**SEPTEMBER 2014**

# FOUNDATION INVESTIGATION AND DESIGN REPORT

## STILL RIVER NBL BRIDGE STRUCTURE, SITE NO. 44-458/1
## HIGHWAY 69 FOUR-LANING FROM 1.7 KM NORTH OF HIGHWAY 529 NORTHERLY TO 3.9 KM NORTH OF HIGHWAY 522
## MINISTRY OF TRANSPORTATION, ONTARIO
## GWP 5404-05-00; WP 5139-08-01

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**GEOCRES NO.:** **41H-121**

**Report Number:** 09-1111-6014-2522

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REPORT

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# PART A

## FOUNDATION INVESTIGATION REPORT
## STILL RIVER NBL BRIDGE STRUCTURE, SITE NO. 44-458/1
## HIGHWAY 69 FOUR-LANING FROM 1.7 KM NORTH OF HIGHWAY 529
## NORTHERLY TO 3.9 KM NORTH OF HIGHWAY 522
## MINISTRY OF TRANSPORTATION, ONTARIO
## GWP 5404-05-00; WP 5139-08-01

# 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 northbound lanes (NBL) structure over the Still River (Site No. 44-458/1), 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 NBL 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 NBL 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 NBL Bridge structure (two-span bridge option) was provided to Golder by URS on April 5, 2011. In addition, a preliminary GA Drawing for the one-span bridge option was provided to Golder by URS on October 1, 2010.

This report addresses the foundation investigation carried out for the Still River NBL Bridge structure and the associated approach embankments only. A two-span 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 evaluted during the initial design stage, this foundation investigation report has been prepared to address the 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 Contract 2 section of the new four-lane Highway 69 alignment is also oriented generally in a south-north direction within the overall project limits, spanning the Township of Wallbridge to the south and the Township of Henvey to the north for a total distance of 4.8 km. The proposed Still River NBL bridge structure is located within the Contract 2 highway alignment and is located approximately 3.2 km from the southern limit of Contract 2, corresponding to approximately 1.5 km northeast of the junction between 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 to densely populated tree covered areas and numerous bedrock outcrops separated by valleys, rivers 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 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 Elevation 178.7 m to 179.1 m at the centre pier to about Elevation 179.0 m to 181.3 m at the north abutment and along the north approach (for the one-span bridge option) and about Elevation 183.9 m to 187.3 m at the north abutment and 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 NBL Bridge structure was carried out between February 10 and 27, and March 3 and 31, 2011 during which time a total of thirteen (13) boreholes, two (2) probeholes (defined as augered boreholes (without sampling) for the purpose of establishing probable bedrock surface), two (2) Dynamic Cone Penetration Tests (DCPTs) and two (2) hand excavations were advanced at the locations of the proposed structure foundation elements and approach embankments. A summary of the respective boreholes, probeholes, DCPTs and hand excavations advanced at each foundation element and approach embankment is presented below.

| Foundation Element/ Approach Embankment | Investigation Type | | | |
|---|---|---|---|---|
| | Borehole No. | Probehole No. | DCPT No. | Hand Excavation |
| South Approach Embankment (One- or Two-Span Option) | S204-18<br>B202-01 | -- | -- | -- |
| South Abutment (One- or Two-Span Option) | B202-02<br>SP1[1] | -- | -- | -- |
| Centre Pier | B202-03<br>B202-14<br>B202-15 | B202-P01<br>B202-P02 | -- | -- |
| North Abutment (One-Span Bridge | B202-04<br>B202-12<br>B202-13 | -- | B202-DC01<br>B202-DC02 | -- |

| Foundation Element/ Approach Embankment | Investigation Type | | | |
|---|---|---|---|---|
| | Borehole No. | Probehole No. | DCPT No. | Hand Excavation |
| North Approach Embankment (One-Span Option) | B202-05 | -- | -- | -- |
| North Abutment (Two-Span Option) | B202-06<br>B202-07<br>B202-08<br>B202-09 | -- | -- | B202-10 |
| North Approach Embankment (Two-Span Option) | -- | -- | -- | B202-11 |

Note: 1. Borehole SP1 was advanced in the vicinity of the south abutment to install a pizometer for monitoring the groundwater level in the area.

In addition, one (1) borehole (Borehole S204-18) advanced within the immediately adjacent Swamp 204 as part of the field investigation work carried out by Golder for the Contract 2 swamp crossings and high fill areas[1] was utilized to supplement this investigation at the south approach embankment. The Record of Borehole sheets and the results of the laboratory testing for all of the boreholes/drillholes advanced for this bridge structure, including Borehole S204-18, are presented in Appendix A and Appendix B, respectively. The locations of the boreholes, probeholes and DCPTs are shown on Drawing 2.

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 or 213 mm outer diameter (O.D.) solid-stem augers, tricone and/or ‘casing (BW, EW, NW or HW) 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 advanced by automatic hammers on the drill rigs, 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 ($^1/_3$) 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 in two (2) boreholes at locations of thin overburden over bedrock knobs. Samples of the cohesive soils were obtained using 76 mm O.D. thin-walled ‘Shelby’ tubes (ASTM D1587, Standard Practice for Thin-Walled Tube Sampling) for relatively undisturbed samples. Field vane shear tests were conducted in cohesive soils for assessment 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 smaller diameter boreholes advanced by portable equipment. Samples of the bedrock were obtained using ‘NQ’ or ‘EQ’ or ‘BQ’ size rock core barrel.

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[1] Golder Associates Ltd. 2012. *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, G.W.P. 5404 05 00; W.P. 5404 05 01. Geocres No. 41H-115.*

The boreholes, probeholes 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 boreholes at the nort-east corner of the north abutment and at the north approach embankment associated with the two-span bridge option were located on bedrock outcrops and refusal condition was confirmed by hand excavation and exposure of bedrock. The boreholes, probeholes and DCPTs were advanced to depths of up to about 53.1 m below existing ground surface, including coring of bedrock. The bedrock was cored for lengths between about 1.6 m and 3.7 m in Boreholes B202-03 to B202-05, B202-07 to B202-09 and B202-12. 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 the north abutment (associated with the two-span bridge option), a piezometer was installed in each of Boreholes B202-03 and B202-08 and in a pre-augered hole located near the proposed south abutment (designated as Borehole SP1) to monitor the ground water levels at these locations. 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 upon completion, in accordance with Ontario Regulation 903, Wells (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, 2006)[2]. 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[3]) 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, probehole 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 locations given in the

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[2] Canadian 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.

Record of Borehole/Drillhole sheets, Record of Probehole sheets and Record of DCPT 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, probehole and DCPT locations and ground surface elevations are summarized below.

| Borehole / Probehole / DCPT No. | Location (MTM NAD 83) | | Ground Surface Elevation (m) | Borehole / Probehole / DCPT Depth (m) |
|---|---|---|---|---|
| | Northing | Easting | | |
| B202-01 | 5074802.7 | 225185.9 | 181.1 | 16.6 |
| B202-02 | 5074821.3 | 225178.6 | 181.0 | 53.1 |
| B202-03 | 5074861.3 | 225162.8 | 178.8 | 27.3 |
| B202-04 | 5074872.5 | 225158.4 | 179.4 | 13.0 |
| B202-05 | 5074886.4 | 225152.9 | 181.3 | 6.2 |
| B202-06 | 5074898.6 | 225140.9 | 183.9 | 0.9 |
| B202-07 | 5074902.4 | 225139.5 | 184.3 | 4.2 |
| B202-08 | 5074901.3 | 225146.9 | 184.2 | 6.6 |
| B202-09 | 5074900.3 | 225154.9 | 186.4 | 3.7 |
| B202-10[1] | 5074904.1 | 225153.3 | 187.1 | 0.1 |
| B202-11[1] | 5074919.8 | 225139.4 | 187.3 | 0.1 |
| B202-12 | 5074873.6 | 225151.0 | 179.5 | 7.5 |
| B202-13 | 5074871.4 | 225166.4 | 179.4 | 15.3 |
| B202-14 | 5074860.2 | 225170.8 | 179.0 | 28.1 |
| B202-15 | 5074862.4 | 225155.3 | 178.7 | 23.1 |
| S204-18 | 5074795.1 | 225189.0 | 181.1 | 42.8 |
| SP1 | 5074819.8 | 225159.0 | 181.1 | 6.1 |
| B202-P01 | 5074858.6 | 225156.8 | 178.9 | 25.6 |
| B202-P02 | 5074864.1 | 225169.3 | 179.1 | 25.7 |
| B202-DC01 | 5074869.7 | 225152.4 | 179.0 | 13.4 |
| B202-DC02 | 5074875.3 | 225164.9 | 180.2 | 12.8 |

Note: 1. B202-10 and B202-11 refers to a shovel excavation carried out at the north-east corner of the north abutment (two-span option) and at the north embankment approach, respectively, to expose the bedrock surface.

# 4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

## 4.1 Regional Geology

As delineated in The Physiography of Southern Ontario[4], 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.

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[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.

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 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 4[5]. 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 excavations carried out by a 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 are shown on Borehole SP1, augered for a piezometer installation, are interpreted based on cuttings and auger samples and are approximate only. These boundaries, therefore, represent transitions between soil types rather than exact planes of geological change. Further, subsurface conditions will vary between and beyond the borehole locations. The thickness of the overburden/depth to refusal as inferred from the resistance to auger, casing and DCPT advancement are shown on Record of Probehole sheets and 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 surficial layer of topsoil, underlain by alternating deposits of cohesive and non-cohesive soils, underlain by inferred cobbles and boulders. Bedrock was not encountered within the maximum depth of investigation (53.1 m). In the areas of the centre pier, north abutment and north approach (to the north of Still River) the subsurface conditions consist of bedrock outcrops and surficial layers of topsoil underlain by deposits of sand to silt and clay, underlain by bedrock at relatively shallow depth. The overburden thickness is variable across the proposed bridge structure, ranging from no cover at the north approach embankment (i.e. bedrock outcrops exposed at ground surface) to 53.1 m or greater at the south abutment.

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

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

## 4.3 South Abutment and Approach Embankment

One (1) borehole (Borehole B202-02) was advanced at the location of the proposed south abutment and one (1) borehole (Boreholes B202 01) was advanced on the alignment centreline of the proposed south approach embankment. In addition, one (1) borehole (Borehole S20418) advanced within Swamp 204, adjacent to the proposed south approach embankment, as part of the field investigation for the Contract 2 swamp crossings and high fill areas has been utilized to describe the subsurface conditions in this area.

In general, the subsurface conditions consist of topsoil underlain by alternating deposits of cohesive and non-cohesive soils with pockets and interlayers at varying depths. The alternating deposits are generally comprised of clayey silt to silty clay, sand to silt, silty clay to clay, silt to silty sand, silty clay to clay, and silt to sand, underlain by inferred 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 in Boreholes B202-01/S204-18 and B202-02, respectively.

A natural water content measured on one (1) sample of the topsoil is about 24 per cent.

### 4.3.2 Sandy Silt

A deposit of brownish grey sandy silt, trace clay was encountered underlying the topsoil in Borehole B202-02. The top of this deposit is at about Elevation 180.8 m and the thickness of the layer is about 0.5 m.

The SPT ‘N’-value measured within the sandy silt deposit is 5 blows per 0.3 m of penetration, indicating a loose relative density.

The natural water content measured on a sample of this deposit is about 20 per cent.

### 4.3.3 Clayey Silt to Silty Clay (Near Surface)

A deposit of brownish grey to brown to grey clayey silt to silty clay, trace to some sand was encountered underlying the topsoil or near surface sandy silt in all boreholes. Sand lenses were encountered within the clayey silt in Borehole B202-02 and rootlets and silty sand layers were encountered in Borehole S204-18. The top of this deposit ranges from about Elevation 180.8 m to 180.3 m and the thickness of the cohesive deposit varies between about 1.7 m and 1.8 m.

The SPT ‘N’-values measured within the clayey silt to silty clay deposit range from 2 blows to 6 blows per 0.3 m of penetration, suggesting a soft to firm consistency. It should be noted that in situ field vanes carried 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 natural water content measured on three (3) samples of the silty clay portion of the cohesive deposit are about 19 per cent and 39 per cent.

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

An Atterberg limits test was carried out on one (1) specimen of the cohesive deposit and indicates a liquid limit of about 37 per cent, a plastic limit of about 18 per cent, and a corresponding plasticity index of about 19 per cent. The results of the Atterberg limits test are shown on the plasticity chart on Figure B2 in Appendix B and indicate the material to be a silty clay of intermediate plasticity.

### 4.3.4 Sand to Silt (Upper)

A deposit of non-cohesive soil comprised of brown to grey sand, silty sand, and silt, trace to some clay, interlayered in places, was encountered underlying the clayey silt to silty clay deposit in all boreholes. The top of this deposit ranges from about Elevation 179.0 m to 178.6 m and the thickness of the deposit varies between about 5.8 m and 7.6 m.

The SPT ‘N’-values measured within this deposit range from 4 blows to 17 blows per 0.3 m of penetration, indicating a loose to compact relative density.

The natural water content measured on seven (7) samples of this deposit range from about 22 per cent to 29 per cent, while he natural water content measured on two (2) samples of the upper portion of the sand and silty sand deposit is 8 per cent and 11 per cent.

The grain size distributions of four (4) samples of the sand and the silt portions of this deposit are shown on Figure B3 in Appendix B. An Atterberg limits test on one (1) sample of the silt deposit indicates this material to be non-plastic.

### 4.3.5 Silty Clay to Clay (Upper)

A deposit of brown to grey silty clay to clay, containing sand lenses in the upper portion of the silty clay in Borehole B202-02, was encountered underlying the upper deposit of sand to silt in all boreholes. The top of the silty clay to clay deposit ranges from about Elevation 172.8 m to 171.4 m and the thickness of the deposit varies between about 7.9 m and 11.6 m. Borehole B202-01 was terminated within this deposit. In Borehole B202-02 a silt interlayer (or pocket) was encountered within the clayey silt to clay deposit, as discussed in Section 4.3.6.

The SPT ‘N’-values recorded within the silty clay to clay deposit 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 deposit measured undrained shear strengths ranging from about 39 kPa to 76 kPa with an average of about 55 kPa and the sensitivity is calculated to range from about 3 to 4. The field vane tests results indicate that the upper silty clay to clay deposit has a firm to stiff consistency.

The natural water content measured on eight (8) specimens of this deposit ranges from about 40 per cent to 74 per cent, but are generally greater than 50 per cent.

The grain size distributions of four (4) samples of the silty clay to clay deposit are shown on Figure B4 in Appendix B.

Atterberg limits tests were carried out on four (4) specimens of the silty clay to clay deposit and indicate liquid limits between about 44 per cent and 64 per cent, plastic limits between about 17 per cent and 21 per cent and corresponding plasticity indices between about 27 per cent and 45 per cent. The results of the Atterberg limits tests are shown on the plasticity chart on Figure B5 in Appendix B and indicate the material to be silty clay of intermediate plasticity to clay of high 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 B202-01. A preconsolidation stress of about 155 kPa was estimated from the void ratio versus logarithmic pressure plot and from the total work versus pressure plot. A bulk unit weight of about 16.6 kN/m$^3$ and a specific gravity of about 2.77 were measured on the consolidation test specimen. Details of the test results are shown on Figure B6 in Appendix B, and the test results are summarized below.

| Borehole Sample No. | Sample Depth / Elevation | $\sigma_{vo}'$ (kPa) | $\sigma_p'$ (kPa) | $\sigma_p' - \sigma_{vo}'$ (kPa) | OCR | $C_c$ | $C_r$ | $e_o$ | $c_v^*$ (cm$^2$/s) |
|---|---|---|---|---|---|---|---|---|---|
| Borehole B202-01<br>Sample 12 | 13.9 m /<br>167.2 m | 135 | 155 | 20 | 1.1 | 0.91 | 0.10 | 1.54 | $1.81 \times 10^{-3}$ |

Note: * For stress range of between effective overburden stress and final stress due to 7.5 m high approach embankment, that is 135 kPa ≤ $\sigma_v'$ ≤ 275 kPa

where:
$\sigma_{vo}'$ is the in situ vertical effective overburden stress in kPa
$\sigma_p'$ is the preconsolidation stress in kPa
$\sigma_v'$ 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 cm$^2$/s

### 4.3.6 Silt Interlayer

An approximately 2.7 m thick interlayer of grey silt, trace to some sand, trace to some clay and containing an approximately 0.6 m thick pocket of brownish grey silty clay was encountered within the upper silty clay to clay deposit in Borehole B202-02 at about Elevation 170.9 m.

Two (2) SPT ‘N’-values measured within the silt interlayer and silty clay pocket are 14 blows and 3 blows per 0.3 m of penetration, respectively, indicating a very loose and compact relative density in the silt and suggesting a soft consistency in the silty clay, respectively.

The natural water content measured on two (2) specimens of the silt interlayer is about 26 per cent and 27 per cent.

A grain size distribution of one (1) sample of this interlayer is shown on Figure B7 in Appendix B.

### 4.3.7 Silt to Silty Sand Interlayer

A non-cohesive interlayer comprised of grey silt, trace to some sand, trace clay to silty sand was encountered underlying the upper silty clay to clay deposit in Boreholes B202-02 and S204-18. The top of this interlayer is at about Elevation 161.2 m and 161.3 m and the thickness of the interlayer is about 5.8 m and 3.1 m in the respective boreholes.

The SPT 'N'-values measured within the silt portion of the non-cohesive interlayer are 7 blows and 9 blows per 0.3 m of penetration, indicating a loose relative density. One (1) SPT 'N'-value measured within the silty sand portion of the non-cohesive interlayer is 45 blows per 0.3 m of penetration, indicating a dense relative density.

The natural water content measured on two (2) samples of this silt to silty sand interlayer is 20 per cent and 24 per cent.

A grain size distribution of one (1) sample of the silt portion of this non-cohesive interlayer is shown on Figure B7 in Appendix B.

### 4.3.8 Silty Clay to Clay (Lower)

A deposit of grey silty clay to clay was encountered underlying the silt to silty sand interlayer in Boreholes B202-02 and S204-18. Silt interlayers were encountered within this deposit in Borehole S204-18 below a depth of 36.1 m. The top of the silty clay to clay deposit is at about Elevation 155.4 m and 158.2 m and the thickness of this deposit is about 16.9 m and 15.0 m in the respective boreholes.

The SPT 'N'-values recorded within the silty clay to clay deposit generally range from 5 blows to 12 blows per 0.3 m of penetration. One (1) SPT 'N'-value measured within the bottom portion of the clay deposit in Borehole B202-02 is 26 blows per 0.3 m of penetration. In situ field vane tests carried out within this deposit measured undrained shear strengths ranging from about 71 kPa to greater than 120 kPa and the sensitivity is calculated to range from about 1 to 4. The field vane tests results indicate that the lower silty clay to clay deposit has a stiff to very stiff consistency.

The natural water content measured on four (4) specimens of this deposit ranges from about 31 per cent to 46 per cent.

The grain size distributions of two (2) samples of the silty clay to clay deposit are shown on Figure B8 in Appendix B

Atterberg limits tests were carried out on two (2) specimens of the silty clay to clay deposit and indicate liquid limits of about 35 per cent and 56 per cent, plastic limits of about 14 per cent and 21 per cent and corresponding plasticity indices of about 21 per cent and 35 per cent. The results of the Atterberg limits tests are shown on the plasticity chart on Figure B9 in Appendix B and indicate the material to be silty clay of intermediate plasticity to clay of high plasticity.

### 4.3.9 Silt to Sand (Lower)

Underlying the lower deposit of silty clay to clay, an interlayered non-cohesive deposit comprised of sand to silty sand to sand and silt to silt was encountered in Boreholes B202-02 and S204-18. The top of the silt to sand deposit is at about Elevation 138.5 m and 143.2 m and the deposit was penetrated for about 6.3 m and 4.9 m in the respective boreholes. Borehole S204-18 was terminated within the sand silt portion of this deposit, while Borehole B202-02 was extended deeper by driving a DCPT to refusal at a depth of about 53.1 m below ground surface (Elevation 127.9 m).

The SPT 'N'-values recorded within this interlayered deposit range from 12 blows to 52 blows per 0.3 m of penetration, generally indicating a compact to very dense relative density.

The natural water content measured on four (4) samples of this deposit range from about 19 per cent to 25 per cent.

A grain size distribution of one (1) sample from the sand and silt portion of this deposit is shown on Figure B10 in Appendix B.

### 4.3.10 Groundwater Conditions

In general, the overburden samples taken in the boreholes were moist to wet. Artesian conditions were encountered in Borehole B202-02 during drilling at a depth of 45.1 m below the ground surface (Elevation 135.9 m) and the groundwater level recorded in the casing on February 16, 2011 was at about Elevation 182 m, measured at 1 m above ground surface. In Boreholes B202-01 and S204-18, the water level observed upon completion of drilling was at about Elevation 177.3 m (measured at a depth of about 3.8 m below ground surface) and 177.4 m (measured at a depth of about 3.7 m below ground surface), respectively.

A standpipe piezometer was installed in Borehole SP1 located on the centreline and to the west of the south abutment to permit monitoring of the groundwater level in this area. The details of the piezometer installations are shown on the Record of Borehole sheets 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 |
|---|---|---|---|---|
| South Abutment | SP1 | 181.1 | 177.1<br>177.1 | February 17, 2011<br>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 B202-03, B202-14 and B202-15) and two (2) probeholes (B202-P01 and B202-P02) 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 deposit of clayey silt to clay and bedrock.

### 4.4.1 Topsoil

An approximately 0.2 m thick layer of topsoil was encountered at the ground surface at all borehole locations. The surface of the topsoil ranges from about Elevation 179 m to 178.7 m.

### 4.4.2 Sand to Silt

A brown to grey non-cohesive deposit varying in composition from sand trace to some silt, to silty sand, to sand and silt trace clay, to silt some clay was encountered below the topsoil in all the boreholes. In general, the upper portion of the deposit contains rootlets and organics and clay lenses in places. The top of the silty sand to silt deposit ranges from about Elevation 178.8 m to 178.5 m and the thickness of this deposit varies between about 5.0 m and 7.9 m.

The SPT 'N'-values measured within this deposit range from 0 blows (weight of hammer) to 6 blows per 0.3 m of penetration, indicating a very loose to loose relative density.

The natural water content measured on twelve (12) samples of this deposit ranges from about 23 per cent to 35 per cent.

The grain size distributions of four (4) samples of the sand and silt to silt portion of this deposit are shown on Figure B11 in Appendix B.

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 of slight plasticity.

### 4.4.3 Clayey Silt and Clay

A deposit of grey clay was encountered underlying the sand to silt deposit at all borehole locations. In Borehole B202-03 the upper 1.5 m portion of the deposit is described as a clayey silt and in Boreholes B202-14 and B202-15 the deposit contains silt interlayers. The top of the clayey silt to clay deposit ranges from about Elevation 173.5 m to 170.9 m and the thickness of the deposit ranges from about 17.2 m to 20.0 m. The bottom of the deposit was defined by refusal to further split-spoon advancement and bedrock coring in Borehole S202-03. Borehole B202-15 was extended deeper by driving a dynamic cone to refusal at a depth of about 23.1 m below ground surface (Elevation 155.6 m), while Borehole B202-14 was extended deeper by first driving a dynamic cone to a depth of about 27.4 m below the ground surface (Elevation 151.6 m) and then advancing a tricone to refusal at a depth of 28.1 m below the ground surface (Elevation 150.9 m).

SPT 'N'-values measured within the clay deposit range from 1 blow to 3 blows per 0.3 m of penetration. In situ field vane tests carried out within the deposit measured undrained shear strengths typically ranging from about 38 kPa to 55 kPa, with one (1) undrained shear strength which is greater than 96 kPa. The sensitivity is calculated to be about 3 and 4. The field vane tests results indicate that the clay deposit generally has a firm to stiff consistency.

The natural water content measured on five (5) samples of this deposit ranges from about 36 per cent to 76 per cent, with the lower water content value being measured in the clayey silt portion of the deposit.

The grain size distributions of four (4) samples of this deposit are shown on Figure B13 in Appendix B.

Atterberg limits tests carried out on five (5) specimens of this deposit indicate a liquid limit of about 28 per cent and a plastic limit of about 18 per cent and a corresponding plasticity index of about 10 per cent for the clayey silt portion of the deposit; and liquid limits between about 51 per cent and 77 per cent, plastic limits between about 18 per cent and 24 per cent and plasticity indices between about 31 per cent and 53 per cent for the main clay deposit 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 clayey silt of low plasticity to clay of high plasticity.

### 4.4.4 Bedrock / Refusal

Bedrock was encountered below the clay deposit and core samples were recovered in Borehole B202-03. The bedrock surface was inferred from refusal to casing advancement in Probeholes B202-P01 and B202-P02, refusal to tricone advancement in Borehole B202-14, and refusal to dynamic cone penetration in Borehole B202-15. 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 summarized below.

| Foundation Element | Borehole No. / Probehole No. | Depth to Bedrock Surface / Refusal (m) | Bedrock Surface / Refusal Elevation (m) | Comments |
|---|---|---|---|---|
| Centre Pier | B202-03 | 24.1 | 154.7 | Bedrock Cored |
| | B202-14 | 28.1 | 150.9 | Tricone Refusal |
| | B202-15 | 23.1 | 155.6 | DCPT Refusal |
| | B202-P01 | 25.6 | 153.3 | Casing Refusal |
| | B202-P02 | 25.7 | 153.4 | Casing Refusal |

In general, the bedrock surface slopes downwards to the south-east towards the river across the proposed centre pier footprint (up to approximately 3.3H:1V slope or a dip of approximately 17° from the horizontal). Across the proposed centre pier, the bedrock drops in elevation by as much as about 2.5 m over a distance of about 4 m (from northeast corner to southeast corner), which is equal to a slope of about 1.6H:1V or a dip of approximately $32^{\circ}$ from the horizontal).

Based on the review of the cored bedrock samples, the bedrock consists of granite gneiss. The bedrock samples are described as fresh, foliated, medium crystalline, slightly porous, strong, pink, grey and black, as presented on the Record of Drillhole sheets in Appendix A. A photograph of the recovered samples is shown on Figure B15 in Appendix B. The degree of weathering of the bedrock samples (i.e. fresh – W1) and the strength classification of the rock mass based on field identification is medium strong to strong (i.e. R2 to R4) are described in accordance with the International Society for Rock Mechanics (ISRM) standard classification system.

The Rock Quality Designation (RQD) measured on the core samples is between 48 per cent and 84 per cent, indicating a rock mass of poor to good quality. The RQD measured on one core sample (only 0.31 m in length) is 48 per cent, indicating a rock mass of poor quality. The Total Core Recovery (TCR) and Solid Core Recovery (SCR) of all samples recovered is 100 per cent and between 75 per cent and 90 per cent, respectively.

Point load tests were carried out on selected samples of the rock 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. An axial test carried out on one (1) core sample of the granite gneiss bedrock measured an $Is_{50}$ value of about 1.7 MPa and the diametral test carried out on one (1) core sample of the granite gneiss bedrock measured an $Is_{50}$ value of 2.7 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 granite gneiss bedrock obtained from Borehole B202-03 and measured a compressive strength of about 69 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) in accordance with 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 granite gneiss bedrock is classified as medium strong (R3, 25 MPa < UCS < 50 MPa) to strong (R4, 50 MPa < UCS < 100 MPa).

### 4.4.5 Groundwater Conditions

In general, the overburden samples taken in the boreholes were moist to wet. Water levels observed in Boreholes B202-14 and B202-15 and Probehole B202-P01 upon completion of drilling range from about Elevation 178.1 m to 177.8 m, measured between about 0.8 m and 1.1 m below ground surface. Artesian conditions were observed in Probehole B202-P02 during drilling and the groundwater level recorded in the casing on March 31, 2011 was at about Elevation 181.4 m, measured at 2.3 m above ground surface.

A standpipe piezometer was installed in Borehole B202-03 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 |
|---|---|---|---|---|
| Centre Pier | B202-03 | 178.8 | 178.6 | March 26, 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.

## 4.5 North Abutment and Approach Embankment (One-Span Bridge)

A total of three (3) boreholes (Boreholes B202-04, B202-12 and B202-13) and two (2) Dynamic Cone Penetration Tests (DCPTs B202-DC01 and B202-DC02) were advanced at the location of the proposed north abutment and one (1) borehole (Borehole B202-05) was advanced on the alignment centerline at the north approach embankment. In general, the subsurface conditions encountered at the north abutment consist of topsoil underlain by a deposit of sand to silt and a deposit of clayey silt to clay which in turn is underlain by bedrock. The subsurface conditions encountered at the north embankment approach are comprised of a deposit of clayey silt to clay with a sandy silt interlayer underlain by bedrock

### 4.5.1 Topsoil

A 0.2 m to 0.3 m thick layer of topsoil was encountered at the ground surface at all borehole locations.

### 4.5.2 Sand to Silt

A non-cohesive deposit comprised of brown to grey sand trace gravel to silty sand trace to some clay to sandy silt to silt trace to some clay was encountered below the topsoil in Boreholes B202-04, B202-12 and B202-13. The upper portion of the silty sand to sandy silt deposit in Boreholes B202-12 and B202-13 contains rootlets and organics. The silt portion of the non-cohesive deposit encountered in Boreholes B202-04 and B202-13 contains silty clay lenses. The top of this deposit is at about Elevation 179.2 m and the thickness of this deposit ranges between about 2.3 m and 5 m.

The SPT 'N'-values measured within this deposit range from 0 blows (weight of hammer) to 8 blows per 0.3 m of penetration, indicating a very loose to loose relative density.

The natural water content measured on nine (9) samples of this deposit ranges from about 20 per cent to 34 per cent.

The grain size distributions of four (4) samples of the silty sand to silt portion of this deposit are shown on Figure B20 in Appendix B.

An Atterberg limits test carried out on a sample of the silt portion of the deposit indicates a liquid limit of about 20 per cent, a plastic limit of about 18 per cent and a plasticity index of 2 per cent, The results of the Atterberg limits test is shown on the plasticity chart on Figure B21 in Appendix B and indicates that the material is classified as silt of slight plasticity.

### 4.5.3 Clayey Silt to Clay

A deposit of cohesive soil comprised of brown to grey clayey silt with sand, silty clay trace sand and clay was encountered underlying the sand to silt deposit or topsoil at all borehole locations. In Boreholes B202-05 the cohesive deposit contains an approximately 0.7 m thick sandy silt interlayer, as discussed in Section 4.5.4. The top of the cohesive deposit ranges from about Elevation 181.1 m to 174.2 m and the thickness of the cohesive deposit ranges from about 3.3 m to 10.1 m. Borehole B202-13 was extended deeper by driving a dynamic cone to refusal at a depth of about 15.3 m below ground surface (Elevation 164.1 m).

The SPT 'N'-values recorded within the clayey silt to clay deposit generally range from 0 blows (weight of hammer) to 8 blows per 0.3 m of penetration. In situ field vane tests carried out within this deposit measured undrained shear strengths ranging from about 24 kPa to 62 kPa and sensitivity is calculated to range from about 3 to 5. The field vane tests results indicate that the clayey silt to clay deposit has a soft to stiff consistency.

The natural water content measured on seven (7) samples of this deposit ranges from about 37 per cent to 74 per cent, but is generally greater than 50 per cent.

The grain size distributions of three (3) samples of the silty clay to clay portion of this deposit are shown on Figure B22 in Appendix B.

Atterberg limits tests were carried out on four (4) specimens of the clayey silt to clay portion of this deposit and indicate liquid limits ranging from about 32 per cent to 63 per cent, plastic limits ranging from about 16 per cent to 21 per cent and plasticity indices ranging from about 16 per cent to 45 per cent. The results of the Atterberg limits tests are shown on the plasticity chart on Figures B23 in Appendix B and indicate the material to be clayey silt of low plasticity to clay of high plasticity.

A laboratory consolidation test was carried out on one (1) specimen of the clay deposit obtained from a Shelby tube sample in Borehole B202-04. A preconsolidation stress of about 155 kPa was estimated from the void ratio versus logarithmic pressure plot and from the total work versus pressure plot. A bulk unit weight of about 16.5 kN/m$^3$ and a specific gravity of about 2.77 were measured on the consolidation test specimen. Details of the test results are shown on Figure B24 in Appendix B, and the test results are summarized below.

| Borehole Sample No. | Sample Depth / Elevation | $\sigma_{vo}'$ (kPa) | $\sigma_p'$ (kPa) | $\sigma_p' - \sigma_{vo}'$ (kPa) | OCR | $C_c$ | $C_r$ | $e_o$ | $c_v^*$ (cm$^2$/s) |
|---|---|---|---|---|---|---|---|---|---|
| Borehole B202-04 Sample 9 | 7.5 m / 171.9 m | 75 | 155 | 80 | 2.1 | 0.95 | 0.11 | 1.56 | $1.97 \times 10^{-3}$ |

Note: * For stress range of between effective overburden stress and final stress due to 9.5 m high approach embankment, that is 75 kPa ≤ $\sigma_v'$ ≤ 255 kPa

where:

| | |
|---|---|
| $\sigma_{vo}'$ | is the in situ vertical effective overburden stress in kPa |
| $\sigma_p'$ | is the preconsolidation stress in kPa |
| $\sigma_v'$ | 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 cm$^2$/s |

### 4.5.4 Sandy Silt Interlayer

An approximately 0.7 m thick interlayer of grey sandy silt was encountered within the clayey silt to silty clay deposit in Borehole B202-05 at about Elevation 179.9 m.

One (1) SPT ‘N’-value measured within the sandy silt interlayer is 28 blows per 0.3 m of penetration, indicating a compact relative density.

### 4.5.5 Bedrock / Refusal

Bedrock was encountered below the clayey silt to clay deposit and core samples were recovered in Boreholes B202-04, B202-05 and B202-12. The bedrock surface was inferred from refusal to dynamic cone penetration in Borehole B202-13 and in DCPTs B202-DC01 and B202-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 summarized below.

| Foundation Element / Approach Embankment | Borehole No. / DCPT No. | Depth to Bedrock Surface / Refusal (m) | Bedrock Surface / Refusal Elevation (m) | Comments |
|---|---|---|---|---|
| North Abutment (One-Span Bridge) | B202-04 | 9.8 | 169.6 | Bedrock Cored |
| | B202-12 | 5.9 | 173.6 | Bedrock Cored |
| | B202-13 | 15.3 | 164.1 | DCPT Refusal |
| | B202-DC01 | 13.4 | 165.6 | DCPT Refusal |
| | B202-DC02 | 12.8 | 167.4 | DCPT Refusal |
| North Approach Embankment | B202-05 | 4.6 | 176.7 | Bedrock Cored |

In general, the bedrock surface slopes down to the south-east towards the river from the borehole advanced at the centreline of the north approach embankment and across the proposed north abutment footprint. Across the proposed one-span north abutment, the bedrock drops in elevation by as much as about 8 m over a distance of about 4 m (from northwest corner to southwest corner), which is equal to a slope of about 0.5H:1V or a dip of approximately 63° from the horizontal).

Based on the review of the bedrock core samples, the bedrock consists of granite gneiss. 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. Photographs of the recovered samples are shown on Figures B25 to B27. The degree of weathering of the bedrock samples is fresh (i.e. W1) and the strength classification of the rock mass based on field identification is strong (i.e. R4).

The Rock Quality Designation (RQD) measured on the core samples ranges from 85 per cent to 100 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 100 per cent and between 63 per cent and 100 per cent, respectively.

Point load tests were carried out on selected samples of the rock 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 two (2) core samples of the granite gneiss bedrock measured $Is_{50}$ values of about 5.2 MPa and 10.7 MPa and the diametral test carried out on two (2) core samples of the granite gneiss bedrock measured $Is_{50}$ value of about 9.3 MPa and 10.6 MPa.

One (1) Unconfined Compression (UC) test (ASTM D7012-10 – Standard Test Method for Compressive Strength and Elastic Moduli of Intact Rock Core Specimens) was carried out on a selected core sample of granite gneiss bedrock obtained in Borehole B202-04 and measured a compressive strength of about 193 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) in accordance with ASTM D5731-08 – 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 granite gneiss bedrock is classified as strong (R4, 50 MPa < UCS < 100 MPa) to 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 ranges between about Elevation 179.5 m and 178.0 m, measured between about 1.3 m and 1.8 m below ground surface.

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 Bridge)

A total of four (4) boreholes (Boreholes B202-06 to B202-09) and one hand shovel excavation (B202-10) were advanced at the location of the proposed north abutment and one (1) hand shovel excavation (B202-11) was advanced on the alignment centerline of the north approach embankment. In general, the subsurface conditions consist of peat/topsoil and/or cobbles and boulders over bedrock.

### 4.6.1 Peat / Topsoil

An approximately 0.2 to 0.6 m thick layer of black amorphous peat was encountered at the ground surface in Boreholes B202-06 to B202-09.

Two (2) SPT ‘N’-values measured within the peat are 10 blows and 39 blows per 0.3 m of penetration, suggesting a stiff and hard consistency, respectively. It should be noted that the high ‘N’-value (39 blows) can be attributed to increased resistance during split-spoon advacment into frozen ground.

The natural water content measured on two (2) specimens of the peat are about 520 per cent and 890 per cent, while the natural content measured on one (1) specimen of the hard consistency peat is about 33 per cent.

A layer of topsoil, about 0.1 m thick, was encountered at the ground surface in the hand shovel excavations, Boreholes B202-10 and B202-11.

### 4.6.2 Cobbles and Boulders

A deposit of cobbles and boulders was encountered below the peat in Boreholes B202-07 and B202-08, interlayered with seams of clayey silt, sand and gravel in Borehole B202-08. The top of the deposit is at Elevation 183.7 m and 183.9 m and the thicknesses of the deposit is about 0.5 m and 2.6 m in the respective boreholes. Photographs of the recovered core of the cobbles and boulders layer are presented on Figures B17 and B18 in Appendix B.

### 4.6.3 Sand to Gravel

A deposit of brown gravelly sand was encountered below the topsoil in Borehole B202-06 and interlayers of brown clayey silt, brown sand and grey gravel were encountered within the deposit of cobbles and boulders in Borehole B202-08. The top of the gravelly sand deposit is at about Elevation 183.7 m and the thickness of the deposit is about 0.7 m. Borehole B202-06 was terminated within this deposit. The top of the clayey silt and sand and gravel interlayers is at about Elevation 183.3 m and the thickness of the interlayers is about 0.1 m.

One (1) SPT 'N'-value measured within the gravelly sand is 10 blows per 0.3 m of penetration, indicating a compact relative density. SPT 'N'-values measured within the sand and gravel interlayers are 100 blows per 0.08 m and 100 blows per 0.1 m, respectively, as a result of driving the split-spoon into the cobbles and boulders.

The natural water content measured on one (1) sample of gravelly sand is about 13 per cent.

A grain size distribution of one (1) sample of the gravelly sand is shown on Figure B16 in Appendix B.

### 4.6.4 Bedrock / Refusal

Bedrock was encountered in Borehole B202-07 to B202-09 below the peat/topsoil and underlying the cobbles and boulders deposit; and core samples were recovered. The bedrock surface was inferred from split-spoon refusal in Borehole B202-06, and exposed by shovel (hand) excavation to confirm refusal at the bottom of the topsoil layer in Boreholes B202-10 and B202-11. The depth to bedrock below ground surface and corresponding bedrock surface elevation (inferred or actual) is summarized below.

| Foundation Element / Approach Embankment | Borehole No. | Depth to Bedrock Surface / Refusal (m) | Bedrock Surface / Refusal Elevation (m) | Comments |
|---|---|---|---|---|
| North Abutment (Two-Span Bridge) | B202-06 | 0.9 | 183.0 | Spoon Refusal |
| | B202-07 | 1.1 | 183.2 | Bedrock Cored |
| | B202-08 | 2.9 | 181.3 | Bedrock Cored |
| | B202-09 | 0.2 | 186.2 | Bedrock Cored |
| | B202-10[1] | 0.1 | 187.0 | Shovel Refusal |
| North Approach Embankment | B202-11[1] | 0.1 | 187.2 | Shovel Refusal |

Note: 1. Boreholes B202-10 and B202-11 refer to a shovel excavation carried out at the north-east corner of the north abutment (two-span bridge) and at the north approach embankment, respectively, to expose the bedrock surface.

In general, the bedrock surface in the area of the north abutment footprint and north approach embankment slopes down to the south-west towards the river, but locally within the abutment footprint the bedrock is at a lower elevation and forms a trough in the centre of the abutment footprint varying by about 5.7 m (approximately 1.4. Therefore, near the proposed north abutment and its approach embankment, the bedrock elevation varies by about 5.9 m over a distance of about 20 m (approximately 3.4H:1V slope or a dip of approximately 16º from the horizontal).

Based on the review of the cored bedrock samples, the bedrock consists of gneiss to granite gneiss with occasional zones of schist. In general the gneiss to granite gneiss bedrock samples are described as predominantly fresh (a slightly weathered zone was encountered in Borehole B202-07 from the bedrock surface to a depth of 3.3 m), generally foliated, medium to coarsely crystalline, slightly porous, strong, pink, grey and black and containing a mafic dyke between a depth of 5.86 m and 5.94 m in Borehole B202-08, as presented on the Record of Drillhole sheets in Appendix A. Photographs of the recovered bedrock core samples are shown on Figures B17 to B19 in Appendix B. Occasional zones of schist were encountered in Borehole B202-09 at depths of 0.2 m and 1.3 m, corresponding to Elevation 186.2 m and 185.1 m, respectively. The schist bedrock samples are generally described as fresh (a highly weathered zone was encountered from a depth of 1.3 m to 1.4 m and contained rootlets), coarsely crystalline, slightly porous, strong, brown and black. The degree of weathering of the bedrock samples is slightly weathered to fresh (i.e. W2 to W1) and the strength classification of the rock mass based on field identification strongto extremely strong (i.e. R4 to R6).

The Rock Quality Designation (RQD) measured on the core samples generally ranges between 62 per cent and 100 per cent, indicating a rock mass of fair to excellent quality. However, the RQD measured on a core sample recovered betwen a depth of about 2.9 m and 3.6 m from Borehole B202-08 was 0 per cent, indicating a rock mass of very poor quality. The Total Core Recovery (TCR) of samples recovered is typically between 94 per cent and 100 per cent. Solid Core Recovery (SCR) of samples recovered varies between 25 per cent and 100 per cent.

Point load tests were carried out on selected samples of the rock 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 two (2) core samples of the granite gneiss bedrock measured $Is_{50}$ values of about 5.3 MPa and 8.3 MPa and the diametral tests carried out on three (3) core samples of the gneiss to granite gneiss bedrock measured $Is_{50}$ values ranging from about 8.1 MPa to 14.1 MPa. An axial test carried out on one (1) core sample of the schist bedrock measured an $Is_{50}$ value of 1.4 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 granite gneiss bedrock obtained in the area of the proposed north abutment and measured a compressive strength of about 175 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) in accordance with 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 gneiss to granite gneiss bedrock is generally classified as very strong (R5, 100 MPa < UCS < 250 MPa) to extremely strong (R6, UCS > 250 MPa), and the schist bedrock (based on one (1) point load test) is classified as medium strong (R3, 25 MPa < UCS < 50 MPa).

### 4.6.5 Groundwater Conditions

In general, the overburden samples taken in the boreholes were moist to wet. Borehole B202-06 was dry upon completion of drilling and the water level observed in Boreholes B202-07 and B202-09 upon completion of drilling was at Elevation 183.9 m to 184.6 m, measured at 0.4 m and 1.8 m below ground surface, respectively.

A standpipe piezometer was installed in Borehole B202-08 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 | B202-08 | 184.2 | 183.4<br>183.5 | February 28, 2011<br>March 31, 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.

## 5.0 CLOSURE

Mr. Matt Rhody, a senior technician with Golder and Messrs. Tony Tomory, E.I.T. and Alexander Mayot, E.I.T., directed the drilling program. This report was prepared by Mr. Tomasz Zalucki, P.Eng., a geotechnical engineer, 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 Principal with Golder, conducted an independent quality control review of the report.

## Report Signature Page

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# PART B

## FOUNDATION DESIGN REPORT
## STILL RIVER NBL BRIDGE STRUCTURE, SITE NO. 44-458/1
## HIGHWAY 69 FOUR-LANING FROM 1.7 KM NORTH OF HIGHWAY 529 NORTHERLY TO 3.9 KM NORTH OF HIGHWAY 522
## MINISTRY OF TRANSPORTATION, ONTARIO
## GWP 5404-05-00; WP 5139-08-01

# 6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides engineering recommendations for detail design of the proposed Still River NBL 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 NBL 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 NBL Bridge crossing. However, since the feasibility of a one-span bridge option was being evaluated during the initial design stage, the following sections include 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 NBL Bridge deck varies between about Elevation 188.9 m and Elevation 189.4 m. The existing ground surface varies between about Elevation 187.3 m and Elevation 178.7 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 location, 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 23 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 firm to stiff and compressible clayey silt to silty clay deposits up to about 19.3 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 the very loose to loose sand to silt deposit overlying firm to stiff and compressible clayey silt to silty clay deposits that vary between about 3.3 m and 10.1 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 granite gneiss bedrock or founded on mass concrete placed on the properly prepared gneiss or granite gneiss bedrock.

The existing ground surface at the borehole locations in the area of the north abutment footprint varies from about Elevation 187.1 m to 183.9 m. At the east end of the proposed abutment, the 0.1 m to 0.3 m thick overburden consists of topsoil or peat. At the center of the proposed abutment, the 2.9 m thick overburden consists of 0.3 m of peat underlain by zones of cobbles and boulders (approximately 0.6 m to 0.9 m thick) interlayered by thin layers of clayey silt, sand, and gravel. At the west end of the proposed abutment, the 0.9 m to 1.1 m thick overburden consist of peat underlain by silty gravelly sand or peat underlain by a 0.5 m thick zone of cobbles and boulders. The bedrock surface elevation as encountered in the boreholes at the proposed north abutment / foundation element varies considerably and is summarized below.

| Location | Relevant Borehole | Ground Surface Elevation (m) | Bedrock Surface Elevation (m) |
|---|---|---|---|
| Northwest Corner | B202-07 | 184.3 | 183.2 |
| Southwest Corner | B202-06 | 183.9 | 183.0[1] |
| Centre | B202-08 | 184.2 | 181.3 |
| Northeast Corner | B202-10 | 187.1 | 187.0 |
| Southeast Corner | B202-09 | 186.4 | 186.1 |

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 183.0 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 0.2 m of bedrock will be required to construct the western portion spread footing on bedrock. At the centre of the proposed abutment, an up to about 2.9 m deep excavation (comprised mostly of zones of cobbles and bedrock) will be required to reach the bedrock. In addition, removal of up to about an additional 0.7 m of broken bedrock (as encountered in Borehole B202-08) may be required. Consequently, placement of mass concrete (ranging from about 1.7 m to 2.4 m thick) on properly prepared bedrock will be required to reach the proposed founding elevation near the centre of the abutment. Along the eastern portion of the proposed abutment, an up to about 4.1 m deep excavation including removal of up to about 4 m of bedrock will be required to construct the spread footing on bedrock.

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 surficial overburden soils and/or 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 granite gneiss bedrock (at east and west side of footing) | 10,000 kPa | N/A[2] |

| 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 footings on mass concrete[1] on properly prepared gneiss or granite gneiss bedrock (at centre 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 will 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 geotechicnical 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 δ, for the interface materials.

| Interface Materials | Coefficient of Friction (tan δ) |
|---|---|
| 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 or mass concrete founded directly on the properly prepared granite or granite 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 about 53 m deep and generally loose to compact sand to silt and soft to stiff clayey silt to silty 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:

| Pile Section | Structural Pile Capacity | Unfactored Dead Load per Pile | Factored Dead Load per Pile | Factored Geotechnial 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 very dense sand deposit underlying lower silty clay to 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. 137 m / 45 m | Piles driven to the top of the very dense sand | 25.5 m | 750 kN | 1,800 kN | 2,150 kN + 400 kN = 2,550 kN | 20 mm |
| HP 310x110 / Elev. 135 m / 47 m | Piles driven 2 m into very dense sand | 26.5 m | 750 kN | 1,900 kN | 2,250 kN + 400 kN = 2,650 kN | 25 mm |
| HP 310x110 / Elev. 134 m / 48 m | Piles driven 3 m into very dense sand | 27.5 m | 750 kN | 2,000 kN | 2,350 kN + 400 kN = 2,750 kN | 25 mm |
| HP 310x110 / Elev. 133 m / 49 m | Piles driven 4 m into very dense sand | 28 m | 750 kN | 2,050 kN | 2,400 kN + 400 kN = 2,800 kN | 20 mm |

| 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. 128 m / 54 m | Piles driven to practical refusal [2] | 30 m | 750 kN | 2,250 kN | 2,750 kN + 250 kN = 3,000 kN | 15 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 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:

| 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. 137 m / 45 m | 2,250 kN + 900 kN = 3,150 kN | 4,200 kN | 1,600 kN | 1,100 kN | 20 mm | Yes |
| HP 310x110 / Elev. 135 m / 47 m | 2,375 kN + 900 kN = 3,275 kN | 4,200 kN | 1,800 kN | 1,100 kN | 25 mm | Yes |
| HP 310x110 / Elev. 134 m / 48 m | 2,500 kN + 900 kN = 3,400 kN | 4,200 kN | 1,800 kN | 1,100 kN | 25 mm | Yes |
| HP 310x110 / Elev. 133 m / 49 m | 2,550 kN + 900 kN = 3,450 kN | 4,200 kN | 1,800 kN | 1,100 kN | 20 mm | Yes |
| HP 310x110 / Elev. 128 m / 54 m | 2,825 kN + 900 kN = 3,725 kN | 4,200 kN | 2,000 kN | 1,100 kN | 15 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 134 m (i.e. 48 m long piles driven about 3 m into the very dense silt and 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 25 mm.

In summary, the following factored geotechnical axial resistance at ULS and the geotechnical reaction at Serviceability Limits States (SLS) are recommended for 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 25 mm of Settlement [2] |
|---|---|---|---|---|
| HP 310x110 piles driven 3 m into very dense silt and sand | 134 m | 48 m | 1,800 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. less than about 25 m) of the very loose to loose sand to silt and firm to stiff clayey silt 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 32° 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 medium strong to 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.

| 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) | 153.4 m to 150.9 m | 23.6 m to 26.1 m | 1,600 kN | N/A [2] |
| HP 310x110 piles driven into bedrock (centre of pier) | 154.7 m | 22.3 m | 1,600 kN | N/A [2] |
| HP 310x110 piles driven into bedrock (west side of pier) | 155.6 m to 153.3 m | 21.4 m to 23.7 m | 1,600 kN | N/A [2] |

Notes:
1. Assuming the underside of the pile cap is at approximately Elevation 177 m as per the April 2011 GA Drawing provided by URS.
2. The geotechnical reaction at SLS for 25 mm of pile settlement on granite 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 15 m) of the very loose to loose sand to silt deposit and firm to stiff clayey silt 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 firm to stiff clayey silt 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.

Considering the steeply sloping bedrock at this location, the piles must be provided with pile points. Specifically, given the estimated steep angle of the bedrock surface (up to a dip of approximately 63° from the horizontal), 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 bedrock. The details of the Olso pile points are as per OPSD 3000.201 *Steel HP 310 Olso Point*. The appropriate NSSP should be included in the Contract Documents; an example is included in Appendix C for reference. The driving procedures to enable pile seating depends on the type of pile driving rig used; these 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 geotechnical reaction at SLS for the steel H-pile foundation option at the proposed north abutment of the one-span option. It should be noted that the factored geotechnical resistance at ULS has been reduced by a factor of 0.8 to take into consideration the steeply sloping nature at the bedrock at the proposed north abutment location.

| 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 164.1 m | 9.4 m to 19.9 m | 1,600 kN | N/A [2] |
| HP 310x110 piles driven into bedrock (centre of abutment) | 169.6 m | 14.4 m | 1,600 kN | N/A [2] |
| HP 310x110 piles driven into bedrock (west side of abutment) | 173.6 m to 173.5 m | 10.4 m to 10.5 m | 1,600 kN | N/A [2] |

Notes:
1. Assuming the underside of the pile cap is at approximately Elevation 184 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 granite 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.

Considering the sloping bedrock and the variability in the top of the probable bedrock surface at the east and west side of the proposed abutment, provisions should be made in the Contract Documents to deal with varying pile lengths at the location of the north abutment.

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 164.1 m | 9.4 m to 19.9 m | 125 kN to 550 kN |
| HP 310x110 piles driven to bedrock (centre of abutment) | 169.6 m | 14.4 m | 300 kN |
| HP 310x110 piles driven to bedrock (west side of abutment) | 173.6 m to 173.5 m | 10.4 m to 10.5 m | 150 kN |

Note:
1. Assuming the underside of the pile cap is at approximately Elevation 184 m as per the preliminary April 2011 GA Drawing provided by URS.

For the EPS embankment construction option, in order to eliminate additional downdrag 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 100 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 granite 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 drivne 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 3,600 kN per pile but must be driven below Elev. 135 m and not below Elev. 133 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:

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

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 Figures 1 and 2) 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 latral 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 (B202-01, B202-02 and S204-18) | Embankment Fill above Water / Groundwater Level (assumed to be Compacted Granular Fill) | 182.0 – 181.0 | 7,500 [1] | - |
| | Loose Silty Sand | 181.0 – 180.3 | 3,000 | |
| | Soft Clayey Silt to Silty Clay (Near Surface) | 180.3 – 178.6 | - | 25 |
| | Loose to Compact Sand to Silt (Upper) | 178.6 – 172.8 | 3,000 | - |
| | Firm Silty Clay to Clay (Upper) | 172.8 – 170.9 | - | 40 |
| | Compact Silt Interlayer | 170.9 – 168.2 | 3,000 | - |
| | Firm Silty Clay to Clay (Upper) | 168.2 – 165.0 | - | 40 |
| | Stiff Silty Clay to Clay (Upper) | 165.0 – 161.2 | - | 60 |
| | Loose Silt to Silty Sand Interlayers | 161.2 – 157.8 | 2,000 | - |
| | Dense Silt to Silt Sand Interlayers | 157.8 – 155.4 | 5,000 | - |
| | Stiff Silty Clay to Clay (Lower) | 155.4 – 150.0 | - | 60 |
| | Stiff to Very Stiff Silty Clay to Clay (Lower) | 150.0 – 138.5 | - | 60 – 100 |
| | Compact Silt to Sand (Lower) | 138.5 – 137.0 | 5,000 | - |
| | Dense to Very dense Silt to Sand (Lower) | 137.0 – 127.9 | 15,000 | - |
| Centre Pier – Two-Span Option (B202-03, B202-14 and B202-15) | Very Loose to Loose Sand to Silt | 178.8 – 171.9 | 2,000 | - |
| | Firm to Stiff Clayey Silt to Clay | 171.9 – 154.7 | - | 35 – 50 |
| North Abutment – One-Span Option (B202-04, B202-12 and B202-13) | Embankment Fill above Water / Groundwater Level (assumed to be Compacted Granular Fill) | 184.0 – 179.4 | 7,500 [1] | - |
| | Very Loose to Loose Sand to Silt | 179.4 – 175.2 | 2,000 | - |
| | Soft to Firm Clayey Silt to Clay | 175.2 – 169.9 | - | 25 – 40 |

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 Figures 1 and 2.

For a single HP 310x110 and HP 310x152 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 (> 46 m long pile) | 85 kN | 25 kN |
| | HP 310x152 (53 m long pile) | 150 kN | 30 kN |
| Centre Pier – Two-Span Option | HP 310x110 | 135 kN | 25 kN |
| North Abutment – One-Span Option | HP 310x110 | 175 kN | 25 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 ($k_h$) 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·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 technique and/or increase the potential arising during caisson construction, such as:

- The difficulty associated with socketing large diameter caissons into the predominantly strong to very strong bedrock at the centre pier and north abutment (one-span option) where the bedrock surface is realtively 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 recommedned 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. Compaction (including type of equipment, target densities, etc.) should be carried out in accordance with Special Provision 105S21 *Compacting*. The granular backfill requirements should be in accordance with OPSD 3101.150 *Walls, Abutment, Backfill, Minimum Granular Requirement* and OPSD 3121.150 *Walls, Retaining, Backfill, Minimum Granular Requirement*. Longitudinal drains and weep holes should be installed in accordance with OPSD 3102.100 *Walls, Abutment, Backfill Drain* and OPSD 3190.100 *Walls, Retaining and Abutment, Wall Drain* to provide positive drainage of the granular backfill.

- For structures that are not comprised of integral or semi-integral abutments, rock fill may be used as backfill behind the walls and the material should meet the specification as outlined in the Northeastern Region Directive (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 unrestrainted 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, $\gamma$ (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, $\gamma$ (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·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;
$\gamma$ = 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 NBL bridge will require placement of up to about 7.5 m of fill within the limits of the south approach embankment and up to about 9.5 m or 3.5 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 loose to very 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 silt underlain by firm to stiff clayey silt 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 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, $\gamma$ (kN/m$^3$) | Effective Friction Angle, $\phi$' (º) | 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)

$\sigma'_p{}'$ = 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:

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

where:

$s_{u(mob)}$ = average mobilized undrained shear strength (kPa)
$s_{u(FV)}$ = undrained shear strength from field vane test (kPa)
$\mu$ = 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 7.5 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 east 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 east slope, a 3 m high by 3.5 m wide stability berm would be required along the outside toe of the NBL 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 1.2 for deep-seated, global failure surfaces of front slope and east 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 east slope, a 5 m high by 8 m wide stability berm would be required along the outside toe of the NBL 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 two (2) consolidation tests that were carried out for the NBL alignment, the analysis also incorporates the results of one (1) consolidation test carried out for the SBL alignment and three (3) consolidation test carried out in Swamp 204 (between STA 11+700 and 11+825) located immediately to the south of the south abutment (refer to 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 Transporattion, Ontario, GWP 5404-05-00; WP 5404-05-01, Geocres No. 41H-115 dated July 2012 by Golder Associates).

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:

$\sigma'_p$ = preconsolidation stress (kPa); and,

$s_{u(mob)}$ = $\mu s_{u(FV)}$ (after Bjerrum, 1973) where $s_{u(mob)}$ = average mobilized undrained shear strength (kPa)

$\mu$ = Bjerrum's correction factor based on Plasticity Index

$s_{u(FV)}$ = undrained shear strength from field vane test (kPa)

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.

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

where:

$S_c$ = secondary consolidation (creep) settlement (mm)

$C_{\alpha\epsilon}$ = modified secondary compression index as estimated from laboratory consolidation tests

$H$ = initial thickness of compressible clay deposit (mm)

$t$ = post-construction period of interest (20 years)

$t_{EoP}$ = time to reach end of primary consolidation (years)

### 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 7.5 m high rock fill embankment is estimated to be about 1,300 mm. The estimated total settlement is comprised of about 235 mm of immediate settlement due to compression of the non-cohesive deposits and about 1,065 mm of primary consolidation of the cohesive deposits.

Based on an average coefficient of consolidation ($c_v$) of about 1.8 x $10^{-3}$ $cm^2/s$ and 3.2 x $10^{-3}$ $cm^2/s$ estimated for the various cohesive deposits (ranging in thickness from about 1.7 m to 16.9 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 4,395 days (or about 12 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 365 mm per log-cycle of time for this area corresponding to about 180 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 7.5 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 795 mm. The estimated total settlement is comprised of about 355 mm of immediate settlement due to compression of the non-cohesive deposits and about 440 mm of primary consolidation of the cohesive deposits.

Based on an average coefficient of consolidation ($c_v$) of about $2 \times 10^{-3}$ $cm^2/s$ estimated for the up to about 10.1 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 390 days (or about 1 year). 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 75 mm per log-cycle of time for this area corresponding to about 95 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 0.2 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 thick cohesive deposits (the bottom of which is up to about 42.5 m below ground surface) and the associated magnitude of primary and secondary consolidation settlement (about 1,095 mm) of the foundation soils under a 7.5 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 (7.5 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 117 years) 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 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 7.5 m high approach embankment constructed to 2H:1V side slopes, consisting of a 1 m thick granular base/levelling pad, an up to about 5.5 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 285 mm. The estimated total settlement is comprised of about 70 mm of immediate settlement due to compression of the non-cohesive deposits, and about 165 mm and 50 mm of primary and secondary consolidation, respectively, 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 Swamp 204.

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 levelling pad beneath the EPS and 1 m thick 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 11.5 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. For this scenario, the depth of sub-excavation along the west slope NBL approach embankment would be up to about 13.5 m, and up to about 15.5 m along the east slope of the NBL approach embankment, as shown on Figure 17. However, given the subsurface conditions/steeply sloping bedrock in this area, the sub-excavation may need extend beyond a depth of 15.5 m (i.e. beyond a corresponding Elevation 164.1 m) in order to fully sub-excavate the cohesive deposit. 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 350 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-contruction settlement (i.e. about 37 years) 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, a 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-piles at the abutment. Given the risks, uncertainties and complexity associated with the staged construction and wick drain design 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, however, given the presence of the thick cohesive deposit below the proposed north approach embankment (i.e. up to about 11.5 m thick), RAPs would not be able to fully penetrate to the bottom of the cohesive deposit and as such are not considered a practical alternative at this 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 thick granular base, an up to about 7.5 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 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 125 mm. The estimated total settlement is comprised of about 85 mm of immediate settlement due to compression of the non-cohesive deposits and about 40 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 embankment constructed (including a 300 mm thick Special Provision 110S13 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 and a two-span bridge configuration was considered for the Still River NBL bridge during the initial design stages.

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 relatively 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 criterion 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 0.6 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 sand and 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 15.5 m below the existing ground surface and would generally be made through very loose to loose non-cohesive deposits (sand to silt) and firm to stiff cohesive deposits (clayey silt to clay), which are considered Type 4 and Type 3 soils, respectively, according to OSHA. Therefore, the excavation through the non-cohesive deposit would have to be carried with side slopes no steeper than 3 horizontal to 1 vertical (3H:1V) while the excavation through the cohesive deposit could be carried out with side slopes no steeper than 1 horizontal to 1 vertical (1H:1V).

At the location of the proposed two-span north abutment, the excavation will extend to a depth of up to about 6.5 m below the existing ground surface and will generally be made through zones of cobbles/boulders and/or through gneiss to granite 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

The use of controlled blasting techniques is recommended for bedrock excavation at the two-span north abutment for the temporary faces, and the use of explosives should be in accordance with OPSS 120 *Use of Explosives*. It is recommended that a separate Special Provision for the control of all blasting operations be prepared (refer to Special Provision 299F06 *Rock Excavation for Controlled Blasting*). The Special Provision should include, but not limited to, the following:

- An outline of the requirements, procedure and extent of a pre-blast survey, including all structures within a radius of about 100 m of the blasting operations, as well as notification to all individuals working or living within 500 m of the blasting area.
- A blast proposal by the blasting contractor or their blast consultant detailing the blast methodology, including drill hole patterns, hole size and depths, size of blasts, explosive and initiation product details, as well as all blast control procedures. Blast control procedures would include details on controlling flyrock, temporary road closures, blast signalling and site clearing procedures, as well as procedures to deal with debris clean-up. This submission would be required prior to the commencement of any blasting operations.
- The requirement for trial blasts for all proposed production and wall control blast procedures.
- The requirements for ground and air vibration monitoring during the blasting operations. This would include details on instrumentation, number and location of monitoring sites, blast recording and reporting procedures, and procedures to be followed in the event of excessive vibration readings.

It is recommended that ground vibration levels be limited to 50 mm/s for any adjacent services and structures. Continuous monitoring of all blasting operations would dictate when changes to the blast procedures become necessary to meet this limit and how close the blasting can be carried out adjacent to any existing services and structures.

It is recommended that the specification for the blasting require a minimum of 80 per cent half barrels (drill hole traces) visible on the cut face after scaling.

### 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 1 m above ground surface (artesian condition) was also recorded during drilling in Borehole B202-02 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. 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 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. by approximately 4.5 m above ground surface corresponding to Elevation 184 m), and as such, dewatering would not be required.

At the north abutment (two-span option), the proposed founding level (about Elevation 183 m) for the spread footing is approximately 0.4 m below the elevation of the groundwater table as measured during the foundation investigation in February 2011. However, it should be noted that an approximately 2.3 m deep excavation below the measured groundwater level will be required at the center of the footing to remove the cobbles and boulders. As such, dewatering will be required to properly prepare the bedrock, place mass concrete to reach the proposed founding elevation (particularly at the centre of the footing) 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 operations.

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 majority of 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 NBL Bridge/09-1111-6014-2522 RPT 14Sep11 Highway 69 Still River NBL Bridge.docx

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ASTM International:

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

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| 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. |
| Special Provision 206S03 | Amendment to OPSS 206 – Rock Excavation, Grading; Rock Embankment. |
| Special Provision 299F06 | Rock Excavation for Controlled Blasting. |

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

Ontario Occupational Health and Safety Act:

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| Ontario Regulation 213 | Construction Projects |

Ontario Provincial Standard Drawings:

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|---|---|
| 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. |
| OPSD 3102.100 | Walls, Abutment, Backfill Drain. |
| OPSD 3121.150 | Walls, Retaining, Backfill, Minimum Granular Requirement. |
| OPSD 3190.100 | Walls, Retaining and Abutment, Wall Drain. |

Ontario Provincial Standard Specifications:

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|---|---|
| OPSS 120 | General Specification for Use of Explosives. |
| OPSS 405 | Construction Specifcation for Pipe Subdrains. |
| OPSS 903 | Construction Specification for Deep Foundations. |
| OPSS 1002 | Material Specification for Aggregaes – Concrete. |
| OPSS 1860 | Material Specification for Geotextiles. |

Ontario Water Resources Act:

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| Ontario Regulation 903 | Wells |

# TABLES

## Table 1A: Evaluation of Foundation Alternatives – Still River NBL 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] | ■ Relative ease of construction. | ■ Allows only for semi-integral abutment design.<br>■ Geotechnical capacity on the thick weak native soils for spread footing design is very low; post-construction settlement likely to occur. | ■ Lower relative cost than piled foundation and caisson options. | ■ Not recommended due to very weak and compressible foundation soils. |
| Steel H-piles driven to the top of dense to very desnse silt and sand | 1 | ■ Allows for integral abutment design. | ■ Long piles will be required to provide adequate geotechnical capacity for pile design.<br>■ Large dragloads will have to be considered in the pile design.<br>■ Partial preloading in the area of south abutment and approach embankment will be required to reduce the dragloads on piles.<br>■ Pile load testing will be required to confirm the geotechnical capacities of the pile. | ■ Lower relative cost than caisson option.<br>■ Higher relative cost than spread footing option.<br>■ Higher cost associated with long piles.<br>■ Additional cost associated with pile load testing. | ■ Difficulties maintaining alignment of very long H-piles while pile driving.<br>■ 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 NBL Bridge Structure – South Abutment
## Highway 69 Four-Laning

| Foundation Option | Rank | Advantages | Disadvantages | Relative Costs | Risks / Consequences |
|---|---|---|---|---|---|
| Caissons augered into compact to very dense sand to silt deposit | NR[1] | ■ Reduced number of deep foundation elements compared to steel H-piles. | ■ Allows only for semi-integral abutment design.<br>■ Temporary or permanent steel liners would be required to control groundwater and provide support through overburden.<br>■ Special measures, such as drilling mud / slurry required to balance groundwater pressures/basal heave and minimize loss of ground during construction.<br>■ Concrete caissons would need to be placed using tremie methods below the groundwater table. | ■ Higher relative cost than spread footing and piled foundation options.<br>■ Additional cost associated with the need for temporary or permanent steel liners and drilling slurry. | ■ Potential for unbalanced head in liners during installation resulting in base heave and possible loss of ground.<br>■ 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 NBL 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 | | ■ Relative ease of construction. | ■ Allows only for semi-integral abutment design.<br>■ Geotechnical capacity on the thick weak native soils for spread footing design is low; post-construction settlement likely to occur.<br>■ Considerable excavation (23.1 m to 28.1 m deep) required if footing is founded on bedrock to achieve greater resistance. | ■ Lower relative cost than piled foundation and caisson options. | ■ Not recommended due to weak and compressible foundation soils, or greater depth of excavation to bedrock. |
| Steel H-piles driven to bedrock | 1 | ■ Negligible post-construction settlement. | ■ Dewatering and concrete placement for the pile cap required within a dry excavation.<br>■ Pile points will be required to facilitate proper seating of the piles on the steeply sloping bedrock. | ■ Lower relative cost than caisson option.<br>■ Higher relative cost than spread footing option.<br>■ Additional cost associated with the need for pile points.<br>■ Additional cost required for dewatering for pile cap construction. | ■ Potential difficulties in seating the steel H-piles into the strong to very strong and sloping bedrock even when pile points are used. |
| Caissons socketted into bedrock | 2 | ■ Reduced number of deep foundation elements compared to steel H-piles.<br>■ Negligible post-construction settlement. | ■ Temporary steel liners would be required to control groundwater and provide support through overburden.<br>■ Concrete inside caissons would need to be placed using tremie methods below the groundwater | ■ Higher relative cost than spread footing piled foundation options.<br>■ Additional cost associated with the need for temporary steel liners. | ■ Increased potential difficulties in achieving adequate seal and drilling a large diameter socket into strong to very strong and sloping bedrock. |

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

| Foundation Option | Rank | Advantages | Disadvantages | Relative Costs | Risks / Consequences |
|---|---|---|---|---|---|
| | | | table.<br>■ Potential for difficulties in socketting caissons into strong to very 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 NBL 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] | ■ Relative ease of construction. | ■ Allows only for semi-integral abutment design.<br>■ Geotechnical capacity on the weak native soils for spread footing design is low; post-construction settlement likely to occur.<br>■ Differential settlement will need to be considered given the variability in the thickness of the cohesive deposit.<br>■ Variable excavation depth (5.9 m to 15.3 m) to be able to found footings on bedrock – requires mass concrete or rock removal to achieve a somewhat level grade. | ■ Lower relative cost than piled foundation and caisson options. | ■ Not recommended due to weak foundation soils, or great/variable depth of excavation to bedrock. |
| Steel H-piles driven to bedrock | 1 | ■ Negligible post-construction settlement. | ■ Pile points will be required to facilitate proper seating of the piles on the steeply sloping bedrock.<br>■ Integral abutment design may not be possible. | ■ Lower relative cost than caisson option.<br>■ Higher relative cost than spread footing option.<br>■ Additional cost associated with the need for pile points. | ■ Integral abutment design may not be possible due to the steeply sloping bedrock.<br>■ Moderate to high risk in seating the steel H-piles into the predominantly strong to very strong and steeply sloping bedrock.<br>■ 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 |

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

| Foundation Option | Rank | Advantages | Disadvantages | Relative Costs | Risks / Consequences |
|---|---|---|---|---|---|
| | | | | | affecting the performance of the north abutment piles. |
| Caissons socketted into bedrock | NR[1] | ■ Reduced number of deep foundation elements compared to steel H-piles.<br>■ Negligible post-construction settlement. | ■ Allows only for semi-integral abutment design.<br>■ Potential for difficulties in socketting caissons into strong to very strong and sloping bedrock.<br>■ Temporary steel liners would be required to control groundwater and support through overburden.<br>■ Concrete for caissons would need to be placed using tremie methods below the groundwater table. | ■ Higher relative cost than spread footing piled foundation options.<br>■ Additional cost associated with the need for temporary steel liners. | ■ Increased potential difficulties in achieving adequate seal and drilling a large diameter socket into strong to very strong sloping bedrock. |

Note: 1. NR – Not Recommended

**Prepared By:** TZ/CN

**Reviewed By:** JPD/JMAC

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

| Foundation Option | Rank | Advantages | Disadvantages | Relative Costs | Risks / Consequences |
|---|---|---|---|---|---|
| Spread footings on properly prepared bedrock and/or mass concrete | 1 | ■ Relative ease of construction.<br>■ Reduced bedrock excavation compared to steel H-pile and caisson options.<br>■ Frost susceptibility is not an issue for footings on bedrock and/or mass concrete. | ■ Allows only for semi-integral abutment design.<br>■ Variable bedrock surface and removal of zones of cobbles and boulders followed by mass concrete placement required to achieve level footing.<br>■ Bedrock will have to be removed using controlled blasting techniques to minimize shattering and over-break. | ■ Lower relative cost than piled foundation and caisson options.<br>■ Additional cost associated with excavation of bedrock and mass concrete placement.<br>■ Additional cost if dowelling into bedrock is required to increase sliding resistance. | ■ Variability in bedrock will impact mass concrete quantities and excavation depth. |
| Steel H-piles in bedrock trenches | NR[1] | ■ Allows for integral abutment design. | ■ Excavation/trenching through strong bedrock will be required for pile installation. | ■ Lower relative cost than caisson option.<br>■ Higher relative cost than spread footing option.<br>■ Additional cost required for bedrock excavation/trenching. | ■ Not recommended due presence of shallow bedrock. |
| Caissons socketted into bedrock | NR[1] | ■ Reduced number of deep foundation elements compared to steel H-piles. | ■ Allows only for semi-integral abutment design.<br>■ Difficulty in drilling through zones of cobbles and boulders in the overburden.<br>■ Drilling through strong bedrock will be required to achieve minimum caisson socket lengths. | ■ Higher relative cost than spread footing and H-piled foundation options.<br>■ Additional cost required for bedrock coring. | ■ Not recommended due presence of shallow bedrock. |

Note: 1. NR – Not Recommended

## Table 2A: Evaluation of Stability/Settlement Mitigation Options – Still River NBL 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 | ■ Reduced total settlement.<br>■ Toe berms are not required. | ■ Generation of very large volume of excess excavation spoil.<br>■ Very large quantity of rock fill required.<br>■ Long delay in construction associated with up to 42.5 m deep sub-excavation and replacement with rock fill operation.<br>■ Specialized equipment and additional effort required for deep sub-excavation and replacement.<br>■ Substantial post-construction settlement of rock fill itself.<br>■ Sub-excavation in close proximity to Still River would require protection measures.<br>■ Will require additional right-of-way to accommodate deep sub-excavation. | ■ Additional costs associated with sub-excavation (specialized drag-line equipment required), disposal and replacement of weak/soft, compressible deposits.<br>■ Additional cost for acquiring additional right-of-way for deep excavation. | ■ Deep sub-excavation in close proximity to Still River may not be permitted.<br>■ Potential difficulties maintaining stability of excavation slopes.<br>■ Preloading would be required to reduce large post-construction settlement of rock fill.<br>■ Would be able to achieve and maintain stability of proposed embankments. |
| Preloading or Surcharging | NR[1] | ■ Not applicable. | ■ Not applicable. | ■ Not applicable. | ■ 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 NBL Bridge Structure – South Approach Embankment Highway 69 Four-Laning

| Stability/Settlement Mitigation Option | Rank | Advantages | Disadvantages | Relative Costs | Risks/Consequences |
|---|---|---|---|---|---|
| Wick Drains | 2 | ■ Reduced time to complete primary consolidation. | ■ Detail wick drain investigation and design would be required.<br>■ Additional time required for installation of wick drains.<br>■ First stage of filling limited to 4.5 m to maintain FoS of 1.3.<br>■ Toe and front slope berms may be required to reduce number of subsequent fill stages.<br>■ Increased magnitude of secondary consolidation (creep) settlement as a result of the accelerated completion of primary consolidation settlement.<br>■ Delay in construction schedule to allow for sufficient settlement to occur to meet post-construction settlement criteria.<br>■ Instrumentation and monitoring program required to monitor staged construction (if possible) and to assess end of preload / surcharge period.<br>■ Increased handling of surcharge fill (Granular ‘B’ Type II) to remove surcharge.<br>■ Potential need for lightweight fill (i.e. EPS) as top-up. | ■ Schedule impacts may increase overall project costs.<br>■ Additional costs associated with detail wick drain investigation and design.<br>■ Additional cost for the installation of wick drains, instrumentation and associated monitoring program.<br>■ Additional costs if toe and front slope berms need to be constructed<br>■ Additional costs associated with construction and materials for surcharge and removal of excess surcharge embankment fill upon completion of surcharge period.<br>■ Additional costs associated with potential EPS top-up. | ■ Potential for instability of embankment on weak/soft foundation soils, even if staged construction is employed.<br>■ Complex wick drain design (potentially in combination with lightweight fill).<br>■ 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. |

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

| Stability/Settlement Mitigation Option | Rank | Advantages | Disadvantages | Relative Costs | Risks/Consequences |
|---|---|---|---|---|---|
| Partial Preloading (4.5 m high for 110 days) followed by Lightweight Fill Construction (5.5 m of EPS) | 1 | ■ Improved stability.<br>■ Reduced total settlement of foundation soils.<br>■ May shorten construction schedule. | ■ Very high cost of EPS construction materials.<br>■ Have to remove partial preload embankment in order to construct the EPS embankment.<br>■ Instrumentation and monitoring program required to assess end of preload period. | ■ Relative cost of EPS fill is about an order of magnitude higher than fill required for the other options.<br>■ Estimated cost for EPS is about $750,000 minus cost of rock fill to construct embankments in base case. | ■ Will achieve stability of partial preload embankments and final EPS embankments on weak/soft foundation soils.<br>■ 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 NBL 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<br>(up to about 15.5 m deep) | 1 | ■ Reduced total settlement.<br>■ Toe berms are not required. | ■ Generation of excess excavation spoil.<br>■ Relatively large quantity of replacement rock fill required.<br>■ Additional effort required for relatively deep sub-excavation and replacement.<br>■ Additional post-construction settlement of rock fill itself.<br>■ Sub-excavation in close proximity to Still River will require special protection measures.<br>■ 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. | ■ Additional costs associated with sub-excavation, disposal and replacement of weak/soft, compressible deposits.<br>■ Additional cost associated with special protection measures.<br>■ Additional cost associated with granular fill to allow pile driving at the north abutment. | ■ Relatively deep sub-excavation in close proximity to Still River may not be permitted.<br>■ Potential difficulties maintaining stability of excavation slopes.<br>■ Preloading would be required to reduce post-construction settlement of rock fill.<br>■ High potential of being able to achieve stability of proposed embankments.<br>■ Potential for lateral soil movement in area of abutment pile foundations. |
| Preloading or Surcharging | NR[1] | ■ Not applicable. | ■ Not applicable. | ■ Not applicable. | ■ 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 2B: Evaluation of Stability/Settlement Mitigation Options – Still River NBL Bridge Structure – North Approach Embankment (One-Span Option) Highway 69 Four-Laning

| Stability/Settlement Mitigation Option | Rank | Advantages | Disadvantages | Relative Costs | Risks/Consequences |
|---|---|---|---|---|---|
| Wick Drains and Staged Construction | NR[1] | ■ Somewhat reduced time to complete primary consolidation. | ■ 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 alternative to accelerate settlements.<br>■ Detail wick drain investigation and design will be required.<br>■ Additional time required for installation of wick drains.<br>■ Toe and front slope berms may be required.<br>■ Increased magnitude of secondary consolidation (creep) settlement as a result of the accelerated completion of primary consolidation settlement.<br>■ Delay in construction schedule to allow for sufficient settlement to occur to meet post-construction settlement criteria.<br>■ Instrumentation and monitoring program required to monitor staged construction (if possible) and to assess end of surcharge period.<br>■ Increased handling of surcharge fill (Granular ‘B’) | ■ Schedule impacts may increase overall project costs.<br>■ Additional costs associated with detail wick drain investigation and design.<br>■ Additional cost for the installation of wick drains, instrumentation and associated monitoring program.<br>■ Additional costs if toe berms need to be constructed<br>■ Additional costs associated with construction and materials for surcharge and removal of excess surcharge embankment fill upon completion of surcharge period.<br>■ Additional costs associated with potential EPS top-up. | ■ Potential for instability of embankment on weak/soft foundation soils, even if staged construction is possible and employed.<br>■ Complex wick drain design (potentially in combination with lightweight fill).<br>■ 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. |

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

| Stability/Settlement Mitigation Option | Rank | Advantages | Disadvantages | Relative Costs | Risks/Consequences |
|---|---|---|---|---|---|
| | | | to remove surcharge.<br>■ Potential need for lightweight fill (i.e. EPS) as top-up. | | |
| Rammed Aggregate Piers (RAPs) | NF | ■ Given the presence of the thick cohesive deposit below the proposed north approach embankment (i.e. up to about 11.5 m thick), RAPs would not be able to fully penetrate to the bottom of the cohesive deposit and as such are not considered a practical alternative at this site. | | | |
| Partial Preloading (2 m high for 45 days) followed by Lightweight Fill Construction (7.5 m of EPS) | 2 | ■ Improved stability.<br>■ Reduced total settlement of foundation soils.<br>■ May shorten construction schedule. | ■ Very high cost of EPS construction materials.<br>■ Some additional effort required to remove the partial preload embankments in order to construct the EPS embankment.<br>■ Instrumentation and monitoring program required to assess end of preload period. | ■ Relative cost of EPS fill is about an order of magnitude higher than fill required for the other options.<br>■ Estimated cost for EPS is about $1,000,000 minus cost of rock fill to construct embankments in base case. | ■ Readily able to achieve stability of partial preload embankments and final EPS embankments on the weak/soft foundation soils.<br>■ No unexpected post-construction settlements. |

Note: 1. NR – Not Recommended
2. NF – Not Feasible

**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) | $\gamma'$ (kN/m$^3$) | $\phi'$ ( $^\circ$ ) | c' (kPa) | $S_u$ (kPa) | $\sigma_p'$ (kPa) | $e_o$ | $C_c$ | $C_r$ | E' (MPa) | $c_v$ (cm$^2$/s) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| South Approach/Abutment (One-Span or Two-Span Bridge Option) | Topsoil | 181.1 – 181.0 | 0.2 – 0.3 | 15.0 | 28 | 1 | - | - | - | - | - | - | - |
| | Sandy Silt (Near Surface) | ~ 180.9 | ~ 0.5 | 18.5 | - | - | - | - | - | - | - | 5 | - |
| | Clayey Silt to Silt Clay (Near Surface) | 180.8 – 180.3 | 1.7 – 1.8 | 17.5 | - | - | 25 | 115 | 1.4 | 0.80 | 0.08 | - | $3.19 \times 10^{-3}$ |
| | Sand to Silt (Upper) | 179.0 – 178.6 | 5.8 – 7.6 | 18.5 | 28 | - | - | - | - | - | - | 10 | - |
| | Silty Clay (Upper) | 172.8 – 171.4 | 7.9 – 11.6 | 17.0 | - | - | 40 – 60 | 180 – 275 | 1.4 | 0.80 – 0.90 | 0.08 – 0.09 | - | $1.81 \times 10^{-3}$ |
| | Silt Interlayer | ~ 170.9 | ~ 2.7 | 18.0 | 29 | - | - | - | - | - | - | 5 | - |
| | Silt to Silty Sand Interlayer | 161.3 – 161.2 | 3.1 – 5.8 | 18.0 | 29 | - | - | - | - | - | - | 5 | - |
| | Silty Clay to Clay (Lower) | 158.2 – 155.4 | 15.0 – 15.0 | 17.0 | - | - | 60 – 100 | 275 – 460 | 1.4 | 0.56 – 0.84 | 0.056 – 0.084 | - | $1.81 \times 10^{-3}$ |
| | Silt to Sand (Lower) | 143.2 – 138.5 | 4.9 – 10.6 | 18.5 | 29 | - | - | - | - | - | - | 25 | - |
| Centre Pier (Two-Span Brideg Option) | Topsoil | 179.0 – 178.7 | ~ 0.2 | 15.0 | 27 | 1 | - | - | - | - | - | - | - |
| | Sand to Silt | 178.8 – 178.5 | 5.0 – 7.9 | 18.5 | 28 | - | - | - | - | - | - | 1 – 5 | - |
| | Clayey Silt to Clay | 173.5 – 170.4 | 17.2 – 20.0 | 17.0 | - | - | 31 – 50 | 140 – 225 | 1.4 | 0.80 | 0.08 | - | $1.97 \times 10^{-3}$ |
| North Approach/Abutment (One-Span Bridge Option) | Topsoil | 179.5 – 179.4 | 0.2 – 0.3 | 15.0 | 27 | 1 | - | - | - | - | - | - | - |
| | Sand to Silt | ~ 179.2 | 2.3 – 5.0 | 18.5 | 28 | - | - | - | - | - | - | 1 – 5 | - |
| | Sandy Silt Interlayer | ~ 179.9 | ~ 0.7 | 18.0 | 29 | - | - | - | - | - | - | - | - |
| | Clayey Silt to Clay | 176.9 – 175.2 | 3.3 – 10.1 | 17.0 | - | - | 20 – 50 | 106 – 225 | 1.4 | 0.80 | 0.08 | - | $1.97 \times 10^{-3}$ |

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 NBL Bridge Structure Highway 69 Four-Laning

| Bridge Alternative | Rank | Advantages | Disadvantages | Relative Costs | Risks / Consequences |
|---|---|---|---|---|---|
| One-Span Option | 2 | ■ Shorter total length of bridge structure.<br>■ 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.<br>■ Readily available rock fill for construction of longer north approach embankments | ■ Deep foundations are required to support all foundation elements compared to the two-span option where shallow foundations are recommended at the north abutment.<br>■ 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.<br>■ Granular fill (i.e. not rock fill) will be required as replacement fill in the north abutment sub-excavation area to allow for the driving of piles.<br>■ To minimize downdrag load on the north abutment piles, a retaining wall would be required in place of a front slope if full sub-excavation option is not adopted. | ■ Lower relative cost for foundation elements given that there is no centre pier for this option.<br>■ 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).<br>■ 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. | ■ 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).<br>■ 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 NBL Bridge Structure
### Highway 69 Four-Laning

| Bridge Alternative | Rank | Advantages | Disadvantages | Relative Costs | Risks / Consequences |
|---|---|---|---|---|---|
| Two-Span Option | 1 | ■ Relative ease of construction at the north abutment and north approach embankment. | ■ Longer total length of bridge structure.<br>■ Footing construction for additional foundation element (i.e. at the north abutment).<br>■ Excavation and dewatering required to construct the pile cap at the centre pier and potentially at the spread footing at the north abutment. | ■ Additional cost associated with construction of an additional foundation element compared to the one-span option.<br>■ Significantly lower relative cost of construction of north approach embankment compared to north approach embankment associated with one-span option.<br>■ Additional cost associated with excavation and dewatering required to construct pile cap for the centre pier.<br>■ The overall cost for two-span option is anticipated to be lower than the one-span option. | ■ No expected instability and long-term settlement of north approach embankment.<br>■ Variability in depth to bedrock at the north abutment will impact excavation depth and mass concrete quantities. |

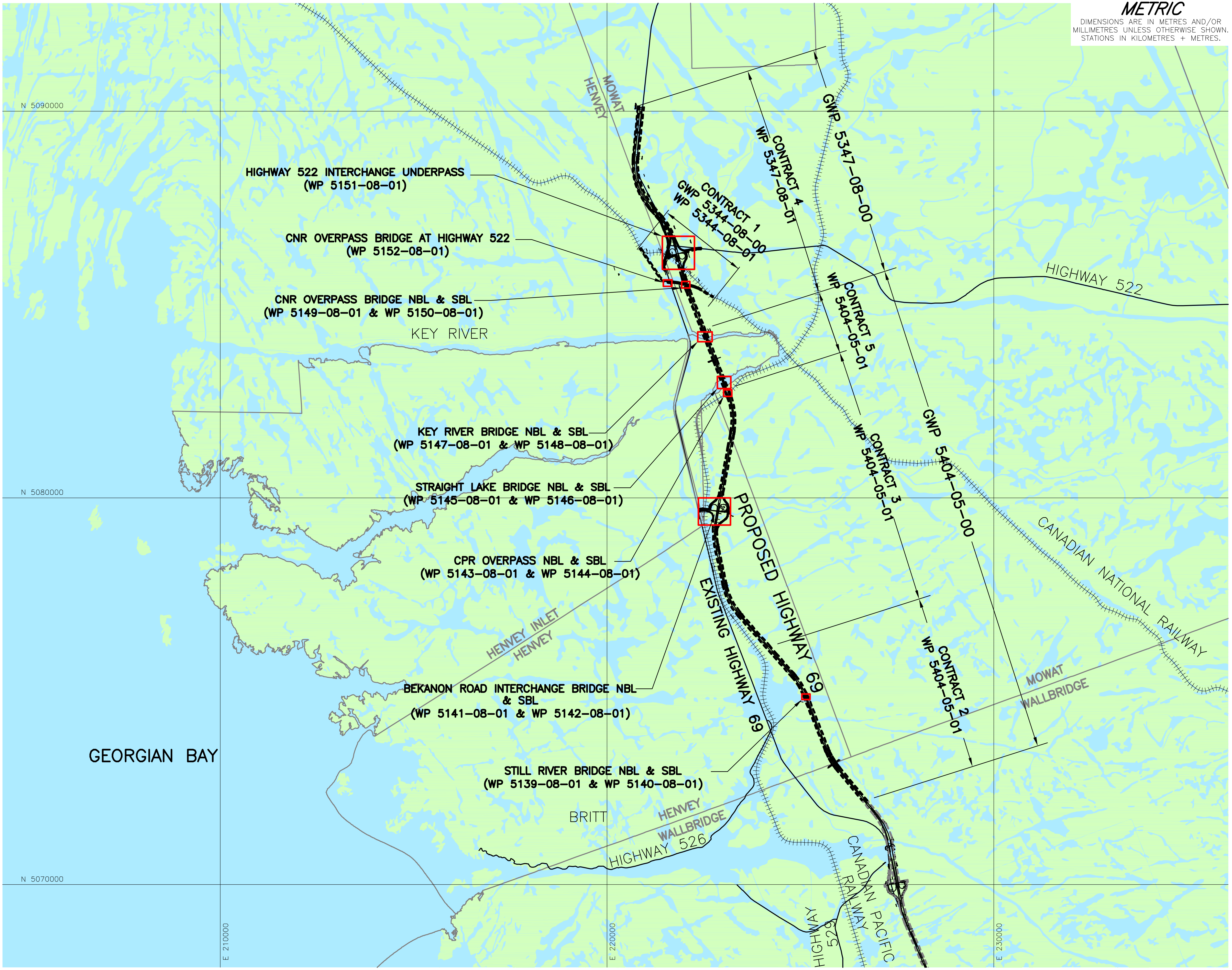
**Prepared By:** TZ/CN

**Reviewed By:** JPD/JMAC

# DRAWINGS

MINISTRY OF TRANSPORTATION, ONTARIO

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



## PLAN

SCALE
1 0 1 2 km

CONT No.
GWP No. 5404-05-00

SHEET

HIGHWAY 69
SITE LOCATION PLAN

Golder Associates

**Golder Associates Ltd.**
MISSISSAUGA, ONTARIO, CANADA

KEY PLAN
NOT TO SCALE

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-121

| | | | |
|---|---|---|---|
| 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 |

PLOT DATE: September 8, 2014
FILENAME: T:\Projects\2009\09-1111-6014 (URS, Hwy 69, Henvey)\-HC- CONTRACT 2 (Still River Bridge (NBL)-FINAL-STAMPED\0911116014HC001.dwg

MINISTRY OF TRANSPORTATION, ONTARIO

| | |
|---|---|
| CONT No. | |
| WP No. | 5139–08–01 |

SHEET

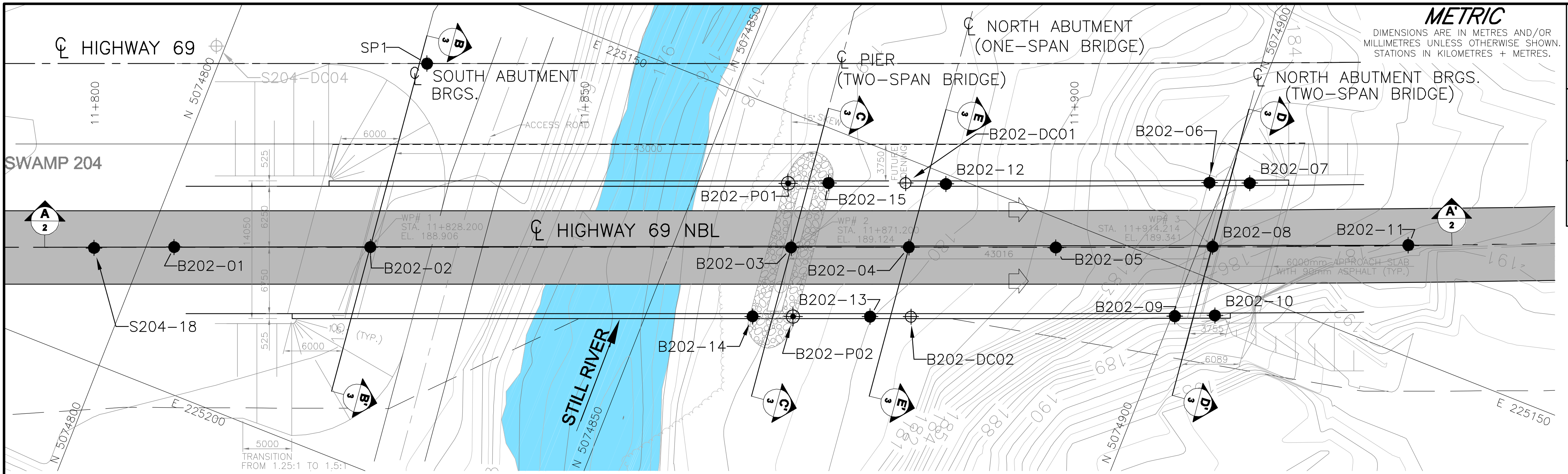
HIGHWAY 69

STILL RIVER NBL BRIDGE STRUCTURE

BOREHOLE LOCATIONS AND SOIL STRATA

**Golder Associates Ltd.**
MISSISSAUGA, ONTARIO, CANADA

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



## PLAN

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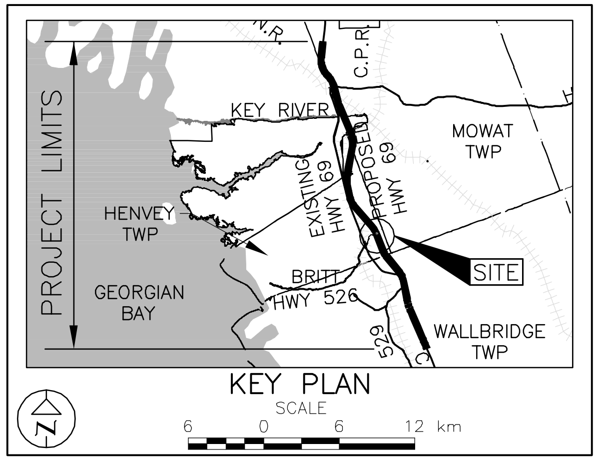


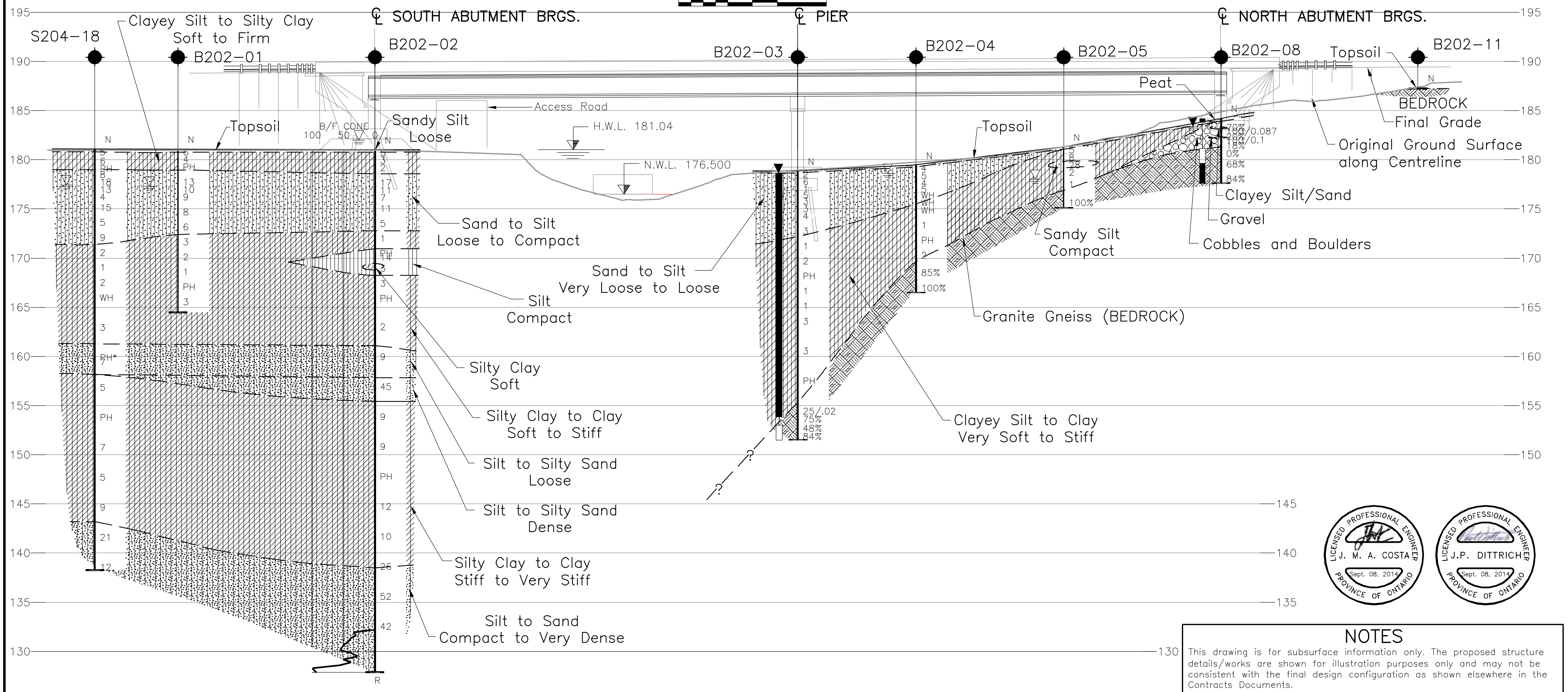
## KEY PLAN

SCALE 6 0 6 12 km

## LEGEND

| | |
|---|---|
| ● | Borehole |
| ⊕ | Dynamic Cone Penetration Test |
| ⊕ | Probehole |
| ▮ | Seal |
| ▯ | Piezometer |
| N<br>16 | Standard Penetration Test Value<br>Blows/0.3m unless otherwise stated<br>(Std. Pen. Test, 475 j/blow) |
| 100% | Rock Quality Designation (RQD) |
| ▼ | WL in piezometer, measured on February 28, 2011 |
| ▽ | WL upon completion of drilling |
| R | Refusal |

| BOREHOLE CO–ORDINATES | | | |
|---|---|---|---|
| No. | ELEVATION | NORTHING | EASTING |
| B202–01 | 181.1 | 5074802.7 | 225185.9 |
| B202–02 | 181.0 | 5074821.3 | 225178.6 |
| B202–03 | 178.8 | 5074861.3 | 225162.8 |
| B202–04 | 179.4 | 5074872.5 | 225158.4 |
| B202–05 | 181.3 | 5074886.4 | 225152.9 |
| B202–06 | 183.9 | 5074898.6 | 225140.9 |
| B202–07 | 184.3 | 5074902.4 | 225139.5 |
| B202–08 | 184.2 | 5074901.3 | 225146.9 |
| B202–09 | 186.4 | 5074900.3 | 225154.9 |
| B202–10 | 187.1 | 5074904.1 | 225153.3 |
| B202–11 | 187.3 | 5074919.8 | 225139.4 |
| B202–12 | 179.5 | 5074873.6 | 225151.0 |
| B202–13 | 179.4 | 5074871.4 | 225166.4 |
| B202–14 | 179.0 | 5074860.2 | 225170.8 |
| B202–15 | 178.7 | 5074862.4 | 225155.3 |
| B202–DC01 | 179.0 | 5074869.7 | 225152.4 |
| B202–DC02 | 180.2 | 5074875.3 | 225164.9 |
| B202–P01 | 178.9 | 5074858.6 | 225156.8 |
| B202–P02 | 179.1 | 5074864.1 | 225169.3 |
| S204–18 | 181.1 | 5074795.1 | 225189.0 |
| SP1 | 181.1 | 5074819.8 | 225159.0 |



J. M. A. COSTA
Sept. 08, 2014

J.P. DITTRICH
Sept. 08, 2014

## CENTRELINE PROFILE HIGHWAY 69 (NBL)

A–A' 2

SCALE 5 0 5 10 m

## REFERENCE

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

## 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 and beyond boreholes the boundaries are assumed from geological evidence.

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

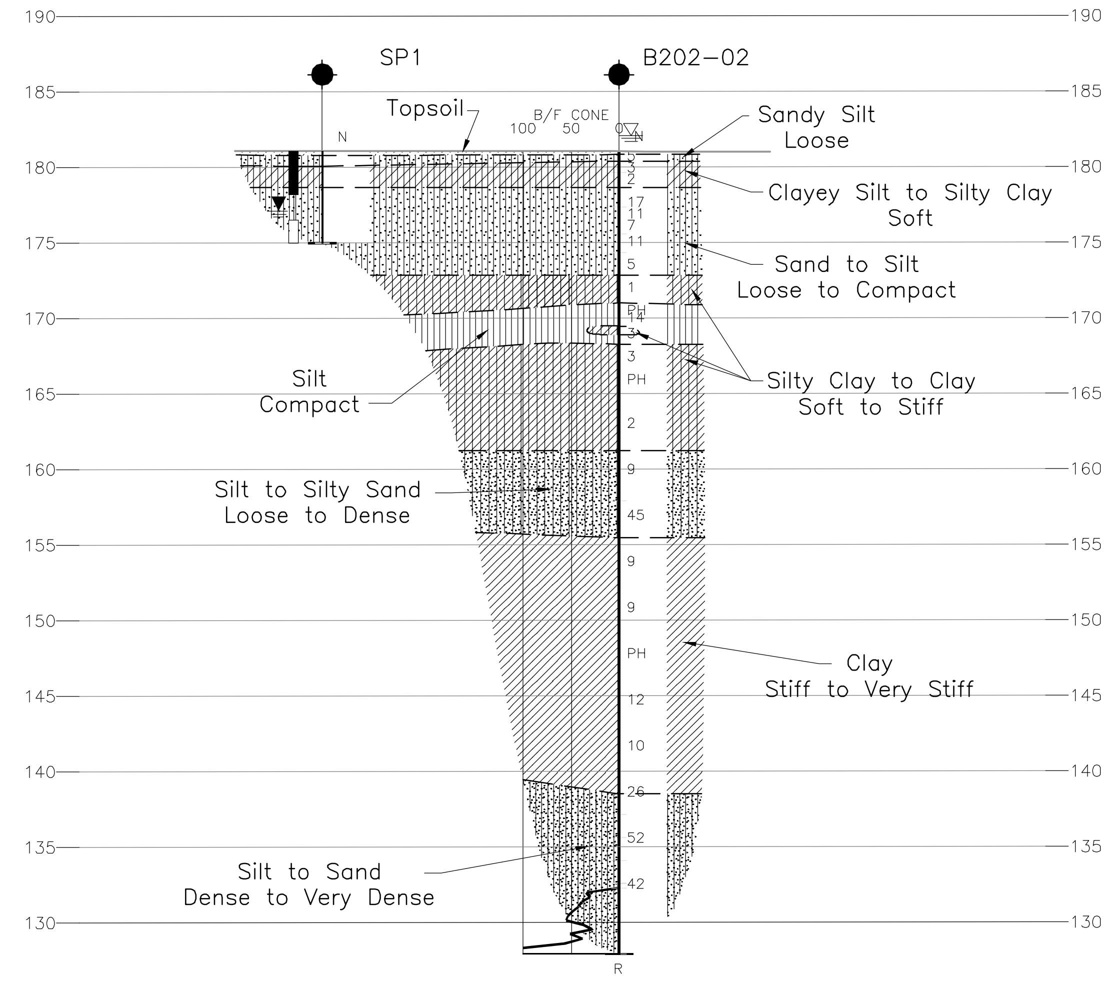
| NO. | DATE | BY | REVISION |
|---|---|---|---|
| | | | |
| | | | |

Geocres No. 41H–121

| | | | |
|---|---|---|---|
| HWY. 69 | | PROJECT NO. 09–1111–6014 | DIST. |
| SUBM'D. TVA | CHKD. TVA/TZ | DATE: Oct. 2012 | SITE: 44–458/1 |
| DRAWN: JFC | CHKD. CN | APPD. JPD/JMAC | DWG. 2 |

PLOT DATE: September 11, 2014

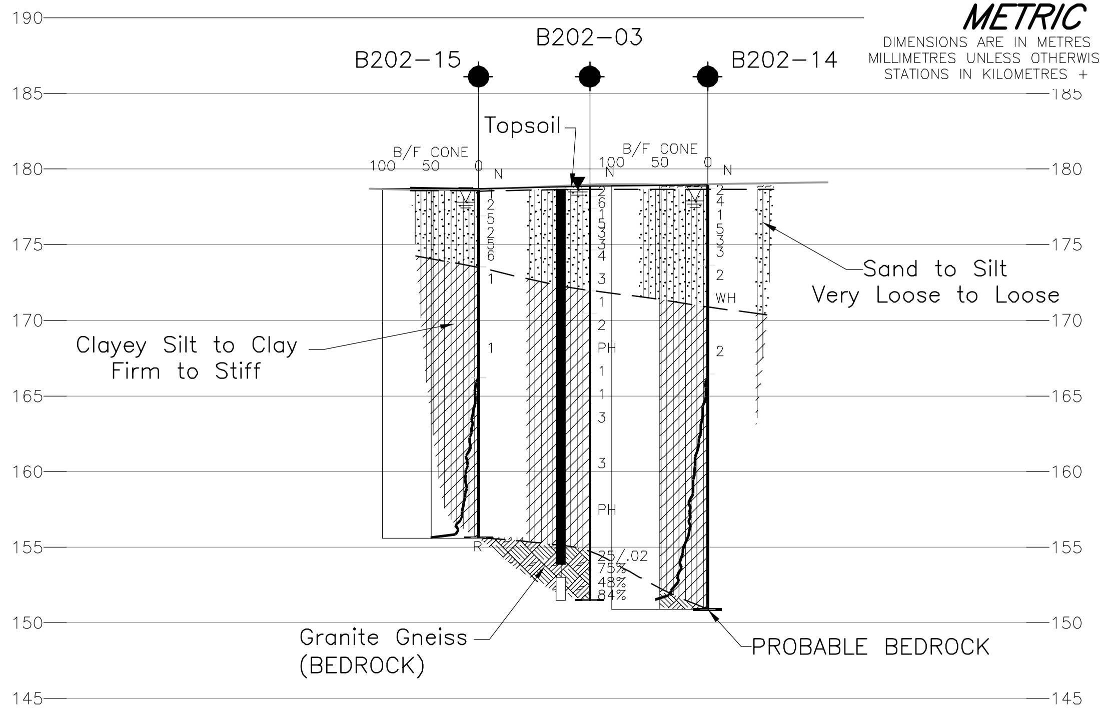
MINISTRY OF TRANSPORTATION, ONTARIO



B-B'
2

## SOUTH ABUTMENT
## HIGHWAY 69 (NBL)

SCALE
5 0 5 10 m

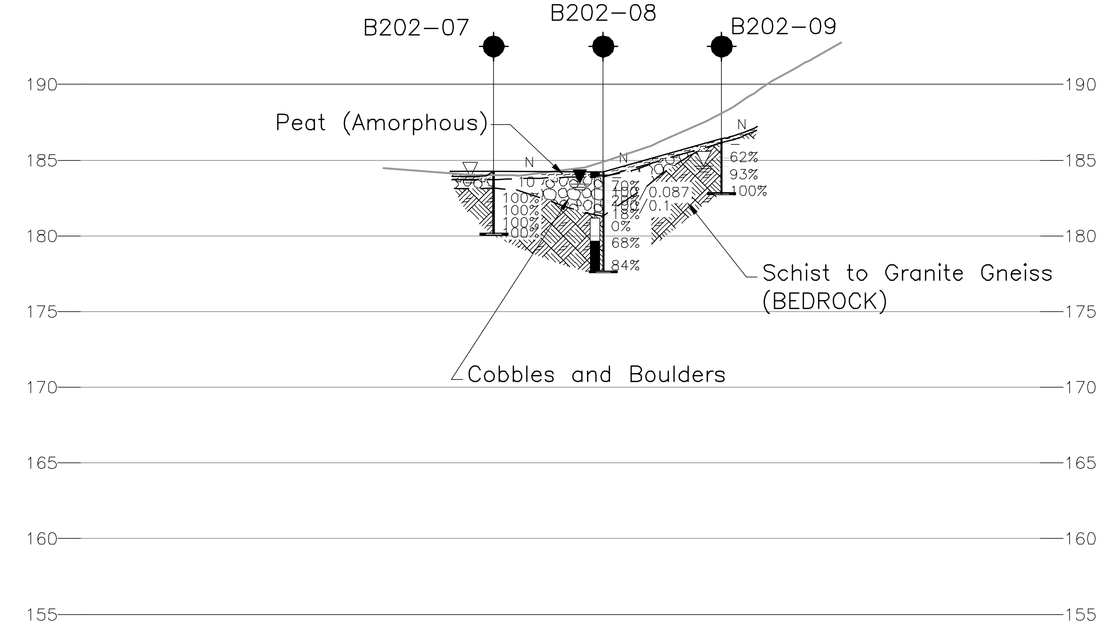


METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

C-C'
2

## CENTRE PIER (TWO-SPAN BRIDGE)
## HIGHWAY 69 (NBL)

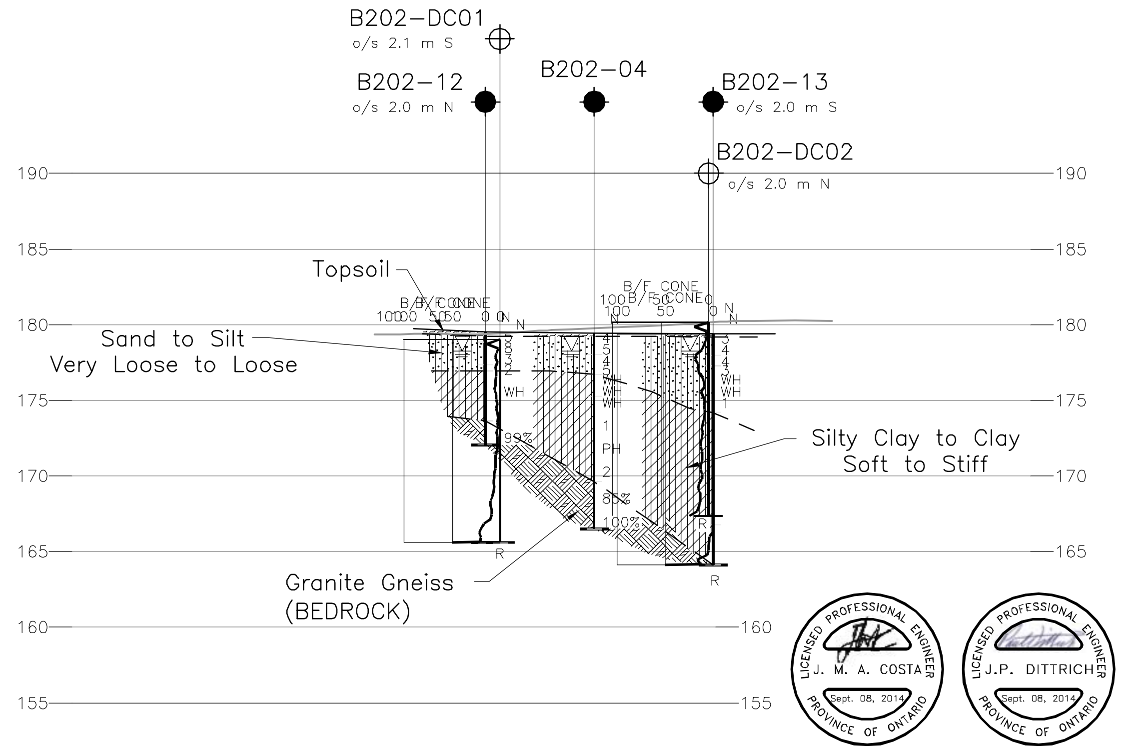
SCALE
5 0 5 10 m



D-D'
2

## NORTH ABUTMENT (TWO-SPAN BRIDGE)
## HIGHWAY 69 (NBL)

SCALE
5 0 5 10 m



E-E'
2

## NORTH ABUTMENT (ONE-SPAN BRIDGE)
## HIGHWAY 69 (NBL)

SCALE
5 0 5 10 m

### REFERENCE

Base plans provided in digital format by URS, drawing files Hwy69_base.dwg, Hwy69_plan.dwg, received December 16, 2009, StillRiver_NBL_ga.dwg, received April 5, 2011 and OG-Survey-Contours C2-3.dwg, received May 30, 2011.

J. M. A. COSTA
Sept. 08, 2014

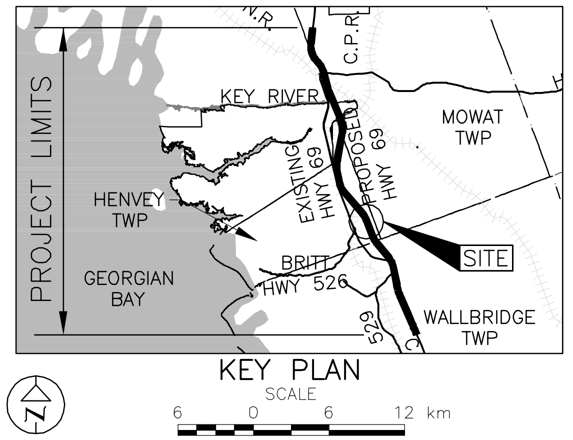
J.P. DITTRICH
Sept. 08, 2014

CONT No.
WP No. 5139-08-01

HIGHWAY 69
STILL RIVER NBL BRIDGE STRUCTURE
SOIL STRATA

SHEET

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



KEY PLAN
SCALE
6 0 6 12 km

### LEGEND

| | |
|---|---|
| ● | Borehole |
| ⊕ | Dynamic Cone Penetration Test |
| ▮ | Seal |
| ▯ | Piezometer |
| N<br>16 | Standard Penetration Test Value<br>Blows/0.3m unless otherwise stated<br>(Std. Pen. Test, 475 j/blow) |
| 100% | Rock Quality Designation (RQD) |
| ▼ | WL in piezometer, measured on February 28, 2011 |
| ▽ | WL upon completion of drilling |
| R | Refusal |

BOREHOLE CO-ORDINATES

| No. | ELEVATION | NORTHING | EASTING |
|---|---|---|---|
| B202-02 | 181.0 | 5074821.3 | 225178.6 |
| B202-03 | 178.8 | 5074861.3 | 225162.8 |
| B202-04 | 179.4 | 5074872.5 | 225158.4 |
| B202-07 | 184.3 | 5074902.4 | 225139.5 |
| B202-08 | 184.2 | 5074901.3 | 225146.9 |
| B202-09 | 186.4 | 5074900.3 | 225154.9 |
| B202-12 | 179.5 | 5074873.6 | 225151.0 |
| B202-13 | 179.4 | 5074871.4 | 225166.4 |
| B202-14 | 179.0 | 5074860.2 | 225170.8 |
| B202-15 | 178.7 | 5074862.4 | 225155.3 |
| B202-DC01 | 179.0 | 5074869.7 | 225152.4 |
| B202-DC02 | 180.2 | 5074875.3 | 225164.9 |
| SP1 | 181.1 | 5074819.8 | 225159.0 |

### NOTES

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

| NO. | DATE | BY | REVISION |
|---|---|---|---|
| | | | |

Geocres No. 41H-121

| | | | |
|---|---|---|---|
| HWY. 69 | | PROJECT NO. 09-1111-6014 | DIST. |
| SUBM'D. TVA | CHKD. TVA | DATE: Oct. 2012 | SITE: 44-458/1 |
| DRAWN: JFC | CHKD. CN | APPD. JPD/JMAC | DWG. 3 |

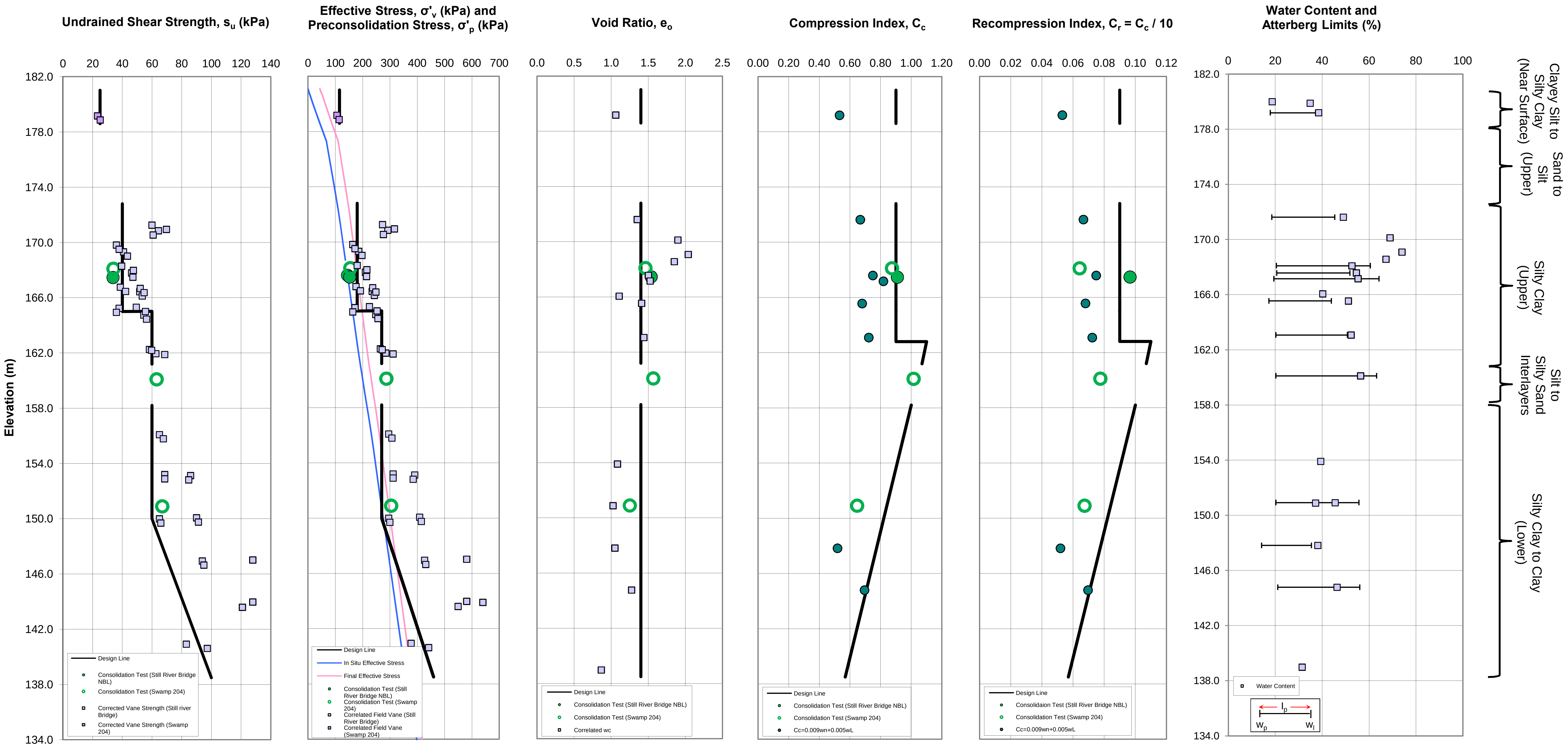
PLOT DATE: September 8, 2014
FILENAME: T:\Projects\2009\09-1111-6014 (URS, Hwy 69, Henvey)\-HC- CONTRACT 2 (Still River Bridge (NBL)-FINAL-STAMPED\0911116014HC003.dwg

# FIGURES

# SUMMARY PLOT OF ENGINEERING PARAMETERS FOR COHESIVE DEPOSITS

## Still River Bridge (NBL) Structure - South Abutment and Approach (One-Span or Two-Span Option)

**FIGURE 1**



**Date:** September 2014
**Project No:** 09-1111-6014-2522

**Prepared By:** MAS/TZ
**Checked By:** JPD/JMAC

Golder Associates

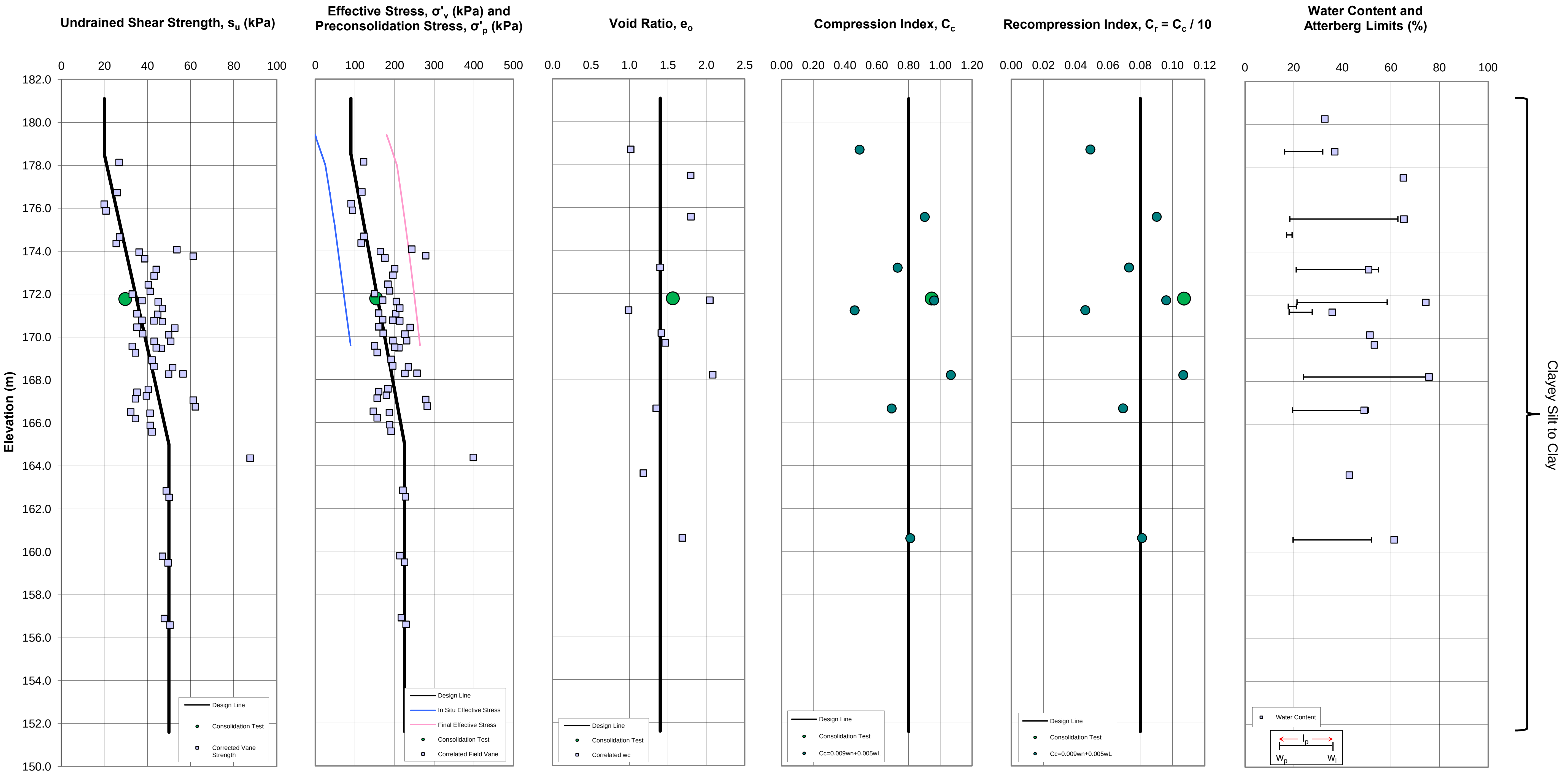
**Golder Associates Ltd.**

\\capws\DavWWWRoot\sites\0911116014highway69FourLaning\Contract 2\Reporting\Final\Still River NBL Bridge\Figures\[09-1111-6014-2522 FIG1 14Sep11 Parameters-Still River Bridge NBL South Abut.xlsx]NBL-South Abutment-Final Plot

# SUMMARY PLOT OF ENGINEERING PARAMETERS FOR COHESIVE DEPOSITS

**Still River Bridge (NBL) Structure - Pier (Two-Span Option) and North Abutment and Approach (One-Span Option)**

**FIGURE 2**



**Date:** September 2014
**Project No:** 09-1111-6014-2522

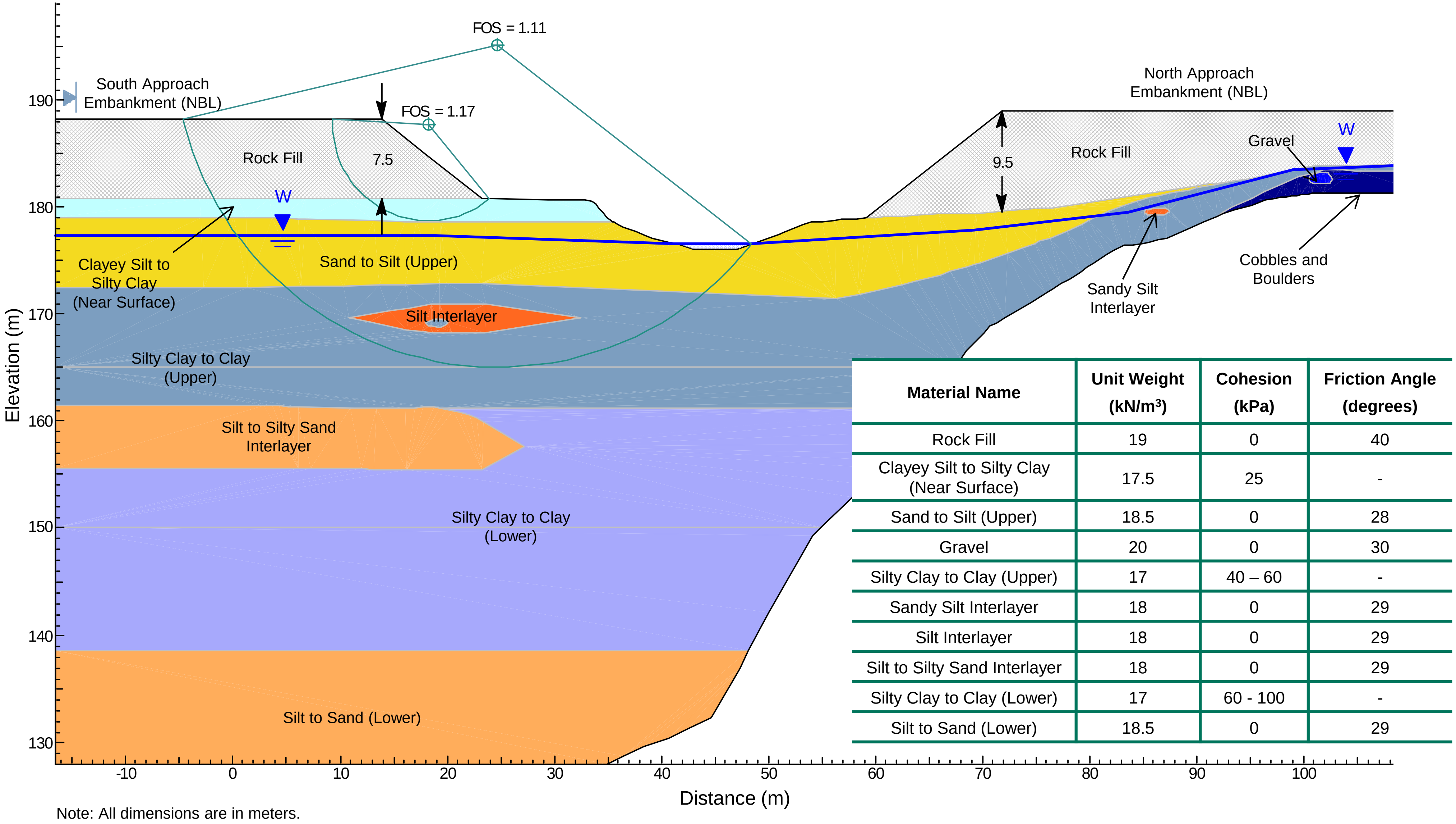
**Prepared By:** MAS/TZ
**Checked By:** JPD/JMAC

**Golder Associates Ltd.**

\\capws\DavWWWRoot\sites\0911116014\highway69FourLaning\Contract 2\Reporting\Final\Still River NBL Bridge\Figures\[09-1111-6014-2522 FIG2 14Sep11 Parameters-Still River Bridge NBL North Abut.xlsx]NBL-Pier&North Abut-Final Plot

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

## Figure 3



| Material Name | Unit Weight (kN/m³) | 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 |
| Gravel | 20 | 0 | 30 |
| Silty Clay to Clay (Upper) | 17 | 40 – 60 | - |
| Sandy Silt Interlayer | 18 | 0 | 29 |
| Silt Interlayer | 18 | 0 | 29 |
| Silt to Silty Sand Interlayer | 18 | 0 | 29 |
| Silty Clay to Clay (Lower) | 17 | 60 - 100 | - |
| Silt to Sand (Lower) | 18.5 | 0 | 29 |

Note: All dimensions are in meters.

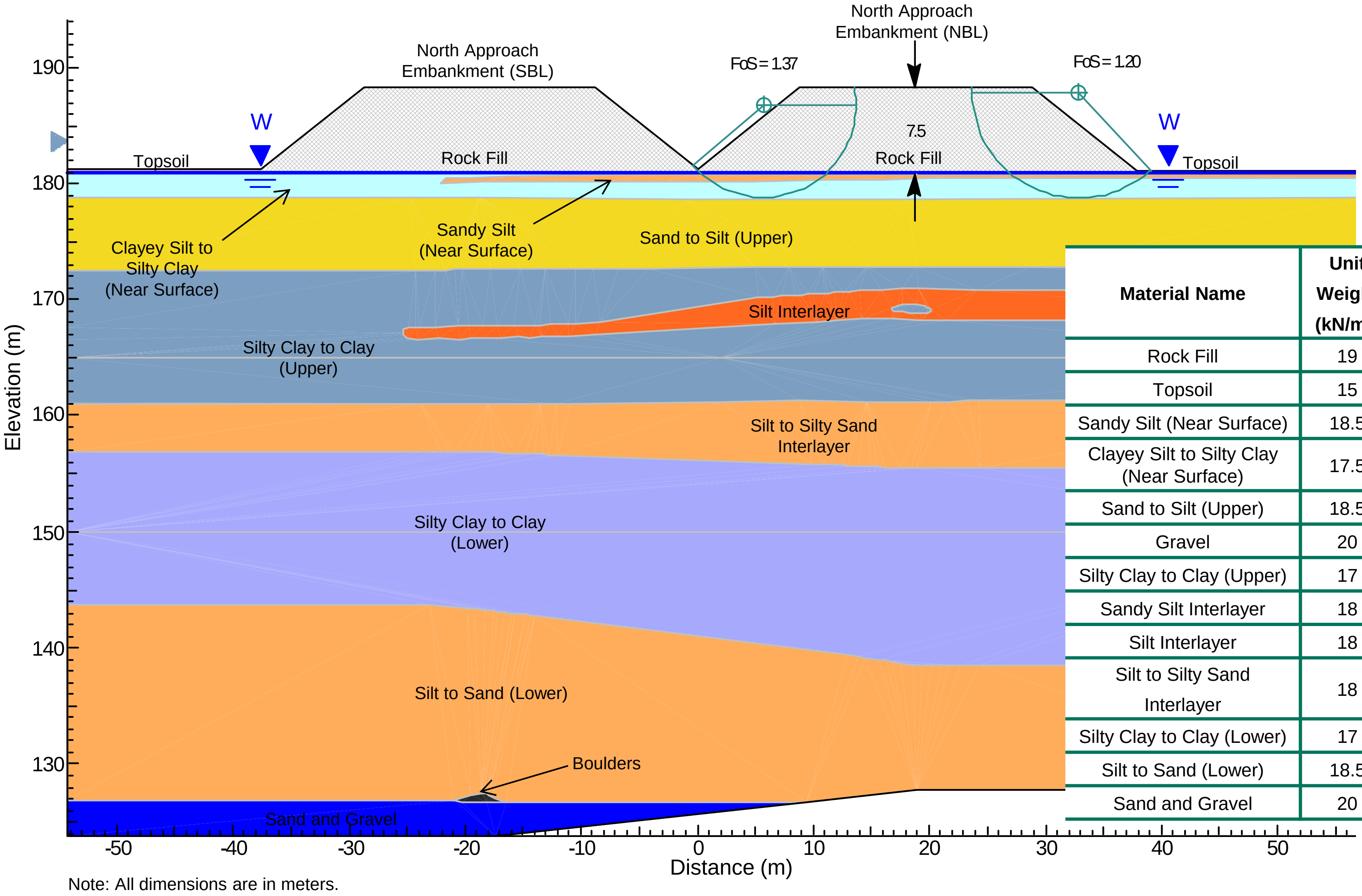
**Date:** September 2014

**Project No:** 09-1111-6014-2522

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

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

**Figure 4**



| Material Name | Unit Weight (kN/m³) | Cohesion (kPa) | Friction Angle (degrees) |
|---|---|---|---|
| Rock Fill | 19 | 0 | 40 |
| Topsoil | 15 | 1 | 27 |
| Sandy Silt (Near Surface) | 18.5 | 0 | 28 |
| Clayey Silt to Silty Clay (Near Surface) | 17.5 | 25 | - |
| Sand to Silt (Upper) | 18.5 | 0 | 28 |
| Gravel | 20 | 0 | 30 |
| Silty Clay to Clay (Upper) | 17 | 40 – 60 | - |
| Sandy Silt Interlayer | 18 | 0 | 29 |
| Silt Interlayer | 18 | 0 | 29 |
| Silt to Silty Sand Interlayer | 18 | 0 | 29 |
| Silty Clay to Clay (Lower) | 17 | 60 – 100 | - |
| Silt to Sand (Lower) | 18.5 | 0 | 29 |
| Sand and Gravel | 20 | 0 | 30 |

Note: All dimensions are in meters.

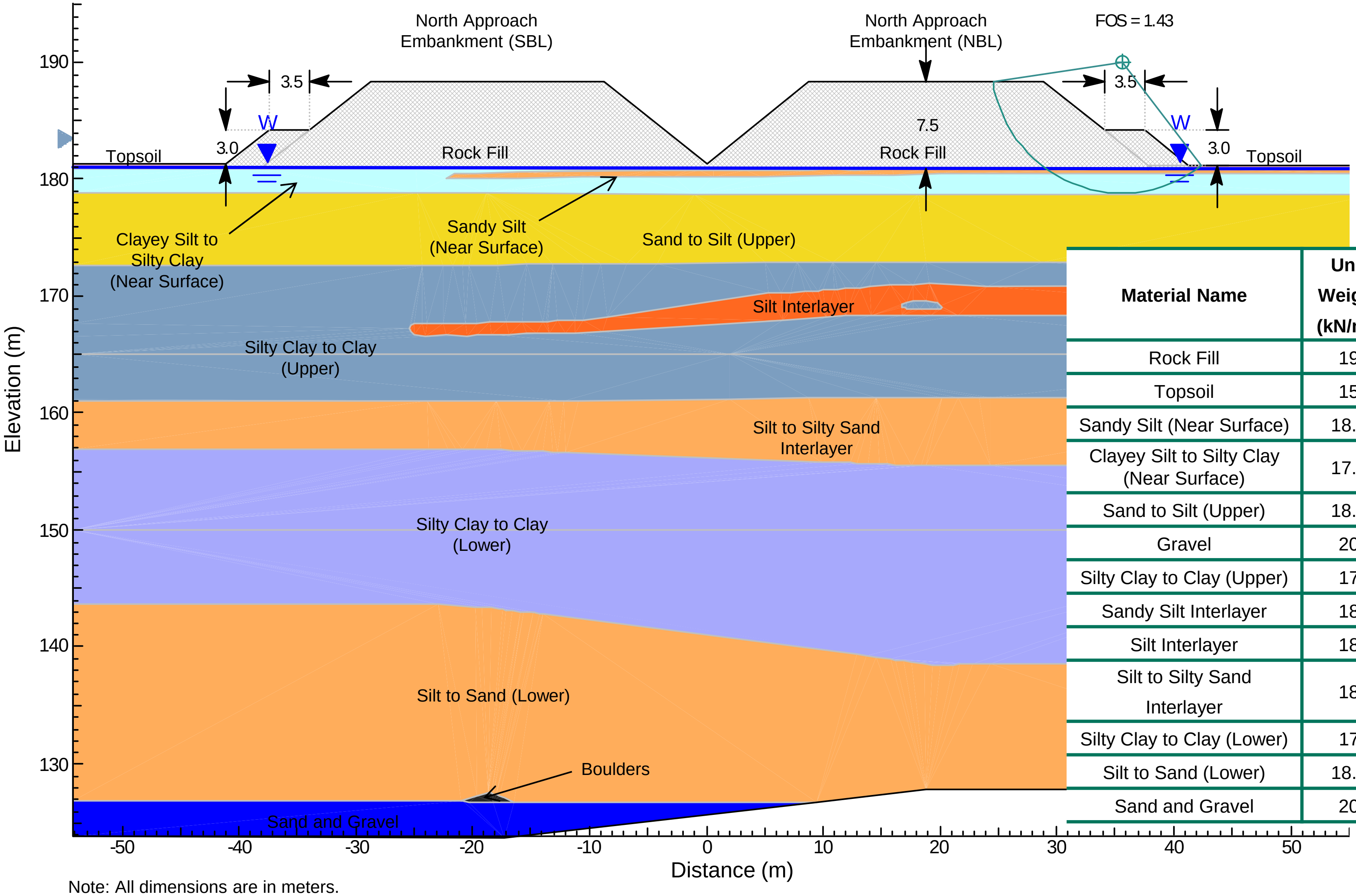
**Date:** September 2014

**Project No:** 09-1111-6014-2522

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

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

# Figure 5



Note: All dimensions are in meters.

| Material Name | Unit Weight (kN/m³) | Cohesion (kPa) | Friction Angle (degrees) |
|---|---|---|---|
| Rock Fill | 19 | 0 | 40 |
| Topsoil | 15 | 1 | 27 |
| Sandy Silt (Near Surface) | 18.5 | 0 | 28 |
| Clayey Silt to Silty Clay (Near Surface) | 17.5 | 25 | - |
| Sand to Silt (Upper) | 18.5 | 0 | 28 |
| Gravel | 20 | 0 | 30 |
| Silty Clay to Clay (Upper) | 17 | 40 – 60 | - |
| Sandy Silt Interlayer | 18 | 0 | 29 |
| Silt Interlayer | 18 | 0 | 29 |
| Silt to Silty Sand Interlayer | 18 | 0 | 29 |
| Silty Clay to Clay (Lower) | 17 | 60 – 100 | - |
| Silt to Sand (Lower) | 18.5 | 0 | 29 |
| Sand and Gravel | 20 | 0 | 30 |

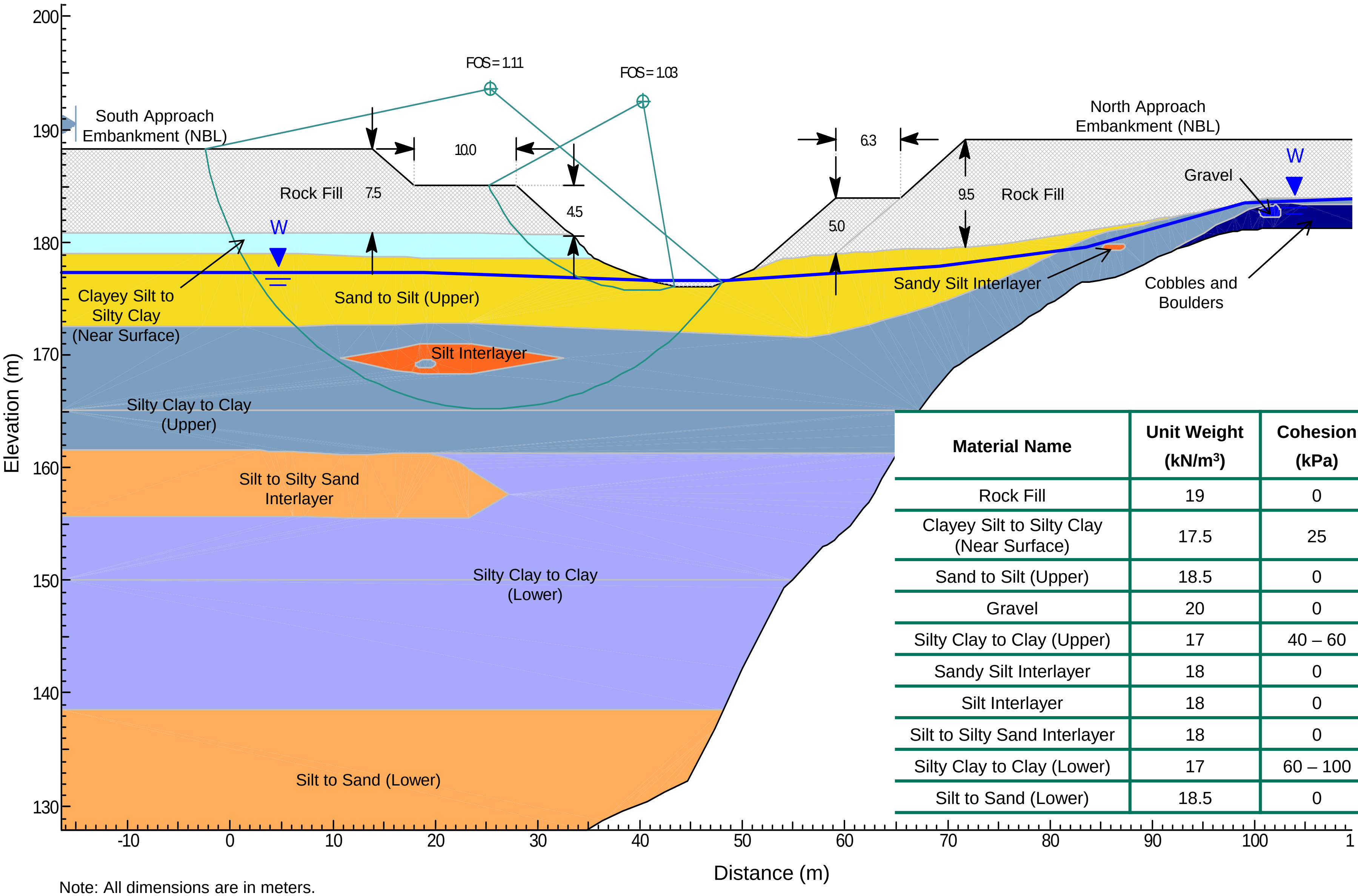
**Date:** September 2014

**Project No:** 09-1111-6014-2522

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

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

## Figure 6



| Material Name | Unit Weight (kN/m³) | 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 |
| Gravel | 20 | 0 | 30 |
| Silty Clay to Clay (Upper) | 17 | 40 – 60 | - |
| Sandy Silt Interlayer | 18 | 0 | 29 |
| Silt Interlayer | 18 | 0 | 29 |
| Silt to Silty Sand Interlayer | 18 | 0 | 29 |
| Silty Clay to Clay (Lower) | 17 | 60 – 100 | - |
| Silt to Sand (Lower) | 18.5 | 0 | 29 |

Note: All dimensions are in meters.

**Date:** September 2014

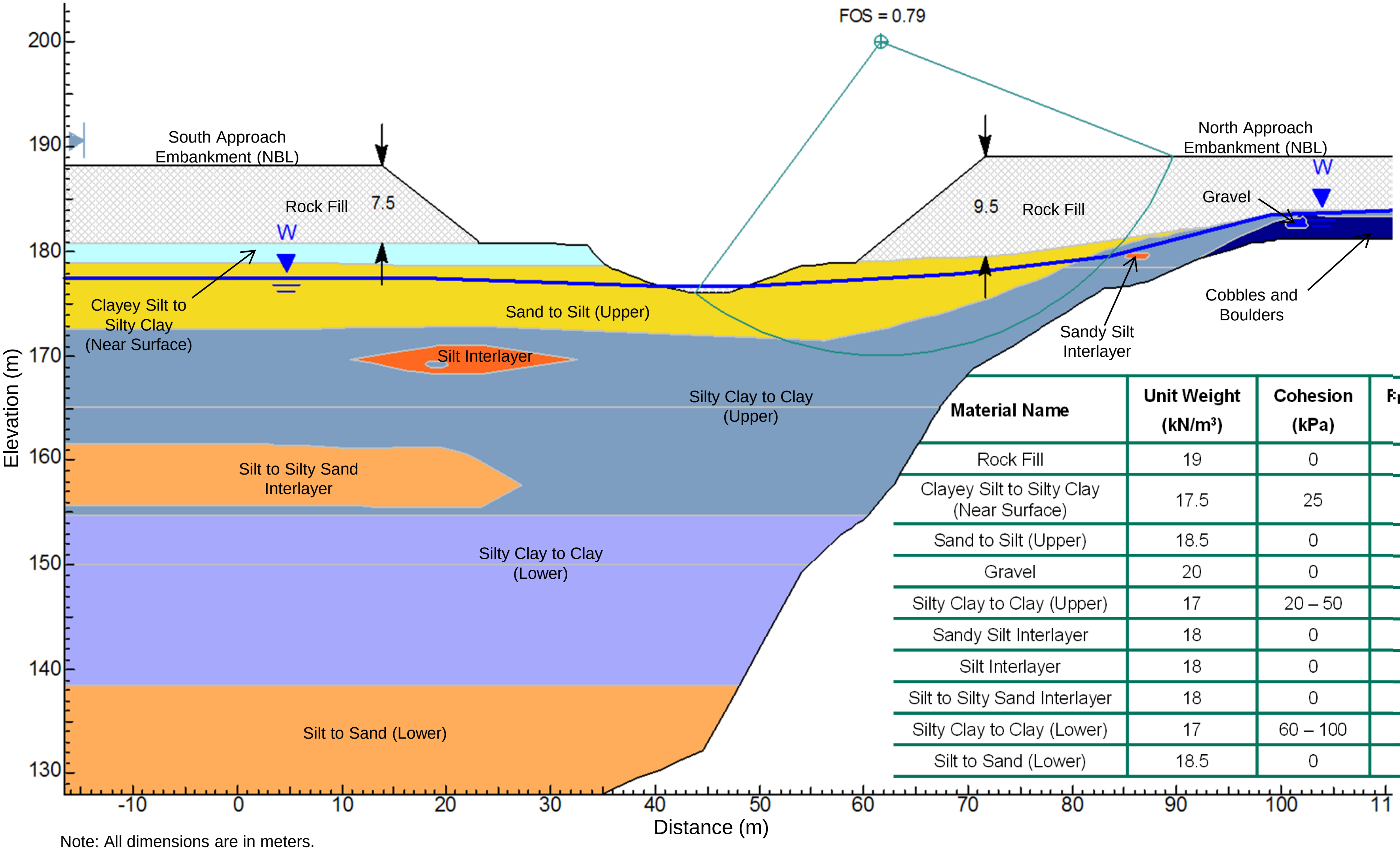
**Project No:** 09-1111-6014-2522

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

Golder Associates

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

## Figure 7



| Material Name | Unit Weight (kN/m³) | 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 |
| Gravel | 20 | 0 | 30 |
| Silty Clay to Clay (Upper) | 17 | 20 – 50 | - |
| Sandy Silt Interlayer | 18 | 0 | 29 |
| Silt Interlayer | 18 | 0 | 29 |
| Silt to Silty Sand Interlayer | 18 | 0 | 29 |
| Silty Clay to Clay (Lower) | 17 | 60 – 100 | - |
| Silt to Sand (Lower) | 18.5 | 0 | 29 |

Note: All dimensions are in meters.

**Date:** September 2014

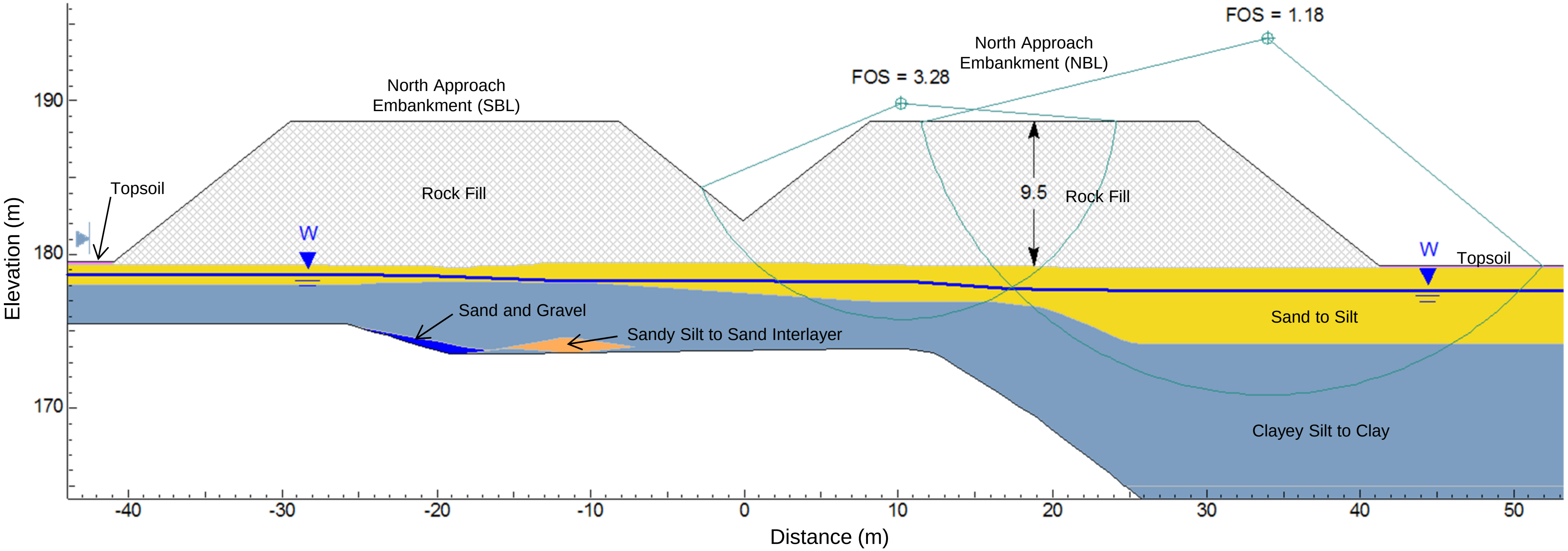
**Project No:** 09-1111-6014-2522

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

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

**Figure 8**

| Material Name | Unit Weight (kN/m³) | Cohesion (kPa) | Friction Angle (degrees) |
|---|---|---|---|
| Rock Fill | 19 | 0 | 40 |
| Topsoil | 15 | 1 | 27 |
| Sand to Silt | 18.5 | 0 | 28 |
| Clayey Silt to Clay | 17 | 20 – 50 | - |
| Sandy Silt to Sand Interlayer | 18.5 | 0 | 29 |
| Sand and Gravel | 20 | 0 | 29 |



Note: All dimensions are in meters.

**Date:** September 2014

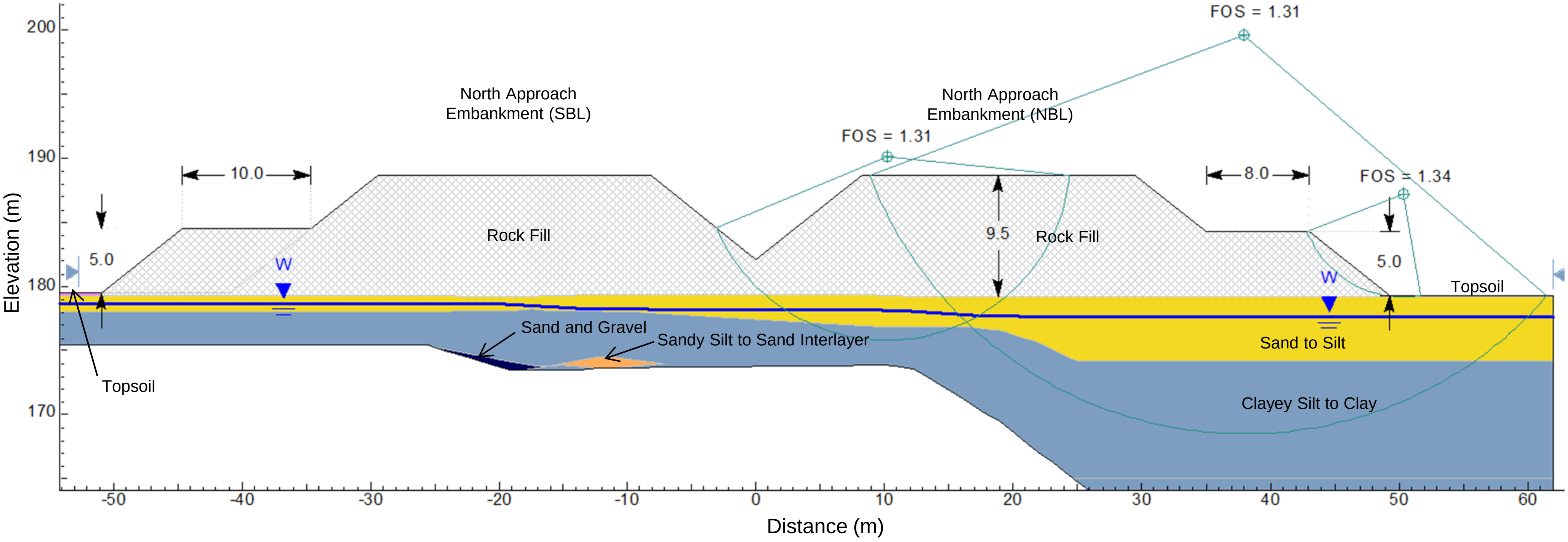
**Project No:** 09-1111-6014-2522

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

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

## Figure 9

| Material Name | Unit Weight (kN/m³) | Cohesion (kPa) | Friction Angle (degrees) |
|---|---|---|---|
| Rock Fill | 19 | 0 | 40 |
| Topsoil | 15 | 1 | 27 |
| Sand to Silt | 18.5 | 0 | 28 |
| Sandy Silt to Sand Interlayer | 18.5 | 0 | 29 |
| Sand and Gravel | 20 | 0 | 29 |
| Clayey Silt to Clay | 17 | 20 – 50 | - |



Note: All dimensions are in meters.

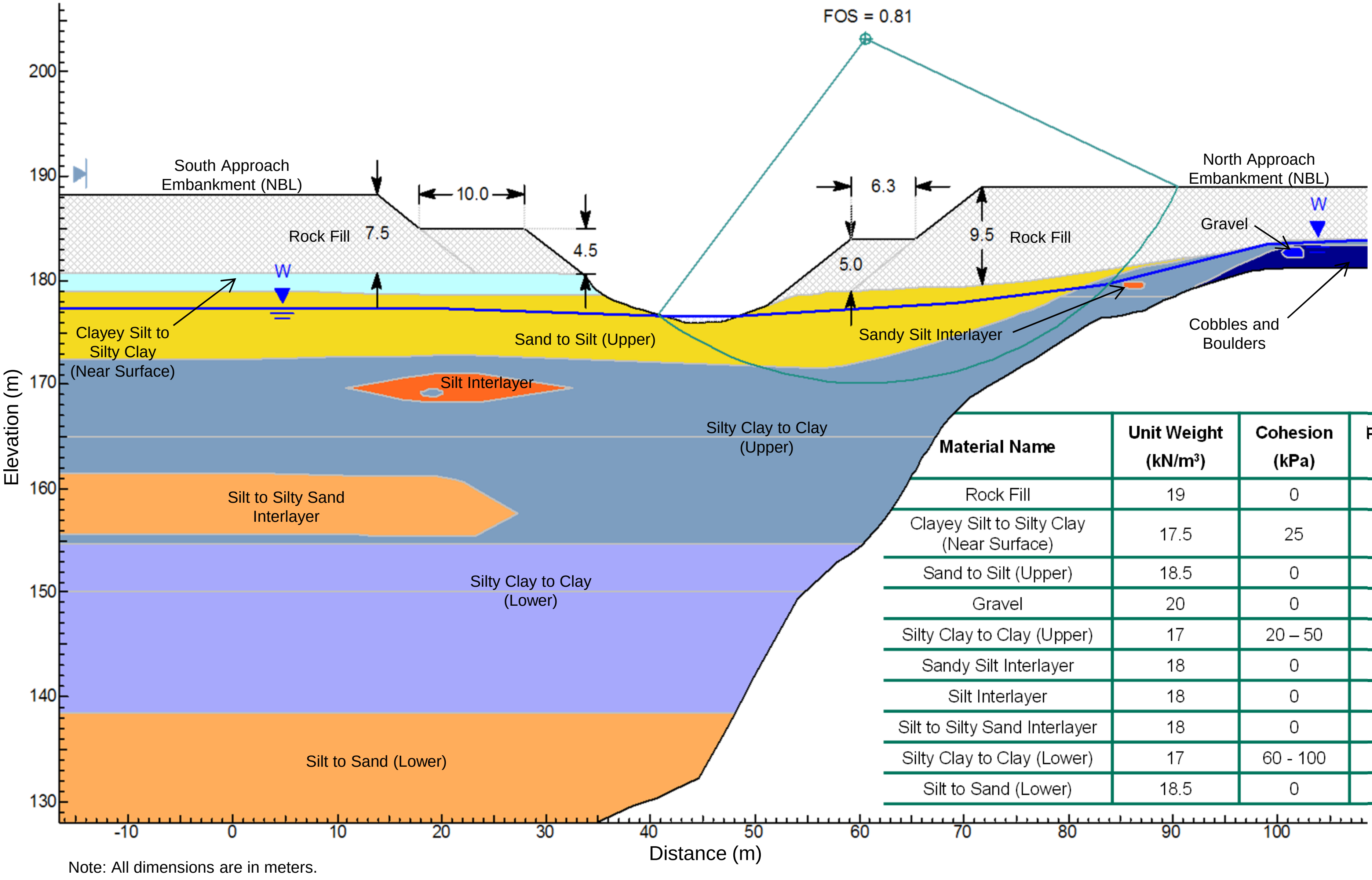
**Date:** September 2014

**Project No:** 09-1111-6014-2522

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

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

## Figure 10



| Material Name | Unit Weight (kN/m³) | 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 |
| Gravel | 20 | 0 | 30 |
| Silty Clay to Clay (Upper) | 17 | 20 – 50 | - |
| Sandy Silt Interlayer | 18 | 0 | 29 |
| Silt Interlayer | 18 | 0 | 29 |
| Silt to Silty Sand Interlayer | 18 | 0 | 29 |
| Silty Clay to Clay (Lower) | 17 | 60 - 100 | - |
| Silt to Sand (Lower) | 18.5 | 0 | 29 |

Note: All dimensions are in meters.

**Date:** September 2014

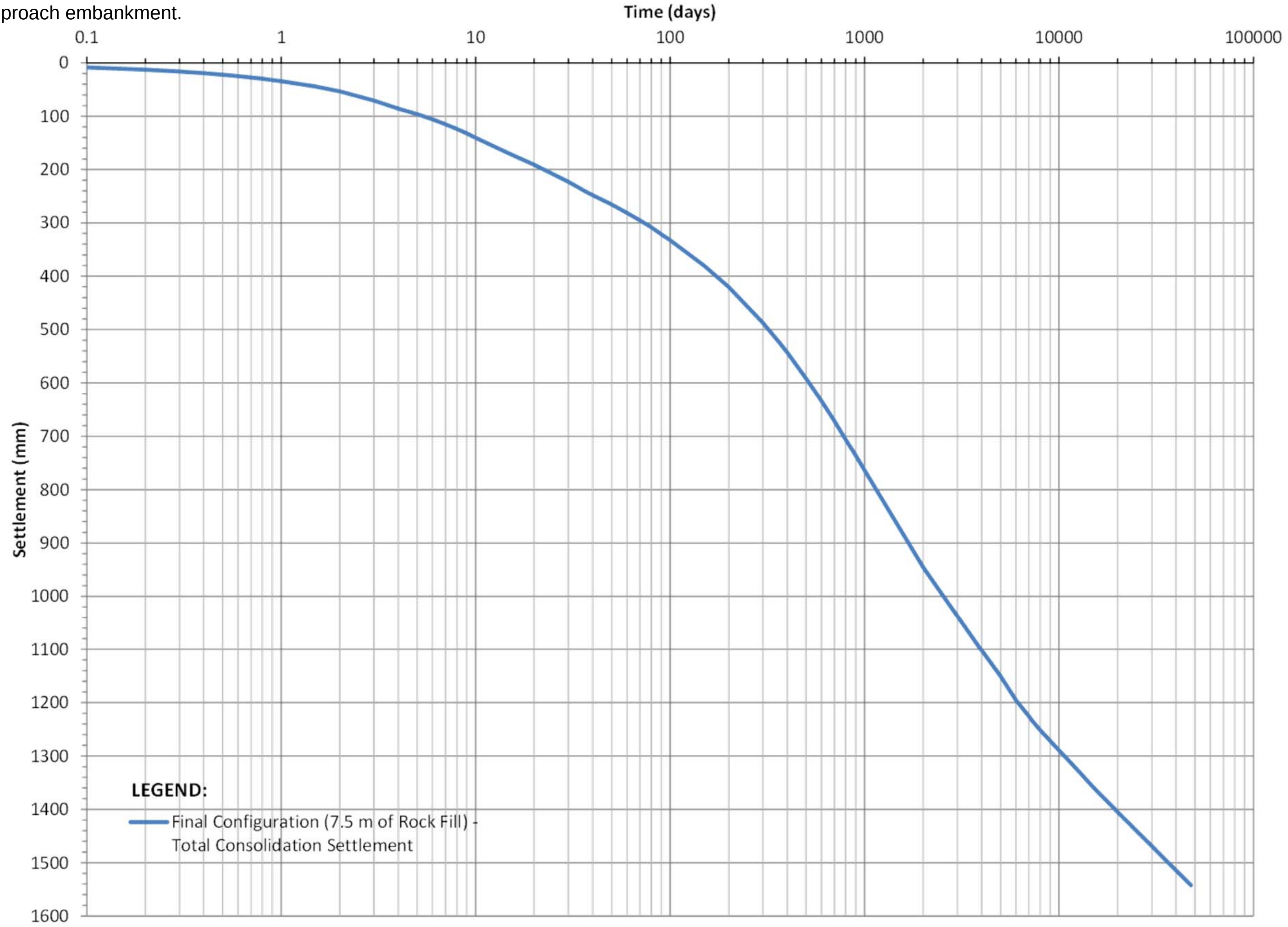
**Project No:** 09-1111-6014-2522

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

# Highway 69 NBL – Still River NBL 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.



**Date:** September 2014

**Project No:** 09-1111-6014-2522

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

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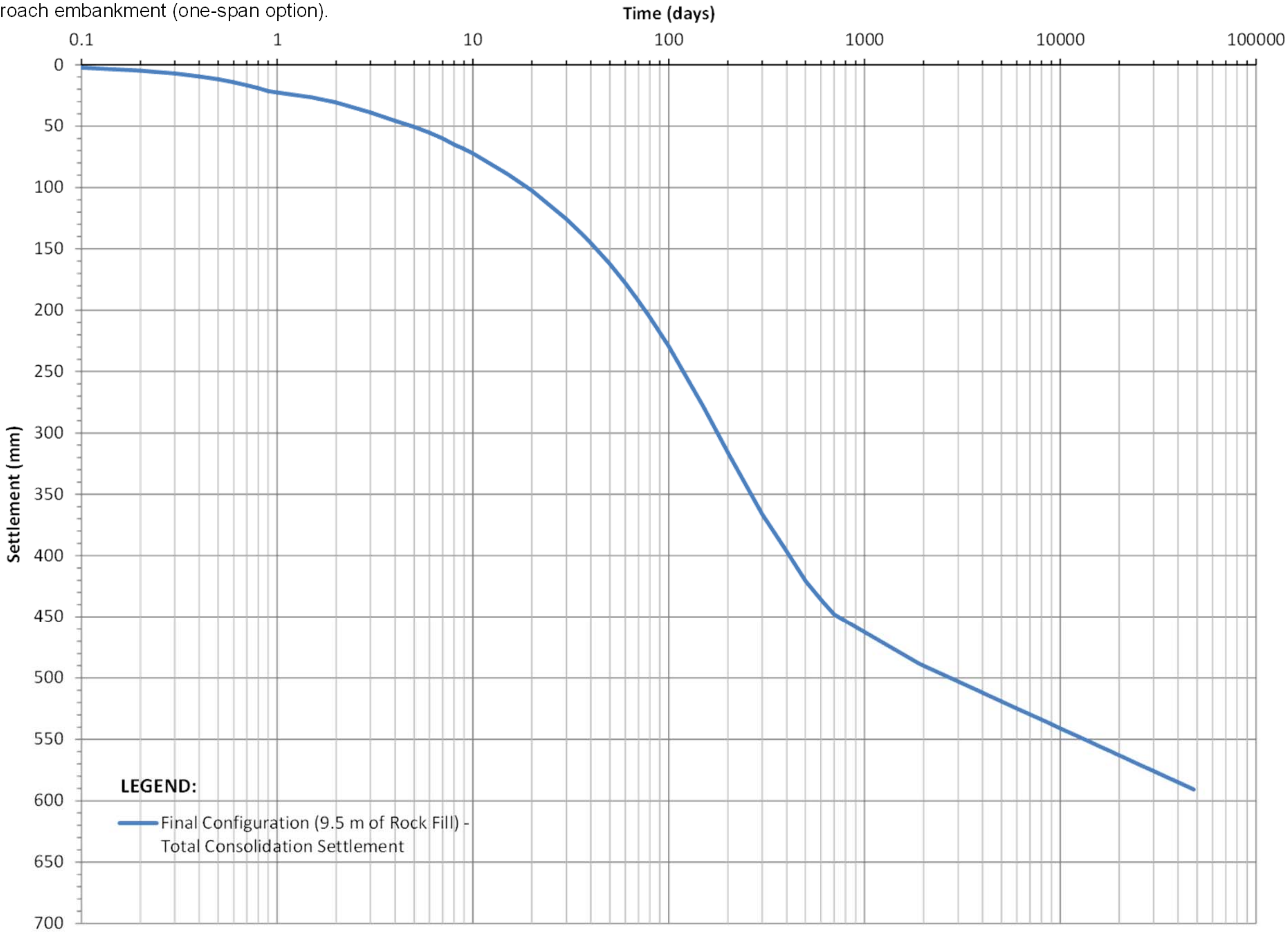
# Highway 69 NBL – Still River NBL Bridge Structure – North Approach (One-Span Option)Time-Rate of Settlement (No Mitigation Options)

## Figure 12

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



**Date:** September 2014

**Project No:** 09-1111-6014-2522

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

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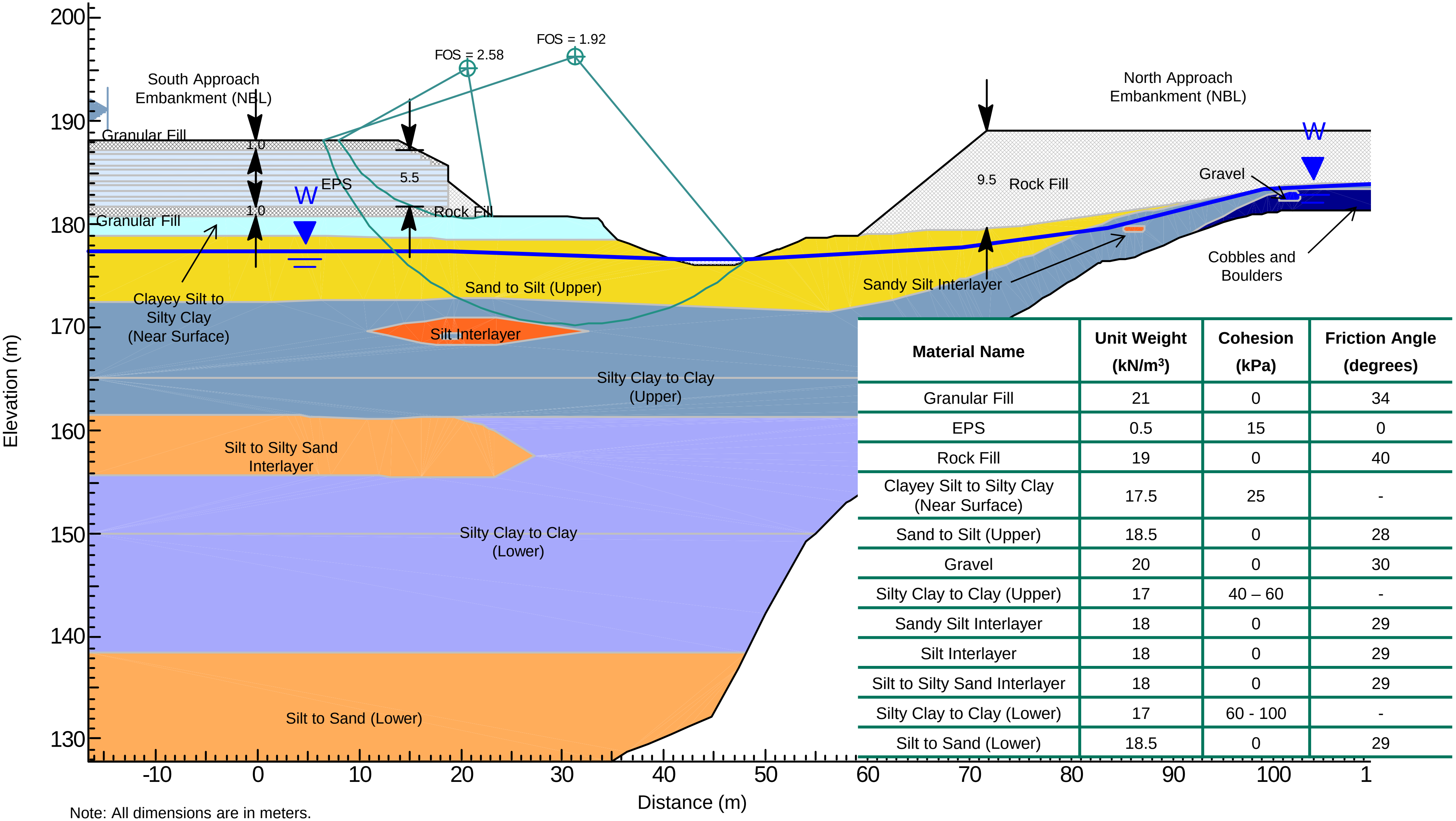
# Highway 69 NBL – Still River NBL Bridge Structure – South Approach Front Slope Stability (5.5 m of EPS Fill)

# Figure 13



| Material Name | Unit Weight (kN/m³) | Cohesion (kPa) | Friction Angle (degrees) |
|---|---|---|---|
| Granular Fill | 21 | 0 | 34 |
| EPS | 0.5 | 15 | 0 |
| Rock Fill | 19 | 0 | 40 |
| Clayey Silt to Silty Clay (Near Surface) | 17.5 | 25 | - |
| Sand to Silt (Upper) | 18.5 | 0 | 28 |
| Gravel | 20 | 0 | 30 |
| Silty Clay to Clay (Upper) | 17 | 40 – 60 | - |
| Sandy Silt Interlayer | 18 | 0 | 29 |
| Silt Interlayer | 18 | 0 | 29 |
| Silt to Silty Sand Interlayer | 18 | 0 | 29 |
| Silty Clay to Clay (Lower) | 17 | 60 - 100 | - |
| Silt to Sand (Lower) | 18.5 | 0 | 29 |

Note: All dimensions are in meters.

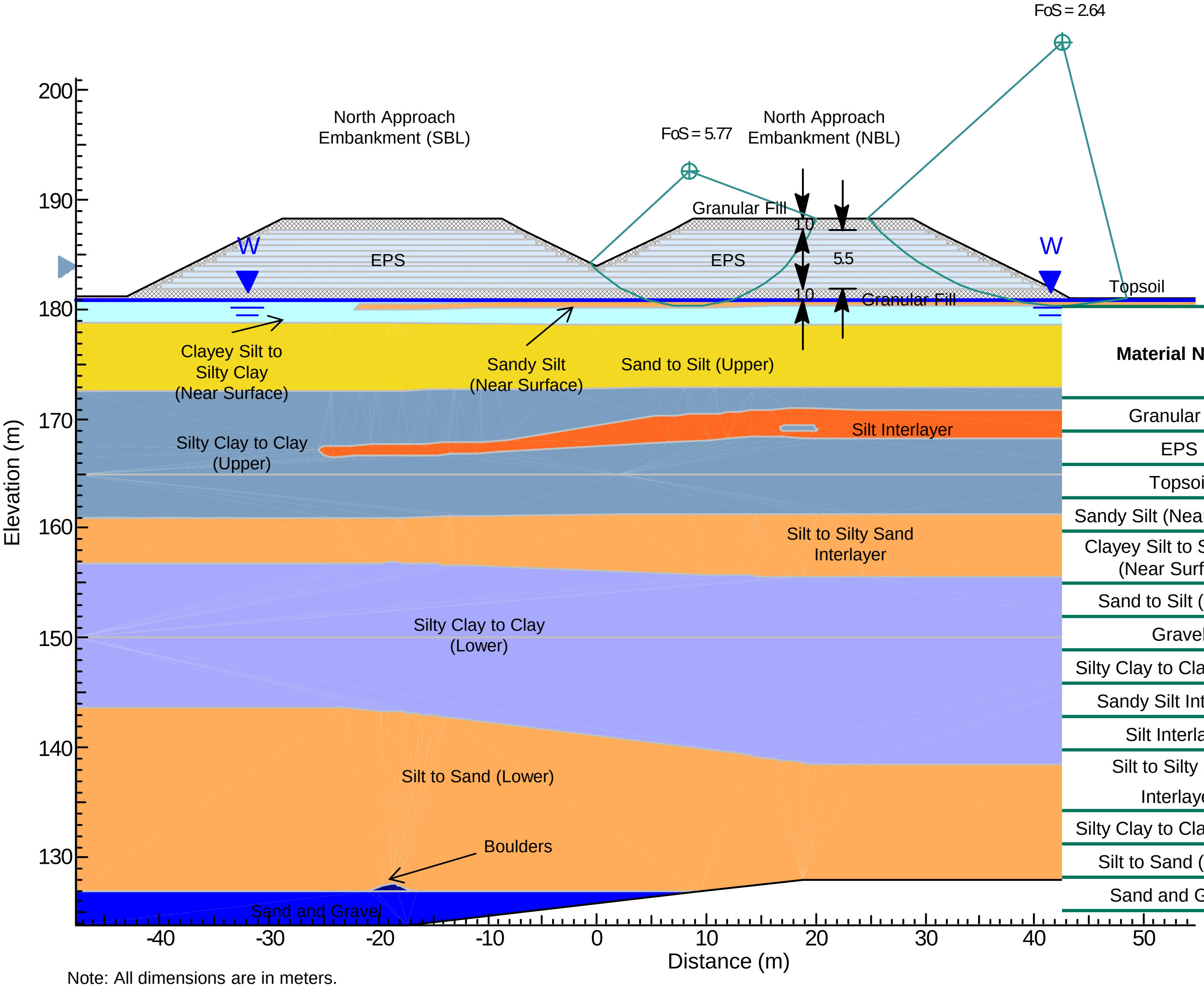
**Date:** September 2014

**Project No:** 09-1111-6014-2522

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

# Highway 69 NBL – Still River NBL Bridge Structure – South Approach Side Slope Stability (5.5 m of EPS Fill)

## Figure 14



| Material Name | Unit Weight (kN/m³) | Cohesion (kPa) | Friction Angle (degrees) |
|---|---|---|---|
| Granular Fill | 21 | 0 | 34 |
| EPS | 0.5 | 15 | 0 |
| Topsoil | 15 | 1 | 27 |
| Sandy Silt (Near Surface) | 18.5 | 0 | 28 |
| Clayey Silt to Silty Clay (Near Surface) | 17.5 | 25 | - |
| Sand to Silt (Upper) | 18.5 | 0 | 28 |
| Gravel | 20 | 0 | 30 |
| Silty Clay to Clay (Upper) | 17 | 40 – 60 | - |
| Sandy Silt Interlayer | 18 | 0 | 29 |
| Silt Interlayer | 18 | 0 | 29 |
| Silt to Silty Sand Interlayer | 18 | 0 | 29 |
| Silty Clay to Clay (Lower) | 17 | 60 - 100 | - |
| Silt to Sand (Lower) | 18.5 | 0 | 29 |
| Sand and Gravel | 20 | 0 | 30 |

Note: All dimensions are in meters.

**Date:** September 2014

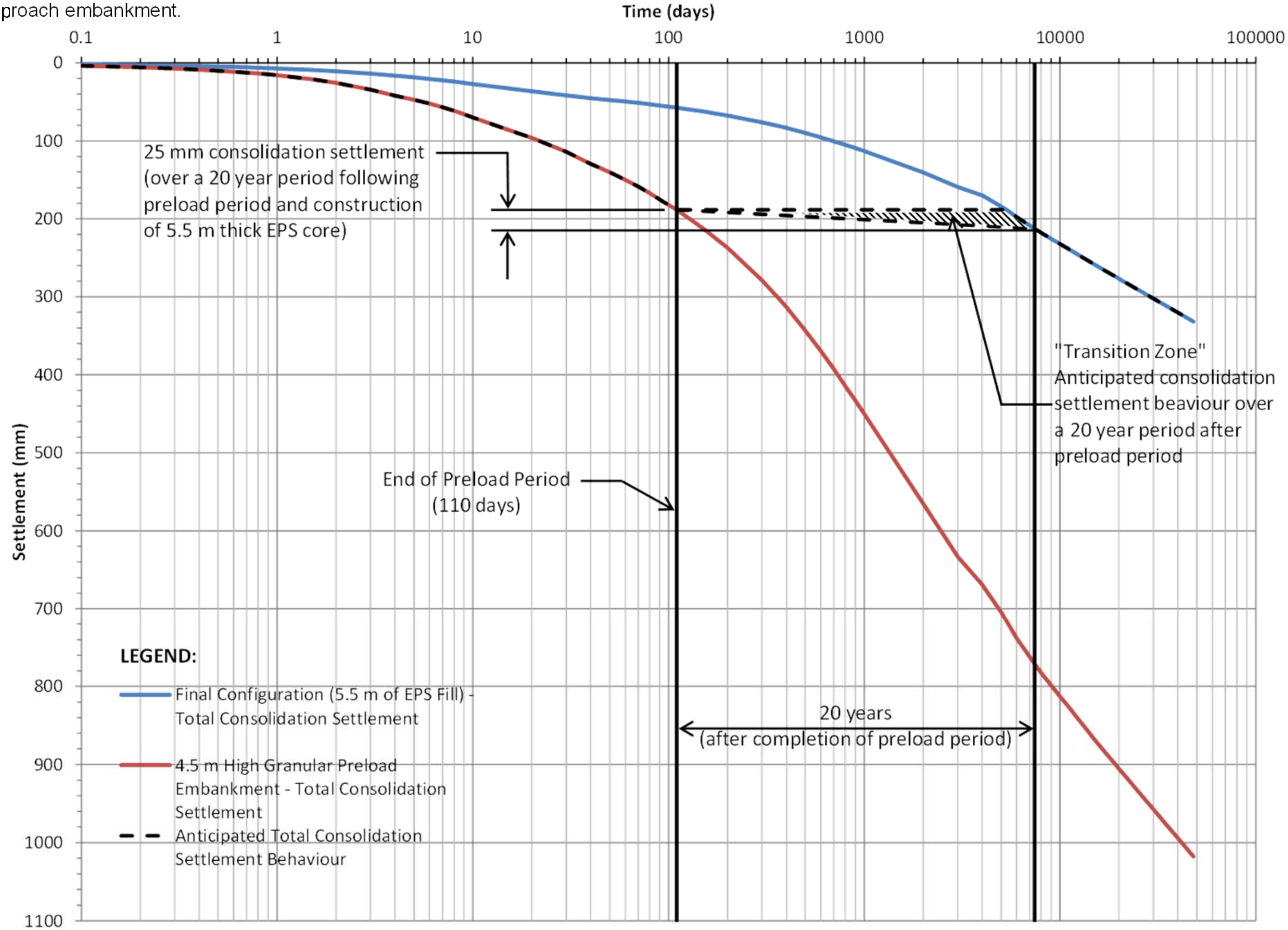
**Project No:** 09-1111-6014-2522

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

## Highway 69 NBL – Still River NBL Bridge Structure – South Approach Time-Rate of Settlement (5.5 m of EPS Fill)

## Figure 15

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



**Date:** September 2014

**Project No:** 09-1111-6014-2522

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

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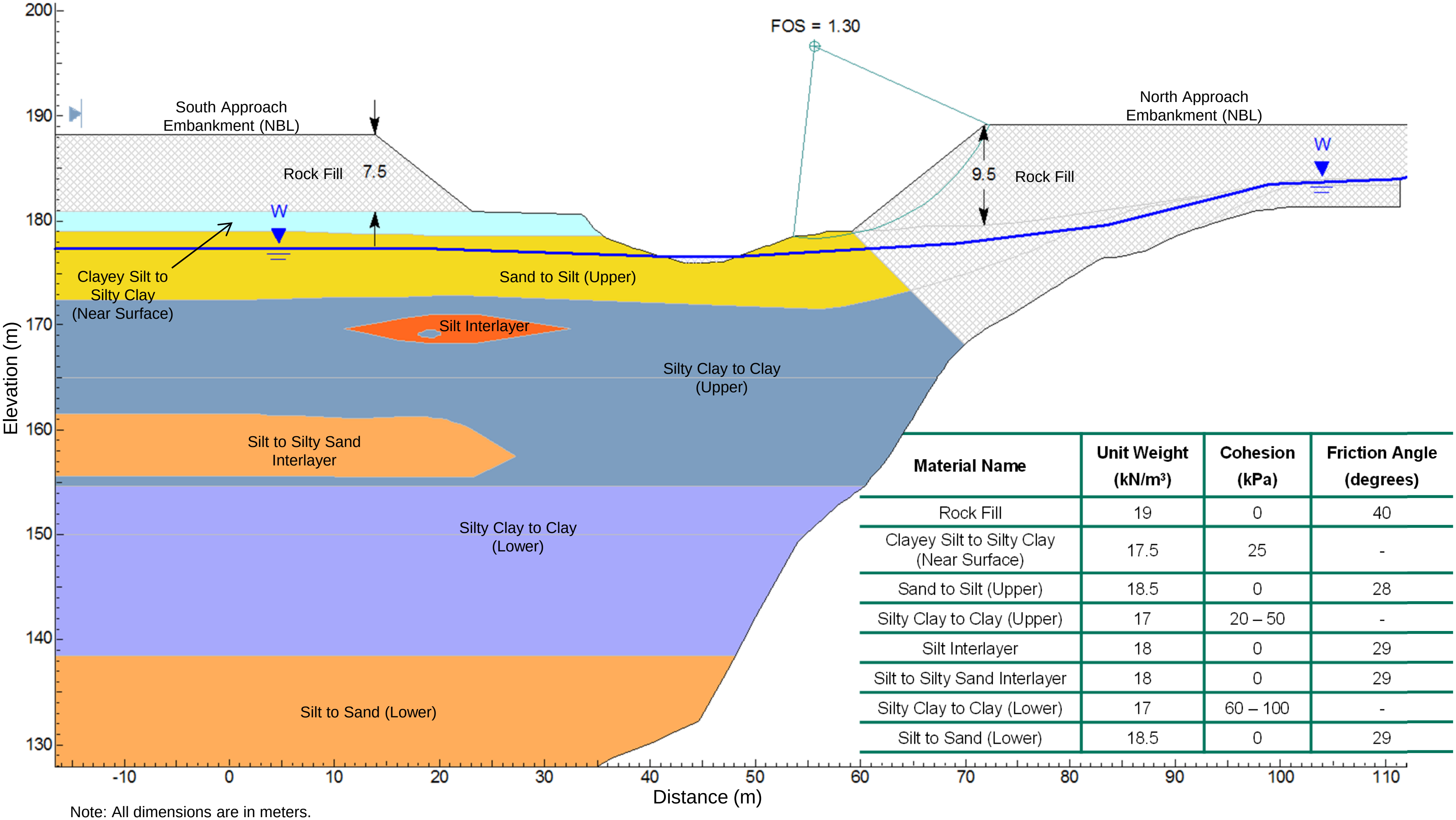
# Highway 69 NBL – Still River NBL Bridge Structure – North Approach (One-Span Bridge)
# Front Slope Stability (Full Sub-Excavation and Backfill)

## Figure 16



| Material Name | Unit Weight (kN/m³) | 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 | 20 – 50 | - |
| Silt Interlayer | 18 | 0 | 29 |
| Silt to Silty Sand Interlayer | 18 | 0 | 29 |
| Silty Clay to Clay (Lower) | 17 | 60 – 100 | - |
| Silt to Sand (Lower) | 18.5 | 0 | 29 |

Note: All dimensions are in meters.

**Date:** September 2014

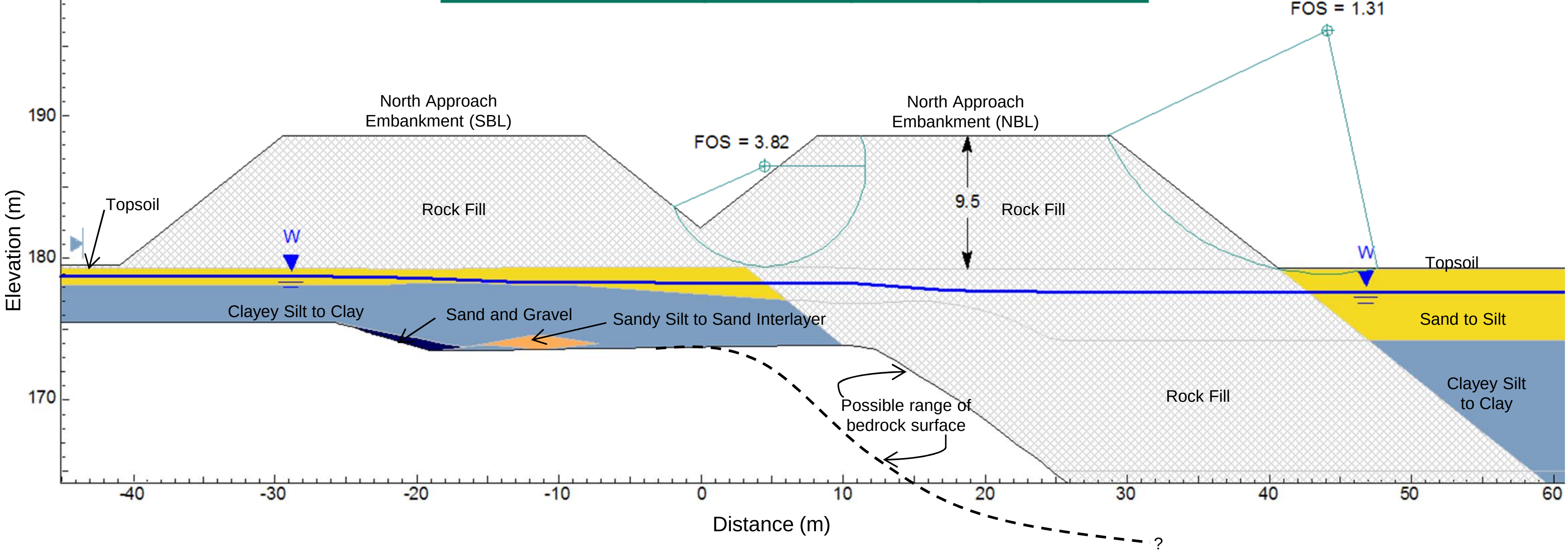
**Project No:** 09-1111-6014-2522

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

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

**Figure 17**

| Material Name | Unit Weight (kN/m³) | Cohesion (kPa) | Friction Angle (degrees) |
|---|---|---|---|
| Rock Fill | 19 | 0 | 40 |
| Topsoil | 15 | 1 | 27 |
| Sand to Silt | 18.5 | 0 | 28 |
| Sandy Silt to Sand Interlayer | 18.5 | 0 | 29 |
| Sand and Gravel | 20 | 0 | 29 |
| Clayey Silt to Clay | 17 | 20 – 50 | - |



NOTE:
Given the subsurface conditions/steeply sloping bedrock in this area, the sub-excavation may need to extend below Elevation 164 m in order to fully remove the cohesive deposit.

Note: All dimensions are in meters.

**Date:** September 2014

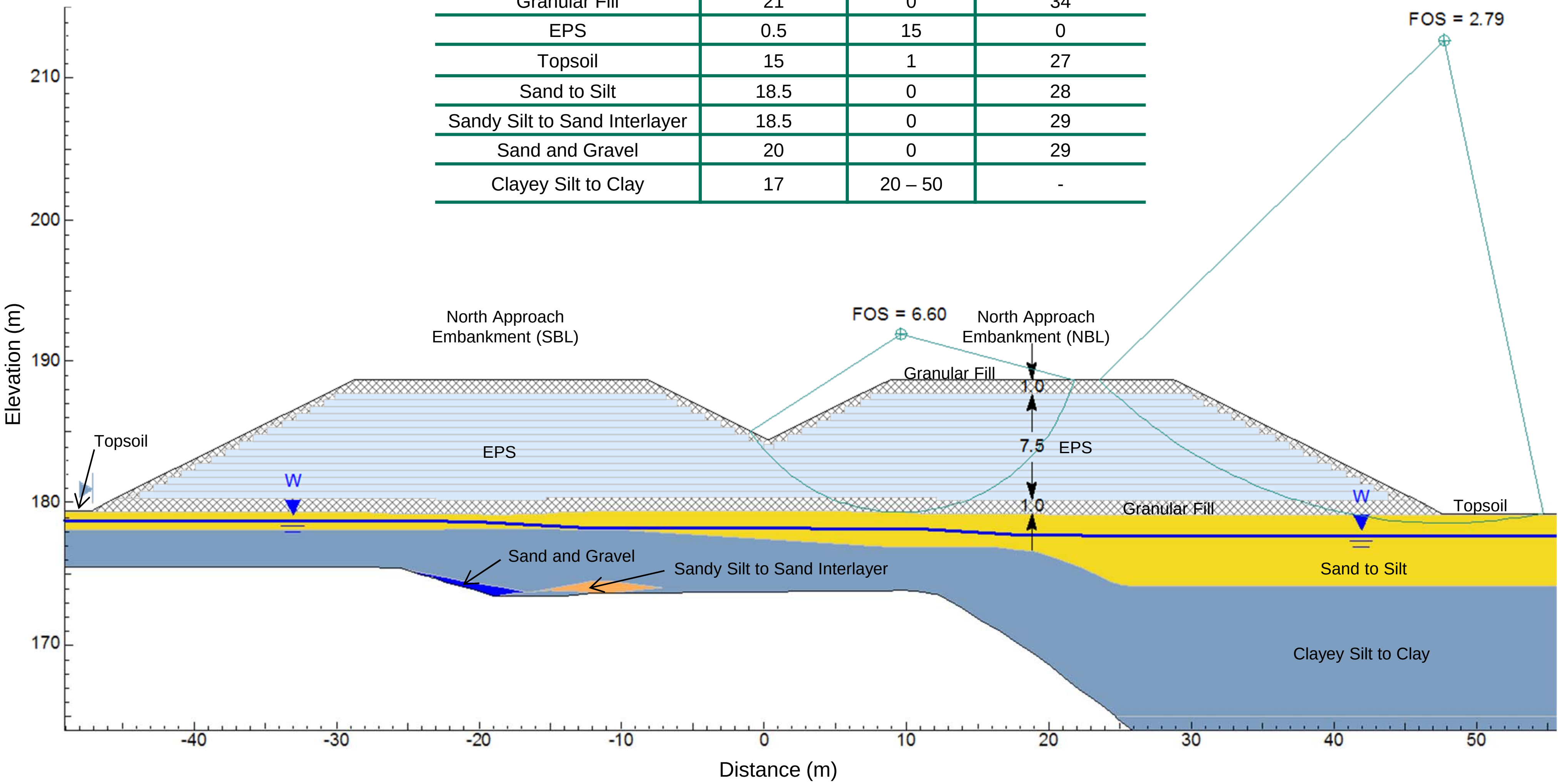
**Project No:** 09-1111-6014-2522

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

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

## Figure 18

| Material Name | Unit Weight (kN/m³) | Cohesion (kPa) | Friction Angle (degrees) |
|---|---|---|---|
| Granular Fill | 21 | 0 | 34 |
| EPS | 0.5 | 15 | 0 |
| Topsoil | 15 | 1 | 27 |
| Sand to Silt | 18.5 | 0 | 28 |
| Sandy Silt to Sand Interlayer | 18.5 | 0 | 29 |
| Sand and Gravel | 20 | 0 | 29 |
| Clayey Silt to Clay | 17 | 20 – 50 | - |



Note: All dimensions are in meters.

**Date:** September 2014
**Project No:** 09-1111-6014-2522

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

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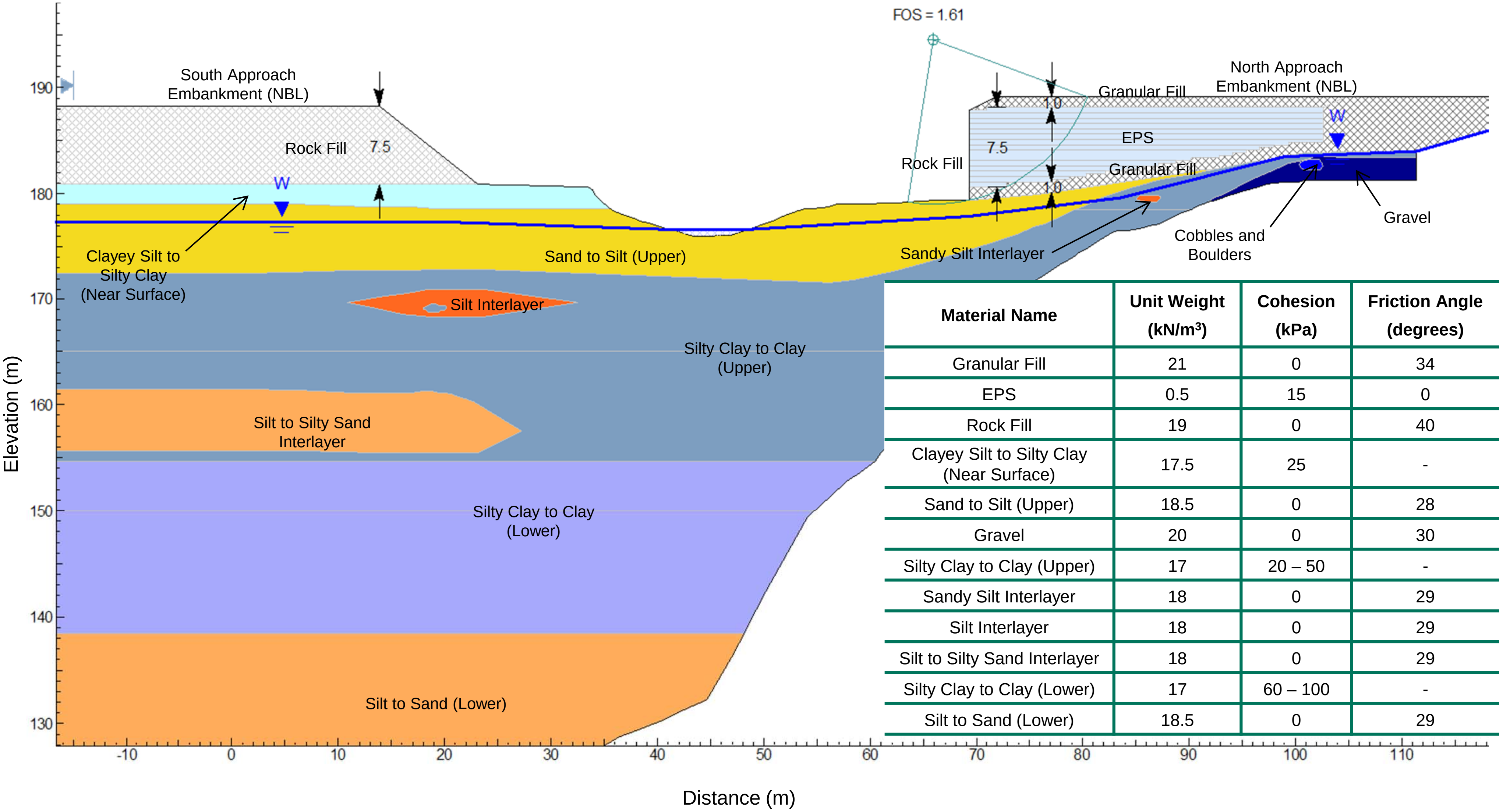
# Highway 69 NBL – Still River NBL Bridge Structure – North Approach (One-Span Bridge)
# Front Slope Stability (7.5 m of EPS Fill)

**Figure 19**



| Material Name | Unit Weight (kN/m$^3$) | Cohesion (kPa) | Friction Angle (degrees) |
|---|---|---|---|
| Granular Fill | 21 | 0 | 34 |
| EPS | 0.5 | 15 | 0 |
| Rock Fill | 19 | 0 | 40 |
| Clayey Silt to Silty Clay (Near Surface) | 17.5 | 25 | - |
| Sand to Silt (Upper) | 18.5 | 0 | 28 |
| Gravel | 20 | 0 | 30 |
| Silty Clay to Clay (Upper) | 17 | 20 – 50 | - |
| Sandy Silt Interlayer | 18 | 0 | 29 |
| Silt Interlayer | 18 | 0 | 29 |
| Silt to Silty Sand Interlayer | 18 | 0 | 29 |
| Silty Clay to Clay (Lower) | 17 | 60 – 100 | - |
| Silt to Sand (Lower) | 18.5 | 0 | 29 |

Note: All dimensions are in meters.

**Date:** September 2014

**Project No:** 09-1111-6014-2522

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

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# 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

| | |
|---|---|
| $\pi$ | 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

| | |
|---|---|
| $\gamma$ | shear strain |
| $\Delta$ | change in, e.g. in stress: $\Delta\ \sigma$ |
| $\varepsilon$ | linear strain |
| $\varepsilon_v$ | volumetric strain |
| $\eta$ | coefficient of viscosity |
| $\upsilon$ | Poisson's ratio |
| $\sigma$ | total stress |
| $\sigma'$ | effective stress ($\sigma' = \sigma - u$) |
| $\sigma'_{vo}$ | initial effective overburden stress |
| $\sigma_1$, $\sigma_2$, $\sigma_3$ | principal stress (major, intermediate, minor) |
| $\sigma_{oct}$ | mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$ |
| $\tau$ | 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 |
| $\gamma'$ | 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 |
| $C_\alpha$ | 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 |
| $\sigma'_p$ | pre-consolidation stress |
| OCR | over-consolidation ratio $= \sigma'_p / \sigma'_{vo}$ |

### (d) Shear Strength

| | |
|---|---|
| $\tau_p$, $\tau_r$ | peak and residual shear strength |
| $\phi'$ | effective angle of internal friction |
| $\delta$ | angle of interface friction |
| $\mu$ | 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 $\rho$. Unit weight symbol is $\gamma$ where $\gamma = \rho g$ (i.e. mass density multiplied by acceleration due to gravity)

**Notes:** 1 $\tau = c' + \sigma' \tan \phi'$
2 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 |

## II. PENETRATION RESISTANCE

**Standard Penetration Resistance (SPT), N:**

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

**Dynamic Cone Penetration Resistance; $N_d$:**

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

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

**Piezo-Cone Penetration Test (CPT)**

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

## III. SOIL DESCRIPTION

### (a) Non-Cohesive Soils

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

### (b) Cohesive Soils

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

## IV. SOIL TESTS

| | |
|---|---|
| w | water content |
| $w_p$ | plastic limit |
| $w_l$ | liquid limit |
| C | consolidation (oedometer) test |
| CHEM | chemical analysis (refer to text) |
| CID | consolidated isotropically drained triaxial test[1] |
| CIU | consolidated isotropically undrained triaxial test with porewater pressure measurement[1] |
| $D_R$ | relative density (specific gravity, $G_s$) |
| DS | direct shear test |
| M | sieve analysis for particle size |
| MH | combined sieve and hydrometer (H) analysis |
| MPC | Modified Proctor compaction test |
| SPC | Standard Proctor compaction test |
| OC | organic content test |
| $SO_4$ | concentration of water-soluble sulphates |
| UC | unconfined compression test |
| UU | unconsolidated undrained triaxial test |
| V | field vane (LV-laboratory vane test) |
| $\gamma$ | 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<br>With (cohesive) | Sand and Gravel<br>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 $90^{o}$ angle is horizontal.

### Description and Notes

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

### Abbreviations

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

# RECORD OF BOREHOLE No B202-01

SHEET 1 OF 2

**METRIC**

PROJECT 09-1111-6014
W.P. 5139-08-01
DIST
HWY 69
DATUM Geodetic

LOCATION N 5074802.7 ;E 225185.9
BOREHOLE TYPE 127 mm O.D. Continuous Flight Hollow Stem Auger
DATE February 17, 2011

ORIGINATED BY MR
COMPILED BY MAS
CHECKED BY TVA

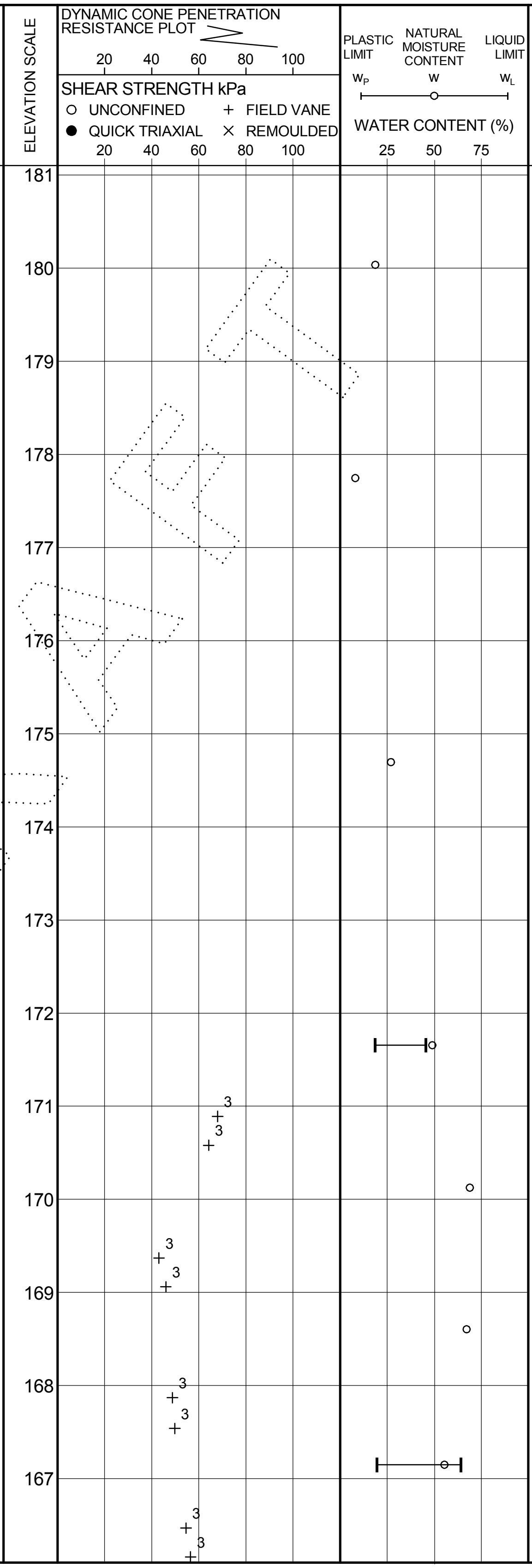
## SOIL PROFILE

| ELEV<br>DEPTH | DESCRIPTION |
|---|---|
| 181.1 | GROUND SURFACE |
| 0.0 | TOPSOIL |
| 180.8<br>0.3 | SILTY CLAY, trace sand<br>Firm<br>Brownish grey to grey<br>Moist |
| 179.0<br>2.1 | SAND, trace gravel, trace silt, trace clay<br>Loose to compact<br>Brown to grey<br>Moist to wet<br>Becoming wet below a depth of 3.7 m<br>Becoming grey at a depth of 4.5 m |
| 172.4<br>8.7 | SILTY CLAY to CLAY, trace sand<br>Firm to stiff<br>Grey to brownish grey<br>Moist |

## SAMPLES

| NUMBER | TYPE | "N" VALUES | UNIT WEIGHT γ kN/m³ | GR | SA | SI | CL |
|---|---|---|---|---|---|---|---|
| 1 | SS | 6 | | | | | |
| 2 | SS | 4 | | | | | |
| 3 | TO | PH | | | | | |
| 4 | SS | 13 | | 1 | 94 | 4 | 1 |
| 5 | SS | 10 | | | | | |
| 6 | SS | 9 | | | | | |
| 7 | SS | 8 | | | | | |
| 8 | SS | 6 | | | | | |
| 9 | SS | 3 | | 0 | 4 | 49 | 47 |
| 10 | SS | 2 | | | | | |
| 11 | SS | 1 | | | | | |
| 12 | SS | PH | 16.6 | | | | |

GROUND WATER CONDITIONS: ∇ at approx. elevation 177.6



Continued Next Page

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

# RECORD OF BOREHOLE No B202-01

SHEET 2 OF 2

**METRIC**

PROJECT 09-1111-6014

W.P. 5139-08-01

LOCATION N 5074802.7 ;E 225185.9

ORIGINATED BY MR

DIST HWY 69

BOREHOLE TYPE 127 mm O.D. Continuous Flight Hollow Stem Auger

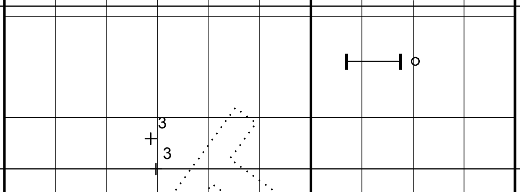
COMPILED BY MAS

DATUM Geodetic

DATE February 17, 2011

CHECKED BY TVA

| SOIL PROFILE | | | SAMPLES | | | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100<br>SHEAR STRENGTH kPa<br>○ UNCONFINED + FIELD VANE<br>● QUICK TRIAXIAL × REMOULDED<br>20 40 60 80 100 | PLASTIC LIMIT $w_P$ / NATURAL MOISTURE CONTENT $w$ / LIQUID LIMIT $w_L$<br>WATER CONTENT (%)<br>25 50 75 | UNIT WEIGHT γ kN/m³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| ELEV DEPTH | DESCRIPTION<br>--- *CONTINUED FROM PREVIOUS PAGE* --- | STRAT PLOT | NUMBER | TYPE | "N" VALUES | | | | | | GR | SA | SI | CL |
| | SILTY CLAY to CLAY, trace sand<br>Firm to stiff<br>Grey to brownish grey<br>Moist | ▨ | 13 | SS | 3 | | 166 | | | | 0 | 1 | 45 | 54 |
| | | ▨ | | | | | 165 | | | | | | | |
| 164.5<br>16.6 | END OF BOREHOLE<br><br>NOTE:<br><br>1. Water level in open borehole at a depth of 3.8 m below ground surface (Elev. 177.3 m) upon completion of drilling. | | | | | | | | | | | | | |



+³, ×³: Numbers refer to Sensitivity

○ ³% STRAIN AT FAILURE

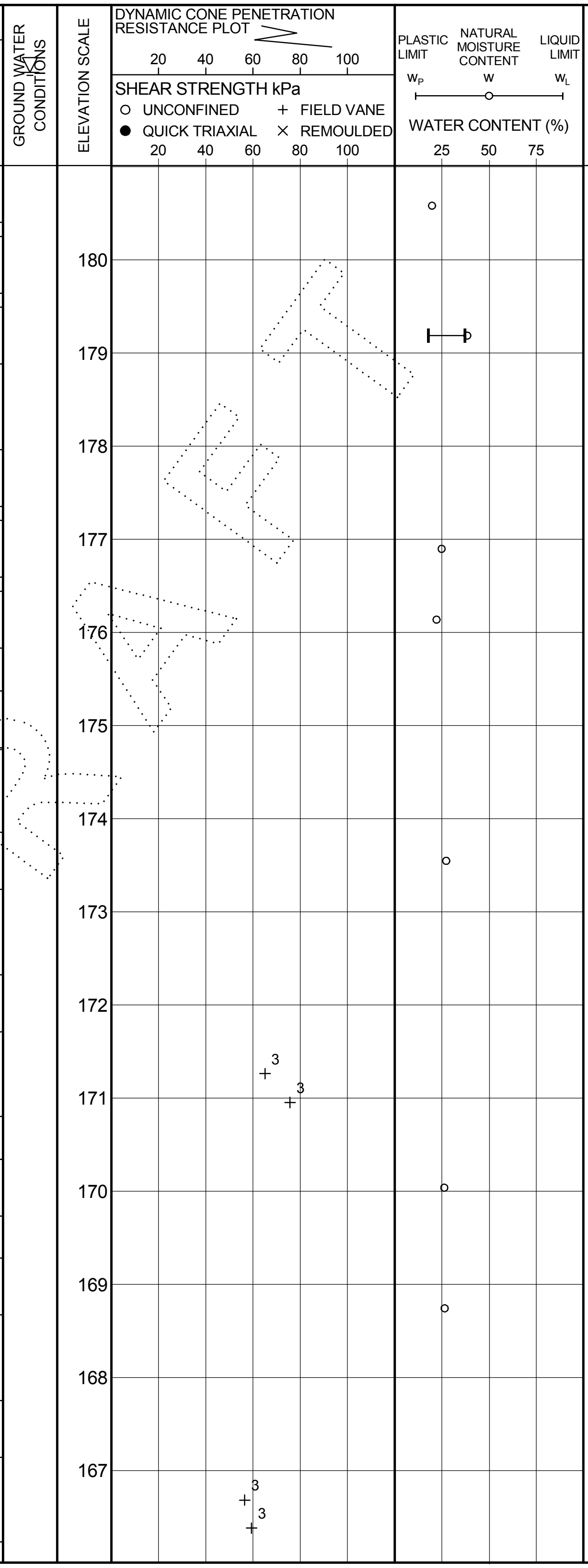
# RECORD OF BOREHOLE No B202-02

SHEET 1 OF 4

**METRIC**

| | | | | | |
|---|---|---|---|---|---|
| PROJECT | 09-1111-6014 | | | | |
| W.P. | 5139-08-01 | LOCATION | N 5074821.3 ;E 225178.6 | ORIGINATED BY | MR |
| DIST HWY | 69 | BOREHOLE TYPE | 213 mm O.D. Cont. Flight Hollow Stem Auger Augers, HW Casing, Wash Boring | COMPILED BY | MAS |
| DATUM | Geodetic | DATE | February 10,14,15 and 16, 2011 | CHECKED BY | TVA |

| ELEV DEPTH | SOIL PROFILE DESCRIPTION | SAMPLES NUMBER | TYPE | "N" VALUES | UNIT WEIGHT γ kN/m³ | GR | SA | SI | CL |
|---|---|---|---|---|---|---|---|---|---|
| 181.0 | GROUND SURFACE | | | | | | | | |
| 0.0 | TOPSOIL | 1A | SS | 5 | | | | | |
| 0.2 | Sandy SILT, trace clay<br>Loose<br>Brownish grey<br>Moist | 1B | | | | | | | |
| 180.3<br>0.7 | CLAYEY SILT, some sand, containing sand lenses<br>Soft<br>Brownish grey<br>Wet | 2 | SS | 3 | | | | | |
| 179.6<br>1.4 | SILTY CLAY, trace sand<br>Soft<br>Brown and grey<br>Moist | 3 | SS | 2 | | 0 | 3 | 53 | 44 |
| 178.6<br>2.4 | SAND, trace silt, trace clay<br>Compact<br>Brown<br>Moist | 4 | SS | 17 | | | | | |
| 177.3<br>3.7 | SILT, some sand, trace to some clay<br>Loose to compact<br>Brown<br>Wet | 5 | SS | 11 | | | | | |
| | | 6 | SS | 7 | | 0 | 18 | 75 | 7 |
| 175.5<br>5.5 | SAND, trace silt<br>Compact<br>Grey<br>Wet | 7 | SS | 11 | | | | | |
| 174.3<br>6.7 | SILT, some sand, trace clay<br>Loose<br>Grey<br>Wet | 8 | SS | 5 | Non-plastic | 0 | 15 | 83 | 2 |
| 172.8<br>8.2 | SILTY CLAY, containing sand lenses<br>Stiff<br>Grey<br>Moist | 9 | SS | 1 | | | | | |
| 170.9<br>10.1 | SILT, some sand, trace to some clay<br>Compact<br>Grey<br>Wet | 10 | TO | PH* | | | | | |
| | | 11 | SS | 14 | | 0 | 17 | 77 | 6 |
| 169.4<br>11.6 | SILTY CLAY, trace sand<br>Soft<br>Brownish grey<br>Moist | 12A | SS | 3 | | | | | |
| 168.8<br>12.2 | SILT, trace sand, trace clay<br>Grey<br>Wet | 12B | | | | | | | |
| 168.2<br>12.8 | CLAY<br>Stiff<br>Brown<br>Wet | 13 | SS | 3 | | | | | |
| | Becoming grey at a depth of 14.3 m | | | | | | | | |



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+ ³, × ³: Numbers refer to Sensitivity

○ ³% STRAIN AT FAILURE

# RECORD OF BOREHOLE No B202-02

SHEET 2 OF 4

**METRIC**

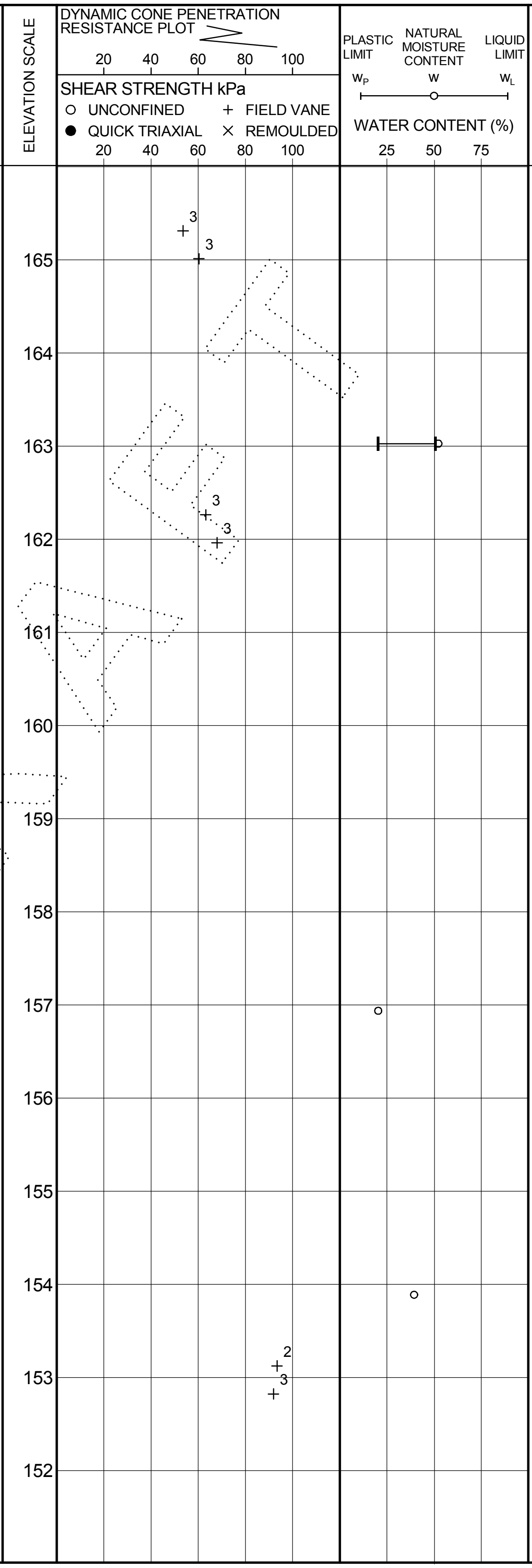
PROJECT 09-1111-6014
W.P. 5139-08-01
LOCATION N 5074821.3 ;E 225178.6
ORIGINATED BY MR
DIST HWY 69
BOREHOLE TYPE 213 mm O.D. Cont. Flight Hollow Stem Auger Augers, HW Casing, Wash Boring
COMPILED BY MAS
DATUM Geodetic
DATE February 10,14,15 and 16, 2011
CHECKED BY TVA

## SOIL PROFILE

| ELEV<br>DEPTH | DESCRIPTION |
|---|---|
| | --- CONTINUED FROM PREVIOUS PAGE --- |
| | CLAY<br>Stiff<br>Brown<br>Wet |
| 161.2<br>19.8 | SILT, trace sand, trace clay<br>Loose<br>Grey<br>Wet |
| 157.8<br>23.2 | Silty SAND<br>Dense<br>Grey<br>Wet |
| 155.4<br>25.6 | CLAY<br>Stiff to very stiff<br>Grey<br>Moist |

## SAMPLES

| NUMBER | TYPE | "N" VALUES | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR | SA | SI | CL |
|---|---|---|---|---|---|---|
| 14 | TO | PH | | | | |
| 15 | SS | 2 | 0 | 1 | 44 | 55 |
| 16 | SS | 9 | | | | |
| 17 | SS | 45 | | | | |
| 18 | SS | 9 | | | | |



UNIT WEIGHT γ kN/m³

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

Golder Associates

# RECORD OF BOREHOLE No B202-02

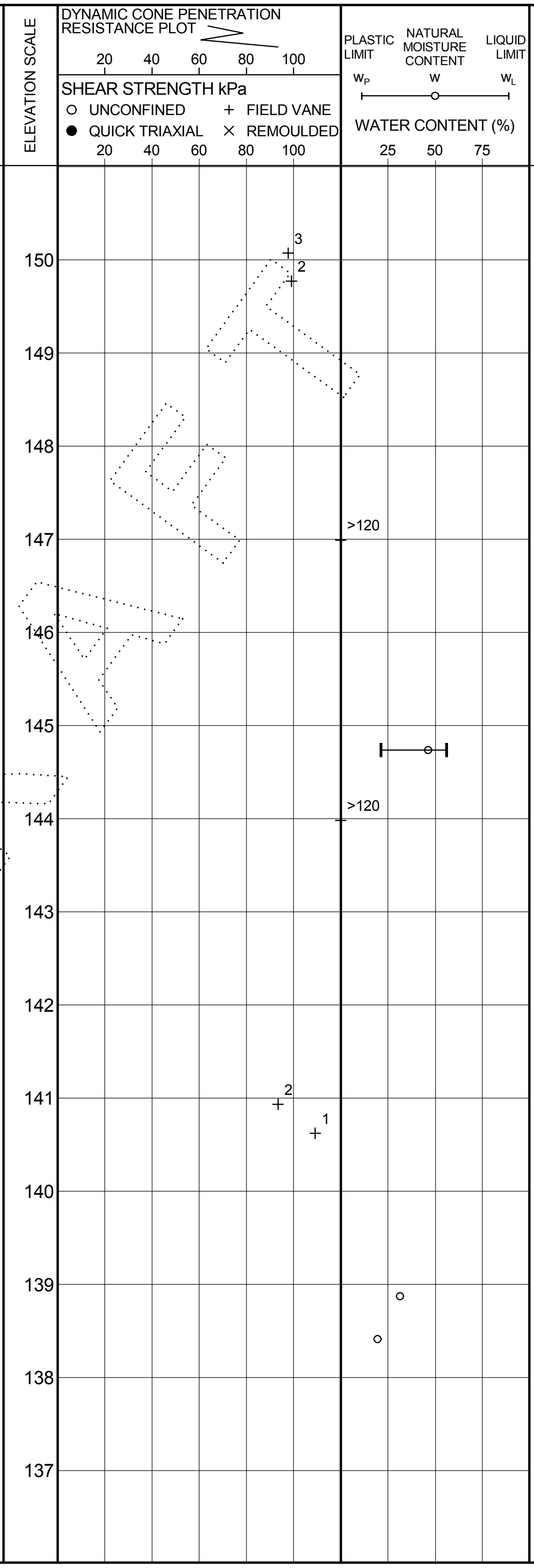
SHEET 3 OF 4

**METRIC**

PROJECT 09-1111-6014
W.P. 5139-08-01
LOCATION N 5074821.3 ;E 225178.6
ORIGINATED BY MR
DIST
HWY 69
BOREHOLE TYPE 213 mm O.D. Cont. Flight Hollow Stem Auger Augers, HW Casing, Wash Boring
COMPILED BY MAS
DATUM Geodetic
DATE February 10,14,15 and 16, 2011
CHECKED BY TVA

--- CONTINUED FROM PREVIOUS PAGE ---

| SOIL PROFILE | | | SAMPLES | | | GROUND WATER CONDITIONS | UNIT WEIGHT γ kN/m³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| ELEV<br>DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | | | GR | SA | SI | CL |
| | CLAY<br>Stiff to very stiff<br>Grey<br>Moist | ▨ | 19 | SS | 9 | | | | | | |
| | | | 20 | TO | PH | | | | | | |
| | | | 21 | SS | 12 | | | 0 | 1 | 34 | 65 |
| | | | 22 | SS | 10 | | | | | | |
| | | | 23A<br>23B<br>23C | SS | 26 | | | | | | |
| 138.5<br>42.5 | SILT, trace sand<br>Grey<br>Wet | | | | | | | | | | |
| 137.1<br>43.9 | SAND<br>Very dense<br>Grey<br>Wet | | | | | | | | | | |



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

# RECORD OF BOREHOLE No B202-02

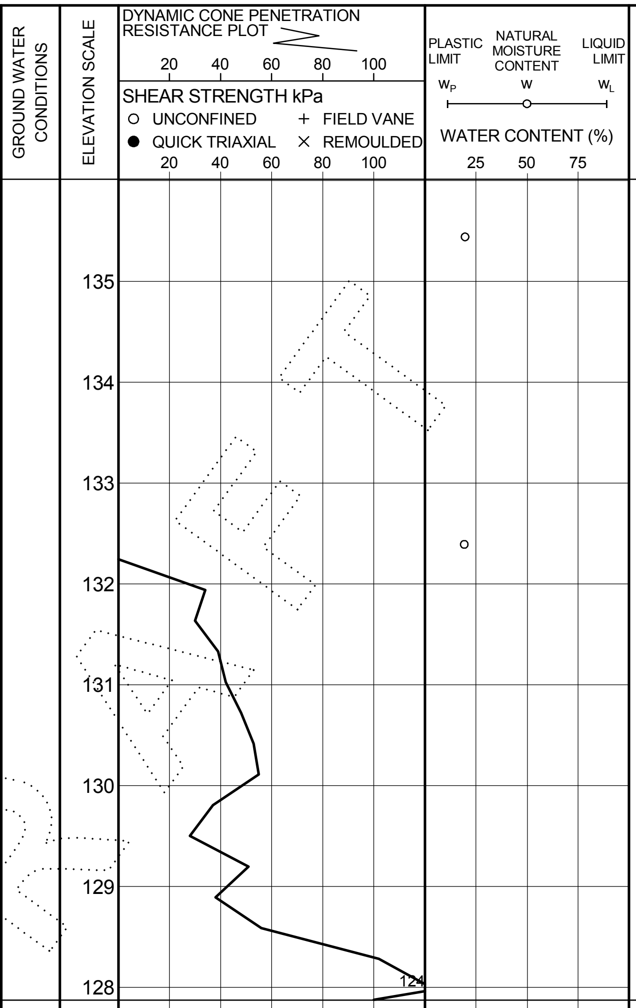
SHEET 4 OF 4

**METRIC**

PROJECT 09-1111-6014
W.P. 5139-08-01
DIST
HWY 69
DATUM Geodetic
LOCATION N 5074821.3 ;E 225178.6
BOREHOLE TYPE 213 mm O.D. Cont. Flight Hollow Stem Auger Augers, HW Casing, Wash Boring
DATE February 10,14,15 and 16, 2011
ORIGINATED BY MR
COMPILED BY MAS
CHECKED BY TVA

--- CONTINUED FROM PREVIOUS PAGE ---

| ELEV<br>DEPTH | DESCRIPTION | SAMPLE NUMBER | TYPE | "N" VALUES | GROUND WATER CONDITIONS | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
|---|---|---|---|---|---|---|
|  |  | 24A<br>24B | SS | 52 |  |  |
| 135.6<br>45.4 | Silty SAND<br>Very dense<br>Grey<br>Wet |  |  |  |  |  |
| 134.1<br>46.9 | SAND<br>Dense<br>Grey<br>Wet |  |  |  |  |  |
| 132.5 | SILT, trace to some sand<br>Dense<br>Grey<br>Wet | 25A<br>25B | SS | 42 |  |  |
| 132.2<br>48.8 | END OF BOREHOLE<br>Dynamic Cone Penetration Test (DCPT) |  |  |  |  |  |
| 127.9<br>53.1 | END OF DCPT<br>Refusal to Further Penetration (100 Blows / 0.1 m) |  |  |  |  |  |



NOTES:

* Unable to recover a Shelby tube sample between depths of 10.2 m and 10.7 m below ground surface (Elev. 170.8 m and 170.3 m).

1. Water level in open borehole rose to 1.0 m above ground surface (Elev. 182.0 m) at a depth of 45.1 m below ground surface (Elev. 135.9 m) during drilling - Artesian Condition.

2. A Dynamic Cone Penetration Test was carried out below a depth of 48.8 m; refusal encountered at a depth of 53.1 m below ground surface (Elev. 127.9 m).

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

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

**Golder Associates**

Foundation Design

## RECORD OF BOREHOLE No B202-03

SHEET 1 OF 2

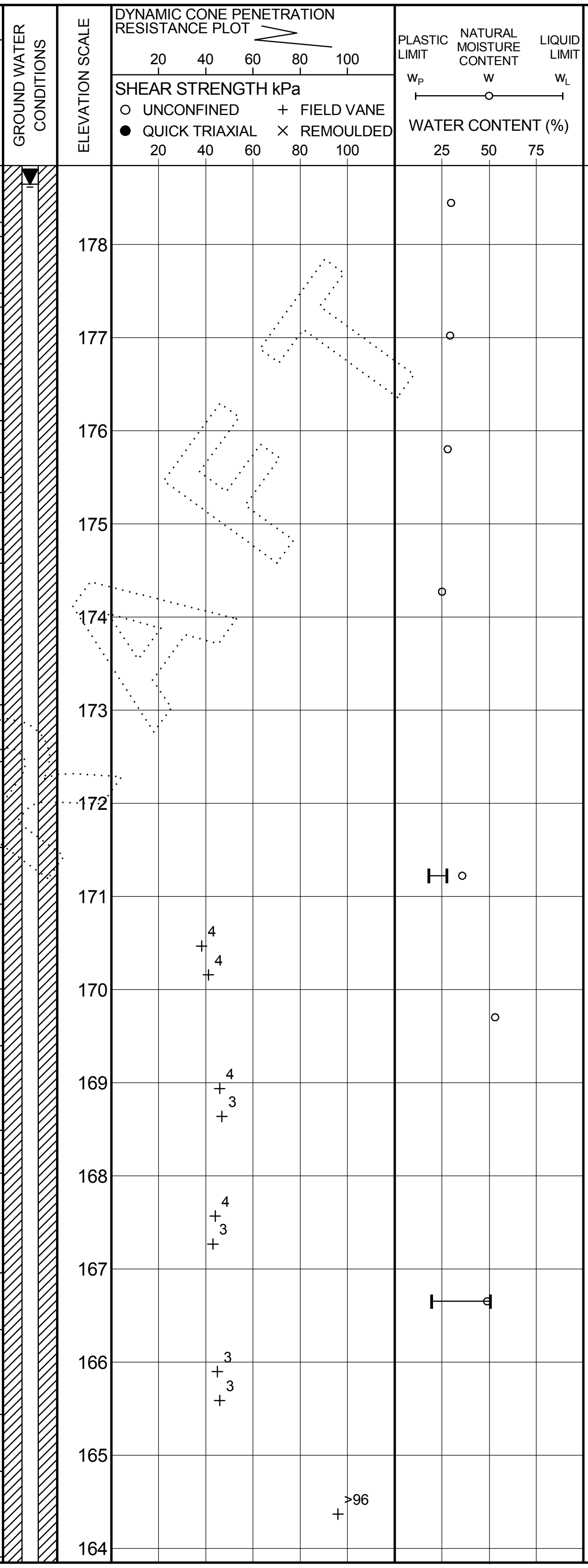
**METRIC**

PROJECT 09-1111-6014
W.P. 5139-08-01
DIST
HWY 69
DATUM Geodetic
LOCATION N 5074861.3 ;E 225162.8
BOREHOLE TYPE 127 mm O.D. Conti. Flight Hollow Stem Auger Augers, HW Casing, Wash Boring
DATE March 26, 2011
ORIGINATED BY MR
COMPILED BY MAS
CHECKED BY TVA

| ELEV<br>DEPTH | SOIL PROFILE – DESCRIPTION |
|---|---|
| 178.8 | GROUND SURFACE |
| 0.0 | TOPSOIL |
| 0.2 | Silty SAND, containing rootlets<br>Very loose<br>Brown<br>Wet |
| 178.1<br>0.7 | SAND, trace to some silt<br>Loose<br>Brown<br>Wet |
| 177.4<br>1.4 | Silty SAND, containing rootlets near surface<br>Very loose to loose<br>Brown to grey<br>Wet<br>Becoming grey below a depth of 2.1 m |
| 174.6<br>4.2 | SAND and SILT, trace clay<br>Very loose<br>Grey<br>Wet |
| 171.9<br>6.9 | CLAYEY SILT, trace sand<br>Firm<br>Grey<br>Moist |
| 170.4<br>8.4 | CLAY<br>Firm to stiff<br>Grey<br>Moist |

| SAMPLES NUMBER | TYPE | "N" VALUES | UNIT WEIGHT γ kN/m³ | GR | SA | SI | CL |
|---|---|---|---|---|---|---|---|
| 1A<br>1B | SS | 2 | | | | | |
| 2 | SS | 6 | | | | | |
| 3 | SS | 1 | | | | | |
| 4 | SS | 5 | | | | | |
| 5 | SS | 3 | | | | | |
| 6 | SS | 3 | | | | | |
| 7 | SS | 4 | | 0 | 31 | 66 | 3 |
| 8 | SS | 3 | | | | | |
| 9 | SS | 1 | | 0 | 2 | 73 | 25 |
| 10 | SS | 2 | | | | | |
| 11 | TO | PH | | | | | |
| 12 | SS | 1 | | 0 | 1 | 49 | 50 |
| 13 | SS | 1 | | | | | |

REMARKS & GRAIN SIZE DISTRIBUTION (%)



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

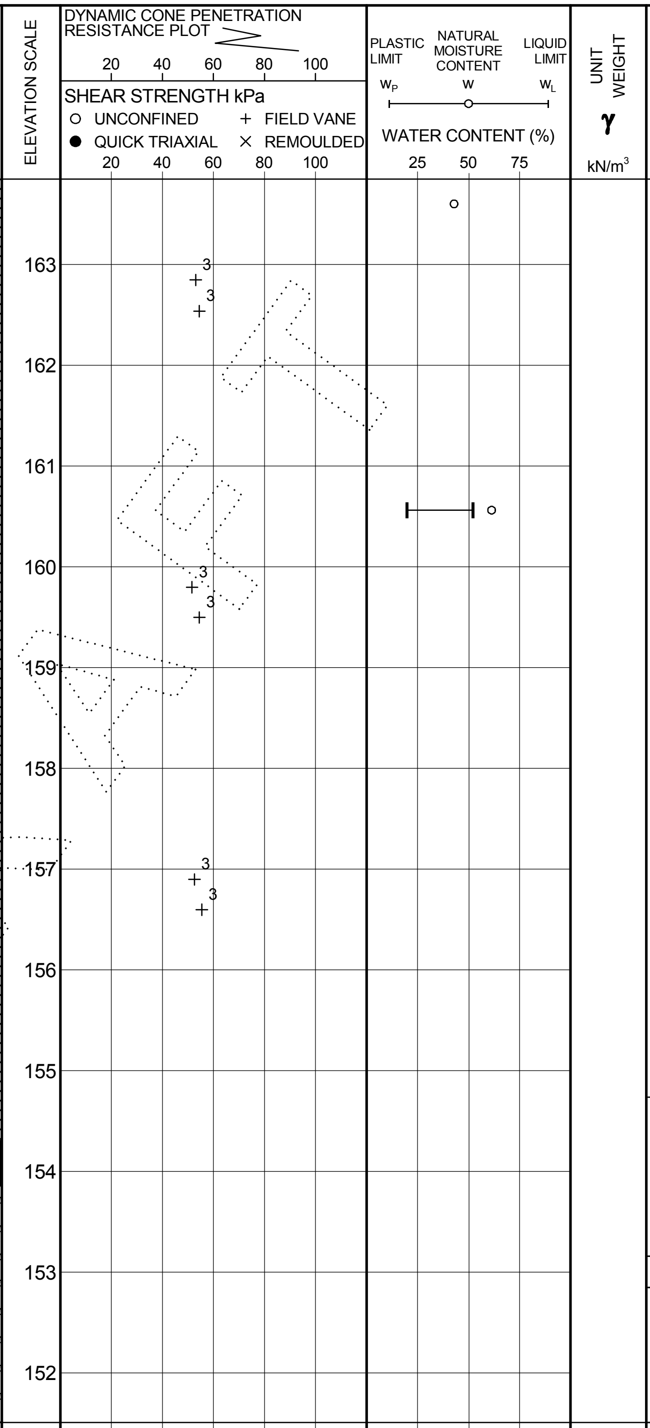
# RECORD OF BOREHOLE No B202-03

SHEET 2 OF 2

**METRIC**

PROJECT 09-1111-6014
W.P. 5139-08-01
LOCATION N 5074861.3 ;E 225162.8
ORIGINATED BY MR
DIST HWY 69
BOREHOLE TYPE 127 mm O.D. Conti. Flight Hollow Stem Auger Augers, HW Casing, Wash Boring
COMPILED BY MAS
DATUM Geodetic
DATE March 26, 2011
CHECKED BY TVA

| SOIL PROFILE | | | SAMPLES | | | GROUND WATER CONDITIONS | ELEVATION SCALE | REMARKS & GRAIN SIZE DISTRIBUTION (%) |
|---|---|---|---|---|---|---|---|---|
| ELEV DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | | | GR SA SI CL |
| | --- CONTINUED FROM PREVIOUS PAGE --- | | | | | | | |
| | CLAY<br>Firm to stiff<br>Grey<br>Moist | | 14 | SS | 3 | | 163 | |
| | | | 15 | SS | 3 | | 160 | 0 1 40 59 |
| | | | 16 | TO | PH | | 157 | |
| 154.7 | | | 17 | SS | 25/.02 | | 155 | |
| 24.1 | Granite Gneiss (BEDROCK)<br>Bedrock cored from depths of 24.1 m to 27.3 m<br>For Bedrock coring details refer to Record of Drillhole B203-03 | | 1 | RC | REC 100% | | 154 | RQD = 75% |
| | | | 2 | RC | REC 100% | | 153 | RQD = 48% |
| | | | 3 | RC | REC 100% | | 152 | RQD = 84% |
| 151.5<br>27.3 | END OF BOREHOLE<br>NOTE:<br>1. Water level measurement in Piezometer:<br>Date Depth (m) Elev. (m)<br>26/03/11 0.2 178.6 | | | | | | | |



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

PROJECT: 09-1111-6014

# RECORD OF DRILLHOLE: B202-03

SHEET 1 OF 1

LOCATION: N 5074861.3 ;E 225162.8

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: March 26, 2011

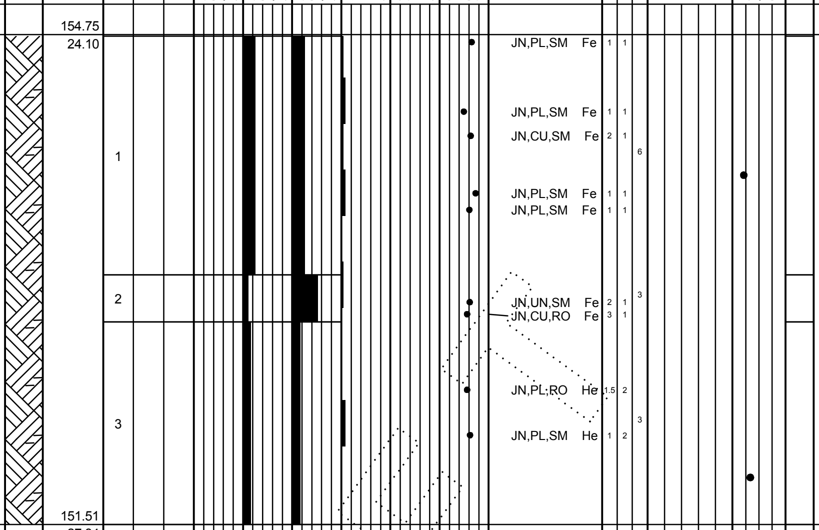
DRILL RIG: D25 Bombardier

DRILLING CONTRACTOR: Walker Drilling

DATUM: Geodetic

| | | | | |
|---|---|---|---|---|
| JN - Joint | BD - Bedding | PL - Planar | PO - Polished | MB - Mechanical Break |
| FLT - Fault | FO - Foliation | CU - Curved | K - Slickensided | BR - Broken Rock |
| SH - Shear | CO - Contact | UN - Undulating | SM - Smooth | **NOTE:** For additional abbreviations refer to list of abbreviations & symbols. |
| VN - Vein | OR - Orthogonal | ST - Stepped | RO - Rough | |
| CJ - Conjugate | CL - Cleavage | IR - Irregular | VR - Very Rough | |

| DEPTH SCALE METRES | DRILLING RECORD | DESCRIPTION | ELEV. DEPTH (m) | RUN No. | TYPE AND SURFACE DESCRIPTION | Jr | Ja | Jn | NOTES |
|---|---|---|---|---|---|---|---|---|---|
| | | Continued from Record of Borehole B202-03 | 154.75 | | | | | | |
| | NW Casing<br>March 26, 2011 | GRANITE GNEISS<br>Fresh, foliated, medium crystalline, slightly porous, medium strong to strong, pink, grey and black | 24.10 | 1 | JN,PL,SM Fe | 1 | 1 | | |
| | | | | | JN,PL,SM Fe | 1 | 1 | | |
| | | | | | JN,CU,SM Fe | 2 | 1 | 6 | |
| 25 | | | | | JN,PL,SM Fe | 1 | 1 | | |
| | | | | | JN,PL,SM Fe | 1 | 1 | | |
| | NQRC<br>March 26, 2011 | | | 2 | JN,UN,SM Fe | 2 | 1 | 3 | |
| 26 | | | | | JN,CU,RO Fe | 3 | 1 | | |
| | | | | 3 | JN,PL,RO He | 1.5 | 2 | | |
| | | | | | JN,PL,SM He | 1 | 2 | 3 | |
| 27 | | | 151.51 | | | | | | (Axial)<br>UC = 69 MPa |
| | | END OF DRILLHOLE | 27.34 | | | | | | |
| 28 | | | | | | | | | |
| 29 | | | | | | | | | |
| 30 | | | | | | | | | |
| 31 | | | | | | | | | |
| 32 | | | | | | | | | |
| 33 | | | | | | | | | |
| 34 | | | | | | | | | |



DEPTH SCALE
1 : 50

Golder Associates

LOGGED: MR
CHECKED: MAS/TVA

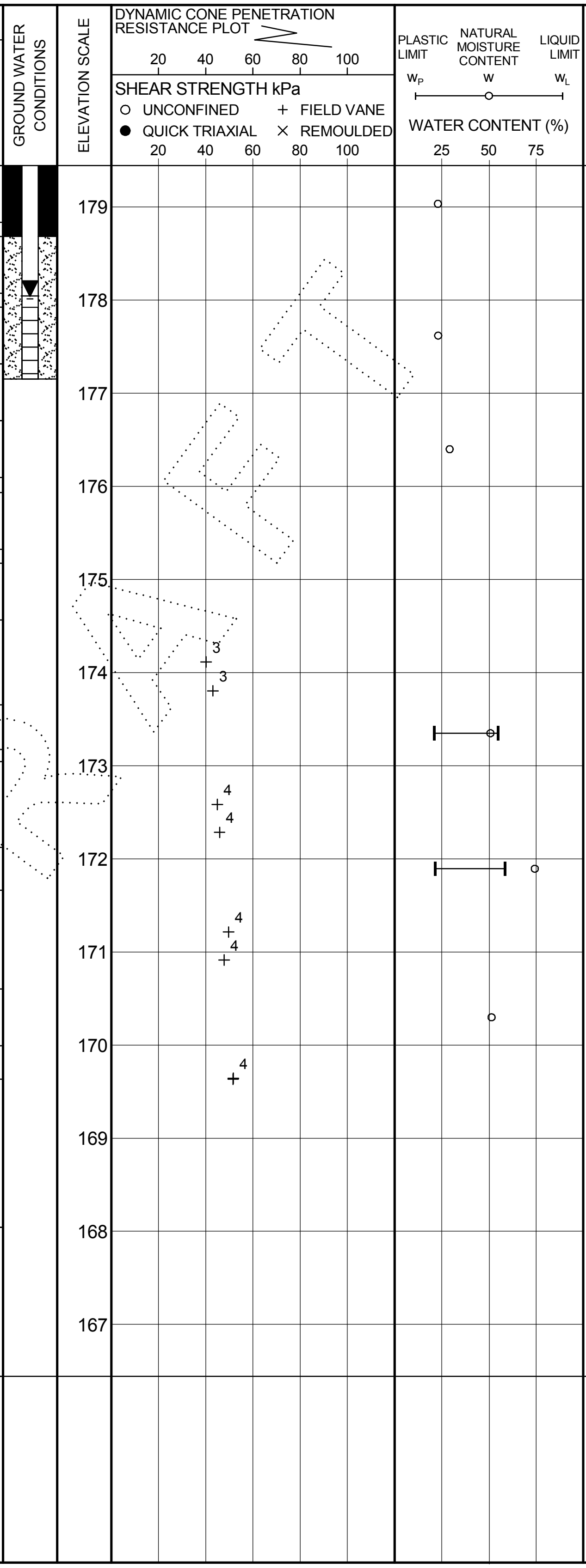
# RECORD OF BOREHOLE No B202-04

SHEET 1 OF 2

**METRIC**

PROJECT 09-1111-6014
W.P. 5139-08-01
LOCATION N 5074872.5 ;E 225158.4
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 27, 2011
CHECKED BY TVA

| SOIL PROFILE | | SAMPLES | | | UNIT WEIGHT γ kN/m³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) | | | |
|---|---|---|---|---|---|---|---|---|---|
| ELEV DEPTH | DESCRIPTION | NUMBER | TYPE | "N" VALUES | | GR | SA | SI | CL |
| 179.4 | GROUND SURFACE | | | | | | | | |
| 0.0 | TOPSOIL | 1A | SS | 4 | | | | | |
| 0.2 | Silty SAND<br>Loose<br>Brown<br>Wet | 1B | | | | | | | |
| | | 2 | SS | 5 | | | | | |
| 178.0<br>1.4 | SAND, trace gravel<br>Loose<br>Brown<br>Wet | 3 | SS | 4 | | | | | |
| | | 4 | SS | 5 | | | | | |
| 176.7<br>2.7 | SILT, trace to some clay, trace sand, containing silty clay lenses<br>Very loose<br>Grey<br>Wet | 5 | SS | WH | | 0 | 1 | 91 | 8 |
| | | 6 | SS | WH | | | | | |
| 175.2<br>4.2 | CLAY<br>Firm<br>Grey<br>Moist | 7 | SS | WH | | | | | |
| | | 8 | SS | 1 | | 0 | 1 | 45 | 54 |
| | | 9 | TO | PH | 16.5 | | | | |
| | | 10 | SS | 2 | | | | | |
| 169.6<br>9.8 | Granite Gneiss (BEDROCK)<br>Bedrock cored from depths of 9.8 m to 13.0 m<br>For bedrock coring details refer to Record of Drillhole B202-04 | 1 | RC | REC 100% | | RQD = 85% | | | |
| | | 2 | RC | REC 100% | | RQD = 100% | | | |
| 166.4<br>13.0 | END OF BOREHOLE | | | | | | | | |



Continued Next Page

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

Golder Associates

# RECORD OF BOREHOLE No B202-04

SHEET 2 OF 2

METRIC

PROJECT 09-1111-6014
W.P. 5139-08-01
LOCATION N 5074872.5 ;E 225158.4
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 27, 2011
CHECKED BY TVA

| SOIL PROFILE | | | SAMPLES | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| ELEV<br>DEPTH | DESCRIPTION<br>--- *CONTINUED FROM PREVIOUS PAGE* --- | STRAT PLOT | NUMBER | TYPE | "N" VALUES | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT<br>20 40 60 80 100<br>SHEAR STRENGTH kPa<br>○ UNCONFINED + FIELD VANE<br>● QUICK TRIAXIAL × REMOULDED<br>20 40 60 80 100 | PLASTIC LIMIT $W_P$ NATURAL MOISTURE CONTENT $W$ LIQUID LIMIT $W_L$<br>WATER CONTENT (%)<br>25 50 75 | UNIT WEIGHT γ<br>$kN/m^3$ | REMARKS & GRAIN SIZE DISTRIBUTION (%)<br>GR SA SI CL |
| | NOTE:<br>1. An additional borehole was drilled 1.0 m North of Borehole B202-04 to carry out installation of piezometer to a depth of 2.3 m below ground surface (Elev. 177.1 m).<br>2. Water level measurement in Piezometer:<br>Date Depth (m) Elev. (m)<br>10/03/12 1.4 178.1 | | | | | | | | | | |

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

$○^{3\%}$ STRAIN AT FAILURE

PROJECT: 09-1111-6014

# RECORD OF DRILLHOLE: B202-04

SHEET 1 OF 1

LOCATION: N 5074872.5 ;E 225158.4

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: March 27, 2011

DRILL RIG: D25 Bombardier

DRILLING CONTRACTOR: Walker Drilling

DATUM: Geodetic

| | | | | |
|---|---|---|---|---|
| JN - Joint | BD - Bedding | PL - Planar | PO - Polished | MB - Mechanical Break |
| FLT - Fault | FO - Foliation | CU - Curved | K - Slickensided | BR - Broken Rock |
| SH - Shear | CO - Contact | UN - Undulating | SM - Smooth | **NOTE:** For additional abbreviations refer to list of abbreviations & symbols. |
| VN - Vein | OR - Orthogonal | ST - Stepped | RO - Rough | |
| CJ - Conjugate | CL - Cleavage | IR - Irregular | VR - Very Rough | |

| DEPTH SCALE METRES | DRILLING RECORD | DESCRIPTION | SYMBOLIC LOG | ELEV. / DEPTH (m) | RUN No. | COLOUR / % RETURN | FLUSH | RECOVERY TOTAL CORE % (80 60 40 20) | RECOVERY SOLID CORE % (80 60 40 20) | R.Q.D. % (80 60 40 20) | FRACT. INDEX PER 0.3 m (5 10 15 20) | DISCONTINUITY DATA B Angle (0 90 180 270) | DIP w.r.t. CORE AXIS (0 30 60 90) | TYPE AND SURFACE DESCRIPTION | Jr | Ja | Jn | HYDRAULIC CONDUCTIVITY K, cm/sec ($10^{-6}$ $10^{-5}$ $10^{-4}$ $10^{-3}$) | Diametral Point Load Index (MPa) (2 4 6) | RMC -Q' AVG. | NOTES |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Continued from Record of Borehole B202-04 | | 169.63 | | | | | | | | | | | | | | | | | |
| | NW Casing<br>March 27, 2011 | GRANITE GNEISS<br>Fresh, foliated, medium crystalline, slightly porous, very strong, pink, grey and black | | 9.81 | | | | | | | | | | | | | | | | | |
| 10 | | | | | | | | | | | | | | JN,PL,SM He | 1 | 1 | | | | | |
| | | Near vertical fracture with silt infilling between depths of 10.7 m and 11.3 m | | | | | | | | | | | | | | | | | | | 10.7 MPa<br>(Axial) |
| | | | | | 1 | | | | | | | | | JN,PL,RO | 1.5 | 1 | | | | | |
| 11 | NQRC<br>March 27, 2011 | | | | | | | | | | | | | JN,PL,SM Fe | 1 | 1 | | | | | |
| | | | | | | | | | | | | | | JN,PL,SM Cl | 1 | 2 | | | | | |
| | | | | | | | | | | | | | | JN,W,SM Cl | 1.5 | 2 | | | | | |
| | | | | | | | | | | | | | | JN,PL,SM Fe | 1 | 2 | | | | | |
| | | | | | | | | | | | | | | JN,PL,RO Fe | 1.5 | 1 | | | | | |
| 12 | | | | | 2 | | | | | | | | | | | | | | | | 10.6 MPa<br>UC = 193 MPa |
| | | | | 166.44 | | | | | | | | | | | | | | | | | |
| 13 | | END OF DRILLHOLE<br>Note:<br>1. Near vertical fracture with silty build up was observed within the recovered bedrock between depths of 10.7 m and 11.3 m, indicating possible water flow into bedrock. | | 13.00 | | | | | | | | | | | | | | | | | |
| 14 | | | | | | | | | | | | | | | | | | | | | |
| 15 | | | | | | | | | | | | | | | | | | | | | |
| 16 | | | | | | | | | | | | | | | | | | | | | |
| 17 | | | | | | | | | | | | | | | | | | | | | |
| 18 | | | | | | | | | | | | | | | | | | | | | |
| 19 | | | | | | | | | | | | | | | | | | | | | |

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: MR

CHECKED: MAS/TVA

Golder Associates

# RECORD OF BOREHOLE No B202-05

SHEET 1 OF 1

**METRIC**

PROJECT 09-1111-6014

W.P. 5139-08-01

LOCATION N 5074886.4 ;E 225152.9

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 28, 2011

CHECKED BY TVA

| ELEV<br>DEPTH | SOIL PROFILE: DESCRIPTION | STRAT PLOT | SAMPLES: NUMBER | SAMPLES: TYPE | SAMPLES: "N" VALUES | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100 | PLASTIC LIMIT $W_P$ / NATURAL MOISTURE CONTENT W / LIQUID LIMIT $W_L$ — WATER CONTENT (%) 25 50 75 | UNIT WEIGHT γ kN/m³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 181.3 | GROUND SURFACE | | | | | | | | | | |
| 0.0 | TOPSOIL | | 1A | | | | | | | | |
| 0.2 | CLAYEY SILT with SAND, containing wood fragments<br>Firm<br>Brown<br>Moist | | 1B | SS | 5 | | 181 | | | | |
| 180.6<br>0.7 | SILTY CLAY<br>Stiff<br>Brown and grey<br>Moist | | 2 | SS | 8 | | 180 | | ○ | | |
| 179.9<br>1.4 | Sandy SILT<br>Compact<br>Grey<br>Wet | | 3 | SS | 28 | ∇ | | | | | |
| 179.2<br>2.1 | CLAYEY SILT, trace sand, containing silt interlayers<br>Firm<br>Grey<br>Moist | | 4 | SS | 2 | | 179 | | ├─┤○ | | 0 2 75 23 |
| | | | | | | | 178 | +5 | | | |
| | | | 5 | SS | 1 | | 177 | +5 | ○ | | |
| 176.7<br>4.6 | Granite Gneiss (BEDROCK)<br>Bedrock cored from a depth of 4.6 m to 6.2 m<br>For bedrock coring details refer to Record of Drillhole B202-05 | | 1 | RC | REC 100% | | 176 | | | | RQD = 100% |
| 175.1<br>6.2 | END OF BOREHOLE<br>NOTE:<br>1. Water level in open borehole at a depth of 1.8 m (Elev. 179.5 m) upon completion of drilling. | | | | | | | | | | |

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

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

GTA-MTO 001 09-1111-6014.GPJ GAL-GTA.GDT 2/6/13 SAC/DD

PROJECT: 09-1111-6014

# RECORD OF DRILLHOLE: B202-05

SHEET 1 OF 1

LOCATION: N 5074886.4 ;E 225152.9

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: March 28, 2011

DRILL RIG: D25 Bombardier

DRILLING CONTRACTOR: Walker Drilling

DATUM: Geodetic

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.

| DEPTH SCALE METRES | DRILLING RECORD | DESCRIPTION | SYMBOLIC LOG | ELEV. DEPTH (m) | RUN No. | COLOUR % RETURN FLUSH | RECOVERY TOTAL CORE % 80 60 40 20 | RECOVERY SOLID CORE % 80 60 40 20 | R.Q.D. % 80 60 40 20 | FRACT. INDEX PER 0.3 m 5 10 15 20 | DISCONTINUITY DATA B Angle 0 90 180 270 | DISCONTINUITY DATA DIP w.r.t. CORE AXIS 0 30 60 90 | DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION | Jr | Ja | Jn | HYDRAULIC CONDUCTIVITY K, cm/sec $10^{-6}$ $10^{-5}$ $10^{-4}$ $10^{-3}$ | Diametral Point Load Index (MPa) 2 4 6 | RMC -Q' AVG. | NOTES |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Continued from Record of Borehole B202-05 | | 176.74 | | | | | | | | | | | | | | | | |
| 5 | NW Casing March 28, 2011 | GRANITE GNEISS<br>Fresh, foliated, medium crystalline, slightly porous, strong, pink, grey, and black | | 4.57 | | | | | | | | ● | JN,PL,RO | 1.5 | 1 | | | | | |
| | NQRC March 28, 2011 | | | | 1 | | | | | | | | | | | 1 | | ● | | (Axial) |
| 6 | | | | 175.12 | | | | | | | | | | | | | | | | |
| | | END OF DRILLHOLE | | 6.19 | | | | | | | | | | | | | | | | |
| 7 | | | | | | | | | | | | | | | | | | | | |
| 8 | | | | | | | | | | | | | | | | | | | | |
| 9 | | | | | | | | | | | | | | | | | | | | |
| 10 | | | | | | | | | | | | | | | | | | | | |
| 11 | | | | | | | | | | | | | | | | | | | | |
| 12 | | | | | | | | | | | | | | | | | | | | |
| 13 | | | | | | | | | | | | | | | | | | | | |
| 14 | | | | | | | | | | | | | | | | | | | | |

DEPTH SCALE
1 : 50

Golder Associates

LOGGED: MR
CHECKED: MAS/TVA

# RECORD OF BOREHOLE No B202-06

SHEET 1 OF 1

**METRIC**

PROJECT 09-1111-6014
W.P. 5139-08-01
DIST
HWY 69
DATUM Geodetic
LOCATION N 5074898.6 ;E 225140.9
BOREHOLE TYPE Portable Equipment
DATE February 26, 2011
ORIGINATED BY TT
COMPILED BY MAS
CHECKED BY TVA

| SOIL PROFILE | | | SAMPLES | | | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100<br>SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100 | PLASTIC LIMIT $w_P$ NATURAL MOISTURE CONTENT w LIQUID LIMIT $w_L$<br>WATER CONTENT (%) 25 50 75 | UNIT WEIGHT γ kN/m³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| ELEV DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | | | | | | GR | SA | SI | CL |
| 183.9 | GROUND SURFACE | | | | | | | | | | | | | |
| 0.0 | PEAT, trace sand (Amorphous)<br>Black<br>Wet | | 1A | SS | 10 | | | | ○ | | 24 | 57 | 16 | 3 |
| 0.2 | Gravelly SAND, some silt, trace clay, containing organics<br>Compact<br>Brown<br>Moist | | 1B | | | | | | | | | | | |
| 183.0<br>0.9 | END OF BOREHOLE<br>SPOON REFUSAL<br><br>NOTE:<br><br>1. Borehole dry upon completion of drilling. | | | | | | | | | | | | | |

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

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

# RECORD OF BOREHOLE No B202-07

SHEET 1 OF 1

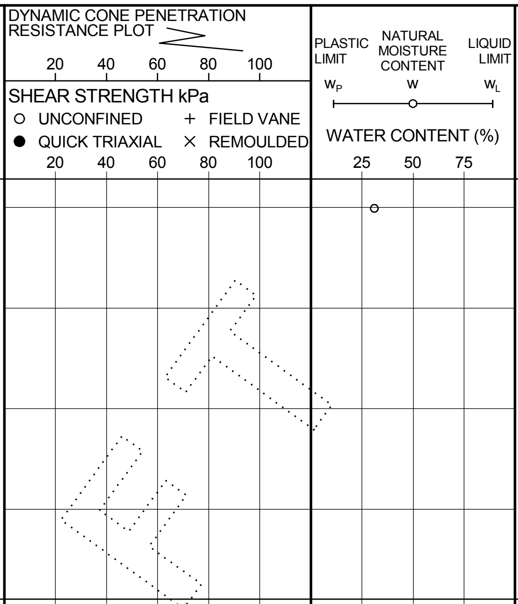
**METRIC**

PROJECT 09-1111-6014
W.P. 5139-08-01
DIST ______ HWY 69
DATUM Geodetic

LOCATION N 5074902.4 ;E 225139.5
BOREHOLE TYPE Portable Equipment, BW Casing
DATE February 27, 2011

ORIGINATED BY TT
COMPILED BY MAS
CHECKED BY TVA

| ELEV DEPTH | SOIL PROFILE DESCRIPTION | STRAT PLOT | SAMPLES NUMBER | SAMPLES TYPE | "N" VALUES | GROUND WATER CONDITIONS | ELEVATION SCALE | UNIT WEIGHT γ kN/m³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
|---|---|---|---|---|---|---|---|---|---|
| 184.3 | GROUND SURFACE | | | | | | | | |
| 0.0<br>183.7 | PEAT (Amorphous)<br>Hard (Frozen)<br>Black | | 1 | SS | 39 | ▽ | 184 | | |
| 0.6<br>183.2 | Cobbles and Boulders | | 1 | RC | REC 50% | | | | |
| 1.1 | Granite Gneiss (BEDROCK)<br>Bedrock cored from depths of 1.1 m to 4.2 m<br>For bedrock coring details refer to Record of Drillhole B202-07 | | 2 | RC | REC 100% | | 183 | | RQD = 100% |
| | | | 3 | RC | REC 100% | | 182 | | RQD = 100% |
| | | | 4 | RC | REC 100% | | 181 | | RQD = 100% |
| 180.1 | | | 5 | RC | REC 100% | | | | RQD = 100% |
| 4.2 | END OF BOREHOLE<br>NOTE:<br>1. Water level in open borehole at a depth of 0.4 m below ground surface (Elev. 183.9 m) upon completion of drilling. | | | | | | | | |



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

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

PROJECT: 09-1111-6014

# RECORD OF DRILLHOLE: B202-07

SHEET 1 OF 1

LOCATION: N 5074902.4 ;E 225139.5

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: February 27, 2011
DRILL RIG: Portable Equipment
DRILLING CONTRACTOR: OGS Inc

DATUM: Geodetic

| | | | | |
|---|---|---|---|---|
| JN - Joint | BD- Bedding | PL - Planar | PO- Polished | MB - Mechanical Break |
| FLT - Fault | FO- Foliation | CU- Curved | K - Slickensided | BR - Broken Rock |
| SH - Shear | CO- Contact | UN- Undulating | SM- Smooth | **NOTE:** For additional abbreviations refer to list of abbreviations & symbols. |
| VN - Vein | OR- Orthogonal | ST - Stepped | RO- Rough | |
| CJ - Conjugate | CL - Cleavage | IR - Irregular | VR- Very Rough | |

| DEPTH SCALE METRES | DRILLING RECORD | DESCRIPTION | SYMBOLIC LOG | ELEV. DEPTH (m) | RUN No. | COLOUR % RETURN FLUSH | RECOVERY TOTAL CORE % 80 60 40 20 | RECOVERY SOLID CORE % 80 60 40 20 | R.Q.D. % 80 60 40 20 | FRACT. INDEX PER 0.3 m 5 10 15 20 | B Angle 0 90 180 270 | DIP w.r.t. CORE AXIS 0 30 60 90 | TYPE AND SURFACE DESCRIPTION | Jr | Ja | Jn | HYDRAULIC CONDUCTIVITY K, cm/sec $10^{-6}$ $10^{-5}$ $10^{-4}$ $10^{-3}$ | Diametral Point Load Index (MPa) 2 4 6 | RMC -Q' AVG. | NOTES |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Continued from Record of Borehole B202-07 | | 183.68 | | | | | | | | | | | | | | | | |
| | BQRC<br>February 27, 2011 | COBBLES AND BOULDERS | | 0.60 | | | | | | | | | | | | | | | | |
| 1 | | | | 183.15 | 1 | | | | | | | | | | | | | | | |
| | | GRANITE GNEISS<br>Fresh, foliated, medium to coarsely crystalline, slightly porous, strong to very strong, pink and black | | 1.13 | 2 | | | | | | | | | | | | | | | 9.5 MPa |
| 2 | | | | | 3 | | | | | | | | | | | | | | | (Axial) |
| 3 | | | | | 4 | | | | | | | | JN,PL,SM Fe | 1 | 2 | 3 | | | | |
| | | | | | | | | | | | | | JN,PL,RO | 1.5 | 1 | | | | | |
| | | | | | 5 | | | | | | | | JN,PL,RO | 1.5 | 1 | | | | | |
| | | | | | | | | | | | | | | | | 2 | | | | |
| 4 | | | | 180.11 | | | | | | | | | | | | | | | | |
| | | END OF DRILLHOLE | | 4.17 | | | | | | | | | | | | | | | | |
| 5 | | | | | | | | | | | | | | | | | | | | |
| 6 | | | | | | | | | | | | | | | | | | | | |
| 7 | | | | | | | | | | | | | | | | | | | | |
| 8 | | | | | | | | | | | | | | | | | | | | |
| 9 | | | | | | | | | | | | | | | | | | | | |
| 10 | | | | | | | | | | | | | | | | | | | | |

DEPTH SCALE
1 : 50

Golder Associates

LOGGED: TT
CHECKED: AM/TVA

Golder Associates

# RECORD OF BOREHOLE No B202-08

SHEET 1 OF 1

**METRIC**

| | | | |
|---|---|---|---|
| PROJECT 09-1111-6014 | | | |
| W.P. 5139-08-01 | LOCATION N 5074901.3 ;E 225146.9 | ORIGINATED BY TT | |
| DIST HWY 69 | BOREHOLE TYPE Portable Equipment, BW Casing and EW Casing | COMPILED BY MAS | |
| DATUM Geodetic | DATE February 24 to 26, 2011 | CHECKED BY TVA | |

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

PLASTIC LIMIT $W_P$ — NATURAL MOISTURE CONTENT $W$ — LIQUID LIMIT $W_L$

WATER CONTENT (%): 25 50 75

UNIT WEIGHT γ kN/m³

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

| ELEV DEPTH | DESCRIPTION | NUMBER | TYPE | "N" VALUES | ELEVATION SCALE | WATER CONTENT (%) | REMARKS |
|---|---|---|---|---|---|---|---|
| 184.2 | GROUND SURFACE | | | | | | |
| 0.0<br>183.9 | Peat (Amorphous) | 1 | CS | - | 184 | 520.2 | |
| 0.3<br>183.3 | Cobbles and Boulders | 1 | RC | REC 94% | | | |
| | CLAYEY SILT, some sand, trace gravel, containing rootlets<br>Brown<br>Moist | 2A | CS | - | | | |
| | | 2B | SS | 100/0.087 | | | |
| 1.2<br>182.3 | SAND, some gravel, trace silt<br>Brown<br>Moist | 2 | RC | REC 53% | 183 | | |
| | | 3 | SS | 100/0.1 | | | |
| 2.0 | Cobbles and Boulders<br>GRAVEL, some sand<br>Grey<br>Moist | 3 | RC | REC 59% | 182 | | |
| 181.3<br>2.9 | Cobbles and Boulders<br>Granite Gneiss (BEDROCK)<br><br>Bedrock cored from depths of 2.9 m to 6.6 m<br><br>For bedrock coring details refer to Record of Drillhole B202-08 | 4 | RC | REC 94% | 181 | | RQD = 0% |
| | | 5 | RC | REC 97% | 180 | | RQD = 68% |
| 177.6 | | 6 | RC | REC 95% | 179<br>178 | | RQD = 84% |
| 6.6 | END OF BOREHOLE | | | | | | |

NOTE:

1. Water level measurement in Piezometer:

| Date | Depth (m) | Elev. (m) |
|---|---|---|
| 28/02/11 | 0.8 | 183.4 |
| 31/03/11 | 0.7 | 183.5 |
| 03/10/12 | 0.8 | 183.4 |

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

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

PROJECT: 09-1111-6014

# RECORD OF DRILLHOLE: B202-08

SHEET 1 OF 1

LOCATION: N 5074901.3 ;E 225146.9

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: February 24 to 26, 2011

DRILL RIG: Portable Equipment

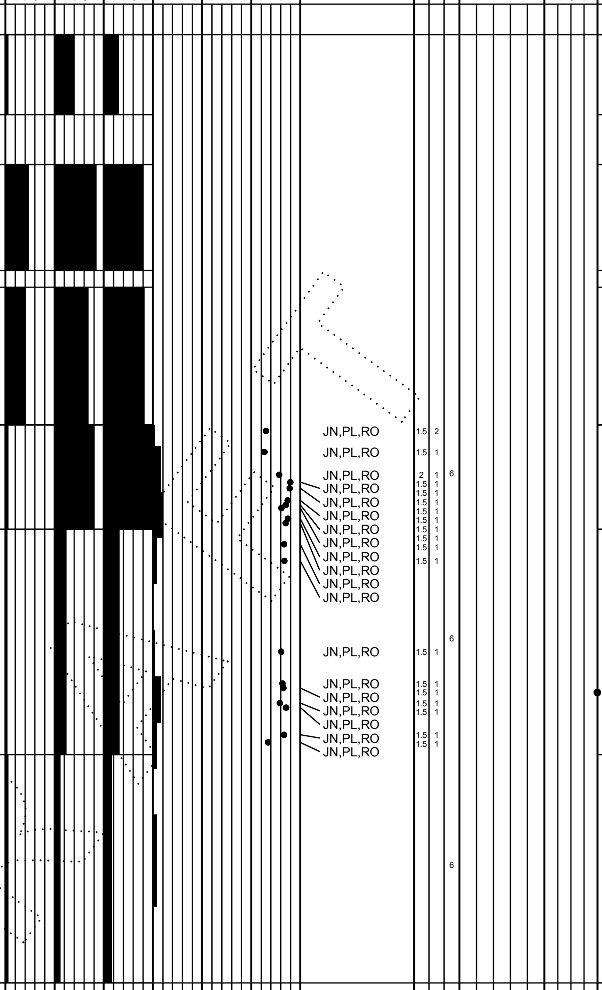
DRILLING CONTRACTOR: OGS Inc

DATUM: Geodetic

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.

| DEPTH SCALE METRES | DRILLING RECORD | DESCRIPTION | ELEV. / DEPTH (m) | RUN No. | NOTES |
|---|---|---|---|---|---|
| | | Continued from Record of Borehole B202-08 | 183.89 | | |
| | BQRC and EQRC<br>February 24 to 26, 2011 | COBBLES AND BOULDERS | 0.33 | 1 | |
| | | | 183.32<br>0.90 | | |
| 1 | | COBBLES AND BOULDERS | 183.03<br>1.19 | 2 | |
| 2 | | | 182.33<br>1.89 | | |
| | | COBBLES AND BOULDERS | 2.03 | 3 | |
| 3 | | GRANITE GNEISS<br>Slightly weathered, foliated, finely crystalline, slightly porous, strong, grey, pink and black | 181.31<br>2.91 | 4 | |
| 4 | | GRANITE GNEISS<br>Fresh, foliated, medium crystalline, slightly porous, strong to very strong, grey, pink and black | 180.91<br>3.31 | 5 | |
| 5 | | | | | 8.3 MPa (Axial) |
| 6 | | Mafic dyke between depths of 5.86 m and 5.94 m | | 6 | |
| 7 | | END OF DRILLHOLE | 177.62<br>6.60 | | |



| TYPE AND SURFACE DESCRIPTION | Jr | Ja | Jn |
|---|---|---|---|
| JN,PL,RO | 1.5 | 2 | |
| JN,PL,RO | 1.5 | 1 | |
| JN,PL,RO | 2 | 1 | 6 |
| JN,PL,RO | 1.5 | 1 | |
| JN,PL,RO | 1.5 | 1 | |
| JN,PL,RO | 1.5 | 1 | |
| JN,PL,RO | 1.5 | 1 | |
| JN,PL,RO | 1.5 | 1 | |
| JN,PL,RO | 1.5 | 1 | |
| JN,PL,RO | 1.5 | 1 | |
| JN,PL,RO | 1.5 | 1 | |
| JN,PL,RO | 1.5 | 1 | |
| | | | 6 |
| JN,PL,RO | 1.5 | 1 | |
| JN,PL,RO | 1.5 | 1 | |
| JN,PL,RO | 1.5 | 1 | |
| JN,PL,RO | 1.5 | 1 | |
| JN,PL,RO | 1.5 | 1 | |
| JN,PL,RO | 1.5 | 1 | |
| JN,PL,RO | 1.5 | 1 | |
| | | | 6 |

HYDRAULIC CONDUCTIVITY K, cm/sec $10^{-6}$ $10^{-5}$ $10^{-4}$ $10^{-3}$

Diametral Point Load Index (MPa) 2 4 6

RMC -Q' AVG.

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: TT

CHECKED: MAS/TVA

Golder Associates

# RECORD OF BOREHOLE No B202-09

SHEET 1 OF 1

**METRIC**

PROJECT 09-1111-6014
W.P. 5139-08-01
DIST ______ HWY 69
DATUM Geodetic

LOCATION N 5074900.3 ;E 225154.9
BOREHOLE TYPE Portable Equipment, BW Casing
DATE February 27, 2011

ORIGINATED BY TT
COMPILED BY MAS
CHECKED BY TVA

| SOIL PROFILE | | SAMPLES | | | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100 | PLASTIC LIMIT $W_P$ NATURAL MOISTURE CONTENT W LIQUID LIMIT $W_L$ / WATER CONTENT (%) 25 50 75 | UNIT WEIGHT γ kN/m³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
|---|---|---|---|---|---|---|---|---|---|---|
| ELEV DEPTH | DESCRIPTION | NUMBER | TYPE | "N" VALUES | | | | | | |
| 186.4 | GROUND SURFACE | | | | | | | | | |
| 0.0 | Peat (Amorphous) | 1 | CS | - | | | | 889.9 | | |
| 186.1<br>0.3 | Schist (BEDROCK)<br>Gneiss (BEDROCK) | 1 | RC | REC 99% | | 186 | | | | RQD = 62% |
| 185.1<br>1.3 | Schist (BEDROCK) | | | | | 185 | | | | |
| 184.6<br>1.8 | Granite Gneiss (BEDROCK)<br>Bedrock cored from depths of 0.3 m to 3.7 m<br>For bedrock coring details refer to Record of Drillhole B202-09 | 2 | RC | REC 100% | ▽ | 184 | | | | RQD = 93% |
| | | 3 | RC | REC 100% | | 183 | | | | RQD = 100% |
| 182.7<br>3.7 | END OF BOREHOLE<br>NOTE:<br>1. Water level in open borehole at a depth of 1.8 m below ground surface (Elev 184.6 m) upon completion of drilling | | | | | | | | | |

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

PROJECT: 09-1111-6014

# RECORD OF DRILLHOLE: B202-09

SHEET 1 OF 1

LOCATION: N 5074900.3 ;E 225154.9

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: February 27, 2011

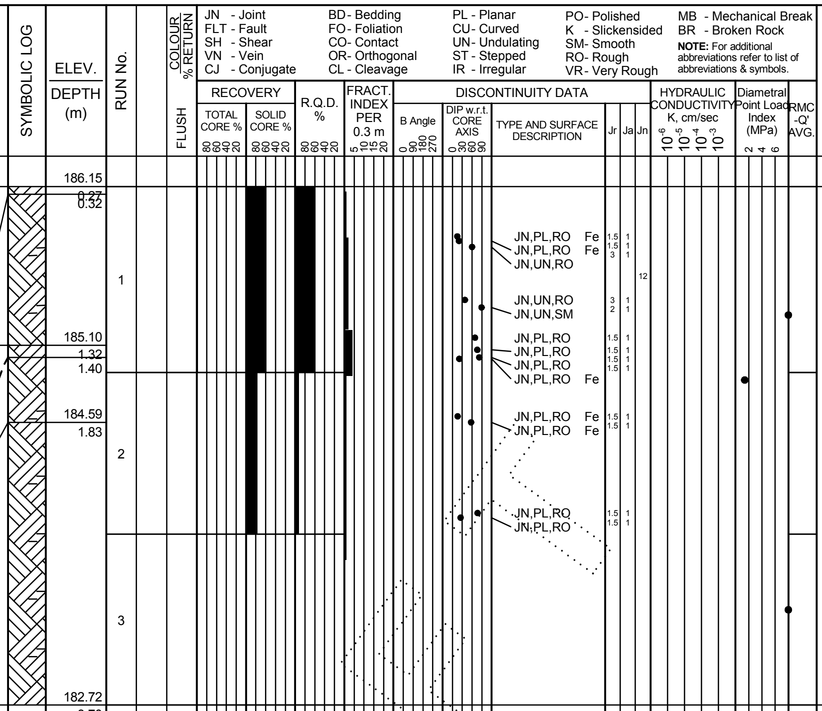
DRILL RIG: Portable Equipment

DRILLING CONTRACTOR: OGS Inc

DATUM: Geodetic

| | | | | |
|---|---|---|---|---|
| JN - Joint | BD- Bedding | PL - Planar | PO- Polished | MB - Mechanical Break |
| FLT - Fault | FO- Foliation | CU- Curved | K - Slickensided | BR - Broken Rock |
| SH - Shear | CO- Contact | UN- Undulating | SM- Smooth | **NOTE:** For additional abbreviations refer to list of abbreviations & symbols. |
| VN - Vein | OR- Orthogonal | ST - Stepped | RO- Rough | |
| CJ - Conjugate | CL - Cleavage | IR - Irregular | VR- Very Rough | |

| DEPTH SCALE METRES | DRILLING RECORD | DESCRIPTION | ELEV. / DEPTH (m) | RUN No. | NOTES |
|---|---|---|---|---|---|
| | | Continued from Record of Borehole B202-09 | 186.15 | | |
| | BQRC<br>February 27, 2011 | **SCHIST**<br>Fresh, coarsely crystalline, slightly porous, medium strong, brown and black | 0.27<br>0.32 | 1 | |
| 1 | | **GNEISS**<br>Fresh, medium to coarsely crystalline, slightly porous, strong to very strong, containing pegmatitic crystal, pink, grey and black | 185.10 | | 8.1 MPa |
| | | **SCHIST**<br>Highly weathered, containing rootlets | 1.32 | | |
| | | **SCHIST**<br>Fresh, coarsely crystalline, slightly porous, medium strong, brown and black | 1.40 | 2 | (Axial) |
| 2 | | **GRANITE GNEISS**<br>Fresh, foliated, medium to coarsely crystalline, slightly porous, strong to extremely strong, grey, pink and black | 184.59<br>1.83 | | |
| 3 | | | 182.72 | 3 | 14.1 MPa<br>UC = 175 MPa |
| | | END OF DRILLHOLE | 3.70 | | |
| 4 | | | | | |
| 5 | | | | | |
| 6 | | | | | |
| 7 | | | | | |
| 8 | | | | | |
| 9 | | | | | |
| 10 | | | | | |



| TYPE AND SURFACE DESCRIPTION | Jr | Ja | Jn |
|---|---|---|---|
| JN,PL,RO Fe | 1.5 | 1 | |
| JN,PL,RO Fe | 1.5 | 1 | |
| JN,UN,RO | 3 | 1 | |
| | | | 12 |
| JN,UN,RO | 3 | 1 | |
| JN,UN,SM | 2 | 1 | |
| JN,PL,RO | 1.5 | 1 | |
| JN,PL,RO | 1.5 | 1 | |
| JN,PL,RO | 1.5 | 1 | |
| JN,PL,RO Fe | 1.5 | 1 | |
| JN,PL,RO Fe | 1.5 | 1 | |
| JN,PL,RO Fe | 1.5 | 1 | |
| JN,PL,RO | 1.5 | 1 | |
| JN,PL,RO | 1.5 | 1 | |

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: TT

CHECKED: MAS/TVA

# RECORD OF BOREHOLE No B202-10

SHEET 1 OF 1

**METRIC**

PROJECT 09-1111-6014

W.P. 5139-08-01

LOCATION N 5074904.1 ;E 225153.3

ORIGINATED BY TT

DIST

HWY 69

BOREHOLE TYPE Hand Excavation

COMPILED BY MAS

DATUM Geodetic

DATE February 26, 2011

CHECKED BY TVA

| SOIL PROFILE | | | SAMPLES | | | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100<br>SHEAR STRENGTH kPa<br>○ UNCONFINED + FIELD VANE<br>● QUICK TRIAXIAL × REMOULDED<br>20 40 60 80 100 | PLASTIC LIMIT $W_P$ NATURAL MOISTURE CONTENT $W$ LIQUID LIMIT $W_L$<br>WATER CONTENT (%)<br>25 50 75 | UNIT WEIGHT γ kN/m³ | REMARKS & GRAIN SIZE DISTRIBUTION (%)<br>GR SA SI CL |
|---|---|---|---|---|---|---|---|---|---|---|---|
| ELEV / DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | | | | | | |
| 187.1 | GROUND SURFACE | | | | | | | | | | |
| 0.0<br>0.1 | TOPSOIL | | | | | | | | | | |
| | END OF EXCAVATION - Bedrock | | | | | | | | | | |
| | NOTE:<br>1. Hand digging carried out at proposed borehole location to expose bedrock. | | | | | | | | | | |

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

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

# RECORD OF BOREHOLE No B202-11

SHEET 1 OF 1

**METRIC**

PROJECT 09-1111-6014
W.P. 5139-08-01
LOCATION N 5074919.8 ;E 225139.4
ORIGINATED BY TT
DIST ______ HWY 69
BOREHOLE TYPE Hand Excavation
COMPILED BY MAS
DATUM Geodetic
DATE March 3, 2011
CHECKED BY TVA

| SOIL PROFILE | | | SAMPLES | | | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100<br>SHEAR STRENGTH kPa<br>○ UNCONFINED + FIELD VANE<br>● QUICK TRIAXIAL × REMOULDED<br>20 40 60 80 100 | PLASTIC LIMIT $W_P$ NATURAL MOISTURE CONTENT $W$ LIQUID LIMIT $W_L$<br>WATER CONTENT (%)<br>25 50 75 | UNIT WEIGHT γ kN/m³ | REMARKS & GRAIN SIZE DISTRIBUTION (%)<br>GR SA SI CL |
|---|---|---|---|---|---|---|---|---|---|---|---|
| ELEV<br>DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | | | | | | |
| 187.3 | GROUND SURFACE | | | | | | | | | | |
| 0.0<br>0.1 | TOPSOIL<br>END OF EXCAVATION - Bedrock | | | | | | | | | | |
| | NOTE: | | | | | | | | | | |
| | 1. Hand digging carried out at proposed borehole location to expose bedrock. | | | | | | | | | | |

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

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

# RECORD OF BOREHOLE No B202-12

SHEET 1 OF 1

**METRIC**

PROJECT 09-1111-6014

W.P. 5139-08-01

DIST HWY 69

DATUM Geodetic

LOCATION N 5074873.6 ;E 225151.0

BOREHOLE TYPE 127 mm O.D. Continuous Flight Solid Stem Augers, NW Casing, Wash Boring

DATE March 28, 2011

ORIGINATED BY MR

COMPILED BY MAS

CHECKED BY TVA

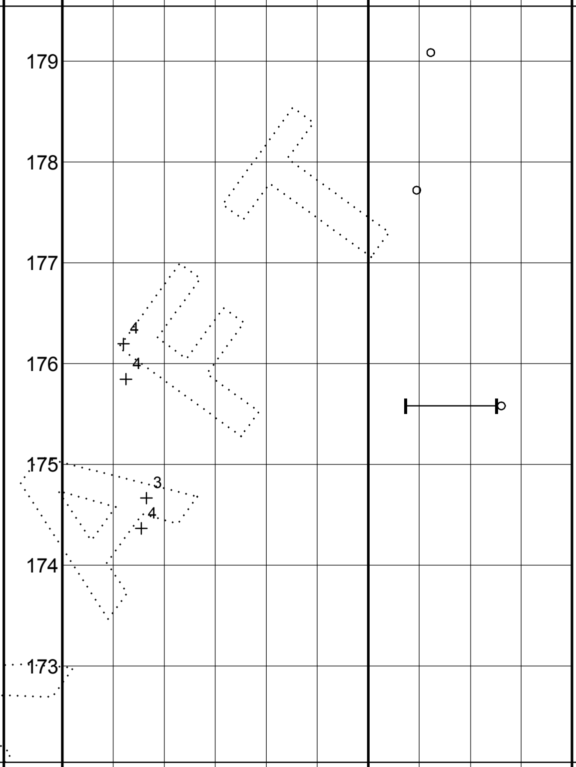
| ELEV<br>DEPTH | SOIL PROFILE – DESCRIPTION | SAMPLES – NUMBER | TYPE | "N" VALUES | GROUND WATER CONDITIONS | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
|---|---|---|---|---|---|---|
| 179.5 | GROUND SURFACE | | | | | |
| 0.0<br>179.2 | TOPSOIL | 1A | SS | 5 | | |
| | Sandy SILT, containing rootlets<br>Loose<br>Grey<br>Moist | 1B | | | | |
| 178.8<br>0.7 | Silty SAND, trace to some clay, containing organics<br>Very loose to loose<br>Brown<br>Moist to Wet | 2 | SS | 8 | ▽ | |
| | | 3 | SS | 3 | | 0 71 21 8 |
| 177.3 | Sandy SILT<br>Very loose<br>Grey<br>Wet | 4A<br>4B | SS | 2 | | |
| 176.9<br>2.6 | CLAY, containing silt interlayers to a depth of 3.4 m<br>Soft to firm<br>Grey<br>Moist | 5 | SS | WH | | 0 1 43 56 |
| 173.6<br>5.9 | Granite Gneiss (BEDROCK)<br>Bedrock cored from depths of 5.9 m to 7.5 m<br>For bedrock coring details refer to Record of Drillhole B202-12 | 1 | RC | REC 100% | | RQD = 99% |
| 172.0<br>7.5 | END OF BOREHOLE<br>NOTE:<br>1. Water level in open borehole at a depth of 1.3 m below ground surface (Elev. 178.2 m) upon completion of drilling. | | | | | |

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

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

PLASTIC LIMIT $W_P$ NATURAL MOISTURE CONTENT W LIQUID LIMIT $W_L$ WATER CONTENT (%) 25 50 75

UNIT WEIGHT γ kN/m³



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

PROJECT: 09-1111-6014

# RECORD OF DRILLHOLE: B202-12

SHEET 1 OF 1

LOCATION: N 5074873.6 ;E 225151.0

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: March 28, 2011

DRILL RIG: D25 Bombardier

DRILLING CONTRACTOR: OGS Inc

DATUM: Geodetic

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.

| DEPTH SCALE METRES | DRILLING RECORD | DESCRIPTION | SYMBOLIC LOG | ELEV. / DEPTH (m) | RUN No. | COLOUR % RETURN / FLUSH | RECOVERY TOTAL CORE % (80 60 40 20) | RECOVERY SOLID CORE % (80 60 40 20) | R.Q.D. % (80 60 40 20) | FRACT. INDEX PER 0.3 m (5 10 15 20) | DISCONTINUITY DATA B Angle (0 90 180 270) | DISCONTINUITY DATA DIP w.r.t. CORE AXIS (0 30 60 90) | DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION | Jr | Ja | Jn | HYDRAULIC CONDUCTIVITY K, cm/sec ($10^{-6}$ $10^{-5}$ $10^{-4}$ $10^{-3}$) | Diametral Point Load Index (MPa) (2 4 6) | RMC -Q' AVG. | NOTES |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Continued from Record of Borehole B202-12 | | 173.65 | | | | | | | | | | | | | | | | |
| 6 | NQRC March 26, 2011 | GRANITE GNEISS<br>Fresh, medium crystalline, slightly porous, strong to very strong, pink, grey and black | | 172.05<br>5.90 | | | | | | | | | | | | | | | | |
| | NW Casing March 26, 2011 | | | | 1 | | | | | | | ● | JN,PL,SM He | 1 | 2 | 1 | | ● | | 9.3 MPa |
| 7 | | | | | | | | | | | | | | | | | | | | |
| | | END OF DRILLHOLE | | 7.50 | | | | | | | | | | | | | | | | |
| 8 | | | | | | | | | | | | | | | | | | | | |
| 9 | | | | | | | | | | | | | | | | | | | | |
| 10 | | | | | | | | | | | | | | | | | | | | |
| 11 | | | | | | | | | | | | | | | | | | | | |
| 12 | | | | | | | | | | | | | | | | | | | | |
| 13 | | | | | | | | | | | | | | | | | | | | |
| 14 | | | | | | | | | | | | | | | | | | | | |
| 15 | | | | | | | | | | | | | | | | | | | | |

DEPTH SCALE
1 : 50

Golder Associates

LOGGED: MR
CHECKED: MAS/TVA

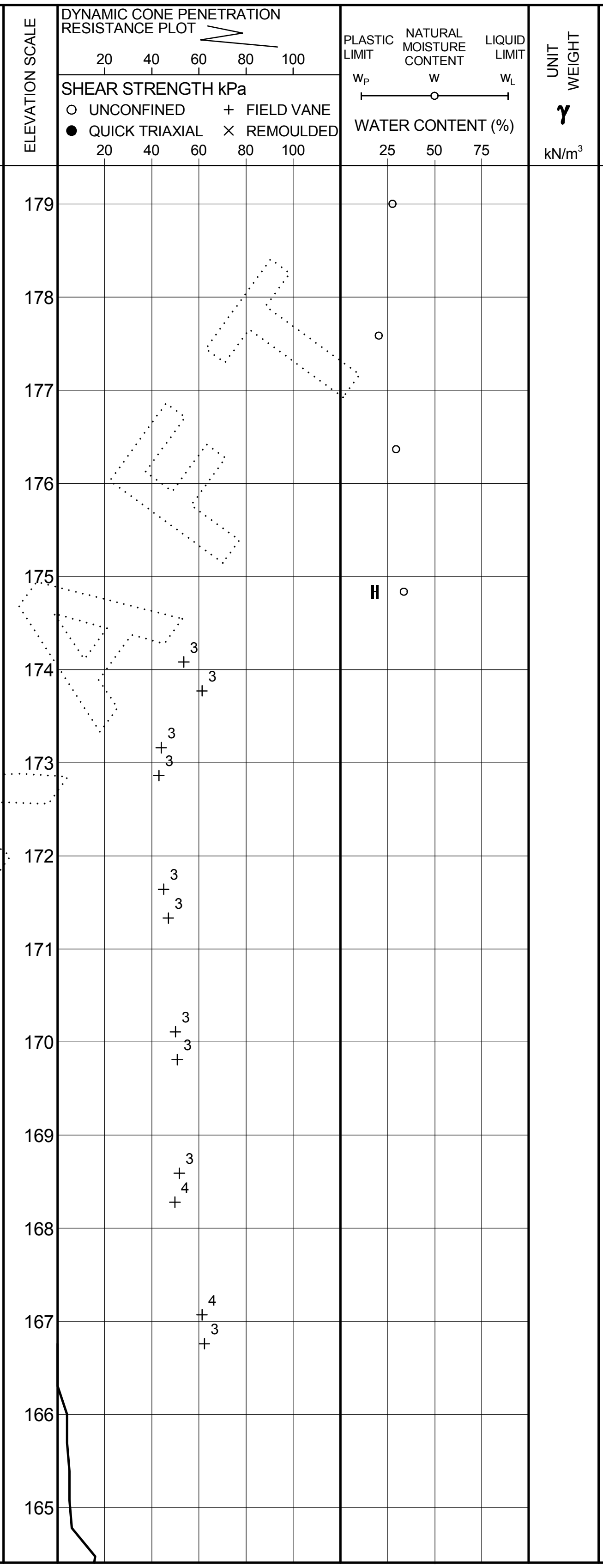
# RECORD OF BOREHOLE No B202-13

SHEET 1 OF 2

**METRIC**

PROJECT 09-1111-6014
W.P. 5139-08-01
LOCATION N 5074871.4 ;E 225166.4
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 29, 2011
CHECKED BY TVA

| ELEV<br>DEPTH | SOIL PROFILE<br>DESCRIPTION | SAMPLES<br>NUMBER | TYPE | "N" VALUES | GROUND WATER CONDITIONS | REMARKS & GRAIN SIZE DISTRIBUTION (%)<br>GR | SA | SI | CL |
|---|---|---|---|---|---|---|---|---|---|
| 179.4 | GROUND SURFACE | | | | | | | | |
| 0.0 | TOPSOIL | 1A | | | | | | | |
| 0.2 | Silty SAND, containing organics and rootlets to a depth of 0.8 m<br>Very loose to loose<br>Brown and grey<br>Wet | 1B | SS | 3 | | | | | |
| | | 2 | SS | 4 | ∇ | | | | |
| 178.0<br>1.4 | SAND, some gravel<br>Loose<br>Brown<br>Wet | 3 | SS | 4 | | | | | |
| 177.3<br>2.1 | SILT, trace clay, trace sand<br>Very loose<br>Grey<br>Wet | 4 | SS | 3 | | | | | |
| | | 5 | SS | WH | | 0 | 5 | 90 | 5 |
| | Containing silty clay lenses between depths of 3.5 m and 4.1 m | 6 | SS | WH | | | | | |
| | | 7 | SS | 1 | | 0 | 3 | 81 | 16 |
| 174.2<br>5.2 | SILTY CLAY to CLAY, trace sand<br>Firm to stiff<br>Grey<br>Moist | | | | | | | | |
| 166.7<br>12.7 | END OF BOREHOLE<br><br>Dynamic Cone Penetration Test (DCPT) | | | | | | | | |



Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

PROJECT 09-1111-6014

# RECORD OF BOREHOLE No B202-13

SHEET 2 OF 2

**METRIC**

W.P. 5139-08-01

LOCATION N 5074871.4 ;E 225166.4

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 29, 2011

CHECKED BY TVA

| SOIL PROFILE | | | SAMPLES | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| ELEV<br>DEPTH | DESCRIPTION<br>--- *CONTINUED FROM PREVIOUS PAGE* --- | STRAT PLOT | NUMBER | TYPE | "N" VALUES | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100<br>SHEAR STRENGTH kPa<br>○ UNCONFINED + FIELD VANE<br>● QUICK TRIAXIAL × REMOULDED<br>20 40 60 80 100 | PLASTIC LIMIT $w_P$ NATURAL MOISTURE CONTENT $w$ LIQUID LIMIT $w_L$<br>WATER CONTENT (%)<br>25 50 75 | UNIT WEIGHT γ<br>kN/m³ | REMARKS & GRAIN SIZE DISTRIBUTION (%)<br>GR SA SI CL |
| 164.1<br>15.3 | END OF DCPT<br>Refusal to Further Penetration<br>(Hammer Bouncing)<br>NOTES:<br>1. Water level in open borehole at a depth of 1.4 m below ground surface (Elev. 178.0 m) upon completion of drilling.<br>2. A Dynamic Cone Penetration Test was carried out below a depth of 12.7 m; refusal encountered at a depth of 15.3 m (Elev. 164.1 m) below ground surface. | | | | | | | | | | |

+ $^3$, × $^3$: Numbers refer to Sensitivity

○ $^{3\%}$ STRAIN AT FAILURE

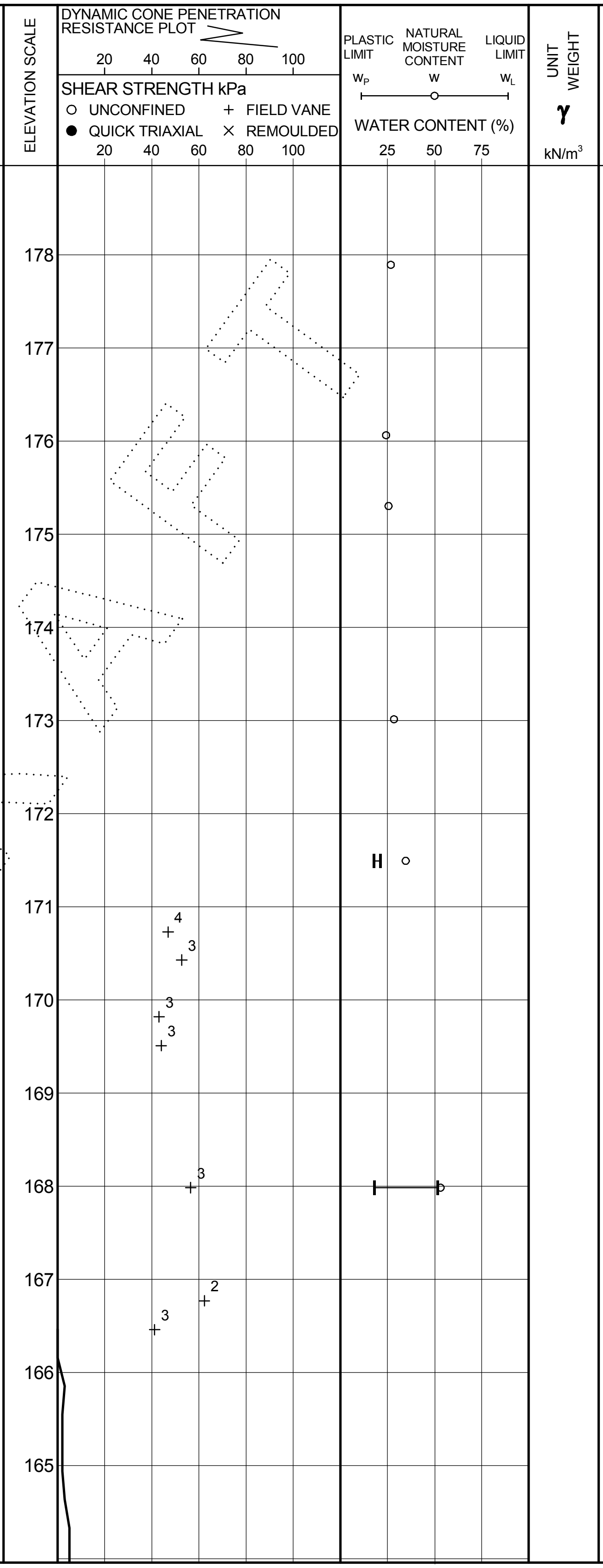
# RECORD OF BOREHOLE No B202-14

SHEET 1 OF 3

**METRIC**

| | | | |
|---|---|---|---|
| PROJECT 09-1111-6014 | | | |
| W.P. 5139-08-01 | LOCATION N 5074860.2 ;E 225170.8 | | 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 29 and 30, 2011 | CHECKED BY TVA |

| ELEV / DEPTH | DESCRIPTION | NUMBER | TYPE | "N" VALUES | GROUND WATER CONDITIONS | GR | SA | SI | CL |
|---|---|---|---|---|---|---|---|---|---|
| 179.0 | GROUND SURFACE | | | | | | | | |
| 0.0 | TOPSOIL | | | | | | | | |
| 0.2 | SAND and SILT, trace clay, containing rootlets to a depth of 0.6 m<br>Very loose to loose<br>Brown to grey<br>Moist to Wet | 1 | SS | 2 | | | | | |
| | | 2 | SS | 4 | ▽ | | | | |
| | | 3 | SS | 1 | | | | | |
| | | 4 | SS | 5 | | 0 | 59 | 38 | 3 |
| | | 5 | SS | 3 | | | | | |
| | Becoming grey below a depth of 4.1 m | 6 | SS | 3 | | | | | |
| 173.7<br>5.3 | SILT, some clay, trace sand, containing clay lenses<br>Very loose<br>Grey<br>Wet | 7 | SS | 2 | | | | | |
| | | 8 | SS | WH | | 0 | 1 | 81 | 18 |
| 170.9<br>8.1 | CLAY, containing silt interlayers<br>Firm to stiff<br>Grey<br>Moist | | | | | | | | |
| | | 9 | SS | 2 | | | | | |
| 166.5<br>12.5 | Dynamic Cone Penetration Test (DCPT)<br>END OF BOREHOLE | | | | | | | | |



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

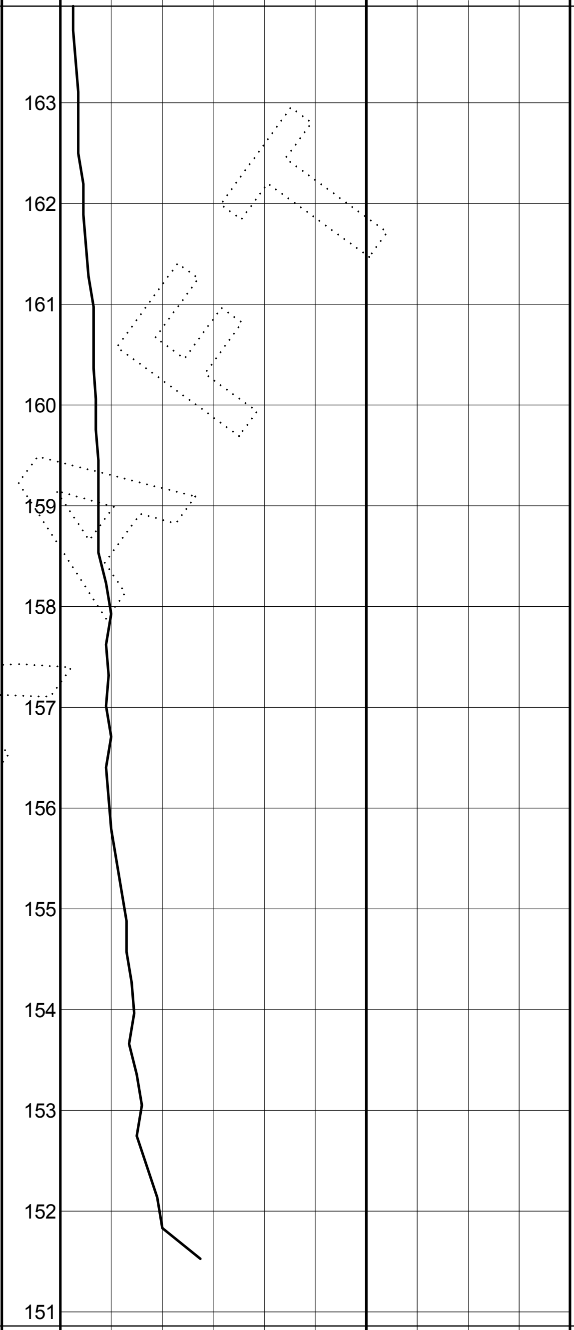
# RECORD OF BOREHOLE No B202-14

SHEET 2 OF 3

**METRIC**

| | | | |
|---|---|---|---|
| PROJECT 09-1111-6014 | | | |
| W.P. 5139-08-01 | LOCATION N 5074860.2 ;E 225170.8 | 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 29 and 30, 2011 | CHECKED BY TVA |

| SOIL PROFILE | | | SAMPLES | | | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100 | PLASTIC LIMIT $W_P$ NATURAL MOISTURE CONTENT W LIQUID LIMIT $W_L$ / WATER CONTENT (%) 25 50 75 | UNIT WEIGHT γ kN/m³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
|---|---|---|---|---|---|---|---|---|---|---|---|
| ELEV / DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | | | | | | |
| | --- CONTINUED FROM PREVIOUS PAGE --- | | | | | | | | | | |
| | Dynamic Cone Penetration Test (DCPT) | | | | | | | | | | |
| 151.6<br>27.4 | END OF DCPT | | | | | | | | | | |
| 150.9<br>28.1 | END OF TRICONE BOREHOLE (PROBABLE BEDROCK) | | | | | | | | | | |



Continued Next Page

+³, ×³: Numbers refer to Sensitivity

○³% STRAIN AT FAILURE

## RECORD OF BOREHOLE No B202-14

SHEET 3 OF 3

**METRIC**

PROJECT 09-1111-6014

W.P. 5139-08-01

LOCATION N 5074860.2 ;E 225170.8

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 29 and 30, 2011

CHECKED BY TVA

| SOIL PROFILE | | | SAMPLES | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| ELEV<br>DEPTH | DESCRIPTION<br>--- CONTINUED FROM PREVIOUS PAGE --- | STRAT PLOT | NUMBER | TYPE | "N" VALUES | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100<br>SHEAR STRENGTH kPa<br>○ UNCONFINED + FIELD VANE<br>● QUICK TRIAXIAL × REMOULDED<br>20 40 60 80 100 | PLASTIC LIMIT $W_P$ NATURAL MOISTURE CONTENT W LIQUID LIMIT $W_L$<br>WATER CONTENT (%)<br>25 50 75 | UNIT WEIGHT γ kN/m$^3$ | REMARKS & GRAIN SIZE DISTRIBUTION (%)<br>GR SA SI CL |
| | NOTES:<br>1. Water level in open borehole at a depth of 1.1 m below ground surface (Elev. 177.9 m) upon completion of drilling.<br>2. A Dynamic Cone Penetration Test was carried out between depths of 12.5 m and 27.4 m below ground surface (Elev. 166.5 m and 151.6 m) .<br>3. Tricone advanced below a depth of 27.4 m; refusal encountered at a depth of 28.1 m below ground surface (Elev. 150.9 m) on probable bedrock. | | | | | | | | | | |

GTA-MTO 001 09-1111-6014.GPJ GAL-GTA.GDT 2/6/13 SAC/DD

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

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

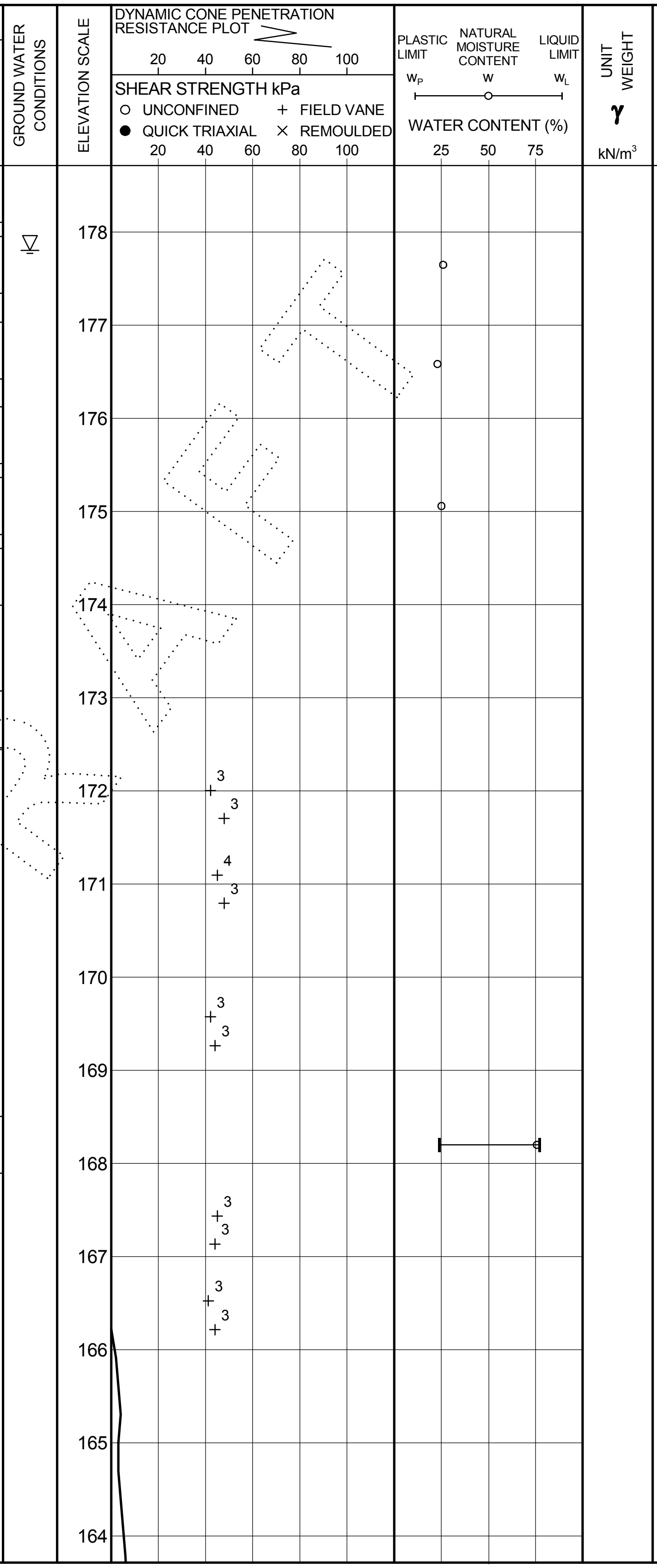
# RECORD OF BOREHOLE No B202-15

SHEET 1 OF 2

**METRIC**

PROJECT 09-1111-6014
W.P. 5139-08-01
LOCATION N 5074862.4 ;E 225155.3
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 30, 2011
CHECKED BY TVA

| SOIL PROFILE | | SAMPLES | | | REMARKS & GRAIN SIZE DISTRIBUTION (%) | | | |
|---|---|---|---|---|---|---|---|---|
| ELEV DEPTH | DESCRIPTION | NUMBER | TYPE | "N" VALUES | GR | SA | SI | CL |
| 178.7 | GROUND SURFACE | | | | | | | |
| 0.0 | TOPSOIL | | | | | | | |
| 0.2 | SAND, some silt, containing rootlets and organics<br>Very loose<br>Grey to brown<br>Wet | 1 | SS | 1 | | | | |
| | | 2 | SS | 2 | | | | |
| 176.9<br>1.8 | SAND and SILT, trace clay<br>Very loose to loose<br>Brown to grey<br>Wet | 3A<br>3B | SS | 5 | | | | |
| | | 4 | SS | 2 | | | | |
| | | 5 | SS | 5 | 0 | 67 | 31 | 2 |
| | | 6 | SS | 6 | | | | |
| 173.5<br>5.2 | CLAY, containing silt interlayers<br>Firm<br>Grey<br>Moist | 7 | SS | 1 | | | | |
| | | 8 | SS | 1 | 0 | 1 | 25 | 74 |
| 166.2<br>12.5 | END OF BOREHOLE<br>Dynamic Cone Penetration Test (DCPT) | | | | | | | |



Continued Next Page

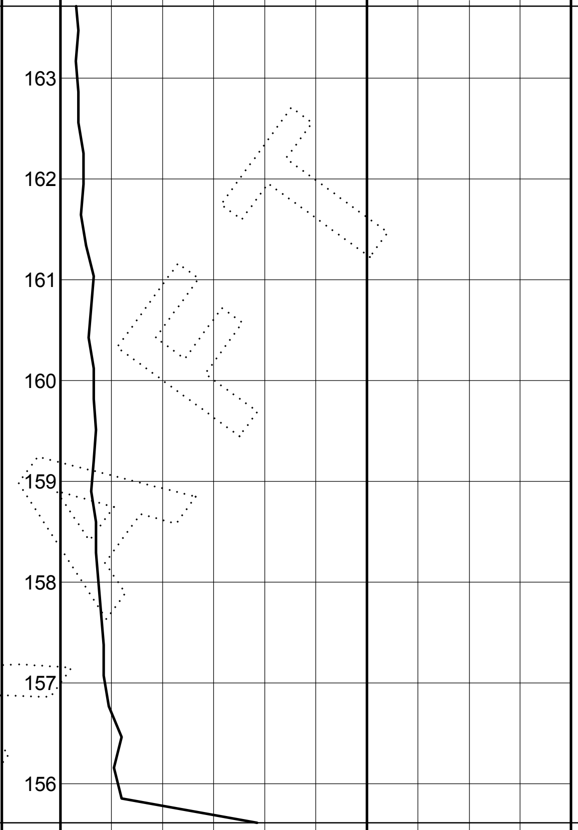
+³, ×³: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

# RECORD OF BOREHOLE No B202-15

SHEET 2 OF 2

**METRIC**

PROJECT 09-1111-6014
W.P. 5139-08-01
LOCATION N 5074862.4 ;E 225155.3
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 30, 2011
CHECKED BY TVA

| SOIL PROFILE | | | SAMPLES | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| ELEV<br>DEPTH | DESCRIPTION<br>--- *CONTINUED FROM PREVIOUS PAGE* --- | STRAT PLOT | NUMBER | TYPE | "N" VALUES | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100<br>SHEAR STRENGTH kPa<br>○ UNCONFINED + FIELD VANE<br>● QUICK TRIAXIAL × REMOULDED<br>20 40 60 80 100 | PLASTIC LIMIT $W_P$ — NATURAL MOISTURE CONTENT $W$ — LIQUID LIMIT $W_L$<br>WATER CONTENT (%)<br>25 50 75 | UNIT WEIGHT γ<br>kN/m$^3$ | REMARKS & GRAIN SIZE DISTRIBUTION (%)<br>GR SA SI CL |
| | Dynamic Cone Penetration Test (DCPT) | | | | | |  | | | | |
| 155.6<br>23.1 | END OF DCPT<br>Refusal to Further Penetration (Hammer Bouncing)<br>NOTES:<br>1. Water level in open borehole at a depth of 0.9 m below ground surface (Elev. 177.8 m) upon completion of drilling.<br>2. A Dynamic Cone Penetration Test was advanced below a depth of 12.5 m; refusal encountered at a depth of 23.1 m below ground surface (Elev. 155.6 m). | | | | | | | | | | |

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

# RECORD OF BOREHOLE No S204-18

SHEET 1 OF 4

**METRIC**

PROJECT 09-1111-6014

W.P. 5404-05-01

LOCATION N 5074795.1 ;E 225189.0

ORIGINATED BY MR/RA

DIST ____ HWY 69

BOREHOLE TYPE 127 mm O.D. Continuous Flight Solid Stem Augers, NW Casing, Wash Boring

COMPILED BY OK

DATUM Geodetic

DATE February 6 and 7, 2010

CHECKED BY TVA

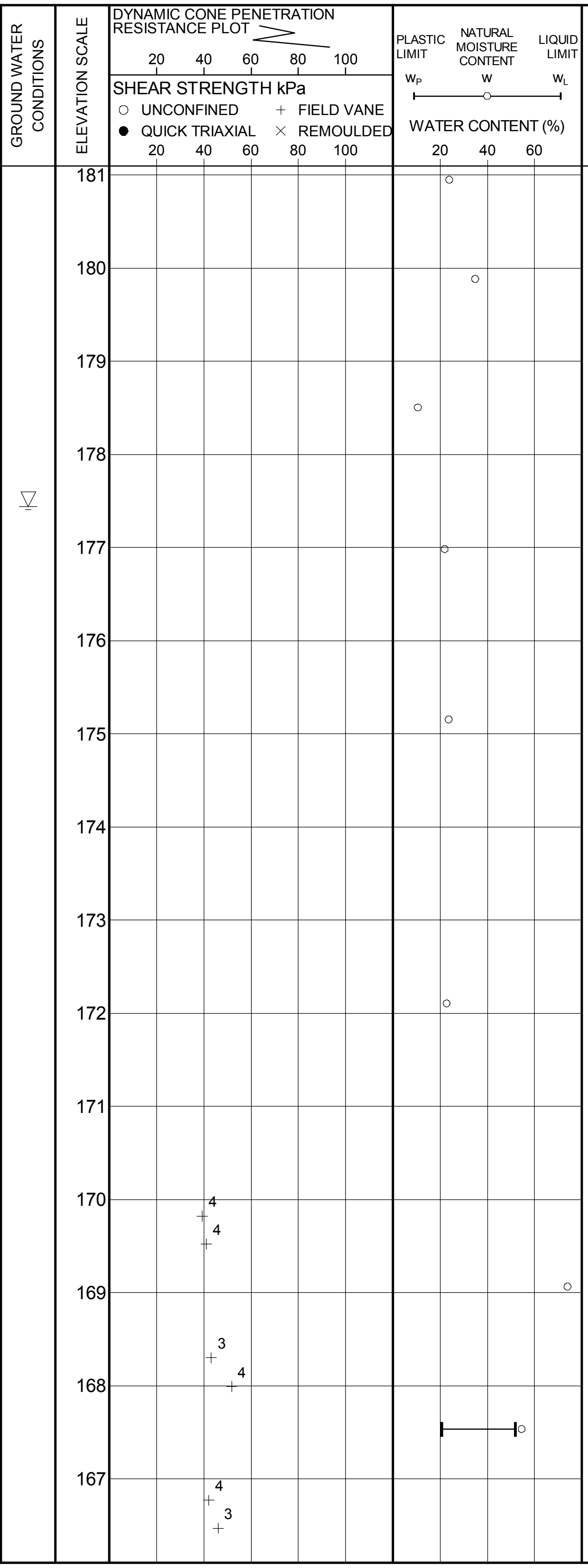
## SOIL PROFILE

| ELEV<br>DEPTH | DESCRIPTION |
|---|---|
| 181.1 | GROUND SURFACE |
| 0.0 | TOPSOIL |
| 180.8<br>0.3 | SILTY CLAY, containing rootlets and silty sand layers to a depth of 0.8 m<br>Firm<br>Brown to grey<br>Moist |
| 179.0<br>2.1 | Silty SAND, trace clay<br>Loose to compact<br>Brown<br>Wet |
| 177.4<br>3.7 | SILT, some sand, trace to some clay<br>Loose to compact<br>Brown<br>Wet<br><br>Becoming grey below a depth of 5.5 m |
| 171.4<br>9.7 | CLAY<br>Firm to stiff<br>Grey<br>Moist |

## SAMPLES

| NUMBER | TYPE | "N" VALUES | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
|---|---|---|---|
| 1A<br>1B | SS | 5 | |
| 2 | SS | 6 | |
| 3A<br>3B | TO | PH | |
| 4 | SS | 8 | |
| 5 | SS | 18 | |
| 6 | SS | 13 | 0 16 78 6 |
| 7 | SS | 4 | |
| 8 | SS | 15 | |
| 9 | SS | 5 | |
| 10 | SS | 9 | |
| 11 | SS | 2 | |
| 12 | SS | 1 | |
| 13 | SS | 2 | 0 1 47 52 |

GROUND WATER CONDITIONS ∇



UNIT WEIGHT γ kN/m³

Continued Next Page

+³, ×³: Numbers refer to Sensitivity

○³% STRAIN AT FAILURE

# RECORD OF BOREHOLE No S204-18

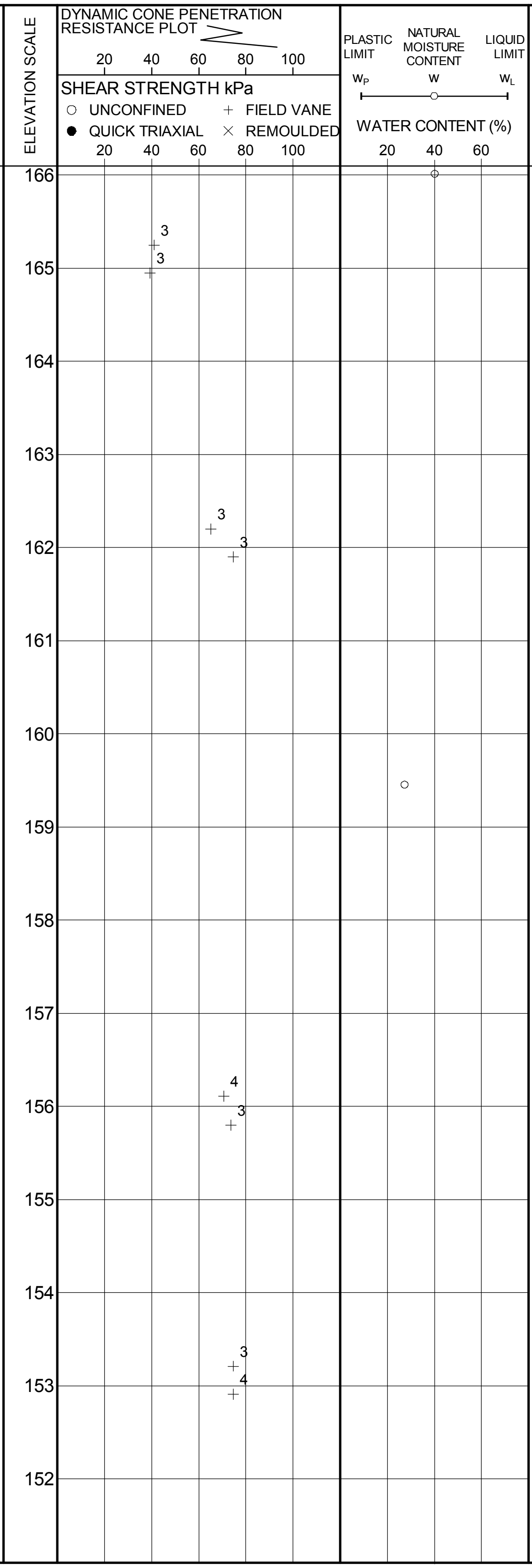
SHEET 2 OF 4

**METRIC**

PROJECT 09-1111-6014
W.P. 5404-05-01
DIST HWY 69
DATUM Geodetic
LOCATION N 5074795.1 ;E 225189.0
BOREHOLE TYPE 127 mm O.D. Continuous Flight Solid Stem Augers, NW Casing, Wash Boring
DATE February 6 and 7, 2010
ORIGINATED BY MR/RA
COMPILED BY OK
CHECKED BY TVA

--- CONTINUED FROM PREVIOUS PAGE ---

| ELEV<br>DEPTH | DESCRIPTION | NUMBER | TYPE | "N" VALUES | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR | SA | SI | CL |
|---|---|---|---|---|---|---|---|---|
| | CLAY<br>Firm to stiff<br>Grey<br>Moist | 14 | SS | WH | | | | |
| | | 15 | SS | 3 | | | | |
| 161.3<br>19.8 | SILT, trace to some sand, trace clay<br>Loose<br>Grey<br>Wet | 16 | TO | PH* | | | | |
| | | 17 | SS | 7 | 0 | 8 | 88 | 4 |
| 158.2<br>22.9 | SILTY CLAY<br>Stiff to very stiff<br>Grey<br>Moist | 18 | SS | 5 | | | | |
| | | 19 | TO | PH | | | | |



UNIT WEIGHT γ kN/m³

Continued Next Page

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

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

# RECORD OF BOREHOLE No S204-18

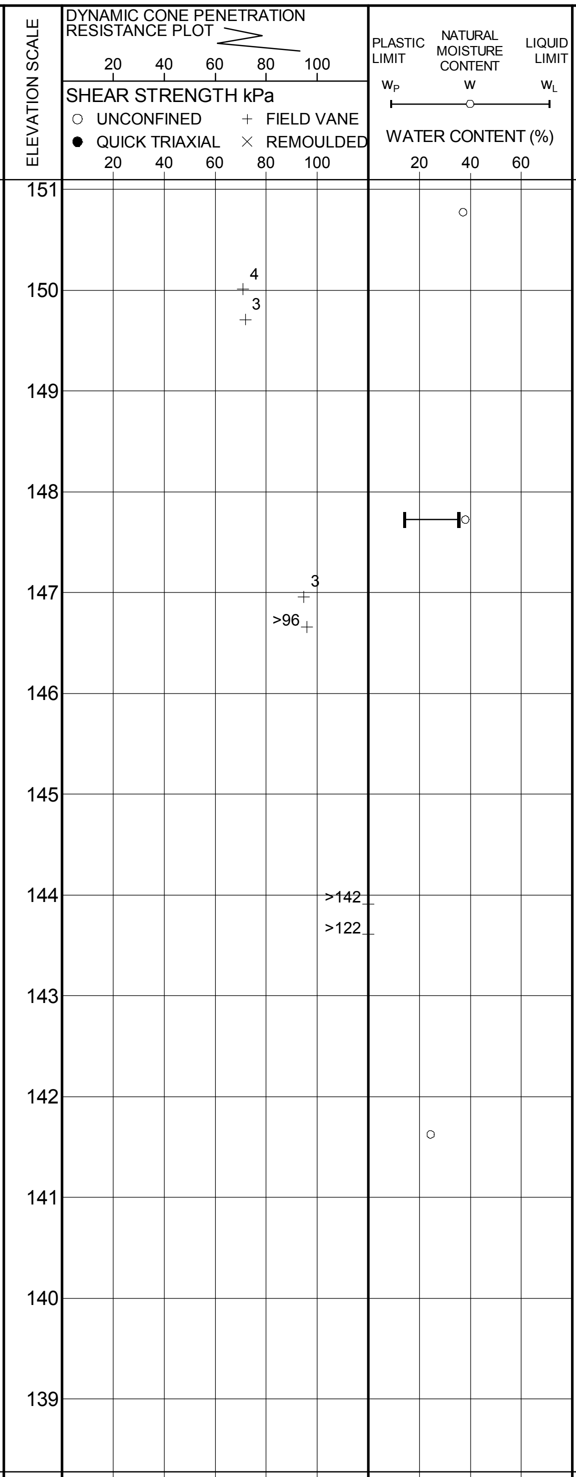
SHEET 3 OF 4

**METRIC**

| | | | |
|---|---|---|---|
| PROJECT 09-1111-6014 | | | |
| W.P. 5404-05-01 | LOCATION N 5074795.1 ;E 225189.0 | ORIGINATED BY MR/RA |
| DIST | HWY 69 | BOREHOLE TYPE 127 mm O.D. Continuous Flight Solid Stem Augers, NW Casing, Wash Boring | COMPILED BY OK |
| DATUM Geodetic | | DATE February 6 and 7, 2010 | CHECKED BY TVA |

--- CONTINUED FROM PREVIOUS PAGE ---

| ELEV<br>DEPTH | DESCRIPTION | SAMPLE NUMBER | TYPE | "N" VALUES | GR | SA | SI | CL |
|---|---|---|---|---|---|---|---|---|
| | SILTY CLAY<br>Stiff to very stiff<br>Grey<br>Moist | 20 | SS | 7 | 0 | 1 | 54 | 45 |
| | | 21 | SS | 5 | | | | |
| | Silt interlayers encountered below a depth of 36.1 m | 22 | SS | 9 | | | | |
| 143.2<br>37.9 | SAND and SILT, trace clay<br>Compact<br>Grey<br>Wet | | | | | | | |
| | | 23 | SS | 21 | 0 | 35 | 65 | 0 |
| | | 24 | SS | 12 | | | | |
| 138.3<br>42.8 | END OF BOREHOLE | | | | | | | |



Continued Next Page

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

PROJECT 09-1111-6014

## RECORD OF BOREHOLE No S204-18

SHEET 4 OF 4

**METRIC**

W.P. 5404-05-01

LOCATION N 5074795.1 ;E 225189.0

ORIGINATED BY MR/RA

DIST HWY 69

BOREHOLE TYPE 127 mm O.D. Continuous Flight Solid Stem Augers, NW Casing, Wash Boring

COMPILED BY OK

DATUM Geodetic

DATE February 6 and 7, 2010

CHECKED BY TVA

| SOIL PROFILE | | | SAMPLES | | | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100<br>SHEAR STRENGTH kPa<br>○ UNCONFINED + FIELD VANE<br>● QUICK TRIAXIAL × REMOULDED<br>20 40 60 80 100 | PLASTIC LIMIT $W_P$ NATURAL MOISTURE CONTENT W LIQUID LIMIT $W_L$<br>WATER CONTENT (%)<br>20 40 60 | UNIT WEIGHT γ kN/m$^3$ | REMARKS & GRAIN SIZE DISTRIBUTION (%)<br>GR SA SI CL |
|---|---|---|---|---|---|---|---|---|---|---|---|
| ELEV DEPTH | DESCRIPTION<br>--- CONTINUED FROM PREVIOUS PAGE --- | STRAT PLOT | NUMBER | TYPE | "N" VALUES | | | | | | |
| | NOTES:<br>* Unable to recover a Shelby tube sample between depths of 20.9 m and 21.3 m (Elev. 160.2 m and 159.8 m) below ground surface.<br>1. Water level in open borehole at a depth of 3.7 m below ground surface (Elev. 177.4 m) upon completion of drilling. | | | | | | | | | | |

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

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

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

Golder Associates

Foundation Design

# RECORD OF BOREHOLE No SP1

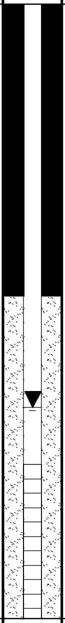
SHEET 1 OF 1

**METRIC**

PROJECT 09-1111-6014
W.P. 5139-08-01
DIST
HWY 69
DATUM Geodetic
LOCATION N 5074819.8 ;E 225159.0
BOREHOLE TYPE 213 mm O.D. Continuous Flight Hollow Stem Augers
DATE February 17, 2011
ORIGINATED BY MR
COMPILED BY TVA
CHECKED BY TVA

| ELEV<br>DEPTH | SOIL PROFILE – DESCRIPTION | ELEVATION SCALE |
|---|---|---|
| 181.1 | GROUND SURFACE | |
| 0.0 | TOPSOIL | 181 |
| 180.8<br>0.3 | Sandy SILT, some clay<br>Brown | |
| 180.2<br>0.9 | SILTY CLAY<br>Grey | 180<br>179 |
| 178.7<br>2.4 | SAND to Silty SAND<br>Brown to grey | 178<br>177<br>176 |
| 175.0<br>6.1 | END OF BOREHOLE | |

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



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:

| Date | Depth (m) | Elev. (m) |
|---|---|---|
| 17/02/11 | 4.0 | 177.1 |
| 27/02/11 | 4.0 | 177.1 |

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

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

## RECORD OF PROBEHOLE No. B202-P01

| | | | | | |
|---|---|---|---|---|---|
| **PROJECT No.:** | 09-1111-6014 | **PROJECT NAME:** | Highway 69 / Still River Bridge (NBL) | **DATE:** | March 31, 2011 |
| **PROBEHOLE NUMBER:** | B202-P01 | **PROBEHOLE SIZE:** | NW Casing | **ELEVATION :** | 178.9 m |
| **MACHINE TYPE:** | D25 Bombardier | **CONTRACTOR:** | Walker Drilling Co. | **DATUM:** | Geodetic |
| **TEMPERATURE:** | -2°C | **WEATHER:** | Sunny | **LOCATION:** | N 5074858.6<br>E 225156.8 |

| **Depth** | | **Soil Description** | **Samples** | | **Remarks** |
|---|---|---|---|---|---|
| **From (m)** | **To (m)** | | **No.** | **Depth (m)** | |
| 0.0 | - | Upper portion of the overburden likely consists of sand based on drilling observation. | - | - | Refusal on probable bedrock at a depth of 25.6 m below ground surface (Elev. 153.3 m). |
| - | 25.6 | Refusal to casing advancement. | - | - | - |

*Comments:*

Water level in open hole at a depth of 0.8 m below ground surface (Elev. 178.1 m) upon completion of drilling.

Probehole backfilled with tremie bentonite grout to the surface.

| | |
|---|---|
| PROJECT No. | 09-1111-6014 |
| PROBEHOLE No.: | B202-P01 |
| ENGINEER: | TVA |

## RECORD OF PROBEHOLE No. B202-P02

| | | | | | |
|---|---|---|---|---|---|
| **PROJECT No.:** | 09-1111-6014 | **PROJECT NAME:** | Highway 69 / Still River Bridge (NBL) | **DATE:** | March 31, 2011 |
| **PROBEHOLE NUMBER:** | B202-P02 | **PROBEHOLE SIZE:** | NW Casing | **ELEVATION :** | 179.1 m |
| **MACHINE TYPE:** | D25 Bombardier | **CONTRACTOR:** | Walker Drilling Co. | **DATUM:** | Geodetic |
| **TEMPERATURE:** | -2°C | **WEATHER:** | Sunny | **LOCATION:** | N 5074864.1<br>E 225169.3 |

| **Depth** | | **Soil Description** | **Samples** | | **Remarks** |
|---|---|---|---|---|---|
| **From (m)** | **To (m)** | | **No.** | **Depth (m)** | |
| 0.0 | - | Upper portion of the overburden likely consists of sand based on drilling observation. | - | - | Refusal on probable bedrock at a depth of 25.7 m below ground surface (Elev. 153.4 m). |
| - | 25.7 | Refusal to casing advancement. | - | - | - |

*Comments:*

Artesian groundwater condition was noted while drilling; water level in open drill casing at 2.3 m above ground surface (Elev. 181.4 m), measured 53 minutes after completion of drilling.

Probehole backfilled with tremie bentonite grout to the surface.

| | |
|---|---|
| PROJECT No. | 09-1111-6014 |
| PROBEHOLE No.: | B202-P02 |
| ENGINEER: | TVA |

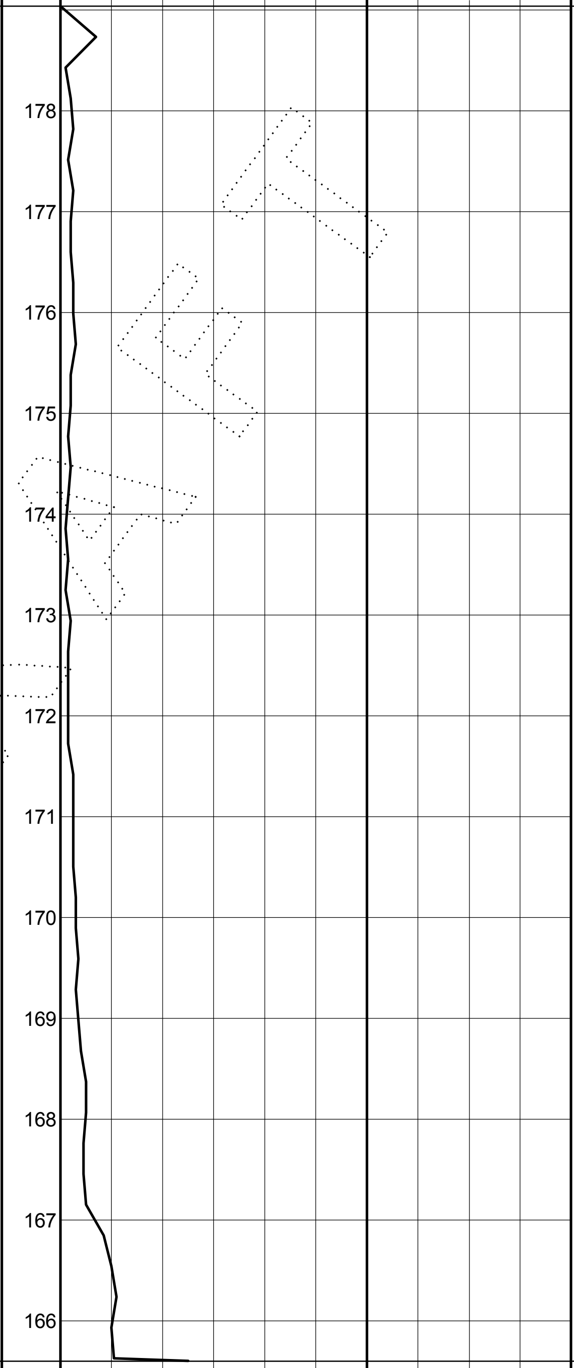
## RECORD OF DCPT No B202-DC01

SHEET 1 OF 1

**METRIC**

PROJECT 09-1111-6014
G.W.P. 5139-08-01
LOCATION N 5074869.7 ;E 225152.4
ORIGINATED BY MR
DIST
HWY 69
BOREHOLE TYPE Dynamic Cone Penetration Test
COMPILED BY MAS
DATUM Geodetic
DATE March 29, 2011
CHECKED BY TVA

| ELEV<br>DEPTH | SOIL PROFILE DESCRIPTION | STRAT PLOT | SAMPLES NUMBER | SAMPLES TYPE | SAMPLES "N" VALUES | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100 | PLASTIC LIMIT $W_P$ NATURAL MOISTURE CONTENT W LIQUID LIMIT $W_L$; WATER CONTENT (%) 25 50 75 | UNIT WEIGHT γ kN/m³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 179.0 | GROUND SURFACE | | | | | | | | | | |
| 0.0 | Dynamic Cone Penetration Test (DCPT) | | | | | | | | | | |
| 165.6<br>13.4 | END OF DCPT<br>Refusal to Further Penetration<br>(Hammer Bouncing) | | | | | | | | | | |



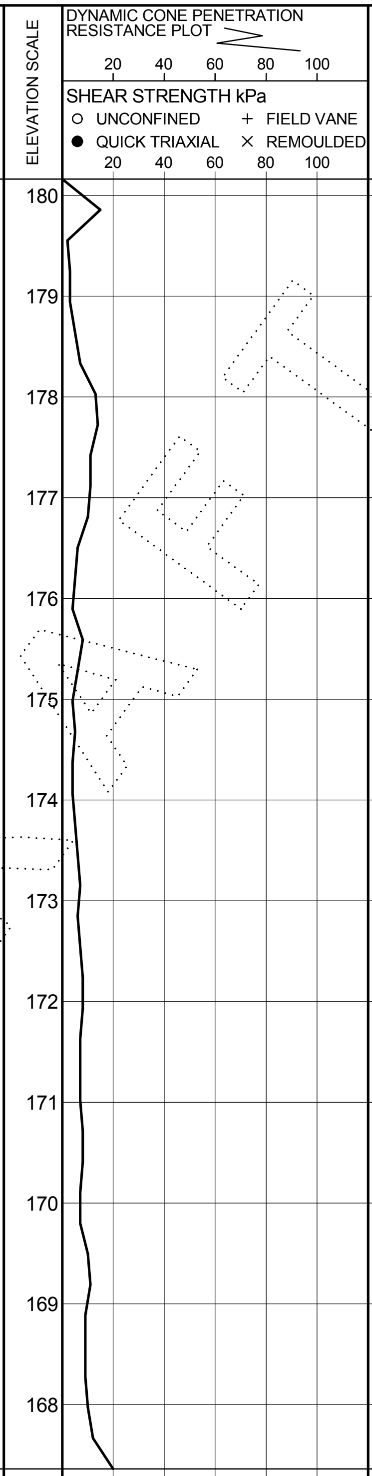
$+^3, \times^3$: Numbers refer to Sensitivity
$\circ^{3\%}$ STRAIN AT FAILURE

# RECORD OF DCPT No B202-DC02

SHEET 1 OF 1

**METRIC**

PROJECT 09-1111-6014
G.W.P. 5139-08-01
LOCATION N 5074875.3 ;E 225164.9
ORIGINATED BY MR
DIST ____ HWY 69
BOREHOLE TYPE Dynamic Cone Penetration Test
COMPILED BY MAS
DATUM Geodetic
DATE March 29, 2011
CHECKED BY TVA



| SOIL PROFILE | | | SAMPLES | | | | PLASTIC LIMIT $w_P$ — NATURAL MOISTURE CONTENT $w$ — LIQUID LIMIT $w_L$ | | UNIT WEIGHT | REMARKS & GRAIN SIZE DISTRIBUTION (%) | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| ELEV DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | GROUND WATER CONDITIONS | WATER CONTENT (%) 25 50 75 | | γ kN/m³ | GR | SA | SI | CL |
| 180.2 | GROUND SURFACE | | | | | | | | | | | | |
| 0.0 | Dynamic Cone Penetration Test (DCPT) | | | | | | | | | | | | |
| 167.4<br>12.8 | END OF DCPT<br>Refusal to Further Penetration<br>(Hammer Bouncing) | | | | | | | | | | | | |

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

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

# APPENDIX B

## Laboratory Test Results and Cobbles/Boulders and Bedrock Core Photographs

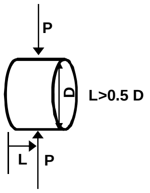
# TABLE B1

# POINT LOAD TEST RESULTS ON ROCK SAMPLES

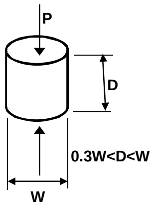
| Borehole Number | Run Number | Sample Depth (m) | Sample Elevation (m) | Bedrock Description | Test Type | Core Length (mm) | Core (2) Diameter (mm) | **Is (50mm)** (MPa) | **Approx. UCS Value (1)** (MPa) |
|---|---|---|---|---|---|---|---|---|---|
| B202-03 | 1 | 25.03 | 153.77 | Granite Gneiss | Diametral | 56.94 | 47.00 | 1.654 | 31 |
| B202-03 | 3 | 27.03 | 151.77 | Granite Gneiss | Axial | 45.20 | 46.99 | 2.719 | 52 |
| B202-04 | 1 | 10.33 | 169.07 | Granite Gneiss | Axial | 42.95 | 47.14 | 10.717 | 204 |
| B202-04 | 2 | 12.33 | 167.07 | Granite Gneiss | Diametral | 60.23 | 42.78 | 10.575 | 201 |
| B202-05 | 1 | 5.43 | 171.31 | Granite Gneiss | Axial | 54.03 | 47.24 | 5.193 | 99 |
| B202-07 | 2 | 1.61 | 182.69 | Granite Gneiss | Diametral | 54.36 | 45.49 | 9.483 | 180 |
| B202-07 | 3 | 2.53 | 181.77 | Granite Gneiss | Axial | 52.09 | 51.37 | 5.304 | 101 |
| B202-08 | 5 | 4.70 | 179.50 | Granite Gneiss | Axial | 31.86 | 37.60 | 8.331 | 158 |
| B202-09 | 1 | 1.15 | 185.25 | Gneiss | Diametral | 49.10 | 45.84 | 8.122 | 154 |
| B202-09 | 2 | 1.58 | 184.82 | Schist | Axial | 40.45 | 50.53 | 1.390 | 26 |
| B202-09 | 3 | 3.10 | 183.30 | Granite Gneiss | Diametral | 57.78 | 45.29 | 14.119 | 268 |
| B202-12 | 1 | 6.53 | 172.97 | Granite Gneiss | Diametral | 59.91 | 42.25 | 9.255 | 176 |
| | | | | | | | | | |
| | | | | | | | | | |
| | | | | | | | | | |
| | | | | | | | | | |
| | | | | | | | | | |
| | | | | | | | | | |

(1) $Is_{50}$ x 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 6 $Is_{50}$ 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/TZ
Reviewed by: JPD/JMAC

# TABLE B2-1
# SUMMARY OF UNCONFINED COMPRESSION STRENGTH TEST RESULTS
# STILL RIVER BRIDGE (NBL) STRUCTURE
# HIGHWAY 69 GWP 5404-05-00; WP 5139-08-01

| Borehole Number (Core Run) | Sample Depth (m) | Sample Elevation (m) | Rock Type | Core Diameter (mm) | Unconfined Compression Strength (MPa) |
|---|---|---|---|---|---|
| B202-03 (3) | 27.4 | 151.4 | Granite Gneiss | 47.1 | 69.0 |
| B202-04 (2) | 12.5 | 166.9 | Granite Gneiss | 47.2 | 193.2 |
| B202-09 (3) | 3.3 | 183.1 | Granite Gneiss | 50.5 | 175.4 |

Compiled By: TZ

Reviewed By: JPD/JMAC

# TABLE B2-2
# UNCONFINED COMPRESSION (UC) TEST
## ASTM D 7012-07

### SAMPLE IDENTIFICATION

| | | | |
|---|---|---|---|
| PROJECT NUMBER | 09-1111-6014 | RUN NUMBER | 3 |
| BOREHOLE NUMBER | B202-03 | SAMPLE DEPTH, m | 27.10-27.25 |

### TEST CONDITIONS

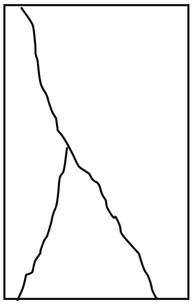
| | | | |
|---|---|---|---|
| MACHINE SPEED, mm/min | 0.00 | TYPE OF SPECIMEN | Rock Core |
| DURATION OF TEST,min | >2 <15 | L/D | 2.04 |

### SPECIMEN INFORMATION

| | | | |
|---|---|---|---|
| SAMPLE HEIGHT, cm | 9.58 | WATER CONTENT, (specimen) % | 0.39 |
| SAMPLE DIAMETER, cm | 4.71 | UNIT WEIGHT, kN/$m^3$ | 24.83 |
| SAMPLE AREA, $cm^2$ | 17.39 | DRY UNIT WT., kN/$m^3$ | 24.74 |
| SAMPLE VOLUME, $cm^3$ | 166.63 | SPECIFIC GRAVITY, assumed | - |
| WET WEIGHT, g | 422.10 | VOID RATIO | - |
| DRY WEIGHT, g | 420.46 | | |

**VISUAL INSPECTION**

**FAILURE SKETCH**



### TEST RESULTS

| | | | |
|---|---|---|---|
| STRAIN AT FAILURE, % | - | COMPRESSIVE STRESS, MPa | 69.0 |
| REMARKS: | N/A | DATE: | 2011-07-14 |
| CHECKED BY: | TZ | REVIEWED BY: | JPD/JMAC |

# TABLE B2-3
# UNCONFINED COMPRESSION TEST (UC)
## ASTM D 7012-07

### SAMPLE IDENTIFICATION

| | | | |
|---|---|---|---|
| PROJECT NUMBER | 09-1111-6014 | RUN NUMBER | 2 |
| BOREHOLE NUMBER | B202-04 | SAMPLE DEPTH, m | 12.40-12.55 |

### TEST CONDITIONS

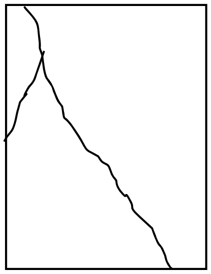
| | | | |
|---|---|---|---|
| MACHINE SPEED, mm/min | 0.00 | TYPE OF SPECIMEN | Rock Core |
| DURATION OF TEST,min | >2 <15 | L/D | 2.32 |

### SPECIMEN INFORMATION

| | | | |
|---|---|---|---|
| SAMPLE HEIGHT, cm | 10.93 | WATER CONTENT, (specimen) % | 0.09 |
| SAMPLE DIAMETER, cm | 4.72 | UNIT WEIGHT, kN/m$^3$ | 27.11 |
| SAMPLE AREA, cm$^2$ | 17.50 | DRY UNIT WT., kN/m$^3$ | 27.09 |
| SAMPLE VOLUME, cm$^3$ | 191.25 | SPECIFIC GRAVITY, assumed | - |
| WET WEIGHT, g | 528.90 | VOID RATIO | - |
| DRY WEIGHT, g | 528.42 | | |

### VISUAL INSPECTION

### FAILURE SKETCH



### TEST RESULTS

| | | | |
|---|---|---|---|
| STRAIN AT FAILURE, % | - | COMPRESSIVE STRESS, MPa | 193.2 |
| REMARKS: N/A | | DATE: | 2011-07-14 |
| CHECKED BY: TZ | | REVIEWED BY: | JPD/JMAC |

# TABLE B2-4
# UNCONFINED COMPRESSION TEST (UC)
## ASTM D 7012-07

### SAMPLE IDENTIFICATION

| | | | |
|---|---|---|---|
| PROJECT NUMBER | 09-1111-6014 | RUN NUMBER | 3 |
| BOREHOLE NUMBER | B202-09 | SAMPLE DEPTH, m | 3.23-3.38 |

### TEST CONDITIONS

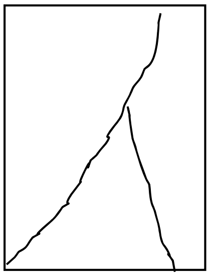
| | | | |
|---|---|---|---|
| MACHINE SPEED, mm/min | - | TYPE OF SPECIMEN | Rock Core |
| DURATION OF TEST,min | >2 <15 | L/D | 2.32 |

### SPECIMEN INFORMATION

| | | | |
|---|---|---|---|
| SAMPLE HEIGHT, cm | 11.70 | WATER CONTENT, (specimen) % | 0.07 |
| SAMPLE DIAMETER, cm | 5.05 | UNIT WEIGHT, kN/m$^3$ | 27.29 |
| SAMPLE AREA, cm$^2$ | 20.03 | DRY UNIT WT., kN/m$^3$ | 27.27 |
| SAMPLE VOLUME, cm$^3$ | 234.35 | SPECIFIC GRAVITY, assumed | - |
| WET WEIGHT, g | 652.45 | VOID RATIO | - |
| DRY WEIGHT, g | 651.98 | | |

### VISUAL INSPECTION

### FAILURE SKETCH



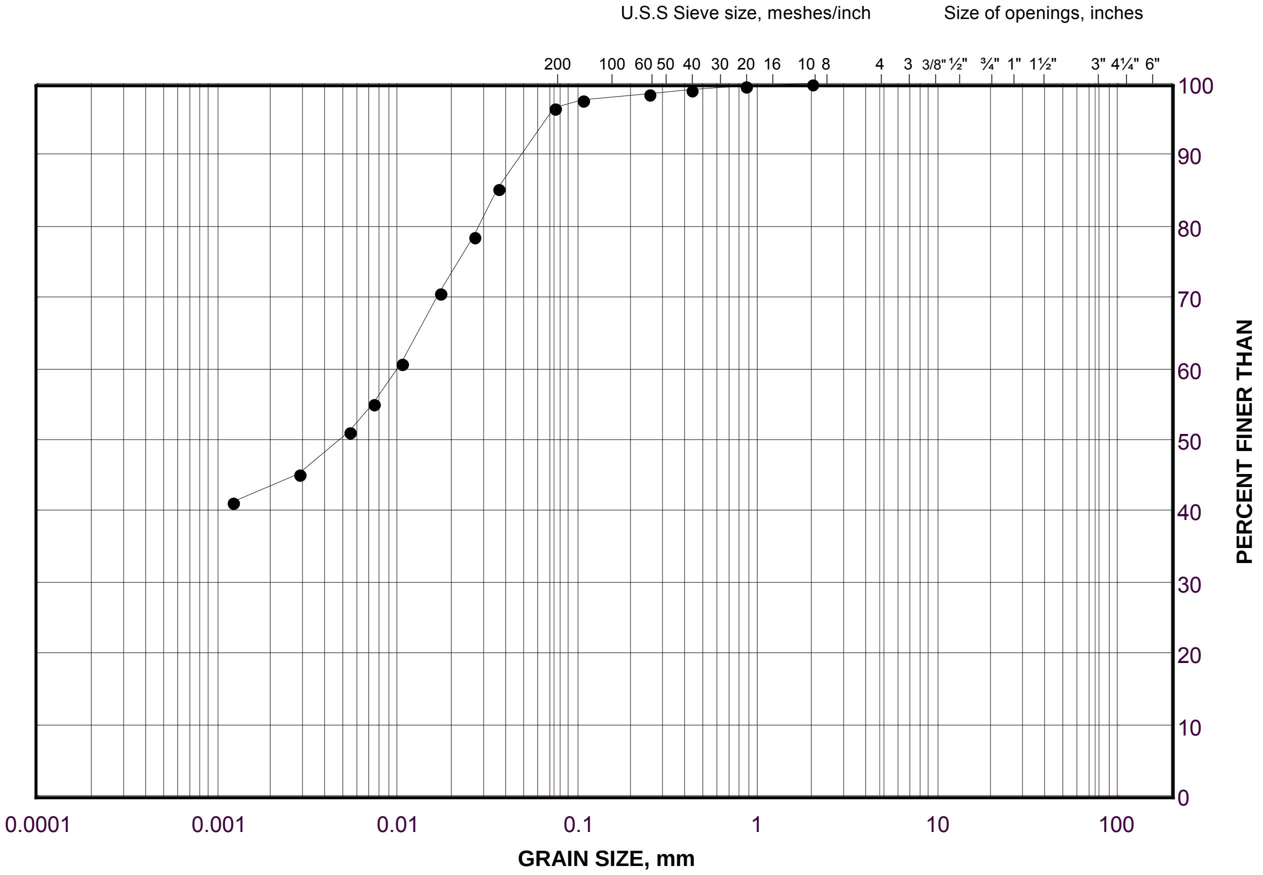
### TEST RESULTS

| | | | |
|---|---|---|---|
| STRAIN AT FAILURE, % | - | COMPRESSIVE STRESS, MPa | 175.4 |
| REMARKS: | N/A | DATE: | 4/8/2011 |
| CHECKED BY: | TZ | REVIEWED BY: | JPD/JMAC |

# GRAIN SIZE DISTRIBUTION

Silty Clay (Near Surface)
South Abutment

FIGURE B1



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

## LEGEND

| SYMBOL | BOREHOLE | SAMPLE | ELEVATION(m) |
|---|---|---|---|
| ● | B202-02 | 3 | 179.2 |

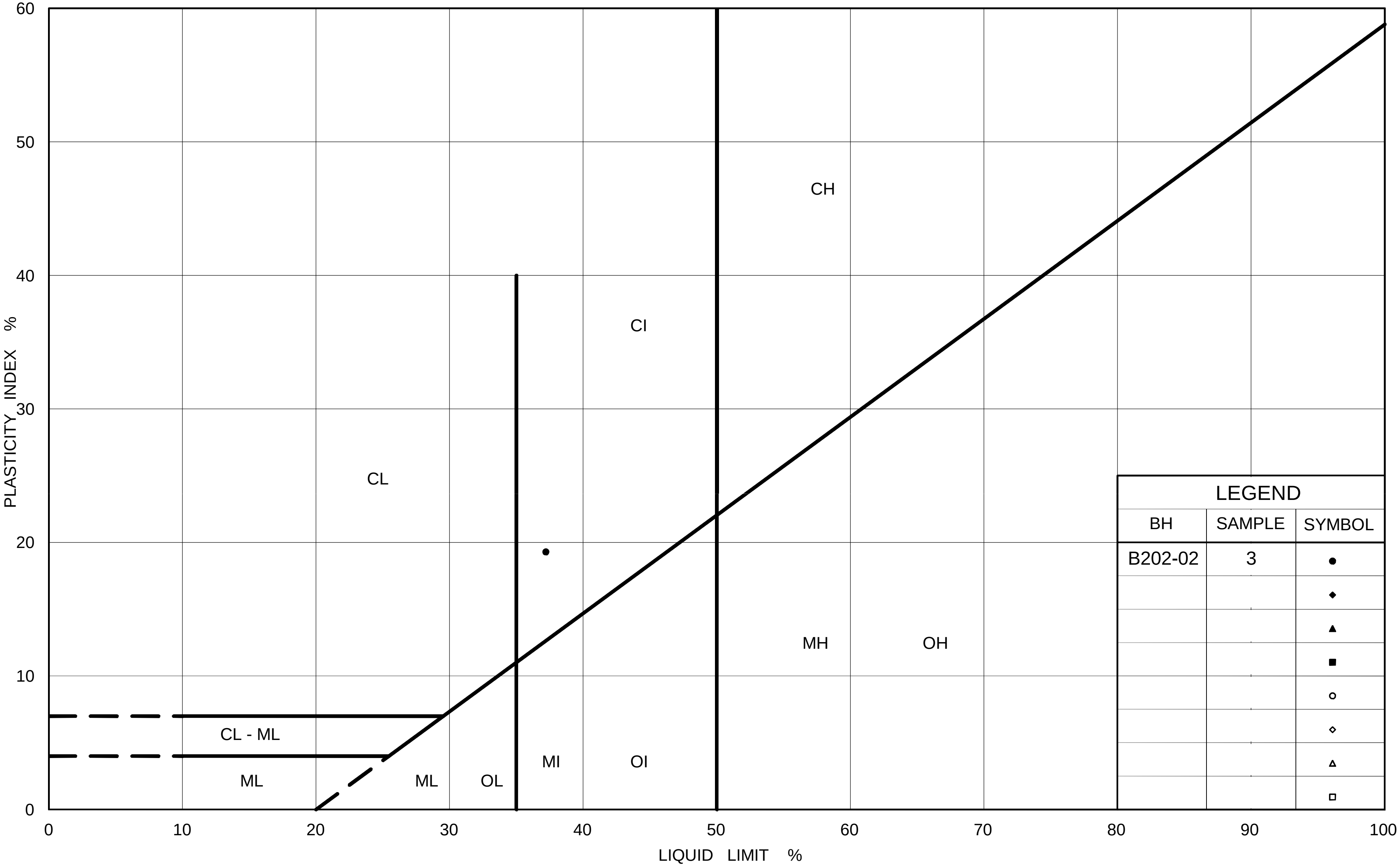
Project Number: 09-1111-6014

Checked By: TZ

**Golder Associates**

Date: 29-Nov-11

Oct 75, FF-S-21



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PLASTICITY CHART
Silty Clay (Near Surface)
South Abutment

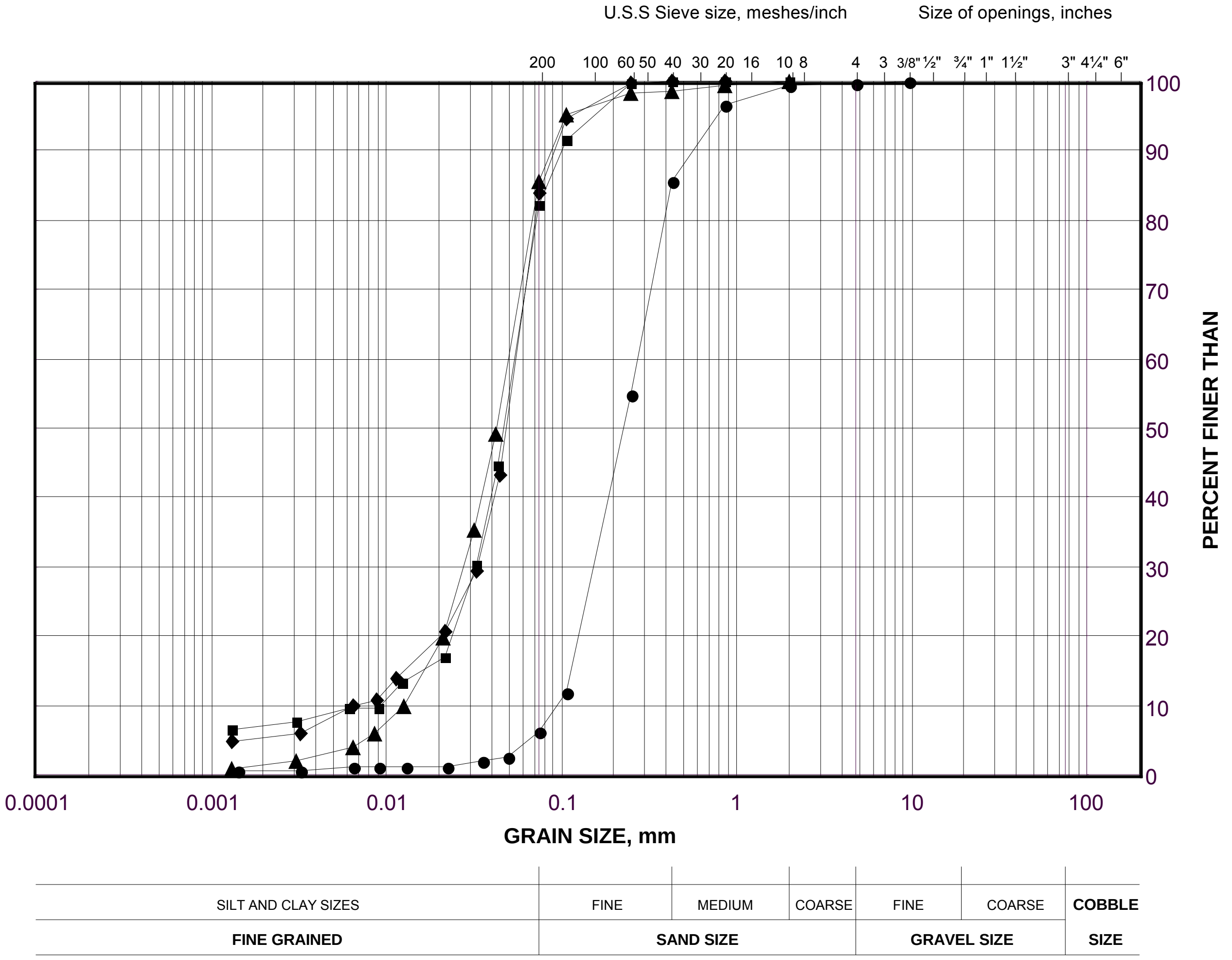
Figure No. B2

Project No. 09-1111-6014

Checked By: TZ

# GRAIN SIZE DISTRIBUTION
Sand to Silt (Upper)
South Abutment and Approach

FIGURE B3



## LEGEND

| SYMBOL | BOREHOLE | SAMPLE | ELEVATION(m) |
|---|---|---|---|
| ● | B202-01 | 4 | 177.8 |
| ■ | B202-02 | 6 | 176.1 |
| ◆ | S204-18 | 6 | 177.0 |
| ▲ | B202-02 | 8 | 173.5 |

Project Number: 09-1111-6014

Checked By: TZ

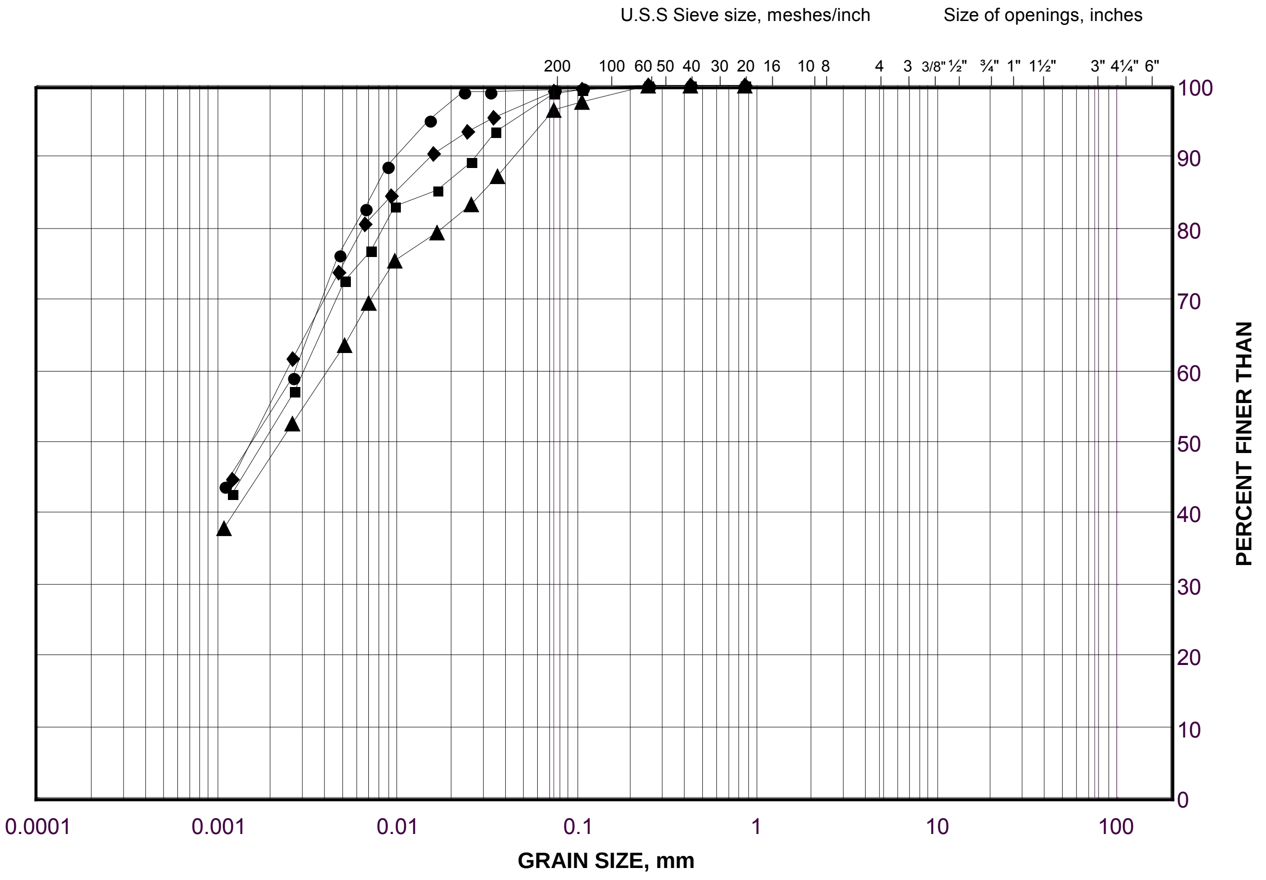
**Golder Associates**

Date: 29-Nov-11

# GRAIN SIZE DISTRIBUTION

Silty Clay to Clay (Upper)
South Abutment and Approach

FIGURE B4



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

## LEGEND

| SYMBOL | BOREHOLE | SAMPLE | ELEVATION(m) |
|---|---|---|---|
| ● | B202-01 | 13 | 165.6 |
| ■ | S204-18 | 13 | 167.5 |
| ◆ | B202-02 | 15 | 163.0 |
| ▲ | B202-01 | 9 | 171.7 |

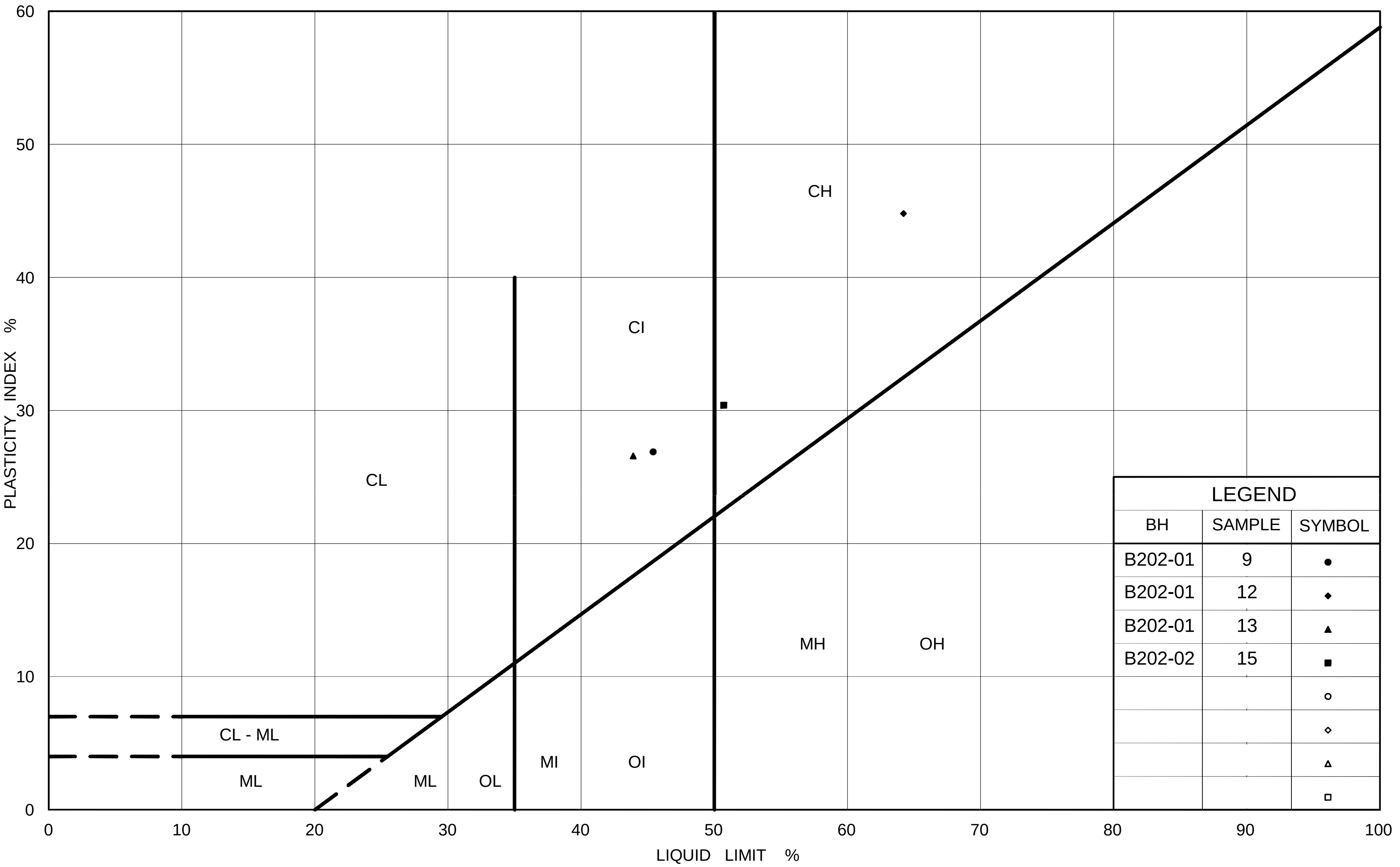
Project Number: 09-1111-6014

Checked By: TZ

**Golder Associates**

Date: 29-Nov-11

Oct 75, FF-S-21



| BH | SAMPLE | SYMBOL |
|---|---|---|
| B202-01 | 9 | ● |
| B202-01 | 12 | ◆ |
| B202-01 | 13 | ▲ |
| B202-02 | 15 | ■ |
|  |  | ○ |
|  |  | ◇ |
|  |  | △ |
|  |  | □ |

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# PLASTICITY CHART
Silty Clay to Clay (Upper)
South Abutment and Approach

Figure No. B5

Project No. 09-1111-6014

Checked By: TZ

# CONSOLIDATION TEST SUMMARY

**FIGURE B6**

**Sheet 1 of 4**

## SAMPLE IDENTIFICATION

| | | | |
|---|---|---|---|
| Project Number | 09-1111-6014 | Sample Number | 12 |
| Borehole Number | B202-01 | Sample Depth, m | 13.72-14.17 |

## TEST CONDITIONS

| | | | |
|---|---|---|---|
| Test Type | Standard | Load Duration, hr | 24 |
| Oedometer Number | 6 | | |
| Date Started | 7/14/2011 | | |
| Date Completed | 8/03/2011 | | |

## SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

| | | | |
|---|---|---|---|
| Sample Height, cm | 1.89 | Unit Weight, kN/$m^3$ | 16.63 |
| Sample Diameter, cm | 6.33 | Dry Unit Weight, kN/$m^3$ | 10.70 |
| Area, $cm^2$ | 31.48 | Specific Gravity, measured | 2.77 |
| Volume, $cm^3$ | 59.62 | Solids Height, cm | 0.746 |
| Water Content, % | 55.35 | Volume of Solids, $cm^3$ | 23.49 |
| Wet Mass, g | 101.10 | Volume of Voids, $cm^3$ | 36.13 |
| Dry Mass, g | 65.08 | Degree of Saturation, % | 99.7 |

## TEST COMPUTATIONS

| Pressure kPa | Corr. Height cm | Void Ratio | Average Height cm | $t_{90}$ sec | $c_v$ $cm^2/s$ | $m_v$ $m^2/kN$ | k cm/s |
|---|---|---|---|---|---|---|---|
| 0.00 | 1.894 | 1.538 | 1.894 | | | | |
| 5.04 | 1.894 | 1.538 | 1.894 | 2 | 3.80E-01 | | |
| 9.99 | 1.893 | 1.537 | 1.894 | 330 | 2.30E-03 | 7.47E-05 | 1.69E-08 |
| 20.49 | 1.889 | 1.531 | 1.891 | 167 | 4.54E-03 | 2.16E-04 | 9.62E-08 |
| 40.01 | 1.880 | 1.519 | 1.885 | 305 | 2.47E-03 | 2.38E-04 | 5.76E-08 |
| 78.94 | 1.860 | 1.492 | 1.870 | 609 | 1.22E-03 | 2.81E-04 | 3.35E-08 |
| 156.84 | 1.810 | 1.426 | 1.835 | 306 | 2.33E-03 | 3.33E-04 | 7.62E-08 |
| 311.51 | 1.617 | 1.166 | 1.714 | 923 | 6.74E-04 | 6.61E-04 | 4.37E-08 |
| 622.54 | 1.449 | 0.941 | 1.533 | 789 | 6.31E-04 | 2.86E-04 | 1.77E-08 |
| 1246.59 | 1.324 | 0.774 | 1.386 | 519 | 7.85E-04 | 1.05E-04 | 8.11E-09 |
| 2494.73 | 1.223 | 0.639 | 1.273 | 383 | 8.98E-04 | 4.27E-05 | 3.76E-09 |
| 1246.59 | 1.238 | 0.659 | 1.230 | | | | |
| 311.51 | 1.269 | 0.700 | 1.253 | | | | |
| 78.94 | 1.313 | 0.759 | 1.291 | | | | |
| 20.49 | 1.354 | 0.815 | 1.333 | | | | |
| 5.04 | 1.385 | 0.855 | 1.369 | | | | |

Note:
k calculated using cv based on $t_{90}$ values.
Specimen swelled under 5kPa

## SAMPLE DIMENSIONS AND PROPERTIES - FINAL

| | | | |
|---|---|---|---|
| Sample Height, cm | 1.38 | Unit Weight, kN/$m^3$ | 19.38 |
| Sample Diameter, cm | 6.33 | Dry Unit Weight, kN/$m^3$ | 14.64 |
| Area, $cm^2$ | 31.48 | Specific Gravity, measured | 2.77 |
| Volume, $cm^3$ | 43.59 | Solids Height, cm | 0.746 |
| Water Content, % | 32.34 | Volume of Solids, $cm^3$ | 23.49 |
| Wet Mass, g | 86.13 | Volume of Voids, $cm^3$ | 20.09 |
| Dry Mass, g | 65.08 | | |

Prepared By: LH

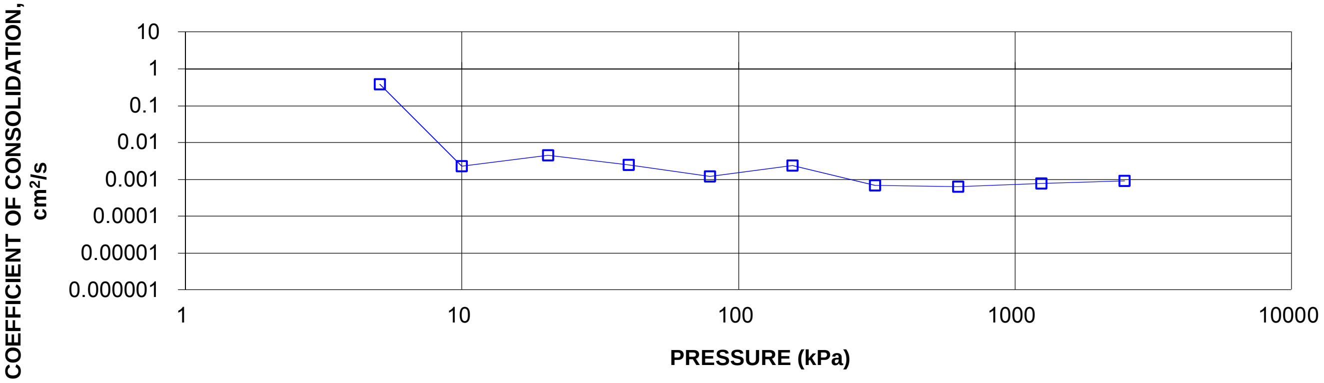
**Golder Associates**

Checked By: TZ

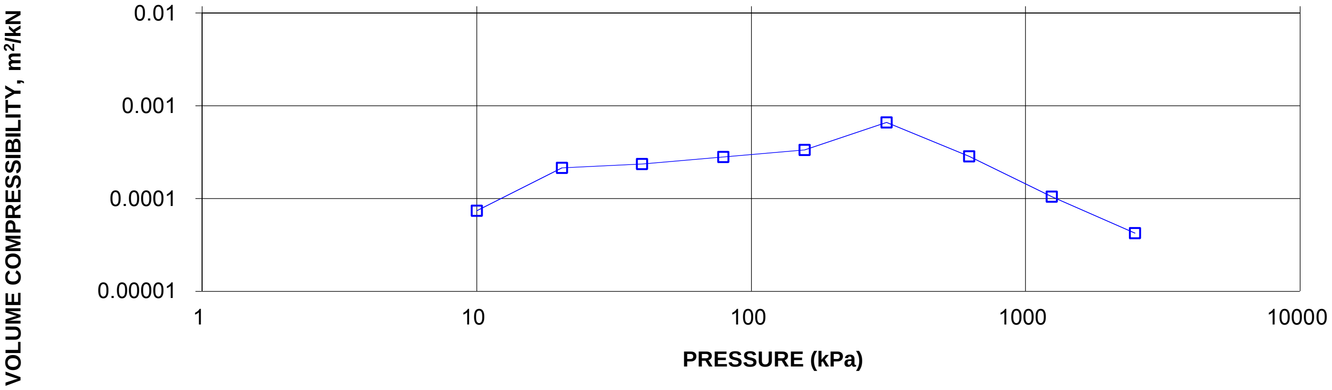
# CONSOLIDATION TEST SUMMARY

**FIGURE B6**

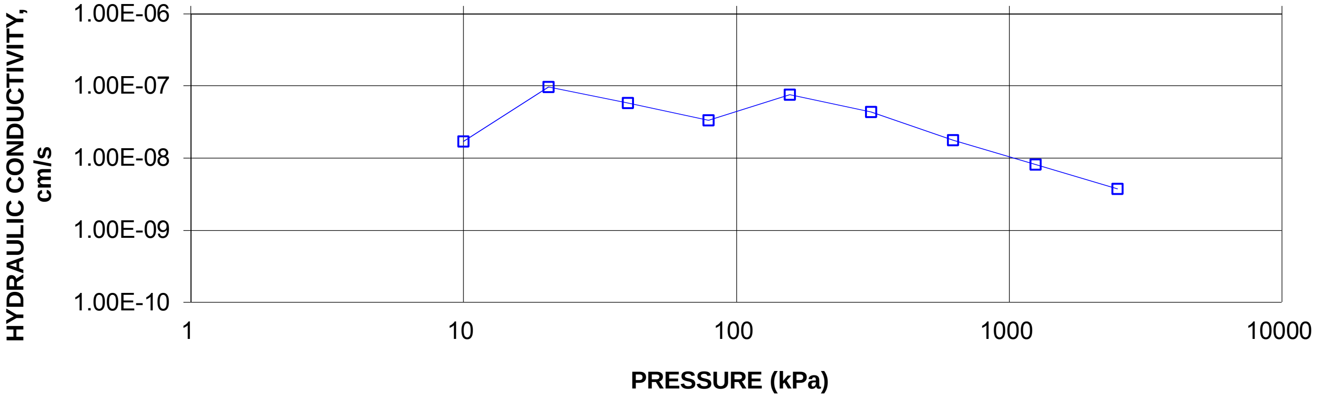
**Sheet 2 of 4**



**CONSOLIDATION TEST**
**$C_V$ cm$^2$/s VS PRESSURE (kPa)**
**BH B202-01 SA 12**



**CONSOLIDATION TEST**
**$M_V$ m$^2$/kN vs PRESSURE (kPa)**
**BH B202-01 SA 12**



**CONSOLIDATION TEST**
**HYDRAULIC CONDUCTIVITY vs PRESSURE**
**BH B202-01 SA 12**

Project No. 09-1111-6014

Prepared By: LH

**Golder Associates**

Checked By: TZ

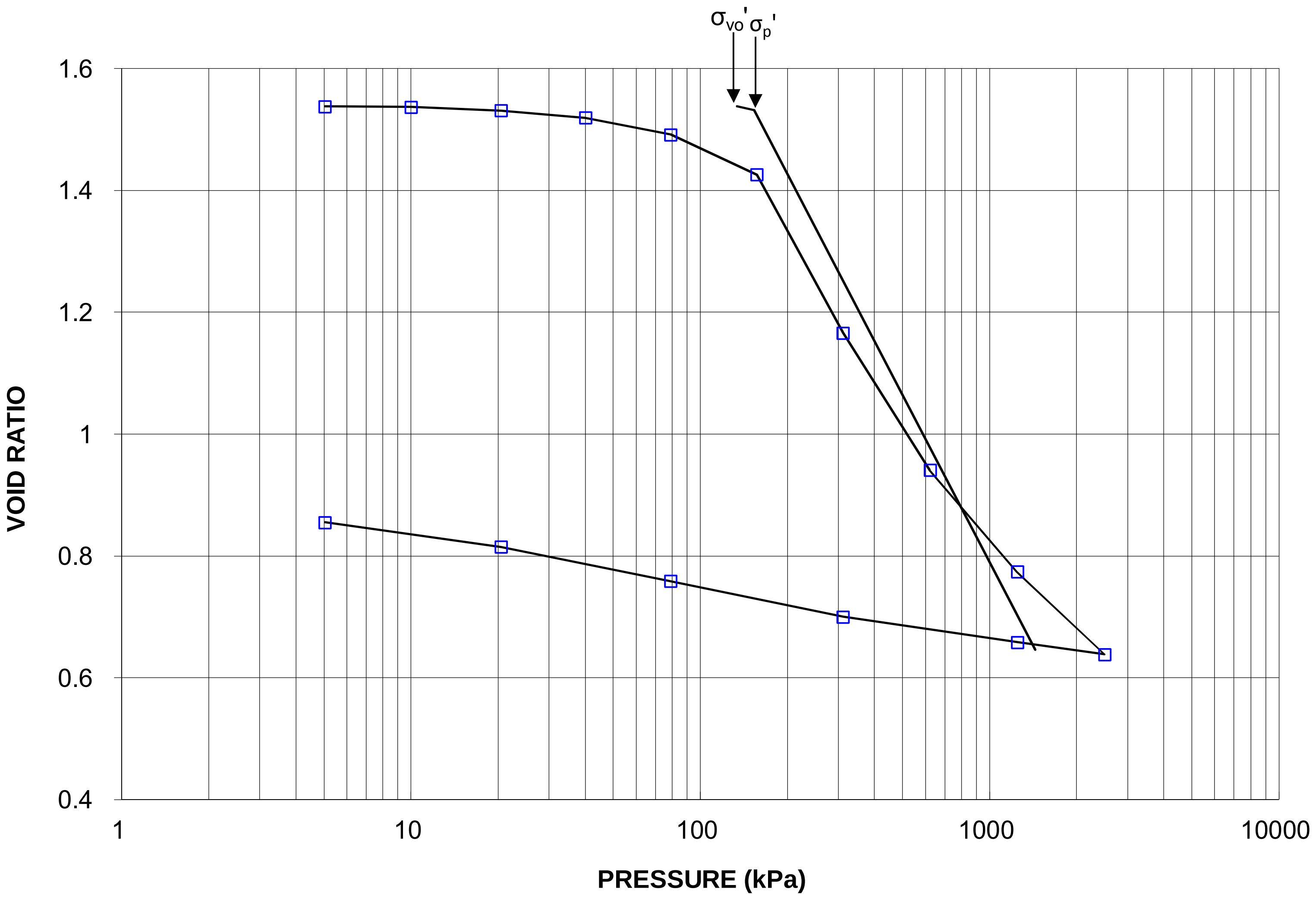
Project No. 09-1111-6014

Prepared By: LH/TZ

**Golder Associates**

Checked By: TZ

# CONSOLIDATION TEST
## VOID RATIO vs PRESSURE
## BH B202-01 SA 12



**CONSOLIDATION TEST VOID RATIO VS LOG PRESSURE**

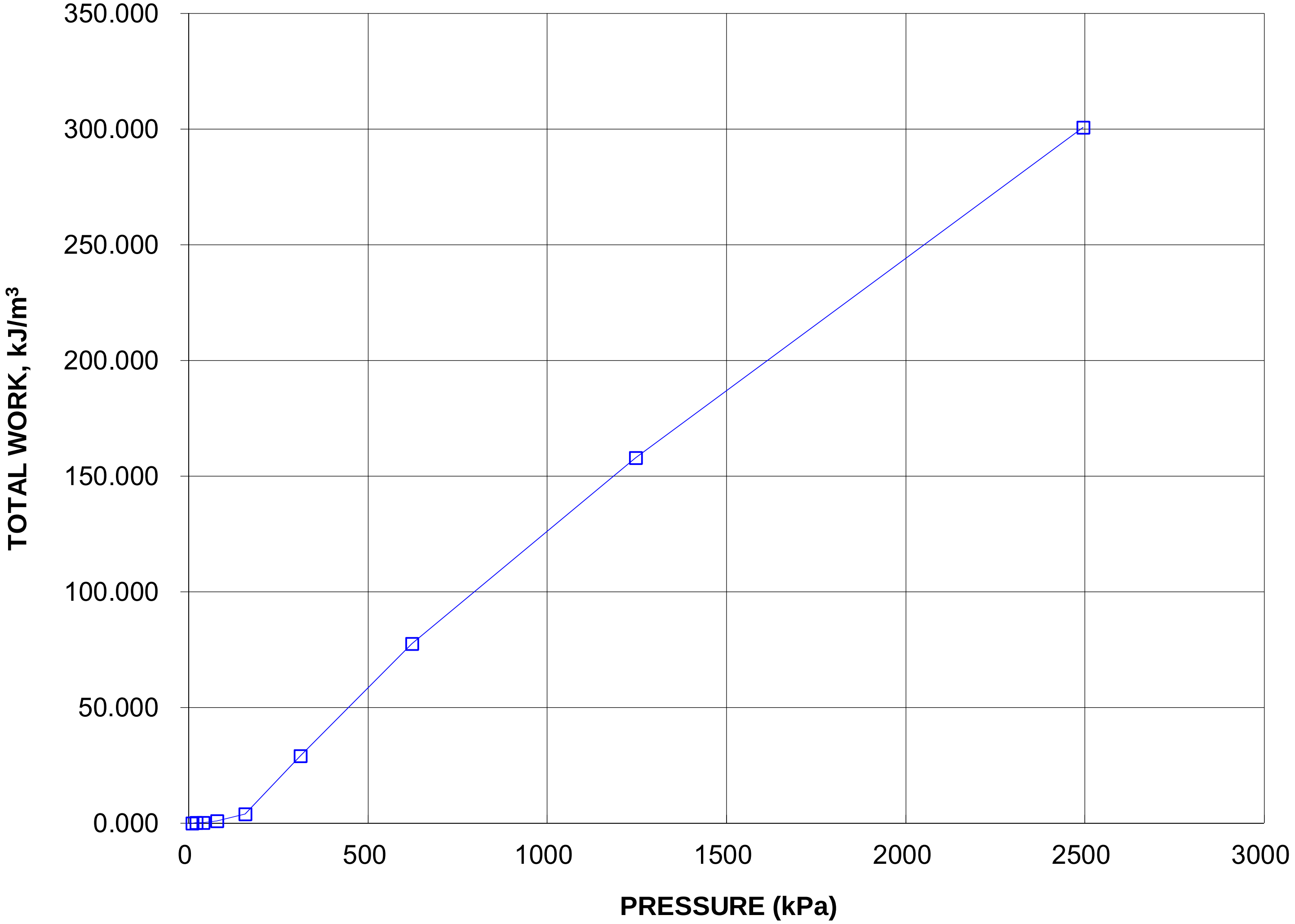
**FIGURE B6**
**Sheet 3 of 4**

Project No. 09-1111-6014
Prepared By: LH

**Golder Associates**

Checked By: TZ

## CONSOLIDATION TEST
## TOTAL WORK, kJ/m$^3$ vs PRESSURE
## BH B202-21 SA 12

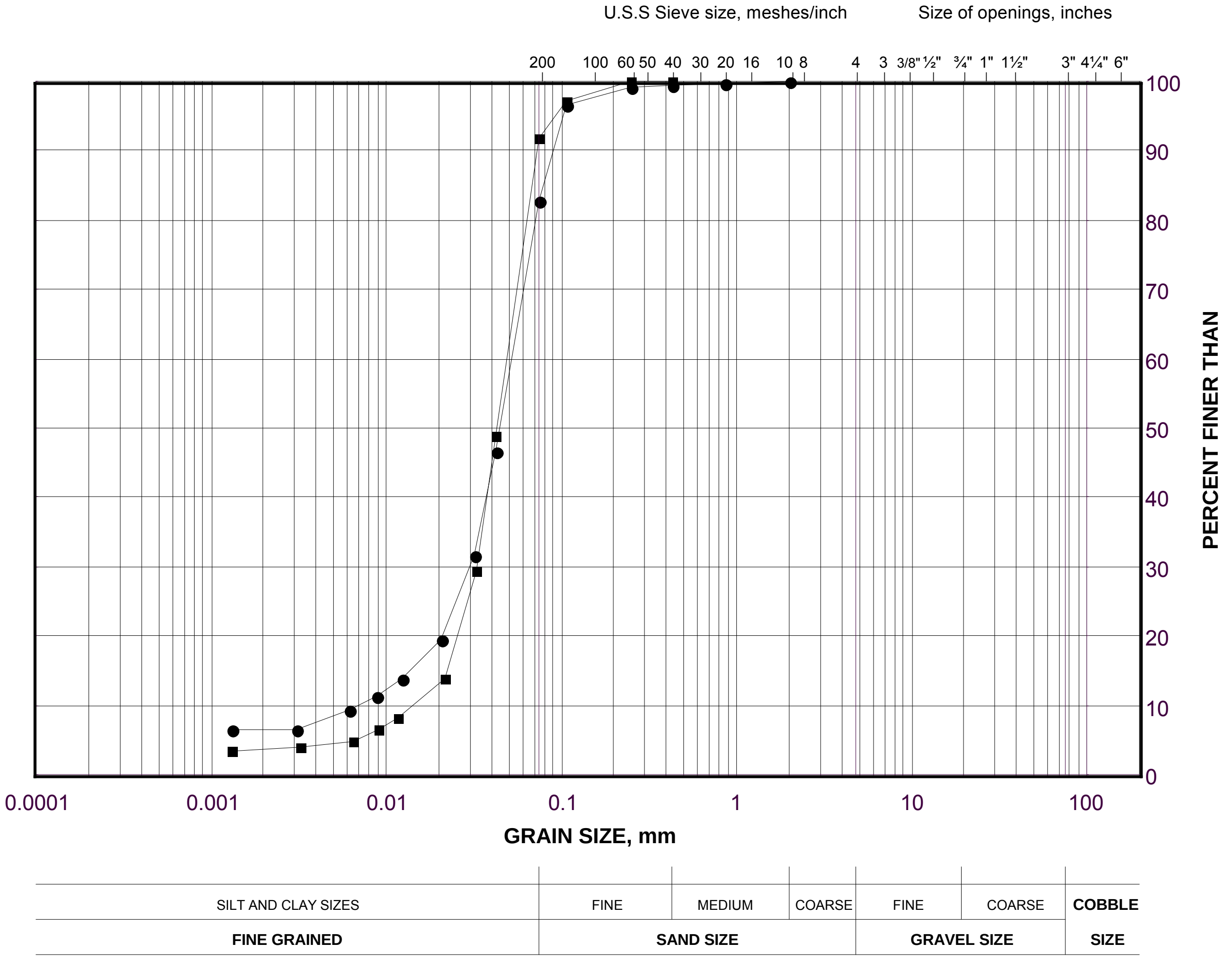


**CONSOLIDATION TEST**
**TOTAL WORK VS PRESSURE**

**FIGURE B6**
**Sheet 4 of 4**

# GRAIN SIZE DISTRIBUTION
Silt Interlayers
South Abutment and Approach

FIGURE B7



## LEGEND

| SYMBOL | BOREHOLE | SAMPLE | ELEVATION(m) |
|---|---|---|---|
| ● | B202-02 | 11 | 170.0 |
| ■ | S204-18 | 17 | 159.5 |

Project Number: 09-1111-6014

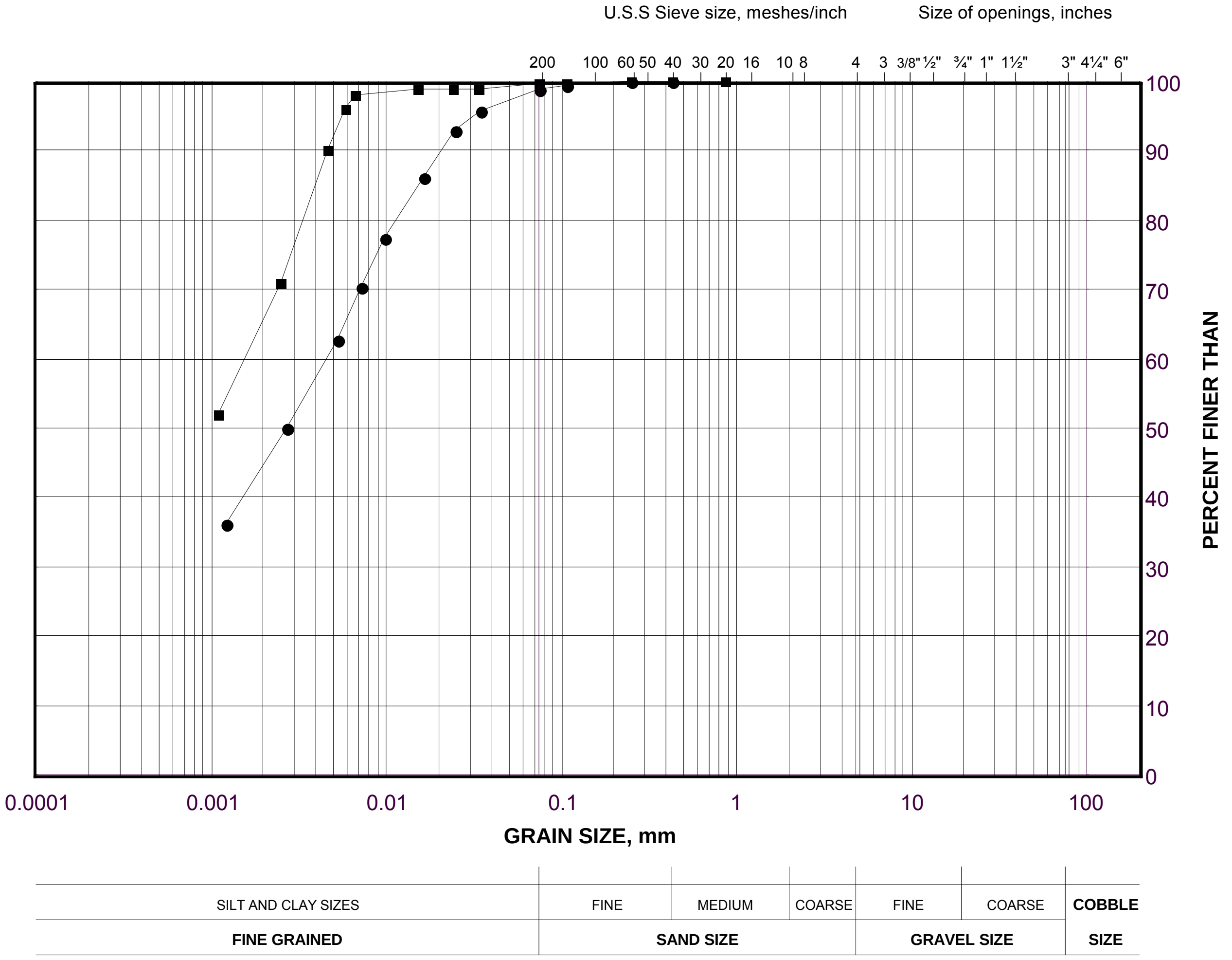
Checked By: TZ

**Golder Associates**

Date: 29-Feb-12

# GRAIN SIZE DISTRIBUTION
Silty Clay to Clay (Lower)
South Abutment and Approach

FIGURE B8



## LEGEND

| SYMBOL | BOREHOLE | SAMPLE | ELEVATION(m) |
|---|---|---|---|
| ● | S204-18 | 20 | 150.8 |
| ■ | B202-02 | 21 | 144.7 |

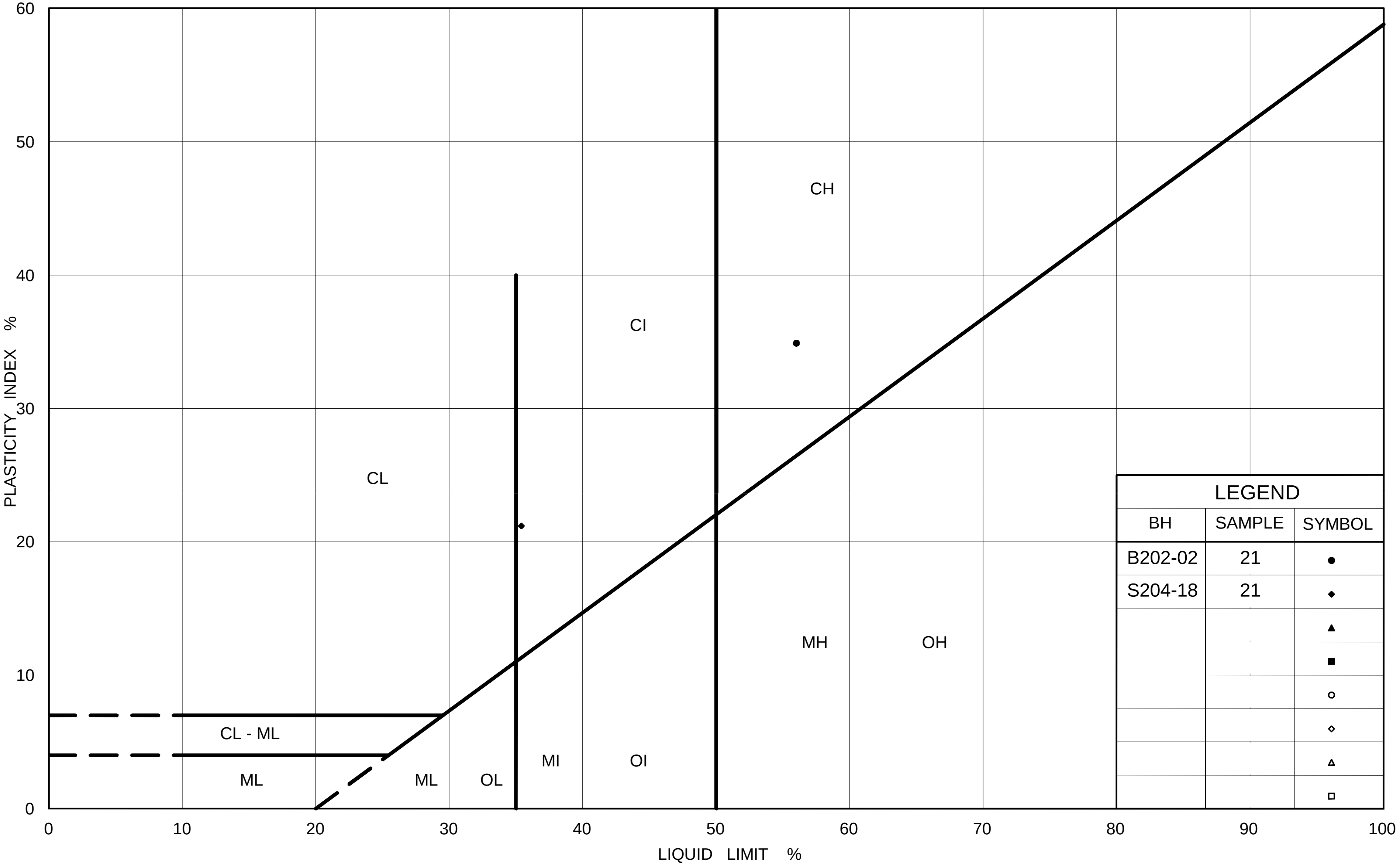
Project Number: 09-1111-6014

Checked By: TZ

**Golder Associates**

Date: 29-Nov-11

Oct 75, FF-S-21



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# PLASTICITY CHART
Silty Clay to Clay (Lower)
South Abutment and Approach

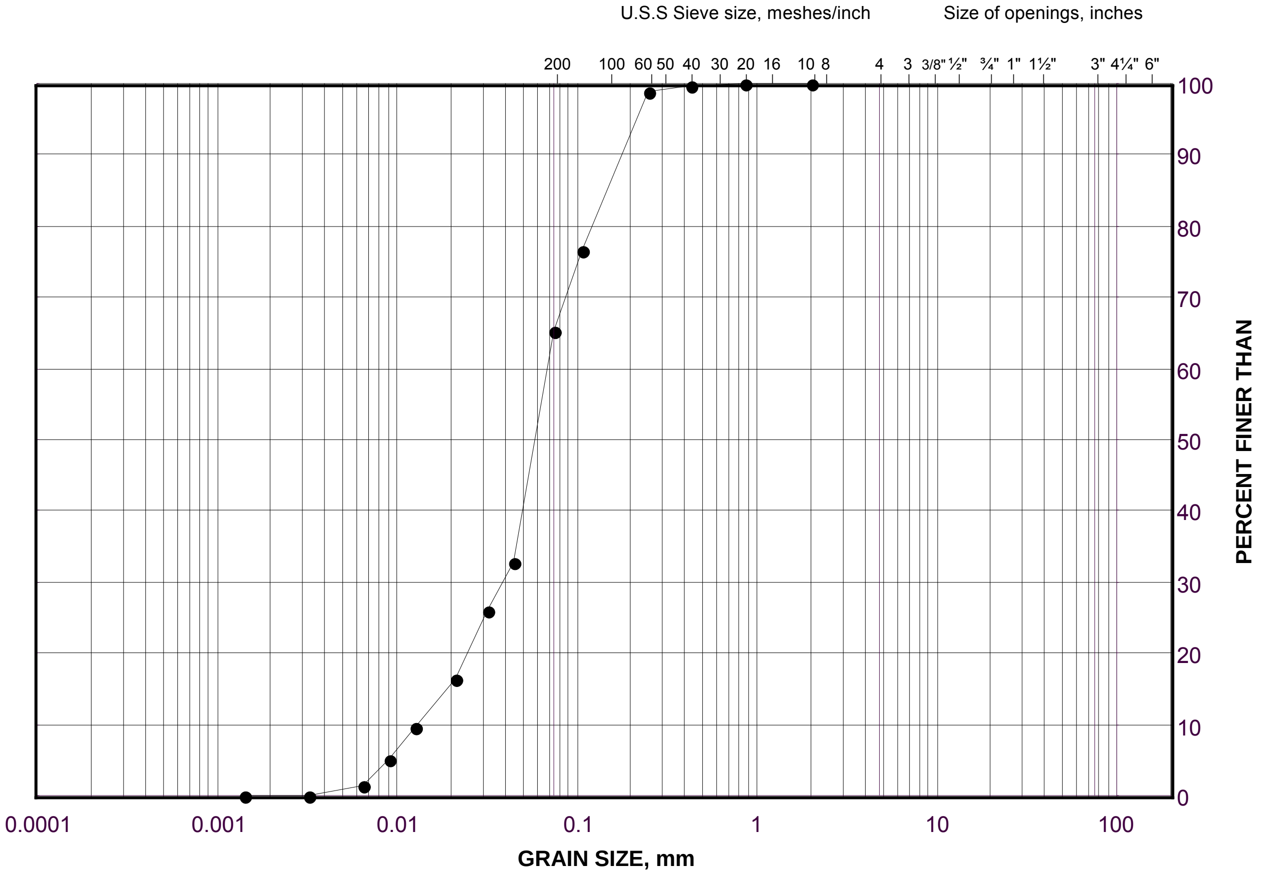
Figure No. B9

Project No. 09-1111-6014

Checked By: TZ

# GRAIN SIZE DISTRIBUTION
Sand and Silt (Lower)
South Approach

FIGURE B10



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

## LEGEND

| SYMBOL | BOREHOLE | SAMPLE | ELEVATION(m) |
|---|---|---|---|
| ● | S204-18 | 23 | 141.6 |

Project Number: 09-1111-6014

Checked By: TZ

**Golder Associates**

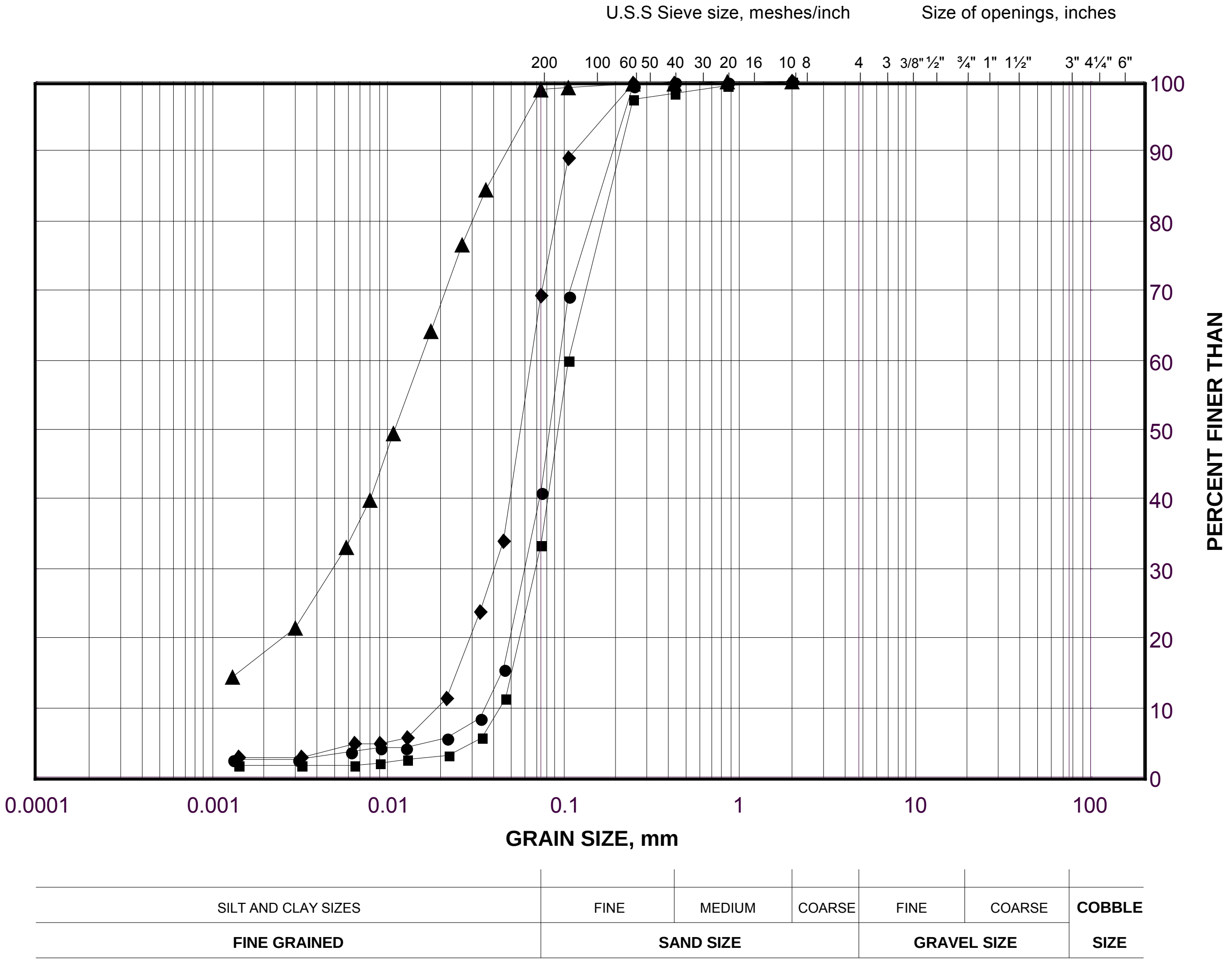
Date: 29-Nov-11

# GRAIN SIZE DISTRIBUTION

Sand and Silt to Silt

Centre Pier

FIGURE B11



## LEGEND

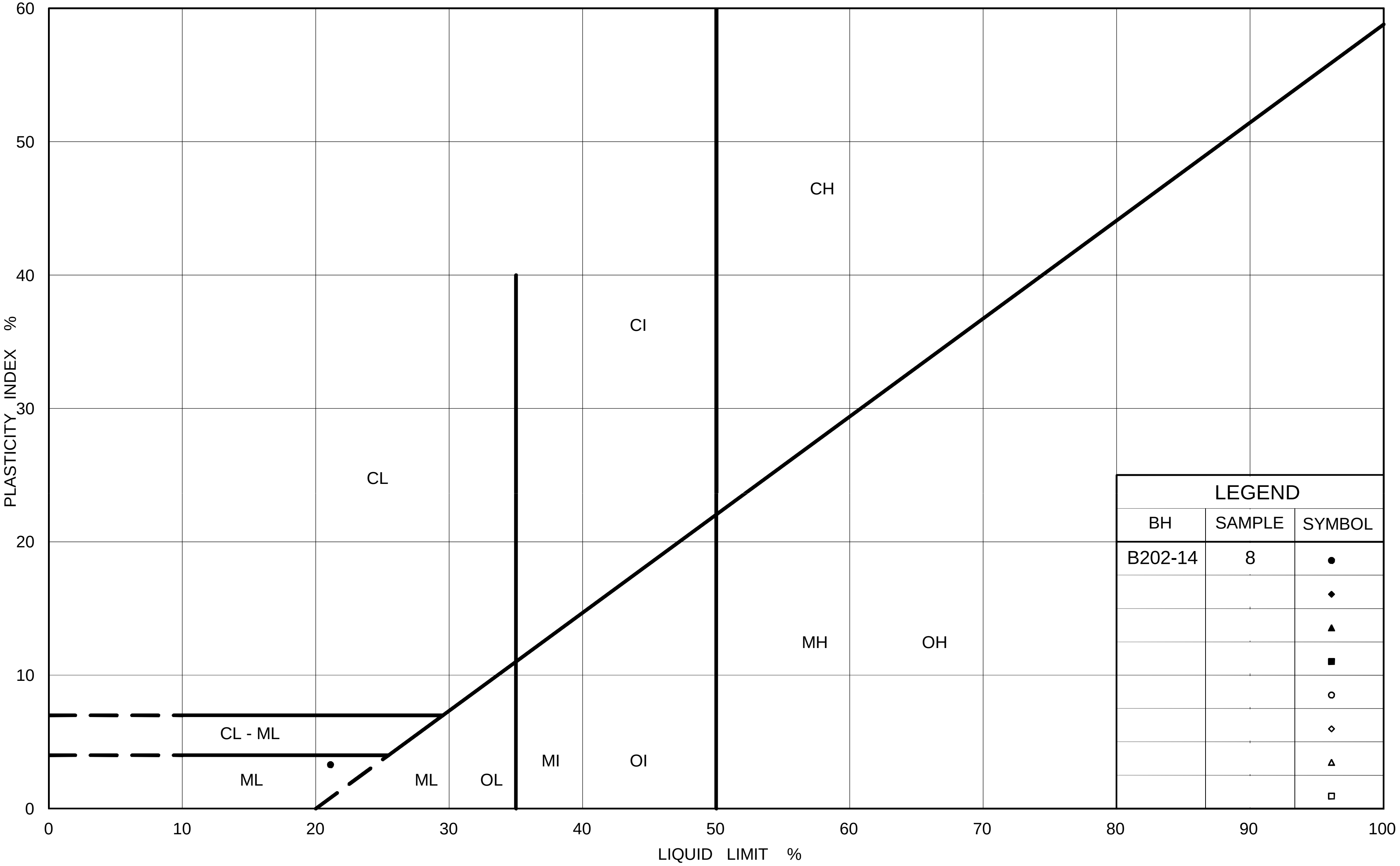
| SYMBOL | BOREHOLE | SAMPLE | ELEVATION(m) |
|---|---|---|---|
| ● | B202-14 | 4 | 176.1 |
| ■ | B202-15 | 5 | 175.1 |
| ◆ | B202-03 | 7 | 174.3 |
| ▲ | B202-14 | 8 | 171.5 |

Project Number: 09-1111-6014

Checked By: TZ

**Golder Associates**

Date: 29-Nov-11



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## PLASTICITY CHART
Silt
Centre Pier

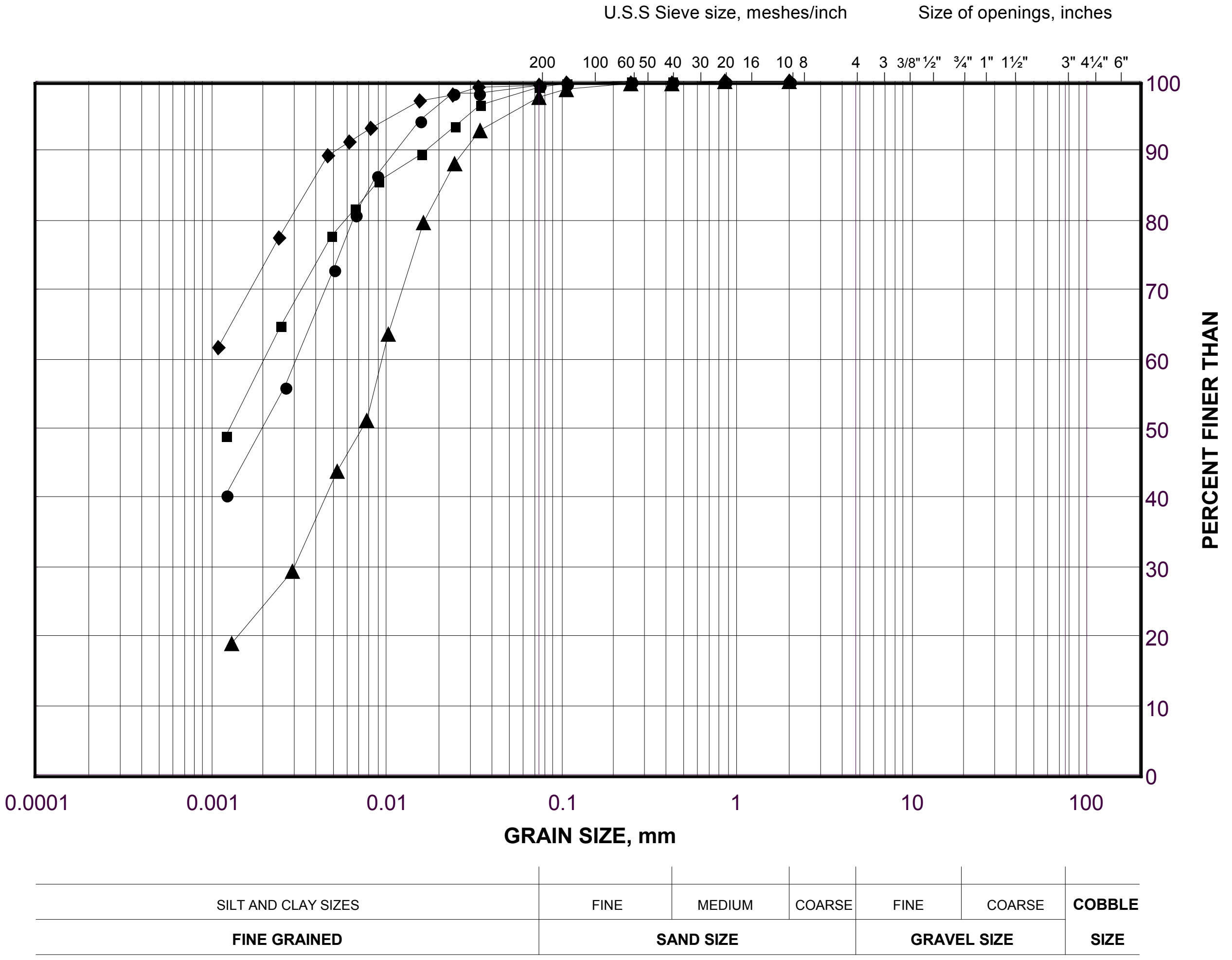
Figure No. B12

Project No. 09-1111-6014

Checked By: TZ

# GRAIN SIZE DISTRIBUTION
Clayey Silt to Clay
Centre Pier

FIGURE B13



## LEGEND

| SYMBOL | BOREHOLE | SAMPLE | ELEVATION(m) |
|---|---|---|---|
| ● | B202-03 | 12 | 166.7 |
| ■ | B202-03 | 15 | 160.6 |
| ◆ | B202-15 | 8 | 168.2 |
| ▲ | B202-03 | 9 | 171.2 |

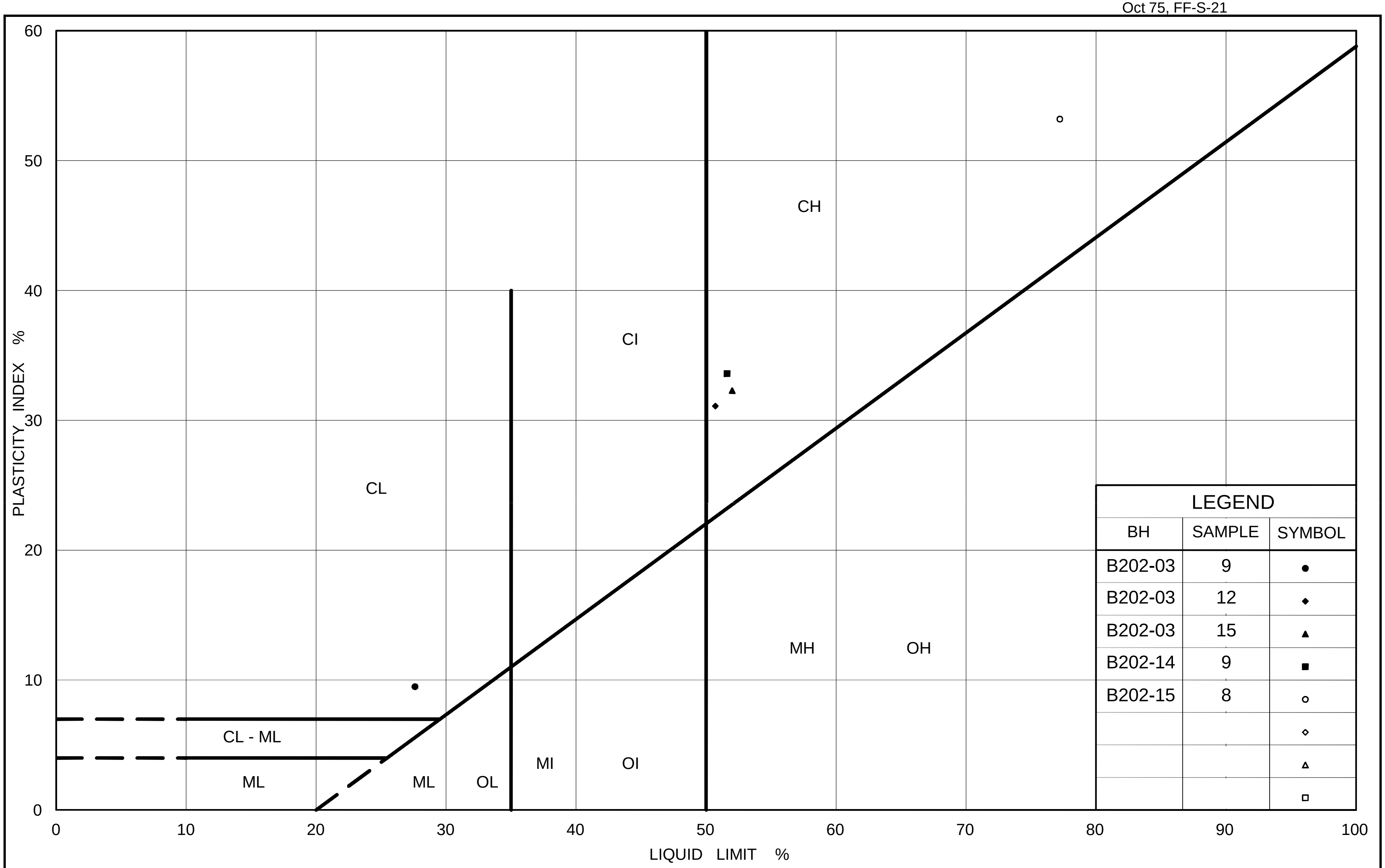
Project Number: 09-1111-6014

Checked By: TZ

**Golder Associates**

Date: 29-Feb-12

Oct 75, FF-S-21



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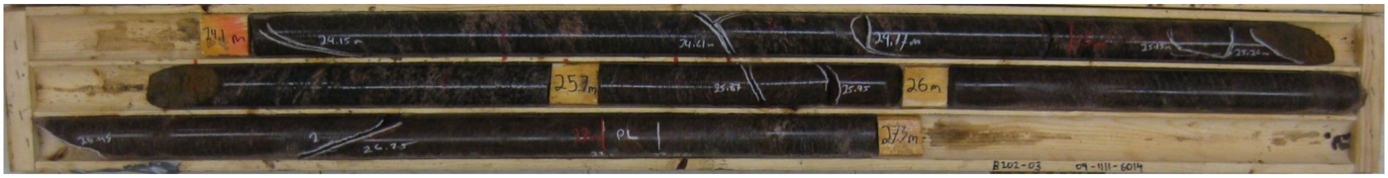
# PLASTICITY CHART
## Clayey Silt to Clay
## Centre Pier

Figure No. B14

Project No. 09-1111-6014

Checked By: TZ

**Borehole B202-03**



Box 1: 24.11 m – 27.34 m

0 m 0.25 m 0.5 m 0.75 m 1.0 m 1.25 m 1.5 m

0 ft 1 ft 2 ft 3 ft 4 ft 5 ft

Scale

| | |
|---|---|
| PROJECT | **Still River Bridge (NBL) Structure**<br>**Highway 69 Four-Laning**<br>**GWP 5404-05-00; WP 5139-08-01** |
| TITLE | **Bedrock Core Photograph – Centre Pier** |

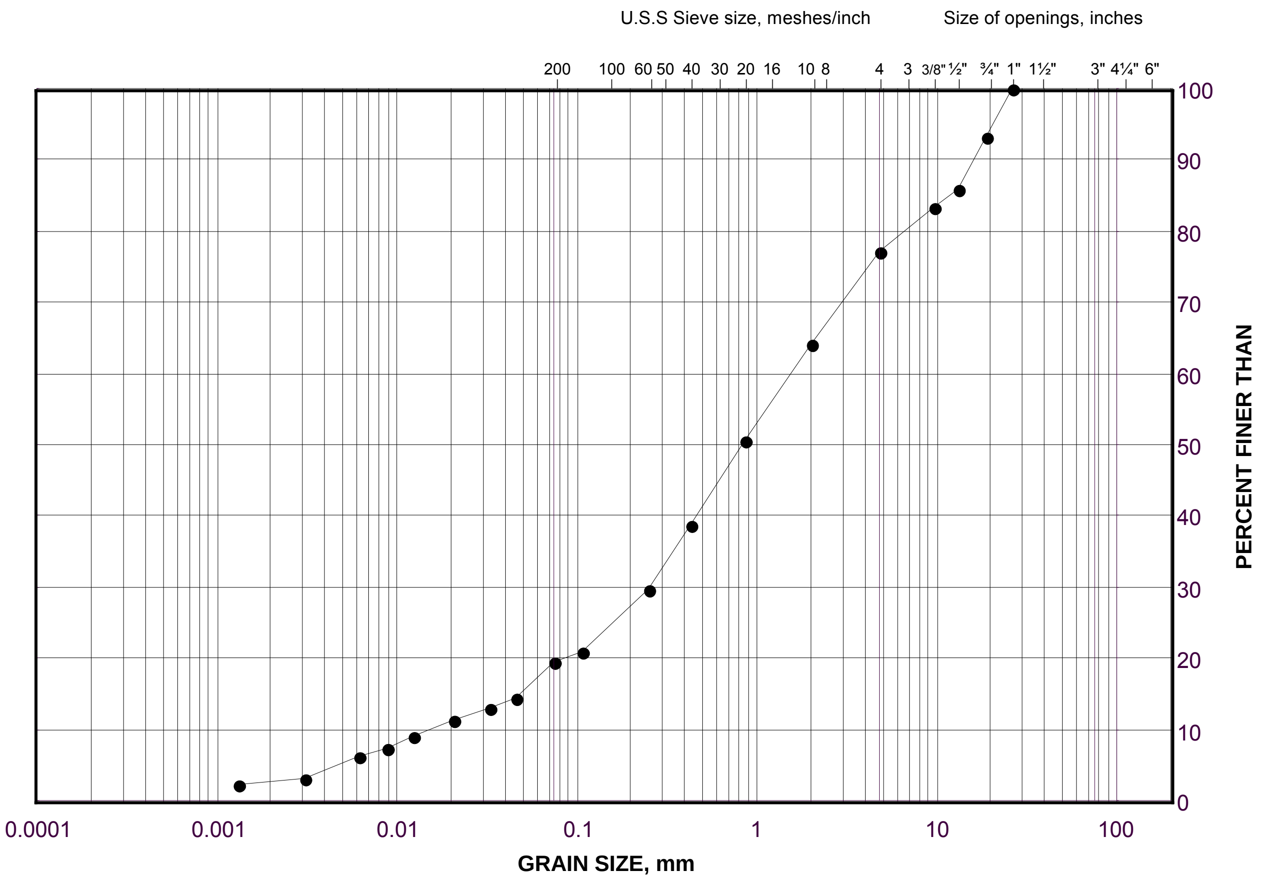
Golder Associates

| | | | | |
|---|---|---|---|---|
| PROJECT No. | 09-1111-6014 | | FILE No. ---- | |
| DESIGN | TZ | | SCALE NTS | REV. |
| CADD | -- | | **FIGURE B15** | |
| CHECK | TZ | | | |
| REVIEW | CN | | | |

REVISION DATE: April 13, 2011 BY: AT Project: 09-1111-6014

# GRAIN SIZE DISTRIBUTION
Gravelly Sand
North Abutment (Two-Span Bridge)

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) |
|---|---|---|---|
| ● | B202-06 | 1B | 183.5 |

Project Number: 09-1111-6014

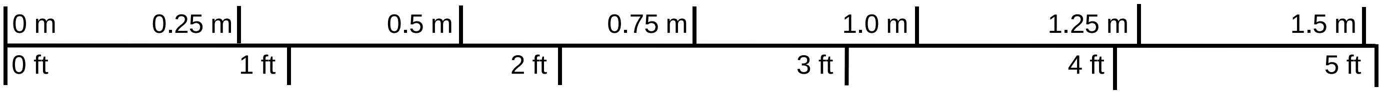
Checked By: TZ

**Golder Associates**

Date: 29-Nov-11

**Borehole B202-07**

Box 1: 0.58 m – 4.17 m



Scale

| | |
|---|---|
| PROJECT | **Still River Bridge (NBL) Structure**<br>**Highway 69 Four-Laning**<br>**GWP 5404-05-00; WP 5139-08-01** |
| TITLE | **Cobbles/Boulders and Bedrock Core Photograph – North Abutment (Two-Span Bridge)** |

| | | | | | |
|---|---|---|---|---|---|
| PROJECT No. | 09-1111-6014 | | FILE No. | ---- | |
| DESIGN | TZ | | SCALE | NTS | REV. |
| CADD | -- | | **FIGURE B17** | | |
| CHECK | TZ | | | | |
| REVIEW | CN | | | | |

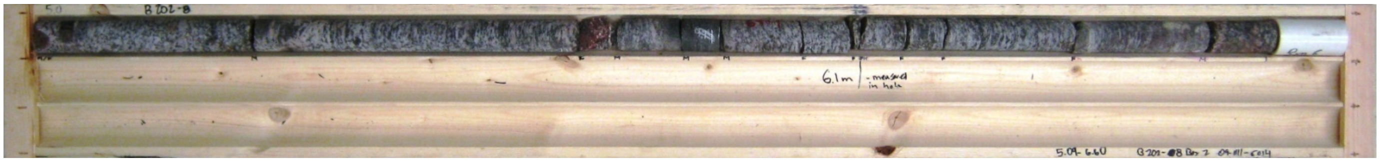
Golder Associates

REVISION DATE: April 13, 2011 BY: AT Project: 09-1111-6014

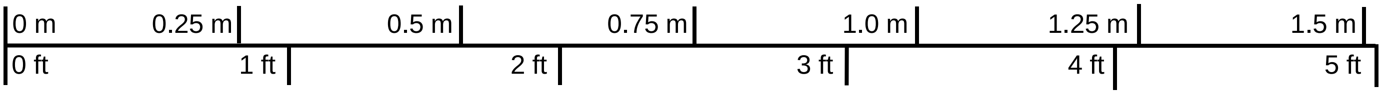
**Borehole B202-08**

Box 1: 0.33 m – 0.86 m; 1.19 m – 1.89 m; 2.00 m – 5.09 m

**Borehole B202-08**



Box 2: 5.09 m – 6.60 m



Scale

PROJECT
**Still River Bridge (NBL) Structure**
**Highway 69 Four-Laning**
**GWP 5404-05-00; WP 5139-08-01**

TITLE
**Cobbles/Boulders and Bedrock Core**
**Photograph – North Abutment**
**(Two-Span Bridge)**

Golder Associates

| | | | |
|---|---|---|---|
| PROJECT No. | 09-1111-6014 | FILE No. | ---- |
| DESIGN | TZ | SCALE | NTS REV. |
| CADD | -- | | |
| CHECK | TZ | **FIGURE B18** | |
| REVIEW | CN | | |

REVISION DATE: April 13, 2011 BY: AT Project: 09-1111-6014

**Borehole B202-09**

Box 1: 0.27 m – 3.70 m

| 0 m | 0.25 m | 0.5 m | 0.75 m | 1.0 m | 1.25 m | 1.5 m |
|---|---|---|---|---|---|---|

| 0 ft | 1 ft | 2 ft | 3 ft | 4 ft | 5 ft |
|---|---|---|---|---|---|

Scale

PROJECT

**Still River Bridge (NBL) Structure**
**Highway 69 Four-Laning**
**GWP 5404-05-00; WP 5139-08-01**

TITLE

## Bedrock Core Photograph – North Abutment (Two-Span Bridge)

Golder Associates

| | | | | | |
|---|---|---|---|---|---|
| PROJECT No. | 09-1111-6014 | | FILE No. | ---- | |
| DESIGN | TZ | | SCALE | NTS | REV. |
| CADD | -- | | | | |
| CHECK | TZ | | **FIGURE B19** | | |
| REVIEW | CN | | | | |

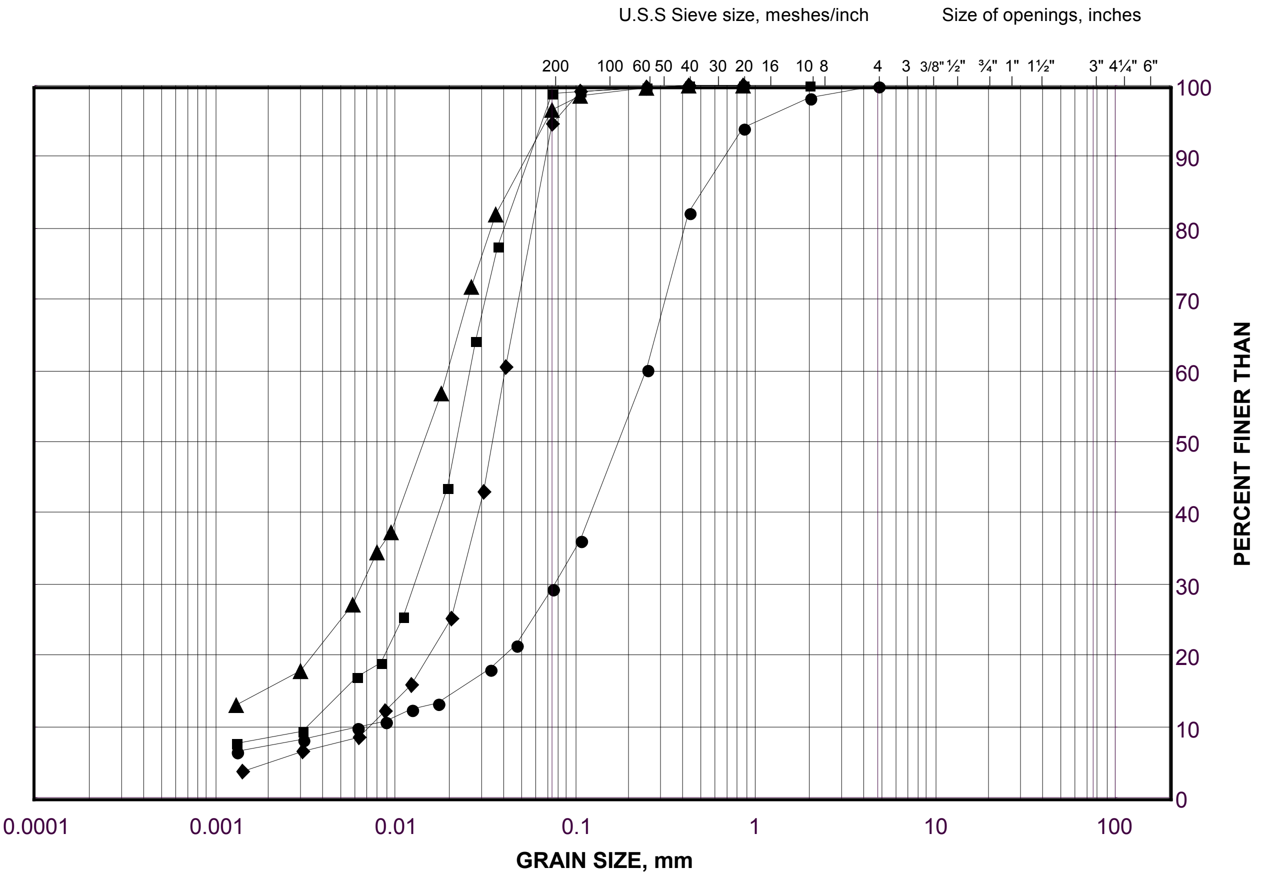
REVISION DATE: April 13, 2011 BY: AT Project: 09-1111-6014

# GRAIN SIZE DISTRIBUTION

Silty Sand to Silt

North Abutment (One-Spand Bridge)

FIGURE B20



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

## LEGEND

| SYMBOL | BOREHOLE | SAMPLE | ELEVATION(m) |
|---|---|---|---|
| ● | B202-12 | 3 | 177.7 |
| ■ | B202-04 | 5 | 176.4 |
| ◆ | B202-13 | 5 | 176.4 |
| ▲ | B202-13 | 7 | 174.8 |

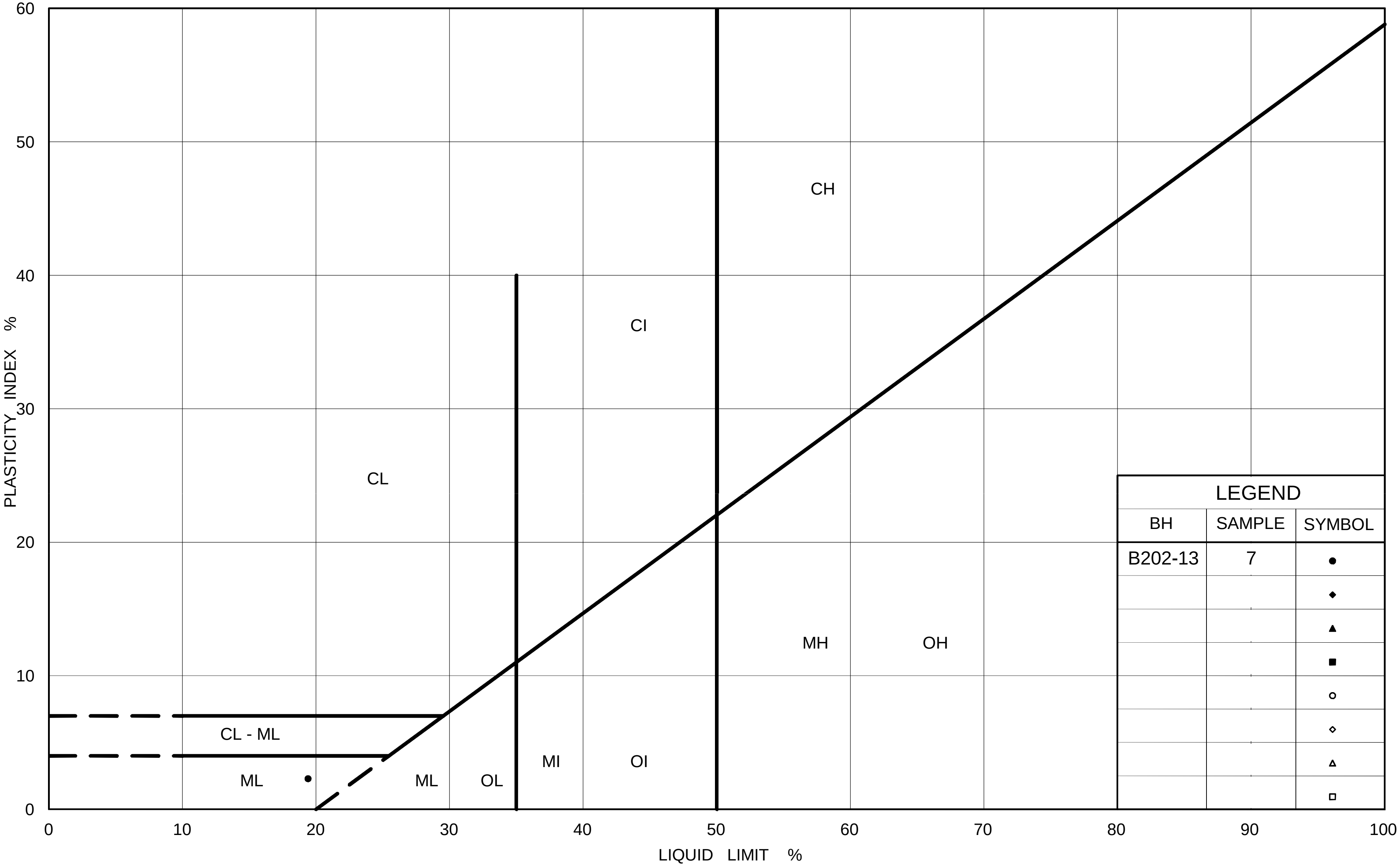
Project Number: 09-1111-6014,09-1111-0018

Checked By: TZ

**Golder Associates**

Date: 29-Feb-12

Oct 75, FF-S-21



Ministry of Transportation

Ontario

# PLASTICITY CHART
## Silt
## North Abutment (One-Span Bridge)

Figure No. B21

Project No. 09-1111-6014

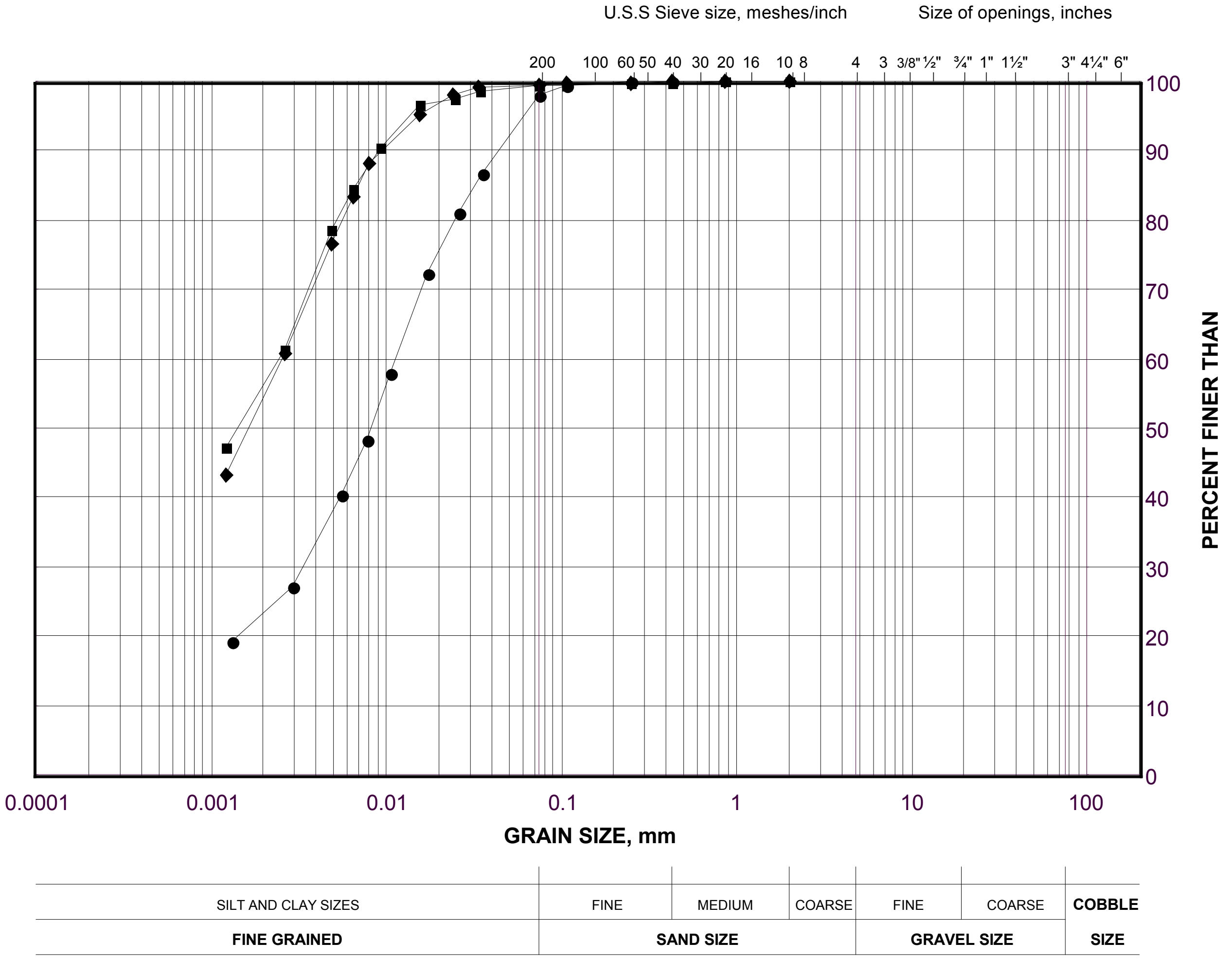
Checked By: TZ

# GRAIN SIZE DISTRIBUTION

Silty Clay to Clay

North Abutment and Approach (One-Spand Bridge)

FIGURE B22



## LEGEND

| SYMBOL | BOREHOLE | SAMPLE | ELEVATION(m) |
|---|---|---|---|
| ● | B202-05 | 4 | 178.7 |
| ■ | B202-12 | 5 | 175.5 |
| ◆ | B202-04 | 8 | 173.3 |

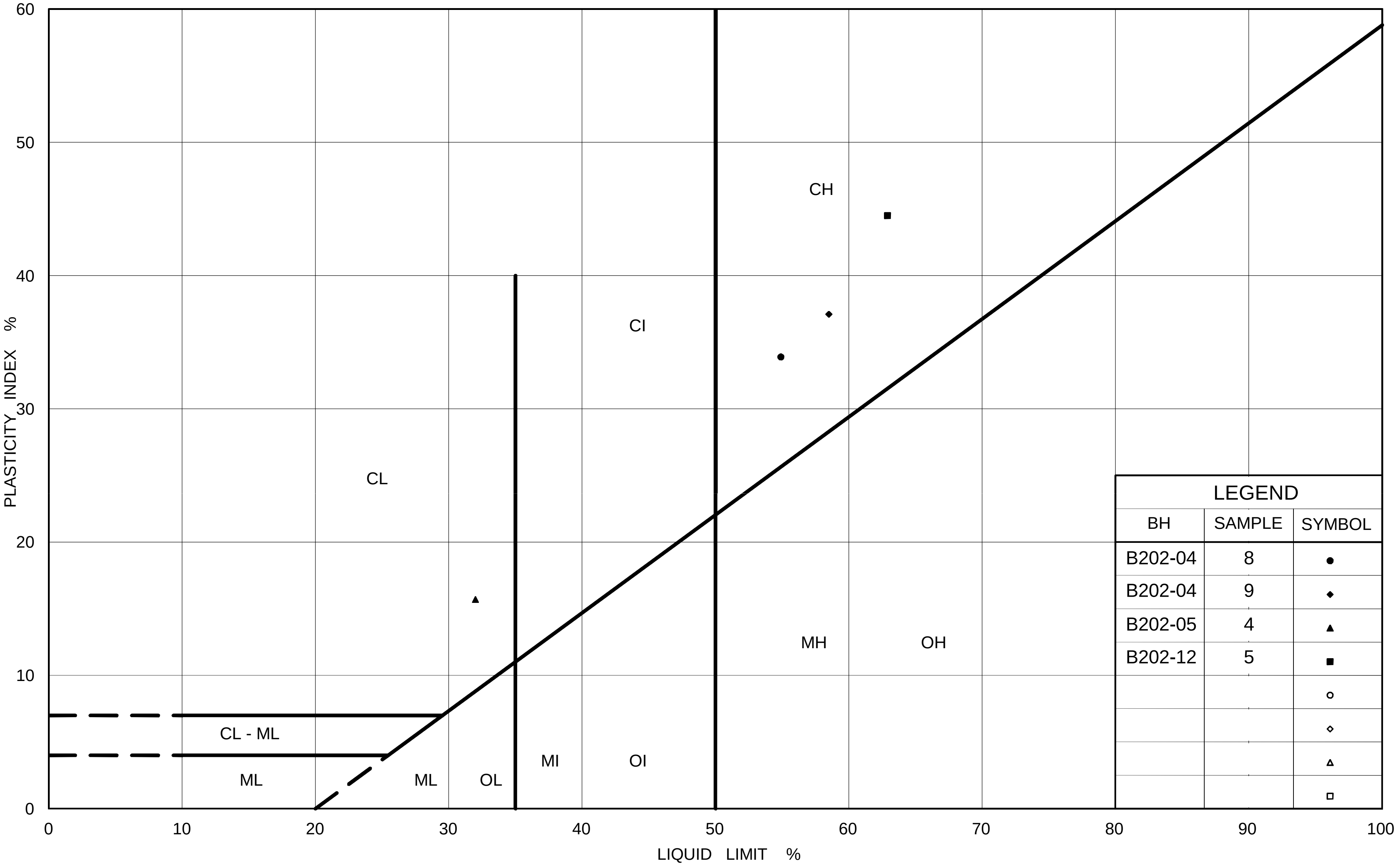
Project Number: 09-1111-6014

Checked By: ______________________

**Golder Associates**

Date: 29-Feb-12

Oct 75, FF-S-21



Ministry of Transportation

Ontario

# PLASTICITY CHART
## Clayey Silt to Clay
North Abutment and Approach (One-Span Bridge)

Figure No. B23

Project No. 09-1111-6014

Checked By: TZ

# CONSOLIDATION TEST SUMMARY

**FIGURE B24**
**Sheet 1 of 4**

## SAMPLE IDENTIFICATION

| | | | |
|---|---|---|---|
| Project Number | 09-1111-6014 | Sample Number | 9 |
| Borehole Number | B202-04 | Sample Depth, m | 7.32-7.75 |

## TEST CONDITIONS

| | | | |
|---|---|---|---|
| Test Type | Standard | Load Duration, hr | 24 |
| Oedometer Number | 7 | | |
| Date Started | 7/14/2011 | | |
| Date Completed | 7/28/2011 | | |

## SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

| | | | |
|---|---|---|---|
| Sample Height, cm | 1.89 | Unit Weight, kN/m$^3$ | 16.54 |
| Sample Diameter, cm | 6.33 | Dry Unit Weight, kN/m$^3$ | 10.60 |
| Area, cm$^2$ | 31.46 | Specific Gravity, measured | 2.77 |
| Volume, cm$^3$ | 59.46 | Solids Height, cm | 0.737 |
| Water Content, % | 56.07 | Volume of Solids, cm$^3$ | 23.20 |
| Wet Mass, g | 100.29 | Volume of Voids, cm$^3$ | 36.26 |
| Dry Mass, g | 64.26 | Degree of Saturation, % | 99.4 |

## TEST COMPUTATIONS

| Pressure kPa | Corr. Height cm | Void Ratio | Average Height cm | $t_{90}$ sec | $c_v$ cm$^2$/s | $m_v$ m$^2$/kN | k cm/s |
|---|---|---|---|---|---|---|---|
| 0.00 | 1.890 | 1.563 | 1.890 | | | | |
| 5.04 | 1.892 | 1.566 | 1.891 | 1 | 7.58E-01 | | |
| 9.99 | 1.896 | 1.571 | 1.894 | 2 | 3.80E-01 | | |
| 20.47 | 1.885 | 1.556 | 1.891 | 113 | 6.71E-03 | 5.65E-04 | 3.72E-07 |
| 40.04 | 1.874 | 1.542 | 1.880 | 383 | 1.96E-03 | 2.89E-04 | 5.54E-08 |
| 79.04 | 1.858 | 1.519 | 1.866 | 420 | 1.76E-03 | 2.28E-04 | 3.93E-08 |
| 156.78 | 1.814 | 1.460 | 1.836 | 290 | 2.46E-03 | 2.98E-04 | 7.20E-08 |
| 315.86 | 1.595 | 1.163 | 1.704 | 1385 | 4.45E-04 | 7.28E-04 | 3.17E-08 |
| 627.57 | 1.424 | 0.931 | 1.509 | 706 | 6.84E-04 | 2.90E-04 | 1.95E-08 |
| 1250.02 | 1.307 | 0.772 | 1.365 | 519 | 7.61E-04 | 9.93E-05 | 7.41E-09 |
| 2493.47 | 1.206 | 0.636 | 1.257 | 452 | 7.41E-04 | 4.27E-05 | 3.10E-09 |
| 1250.02 | 1.220 | 0.654 | 1.213 | | | | |
| 315.86 | 1.256 | 0.703 | 1.238 | | | | |
| 79.04 | 1.307 | 0.773 | 1.281 | | | | |
| 20.47 | 1.349 | 0.830 | 1.328 | | | | |
| 5.04 | 1.378 | 0.869 | 1.364 | | | | |

Note:
k calculated using cv based on $t_{90}$ values.
Specimen swelled under 10 kPa

## SAMPLE DIMENSIONS AND PROPERTIES - FINAL

| | | | |
|---|---|---|---|
| Sample Height, cm | 1.38 | Unit Weight, kN/m$^3$ | 19.28 |
| Sample Diameter, cm | 6.33 | Dry Unit Weight, kN/m$^3$ | 14.53 |
| Area, cm$^2$ | 31.46 | Specific Gravity, measured | 2.77 |
| Volume, cm$^3$ | 43.36 | Solids Height, cm | 0.737 |
| Water Content, % | 32.66 | Volume of Solids, cm$^3$ | 23.20 |
| Wet Mass, g | 85.25 | Volume of Voids, cm$^3$ | 20.17 |
| Dry Mass, g | 64.26 | | |

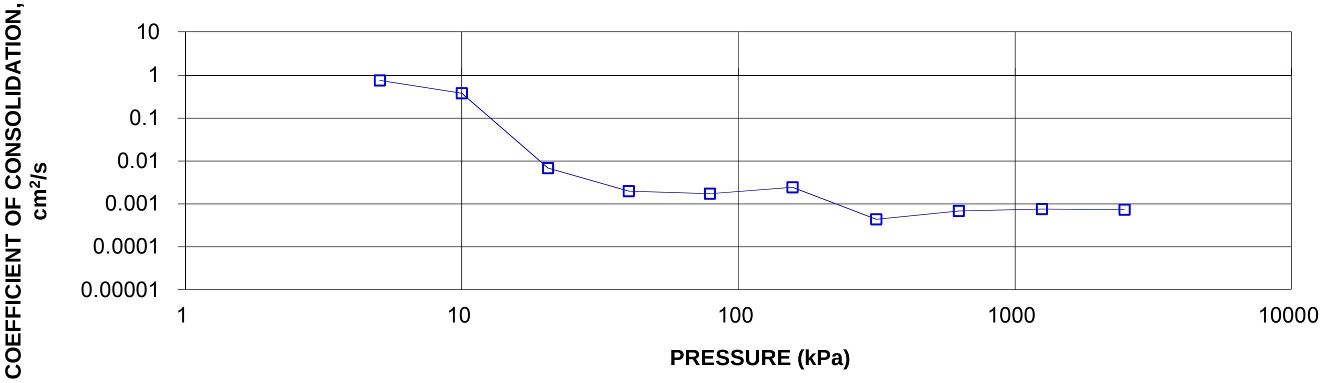
Prepared By: LH **Golder Associates** Checked By: TZ

# CONSOLIDATION TEST SUMMARY

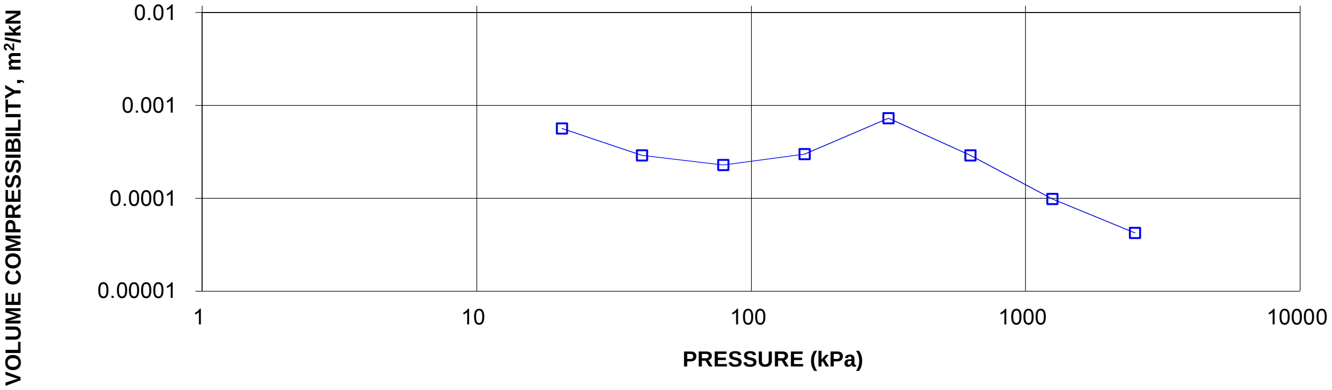
## FIGURE B24

Sheet 2 of 4

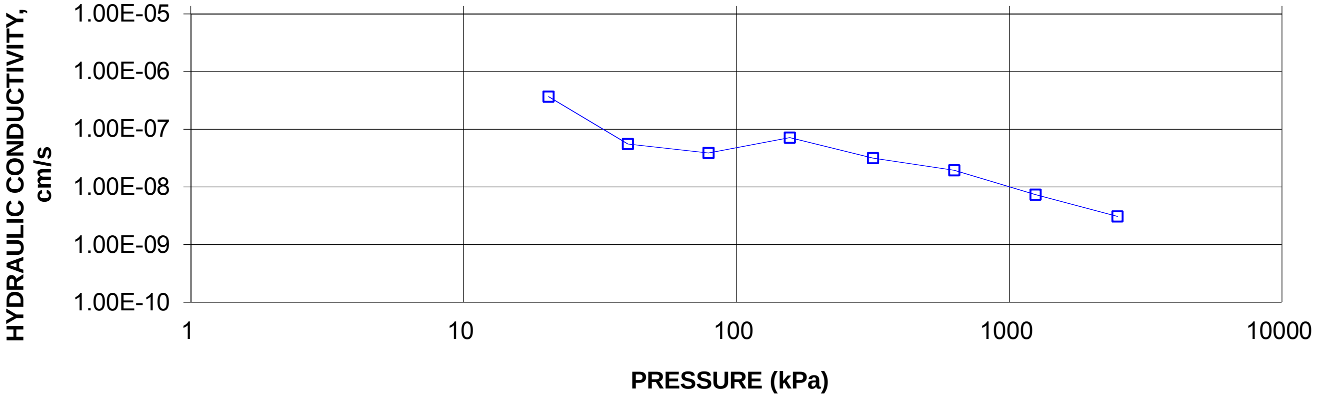
CONSOLIDATION TEST
$C_V$ cm$^2$/s VS PRESSURE (kPa)
BH B202-04 SA 9



CONSOLIDATION TEST
$M_V$ m$^2$/kN vs PRESSURE (kPa)
BH B202-04 SA 9



CONSOLIDATION TEST
HYDRAULIC CONDUCTIVITY vs PRESSURE
BH B202-04 SA 9



Project No. 09-1111-6014
Prepared By: LH

**Golder Associates**

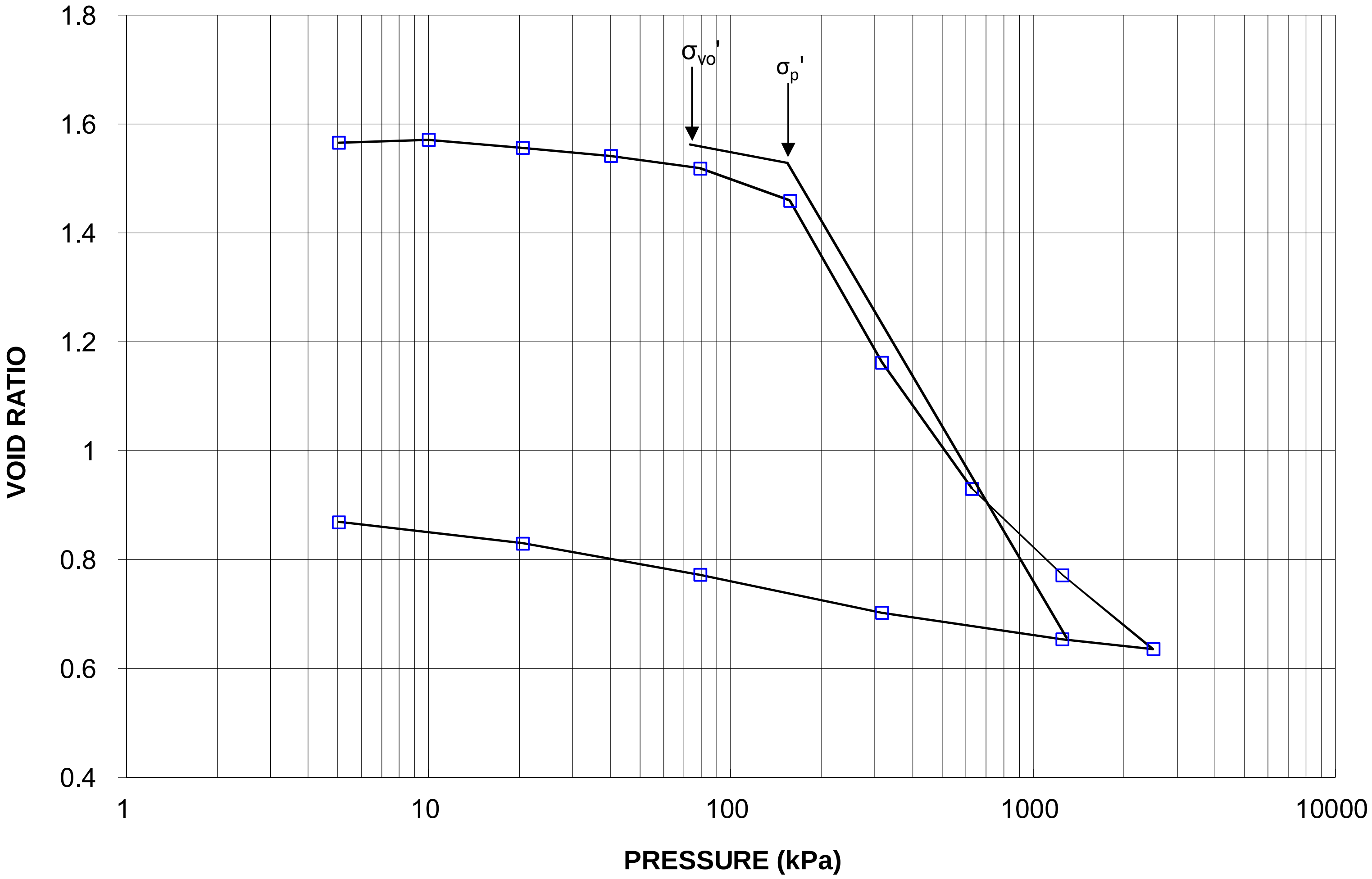
Checked By: TZ

Project No. 09-1111-6014
Prepared By: LH/TZ

**Golder Associates**

Checked By: TZ

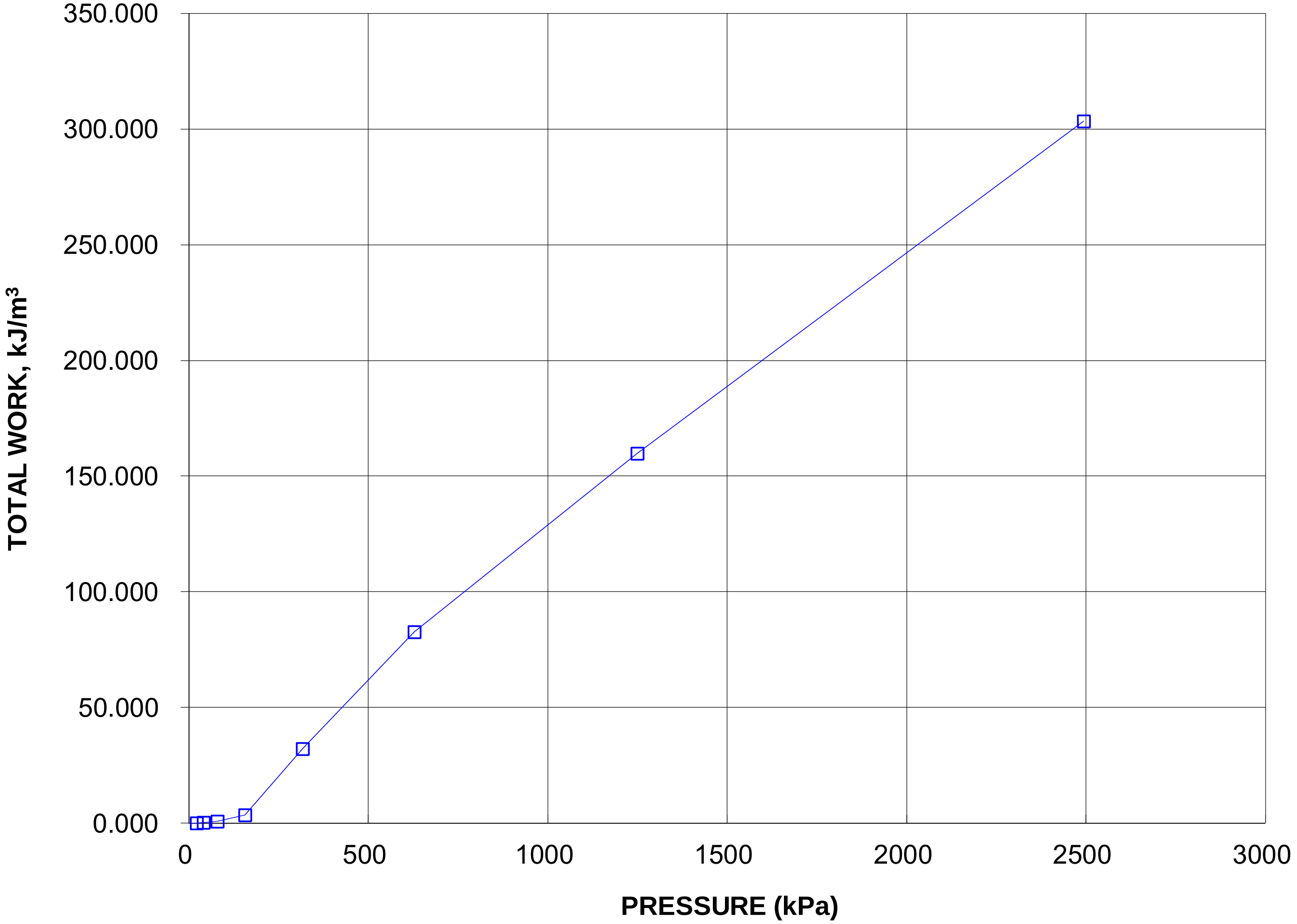
# CONSOLIDATION TEST
## VOID RATIO vs PRESSURE
## BH B202-04 SA 9



**CONSOLIDATION TEST**
**VOID RATIO VS LOG PRESSURE**

**FIGURE B24**
**Sheet 3 OF 4**

## CONSOLIDATION TEST
## TOTAL WORK, kJ/m³ vs PRESSURE
## BH B202-04 SA 9



Project No. 09-1111-6014
Prepared By: LH

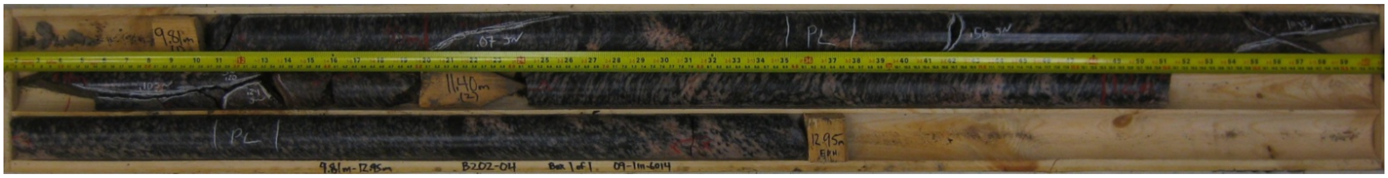
**Golder Associates**

Checked By: TZ

**CONSOLIDATION TEST**
**TOTAL WORK VS PRESSURE**

**FIGURE B24**
**Sheet 4 of 4**

**Borehole B202-04**



Box 1: 9.81 m – 13.00 m

| 0 m | 0.25 m | 0.5 m | 0.75 m | 1.0 m | 1.25 m | 1.5 m |
|---|---|---|---|---|---|---|

| 0 ft | 1 ft | 2 ft | 3 ft | 4 ft | 5 ft |
|---|---|---|---|---|---|

Scale

PROJECT
**Still River Bridge (NBL) Structure**
**Highway 69 Four-Laning**
**GWP 5404-05-00; WP 5139-08-01**

TITLE
## Bedrock Core Photograph – North Abutment (One-Span Bridge)

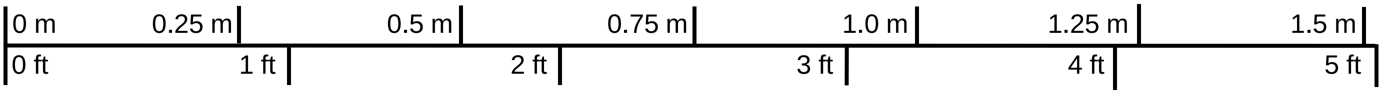
Golder Associates

| | | | | |
|---|---|---|---|---|
| PROJECT No. 09-1111-6014 | | | FILE No. ---- | |
| DESIGN | TZ | | SCALE NTS | REV. |
| CADD | -- | | **FIGURE B25** | |
| CHECK | TZ | | | |
| REVIEW | CN | | | |

REVISION DATE: April 13, 2011 BY: AT Project: 09-1111-6014

**Borehole B202-05**

Box 1: 4.57 m – 6.19 m



Scale

PROJECT

**Still River Bridge (NBL) Structure**
**Highway 69 Four-Laning**
**GWP 5404-05-00; WP 5139-08-01**

TITLE

**Bedrock Core Photograph – North Approach**
**(One-Span Bridge)**

Golder Associates

| | | | | |
|---|---|---|---|---|
| PROJECT No. | 09-1111-6014 | | FILE No. | ---- |
| DESIGN | TZ | | SCALE | NTS |
| CADD | -- | | REV. | |
| CHECK | TZ | | **FIGURE B26** | |
| REVIEW | CN | | | |

REVISION DATE: April 13, 2011 BY: AT Project: 09-1111-6014

**Borehole B202-12**

Box 1: 5.90 m – 7.50 m

Scale

| | |
|---|---|
| PROJECT | Still River Bridge (NBL) Structure<br>Highway 69 Four-Laning<br>GWP 5404-05-00; WP 5139-08-01 |
| TITLE | Bedrock Core Photograph – North Abutment<br>(One-Span Bridge) |

Golder Associates

| | | | | |
|---|---|---|---|---|
| PROJECT No. | 09-1111-6014 | | FILE No. | ---- |
| DESIGN | TZ | | SCALE | NTS REV. |
| CADD | -- | | FIGURE B27 | |
| CHECK | TZ | | | |
| REVIEW | CN | | | |

REVISION DATE: April 13, 2011 BY: AT Project: 09-1111-6014

# 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.<br>6767 Invader Crescent<br>Mississauga, Ontario<br>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 Structures[i]. All reinforcing steel supplied shall be in accordance with OPSS 1440 Steel Reinforcement for Concrete[ii] (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<br>- Linear Dimensions<br>- Flatness<br>- Squareness | mm (min) | 1200 x 600 x 300 ± 1%<br>10 mm in 3 m<br>± 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$^2$.$^o$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 ± 1%.

The flatness of the block faces shall be within ± 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 $m^2 \cdot {}^{o}C/W$ for a 25 mm thickness using the following equation and using the average value from three specimens:

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

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 then 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 90º 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.

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