



October 25, 2013

FOUNDATION INVESTIGATION AND DESIGN REPORT

REPLACEMENT OF EVELYN CREEK BRIDGE
SITE NO. 39W-100
HIGHWAY 11, TOWNSHIP OF DEVITT, ONTARIO
MINISTRY OF TRANSPORTATION, ONTARIO
GWP 5150-05-00

Submitted to:
LEA Consulting Ltd.
625 Cochrane Drive, Suite 900
Markham, Ontario
L3R 9R9



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REPORT





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PART A

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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 detailed foundation engineering services for the replacement of the Evelyn Creek Bridge (Site No. 39W-100), located on Highway 11 (east of Hearst, Ontario) in the Township of Devitt.

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 Evelyn Creek Bridge structure is contained in Section 6.8 of LEA's 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 for this project, dated August 2011.

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

2.0 SITE DESCRIPTION

The Evelyn Creek Bridge site is situated in the Township of Devitt on Highway 11 approximately 3.7 km east of Val Côté, Ontario. The surrounding land is generally flat and gently slopes down towards the creek and from the highway towards the creek banks on either side of the existing bridge. The area is vegetated with grass and small shrubs. The creek is about 5 m wide at the existing bridge location and flows in a northerly direction.

The existing structure is a two-lane, single-span steel girder bridge with concrete deck and timber substructure/wing walls, constructed in 1960. The structure is supported on timber piles, likely founded in the native cohesionless soils. The existing roadway surface is at about Elevation 237.8 m as it crosses the creek. The existing embankment side slopes are generally formed at about 2 Horizontal to 1 Vertical (2H:1V).

A creek water level at Elevation 235.2 m was measured by others on October 24, 2011. The creek water levels measured by Golder during the field investigations, which took place in October 2012 and July 2013, were Elevation 235.3 m and 235.9 m, respectively.

3.0 INVESTIGATION PROCEDURES

The fieldwork for this subsurface investigation was carried out on October 19 and 20, 2012, June 6, 18, 19, 24, 25 and July 10, 2013, at which time eight boreholes, Boreholes E1 to E8, were advanced at the site. The boreholes were advanced using both a CME 55 track-mounted drill rig supplied and operated by Landcore Drilling Inc. of Sudbury, Ontario, and a D-25 semi-portable drill rig supplied and operated by Walker Drilling Ltd. of Barrie, Ontario. Boreholes E1 and E6 were advanced at the toes of slope, on the northwest and northeast sides of the creek, respectively, in the vicinity of the proposed north wing walls. Boreholes E3 and E4 were advanced through the existing Highway 11 embankment on the west side of the proposed replacement culvert structure. Boreholes E2 and E5 were advanced through the existing Highway 11 embankment along the east side of the proposed replacement (culvert) structure. Boreholes E7 and E8 were advanced for the proposed



embankment approaches up to about 20 m from the replacement structure. The borehole locations are shown on Drawing 1.

The boreholes were advanced using 108 mm inner diameter hollow-stem augers and/or NW casing and a NQ size core barrel where coring through cobbles/boulders 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 (track rig) or cathead hammer (D-25 rig), in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586). Field vane shear tests were carried out in cohesive soils (strata) for determination of undrained shear strengths (ASTM D2573) using an MTO Standard 'N' size vane. All open boreholes were backfilled upon completion in accordance with Ontario Regulation 903 Wells (as amended).

The groundwater conditions were observed in the open boreholes during and immediately following the drilling operations and a standpipe piezometer was installed in Borehole E1 to permit monitoring of the groundwater level. The piezometer consists of a 50 mm diameter polyvinyl chloride (PVC) pipe, with a 1.5 m long slotted screen, sealed within a sand filter pack at a selected depth interval within the borehole. Below the piezometer screen the borehole was backfilled with bentonite pellets, and 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 borehole and bentonite. A seal of bentonite was placed to ground surface. The piezometer installation details and water level readings are indicated on the Record of Borehole sheets contained in Appendix A. The non-instrumented boreholes were backfilled with bentonite as per Ontario Regulation 903 (as amended) upon completion of drilling. The piezometer was decommissioned on June 25, 2013, in accordance with the regulations.

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 operations 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. The geotechnical laboratory testing was completed according to applicable MTO LS standards. The results of the laboratory testing are shown on the Record of Borehole sheets in Appendix A and on figures contained in Appendix B.

A sample of the creek 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.

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 J.D. 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 Drawing 1, 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)
E1	5499526.1	351274.2	236.1	21.4
E2	5499510.9	351281.0	237.8	17.1
E3	5499510.6	351270.4	237.7	21.5
E4	5499507.0	351267.6	237.7	19.9
E5	5499502.5	351284.5	237.7	18.6
E6	5499523.8	351286.1	236.1	19.9
E7	5499513.4	351250.4	237.8	15.8
E8	5499505.5	351299.6	237.8	15.8

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

Based on NOEGTS¹ Mapping, the subsoils in the vicinity of the Evelyn Creek 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)². 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.

4.2 Subsurface Conditions

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 presented on the Record of Borehole sheets, contained in Appendix A. The results of geotechnical laboratory testing are also presented on Figures B1 to B8, contained 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 profiles and cross-sections on Drawings 1 and 2, are inferred from non-continuous sampling, observation of drilling progress and the soil cuttings 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 generally consist of the existing pavement structure and fill comprising the existing embankment, underlain by a deposit of peat/organic clay, which was also

¹ Northern Ontario Engineering Geology Terrain Study. Ontario Geological Society Map Reference Number 42GNW.

² 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.



encountered from ground surface at the toe of the existing embankment slope. Underlying the organic deposit the boreholes penetrated a deposit of soft to stiff clay, underlain by a deposit of very soft to stiff sandy clayey silt to silt, underlain by deposits of loose to dense gravelly sand and silt to sand and silt and very dense sand and silt till. Boulders were encountered within the sand and silt deposit in one borehole.

4.2.1 Asphalt, Concrete

Boreholes E2 to E4, advanced through the approach slabs, penetrated a 100 mm to 150 mm thick layer of asphalt, underlain by a 230 mm to 280 mm thick layer of concrete. A 420 mm and 115 mm void was noted below the concrete in Boreholes E2 and E3.

Boreholes E5, E7 and E8, advanced through the roadway approach embankments and shoulder, penetrated an asphalt layer ranging between 120 mm to 150 mm thick.

4.2.2 Fill

Embankment fill consisting of brown, gravelly sand to silty sand to sand was encountered below the concrete in Borehole E4, at the bottom of the void in Boreholes E2 and E3 and underlying the asphalt in Boreholes E5, E7 and E8. The surface of the fill was encountered between Elevation 237.7 m and 237.0 m and ranges in thickness from 1.8 m to 2.2 m.

The SPT 'N'-values measured within the fill deposit range from 4 blows to 31 blows per 0.3 m of penetration indicating a loose to dense relative density.

The results of grain size distribution testing completed on two samples of the sand fill are shown on Figure B1 in Appendix B.

The natural moisture content measured on two samples of the fill are about 9 per cent and 12 per cent.

4.2.3 Peat/Organic Silt

A deposit of black, fibrous to amorphous peat or organic silt was encountered from ground surface in Boreholes E1 and E6 north of the roadway area and underlying the fill in Boreholes E3, E4, E5 and E8. The top of this organic layer was encountered between Elevation 236.1 m and 235.1 m, and the thickness of the deposit ranges between 0.4 m and 2.0 m.

The SPT 'N'-values measured within the organic deposit range from 1 blow to 14 blows per 0.3 m of penetration, suggesting a very soft to stiff consistency.

Atterberg limits testing carried out on two samples of the organic silt returned liquid limits of about 101 per cent and 131 per cent, plastic limits of about 43 per cent and 74 per cent and a plasticity index of about 58 per cent. The results of the Atterberg limits test are shown in the plasticity chart on Figure B2 in Appendix B and indicate that the material is comprised of organic silt of high plasticity.



The natural moisture content measured on four samples of the organic deposit ranges from about 42 per cent to 130 per cent. The organic content measured on one sample of the organic silt is 7.2 per cent.

4.2.4 Clay

A deposit of brown to grey clay was encountered underlying the fill and/or organic deposit in Boreholes E1, E2, E4, E7 and E8. The top of the clay deposit was encountered between Elevation 235.6 m and Elevation 234.7 m, and the thickness of the deposit ranges between 1.4 m and 3.1 m.

SPT 'N'-values measured within the clay deposit range from 2 blows to 13 blows per 0.3 m of penetration. In situ field vane tests carried out within this stratum measured undrained shear strengths ranging from 35 kPa to greater than 100 kPa with the calculated sensitivity ranging from about 6 to 9. The SPT 'N'-values together with the in situ vane test result suggest that the clay deposit generally has a firm to stiff consistency.

The grain size distributions for two samples of the clay deposit are shown on Figure B3 in Appendix B.

Atterberg limits testing carried out on five samples of the clay deposit yielded liquid limits ranging from about 50 per cent to 67 per cent, plastic limits ranging from about 19 per cent to 22 per cent and plasticity indices ranging from about 29 per cent to 45 per cent. The results of the Atterberg limits testing are shown on the plasticity chart on Figure B4 and indicate that the deposit is classified as clay of high plasticity.

The natural moisture content measured on five samples of the clay deposit ranges from about 26 per cent to 38 per cent.

4.2.5 Sandy Clayey Silt to Silt

A deposit of grey sandy clayey silt to silt, containing trace to some gravel was encountered underlying the clay and/or the organic deposit in all of the boreholes. The surface of this deposit was encountered between Elevation 234.7 m and 231.9 m, and where penetrated, the thickness of the deposit ranges from 6.1 m to 14.0 m. Borehole E8 did not penetrate the deposit after exploring for 9.9 m. In Borehole E1, approximately 6.1 m of heaved material was noted inside the augers when advanced to a depth of 15.8 m and in Borehole E5, approximately 1.5 m of heaved material was noted inside the augers at a depth of 15.8 m. A sand and silt seam was encountered within this deposit at 7.9 m below ground surface (Elevation 230.1 m) in Borehole E2.

SPT 'N'-values measured within this deposit range from 0 blows (weight of hammer) to 36 blows per 0.3 m of penetration. In situ field vane tests carried out in this deposit measured undrained shear strengths ranging from 19 kPa to 98 kPa with 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 silt deposit generally has a soft to very stiff consistency.

The grain size distributions of sixteen samples of the sandy clayey silt to silt deposit are shown on Figure B5 in Appendix B.

Atterberg limits testing carried out on twelve samples of the clayey silt to silt yielded liquid limits ranging from about 15 per cent to 32 per cent, plastic limits ranging from about 11 per cent to 19 per cent, and plasticity indices ranging from about 4 per cent to 15 per cent. The results of the Atterberg limits testing are shown on the



plasticity chart on Figure B6 and indicate that the deposit consists of clayey silt of low plasticity to silt of slight plasticity. One sample was determined to be non-plastic.

The natural moisture content measured on samples of the sandy clayey silt to silt deposit ranges from about 12 per cent to 51 per cent.

The grain size distribution of a sample of the sand and silt seam is shown on Figure B7 in Appendix B. The natural moisture content measured on the sample of the sand and silt seam is about 20 per cent.

4.2.6 Boulders

In Borehole E3, at the bottom of the clayey silt deposit, a 0.3 m boulder was encountered at Elevation 225.1 m. Within the underlying gravely sand and silt deposit discussed further below, a 1.1 m boulder was encountered at Elevation 222.8 m.

4.2.7 Sand and Silt to Gravely Sand and Silt

A deposit of grey sand and silt to gravely sand and silt was encountered below the sandy clayey silt to silt in Boreholes E3 to E7. The surface of this deposit was encountered between Elevations 227.4 m and 224.4 m and the thickness of the deposit ranges from 1.6 m to 4.7 m. Borehole E7 was terminated after exploring the deposit for 5.4 m.

The measured SPT 'N' values within the sand and silt deposit range from 6 blows to 47 blows per 0.3 m of penetration, indicating a loose to dense relative density.

The grain size distributions for three samples of the sand and silt deposit are shown on Figure B7 in Appendix B

An Atterberg limits test on a sample from this deposit indicated that the material is non-plastic.

The natural moisture content measured on three samples of the sand and silt deposit range from about 14 per cent to 44 per cent.

4.2.8 Sand and Silt (Till)

A deposit of grey sand and silt to gravel and sand and silt till was encountered below the sandy clayey silt to silt deposit in Boreholes E1 and E2 and below the sand and silt or gravely sand and silt deposits in Borehole E3 to E6. The surface of this deposit was encountered between 224.5 m and 218.3 m and the boreholes were terminated after exploring the deposit between 3.5 m and 5.2 m into the deposit.

Difficult casing advancement was noted throughout this till deposit and coring techniques were required to advance the borehole at several depths throughout. A 0.2 m size cobble was encountered in Borehole E2 at a depth of 14.9 m below ground surface (Elevation 222.9 m).

The measured SPT 'N' values within the till deposit range from 71 blows to greater than 100 blows per 0.3 m of penetration, indicating a very dense relative density.

The grain size distributions for five samples of the till deposit are shown on Figure B8 in Appendix B.



The natural moisture content measured on five samples of the till deposit range from about 9 per cent to 13 per cent.

4.3 Groundwater Conditions

Groundwater levels were measured in the open boreholes during and upon completion of drilling and a piezometer was installed in Borehole E1 and sealed within the clay and sandy clayey silt to silt deposits. The measured groundwater levels in the open boreholes and piezometer are presented below.

Borehole	Installation	Date	Groundwater Depth (m)	Groundwater Elevation (m)
E1	Open Borehole	October 20, 2012	1.2	234.9
	Piezometer	December 13, 2012	1.1	235.0
	Piezometer	June 25, 2013	0.5	235.6
E2	Open Borehole	October 19, 2012	2.4	235.4
E3	Open Borehole	June 25, 2013	3.7	234.0
E4	Open Borehole	June 19, 2013	2.7	235.0
E5	Open Borehole	June 18, 2013	3.0	234.7
E6	Open Borehole	July 10, 2013	3.6	232.5
E7	Open Borehole	June 19, 2013	8.8	229.0
E8	Open Borehole	June 19, 2013	11.4	226.4

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 creek water) levels in the area are subject to seasonal fluctuations and to fluctuations after precipitation events and snowmelt. The water level in Evelyn Creek was measured at Elevation 235.3 m on October 20, 2012, and at Elevation 235.9 m on July 26, 2013.

5.0 CLOSURE

The field drilling program was supervised by Mr. Indulis Dumpis and Mr. Ed Savard. This Detail Foundation Investigation Report was prepared by Mr. Adam Core, E.I.T. and Mr. Evan Childerhose, P.Eng., and the technical aspects were reviewed by Ms. Sarah Coyne, P.Eng. Messrs. Fintan Heffernan, P.Eng., and Jorge Costa, P.Eng., Designated MTO Foundations Contacts and Principals with Golder, conducted independent quality control reviews of this report.



Report Signature Page

GOLDER ASSOCIATES LTD.

Evan Childerhose, P.Eng.
Geotechnical Engineer



Sarah E. M. Coyne, P.Eng.
Senior Geotechnical Engineer, Associate



Jorge M.A. Costa, P.Eng.
Designated MTO Contact, Principal

AC/EC/SEMC/JMAC/kp

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6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides foundation design recommendations for the proposed replacement of the Evelyn Creek Bridge on Highway 11, east of Hearst, Ontario. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the subsurface investigations at the site.

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 single span Evelyn Creek Bridge structure was constructed in 1960 and the abutments are supported on piles, likely founded within the native cohesionless soils. We understand that the existing bridge will be replaced with a precast concrete open footing culvert with a clear span of 7.7 m and will be constructed in two stages. The existing embankment at the proposed culvert location is approximately 1.7 m high above the surrounding ground surface. The finished grade for the new Highway 11 alignment will be Elevation 238.3 m, which is up to 0.6 m higher than the existing grade. The culvert will be 15.0 m long to accommodate both a 1.9 m highway alignment shift to the north as well as the embankment grade raise. Approximately 4.5 m long wing walls are required at the culvert ends.

The subsurface conditions at the culvert location generally consist of sand to gravelly sand fill comprising the existing embankment, underlain by a deposit of peat/organic silt, which was also encountered from ground surface at the toe of the existing embankment slope. Underlying the organic deposit is a deposit of a soft to stiff clay, underlain in turn by deposits of soft to hard sandy clayey silt to silt, loose to dense sand and silt, and very dense sand and silt till.

6.2 Foundation Options

Based on the proposed culvert geometry and the subsurface conditions at this site, both shallow and deep foundation options have been considered for support of the footings of the replacement structure. The following sections provide recommendations for both shallow and deep foundation options. A comparison of the foundation options based on advantages, disadvantages, risks/consequences and relative costs is provided in Table 1 following the text of this report. From a foundations perspective, we recommend founding the culvert and wing walls on strip footings founded on the firm sandy clayey silt to silt deposit on an engineered fill pad (i.e., tremie concrete plug) constructed on this deposit. If the geotechnical resistances available are inadequate for the proposed open footing design, then deep foundations comprised of driven steel H-piles will need to be utilized.

Based on the subsurface conditions and the proposed size of the culvert at this site, a concrete box culvert is not recommended for the replacement structure.



6.3 Shallow Foundations

From a foundations perspective, strip/spread footings would be feasible for supporting the culvert and wing walls at this site. Due to the depth to competent material below the groundwater level at this site, and the requirement for footings to be constructed in-the-dry, consideration could be given to installation of a tremie concrete plug (within steel sheeting) placed at the proposed founding elevation and then founding the concrete footing on top of the tremie plug. The following sections provide details for foundation elevations, geotechnical axial resistances, resistance to lateral loads and frost protection.

6.3.1 Founding Elevations

Footings for support of the culvert and wing walls could be founded on the firm sandy clayey silt to silt deposit at the founding elevations given below. Alternatively, provided the thickness of the tremie concrete plug meets the hydraulic requirements, the base of the tremie concrete plug could be placed at the elevations given below.

Location	Reference Boreholes	Maximum (Highest) Founding Elevation (m)	Approximate Excavation Depth (m)		Approximate Depth of Excavation Below Creek Level ¹ (m)
			Below Existing Road Grade (midpoint)	Below Surrounding Ground (north side)	
West Footing	E1, E3, E4	232.0	5.7	4.1	3.2
East Footing	E2, E5, E6	232.0	5.8	4.1	3.2

1. Assumes a creek water level at Elevation 235.2 m (measured by LEA October 24, 2011).

6.3.2 Geotechnical Resistance

Given that the elevations provided above are likely too deep to allow for construction of strip footings in-the-dry, it is likely that a tremie concrete plug will be required and placed directly on the sandy clayey silt to silt deposit below the water level.

The tremie concrete plug and concrete footing system should be designed based on a factored geotechnical axial resistance at Ultimate Limit States (ULS) of 300 kPa and a geotechnical reaction at Serviceability Limit States (SLS) (for 25 mm of settlement) of 100 kPa for tremie concrete plug widths up to about 3 m. The thickness of the tremie plug is discussed in Section 6.7.3. If settlements of up to 37 mm can be accommodated, an SLS value of 120 kPa can be used. These geotechnical resistances are provided for loads applied perpendicular to the surface of the footings; where applicable, inclination of the load should be taken into account in accordance with Section 6.7.4 of the Canadian Highway Bridge Design Code (CHBDC 2006) and its Commentary.

Further, given that the tremie concrete is placed under water, the subgrade soils at Elevation 232 m will not be able to be examined and the geotechnical resistances reflect this.



6.3.3 Resistance to Lateral Loads/Sliding Resistance

Resistance to lateral forces/sliding resistance between the base of the tremie concrete plug and the firm clayey silt to silt deposit should be calculated in accordance with Section 6.7.5 of the CHBDC. The coefficient of friction, $\tan \delta$, may be taken as 0.3 between tremie concrete plug and the subgrade. We understand that pre-cast footings are being proposed for the structure. We do not recommend that pre-cast units be placed directly on the tremie concrete plug unless a 100 mm thick concrete working slab is placed on the tremie concrete plug or the pre-cast unit is dowelled into the tremie concrete plug or the gap between the pre-cast unit and tremie concrete plug is grouted. Dowelling or grouting may be difficult to carry out in this case. For pre-cast concrete units placed over a concrete working slab, $\tan \delta$ may be taken as 0.6. For cast-in-place concrete units over the tremie concrete plug, $\tan \delta$ may be taken as 0.7. These values represent unfactored values.

6.3.4 Frost Protection

All footings 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 Deep Foundations

As an alternative to shallow foundations, the culvert footings and wing walls may be supported on steel HP 310X110 piles driven to terminate into the very dense sand and silt to sand and gravel till deposit ('N'-values greater than about 100 blows per 0.3 m of penetration). The following sections provide details for geotechnical axial resistances/reactions, set criteria and pile driving notes, resistance to lateral loads and frost protection for deep foundations.

6.4.1 Founding Elevations

In very dense till deposits, driven piles typically penetrate 2 m to 3 m into the deposit. The surface of the very dense till varies across the length of the footings being highest towards the south. For design, the lowest pile tip elevation has been provided (i.e., towards the north), assuming a penetration of about 3 m into the very dense till.

Location	Reference Boreholes	Estimated Design Pile Tip Elevation* (m)	Assumed Underside of Pile Cap Elevation (m)	Estimated Approx. Pile Length (m)
West Foundation	E1, E3, E4	215 - 220	234	14 - 19
East Foundation	E2, E5, E6	217 - 220	234	14 - 17

*Pile lengths would be shorter at the south end of the structure where the estimated tip elevation could be as high as Elevation 220 m.



The presence of cobbles and boulders within the sand and silt deposit and potentially within the till deposits could result in piles ‘hanging-up’ or being deflected from their intended vertical alignment. Therefore, consideration should be given to using a heavier H-pile section, such as HP310x132 as an alternative to HP310X110. Steel H-piles are preferred over steel pipe piles, as steel H-piles are considered to have a lesser potential of ‘hanging-up’ or being deflected away from their vertical or battered orientation during installation.

If corrugated steel pipes (CSPs) are installed as part of the integral abutment design (through which the piles will be driven), which we understand is the case for this site, the CSPs should be backfilled with a loose, fine to medium sand.

6.4.2 Axial Geotechnical Resistance

For end bearing piles, the factored geotechnical axial resistance at Ultimate Limit States (ULS) is a combination of the shaft and toe resistance, utilizing a factor of 0.5 on the calculated ultimate resistance in accordance with the CHBDC (2006) and current MTO Foundations practice. The axial resistance at SLS (for 25 mm of settlement) assumes that the pile will settle approximately 10 mm to 15 mm to mobilize shaft friction. The ULS and SLS values for two different pile types driven to the elevations given above are presented below. Generally, HP310X110 piles are used; however, a heavier pile section, HP310X132, is recommended due to the cobbles and boulders likely present within or above the till deposit.

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	1,200

The estimated tip elevations assume that the piles will penetrate about 3 m into the very dense till deposit. Due to the presence of cobbles and boulders (up to 1.1 m diameter boulder size particles) within the sand and silt deposit and potentially within the till deposit, the piles could “hang-up” or be deflected prior to reaching the design tip elevation. At the west abutment (Borehole E3), boulders were encountered at 12.6 m and 14.9 depths (Elevation 225.1 m and 222.8 m, respectively). At the east abutment (Borehole E2), a 0.2 m thick cobble was encountered at 14.9 m depth (Elevation 222.9 m). It may be possible for the piles to “hang-up” within these zones, however, these zones appear to be discontinuous across the abutments.

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. A Non Standard Special Provision (NSSP) detailing the installation method and gradation of this sand should be included in the Contract Documents; an example is provided in Appendix C.

6.4.3 Set Criteria and Pile Driving Note

All pile installation/driving should be in accordance with OPSS 903 (Deep Foundations). The piles should be provided with driving shoes 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 sand and silt deposit and the till deposit and potential for damage to the pile tip during driving, consideration could also 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 into the very dense till deposit and to also avoid overdriving and possibly damaging the piles.

The pile capacity must be verified in the field by the use of the Hiley Formula in accordance with Standard Drawing SS103-11 (April, 2008) "Pile Driving Control", during the final stages of driving, starting about 2 m to 3 m higher than the tip elevations provided in Section 6.4.1. The ultimate geotechnical axial resistance predicted from the Hiley Formula should then be multiplied by a geotechnical resistance factor equal to 0.5 in accordance with Table 6.1 in the CHBDC (2006) 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 Note 2 in Clause 3.3.3 of the Structural Manual (MTO, 2008).

For HP310X132 piles, the note should read:

- Piles to be driven in accordance with Standard Structural Drawing SS 103-11 using an ultimate geotechnical resistance of 3,600 kN per pile but must be driven below Elevation 220 m.

For HP310X110 piles, the note should read:

- Piles to be driven in accordance with Standard Structural Drawing SS 103-11 using an ultimate geotechnical resistance of 3,200 kN per pile but must be driven below Elevation 220.

6.4.4 Resistance to Lateral Loads

The design of piles subjected to lateral loads should take into account such factors as the batter of the piles (if any), the relative rigidity of the pile to the surrounding soil, the fixity condition at the head of the pile (pile cap level), the structural capacity of the pile to withstand bending moments, the soil resistance that can be mobilized, the tolerable lateral deflections at the head of the pile and pile group effects. For design purposes, both the structural and geotechnical resistances should be evaluated to establish the governing case. Lateral loading could be resisted fully or partially by the use of battered piles.

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



for non-cohesive soils:

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

where:

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

and for cohesive soils:

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

where:

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

Where an integral abutment 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 at this site are summarized below.

Location (Relevant Boreholes)	Soil Unit	Elevation (m)	n_h (kPa/m)	s_u (kPa)
West Foundation (E3)	Loose Sand within CSP	234.0 to 231.0	1,300	-
	Firm to Stiff Clayey Silt	231.0 to 225.1	-	40
	Compact to Very Dense Sand and Silt (Boulders at Elev. 225.1 m and 223.8 m)	225.1 to 219.9	11,000	-
	Very Dense Sand and Silt Till	219.9 to 216.2	11,000	-
East Foundation (E2)	Loose Sand within CSP	234.0 to 231.0	1,300	-
	Firm to Stiff Clayey Silt to Silt	231.0 to 224.5	-	40
	Very Dense Sand and Silt Till (Cobbles at Elev. 223 m)	224.5 to 221.0	11,000	-

For a single HP310X110 or HP310X132 extending to the design tip elevations provided in Section 6.4.1 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	55	20
HP310X132	60	20

The lateral resistances give above are based on a vertical load of 1,000 kN per pile. The lateral resistance should be reviewed for vertical loads greater or less than 1,000 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.

6.4.5 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).

6.5 Lateral Earth Pressures

The lateral earth pressures acting on the side walls and wing walls of the culverts will depend on the type and method of placement of backfill materials, the nature of 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 the culverts and wing walls. It should be noted that these design recommendations and parameters assume level backfill and ground surface behind the walls. Where there is sloping ground behind the walls, the coefficient of lateral earth pressure must be adjusted to account for the slope.

- Select, free draining granular fill meeting the requirements of SP110S13 (Aggregates) Granular 'A' or Granular 'B' Type II but with less than 5 per cent passing the 200 sieve (0.075 mm) should be used as backfill behind the culverts. Longitudinal drains and weep holes should be installed in the walls to provide positive drainage of the granular backfill. Compaction (including type of equipment, target densities, etc.) should be carried out in accordance with SP 105S21 (Compacting). Other aspects of the granular backfill requirements with respect to sub drains and frost taper should be in accordance with OPSD 3121.150 (Walls Retaining, Backfill).
- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the culverts and wing walls, 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 may be placed either in a zone with the width equal to at least 2.6 m behind the back of the wall (see 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 wall (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 culvert structure and wing walls allow for lateral yielding, active earth pressures may be used in the geotechnical design of the structures. If the culvert structure and wing walls do not allow lateral yielding, at-rest earth pressures should be assumed for geotechnical 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

As part of the replacement of the Evelyn Creek structure, the new Highway 11 centreline will be located 1.9 m to the north of the existing centreline, with the embankment grade at Elevation 238.3 m, approximately 0.6 m higher than the existing grade.



Along the north toe of slope, below the proposed widened embankment footprint, peat/organic silt was encountered from ground surface and extends up to 0.8 m below the ground surface at the west abutment (Elevation 235.3 m) and up to 2.0 m below existing ground surface at the east abutment (Elevation 234.1 m). The analysis assumes that the peat below the widened embankment footprint will be removed.

Below the existing highway alignment, the peat/organic silt deposit was encountered below the existing highway embankment. The organic deposit is up to 1.0 m thick with the base of the organics between about 2.8 m and 3.2 m depth below the existing highway grade (between Elevations 234.5 m and 235.0 m). The peat deposit was not encountered in all of the boreholes, namely Borehole E2 at the centre of the east abutment and Borehole E7 at the west approach. It is assumed that the peat/organic silt deposit will be left in place below the existing embankment except where removal is required to facilitate the excavation in the widening areas.

The stability and settlement analysis for the embankments are discussed in Section 6.6.2 and 6.6.3, respectively. The geometry of the proposed approach embankments, existing ground surface and existing creek bed included in the analyses are based on the information from the GA drawing.

6.6.1 Approach Embankment Stability

Limit equilibrium slope stability analyses were performed for the proposed embankment geometry 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 on MTO projects for the design of embankment slopes under static conditions. This FoS is considered adequate for the embankment at this site considering the design requirements and the field data available and is based on deep-seated, global failure surfaces that would affect the operation of the roadway. The stability analyses were performed to check that the target minimum FoS was achieved for the embankment height and geometry at the culvert location.

For the existing cohesionless fill and native cohesionless soils, effective stress parameters were employed in the stability 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 NAVFAC (1986) were employed and the results were tempered by engineering judgment based on precedent experience in similar soils.

For the cohesive deposits, total stress parameters were employed in the analysis. The total stress parameters (i.e., undrained shear strength – s_u) for the cohesive soil were assessed based primarily on the results of the in situ field vane tests.

The simplified stratigraphy together with the associated strength and unit weight employed for the different soil types are summarized below.



Soil Deposit	Bulk Unit Weight (kN/m ³)	Effective Friction Angle	Undrained Shear Strength (kPa)
New Granular 'B' Type I or II (embankment fill around the culvert and for sub-excavated areas)	21	35°	-
Existing Gravelly Sand to Silty Sand (Fill)	21	32°	-
Peat/organic Silt (at toe of north slope)	12	27°	1
Peat/organic Silt (below existing embankment)	12	28°	1
Firm to Stiff Clay	17	-	35
Firm to Stiff Sandy Clayey Silt to Silt	18	-	40
Loose to Very Dense Sand and Silt	21	32°	-
Very Dense Sand and Silt (Till)	21	35°	-

The stability analysis performed on the proposed embankment at the culvert location indicates that after embankment widening, the embankment will have a FoS of 1.3 or greater for deep-seated, global failure surfaces that would impact the operation of the roadway, as shown on Figure 1.

6.6.2 Approach Embankment Settlement

Settlement of the new approach embankments can be expected as a result of the loading from the new fills on the foundation soils at this site. In addition, settlements may also occur due to compression of the embankment fill itself; however, typically for well compacted granular fill (i.e., Granular 'B' Type I or Type II), settlement of the new granular embankment fill is minimal. To estimate the magnitude of the expected settlements, analyses were carried out on the critical section of the proposed embankment using hand and spreadsheet calculations.

Although a culvert will replace the existing bridge structure, the MTO's post-construction settlement guidelines for a bridge structure have been used for settlement design guidelines for the embankments on either side of the culvert. Based on MTO's "*Embankment Settlement Criteria for Design*" Final Draft dated March 2, 2010, the following post-construction settlement for transitions 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 (Non-Freeways)	0 m to 20 m	25
	20 m to 50 m	50
	50 m to 75 m	100

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



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

The following correlation relating in-situ undrained shear strength to preconsolidation 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 upper clay and the clayey silt to silt deposit 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 consolidation parameters for the peat/organic silt were estimated using correlations with water content from the Muskeg Engineering Handbook (1969). Due to the minimal widening and grade raise required at this site, the preconsolidation pressures of the organic and cohesive deposits are not anticipated to be exceeded and are therefore considered to be overconsolidated.

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

Soil Type	Maximum Thickness (m)	γ (kN/m ³)	E (MPa)	σ_{vo}' (kPa)	σ_p' (kPa)	OCR	e_o	C_r	C_c
Peat	1.0 (under existing embankment)	12	-	55	~100	~2	0.70	0.5	5.0
Clay (Firm to stiff)	3.0	17	-	60	160	2.7	0.94	0.06	0.6
Sandy Clayey Silt to Silt (firm to stiff)	14.0	18	-	90	180	2.0	0.60	0.03	0.3
Sand and Silt (loose to very dense)		21	25	-	-	-	-	-	-

The immediate settlement of the existing and new fill materials, properly benched and compacted, is expected to occur during construction and will be less than 25 mm. If the backfill material will be placed sub-aqueously, the Granular 'B' Type II could settle up to about 25 mm for every 1 m of material and will generally occur immediately following embankment construction. The immediate settlement of the firm to stiff clayey silt to silt deposit is expected to occur during construction and will be less than 50 mm under the widened portion of the embankment and less than about 10 mm under the existing embankment. Settlement of the compact to very dense sand and silt and very dense sand and silt till deposits is expected to be negligible.



In the widened area, where the peat is to be removed, the settlement of the clay is estimated to be approximately 50 mm and is expected to occur during or shortly after construction. It is estimated that the settlement will occur within about one month after construction of the widened embankment to the final grade. We understand that approximately 2 weeks will transpire between construction to final grade and paving of each half of the highway (staged construction). Therefore, the estimated post-construction settlement of the clay deposit after a minimum 2 week period is estimated to be less than 25 mm and therefore meets the post-construction settlement criteria and therefore mitigation is not required.

Under the existing embankment, the settlement of the clay deposit under the 0.6 m grade raise is expected to be about 5 mm and will occur during construction. If the peat is to be left in place, settlement of the peat/organic silt at the east approach (i.e., from immediately behind the cofferdam to 20 m behind the abutments) is estimated to be about 25 mm. At the west approach, immediately behind the cofferdam, the settlement of the peat is estimated to be about 15 mm. Settlement of the peat is expected to occur shortly after construction, however, some or all of the estimated settlement is expected to be post-construction. The magnitude of post-construction peat settlement meets the settlement criteria and therefore mitigation is not required.

Due to the proposed staged construction, the post-construction settlement of the peat and clay deposits, where present, will occur as differential settlement. Since more settlement is estimated to occur under the widened portion of the embankment compared to under the existing embankment, we recommend that the north half of the structure be constructed first to allow the maximum passage of time before final paving.

6.7 Construction Considerations

6.7.1 Embankment Construction

The new embankment fill should be placed and compacted in accordance with OPSS 501 (Compacting) and MTO's SP 206S03 (Earth Excavation and Grading). The new fill should be "keyed-in" or benched into the existing fills, in accordance with OPSD 208.010 (Benching of Earth Slopes).

Below the water table, granular fill for construction of the new embankment should consist of SP 110S13 (Aggregates) Granular 'B' Type II material; above the water table, Granular 'B' Type I material may be used. Side slopes for granular fill should be no steeper than 2H:1V. Based on the minimal widening and grade raise at this site, rock fill is not suggested for embankment construction or backfilling at this site.

The culvert side slopes beyond the wing walls and adjacent to the creek 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. 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 footings or pile caps in the design of the foundations.

To reduce surface water erosion on the embankment side slopes, topsoil and seeding as per OPSS 802 (Topsoil) and OPSS 804 (Seed and Cover) should be carried out as soon as possible after construction of the embankments. 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.

6.7.2 Excavations and Temporary Cut Slopes

The proposed works will require excavations through the embankment fill behind the existing abutments in order to replace the existing abutments and other components of the bridge with the footings or pile caps for the proposed culvert.

The widening to the north will require sub-excavation of the peat/organic silt deposit, the base of which was encountered up to 0.8 m below the existing ground surface at the west abutment (Elevation 235.3 m) and up to 2.0 m below existing ground surface at the east abutment (Elevation 234.1 m). Sub-excavation of the organics should be carried out for 20 m behind each abutment and in the widened construction areas in accordance with OPSD 203.020 (Embankments over Swamp, Existing Slope Excavated to 1H:1V). The sub-excavation can be carried out sub-aqueously using Granular 'B' Type II as discussed above. Peat removal is not required below the existing embankment outside the widening area.

Excavations for pile cap or footing construction will extend to Elevation 232 m. This will result in excavations up to 5.8 m below the top of roadway surface or up to 4.1 m below the ground surface at the north toe of slope. It is expected that the excavations will require temporary shoring and groundwater control, as discussed in Section 6.7.3.

The depth of excavation required for the peat sub-excavation and for the strip footings or pile caps will encounter groundwater, as the stabilized water level measured in the piezometer installed at Borehole E1 and the water level in the creek are higher than the underside of the proposed footings/pile caps. The groundwater level is subject to fluctuations and the depth of excavation below the groundwater will depend on the time of year that the rehabilitation works are carried out. Also, perched groundwater may be present within the granular fill layers. Surficial water seepage into the excavations should be expected and will be greater during periods of sustained precipitation. Control of groundwater is discussed in Section 6.7.3.

All excavations should be carried out in accordance with the latest edition of the Ontario Occupational Health and Safety Act and Regulations for Construction Projects and good construction practice. Existing fill, peat, clay and sandy clayey silt to silt would be classified as Type 3 soil, according to the OHSA. Temporary excavations (i.e., those that are open for a relatively short time period) should be made with side slopes no steeper than 1H:1V above the water table and 3H:1V below the water table.

6.7.3 Temporary Excavation Support Systems and Groundwater Control

Sub-excavation of the peat requires excavations up to about 1 m below the normal creek water level at Elevation 235.2 m and up to about 2 m below the high water level at Elevation 236.0 m. As discussed above, excavation and backfilling can be carried out sub-aqueously and ground water control is not required. Surface water should be directed away from the excavations at all times.

Excavations for the footings/pile caps will require excavations up to about 3 m below the normal water level or up to about 4 m below the high water level. In this regard, 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 creek. 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 2.2 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 footings or pile caps. If an H-pile foundation is to be used, the H-piles should be driven prior to placement of the tremie concrete plug and a balanced head of water should be maintained until the tremie plug is placed to guard against the basal heave and/or piping methods of failure.

It should be noted that at this site, the tremie concrete plug forms an integral part of the footing system (see Section 6.3) and it is the tremie concrete that will extend to the recommended spread footing founding elevation and therefore, placement of this tremie concrete is not optional.

Temporary roadway protection will also be required for backfilling behind the cofferdams.

These 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:

	P	=	0 at ground surface increasing linearly to a depth of $0.25 H_T$ to:
	p	=	$\gamma H_T - 4 \text{ m } S_u$ 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/m²).

The support systems may be designed using the following parameters:

	COEFFICIENT OF EARTH PRESSURE			INTERNAL ANGLE OF	UNIT	UNDRAINED SHEAR
SOIL TYPE	Active, K _a	At Rest, K _o	Passive, K _p **	FRICTION	WEIGHT	STRENGTH
				(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	-
Existing Gravelly Sand to Silty Sand (Fill)	0.31	0.47	3.3	32	21	-
Clay*	0.41	0.58	2.5	25	17	-
	1.0	1.0	1.0	-	17	35
Sandy Clayey Silt to Silt*	0.38	0.55	2.7	27	18	-
	1.0	1.0	1.0	-	18	40
Sand and Silt	0.31	0.47	3.3	32	18	-
Sand and Silt (Till)	0.27	0.43	3.7	35	21	-

Notes: *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.

6.7.4 Temporary Support of Existing Timber Piles

The structural designer should consider the condition of the existing timber piles and the temporary nature of the various phases of construction to ensure that the integrity of the existing abutments is not compromised while the existing bridge is still in operation.

6.7.5 Backfilling

Backfill to the culvert walls should consist of granular fill meeting the specifications for SP110S13 (Aggregates) Granular 'B' Type II (but with less than 5 per cent passing the 200 sieve). The backfill should be placed in lifts not exceeding 200 mm loose thickness and compacted to 95 per cent of the Standard Proctor maximum dry density (SPMDD) of the material. The fill depth during placement should be maintained equal on both sides of the culvert with one side not exceeding the other by more than 500 mm. Granular fill materials and placement



should be carried out in accordance with the requirements as outlined in SP 206S03 (Earth Excavation, Grading; Earth Embankment).

The culverts should be designed for the full overburden stress and appropriate live loads, assuming a fill unit weight of 22 kN/m³ for Granular 'A' and 21 kN/m³ for Granular 'B' Type II backfill above and surrounding the culvert.

Prior to placement of the roadway granular subbase and base courses, the final lift of embankment fill should be compacted to 100 per cent of the SPMDD. 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.

6.7.6 Obstructions

The till at this site is glacially derived and as such contains cobbles and boulders, as were also encountered in the overlying gravelly sand and silt to sand and silt deposit, which could affect the installation of deep foundations. 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.

6.7.7 Vibration Monitoring During Pile Installation

A maximum peak particle velocity (PPV) of 100 mm/s is generally considered applicable for bridge structures in good condition. Based on vibration monitoring experience, it is considered unlikely that vibrations induced by conventional construction activities (such as pile driving or cofferdam installation) will reach this threshold level. Therefore, vibration monitoring is not required during construction at this site.

6.7.8 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 culvert footings and during pile driving into the till of the pile for the following reasons:

- the age and condition of the existing structure;
- the existing abutments are founded on timber piles;
- the close proximity of the existing and proposed structure;
- 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.



6.7.9 Analytical Testing for Construction Materials

The analytical test results on a sample of creek water are presented in Appendix B. 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 Detail 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., Designated MTO Foundations Contacts and Principals with Golder, conducted independent quality control reviews of this report.



Report Signature Page

GOLDER ASSOCIATES LTD.

Evan Childerhose, P.Eng.
Geotechnical Engineer



Sarah E. M. Coyne, P.Eng.
Senior Geotechnical Engineer, Associate



Jorge M.A. Costa, P.Eng.
Designated MTO Contact, Principal

AC/EC/SEMC/JMAC/kp

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- GeoStudio (Version 7.19) by Geo-Slope International Ltd.
 - LPile (Version 5.0) by Ensoft Inc.
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Ministry of Transportation Ontario Special Provisions

- SP 105S21 Amendment to OPSS 501, November 2010 – Water Requirements and Quality Control for Compaction – Method B.
- SP 110S13 Material Specification for Aggregates – Base, Subbase, Select Subgrade and Backfill Material
- SP 206S03 Earth Excavation, Grading; Rock Excavation, Grading
- SP 511S01 Rip Rap; Rock Protection; Gravel Sheetting

Ontario Provincial Standard Structural Drawings

- SS 103-11 Pile Driving Control, 2008

Ontario Provincial Standard Drawings

- OPSD 203.020 Embankments Over Swamp, Existing Slope Excavated to 1H:1V
- OPSD 208.010 Benching of Earth Slopes
- OPSD 3000.100 Foundation, Piles, Steel H-Pile, Driving Shoe
- OPSD 3090.100 Foundation, Frost Penetration Depths for Northern Ontario
- OPSD 3121.150 Walls Retaining, Backfill Minimum Granular Requirement

Ontario Provincial Standard Specifications

- OPSS 501 Construction Specification for Compacting
- OPSS 511 Construction Specification for Rip Rap, Rock Protection and Granular Sheetting
- OPSS 539 Construction Specification for 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

Ontario Water Resources Act

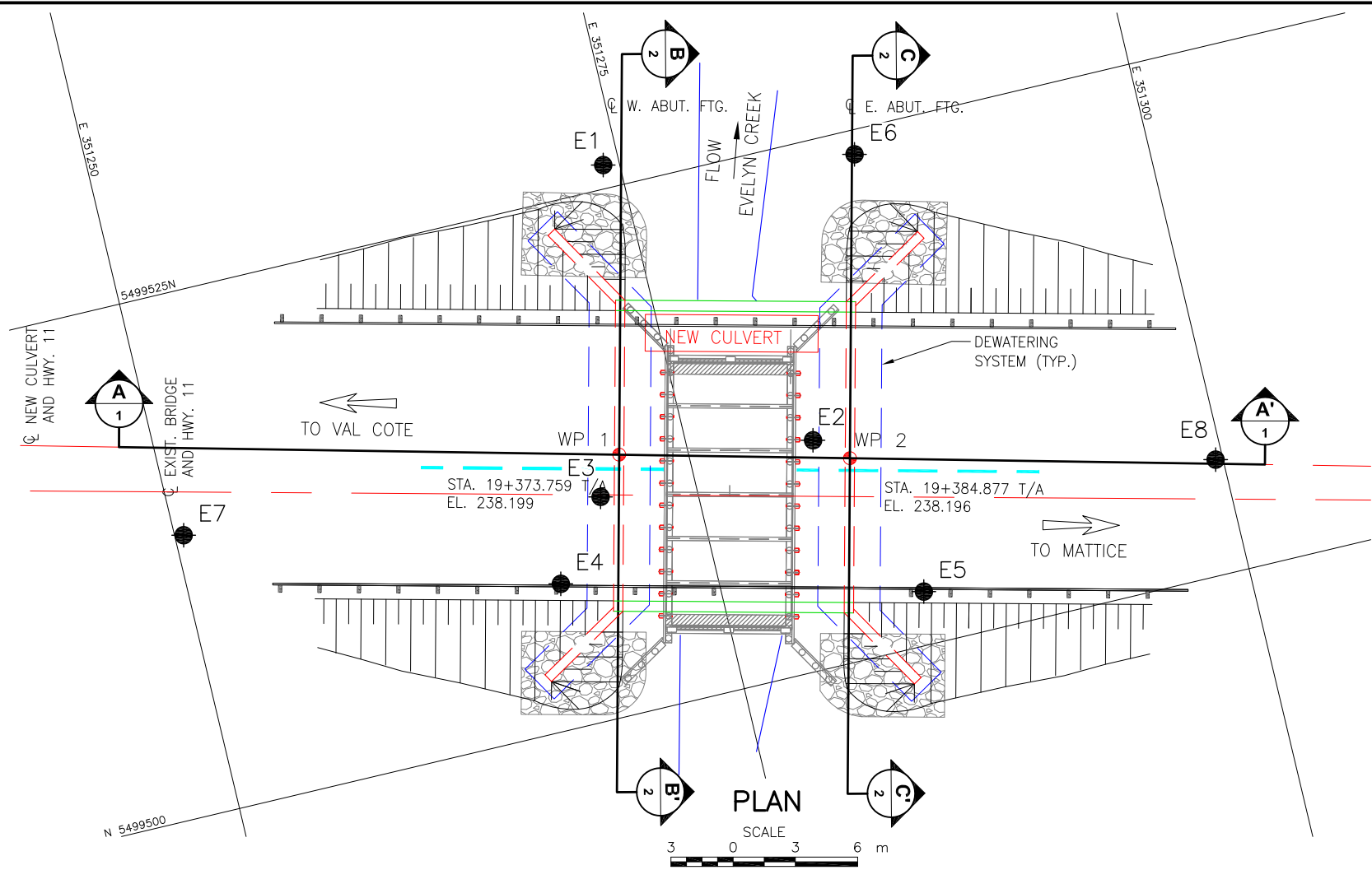
- Ontario Regulation 903/90 Wells: O. Reg. 468/10 Amendment to Ontario Regulation 903



**FOUNDATION REPORT REPLACEMENT OF EVELYN CREEK BRIDGE
HIGHWAY 11, SITE NO. 39W-100, GWP 5150-05-00**

Table 1: Evaluation of Culvert Foundation Alternatives

Foundation Type	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Open Footing Culvert supported on Strip Footings founded on tremie plug above native deposits	1	<ul style="list-style-type: none"> ■ Straightforward construction. ■ Can be used for open footings for culvert and wing walls. ■ Cobbles and boulders will not impact footing construction. ■ Suitable for sites with low loading conditions. 	<ul style="list-style-type: none"> ■ Excavation and sheeting (cofferdam) required for footings adjacent to the creek to allow for removal of peat and organic silt. ■ Lower axial resistance compared to deep foundations due to firm to stiff clayey silt to silt founding stratum. ■ Potential for differential settlement along strip footing. 	<ul style="list-style-type: none"> ■ Typically spread footings are lower cost than deep foundations. 	<ul style="list-style-type: none"> ■ Insufficient bearing resistance potentially available due to poor subsoil conditions.
Open Footing Culvert supported on Driven Steel H-Piles	2	<ul style="list-style-type: none"> ■ Straightforward construction. ■ Higher axial resistance compared to spread footings. ■ Minimal settlement. 	<ul style="list-style-type: none"> ■ Excavation and sheeting (cofferdam) required for pile caps adjacent to the creek to allow for removal of peat and organic silt. ■ Potential for “hanging-up” on cobbles and boulders within the very dense till deposit. ■ More specialized construction equipment/materials required than for footing construction. 	<ul style="list-style-type: none"> ■ Typically deep foundations are higher cost than shallow foundations. 	<ul style="list-style-type: none"> ■ Potential for varying pile tip elevations if piles “hang-up” on cobbles/boulders.
Box Culvert	NF	<ul style="list-style-type: none"> ■ Lesser depth of excavation required. 	<ul style="list-style-type: none"> ■ Very large open space cannot be easily accommodated by a box structure. 	<ul style="list-style-type: none"> ■ 	<ul style="list-style-type: none"> ■



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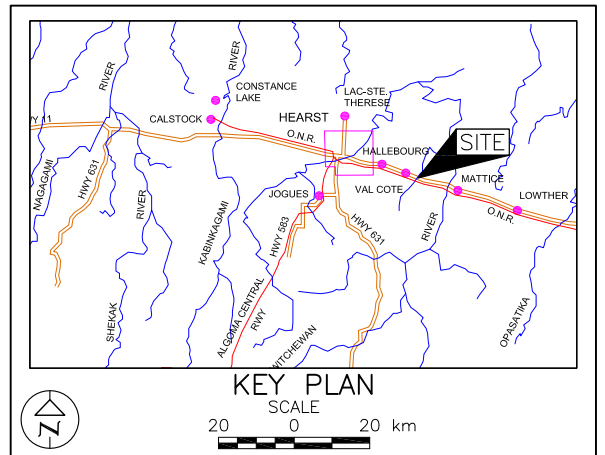
CONT No.
WP No. 5150-05-00

HIGHWAY 11
EVELYN CREEK CULVERT
BOREHOLE LOCATION AND
SOIL STRATA

SHEET
44

Golder Associates

Golder Associates Ltd.
SUDBURY, ONTARIO, CANADA



LEGEND

- Borehole
- Seal
- Piezometer
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- 100% Recovery
- WL upon completion of drilling

BOREHOLE CO-ORDINATES			
No.	ELEVATION	NORTHING	EASTING
E1	236.1	5499526.1	351274.2
E2	237.8	5499510.9	351281.0
E3	237.7	5499510.6	351270.4
E4	237.7	5499507.0	351267.6
E5	237.7	5499502.5	351284.5
E6	236.1	5499523.8	351286.1
E7	237.8	5499513.4	351250.4
E8	237.8	5499505.5	351299.6

NOTES

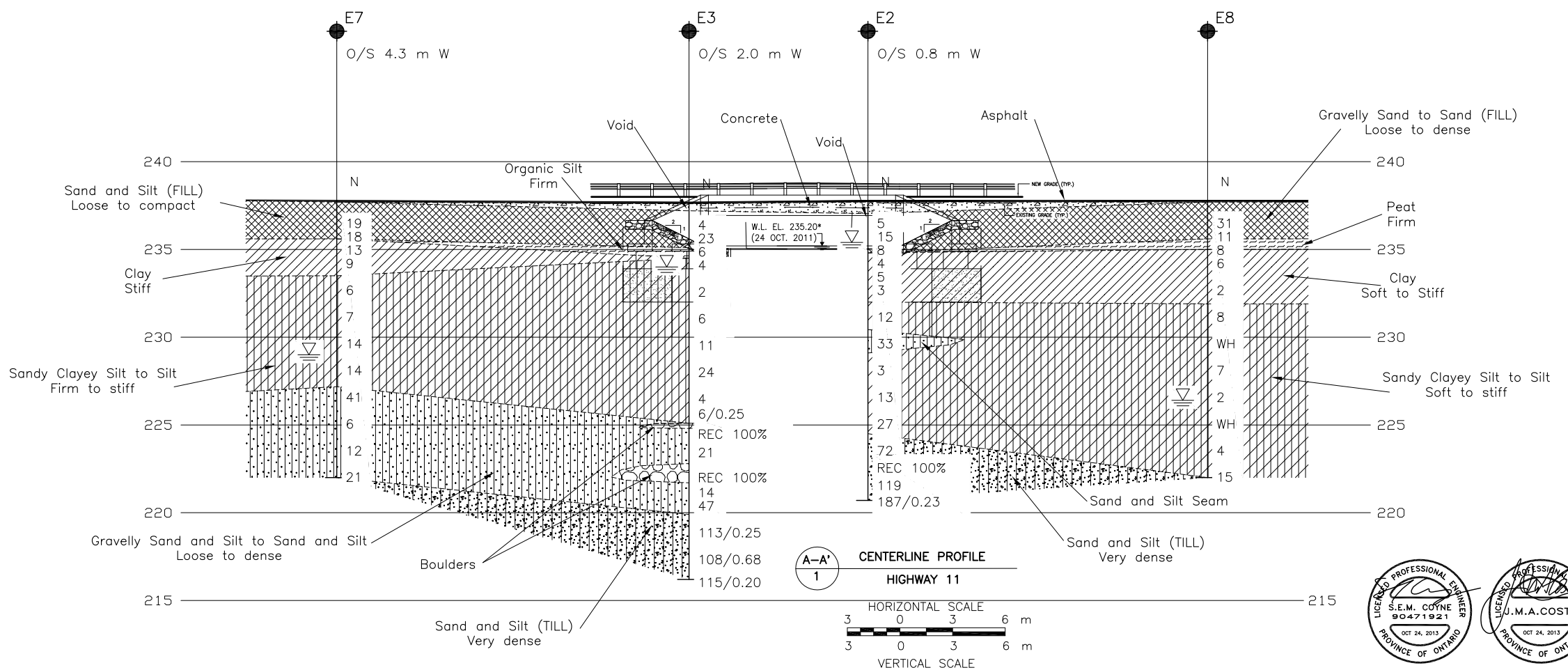
This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contracts Documents.

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

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

REFERENCE

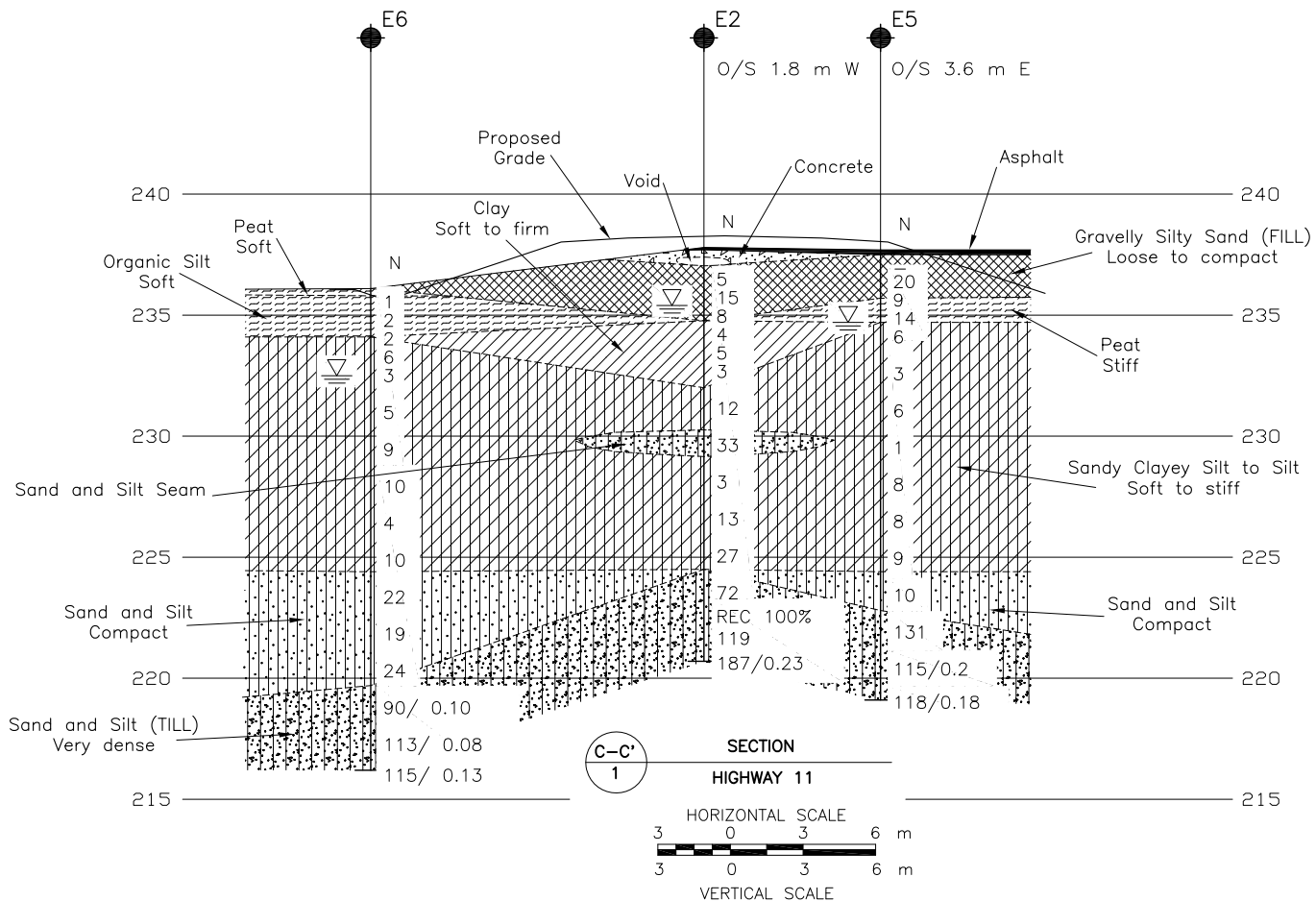
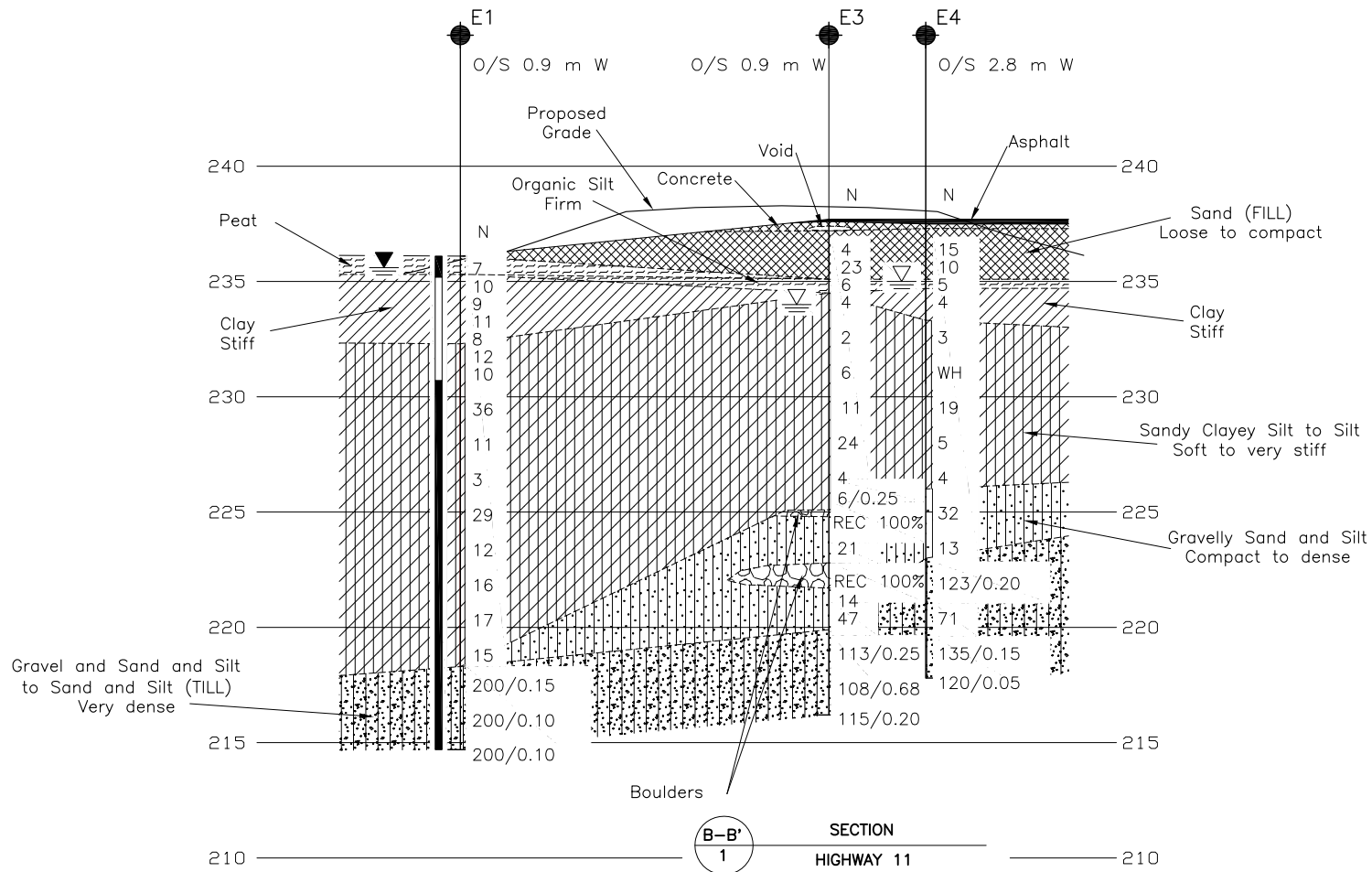
Base plans provided in digital format by LEA Consulting Ltd., drawing file no. 8960-Evelyn-S01.dwg, received Apr. 29, 2013 and x8960 EVE xsections.dwg received Oct. 22, 2013.



S.E.M. COYNE
90471921
OCT 24, 2013
PROVINCE OF ONTARIO

J.M.A. COSTA
OCT 24, 2013
PROVINCE OF ONTARIO

NO.	DATE	BY	REVISION
Geocres No. 42G-45			
HWY. 11	PROJECT NO. 11-1191-0008		DIST.
SUBM'D. EC	CHKD.	DATE: OCT 2013	SITE: 39W-100
DRAWN: TB	CHKD. SEMC	APPD. JMAC	DWG. 1



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STATIONS IN KILOMETRES + METRES.

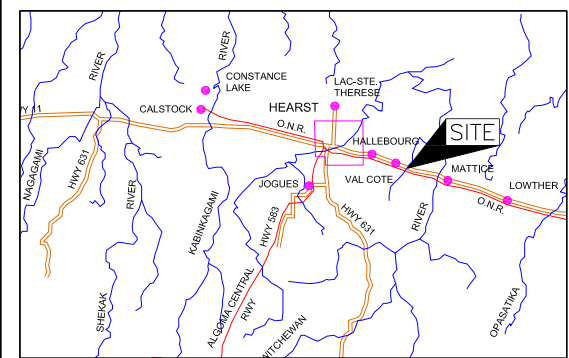
CONT No.
WP No. 5150-05-00

HIGHWAY 11
EVELYN CREEK BRIDGE
SOIL STRATA

SHEET
45



Golder Associates Ltd.
SUDBURY, ONTARIO, CANADA



LEGEND			
	Borehole		
	Seal		
	Piezometer		
	Standard Penetration Test Value		
	Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)		
	100% Recovery		
	WL in piezometer, measured on June 25, 2013.		
	WL upon completion of drilling		

BOREHOLE CO-ORDINATES			
No.	ELEVATION	NORTHING	EASTING
E1	236.1	5499526.1	351274.2
E2	237.8	5499510.9	351281.0
E3	237.7	5499510.6	351270.4
E4	237.7	5499507.0	351267.6
E5	237.7	5499502.5	351284.5
E6	236.1	5499523.8	351286.1

NOTES

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REFERENCE

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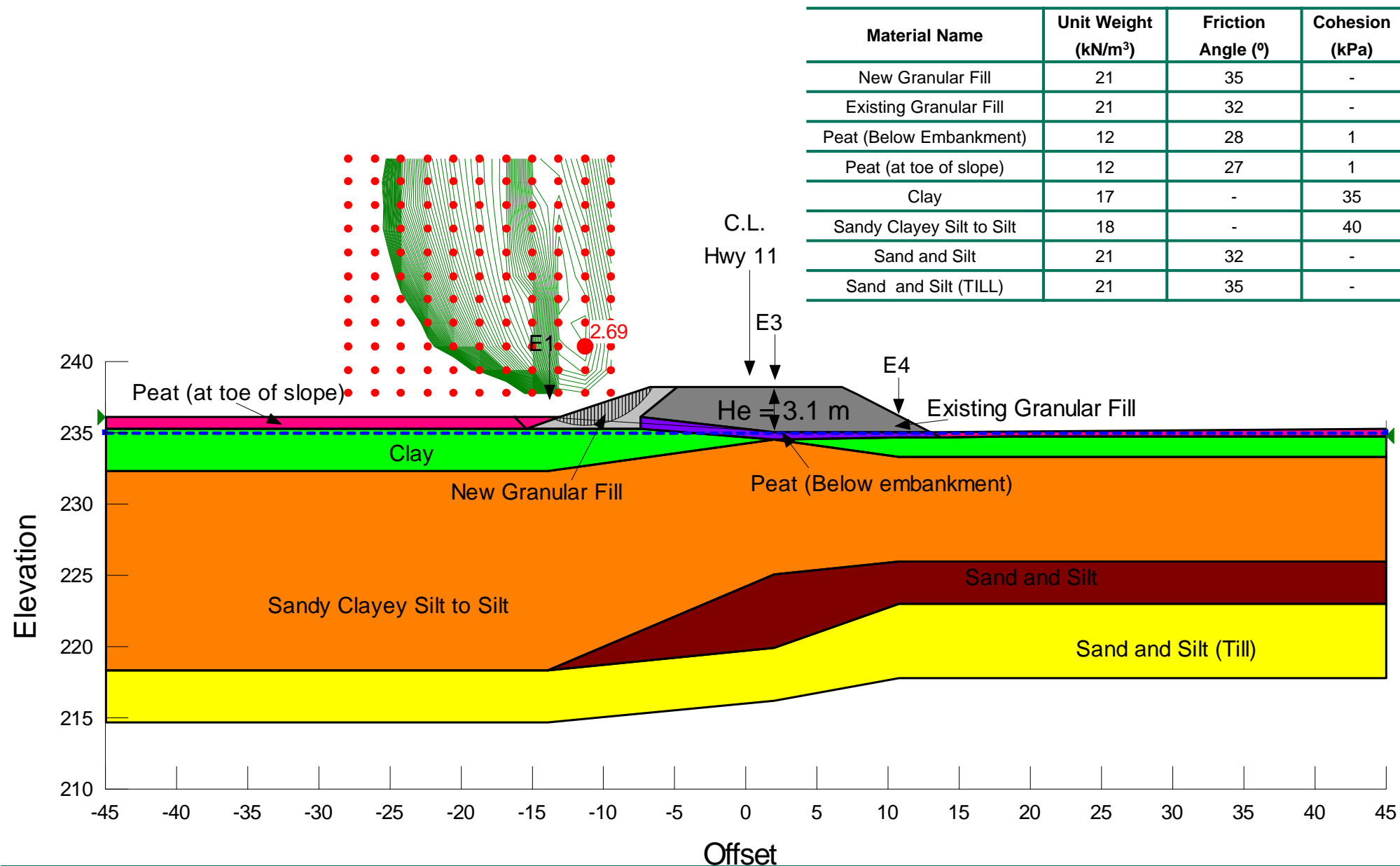


NO.	DATE	BY	REVISION
Geocres No. 42G-45			
HWY. 11	PROJECT NO. 11-1191-0008		DIST.
SUBM'D. EC	CHKD.	DATE: OCT 2013	SITE: 39W-100
DRAWN: TB	CHKD. SEMC	APPD. JMAC	DWG. 2



Evelyn Creek Culvert – Highway 11 Stability Analysis (Side Slopes)

Figure 1





APPENDIX A

Record of Boreholes (E1 to E8)



LIST OF SYMBOLS

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

I. GENERAL

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

II. STRESS AND STRAIN

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

III. SOIL PROPERTIES

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

(a) Index Properties (continued)

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

(b) Hydraulic Properties

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

(c) Consolidation (one-dimensional)

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

(d) Shear Strength

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

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

Notes: 1
2

$$\tau = c' + \sigma' \tan \phi'$$
$$\text{shear strength} = (\text{compressive strength})/2$$



LIST OF ABBREVIATIONS

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

I. SAMPLE TYPE

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

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

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

Dynamic Cone Penetration Resistance; N_d :

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

PH: Sampler advanced by hydraulic pressure

PM: Sampler advanced by manual pressure

WH: Sampler advanced by static weight of hammer

WR: Sampler advanced by weight of sampler and rod

Piezo-Cone Penetration Test (CPT)

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

III. SOIL DESCRIPTION

(a) Non-Cohesive (Cohesionless) Soils

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

(b) Cohesive Soils Consistency

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

IV. SOIL TESTS

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

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

V. MINOR SOIL CONSTITUENTS

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

PROJECT 11-1191-0008				RECORD OF BOREHOLE No E1				1 OF 2 METRIC								
W.P. 5150-05-00				LOCATION N 5499526.1; E 351274.2				ORIGINATED BY ID								
DIST _____ HWY 11				BOREHOLE TYPE 108 mm ID Continuous Flight Hollow Stem Augers, NW Casing				COMPILED BY EC								
DATUM Geodetic				DATE October 20, 2012				CHECKED BY SEMC								
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								
236.1	GROUND SURFACE															
0.0	PEAT (Fibrous) Firm Black Moist		1	SS	7											
235.3																
0.8	CLAY, trace sand Stiff Brown to grey Moist to wet Trace organics to 2.1 m depth.		2	SS	10											
			3	SS	9											
			4	SS	11											
			5	SS	8											
232.3																
3.8	Sandy CLAYEY SILT to SILT, trace to some gravel Soft to hard Grey Wet		6	SS	12											
			7	SS	10											
			8	SS	36											
			9	SS	11											
			10	SS	3											
			11	SS	29											
			12	SS	12											
			13	SS	16											

Continued Next Page

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

SUD_MTO 003 1111910008DET.GPJ GAL-MISS.GDT 22/10/13 DATA INPUT:

PROJECT 11-1191-0008			RECORD OF BOREHOLE No E1				2 OF 2 METRIC									
W.P. 5150-05-00		LOCATION N 5499526.1; E 351274.2				ORIGINATED BY ID										
DIST _____ HWY 11		BOREHOLE TYPE 108 mm ID Continuous Flight Hollow Stem Augers, NW Casing				COMPILED BY EC										
DATUM Geodetic		DATE October 20, 2012				CHECKED BY SEMC										
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa								
	--- CONTINUED FROM PREVIOUS PAGE ---															
	Sandy CLAYEY SILT to SILT, trace to some gravel Soft to hard Grey Wet		14	SS	17											
	Approximately 6.1 m of heave encountered at 15.8 m depth.															
218.3			15	SS	15											
17.8	GRAVEL and SAND and SILT, trace clay (TILL) Very dense Grey Moist		16	SS	200/0.15											34 31 31 4
			17	SS	200/0.1											
214.7																
21.4	END OF BOREHOLE		18	SS	200/0.1											
	Note: 1. Water level at a depth of 1.2 m below ground surface (Elev. 234.9 m) upon completion of drilling. 2. Water level in piezometer at a depth of 1.1 m below ground surface (Elev. 235.0 m) on December 13, 2012. 3. Water level in piezometer at a depth of 0.5 m below ground surface (Elev. 235.6 m) on June 25, 2013.															

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




PROJECT <u>11-1191-0008</u>				RECORD OF BOREHOLE No E2				2 OF 2 METRIC										
W.P. <u>5150-05-00</u>		LOCATION <u>N 5499510.9; E 351281.0</u>				ORIGINATED BY <u>ID</u>												
DIST <u> </u> HWY <u>11</u>		BOREHOLE TYPE <u>NW Casing, NQ Coring</u>				COMPILED BY <u>EC</u>												
DATUM <u>Geodetic</u>		DATE <u>October 19, 2012</u>				CHECKED BY <u>SEMC</u>												
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa					WATER CONTENT (%)					
	--- CONTINUED FROM PREVIOUS PAGE ---						<div style="display: flex; justify-content: space-between;"> 20 40 60 80 100 20 40 60 80 100 </div> <div style="display: flex; justify-content: space-between;"> ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED </div>					<div style="display: flex; justify-content: space-between;"> W_p W W_L </div>						
	SAND and SILT, some gravel (TILL) Very dense Grey Wet		1	RC	100%													
			13	SS	119													
	Refusal to further casing advancement at 14.9 m depth. Cored between 14.9 m and 15.1 m depth: Recovered cobble (200 mm thick).																	
220.7			14	SS	187/0.23													
17.1	END OF BOREHOLE Note: 1. Water level at a depth of 2.4 m below ground surface (Elev. 235.4 m) upon completion of drilling.																	

PROJECT 11-1191-0008				RECORD OF BOREHOLE No E3				1 OF 2 METRIC					
W.P. 5150-05-00				LOCATION N 5499510.6; E 351270.4				ORIGINATED BY EHS					
DIST _____ HWY 11				BOREHOLE TYPE NW Casing, NQ Coring				COMPILED BY EC					
DATUM Geodetic				DATE June 24 and 25, 2013				CHECKED BY AB					
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT		UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		WATER CONTENT (%)			
237.7	GROUND SURFACE							20 40 60 80 100					
0.0	ASPHALT (125 mm)							20 40 60 80 100					
	CONCRETE (260 mm)												
0.5	VOID (115 mm)												
	Sand, some gravel, trace silt (FILL)												
	Loose to compact												
	Brown												
	Moist to wet												
235.1			1	SS	4		237						
			2	SS	23		236						
			3A	SS	6								
234.5	ORGANIC SILT		3B				235						
	Firm												
	Black												
	Wet												
234.5			4	SS	4		234						
	CLAYEY SILT to SILT, trace to some												
	sand												
	Soft to very stiff												
	Grey												
	Wet												
			5	SS	2		233						
			6	SS	6		232						
			7	SS	11		231						
			8	SS	24		230						
							229						
							228						
							227						
			9	SS	4								
							226						
225.1			10	SS	6/0.25		225						
	BOULDER			RC	REC 100%								
224.8													
	Gravelly SAND and SILT, trace clay												
	Compact to dense												
	Grey												
	Wet												
			11	SS	21		224						
							223						
222.8													

Continued Next Page

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

SUD_MTO 003 1111910008DET.GPJ GAL-MISS.GDT 22/10/13 DATA INPUT:

PROJECT		RECORD OF BOREHOLE				No E3		2 OF 2		METRIC							
W.P. 5150-05-00		LOCATION N 5499510.6; E 351270.4				ORIGINATED BY		EHS									
DIST _____ HWY 11		BOREHOLE TYPE NW Casing, NQ Coring				COMPILED BY		EC									
DATUM Geodetic		DATE June 24 and 25, 2013				CHECKED BY		AB									
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
	--- CONTINUED FROM PREVIOUS PAGE ---																
14.9	BOULDER (Gneiss)		-	RC	REC 100%												
221.7																	
16.0	Gravelly SAND and SILT, some gravel, trace clay Compact to dense Grey Wet		12	SS	14												
			13	SS	47												
219.9																	
17.8	SAND and SILT, some gravel, trace clay (TILL) Very dense Grey Wet		14	SS	113/0.25												
			15	SS	108/0.1												
216.2																	
21.5	END OF BOREHOLE		16	SS	115/0.2												
	Note: 1. Water level at a depth of 3.7 m below ground surface (Elev. 234.0 m) upon completion of drilling.																

PROJECT 11-1191-0008				RECORD OF BOREHOLE No E4				1 OF 2 METRIC						
W.P. 5150-05-00				LOCATION N 5499507.0; E 351267.6				ORIGINATED BY EHS						
DIST _____ HWY 11				BOREHOLE TYPE NW Casing, NQ Coring				COMPILED BY EC						
DATUM Geodetic				DATE June 19, 2013				CHECKED BY AB						
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
237.7	GROUND SURFACE													
0.0	ASPHALT (150 mm)													
	CONCRETE (230 mm)													
0.4	Sand, some gravel (FILL) Compact Brown Wet		1	SS	15									
			2	SS	10									
235.1			3	SS	5									
234.7	PEAT (Amorphous) Firm Black Wet		4	SS	4									
3.0	CLAY, varved Stiff Grey Wet													
233.3	Sandy CLAYEY SILT, trace gravel Very soft to stiff Grey Wet		5	SS	3									
4.4			6	SS	WH									
			7	SS	19									
			8	SS	5									
			9	SS	4									
226.0	SAND and SILT, trace to some gravel Compact to dense Grey Wet		10	SS	32									
11.7			11	SS	13									
223.0														
14.7														

SUD_MTO_003_1111910008DET.GPJ GAL-MISS.GDT 22/10/13 DATA INPUT:

Continued Next Page

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



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

SUD MTO 003 111910008DET.GPJ GAL-MISS.GDT 22/10/13 DATA INPUT:

PROJECT 11-1191-0008			RECORD OF BOREHOLE No E5			1 OF 2 METRIC											
W.P. 5150-05-00			LOCATION N 5499502.5; E 351284.5			ORIGINATED BY EHS											
DIST _____ HWY 11			BOREHOLE TYPE 108 mm ID Continuous Flight Hollow Stem Augers			COMPILED BY EC											
DATUM Geodetic			DATE June 18, 2013			CHECKED BY AB											
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa					WATER CONTENT (%)			γ kN/m³	GR SA SI CL
							20 40 60 80 100	20 40 60 80 100	Wp	W	WL	20 40 60					
237.7	GROUND SURFACE																
0.0	ASPHALT (150 mm)																
0.2	Silty sand, trace gravel (FILL) Loose to compact Brown Moist		1	AS	-		237										1 78 (21)
			2	SS	20												
235.7			3A	SS	9		236										
2.0	PEAT (Amorphous) Stiff Black Wet		3B														
			4	SS	14		235										
234.7																	
3.0	Sandy CLAYEY SILT to SILT, trace to some gravel Soft to stiff Grey Wet		5	SS	6		234										
			6	SS	3		233										
			7	SS	6		232										
			8	SS	1		231										0 3 87 10
			9	SS	8		230										
			10	SS	8		229										
			11	SS	9		228										
			12	SS	10		227										10 22 52 16
224.4							226										
13.3	SAND and SILT Compact Grey Moist						225										
							224										
222.8							223										




SUD_MTO_003 1111910008DET.GPJ GAL-MISS.GDT 22/10/13 DATA INPUT:

Continued Next Page

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

PROJECT <u>11-1191-0008</u>		RECORD OF BOREHOLE No E5				2 OF 2 METRIC											
W.P. <u>5150-05-00</u>		LOCATION <u>N 5499502.5; E 351284.5</u>				ORIGINATED BY <u>EHS</u>											
DIST <u> </u> HWY <u>11</u>		BOREHOLE TYPE <u>108 mm ID Continuous Flight Hollow Stem Augers</u>				COMPILED BY <u>EC</u>											
DATUM <u>Geodetic</u>		DATE <u>June 18, 2013</u>				CHECKED BY <u>AB</u>											
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa					WATER CONTENT (%)				
--- CONTINUED FROM PREVIOUS PAGE ---																	
14.9	SAND and SILT, trace to some gravel, trace clay (TILL) Very dense Grey Moist Approximately 1.5 m of heave encountered at 15.8 m depth.	13	SS		131		222										
							221										
		14	SS		115/0.2		220										
219.1		15	SS		118/0.2												
18.6	END OF BOREHOLE Note: 1. Water level at a depth of 3.0 m below ground surface (Elev. 234.7 m) upon completion of drilling.																


SUD_MTO_003 1111910008DET.GPJ GAL-MISS.GDT 22/10/13 DATA INPUT:

PROJECT 11-1191-0008				RECORD OF BOREHOLE No E6				1 OF 2 METRIC									
W.P. 5150-05-00				LOCATION N 5499523.8; E 351286.1				ORIGINATED BY EHS									
DIST _____ HWY 11				BOREHOLE TYPE 108 mm ID Continuous Flight Hollow Stem Augers				COMPILED BY EC									
DATUM Geodetic				DATE June 6 and July 10, 2013				CHECKED BY AB									
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								WATER CONTENT (%)	
236.1	GROUND SURFACE							20	40	60	80	100					
0.0	PEAT (Fibrous)		1	SS	1		236										
0.3	Soft Black Wet		2	SS	2		235										
	ORGANIC SILT		3	SS	2		234										
234.1	Sandy CLAYEY SILT to SILT, trace gravel		4	SS	6		233										
2.0	Firm to stiff Grey Wet		5	SS	3		232										
			6	SS	5		231										
			7	SS	9		230										
			8	SS	10		229										
			9	SS	4		228										
			10	SS	10		227										
			11	SS	22		226										
224.4	SAND and SILT		12	SS	19		225										
11.7	Compact Grey Wet					224											
						223											
						222											

Continued Next Page

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

SUD_MTO 003 1111910008DET.GPJ GAL-MISS.GDT 22/10/13 DATA INPUT:

PROJECT 11-1191-0008				RECORD OF BOREHOLE No E6				2 OF 2 METRIC									
W.P. 5150-05-00				LOCATION N 5499523.8; E 351286.1				ORIGINATED BY EHS									
DIST _____ HWY 11				BOREHOLE TYPE 108 mm ID Continuous Flight Hollow Stem Augers				COMPILED BY EC									
DATUM Geodetic				DATE June 6 and July 10, 2013				CHECKED BY AB									
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
	--- CONTINUED FROM PREVIOUS PAGE ---							20	40	60	80	100					
219.7	SAND and SILT Compact Grey Wet		13	SS	24												
16.4	SAND and SILT, some gravel, trace clay (TILL) Very dense Grey Wet																
			14	SS	90/0.1												
			15	SS	113/0.1												
216.2	END OF BOREHOLE		16	SS	115/0.1												
19.9	Note: 1. Water level at a depth of 3.6 m below ground surface (Elev. 232.5 m) upon completion of drilling.																

SUD_MTO_003 1111910008DET.GPJ GAL-MISS.GDT 22/10/13 DATA INPUT:

SUD MTO 003 111910008DET.GPJ GAL-MISS.GDT 22/10/13 DATA INPUT:

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



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

SUD MTO 003 111910008DET.GPJ GAL-MISS.GDT 22/10/13 DATA INPUT:

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

SUD MTO 003 111910008DET.GPJ GAL-MISS.GDT 22/10/13 DATA INPUT:

PROJECT <u>11-1191-0008</u>		RECORD OF BOREHOLE No E8				2 OF 2 METRIC											
W.P. <u>5150-05-00</u>		LOCATION <u>N 5499505.5; E 351299.6</u>				ORIGINATED BY <u>EHS</u>											
DIST <u> </u> HWY <u>11</u>		BOREHOLE TYPE <u>108 mm ID Continuous Flight Hollow Stem Augers</u>				COMPILED BY <u>EC</u>											
DATUM <u>Geodetic</u>		DATE <u>June 19, 2013</u>				CHECKED BY <u>AB</u>											
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa					WATER CONTENT (%)				
	--- CONTINUED FROM PREVIOUS PAGE ---						<div style="display: flex; justify-content: space-between;"> 20 40 60 80 100 20 40 60 80 100 </div> <div style="display: flex; justify-content: space-between;"> ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED </div>					<div style="display: flex; justify-content: space-between;"> W_p W W_L </div>					
222.0 15.8	END OF BOREHOLE Note: 1. Water level at a depth of 11.4 m below ground surface (Elev. 226.4 m) upon completion of drilling.	12	SS	15		222											



APPENDIX B

Laboratory Test Results



**DETAIL FOUNDATION REPORT REPLACEMENT OF EVELYN CREEK BRIDGE
HIGHWAY 11, SITE NO. 39W-100, GWP 5150-05-00**

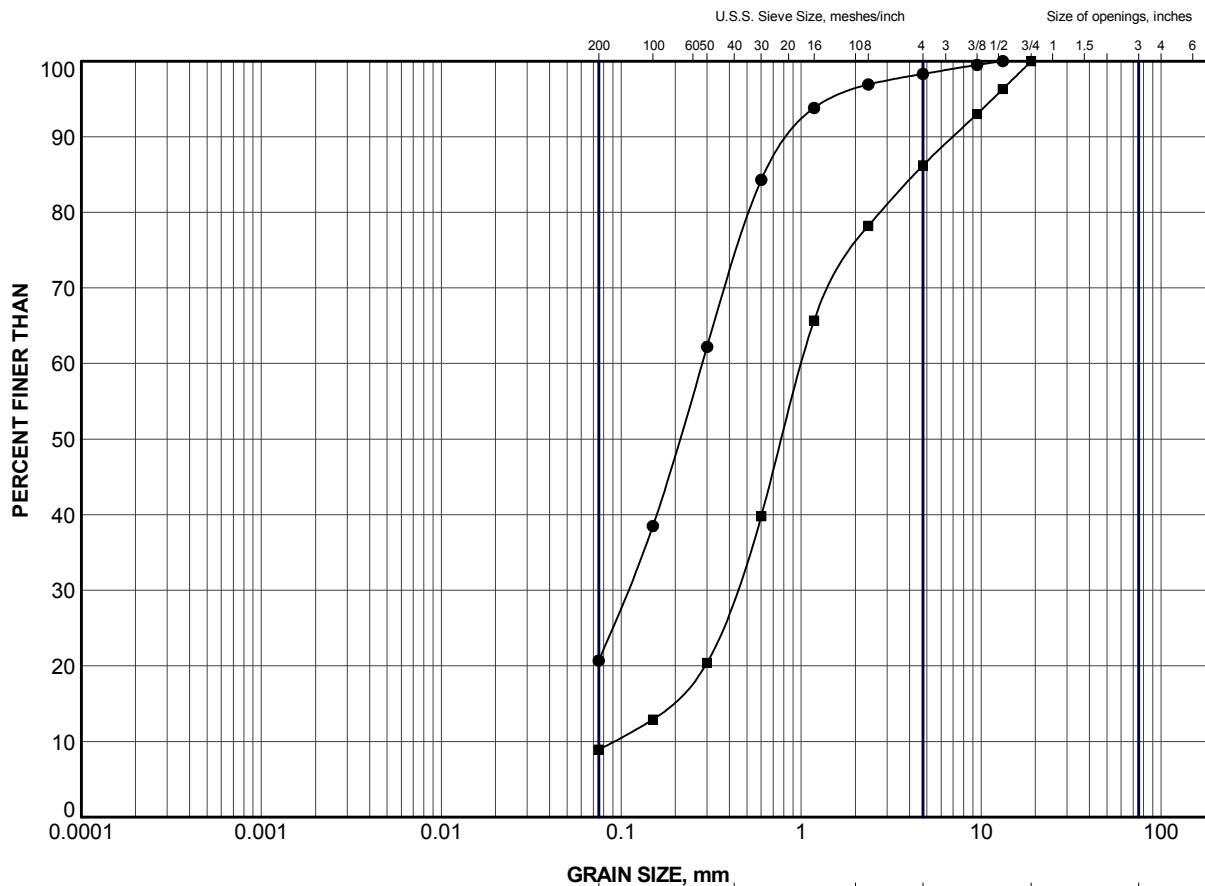
Table B1 - Summary of Analytical Testing of Creek Water

Parameter	Units	Result
Resistivity	ohm-cm	13,000
Conductivity	µmho/cm	75
pH	pH	6.81
Sulphate	mg/L	Not Detected
Chloride	mg/L	2

Notes:

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


Prepared by: EC
Reviewed by: SFMC

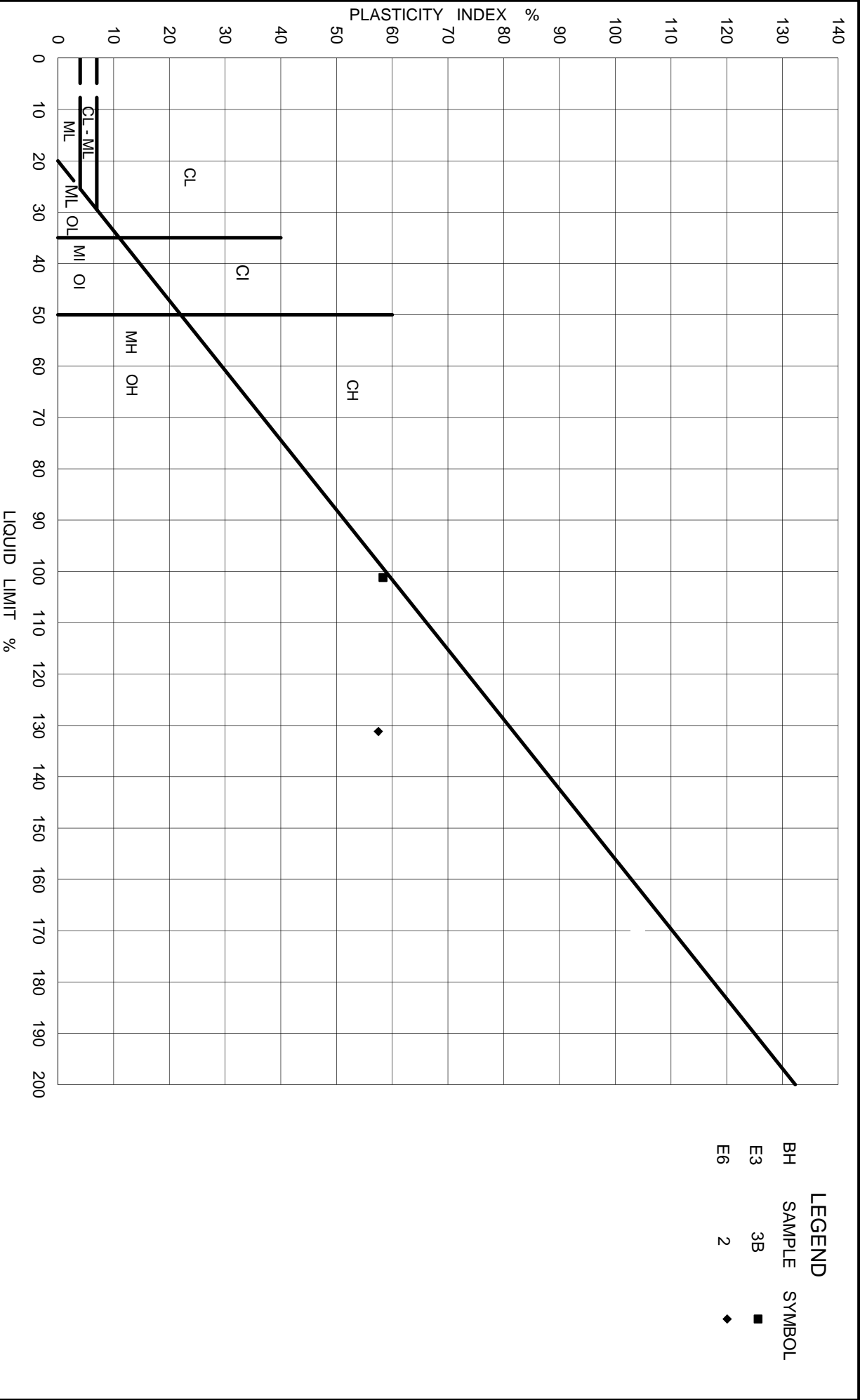


CLAY AND SILT		GRAVEL SIZE, mm					Cobble Size
		fine	medium	coarse	fine	coarse	
		SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	E5	1	237.2
■	E8	2A	236.2

PROJECT					
HIGHWAY 11 EVELYN CREEK CULVERT					
TITLE					
GRAIN SIZE DISTRIBUTION SILTY SAND to SAND (FILL)					
PROJECT No.		11-1191-0008		FILE No. 1111910008DET.GPJ	
DRAWN	JJL	Oct 2013	SCALE	N/A	REV.
CHECK	SEMC	Oct 2013			
APPR	FJH	Oct 2013			
 Golder Associates SUDBURY, ONTARIO			FIGURE B1		



Ministry of Transportation
Ontario

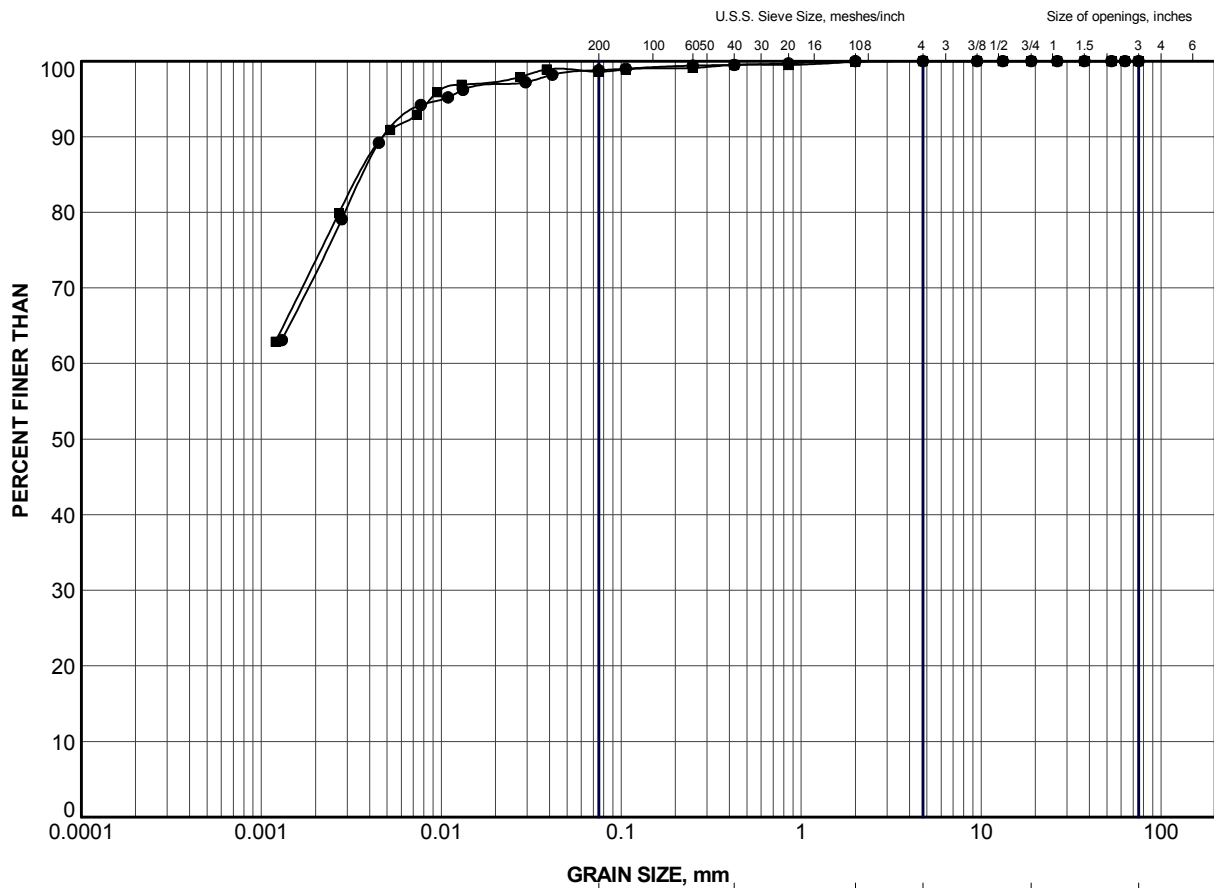
PLASTICITY CHART

ORGANIC SILT

Figure B2

Project No. 11-1191-0008


Checked By: SEMC

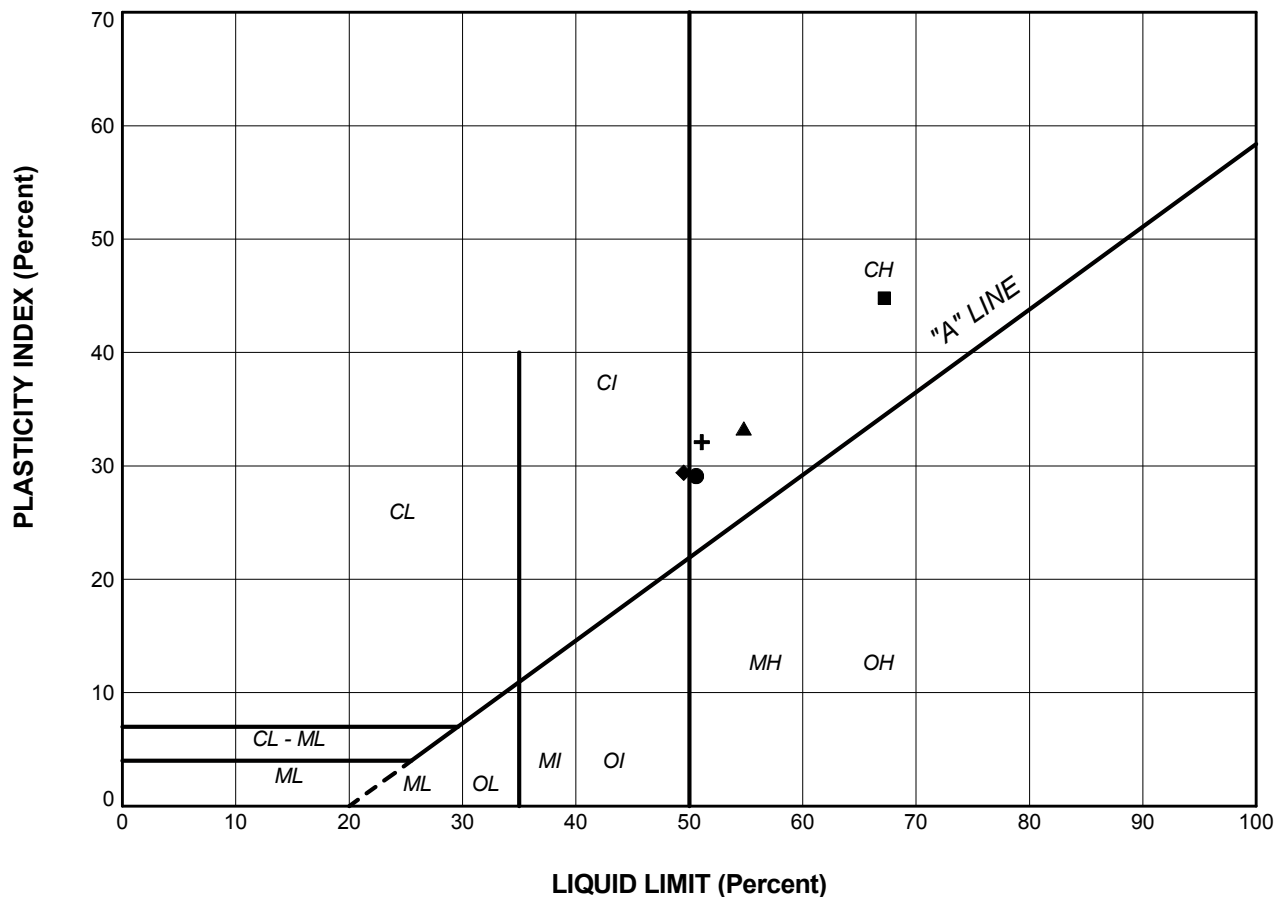


GRAIN SIZE, mm						
CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	E1	3	234.3
■	E2	5	233.7

PROJECT						HIGHWAY 11 EVELYN CREEK CULVERT					
TITLE						GRAIN SIZE DISTRIBUTION CLAY					
PROJECT No.			11-1191-0008			FILE No.			1111910008DET.GPJ		
DRAWN		JJL		Oct 2013		SCALE		N/A		REV.	
CHECK		SEMC		Oct 2013							
APPR		FJH		Oct 2013							
 Golder Associates SUDBURY, ONTARIO						FIGURE B3					

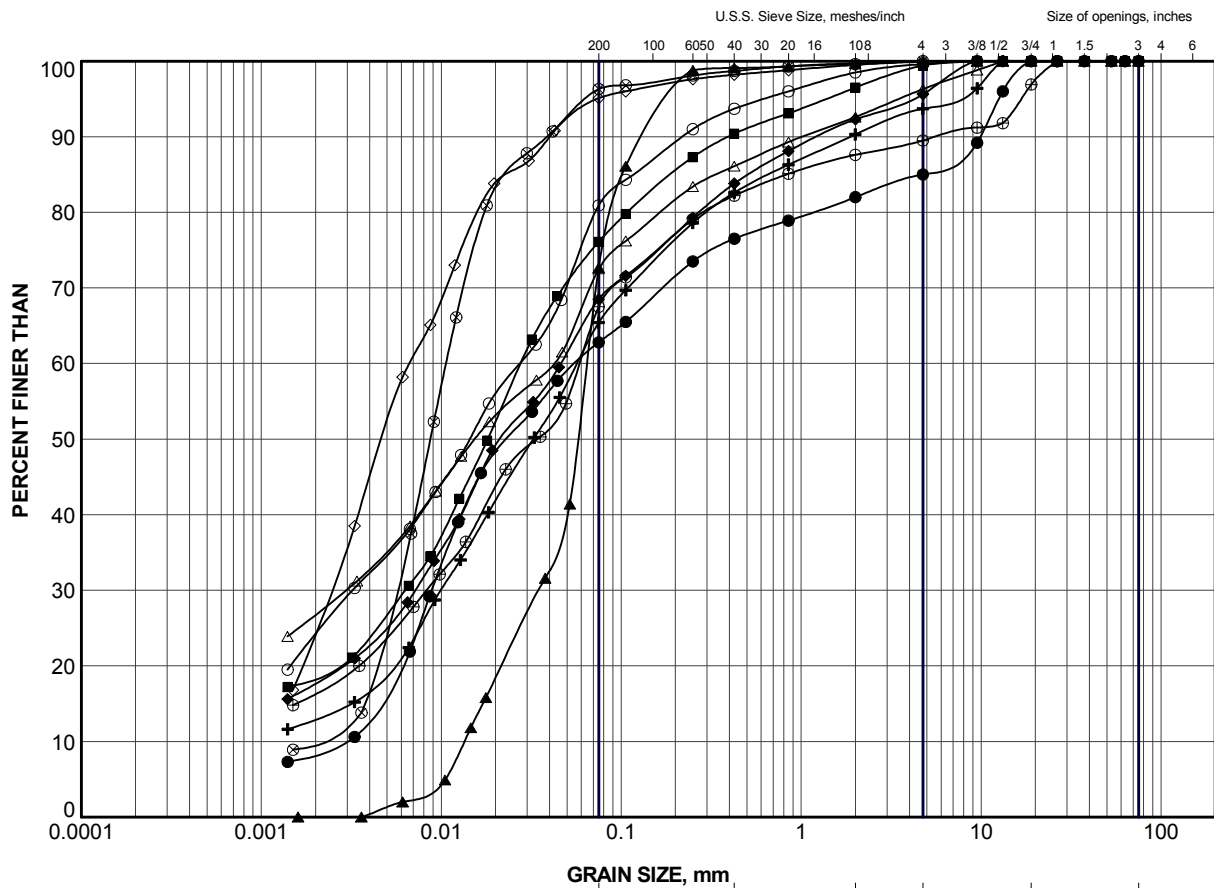


LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	E1	3	50.6	21.5	29.1
■	E2	5	67.2	22.4	44.8
▲	E4	4	54.8	21.5	33.3
+	E7	4	51.1	19.0	32.1
◆	E8	5	49.5	20.1	29.4

PROJECT			HIGHWAY 11 EVELYN CREEK CULVERT		
TITLE			PLASTICITY CHART CLAY		
PROJECT No.		11-1191-0008	FILE No.		1111910008DET.GPJ
DRAWN	JJL	Oct 2013	SCALE	N/A	REV.
CHECK	SEMC	Oct 2013	FIGURE B4		
APPR	FJH	Oct 2013			





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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	E1	6	232.0
■	E1	9	228.2
▲	E1	13	222.1
+	E2	9	228.4
◆	E2	11	225.3
◇	E3	5	232.8
○	E3	9	226.7
△	E4	9	226.7
⊗	E5	7	231.3
⊕	E5	10	226.7

PROJECT

HIGHWAY 11
EVELYN CREEK CULVERT

TITLE

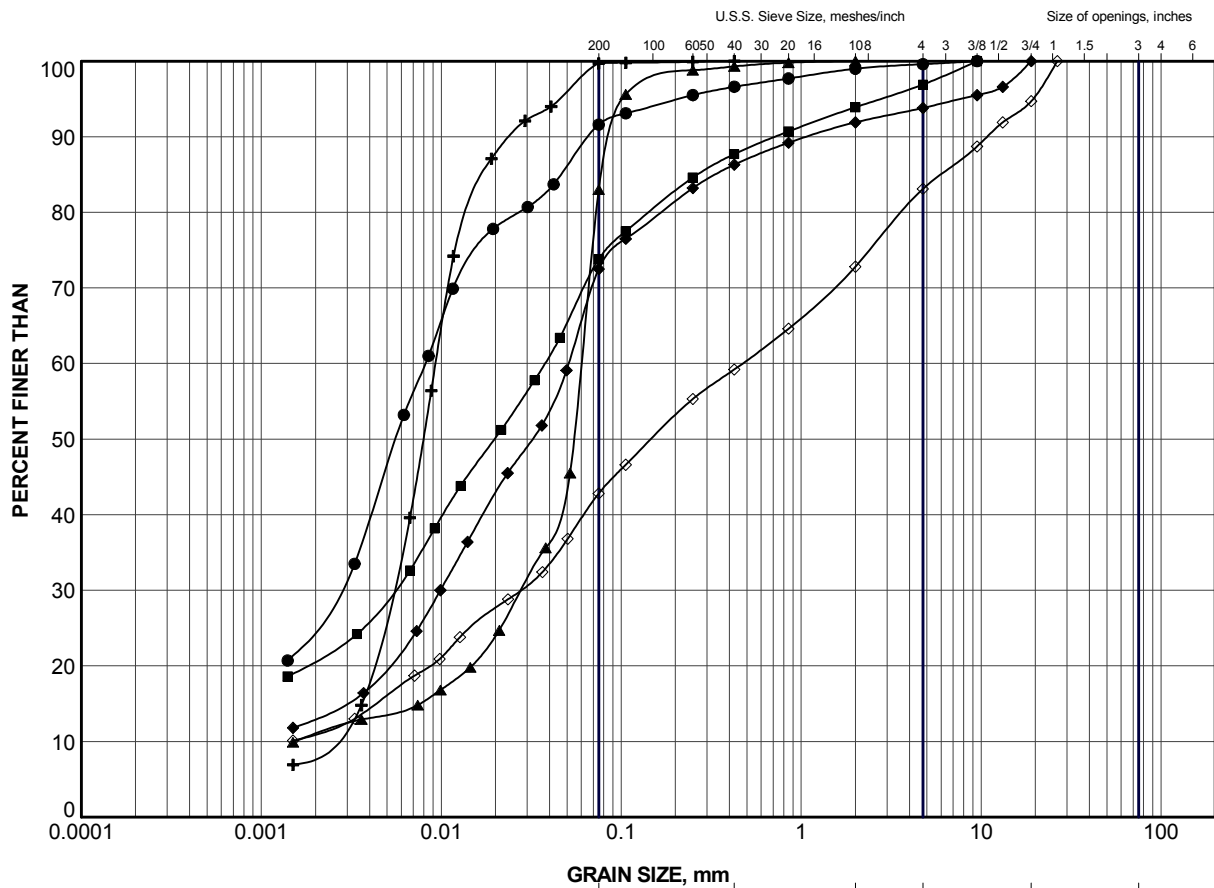
GRAIN SIZE DISTRIBUTION

SANDY CLAYEY SILT to SILT



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


FIGURE B5.1

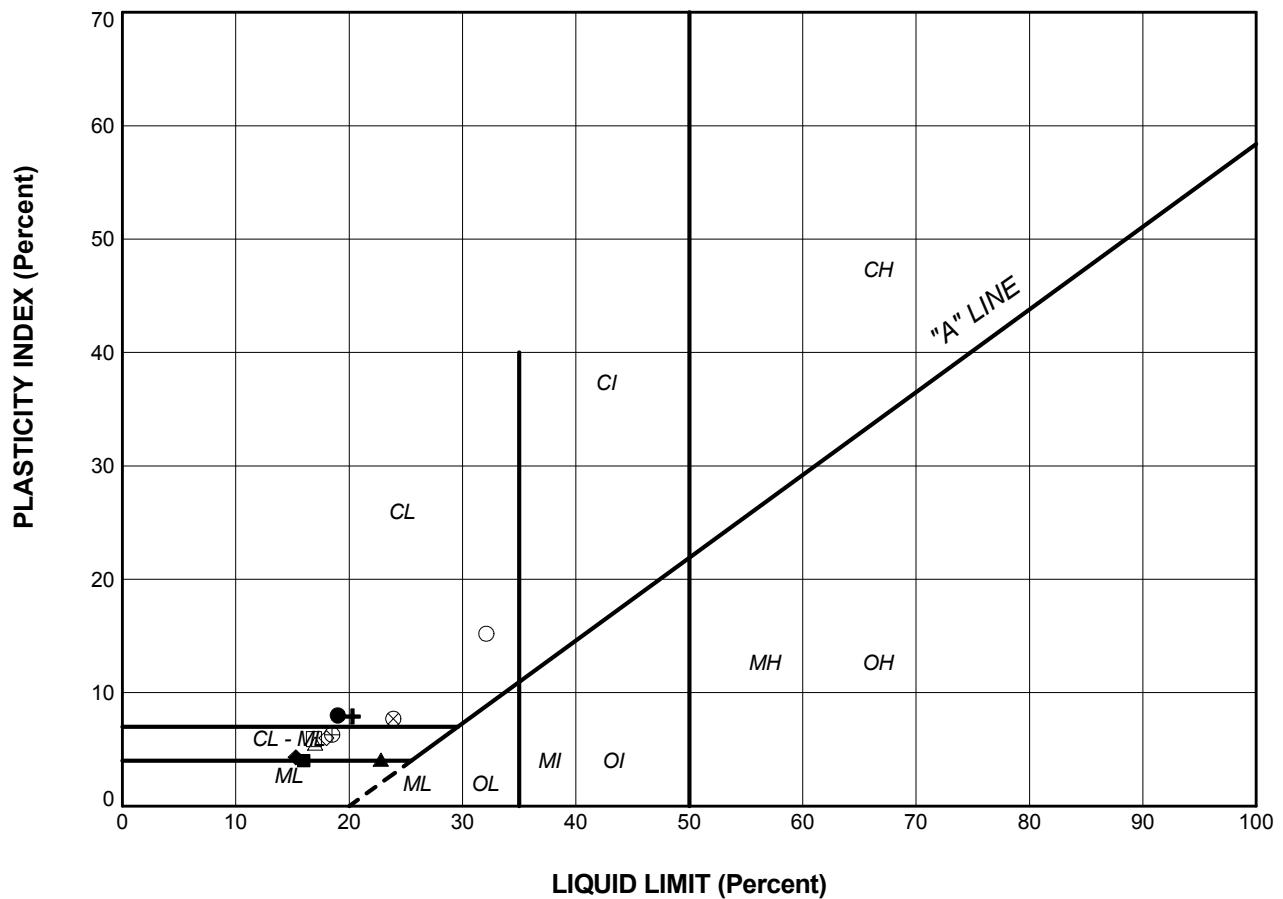


CLAY AND SILT	GRAVEL SIZE, mm					Cobble Size
	fine	medium	coarse	fine	coarse	
	SAND SIZE			GRAVEL SIZE		

LEGEND


SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	E6	5	232.8
■	E6	9	226.7
▲	E7	7	229.9
+	E8	6	231.4
◆	E8	8	228.4
◇	E8	11	223.8

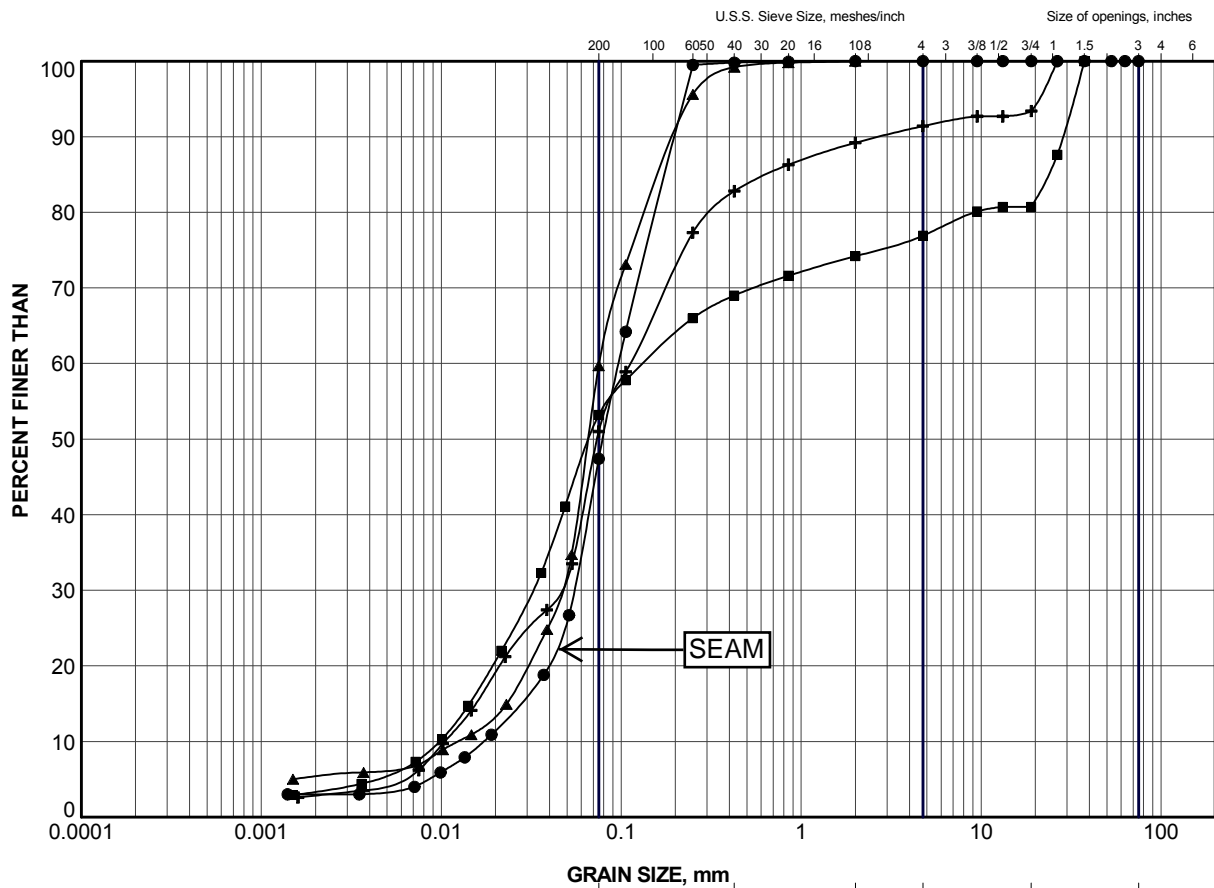
PROJECT					
HIGHWAY 11 EVELYN CREEK CULVERT					
TITLE					
GRAIN SIZE DISTRIBUTION SANDY CLAYEY SILT to SILT					
PROJECT No.		11-1191-0008		FILE No. 1111910008DET.GPJ	
DRAWN	JJL	Oct 2013	SCALE	N/A	REV.
CHECK	SEMC	Oct 2013			
APPR	FJH	Oct 2013			
 Golder Associates SUDBURY, ONTARIO			FIGURE B5.2		



LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	E1	11	19.0	11.0	8.0
■	E2	9	16.0	12.0	4.0
▲	E3	5	22.8	18.7	4.1
+	E3	9	20.3	12.4	7.9
◆	E4	6	15.3	11.0	4.3
◇	E4	9	18.0	12.0	6.0
○	E5	6	32.1	16.9	15.2
△	E5	10	17.0	11.5	5.5
⊗	E6	5	23.9	16.2	7.7
⊕	E6	9	18.5	12.2	6.3
□	E8	8	16.9	11.0	5.9


PROJECT					HIGHWAY 11 EVELYN CREEK CULVERT				
TITLE					PLASTICITY CHART SANDY CLAYEY SILT to SILT				
PROJECT No. 11-1191-0008			FILE No. 1111910008DET.GPJ						
DRAWN	JJL	Oct 2013	SCALE	N/A	REV.				
CHECK	SEMC	Oct 2013							
APPR	FJH	Oct 2013							
 Golder Associates SUDBURY, ONTARIO			FIGURE B6						

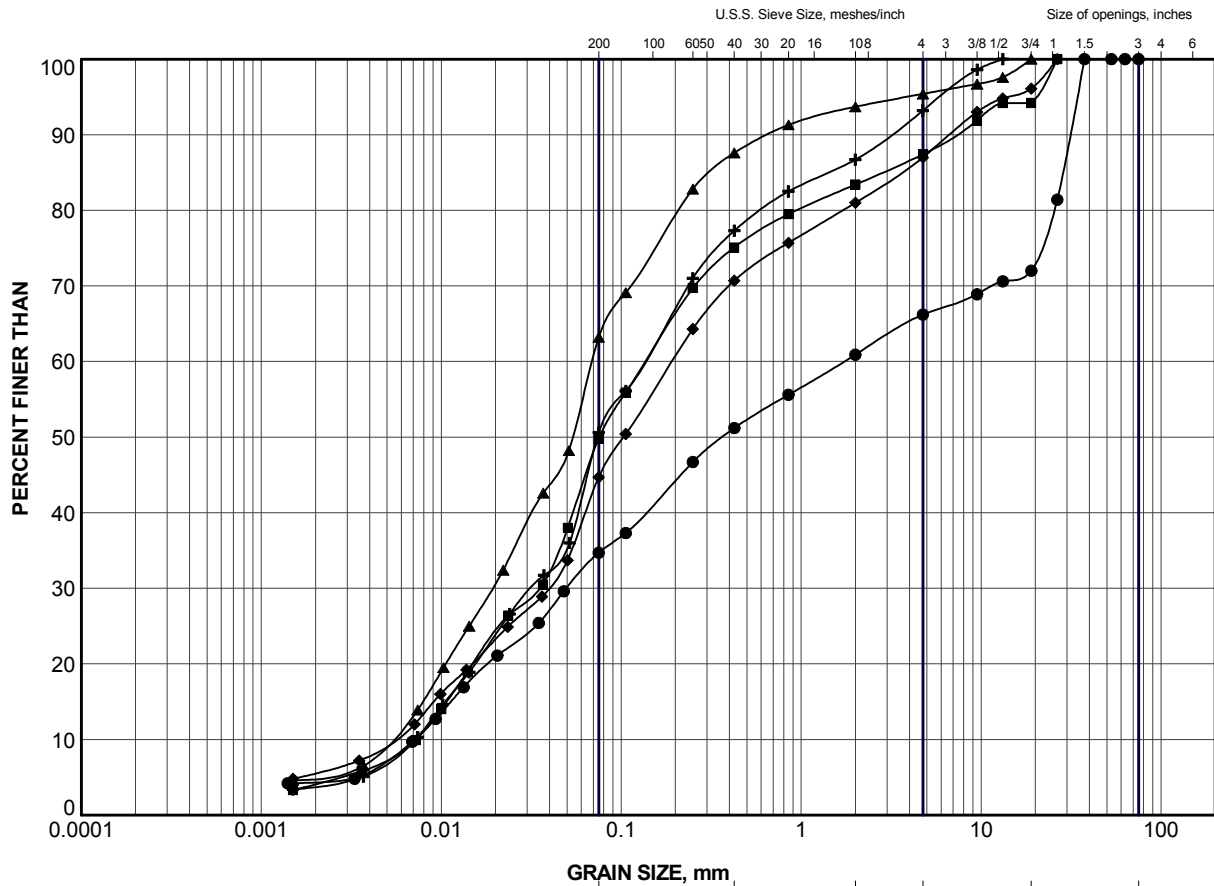


GRAIN SIZE, mm						
CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	E2	8	229.9
■	E3	11	223.7
▲	E7	9	226.8
✚	E7	12	222.3

PROJECT					HIGHWAY 11 EVELYN CREEK CULVERT				
TITLE					GRAIN SIZE DISTRIBUTION SAND and SILT				
PROJECT No.		11-1191-0008		FILE No.		1111910008DET.GPJ			
DRAWN	JJL	Oct 2013	SCALE	N/A	REV.				
CHECK	SEMC	Oct 2013							
APPR	FJH	Oct 2013							
 Golder Associates SUDBURY, ONTARIO			FIGURE B7						



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	E1	16	217.7
■	E3	14	219.1
▲	E4	12	222.2
+	E5	14	220.7
◆	E6	14	219.0

PROJECT

HIGHWAY 11
EVELYN CREEK CULVERT

TITLE

GRAIN SIZE DISTRIBUTION

SAND and SILT (TILL)



**Golder
Associates**
SUDBURY, ONTARIO

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

FIGURE B8



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, April 2008) 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.

OBSTRUCTIONS

Non-Standard Special Provision

The Contactor is hereby notified that the soils at the site of the Evelyn Creek Culvert 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.

For more information, visit golder.com

Africa	+ 27 11 254 4800
Asia	+ 86 21 6258 5522
Australasia	+ 61 3 8862 3500
Europe	+ 356 21 42 30 20
North America	+ 1 800 275 3281
South America	+ 56 2 2616 2000

solutions@golder.com
www.golder.com

Golder Associates Ltd.
1010 Lorne Street
Sudbury, Ontario, P3C 4R9
Canada
T: +1 (705) 524 6861

