



September 28, 2011

FOUNDATION INVESTIGATION AND DESIGN REPORT

**REPLACEMENT OF WICKLOW RIVER BRIDGE SOUTH
HIGHWAY 7036, SITE NO. 39E-073
TOWNSHIP OF NEWMARKET, ONTARIO
MINISTRY OF TRANSPORTATION, ONTARIO
GWP 5139-06-00, AGREEMENT NO. 5008-E-0037**

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REPORT





Table of Contents

PART A – FOUNDATION INVESTIGATION REPORT

1.0 INTRODUCTION.....	1
2.0 SITE DESCRIPTION.....	1
3.0 INVESTIGATION PROCEDURES	2
4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS	3
4.1 Regional Geology	3
4.2 Subsurface Conditions.....	4
4.2.1 Asphalt.....	4
4.2.2 Fill	4
4.2.3 Silty Clay to Clay (Alluvium)	5
4.2.4 Silty Clay to Clay.....	6
4.2.5 Silt.....	7
4.2.6 Sand to Sand and Gravel.....	7
4.2.7 Bedrock.....	7
4.2.8 Groundwater Conditions	8
5.0 CLOSURE.....	9

PART B - FOUNDATION DESIGN REPORT

6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS.....	11
6.1 General.....	11
6.2 Foundations	11
6.3 Deep Foundations	12
6.3.1 Geotechnical Axial Resistance.....	12
6.3.2 Downdrag.....	13
6.3.3 Pile Driving Notes and Set Criteria.....	13
6.3.4 Resistance to Lateral Loads.....	14
6.3.5 Frost Protection.....	16
6.4 Seismic Considerations	16
6.4.1 Site Coefficient.....	16
6.4.2 Seismic Analysis Coefficient	16



6.5	Lateral Earth Pressures for Design	17
6.6	Approach Embankment Design	18
6.6.1	Design Assumptions	19
6.6.2	Stability	19
6.6.2.1	Methodology	19
6.6.2.2	Parameter Selection	20
6.6.2.3	Results of Analysis	21
6.6.3	Liquefaction Potential and Seismic Analysis	21
6.6.4	Settlement.....	21
6.6.4.1	Methodology	21
6.6.4.2	Settlement Criteria	21
6.6.4.3	Parameter Selection	22
6.6.4.4	Results of Analysis	24
6.6.5	Mitigation of Time-Dependent Settlements	25
6.6.5.1	Lightweight Fill.....	25
6.6.5.2	Bridge/Highway Geometry	26
6.6.5.3	Preloading	26
6.6.5.4	Sub-Excavation of Silty Clay to Clay Deposit	26
6.6.5.5	Wick Drains	26
6.7	Subgrade Preparation and Embankment Construction.....	26
6.8	Design and Construction Considerations.....	27
6.8.1	Excavations.....	27
6.8.2	Groundwater and Surface Water Control	27
6.8.3	Temporary Shoring	28
6.8.4	Filter Blanket	28
6.8.5	Obstructions	28
7.0	CLOSURE.....	29



TABLES

Table 1	Evaluation of Foundation Alternatives
Table 2	Evaluation of Stability and Settlement Mitigation Alternatives

DRAWINGS

Drawing 1	Wicklow River Bridge South, Highway 7036, Borehole Locations and Soil Strata
Drawing 2	Wicklow River Bridge South, Highway 7036, Soil Strata

FIGURES

Figure 1	Undrained Shear Strength vs. Elevation
Figure 2	Stability Analysis - West Front Slope
Figure 3	Stability Analysis - East Front Slope
Figure 4	Preconsolidation Pressure vs. Elevation
Figure 5	Water Content and Atterberg Limits vs. Elevation
Figure 6	Estimated Consolidation Settlement vs. Time
Figure 7	Recommended EPS Configuration

APPENDICES

Appendix A Record of Boreholes and Drillholes

List of Symbols and Abbreviations
Lithological and Geotechnical Rock Description Terminology
Record of Borehole Sheets (WS-1 to WS-4 and WS-3A)
Record of Drillhole Sheets (WS-2 and WS-3A)

Appendix B Laboratory Test Results

Figure B-1	Grain Size Distribution – Sand (Fill)
Figure B-2	Grain Size Distribution – Silty Clay (Fill)
Figure B-3	Plasticity Chart – Clayey Silt to Silty Clay (Fill)
Figure B-4	Grain Size Distribution – Silty Clay to Clay (Alluvium)
Figure B-5	Plasticity Chart – Silty Clay to Clay (Alluvium)
Figure B-6	Grain Size Distribution – Silty Clay
Figure B-7	Plasticity Chart – Silty Clay to Clay
Figure B-8	Consolidation Test – WS-2, Sa 10
Figure B-9	Consolidation Test – WS-3, Sa 8
Figure B-10	Grain Size Distribution – Silt
Figure B-11	Plasticity Chart – Silt
Figure B-12	Grain Size Distribution – Sand to Sand and Gravel
Figure B-13	Bedrock Core (WS-2)
Figure B-14	Bedrock Core (WS-3A)

Appendix C Non-Standard Special Provisions

NSSP	Control of Artesian Water Seepage
NSSP	CSP for Integral Abutment
NSSP	Pile Capacity Verification Procedure
NSSP	Rigid Expanded Polystyrene Embankment Fill



PART A

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1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by LEA Consulting Ltd. (LEA) on behalf of Ministry of Transportation, Ontario (MTO) to provide foundation engineering services for the replacement of the Wicklow River South Bridge (Site No. 39E-073), located on Highway 7036 (southeast of Cochrane) in the Township of Newmarket.

The terms of reference and scope of work for the foundation investigation are outlined in MTO's Request for Proposal (RFP) dated November 17, 2008. Golder's proposal P81-1685, dated December 2008, for foundation engineering services associated with the replacement is contained in Sections 5.8 and 6.8 of LEA's Technical Proposal that forms part of the Consultant's Agreement Number 5008-E-0037 for this project. Subsequent to the award of the engineering services contract, the Preliminary and Detail Design investigation phases were combined to Detail Design level only. The work was carried out in accordance with Golder's Supplemental Specialty Quality Control Plan for this project dated September 16, 2009. The General Arrangement drawing for the replacement bridge was provided to Golder by LEA on April 14, 2011.

Information on the subsurface conditions for the existing bridge site is not available on MTO's GEOCREST library.

The purpose of this investigation is to establish the subsurface conditions at the proposed replacement location by borehole drilling, in situ testing and laboratory testing on selected samples. The location of the investigated area is shown on Drawing 1.

2.0 SITE DESCRIPTION

The site is situated in the Township of Newmarket on Highway 7036 crossing the Wicklow River, approximately 1.2 km west of the junction with Highway 11. The surrounding table land is generally flat but slopes down towards the river banks along the west and east sides of the river. The area is occupied mainly by residential development with grass and tree covered terrain beyond the various property limits. The river banks adjacent to the existing bridge area are vegetated with landscaped grass and small shrubs. The river flows in a northerly direction and is less than 9 m wide at the existing bridge location.

The existing structure consists of a 27 m long by 9 m wide, two-lane bridge constructed in 1941. The existing structure is founded on timber piles. The length of the piles is unknown. The existing ground surface along the existing highway alignment ranges from Elevation 269.2 m to 269.8 m sloping upwards from west to east. The existing embankment front slopes are formed at approximately 2.9 horizontal to 1 vertical (2.9H:1V) and 2.5H:1V on the west and east sides of the river, respectively. The existing embankment side slopes are greater than about 2H:1V.

The water level in the river was measured between Elevation 266.0 m and 266.2 m during the field investigation (i.e. April 16 to 27, 2011). The high water level is reported to be Elevation 266.9 m. The existing highway is approximately 3.5 m above the river water level or about 3 m above the surrounding ground surface at the west side of the bridge and about 1 m to 2 m above the surrounding ground surface at the east side of the bridge.



3.0 INVESTIGATION PROCEDURES

The fieldwork at the bridge site was carried out between April 16 and 27, 2011, at which time a total of five (5) boreholes (WS-1 to WS-3, WS-3A and WS-4) were advanced: two boreholes (WS-1 and WS-4) at the proposed bridge approaches; and three boreholes (WS-2, WS-3 and WS-3A) at the bridge abutments. The locations of and ground surface elevations at the boreholes are shown on Drawing 1.

All boreholes were drilled using a CME 55 track-mounted drill supplied and operated by George Downing Estate Drilling Ltd. (Downing) of Grenville-Sur-La-Rouge, Quebec. The boreholes were advanced using 108 mm inside diameter (I.D.) continuous flight hollow stem augers, NW casing with wash boring and NQ size core barrels. Soil samples were obtained at intervals of depth of about 0.75 m to 3.0 m, using a 50 mm outer diameter (O.D.) split-spoon sampler operated by an automatic hammer on the drill rig, in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586). Selected 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 determination of undrained shear strengths (ASTM D2573, Standard Test Method for Field Vane Strength Shear Test) using MTO Standard 'N' size vanes. All boreholes were backfilled upon completion in accordance with Ontario Regulation 903 Wells (as amended).

The boreholes for the bridge approaches, WS-1 and WS-4, were advanced to a depth of 15.8 m below ground surface. Borehole WS-2 was advanced at the south side of the west abutment to a depth of 33.6 m below ground surface, including 3.1 m of bedrock coring.

Borehole WS-3 was advanced at the north side of the east abutment to a depth of 34.1 m below ground surface. Difficulty was encountered during the advancement of Borehole WS-3 through the gravelly sand to sand deposit below a depth of about 30 m. Casing refusal was initially encountered at a depth of 30.0 m and a split-spoon was attempted and was noted to be bouncing (i.e. no penetration). An NQ size core barrel was then used to advance the borehole to a depth of 30.5 m. After coring to 30.5 m depth, the casing was further advanced to a depth of 33.5 m. While attempting to obtain a split-spoon sample at a depth of 33.5 m, it was noted that there was approximately 0.6 m of material (gravel and cobbles) inside the casing (i.e. from 32.9 m to 33.5 m). The NQ core barrel was advanced in an attempt to "clean out" the material from inside the casing but the core barrel became lodged inside the casing. After dislodging the core barrel, an attempt was made to remove the gravel and cobbles and dislodge the casing using a "tricone" bit. The tricone was advanced below the bottom of the NW casing to a depth of 34.1 m at which depth the tricone became lodged inside the casing. Several attempts were made to dislodge the tricone but were unsuccessful and the AW drill rods (for the tricone advance) broke at a depth of 12.2 m. Borehole WS-3 was abandoned by tremie grouting with cement grout and Borehole WS-3A was advanced at the south side of the east abutment to penetrate this deposit using drilling mud. Borehole WS-3A was advanced to a depth of 35.0 m below ground surface, including 3.2 m of bedrock coring. It should be noted that in Borehole WS-3A, soil sampling commenced at a depth of 22.9 m below ground surface.

The groundwater conditions and water levels in the open boreholes were observed during the drilling operations and are described on the Record of Borehole sheets in Appendix A. Piezometers were installed in Boreholes WS-1 and WS-4 to allow monitoring of the groundwater level at these locations. The piezometers consist of 19 mm O.D. rigid PVC tubing with 1.5 m and 3 m long slotted screen sealed within the silty clay deposit. Flush mounted caps were used at the ground surface. Details of the piezometer installations and water level readings are presented on the attached Record of Borehole sheets in Appendix A. The piezometers will be decommissioned at a later date.



Flowing artesian groundwater conditions were encountered in Boreholes WS-2, WS-3 and WS-3A upon encountering the silt deposit underlying the silty clay deposit. Details of the sealing of the artesian boreholes are given in Section 4.2.8.

Traffic protection was implemented for the boreholes drilled within the roadway in accordance with the Traffic Protection Plan for this project and MTO Book 7 "Temporary Conditions Manual of the Ontario Traffic Manual" (2001).

The fieldwork was supervised throughout by a member of our technical staff, who located the boreholes, arranged for the clearance of underground services at the borehole locations, supervised the drilling and sampling operations, logged the boreholes, and examined and cared for the soil and rock core samples. The samples were identified in the field, placed in appropriate containers, labelled and transported to our Sudbury 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. Two one-dimensional consolidation (oedometer) tests were carried out on one Shelby tube samples of the cohesive soil. Uniaxial compressive strength (UCS) testing was carried out on two selected specimens of the bedrock core recovered from the boreholes.

The locations of the boreholes were laid out in the field by Golder relative to the existing bridge features. Golder surveyed the geodetic ground surface elevation of the boreholes once completed, referencing an existing benchmark located approximately 18.8 m south of the roadway centreline and approximately 79.0 m west of the west limit of the existing bridge. The northing and easting coordinates were determined by plotting the boreholes relative to the existing bridge features shown on the General Arrangement drawing. The MTM NAD 83 northing and easting coordinates, ground surface elevations referenced to Geodetic datum and depth of each borehole are presented on the Record of Borehole sheets in Appendix A and summarised below.

Borehole	Borehole Location		Ground Surface Elevation (m)	Borehole Depth (m)
	Northing	Easting		
WS-1	5414654.3	313731.7	269.2	15.8
WS-2	5414648.9	313753.7	269.4	33.6
WS-3	5414655.7	313780.8	269.5	34.1
WS-3A	5414649.8	313781.0	269.6	35.0
WS-4	5414649.7	313801.4	269.8	15.8

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

Based on terrain mapping by the Ontario Geological Survey¹, the subsurface soils in the vicinity of the site consist of glaciolacustrine plain deposits comprised of clay bordering with areas of organic terrain.

¹ Northern Ontario Engineering Geology Terrain Study, OGS Survey Map 5027



Based on bedrock geology mapping by the Ministry of Northern Development and Mines², the bedrock of this domain consists of massive granodiorite to granite from the neoarchean to mesoarchean era.

4.2 Subsurface Conditions

The detailed subsurface soil and groundwater conditions as encountered in the boreholes advanced during this investigation, together with the results of the laboratory tests carried out on selected soil samples, are presented on the Record of Borehole and Drillhole sheets in Appendix A. The stratigraphic boundaries shown on the Record of Borehole sheets are inferred from non-continuous sampling and observations of drilling progress and cuttings. 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 inferred soil stratigraphy based on the results of the boreholes is shown in profile on Drawings 1 and 2.

The existing ground surface at the boreholes along Highway 7036 ranges from Elevation 269.2 m to 269.8 m sloping upwards from west to east.

In general, the subsoils consist of fill and alluvium underlain generally by a soft to firm silty clay to clay deposit. Underlying the silty clay to clay deposit are cohesionless deposits of silt and sand to sand and gravel underlain by granite bedrock. A more detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

4.2.1 Asphalt

A 50 mm to 200 mm thick layer of asphalt was encountered from ground surface in all boreholes (WS-1 to WS-3, WS-3A and WS-4).

4.2.2 Fill

Boreholes WS-1 to WS-4 encountered embankment fill consisting of granular and/or clayey soils underlying the asphalt. The total thickness of the fill is between 2.3 m and 3.9 m.

Granular Fill

Granular fill consisting of frozen, brown sand and gravel to sand containing some gravel was encountered in Boreholes WS-1 to WS-4. The granular fill contains trace silt, trace clay and in Boreholes WS-3 and WS-4 is slightly organic. The granular fill is between 0.05 m and 0.8 m thick.

A grain size distribution test was carried out on one sample of the granular fill and the result is shown on Figure B-1.

The natural water content measured on one sample of the granular fill is 11 percent.

² Ministry of Northern Development and Mines, Bedrock Geology of Ontario, East-Central Sheet, Map 2543



Silty Clay Fill

Cohesive fill consisting of frozen to moist, brown silty clay was encountered in Boreholes WS-1 to WS-4 underlying the granular fill. The cohesive fill contains trace to with sand, trace to some gravel and is slightly organic. The surface of the cohesive fill was encountered between Elevation 269.7 m and 268.6 m and the thickness of the deposit ranges from 2.2 m to 3.0 m.

SPT 'N'-values recorded in the frozen cohesive fill range from 11 blows to 36 blows per 0.3 m of penetration. The SPT 'N'-values recorded in the non-frozen cohesive fill range from 3 blows to 10 blows per 0.3 m of penetration suggesting a soft to stiff consistency.

Grain size distribution tests were carried out on two samples of the cohesive fill and the results are presented on Figure B-2. Atterberg limits tests were carried out on five samples of the cohesive fill and test results are presented on Figure B-3. The liquid limits range from about 34 percent to 47 percent, the plastic limits range from about 15 percent to 19 percent and the plasticity indices range between about 16 percent and 28 percent. These results indicate the fill deposit is generally classified as silty clay of intermediate plasticity with one test result plotting in the range of a clayey silt of low plasticity.

The natural water content measured on several samples of the cohesive fill ranges from 23 percent to 27 percent.

An organic content test was carried out on two sample of the cohesive fill and indicates about 1 percent and 3 percent organics.

4.2.3 Silty Clay to Clay (Alluvium)

A deposit of moist, brown grey silty clay to clay alluvium containing trace to some sand was encountered below the fill materials in Boreholes WS-1 and WS-2, both located on the west side of the river. The surface of the alluvium deposit was encountered at Elevation 266.2 m and the thickness of the deposit is between 2.6 m and 2.4 m at the respective boreholes.

The SPT 'N'-values measured within the silty clay to clay alluvium range from 6 blows to 14 blows per 0.3 m of penetration suggesting a firm to stiff consistency.

Grain size distribution tests were carried out on two samples of the alluvium and the results are presented on Figure B-4. Atterberg limits tests were carried out on two samples of the alluvium deposit and the results are presented on Figure B-5. The liquid limits are about 38 percent and 53 percent, the plastic limits are about 22 percent and 23 percent and the plasticity indices are about 16 percent and 30 percent. The results indicate the deposit is classified as silty clay of intermediate plasticity to clay of high plasticity.

The natural moisture content measured on five samples of the alluvium ranges from about 25 percent to 43 percent.

An organic content test carried out on one sample of the alluvium indicates about 8 percent organics.



4.2.4 Silty Clay to Clay

A cohesive deposit of moist to wet, brown to grey, silty clay to clay containing trace sand was encountered below the alluvium in Boreholes WS-1 and WS-2 on the west side of the river and below the fill in Borehole WS-3 and WS-4 on the east side of the river. In Boreholes WS-2 and WS-3, silt layers were noted within the silty clay to clay deposit below about Elevation 252 m. The surface of the silty clay to clay deposit was encountered between Elevation 267.5 m and 263.6 m and the thickness of the deposit is 17.3 m and 19.0 m where the deposit was fully penetrated. Boreholes WS-1 and WS-4 were terminated within this deposit.

The SPT 'N'-values measured in the upper 0.8 m to 2.6 m of the silty clay to clay range from 1 blow to 6 blows per 0.3 m of penetration. Below generally the upper 0.8 m, the SPT 'N'-values are 0 blows per 0.3 m of penetration (i.e. weight of hammer). In situ field vane test carried out within the silty clay to clay ranged from 11 kPa to 57 kPa suggesting a soft to stiff consistency. Generally, the in situ vanes measured undrained shear strengths between 17 kPa and 34 kPa above Elevation 254 m and between 38 kPa and 42 kPa below Elevation 254 m. In Borehole WS-3, the vanes were noted to sink under the weight of the rods on three test attempts between about Elevation 261 m and 254 m. The in situ vane test results indicate that the silty clay to clay deposit generally has a soft to firm consistency. The sensitivity is calculated to be between 2 and 5.

Grain size distribution tests were carried out on two samples of this deposit and the results are presented on Figure B-6. Atterberg limits tests were carried out on thirteen samples of the cohesive deposit and the test results are presented on Figure B-7. The liquid limits range from about 35 percent to 59 percent, the plastic limits range from about 17 percent to 21 percent and the plasticity indices range between about 19 percent and 38 percent. The results indicate this deposit is classified as silty clay of intermediate plasticity to clay of high plasticity.

The natural moisture content measured on twenty-one samples of the silty clay range from 29 percent to 61 percent.

Two laboratory consolidation (oedometer) tests were carried out on specimens of the silty clay to clay obtained from Boreholes WS-2 and WS-3 and the test results are shown on Figures B-8 and B-9, respectively. The preconsolidation stresses were estimated from the Void Ratio versus logarithmic Pressure plots using the Casagrande method as well as from the Total Work versus Pressure plots. The unit weight of the two consolidation samples as well as additional tests from other Shelby tubes was measured between 18.0 kN/m³ and 18.8 kN/m³, and the measured specific gravity was 2.7. The relevant consolidation test results are summarized below:

Borehole/ Sample Number	Elevation (m)	σ_{vo}' (kPa)	σ_p' (kPa)	$\sigma_p' - \sigma_{vo}'$ (kPa)	OCR	e_o	C_r	C_c	c_v^* (cm ² /s)
WS-2/10	260.0	79	133	54	1.7	1.1	0.04	0.3	8.0×10^{-4}
WS-3/8	261.6	91	125	34	1.4	1.1	0.02	0.2	3.0×10^{-4}

Note: *For approximate stress range between the effective overburden stress and the final stress due to a 2.0 m high embankment, that is $80 \text{ kPa} \leq \sigma_v' \leq 150 \text{ kPa}$

where:

- σ_{vo}' effective overburden stress in kPa
- σ_p' preconsolidation stress in kPa
- OCR overconsolidation ratio
- e_o initial void ratio
- C_c compression index (based on void ratio)
- C_r recompression index (based on void ratio)
- c_v coefficient of consolidation in cm²/s in the normally consolidated range



4.2.5 Silt

A deposit of wet, grey silt containing some clay and trace sand was encountered underlying the silty clay to clay deposit in Boreholes WS-2 and WS-3. The surface of the silt deposit was encountered at Elevation 246.5 m and 246.6 m and the thickness of the deposit was 4.5 m and 6.0 m in Boreholes WS-2 and WS-3, respectively. Sampling in Borehole WS-3A commenced within the silt deposit at Elevation 246.7 m and the thickness of the deposit at this location is about 6.5 m. Since sampling was not performed prior to encountering the silt deposit in Boreholes WS-3A, the surface of the silt deposit may be higher and the deposit may be thicker than indicated above.

The SPT 'N'-values measured in the silt deposit range from 4 blows to 11 blows per 0.3 m of penetration indicating a loose to compact relative density.

Grain size distribution tests were carried out on three samples of the silt deposit and the results are shown on Figure B-10. An Atterberg limits test was carried out on one sample of the silt deposit and the test result is presented on Figure B-11. The liquid limit is about 23 percent, the plastic limit is about 18 percent and the plasticity index is about 5 percent and indicate that the (upper portion of the) silt deposit where it transitions from the upper silty clay to clay deposit, is classified as a clayey silt of slight plasticity.

The natural moisture content measured on six samples of the silt ranges from 23 percent to 29 percent.

4.2.6 Sand to Sand and Gravel

A deposit of wet, brown to grey sand to sand and gravel was encountered underlying the silt deposit in Boreholes WS-2, WS-3 and WS-3A. The sand to sand and gravel deposit contains trace silt, trace clay, and cobbles were noted at various depths. The surface of the sand to sand and gravel deposit was encountered between Elevation 242.0 m and 240.2 m and, in Boreholes WS-2 and WS-3A, the deposit is 3.1 m and 2.4 m thick, extending to the bedrock surface. In Borehole WS-3, the sand to sand and gravel was not fully penetrated, as described in Section 3.0, and the deposit is at least 5.2 m thick, extending to the borehole termination depth.

The SPT 'N'-values measured in the sand to sand and gravel deposit range from 29 blows to 69 blows per 0.3 m of penetration indicating a compact to very dense relative density. In two samples, soil/rock coring using an NQ sized core barrel was required to advance the borehole. The recovered soil cores consist of sand to sand and gravel containing cobbles between 0.04 m and 0.18 m thick. Split-spoon refusal (i.e. greater than 100 blows per 0.3 m of penetration) was encountered at the bedrock surface contact at the bottom of the sand to sand and gravel deposit in Boreholes WS-2 and WS-3A.

Grain size distribution tests were carried out on three samples of the sand to sand and gravel deposit and the results are shown on Figure B-12.

The natural moisture content measured on four samples of the sand to sand and gravel deposit range from 10 percent to 23 percent.

4.2.7 Bedrock

Bedrock was encountered at Elevation 238.9 m and 237.8 m (i.e. at depths of 30.5 m and 31.8 m below existing ground surface) in Boreholes WS-2 and WS-3A, respectively. As noted in Section 3.0, Borehole WS-3 could not be advanced deeper than Elevation 235.4 m (i.e. corresponding to a depth of 34.1 m below ground surface) through the overburden, indicating that the bedrock is sloping downwards to the north.



Bedrock was cored for 3.1 m and 3.2 m lengths in Boreholes WS-2 and WS-3A, respectively. The retrieved bedrock core is described as coarse grained, slightly weathered, pinkish grey, granite bedrock, as presented in the Record of Drillhole sheets in Appendix A. Photographs of the retrieved bedrock cores are shown on Figures B-13 and B-14.

The Rock Quality Designation (RQD) measured on the core samples ranges from 80 percent to 100 percent, indicating a rock mass of good to excellent quality as per Table 3.10 of the Canadian Foundation Engineering Manual (CFEM, 2006). The Total Core Recovery (TCR) during bedrock coring was 100 percent.

Laboratory Uniaxial Compression Strength (UCS) testing was carried out on two core samples of the bedrock. The UCS values are presented on the Record of Drillhole Sheets in Appendix A and are summarized below and indicate that the bedrock is strong to very strong as per Table 3.5 of CFEM (2006).

Borehole	Elevation (m)	UCS (MPa)
WS-2	237.5	86
WS-3A	236.2	128

4.2.8 Groundwater Conditions

Groundwater levels were measured in the open boreholes during and upon completion of drilling. Piezometers were installed in Boreholes WS-1 and WS-4 and sealed within the silty clay to clay deposit to allow for monitoring of the groundwater levels. The measured groundwater levels in the open boreholes and piezometers are presented below.

Borehole	Installation	Time and/or Date	Groundwater Depth	Groundwater Elevation
WS-1	Open borehole	Upon completion of drilling	13.5 m	255.7 m
	Piezometer	April 28, 2011	3.5 m	265.7 m
		July 3, 2011	2.3 m	266.9 m
WS-2	Open borehole	Prior to coring bedrock	1.1 m above ground surface (4.4 m above river level at time of drilling)	270.5 m
		After coring bedrock	3.7 m	265.7 m
WS-3	Open borehole	Upon completion of drilling	2.1 m above ground surface (5.5 m above river level at time of drilling)	271.6 m
WS-3A	Open borehole	Upon completion of drilling	2.4 m above ground surface (5.9 m above river level at time of drilling)	272.0 m
WS-4	Open borehole	Dry to bottom of borehole at 15.8 m depth (Elev. 254.0 m) upon completion of drilling	--	--
	Piezometer	April 28, 2011	4.2 m	265.6 m
		July 03, 2011	1.7 m	268.1



Groundwater levels encountered in the boreholes during and shortly after drilling may not be representative of static levels since the groundwater levels in the boreholes may not have stabilized on completion of drilling.

The water level in Wicklow River was measured between Elevation 266.0 m and 266.2 m (shown on Drawing 1) during the field investigation from April 16 to 27, 2011 and at Elevation 265.6 m on July 3, 2011. The high water level is reported to be Elevation 266.9 m.

Groundwater and river water levels in the area are subject to seasonal fluctuations and to fluctuations after precipitation events and snowmelt.

Artesian groundwater conditions were encountered in Boreholes WS-2, WS-3 and WS-3A upon penetrating into the silt deposit. The groundwater levels were measured between Elevation 270.5 m and 272.0 m (corresponding to 1.1 m and 2.4 m above the river level, respectively). These boreholes were sealed full column with cement grout, consistent with Ontario Regulation 903 Wells (as amended by Ontario Regulation 372).

Water level readings were obtained in the piezometers prior to leaving the site on April 28, 2011 and upon returning to the site on July 3, 2011. At the time of the piezometer readings, it was confirmed visually that Boreholes WS-2, WS-3 and WS-3A did not show flowing artesian groundwater conditions.

5.0 CLOSURE

The field drilling program was supervised by Mr. Indulis Dumpis and Mr. David Muldowney, P.Eng. This report was prepared by Mr. David Muldowney, P.Eng., and the technical aspects were reviewed by Ms. Sarah E.M. Coyne, P.Eng., Associate. A quality control review of the report was provided by Mr. Jorge M.A. Costa, P.Eng., Principal and Golder's Designated MTO Contact for this project.



Report Signature Page

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PART B

FOUNDATION DESIGN REPORT
REPLACEMENT OF WICKLOW RIVER BRIDGE SOUTH
HIGHWAY 7036, SITE NO. 39E-073
TOWNSHIP OF NEWMARKET, ONTARIO
MINISTRY OF TRANSPORTATION, ONTARIO
GWP 5139-06-00, AGREEMENT NO. 5008-E-0037



6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides design recommendations on the foundation aspects of the proposed new Highway 7036 bridge structure over the Wicklow River Bridge South. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the subsurface investigation at the site.

The interpretation and recommendations presented are intended only to provide the designers with sufficient information to assess the feasible foundation alternatives and to design the proposed structure foundations. As such, where comments are made on construction, they are provided only in order to highlight those aspects which could affect the design of the project. Those requiring information on aspects of construction should make their own interpretation of the factual information provided as it may affect equipment selection, proposed construction methods, scheduling and the like.

6.1 General

We understand that the existing Wicklow River Bridge South was constructed in 1941 and consists of a 27 m long by 9 m wide, two-lane, four-span structure. The existing bridge is supported by timber piles of unknown length.

We understand that the proposed replacement bridge will be a 22 m long by 6 m wide, single-lane, single-span, integral abutment bridge with 6 m long approach slabs constructed along the existing alignment. The proposed grade at the replacement bridge is between Elevation 269.4 m and 269.5 m at the west and east abutments, respectively. Given that the abutments have been moved closer to the river, the grade raise is approximately 2.0 m high above the existing ground surface at the abutments, while beyond the abutments the grade raise is less than about 0.1 m high above the existing ground surface. The new bridge will be about 3 m narrower than the existing structure and embankment widening will not be required. We understand that the roadway will be closed and traffic will be detoured along the old Highway 11 alignment during construction of the replacement bridge.

The subsurface conditions in the vicinity of the proposed bridge generally consist of fill and/or stiff silty clay alluvium underlain by an up to 19 m thick deposit of soft to firm silty clay to clay. Underlying the cohesive strata are cohesionless deposits of silt and sand to sand and gravel overlying granite bedrock. Bedrock was encountered at depths of about 31 m and 32 m below ground surface (Elevation 238.9 m and 237.8 m) on the west and east sides of the river, respectively, although one borehole at the east abutment was advanced to about 34 m (Elevation 235.0 m) without encountering the bedrock surface. Artesian conditions were noted during the advancement of the boreholes at the abutments, between about Elevation 243 m and 245 m, upon penetrating into the silt deposit.

6.2 Foundations

Given that bedrock was encountered at depths of about 31 m and 32 m below ground surface, and may be at depths of 34 m, we recommend that the replacement bridge be supported on steel H-piles driven to bedrock. Shallow spread footings are not recommended due to the presence of fill and alluvium at the abutments and the



low geotechnical resistance available. Spread footings on a granular pad are also not recommended due to the potential for settlement of the subsoils due to the higher unit weight of the granular fill relative to the existing soils and the pressures exerted by the footings. Caisson foundations are not considered feasible due to the artesian groundwater conditions and the presence of cobbles and possible boulders (as inferred from difficult drilling advance) in the lower cohesionless deposits. Friction piles driven to tip elevations within the silty clay to clay stratum are also not considered suitable due to the low geotechnical axial resistances available and potential downdrag loads imposed by the abutment/approach embankments and hence potential for settlement of the abutments. Table 1 summarizes the advantages, disadvantages, relative costs and risks/consequences of the foundation alternatives for the replacement structure. Design recommendations for the recommended option are given in the sections below.

6.3 Deep Foundations

We recommend that the replacement bridge be supported on steel H-piles driven to bedrock. This will require the piles to penetrate the artesian groundwater bearing deposits encountered below the silty clay to clay deposit. As noted above, friction piles terminating sufficiently above the artesian groundwater deposit within the silty clay to clay deposit will not achieve a sufficient geotechnical axial resistance to support the structure.

6.3.1 Geotechnical Axial Resistance

A factored geotechnical axial resistance at Ultimate Limit States (ULS) of 2,000 kN may be used for the design of steel HP310X110 piles driven to the surface of the granite bedrock. For steel HP310X132 piles driven to the bedrock surface, a factored axial resistance of 2,300 kN may be used for design. These values represent a structural limitation for the piles rather than a geotechnical limitation. The geotechnical resistance at Serviceability Limit States (SLS) for 25 mm of settlement (for the length of piles required at this site) will be greater than the factored geotechnical axial resistances at ULS. Since the bedrock is considered to be an unyielding material, ULS conditions will govern for this foundation type.

For piles driven to bedrock, the estimated tip elevations are presented below. The pile lengths are based on the depth to bedrock encountered in the boreholes advanced for the proposed abutments and the elevations at the proposed underside of the pile caps, as shown on the General Arrangement drawing.

Foundation Unit	Borehole Number	Proposed Underside of Pile Cap	Bedrock Surface Elevation	Approximate Design Pile Length
West Abutment	WS-2	263.7	236.5 m*	27.2 m
East Abutment	WS-3	263.8	235.4 m**	28.4 m
	WS-3A		237.8	

*Depth to the bedrock surface at the west abutment is estimated based on the depth to the bedrock surface in Borehole WS-2 at the south side of the abutment and an assumed bedrock slope towards the north according to the tricone refusal and bedrock surface depths encountered in Boreholes WS-3 and WS-3A at the east abutment.

**Depth of tricone refusal on sand to sand and gravel deposit at the north side of the abutment. Bedrock surface at the south side of this abutment is at Elevation 237.8 m; pile length estimated for greater depth to refusal.



Since the boreholes could not be advanced through the existing bridge deck and the ground topography restricted access along the existing embankment side slopes, the boreholes were located approximately 3 m behind the proposed abutments. Therefore, the depths to bedrock at the location of the new bridge abutments may differ from the bedrock depths at the borehole locations given above. Based on the General Arrangement drawing showing the location of the proposed abutments and site observation of the relative location of the abutments to the existing timber pile bents, it is not expected that the steel H-piles for the new bridge abutments would intercept the existing timber piles.

Because the piles will be driven to bedrock through the artesian groundwater bearing deposit, a filter sand blanket (see Section 6.8.4) should be constructed immediately below the pile cap to dissipate artesian groundwater and filter soil fines that may be carried upwards to the surface of the native soils. A contingency for grouting of any space/voids created along the piles should be included in the contract documents. Four Non Standard Special Provision (NSSP) items detailing the grouting method should be included in the Contract Documents; an example is provided in Appendix C. In addition, flange reinforcement or driving shoes/rock points on the piles should not be used, as discussed in Section 6.3.3.

If corrugated steel pipes (CSPs) are installed as part of the integral abutment design for the bridge (through which the piles will be driven), which we understand is not the case for this site, the CSPs should be backfilled with a loose, fine to medium sand. A NSSP detailing the installation method and gradation of this sand should be included in the Contract Documents; an example is provided in Appendix C.

6.3.2 Downdrag

The difference between the unit weights of the existing and new fill combined with the 2.0 m grade raise will induce settlement of the soft to firm silty clay to clay deposit. Downdrag loads (negative skin friction) will be induced on the friction piles as a result of the addition of approach embankment fill after pile installation is complete, as well as from the resulting settlement of the cohesive soil relative to the piles. Downdrag loads will need to be taken into account for design of the piles supporting the abutments unless mitigation of settlement is carried out prior to pile installation.

The structural design of the abutment piles should be based on an estimated unfactored downdrag load of 200 kN acting on the piles (HP310X110 or HP310X132). The downdrag loads noted above are unfactored loads. The structural capacity of the piles must be checked for the factored dead and downdrag loads in accordance with Section C6.8.4 of the Commentary to the CHBDC (2006) for ULS conditions.

6.3.3 Pile Driving Notes and Set Criteria

All pile installation/driving should be in accordance with OPSS 903 (Deep Foundations). The piles should not be fitted with rock points, driving shoes or flange plates (reinforced tips) in order to minimize the width of the gap that may be created as the piles are driven through the silty clay to clay stratum. This will increase the likelihood of the silty clay to clay to “self-seal” against the pile and hence reduce the potential for the creation of a pathway for artesian groundwater along the pile. Further, given that the piles are long and require splicing, the “butt welds” from OPSD 300.150 (Steel H-Pile Splice) should be used to splice the pile sections to minimize the pathway as opposed to the splice plates. For piles driven to bedrock, Note 5 in Clause 3.3.3 of the Structural Manual (MTO, 2008) should be used on the drawings:

- Piles to be driven to bedrock.



For piles driven to bedrock, set criteria are highly dependent on pile driving hammer type and the selected pile type. 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 choice of set criteria is dependent on the experience of the engineer and traditional use where a substantial database has been developed over the years. The criteria need to be set to also avoid overdriving and possibly damaging the piles.

Based on our experience, consideration should be given to the following preliminary criteria for piles driven to bedrock: the piles should be driven to an initial set equal to or greater than 10 blows per 12 mm of penetration (unless abrupt peaking occurs) using a hammer with rated energy of about 50 kilojoules but not exceeding 60 kilojoules; on reaching the required set, the hammer energy should be reduced by about 75 percent and the pile should then be re-driven in 2 sets of 10 blows and the penetration recorded after each set of 10 blows. The hammer energy shall then be increased to 100 percent and the pile re-driven for 10 blows and the penetration recorded. This procedure is intended to improve the process of seating the pile on the bedrock surface. A final set of no less than 10 blows per 12 mm of penetration should be obtained at the maximum hammer energy.

A NSSP, which indicates that “butt” welds are to be used for splicing the piles and outlines the above criteria for seating the piles on bedrock, should be included in the Contract and an example is included in Appendix C.

6.3.4 Resistance to Lateral Loads

Lateral loads can be resisted fully or partially by the use of battered steel H-piles. If vertical piles are used, the resistance to lateral loading will have to be derived from the soil in front of the piles.

The evaluation of the piles subjected to lateral loads should take into account such factors as 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 moment, the soil resistance that can be mobilized, the tolerable lateral deflection at the head of the pile and the pile group effects.

The lateral load response of a single pile may be calculated using subgrade reaction theory where the coefficient of horizontal subgrade reaction, k_h , (kPa/m) is determined in accordance with Section C6.8.7 in the Commentary to the CHBDC based on the equation for cohesionless soils given below (CFEM, 1992).

For cohesionless soils:

$$k_h = \frac{n_h z}{B} \quad \text{where} \quad \begin{array}{l} n_h \text{ is the constant of horizontal subgrade reaction (kPa/m)} \\ z \text{ is the depth (m)} \\ B \text{ is the pile diameter or width (m)} \end{array}$$

and for cohesive soils:

$$k_h = \frac{6.7 s_u}{B} \quad \text{where} \quad \begin{array}{l} s_u \text{ is the undrained shear strength of the soil (kPa)} \\ B \text{ is the pile diameter or width (m)} \end{array}$$

It is understood that an integral abutment foundation design is being considered, however, we understand CSP liners are not required at this site. Where the integral design includes the installation of 3 m long CSP liners (with the annular space between the pile and the liner filled with uniform grained, uncompacted sand), the upper portion of the H-piles will be generally free to flex and move laterally within the limits of the CSP. With this design, the passive lateral resistance over the length of the pile within the CSP liner should be based on the resistance provided by loose sand. The passive lateral resistance on the exterior of the CSP should be based on the resistance provided by the surrounding soil conditions.



The lateral resistance of the piles should be developed primarily from the passive resistance of the soil. The values of n_h and s_u to be used to calculate the coefficient of horizontal subgrade reaction (k_h) to be utilized in the structural analysis for the piles at this location are given below.

Foundation Element (Relevant Borehole)	Soil Unit	Elevation (m)	n_h (kPa/m)	s_u (kPa)
West Abutment (WS-2)	Loose Sand (Filter Blanket)	263.7 to 263.2	1,300	-
	Soft to Firm Silty Clay to Clay	263.2 to 254.0	-	22 (see Fig. 1)
		254.0 to 246.5	-	35 (see Fig. 1)
	Loose Silt	246.5 to 242.0	2,800	-
	Very Dense Sand to Sand and Gravel (instances of cobbles recorded)	242.0 to 238.9	11,000	-
East Abutment (WS-3, WS-3A)	Loose Sand (Filter Blanket)	263.8 to 263.3	1,300	-
	Soft to Firm Silty Clay to Clay	263.3 to 254.0	-	22 (see Fig. 1)
		254.0 to 246.6	-	35 (see Fig. 1)
	Loose to Compact Silt/Gravelly Sand	246.6 to 240.2	2,800	-
	Dense to Very Dense Gravelly Sand (instance of a cobbles recorded)	240.2 to 235.0	11,000	-

For a single HP310X110 or HP310X132 extending to the bedrock surface to the tip elevations provided in Section 6.3.1, an estimated factored lateral resistance at ULS of 75 kN and at SLS (for 10 mm of horizontal deflection at the pile cap) of 25 kN may be used for design. These values are based on analysis carried out using the Broms' (1964) method as outlined in the CFEM (2006) and the commercially available program LPILE Plus (Version 5.0), produced by Ensoft Inc.

It is recommended that both the 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 soil (down to a depth below the pile cap equal to about $1.5 \times B$ after Broms (1964), where B = 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 the loading is less than six to eight pile diameters. Group action can be evaluated by reducing the coefficient of horizontal subgrade reaction (NAVFAC, 1982) in the direction of loading by a reduction factor, R , as follows:



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

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

Reduction for group effects is negligible when the centre to centre pile spacing exceeds three pile diameters measured in the direction perpendicular to loading.

6.3.5 Frost Protection

At this site, the pile caps should be provided with a minimum of 2.5 m of conventional soil cover for frost protection (as per OPSD 3090.100 Foundation Frost Penetration Depths for Northern Ontario). Alternatively, rigid polystyrene insulation could be used to reduce the required thickness of soil cover. As a guideline for design, it is generally adopted by the MTO that a thickness of 25 mm of rigid polystyrene insulation should be assumed to be equivalent to about 300 mm of conventional soil cover. The insulation, if used, should be placed vertically along the face of the foundation (to the base of the pile cap) and extend horizontally for a distance of 2.5 m beyond the face. A minimum of 1 m of soil cover should be placed over the rigid insulation.

6.4 Seismic Considerations

6.4.1 Site Coefficient

For seismic design purposes, the Site Coefficient, S , for this site, based on experience and considering the guidelines in Section 4.4.6 of the CHBDC may be taken as 1.5, consistent with Soil Profile Type III.

6.4.2 Seismic Analysis Coefficient

The potential for seismic (earthquake) loading must also be considered for the design of abutment stems/retaining walls in accordance with Section 4.6 of the CHBDC. According to Table A3.1.1 of the CHBDC and Table C4.2 of the Commentary, this site is located in Seismic Zone 1. The site-specific zonal acceleration ratio (A) for the Cochrane area is 0.05. The reference Peak Ground Acceleration (PGA) is 0.019 g based on the NRE website; however, the more conservative CHBDC value has been used in the assessment. Based on experience, for the subsurface conditions at this site, a 50 percent amplification of the ground motion may occur (i.e. Site Coefficient, $S=1.5$ for Soil Profile III from Table 4.4 of CHBDC), resulting in an increase in the Peak Ground Acceleration (PGA) from 0.05 g to 0.075 g at the ground surface.

We understand based on Section 4.4.4 of the CHBDC, that this bridge structure is assigned Seismic Performance Zone 1. Given this, and in accordance with Section 4.4.5.1 of the CHBDC, no seismic analysis is required for structures located in Seismic Zone Performance 1.



6.5 Lateral Earth Pressures for Design

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 materials, 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. As discussed in Section 6.4.2, seismic (earthquake) loading need not be analyzed for this structure.

The following recommendations are made concerning the design of the abutment walls. 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 SP 110S13 Granular 'A' or Granular 'B' Type II but containing less than 5 percent passing the No. 200 sieve size should be used as backfill behind the walls. Compaction (including type of equipment, target densities, etc.) should be carried out in accordance with OPSS 501 (Compacting). Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the granular backfill requirements with respect to sub-drains and frost taper should be in accordance with OPSD 3101.150 (Walls Abutment, Backfill) and OPSD 3121.150 (Walls Retaining, 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 specifications as outlined in the Northern Region Directive (2002) for backfill to structures adjacent to rock fill 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 structures, the granular fill should be placed in a zone with width equal to at least 2.5 m behind the back of the walls (in accordance with Figure C6.20(a) of the Commentary to the CHBDC). For unrestrained structures, granular fill should be placed within the wedge shaped zone defined by a line drawn at no steeper than 1.5H:1V extending up and back from the rear face of the base of the footing (in accordance with Figure C6.20(b), Case II, of the Commentary to the CHBDC).
- For restrained structures, the pressures are based on the proposed embankment fill materials and the existing overburden soils and the following parameters (unfactored) may be used assuming the use of granular fill or rock fill:

	Earth Fill 21 kN/m ³	Rock Fill 19 kN/m ³
Soil unit weight:		
Coefficients of static lateral earth pressure:		
Active, K_a	0.31	0.22
At rest, K_o	0.47	0.35



- For unrestrained structures, the pressures are based on the rock fill as above or on the granular fill as placed and the following parameters (unfactored) may be assumed:

	Granular 'A'	Granular 'B' Type II
Soil unit weight:	22 kN/m ³	21 kN/m ³
Coefficients of static lateral earth pressure:		
Active, K_a	0.27	0.27
At rest, K_o	0.43	0.43

- Where EPS is installed behind the abutment wall, the pressure acting over the thickness of the EPS may be calculated using the following values:
 - EPS unit weight: 0.5 kN/m³
 - Coefficients of static lateral earth pressure:
 - Active, K_a 0.11
 - At rest, K_o 0.11

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

A restrained structure is typically a concrete box culvert or a rigid frame bridge where the rotational and/or horizontal movement is not sufficient to mobilize the active earth pressure condition. For this condition, an at-rest pressure plus any compaction surcharge should be included in the design of the structure.

6.6 Approach Embankment Design

The Wicklow River Bridge South replacement structure will be located along the same alignment as the existing structure. The replacement bridge will be about 3 m narrower than the existing bridge and the highway grades will be raised up to about 0.1 m. At the location of the proposed west and east abutments, which are located about 2.5 m closer to the river than the current west and east ends of the existing bridge, the existing sloping ground surface is at approximately Elevation 267.4 m and 267.5 m, respectively. Therefore, the proposed highway grade will be raised 2.0 m above the existing ground surface to Elevation 269.4 m and 269.5 m at/behind the proposed west and east abutments. The proposed grade is approximately 3 m higher than the surrounding ground surface adjacent to the river.

The existing embankments are formed at about 2.9H:1V and 2.5H:1V on the west and east front slopes to the river, respectively. The existing embankment side slopes are formed at between 1.9H:1V and 3.6H:1V. We understand the proposed front slopes will remain at approximately their existing slope angle (i.e. greater than 2H:1V) and the sides will be formed no steeper than 2H:1V.



The subsurface conditions in the vicinity of the proposed bridge generally consist of fill and/or stiff alluvium underlain by an up to 19 m thick deposit of soft to firm silty clay to clay. Underlying the cohesive deposit are cohesionless deposits of silt, and sand to sand and gravel overlying bedrock. Details of the soil and groundwater conditions are given in Section 4.2. The design lines for the magnitude of undrained shear strength, pre-consolidation pressure and index properties (water content and Atterberg limits) used for correlation purposes, versus elevation, for the assessment of stability and settlement of the west and east approach embankments are summarized on Figures 1 to 3.

The following sections present the design assumptions and methodology, parameter selection and results of stability and settlement analysis for the new approach embankments, including recommendations for stability and settlement mitigation measures, as required.

6.6.1 Design Assumptions

For the bridge approach embankments within 20 m of the abutments, the analyses assume that the existing fill and alluvium will only be removed to the underside of the proposed 0.5 m thick filter blanket recommended under the pile cap (i.e. to Elevation 263.2 m and 263.3 m at the west and east abutments, respectively) and within the frost taper zone. Elsewhere, the alluvium material, which is slightly organic, will be left in place, if encountered. The low river water level used for design is Elevation 265.5 m and the high river water level used for design is Elevation 266.9 m. Groundwater levels beyond the river have been assumed to rise approximately 1 m higher behind the approaches.

Granular 'B' Type II fill is recommended and has been assumed for the construction of the approach embankments since this type of material will be used as backfill behind the abutments. Rock fill is not being considered as there is not a local source available and due to the limited quantity required. Embankment side slopes constructed with Granular 'B' Type II fill will be formed at 2H:1V. The proposed front slopes will be formed at approximately 2.9H:1V and 2.5H:1V at the west and east sides of the river, respectively, following the existing slope angle.

6.6.2 Stability

Analyses were performed on the critical sections of the proposed approach embankments for conditions during and after construction to assess the stability and liquefaction potential for the proposed embankment height, existing geometry and soil stratigraphy. The critical embankment sections at this site are the front slopes (towards the river). The side slopes have been checked but are not considered critical. The geometry of the proposed approach embankments, existing ground surface and existing river bed included in the analyses are based on the information from the General Arrangement drawing provided by LEA.

6.6.2.1 Methodology

Limit equilibrium slope stability analyses were performed using the commercially available program GeoStudio 2007 (Version 7.17), produced by Geo-Slope International Ltd., employing the Morgenstern-Price method of analysis. For all analyses, the Factor of Safety (FoS) of numerous potential failure surfaces was computed in order to establish the minimum FoS. The FoS is defined as the ratio of the forces tending to resist failure to the



driving forces tending to cause failure. A target minimum FoS of 1.3 is normally adopted for the design of embankment slopes under static conditions at the end of construction. This FoS is considered adequate for the embankments at this site considering the design requirements and the field data available. The stability analyses were performed to check that the target minimum FoS was achieved for the design embankment height and geometries. In general, circular slip surfaces were analysed in the design. In addition, effective stress (drained) analyses were conducted to assess long-term conditions applying a FoS of 1.3.

6.6.2.2 Parameter Selection

For the cohesionless deposits and granular fill, effective stress parameters were employed in the analysis assuming drained conditions and the parameters were estimated from empirical correlations using the results of the in situ SPT 'N'-values. The correlations proposed by Peck et al. (1974), Schmertmann (1975) and NAVFAC (1982) were employed and the results were tempered by engineering judgment based on precedent experience in similar soils.

For the cohesive fill and native soils, total stress parameters were employed in the analysis. The total stress parameters (i.e. average mobilized undrained shear strength – s_u) for the cohesive soils were assessed based on the results of the in situ field vane tests and estimated from correlations with the SPT 'N'-value results and other laboratory test data. Where appropriate, Bjerrum's (1973) correction factor as a function of the plasticity index of the soil (Figure 3) was employed to estimate the average mobilized undrained shear strength from the results of the in situ field vane tests (Figure 1).

The effective stress parameters for the cohesive strata (effective friction angle and cohesion) for evaluating long-term drained conditions were estimated using empirical correlations with plasticity index (PI) proposed by Mitchell (1993), Ladd et al. (1977) and Kulhawy and Mayne (1990).

Summarized below are the simplified stratigraphy and the associated strengths and unit weights employed for the different soil types in the proposed approach areas. The undrained shear strength design line values for the silty clay to clay deposit used in the analysis are shown graphically on Figure 1. The slope stability analyses model geometry and stratigraphy are shown on Figures 4 and 5 for the critical sections identified above.

Soil Type	Unit Weight (kN/m ³)	Undrained Shear Strength (kPa)	Angle of Internal Friction
New Granular 'B' Type II Fill	21	--	35°
Existing Silty Clay Fill	18	50 kPa	30°
Silty Clay Alluvium	17	50 kPa	30°
Silty Clay to Clay	18	20 kPa above Elev. 254.0 m 33 kPa below Elev. 254.0 m (See Figure 1)	27°
Silt	19	--	27°



6.6.2.3 Results of Analysis

The results of the stability analyses indicate that the critical front slopes, formed at their existing slope angle according to the General Arrangement drawing from LEA, have a FoS of greater than the target of 1.3 for both the drained and undrained case and are presented on Figures 4 and 5. Therefore, stability mitigation is not required at this site.

6.6.3 Liquefaction Potential and Seismic Analysis

As noted in Section 6.4.2, this site is located in Seismic Zone 1 with a $PGA < 0.08$. Further, the bridge structure is not a lifeline structure. As such, based on Section 4.4.4 of the CHBDC, the site is assigned a Seismic Performance of 1 and, therefore, in accordance with Section 4.4.5.1 of the CHBDC, liquefaction analysis is not required.

6.6.4 Settlement

Settlement of the approach embankments can be expected as a result of the loading from the new fills on the existing fill and compressible foundation soils at this site. The critical section is considered to be the immediate abutment area where the embankment is being raised by 2.0 m, as a result of the abutments being moved closer to the river. Beyond the immediate abutment area, the grade raise decreases to zero as it transitions to the existing top of roadway level. Due to this grade raise at the abutments, time-dependent consolidation settlement of the soft to firm silty clay to clay deposit is expected. Settlement of the existing fill, firm to stiff alluvium and cohesionless deposits is expected to be elastic, occurring during or shortly after construction. In addition, settlement of the new embankment fill (Granular 'B' Type II) will also occur, but is expected to take place during or shortly after construction.

The following sections summarize the methodology, criteria, simplified stratigraphy, unit weights and deformation parameters employed for the different soils types in the approach areas. The maximum estimated settlement of the foundation soils in these areas (due to the loading imposed by the new approach embankment fills) and a discussion on the rate of settlement is presented below.

6.6.4.1 Methodology

To estimate the magnitude of the expected settlements, analyses were carried out on the critical sections of the proposed approach embankments using the commercially available program Settle3D (Version 2.003) produced by Rocscience Inc. as well as hand/spreadsheet calculations. The rate of settlement of the cohesive foundation soils was assessed using Terzaghi's one-dimensional consolidation theory. The model geometry and stratigraphy are shown on Figures 4 and 5, as used for the stability analyses for the west and east front slopes, respectively.

6.6.4.2 Settlement Criteria

Based on MTO's "Embankment Settlement Criteria for Design" Final Draft dated March 2, 2010, the following post-construction settlement and differential settlement criteria are considered acceptable to occur within 20 years post-paving for the bridge approach embankments.



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

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

6.6.4.3 Parameter Selection

The immediate compression of the existing fill and native cohesionless deposits was assessed by estimating an elastic modulus of deformation based on the SPT 'N'-values and empirical correlations found in literature by Bowles (1984) and Kulhawy and Mayne (1990).

The consolidation settlement of the very soft to firm clayey silt to clay deposit was assessed using the results of the in situ field vane tests and the laboratory consolidation tests to estimate the deformation parameters for these soils. In addition, the results of the laboratory index testing were also employed to estimate deformation parameters using empirical correlations proposed in literature by Terzaghi and Peck (1967), Kulhawy and Mayne (1990), Azzouz et al. (1976) and Koppula (1986).

The degree of over-consolidation in the cohesive strata, required in the analyses, was estimated from the results of the in situ field vane tests and the following correlations relating mobilized undrained shear strength to preconsolidation pressure:

$$s_{u(mob)} = 0.22\sigma_p' \text{ (after Mesri 1975)}$$

where: $s_{u(mob)}$ = average mobilized undrained shear strength (kPa)
 σ_p' = pre-consolidation stress (kPa) (see Figure 2)

and

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

where: $s_{u(mob)}$ = average mobilized undrained shear strength (kPa)
 $s_{u(FV)}$ = undrained shear strength from field vane test (kPa)
 μ = Bjerrum's correction factor based on Plasticity Index (i.e. ranging from 0.88 to 1 for this site as Plasticity Indices range from about 19 percent to about 38 percent), as shown on Figure 3.

It is known that some secondary consolidation settlement occurs following the completion of primary settlement. This secondary settlement, or creep settlement, occurs over the long term (i.e. decades) for the normally consolidated clays at this site. The magnitude of secondary (creep) settlement (Mesri 1973 as quoted in Holtz and Kovacs 1981) was estimated using the following:



$$S_c = C_{\alpha\epsilon} \times L_o \times (\Delta \log t)$$

Based on Mesri (1973), the following empirical correlation was utilized to estimate $C_{\alpha\epsilon}$ from water content:

$$C_{\alpha\epsilon} = w_n / 100$$

where: S_c = secondary (creep) settlement (mm)
 $C_{\alpha\epsilon}$ = modified secondary compression index (%)
 L_o = initial thickness of compressible deposit (mm) in the normally consolidated portion of the deposit
 w_n = water content (decimal)
 t = time period of interest

The following simplified stratigraphy, unit weights and deformation parameters have been employed in the settlement analysis of the proposed approach embankments. The thickness of the materials varies across the site and the maximum thicknesses have been used in the settlement estimates. Given the relatively low embankment heights, the full thickness of the silty clay to clay deposit (up to about 19 m) will not be impacted by the fill loading and the estimated effective compressible thickness has been used in the analyses as noted below and has been confirmed by computer analysis. Further, the lower cohesionless soils are not anticipated to settle under the imposed loading due to the great depth below ground surface that these strata are present at. The thicknesses given below assumed that only the existing fill and alluvium have been removed in the abutment, filter blanket and frost taper zones (as per OPSD 3101.159, Walls, Abutment, Backfill) and therefore some of these materials may be left in place.

Deposit	Maximum Thickness at West or East Abutment (m)	Maximum Thickness at West or East Approach ~20 m Behind Abutment (m)	Unit Weight (kN/m ³)	Deformation Properties
New Granular* Fill	W = 4.2 (2.4)* E = 4.2 (2.4)*	W = ~1.0 (~0)* E = ~1.0 (~0)*	21	See Section 6.6.4.4
Existing Sand to Sand and Gravel Fill	W = n/a E = n/a	W = n/a E = n/a	20	E' = 10 MPa
Existing Silty Clay Fill	W = n/a E = n/a	W = n/a E = n/a	18	E' = 15 MPa
Silty Clay to Clay Alluvium	W = n/a E = n/a	W = 2.6 E = n/a	17	E' = 15 MPa
Silty Clay to Clay	W = 17.3 E = 19.0 (impacted thickness is typically about 5 m to 7 m)**	W = >10.2 E = > 13.5 (not expected to be impacted by low fill height)	18	(see below)

* This value represents the total thickness of new fill. The effective fill thickness (in brackets) considers the total loading from the existing fill and the native soils that are to be removed.

** The maximum thickness of main cohesive deposit that will be influenced by embankment loading (approximately two times the effective height of the embankment below the base of the fill) is estimated to be between 5 m and 7 m of the total thickness of the stratum.

n/a Indicates deposit not encountered below the new fill.



The following consolidation parameters were estimated for the silty clay to clay deposit based on the results of laboratory consolidation tests performed on specimens of the silty clay to clay obtained from Boreholes WS-2 and WS-3 and compared with values estimated from empirical correlations using the results of the in situ tests and laboratory index testing as described above as well as review of the data contained in Quigley and Ogunbadejo (1972). The plot of preconsolidation pressure and in situ vertical effective stress versus elevation is provided on Figure 2.

Approximate Elevation (m)	σ_{vo}' (kPa)	σ_p' (kPa)	OCR	e_o	C_r	C_c
265.0 – 254.0	(see Figure 4)		1.1	1.2	0.04	0.4
254.0 – 247.0	(see Figure 4)		1.1	1.6	0.08	0.8

Figure 5 shows the results of the water content and Atterberg Limits versus elevation. Based on the results of the consolidation tests and empirical correlations with this laboratory data (NAVFAC, 1982), a coefficient of consolidation, $c_v (n/c)$, equal to $8.0 \times 10^{-4} \text{ cm}^2/\text{s}$ is considered appropriate for the normally consolidated range and a $c_v (o/c)$ of $6.3 \times 10^{-3} \text{ cm}^2/\text{s}$ is considered appropriate for the recompression range. The modified secondary compression index, $C_{\alpha(\epsilon)}$, used in the analysis to calculate creep is 1.3.

6.6.4.4 Results of Analysis

A summary of the results of the settlement analysis at the critical sections (i.e. at the abutments and the approaches 20 m behind the abutments) is presented below. Settlement of the new Granular 'B' Type II fill above the water level is typically less than 25 mm if placed and compacted properly (see Section 6.8). Figure 6 shows the estimated consolidation settlement versus time for the silty clay to clay deposit at the abutments.

Critical Section	Relevant Borehole	Estimated Immediate Settlement (mm)		Consolidation Settlement (mm)		Total Settlement	Post-Construction Settlement**
		New Granular Fill*	Existing Fill/Alluvium	Silty Clay to Clay			
				Primary	Creep		
West Approach	WS-1	<25 mm	<25 mm	0	0	<25 mm	<25 mm
West Abutment	WS-2		<25 mm	150	30	205	180
East Abutment	WS-3		<25 mm	150	30	205	180
East Approach	WS-4		<25 mm	0	0	<25 mm	<25 mm

* Granular 'B' Type II.

** Assumes that post-construction settlement begins when the embankment has reached its final height; in this case, when the abutment is completely backfilled.



Based on the c_v values given in Section 6.6.4.3, it is estimated that about 90 percent of the primary consolidation settlement will be completed in about 12 years at the west and east abutments. The magnitude of secondary (creep) settlement is expected to be about 130 mm per log cycle. Therefore, less than one log cycle of creep will occur from completion of about 90% of primary consolidation to the design life of the approach embankment (i.e. 20 years) as indicated above.

Since the post-construction settlement at the abutments is greater than the 25 mm allowable criteria (see Section 6.6.4.2), settlement mitigation will be required.

6.6.5 Mitigation of Time-Dependent Settlements

In order to reduce the post-construction settlement at the abutments, the alternatives presented below can be considered. The alternatives described have been evaluated and ranked on the basis of the advantages, disadvantages, relative costs and risk/consequences and are summarised in Table 2. The preferred alternative from a foundations perspective to provide the best technical solution in terms of the long-term operational performance of the roadway is the use of lightweight fill comprised of expanded polystyrene (EPS) within a portion of the approach embankments behind the abutments to reduce the loading on the underlying soft to firm silty clay to clay deposit. Other alternatives such as changing the bridge span lengths and/or highway geometry, preloading, sub-excavation and wick drains have also been considered, but do not appear to be of advantage over the use of EPS.

6.6.5.1 Lightweight Fill

We recommend that EPS fill be used to reduce the embankment fill load on the subsoils thereby reducing post-construction settlement and eliminating downdrag loads on the piles. Typically, EPS is considered to be too expensive to consider as a solution, however, given the small volume of material to be used at this site, it is not precluded as an option based on cost alone.

The EPS should be 2.0 m thick from the abutment to 5 m behind the abutments, stepping up (decreasing in thickness) in 0.5 m increments from 5 m behind the abutments at approximately 5H:1V (or approximately 0.5 m by 2.5 m long steps), for a taper distance of about 7.5 m. This will result in the EPS extending a total distance of about 12.5 m behind the abutments. Figure 7 shows the recommended configuration of the EPS behind the abutments in the approach areas in both plan and cross-section. EPS is typically provided in blocks of 0.5 m thick to accommodate the steps. Appropriate staggered layout, ties and spacers should be used to ensure the EPS block mass acts as a single unit. The finished top of the EPS blocks should be covered with a 6 mil mm thick polyethylene sheet and provided with a minimum 1 m of conventional granular cover (pavement structure), and include a 125 mm thick concrete slab, for ballast and for protection against differential icing at the roadway structure. It is estimated that a volume of EPS of about 250 m³ will be required to satisfy the recommendations for this site.

Buoyancy of the EPS is not a consideration in this case since the base of the EPS blocks will be well above the highest measured water level.

An NSSP should be included in the Contract for Rigid Expanded Polystyrene Embankment Fill and an example is included in Appendix C.



6.6.5.2 Bridge/Highway Geometry

Consideration could be given to moving the abutments further back from the river to reduce the post-construction settlement at the abutments. Further, lowering the highway grade at the bridge could also be considered. In either case, some post-construction settlement would still occur due to the settlement resulting from the difference in density between the existing soils and new fill and any additional regrading fill that may be used under the pavement structure. However, lowering the grade would result in a lower underside of pile cap excavation, which would extend further below the water level and potentially result in the need for additional temporary shoring/dewatering.

6.6.5.3 Preloading

It is estimated that 90 percent of primary consolidation settlement would be completed in about 12 years for the west and east approach embankments. The total post-construction settlement would be about 45 mm (including remaining primary and creep settlement) and additional mitigation measures would be required to reduce this value below the settlement criteria.

Given the logistics of and time required to construct the preload embankment and the requirement for an embankment instrumentation and monitoring program, preloading is not recommended as the preferred alternative.

6.6.5.4 Sub-Excavation of Silty Clay to Clay Deposit

Due to the depth of the silty clay to clay deposit (approximately 17 m below the underside of pile cap) and the potential for encountering artesian groundwater conditions, sub-excavation of the soft to firm compressible soils, and subsequent backfilling of the excavated area with Granular 'B' Type II (or rock fill) is not considered practical for the limited area over which such remedial works would be required.

6.6.5.5 Wick Drains

Wick drains would decrease the time required for primary settlement. However, since this option would have to be combined with the preloading, this option is not recommended.

6.7 Subgrade Preparation and Embankment Construction

For the bridge approach embankments within 20 m of the abutments, removal of the topsoil and fill and partial removal of the slightly organic silty clay to clay alluvium is required below the reconstructed embankment footprint prior to placement of new fill. All softened/loosened soils should also be stripped from below the approach embankments, prior to placement of new fill. The backfill in the frost taper zone should be constructed in accordance with OPSD 3101.150 (Walls, Abutment, Backfill).

Granular fill materials and placement should be in accordance with the requirements as outlined in SP 206S03 (Earth Excavation, Grading). All granular fill should be placed in lifts with loose thickness not exceeding 300 mm and compacted to at least 95 percent of the standard Proctor maximum dry density. Prior to placement of the



granular subbase and base courses, the final lift of embankment fill should be compacted to not less than 100 percent of the Standard Proctor maximum dry density. Inspection and field density testing should be carried out by qualified personnel during fill placement operations to ensure that appropriate materials are used and that adequate levels of compaction have been achieved. The new fill should be keyed into the existing embankment side slopes per the requirements of OPSD 208.010 (Benching of Earth Slopes).

The abutment front slopes and side slopes adjacent to the river require erosion protection in accordance with OPSS 511 (Rip Rap, Granular Sheeting) and SP 511S01 (Rip Rap, Gravel Sheeting). Erosion protection should be placed on the slopes to at least 0.5 m above the design high water level. Erosion protection could consist of a minimum 0.6 m thick layer of R-10 Rip Rap (300 mm diameter as per OPSS 1004), rock protection or concrete slope paving. The designer should address the potential for scour below the pile caps in the design of the bridge foundations.

To reduce surface water erosion on the embankment side slopes, topsoil and seeding as per OPSS 802 (Topsoil) and OPSS 804 (Seed and Cover) should be carried out as soon as possible after construction where earth fill is used. If this slope protection is not in place before winter, then alternate protection measures, such as covering the slope with straw or gravel sheeting as per OPSS 511 (Granular Sheeting) to prevent erosion, will be required to reduce the potential for remedial works on the side slopes in the Spring prior to topsoil dressing and seeding.

6.8 Design and Construction Considerations

6.8.1 Excavations

The excavation for pile cap and filter blanket construction will extend to Elevation 263.2 m and 263.3 m at the west and east abutments, respectively, resulting in excavations up to 6.2 m below the existing ground surface at the abutments. Since the excavations will extend up to 2.3 m below the river low water level Elevation 265.5 m, a cofferdam and/or temporary shoring will likely be required to allow for construction of the pile caps in dry conditions, as discussed in Section 6.8.2.

As it is proposed that the road will be closed during construction, it will be possible to carry out the excavations in open cut. Open cut slopes within the fill materials, silty clay to clay alluvium and the very upper portion of the silty clay to clay deposit should be maintained at no steeper than 2H:1V above the water level and below the water level, if encountered.

All excavations must be carried out in accordance with the latest edition of the Ontario Occupational Health and Safety Act (OHSA) and Regulations for Construction Projects and good construction practice. The existing fill materials and the native soils should be classified as Type 3 soil, according to the OHSA. □

6.8.2 Groundwater and Surface Water Control

Although perched water was not encountered within the fill during the investigation, it is possible that water is perched within the fill materials, and the Contractor should anticipate perched water within the fill sub-excavation for the bridge approach embankments. Surface water should be directed away from the excavation at all times.



The excavation for the abutments will be located adjacent to the Wicklow River. The excavation for the pile cap and filter blanket will be 2.3 m below the river low water level and up to 3.7 m below the river high water level. Therefore, temporary shoring with dewatering will likely be required to allow for construction of the pile caps in dry conditions. Temporary shoring and dewatering could be in the form of a sheet-pile cut off wall, a cofferdam or temporary pumping from properly filtered sumps below the base of the excavation for localized groundwater control.

6.8.3 Temporary Shoring

A temporary excavation support system should be designed and constructed in accordance with OPSS 539 (Temporary Protection Systems). The lateral movement of the temporary shoring system should meet Performance Level 2, as specified in OPSS 539 (Temporary Protection Systems).

Given the proximity to the river and the depth of excavation required for the bridge abutments, consideration should be given to the use of temporary shoring such as a sheet-pile cut-off wall or other cofferdam type construction. Above the water level, other types of temporary shoring could be considered including braced excavations such as sheet piling, soldier pile and lagging or tied-back walls.

6.8.4 Filter Blanket

Given that the H-piles for the abutments will penetrate through the artesian groundwater bearing layer(s) and will be founded on bedrock, we recommend that a drainage/filter blanket consisting of a minimum 0.5 m thick layer of concrete fine aggregate (OPSS 1002, Aggregates, Concrete) be placed below the underside of the pile caps encasing all the piles. The base of the filter blanket, 0.5 m below the underside of pile cap, will extend to Elevation 263.2 m and 263.3 m at the west and east abutments, respectively. The concrete fine aggregate layer should extend a minimum of 0.5 m horizontally beyond each of the pile caps. Further, the excavation at the front of the abutments (towards the river) should be backfilled with free draining material extending at least 0.5 m horizontally from the front face of the abutment.

6.8.5 Obstructions

As part of the design and construction of the new abutment foundations, careful consideration should be given to the location of the existing piles relative to temporary shoring, cofferdams and replacement bridge piles. Specifically, the designer should check that the new piles (batter and orientation) and temporary shoring do not interfere with the existing piles. This should be checked to the full extent of the pile/shoring length.

The depth of the existing timber piles is not known. Given the thickness of the cohesive deposit, it is likely that the timber piles were terminated above the artesian groundwater bearing silt deposit, which was encountered at about 23 m below the existing ground surface. It is, however, possible that the piles extend into the artesian groundwater bearing deposit. If the timber piles are removed, there may only be a portion of the thickness of the silty clay to clay stratum resisting the artesian pressure at the base of the extraction hole and, therefore, there would be a risk of creating a pathway for artesian groundwater and for potential ground loss. Further, backfilling the pile holes, where removed within the river channel, may not be feasible or practical below the water level. We recommend that the existing timber piles be left in place and "cut off" at the river bed level and not be pulled out.



7.0 CLOSURE

This report was prepared by Mr. David Muldowney, P.Eng., and the technical aspects were reviewed by Ms. Sarah E.M. Coyne, P.Eng., Associate. A quality control review of the report was provided by Mr. Jorge M.A. Costa, P.Eng., Principal and Golder's Designated MTO Contact for this project.



Report Signature Page

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ASTM International

- ASTM D1586 Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils
- ASTM D1587 Standard Practice for Thin-Walled Tube Sampling of Soils for Geotechnical Purposes
- ASTM D2573 Standard Test Method for Field Vane Shear Test in Cohesive Soil

Commercial Software

- LPILE (Version 5.0) by Ensoft Inc.
- GeoStudio (Version 7.17) by Geo-Slope International Ltd.
- Settle 3D (Version 2.003) by Rocscience Inc.

Ministry of Transportation Ontario Special Provisions

- SP 110S13 Material Specification for Aggregates – Base, Subbase, Select Subgrade and Backfill Material
- SP 206S03 Earth Excavation, Grading
- SP 511S01 Rip Rap; Rock Protection; Gravel Sheeting

Ontario Provincial Standard Drawings

- OPSD 208.010 Benching of Earth Slopes
- OPSD 3000.150 Foundation Piles, Steel H-Pile Splice
- OPSD 3090.100 Foundation, Frost Penetration Depths for Northern Ontario
- OPSD 3101.150 Walls Abutment, Backfill Minimum Granular Requirement
- OPSD 3121.150 Walls Retaining, Backfill Minimum Granular Requirement

Ontario Provincial Standard Specifications

- OPSS 501 Construction Specification for Compacting
- OPSS 511 Construction Specification for Rip Rap, Rock Protection and Granular Sheeting
- OPSS 539 Construction Specification for Temporary Protection Systems
- OPSS 802 Construction Specification for Topsoil
- OPSS 804 Construction Specification for Seed and Cover
- OPSS 903 Construction Specification for Deep Foundations
- OPSS 1002 Material Specification for Aggregates – Concrete
- OPSS 1004 Material Specification for Aggregates – Miscellaneous

Ontario Water Resources Act

- Ontario Regulation 468/10 Amendment to Ontario Regulation 903
- Ontario Regulation 903/90 Wells



Table 1: Evaluation of Foundation Alternatives

Foundation Type	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Driven Steel H-Piles	1	<ul style="list-style-type: none">■ Straightforward construction.■ Higher axial resistance compared to spread footings.■ Fewer problems associated with artesian groundwater compared to caissons.■ Suitable for integral abutment design.	<ul style="list-style-type: none">■ Risk of problems associated with penetrating the artesian deposit, such as pile settlement or ground loss.■ Requires excavation for pile cap construction.	<ul style="list-style-type: none">■ Relative costs lower than caissons but higher than spread footings.■ Contingency for grouting along piles.	<ul style="list-style-type: none">■ Low risk of not achieving design resistance at design pile tip elevation.■ Low risk of problems associated with artesian groundwater during pile driving, but grouting along piles could be necessary.
Caissons	2	<ul style="list-style-type: none">■ Higher axial resistances compared to steel H-piles.■ Pile cap can be constructed at bridge deck and, therefore, excavation is not required.	<ul style="list-style-type: none">■ Higher risk of problems associated with penetrating the artesian deposits and cobbles and boulders compared to piles.■ Not suitable for integral abutment design.	<ul style="list-style-type: none">■ Relative costs much higher than steel H-piles.	<ul style="list-style-type: none">■ High risk of not reaching the required termination depth due to the presence of cobbles and boulders.■ High risk of construction problems associated with artesian groundwater during caisson installation.
Shallow Spread Footings on granular pad	3	<ul style="list-style-type: none">■ Conventional construction.■ Removes potential complications associated artesian groundwater conditions.	<ul style="list-style-type: none">■ Construction of granular pad will cause consolidation settlement of the underlying native soils.■ Front slope will likely not be stable with extra load from footings.■ Dewatered excavation (cofferdam) required adjacent to the river to allow for construction of a granular pad in-the-dry.■ Not suitable for integral abutment design.	<ul style="list-style-type: none">■ Typically lower cost than deep foundations.■ Cost of extra fill for granular pad.	<ul style="list-style-type: none">■ Post-construction settlement of subsoils will occur and will require mitigation prior to footing construction.■ Some risk of settlement of the granular pad if sub-aqueous filling is carried out (without compaction).



Table 2: Evaluation of Stability and Settlement Mitigation Alternatives

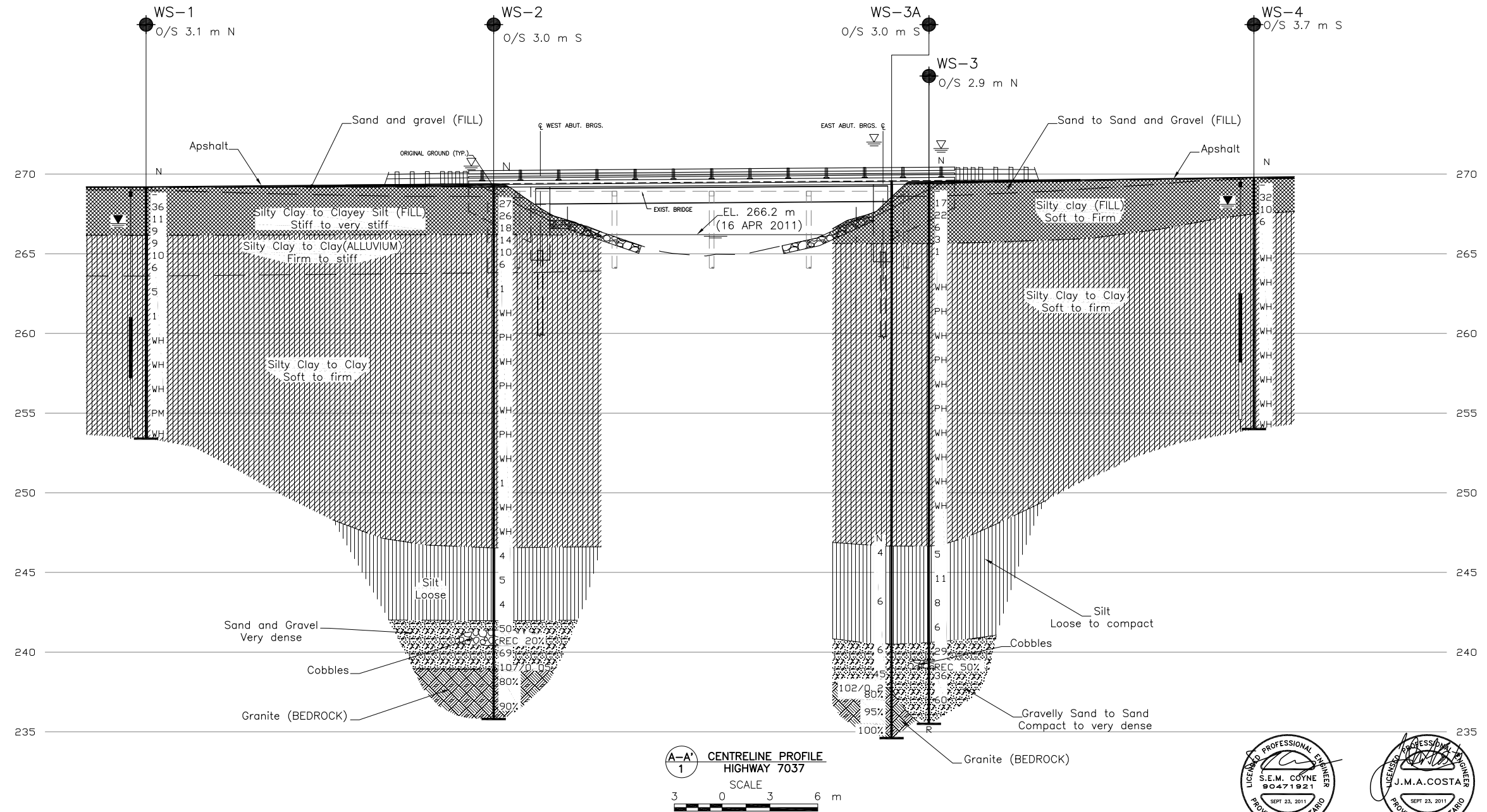
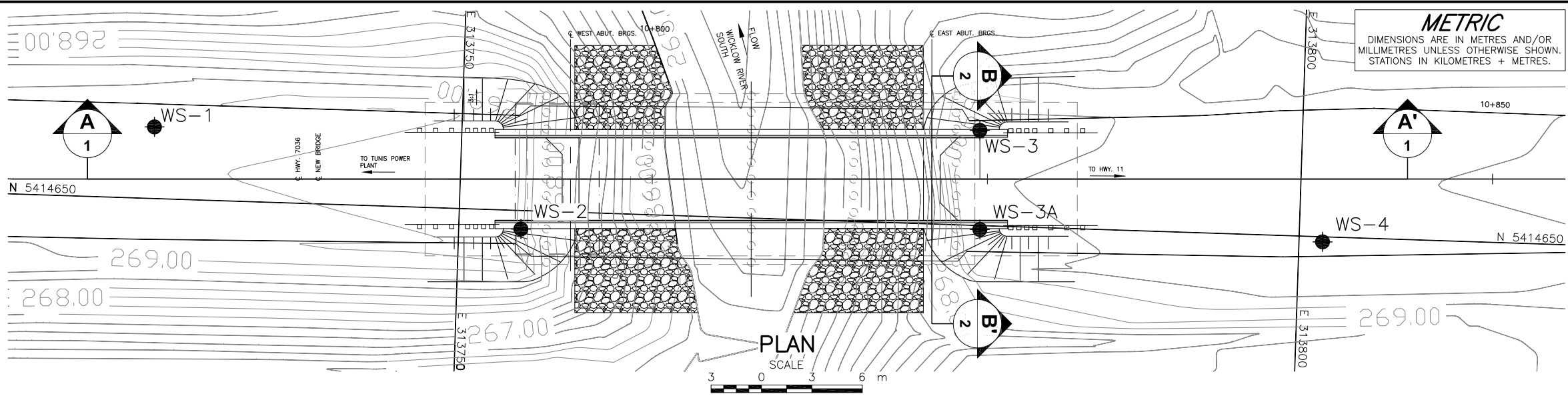
Mitigation Option	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Lightweight Fill (EPS)	1	<ul style="list-style-type: none"> Reduces load on compressible subsoils thereby reducing post-construction settlement of foundation subsoils. Straightforward installation of EPS blocks within embankment fill. No time delay. Eliminates downdrag loading on piles. 	<ul style="list-style-type: none"> High cost of EPS material. Requires concrete slab and minimum of 1 m of conventional soil cover to mitigate potential for differential icing. Use restricted to above groundwater level. 	<ul style="list-style-type: none"> EPS cost is up to an order of magnitude higher than other fill materials. 	
Change in Bridge Geometry <ul style="list-style-type: none"> Move abutments back Lower the grade 	2	<ul style="list-style-type: none"> Moving abutments back reduces magnitude of post-construction settlement of subsoils at the abutment locations. Lowering grade would reduce post-construction consolidation settlement of subsoils. 	<ul style="list-style-type: none"> Consolidation settlement of subsoils would still occur due to difference between existing and new fill, however, would be less than for abutments in currently proposed location. If grade is lowered, resulting lower pile caps would require excavation below the water level and additional/deeper shoring/dewatering for construction in-the-dry. 	<ul style="list-style-type: none"> Possible additional costs for shoring/cofferdam. Longer bridge span would be more expensive. 	<ul style="list-style-type: none"> Mitigation of settlement may still be required. Longer bridge could require central pier.
Preloading	NF	<ul style="list-style-type: none"> Reduces post-construction settlement. 	<ul style="list-style-type: none"> Time delay in schedule to allow for preloading (or staged preloading). Logistically difficult to construct staged preload in limited area beside river. Instrumentation and monitoring program would be required. 	<ul style="list-style-type: none"> Increased overall costs due to extra time required in schedule. Cost of instrumentation and monitoring program. 	<ul style="list-style-type: none"> Creep settlement will occur.



**FOUNDATION REPORT, REPLACEMENT OF WICKLOW RIVER BRIDGE SOUTH
HIGHWAY 7036, SITE NO. 39E-073 GWP 5139-06-00**

Mitigation Option	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Sub-excavation of Silty Clay to Clay Deposit	NF	<ul style="list-style-type: none">■ Reduces post-construction settlement.	<ul style="list-style-type: none">■ Requires excavation below the water level.■ Additional/deeper shoring/dewatering may be required.■ Requires site for disposal of excavated material.	<ul style="list-style-type: none">■ Cost of disposal of excess material and replacement backfill.■ Cost of excavation/shoring.	<ul style="list-style-type: none">■ High risk of post-construction settlement of fill material placed below the water level.
Wick Drains	NF	<ul style="list-style-type: none">■ Reduces time for primary consolidation to occur.	<ul style="list-style-type: none">■ Increases magnitude of creep settlement.■ Must be used in conjunction with preloading/staged construction due to stability.	<ul style="list-style-type: none">■ Additional cost of foundation investigation and design and instrumentation and monitoring program.	<ul style="list-style-type: none">■ Creep settlement will occur.

NF: Considered not technically, cost or schedule feasible to mitigate stability and/or settlement.



CONT No.
WP No. 5139-06-00

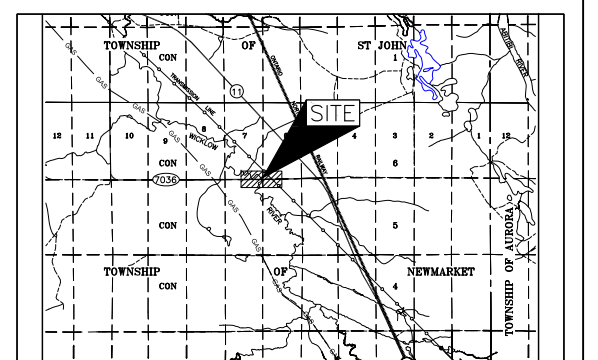
WICKLOW RIVER BRIDGE SOUTH
HIGHWAY 7036
BOREHOLE LOCATIONS AND
SOIL STRATA



SHEET



Golder Associates Ltd.
SUDBURY, ONTARIO, CANADA



LEGEND

- Borehole
- Seal
- Piezometer
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- R Refusal
- 100% Rock Quality Designation (RQD)
- WL in piezometer, measured on JULY 03, 2011
- WL upon completion of drilling

BOREHOLE CO-ORDINATES			
No.	ELEVATION	NORTHING	EASTING
WS-1	269.2	5414654.3	313731.7
WS-2	269.4	5414648.9	313753.7
WS-3	269.5	5414655.7	313780.8
WS-3A	269.6	5414649.8	313781.0
WS-4	269.8	5414649.7	313801.4

NOTES

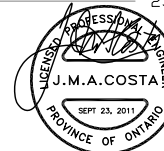
This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contracts Documents.

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

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

REFERENCE

Base plan provided in digital format by MMM, drawing file no. 8743-WS-S01.dwg, dated MAR 2011, received APR 26, 2011, x8743-Wicklow S-Contour.dwg, received APR 27, 2011, x8743 WS Base.dwg received APR 28, 2011.



NO.	DATE	BY	REVISION
1	SEP 23, 2011	J.J.L.	ISSUED FOR CONSTRUCTION
Geocres No. 42A-87			
HWY. 7036		PROJECT NO. 09-1191-0022	DIST.
SUBM'D. DAM	CHKD. SEMC	DATE: SEPT 2011	SITE: 39E-073
DRAWN: J.J.L.	CHKD.	APPD. JMAC	DWG. 1

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

CONT No.
WP No. 5139-06-00



WICKLOW RIVER BRIDGE SOUTH
HIGHWAY 7036

SHEET

SOIL STRATA



Golder Associates Ltd.
SUDBURY, ONTARIO, CANADA

LEGEND

- Borehole
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated
(Std. Pen. Test, 475 j/blow)
- R Refusal
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- WL in piezometer, measured on JULY 03, 2011
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WS-3	269.5	5414655.7	313780.8
WS-3A	269.6	5414649.8	313781.0
WS-4	269.8	5414649.7	313801.4

NOTES

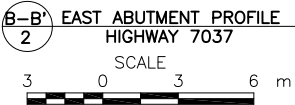
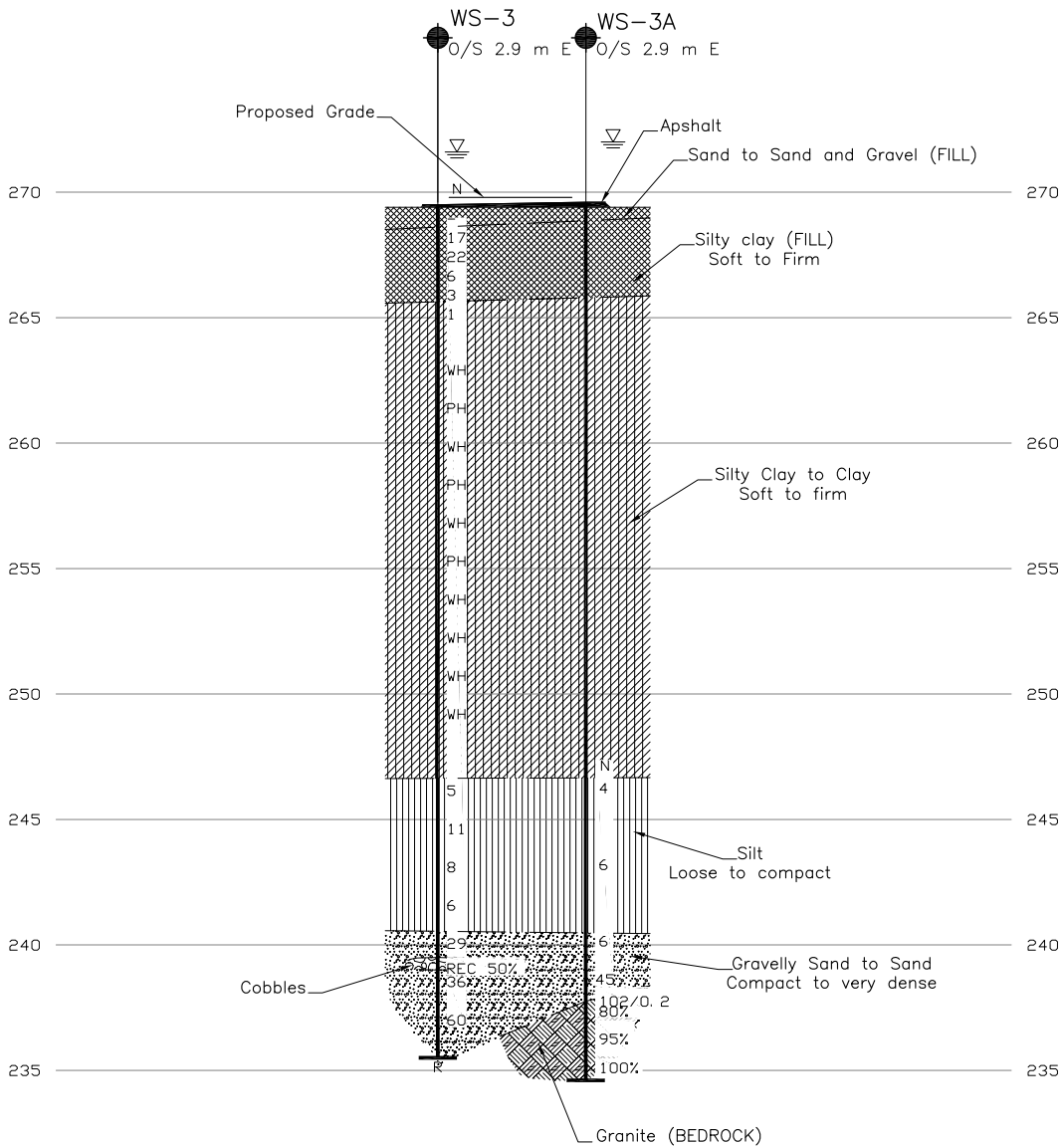
This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contracts Documents.

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

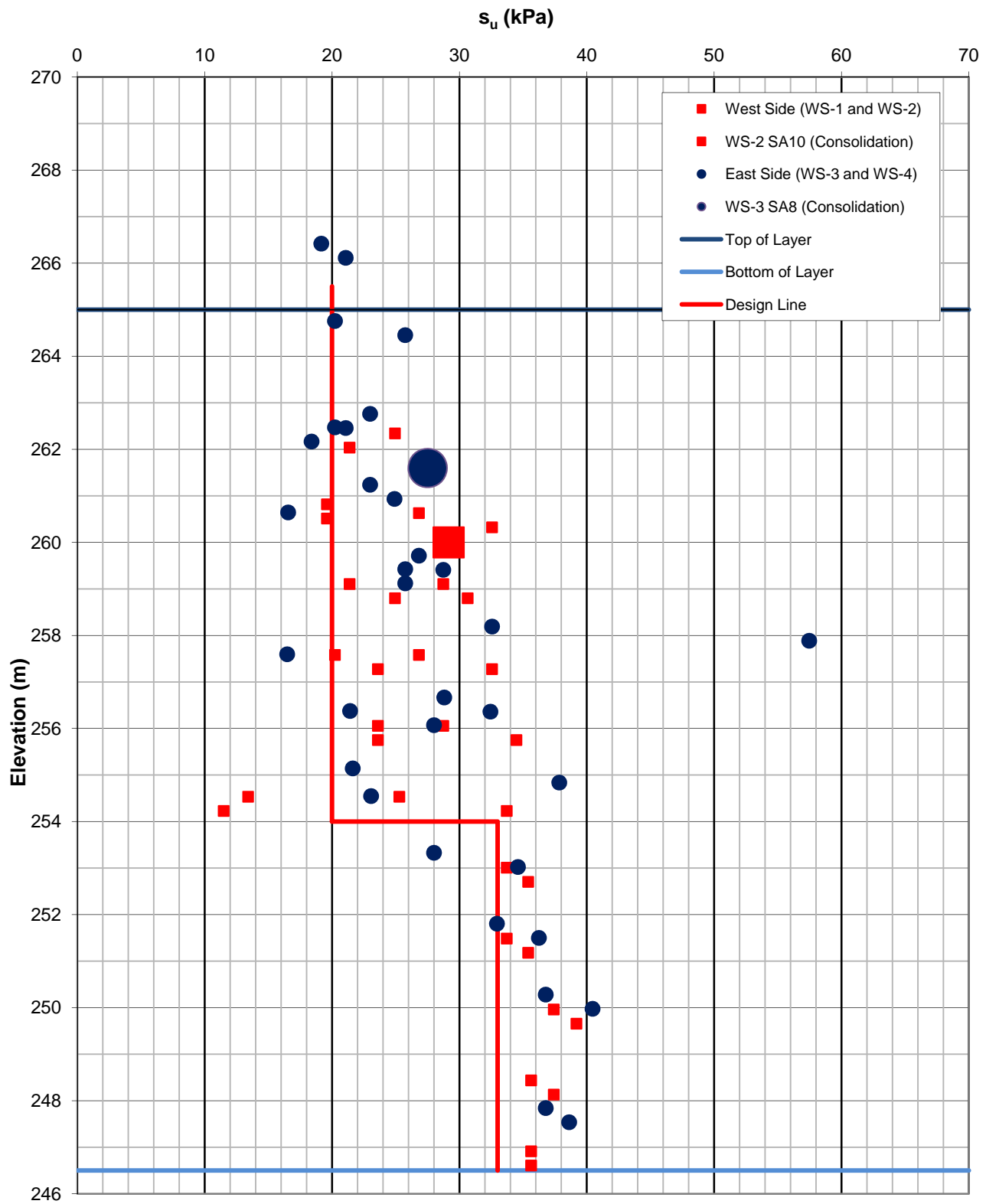
The complete Foundation Investigation and Design Report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with Section GC 2.01 of OPS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


REFERENCE

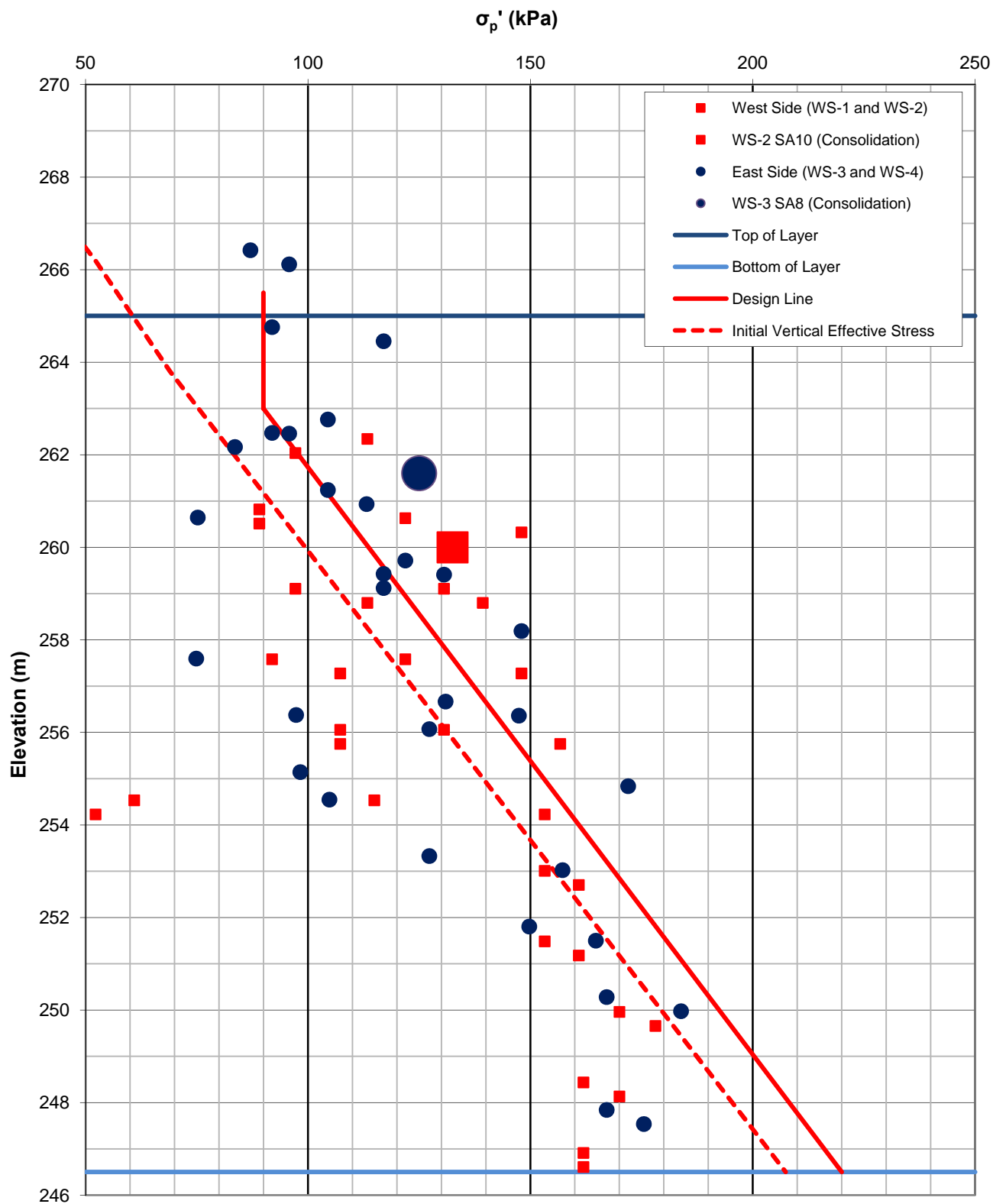
Base plan provided in digital format by MMM, drawing file no. 8743-WS-S01.dwg, dated MAR 2011, received APR 26, 2011, x8743-Wicklow S-Contour.dwg, received APR 27, 2011, x8743 WS Base.dwg received APR 28, 2011.



NO.	DATE	BY	REVISION
Geocres No. 42A-87			
HWY. 7036	PROJECT NO. 09-1191-0022	DIST.	
SUBM'D. DAM	CHKD. SEMC	DATE: SEPT 2011	SITE: 39E-073
DRAWN: JJL	CHKD.	APPD. JMAC	DWG. 2



PROJECT		WICKLOW RIVER BRIDGE SOUTH HIGHWAY 7036			
TITLE		UNDRAINED SHEAR STRENGTH VERSUS ELEVATION			
		PROJECT No. 09-1191-0022		FILE No. ----	
		DESIGN	DAM	SEPT 2011	SCALE AS SHOWN REV.
		CADD	--		
		CHECK	SEMC	SEPT 2011	
		REVIEW	JMAC	SEPT 2011	
					Figure 1



Note: Pre-consolidation pressure values shown include correction suggested by Bjerrum for plasticity.

PROJECT

WICKLOW RIVER BRIDGE SOUTH
HIGHWAY 7036

TITLE

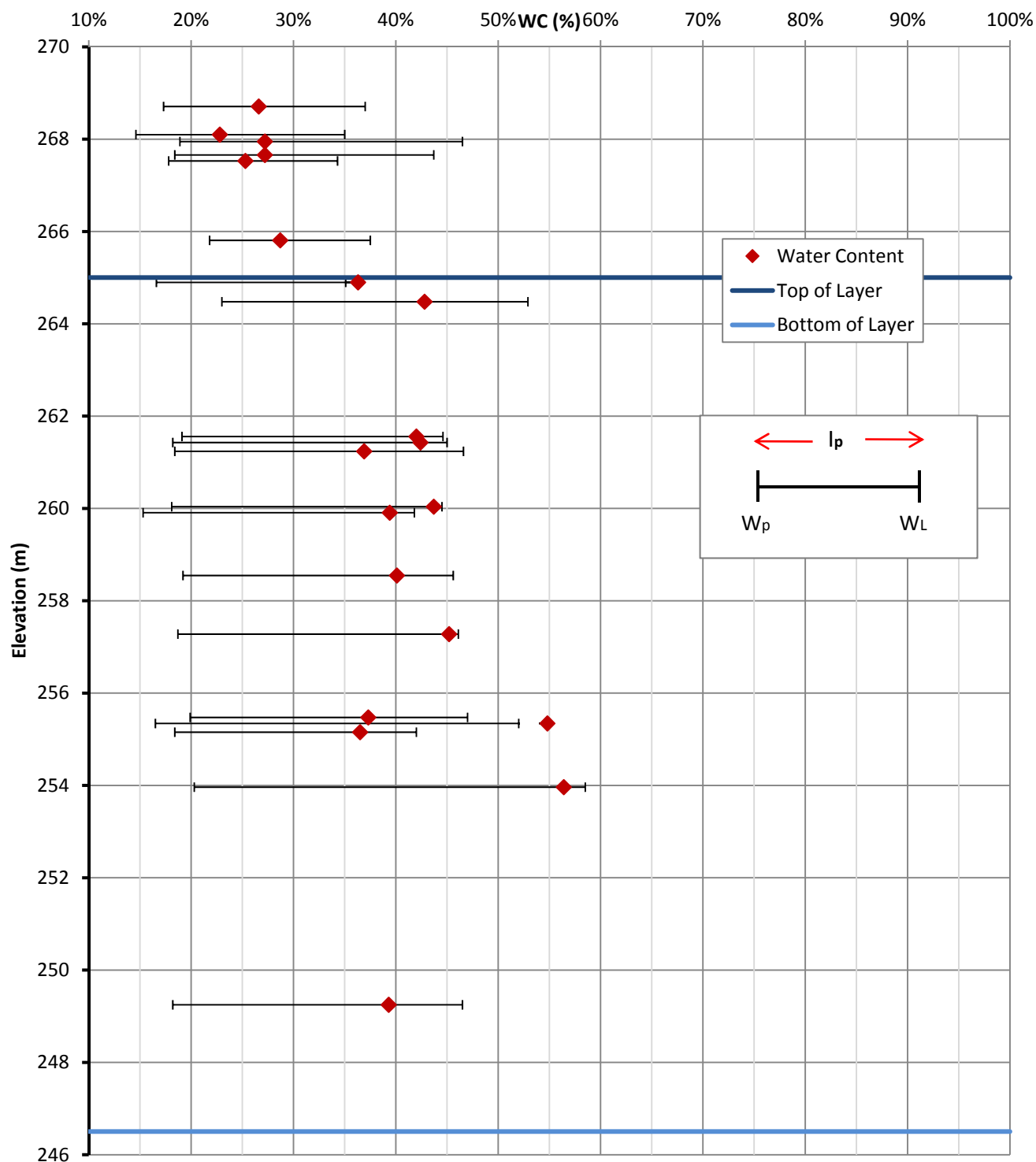
PRE-CONSOLIDATION PRESSURE VERSUS
ELEVATION




DESIGN	DAM	SEPT 2011
CADD	--	
CHECK	SEMC	SEPT 2011
REVIEW	JMAC	SEPT 2011

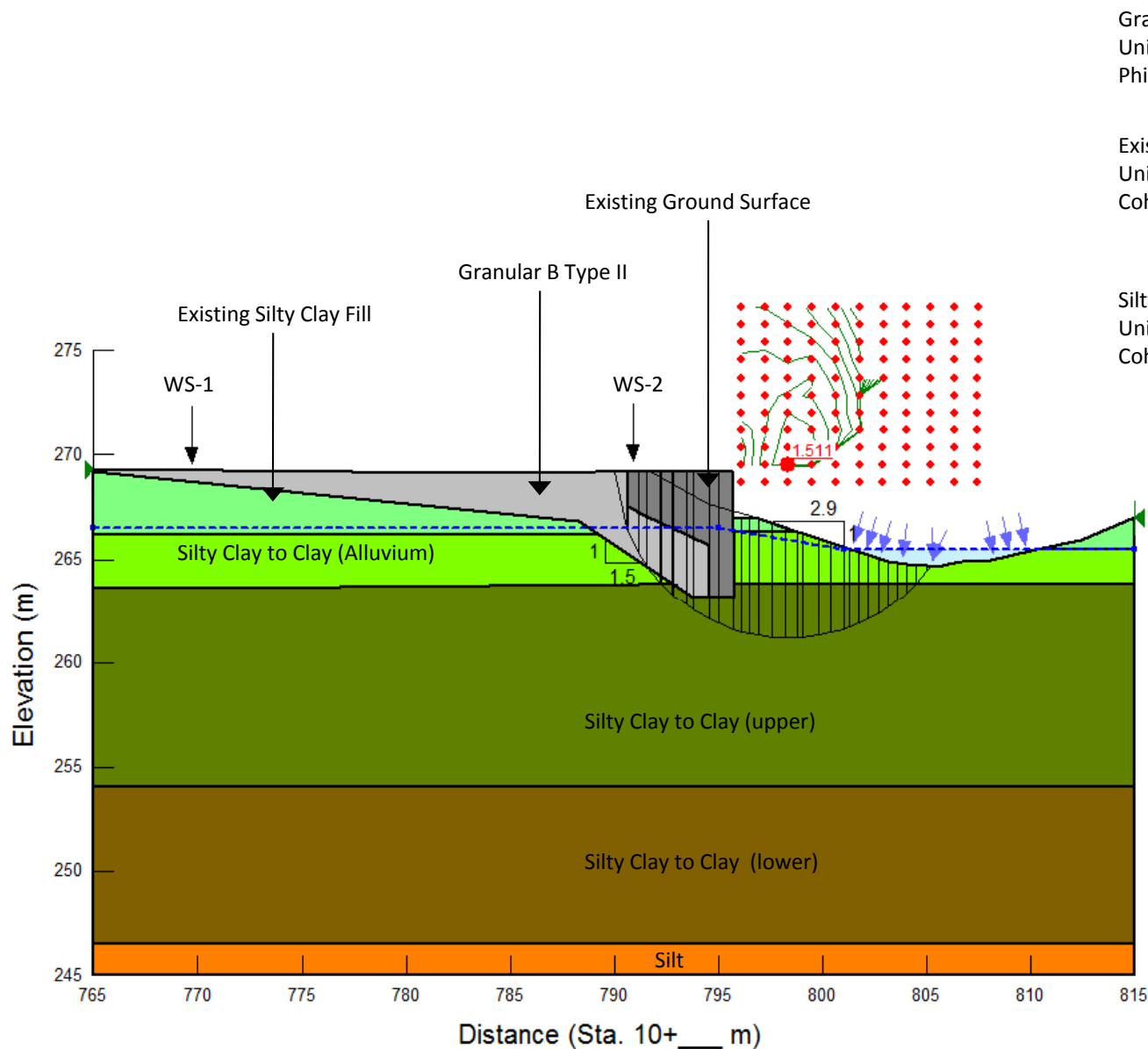
FILE No.	----
SCALE	AS SHOWN
REV.	

Figure 2



Note: Water content and Atterberg limits for the silty clay to clay layer between elevation 265 m and 247 m were used for collection of C_v and $C_{\alpha\epsilon}$ values, as noted in the text of the report.

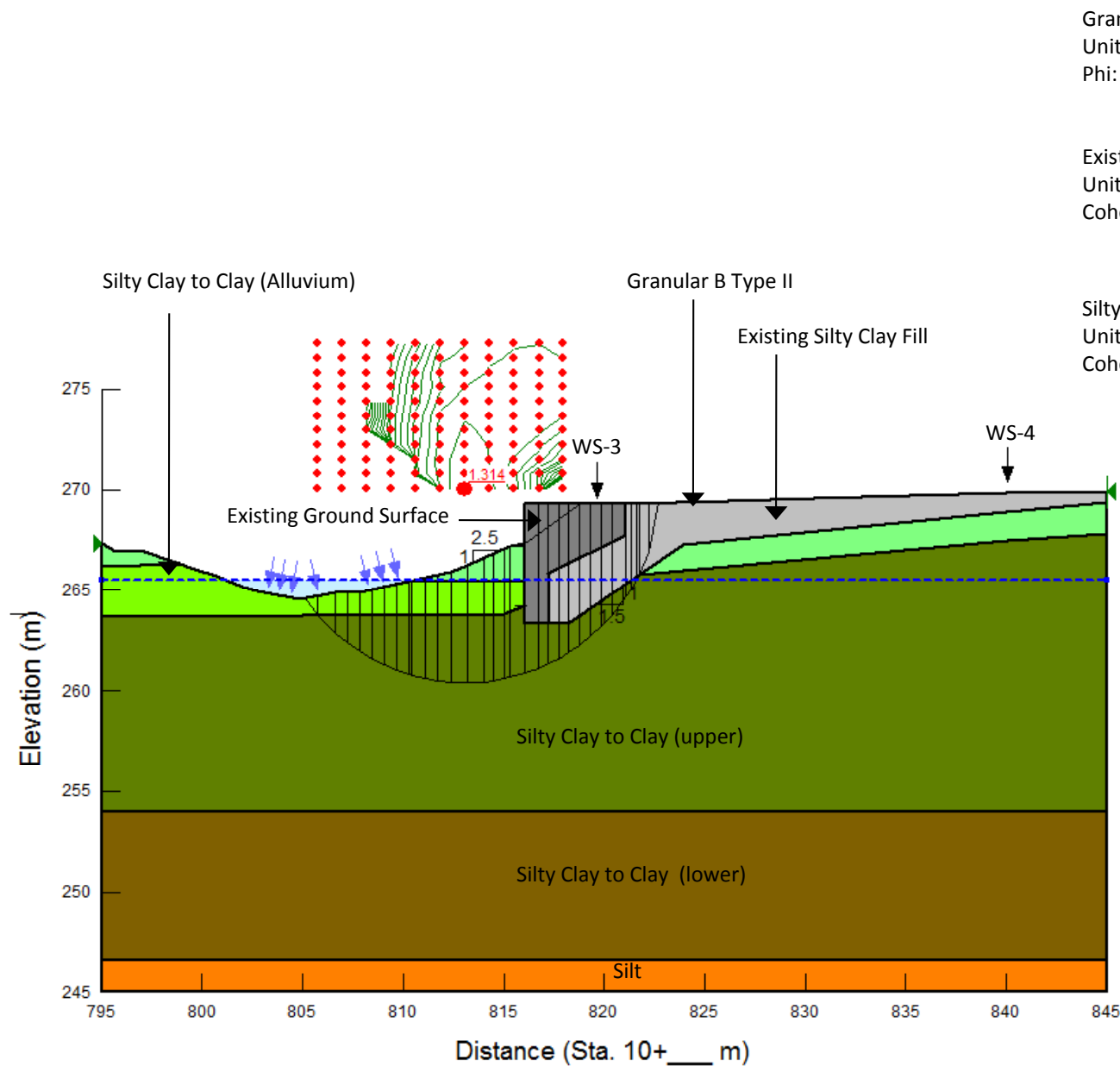
PROJECT		WICKLOW RIVER BRIDGE SOUTH HIGHWAY 7036			
TITLE		WATER CONTENT AND ATTERBERG LIMITS VERSUS ELEVATION			
		PROJECT No. 09-1191-0022		FILE No. ----	
		DESIGN	DAM	SEPT 2011	SCALE AS SHOWN REV.
		CADD	--		
		CHECK	SEMC	SEPT 2011	
		REVIEW	JMAC	SEPT 2011	
					Figure 3



PROJECT				WICKLOW RIVER BRIDGE SOUTH HIGHWAY 7036			
TITLE				STABILITY ANALYSIS WEST FRONT SLOPE			
PROJECT No. 09-1191-0022				FILE No. ----			
DESIGN	DAM	SEPT 2011		SCALE	AS SHOWN	REV.	
CADD	--						
CHECK	SEMC	SEPT 2011					
REVIEW	JMAC	SEPT 2011					



Figure 4



Granular B Type II
Unit Weight: 21/m³
Phi: 35 °

Silty Clay to Clay (upper)
Unit Weight: 18 kN/m³
Cohesion: 20 kPa

Existing Silty Clay Fill
Unit Weight: 18 kN/m³
Cohesion : 50 kPa

Silty Clay to Clay (lower)
Unit Weight: 18 kN/m³
Cohesion : 33 kPa

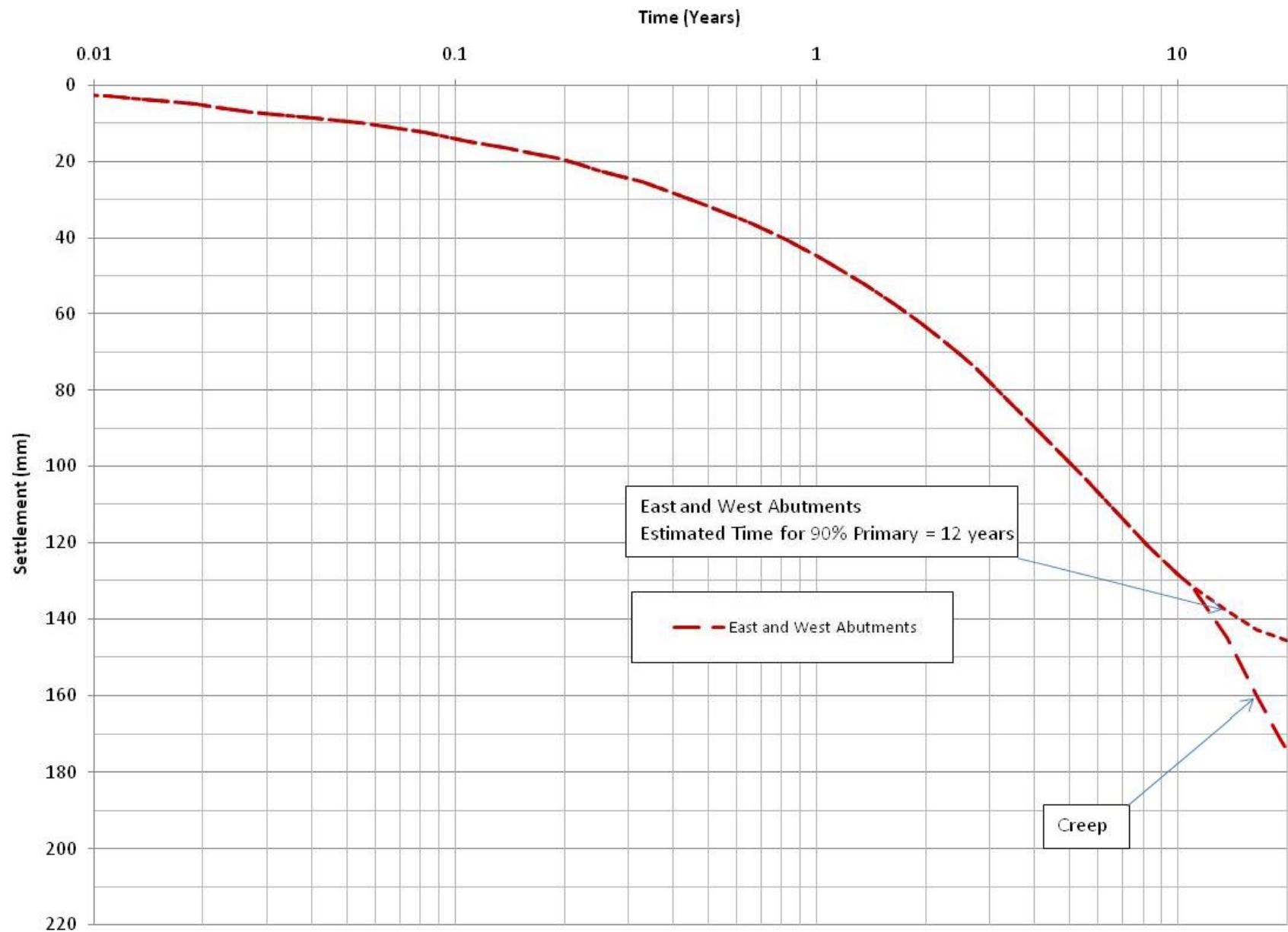
Silty Clay to Clay (Alluvium)
Unit Weight: 17 kN/m³
Cohesion: 50 kPa

Silt
Unit Weight: 19 kN/m³
Phi: 27 °

PROJECT		WICKLOW RIVER BRIDGE SOUTH HIGHWAY 7036			
TITLE		STABILITY ANALYSIS EAST FRONT SLOPE			
		PROJECT No. 09-1191-0022		FILE No. ----	
DESIGN	DAM	SEPT 2011	SCALE	AS SHOWN	REV.
CADD	--				
CHECK	SEMC	SEPT 2011			
REVIEW	JMAC	SEPT 2011			



Figure 5



PROJECT

WICKLOW RIVER BRIDGE SOUTH
HIGHWAY 7036

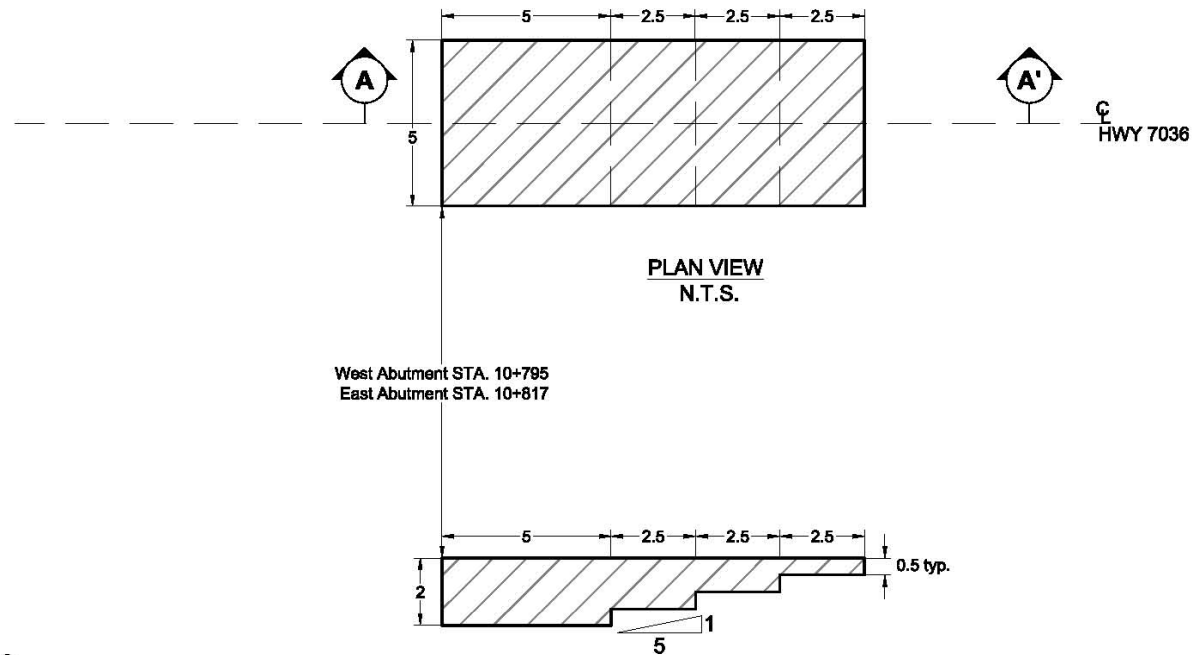
TITLE

ESTIMATED CONSOLIDATION SETTLEMENT VERSUS TIME



PROJECT No.	09-1191-0022	FILE No.	----
DESIGN	DAM	SCALE	AS SHOWN
CADD	--	REV.	
CHECK	SEMC	SEPT 2011	
REVIEW	JMAC	SEPT 2011	

Figure 6



NOTES:

1. All units are in metres.
2. Refer to accompanying Foundation Design Report, Section 6.6.5.1
3. Bedding of 300 mm at GBT2 and 100 mm mortar sand, levelling pad shall be placed below the EPS.
4. A minimum 300 mm thick layer of Granular "B" Type I should be placed above the side slopes of the EPS.
5. All EPS, including side slopes, should be protected with a minimum 1.2 m of soil cover and 6 mm thick polyethylene sheeting.
6. A 125 mm thick concrete slab is required on the top of the highest level of EPS blocks.

SECTION A-A'
N.T.S.


PROJECT		WICKLOW RIVER BRIDGE SOUTH HIGHWAY 7036			
TITLE		RECOMMENDED EPS CONFIGURATION			
		PROJECT No.	09-1191-0022	FILE No.	----
		DESIGN	DAM	SEPT 2011	SCALE AS SHOWN
		CADD	--		REV.
		CHECK	SEMC	SEPT 2011	
		REVIEW	JMAC	SEPT 2011	

Figure 7



APPENDIX A

Record of Boreholes and Drillholes



LIST OF SYMBOLS

Unless otherwise stated, the symbols employed in the report are as follows:

1. GENERAL

π	3.1416
$\ln x$,	natural logarithm of x
\log_{10}	x or log x, logarithm of x to base 10
g	acceleration due to gravity
t	time
FoS	Factor of Safety
V	volume
W	weight

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. stress: $\Delta\sigma$
ε	linear strain
ε_v	volumetric strain
η	coefficient of viscosity
ν	Poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma - u$)
σ_{vo}	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stress (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

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

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

(a) Index Properties (continued)

w	water content
w_l	liquid limit
w_p	plastic limit
I_p	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_a	coefficient of secondary consolidation
m_v	coefficient of volume change
c_v	coefficient of consolidation
T_v	time factor (vertical direction)
U	degree of consolidation
σ'_p	pre-consolidation pressure
OCR	over-consolidation ratio $= \sigma'_p / \sigma'_{vo}$

(d) Shear Strength

τ_p, τ_r	peak and residual shear strength
ϕ'	effective angle of internal friction
δ	angle of interface friction
μ	coefficient of friction $= \tan \delta$
c'	effective cohesion
c_u, s_u	undrained shear strength ($\phi = 0$ analysis)
p	mean total stress $(\sigma_1 + \sigma_3)/2$
p'	mean effective stress $(\sigma'_1 + \sigma'_3)/2$
q	$(\sigma_1 + \sigma_3)/2$ or $(\sigma'_1 + \sigma'_3)/2$
q_u	compressive strength $(\sigma_1 + \sigma_3)$
S_t	sensitivity

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
SS	Split-spoon
DS	Denison type sample
FS	Foil sample
RC	Rock core
SC	Soil core
ST	Slotted tube
TO	Thin-walled, open
TP	Thin-walled, piston
WS	Wash sample

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

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

Dynamic Cone Penetration Resistance; N_d :

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

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

Piezo-Cone Penetration Test (CPT)

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

III. SOIL DESCRIPTION

(a) Cohesionless Soils

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

(b) Cohesive Soils Consistency

	C_u, S_u	
	kPa	psf
Very soft	0 to 12	0 to 250
Soft	12 to 25	250 to 500
Firm	25 to 50	500 to 1,000
Stiff	50 to 100	1,000 to 2,000
Very stiff	100 to 200	2,000 to 4,000
Hard	over 200	over 4,000

IV. SOIL TESTS

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

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

V. MINOR SOIL CONSTITUENTS

Percent 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 (cohesionless) or With (cohesive)	Sand and Gravel Silty Clay with sand / Clayey Silt with sand



LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

WEATHERING 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 texture and structure are preserved.

BEDDING THICKNESS

<u>Description</u>	<u>Bedding Plane Spacing</u>
Very thickly bedded	> 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	< 6 mm

JOINT OR FOLIATION SPACING

<u>Description</u>	<u>Spacing</u>
Very wide	> 3 m
Wide	1 – 3 m
Moderately close	0.3 – 1 m
Close	50 – 300 mm
Very close	< 50 mm

GRAIN SIZE

<u>Terms</u>	<u>Size*</u>
Very Coarse Grained	> 60 mm
Coarse Grained	2 – 60 mm
Medium Grained	60 microns – 2 mm
Fine Grained	2 – 60 microns
Very Fine Grained	< 2 microns

* Note: Grains > 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 varies 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 separation) in the rock core, including both naturally occurring fractures and mechanically induced breaks caused by drilling.

Dip with Respect to (W.R.T.) Core Axis

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

Description and Notes

An abbreviated description of the discontinuities, whether naturally occurring separation such as fractures, bedding planes and foliation planes or mechanically induced fractures 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



B - Bedding	⊥ - Perpendicular To
FO - Foliation / Schistosity	- Parallel To
CL - Cleavage	P - Polished
SH - Shear Plane / Zone	K - Slickensided
VN - Vein	SM - Smooth
F - Fault	R - Rough
CO - Contact	ST - Stepped
J - Joint	PL - Planar
FR - Fracture	U - Undulating
MF - Mechanical Fracture	C - Curved

PROJECT 09-1191-0022				RECORD OF BOREHOLE No WS-1				1 OF 2 METRIC							
W.P. 5139-06-00				LOCATION N 5414654.3; E 313731.7				ORIGINATED BY DAM							
DIST Cochrane HWY 7036				BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers				COMPILED BY JLL							
DATUM Geodetic				DATE April 17, 2011				CHECKED BY SC							
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE		DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES		20	40	60	80					
269.2	GROUND SURFACE														
0.0	APSHALT (Surface Treatment)		1	AS	-										
0.2	Sand and gravel, trace silt (FILL) Frozen Brown		2	SS	36										
	Silty clay with sand, some gravel, slightly organic (FILL) (Frozen) Stiff and moist below 2.3 m depth Brown to black		3	SS	11										
			4	SS	9										
266.2															
3.0	SILTY CLAY, trace to some sand, slightly organic (ALLUVIUM) Firm to stiff Grey Moist		5	SS	9										
			6	SS	10										
			7	SS	6										
263.6															
5.6	SILTY CLAY, trace sand Firm Grey Wet		8	SS	5										
			9	SS	1										
			10	SS	WH										
			11	SS	WH										
			12	SS	WH										
			13	SS	PM										
	Soft below 14.6 m depth.														

Continued Next Page

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

SUD-MTO 001 09-1191-0022.GPJ GAL-MISS.GDT 28/09/11 DATA INPUT:


PROJECT		RECORD OF BOREHOLE				No WS-1		2 OF 2		METRIC							
W.P. 5139-06-00		LOCATION N 5414654.3; E 313731.7				ORIGINATED BY		DAM									
DIST Cochrane HWY 7036		BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers				COMPILED BY		JJL									
DATUM Geodetic		DATE April 17, 2011				CHECKED BY		SC									
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
	--- CONTINUED FROM PREVIOUS PAGE ---							20	40	60	80	100					
253.4	SILTY CLAY, trace sand Firm Grey Wet		14	SS	WH		254										
15.8	END OF BOREHOLE																
	Note: 1. Water level at a depth of 13.5 m below ground surface (Elev. 255.7 m) upon completion of drilling. 2. Water level in piezometer at a depth of 3.5 m (Elev. 265.7 m) on April 28, 2011, and 2.3 m (Elev. 266.9 m) on July 3, 2011.																

PROJECT 09-1191-0022		RECORD OF BOREHOLE No WS-2		1 OF 3 METRIC													
W.P. 5139-06-00		LOCATION N 5414648.9; E 313753.7		ORIGINATED BY ID													
DIST Cochrane HWY 7036		BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers, NW Casing, NQ Coring		COMPILED BY JJL													
DATUM Geodetic		DATE April 25 and 26, 2011		CHECKED BY SC													
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa		WATER CONTENT (%)		γ		GR SA SI CL			
269.4	GROUND SURFACE							20 40 60 80 100	○ UNCONFINED + FIELD VANE	W _p W W _L	20 40 60						
0.0	ASPHALT (Surface Treatment)		1	AS	-		269		● QUICK TRIAXIAL × REMOULDED								
0.2	Sand and gravel, trace silt (FILL) Frozen Brown		2	SS	27		268										
268.6	Clayey silt to silty clay, trace to some sand, trace to some gravel, slightly organic (FILL) (Frozen) Very stiff and moist below 2.3 m depth Brown		3	SS	26		267										
			4	SS	18		266										
266.2	SILTY CLAY to CLAY, some sand, organic (ALLUVIUM) Firm to stiff Brown to grey Moist		5a				265										
3.2			5b	SS	14		264										
			6	SS	10		263										
			7	SS	6		262										
263.8	SILTY CLAY to CLAY, trace sand Soft to firm Grey Wet Switched to NW Casing at 15.2 m depth.		8	SS	1		261										
5.6			9	SS	WH		260										
			10	TO	PH		259										
			11	SS	WH		258										
			12	TO	PH		257										
			13	SS	WH		256										
							255										

SUD-MTO 001 09-1191-0022.GPJ GAL-MISS.GDT 28/09/11 DATA INPUT:

Continued Next Page


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

PROJECT 09-1191-0022			RECORD OF BOREHOLE No WS-2			2 OF 3 METRIC																
W.P. 5139-06-00			LOCATION N 5414648.9; E 313753.7			ORIGINATED BY ID																
DIST Cochrane HWY 7036			BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers, NW Casing, NQ Coring			COMPILED BY JJL																
DATUM Geodetic			DATE April 25 and 26, 2011			CHECKED BY SC																
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)							
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa					WATER CONTENT (%)			γ			GR SA SI CL			
	--- CONTINUED FROM PREVIOUS PAGE ---							20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					W _p W W _L 20 40 60			kN/m ³						
	SILTY CLAY to CLAY, trace sand Soft to firm Grey Wet		14	TO	PH		254															
	Switched to NW Casing at 15.2 m depth.						253															
	Silt layers below 16.8 m depth.		15	SS	WH		252															
			16	SS	1		251															
			17	SS	WH		250															
			18	SS	WH		249															
			19	SS	4		248															
			20	SS	5		247															
246.5	SILT, some clay and / or clay layers, trace sand Loose Grey Wet		21	SS	4		246															
242.0	Artesian conditions (water flowing out of casing) below 25.9 m depth.		22	SS	50		245															
27.4	SAND and GRAVEL, trace to some silt, trace clay, containing cobbles Very dense Grey Wet		23	SS	69		244															
	Casing refusal at 28.0 m depth. Switched to NQ coring between 28.0 m and 29.0 m depth. Recovered gravel and cobbles between 0.05 m and 0.18 m sizes.			RC	REC 20%		243															
	NQ coring used to pilot borehole between 29.5 m and 30.5 m depth prior to advancing casing.						242															
							241															
							240															

SUD-MTO 001 09-1191-0022.GPJ GAL-MISS.GDT 28/09/11 DATA INPUT:

Continued Next Page

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

PROJECT		RECORD OF BOREHOLE				No WS-2		3 OF 3		METRIC								
W.P. 09-1191-0022		LOCATION				N 5414648.9; E 313753.7		ORIGINATED BY ID										
DIST Cochrane HWY 7036		BOREHOLE TYPE				108 mm I.D. Continuous Flight Hollow Stem Augers, NW Casing, NQ Coring		COMPILED BY JJL										
DATUM Geodetic		DATE				April 25 and 26, 2011		CHECKED BY SC										
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa										
	--- CONTINUED FROM PREVIOUS PAGE ---						20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					WATER CONTENT (%) 20 40 60						
238.9 30.5	Spoon refusal at 30.5 m depth. GRANITE (BEDROCK)		24	SS	07/0.65													
	Bedrock cored from 30.5 m depth to 33.6 m depth. For coring details see Record of Drillhole WS-2.		1	RC	REC 100%													
			2	RC	REC 100%													
235.8 33.6	END OF BOREHOLE Note: 1. Water level at 1.1 m above ground surface (Elev. 270.5 m) upon penetrating the silt deposit. 2. Water level at a depth of 3.7 m below ground surface (Elev. 265.7 m) upon penetrating the gravelly sand deposit.																	

SUD-MTO 001 09-1191-0022.GPJ GAL-MISS.GDT 28/09/11 DATA INPUT:

PROJECT: 09-1191-0022

RECORD OF DRILLHOLE: WS-2

SHEET 1 OF 1

LOCATION: N 5414648.9 ;E 313753.7

DRILLING DATE: April 26, 2011

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: —

DRILL RIG: CME 55

DRILLING CONTRACTOR: George Downing Estate Drilling Ltd.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	COLOUR % RETURN	JN - Joint FLT - Fault SHR - 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 MB - Mechanical Break												BR - Broken Rock	NOTE: For additional abbreviations refer to list of abbreviations & symbols.	NOTES WATER LEVELS INSTRUMENTATION																																																																																																																																																																																																																																																																					
							RECOVERY				R.Q.D. %	FRACT. INDEX METRES	DISCONTINUITY DATA												HYDRAULIC CONDUCTIVITY k, cm/s				Diametral Point Load Index (MPa)	RMC -Q AVG																																																																																																																																																																																																																																																																																																
							FLUSH	TOTAL CORE %	SOLID CORE %	DISCONTINUITY DATA			Jr	Ja	Jn	DISCONTINUITY DATA	DISCONTINUITY DATA	DISCONTINUITY DATA	DISCONTINUITY DATA	DISCONTINUITY DATA	DISCONTINUITY DATA	DISCONTINUITY DATA	DISCONTINUITY DATA	DISCONTINUITY DATA	DISCONTINUITY DATA	DISCONTINUITY DATA	DISCONTINUITY DATA	DISCONTINUITY DATA			DISCONTINUITY DATA	DISCONTINUITY DATA	DISCONTINUITY DATA	DISCONTINUITY DATA	DISCONTINUITY DATA	DISCONTINUITY DATA	DISCONTINUITY DATA	DISCONTINUITY DATA	DISCONTINUITY DATA	DISCONTINUITY DATA	DISCONTINUITY DATA	DISCONTINUITY DATA	DISCONTINUITY DATA	DISCONTINUITY DATA	DISCONTINUITY DATA	DISCONTINUITY DATA	DISCONTINUITY DATA	DISCONTINUITY DATA	DISCONTINUITY DATA	DISCONTINUITY DATA	DISCONTINUITY DATA	DISCONTINUITY DATA	DISCONTINUITY DATA	DISCONTINUITY DATA				DISCONTINUITY DATA	DISCONTINUITY DATA	DISCONTINUITY DATA	DISCONTINUITY DATA	DISCONTINUITY DATA	DISCONTINUITY DATA	DISCONTINUITY DATA	DISCONTINUITY DATA	DISCONTINUITY DATA	DISCONTINUITY DATA	DISCONTINUITY DATA	DISCONTINUITY 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DEPTH SCALE

1 : 50



LOGGED: ID

CHECKED: SC


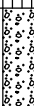
SUD-RCK 09-1191-0022.GPJ GAL-MISS.GDT 2809/11 DATA INPUT:

PROJECT		09-1191-0022		RECORD OF BOREHOLE No WS-3		1 OF 3 METRIC							
W.P.		5139-06-00		LOCATION		N 5414655.7; E 313780.8							
DIST		Cochrane HWY 7036		BOREHOLE TYPE		108 mm I.D. Continuous Flight Hollow Stem Augers							
DATUM		Geodetic		DATE		April 17 - 20, 2011							
						ORIGINATED BY ID							
						COMPILED BY JJL							
						CHECKED BY SC							
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT		UNIT WEIGHT		REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100	W _p W W _L	WATER CONTENT (%)	γ	GR SA SI CL	
269.5	GROUND SURFACE												
0.0	ASPHALT (Surface Treatment)		1	AS	-		269					13 52 13 12	
0.1	Sand, some gravel, silt and clay, slightly organic (FILL)												
268.6	Frozen Brown		2	SS	17		268						
0.9	Silty clay, trace to some sand, trace to some gravel, slightly organic (FILL) (Frozen)		3	SS	22		267						
	Soft to firm and moist below 2.3 m depth		4	SS	6		266						
	Brown		5	SS	3		265						
265.6	SILTY CLAY to CLAY, trace sand		6	SS	1		264						
3.9	Soft to firm						263						
	Grey		7	SS	WH		262						
	Wet		8	TO	PH		261						
	Vane sank 0.3 m at 8.5 m depth.		9	SS	WH		260						
			10	TO	PH		259						
	Vane sank 0.3 m at 11.6 m depth.		11	SS	WH		258						
			12	TO	PH		257						
	Vane sank 0.3 m at 14.6 m depth.						256						
							255						

Continued Next Page

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

SUD-MTO 001 09-1191-0022.GPJ GAL-MISS.GDT 28/09/11 DATA INPUT:

PROJECT 09-1191-0022			RECORD OF BOREHOLE No WS-3			2 OF 3 METRIC											
W.P. 5139-06-00			LOCATION N 5414655.7; E 313780.8			ORIGINATED BY ID											
DIST Cochrane HWY 7036			BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers			COMPILED BY JLL											
DATUM Geodetic			DATE April 17 - 20, 2011			CHECKED BY SC											
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa					WATER CONTENT (%)			γ kN/m³	GR SA SI CL
							20 40 60 80 100	20 40 60 80 100	W _p	W	W _L	20 40 60					
--- CONTINUED FROM PREVIOUS PAGE ---																	
	SILTY CLAY to CLAY, trace sand Soft to firm Grey Wet Switched to NW Casing at 15.2 m depth.		13	SS	WH		254										
								253	4								
								252	3								
								251	4								
								250	3								
								249	4								
			14	SS	WH		248	4									
							247										
246.6	SILT, some clay and / or clay layers Loose to compact Grey Wet		17	SS	5		246										
22.9							245										
	Artesian conditions noted below 24.4 m depth.		18	SS	11		244										
							243										
			19	SS	8		242										
							241										
			20	SS	6		240										
240.6	Gravelly SAND to SAND, some silt, trace clay, containing cobbles Compact to very dense Grey Wet		21	SS	29		240										
28.9																	

Continued Next Page

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

SUD-MTO 001 09-1191-0022.GPJ GAL-MISS.GDT 28/09/11 DATA INPUT:

PROJECT		09-1191-0022		RECORD OF BOREHOLE No WS-3				3 OF 3 METRIC									
W.P.		5139-06-00		LOCATION		N 5414655.7; E 313780.8		ORIGINATED BY ID									
DIST		Cochrane HWY 7036		BOREHOLE TYPE		108 mm I.D. Continuous Flight Hollow Stem Augers		COMPILED BY JJL									
DATUM		Geodetic		DATE		April 17 - 20, 2011		CHECKED BY SC									
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
	--- CONTINUED FROM PREVIOUS PAGE ---																
	Gravelly SAND to SAND, some silt, trace clay, containing cobbles Compact to very dense Grey Wet		22	RC	REC 50%		239										
	Casing refusal at 30.0 m depth. Spoon attempted / bouncing.		23	SS	36												1 86 10 3
	Switched to NQ coring between 30.0 m and 30.5 m depth. Recovered gravel and cobbles between 0.04 m and 0.13 m sizes, advanced casing.		24	SS	60		238										
	Casing refusal at 33.5 m depth.						237										
235.4	END OF BOREHOLE Tricone equipment lodged / lost at bottom of borehole						236										
34.1	Note: 1. Water level at 2.1m above ground surface (Elev. 271.6 m) upon penetrating the silt deposit. 2. Casing refusal encountered at 33.5 m depth. About 0.6 m of material (gravel and cobbles) inside the casing. Core barrel lodged inside casing after attempting to remove gravel and cobbles. After dislodging core barrel, tricone was used to remove cobbles from casing. Tricone advanced to 34.1 m depth and became lodged inside borehole. Tricone could not be dislodged. 3. Moved 5.9 m south and advanced new Borehole 22.9 m without sampling. See Record of Borehole WS-3A.																

SUD-MTO 001 09-1191-0022.GPJ GAL-MISS.GDT 28/09/11 DATA INPUT:



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

SUD-MTO 001 09-1191-0022.GPJ GAL-MISS.GDT 28/09/11 DATA INPUT:

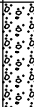


PROJECT <u>09-1191-0022</u>		RECORD OF BOREHOLE No WS-3A			2 OF 3 METRIC	
W.P. <u>5139-06-00</u>		LOCATION <u>N 5414649.8; E 313781.0</u>			ORIGINATED BY <u>ID</u>	
DIST <u>Cochrane</u> HWY <u>7036</u>		BOREHOLE TYPE <u>108 mm I.D. Continuous Flight Hollow Stem Augers, NW Casing, NQ Coring</u>			COMPILED BY <u>JJL</u>	
DATUM <u>Geodetic</u>		DATE <u>April 27, 2011</u>			CHECKED BY <u>SC</u>	

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa					W _p	W		
						○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					WATER CONTENT (%)					
	--- CONTINUED FROM PREVIOUS PAGE ---					20	40	60	80	100	20	40	60			
254																
253																
252																
251																
250																
249																
248																
247																
246.7 22.9	SILT, some clay and / or clay layers, trace sand Loose Grey Wet Artesian conditions noted at 25.9 m depth.		1	SS	4											
246																
245																
244																
243			2	SS	6										o	0 2 80 18
242																
241																
240.2 29.4		3	SS	6												
240																

SUD-MTO 001 09-1191-0022.GPJ GAL-MISS.GDT 28/09/11 DATA INPUT:

Continued Next Page

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

PROJECT		RECORD OF BOREHOLE No WS-3A				3 OF 3 METRIC											
W.P. 09-1191-0022		LOCATION N 5414649.8; E 313781.0				ORIGINATED BY ID											
DIST Cochrane HWY 7036		BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers, NW Casing, NQ Coring				COMPILED BY JJL											
DATUM Geodetic		DATE April 27, 2011				CHECKED BY SC											
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa									WATER CONTENT (%)
	--- CONTINUED FROM PREVIOUS PAGE ---																
237.8	Gravelly SAND to SAND, trace to some silt, trace clay Dense to very dense Grey Wet		4	SS	45		239										
31.8	Spoon refusal at 31.8 m depth. GRANITE (BEDROCK)		5	SS	102/0.2		238										
	Bedrock cored from 31.8 m depth to 35.0 m depth. For coring details see Record of Drillhole WS-3A		1	RC	REC 100%		237										RQD = 80%
			2	RC	REC 100%		236										RQD = 95%
			3	RC	REC 100%		235										RQD = 100%
234.6	END OF BOREHOLE																
35.0	Note: 1. Water level at 2.4 m above ground surface (Elev. 272.0 m) upon penetrating the silt deposit.																

SUD-MTO 001 09-1191-0022.GPJ GAL-MISS.GDT 28/09/11 DATA INPUT:

PROJECT: 09-1191-0022

RECORD OF DRILLHOLE: WS-3A

SHEET 1 OF 1

LOCATION: N 5414649.8 ;E 313781.0

DRILLING DATE: April 27, 2011

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: —

DRILL RIG: CME 55

DRILLING CONTRACTOR: George Downing Estate Drilling Ltd.

DEPTH SCALE METRES	DRILLING RECORD		DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	COLOUR % RETURN	JN - Joint FLT - Fault SHR - 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 MB - Mechanical Break																BR - Broken Rock	NOTE: For additional abbreviations refer to list of abbreviations & symbols.	NOTES WATER LEVELS INSTRUMENTATION																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																									
								RECOVERY				R.Q.D. %	FRACT INDEX METRES	DISCONTINUITY DATA												HYDRAULIC CONDUCTIVITY k, cm/s				Diametral Point Load Index (MPa)	RMC -Q AVG																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																				
								TOTAL CORE %	SOLID CORE %	FLUSH	FRACT INDEX METRES			B Angle	DIP w.r.t CORE AXIS	TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jn	K	SM	Ro	MB	K	SM	Ro	MB																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																								
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32	NW	REFER TO PREVIOUS PAGE		237.8 31.8	1	Grey/100%																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																													</

DEPTH SCALE

1 : 50

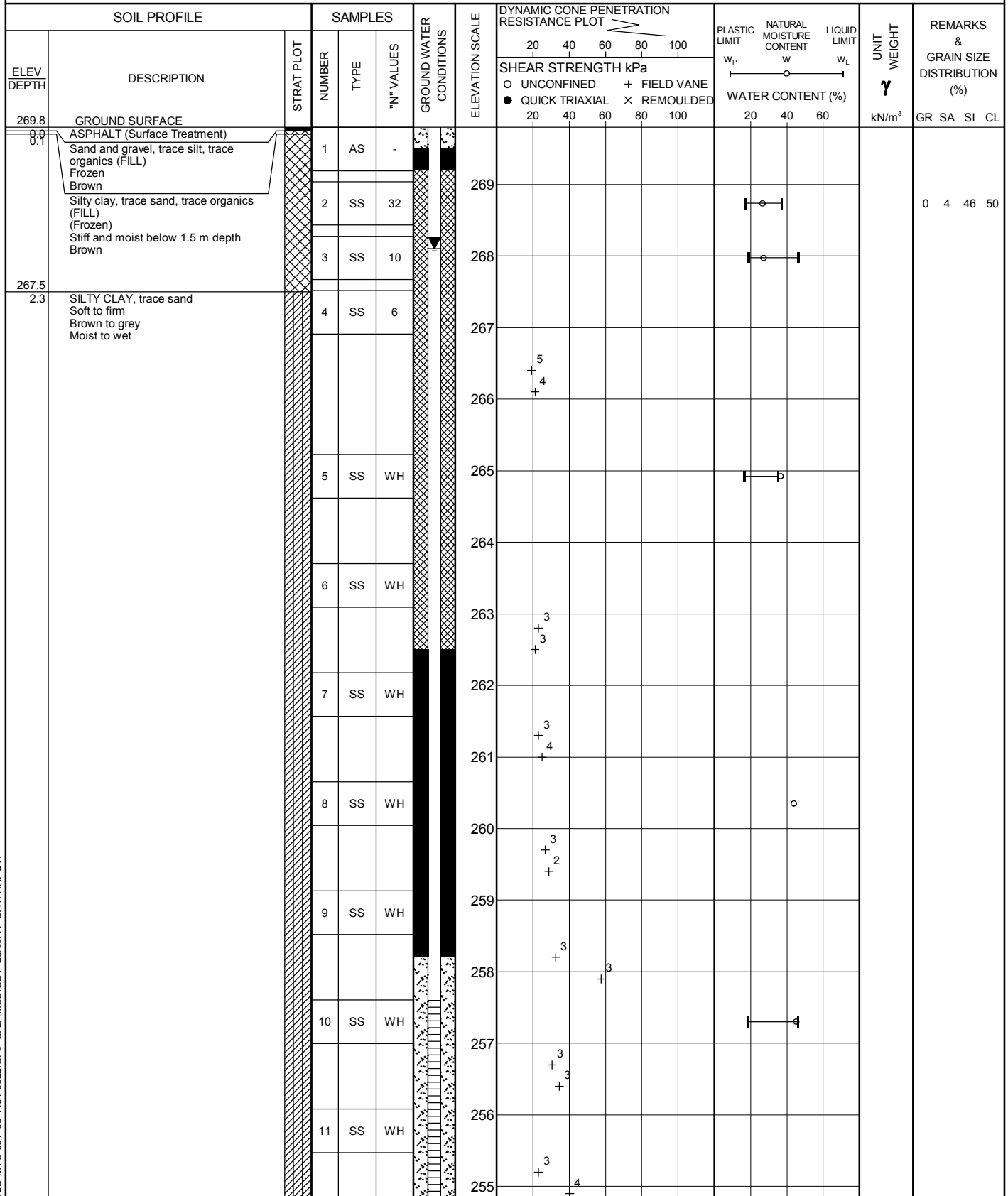


LOGGED: ID

CHECKED: SC

SUD-RCK 09-1191-0022.GPJ GAL-MISS.GDT 2809/11 DATA INPUT:

PROJECT 09-1191-0022		RECORD OF BOREHOLE No WS-4		1 OF 2 METRIC
W.P. 5139-06-00		LOCATION N 5414649.7; E 313801.4		ORIGINATED BY DAM
DIST Cochrane HWY 7036		BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers		COMPILED BY JLL
DATUM Geodetic		DATE April 16, 2011		CHECKED BY SC



Continued Next Page

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

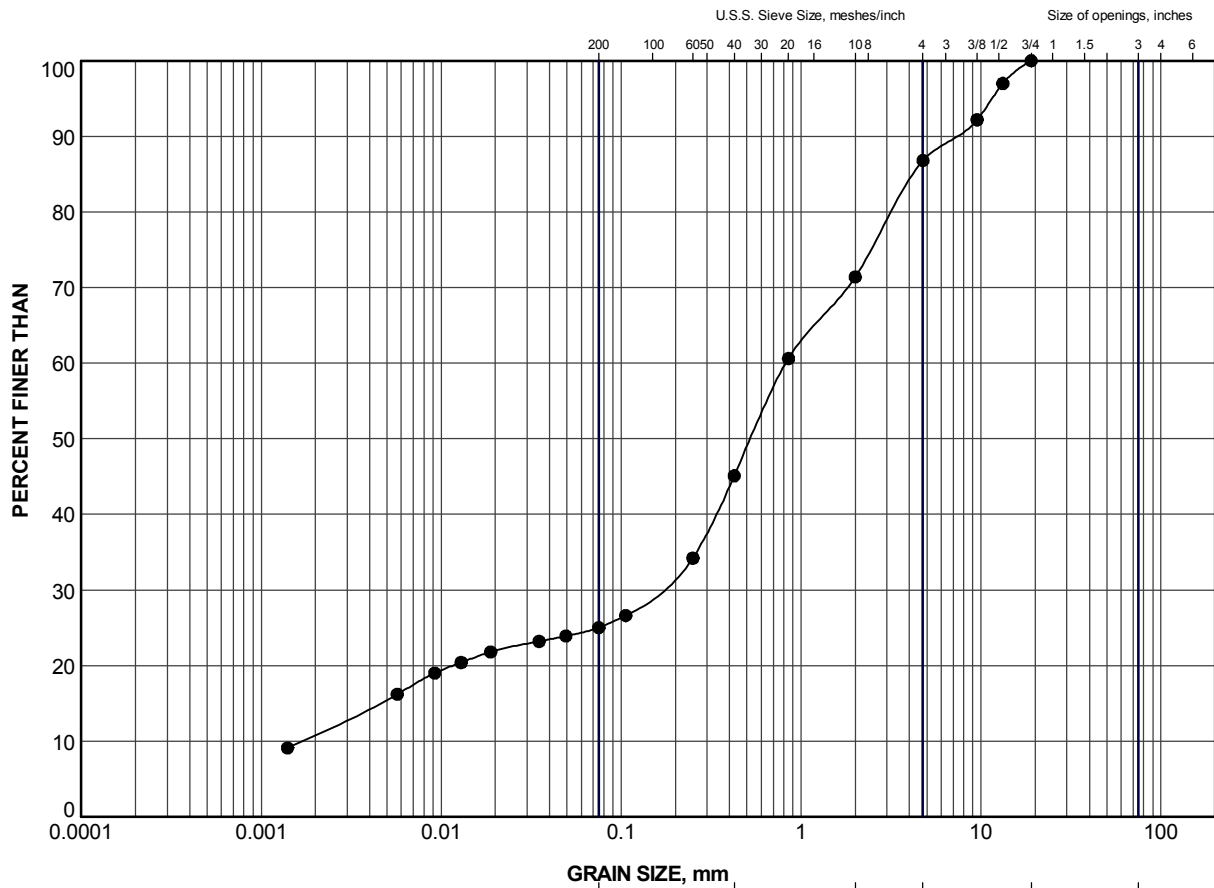
+³, ×³: Numbers refer to Sensitivity ○^{3%} STRAIN AT FAILURE

CSUD-MTO 001 09-1191-0022.GPJ GAL-MISS.GDT 28/09/11 DATA INPUT:



APPENDIX B


Laboratory Test Results

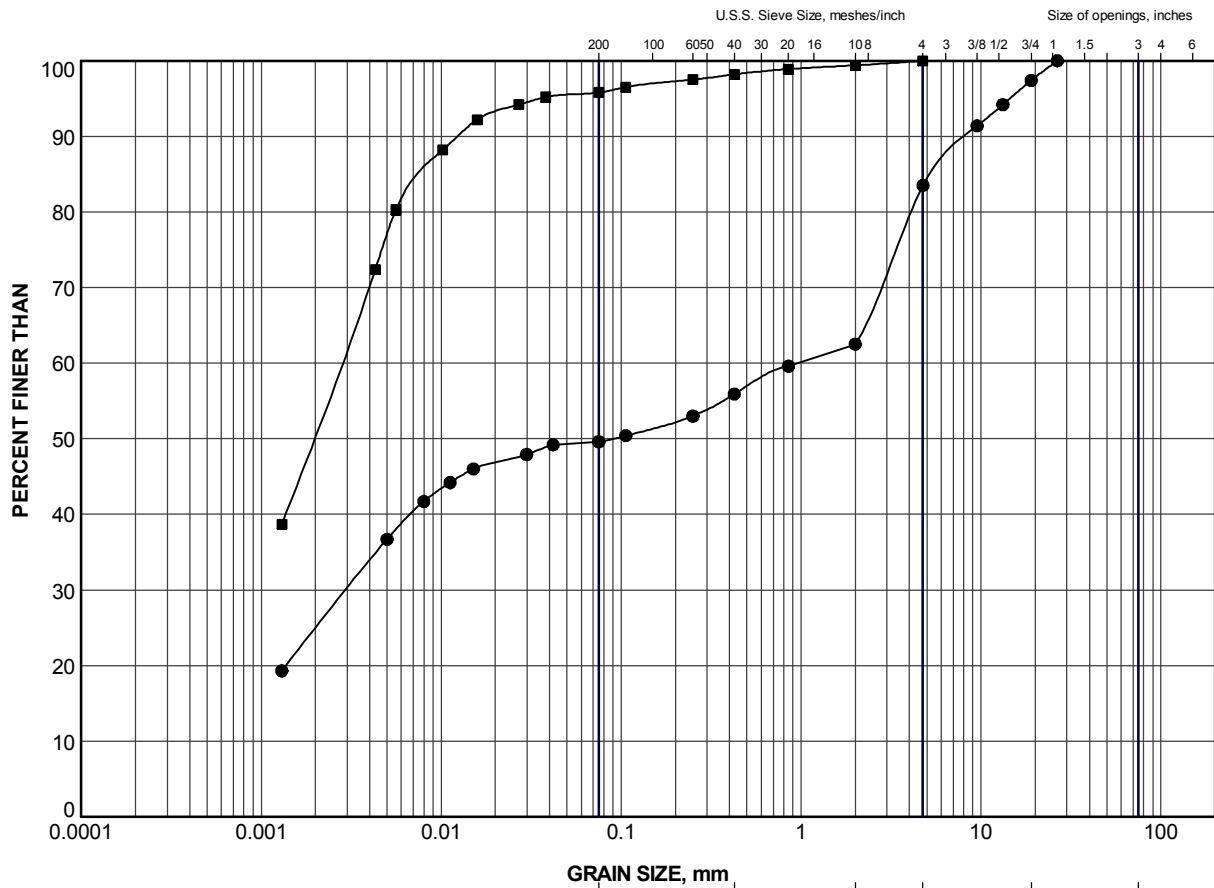


GRAVEL SIZE, mm							Cobble Size
CLAY AND SILT	fine	medium	coarse	fine	coarse		
	SAND SIZE			GRAVEL SIZE			

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	WS-3	1	269.1


PROJECT					WICKLOW RIVER BRIDGE SOUTH HIGHWAY 7036				
TITLE					GRAIN SIZE DISTRIBUTION SAND (FILL)				
PROJECT No.		09-1191-0022		FILE No.		09-1191-0022.GPJ			
DRAWN	JJL	Sep 2011	SCALE	N/A	REV.				
CHECK	DAM	Sep 2011							
APPR		Sep 2011							
 Golder Associates SUDBURY, ONTARIO			FIGURE B-1						

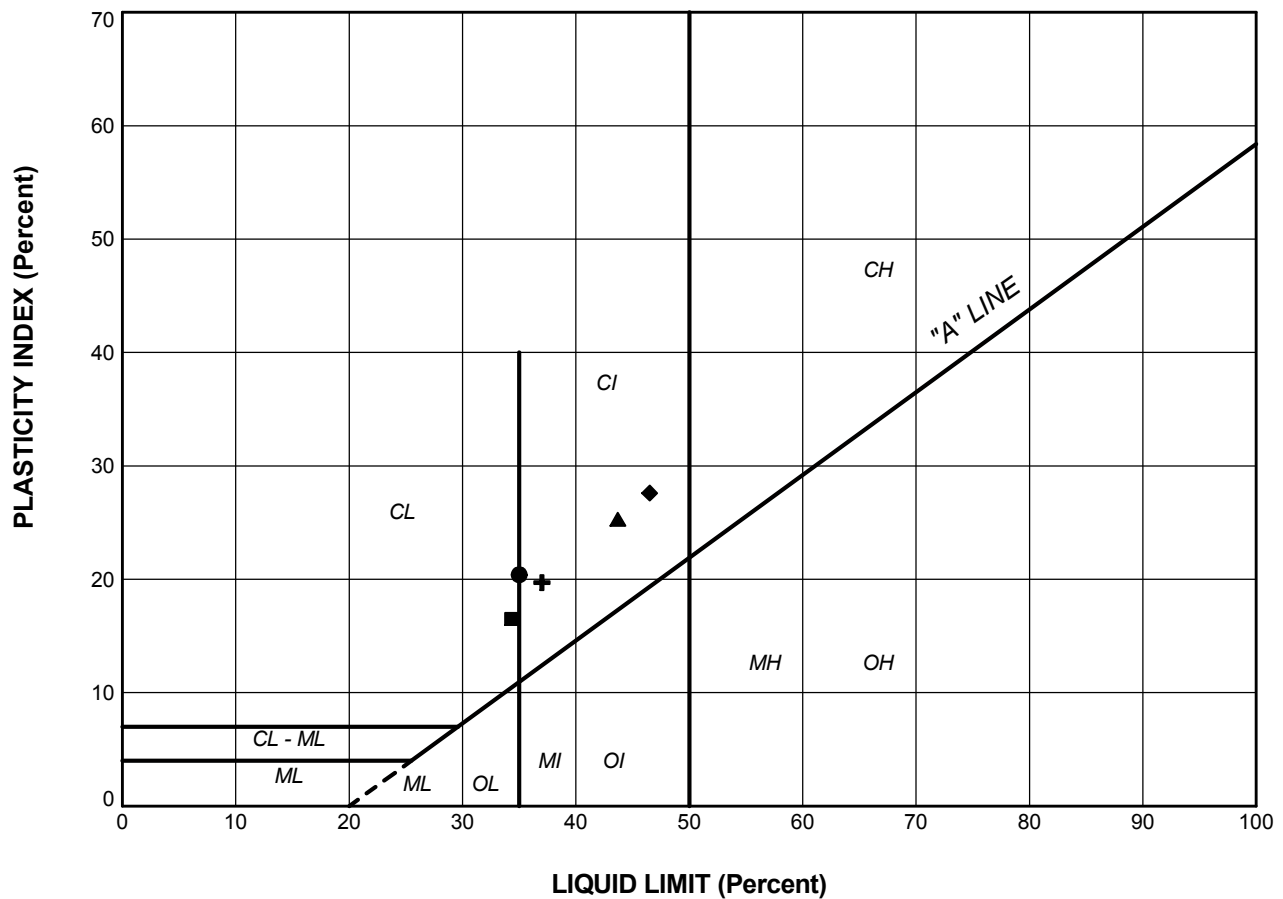


CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

LEGEND


SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	WS-1	1	268.8
■	WS-4	2	268.7

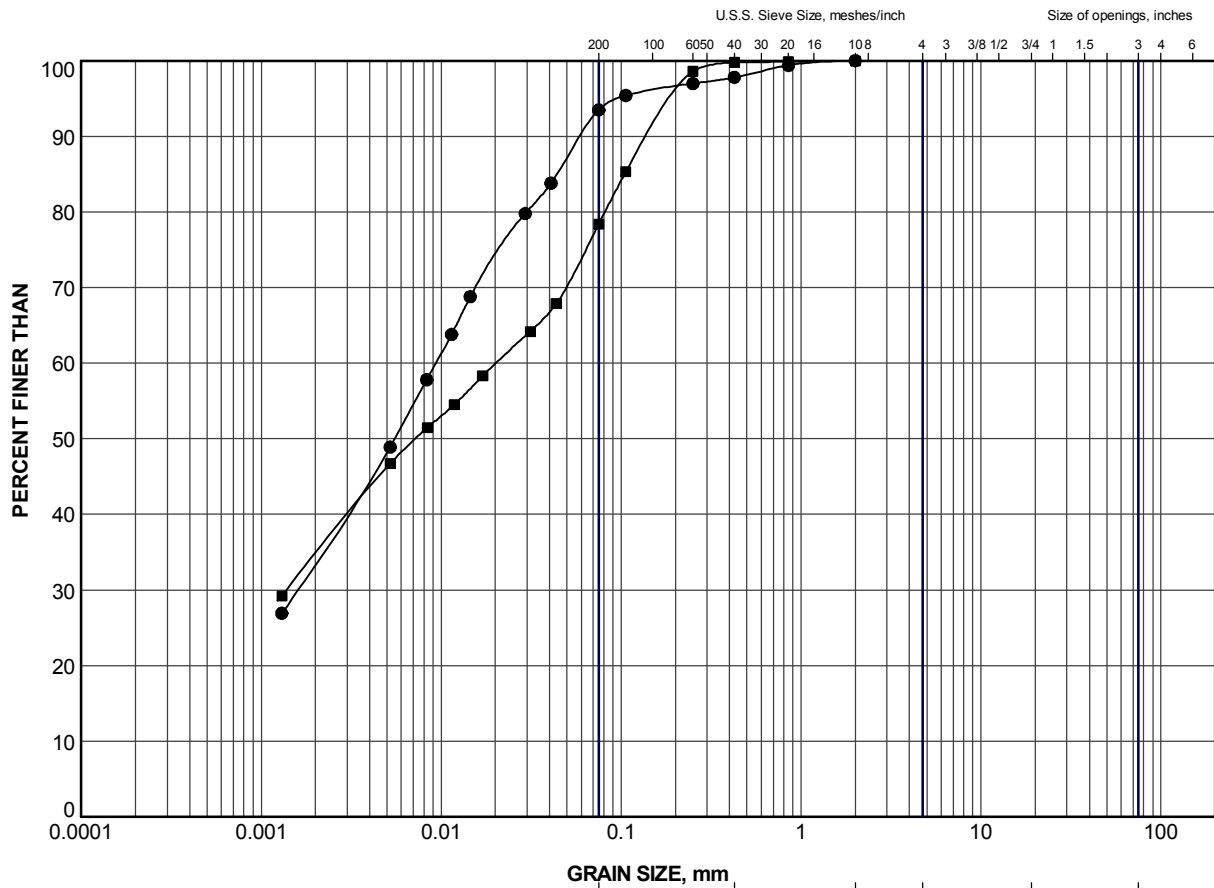
PROJECT					
WICKLOW RIVER BRIDGE SOUTH HIGHWAY 7036					
TITLE					
GRAIN SIZE DISTRIBUTION SILTY CLAY (FILL)					
PROJECT No.		09-1191-0022		FILE No. 09-1191-0022.GPJ	
DRAWN	JJL	Sep 2011	SCALE	N/A	REV.
CHECK	DAM	Sep 2011			
APPR		Sep 2011			
 Golder Associates SUDBURY, ONTARIO			FIGURE B-2		



LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	WS-1	1	35.0	14.6	20.4
■	WS-2	3	34.3	17.8	16.5
▲	WS-3	3	43.7	18.4	25.3
+	WS-4	2	37.0	17.3	19.7
◆	WS-4	3	46.5	18.9	27.6


PROJECT					
WICKLOW RIVER BRIDGE SOUTH HIGHWAY 7036					
TITLE					
PLASTICITY CHART CLAYEY SILT TO SILTY CLAY (FILL)					
PROJECT No.		09-1191-0022		FILE No.	
DRAWN		JJL		Sep 2011	
CHECK		DAM		Sep 2011	
APPR				Sep 2011	
SCALE		N/A		REV.	
 Golder Associates SUDBURY, ONTARIO				FIGURE B-3	

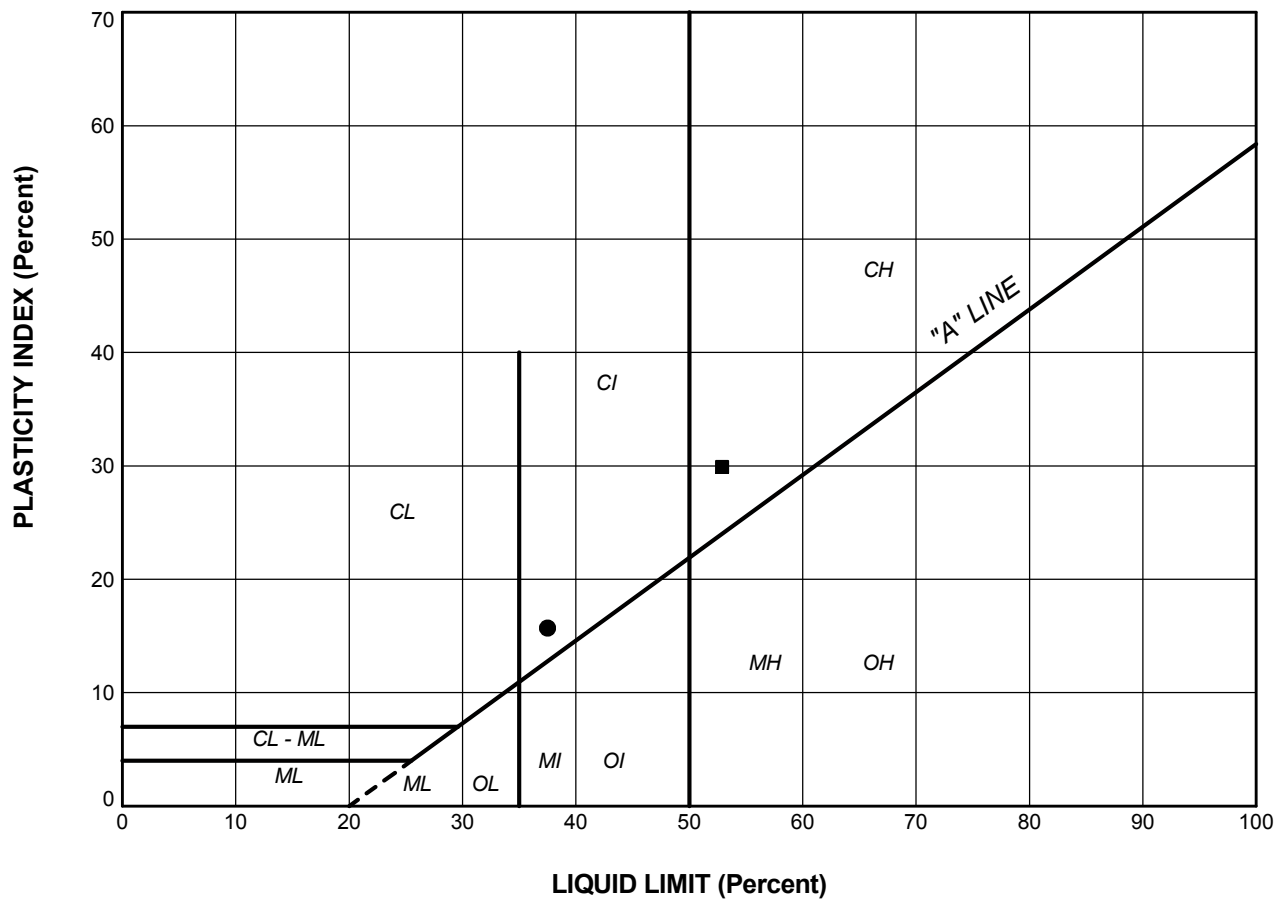


GRAVEL SIZE, mm						
CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

LEGEND


SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	WS-1	5	265.9
■	WS-2	6	265.3

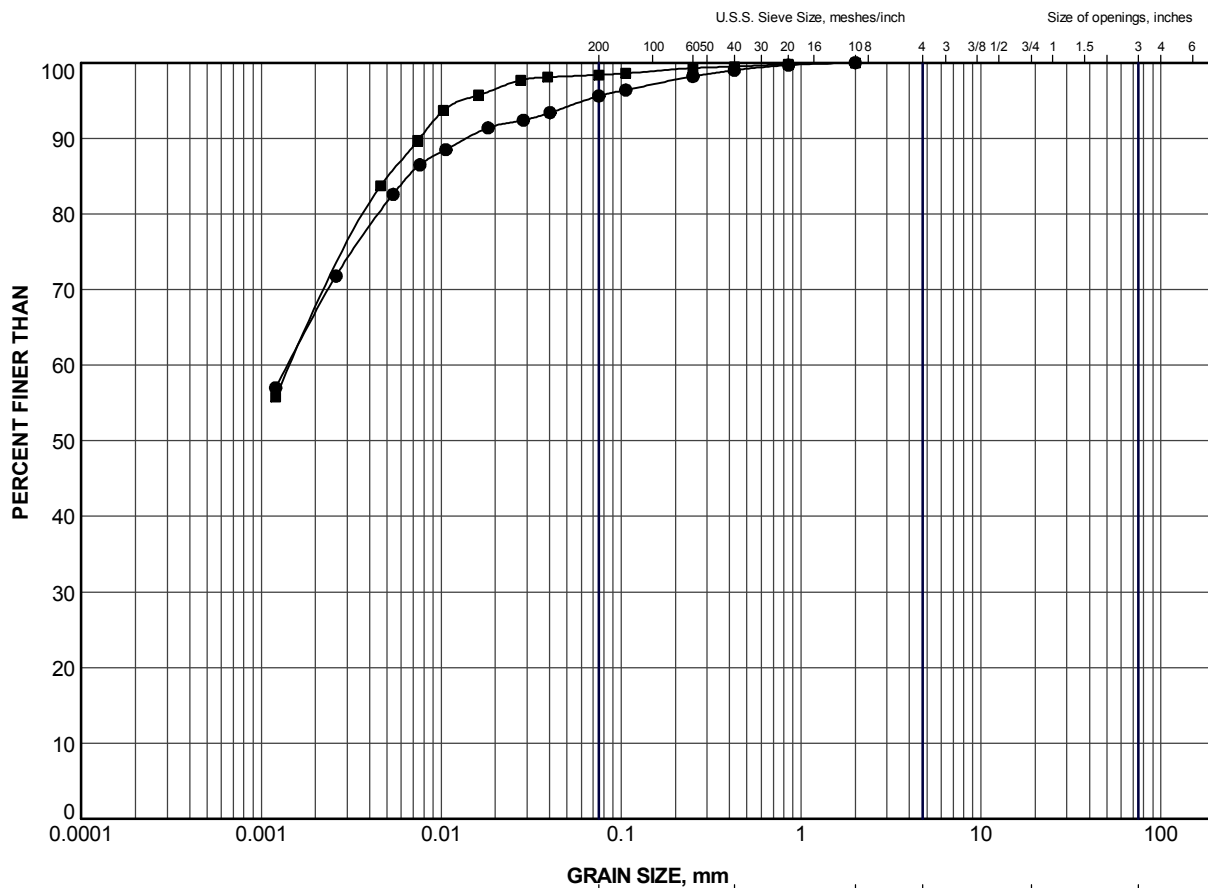
PROJECT						WICKLOW RIVER BRIDGE SOUTH HIGHWAY 7036					
TITLE						GRAIN SIZE DISTRIBUTION SILTY CLAY TO CLAY (ALLUVIUM)					
PROJECT No.			09-1191-0022			FILE No.			09-1191-0022.GPJ		
DRAWN	JJL	Sep 2011	CHECK		DAM	Sep 2011	SCALE		N/A	REV.	
APPR		Sep 2011								FIGURE B-4	
 Golder Associates SUDBURY, ONTARIO											



LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	WS-1	5	37.5	21.8	15.7
■	WS-2	7	52.9	23.0	29.9


PROJECT					
WICKLOW RIVER BRIDGE SOUTH HIGHWAY 7036					
TITLE					
PLASTICITY CHART SILTY CLAY TO CLAY (ALLUVIUM)					
PROJECT No.		09-1191-0022		FILE No.	
				09-1191-0022.GPJ	
DRAWN	JJL	Sep 2011	SCALE	N/A	REV.
CHECK	DAM	Sep 2011			
APPR		Sep 2011			
 Golder Associates SUDBURY, ONTARIO			FIGURE B-5		

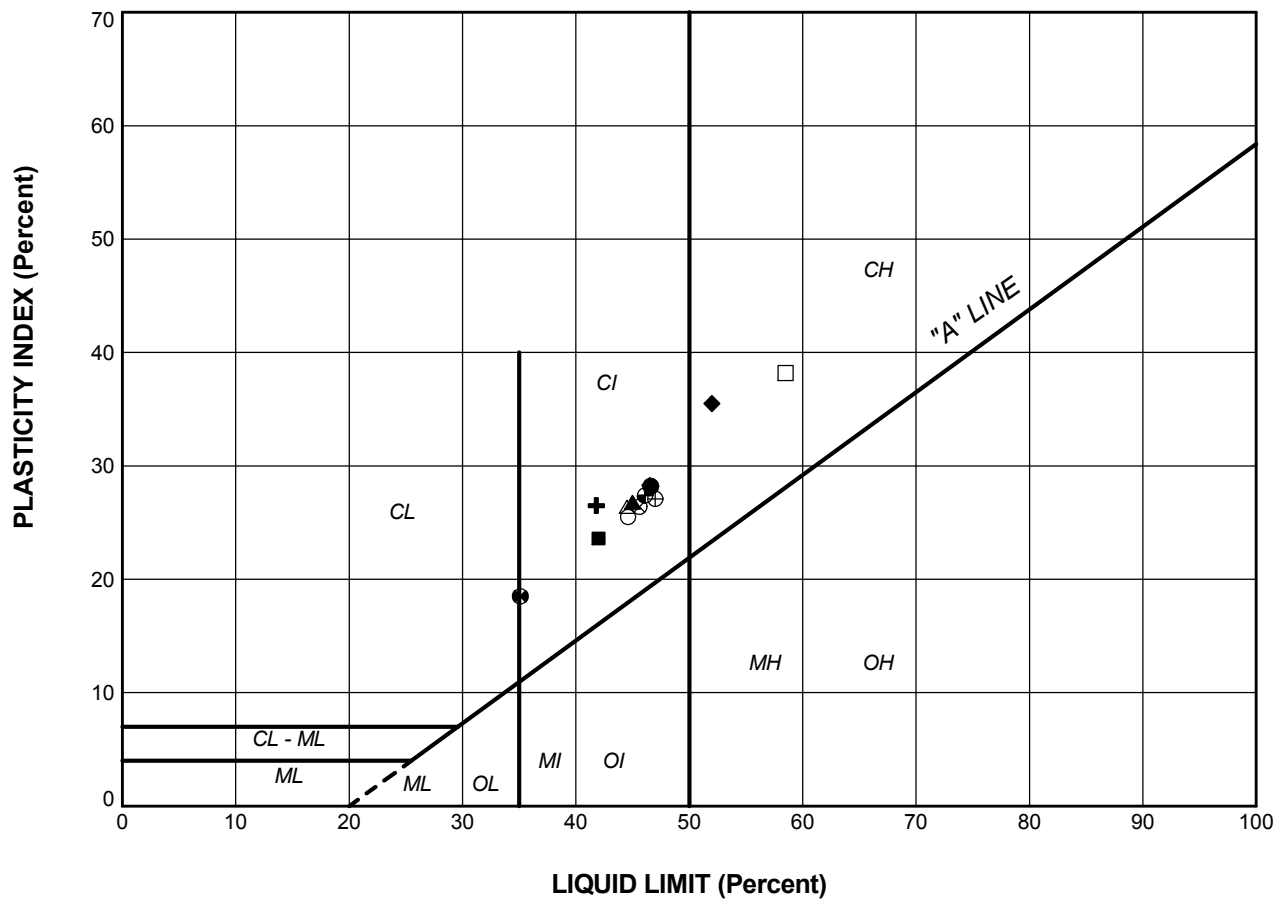


CLAY AND SILT		GRAVEL SIZE, mm					Cobble Size
		fine	medium	coarse	fine	coarse	
		SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	WS-2	10	260.0
■	WS-3	9	260.1

PROJECT						WICKLOW RIVER BRIDGE SOUTH HIGHWAY 7036					
TITLE						GRAIN SIZE DISTRIBUTION SILTY CLAY					
PROJECT No.			09-1191-0022			FILE No.			09-1191-0022.GPJ		
DRAWN		J.J.L.		Sep 2011		SCALE		N/A		REV.	
CHECK		DAM		Sep 2011							
APPR				Sep 2011							
 Golder Associates SUDBURY, ONTARIO						FIGURE B-6					



LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	WS-1	9	46.6	18.4	28.2
■	WS-1	13	42.0	18.4	23.6
▲	WS-2	9	45.0	18.2	26.8
+	WS-2	10	41.8	15.3	26.5
◆	WS-2	13	52.0	16.5	35.5
◇	WS-2	17	46.5	18.2	28.3
○	WS-3	8	44.6	19.1	25.5
△	WS-3	9	44.5	18.1	26.4
⊗	WS-3	10	45.6	19.2	26.4
⊕	WS-3	12	47.0	19.9	27.1
□	WS-3	13	58.5	20.3	38.2
⊙	WS-4	5	35.1	16.6	18.5
●	WS-4	10	46.1	18.7	27.4

PROJECT					
WICKLOW RIVER BRIDGE SOUTH HIGHWAY 7036					
TITLE					
PLASTICITY CHART SILTY CLAY TO CLAY					
PROJECT No.		09-1191-0022		FILE No.	
DRAWN		J.J.L.		Sep 2011	
CHECK		DAM		Sep 2011	
APPR				Sep 2011	
SCALE		N/A		REV.	
FIGURE B-7					



CONSOLIDATION TEST SUMMARY**FIGURE B-8**

Pg. 1 of 4

SAMPLE IDENTIFICATION

Project Number: 09-1191-0022

Sample Number: 10

Borehole Number: WS-2

Sample Depth, m: 9.1

TEST CONDITIONS

Test Type Standard

Load Duration, hr 24

Oedometer Number 1

Date Started May 26/11

Date Completed June 9/11

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	2.550	Unit Weight, kN/m ³	18.21
Sample Diameter, cm	6.330	Dry Unit Weight, kN/m ³	13.01
Area, cm ²	31.47	Specific Gravity, measured	2.74
Volume, cm ³	80.25	Solids Height, cm	1.234
Water Content, %	39.97	Volume of Solids, cm ³	38.84
Wet Mass, g	149.05	Volume of Voids, cm ³	41.41
Dry Mass, g	106.49	Degree of Saturation, %	102.8

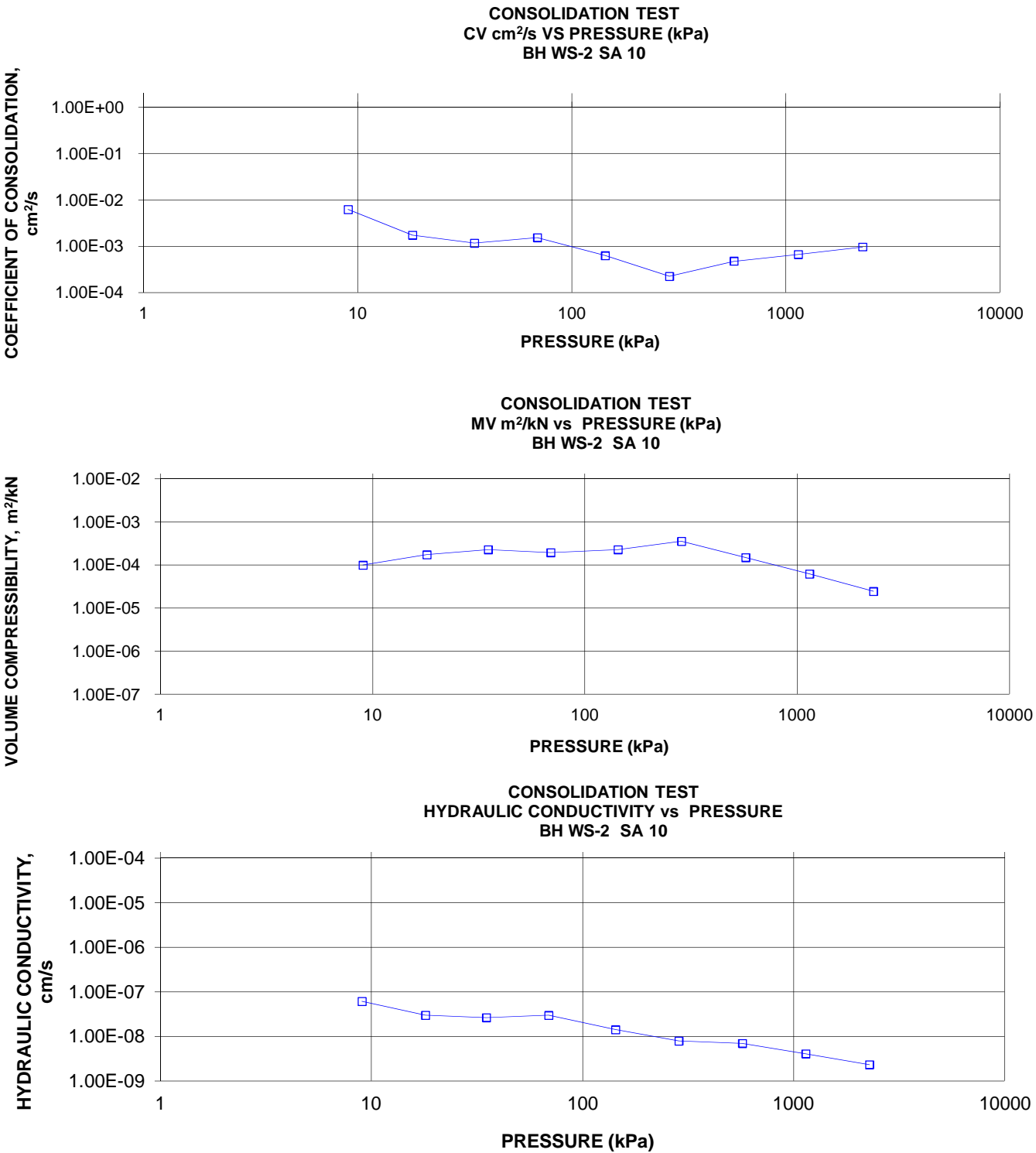
TEST COMPUTATIONS

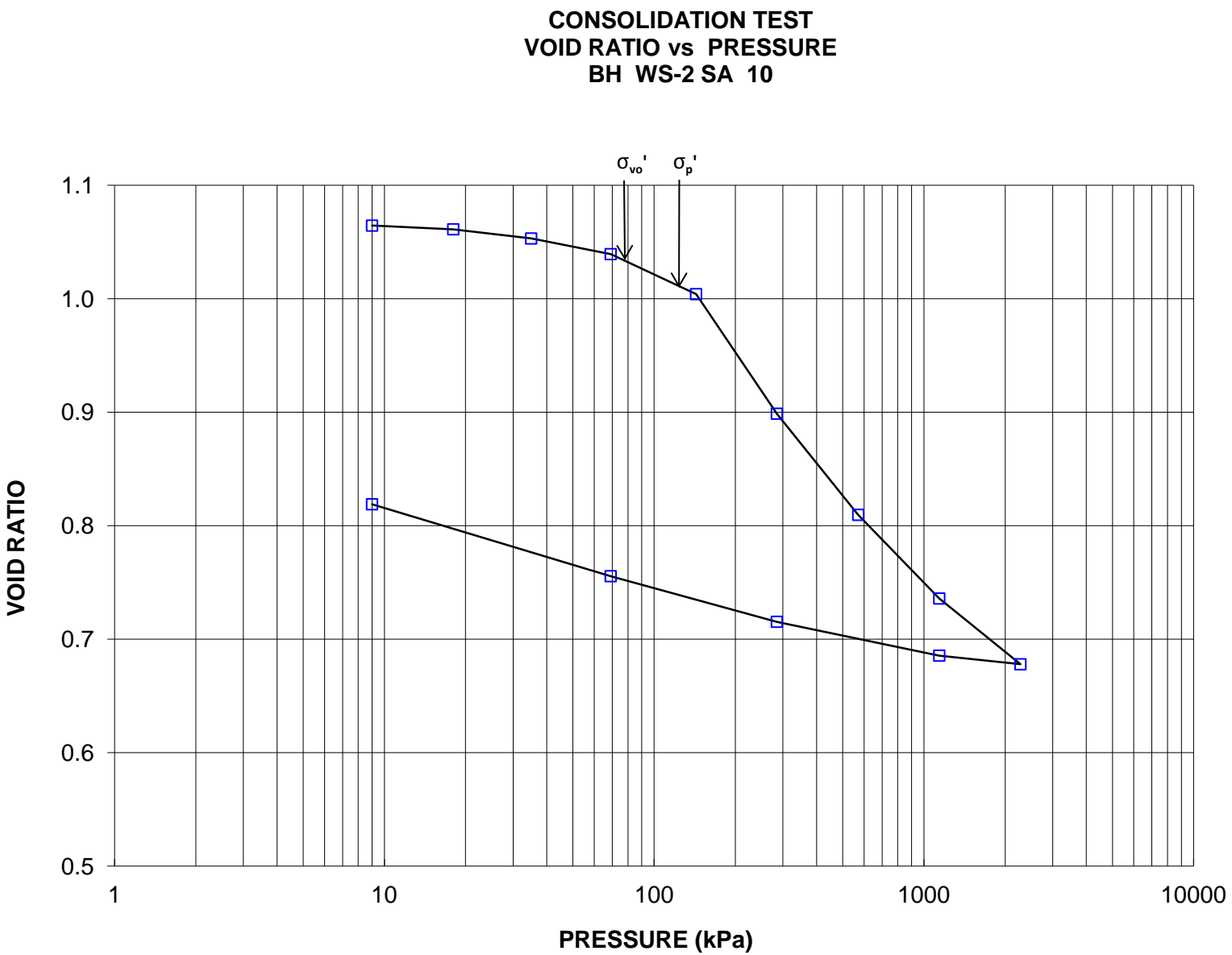
Pressure kPa	Primary Consolidation	Corr. Height cm	Void Ratio	Average Height cm	t ₉₀ sec	cv. cm ² /s	mv m ² /kN	k cm/s	Total Work kJ/m ³
0	0	2.550	1.066	2.550					
9	0.02	2.548	1.064	2.549	220	0.0063	1.00E-04	6.15E-08	0.004
18	0.04	2.544	1.061	2.546	780	0.0018	1.74E-04	3.01E-08	0.025
35	0.10	2.534	1.053	2.539	1160	0.0012	2.31E-04	2.66E-08	0.129
69	0.17	2.517	1.039	2.525	870	0.0016	1.96E-04	2.99E-08	0.478
143	0.44	2.473	1.004	2.495	2090	0.0006	2.31E-04	1.43E-08	2.310
285	1.30	2.343	0.899	2.408	5415	0.0002	3.59E-04	7.99E-09	13.559
571	1.10	2.233	0.810	2.288	2310	0.0005	1.51E-04	7.10E-09	33.651
1140	0.91	2.142	0.736	2.188	1500	0.0007	6.27E-05	4.16E-09	68.512
2279	0.72	2.071	0.678	2.106	960	0.0010	2.46E-05	2.36E-09	125.570
1140	-0.09	2.080	0.686	2.075					
285	-0.37	2.117	0.715	2.099					
69	-0.50	2.167	0.756	2.142					
9	-0.782	2.245	0.819	2.206					

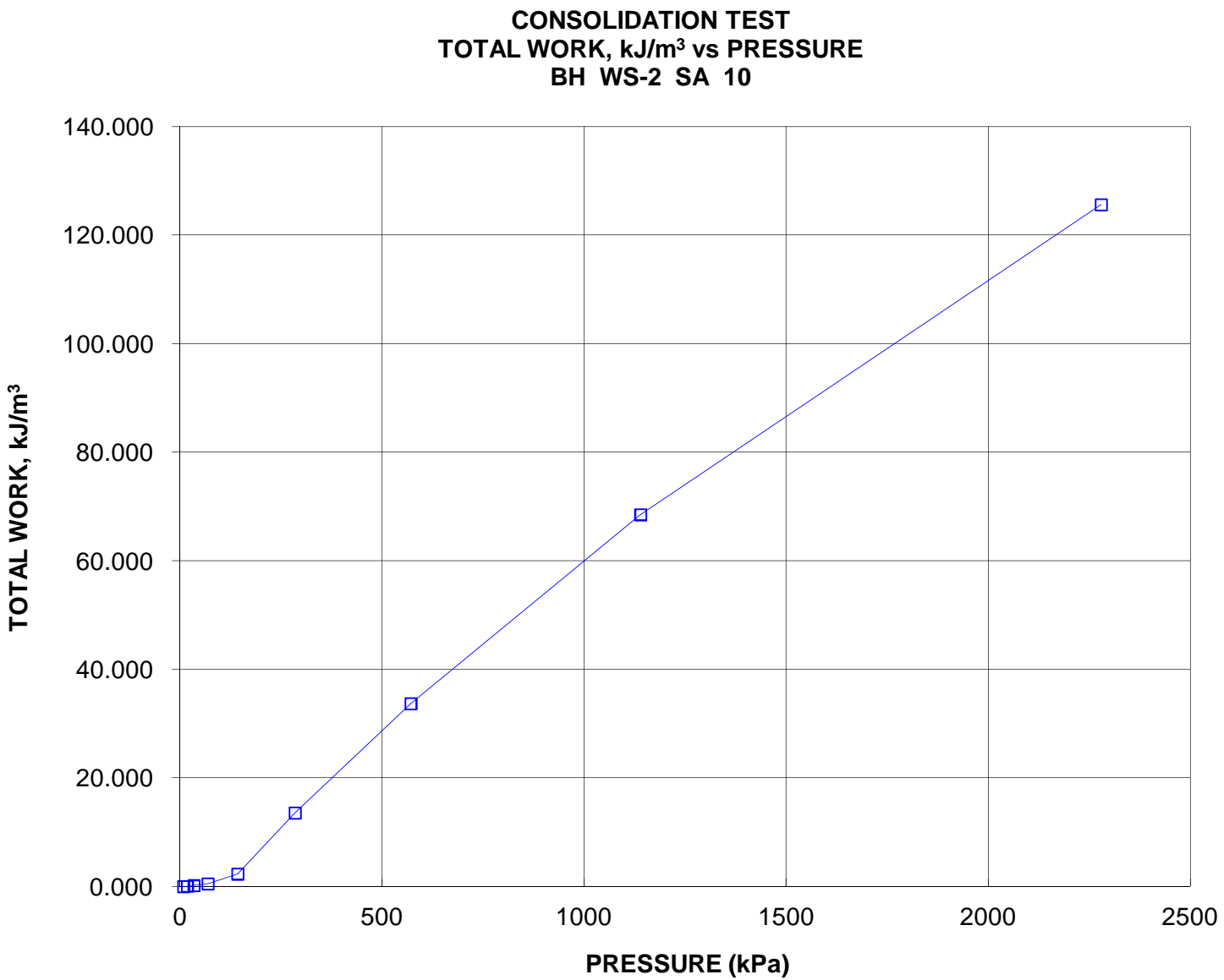
Note:

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

Sample Height, cm	2.245	Unit Weight, kN/m ³	18.83
Sample Diameter, cm	6.33	Dry Unit Weight, kN/m ³	14.78
Area, cm ²	31.47	Specific Gravity, measured	2.74
Volume, cm ³	70.64	Solids Height, cm	1.234
Water Content, %	27.36	Volume of Solids, cm ³	38.84
Wet Mass, g	135.63	Volume of Voids, cm ³	31.80
Dry Mass, g	106.49		







CONSOLIDATION TEST SUMMARY**FIGURE B-9****Pg. 1 of 4****SAMPLE IDENTIFICATION**Project Number: 09-1191-0022
Borehole Number: WS-3Sample Number: 8
Sample Depth, m: 7.6**TEST CONDITIONS**Test Type Standard Load Duration, hr 24
Oedometer Number 1
Date Started 4-Jul-11
Date Completed 19-Jul-11**SAMPLE DIMENSIONS AND PROPERTIES - INITIAL**Sample Height, cm 2.550 Unit Weight, kN/m³ 17.95
Sample Diameter, cm 6.330 Dry Unit Weight, kN/m³ 12.73
Area, cm² 31.47 Specific Gravity, measured 2.71
Volume, cm³ 80.25 Solids Height, cm 1.221
Water Content, % 41.06 Volume of Solids, cm³ 38.42
Wet Mass, g 146.89 Volume of Voids, cm³ 41.82
Dry Mass, g 104.13 Degree of Saturation, % 102.2**TEST COMPUTATIONS**

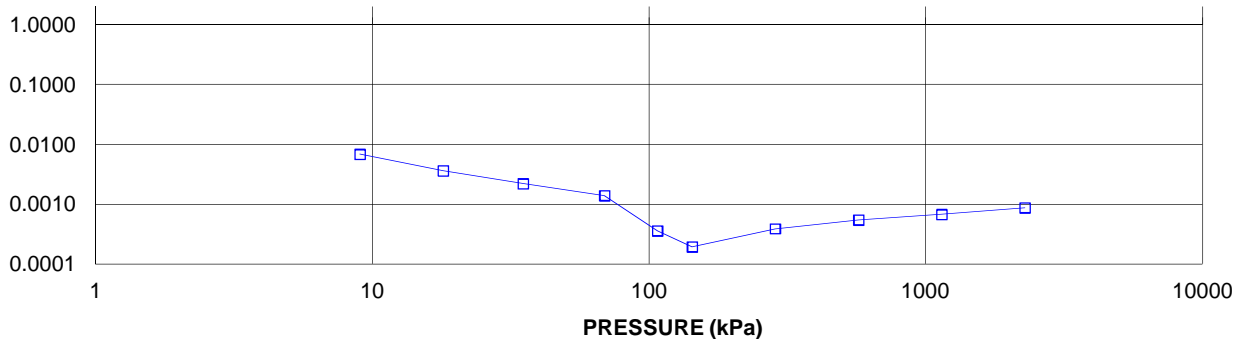
Pressure	Primary	Corr.		Average					Total
kPa	Consolidation	Height	Void	Height	t ₉₀	cv.	mv	k	Work
		cm	Ratio	cm	sec	cm ² /s	m ² /kN	cm/s	kJ/m3
0	0	2.550	1.088	2.550					
9	0.02	2.548	1.087	2.549	200	0.0069	1.05E-04	7.06E-08	0.004
18	0.04	2.544	1.084	2.546	375	0.0037	1.58E-04	5.68E-08	0.023
35	0.10	2.534	1.075	2.539	614	0.0022	2.28E-04	4.98E-08	0.127
69	0.19	2.515	1.060	2.524	960	0.0014	2.22E-04	3.06E-08	0.522
107	0.24	2.491	1.040	2.503	3650	0.0004	2.50E-04	8.90E-09	1.368
143	0.28	2.463	1.017	2.477	6615	0.0002	2.74E-04	5.28E-09	2.701
285	1.14	2.349	0.924	2.406	3110	0.0004	3.15E-04	1.22E-08	12.605
571	1.10	2.239	0.834	2.294	2018	0.0006	1.51E-04	8.17E-09	32.647
1140	0.92	2.147	0.758	2.193	1500	0.0007	6.37E-05	4.24E-09	67.950
2279	0.94	2.053	0.682	2.100	1058	0.0009	3.22E-05	2.79E-09	142.407
1140	-0.08	2.062	0.688	2.057					
285	-0.34	2.095	0.716	2.079					
69	-0.46	2.142	0.754	2.119					
9	-0.93	2.235	0.830	2.188					

Note:

k calculated using α based on t₉₀ values.**SAMPLE DIMENSIONS AND PROPERTIES - FINAL**Sample Height, cm 2.235 Unit Weight, kN/m³ 18.41
Sample Diameter, cm 6.33 Dry Unit Weight, kN/m³ 14.52
Area, cm² 31.47 Specific Gravity, measured 2.71
Volume, cm³ 70.33 Solids Height, cm 1.221
Water Content, % 26.82 Volume of Solids, cm³ 38.42
Wet Mass, g 132.06 Volume of Voids, cm³ 31.91
Dry Mass, g 104.13

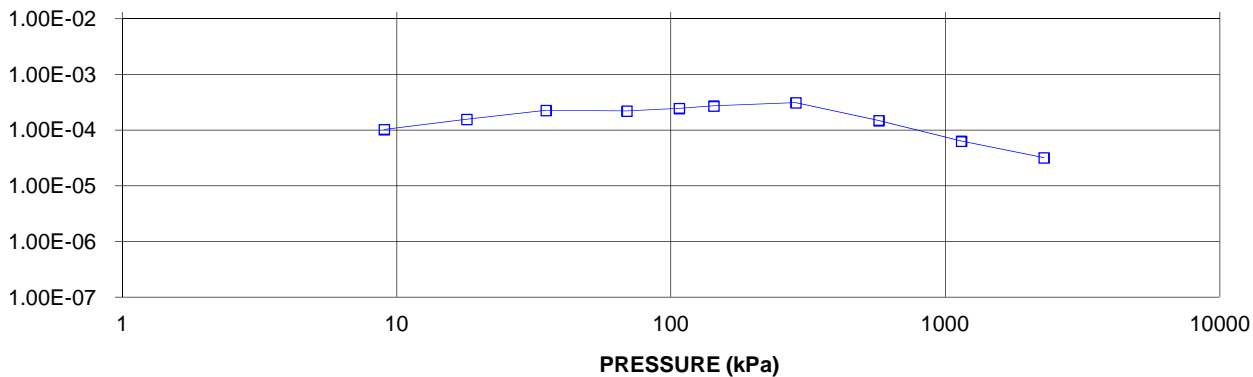
COEFFICIENT OF CONSOLIDATION,
cm²/s

CONSOLIDATION TEST
CV cm²/s VS PRESSURE (kPa)
BH WS-3 SA 8



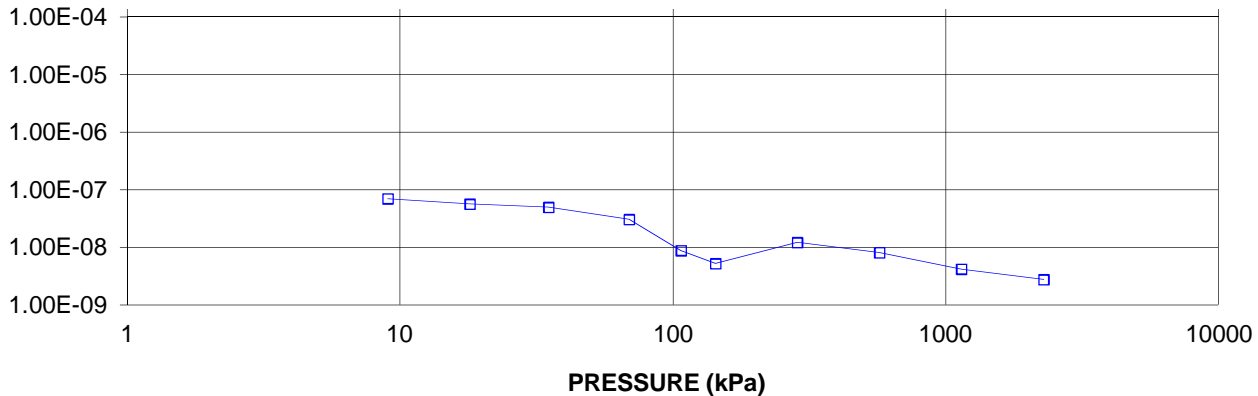
VOLUME COMPRESSIBILITY, m²/kN

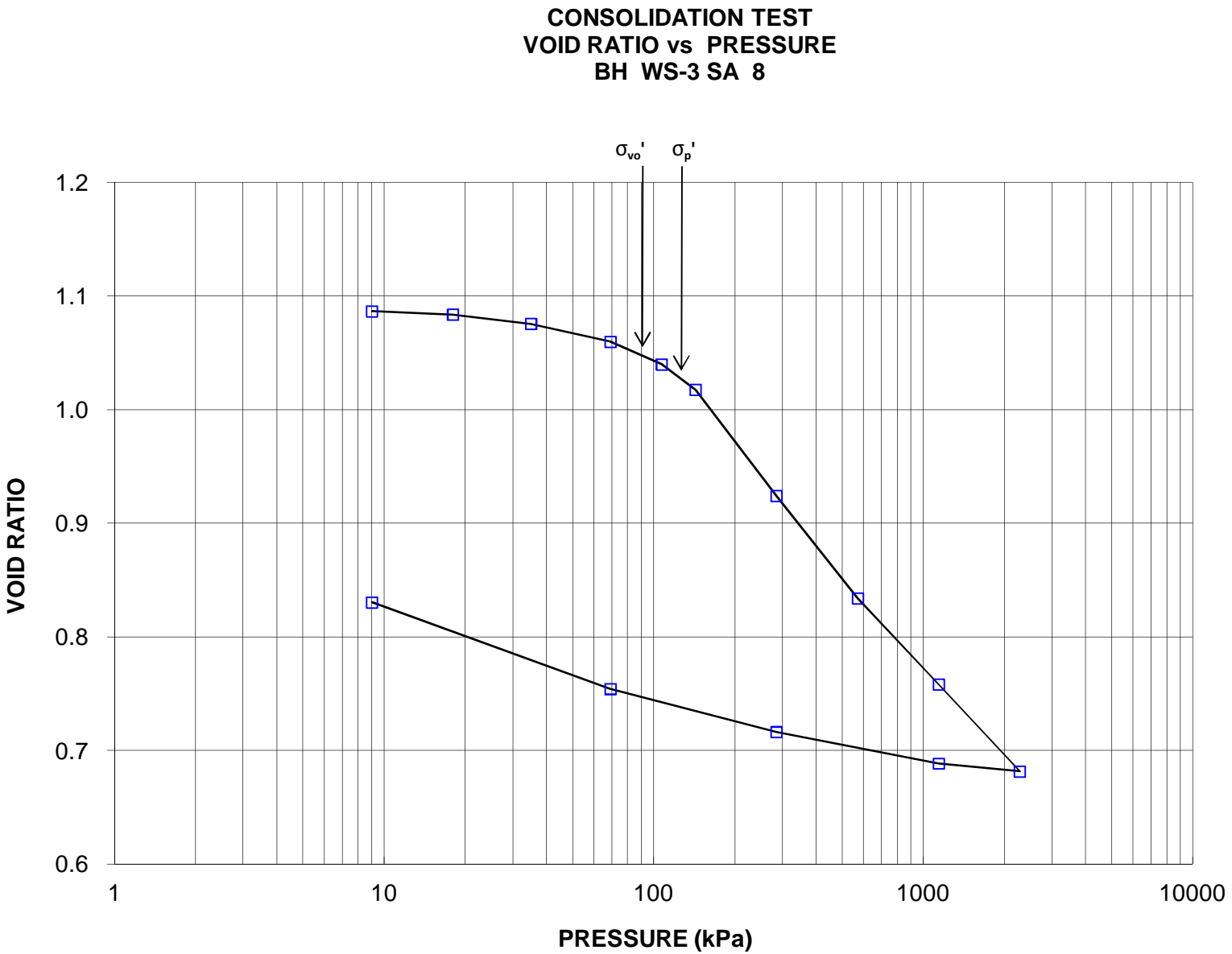
CONSOLIDATION TEST
MV m²/kN vs PRESSURE (kPa)
BH WS-3 SA 8

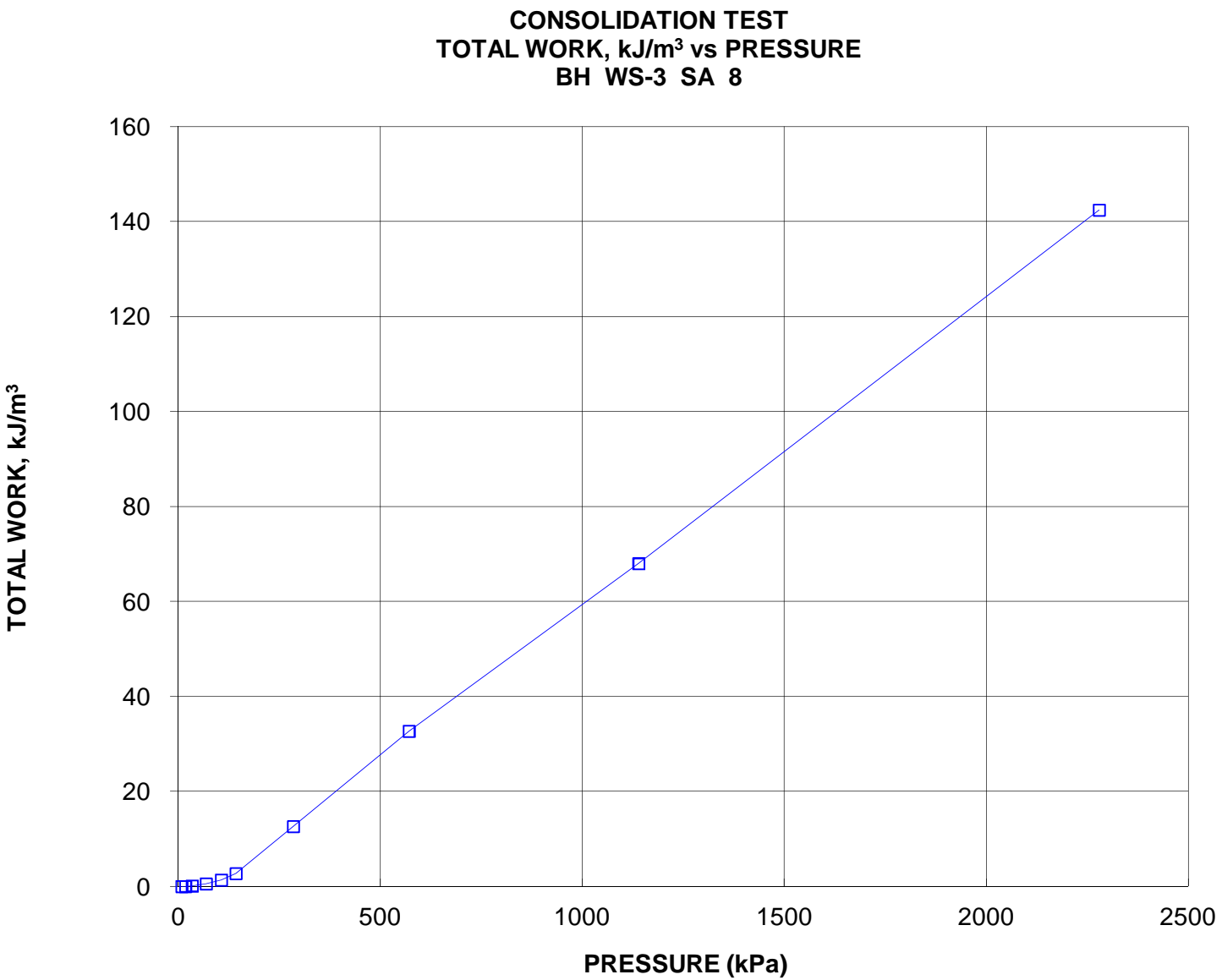


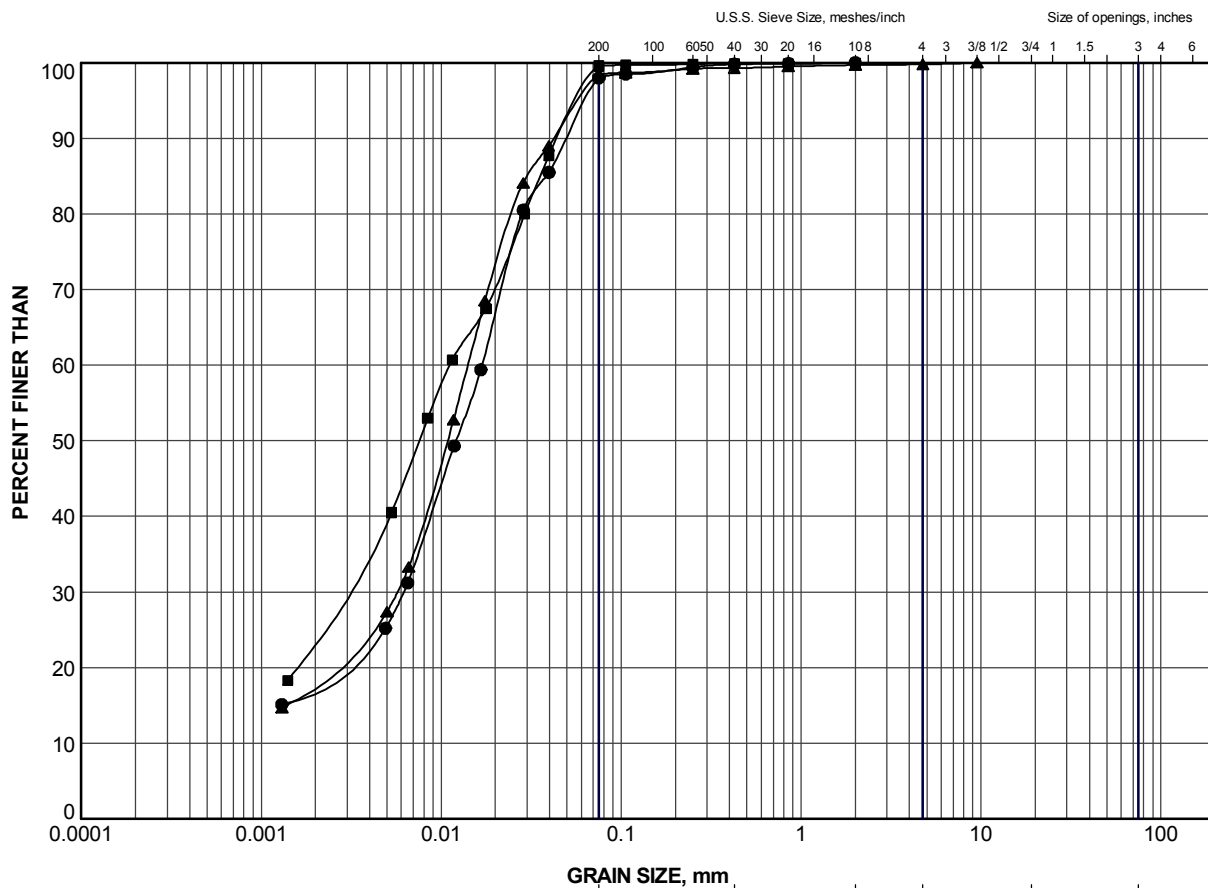
HYDRAULIC CONDUCTIVITY,
cm/s

CONSOLIDATION TEST
HYDRAULIC CONDUCTIVITY vs PRESSURE
BH WS-3 SA 8








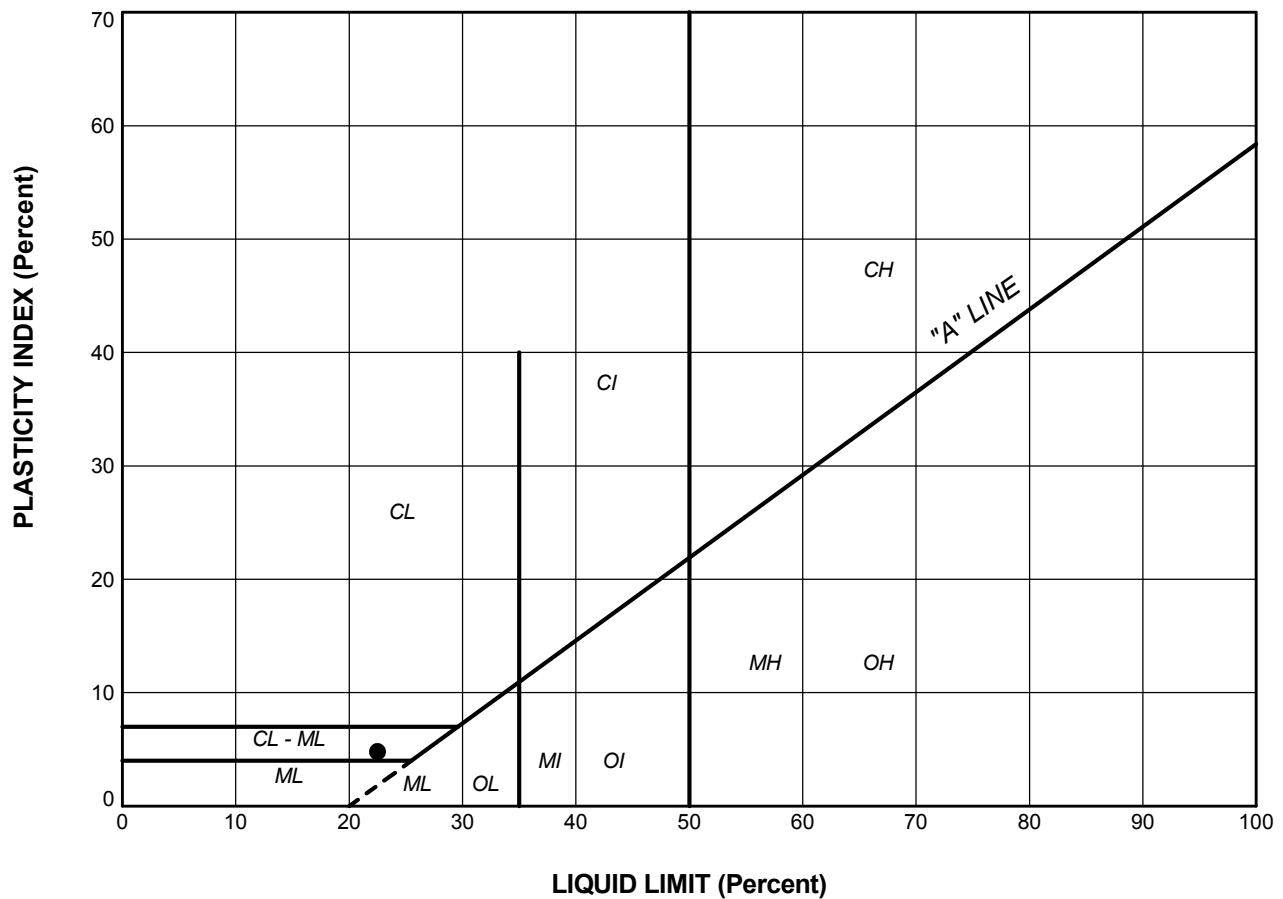


GRAVEL SIZE, mm						Cobble Size
CLAY AND SILT	fine	medium	coarse	fine	coarse	
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	WS-2	21	243.2
■	WS-3	17	246.3
▲	WS-3A	2	243.4

PROJECT					
WICKLOW RIVER BRIDGE SOUTH HIGHWAY 7036					
TITLE					
GRAIN SIZE DISTRIBUTION SILT					
PROJECT No.		09-1191-0022		FILE No. 09-1191-0022.GPJ	
DRAWN	JJL	Sep 2011	SCALE	N/A	REV.
CHECK	DAM	Sep 2011			
APPR		Sep 2011			
 Golder Associates SUDBURY, ONTARIO			FIGURE B-10		



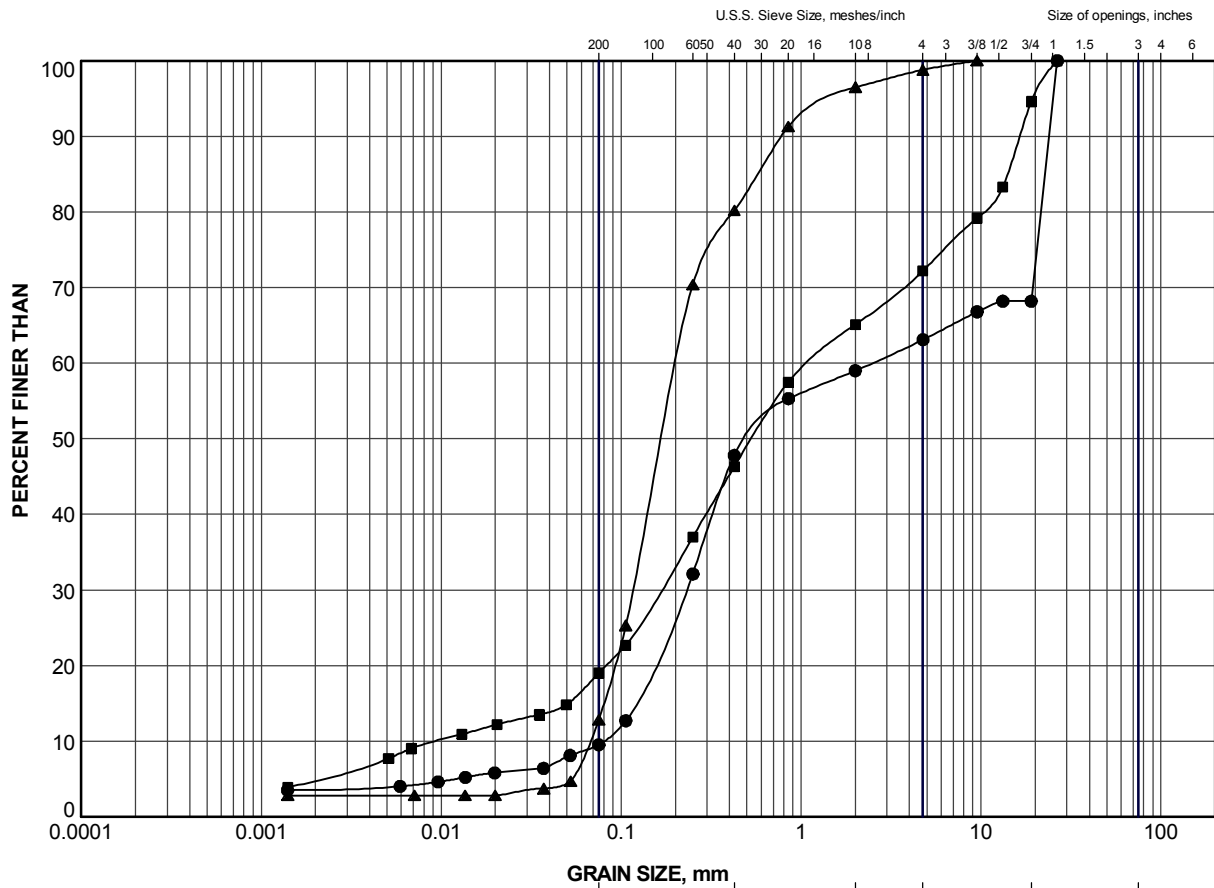
LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	WS-3	17	22.5	17.7	4.8

PROJECT					
WICKLOW RIVER BRIDGE SOUTH HIGHWAY 7036					
TITLE					
PLASTICITY CHART silt					
PROJECT No.		09-1191-0022		FILE No.	
				09-1191-0022.GPJ	
DRAWN	JJL	Sep 2011	SCALE	N/A	REV.
CHECK	DAM	Sep 2011			
APPR		Sep 2011			




FIGURE B-11



CLAY AND SILT	GRAVEL SIZE, mm						Cobble Size
	fine	medium	coarse	fine	coarse		
	SAND SIZE			GRAVEL SIZE			

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	WS-2	22	241.7
■	WS-3	21	240.2
▲	WS-3	23	238.7

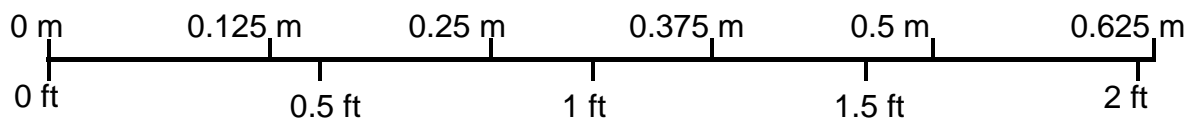
PROJECT						WICKLOW RIVER BRIDGE SOUTH HIGHWAY 7036					
TITLE						GRAIN SIZE DISTRIBUTION SAND TO SAND AND GRAVEL					
PROJECT No.			09-1191-0022			FILE No.			09-1191-0022.GPJ		
DRAWN		J.J.L.		Sep 2011		SCALE		N/A		REV.	
CHECK		DAM		Sep 2011							
APPR				Sep 2011							
 Golder Associates SUDBURY, ONTARIO						FIGURE B-12					




Box 1: 30.5 m – 32.0 m



Box 2: 32.0 m – 33.6 m



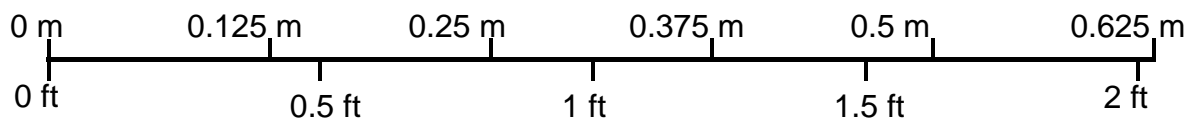
PROJECT		WICKLOW RIVER BRIDGE SOUTH HIGHWAY 7036			
TITLE		BEDROCK CORE (Borehole WS-2)			
	PROJECT No. 09-1191-0022			FILE No. ----	
	DESIGN	DAM	SEPT 2011	SCALE	AS SHOWN
	CADD	--		REV.	
	CHECK	SEMC	SEPT 2011	Figure B-13	
	REVIEW				




Box 1: 31.8 m – 34.0 m



Box 2: 34.0 m – 35.0 m



PROJECT		WICKLOW RIVER BRIDGE SOUTH HIGHWAY 7036	
TITLE		BEDROCK CORE (Borehole WS-3A)	
	PROJECT No.	09-1191-0022	FILE No. ----
	DESIGN	DAM	SEPT 2011
	CADD	--	
	CHECK	SEMC	SEPT 2011
REVIEW			
		SCALE	AS SHOWN
		REV.	
Figure B-14			



APPENDIX C

Non-Standard Special Provisions

CONTROL OF ARTESIAN WATER SEEPAGE –
MOBILIZATION/DEMOBILIZATION - Item No.

Special Provision

Scope

Work under this item shall include mobilization and demobilization of the grouting plant to/from the sites for controlling artesian water seepage during pile driving at the abutments. The grout plant and drilling equipment shall be mobilized once prior to commencing pile driving and demobilize from site upon completing the pile driving. The work shall also include moving the plant from one abutment location to the other.

Payment

Payment at the contract price shall be for all labour, equipment, and materials necessary to mobilize and demobilize the drilling and grouting plant to/from the Wicklow River Bridge South sites, and relocating the equipment from one abutment to the other during the pile driving operation.

CONTROL OF ARTESIAN WATER SEEPAGE – STANDBY - Item No.

Special Provision

Scope

Work under this item shall include standby costs for all labour, equipment, and material for drilling and grouting plant after mobilization during the pile driving at the abutments.

Measurement and Payment

Measurement for payment shall be paid at an hourly rate for the actual standby time incurred during the installation of the piles at each abutment.

Payment at the contract price shall be for all labour, equipment, and materials necessary for the grouting plant and drilling operation to remain on standby during installation of the piles at the abutments.

CONTROL OF ARTESIAN WATER SEEPAGE – DRILLING AND GROUTING -
Item No.

Special Provision

Scope

Work under the above item shall be carried out at the abutment piles. Work shall be performed by an experienced contractor with adequate specialized experience in the control of artesian water seepage.

The Contractor shall provide all labour, materials, tools, equipment and perform all work necessary for augering/drilling with casing and chemical grouting to control artesian water seepage at the direction of the Contract Administrator as follows:

1. The Contractor shall auger/drill cased chemical grouting holes immediately adjacent to the HP piles, install a Multi Port Sleeve Pipe system (MPSP) in each drilled hole and remove the casing. The tip elevation of the grouting hole shall be at EL. 252 m. Under no circumstances shall the tip elevation of the grouting hole reach the silt layer underlying the silty clay to clay stratum.
2. The MPSP system shall be plastic or steel with a minimum 50 mm o.d. Each length of pipe shall be screwed and socketted to fit together flush. The pipe shall have holes drilled and covered with minimum 4 mm thick rubber sleeves along its length at 500 mm spacing.
3. A polypropylene non-woven geotextile fabric bag shall be placed over every second set of sleeved holes at one metre spacing and fastened by pipe clamps above and below the sleeve location. Each polypropylene non-woven geotextile fabric bag shall have a minimum volume of 0.03 m³ (1 ft³).
4. A cementitious grout to inflate the bags shall be pumped into the bags utilizing an inflatable double packer system which is inserted into the sleeved pipe to isolate each grout bag location. The maximum pressure to be used to inflate the MPSP system grout bags should not exceed 1500 kPa.
5. The artesian water seepage at each grouting zone between the grout bags shall be verified by water pressure testing. A fluorescent dye may be introduced into the test water to visually verify the zone connection to the artesian water seepage to the surface.
6. Each grouting zone found to connect to the artesian water seepage shall be pressure grouted with a single component water reactive polyurethane chemical grout.
7. If after completion of the grouting on one side of the H-Pile, the artesian water seepage is not completely sealed, the same procedure shall be repeated on the other side of the H-Pile.

8. When the artesian water seepage in one location is satisfactorily sealed, the Contractor shall repeat the same procedure at other locations where artesian water seepage was observed.

Measurement for Payment

Measurement for payment shall be for installing sufficient MRSP for sealing the artesian water seepage at each pile location, as measured on site by the Contract Administrator. For bidding purposes, the Contractor shall assume that control of artesian water seepage will be required at five (5) pile locations at each site. The estimated tender quantity may be increased or decreased as required.

Basis of Payment

Payment shall be at the unit price bid for successfully sealing the artesian water seepage at each pile location and shall be payment in full for all labour, equipment and materials necessary to complete the work.

CONTROL OF ARTESIAN WATER SEEPAGE – GROUT - Item No.

Special Provision

Scope

Work under the above tender item shall include the supply of grout for the control of Artesian Water Seepage. The grout to be used shall be either MME-UNIVERSAL, distributed by MME Multiurethanes, Mississauga, Ontario or Hydro-Active Cut, manufactured by DeNeef Const. Chemical, distributed by Janac Sales, Toronto, Ontario.

Method of Operation

The requirements for the usage of the grout shall conform to Control of Artesian Water Seepage – Drilling and Grouting.

Measurement for Payment

Measurement is cubic metre (m³) of grout installed to seal the artesian water seepage up through the pile locations.

Basis of Payment

Payment at the contract price for the above tender item shall be full compensation for all labour, equipment and material required to supply the grout.

CSP FOR INTEGRAL ABUTMENTS – Item No.

Non-Standard Special Provision

Scope

This specification covers the requirements for the installation of the corrugated steel pipes (CSPs) at the integral abutments.

SUBMISSION AND DESIGN REQUIREMENTS

All submissions shall bear the seal and signature of an Engineer.

At least two weeks prior to commencement of installation of the abutment piles, the Contractor shall submit to the Contract Administrator, for information purposes only, three (3) sets of the working drawings.

The Contractor shall have a copy of the submitted working drawings on site at all times. Working drawings shall include at least the following:

1. Layout and elevations of the CSPs;
2. Location of reference points, and location of the centroid of each pile with respect to the reference points;
3. Construction sequence and details;
4. Source of the sand fill, and description of placing methods and equipment;
5. Location and details of all temporary bracing and spacers for the piles and CSPs;
6. Method for preventing water and debris from entering the CSP prior to placing sand; and
7. Method for preventing concrete from abutment pours from entering the CSPs during placement.

The Contractor shall be responsible for the complete detailed design of all temporary bracing, including spacers required to maintain the piles, CSP spacing and abutment stems in their specified positions through all stages of construction until the CSPs have been backfilled. All temporary bracing shall be removed.

MATERIAL

Corrugated Steel Pipe

CSP shall be in accordance with OPSS 1801 and shall be from a supplier listed under DSM#4.60.80. The CSP shall be of the diameter and wall thickness specified on the Contract Drawings, and shall be galvanized in accordance with CSA G164-M.

CSPs shall be supplied in the lengths and with the end treatments, either square or skew, as specified on the Contract Drawings; field cutting and splicing of CSPs will not be permitted. Cut ends shall be neat and free of burrs. The planes defined by the end treatments of each CSP shall be parallel to each other.

Handling and storage of CSPs shall be in accordance with the manufacturer's recommendations. Damaged CSPs shall be rejected. Localized areas of damaged galvanizing on otherwise acceptable CSPs shall be repaired with two coats of zinc-rich paint.

Sand Fill

The sand fill for backfilling the CSP shall meet the gradation requirements of Table 1 below:

Table 1 – Sand Fill Gradation Requirements

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

CONSTRUCTION

The sequence of construction shall be in accordance with the working drawings and as follows, unless otherwise approved:

1. Form concrete levelling pad and place CSPs and spacers.
2. Construct concrete levelling pads.
3. Install piles by driving to the design tip elevation or bedrock if end-bearing piles are selected.
4. Place loose sand into the CSP.
5. Remove temporary spacers.

The CSP shall be positioned such that the piles are centrally positioned within the CSP. Temporary blocking and bracing shall be used to hold the CSP in position.

The Contractor shall ensure the full perimeters of the top of all CSPs at each abutment are at the elevation and orientation shown on the working drawings.

The CSP at each pile shall be constructed to the following tolerances:

<u>Criteria</u>	<u>Tolerance</u>
Maximum deviation of CSP from pile centroid	+/- 50 mm
Maximum deviation of any point on the top perimeter of the CSP from the specified elevation	+/- 10 mm

The sand fill shall be placed dry of optimum and free-flowing, completely filling the volume between the CSP and pile. No additional compaction effort other than the action of placing the sand itself shall be applied to the sand fill.

The placing of the sand fill shall be carried out in a manner such as to not damage and displace the CSP.

Basis of Payment

Payment at the contract price for the above tender item shall include all labour, equipment and material required to do the work.

H-PILES – HP310 X 110 - Item No.

Non-Standard Special Provision

903.07.02.03.03 H-Piles, Tube Piles and Sheet Piles

Section 903.07.02.03.03 of OPSS 903 is amended by the addition of the following:

HP 310x110 pile splices shall be butt welded to the details and provisions of OPSD 3000.150.

903.07.02.07.03.03 Driving to Bedrock

Section 903.07.02.07.03.03 of OPSS 903 is deleted and replaced with the following:

When driving piles to bedrock, the Contractor shall adequately seat the pile on bedrock without damaging the pile.

In order to avoid overdriving and possibly damaging the piles when seating onto bedrock, the piles shall 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. The bedrock elevation shall be recorded. On reaching the required set, the hammer energy shall be reduced to 75 percent of the maximum energy and the pile shall then be re-driven in 2 sets of 10 blows and the penetration recorded after each set of 10 blows. The hammer energy shall then be increased to 100 percent and the pile re-driven for 10 blows and the penetration recorded. A final set of no less than 10 blows per 12 mm of penetration shall be obtained at the maximum hammer energy.

If unrealistic excessive penetration per blow is observed, driving shall be stopped and this excessive penetration immediately reported to the Contract Administrator.

The Quality Verification Engineer shall determine when the hammer energy can be increased and when the driving is complete for each pile.

RIGID EXPANDED POLYSTYRENE EMBANKMENT FILL - Item No.

Special Provision

1. SCOPE

This special provision covers the requirements for the supply and construction of the rigid expanded polystyrene backfill and associated works as shown on the Contract Drawings.

As part of the work under this item, the Contractor shall supply and place 300 mm Granular B, Type I side cushion material, at least 300 mm of Granular B, Type II bedding and a 100 mm mortar sand levelling pad, as well as polyethylene sheeting, geotextile underlay, and reinforced concrete top slab above the EPS, at the abutments as shown on the Contract Drawings.

2. REFERENCES

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

National Standards of Canada

CAN/CGSB - 51.20 M87

ASTM

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

OPSS - Ontario Provincial Standard Specification

OPSS 212	Construction Specification for Borrow
OPSS 501	Construction Specification for Compacting
OPSS 517	Construction Specification for Dewatering of Pipeline, Utility, and Associated Structure Excavations
OPSS PROV 904	Construction Specification for Concrete Structures

OPSS 1004	Material Specification for Aggregates, Miscellaneous
OPSS 1010	Material Specification for Aggregates, Base, Subbase, Select Subgrade and Backfill Material
OPSS 1605	Material Specification for Extruded Expanded Polystyrene Pavement Insulation
OPSS 1860	Material Specification for Geotextiles

3. SUBSURFACE CONDITIONS

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

4. DEFINITIONS

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

Rigid Expanded Polystyrene

Molded 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

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. 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. SUBMISSION AND DESIGN REQUIREMENTS

6.1 Submission of Shop Drawings

At least three weeks before the commencement of work, the Contractor shall submit to the Contract Administrator six 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 manufacturers' requirement.

6.3 Construction

The contractor shall submit full details of the following:

- a. The method of base preparation.
- b. Construction of leveling/drainage layer (granular base and mortar sand) as shown on the Contract Drawings.
- 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 the reinforced 30MPa concrete top slab.
- f. The method of placement of subbase material.
- g. The method of placement for Granular B, Type I side cushion material.
- h. 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 Backfill 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 and Side Cushion

The levelling pad shall consist of 100 mm thick layer of mortar sand with gradation and physical requirements as specified in OPSS 1004. Bedding below the levelling pad shall consist of 300 mm of Granular B, Type II, as indicated in the drawings. The Granular B, Type I side cushion material should conform to OPSS 1010.

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 experience background 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 week prior to commencement of work under this item. The Certificate shall state that the expanded polystyrene material is in conformance with the requirements and specifications of the contract documents.

7.2.1.2 Production Lots

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

7.2.2 Detail Requirements

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

Table 1 – Material Properties

Property	Unit	Requirements	Test Procedure
Geometry - Linear - Flatness - Squareness - Thickness	mm	1200 x 600 x 300 with tolerances $\pm 1\%$ 10 mm in 3 m \pm 0.5% -3, +5	
Compressive Strength	kPa (min)	115	ASTM D1621 (Procedure A)
Flexural Strength	kPa (min)	240	ASTM C203
Dimensional Stability	% linear change (max)	1.5	ASTM D2126
Thermal Resistance	m ² .°C/W (min for 25 mm thickness)	0.7	ASTM C177 or C518
Flammability	Limiting Oxygen Index (min)	24	ASTM D2863
Water Absorption	% by Volume (max)	4	ASTM D2842

7.2.2.1 Geometry

The expanded polystyrene shall be supplied in the form of rectangular parallel blocks of minimum acceptable dimensions of 1200 mm x 600 mm x 300 mm.

The maximum deviation from the specified linear dimensions shall be $\pm 1\%$. The flatness of the block faces shall be within ± 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%. The thickness shall be within -3 to $+5$ mm.

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% strain.

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.

7.2.2.4 Dimensional Stability

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

7.2.2.5 Thermal Resistance

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

$$R_{25\text{mm}} = \frac{R_{\text{measured}}}{\text{thickness (mm)}} \times 25$$

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

7.2.2.6 Flammability

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

7.2.2.7 Water Absorption

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

7.2.2.8 Chemical Resistance

The expanded polystyrene shall be resistant to common inorganic acids and alkalies. 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.

7.3 Polyethylene Sheeting

The plastic sheeting shall be 6 mil thick polyethylene sheeting or equivalent.

7.4 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 Preparation

Foundation preparation of the base upon which the polystyrene blocks are to be placed shall be carried out to the design elevations shown on the drawings as earth excavation or embankment construction. Any softened, loosened or deleterious materials at the base of the excavation or embankment level shall be sub-excavated and replaced with Granular 'A' or Granular 'B' Type II material in accordance with OPSS 1010.

9.2 Levelling Pad

Place, level and compact a 100 mm thick layer of OPSS 1004 mortar sand material on geotextile underlay in accordance with OPSS 501 to within ± 30 mm of the design elevation. Bedding below the mortar sand shall consist of 300 mm of Granular B, Type II on geotextile underlay. The levelling 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 levelling pad shall not be placed on frozen ground. The levelling pad must be placed in-the-dry.

9.3 Installation of Blocks

- (1) The individually marked blocks shall be placed on the prepared levelling 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. Contractor shall ensure all trimmed material is disposed of in accordance with all applicable regulations and that no trimmed debris enters the watercourse.
- (2) Subsequent successive layers shall be oriented with the long axis of blocks positioned at 90° to the previous layer in order to avoid continuous joints. Block joints shall be offset and staggered between layers except at the vertical construction joints.
- (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 levelling 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 6 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.

9.4 Concrete Top Slab

The concrete top slab shall be poured after the polyethylene sheeting is fixed in place. Place 125 mm thick layer of concrete in accordance with OPSS 904 to within ± 30 mm of the design elevation.

9.5 Backfill

Backfill over the top of the concrete slab and on the sides of the embankment shall be as shown on the Contract Drawings.

10. 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. 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. The testing shall be conducted by a recognized testing laboratory accredited by the Standards Council of Canada.

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. At a minimum, three 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. MEASUREMENT FOR PAYMENT

12.1 Actual Measurement

Measurement will be by volume in cubic metres of rigid expanded polystyrene backfill material measured in its original position based on theoretical dimensions.

13. PAYMENT

13.1 Basis of Payment

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.

At Golder Associates we strive to be the most respected global group of companies specializing in ground engineering and environmental services. Employee owned since our formation in 1960, we have created a unique culture with pride in ownership, resulting in long-term organizational stability. Golder professionals take the time to build an understanding of client needs and of the specific environments in which they operate. We continue to expand our technical capabilities and have experienced steady growth with employees now operating from offices located throughout Africa, Asia, Australasia, Europe, North America and South America.

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