



SEPTEMBER 2014

FOUNDATION INVESTIGATION AND DESIGN REPORT

STILL RIVER SBL BRIDGE STRUCTURE, SITE NO. 44-458/2
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 5404-05-00; WP 5140-08-01

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REPORT



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PART A

FOUNDATION INVESTIGATION REPORT

STILL RIVER SBL BRIDGE STRUCTURE, SITE NO. 44-458/2

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 5404-05-00; WP 5140-08-01



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 detail foundation engineering services for the proposed Highway 69 southbound lanes (SBL) structure over the Still River (Site No. 44-458/2), which is within the Contract 2 limits of the new Highway 69 alignment to the north of the junction with Highway 529. The proposed work in Contract 2 is part of the 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. The foundation engineering components within the overall project limits include the engineering of: 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 (CPR) and Canadian National Railway (CNR) structures; as well as culvert crossings. The Still River SBL Bridge structure is located approximately 1.2 km east of the existing Highway 69. The general location of this bridge along the new Highway 69 four-laning alignment is shown on the Site Location Plan on Drawing 1.

The Terms of Reference (TOR) and the scope of work for the foundation investigation are outlined in MTO's Request for Proposal, dated January 2009. Golder's proposal for foundation engineering services associated with the Contract 2 Still River SBL Bridge structure 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 Quality Control Plan for foundation engineering services for this project, dated April 19, 2010. The General Arrangement (GA) Drawing for the proposed Still River SBL Bridge structure (two-span option) was provided to Golder by URS on April 5, 2011. In addition, a preliminary GA Drawing for the one-span option was provided to Golder by URS on October 1, 2010.

This report addresses the foundation investigation carried out for the Still River SBL Bridge structure and the associated approach embankments only. A two-span bridge structure was proposed in the Environmental Assessment Report. During the initial stage of the detail design assignment one-span and two-span options were evaluated. In September 2011 Golder conducted field investigations and prepared a Technical Memorandum summarizing the findings and provided preliminary recommendations with respect to both a one-span and two-span alternative. Following discussions with the Ministry, a two-span option has been chosen for the Still River crossing. Since the feasibility of a one-span bridge option was being evaluated during the initial design stage, this foundation investigation report has been prepared to address these two options. 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 structure location, including the associated approach embankments, 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.

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 Township of Mowat to the north. The proposed Still River SBL Bridge structure is located in the Township of Henvey along the new Highway 69 SBL alignment, approximately 1.2 km east of the existing Highway 69 alignment and about 6.5 km northeast of the junction of



existing Highway 69 and Highway 526. The proposed new four-lane Highway 69 alignment is oriented generally in a south-north direction and parallel to the east side of the existing Highway 69 within the Contract 2 project limits.

In general, the topography of this section of the overall project limits consists of rolling terrain, including sparsely or densely populated tree covered areas and numerous bedrock outcrops separated by valleys and swamps containing areas of standing water and various types of vegetation and organic soils. The proposed bridge structure and associated approach embankments are to be situated on a relatively flat and low-lying swamp/open field area on the south side of the Still River and on the moderately to densely tree covered sloping ground and bedrock outcrop on the north side of the River. On the south side of the River, the ground surface within the limits of the proposed structure is relatively flat at about Elevation 181.0 m along the south approach embankment and at the south abutment. On the north side of the river, the ground surface ranges from about Elevations 179.0 m to 178.6 m at the centre pier to about Elevations 185.4 m to 179.5 m at the north abutments (for one- and two-span bridge options) and about Elevation 192.2 m along the north approach embankment (for the two-span bridge option). All elevations are referenced to Geodetic datum.

3.0 INVESTIGATION PROCEDURES

3.1 Foundation Investigation

The field work for the proposed Still River SBL Bridge structure was carried out between February 3 and 8, 2011 and between March 1 and 25, 2011 during which time a total of twelve (12) boreholes, four (4) Dynamic Cone Penetration Tests (DCPTs) and two (2) hand excavations were advanced at the locations of the structure and approach embankments. An additional borehole was (Borehole SP1) was advanced in the vicinity of the south abutment and instrumented with a piezometer to allow for monitoring of the groundwater level.

In addition, one (1) borehole (Borehole S204-09) advanced within the adjacent Swamp 204 as part of the field investigation work carried out by Golder for the Contract 2 swamp crossings and high fill areas¹ was utilized to supplement this investigation at the south approach embankment. The Record of Borehole sheet and the results of the laboratory testing for Borehole S204-09 are presented together with the details of the boreholes advanced for this bridge structures in Appendix A and Appendix B, respectively. The locations of the boreholes and DCPTs are shown on Drawing 2.

A summary of the respective boreholes and DCPTs advanced at each foundation element and approach embankment is presented below.

Foundation Element/ Approach Embankment	Investigation Type		
	Borehole No.	DCPT No.	Hand Excavation
South Approach Embankment (One- or Two-Span Option)	S204-09	-	-

¹ Golder Associates Ltd. 2012. *Foundation Investigation and Design Report, Swamp Crossings and High Fill Areas – Contract 2, Highway 69 Four Lining from 1.7 km North of Highway 529 Northerly to 3.9 km North of Highway 522, Ministry of Transportation, Ontario, G.W.P. 5404 05 00; W.P. 5404 05 01. Geocres No. 41H-115.*



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Foundation Element/ Approach Embankment	Investigation Type		
	Borehole No.	DCPT No.	Hand Excavation
South Abutment (One- or Two-Span Option)	B201-01 SP1 ¹	-	-
Centre Pier (Two-Span Option)	B201-02 B201-13 B201-14	B201-DC03 B201-DC04	-
North Abutment (One-Span Bridge)	B201-03 B201-11 B201-12	B202-DC01 B202-DC02	-
North Approach Embankment (One-Span Option)	B201-04	-	-
North Abutment (Two-Span Option)	B201-05 B201-06 B201-07 B201-08	-	B201-09
North Approach Embankment (Two-Span Option)	-	-	B201-10

Note: 1. Borehole SP1 was advanced in the vicinity of the south abutment to monitor the groundwater level in the area.

The field investigation was carried out using a track-mounted Diedrich D-25 or D-50 Turbo drill rig supplied and operated by Walker Drilling Co. Ltd. of Utopia, Ontario and by portable equipment supplied and operated by OGS Inc. of Almonte, Ontario. Hand excavation methods were used as appropriate depending on the terrain to confirm refusal conditions at shallow borehole locations. The boreholes were advanced through the overburden using 127 mm outer diameter (O.D.) solid-stem augers, tricone and/or 'BW', 'NW' or 'HW' casing with wash boring techniques. In general, soil samples were obtained at intervals of depth of about 0.75 m and 1.5 m, using a 50 mm O.D. split-spoon sampler driven by an automatic hammer on the drill rig, performed in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586, Standard Test Method for Standard Penetration Test). Boreholes advanced by portable equipment employed one-third ($\frac{1}{3}$) weight hammers lifted manually and dropped from the SPT height. The SPT 'N'-value obtained by the use of the lesser weight hammer were then adjusted down by a factor of 3 to correspond to the SPT 'N'-values that would be expected to be obtained had a full-weight hammer been used. Chunk samples were obtained from two (2) boreholes at locations of thin overburden over shallow bedrock. Samples of the cohesive soils were obtained using 76 mm O.D. thin-walled 'Shelby' tubes (ASTM D1587-08, Standard Practice for Thin-Walled Tube Sampling) for relatively undisturbed samples. Field vane shear tests were conducted in cohesive soils for determination of undrained shear strengths (ASTM D2573, Standard Test Method for Field Vane Strength Shear Test) using MTO Standard 'N' size vanes and 'B' size vanes in the smaller diameter boreholes advanced by portable equipment. Samples of the bedrock were obtained using a 'NQ' or 'BQ' size rock core barrel.

The boreholes and DCPTs at the locations of the foundation elements were typically advanced to casing and/or split-spoon sampler refusal (i.e. inferred bedrock) and bedrock was confirmed by coring in selected boreholes. The borehole at the north approach embankment for the two-span bridge option is located on a bedrock outcrop and refusal condition was confirmed by hand excavation. The boreholes and DCPTs were advanced to depths of up to about 57.3 m below existing ground surface, including coring of bedrock for core lengths between about



2.5 m and 4.8 m in Boreholes B201-02 to B202-04, B201-06 to B201-08 and B201-11 to B201-14. Photographs of the recovered rock core samples are provided in Appendix B.

The groundwater conditions and water levels in the open boreholes were observed during the drilling operations. Within the limits of the centre pier and north abutment foundation elements, a piezometer was installed in each of Boreholes B201-02, B201-03 and B201-07 as well as Borehole SP1 located near the proposed south abutment, to monitor the ground water levels at these locations. No piezometer was installed within the limits of the south abutment foundation element due to the presence of artesian groundwater conditions. The piezometers consist of 35 mm diameter PVC pipe, with a slotted screen sealed at a select depth within the boreholes. The boreholes and annulus surrounding the piezometer pipe above the screen sand pack were backfilled to the surface with bentonite pellets/grout. Piezometer installation details and water level readings are described on the Record of Borehole sheets presented in Appendix A. All boreholes in which standpipe piezometers were not installed were backfilled with bentonite or a bentonite/cement grout mixture using tremie methods upon completion, in accordance with Ontario Regulation 903 (as amended). The piezometer installed in Borehole SP1 was decommissioned once a final water level reading was taken ten (10) days after installation.

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, Atterberg limits and grain size distribution) was carried out on selected samples. A consolidation (oedometer) test was also carried out on a sample of the cohesive deposit. Strength testing, such as 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.

Classification of the rock mass quality of the bedrock with respect to the Rock Quality Designation (RQD) is described based on Table 3.10 of the Canadian Foundation Engineering Manual (CFEM, 2006²). The degree of weathering of the bedrock samples (i.e. fresh to slightly weathered – W1 to W2) and the strength classification of the intact rock mass based on field identification (i.e. strong to extremely strong – R4 to R6) are described in accordance with Table B.3 and Table B.6, respectively, of the International Society for Rock Mechanics (ISRM, 1985³) standard classification system.

The perimeter limits of each foundation unit were located in the field by Callon Dietz prior to drilling. The as-drilled borehole and DCPT locations and ground surface elevations were surveyed by a member of our technical staff, referenced to the survey stakes put down by Callon Dietz. The borehole locations given in the Record of Borehole/Drillhole sheets and shown on Drawings 2 and 3 are positioned relative to MTM NAD 83 northing and easting coordinates and the ground surface elevations are referenced to Geodetic datum. The borehole and DCPT locations and ground surface elevations are summarized below.

²Canadian Geotechnical Society, 2006. Canadian Foundation Engineering Manual, 4th Edition.

³ 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.



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Borehole / DCPT No.	Location (MTM NAD 83)		Ground Surface Elevation (m)	Borehole / DCPT Depth (m)
	Northing	Easting		
B201-01	5074818.7	225139.3	181.1	57.3
B201-02	5074858.6	225123.5	178.6	9.1
B201-03	5074874.5	225117.3	179.5	9.1
B201-04	5074886.1	225112.7	180.6	5.0
B201-05	5074896.5	225100.8	183.9	0.5
B201-06	5074900.4	225099.3	185.4	3.4
B201-07	5074898.7	225107.6	184.4	5.2
B201-08	5074896.9	225115.3	183.4	4.0
B201-09 ¹	5074900.8	225113.7	185.5	0.1
B201-10 ¹	5074917.2	225100.0	192.2	0.1
B201-11	5074876.3	225109.0	179.5	7.1
B201-12	5074872.7	225124.9	179.6	8.8
B201-13	5074856.9	225131.2	178.7	10.8
B201-14	5074860.4	225115.2	179.0	9.7
B201-DC01	5074872.3	225110.6	179.4	5.9
B201-DC02	5074876.7	225123.4	179.8	5.2
B201-DC03	5074860.8	225129.7	178.3	5.4
B201-DC04	5074856.4	225116.8	178.6	9.2
S204-09	5074804.5	225144.9	181.1	45.9
SP1	5074819.8	225159.0	181.1	6.1

Note: 1. B201-09 and B201-10 refer to hand excavations carried out at the north-east corner of the north abutment (two-span option) and at the north approach embankment, respectively, to expose the shallow bedrock surface.

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

As delineated in The Physiography of Southern Ontario⁴, 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 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 and swampy areas, containing peat and/or organic soils, underlain by soft/loose native soils, 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 4⁵.

⁴ 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.

⁵ Geology of Ontario, 1991. Ontario Geological 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 General Overview of Local Subsurface Conditions

The detailed subsurface soil and groundwater conditions as encountered in the boreholes advanced during this investigation (including hand excavations by hand shovel), together with the results of the laboratory tests carried out on selected soil and bedrock core samples, are presented on the attached Record of Borehole and Drillhole sheets and 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 undrained shear strengths from the field vanes) as presented on the Record of Borehole sheets and in Section 4.0 are uncorrected. The stratigraphic boundaries shown on the Record of Borehole sheets are inferred from non-continuous sampling, observations of drilling progress and the results of SPTs and in situ testing. The stratigraphic boundaries shown in Borehole SP1, augered for a piezometer installation, are interpreted based on cuttings and auger samples. All boundaries, therefore, represent transitions between soil types rather than exact planes of geological change. Further, subsurface conditions will vary between and beyond the borehole locations. The thickness of the overburden/depth to refusal as inferred from the resistance to DCPT advancement are shown on the Record of Penetration Test sheets in Appendix A. It should be noted that the interpreted stratigraphy shown on Drawings 2 and 3 is a simplification of the subsurface conditions.

In general, the subsurface conditions in the area of the south approach and south abutment (to the south of Still River) consist of a surface layer of topsoil, underlain by alternating deposits of cohesive and non-cohesive soils, underlain by cobbles and boulders. Bedrock was not encountered within the maximum depth of investigation (57.3 m). In the areas of the centre pier, north abutment and north approach (to the north of Still River), the subsurface conditions consists of a surficial layer of topsoil, underlain by deposits of sand to silt and clay, underlain by bedrock at shallow depths. The overburden thickness is variable across the proposed bridge structure, ranging from greater than 57.3 m at the south abutment to no cover at the north approach embankment (i.e. bedrock outcrops exposed at ground surface).

A detailed description of the subsurface conditions encountered in the boreholes at the abutments, center pier and approach embankments is provided in the following sections.

4.3 South Abutment and Approach Embankment

One (1) borehole (Borehole B201-01) was advanced at the location of the proposed south abutment as part of this current investigation. At the south approach embankment, one (1) borehole (Borehole S204-09) was advanced on the centreline of the proposed Highway 69 SBL alignment (within Swamp 204) as part of the field investigation for the Contract 2 swamp crossings and high fill areas.

In general, the subsurface conditions consist of topsoil underlain by alternating deposits comprised of clayey silt to silty clay, sand to sandy silt to sand and silt, silty clay to clay, silt to sandy silt, silty clay, silt to sand, and sand and gravel, underlain by cobbles and boulders.



4.3.1 Topsoil

An approximately 0.3 m and 0.2 m thick layer of topsoil was encountered at the ground surface at about Elevation 181.1 m in Boreholes B201-01 and S204-09.

The natural water content measured on a specimen of this layer is about 34 per cent.

4.3.2 Clayey Silt to Silty Clay (Near Surface)

A near surface stratum of brown to grey clayey silt to silty clay, trace to some sand containing rootlets was encountered underlying the topsoil in Boreholes B201-01 and S204-09. The clayey silt to silty clay stratum is intersected by an approximately 0.4 m thick pocket of sandy silt in Borehole B201-01. The top of the cohesive stratum is at about Elevations 180.8 m and 180.9 m and the total thickness of the cohesive stratum is about 2.0 m and 2.7 m in the respective boreholes.

The SPT 'N'-values measured within the clayey silt to silty clay stratum range from 1 blow to 5 blows per 0.3 m of penetration. In situ field vane tests carried out within this stratum measured undrained shear strengths of about 46 kPa and 50 kPa, and the sensitivity is calculated to be about 4 and 5. It should be noted that in situ filed vanes carried out within the near surface cohesive deposit in Swamp 204, adjacent to the south abutment and south approach embankment, measured undrained shear strengths as low as about 25 kPa. The field vane tests results together with the SPT 'N'-values indicate that the clayey silt to silty clay stratum has a very soft to firm consistency.

The natural water content measured on five (5) samples of the clayey silt to silty clay stratum ranges from about 24 per cent to 45 per cent.

A grain size distribution of one (1) sample of the silty clay portion of this stratum is shown on Figure B1 in Appendix B.

Atterberg limits tests were carried out on two (2) specimens of the clayey silt to silty clay stratum and indicate liquid limits of about 37 per cent and 29 per cent, plastic limits of about 17 per cent, and corresponding plasticity indices of about 20 per cent and 12 per cent. The results of the Atterberg limits tests are shown on the plasticity chart on Figure B2 in Appendix B and indicate the material to be clayey silt of low plasticity to silty clay of intermediate plasticity.

4.3.3 Sand to Sandy Silt (Upper)

A deposit of non-cohesive soil comprised of brown sand trace silt to sand and silt to sandy silt was encountered underlying the near surface stratum of clayey silt to silty clay in both boreholes. The deposit generally contains trace to some clay, and the upper portion of the deposit in Borehole S204-09 contains organics. The top of this deposit is at about Elevations 178.8 m and 178.2 m and the overall thickness of the deposit is about 6.1 m and 6.8 m in Boreholes B201-01 and S204-09, respectively.

The SPT 'N'-values measured within this deposit range from 1 blow to 21 blows per 0.3 m of penetration, with the higher 'N'-values generally being recorded within the sand portion of this deposit, indicating a very loose to compact relative density.



The natural water content measured on seven (7) samples of this deposit ranges from about 9 per cent to 28 per cent.

The grain size distributions of two (2) samples of the sand and silt to sandy silt portion of this deposit are shown on Figure B3 in Appendix B.

4.3.4 Silty Clay to Clay (Upper)

A stratum of grey silty clay to clay trace silt, trace sand and containing sand lenses was encountered underlying the upper deposit of sand to sandy silt in both boreholes. Within the clay portion of this stratum in Boreholes S204-09 and B201-01, silt to sandy silt interlayers were encountered at varying depths. The top of the silty clay to clay stratum is at about Elevations 172.7 m and 171.4 m and the thickness of the deposit is about 5.0 m and 4.6 m in Boreholes B201-01 and S204-09, respectively.

The SPT 'N'-values recorded within the silty clay to clay deposit range from 1 blow to 5 blows per 0.3 m of penetration. In situ field vane tests carried out within this stratum measured undrained shear strengths ranging from about 18 kPa to 80 kPa, but generally greater than 50 kPa, and the sensitivity is calculated to range from about 2 to 5. The field vane test results indicate that the silty clay to clay stratum has a soft to stiff consistency, but generally a stiff consistency.

The natural water content measured on four (4) specimens of this stratum ranges from about 29 per cent to 118 per cent, but is generally less than 48 per cent.

Atterberg limits tests were carried out on two (2) specimens of this cohesive stratum and indicate liquid limits of about 50 per cent and 51 per cent, plastic limits of about 20 per cent and 19 per cent and corresponding plasticity indices of about 30 per cent and 32 per cent. The results of the Atterberg limits tests are shown on the plasticity chart on Figure B4 in Appendix B and indicate the material to be a silty clay of intermediate plasticity to clay of high plasticity.

4.3.5 Silt to Sandy Silt Interlayer

An interlayer of grey silt some sand to sandy silt, containing trace clay was encountered underlying the silty clay to clay stratum in Boreholes B201-01 and S204-09. The top of this interlayer is at about Elevations 167.7 m and 166.8 m and the thickness of the deposit is about 1.1 m and 2.3 m in the respective boreholes.

Two (2) SPT 'N'-values measured within this deposit are 5 blows and 19 blows per 0.3 m of penetration, indicating a loose to compact relative density.

The natural water content measured on two (2) samples of this interlayer is about 24 per cent and 26 per cent.

The grain size distributions of two (2) samples of this interlayer are shown on Figure B5 in Appendix B.

4.3.6 Silty Clay (Lower)

A stratum of grey silty clay trace sand was encountered underlying the silt to sandy silt interlayer in Boreholes B201-01 and S204-09. The lower silty clay stratum is separated into an upper and lower segment by sand and silt to silt interlayers up to about 4.1 m thick, as described below. The top of the silty clay stratum is at



about Elevations 166.6 m and 164.5 m and the overall thickness of the cohesive stratum is about 23.3 m and 25.8 m in the respective boreholes.

The SPT 'N'-values recorded within the silty clay stratum range from 1 blow to 9 blows per 0.3 m of penetration. In situ field vane tests carried out within this stratum measured undrained shear strengths ranging from about 55 kPa to greater than 120 kPa, but are typically less than 100 kPa. The sensitivity is calculated to range from about 2 to 4. The field vane tests results indicate that the silty clay stratum has a stiff to very stiff consistency.

The natural water content measured on seven (7) samples of this stratum ranges from about 37 per cent to 53 per cent.

The grain size distributions of two (2) samples of this silty clay stratum are shown on Figure B6 in Appendix B.

Atterberg limits tests were carried out on three (3) specimens of the silty clay stratum and indicate liquid limits ranging from about 45 per cent to 48 per cent, plastic limits between about 17 per cent and 19 per cent and plasticity indices ranging from about 27 per cent to 29 per cent. The results of the Atterberg limits tests are shown on the plasticity chart on Figure B7 in Appendix B and indicate the material to be silty clay of intermediate plasticity.

A laboratory consolidation test was carried out on one (1) specimen of the silty clay deposit obtained from a Shelby tube sample in Borehole B201-01. A preconsolidation stress of about 210 kPa was estimated from the void ratio versus logarithmic stress plot and from the total work versus stress plot. A bulk unit weight of about 15.7 kN/m³ and a specific gravity of about 2.81 were measured on the consolidation test specimen. Details of the test results are shown on Figure B8 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 B201-01 Sample 17	27.5 m / 153.6 m	218	218	~ 0	~ 1.0	1.12	0.12	1.95	1.81×10^{-3}

Note: * For stress range of between effective overburden stress and final stress due to 7.5 m high approach embankment, that is $215 \text{ kPa} \leq \sigma_v' \leq 360 \text{ kPa}$

where: σ_{vo}' is the in situ vertical effective overburden stress in kPa
 σ_p' is the preconsolidation stress in kPa
 σ_v' is the vertical effective stress in kPa
OCR is overconsolidation ratio
 e_o is 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

4.3.7 Sand and Silt to Silt Interlayers

An approximately 4.1 m thick interlayer of grey silt, trace sand, trace clay and an approximately 2.7 m thick interlayer of grey sand and silt trace clay were encountered within the silty clay stratum in Boreholes B201-01 and S204-09 at about Elevations 161.0 m and 161.1 m, respectively.

Two SPT 'N'-values measured within the sand and silt to silt interlayers are 4 blows and 6 blows per 0.3 m of penetration, indicating a loose relative density.



The natural water content measured on a specimen of the sand and silt interlayer is about 21 per cent.

4.3.8 Silt to Sand (Lower)

Underlying the lower stratum of silty clay, a non-cohesive soil grading from silt, trace to some sand, trace to some clay, trace gravel to silty sand, trace clay. to sand trace to some silt was encountered in both boreholes. The top of the silty sand to silt to sand deposit is at about Elevations 143.3 m and 138.7 m and the thickness of the deposit is about 15.3 m and greater than 3.5 m in Boreholes B201-01 and S204-09, respectively. Borehole S204-09 was terminated within this deposit.

The SPT 'N'-values recorded within this deposit ranges from 19 blows to 49 blows per 0.3 m of penetration, generally indicating a compact to dense relative density.

The natural water content measured on five (5) samples of this deposit ranges from about 18 per cent to 29 per cent.

The grain size distributions of two (2) samples from the silt portion of this deposit are shown on Figure B9 in Appendix B. An Atterberg limits test on one (1) sample of the silt deposit indicates this material to be non-plastic.

4.3.9 Boulders

Below the silt to sand deposit in Borehole B201-01, an approximately 0.3 m and 0.8 m thick layer of boulders was encountered at about Elevation 128.0 m. An approximately 0.2 m thick layer of sand, trace silt was encountered between the boulder layers. A photograph of the recovered cobbles and boulders samples is shown on Figure B10 in Appendix B.

4.3.10 Sand and Gravel

Underlying the boulder layer in Borehole B201-01, a deposit of sand and gravel containing trace silt and cobbles / boulders at varying depths was encountered. The top of the sand and gravel deposit is at about Elevation 126.7 m and the thickness of the deposit is greater than 2.9 m. The lower portion of the deposit was penetrated by coring through the cobbles and boulders. Borehole B201-01 was terminated within the sand and gravel deposit.

One SPT 'N'-value of 20 blows per 0.3 m of penetration was recorded within the upper portion of the deposit, indicating a compact relative density and about 20 per cent of recovery was obtained within the cored portion of the overburden. The natural water content measured on a specimen of this deposit is about 17 per cent.

4.3.11 Groundwater Conditions

In general, the overburden samples taken in the boreholes were moist to wet. Artesian groundwater conditions were observed in Borehole B201-01 when the drill casing was advanced between about Elevations 133.1 m and 129.3 m; the artesian flow stopped when advancing the casing below Elevation 129.3 m. The series of groundwater levels recorded in the drill casing during and upon completion of drilling range from about



Elevations 183.4 m to 177.0 m, measured at depths ranging from about 2.3 m above ground surface to 4.1 m below ground surface. In Borehole S204-09, the water level observed upon completion of drilling was at about Elevation 177.7 m, measured at a depth of about 3.4 m below ground surface.

A shallow standpipe piezometer was installed in Borehole SP1 located in the area to the east of the south abutment to allow monitoring of the near surface groundwater level in the area. The water level in the piezometer was monitored about ten (10) days after installation and then the piezometer was decommissioned. The details of the piezometer installations and the groundwater levels are shown on the Record of Borehole sheets in Appendix A, and the groundwater levels are summarized below.

Foundation Element	Borehole No.	Ground Surface Elevation (m)	Groundwater Elevation (m)	Date of Measurement
South Abutment	SP1	181.1	177.1 177.1	February 17, 2011 February 27, 2011

It should be noted that groundwater level in the area is subject to seasonal fluctuations and precipitation events, and should be expected to be higher during wet periods of the year.

4.4 Centre Pier (Two-Span Option)

A total of three (3) boreholes (Boreholes B201-02, B201-13 and B201-14) and two (2) Dynamic Cone Penetration Tests (DCPTs B201-DC03 and B201-DC04) were advanced at the location of the proposed centre pier. In general, the subsurface conditions consist of topsoil underlain by a deposit of silty sand to silt, underlain by a stratum of silty clay to clay, underlain by bedrock.

4.4.1 Topsoil

An approximately 0.2 m to 0.3 m thick layer of topsoil was encountered at the ground surface at all borehole locations. The surface of the topsoil ranges from about Elevations 179.0 m to 178.6 m.

4.4.2 Sand to Silt

A deposit of grey silty sand to sand to silt, generally containing trace gravel, trace to some clay was encountered below the topsoil in all the boreholes. In general, the upper portion of the deposit contains rootlets, organics and wood fragments and in Borehole B201-13, the silt portion of this deposit contains silty clay interlayers. The top of the silty sand to sand to silt deposit ranges from about Elevations 178.7 m to 178.4 m and the thickness of this deposit ranges from about 2.0 m to 4.0 m.

The SPT 'N'-values measured within this deposit range from 0 blows (weight of hammer) to 9 blows per 0.3 m of penetration, indicating a very loose to loose relative density.

The natural water content measured on samples of this deposit ranges from about 20 per cent to 37 per cent.

The grain size distributions of two (2) samples of the silty sand to sand portion of this deposit and one (1) sample of the silt portion of this deposit are shown on Figures B11A and B11B in Appendix B, respectively.



An Atterberg limits test was carried out on one (1) sample of the silt portion of this deposit and measured a liquid limit of about 21 per cent, a plastic limit of about 18 per cent and a corresponding plasticity index of about 3 per cent. The result of the Atterberg limits test is shown on the plasticity chart on Figure B12 in Appendix B and indicates that the material is classified as silt with slight plasticity.

4.4.3 Silty Clay to Clay

A stratum of grey silty clay to clay some silt, trace sand was encountered underlying the silty sand to sand to silt deposit at all borehole locations. The top of the silty clay to clay stratum ranges from about Elevations 176.7 m to 174.5 m and the thickness of the stratum ranges from about 3.3 m to 4.3 m.

The SPT 'N'-values recorded within the silty clay to clay stratum range from 0 blows (weight of hammer) to 3 blows per 0.3 m of penetration. In situ field vane tests carried out within this stratum measured undrained shear strengths ranging from about 27 kPa to 55 kPa, but are typically less than 37 kPa. The sensitivity is calculated to be about 3 and 4. The field vane tests results indicate that the silty clay to clay stratum generally has a firm to stiff consistency.

The natural water content measured on samples of this stratum ranges from about 37 per cent to 57 per cent.

The grain size distributions of three (3) samples of this stratum are shown on Figure B13 in Appendix B.

Atterberg limits tests were carried out on three (3) specimens of the silty clay to clay stratum and indicate liquid limits between about 50 per cent and 53 per cent, plastic limits between about 18 per cent and 20 per cent and plasticity indices between about 32 per cent and 35 per cent. The results of the Atterberg limits tests are shown on the plasticity chart on Figure B14 in Appendix B and indicate the material to be silty clay of intermediate plasticity to clay of high plasticity.

4.4.4 Bedrock / Refusal

Bedrock was encountered below the silty clay to clay stratum and core samples were recovered at all borehole locations. The bedrock surface is inferred from refusal to dynamic cone penetration in DCPTs B201-DC03 and B201-DC04. The refusal depths at the DCPTs, while they do not confirm bedrock elevations, may be inferred to indicate potential proximity to the bedrock interface at this location. The depth to bedrock below ground surface and corresponding bedrock surface elevation (inferred or actual) is summarised below.

Foundation Element / Approach Embankment	Borehole No.	Depth to Bedrock Surface / Refusal (m)	Bedrock Surface / Refusal Elevation (m)	Comments
Centre Pier (Two-Span Option)	B201-02	6.2	172.4	Bedrock Cored
	B201-13	7.8	170.9	Bedrock Cored
	B201-14	6.6	172.4	Bedrock Cored
	B201-DC03	5.4	172.9	DCPT Refusal
	B201-DC04	9.2	169.4	DCPT Refusal

In general, the bedrock surface slopes downwards towards the river across the proposed centre pier footprint (up to approximately 1.3H:1V slope or a dip of approximately 37° from the horizontal). Based on the review of



the cored bedrock samples, the bedrock consists of granitic gneiss. The bedrock samples are described as slightly weathered to fresh, foliated, medium crystalline, slightly porous, strong, pink, grey and black, as presented in the Record of Drillhole sheets in Appendix A, and as shown on the photograph on Figure B15 in Appendix B. The degree of weathering of the bedrock samples is slightly weathered to fresh (i.e. W2 to W1) and the strength classification of the intact rock mass based on field identification is very strong to extremely strong (i.e. R5 to R6).

The Rock Quality Designation (RQD) measured on the core samples ranges between 77 per cent and 99 per cent, indicating a rock mass of good to excellent quality. The Total Core Recovery (TCR) and Solid Core Recovery (SCR) of samples recovered are between 97 per cent and 100 per cent and between 60 per cent and 100 per cent, respectively.

Point load strength tests were carried out on selected samples of the rock core and 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. The axial tests carried out on three (3) core samples of the granitic gneiss bedrock measured Is_{50} values ranging from about 7.7 MPa to 8.8 MPa and the diametral tests carried out on three (3) core samples of the granitic gneiss bedrock measured Is_{50} values ranging from about 6.9 MPa to 14.4 MPa.

One (1) Unconfined Compression (UC) test (ASTM D7012 – Standard Test Method for Compressive Strength and Elastic Moduli of Intact Rock Core Specimens) was carried out on a selected core sample of granitic gneiss bedrock obtained in Borehole B201-02 and measured a compressive strength of about 141 MPa, as summarized in Table B2-1 and detailed in Table B2-2 in Appendix B.

Also presented in Table B1 are the estimated Uniaxial Compressive Strength (UCS) values for each sample tested for point load strength based on a relationship between Is_{50} and UCS which is given by a correlation factor (K) (ASTM D5731 – Standard Test Method for Determination of the Point Load Strength Index of Rock and Application to Rock Strength Classification), which varies depending on the size of the core sample and the strength of the rock. For this site, the UCS values are based on an estimated average correlation factor (K) of 19.

Based on the laboratory UC test and the point load test results, in accordance with Table 3.5 in CFEM (2006), the granitic gneiss bedrock is classified as very strong (R5, 100 MPa < UCS < 250 MPa) to extremely strong (R6, UCS > 250 MPa).

4.4.5 Groundwater Conditions

In general, the overburden samples taken in the boreholes were moist to wet. Water levels observed in the boreholes upon completion of drilling ranged from about Elevations 178.0 m to 177.8 m, measured between about 0.7 m and 1.2 m below ground surface.

A standpipe piezometer was installed in Borehole B201-02 to allow monitoring of the groundwater level at the site. Details of the piezometer installation are shown on the Record of Borehole sheet in Appendix A. The groundwater level measured in the piezometer installation is summarized below.



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Foundation Element	Borehole No.	Ground Surface Elevation (m)	Groundwater Elevation (m)	Date of Measurement
Centre Pier (Two-Span Option)	B201-02	178.6	177.9 177.9	March 23, 2011 October 3, 2012

It should be noted that groundwater level in the area is subject to seasonal fluctuations and precipitation events, and should be expected to be higher during wet periods of the year.

4.5 North Abutment and Approach Embankment (One-Span Option)

A total of three (3) boreholes (Boreholes B201-03, B201-11 and B201-12) and two (2) Dynamic Cone Penetration Tests (DCPTs B201-DC01 and B201-DC02) were advanced at the alternative location of the proposed north abutment and one (1) borehole (Borehole B201-04) was advanced on the centerline of the north approach embankment of the one-span bridge configuration. In general, the subsurface conditions consist of topsoil underlain by a deposit of silty sand to sand, underlain by a stratum of silty clay to clay, which in turn is underlain in places by pockets of either sand and gravel or sandy silt to sand, and/or bedrock.

4.5.1 Topsoil

A layer of topsoil was encountered at the ground surface at all borehole locations. The surface of topsoil ranges from about Elevations 180.6 m to 179.5 m and its thickness ranges from 0.1 m to 0.3 m.

4.5.2 Silty Sand to Sand

A deposit of brown silty sand to sand trace gravel was encountered below the topsoil in Boreholes B201-03, B201-11 and B201-12. The upper portion of the silty sand deposit contains rootlets. The top of this deposit varies between about Elevations 179.4 m and 179.2 m and the thickness of the deposit is between about 0.9 m and 1.2 m.

The SPT 'N'-values measured within this deposit range from 3 blows to 8 blows per 0.3 m of penetration, indicating a very loose to loose relative density.

The natural water content measured on samples of this deposit ranges from about 17 per cent to 25 per cent.

4.5.3 Clayey Silt to Clay

A stratum of cohesive soil comprised of brown to grey clayey silt to silty clay to clay, some silt, trace to some sand was encountered either below the silty sand to sand deposit or topsoil at all borehole locations. In Borehole B201-04, the stratum is interlayered with an approximately 0.3 m thick pocket of sandy silt. The top of the cohesive stratum ranges from about Elevations 180.5 m to 178.1 m and the thickness of the cohesive stratum ranges from about 2.4 m to 4.1 m.

The SPT 'N'-values measured within the clayey silt to clay stratum range from 0 blows (weight of hammer) to 17 blows per 0.3 m of penetration, with the higher 'N'-values generally being recorded in Borehole B201-04



within the clayey silt portion of the deposit and at the interface of the sandy silt pocket below the clayey silt stratum. In situ field vane tests carried out within this stratum measured undrained shear strengths ranging from about 17 kPa to 77 kPa, but are typically less than 27 kPa. The sensitivity is calculated to range from about 2 to 13, but is mostly 3 and 4. The field vane tests results indicate that the clayey silt to clay stratum generally has a soft to stiff consistency.

The natural water content measured on samples of this stratum ranges from about 24 per cent to 75 per cent.

The grain size distributions of three (3) samples of the silty clay to clay portion of this stratum are shown on Figure B16 in Appendix B.

Atterberg limits tests were carried out on three (3) specimens of the silty clay to clay portion of this stratum and indicate liquid limits ranging from about 49 per cent to 51 per cent, plastic limits ranging from about 18 per cent to 20 per cent and plasticity indices ranging from about 29 per cent to 32 per cent. The results of the Atterberg limits tests are shown on the plasticity chart on Figure B17 in Appendix B and indicate the material to be silty clay of intermediate plasticity to clay of high plasticity.

4.5.4 Sand and Gravel / Sandy Silt to Sand

In Boreholes B201-03 and B201-12, an approximately 0.7 m and 0.9 m thick deposit of sand and gravel to sandy silt to sand some silt was encountered underlying the clay and silty clay strata at about Elevations 174.2 m and 174.6 m, respectively.

An SPT 'N'-value recorded within the sand and gravel layer is 50 blows per 0.08 m of penetration, indicating a very dense relative density. An SPT 'N'-value recorded within the sandy silt to sand layer is 11 blows per 0.3 m of penetration, indicating a compact relative density, with one SPT 'N'-value of 25 blows per 0.01 m of penetration measured at the contact with the underlying bedrock.

The natural water content measured on a specimen of the sand and gravel layer is 23 per cent.

4.5.5 Bedrock / Refusal

Bedrock was encountered below the clayey silt to clay stratum or below the sand and gravel/sandy silt to sand deposit and core samples were recovered at all borehole locations. The bedrock surface is inferred from refusal to dynamic cone penetration in DCPTs B201-DC01 and B201-DC02. These refusal depths, while they do not confirm bedrock elevations, may be inferred to indicate potential proximity to the bedrock interface. The depth to bedrock below ground surface and corresponding bedrock surface elevation (inferred or actual) is summarised below.

Foundation Element / Approach Embankment	Borehole	Depth to Bedrock Surface / Refusal (m)	Bedrock Surface / Refusal Elevation (m)	Comments
North Abutment (One-Span Option)	B201-03	6.0	173.5	Bedrock Cored
	B201-11	4.1	175.4	Bedrock Cored
	B201-12	5.9	173.7	Bedrock Cored
	B201-DC01	5.9	173.5	DCPT Refusal
	B201-DC02	5.2	174.6	DCPT Refusal



North Approach Embankment	B201-04	2.5	178.1	Bedrock Cored
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The bedrock surface slopes downwards towards the Still River from the borehole advanced at the centreline of the north approach embankment (Borehole B201-04) to the borehole advanced at the centreline of the proposed north abutment (Borehole B201-03), with the bedrock elevation decreasing by about 4.5 m towards the river (approximately 2.7H:1V slope or a dip of approximately 21° from the horizontal).

Based on the review of the bedrock core samples, the bedrock consists of granitic gneiss. In general the bedrock samples are described as fresh, foliated, medium crystalline, slightly porous, strong, pink, grey and black, as presented in the Record of Drillhole sheets in Appendix A, and shown on the photograph of the recovered samples on Figure B18 in Appendix B. The degree of weathering of the bedrock samples is fresh (i.e. W1) and the strength classification of the intact rock mass based on field identification is very strong to extremely strong (i.e. R5 to R6).

The Rock Quality Designation (RQD) measured on the core samples ranges from 90 per cent to 100 per cent, indicating a rock mass of excellent quality. The Total Core Recovery (TCR) and Solid Core Recovery (SCR) of samples recovered are between 92 per cent and 100 per cent and between 85 per cent and 100 per cent, respectively.

Point load strength tests 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. The axial test carried out on four (4) core samples of the granitic gneiss bedrock measured Is_{50} values ranging from about 6.2 MPa to 8.8 MPa and the diametral test carried out on three (3) core samples of the granitic gneiss bedrock measured Is_{50} value ranging from about 7.2 MPa to 10.7 MPa.

One (1) Unconfined Compression (UC) test (ASTM D7012 – Standard Test Method for Compressive Strength and Elastic Moduli of Intact Rock Core Specimens) was carried out on a selected core sample of granitic gneiss bedrock obtained in Borehole B201-03 and measured a compressive strength of about 127 MPa, as summarized in Table B2-1 and detailed in Table B2-4 in Appendix B.

Also presented in Table B1 are the estimated Uniaxial Compressive Strength (UCS) values for each sample tested for point load strength based on a relationship between Is_{50} and UCS which is given by a correlation factor (K) (ASTM D5731 – Standard Test Method for Determination of the Point Load Strength Index of Rock and Application to Rock Strength Classification) which varies depending on the size of the core sample and the strength of the rock. For this site, the UCS values are based on an estimated average correlation factor (K) of 19.

Based on the laboratory UC test and the point load test results, in accordance with Table 3.5 in CFEM (2006), the granitic gneiss bedrock is classified as very strong (R5, 100 MPa < UCS < 250 MPa).

4.5.6 Groundwater Conditions

In general, the overburden samples taken in the boreholes were moist to wet. The water level observed in the boreholes upon completion of drilling was at about Elevation 178.7 m, measured at about 0.8 m and 0.9 m below ground surface, except in Borehole B201-04 where the water level was not recorded.



A standpipe piezometer was installed in Borehole B201-03 to allow monitoring of the groundwater level at the site. Details of the piezometer installation are shown on the Record of Borehole sheet in Appendix A. The groundwater levels measured in the piezometer installation are summarized below.

Foundation Element	Borehole No.	Ground Surface Elevation (m)	Groundwater Elevation (m)	Date of Measurement
North Abutment (One-Span Option)	B201-03	179.5	179.5	March 21, 2011
			179.4	March 24, 2011
			179.0	October 3, 2012

It should be noted that groundwater level in the area is subject to seasonal fluctuations and precipitation events, and should be expected to be higher during wet periods of the year.

4.6 North Abutment and Approach Embankment (Two-Span Option)

A total of five (5) boreholes (Boreholes B201-05 to B201-09) were advanced at the location of the proposed north abutment and one (1) borehole (Borehole B201-10) was advanced on the centerline of the proposed Highway 69 alignment at the north approach embankment for the two-span bridge configuration. In general, the subsurface conditions consist of peat or topsoil over cobbles and boulders over bedrock.

4.6.1 Peat / Topsoil

An approximately 0.1 m and 0.2 m thick layer of black amorphous peat was encountered at the ground surface in Boreholes B201-06 and B201-08. A layer of topsoil between about 0.1 m and 0.5 m thick was encountered at the ground surface in Boreholes B201-05, B201-07, B201-09 and B201-10. The topsoil sample retrieved in Borehole B201-05 contains highly weathered schist rock fragments. The top of the peat and topsoil deposits range from about Elevations 192.2 m to 183.4 m.

The natural water content measured on two (2) specimens of the peat is about 478 per cent and 1181 per cent, and the organic content measured on these specimens is about 84 per cent and 25 per cent, respectively.

4.6.2 Cobbles and Boulders

An approximately 0.2 m thick layer of cobbles and 0.3 m thick layer of boulders were encountered below the peat in Boreholes B201-06 and B201-08. The cobbles and boulders were encountered at about Elevations 185.3 m and 183.2 m in the respective boreholes and the layer was cored through to the bedrock. Photographs of the recovered cobbles/boulder core from Boreholes B201-06 and B201-08 are presented on Figure B19 in Appendix B.

4.6.3 Bedrock / Refusal

Bedrock was encountered below the peat/topsoil and/or underlying cobbles and boulders layers, and core samples were recovered in Borehole B201-06 to B201-08. The bedrock surface is inferred from split-spoon refusal in Borehole B201-05, and exposed by shovel (hand) excavation to confirm refusal at the bottom of the



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topsoil layer in Boreholes B201-09 and B201-10. The depth to bedrock below ground surface and corresponding bedrock surface elevation is summarised below.

Foundation Element / Approach Embankment	Borehole	Depth to Bedrock Surface / Refusal (m)	Bedrock Surface / Refusal Elevation (m)	Comments
North Abutment (Two-Span Option)	B201-05	0.5	183.4	Split-Spoon Refusal
	B201-06	0.3	185.1	Bedrock Cored
	B201-07	0.2	184.2	Bedrock Cored
	B201-08	0.5	182.9	Bedrock Cored
	B201-09	0.1	185.4	Shovel Refusal
North Approach Embankment	B201-10	0.1	192.1	Shovel Refusal

In general, the bedrock surface slopes downwards towards Still River from the borehole advanced at the centreline of the north approach embankment (Borehole B210-10) to the borehole advanced at the centreline of the proposed north abutment (Borehole B201-07), with the bedrock elevation decreasing by about 8.0 m towards the river (approximately 2.4H:1V slope or a dip of approximately 23° from the horizontal).

Based on the review of the bedrock core samples, the bedrock consists of schist to granitic gneiss. In general the bedrock samples are described as fresh to slightly weathered, foliated, medium to coarse crystalline, slightly to moderately porous, strong, pink, grey, brown and black and containing quartzite banding, as presented in the Record of Drillhole sheets in Appendix A, and shown on the photograph of the recovered samples on Figure B19 in Appendix B. The degree of weathering of the bedrock samples is fresh to slightly weathered (i.e. W1 to W2) and the strength classification of the intact rock mass based on field identification is very strong to extremely strong (i.e. R5 to R6).

The Rock Quality Designation (RQD) measured on the core samples ranges between 67 per cent and 100 per cent, indicating a rock mass of fair to excellent quality. The Total Core Recovery (TCR) and Solid Core Recovery (SCR) of samples recovered are between 47 per cent and 100 per cent and between 40 per cent and 100 per cent, respectively.

Point load strength tests 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. The axial tests carried out on three (3) core samples of the granitic gneiss bedrock measured Is_{50} values ranging from about 6.7 MPa to 9.0 MPa and the diametral tests carried out on three (3) core samples of the granitic gneiss bedrock measured Is_{50} values ranging from about 7.2 MPa to 12.8 MPa.

One (1) Unconfined Compression (UC) test (ASTM D7012 – Standard Test Method for Compressive Strength and Elastic Moduli of Intact Rock Core Specimens) was carried out on a selected core sample of granitic gneiss bedrock obtained in Borehole B201-08 and measured a compressive strength of about 169 MPa, as summarized in Table B2-1 and detailed in Table B2-3 in Appendix B.

Also presented in Table B1 are the estimated Uniaxial Compressive Strength (UCS) values for each sample tested for point load strength based on a relationship between Is_{50} and UCS which is given by a correlation factor (K) (ASTM D5731 – Standard Test Method for Determination of the Point Load Strength Index of Rock and Application to Rock Strength Classification) which varies depending on the size of the core sample and the



strength of the rock. For this site, the UCS values are based on an estimated average correlation factor (K) of 19.

Based on the laboratory UC test and the point load test results, in accordance with Table 3.5 in CFEM (2006), the granitic gneiss bedrock is classified as very strong (R5, 100 MPa < UCS < 250 MPa).

4.6.4 Groundwater Conditions

In general, the overburden samples taken in the boreholes were moist. The water level observed in Boreholes B201-06 and B201-08 upon completion of drilling was at about Elevations 184.5 m and 182.5 m, measured at about 0.9 m below ground surface.

A standpipe piezometer was installed in Borehole B201-07 to permit monitoring of the groundwater level at the site. Details of the piezometer installation are shown on the Record of Borehole sheet in Appendix A. The groundwater level measured in the piezometer installation is summarized below.

Foundation Element	Borehole No.	Ground Surface Elevation (m)	Groundwater Elevation (m)	Date of Measurement
North Abutment (Two-Span Option)	B201-07	184.4	183.9 183.9	March 4, 2011 October 3, 2012

It should be noted that groundwater level in the area is subject to seasonal fluctuations and precipitation events, and should be expected to be higher during wet periods of the year.

5.0 CLOSURE

Mr. Matt Rhody, senior technician with Golder and Mr. Tony Tomory, E.I.T., directed the drilling program. This report was prepared by Ms. T. Veronica Ayetan, P.Eng., a geotechnical engineer, and reviewed by Mr. J. Paul Dittrich, Ph.D., P.Eng., a senior geotechnical engineer and Principal with Golder. Mr. Jorge M. A. Costa, P.Eng., Golder's Designated MTO Contact for this project and Principal with Golder, conducted an independent quality control review of the report.



Report Signature Page

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[Http://capws/sites/0911116014highway69FourLaning/Contract 2/Reporting/Final/Still River SBL Bridge/09-1111-6014-2523 RPT 14Sep11 Highway 69 Still River SBL Bridge.docx](http://capws/sites/0911116014highway69FourLaning/Contract%20Reporting/Final/Still%20River%20SBL%20Bridge/09-1111-6014-2523%20RPT%2014Sep11%20Highway%2069%20Still%20River%20SBL%20Bridge.docx)



PART B

FOUNDATION DESIGN REPORT

STILL RIVER SBL BRIDGE STRUCTURE, SITE NO. 44-458/2

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 5404-05-00; WP 5404-08-01



6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides engineering recommendations for detail design of the proposed Still River SBL bridge 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 approach embankments. Where comments are made on construction, they are provided in order to highlight those aspects which could affect the design of the project. Those requiring information on the aspects of construction should 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 – Structure Alternatives

Golder Associates Ltd. (Golder) has been retained by URS Canada Inc. (URS) on behalf of the Ministry of Transportation, Ontario (MTO) to provide recommendations on foundation aspects for the Detail Design of the Still River SBL Bridge within Contract 2 along the proposed Highway 69 alignment, associated with the four-laning of Highway 69 in the Townships of Wallbridge, Henvey and Mowat. From a foundation perspective (as discussed in Section 6.9), a two-span structure is the preferred alternative for the Still River SBL Bridge crossing. However, since the feasibility of a one-span bridge was being evaluated during the initial design stage, the following sections provide recommendations for the two-span bridge option and for the one-span bridge option (for comparison/information purposes).

Based on the General Arrangement (GA) Drawings provided by URS on October 1, 2010 and April 5, 2011 for the one-span and two-span bridge structures, respectively, the grade of the proposed Still River SBL Bridge deck varies between about Elevations 188.9 m and 189.3 m. The existing ground surface varies between about Elevations 192.2 m and 178.6 m at the borehole locations.

6.2 Foundation Options

Given the significant variation in the thickness of weak/soft cohesive deposits at the site as well as the variation in the depth to bedrock, a shallow foundation comprised of a strip/spread footing founded directly on bedrock is considered the only appropriate foundation alternative to support the two-span bridge option at the north abutment.

For the south abutment, two-span centre pier and one-span north abutment, deep foundations comprised of either steel H-piles or caissons are considered the preferred alternatives for the design of the bridge structure.

The following sections provide recommendations for spread footing, steel H-pile and caisson foundations, where applicable, to support the bridge foundation elements.

The advantages, disadvantages, relative costs and risk/consequences for each of the foundation options are summarized in Tables 1A to 1D.



6.3 Spread Footings

At this site, shallow foundations comprised of a strip or spread footing are only feasible founded directly on bedrock at the two-span north abutment. The overburden deposits at the location of the other foundation elements (i.e. south abutment, two-span centre pier and one-span north abutment) are too weak/soft to support the bridge structure.

6.3.1 Footing Options and Geotechnical Resistances/Reactions

Shallow foundation alternatives for support of the foundation elements associated with the one-span and two-span option, where applicable, are summarized below.

6.3.1.1 South Abutment

Given the presence of the alternating deposits of generally soft to firm and compressible clayey silt to silty clay deposits and loose to compact sand to silt deposit within the upper 24 m of the overburden, a shallow foundation comprised of a strip/spread footing founded on either the native soil deposits or perched within the new embankment fill is not recommended at the location of the south abutment for either the two-span or one-span bridge options.

6.3.1.2 Centre Pier (Two-Span Option)

Given the presence of the very loose to loose sand to silt deposit overlying generally firm and compressible silty clay to clay deposits up to about 4.3 m thick, a shallow foundation comprised of a strip/spread footing founded on the native soil deposits is not recommended at the location of the two-span centre pier.

6.3.1.3 North Abutment (One-Span Option)

Given the presence of very loose to loose sand to silt deposit overlying firm to stiff and compressible silty clay to clay deposits up to about 4.1 m thick, a shallow foundation comprised of a strip/spread footing founded on either the native soil deposits or perched within the new embankment fill is not recommended at the location of the one-span north abutment.

6.3.1.4 North Abutment (Two-Span Option)

The north abutment of the two-span bridge may be supported on a strip/spread footing placed on the properly prepared gneiss or granitic gneiss bedrock or founded on mass concrete placed on the properly prepared gneiss or granitic gneiss bedrock.

The existing ground surface at the borehole locations in the area of the north abutment footprint varies from about Elevations 185.5 m to 183.4 m. Along the proposed abutment, the overburden consists of deposit of topsoil and/or peat between about 0.2 m to 0.6 m thick, underlain by about an up to 0.3 m thick zone of cobbles or boulders, underlain by bedrock. The bedrock surface elevation as encountered in the boreholes at the



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proposed north abutment / foundation element varies by up to 2.2 m between the northwest and the southeast corner of the abutment and is summarized below.

Location	Relevant Borehole	Ground Surface Elevation (m)	Bedrock Surface Elevation (m)
Northwest Corner	B201-06	185.4	185.1
Southwest Corner	B201-05	183.9	183.4 ¹
Centre	B201-07	184.4	184.2
Northeast Corner	B201-09	185.5	185.4
Southeast Corner	B201-08	183.4	182.9

Note: 1. SPT refusal only. Bedrock not confirmed by coring.

Based on the GA Drawing (April 2011) provided by URS, the underside of the north abutment footing for the two-span bridge is proposed to be founded at approximately Elevation 184.0 m. Given this, foundation elevation of the proposed abutment footing, an up to about 1.5 m deep excavation including removal of up to about 1.4 m of bedrock will be required to construct the northern portion of the footing on bedrock. Along the southern portion of the abutment, placement of an about 1.1 m thick pad of mass concrete on properly prepared bedrock will be required to achieve the proposed founding elevation.

A Non-Standard Special Provision (NSSP) should be included in the Contract Documents for mass concrete for the proposed north abutment strip/spread footing to accommodate the variations in the bedrock surface; an example is provided in Appendix C. Furthermore, following excavation of the overburden organic soils and zones of cobbles/boulders and prior to placing any concrete, it will be necessary to clean, scale and remove all loose, shattered and/or fractured rock within the area of the footing to ensure a proper bond to the bedrock. A provision should be included in the Contract Documents to address the requirements for field inspection. In order to carry out this inspection, the excavation should be dry.

The factored geotechnical axial resistance at Ultimate Limits States (ULS) and the geotechnical reaction at Serviceability Limits States (SLS) for strip/spread footings founded directly on properly prepared bedrock or strip/spread footings founded on mass concrete placed on properly prepared bedrock are given below.

Shallow Foundation Alternative for the North Abutment (Two-Span Option)	Factored Geotechnical Axial Resistance at Ultimate Limit State (ULS)	Geotechnical Reaction at Serviceability Limit State (SLS) for 25 mm of Settlement
Spread footing on properly prepared gneiss or granitic gneiss bedrock (at centre, northeast and northwest corner of footing)	10,000 kPa	N/A ²
Spread footings on mass concrete ¹ on properly prepared gneiss or granitic gneiss bedrock (at southwest and southeast corner of footing)	10,000 kPa	N/A ²

- Note:
1. Assuming that the strength of the mass concrete is at least 25 MPa.
 2. The geotechnical reaction at SLS for 25 mm of settlement will be greater than the factored geotechnical axial resistance at ULS for spread footings on bedrock or on mass concrete on bedrock and as a result the SLS condition does not apply.



The geotechnical resistances provided above are given for loads applied perpendicular to the surface of the footing. Where the load is not applied perpendicular to the surface of the footing, 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)* and its *Commentary*.

For a spread footing founded on the properly prepared and inspected bedrock (or on mass concrete over bedrock), the geotechnical reaction at SLS for 25 mm of settlement will be greater than the factored geotechnical resistance at ULS and as a result the SLS condition does not apply. For a footing placed on mass concrete, the factored geotechnical axial resistance at ULS, as given above, assumes that the strength of the concrete used to form the pad has an unconfined compression strength of at least 25 MPa.

6.3.1.5 Resistance to Lateral Loads

The resistance to lateral forces/sliding resistance between the concrete footing or mass concrete and the bedrock (applicable to the north abutment of the two-span bridge only) should be calculated in accordance with Section 6.7.5 of the *CHBDC*. The following summarizes the coefficient of friction, $\tan \delta$, for the interface materials.

Interface Materials	Coefficient of Friction ($\tan \delta$)
Concrete footing or mass concrete on properly prepared bedrock	0.70

If necessary, the sliding resistance between the concrete footing and/or mass concrete and the bedrock can be supplemented by dowelling into the bedrock. The horizontal resistance of the dowels is dependent on the strength of the bedrock, grout and steel. For this site, where the rock mass is essentially as strong or stronger than concrete, the design of dowels into the rock may be handled in the same way as the dowel embedment into the concrete for unconfined compressive strength (UCS) of the grout similar to that of the concrete. The dowels should have a minimum embedded length within the sound bedrock of 1 m, and the structural strength of the dowels and compressive strength of the grout should not be exceeded. If dowelling into bedrock is adopted at this site, an NSSP should be included in the Contract Documents to specify the installation, materials and testing of the dowels, such as the example provided in Appendix C.

6.3.1.6 Frost Protection

For a footing founded directly on the properly prepared granite or granitic gneiss bedrock or a footing on mass concrete over bedrock at this site, a minimum soil cover for frost protection is not required.

6.4 Steel H-Pile Foundations

Given the presence of the thick and compressible foundation soils encountered throughout the majority of this site, deep foundations consisting of driven steel H-piles are considered to be the preferred alternative for the support of the south abutment and for the centre pier of the two-span bridge option. At the north abutment of the two-span option where bedrock is present at shallow depth, a shallow foundation is the recommended option,



although a pile supported foundation could be considered; however, trenching of the shallow bedrock across the limits of the proposed pile cap to achieve the minimum required pile lengths would be required.

Pile installation should be carried out in accordance with OPSS 903 *Deep Foundations*.

6.4.1 Geotechnical Axial Resistance/Reaction and Downdrag (Negative Skin Friction)

Steel H-pile foundation alternatives for support of the foundation elements associated with the one-span and two-span option, where applicable, are summarized below.

6.4.1.1 South Abutment

Given the extent and the nature of the foundation soils (i.e. up to at least 57.5 m deep and generally loose to compact sand to silt and soft to stiff silty clay to clay deposits), friction piles are recommended for support of the proposed south abutment for the one-span and two-span options.

As a result of the loading from the south approach embankment, consolidation settlement of the underlying cohesive deposits will occur. The difference in the vertical movement between the thick overburden (i.e. from the consolidation settlement and creep of the cohesive deposits) and the long friction piles (i.e. from the elastic deformation of the piles under the load from the bridge structure and from the punching of the piles into the soil deposit below the pile tip) will result in the development of downdrag on the piles (negative skin friction). If the piles for the abutment are installed prior to the construction of the approach embankment, large dragloads will develop that may exceed the structural capacity of the pile or result in a low geotechnical axial resistance for structural design consideration. Therefore, it is recommended that settlement mitigation be adopted at the south abutment and the approach embankment area to reduce the differential vertical movement between the overburden soils and the friction piles (H-piles), which will in turn reduce dragloads on the piles. For the south approach embankment, it is recommended that a partial preload embankment be constructed and left in place for a specified preload period, prior to the construction of the remainder of the south approach embankment with lightweight fill (Expanded Polystyrene Styrofoam (EPS)). This settlement mitigation method is consistent with that recommended for the construction of the immediately adjacent embankments crossing Swamp 204. The details of the settlement mitigation options (together with stability mitigation options) for the south approach embankment are discussed in Section 6.8.3.1.

Analyses to estimate dragloads and geotechnical resistances for the recommended friction pile foundation option at the south abutment were carried out in accordance with Section 6.8.4 of *CHBDC and its Commentary* using the method proposed by Briaud and Tucker (1996). It is noted that the methodology employed to assess the pile reaction at SLS of the pile and the associated dragload is dependent on a number of factors including the pile length, foundation conditions at the pile tip, the unfactored dead load on the pile and the anticipated post-construction settlement profile of the foundation soils. If any of these factors and/or the recommended embankment settlement mitigation option is different from those assumed in the analysis, the dragload and pile capacity presented below would need to be reassessed.

In order to estimate the dragloads and the geotechnical axial resistances, the following structural pile capacities and the unfactored and factored dead load per pile, were provided by URS as inputs to the analyses.



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Pile Section	Structural Pile Capacity	Unfactored Dead Load per Pile	Factored Dead Load per Pile	Factored Geotechnical Axial Resistance at ULS Required for Structural Design
HP 310x110	4,200 kN	750 kN	900 kN	1,100 kN

Detail dragload/geotechnical axial resistance analyses were carried out for various pile lengths driven to or into the compact to dense sand deposit underlying the lower silty clay deposit and the results are summarized below. Consideration was given to founding driven piles within the upper sand layer or within the lower portions of the soft to stiff silty clay to clay deposit during design; however, given the risk and uncertainty associated with the magnitude of the long-term post-construction settlement below these pile tip elevations as well as the lower geotechnical axial resistance at ULS, piles founded within the upper sand layer or within the lower silty clay to clay deposit were not considered an appropriate alternative.

Pile Type / Pile Tip Elevation / Approximate Pile Length ¹	Founding Condition	Estimated Depth of Neutral Plane from Top of Pile	Unfactored Dead Load per Pile	Unfactored Dragload (Negative Skin Friction)	Unfactored Load Distribution (Positive Skin Friction + Tip Resistance)	Estimated Vertical Movement of the Top of Pile
HP 310x110 / Elev. 140 m / 42 m	Piles driven to the top of the compact to dense silt and sand	21.5 m	750 kN	1,450 kN	1900 kN + 300 kN = 2,200 kN	20 mm
HP 310x110 / Elev. 138 m / 44 m	Piles driven 2 m into compact to dense silt and sand	22.5 m	750 kN	1,550 kN	2,000 kN + 300 kN = 2,300 kN	20 mm
HP 310x110 / Elev. 137 m / 45 m	Piles driven 3 m into compact to dense silt and sand	23.5 m	750 kN	1,600 kN	2,050 kN + 300 kN = 2,350 kN	20 mm
HP 310x110 / Elev. 136 m / 46 m	Piles driven 4 m into compact to dense silt and sand	24 m	750 kN	1,650 kN	2,100 kN + 300 kN = 2,400 kN	20 mm
HP 310x110 / Elev. 128 m / 54 m	Piles driven to practical refusal on probable boulders ²	28 m	750 kN	2,050 kN	2,550 kN + 250 kN = 2,800 kN	20 mm

- Note:
1. Assuming the underside of the pile cap is at approximately Elevation 182 m as per the April 2011 GA Drawing provided by URS.
 2. The actual pile length may vary during piling operations depending on the thickness and composition of the boulder zone and the actual resistance achieved as calculated by the Hiley method.

Using the results above, the following is an evaluation of the different pile length alternatives for the south abutment:



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Pile Type / Pile Tip Elevation / Approximate Pile Length ¹	Maximum Load at Neutral Plane (Factored) ²	Structural Pile Capacity	Factored Geotechnical Axial Resistance at ULS ³	Factored Geotechnical Axial Resistance at ULS Required for Structural Design	Estimated Vertical Movement of the Top of Pile	All Design Criteria Satisfied
HP 310x110 / Elev. 140 m / 42 m	1,450 kN + 900 kN = 2,350 kN	4,200 kN	1,400 kN	1,100 kN	20 mm	Yes
HP 310x110 / Elev. 138 m / 44 m	1,950 kN + 900 kN = 2,850 kN	4,200 kN	1,500 kN	1,100 kN	20 mm	Yes
HP 310x110 / Elev. 137 m / 45 m	2,000 kN + 900 kN = 2,900 kN	4,200 kN	1,500 kN	1,100 kN	20 mm	Yes
HP 310x110 / Elev. 136 m / 46 m	2,050 kN + 900 kN = 2,950 kN	4,200 kN	1,500 kN	1,100 kN	20 mm	Yes
HP 310x110 / Elev. 128 m / 54 m	2,550 kN + 900 kN = 3,450 kN ⁴	4,200 kN	1,800 kN	1,100 kN	20 mm	Yes

- Notes:
1. Assuming the underside of the pile cap is at approximately Elevation 182 m as per the April 2011 GA Drawing provided by URS.
 2. The maximum load at the neutral plane is the sum of the dragload (negative skin friction) multiplied by 1.25 (in accordance with the load factor for negative skin friction on piles in Table 3.2 of the *CHBDC*) and the factored dead load (900 kN as provided by URS).
 3. The factored geotechnical axial resistance at ULS is the sum of the ultimate shaft and tip resistance, multiplied by 0.4 (in accordance with the resistance factor for static analysis on piles in compression on Table 6.1 of the *CHBDC*).

Based on the evaluation above, all of the alternative pile tip elevations/pile lengths considered are able to satisfy the required factored geotechnical axial resistance at ULS without exceeding the structural capacity of the pile at the neutral plane while limiting the estimated vertical movement at the top of the pile to less than 25 mm. From a foundations perspective, the HP 310x110 pile driven to Elevation 137 m (i.e. 45 m long piles driven about 3 m into the compact to dense sand deposit) is recommended for adoption as the pile foundation for the south abutment as it provides an appropriate magnitude of geotechnical axial resistance while maintaining an estimated vertical movement of the top of the pile to about 20 mm.

In summary, the following factored geotechnical axial resistance at ULS and the geotechnical reaction at Serviceability Limits States (SLS) are recommended for the design of the pile foundation:

Pile Foundation for the South Abutment	Approximate Pile Tip Elevation	Approximate Pile Length ¹	Factored Geotechnical Resistance at Ultimate Limit States (ULS)	Geotechnical Reaction at Serviceability Limit States (SLS) for about 25 mm of Settlement ²
HP 310x110 piles driven 3 m into compact to dense silt and sand	137 m	45 m	1,500 kN	750 kN

- Notes:
1. Assuming the underside of the pile cap is at approximately Elevation 182 m as per the April 2011 GA Drawing provided by URS.
 2. The geotechnical reaction at SLS for 25 mm of settlement includes the effect of the estimated unfactored dragload.



Given the uncertainty in terms of the geotechnical axial resistance associated with a friction pile compared to an end-bearing pile, it is recommended that a pile load test be completed at the proposed south abutment to confirm the geotechnical axial resistance and settlement behaviour of a single friction pile.

Furthermore, as a result of artesian conditions encountered at the south abutment during the borehole investigation, a seepage control system/sand filter will be required immediately under the pile cap as described in Section 6.11.4.1.

6.4.1.2 Centre Pier (Two-Span Option)

Given the presence of a reasonable thickness (i.e. between about 5.4 m and 9.2 m) of the very loose to loose sand to silt and generally firm silty clay to clay deposit overlying bedrock at this location, piles driven to bedrock are recommended for support of the centre pier for the two-span bridge structure.

Considering the sloping nature of the bedrock surface at the location of this foundation element, the piles should be provided with pile points. Specifically, given the estimated angle of the bedrock surface (a dip of up to approximately 37° from the horizontal towards the river), it is recommended that the tips of the piles be fitted with either Titus or Oslo pile points to facilitate proper seating on the sloping very strong to extremely strong granitic gneiss bedrock. The appropriate NSSP should be included in the Contract Documents, such as the example included in Appendix C. The driving procedures to enable pile seating depend on the type of pile driving rig used and such procedures need to be established at the time of construction. Generally, the procedures will involve a reduction in hammer energy once abrupt peaking is met to ease the pile point into the rock.

The following summarizes the approximate pile tip elevation(s), approximate pile length(s), and the factored geotechnical axial resistance at ULS as well as the geotechnical reaction at SLS for the steel H-pile foundation option at the proposed centre pier. It should be noted that a factor of 0.8 was used to reduce the factored geotechnical resistance at ULS to take into consideration the sloping bedrock at the proposed centre pier location. Further, considering the sloping nature of the bedrock and the variability in the top of the probable bedrock surface at the proposed pier, provisions should be made in the Contract Documents to deal with varying pile lengths at the location of the centre pier.



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Pile Foundation for the Centre Pier (Two-Span Option)	Approximate Pile Tip Elevation(s)	Approximate Pile Length ¹	Factored Geotechnical Axial Resistance at Ultimate Limit State (ULS)	Geotechnical Reaction at Serviceability Limit State (SLS) for 25 mm of Settlement
HP 310x110 piles driven into bedrock (east side of pier)	172.9 m to 170.9 m	4.6 m to 6.6 m	1,600 kN	N/A ²
HP 310x110 piles driven into bedrock (centre of pier)	172.4 m	5.1 m	1,600 kN	N/A ²
HP 310x110 piles driven into bedrock (west side of pier)	172.4 m to 169.4 m	5.1 m to 8.1 m	1,600 kN	N/A ²

- Notes:
1. Assuming the underside of the pile cap is at approximately Elevation 177.5 m as per the April 2011 GA Drawing provided by URS.
 2. The geotechnical reaction at SLS for 25 mm of pile settlement on granitic gneiss bedrock will be greater than the factored axial resistance at ULS for piles on bedrock and as a result the SLS condition does not apply.

6.4.1.3 North Abutment (One-Span Option)

Given the limited thickness (i.e. less than about 6 m) of the very loose to loose sand to silt deposit and soft silty clay to clay deposit overlying bedrock at this location, piles driven to bedrock are recommended for support of the north abutment for the one-span bridge structure.

As discussed in more detail in Section 6.8.3.2, a Factor of Safety (FoS) greater than or equal to 1.3 for the up to about 9.5 m high north approach embankment can only be achieved by full sub-excavation and replacement of the generally firm silty clay to clay deposit in this area. In the area of the abutment, granular material should be used for backfilling in order to allow driving the piles through the fill to bedrock. Prior to the installation of the piles, settlement mitigation (i.e. preloading) would also be required at the north abutment and approach embankment area to reduce the magnitude of subsequent lateral soil movement affecting the abutment piles (associated with the irregular geometry of the sub-excavation and replacement zone below the front slopes of the approach embankment). For the north approach embankment, a rock fill preload embankment must be required to be constructed well ahead of the construction of the abutment and left in place for a specified preload period to allow for the settlement of the rock fill and lateral squeezing of the cohesive deposit to occur prior to partial removal of the fill, pile driving and construction of the abutment. The details of the stability and settlement mitigation options for the north approach embankment for the one-span bridge option are discussed in Section 6.8.3.2.

The following summarizes the approximate pile tip elevation(s), approximate pile length(s), and the factored geotechnical axial resistance at ULS as well as geotechnical reaction at SLS for the steel H-pile foundation option at the proposed north abutment of the one-span option.



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Pile Foundation for the North Abutment (One-Span Option)	Approximate Pile Tip Elevation	Approximate Pile Length ¹	Factored Geotechnical Resistance at Ultimate Limit State (ULS)	Geotechnical Reaction at Serviceability Limit State (SLS) for 25 mm of Settlement
HP 310x110 piles driven into bedrock (east side of abutment)	174.6 m to 173.7 m	8.4 m to 9.3 m	2,000 kN	N/A ²
HP 310x110 piles driven into bedrock (centre of abutment)	173.5 m	9.5 m	2,000 kN	N/A ²
HP 310x110 piles driven into bedrock (west side of abutment)	175.4 m to 173.5 m	7.6 m to 9.5 m	2,000 kN	N/A ²

- Notes:
1. Assuming the underside of the pile cap is at approximately Elevation 183 m as per the preliminary April 2011 GA Drawing provided by URS.
 2. The geotechnical reaction at SLS for 25 mm of pile settlement on granitic gneiss bedrock will be greater than the factored axial resistance at ULS for piles on bedrock and as a result the SLS condition does not apply.

If consideration is given to constructing the one-span north approach embankment with expanded polystyrene (EPS) (including a granular levelling course and a minimum 1 m thick conventional soil cover on top of the EPS) rather than full sub-excavation and replacement of the cohesive deposit, downdrag will need to be taken into account for the pile design, as follows:

Pile Foundation for the North Abutment (One-Span Option)	Approximate Pile Tip Elevation	Approximate Pile Length ¹	Unfactored Dragload (Negative Skin Friction)
HP 310x110 piles driven to bedrock (east side of abutment)	174.6 m to 173.6 m	8.4 m to 9.3 m	100 kN to 200 kN
HP 310x110 piles driven to bedrock (centre of abutment)	173.5 m	9.5 m	200 kN
HP 310x110 piles driven to bedrock (west side of abutment)	175.4 m to 173.5 m	7.6 m to 9.5 m	75 kN to 200 kN

- Note:
1. Assuming the underside of the pile cap is at approximately Elevation 183 m as per the preliminary April 2011 GA Drawing provided by URS.

For the EPS embankment construction option, in order to eliminate additional dragloads and potential eccentric loads on the piles, consideration would have to be given to constructing a Retained Soil System (RSS) wall in front of the abutment in place of a 2 horizontal to 1 vertical (2H:1V) front slope. Assuming a 0.6 m wide strip footing embedded 1.9 m below ground surface (for frost protection), the factored geotechnical axial resistance at ULS for the RSS wall facing footing is estimated to be 60 kPa.

Given the additional cost associated with the construction of a RSS wall together with the high cost for constructing the approach embankment with EPS, full sub-excavation of the cohesive deposit and replacement with granular fill (at the abutment) is considered the more effective alternative for the design of the north abutment and approach embankment for a one-span bridge structure option.



6.4.1.4 North Abutment (Two-Span Option)

Due to the shallow nature of the overburden overlying the granitic gneiss bedrock at the two-span north abutment location, pile foundations will not be practical and a significant amount of excavation/trenching into the strong bedrock would be required to achieve minimum pile lengths for an integral abutment design. As such, a pile foundation at the two-span north abutment is not recommended and has not been considered further herein.

6.4.2 Set Criteria

For piles driven to bedrock, set criteria are highly dependent on pile driving hammer type and the selected pile. The set criteria can be established through a variety of methods, including empirical correlations and wave equation analyses, at the time of construction once the hammer and pile types are known. The criteria need to be set to also avoid overdriving and possibly damaging the pile. Based on our experience, consideration should be given to the following preliminary criteria for piles driven to bedrock:

- The piles should be driven to an initial set equal to or greater than 10 blows per 12 mm of penetration (unless abrupt peaking occurs) using a hammer with rated energy of about 50 kilojoules, but not exceeding 60 kilojoules.
- On reaching the required set, the hammer energy should be reduced by about 75 per cent and the pile should be re-driven by increasing the hammer energy slowly up to the maximum rated energy over about 40 blows to improve the process of seating the pile on the sloping bedrock surface.
- A final set of no less than 10 blows per 12 mm of penetration should be obtained at the maximum hammer energy. Provision should be made to re-tap all piles to confirm the set after adjacent piles have been driven.

For friction piles, the pile capacity should be checked in the field by the use of the Hiley formula (MTO Standard Drawing SS 103-11) during the final stages of driving, starting about 1.5 m above the design pile tip elevation to verify that the ultimate capacity specified has been achieved. All pile installation/driving should be in accordance with OPSS 903 *Deep Foundations*.

6.4.3 Pile Driving Note(s)

The pile driving notes that should be added to the Contract Drawings are Notes 4 and 5 in Clause 3.3.3 of the MTO Structural Manual (2008), as follows:

At the south abutment (Note 4):

- “Piles to be driven in accordance with Standard SS 103-11 using an ultimate geotechnical resistance of 3,000 kN per pile but must be driven below Elev. 138 m and not below Elev. 136 m without approval of the engineer.”

At the centre pier (Note 5):

- “Piles to be driven to bedrock.”



6.4.4 Resistance to Lateral Loads

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

Lateral loading could be resisted fully or partially by the use of battered piles.

The resistance to lateral loading in front of a single pile may be calculated using subgrade reaction theory where the coefficient of horizontal subgrade reaction, k_h (kPa/m), is based on the 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)
z	=	depth (m)
B	=	pile diameter or width (m)

and for cohesive soils:

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

where:

s_u	=	undrained shear strength of the soil (kPa)
B	=	pile diameter or width (m)

The values of n_h (Terzaghi, 1955 and Reese, 1975) and s_u (values taken from the design line shown on Figure 1) for use in the calculations of the coefficient of horizontal subgrade reaction (k_h) within the native subsoils/fills to be utilized for the structural lateral analysis of the piles at this site are summarized below.

Foundation Element (Relevant Boreholes)	Soil Unit	Elevation (m)	n_h (kPa/m)	s_u^2 (kPa)
South Abutment (B201-01 and S204-09)	Embankment Fill above Water / Groundwater Level (assumed to be Compacted Granular Fill)	182.1 – 180.9	7,500 ¹	-
	Soft Clayey Silt to Silty Clay (Near Surface)	180.9 – 178.5	-	25
	Loose to Compact Sand to Sandy Silt (Upper)	178.5 – 172.1	3,000	-
	Firm Silty Clay to Clay (Upper)	172.1 – 167.3	-	40
	Loose to Compact Silt to Sandy Silt Interlayer	167.3 – 165.6	2,000	-



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Foundation Element (Relevant Boreholes)	Soil Unit	Elevation (m)	n_h (kPa/m)	s_u^2 (kPa)
South Abutment (B201-01 and S204-09)	Firm Silty Clay (Upper)	165.6 – 165.0	-	40
	Stiff Silty Clay (Upper)	165.0 – 161.1	-	60
	Loose Sand and Silt to Silty Interlayer	161.1 – 157.7	2,000	-
	Stiff Silty Clay (Lower)	157.7 – 150.0	-	60
	Stiff to Very Stiff Silty Clay (Lower)	150.0 – 141.7	-	60 – 90
	Compact to Dense Silt (Lower)	141.7 – 136.8	5,000	-
	Compact to Dense Sand to Silty Sand (Lower)	136.8 – 131.1	15,000	-
	Dense Silt (Lower)	131.1 – 128.0	8,000	-
Centre Pier – Two-Span Option (B201-02, B201-13 and B201-14))	Very Loose to Loose Sand to Silt	178.4 – 175.7	2,000	-
	Soft to Firm Silty Clay to Clay	175.7 – 172.4	-	24 – 38
North Abutment – One-Span Option (B201-03, B201-11 and B201-12)	Embankment Fill above Water / Groundwater Level (assumed to be Compacted Granular Fill)	183.0 – 179.5	7,500 ¹	-
	Very Loose to Loose Sand to Sandy Silt	179.5 – 178.3	2,000	-
	Soft to Firm Clay	178.3 – 174.2	-	15 – 31
	Very Dense Sand and Gravel	174.2 – 173.5	25,000	-

- Notes: 1. The new granular fill must extend a minimum distance of five (5) pile diameters away from the outer edges of the piles in all directions.
2. Values taken as the design line shown on Figure 1.

For a single HP 310x110 vertical pile, the estimated factored lateral resistances at ULS as well as the estimated 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 5.0), developed by Ensoft Inc.

Foundation Element	Pile Type ¹	Factored Geotechnical Lateral Resistance at Ultimate Limit State (ULS)	Geotechnical Lateral Resistance at Serviceability Limit State (SLS) for 10 mm of Deflection
South Abutment	HP 310x110 (> 45 m long pile)	75 kN	25 kN
	HP 310x152 (54 m long pile)	165 kN	30 kN
Centre Pier – Two-Span Option	HP 310x110	45 kN	30 kN
North Abutment – One-Span Option	HP 310x110	175 kN	30 kN

- Note: 1. All cases assume a 750 kN unfactored dead load applied at the top of the pile.

Based on the above, it is considered that both structural and geotechnical resistances of the piles should be evaluated to establish the governing case at ULS. At SLS, the horizontal resistance of the piles will be controlled by deflections and the horizontal resistance of the pile should be calculated based on the coefficient of



horizontal subgrade reaction (k_n) of the soil as discussed above. The SLS resistance should be taken as that corresponding to a horizontal deflection of 10 mm at the underside of the pile cap for units supporting abutments (*CHBDC Commentary C6.8.7.1*).

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

Group action for lateral loading should also be considered when the pile spacing in the direction of loading is less than eight (8) pile diameters. Group action can be evaluated by reducing the coefficient of lateral subgrade reaction in the direction of loading by a reduction factor, R (NAVFAC DM 7.02), as follows:

Pile Spacing in Direction of Loading (d = pile diameter)	Subgrade Reaction Reduction Factor, R
8d	1.00
6d	0.70
4d	0.40
3d	0.25

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

6.4.5 Frost Protection

All pile caps should be provided with a minimum of 1.9 m of soil cover for frost protection as per OPSD 3090.01 *Foundation Frost Penetration Depths for Southern Ontario*.

6.5 Caissons

Consideration could be given to the use of caissons for support of the abutments (both one-span and two-span options) and the centre pier (two-span option). However, there are several factors at this site that likely will require specific construction techniques and/or increase the potential arising during caisson construction, such as:

- The difficulty associated with socketing large diameter caissons into the predominantly very strong to extremely strong bedrock at the centre pier and north abutment (one-span option) where the bedrock surface is relatively steeply sloping;
- The need for temporary or permanent steel liners to control groundwater and support through the overburden;
- The requirement to maintain a balanced head to reduce the chance of base heave;
- Requirements for tremie concrete placement;
- The presence of artesian groundwater conditions which could increase the chance of base heave inside the liner during construction and increase the difficulty associated with drilling and properly sealing a large diameter pile; and,



- The requirement for bedrock drilling for the entire length of the caisson at the north abutment (two-span option).

Given these factors, the use of caissons is not recommended at this site.

6.6 Site Coefficient

In accordance with Section 4.4.6 of the *CHBDC*, the soils at the proposed bridge structure are categorized as Soil Profile Type IV and as such, the Site Coefficient, *S*, is 2.0.

6.7 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. Seismic (earthquake) loading must also be taken into account in design.

The following recommendations are made concerning the design of walls for 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 Special Provision 110S13 Aggregates Granular 'A' or Granular 'B' Type II, but with less than 5 per cent passing the No. 200 sieve, should be used as backfill behind the walls. Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Compaction (including type of equipment, target densities, etc.) should be carried out in accordance with Special Provision 105S21 *Compacting*. Other aspects of the granular backfill requirements with respect to sub-drains and frost taper should be in accordance with OPSD 3101.150 *Walls, Abutment, Backfill, Minimum Granular Requirement*.
- 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 Northern Region 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*.
- 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 1.9 m behind the back of the wall (in accordance with Figure C6.20(a) of the *Commentary* to the *CHBDC*). For unrestrained walls, fill should be placed within the wedge shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical (1.5H:1V) extending up and back from the rear face of the footing (in accordance with Figure C6.20(b) of the *Commentary* to the *CHBDC*). The pressures are based on the proposed embankment fill material and the following parameters (unfactored) may be used:



Fill Type	Unit Weight, γ (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

- Where lightweight fill (EPS) is installed behind the abutment wall, the pressure acting over the depth of the EPS may be calculated as follows:

Fill Type	Unit Weight, γ (kN/m^3)	Coefficients of Static Lateral Earth Pressure	
		At-Rest, K_o	Active, K_a
Lightweight Fill (EPS)	0.5	0.11	0.11

If the wall support and superstructure allow lateral yielding of the stem, active earth pressures may be used in the foundation design of the structure. If the wall support and superstructure does not allow lateral yielding, at-rest earth pressures should be assumed for foundation 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*.

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

- Seismic loading will result in increased lateral earth pressures acting on the abutment stem and retaining walls. The walls should be designed to withstand the combined lateral loading for the appropriate static pressure conditions given above, plus the earthquake-induced dynamic earth pressure. According to the National Building Code of Canada (1995) seismic hazard values (as referenced in the *CHBDC* and its *Commentary*), the site specific peak horizontal ground acceleration for Parry Sound and Sudbury area is 0.054 (for a probability of exceedance of 10 per cent in 50 years). For the thicknesses and type of overburden soils at the site, an amplification factor of 2.0 of the ground motion is recommended for design. As such the ground surface acceleration would be 0.108.
- Based on the above, according to Table C4.2 of the *Commentary* to the *CHBDC*, this site would be located in Seismic Performance Zone 2 and the corresponding Zonal Acceleration Ratio, A , would be 0.10. The seismic lateral earth pressure coefficient given below has been derived based on a design zonal acceleration ratio of 0.10.
- In accordance with Sections 4.6.4 and C.4.6.4 of the *CHBDC* and its *Commentary*, for structures which allow lateral yielding, the horizontal seismic coefficient, k_h , used in the calculation of the seismic active pressure coefficient, is taken as 0.5 times the zonal acceleration ratio (i.e. $k_h = 0.05$). For structures that do not allow lateral yielding, k_h is taken as 1.5 times the zonal acceleration ratio (i.e. $k_h = 0.15$). The seismic active earth pressure coefficient is also dependent on the vertical component of the earthquake acceleration, k_v . Three discrete values of vertical acceleration are typically selected for analysis, corresponding to $k_v = +2/3 k_h$, $k_v = 0$, and $k_v = -2/3 k_h$.
- The seismic active pressure coefficients (K_{AE}) given below for unrestrained walls (in accordance with Figure C6.20(b) of the *Commentary* to the *CHBDC*) may be used in design. These coefficients reflect the maximum K_{AE} obtained using the k_h and three values of k_v as described above. It should be noted that these



seismic earth pressure coefficients assume that the back of the wall is vertical and the ground surface behind the wall is level.

Seismic Active Pressure Coefficients, K_{AE}

Wall Type	Fill Type	
	Granular 'A'	Granular 'B' Type II
Yielding Wall	0.28	0.28
Non-yielding Wall	0.36	0.36

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

Seismic Active Pressure Coefficients, K_{AE}

Wall Type	Fill Type
	Lightweight Fill (EPS)
Yielding Wall	0.07
Non-yielding Wall	0.10

- The above K_{AE} values for yielding walls are applicable provided that the wall can move up to $250 \cdot A$ (mm), where A is the design zonal acceleration ratio of 0.10. This corresponds to displacements of up to 25 mm at this site.
- The earthquake-induced dynamic pressure distribution, which is to be added to the static earth pressure distribution, is a linear distribution with maximum pressure at the top of the wall and minimum pressure at its toe (i.e. an inverted triangular pressure distribution). The total pressure distribution (static plus seismic) may be determined as follows:

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

where:

K	=	is either the static active earth pressure coefficient (K_a) or the static at-rest earth pressure coefficient (K_o);
K_{AE}	=	is the seismic active earth pressure coefficient;
γ	=	is the unit weight of fill materials (kN/m^3);
z	=	is the depth below the top of the wall (m); and,
H	=	is the total height of wall (m).

6.8 Approach Embankment Design

The construction of the Still River SBL bridge will require placement of up to about 8.0 m of fill within the limits of the south approach embankment and up to about 9.5 m or 2.5 m of fill within the limits of the north approach embankment for the one-span option or two-span option, respectively. Based on the foundation investigation results at this site, the south approach embankment will be founded on a near surface deposit of soft to firm clayey silt to silty clay underlain by alternating deposits of very loose to dense sand to silt and soft to very stiff silty clay to clay. The north approach embankment associated with the one-span option will generally be founded on deposits of very loose to loose sand to silty sand underlain by soft silt clay to clay which in turn is



underlain by bedrock. The north approach embankment associated with the two-span option will essentially be founded on bedrock. All topsoil and organic matter should be stripped from below the approach embankment areas and all subgrade soils, if encountered, should be proof-rolled prior to fill placement.

The results of stability and settlement analyses for the new approach embankments are presented in the following sections.

The advantages, disadvantages, relative costs and risk/consequences associated with the stability/settlement foundation mitigation options for the south approach embankment and the north approach embankment (one-span option) are summarized in Tables 2A and 2B, respectively. The north approach embankment (two-span option) is expected to be founded essentially on bedrock after removal of any surficial organic deposits (i.e. topsoil and peat) as well as zones of cobbles and boulders and, as such, foundation mitigation options are not required.

6.8.1 Stability

Analyses were performed on the critical sections (i.e. the greatest new approach embankment height and/or the maximum thickness of soft, compressible cohesive deposits) of the proposed new approach embankments to assess the stability and liquefaction potential for the proposed heights and geometries. Critical sections include those through the front slope and side slopes of the new approaches.

6.8.1.1 Methodology

All limit equilibrium slope stability analyses were performed using the commercially available program Slide (Version 6.0), developed by Rocscience Inc., employing the Morgenstern-Price method of analysis. For all analyses, the Factors of Safety (FoS) of numerous potential failure surfaces were 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 in the design of embankment slopes under static conditions.

6.8.1.2 Parameter Selection

The simplified stratigraphy together with the foundation engineering parameters employed for the cohesive deposits (i.e. clayey silt / silty clay / clay) encountered at the south approach and north approach (one-span option only) are provided on Figures 1 and 2 and summarized for all soil layers in Table 3. The following is a summary of slope geometries (i.e. front slope and side slopes), unit weights and effective friction angles or cohesion values for various fill types, where applicable, modelled in the analyses.

Fill Type	Recommended Slope Profile	Unit Weight, γ (kN/m³)	Effective Friction Angle, ϕ' (°)	Cohesion, c' (kPa)
Rock Fill	1.25H:1V	19	40	-
Granular Fill	2H:1V	21	34	-
Lightweight (EPS) Fill	2H:1V	0.5	-	15



The overburden encountered at this site is composed of cohesive deposits (clayey silt, silty clay and/or clay) and granular soils (silt, sand, sandy silt/silty sand, and/or sand and gravel), except at the north approach of the two-span option where very little overburden was encountered. For granular soils, effective stress parameters were employed in the analyses assuming drained conditions. The effective stress parameters (effective friction angle and effective cohesion) for the granular soils were estimated from empirical correlations using the results of in situ SPTs, in conjunction with engineering judgement based on experience in similar soil conditions.

For cohesive deposits, total stress parameters were employed in the analyses assuming 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. For 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

6.8.1.3 Results of Analysis

The results of stability analysis for the south and north approach embankments are summarized below.

6.8.1.3.1 South Approach Embankment

The stability analysis for the up to about 8.0 m high rock fill south approach embankment indicates that after completion of construction (including removal and replacement of topsoil), the embankment will have a FoS of between about 1.1 and 1.2 for deep-seated, global failure surfaces of the front slope and west slope, that would impact the operation of the highway (see Figures 3 and 4).

To achieve a FoS greater than or equal to 1.3 for the west slope, a 3 m high by 3.6 m wide stability berm would be required along the outside toe of the SBL south approach embankment, as shown on Figure 5. However, it should be noted that as a result of the proximity of the proposed south abutment to the south bank of Still River, there is insufficient space to construct stability berms large enough to achieve a FoS greater than or equal to 1.3 for the front slope, as shown on Figure 6. As such, an alternative stability mitigation option for the south approach embankment will need to be implemented as discussed in Section 6.8.3.



6.8.1.3.2 North Approach Embankment (One-Span Option)

The stability analysis for the up to about 9.5 m high rock fill north approach embankment for the one-span bridge option indicates that after completion of construction (including removal and replacement of topsoil), the embankment will have a FoS of approximately 0.8 and 0.9 for deep-seated, global failure surfaces of front slope and west slope, respectively, that would impact the operation of the highway (see Figures 7 and 8).

To achieve a FoS greater than or equal to 1.3 for the west slope, a two-tiered stability berm, consisting of a 2.0 m high by 10.5 m wide bottom tier and a 3.0 m high by 8.0 m wide upper tier, would be required along the outside toe of the SBL north approach embankment, as shown on Figure 9. However, it should be noted that as a result of the proximity of the north abutment to the north bank of Still River, there is insufficient space to construct stability berms large enough to achieve a FoS greater than or equal to 1.3 for the front slope, as shown on Figure 10. As such, an alternative stability mitigation option for the north approach embankment will need to be implemented as discussed in Section 6.8.3.

6.8.1.3.3 North Approach Embankment (Two-Span Option)

The stability analysis for the up to about 3.5 m high rock fill north approach embankment for the two-span bridge option indicates that after the completion of construction (including removal and replacement of organic deposits), the embankment will have a FoS of 1.3 or greater for deep-seated, global failure surfaces that would impact the operation of the highway. As such, no stability mitigation options are required at the north approach embankment.

6.8.2 Settlement

Large settlements of the south approach embankment and one-span north approach embankment are expected as a result of the loading from the new fills on the compressible foundation soils at this site. In addition, depending on the type of fill materials employed in the construction, settlements may also occur due to compression of the new embankment fill itself.

6.8.2.1 Methodology

To estimate the magnitude of the expected settlements, analyses were carried out on the critical sections of the proposed approach embankments using both the commercially available program Settle^{3D} (Version 2.0), developed by Rocscience Inc., and hand/spreadsheet calculations.

For the settlement analyses, the critical sections at each approach area were assessed considering the location of the following:

- The greatest new embankment height; and/or,
- The thickest cohesive deposit.



The sources of settlement are considered to include:

- Immediate settlement of the native granular soils;
- Primary time-dependent consolidation of the cohesive deposits (using Terzaghi's one-dimensional consolidation theory);
- Secondary time-dependent (creep) consolidation of the cohesive deposits (long-term); and,
- Self-weight compression of the embankment fill materials (long-term).

6.8.2.2 *Parameter Selection*

At the south approach and north approach(one-span option), the foundation soils are composed of loose to dense sand to silt and soft to very stiff clayey silt to clay deposits.

The simplified stratigraphy together with the associated deformation and time-rate consolidation parameters employed for the cohesive deposits (i.e. clayey silt / silty clay / clay) encountered at the south approach/abutment (one-span or two-span option) and north approach/abutment (one-span option) are provided on Figures 1 and 2 and summarized for all soil layers in Table 3.

The immediate compression of the non-cohesive deposits (i.e. silt, sandy silt to silty sand, sand, and sand and gravel) were modelled by estimating an elastic modulus of deformation based on the SPT 'N'-values and using correlations proposed by Bowles (1984) and Kulhawy and Mayne (1990). These estimated values were compared with the typical range of expected values for similar soil types, as outlined in *CHBDC* and adjusted, as appropriate.

The consolidation settlement of the cohesive deposits was assessed using the results of the laboratory consolidation tests and in situ field vane tests to estimate the deformation parameters for the cohesive deposits. It should be noted that, in addition to the one (1) consolidation test that was carried out for the SBL alignment, the analysis also incorporates the results of two (2) consolidation tests carried out for the NBL alignment and three (3) consolidation test carried out in Swamp 204 (between STA 11+700 and STA 11+825) located immediately to the south of the south abutment (refer to Golder's Foundation Investigation and Design Report, Swamp Crossings and High Fill Areas – Contract 2, 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 5404-05-00; WP 5404-05-01, Geocres No. 41H-115 dated July 2012).

In addition, the results of the laboratory index testing were also employed to further assess deformation parameters (i.e. compression and recompression indices) using empirical correlations proposed in literature by Koppula (1986), Terzaghi and Peck (1967), Kulhawy and Mayne (1990) and Azzouz et al. (1976). The correlation by Koppula (1986) relating the natural water content and liquid limit to the compression index was found to be the most consistent with the results of laboratory consolidation tests for the clayey soils at this site.

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



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

where:

$$\begin{aligned}\sigma'_p &= \text{preconsolidation stress (kPa); and,} \\ s_{u(mob)} &= \mu s_{u(FV)} \text{ (after Bjerrum, 1973) where } s_{u(mob)} = \text{average mobilized undrained shear strength (kPa)} \\ \mu &= \text{Bjerrum's correction factor based on Plasticity Index} \\ s_{u(FV)} &= \text{undrained shear strength from field vane test (kPa)}\end{aligned}$$

The coefficient of consolidation, c_v (cm^2/s), required in the time-rate settlement analysis was established using the results of the laboratory consolidation tests and also estimated from the U.S. Navy (1986) correlation with liquid limit assuming normally consolidated soils.

In addition to primary consolidation within the cohesive deposits (i.e. clayey silts to clays), secondary compression may also occur. Secondary compression is referred to as creep settlement and occurs over a long period of time, after full dissipation of excess pore pressure under a constant stress. The following relationships have been employed for estimating the magnitude of creep settlement over the life of the embankment following the completion of primary settlement at each location.

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

where:

$$\begin{aligned}S_c &= \text{secondary consolidation (creep) settlement (mm)} \\ C_{\alpha\epsilon} &= \text{modified secondary compression index as estimated from laboratory consolidation tests} \\ H &= \text{initial thickness of compressible clay deposit (mm)} \\ t &= \text{post-construction period of interest (20 years)} \\ t_{EOP} &= \text{time to reach end of primary consolidation (years)}\end{aligned}$$

6.8.2.3 Settlement of Approach Embankment Fill

Where rock fill is used for the construction of the proposed approach 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); and,
- 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 Special Provision 206S03, *Rock Excavation, 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 (6) 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 (1) year following the completion of construction over the life of the embankment.

6.8.2.4 Settlement Performance Requirements

The settlement performance criterion for design of approach embankments is in accordance with Section 1.2 of MTO’s Embankment Settlement Criteria for Design, dated March 2010. In general, new approach embankments are to be designed as follows:

- Total settlement is to be less than 25 mm within 20 m of a transition point over a 20-year period following completion of construction for a “Freeway”.



6.8.2.5 Results of Analysis

The results of settlement analyses for the south and north approach embankments are summarized below.

6.8.2.5.1 South Approach Embankment

Based on the results of the settlement analysis (with the topsoil removed and replaced), the settlement of the foundation soils under the loading imposed by a 8.0 m high rock fill embankment is estimated to be about 1,020 mm. The estimated total settlement is comprised of about 280 mm of immediate settlement due to compression of the non-cohesive deposits and about 740 mm of primary consolidation of the cohesive deposits.

Based on an average coefficient of consolidation (c_v) of about $1.8 \times 10^{-3} \text{ cm}^2/\text{s}$ and $3.2 \times 10^{-3} \text{ cm}^2/\text{s}$ estimated for the various cohesive deposits (ranging in thickness from about 2.0 m to 25.8 m), and assuming two-way drainage for these cohesive deposits, it is estimated that about 90 per cent of the primary consolidation settlement will be completed in about 3,470 days (or about 9.5 years). A longer duration would be required to meet the settlement performance criteria.

The magnitude of total secondary consolidation (creep) settlement for the various cohesive deposits is estimated to be about 255 mm per log-cycle of time for this area corresponding to about 200 mm over a 20-year period following completion of construction.

A plot illustrating the rate of total consolidation settlement of the cohesive deposit over a 20-year period following the construction of the approach embankment is shown on Figure 11.

In addition, the total settlement of the rock fill embankment (based on a 8.0 m high embankment plus about 0.3 m of additional fill required after removal of organic deposits) is estimated to be about 70 mm, with about 55 mm expected to occur within six (6) months of construction of the embankment, 5 mm occurring during the next six (6) months and about 10 mm expected to occur over the remaining design life of the approach embankment.

As such, a settlement mitigation option for the south approach embankment will need to be implemented as discussed in Section 6.8.3.

6.8.2.5.2 North Approach Embankment (One-Span Option)

Based on the results of the settlement analysis (with the topsoil removed and replaced), the settlement of the foundation soils under the loading imposed by a 9.5 m high rock fill embankment is estimated to be about 525 mm. The estimated total settlement is comprised of about 55 mm of immediate settlement due to compression of the non-cohesive deposits and about 470 mm of primary consolidation of the cohesive deposits.

Based on an average coefficient of consolidation (c_v) of about $2 \times 10^{-3} \text{ cm}^2/\text{s}$ estimated for the up to about 4.1 m thick cohesive deposits, and assuming two-way drainage for these cohesive deposits, it is estimated that about 90 per cent of the primary consolidation settlement will be completed in about 210 days (or about 7 months). A longer duration would be required to meet the settlement performance criteria.

The magnitude of total secondary consolidation (creep) settlement for the cohesive deposit is estimated to be about 55 mm per log-cycle of time for this area corresponding to about 85 mm over a 20-year period following completion of construction.



A plot illustrating the rate of total consolidation settlement of the cohesive deposit over a 20-year period following the construction of the approach embankment is shown on Figure 12.

In addition, the total settlement of the rock fill embankment (based on a 9.5 m high embankment plus about 0.3 m of additional fill required after removal of organic deposits) is estimated to be about 85 mm, with about 70 mm expected to occur within six (6) months of construction of the embankment, 5 mm occurring during the next six (6) months and about 10 mm expected to occur over the remaining design life of the approach embankment.

As such, a settlement mitigation option for the north approach embankment for the one-span bridge option would need to be implemented as discussed in Section 6.8.3.

6.8.2.5.3 North Approach Embankment (Two-Span Option)

The north approach embankment (two-span option) is expected to be founded essentially on bedrock after removal of any surficial organic deposits (i.e. topsoil and peat) as well as zones of cobbles and boulders. As such, the total embankment settlement will be comprised entirely of rock fill settlement.

The total settlement of the 3.5 m high rock fill embankment is estimated to be about 25 mm, with about 20 mm expected to occur within six (6) months of construction of the embankment and about 5 mm expected to occur over the remaining design life of the approach embankment. As a result, no foundation mitigation options are required for the north approach embankment.

6.8.3 Stability and Settlement Mitigation Measures

As discussed in Section 6.8.2.5, foundation mitigation measures are required at the south approach embankment and would need to be implemented at the north approach embankment for the one-span bridge option in order to mitigate stability and settlement issues. The following sections provided a discussion of the preferred foundation mitigation measures.

6.8.3.1 South Approach Embankment

Given that the south approach embankment will be subject to stability and settlement issues similar to those in the swamp crossing/high fill area immediately to the south (i.e. Swamp 204), it is recommended that the south approach embankment adopt a similar stability and settlement mitigation scheme (i.e. partial preload and EPS fill) as a continuation of the embankment over Swamp 204 (Golder, 2012).

Given the presence of the up to 32.5 m thick cohesive deposit (the bottom of which is up to about 42.5 m below ground surface) and the associated magnitude of primary and secondary consolidation settlement (combined total of about 940 mm) of the foundation soils under a 8.0 m high approach embankment, extensive stability and settlement analysis has been carried out for the embankment in Swamp 204 considering combinations of stability berms, staged construction, surcharging, sub-excavation of near surface cohesive deposits, wick drains and lightweight fill (i.e. expanded polystyrene (EPS)).

Although partial sub-excavation of the near surface portion of the clayey silt to silty clay deposit is considered practical, the FoS associated with the stability of the front slope would still be less than 1.3 due to the presence



of the greater (thicker) portion of the cohesive deposit at depth. Full sub-excavation of the cohesive deposit would not be feasible given the depth to the bottom of the cohesive deposit (about 42.5 m). Preloading (8 m high) and surcharging (additional 2 m) are not feasible due to insufficient space between the toe of the front slope of the approach embankment and the south bank of Still River to construct a stability berm large enough to achieve a FoS of 1.3 along the front slope. In addition, the estimated preload and surcharge periods to achieve the long-term post-construction settlement criteria (i.e. about 78.4 years and 1.5 years, respectively) are not practical. Wick drains are not recommended given the limited height of the preload/surcharge embankment (4.5 m to maintain a FoS of 1.3) that can be constructed for the purpose of expediting consolidation settlements, the estimated high rate of secondary consolidation (creep) settlement for the various cohesive deposits, the need for and uncertainties associated with deep wick drain installations as well as the requirement for some amount of EPS despite the use of wick drains.

To satisfy the requirements for both stability and the long-term post-construction settlement criterion for the approach embankment, the preferred mitigation option is to construct a partial preload granular embankment which is to be left in place for a specific period of time and upon completion of the preload period, the final upper section of the embankment is to be reconstructed with EPS (consistent with the foundation mitigation recommendations for Swamp 204).

For the final up to 8.0 m high approach embankment constructed to 2H:1V side slopes, consisting of a 1 m thick granular base/levelling pad, an up to about 6.0 m thick core of EPS and a 1 m thick granular protective cover/pavement structure, the stability analysis indicates that the approach embankment will have a FoS of 1.3 or greater for deep-seated, global failure surfaces, of the front and side slopes, as shown on Figures 13 and 14, respectively.

Based on the results of the settlement analysis, the settlement of the foundation soils for this scenario is estimated to be about 200 mm. The estimated total settlement is comprised of about 80 mm of immediate settlement due to compression of the non-cohesive deposits, and about 120 mm of primary consolidation, settlement for the cohesive deposits.

In order to satisfy the long-term post-construction settlement performance criterion of 25 mm of settlement over a 20-year period (post-construction), it is recommended that a 4.5 m high Granular 'B' Type II preload embankment be initially constructed and left in place for a preload period of 110 days, which is consistent with the height of the partial preload embankment and preload period associated with the Still River Bridge NBL south approach embankment.

As a result of the presence of the thick deposits of compressible cohesive soils and to facilitate the assessment for the end of the preload period, instrumentation and monitoring during and after construction will be required. Monitoring instrumentation should consist of settlement plates (SPs), vibrating wire piezometers (VWPs) and standpipe piezometers (SPPs).

Upon the completion of the preload period, the partial preload granular embankment is to be reduced to a height of 0.7 m above ground surface and the final EPS embankment constructed (including a 300 mm thick Special Provision 110S13 Granular 'A' levelling pad beneath the EPS and 1 m granular protective cover/pavement structure over the EPS core).

A plot illustrating the rate of total consolidation settlement of the cohesive deposit during the preload period (i.e. within the first 110 days) and over a 20-year period following the preload period and construction of the EPS embankment is shown on Figure 15.



6.8.3.2 North Approach Embankment (One-Span Option)

Considering that there is insufficient space between the toe of the front slope of the approach embankment and the north bank of Still River to construct a stability berm large enough to achieve a FoS of 1.3, full sub-excavation of the firm clay deposit is recommended as the preferred stability and settlement mitigation option at this location.

Full Sub-Excavation

Full sub-excavation of the cohesive deposit, as defined by a line drawn at a 1H:1V slope from the toe of the front slope of the embankment down to the bedrock surface in the northerly direction away from Still River (up to about 5.5 m deep) and replacement with rockfill as part of the overlying embankment, will achieve a FoS greater than or equal to 1.3 for the front slope stability, as shown on Figure 16. However, given that the limit of the excavation will be in close proximity to the north bank of Still River (about 5 m away), temporary protection systems will be required to support the excavation and to protect the river. The full sub-excavation and replacement would also have to extend to the limits of the side slope embankment toes to satisfy side slope stability requirements, as shown on Figure 17. Further, if pile foundations are adopted for the north abutment, granular fill (i.e. such as Special Provision 110S13 Granular 'B' Type II but not rock fill) should be used as replacement fill for the sub-excavation in the abutment area to allow for the driving of the H-piles.

In order to satisfy the long-term post-construction settlement performance criterion of 25 mm of settlement over a 20-year period and reduce the potential for lateral soil movements on the abutment piles (associated with the irregular sub-excavation and replacement zone below the front slope), it is recommended that the rock fill embankment be constructed and left in place for a preload period of 250 days to allow for the settlement of rock fill and lateral squeezing of any remaining or adjacent cohesive deposit to occur, prior to the piling and construction of the north abutment as well as the paving of the final embankment. The remaining portion of the cohesive deposit along the transition zone of the sub-excavated area below the front slope will experience some post-construction settlement; however, it is not expected to affect the performance of the abutment or the travelled portion of the highway within the north approach embankment area.

Preloading and Surcharging

Preloading and surcharging without full sub-excavation is not feasible due to the insufficient space between the toe of the approach embankment and the north bank of Still River to construct a stability berm large enough to achieve a FoS of 1.3 along the front slope. In addition, the estimated preload and surcharge periods to reach the long-term post-construction settlement (i.e. about 4.3 years and 1.4 years, respectively) are not practical.

Wick Drains and Staged Construction

Preliminary analysis suggests that the embankment cannot be constructed at the proposed north approach area using wick drains and staged construction. As such, additional stability analysis and detailed wick drain design would be required to confirm the feasibility/practicality of staged construction or if additional stability berms would be required. In addition, based on the laboratory consolidation test results, the rate of secondary consolidation settlement of the cohesive foundation soils at this location is estimated to be high. Consequently, longer surcharge period and possibly expanded polystyrene (EPS) fill top-ups would be required to satisfy the long-term



post-construction settlement criterion. Further, the long-term creep settlements could result in additional dragloads imposed on the driven steel H-pile at the one-span north abutment. Given the risks, uncertainties and complexity associated with the staged construction and wick drain design at this location, and the relatively thin nature of the clay deposit at this location, wick drains are not recommended for the north approach embankment.

Rammed Aggregate Piers

Rammed Aggregate Piers (RAPs) have also been considered as a potential stability and settlement mitigation measure. The cohesive deposit below the proposed north approach embankment extends to a depth of up to about 5.5 m below existing ground surface, and as such, RAPs could be considered; however, given that the clay stratum is soft, temporary liners would likely be required for construction and a relatively high area replacement ratio may be required to satisfy stability and achieve an adequate FoS. In addition, given that the foundation area for the north approach embankment is relatively small and RAPs are not feasible along the adjacent NBL embankment due to the presence of a much thicker clay deposit (i.e. up to about 15.5 m thick), RAPs are not considered a practical alternative for construction at this relatively small area of the site.

Lighweight (EPS) Fill

An approach embankment constructed with expanded polystyrene (EPS) was also considered as a potential stability and settlement mitigation option. For the up to 9.5 m high approach embankment constructed to 2H:1V side slopes, consisting of a 1 m granular base, an up to about 7.5 m thick EPS core and a 1 m granular protective cover/pavement structure, the stability analysis indicates that the approach embankment will have a FoS of 1.3 or greater for deep-seated, global failure surfaces of the side slopes, as shown on Figure 18. As discussed in Section 6.4.1.3, in order to eliminate additional dragloads and potential eccentric loads on the pile for the one-span north abutment, a Retained Soil System (RSS) wall could be constructed in front of the abutment in place of a 2H:1V front slope. The stability analysis of the north abutment towards Still River (including EPS behind an RSS wall) indicates a FoS of 1.3 or greater for deep-seated, global failure surfaces, as shown on Figure 19.

Based on the settlement analysis, the settlement of the foundation soils for the EPS option is estimated to be about 80 mm. The estimated total settlement is comprised of about 15 mm of immediate settlement due to compression of the non-cohesive deposits and about 65 mm of primary consolidation for the cohesive deposits. In order to satisfy the long-term post-construction settlement performance criterion of 25 mm of settlement over a 20-year period, a 2 m high Special Provision 110S13 Granular 'B' Type II partial preload embankment should be initially constructed and left in place for a preload period of 45 days. To facilitate the assessment for the end of the preload period, instrumentation and monitoring during and after construction will be required. Monitoring instrumentation should consist of settlement plates (SPs), vibrating wire piezometers (VWPs) and standpipe piezometers (SPPs). Upon completion of the preload period, the granular preload embankment would be reduced to a height of 0.7 m above ground surface and the final EPS embankments constructed (including a 300 mm thick Special Provision 110S13 Granular 'A' levelling pad beneath the EPS and 1 m thick granular protective cover/pavement structure over the EPS core). Given the high cost associated with the EPS option, the lightweight fill with partial preloading option would likely not make this alternative cost effective compared to the full sub-excavation option.



6.8.3.3 North Approach Embankment (Two-Span Option)

As discussed in Sections 6.8.1.3.3 and 6.8.2.5.3, there are no stability issues associated with the north approach embankment for the two-span bridge option and the settlements are expected to be within the settlement performance criterion, as such, no foundation mitigation options are required at this location.

6.8.4 Liquefaction Potential Below Embankments

The liquefaction potential of the soils below the approach embankments under seismic loading has been considered using the empirical method outlined in Section C.4.6.2 of the *CHBDC Commentary*, which correlates the cyclic resistance ratio (CRR) of the soils with their normalized penetration resistance and fines content. Based on this assessment and with a site specific peak horizontal acceleration ratio of 0.10g (as discussed in Section 6.7), the subsoils are not considered liquefiable for an earthquake of magnitude 7.0. Localized failures at the embankment toe, resulting in steepening of the embankment side slopes, could occur, however, the probability of this occurrence is considered to be low.

6.9 Preferred Structure Alternative

As noted in Section 6.1, a one-span or a two-span bridge configuration was considered for the Still River SBL bridge during the initial design stage of the project.

The main advantage of the one-span alternative is a shorter length of the bridge structure and consequently a lower cost of the superstructure. However, a major drawback of the one-span alternative pertains to the high level of complexity of design associated with the stability and settlement mitigation option required at the north abutment and higher approach embankment as summarized in Section 6.8.3.2.

In comparison, the cost of the two-span configuration is higher due to the greater length of the bridge structure and the requirement for an additional foundation element. However, this configuration offers a standard foundation design at the north abutment and approach embankment due to the very shallow depth to bedrock as described in Section 6.3.1.4. The foundation design associated with the additional foundation element (i.e. centre pier) is also relatively standard (i.e. steel H-piles driven to bedrock) and is summarized in Section 6.4.1.2.

Overall, upon comparison of the two structure alternatives from a foundations perspective, the one-span configuration adds a high degree of risk and uncertainty to the design of the one-span bridge structure as a result of the presence and thickness of the weak/soft foundation soils, proximity to the Still River, as well as mitigation measures required to achieve the long-term post-construction settlement criteria at the north abutment and higher approach embankment. Furthermore, the possibility of lateral soil movement which can have an adverse impact on the performance of the north abutment piles also increases the risk. Finally, the cost associated with the foundation mitigation option at the north abutment and higher approach embankment (one-span option) may also be significant compared to the overall cost for construction of the bridge. As such, the one-span bridge configuration is not the preferred alternative from a foundation perspective.

The advantages, disadvantages, relative costs and risk/consequences associated with the one-span and two-span structure alternatives are summarized in Table 4.



6.10 Subgrade Preparation and Embankment Construction

The existing native subsoils are considered to be appropriate sub-base for the proposed approach embankments; however, prior to the placement of any fill, any surface or near surface layers of topsoil/organic deposits and any softened soil should be stripped from the plan limits of the proposed works and the subgrade should be proof-rolled.

6.10.1 Removal of Organics

Based on the information from the borings obtained during the field investigation, organic deposits (i.e. topsoil and peat) of up to 0.5 m thick can be expected in some areas of the new approach embankments. These organic layers should be stripped from the plan limits of the approach embankment footprints prior to fill placement.

6.10.2 Approach Embankment Fill Placement

Placement of rock fill and granular fill above the water table for construction of new embankments should be carried out in accordance with the requirements as outlined in Special Provision 206S03 *Rock Excavation, Grading*. The rock fill should not be dumped into final position, but should be deposited on and pushed forward over the end of the layer being constructed. Voids and bridging should be minimized by blading, dozing and 'chinking' the rock to form a dense, compacted mass. Side slopes for rock fill embankments should be no steeper than 1.25H:1V. Where expanded polystyrene (EPS) levelling pads are required, granular fill should be placed in lifts with loose thickness not exceeding 300 mm and compacted to at least 95 per cent of the standard Proctor maximum dry density. Side slopes for granular fill should be no steeper than 2H:1V.

The EPS fill should be installed in accordance with the Non-Standard Special Provision for Expanded Polystyrene Embankment presented in Appendix C. It is recommended that a levelling pad comprised of at least 300 mm of Special Provision 110S13 Granular 'A' be placed prior to the installation of the EPS. The EPS should be covered with a 6 mil thick polyethylene sheet and overlain with a minimum 125 mm thick reinforced concrete slab constructed on top of the EPS and a minimum 1 m protective cover/pavement structure over the slab. The EPS on the side slopes of the embankment should be covered with a 1 m thick layer of conventional soil/granular material.

6.11 Design and Construction Considerations

6.11.1 Excavation

At the location of the proposed two-span centre pier, the excavation will extend to depths of up to about 2 m below the existing ground surface and will be made through very loose to loose non-cohesive deposits (sand to silty sand to silt), which are considered Type 4 soils according to the Occupation Health and Safety Act and Regulations for Construction Projects (OHSA). Therefore, the excavation through the overburden should be carried out with side slopes no steeper than 3 horizontal to 1 vertical (3H:1V).

At the location of the proposed one-span north abutment and approach embankment, the excavation would extend to a depth of up to about 6.0 m below the existing ground surface and would generally be made through very loose to loose non-cohesive deposits (sand to silty sand) and soft cohesive deposits (silty clay to clay),



which are considered Type 4 soils, according to OSHA. Therefore, the excavation through the non-cohesive and cohesive deposit would have to be carried with side slopes no steeper than 3 horizontal to 1 vertical (3H:1V).

At the location of the proposed two-span north abutment, the excavation will extend to a depth of up to about 2.5 m below the existing ground surface and will generally be made through zones of cobbles/boulders and/or through gneiss to granitic gneiss and schist bedrock (for footing construction). The excavation in the cobbles and boulders should be maintained with side slopes no steeper than 1H:1V, while for cuts through the bedrock the overall slope of the cut face may be formed vertically or near vertically (i.e. about 0.25H:1V). The use of carefully controlled excavation techniques will be required to ensure a neat excavation line and minimize face instabilities.

All excavations must be carried out in accordance with the latest edition of the OHSA.

6.11.2 Temporary Protection System

Given the proximity of the sub-excavation limits for the proposed one-span north abutment and approach embankment to the north bank of Still River, a temporary support/protection system would be required for this option to protect the river. The temporary excavation protection system would have to be designed and constructed to Performance Level 3 in accordance with Special Provision 539S02 *Protection System*.

6.11.3 Blasting

Good blasting practices will be critical to maintaining the excavation lines and preserving the integrity of the rock mass in the area of the structure foundations. Abutment footings should be located away from the rock face at least a distance as defined by an imaginary line projected at 0.5 horizontal to 1 vertical (0.5H:1V) from the toe of the rock cut or a minimum of 2 m. If the layout does not allow for this setback zone, special measures are required to control and pre-support the rock face.

The use of explosives should follow the specifications outlined in OPSS 120 *Use of Explosives*. It is recommended that control of all blasting operation be carried out in accordance with Special Provision 299F06 *Rock Excavation for Controlled Blasting*.

It is recommended that all new rock cut faces in the area of the proposed structure foundations be inspected by a Quality Verification Engineer (QVE) soon after blasting to assess if the blasting operations have affected the integrity of the rock mass that will ultimately be supporting the new spread footings. All loose, unstable rock should be removed from the cut faces in accordance with Special Provision 299F03 *Rock Excavation (Machine Scaling)* before access to the toe area of the slope is permitted. After rock blasting, where potentially unstable rock blocks or wedges cannot be removed safely or where the removal of such blocks/wedges could undermine the rock mass above, rock bolting may be required. As such, if rock bolting be necessary, it should be carried out in accordance with Special Provision 299S07 *Rock Bolting*.



6.11.4 Control of Groundwater and Surface Water

The groundwater level at this site is generally between 0 m (i.e. at the existing ground surface) and about 4 m below the existing ground surface, but an excess head of 2.3 m above ground surface (artesian condition) was also recorded during drilling in Borehole B201-01 at the proposed south abutment.

At the south abutment, the proposed level of the underside of the pile cap (about Elevation 182 m) is above ground surface (i.e. by approximately 1 m), and as such, dewatering will not be required.

At the centre pier (two-span option), the proposed level of the underside of the pile cap (about Elevation 177.5 m) is below ground surface (i.e. by approximately 2 m), and as such, dewatering will be required.

At the north abutment (one-span option), the pile cap is expected to be perched within the approach embankment (i.e. approximately 4.5 m above ground surface corresponding to Elevation 183 m), and as such, dewatering would not be required.

At the north abutment (two-span option), the proposed founding level (about Elevation 184 m) for the spread footing is approximately 0.9 m below the elevation of the groundwater table as measured during the foundation investigation in March 2011. As such, dewatering will be required to properly prepare the bedrock and construct the footing in the dry.

Given the relative density and grain size distribution for the non-cohesive soils at the centre pier (two-span option), and using the limits of dewatering proposed by Powers (1992), it is considered likely that pumping from within trenches/ditching with adequately sized and properly filtered pumps will be sufficient to control the groundwater inflow. At the north abutment (two-span option) it is considered likely that pumping from properly filtered pumps will be adequate to control groundwater inflow.

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

6.11.4.1 Control of Fines Migration

As a result of the artesian conditions encountered at the south abutment during the borehole investigation, a seepage control system/sand filter comprised of a concrete sand drainage blanket wrapped in a geotextile and containing collector pipes is recommended to control migration of fines that may be brought up along the pile due to water flow under artesian pressure, during and following the pile driving operation.

The drainage blanket should consist of a minimum 0.5 m thick layer of concrete fine aggregate meeting the gradation requirements of OPSS 1002, *Aggregates – Concrete*. The concrete fine aggregate should extend a minimum of 0.5 m horizontally beyond each of the piles. Appropriate drainage from under the pile cap should be provided for the granular blanket, such as by using a 100 mm perforated subdrain in accordance with OPSS 405, *Pipe Subdrains*, wrapped in a knitted sock geotextile and draining to an adjacent ditch. The geotextile surrounding the drainage blanket should consist of a non-woven, Class 1 geotextile with filtration opening size (FOS) of 75 µm to 115 µm in accordance with OPSS 1860, *Geotextiles*.



6.11.5 Obstructions

The native subsoils at the location of the proposed north abutment (two-span option) contain zones/nests of cobbles and boulders (as encountered by coring).

Conventional excavating equipment, where required, should be suitable for the excavation through the subsoils on site. However, the presence of boulders may interfere or slow the progress of stripping and excavation. It is recommended that a NSSP be included in the Contract Documents to warn the Contractor of these obstructions and to ensure that the Contractor is equipped to handle such obstructions (an example NSSP is included in Appendix C).

7.0 CLOSURE

This report was prepared by Messrs. Tomasz Zalucki, P.Eng. and Christopher Ng, P.Eng., and was reviewed by Mr. J. Paul Dittrich, Ph.D., P.Eng., a senior geotechnical engineer and Principal with Golder. Mr. Jorge M. A. Costa, P.Eng., Golder's Designated MTO Contact for this project and a Principal with Golder, conducted an independent quality control review of the report.



Report Signature Page

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[Http://capws/sites/0911116014highway69FourLaning/Contract 2/Reporting/Final/Still River SBL Bridge/09-1111-6014-2523 RPT 14Sep11 Highway 69 Still River SBL Bridge.docx](http://capws/sites/0911116014highway69FourLaning/Contract%20Reporting/Final/Still%20River%20SBL%20Bridge/09-1111-6014-2523%20RPT%2014Sep11%20Highway%2069%20Still%20River%20SBL%20Bridge.docx)



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ASTM International:

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
ASTM D7012	Standard Test Method for Compressive Strength and Elastic Moduli of Intact Rock Core Specimens under Varying States of Stress and Temperatures

Commercial Software:

LPILE Plus (Version 5.0) by Ensoft Inc.

Settle3D (Version 2.0) by Rocscience Inc.

Slide (Version 6.0) by Rocscience Inc.

Ministry of Transportation Ontario:

Northern Region Directive, Backfill to Structures Adjacent to Rock Embankment Approaches. November 2002.

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

Embankment Settlement Criteria for Design. March 2010.

Structural Manual, Ministry of Transportation, Provincial Highways Management Division, Highway Standards Branch, Bridge Office. April 2008.

Ministry of Transportation Ontario Special Provisions:

Special Provision 105S21 Amendment to OPSS 501 – Compacting

Special Provision 110S13 Amendment to OPSS 1010 – Material Specification for Aggregates –
Base, Subbase Select Subgrade and Backfill Material.



**FOUNDATION REPORT – STILL RIVER SBL BRIDGE STRUCTURE –
HIGHWAY 69 GWP 5404-08-00; WP 5140-08-01**

Special Provision 206S03 Amendment to OPSS 206 – Rock Excavation, Grading; Rock Embankment.

Special Provision 299F06 Rock Excavation for Controlled Blasting.

Special Provision 539S02 Amendment to OPSS 539 – Protection System.

Ontario Occupational Health and Safety Act:

Ontario Regulation 213 Construction Projects

Ontario Provincial Standard Drawings:

OPSD 3000.201 Foundation, Piles, Steel HP 310 Oslo Point.

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

Ontario Provincial Standard Specifications:

OPSS 120 General Specification for Use of Explosives.

OPSS 405 Construction Specification for Pipe Subdrains.

OPSS 903 Construction Specification for Deep Foundations.

OPSS 1002 Material Specification for Aggregates – Concrete.

OPSS 1860 Material Specification for Geotextiles.

Ontario Water Resources Act:

Ontario Regulation 903 Wells



TABLES



Table 1A: Evaluation of Foundation Alternatives – Still River SBL Bridge Structure – South Abutment Highway 69 Four-Laning

Foundation Option	Rank	Advantages	Disadvantages	Relative Costs	Risks / Consequences
Spread footing founded on native soils or perched on Granular 'A' pad	NR ¹	<ul style="list-style-type: none"> Relative ease of construction. 	<ul style="list-style-type: none"> Allows only for semi-integral abutment design. Geotechnical capacity on the thick weak native soils for spread footing design is very low; post-construction settlement likely to occur. 	<ul style="list-style-type: none"> Lower relative cost than piled foundation and caisson options. 	<ul style="list-style-type: none"> Not recommended due to very weak and compressible foundation soils.
Steel H-piles driven to the top of compact to dense sand deposit	1	<ul style="list-style-type: none"> Allows for integral abutment design. 	<ul style="list-style-type: none"> Long piles will be required to provide adequate geotechnical capacity for pile design. Large dragloads will have to be considered in the pile design. Partial preloading in the area of south abutment and approach embankment will be required to reduce the dragloads on piles. Pile load testing will be required to confirm the geotechnical capacities of the pile. 	<ul style="list-style-type: none"> Lower relative cost than caisson option. Higher relative cost than spread footing option. Higher cost associated with long piles. Additional cost associated with pile load testing. 	<ul style="list-style-type: none"> Difficulties maintaining alignment of very long H-piles while pile driving. The abutment design should be flexible enough to accommodate installation of extra piles, if required based on the pile load test results.



Table 1A: Evaluation of Foundation Alternatives – Still River SBL Bridge Structure – South Abutment
Highway 69 Four-Laning

Foundation Option	Rank	Advantages	Disadvantages	Relative Costs	Risks / Consequences
Caissons augered into compact to dense sand deposit	NR ¹	<ul style="list-style-type: none">■ Reduced number of deep foundation elements compared to steel H-piles.	<ul style="list-style-type: none">■ Allows only for semi-integral abutment design.■ Temporary or permanent steel liners would be required to control groundwater and provide support through overburden.■ Special measures, such as drilling mud / slurry required to balance groundwater pressures/basal heave and minimize loss of ground during construction.■ Concrete caissons would need to be placed using tremie methods below the groundwater table.	<ul style="list-style-type: none">■ Higher relative cost than spread footing and piled foundation options.■ Additional cost associated with the need for temporary or permanent steel liners and drilling slurry.	<ul style="list-style-type: none">■ Potential for unbalanced head in liners during installation resulting in base heave and possible loss of ground.■ Difficulties maintaining alignment of caissons due to the great depth of overburden.

Note: 1. NR – Not Recommended

Prepared By: TZ/CN

Reviewed By: JPD/JMAC



**Table 1B: Evaluation of Foundation Alternatives – Still River SBL Bridge Structure – Centre Pier (Two-Span Option)
Highway 69 Four-Laning**

Foundation Option	Rank	Advantages	Disadvantages	Relative Costs	Risks / Consequences
Spread footing founded on native soils or perched on Granular 'A' pad	NR ¹	<ul style="list-style-type: none"> Relative ease of construction. 	<ul style="list-style-type: none"> Allows only for semi-integral abutment design. Geotechnical capacity on the weak native soils for spread footing design is low; post-construction settlement likely to occur. Considerable excavation (6.2 m to 7.8 m deep) required if footing is founded on bedrock to achieve greater resistance. 	<ul style="list-style-type: none"> Lower relative cost than piled foundation and caisson options. 	<ul style="list-style-type: none"> Not recommended due to weak and compressible foundation soils, or greater depth of excavation to bedrock.
Steel H-piles driven to bedrock	1	<ul style="list-style-type: none"> Negligible post-construction settlement. 	<ul style="list-style-type: none"> Dewatering and concrete placement for the pile cap required within a dry excavation. Pile points will be required to facilitate proper seating of the piles on the steeply sloping bedrock. 	<ul style="list-style-type: none"> Lower relative cost than caisson option. Higher relative cost than spread footing option. Additional cost associated with the need for pile points. Additional cost required for dewatering for pile cap construction. 	<ul style="list-style-type: none"> Potential difficulties in seating the steel H-piles into the very strong to extremely strong and sloping bedrock even when pile points are used.
Caissons socketted into bedrock	2	<ul style="list-style-type: none"> Reduced number of deep foundation elements compared to steel H-piles. Negligible post-construction settlement. 	<ul style="list-style-type: none"> Allows only for semi-integral abutment design. Temporary steel liners would be required to control groundwater and provide support through overburden. Concrete inside caissons would need to be placed 	<ul style="list-style-type: none"> Higher relative cost than spread footing and driven pile foundation options. Additional cost associated with the need for temporary steel liners. 	<ul style="list-style-type: none"> Increased potential difficulties in achieving adequate seal and drilling a large diameter socket into very strong to extremely strong and sloping bedrock.



Table 1B: Evaluation of Foundation Alternatives – Still River SBL Bridge Structure – Centre Pier (Two-Span Option)
Highway 69 Four-Laning

Foundation Option	Rank	Advantages	Disadvantages	Relative Costs	Risks / Consequences
			using tremie methods below the groundwater table. <ul style="list-style-type: none">■ Potential for difficulties in socketting caissons into very strong to extremely strong and sloping bedrock.		

Note: 1. NR – Not Recommended

Prepared By: TZ/CN

Reviewed By: JPD/JMAC



**Table 1C: Evaluation of Foundation Alternatives – Still River SBL Bridge Structure – North Abutment (One-Span Option)
Highway 69 Four-Laning**

Foundation Option	Rank	Advantages	Disadvantages	Relative Costs	Risks / Consequences
Spread footing founded on native subsoils or perched on Granular 'A' pad	NR ¹	<ul style="list-style-type: none"> ■ Relative ease of construction. 	<ul style="list-style-type: none"> ■ Allows only for semi-integral abutment design. ■ Geotechnical capacity on the weak native soils for spread footing design is low; post-construction settlement likely to occur. ■ Differential settlement will need to be considered given the variability in the thickness of the cohesive deposit. ■ Variable excavation depth (4.1 m to 6.0 m) to be able to found footings on bedrock – requires mass concrete or rock removal to achieve a somewhat level grade. 	<ul style="list-style-type: none"> ■ Lower relative cost than piled foundation and caisson options. 	<ul style="list-style-type: none"> ■ Not recommended due to weak and compressible foundation soils or great/variable depth to excavation to bedrock.
Steel H-piles driven to bedrock	1	<ul style="list-style-type: none"> ■ Negligible post-construction settlement. 	<ul style="list-style-type: none"> ■ Integral abutment design may not be possible since some piles may not achieve minimum pile length for integral abutment design. ■ Difficult drilling into very strong bedrock for pile sockets to achieve required pile length. 	<ul style="list-style-type: none"> ■ Lower relative cost than caisson option. ■ Higher relative cost than spread footing option. 	<ul style="list-style-type: none"> ■ Likely no difficulty in seating the steel H-piles into the sloping and predominantly very strong to extremely strong bedrock. ■ Depending on which approach embankment stability/settlement mitigation is adopted, there is some potential for lateral soil movement at the north abutment due to approach embankment construction affecting the performance of the north abutment piles.



**Table 1C: Evaluation of Foundation Alternatives – Still River SBL Bridge Structure – North Abutment (One-Span Option)
Highway 69 Four-Laning**

Foundation Option	Rank	Advantages	Disadvantages	Relative Costs	Risks / Consequences
Caissons socketted into bedrock	NR ¹	<ul style="list-style-type: none">■ Reduced number of deep foundation elements compared to steel H-piles.■ Negligible post-construction settlement.	<ul style="list-style-type: none">■ Allows only for semi-integral abutment design.■ Potential for difficulties in socketting caissons into very strong to extremely strong bedrock.■ Temporary steel liners would be required to control groundwater and support through overburden.■ Concrete for caissons would need to be placed using tremie methods below the groundwater table.	<ul style="list-style-type: none">■ Higher relative cost than spread footing and driven pile foundation options.■ Additional cost associated with the need for temporary steel liners.	<ul style="list-style-type: none">■ Potential difficulties in achieving adequate seal and drilling a large diameter socket into very strong to extremely strong bedrock.

Note: 1. NR – Not Recommended

Prepared By: TZ/CN

Reviewed By: JPD/JMAC



**Table 1D: Evaluation of Foundation Alternatives – Still River SBL Bridge Structure – North Abutment (Two-Span Option)
Highway 69 Four-Laning**

Foundation Option	Rank	Advantages	Disadvantages	Relative Costs	Risks / Consequences
Spread footing founded on properly prepared gneiss or granitic gneiss bedrock or founded on mass concrete placed on properly prepared gneiss or granitic gneiss bedrock	1	<ul style="list-style-type: none"> Relative ease of construction. 	<ul style="list-style-type: none"> Allows only for semi-integral abutment design. Variable excavation depth (up to about 1.5 m deep, including removal of up to about 1.4 m of bedrock) and/or mass concrete required to found footings on bedrock or on mass concrete founded on properly prepared bedrock. 	<ul style="list-style-type: none"> Lower relative cost than piled foundation and caisson options. 	<ul style="list-style-type: none"> Rock excavation and/or mass concrete required to achieve a relatively level founding grade.
Steel H-piles set into trench bedrock.	NR ¹	<ul style="list-style-type: none"> Allows for full integral abutment design. 	<ul style="list-style-type: none"> Excavation required to remove cobbles and boulders near the ground surface. Significant amount of excavation / trenching into the very strong bedrock would be required to achieve minimum pile lengths. Bedrock trench would have to be backfilled with concrete. 	<ul style="list-style-type: none"> Higher relative cost than spread footing option. Additional cost associated with specialized drilling and excavating / trenching equipment. 	<ul style="list-style-type: none"> Specialized drilling equipment will be required to penetrate cobbles and boulders and excavate / trench very strong bedrock. Significant amount of excavation / trenching into the strong bedrock would be required to achieve minimum pile lengths.



**Table 1D: Evaluation of Foundation Alternatives – Still River SBL Bridge Structure – North Abutment (Two-Span Option)
Highway 69 Four-Laning**

Foundation Option	Rank	Advantages	Disadvantages	Relative Costs	Risks / Consequences
Caissons socketted into bedrock	NR ¹	<ul style="list-style-type: none">■ Reduced number of deep foundation elements compared to steel H-piles.	<ul style="list-style-type: none">■ Allows only for semi-integral abutment design.■ Potential for difficulties in socketting caissons into very strong bedrock.■ Temporary steel liners would be required to control groundwater and support through zones of cobbles and boulders.■ Concrete for caissons may need to be placed using tremie methods below the groundwater table.	<ul style="list-style-type: none">■ Higher relative cost than spread footing option.■ Additional cost associated with the need for temporary steel liners.	<ul style="list-style-type: none">■ Potential difficulties in achieving adequate seal and drilling a large diameter socket into very strong to bedrock.

Note: 1. NR – Not Recommended

Prepared By: TZ/CN

Reviewed By: JPD/JMAC



Table 2A: Evaluation of Stability/Settlement Mitigation Options – Still River SBL Bridge Structure – South Approach Embankment Highway 69 Four-Laning

Stability/Settlement Mitigation Option	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Full Sub-Excavation of Cohesive Deposits (up to 42.5 m deep)	Not feasible	<ul style="list-style-type: none"> ■ Reduced total settlement. ■ Toe berms are not required. 	<ul style="list-style-type: none"> ■ Generation of very large volume of excess excavation spoil. ■ Very large quantity of rock fill required. ■ Long delay in construction associated with up to 42.5 m deep sub-excavation and replacement with rock fill operation. ■ Specialized equipment and additional effort required for deep sub-excavation and replacement. ■ Substantial post-construction settlement of rock fill itself. ■ Sub-excavation in close proximity to Still River would require protection measures. ■ Will require additional right-of-way to accommodate deep sub-excavation. 	<ul style="list-style-type: none"> ■ Additional costs associated with sub-excavation (specialized drag-line equipment required), disposal and replacement of weak/soft, compressible deposits. ■ Additional cost for acquiring additional right-of-way for deep excavation. 	<ul style="list-style-type: none"> ■ Deep sub-excavation in close proximity to Still River may not be permitted. ■ Potential difficulties maintaining stability of excavation slopes. ■ Preloading would be required to reduce large post-construction settlement of rock fill. ■ Would be able to achieve and maintain stability of proposed embankments.
Preloading or Surcharging	NR ¹	<ul style="list-style-type: none"> ■ Not applicable. 	<ul style="list-style-type: none"> ■ Not applicable. 	<ul style="list-style-type: none"> ■ Not applicable. 	<ul style="list-style-type: none"> ■ Preloading and surcharging are not feasible as a result of insufficient space between the toe of the approach embankment and the north bank of Still River to construct a stability berm large enough to achieve an adequate FoS along the front slope.



Table 2A: Evaluation of Stability/Settlement Mitigation Options – Still River SBL Bridge Structure – South Approach Embankment Highway 69 Four-Laning

Stability/Settlement Mitigation Option	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Wick Drains	2	<ul style="list-style-type: none"> ■ Reduced time to complete primary consolidation. 	<ul style="list-style-type: none"> ■ Detail wick drain investigation and design would be required. ■ Additional time required for installation of wick drains. ■ First stage of filling limited to 4.5 m to maintain FoS of 1.3. ■ Toe and front slope berms may be required to reduce number of subsequent fill stages. ■ 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 (if possible) and to assess end of preload / surcharge period. ■ Increased handling of surcharge fill (Granular 'B' Type II) to remove surcharge. ■ Potential need for lightweight fill (i.e. EPS) as top-up. 	<ul style="list-style-type: none"> ■ Schedule impacts may increase overall project costs. ■ 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 if toe and front slope berms need to be constructed ■ Additional costs associated with construction and materials for surcharge and removal of excess surcharge embankment fill upon completion of surcharge period. ■ Additional costs associated with potential EPS top-up. 	<ul style="list-style-type: none"> ■ Potential for instability of embankment on weak/soft foundation soils, even if staged construction is employed. ■ Complex wick drain design (potentially in combination with lightweight fill). ■ 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.
Partial Preloading (4.5 m high for 110 days) followed by Lightweight Fill	1	<ul style="list-style-type: none"> ■ Improved stability. ■ Reduced total settlement of foundation soils. 	<ul style="list-style-type: none"> ■ Very high cost of EPS construction materials. ■ Have to remove partial preload 	<ul style="list-style-type: none"> ■ Relative cost of EPS fill is about an order of magnitude higher than fill required for 	<ul style="list-style-type: none"> ■ Will achieve stability of partial preload embankments and final EPS embankments



Table 2A: Evaluation of Stability/Settlement Mitigation Options – Still River SBL Bridge Structure – South Approach Embankment Highway 69 Four-Laning

Stability/Settlement Mitigation Option	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Construction (6.0 m of EPS)		<ul style="list-style-type: none">■ May shorten construction schedule.	<ul style="list-style-type: none">embankment in order to construct the EPS embankment.■ Instrumentation and monitoring program required to assess end of preload period.	<ul style="list-style-type: none">the other options.■ Estimated cost for EPS is about \$800,000 minus cost of rock fill to construct embankments in base case.	<ul style="list-style-type: none">on weak/soft foundation soils.■ Reduce potential of unexpected post-construction settlements.

Note: 1. NR – Not Recommended

Prepared By: TZ/CN

Reviewed By: JPD/JMAC



**Table 2B: Evaluation of Stability/Settlement Mitigation Options – Still River SBL Bridge Structure – North Approach Embankment (One-Span Option)
Highway 69 Four-Laning**

Stability/Settlement Mitigation Option	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Full Sub-Excavation as defined by a line drawn at a 1H:1V slope from the front slope toe of the embankment down to the bedrock surface in the northerly direction away from Still River (up to about 5.5 m deep)	1	<ul style="list-style-type: none"> ■ Reduced total settlement. ■ Toe berms are not required. 	<ul style="list-style-type: none"> ■ Generation of excess excavation spoil. ■ Relatively large quantity of replacement rock fill required. ■ Additional effort required for relatively deep sub-excavation and replacement. ■ Additional post-construction settlement of rock fill itself. ■ Sub-excavation in close proximity to Still River will require special protection measures. ■ If pile foundations are adopted at the north abutment, granular fill (i.e. not rock fill) must be used as replacement fill to allow for driving of H-piles. 	<ul style="list-style-type: none"> ■ Additional costs associated with sub-excavation, disposal and replacement of weak/soft, compressible deposits. ■ Additional cost associated with special protection measures. ■ Additional cost associated with granular fill to allow pile driving at the north abutment. 	<ul style="list-style-type: none"> ■ Relatively deep sub-excavation in close proximity to Still River may not be permitted. ■ Potential difficulties maintaining stability of excavation slopes. ■ Preloading would be required to reduce post-construction settlement of rock fill. ■ High potential of being able to achieve stability of proposed embankments. ■ Potential for lateral soil movement in area of abutment pile foundations.
Preloading or Surcharging	NR ¹	<ul style="list-style-type: none"> ■ Not applicable. 	<ul style="list-style-type: none"> ■ Not applicable. 	<ul style="list-style-type: none"> ■ Not applicable. 	<ul style="list-style-type: none"> ■ Preloading and surcharging are not feasible as a result of insufficient space between the toe of the approach embankment and the north bank of Still River to construct a stability berm large enough to achieve an adequate FoS along the front slope.
Wick Drains and Staged Construction	NR ¹	<ul style="list-style-type: none"> ■ Somewhat reduced time to complete primary consolidation. 	<ul style="list-style-type: none"> ■ Relative thinness of clayey stratum at this location (i.e. less than about 5 m thick) reduces the efficiency and viability of wick drains as an 	<ul style="list-style-type: none"> ■ Schedule impacts may increase overall project costs. ■ Additional costs associated 	<ul style="list-style-type: none"> ■ Potential for instability of embankment on weak/soft foundation soils, even if staged construction is possible and employed.



**Table 2B: Evaluation of Stability/Settlement Mitigation Options – Still River SBL Bridge Structure – North Approach Embankment
(One-Span Option)
Highway 69 Four-Laning**

Stability/Settlement Mitigation Option	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
			<p>alternative to accelerate settlements.</p> <ul style="list-style-type: none"> ■ Detail wick drain investigation and design will be required. ■ Additional time required for installation of wick drains. ■ Toe and front slope berms may be required. ■ 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 (if possible) and to assess end of surcharge period. ■ Increased handling of surcharge fill (Granular 'B') to remove surcharge. ■ Potential need for lightweight fill (i.e. EPS) as top-up. 	<p>with detail wick drain investigation and design.</p> <ul style="list-style-type: none"> ■ Additional cost for the installation of wick drains, instrumentation and associated monitoring program. ■ Additional costs if toe berms need to be constructed ■ Additional costs associated with construction and materials for surcharge and removal of excess surcharge embankment fill upon completion of surcharge period. ■ Additional costs associated with potential EPS top-up. 	<ul style="list-style-type: none"> ■ Complex wick drain design (potentially in combination with lightweight fill). ■ 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.
Rammed Aggregate Piers (RAPs) (up to about 6 m deep)	3	<ul style="list-style-type: none"> ■ Reduced total settlement. ■ Toe berms are not required. 	<ul style="list-style-type: none"> ■ Relatively high area replacement ratio may be required to satisfy stability. 	<ul style="list-style-type: none"> ■ Additional costs associated with mobilization of specialized equipment and design / installation of RAPs. 	<ul style="list-style-type: none"> ■ Readily able to maintaining stability of proposed embankment.



**Table 2B: Evaluation of Stability/Settlement Mitigation Options – Still River SBL Bridge Structure – North Approach Embankment (One-Span Option)
Highway 69 Four-Laning**

Stability/Settlement Mitigation Option	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
		<ul style="list-style-type: none">■ Relatively small area requiring ground improvement treatment.	<ul style="list-style-type: none">■ Installation of RAPs in close proximity to Still River would require protection measures.■ Temporary liners may be required for construction of RAPs as a result of soft clay deposit.	<ul style="list-style-type: none">■ Additional cost associated with the potential need for temporary liners; slows down rate of construction.■ Additional cost if protection measures are required.	<ul style="list-style-type: none">■ Inconsistent stability / settlement mitigation option compared to the mitigation option at the NBL north approach embankment (one-span option) where RAPs are not feasible due to much thicker clay deposit (up to about 15.5 m below existing ground surface).
Partial Preloading (2 m high for 45 days) followed by Lightweight Fill Construction (7.5 m of EPS)	2	<ul style="list-style-type: none">■ Improved stability.■ Reduced total settlement of foundation soils.■ May shorten construction schedule.	<ul style="list-style-type: none">■ Very high cost of EPS construction materials.■ Some additional effort required to remove the partial preload embankments in order to construct the EPS embankment.■ Instrumentation and monitoring program required to assess end of preload period.	<ul style="list-style-type: none">■ Relative cost of EPS fill is about an order of magnitude higher than fill required for the other options.■ Estimated cost for EPS is about \$1,000,000 minus cost of rock fill to construct embankments in base case.	<ul style="list-style-type: none">■ Readily able to achieve stability of partial preload embankments and final EPS embankments on the weak/soft foundation soils.■ No unexpected post-construction settlements.

Note: 1. NR – Not Recommended

Prepared By: TZ/CN

Reviewed By: JPD/JMAC



Table 3: Summary of Foundation Engineering Parameters
Highway 69 Four-Laning

Location	Stratigraphic Unit	Top Elevation (m)	Thickness (m)	γ' (kN/m ³)	ϕ' (°)	c' (kPa)	S_u (kPa)	$\sigma_{p'}$ (kPa)	e_o	C_c	C_r	E' (MPa)	C_y (cm ² /s)
South Approach/Abutment (One-Span or Two-Span Bridge Option)	Topsoil	~ 181.1	0.2 – 0.3	15.0	28	1	-	-	-	-	-	-	-
	Clayey Silt to Silt Clay (Near Surface)	180.9 – 180.8	1.6 – 2.7	17.5	-	-	25	115	1.4	0.70	0.07	-	3.19×10^{-3}
	Sand to Sandy Silt (Upper)	178.8 – 178.2	6.1 – 6.8	18.5	28	-	-	-	-	-	-	6	-
	Silty Clay to Clay (Upper)	172.7 – 171.4	5.0 – 4.6	17.0	-	-	40	180	1.4	0.90	0.09	-	1.81×10^{-3}
	Silt to Sandy Silt (Lower)	167.7 – 166.8	1.1 – 2.3	18.0	29	-	-	-	-	-	-	5	-
	Silty Clay (Lower)	166.6 – 164.5	19.1 – 23.1	17.0	-	-	40 – 100	180 – 460	1.4	0.57 – 1.10	0.06 – 0.11	-	1.81×10^{-3}
	Silt to Silty Sand Interlayer	161.1 – 161.0	2.7 – 4.1	18.0	29	-	-	-	-	-	-	5	-
	Silt to Sand	143.3 – 138.7	3.5 – 15.5	18.5	29	-	-	-	-	-	-	15 – 25	-
	Sand and Gravel	~ 126.7	~ 2.9	21.0	32	-	-	-	-	-	-	30	-
Centre Pier (Two-Span Bridge Option)	Topsoil	179.0 – 178.6	0.2 – 0.3	15.0	27	1	-	-	-	-	-	-	-
	Sand to Silt	178.7 – 178.4	2.0 – 4.0	18.5	28	-	-	-	-	-	-	1 – 5	-
	Silty Clay to Clay	176.7 – 174.5	3.3 – 4.3	17.0	-	-	20 – 50	90 – 225	1.4	0.80	0.08	-	1.97×10^{-3}
North Approach/Abutment (One-Span Bridge Option)	Topsoil	180.6 – 179.5	0.1 – 0.3	15.0	27	1	-	-	-	-	-	-	-
	Silty Sand to Sand	179.4 – 179.2	0.9 – 1.2	18.5	28	-	-	-	-	-	-	1 – 5	-
	Clayey Silt to Clay	180.5 – 178.1	2.1 – 4.1	17.0	-	-	15 – 40	68 – 180	1.4	0.80	0.08	-	1.97×10^{-3}

Note: Foundation Engineering Parameters are not provided for the north approach and abutment location for two-span option due to the thin overburden and bedrock at/near ground surface.
See Figures 1 and 2 for details of engineering parameters for cohesive deposits.

Prepared By: TZ

Reviewed By: JPD/JMAC



**Table 4: Evaluation of Bridge Alternatives – Still River SBL Bridge Structure
Highway 69 Four-Laning**

Bridge Alternative	Rank	Advantages	Disadvantages	Relative Costs	Risks / Consequences
One-Span Option	2	<ul style="list-style-type: none"> ■ Shorter total length of bridge structure. ■ Only two (2) foundation elements are required to support the one-span bridge compared with three (3) foundation elements required to support the two-span bridge. ■ Readily available rock fill for construction of longer north approach embankments 	<ul style="list-style-type: none"> ■ Deep foundations are required to support all foundation elements compared to the two-span option where shallow foundations are recommended at the north abutment. ■ If the full sub-excavation and replacement foundation mitigation option is chosen at the north abutment and approach embankment area, temporary protection systems will be required to protect Still River and support the excavation. ■ Granular fill (i.e. not rock fill) will be required as replacement fill in the north sub-excavation area to allow for the driving of piles. ■ To minimize downdrag load on the north abutment piles, a retaining wall would be required in place of a front slope. 	<ul style="list-style-type: none"> ■ Lower relative cost for foundation elements given that there is no centre pier for this option. ■ High cost associated with stability and settlement mitigation measures required at the north abutment and approach embankment (i.e. full sub-excavation with temporary protection system and replacement with granular fill, or EPS embankment with retaining wall) compared to the two-span option (i.e. where no foundation mitigation is required at centre pier or at north abutment and approach). ■ The overall cost for foundations and foundation mitigation measures for the one-span option is anticipated to be higher compared to the two-span option. 	<ul style="list-style-type: none"> ■ Potential for instability of north approach embankment as a result of proximity to the north bank of Still River and complexity associated with the stability mitigation measure(s). ■ Depending on which approach embankment stability/settlement mitigation option is adopted, there is additional potential for lateral soil movement at the north abutment adding to the complexity of the pile design at the north abutment.



**Table 4: Evaluation of Bridge Alternatives – Still River SBL Bridge Structure
Highway 69 Four-Laning**

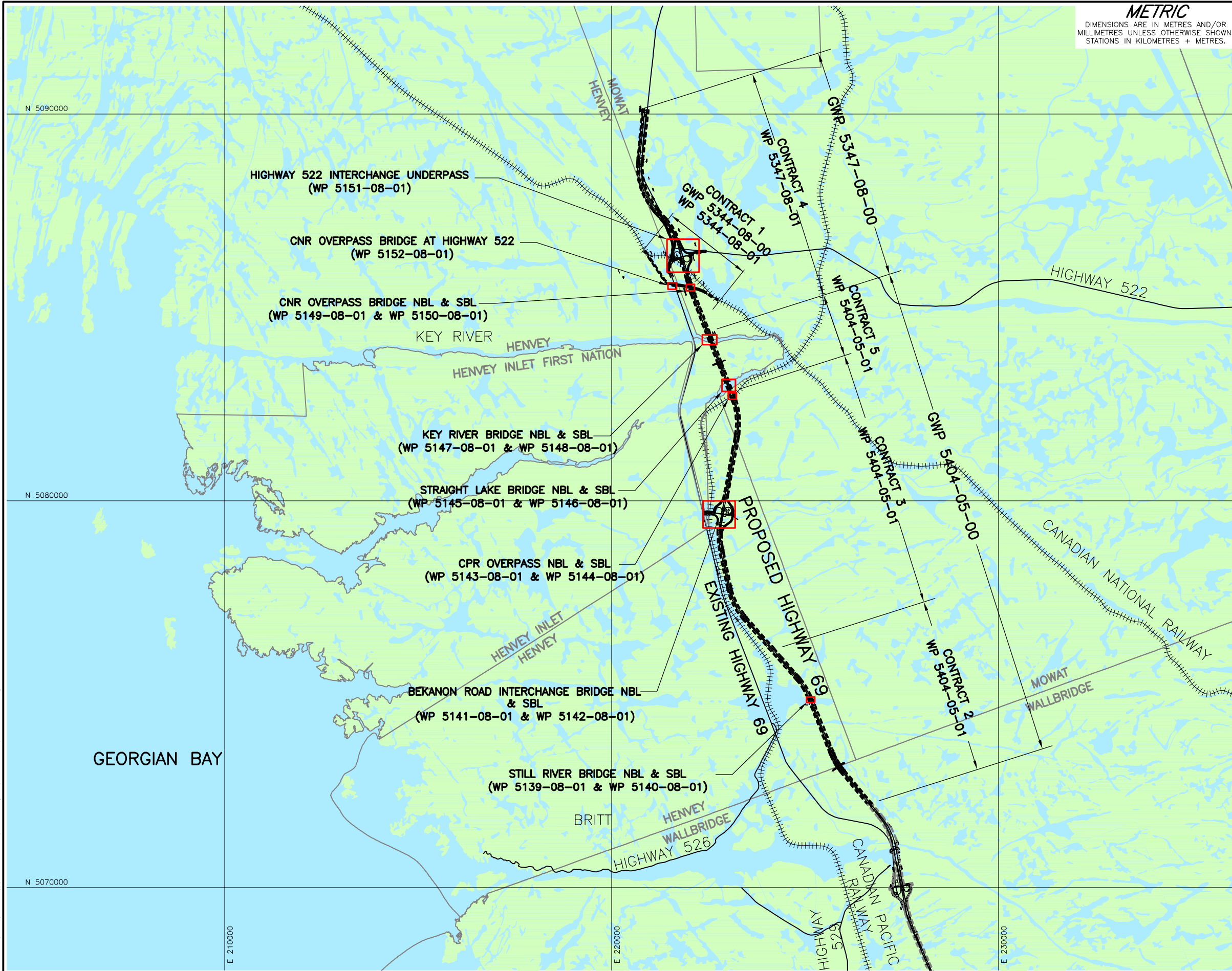
Bridge Alternative	Rank	Advantages	Disadvantages	Relative Costs	Risks / Consequences
Two-Span Option	1	<ul style="list-style-type: none">■ Relative ease of construction at the north abutment and north approach embankment.	<ul style="list-style-type: none">■ Longer total length of bridge structure.■ Footing construction required for additional foundation element (i.e. at the north abutment).■ Excavation and dewatering required to construct the pile cap at the centre pier and potentially at the spread footing at the north abutment.	<ul style="list-style-type: none">■ Additional cost associated with construction of an additional foundation element compared to the one-span option.■ Significantly lower relative cost of construction of north approach embankment compared to north approach embankment associated with one-span option.■ Additional cost associated with excavation and dewatering required to construct pile cap for the centre pier.■ The overall cost for two-span option is anticipated to be lower than the one-span option.	<ul style="list-style-type: none">■ No expected instability and long-term settlement of north approach embankment.■ Variability in depth to bedrock at the north abutment will impact excavation depth and mass concrete quantities.

Prepared By: TZ/CN

Reviewed By: JPD/JMAC



DRAWINGS



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

CONT No.
GWP No. 5404-05-00

HIGHWAY 69
SITE LOCATION PLAN

SHEET

Golder Associates

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA

KEY PLAN
NOT TO SCALE

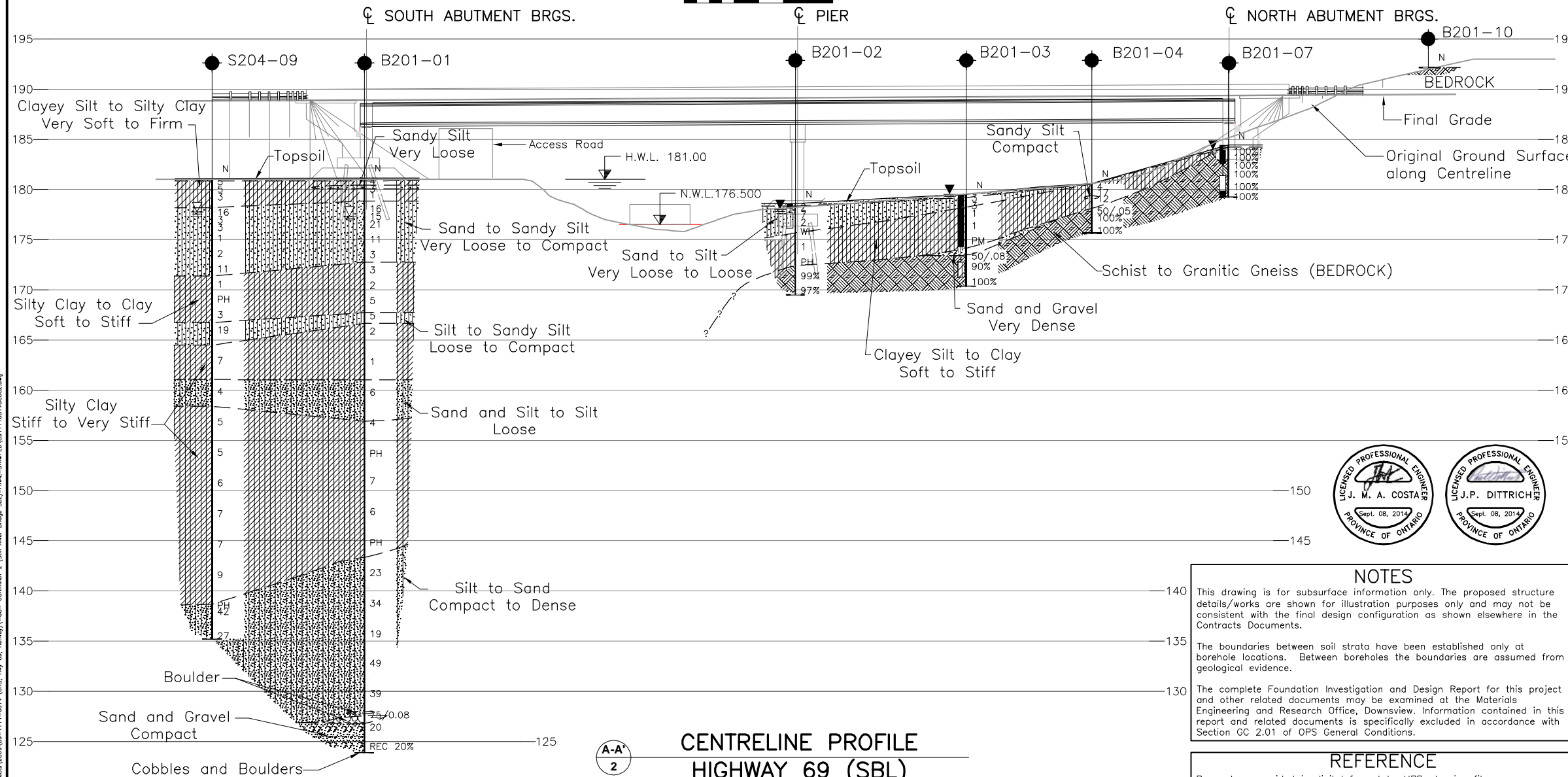
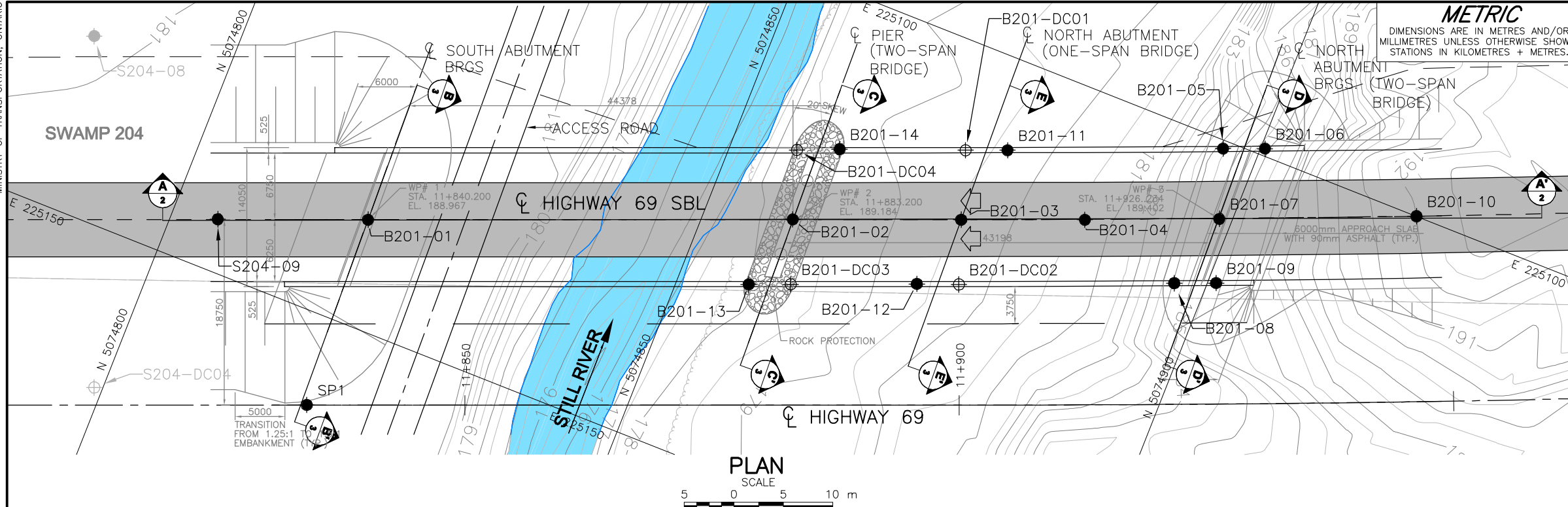
PLAN



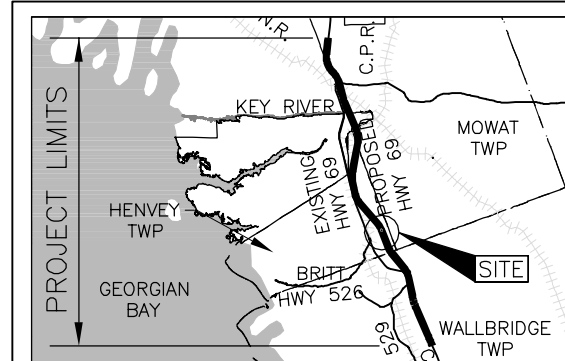
REFERENCE

Base Data - MNR NRVIS, obtained 2004, CANMAP v2008
Produced by Golder Associates Ltd under licence from
Ontario Ministry of Natural Resources, © Queens Printer 2008
Datum : NAD 83 Projection : MTM Zone 10

NO.	DATE	BY	REVISION
Geocres No. 41H-137			
HWY. 69	PROJECT NO. 09-1111-6014		DIST.
SUBM'D. TVA	CHKD. TVA	DATE: June 2011	SITE:
DRAWN: JFC	CHKD. CN	APPD. JPD/JMAC	DWG. 1

CENTRELINE PROFILE
HIGHWAY 69 (SBL)SCALE
5 0 5 10 mCONT No.
WP No. 5140-08-01HIGHWAY 69
STILL RIVER SBL BRIDGE STRUCTURE
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA

KEY PLAN

SCALE
6 0 6 12 km

LEGEND

- Borehole - Current Investigation
- ⊕ Dynamic Cone Penetration Test
- 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)
- REC Recovery (%)
- WL in piezometer, measured on March 26, 2011
- WL upon completion of drilling

BOREHOLE CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
B201-01	181.1	5074818.7	225139.3
B201-02	178.6	5074858.6	225123.5
B201-03	179.5	5074874.5	225117.3
B201-04	180.6	5074886.1	225112.7
B201-05	183.9	5074896.5	225100.8
B201-06	185.4	5074900.4	225099.3
B201-07	184.4	5074898.7	225107.6
B201-08	183.4	5074896.9	225115.3
B201-09	185.5	5074900.8	225113.7
B201-10	192.2	5074917.2	225100.0
B201-11	179.5	5074876.3	225109.0
B201-12	179.6	5074872.7	225124.9
B201-13	178.7	5074856.9	225131.2
B201-14	179.0	5074860.4	225115.2
B201-DC01	179.4	5074872.3	225110.6
B201-DC02	179.8	5074876.7	225123.4
B201-DC03	178.3	5074860.8	225129.7
B201-DC04	178.6	5074856.4	225116.8
S204-09	181.1	5074804.5	225144.9
SP1	181.1	5074819.8	225159.0

NOTES

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

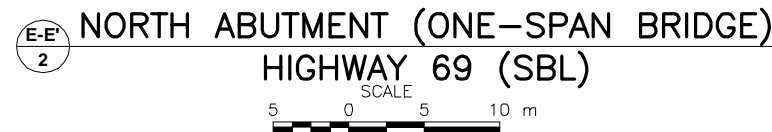
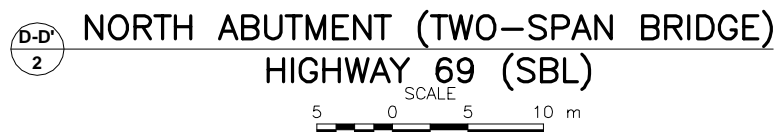
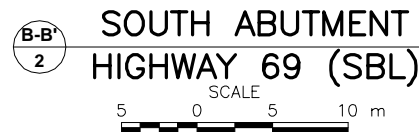
The boundaries between soil strata have been established only at borehole locations. Between boreholes the boundaries are assumed from geological evidence.

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

REFERENCE

Base plans provided in digital format by URS, drawing files Hwy69_base.dwg, Hwy69_plan.dwg, received December 16, 2009, StillRiver_SBL_ga.dwg, received April 5, 2011 and Hwy69_Contour-Plan_C2_C3.dwg, received July 14, 2011.

NO.	DATE	BY	REVISION
Geocres No. 41H-137			
HWY. 69		PROJECT NO. 09-1111-6014	DIST.
SUBM'D. TVA	CHKD. TVA/TZ	DATE: Dec. 2012	SITE: 44-458/2
DRAWN: JFC/CD	CHKD. CN	APPD. JPD/JMAC	DWG. 2



Ground Surface cut from OG-Survey-Contours C2-3.dwg,
received May 30, 2011.



NO.	DATE	BY	REVISION		
Geocres No. 41H-137					
HWY. 69		PROJECT NO. 09-1111-6014			DIST.
SUBM'D. TVA	CHKD. TVA/TZ	DATE: Dec. 2012		SITE: 44-458/2	
DRAWN: JFC	CHKD. CN	APPD. JPD/JMAC		DWG. 3	

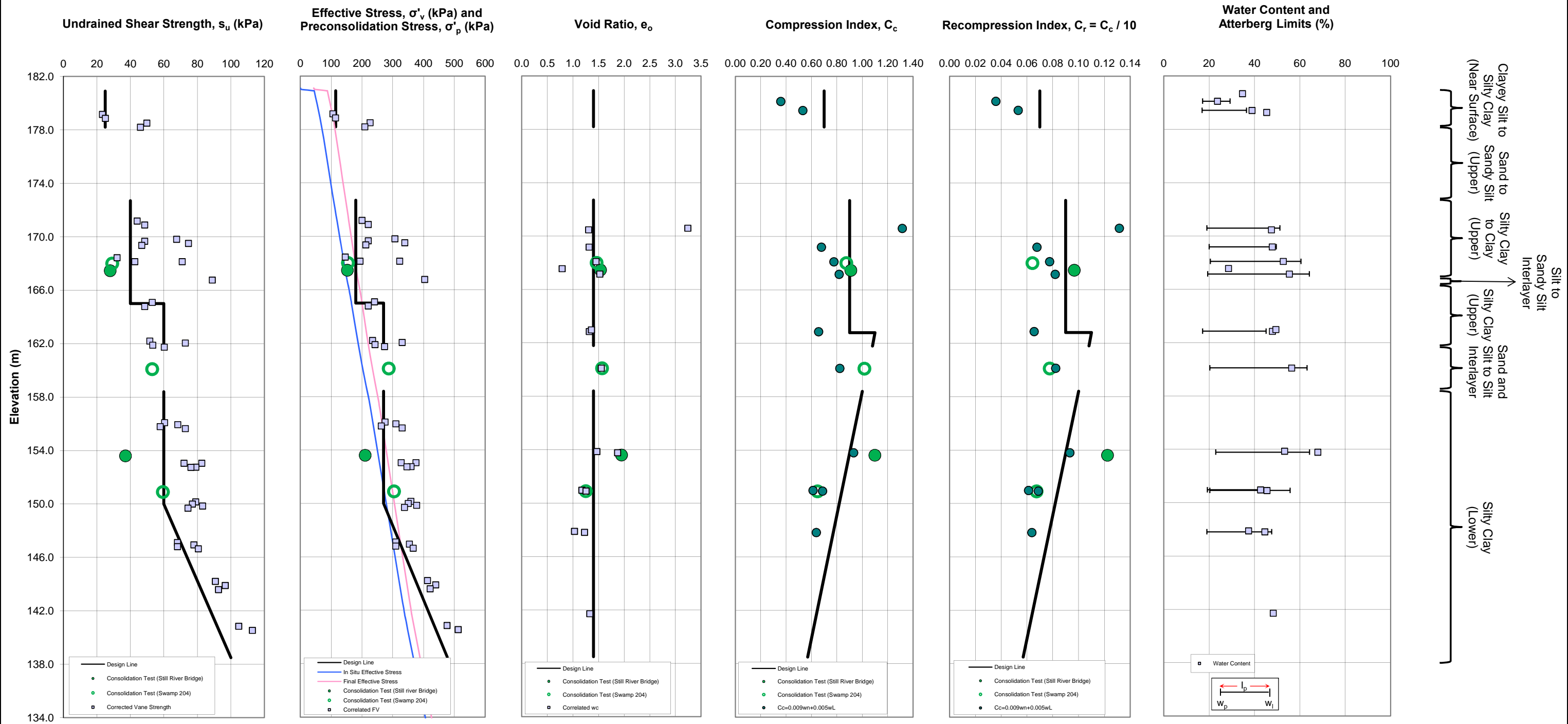


FIGURES

\\capws\dav\WWW\Root\sites\0911116014\highway69\FourLaning\Contract 2\Reporting\Final\Still River SBL Bridge\Figures\09-1111-6014-2523 FIG1 Parameters-Still River Bridge SBL South Abutment-Final Plot

**SUMMARY PLOT OF ENGINEERING PARAMETERS FOR
COHESIVE DEPOSITS**
Still River Bridge (SBL) Structure - South Abutment and Approach (One-
Span or Two-Span Option)

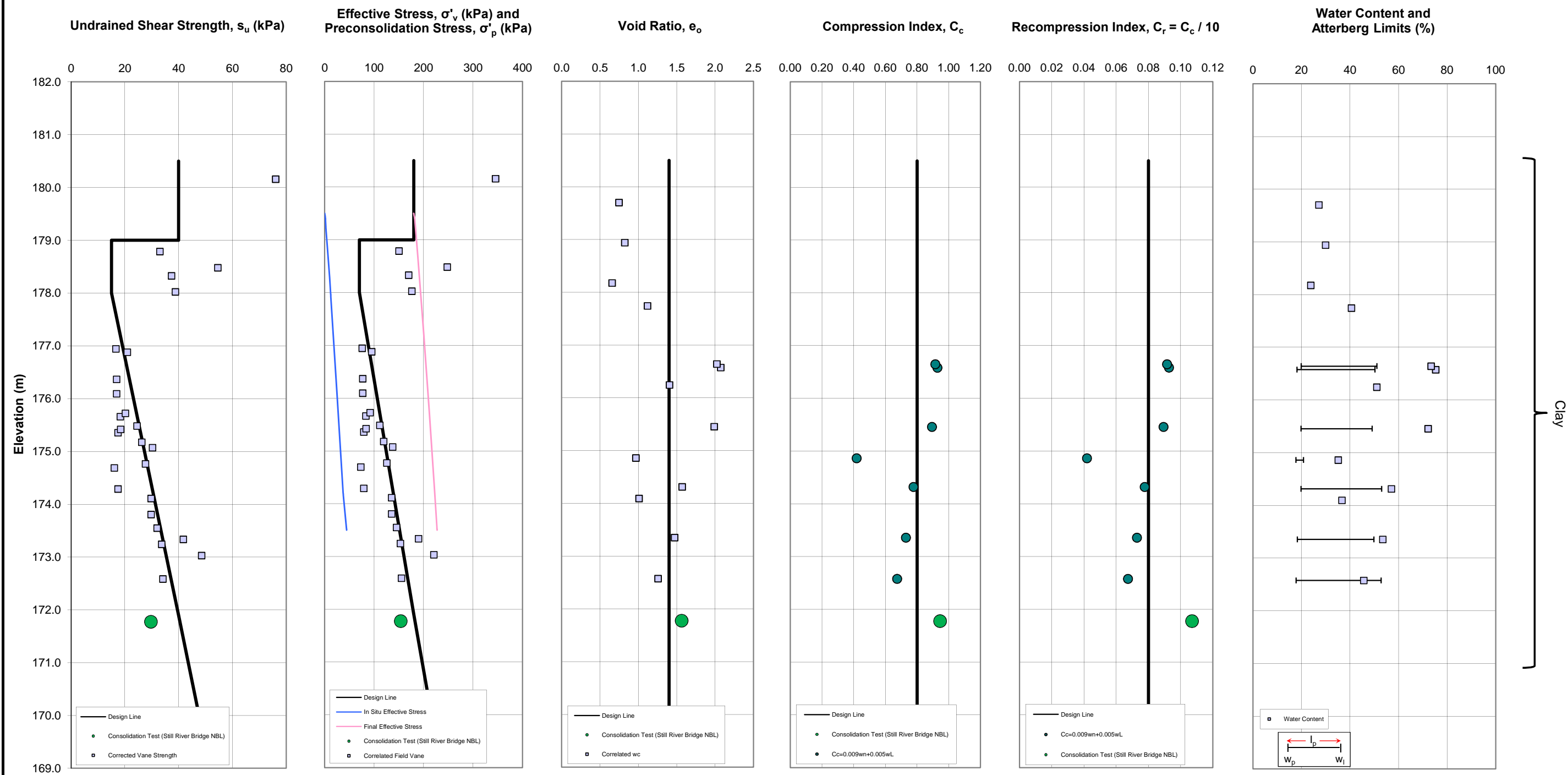
FIGURE 1



\\capws\dav\WWWRoot\Sites\0911116014\highway69\FourLaning\Contract 2\Reporting\Final\Still River SBL Bridge\Figures\09-1111-6014-2523 FIG2 Parameters-Still River Bridge SBL North Abutment-Final Plot

SUMMARY PLOT OF ENGINEERING PARAMETERS FOR
COHESIVE DEPOSITS
Still River Bridge (SBL) Structure - Pier (Two-Span Option) and North
Abutment and Approach (One-Span Option)

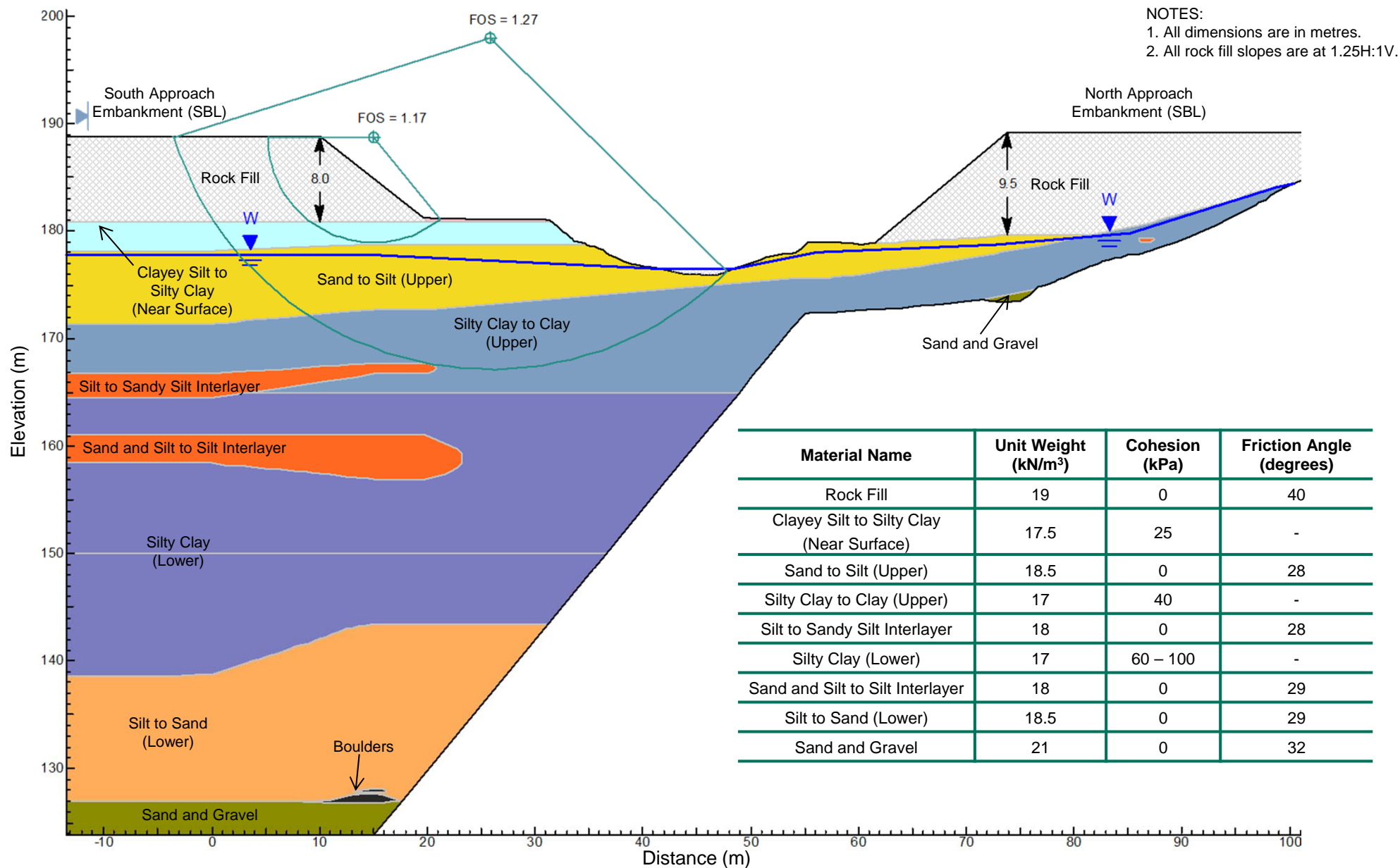
FIGURE 2





Highway 69 SBL – Still River SBL Bridge Structure – South Approach Front Slope Stability (No Stability Mitigation Options)

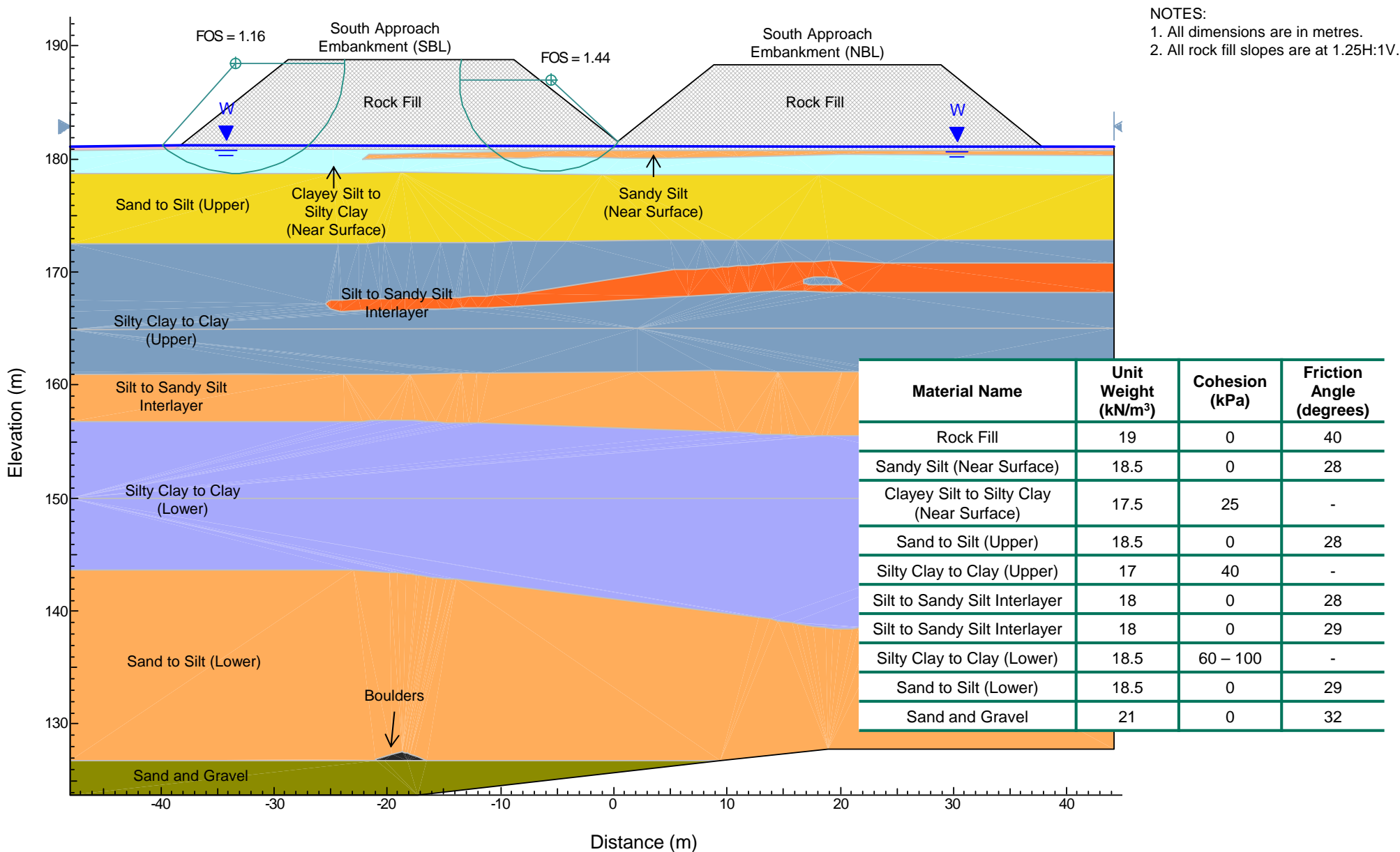
Figure 3





Highway 69 SBL – Still River SBL Bridge Structure – South Approach Side Slope Stability (No Stability Mitigation Options)

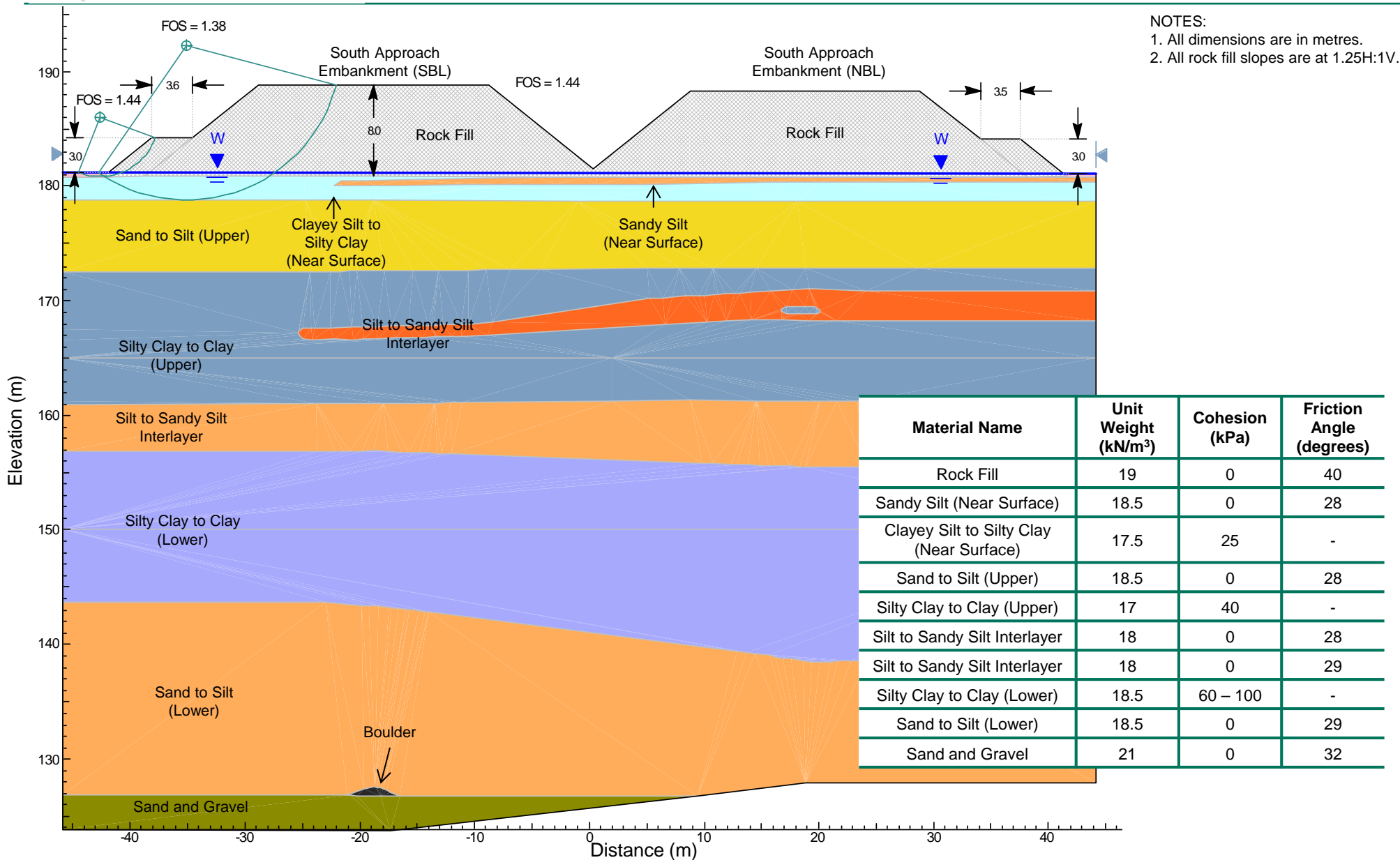
Figure 4





Highway 69 SBL – Still River SBL Bridge Structure – South Approach Side Slope Stability (Outside Side Slope Toe Berm)

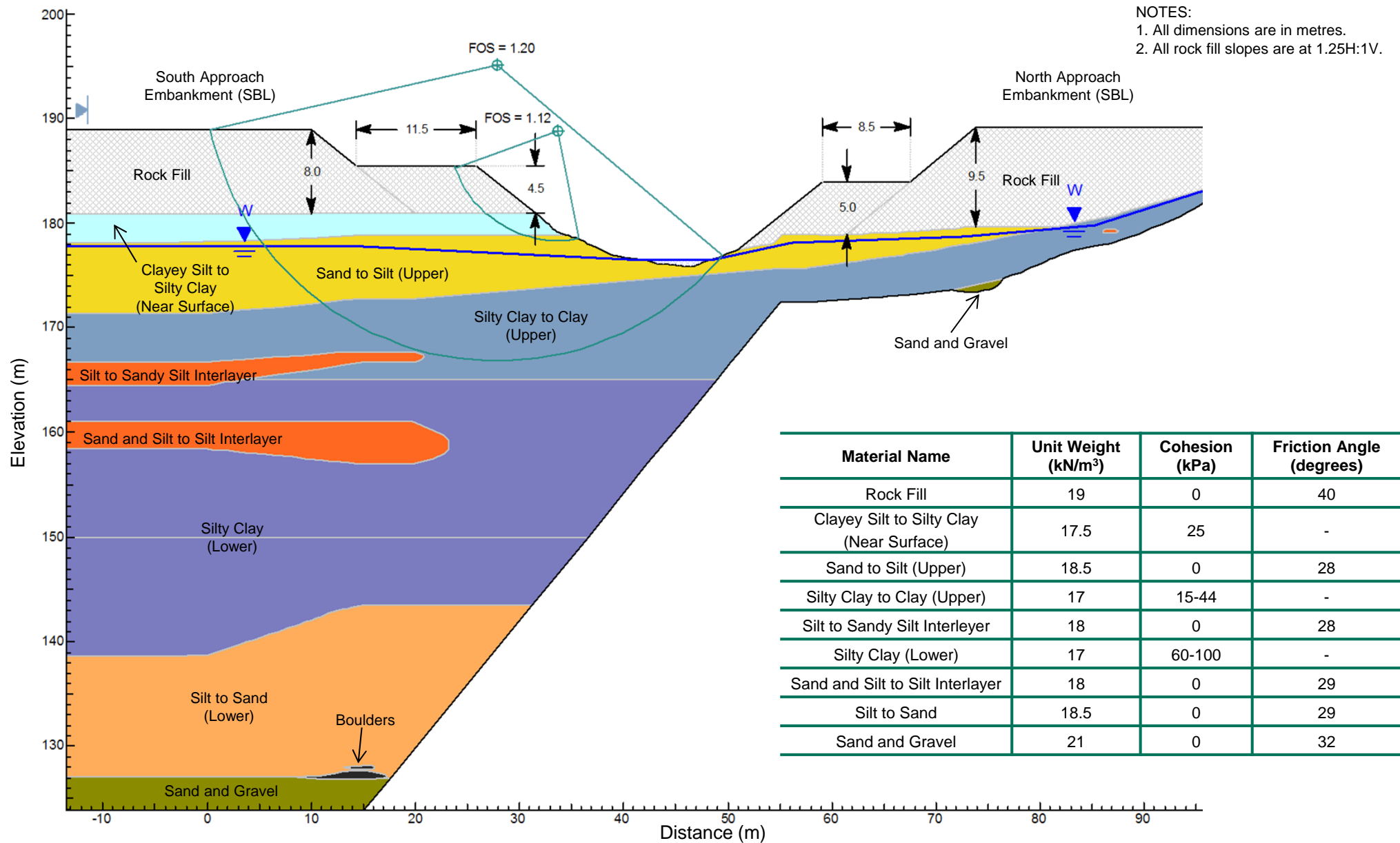
Figure 5





Highway 69 SBL – Still River SBL Bridge Structure – South Approach Front Slope Stability (Front Slope Toe Berm)

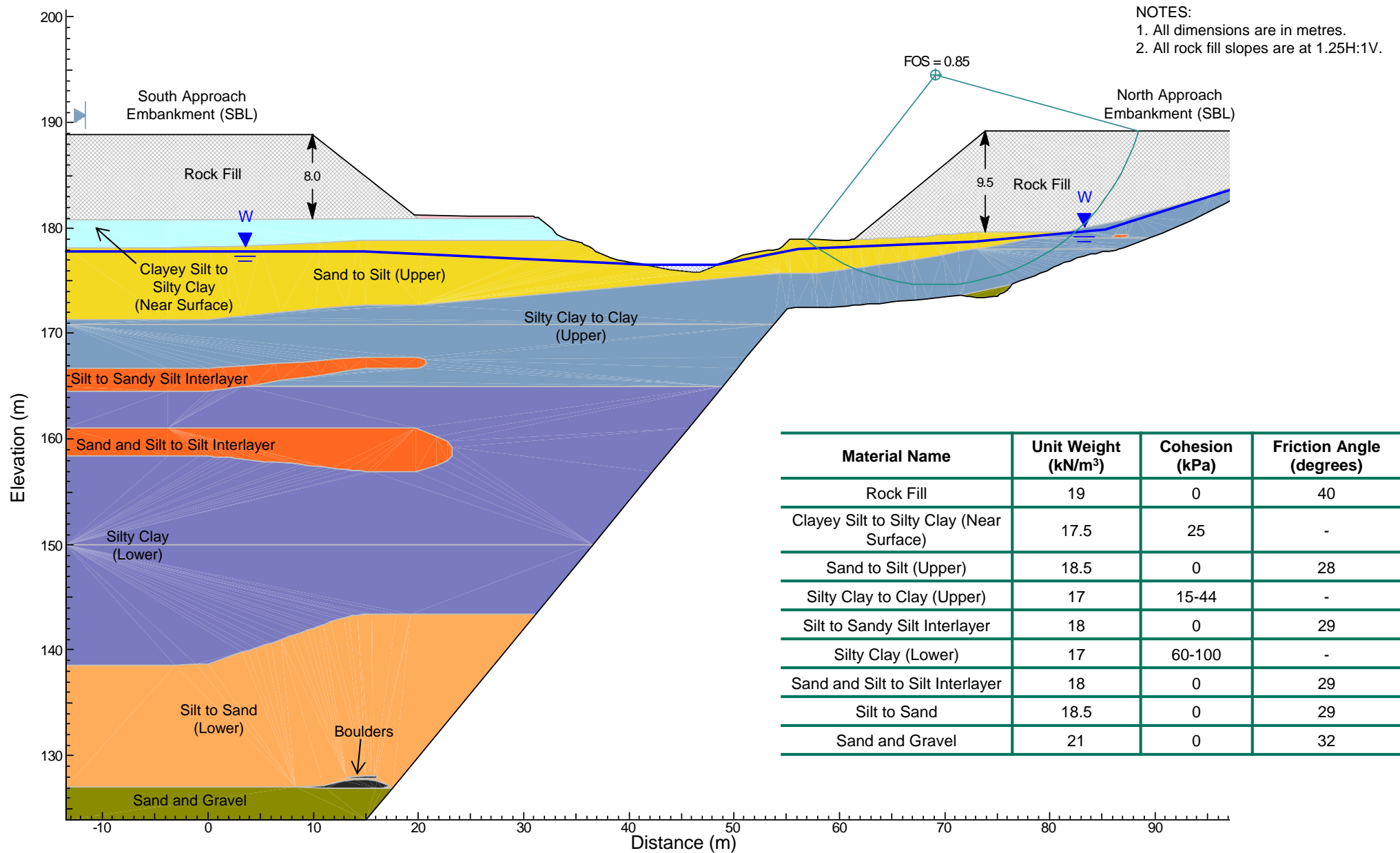
Figure 6





Highway 69 SBL – Still River SBL Bridge Structure – North Approach (One-Span Bridge) Front Slope Stability (No Stability Mitigation Options)

Figure 7



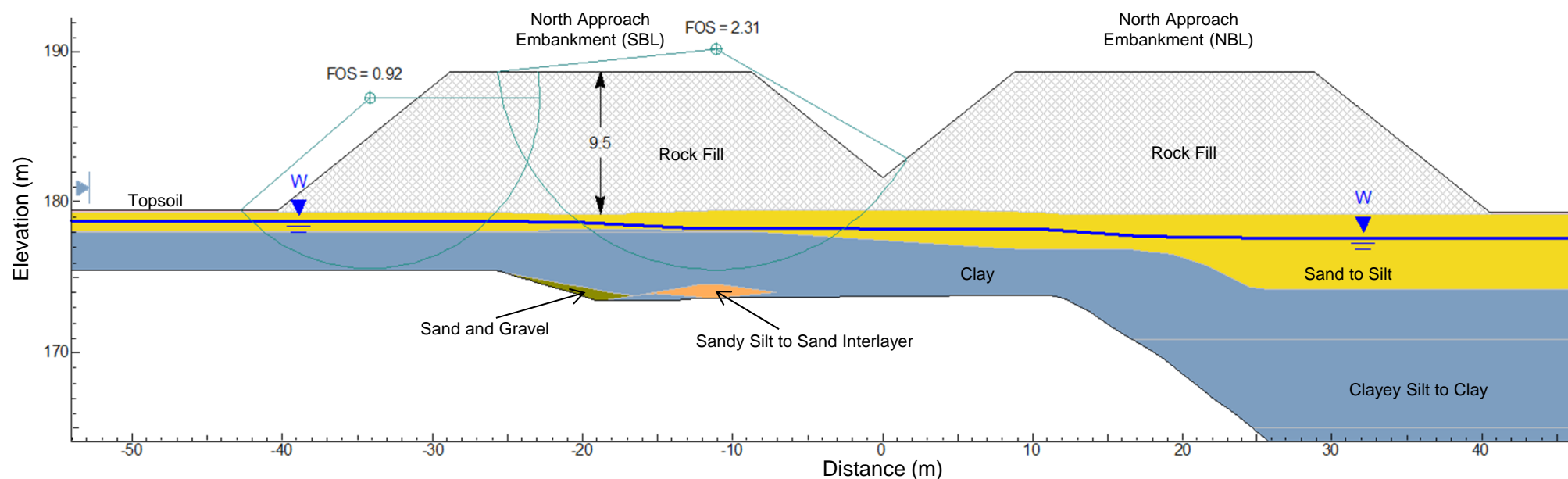


Highway 69 SBL – Still River SBL Bridge Structure – North Approach (One-Span Bridge) Side Slope Stability (No Stability Mitigation Options)

Figure 8

Material Name	Unit Weight (kN/m ³)	Cohesion (kPa)	Friction Angle (degrees)
Rock Fill	19	0	40
Sand to Silt	18.5	0	28
Clay	16.5	15-44	-
Sandy Silt to Sand Interlayer	18.5	0	28
Sand and Gravel	20	0	29
Clayey Silt to Clay	17	44-50	-

NOTES:
1. All dimensions are in metres.
2. All rock fill slopes are at 1.25H:1V.



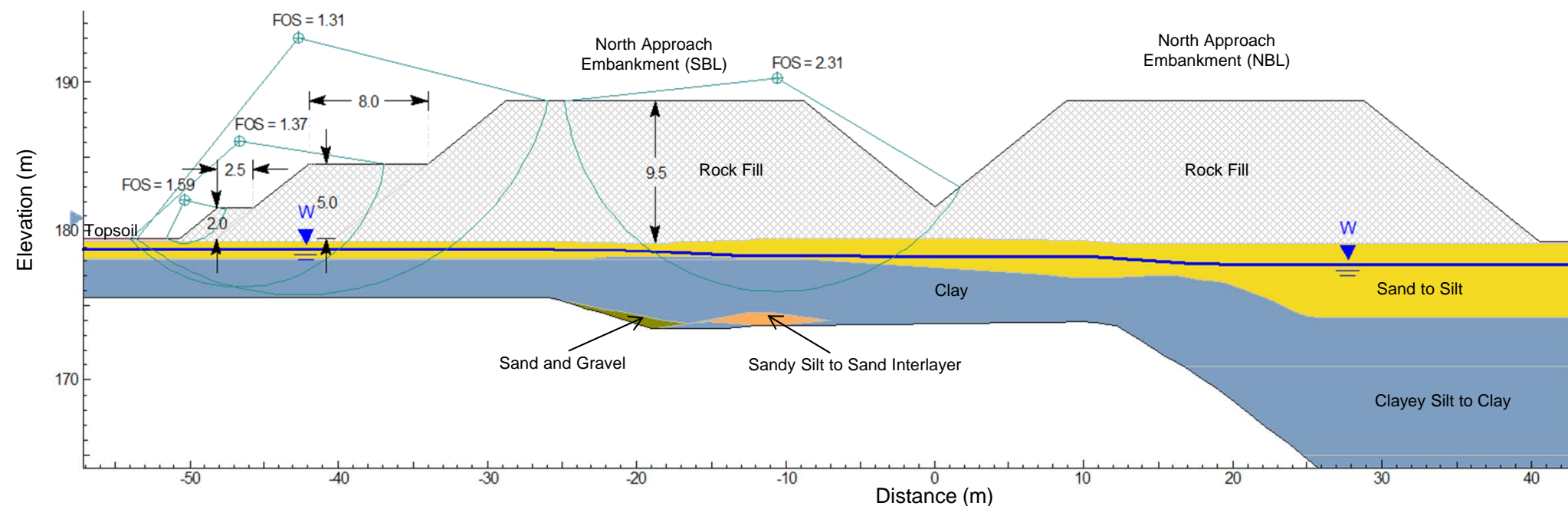


Highway 69 SBL – Still River SBL Bridge Structure – North Approach (One-Span Bridge) Side Slope Stability (Outside Toe Berm)

Figure 9

Material Name	Unit Weight (kN/m ³)	Cohesion (kPa)	Friction Angle (degrees)
Rock Fill	19	0	40
Sand to Silt	18.5	0	28
Clay	16.5	15 – 44	-
Sandy Silt to Sand Interlayer	18.5	0	28
Sand and Gravel	20	0	29
Clayey Silt to Clay	17	44 – 50	-

NOTES:
1. All dimensions are in metres.
2. All rock fill slopes are at 1.25H:1V.

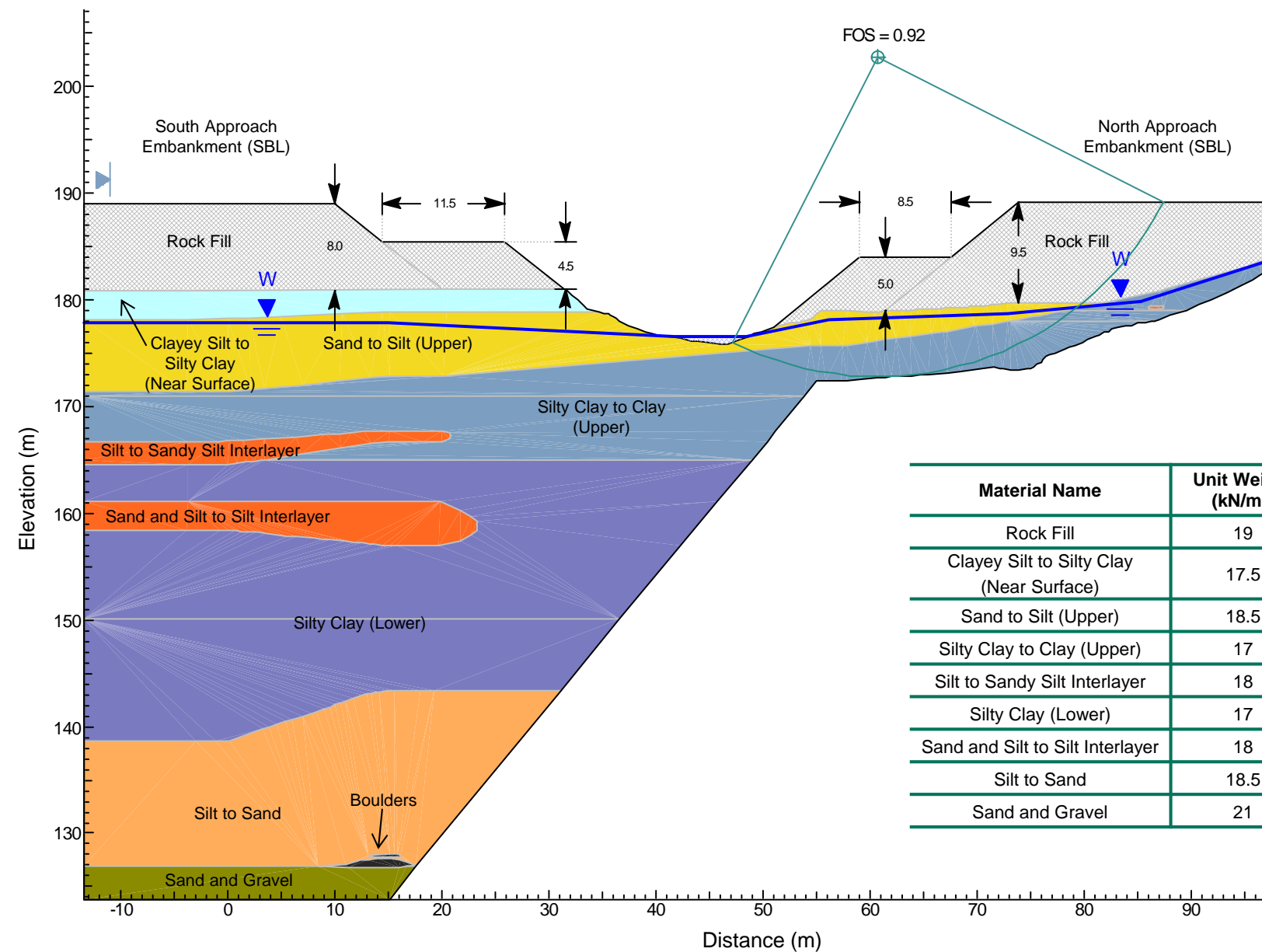




Highway 69 SBL – Still River SBL Bridge Structure – North Approach (One-Span Bridge) Front Slope Stability (Front Slope Toe Berm)

Figure 10

NOTES:
1. All dimensions are in metres.
2. All rock fill slopes are at 1.25H:1V.



Material Name	Unit Weight (kN/m ³)	Cohesion (kPa)	Friction Angle (degrees)
Rock Fill	19	0	40
Clayey Silt to Silty Clay (Near Surface)	17.5	25	-
Sand to Silt (Upper)	18.5	0	28
Silty Clay to Clay (Upper)	17	15 – 44	-
Silt to Sandy Silt Interlayer	18	0	28
Silty Clay (Lower)	17	60 – 100	-
Sand and Silt to Silt Interlayer	18	0	29
Silt to Sand	18.5	0	29
Sand and Gravel	21	0	32

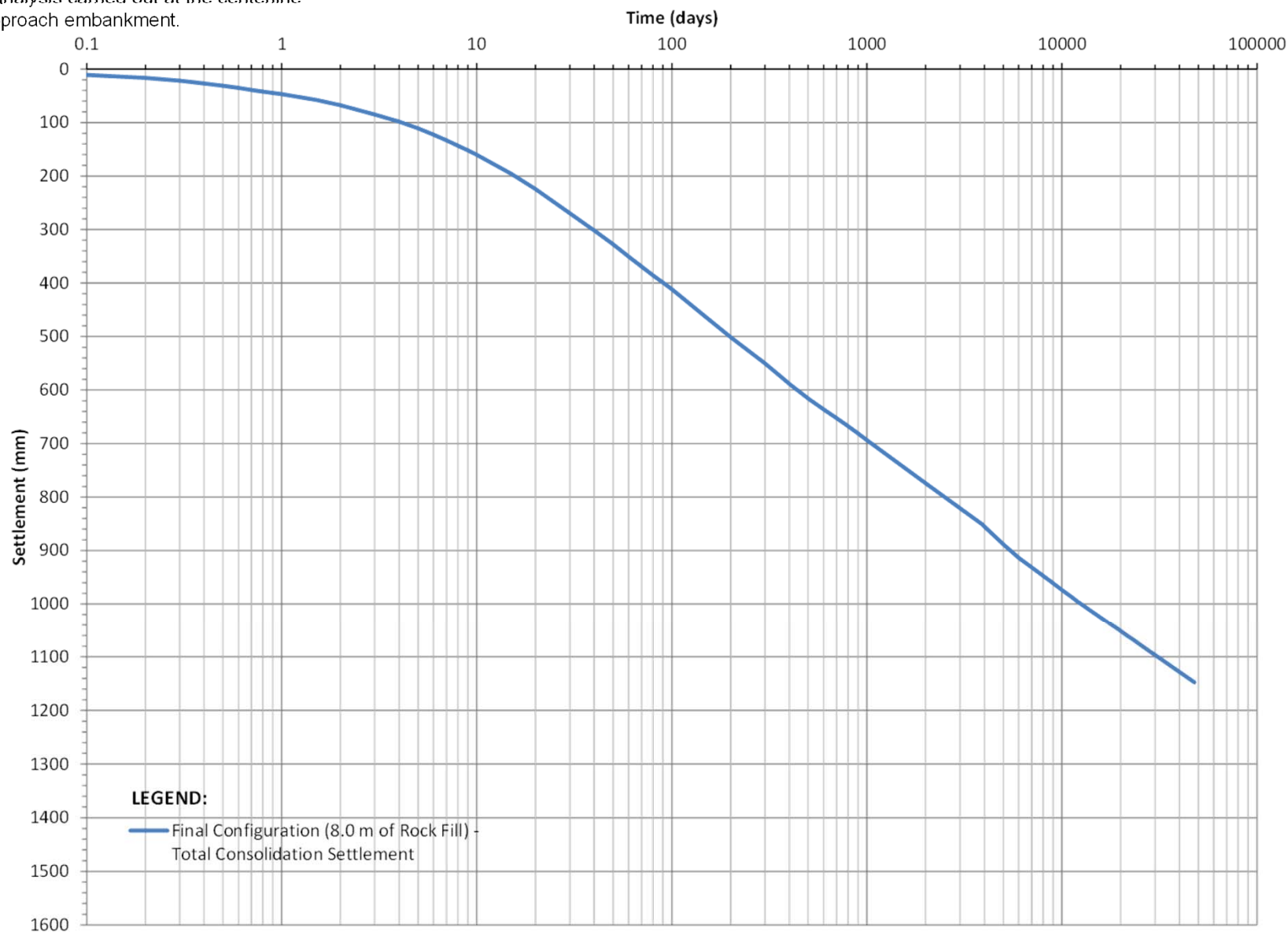


Highway 69 SBL – Still River SBL Bridge Structure – South Approach Time-Rate of Settlement (No Mitigation Options)

Figure 11

NOTE:

1. Settlement analysis carried out at the centerline of the south approach embankment.





Highway 69 SBL – Still River SBL Bridge Structure – North Approach (One-Span Option) Time-Rate of Settlement (No Mitigation Options)

Figure 12

NOTE:

1. Settlement analysis carried out at the centerline of the north approach embankment (one-span option).

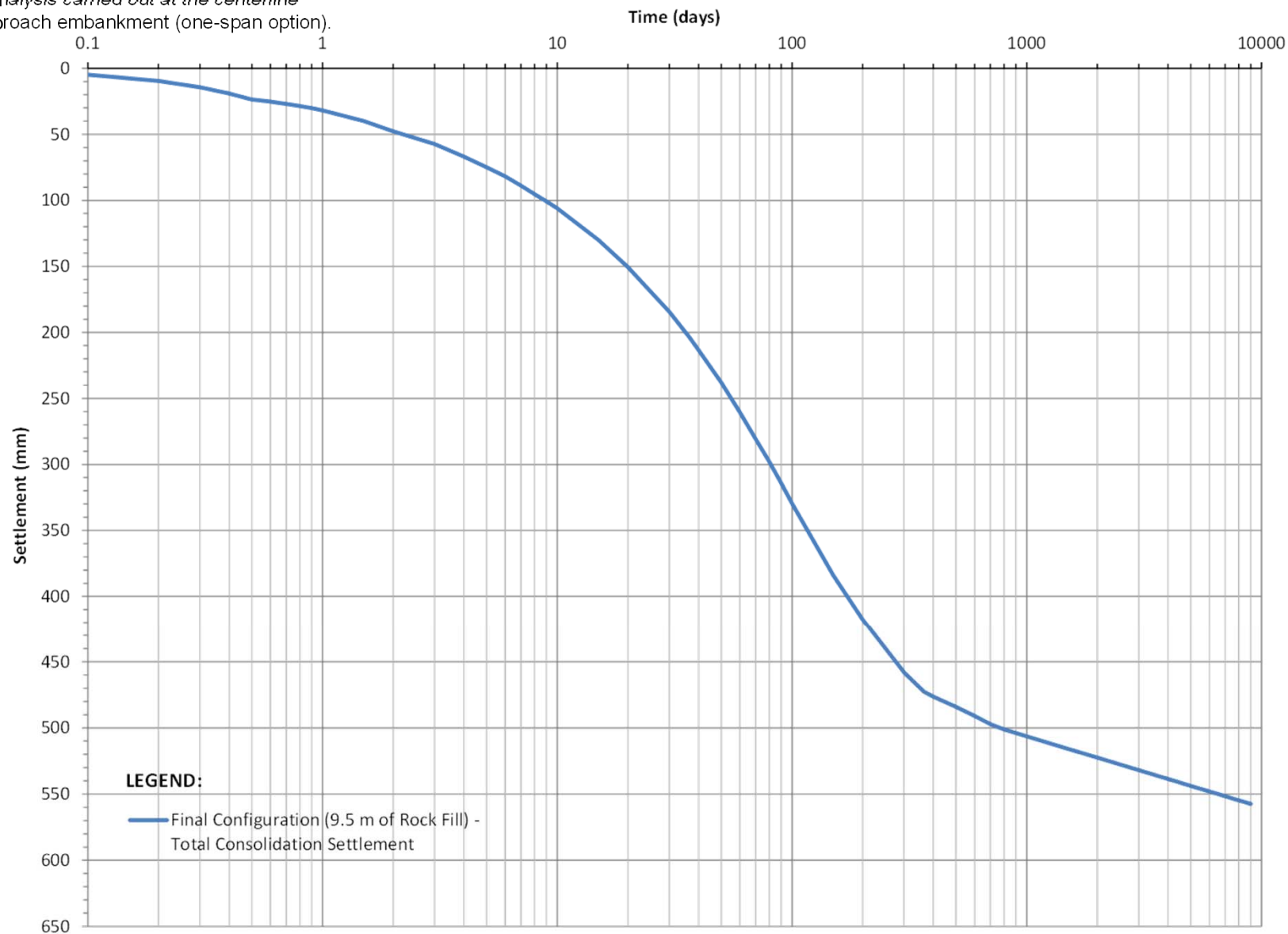
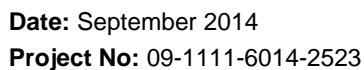




Figure 13



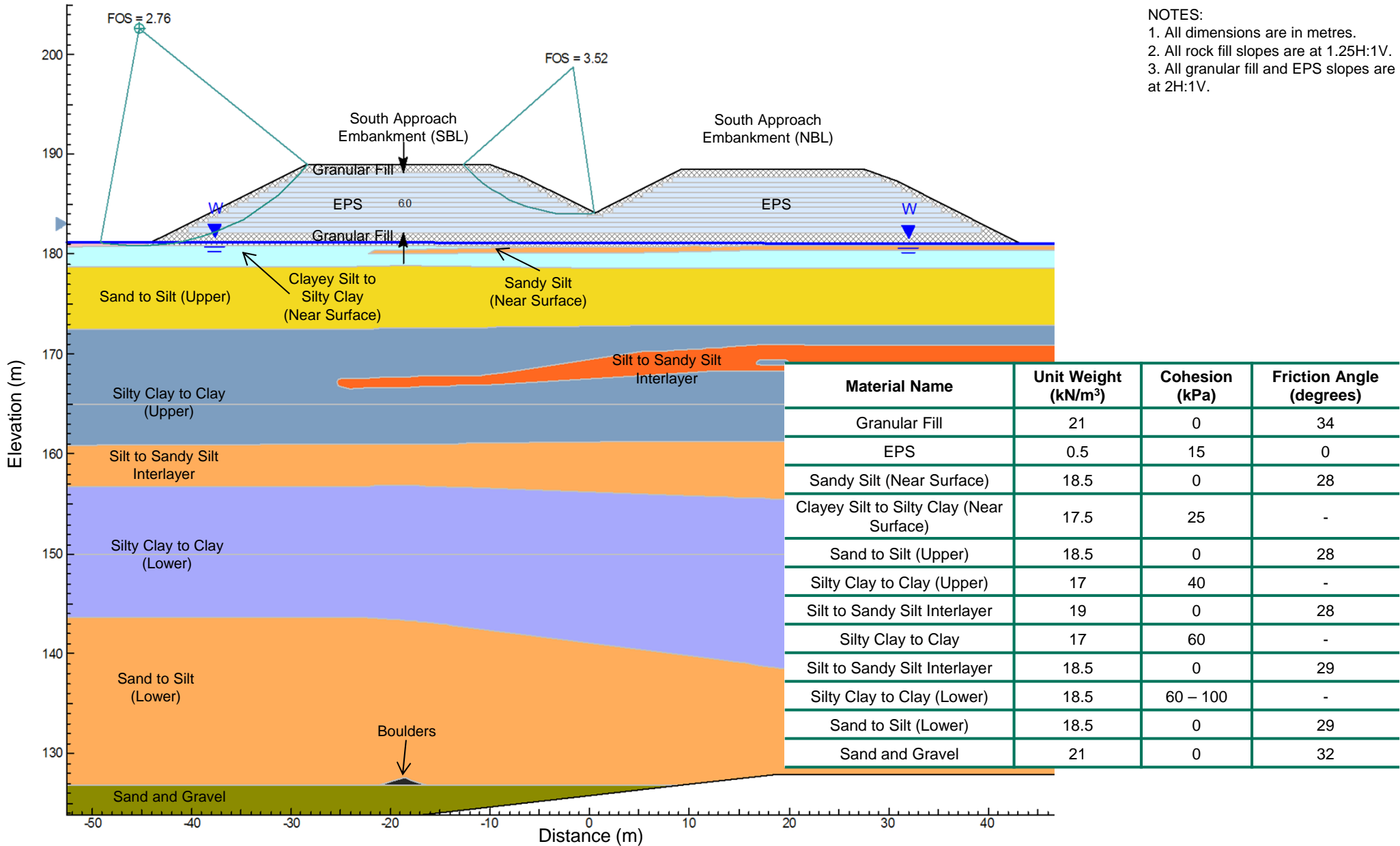
Analysis By: TZ **Reviewed By:** JPD/JMAC





Highway 69 SBL – Still River SBL Bridge Structure – South Approach Side Slope Stability (6.0 m of EPS Fill)

Figure 14



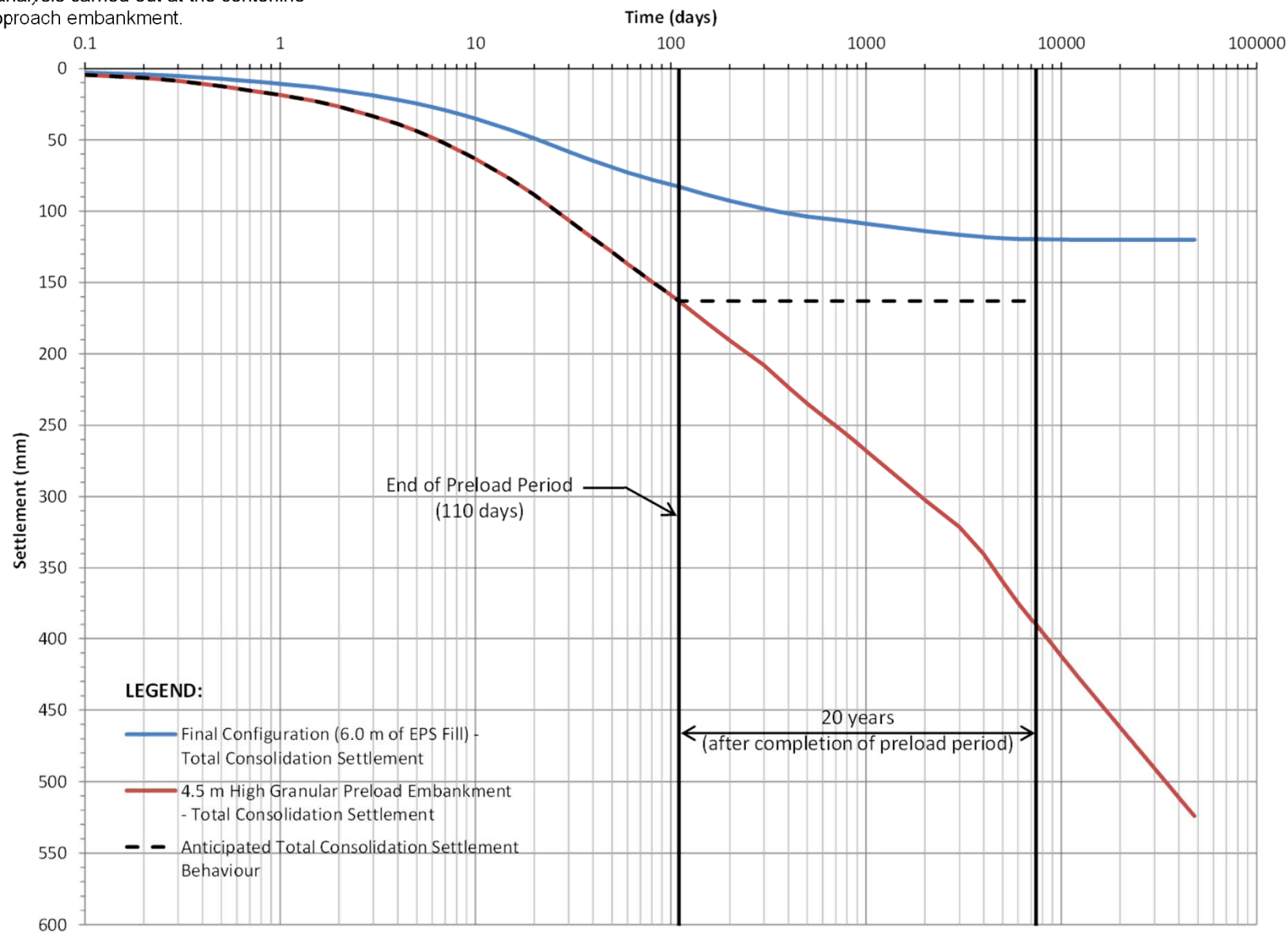


Highway 69 SBL – Still River SBL Bridge Structure – South Approach Time-Rate of Settlement (6.0 m of EPS Fill)

Figure 15

NOTE:

1. Settlement analysis carried out at the centerline of the south approach embankment.

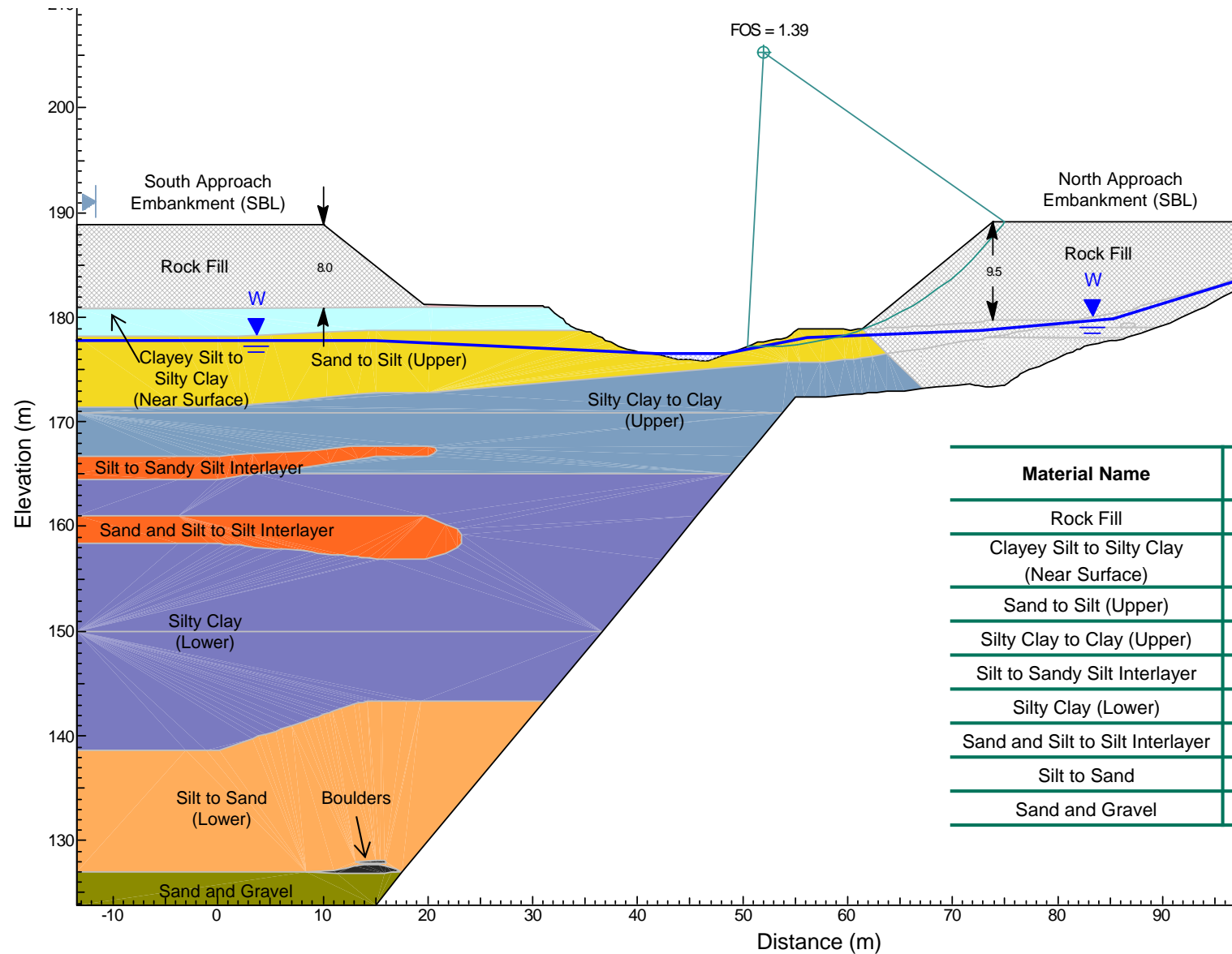




Highway 69 SBL – Still River SBL Bridge Structure – North Approach (One-Span Bridge) Front Slope Stability (Full Sub-Excavation and Backfill)

Figure 16

NOTES:
 1. All dimensions are in metres.
 2. All rock fill slopes are at 1.25H:1V.



Material Name	Unit Weight (kN/m ³)	Cohesion (kPa)	Friction Angle (degrees)
Rock Fill	19	0	40
Clayey Silt to Silty Clay (Near Surface)	17.5	25	-
Sand to Silt (Upper)	18.5	0	28
Silty Clay to Clay (Upper)	17	15-44	-
Silt to Sandy Silt Interlayer	18	0	28
Silty Clay (Lower)	17	60-100	-
Sand and Silt to Silt Interlayer	18	0	29
Silt to Sand	18.5	0	29
Sand and Gravel	21	0	32



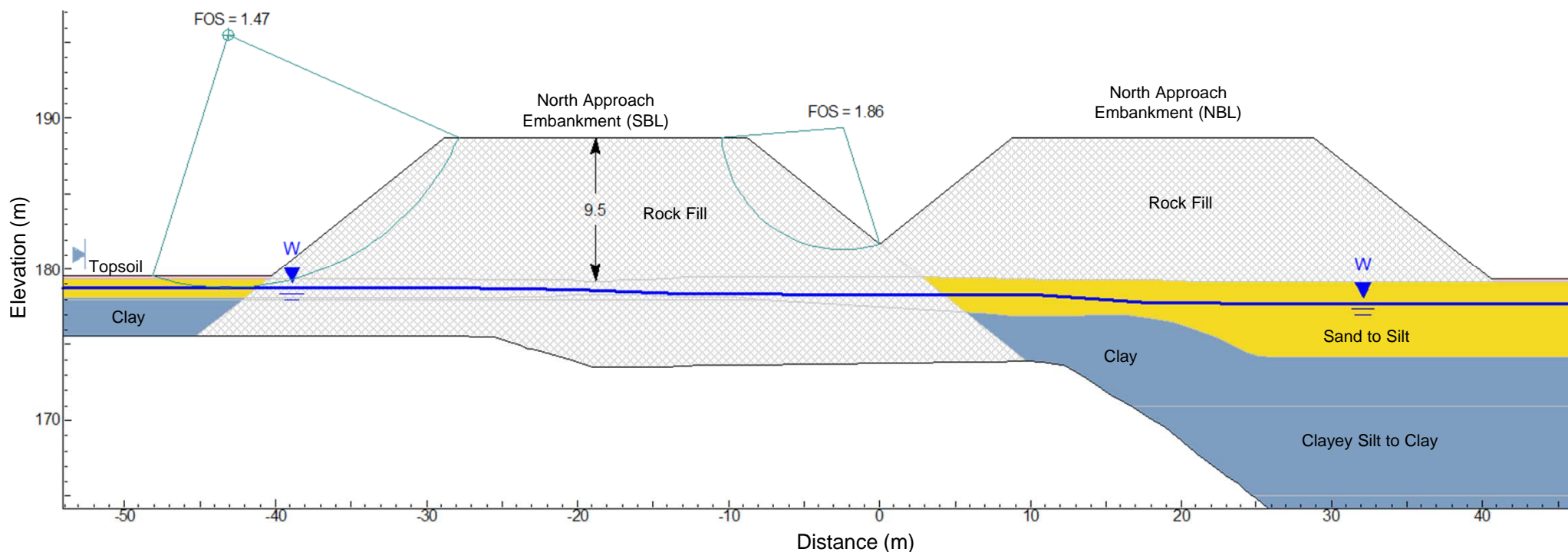
Highway 69 SBL – Still River SBL Bridge Structure – North Approach (One-Span Bridge) Side Slope Stability (Full Sub-Excavation and Backfill)

Figure 17

Material Name	Unit Weight (kN/m ³)	Cohesion (kPa)	Friction Angle (degrees)
Rock Fill	19	0	40
Sand to Silt	18.5	0	28
Clay	16.5	15 – 44	-
Clayey Silt to Clay	17	44 – 50	-

NOTES:

1. All dimensions are in metres.
2. All rock fill slopes are at 1.25H:1V.



Note: All dimensions are in meters.

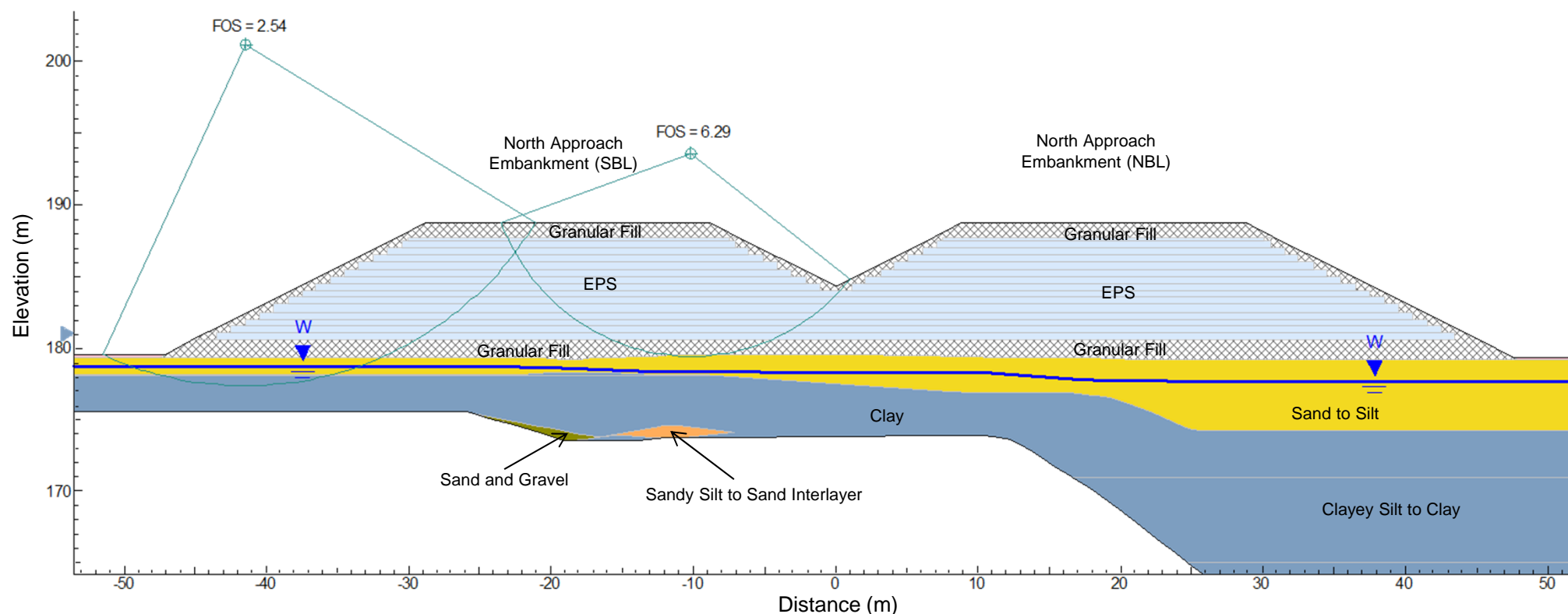


Highway 69 SBL – Still River SBL Bridge Structure – North Approach (One-Span Bridge) Side Slope Stability (7.5 m of EPS Fill)

Figure 18

Material Name	Unit Weight (kN/m ³)	Cohesion (kPa)	Friction Angle (degrees)
Granular Fill	21	0	34
EPS	0.5	15	0
Sand to Silt	18.5	0	28
Clay	16.5	15-44	-
Sandy Silt to Sand Interlayer	18.5	0	28
Sand and Gravel	20	0	29
Clayey Silt to Clay	17	44-50	-

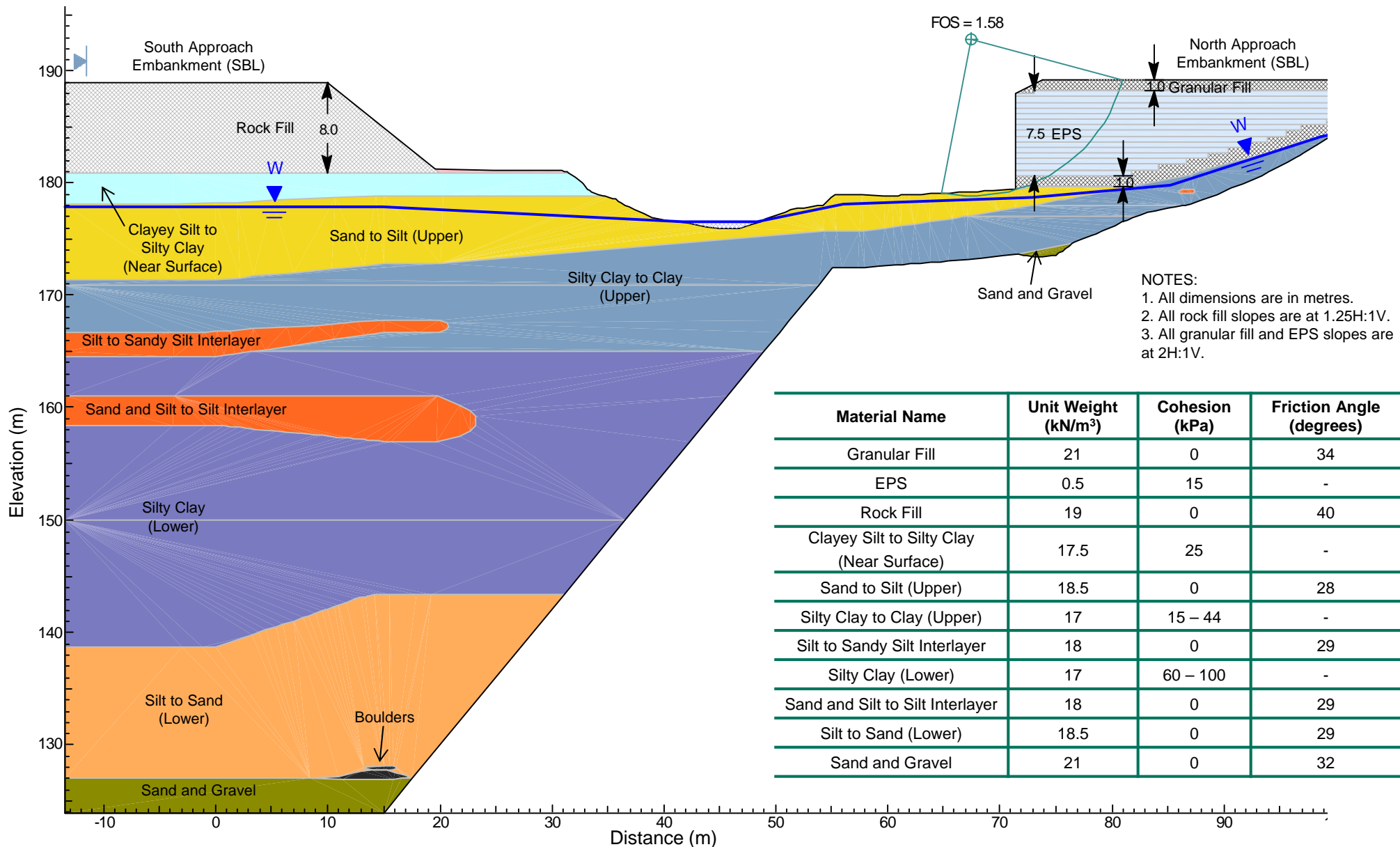
NOTES:
1. All dimensions are in metres.
2. All rock fill slopes are at 1.25H:1V.
3. All granular fill and EPS slopes are at 2H:1V.





Highway 69 SBL – Still River SBL Bridge Structure – North Approach (One-Span Bridge) Front Slope Stability (7.5 m of EPS Fill)

Figure 19





APPENDIX A

Record of Boreholes, Drillholes, Probeholes and Dynamic Cone Penetration Tests



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

$\tau = c' + \sigma' \tan \phi'$
shear strength = (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 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	C _u , S _u	psf
Very soft	0 to 12		0 to 250
Soft	12 to 25		250 to 500
Firm	25 to 50		500 to 1,000
Stiff	50 to 100		1,000 to 2,000
Very stiff	100 to 200		2,000 to 4,000
Hard	over 200		over 4,000

Dynamic Cone Penetration Resistance; N_d:

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

PH: Sampler advanced by hydraulic pressure

PM: Sampler advanced by manual pressure

WH: Sampler advanced by static weight of hammer

WR: Sampler advanced by weight of sampler and rod

Piezo-Cone Penetration Test (CPT)

A electronic cone penetrometer with a 60° conical tip and a project end area of 10 cm² pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (Q_t), porewater pressure (PWP) and friction along a sleeve are recorded electronically at 25 mm penetration intervals.

IV. SOIL TESTS

w	water content
w _p	plastic limit
w _l	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test ¹
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement ¹
D _R	relative density (specific gravity, G _s)
DS	direct shear test
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
SO ₄	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V	field vane (LV-laboratory vane test)
γ	unit weight

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

V. MINOR SOIL CONSTITUENTS

Per cent by Weight	Modifier	Example
0 to 5	Trace	Trace sand
5 to 12	Trace to Some (or Little)	Trace to some sand
12 to 20	Some	Some sand
20 to 30	(ey) or (y)	Sandy
over 30	And (non-cohesive) or With (cohesive)	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 <u>09-1111-6014</u>		RECORD OF BOREHOLE No B201-01		SHEET 1 OF 5	METRIC
W.P. <u>5140-05-01</u>	LOCATION <u>N 5074818.7 ; E 225139.3</u>	ORIGINATED BY <u>MR</u>			
DIST <u> </u> HWY <u>69</u>	BOREHOLE TYPE <u>127 mm O.D. Continuous Flight Solid Stem Augers, HW Casing, Wash Boring</u>	COMPILED BY <u>MAS</u>			
DATUM <u>Geodetic</u>	DATE <u>February 3 to 8, 2011</u>	CHECKED BY <u>TVA</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		WATER CONTENT (%)					
								<div><div></div><div>20 40 60 80 100</div></div>	<div><div></div><div>20 40 60</div></div>	<div><div></div><div>20 40 60</div></div>	<div><div></div><div>20 40 60</div></div>				
181.1	GROUND SURFACE														
0.0 180.8	TOPSOIL		1A	SS	4		181								
180.4	CLAYEY SILT, trace sand, containing rootlets Firm Brown Moist		1B												
180.0			2A	SS	3		180								
1.1	SANDY SILT, trace clay, containing sand layers Very loose Brown and Grey Wet		2B												
			3	SS	1										
178.8	SILTY CLAY, trace sand Very soft to soft Grey Moist						179								
2.3															
	SAND, trace silt, trace clay Compact Brown Moist		4	SS	16		178								
			5	SS	15										
176.9							177								
4.2	SAND and SILT, trace clay Very loose to compact Grey Wet		6	SS	21										
							176								
			7	SS	11		175								
			8	SS	3	174									
						173									
172.7															
8.4	SILTY CLAY, containing sand lenses Soft to stiff Grey Moist		9	SS	3	172									
			10	SS	2	171									
			11	SS	5	170									
167.7						169									
											</				

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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 09/02/14

PROJECT 09-1111-6014		RECORD OF BOREHOLE No B201-01		SHEET 2 OF 5		METRIC																
W.P. 5140-05-01		LOCATION N 5074818.7 ; E 225139.3		ORIGINATED BY MR																		
DIST HWY 69		BOREHOLE TYPE 127 mm O.D. Continuous Flight Solid Stem Augers, HW Casing, Wash Boring		COMPILED BY MAS																		
DATUM Geodetic		DATE February 3 to 8, 2011		CHECKED BY TVA																		
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			
	--- CONTINUED FROM PREVIOUS PAGE ---							20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100					W _p — W — W _L 20 40 60			kN/m ³						
161.0	SILTY CLAY, trace sand Stiff Grey Moist		13	SS	2		166															
20.1							165															
			14	SS	1		163															
							162															
							161															
	SILT, trace sand, trace clay Loose Grey Wet		15	SS	6		160															
							159															
							158															
156.9			16A				157															
24.2	SILTY CLAY, trace sand Stiff Grey Moist		16B	SS	4		156															
							155															
							154															
			17	TO	PH		153															
							152															

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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 09/02/14

PROJECT		RECORD OF BOREHOLE		No B201-01		SHEET 3 OF 5		METRIC							
W.P. 09-1111-6014		LOCATION		N 5074818.7 ; E 225139.3		ORIGINATED BY		MR							
DIST		HWY 69		BOREHOLE TYPE		127 mm O.D. Continuous Flight Solid Stem Augers, HW Casing, Wash Boring		COMPILED BY							
DATUM		Geodetic		DATE		February 3 to 8, 2011		CHECKED BY							
								TVA							
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							
	--- CONTINUED FROM PREVIOUS PAGE ---							20 40 60 80 100							
	SILTY CLAY, trace sand Stiff Grey Moist		18	SS	7		151								0 2 40 58
							150								
							149								
	Containing silt lenses between depths of 32.9 m and 33.5 m		19	SS	6		148								
							147								
							146								
			20	TO	PH		145								
							144								
143.3 37.8	SILT, some clay, trace sand Compact Grey Wet						143								
			21	SS	23		142								Non-Plastic 0 3 83 14
							141								
140.3 40.8	SAND, trace to some silt Compact to dense Grey Wet						140								
			22	SS	34		139								
							138								
							137								

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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)\LOG09-1111-6014.GPJ GAL-GTA.GDT 09/02/14

PROJECT 09-1111-6014		RECORD OF BOREHOLE No B201-01		SHEET 4 OF 5		METRIC							
W.P. 5140-05-01		LOCATION N 5074818.7 ; E 225139.3		ORIGINATED BY MR									
DIST HWY 69		BOREHOLE TYPE 127 mm O.D. Continuous Flight Solid Stem Augers, HW Casing, Wash Boring		COMPILED BY MAS									
DATUM Geodetic		DATE February 3 to 8, 2011		CHECKED BY TVA									
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 ---												
	SAND, trace to some silt Compact to dense Grey Wet		23	SS	19								
			24	SS	49								
131.1													
50.0	SILT, some sand, trace clay, trace gravel Dense Grey Wet												
			25	SS	39								
127.9													
	Boulder		26	SS	75/0.08								
	SAND, trace silt		1	RC	-								
53.6	Grey Wet Boulders		27	SS	-								
			2	RC	-								
126.7													
54.4	SAND and GRAVEL, trace silt Compact Grey Wet		28	SS	20								
	Containing cobbles and boulders below a depth of 55.3 m												
			3	RC	REC 20%								RQD = 0%
123.8													
57.3	END OF BOREHOLE												

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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 09/02/14

PROJECT 09-1111-6014		RECORD OF BOREHOLE No B201-01				SHEET 5 OF 5		METRIC																																									
W.P. 5140-05-01		LOCATION N 5074818.7 ; E 225139.3				ORIGINATED BY MR																																											
DIST HWY 69		BOREHOLE TYPE 127 mm O.D. Continuous Flight Solid Stem Augers, HW Casing, Wash Boring				COMPILED BY MAS																																											
DATUM Geodetic		DATE February 3 to 8, 2011				CHECKED BY TVA																																											
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 ---							<div style="display: flex; justify-content: space-between;"> 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>					<div style="display: flex; justify-content: space-between;"> 20 40 60 </div> <div style="display: flex; justify-content: space-between;"> W_p W W_L </div>					WATER CONTENT (%)		kN/m ³	GR SA SI CL																													
<p>NOTES:</p> <p>1. On February 5, 2011, water was flowing from top of casing when advanced to a depth of 42.0 m below ground surface (Elev. 139.1 m), then stopped after advancing and pulling rods in/out of the borehole up to a depth of 51.8 m - Possible artesian condition.</p> <p>2. Grouted borehole with portland cement to a depth of 51.8 m to continue advancing borehole.</p> <p>3. Water level measurements in casing during and upon completion of drilling:</p> <table border="1" style="width: 100%; border-collapse: collapse;"> <thead> <tr> <th>Date</th> <th>Depth (m) Water/Casing</th> <th>W.L. Elev</th> </tr> </thead> <tbody> <tr> <td>(m)</td> <td></td> <td></td> </tr> <tr> <td>03/02/11</td> <td>4.1/11.9</td> <td>177.0</td> </tr> <tr> <td>04/02/11</td> <td>0.2/36.0</td> <td>180.9</td> </tr> <tr> <td>05/02/11</td> <td>0.9/36.0</td> <td>180.2</td> </tr> <tr> <td>06/02/11</td> <td>-1.1*/48.0</td> <td></td> </tr> <tr> <td></td> <td>182.2*</td> <td></td> </tr> <tr> <td>06/02/11</td> <td>-2.3*/51.8</td> <td>183.4*</td> </tr> <tr> <td>07/02/11</td> <td>3.7/53.2</td> <td>177.4</td> </tr> <tr> <td>08/02/11</td> <td>1.3/54.3</td> <td>179.8</td> </tr> <tr> <td>10/02/11</td> <td>3.8/55.3</td> <td>177.3</td> </tr> </tbody> </table> <p>* Artesian condition</p>		Date	Depth (m) Water/Casing	W.L. Elev	(m)			03/02/11	4.1/11.9	177.0	04/02/11	0.2/36.0	180.9	05/02/11	0.9/36.0	180.2	06/02/11	-1.1*/48.0			182.2*		06/02/11	-2.3*/51.8	183.4*	07/02/11	3.7/53.2	177.4	08/02/11	1.3/54.3	179.8	10/02/11	3.8/55.3	177.3															
Date	Depth (m) Water/Casing	W.L. Elev																																															
(m)																																																	
03/02/11	4.1/11.9	177.0																																															
04/02/11	0.2/36.0	180.9																																															
05/02/11	0.9/36.0	180.2																																															
06/02/11	-1.1*/48.0																																																
	182.2*																																																
06/02/11	-2.3*/51.8	183.4*																																															
07/02/11	3.7/53.2	177.4																																															
08/02/11	1.3/54.3	179.8																																															
10/02/11	3.8/55.3	177.3																																															

GTA-MTO 001 T:\PROJECTS\2009\09-1111-6014 (URS, HWY 69, HENVEY)\LOG\09-1111-6014.GPJ GAL-GTA.GDT 09/02/14

PROJECT		RECORD OF BOREHOLE		No B201-02		SHEET 1 OF 1		METRIC					
W.P.		LOCATION		ORIGINATED BY		DIST		BOREHOLE TYPE					
5140-05-01		N 5074858.6 ; E 225123.5		MR		HWY 69		127 mm O.D. Continuous Flight Solid Stem Augers, NW Casing, Wash Boring					
DATUM		DATE		CHECKED BY		COMPILED BY		MAS					
Geodetic		March 23, 2011		TVA									
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100	PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	GR SA SI CL
178.6	GROUND SURFACE												
0.0	TOPSOIL												
0.2	Silty SAND, trace clay, containing rootlets, organics and wood fragments Very loose to loose Brown Wet		1A	SS	2		178						
			1B										
			2	SS	7								
			3	SS	2		177						0 74 23 3
176.3	SILT, trace sand, trace clay Very loose Grey Wet		4A	SS	WH		176						
175.7	CLAY, some silt, trace sand Firm Grey Moist		4B										
2.9							175						
			5	SS	1								0 1 39 60
			6	TO	PH		173						
172.4	Granitic Gneiss (BEDROCK)												
6.2	Bedrock cored from depths of 6.2 m to 9.1 m For bedrock coring details refer to Record of Drillhole B201-02		1	RC	REC 100%		172						RQD = 99%
			2	RC	REC 97%		171						RQD = 97%
169.5	END OF BOREHOLE						170						
9.1	NOTES: 1. Water level in open borehole at a depth of 0.7 m below ground surface (Elev. 177.9 m), measured during drilling. 2. Water level measurement in Piezometer installed 0.3 m north of Borehole B201-02: Date Depth (m) Elev. (m) 23/03/11 0.7 177.9 03/10/12 0.7 177.9												

SHEET 1 OF 1

DATUM: Geodetic

DRILLING CONTRACTOR: Walker Drilling

CHECKED: MAS/TVA

PROJECT		RECORD OF BOREHOLE		No B201-03		SHEET 1 OF 1		METRIC											
W.P. 09-1111-6014		LOCATION		N 5074874.5 ; E 225117.3		ORIGINATED BY		MR											
DIST		HWY 69		BOREHOLE TYPE		127 mm O.D. Continuous Flight Solid Stem Augers, NW Casing, Wash Boring		COMPILED BY											
DATUM		Geodetic		DATE		March 21, 2011		CHECKED BY											
								TVA											
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			20	40						60	80	100	20	40
179.5	GROUND SURFACE																		
179.2	TOPSOIL		1A	SS	3														
0.3	Silty SAND, containing rootlets to a depth of 0.7 m Very loose Brown Wet		1B																
178.3			2A	SS	3														
1.2	CLAY, some silt, trace sand Soft Brown to grey Moist		2B																
			3	SS	1														
	Becoming grey at a depth of 2.4 m																		
			4	SS	1														
			5	SS	PM														
174.2																			
5.3	SAND and GRAVEL, some silt Very dense Grey Wet		6	SS	50/0.08														
173.5																			
6.0	Granitic Gneiss (BEDROCK)																		
	Bedrock cored from depths of 6.0 m to 9.1 m		1	RC	REC 92%													RQD = 90%	
	For bedrock coring details refer to Record of Drillhole B201-03		2	RC	REC 100%													RQD = 100%	
170.4																			
9.1	END OF BOREHOLE																		
	NOTES: 1. Water level in open borehole at a depth of 0.8 m below ground surface (Elev. 178.7 m) upon completion of drilling. 2. Water level measurements in Piezometer: Date Depth (m) Elev. (m) 21/03/11 0.0 179.5 24/03/11 0.1 179.4 03/10/12 0.5 179.0																		

PROJECT: 09-1111-6014

RECORD OF DRILLHOLE: B201-03

SHEET 1 OF 1

LOCATION: N 5074874.5 ;E 225117.3

DRILLING DATE:

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: —

DRILL RIG: D25 Bombardier

DRILLING CONTRACTOR: Walker Drilling

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	FLUSH	COLOUR % RETURN										NOTES				
							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.						
							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
6	NW Casing March 21, 2011	Continued from Record of Borehole B201-03 Fresh, pink, grey and black, medium crystalline, slightly porous, very strong GRANITIC GNEISS		173.47 6.00																	
7					1																
8	NQRC March 21, 2011																				8.8 MPa (Axial)
9					2																UCS=127 MPa
9		END OF DRILLHOLE		170.36 9.11																	
10																					
11																					
12																					
13																					
14																					
15																					
16																					

DEPTH SCALE

1 : 50




LOGGED: MR

CHECKED: MAS/TVA

GTA-RCK 018 T:\PROJECTS\2009\09-1111-6014 (URS, HWY 69, HENVEY)\LOG\09-1111-6014.GPJ GAL-MASS.GDT 09/02/14

PROJECT		RECORD OF BOREHOLE		No B201-04		SHEET 1 OF 1		METRIC						
W.P. 09-1111-6014		LOCATION		N 5074886.1 ; E 225112.7		ORIGINATED BY		TT						
DIST		HWY 69		BOREHOLE TYPE		127 mm O.D. Continuous Flight Solid Stem Augers, NW Casing, Wash Boring		COMPILED BY						
DATUM		Geodetic		DATE		March 3 and 4, 2011		CHECKED BY						
								TVA						
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						
180.6	GROUND SURFACE							20 40 60 80 100	20 40 60					
180.9	TOPSOIL													
	CLAYEY SILT, trace to some sand, containing rootlets		1	SS	4			7						
	Stiff													
	Brown and grey		2	SS	17			13						
	Moist													
179.4	Sandy SILT													
179.1	Compact		3A	SS	12			3						
1.5	Brown		3B											
	Moist													
	Silty Clay, trace sand							2						
	Firm to stiff													
178.1	Grey							2						
	Moist													
2.5	Granitic Gneiss (BEDROCK)													
	Bedrock cored from depths of 2.5 m to 5.0 m.		2	RC	REC 100%									RQD = 100%
	For bedrock coring details refer to Record of Drillhole B201-04													
			3	RC	REC 100%									RQD = 100%
175.6	END OF BOREHOLE													
5.0	NOTES:													
	1. Water level in open borehole not measured upon completion of drilling.													
	2. Additional boreholes were advanced 1.5 m south and 0.5 m east of Borehole B201-04 to carry out in-situ vane tests between depths of 0.5 m and 2.6 m.													

PROJECT		RECORD OF BOREHOLE				No B201-05		SHEET 1 OF 1		METRIC							
W.P. 09-1111-6014		LOCATION				N 5074896.5 ; E 225100.8		ORIGINATED BY TT									
DIST		HWY 69		BOREHOLE TYPE		Portable Equipment		COMPILED BY MAS									
DATUM Geodetic		DATE		March 2, 2011		CHECKED BY		TVA									
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									
183.9	GROUND SURFACE																
0.0	TOPSOIL		1	SS	8												
183.4	END OF BOREHOLE SPOON REFUSAL																
0.5	NOTES: 1. Borehole advanced using Portable drilling equipment with a one-third weight hammer. SPT N value shown has been adjusted to reflect value that would be obtained with standard weight hammer. 2. Highly weathered Schist fragments were encountered within the Topsoil. 3. Water level in borehole not noted 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 09/02/14

PROJECT		RECORD OF BOREHOLE		No B201-06		SHEET 1 OF 1		METRIC								
W.P. 09-1111-6014		LOCATION		N 5074900.4 ; E 225099.3		ORIGINATED BY		TT								
DIST		HWY 69		BOREHOLE TYPE		Portable Equipment		COMPILED BY								
AM		DATE		March 2, 2011		CHECKED BY		TVA								
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								
185.4	GROUND SURFACE															
0.0	PEAT (Amorphous)		1	RC	REC											
0.3	Black Moist Cobbles Granitic Gneiss (BEDROCK)		2	RC	REC											
	Bedrock cored from depths of 0.3 m to 3.4 m.															
	For bedrock coring details, refer to Record of Drillhole B201-06.		3	RC	REC											
			4	RC	REC											
182.0	END OF BOREHOLE															
3.4	NOTE: 1. Water level in open borehole at a depth of 0.9 m below ground surface (Elev. 184.5 m) upon completion of drilling.															

PROJECT: 09-1111-6014

RECORD OF DRILLHOLE: B201-06

SHEET 1 OF 1

LOCATION: N 5074900.4 ;E 225099.3

DRILLING DATE:

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: —

DRILL RIG: Portable Equipment

DRILLING CONTRACTOR: OGS Inc

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 m	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	
								TOTAL CORE %	SOLID CORE %												
								888888	888888												
								888888	888888												
		Continued from Record of Borehole B201-06		185.19																	
		COBBLES		0.20	1																
		Fresh, foliated, pink, grey and black, medium crystalline, slightly porous, very strong GRANITIC GNEISS		0.29																	
1					2																
2	BQRC March 2, 2011				3								0.5								
3					4								1.5 1 6								
													1.5 1								
		END OF DRILLHOLE		182.01									1.5 1								
				3.38									1.5 1								
4																					
5																					
6																					
7																					
8																					
9																					
10																					

8.3 MPa

DEPTH SCALE

1 : 50



LOGGED: TT

CHECKED: MAS/TVA

PROJECT		RECORD OF BOREHOLE		No B201-07		SHEET 1 OF 1		METRIC									
W.P. 09-1111-6014		LOCATION		N 5074898.7 ; E 225107.6		ORIGINATED BY		TT									
DIST		HWY 69		BOREHOLE TYPE		Portable Equipment		COMPILED BY									
DATUM		Geodetic		DATE		March 1, 2011		CHECKED BY									
								TVA									
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa					WATER CONTENT (%)			γ	GR SA SI CL
								20 40 60 80 100	20 40 60 80 100	20 40 60	W _p	W	W _L				
184.4	GROUND SURFACE		1	CS	REC												
0.0	TOPSOIL		1	RC	100%												
	Schist (BEDROCK)																
0.4	Granitic Gneiss (BEDROCK)		2	RC	100%		184										
	Bedrock cored from depths of 0.2 m to 5.2 m																
	For bedrock coring details refer to Record of Drillhole B201-07.		3	RC	100%		183										
			4	RC	100%		182										
			5	RC	100%		181										
			6	RC	100%		180										
179.2	END OF BOREHOLE																
5.2	NOTE:																
	1. Water level measurement in Piezometer:																
	Date Depth (m) Elev. (m)																
	04/03/11 0.5 183.9																
	03/10/12 0.5 183.9																

GTA-MTO 001 T:\PROJECTS\2009\09-1111-6014 (URS, HWY 69, HENVEY)\LOG\09-1111-6014.GPJ GAL-GTA.GDT 09/02/14

PROJECT: 09-1111-6014

RECORD OF DRILLHOLE: B201-07

SHEET 1 OF 1

LOCATION: N 5074898.7 ;E 225107.6

DRILLING DATE:

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: —

DRILL RIG: Portable Equipment

DRILLING CONTRACTOR: OGS Inc

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 %			TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jn	K ₁₀	K ₅	K ₁	K _{0.5}	P ₁₀	P ₅		P ₁	P _{0.5}																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																							
		Continued from Record of Borehole B201-07		184.23																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																										</

DEPTH SCALE

1 : 50



LOGGED: TT

CHECKED: MAS/AM

GTA-RCK 018 T:\PROJECTS\2009\09-1111-6014 (URS, HWY 69, HENVEY)\LOG\09-1111-6014.GPJ GAL-MISS.GDT 09/02/14

PROJECT		RECORD OF BOREHOLE		No B201-08		SHEET 1 OF 1		METRIC							
W.P. 09-1111-6014		LOCATION		N 5074896.9 ; E 225115.3		ORIGINATED BY		TT							
DIST		HWY 69		BOREHOLE TYPE		Portable Equipment		COMPILED BY							
DATUM		Geodetic		DATE		March 3, 2011		CHECKED BY							
								TVA							
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							
183.4	GROUND SURFACE														
0.0	PEAT, trace sand (Amorphous)		1	SS	20/0.15										
182.9	Black Moist		1	RC	REC 42%										
0.5	Boulders														
	Granitic Gneiss (BEDROCK)														
	Bedrock cored from depths of 0.2 m to 4.0 m.		2	RC	REC 100%										RQD = 71%
	For bedrock coring details, refer to Record of Drillhole B201-08.		3	RC	REC 100%										RQD = 100%
			4	RC	REC 100%										RQD = 97%
179.4	END OF BOREHOLE														
4.0	NOTES:														
	1. Borehole advanced using Portable drilling equipment with a one-third weight hammer. SPT N value shown has been adjusted to reflect value that would be obtained with standard weight hammer.														
	2. Water level in open borehole at a depth of 0.9 m below ground surface (Elev. 182.5 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 09/02/14

SHEET 1 OF 1

DATUM: Geodetic

DRILLING CONTRACTOR: Walker Drilling

CHECKED: MAS/TVA

PROJECT 09-1111-6014		RECORD OF BOREHOLE No B201-09				SHEET 1 OF 1		METRIC								
W.P. 5140-05-01		LOCATION N 5074900.8 ; E 225113.7				ORIGINATED BY TT										
DIST _____ HWY 69		BOREHOLE TYPE Hand Excavation				COMPILED BY MAS										
DATUM Geodetic		DATE March 1, 2011				CHECKED BY TVA										
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								
185.5	GROUND SURFACE															
8.9	TOPSOIL															
	END OF EXCAVATION - BEDROCK															
	NOTE: 1. Hand digging carried out at proposed borehole location to expose bedrock.															

GTA-MTO 001 T:\PROJECTS\2009\09-1111-6014 (URS, HWY 69, HENVEY)\LOG\09-1111-6014.GPJ GAL-GTA.GDT 09/02/14

PROJECT <u>09-1111-6014</u>		RECORD OF BOREHOLE No B201-10		SHEET 1 OF 1		METRIC											
W.P. <u>5140-05-01</u>		LOCATION <u>N 5074917.2 ; E 225100.0</u>		ORIGINATED BY <u>TT</u>													
DIST <u> </u> HWY <u>69</u>		BOREHOLE TYPE <u>Hand Excavation</u>		COMPILED BY <u>MAS</u>													
DATUM <u>Geodetic</u>		DATE <u>March 1, 2011</u>		CHECKED BY <u>TVA</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
192.2	GROUND SURFACE						20	40	60	80	100						
8.9	TOPSOIL																
	END OF EXCAVATION - BEDROCK																
	NOTE: 1. Hand digging carried out at proposed borehole location to expose bedrock.																

GTA-MTO 001 T:\PROJECTS\2009\09-1111-6014 (URS, HWY 69, HENVEY)\LOG\09-1111-6014.GPJ GAL-GTA.GDT 09/02/14

PROJECT		RECORD OF BOREHOLE		No B201-11		SHEET 1 OF 1		METRIC						
W.P. 09-1111-6014		LOCATION		N 5074876.3 ; E 225109.0		ORIGINATED BY		MR						
DIST		HWY 69		BOREHOLE TYPE		127 mm O.D. Continuous Flight Solid Stem Augers, NW Casing, Wash Boring		COMPILED BY						
DATUM		Geodetic		DATE		March 21 and 22, 2011		CHECKED BY						
								TVA						
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						
179.5	GROUND SURFACE													
0.0	TOPSOIL		1A	SS	3									
0.2	Silty SAND Very loose Brown Wet		1B											
178.6			2A											
0.9	SAND, trace gravel Loose Brown Wet		2B	SS	7									
178.1														
1.4	CLAY Soft Brown to grey Moist Becoming grey at a depth 2.4 m		3	SS	WH									
			4	SS	WH									
175.4	Granitic Gneiss (BEDROCK)													
4.1	Bedrock cored from depths of 4.1 m to 7.1 m For bedrock coring details refer to Record of Drillhole B201-11		1	RC	REC 97%									
			2	RC	REC 93%									
172.4	END OF BOREHOLE													
7.1	NOTE: 1. Water level in open borehole at a depth of 0.8 m below ground surface (Elev. 178.7 m) upon completion of drilling.													

PROJECT: 09-1111-6014

RECORD OF DRILLHOLE: B201-11

SHEET 1 OF 1

LOCATION: N 5074876.3 ;E 225109.0

DRILLING DATE:

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: —

DRILL RIG: D25 Bombardier

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																																																																																																																																																																														
								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																																																																																																																																																																											
								80 60 40 20 0	80 60 40 20 0					0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00				0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00	0.25 0.20 0.15 0.10 0.05 0.00

DEPTH SCALE

1 : 50



LOGGED: MR

CHECKED: MAS/TVA

GTA-RCK 018 T:\PROJECTS\2009\09-1111-6014 (URS, HWY 69, HENVEY)\LOG\09-1111-6014.GPJ GAL-MISS.GDT 09/02/14

PROJECT		RECORD OF BOREHOLE		No B201-12		SHEET 1 OF 1		METRIC						
W.P. 09-1111-6014		LOCATION		N 5074872.7 ; E 225124.9		ORIGINATED BY		MR						
DIST		HWY 69		BOREHOLE TYPE		127 mm O.D. Continuous Flight Solid Stem Augers, NW Casing, Wash Boring		COMPILED BY						
DATUM		Geodetic		DATE		March 22 and 23, 2011		CHECKED BY						
								TVA						
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					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						
179.6	GROUND SURFACE						20	40	60	80	100			
0.0	TOPSOIL		1A		3									
0.2	Silty SAND, containing rootlets to a depth of 0.5 m Very loose to loose Brown Wet		1B	SS										
178.2			2	SS	8									
1.4	SILTY CLAY, trace sand Soft Brown to grey Moist Becoming grey at a depth 2.3 m		3	SS	1									
			4	TO	PM									
			5	SS	WH									
174.6														
5.0	Sandy SILT Compact Grey Wet		6	SS	11									
173.8			7	SS	25/0.0									
5.9	SAND, some silt Compact Grey Wet Granitic Gneiss (BEDROCK) Bedrock cored from depths of 5.9 m to 8.8 m For bedrock coring details, refer to Record of Drillhole B201-12		1	RC	REC 96%									
			2	RC	REC 100%									
170.8														
8.8	END OF BOREHOLE													
	NOTE: 1. Water level in open borehole at a depth of 0.9 m below ground surface (Elev. 178.7 m) upon completion of drilling.													

SHEET 1 OF 1

DATUM: Geodetic

DRILLING CONTRACTOR: Walker Drilling

CHECKED: MAS/TVA

PROJECT 09-1111-6014		RECORD OF BOREHOLE No B201-13		SHEET 1 OF 1		METRIC															
W.P. 5140-05-01		LOCATION N 5074856.9; E 225131.2		ORIGINATED BY MR																	
DIST HWY 69		BOREHOLE TYPE 127 mm O.D. Continuous Flight Solid Stem Augers, NW Casing, Wash Boring		COMPILED BY MAS																	
DATUM Geodetic		DATE March 24, 2011		CHECKED BY TVA																	
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		
178.7	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	TOPSOIL		1A	SS	2		178														
0.2	Silty SAND, containing rootlets, organics and wood fragments Very loose to loose Brown Moist to wet Becoming wet below a depth of 0.8 m		1B	SS																	
			2	SS	8																
			3	SS	9		177														
176.5	SAND, trace gravel, containing wood fragments Loose Grey Wet		4	SS	4		176														
			5	SS	5																
175.3	SILT, some clay, trace sand, containing silty clay interlayers Very loose Grey Wet		6	SS	2		175														
174.5	CLAY, some silt, trace sand Firm to stiff Grey Moist		7	SS	WH		174														
4.2			8	SS	1		173														
			9	SS	3		171														
170.9	Granitic Gneiss (BEDROCK)		1	RC	REC 99%		170														
7.8	Bedrock cored from depths of 7.8 m to 10.8 m For Bedrock coring details refer to Record of Drillhole B201-13		2	RC	REC 97%		169														
167.9	END OF BOREHOLE						168														
10.8	NOTES: 1. Unable to push N vane past a depth of 6.9 m below ground surface (Elev. 171.8 m). 2. Water level in open borehole at a depth of 0.7 m below ground surface (Elev. 178.0 m) upon completion of drilling.																				

PROJECT		RECORD OF BOREHOLE		No B201-14		SHEET 1 OF 1		METRIC									
W.P. 09-1111-6014		LOCATION		N 5074860.4 ; E 225115.2		ORIGINATED BY		MR									
DIST		HWY 69		BOREHOLE TYPE		127 mm O.D. Continuous Flight Solid Stem Augers, NW Casing, Wash Boring		COMPILED BY									
DATUM		Geodetic		DATE		March 25, 2011		CHECKED BY									
								TVA									
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									
179.0	GROUND SURFACE																
0.0	TOPSOIL		1A	SS	4												
178.7			1B														
0.3	Silty SAND, containing rootlets and organics Loose Brown to grey Wet		2	SS	9												
177.6																	
1.4	SAND, trace to some silt, trace clay Loose Brown Wet		3	SS	5												0 92 7 1
176.7																	
2.3	SILTY CLAY, trace sand Firm Grey Moist		4	SS	WH												
			5	TO	PM												
			6	SS	1												0 1 41 58
172.4																	
6.6	Granitic Gneiss (BEDROCK)																
	Bedrock cored from depths of 6.6 m to 9.7 m		1	NQ RC	REC 100%												RQD = 92%
	For bedrock coring details refer to Record of Drillhole B201-14																
			2	NQ RC	REC 98%												RQD = 77%
169.3																	
9.7	END OF BOREHOLE																
	NOTE: 1. Water level in open borehole at a depth of 1.2 m below ground surface (Elev. 177.8 m) upon completion of drilling.																

PROJECT <u>09-1111-6014</u>		RECORD OF BOREHOLE No S204-09		SHEET 1 OF 4		METRIC	
W.P. <u>5404-05-01</u>		LOCATION <u>N 5074804.5 ;E 225144.9</u>		ORIGINATED BY <u>MR</u>			
DIST <u> </u> HWY <u>69</u>		BOREHOLE TYPE <u>127 mm O.D. Continuous Flight Solid Stem Augers, NW Casing, Wash Boring</u>		COMPILED BY <u>OK</u>			
DATUM <u>Geodetic</u>		DATE <u>February 3 and 4, 2010</u>		CHECKED BY <u>TVA</u>			

[illegible]

GT-A-MTO 001 09-1111-6014.GPJ GAL-MISS.GDT 7/25/12 SAC/DD

Continued Next Page

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



PROJECT		RECORD OF BOREHOLE		No S204-09		SHEET 2 OF 4		METRIC												
W.P.		LOCATION		ORIGINATED BY																
DIST		BOREHOLE TYPE		COMPILED BY																
DATUM		DATE		CHECKED BY																
09-1111-6014		N 5074804.5 ; E 225144.9		MR																
5404-05-01		127 mm O.D. Continuous Flight Solid Stem Augers, NW Casing, Wash Boring		OK																
Geodetic		February 3 and 4, 2010		TVA																
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 kN/m ³	GR SA SI CL			
								20 40 60 80 100	20 40 60	20 40 60	20 40 60									
--- CONTINUED FROM PREVIOUS PAGE ---																				
164.5	Sandy SILT, trace clay Compact Grey Wet		13	SS	19		166													0 29 68 3
16.6	SILTY CLAY Stiff Grey Moist						165													
			14	SS	7		164													
							163													
							162													
161.1	SAND and SILT, trace clay Loose Grey Moist						161													
20.0			15A	SS	4		160													
			15B				159													
							158													
158.4	SILTY CLAY Stiff to very stiff Grey Moist						157													
22.7			16	SS	5		156													
							155													
							154													
	Sandy silt interlayers encountered between depths of 27.0 m and 27.6 m		17	SS	5		153													
							152													

Continued Next Page

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

GTA-MTO 001 09-1111-6014.GPJ GAL-MASS.GDT 7/25/12 SAC/DD

PROJECT <u>09-1111-6014</u>		RECORD OF BOREHOLE No S204-09		SHEET 3 OF 4		METRIC	
W.P. <u>5404-05-01</u>		LOCATION <u>N 5074804.5 ; E 225144.9</u>		ORIGINATED BY <u>MR</u>			
DIST <u> </u> HWY <u>69</u>		BOREHOLE TYPE <u>127 mm O.D. Continuous Flight Solid Stem Augers, NW Casing, Wash Boring</u>		COMPILED BY <u>OK</u>			
DATUM <u>Geodetic</u>		DATE <u>February 3 and 4, 2010</u>		CHECKED BY <u>TVA</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/m³	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 ● QUICK TRIAXIAL × REMOULDED						
	--- CONTINUED FROM PREVIOUS PAGE ---						20 40 60 80 100		20 40 60					
	SILTY CLAY Stiff to very stiff Grey Moist		18	SS	6		151							
							150							
							149							
							148							
			19	SS	7		147							
							146							
							145							
			20	SS	7		144							
							143							
							142							
	SILT, trace to some clay, trace sand Dense Grey Wet		21	SS	9		141						0 2 92 6	
							140							
							139							
							138							
							137							
138.7 42.4			22	TO	PH									
			23	SS	42									
136.8 44.3														

Continued Next Page

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

GTA-MTO 001 09-1111-6014.GPJ GAL-MISS.GDT 7/25/12 SAC/DD

PROJECT <u>09-1111-6014</u>		RECORD OF BOREHOLE No S204-09		SHEET 4 OF 4		METRIC	
W.P. <u>5404-05-01</u>		LOCATION <u>N 5074804.5 ; E 225144.9</u>		ORIGINATED BY <u>MR</u>			
DIST <u> </u> HWY <u>69</u>		BOREHOLE TYPE <u>127 mm O.D. Continuous Flight Solid Stem Augers, NW Casing, Wash Boring</u>		COMPILED BY <u>OK</u>			
DATUM <u>Geodetic</u>		DATE <u>February 3 and 4, 2010</u>		CHECKED BY <u>TVA</u>			

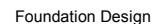
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	20	40	60	80	100	W _p	W		
135.2	Silty SAND, trace clay Compact Grey Wet		24	SS	27	136										
45.9	END OF BOREHOLE NOTE: 1. Water level in open borehole at a depth of 3.4 m below ground surface (Elev. 177.7 m) upon completion of drilling.															

GTA-MTO 001 09-1111-6014.GPJ GAL-MISS.GDT 7/25/12 SAC/DD

PROJECT 09-1111-6014				RECORD OF BOREHOLE No SP1				SHEET 1 OF 1				METRIC														
W.P. 5139-08-01				LOCATION N 5074819.8 ; E 225159.0				ORIGINATED BY MR																		
DIST HWY 69				BOREHOLE TYPE 213 mm O.D. Continuous Flight Hollow Stem Augers				COMPILED BY TVA																		
DATUM Geodetic				DATE February 17, 2011				CHECKED BY TVA																		
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT			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					W _p — W — W _L						WATER CONTENT (%)							
181.1	GROUND SURFACE																									
0.0	TOPSOIL																									
180.8																										
0.3	Sandy SILT, some clay Brown																									
180.2																										
0.9	SILTY CLAY Grey																									
178.7																										
2.4	SAND to Silty SAND Brown to grey																									
175.0																										
6.1	END OF BOREHOLE																									
NOTES: 1. General stratigraphy interpretations are based on cuttings and auger samples only. Soil types and boundaries shown between soil types are approximate. 2. Water level measurement in Piezometer: <table border="1" style="width:100%; border-collapse: collapse;"> <thead> <tr> <th>Date</th> <th>Depth (m)</th> <th>Elev. (m)</th> </tr> </thead> <tbody> <tr> <td>17/02/11</td> <td>4.0</td> <td>177.1</td> </tr> <tr> <td>27/02/11</td> <td>4.0</td> <td>177.1</td> </tr> </tbody> </table>																		Date	Depth (m)	Elev. (m)	17/02/11	4.0	177.1	27/02/11	4.0	177.1
Date	Depth (m)	Elev. (m)																								
17/02/11	4.0	177.1																								
27/02/11	4.0	177.1																								

PROJECT 09-1111-6014					RECORD OF DCPT No B201-DC01					SHEET 1 OF 1					METRIC				
W.P. 5140-05-01					LOCATION N 5074872.3 ; E 225110.6					ORIGINATED BY MR									
DIST _____ HWY 69					BOREHOLE TYPE Dynamic Cone Penetration Test					COMPILED BY MAS									
DATUM Geodetic					DATE March 22, 2011					CHECKED BY TVA									
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT			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				
179.4 0.0	GROUND SURFACE Dynamic Cone Penetration Test (DCPT)						179												
							178												
							177												
							176												
							175												
							174												
173.5 5.9	END OF DCPT Refusal to Further Penetration (Hammer Bouncing)																		

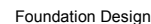
GTA-MTO 001 T:\PROJECTS\2009\09-1111-6014 (URS, HWY 69, HENVEY)\LOG\09-1111-6014.GPJ GAL-GTA.GDT 09/02/14



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

PROJECT <u>09-1111-6014</u>										RECORD OF DCPT No B201-DC03 SHEET 1 OF 1										METRIC			
W.P. <u>5140-05-01</u>					LOCATION <u>N 5074860.8 ; E 225129.7</u>					ORIGINATED BY <u>MR</u>													
DIST <u> </u> HWY <u>69</u>					BOREHOLE TYPE <u>Dynamic Cone Penetration Test</u>					COMPILED BY <u>MAS</u>													
DATUM <u>Geodetic</u>					DATE <u>March 25, 2011</u>					CHECKED BY <u>TVA</u>													
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT			LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)					
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					W _p	W									
178.3	GROUND SURFACE							20	40	60	80	100											
0.0	Dynamic Cone Penetration Test (DCPT)						178																
							177																
							176																
							175																
							174																
172.9	END OF DCPT						173																
5.4	Refusal to Further Penetration (Hammer Bouncing)																						

GTA-MTO 001 T:\PROJECTS\2009\09-1111-6014 (URS, HWY 69, HENVEY)\LOG\09-1111-6014.GPJ GAL-GTA.GDT 09/02/14



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



APPENDIX B

Laboratory Test Results and Cobbles/Boulders and Bedrock Core Photographs

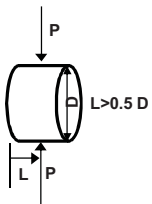
TABLE B1
POINT LOAD TEST RESULTS ON ROCK SAMPLES

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)
B201-02	1	6.8	171.8	Granite Gneiss	Axial	44.44	47.41	7.677	146
B201-02	2	8.0	170.6	Granite Gneiss	Diametral	52.86	44.64	6.926	132
B201-03	2	7.7	171.8	Granite Gneiss	Axial	43.64	47.49	8.837	168
B201-03	2	8.5	171.0	Granite Gneiss	Diametral	51.63	43.74	7.213	137
B201-04	2	3.0	177.6	Granite Gneiss	Diametral	61.38	48.20	7.209	137
B201-04	3	4.2	176.4	Granite Gneiss	Axial	45.55	50.72	6.238	119
B201-06	3	1.5	183.9	Granite Gneiss	Diametral	51.15	45.75	8.277	157
B201-07	2	1.0	183.4	Granite Gneiss	Axial	37.85	50.83	6.703	127
B201-07	4	2.5	181.9	Granite Gneiss	Diametral	46.13	41.21	12.844	244
B201-07	6	4.5	179.9	Granite Gneiss	Axial	50.58	50.70	8.954	170
B201-08	3	2.0	181.4	Granite Gneiss	Diametral	54.13	46.64	7.206	137
B201-08	4	3.5	179.9	Granite Gneiss	Axial	44.89	51.11	7.025	133
B201-11	1	5.0	174.5	Granite Gneiss	Axial	40.81	47.37	7.676	146
B201-11	2	6.0	173.5	Granite Gneiss	Diametral	61.06	41.61	10.677	203
B201-12	2	8.0	171.6	Granite Gneiss	Axial	26.570	47.420	8.022	152
B201-13	1	8.5	170.2	Granite Gneiss	Axial	54.17	47.44	8.490	161
B201-13	2	9.9	168.8	Granite Gneiss	Diametral	61.31	43.36	14.433	274
B201-14	1	7.2	171.8	Granite Gneiss	Diametral	72.65	38.29	12.650	240
B201-14	2	8.9	170.1	Granite Gneiss	Axial	37.90	46.90	8.848	168

⁽¹⁾ $I_{S50} \times K$, 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 $K = 19$ has been used and is based on the average of six (6) I_{S50} tests and the average of 3 UCS tests, for similar bedrock core zones.

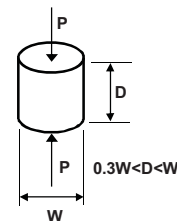
DIAMETRAL SPECIMEN SHAPE REQUIREMENTS

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: TVA
Reviewed by: JPD/JMAC

TABLE B2-1
SUMMARY OF UNIAXIAL COMPRESSIVE STRENGTH TEST RESULTS
STILL RIVER BRIDGE (SBL) STRUCTURE
HIGHWAY 69 GWP 5404-05-00; WP 5140-08-01

Borehole Number (Core Run)	Sample Depth (m)	Sample Elevation (m)	Rock Type	Core Diameter (mm)	Uniaxial Compressive Strength (MPa)
B201-02 (2)	8.2	170.4	Granitic Gneiss	47.5	140.6
B201-03 (2)	8.7	170.8	Granitic Gneiss	47.3	126.6
B201-08 (3)	1.8	181.6	Granitic Gneiss	50.5	168.8

Compiled By: TVAReviewed By: JPD/JMAC

TABLE B2-2
UNCONFINED COMPRESSION (UC) TEST
ASTM D 7012-10

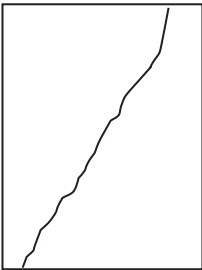
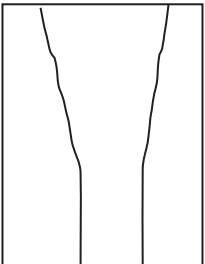
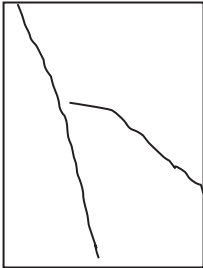
SAMPLE IDENTIFICATION			
PROJECT NUMBER	09-1111-6014	RUN NUMBER	2
BOREHOLE NUMBER	B201-02	SAMPLE DEPTH, m	8.10-8.25
TEST CONDITIONS			
MACHINE SPEED, mm/min	0.00	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST,min	>2 <15	L/D	2.41
SPECIMEN INFORMATION			
SAMPLE HEIGHT, cm	11.46	WATER CONTENT, (specimen) %	0.09
SAMPLE DIAMETER, cm	4.75	UNIT WEIGHT, kN/m ³	26.76
SAMPLE AREA, cm ²	17.72	DRY UNIT WT., kN/m ³	26.74
SAMPLE VOLUME, cm ³	203.08	SPECIFIC GRAVITY, assumed	2.75
WET WEIGHT, g	554.40	VOID RATIO	0.01
DRY WEIGHT, g	553.90		
VISUAL INSPECTION		FAILURE SKETCH	
			
TEST RESULTS			
STRAIN AT FAILURE, %	-	COMPRESSIVE STRESS, MPa	140.6
REMARKS:	N/A	DATE:	2011-07-14
CHECKED BY:	TVA	REVIEWED BY:	JPD/JMAC

TABLE B2-3
UNCONFINED COMPRESSION (UC) TEST
ASTM D 7012-10

SAMPLE IDENTIFICATION			
PROJECT NUMBER	09-1111-6014	RUN NUMBER	2
BOREHOLE NUMBER	B201-03	SAMPLE DEPTH, m	8.60-8.75
TEST CONDITIONS			
MACHINE SPEED, mm/min	0.00	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST,min	>2 <15	L/D	2.28
SPECIMEN INFORMATION			
SAMPLE HEIGHT, cm	10.80	WATER CONTENT, (specimen) %	0.12
SAMPLE DIAMETER, cm	4.73	UNIT WEIGHT, kN/m ³	27.08
SAMPLE AREA, cm ²	17.57	DRY UNIT WT., kN/m ³	27.05
SAMPLE VOLUME, cm ³	189.77	SPECIFIC GRAVITY, assumed	2.75
WET WEIGHT, g	524.30	VOID RATIO	0.00
DRY WEIGHT, g	523.67		
VISUAL INSPECTION		FAILURE SKETCH	
			
TEST RESULTS			
STRAIN AT FAILURE, %		- COMPRESSIVE STRESS, MPa	
126.6			
REMARKS:	N/A	DATE:	2011-07-14
CHECKED BY:	TVA	REVIEWED BY:	JPD/JMAC

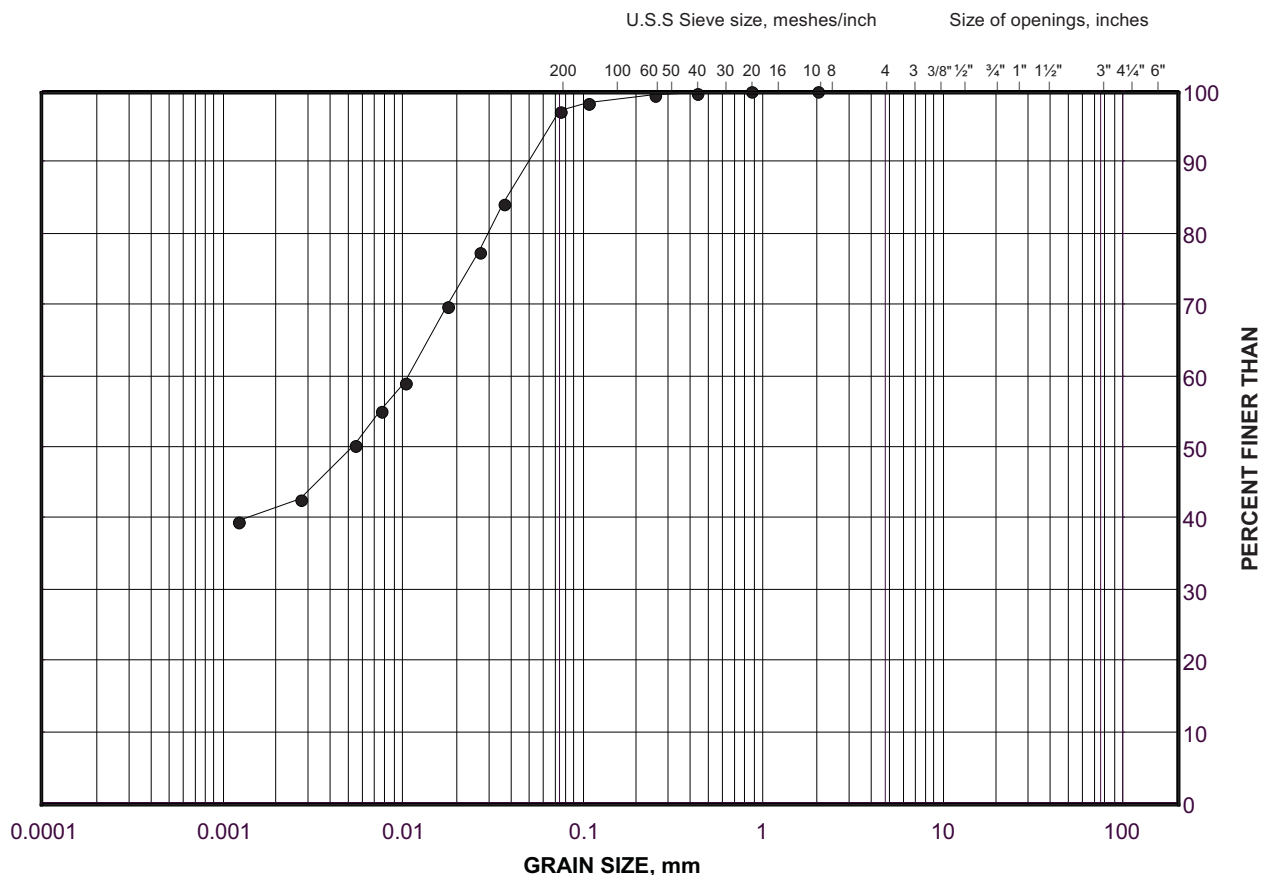
**TABLE B2-4
UNCONFINED COMPRESSION (UC) TEST
ASTM D 7012-10**

SAMPLE IDENTIFICATION			
PROJECT NUMBER	09-1111-6014	RUN NUMBER	3
BOREHOLE NUMBER	B201-08	SAMPLE DEPTH, m	1.75-1.90
TEST CONDITIONS			
MACHINE SPEED, mm/min	-	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST,min	>2 <15	L/D	2.31
SPECIMEN INFORMATION			
SAMPLE HEIGHT, cm	11.65	WATER CONTENT, (specimen) %	0.10
SAMPLE DIAMETER, cm	5.05	UNIT WEIGHT, kN/m ³	27.42
SAMPLE AREA, cm ²	20.03	DRY UNIT WT., kN/m ³	27.39
SAMPLE VOLUME, cm ³	233.35	SPECIFIC GRAVITY, assumed	2.80
WET WEIGHT, g	652.75	VOID RATIO	0.00
DRY WEIGHT, g	652.09		
VISUAL INSPECTION		FAILURE SKETCH	
			
TEST RESULTS			
STRAIN AT FAILURE, %		COMPRESSIVE STRESS, MPa	
-		168.8	
REMARKS:	N/A	DATE:	2011-08-04
CHECKED BY:	TVA	REVIEWED BY:	JPD/JMAC

GRAIN SIZE DISTRIBUTION

Silty Clay (Near Surface)
South Abutment - Still River Bridge (SBL)

FIGURE B1



LEGEND

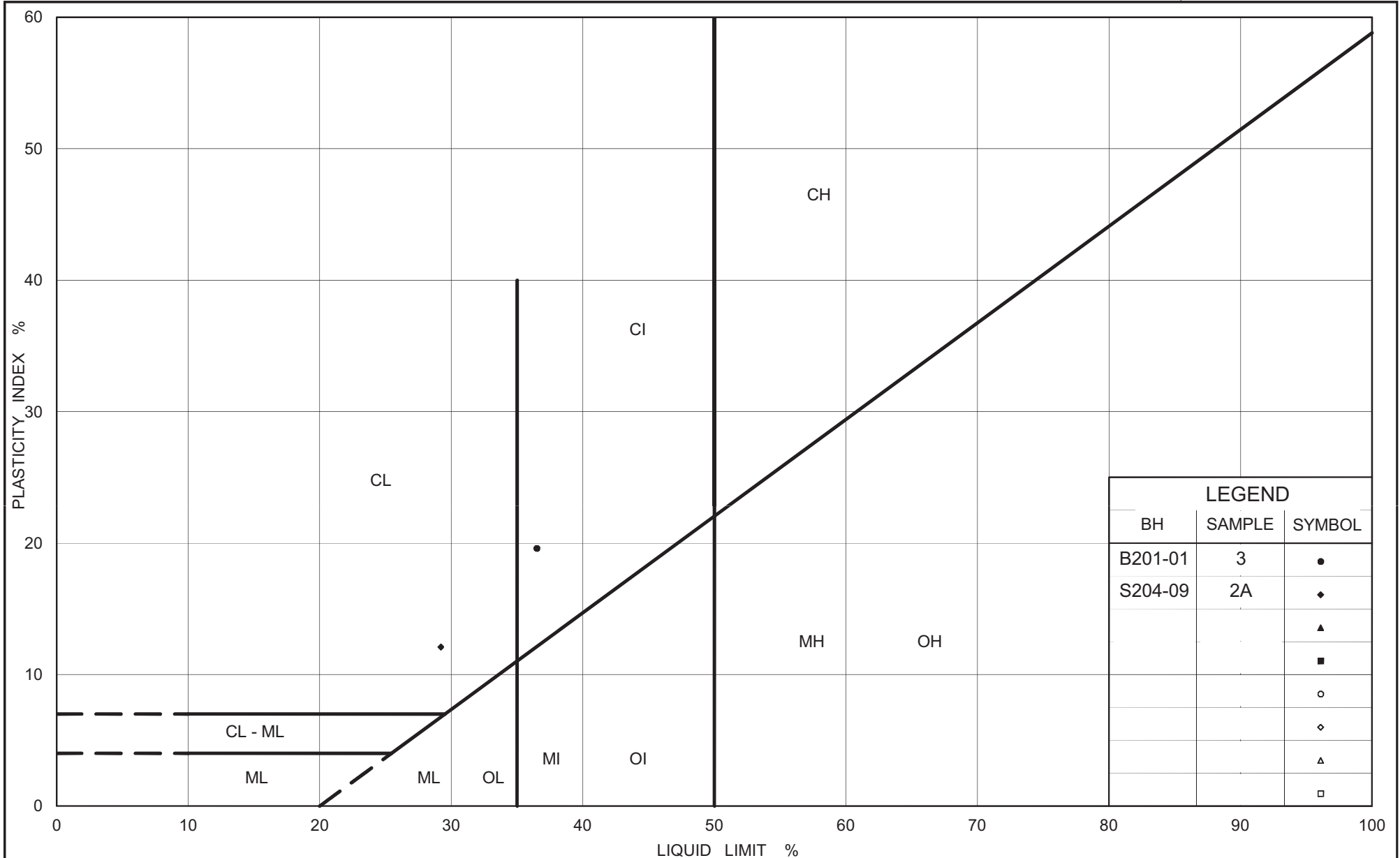
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
•	B201-01	3	179.4

Project Number: 09-1111-6014

Checked By: TZ

Golder Associates

Date: 13-Oct-11



Ministry of Transportation

Ontario

PLASTICITY CHART
 Clayey Silt to Silty Clay (Near Surface)
 South Abutment and Approach - Still River Bridge (SBL)

Figure No. B2

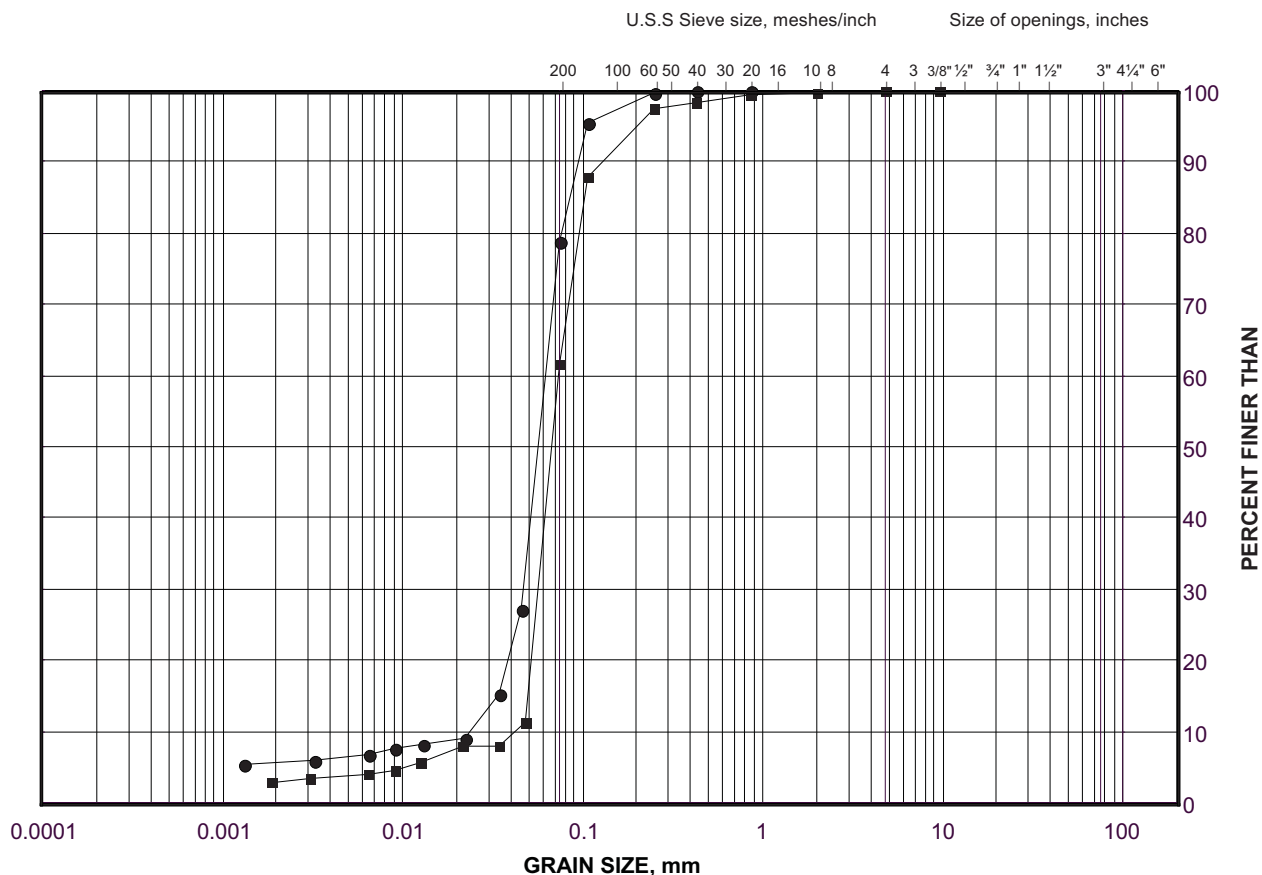
Project No. 09-1111-6014

Checked By: TZ

GRAIN SIZE DISTRIBUTION

Sand and Silt to Sandy Silt (Upper)
South Abutment and Approach - Still River Bridge (SBL)

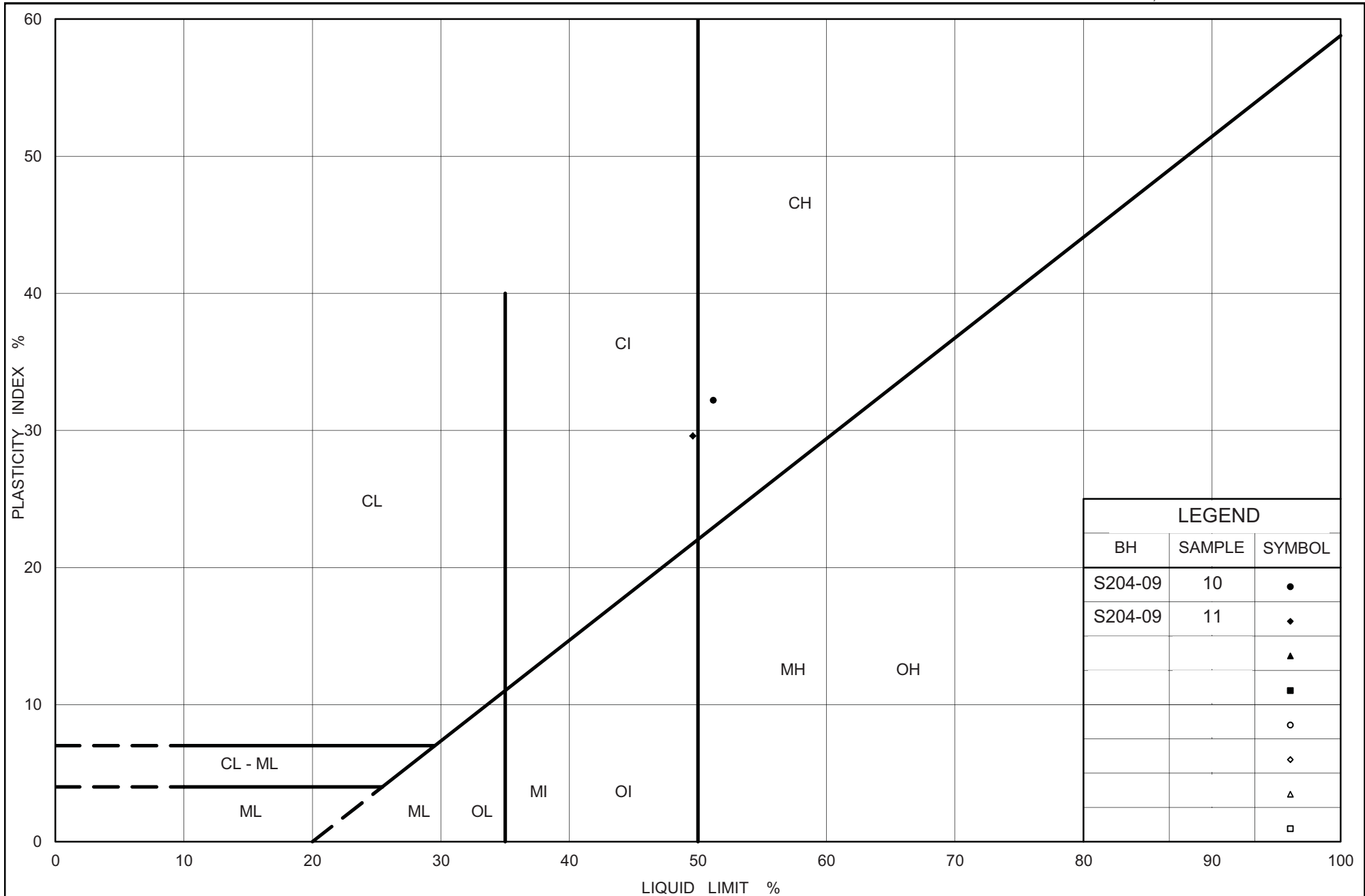
FIGURE B3



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	S204-09	5	177.0
■	B201-01	7	175.0



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Ontario

PLASTICITY CHART
Silty Clay to Clay (Upper)
South Approach - Still River Bridge (SBL)

Figure No. B4

Project No. 09-1111-6014

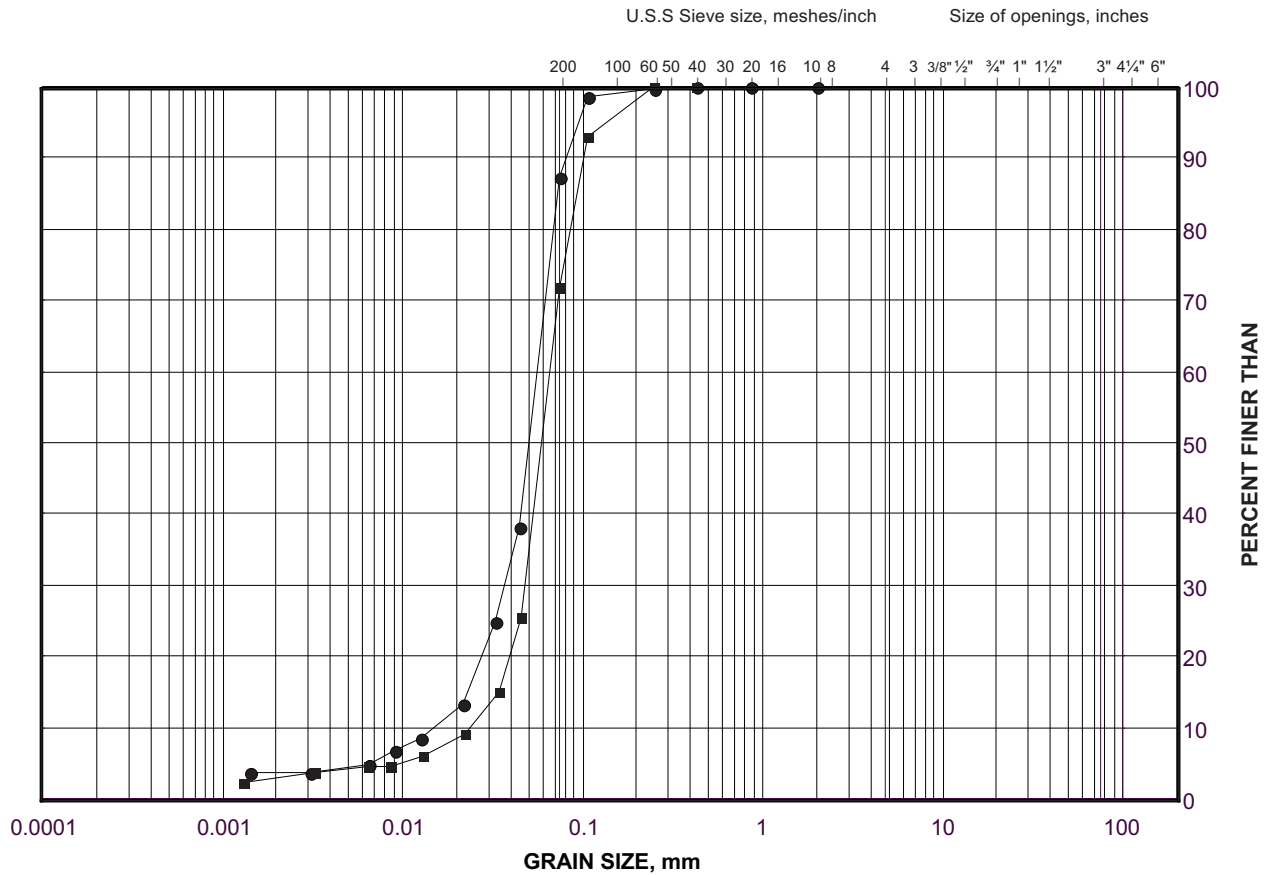
Checked By: TZ

GRAIN SIZE DISTRIBUTION

Silt to Sandy Silt

South Abutment and Approach - Still River Bridge (SBL)

FIGURE B5



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	B201-01	12	167.4
■	S204-09	13	166.0

Project Number: 09-1111-6014

Checked By: TZ

Golder Associates

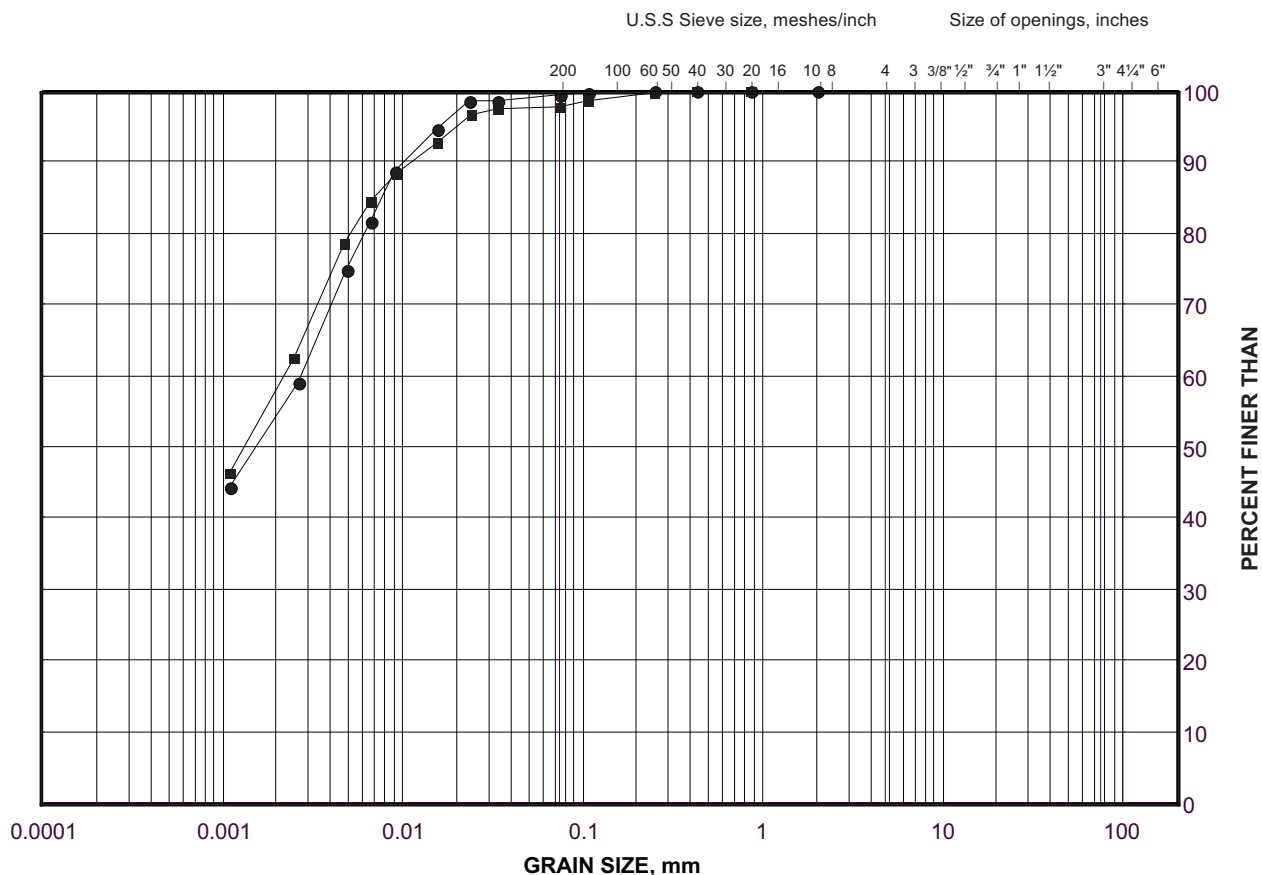
Date: 13-Oct-11

GRAIN SIZE DISTRIBUTION

Silty Clay (Lower)

South Abutment and Approach - Still River Bridge (SBL)

FIGURE B6



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

LEGEND

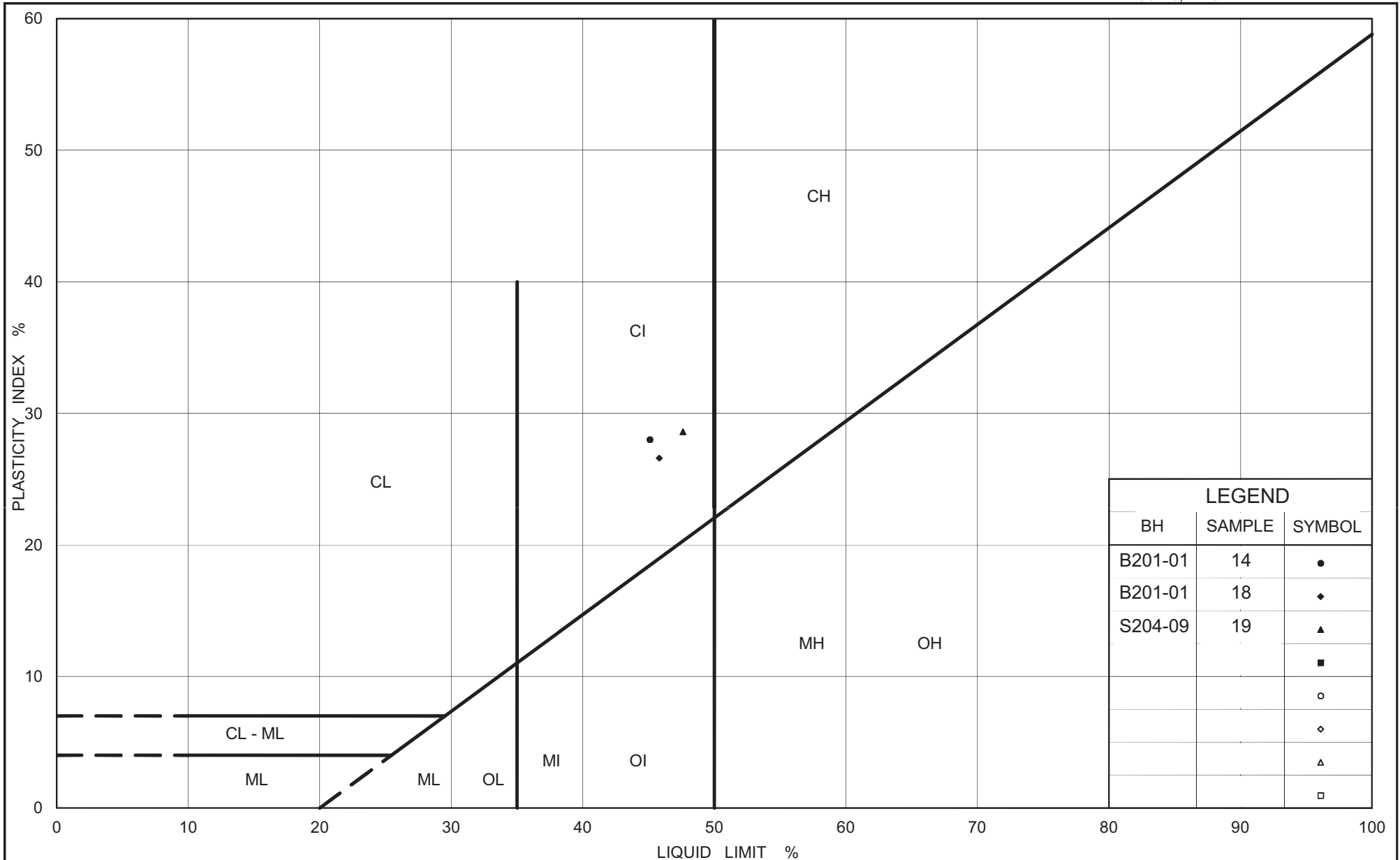
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	B201-01	14	162.8
■	B201-01	18	150.9

Project Number: 09-1111-6014

Checked By: TZ

Golder Associates

Date: 13-Oct-11



Ministry of Transportation

Ontario

PLASTICITY CHART Silty Clay (Lower)

South Abutment and Approach - Still River Bridge (SBL)

Figure No. B7

Project No. 09-1111-6014

Checked By: TZ

CONSOLIDATION TEST SUMMARY**FIGURE B8****Sheet 1 of 4****SAMPLE IDENTIFICATION**

Project Number	09-1111-6014	Sample Number	17
Borehole Number	B201-01	Sample Depth, m	27.1-27.6

TEST CONDITIONS

Test Type	Standard	Load Duration, hr	24
Oedometer Number	3		
Date Started	12/18/2012		
Date Completed	01/02/2013		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	2.53	Unit Weight, kN/m ³	15.69
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m ³	9.34
Area, cm ²	31.68	Specific Gravity, measured	2.81
Volume, cm ³	80.28	Solids Height, cm	0.859
Water Content, %	67.93	Volume of Solids, cm ³	27.22
Wet Mass, g	128.43	Volume of Voids, cm ³	53.06
Dry Mass, g	76.48	Degree of Saturation, %	97.9

TEST COMPUTATIONS

Stress kPa	Corr. Height cm	Void Ratio	Average Height cm	t ₉₀ sec	cv. cm ² /s	mv m ² /kN	k cm/s
0.00	2.534	1.949	2.534				
6.00	2.532	1.947	2.533	2	6.80E-01	1.25E-04	8.33E-06
10.63	2.531	1.946	2.531	290	4.68E-03	1.11E-04	5.09E-08
20.65	2.526	1.940	2.528	265	5.11E-03	2.01E-04	1.01E-07
40.04	2.505	1.916	2.516	540	2.48E-03	4.13E-04	1.01E-07
78.65	2.467	1.871	2.486	437	3.00E-03	3.98E-04	1.17E-07
155.81	2.403	1.796	2.435	463	2.71E-03	3.27E-04	8.71E-08
310.26	2.232	1.598	2.317	1009	1.13E-03	4.36E-04	4.82E-08
619.36	1.953	1.273	2.093	1779	5.22E-04	3.56E-04	1.82E-08
1237.41	1.756	1.044	1.855	1109	6.57E-04	1.26E-04	8.11E-09
2474.60	1.598	0.860	1.677	913	6.53E-04	5.04E-05	3.22E-09
1237.41	1.607	0.870	1.602				
310.26	1.662	0.935	1.635				
78.65	1.734	1.018	1.698				
20.65	1.794	1.088	1.764				
6.00	1.838	1.140	1.816				

Note:

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

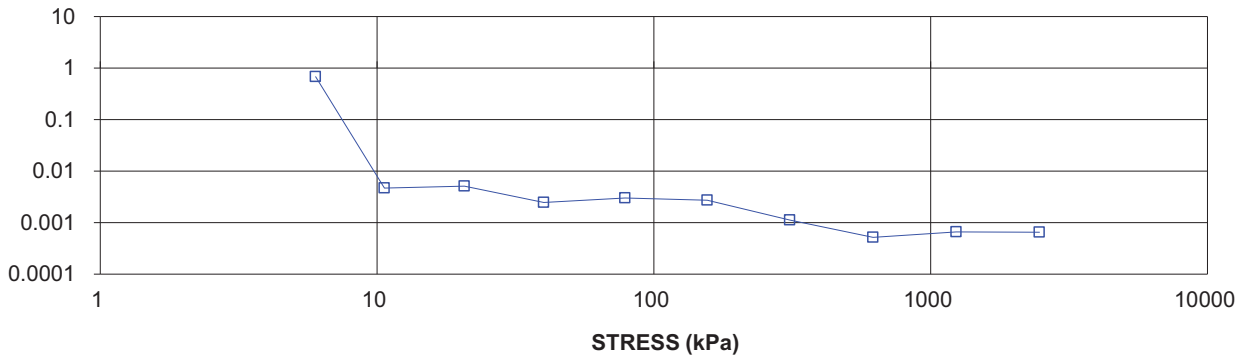
Sample Height, cm	1.84	Unit Weight, kN/m ³	18.24
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m ³	12.88
Area, cm ²	31.68	Specific Gravity, measured	2.81
Volume, cm ³	58.24	Solids Height, cm	0.859
Water Content, %	41.64	Volume of Solids, cm ³	27.22
Wet Mass, g	108.33	Volume of Voids, cm ³	31.02
Dry Mass, g	76.48		

CONSOLIDATION TEST SUMMARY

FIGURE B8
Sheet 2 of 4

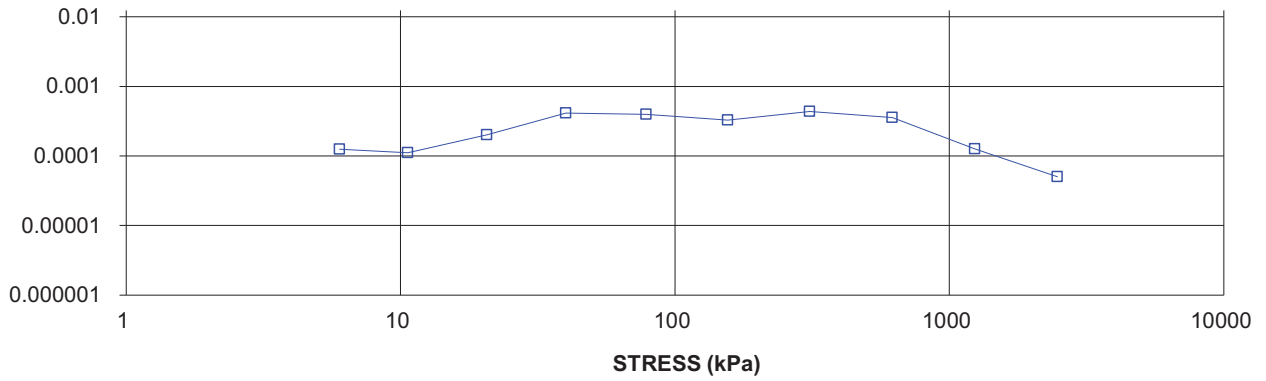
COEFFICIENT OF CONSOLIDATION,
cm²/s

CONSOLIDATION TEST
CV cm²/s VS STRESS (kPa)
BH B201-01 SA 17



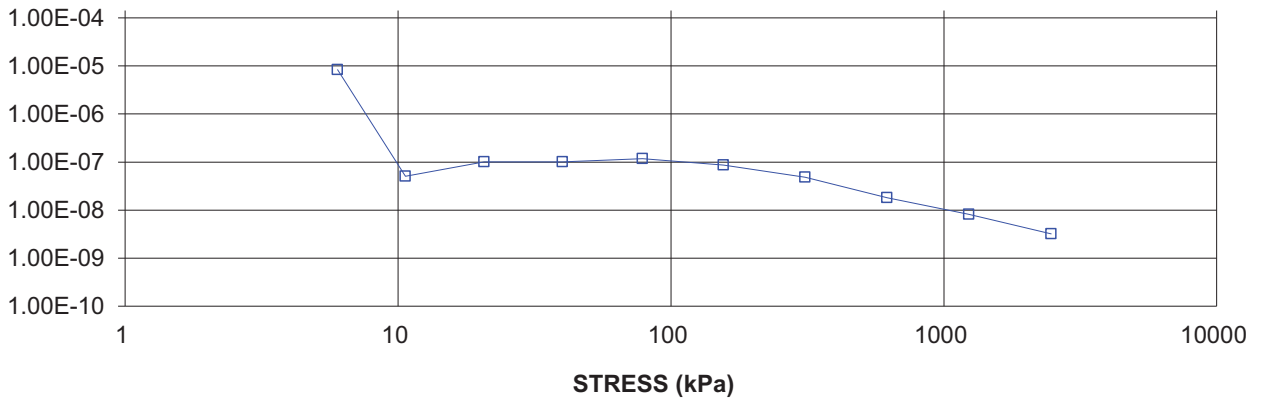
VOLUME COMPRESSIBILITY, m²/kN

CONSOLIDATION TEST
MV m²/kN vs STRESS (kPa)
BH B201-01 SA 17



HYDRAULIC CONDUCTIVITY,
cm/s

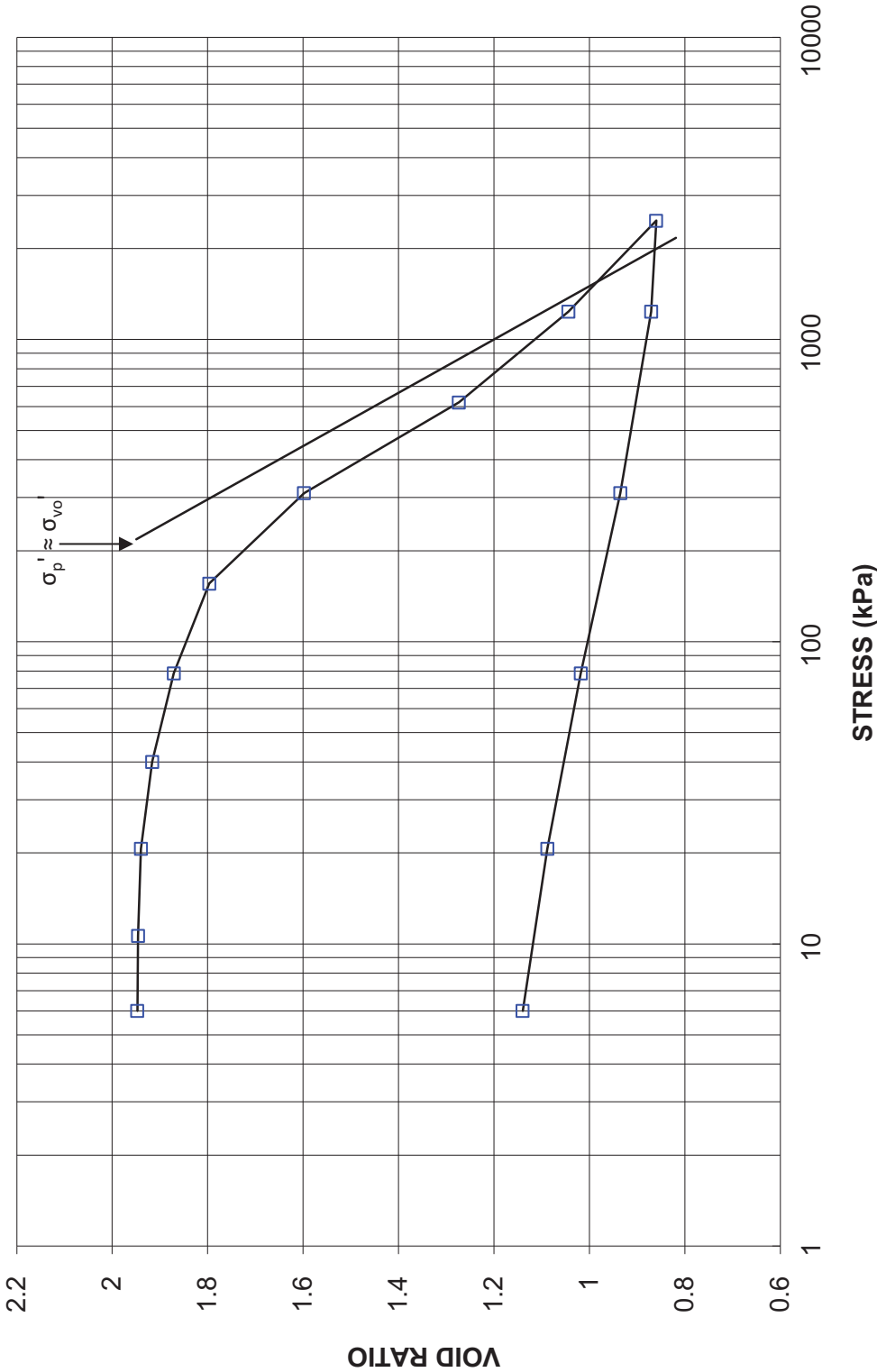
CONSOLIDATION TEST
HYDRAULIC CONDUCTIVITY vs STRESS
BH B201-01 SA 17

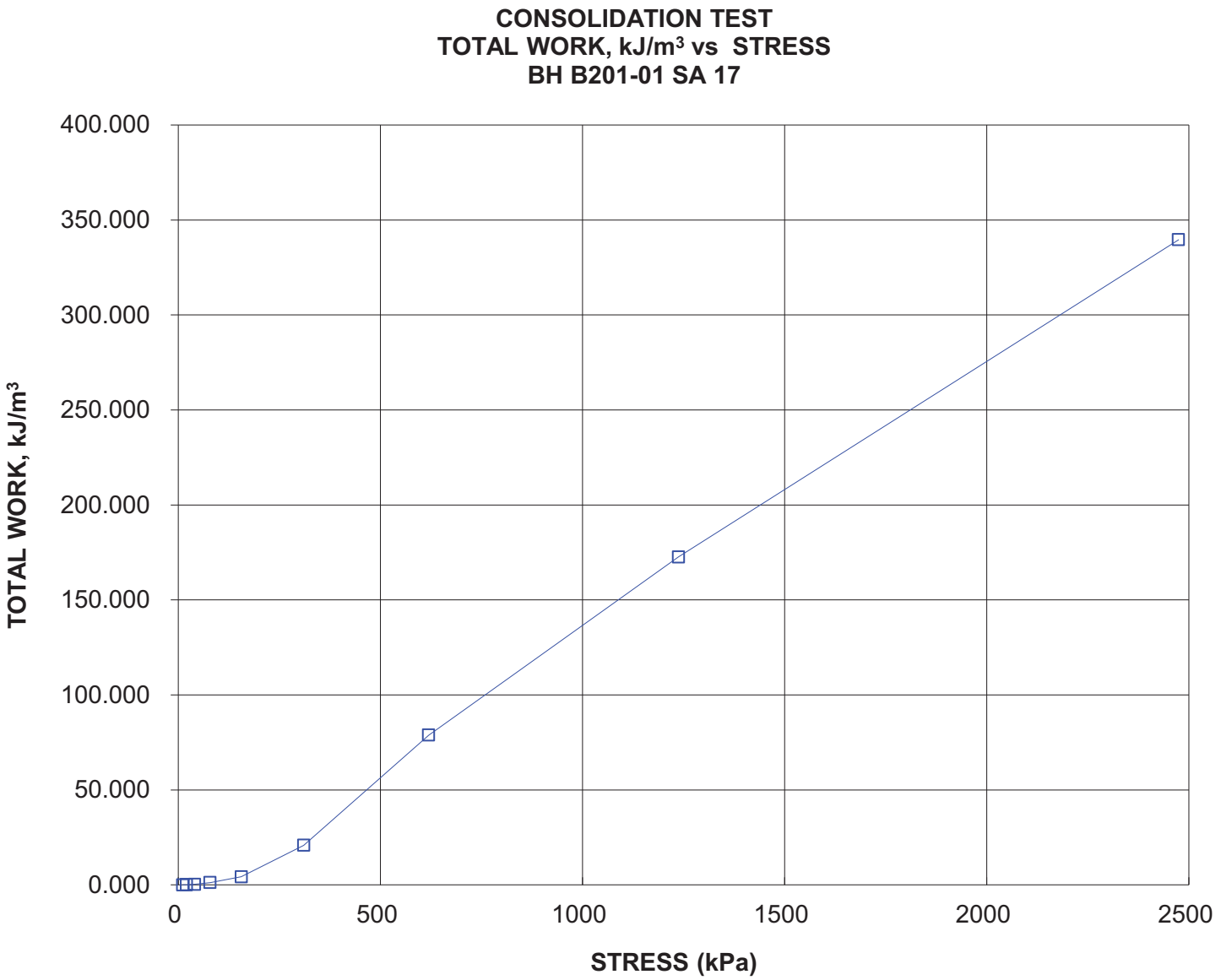


CONSOLIDATION TEST
VOID RATIO VS LOG STRESS

FIGURE B8
Sheet 3 of 4

CONSOLIDATION TEST
VOID RATIO vs STRESS
BH B201-01 SA 17



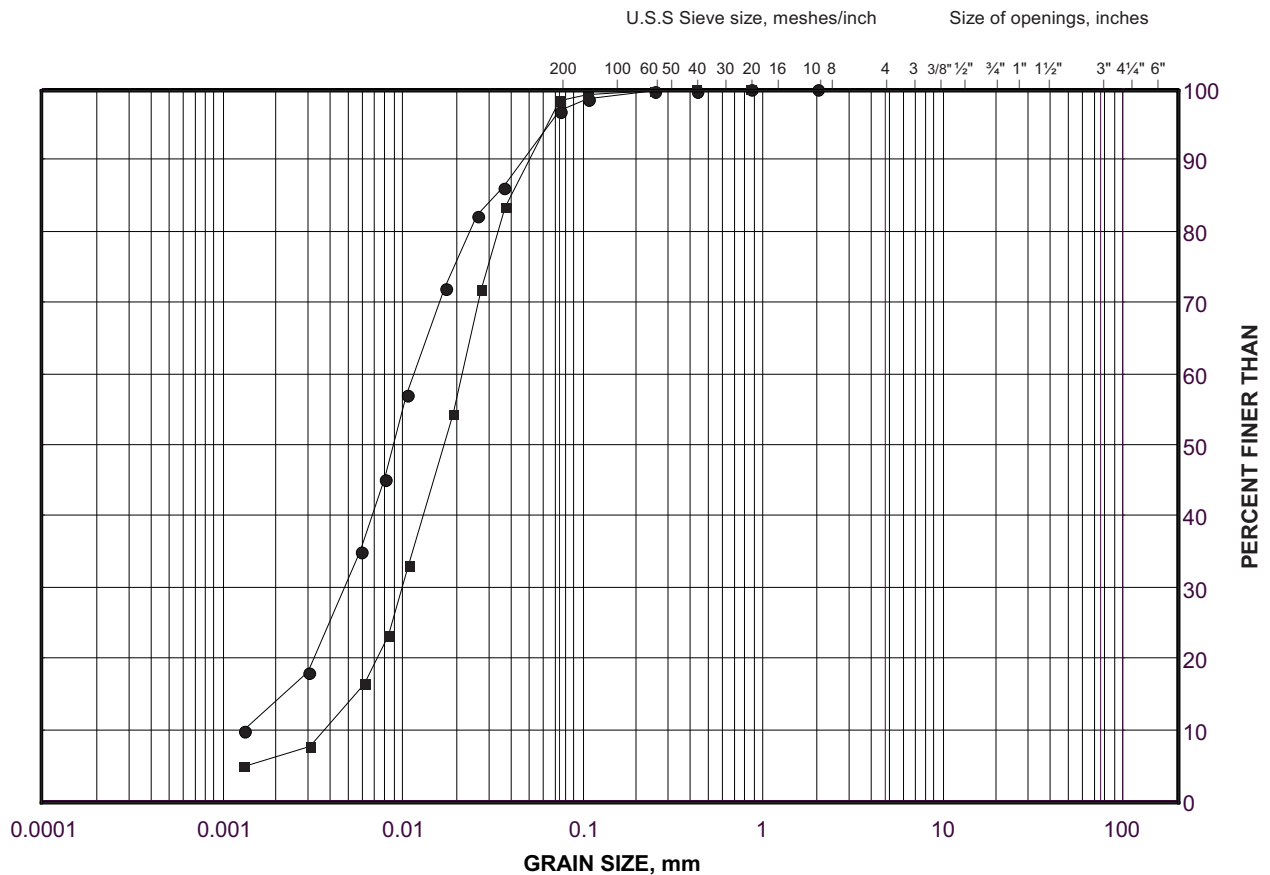


GRAIN SIZE DISTRIBUTION

Silt

South Abutment and Approach - Still River Bridge (SBL)

FIGURE B9



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	B201-01	21	141.8
■	S204-09	23	138.0

Project Number: 09-1111-6014

Checked By: TZ

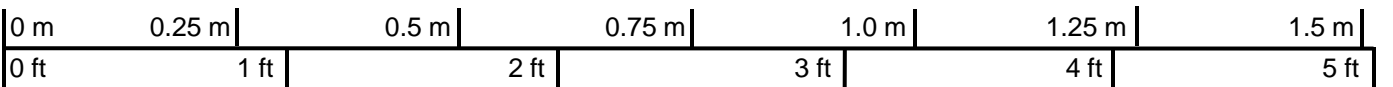
Golder Associates

Date: 13-Oct-11


Borehole B201-01



Box 1: 53.2 m – 57.3 m



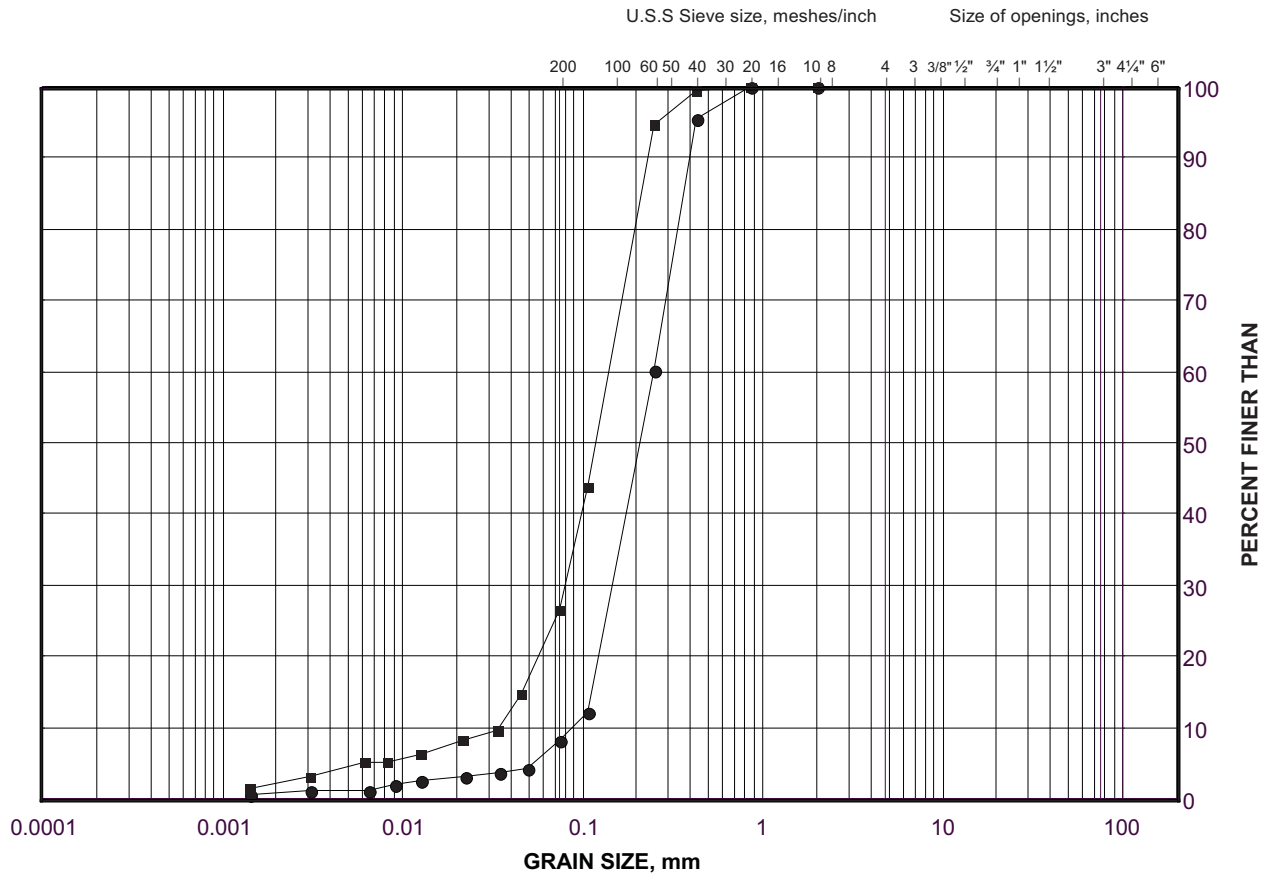
Scale

PROJECT				Still River Bridge SBL Structure Highway 69 Four-Laning GWP 5404-05-00; WP 5140-05-01			
TITLE				Cobbles and Boulder Rock Core Photograph – South Abutment			
				PROJECT No. 09-1111-6014		FILE No. ----	
				DESIGN	AT		SCALE NTS REV.
				CADD	--		
				CHECK	TVA		
				REVIEW	TVA		
FIGURE B10							

GRAIN SIZE DISTRIBUTION

Silty Sand to Sand
Centre Pier - Still River Bridge (SBL)

FIGURE B11A



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	B201-14	3	177.2
■	B201-02	3	176.8

Project Number: 09-1111-6014

Checked By: TZ

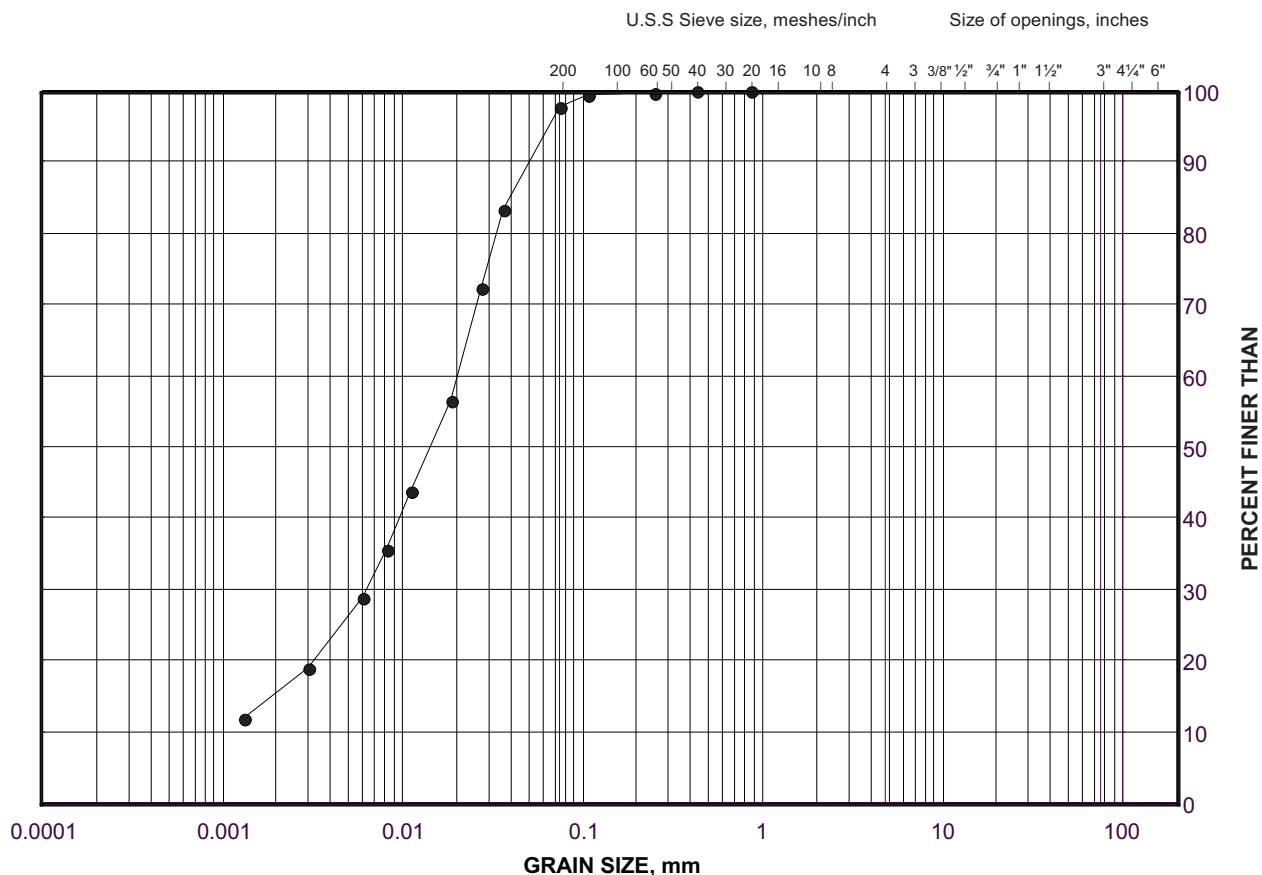
Golder Associates

Date: 13-Oct-11

GRAIN SIZE DISTRIBUTION

Silt
Centre Pier - Still River Bridge (SBL)

FIGURE B11B



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

LEGEND

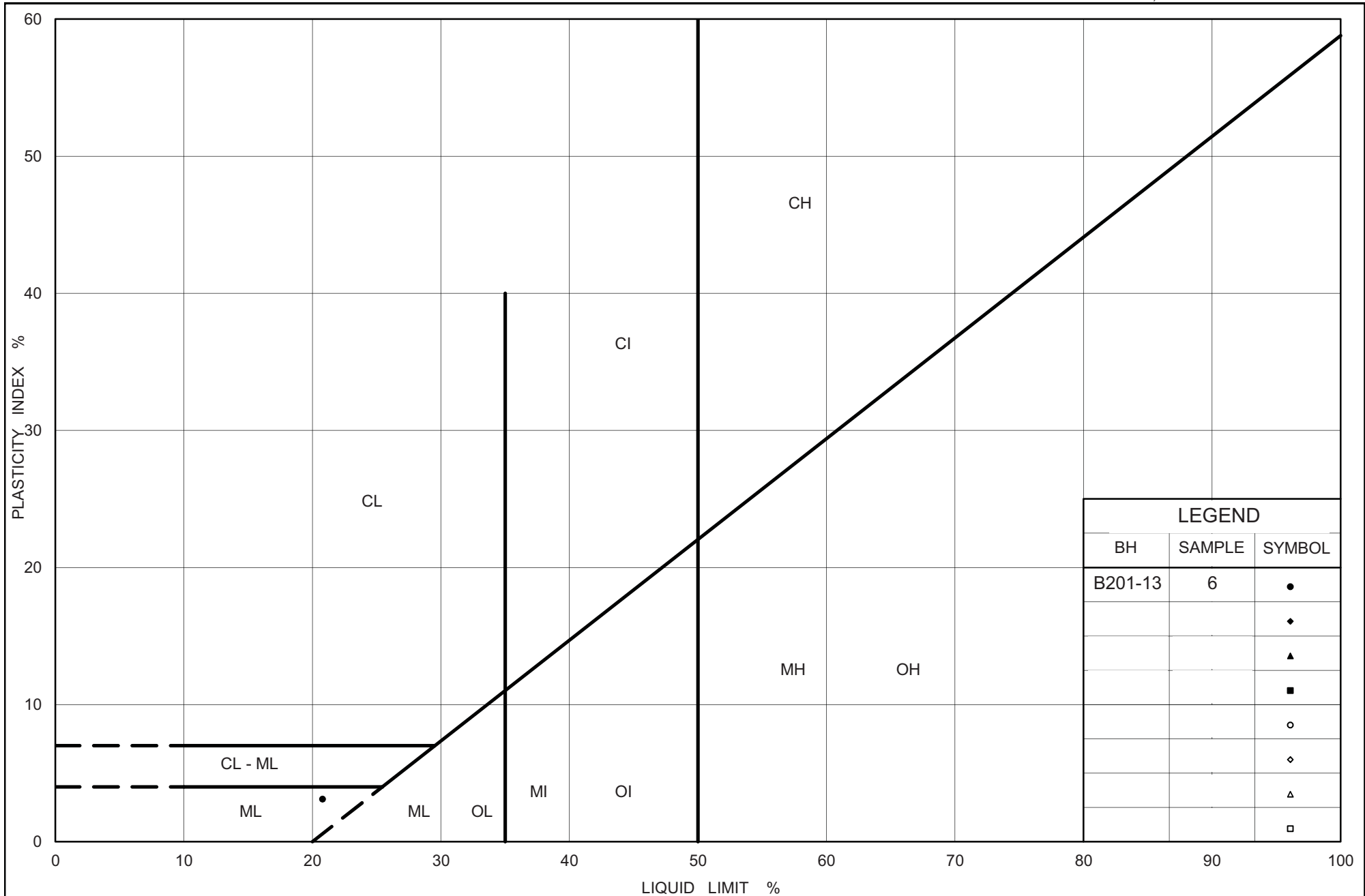
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
•	B201-13	6	174.9

Project Number: 09-1111-6014

Checked By: TZ

Golder Associates

Date: 13-Oct-11



Ministry of Transportation

Ontario

PLASTICITY CHART
Silt (Slight Plasticity)
Centre Pier - Still River Bridge (SBL)

Figure No. B12

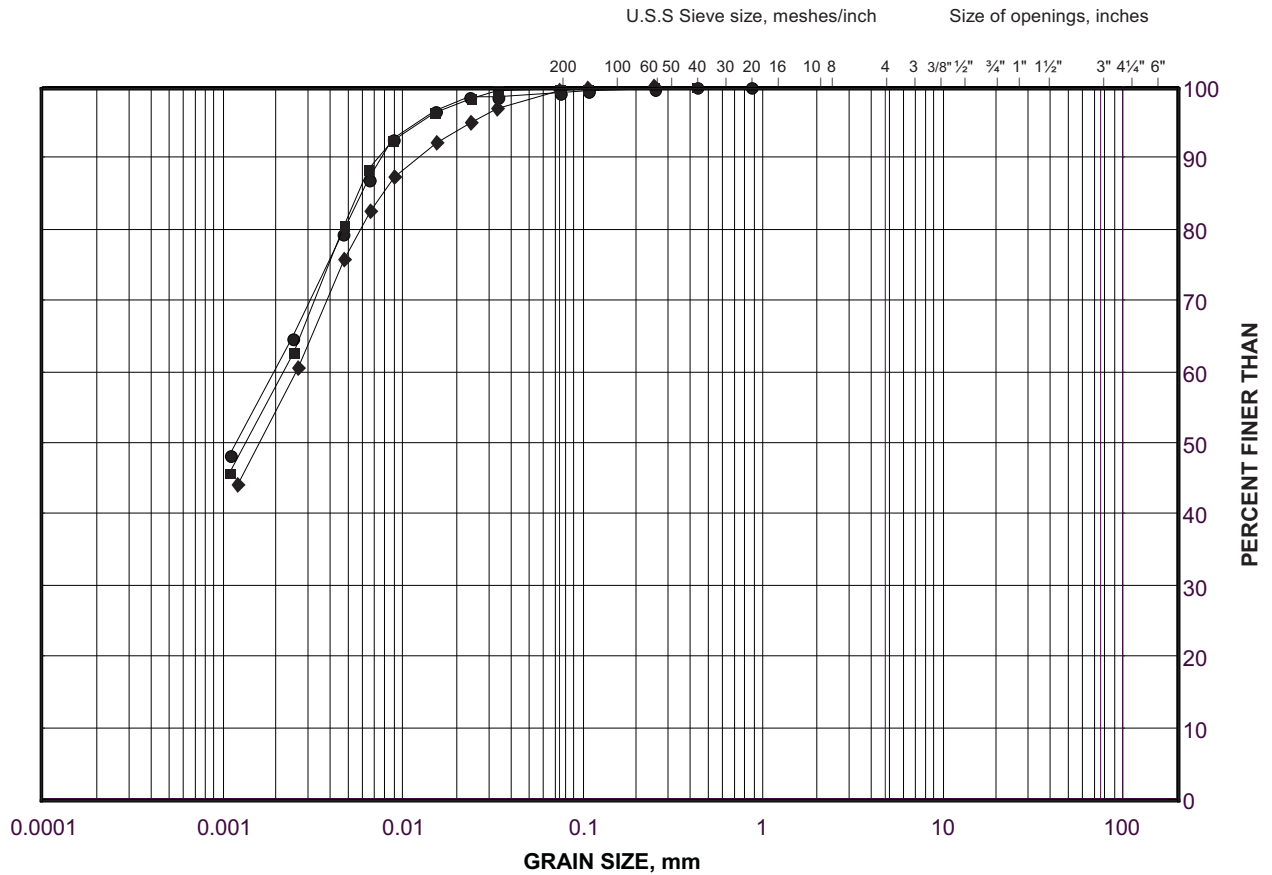
Project No. 09-1111-6014

Checked By: TZ

GRAIN SIZE DISTRIBUTION

Silty Clay to Clay
Centre Pier - Still River Bridge (SBL)

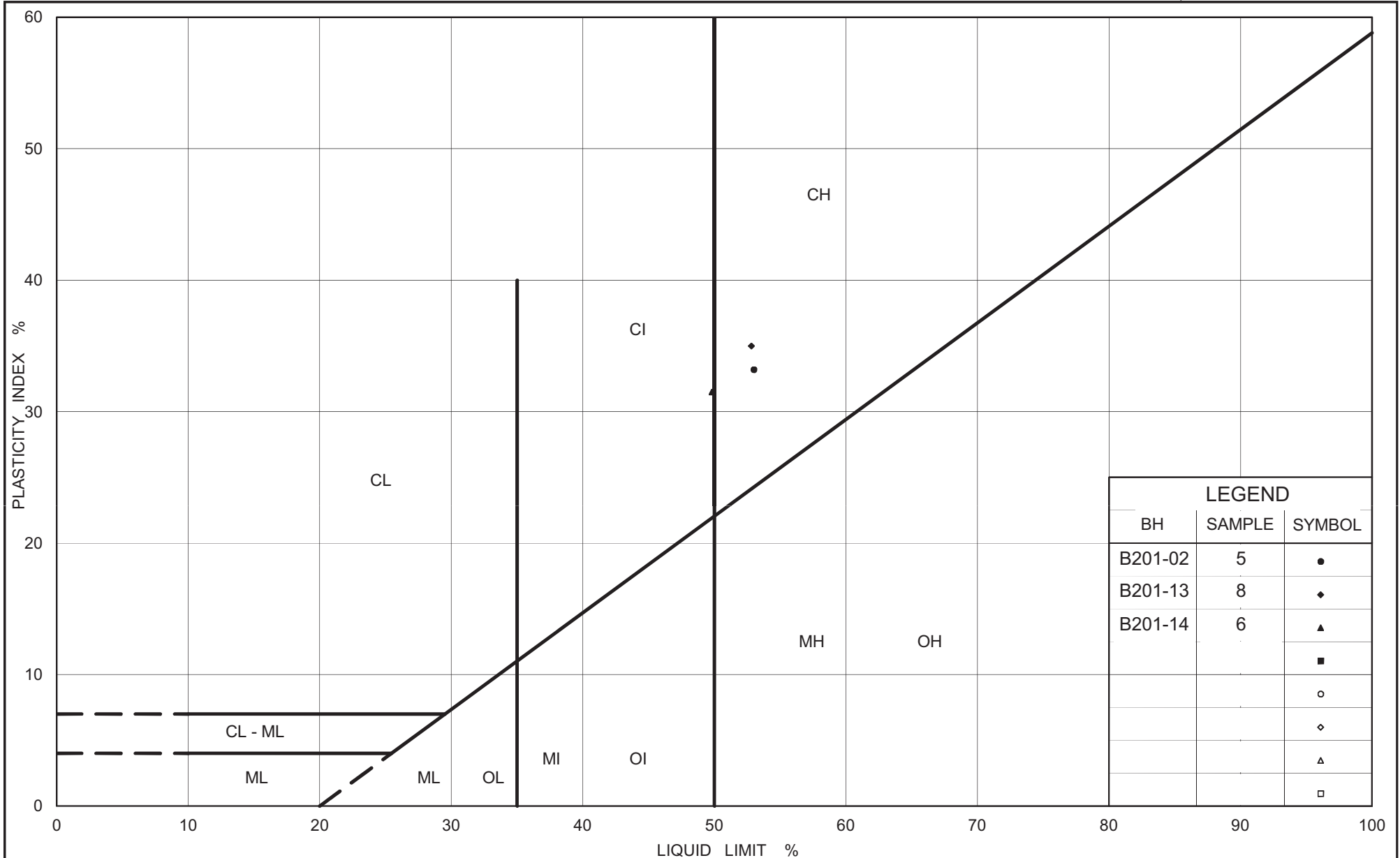
FIGURE B13



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	B201-02	5	174.3
■	B201-14	6	173.4
◆	B201-13	8	172.6



Ministry of Transportation

Ontario

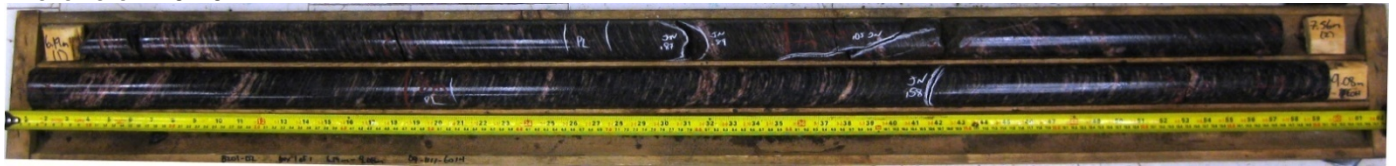
PLASTICITY CHART
 Silty Clay to Clay
 Centre Pier - Still River Bridge (SBL)

Figure No. B14

Project No. 09-1111-6014

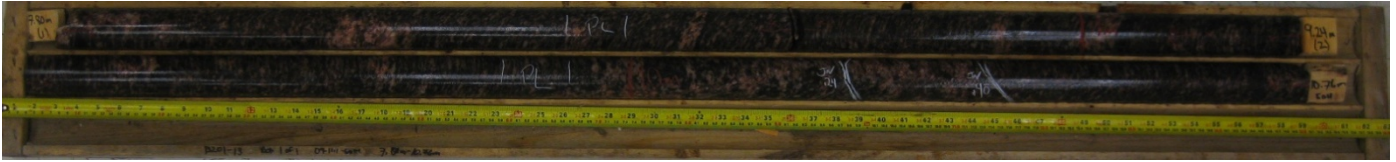
Checked By: TZ

Borehole B201-02



Box 1: 6.2 m – 9.1 m

Borehole B201-13

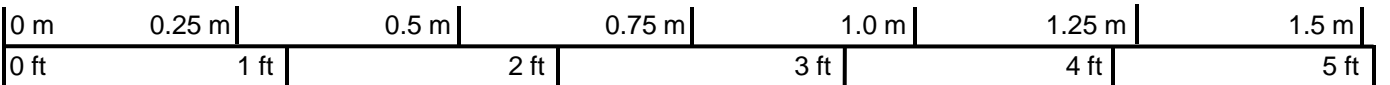


Box 1: 7.8 m – 10.8 m


Borehole B201-14



Box 1: 6.6 m – 9.7 m



Scale

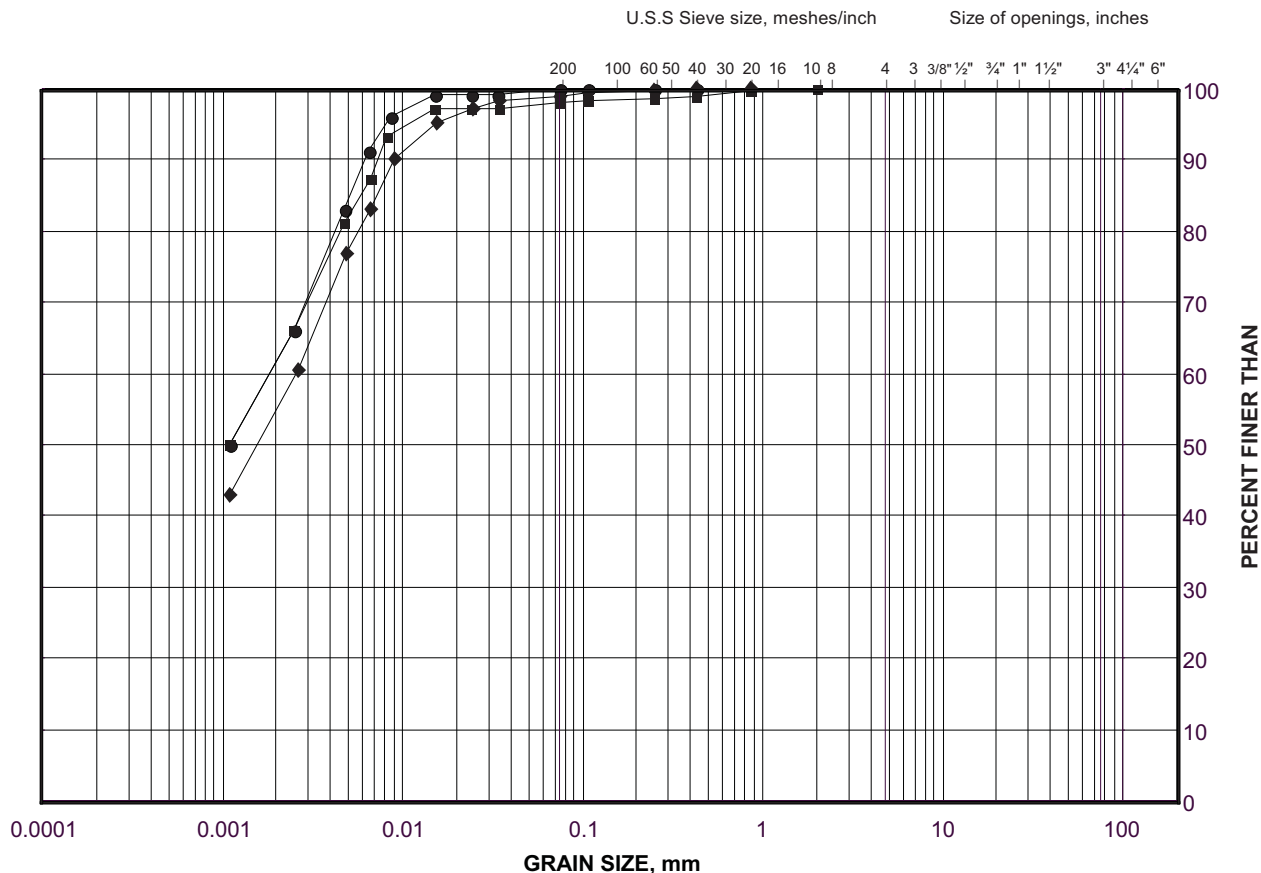
PROJECT		Still River Bridge SBL Structure Highway 69 Four-Laning GWP 5404-05-00; WP 5140-05-01		
TITLE		Bedrock Core Photographs – Centre Pier		
		PROJECT No. 09-1111-6014		FILE No. ----
		DESIGN	AT	SCALE NTS
		CADD	--	REV.
		CHECK	TVA	FIGURE B15
		REVIEW	TVA	

GRAIN SIZE DISTRIBUTION

Silty Clay to Clay

North Abutment (One-Span Option) - Still River Bridge (SBL)

FIGURE B16



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

LEGEND

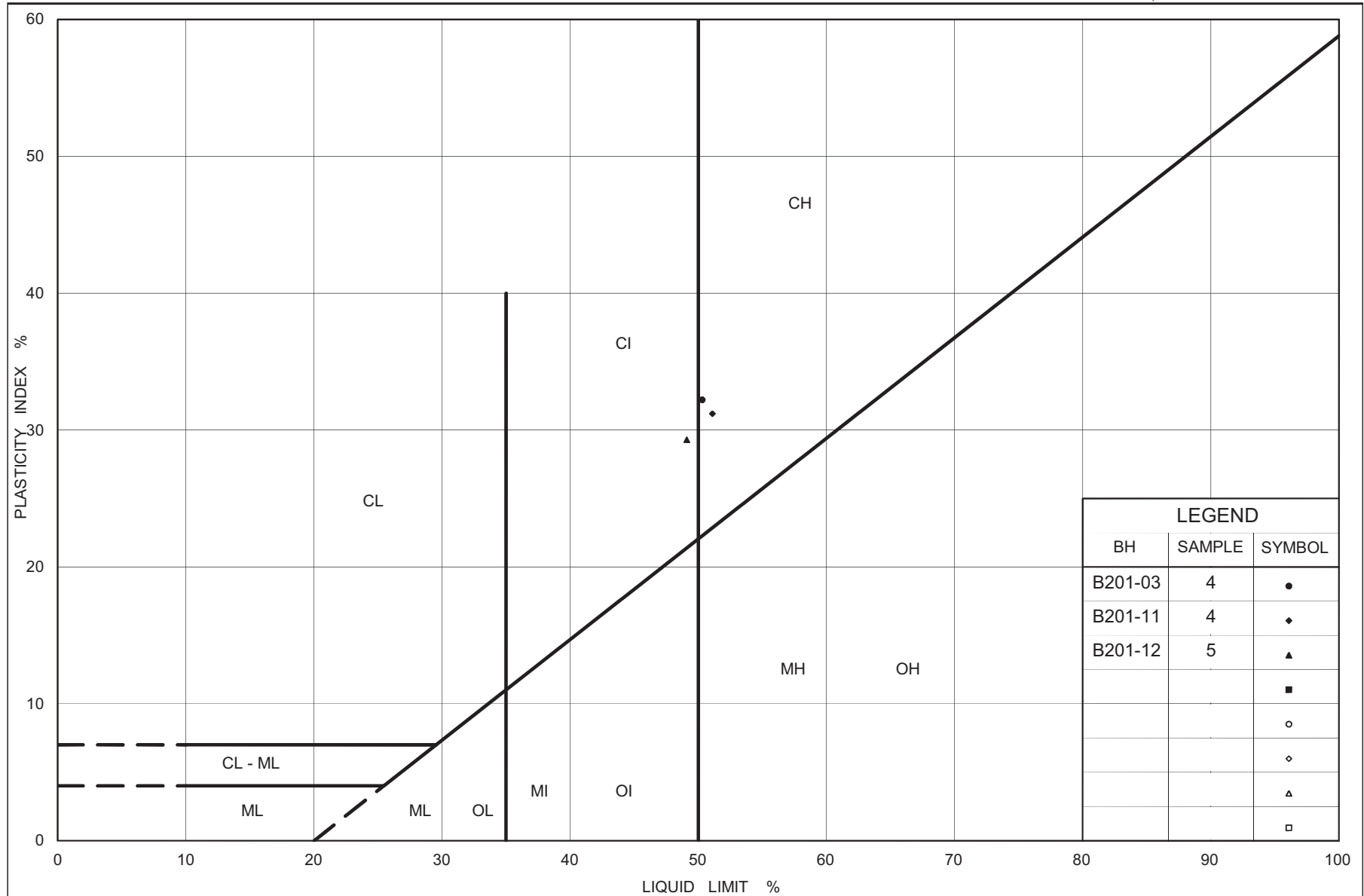
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	B201-11	4	176.7
■	B201-03	4	176.4
◆	B201-12	5	175.5

Project Number: 09-1111-6014

Checked By: TZ

Golder Associates

Date: 13-Oct-11



Ministry of Transportation

Ontario

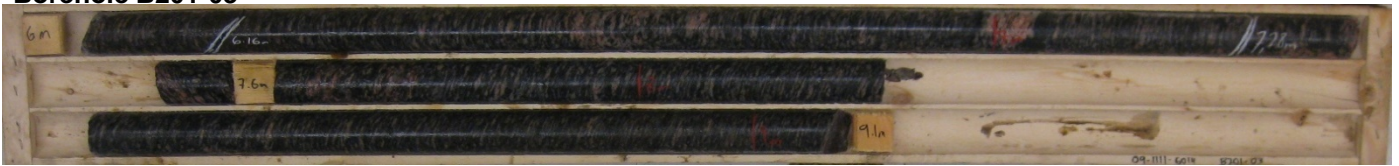
PLASTICITY CHART
 Silty Clay to Clay
 North Abutment (One-Span Option) - Still River Bridge (SBL)

Figure No. B17

Project No. 09-1111-6014

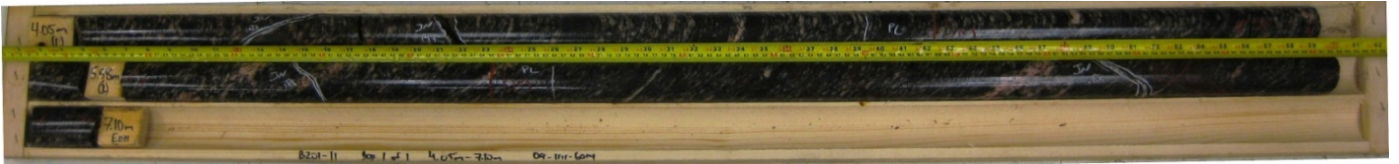
Checked By: TZ

Borehole B201-03



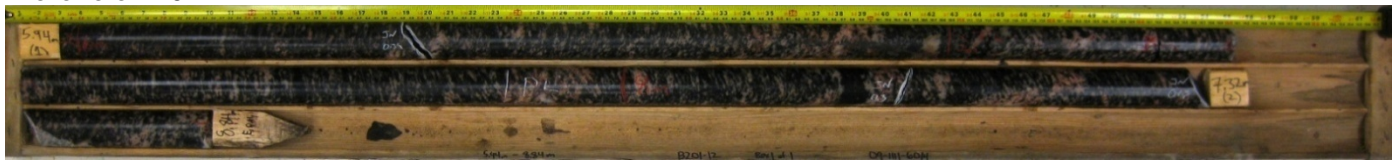
Box 1: 6.0 m – 9.1 m

Borehole B201-11

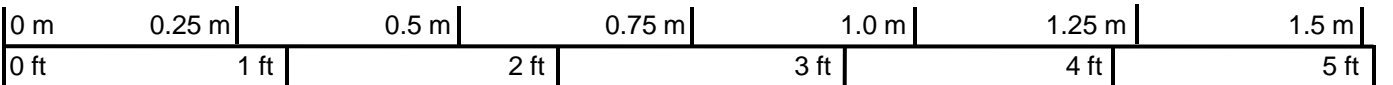


Box 1: 4.1 m – 7.1 m


Borehole B201-12



Box 1: 5.9 m – 8.8 m



Scale

PROJECT				Still River Bridge SBL Structure Highway 69 Four-Laning GWP 5404-05-00; WP 5139&40-05-01		
TITLE				Bedrock Core Photographs – North Abutment (One-Span Option)		
				PROJECT No. 09-1111-6014		FILE No. ----
				DESIGN	AT	
				CADD	--	
				CHECK	TVA	
				REVIEW	TVA	
				SCALE	NTS	REV.
				FIGURE B18		

Borehole B201-06



Box 1: 0.1 m – 3.4 m

Borehole B201-07



Box 1: 0.2 m – 3.2 m

Borehole B201-07

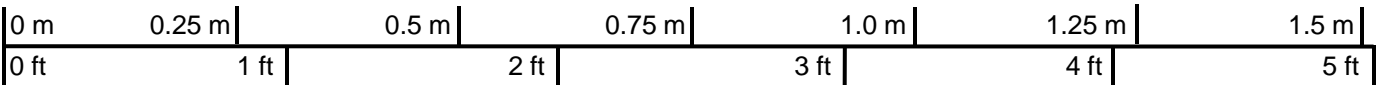


Box 2: 3.2 m – 5.2 m


Borehole B201-08



Box 1: 0.2 m – 4.0 m



Scale

PROJECT		Still River Bridge SBL Structure Highway 69 Four-Laning GWP 5404-05-00; WP 5140-05-01		
TITLE		Cobbles/Boulder and Bedrock Core Photographs – North Abutment and Approach (Two-Span Option)		
		PROJECT No. 09-1111-6014		FILE No. ----
		DESIGN	AT	
		CADD	--	
		CHECK	TVA	
		REVIEW	TVA	
		SCALE	NTS	REV.
FIGURE B19				



APPENDIX C

Non-Standard Special Provisions

PILE POINTS – Item No.

Non-Standard Special Provision

As part of the work under the above tender item, the Contractor shall supply Titus “Rock Injector Design” Pile Points on HP 310x110 Piles or equivalent. Piles will be driven to bedrock.

References

OPSS 906 – Structural Steel

Materials

The pile points shall be of the following:

Product

Manufacturer

HPP-R-12-HD

Titus Steel Company Ltd.
6767 Invader Crescent
Mississauga, Ontario
Tel. 905-564-2446

(Or approved equivalent which includes Oslo Points as per OPSD 3000.201)

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

MASS CONCRETE – Item No.

Non-Standard Special Provision

Scope of Work

The scope of work for the above noted tender item includes mass concrete under the north abutment (two-span structure) footings.

Construction

Concrete shall be of the same strength as the footing concrete and placed in accordance with OPSS 904.

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

DOWELS INTO ROCK – Item No.

Non-Standard Special Provision

Scope of Work

This special provision covers the requirements for the placement and field testing of dowels into rock.

Construction

Dowels into rock shall be constructed in accordance with OPSS.PROV 904 Concrete Structuresⁱ. All reinforcing steel supplied shall be in accordance with OPSS 1440 Steel Reinforcement for Concreteⁱⁱ (dowel bars conforming to CAN/CSA G30.18, Grade 400).

Where dowels are to be placed in rock, hole shall be drilled to the required depth and size. Hole diameter shall be two times the nominal diameter of the dowel. Each hole shall be cleaned out, grouted and the dowel set in place. Grout shall be of the same strength as the footing concrete or at least 25 MPa at 28 days.

If hole contains water, the Contractor shall remove the water, otherwise a tremie procedure shall be used to completely fill the hole with grout. The dowel shall be forced into the hole after the grout has been placed and while it is still fresh.

Rock Dowel Testing

All proposed testing procedures shall be in general conformance with ASTM D3689, ASTM D1143/D1143M and ASTM D4435. Field testing must be carried out in the presence of, and the results reviewed and approved by, the Contract Administrator.

Performance Tests

The following table summarizes the number of rock dowels where performance testing shall be carried out to confirm that the design load of the rock dowels can be achieved. The Contract Administrator will select the rock dowels to be tested.

Bridge	Foundation	Number of Dowels for Performance Testing
Highway 69 / Still River Bridge (SBL)	North Abutment	2

Performance test shall be by axial tensioning using a hydraulic jack with a capacity of at least 1.5 times the ultimate strength of the dowels.

Rock dowels shall be loaded and unloaded in 3 cycles and measurements of the displacement of the dowel shall be carried out at each load increment (step) in accordance with the following schedule:

Cycle-Step	1-1	1-2	1-3	2-1	2-2	2-3	2-4
% Design Load	50	75	25	50	75	100	25
Cycle-Step	3-1	3-2	3-3	3-4	3-5		
% Design Load	50	75	100	110	25		

The design load shall be taken as 360 kN for 35M dowels, 252 kN for 30M dowels, 180 kN, for 25M dowels, and 108 kN for 20M dowels.

Displacement measurements shall be carried out at each load increment using calibrated displacement gauges capable of measuring movements of 0.0025 cm. Measurements shall be referenced to an independent fixed referenced pint.

Rock dowels which fail to meet the acceptance criteria shall be replaced at the Contractor's expense and re-tested. If a rock dowel fails, three (3) additional rock dowels shall be tested at the same abutment and pier footing as directed by the Contract Administrator.

Acceptance criteria for the rock dowels will be in accordance with the Post-Tensioning Institute (1985) as follows:

- The dowels are acceptable if the total elastic movement is greater than 80 percent of the theoretical elastic elongation of the free stressing and is less than the theoretical elongation of the free stressing length plus 50 percent of the bond length.

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

ⁱ OPSS.PROV 904 Construction Specification for Concrete Structures

ⁱⁱ OPSS 1440 Material Specification for Steel Reinforcement for Concrete

OBSTRUCTIONS – Item No.

Special Provision

The overburden at the location of the proposed north abutment/approach embankment (two-span structure) contains numerous zones/nests of cobbles and boulders. Consideration of the presence of these obstructions must be made in the selection of appropriate equipment and procedures for sub-excavation for construction of spread footings.

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

EXPANDED POLYSTYRENE EMBANKMENT – Item No.

Special Provision

REQUIREMENTS FOR EXPANDED POLYSTYRENE EMBANKMENT FILL

1.0 SCOPE

This special provision covers the requirements for the supply and construction of the rigid expanded polystyrene embankment fill and associated works as shown on the contract drawings.

2.0 REFERENCES

This special provision refers to the following standards, specifications or publications.

2.1 National Standards of Canada

CAN/CGSB - 51.20 M87

2.2 ASTM

ASTM D6817 Standard Specification for Rigid Cellular Polystyrene Geofoam

ASTM D1621 Test Method for Compressive Properties of Rigid Cellular Plastics

ASTM C203 Test Method for Breaking Load and Flexural Properties of Block Type Thermal Insulation

ASTM C177 Test Method for Steady State Heat Flux Measurements and Thermal Transmission Properties by Means of the Heat Flow Apparatus

ASTM D2842 Test Method for Water Absorption by Rigid Cellular Plastics

ASTM D2863 Test Method for Measuring the Minimum Oxygen Content

ASTM D2126 Test Method for Response of Rigid Cellular Plastics to Thermal and Humid Aging

2.3 OPSS - Ontario Provincial Standard Specification

OPSS 212 Borrow

OPSS 501 Compaction

OPSS 517 Dewatering

OPSS 1010 Aggregates – Granular A, B, M, and Selected Subgrade Material

OPSS 1860 Geotextiles

3.0 SUBSURFACE CONDITIONS

The subsurface conditions at the site are described in the Foundation Investigation Report for this Contract.

4.0 DEFINITIONS

For the purpose of this special provision, the following definitions apply:

Rigid Expanded Polystyrene: Moulded rigid blocks produced by a process of pre-expansion, aging and forming of petroleum based raw material.

Rigid Extruded Expanded Polystyrene: Rigid boards made by extrusion of expanded polystyrene beads.

Production Lot: The quantity of rigid polystyrene blocks produced in a continuous period of manufacturing the same grade and thickness of product within the same production day.

Quality Verification Engineer: Quality Verification Engineer means an Engineer with a minimum of five (5) years experience related to the design and/or construction of expanded polystyrene systems of similar scope to that in the Contract, or alternatively has demonstrated expertise by providing satisfactory quality verification services for the work at a minimum of two (2) projects of similar scope to the Contract. The Quality Verification Engineer shall be retained by the Contractor to ensure conformance with the contract documents and issue of certificate(s) of conformance.

5.0 QUALIFICATION

The Contractor shall have on site at the commencement of the work, a representative of the supplier of the rigid expanded polystyrene to advise on recommended construction procedure.

The Contractor shall maintain liaison with the supplier throughout the construction of the embankment for advice and guidance as required. Periodic site visits by the supplier should be coordinated as required.

6.0 SUBMISSION AND DESIGN REQUIREMENTS

6.1 Submission of Shop Drawings

At least three (3) weeks before the commencement of work, the Contractor shall submit to the Contract Administrator six (6) copies of the shop drawings and method statement signed and sealed by the Quality Verification Engineer that provides full details of materials and construction procedure.

6.2 Delivery, Storage, Handling, and Protection

The Contractor shall submit the method of delivery, storage, handling and protection from damage by weather, traffic, construction staging and other causes as per the rigid expanded polystyrene manufacturer's requirement.

6.3 Construction

The contractor shall submit full details of the following.

- a) The method of foundation excavation and preparation.
- b) Construction of 300 mm thick levelling pad.
- c) The method of placement of expanded polystyrene blocks including temporary ballasting and protection of blocks during installation. The shop drawings shall indicate laying pattern and block dimensions on a layer-by-layer basis.
- d) The method and limits of placement of polyethylene sheeting.
- e) The method of placement of 125 mm reinforced concrete base pad (or equivalent).
- f) The method of placement of subbase material.
- g) The method of placement of side slope cover.

6.4 Quality Verification Engineer

- (1) The Contractor shall submit details of the sequence and method of installation to the Quality Verification Engineer for review. The submittals shall satisfy the specifications and at a minimum include a detailed description of proposed installation procedures. The details shall be submitted at least three weeks prior to the installation of the rigid expanded polystyrene embankments. The Contractor shall also submit to the Contract Administrator, for information purposes, details of the sequence and method of installation. The submittals shall satisfy the specifications and at a minimum contain the above information as provided to the Contractor's Quality Verification Engineer.
- (2) The Contractor shall submit to the Contract Administrator a Certificate of Conformance sealed and signed by the Quality Verification Engineer a minimum of one week prior to commencement of work under this item. The Certificate shall state that the installation procedures are in conformance with the requirements and specifications of the contract documents. Quality test certificates for each production lot supplied, showing compliance with all requirements of this special provision shall be obtained by the Contractor and submitted to the Contract Administrator prior to installation. Upon completion of the Expanded Polystyrene Embankment the Contractor shall submit to the Contract Administrator a Certificate of Conformance sealed and signed by the Quality Verification Engineer stating that the Expanded Polystyrene Embankment has been constructed in conformance with the installation procedures and specifications of the contract documents.

7.0 MATERIALS

7.1 Granular Levelling Pad

The levelling pad shall consist of a Granular 'A' material with gradation and physical requirements as specified in Special Provision 110S13.

7.2 Rigid Expanded Polystyrene

7.2.1 General

7.2.1.1 The Contractor shall submit:

1. A general statement as to the type, composition, and method of production of the material.
2. The manufacturer's name, address, phone number, identification of a contact person and description of background experience in the manufacturing of the rigid expanded polystyrene.
3. Certification of compliance of physical and mechanical properties.
4. An identification of a laboratory accredited by the Standards Council of Canada to conduct the testing of the physical and mechanical properties of the rigid expanded polystyrene.
5. The physical and mechanical properties of the rigid expanded polystyrene including:
 1. Geometry
 2. Nominal Density
 3. Compressive Strength
 4. Flexural Strength
 5. Thermal Resistance
 6. Dimensional Stability
 7. Flammability
 8. Water Absorption
6. Aging and durability characteristics of the polystyrene including the chemical, biological and ultra-violet degradation resistance of the rigid polystyrene.
7. A sample of the expanded polystyrene material to the Quality Verification Engineer for review.
8. To the Contract Administrator a Certificate of Conformance sealed and signed by the Quality Verification Engineer a minimum of one (1) week prior to commencement of work under this item. The Certificate shall state that the expanded polystyrene material is in conformance with the requirements and specifications of the contract documents.

7.2.1.2 Production Lots

Each block of the same production lot shall be stamped with the same production code showing plant identification, type and date of production. The polystyrene shall be free from defects affecting serviceability.

7.2.2 Detail Requirements

Requirements shall be as shown in Table 1 and as described below.

Table 1 – Material Properties

PROPERTY	UNIT	REQUIREMENTS	TEST PROCEDURE
Geometry - Linear Dimensions - Flatness - Squareness	mm (min)	1200 x 600 x 300 \pm 1% 10 mm in 3 m \pm 0.5%	--
Nominal Density	kg/m ³ (max)	50	--
Compressive Strength at 5% Deformation	kPa (min)	115	ASTM D1621 (Procedure A)
Flexural Strength	kPa (min)	240	ASTM C203
Dimensional Stability	% linear change (max)	1.5	ASTM D2126
Thermal Resistance	m ² .°C/W (min for 25 mm thickness)	0.7	ASTM C177 or C518
Flammability	Limiting Oxygen Index (min)	24	ASTM D2863
Water Absorption	% by Volume (max)	4	ASTM D2842

7.2.2.1 Geometry

The expanded polystyrene shall be supplied in the form of rectangular parallel blocks of minimum acceptable dimensions of 1200 mm x 600 mm x 300 mm. The maximum deviation from the specified linear dimensions shall be \pm 1%.

The flatness of the block faces shall be within \pm 10 mm of a line formed by a 3 m straight edge.

The maximum difference in corner-to-corner dimensions (squareness) shall be 0.5%.

7.2.2.2 Compressive Strength

The minimum compressive strength, measured in accordance with ASTM D1621, Procedure A, shall be 115 kPa at a strain of not more than 5%. The maximum permissible permanent stress level should not exceed 30% of the compressive strength of the material at 5% deformation.

7.2.2.3 Flexural Strength

The minimum flexural strength of the polystyrene shall be 240 kPa. The flexural strength shall be determined in accordance to ASTM C203, Method 1, Procedure B.2.7.4 Dimensional Stability.

7.2.2.4 Dimensional Stability

Dimensional Stability shall be determined in accordance with ASTM D2126, Procedure G. A tolerance of 1.5% shall be satisfied.

7.2.2.5 Thermal Resistance

The thermal resistance shall be $0.7 \text{ m}^2 \cdot \text{C}/\text{W}$ for a 25 mm thickness using the following equation and using the average value from three specimens:

$$R_{25 \text{ mm}} = \frac{R}{\text{thickness (mm)}} \cdot 25 \text{ mm}$$

The thermal resistance shall be measured in accordance with ASTM C177 or C518.

7.2.2.6 Flammability

The expanded polystyrene shall be classified as to surface burning characteristics in accordance with CAN/ULC – 51022 having a flame spread rating less than 500. The expanded polystyrene shall have a minimum limiting oxygen index measured in accordance with ASTM D2863.

7.2.2.7 Water Absorption

The water absorption as measured by ASTM D2842 shall be limited to 4% by volume.

7.2.2.8 Chemical Resistance

The expanded polystyrene shall be resistant to common inorganic acids and alkalis. A table identifying the chemical resistance as either resistant limited or not resistant shall be submitted.

7.2.2.9 Biological Resistance

The expanded polystyrene shall be resistant to biological degradation caused by organisms or enzymes.

7.2.2.10 Environmental

The expanded polystyrene shall be inert, non-nutritive and highly stable and shall not produce undesirable gases or leachate.

7.2 Polyethylene Sheeting

The plastic sheeting shall be 10 mil polyethylene sheeting or equivalent.

7.2 Concrete Top Slab

The concrete top slab shall consist of 30 MPa reinforced concrete as shown on the Contract Drawings.

8.0 DELIVERY, STORAGE AND HANDLING

The product shall be suitably marked to identify its type, number and the manufacturer's name or trademark.

The Contractor shall protect the expanded polystyrene from exposure to sunlight to avoid ultraviolet degradation as per manufacturer's recommendation.

Protection of materials and works from damage by weather, traffic, construction staging, fire or vandalism and other causes shall be the responsibility of the Contractor.

9.0 CONSTRUCTION

9.1 Foundation Excavation

Foundation excavation shall be carried out to the design elevations shown on the Contract Drawings. Any softened, loosened or deleterious materials at the foundation footing elevation shall be subexcavated and replaced with Granular 'A' or Granular 'B' material.

9.2 Leveling Pad

Place, level and compact a layer of Granular 'A' material in accordance with OPSS 501 to within ± 30 mm of the design elevation. The leveling pad shall not deviate by more than 10 mm at any place on a 3 m straight edge over the limits of the bottom course of blocks. The leveling pad shall not be placed on frozen ground.

9.3 Installation of Blocks

- (1) The individually marked blocks shall be placed on the prepared leveling pad. The top surface of the first layer of blocks is to be set plane and level. Local trimming of the blocks may be necessary.
- (2) Subsequent successive layers shall be oriented with the long axis of blocks positioned at 90° to the previous layer in order to avoid continuous joints. Block joints shall be offset and staggered between layers.
- (3) A continuous check shall be kept to ensure the evenness of the blocks is satisfactory in each layer. Blocks shall be laid with joints with maximum opening of 10 mm between blocks. Differences in heights between adjacent blocks in the same layer should not exceed 5 mm.
- (4) Sloping end adjustments at the abutments shall be accomplished by leveling terraces in the subsoil in accordance with the block thickness.
- (5) Temporary ballast shall be provided as necessary to prevent movement of expanded polystyrene both in storage and as placed due to windy conditions. Timber fasteners or equivalent shall be used as necessary.
- (6) The expanded polystyrene embankment shall be protected from accidental ignition due to welding, smoking, grinding or cutting tools, etc. The Contractor shall take all necessary precautions to prevent ignition of the expanded polystyrene.
- (7) The expanded polystyrene shall be protected from organic solvents and other aggressive, harmful chemicals during construction. The proposed method of protection during construction shall be submitted to the Contractor's Quality

Verification Engineer for review and to the Contract Administrator for information purposes.

- (8) Exposed blocks shall be covered immediately to avoid possible burrowing by animals.
- (9) Individually marked blocks shall be fabricated and placed to ensure the top surface matches the elevation and crossfall shown on the drawings.
- (10) The top surface and side surfaces of the expanded polystyrene shall be covered with 10 mil polyethylene sheeting extending onto adjacent work at the longitudinal ends of the embankment. All joints shall be lapped a minimum of 300 mm to provide a fully sealed enclosure.
- (11) The contractor shall install the concrete base pad as detailed elsewhere in the contract.
- (12) The side slope of the rigid expanded polystyrene embankment shall be covered with granular fill as detailed elsewhere in the Contract Drawings.

10.0 EQUIPMENT

All cutting of polystyrene materials shall be by electric equipment or by hand.

Heavy equipment shall be limited in weight and size and restricted in operation to avoid damaging the expanded polystyrene as per the manufacturer's requirement.

11.0 QUALITY ASSURANCE

11.1 General

The Contract Administrator may undertake an independent testing program of the expanded polystyrene. Sampling and testing will be carried out in conformance with the relevant test procedure. The physical and thermal property testing identified in Table 1 will be conducted. A recognized testing laboratory accredited by the Standards Council of Canada shall conduct the testing.

11.2 Sampling Frequency

Sufficient sample material shall be obtained from blocks randomly selected by the Contract Administrator from each production lot as soon as the material arrives on site. As a minimum, three (3) blocks shall be tested.

11.3 Acceptance/Rejection

Failure of any one of the sample blocks to comply with any requirements of this special provision shall be cause for rejection of the production lot from which it was taken. Replacement of the blocks shall be at the Contractor's expense.

12.0 MEASUREMENT FOR PAYMENT

12.1 Actual Measurement

Measurement will be by volume in cubic metres measured in its original position and based on cross-sections.

13.0 PAYMENT

13.1 Basis of Payment

The Concrete Base pad and granular leveling pad shall be paid for with the appropriate tender items as detailed elsewhere in the contract.

Payment at the contract price for the above tender item shall be full compensation for all labour, materials and equipment to do the work as described above and no extra payments will be made.

At Golder Associates we strive to be the most respected global company providing consulting, design, and construction services in earth, environment, and related areas of energy. Employee owned since our formation in 1960, our focus, unique culture and operating environment offer opportunities and the freedom to excel, which attracts the leading specialists in our fields. Golder professionals take the time to build an understanding of client needs and of the specific environments in which they operate. We continue to expand our technical capabilities and have experienced steady growth with employees who operate from offices located throughout Africa, Asia, Australasia, Europe, North America, and South America.

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