-1111-6014 (URS, HWY 69, HENVEY)\LOG\09-1111-6014.GPJ GAL-GTA.GDT 07/25/16

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

PROJECT 09-1111-6014		RECORD OF BOREHOLE No S502-20		SHEET 1 OF 2		METRIC															
W.P. 5005-10-01		LOCATION N 5083172.0 ; E 222978.4		ORIGINATED BY ID																	
DIST HWY 69		BOREHOLE TYPE 106 mm I.D. Continuous Flight Hollow Stem Augers		COMPILED BY GRL																	
DATUM Geodetic		DATE March 1 and 4, 2013		CHECKED BY CN																	
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		
184.1	GROUND SURFACE							20 40 60 80 100	20 40 60	20 40 60	20 40 60	20 40 60	20 40 60	20 40 60	20 40 60	20 40 60	20 40 60	20 40 60	20 40 60	20 40 60	20 40 60
0.0	TOPSOIL		1A	SS	6		184														
183.8	Black		1B	SS	7		183														
0.3	SAND, trace silt, trace organics Very loose to loose Brown becoming grey below a depth of 2.3 m Moist to wet		2	SS	6		182														
			3	SS	6		181														
			4	SS	WH		180														
			5	SS	WH		179														
180.4	SILT and SAND, trace clay Loose Grey Wet		6	SS	6		178														
3.7	Clayey silt seams below a depth of 4.6 m		7	SS	5		177														
			8	SS	WH		176														
178.0	CLAY Soft to stiff Grey Wet		9	SS	1		175														
6.1			10	TO	PH		174														
			11A	SS	WH		173														
173.6	CLAYEY SILT Very soft Grey Wet		11B	SS	WH		172														
10.5	SILT, some sand, trace to some clay Very loose Grey Wet		12	SS	3		171														
173.0	SILT and SAND, trace clay Very loose Grey Wet		13	SS	10/0.05																
11.1																					
172.4																					
11.7																					
170.6																					
13.5																					

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

GTA-MTO 001 T:\PROJECTS\2009\09-1111-6014 (URS, HWY 69, HENVEY)\LOG\09-1111-6014.GPJ GAL-GTA.GDT 07/25/16



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

GTA-MTO 001 T:\PROJECTS\2009\09-1111-6014 (URS, HWY 69, HENVEY)\LOG\09-1111-6014.GPJ GAL-GTA.GDT 07/25/16

PROJECT		RECORD OF BOREHOLE		No S502-20A		SHEET 1 OF 2		METRIC	
W.P.		LOCATION		ORIGINATED BY		DIST		BOREHOLE TYPE	
DATE		COMPILED BY		CHECKED BY		DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT	
Geodetic		March 9, 2014		JPD				W _p	
SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS		ELEVATION SCALE		SHEAR STRENGTH kPa	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	ELEVATION	20 40 60 80 100	20 40 60 80 100	20 40 60
184.2	GROUND SURFACE		1A	SS	12 *	184			
0.0	TOPSOIL Dark brown Moist		1B	SS					
183.6	SAND, some gravel, trace silt Loose to compact Brown Moist		2	SS	4	183			
0.6	SILT, trace sand, trace clay Loose Brown to grey Wet		3A	SS					
182.7	CLAYEY SILT, trace sand Soft to firm Grey Wet		3B	SS	5	182			
1.5	SAND Very loose to loose Grey Wet		4	SS	4				
181.6			5	SS	2	181			
2.6			6	SS	7				
			7	SS	6	180			
179.0						179			
5.2	SILT CLAY to CLAY, trace sand Soft to firm Grey Wet		8	SS	2	178			
177.1	SILT, trace to some sand, trace clay Loose Grey Wet		9	TO	PH	177			
7.1									
			10	SS	9	176			
172.9			11	SS	10	175			
11.3	SAND, some gravel Very dense Grey Wet								
171.8			12	SS	50/0.15	174			
12.4						173			
						172			

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

GTA-MTO 001 T:\PROJECTS\2009\09-1111-6014 (URS, HWY 69, HENVEY)\LOG09-1111-6014.GPJ GAL-GTA-GDT 07/25/16

PROJECT <u>09-1111-6014</u>		RECORD OF BOREHOLE No S502-20A				SHEET 2 OF 2		METRIC									
W.P. <u>5005-10-01</u>		LOCATION <u>N 5083189.5 ;E 222993.4</u>				ORIGINATED BY <u>MR</u>											
DIST <u> </u> HWY <u>69</u>		BOREHOLE TYPE <u>150 mm O.D. Continuous Flight Solid Stem Augers, NW Casing, Wash Boring</u>				COMPILED BY <u>BM</u>											
DATUM <u>Geodetic</u>		DATE <u>March 9, 2014</u>				CHECKED BY <u>JPD</u>											
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 (%)
	--- CONTINUED FROM PREVIOUS PAGE ---																
	END OF BOREHOLE NOTES: 1. Water level in open borehole at a depth of 0.6 m below ground surface (Elev. 183.6 m) upon completion of drilling. 2. An (S502-20B) additional borehole was advanced 0.5 m east of borehole S502-20A to obtain a Shelby Tube sample at a depth of 5.5 m (Elev. 178.7 m) and to carry out in situ field vanes at depths of 6.4 m (Elev. 177.8 m) and 6.7 m (Elev. 177.5 m). * "N" Value impacted by frozen ground.																

GTA-MTO 001 T:\PROJECTS\2009\09-1111-6014 (URS, HWY 69, HENVEY)\LOG\09-1111-6014.GPJ GAL-GTA.GDT 07/25/16



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

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

PROJECT 09-1111-6014		RECORD OF BOREHOLE No S502-21				SHEET 2 OF 2		METRIC								
W.P. 5005-10-01		LOCATION N 5083176.3 ; E 222956.2				ORIGINATED BY ID										
DIST _____ HWY 69		BOREHOLE TYPE 106 mm I.D. Continuous Flight Hollow Stem Augers				COMPILED BY GRL										
DATUM Geodetic		DATE February 27, 2013				CHECKED BY CN										
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 (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa								
	--- CONTINUED FROM PREVIOUS PAGE ---						20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100					WATER CONTENT (%) 20 40 60				
170.8	SILT, some sand, trace clay Very loose to loose Grey Wet		14	SS	4		172									
16.5	END OF BOREHOLE SPLIT-SPOON AND AUGER REFUSAL NOTE: 1. Water level in open borehole at a depth of 4.0 m below ground surface (Elev. 183.3 m) upon completion of drilling.		15	SS	-		171									

GTA-MTO 001 T:\PROJECTS\2009\09-1111-6014 (URS, HWY 69, HENVEY)\LOG\09-1111-6014.GPJ GAL-GTA.GDT 07/25/16

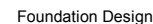
PROJECT <u>09-1111-6014</u>		RECORD OF BOREHOLE No S502-22		SHEET 1 OF 2		METRIC	
W.P. <u>5005-10-01</u>		LOCATION <u>N 5083195.1 ; E 222968.7</u>		ORIGINATED BY <u>SA</u>			
DIST <u> </u> HWY <u>69</u>		BOREHOLE TYPE <u>106 mm I.D. Continuous Flight Hollow Stem Augers</u>		COMPILED BY <u>CC/AV</u>			
DATUM <u>Geodetic</u>		DATE <u>March 4 and 5, 2013</u>		CHECKED BY <u>CN</u>			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		W _p	W	W _L		
								○ UNCONFINED ● QUICK TRIAXIAL	+ FIELD VANE × REMOULDED					
188.0	GROUND SURFACE													
0.0	TOPSOIL		1A	SS	5									
0.1	SILTY CLAY, trace organics Firm Brown and grey Wet		1B	SS										
187.1			2A	SS	7									
0.9	Silty SAND, trace clay Very loose to compact Brown Moist		2B	SS										
			3	SS	9									
			4	SS	11									
185.0			5	SS	10									0 75 23 2
3.0	SILT and SAND, trace clay Very loose to compact Grey Moist to wet		6	SS	2									0 51 47 2
			7	SS	4									
			8	SS	2									
			9	SS	5									
178.6			10A	SS	4									
9.4	CLAYEY SILT, trace sand Firm Grey Wet		10B	SS										
177.5			11A	SS	2									
10.5	SILT, some sand Very loose Grey Wet		11B	SS										
177.0			12	TO	PH									
11.0	SILTY CLAY, trace sand Firm to stiff Grey Wet													
174.7			13	SS	7									0 10 86 4
13.3	SILT, trace to some sand, trace clay Loose Grey Wet													
173.3														
14.7														

Continued Next Page

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

GTA-MTO 001 T:\PROJECTS\2009\09-1111-6014 (URS, HWY 69, HENVEY)\LOG\09-1111-6014.GPJ GAL-GTA.GDT 07/25/16



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

GTA-MTO 001 T:\PROJECTS\2009\09-1111-6014 (URS, HWY 69, HENVEY)\LOG\09-1111-6014.GPJ GAL-GTA.GDT 07/25/16

PROJECT 09-1111-6014		RECORD OF BOREHOLE No S502-23		SHEET 1 OF 2		METRIC											
W.P. 5005-10-01		LOCATION N 5083212.8 ; E 222978.6		ORIGINATED BY SA													
DIST HWY 69		BOREHOLE TYPE 106 mm I.D. Continuous Flight Hollow Stem Augers		COMPILED BY GRL													
DATUM Geodetic		DATE March 5 and 6, 2013		CHECKED BY CN													
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 kN/m³	GR SA SI CL
							20 40 60 80 100	20 40 60 80 100	20 40 60	W _p W W _L							
188.7	GROUND SURFACE																
0.0	TOPSOIL		1A		3												
0.2	CLAYEY SILT, trace sand, trace organics Soft to firm Brown Moist		1B	SS			188										
187.3			2	SS	7												
1.4	SILT and SAND, trace to some clay Loose to compact Brown, becoming grey below a depth of 3.8m Wet		3	SS	11		187									0 58 36 6	
			4	SS	14		186										
			5	SS	14		185										
			6	SS	6											0 57 41 2	
184.2	SAND, some silt Very loose Grey Wet		7	SS	3	▽	184									0 82 18 0	
							183										
182.6	SILT, trace to some clay, containing silty clay seams Very loose Grey Wet		8	SS	1		182										
181.4	SILT and SAND, trace clay Loose to compact Grey wet		9	SS	7		181										
			10	SS	8		180									0 58 41 1	
			11A	SS	19		178										
177.6	SILTY CLAY Firm to stiff Grey Wet		11B				177										
176.5																	
12.2	SILT, trace to some sand, trace clay Very loose to loose Grey Wet		12	SS	1		176									Non-Plastic	
			13	SS	7		175										
173.7							174										

Continued Next Page

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

GTA-MTO 001 T:\PROJECTS\2009\09-1111-6014 (URS, HWY 69, HENVEY)\LOG\09-1111-6014.GPJ GAL-GTA.GDT 07/25/16

PROJECT		RECORD OF BOREHOLE		No S502-23		SHEET 2 OF 2		METRIC								
W.P. 5005-10-01		LOCATION		N 5083212.8 ; E 222978.6		ORIGINATED BY		SA								
DIST		HWY 69		BOREHOLE TYPE		106 mm I.D. Continuous Flight Hollow Stem Augers		COMPILED BY								
GRL		DATE		March 5 and 6, 2013		CHECKED BY		CN								
DATUM		Geodetic														
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								
	--- CONTINUED FROM PREVIOUS PAGE ---															
15.0	SILT Very loose to loose Grey wet		14	SS	WH		173									
172.4																
16.3	END OF BOREHOLE SPLIT-SPOON AND AUGER REFUSAL NOTE: 1. Water level in open borehole at a depth of 4.9 m below ground surface (Elev. 183.8 m) upon completion of drilling.		15	SS	10/0.0											

GTA-MTO 001 T:\PROJECTS\2009\09-1111-6014 (URS, HWY 69, HENVEY)\LOG\09-1111-6014.GPJ GAL-GTA.GDT 07/25/16

PROJECT <u>09-1111-6014</u>		RECORD OF BOREHOLE No S502-24		SHEET 1 OF 2		METRIC	
W.P. <u>5005-10-01</u>		LOCATION <u>N 5083218.2 ; E 222959.0</u>		ORIGINATED BY <u>ID</u>			
DIST <u> </u> HWY <u>69</u>		BOREHOLE TYPE <u>106 mm I.D. Continuous Flight Hollow Stem Augers</u>		COMPILED BY <u>GRL</u>			
DATUM <u>Geodetic</u>		DATE <u>February 27 and 28, 2013</u>		CHECKED BY <u>CN</u>			

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 (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)
								○ UNCONFINED	+ FIELD VANE						
190.3	GROUND SURFACE														
0.0	TOPSOIL Soft Dark brown to black Wet		1	SS	3		190						OC = 6.3%		
189.5															
0.8	SAND, trace to some silt, trace clay Loose to compact Brown Moist		2	SS	9		189								
			3	SS	13										
			4	SS	8		188							0 88 9 3	
			5	SS	10		187								
			6	SS	15		186								
			7	SS	17										
184.7							185								
5.6	SILT and SAND, trace clay Very loose to loose Grey Wet Clayey silt seams below a depth of 7.6 m		8	SS	5		184						0 62 36 2		
			9	SS	2		183							0 47 49 4	
181.6							182								
8.7	SAND, some silt Very loose to loose Grey Wet		10	SS	5		181								
			11	SS	3		180								
178.6							179								
11.7	CLAY, with silt seams Firm Grey Wet		12	SS	1		178								
176.8							177	5							
13.5	SILT, trace clay Very loose Grey Wet		13	SS	1		176							Non-Plastic	

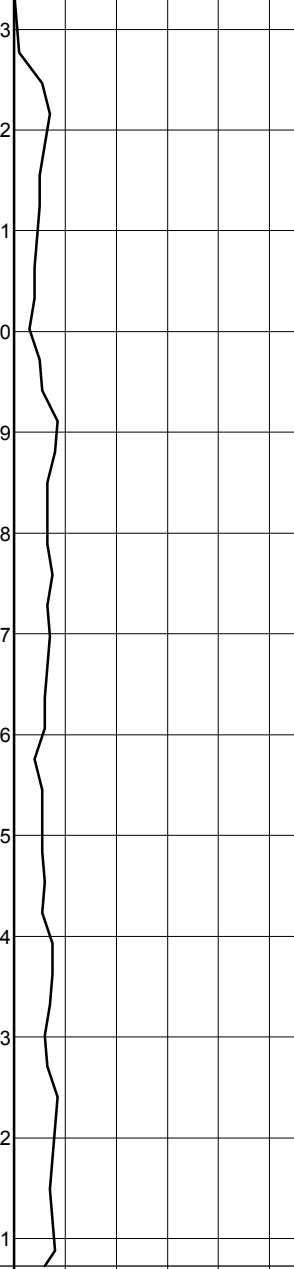
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+ 3, × 3: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

GTA-MTO 001 T:\PROJECTS\2009\09-1111-6014 (URS, HWY 69, HENVEY)\LOG\09-1111-6014.GPJ GAL-GTA.GDT 07/25/16

PROJECT 09-1111-6014		RECORD OF BOREHOLE No S502-24				SHEET 2 OF 2		METRIC									
W.P. 5005-10-01		LOCATION N 5083218.2; E 222959.0				ORIGINATED BY ID											
DIST HWY 69		BOREHOLE TYPE 106 mm I.D. Continuous Flight Hollow Stem Augers				COMPILED BY GRL											
DATUM Geodetic		DATE February 27 and 28, 2013				CHECKED BY CN											
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 (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
	--- CONTINUED FROM PREVIOUS PAGE ---																
174.0	SILT, trace clay Very loose Grey Wet		14	SS	WH		175										
16.3	SILT and SAND, trace gravel, trace clay Loose Grey Wet		15	SS	8		174										
172.6							173									5 45 47 3 Non-Plastic	
17.7	END OF BOREHOLE SPLIT-SPOON AND AUGER REFUSAL NOTE: 1. Water level in open borehole at a depth of 6.0 m below ground surface (Elev. 184.3 m) upon completion of drilling.		16	SS													

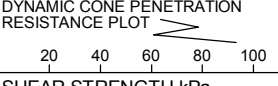
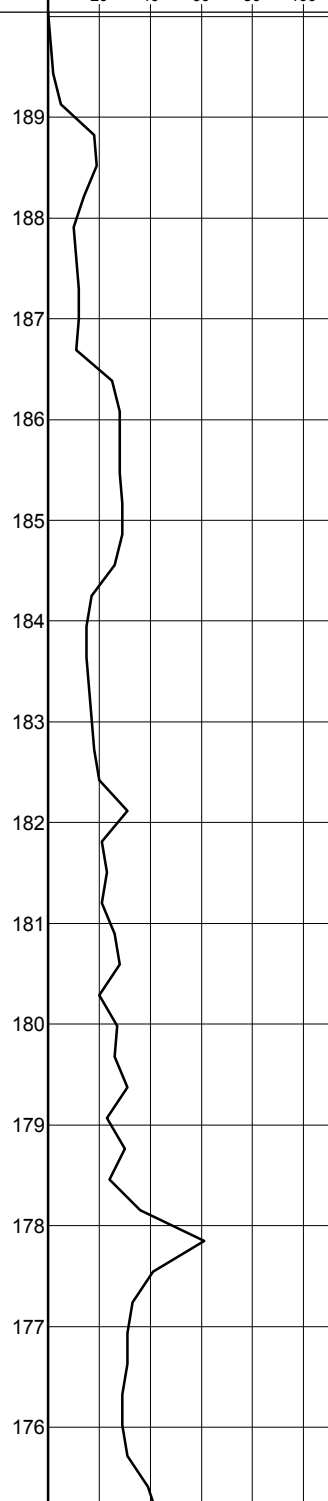
GTA-MTO 001 T:\PROJECTS\2009\09-1111-6014 (URS, HWY 69, HENVEY)\LOG\09-1111-6014.GPJ GAL-GTA.GDT 07/25/16

PROJECT <u>09-1111-6014</u>					RECORD OF DCPT No S502-DC02					SHEET 1 OF 1		METRIC					
W.P. <u>5005-10-01</u>		LOCATION <u>N 5083151.2 ; E 222966.7</u>		ORIGINATED BY <u>ID</u>													
DIST <u> </u> HWY <u>69</u>		BOREHOLE TYPE <u>Dynamic Cone Penetration Test</u>						COMPILED BY <u>AV</u>									
DATUM <u>Geodetic</u>		DATE <u>March 14, 2013</u>						CHECKED BY <u>JPD</u>									
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 (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa									
183.4	GROUND SURFACE						<div style="display: flex; justify-content: space-between;"> 20 40 60 80 100 20 40 60 80 100 </div> <div style="display: flex; justify-content: space-between;"> ○ UNCONFINED + FIELD VANE </div> <div style="display: flex; justify-content: space-between;"> ● QUICK TRIAXIAL × REMOULDED </div>										
0.0	Dynamic Cone Penetration Test (DCPT)						<div style="display: flex; justify-content: space-between;"> 20 40 60 80 100 20 40 60 </div> <div style="display: flex; justify-content: space-between;"> W_p W W_L </div> <div style="display: flex; justify-content: space-between;"> 20 40 60 20 40 60 </div>										
						183											
						182											
						181											
						180											
						179											
						178											
						177											
						176											
						175											
						174											
						173											
						172											
						171											
170.8	END OF DCPT Refusal to Further Penetration																
12.7																	

GTA-MTO 001 T:\PROJECTS\2009\09-1111-6014 (URS, HWY 69, HENVEY)\LOG\09-1111-6014.GPJ GAL-GTA.GDT 07/25/16



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

PROJECT <u>09-1111-6014</u>		RECORD OF DCPT No S502-DC09		SHEET 1 OF 2		METRIC					
W.P. <u>5005-10-01</u>		LOCATION <u>N 5083199.0 ; E 222951.4</u>		ORIGINATED BY <u>EHS</u>							
DIST <u> </u> HWY <u>69</u>		BOREHOLE TYPE <u>Dynamic Cone Penetration Test</u>		COMPILED BY <u>AV</u>							
DATUM <u>Geodetic</u>		DATE <u>March 4, 2013</u>		CHECKED BY <u>JPD</u>							
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT  SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED	PLASTIC LIMIT W _p NATURAL MOISTURE CONTENT W LIQUID LIMIT W _L WATER CONTENT (%)	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE							"N" VALUES
190.0 0.0	GROUND SURFACE Dynamic Cone Penetration Test (DCPT)										
											

GTA-MTO 001 T:\PROJECTS\2009\09-1111-6014 (URS, HWY 69, HENVEY)\LOG\09-1111-6014.GPJ GAL-GTA.GDT 07/25/16

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

RECORD OF BOREHOLE No ST-37

G.W.P. 5377-02-00 LOCATION Center of Straight Lake, Mowat Township, Co-ords: 5083027 N; 223022 E 1 OF 5
 DIST 54 HWY 69 BOREHOLE TYPE Portable Drilling Equipment - Wash Boring ORIGINATED BY JF
 DATUM Geodetic DATE 18 February 2006 - 21 February 2006 COMPILED BY SN
 PROJECT Highway 69 Route Selection Study, from 3.5 km North of HWY 559 to 3.8 km North of HWY 522 CHECKED BY IH
 JOB NO. TT53126

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	DEPTH m	ELEVATION SCALE ELEVATION m	DYNAMIC CONE PENETRATION RESISTANCE PLOT <div><div><div></div><div>20406080100</div></div><div>SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE</div></div>	PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH (m)	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES										WATER CONTENT (%) W _p — W — W _L
178.5 0.0	about 500 mm ICE														
178.0 0.5	WATER							178							
							1								
								177							
							2								
								176							
175.5 3.1	SILTY CLAY / SILT AND CLAY / CLAYEY SILT trace sand, trace organics grey, very soft to hard, medium plasticity, wet CI		1	SS	0		3								
								175							
							4								
								174							
							5								
								173							
							6								
								172							
							7								
								171							
			2	SS	4										

For SS2:
w_L=44%

054550

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

For SS2:
W_p=44%

0 5 45 50

2 OF 5

G.W.P. 5377-02-00 LOCATION Center of Straight Lake, Mowat Township, Co-ords: 5083027 N; 223022 E ORIGINATED BY JF
 DIST 54 HWY 69 BOREHOLE TYPE Portable Drilling Equipment - Wash Boring COMPILED BY SN
 DATUM Geodetic DATE 18 February 2006 - 21 February 2006 CHECKED BY IH
 PROJECT Highway 69 Route Selection Study, from 3.5 km North of HWY 559 to 3.8 km North of HWY 522 JOB NO. TT53126

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	DEPTH m	ELEVATION SCALE m	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH (m)	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES				SHEAR STRENGTH kPa			WATER CONTENT (%)				
									○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE							
	SILTY CLAY / SILT AND CLAY / CLAYEY SILT trace sand grey, soft to hard, medium plasticity, wet CI						170									
			3	SS	4			169								
								168								
			4	SS	4			167								
								166								
								165								
			5	SS	110/23			164								
								163								
			6	SS	5			162								
								161								

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

G.W.P. 5377-02-00 LOCATION Center of Straight Lake, Mowat Township, Co-ords: 5083027 N; 223022 E ORIGINATED BY JF
 DIST 54 HWY 69 BOREHOLE TYPE Portable Drilling Equipment - Wash Boring COMPILED BY SN
 DATUM Geodetic DATE 18 February 2006 - 21 February 2006 CHECKED BY IH
 PROJECT Highway 69 Route Selection Study, from 3.5 km North of HWY 559 to 3.8 km North of HWY 522 JOB NO. TT53126

[illegible]

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

RECORD OF BOREHOLE No ST-37

G.W.P. 5377-02-00 LOCATION Center of Straight Lake, Mowat Township, Co-ords: 5083027 N; 223022 E 4 OF 5
 DIST 54 HWY 69 BOREHOLE TYPE Portable Drilling Equipment - Wash Boring ORIGINATED BY JF
 DATUM Geodetic DATE 18 February 2006 - 21 February 2006 COMPILED BY SN
 PROJECT Highway 69 Route Selection Study, from 3.5 km North of HWY 559 to 3.8 km North of HWY 522 CHECKED BY IH
 JOB NO. TT53126

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	DEPTH m	ELEVATION m	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH (m)	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES				20	40	60	80	100		
153.9	SILTY CLAY / SILT AND CLAY / CLAYEY SILT trace sand and gravel grey, firm, high plasticity, wet CH						154								
24.6	SILT / SANDY SILT / SILT AND SAND with clay, trace sand grey, compact to very dense, low plasticity, wet CL-ML						25								
			10	SS	45		26								
							153								
							27								
			11	SS	12		151								
							28								
							150								
							29								
							149								
							30								
			12	SS	51		148								
							31								
							147								

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G.W.P. 5377-02-00 LOCATION Center of Straight Lake, Mowat Township, Co-ords: 5083027 N; 223022 E ORIGINATED BY JF
 DIST 54 HWY 69 BOREHOLE TYPE Portable Drilling Equipment - Wash Boring COMPILED BY SN
 DATUM Geodetic DATE 18 February 2006 - 21 February 2006 CHECKED BY IH
 PROJECT Highway 69 Route Selection Study, from 3.5 km North of HWY 559 to 3.8 km North of HWY 522 JOB NO. TT53126

[illegible]

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



APPENDIX B

Laboratory Test Results and Bedrock Core Photographs

TABLE B1
POINT LOAD TEST RESULTS ON ROCK SAMPLES

Foundation Element	Borehole Number	Run Number	Sample Depth (m)	Sample Elevation (m)	Bedrock Description	Test Type	Core Length (mm)	Core ⁽²⁾ Diameter (mm)	Is (50mm) (MPa)	Approx. UCS Value ⁽¹⁾ (MPa)
South Abutment	B502-05	1	5.0	180.7	Granitic Gneiss	Axial	42	45	10.9	153
	B502-05	1	5.0	180.7	Granitic Gneiss	Diametral	90	45	10.5	147
	B502-05	2	6.0	179.7	Granitic Gneiss	Axial	35	45	12.7	178
	B502-05	2	6.0	179.7	Granitic Gneiss	Diametral	71	45	8.9	125
	B502-05	3	6.3	179.4	Granitic Gneiss	Diametral	55	45	8.2	114
	B502-05	3	6.3	179.4	Granitic Gneiss	Axial	33	45	9.9	138
	B502-05	4	7.0	178.7	Granitic Gneiss	Diametral	68	45	9.8	138
	B502-05	4	7.0	178.7	Granitic Gneiss	Axial	35	45	11.0	154
	B502-05	4	7.4	178.3	Granitic Gneiss	Diametral	76	45	7.7	107
	B502-06	1	8.5	177.5	Granitic Gneiss	Diametral	60	45	5.7	80
	B502-06	2	8.8	177.2	Granitic Gneiss	Diametral	67	45	7.7	108
	B502-07	2	8.0	178.4	Granitic Gneiss	Axial	48	45	1.4	19
	B502-07	3	8.1	178.3	Granitic Gneiss	Diametral	140	45	2.9	41
	B502-07	4	9.4	177.0	Granitic Gneiss	Axial	45	45	9.2	128
	B502-07	4	9.5	176.9	Granitic Gneiss	Diametral	120	45	8.7	122
Between South Abutment and Pier 1	B502-09	1	3.7	174.9	Granitic Gneiss	Diametral	78	45	13.2	185
	B502-09	1	4.3	174.3	Granitic Gneiss	Diametral	112	45	7.5	105
	B502-09	1	4.3	174.3	Granitic Gneiss	Axial	56	45	6.4	89
	B502-09	2	5.5	173.1	Granitic Gneiss	Diametral	95	45	11.5	161
	B502-09	2	5.7	172.9	Granitic Gneiss	Axial	58	45	8.1	113
	B502-09	3	6.1	172.5	Granitic Gneiss	Diametral	71	45	10.5	147
	B502-09	3	6.1	172.5	Granitic Gneiss	Axial	30	45	8.4	117
	B502-15	1	3.7	174.9	Granitic Gneiss	Axial	45	45	10.3	144
	B502-15	1	3.8	174.8	Granitic Gneiss	Diametral	125	45	10.6	149
	B502-15	2	5.2	173.5	Granitic Gneiss	Axial	45	45	8.1	113
	B502-15	2	5.3	173.3	Granitic Gneiss	Diametral	100	45	9.2	129
	B502-18	1	7.7	170.9	Granitic Gneiss	Axial	49	45	8.3	116
	B502-18	1	7.9	170.7	Granitic Gneiss	Diametral	77	45	11.0	154
	B502-18	2	8.5	170.1	Granitic Gneiss	Diametral	56	45	6.2	87
	B502-18	2	8.6	170.0	Granitic Gneiss	Axial	34	45	5.0	70
	B502-18	3	9.4	169.2	Granitic Gneiss	Diametral	82	45	8.3	116
	B502-18	3	9.7	168.9	Granitic Gneiss	Axial	46	45	7.6	107
Pier 1	B502-01	1	12.9	165.7	Granitic Gneiss	Axial	47	47	8.0	112
	B502-01	1	12.9	165.7	Granitic Gneiss	Diametral	121	47	9.9	139
	B502-01	2	14.4	164.2	Granitic Gneiss	Axial	47	47	6.9	97
	B502-01	2	14.4	164.2	Granitic Gneiss	Diametral	106	47	14.3	200
	B502-01	2	15.2	163.4	Granitic Gneiss	Diametral	64	47	12.4	174
Pier 2	B502-10	1	50.6	127.9	Granitic Gneiss	Axial	34	45	9.3	130
	B502-10	1	50.7	127.8	Granitic Gneiss	Diametral	70	45	4.7	66
	B502-10	2	51.3	127.2	Granitic Gneiss	Axial	50	45	8.0	112
	B502-10	2	51.3	127.2	Granitic Gneiss	Diametral	97	45	7.3	102
	B502-10	3	53.0	125.5	Granitic Gneiss	Axial	40	45	9.4	132
	B502-10	3	53.0	125.5	Granitic Gneiss	Diametral	80	45	6.9	97
	B502-11	1	37.1	141.4	Granitic Gneiss	Axial	45	45	7.7	107
	B502-11	1	37.1	141.4	Granitic Gneiss	Diametral	97	45	10.1	142
	B502-11	1	37.1	141.4	Granitic Gneiss	Axial	49	45	8.1	113
	B502-11	2	38.7	139.8	Granitic Gneiss	Axial	31	45	3.4	48
	B502-11	3	39.8	138.7	Granitic Gneiss	Axial	40	45	1.7	24
	B502-11	3	39.8	138.7	Granitic Gneiss	Diametral	54	45	0.3	5

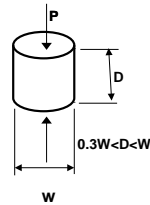
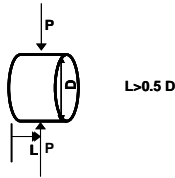
TABLE B1
POINT LOAD TEST RESULTS ON ROCK SAMPLES

Foundation Element	Borehole Number	Run Number	Sample Depth (m)	Sample Elevation (m)	Bedrock Description	Test Type	Core Length (mm)	Core Diameter (mm) ⁽²⁾	Is (50mm) (MPa)	Approx. UCS Value ⁽¹⁾ (MPa)
Pier 3	B502-02	2	29.0	149.6	Granitic Gneiss	Axial	34	48	9.5	133
	B502-02	2	29.0	149.6	Granitic Gneiss	Diametral	97	48	5.6	79
	B502-02	3	30.2	148.4	Granitic Gneiss	Axial	40	47	10.8	151
	B502-02	3	30.2	148.4	Granitic Gneiss	Diametral	85	47	8.9	124
	B502-02	3	31.3	147.3	Granitic Gneiss	Axial	40	47	9.4	131
	B502-02	3	31.3	147.3	Granitic Gneiss	Diametral	90	47	7.9	111
Between Pier 3 and Pier 4	B502-12	1	15.5	163.8	Granitic Gneiss	Diametral	105	45	7.9	111
	B502-12	1	15.6	163.7	Granitic Gneiss	Axial	53	45	9.5	133
	B502-12	3	17.3	162.0	Granitic Gneiss	Diametral	115	45	4.7	65
Pier 4	B502-13	1	16.4	167.1	Granitic Gneiss	Diametral	160	45	7.2	101
	B502-13	2	17.2	166.3	Granitic Gneiss	Diametral	115	45	7.5	105

⁽¹⁾ $IS_{50} \times K$, from ASTM Designation: D 5731 "Standard Test Method for Determination of the Point Load Strength Index of Rock and Application to Rock Strength Classifications". A value of $K = 14$ has been used based on 14 UCS tests for both the SBL and NBL bridges.

DIAMETRAL SPECIMEN SHAPE REQUIREMENTS

note: Diametral tests are perpendicular to core axis
(planes of weakness)



AXIAL SPECIMEN SHAPE REQUIREMENTS

note: Axial tests are parallel to core axis
(planes of weakness)

Compiled By: MCK/MT
Checked By: AB
Reviewed By: JPD/JMAC

TABLE B2-1
SUMMARY OF UNIAXIAL COMPRESSIVE STRENGTH TEST RESULTS
STRAIGHT LAKE NBL BRIDGE
HIGHWAY 69 GWP 5404-05-00; WP 5146-08-01

Borehole Number (Core Run)	Sample Depth (m)	Sample Elevation (m)	Rock Type	Core Diameter (mm)	Uniaxial Compressive Strength (MPa)
B502-01 (1)	13.1	165.5	Granitic Gneiss	47.4	68.1
B502-09 (3)	6.2	172.4	Granitic Gneiss	47.6	85.1
B502-11 (3)	39.1	139.4	Granitic Gneiss	47.4	22.7
B502-13 (2)	16.6	166.9	Granitic Gneiss	47.5	110.6

Compiled By: MTReviewed By: AB

Table B2-2

UNCONFINED COMPRESSION TEST (UC)**ASTM D 7012-07****SAMPLE IDENTIFICATION**

PROJECT NUMBER	09-1111-6014	SAMPLE NUMBER	Run 1
BOREHOLE NUMBER	B502-01	SAMPLE DEPTH, m	13.00-13.12

TEST CONDITIONS

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

SPECIMEN INFORMATION

SAMPLE HEIGHT, cm	10.56	WATER CONTENT, (specimen) %	0.05
SAMPLE DIAMETER, cm	4.74	UNIT WEIGHT, kN/m ³	27.60
SAMPLE AREA, cm ²	17.63	DRY UNIT WT., kN/m ³	27.59
SAMPLE VOLUME, cm ³	186.11	SPECIFIC GRAVITY	-
WET WEIGHT, g	524.00	VOID RATIO	-
DRY WEIGHT, g	523.74		

VISUAL INSPECTION**FAILURE SKETCH****TEST RESULTS**

STRAIN AT FAILURE, %	-	COMPRESSIVE STRESS, MPa	68.1
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REMARKS:	DATE:	3/15/2013
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Table B2-3

UNCONFINED COMPRESSION TEST (UC)**ASTM D 7012-07****SAMPLE IDENTIFICATION**

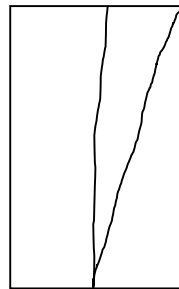
PROJECT NUMBER	09-1111-6014	SAMPLE NUMBER	Run 3
BOREHOLE NUMBER	B502-09	SAMPLE DEPTH, m	6.10-6.30

TEST CONDITIONS

MACHINE SPEED, mm/min	-	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST, min	>2 <15	L/D	2.11

SPECIMEN INFORMATION

SAMPLE HEIGHT, cm	10.06	WATER CONTENT, (specimen) %	0.07
SAMPLE DIAMETER, cm	4.76	UNIT WEIGHT, kN/m ³	26.96
SAMPLE AREA, cm ²	17.78	DRY UNIT WT., kN/m ³	26.94
SAMPLE VOLUME, cm ³	178.85	SPECIFIC GRAVITY	-
WET WEIGHT, g	491.80	VOID RATIO	-
DRY WEIGHT, g	491.46		

VISUAL INSPECTION**FAILURE SKETCH****TEST RESULTS**

STRAIN AT FAILURE, %	-	COMPRESSIVE STRESS, MPa	85.1
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REMARKS:

DATE:

4/17/2014

Table B2-4

UNCONFINED COMPRESSION TEST (UC)**ASTM D 7012-07****SAMPLE IDENTIFICATION**

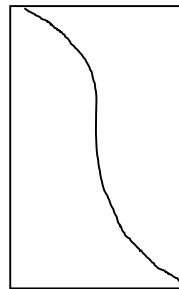
PROJECT NUMBER	09-1111-6014	SAMPLE NUMBER	Run 3
BOREHOLE NUMBER	B502-11	SAMPLE DEPTH, m	38.98-39.16

TEST CONDITIONS

MACHINE SPEED, mm/min	-	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST, min	>2 <15	L/D	2.10

SPECIMEN INFORMATION

SAMPLE HEIGHT, cm	9.97	WATER CONTENT, (specimen) %	1.30
SAMPLE DIAMETER, cm	4.74	UNIT WEIGHT, kN/m ³	26.70
SAMPLE AREA, cm ²	17.63	DRY UNIT WT., kN/m ³	26.35
SAMPLE VOLUME, cm ³	175.80	SPECIFIC GRAVITY	-
WET WEIGHT, g	478.74	VOID RATIO	-
DRY WEIGHT, g	472.60		

VISUAL INSPECTION**FAILURE SKETCH****TEST RESULTS**

STRAIN AT FAILURE, %	-	COMPRESSIVE STRESS, MPa	22.7
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REMARKS:

DATE:

4/17/2014

Table B2-5

UNCONFINED COMPRESSION TEST (UC)**ASTM D 7012-07****SAMPLE IDENTIFICATION**

PROJECT NUMBER	09-1111-6014	SAMPLE NUMBER	Run 2
BOREHOLE NUMBER	B502-13	SAMPLE DEPTH, m	16.52-16.74

TEST CONDITIONS

MACHINE SPEED, mm/min	-	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST,min	>2 <15	L/D	2.20

SPECIMEN INFORMATION

SAMPLE HEIGHT, cm	10.47	WATER CONTENT, (specimen) %	0.08
SAMPLE DIAMETER, cm	4.75	UNIT WEIGHT, kN/m ³	26.15
SAMPLE AREA, cm ²	17.70	DRY UNIT WT., kN/m ³	26.13
SAMPLE VOLUME, cm ³	185.23	SPECIFIC GRAVITY	-
WET WEIGHT, g	494.15	VOID RATIO	-
DRY WEIGHT, g	493.75		

VISUAL INSPECTION**FAILURE SKETCH****TEST RESULTS**

STRAIN AT FAILURE, %	-	COMPRESSIVE STRESS, MPa	110.6
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REMARKS:

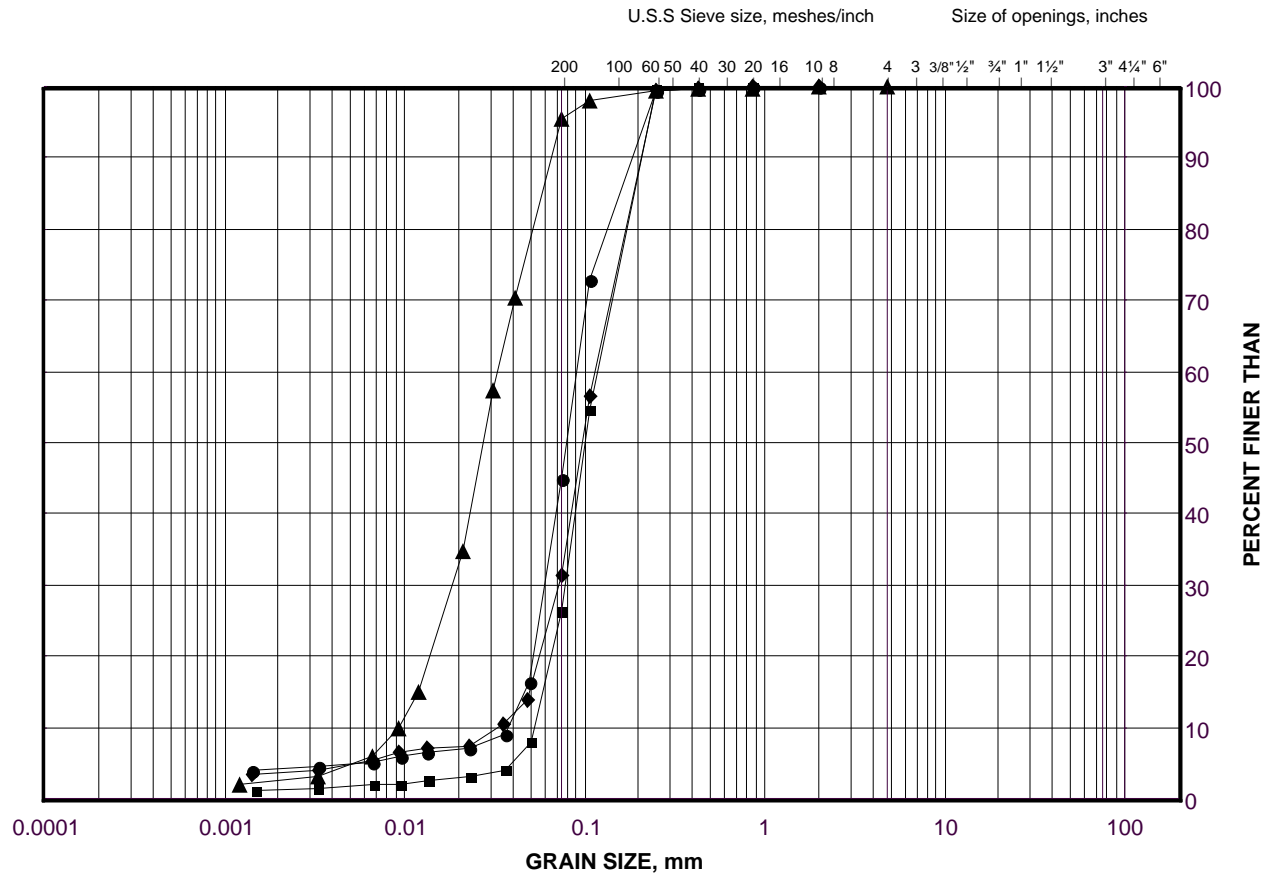
DATE:

4/17/2014

GRAIN SIZE DISTRIBUTION

Silt to Silty Sand

FIGURE B1-A



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	B502-04	3	183.9
■	B502-06	3	184.3
◆	B502-05	3	183.9
▲	B502-01	8	166.4

Project Number: 09-1111-6014

Checked By: AB/JMAC

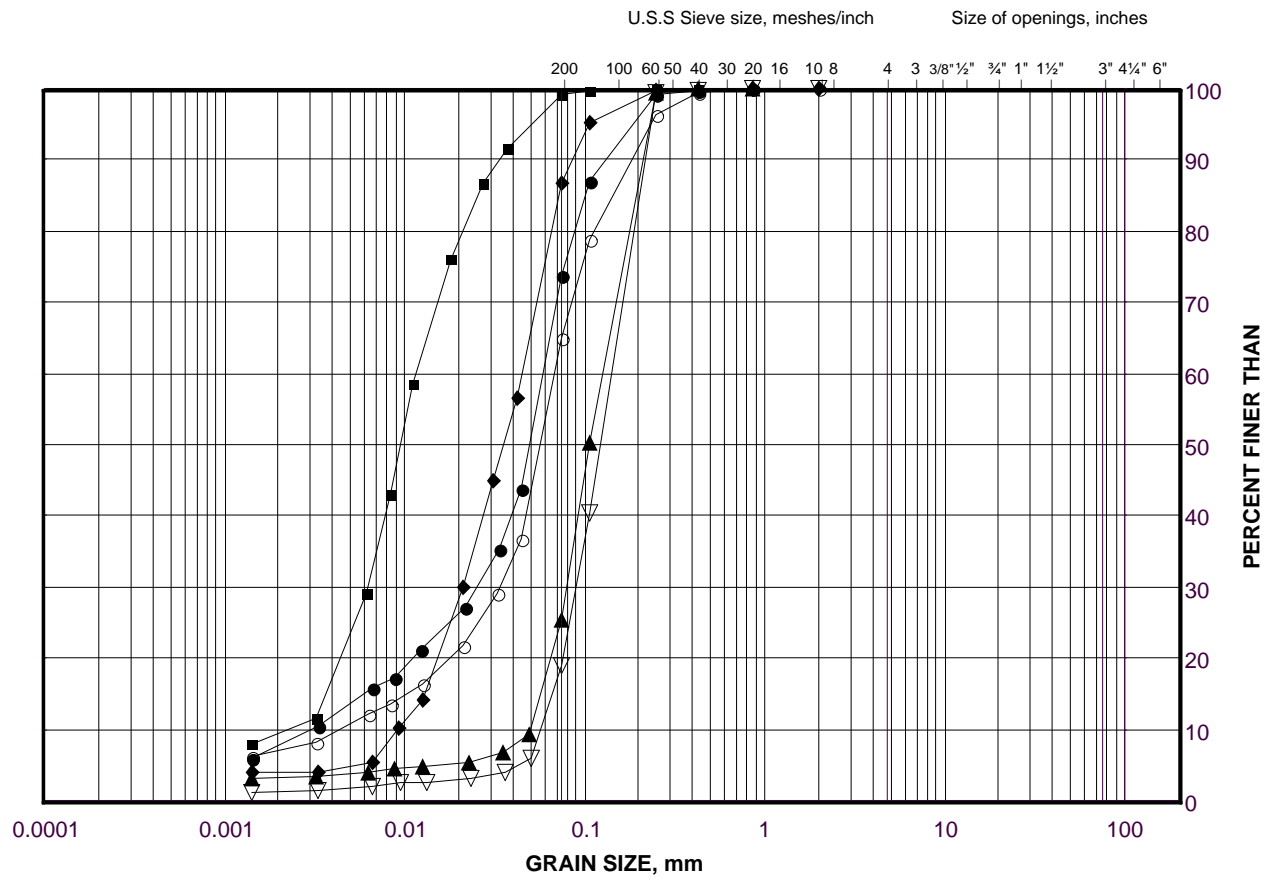
Golder Associates

Date: 28-Jun-16

GRAIN SIZE DISTRIBUTION

Silt to Sand

FIGURE B1-B



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

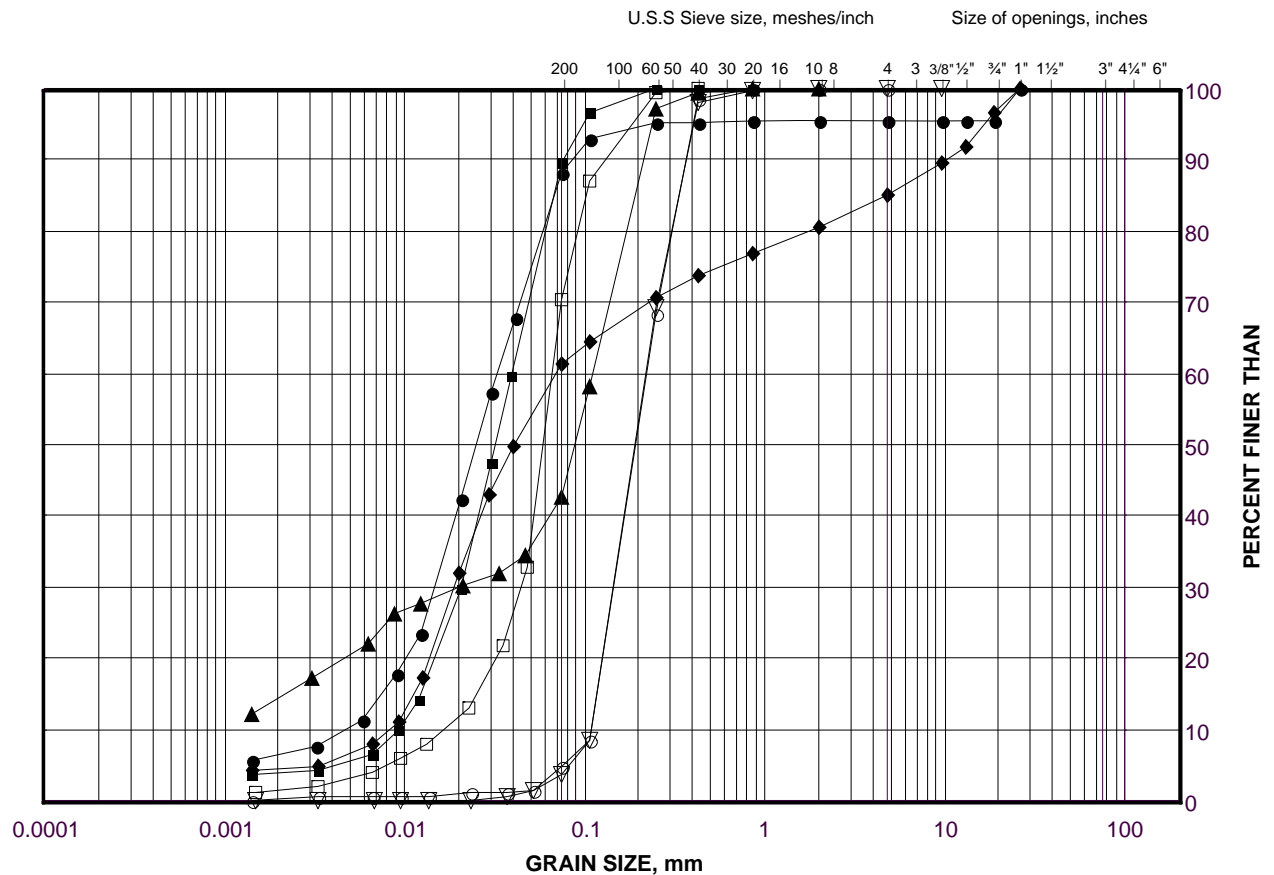
LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	B502-11	10	167.5
■	B502-10	16	155.6
◆	B502-10	18	151.1
▲	B502-07	3	184.7
▽	B502-08	3	184.6
○	B502-10	9	166.3

GRAIN SIZE DISTRIBUTION

Silt to Sand

FIGURE B1-C



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	B502-13	13	171.2
■	B502-11	18	153.8
◆	B502-11	22	141.8
▲	B502-16	3	173.4
▽	B502-13	5	180.9
○	S502-20	5	180.8
□	S502-20	6	180.0

Project Number: 09-1111-6014

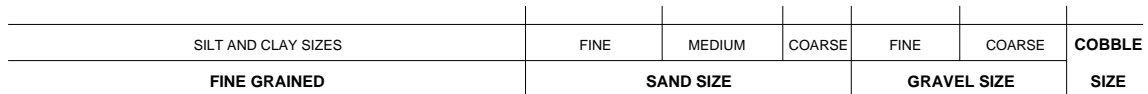
Checked By: AB/JMAC

Golder Associates

Date: 26-Nov-15

Silt to Silty Sand

FIGURE B1-D



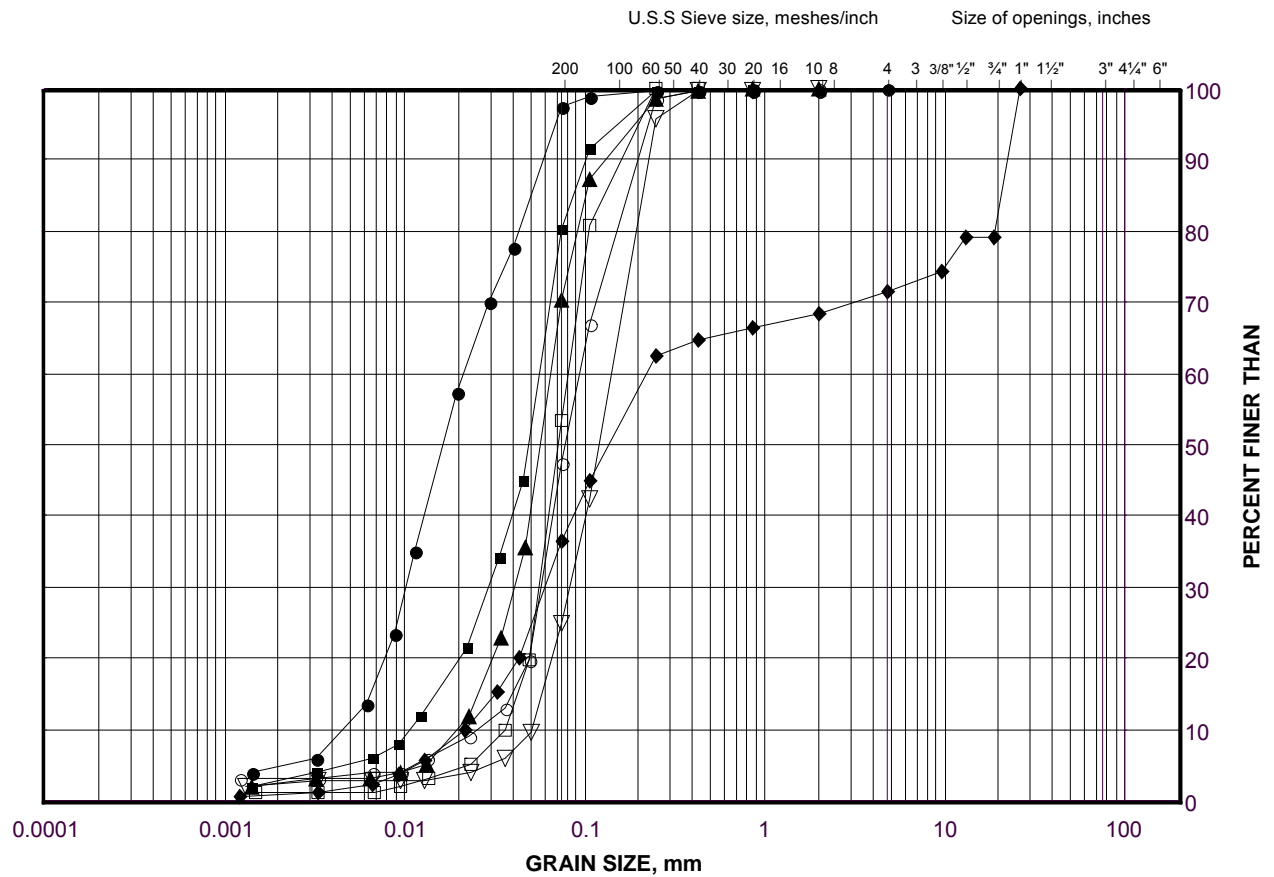
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	S502-20	11B	172.9
■	S502-20	12	171.6
◆	B502-02	16	154.4
▲	B502-11	20	147.7
▽	B502-02	8	168.1

Date: 30-Jun-16

GRAIN SIZE DISTRIBUTION

Silt to Silty Sand

FIGURE B1-E



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	S502-18	12	168.8
■	S502-18	13	167.3
◆	B502-02	17	151.3
▲	B502-14	3	184.5
▽	S502-22	4	185.4
○	S502-18	4	178.7
□	B502-14	8	178.4

Project Number: 09-1111-6014

Checked By: AB/JMAC

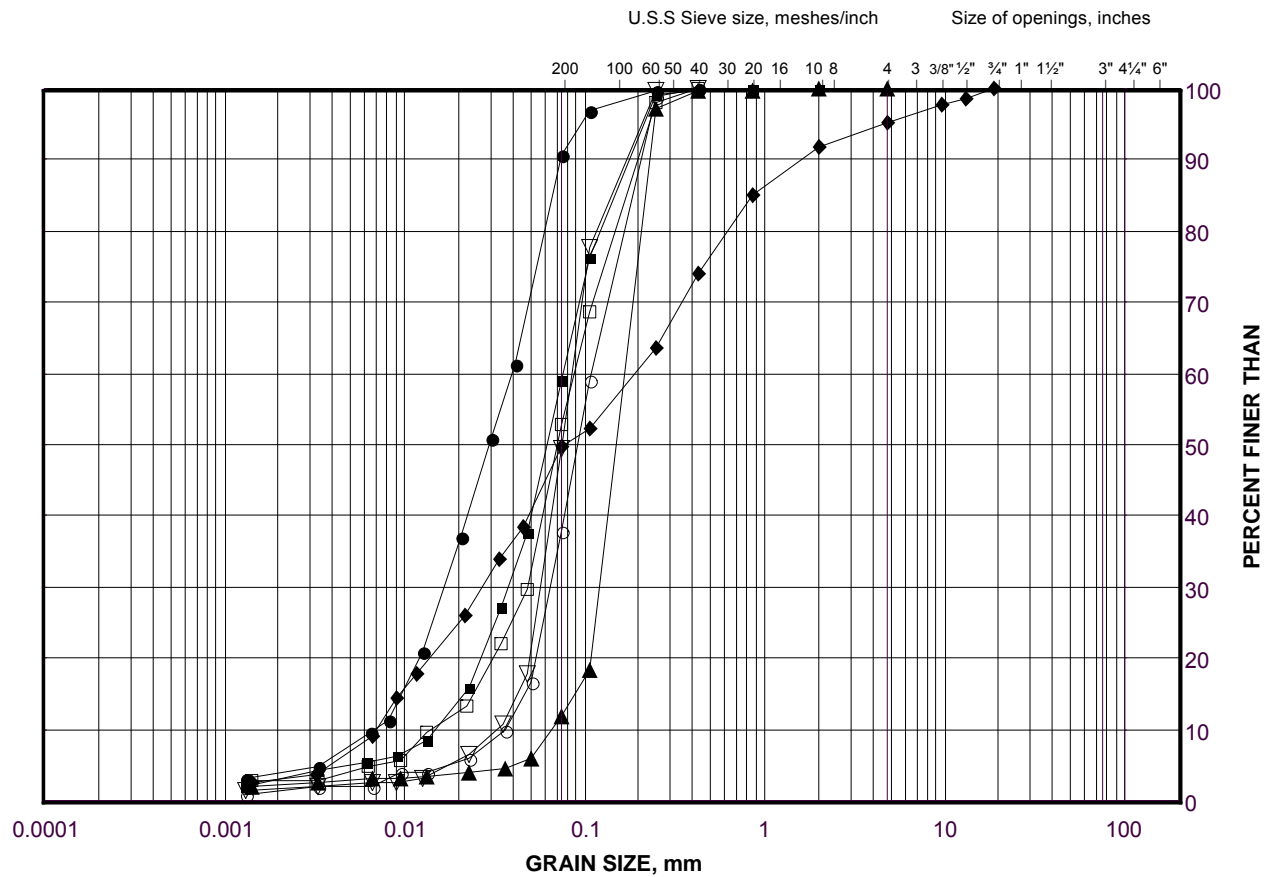
Golder Associates

Date: 07-Jan-16

GRAIN SIZE DISTRIBUTION

Silt to Silty Sand

FIGURE B1-F



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	S502-22	13	174.0
■	S502-22	15	170.9
◆	S502-24	15	173.2
▲	S502-24	4	187.7
▽	S502-22	5	184.7
○	S502-24	8	183.9
□	S502-24	9	182.4

Project Number: 09-1111-6014

Checked By: AB/JMAC

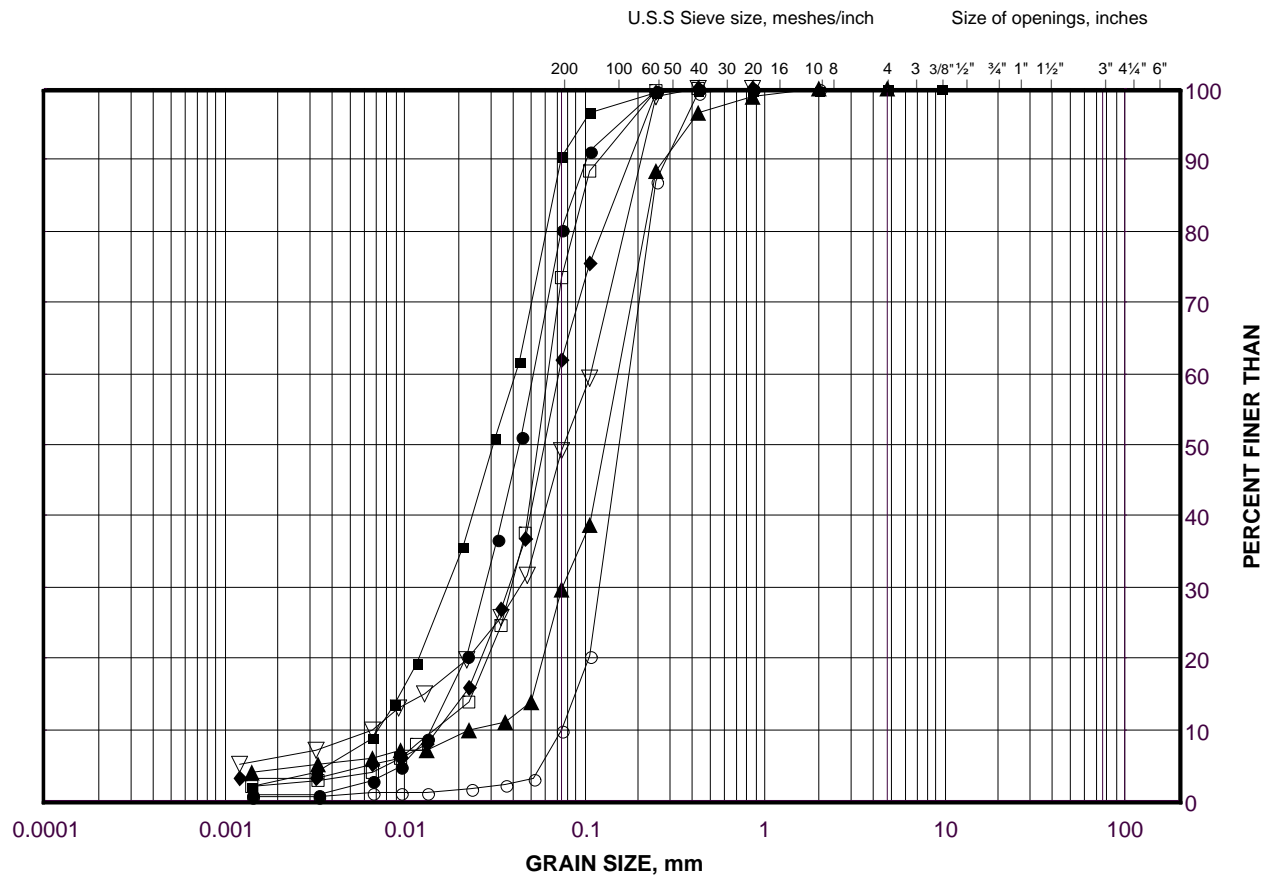
Golder Associates

Date: 08-Jan-16

GRAIN SIZE DISTRIBUTION

Silt to Silty Sand

FIGURE B1-G



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	S502-21	13	173.6
■	S502-08	14	175.1
◆	S502-08	16	172.0
▲	S502-21	3	185.7
▽	S502-21	7	182.7
○	S502-08	7	185.8
□	S502-08	8	184.2

Project Number: 09-1111-6014

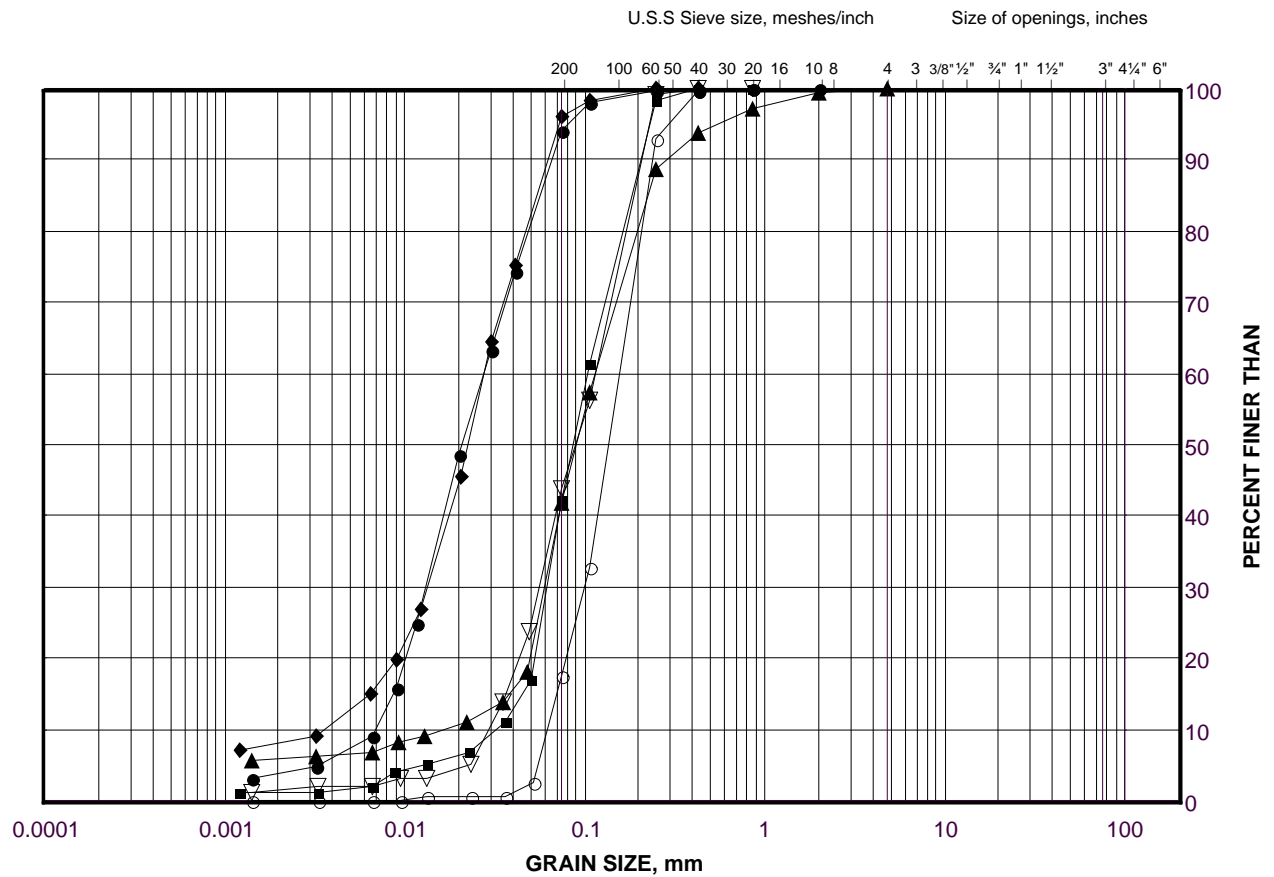
Checked By: _____

Golder Associates

Date: 12-Apr-16

Silt to Silty Sand

FIGURE B1-H



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	S502-20A	10	175.2
■	S502-23	10	179.3
◆	S502-18	11B	170.1
▲	S502-23	3	186.9
▽	S502-23	6	184.6
○	S502-23	7	183.8

Project Number: 09-1111-6014

Checked By: _____

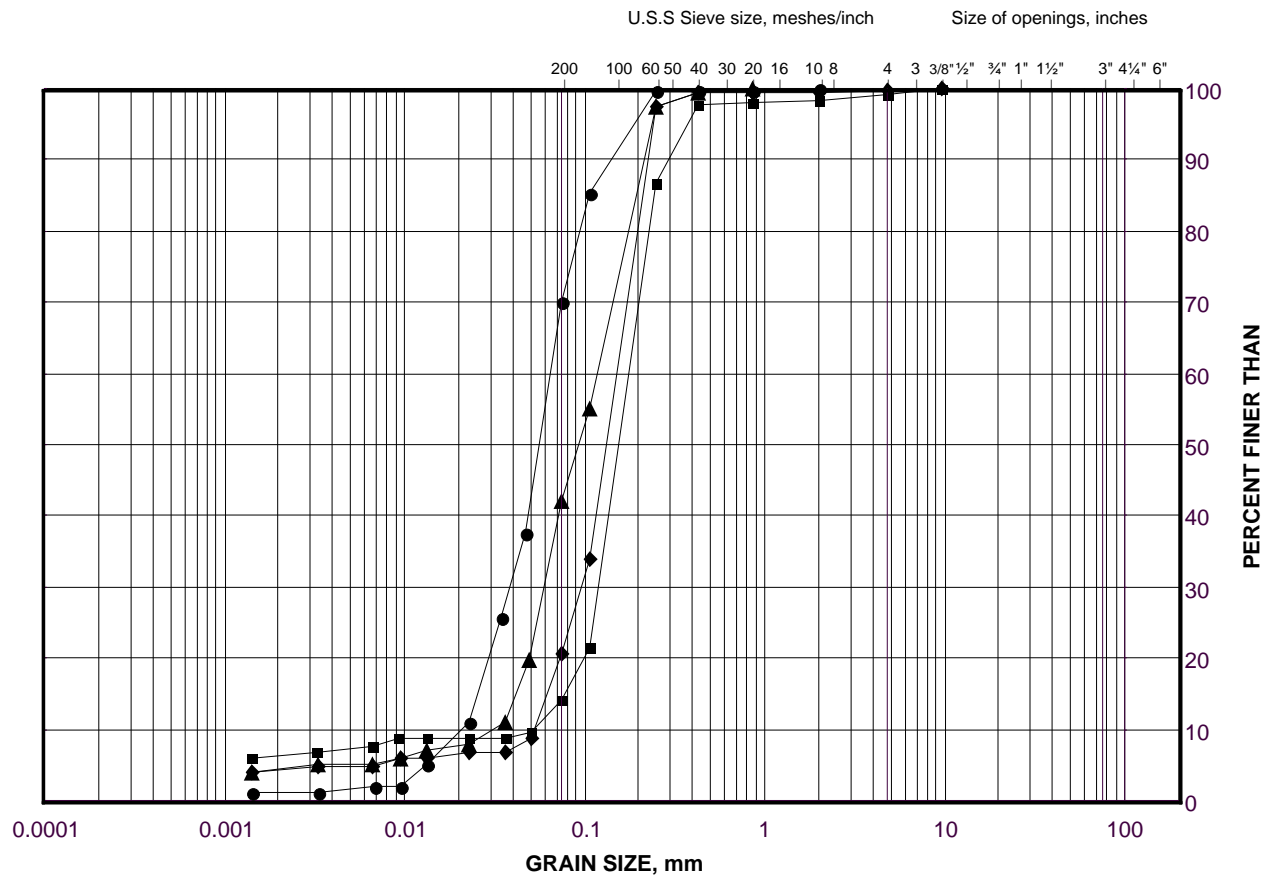
Golder Associates

Date: 13-Apr-16

GRAIN SIZE DISTRIBUTION

Silt and Sand

FIGURE B1-I



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	S502-12	11	181.1
■	S502-12	4	189.5
◆	S502-12	6A	188.1
▲	S502-12	7	187.2

Project Number: 09-1111-6014

Checked By: _____

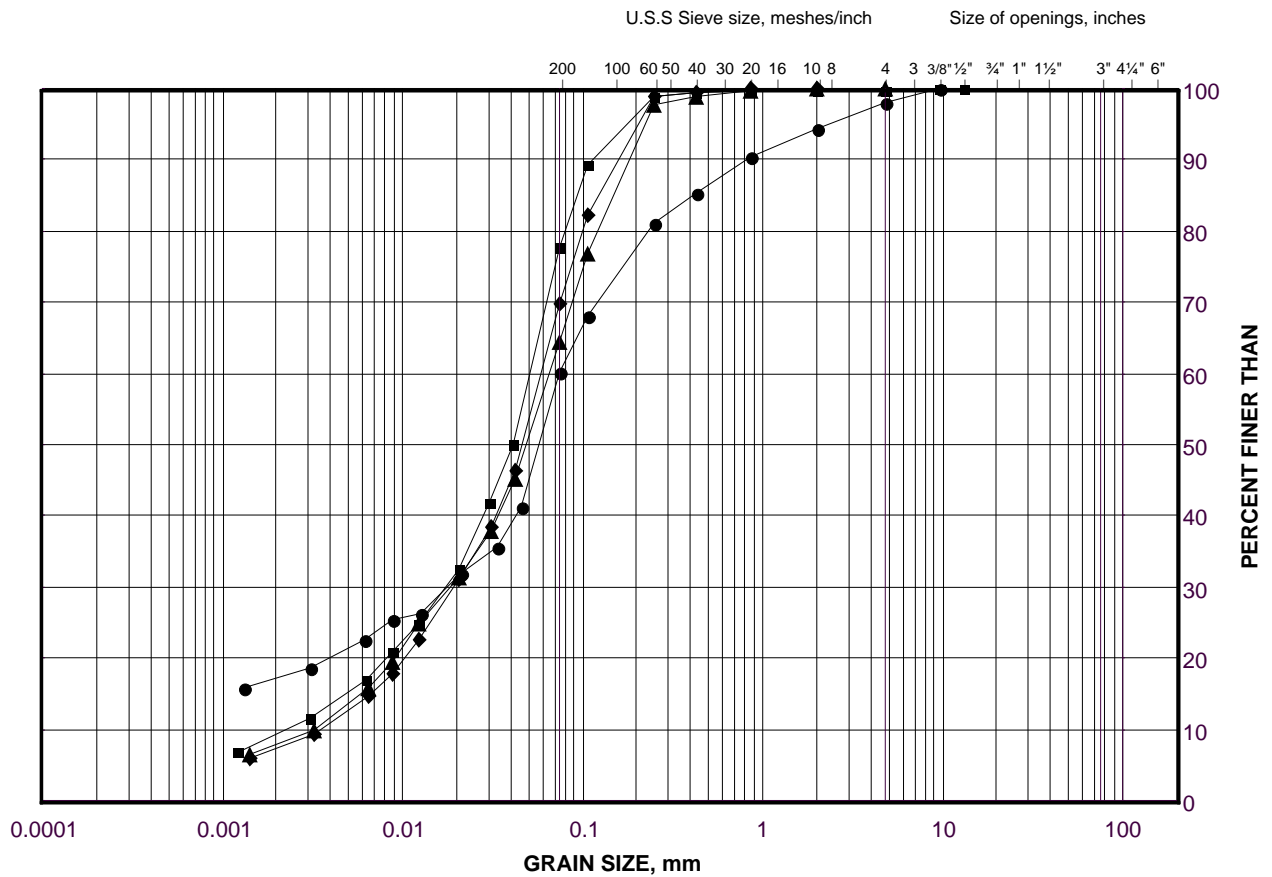
Golder Associates

Date: 19-May-16

GRAIN SIZE DISTRIBUTION

Organic Silt to Organic Silt and Sand

FIGURE B1-J



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

LEGEND

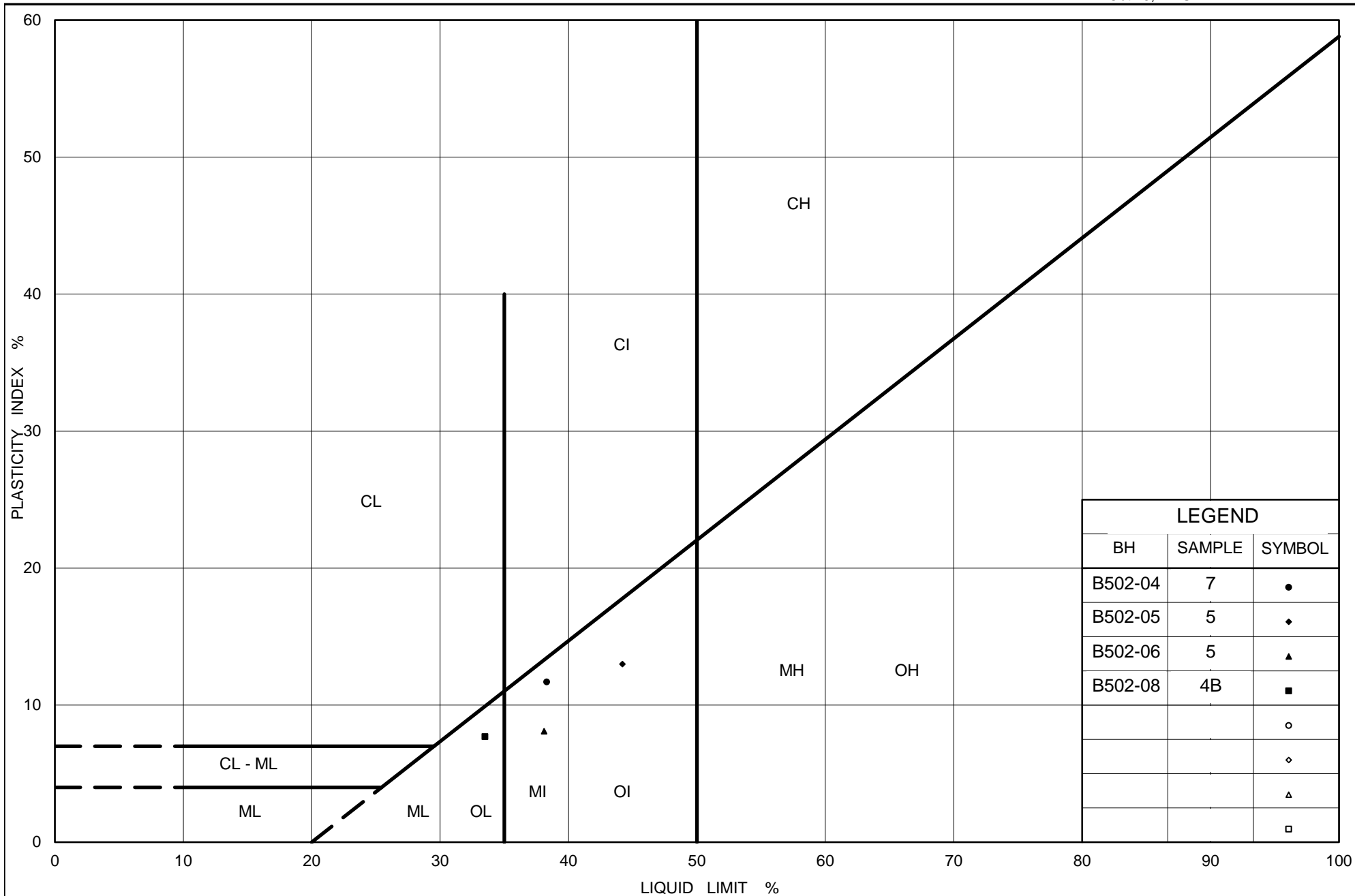
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	B502-07	5	183.2
■	B502-05	5	182.4
◆	B502-06	5	182.7
▲	B502-04	7	180.8

Project Number: 09-1111-6014

Checked By: AB/JMAC

Golder Associates

Date: 28-Jun-16



Ministry of Transportation

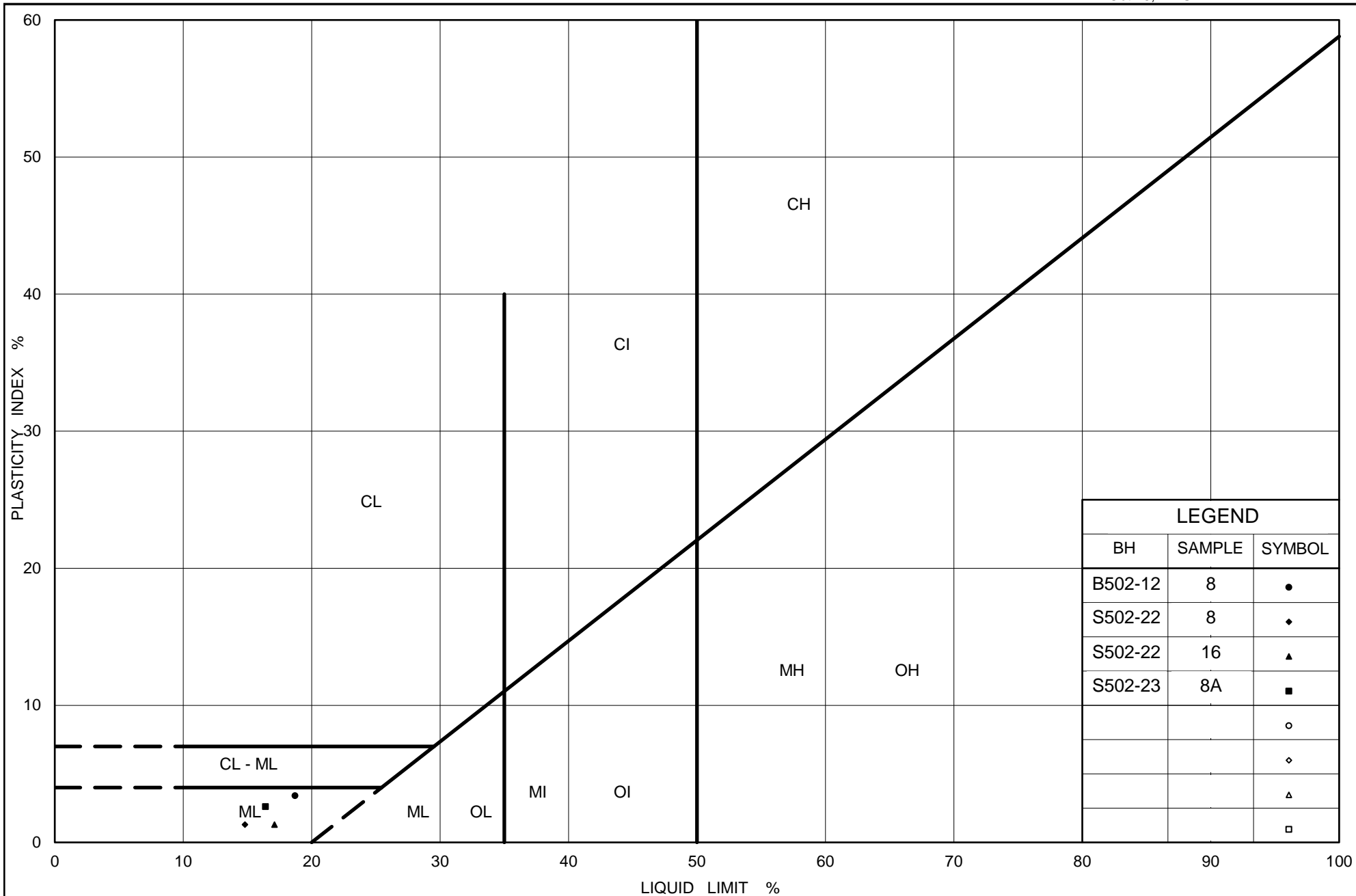
Ontario

PLASTICITY CHART Organic Silt

Figure No. B2-A

Project No. 09-1111-6014

Checked By: AB/JMAC



Ministry of Transportation

Ontario

PLASTICITY CHART Silt (Interlayer) to Silt and Sand

Figure No. B2-B

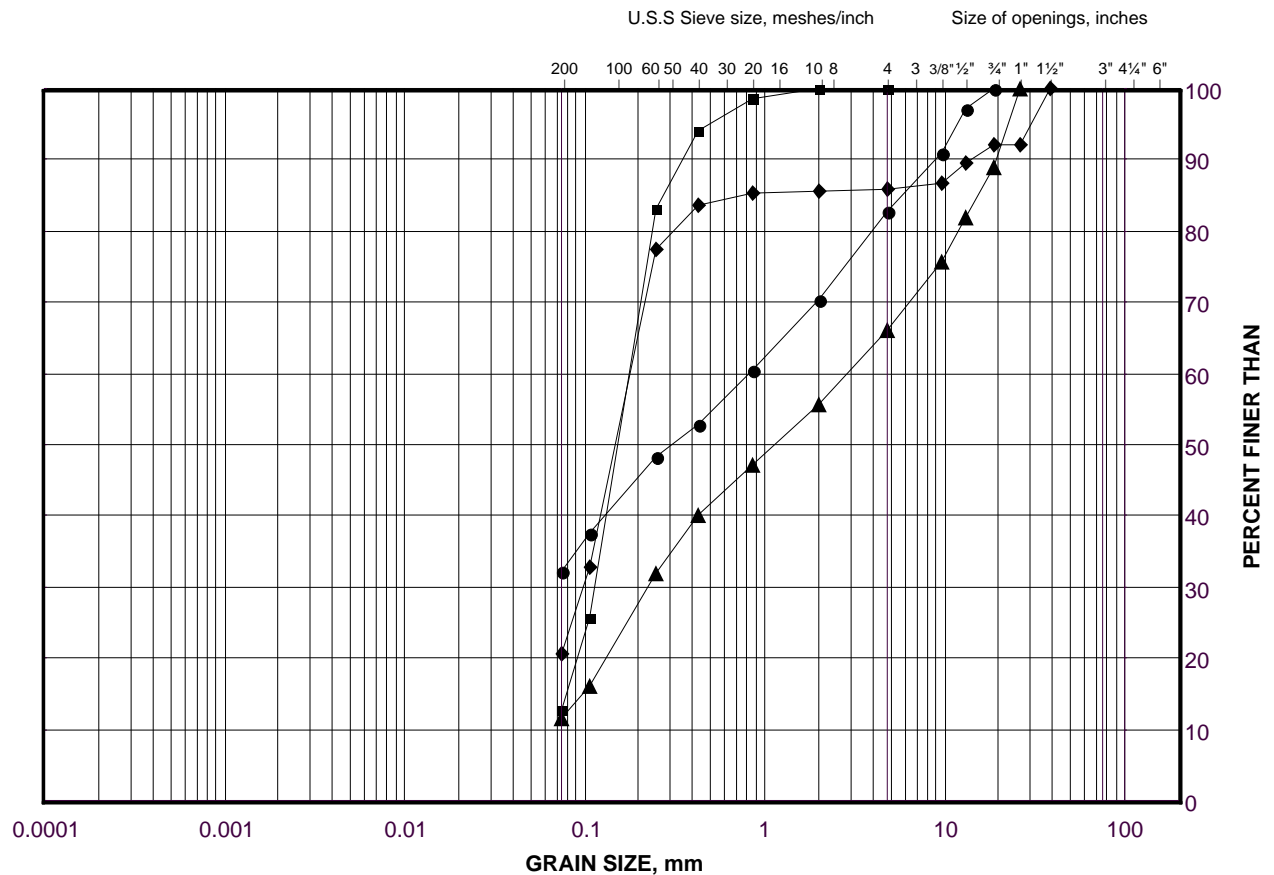
Project No. 09-1111-6014

Checked By: AB/JMAC

GRAIN SIZE DISTRIBUTION

Silty Sand to Sand and Gravel

FIGURE B3



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

LEGEND

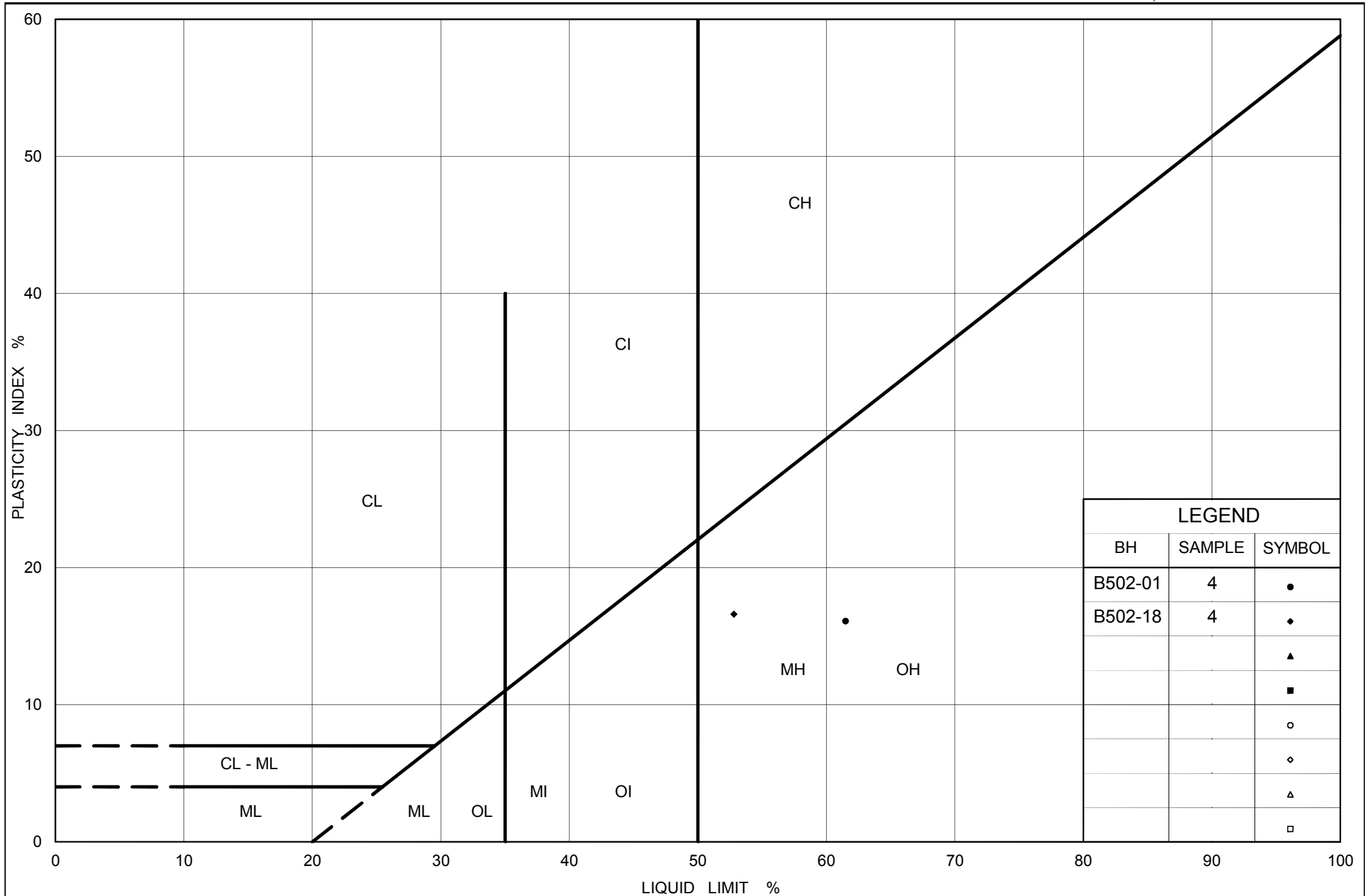
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	B502-04	11	177.9
■	B502-10	20	145.0
◆	B502-10	22	138.9
▲	B502-06	8	180.5

Project Number: 09-1111-6014

Checked By: AB/JMAC

Golder Associates

Date: 30-Jun-16



Ministry of Transportation

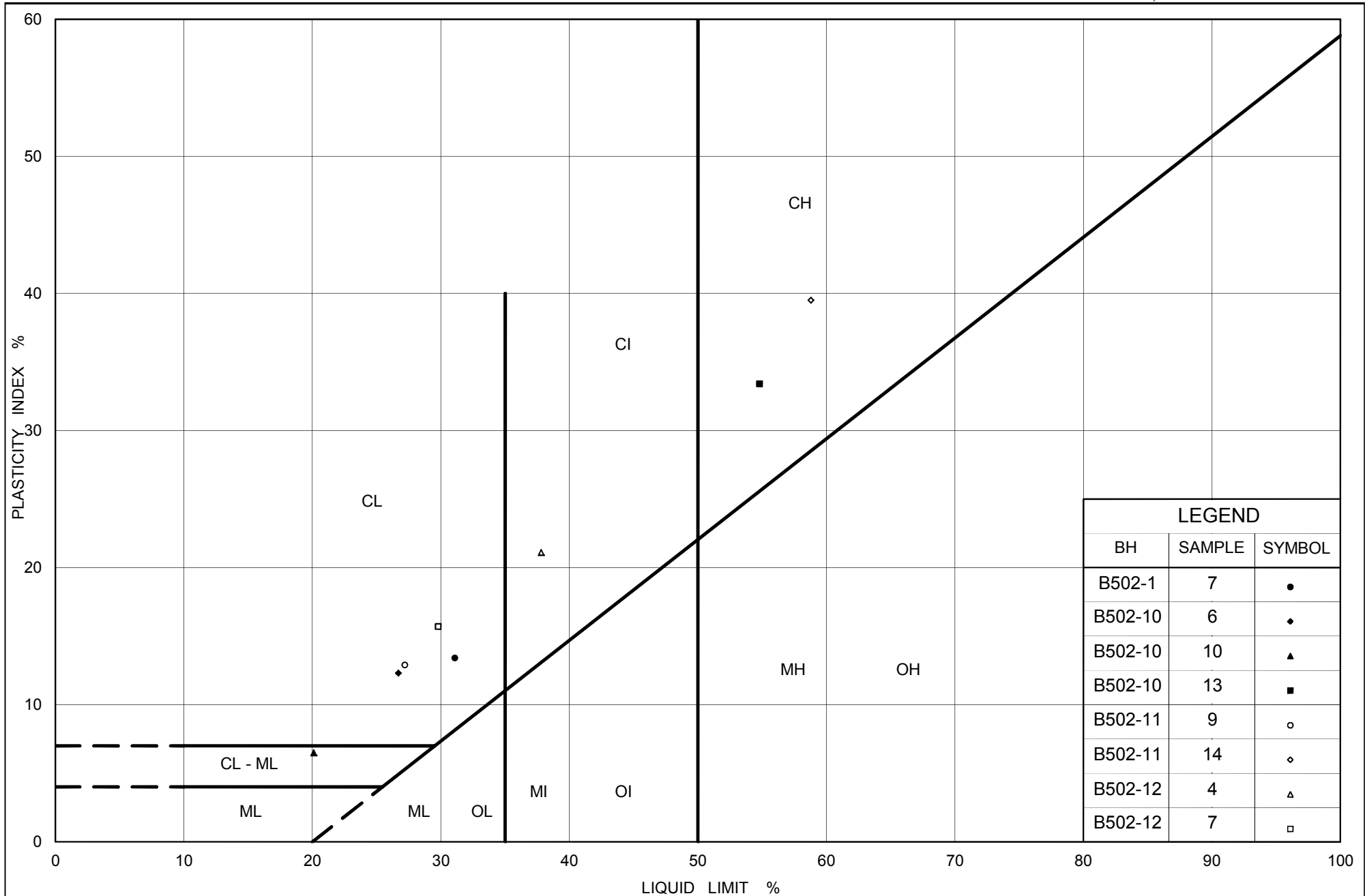
Ontario

PLASTICITY CHART Organic Silt

Figure No. B4

Project No. 09-1111-6014

Checked By: AB/JMAC



Ministry of Transportation

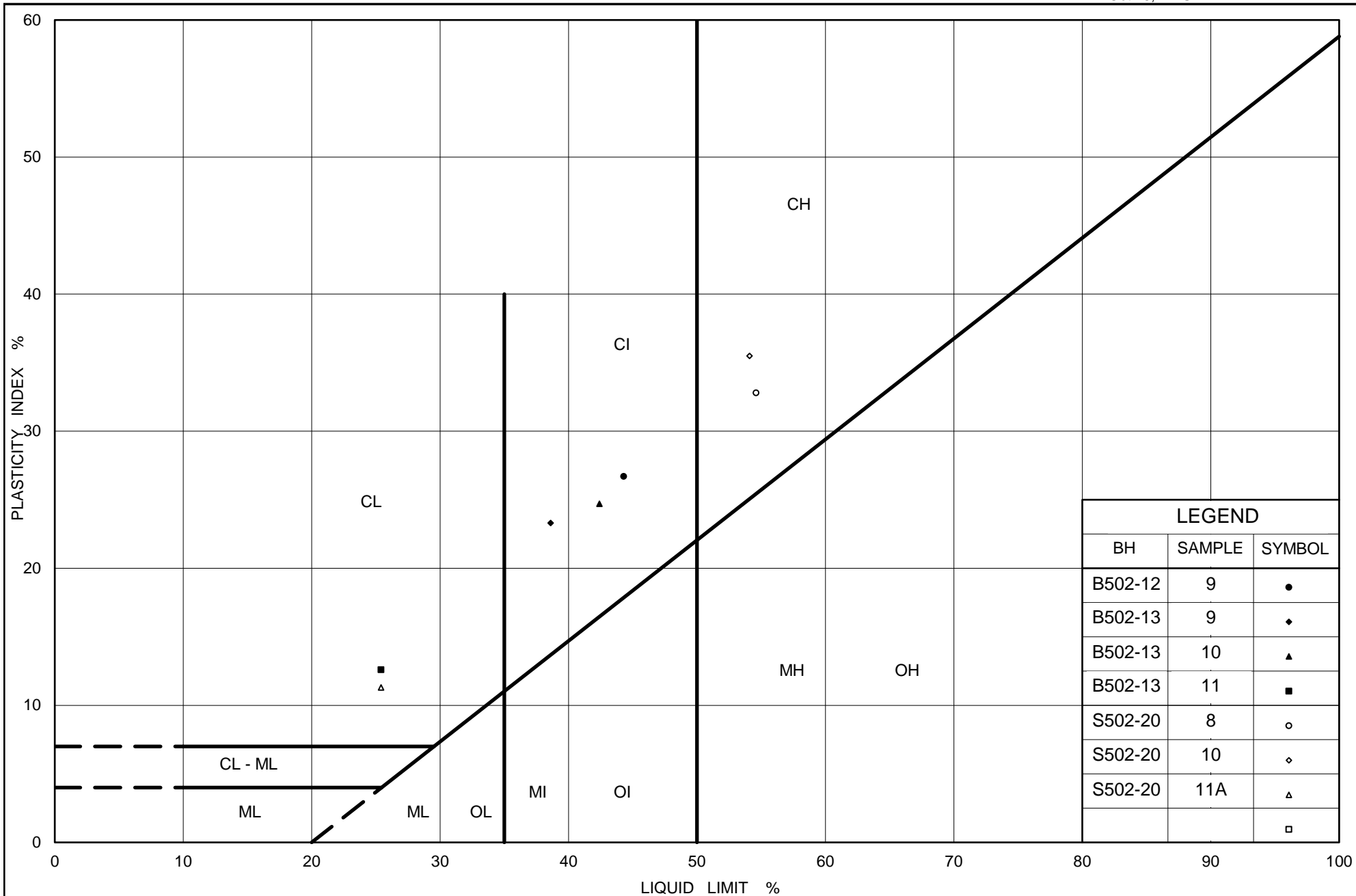
Ontario

PLASTICITY CHART Clayey Silt to Clay

Figure No. B5-A

Project No. 09-1111-6014

Checked By: AB/JMAC



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Ontario

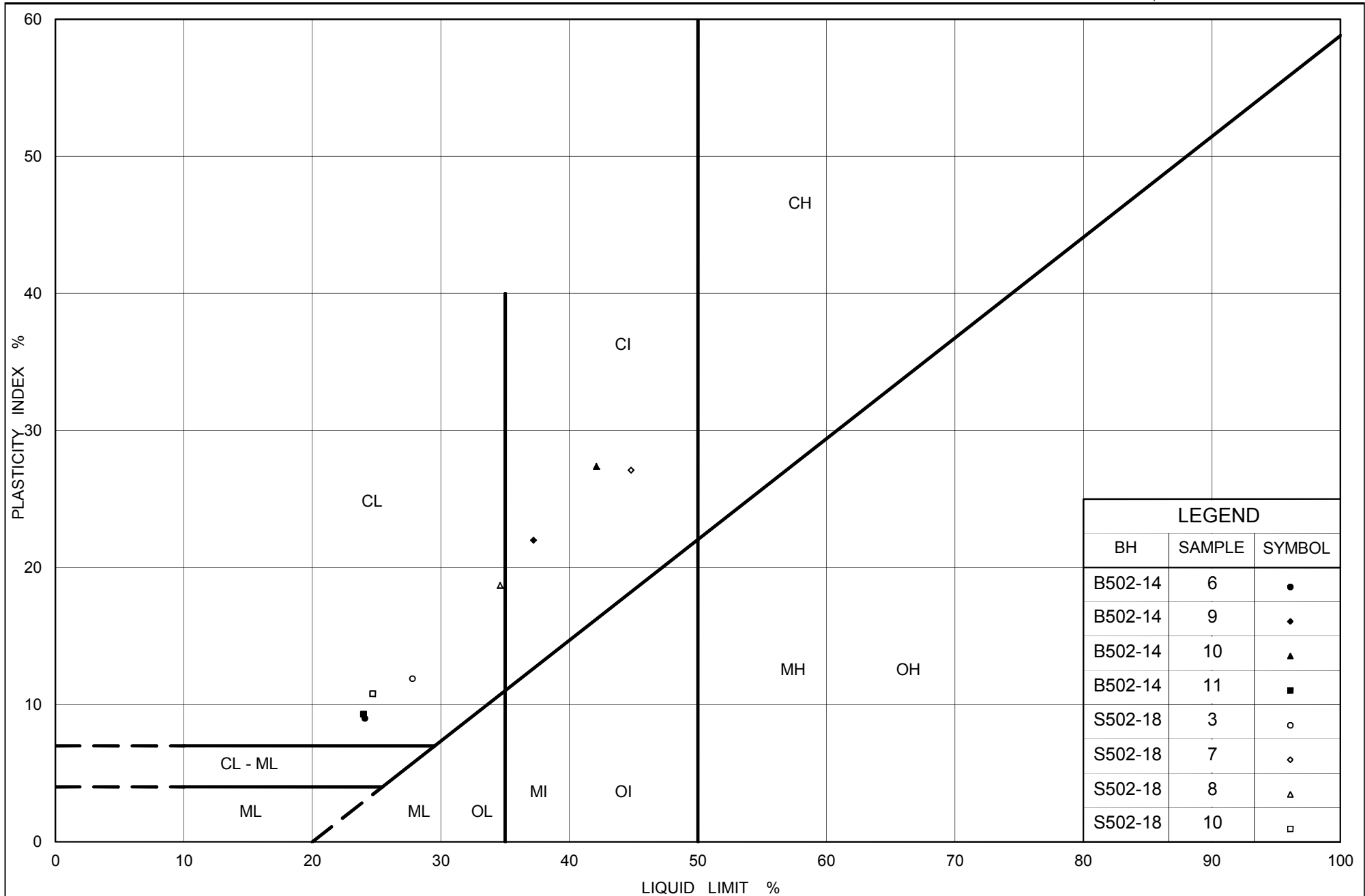
PLASTICITY CHART

Clayey Silt to Clay

Figure No. B5-B

Project No. 09-1111-6014

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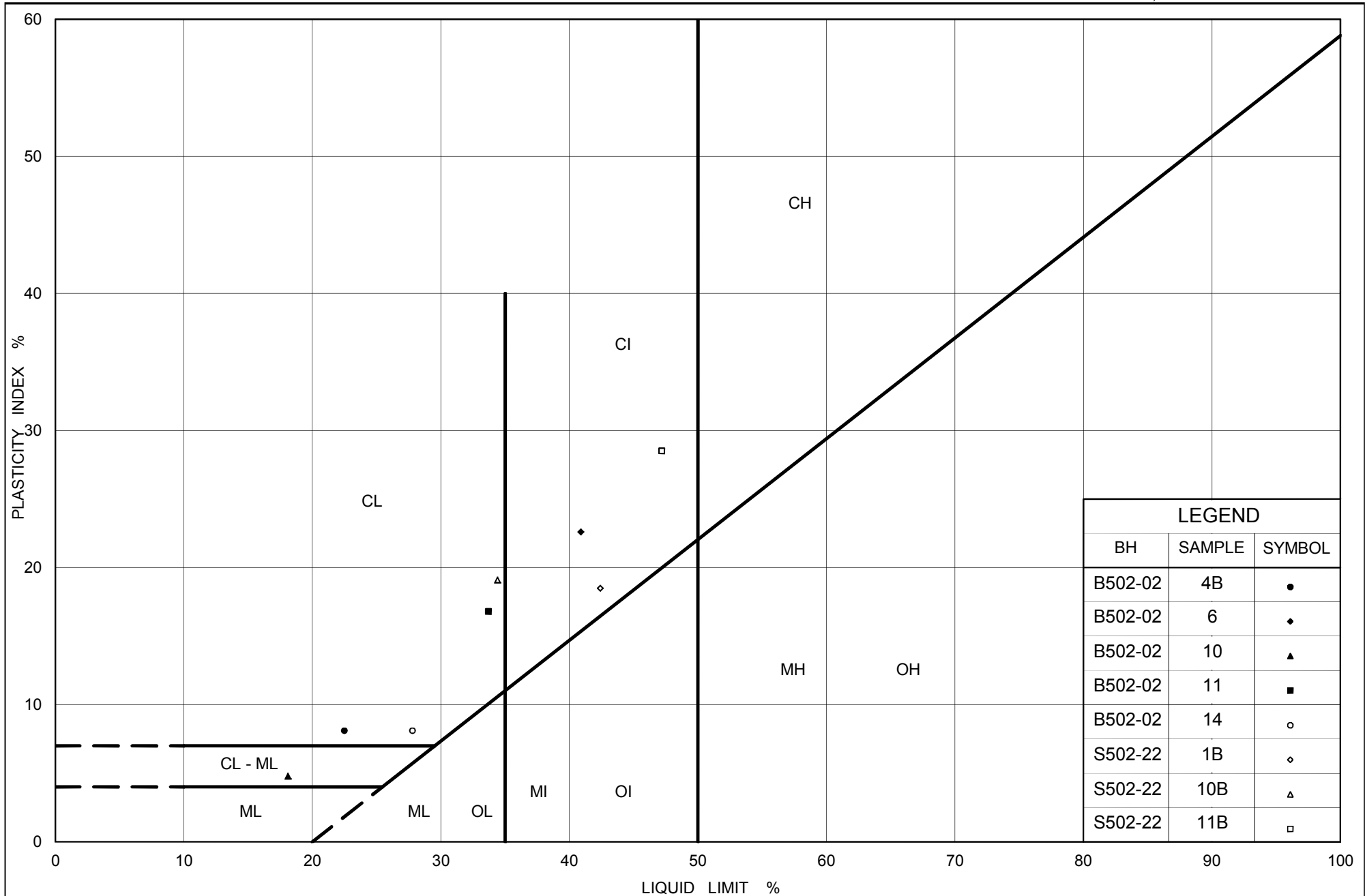
Ontario

PLASTICITY CHART Clayey Silt to Silty Clay

Figure No. B5-C

Project No. 09-1111-6014

Checked By: AB/JMAC



Ministry of Transportation

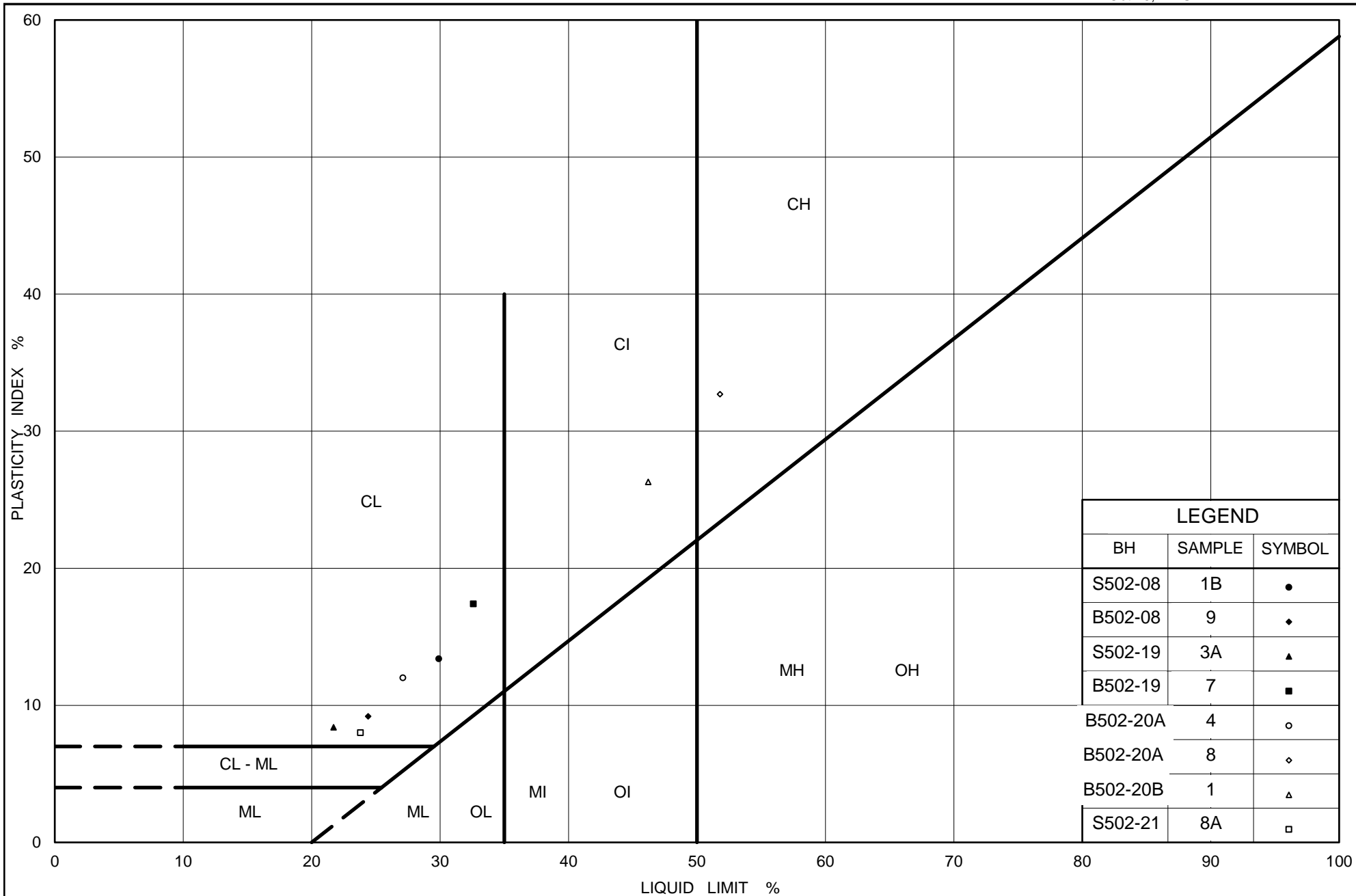
Ontario

PLASTICITY CHART Clayey Silt to Silty Clay

Figure No. B5-D

Project No. 09-1111-6014

Checked By: AB/JMAC



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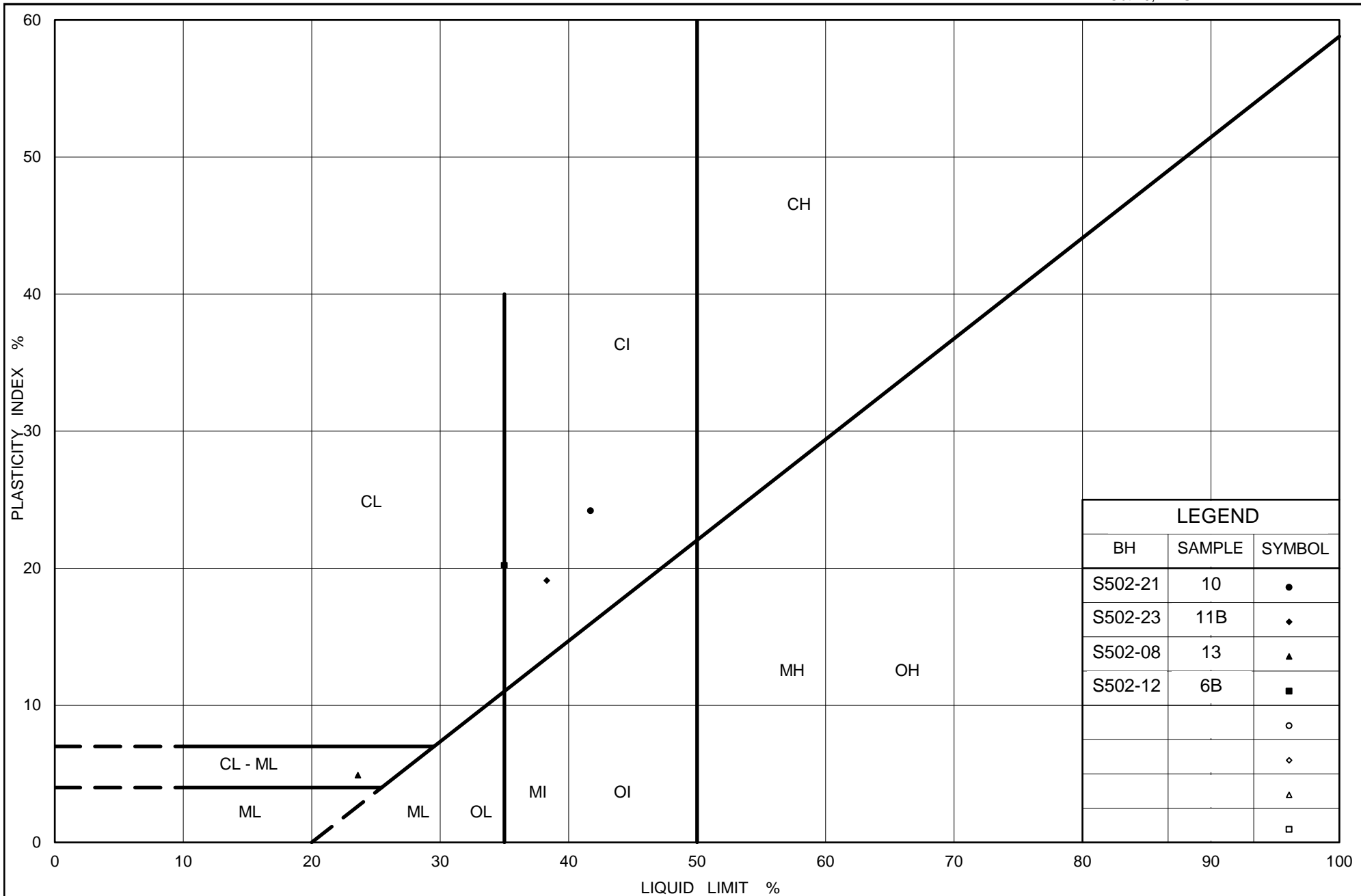
Ontario

PLASTICITY CHART Clayey Silt to Silty Clay

Figure No. B5-E

Project No. 09-1111-6014

Checked By: AB/JMAC



Ministry of Transportation

Ontario

PLASTICITY CHART Clayey Silt to Silty Clay

Figure No. B5-F

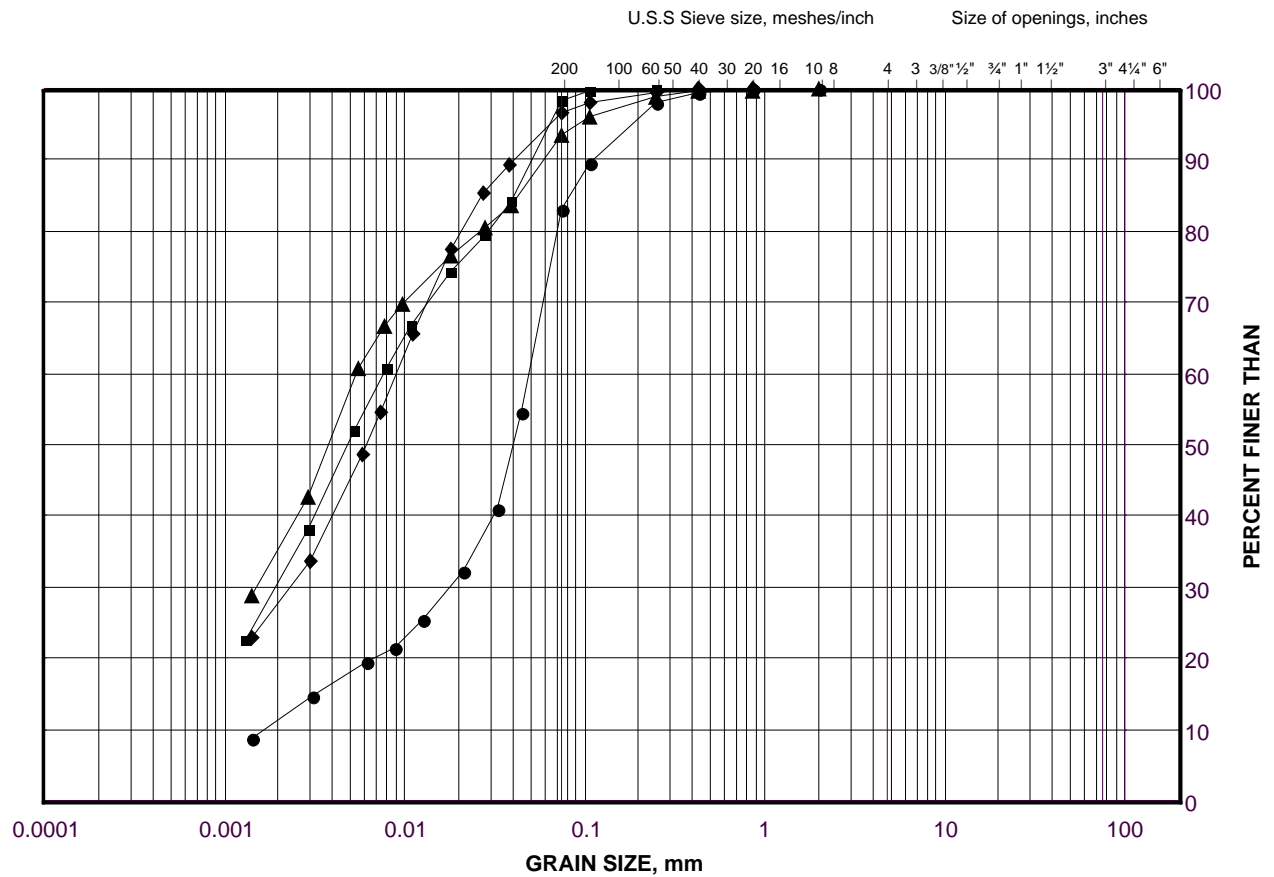
Project No. 09-1111-6014

Checked By: AB/JMAC

GRAIN SIZE DISTRIBUTION

Clayey Silt

FIGURE B6



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	B502-02	10	165
■	B502-02	11	163.7
◆	S502-20A	4	181.8
▲	B502-12	7	171.7

Project Number: 09-1111-6014

Checked By: _____

Golder Associates

Date: 13-Apr-16

CONSOLIDATION TEST SUMMARY**FIGURE B7**

Sheet 1 of 4

SAMPLE IDENTIFICATION

Project Number	09-1111-6014	Sample Number	10
Borehole Number	B502-13	Sample Depth, m	7.47-7.92

TEST CONDITIONS

Test Type	Standard	Load Duration, hr	24
Oedometer Number	8		
Date Started	3/18/2013		
Date Completed	3/31/2014		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	1.90	Unit Weight, kN/m ³	17.57
Sample Diameter, cm	6.30	Dry Unit Weight, kN/m ³	12.11
Area, cm ²	31.19	Specific Gravity, measured	2.76
Volume, cm ³	59.27	Solids Height, cm	0.850
Water Content, %	45.07	Volume of Solids, cm ³	26.51
Wet Mass, g	106.16	Volume of Voids, cm ³	32.75
Dry Mass, g	73.18	Degree of Saturation, %	100.7

TEST COMPUTATIONS

Stress	Corr. Height	Void	Average Height	t ₉₀	cv.	mv	k
kPa	cm	Ratio	cm	sec	cm ² /s	m ² /kN	cm/s
0.00	1.900	1.235	1.900				
6.35	1.903	1.238	1.901	11	6.97E-02		
11.33	1.902	1.238	1.902	231	3.32E-03	5.71E-05	1.86E-08
21.23	1.900	1.235	1.901	638	1.20E-03	1.06E-04	1.25E-08
40.93	1.891	1.224	1.895	1134	6.72E-04	2.56E-04	1.69E-08
80.23	1.874	1.204	1.882	1329	5.65E-04	2.24E-04	1.24E-08
158.60	1.846	1.171	1.860	1192	6.15E-04	1.90E-04	1.15E-08
319.05	1.697	0.997	1.772	3244	2.05E-04	4.86E-04	9.77E-09
632.89	1.547	0.820	1.622	1745	3.20E-04	2.52E-04	7.90E-09
1262.00	1.435	0.688	1.491	919	5.13E-04	9.39E-05	4.72E-09
2517.59	1.335	0.571	1.385	919	4.42E-04	4.18E-05	1.81E-09
1262.00	1.347	0.585	1.341				
319.05	1.368	0.609	1.357				
80.23	1.405	0.653	1.386				
21.23	1.436	0.689	1.421				
6.35	1.467	0.726	1.452				

Note:

Specimen taken 12-19 cm from bottom of the tube

k calculated using cv based on ϕ_0 values.**SAMPLE DIMENSIONS AND PROPERTIES - FINAL**

Sample Height, cm	1.47	Unit Weight, kN/m ³	19.91
Sample Diameter, cm	6.30	Dry Unit Weight, kN/m ³	15.68
Area, cm ²	31.19	Specific Gravity, measured	2.76
Volume, cm ³	45.76	Solids Height, cm	0.850
Water Content, %	26.93	Volume of Solids, cm ³	26.51
Wet Mass, g	92.89	Volume of Voids, cm ³	19.24
Dry Mass, g	73.18		

Prepared By: RD

Golder Associates

Checked By: AV

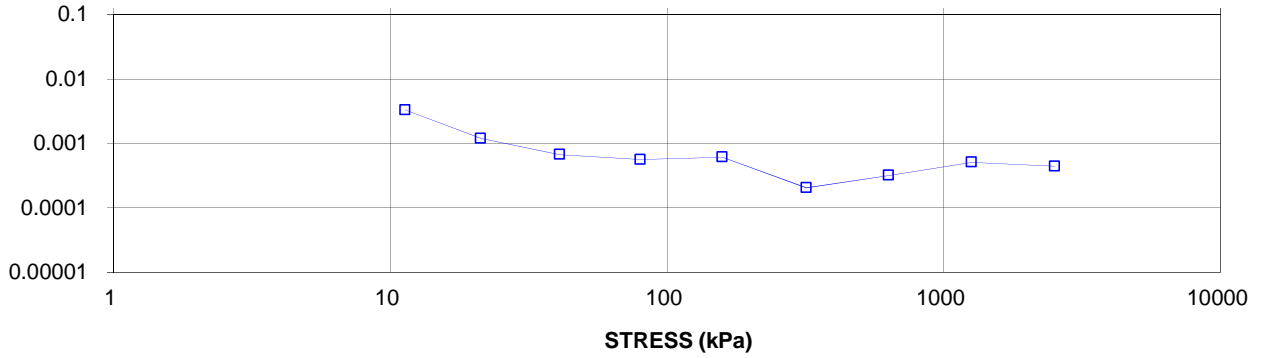
CONSOLIDATION TEST SUMMARY

FIGURE B7

Sheet 2 of 4

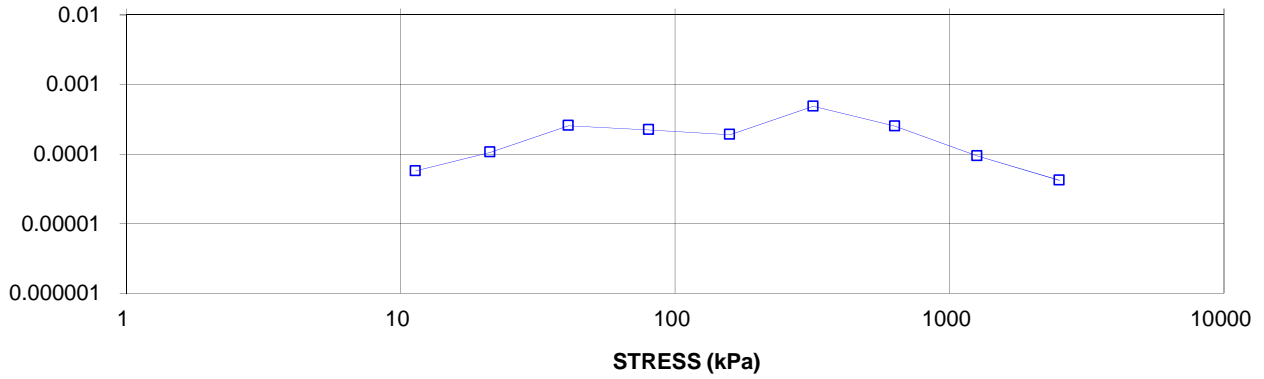
COEFFICIENT OF CONSOLIDATION,
cm²/s

CONSOLIDATION TEST
CV cm²/s VS STRESS (kPa)
BH B502-13 SA 10



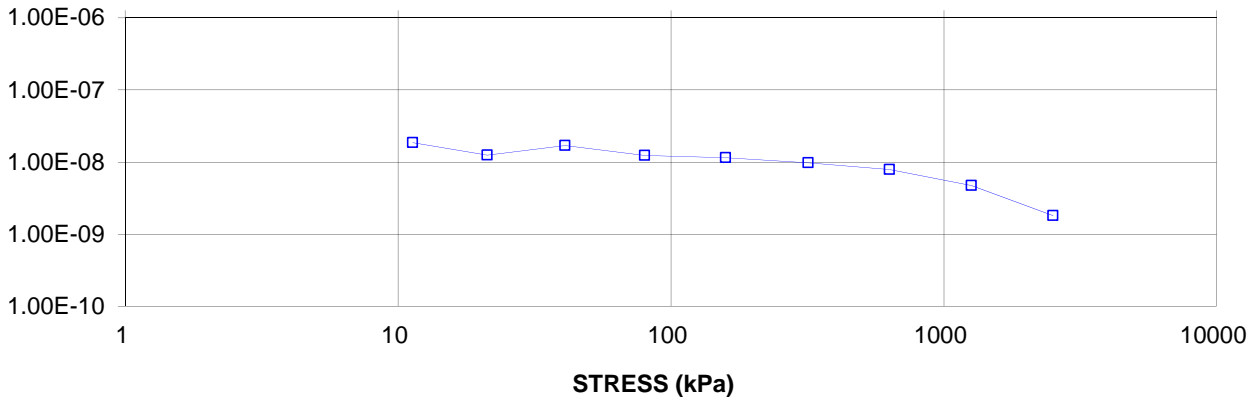
VOLUME COMPRESSIBILITY, m²/kN

CONSOLIDATION TEST
MV m²/kN vs STRESS (kPa)
BH B502-13 SA 10



HYDRAULIC CONDUCTIVITY,
cm/s

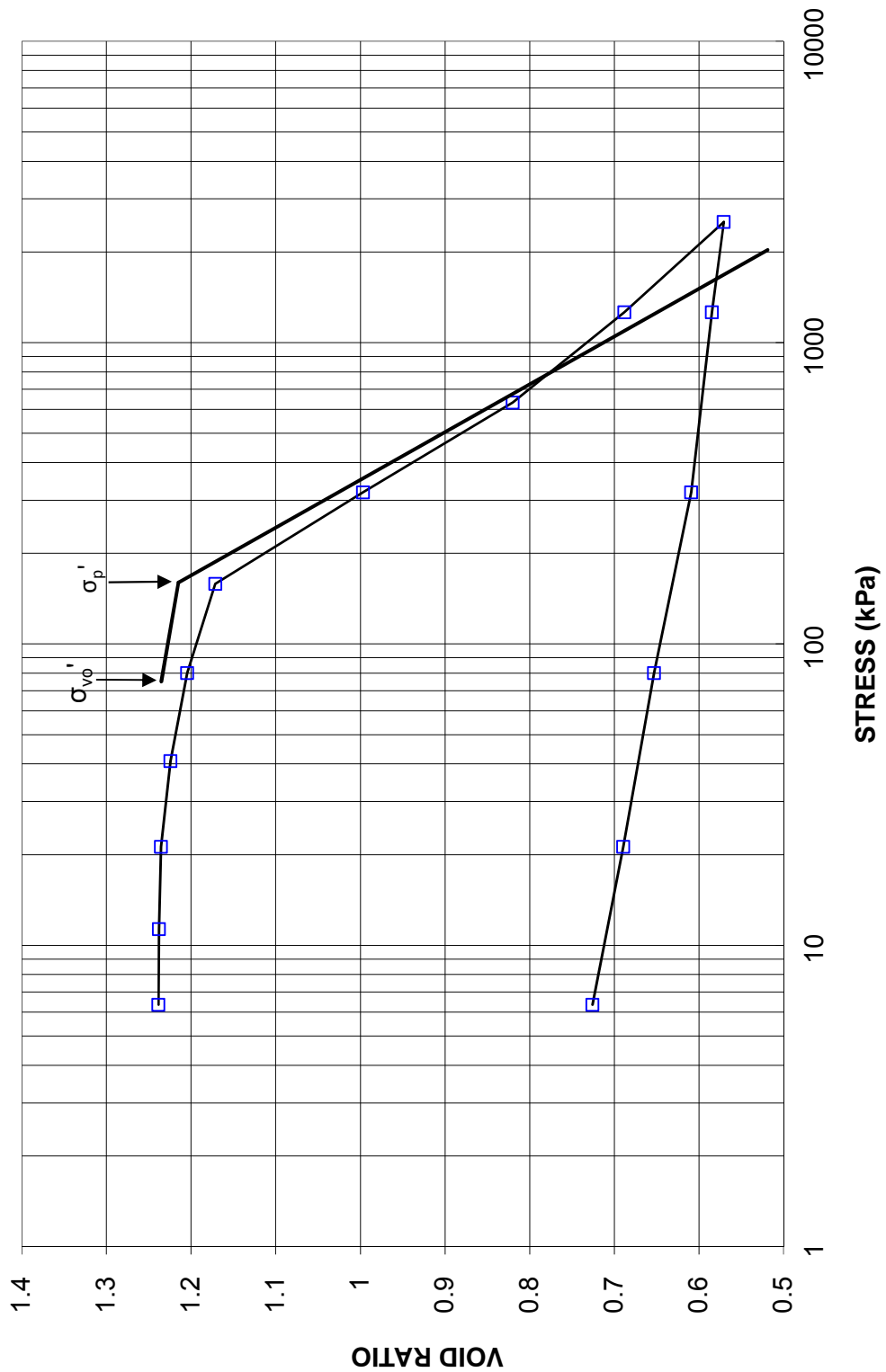
CONSOLIDATION TEST
HYDRAULIC CONDUCTIVITY vs STRESS
BH B502-13 SA 10



**CONSOLIDATION TEST
VOID RATIO VS LOG STRESS**

FIGURE B7
Sheet 3 of 4

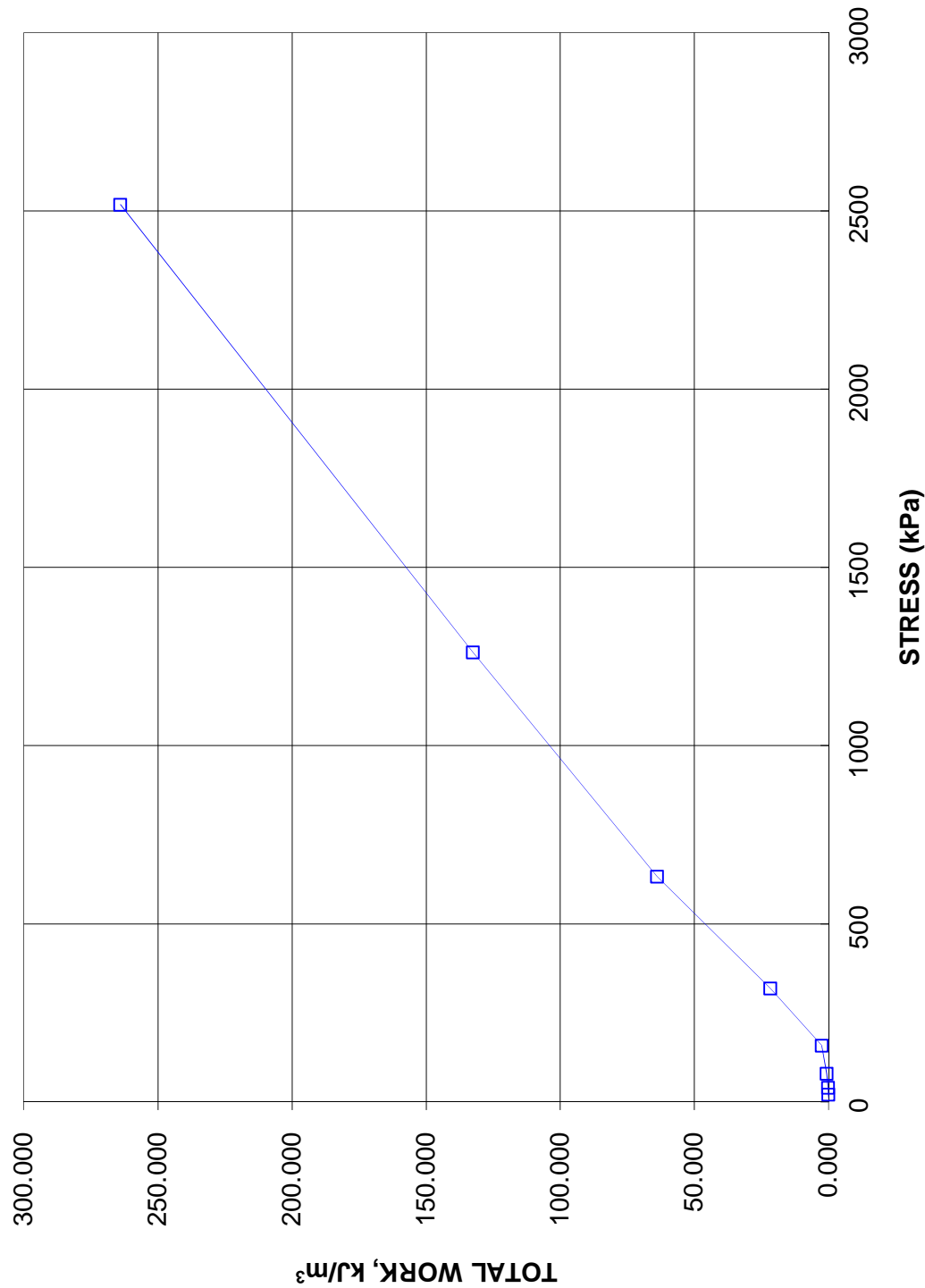
**CONSOLIDATION TEST
VOID RATIO vs STRESS
BH B502-13 SA 10**



CONSOLIDATION TEST
TOTAL WORK VS STRESS

FIGURE B7
Sheet 4 of 4

CONSOLIDATION TEST
TOTAL WORK, kJ/m^3 vs STRESS
BH B502-13 SA 10



CONSOLIDATION TEST SUMMARY**FIGURE B8**

Sheet 1 of 4

SAMPLE IDENTIFICATION

Project Number	09-1111-6014	Sample Number	10
Borehole Number	S502-20	Sample Depth, m	9.14-9.74

TEST CONDITIONS

Test Type	Standard	Load Duration, hr	24
Oedometer Number	1		
Date Started	12/11/2013		
Date Completed	01/02/2014		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	2.56	Unit Weight, kN/m ³	16.91
Sample Diameter, cm	6.34	Dry Unit Weight, kN/m ³	11.24
Area, cm ²	31.54	Specific Gravity, measured	2.75
Volume, cm ³	80.58	Solids Height, cm	1.065
Water Content, %	50.49	Volume of Solids, cm ³	33.57
Wet Mass, g	138.95	Volume of Voids, cm ³	47.01
Dry Mass, g	92.33	Degree of Saturation, %	99.2

TEST COMPUTATIONS

Stress	Corr. Height	Void	Average Height	t ₉₀	cv.	mv	k
kPa	cm	Ratio	cm	sec	cm ² /s	m ² /kN	cm/s
0.00	2.555	1.400	2.555				
6.26	2.553	1.399	2.554	1	1.38E+00	1.00E-04	1.36E-05
10.91	2.551	1.396	2.552	34	4.06E-02	2.27E-04	9.04E-07
20.98	2.547	1.392	2.549	94	1.46E-02	1.63E-04	2.34E-07
40.45	2.535	1.382	2.541	167	8.20E-03	2.23E-04	1.79E-07
79.24	2.511	1.359	2.523	184	7.33E-03	2.49E-04	1.79E-07
98.58	2.499	1.348	2.505	1815	7.33E-04	2.37E-04	1.70E-08
40.45	2.507	1.355	2.503				
10.91	2.519	1.366	2.513				
40.45	2.510	1.358	2.515	42	3.19E-02	1.15E-04	3.61E-07
79.24	2.501	1.349	2.506	43	3.10E-02	9.79E-05	2.97E-07
98.58	2.495	1.344	2.498	2953	4.48E-04	1.15E-04	5.06E-09
156.82	2.462	1.313	2.478	1717	7.58E-04	2.24E-04	1.66E-08
311.93	2.232	1.097	2.347	1270	9.19E-04	5.80E-04	5.22E-08
622.63	2.000	0.878	2.116	508	1.87E-03	2.93E-04	5.36E-08
1246.39	1.855	0.742	1.927	227	3.47E-03	9.09E-05	3.09E-08
2491.36	1.728	0.623	1.791	194	3.51E-03	4.00E-05	1.37E-08
1246.39	1.733	0.628	1.730				
311.93	1.768	0.661	1.750				
98.58	1.796	0.687	1.782				
20.98	1.841	0.730	1.819				
6.26	1.878	0.764	1.859				

Note:

Specimen taken 19-29 cm from bottom of the tube
k calculated using cv based on $\dot{\epsilon}_0$ values.

SAMPLE DIMENSIONS AND PROPERTIES - FINAL

Sample Height, cm	1.88	Unit Weight, kN/m ³	19.83
Sample Diameter, cm	6.34	Dry Unit Weight, kN/m ³	15.29
Area, cm ²	31.54	Specific Gravity, measured	2.75
Volume, cm ³	59.22	Solids Height, cm	1.065
Water Content, %	29.69	Volume of Solids, cm ³	33.57
Wet Mass, g	119.74	Volume of Voids, cm ³	25.64
Dry Mass, g	92.33		

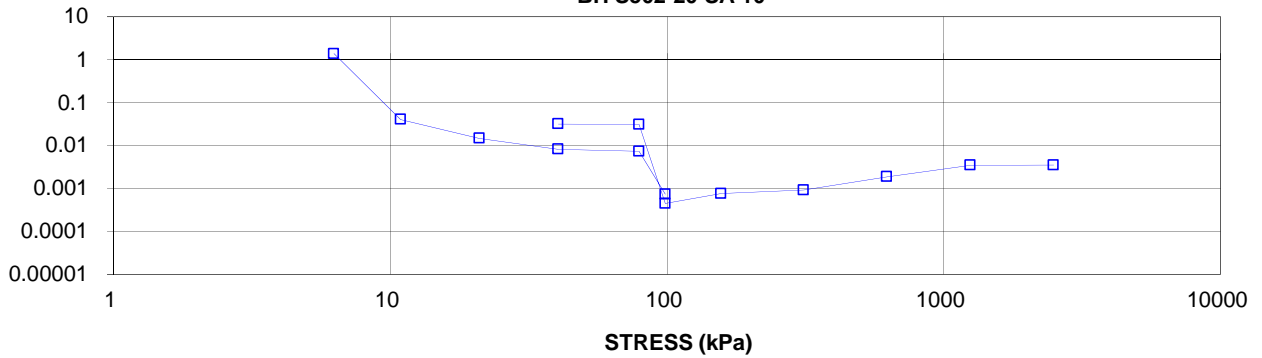
CONSOLIDATION TEST SUMMARY

FIGURE B8

Sheet 2 of 4

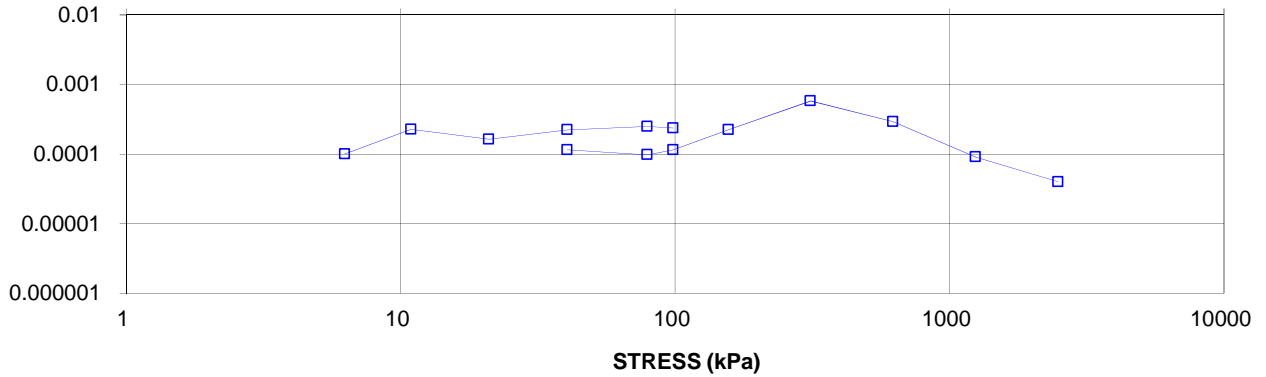
COEFFICIENT OF CONSOLIDATION,
cm²/s

CONSOLIDATION TEST
CV cm²/s VS STRESS (kPa)
BH S502-20 SA 10



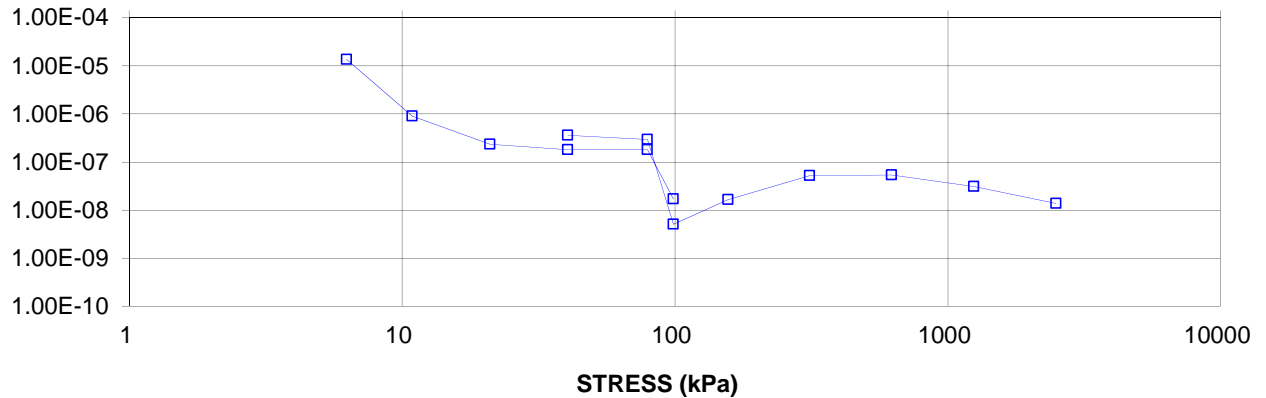
VOLUME COMPRESSIBILITY, m²/kN

CONSOLIDATION TEST
MV m²/kN vs STRESS (kPa)
BH S502-20 SA 10



HYDRAULIC CONDUCTIVITY,
cm/s

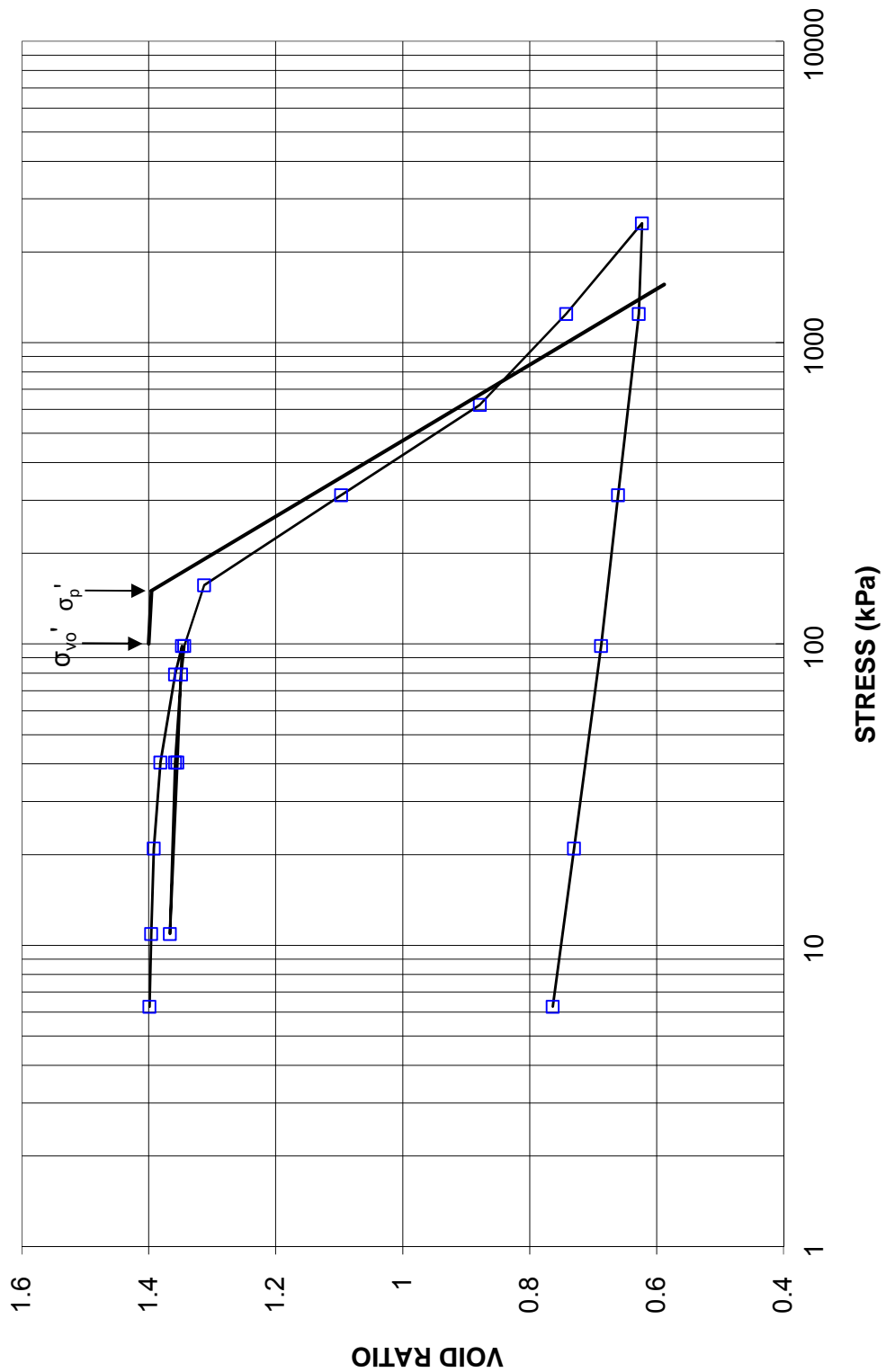
CONSOLIDATION TEST
HYDRAULIC CONDUCTIVITY vs STRESS
BH S502-20 SA 10



CONSOLIDATION TEST VOID RATIO VS LOG STRESS

FIGURE B8
Sheet 3 of 4

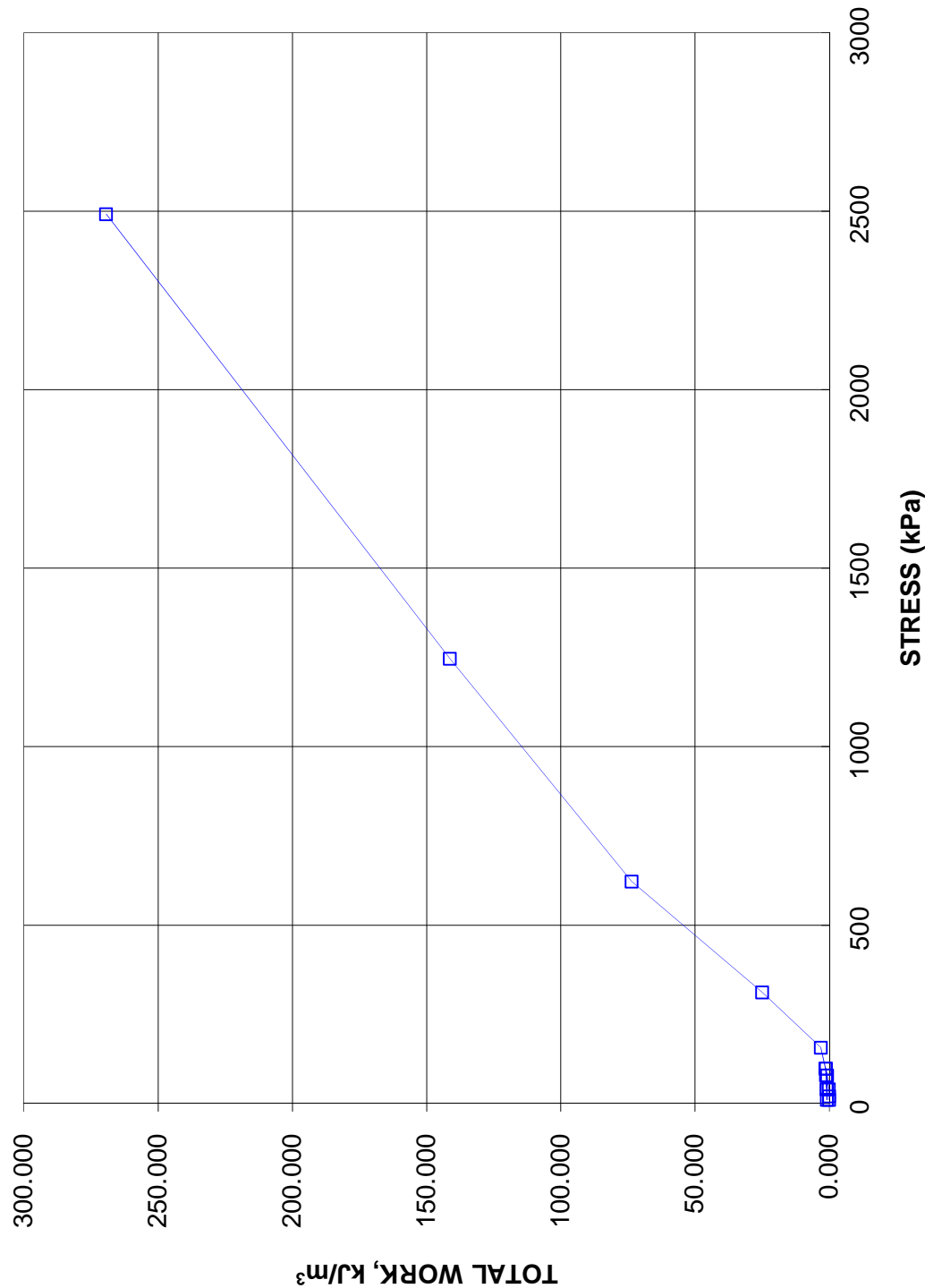
CONSOLIDATION TEST
VOID RATIO vs STRESS
BH S502-20 SA 10



CONSOLIDATION TEST
TOTAL WORK VS STRESS

FIGURE B8
Sheet 4 of 4

CONSOLIDATION TEST
TOTAL WORK, kJ/m³ vs STRESS
BH S502-20 SA 10



CONSOLIDATION TEST SUMMARY**FIGURE B9**

Sheet 1 of 4

SAMPLE IDENTIFICATION

Project Number	09-1111-6014	Sample Number	1
Borehole Number	S502-20B	Sample Depth, m	5.5-5.9

TEST CONDITIONS

Test Type	Standard	Load Duration, hr	24
Oedometer Number	3		
Date Started	3/18/2013		
Date Completed	3/31/2014		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	2.54	Unit Weight, kN/m ³	16.95
Sample Diameter, cm	6.33	Dry Unit Weight, kN/m ³	11.13
Area, cm ²	31.43	Specific Gravity, measured	2.76
Volume, cm ³	79.77	Solids Height, cm	1.043
Water Content, %	52.30	Volume of Solids, cm ³	32.80
Wet Mass, g	137.86	Volume of Voids, cm ³	46.97
Dry Mass, g	90.52	Degree of Saturation, %	100.8

TEST COMPUTATIONS

Stress	Corr. Height	Void Ratio	Average Height	t ₉₀	cv.	mv	k
kPa	cm		cm	sec	cm ² /s	m ² /kN	cm/s
0.00	2.538	1.432	2.538				
5.85	2.541	1.435	2.540				
10.81	2.538	1.432	2.540	1220	1.12E-03	2.70E-04	2.97E-08
20.59	2.531	1.425	2.534	1192	1.14E-03	2.90E-04	3.25E-08
40.17	2.520	1.415	2.525	1250	1.08E-03	2.23E-04	2.37E-08
79.16	2.497	1.393	2.508	1117	1.19E-03	2.32E-04	2.72E-08
157.05	2.457	1.354	2.477	1024	1.27E-03	2.02E-04	2.52E-08
312.87	2.311	1.215	2.384	2289	5.26E-04	3.68E-04	1.90E-08
624.42	2.070	0.984	2.190	1745	5.83E-04	3.05E-04	1.74E-08
1265.50	1.917	0.837	1.993	923	9.13E-04	9.43E-05	8.43E-09
2513.86	1.786	0.712	1.851	638	1.14E-03	4.12E-05	4.60E-09
1265.50	1.800	0.725	1.793				
312.87	1.833	0.756	1.816				
79.16	1.880	0.801	1.856				
20.59	1.918	0.838	1.899				
5.85	1.959	0.877	1.938				

Note:

Specimen taken 13-18 cm from bottom of the tube

k calculated using cv based on t₉₀ values.**SAMPLE DIMENSIONS AND PROPERTIES - FINAL**

Sample Height, cm	1.96	Unit Weight, kN/m ³	19.14
Sample Diameter, cm	6.33	Dry Unit Weight, kN/m ³	14.42
Area, cm ²	31.43	Specific Gravity, measured	2.76
Volume, cm ³	61.56	Solids Height, cm	1.043
Water Content, %	32.74	Volume of Solids, cm ³	32.80
Wet Mass, g	120.16	Volume of Voids, cm ³	28.77
Dry Mass, g	90.52		

Prepared By: RD

Golder Associates

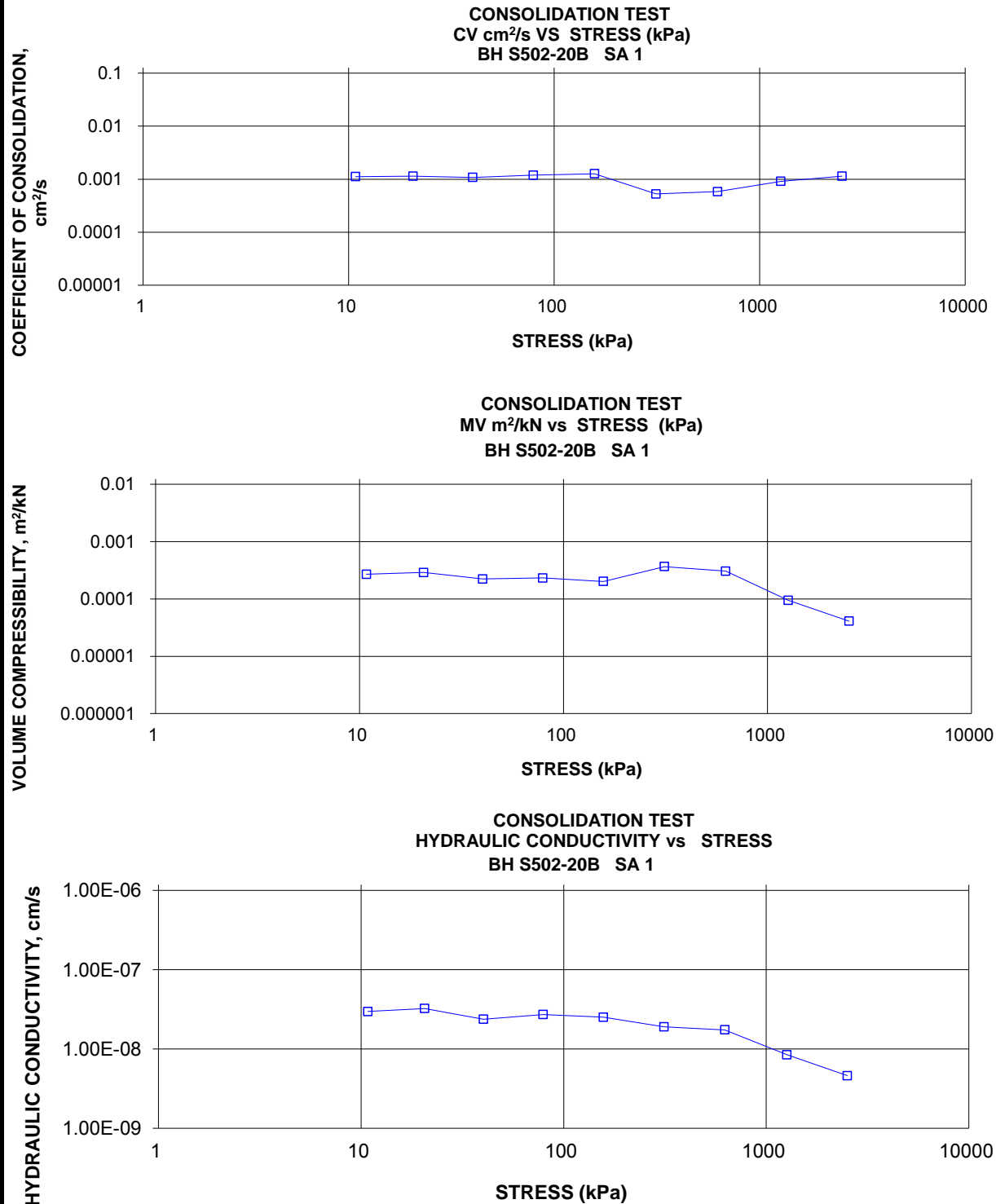
Checked By:

AV

CONSOLIDATION TEST SUMMARY

FIGURE B9

Sheet 2 of 4



Project No. 09-1111-6014

Prepared By: RD

Golder Associates

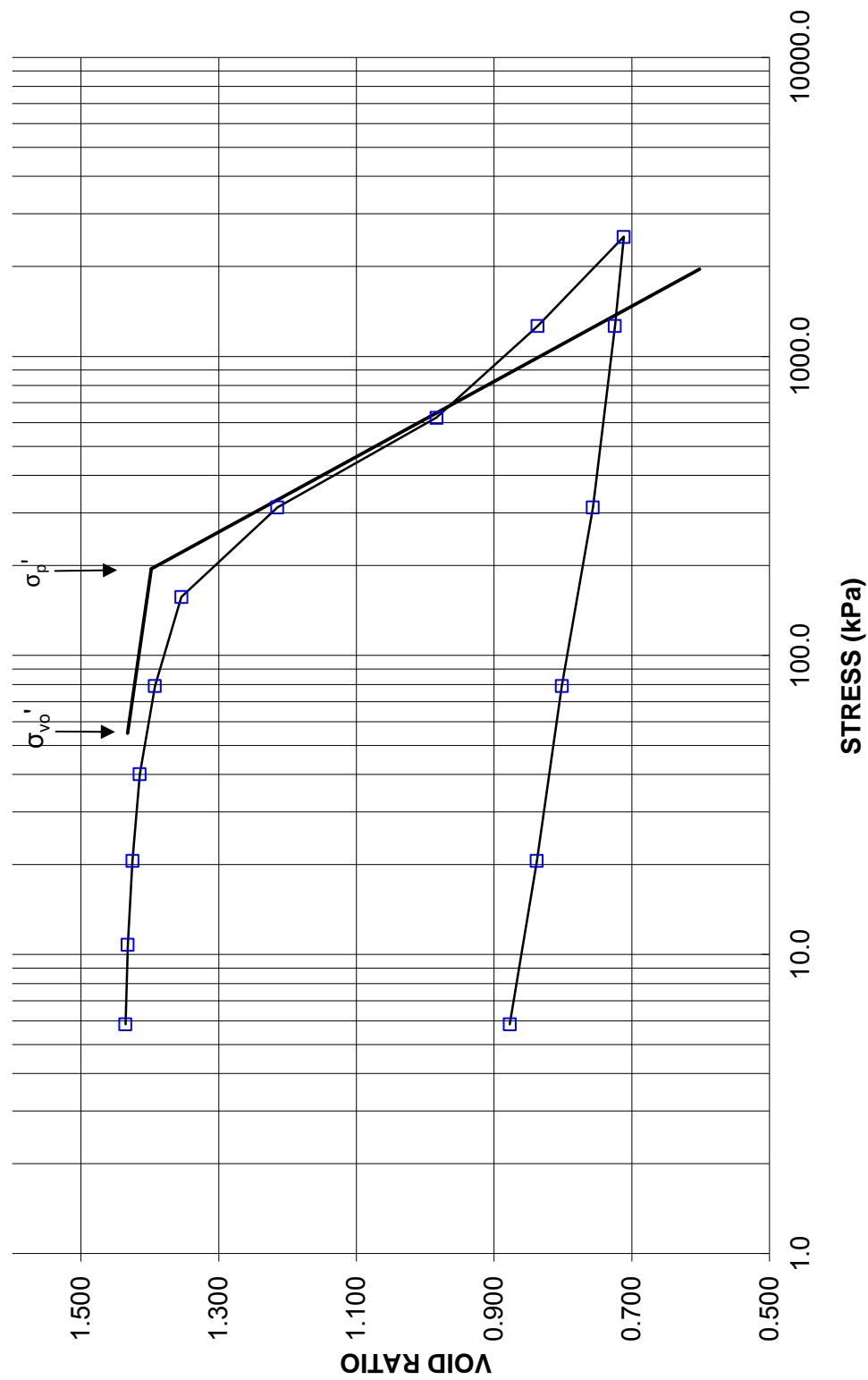
Checked By:

AV

CONSOLIDATION TEST
VOID RATIO VS LOG STRESS

FIGURE B9
Sheet 3 of 4

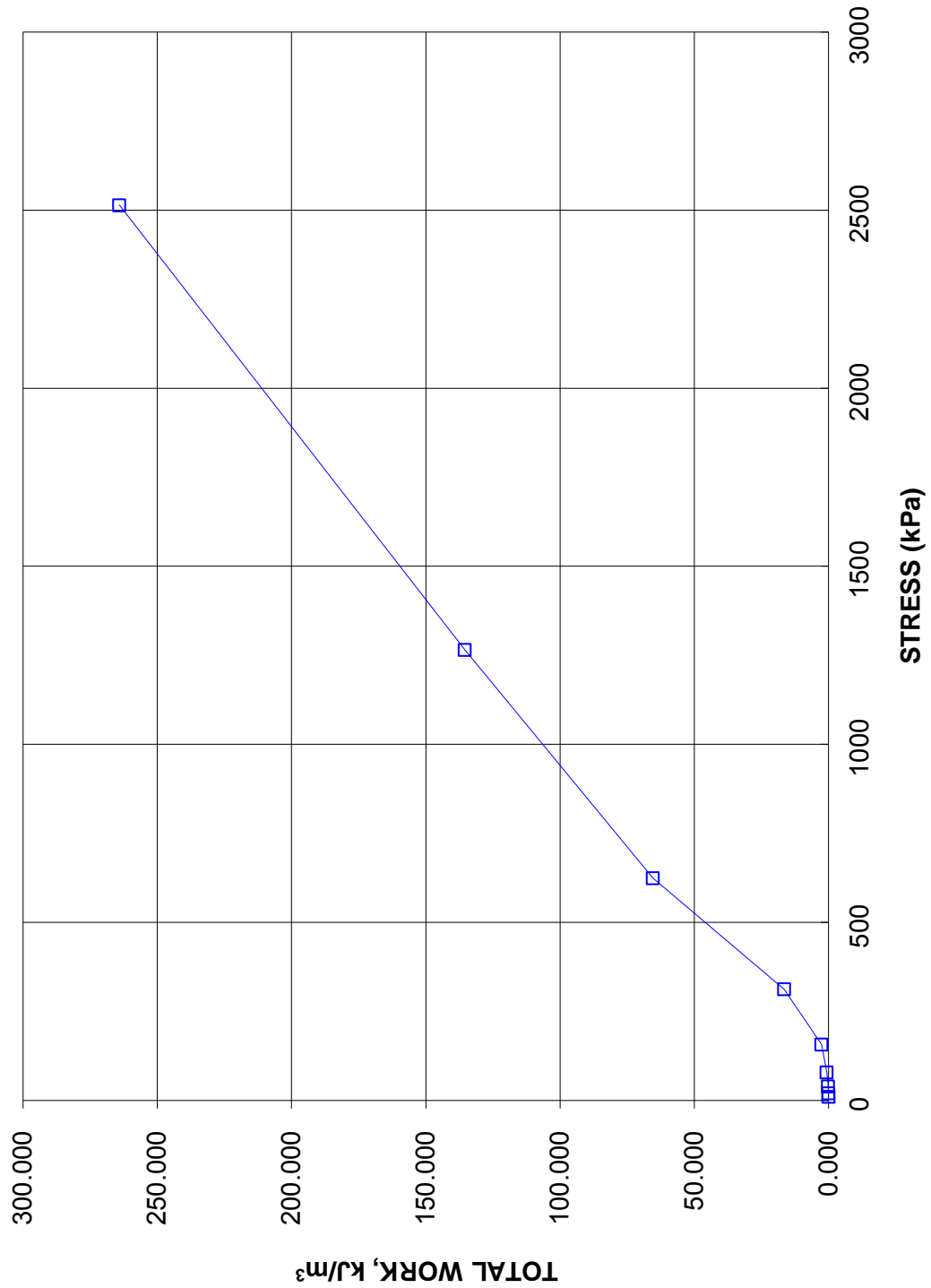
CONSOLIDATION TEST
VOID RATIO vs STRESS
BH S502-20B SA 1



**CONSOLIDATION TEST
TOTAL WORK VS STRESS**

**FIGURE B9
Sheet 4 of 4**

**CONSOLIDATION TEST
TOTAL WORK, kJ/m^3 vs STRESS
BH S502-20B SA 1**



Three sections of a geological core sample are shown, labeled 1-3, 1-25, and 1-50. The sections are dark, cylindrical, and show various sedimentary layers and textures. Handwritten annotations in red and black ink are visible on the sections. The top section (1-3) has a red 'X' and a red '1-3' at the right end. The middle section (1-25) has a red 'X' and a red '1-25' at the right end. The bottom section (1-50) has a red 'X' and a red '1-50' at the right end. A handwritten note 'END 2ND RUN 51.5'' is visible on the right side of the bottom section.

93.5' = Boulder

END 1ST RUN 98.0'

END 2ND RUN 103.1'

Figure 1 displays two sets of rock samples, likely from the same geological formation, showing varying degrees of fracturing and discoloration. The samples are arranged in wooden boxes with rulers for scale.


Left Set of Samples:

- Sample 1 (top): Labeled "17.12" and "5m".
- Sample 2: Labeled "16.40" and "25".
- Sample 3: Labeled "16.40" and "50".
- Sample 4 (bottom): Labeled "20.35" and "22.0".

Right Set of Samples:

- Sample 1 (top): Labeled "22.0" and "7m".
- Sample 2 (bottom): Labeled "22.0" and "15".

0 m	0.25 m	0.5 m	0.75 m	1.0 m	1.25 m	1.5 m
0 ft	1 ft	2 ft	3 ft	4 ft	5 ft	

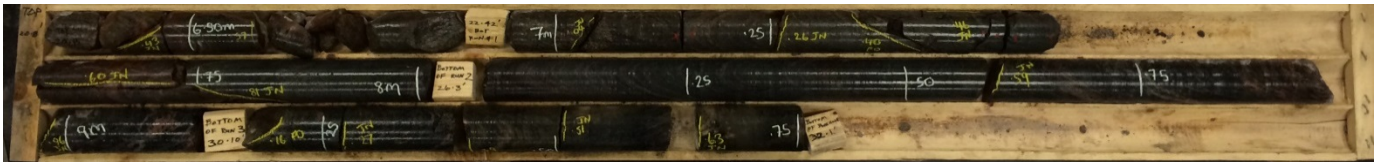
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TITLE Bedrock Core Photographs Boreholes B502-01, B502-02 & B502-05							
		PROJECT No. 09-1111-6014				FILE No. ----	
		DESIGN	MT	OCT 15	SCALE	NTS	REV.
		CADD	-- --				
		CHECK	AB	OCT 15	FIGURE B10		
		REVIEW	JMAC	OCT 15			

Borehole B502-06



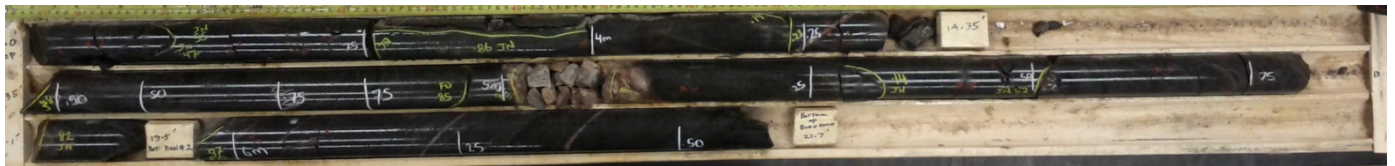
Box 1: 7.9 m – 9.4 m

Borehole B502-07

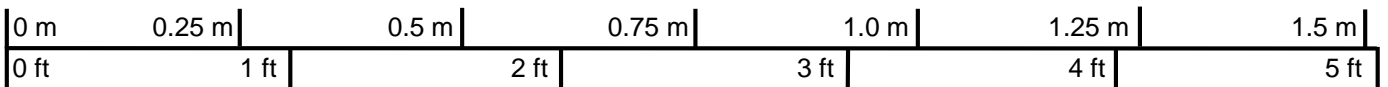


Box 1: 6.3 m – 9.8 m


Borehole B502-09



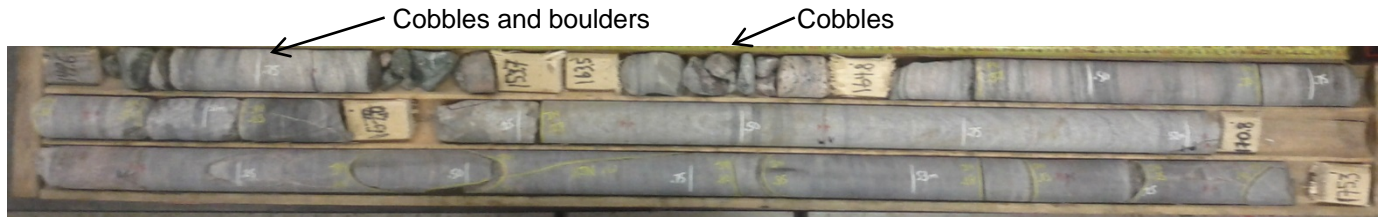
Box 1: 3.4 m – 6.6 m



Scale

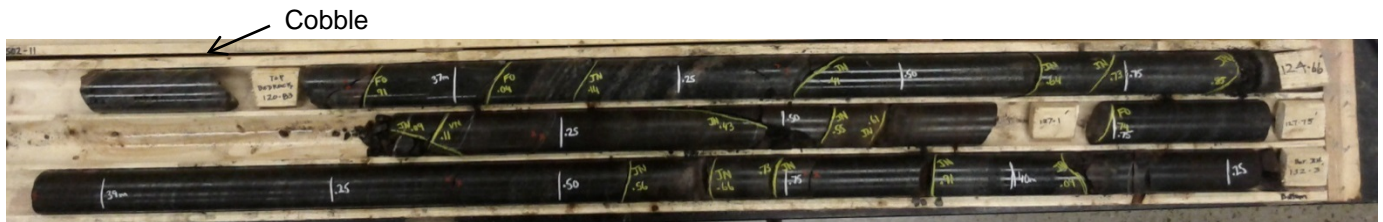
PROJECT		Straight Lake NBL Bridge Structure Highway 69 GWP 5005-10-00; WP 5145-08-01		
TITLE		Bedrock Core Photographs Boreholes B502-06, B502-07 & B502-09		
		PROJECT No. 09-1111-6014		FILE No. ----
		DESIGN	MT	OCT 15
		CADD	--	--
		CHECK	AB	OCT 15
		REVIEW	JMAC	OCT 15
		SCALE NTS		
		FIGURE B11		

Borehole B502-10



Box 1: 50.2 m – 53.4 m

Borehole B502-11

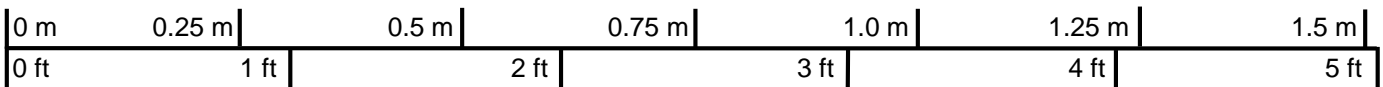


Box 1: 36.8 m – 40.3 m


Borehole B502-12



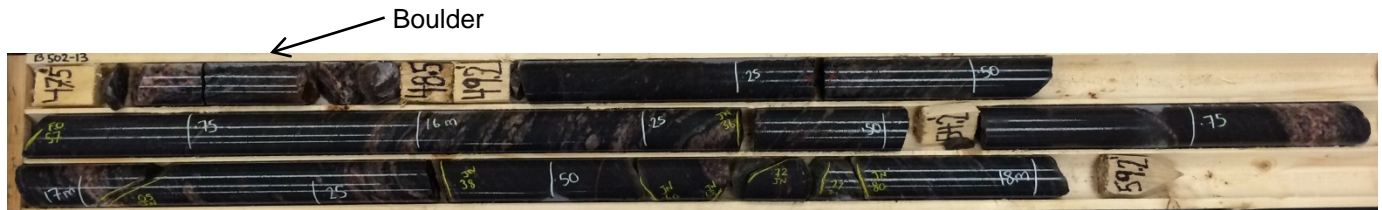
Box 1: 14.7 m – 18.0 m



Scale

PROJECT		Straight Lake NBL Bridge Structure Highway 69 GWP 5005-10-00; WP 5145-08-01			
TITLE		Bedrock Core Photographs Boreholes B502-10, B502-11 & B502-12			
		PROJECT No. 09-1111-6014		FILE No. ----	
		DESIGN	MT	OCT 15	SCALE NTS
		CADD	--		
		CHECK	AB	OCT 15	FIGURE B12
		REVIEW	JMAC	OCT 15	

Borehole B502-13



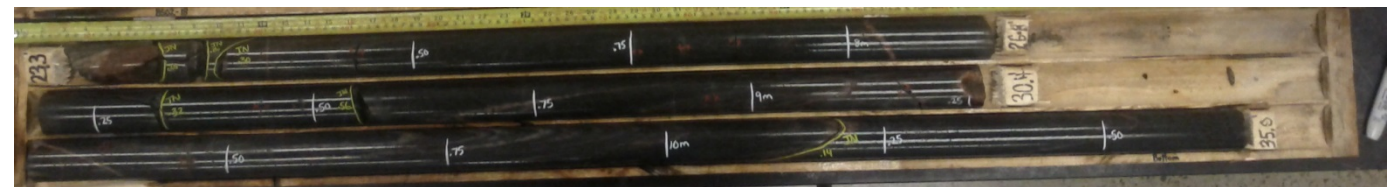
Box 1: 15.0 m – 18.0 m

Borehole B502-15

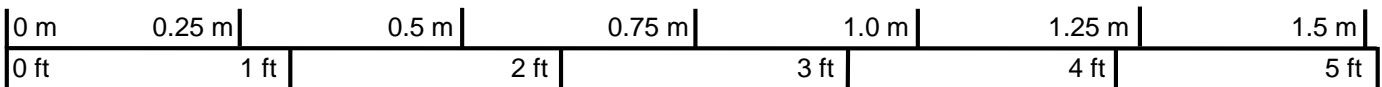


Box 1: 2.4 m – 5.6 m


Borehole B502-18



Box 1: 7.1 m – 10.7 m



Scale

PROJECT		Straight Lake NBL Bridge Structure Highway 69 GWP 5005-10-00; WP 5145-08-01			
TITLE		Bedrock Core Photographs Boreholes B502-13, B502-15 & B502-18			
		PROJECT No. 09-1111-6014		FILE No. ----	
		DESIGN	MT	OCT 15	SCALE NTS
		CADD	--		REV.
		CHECK	AB	OCT 15	FIGURE B13
		REVIEW	JMAC	OCT 15	



APPENDIX C

Background Information Related to Changes in Bridge Configuration

May 31, 2013

Project No. 09-1111-6014

Mr. Dragan Illic, P.Eng.
URS Canada Inc.
4th Floor, 30 Leek Crescent
Richmond Hill, Ontario
L4B 4N4

**STRAIGHT LAKE BRIDGE STRUCTURES - HIGHWAY 69 (NEW) NBL & SBL
PRELIMINARY SUMMARY OF SUBSURFACE CONDITIONS AND FOUNDATION ALTERNATIVES
HIGHWAY 69 (NEW) FOUR-LANING FROM 1.7 KM NORTH OF HIGHWAY 529, NORTHERLY TO 3.9 KM
NORTH OF HIGHWAY 522, 19.7 KM
G.W.P. 5404-05-01; W.P. 5145-08-01 AND 5146-08-01; AGREEMENT 5008-E-0009
DRAFT - FOR INTERNAL DISCUSSION ONLY**

Dear Sirs:

Golder Associates Ltd. (Golder) was requested by URS Canada Inc. (URS) to provide a summary of the subsurface conditions encountered and a preliminary assessment of foundation alternatives at the site of the proposed Straight Lake Bridge Structures located within the Henvey Inlet First Nations land (to the north) and the Township of Henvey (to the south). The proposed structures are part of Contract 5 of the above noted project.

The Straight Lake NBL and SBL bridge structures will carry the new Highway 69 Four-Laning over Straight Lake at a location approximately 1.3 km east of the existing Highway 69 alignment and about 4.0 km south of the existing Highway 552 connection. The approximate location of the site along the new Highway 69 alignment is shown on the Key Plan on Drawings 1 and 2. A preliminary foundation investigation in the vicinity of the proposed bridge structures was carried out by AMEC Earth and Environmental (AMEC) in 2006 as part of the Preliminary Foundation Investigation and Design for the Highway 69 Route Selection Study. In 2013, Golder carried out the Phase 1 foundation investigation for the detail design of the structures. The locations of all of the boreholes advanced in this area, together with a stratigraphic profile along the centreline of the proposed structure alignments are shown on Drawing 1 (SBL) and Drawing 2 (NBL). This letter summarizes the subsurface soil conditions at the proposed structures based on the subsurface investigations completed to date.

In general, the topography at/in the vicinity of the bridge sites consists of rolling terrain including densely treed areas, bedrock outcrops at or near the south shore of the lake, covered in places with low scrub-brush, and some low-lying areas adjacent to north shore of the lake. The proposed Straight Lake bridge structures are located within an area where the ground surface rises about 6 m above the water surface on the south shore of the lake but only about 1 m to 2 m above the water surface on the north shore of the lake. The ground surface



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generally slopes steeply upward from the south shore of the lake to the proposed south abutment and approach areas (where it is as much as 14 m above the water surface) but slopes gently upward from the north shore of the lake to the proposed north abutment and approach areas. Bedrock outcrops at various points on the south abutment areas, but was not seen to be outcropping in the north abutment areas.

1.0 SUBSURFACE CONDITIONS

1.1 South Shore

As part of the preliminary investigation carried out by AMEC in 2006 for these structures, one (1) borehole (Borehole ST-36) was advanced on the south shore in the vicinity of the south abutment and approach of the proposed Straight Lake SBL bridge structure. In this area, bedrock was found to be overlain by a thin layer of topsoil up to about 0.2 m thick, with the bedrock surface at about Elevation 183.5 m at the investigated location. Bedrock outcrops at various points on the south shore in the area of the south abutments and approach embankments. In general, the topography around the south abutments and approach embankments shows that the ground surface rises from the lake surface at about Elevation 178 m and extends as high as about Elevation 192 m, resulting in outcrop heights up to about 14 m above the lake level.

1.2 Straight Lake (On-Ice/Over-Water Investigations)

A total of five (5) in-water boreholes and one (1) dynamic cone penetration test (DCPT) have been advanced from the ice surface of Straight Lake as part of the various investigations completed to date for the proposed SBL and NBL structures at this location. The investigation area and borehole locations are shown in plan on Drawings 1 and 2. Borehole ST-37 was advanced by AMEC in 2006; Boreholes B501-01, B501-02, B502-01 and B502-02, and DCPT B501-DC01 were advanced by Golder in 2013.

At the time of the most recent investigation in 2013, the frozen lake level was at about Elevation 178.6 m; the lake level was at Elevation 178.5 m during the AMEC 2006 investigation. The elevation of the lake level is expected to fluctuate on a seasonal basis.

1.2.1 Near Shore – South Side

At the location of the one borehole (Borehole B502-01) and one DCPT (B501-DC01) advanced as part of the recent Phase 1 investigation by Golder, within the near shore area at the south side of the lake, the subsoils immediately beneath the lakebed along the proposed bridge structure alignments generally consist of a stratum of very loose organic silt to silty sand trace organics up to about 4.8 m thick. Underlying the organic silt to silty sand is a non-cohesive deposit of very loose silt about 1.1 m thick, which is underlain by a cohesive deposit of soft to firm clayey silt about 1.9 m thick. Underlying the cohesive deposit is a non-cohesive deposit of loose silt about 0.9 m thick. Bedrock was inferred from the DCPT advanced to a depth of about 15.6 m below the ice surface (Elevation 163.0 m) at DCPT B501-DC01 – SBL structure alignment. Granite gneiss bedrock was encountered beneath the overburden soils at a depth of about 12.7 m below the ice surface (Elevation 165.9 m) at Borehole B502-01 – NBL structure alignment. Given that the bedrock outcrops at about Elevation 180 m on the south shore of the lake, it appears that the sloping nature of the bedrock (downwards from south to north) is relatively consistent at both structures and the thicknesses of the overburden on the south side of the lake in the area of the proposed Pier 1 may be similar for both the SBL and NBL structures. The results of point load tests carried out on selected samples of the bedrock core indicate that the granite gneiss is strong to very strong. The depth of ice/water to the lakebed in this area varied from about 4.0 m to 4.3 m.

1.2.2 Middle of Lake

At the location of the two (2) boreholes advanced within the middle area of the lake (Borehole B501-01 as part of Golder's Phase 1 investigation; and Borehole ST-37 as part of AMEC's 2006 investigation), the subsoils immediately beneath the lakebed along the proposed bridge structure alignments generally consist of a stratum of very loose organic silt to very soft clayey silt to clay about 4.5 m thick. This stratum is underlain (at some locations) by a deposit of compact to dense sand to silty sand about 4.3 m thick, underlain by a deposit of very soft to stiff silty clay to clay between about 9.5 m and 21.6 m thick, which is in turn underlain by a deposit of very loose to very dense silt to sand and silt to sand. The thickness of the silt to sand deposit was measured to be approximately 27.0 m in Borehole B501-01. Borehole ST-37 was terminated within the silt to sand and silt to sand deposit after penetrating it for approximately 12.4 m. Granite gneiss bedrock was encountered beneath the overburden soils at a depth of approximately 49.3 m below the ice surface (Elevation 129.3 m at Borehole B501-01 – SBL structure alignment). The results of point load tests carried out on selected samples of the bedrock core indicate that the granite gneiss is strong. The depth of ice/water to the lakebed in this area varied from about 3.1 m to 4.0 m.

1.2.3 Near Shore – North Side

At the location of the two (2) boreholes (Borehole B501-02 and B502-02) advanced as part of the recent Phase 1 investigation by Golder, within the near shore area on the north side of the lake, the subsoils immediately beneath the lakebed along the proposed bridge structure alignments generally consist of a stratum of very soft peat/very loose organic silt ranging in thickness from about 2.4 m to 2.9 m. Underlying the peat/organic silt is a deposit of very soft to stiff clayey silt to silty clay between about 16.4 and 17.2 m thick containing interlayers of very loose to compact silts to sands up to about 2.8 m thick. The cohesive stratum is underlain by a non-cohesive deposit of compact to dense silt to sand and silt to sand between about 7.7 m and 16.7 m thick. Cobbles and boulders were encountered within this non-cohesive deposit at depths ranging from about 28.0 m below the ice surface (Elevation 150.6 m at Borehole B502-02 – NBL structure alignment) to about 33.2 m below the ice surface (Elevation 145.4 m at Borehole B501-02 – SBL structure alignment). A granite boulder was cored in borehole B501-02 for a total thickness of approximately 0.5 m. Granite gneiss bedrock was encountered beneath the overburden soils at depths of about 37.4 m below the ice surface (Elevation 141.2 m at Borehole B501-02 – SBL structure alignment) and about 28.5 m below the ice surface (Elevation 150.1 m at Borehole B502-02 – NBL structure alignment). The results of point load tests carried out on selected samples of the bedrock core indicate that the granite gneiss is strong. The depth of ice/water to the lakebed in this area varied from about 1.1 m to 1.4 m.

1.3 North Shore

Boreholes advanced for the investigation of Swamp 502 immediately to the north of Straight Lake near the north abutment areas have been used for the Straight Lake preliminary or Phase 1 investigations completed to date. At the location of three (3) of the boreholes (Boreholes S502-01, S502-03, and S502-18) advanced as part of the recent Swamp 502 investigation by Golder, the subsoils immediately beneath the ground surface in the north abutment areas along the proposed bridge structure alignments generally consist of a stratum of very soft peat/very loose organic silt ranging in thickness from about 0.3 m to 1.2 m. Underlying the peat/organic silt is a deposit of interlayered non-cohesive and cohesive materials comprised of very loose to loose silts to sands and very soft clayey silt to silty clay between about 0.9 m and 7.4 m thick, becoming thicker in a northerly direction beyond the north abutment areas. This interlayered stratum is underlain by a cohesive deposit of soft to stiff clayey silt to silty clay between about 7.0 and 12.4 m thick, which in turn is underlain by a non-cohesive deposit

of loose to dense silt to sandy silt between about 1.4 m and 5.9 m thick, which appears to become substantially thinner in a northerly direction along the structure alignments within the north abutment areas. All three boreholes were terminated upon auger or casing refusal on inferred bedrock at depths ranging from about 27.4 m below the existing ground surface (Elevation 152.1 m) at Borehole S502-01 – SBL structure alignment) to about 14.7 m below existing ground surface (Elevation 166.6 m) at Borehole S502-18 – NBL structure alignment. Given that the bedrock as encountered in the near shore boreholes is between about Elevations 140 m (SBL) and 150 m (NBL), it appears that the sloping nature of the bedrock (upwards from south to north) can result in significant variability in the thickness of the overburden over relatively short distances north of the lake shore. The bedrock was not cored during the investigation in these areas. In general, the topography around the north abutments and approach embankments shows that the ground surface gently rises from the lake surface at about Elevation 178 m and extends to as high as about Elevation 184 m, resulting in a height of up to about 6 m above the lake level.

2.0 PRELIMINARY FOUNDATION ALTERNATIVES

Based on the subsurface conditions at the Straight Lake bridge structure sites as described above, a discussion on the preliminary foundation alternatives for the Abutment areas and for the Piers are detailed in Tables 1 to 4 including advantages, disadvantages, relative costs and risks/consequences. A summary of the preferred foundation alternative for the abutments and in-water pier areas are described below.

2.1 Abutment Areas

At the south abutments, considering the presence of the bedrock outcrops and/or the shallow nature of the overburden near the south shore of the lake at the location of the structure crossing alignments, shallow foundations consisting of spread footings founded directly on the bedrock or founded on compacted granular pads overlying the bedrock are considered to be the preferred foundation alternative for both the SBL and NBL bridge structures.

At the north abutments, the overburden thickness is significantly greater near the north shore of the lake at the location of the structure crossing alignments. Considering the presence of the very loose/soft to firm overburden soils and significant depth to bedrock, it is considered that deep foundations (i.e. steel H-piles) driven to refusal on bedrock are the preferred foundation alternative for both the SBL and NBL bridge structures.

2.2 In-water Piers

It is our understanding that at present, consideration is being given to designing the Straight Lake SBL and NBL structures as either:

- 4-span structures (employing two ≈57 m end-spans and two ≈75 m central spans), with two piers, each located near the north and the south shores of Straight Lake and one (1) pier located near the center of Straight Lake; or,
- 5-span structures (employing two ≈45 m end-spans and three ≈58 m central spans), with two piers, each located near the north and the south shores of Straight Lake and two (2) piers located near the center of Straight Lake.

The construction of deep foundations for the in-water piers is considered feasible at this site regardless of the structure arrangement selected. The in-water work would most likely be carried out from floating platforms but it is noted that some dredging may be necessary near the north shore of the lake for the piers at this location given

the shallow depth of water in this area. A list of the potential challenges associated with foundation design and construction for the in-water piers are discussed below.

Challenges

- Depth of lake water (ranging from about 1.1 m to 4.3 m deep at the investigated locations), variable overburden thickness (ranging from about 9 m to 45 m thick) and presence of weak soils, especially within the zone of influence for lateral pile design (i.e. at least 6 to 8 pile diameters below lake bed), will make design of foundations to resist lateral loads difficult (especially for vertical piles). At the south shore of the lake in particular, the overburden is relatively thin (shallow depth to bedrock) and weak and will provide very low lateral resistance. More rigorous non-linear, soil-structure interaction modelling to predict foundation response under lateral loads and/or moments will be required.
- Highly variable depths to bedrock (i.e. ranging from about 13 m to 49 m below lake level at the investigated locations; unknown depth to bedrock for center-most piers for the 5-span option) will result in widely varying pile lengths, especially if battered. It will likely not be possible to predict the range in pile length variability based on foundation investigations. Great depth to refusal in the center portion of lake will require long pile elements.
- Based on the boreholes drilled to date, the bedrock in the near shore areas appears to be sloping in the near shore areas at angles ranging from about 30° to 40°, and possibly up to 45°. As a result of the bedrock sloping at up to about 40° (or more) near the south shore in particular, seating driven piles and/or large-diameter caissons into the strong to very strong granite gneiss bedrock is expected to be difficult, especially for battered piles.
- Where caissons/drilled steel casings are used, if liners/casings are not adequately sealed into bedrock, there is a risk of debris and materials impeding rock socket construction.
- Presence of cobbles and boulders at depths ranging from 28 m to 33 m below the lake surface (about Elevations 145 m to 150 m) near the north shore of the lake will require consideration for driven pile foundations; the use of driven steel tube piles could be problematic. Heavier H-pile sections, fitted with pile shoes and flange stiffeners may be required to deal with obstructions.

Based on the subsurface conditions encountered at the site, the 4-span alternative is the most preferred from a foundations perspective. The 5-span alternative has the disadvantage of requiring one (1) additional in-water pier construction for each bridge as well as having the first pier(s) on the south side of the lake closer to the shore where the overburden is thinner and the bedrock surface is anticipated to be sloped at a steeper angle making foundation installation at this location more difficult.

Given that the subsurface soils immediately below the lake bed are comprised of very loose organic silts and soft to firm silty clays, developing sufficient lateral resistance from smaller diameter pile elements may be difficult. As such, a large number of battered piles may be required or consideration would have to be given to installing groups of smaller piles (i.e. sets of 3 or 4 pile elements) through 1.2 m diameter steel casings driven through the organics and clayey deposits into the upper portion of the lower sand and silt stratum. The 1.2 m diameter casing would be driven first, cleaned-out prior to installing the 3 or 4 internal pile elements and filled with tremie concrete upon completion of piling. These large diameter casing elements would provide a larger lateral resistance than individual smaller pile elements.

It is anticipated that conventional cofferdam construction will be difficult at this site given the presence of the weak/very loose upper soils and considering that the depth of the water at most of the pier locations is on the

order of about 3 m to 4 m. If conventional steel sheet pile cofferdam construction is used, all excavation and foundation piling installation would have to be carried out in-the-wet until a sufficiently thick/heavy tremie-plug is established on the base of the cofferdams otherwise there is a high risk that base heave failure will occur during construction. As such, it is recommended that consideration be given to using prefabricated cofferdam, constructed with pre-drilled holes and steel tube sleeves large enough to accommodate the foundation pile elements. These cofferdams could be floated into place, act as a template during pile installation and upon completion of piling, could be filled with concrete to form part of the pile cap.

After considering the potential difficulties associated with the subsurface conditions, with the installation of the different foundation types at the in-water pier locations and with the construction of cofferdams, the preferred foundation types for the in-water piers of the 4-span alternative are as follows:

- Near shore piers on the south side of the lake - founded on either:
 - drilled steel casings (600 mm or 760 mm diameter), socketed into bedrock and filled with concrete; or,
 - micropiles (273 mm diameter), arranged in sets of 3 or 4, drilled and socketed into bedrock, installed within a 1.2 m diameter outer steel casing driven into the upper portion of the sand and silt stratum .
- Central piers and near shore piers on the north side of the lake - founded on steel H-Piles driven to refusal on bedrock, arranged in sets of 3 installed within a 1.2 m diameter outer steel casing driven into the upper portion of the sand and silt stratum.

Based on the limited subsurface information presently available, it is noted that piling at the currently proposed location of the near shore pier(s) on the south side of the lake may be challenging given the estimated slope of the bedrock surface as well as the thinness and composition of the overburden. If possible, consideration should be given to lengthening the end-span of the structures and locating the southerly most piers further into the lake where the overburden is expected to be thicker and the bedrock surface slope might be less steep than nearer to the shore.

2.3 Approach Embankments

The current highway profile as shown in the preliminary General Arrangement drawings indicates that approach embankments up to about 15 m high will be required at the south and north approaches to the new Straight Lake SBL and NBL structures. At the south approach area, given that the overburden is relatively thin and bedrock is located at/near the ground surface, construction of the approach embankment should be relatively straight-forward with no significant settlement or stability issues. At the north approach area, however, the presence of the 10 m to 15 m thick firm silty clay stratum results in the design of these embankments being challenging.

Preliminary stability analyses for the north approach embankments indicate that a Factor of Safety (FoS) less than 1.0 will result for the proposed embankments constructed in a single-stage with conventional fill. It is not possible to achieve a $FoS \geq 1.3$ using a stability berm alone at the toe of the front slopes without the berm fill(s) encroaching into Straight Lake. Staged construction, in combination with a toe berm(s) and wick drains to accelerate consolidation and strength gain of the cohesive deposits between stages will likely be required to achieve an adequate FoS. In addition, some amount of lightweight (EPS) fill will also likely be required to reduce post-construction settlements to an acceptable level. The settlements associated with the construction of the north approach embankments will result in downdrag forces and dragloads acting on the pile foundations at the north abutments which will have to be considered in the design and/or the construction will have to be staged such that the approach fills and associated settlements are completed prior to the installation of the piles.

Given the expected challenges associated with the design and construction of the embankments at the north approaches, if possible, consideration should be given to lowering the grade of the highway in this area.

3.0 CLOSURE

We trust that this letter is sufficient for your immediate requirements. Please contact us if you have any questions.

Yours truly,

DRAFT

DRAFT

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[https://capws.golder.com/sites/0911116014highway69fourlaning/contract 5/correspondence/09-1111-6014 13may30 summary of the subsurface conditions at straight lake structures_mas \(3\)_jpd_2.docx](https://capws.golder.com/sites/0911116014highway69fourlaning/contract%205/correspondence/09-1111-6014%2013may30%20summary%20of%20the%20subsurface%20conditions%20at%20straight%20lake%20structures_mas%20(3)_jpd_2.docx)

Attachment(s): Drawing 1 – Straight Lake SBL Crossing – Borehole Locations and Soil Strata
Drawing 2 – Straight Lake NBL Crossing – Borehole Locations and Soil Strata
Table 1 – Evaluation of Foundation Alternatives – South Abutments
Table 2 – Evaluation of Foundation Alternatives – North Abutments
Table 3 – Evaluation of Foundation Alternatives – In-water Piers – Near Shore South Side
Table 4 – Evaluation of Foundation Alternatives – In-water Piers – Central and Near Shore North Side

**Drawings
Superceded**

TABLE 1
EVALUATION OF FOUNDATION ALTERNATIVES – SOUTH ABUTMENTS
Highway 69 Four Laning – Straight Lake Bridge – NBL and SBL Structures

Foundation Option	Rank	Advantages	Disadvantages	Relative Costs	Risks / Consequences
Spread Footings founded on Bedrock or on Compacted Granular Pads over Bedrock	1	<ul style="list-style-type: none"> ■ Relative ease of construction. ■ Negligible post-construction settlement. 	<ul style="list-style-type: none"> ■ Some excavation in strong to very strong bedrock (or additional mass concrete) may be required to achieve a level bearing surface for footings bearing on bedrock. ■ Bedrock excavation, if required, may have to be blasted using controlled blasting techniques to minimize shattering and over-break. ■ Fully integral abutment design not achievable. 	<ul style="list-style-type: none"> ■ Lower relative cost than piled foundation option. ■ Additional costs for vertical dowels, if required to improve lateral resistance. 	<ul style="list-style-type: none"> ■ Must take measures to ensure integrity of rock below the footings or repairing using mass concrete may be required during construction in areas of over-break / over-shatter if/where bedrock excavation required.
Steel H-piles driven through Compacted Granular Pads to refusal on Bedrock	2	<ul style="list-style-type: none"> ■ Negligible post-construction settlement; and ■ Fully integral abutment design achievable. 	<ul style="list-style-type: none"> ■ Minimum 8 m thick granular pad and/or overburden required (below pile cap) to achieve adequate lateral resistance for integral abutment design. 	<ul style="list-style-type: none"> ■ Higher relative cost than spread footings due to additional costs for piling. 	<ul style="list-style-type: none"> ■ Compacted granular pad must extend laterally at least 10 pile diameters in all directions around the pile cap.

Prepared By: MAS

Checked By: JPD

Reviewed By: JMAC

TABLE 2
EVALUATION OF FOUNDATION ALTERNATIVES – NORTH ABUTMENTS
Highway 69 Four Laning – Straight Lake Bridge – NBL and SBL Structures

Foundation Option	Rank	Advantages	Disadvantages	Relative Costs	Risks / Consequences
Steel H-piles driven to refusal on bedrock	1	<ul style="list-style-type: none"> ■ Negligible post-construction settlement. ■ Fully integral abutment design achievable. 	<ul style="list-style-type: none"> ■ Potential for encountering obstructions (cobbles and boulders) during pile driving; may require heavier pile section and/or pile shoes/flange stiffeners. ■ Potential for difficulties seating driven piles on sloping, strong to very strong bedrock; rock points will be required. ■ Dragloads and/or appropriate approach embankment construction staging need to be considered in design. 	<ul style="list-style-type: none"> ■ Lower relative cost than drilled steel casings. 	<ul style="list-style-type: none"> ■ Difficulties seating piles on strong to very strong, sloping bedrock. Rock points likely required.
Drilled Steel Casings (0.6 m to 0.76 m Ø) socketed into bedrock using DTH drilling	2	<ul style="list-style-type: none"> ■ Negligible post-construction settlement; and ■ Higher bearing resistances than for driven steel piles; requires fewer pile elements. 	<ul style="list-style-type: none"> ■ Requires specialty contractor to install piles. ■ Permanent steel casings required during construction to control potential ground losses in non-cohesive soils. 	<ul style="list-style-type: none"> ■ Higher relative cost than driven Steel H-piles. 	<ul style="list-style-type: none"> ■ Risk of difficulties drilling rock sockets on sloping and strong to very strong bedrock which could raise costs and potentially affect schedule.

Foundation Option	Rank	Advantages	Disadvantages	Relative Costs	Risks / Consequences
			<ul style="list-style-type: none"> Fully integral abutment design may not be possible. 		
Shallow Spread Footings founded on overburden or on engineered granular pads	NF	<ul style="list-style-type: none"> Relatively straightforward footing construction. 	<ul style="list-style-type: none"> Presence of very loose/soft to firm overburden soils and significant depth to competent founding stratum will result in very low geotechnical resistance(s). Significant settlement expected to occur. Fully integral abutment design not achievable. Cofferdam construction for working in-the-dry will be required but will be difficult to construct. 	<ul style="list-style-type: none"> Lower relative cost than piled foundation options, but cost of cofferdam and unwatering will be high. 	<ul style="list-style-type: none"> Not feasible due to presence of very weak and compressible overburden soils. May not be able to properly seal cofferdam thereby increasing cost for unwatering

NF: Foundation option is not feasible

DTH: Down-The-Hole (hammer-type for pile installation)

Prepared By: MAS

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Reviewed By: JMAC

TABLE 3
EVALUATION OF FOUNDATION ALTERNATIVES – IN-WATER PIERS – Near Shore South Side
Highway 69 Four Laning – Straight Lake Bridge – NBL and SBL Structures

Foundation Option	Rank	Advantages	Disadvantages	Relative Costs	Risks / Consequences
Drilled Steel Casings (0.6 m to 0.76 m Ø) socketed into bedrock using DTH drilling	2	<ul style="list-style-type: none"> High axial capacity. Higher lateral capacity than small diameter pile elements. Smaller number of pile elements required per pier. 	<ul style="list-style-type: none"> Cannot be easily battered. Requirement for large drill rig set-up on barge in lake may make over-water construction difficult. Requires specialty contractor to install piles. 	<ul style="list-style-type: none"> Higher relative cost than driven piles. Smaller number of pile elements may result in some cost savings. 	<ul style="list-style-type: none"> Difficult construction conditions and potential for difficulties seating larger diameter steel casings on sloping, strong to very strong bedrock and drilling rock sockets which will raise costs and potentially affect schedule. If casings not adequately sealed, and the base not properly cleaned, there is a potential of debris and materials impeding rock socket construction.
Micropiles (0.273 m Ø) socketed into bedrock using DTH drilling; arranged in sets of 3 or 4; installed within 1.2 m diameter outer steel casing	1	<ul style="list-style-type: none"> Relatively straight forward construction. Small diameter micropiles drilled using DTH offers best chance of seating and socketing piles in sloping, strong to very strong bedrock. Installed in groups of 3 or 	<ul style="list-style-type: none"> Requires specialty contractor to install piles. Requires more structural design consideration at interface between micropiles and large diameter 	<ul style="list-style-type: none"> Higher relative cost than driven piles. Lower cost per pile than larger diameter drilled steel casings, but more piles may be required as a result of lower individual axial 	<ul style="list-style-type: none"> Construction difficulties when installing and socketing 3 or 4 micropiles in close proximity in a single group. May require input/review from Micropile Contractor at

Foundation Option	Rank	Advantages	Disadvantages	Relative Costs	Risks / Consequences
		<p>4 within 1.2 m diameter upper steel casings, backfilled with grout, structurally advantageous to provide large, stiff cross-section for higher lateral resistance in weak overburden.</p> <ul style="list-style-type: none"> ■ Pile cap/cofferdam design would be similar to other piers. 	<p>outer casing.</p>	<p>capacity.</p> <ul style="list-style-type: none"> ■ Additional cost for installing 1.2 m diameter outer steel casing. 	<p>design stage.</p>
Steel H-piles (HP310x110) driven to refusal on bedrock; arranged in sets of 3; installed within 1.2 m diameter out steel casing	NR	<ul style="list-style-type: none"> ■ Relatively straight forward construction. ■ Installed in groups of 3 within 1.2 m diameter upper steel casings, backfilled with grout, structurally advantageous to provide large, stiff cross-section for higher lateral resistance in weak overburden. 	<ul style="list-style-type: none"> ■ Weak and thin overburden soils may result in near-shore piles having insufficient length to support piers (since piles not socketed into bedrock). ■ May be difficult to seat driven piles on sloping (up to 40° or more), strong to very strong bedrock (even with rock points) considering the presence of the weak, thin overburden. 	<ul style="list-style-type: none"> ■ Lower cost per pile than drilled steel casings or micropiles. ■ Additional cost for installing 1.2 m diameter outer steel casing. 	<ul style="list-style-type: none"> ■ Shallow, weak overburden may result in insufficient length of piles to develop required lateral resistance since piles not socketed into bedrock. ■ Difficulties seating driven piles on sloping, strong to very strong bedrock which will raise costs and potentially affect schedule. ■ Variable pile lengths during construction due to variable depth to bedrock (not readily determined at design stage).

Foundation Option	Rank	Advantages	Disadvantages	Relative Costs	Risks / Consequences
Steel H-piles (HP310x110) or concrete filled Steel Tube Piles (300 mm Ø) driven to refusal on bedrock	NR	<ul style="list-style-type: none"> Relatively straight forward construction. 	<ul style="list-style-type: none"> Weak and thin overburden soils may result in near shore piles having insufficient length to support piers or to provide sufficient lateral resistance (since piles are small diameter and not socketed into bedrock). May be difficult to seat driven piles on sloping (up to 40° or more), strong to very strong bedrock (even with rock points) considering the presence of the weak, thin overburden. 	<ul style="list-style-type: none"> Lower cost per pile than drilled steel casings or micropiles, but large number of pile elements may be required to satisfy lateral resistance requirements. 	<ul style="list-style-type: none"> Difficulties seating driven piles on sloping, strong to very strong bedrock which will raise costs and potentially affect schedule Variable pile lengths during construction due to variable depth to bedrock (not readily determined at design stage). Insufficient lateral resistance offered by small diameter pile elements.
Shallow Spread Footings on Overburden in Lakebed	NF	<ul style="list-style-type: none"> Relatively straightforward footing construction. 	<ul style="list-style-type: none"> Very low factored axial geotechnical resistance at ULS and geotechnical reaction at SLS. Cofferdam construction for working in-the-dry will be difficult. 	<ul style="list-style-type: none"> Lower relative cost than piled foundation options, but cost of cofferdam will be high. 	<ul style="list-style-type: none"> Not feasible due to depth of lake water and presence of very weak and compressible near surface overburden soils below lakebed.

NF: Foundation option is not feasible

NR: Foundation option is not recommended

DTH: Down-The-Hole (hammer-type for pile installation)

Prepared By: MAS

Checked By: JPD

Reviewed By: JMAC

TABLE 4
EVALUATION OF FOUNDATION ALTERNATIVES – IN-WATER PIERS – Central and Near Shore North Side
Highway 69 Four Laning – Straight Lake Bridge – NBL and SBL Structures

Foundation Option	Rank	Advantages	Disadvantages	Relative Costs	Risks / Consequences
Steel H-piles (HP310x110) driven to refusal on bedrock; arranged in sets of 3; installed within 1.2 m diameter outer steel casing	1	<ul style="list-style-type: none"> ■ Relatively straight forward construction. ■ Installed in groups of 3 within 1.2 m diameter upper steel casings, backfilled with grout, structurally advantageous to provide large, stiff cross-section for higher lateral resistance in weak overburden. 	<ul style="list-style-type: none"> ■ Potential for encountering obstructions (cobbles and boulders) during pile driving; heavier pile sections, pile shoes and flange stiffeners may be required. ■ Rock points required to ensure proper seating on sloping bedrock. 	<ul style="list-style-type: none"> ■ Lower cost per pile than drilled steel casings. ■ Additional cost for installing 1.2 m diameter outer steel casing. 	<ul style="list-style-type: none"> ■ Variable pile lengths during construction due to variable depth to bedrock (not readily determined at design stage).
Drilled Steel Casings (0.6 m to 0.76 m Ø) socketed into bedrock using DTH drilling	2	<ul style="list-style-type: none"> ■ High axial capacity. ■ Higher lateral capacity than small diameter pile elements. ■ Smaller number of pile elements required per pier. 	<ul style="list-style-type: none"> ■ Cannot be easily battered. ■ Requirement for large drill rig set-up on barge in lake may make over-water construction difficult. ■ Requires specialty contractor to install piles. 	<ul style="list-style-type: none"> ■ Higher relative cost than driven piles. ■ Smaller number of pile elements may result in some cost savings. 	<ul style="list-style-type: none"> ■ If casings not adequately sealed, and the base not properly cleaned, there is a potential of debris and materials impeding rock socket construction.
Steel H-piles (HP310x110) or concrete filled Steel Tube Piles (300 mm Ø) driven to refusal on bedrock	3	<ul style="list-style-type: none"> ■ Relatively straight forward construction. 	<ul style="list-style-type: none"> ■ Potential for encountering obstructions (cobbles and boulders) during pile driving; heavier pile sections, shoes and flange 	<ul style="list-style-type: none"> ■ Lower cost per pile than drilled steel casings, but large number of pile elements may be required to satisfy lateral resistance 	<ul style="list-style-type: none"> ■ Variable pile lengths during construction due to variable depth to bedrock (not readily determined at design stage).

Foundation Option	Rank	Advantages	Disadvantages	Relative Costs	Risks / Consequences
			<p>stiffeners may be required.</p> <ul style="list-style-type: none"> Weak overburden soils do not provide adequate lateral resistance for small diameter pile elements; large number of battered piles may be required for design. 	requirements.	
Shallow Spread Footings on Overburden in Lakebed	NF	<ul style="list-style-type: none"> Relatively straightforward footing construction. 	<ul style="list-style-type: none"> Very low factored axial geotechnical resistance at ULS and geotechnical reaction at SLS. Cofferdam construction for working in-the-dry will be difficult. 	<ul style="list-style-type: none"> Lower relative cost than piled foundation options; but cost of cofferdam and unwatering will be high. 	<ul style="list-style-type: none"> Not feasible due to depth of lake water and presence of very weak and compressible near surface overburden soils below lakebed. May not be able to properly seal cofferdam thereby increasing cost for unwatering.

NF: Foundation option is not feasible

DTH: Down-The-Hole (hammer-type for pile installation)

Prepared By: MAS

Checked By: JPD

Reviewed By: JMAC

December 13, 2013

Project No. 09-1111-6014

Mr. Glenn Scheepstra
URS Canada Inc.
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L4B 4N4

**PRELIMINARY FOUNDATION MITIGATION OPTIONS – STRAIGHT LAKE NORTH APPROACHES AND SWAMP 502 EMBANKMENTS
HIGHWAY 69 FOUR-LANING FROM 1.7 KM NORTH OF HIGHWAY 529, NORTHERLY TO 3.9 KM NORTH OF HIGHWAY 522, 19.7 KM
GWP 5404-05-01; WP 5145-08-01 AND 5146-08-01 (STRAIGHT LAKE NBL AND SBL BRIDGE STRUCTURES) ; WP 5005-10-01 (CONTRACT 5 SWAMPS)**

Dear Sirs:

On November 27, 2013, Golder Associates Ltd. (Golder) attended a meeting with URS Canada Inc. (URS) to present and discuss potential design challenges and foundation mitigation options for the north approach embankments associated with the proposed Straight Lake bridge structures and the contiguous embankments in the adjacent Swamp 502 for the new Highway 69. Golder also attended a separate meeting on December 3, 2013 with MTO Foundations Section (MTO) to discuss the same issues and to provide a summary of the potential foundation mitigation options and discuss the feasibility of these alternatives from MTO's perspective. This letter has been prepared to summarize the items discussed during these two meetings. The contents of this document include a brief summary of subsurface conditions encountered at this site, an outline of foundation mitigation options considered for evaluation, including a table summarizing the advantages, disadvantages, relative costs and risks/consequences of each foundation mitigation alternative, and a brief discussion on the preferred alternative from a foundations perspective.

1.0 SUBSURFACE CONDITIONS

In general, the subsurface conditions encountered immediately beneath the north approach embankments of the proposed Straight Lake bridge structures, which are located near the north shoreline of Straight Lake (i.e. the southern limit of the adjacent Swamp 502), consist of the following:

- up to 4 m thick deposit of very soft peat/very loose organic silt overlying;
- up to 16.4 m thick deposit of soft to firm (s_u from 20 kPa to 45 kPa) clayey silt to clay overlying;
- up to 16.7 m thick deposit of silt to silt and sand underlain by;



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- granitic gneiss bedrock at a depth of about 27.1 m and 36.3 m below existing ground surface at the NBL and SBL alignment, respectively.

It should be noted that the bedrock surface rises and the cohesive deposit thins out in the area to the north of the proposed north abutments and approaches. Along the NBL alignment, the cohesive deposit is about 5 m thick at a distance of about 30 m north of the proposed abutment location. Along the SBL alignment, the cohesive deposit thins out to less than 3 m thick at 30 m north of the proposed abutment location.

2.0 POTENTIAL FOUNDATION MITIGATION OPTIONS

The current General Arrangement (GA) drawings of the Straight Lake bridge structures, received from URS on October 9, 2013, show approach embankments up to about 16.5 m high. In addition, the GA drawings illustrate an up to about 11.5 m high granular core beneath the abutment stem walls supporting perched pile caps.

Based on the subsurface conditions below the footprint of the proposed embankments (as discussed in Section 1.0), the embankment geometry and the abutment configuration, stability analyses indicate that the Factor of Safety (FoS) will be much less than 1.0 for deep-seated global failure surfaces that would impact the operation of the roadway at the front slopes of both embankments, on the east side slope of the NBL embankment and on the west side slope of the SBL embankment. As such, the proposed embankments cannot be constructed without implementing significant foundation mitigation measure(s) that will address the stability issues. In addition, based on the results of settlement analyses, which assume that the near surface organic materials are sub-excavated and replaced with rock fill, the settlement of the foundation soils (not including settlement of the rock fill itself) is expected to be between about 2 m and 2.5 m if the embankments were constructed out of conventional fill materials. The majority of this settlement is due to primary consolidation within the cohesive deposit. In addition, the magnitude of secondary consolidation (creep) settlement is expected to be up to about 115 mm per log-cycle of time for this area. It should be noted that the post-construction settlement criterion for the north approach embankments is only 25 mm over a 20-year period following completion of construction. Consequently, significant foundation mitigation measure(s) will also be required to minimize post-construction settlements in order to ensure long-term performance of the roadway.

Multiple foundation mitigation alternatives have been considered to mitigate the stability and time dependent settlement issues, including:

- | | |
|---|--|
| ■ Construction of toe berms | ■ Sub-excavation and replacement |
| ■ Installation of wick drains | ■ Staged construction |
| ■ Installation of Rammed Aggregate Piers (RAPs) | ■ Preloading / surcharging |
| ■ Use of lightweight fill | ■ Lengthening the bridges by relocating the currently proposed north abutments approximately 30 m to the north |

These alternatives, including various combinations of the alternatives presented above, have been evaluated and ranked on the basis of advantages, disadvantages, relative costs and risks/consequence and are summarized in Table 1 following the text of this letter.

3.0 PRELIMINARY RECOMMENDATIONS

Based on our discussions with URS and MTO and assuming that the currently proposed north abutment location remains unaltered, one of the following foundation mitigation options could potentially be adopted:

- Full sub-excavation to a depth of up to 20 m below ground surface and replacement with granular fill beneath the abutment area (refer to Figure 1);
- Reduction in the height of the granular core and use of lightweight fill (expanded polystyrene (EPS)) (refer to Figure 2); or,
- Installation of wick drains in combination with staged construction and use of lightweight fill (EPS) (refer to Figure 3).

Although each of the alternatives presented above is considered technically feasible, the costs associated with all three options are very high, and more importantly, the risk level in terms of potential for instability, poor long-term performance of the roadway and encountering problems during construction is high as well, in particular for the full sub-excavation and wick drains/staged construction options.

A lower risk solution for achieving ground improvement to minimize stability/settlement issues is to increase the span lengths of the bridge structures and consequently shift the north abutments further away from Straight Lake. By relocating the abutments approximately 30 m to the north of their currently proposed locations, the following less expensive foundation mitigation options can be considered:

- Full sub-excavation to depths of up to about 15 m below ground surface and replacement with granular fill beneath the abutment area (refer to Figure 4);
- Reduction in the height of the granular core and use of lightweight fill (EPS) (refer to Figure 5);
- Installation of wick drains and staged construction with toe berms (refer to Figure 6); or,
- Installation of grouted RAPs and wick drains (or non-grouted RAPs) in combination with staged construction (refer to Figure 7).

All of these alternatives are feasible from a technical consideration, but the full sub-excavation option may be preferred based on it having the lowest cost and lowest risk so long as the environmental concerns in terms of sub-excavation and disposal of excavated spoil can be addressed. If not, then lightweight fill, or either the RAPs or wick drains in conjunction with staged construction could be adopted.

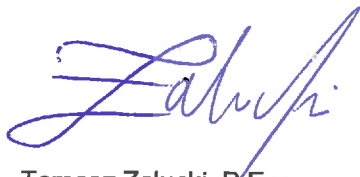
In conclusion, the preferred foundation mitigation option involves shifting the north abutments approximately 30 m to the north of their original location in conjunction with one of the above (second set of) mitigation options. This design change reduces the cost and mitigates the risks associated with slope stability and long-term settlement performance of the roadway compared to the foundation mitigation options associated with the originally proposed abutment configuration. While the costs associated with constructing longer bridge structures will be higher, at the same time the costs associated with the foundation mitigation option will be lower and the overall costs will likely be comparable to the costs corresponding to foundation mitigation options associated with the originally proposed abutment location. The selection of the most preferred foundation mitigation alternative (i.e. full sub-excavation, RAPs or wick drains and staged construction) will depend on a comparison of better defined costs, duration of construction and environmental restrictions, details of which will be provided in the Foundation Design Report for the Contract 5 swamp crossings / high fill areas.

4.0 CLOSURE

We trust that this letter is sufficient for your immediate requirements. Please do not hesitate to contact us if you require clarifications or have any questions.

Regards,

GOLDER ASSOCIATES LTD.



Tomasz Zalucki, P.Eng.,
Geotechnical Engineer



J. Paul Dittrich, P.Eng.,
Senior Geotechnical Engineer, Principal

TZ/JPD/JMAC/tz

CC: Mr. Dragan Ilic, P.Eng.
Mr. Sardar A. Nabi, P.Eng.

Attachments: Table 1 – Evaluation of Stability/Settlement Mitigation Options (5 pages)
Figure 1 – Front Slope Stability – Full Sub-Excavation (Undrained Conditions)
Figure 2 – Front Slope Stability – Lightweight Fill (Undrained Conditions)
Figure 3 – Front Slope Stability – Wick Drains and Lightweight Fill (Drained Conditions)
Figure 4 – Front Slope Stability – Relocated North Abutment and Full Sub-Excavation (Undrained Conditions)
Figure 5 – Front Slope Stability – Relocated North Abutment and Lightweight Fill (Undrained Conditions)
Figure 6 – Front Slope Stability – Relocated North Abutment and Wick Drains (Drained Conditions)
Figure 7 – Front Slope Stability – Relocated North Abutment and Rammed Aggregate Piers and Wick Drains (Drained Conditions)

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STRAIGHT LAKE NORTH APPROACHES AND SWAMP 502 EMBANKMENTS HIGHWAY 69 FOUR-LANING

Table 1: Evaluation of Stability/Settlement Mitigation Options

Stability/Settlement Mitigation Option	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Toe Berms and preloading / surcharging	Not feasible	<ul style="list-style-type: none"> Due to the very close proximity of the proposed north approach embankments to Straight Lake, toe berms along the front slope of the approaches would need to extend well into Straight Lake in order to ensure an adequate Factor of Safety against deep-seated global failure surfaces that would impact the operation of the roadway. Excessive delay in construction (over 10 years) to meet the post-construction settlement criteria due to great thickness of clay deposit. 			
Installation of wick drains and staged construction	Not feasible	<ul style="list-style-type: none"> Assuming completely drained conditions due to installation of wick drains in combination with staged construction, an adequate Factor of Safety against deep-seated global failure surfaces that would impact the operation of the roadway cannot be achieved with this stand-alone alternative. 			
Installation of Rammed Aggregate Piers and staged construction	Not feasible	<ul style="list-style-type: none"> Assuming completely drained conditions and an increase in the overall strength/stiffness of the foundation soils due to installation of Rammed Aggregate Piers in combination with staged construction, an adequate Factor of Safety against deep-seated global failure surfaces that would impact the operation of the roadway cannot be achieved with this stand-alone alternative. 			
Full sub-excavation near the abutment area (up to 21.5 m deep)	7	<ul style="list-style-type: none"> Toe berms are not required. Potential elimination of deep foundations at north abutments (i.e. perched spread footings could be considered). 	<ul style="list-style-type: none"> Generation of very large volume of excess excavation spoil. Significant quantity of granular fill will be required for backfilling. Delay in construction associated with up to 21.5 m deep sub-excavation and replacement with granular fill operation. Specialized equipment (i.e. dragline) and additional effort required for deep sub-excavation and replacement. May require additional right-of-way to accommodate limits of deep sub-excavation. Installation of a sheet pile wall required along the north shoreline of Straight Lake to provide groundwater cut off during sub-excavation. 	<ul style="list-style-type: none"> Additional costs associated with sub-excavation (specialized dragline equipment required), disposal and replacement of weak/soft excavated spoil. Additional costs associated with installation of a sheet pile wall. Possible additional cost for acquiring additional right-of-way for deep sub-excavation. Possible reduced costs for construction of shallow foundations compared to deep foundations. 	<ul style="list-style-type: none"> High risk with maintaining stability of excavation slopes. Low risk with respect to maintaining stability of proposed embankments. Large sub-excavation may not be permitted due to environmental restrictions associated with the proximity of the north approach embankments to Straight Lake and other constraints associated with working within First Nations lands.



STRAIGHT LAKE NORTH APPROACHES AND SWAMP 502 EMBANKMENTS HIGHWAY 69 FOUR-LANING

Table 1: Evaluation of Stability/Settlement Mitigation Options

Stability/Settlement Mitigation Option	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Partial preloading and lightweight fill (EPS)	5	<ul style="list-style-type: none"> Improved stability – toe berms are not required. Reduced total settlement of foundation soils. Eliminates generation of large volume of excess excavation spoil. Faster construction than Rammed Aggregate Piers and staged construction due to fewer requirements for staging. Reduced lateral loads against the abutment wall. 	<ul style="list-style-type: none"> Very high cost of EPS construction materials due to large volume required – 16.5 m high embankments. Requires embankments to be constructed with a 1.5H:1V front slope to accommodate site constraints which likely is more difficult to maintain and does not meet MTO's adoption of 2H:1V slope inclination. Not feasible to install below the groundwater table (due to buoyancy forces). A minimum 125 mm thick reinforced concrete pad should be constructed on top of the EPS. The height of the proposed granular cores beneath the north abutments must be reduced, resulting in either a taller abutment stem wall or tall, large diameter caissons extending above original ground surface up to proposed underside of abutment. Instrumentation and monitoring program required to assess end of partial preload period. 	<ul style="list-style-type: none"> Cost of EPS fill is estimated to be between approximately \$5.5 million and \$6.5 million. Additional costs associated with the construction of a protective concrete cap on top of the EPS. Additional cost associated with the construction of a taller abutment wall or tall, large diameter caissons to support abutment. Additional cost for instrumentation and associated monitoring program. Reduced costs for disposal / management of excavation spoils as compared with full sub-excavation option. 	<ul style="list-style-type: none"> Low risk with respect to stability and post-construction settlement of foundation soils. Subject to the monitoring data collected during the partial preload period, the preload embankment may need to be left in place for an extended period of time.
Installation of Rammed Aggregate Piers in combination with staged construction and lightweight fill (EPS)	6	<ul style="list-style-type: none"> Size of toe berms may be reduced or eliminated by staged construction. Requires confirmation during detail Rammed Aggregate Pier design. Reduced time for primary consolidation settlement to complete. 	<ul style="list-style-type: none"> Detail Rammed Aggregate Pier design will be required. Additional time required for installation of Rammed Aggregate Piers. Delay in construction schedule to allow for sufficient settlement to occur to meet post-construction settlement criteria. 	<ul style="list-style-type: none"> Schedule impacts may increase overall project costs. Additional costs associated with Rammed Aggregate Pier design. Additional cost for the installation of Rammed Aggregate Piers. 	<ul style="list-style-type: none"> Some risk associated with the complexity of a Rammed Aggregate Pier design. Subject to the monitoring data collected during the staged construction phase, the embankment stages may need to be left in place for an extended period of time.



STRAIGHT LAKE NORTH APPROACHES AND SWAMP 502 EMBANKMENTS HIGHWAY 69 FOUR-LANING

Table 1: Evaluation of Stability/Settlement Mitigation Options

Stability/Settlement Mitigation Option	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
		<ul style="list-style-type: none"> Significant reduction or complete elimination of excavation spoil. 	<ul style="list-style-type: none"> Instrumentation and monitoring program required to monitor staged construction and to assess end of preload period. Requires specialist contractor to construct system. 	<ul style="list-style-type: none"> Additional cost of EPS fill. Additional costs associated with the construction of a protective cap on top of the EPS. Additional cost for instrumentation and associated monitoring program. 	
North abutments relocated 30 m to the north plus toe berms and preloading / surcharging	Not feasible	<ul style="list-style-type: none"> An adequate Factor of Safety against deep-seated global failure surfaces that would impact the operation of the roadway cannot be achieved with this alternative, even if the toe berms (approximately 40 m long) along the front slope extend to the shore of Straight Lake. Excessive delay in construction (over 2 years) to meet the post-construction settlement criteria. 			
North abutments relocated 30 m to the north plus partial preloading and lightweight fill (EPS)	4	<ul style="list-style-type: none"> Improved stability – toe berms are not required. Reduced total settlement of foundation soils. Eliminates generation of large volume of excess excavation spoil. Faster construction than Rammed Aggregate Piers and staged construction due to fewer requirements for staging. Reduced lateral loads against the abutment wall. 	<ul style="list-style-type: none"> Very high cost of EPS construction materials due to large volume required - 13.5 m high embankments. Requires embankments to be constructed with 2H:1V side and front slopes given the need for granular fill for levelling pad and conventional soil cover on side and front slopes. Not feasible to install below the groundwater table (due to buoyancy forces). A minimum 125 mm thick reinforced concrete pad should be constructed on top of the EPS. The height of the proposed granular cores beneath the north abutments must be reduced, resulting in either a taller abutment stem wall or tall, large diameter caissons extending above original ground surface up to proposed underside of abutment. 	<ul style="list-style-type: none"> Additional cost of EPS fill. Additional costs associated with the construction of a protective concrete cap on top of the EPS. Additional cost associated with the construction of a taller abutment wall or tall, large diameter caissons to support abutment. Additional cost for instrumentation and associated monitoring program. Reduced costs for disposal / management of excavation spoils as compared with full sub-excavation option. 	<ul style="list-style-type: none"> Low risk with respect to stability and post-construction settlement of foundation soils. Subject to the monitoring data collected during the partial preload period, the preload embankment may need to be left in place for an extended period of time.



STRAIGHT LAKE NORTH APPROACHES AND SWAMP 502 EMBANKMENTS HIGHWAY 69 FOUR-LANING

Table 1: Evaluation of Stability/Settlement Mitigation Options

Stability/Settlement Mitigation Option	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
			<ul style="list-style-type: none"> Instrumentation and monitoring program required to assess end of partial preload period. 		
North abutments relocated 30 m to the north plus full sub-excavation near abutment area (up to about 15.5 deep)	1	<ul style="list-style-type: none"> Toe berms are not required. Potential elimination of deep foundations at north abutments (i.e. perched spread footings could be considered instead). Basic construction method with few, if any, complexities. 	<ul style="list-style-type: none"> Generation of large volume of excess excavation spoil. Significant quantity of granular fill will be required for backfilling. Delay in construction associated with up to 15.5 m deep sub-excavation and replacement with granular fill operation. Specialized equipment (i.e. dragline) may be required, and additional effort required for deep sub-excavation and replacement. May require additional right-of-way to accommodate deep sub-excavation. Would require a sheet pile cut-off system to minimize water inflow from Straight Lake through the upper silt to sandy silt deposit. 	<ul style="list-style-type: none"> Additional costs associated with sub-excavation (specialized dragline equipment required), disposal and replacement of weak/soft, excavated spoil. Possible additional cost for acquiring additional right-of-way for deep sub-excavation. Additional cost associated with installation of a sheet pile wall, if required. Possible reduced costs for construction of shallow foundations compared to deep foundations. 	<ul style="list-style-type: none"> Some risk with maintaining stability of excavation slopes. Low risk with respect to maintaining stability of proposed embankments. Sub-excavation may not be permitted due to environmental restrictions and other cultural restrictions associated with working within First Nations' lands.
North abutments relocated 30 m to the north plus installation of grouted Rammed Aggregate Piers and wick drains (or non-grouted Rammed Aggregate Piers) with staged construction.	2	<ul style="list-style-type: none"> Size of toe berms may be reduced or eliminated by staged construction. Requires confirmation during detail Rammed Aggregate Pier and wick drain design. Reduced time for primary consolidation settlement to complete. Significant reduction or complete elimination of excavation spoil. 	<ul style="list-style-type: none"> Toe berms will be required if preload embankment (plus surcharge, if required) is constructed continuously in one stage. Detail Rammed Aggregate Pier and wick drain design will be required. Additional time required for installation of grouted Rammed Aggregate Piers and wick drains (or non-grouted Rammed Aggregate Piers). Delay in construction schedule to allow for sufficient settlement to 	<ul style="list-style-type: none"> Schedule impacts may increase overall project costs. Additional costs associated with Rammed Aggregate Pier and wick drain design. Additional cost for the installation of grouted Rammed Aggregate Piers, wick drains (or non-grouted Rammed Aggregate Piers), and instrumentation and associated monitoring program. 	<ul style="list-style-type: none"> Some risk associated with the complexity of a Rammed Aggregate Pier in combination with wick drain design. Subject to the monitoring data collected during the staged construction phase, the stage embankment may need to be left in place for an extended period of time.



STRAIGHT LAKE NORTH APPROACHES AND SWAMP 502 EMBANKMENTS HIGHWAY 69 FOUR-LANING

Table 1: Evaluation of Stability/Settlement Mitigation Options

Stability/Settlement Mitigation Option	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
			<ul style="list-style-type: none"> occur to meet post-construction settlement criteria. ■ Instrumentation and monitoring program required to monitor staged construction and to assess end of preload/surcharge period. ■ Specialist contractors required for either ground improvement system. 		
North abutments relocated 30 m to the north plus installation of wick drains and staged construction with toe berms	3	<ul style="list-style-type: none"> ■ Reduced time for primary consolidation settlement to complete. ■ Size of toe berms may be reduced by staged construction. Requires confirmation during detail wick drain design. ■ Avoids generation of large volume of excess excavation spoil. 	<ul style="list-style-type: none"> ■ Large toe berms may be required if preload embankment plus surcharge is constructed too quickly (i.e. not enough stages and delay periods) ■ Additional right-of-way may be required to accommodate large toe berms. ■ Detail wick drain investigation and design will be required. ■ Additional time required for installation of wick drains. ■ Increased magnitude of secondary consolidation (creep) settlement as a result of the accelerated completion of primary consolidation settlement. ■ Delay in construction schedule to allow for sufficient settlement to occur to meet post-construction settlement criteria. ■ Instrumentation and monitoring program required to monitor staged construction and to assess end of surcharge period. ■ Increased handling of surcharge fill (Granular 'B') to remove surcharge. ■ Specialist contractor required to install system. 	<ul style="list-style-type: none"> ■ Additional costs associated with detail wick drain investigation and design. ■ Additional cost for the installation of wick drains, instrumentation and associated monitoring program. ■ Additional costs associated with construction and materials for 2 m high surcharge and partial deconstruction of surcharge embankment upon completion of surcharge period. ■ Potential additional cost for acquiring additional right-of-way to accommodate large berms. 	<ul style="list-style-type: none"> ■ Some risk with respect to stability of surcharge embankment on weak/soft foundation soils if staged construction is employed. ■ Some risk associated with the complexity of a wick drain design. ■ Additional right-of-way may be required for large toe berms. ■ Subject to the monitoring data collected during the surcharge period, the surcharge embankment may need to be left in place for an extended period of time.



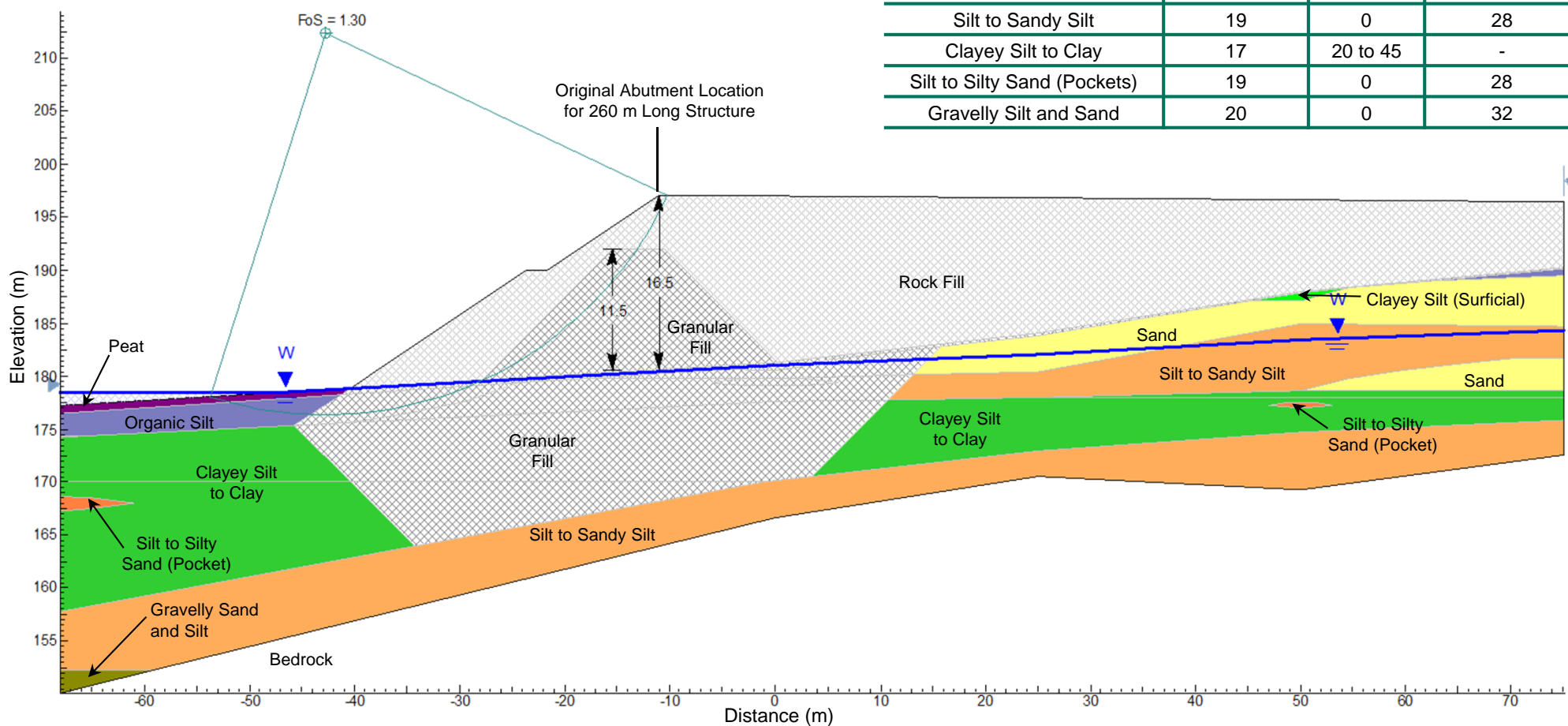
Highway 69 NBL – STA 20+990 to 21+075 (Swamp 502) Front Slope Stability – Full Sub-Excavation (Undrained Conditions)

Figure 1

NOTES:

1. All dimensions are in metres.
2. All rock fill slopes are constructed at 1.25H:1V, except for the front slopes, which are constructed at 1.5H:1V.
3. Granular fill core slopes are constructed at 1H:1V.

Material Name	Unit Weight (kN/m ³)	Cohesion (kPa)	Friction Angle (degrees)
Rock Fill	19	0	40
Granular Fill	21	0	34
Peat	12	1	28
Organic Silt	15	0	27
Clayey Silt (Surficial)	18	30	-
Sand	18.5	0	29
Silt to Sandy Silt	19	0	28
Clayey Silt to Clay	17	20 to 45	-
Silt to Silty Sand (Pockets)	19	0	28
Gravelly Silt and Sand	20	0	32





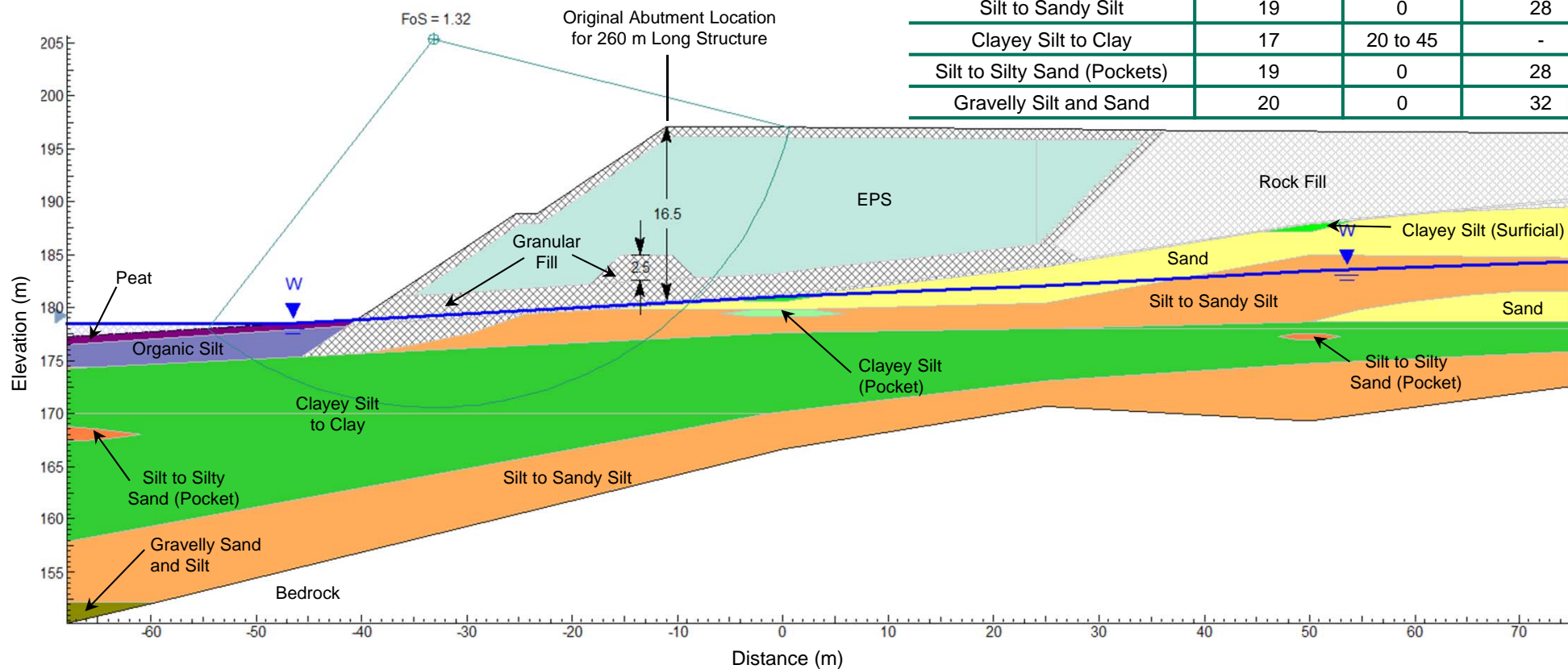
Highway 69 NBL – STA 20+990 to 21+075 (Swamp 502) Front Slope Stability – Lightweight Fill (Undrained Conditions)

Figure 2

NOTES:

1. All dimensions are in metres.
2. All granular fill slopes are constructed at 2H:1V, except for the front slopes, and the granular fill core slopes, which are constructed at 1.5H:1V and 1H:1V, respectively.

Material Name	Unit Weight (kN/m ³)	Cohesion (kPa)	Friction Angle (degrees)
Rock Fill	19	0	40
Granular Fill	21	0	34
Lightweight Fill (EPS)	0.5	15	0
Peat	12	1	28
Organic Silt	15	0	27
Clayey Silt (Surficial)	18	30	-
Sand	18.5	0	29
Clayey Silt (Pocket)	18	30	0
Silt to Sandy Silt	19	0	28
Clayey Silt to Clay	17	20 to 45	-
Silt to Silty Sand (Pockets)	19	0	28
Gravelly Silt and Sand	20	0	32





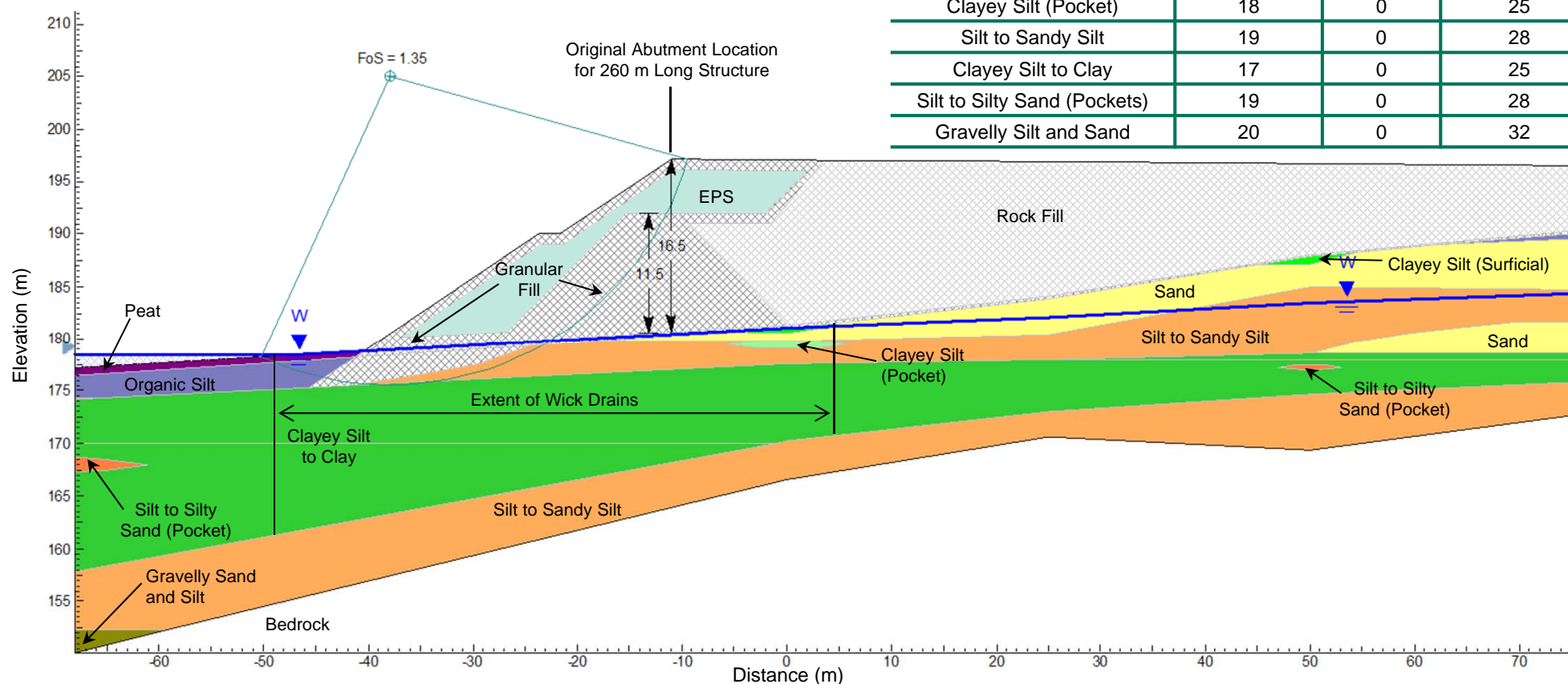
Highway 69 NBL – STA 20+990 to 21+075 (Swamp 502) Front Slope Stability – Lightweight Fill and Wick Drains (Drained Conditions)

Figure 3

NOTES:

1. All dimensions are in metres.
2. All granular fill slopes are constructed at 2H:1V, except for the front slopes, and the granular fill core slopes, which are constructed at 1.5H:1V and 1H:1V, respectively.
3. The slope stability model assumes that drained (long-term) conditions have been reached.

Material Name	Unit Weight (kN/m ³)	Cohesion (kPa)	Friction Angle (degrees)
Rock Fill	19	0	40
Granular Fill	21	0	34
Lightweight Fill (EPS)	0.5	15	0
Peat	12	1	28
Organic Silt	15	0	27
Clayey Silt (Surficial)	18	0	25
Sand	18.5	0	29
Clayey Silt (Pocket)	18	0	25
Silt to Sandy Silt	19	0	28
Clayey Silt to Clay	17	0	25
Silt to Silty Sand (Pockets)	19	0	28
Gravelly Silt and Sand	20	0	32





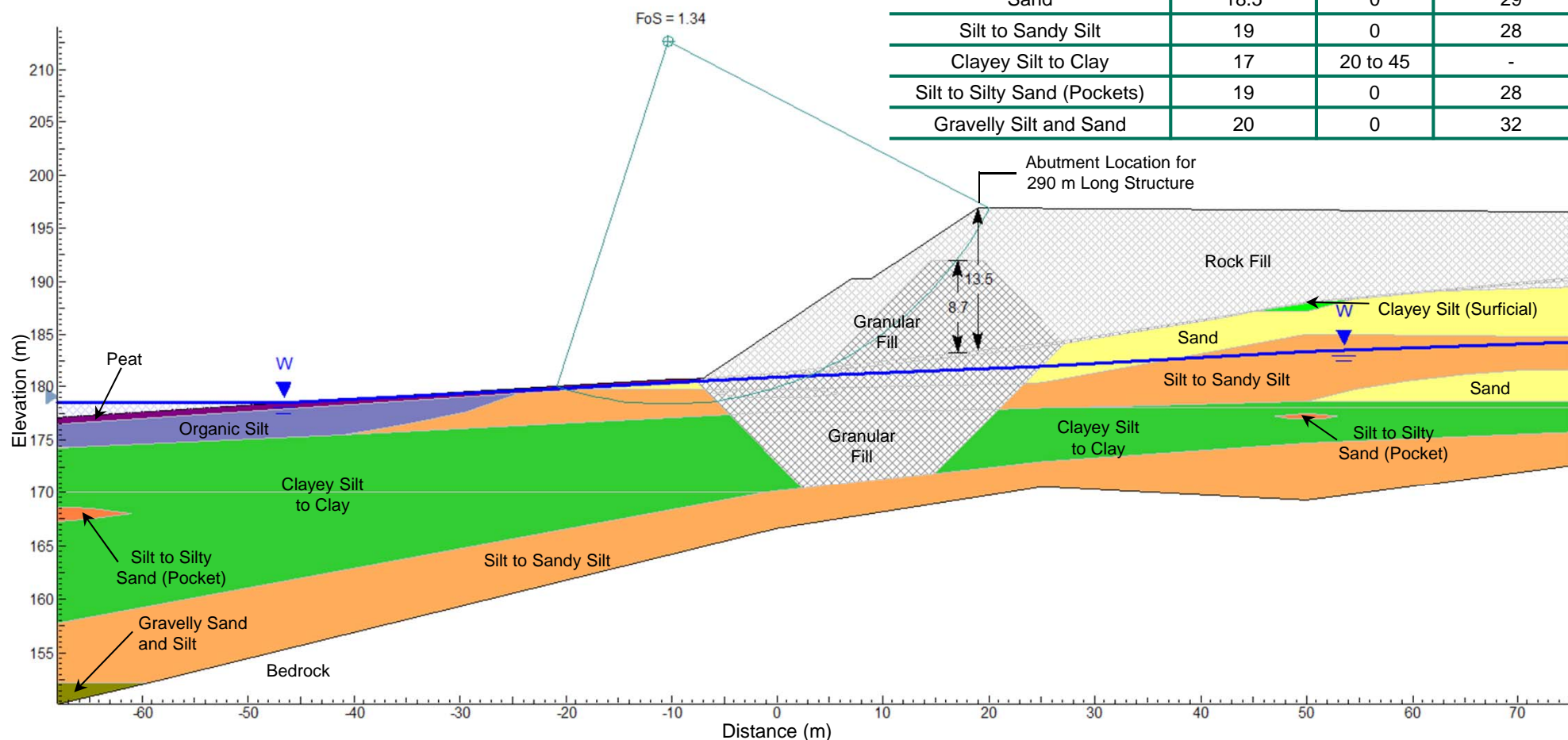
Highway 69 NBL – STA 20+990 to 21+075 (Swamp 502) Front Slope Stability – Relocated Abutment and Full Sub-Excavation (Undrained Condition)

Figure 4

NOTES:

1. All dimensions are in metres.
2. All rock fill slopes are at constructed at 1.25H:1V, except for the front slopes, which are constructed at 1.5H:1V.
3. Granular fill core slopes are constructed at 1H:1V.
4. The north abutment has been shifted 30 m to the north of its original location.

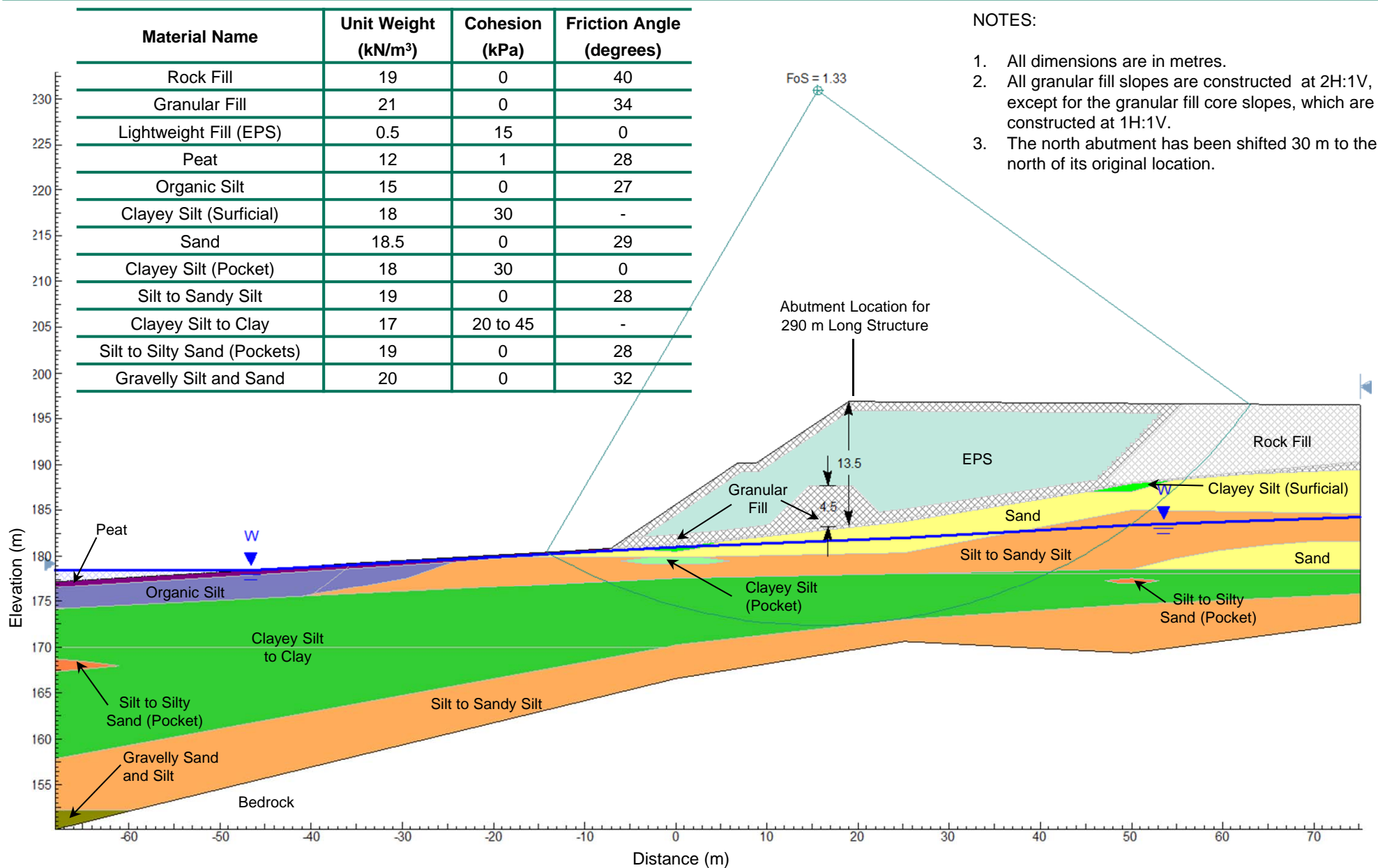
Material Name	Unit Weight (kN/m ³)	Cohesion (kPa)	Friction Angle (degrees)
Rock Fill	19	0	40
Granular Fill	21	0	34
Peat	12	1	28
Organic Silt	15	0	27
Clayey Silt (Surficial)	18	30	-
Sand	18.5	0	29
Silt to Sandy Silt	19	0	28
Clayey Silt to Clay	17	20 to 45	-
Silt to Silty Sand (Pockets)	19	0	28
Gravelly Silt and Sand	20	0	32





Highway 69 NBL – STA 20+990 to 21+075 (Swamp 502) Front Slope Stability – Relocated Abutment and Lightweight Fill (Undrained Conditions)

Figure 5





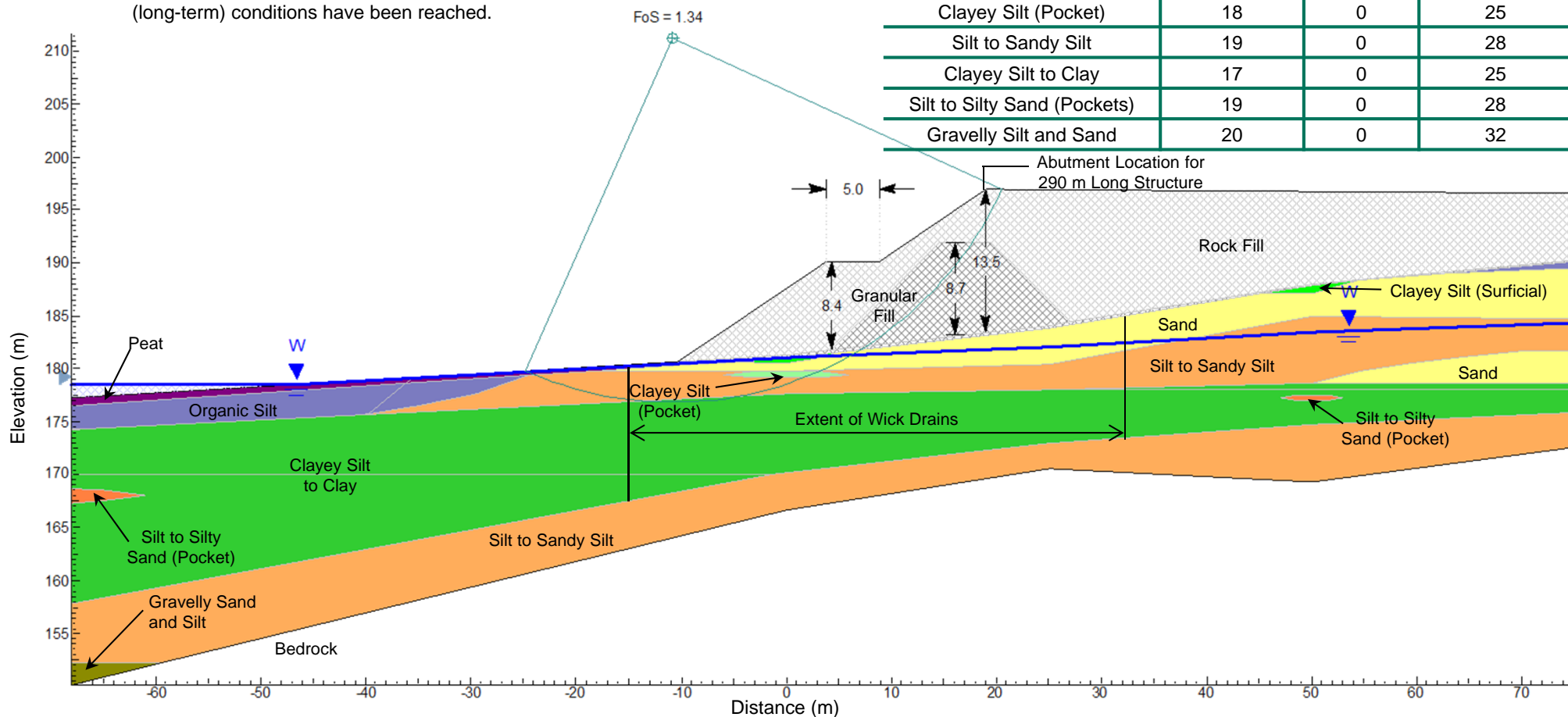
Highway 69 NBL – STA 20+990 to 21+075 (Swamp 502) Front Slope Stability – Relocated Abutment and Wick Drains (Drained Conditions)

Figure 6

NOTES:

1. All dimensions are in metres.
2. All rock fill slopes are at constructed at 1.25H:1V, except for the front slopes, which are constructed at 1.5H:1V.
3. Granular fill core slopes are constructed at 1H:1V.
4. The north abutment has been shifted 30 m to the north of its original location.
5. The slope stability model assumes that drained (long-term) conditions have been reached.

Material Name	Unit Weight (kN/m ³)	Cohesion (kPa)	Friction Angle (degrees)
Rock Fill	19	0	40
Granular Fill	21	0	34
Peat	12	1	28
Organic Silt	15	0	27
Clayey Silt (Surficial)	18	0	25
Sand	18.5	0	29
Clayey Silt (Pocket)	18	0	25
Silt to Sandy Silt	19	0	28
Clayey Silt to Clay	17	0	25
Silt to Silty Sand (Pockets)	19	0	28
Gravelly Silt and Sand	20	0	32





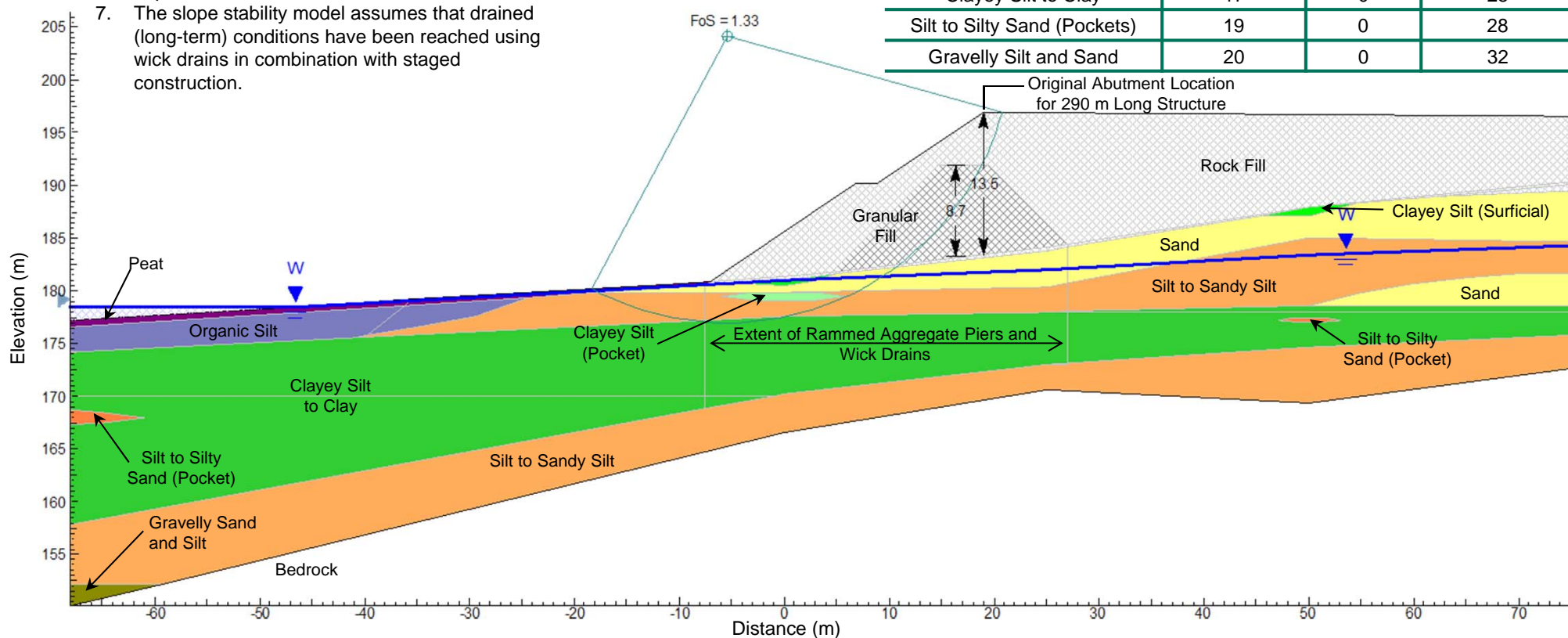
Highway 69 NBL – STA 20+990 to 21+075 (Swamp 502) Front Slope Stability – Relocated Abutment and Rammed Aggregate Piers and Wick Drains (Drained Conditions)

Figure 7

NOTES:

1. All dimensions are in metres.
2. All rock fill slopes are at constructed at 1.25H:1V, except for the front slopes, which are constructed at 1.5H:1V.
3. Granular fill core slopes are constructed at 1H:1V.
4. The north abutment has been shifted 30 m to the north of its original location.
5. The slope stability model assumes that ground improvement will consist of Rammed Aggregate Piers .
6. Composite soil parameters were assigned to soil deposits within the foundation treatment area.
7. The slope stability model assumes that drained (long-term) conditions have been reached using wick drains in combination with staged construction.

Material Name	Unit Weight (kN/m ³)	Cohesion (kPa)	Friction Angle (degrees)
Rock Fill	19	0	40
Granular Fill	21	0	34
Peat	12	1	28
Organic Silt	15	0	27
Clayey Silt (Surficial)	18	0	25
Sand	18.5	0	29
Clayey Silt (Pocket)	18	0	25
Silt to Sandy Silt	19	0	28
Clayey Silt to Clay	17	0	25
Silt to Silty Sand (Pockets)	19	0	28
Gravelly Silt and Sand	20	0	32





**PRE-DRAFT FOUNDATION REPORT – SWAMP CROSSING, HIGH FILL
AREAS AND DEEP CUT - HIGHWAY 69 GWP 5347-08-00; WP 5005-10-01**

**Table B1: Evaluation of Stability/Settlement Mitigation Options
Highway 69 SBL – STA 20+980 to STA 21+055 (High Fill 502)**

Stability/Settlement Mitigation Option	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Add Span to Bridge Structure (35 m long) and Preloading of Rock Fill (80 days)	1	<ul style="list-style-type: none"> Relocating abutment 35 m to the north will result in approach and highway embankment located on non-clayey foundation soils thereby reducing the risks associated with stability and long-term settlement. Abutment foundation can be constructed almost at existing ground surface (only a very small compacted granular pad required – about 1 m thick) Abutment relocation will result in lower maximum embankment height (on the order of 6.0 m) which reduces front slope stability issues; no toe berms required. Almost no delays associated with embankment construction; only short preload period required prior to completion of construction. Very low impact on the environmental footprint as less filling required and no excavation spoil produced. 	<ul style="list-style-type: none"> Rock fill embankment will still require short preload period to reduce post-construction settlement to acceptable limit. Additional pier and associated foundation element likely required. 	<ul style="list-style-type: none"> Estimate \$2.25M to \$2.75M (for structure) Possible additional costs for foundation investigation at new pier and bridge abutment location. 	<ul style="list-style-type: none"> Very low risk associated with embankment stability and post-construction settlement.



**PRE-DRAFT FOUNDATION REPORT – SWAMP CROSSING, HIGH FILL
AREAS AND DEEP CUT - HIGHWAY 69 GWP 5347-08-00; WP 5005-10-01**

**Table B1: Evaluation of Stability/Settlement Mitigation Options
Highway 69 SBL – STA 20+980 to STA 21+055 (High Fill 502)**

Stability/Settlement Mitigation Option	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
<p>Wick Drains (1 m spacing from STA 20+970 to 21+055) with Staged Embankment Construction and Toe Berm (12.5 m wide x 4.5 m) on Front Slope, Surcharging (2m high) with 4 m thick EPS installed behind abutment only</p> <p>Preliminary Analysis Estimates:</p> <ul style="list-style-type: none"> • 3 stages of embankment construction (incl. surcharge) • 2 delay periods of 330 days each between Stages 1 to 3 • Up to 460 days for Surcharge Period • Total construction time = 3.1 years 	2	<ul style="list-style-type: none"> ■ Lower cost compared with other options. ■ Avoids generation and handling of large volume of excess excavation spoil. ■ Avoids requirement for large volume of granular fill and rock fill for backfilling. 	<ul style="list-style-type: none"> ■ Long delay in construction schedule to allow for dissipation of excess pore pressures between each stage of construction plus additional delay during surcharge period to allow sufficient settlement to occur to meet post-construction settlement criteria. ■ Detail wick drain investigation and design will be required. ■ Additional time required for installation of wick drains. ■ Increased magnitude of secondary consolidation (creep) settlement in the short term as a result of the accelerated completion of primary consolidation settlement. ■ Instrumentation and monitoring program required to monitor staged construction and to assess end of surcharge period. ■ Increased handling of surcharge fill (Granular 'B') to remove surcharge. ■ North abutment and Pier #4 foundations cannot be constructed until after completion of embankment construction and surcharge period due to drag loads and lateral loads on pile foundations (caused by foundation soil settlement and lateral spreading). ■ Possible need for erosion protection (rip-rap) on front slope of toe berm adjacent to Straight Lake. 	<ul style="list-style-type: none"> ■ Estimate \$1.1M (includes wick drains, 2 m surcharge, toe berm and EPS behind abutment, detail wick drain investigation and design, instrumentation and monitoring). ■ Possible additional costs for EPS fill top-up (\$0.1M to \$0.2M) after surcharge period to satisfy post-construction settlement criteria. ■ Possible additional costs for pre-drilling at some wick drain locations to facilitate installation through compact sands/silts up to 10 m thick. ■ Relatively long delays (up to 3.1 years) for staged construction and surcharging impacts schedule and may further increase overall project costs. ■ Possible additional cost on adjacent abutment foundation design if need to design piles for dragloads. 	<ul style="list-style-type: none"> ■ Size of toe berm, number of stages and length of surcharge period may differ from the results of the preliminary analysis, thereby changing the estimated construction time. Requires confirmation during detail wick drain design. ■ Risk of instability of high surcharge embankment on weak/soft foundation soils even when staged construction is employed; additional instrumentation (such as Slope Inclinometers) may be required. ■ Higher risk associated with the complexity of a staged construction and wick drain design. ■ Subject to the monitoring data collected during the construction and surcharge period, the delay times between stages and the surcharge period may be longer than that estimated in the preliminary analysis; fill may need to be left in place for extended periods of time. ■ Risk that settlements during surcharge period are greater than estimated; may necessitate installation of EPS fill to top-up embankment and limit post-construction settlements; may also result in need to consider dragloads in abutment foundation design.



**PRE-DRAFT FOUNDATION REPORT – SWAMP CROSSING, HIGH FILL
AREAS AND DEEP CUT - HIGHWAY 69 GWP 5347-08-00; WP 5005-10-01**

**Table B1: Evaluation of Stability/Settlement Mitigation Options
Highway 69 SBL – STA 20+980 to STA 21+055 (High Fill 502)**

Stability/Settlement Mitigation Option	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
<p>Localized Sub-Excavation between STA 20+985 and 21+065 (up to a depth of 15 m below existing ground surface within the north abutment / front slope area; stepped excavation in benches north of abutment area);</p> <p>Surcharging (2 m high) for 375 days; installation of lightweight fill (3 m thick EPS core from STA 21+020 to 21+055)</p>	3	<ul style="list-style-type: none"> Slightly less volume of sub-excavated material and backfill compared to full subexcavation alternative. <p>Combination of mitigation measures satisfies both slope stability of front/side slopes and sufficiently reduces settlement during the preload/surcharge period and minimizes long-term settlement.</p>	<ul style="list-style-type: none"> Generation of large volume of excess excavation spoil (approx. 27,000 m³). Large quantity of Granular B Type II fill (20,000 m³ in abutment area to allow installation of driven piles at the proposed north abutment) and rock fill (7,000 m³ beyond abutment area) will be required for backfilling below existing ground surface. Delay in construction associated with up to 15 m deep sub-excavation and fill replacement operation. Additional effort for partial dewatering and staged excavation in benches with specialized dragline equipment (or possibly with long-stick backhoe) required for deep sub-excavation and replacement. Additional excavation in benches (5 m wide, 3 to 5 m high at 1.5:1V slopes) beyond immediate abutment area required to maintain stability of excavation face; still results in relatively large volume of excavation required. May require additional right-of-way to accommodate deep sub-excavation. Increased handling of surcharge fills (granular fill) upon completion of surcharge period and the rock fill to be replaced with EPS (3 m thick core and pavement structure). 	<ul style="list-style-type: none"> Estimate \$2.5M (includes granular fill in abutment area and rock fill north of abutment area, 2 m surcharge, 3 m thick EPS fill with concrete cap, instrumentation and monitoring) Additional costs associated with sub-excavation (partial dewatering and staged excavation using long-stick excavator equipment OR specialized dragline equipment required); Possible additional costs for disposal of large volume of weak/soft/wet, compressible deposits, depending on location of soil management area. Possible additional cost for acquiring additional right-of-way for deep sub-excavation. Schedule impacts for construction of different mitigation components may increase overall project costs. Somewhat reduced costs for disposal / management of excavation spoils as compared with full sub-excavation option. 	<ul style="list-style-type: none"> Potential for instability of below grade excavation slopes and difficulties excavating to required depths; may result in expansion of excavation footprint. Will achieve/ maintain stability of proposed embankments. Subject to the monitoring data collected during the surcharge period, the surcharge embankment may need to be left in place for an extended period of time (or may be shortened).



**PRE-DRAFT FOUNDATION REPORT – SWAMP CROSSING, HIGH FILL
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**Table B1: Evaluation of Stability/Settlement Mitigation Options
Highway 69 SBL – STA 20+980 to STA 21+055 (High Fill 502)**

Stability/Settlement Mitigation Option	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
			<ul style="list-style-type: none"> Delay in construction schedule to allow for sufficient settlement to occur to meet post-construction settlement criteria. Instrumentation and monitoring program required to monitor staged construction and to assess end of surcharge period. Requires 125 mm thick reinforced concrete pad constructed on top of the EPS. 		
Full Sub-Excavation (up to 15 m deep for entire footprint: from STA 20+985 to 21+075) and Preloading of Rock Fill (365 days)	4	<ul style="list-style-type: none"> Toe berms are not required. Mitigates instability of the embankment and minimizes long-term post construction settlement. Preload period could be reduced to 160 days by using granular material as backfill in subexcavation area north of abutment area (however, additional cost premium for this option). 	<ul style="list-style-type: none"> Generation of very large volume of excess excavation spoil (approx. 35,000 m³). Large quantity of Granular B Type II fill (20,000 m³ in abutment area to allow installation of driven piles at the proposed north abutment) and rock fill (15,000 m³ beyond abutment area) will be required for backfilling below existing ground surface. Delay in construction associated with up to 15 m deep sub-excavation and fill replacement operation. Additional effort for partial dewatering and staged excavation with specialized dragline equipment (or possibly with long-stick backhoe) required for deep sub-excavation and replacement. Additional post-construction settlement of rock fill itself requires substantial preloading period to reduce overall settlement in the short-term. 	<ul style="list-style-type: none"> Estimate \$2.6M (includes excavation and granular fill in abutment area and rock fill north of abutment area). Additional costs associated with sub-excavation (partial dewatering and staged excavation using specialized dragline equipment required); Possible additional costs for disposal of large volume of weak/soft/wet, compressible deposits, depending on location of soil management area. Possible additional cost for acquiring additional right-of-way for deep sub-excavation. Schedule impacts for preload period may increase overall project costs. 	<ul style="list-style-type: none"> Preloading would be required to reduce large post-construction settlement of rock fill. Potential for instability of below grade excavation slopes and difficulties excavating to required depths; may result in expansion of excavation footprint. Will achieve/ maintain stability of proposed embankments. Better understanding of risks and potential future maintenance requirement than for other alternatives Specialized dragline equipment is required for sub-excavation of deep and wide area (extents dictated by safe backslope angles) . If long-stick excavator used, high risk of trapping softened clay material at base of excavation leading to FoS<1.3 for front slope.



**PRE-DRAFT FOUNDATION REPORT – SWAMP CROSSING, HIGH FILL
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**Table B1: Evaluation of Stability/Settlement Mitigation Options
Highway 69 SBL – STA 20+980 to STA 21+055 (High Fill 502)**

Stability/Settlement Mitigation Option	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
			<ul style="list-style-type: none"> May require additional right-of-way to accommodate deep sub-excavation footprint. 		
Partial Preloading (1.5 m high for 30 days) and Lightweight Fill (11.0 m thick EPS core from STA 21+005 to 21+055)	5	<ul style="list-style-type: none"> Substantially faster construction period (small delay as compared with time for large excavation or staged embankment construction). Improved stability – toe berms are not required. Reduced total settlement of foundation soils. Avoids generation of large volume of excess excavation spoil. 	<ul style="list-style-type: none"> High EPS fill will require redesign of adjacent bridge abutment stem wall and possibly supporting foundation type. Very high cost of EPS construction materials. Not feasible to install below the groundwater table (due to buoyancy forces). A minimum 125 mm thick reinforced concrete pad should be required on top of the EPS. Still requires a preload of limited thickness, with some delay in construction schedule to allow for sufficient settlement of below grade soils to occur to meet post-construction settlement criteria. 	<ul style="list-style-type: none"> Estimate \$4.3M (includes granular fill for preload, EPS fill, concrete cap, granular top up after preload settlement). Additional cost for redesign of abutment stem wall and possibly higher costs for abutment foundation support). Additional cost for supply and removal of preload fill. Reduced costs for disposal / management of excavation spoils as compared with the sub-excavation options. 	<ul style="list-style-type: none"> Potential risk of not achieving/maintaining stability of embankment on weak/soft foundation soils during the preload period. Risk that large thickness of EPS surrounding abutment will complicate design of bridge structure.



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**Table B1: Evaluation of Stability/Settlement Mitigation Options
Highway 69 SBL – STA 20+980 to STA 21+055 (High Fill 502)**

Stability/Settlement Mitigation Option	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
<p>Deep Soil Mixing (55 m x 30 m area x 15 m deep in abutment area);</p> <p>Surcharging (2 m high) for 375 days; installation of lightweight fill (3 m thick EPS core from STA 21+020 to 21+055)</p>	6	<ul style="list-style-type: none"> ■ Soil improvement achieved by installation of either a 'grillage' of cemented soil panels or installation of discrete 'barrettes'. ■ Soil treatment area forms strengthened/stiffened shear key through weak and compressible soils to increase stability of front slope of high embankments. ■ Soil mixed area can be designed for direct support of adjacent bridge abutment foundation; reduces cost of abutment foundation design. ■ No substantial soil disposal required. 	<ul style="list-style-type: none"> ■ Additional investigation, testing (bench scale mix trials) and design would be required to develop the most cost-effective design (i.e. to optimize the amount of cement required in the mixed panels); ■ Specialty contractor would need to be retained to carry-out the ground improvement works. ■ Embankment area behind soil treatment zone still requires surcharging and then installation of 3 m thick EPS zone to reduce post-construction settlements to acceptable level. 	<ul style="list-style-type: none"> ■ Estimate \$4.0M to \$4.5M (includes DSM panel or barrette mixing in abutment area; 2 m surcharge, 3 m thick EPS fill with concrete cap, instrumentation and monitoring north of abutment area) 	<ul style="list-style-type: none"> ■ Subject to the monitoring data collected during the surcharge period, the surcharge embankment may need to be left in place for an extended period of time (or may be shortened).



**PRE-DRAFT FOUNDATION REPORT – SWAMP CROSSING, HIGH FILL
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**Table B1: Evaluation of Stability/Settlement Mitigation Options
Highway 69 SBL – STA 20+980 to STA 21+055 (High Fill 502)**

Stability/Settlement Mitigation Option	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
<p>Aggregate Piers (0.76 m diameter piers at 1.8 m spacing on triangular grid; 55 m x 50 m treatment area over abutment and front slope area)</p> <p>Surcharging (2 m high) for 375 days; installation of lightweight fill (3 m thick EPS core from STA 21+020 to 21+055)</p>	Not practical	<ul style="list-style-type: none"> Potentially suitable (mitigation) ground improvement measured in combination with other mitigation measures. 	<ul style="list-style-type: none"> Would not provide adequate ground improvement to enhance stability or mitigate settlement unless piers are installed at a tight spacing in plan. Requires substantial volume of aggregate material. May require the use of temporary liners to support the wet sand/silt deposit overlying the clay stratum and possibly also to support the clay stratum during pier construction which would increase the construction time and costs. Alternative displacement-type aggregate pier construction may be possible. Potentially will have a large volume of augered spoil material to be removed off site (unless displacement-type pier aggregate pier construction can be used). 	<ul style="list-style-type: none"> Estimate \$3.4M to \$3.9M (includes aggregate piers in abutment area; 2 m surcharge, 3 m thick EPS fill with concrete cap, instrumentation and monitoring north of abutment area). Possible additional costs for cement grouting in aggregate pier columns to provide increased strength/stiffness and therefore increased stability and reduced total settlement of treated soil mass. Possible additional costs for wick drains installed between aggregate columns 	<ul style="list-style-type: none"> Based on preliminary analysis, even in combination with other mitigation measure may not result in adequate ground improvement without impractically high pier area replacement ratio. High cost with high uncertainty in effectiveness.
Preloading (single stage construction)	Not feasible	<ul style="list-style-type: none"> Given the proximity of Pier 4 to the proposed north abutment, a toe berm of adequate width to achieve the minimum Factor of Safety against instability cannot be constructed along the front slope. As such, this option is considered to be not feasible. 			
Surcharging (single stage construction)	Not feasible	<ul style="list-style-type: none"> Given the proximity of Pier 4 to the proposed north abutment, a toe berm of greater width than would be required for preloading to achieve the minimum Factor of Safety against instability cannot be constructed along the front slope. As such, this option is considered to be not feasible. 			



**PRE-DRAFT FOUNDATION REPORT – SWAMP CROSSING, HIGH FILL
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**Table B2: Evaluation of Stability/Settlement Mitigation Options
Highway 69 NBL – STA 20+990 to STA 21+075 (High Fill 502)**

Stability/Settlement Mitigation Option	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
<p>Add Span to Bridge Structure (35 m long)</p> <p>Plus:</p> <p>Wick Drains (1 m spacing from STA 21+040 to 21+065) with Partial Preload (4.0 m high for 220 days), 5.5 m thick EPS core from STA 21+055 to 21+075)</p>	1(A)	<ul style="list-style-type: none"> Relocating abutment 35 m to the north will result in a much lower approach embankment (8 m high) and highway embankment located on much thinner clayey foundation soils thereby reducing the risks associated with stability and long-term settlement. Abutment foundation can be constructed almost at existing ground surface (only a very small compacted granular pad required – about 1 m thick) Abutment relocation will result in lower maximum embankment height (on the order of 8 m) which reduces front slope stability issues. Very low impact on the environmental footprint as less filling required and no excavation spoil produced. 	<ul style="list-style-type: none"> Preload period of 220 days still required to satisfy post-construction settlement criteria. Detail wick drain investigation and design will be required. Up to 5.5 m thickness of EPS for a distance of about 20 m behind abutment required to satisfy post-construction settlement criteria. Additional pier and associated foundation element likely required. 	<ul style="list-style-type: none"> Estimate \$2.25M to \$2.75M (for structure + \$0.9M (includes wick drains, preload, EPS, detail wick drain investigation and design, instrumentation and monitoring) Possible additional costs for pre-drilling at some wick drain locations to facilitate installation through compact sands/silts up to 10 m thick. Possible additional costs for foundation investigation at new pier and bridge abutment location. 	<ul style="list-style-type: none"> Lower risk associated with embankment stability and post-construction settlement as compared with other options.
<p>Add Span to Bridge Structure (60 m long)</p>	1(B)	<ul style="list-style-type: none"> Same advantages as above, but even lower approach embankment height (6 m) on stronger foundation soils No additional foundation mitigation measures required for stability or settlement. 		<ul style="list-style-type: none"> Estimate \$2.5M + \$1.9M = \$4.4M (for structure) \$1M premium over Option 1(A) 	<ul style="list-style-type: none"> Lowest Risk



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**Table B2: Evaluation of Stability/Settlement Mitigation Options
Highway 69 NBL – STA 20+990 to STA 21+075 (High Fill 502)**

Stability/Settlement Mitigation Option	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
<p>Wick Drains (1 m spacing from STA 20+970 to 21+065) with Staged Embankment Construction and Toe Berm (11.5 m wide x 4.0 m) on Front Slope, Surcharging (2m high) with 4 m thick EPS installed behind abutment only</p> <p>Preliminary Analysis Estimates:</p> <ul style="list-style-type: none"> • 3 stages of embankment construction (incl. surcharge) • 2 delay periods of 330 days each between Stages 1 to 3 • Up to 500 days for Surcharge Period • Total construction time = 3.2 years 	2	<ul style="list-style-type: none"> ■ Lower cost compared with other options. ■ Avoids generation and handling of large volume of excess excavation spoil. ■ Avoids requirement for large volume of granular fill and rock fill for backfilling. 	<ul style="list-style-type: none"> ■ Long delay in construction schedule to allow for dissipation of excess pore pressures between each stage of construction plus additional delay during surcharge period to allow sufficient settlement to occur to meet post-construction settlement criteria. ■ Detail wick drain investigation and design will be required. ■ Additional time required for installation of wick drains. ■ Increased magnitude of secondary consolidation (creep) settlement in the short term as a result of the accelerated completion of primary consolidation settlement. ■ Instrumentation and monitoring program required to monitor staged construction and to assess end of surcharge period. ■ Increased handling of surcharge fill (Granular 'B') to remove surcharge. ■ North abutment and Pier #4 foundations cannot be constructed until after completion of embankment construction and surcharge period due to drag loads and lateral loads on pile foundations (caused by foundation soil settlement and lateral spreading). ■ Possible need for erosion protection (rip-rap) on front slope of toe berm adjacent to Straight Lake. 	<ul style="list-style-type: none"> ■ Estimate \$1.1 (includes wick drains, 2 m surcharge, toe berm and EPS behind abutment, detail wick drain investigation and design, instrumentation and monitoring). ■ Possible additional costs for EPS fill top-up (\$0.1M to \$0.2M) after surcharge period to satisfy post-construction settlement criteria. ■ Possible additional costs for pre-drilling at some wick drain locations to facilitate installation through compact sands/silts up to 10 m thick. ■ Relatively long delays (up to 3.2 years) for staged construction and surcharging impacts schedule and may further increase overall project costs. ■ Possible additional cost on adjacent abutment foundation design if need to design piles for dragloads. 	<ul style="list-style-type: none"> ■ Size of toe berm, number of stages and length of surcharge period may differ from the results of the preliminary analysis, thereby changing the estimated construction time. Requires confirmation during detail wick drain design. ■ Risk of instability of high surcharge embankment on weak/soft foundation soils even when staged construction is employed; additional instrumentation (such as Slope Inclinometers) may be required. ■ Higher risk associated with the complexity of a staged construction and wick drain design. ■ Subject to the monitoring data collected during the construction and surcharge period, the delay times between stages and the surcharge period may be longer than that estimated in the preliminary analysis; fill may need to be left in place for extended periods of time. ■ Risk that settlements during surcharge period are greater than estimated; may necessitate installation of EPS fill to top-up embankment and limit post-construction settlements; may also result in need to consider dragloads in abutment foundation design.



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AREAS AND DEEP CUT - HIGHWAY 69 GWP 5347-08-00; WP 5005-10-01**

**Table B2: Evaluation of Stability/Settlement Mitigation Options
Highway 69 NBL – STA 20+990 to STA 21+075 (High Fill 502)**

Stability/Settlement Mitigation Option	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
<p>Localized Sub-Excavation between STA 20+985 and 21+065 (up to a depth of 13 m below existing ground surface within the north abutment / front slope area; stepped excavation in benches north of abutment area);</p> <p>Wick Drains (1.25 m spacing from STA 21+020 to 21+065), Surcharging (2 m high) for 375 days; installation of lightweight fill (3 m thick EPS core from STA 21+020 to 21+075)</p>	3	<ul style="list-style-type: none"> ■ Slightly less volume of sub-excavated material and backfill compared to full subexcavation alternative. ■ Combination of mitigation measures satisfies both slope stability of front/side slopes and sufficiently reduces settlement during the preload/surcharge period and minimizes long-term settlement. 	<ul style="list-style-type: none"> ■ Generation of large volume of excess excavation spoil (approx. 21,000 m³). ■ Large quantity of Granular B Type II fill (16,000 m³ in abutment area to allow installation of driven piles at the proposed north abutment) and rock fill (5,000 m³ beyond abutment area) will be required for backfilling below existing ground surface. ■ Delay in construction associated with up to 13 m deep sub-excavation and fill replacement operation. ■ Additional effort for partial dewatering and staged excavation in benches with long-stick backhoe or specialized dragline equipment required for deep sub-excavation and replacement. ■ Additional excavation in benches (5 m to 8 m wide, 2 m to 4 m high at 1.5:1V slopes) beyond immediate abutment area required to maintain stability of excavation face; still results in relatively large volume of excavation required. ■ May require additional right-of-way to accommodate deep sub-excavation. ■ Installation of wick drains required to accelerate settlements in area behind (north of) localized sub-excavation in order to achieve post-construction settlement criteria. ■ Detail wick drain investigation and design will be required. 	<ul style="list-style-type: none"> ■ Estimate \$2.5 (includes granular fill in abutment area and rock fill north of abutment area, wick drains at 1.25 m spacing, 2 m surcharge, 3 m thick EPS fill with concrete cap, instrumentation and monitoring) ■ Additional costs associated with sub-excavation (partial dewatering and staged excavation using long-stick excavator equipment OR specialized dragline equipment required); ■ Possible additional costs for disposal of large volume of weak/soft/wet, compressible deposits, depending on location of soil management area. ■ Possible additional cost for acquiring additional right-of-way for deep sub-excavation. ■ Possible additional costs for pre-drilling at some wick drain locations to facilitate installation through granular fill and compact sands/silts up to 10 m thick. ■ Schedule impacts for construction of different mitigation components may increase overall project costs. ■ Somewhat reduced costs for disposal / management of 	<ul style="list-style-type: none"> ■ Potential for instability of below grade excavation slopes and difficulties excavating to required depths; may result in expansion of excavation footprint. ■ Will achieve/ maintain stability of proposed embankments. ■ Subject to the monitoring data collected during the surcharge period, the surcharge embankment may need to be left in place for an extended period of time (or may be shortened).



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**Table B2: Evaluation of Stability/Settlement Mitigation Options
Highway 69 NBL – STA 20+990 to STA 21+075 (High Fill 502)**

Stability/Settlement Mitigation Option	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
			<ul style="list-style-type: none"> Increased handling of surcharge fills (granular fill) upon completion of surcharge period and the rock fill to be replaced with EPS (3 m thick core and pavement structure). Delay in construction schedule to allow for sufficient settlement to occur to meet post-construction settlement criteria. Instrumentation and monitoring program required to monitor staged construction and to assess end of surcharge period. Requires 125 mm thick reinforced concrete pad constructed on top of the EPS. 	excavation spoils as compared with full sub-excavation option.	
Full Sub-Excavation (up to 14 m deep for entire footprint: from STA 20+985 to 21+100) and Preloading of Rock Fill (365 days)	4	<ul style="list-style-type: none"> Toe berms are not required. Mitigates instability of the embankment and minimizes long-term post construction settlement. Preload period could be reduced to 200 days by using granular material as backfill in subexcavation area north of abutment area (however, additional cost premium for this option). 	<ul style="list-style-type: none"> Generation of very large volume of excess excavation spoil (approx. 49,000 m³). Large quantity of Granular B Type II fill (16,000 m³ in abutment area to allow installation of driven piles at the proposed north abutment) and rock fill (33,000 m³ beyond abutment area) will be required for backfilling below existing ground surface. Delay in construction associated with up to 14 m deep sub-excavation and fill replacement operation. Additional effort for partial dewatering and staged excavation with long-stick backhoe or specialized dragline equipment required for deep sub-excavation 	<ul style="list-style-type: none"> Estimate \$3.1M (includes excavavation and granular fill in abutment area and rock fill north of abutment area). Additional costs associated with sub-excavation (partial dewatering and staged excavation using specialized dragline equipment required); Possible additional costs for disposal of large volume of weak/soft/wet, compressible deposits, depending on location of soil management area. Possible additional cost for acquiring additional right-of-way for deep 	<ul style="list-style-type: none"> Preloading would be required to reduce large post-construction settlement of rock fill. Potential for instability of below grade excavation slopes and difficulties excavating to required depths; may result in expansion of excavation footprint. Will achieve/ maintain stability of proposed embankments. Better understanding of risks and potential future maintenance requirement than for other alternatives Specialized dragline equipment is required for sub-excavation of deep and wide area (extents dictated by safe



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**Table B2: Evaluation of Stability/Settlement Mitigation Options
Highway 69 NBL – STA 20+990 to STA 21+075 (High Fill 502)**

Stability/Settlement Mitigation Option	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
			<p>and replacement.</p> <ul style="list-style-type: none"> ■ Additional post-construction settlement of rock fill itself requires substantial preloading period to reduce overall settlement in the short-term. ■ May require additional right-of-way to accommodate deep sub-excavation footprint. 	<p>sub-excavation.</p> <ul style="list-style-type: none"> ■ Schedule impacts for preload period may increase overall project costs. 	<p>backslope angles) . If long-stick excavator used, high risk of trapping softened clay material at base of excavation leading to FoS<1.3 for front slope.</p>
Partial Preloading (2.0 m high for 120 days) and Lightweight Fill (12.5 m thick EPS core from STA 21+005 to 21+075)	5	<ul style="list-style-type: none"> ■ Substantially faster construction period (relatively small delay as compared with time for large excavation or staged embankment construction). ■ Improved stability – toe berms are not required. ■ Reduced total settlement of foundation soils. ■ Avoids generation of large volume of excess excavation spoil. 	<ul style="list-style-type: none"> ■ High EPS fill will require redesign of adjacent bridge abutment stem wall and possibly supporting foundation type. ■ Very high cost of EPS construction materials. ■ Not feasible to install below the groundwater table (due to buoyancy forces). ■ A minimum 125 mm thick reinforced concrete pad should be required on top of the EPS. ■ Still requires a preload of limited thickness, with some delay in construction schedule to allow for sufficient settlement of below grade soils to occur to meet post-construction settlement criteria. 	<ul style="list-style-type: none"> ■ Estimate \$7.2M (includes granular fill for preload, EPS fill, concrete cap, granular top up after preload settlement). ■ Additional cost for redesign of abutment stem wall and possibly higher costs for abutment foundation support). ■ Additional cost for supply and removal of preload fill. ■ Reduced costs for disposal / management of excavation spoils as compared with the sub-excavation options. 	<ul style="list-style-type: none"> ■ Potential risk of not achieving/maintaining stability of embankment on weak/soft foundation soils during the preload period. ■ Risk that large thickness of EPS surrounding abutment will complicate design of bridge structure.



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**Table B2: Evaluation of Stability/Settlement Mitigation Options
Highway 69 NBL – STA 20+990 to STA 21+075 (High Fill 502)**

Stability/Settlement Mitigation Option	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
<p>Deep Soil Mixing (55 m x 30 m area x 15 m deep in abutment area);</p> <p>Wick Drains (1.25 m spacing from STA 21+020 to 21+065), Surcharging (2 m high) for 375 days; installation of lightweight fill (3 m thick EPS core from STA 21+020 to 21+075)</p>	6	<ul style="list-style-type: none"> ■ Soil improvement achieved by installation of either a 'grillage' of cemented soil panels or installation of discrete 'barrettes'. ■ Soil treatment area forms strengthened/stiffened shear key through weak and compressible soils to increase stability of front slope of high embankments. ■ Soil mixed area can be designed for direct support of adjacent bridge abutment foundation; reduces cost of abutment foundation design. ■ No substantial soil disposal required. 	<ul style="list-style-type: none"> ■ Additional investigation, testing (bench scale mix trials) and design would be required to develop the most cost-effective design (i.e. to optimize the amount of cement required in the mixed panels); ■ Specialty contractor would need to be retained to carry-out the ground improvement works. ■ Installation of wick drains required to accelerate settlements in area behind (north of) deep soil mixing area in order to achieve post-construction settlement criteria. ■ Detail wick drain investigation and design will be required. ■ Embankment area behind soil treatment zone still requires surcharging and then installation of 3 m thick EPS zone to reduce post-construction settlements to acceptable level. 	<ul style="list-style-type: none"> ■ Estimate \$4.7M to \$5.2M (includes DSM panel or barrette mixing in abutment area; wick drains at 1.25 m spacing, 2 m surcharge, 3 m thick EPS fill with concrete cap, instrumentation and monitoring north of abutment area). ■ Possible additional costs for pre-drilling at some wick drain locations to facilitate installation through compact sands/silts up to 10 m thick. 	<ul style="list-style-type: none"> ■ Subject to the monitoring data collected during the surcharge period, the surcharge embankment may need to be left in place for an extended period of time (or may be shortened).



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**Table B2: Evaluation of Stability/Settlement Mitigation Options
Highway 69 NBL – STA 20+990 to STA 21+075 (High Fill 502)**

Stability/Settlement Mitigation Option	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
<p>Aggregate Piers (0.76 m diameter piers at 1.8 m spacing on triangular grid; 55 m x 50 m treatment area over abutment and front slope area)</p> <p>Wick Drains (1.25 m spacing from STA 21+020 to 21+065), Surcharging (2 m high) for 375 days; installation of lightweight fill ((3 m thick EPS core from STA 21+020 to 21+075)</p>	Not practical	<ul style="list-style-type: none"> Potentially suitable (mitigation) ground improvement measured in combination with other mitigation measures. 	<ul style="list-style-type: none"> Would not provide adequate ground improvement to enhance stability or mitigate settlement unless piers are installed at a tight spacing in plan. Requires substantial volume of aggregate material. May require the use of temporary liners to support the wet sand/silt deposit overlying the clay stratum and possibly also to support the clay stratum during pier construction which would increase the construction time and costs. Alternative displacement-type aggregate pier construction may be possible. Installation of wick drains required to accelerate settlements in area behind (north of) aggregate pier area in order to achieve post-construction settlement criteria. Detail wick drain investigation and design will be required. Potentially will have a large volume of augered spoil material to be removed off site (unless displacement-type pier aggregate pier construction can be used). 	<ul style="list-style-type: none"> Estimate \$3.9M to \$4.4M (includes aggregate piers in abutment area; wick drains at 1.25 m spacing, 2 m surcharge, 3 m thick EPS fill with concrete cap, instrumentation and monitoring north of abutment area). Possible additional costs for cement grouting in aggregate pier columns to provide increased strength/stiffness and therefore increased stability and reduced total settlement of treated soil mass. Possible additional costs for pre-drilling at some wick drain locations to facilitate installation through compact sands/silts up to 10 m thick. Possible additional costs for wick drains installed between aggregate columns. 	<ul style="list-style-type: none"> Based on preliminary analysis, even in combination with other mitigation measure may not result in adequate ground improvement without impractically high pier area replacement ratio. High cost with high uncertainty in effectiveness.
Preloading (single stage construction)	Not feasible	<ul style="list-style-type: none"> Given the proximity of Pier 4 to the proposed north abutment, a toe berm of adequate width to achieve the minimum Factor of Safety against instability cannot be constructed along the front slope. As such, this option is considered to be not feasible. 			
Surcharging (single stage construction)	Not feasible	<ul style="list-style-type: none"> Given the proximity of Pier 4 to the proposed north abutment, a toe berm of greater width than would be required for preloading to achieve the minimum Factor of Safety against instability cannot be constructed along the front slope. As such, this option is considered to be not feasible. 			



APPENDIX D

Operational Constraints and Non-Standard Special Provision

Operational Constraint - Obstructions

The Contactor is hereby notified that cobbles and boulders are present within/underlying the silt and sand to sand deposit (overlying bedrock) below the lake bed in Straight Lake and encountered within the sand to sand and gravel deposit at the south abutment. Consideration of the presence of these obstructions must be made in selection of appropriate equipment for installation of the piles at the piers and abutments.

Operational Constraint – Artesian Conditions

The Contactor is hereby notified that artesian conditions were encountered within the silt and sand and cobbles and boulders deposits (overlying bedrock) in Boreholes B501-10 and B501-12 advanced for the SBL structure. Given the proximity to these boreholes to the NBL structure alignment, the presence of the artesian conditions must be considered when selecting the appropriate equipment and methodology for installation of the piles at the piers.

Drilled Steel Casings – Drilling and Cleaning/Flushing of Rock Sockets

Non-Standard Special Provision

Scope of Work

This special provision covers the requirement for the cleaning/flushing of the uncased rock socket at the bottom of the drilled steel casings socketed into bedrock.

Construction

The steel casing shall be advanced through the overburden and seated into the bedrock using Down-the-Hole (DTH) hammer drilling equipment and methods. After seating the steel casing to the required depth into bedrock (i.e. minimum 1 m below the lowest point of contact with the bedrock surface and a minimum of 1 m into fair quality bedrock), the uncased rock socket shall be drilled for a minimum length of 2 m into fair to good quality bedrock. Upon completion of drilling, the uncased rock socket shall be cleaned/flushed by whatever means necessary to remove all sediment, debris and loose rock fragments from the socket walls and base. The rock socket shall be inspected (with a suitable Shaft Inspection Device [SID]) to confirm proper cleaning prior to placing the steel reinforcement cage and concrete by tremie methods.

Basis of Payment

Payment at the lump sum contract price for this tender item shall be full compensation for all labour, equipment and materials for completion of the work.

END OF SECTION

Laboratory Testing of Granular Materials for Pad Construction (to Support Spread Footings)

Non-Standard Special Provision

Scope of Work

This special provision covers the requirement for testing of granular materials for the pad construction to support spread footings at the South Abutment. This testing is only required if the South Abutment is designed to be supported on a shallow foundation / spread footing perched within the high approach embankment fill.

Construction

Prior to the start of construction of the granular pad for the South Abutment spread footing, laboratory testing shall be carried out on the granular materials to be used for pad construction to verify that a minimum internal angle of friction of 37 degrees will be achieved for that material.

Basis of Payment

Payment at the lump sum contract price for this tender item shall be full compensation for all labour, equipment and materials for completion of the work.

END OF SECTION

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