



November 1, 2013

FOUNDATION INVESTIGATION AND DESIGN REPORT

**HIGHWAY 522 INTERCHANGE UNDERPASS STRUCTURE, SITE 44-464
HIGHWAY 69 FOUR-LANING FROM 1.7 KM NORTH OF HIGHWAY 529
NORTHERLY TO 3.9 KM NORTH OF HIGHWAY 522
MINISTRY OF TRANSPORTATION, ONTARIO
GWP 5347-08-00; WP 5151-08-01**

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REPORT

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PART A

FOUNDATION INVESTIGATION REPORT

HIGHWAY 522 INTERCHANGE UNDERPASS STRUCTURE, SITE 44-464

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

NORTHERLY TO 3.9 KM NORTH OF HIGHWAY 522

MINISTRY OF TRANSPORTATION, ONTARIO

GWP 5347-08-00; WP 5151-08-01



1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by URS Canada Inc. (URS) on behalf of the Ministry of Transportation, Ontario (MTO) to provide foundation engineering services for the proposed Highway 522 Interchange Underpass structure (Site No. 44-464), which is within the Contract 4 limits of the new Highway 69 alignment. The proposed work in Contract 4 is part of the four-laning of Highway 69 from 1.7 km north of Highway 529 northerly to 3.9 km north of Highway 522, for a total distance of 19.7 km. The foundation engineering components within the overall project limits include the engineering of: high fill embankments and embankments over swamps; the Canadian National Railway (CNR) re-alignment; the Bekanon Road and Highway 522 interchange and structures; the Still River, Straight Lake and Key River structures; the Canadian Pacific Railway (CPR) and CNR structures, as well as culvert crossings. The general location of this bridge along the new Highway 69 four-laning alignment is shown on the Site Location Plan on Drawing 1.

The Terms of Reference and the Scope of Work for the foundation investigation are outlined in MTO's Request for Proposal (RFP), dated January 2009. Golder's proposal for foundation engineering services associated with the Contract 4 Highway 522 Interchange Underpass structure is contained in Section 6.8 of URS's Technical Proposal for this assignment. The work has been carried out in accordance with Golder's Supplementary Specialty Plan for foundation engineering services for this project, dated April 19, 2010. The General Arrangement (GA) drawing for the proposed Highway 522 Interchange Underpass structure was provided to Golder by URS on July 23, 2012.

This report addresses the investigation carried out for the two-span Highway 522 Interchange Underpass structure and the associated approach embankments only. Separate reports address the foundation investigations for the related swamp crossings and high fill areas, culverts and other bridge structures for the project.

The purpose of this investigation is to establish the subsurface conditions at the proposed bridge structure location, including the associated approach embankments, by borehole drilling, rock coring, in situ testing and laboratory testing on selected soil and rock core samples. The investigation area is shown in plan on Drawing 2.

Preliminary subsurface information for this project is available and was supplied by MTO, in a report titled:

- *Preliminary Foundation Investigation and Design Report for Selected Structures, Highway 69 Four Laning, From 3.5 km North of Hwy 559 to 3.8 km North of Hwy 522, GWP 5377-02-00, Highway 69, GEOCRE No. 41H-50, dated September 2005, by Trow Associates Inc.*

2.0 SITE DESCRIPTION

The proposed 86.5 m long two-span structure is located in the Township of Mowat and is oriented in an east-west direction essentially perpendicular to the new Highway 69 alignment. This section of proposed Highway 69 alignment is oriented generally in a north-south direction extending from the Township of Wallbridge and to the south, along the boundary between the Township of Henvey and the Township of Mowat. The Contract 4 section of the new four-lane Highway 69 alignment is also oriented generally in a north-south direction within the overall project limits.



In general, the topography of this section of the overall project limits consists of rolling terrain, including sparsely to densely populated tree covered areas and numerous bedrock outcrops separated by valleys, rivers and swamps containing areas of standing water and various types of vegetation and organic soils. The proposed bridge structure and associated approach embankments are to be situated in an area of relatively flat ground with moderate to dense tree cover. The ground surface within the limits of the proposed structure ranges from about Elevation 185.5 m to 185.2 m along the centerline of the structure.

3.0 INVESTIGATION PROCEDURES

The field work for the proposed Highway 522 Interchange Underpass structure was carried out on October 24, 2011 and between August 13 and 16, 2012, during which time a total of five boreholes (H402-1 and B4-1 to B4-4) were advanced at the locations of the proposed structure foundation elements and approach embankments. The Record of Borehole and Drillhole sheets and the results of the laboratory testing for all of the boreholes/drillholes advanced for this bridge structure are presented in Appendix A. The locations of the boreholes and drillholes are shown on Drawing 2.

The field investigation was carried out using track mounted CME 55 and CME 850 drill rigs supplied and operated by Landcore Drilling Inc. of Chelmsford, Ontario. The boreholes were advanced using 108 mm inner diameter (I.D.) hollow-stem augers, NW casing using wash boring techniques and NQ size core barrel for bedrock coring. Soil samples were generally obtained at intervals of depth of about 0.75 m and 1.5 m, using a 50 mm outer diameter (O.D.) split-spoon sample operated by an automatic hammer on the drill rigs, in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586). Samples of the cohesive soils were obtained using 76 mm O.D. thin-walled 'Shelby' tubes (ASTM D1587, Standard Practice for Thin-Walled Tube Sampling) for relatively undisturbed samples. Field vane shear tests were conducted in cohesive soils for assessment of undrained shear strengths (ASTM D2573, Standard Test Method for Field Vane Strength Shear Test) using MTO Standard 'N' size. All boreholes were backfilled upon completion in accordance with Ontario Regulation 903 Wells (as amended).

The boreholes were advanced to depths up to 20.7 m below existing ground surface including coring of bedrock between 3.0 m and 3.2 m at Boreholes B4-2, B4-3 and B4-4.

The groundwater conditions were observed in the open boreholes during and immediately following the drilling operations and are described on the Record of Borehole sheets in Appendix A. A standpipe piezometer was installed in Borehole B4-3 to permit monitoring of the groundwater level. The piezometer consists of 50 mm diameter polyvinyl chloride (PVC) pipe, with a 1.5 m long slotted screen, sealed within a sand filter pack at a selected depth interval within the borehole. Above the sand filter pack and piezometer screen, the annulus surrounding the piezometer was partially backfilled with bentonite pellets to create a seal, then backfilled to near the surface with cuttings from the borehole and bentonite. A seal of bentonite was placed to ground surface. The piezometer installation details and water level readings are indicated on the Record of Borehole sheets contained in Appendix A.

The fieldwork was observed by members of our engineering and technical staff, who located the boreholes, arranged for the clearance of underground services, observed the drilling, sampling and in situ testing operations, logged the boreholes and examined and cared for the soil and rock samples. The samples were identified in the field, placed in appropriate containers, labelled and transported to our Sudbury geotechnical



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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. Strength testing, such as uniaxial compression strength (UCS), was carried out on selected specimens of the rock core.

The proposed centreline of the Highway 69 alignment was staked in the field by Callon Dietz Inc. (Callon Dietz), in the fall 2011, prior to drilling. Borehole locations, in stations and offsets, were measured in reference to the centreline alignment and were subsequently converted into MTM NAD 83 coordinates in AutoCAD. Where ground surface elevations were not surveyed by Callon Dietz, borehole elevations were surveyed by a member of our technical staff in reference to the ground surface elevations at the centreline median and to temporary benchmarks, which were provided by Callon Dietz. The borehole/drillhole locations shown on Drawing 1 are positioned relative to MTM NAD 83 northing and easting coordinates and the ground surface elevations are referenced to Geodetic datum. The borehole/drillhole locations and ground surface elevations are as follows:

Borehole No.	Location (MTM NAD 83)		Ground Surface Elevation (m)	Borehole Depth (m)
	Northing	Easting		
B4-1	5086291.2	221758.0	185.3	16.0
B4-2	5086301.9	221778.4	185.3	20.7
B4-3	5086307.5	221821.4	185.5	18.9
B4-4	5086318.7	221862.4	185.5	7.8
H402-1	5086323.3	221879.3	185.2	6.1

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

As delineated in *The Physiography of Southern Ontario*¹, this section of the new Highway 69 lies within the physiographic region known as the Georgian Bay Fringe, which extends along the east side of Georgian Bay through the Parry Sound and Muskoka areas, then eastward from Muskoka in patches into the area north of the Kawartha Lakes.

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

The bedrock in the area consists typically of crystalline gneisses of the Britt Domain of the Central Gneiss Belt, a subdivision of the Grenville Structural Province, as described in *Geology of Ontario*, OGS Special Volume 4². Deposition of Paleozoic strata initially covered the bedrock and later erosion during glaciation exposed these Precambrian rocks.

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

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



4.2 General Overview of Local 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 and bedrock core samples are presented on the Record of Borehole and Drillhole sheets and the laboratory test figures provided in Appendix A. The stratigraphic boundaries shown on the Record of Borehole sheets are inferred from non-continuous sampling, observations of drilling progress and the results of SPTs and in situ testing. 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 as encountered in the boreholes/drillholes is shown in profile on Drawings 2.

In general, the subsurface conditions in the underpass structure area consist of a surficial layer of topsoil or organic soil/pine needles underlain by a near surface relatively thin deposit of silty clay to clay, underlain by a relatively thick sand deposit. In the area of the west approach, west abutment and centre pier the sand deposit is underlain by interlayered deposits of silt and silty clay to clay, silt and sand whereas in the area of the east abutment and east approach, the sand deposit transitions to silt and sand, in turn underlain by a deposit of silty sand till. The lower sand deposit and silty sand deposit are underlain by granite gneiss bedrock. The overburden thickness is variable across the proposed structure, generally decreasing from west to east.

A detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

4.2.1 Topsoil/Organics

An approximately 25 mm to 75 mm thick layer of topsoil or organics consisting mainly of pine needles was encountered at the ground surface in Boreholes B4-1, B4-2, B4-4 and H402-1.

4.2.2 Silty Clay to Clay (Near Surface)

A deposit of brown oxidized silty clay to clay, trace sand, trace organics was encountered underlying the topsoil/organics in Boreholes B4-1, B4-2, B4-4 and H402-1, and from ground surface in Borehole B4-3. The top of this deposit ranges from about Elevation 185.5 m to 185.1 m and the thickness of the cohesive deposit varies between 0.4 m and 0.7 m.

The SPT 'N'-values measured within the surficial silty clay to clay deposit range from 5 blows to 17 blows per 0.3 m of penetration, suggesting a firm to very stiff consistency.

Grain size distributions of two samples of the silty clay deposit are shown on Figure A1 in Appendix A.

An Atterberg limits test was carried out on one specimen of the cohesive deposit and indicates a liquid limit of about 53 per cent, a plastic limit of about 24 per cent and a corresponding plasticity index of about 29 per cent. The results of the Atterberg limits test are shown on the plasticity chart on Figure A2 in Appendix A and classify this material as clay of high plasticity.

The natural water content measured on two samples of the silty clay to clay deposit are about 24 per cent and 30 per cent.



4.2.3 Sand

A deposit of brown to grey sand, trace to some silt was encountered in all Boreholes below the surficial silty clay to clay deposit. The top of this deposit ranges from about Elevation 184.8 m to 184.6 m and the thickness of the deposit varies between about 2.8 m and 9.7 m. In Boreholes B4-1 and B4-2, silty clay seams were encountered at depths between 6.2 m and 7.8 m below ground surface, and ranged between 200 mm and 450 mm thick.

The SPT 'N'-values measured within this deposit range from 1 blow to 17 blows per 0.3 m of penetration, indicating a very loose to compact relative density.

The grain size distributions of ten samples of the sand deposit are shown on Figure A3 in Appendix A.

The natural water content measured on ten samples of sand deposit range from about 2 per cent to 22 per cent.

The grain size distribution of one sample of a silty clay seam is shown on Figure A4 in Appendix A. Atterberg limits tests were carried out on two samples of the silty clay seams and indicate liquid limits of about 35 per cent and 49 per cent, plastic limits of about 18 per cent and 20 per cent and corresponding plasticity indices of about 17 per cent and 30 per cent. The results of the Atterberg limits tests are shown on the plasticity chart on Figure 5 in Appendix A and classify the material as silty clay of intermediate plasticity. The natural water content measured on three samples of the silty clay seams range between 32 per cent and 41 per cent.

4.2.4 Silt and Sand to Silt

A deposit comprised of grey silt and sand to silt, trace clay was encountered in Boreholes B4-3, B4-4 and H402-1. The top of the silt and sand to silt deposit is at Elevation 181.8 m to 175.3 m, and the thickness of the deposit ranges between 0.1 m and 3.1 m.

The SPT 'N'-values recorded within this deposit range from 1 blow to 4 blows per 0.3 m of penetration, indicating a very loose relative density.

Grain size distributions of two samples from the silt and sand to silt deposit are shown on Figure A4 in Appendix A.

The natural water content measured on three samples of this deposit are about 24 per cent.

4.2.5 Silty Clay to Clay

A deposit of grey silty clay to clay was encountered below the sand deposit in Boreholes B4-1 and B4-2 and below the silt deposit in Borehole B4-3. The top of the silty clay to clay deposit ranges from about Elevation 175.1 m to 172.2 m and the thickness of the deposit varies between about 1.9 m and 3.1 m. The clay deposit is described as having grey to brown varves in Borehole B4-1.

The SPT 'N'-values recorded within the silty clay to clay deposit range from 0 blows (weight of hammer) to 2 blows per 0.3 m of penetration. In situ field vane tests carried out within this deposit measured undrained shear strengths ranging from about 32 kPa to 38 kPa and the sensitivity is calculated to range from about 2 to 4. The field vane tests results indicate that the silty clay to clay deposit has a firm consistency.



Atterberg limits tests were carried out on three samples of the silty clay to clay deposit and indicate liquid limits between about 38 per cent and 54 per cent, plastic limits between about 19 per cent and 22 per cent and plasticity indices between about 17 per cent and 35 per cent. The results of the Atterberg limits tests are shown on the plasticity chart on Figure A5 in Appendix A and classify the material as silty clay of intermediate plasticity to clay of high plasticity.

The natural water content measured on three samples of this deposit ranges from about 42 per cent to 52 per cent.

4.2.6 Silt

A deposit of grey, silt, trace sand, trace clay was encountered beneath the silty clay to clay deposit in Boreholes B4-1 and B4-2. The top of the silt deposit is at Elevation 172.2 m and 172.0 m, and the thickness of the deposit is about 1.7 m and 2.4 m at the respective boreholes.

The SPT 'N'-values measured within the silt deposit range from 3 blows to 17 blows per 0.3 m of penetration, indicating a very loose to compact relative density.

A grain size distribution of one sample of this deposit is shown on Figure A6 in Appendix A.

The natural water content measured on one sample of the silt deposit is about 24 per cent.

4.2.7 Sand to Silty Sand (Till)

A deposit of sand to silty sand till was encountered below the silt deposit in Boreholes B4-1 and B4-2, below the silty clay to clay deposit in Borehole B4-3 and below the silt and sand deposit in Borehole H402-1. The top of this deposit ranges from about Elevation 179.6 m to 169.6 m and the thickness of the deposit ranges between about 0.5 m and 2.0 m. In Borehole B4-2, cobbles between 100 mm and 250 mm in size were recovered in the NQ core barrel below a depth of 16.5 m (Elevation 168.8 m).

The SPT 'N'-values within the sand to silty sand till deposit are 12 blows and 32 blows per 0.3 m of penetration, indicating a compact to dense relative density.

4.2.8 Bedrock/Refusal

The bedrock surface is inferred from refusal to casing advancement in Borehole B4-1 and refusal to split-spoon and auger penetration Borehole H402-1. These refusal depths, while they do not confirm bedrock elevations, may be inferred to indicate potential proximity to the bedrock interface. Bedrock was encountered below the sand to silty sand till deposit in Boreholes B4-2 and B4-3, and below the silt and sand deposit in Borehole B4-4 and was cored for between 3.0 m and 3.2 m lengths. The depth to bedrock/refusal below ground surface and corresponding bedrock surface elevation (inferred or actual) is summarized below.



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Foundation Element/ Approach Embankment	Borehole No.	Depth to Bedrock Surface/Refusal (m)	Bedrock Surface/ Refusal Elevation (m)	Comments
West Approach	B4-1	16.0	169.3	Refusal to further Casing advancement
West Abutment	B4-2	17.7	167.6	Bedrock cored
Center Pier	B4-3	15.9	169.6	Bedrock cored
East Abutment	B4-4	4.6	180.9	Bedrock cored
East Approach	H402-1	6.1	179.1	Spilt-spoon and auger refusal

The recovered bedrock core from Borehole B4-2 is described as fine to medium grained, fresh, foliated, grey (with pink seams) granitic gneiss. The recovered bedrock core from Borehole B4-3 is described as fine grained, weakly foliated, slightly weathered, pinkish grey granite. The recovered bedrock core from Borehole B4-4 is describes as fine to medium grained, slightly weathered, foliated, pinkish grey granitic gneiss, with moderate vuggy zones encountered between the depths of 4.9 m and 5.1 m, and 7.0 m and 7.3 m. Photographs of the recovered core samples are shown on Figure A7 in Appendix A.

The Rock Quality Designation (RQD) measured on the core samples is between 35 per cent and 100 per cent, indicating a rock mass of poor to excellent quality as per Table 3.10 of the Canadian Foundation Engineering Manual (CFEM, 2006). The RQD is generally between 76 per cent and 100 per cent (i.e., good to excellent quality) with one measured RQD of 35 per cent (poor quality) in the bottom 0.8 m core run in Borehole B4-3. The Total Core Recovery (TCR) of all samples recovered is 100 per cent and the Solid Core Recovery (SCR) was between 42 per cent and 100 per cent, respectively.

Laboratory Unconfined Compression Strength (UCS) testing was carried out on three selected samples of the recovered bedrock core. 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 (R4 to R5, 50 MPa < UCS < 250 MPa) in accordance with Table 3.5 of CFEM (2006).

Borehole/ Core Run	Elevation (m)	UCS (MPa)
B4-2/#2	166.4	51
B4-3/#2	168.4	144
B4-4/#1	179.8	130

4.2.9 Groundwater Conditions

In general, the overburden samples taken in the boreholes were moist to wet. The water level observed in the boreholes upon completion of drilling varies between about Elevation 181.8 m and 177.1 m, measured at 3.4 m to 8.2 m below ground surface, respectively. Borehole B4-4 was dry upon completion of drilling.



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A standpipe piezometer was installed in Borehole B4-3 to allow monitoring of the ground water level at the site. Details of the piezometer installation are shown on the Record of Borehole sheets in Appendix A. The groundwater level measured in the piezometer installation is summarized below.

Foundation Element	Borehole No.	Ground Surface Elevation (m)	Groundwater Elevation (m)	Date of Measurement
Center Pier	B4-3	185.5	180.6 180.5	August 15, 2012 August 29, 2012

The groundwater level in the area is subject to seasonal fluctuations and precipitation events, and should be expected to be higher during wet periods of the year. The August period of the field investigation was relatively dry and in comparison, the water level observed in open Borehole H402-1, drilled in October 2011, recorded the highest (unstabilized) water level at Elevation 181.8 m that is between about 0.9 m and 4.7 m higher than in the other boreholes/piezometer.

5.0 CLOSURE

The field personnel supervising the drilling program were Messrs. Indulis Dumpis and Adam Core, E.I.T. This report was prepared by Mr. Adam Core, E.I.T., and the technical aspects were reviewed by Ms. Sarah Coyne, P.Eng., a senior geotechnical engineer and Associate with Golder. Mr. Jorge M. A. Costa, P.Eng., Golder's Designated MTO Contact for this project and Principal with Golder, conducted an independent quality control review of the report.



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Report Signature Page

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PART B

FOUNDATION DESIGN REPORT

HIGHWAY 522 INTERCHANGE UNDERPASS STRUCTURE, SITE 44-464

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

NORTHERLY TO 3.9 KM NORTH OF HIGHWAY 522

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6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides engineering recommendations for detail design of the proposed Highway 522 Interchange Underpass structure. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the subsurface investigation. The discussion and recommendations presented are intended to provide the designers with sufficient information to assess the feasible foundation alternatives and to carry out the design of the structure foundation and approach embankments. Where comments are made on construction, they are provided in order to highlight those aspects which could affect the design of the project. Those requiring information on the aspects of construction should make their own interpretation of the factual information provided as such interpretation may affect equipment selection, proposed construction methods, scheduling and the like.

6.1 General

Golder Associates Ltd. (Golder) has been retained by URS Canada Inc. (URS) on behalf of the Ministry of Transportation, Ontario (MTO) to provide recommendations on foundation aspects for the Detail Design of the Highway 522 Interchange Underpass structure within Contract 4 of the proposed Highway 69 alignment. The proposed structure is one component of the four-laning of Highway 69 extending from the Township of Wallbridge to the south and along the boundary between the Township of Henvey and the Township of Mowat.

The proposed structure is located in the Township of Mowat and is oriented in a west-east direction essentially perpendicular to the new Highway 69 alignment. Based on the General Arrangement (GA) Drawing provided by URS, we understand the proposed underpass will be an 86.5 m long by 11.6 m wide, two-span structure. The grade of the proposed bridge deck varies between about Elevation 192.8 m and Elevation 192.1 m at the west and east abutments, respectively. The existing ground surface varies between about Elevation 185.2 m and Elevation 185.5 m at the borehole locations. The proposed approach embankments will be up to about 7.5 m high above the existing ground surface and the new Highway 69 grade will be in an approximately 1 m to 3 m cut, for a total embankment height of about 10.5 m, relative to the outside ditches of Highway 69.

The subsurface conditions in the vicinity of the proposed bridge structure generally consist of a very loose to compact deposit of sand extending to depths of between about 9.7 m below ground surface at the west abutment and about 2.8 m below ground surface at the east abutment. Near the west abutment and the center pier, the sand deposit is underlain by an upper deposit of loose silt, a deposit of firm silty clay to clay and a lower deposit of very loose to loose silt. The silt deposits are between about 1.7 m and 3.1 m thick and the silty clay to clay deposit is between about 1.9 m and 3.1 m thick. The lower silt deposit and silty clay to clay deposit in this area are in turn underlain by a 0.6 m to 1.2 m thick layer of compact to dense sand till, overlying bedrock. The bedrock surface, as determined by drilling (i.e., NW casing or auger and split spoon) refusal and bedrock coring, is 4.6 m and 17.7 m below ground surface at the east and west abutments, respectively. Near the east abutment and approach, the sand deposit is underlain by a 0.1 m to 2.7 m thick deposit of very loose to loose silt and sand and/or very dense silty sand till, overlying bedrock. Details of the stratigraphic profile and soil conditions are provided on Drawing 1 and on the Record of Borehole sheets in Appendix A.



6.2 Foundation Recommendations

Consideration has been given to the use of either shallow or deep foundations for the support of the new bridge structure. Given the presence of the compressible cohesive deposit (firm silty clay to clay at depth) at the west abutment, shallow foundations are not recommended due to low geotechnical resistance that would be available to support the abutment as well as the potential settlement of the subsoils as a result of the footing pressure and the embankment loading. Although the silty clay to clay deposit is not present in the boreholes at the east abutment, and shallow foundations could be suitable for support of the abutment, it is not common practice to mix foundation types due to the potential for differential settlement between foundation elements.

We recommend that deep foundations be utilized for support of the bridge structure. Deep foundations could consist of driven steel H-piles, HSS (pipe) piles or concrete caissons. Table 1 summarizes the advantages, disadvantages, relative costs and risks/consequences of the foundation alternatives for the proposed bridge structure. Discussion and design recommendations for the various alternatives, where appropriate, are given in the sections below.

6.2.1 Steel H-Pile Foundations

We recommend that the bridge be supported on steel HP310X110 piles driven to bedrock. The following sections provide details for geotechnical axial resistances/reactions and downdrag loads, set criteria and pile driving notes, resistance to lateral loads and frost protection.

Deep foundations comprised of steel pipe piles are not considered appropriate for this site due to the higher risk of pile damage associated with seating the piles on the potential sloping bedrock. Further, there is limited data on axial resistance of pipe piles and uncertainty at this time whether an integral abutment design is applicable for pipe pile foundations. Therefore, pipe piles are not considered suitable for this site and are not discussed further herein.

6.2.1.1 Geotechnical Axial Resistances/Reactions

The following summarizes the approximate pile tip elevation(s), approximate pile length(s) and the factored geotechnical axial resistance at ultimate limit states (ULS) and geotechnical reaction at serviceability limit states (SLS) for steel HP310X110 piles driven to bedrock. The estimated tip elevations and estimated pile lengths are based on the underside of pile caps given on the GA drawing. There should be a provision made in the Contract for dealing with varying pile lengths due to the variability in depth to the bedrock surface and the lengths given below should be considered minimum lengths.



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Foundation Element	Approximate Pile Tip Elevation(s)	Approximate Pile Length ¹	Factored Geotechnical Axial Resistance at Ultimate Limit States (ULS)	Geotechnical Reaction at Serviceability Limit States (SLS) for 25 mm of Settlement
West Abutment	167.6 m	19.4	2,000 kN	N/A ²
Centre Pier	169.7 m	10.9	2,000 kN	N/A ²
East Abutment	180.9 m	5.2	2,000 kN	N/A ²

- Notes:
1. Assuming the underside of the pile cap is at approximately Elevation 187.0 m and 186.1 m for the west and east abutments, respectively and 180.6 m for the centre pier as per the General Arrangement Drawing.
 2. The geotechnical reaction at SLS for 25 mm of pile settlement on granite or granitic gneiss bedrock will be greater than the factored axial resistance at ULS for piles on bedrock and as a result the SLS condition does not apply.

Due to the variability in elevation of the bedrock surface, bedrock excavation consisting of trenching socketing will be required at the east abutment to achieve a minimum 5 m pile length required for integral abutment design. For piles installed in a trench or socket in bedrock, the piles will have to be fixed in concrete at the base of the trench/pre-drilled holes for a sufficient depth (to be determined by the structural engineer) to achieve fixity of the lower section of the pile. The thickness of the concrete would need to be considered when determining the bedrock excavation depths. The bedrock at the founding depth is expected to be of fair quality and the founding surface should be properly prepared using controlled rock excavation/blasting techniques for a trench or pre-drilled sockets. Recommendations for bedrock blasting are provided in Section 6.6.6.

In addition, once bedrock excavation is complete and the piles have been “fixed” in concrete at the base of the pre-drilled sockets or bedrock trench, a cone of compacted OPSS.PROV1010 Granular B Type II backfill will be required around the piles to the underside of the abutment/pile cap, as discussed in Section 6.6.5.

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 may be the case for this site, the CSPs should be backfilled with a loose, fine to medium sand. A Non Standard Special Provision (NSSP) detailing the installation method and gradation of this sand should be included in the Contract Documents; an example is provided in Appendix B.

Due to the required grade raise at the west and east abutments for the respective approach embankments, a compacted granular pad will be required below the abutments through which the piles will be driven. Based on the GA drawing, the underside of the pile caps are 1.7 m and 0.6 m above the existing ground surface at the west and east abutments, respectively. Since the new Highway 69 cut will be made prior to construction of the bridge abutments, the thickness of the granular pad will need to extend down towards the future Highway 69 ditches, which are 3.0 m and 3.2 m below the existing ground surface at the west and east abutments, respectively. The total pad thickness is variable across the width of the proposed Highway 69 cut. The granular pad should be designed and constructed as described in Section 6.6.5.

6.2.1.2 Downdrag

The proposed Highway 522 approach embankment, which will be up to about 7.5 m above the existing ground surface for the side slopes and up to 10.5 m above the future Highway 69 ditches, will induce settlement of the underlying firm silty clay to clay deposit at the west abutment. Downdrag loads (negative skin friction) will be induced on the piles supporting the abutment as a result of the addition of approach embankment fill after pile



installation is complete, causing 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 west abutment unless mitigation of settlement is carried out prior to pile installation (Section 6.5.3).

The structural design of the west abutment piles should be based on an estimated unfactored downdrag load of 120 kN acting on the piles (HP310X110). 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 Canadian Highway Bridge Design Code (CHBDC, 2006) for ULS conditions. Downdrag loads do not have to be considered at the center pier since there will not be an embankment constructed at this location. Downdrag loads do not have to be considered at the east abutment, since cohesive deposits were not encountered at this location.

6.2.1.3 Set Criteria

Pile installation should be carried out in accordance with Ontario Provincial Standard Specification (OPSS) 903 (Deep Foundations). The piles should be provided with rock points to assist in seating the pile on potentially sloping bedrock. The piles should be provided with rock points such as Titus Injector or Oslo Point as per Ontario Provincial Standard Drawing (OPSD) 3000.201 (HP310 Oslo Point) or equivalent. An NSSP should be included in the Contract Documents to address the requirements for rock points; an example is included in Appendix B.

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

- The piles should be driven to an initial set equal to or greater than 10 blows per 12 mm of penetration (unless abrupt peaking occurs) using a hammer with rated energy of about 50 kilojoules, but not exceeding 60 kilojoules.
- On reaching the required set, the hammer energy should be reduced to 75 per cent and the pile should be re-driven in 2 sets of 10 blows to improve the process of seating the pile on the sloping bedrock surface.
- A final set of no less than 10 blows per 12 mm of penetration should be obtained at the maximum hammer energy. Provision should be made to re-tap all piles to confirm the set after adjacent piles have been driven.

A NSSP, which outlines the above criteria for seating the piles on bedrock, should be included in the Contract and an example is included in Appendix B.

6.2.1.4 Pile Driving Note

The pile driving note that should be added to the drawings for this project is Note 5 in Clause 3.3.3 of the MTO's Structural Manual (MTO 2008), as follows:

- "Piles to be driven to bedrock."



6.2.1.5 Resistance to Lateral Loads

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

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

The resistance to lateral loading in front of a single pile may be calculated using subgrade reaction theory where the coefficient of horizontal subgrade reaction, k_h (kPa/m), is based on the following equations (CFEM, 1992 as referenced in the CHBDC Commentary, 2006):

for non-cohesive soils:

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

where:

$$\begin{aligned} n_h &= \text{constant of subgrade reaction (kPa/m)} \\ z &= \text{depth (m)} \\ B &= \text{pile diameter or width (m)} \end{aligned}$$

and for cohesive soils:

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

where:

$$\begin{aligned} s_u &= \text{undrained shear strength of the soil (kPa)} \\ B &= \text{pile diameter or width (m)} \end{aligned}$$

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

Foundation Element (Relevant Boreholes)	Soil Unit	Elevation (m)	n_h (kPa/m)	s_u^1 (kPa)
West Abutment (Borehole B4-2)	Loose Sand within CSP	187.0 – 184.0	2,200	-
	Very Loose to Compact Sand	184.0 – 175.1	1,300	-
	Firm Silty Clay	175.1 – 172.0	-	30 to 35
	Loose to Compact Silt	172.0 – 169.6	1,300	-
	Compact to Dense Sand (Till)	169.6 – 167.6	4,400	-
Centre Pier (Borehole B4-3)	Very Loose to Compact Sand	180.6 – 175.3	1,300	-
	Loose Silt	175.3 – 172.2	1,300	-
	Firm Silty Clay	172.2 – 170.3	-	35 to 38
	Compact to Dense Sand (Till)	170.3 – 169.7	4,400	



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Foundation Element (Relevant Boreholes)	Soil Unit	Elevation (m)	n_h (kPa/m)	s_u^1 (kPa)
East Abutment (Borehole B4-4)	Loose Sand within CSP	186.1 – 183.1	2,200	-
	Loose to Compact Sand to Silt and Sand	183.1 – 180.9	1,300	-

Note ¹: Values taken as per the design line shown on Figure 1.

For a single HP 310x110 vertical pile, the estimated factored lateral resistances at ULS as well as the estimated lateral reactions at SLS (for 10 mm of horizontal deflection at the pile cap) are presented below. These values are based on analysis carried out using the commercially available program LPILE Plus (Version 5.0), developed by Ensoft Inc.

Foundation Element	Factored Geotechnical Lateral Resistance at ULS	Geotechnical Lateral Reaction at SLS for 10 mm of Deflection
West Abutment	100 kN	20 kN
Centre Pier	100 kN	20 kN
East Abutment	100 kN	20 kN

Note ¹: All scenarios assume a 2,000 kN dead load applied at the top of the pile.

Both structural and geotechnical resistances of the piles should be evaluated to establish the governing case at ULS. At SLS, the horizontal reaction 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 reaction 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).

Group action for lateral loading should also be considered when the pile spacing in the direction of loading is less than six to eight pile diameters. Group action can be evaluated by reducing the coefficient of lateral 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, R
8d	1.00
6d	0.70
4d	0.40
3d	0.25

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



6.2.1.6 Frost Protection

All pile caps should be provided with a minimum of 1.9 m of soil cover for frost protection as per OPSD 3090.101 (Foundation Frost Penetration Depths for Southern Ontario).

6.2.2 Caissons

Consideration could be given to the use of caissons for support of the abutments although caissons are not considered to be practical at this site due to the high water level and because caisson foundations preclude integral abutment design. The high axial capacity of the caissons would result in fewer units being required to support the abutments than that required for the H-pile design and the possible elimination of a pile cap. It should be noted, however, that there may be difficulty in socketing the large diameter caissons within the strong to very strong granite gneiss bedrock and achieving an adequate seal. Temporary liners and tremie concrete will likely be required to install caissons at this site. As well, the compact to dense sand till at the west abutment and the sloping bedrock surface at the east abutment, may also present some difficulties in caisson installation.

6.2.2.1 Geotechnical Axial Resistance

If caissons are considered as a foundation alternative, the caissons at this site will derive their axial resistance mainly from the shaft resistance of the rock socket. The contribution from end-bearing has been neglected in the analysis due to the difficulties in cleaning and inspecting the base of the sockets, which will be below the water level and at great depth. The factored geotechnical axial resistances at ULS for two different caisson diameters socketed a minimum of 2 m into the bedrock are given below.

Caisson Diameter (m)	Gneiss Bedrock (minimum 2 m socket)	
	ULS (kN)	SLS for 25 mm
1.2	6,000	n/a
1.5	8,000	n/a

The geotechnical reaction required to achieve 25 mm of settlement is greater than that given for ULS for caissons socketed into the bedrock and, therefore, SLS conditions do not apply.

It should be noted that basal heaving within the caisson could occur during installation through the overburden and a sufficient head of water should be maintained inside the liner at all times to balance the hydrostatic pressures.

6.2.2.2 Downdrag

Downdrag loads will need to be taken into account for design of the caissons supporting the west abutment unless mitigation of settlement is carried out prior to pile installation (Section 6.5.3). As caissons are not



proposed for the abutments at this time, downdrag loads for the foundation elements at this location are not required by the designer.

6.2.2.3 *Resistance to Lateral Loads*

The geotechnical resistance to lateral loading for the caissons should be calculated in accordance with Section 6.2.1.5 using the horizontal subgrade reaction formulas. As caissons are not proposed for the abutments at this time, lateral capacities for the foundation elements at this location are not required by the designer.

6.2.2.4 *Frost Protection*

The pile caps for the caissons at the abutments should be provided with a minimum of 1.9 m of conventional soil cover for frost protection as described in Section 6.2.1.6.

6.3 Seismic Considerations

6.3.1 Site Coefficient

In accordance with Section 4.4.6, Table 4.4 of the CHBDC, the soils at the proposed bridge structure are categorized as Soil Profile Type IV and as such, the Site Coefficient, S , is 2.0.

6.3.2 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, this site is located in Seismic Performance Zone 1. The site-specific zonal acceleration ratio for the Sudbury and Parry Sound areas is 0.05. Based on the subsurface conditions at this site, a 100 per cent amplification of the ground motion may occur (i.e., Site Coefficient, $S=2.0$ for Soil Profile Type IV from Table 4.4 of CHBDC), resulting in an increase in the Peak Horizontal Acceleration (PHA) from 0.05 g to 0.10 g at the ground surface.

We understand based on Section 4.4.4, Table 4.1 of the CHBDC, that this bridge structure is assigned Seismic Performance Zone 1. Given this, and in accordance with Section 4.4.5.1, Table 4.2 of the CHBDC, no seismic analysis is required for structures located in Seismic Performance Zone 1.

6.4 Lateral Earth Pressures

The lateral earth pressures acting on the abutment stems and any associated wing walls/retaining walls will depend on the type and method of placement of the backfill material, the nature of the soils behind the backfill, the magnitude of surcharge including construction loadings, the freedom of lateral movement of the structure, and the drainage conditions behind the walls. As discussed in Section 6.3.2, seismic (earthquake) loading need not be analyzed for this structure.



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

- Select, free draining granular fill meeting the specifications of Special Provision (SP) 110S13 Granular 'A' or Granular 'B' Type II, but with less than 5 per cent passing the No. 200 sieve, should be used as backfill behind the walls. Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Compaction (including type of equipment, target densities, etc.) should be carried out in accordance with SP 105S21 (Compacting). Other aspects of the granular backfill requirements with respect to sub-drains and frost taper should be in accordance with OPSD 3121.150 (Walls, Retaining, Backfill, Minimum Granular Requirement).
- For structures that are not comprised of integral or semi-integral abutments, rock fill may be used as backfill behind the walls and the material should meet the specification as outlined in the Northern Region Directive (2002) for backfill of structures adjacent to rock embankments. Other aspects of rock backfill requirements should be in accordance with OPSD (3101.200 Walls, Abutment, Backfill, Rock).
- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the wall stem, in accordance with CHBDC Section 6.9.3 and Figure 6.6. Other surcharge loadings should be accounted for in the design as required.
- For restrained walls, granular fill should be placed in a zone with the width equal to at least 1.9 m behind the back of the wall (in accordance with Figure C6.20(a) of the Commentary to the CHBDC). For unrestrained walls, fill should be placed within the wedge shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical (1.5H:1V) extending up and back from the rear face of the footing (in accordance with Figure C6.20(b) of the Commentary to the CHBDC). The pressures are based on the proposed embankment fill material and the following parameters (unfactored) may be used:

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

If the wall support and superstructure allow lateral yielding of the stem, active earth pressures may be used in the foundation design of the structure. If the wall support and superstructure does not allow lateral yielding, at-rest earth pressures should be assumed for foundation design. The movement required to allow active pressures to develop within the backfill, and thereby assume an unrestrained structure for design, should be calculated in accordance with Section C6.9.1 and Table C6.6 of the Commentary to the CHBDC.



6.5 Approach Embankment Design

Based on the GA drawing by URS, the Highway 522 underpass crossing the new Highway 69 will require fill embankments up to 7.5 m above the existing ground surface. Since the new Highway 69 will be in a cut section at the bridge location, the front embankment at the bridge abutments will be up to 10.5 m above the existing ground surface.

Sections 6.5.1 and 6.5.2 of this report summarize the methods used to analyze the stability and settlement, and the criteria, parameters and results of analysis for the critical sections of the approach embankments. Recommendations for mitigating stability and/or time-dependent settlements are presented in Section 6.5.3. General aspects of subgrade preparation and embankment construction are presented in Section 6.6.

At all areas, the analyses assume that the 0.5 m to 0.8 m thick layer of organics (topsoil) and near surface cohesive soils will be removed below the approach embankments prior to construction of the new embankments (as discussed in Section 6.6.1).

The piezometric conditions required in the analyses are based on a groundwater level at Elevation 182.5 m, which corresponds to the elevation of the proposed Highway 69 median. It should be noted that this water level is higher than that measured in the boreholes and piezometers, however, it was considered to be an extremely dry year at the time of drilling and it is anticipated that the water level could rise as high as the level of the proposed Highway 69 cut in this area.

It is understood that rock fill is the preferred embankment fill material for this project due to the availability from rock blasting for road cuts required elsewhere on the project. In this regard, the stability and settlement analyses discussed in Sections 6.5.1 and 6.5.2 have been carried out on the basis that the highway embankment will be constructed of rock fill formed at side slopes of 1.25 Horizontal to 1 Vertical (1.25H:1V). In the immediate abutment area, the embankment will be constructed of either Granular 'B' Type II or the Granular 'A' pad to facilitate pile driving.

6.5.1 Stability

The critical sections (i.e., the greatest embankment height and maximum thickness of compressible soils) for the Highway 522 Underpass structure are the northwest side slope and the west front slope. Analyses were performed on the critical sections of the new approach embankments to assess the stability of the proposed embankment heights and geometries. The following sections summarize the methodology, simplified stratigraphy, unit weights and strength parameters employed for the different soil types in the approach areas.

6.5.1.1 Methodology

All limit equilibrium slope stability analyses were performed using the commercially available program GeoStudio 2007 (Version 7.19) produced by Geo-Slope International Ltd., employing the Morgenstern-Price method of analysis. For all analyses, the Factors of Safety (FoS) of numerous potential failure surfaces were computed in order to establish the minimum FoS. The FoS is defined as the ratio of the forces tending to resist failure to the driving forces tending to cause failure. A target minimum FoS of 1.3 is normally adopted for the design of embankment slopes under static conditions for MTO embankments. This FoS is considered adequate for the



embankments at this site considering the design requirements and the field data available and is based on deep-seated, global failure surfaces that would affect the operation of the highway. The stability analyses were carried out to check that the target minimum FoS was achieved for the design embankment heights and geometries.

6.5.1.2 Parameter Selection

The simplified stratigraphy together with the associated strengths and unit weights employed for the different soil types at the critical sections are summarized below.

Soil Type	Unit Weight, γ (kN/m ³)	Undrained Shear Strength (kPa)	Effective Friction Angle, ϕ' (°)
Rock Fill	19	0	40
Granular 'A' Fill	21	0	35
Organics	12	1	27
Silty Clay (Near Surface)	15	20	-
Sand	19	0	28
Silty Clay to Clay	16	30 – 35 (see Figure 1)	-
Silt	18	0	28
Sand Till	20	-	30

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

For cohesive deposits, total stress parameters were employed in the analyses assuming undrained conditions. The total stress parameters (i.e., average mobilized undrained shear strength – s_u) for the cohesive soils were assessed based on the results of in situ field vane shear tests, and estimated from correlations with the SPT results and other laboratory test data (i.e., natural water content), where appropriate. Where appropriate, Bjerrum's correction factor was employed to estimate the average mobilized undrained shear strength from the results of the in situ field vane tests as follows:

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

where:

$$s_{u(mob)} = \text{average mobilized undrained shear strength (kPa)}$$

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

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

6.5.1.3 Results of Analysis

Analysis for the northwest side slope was carried out at Sta. 9 +940 (about 10 m west of the proposed west abutment). The proposed approach embankment side slope is about 7.5 m high above the existing ground



surface and the cohesive deposit, which was encountered at a depth of about 10 m below the existing ground surface, is up to about 3.1 m thick. The stability analysis indicates that after completion of construction, the approach embankment side slope will have a FoS of approximately 1.4 for a deep-seated, global failure surface that would impact the operation of the highway (see Figure 2).

Analysis for the west abutment front slope of the proposed bridge abutment was also carried out for an embankment height of about 10.5 m above the future Highway 69 ditch grade. The cohesive deposit is up to about 3.1 m thick at the abutment location. The stability analysis indicates that after completion of construction, the approach embankment front slope will have a FoS of approximately 1.4 for a deep-seated, global failure surface that would impact the operation of the highway (see Figure 3).

The results of the analyses for side slope and front slope sections typically indicate shallow surficial slip surfaces with a Factor of Safety less than 1.3. However, these surficial slip surfaces are not representative of true deep seated failure conditions. Selection of the minimum Factor of Safety involves engineering judgement on the results generated by the computer program and selection of a realistic failure surface that would impact the operation of the roadway.

Based on the results of the stability analyses carried out at the critical sections, stability mitigation measures will not be required for approach embankments to the Highway 522 Underpass structure.

6.5.2 Settlement

Settlement of the approach embankments can be expected as a result of the loading from the new fills on the compressible foundation soils at this site, particularly at the west approach embankment where such soils are present. In addition, depending on the type of fill materials employed in the construction, settlements may also occur due to compression of the new embankment fill itself. The following sections summarize the methodology, criteria, simplified stratigraphy, unit weights and deformation parameters employed for the different soil 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.

6.5.2.1 Methodology

To estimate the magnitude of the expected settlements, analyses were carried out on the critical sections of the proposed approach embankments using both the commercially available program Settle^{3D} (Version 2.016), developed by Rocscience Inc., and hand/spreadsheet calculations.

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

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

The sources of settlement were considered to include:



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- immediate settlement of the native granular soils;
- primary time-dependent consolidation of the cohesive deposits (using Terzaghi's one-dimensional consolidation theory);
- secondary time-dependent (creep) consolidation of the cohesive deposits (long-term); and
- self-weight compression of the embankment fill materials (long-term).

6.5.2.2 Settlement Criteria

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

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

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

6.5.2.3 Parameter Selection

The simplified stratigraphy together with the associated strengths and unit weights employed for the different soil types at the approach embankments are summarized below.

Soil Type	Location	Elevation (m)	γ	E	e_o	C _c	C _r	σ_{vo}'	σ_p'
Sand	West Appr.	184.8 to 175.1	19	12	-	-	-	-	-
	East Appr.	184.6 to 181.8							
Clay	West Appr.	175.1 to 172.0	16	-	1.3	0.5	0.03	130-155	135-160
Silt to Silt and Sand	West Appr.	172.0 to 169.6	18	8	-	-	-	-	-
	East Appr.	181.8 to 179.6							
Sand to Silty Sand Till	West Appr.	169.6 to 167.6	20	50	-	-	-	-	-
	East Appr.	179.6 to 179.1							

γ unit weight (kN/m³)
 E Young's Modulus (MPa)
 e_o initial void ratio
 C_c compression index (based on void ratio)
 C_r recompression index (based on void ratio)
 σ_{vo}' initial vertical effective stress (kPa)
 σ_p' preconsolidation stress (kPa)



The immediate compression of the non-cohesive deposits was modelled by estimating an elastic modulus of deformation based on the SPT 'N'-values and using correlations proposed by Bowles (1984) and Kulhawy and Mayne (1990).

The consolidation settlement of the cohesive deposits was assessed using the results of the laboratory index testing to determine deformation parameters (i.e., recompression and compression indices) using empirical correlations proposed in literature by Koppula (1986), Terzaghi and Peck (1967), Kulhawy and Mayne (1990) and Azzouz et al. (1976). The correlations by Koppula (1986) and Terzaghi and Peck (1967) relating the natural water content and liquid limit to the compression index was found to be the most consistent with the correlated laboratory index testing.

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

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

where:

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

The coefficient of consolidation, c_v (cm^2/s), required in the time-rate settlement analysis was estimated from the NAVFAC (1982) correlation with liquid limit assuming normally consolidated soils.

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

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

where:

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

6.5.2.4 Settlement of Approach Embankment Fill

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

- type of rock/strength of particles;
- size and shape of rock particles;



- gradation of rock fill;
- total height/thickness of rock fill (stress level); and
- method of construction and sequence of placement (including lift thickness, compactive effort and state of packing).

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

Rock fill should be placed, whenever possible, in a controlled manner (i.e., not end-dumped) in accordance with SP 206S03 (Rock Excavation, Grading). Blading, dozing and 'chinking' the rock fill to form a dense, compact mass is required to minimize voids and bridging and reduce settlements and should be used to construct rock fill embankments above the existing groundwater table. Where rock fill cannot be placed in a controlled manner (i.e., below the groundwater table), the post-construction settlement of the rock fill is expected to be greater.

Short-Term Rock Fill Settlement

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

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

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

Long-Term Rock Fill Settlement

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

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



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

6.5.2.5 Settlement Results

The estimated magnitude of immediate settlement of the cohesionless soils, the primary and secondary consolidation settlement of the cohesive soils as well as the short-term and long-term rock fill settlement at each approach embankment is presented below.

Approach	Estimated Settlement (mm)						Post-Construction ¹
	Non-Cohesive Deposit(s)	Cohesive Deposit(s)		Rock Fill		Total	
	Immediate	Primary	Creep	Short Term	Long Term		
West	150	160	20	60	~10	400	250
East	60	n/a	n/a	55	~10	125	65

Note ¹: Assumes that post-construction settlement begins when the embankment has reached its final height. In this case, this includes all of the primary and secondary (creep) consolidation settlement and all rock fill settlement.

Based on an estimated coefficient of consolidation (c_v) of about $2.0 \times 10^{-3} \text{ cm}^2/\text{s}$ estimated for the cohesive deposit based on correlations with laboratory test data, the imposed loading conditions and assuming two-way drainage of the cohesive deposit, it is estimated that about 90 per cent of the primary consolidation settlement will be completed in about 4 months.

Based on an estimated modified secondary compression index ($C_{\alpha\epsilon}$) of 0.004, the magnitude of secondary (creep) settlement for the cohesive deposit at the west approach is estimated to be about 10 mm per log cycle of time. Assuming creep settlement begins at about 90 per cent of the primary consolidation, about 20 mm of creep will occur over the design life of the approach embankment (i.e., 20 years).

The total settlement of the rock fill embankment at the west approach is based on a 7.5 m high embankment plus about 0.5 m of additional fill required after removal of organics and near surface cohesive deposit. At the east approach, the total settlement of the rock fill embankment is based on a 7.0 m high embankment plus about 0.8 m of additional fill required after removal of organics and near surface cohesive deposit.

Approximately 90 per cent of the estimated short-term rock fill settlement is expected to occur within six months following construction, with the remaining settlement expected to occur over the remaining design life of the roadway embankment.

Since the estimated post-construction settlement of the subsoils and embankment rock fill is greater than the settlement criteria, mitigation measures to reduce the magnitude of post-construction settlement are required at both approaches.



6.5.3 Mitigation of Time-Dependent Settlement

The presence of the up to 3.1 m thick silty clay to clay deposit at the west approach influences the magnitude of settlement of the up to 7.5 m high approach embankment. In order to minimize post-construction settlement of both the silty clay to clay deposit and the rock fill embankment itself, 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. Compressible soils were not encountered at the east abutment so settlement mitigation is only required to mitigate rock fill settlement. We recommend preloading for 12 months and 6 months at the west and east approaches, respectively, as the preferred technical solution for long-term performance of the roadway embankment.

6.5.3.1 Preloading

We recommend preloading the approach embankments in order to reduce the post-construction settlements of the proposed embankments. Preloading refers to the placement of fill to the proposed profile grade of the embankment in advance of final construction in order to preconsolidate the underlying compressible soils. Preloading reduces the magnitude of long-term, post-construction settlements by promoting such settlements to occur under embankment fill loads in advance of final grading of the embankment. This option is most suited for areas where a delay in the construction schedule is acceptable or can be accommodated.

Preloading requires placement of embankment fill and monitoring of settlements, and possibly pore pressures, for a period of time corresponding to approximately the 'End of Primary' (EoP) consolidation of clayey subsoils. Long-term secondary (creep) settlements will still continue to occur over the design life of the embankment; however, such settlements would be less than primary consolidation settlements.

We recommend a 12-month preload period at the west approach embankment and a six month preload period for the east approach embankment.

At the west approach embankment, it is estimated that 90 per cent of primary consolidation settlement will be completed in about four months after construction of the embankment to the design surface elevation at which time creep begins. If only four months of preloading is applied, then the total post-construction settlement after the preload period will be about 50 mm, consisting of about 15 mm of remaining primary settlement, 20 mm of creep settlement still to occur and about 15 mm of short and long-term rock fill settlement. Therefore, we recommend a 12-month preload period after which time the total post-construction settlement will be reduced to about 25 mm, consisting of 15 mm of creep settlement and 10 mm of long term rock fill settlement. At the east approach, the post-construction settlement will be 15 mm consisting of about 5 mm of remaining short term-term rock fill settlement and 10 mm long-term rock fill settlement.

Instrumentation and settlement monitoring during and after the construction of the west approach embankment will be required (Section 6.6.6); however, monitoring of the east approach embankment is not required. If the construction schedule can accommodate this preload period, by constructing the embankment as early as possible, preloading the foundation soils should be considered as the settlement mitigation measure.



6.5.3.2 *Surcharging with Toe Berms*

Similar to preloading, surcharging refers to the placement of embankment fill in advance of final pavement construction to reduce long-term, post-construction settlements (including creep). The difference between preloading and surcharging is the amount of fill placed and the time required for consolidation to be achieved. With surcharging, the preload is placed as described in Section 6.5.3.1, followed by an additional lift of fill (the surcharge) above that required to construct the final embankment geometry. This additional lift of fill applies greater stress to the underlying clayey deposit and increases the rate of primary consolidation over that achieved by preloading only, resulting in over consolidation of the underlying compressible foundations soils. At the EoP consolidation, the portion of the surcharge fill remaining above the required embankment height (sub base level) is removed. The surcharge fill can also be left in place for a longer duration to further reduce the long-term, secondary (creep) settlements. The major advantage of surcharging is that the total amount of creep is reduced and delay to the schedule is decreased. However, surcharging will not assist with reducing the amount of time required for rock fill settlement to occur.

If there is insufficient time in the schedule to allow for the full 12 month preload period at the west approach embankment, consideration could be given to adding a 2 m surcharge onto the embankment and reducing the preload period to four to six months. In this case, there would be no further primary or creep settlement, however, there would still be settlement due to the rock fill, in the amount of about 20 mm.

At this site, the west approach embankment will not be stable with a 2 m surcharge and therefore temporary toe berms would be required and would need to be approximately 3 m high and 10 m wide. Upon completion of the surcharge period, the toe berm and surcharge material may be re used on other parts of the site. A monitoring program would be required during the surcharge period.

6.5.3.3 *Lightweight Fill*

Another alternative for reducing the magnitude of long-term settlement at the west approach due to the presence of soft, compressible foundation soils is to use lightweight fill, such as expanded polystyrene (EPS), for embankment construction. The use of lightweight fill reduces the load applied to the foundation soils due to the low density of the fill materials. This in turn reduces the magnitude of post-construction settlement. The major advantage of using lightweight fill is that there will be no delay in the construction schedule. The major disadvantage is that the cost of EPS is typically an order of magnitude higher than other fill types.

At this site, although EPS would only be required at the west approach due to the presence of the silty clay to clay deposit and not at the east approach where these compressible soils are not present, EPS would still be required behind the east abutment to equalize the lateral loading on either side of the bridge.

6.5.3.4 *Full Sub-Excavation*

Taking into consideration the depth to the bottom of the cohesive deposit in the west abutment/approach area (i.e., up to about 13 m below the existing ground surface), full sub-excavation of the cohesive deposit is not considered the most practical settlement mitigation alternative for this area. Although there would no longer be consolidation settlement, the long-term settlement of about 250 mm from the 13 m of non-compacted rock fill below groundwater level, would need to be mitigated by preloading. In any case, a minimum of 6 months of



preloading would be required at both the east and west approaches to mitigate rock fill settlement, and likely even longer at the west approach due to the large magnitude of long-term settlement associated with the below ground rock fill.

6.6 Subgrade Preparation and Embankment Construction

The following sections discuss general aspects of subgrade preparation and embankment construction for the new Highway 522 approach embankments on both sides of the Highway 522 Underpass structure. These include: removal of surficial and near surface cohesive materials; excavation and replacement of very loose or disturbed cohesionless deposits, where applicable; groundwater control, where required; embankment fill placement; backfilling of sub-excavated areas; and the construction of the granular pad to facilitate pile driving at the abutments. Discussion on the instrumentation and monitoring program required for preloading is also provided.

6.6.1 Excavation and Removal of Organics

Based on the information from the boreholes advanced during the field investigation, the thickness of organic deposit and near surface cohesive deposit is up to about 0.8 m. The areas requiring removal of these near surface unsuitable soil deposits do not include the stripping or removal associated with the Highway 69 cut which is addressed in a separate report. After clearing and grubbing and prior to the placement of any fill for the new construction, these deposits within embankment areas should be sub-excavated from the plan limits of the proposed approach embankments, including toe berms, if applicable. The organic materials should be removed using construction procedures in accordance with OPSS 209 (Embankments Over Swamps and Compressible Soils). The excavation limits should be consistent with OPSD 203.010 (Embankments Over Swamp, New Construction) modified to remove the restrictions on the height of the embankment and the depth of excavation (i.e., Note A).

The native soils at this site may be classified as Type 4 soil. All excavations must be carried out in accordance with the latest edition of the Occupational Health and Safety Act (OHSA 2006) and Regulation for Construction Projects and good construction practices, which, through/into Type 4 soil, should have walls sloped from the bottom of the excavation no steeper than 3H:1V.

6.6.2 Groundwater and Surface Water Control

Excavation within the plan limits of the proposed works will be required to remove organic and/or near surface cohesive deposits prior to embankment fill placement, which, based on the available borehole information, likely will not extend to the groundwater table. If perched groundwater or the groundwater table is encountered, flow into the excavations could occur due to the presence of highly permeable subsoils. Dewatering is not required for the excavation and backfilling in the approach areas as rock fill will be used for embankment construction below grade (or Granular 'B' Type II will be used as backfill in the immediate abutment area). Surface water should be directed away from the excavations at all times.



6.6.3 Embankment Fill Placement

Placement of all rock fill material above the water table for construction of new approach embankments should be carried out in accordance with the requirements as outlined in SP 206S03 (Rock Excavation, Grading, Rock Embankment). As noted in the Special Provision, the rock should not be dumped in final position, but should be deposited on and pushed forward over the end of the layer being constructed. Voids and bridging should be minimized by blading, dozing and 'chinking' the rock to form a dense, compacted mass. Side slopes for rock fill embankments should be no steeper than 1.25H:1V for the highway embankments.

For embankments constructed using rock fill, the incorporation of 2 m wide benches (or successive berms) into the uniform side slope profile is required wherever the embankment will exceed a height of 10 m such that the uninterrupted rock fill slope does not exceed a height of 10 m as per OPSD 202.010 (Slope Flattening). Given that the height of the embankments is generally less than 7.5 m, benches will not be required. At the front slopes, the embankments will be about 10.5 m high due to the cut required for the proposed Highway 69 ditches, however, these slopes will, for the most part, be interrupted by the bridge abutments, with the exception of the small portion immediately north and south of the abutments and therefore, mid-height berms are not required.

6.6.4 Backfilling

It is recommended that rock fill be used for replacement of the sub-excavated material except as indicated in Section 6.6.2 in the abutment area. The rock fill is anticipated to be end dumped as the excavation advances and, in the event that the silty clay to clay deposit is sub-excavated, the rock fill would extend below the water table. Settlement of the overall rock fill embankment, including the portion underwater, has been estimated accordingly in Section 6.5.2.5.

6.6.5 Granular Pad

At the abutments, the compacted granular pad required for installation of the piles should be constructed such that it extends to the base of the sub-excavation and is integrated into the rock fill embankment. We recommend the pad be constructed using OPSS.PROV 1010 (Aggregates) Granular 'A' or Granular 'B' Type II with maximum particle size of 75 mm. The granular pad/core should extend at least 1 m beyond the plan limits of the pile cap and be sloped no steeper than 1H:1V. The granular pad should be constructed in accordance with SP 206S03 (Earth Excavation, Grading) and compacted in accordance with SP 105S21 (Water Requirements and Quality Control for Compaction - Method B). Given the requirement for sub-excavation of the organic and cohesive soils at the abutments, the granular pad is expected to be about 1.7 m and 0.6 m thick at the west and east abutments, respectively, based on the proposed underside of pile cap noted in Section 6.2.1.1, and sloped downwards to meet the base of the Highway 69 ditches. If bedrock excavation (trenching or socketing) is required for pile installation at the east abutment, the granular pad is expected to be at least 5 m thick.

6.6.6 Blasting for Rock Excavations

Bedrock excavation may be required to allow for the installation of 5 m long piles for integral abutment design at the east abutment. For bedrock excavation, the overall slope of the cut face may be formed vertical or at a steep



slope (i.e., 0.25H:1V). All bedrock excavation within and near the foundations areas should be carried out using controlled blasting techniques in order to minimize shattering and over-break. The use of line drilling, pre-shearing or cushion blasting is recommended in order to provide a neat excavation line. Good blasting practices will be critical to maintaining the excavation lines in the area of the structure foundations and all blasting operations should be carried out consistent with OPSS 120 (Use of Explosives).

6.6.7 Instrumentation and Monitoring

For areas where the preloading or surcharging options are adopted, the magnitude and time rate of settlement as well as dissipation of pore pressures during and after construction of the approach embankments should be assessed with monitoring instrumentation. Such monitoring should consist of installing settlement pins/stakes (SSs), settlement plates (SPs) and vibrating wire piezometers (VWPs) below the embankment and taking regular measurements/readings at given intervals of time during and after construction of the embankment for the duration of the preloading/surcharging period. In addition, standpipe piezometers (SPPs) may be required and are usually installed to provide background pore pressure readings for comparison of the data from the vibrating wire piezometers.

The extent of instrumentation and the frequency of monitoring required will depend on the foundation treatment alternative chosen for a given site and the height of the proposed embankment fill. Specifications for the type, number and layout of the instrumentation, together with the supply, installation, protection and monitoring should be included as a Non-Standard Special Provision in the Contract.

6.6.8 Temporary Protection System

A temporary excavation protection system, if required, should be designed and constructed to Performance Level 3 in accordance with SP 539S02 (Protection System).

7.0 CLOSURE

This report was prepared by Mr. Adam Core, E.I.T. and David Muldowney, P.Eng., and the technical aspects were reviewed by Ms. Sarah Coyne, P.Eng., a senior geotechnical engineer and Associate with Golder. Mr. Jorge M. A. Costa, P.Eng., Golder's Designated MTO Contact for this project and a Principal with Golder, conducted an independent quality control review of the report.

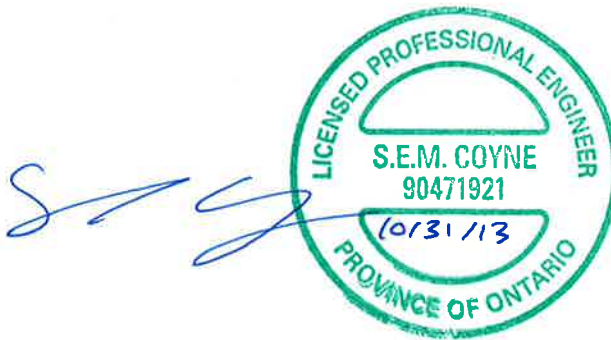


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 - ASTM D1587 Standard Practice for Thin-Walled Tube Sampling of Soils for Geotechnical Purposes
 - ASTM D2573 Standard Test Method for Field Vane Shear Test in Cohesive Soil
- Commercial Software
- LPile (Version 5.0) by Ensoft Inc.



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Settle^{3D} (Version 2.016) by Rocscience Inc.

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Embankment Settlement Criteria for Design, Final Draft, March 2, 2010

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SP 105S21 Amendment to OPSS 501, November 2010 – Water Requirements and Quality Control for Compaction – Method B.

SP 206S03 Rock Excavation, Grading; Rock Embankment

SP 539S02 Protection System – Item No. – Amendment to OPSS 539, November 2009.

Ontario Provincial Standard Drawings

OPSD 202.010 Slope Flattening Using Surplus Excavated Material on Earth or Rock Embankment

OPSD 203.010 Embankments Over Swamp, New Construction

OPSD 3090.101 Foundation, Frost Penetration Depths for Southern Ontario

OPSD 3121.150 Walls Retaining, Backfill, Minimum Granular Requirement

Ontario Provincial Standard Specifications

OPSS 120 General Specification for Use of Explosives

OPSS 209 Construction Specification for Embankments over Swamps and Compressible Soils

OPSS 501 Construction Specification for Compacting

OPSS.PROV 1010 Material Specification for Aggregates – Base, Subbase, Select Subgrade and Backfill Material

Ontario Water Resources Act

Ontario Regulation 903/90 Wells; O.Reg 468/10 Amendment to Ontario Regulation 903



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Table 1: Evaluation of Foundation Alternatives - Highway 522 Underpass

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 groundwater/flowing soil conditions compared to caissons. ■ Suitable for integral abutment design. 	<ul style="list-style-type: none"> ■ Requires excavation for pile cap construction. 	<ul style="list-style-type: none"> ■ Lower cost than caissons but higher than spread footings. ■ $[(9 \times 19.5\text{m}) + (8 \times 11\text{m}) + (9 \times 5.5\text{m})] @ \\$200/\text{m} = \sim \\$85,000$ 	<ul style="list-style-type: none"> ■ Low risk of not achieving design resistance at design pile tip elevation.
Caissons	2	<ul style="list-style-type: none"> ■ Higher axial resistances compared to steel H-piles. ■ Pile cap can be constructed at the bridge deck level, therefore excavation is not required. 	<ul style="list-style-type: none"> ■ Not suitable for integral abutment design. ■ Potential problems associated with penetrating cobbles and boulder and seating/socketing into strong sloping bedrock. 	<ul style="list-style-type: none"> ■ Much higher cost than steel H-piles. ■ $[(4 \times 19.5\text{m}) + (2 \times 11\text{m}) + (4 \times 5.5\text{m}) + (10 \times 2.5\text{m sockets})] @ \\$900/\text{m} = \sim \\$130,000$ 	<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 groundwater during caisson installation.
Shallow Spread Footings on granular pad	NF	<ul style="list-style-type: none"> ■ Conventional construction. ■ No issues associated with groundwater conditions. 	<ul style="list-style-type: none"> ■ Significantly lower geotechnical axial resistance. ■ Will be subjected to consolidation settlements of the cohesive deposit from embankment and bridge loads ■ Not suitable for integral abutment design. 	<ul style="list-style-type: none"> ■ Typically lower cost than deep foundations. ■ $[(3 \times 12\text{m} \times 1\text{m}) \times 3 \text{ elements}] @ \\$600/\text{m} = \sim \\$65,000$ ■ 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). ■ Potential for differential settlement between foundation elements.



FOUNDATION REPORT HIGHWAY 522 INTERCHANGE UNDERPASS STRUCTURE - GWP 5347-08-00; WP 5151-08-01

Table 2: Evaluation of Settlement Mitigation Options

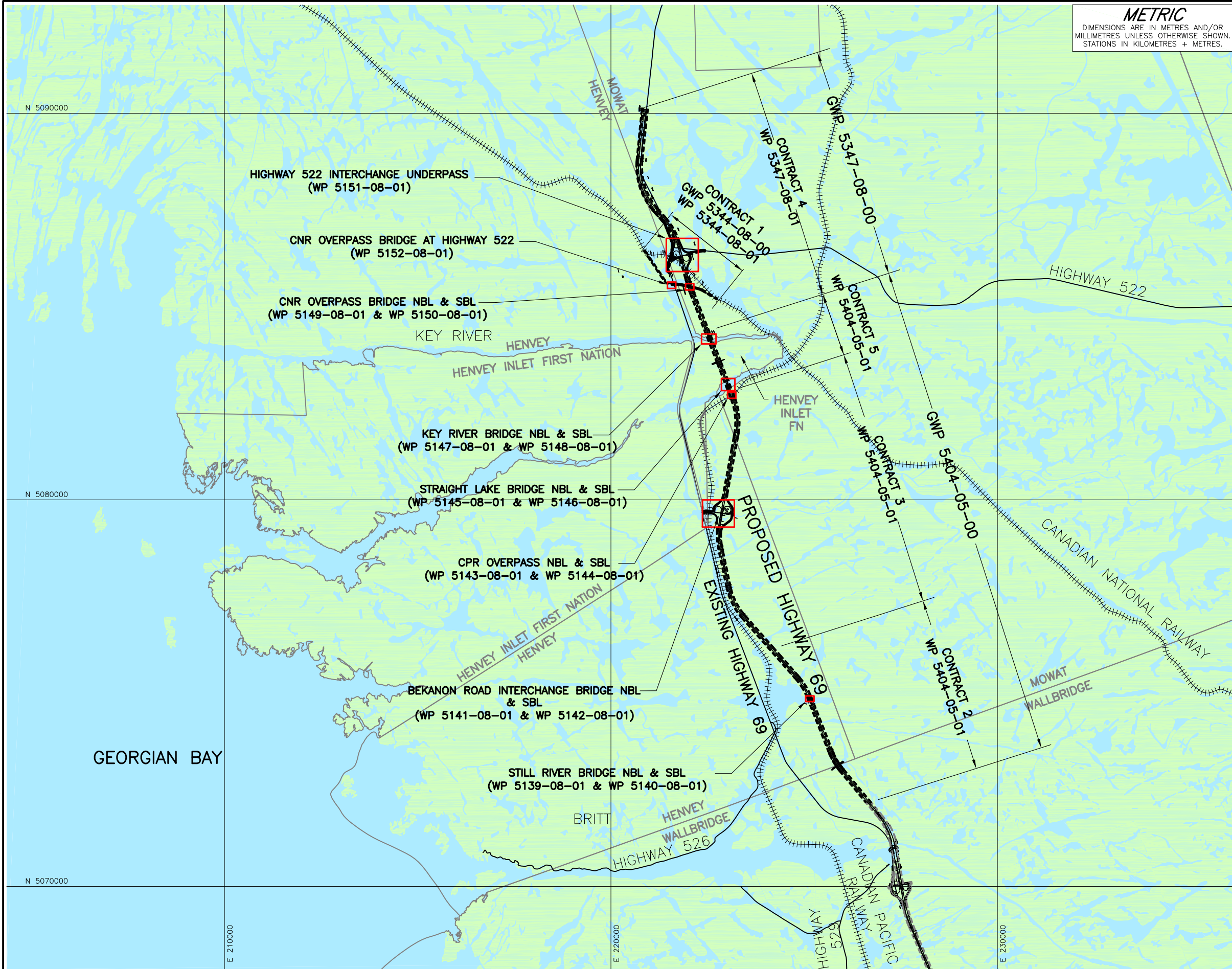
Mitigation Option	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Preloading	1	<ul style="list-style-type: none"> Standard construction operation. Reduces post-construction settlement of cohesive soils and rock fill. Does not require toe berm for overall embankment stability. 	<ul style="list-style-type: none"> Delay in construction schedule (6 months for east approach and 12 months for west approach) to allow for sufficient settlement to occur to meet post-construction settlement criteria. Instrumentation and monitoring program required to assess end of preload. Re-grading is required prior to final pavement structure construction to account for settlement associated with preloading. 	<ul style="list-style-type: none"> Schedule impacts may increase overall project costs. Additional cost for instrumentation and associated monitoring program. 	<ul style="list-style-type: none"> Subject to the monitoring data collected during the preloading period, additional preload time may be required. Reduced potential of unexpected post-construction settlements (i.e., creep).
Surcharging with Toe Berms	2	<ul style="list-style-type: none"> Standard construction operation. Reduces post-construction secondary settlement due to creep. Reduces time to reach end of primary settlement. 	<ul style="list-style-type: none"> Delay in construction schedule to allow for sufficient settlement to occur to meet post-construction settlement criteria, but typically less than for preloading alone. Increased handling of surcharge fill upon completion of surcharge period. Toe berms are required to maintain stability of surcharged embankment. Additional rock fill required for surcharge and toe berm construction. Instrumentation and monitoring program required to assess end of surcharge period. Re-grading and disposal of excess material is required prior to final pavement structure construction. 	<ul style="list-style-type: none"> Additional cost associated with construction of embankment surcharge and toe berms. Schedule impacts may increase overall project costs. Additional cost for instrumentation and associated monitoring program. 	<ul style="list-style-type: none"> Toe berms required for maintaining stability of surcharged embankment on weak/soft foundation soils. Subject to the monitoring data collected during the surcharge period, additional surcharge time may be required. Significantly reduced potential of unexpected post-construction settlements (i.e., creep).



FOUNDATION REPORT HIGHWAY 522 INTERCHANGE UNDERPASS STRUCTURE - GWP 5347-08-00; WP 5151-08-01

Table 2: Evaluation of Settlement Mitigation Options

Mitigation Option	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Lightweight Fill	3	<ul style="list-style-type: none"> ■ Reduced load on subsoils resulting in lower post-construction settlement of foundation soils and rock fill. ■ No delay in schedule. ■ Not additional rock fill required for surcharge or toe berm construction. 	<ul style="list-style-type: none"> ■ Increased handling of embankment fill required to install EPS and granular fill cover. 	<ul style="list-style-type: none"> ■ Relative cost of EPS fill is about an order of magnitude higher than fill required for other options. 	<ul style="list-style-type: none"> ■ Significantly reduced potential of unexpected post-construction settlements (i.e., creep).
Full Sub-Excavation of Compressible Deposits (to up to about 13 m below ground surface)	Not Feasible	<ul style="list-style-type: none"> ■ .Would remove component of settlement associated with the cohesive deposit. 	<ul style="list-style-type: none"> ■ Generation of very large volume of excess excavation spoil. ■ Very large quantity of rock fill required. ■ Substantial increase in post-construction settlement due to rock fill end-dumped below the water level. Delay in construction associated to allow for sufficient settlement of rock fill to occur to meet post-construction settlement (12 months or greater). ■ Specialized equipment and additional effort required for deep sub-excavation and replacement. ■ Likely require additional right-of-way to accommodate deep sub-excavation. 	<ul style="list-style-type: none"> ■ Substantial additional costs associated with sub-excavation (specialized drag line equipment required), disposal and replacement of weak/soft compressible deposits. ■ Additional cost of rock fill for backfilling excavation. 	<ul style="list-style-type: none"> ■ High potential of unexpected post-construction settlements due to rock fill settlement. ■ Potential increased size of excavation due to sloughing of excavation walls from groundwater inflows.



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

CONT No.
GWP No. 5347-08-00

HIGHWAY 69
SITE LOCATION PLAN

SHEET



Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



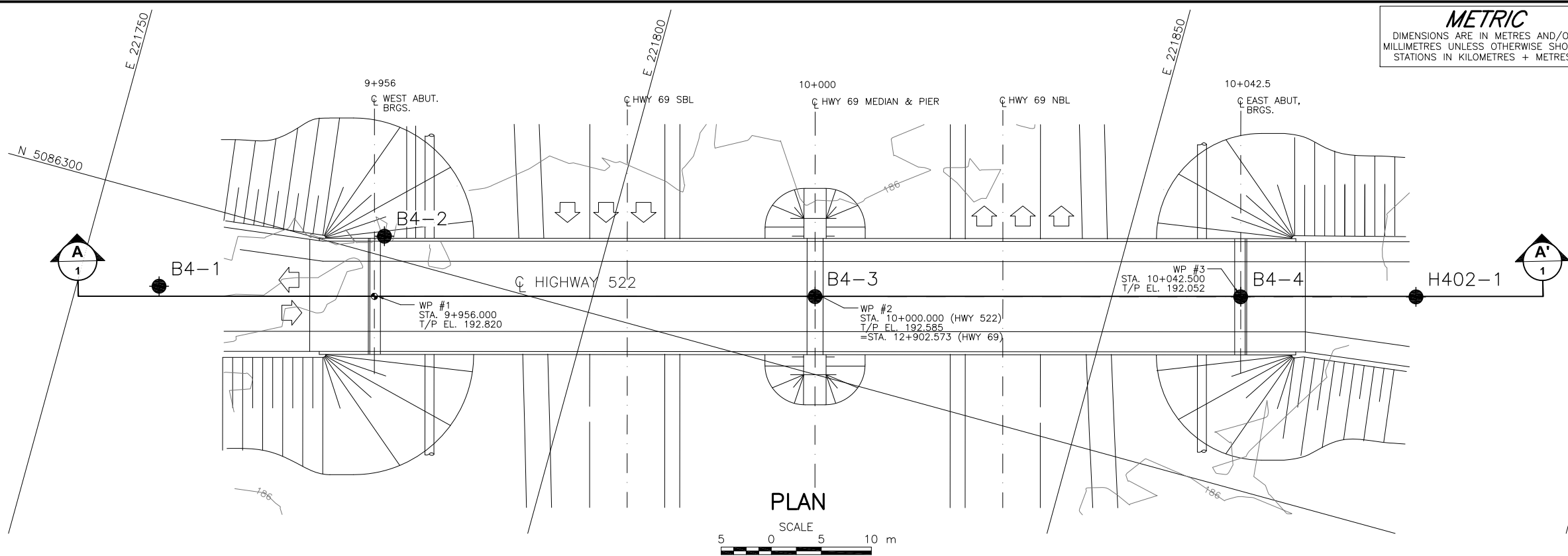
KEY PLAN
NOT TO SCALE



REFERENCE

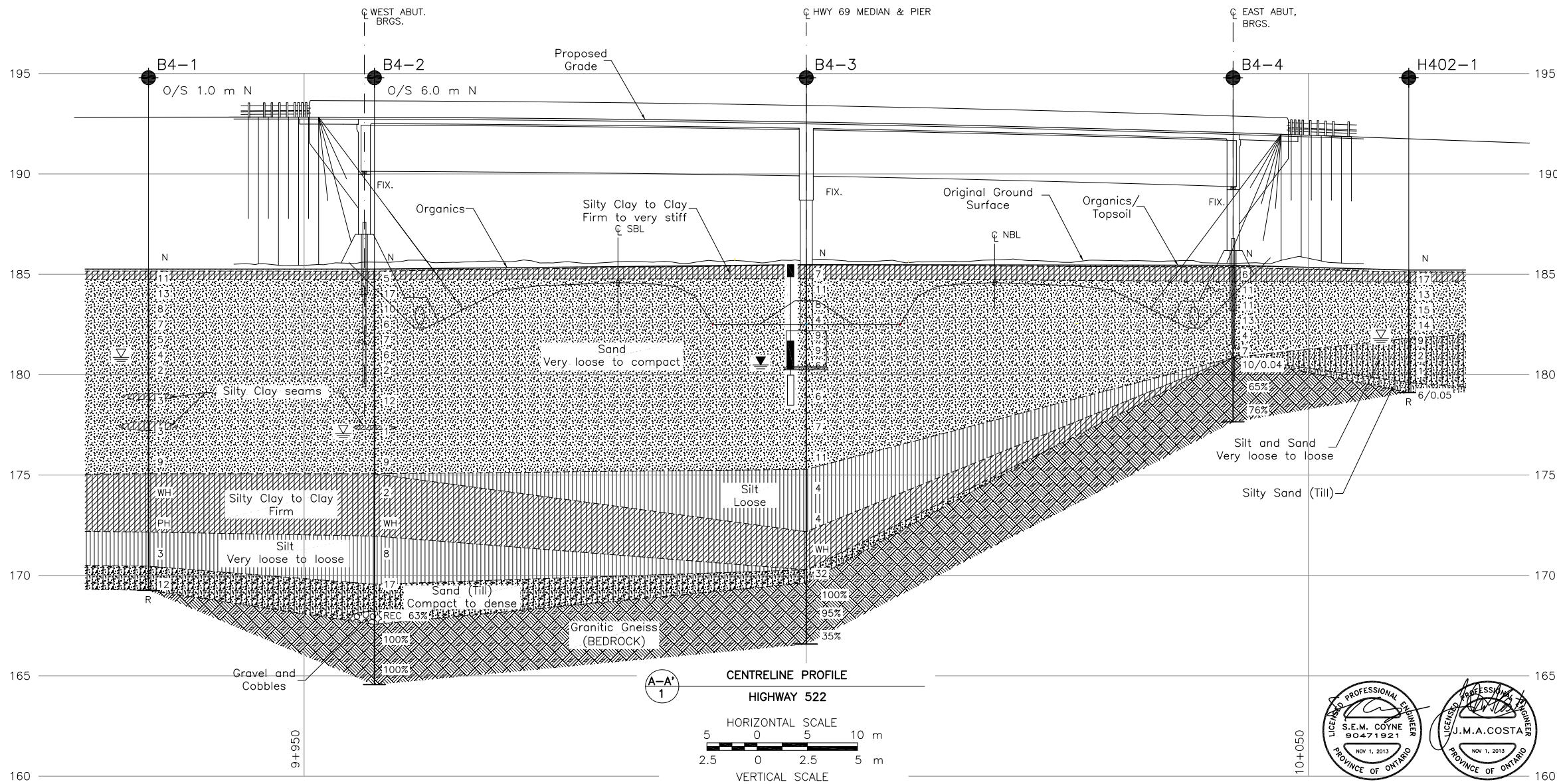
Base Data – MNR NRVIS, obtained 2004, CANMAP v2008
Produced by Golder Associates Ltd under licence from
Ontario Ministry of Natural Resources, © Queens Printer 2008
Datum : NAD 83 Projection : MTM Zone 10

NO.	DATE	BY	REVISION
Geocres No.			
HWY. 69	PROJECT NO. 09-1111-6014		DIST.
SUBM'D. TVA	CHKD. TVA	DATE: NOV 2013	SITE:
DRAWN: JFC	CHKD. CN	APPD. JMAC	DWG. 1



PLAN

SCALE
5 0 5 10 m



CENTRELINE PROFILE

HIGHWAY 522

HORIZONTAL SCALE
5 0 5 10 m

VERTICAL SCALE
2.5 0 2.5 5 m

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

CONT No.
WP No. 5151-08-01

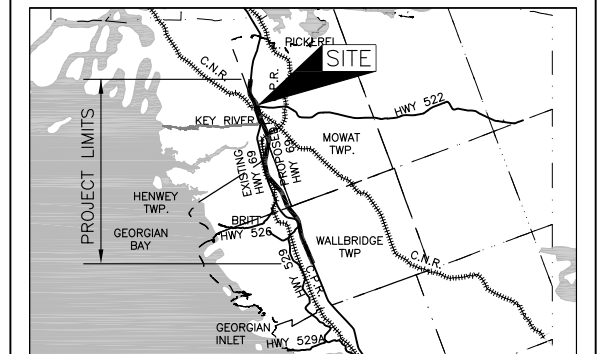
HIGHWAY 69/522 INTERCHANGE
HIGHWAY 522 UNDERPASS STRUCTURE
BOREHOLE LOCATIONS AND SOIL STRATA



SHEET



Golder Associates Ltd.
SUDBURY, ONTARIO, CANADA



KEY PLAN

SCALE

10 0 10 20 km

LEGEND

- Borehole
- Seal
- Piezometer
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- REC Recovery (%)
- R Refusal
- 100% Rock Quality Designation (RQD)
- WL in piezometer, measured on AUG 29, 2012
- WL upon completion of drilling

BOREHOLE CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
B4-1	185.3	5086291.2	221758.0
B4-2	185.3	5086301.9	221778.4
B4-3	185.5	5086307.5	221821.4
B4-4	185.5	5086318.7	221862.4
H402-1	185.2	5086323.3	221879.3

NOTES

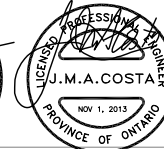
This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contracts Documents.

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

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

REFERENCE

Base plans provided in digital format by URS, drawing file GA HWY 522 19122012.dwg received JAN 21, 2013, Keyplan received APR 16, 2010.

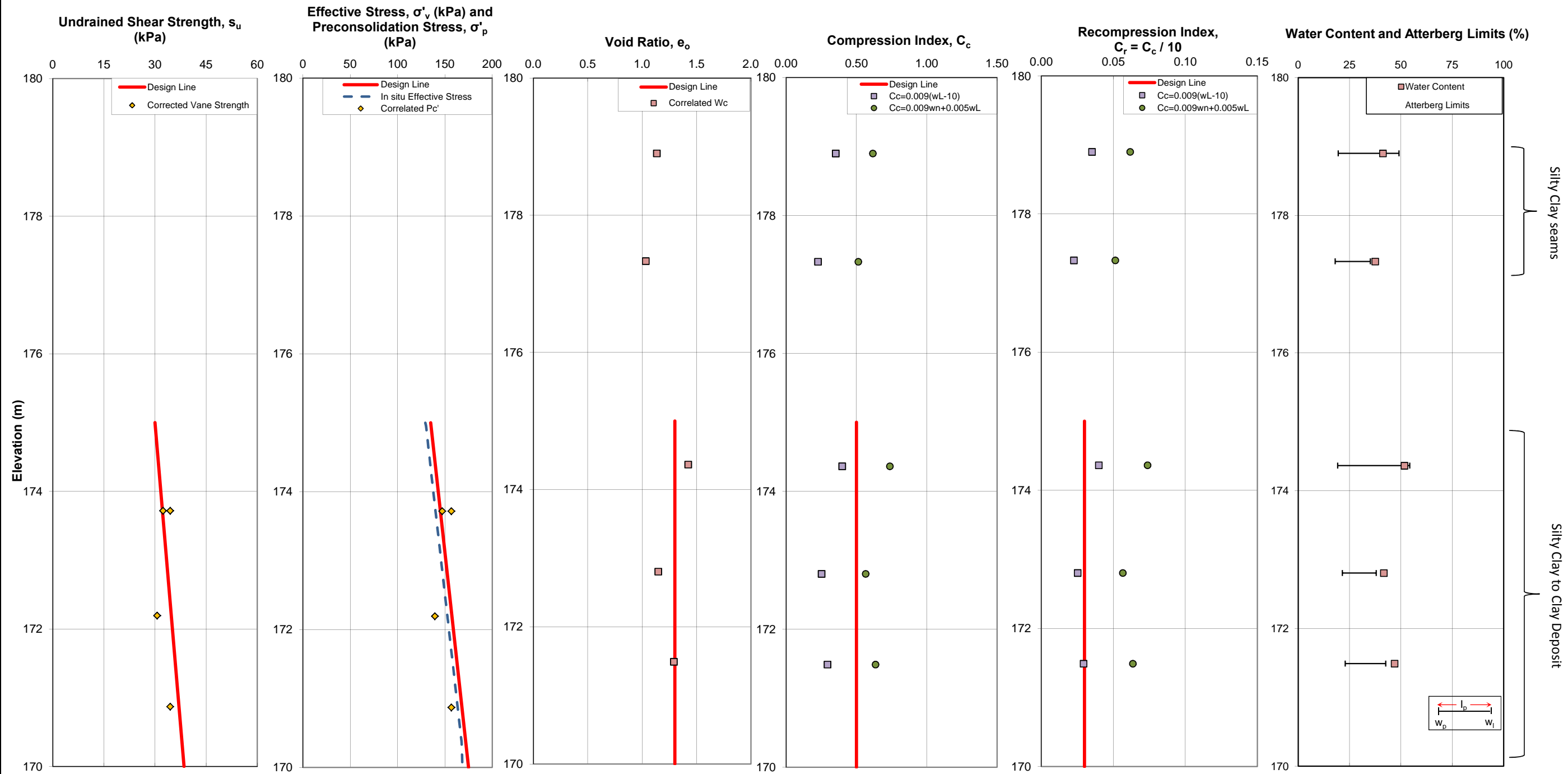


NO.	DATE	BY	REVISION
Geocres No. 41H-124			
HWY. 69/522		PROJECT NO. 09-1111-6014	DIST.
SUBM'D. AC	CHKD.	DATE: NOV 2013	SITE: 44-464
DRAWN: TB	CHKD. SEMC	APPD. JMAC	DWG. 2

https://capws.golder.com/sites/0911116014highway699FourLaning/Contract 4/Analysis/09-1111-6014-C4-Parameters and Design Lines-Swamp and HighFill areas - Revised 13Oct30 Final.xlsx[S401 Plots - Final

SUMMARY PLOT OF ENGINEERING PARAMETERS FOR
SILTY CLAY TO CLAY DEPOSIT
Highway 522 Underpass

FIGURE 1



Golder Associates

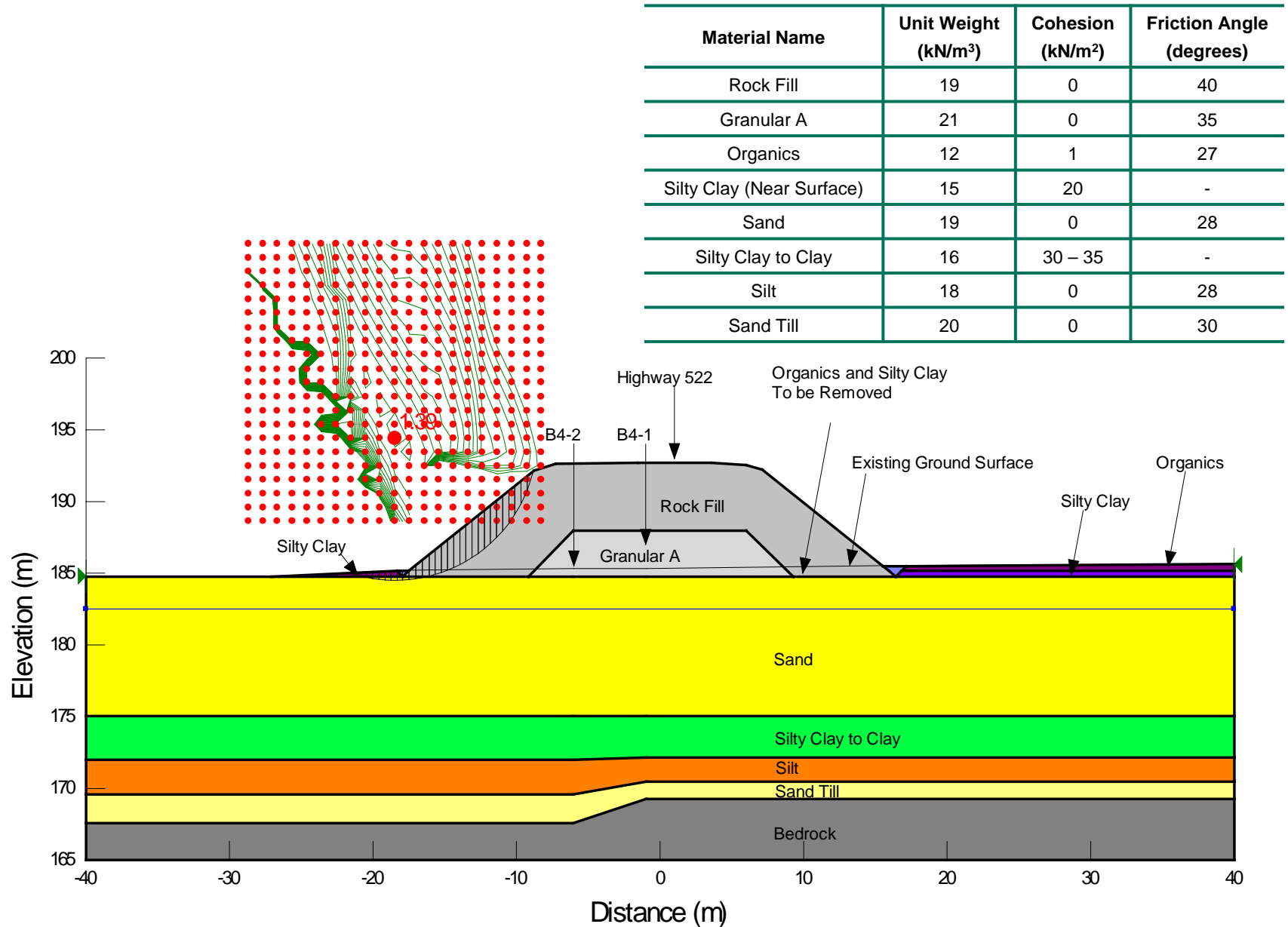
Date: November 2013
Project No: 09-1111-6014

Prepared By: DAM
Checked By: SEMC



Highway 522 Underpass Stability Analysis (West Approach Side Slope)

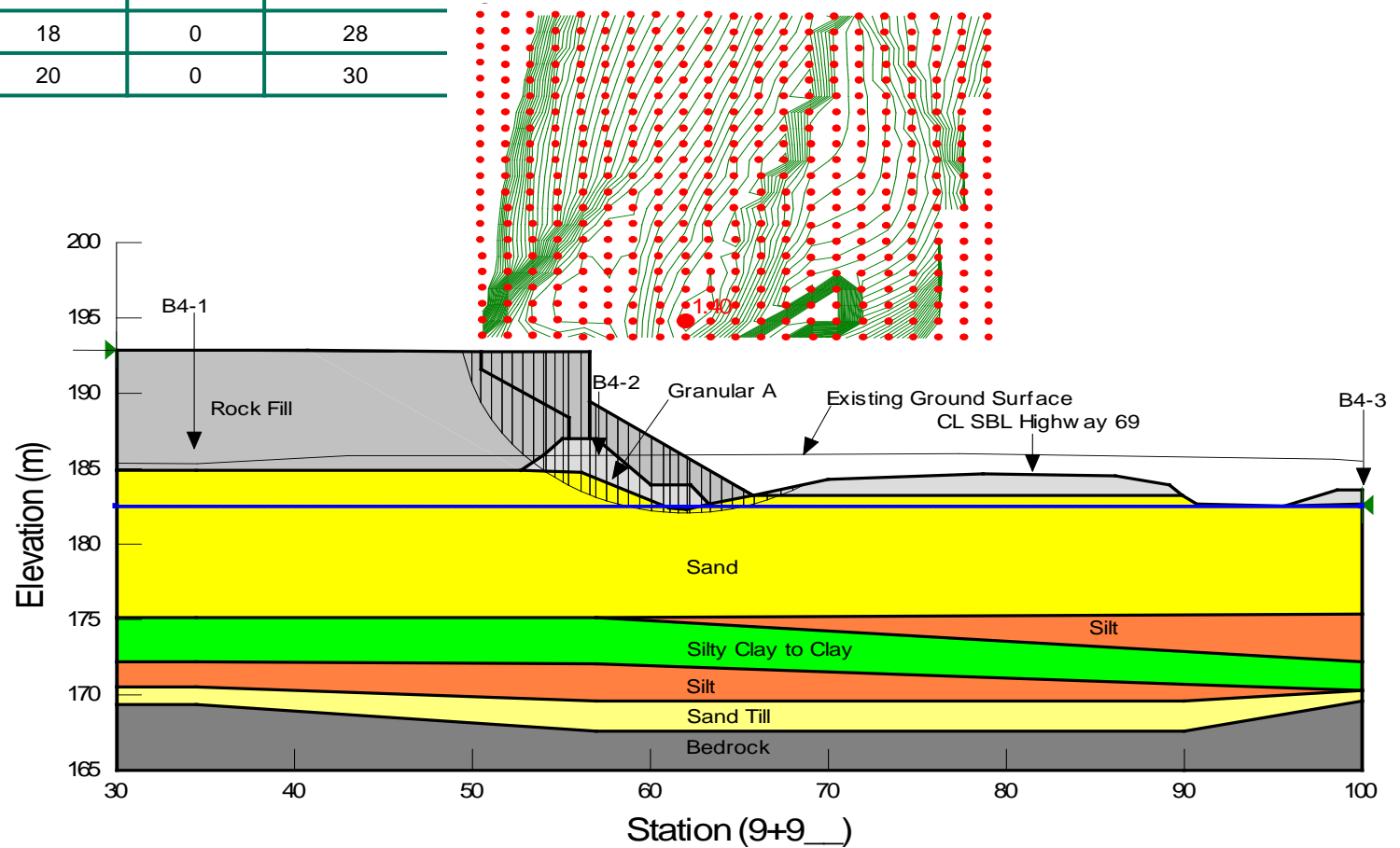
Figure 2



Highway 522 Underpass Stability Analysis (West Abutment Front Slope)

Figure 3

Material Name	Unit Weight (kN/m ³)	Cohesion (kN/m ²)	Friction Angle (degrees)
Rock Fill	19	0	40
Granular A	21	0	35
Sand	19	0	28
Silty Clay to Clay	16	30 – 38	-
Silt	18	0	28
Sand Till	20	0	30





APPENDIX A

Record of Boreholes, Drillholes and Figures



LIST OF SYMBOLS

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

I. GENERAL

π	3.1416
$\ln x$,	natural logarithm of x
\log_{10}	x or log x, logarithm of x to base 10
g	acceleration due to gravity
t	time
FoS	factor of safety

II. STRESS AND STRAIN

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

III. SOIL PROPERTIES

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

(a) Index Properties (continued)

w	water content
w_l or LL	liquid limit
w_p or PL	plastic limit
I_p or PI	plasticity index = $(w_l - w_p)$
w_s	shrinkage limit
I_L	liquidity index = $(w - w_p) / I_p$
I_C	consistency index = $(w_l - w) / I_p$
e_{max}	void ratio in loosest state
e_{min}	void ratio in densest state
I_D	density index = $(e_{max} - e) / (e_{max} - e_{min})$ (formerly relative density)

(b) Hydraulic Properties

h	hydraulic head or potential
q	rate of flow
v	velocity of flow
i	hydraulic gradient
k	hydraulic conductivity (coefficient of permeability)
j	seepage force per unit volume

(c) Consolidation (one-dimensional)

C_c	compression index (normally consolidated range)
C_r	recompression index (over-consolidated range)
C_s	swelling index
C_α	secondary compression index
m_v	coefficient of volume change
C_v	coefficient of consolidation (vertical direction)
C_h	coefficient of consolidation (horizontal direction)
T_v	time factor (vertical direction)
U	degree of consolidation
σ'_p	pre-consolidation stress
OCR	over-consolidation ratio = σ'_p / σ'_{vo}

(d) Shear Strength

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

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

Notes: 1
2

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

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



LIST OF ABBREVIATIONS

The abbreviations commonly employed on Records of Boreholes, on figures and in the text of the report are as follows:

I. SAMPLE TYPE

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

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

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

Dynamic Cone Penetration Resistance; N_d :

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

PH: Sampler advanced by hydraulic pressure

PM: Sampler advanced by manual pressure

WH: Sampler advanced by static weight of hammer

WR: Sampler advanced by weight of sampler and rod

Piezo-Cone Penetration Test (CPT)

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

III. SOIL DESCRIPTION

(a) Non-Cohesive (Cohesionless) Soils

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

(b) Cohesive Soils Consistency

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

IV. SOIL TESTS

w	water content
w_p	plastic limit
w_l	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test ¹
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement ¹
D_R	relative density (specific gravity, G_s)
DS	direct shear test
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
SO_4	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V	field vane (LV-laboratory vane test)
γ	unit weight

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

V. MINOR SOIL CONSTITUENTS

Per cent by Weight	Modifier	Example
0 to 5	Trace	Trace sand
5 to 12	Trace to Some (or Little)	Trace to some sand
12 to 20	Some	Some sand
20 to 30	(ey) or (y)	Sandy
over 30	And (non-cohesive (cohesionless)) or With (cohesive)	Sand and Gravel Silty Clay with sand / Clayey Silt with sand



LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

WEATHERINGS STATE

Fresh: no visible sign of weathering

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

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

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

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

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

BEDDING THICKNESS

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

JOINT OR FOLIATION SPACING

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

GRAIN SIZE

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

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

CORE CONDITION

Total Core Recovery (TCR)

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

Solid Core Recovery (SCR)

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

Rock Quality Designation (RQD)

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

DISCONTINUITY DATA

Fracture Index

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

Dip with Respect to Core Axis

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

Description and Notes

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

Abbreviations

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

[illegible]

Continued Next Page

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

PROJECT <u>09-1111-6014</u>				RECORD OF BOREHOLE No B4-1				2 OF 2 METRIC									
W.P. <u>5151-08-01</u>		LOCATION <u>N 5086291.2; E 221758.0</u>				ORIGINATED BY <u>AC</u>											
DIST <u></u> HWY <u>522 UNDERPASS</u>		BOREHOLE TYPE <u>108 mm I.D. Continuous Flight Hollow Stem Augers, NW casing and Wash Boring</u>				COMPILED BY <u>AC</u>											
DATUM <u>GEODETIC</u>		DATE <u>August 16, 2012</u>				CHECKED BY <u>SEMC</u>											
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
	--- CONTINUED FROM PREVIOUS PAGE ---							20	40	60	80	100					
169.3	SAND, some silt (TILL) Compact Grey Wet	14	SS	12		170											
16.0	END OF BOREHOLE REFUSAL TO FURTHER CASING ADVANCEMENT Note: 1. Water level at a depth of 4.4 m below ground surface (Elev. 180.9 m) upon completion of drilling.																

SUD-MTO 001 09-1111-6014_BRIDGE.GPJ GAL-MISS.GDT 31/10/13 DATA INPUT:

PROJECT 09-1111-6014			RECORD OF BOREHOLE No B4-2			1 OF 2 METRIC														
W.P. 5151-08-01			LOCATION N 5086301.9; E 221778.4			ORIGINATED BY AC														
DIST _____ HWY 522 UNDERPASS			BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers, NW casing and NQ Coring			COMPILED BY AC														
DATUM GEODETIC			DATE August 15 and 16, 2012			CHECKED BY SEMC														
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)					
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					WATER CONTENT (%) W _p — W — W _L			γ	GR	SA	SI	CL
185.3	GROUND SURFACE							20 40 60 80 100												
0.0	ORGANICS (Pine needles)		1	SS	5		185													
184.8	SILTY CLAY, trace sand, trace organics, oxidized		2	SS	17		184													
0.5	Firm Brown Dry to moist		3	SS	10		183													
	SAND, trace to some silt		4	SS	6		182													
	Very loose to compact		5	SS	7		181													
	Brown		6	SS	6		180													
	Wet		7	SS	2		179													
			8	SS	12		178													
	Grey below 6.1 m depth.		9	SS	1		177													
	An approximately 200 mm silty clay seam encountered at 7.8 m depth.		10	SS	9		176													
			11	SS	2		175													
	SILTY CLAY, trace sand		12	SS	WH		174													
	Firm						173													
	Grey						172													
	Wet						171													
	An approximately 250 mm silty sand seam encountered at 11.0 m depth.		13	SS	8															
	SILT, trace to some sand, trace to some clay																			
	Loose to compact																			
	Grey																			
	Wet																			

Continued Next Page

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

SUD-MTO 001 09-1111-6014-BRIDGE.GPJ GAL-MISS.GDT 31/10/13 DATA INPUT:

PROJECT 09-1111-6014				RECORD OF BOREHOLE No B4-2				2 OF 2 METRIC									
W.P. 5151-08-01		LOCATION N 5086301.9; E 221778.4				ORIGINATED BY AC											
DIST _____ HWY 522 UNDERPASS		BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers, NW casing and NQ Coring				COMPILED BY AC											
DATUM GEODETIC		DATE August 15 and 16, 2012				CHECKED BY SEMC											
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					
169.6			14	SS	17		170										
15.7	SAND, some silt, trace to some gravel (TILL) Grey Wet						169										
	Gravel and cobbles encountered between 16.5 m and 17.7 m depth as follows:																
	Depth (m) Thickness (mm)																
	16.5 250		1	RC	REC 63%		168										
167.6	16.8 100																
	17.1 125																
17.7	17.5 100																
	GRANITIC GNEISS (BEDROCK)																
	Bedrock cored from 17.7 m depth to 20.7 m depth.		2	RC	REC 100%		167										RQD = 100%
	For coring details see Record of Drillhole B4-2.																
							166										RQD = 100%
			3	RC	REC 100%		165										
164.6	END OF BOREHOLE																
20.7	Note: 1. Water level at a depth of 8.2 m below ground surface (Elev. 177.1 m) inside casing upon completion of drilling.																

SUD-MTO 001 09-1111-6014_BRIDGE.GPJ GAL-MISS.GDT 31/10/13 DATA INPUT:

PROJECT: 09-1111-6014

RECORD OF DRILLHOLE: B4-2

SHEET 1 OF 1

LOCATION: N 5086301.9; E 221778.4

DRILLING DATE: August 15 and 16, 2012

DATUM: GEODETIC

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 850

DRILLING CONTRACTOR: Landcore Drilling Inc.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV.		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																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																
				DEPTH (m)	FLUSH									RECOVERY			R.Q.D. %	FRACT. INDEX METRES	DISCONTINUITY DATA					HYDRAULIC CONDUCTIVITY					Diameter Point Load Index (MPa)	RMC -Q AVG.																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																															
														TOTAL CORE %	SOLID CORE %	B Angle			DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION	Ur	Ja	Jn	10°	10°	10°	2	4			8																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																														
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DEPTH SCALE

1 : 50



LOGGED: AC

CHECKED: SEMC

SUD-RCK 09-1111-6014 BRIDGE GPJ GAL-MISS GDT 31/10/13 DATA INPUT:

1 OF 2 **METRIC**

ORIGINATED BY AC

COMPILED BY AC

CHECKED BY _____ SEMC

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

MSUD-MTO 001 09-1111-6014 BRIDGE.GPJ GAL-MISS.GDT 31/10/13 DATA INPUT:

PROJECT		RECORD OF BOREHOLE No B4-3				2 OF 2 METRIC											
W.P. 09-1111-6014		LOCATION N 5086307.5; E 221821.4				ORIGINATED BY AC											
DIST _____ HWY 522 UNDERPASS		BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers, NW casing and NQ Coring				COMPILED BY AC											
DATUM GEODETIC		DATE August 13 and 14, 2012				CHECKED BY SEMC											
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					
170.3																	
15.2	SAND, some silt, some gravel, trace clay (TILL)		14	SS	32		170										
169.7	Dense Wet																
15.8	GRANITE (BEDROCK)		1	RC	REC 100%		169										RQD = 100%
	Bedrock cored from 15.9 m depth to 18.9 m depth.																
	For Coring details see Record of Drillhole B4-3.		2	RC	REC 100%		168										RQD = 95%
			3	RC	REC 100%		167										RQD = 35%
166.6																	
18.9	END OF BOREHOLE																
	Note: 1. Water level at a depth of 4.9 m below ground surface (Elev. 180.6 m) upon completion of drilling. 2. Water level in piezometer at a depth of 4.9 m below ground surface (Elev. 180.6 m) on August 15, 2012 and at a depth of 5.0 m below ground surface (Elev. 180.5 m) on August 29, 2012.																

SUD-MTO 001 09-1111-6014_BRIDGE.GPJ GAL-MISS.GDT 31/10/13 DATA INPUT:

PROJECT: 09-1111-6014

RECORD OF DRILLHOLE: B4-3

SHEET 1 OF 1

LOCATION: N 5086307.5;E 221821.4

DRILLING DATE: August 14, 2012

DATUM: GEODETIC

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 850

DRILLING CONTRACTOR: Landcore Drilling Inc.

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												NOTES WATER LEVELS INSTRUMENTATION																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																												
							RECOVERY				R.Q.D. %				FRACT. INDEX METRES				DISCONTINUITY DATA								HYDRAULIC CONDUCTIVITY				Diametral Point Load Index (MPa)				RMC -Q AVG.																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																												
							TOTAL CORE %		SOLID CORE %		R.Q.D. %		FRACT. INDEX METRES		B Angle		DIP w.r.t. CORE AXIS		TYPE AND SURFACE DESCRIPTION		Jr		Ja		Jn		k, cm/s		10 ⁻⁶		10 ⁻⁵		10 ⁻⁴		10 ⁻³		10 ⁻²		10 ⁻¹		10 ⁰		10 ¹		10 ²		10 ³		10 ⁴		10 ⁵		10 ⁶		10 ⁷		10 ⁸		10 ⁹		10 ¹⁰		10 ¹¹		10 ¹²			10 ¹³		10 ¹⁴		10 ¹⁵		10 ¹⁶		10 ¹⁷		10 ¹⁸		10 ¹⁹		10 ²⁰		10 ²¹		10 ²²		10 ²³		10 ²⁴		10 ²⁵		10 ²⁶		10 ²⁷		10 ²⁸		10 ²⁹		10 ³⁰		10 ³¹		10 ³²		10 ³³		10 ³⁴		10 ³⁵		10 ³⁶		10 ³⁷		10 ³⁸		10 ³⁹		10 ⁴⁰		10 ⁴¹		10 ⁴²		10 ⁴³		10 ⁴⁴		10 ⁴⁵		10 ⁴⁶		10 ⁴⁷		10 ⁴⁸		10 ⁴⁹		10 ⁵⁰		10 ⁵¹		10 ⁵²		10 ⁵³		10 ⁵⁴		10 ⁵⁵		10 ⁵⁶		10 ⁵⁷		10 ⁵⁸		10 ⁵⁹		10 ⁶⁰		10 ⁶¹		10 ⁶²		10 ⁶³		10 ⁶⁴		10 ⁶⁵		10 ⁶⁶		10 ⁶⁷		10 ⁶⁸		10 ⁶⁹		10 ⁷⁰		10 ⁷¹		10 ⁷²		10 ⁷³		10 ⁷⁴		10 ⁷⁵		10 ⁷⁶		10 ⁷⁷		10 ⁷⁸		10 ⁷⁹		10 ⁸⁰		10 ⁸¹		10 ⁸²		10 ⁸³		10 ⁸⁴		10 ⁸⁵		10 ⁸⁶		10 ⁸⁷		10 ⁸⁸		10 ⁸⁹		10 ⁹⁰		10 ⁹¹		10 ⁹²		10 ⁹³		10 ⁹⁴		10 ⁹⁵		10 ⁹⁶		10 ⁹⁷		10 ⁹⁸		10 ⁹⁹		10 ¹⁰⁰		10 ¹⁰¹		10 ¹⁰²		10 ¹⁰³		10 ¹⁰⁴		10 ¹⁰⁵		10 ¹⁰⁶		10 ¹⁰⁷		10 ¹⁰⁸		10 ¹⁰⁹		10 ¹¹⁰		10 ¹¹¹		10 ¹¹²		10 ¹¹³		10 ¹¹⁴		10 ¹¹⁵		10 ¹¹⁶		10 ¹¹⁷		10 ¹¹⁸		10 ¹¹⁹		10 ¹²⁰		10 ¹²¹		10 ¹²²		10 ¹²³		10 ¹²⁴		10 ¹²⁵		10 ¹²⁶		10 ¹²⁷		10 ¹²⁸		10 ¹²⁹		10 ¹³⁰		10 ¹³¹		10 ¹³²		10 ¹³³		10 ¹³⁴		10 ¹³⁵		10 ¹³⁶		10 ¹³⁷		10 ¹³⁸		10 ¹³⁹		10 ¹⁴⁰		10 ¹⁴¹		10 ¹⁴²		10 ¹⁴³		10 ¹⁴⁴		10 ¹⁴⁵		10 ¹⁴⁶		10 ¹⁴⁷		10 ¹⁴⁸		10 ¹⁴⁹		10 ¹⁵⁰		10 ¹⁵¹		10 ¹⁵²		10 ¹⁵³		10 ¹⁵⁴		10 ¹⁵⁵		10 ¹⁵⁶		10 ¹⁵⁷		10 ¹⁵⁸		10 ¹⁵⁹		10 ¹⁶⁰		10 ¹⁶¹		10 ¹⁶²		10 ¹⁶³		10 ¹⁶⁴		10 ¹⁶⁵		10 ¹⁶⁶		10 ¹⁶⁷		10 ¹⁶⁸		10 ¹⁶⁹		10 ¹⁷⁰		10 ¹⁷¹		10 ¹⁷²		10 ¹⁷³		10 ¹⁷⁴		10 ¹⁷⁵		10 ¹⁷⁶		10 ¹⁷⁷		10 ¹⁷⁸		10 ¹⁷⁹		10 ¹⁸⁰		10 ¹⁸¹		10 ¹⁸²		10 ¹⁸³		10 ¹⁸⁴		10 ¹⁸⁵		10 ¹⁸⁶		10 ¹⁸⁷		10 ¹⁸⁸		10 ¹⁸⁹		10 ¹⁹⁰		10 ¹⁹¹		10 ¹⁹²		10 ¹⁹³		10 ¹⁹⁴		10 ¹⁹⁵		10 ¹⁹⁶		10 ¹⁹⁷		10 ¹⁹⁸		10 ¹⁹⁹		10 ²⁰⁰		10 ²⁰¹		10 ²⁰²		10 ²⁰³		10 ²⁰⁴		10 ²⁰⁵		10 ²⁰⁶		10 ²⁰⁷		10 ²⁰⁸		10 ²⁰⁹		10 ²¹⁰		10 ²¹¹		10 ²¹²		10 ²¹³		10 ²¹⁴		10 ²¹⁵		10 ²¹⁶		10 ²¹⁷		10 ²¹⁸		10 ²¹⁹		10 ²²⁰		10 ²²¹		10 ²²²		10 ²²³		10 ²²⁴		10 ²²⁵		10 ²²⁶		10 ²²⁷		10 ²²⁸		10 ²²⁹		10 ²³⁰		10 ²³¹		10 ²³²		10 ²³³		10 ²³⁴		10 ²³⁵		10 ²³⁶		10 ²³⁷		10 ²³⁸		10 ²³⁹		10 ²⁴⁰		10 ²⁴¹		10 ²⁴²		10 ²⁴³		10 ²⁴⁴		10 ²⁴⁵		10 ²⁴⁶		10 ²⁴⁷		10 ²⁴⁸		10 ²⁴⁹		10 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LOGGED: AC

CHECKED: SEMC

DEPTH SCALE

1 : 50



SUD-RCK 09-1111-6014 BRIDGE GPJ GAL-MISS GDT 31/10/13 DATA INPUT:

PROJECT 09-1111-6014			RECORD OF BOREHOLE No B4-4			1 OF 1 METRIC														
W.P. 5151-08-01			LOCATION N 5086318.7; E 221862.4			ORIGINATED BY AC														
DIST _____ HWY 522 UNDERPASS			BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers, NW casing and NQ Coring			COMPILED BY AC														
DATUM GEODETIC			DATE August 14, 2012			CHECKED BY SEMC														
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									
185.5	GROUND SURFACE																			
0.0	TOPSOIL		1	SS	6		185											0 0 47 53		
184.7	CLAY, trace sand, trace organics (oxidized) Firm Brown Moist		2	SS	11															
0.8	SAND, trace silt Loose to compact Brown Moist		3	SS	11		184													
			4	SS	13		183											0 98 (2)		
			5	SS	4		182													
			6	SS	7															
181.0	SILT and SAND, some gravel, trace clay Loose Grey Wet		7	SS	10/0.04		181													
4.6	GRANITIC GNEISS (Bedrock)		1	RC	REC 100%		180											RQD = 65%		
	Bedrock cored from 4.6 m depth to 7.8 m depth.																			
	For coring details see Record of Drillhole B4-4.		2	RC	REC 100%		179											RQD = 76%		
177.7	END OF BOREHOLE						178													
7.8	Note: 1. Borehole dry upon completion of drilling.																			

SUD-MTO 001 09-1111-6014_BRIDGE.GPJ GAL-MISS.GDT 31/10/13 DATA INPUT:

PROJECT: 09-1111-6014

RECORD OF DRILLHOLE: B4-4

SHEET 1 OF 1

LOCATION: N 5086318.7 ; E 221862.4

DRILLING DATE: August 14, 2012

DATUM: GEODETIC

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 850

DRILLING CONTRACTOR: Landcore Drilling Inc.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	COLOUR FLUSH % 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		Diameter Point Load Index (MPa)	RMC -Q AVG.																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																											
							TOTAL CORE %	SOLID CORE %			B Angle		DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION		Jr	Ja	Jn	k, cm/s																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																
							888888	888888			0-90°	0-90°		0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°			0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°				0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°	0-90°

DEPTH SCALE

1 : 50



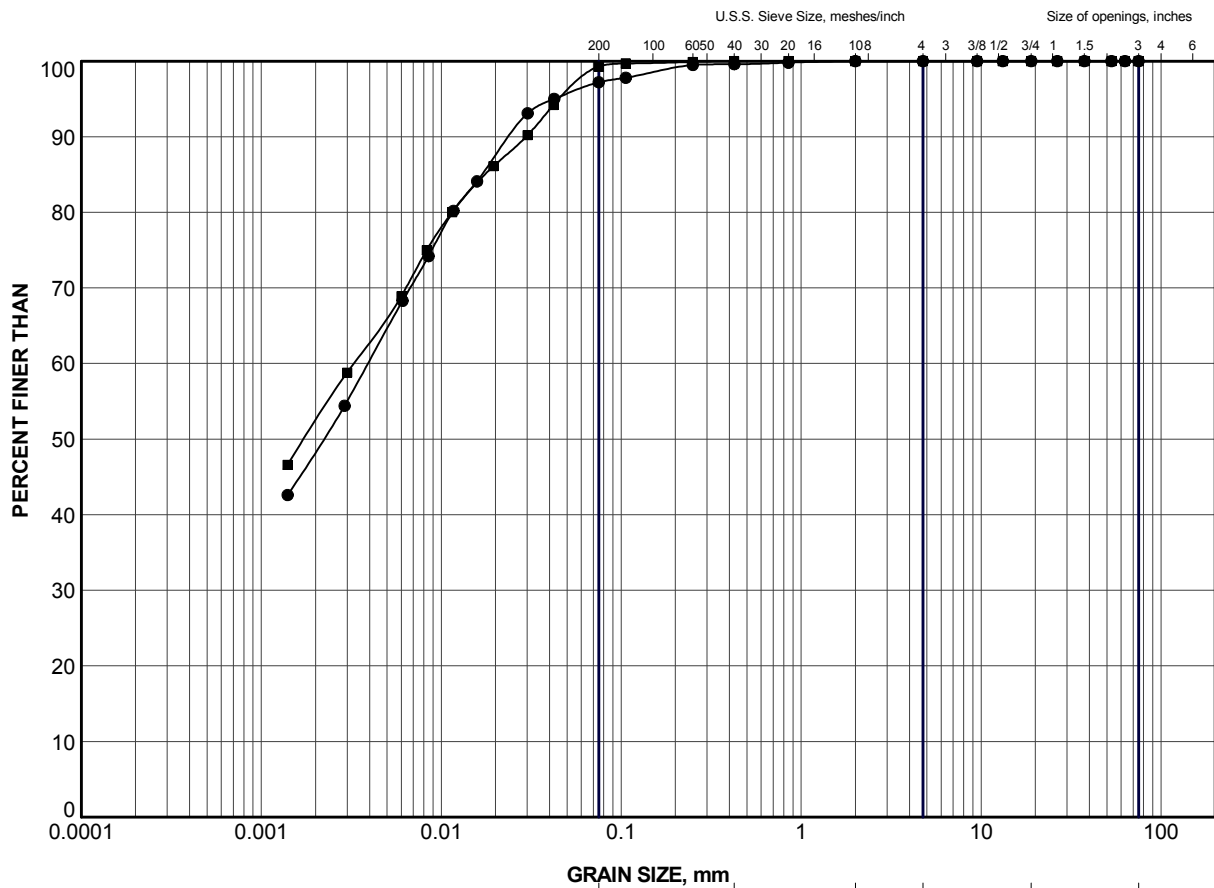
LOGGED: AC

CHECKED: SEMC

SUD-RCK 09-1111-6014 BRIDGE GPJ GAL-MISS GDT 31/10/13 DATA INPUT:

PROJECT		09-1111-6014		RECORD OF BOREHOLE No H402-1				1 OF 1 METRIC					
W.P.		5347-08-01		LOCATION		N 5086323.3; E 221879.3		ORIGINATED BY ID					
DIST		HWY 522 UNDERPASS		BOREHOLE TYPE		108 mm I.D. Continuous Flight Hollow Stem Augers		COMPILED BY AC					
DATUM		GEODETIC		DATE		October 24, 2011		CHECKED BY SEMC					
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT		UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		WATER CONTENT (%)			
185.2	GROUND SURFACE												
0.0	ORGANICS (Topsoil)		1	SS	17								0 3 49 48
184.6	CLAY, trace sand, trace organics Very stiff Brown Moist		2	SS	13								
0.6	SAND, trace silt Loose to compact Brown Moist		3	SS	15								0 97 (3)
			4	SS	14								
181.8	SILT and SAND, trace to some clay Very loose to loose Grey Wet		5	SS	9								
3.4			6	SS	2								0 42 52 6
			7	SS	1								
	Augers grinding below 5.8 m depth.												
179.6	Silty SAND, some gravel, trace clay (TILL) Grey Wet		8	SS	6/0.06								
179.1	Split spoon bouncing at 6.1 m depth.												
6.1	END OF BOREHOLE AUGER AND SPOON REFUSAL												
Note: 1. Water level at a depth of 3.4 m below ground surface (Elev. 181.8 m) upon completion of drilling.													

SUD-MTO 001 09-1111-6014_BRIDGE.GPJ GAL-MISS.GDT 31/10/13 DATA INPUT:



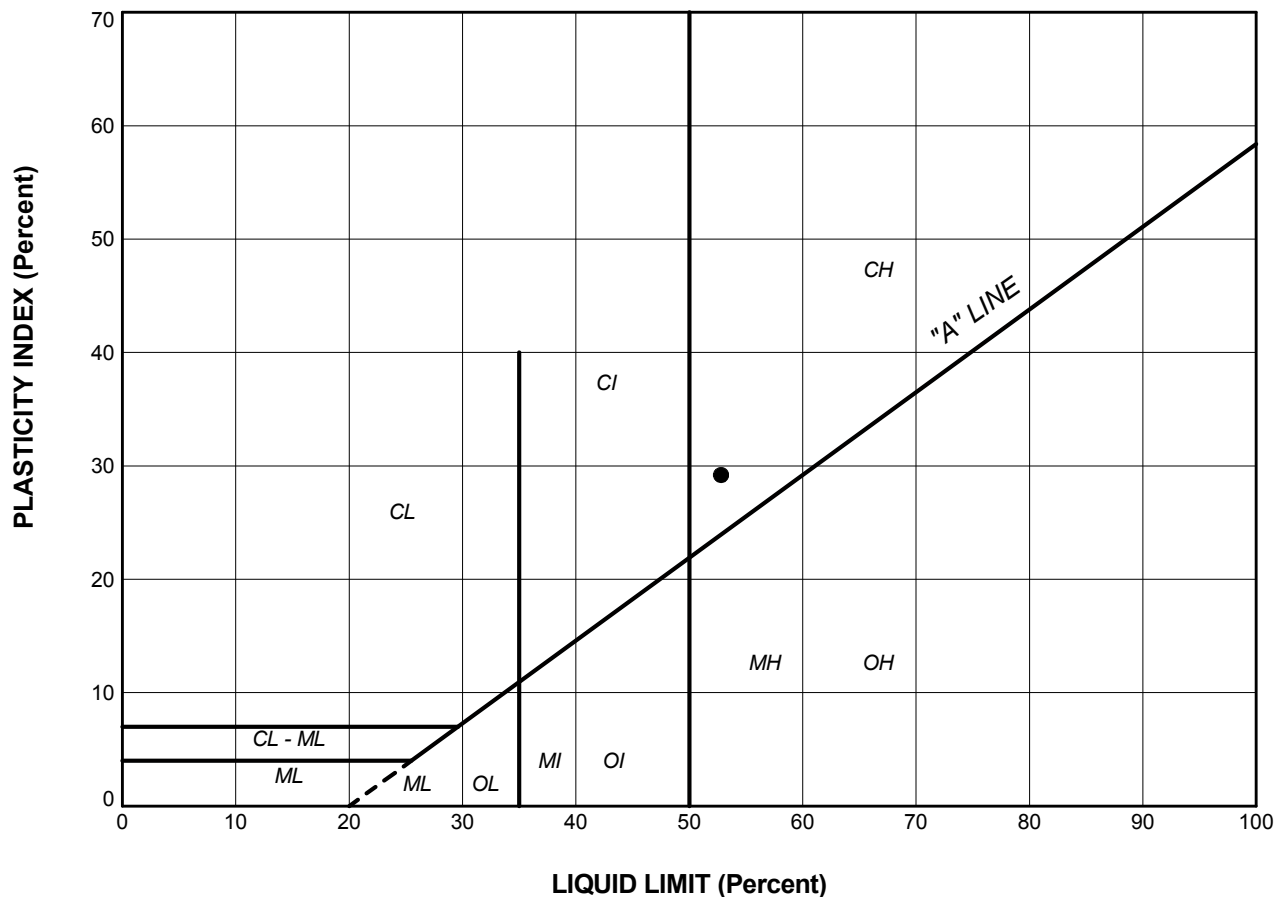
GRAIN SIZE, mm						
CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	H402-1	1	184.9
■	B4-4	1	185.2

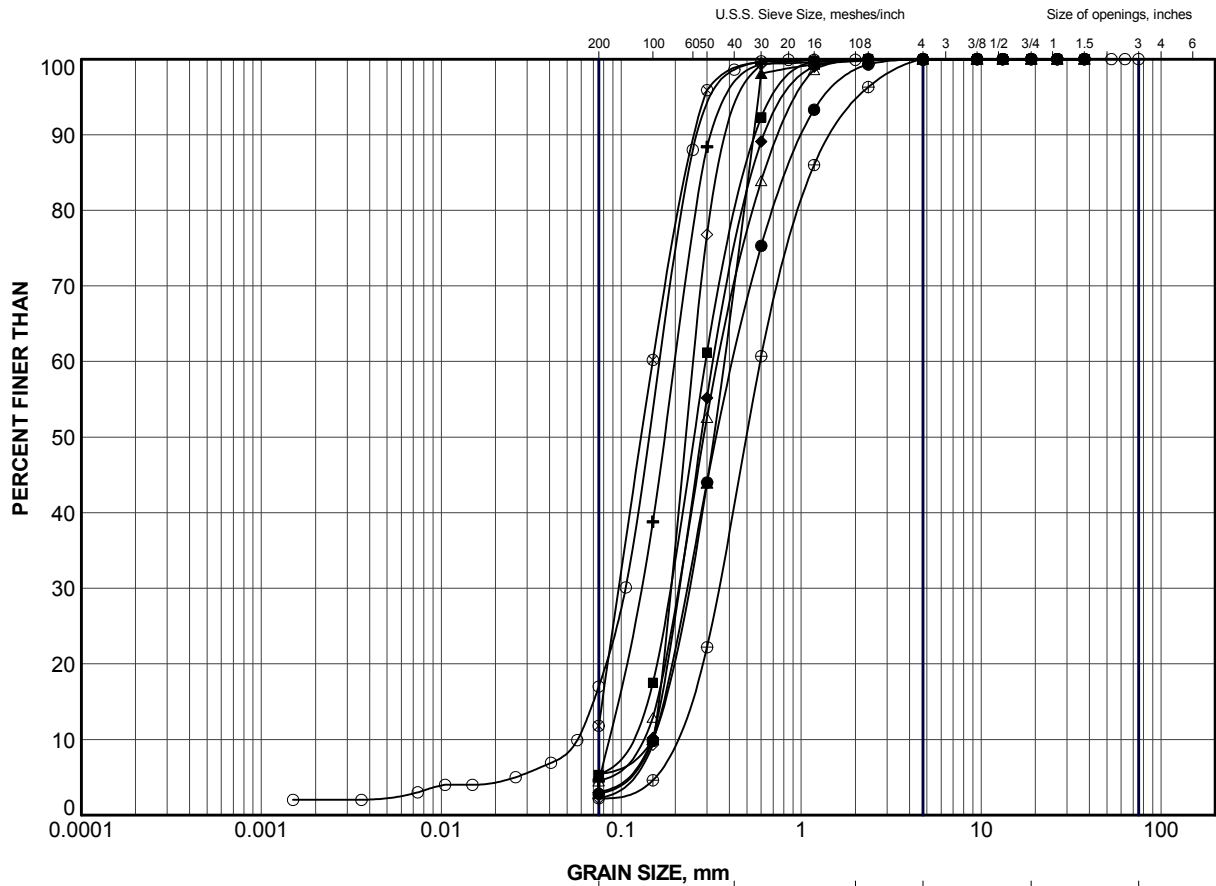
PROJECT						HIGHWAY 522 UNDERPASS (STA 12+900 HIGHWAY 69)											
TITLE						GRAIN SIZE DISTRIBUTION CLAY (NEAR SURFACE)											
PROJECT No.						09-1111-6014						FILED 09-1111-6014_BRIDGE.GPJ					
DRAWN		TB		Nov 2013		SCALE		N/A		REV.							
CHECK		AC		Nov 2013													
APPR		SEMC		Nov 2013								FIGURE A1					





PROJECT				
HIGHWAY 522 UNDERPASS (STA 12+900 HIGHWAY 69)				
TITLE				
PLASTICITY CHART CLAY (NEAR SURFACE)				
PROJECT No.		09-1111-6014		FILE No09-1111-6014_BRIDGE.GPJ
DRAWN	TB	Nov 2013		SCALE N/A
CHECK	AC	Nov 2013		REV.
APPR	SEMC	Nov 2013		FIGURE A2





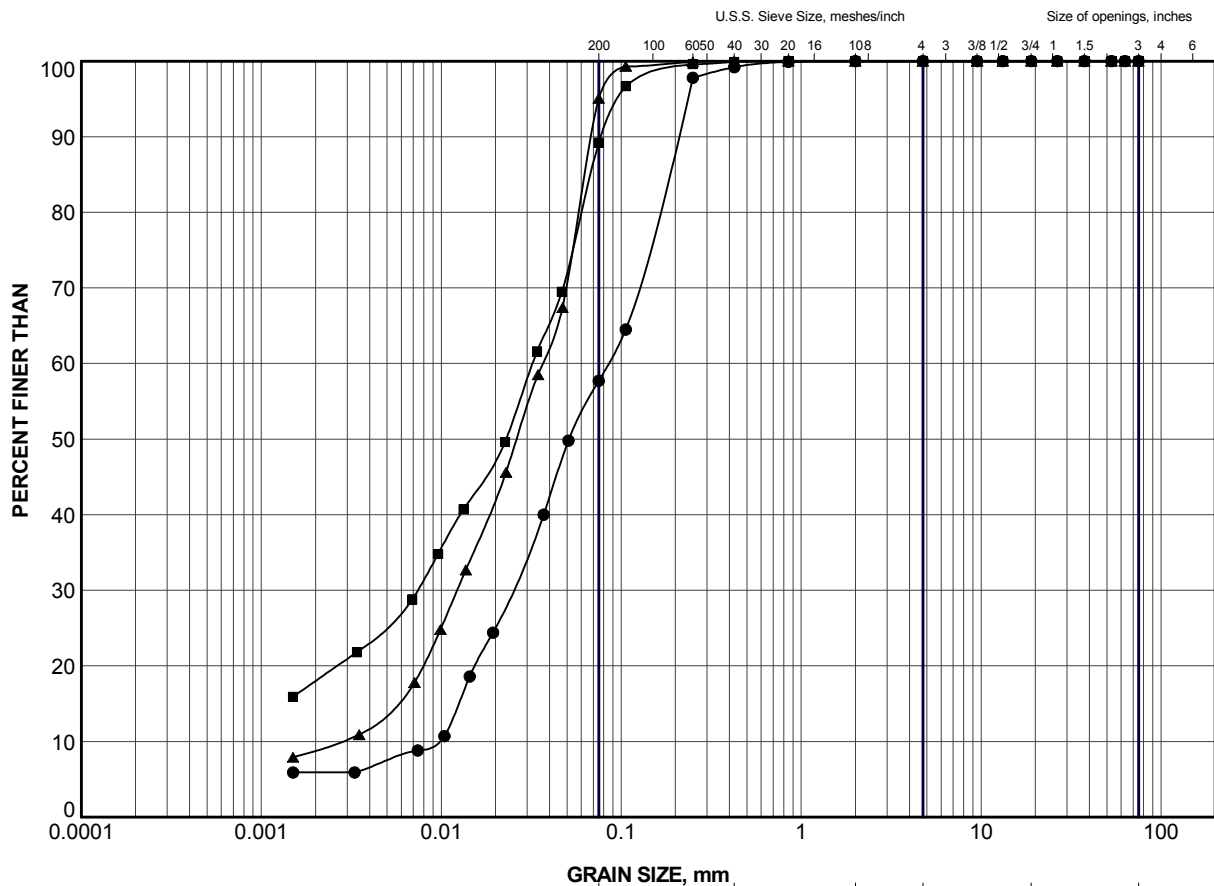
CLAY AND SILT	GRAVEL SIZE, mm					Cobble Size
	fine	medium	coarse	fine	coarse	
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	H402-1	3	183.4
■	B4-1	2	184.2
▲	B4-1	6	181.2
+	B4-1	10	175.8
◆	B4-2	3	183.5
◇	B4-2	6	181.2
○	B4-2	10	175.9
△	B4-3	3	183.7
⊗	B4-3	9	177.6
⊕	B4-4	4	182.9

PROJECT				
HIGHWAY 522 UNDERPASS (STA 12+900 HIGHWAY 69)				
TITLE				
GRAIN SIZE DISTRIBUTION SAND				
PROJECT No.		09-1111-6014		FILE 09-1111-6014_BRIDGE.GPJ
DRAWN	TB	Nov 2013	SCALE	N/A
CHECK	AC	Nov 2013	REV.	
APPR	SEMC	Nov 2013	FIGURE A3	




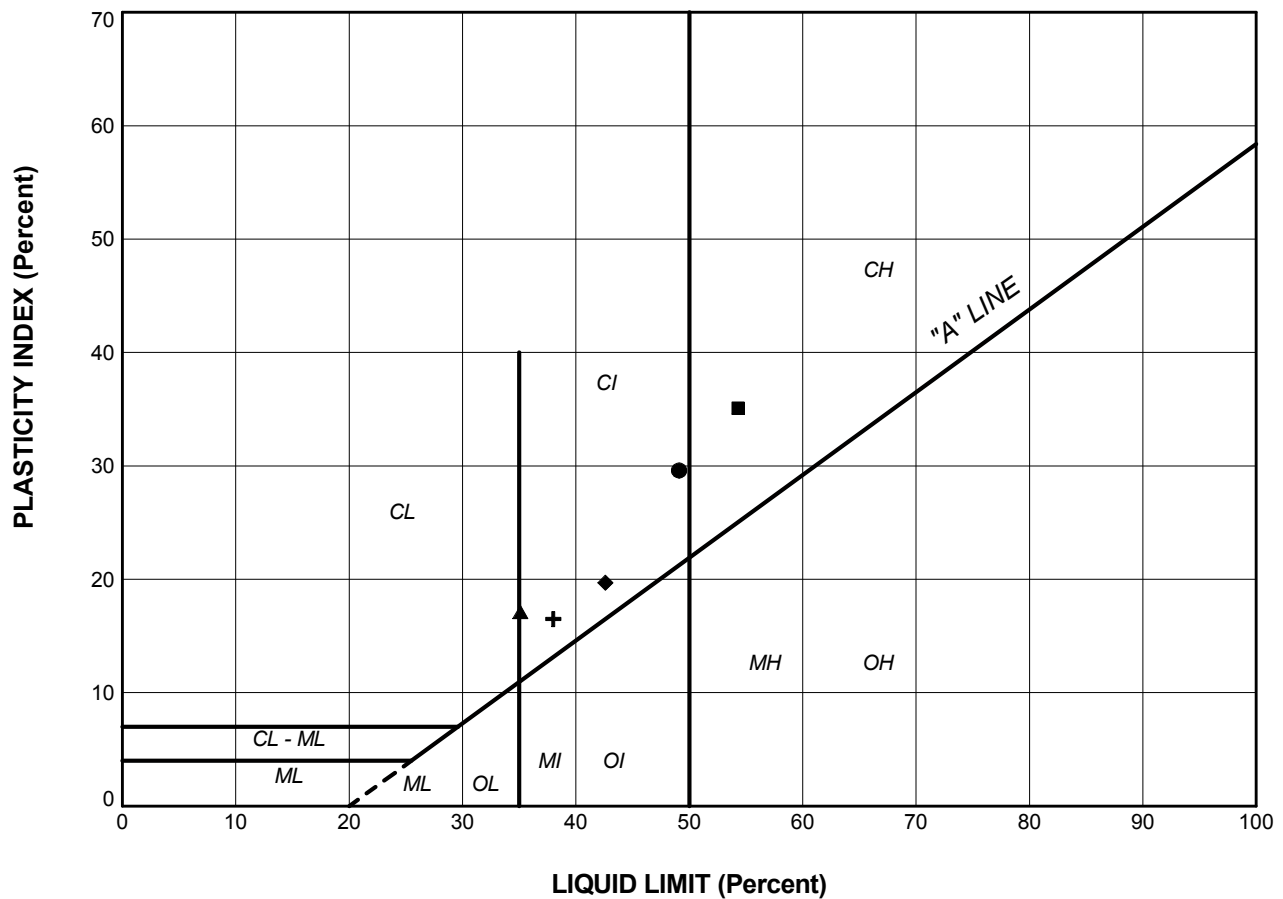


CLAY AND SILT	SAND SIZE			GRAVEL SIZE		Cobble Size
	fine	medium	coarse	fine	coarse	

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	H402-1	6	181.1
■	B4-1	9	177.4
▲	B4-3	12	173.0

PROJECT						HIGHWAY 522 UNDERPASS (STA 12+900 HIGHWAY 69)					
TITLE						GRAIN SIZE DISTRIBUTION SILT AND SAND TO SILT					
PROJECT No. 09-1111-6014						FILED 09-1111-6014_BRIDGE.GPJ					
DRAWN		TB		Nov 2013		SCALE		N/A		REV.	
CHECK		AC		Nov 2013							
APPR		SEMC		Nov 2013							
 Golder Associates SUDBURY, ONTARIO						FIGURE A4					

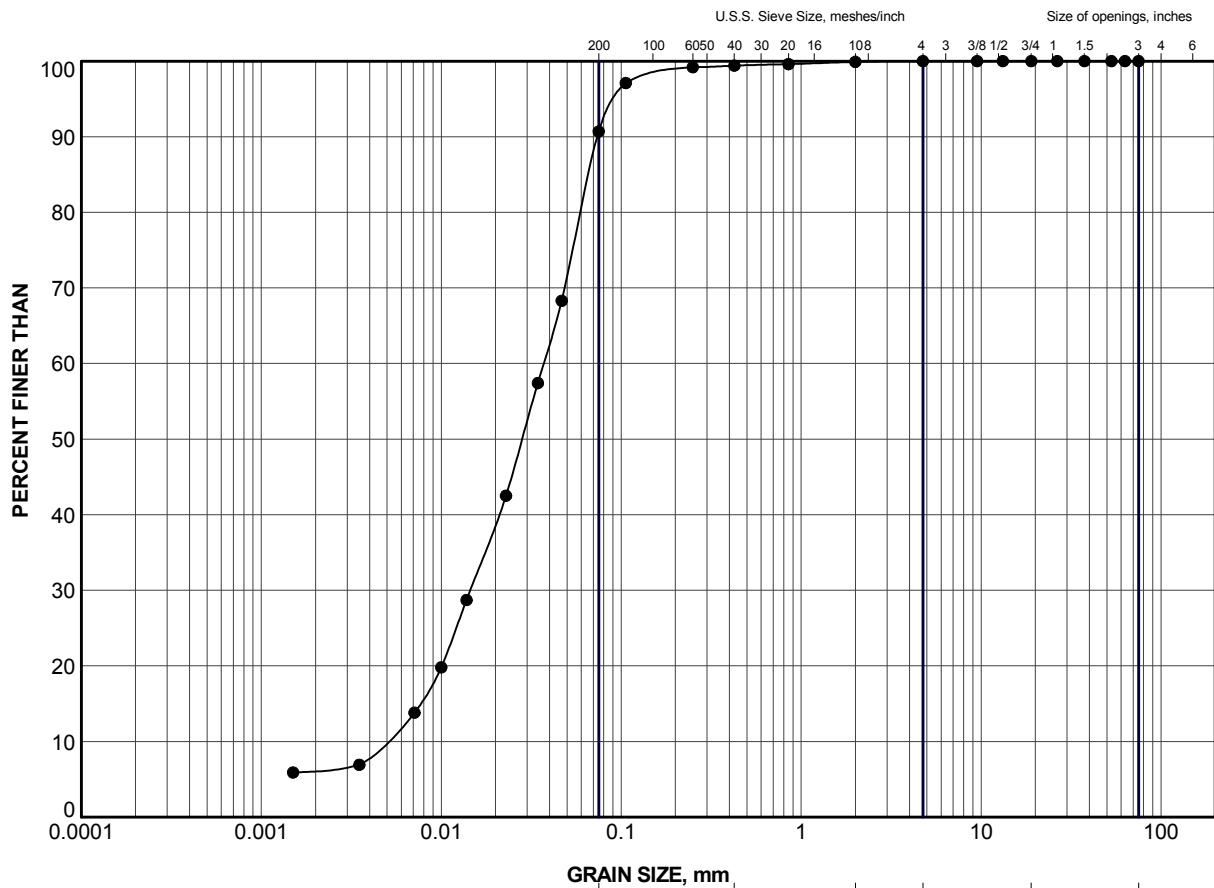


LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	B4-1	8	49.1	19.5	29.6
■	B4-1	11	54.3	19.2	35.1
▲	B4-2	9	35.1	18.0	17.1
+	B4-2	12	38.0	21.5	16.5
◆	B4-3	13	42.6	22.9	19.7

PROJECT					HIGHWAY 522 UNDERPASS (STA 12+900 HIGHWAY 69)				
TITLE					PLASTICITY CHART SILTY CLAY TO CLAY				
PROJECT No.			09-1111-6014		FILE No			09-1111-6014_BRIDGE.GPJ	
DRAWN	TB	Nov 2013		SCALE	N/A		REV.		
CHECK	AC	Nov 2013		FIGURE A5					
APPR	SEMC	Nov 2013							




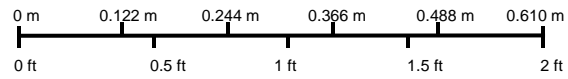


GRAIN SIZE, mm						
CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

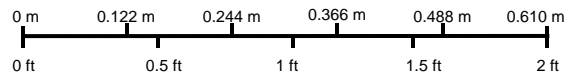
LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	B4-2	13	171.3

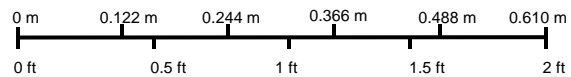
PROJECT						HIGHWAY 522 UNDERPASS (STA 12+900 HIGHWAY 69)					
TITLE						GRAIN SIZE DISTRIBUTION SILT					
PROJECT No. 09-1111-6014						FILE 09-1111-6014_BRIDGE.GPJ					
DRAWN		TB		Nov 2013		SCALE		N/A		REV.	
CHECK		AC		Nov 2013							
APPR		SEMC		Nov 2013							
 Golder Associates SUDBURY, ONTARIO						FIGURE A6					




Borehole B4-2
Elevation 167.6 m to 164.6 m



Borehole B4-3
Elevation 169.6 m to 166.6 m



Borehole B4-4
Elevation 180.9 m to 177.7 m

PROJECT		HIGHWAY 522 UNDERPASS (STA 12+900 HIGHWAY 69)				
TITLE		ROCK CORE PHOTOGRAPHS				
	PROJECT No.		09-1111-6014		FILE No.	----
	DESIGN	AC	NOV 2013		SCALE	AS SHOWN REV.
	CADD	--			FIGURE A7	
	CHECK	SEMC	NOV 2013			
	REVIEW					



APPENDIX B

Non-Standard Special Provisions

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.

CSP FOR INTEGRAL ABUTMENTS – Item No.

Special Provision

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 Mass
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 bedrock.
4. Place loose sand into 600 diameter 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.

CSP FOR INTEGRAL ABUTMENTS – Item No.

Special Provision

The Contractor shall ensure the full perimeters of the tops 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.

ROCK POINTS - Item No.

Non-Standard Special Provision

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

References

OPSS 906 – Structural Steel
SP903S01

Materials

The pile points shall be of the following:

Product

Manufacturer

HPP-R-12

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

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

Basis of Payment

Payment at the lump sum contract price for this tender item shall be full compensation for all labour, equipment and materials for completion of the work.

END OF SECTION

H-PILES – HP310 X 110 - Item No.

Non-Standard Special Provision

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.

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