October 31, 2013

FOUNDATION INVESTIGATION AND DESIGN REPORT

REPLACEMENT OF VALENTINE RIVER BRIDGE - SITE NO. 39W-010
HIGHWAY 11, TOWNSHIP OF STODDART, ONTARIO
MINISTRY OF TRANSPORTATION, ONTARIO
GWP 5150-05-00

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GEOCRES NO.: 42G-48

Report Number: 11-1191-0008-5

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REPORT

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FOUNDATION INVESTIGATION REPORT
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1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by LEA Consulting Ltd. (LEA) on behalf of the Ministry of Transportation, Ontario (MTO) to provide detail foundation engineering services for the replacement of the Valentine River Bridge (Site No. 39W-010), located on Highway 11 west of Hearst in the Township of Stoddart, Ontario.

The Terms of Reference for the Foundation Investigation are outlined in MTO's Request for Proposal dated, March 2011. Golder's proposed Scope of Work for foundation engineering services associated with replacement of the Valentine River Bridge structure is contained in Section 6.8 of LEAs Technical Proposal for this assignment. The work has been carried out in accordance with Golder's Project Specific Supplementary Specialty Plan for foundations engineering services, dated August 2011.

The purpose of this investigation is to establish the subsurface conditions at the location of the proposed replacement structure, including the associated approach embankments, by borehole drilling, rock coring, in situ testing and laboratory testing on selected soil and rock core samples. The location of the investigation area is shown in plan on Drawing 1.

2.0 SITE DESCRIPTION

The Valentine River Bridge site is situated in the Township of Stoddart on Highway 11, approximately 22 km west of the west junction of Highway 11 and Highway 583 in Hearst, Ontario. The surrounding land is generally flat but slopes down towards the riverbanks along the east and west sides of the river. The riverbanks adjacent to the existing bridge area are vegetated with grass and small shrubs, with moderate tree-covered terrain beyond the highway right-of-way limits. The river flows in a northerly direction and is about 35 m wide at the existing bridge location.

The existing structure is a two-lane, thirteen-span, timber bridge with a stressed laminated timber sub-struction with an asphalt surfaced deck and was constructed in 1956. The structure is founded on timber crib abutments and the abutments and piers are supported on timber piles founded at unknown depths. The existing ground surface along the existing structure alignment is about Elevation 239.5 m. The existing embankment front slopes are formed at approximately 1.1 horizontal to 1 vertical (1.1H:1V) and the east and west side slopes adjacent to the abutments are currently at about 1H:1V. The existing approach embankment side slopes are generally flatter than about 2H:1V. There are no visible signs of approach embankment instability or settlement.

The water level shown on the General Arrangement (GA) drawing is Elevation 235.3 m (from November 2011). The water level measured in Valentine River during the field investigations, which took place in October 2012 and July 2013 varied between Elevation 235.3 m and Elevation 234.9 m, respectively. The high water level is reported to be at Elevation 236.1 m. The existing highway embankment grade is approximately 3.7 m above the surrounding ground surface adjacent to the river.

3.0 INVESTIGATION PROCEDURES

The fieldwork for this subsurface investigation was carried out between October 16 to 18, 2012, and June 6 to July 6, 2013, at which time twelve boreholes (Boreholes VS1 to VS12) were advanced using a CME 55 track-mounted drill rig supplied and operated by Landcore Drilling Inc. of Sudbury, Ontario. Boreholes VS1, VS3 and VS4 were advanced at the west abutment, Boreholes VS2, VS5 and VS6 were advanced at the east abutment, VS7 to VS10 were advanced north of the proposed alignment for potential roadway protection, and VS11 and VS12 were advanced at the west and east approaches, respectively. The locations of the boreholes are shown on Drawing 1.

The boreholes were advanced using 108 mm inner diameter hollow-stem augers, as well as NW casing and NQ size core barrel where coring through boulders/bedrock was required. Soil samples were obtained at intervals of depth of about 0.75 m to 1.5 m, using a 50 mm outer diameter split-spoon sampler operated by an automatic hammer on the drill rig, in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586). Field vane shear tests were carried out in cohesive soils for determination of undrained shear strengths (ASTM D2573, Field Vane Strength Sear Test) using MTO Standard 'N' size vanes.

The groundwater conditions were observed in the open boreholes during and immediately following the drilling operations and a standpipe piezometer was installed in Borehole VS2 to permit monitoring of the groundwater level. The piezometer consists of a 50 mm diameter polyvinyl chloride (PVC) pipe, with a 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 pipe was partially backfilled with bentonite pellets to create a seal then backfilled to near surface with cuttings from the boreholes 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 open boreholes were backfilled upon completion of drilling and the piezometer was decommissioned in accordance with Ontario Regulation 903 Wells (as amended).

The fieldwork was supervised on a full-time basis by a member of Golder's staff, who located the boreholes in the field, directed the drilling and sampling and logged the boreholes. The soil samples were identified in the field, placed in labelled containers and transported to Golder's Sudbury Laboratory for further examination and laboratory testing. Index and classification tests consisting of water content, Atterberg limits and grain size distribution were carried out on selected soil samples. In addition, uniaxial compressive strength (UCS) testing was carried out on specimens of the recovered bedrock core. The results of the laboratory testing are shown on the Record of Borehole and Drillhole sheets in Appendix A and on the figures contained in Appendix B.

Classification of the rock mass quality of the bedrock with respect to the Rock Quality Designation (RQD) and UCS are described based on Table 3.10 and Table 3.5, respectively, of the Canadian Foundation Engineering Manual (CFEM, 2006)[1]. The degree of weathering of the bedrock samples (i.e., fresh to completely weathered) and the strength classification of the intact rock mass based on field identification (i.e., strong to very strong) are described in accordance with Table B.3 and Table B.6, respectively, of the International Society for Rock Mechanics (ISRM[2]) standard classification system.

A sample of the river water was obtained during the field investigation using appropriate sampling protocols and submitted to a specialist analytical laboratory under chain of custody procedures for testing for a suite of inorganic parameters. The results of the analytical testing are summarized in Table B1 in Appendix B.

[1]Canadian Geotechnical Society, 2006. Canadian Foundation Engineering Manual, 4th Edition.

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

The borehole locations and elevations were measured in the field by Golder personnel, relative to existing site features and surveyed to stakes placed in the field by JD Barnes Ltd. The borehole locations (referenced to the MTM NAD83 co-ordinate system), ground surface elevations (referenced to Geodetic datum), and borehole depths are shown on Drawings 1 and 2, presented on the Record of Borehole sheets in Appendix A and are summarized below.

Borehole Number	MTM NAD83 Northing (m)	MTM NAD83 Easting (m)	Ground Surface Elevation (m)	Borehole Depth (m)
VS1	5511323.7	307587.9	236.6	13.1
VS2	5511309.6	307639.2	235.8	12.0
VS3	5511324.2	307585.9	236.5	19.4
VS4	5511313.5	307577.2	236.5	16.1
VS5	5511316.1	307635.4	235.8	12.6
VS6	5511304.7	307630.5	236.1	14.2
VS7	5511330.2	307581.7	237.6	10.3
VS8	5511326.6	307593.6	237.4	14.0
VS9	5511321.6	307635.1	235.5	10.8
VS10	5511316.9	307654.3	239.4	15.5
VS11	5511330.7	307561.2	237.0	10.7
VS12	5511305.0	307657.3	236.5	10.5

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

Based on NOEGTS[3] Mapping, the subsoils in the vicinity of the Valentine River Bridge site generally consist of clayey till deposited as a ground moraine.

Published literature indicates that the site is located in the Quetico Subprovince of the Superior Province (OGS, 1991)[4]. The bedrock of this domain consists of muscovite-bearing granitic rocks (peraluminous), and may include biotite granite. Beyond the muscovite-bearing granitic boundary, bedrock consists of meta-sedimentary rocks.

[3] Northern Ontario Engineering Geology Terrain Study. Ontario Geological Society, Map Reference Number 42GNW.

[4] Ontario Geological Survey, 1991, Geology of Ontario. Special Volume 4, Part 1. Eds P.C. Thurston, H.R. Williams, R.H. Sutcliffe and G.M. Stott, Ministry of Northern Development and Mines, Ontario.

4.2 Subsurface Conditions

For the detailed subsurface investigation, twelve boreholes were advanced in the vicinity of Valentine River Bridge. The borehole locations, ground surface elevations and interpreted stratigraphic conditions at the site are shown on Drawings 1 and 2. The detailed subsurface soil and groundwater conditions encountered in the boreholes and the results of in situ and laboratory testing are given on the Record of Borehole sheets, contained in Appendix A. The results of geotechnical laboratory testing are presented in Appendix B. The results of the in situ field tests (i.e., SPT 'N'-values and undrained shear strengths from the field vanes) as presented on the Record of Borehole sheets and in Section 4 are uncorrected. The stratigraphic boundaries shown on the Record of Borehole sheets, and on the interpreted stratigraphic profile on Drawings 1 and 2, are inferred from non-continuous sampling and observation of drilling progress and soil cutting returns and, therefore, represent transitions between soil types rather than exact planes of geological change. The subsoil conditions will vary between and beyond the borehole locations.

In summary, the subsoil conditions encountered at the site consist of fill and/or peat/organic clay/topsoil, underlain by deposits of soft to firm clayey silt to silty clay, firm clayey silt to silt, very loose to compact sandy silt to sand and silt, and hard sandy clayey silt to clayey silt with sand till. A more detailed description of the soil deposits encountered in these boreholes is provided in Sections 4.2.1 to 4.2.7.

4.2.1 Fill

Fill consisting of sand to gravelly sand and/or clayey silt to clay, was encountered from ground surface in Boreholes VS7 and VS8 and underlying a 180 mm thick layer of asphalt in Borehole VS10. The total thickness of the fill ranges from 1.1 m to 3.0 m, and was encountered between Elevation 239.2 m and 237.4 m.

The granular fill consists of moist, brown gravelly sand to sand containing trace to some silt. The SPT 'N'-values measured within the granular fill portion of the deposit range from 4 blows to 10 blows per 0.3 m of penetration indicating a loose relative density.

The clayey silt to clay fill was moist, brown containing trace to some sand. The SPT 'N'-values measured within the clayey silt to clay fill range from 7 blows to 12 blows per 0.3 m of penetration suggesting a firm to stiff consistency.

The results of grain size distribution testing completed on one selected sample of the sand fill is shown on Figure B1 in Appendix B.

Atterberg limits testing carried out on two samples of the clayey silt to clay fill gave liquid limits of about 33 per cent and 51 per cent, plastic limits of about 18 per cent and 20 per cent and plasticity indices of about15 per cent to 31 per cent. The results of the Atterberg limits tests are shown in the Plasticity chart on Figure B2 in Appendix B, and indicate the fill material to be clayey silt of low plasticity to clay of high plasticity.

The natural moisture content measured on one sample of the granular fill is about 4 per cent. The natural moisture content measured on two samples of the clayey silt to clay fill is about 19 per cent and 24 per cent.

4.2.2 Peat/Organic Clay/Topsoil

A deposit containing black, fibrous to amorphous peat or topsoil was encountered from ground surface or underlying the fill deposit in all Boreholes except VS8. The top of this organic layer was encountered between Elevation 237.0 m and 235.5 m, and the thickness of the deposit ranges between 0.2 m and 2.2 m. In Borehole VS9, the amorphous peat transitioned to organic clay at a depth of 0.7 m below ground surface (Elevation 234.8 m).

The SPT 'N'-values measured within the organic deposit range from 0 blows (weight of hammer) to 6 blows per 0.3 m of penetration, suggesting a very soft to firm consistency.

Atterberg limits testing carried out on one sample of the organic clay gave a liquid limit of about 59 per cent, a plastic limit of about 31 percent and a plasticity index of about 28 per cent. The result of the Atterberg limits test is shown in the plasticity chart on Figure B3 in Appendix B and is below the A-line.

The organic content measured on one sample of the organic clay is about 4 per cent.

The natural moisture content measured on one sample of the organic clay is about 38 per cent.

4.2.3 Clayey Silt to Silty Clay

A deposit of brown to grey clayey silt to silty clay was encountered below the peat/organic clay/topsoil in all Boreholes except VS9. The surface of this deposit was encountered between Elevation 236.5 m and 235.2 m and the thickness of the deposit ranges from 0.8 m to 3.5 m.

The SPT 'N'-values measured within the clayey silt to silty clay deposit range from 0 blows (weight of hammer) to 10 blows per 0.3 m of penetration. In situ field vane tests carried out in this deposit measured undrained shear strengths ranging from 24 kPa to 45 kPa and calculated sensitivities ranging from 1 to 4. The in situ vane test results, together with the SPT 'N'-values, suggest that the clayey silt to silty clay deposit generally has a soft to firm consistency.

The results of grain size distribution tests completed on three samples of the clayey silt to silty clay are shown on Figure B4.

Atterberg limits testing carried out on twelve samples of the clayey silt to silty clay deposit yielded liquid limits ranging from about 27 per cent to 41 per cent, plastic limits ranging from about 14 per cent to 20 per cent and plasticity indices ranging from about 9 per cent to 23 per cent. The results of the Atterberg limits testing are shown on the plasticity chart on Figure B5 and indicate that the deposit is classified as a clayey silt of low plasticity to silty clay of intermediate plasticity.

The natural moisture content measured on fifteen samples of the clayey silt to silty clay deposit ranges from about 19 per cent to 31 per cent.

4.2.4 Clayey Silt to Silt

A deposit of clayey silt to silt, trace to some sand, trace to some gravel, was encountered underlying the clayey silt to silty clay deposit in Boreholes VS1 to VS3, VS5 to VS8, VS10 and VS12 and underlying the organic clay in

Borehole VS9. The surface of the clayey silt to silt deposit was encountered between Elevation 235.2 m and 232.1 m and the thickness of the deposit ranges from 0.7 m to 3.2 m.

The SPT 'N'-values measured within the clayey silt to silt deposit range from 1 blow to 9 blows per 0.3 m of penetration. In situ field vane tests carried out in this deposit measured undrained shear strengths ranging from 43 kPa to 48 kPa and calculated sensitivities ranging from 3 to 5. The in situ vane test results, together with the SPT 'N'-values, suggest that the clayey silt to silt deposit generally has a soft to firm consistency.

The results of grain size distribution testing completed on nine selected samples of the clayey silt to silt deposit are shown on Figure B6.

Atterberg limits testing carried out on ten samples of the clayey silt to silt deposit yielded liquid limits ranging from about 17 per cent to 23 per cent, plastic limits ranging from about 12 per cent and 17 per cent and plasticity indices ranging from about 4 per cent to 10 per cent. The results of the Atterberg limits testing are shown on the plasticity chart on Figure B7 and indicate that the deposit consists of clayey silt of low plasticity to silt of slight plasticity.

The natural moisture content measured on ten samples of the clayey silt to silt deposit ranges from about 11 per cent and 31 per cent.

4.2.5 Sandy Silt to Sand and Silt

A deposit of grey sandy silt to sand and silt was encountered underlying the cohesive deposits in all of the boreholes. The surface of this deposit was encountered between Elevation 234.6 m and 230.2 m and the thickness of the deposit ranges from 2.7 m to 5.7 m.

SPT 'N'-values measured within this deposit range from 0 blows (weight of hammer) to 26 blows per 0.3 m of penetration, indicating a very loose to compact relative density.

The results of grain size distribution testing completed on nine samples of the sandy silt to sand and silt deposit are shown on Figure B8.

The natural moisture content measured on ten samples of the sandy silt to silty sand deposit ranges from about 13 per cent to 23 per cent.

4.2.6 Sandy Clayey Silt to Clayey Silt with Sand (Till)

A deposit of grey sandy clayey silt to clayey silt with sand till containing trace to some gravel was encountered below the sandy silt to sand and silt deposit in all of the boreholes. The surface of this deposit was encountered between Elevation 230.4 m and 226.8 m and the thickness of the deposit ranges from 1.6 m and 7.4 m where the deposit was fully penetrated. Boreholes VS1, VS2 and VS5 to VS12 were terminated within this deposit.

Difficult auger and/or casing advancement was noted throughout this deposit and coring techniques were required to advance some boreholes at various depths throughout this deposit, resulting in sample recoveries ranging from 0 per cent to 100 per cent. A granite boulder, 1.0 m thick, was encountered at a depth of 12.1 m below ground surface (Elevation 224.5 m) in Borehole VS1, and cored through. In Borehole VS8, a 0.4 m thick granite boulder was encountered and cored through at a depth of 9.7 m below ground surface (Elevation 227.7 m).

The measured SPT 'N'-values within the till deposit range from 36 blows to greater than 289 blows per 0.3 m of penetration with several split spoons that did not penetrate the full 0.3 m. The SPT 'N'-values together with the requirement for coring through boulders in the deposit indicate that the till deposit has a hard consistency.

The results of grain size distribution testing completed on eleven samples of the till deposit are shown on Figure B9.

Atterberg limits testing carried out on six samples of the sandy clayey silt to clayey silt with sand deposit yielded liquid limits ranging from about 13 per cent to 18 per cent, plastic limits ranging from about 6 per cent and 13 per cent and plasticity indices ranging from about 4 per cent to 9 per cent. The results of the Atterberg limits testing are shown on the plasticity chart on Figure B10 and indicate that the fines portion of the till deposit consists of clayey silt of low plasticity to silt of slight plasticity.

The natural moisture content measured on eleven samples of the till deposit ranges from about 8 per cent to 11 per cent.

4.2.7 Bedrock

The bedrock surface was encountered in Boreholes VS3 and VS4 at depths of 16.1 m and 12.9 m below ground surface (Elevation 220.4 m and 223.6 m), and was cored for a length of 3.3 m and 3.2 m, respectively. The retrieved bedrock is described as fine grained, slightly to moderately weathered grey gneiss. Photographs of the retrieved bedrock core samples are shown on Figure B10.

The Total Core Recovery (TCR) in both Boreholes is 100 per cent. The Rock Quality Designation (RQD) measured ranges from about 77 per cent to 100 per cent, indicating a rock mass of good to excellent quality (CFEM, 2006).

Laboratory UCS testing was carried out on two core samples of the bedrock. The UCS values are presented on the Record of Drillhole sheets in Appendix A and summarized below and indicate that the bedrock is strong (R4, 50 MPa < UCS < 100 MPa) to very strong (R5, 100 MPa < UCS < 250 MPa) (ISRM, 1985).

Borehole	Elevation (m)	UCS (MPa)
VS3	219.0	75
VS4	223.2	126

4.2.8 Groundwater Conditions

Groundwater levels were measured in the open boreholes during and upon completion of drilling and a piezometer was installed in Borehole VS2, sealed within the sand and silt deposit, to monitor the groundwater level. The measured groundwater levels in the open boreholes and piezometer are presented below.

Borehole	Installation	Time and/or Date	Groundwater Depth (m)	Groundwater Elevation (m)
VS1	Open Borehole	October 17, 2012	1.4	235.2
VS2	Open Borehole	October 18, 2012	1.1	234.7
	Piezometer	December 6, 2012	0.4	235.4
	Piezometer	July 5, 2013	0.9	234.9
VS3	Open Borehole	June 8, 2013	-0.6[1]	237.1
VS4	Open Borehole	June 10, 2013	5.0	231.5
VS5	Open Borehole	July 4, 2013	1.1	234.7
VS6	Open Borehole	June 6, 2013	1.3	234.8
VS7	Open Borehole	June 7, 2013	1.8	235.8
VS8	Open Borehole	June 17, 2013	2.4	235.0
VS9	Open Borehole	June 27, 2013	6.1	229.4
VS10	Open Borehole	June 26, 2013	4.9	234.5
VS11	Open Borehole	June 6, 2013	2.9	234.1
VS12	Open Borehole	June 27, 2013	8.8	227.7

Note:
1. Water level above ground surface upon completion of drilling.

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

Groundwater and river water levels in the area are subject to seasonal fluctuations and to fluctuations after precipitation events and snowmelt. The water level in Valentine River was measured at Elevation 235.3 m on October 2012 and at Elevation 234.9 m in July 2013, which is near that of the groundwater level measured in the piezometer.

5.0 CLOSURE

The field drilling program was supervised by Mr. Indulis Dumpis and Mr. Ed Savard. This report was prepared by Mr. Adam Core, E.I.T. and by Mr. Evan Childerhose, P.Eng. The technical aspects were reviewed by Ms. Sarah Coyne, P.Eng., Associate. Messrs. Fintan Heffernan, P.Eng., and Jorge Costa, P.Eng., Principal, Designated MTO Foundations Contacts, conducted an independent quality control review of this report.

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PART B

FOUNDATION DESIGN REPORT
REPLACEMENT OF VALENTINE RIVER BRIDGE
HIGHWAY 11, SITE NO. 39W-010
TOWNSHIP OF STODDART, ONTARIO
MINISTRY OF TRANSPORTATION, ONTARIO
GWP 5150-05-00

6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides foundation design recommendations for the proposed replacement of the Valentine River Bridge (Site No. 39W-010) located on Highway 11, west of Hearst, Ontario. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the subsurface investigation.

The interpretation of the subsurface information and recommendations presented are intended to provide the designers with sufficient information to assess the feasible foundation alternatives and to design the proposed structure foundations and approach embankments. As such, where comments are made on construction, they are provided to highlight those aspects that could affect the design of the project. Those requiring information on construction aspects 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

The existing thirteen-span Valentine River Bridge was constructed in 1956, with the piers and timber crib abutments supported on timber piles founded at unknown depth, as shown on the existing bridge drawings. We understand that due to the age and poor condition of the existing bridge, replacement will be required. We understand that the replacement bridge will be a single-span structure, on a new alignment located 18 m south of the existing centreline. The current General Arrangement (GA) drawing indicates that an integral abutment structure founded on driven steel piles is the preferred bridge support system alternative from a structural perspective.

The finished grade for the realigned Highway 11 will be approximately Elevation 240.4 m at the west abutment and Elevation 240.1 m at the east abutment, requiring approach embankments up to 3.9 m and 4.3 m high, respectively, relative to the existing natural ground surface.

The subsurface conditions within the existing roadway generally consist of a layer of asphalt surface treatment underlain by granular fill and clayey silt to clay fill. An organic deposit comprised of peat and/or topsoil was encountered from ground surface where the boreholes were advanced beyond the existing embankment. Cohesive deposits of silty clay to clayey silt and clayey silt to silt were encountered underlying the embankment fill or peat, underlain by a deposit of sandy silt to sand and silt which is in-turn underlain by a deposit of hard sandy clayey silt to clayey silt with sand till. In Boreholes VS3 and VS4, where the till layer were fully penetrated, the bedrock surface was encountered at depths of 16.1 m and 12.9 m below ground surface, at Elevation 220.4 m and Elevation 223.6 m, respectively. At the time of the borehole investigations, the groundwater level varied from Elevation 235.3 m (October 2012) to Elevation 234.9 m (July 2013). Borehole VS3 exhibited artesian groundwater conditions.

6.2 Foundations

In order to provide sufficient geotechnical resistance to support the proposed bridge abutments, shallow strip/spread footings would need to be founded on the hard till deposit. Given that the depth to the hard sandy clayey silt to clayey silt with sand till deposit, is up to 8.7 m below the existing grade at the east and west abutments, strip/spread footings are not considered to be practical for this site. In addition, the base of these excavations would be up to approximately 8 m below the high water level and extensive shoring/dewatering would be required to facilitate such an excavation and construction in-the-dry. Spread footings at a higher elevation are not recommended due to the low geotechnical resistance that would be available from both the very loose to compact sandy silt to sand and silt or the upper soft to firm cohesive deposits, as well as the need for dewatering/shoring prior to excavating operations. Spread footings on a granular pad are also not recommended due to the potential for settlement of the subsoils and embankment instability as a result of footing pressure and increased soil weight.

Caisson foundations are not considered practical due to the artesian groundwater conditions and the presence of cobbles and boulders (as inferred from difficult drilling advance and coring) in the sandy clayey silt to clayey silt with sand till deposit.

Deep foundations comprised of driven steel H-piles are considered to be the preferred alternative from a foundations perspective and suitability for integral abutment design. Since bedrock was not encountered at the proposed east abutment, the foundation design should be based on the use of end bearing piles in the hard sandy clayey silt to clayey silt with sand till. Steel tube piles are not considered appropriate for this site as these piles are displacement piles which potentially could create a void along the length of the piles leading to artesian groundwater flow along the pile as well. Also they pose a higher risk of "hanging up" or being deflected away from their vertical or battered orientation due to the presence of cobbles and boulders (due to their larger end area).

Table 1 summarizes the advantages, disadvantages, relative costs and risks/consequences of the foundation alternatives for the replacement structure. Design recommendations for the recommended option are given in the sections below.

6.3 Deep Foundations

We recommend that the bridge be supported on steel HP310X110 piles driven to penetrate into the hard sandy clayey silt to clayey silt with sand till (having Standard Penetration Test "N"-values greater than about 100 blows per 0.3 m of penetration). Driven steel pile foundations allow for the pile caps to be constructed at a higher elevation than footings, resulting in less excavation and unwatering needs. Given that the till deposit is glacially derived and contains cobbles and boulders, the piles could "hang up" or be deflected from their intended vertical alignment. Therefore, consideration should be given to using a heavier H-pile section, such as HP310x132 piles, to reduce the potential for damage to the piles during driving to the required tip elevation. The piles will also penetrate the till deposit under potentially artesian groundwater pressure encountered near the base of the till deposit (as encountered in Borehole VS3) at the west abutment. Artesian conditions were not encountered in the east abutment boreholes.

The following sections provide details regarding the tip elevation, geotechnical axial resistances/reactions, set criteria and pile driving notes, resistance to lateral loads and frost protection for driven steel H-piles.

6.3.1 Design Tip Elevation

The estimated pile lengths given below are based on the underside of pile cap elevations shown on the General Arrangement drawing. The tip elevations correspond to the estimated termination depth of the piles, approximately 3 m into the hard sandy clayey silt to clayey silt with sand till deposit or to the bedrock surface at the west abutment.

Foundation Element	Borehole Numbers	Proposed Underside of Pile Cap Elevation (m)	Design Pile Tip Elevation (m)	Estimated Approximate Pile Length (m)
West Abutment	VS1, VS3, VS4 and VS7	234.5	220.4 to 223.6	11 to 14
East Abutment	VS2, VS5, VS6 and VS9	234.5	222 to 225	9.5 to 12.5

Note: As per comments received from LEA on September 5, 2013.

It should be noted that the bedrock surface was encountered at Elevation 233.6 m in Borehole VS4 and Elevation 220.4 m in Borehole VS3 located at the west abutment, indicating that at this location, the piles could terminate on the bedrock surface, however, it is likely that, in places, the piles will terminate within the hard sandy clayey silt to clayey silt with sand till deposit overlying the bedrock surface. At the east abutment, high SPT 'N'-values or refusal were obtained below Elevation 225.0 m and the piles will terminate within this hard till deposit.

6.3.2 Geotechnical Axial Resistance

For steel H-piles end-bearing in the hard sandy clayey silt to clayey silt with sand till deposit, the geotechnical axial resistance at Ultimate Limit States (ULS) is achieved by a combination of shaft resistance and toe resistance, and the factored ULS may be estimated by applying a factor of 0.5 on the ultimate resistance in accordance with current MTO Foundations practice. The axial reaction at Serviceability Limit States (SLS) (for 25 mm of settlement) assumes that the pile will settle approximately 10 mm to 15 mm to mobilize shaft friction. The factored ULS and SLS values for two different pile types driven to the elevations given above are as follows.

Pile Section	Factored Geotechnical Axial Resistance at ULS	Geotechnical Axial Resistance at SLS (for 25 mm settlement)
HP310X110	1,600	1,100
HP310X132	1,800	1,100

Artesian conditions were encountered when advancing Borehole VS3 through the hard sandy clayey silt to clayey silt with sand till deposit overlying the bedrock surface and the artesian groundwater level was recorded at Elevation 237.1 m (corresponding to 0.6 m above the ground surface or about 1.8 m above the river level) upon completion of drilling this borehole. A filter sand blanket (see Section 6.7.2) should be constructed immediately below the pile cap at the west abutment to dissipate artesian groundwater and filter soil fines that may be carried upwards to the surface of the native soils in the event that piles driven to terminate within the till deposit penetrate through an artesian layer.

If corrugated steel pipes (CSPs) are installed as part of the integral abutment design (through which the piles will be driven), the CSPs should be backfilled with a loose, fine to medium sand. An NSSP detailing the installation method and gradation of this sand should be included in the Contract Documents; an example is provided in Appendix C. Given that the settlement of the subsoils under the proposed embankment loading is estimated to occur rapidly during construction (see Section 6.6.2.3), downdrag loads need not be considered for design.

6.3.3 Set Criteria and Pile Driving Note

All pile installation/driving should be in accordance with OPSS 903 (Deep Foundations). The piles should be fitted with driving shoes or flange plates (reinforced tips) in accordance with OPSD 3000.100 (Steel H-Driving Shoe) to minimize damage to the pile tip during driving. Given the presence of cobbles and boulders within the till deposits and potential for damage to the pile tip during driving, we recommend consideration be given to using the heavier pile section (HP310x132).

The pile termination or set criteria will be dependent on the pile driving hammer type and the selected pile type. The set criteria can be established through a variety of methods including empirical correlations, such as the use of the Hiley Formula, and wave equation analyses, at the time of construction once the hammer and pile types are known. The criteria need to be set to allow for founding of the piles on bedrock and into the hard sandy clayey silt to clayey silt with sand till deposit and to also avoid overdriving and possibly damaging the piles.

For end-bearing piles not founded on bedrock, the pile capacity must be verified in the field by the use of the Hiley Formula in accordance with Standard Structural Drawing SS103-11 (April 2008) “Pile Driving Control” during the final stages of driving, starting at about 2 m to 3 m higher than the tip elevations provided in Section 6.3.1. The ultimate geotechnical axial resistance predicted from the Hiley Formula should then be multiplied by a geotechnical resistance factor equal to 0.5 as per current MTO practice to verify the factored ULS design value. An NSSP, which outlines the above set criteria, should be included in the Contract; an example is included in Appendix C.

The pile driving note that should be added to the drawings for this project is Notes 2 to 5, as applicable, in Clause 3.3.3 of the Structural Manual (MTO, 2008).

For HP310X110 piles, the note should read:

- Piles to be driven in accordance with Standard Structural Drawing SS103-11 using an ultimate geotechnical resistance of 3,200 kN per pile but must be driven below EL 225 m.

For HP310X132 piles, the note should read:

- Piles to be driven in accordance with Standard Structural Drawing SS103-11 using an ultimate geotechnical resistance of 3,600 kN per pile but must be driven below EL 226 m.

6.3.4 Resistance to Lateral Loads

The design of piles subjected to lateral loads should take into account such factors as the batter of the piles (if any), the relative rigidity of the pile to the surrounding soil, the fixity condition at the head of the pile (pile cap level), the structural capacity of the pile to withstand bending moments, the soil resistance that can be mobilized, the tolerable lateral deflections at the head of the pile and pile group effects. For a longer, more flexible pile, the maximum yield moment of the pile may be reached prior to 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 Canadian Highway Bridge Design Code (CHBDC) and it's Commentary (2006):

for non-cohesive soils:

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

where:

n_h = constant of subgrade reaction (kPa/m)
z = depth (m)
B = pile diameter or width (m)

and for cohesive soils:

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

where:

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

It is understood that an integral abutment foundation design is being considered and CSP liners may be required at this site. Where the integral design includes the installation of 3 m long CSP liners (with the annular space between the pile and the liner filled with uniform grained, uncompacted sand), the upper portion of the H-piles will be generally free to flex and move laterally within the limits of the CSP. With this design, the passive lateral resistance over the length of the pile within the CSP liner should be based on the resistance provided by loose sand. The passive lateral resistance on the exterior of the CSP should be based on the resistance provided by the surrounding soil conditions.

The lateral resistance of the piles should be developed primarily from the passive resistance of the soil. The values of n_h (Terzaghi, 1955) and s_u 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 (with and without CSP liners) at this site are summarized below.

Foundation Element (Relevant Boreholes)	CSP Liner Options	Soil Unit	Elevation (m)	n_h (kPa/m)	s_u (kPa)
West Abutment (VS3 and VS4)	With CSP Liners	Loose Sand (CSP and Filter Blanket)	234.5 to 231.5	1,300	-
		Soft Clayey Silt to Silt	231.5 to 231.0	-	27
		Very Loose to Compact Sand and Silt	231.0 to 227.8	4,400	-
	Without CSP Liners	Filter Blanket	234.5 to 234.0	1,300	-
		Soft Clayey Silt to Silt	234.0 to 231.0	-	27
		Very Loose to Compact Sand and Silt	231.0 to 227.8	4,400	-
	With or Without CSP Liners	Hard Sandy Clayey Silt to Clayey Silt with Sand (Till)	227.8 to 224.0	11,000	-
East Abutment (VS5 and VS6)	With CSP Liners	Loose Sand within CSP	234.5 to 231.5	1,300	-
		Very Soft to Stiff Sandy Clayey Silt to Silt	231.5 to 230.2	-	40
	Without CSP Liners	Soft to Firm Clayey Silt to Silty Clay	234.5 to 232.1	-	27
		Soft to Stiff Sandy Clayey Silt to Silt	232.1 to 230.2	-	40
	With or Without CSP Liners	Very Loose to Compact Sand and Silt	230.2 to 227.1	1,300	-
		Hard Sandy Clayey Silt to Clayey Silt with Sand (Till)	227.1 to 226.3	11,000	-

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

Pile Size	Lateral Resistance/Reaction (kN)	
	ULS (Factored)	SLS (10 mm of deflection)
HP310X110	69	23
HP310X132	75	25

The lateral resistances give above are based on a vertical load of 1,100 kN per pile. The lateral resistance should be reviewed for vertical loads greater than 1,100 kN per pile.

It is recommended that both the structural and geotechnical resistances of the piles should be evaluated to establish the governing case at ULS. At SLS, the horizontal 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 resistance should be taken as that corresponding to a horizontal deflection of 10 mm at the underside of the pile cap for units supporting abutments (CHBDC Commentary C6.8.7.1).

Group action for lateral loading should also be considered when the pile spacing in the direction of the loading is less than six to eight pile diameters. Group action can be evaluated by reducing the coefficient of horizontal subgrade reaction (NAVFAC, 1986) in the direction of loading by a reduction factor, R, as follows:

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

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

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

6.3.5 Frost Protection

All pile caps should be provided with a minimum of 2.6 m of soil cover for frost protection as per OPSD 3090.100 (Foundation Frost Penetration Depths for Northern Ontario).

6.4 Seismic Considerations

Based on the latitude and longitude of the site, (49.7398^{o} N and 83.9608 ^{o} W) the peak horizontal acceleration (PHA) is reported to be equal to 0.075 g at the bedrock level at the site based on the information obtained from the NRCan website for a probability of exceedance of 10% in 50 years. According to Table 4.1 of the CHBDC, this site is located in Seismic Performance Zone 1 and the corresponding site–specific zonal acceleration ratio, A, is 0.05. Given this assessment, and in accordance with Section 4.4.5.1 of the *CHBDC*, no seismic analysis is required for structures located in Seismic Performance Zone 1.

6.5 Lateral Earth Pressures

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

The following recommendations are made concerning the design of walls for this site. It should be noted that these design recommendations and parameters assume level backfill and ground surface behind the walls.

Where there is sloping ground behind the walls, the coefficient of lateral earth pressure must be adjusted to account for the slope.

- Select, free draining granular fill meeting the specifications of OPSS.Prov.1010 (Aggregates) Granular ‘A’ or Granular ‘B’ Type II, but with less than 5 per cent passing the No. 200 sieve, should be used as backfill behind the walls. Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Compaction (including type of equipment, target densities, etc.) should be carried out in accordance with SP 105S21 (Compacting). Other aspects of the granular backfill requirements with respect to sub-drains and frost taper should be in accordance with OPSD 3101.150 (Walls, Abutment, Backfill, Minimum Granular Requirement) or OPSD 3121.150 (Walls, Retaining, Backfill, Minimum Granular Requirement), as applicable.
- 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 2.6 m behind the back of the wall (in accordance with Figure C6.20 (a) of the Commentary to the CHBDC). For unrestrained walls, granular fill should be placed within the wedge shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical (1.5H:1V) extending up and back from the rear face of the footing (in accordance with Figure C6.20(b) of the Commentary to the CHBDC). The pressures are based on the proposed embankment fill material and the following parameters (unfactored) may be used:

Fill Type	Unit Weight	Coefficients of Static Lateral Earth Pressure	
		At-Rest, K_o	Active, K_a
Granular ‘A’	22 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.

6.6 Approach Embankments

The replacement bridge will be constructed on a new alignment located 18 m to the south of the existing structure. The grade of the new embankments will be up to 4.3 m above the existing ground surface.

The analyses assume that the approach embankments will be constructed of granular fill and that the peat, where encountered, will be removed from below the footprint of the new approach embankments. Beyond the abutments, rock fill could also be used for embankment construction. The geometry of the proposed approach embankments, existing ground surface and existing river bed included in the analyses are based on the information from the GA drawing and cross-sections provided by LEA. The piezometric conditions required in the stability and settlement analyses are based on the groundwater level as encountered during the subsurface investigation.

6.6.1 Stability

Analyses were performed on the critical sections of the proposed approach embankments for conditions during and after construction to assess the stability for the proposed embankment height, geometry and soil stratigraphy. The critical embankment sections at this site are the front slope, where the base of cohesive deposits is at the lowest elevation, and the side slope, where the grade raise for the proposed embankment widening is the highest.

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 Factor of Safety (FoS) of numerous potential failure surfaces was computed in order to establish the minimum FoS. The FoS is defined as the ratio of the forces tending to resist failure to the driving forces tending to cause failure. A target minimum FoS of 1.3 is normally adopted for the design of embankment slopes under static conditions at the end of construction. This FoS is considered adequate for the embankments at this site considering the design requirements and the field data available. The stability analyses were performed to check that the target minimum FoS was achieved for the design embankment height and geometries. In general, circular slip surfaces were analysed in the design.

For the rock fill, granular fill, existing fill and cohesionless deposits, effective stress parameters were employed in the analysis assuming drained conditions and the parameters were estimated from empirical correlations using the results of the in-situ SPT ‘N’-values. The correlations proposed by Terzaghi and Peck (1967) were employed and the results were tempered by engineering judgment based on precedent experience in similar soils.

For the cohesive soils, total stress parameters were employed in the analysis 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).

Summarized below are the simplified stratigraphy and the associated strengths and unit weights employed for the different soil types in the proposed approach areas. The slope stability analyses model geometry and stratigraphy are shown on Figures 1 and 2 for the critical sections identified above.

Soil Deposit	Bulk Unit Weight (kN/m^3)	Effective Friction Angle (°)	Undrained Shear Strength (kPa)
New Rock Fill	19	40	-
New Granular 'B' Type I or II Fill	21	35	-
Existing Granular Fill	21	32	-
Soft to Firm Clayey Silt to Silty Clay	17	-	27
Firm Clayey Silt to Silt	18	-	40
Very Loose to Compact Sandy Silt to Sand and Silt	21	32	-
Hard Sandy Clayey Silt to Clayey Silt with Sand Till	21	35	-

6.6.1.1 Results of Analysis

The results of the stability analysis indicate that for the embankments constructed of granular fill, the critical sections have a FoS greater than the target 1.3 for embankments constructed at slopes of 2H:1V. Therefore, stability mitigation measures will not be required for this site. The results of the analysis are shown on Figures 1 and 2 for the west front slope and the south east side slope, respectively. The results of the stability analysis for the embankments constructed of rock fill at 1.25H:1V side slopes also indicate a FoS greater than 1.3.

6.6.2 Settlement

Settlement of the approach embankments can be expected as a result of the loading from the up to 4.3 m high new embankment fill and from the loading associated with the sub-excavation/replacement of the up to 0.9 m thick peat deposit in the embankment footprint area on the compressible foundation soils at this site. Settlement of the cohesionless deposits is expected to occur during or shortly after construction. Time-dependent consolidation settlement of the cohesive deposit under the approach embankments will occur but is also expected to primarily occur during construction as the deposits are considered to be overconsolidated. In addition, settlement of the new embankment fill will also occur.

To estimate the magnitude of the expected settlements, analyses were carried out on the critical sections of the proposed approach embankments using the commercially available program Settle^{3D} (Version 2.016) produced by Rocscience Inc. as well as hand calculations. The rate of settlement of the cohesive foundation soils was assessed using Terzaghi's one-dimensional consolidation theory. The model geometry and stratigraphy at the abutments are shown on Figures 1 and 2, as used for the stability analyses. For the settlement analyses at each approach, the critical sections were assessed for the new embankment height and geometry. The sources of settlement were considered to include:

- immediate settlement of the cohesionless deposits;
- time-dependent consolidation of the cohesive deposits; and
- self-weight compression of the embankment rock fill, if used for embankment construction.

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

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).

Soil Type	Location	Thickness (m)	γ (kN/m^3)	E (MPa)
New Rock Fill	West Approach	3.9	19	-
	East Approach	4.3		
New Granular 'B' Type II or I Fill	West Approach	3.9	21	10
	East Approach	4.3		
Silty Clay to Clayey Silt (soft to firm)	West Approach	2.4	17	see below
	East Approach	3.5		
Clayey Silt to Silt (firm)	West Approach	2.7	18	see below
	East Approach	2.2		
Sandy Silt to Sand and Silt (very loose to compact)	West Approach	4.3	21	10
	East Approach	4.3		
Sandy Clayey Silt to Clayey Silt with Sand (Till) (hard)	West Approach	>4.1	21	90
	East Approach	>7.0		

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

$$\sigma_p' = s_{u(mob)} / 0.22$$

where: σ_p' = pre-consolidation stress (kPa)

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

The consolidation settlement of the cohesive deposits (organic clay, silty clay to clayey silt and clayey silt to silt) was assessed using the results of the laboratory index testing to estimate the deformation parameters (i.e. recompression and compression indices) using empirical correlations proposed in literature by Koppula (1986).

The coefficient of consolidation, c_v (cm^2/s), required in the time-rate settlement analysis was estimated from the NAVFAC (1986) correlation with liquid limit assuming the deposits are over consolidated.

Soil Type	σ_{vo}' (kPa)	σ_p' (kPa)	OCR	e_o	C_c	C_r	c_v (cm^2/s)
Silty Clay to Clayey Silt	22	115	6.7	0.8	0.4	0.04	0.025
Clayey Silt to Silt	54	180	3.3	0.6	0.3	0.03	-

Where: σ_{vo}' initial vertical effective stress (kPa)
σ_p' pre-consolidation stress (kPa)
OCR overconsolidation ratio
e_o initial void ratio (based on water content)
C_c compression index (based on water contents and liquid limits)
C_r recompression index (based on water content and liquid limits)
C_v coefficient of consolidation (based on water content and liquid limits)

6.6.2.1 Rock Fill Settlement

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

Short-Term Rock Fill Settlement

The magnitude of short-term post-construction settlement associated with compacted and end-dumped rock fill may be estimated in accordance with the MTO's 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 MTO's 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.

6.6.2.2 Settlement Criteria

Based on MTO's "Embankment Settlement Criteria for Design Final Draft", dated March 2, 2010, the following post-construction settlement and differential settlement criteria are considered acceptable for settlements 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	75

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

6.6.2.3 Results of Analysis

The results of the analysis indicate that up to 35 mm of immediate settlement of the cohesionless deposits will occur during embankment construction. Further, up to 50 mm of consolidation settlement of the cohesive deposits is expected to occur. However, based on the c_v value given above, it is estimated that the consolidation settlement will be completed during or immediately following construction.

If the embankment is constructed of granular fill (i.e., Granular 'B' Type I or Type II), then the fill settlement itself is not a concern as the settlement of granular fill, that is properly placed and compacted, is considered nominal and would occur during construction.

If rock fill is used for embankment construction, approximately 25 mm of post-construction settlement will occur (20 mm short term and 5 mm long term) of which 20 mm is expected to occur in the first 6 months after construction. It should be noted that rock fill is not to be used for backfill to the abutments, but could be used beyond the abutment backfill zone.

Since the total post-construction settlement is less than the criteria of 25 mm, settlement mitigation measures are not required.

6.7 Construction Considerations

6.7.1 Subgrade Preparation and Embankment Construction

For the bridge approach embankments, removal of the peat and organic clay is recommended prior to construction of the new approach embankments. Also, all softened/loosened soils should be stripped from below the approach embankment, prior to placement of new fill.

Fill for construction of the new embankments should consist of a Granular 'B' Type I or Type II meeting the specifications of OPSS.Prov.1010 (Aggregates) or rock fill. The embankment fill for the realigned Highway 11 should be placed and compacted in accordance with SP 105S21 (Compacting) and SP 206S03 (Earth, or Rock, Excavation and Grading), as applicable. Where new fill is to tie into existing fill along and beyond the approaches, the new fill should be "keyed-in" or benched into the existing fills, in accordance with OPSD 208.010 (Benching of Earth Slopes). Side slopes in granular fill should be no steeper than 2H:1V and in rock fill, no steeper than 1.25H:1V.

All granular fill should be placed in lifts with loose thickness not exceeding 300 mm and compacted to at least 95 per cent of the standard Proctor maximum dry density. Inspection and field density testing should be carried out by qualified personnel during fill placement operations to ensure that appropriate materials are used and that adequate levels of compaction have been achieved.

Rock fill, if utilized, 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 below the groundwater table.

The abutment front slopes and side slopes adjacent to the river require erosion protection in accordance with OPSS 511 (Rip Rap, Rock Protection and Granular Sheeting) and SP 511S01 (Rip Rap, Rock Protection, Gravel Sheeting). Erosion protection should be placed on the slopes to at least 0.5 m above the design high water level. Subject to modifications based on the hydrology reports (by others), erosion protection could consist of a minimum 0.6 m thick layer of R-10 Rip Rap (180 mm diameter as per OPSS.PROV 1004 (Aggregates - Miscellaneous), rock protection or concrete slope paving. The designer should address the potential for scour below the pile caps in the design of the bridge foundations.

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

The cohesive soils that will be exposed within the pile cap excavation at the pile cap foundation subgrade level will be susceptible to disturbance from construction traffic and/or ponded water. To limit the effects of this disturbance, it is recommended that a concrete working slab be placed on the subgrade within four hours after preparation, inspection and approval of the pile cap subgrade. It is anticipated that the piles would be driven through the working slab in this case. This requirement can be addressed with a note on the General Arrangement drawing and/or with an NSSP; an example of which is included in Appendix C. If a tremie plug is to be used for groundwater control within the pile cap excavation (see Section 6.7.3), then the tremie plug can be placed directly on the subgrade below the water level (assuming that the piles will be driven first through water).

6.7.2 Control of Artesian Groundwater Pressure during Piling

Given that artesian groundwater was encountered during the subsurface investigation in Borehole VS3, we recommend that a drainage/filter blanket consisting of a minimum 0.5 m thick layer of concrete fine aggregate (OPSS 1002, Aggregates - Concrete) be placed below the underside of the west abutment pile cap (or tremie plug -see below) encasing all the piles. The base of the filter blanket, 0.5 m below the underside of pile cap, will extend to Elevation 233.0 m at the west abutment. The concrete fine aggregate layer should extend a minimum of 0.5 m horizontally beyond each of the pile caps or be contained within a cofferdam. Further, the excavation at the front of the abutments (i.e., towards the river) should be backfilled with free draining material such as OPSS.Prov.1010 (Aggregates) Granular 'B' Type II, extending at least 0.5 m horizontally from the front face of the abutment.

6.7.3 Excavations and Groundwater Control

The proposed excavation depths for the removal of peat and organic clay and for construction of the pile caps are presented below, together with the depth of the excavations below the design river High Water Level (provided by LEA) at Elevation 236.1 m.

Element	Base of Excavation		
	Elevation (m)	Depth Below Ground Surface (m)	Depth Below High Water Level (m)
West Abutment (for pile cap construction)	232.5[1,2]	4.0	3.6
East Abutment (for pile cap construction)	233.0[1]	3.1	3.1
West Approach Embankment (for peat removal)	236.5	0.5	n/a[3]
East Approach Embankment (for peat removal)	235.8	0.7	0.3

Notes:
1. Base of excavation assumes that with pile caps constructed at Elevation 234.5 m, the base extends 1.5 m below underside of the pile cap to allow for the installation of a 1.5 m thick tremie concrete plug to allow installation of a granular filter blanket at the west abutment.
2. Base of excavation at the east abutment extends 0.5 m below underside of the tremie concrete plug to allow installation of a granular filter blanket.
3. The base of excavation will be above the river High Water Level (Elev. 236.1 m).

The tremie plug must be in place not only for hydraulic/shoring requirements but also to achieve the new frost protection depth.

Temporary excavations to remove the peat and/or to construct the pile caps should be made with side slopes no steeper than 1H:1V above the water table and 3H:1V below the water table through the peat and upper cohesive deposits.). The width of the peat excavation should extend to a lateral distance from a line projected down from the crest of the widened embankment at the projected embankment side slope (1.25H:1V for rock fill and 2H:1V for granular fill) to the base of the sub-excavation. Excavations for this purpose should be in accordance with OPSS 902 (Excavating and Backfilling – Structures). If open-cut excavations are adopted, the excavations should be carried out in accordance with the guidelines in the latest version of the *Occupational Health and Safety Act* (OHSA) for Construction Activities. The peat and upper cohesive soils would be classified as Type 3 soil, according to the OHSA.

Removal of the organic deposits beyond the abutment locations is not anticipated to be below the groundwater level and therefore groundwater control is not required in these areas. Surface water should be directed away from the excavations at all times.

Given the depth of the pile cap excavation below the water level as indicated in the table above, groundwater control will be required and could be in the form of a sheet-pile cut-off wall or cofferdam advanced to an appropriate depth to control groundwater inflow from the river. At this site, we recommend placement of a tremie concrete plug within the sheet-pile cofferdam to guard against the basal heave method of failure. The tremie concrete plug should be a minimum of 3.0 m thick and should have a minimum compressive strength of 1 MPa. Once the tremie plug is in place, water can be pumped out of the excavation for construction of the pile caps. Piles should be driven prior to placement of the tremie plug. The filter blanket may be installed in the wet below the tremie plug. A balanced head of water should be maintained on both sides of the cofferdam until the tremie concrete plug is in place to prevent basal heave or piping.

6.7.4 Temporary Excavation Support Systems

Temporary shoring is required to construct the pile caps and temporary roadway protection systems are required to allow for peat removal and construction of the new approach embankments adjacent to the existing highway embankment, which will need to remain in operation during construction of the new bridge.

Temporary excavation support systems should be designed and constructed in accordance with OPSS 539 (Temporary Protection Systems). The lateral movement of the temporary shoring systems should meet Performance Level 2 as specified in OPSS 539. The contractor is responsible for the complete detailed design of the shoring/protection systems.

The temporary support system could consist of either driven steel sheet piling (for the cofferdam and temporary roadway protection) or soldier piles and lagging (temporary roadway protection) where the H-piles would be driven to a suitable depth and horizontal lagging installed as the excavation proceeds. If soldier piles and lagging is selected, pile installation should be in accordance with OPSS 903 (Deep Foundations). Support to the cofferdam could be in the form of struts and walers; bracing is likely not required for the temporary roadway protection, depending on the unsupported height of the excavation required for backfilling behind the cofferdam.

The design of braced sheet pile or soldier pile and lagging walls should be based on a rectangular earth pressure distribution using the design parameters given below.

For a braced excavation in granular fill and native cohesionless soils, the unfactored rectangular earth pressure distribution (p in kN/m^2; constant with depth), can be calculated as follows (CFEM 2006):

	P	=	$K_a(0.65\ \gamma\ H + q)$
where	K_a	=	active coefficient of earth pressure
	H	=	the total depth of the excavation (m)
	γ	=	soil unit weight (kN/m^3)
	q	=	surcharge for traffic and other loading (kN/m^2)

For a braced excavation in soft to firm cohesive soil, the unfactored rectangular earth pressure distribution (p in kN/m^2; varying with depth), can be calculated as follows (CFEM 2006):

	P	=	0 at ground surface increasing linearly to a depth of 0.25 H_T to:
	p	=	$\gamma\ H_T - 4\ m\ S_u$ at 0.25 H_T and from 0.25 H_T to H_T below ground surface
where	H_T	=	the total depth of the excavation (m)
	γ	=	soil unit weight (kN/m^3)
	q	=	surcharge for traffic and other loading (kN/m^2)
	m	=	0.4 if an extensive soft clay layer underlies the excavation
			1.0 if more resistant layer is present at the excavation base
	S_u	=	undrained shear strength (kN/m^2).

The support systems may be designed using the following parameters:

Soil Type	Coefficient of Earth Pressure			Internal Angle of Friction	Unit Weight	Undrained Shear Strength
	Active, K_a	At Rest, K_o	Passive, K_p	(ϕ, degrees)	(γ, kN/m^2)	(S_u, kPa)
New Granular 'B' Type I or II (Fill)	0.27	0.43	3.7	35	21	-
New Rock Fill	0.22	0.36	4.5	40	19	-
Silty Clay to Clayey Silt*	0.41	0.58	2.5	25	17	-
	1.0	1.0	1.0	-	17	27
Clayey Silt to Silt*	0.38	0.55	2.7	27	18	-
	1.0	1.0	1.0	-	18	40
Sandy Silt to Sand and Silt	0.31	0.47	3.3	32	19	-
Sandy Clayey Silt to Clayey Silt with Sand (Till)	0.27	0.43	3.7	35	21	-

Note:
*Temporary Protection Systems should be designed based on the more conservative (higher) earth pressure value.

The total passive resistance below the base of the excavation within the sheet pile cofferdam should be calculated based on the values of K_p given above and reduced by an appropriate factor of safety which considers the allowable wall movement as extrapolated from Figure C6.16 of the CHBDC (2006) to account for the fact that a large strain would be required for full mobilization of the passive resistance.

The earth pressure coefficients noted above are based on a horizontal surface adjacent to the excavation. If sloped surfaces are present, the coefficients should be adjusted accordingly.

6.7.5 Obstructions

The soils at this site are glacially derived and as such contain coarse gravel, cobbles and possibly boulders as noted in the Record of Borehole sheets, which could affect the installation of deep foundations and/or temporary roadway protection systems. An NSSP should be included in the Contract Documents to identify to the contractor the possible presence of cobbles and/or boulders within the overburden soils, an example of which is included in Appendix C.

6.7.6 Existing Structure Monitoring

We recommend that the abutments of the existing structure be monitored for settlement and lateral movement during the new construction, especially during installation of temporary shoring or roadway protection, excavation for the new abutments/peat removal and during pile driving (during advancement through cobbles and boulders) for the following reasons:

- the old age and deteriorated condition of the existing structure;
- the existing abutments are founded on timber piles;
- the close proximity of the existing and proposed abutments;
- the requirement for staged construction; and
- the requirement for the existing structure to carry traffic during construction of the new structure.

The foundation monitoring should be carried out by a qualified foundations consultant reporting to the Contract Administrator.

6.7.7 Analytical Testing for Construction Materials

The analytical test results on a sample of river water are presented in Table B1. The suite of parameters tested is intended to allow the structural engineer to assess the requirements for the appropriate type of cement to be used in construction and the need for corrosion protection.

7.0 CLOSURE

This Foundation Design Report was prepared by Mr. Evan Childerhose, P.Eng., and the technical aspects were reviewed by Ms. Sarah Coyne, P.Eng., Associate. Messrs. Fintan Heffernan, P.Eng., and Jorge Costa, P.Eng., Principal, Designated MTO Foundations Contacts for Golder, conducted independent quality control reviews of this report.

Signature Page

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EC/SEMC/FJH/JMAC//kp

http://capws.golder.com/sites/capws2/p111910008mtosixbridgesnearhearst/reports/reports/5valentine river/2c final detail report/11-1191-0008-5 rpt 13oct31 final fidr valentine river.docx

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Koppula, S.D. 1986. Discussion: Statistical Estimation of Compression Index, Geotechnical Testing Journal, ASTM, Vol. 4, No. 2, pp. 68-73.

Mesri, G., 1975. Discussion on New Design Procedure for Stability of Soft Clays. ASCE Journal of the Geotechnical Engineering Division, Vol. 101, GT4, pp. 409-412.

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Terzaghi, K., 1955. Evaluation of Coefficients of Subgrade Reaction/ Geotechnique, Vol. 5, No. 4, pp. 297-326. Discussion in Vol. 6, No. 2, pp. 94-98.

Terzaghi, K. and Peck, R.B. 1967. Soil Mechanics in Engineering Practice, 2nd Edition, John Wiley and Sons, New York.

Unified Facilities Criteria, U.S. Navy. 1986. NAVFAC Design Manuals 7.01 and 7.02. Soil Mechanics, Foundation and Earth Structures. Alexandria, Virginia.

ASTM International

ASTM D1586 Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils

ASTM D2573 Standard Test Method for Field Vane Shear Test in Cohesive Soil

Commercial Software

GeoStudio (Version 7.19) by Geo-Slope International Ltd.

LPile (Version 5.0) by Ensoft Inc.

Settle^{3D} (Version 2.0) by Rocscience Inc.

Ministry of Transportation, Ontario

Embankment Settlement Criteria for Design. March 2010.

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

Northern Region Engineering Directive NRE 98-200. Northern Region Embankment Design Guidelines. 2002.

Structural Manual. Quality and Standards, Transportation Engineering Branch, Bridge Office, Design Section. April 2008.

Ministry of Transportation Ontario Special Provisions

SP 105S21	Compacting
SP 206S03	Earth Excavation, Grading
SP 511S01	Rip Rap, Rock Protection, Gravel Sheeting

Ontario Provincial Standard Drawings

OPSD 203.010	Embankments Over Swamp, New Construction
OPSD 208.010	Benching of Earth Slopes
OPSD 3000.100	Steel H-Driving Shoe
OPSD 3090.100	Foundation, Frost Penetration Depths for Northern Ontario
OPSD 3101.150	Walls, Abutment, Backfill, Minimum Granular Requirement
OPSD 3121.150	Walls Retaining, Backfill Minimum Granular Requirement
OPSD 3101.200	Walls, Abutment, Backfill, Rock

Ontario Provincial Standard Specifications

OPSS 209	Construction Specification for Embankments Over Swamps and Compressible Soils
OPSS 501	Construction Specification for Compacting
OPSS 511	Construction Specification for Rip Rap, Rock Protection and Granular Sheeting
OPSS 539	Temporary Protection Systems
OPSS 802	Construction Specification for Topsoil
OPSS 804	Construction Specification for Seed and Cover
OPSS 902	Construction Specification for Excavating and Backfilling - Structures
OPSS 903	Construction Specification for Deep Foundations
OPSS.PROV 1004	Material Specification for Aggregates – Miscellaneous
OPSS.PROV 1010	Material Specification for Aggregates – Base, Subbase, Select Subgrade and Backfill Material

OPSS 1002 Material Specification for Aggregates – Concrete

Ontario Water Resources Act

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

Table 1: Evaluation of Foundation Alternatives

Foundation Type	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Driven steel H-piles	1	■ Straightforward construction. ■ Higher axial resistance compared to shallow foundation. ■ Suitable for integral abutment design.	■ Requires excavation below groundwater level for pile cap construction. ■ Potential for piles “hanging-up” on cobbles and boulders within till deposit, but likely easier to advance than caissons. ■ Filter blanket required to account for artesian groundwater conditions at west abutment.	■ Relative costs lower than for caissons. ■ Cost of temporary roadway protection/ dewatering for pile cap.	■ Potential for not achieving design resistance above the design pile tip elevation due to the presence of cobbles and boulders.
Driven steel Tube Piles	2	■ Straightforward construction. ■ Higher axial resistance compared to shallow foundation.	■ Requires excavation below groundwater level for pile cap construction. ■ Greater potential for piles “hanging-up” on cobbles and boulders within till deposit and for deflecting from their intended alignment compared to H-piles. ■ Potential larger void created along pile compared to H-piles which could create a preferential pathway for artesian conditions. ■ Not suitable for integral abutment design. ■ Not readily adopted by MTO	■ Relative costs lower than for caissons. ■ Cost of temporary roadway protection/ dewatering for pile cap.	■ Potential for not achieving design resistance at above the design pile tip elevation due to the presence of cobbles and boulders.

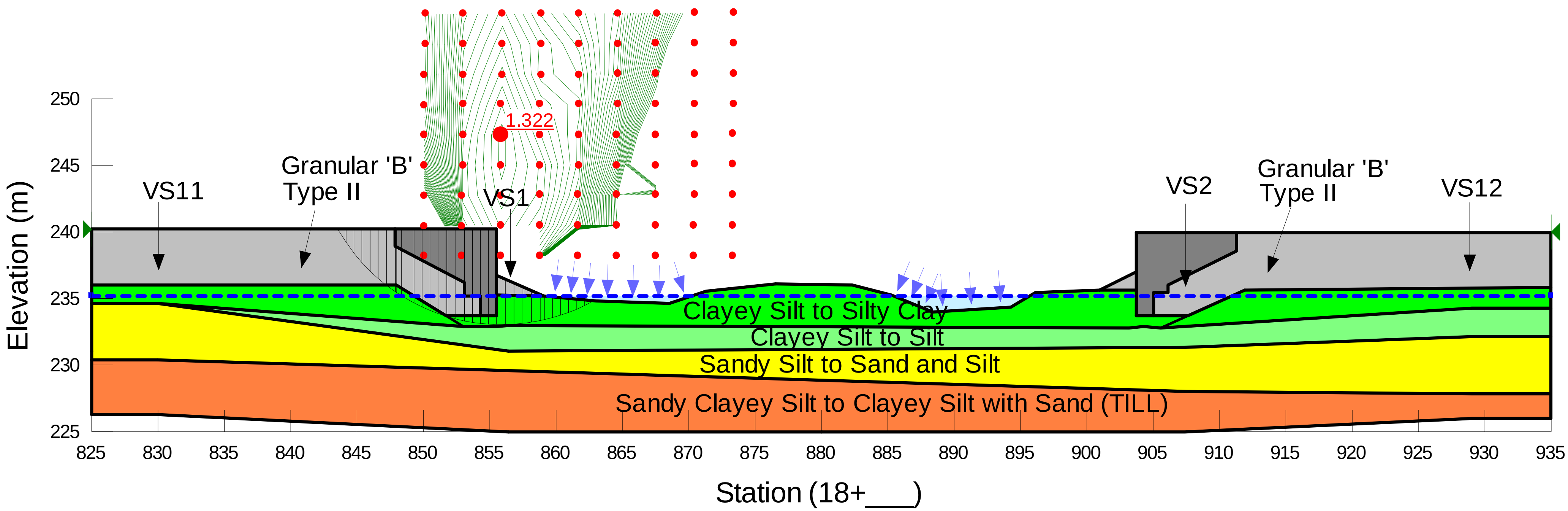
Foundation Type	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Caissons	3	■ Higher axial resistances compared to steel H-piles or tube piles or shallow foundations. ■ Possible elimination of pile cap and associated excavation as caissons can be extended to underside of bridge.	■ Potential for difficulties penetrating cobbles and boulders compared to piles. ■ Potential for issues with artesian groundwater conditions during caisson advancement. ■ Not suitable for integral abutment design. ■ Require caisson protection and likely a head of water to mitigate subsoil inflow due to groundwater conditions.	■ Relative costs higher than steel H-piles or pipe piles or shallow foundations.	■ Potential for difficulties reaching the required termination depth due to the presence of cobbles and boulders. ■ Potential for difficulties with respect to artesian groundwater conditions.
Shallow Foundations	NF	■ Conventional construction.	■ Requires deeper excavation and dewatering (cofferdam) adjacent to the river to allow for construction in-the-dry compared to pile caps. ■ Axial resistances too low for this option to be technically feasible. ■ Potential for settlement or differential settlement between abutments. ■ Not suitable for integral abutment design.	■ Typically lower cost than deep foundations; however increased cost for deeper excavation and dewatering to greater depths than excavation for pile caps.	■ Potential for instability of, or need to, advance shoring with deeper excavation adjacent to existing highway and river compared to deep foundations.

Note: NF – Not Feasible

Valentine River Bridge – Highway 11
Stability Analysis (West Front Slope)

Figure 1

Material Name	Unit Weight (kN/m^3)	Friction Angle (º)	Cohesion (kPa)
Granular B Type II	21	35	-
Clayey Silt to Silty Clay	17	-	27
Clayey Silt to Silt	18	-	40
Sandy Silt to Sand and Silt	21	32	-
Sandy Clayey Silt to Clayey Silt with Sand (TILL)	21	35	-



Date: October 2013

Project No: 11-1191-0008

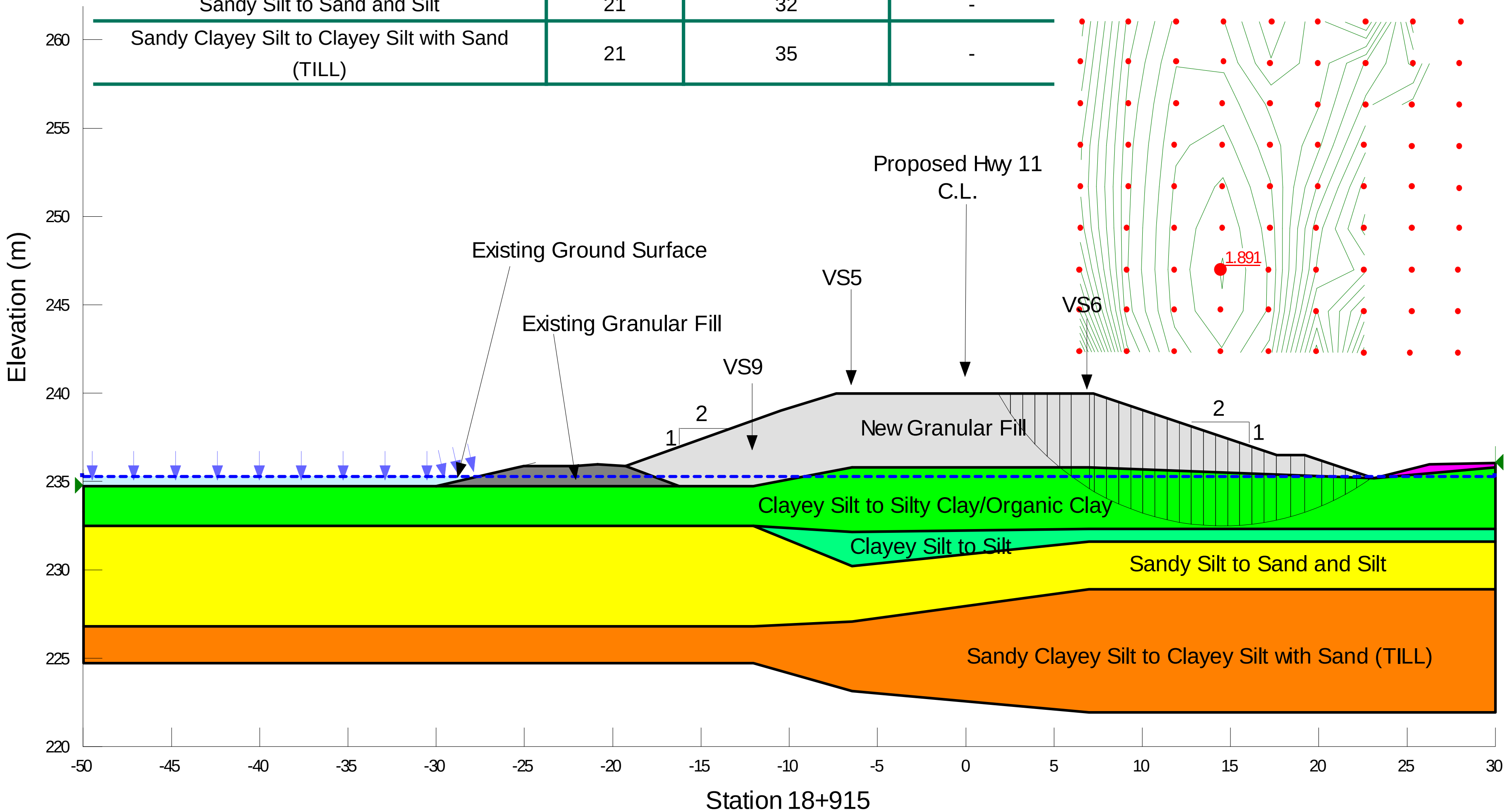
Analysis By: EC **Reviewed By:** SEMC

Golder Associates

Valentine River Bridge – Highway 11
Stability Analysis (South East Side Slope)

Figure 2

Material Name	Unit Weight (kN/m³)	Friction Angle (º)	Cohesion (kPa)
Granular B Type II	21	35	-
Clayey Silt to Silty Clay/Organic Clay	17	-	27
Clayey Silt to Silt	18	-	40
Sandy Silt to Sand and Silt	21	32	-
Sandy Clayey Silt to Clayey Silt with Sand (TILL)	21	35	-



Date: October 2013

Project No: 11-1191-0008

Analysis By: EC **Reviewed By:** SEMC

Golder Associates

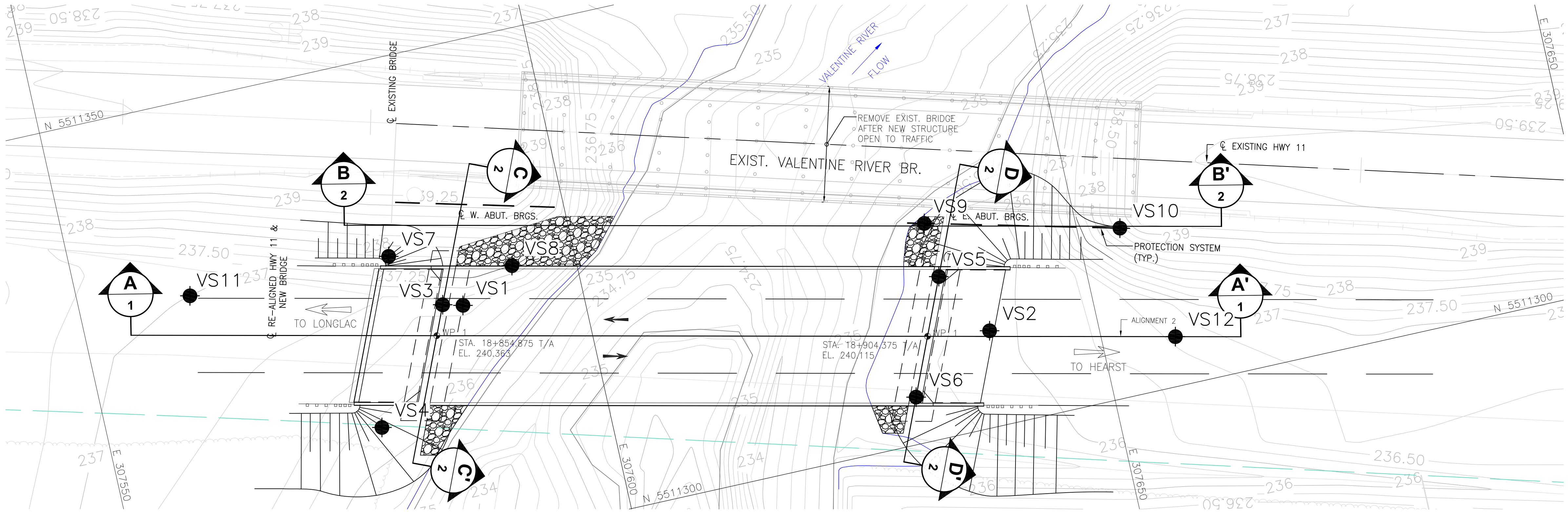
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.
WP No. 5150-05-00

HIGHWAY 11
VALENTINE RIVER BRIDGE
BOREHOLE LOCATIONS AND SOIL STRATA

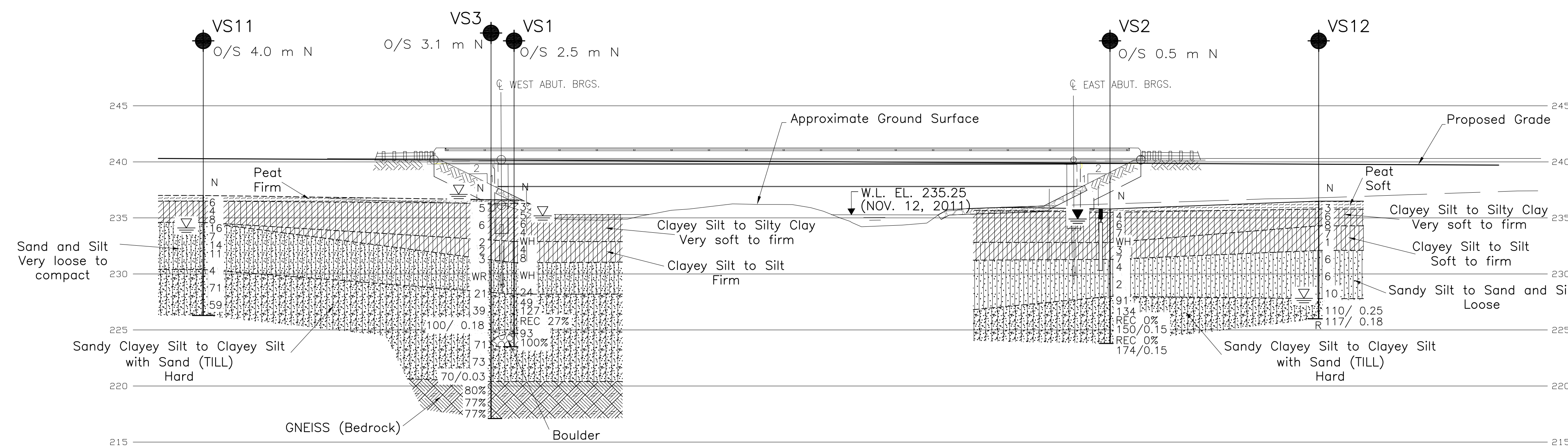
SHEET

Golder Associates Ltd.
SUDBURY, ONTARIO, CANADA



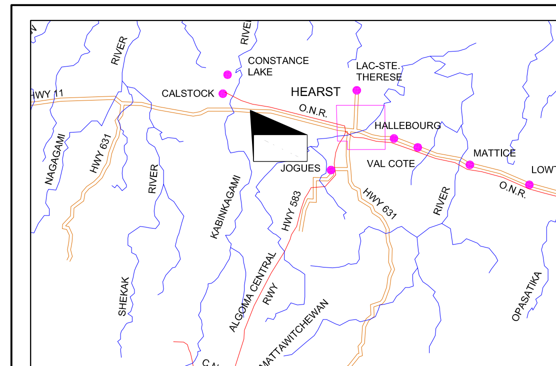
PLAN

SCALE 5 0 5 10 m



A-A' 1 CENTERLINE PROFILE
HIGHWAY 11

HORIZONTAL SCALE 5 0 5 10 m
5 0 5 10 m VERTICAL SCALE



KEY PLAN
SCALE 20 0 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
100%	Rock Quality Designation (RQD)
	WL upon completion of drilling
	WL in piezometer, measured on July 5, 2013
R	Refusal

BOREHOLE CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
VS1	236.6	5511323.7	307587.9
VS2	235.8	5511309.6	307639.2
VS3	236.5	5511324.2	307585.9
VS4	236.5	5511313.5	307577.2
VS5	235.8	5511316.1	307635.4
VS6	236.1	5511304.7	307630.5
VS7	237.6	5511330.2	307581.7
VS8	237.4	5511326.6	307593.6
VS9	235.5	5511321.6	307635.1
VS10	239.4	5511316.9	307654.3
VS11	237.0	5511330.7	307561.2
VS12	236.5	5511305.0	307657.3

REFERENCE

Base plans provided in digital format by LEA Consulting Ltd., drawing file no. 8960-VAL-S01(Ehx-40).dwg, received May. 30, 2013 and Inroads-X-Sections.dwg, received August 21, 2013.

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.

LICENSED PROFESSIONAL ENGINEER S.E.M. COYNE 90471921 OCT 31, 2013 PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER J.M.A. COSTA OCT 31, 2013 PROVINCE OF ONTARIO

NO.	DATE	BY	REVISION

Geocres No. 42G-48

HWY. 11		PROJECT NO. 11-1191-0008	DIST.
SUBM'D. EC	CHKD.	DATE: OCT 2013	SITE: 39W-010
DRAWN: TB	CHKD. SEMC	APPD. JMAC	DWG. 1

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

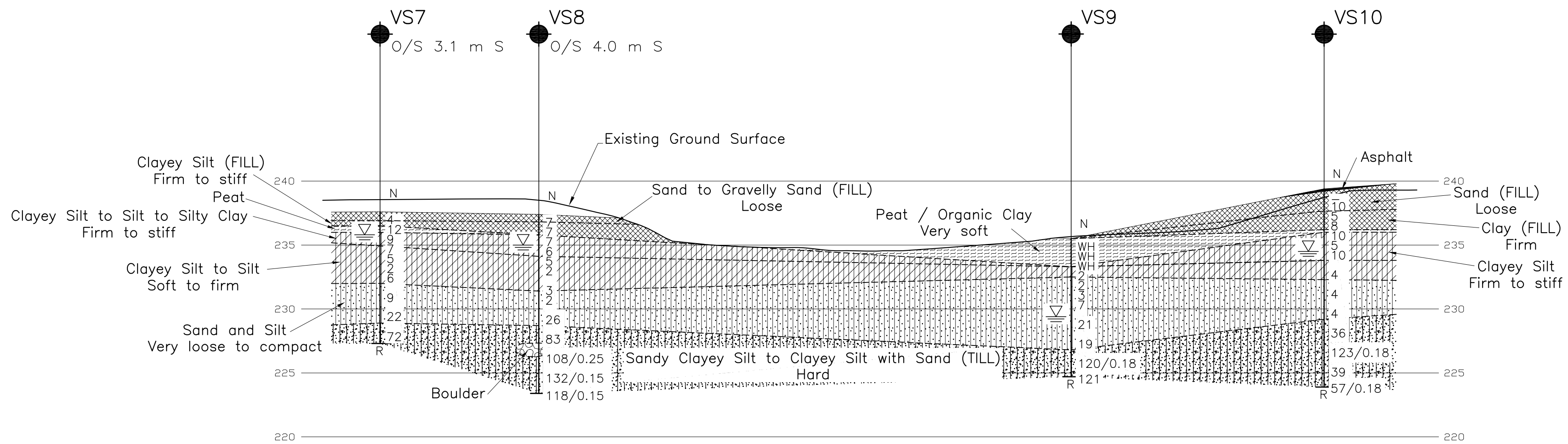
CONT No.
WP No. 5150-05-00

HIGHWAY 11
VALENTINE RIVER BRIDGE

SOIL STRATA

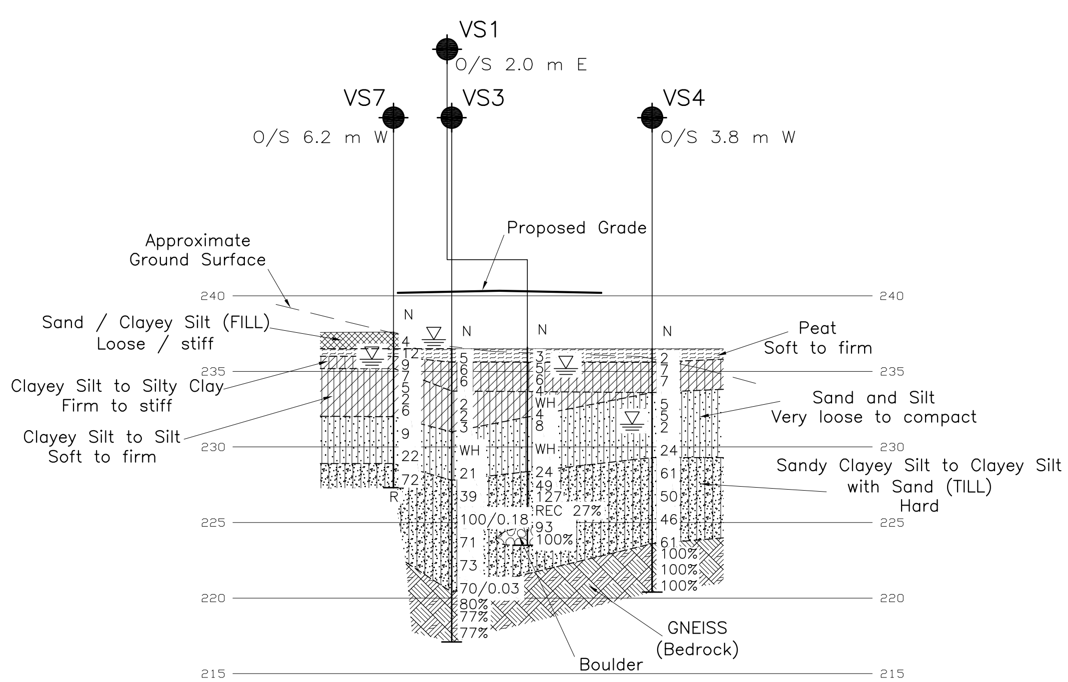
SHEET

Golder Associates Ltd.
SUDBURY, ONTARIO, CANADA



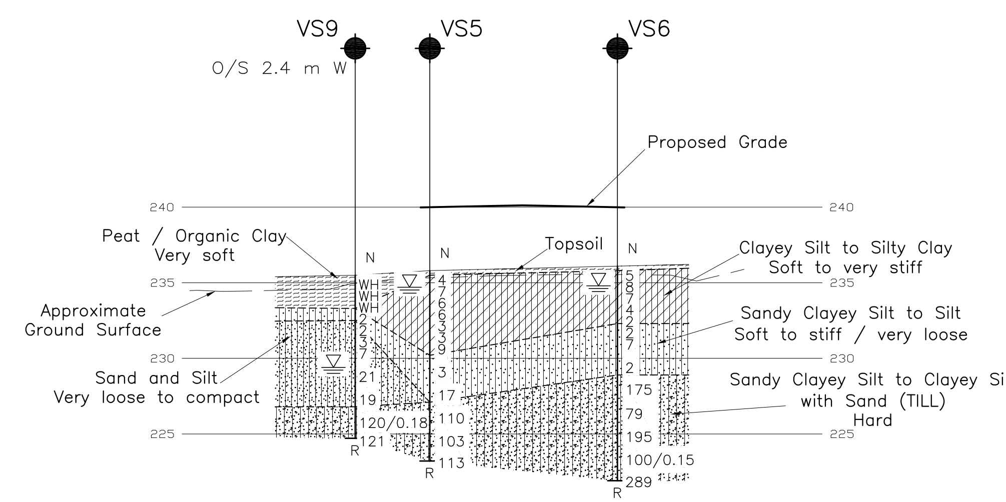
B-B' 1 PROFILE
HIGHWAY 11

HORIZONTAL SCALE 5 0 5 10 m
5 0 5 10 m VERTICAL SCALE



C-C' 1 WEST ABUTMENT SECTION
HIGHWAY 11

HORIZONTAL SCALE 5 0 5 10 m
5 0 5 10 m VERTICAL SCALE



D-D' 1 EAST ABUTMENT SECTION
HIGHWAY 11

HORIZONTAL SCALE 5 0 5 10 m
5 0 5 10 m VERTICAL SCALE

LEGEND

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

BOREHOLE CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
VS1	236.6	5511323.7	307587.9
VS3	236.5	5511324.2	307585.9
VS4	236.5	5511313.5	307577.2
VS5	235.8	5511316.1	307635.4
VS6	236.1	5511304.7	307630.5
VS7	237.6	5511330.2	307581.7
VS8	237.4	5511326.6	307593.6
VS9	235.5	5511321.6	307635.1
VS10	239.4	5511316.9	307654.3

NOTES

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LICENSED PROFESSIONAL ENGINEER S.E.M. COYNE 90471921 OCT 31, 2013 PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER J.M.A.COSTA OCT 31, 2013 PROVINCE OF ONTARIO

NO.	DATE	BY	REVISION

Geocres No. 42G-48

HWY. 11		PROJECT NO. 11-1191-0008	DIST.
SUBM'D. EC	CHKD.	DATE: OCT 2013	SITE: 39W-010
DRAWN: TB	CHKD. SEMC	APPD. JMAC	DWG. 2

APPENDIX A

Record of Boreholes (VS1 to VS12)
Record of Drillholes (VS3 and VS4)

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
υ	Poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma - u$)
σ'_{vo}	initial effective overburden stress
σ_1, σ_2, σ_3	principal stress (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

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

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

(a) Index Properties (continued)

w	water content
w_l 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 $= \sigma'_p / \sigma'_{vo}$

(d) Shear Strength

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

Notes:
1. $\tau = c' + \sigma' \tan\phi'$
2. shear strength = (compressive strength)/2

LIST OF ABBREVIATIONS

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

I. SAMPLE TYPE

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

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

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

Dynamic Cone Penetration Resistance; N_d:

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

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

Piezo-Cone Penetration Test (CPT)

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

III. SOIL DESCRIPTION

(a) Non-Cohesive (Cohesionless) Soils

Density Index **Relative Density**	**N** **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	Sand and Gravel
	With (cohesive)	Silty Clay with sand / Clayey Silt with sand

WEATHERINGS STATE

Fresh: no visible sign of weathering

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

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

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

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

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

BEDDING THICKNESS

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

JOINT OR FOLIATION SPACING

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

GRAIN SIZE

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

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

CORE CONDITION

Total Core Recovery (TCR)

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

Solid Core Recovery (SCR)

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

Rock Quality Designation (RQD)

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

DISCONTINUITY DATA

Fracture Index

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

Dip with Respect to Core Axis

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

Description and Notes

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

Abbreviations

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

RECORD OF BOREHOLE No VS1

 METRIC

PROJECT 11-1191-0008
W.P. 5150-05-00
LOCATION N 5511323.7; E 307587.9
ORIGINATED BY ID
DIST HWY 11
BOREHOLE TYPE 108 mm ID Continuous Flight Hollow Stem Augers, NW Casing, NQ Coring
COMPILED BY EC
DATUM Geodetic
DATE October 16 and 17, 2012
CHECKED BY SEMC

ELEV DEPTH (m)	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
236.6	GROUND SURFACE					
236.3 / 0.0	PEAT (Fibrous) Black Moist	1	SS	3		
0.3	CLAYEY SILT to SILTY CLAY, trace to some sand Very soft to firm Brown to grey Wet	2	SS	5	∇	
		3	SS	6		
		4	SS	4		
		5	SS	WH		0 9 59 32
232.9 / 3.7	CLAYEY SILT to SILT, some gravel, some sand Firm Grey Wet	6	SS	4		
		7	SS	8		17 13 61 9
231.0 / 5.6	SAND and SILT, trace gravel Very loose to compact Grey Wet	8	SS	WH		
		9	SS	24		
228.2 / 8.4	SAND and SILT, some gravel, trace clay (TILL) Dense to very dense Grey Moist to wet	10	SS	49		
		11	SS	127		13 45 37 5
	Spoon refusal (hammer bouncing) and auger refusal at 9.6 m depth. Coring between 9.7 m and 11.3 m depth.	1	REC	REC 27%		
		12	SS	93		
	A 1.0 m thick granite boulder was encountered at a 12.1 m depth.	2	REC	REC 100%		
223.5 / 13.1	END OF BOREHOLE					

Note:

1. Water level at a depth of 1.4 m below ground surface (Elev. 235.2 m) upon completion of drilling.

Column headings: STRAT PLOT; ELEVATION SCALE (236, 235, 234, 233, 232, 231, 230, 229, 228, 227, 226, 225, 224); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³

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

RECORD OF BOREHOLE No VS2

 METRIC

PROJECT 11-1191-0008

W.P. 5150-05-00

LOCATION N 5511309.6; E 307639.2

ORIGINATED BY ID

DIST HWY 11

BOREHOLE TYPE 108 mm ID Continuous Flight Hollow Stem Augers, NW Casing, NQ Coring

COMPILED BY EC

DATUM Geodetic

DATE October 17 and 18, 2012

CHECKED BY SEMC

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
235.8	GROUND SURFACE
235.5 / 0.0	PEAT (Fibrous) Black Moist
0.3	CLAYEY SILT Very soft to firm Brown to grey Moist to wet Trace organics to 2.1 m depth.
232.8 / 3.0	CLAYEY SILT to SILT, some sand, trace gravel Soft to firm Grey Wet
231.3 / 4.5	SAND and SILT, trace gravel, trace clay Very loose Grey Wet
228.0 / 7.8	Sandy CLAYEY SILT to CLAYEY SILT with SAND, trace to some gravel (TILL) Hard Grey Moist to wet Auger refusal at 8.8 m depth. Coring between 9.2 m and 10.4 m depth. Coring between 10.7 m and 11.9 m depth.
223.8 / 12.0	END OF BOREHOLE

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	4	
2	SS	6	
3	SS	7	
4	SS	WH	
5	SS	3	5 19 60 16
6	SS	7	
7	SS	4	
8	SS	2	3 63 31 3
9	SS	91	
10	SS	134	8 27 51 14
1	SC	REC 0%	
11	SS	150/0.15	
2	SC	REC 0%	
12	SS	174/0.15	

ELEVATION SCALE: 235, 234, 233, 232, 231, 230, 229, 228, 227, 226, 225, 224

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

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

WATER CONTENT (%): 20 40 60

UNIT WEIGHT γ kN/m³

GROUND WATER CONDITIONS

STRAT PLOT

Continued Next Page

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

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PROJECT 11-1191-0008	RECORD OF BOREHOLE No VS2	2 OF 2 **METRIC**
W.P. 5150-05-00	LOCATION N 5511309.6; E 307639.2	ORIGINATED BY ID
DIST HWY 11	BOREHOLE TYPE 108 mm ID Continuous Flight Hollow Stem Augers, NW Casing, NQ Coring	COMPILED BY EC
DATUM Geodetic	DATE October 17 and 18, 2012	CHECKED BY SEMC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	--- CONTINUED FROM PREVIOUS PAGE ---										
	Note: 1. Water level at a depth of 1.1 m below ground surface (Elev. 234.7 m) upon completion of drilling. 2. Water level in piezometer at a depth of 1.0 m below ground surface (Elev. 234.3 m) on October 18, 2012. 3. Water level in piezometer at a depth of 0.4 m below ground surface (Elev. 235.4 m) on December 6, 2012. 4. Water level in piezometer at a depth of 0.9 m below ground surface (Elev. 234.9 m) on July 5, 2013.										

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

SUD_MTO 003 111191000008DET.GPJ GAL-MISS.GDT 31/10/13 DATA INPUT:

Golder Associates

RECORD OF BOREHOLE No VS3

 METRIC

PROJECT 11-1191-0008
W.P. 5150-05-00
LOCATION N 5511324.2; E 307585.9
ORIGINATED BY EHS
DIST HWY 11
BOREHOLE TYPE 108 mm ID Continuous Flight Hollow Stem Augers, NW Casing, NQ Coring
COMPILED BY EC
DATUM Geodetic
DATE June 7 and 8, 2013
CHECKED BY SEMC

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	SHEAR STRENGTH kPa	WATER CONTENT	GR	SA	SI	CL
236.5	GROUND SURFACE									
0.0	PEAT (Fibrous) Firm Black Moist	1	SS	5						
235.6 / 0.9	SILTY CLAY, trace sand Firm Brown Wet	2	SS	6		W_P–w–W_L plotted				
		3	SS	6	Field vane 4					
233.7 / 2.8	CLAYEY SILT to SILT, some sand Soft Grey Wet	4	SS	2		W_P–w–W_L plotted	0	16	62	22
		5	SS	2						
		6	SS	3						
231.0 / 5.5	SAND and SILT Very loose to compact Brown Wet. Between 0.3 m and 2.1 m of heave encountered below 6.1 m depth.	7	SS	WR						
		8	SS	21						
227.8 / 8.7	Sandy CLAYEY SILT to CLAYEY SILT with SAND, trace to some gravel (TILL) Hard Grey Moist to wet	9	SS	39		w plotted	11	31	48	10
	Spoon refusal (hammer bouncing) at 11.0 m depth.	10	SS	100/0.18						
		11	SS	71						
		12	SS	73		W_P–w–W_L plotted	13	24	46	17

Continued Next Page

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

SUD_MTO 003 11119100008DET.GPJ GAL-MISS.GDT 31/10/13 DATA INPUT:

Golder Associates

RECORD OF BOREHOLE No VS3

 METRIC

PROJECT 11-1191-0008
W.P. 5150-05-00
LOCATION N 5511324.2; E 307585.9
ORIGINATED BY EHS
DIST HWY 11
BOREHOLE TYPE 108 mm ID Continuous Flight Hollow Stem Augers, NW Casing, NQ Coring
COMPILED BY EC
DATUM Geodetic
DATE June 7 and 8, 2013
CHECKED BY SEMC

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%)
220.4	Sandy CLAYEY SILT to CLAYEY SILT with SAND, trace to some gravel (TILL) Hard Grey Moist to wet	13	SS	70/0.03	221	
16.1	GNEISS (BEDROCK) Bedrock cored from 16.1 m depth to 19.4 m depth. For coring details see Record of Drillhole VS-3.	1	RC	REC 100%	220	RQD = 80%
		2	RC	REC 100%	219	RQD = 77%
217.1		3	RC	REC 100%	218	RQD = 77%
19.4	END OF BOREHOLE					

Column headings also printed: SOIL PROFILE; SAMPLES; STRAT PLOT; GROUND WATER CONDITIONS; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m^3; GR SA SI CL

Note:

1. Water level at a depth of 2.9 m below ground surface (Elev. 233.6 m) on the morning of June 8, 2013.

2. Water level at 0.6 m above ground surface (Elev. 237.1 m) upon completion of drilling.

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

SUD_MTO 003 111910008DET.GPJ GAL-MISS.GDT 31/10/13 DATA INPUT:

RECORD OF DRILLHOLE: VS3

PROJECT: 11-1191-0008

SHEET 1 OF 1

LOCATION: N 5511324.2 ;E 307585.9

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: June 7 and 8, 2013

DRILL RIG: CME55

DRILLING CONTRACTOR: Landcore Drilling Ltd.

DATUM: Geodetic

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.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. / DEPTH (m)	RUN No.	COLOUR % RETURN	DISCONTINUITY DATA: TYPE AND SURFACE DESCRIPTION	NOTES WATER LEVELS INSTRUMENTATION
		REFER TO PREVIOUS PAGE					
	NW; June 8, 2013; NQ Coring	GNEISS Strong Fine to coarse grained Slightly weathered Grey	220.4 16.1	1	GREY 100%	JNPLRo JNPLRo JNPLRo	
17–18				2	GREY 100%	JNFORo JNFORo JNFORo JNPLRo JNPLRo JNUNRo JNIRRo JNSTRo JNFORo JNUNRo JNPLK	UCS = 75 MPa
18–19			217.1 19.4	3	GREY 100%	JNPLRo JNIRRo JNIRRo JNIRRo JNIRRo JNIRRo JNIRRo	
		END OF DRILLHOLE					

Depth scale: 17, 18, 19, 20, 21, 22, 23, 24, 25, 26

Column headings: FLUSH; RECOVERY (TOTAL CORE %, SOLID CORE %) 80 60 40 20; R.Q.D. % 80 60 40 20; FRACT. INDEX METRES 5 10 15 20; B Angle 0 90 180 270; DIP w.r.t. CORE AXIS 0 30 60 90; Jr; Ja; Jn; HYDRAULIC CONDUCTIVITY k, cm/s 10^{-6} 10^{-5} 10^{-4} 10^{-3}; Diametral Point Load Index (MPa) 2 4 6; RMC -Q' AVG.

DEPTH SCALE 1 : 50

Golder Associates

LOGGED: EHS

CHECKED: SEMC

Golder Associates

RECORD OF BOREHOLE No VS4

 METRIC

PROJECT 11-1191-0008
W.P. 5150-05-00
DIST ______ HWY 11
DATUM Geodetic

LOCATION N 5511313.5; E 307577.2
BOREHOLE TYPE 108 mm ID Continuous Flight Hollow Stem Augers, NW Casing, NQ Coring
DATE June 9 and 10, 2013

ORIGINATED BY EHS
COMPILED BY EC
CHECKED BY SEMC

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
236.5	GROUND SURFACE								
0.0	PEAT (Amorphous) Soft Black Moist	1	SS	2		236			
235.6 / 0.9	CLAYEY SILT, trace sand, trace organics Firm Brown Wet	2	SS	7		235			
		3	SS	7				w_P–w–w_L	
						234	+ 4 (field vane)		
233.6 / 2.9	SAND and SILT, trace gravel, trace to some clay Very loose to loose Grey Wet	4	SS	5		233			
		5	SS	5				○	4 33 56 7
		6	SS	2	∇	232			
						231			
		7	SS	24		230			
229.3 / 7.2	Sandy CLAYEY SILT to CLAYEY SILT with SAND, trace gravel (TILL) Hard Grey Moist					229			
		8	SS	61				○H	3 27 55 15
						228			
		9	SS	50		227			
						226			
		10	SS	46				○	3 28 56 13
						225			
		11	SS	61		224			
223.6 / 12.9	GNEISS (BEDROCK) Bedrock cored from 12.9 m depth to 16.1 m depth. For coring details see Record of Drillhole VS4.	1	RC	REC 100%		223			RQD = 100%
		2	RC	REC 100%		222			RQD = 100%

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED; 20 40 60 80 100
PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60
UNIT WEIGHT γ kN/m³

Continued Next Page

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

Golder Associates

RECORD OF BOREHOLE No VS4

 METRIC

PROJECT 11-1191-0008

W.P. 5150-05-00 LOCATION N 5511313.5; E 307577.2 ORIGINATED BY EHS

DIST HWY 11 BOREHOLE TYPE 108 mm ID Continuous Flight Hollow Stem Augers, NW Casing, NQ Coring COMPILED BY EC

DATUM Geodetic DATE June 9 and 10, 2013 CHECKED BY SEMC

SOIL PROFILE: ELEV DEPTH	SOIL PROFILE: DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: "N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L / WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	--- CONTINUED FROM PREVIOUS PAGE ---										
			2	RC							RQD = 100%
			3	RC	REC 100%		221				RQD = 100%
220.4 16.1	END OF BOREHOLE Note: 1. Water level at a depth of 5.0 m below ground surface (Elev. 231.5 m) upon completion of drilling.										

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

SUD_MTO 003 111910008DET.GPJ GAL-MISS.GDT 31/10/13 DATA INPUT:

PROJECT: 11-1191-0008

RECORD OF DRILLHOLE: VS4

SHEET 1 OF 1

LOCATION: N 5511313.5 ;E 307577.2

DRILLING DATE: June 9 and 10, 2013

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME55

DRILLING CONTRACTOR: Landcore Drilling Ltd.

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.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. / DEPTH (m)	RUN No.	COLOUR % RETURN	DISCONTINUITY TYPE AND SURFACE DESCRIPTION	NOTES WATER LEVELS INSTRUMENTATION
		REFER TO PREVIOUS PAGE					
13	June 10, 2013 NQ Coring (NW)	GNEISS Very Strong Fine to coarse grained Slightly weathered Grey	223.6 12.9	1	GREY 100%	JNIRRo JNSTRo JNIRRo JNIRRo	UCS = 126 MPa
14				2	GREY 100%	JNFORo	
15				3	GREY 100%		
16		END OF DRILLHOLE	220.4 16.1				
17							
18							
19							
20							
21							
22							

Column headers also include: SYMBOLIC LOG, FLUSH, RECOVERY (TOTAL CORE %, SOLID CORE %), R.Q.D. %, FRACT. INDEX PER 0.3 METRES, DISCONTINUITY DATA (B Angle, DIP w.r.t. CORE AXIS, Jr, Ja, Jn), HYDRAULIC CONDUCTIVITY k, cm/s, Diametral Point Load Index (MPa), RMC -Q' AVG.

SUD-RCK 1111910008DET.GPJ GAL-MISS.GDT 23/10/13 DATA INPUT:

DEPTH SCALE 1 : 50

Golder Associates

LOGGED: EHS

CHECKED: SEMC

RECORD OF BOREHOLE No VS5

 METRIC

PROJECT 11-1191-0008
W.P. 5150-05-00
LOCATION N 5511316.1; E 307635.4
ORIGINATED BY ID
DIST HWY 11
BOREHOLE TYPE 108 mm ID Continuous Flight Hollow Stem Augers
COMPILED BY EC
DATUM Geodetic
DATE July 4, 2013
CHECKED BY SEMC

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
235.8	GROUND SURFACE					
0.0 / 235.5	TOPSOIL	1A	SS	4		
0.3	CLAYEY SILT, trace sand Soft to firm Brown to grey Wet	1B				
		2	SS	7	▽	
		3	SS	6		
		4	SS	6		0 3 80 17
		5	SS	3		
232.1 / 3.7	Sandy CLAYEY SILT to SILT, trace to some gravel Soft to stiff Grey Wet	6	SS	3		10 39 38 13
		7	SS	9		
230.2 / 5.6	SAND and SILT, some gravel Very loose to compact Grey Wet	8	SS	3		
		9	SS	17		
227.1 / 8.7	Sandy CLAYEY SILT to CLAYEY SILT with SAND, trace gravel (TILL) Hard Grey Moist	10	SS	110		4 26 55 15
		11	SS	103		
		12	SS	113		
223.2 / 12.6	END OF BOREHOLE SPOON REFUSAL (HAMMER BOUNCING) Note: 1. Water level at a depth of 1.1 m below ground surface (Elev. 234.7 m) upon completion of drilling.					

Column headings: STRAT PLOT; ELEVATION SCALE (235, 234, 233, 232, 231, 230, 229, 228, 227, 226, 225, 224); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³.

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

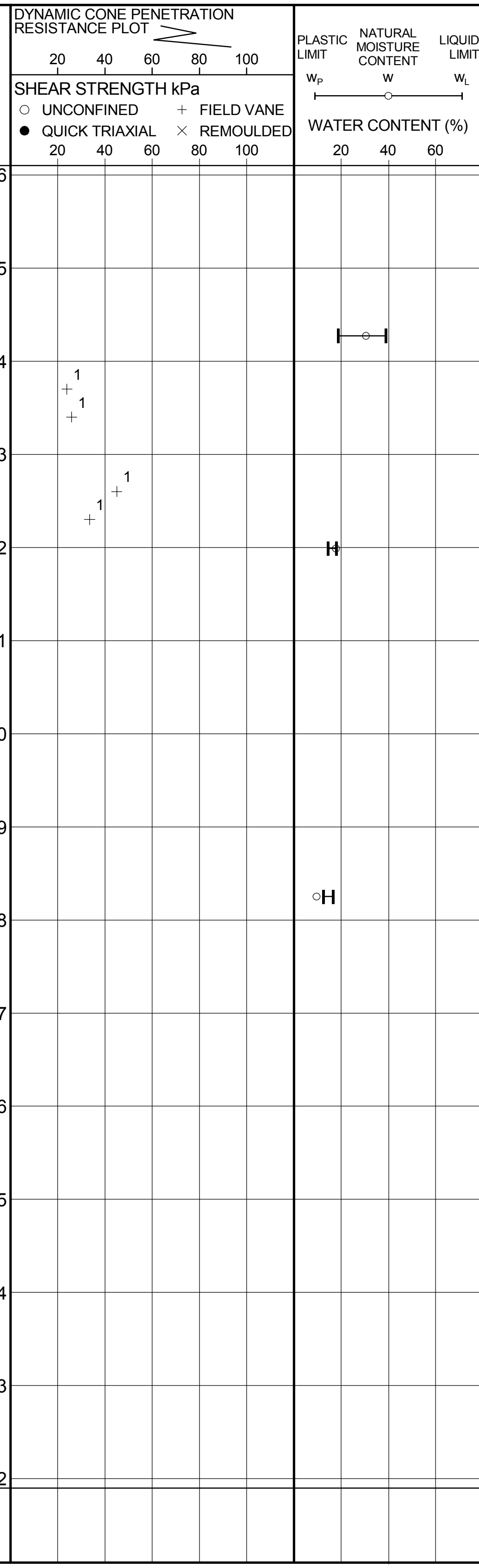
Golder Associates

RECORD OF BOREHOLE No VS6

 METRIC

PROJECT 11-1191-0008
W.P. 5150-05-00 LOCATION N 5511304.7; E 307630.5 ORIGINATED BY ID
DIST HWY 11 BOREHOLE TYPE 108 mm ID Continuous Flight Hollow Stem Augers COMPILED BY EC
DATUM Geodetic DATE July 5 and 6, 2013 CHECKED BY SEMC

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
236.1	GROUND SURFACE					236	
0.0	TOPSOIL	1A	SS	5			
235.8 / 0.3	SILTY CLAY Soft to firm Brown to grey Moist to wet	1B					
		2	SS	8	∇	235	
		3	SS	7			
		4	SS	4		234	
		5	SS	2		233	
232.3 / 3.8	SILT, some sand, trace to some clay, trace to some gravel Very loose Grey Wet	6	SS	2		232	6 22 61 11
231.6 / 4.5	Sandy SILT, trace to some clay, trace to some gravel Very loose to loose Grey Wet	7	SS	7		231	
		8	SS	2		230	
228.9 / 7.2	Sandy CLAYEY SILT to CLAYEY SILT with SAND, trace gravel (TILL) Hard Grey Moist Between 0.9 m and 1.2 m of heave encountered below 7.6 m depth. Spoon refusal (hammer bouncing) at 8.1 m depth.	9	SS	175		229 / 228	3 30 55 12
		10	SS	79		227 / 226	
	Spoon refusal (hammer bouncing) at 11.1 m depth.	11	SS	195		225	
	Spoon refusal (hammer bouncing) at 12.5 m depth.	12	SS	100/ 0.15		224 / 223	
221.9 / 14.2		13	SS	289		222	



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

Golder Associates

RECORD OF BOREHOLE No VS6

 METRIC

PROJECT 11-1191-0008

W.P. 5150-05-00 LOCATION N 5511304.7; E 307630.5 ORIGINATED BY ID

DIST HWY 11 BOREHOLE TYPE 108 mm ID Continuous Flight Hollow Stem Augers COMPILED BY EC

DATUM Geodetic DATE July 5 and 6, 2013 CHECKED BY SEMC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L / WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	--- CONTINUED FROM PREVIOUS PAGE ---										
	END OF BOREHOLE SPOON REFUSAL (HAMMER BOUNCING) Note: 1. Water level at a depth of 1.3 m below ground surface (Elev. 234.8 m) upon completion of drilling. 2. Moved 1.0 m east of Borehole VS6 and turned N-vanes from 2.4 m depth to 3.8 m depth.										

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

RECORD OF BOREHOLE No VS7

 METRIC

PROJECT 11-1191-0008
W.P. 5150-05-00
LOCATION N 5511330.2; E 307581.7
ORIGINATED BY EHS
DIST ___ HWY 11
BOREHOLE TYPE 108 mm ID Continuous Flight Hollow Stem Augers
COMPILED BY EC
DATUM Geodetic
DATE June 7, 2013
CHECKED BY SEMC

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
237.6	GROUND SURFACE						
0.0	Sand, trace to some gravel (FILL) Loose Brown Moist	1	SS	4		237	
236.9	Clayey silt (FILL) Stiff Brown Moist	2A / 2B	SS	12			
236.5 / 1.1	PEAT (Amorphous) Black Moist						
236.0 / 1.6	SILTY CLAY Stiff Brown Moist to wet	3	SS	9	▽	236	
235.2 / 2.4	CLAYEY SILT, some sand, trace gravel Soft to firm Brown to grey Wet	4	SS	7		235	
		5	SS	5		234	
		6	SS	2			2 20 56 22
		7	SS	6		233	
232.0 / 5.6	SAND and SILT, trace to some clay, trace gravel Loose to compact Grey Wet					232	
		8	SS	9		231	2 38 54 6
		9	SS	22		230	
	Spoon refusal (hammer bouncing) at 8.1 m depth.					229	
228.9 / 8.7	Sandy CLAYEY SILT to CLAYEY SILT with SAND, some gravel (TILL) Hard Grey Moist	10	SS	72		228	
	Augers grinding below 10.0 m depth.						
227.3 / 10.3	END OF BOREHOLE AUGER REFUSAL						

Note:

1. Water level at a depth of 1.8 m below ground surface (Elev. 235.8 m) upon completion of drilling.

Plot headings: DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³

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

Golder Associates

RECORD OF BOREHOLE No VS8

 METRIC

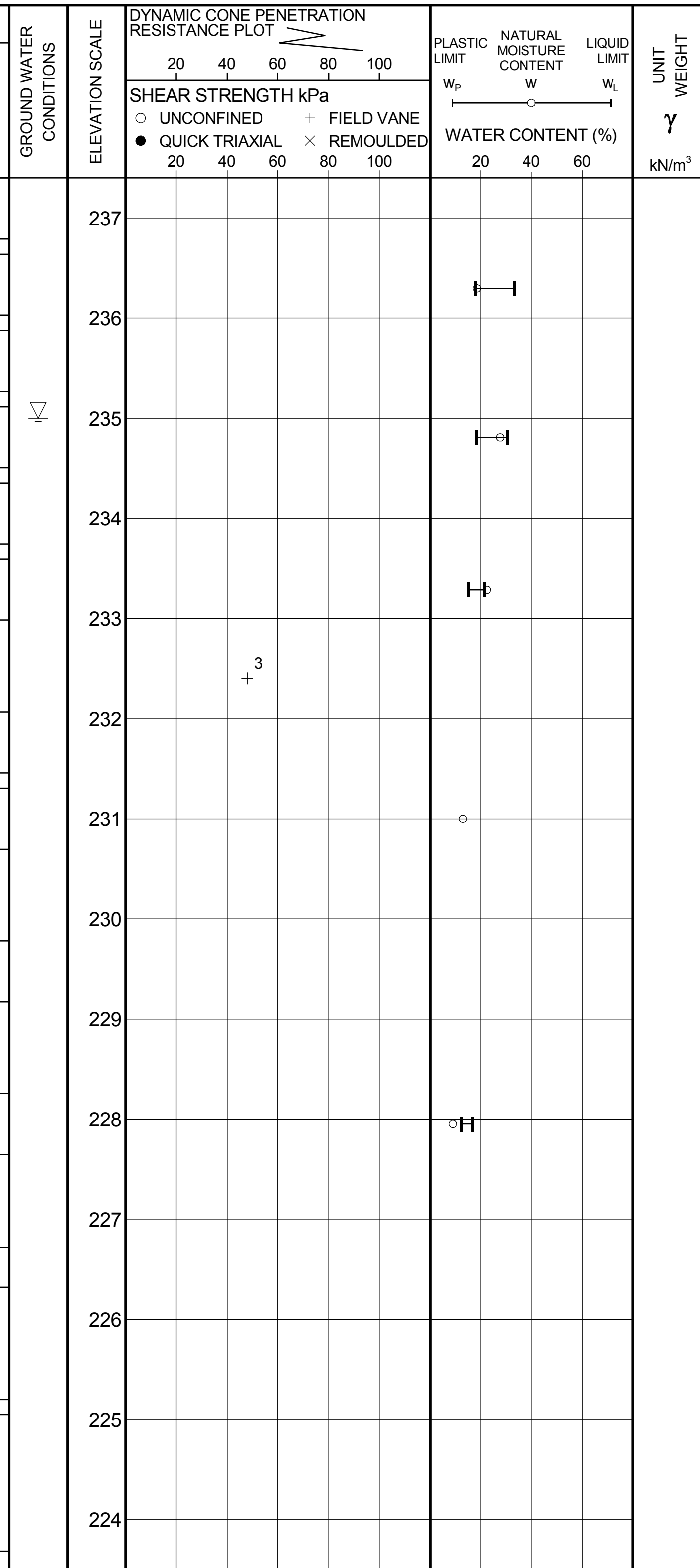
PROJECT 11-1191-0008
W.P. 5150-05-00
LOCATION N 5511326.6; E 307593.6
ORIGINATED BY EHS
DIST HWY 11
BOREHOLE TYPE 108 mm ID Continuous Flight Hollow Stem Augers, NW Casing, NQ Coring
COMPILED BY EC
DATUM Geodetic
DATE June 10 and 17, 2013
CHECKED BY SEMC

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
237.4	GROUND SURFACE
0.0	Gravelly sand, trace silt (FILL) Loose Brown Moist
236.5 0.9	Clayey silt, trace to some sand, trace organics (FILL) Firm Grey Moist to wet
235.2 2.2	CLAYEY SILT Firm Brown to grey Wet
233.7 3.7	CLAYEY SILT to SILT Firm Grey Brown
231.4 6.0	SAND and SILT, trace to some clay, trace gravel Very loose to compact Brown to grey Wet
228.7 8.7	Sandy CLAYEY SILT to CLAYEY SILT with SAND, trace gravel (TILL) Hard Grey Wet A 0.4 m thick granite boulder was encountered at 9.7 m depth.
223.4 14.0	

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
1	SS	7				
2	SS	7				
3	SS	7				
4	SS	6				
5	SS	5				
6	SS	2				
7	SS	3				
8	SS	2	5	44	41	10
9	SS	26				
10	SS	83	5	25	56	14
11	SS	108/ 0.25				
12	SS	132/ 0.15				
13	SS	118/ 0.15				



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

Golder Associates

RECORD OF BOREHOLE No VS8

 METRIC

PROJECT 11-1191-0008

W.P. 5150-05-00 LOCATION N 5511326.6; E 307593.6 ORIGINATED BY EHS

DIST HWY 11 BOREHOLE TYPE 108 mm ID Continuous Flight Hollow Stem Augers, NW Casing, NQ Coring COMPILED BY EC

DATUM Geodetic DATE June 10 and 17, 2013 CHECKED BY SEMC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L / WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m^3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	--- CONTINUED FROM PREVIOUS PAGE ---										
	END OF BOREHOLE Note: 1. Water level at a depth of 2.4 m below ground surface (Elev. 235.0 m) on June 17 prior to resuming drilling completion below 9.1 m depth.										

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

Golder Associates

RECORD OF BOREHOLE No VS9

 METRIC

PROJECT	11-1191-0008		
W.P.	5150-05-00	LOCATION	N 5511321.6; E 307635.1
DIST	HWY 11	BOREHOLE TYPE	108 mm ID Continuous Flight Hollow Stem Augers
DATUM	Geodetic	DATE	June 27, 2013
ORIGINATED BY	EHS	COMPILED BY	EC
CHECKED BY	SEMC		

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS	GR	SA	SI	CL
235.5	GROUND SURFACE										
0.0	PEAT (Amorphous) Very soft Black Wet	1	SS	WH		235					
234.8 / 0.7	ORGANIC CLAY Very soft Black Wet	2	SS	WH		234	OC=4.1%				
		3	SS	WH							
233.3 / 2.2	CLAYEY SILT to SILT, some sand Soft Grey Wet	4	SS	2		233		0	13	66	21
232.5 / 3.0	SAND and SILT, trace to some clay, trace to some gravel Very loose to compact Grey Wet	5	SS	2		232					
		6	SS	3		231		4	31	57	8
		7	SS	7		230					
		8	SS	21	▽	229					
		9	SS	19		228, 227		11	50	33	6
226.8 / 8.7	Sandy CLAYEY SILT to CLAYEY SILT with SAND, trace to some gravel (TILL) Hard Grey Wet Spoon refusal (hammer bouncing) at 9.3 m depth.	10	SS	120/0.18		226					
		11	SS	121		225					
224.7 / 10.8	END OF BOREHOLE SPOON REFUSAL (HAMMER BOUNCING) Note: 1. Water level at a depth of 6.1 m below ground surface (Elev. 229.4 m) upon completion of drilling.										

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60

UNIT WEIGHT γ kN/m³

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

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

Golder Associates

RECORD OF BOREHOLE No VS10

 METRIC

PROJECT 11-1191-0008

W.P. 5150-05-00 LOCATION N 5511316.9; E 307654.3 ORIGINATED BY EHS

DIST HWY 11 BOREHOLE TYPE 108 mm ID Continuous Flight Hollow Stem Augers COMPILED BY EC

DATUM Geodetic DATE June 25 and 26, 2013 CHECKED BY SEMC

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT / LIMITS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
239.4	GROUND SURFACE							
0.0	ASPHALT (180 mm)							
0.2	Sand, some gravel, trace to some silt (FILL) Loose Brown Moist	1	AS	-		239		
		2	SS	10			w	18 74 (8)
		3	SS	5		238		
237.4 2.0	Clay, trace to some sand (FILL) Firm Brown Moist	4	SS	8		237	wP–w–wL	
236.2	PEAT (Fibrous) Black Moist	5	SS	10		236		
3.4	CLAYEY SILT, trace sand Firm to stiff Brown to grey Wet	6	SS	5		235	wP–w–wL	0 2 53 45
		7	SS	10	∇			
233.8 5.6	CLAYEY SILT to SILT Firm Brown to grey Wet	8	SS	4		234 / 233	wP–w–wL	8 43 39 10
232.2 7.2	SAND and SILT, trace to some clay, trace to some gravel Loose Brown to grey Wet	9	SS	4		232 / 231		
		10	SS	4		230		
229.2 10.2	Sandy CLAYEY SILT to CLAYEY SILT with SAND, trace gravel (TILL) Hard Grey Wet	11	SS	36		229 / 228	w	2 37 52 9
	Spoon refusal (hammer bouncing) at 12.4 m depth.	12	SS	123/0.18		227		
		13	SS	39		226 / 225		

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60

UNIT WEIGHT γ kN/m^3

Continued Next Page

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

Golder Associates

RECORD OF BOREHOLE No VS10

 METRIC

PROJECT 11-1191-0008

W.P. 5150-05-00 LOCATION N 5511316.9; E 307654.3 ORIGINATED BY EHS

DIST HWY 11 BOREHOLE TYPE 108 mm ID Continuous Flight Hollow Stem Augers COMPILED BY EC

DATUM Geodetic DATE June 25 and 26, 2013 CHECKED BY SEMC

ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L / WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	--- CONTINUED FROM PREVIOUS PAGE ---										
223.9			14	SS	57/ 0.20		224				
15.5	END OF BOREHOLE SPOON REFUSAL AND AUGER REFUSAL (HAMMER BOUNCING) Note: 1. Water level at a depth of 4.9 m below ground surface (Elev. 234.5 m) upon completion of drilling.										

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

Golder Associates

RECORD OF BOREHOLE No VS11

 METRIC

PROJECT 11-1191-0008
W.P. 5150-05-00
LOCATION N 5511330.7; E 307561.2
ORIGINATED BY EHS
DIST HWY 11
BOREHOLE TYPE 108 mm ID Continuous Flight Hollow Stem Augers
COMPILED BY EC
DATUM Geodetic
DATE June 6, 2013
CHECKED BY SEMC

ELEV DEPTH (m)	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
237.0	GROUND SURFACE						
0.0	PEAT (Amorphous), Firm, Black, Wet	1	SS	6			
236.5 / 0.5	CLAYEY SILT, trace to some sand, Firm, Brown, Wet	2	SS	4			
		3	SS	8		Wp–w–WL	
234.6 / 2.4	SAND and SILT, trace to some clay, trace to some gravel, Loose to compact, Brown to grey, Wet	4	SS	16	▽ (Elev. 234.1)		
		5	SS	7			
		6	SS	14		○	9 39 46 6
		7	SS	11			
		8A / 8B	SS	4		○	0 35 64 4
230.4 / 6.6	Sandy CLAYEY SILT to CLAYEY SILT with SAND, trace gravel (TILL), Hard, Grey, Wet						
		9	SS	71			
		10	SS	59		○	3 29 57 11
		11	AS	-			
226.3 / 10.7	END OF BOREHOLE						

Note:

1. Water level at a depth of 2.9 m below ground surface (Elev. 234.1 m) upon completion of drilling.

Elevation scale: 236, 235, 234, 233, 232, 231, 230, 229, 228, 227

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED; 20 40 60 80 100
PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60
UNIT WEIGHT γ kN/m³

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

RECORD OF BOREHOLE No VS12

 METRIC

PROJECT 11-1191-0008

W.P. 5150-05-00 LOCATION N 5511305.0; E 307657.3 ORIGINATED BY EHS

DIST HWY 11 BOREHOLE TYPE 108 mm ID Continuous Flight Hollow Stem Augers COMPILED BY EC

DATUM Geodetic DATE June 26 and 27, 2013 CHECKED BY SEMC

ELEV DEPTH	SOIL PROFILE DESCRIPTION
236.5	GROUND SURFACE
0.0	PEAT (Fibrous) Soft Black Wet
235.8 0.7	SILTY CLAY Firm Brown to grey Moist to wet
234.3 2.2	CLAYEY SILT to SILT, trace to some sand Firm Brown to grey Wet
232.1 4.4	Sandy SILT to SAND and SILT, trace to some clay, trace gravel Loose Grey Wet Approximately 0.6 m of heave encontered at 7.6 m depth.
227.8 8.7	Sandy CLAYEY SILT to CLAYEY SILT with SAND, some gravel (TILL) Hard Grey Wet Spoon refusal (hammer bouncing) at 9.6 m depth.
226.0 10.5	END OF BOREHOLE SPOON AND AUGER REFUSAL (HAMMER BOUNCING) Note: 1. Water level at a depth of 8.8 m below ground surface (Elev. 227.7 m) upon completion of drilling.

SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
1	SS	3				
2	SS	6				
3	SS	8				
4	SS	7				
5	SS	1	0	6	65	29
6	SS	6				
7	SS	6	3	27	64	6
8	SS	10				
9	SS	110/0.25				
10	SS	30/0.03				

Column headings: STRAT PLOT; GROUND WATER CONDITIONS; ELEVATION SCALE (236, 235, 234, 233, 232, 231, 230, 229, 228, 227, 226); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

5

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

APPENDIX B

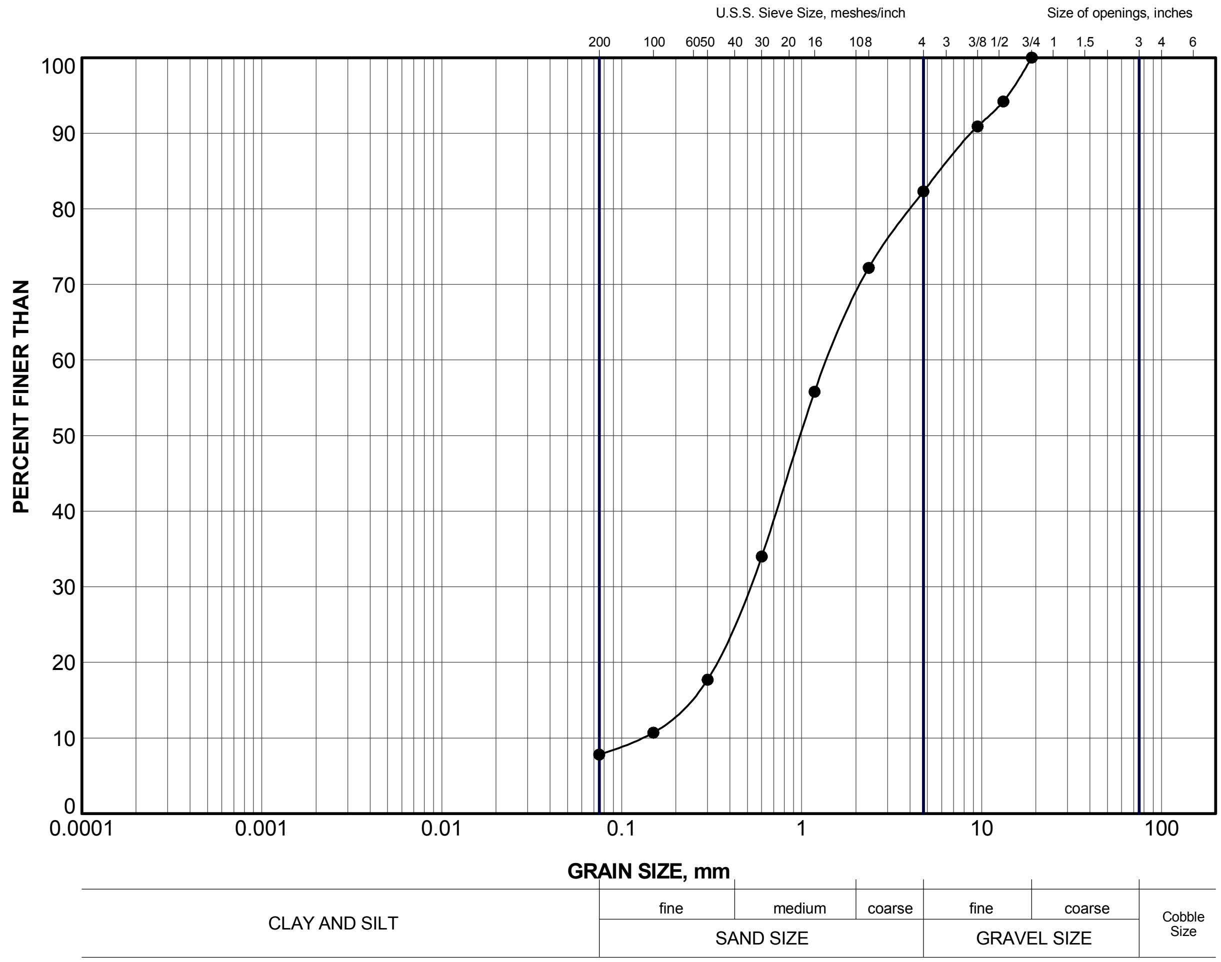
Laboratory Test Results

Table B1 - Summary of Analytical Testing of Creek Water

Parameter	Units	Result
Resistivity	ohm-cm	9,100
Conductivity	µmho/cm	110
pH	pH	7.15
Sulphate	mg/L	Not Detected
Chloride	mg/L	2

Prepared by: EC
Reviewed by: SEMC

Notes:
1. Sample obtained July 6, 2013
2. Analytical testing carried out by Maxxam Analytics Inc.



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	VS10	2	238.3

PROJECT

HIGHWAY 11
VALENTINE RIVER BRIDGE

TITLE

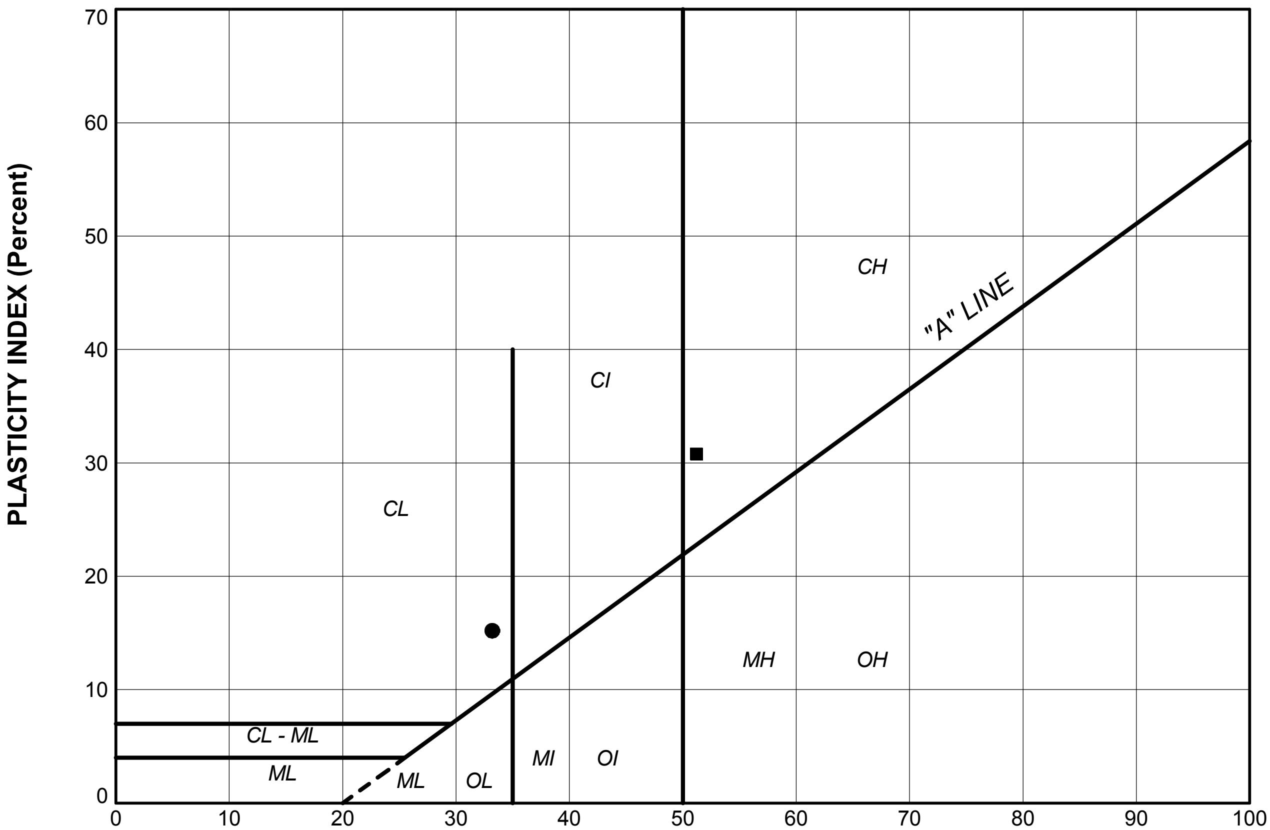
GRAIN SIZE DISTRIBUTION

SAND (FILL)

Golder Associates
SUDBURY, ONTARIO

PROJECT No.		11-1191-0008	FILE No. 1111910008DET.GPJ	
DRAWN	JJL	Oct 2013	SCALE N/A	REV.
CHECK	SEMC	Oct 2013		
APPR	JMAC	Oct 2013	FIGURE B1	

SUD-MTO GSD (NEW) GLDR_LDN.GDT



SOIL TYPE
C = Clay
M = Silt
O = Organic

PLASTICITY
L = Low
I = Intermediate
H = High

LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	VS8	2	33.2	18.0	15.2
■	VS10	4	51.2	20.4	30.8

PROJECT

HIGHWAY 11
VALENTINE RIVER BRIDGE

TITLE

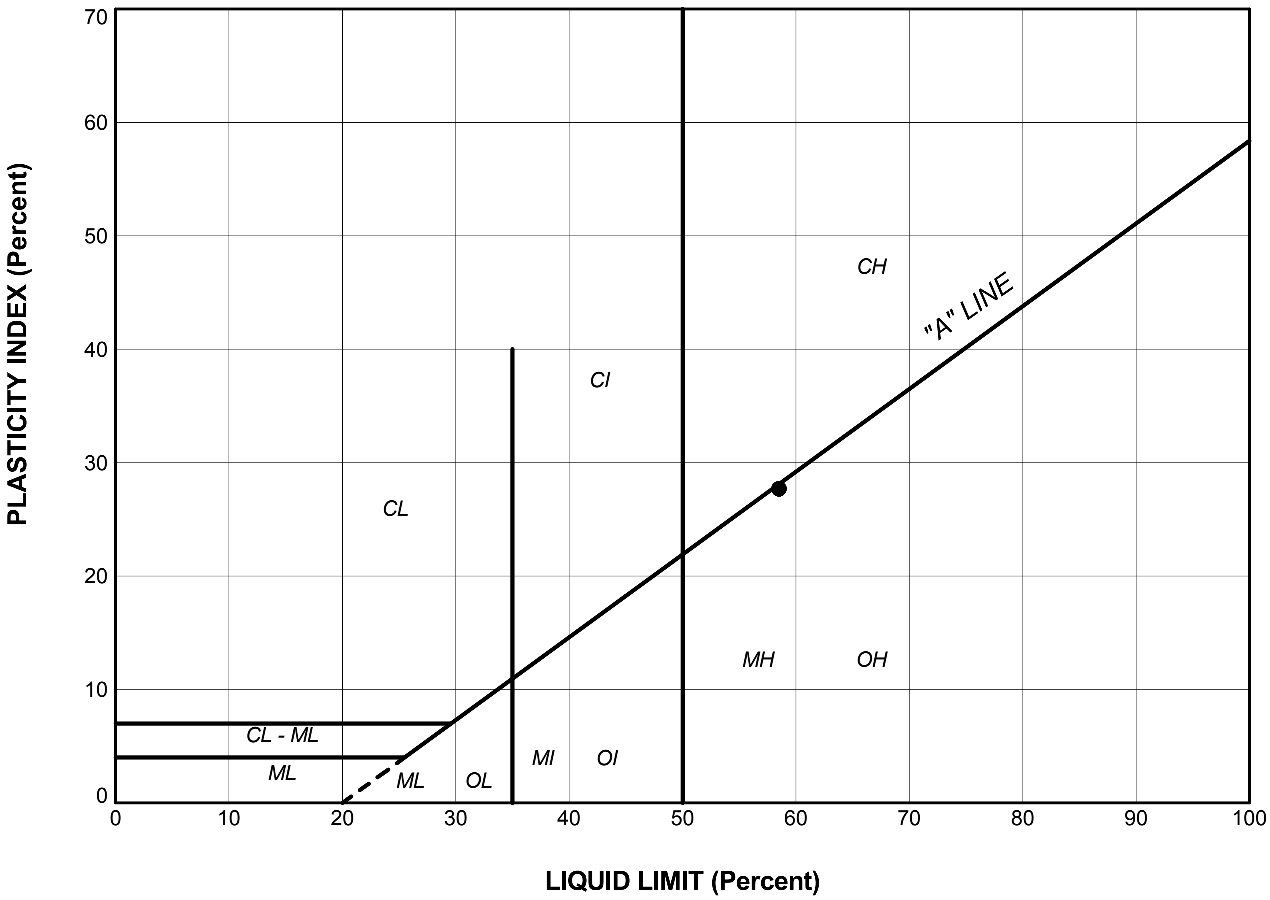
PLASTICITY CHART

CLAYEY SILT to CLAY (FILL)

PROJECT No.	11-1191-0008		FILE No. 1111910008DET.GPJ
DRAWN	JJL	Oct 2013	SCALE N/A REV.
CHECK	SEMC	Oct 2013	
APPR	JMAC	Oct 2013	FIGURE B2

Golder Associates
SUDBURY, ONTARIO

SUD-MTO PL (NEW) GLDR_LDN.GDT



SOIL TYPE
C = Clay
M = Silt
O = Organic

PLASTICITY
L = Low
I = Intermediate
H = High

LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	VS9	2	58.5	30.8	27.7

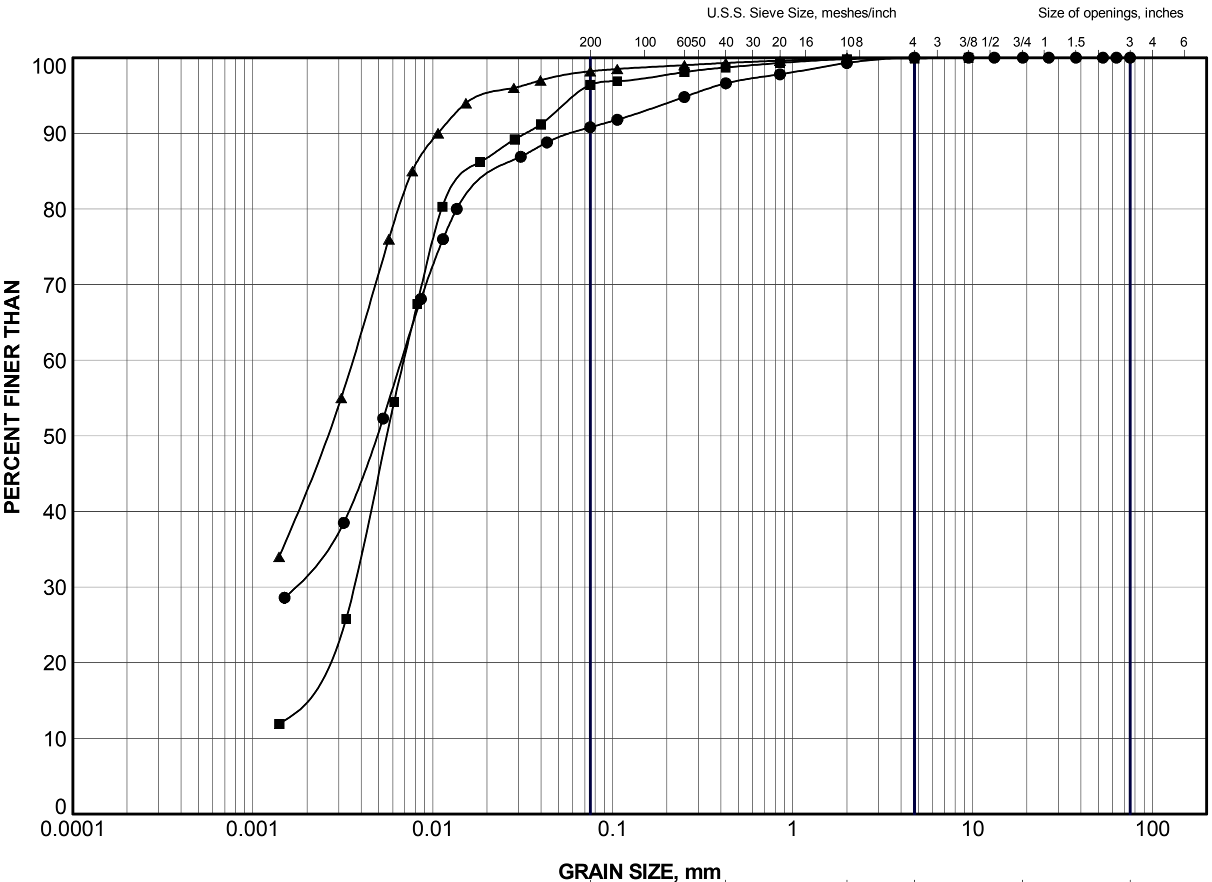
PROJECT
HIGHWAY 11
VALENTINE RIVER BRIDGE

TITLE
PLASTICITY CHART
ORGANIC CLAY

Golder Associates
SUDBURY, ONTARIO

PROJECT No.	11-1191-0008		FILE No.	1111910008DET.GPJ
DRAWN	JJL	Oct 2013	SCALE N/A	REV.
CHECK	SEMC	Oct 2013	**FIGURE B3**	
APPR	JMAC	Oct 2013		

SUD-MTO PL (NEW) GLDR_LDN.GDT



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	VS1	5	233.2
■	VS5	4	233.2
▲	VS10	6	235.3

PROJECT

HIGHWAY 11
VALENTINE RIVER BRIDGE

TITLE

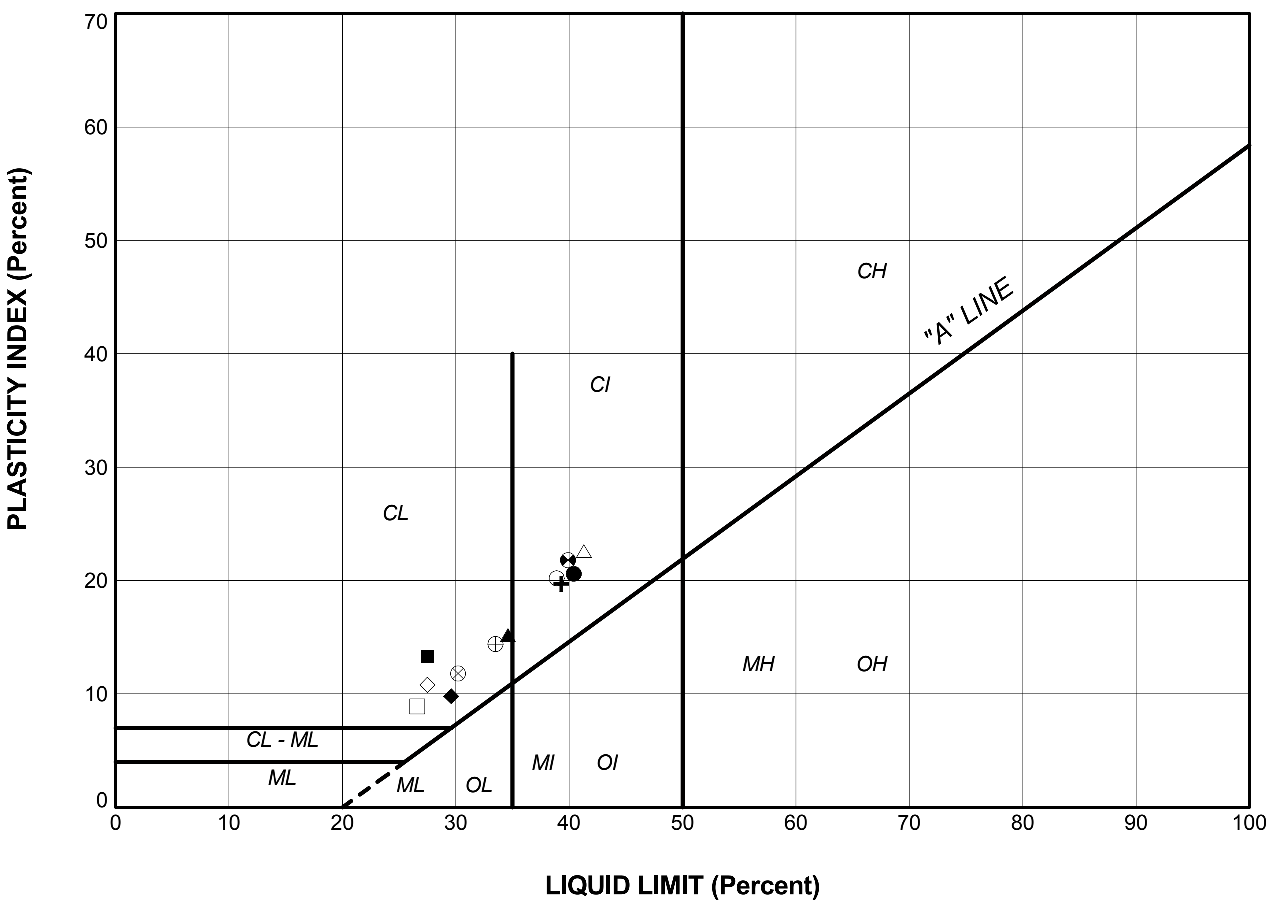
GRAIN SIZE DISTRIBUTION

CLAYEY SILT to SILTY CLAY

Golder Associates
SUDBURY, ONTARIO

PROJECT No.	11-1191-0008		FILE No. 1111910008DET.GPJ	
DRAWN	JJL	Oct 2013	SCALE N/A	REV.
CHECK	SEMC	Oct 2013		
APPR	JMAC	Oct 2013	FIGURE B4	

SUD-MTO GSD (NEW) GLDR_LDN.GDT



SOIL TYPE	PLASTICITY
C = Clay	L = Low
M = Silt	I = Intermediate
O = Organic	H = High

LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	VS1	2	40.4	19.8	20.6
■	VS1	5	27.5	14.2	13.3
▲	VS2	3	34.6	19.4	15.2
✚	VS3	2	39.3	19.6	19.7
◆	VS4	3	29.6	19.8	9.8
◇	VS5	2	27.5	16.7	10.8
○	VS6	3	38.9	18.7	20.2
△	VS7	3	41.3	18.7	22.6
⊗	VS8	4	30.2	18.4	11.8
⊕	VS10	6	33.5	19.1	14.4
□	VS11	3	26.6	17.7	8.9
◓	VS12	2	39.9	18.1	21.8

PROJECT

HIGHWAY 11
VALENTINE RIVER BRIDGE

TITLE

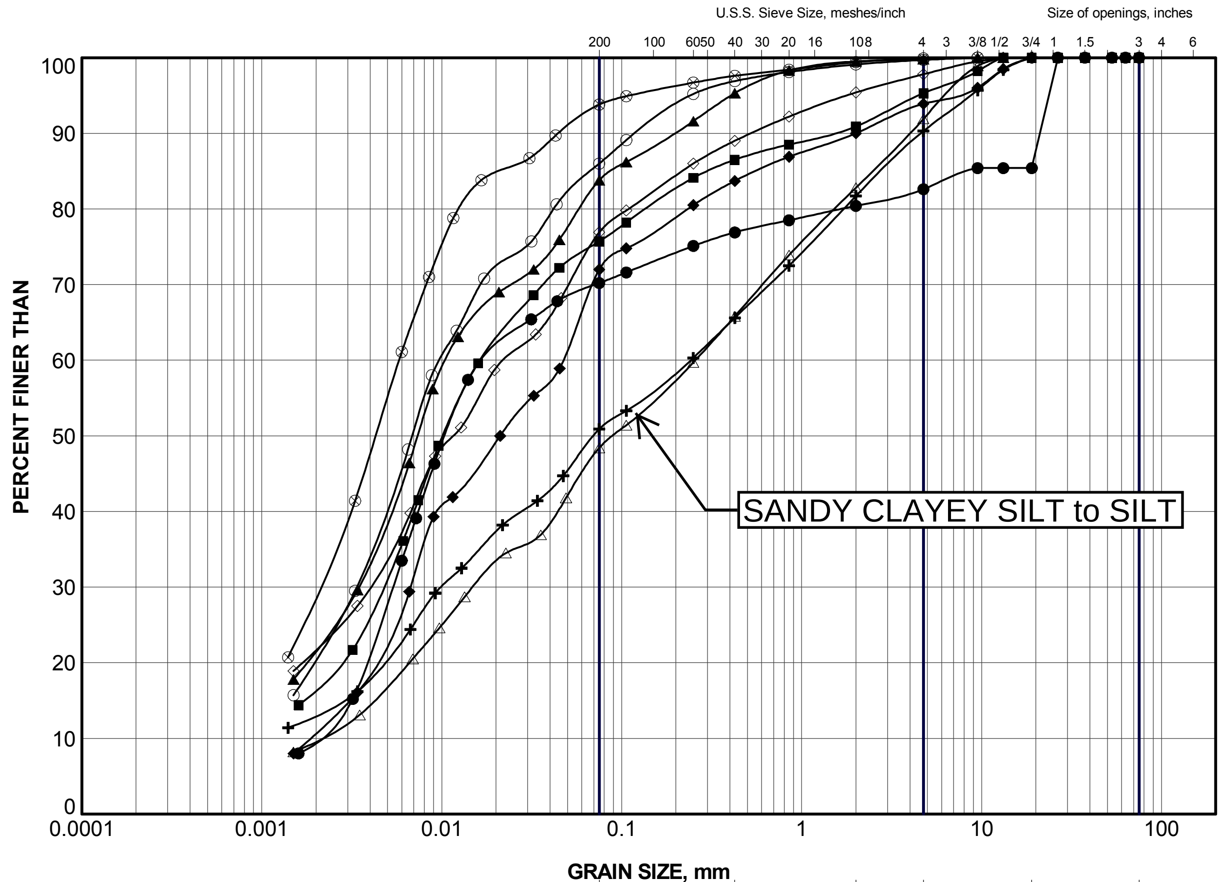
PLASTICITY CHART

CLAYEY SILT to SILTY CLAY

Golder Associates
SUDBURY, ONTARIO

PROJECT No.	11-1191-0008		FILE No.	1111910008DET.GPJ
DRAWN	JJL	Oct 2013	SCALE N/A	REV.
CHECK	SEMC	Oct 2013		
APPR	JMAC	Oct 2013	**FIGURE B5**	

SUD-MTO PL (NEW) GLDR_LDN.GDT



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	VS1	7	231.7
■	VS2	5	232.4
▲	VS3	4	233.1
+	VS5	6	231.7
◆	VS6	6	232.0
◇	VS7	6	233.5
○	VS9	4	232.9
△	VS10	8	233.0
⊗	VS12	5	233.1

PROJECT

HIGHWAY 11
VALENTINE RIVER BRIDGE

TITLE

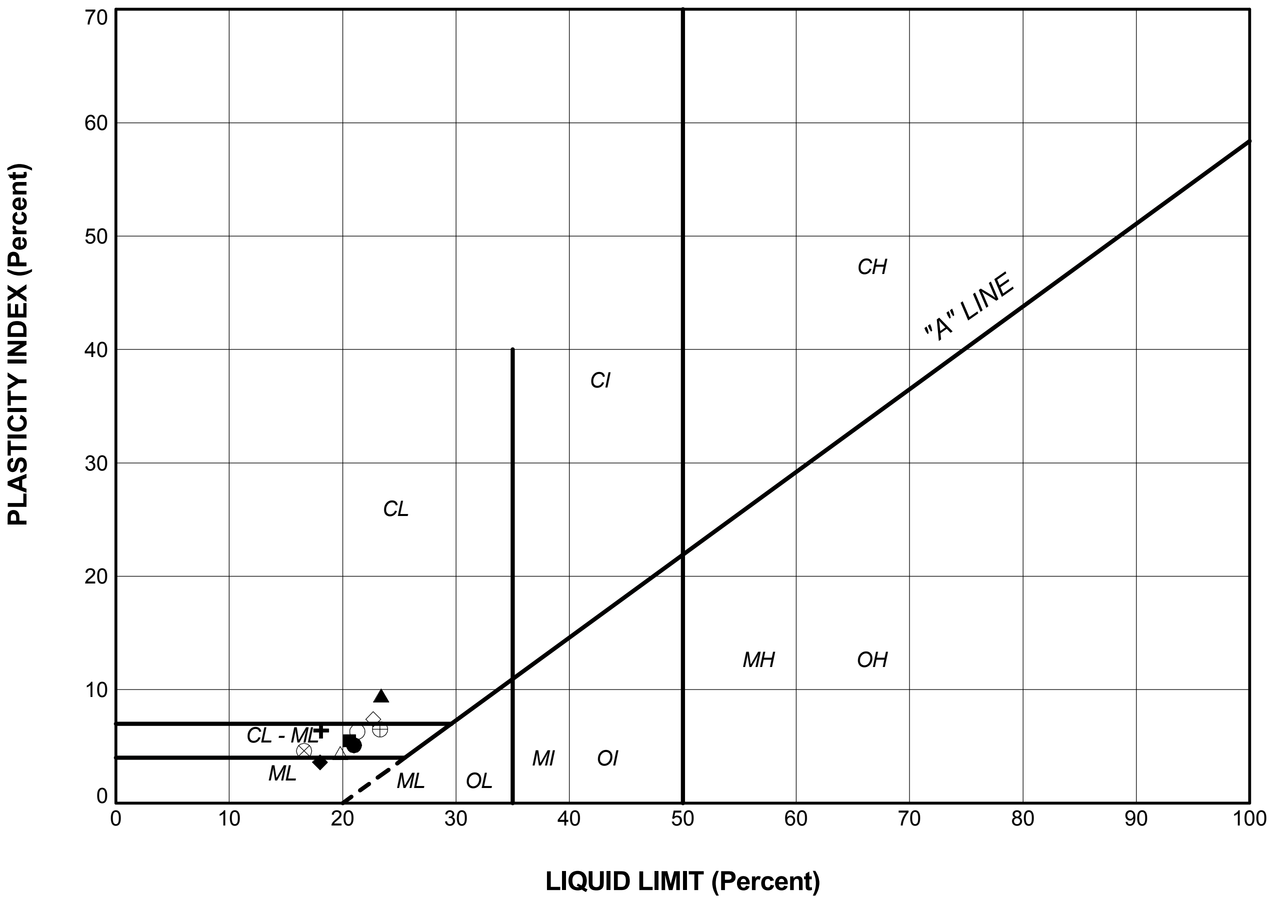
GRAIN SIZE DISTRIBUTION

CLAYEY SILT to SILT

Golder Associates
SUDBURY, ONTARIO

PROJECT No.	11-1191-0008		FILE No. 1111910008DET.GPJ
DRAWN	JJL	Oct 2013	SCALE N/A REV.
CHECK	SEMC	Oct 2013	
APPR	JMAC	Oct 2013	FIGURE B6

SUD-MTO GSD (NEW) GLDR_LDN.GDT



SOIL TYPE	PLASTICITY
C = Clay	L = Low
M = Silt	I = Intermediate
O = Organic	H = High

LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	VS1	7	21.0	15.9	5.1
■	VS2	5	20.6	15.1	5.5
▲	VS3	4	23.4	13.9	9.5
✚	VS5	6	18.1	11.7	6.4
◆	VS6	6	18.0	14.4	3.6
◇	VS7	6	22.7	15.3	7.4
○	VS8	6	21.3	15.0	6.3
△	VS9	4	19.8	15.4	4.4
⊗	VS10	8	16.6	12.0	4.6
⊕	VS12	5	23.3	16.8	6.5

PROJECT

HIGHWAY 11
VALENTINE RIVER BRIDGE

TITLE

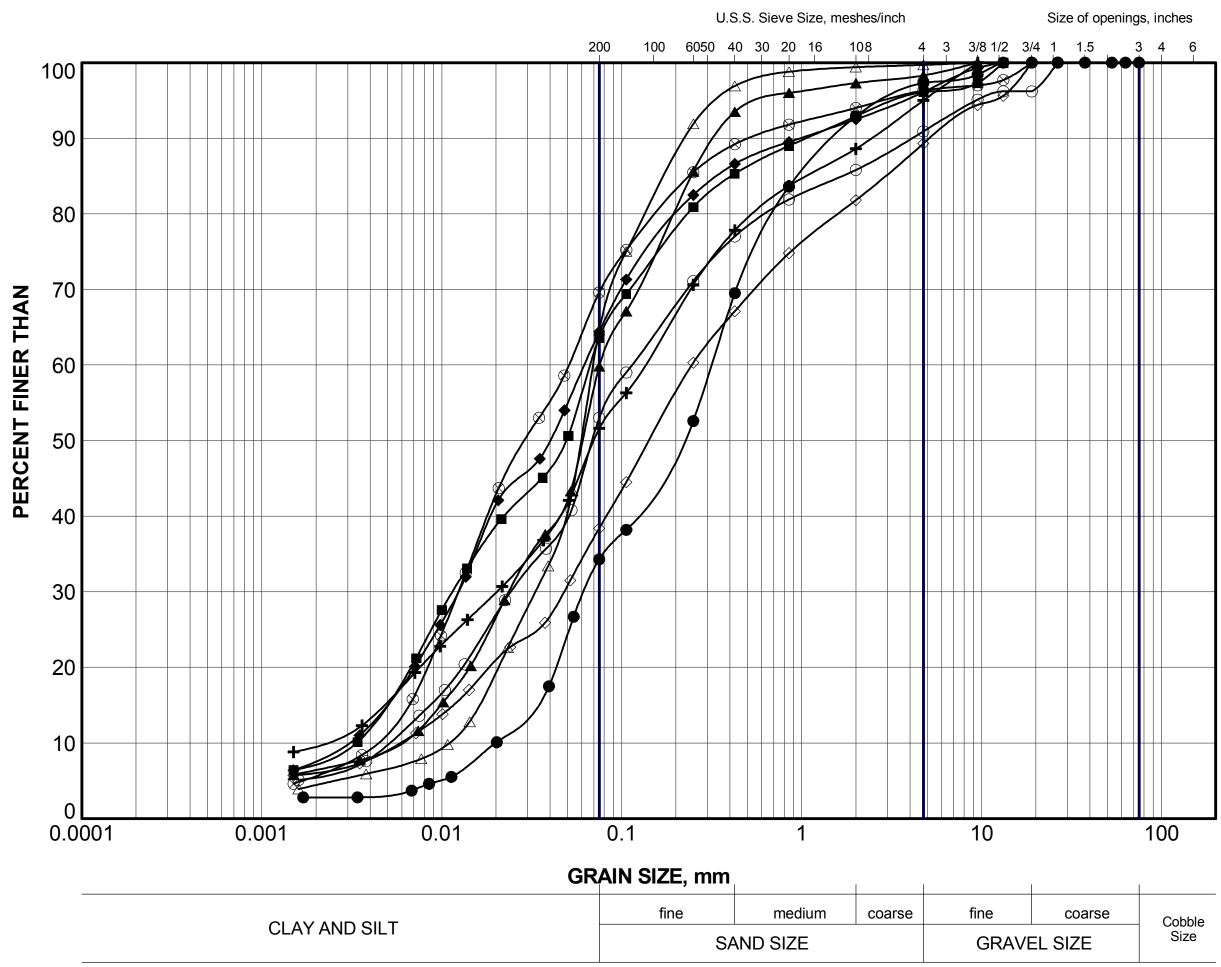
PLASTICITY CHART

CLAYEY SILT to SILT

Golder Associates
SUDBURY, ONTARIO

PROJECT No.	11-1191-0008		FILE No.	1111910008DET.GPJ
DRAWN	JJL	Oct 2013	SCALE N/A	REV.
CHECK	SEMC	Oct 2013		
APPR	JMAC	Oct 2013	FIGURE B7	

SUD-MTO PL (NEW) GLDR_LDN.GDT



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	VS2	8	229.4
■	VS4	5	232.4
▲	VS7	8	231.2
✚	VS8	8	231.0
◆	VS9	6	231.4
◇	VS9	9	227.6
○	VS11	6	232.9
△	VS11	8A	230.8
⊗	VS12	7	230.1

PROJECT

HIGHWAY 11
VALENTINE RIVER BRIDGE

TITLE

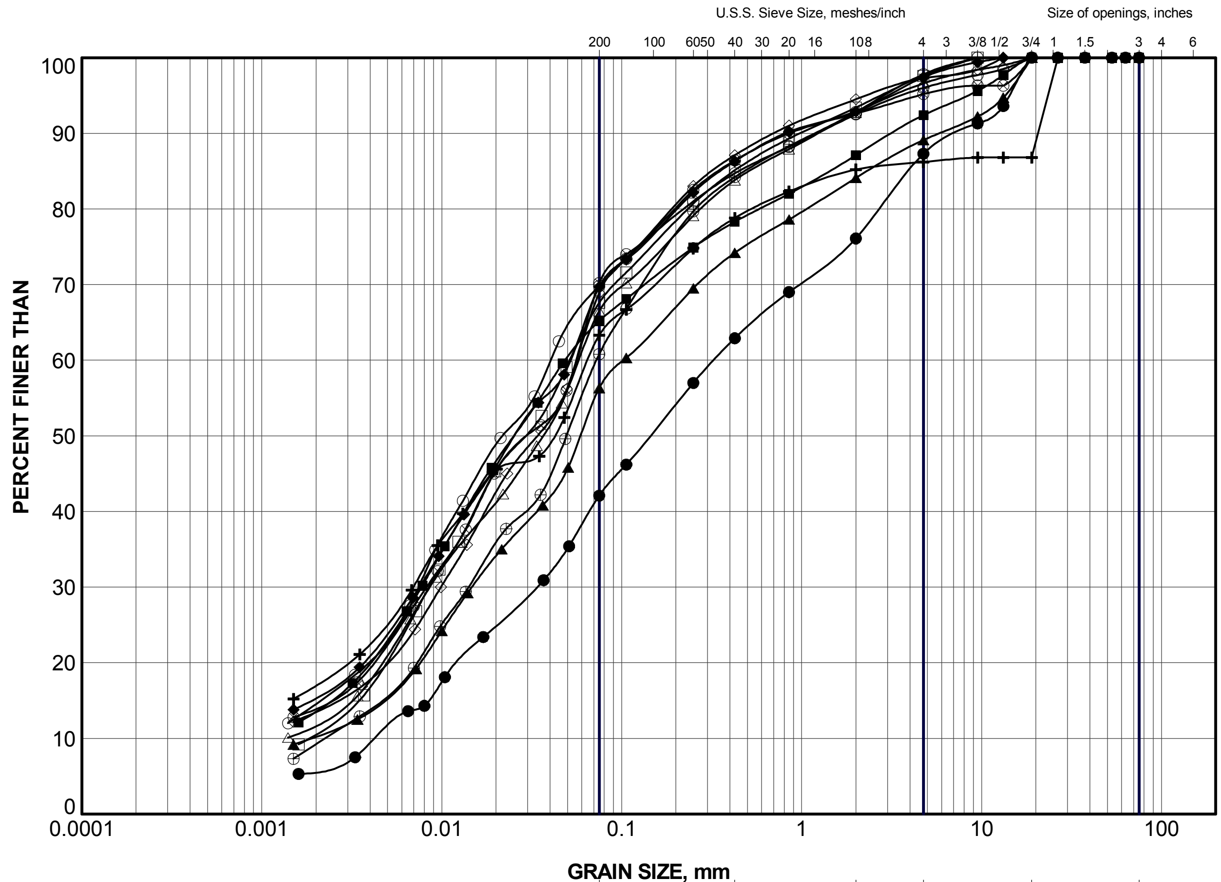
GRAIN SIZE DISTRIBUTION

SANDY SILT to SAND and SILT

Golder Associates
SUDBURY, ONTARIO

PROJECT No.	11-1191-0008		FILE No. 1111910008DET.GPJ
DRAWN	JJL	Oct 2013	SCALE N/A REV.
CHECK	SEMC	Oct 2013	
APPR	JMAC	Oct 2013	FIGURE B8

SUD-MTO GSD (NEW) GLDR_LDN.GDT



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	VS1	11	227.2
■	VS2	10	226.8
▲	VS3	9	227.1
✚	VS3	12	222.5
◆	VS4	8	228.6
◇	VS4	10B	225.4
○	VS5	10	226.4
△	VS6	9	228.3
⊗	VS8	10	228.0
⊕	VS10	11	228.4
□	VS11	10	227.6

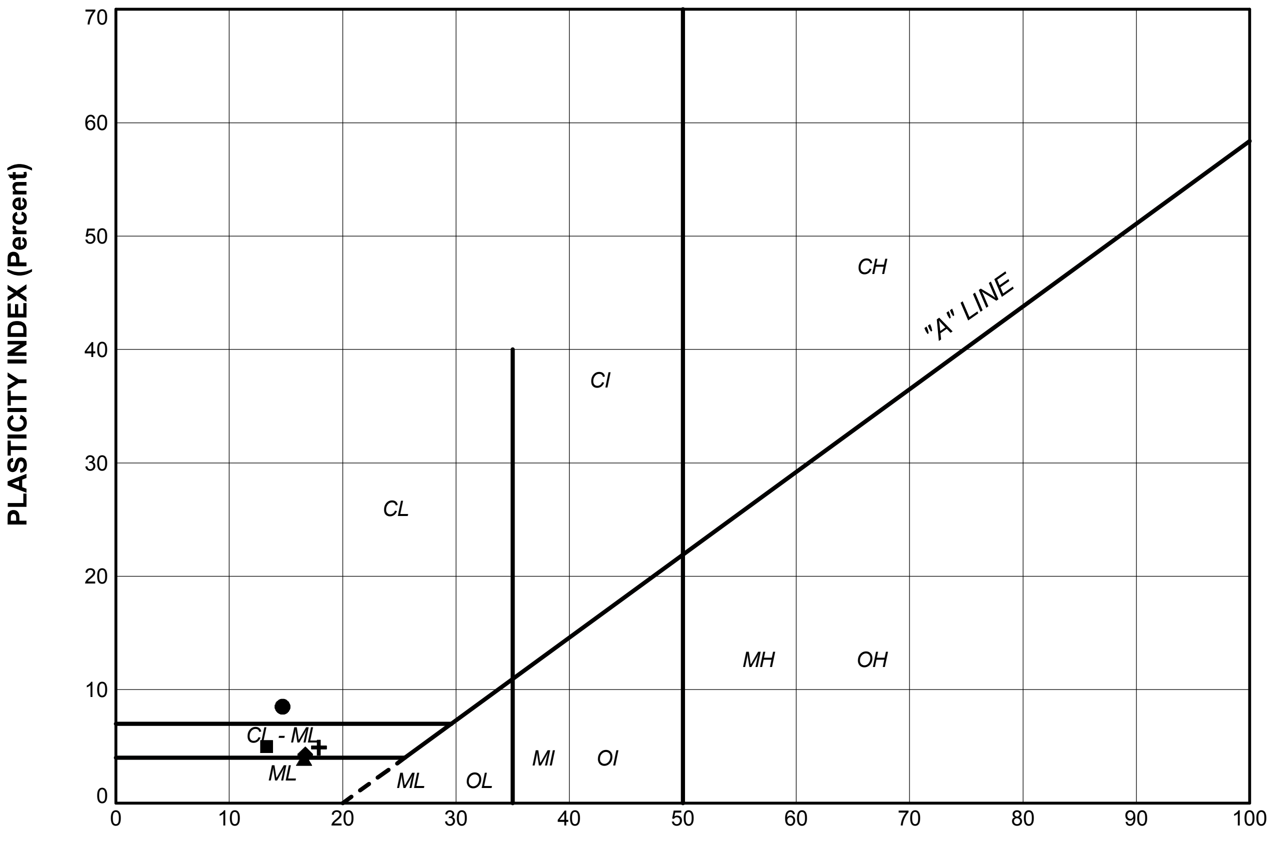
PROJECT
HIGHWAY 11
VALENTINE RIVER BRIDGE

TITLE
GRAIN SIZE DISTRIBUTION
SANDY CLAYEY SILT to CLAYEY SILT with SAND (TILL)

PROJECT No.		11-1191-0008	FILE No.	1111910008DET.GPJ
DRAWN	JJL	Oct 2013	SCALE N/A	REV.
CHECK	SEMC	Oct 2013		
APPR	JMAC	Oct 2013	FIGURE B9	

Golder Associates
SUDBURY, ONTARIO

SUD-MTO GSD (NEW) GLDR_LDN.GDT



SOIL TYPE
C = Clay
M = Silt
O = Organic

PLASTICITY
L = Low
I = Intermediate
H = High

LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	VS2	10	14.7	6.2	8.5
■	VS3	12	13.3	8.3	5.0
▲	VS4	8	16.6	12.7	3.9
✚	VS5	10	17.9	13.0	4.9
◆	VS6	9	16.7	12.4	4.3
◇	VS8	10	16.7	12.5	4.2

PROJECT
HIGHWAY 11
VALENTINE RIVER BRIDGE

TITLE
PLASTICITY CHART
SANDY CLAYEY SILT to CLAYEY SILT with SAND
(TILL)

Golder Associates
SUDBURY, ONTARIO

PROJECT No.	11-1191-0008		FILE No.	1111910008DET.GPJ	
DRAWN	JJL	Oct 2013	SCALE	N/A	REV.
CHECK	SEMC	Oct 2013			
APPR	JMAC	Oct 2013	FIGURE B10		

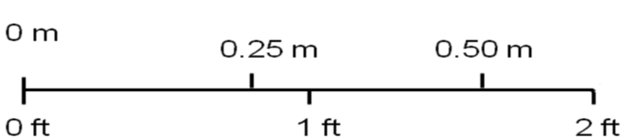
SUD-MTO PL (NEW) GLDR_LDN.GDT

Borehole VS3

Elevation 220.4 m to 217.1 m

Borehole VS4

Elevation 223.6 m to 220.4 m



PROJECT	HWY 11 Valentine River Bridge Site # 39W-010				
TITLE	BEDROCK CORE PHOTOGRAPHS (VS3 and VS4)				
Golder Associates	PROJECT No.	11-1191-0008		FILE No.	----
	DESIGN	EC	Aug 2013	SCALE AS SHOWN	REV.
	CADD	--		FIGURE B11	
	CHECK	SEMC	Aug 2013		
	REVIEW				

APPENDIX C

Non Standard Special Provisions

CSP FOR INTEGRAL ABUTMENTS – Item No.

Non-Standard Special Provision

Scope

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

SUBMISSION AND DESIGN REQUIREMENTS

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

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

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

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

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

MATERIAL

Corrugated Steel Pipe

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

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

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

Sand Fill

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

Table 1 – Sand Fill Gradation Requirements

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

CONSTRUCTION

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

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

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

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

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

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

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

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

Basis of Payment

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

H-PILES - Item No.

Non-Standard Special Provision

903.07.02 Driven Piles

903.07.02.01 Pile Driving Requirements and Restrictions

Section 903.07.02.01 of OPSS 903 is amended by the addition of the following:

The Contractor shall commence assessment of the ultimate axial resistance of the pile by the Hiley Formula (Standard Drawing SS-103-11) once the pile reaches a depth of 3.0 m above the design pile tip elevation shown in the Contract Drawings and at subsequent 0.5 m intervals of depth until the ultimate axial resistance is achieved. If the ultimate axial resistance as determined by the Hiley Formula is not achieved within the 3.0 m interval down to the design pile tip elevation the Contractor shall stop pile driving and notify the Contract Administrator. At this depth the pile should be allowed to rest for 48 hours, and the Hiley Formula shall then be applied immediately upon re-striking of the pile. If the ultimate axial resistance is still not achieved after the 48 hour wait period, the Contract Administrator shall be notified and authorization given prior to driving the pile below the design pile tip elevation.

The contractor shall have materials and equipment available on site to deal with varying pile lengths as the pile tip elevation (and hence length of pile) will depend on achieving the required geotechnical axial resistance as specified in the contract.

WORKING SLAB – Item No.

Non-Standard Special Provision

Scope of Work

This Special Provision covers the requirements for the supply and placement of a concrete working slab on a soil subgrade under the structure foundation for the Valentine River Bridge. The purpose of the working slab is to protect the subgrade from disturbance and loosening due to construction traffic and ponded water and also to provide a level working surface.

Construction

Protection of Founding Soil

- Following inspection and approval of the prepared soil subgrade by the Quality Verification Engineer, a working slab, with a minimum thickness of 100 mm shall be placed on the foundation subgrade as per the contract drawings and documents. The concrete shall have a minimum 28 day compressive strength of 20 MPa.

Unwatering carried out for the pile cap excavation shall be done in such a manner as to prevent any disturbance to the surrounding original soil.

Basis of Payment

Payment at the Contract Price for the above tender item includes full compensation for all labour, equipment and material to do the required work.

OBSTRUCTIONS

Non-Standard Special Provision

The Contactor is hereby notified that the soils at the site of the Valentine River Bridge are glacially derived and as such should be expected to contain cobbles and boulders, which could affect the installation of deep foundations and/or temporary shoring and roadway protection systems. Consideration of the presence of these obstructions must be made in selection of appropriate equipment and procedures for sub-excavation and installation of the foundation and temporary shoring and roadway protection systems.

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

As a global, employee-owned organisation with over 50 years of experience, Golder Associates is driven by our purpose to engineer earth's development while preserving earth's integrity. We deliver solutions that help our clients achieve their sustainable development goals by providing a wide range of independent consulting, design and construction services in our specialist areas of earth, environment and energy.

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