REPORT

October 31, 2013

FOUNDATION INVESTIGATION AND DESIGN REPORT

REPLACEMENT OF PRUNE CREEK BRIDGE
HIGHWAY 583, SITE NO. 39W-046
TOWNSHIP OF WAY, ONTARIO
MINISTRY OF TRANSPORTATION, ONTARIO
GWP 5149-06-00 WP 5484-09-01

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FOUNDATION INVESTIGATION REPORT

REPLACEMENT OF PRUNE CREEK BRIDGE

HIGHWAY 583, SITE NO. 39W-046

TOWNSHIP OF WAY, ONTARIO

MINISTRY OF TRANSPORTATION, ONTARIO

GWP 5149-06-00

1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by LEA Consulting Ltd. (LEA) on behalf of the Ministry of Transportation, Ontario (MTO) to provide detail foundation engineering services for the replacement of the Prune Creek Bridge (Site No. 39W-046), located on Highway 583 (south of Hearst, Ontario) in the Township of Way.

The Terms of Reference for the Foundation Investigation are outlined in MTO's Request for Proposal dated, March 2011. The Scope of Work has been implemented in accordance with Golder's Supplementary Specialty Plan for foundations engineering services for this project, dated August 2011. The Base Plan (General Arrangement [GA] Drawing) showing the alignment of Prune Creek Bridge was provided to Golder by LEA in October 2013.

The purpose of this investigation is to establish the subsurface conditions at the location of the proposed replacement bridge structure, including the associated approach embankments, by borehole drilling, rock coring, in situ testing and laboratory testing on selected soil and rock core samples. The location of the investigation area is shown in plan on Drawings 1 and 4.

2.0 SITE DESCRIPTION

The Prune Creek Bridge site is situated in the Township of Way, on Highway 583 approximately 16.2 km south of the west junction of Highway 11 and Highway 583, in Hearst, Ontario. The surrounding land is generally flat, but slopes down towards the creek banks along the north and south sides of the creek, with nearby residential development and sparse tree covered terrain beyond the highway right-of-way limits. Immediately east and west of the existing bridge, the creek bends northerly. The creek banks in the area adjacent to the existing bridge are vegetated with landscaped grass and small shrubs. The creek flows in a westerly direction and is about 10 m wide at the existing bridge location. The creek meanders to the north immediately west of the existing bridge and runs essentially parallel to the highway at the west toe of slope.

The existing structure consists of an approximately 10 m wide by 33 m long, three-span bridge, constructed in 1962. The structure is founded on timber piles driven to unknown depths. The existing ground surface along the structure ranges from about Elevation 249.1 m to 248.9 m sloping downwards from south to north. The existing embankment front slopes are formed at approximately 2 horizontal to 1 vertical (2H:1V) on both the north and south sides of the creek. The existing embankment side slopes are generally about 2H:1V. There are no visible signs of approach embankment instability or settlement.

The water level shown on the GA drawing for November 2011 is at Elevation 244.8 m. The creek level measured by Golder during the field investigations, which took place in March to April and in December 2012, varied between Elevation 245.5 m and 247.6 m. The high water level is reported to be Elevation 247.3 m; however, a creek water level at Elevation 247.6 m was recorded on March 22, 2012 during the spring freshet. The existing highway embankment grade is about 4 m above the surrounding ground surface adjacent to the creek.

3.0 INVESTIGATION PROCEDURES

The fieldwork for this subsurface investigation was carried out on March 22, April 19 to 21 and December 6 to 12, 2012, at which time eight boreholes (Boreholes P1 to P8) were advanced. On June 4 2013, a supplementary borehole was advanced immediately adjacent to Borehole P8 to confirm the refusal condition encountered originally at a shallower depth. Boreholes P1 and P6 to P8/8A were advanced using a truck-mounted drill rig supplied and operated by Landcore Drilling Inc. of Sudbury, Ontario; whereas, Boreholes P2 to P5 were advanced using a D-25 semi-portable drill rig supplied and operated by Walker Drilling Ltd. of Barrie, Ontario. Boreholes P1, P2, P7 and P8/P8A were advanced approximately at the corners of the proposed north and south abutments of the bridge, which partially overlap the existing Highway 583 embankment. Boreholes P3 and P6 were advanced along the proposed approach embankments. Boreholes P4 and P5 were advanced along the west toe of slope at the north approach on the alignment of an originally proposed retaining wall, where the existing creek meanders towards the existing roadway. The approximate locations of the Boreholes advanced as part of the field investigations are shown on Drawings 1 and 4.

Borehole P1 and P6 to P8/P8A were advanced using 108 mm ID continuous flight hollow stem augers and NW casing, while Boreholes P2 to P5 were advanced using NW casing and wash boring techniques. Where coring was required, a NQ size core barrel was used. Soil samples were obtained at intervals of depth of about 0.75 m to 1.5 m, using a 50 mm outer diameter (O.D.) split-spoon sampler (operated by an automatic hammer on the drill rig), in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586). Boreholes advanced by semi-portable equipment employed full weight hammers lifted manual and dropped from the SPT height. Selected samples of the cohesive soils were obtained using 76 mm O.D. thin-walled 'Shelby' tubes (ASTM D1587, Thin-Walled Tube Sampling) for relatively undisturbed samples. Field vane shear tests were carried out in cohesive soils for determination of undrained shear strengths (ASTM D2573, Field Vane Strength Shear Test) using MTO Standard "N" size vanes. All open boreholes were backfilled upon completion in accordance with Ontario Regulation 903 Wells (as amended).

A supplemental investigation consisting of two test pits (TP1 and TP2) was carried out on September 25, 2013 to obtain soil samples in chemical characterization for soil disposal purposes. The test pits were advanced using a Komatsu 228 excavator operated by Villeneuve Construction Co. Ltd. of Hearst, Ontario. Test pits TP1 and TP2 were located on the west bank of the existing creek alignment where the creek meanders towards the north parallel to the existing highway embankment, as shown on Drawing 4.

The groundwater conditions were observed in the open boreholes and test pits during and immediately following the drilling/excavation operations and a standpipe piezometer was installed in each of Boreholes P1 and P2 to permit monitoring of the groundwater level. The piezometers consist of a 50 mm diameter PVC pipe, with a 3 m long slotted screen sealed within a sand filter pack at a selected depth interval within the boreholes. 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 a mixture of cuttings from the boreholes and bentonite. A seal of bentonite was placed to ground surface. The piezometer installation details and water level readings are indicated on the borehole records contained in Appendix A. The piezometers were decommissioned on June 4 and June 5, 2013.

The fieldwork was supervised on a full-time basis by a member of Golder's technical staff who located the boreholes/test pits in the field, arranged for the clearance of underground service locations, directed the drilling/excavation, sampling and in situ testing operations and logged the boreholes/test pits. The soil samples were identified in the field, placed in labelled containers and transported to Golder's laboratory in Sudbury for further examination and laboratory testing. Index and classification tests consisting of water content, organic content, Atterberg limits and grain size distribution were carried out on selected soil samples. In addition, a one-dimensional consolidation (oedometer) test was carried out on one sample of the silty clay to clay deposit. Uniaxial compressive strength (UCS) testing was carried out on specimens of the recovered bedrock core. 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 and Drillhole sheets in Appendix A and on the figures contained in Appendix B.

A sample of the creek water was obtained during the borehole 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 including: resistivity/conductivity; ph; sulphate; and cholride. The results of the analytical testing are summarized in Table C1 in Appendix C.

During the supplemental test pit investigation, soil samples were obtained from each soil horizon, field screened for evidence of potential petroleum impacts (head space readings) and immediately placed into laboratory-supplied jars and vials. One soil sample from each test pit, which indicated the highest head space readings for hydrocarbons, was submitted to a Canadian Association for Laboratory Accreditation Inc. (CALA) accredited laboratory under change of custody procedures for analysis of a suite of parameters for soil disposal purposes, namely: benzene, toluene, ethylbenzene and xylenes (BTEX); petroleum hydrocarbon fractions F_1 to F_4 (PHC F_1-F_4); metals, chloride and sodium and toxicity leachate characterization procedure (TCLP). The results of the analytical testing are included in Appendix C.

The borehole and test pit locations and elevations were measured in the field by Golder personnel relative to existing site features and surveyed to an existing temporary benchmark. The borehole and test pit locations (referenced to the MTM NAD83 co-ordinate system), ground surface elevations (referenced to Geodetic datum) and depths are shown on Drawings 1 and 4 and presented on the Record of Borehole and Test Pit sheets in Appendix A and are summarized below.

Borehole Number	MTM NAD83 Northing (m)	MTM NAD83 Easting (m)	Ground Surface Elevation (m)	Borehole/Test Pit Depth (m)
P1	5495077.0	323212.5	249.1	17.4
P2	5495106.2	323199.2	245.7	18.3
P3	5495124.1	323197.4	246.7	9.8
P4	5495150.0	323194.7	245.6	8.2
P5	5495168.9	323192.7	246.4	8.1
P6	5495061.1	323209.9	249.1	11.3
P7	5495080.2	323201.7	248.0	15.4
P8/8A	5495112.1	323211.7	248.8	19.8
TP1	5495134.0	323187.0	245.8	3.7
TP2	5495143.9	323185.4	245.6	3.7

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

Based on NOEGTS[1] Mapping, the subsoils in the vicinity of the Prune Creek Bridge site generally consist of clayey till deposited as a ground moraine. A bedrock plain is located about 40 m south of the bridge site and an area of peat/organics is located about 100 m north of the bridge site.

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/test pit locations, ground surface elevations and interpreted stratigraphic conditions at the site are shown on Drawings 1 to 4. The detailed subsurface soil and groundwater conditions encountered in the boreholes/test pits and the results of in situ and laboratory testing are given on the Record of Borehole, Drillhole and Test Pit sheets contained in Appendix A. The results of geotechnical laboratory testing are also presented on Figures B1 to B10 and photographs of the bedrock core are shown on Figure B11, contained in Appendix B. The stratigraphic boundaries shown on the borehole/test pit records and on the interpreted stratigraphic profiles on Drawings 1 to 4 are inferred from non-continuous sampling 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 embankment fill and/or peat, overlying a soft to stiff deposit of silty clay to clay. The upper cohesive deposit is underlain by a very loose to compact deposit of sandy silt to sand and silt till and a hard deposit of sandy clayey silt to clayey silt till. A more detailed description of the soil deposits encountered in these boreholes is provided in Sections 4.2.1 to 4.2.8.

4.2.1 Asphalt Surface Treatment

A 150 mm to 200 mm thick layer of asphalt surface treatment material was encountered from ground surface (Elevation 249.1 m to 248.8 m) in Boreholes P1, P6 and P8, which were advanced through the existing Highway 583.

4.2.2 Fill

Embankment fill consisting of granular material and/or cohesive soil fill was encountered underlying the asphalt surface treatment layer in Boreholes P1, P6 and P8/8A and at ground surface in Borehole P4. The total thickness of fill the below the existing roadway surface layer is between 2.1 m and 2.8 m and the top of the fill is between about Elevation 248.9 m and 248.6 m. Borehole P4, which was advanced at the northwest toe of slope where the creek meanders towards the roadway, encountered fibrous peat fill and granular fill from ground

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

[2] Ontario Geological Survey, Geology of Ontario, 1991. , 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.

surface, at Elevation 245.6 m. The total thickness of the fill at the northwest toe of slope in Borehole P4 is 1.5 m, comprised of a 0.6 m thick layer of peat and a 0.9 m thick layer of silty sand.

The granular fill under the roadway surface layer consists of moist to wet, brown silty sand, sand or sand and gravel to sand, trace to some silt. The Standard Penetration Test (SPT) “N”-values measured within the granular fill range from 5 blows to 23 blows per 0.3 m of penetration indicating a loose to compact relative density.

The natural moisture content measured on two samples of the granular fill is 3 per cent and 4 per cent.

The cohesive fill is comprised of moist, brown clayey silt, trace to some gravel, trace organics. The SPT “N”-values measured within the clayey silt fill are 13 blows and 21 blows per 0.3 m suggesting a stiff to very stiff consistency

A grain size distribution test completed on one selected sample of the cohesive fill is shown on Figure B1 in Appendix B. The results of an Atterberg limits test completed on the same sample of the cohesive fill yielded a liquid limit of about 24 per cent, a plastic limit of about 16 per cent and a corresponding plastic index of 8 per cent, as shown on Figure B2 in Appendix B, and indicates that the material is a clayey silt of low plasticity.

The natural moisture content measured on one sample of the cohesive fill is 11 per cent.

4.2.3 Silty Peat to Peat

A 0.6 m to 2.3 m thick deposit of moist to wet, brown to black, amorphous or fibrous silty peat to peat some silt was encountered from ground surface, between Elevations 248.0 m and 245.6 m in Boreholes P2, P3, P5 and P7 and Test Pits TP1 and TP2.

The SPT “N”-values measured within the peat to silty peat deposit range from 3 blows to 6 blows per 0.3 m of penetration, suggesting a soft to firm consistency.

The natural moisture content measured on one sample of the silty peat is 47 per cent.

4.2.4 Silty Clay to Clay

A 1.8 m to 5.0 m thick deposit of wet brown to grey silty clay to clay, trace to some sand, trace to some gravel, trace to some organics was encountered below the fill in Borehole P1, P4, P6 and P8/8A and below the peat deposit in Boreholes P2, P3, P5 and P7 where the deposit was fully penetrated. In Test Pits TP1 and TP2, where the silty clay to clay deposit was not fully penetrated, the deposit is up to 3.7 m thick. The surface of the deposit was encountered at depths between 0.6 m and 3.0 m below ground surface, between Elevations 247.2 m and 244.1 m.

The SPT “N”-values measured within the silty clay to clay deposit range from 0 blows (i.e. weight of hammer) to 12 blows per 0.3 m of penetration and indicate a very soft to stiff consistency. The higher “N”-values were encountered near the surface of the deposit (i.e. directly beneath the fill) or at the bottom of the deposit (directly over the sandy silt to sand and silt till deposit). In situ field vane tests within the silty clay to clay deposit measured undrained shear strengths ranging from 19 kPa to 30 kPa, indicating a soft to firm consistency. The in situ vane test results, together with the SPT “N”-values excluding those within the upper or lower portions of the deposit, suggest that the silty clay to clay deposit generally has a soft to firm consistency.

The natural moisture content measured on twenty-three samples of the silty clay to clay deposit ranges from 22 per cent to 79 per cent. The natural moisture content measured on one sample of the clayey silt portion of the deposit is 19 per cent.

The results of grain size distribution tests completed on nine selected samples of the silty clay to clay deposit are shown on Figure B3 in Appendix B. It should be noted that Sample 6 from Borehole P2, taken across the silty clay to clay deposit and the underlying sandy silt to sand till deposit, returned a grain size distribution similar to that of the underlying sandy silt silt till deposit and it is considered that this test result was influenced by the composition of the underlying deposit.

Atterberg limits tests were carried out on nineteen selected samples of the cohesive deposit, eighteen of which returned liquid limits ranging from about 38 per cent to 82 per cent, plastic limits ranging from about 18 per cent to 36 per cent and plasticity indices ranging from about 20 per cent to 46 per cent, indicating a silty clay to clay of intermediate to high plasticity as shown on Figure B4 in Appendix B. The Atterberg limits test on Sample 6 from Borehole P2, near the transition zone with the underlying till deposit, returned a liquid limit of about 22 per cent, a plastic limit of about 13 per cent and a plasticity index of about 9 per cent, indicating a clayey silt of low plasticity.

A laboratory consolidation test was carried out on one sample of the silty clay to clay deposit obtained from a Shelby tube sample in Borehole P7. A preconsolidation stress of about 80 kPa was estimated from the void ratio versus logarithmic pressure plot and from the total work versus pressure plot. A bulk unit weight of about 15.9 kN/m^3 and a specific gravity of about 2.74 were measured on the consolidation test sample. Details of the test results are shown on Figure B5 in Appendix B, and the test results are summarized below.

Borehole/ Sample No.	Sample Depth/ Elevation	σ_{vo}' (kPa)	σ_p' (kPa)	$\sigma_p' - \sigma_{vo}'$ (kPa)	OCR	C_c	C_r	e_o	c_v^* (cm^2/s)
Borehole P7/ Sample 4	3.4 m/ 244.7 m	32	80	48	2.5	0.78	0.069	1.88	1.5 x 10^{-3}

Note: For the overly consolidated stress range, 13 kPa ≤ σ_v' ≥ 31 kPa

where:
- σ_{vo}' is the in situ vertical effective overburden stress in kPa
- σ_p' is the preconsolidation stress in kPa
- OCR is over consolidation ratio
- e_o is initial void ratio
- C_c is the compression index
- C_r is the recompression index
- C_v is the coefficient of consolidation in cm^2/s

4.2.5 Sand and Gravel and Sand

A 1.0 m thick deposit of wet, brown sand and gravel and a 1.3 m thick deposit of wet, brown/grey sand some gravel trace to some silt was encountered below the cohesive deposit in Boreholes P3 and P7, respectively. The surface of these deposits was encountered at depths of 4.1 m and 5.8 m below ground surface, corresponding to Elevation 242.6 m and Elevation 242.2 m in the respective boreholes.

The SPT "N"-values measured within the sand to sand and gravel deposit are 10 blows and 16 blows per 0.3 m of penetration, indicating a compact relative density.

The natural moisture content measured on one sample of the sand deposit is 17 per cent.

The result of a grain size distribution test completed on one selected sample of the sand deposit is shown on Figure B6 in Appendix B.

4.2.6 Sandy Silt to Sand and Silt (Till)

A till deposit comprised of moist to wet, brown to grey sandy silt to sand and silt, trace to some gravel, trace to some clay was encountered below the silty clay to clay deposit in Boreholes P1, P2, P4 to P6 and P8 and below the sand and gravel and the sand deposits in Boreholes P3 and P7. The surface of this deposit was encountered at depths between 4.4 m and 7.6 m below ground surface, between Elevations 243.0 and 240.9 m. The deposit is between 1.6 and 3.9 m thick where it was fully penetrated in Boreholes P1 to P3, P7 and P8. In boreholes P4 to P6, where the boreholes were terminated within this deposit, the deposit is up to 5.2 m thick.

The SPT "N"-values measured within the sandy silt to sand and silt till deposit range from 0 blows (i.e. weight of hammer) to 237 blows per 0.3 m of penetration and indicate that the deposit is generally very loose to loose in the upper 2 m to 3 m becoming compact to very dense with depth.

The natural moisture content measured on eleven samples of the sandy silt to sand and silt till deposit range from 9 per cent to 18 per cent. In general, the upper very loose to loose portion of the deposit was wet, while the lower compact to dense portion of the deposit was moist.

The results of grain size distribution tests completed on eight selected samples of the sandy silt to sand and silt till deposit are shown on Figure B7 in Appendix B.

Atterberg limits tests were carried out on seven selected samples of the deposit, three of which classify the material as non-plastic. The results of the remaining four Atterberg limits tests yielded liquid limits ranging from about 15 per cent to 17 per cent, plastic limits ranging from about 12 per cent to 13 per cent and plasticity indices ranging from 2 per cent to 4 per cent, as shown on Figure B8 in Appendix B, and indicate that the material is classified as silt of slight plasticity.

4.2.7 Sandy Clayey Silt to Clayey Silt (Till)

A till deposit comprised of moist to dry, grey sandy clayey silt to clayey silt, trace to some gravel was encountered below the sandy silt to sand and silt till deposit in Boreholes P1 to P3, P7 and P8/8A. The surface of the deposit was encountered at depths between 7.6 m and 10.2 m below ground surface, between Elevations 239.3 m and 237.0 m. The deposit is 6.6 m thick where it was fully penetrated in Boreholes P2 and up to 7.2 m thick in Boreholes P1 to P3 and P8/8A, which were terminated within this deposit.

SPT "N"-values measured within this deposit range from 43 blows to 155 blows per 0.3 m of penetration suggesting a hard consistency. In seven instances, the split-spoon sampler did not penetrate the full sample depth due to the presence of inferred gravel/cobbles. In eight instances, NQ coring was required to advance the boreholes through the deposit. Grinding of the augers and/or casing was noted throughout this deposit in Boreholes P1 and P2.

At the north abutment in Borehole P2, cobbles were encountered at a depth of 11.6 m below ground surface corresponding to Elevation 234.1 m. At the south abutment in Boreholes P7 and P1, a 2.7 m thick zone of coarse gravel and cobbles was encountered between 9.8 m and 12.5 m depth (Elevation 238.2 m to 235.5 m) and a 1.4 m thick zone was encountered between 15.4 m and 16.0 m depth (Elevation 233.7 m and 233.1 m), in the respective boreholes.

The natural moisture content measured on nine samples of the sandy clayey silt to clayey silt till deposit range from 8 per cent to 11 per cent.

The results of grain size distribution tests completed on six selected samples of the sandy clayey silt to clayey silt till are show on Figure B9 in Appendix B.

Atterberg limits tests, carried out on six selected samples, yielded liquid limits ranging from about 18 per cent to 21 per cent, plastic limits ranging from about 11 per cent to 12 per cent and plasticity indices ranging from about 6 per cent to 10 per cent, as shown in Figure B10 in Appendix B. These test results indicate that the material is classified as clayey silt of low plasticity.

4.2.8 Refusal/Bedrock

Refusal to further split spoon advancement was recorded in Borehole P7 at a depth of about 15.4 m below ground surface, Elevation 232.6 m. The bedrock surface was encountered in Borehole P2 and P8/8A at depths of 15.3 m and 16.6 m below ground surface, respectively, corresponding to Elevations 230.4 m and 232.2 m. The bedrock was cored for lengths of 3.0 m and 3.2 m in the respective boreholes. The retrieved bedrock core is described as fine grained, slightly to moderately weathered, grey, gneiss. In Borehole P2, a 100 mm thick vein of white quartz was noted within the core at a depth of 16.1 m below ground surface (Elevation 229.6 m) and a 200 mm thick mica schistose zone was noted at a depth of 16.3 m below ground surface (Elevation 229.3 m). In Borehole P8/8A, a 400 mm thick quartz vein was encountered at a depth of 19.4 m below ground surface (Elevation 229.4 m). Photographs of the retrieved bedrock core are shown on Figure B11 in Appendix B.

The Total Core Recovery (TCR) from Boreholes P2 and P8/8A is 100 per cent. The Rock Quality Designation (RQD) measured on the core runs is 79 per cent to 31 per cent, indicating a rock mass of poor to good quality as per Table 3.10 of the Canadian Foundation Engineering Manual (CFEM, 2006)[3].

Laboratory Uniaxial Compression Strength (UCS) testing was carried out on two core samples of the bedrock from Borehole P2. The UCS values are presented on the Record of Drillhole sheet in Appendix A and summarized below. The bedrock is considered to be strong as per Table 3.5 of CFEM (2006)[3].

Borehole	Elevation (m)	UCS (MPa)
P2	230.2	60
P2	229.1	77

4.3 Groundwater Conditions

Groundwater levels were measured in the open boreholes and test pits during and upon completion of drilling/excavation and a piezometer was installed in each of Boreholes P1 and P2 to monitor the groundwater

[3] Canadian Geotechnical Society 2006. Canadian Foundation Engineering Manual, 4th Edition, BiTech Publications.

levels. The piezometers are sealed within the sandy silty clay to clay and silty peat deposits in the respective boreholes. Details of the piezometer installations and water level readings are presented on the Record of Borehole sheets in Appendix A. The piezometers were decommissioned on June 4 and June 5, 2013.

The measured groundwater levels in the open boreholes, test pits and piezometers are presented below.

Borehole	Installation	Time and/or Date	Groundwater Depth (m)	Groundwater Elevation (m)
P1	Open Borehole	March 22, 2012	2.3	246.8
	Piezometer	April 20, 2012	1.8	247.3
		December 6, 2012	2.4	246.7
		June 5, 2013	1.8	247.3
P2	Open Borehole	April 20, 2012	0.4	245.3
	Piezometer	December 6, 2012	0.2	245.5
		June 4, 2013	0.0	245.7
P3	Open Borehole	April 20, 2012	0.7	246.0
P4	Open Borehole	April 21, 2012	0.8	244.8
P5	Open Borehole	April 21, 2012	1.2	245.2
P6	Open Borehole	December 6, 2012	2.4	246.7
P7	Open Borehole	December 9, 2012	1.5	246.5
P8/8A	Open Borehole	December 12, 2012	6.2	242.6
TP1	Open Test Pit	September 25, 2013	Dry	-
TP2	Open Test Pit	September 25, 2013	3.7	241.9

Groundwater levels encountered in the boreholes/test pits during and shortly after drilling/excavating 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 shown on the GA drawing for November 2011 is at Elevation 244.8 m. The high water level is reported to be at Elevation 247.3 m. During the preliminary field investigation, the water level in Prune Creek was measured at Elevation 247.3 m on March 20, 2012, and due to the spring freshet, the creek level rose to Elevation 247.6 m as measured on March 22, 2012. The creek level was measured at Elevation 245.1 m between April 17 and 21, 2012, and on December 9, 2012. The creek level was measured at Elevation 245.7 m on June 4, 2013 when the piezometers were decommissioned.

4.4 Analytical Testing

The analytical test results on a sample of creek water are presented in Table C1 in Appendix C. The creek water was tested for suite of parameters including: resistivity/conductivity; ph; sulphate; and chloride.

The analytical test results on two soil samples, which indicated the highest head space readings for hydrocarbons from the test pit investigation, are also presented in Appendix C. The results of the analytical laboratory testing of the soil samples indicate the following:

- BTEX and F_1-F_4 fractions are below the method detection limits;
- The concentration of the metals are lower than the MOE Generic Site Conditions Standards Table 1 (Full Residential/Parkland/Instituitional/Industrial/Commercial/Community Property Use) as outlined in the Soil, Groundwater and Sediment Standards for Use Under Part XV.1 of the Environmental Protection Act (April 15, 2011); and
- TCLP sample concentrations are lower than O. Reg 558 – Schedule 4 (Leachate Quality Criteria) levels.

5.0 CLOSURE

The field drilling program was supervised by Mr. Indulis Dumpis, Mr. Ed Savard and Mr. Shane Albert. This Foundation Investigation Report was prepared by Mr. David Muldowney, P.Eng. and reviewed by Ms. Sarah Coyne, P.Eng., a geotechnical engineer and Associate. Mr. Jorge Costa, P.Eng., a Designated MTO Foundations Contact and Principal with Golder, conducted an independent quality control review of this report.

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

PART B

FOUNDATION DESIGN REPORT

REPLACEMENT OF PRUNE CREEK BRIDGE

HIGHWAY 583, SITE NO. 39W-049

TOWNSHIP OF WAY, ONTARIO

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GWP 5149-06-00

October 31, 2013
Report No. 11-1191-0008-2

6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides foundation design recommendations for the proposed replacement of the Prune Creek Bridge on Highway 583, south 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 three-span Prune Creek Bridge structure was constructed in 1962 and is supported on timber piles of unknown length. We understand that this structure is nearing the end of its service life and therefore replacement will be required. We understand that the replacement bridge will be a single-span structure, on an alignment which has been shifted to the west of the existing bridge. The new structure will be constructed on a slight skew relative to the existing structure, shifted by about 5 m at north abutment (centreline to centreline) and about 4 m at the south abutment. The east half (northbound lane) of the proposed bridge and approach embankments will coincide with the west half of the existing bridge and approach embankments. The west half (southbound lane) of the proposed bridge and approach embankments will be located over native ground extending to or beyond the existing west toe of slope. The current General Arrangement (GA) drawing indicates that an integral abutment structure founded on driven steel piles is the preferred bridge support system alternative from a structural perspective.

The finished grade for the realigned Highway 583 will be approximately Elevation 249.1 m at the south abutment and Elevation 248.9 m at the north abutment, requiring approach embankments up to 3.3 m high relative to the existing natural ground surface at the existing west toe of slope. The existing and proposed highway grades will essentially remain the same with only a minor grade raise (i.e. about 0.1 m) required at the north abutment. Given the present meander of the creek along the north approach embankment at the west toe of slope, we understand that the existing Prune Creek will be re-aligned further to the west to accommodate the proposed embankment widening.

The subsurface conditions within the existing roadway generally consist of a layer of asphalt surface treatment underlain by granular fill and clayey silt fill. An organic deposit comprised of peat was encountered from ground surface where the boreholes were advanced beyond the existing embankment and a cohesive deposit of silty clay to clay was encountered in most boreholes and test pits underlying the fill or organic deposit, which is in-turn underlain by a cohesionless deposit of generally loose sandy silt to sand and silt till and a cohesive deposit of hard sandy clayey silt to clayey silt till. In Boreholes P2 and P8/8A, where the till layers where fully penetrated, the bedrock surface was encountered at depths of 15.3 m and 16.6 m below ground surface, at Elevation 230.4 m and Elevation 232.2 m, respectively. At the time of the investigations, the groundwater/creek level varied from Elevation 244.8 m to 247.6 m.

6.2 Foundations

Based on the proposed bridge geometry and the subsurface conditions at this site, both shallow and deep foundation options have been considered for support of the abutments 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. Based on the comparison of the foundation alternatives, we recommend that the bridge be supported on deep foundations comprised of steel H-piles extending into the hard clayey silt to sand clay silt till deposits.

6.3 Shallow Foundations

In order to provide sufficient geotechnical resistance to support the proposed bridge abutments and wing walls, shallow strip/spread footings would need to be founded on the hard till layer. Given that the depth to the hard sandy clayey silt till deposit, which is up to 11.8 m below the existing roadway and 8.7 m below existing ground surface at the west toe of slope, strip or spread footings are not considered to be practical for this site. In addition, the base of these excavations would be up to 10.3 m below the high water level and extensive shoring/dewatering would be required to facilitate such an excavation and construction in-the-dry. Spread footings at a higher elevation are not recommended due to the low geotechnical resistance that would be available from both the loose sandy silt to sand and silt till or the upper soft to firm cohesive deposits, as well as the need for dewatering/shoring prior to excavating operations.

6.4 Deep Foundations

Deep foundations, consisting of concrete caissons or driven steel piles have been considered for support of the abutments for the replacement structure and are discussed in the following sections.

6.4.1 Concrete Caissons

Concrete caissons socketed into bedrock are considered feasible and will provide increased geotechnical axial resistance compared to driven steel piles, and it may be possible to eliminate the below-grade pile cap and associated excavation. However, due to the presence of cobbles and boulders within the till deposits, it may be difficult to install and inspect concrete caissons at this site, even with relatively small diameter caissons (i.e. less than 0.9 m diameter). Furthermore, due to the high groundwater levels, temporary or permanent liners would be required, and the caisson would have to be constructed by tremie methods. As such, concrete caissons are not considered practical for this site.

6.4.2 Driven Steel Piles

We recommend that the bridge be supported on steel HP310X110 piles driven to about 3 m into the hard sandy clayey silt to clayey silt till (having Standard Penetration Test “N”-values greater than about 100 blows per 0.3 m of penetration). Driven steel pile foundations also allow for the pile caps to be constructed at a higher elevation than footings, resulting in less excavation and unwatering needs. Given that the till deposits are glacially derived and contain cobbles as encountered in the boreholes advanced at the abutments (and potentially boulders), the piles could “hang up” or be deflected from their intended vertical alignment. Therefore, consideration should be given to using a heavier H-pile section, such as HP310x132 piles, to reduce the potential for damage to the piles during driving to the required tip elevation. Hollow Structural Section (HSS) “pipe” piles are considered to pose a higher risk of “hanging up” or being deflected away from their vertical or battered orientation (due to their larger end area) and are not considered as practical for this site. The following sections provide details regarding the tip elevation, geotechnical axial resistances/reactions and downdrag loads, set criteria and pile driving notes, resistance to lateral loads and frost protection for driven steel H-piles.

6.4.2.1 Design Tip Elevation

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

Foundation Element	Borehole Numbers	Proposed Underside of Pile Cap Elevation[1] (m)	Design Pile Tip Elevation (m)	Estimated Approximate Pile Length (m)
South Abutment	P1 and P7	245.0	234.0	11.0
North Abutment	P2 and P8/8A	245.0	233.0 – 232.2	12.0 – 12.8

Note 1. As taken from the GA drawing provided by LEA in October 2013

6.4.2.2 Geotechnical Axial Resistance

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

Pile Section	Factored Geotechnical Axial Resistance at ULS	Geotechnical Axial Resistance at SLS (for 25 mm settlement)
HP310X110	1,600	1,100
HP310X132	1,800	1,200

Generally, HP310X110 piles are used; however, we understand that a heavier pile section, HP310X132, is being considered for structural reasons, and is considered appropriate from a foundations perspective. The estimated tip elevations assume that the piles will penetrate about 3 m into the hard sandy clayey silt to clayey silt till deposit. Due to the presence of cobbles (up to 275 mm sizes) and the potential for boulders to be present within the till deposits, the piles could “hang up” or be deflected prior to reaching the design tip elevation. At the north abutment (Borehole P2), cobbles were encountered at a depth of 11.6 m below ground surface (Elevation

234.1 m). At the south abutment, zones of coarse gravel and cobbles were encountered between the depths of 9.8 m and 12.5 m below ground surface (Elevations 238.2 m and 235.5 m) in Borehole P7, and between 15.4 m and 16.0 m (Elevations 233.7 m and 233.1 m) in Borehole P1.

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. If CSPs are used, an NSSP detailing the installation method and gradation of this sand should be included in the Contract Documents; an example is provided in Appendix D.

6.4.2.3 Downdrag

The proposed Highway 538 approach embankments, which will be up to 3.3 m above the existing ground surface at the west toe of slope, will induce settlement of the underlying soft to firm silty clay to clay deposit. Downdrag loads (negative skin friction) will be induced on the piles supporting the abutments as a result of the addition of the approach embankment fill loads after pile installation is complete, causing settlement of the cohesive soil relative to the piles. Downdrag loads will need to be taken into account for design of the piles.

The structural design of the west abutment piles should be based on an estimated unfactored downdrag load of 75 kN acting on the piles. The structural capacity of the piles must be checked for the factored dead and downdrag loads in accordance with Section C6.8.4 of the Commentary to the Canadian Highway Bridge Design Code (CHBDC, 2006) for ULS conditions.

6.4.2.4 Set Criteria and Pile Driving Note

All pile installation/driving should be in accordance with Ontario Provincial Standard Specification (OPSS) 903 (Deep Foundations). The piles should be provided with driving shoes or flange plates (reinforced tips) in accordance with OPSD 3000.100 (Steel H-Driving Shoe) to minimize damage to the pile tip during driving. Given the presence of cobbles (and potential boulders) within the till deposits and potential for damage to the pile tip during driving, consideration could be given to using the heavier pile section (HP310x132), as noted in Section 6.4.2.2.

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 hard sandy clayey silt to clayey silt deposit and to also avoid overdriving and possibly damaging the piles.

For friction piles, the pile capacity must be verified in the field by the use of the Hiley Formula in accordance with MTO's Standard Drawing SS103-11 (Pile Driving Control, dated April 2008) during the final stages of driving for the ultimate capacity at the elevations provided in Section 6.4.2.1. The ultimate geotechnical axial resistance predicted from the Hiley Formula should then be multiplied by a geotechnical resistance factor equal to 0.5 as per current MTO practice to verify the factored ULS design value. An NSSP, which outlines the above set criteria, should be included in the Contract; an example is included in Appendix D.

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 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 EL 234.0 m (South Abutment) and EL 233.0 m (North Abutment).

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 EL 234.0 m (South Abutment) and EL 233.0 m (North Abutment).

As noted in Section 6.4.2.1, bedrock was encountered in Borehole P8/P8A on the east side of the North Abutment at about Elevation 232.2 m, as confirmed by coring. Therefore, the Contract must make provision for varying pile lengths.

6.4.2.5 Resistance to Lateral Loads

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

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

for non-cohesive soils:

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

where:

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

and for cohesive soils:

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

where:

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

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

The lateral resistance of the piles should be developed primarily from the passive resistance of the soil. The values of n_h (Terzaghi, 1955) and s_u to be incorporated into the calculations of the coefficient of horizontal subgrade reaction (k_h) within the native subsoils/fills to be utilized for the structural lateral analysis of the piles (with and without CSP liners) at this site are summarized below.

Foundation Element (Relevant Boreholes)	CSP Liner Options	Soil Unit	Elevation (m)	n_h (kPa/m)	s_u (kPa)
South Abutment (P1 and P7)	With CSP Liners[1]	Loose Sand within CSP	245.0 to 242.0	1,300	-
		Soft to Firm Silty Clay to Clay	242.0 to 241.5	-	22
	Without CSP Liners[2]	Soft to Firm Silty Clay to Clay	243.0 to 241.5	-	22
	With or Without CSP Liners	Loose to Compact Sandy Silt to Sand and Silt (Till)	241.5 to 238.9	1,300	-
		Hard Sandy Clayey Silt to Clayey Silt (Till)	238.9 to 234.0	-	200
North Abutment (P2 and P8/8A)	With CSP Liners[1]	Loose Sand within CSP	245.0 to 242.0	1,300	-
		Soft to Stiff Silty Clay to Clay	242.0 to 240.9	-	22
	Without CSP Liners[1]	Soft to Stiff Sandy Silty Clay to Clay	243.0 to 240.9	-	22
	With or Without CSP Liners	Loose to Compact Sandy Silt to Sand and Silt (Till)	240.9 to 237.0	1,300	-
		Hard Sandy Clayey Silt to Clayey Silt (Till)	237.0 to 233.5	-	200

1. Base of pile cap at Elevation 245.0 m
2. For the options without the CSP liner, the soil information is provided from the underside of the tremie concrete plug (see section 6.7.1).

For a single HP310X110 or HP310X132 extending to the design tip elevations provided in Section 6.4.2.1, 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	90	30
HP310X132	105	35

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 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.

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

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

- Select, free draining granular fill meeting the specifications of OPSS.PROV 1010 (Aggregates) Granular ‘A’ or Granular ‘B’ Type II, but with less than 5 per cent passing the No. 200 sieve, should be used as backfill behind the walls. Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Compaction (including type of equipment, target densities, etc.) should be carried out in accordance with SP 105S21 (Compacting). Other aspects of the granular backfill requirements with respect to sub-drains and frost taper should be in accordance with OPSD 3101.150 (Walls, Abutment, Backfill, Minimum Granular Requirement) or OPSD 3121.150 (Walls, Retaining, Backfill, Minimum Granular Requirement), as applicable.

- For structures that are not comprised of integral or semi-integral abutments, rock fill may be used as backfill behind the walls and the material should meet the specification as outlined in the Northern Region Directive (2002) for backfill of structures adjacent to rock embankments. Other aspects of rock backfill requirements should be in accordance with OPSD 3101.200 (Walls, Abutment, Backfill, Rock).
- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the wall stem, in accordance with CHBDC Section 6.9.3 and Figure 6.6. Other surcharge loadings should be accounted for in the design as required.
- For restrained walls, granular fill should be placed in a zone with the width equal to at least 2.6 m behind the back of the wall (in accordance with Figure C6.20 (a) of the Commentary to the CHBDC). For unrestrained walls, granular fill should be placed within the wedge shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical (1.5H:1V) extending up and back from the rear face of the footing (in accordance with Figure C6.20(b) of the Commentary to the CHBDC). The pressures are based on the proposed embankment fill material and the following parameters (unfactored) may be used:

Fill Type	Unit Weight	Coefficients of Static Lateral Earth Pressure	
		At-Rest, K_o	Active, K_a
Granular ‘A’	22 kN/m^3	0.43	0.27
Granular ‘B’ Type II	21 kN/m^3	0.43	0.27

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

Seismic (earthquake) loading must also be taken into account in the design in accordance with Section 4.6 of the CHBDC. In this regard, the following should be included in the assessment of lateral earth pressures:

- Seismic loading will result in increased lateral earth pressures acting on the abutment stem and retaining walls. The walls should be designed to withstand the combined lateral loading for the appropriate static pressure conditions given above, plus the earthquake-induced dynamic earth pressure. According to the Figure A3.1.6 of the CHBDC, the site specific peak horizontal ground acceleration PHA for Hearst is 0.04 g (for a probability of exceedance of 10 per cent in 50 years). Based on Table A3.1.1 of the CHBDC, the zonal acceleration ratio for Hearst is 0.00.
- Based on the above, this site is located in Seismic Performance Zone 1 in accordance with Table 4.1 of the CHBDC. Further, we understand that there are no bridges that are classified as a Lifeline bridge in Ontario

(MTO, 2011). In accordance with Section 4.4.5 (and Table 4.2 in Section 4.4.5.3.1) of the CHBDC, no seismic analysis is required for single-span bridges or structures located in Seismic Performance Zone 1.

6.6 Approach Embankments

The replacement bridge will be constructed to the west of the existing structure with new approach embankments to be constructed essentially as a 4 m to 5 m westward widening of the existing embankments. The grade of the new embankments will be essentially the same as the current highway grade and up to about 3.3 m above the existing ground surface in the area west of the existing west toe of slope.

Since consideration is being given to realigning the creek on the northwest side of the embankment, we understand that it is desirable to have embankment slopes constructed as steep as possible. Therefore, we recommend that the approach embankments be constructed of rock fill having side slopes of 1.25H:1V. The embankments could also be constructed of granular fill having side slopes of 2H:1V, for locations not impacted by the creek (i.e. the south approach).

The analyses assume that the approach embankments will be constructed of rock fill and that the peat, where encountered, will be removed from below the footprint of the embankments. The geometry of the proposed approach embankments, existing ground surface and existing/re-aligned creek bed included in the analyses are based on the information from the GA drawing and cross-sections provided by LEA. The piezometric conditions required in the stability analyses are based on the low groundwater level at Elevation 244.8 m, which corresponds to creek level shown on the GA drawing, and the stability was also checked for a high water level at Elevation 247.6 m, which corresponds to the creek level measured during the spring freshet. For the settlement analyses, the piezometric conditions were based on the stabilized groundwater level at Elevation 246.5 m (measured in in the piezometers on December, 2012).

6.6.1 Approach Embankment Stability

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

6.6.1.1 Methodology

Limit equilibrium slope stability analyses were performed using the commercially available program GeoStudio 2007 (Version 7.19), produced by Geo-Slope International Ltd., employing the Morgenstern-Price method of analysis. For all analyses, the Factor of Safety of numerous potential failure surfaces was computed in order to establish the minimum Factor of Safety (FoS). The FoS is defined as the ratio of the forces tending to resist failure to the driving forces tending to cause failure. A target minimum FoS of 1.3 is normally adopted for the design of embankment slopes under static conditions at the end of construction. This FoS is considered adequate for the embankments at this site considering the design requirements and the field data available. The stability analyses were performed to check that the target minimum FoS was achieved for the design embankment height and geometries. In general, circular slip surfaces were analysed in the design.

6.6.1.2 Parameter Selection

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

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

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

where:

$s_{u(mob)}$ = average mobilized undrained shear strength (kPa)
$s_{u(FV)}$ = undrained shear strength from field vane test (kPa)
μ = Bjerrum's correction factor based on Plasticity Index

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

Soil Deposit	Bulk Unit Weight (kN/m^3)	Effective Friction Angle (°)	Undrained Shear Strength (kPa)
New Rock Fill	19	40	-
New Granular 'B' Type II Fill	21	35	-
Existing Granular Fill	20	32	-
Silty Peat to Peat	12	27	1
Silty Clay to Clay (soft to firm)	16	-	22
Sand to Sand and Gravel (compact)	19	32	-
Sandy Silt to Sand and Silt Till (very loose to compact)	19	30	-
Sandy Clayey Silt to Clayey Silt Till (hard)	21	-	200

6.6.1.3 Results of Analysis

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

6.6.2 Approach Embankment Settlement

Settlement of the approach embankments can be expected as a result of the loading from the up to 3.3 m high new embankment fill and from the loading associated with the sub-excavation/replacement of the up to 2.3 m thick deposit of peat on the compressible foundation soils at this site, especially in the area of the proposed embankment widening to the west. Settlement of the cohesionless deposits is expected to occur during or shortly after construction. Time-dependent consolidation settlement of the cohesive deposit along the west side of the approach embankments will occur but is also expected to primarily occur during construction. In addition, settlement of the new embankment fill will also occur.

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

6.6.2.1 Methodology

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

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

6.6.2.2 Rock Fill Settlement

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

Short-Term Rock Fill Settlement

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

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

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

Long-Term Rock Fill Settlement

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

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

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

6.6.2.3 Settlement Criteria

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

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

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

6.6.2.4 Parameter Selection

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

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

Soil Type	Location	Thickness (m)	γ (kN/m^3)	E (MPa)
New Rock Fill	North Approach	3.1 to 4.7	19	-
	South Approach	1.9 to 2.7		
Silty Clay to Clay (soft to firm)	North Approach	1.8 to 4.0	16	see below
	South Approach	3.8 to 5.0		
Sand to Sand and Gravel (compact)	North Approach	0.0 to 1.0	19	10
	South Approach	0.0 to 1.3		
Sandy Silt to Sand and Silt (Till) (very loose to compact)	North Approach	2.5 to 3.9	19	5
	South Approach	1.6 to 5.2		
Sandy Clayey Silt to Clayey Silt (Till) (hard)	North Approach	0.0 to 6.6	21	75
	South Approach	0.0 to 7.2		

n/a Indicates the deposit was not encountered.

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

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

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

The consolidation settlement of the silty clay to clay 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). A summary plot of the engineering parameters for the cohesive deposits is shown on Figure 3.

The coefficient of consolidation, c_v (cm^2/s), based on the results of the consolidation test, is estimated to be 1.5x10^{-3} cm^2/s within the anticipated stress range imposed by the new approach embankments. However, as the silty clay to clay deposit is over-consolidated, the laboratory test result is considered conservatively low and therefore an estimated c_v value of 5.5x10^{-3} cm^2/s cm^2/s, is considered to be more appropriate based on the Unified Facilities Criteria (U.S. Navy, NAVFAC 1986) correlation with liquid limit, and has been used for design.

6.6.2.5 Results of Analysis

A summary of the results of the settlement analysis at the abutments and the approaches is presented below.

Critical Section	Relevant Borehole	Estimated Settlements (mm)						
		Cohesionless Deposits	Cohesive Deposits	Rock Fill*		Total	Post-Construction	Post-Construction after 2 month paving delay
		Immediate	Primary	Short Term	Long Term			
South Approach	P6	30	50	5	0	85	55	10
South Abutment	P7	10	35	5	0	50	40	15
North Approach	P3	35	45	20	5	105	70	25
North Abutment	P2	55	80	20	5	160	105	25

*Assumes that granular fill will be used immediately behind the abutments as backfill.

Based on the c_v value given in Section 6.6.2.4, it is estimated that about 90 per cent of the primary consolidation settlement will be completed in about 2 months.

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

Since the post-construction settlement criterion is exceeded, settlement mitigation will be required. However, given the relatively short period of time estimated to complete 90 per cent of the primary consolidation settlement, it is anticipated that most of the primary settlement, and a portion of the short-term rock fill settlement, will occur during construction. Provided that paving can be delayed for at least 2 months after construction of the embankments, the post-construction settlement estimated as noted above can be reduced to meet the settlement criteria.

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

6.7 Construction Considerations

6.7.1 Subgrade Preparation and Embankment Construction

For the bridge approach embankments, removal of the peat is recommended prior to construction of the realigned embankment (see Section 6.7.2). Also, all softened/loosened soils should be stripped from below the approach embankment, prior to placement of new fill.

Fill for construction of the new embankments should consist of a Granular 'B' Type II meeting the specifications of OPSS.PROV 1010 (Aggregates) or rock fill. The embankment fill for the realigned Highway 583 should be placed and compacted in accordance with SP 105S21 (Compacting) and SP 206S03 (Earth, or Rock, Excavation and Grading), as applicable. Where new fill is to tie into existing fill along and beyond the approaches, the new fill should be "keyed-in" or benched into the existing fills, in accordance with OPSD 208.010 (Benching of Earth Slopes).

We understand that rock fill is proposed for the new embankment construction to limit the horizontal extent of the embankment widening at the northwest portion of the approach embankment where the existing Prune Creek will be re-aligned. We further understand that the upper 2 m of granular fill within the proposed north approach embankment will be replaced with rock fill to mitigate potential differential frost heaving between the existing granular embankment and the proposed new rock fill embankment. Rock fill for the proposed new embankment can be placed sub-aqueously, potentially avoiding extensive dewatering during construction. However, there will be settlement associated with rock fill as discussed in Section 6.6.2. Where granular fill other than Granular 'B' Type II is used, temporary shoring and dewatering (see Section 6.7.2) will be required to allow placement and compaction of the granular fill in dry conditions. Therefore, for the portion of under-water construction of the embankments, Granular 'B' Type II should be used.

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

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

The abutment front slopes and side slopes adjacent to the 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 pile caps in the design of the bridge foundations.

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

6.7.2 Excavation, Temporary Shoring and Groundwater Control

Along the west toe of the existing highway embankment, a 0.6 m to 2.3 m thick deposit of peat was encountered from ground surface. Peat was not encountered below the existing embankment. Given the compressible nature of the peat, we recommended the peat be removed from below the widened embankment footprint extending the full length of the embankment widening. Removal of the existing fill is not required beyond that required for pile cap construction and for protection of the embankment subgrade from frost protection (i.e. upper 2 m along the north approach embankment) as discussed in above Section 6.7.1.

The proposed excavation depths for the construction of the pile caps and peat removal are presented below, along with the depth of the excavations below the reported high groundwater level at Elevation 247.3 m.

Element	Base of Excavation		Depth Below High Water Level (m)
	Elevation (m)	Depth Below Existing Roadway (m)	
Pile Cap (South Abutment)[1]	243.0	6.1	4.3
Pile Cap (North Abutment)[1]	243.0	5.8	4.3
South Approach Embankment	247.2	1.9	0.1
North Approach Embankment	244.2	4.7	3.1

1. The base of the pile cap is assumed to be at Elevation 245.0 m as taken prom the GA Drawing provided in October 2013, and the excavation for the pile caps is based on the underside of the tremie concrete plug (see section 6.7.1).

If spread footing foundations are used to support the bridge, the excavations would be up to 11.8 m below the existing roadway surface and up to 10.3 m below the high water level.

If open-cut excavations are adopted, the excavations should be carried out in accordance with the guidelines in the latest version of the Occupational Health and Safety Act (OHSA) for Construction Activities. The existing fill, peat, silty clay to clay and the very loose to compact sandy silt to sand and silt till would be classified as Type 3 soil, according to the OHSA, while the hard sandy clayey silt to clayey silt till would be classified as a Type 2 material. Temporary excavations (i.e. those that are open for a relatively short time period) in Type 3 soils at this site should be made with side slopes no steeper than 2H:1V while in Type 2 soil, may be made with side slopes not steeper than 1H:1V. If sufficient space is not available to allow open-cut excavations, temporary shoring will be required along Highway 583. To allow for sub-aqueous fill placement, it is recommended that rock fill be used below the groundwater level.

Given the depths of the sub-excavations, the depths below the groundwater level and the proximity to the creek, temporary shoring and unwatering will be required for construction of the bridge foundations and to facilitate peat removal during embankment construction. Temporary shoring and dewatering for the bridge foundations 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 and/or piping methods of failure. The tremie concrete plug should be a minimum of 2.0 m thick and should have a minimum compressive strength of 1 MPa. A balanced head of water should be maintained on both sides of the cofferdam until the tremie concrete plug is place to prevent basal heave or piping. Water should only be pumped out of the excavation for construction of the footings or pile caps once the tremie plug is in place.

The temporary support to facilitate peat removal for construction of the approach embankments (i.e. within the 20 m of the abutments) could consist of either driven steel sheet piling or soldier piles and lagging 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).

Beyond the limits of the approach embankment (i.e. beyond about 20 m from the abutments), removal of the peat should be carried out as per OPSD 203.020 (Embankments Over Swamp, Existing Slope Excavated to 1H:1V). The width of the peat excavation should extend to a lateral distance from a line projected down from the crest of the widened embankment at the projected embankment side slope (1.25H:1V for rock fill and 2H:1V for granular fill) to the base of the sub-excavation. Excavations for this purpose should be in accordance with OPSS 902 (Excavating and Backfilling – Structures).

Removal of the peat for the for the first stage of construction should extend the full width of the cofferdam, as well as outside the cofferdam on the west side, as shown schematically in Section A-A' on Figure 4. Beyond the cofferdam, where temporary roadway protection will be in place, full removal of the peat is also required as shown schematically in Section B-B' on Figure 4. Beyond the temporary roadway protection (i.e. beyond 20 m from the abutments), peat removal should be carried out as per OPSD 203.020 as shown schematically in Section C-C' on Figure 4.

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 non-cohesive soils, the unfactored rectangular earth pressure distribution (p in kN/m^2; constant with depth), can be calculated as follows:

$$P = K_a(0.65\,\gamma\,H + q)$$

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	=	$\gamma H_T - 4\,m\,S_u$ at 0.25 H_T and from 0.25 H_T to H_T below ground surface

where	H_T	=	the total depth of the excavation (m)
	γ	=	soil unit weight (kN/m^3)
	q	=	surcharge for traffic and other loading (kN/m^2)
	m	=	0.4 if an extensive soft clay layer underlies the excavation 1.0 if more resistant layer is present at the excavation base
	S_u	=	undrained shear strength (kN/m^2).

Support to the temporary roadway protection could be in the form of struts and walers although bracing may not be required depending on the unsupported height of the excavation required for backfilling behind the cofferdam. If support to the wall is to be provided by anchors or rakers, the wall design should be based on a triangular earth pressure distribution using the design parameters given below. The raker/anchor support must be designed to accommodate the loads applied from earth and groundwater pressures and surcharge pressures from area, line or point loads as well as the impact of sloping ground behind the system. Passive toe restraint to the soldier piles may be determined using a triangular pressure distribution acting over an equivalent width equal to three times the pile socket diameter.

The unfactored triangular earth pressure distribution (p in kN/m^2; increasing with depth), can be calculated as follows:

	p	=	Ka (γ H + q)
where	Ka	=	active coefficient of earth pressure
	H	=	the depth of the excavation at any point (m)
	γ	=	soil unit weight (kN/m^3)
	q	=	surcharge for traffic and other loading (kN/m^2)

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 Rock Fill	0.22	0.36	4.6	40	19	-
New Granular Fill	0.27	0.43	3.7	35	21	-
Existing Granular Fill	0.31	0.47	3.3	32	21	-
Existing Clayey Silt Fill*	0.33	0.53	2.8	28	18	-
	1.0	1.0	1.0	-	18	22
Silty Peat to Peat	0.38	0.55	2.7	27	12	1
Silty Clay to Clay* (soft to stiff)	0.36	0.53	2.8	28	16	-
	1.0	1.0	1.0	-	16	22
Sand to Sand and Gravel (compact)	0.31	0.47	3.3	32	19	-
Sandy Silt to Sand and Silt Till (very loose to compact)	0.33	0.50	3.0	30	19	-
Sandy Clayey Silt to Clayey Silt Till (hard)	0.27	0.43	3.7	35	21	-
	1.0	1.0	1.0	-	21	200

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.

6.7.3 Obstructions

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

6.7.4 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) will reach this threshold level. Therefore, vibration monitoring is not required during construction at this site.

6.7.5 Existing Structure Monitoring

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

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

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

6.7.6 Analytical Testing for Construction Materials

The analytical test results on a sample of creek water are presented in Table C1 in Appendix C. 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.

The analytical test results on two soil samples from the test pit investigation are also presented in Appendix C. The suite of parameters is intended for others to determine a suitable soil disposal site for the material to be removed as part of the proposed creek re-alignment.

7.0 CLOSURE

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

Report Signature Page

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

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ASTM International

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

Commercial Software

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

LPile Plus (Version 5.0) by Ensoft Inc.

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

Ministry of Transportation, Ontario

Structural Manual. Provincial Highways Management Division, Highway Standards Branch, Bridge Office, April, 2008

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

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

Backfill of Structures Adjacent to Rock Embankment Approaches, Northern Region Directive, November, 2002.

Ministry of Transportation Ontario Special Provisions

SP 105S21	Amendment to OPSS 501, November 2010 – Water Requirements and Quality Control for Compaction – Method B.
SP 206S03	Earth Excavation, Grading; Rock Excavation, Grading
SP 511S01	Rip Rap; Rock Protection; Gravel Sheeting

Ministry of the Environment, Ontario

Soil, Ground Water and Sediment Standards for Use Under Part XV.1 of the Environmental Act, revised version April 15, 2011 (Amendment to Ontario Regulation 153/04)

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 3101.150	Walls Abutment, Backfill Minimum Granular Requirement
OPSD 3101.200	Walls Abutment, Backfill Rock
OPSD 3121.150	Walls Retaining, Backfill Minimum Granular Requirement

Ontario Provincial Standard Structural Drawings

SS103-11	Pile Driving Control, April 2008Ontario Provincial Standard Specifications
OPSS 501	Construction Specification for Compacting
OPSS 511	Construction Specification for Rip Rap, Rock Protection and Granular Sheeting
OPSS 539	Construction Specification for Temporary Protection Systems
OPSS 802	Construction Specification for Topsoil
OPSS 804	Construction Specification for Seed and Cover

OPSS 902	Construction Specification for Excavating and Backfilling-Structures
OPSS 903	Construction Specification for Deep Foundations
OPSS.PROV 1004	Material Specification for Aggregates – Miscellaneous
OPSS.PROV 1010	Material Specification for Aggregates – Base, Subbase, Select Subgrade and Backfill Material

Ontario Water Resources Act

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

Ontario Environmental Protection Act

Ontario Regulation 153/04	Environmental Protection Act, Records of Site Conditions –Part XV.1 of the Act
Ontario Regulation 558/00	Environmental Protection Act, Amendment to Ontario Regulation 347

Table 1: Comparison of Foundation Alternatives

Foundation Type	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Driven Steel H-Piles	1	■ Straightforward construction. ■ Higher axial resistance compared to spread footings. ■ Shallower excavation for pile cap compared to spread footing. ■ Allow for integral abutment design ■ Very little/negligible settlement expected	■ Requires excavation and dewatering for pile cap construction below groundwater level. ■ Potential for “hanging up” on cobbles and potential boulders within till deposits but likely easier to advance than pipe piles.	■ Relative costs lower than for caissons and spread footings. ■ Cost of shoring/ dewatering for pile cap and temporary roadway protection.	■ Potential risk for not reaching the design pile tip elevation due to the presence of cobbles and potential boulders – variable pile lengths.
Driven Steel Tube Piles	2	■ Straightforward construction. ■ Higher axial resistance compared to spread footings. ■ Shallower excavation for pile cap compared to spread footing. ■ Very little/negligible settlement expected	■ Requires excavation and dewatering for pile cap construction below groundwater level. ■ Displacement piles greater potential for “hanging up” or deflecting on cobbles and potential boulders within upper and lower till deposits. ■ Not acceptable by MTO for integral abutment design.	■ Relative costs lower than caissons and spread footings. ■ Cost of shoring/ dewatering for pile cap and temporary roadway protection.	■ Greater potential than the steel H-Piles for not reaching the design pile tip elevation due to the presence of cobbles and potential boulders - variable pile lengths ■ Greater potential than H-piles for deflection due to the presence of cobbles and boulders.
Caissons	3	■ Higher axial resistances compared to steel H-piles or tube piles. ■ Shallower excavation for caisson cap compared to spread footing or possible elimination of pile cap and associated excavation	■ Higher risk of problems associated with high groundwater conditions compared to piles. ■ Potential for difficulties penetrating the cobbles and potential boulders compared to piles. ■ Does not allow for integral abutment design.	■ Relative costs much higher than for steel H-piles or pipe piles.	■ Highest potential risk of difficulties reaching the required termination depth in the hard till deposits due to the presence of cobbles and potential boulders. ■ Potential for construction problems associated with high groundwater during caisson installation.

Table 1: Comparison of Foundation Alternatives

Foundation Type	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Spread Footings on Dense to Very Dense Soil	Not Feasible	■ Conventional construction.	■ Requires deep excavations and dewatering (cofferdam) adjacent to the creek to achieve sufficient geotechnical strength. ■ Low geotechnical axial resistances requires large footing.	■ Typically lower cost than deep foundations; however much increased cost of shoring/ dewatering for deeper excavation than for pile caps.	■ Potential difficulties advancing shoring/dewatering deeper into the dense to very dense till material containing cobbles and potential boulders. ■ Very large/deep excavation required.

CONT No.
WP No. 5484-09-01

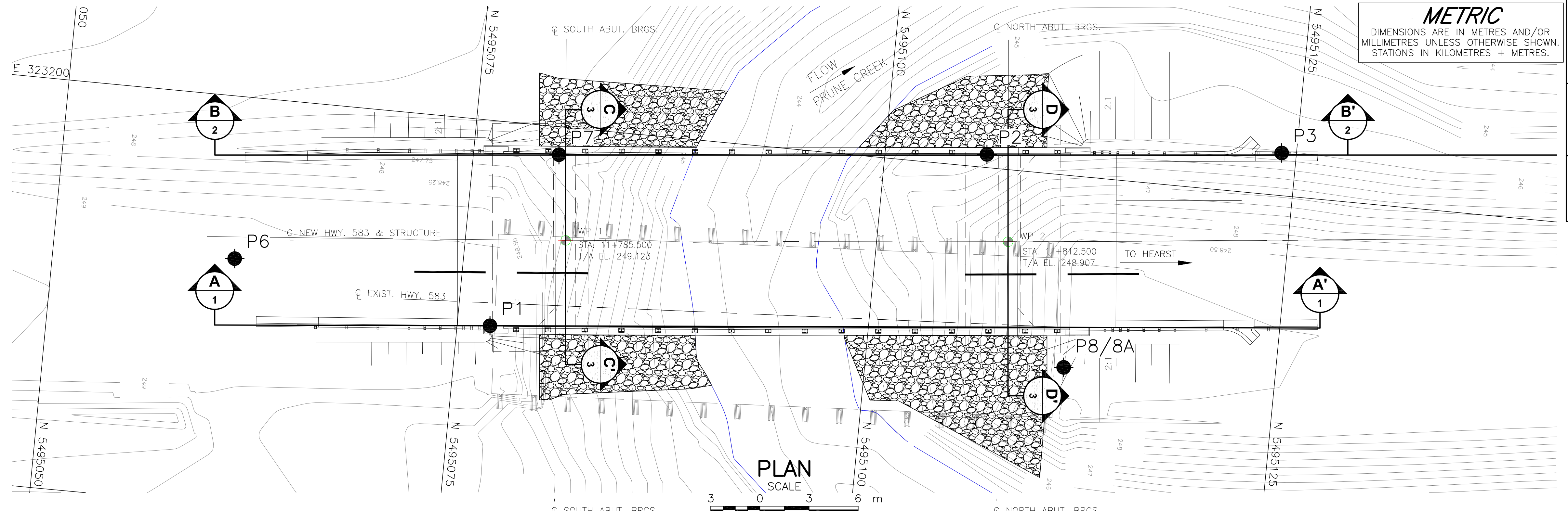
SHEET

HIGHWAY 583
PRUNE CREEK BRIDGE

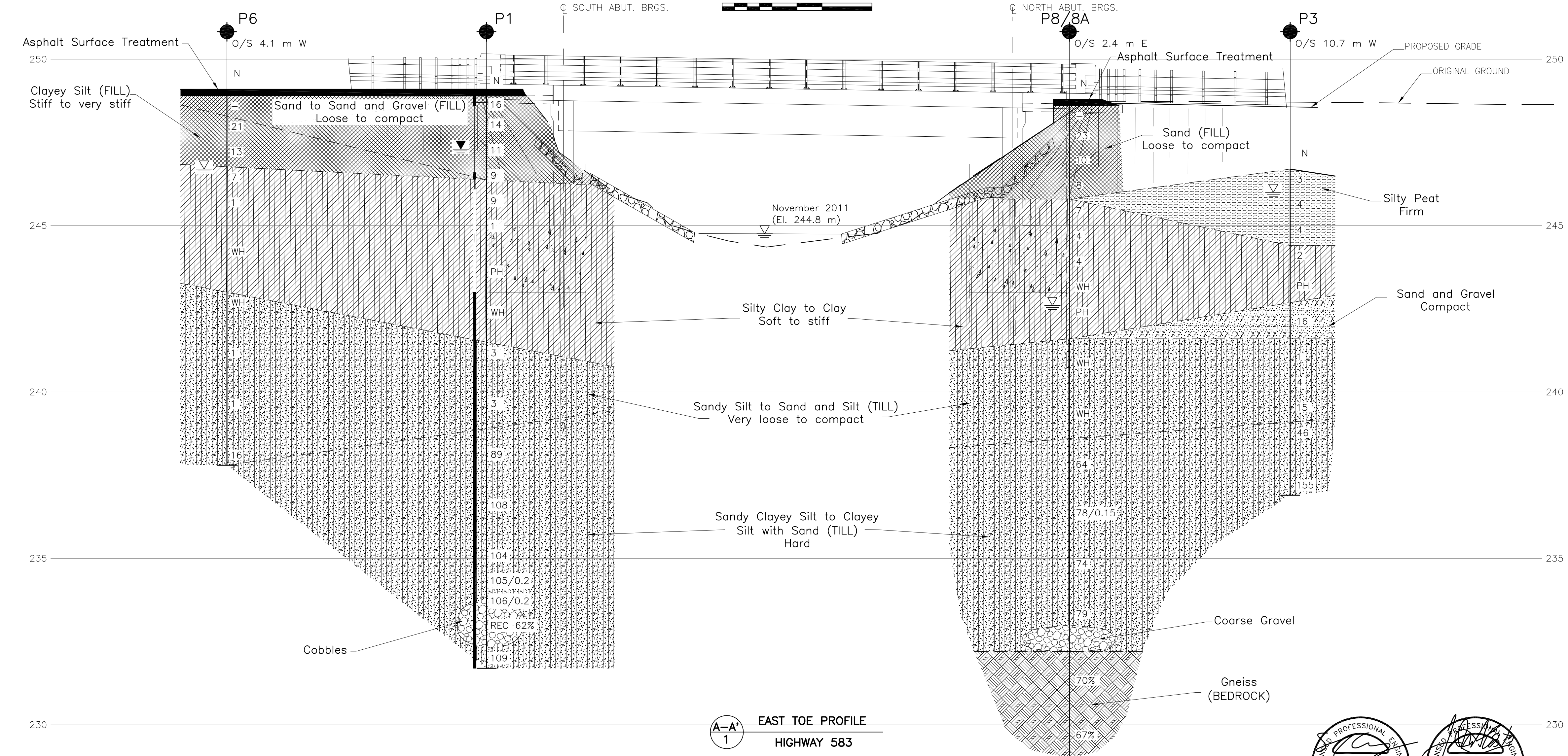
BOREHOLE LOCATIONS AND SOIL STRATA

Golder Associates Ltd.
SUDBURY, ONTARIO, CANADA

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

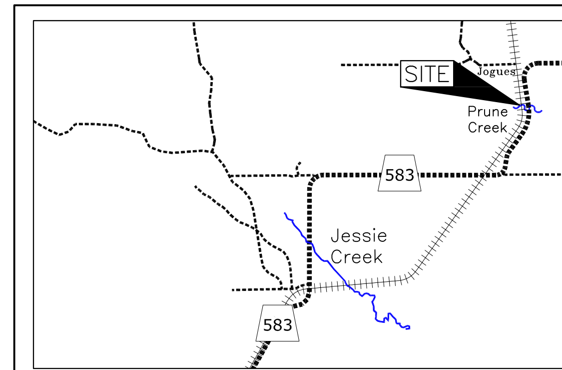


PLAN
SCALE
3 0 3 6 m



EAST TOE PROFILE
HIGHWAY 583
A-A'
1

HORIZONTAL SCALE
3 0 3 6 m
1.5 0 1.5 3 m
VERTICAL SCALE



KEY PLAN
SCALE
1.6 0 1.6 km

LEGEND

- Borehole
- Seal
- Piezometer
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- R Refusal
- 100% Rock Quality Designation (RQD)
- WL in piezometer, measured on June 5, 2013
- WL upon completion of drilling

BOREHOLE CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
P1	249.1	5495077.0	323212.5
P2	245.7	5495106.2	323199.2
P3	246.7	5495124.1	323197.4
P6	249.1	5495061.1	323209.9
P7	248.0	5495080.2	323201.7
P8/8A	248.8	5495112.1	323211.7

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, drawing file nos. 8960-Prune-S01.dwg, received October 22, 2013.

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

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

NO.	DATE	BY	REVISION

Geocres No. 42G-46

HWY. 583	PROJECT NO. 11-1191-0008		DIST.
SUBM'D. DAM	CHKD. SEMC	DATE: OCT 2013	SITE: 39W-046
DRAWN: JJL	CHKD.	APPD. JMAC	DWG. 1

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

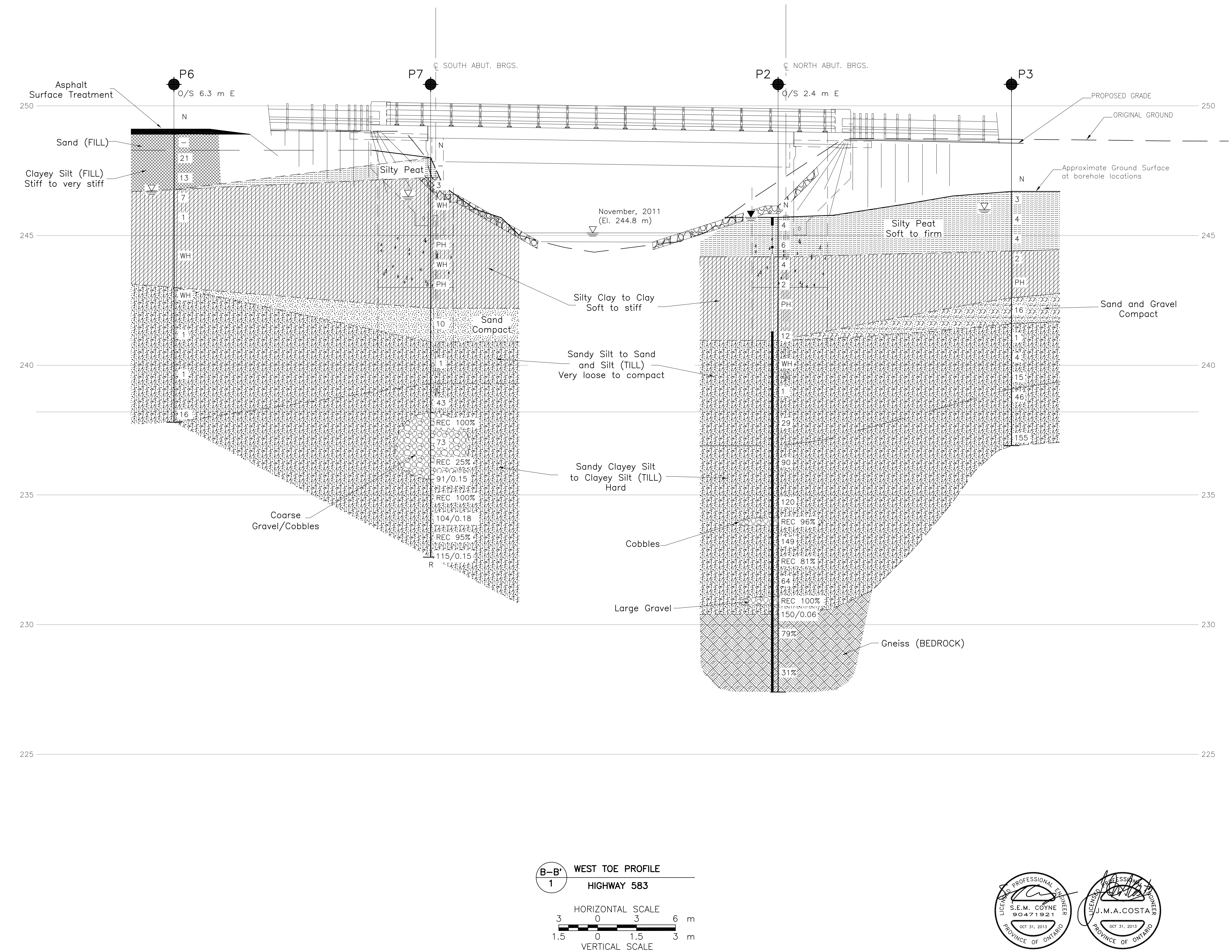
CONT No.
WP No. 5484-09-01

HIGHWAY 583
PRUNE CREEK BRIDGE

SOIL STRATA

SHEET

Golder Associates Ltd.
SUDBURY, ONTARIO, CANADA



LEGEND

●	Borehole
	Seal
	Piezometer
N	Standard Penetration Test Value
16	Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
R	Refusal
100%	Rock Quality Designation (RQD)
	WL in piezometer, measured on June 4, 2013
	WL upon completion of drilling

BOREHOLE CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
P2	245.7	5495106.2	323199.2
P3	246.7	5495124.1	323197.4
P6	249.1	5495061.1	323209.9
P7	248.0	5495080.2	323201.7

NOTES

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

NO.	DATE	BY	REVISION

Geocres No. 42G-46		
HWY. 583	PROJECT NO. 11-1191-0008	DIST.
SUBM'D. DAM	CHKD. SEMC	DATE: OCT 2013	SITE: 39W-046
DRAWN: JJL	CHKD.	APPD. JMAC	DWG. 2

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

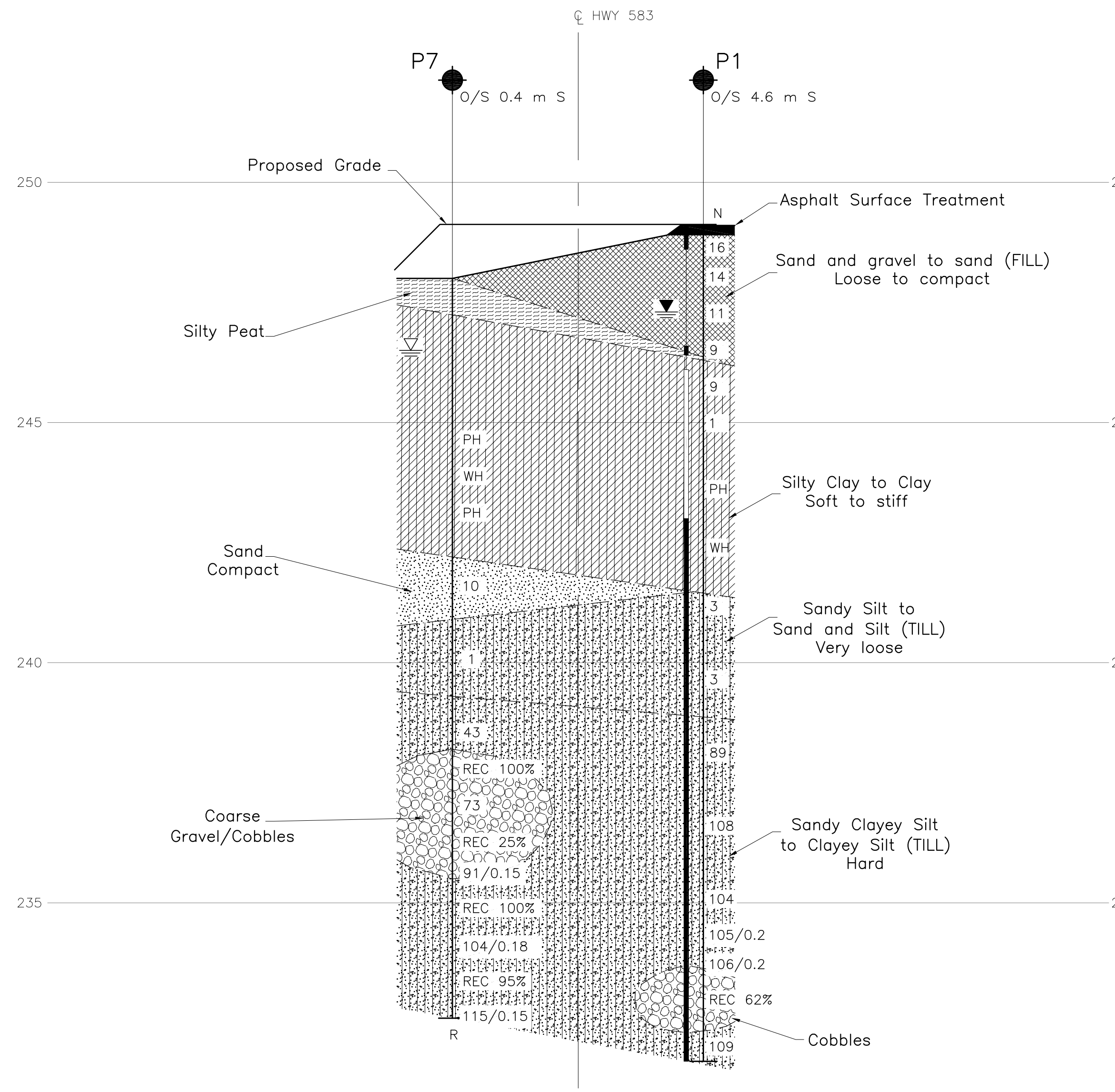
CONT No.
WP No. 5484-09-01

HIGHWAY 583
PRUNE CREEK BRIDGE

SOIL STRATA

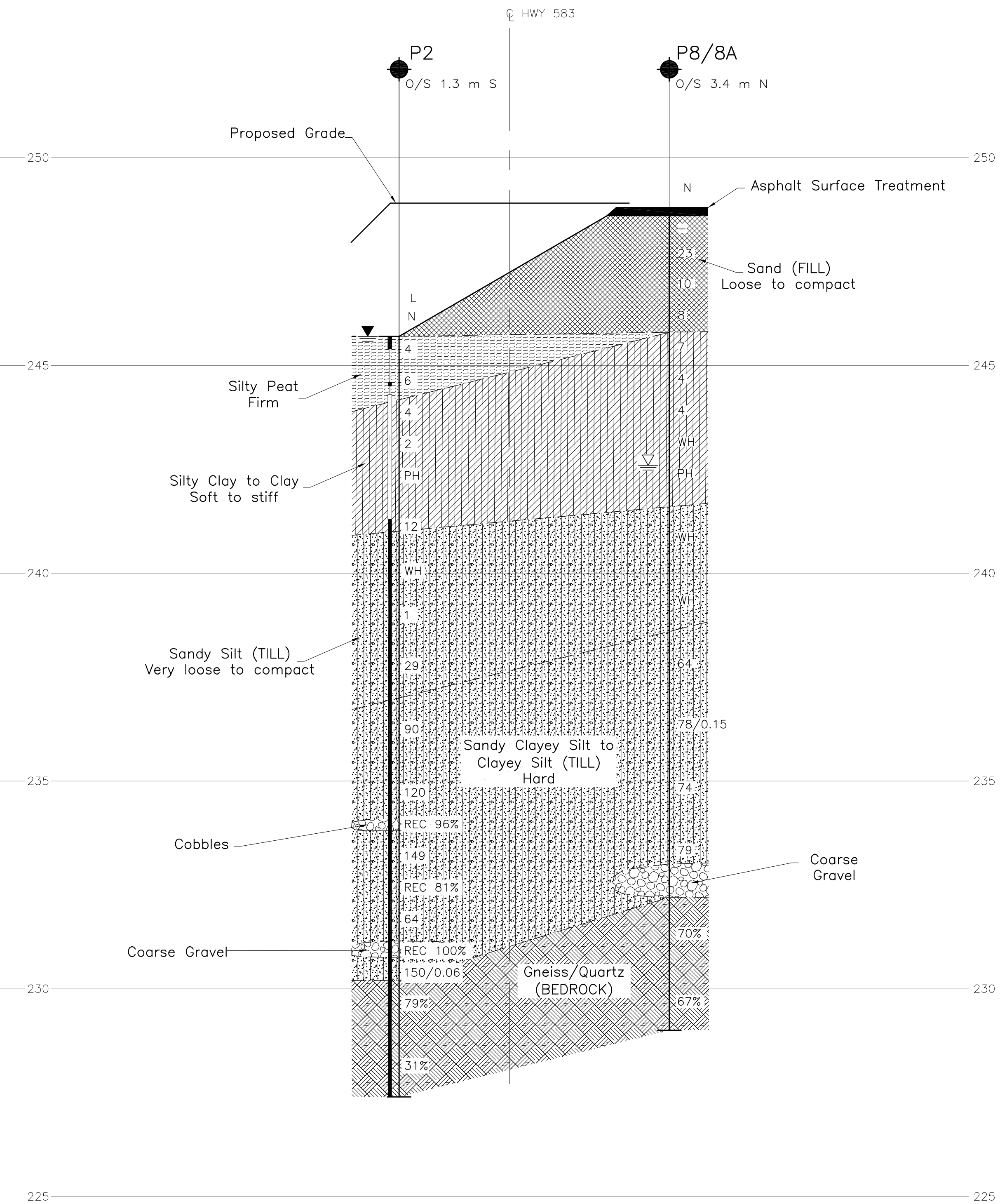
SHEET

Golder Associates Ltd.
SUDBURY, ONTARIO, CANADA



C-C' 1 SOUTH ABUTMENT
HIGHWAY 583

HORIZONTAL SCALE 3 0 3 6 m
1.5 0 1.5 3 m VERTICAL SCALE



D-D' 1 NORTH ABUTMENT
HIGHWAY 583

HORIZONTAL SCALE 3 0 3 6 m
1.5 0 1.5 3 m VERTICAL SCALE

LEGEND

- Borehole
- Seal
- Piezometer
- N — Standard Penetration Test Value
- 16 — Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- R — Refusal
- 100% — Rock Quality Designation (RQD)
- WL in piezometer, measured on June 4, 2013
- WL upon completion of drilling

BOREHOLE CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
P1	249.1	5495077.0	323212.5
P2	245.7	5495106.2	323199.2
P7	248.0	5495080.2	323201.7
P8/8A	248.8	5495112.1	323211.7

NOTES

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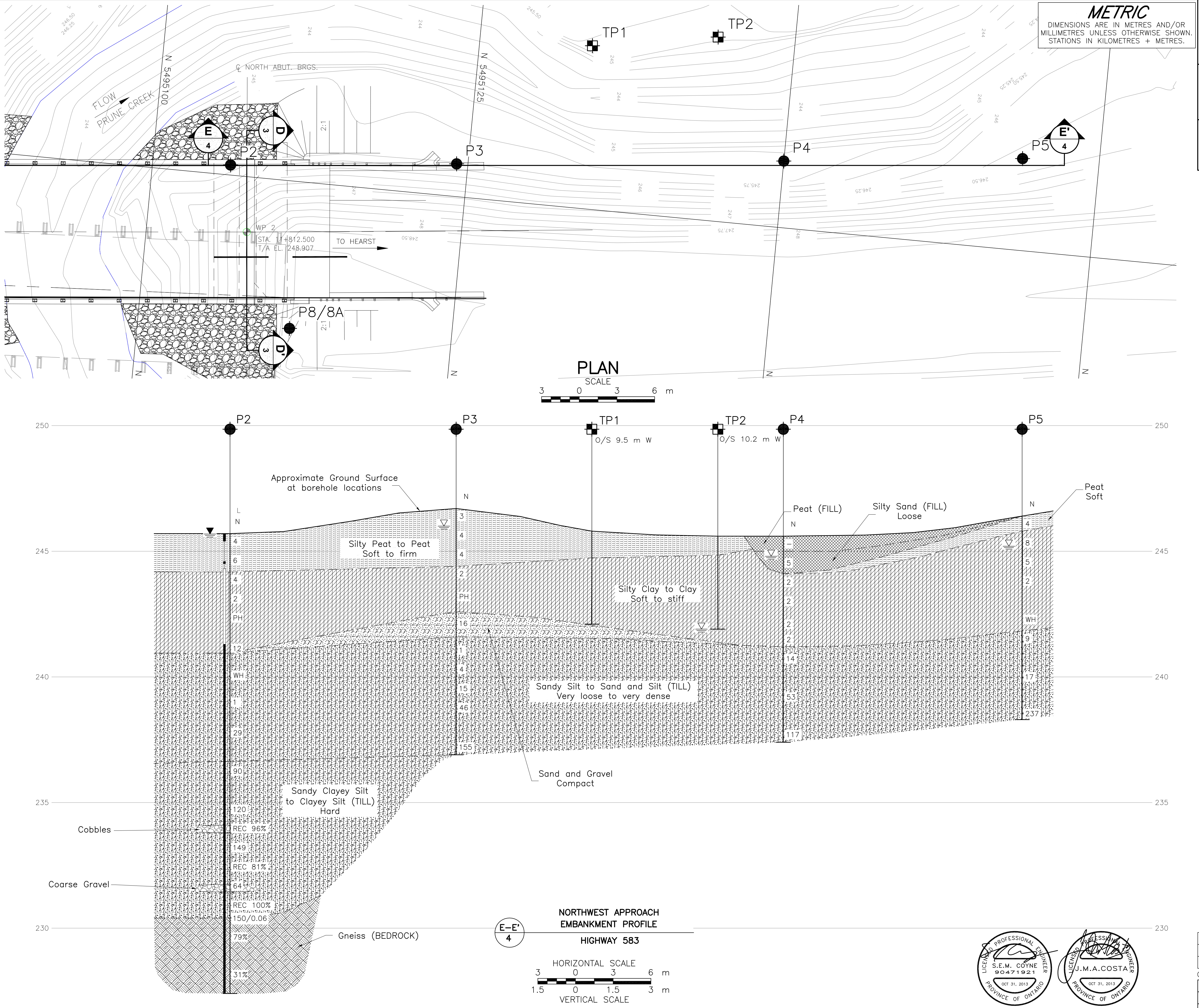
LICENSED PROFESSIONAL ENGINEER J.M.A.COSTA OCT 31, 2013 PROVINCE OF ONTARIO

NO.	DATE	BY	REVISION

Geocres No. 42G-46

HWY. 583	PROJECT NO. 11-1191-0008		DIST.
SUBM'D. DAM	CHKD. SEMC	DATE: OCT 2013	SITE: 39W-046
DRAWN: JJL	CHKD.	APPD. JMAC	DWG. 3

MINISTRY OF TRANSPORTATION, ONTARIO



CONT No.
WP No. 5484–09–01

HIGHWAY 583
PRUNE CREEK BRIDGE RETAINING WALL
BORIHOLE/TEST PIT LOCATIONS AND SOIL STRATA

SHEET

Golder Associates
Golder Associates Ltd.
SUDBURY, ONTARIO, CANADA

LEGEND

- Borehole
- Test Pit
- Seal
- Piezometer
- N — Standard Penetration Test Value
- 16 — Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- R — Refusal
- 100% — Rock Quality Designation (RQD)
- WL in piezometer, measured on December 6, 2012
- WL upon completion of drilling

BOREHOLE CO–ORDINATES

No.	ELEVATION	NORTHING	EASTING
P2	245.7	5495106.2	323199.2
P3	246.7	5495124.1	323197.4
P4	245.6	5495150.0	323194.7
P5	246.4	5495168.9	323192.7
P8/8A	248.8	5495112.1	323211.7
TP1	245.8	5495134.0	323187.0
TP2	245.6	5495143.9	323185.4

NOTES

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REFERENCE

Base plans provided in digital format by LEA, drawing file nos. 8960–Prune–S01.dwg, received October 22, 2013.

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

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

NO.	DATE	BY	REVISION

Geocres No. 42G–46

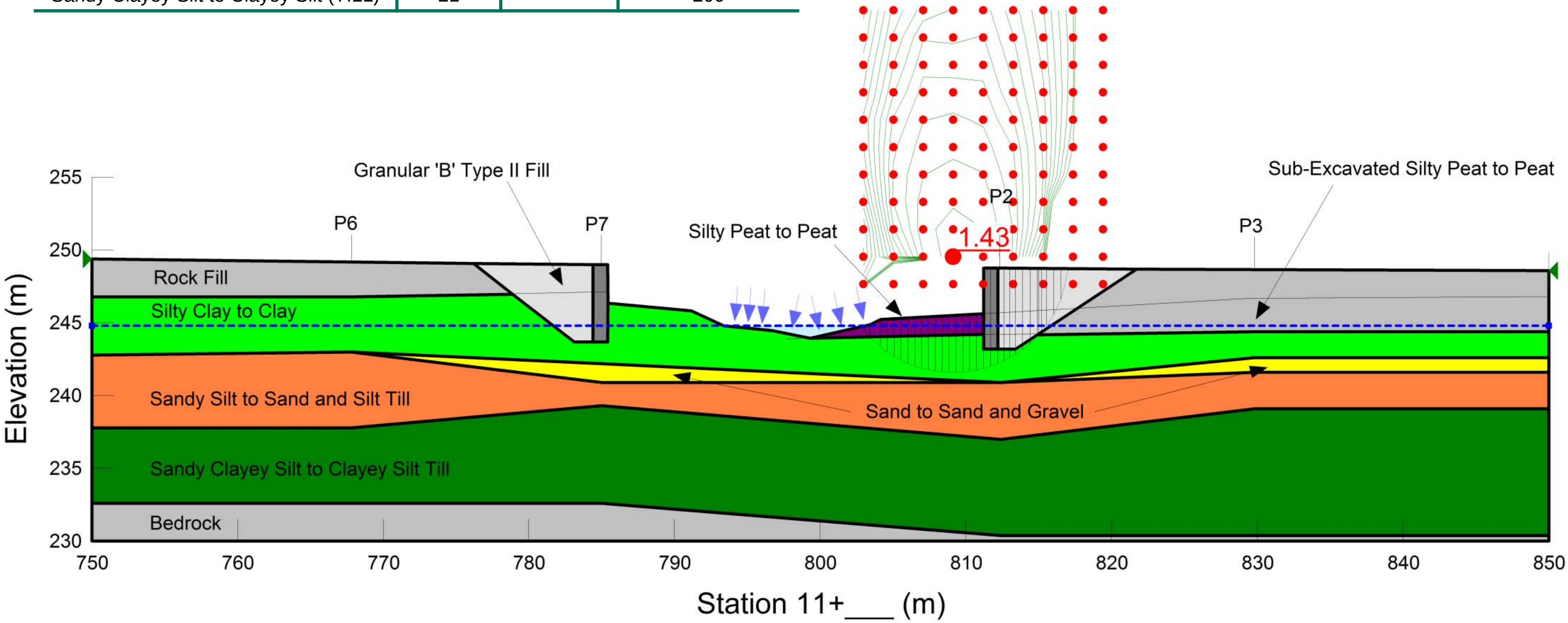
HWY. 583	PROJECT NO. 11–1191–0008		DIST.
SUBM'D. DAM	CHKD. SEMC	DATE: OCT 2013	SITE: 39W–046
DRAWN: JJL	CHKD.	APPD. JMAC	DWG. 4

PLOT DATE: November 1, 2013

Prune Creek Bridge – Highway 583
Stability Analysis (North Front Slope)

Figure 1

Material Name	Unit Weight (kN/m³)	Friction Angle (º)	Cohesion (kPa)
Rock Fill	19	40	0
New Granular B Type II Fill	21	35	
Silty Peat to Peat	12	27	1
Silty Clay to Clay	16	-	22
Sand to Sand and Gravel	19	32	-
Sandy Silt to Sand and Silt (TILL)	19	30	-
Sandy Clayey Silt to Clayey Silt (TILL)	21	-	200



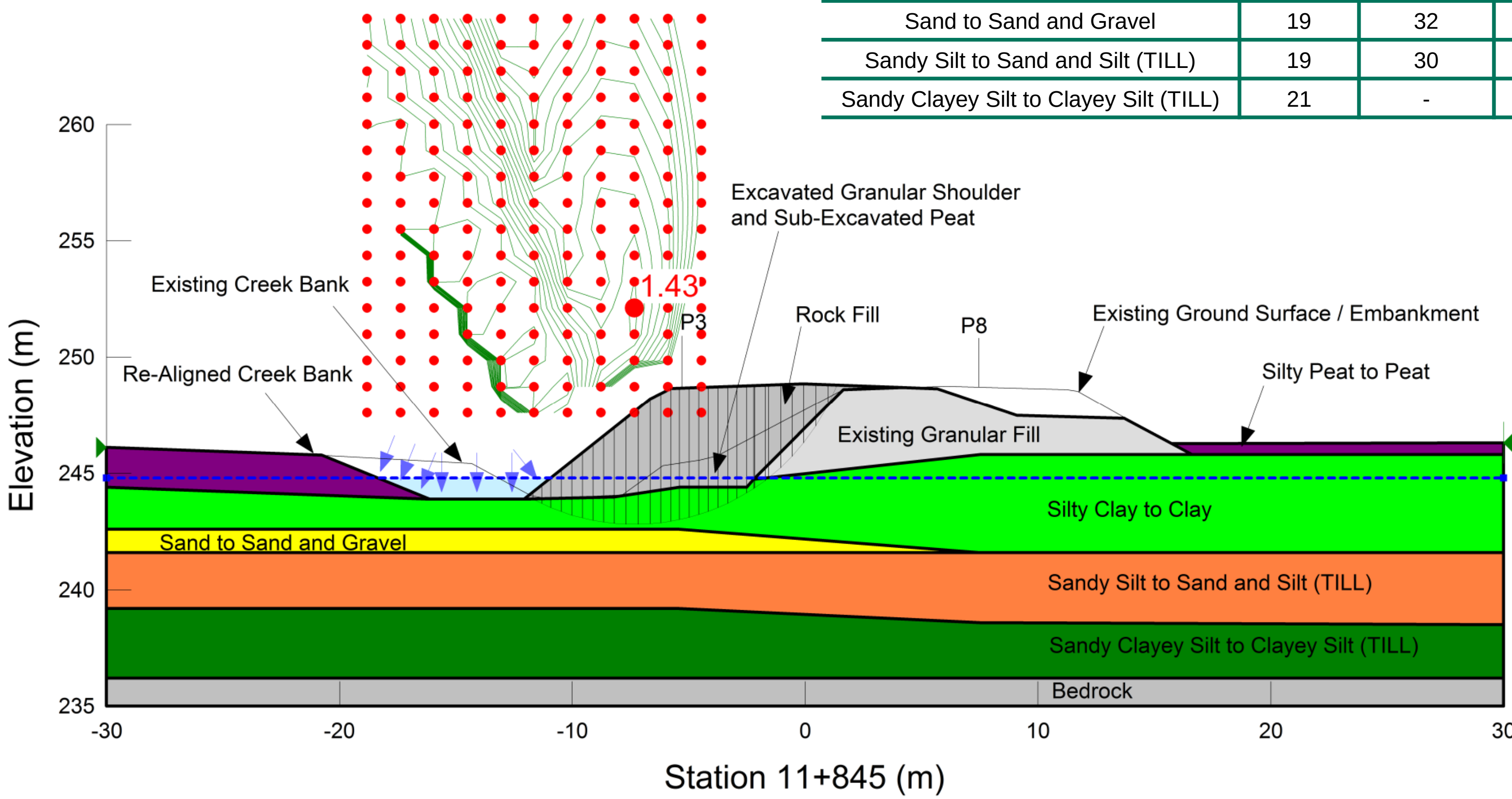
Date: October, 2013

Project No: 11-1191-0008

Analysis By: DAM **Reviewed By:** SEMC

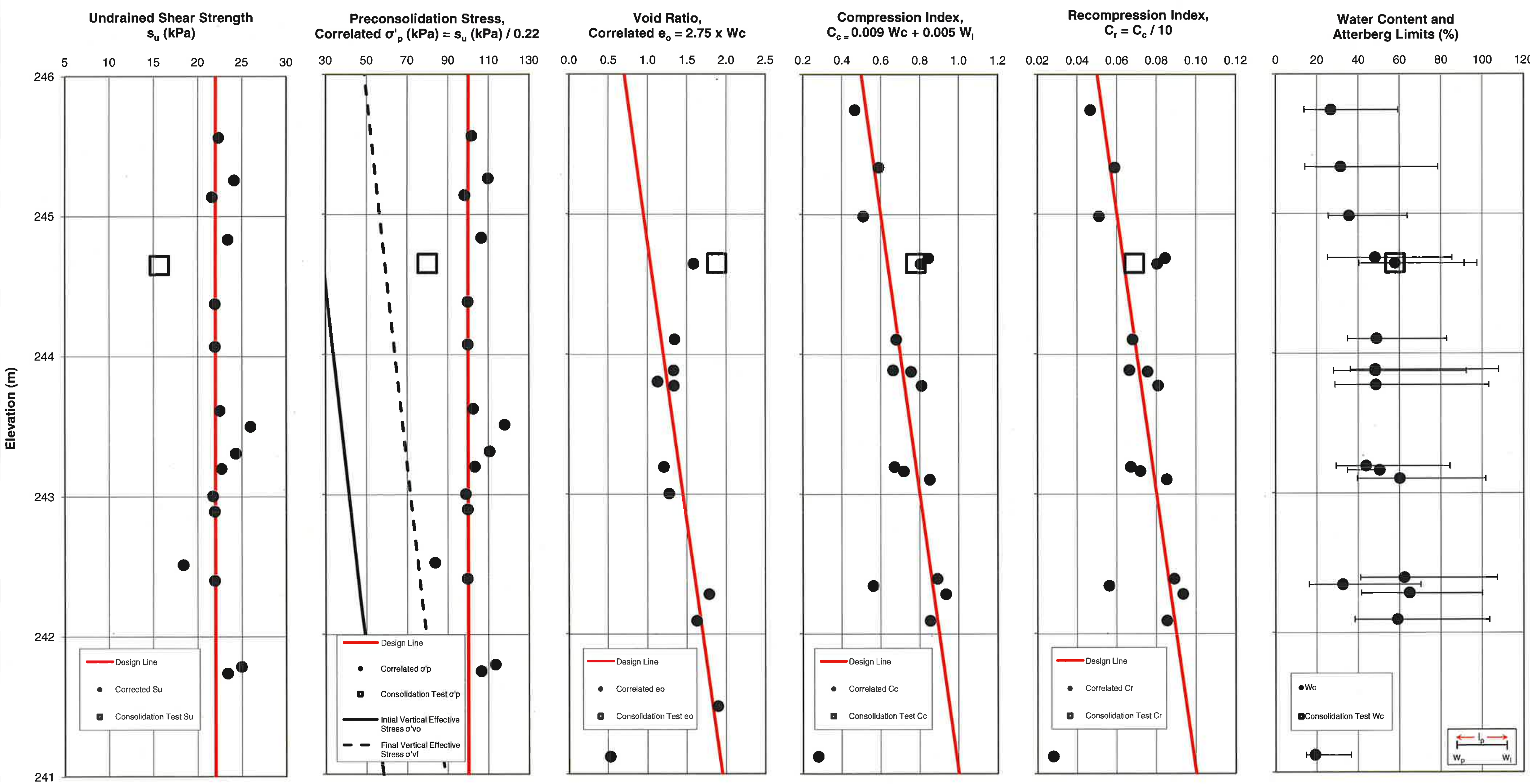
Golder Associates

Material Name	Unit Weight (kN/m³)	Friction Angle (º)	Cohesion (kPa)
Rock Fill	19	40	-
Existing Granular Fill	20	32	-
Silty Peat to Peat	12	27	1
Silty Clay to Clay	16	-	22
Sand to Sand and Gravel	19	32	-
Sandy Silt to Sand and Silt (TILL)	19	30	-
Sandy Clayey Silt to Clayey Silt (TILL)	21	-	200



SUMMARY PLOT OF ENGINEERING PARAMETERS FOR COHESIVE DEPOSIT
Prune Creek Bridge

FIGURE 3

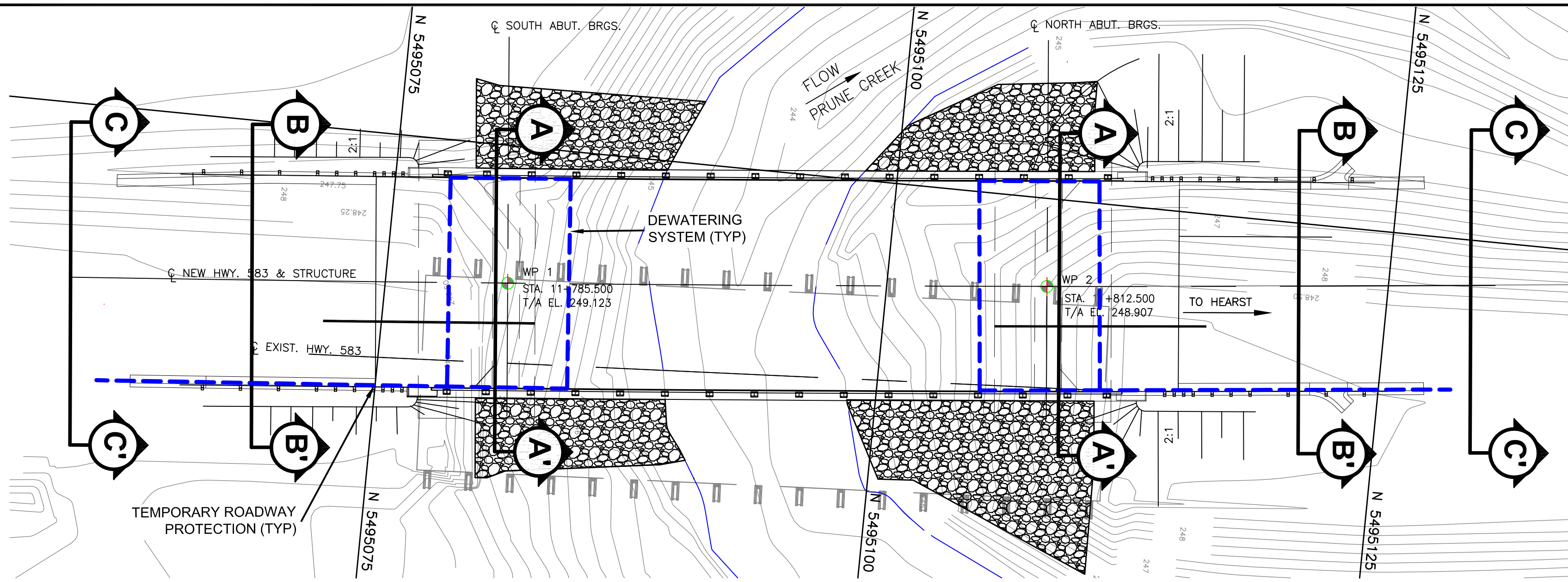


Date: October, 2013
Project No: 11-1191-0008

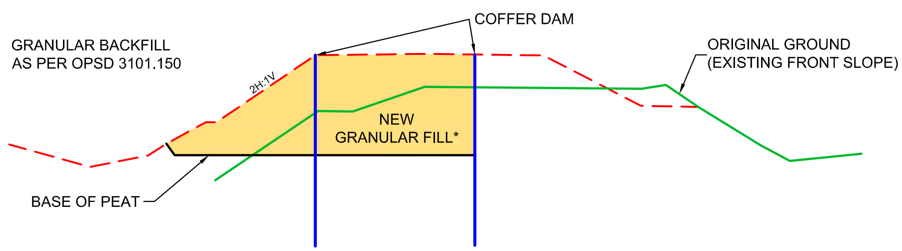
Prepared By: DAM
Checked By: SEMC

Golder Associates

PLOT DATE: November 1, 2013
FILENAME: \\golder.gds\GAL\Sudbury\CAD-GIS\CAD\Projects\2011\11-1191-0008 6 Bridges Hearst\1- Jessie Prune\DETAILED\Embank.Constr\1111910008_Prune Emb.dwg



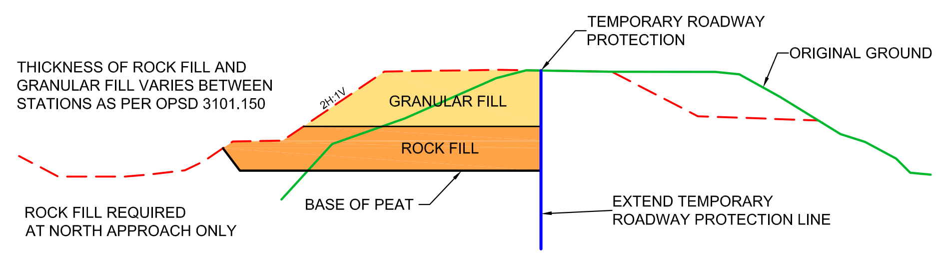
(A-A') WITHIN DEWATERING SYSTEM



* GRANULAR B TYPE II TO BE USED BELOW GROUNDWATER LEVEL

GRANULAR B TYPE I OR TYPE II TO BE USED ABOVE THE GROUNDWATER LEVEL

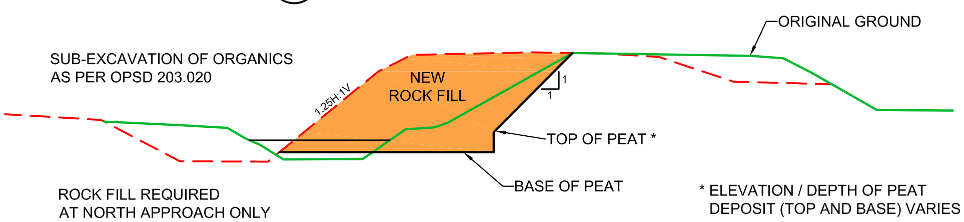
(B-B') WITHIN TEMPORARY ROADWAY PROTECTION AND ABUTMENT BACKFILL ZONE



REFERENCES:

MAPPING BASED ON "8960-Prune-S01.dwg" RECEIVED FROM LEA ON OCTOBER 22, 2013.

(C-C') BEYOND TEMPORARY ROADWAY PROTECTION



* ELEVATION / DEPTH OF PEAT DEPOSIT (TOP AND BASE) VARIES

N.T.S

PROJECT

HIGHWAY 583
PRUNE CREEK BRIDGE

TITLE

TYPICAL PEAT SUB-EXCAVATION AND EMBANKMENT CONSTRUCTION SCHEMATIC CROSS-SECTION

Golder Associates
Sudbury, Ontario, Canada

PROJECT No.		11-1191-0008
DESIGN		
CAD	TB	OCT 2013
CHECK	SEMC	OCT 2013
REVIEW	JMAC	OCT 2013

FILE No. 1111910008_Prune Emb.dwg
SCALE NTS REV.
FIGURE No. 4

APPENDIX A

Record of Boreholes and Drillholes

LIST OF SYMBOLS

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

I. GENERAL

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

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. in stress: $\Delta\,\sigma$
ε	linear strain
ε_v	volumetric strain
η	coefficient of viscosity
υ	Poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma - u$)
σ'_{vo}	initial effective overburden stress
σ_1, σ_2, σ_3	principal stress (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

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

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

(a) Index Properties (continued)

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

(b) Hydraulic Properties

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

(c) Consolidation (one-dimensional)

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

(d) Shear Strength

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

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

LIST OF ABBREVIATIONS

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

I. SAMPLE TYPE

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

II. PENETRATION RESISTANCE

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

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

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

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

III. SOIL DESCRIPTION

(a) Non-Cohesive (Cohesionless) Soils

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

(b) Cohesive Soils
Consistency

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

IV. SOIL TESTS

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

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

V. MINOR SOIL CONSTITUENTS

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

LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

WEATHERINGS STATE

Fresh: no visible sign of weathering

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

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

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

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

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

BEDDING THICKNESS

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

JOINT OR FOLIATION SPACING

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

GRAIN SIZE

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

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

CORE CONDITION

Total Core Recovery (TCR)

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

Solid Core Recovery (SCR)

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

Rock Quality Designation (RQD)

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

DISCONTINUITY DATA

Fracture Index

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

Dip with Respect to Core Axis

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

Description and Notes

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

Abbreviations

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

Golder Associates

RECORD OF BOREHOLE No P1

 METRIC

PROJECT 11-1191-0008

W.P. 5149-06-00 | LOCATION N 5495077.0; E 323212.5 | ORIGINATED BY ID

DIST ___ HWY 583 | BOREHOLE TYPE 108mm ID Continuous Flight Hollow Stem Augers, NW Casing, NQ Coring | COMPILED BY DM

DATUM Geodetic | DATE March 22, 2012 | CHECKED BY SEMC

ELEV / DEPTH (m)	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS (GR SA SI CL)	UNIT WEIGHT
249.1	GROUND SURFACE					
0.0	Asphalt Surface Treatment (150 mm)					
0.2	Sand to sand and gravel, trace to some silt (FILL) Loose to compact Brown Moist to wet	1	SS	16		
		2	SS	14		
		3	SS	11		
		4	SS	9		
246.4 / 2.7	SILTY CLAY, trace sand, layered above 3.7 m depth Soft to stiff Brown to grey below 4.3 m depth Wet	5	SS	9		
		6	SS	1		
		7	TO	PH		
		8	SS	WH	0 2 20 78	
241.5 / 7.6	SAND and SILT, trace to some gravel, trace to some clay (TILL) Very loose Grey Wet	9	SS	3	4 30 58 8	NP
		10	SS	3		
238.9 / 10.2	CLAYEY SILT, with sand, trace to some gravel (TILL) Hard Grey Moist Swithced to NW casing at 10.7 m depth. Intermittent grinding of casing below 10.7 m depth.	11	SS	89		
		12	SS	108	12 34 37 17	
		13	SS	104		
		14	SS	105/0.2		

Field vane sensitivity values (shear strength plot): 3, 3, 3, 2.

Elevation scale: 249, 248, 247, 246, 245, 244, 243, 242, 241, 240, 239, 238, 237, 236, 235

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

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

UNIT WEIGHT γ kN/m³

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

Continued Next Page

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

RECORD OF BOREHOLE No P1

 METRIC

PROJECT 11-1191-0008
W.P. 5149-06-00
LOCATION N 5495077.0; E 323212.5
ORIGINATED BY ID
DIST
HWY 583
BOREHOLE TYPE 108mm ID Continuous Flight Hollow Stem Augers, NW Casing, NQ Coring
COMPILED BY DM
DATUM Geodetic
DATE March 22, 2012
CHECKED BY SEMC

ELEV DEPTH	DESCRIPTION (SOIL PROFILE)	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	--- CONTINUED FROM PREVIOUS PAGE ---										
	CLAYEY SILT, with sand, trace to some gravel (TILL) Hard Grey Moist		15	SS	106/0.2		234				
	Spoon bouncing at 15.0 m depth.		1	RC	REC 62%		233				
	Casing refusal at 15.2 m depth. Casing advance noted harder and softer layers below 15.2 m depth.										
231.7 17.4	Cobbles encountered between 15.4 m and 16.8 m depth as follows:		16	SS	109		232		○		

Depth (m)	Thickness (mm)	Type
15.4	210	Granite
16.0	275	Granite/gneiss

END OF BOREHOLE

Note:

1. Water level at a depth of 2.3 m below ground surface (Elev. 246.8 m) upon completion of drilling.

2. Water level in piezometer at a depth of 1.8 m below ground surface (Elev. 247.3 m) on April 20, 2012.

3. Water level in piezometer at a depth of 2.4 m below ground surface (Elev. 246.7 m) on December 6, 2012.

4. Water level in piezometer at a depth of 1.8 m below ground surface (Elev. 247.3 m) on June 5, 2013.

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

RECORD OF BOREHOLE No P2

 METRIC

PROJECT 11-1191-0008

W.P. 5149-06-00 LOCATION N 5495106.2; E 323199.2 ORIGINATED BY ID

DIST HWY 583 BOREHOLE TYPE NW Casing, Wash Boring, NQ Coring COMPILED BY DM

DATUM Geodetic DATE April 19 and 20, 2012 CHECKED BY SEMC

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	SHEAR STRENGTH kPa	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
245.7	GROUND SURFACE											
0.0	Silty PEAT (AMORPHOUS), some sand, containing wood fibres Firm Brown Wet	1	SS	4	245							
		2	SS	6								
244.2 1.5	Sandy SILTY CLAY to CLAY, trace sand, trace to some gravel, trace organics Soft to stiff Brown to dark grey Wet	3	SS	4	244				2	3	32	63
		4	SS	2	243			OC=2.3%				
		5	TO	PH	242	5 (field vane)						
		6	SS	12					16	22	48	14
240.9 4.8	Sandy SILT, trace to some gravel, trace to some clay (TILL) Very loose to compact Brown to grey Wet Sand and gravel layer 350 mm thick at 4.8 m depth. Clayey silt seam at 6.4 m depth. Becoming moist to dry below 7.2 m depth.	7	SS	WH	241, 240			NP	6	28	57	9
		8	SS	1	239							
		9	SS	29	238							
237.0 8.7	Sandy CLAYEY SILT, trace gravel (TILL) Hard Grey Moist Casing grinding between 9.1 m and 11.6 m depth. Casing refusal at 11.6 m depth. Cobbles 125 mm and 175 mm thick encountered at 11.6 m depth (granite). Three coarse gravel pieces 50 mm to 75 mm thick encountered at 14.3 m depth (granite/meta sediment).	10	SS	90	237, 236				6	25	47	22
		11	SS	120	235							
		1	RC	REC 96%	234							
		12	SS	149	233							
		2	RC	REC 81%	232							
		13	SS	64								
		3	RC	REC 100%	231							

Continued Next Page

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

Foundation Design

Golder Associates

RECORD OF BOREHOLE No P2

 METRIC

PROJECT 11-1191-0008
W.P. 5149-06-00
LOCATION N 5495106.2; E 323199.2
ORIGINATED BY ID
DIST
HWY 583
BOREHOLE TYPE NW Casing, Wash Boring, NQ Coring
COMPILED BY DM
DATUM Geodetic
DATE April 19 and 20, 2012
CHECKED BY SEMC

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%)
230.4		~~14~~	~~SS~~	~~150/0.0~~		
15.3	GNEISS (BEDROCK) Bedrock cored from 15.3 m depth to 18.3 m depth. For coring details see Record of Drillhole P2.	4	RC	REC 100%	230, 229	RQD = 79%
		5	RC	REC 100%	228	RQD = 31%
227.4 18.3	END OF BOREHOLE Note: 1. Water level at a depth of 0.4 m below ground surface (Elev. 245.3 m) upon completion of drilling. 2. Water level in piezometer at a depth of 0.2 m below ground surface (Elev. 245.5 m) on December 6, 2012. 3. Water level in piezometer at gound surface (Elev. 245.7 m) on June 4, 2013.					

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

SUD_MTO 003 11119100008D.GPJ GAL-MISS.GDT 22/10/13 DATA INPUT:

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

PROJECT: 11-1191-0008

RECORD OF DRILLHOLE: P2

SHEET 1 OF 1

LOCATION: N 5495106.2 ;E 323199.2

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: April 19 and 20, 2012

DRILL RIG: D-25

DRILLING CONTRACTOR: Walker Drilling Ltd.

DATUM: Geodetic

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

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	COLOUR % RETURN (FLUSH)	DISCONTINUITY DATA: TYPE AND SURFACE DESCRIPTION	NOTES WATER LEVELS INSTRUMENTATION
		REFER TO PREVIOUS PAGE		230.4				
	NW	GNEISS Fine grained Grey Slightly to moderately weathered		15.3			JNFORo	UCS=60 MPa
							JNFORo	
16	April 20, 2012 NQ Coring	Quartz vein between 16.1 m and 16.2 m depth. Mica-schistose zone between 16.4 and 16.6 m depth.			4	GREY 100	JNFORo	
							JNIRRo	UCS=77 MPa
							JNFORo JNFORo JNFORo JNIRRo	
17							JNIRRo	
					5	GREY 100	BR	
							BR	
18							JNIRRo JNIRRo	
				227.4			BR	
		END OF DRILLHOLE		18.3				
19								
20								
21								
22								
23								
24								
25								

DEPTH SCALE
1 : 50

Golder Associates

LOGGED: ID
CHECKED: SEMC

Golder Associates

RECORD OF BOREHOLE No P3

 METRIC

PROJECT	11-1191-0008		
W.P.	5149-06-00	LOCATION	N 5495124.1; E 323197.4
ORIGINATED BY	ID		
DIST		HWY	583
BOREHOLE TYPE	NW Casing, Wash Boring	COMPILED BY	DM
DATUM	Geodetic	DATE	April 20, 2012
CHECKED BY	SEMC		

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
246.7	GROUND SURFACE
0.0	Silty PEAT (Fibrous) containing wood Soft to firm Brown to black Wet
244.4 / 2.3	SILTY CLAY, trace sand, trace gravel, trace organics Firm Dark grey Wet
242.6 / 4.1	SAND and GRAVEL Compact Brown Wet
241.6 / 5.1	Sandy SILT to SAND and SILT, trace to some clay, trace gravel (TILL) Very loose to compact Brown Wet Moist and grey below 6.9 m depth
239.1 / 7.6	Sandy CLAYEY SILT, trace gravel (TILL) Hard Grey Moist
236.9 / 9.8	END OF BOREHOLE Note: 1. Water level at a depth of 0.7 m below ground surface (Elev. 246.0 m) upon completion of drilling.

SAMPLES

NUMBER	TYPE	"N" VALUES	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
1	SS	3					
2	SS	4					
3	SS	4					
4	SS	2	OC=1.7%	1	1	24	74
5	TO	PH					
6	SS	16					
7	SS	1					
8	SS	4	NP	2	32	58	8
9	SS	15					
10	SS	46		4	23	48	25
11	SS	155					

GROUND WATER CONDITIONS: ∇ (approx. Elev. 246.0)

ELEVATION SCALE: 246, 245, 244, 243, 242, 241, 240, 239, 238, 237

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

Field vane: + 6

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

WATER CONTENT (%): 20 40 60

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

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

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

Golder Associates

RECORD OF BOREHOLE No P4

 METRIC

PROJECT 11-1191-0008
W.P. 5149-06-00
LOCATION N 5495150.0; E 323194.7
ORIGINATED BY ID
DIST
HWY 583
BOREHOLE TYPE NW Casing, Wash Boring
COMPILED BY DM
DATUM Geodetic
DATE April 21, 2012
CHECKED BY SEMC

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE / SHEAR STRENGTH kPa	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
245.6	GROUND SURFACE												
0.0	Peat (Fibrous), some sand, some silt (FILL) Black / brown Moist	1	AS	-									
245.0 / 0.6	Silty sand, some gravel (FILL) Loose Brown	2	SS	5	▽	245							
244.1 / 1.5	CLAY, trace to some sand, trace gravel, trace to some organics Soft to firm Brown to dark grey Wet	3	SS	2		244			OC=8.5%	4	20	36	40
		4	SS	2		243							
							+ 5 (field vane)						
		5	SS	2		242				0	2	31	67
		6	SS	2									
241.2 / 4.4	SAND and SILT, trace to some gravel, trace clay (TILL) Compact to very dense Brown to grey Wet to moist	7	SS	14		241							
						240							
		8	SS	53		239				11	36	48	5
						238							
		9	SS	117									
237.4 / 8.2	END OF BOREHOLE												

Note:

1. Water level at a depth of 0.8 m below ground surface (Elev. 244.8 m) upon completion of drilling.

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100
SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100
PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L
WATER CONTENT (%) 20 40 60
REMARKS & GRAIN SIZE DISTRIBUTION (%)

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

Golder Associates

RECORD OF BOREHOLE No P5

 METRIC

PROJECT 11-1191-0008

W.P. 5149-06-00 LOCATION N 5495168.9; E 323192.7 ORIGINATED BY ID

DIST HWY 583 BOREHOLE TYPE NW Casing, Wash Boring COMPILED BY DM

DATUM Geodetic DATE April 21, 2012 CHECKED BY SEMC

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
246.4	GROUND SURFACE												
0.0	PEAT (Fibrous), some silt Soft Brown Moist	1	SS	4		246							
245.8 0.6	CLAY, trace sand, trace gravel Soft to firm Brown to grey Wet	2	SS	8	▽	245				0	4	47	49
		3	SS	5									
		4	SS	2		244							
						243	+ 5, + 6 (field vane)						
		5	SS	WH		242							
241.8 4.6	SAND and SILT trace to some gravel, trace clay (TILL) Loose to very dense Brown to grey Wet to moist	6	SS	9		241							
	Casing grinding below 6.1 m depth.	7	SS	17		240				32	34	31	3
						239							
		8	SS	237									
238.3 8.1	END OF BOREHOLE												

Note:

1. Water level at a depth of 1.2 m below ground surface (Elev. 245.2 m) upon completion of drilling.

Legend: DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100); SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 20, 40, 60.

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

Golder Associates

RECORD OF BOREHOLE No P6

 METRIC

PROJECT 11-1191-0008 | LOCATION N 5495061.1; E 323209.9 | ORIGINATED BY ID

W.P. 5149-06-00 | BOREHOLE TYPE 108mm ID Continuous Flight Hollow Stem Augers | COMPILED BY DM

DIST | HWY 583 | DATE December 6, 2012 | CHECKED BY SEMC

DATUM Geodetic

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	FIELD VANE SHEAR STRENGTH (kPa)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
249.1	GROUND SURFACE						
0.0	Asphalt Surface Treatment (200 mm)						
0.2	Sand, trace silt (FILL) Brown Frozen	1	AS	-			
248.3 / 0.8	Clayey silt, with sand, trace to some gravel, trace organics (FILL) Stiff to very stiff Brown Moist	2	SS	21			10 47 27 16
		3	SS	13			
246.8 / 2.3	SILTY CLAY Soft to firm Grey Wet	4	SS	7	▽		
		5	SS	1			
						+ (sensitivity 3); + (sensitivity 3)	
		6	SS	WH			
						+ (sensitivity 3); + (sensitivity 3)	
243.0 / 6.1	Sandy SILT, trace to some clay, trace to some gravel (TILL) Very loose to compact Grey Wet	7	SS	WH			2 22 63 13
		8	SS	1			
		9	SS	1			
		10	SS	16			8 27 53 12
237.8 / 11.3	END OF BOREHOLE						

Note:

1. Water level at a depth of 2.4 m below ground surface (Elev. 246.7 m) upon completion of drilling.

Legend: SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED. WATER CONTENT (%): PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L. UNIT WEIGHT γ kN/m³.

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

RECORD OF BOREHOLE No P7

 METRIC

PROJECT 11-1191-0008
W.P. 5149-06-00 LOCATION N 5495080.2; E 323201.7 ORIGINATED BY ID
DIST HWY 583 BOREHOLE TYPE 108mm ID Continuous Flight Hollow Stem Augers, NW Casing, NQ Coring COMPILED BY DM
DATUM Geodetic DATE December 8 and 9, 2012 CHECKED BY SEMC

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	DYNAMIC CONE / SHEAR STRENGTH	WATER CONTENT	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
248.0	GROUND SURFACE											
0.0	Silty PEAT (Amorphous) Black Frozen	1	AS	-								
247.2 / 0.8	SILTY CLAY to CLAY Soft to firm Brown / Grey Wet	2	SS	3	▽							
		3	SS	WH		Field vane 5, 4						
		4	TO	PH				15.9				
		5	SS	WH					0	2	38	60
		6	TO	PH		Field vane 2						
242.2 / 5.8	SAND, some silt, trace to some gravel Compact Brown / Grey Wet	7	SS	10					7	77	(16)	
240.9 / 7.1	Sandy SILT, trace to some clay, trace to some gravel (TILL) Very loose Grey Wet Swithed to NW Casing at 7.6 m depth.	8	SS	1								
239.3 / 8.7	Sandy CLAYEY SILT, trace to some gravel (TILL) Hard Moist Swithed to NW casing at 9.8 m depth. Gravel and cobbles encountered below 9.8 m as follows:	9	SS	43								
			RC	REC 100%								
		10	SS	73					3	23	50	24
			RC	REC 25%								
		11	SS	91/0.15								
			RC	REC 100%								
		12	SS	104/0.18					5	26	48	21
			RC	REC 95%								

Depth (m)	Thickness (mm)
9.8	180
10.3	120
11.6	65
11.9	40
12.5	50

Elevation scale: 247, 246, 245, 244, 243, 242, 241, 240, 239, 238, 237, 236, 235, 234

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED: 20 40 60 80 100
PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%): 20 40 60
REMARKS & GRAIN SIZE DISTRIBUTION (%)

Continued Next Page

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

Golder Associates

RECORD OF BOREHOLE No P7

 METRIC

PROJECT 11-1191-0008

W.P. 5149-06-00 | LOCATION N 5495080.2; E 323201.7 | ORIGINATED BY ID

DIST | HWY 583 | BOREHOLE TYPE 108mm ID Continuous Flight Hollow Stem Augers, NW Casing, NQ Coring | COMPILED BY DM

DATUM Geodetic | DATE December 8 and 9, 2012 | CHECKED BY SEMC

ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L — WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	--- CONTINUED FROM PREVIOUS PAGE ---										
232.6			13	SS	115/0.15						
15.4	END OF BOREHOLE SPOON REFUSAL (HAMMER BOUNCING) Note: 1. Water level at a depth of 1.5 m below ground surface (Elev. 246.5 m) upon completion of drilling.										

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

Golder Associates

RECORD OF BOREHOLE No P8/P8A

 METRIC

PROJECT	11-1191-0008		
W.P.	5149-06-00	LOCATION	N 5495112.1; E 323211.7
DIST		HWY	583
BOREHOLE TYPE	108mm ID Continuous Flight Hollow Stem Augers		
DATUM	Geodetic	DATE	December 12, 2012 and June 4, 2013
ORIGINATED BY	ID/EHS	COMPILED BY	DM
CHECKED BY	SEMC		

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
248.8	GROUND SURFACE
0.0	Asphalt Surface Treatment (200 mm)
0.2	Sand, trace to some gravel, trace silt (FILL) Loose to compact Brown Moist
245.8 / 3.0	CLAY, trace to some sand, trace to some organics Soft to firm Grey Wet
241.6 / 7.2	Sandy SILT, trace to some clay, trace to some gravel (TILL) Very loose Grey Wet Grinding of augers noted at 7.6 m and 9.1 m depth. Grinding of augers noted below 10.1 m depth.
238.6 / 10.2	Sandy CLAYEY SILT, trace gravel (TILL) Hard Grey Moist Switched to NW casing at 12.2 m depth.

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS
1	AS	-	
2	SS	23	
3	SS	10	
4	SS	8	
5	SS	7	
6	SS	4	OC=7.4%; GR 0, SA 10, SI 43, CL 47; liquid limit 82
7	SS	4	
8	SS	WH	Field vane, sensitivity 3
9	TO	PH	
10	SS	WH	GR 6, SA 28, SI 58, CL 8
11	SS	WH	
12	SS	64	GR 1, SA 26, SI 53, CL 20
13	SS	78/0.15	
14	SS	74	

Groundwater level symbol shown at approx. elev. 242.7.

Elevation scale: 248 to 234.

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20, 40, 60, 80, 100

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

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

UNIT WEIGHT γ kN/m³

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

Continued Next Page

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

Golder Associates

RECORD OF BOREHOLE No P8/P8A

 METRIC

PROJECT: 11-1191-0008			
W.P.: 5149-06-00	LOCATION: N 5495112.1; E 323211.7		ORIGINATED BY: ID/EHS
DIST:	HWY: 583	BOREHOLE TYPE: 108mm ID Continuous Flight Hollow Stem Augers	COMPILED BY: DM
DATUM: Geodetic	DATE: December 12, 2012 and June 4, 2013		CHECKED BY: SEMC

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%)
	Sandy CLAYEY SILT, trace gravel (TILL) Hard Grey Moist	15	SS	79	233	
	Coarse gravel encountered between 15.8 and 16.6 m depth as follows:					
232.2 / 16.6	GNEISS (BEDROCK)	1	RC	REC 100%	232, 231	RQD = 70%
	Bedrock cored from 16.6 m depth to 19.8 m depth. For coring details see Record of Drillhole P2.	2	RC	REC 100%	230	RQD = 67%
229.0 / 19.8	END OF BOREHOLE				229	

Depth (m)	Thickness (mm)	Type
15.8	50	Gneiss
16.0	75	Gneiss
16.2	50	Gneiss
16.4	63	Gneiss
16.5	75	Gneiss
16.6	25	Gneiss

Note:

1. Water level at a depth of 6.2 m below ground surface (Elev. 242.6 m) upon completion of drilling.

2. Borehole moved 1.2 m north to obtain field vane at 5.8 m depth.

3. Borehole moved to 1.5 m north on June 4, 2013 to obtain Sample 14 and 15 and bedrock core (as Borehole P8A).

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100; PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³; GR SA SI CL

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

PROJECT: 11-1191-0008

RECORD OF DRILLHOLE: P8/P8A

SHEET 1 OF 1

LOCATION: N 5495112.1 ;E 323211.7

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: December 12, 2012 and June 4, 2013

DRILL RIG: CME 850

DRILLING CONTRACTOR: Landcore Drilling Ontario Inc.

DATUM: Geodetic

JN - Joint, FLT - Fault, SHR- Shear, VN - Vein, CJ - Conjugate, BD- Bedding, FO- Foliation, CO- Contact, OR- Orthogonal, CL - Cleavage, PL - Planar, CU- Curved, UN- Undulating, ST - Stepped, IR - Irregular, PO- Polished, K - Slickensided, SM- Smooth, Ro - Rough, MB- Mechanical Break, BR - Broken Rock

NOTE: For additional abbreviations refer to list of abbreviations & symbols.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. / DEPTH (m)	RUN No.	FLUSH COLOUR % RETURN	RECOVERY TOTAL CORE %	RECOVERY SOLID CORE %	R.Q.D. %	FRACT. INDEX PER 0.25 m	DISCONTINUITY DATA B Angle	DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jn	HYDRAULIC CONDUCTIVITY k, cm/s	Diametral Point Load Index (MPa)	RMC -Q' AVG.	NOTES WATER LEVELS INSTRUMENTATION
		REFER TO PREVIOUS PAGE		232.2																
17, 18	June 4, 2013 NQ Coring; NW	GNEISS Fine to coarse grained Grey		16.6	1	GREY 100%							JNFORo JNFORo JNFORo JNFORo JNFORo JNFORo JNFORo JNFORo JNIRRo JNFORo JNFORo							
19	June 4, 2013 NQ Coring	Quartz vein between 19.4 m and 19.8 m depth.		230.6 18.2	2	GREY 100%							JNFORo JNFORo JNFORo JNFORo JNFORo JNSTRo JNFORo							
20		END OF DRILLHOLE		229.0 19.8																

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: ID/EHS

CHECKED: SEMC

RECORD OF TEST PIT No TP1

1 OF 1 **METRIC**

PROJECT 11-1191-0008
W.P. 5149-06-00 LOCATION N 5495134.0 ;E 323187.0 ORIGINATED BY SA
DIST HWY 583 BOREHOLE TYPE Komatsu 228 Excavator COMPILED BY DM
DATUM Geodetic DATE September 25, 2013 CHECKED BY SEMC

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS
245.8	GROUND SURFACE						
0.0	Silty PEAT Brown Moist to wet	1					
		2				245	
244.7 / 1.1	SILTY CLAY to CLAY, trace to some sand Dark grey to grey Wet	3				244	
		4				243	
242.1 / 3.7	END OF TEST PIT Note: 1. Test pit dry upon completion.						

Columns: DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED; 20 40 60 80 100); PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

RECORD OF TEST PIT No TP2

1 OF 1 **METRIC**

PROJECT 11-1191-0008
W.P. 5149-06-00 LOCATION N 5495143.9 ;E 323185.4 ORIGINATED BY SA
DIST HWY 583 BOREHOLE TYPE Komatsu 228 Excavator COMPILED BY DM
DATUM Geodetic DATE September 25, 2013 CHECKED BY SEMC

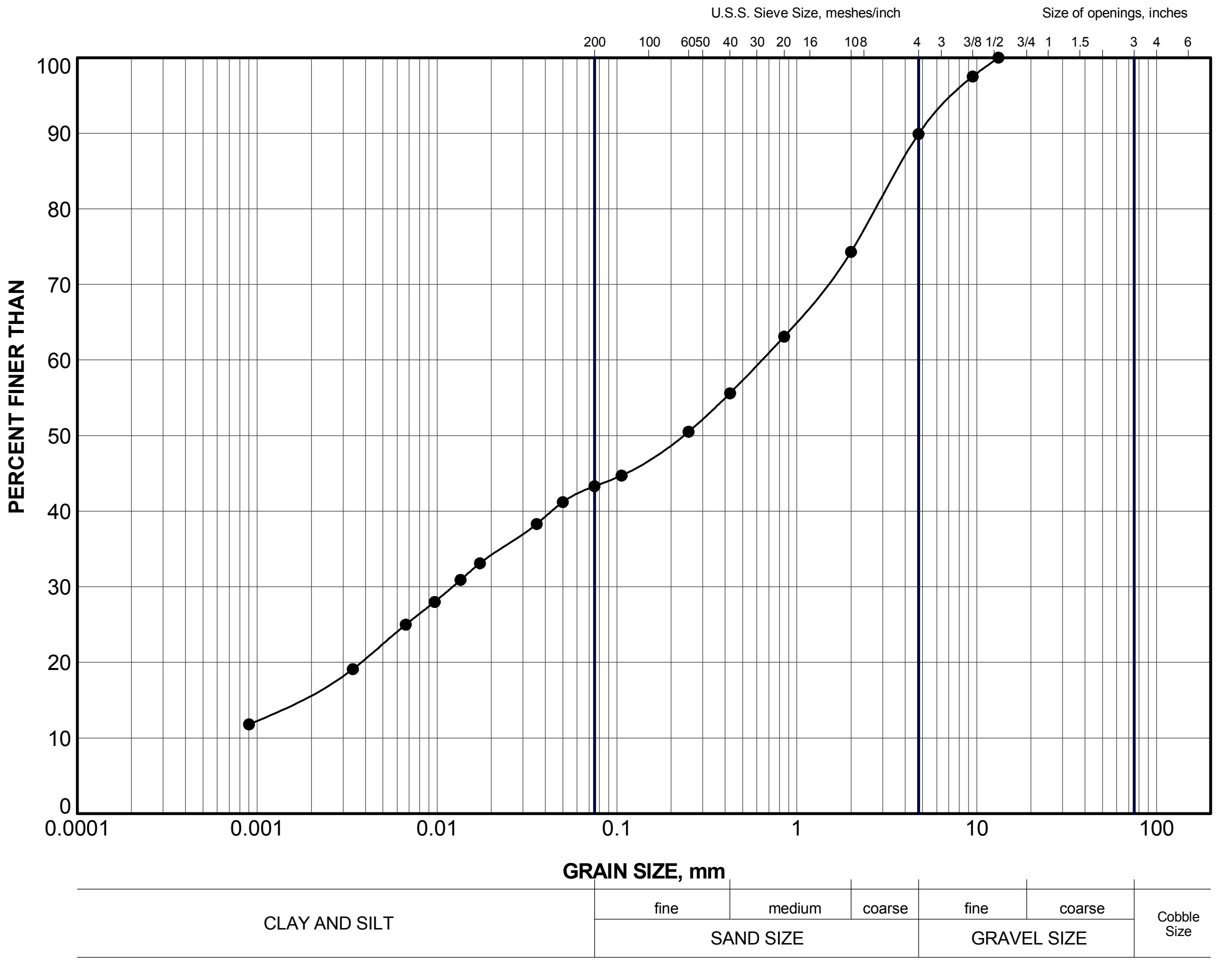
ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS
245.6	GROUND SURFACE						
0.0	Silty PEAT Brown Moist	1				245	
244.8 / 0.8	SILTY CLAY to CLAY, trace sand Brown to grey Wet	2				244	
		3				243	
241.9 / 3.7	END OF TEST PIT Note: 1. Water seepage noted at base of test pit upon completion.				▽	242	

Columns: DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED; 20 40 60 80 100); PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

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

APPENDIX B

Laboratory Test Results



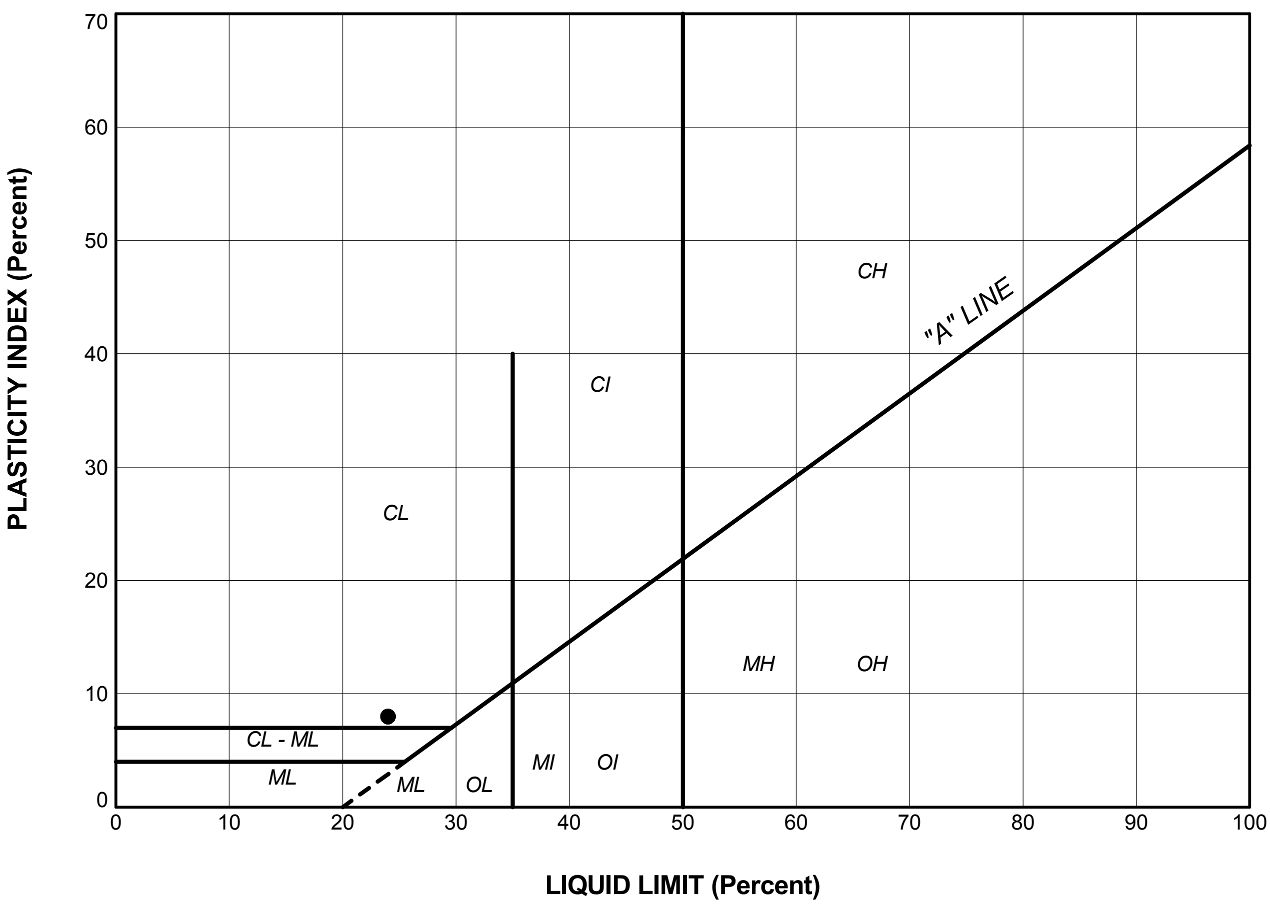
LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	P6	2	248.0

PROJECT	HIGHWAY 583 PRUNE CREEK BRIDGE
TITLE	**GRAIN SIZE DISTRIBUTION** CLAYEY SILT (FILL)

Golder Associates SUDBURY, ONTARIO	PROJECT No.	11-1191-0008		FILE No.	1111910008D.GPJ
	DRAWN	JJL	Oct 2013	SCALE N/A	REV.
	CHECK	DAM	Oct 2013	**FIGURE B1**	
	APPR	SEMC	Oct 2013		

SUD-MTO GSD (NEW) GLDR_LDN.GDT



SOIL TYPE
C = Clay
M = Silt
O = Organic

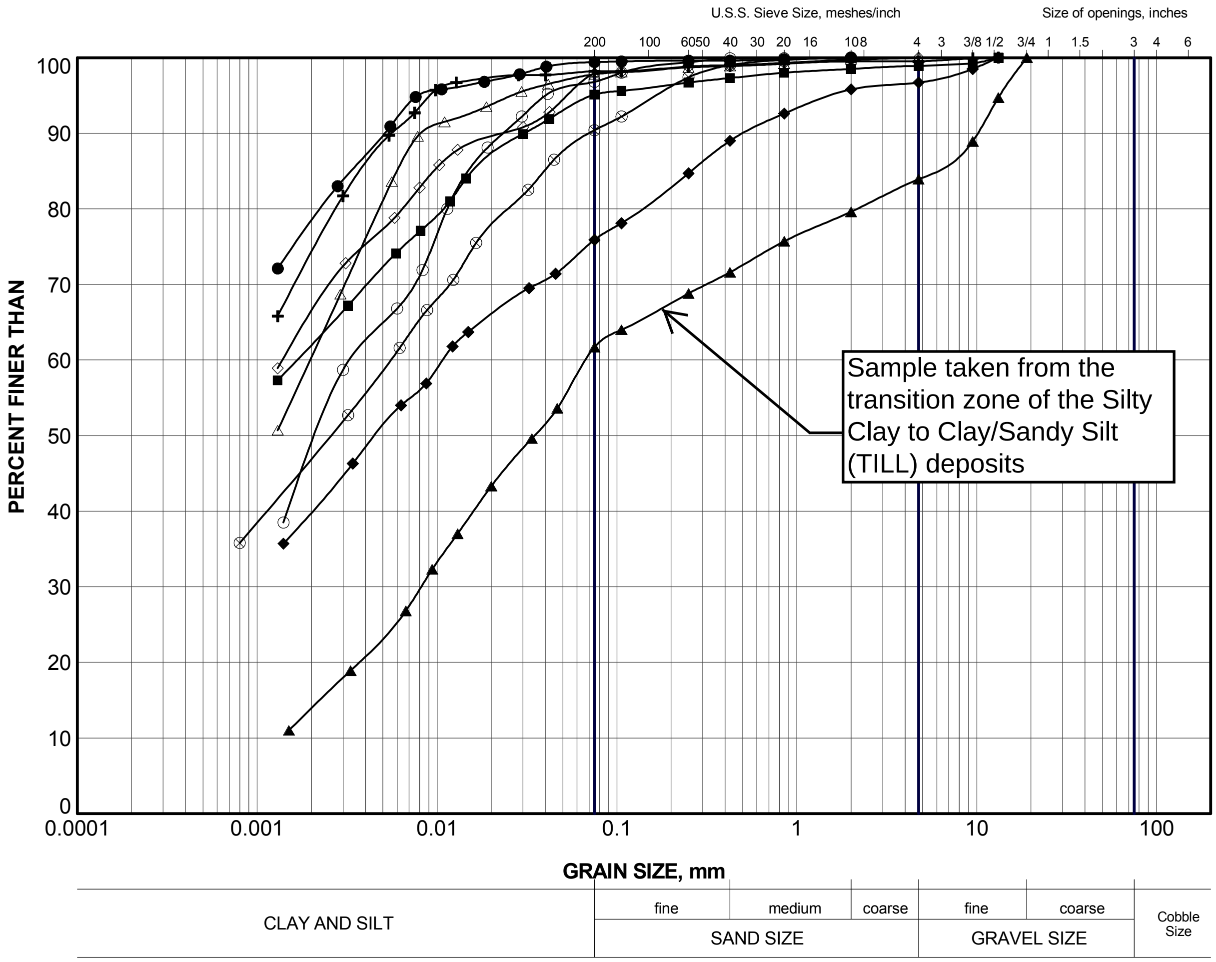
PLASTICITY
L = Low
I = Intermediate
H = High

LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	P6	2	24.0	16.0	8.0

PROJECT	HIGHWAY 583 PRUNE CREEK BRIDGE				
TITLE	**PLASTICITY CHART** CLAYEY SILT (FILL)				
Golder Associates SUDBURY, ONTARIO	PROJECT No.	11-1191-0008		FILE No.	1111910008D.GPJ
	DRAWN	JJL	Oct 2013	SCALE N/A	REV.
	CHECK	DAM	Oct 2013	**FIGURE B2**	
	APPR	SEMC	Oct 2013		

SUD-MTO PL (NEW) GLDR_LDN.GDT



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	P1	8	242.4
■	P2	3	243.9
▲	P2	6	241.1
✚	P3	4	244.1
◆	P4	3	243.8
◇	P4	5	242.1
○	P5	2	245.3
△	P7	5	243.9
⊗	P8/P8A	6	244.7

PROJECT

HIGHWAY 583
PRUNE CREEK BRIDGE

TITLE

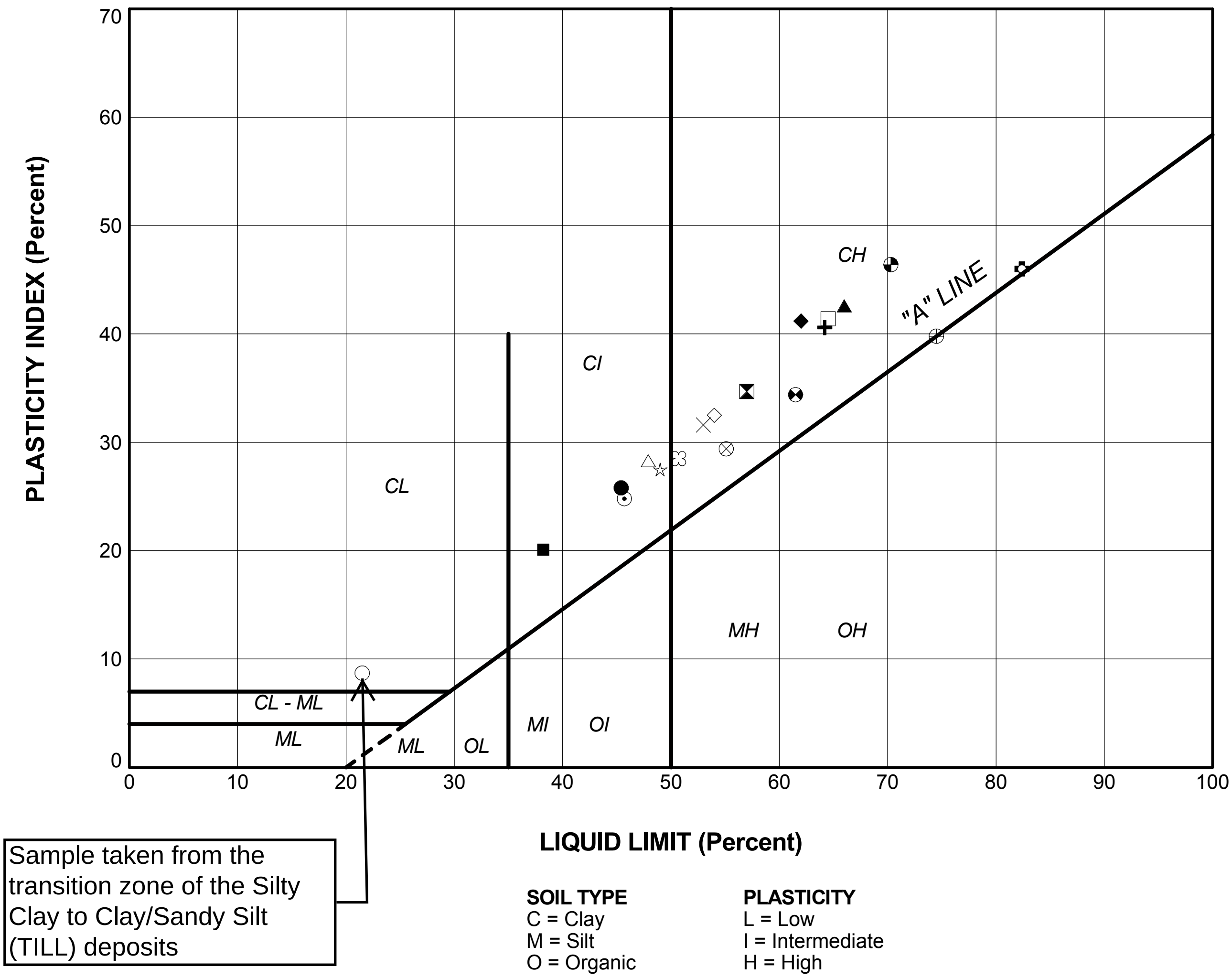
GRAIN SIZE DISTRIBUTION

SILTY CLAY to CLAY

Golder Associates
SUDBURY, ONTARIO

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

SUD-MTO GSD (NEW) GLDR_LDN.GDT



SOIL TYPE
C = Clay
M = Silt
O = Organic

PLASTICITY
L = Low
I = Intermediate
H = High

LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	P1	5	45.4	19.6	25.8
■	P1	6	38.2	18.1	20.1
▲	P1	8	66.0	23.4	42.6
✚	P2	3	64.2	23.6	40.6
◆	P2	4	62.0	20.8	41.2
◇	P2	5	54.0	21.5	32.5
○	P2	6	21.5	12.8	8.7
△	P3	4	47.9	19.6	28.3
⊗	P3	5	55.1	25.7	29.4
⊕	P4	3	74.5	34.7	39.8
□	P4	5	64.5	23.1	41.4
	P5	2	61.5	27.1	34.4
	P5	5	70.3	23.9	46.4
☆	P6	4	49.0	21.5	27.5
	P7	2	50.7	22.2	28.5
⧗	P7	4	57.0	22.3	34.7
⊙	P7	5	45.7	20.9	24.8
	P8/P8A	6	82.4	36.4	46.0
×	P8/P8A	8	53.0	21.4	31.6

PROJECT
HIGHWAY 583
PRUNE CREEK BRIDGE

TITLE
PLASTICITY CHART
SILTY CLAY to CLAY

Golder Associates
SUDBURY, ONTARIO

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

SUD-MTO PL (NEW) 25 GLDR_LDN.GDT

CONSOLIDATION TEST SUMMARY

SAMPLE IDENTIFICATION

Project Number:	11-1191-0008	Sample Number:	4
Borehole Number:	7	Sample Depth, m:	3.51

TEST CONDITIONS

Test Type	Standard	Load Duration, hr	24
Oedometer Number	2		
Date Started	June 13, 2013		
Date Completed	June 20, 2013		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	2.517	Unit Weight, kN/m^3	15.88
Sample Diameter, cm	6.347	Dry Unit Weight, kN/m^3	9.33
Area, cm^2	31.64	Specific Gravity, Measured	2.743
Volume, cm^3	79.64	Solids Height, cm	0.873
Water Content, %	70.20	Volume of Solids, cm^3	27.62
Wet Mass, g	128.92	Volume of Voids, cm^3	52.02
Dry Mass, g	75.75		

TEST COMPUTATIONS

Pressure kPa	Primary Consolidation	Corr. Height cm	Void Ratio	Average Height cm	t_{90} sec	cv. cm^2/s	mv m^2/kN	k cm/s	Total Work kJ/m3
0	0	2.517	1.884	2.517					
4	0.03	2.514	1.880	2.516	2160	0.0006	3.09E-04	1.88E-08	0.003
13	0.05	2.509	1.874	2.511	735	0.0018	2.28E-04	4.06E-08	0.020
31	0.12	2.497	1.860	2.503	1162	0.0011	2.70E-04	3.02E-08	0.126
66	0.28	2.469	1.828	2.483	1441	0.0009	3.16E-04	2.81E-08	0.674
137	1.76	2.293	1.626	2.381	10140	0.0001	9.94E-04	1.15E-08	7.906
277	2.10	2.083	1.386	2.188	6000	0.0002	5.92E-04	9.82E-09	26.815
558	1.33	1.950	1.234	2.017	3650	0.0002	1.88E-04	4.36E-09	53.493
1117	1.00	1.850	1.119	1.900	2018	0.0004	7.11E-05	2.64E-09	96.492
558	-0.14	1.864	1.135	1.857					
137	-0.55	1.919	1.198	1.891					
31	-0.61	1.979	1.267	1.949					
4	-0.52	2.031	1.327	2.005					

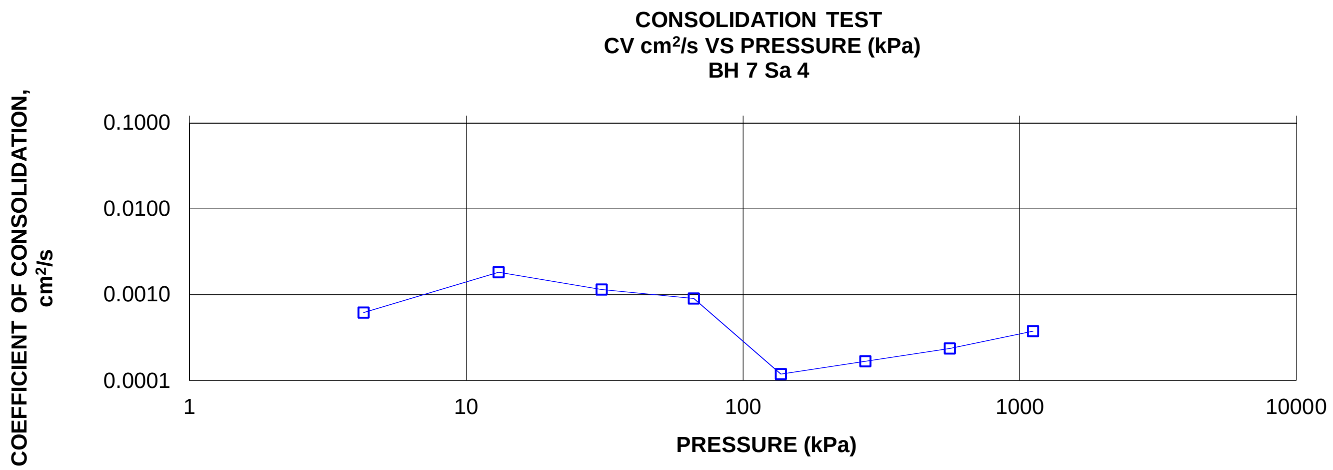
Note:
k calculated using c_v based on t_{90} values.

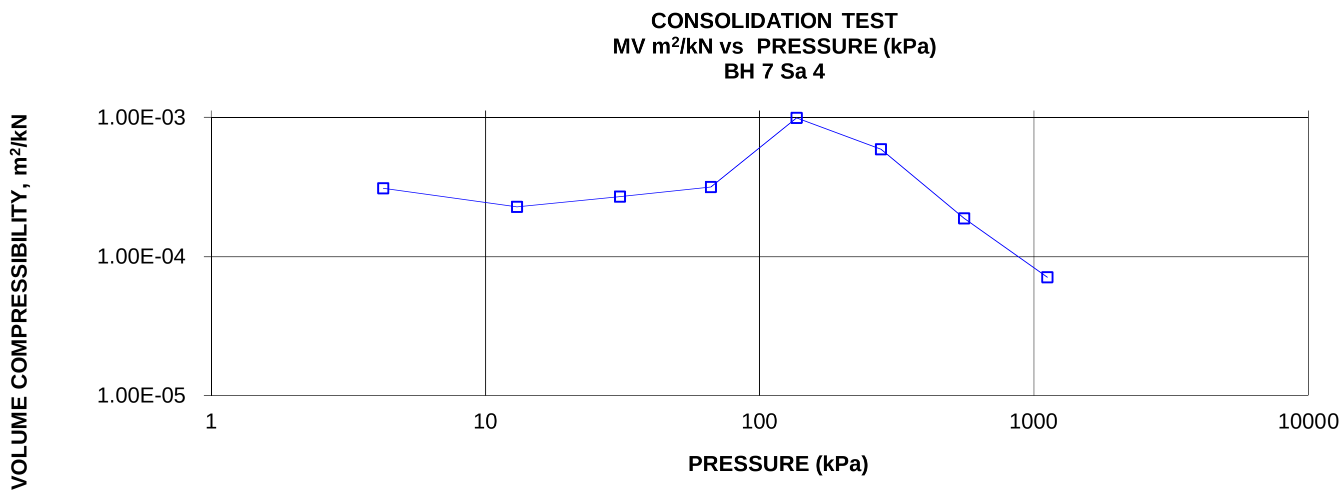
SAMPLE DIMENSIONS AND PROPERTIES - FINAL

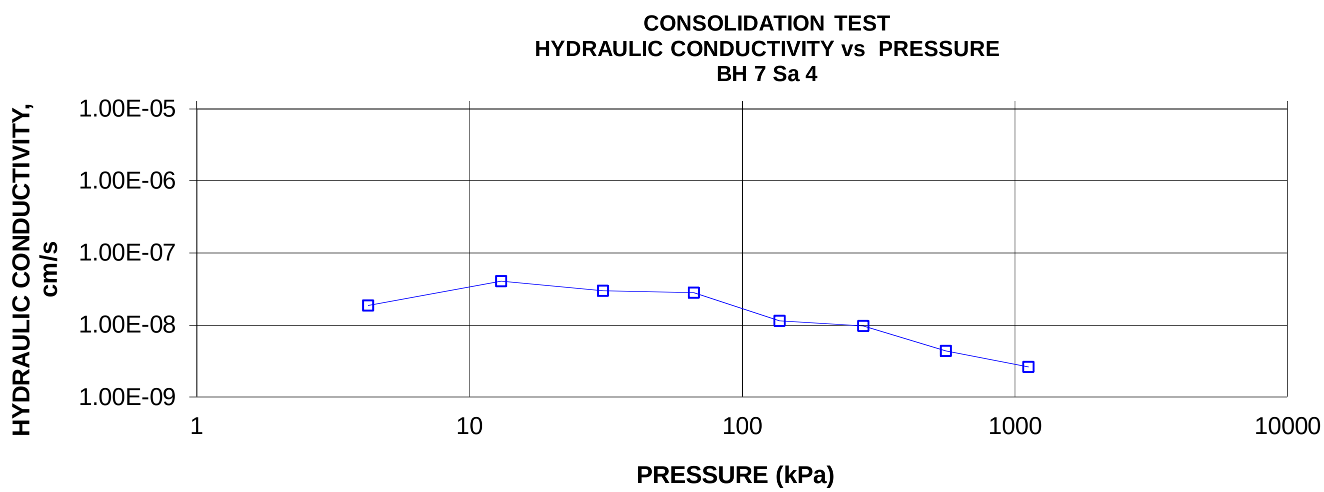
Sample Height, cm	2.031	Unit Weight, kN/m^3	16.68
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m^3	11.56
Area, cm^2	31.64	Specific Gravity, Measured	2.743
Volume, cm^3	64.26	Solids Height, cm	0.873
Water Content, %	44.27	Volume of Solids, cm^3	27.62
Wet Mass, g	109.28	Volume of Voids, cm^3	36.64
Dry Mass, g	75.75		

CONSOLIDATION TEST SUMMARY

FIGURE B5







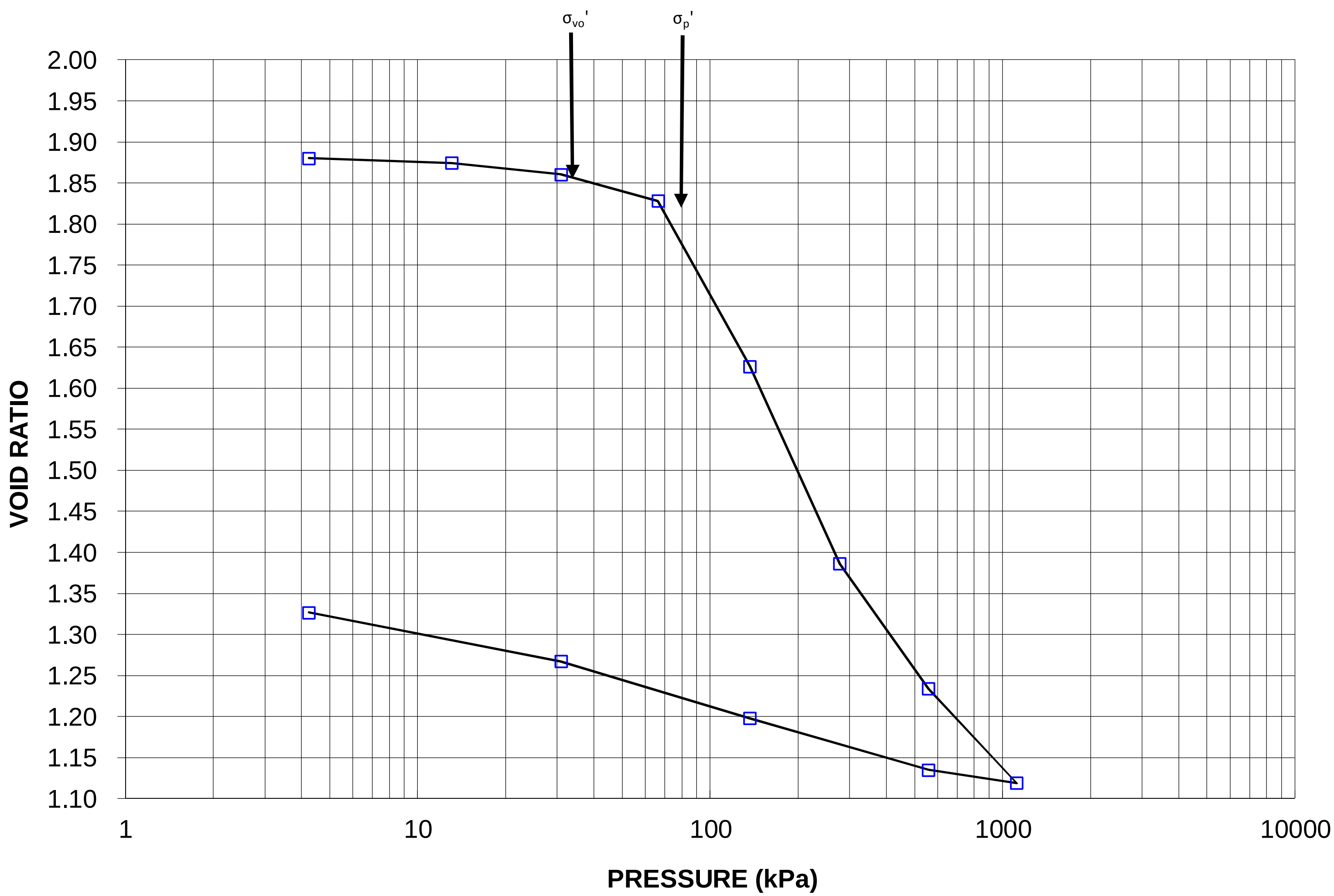
Project No.11-1191-0008
Prepared By: TG

Golder Associates

Checked By: SL

Project No. 11-1191-0008
Prepared By TG

CONSOLIDATION TEST
VOID RATIO vs PRESSURE
BH 7 Sa 4



Golder Associates

Checked By: SL

CONSOLIDATION TEST
VOID RATIO VS LOG PRESSURE

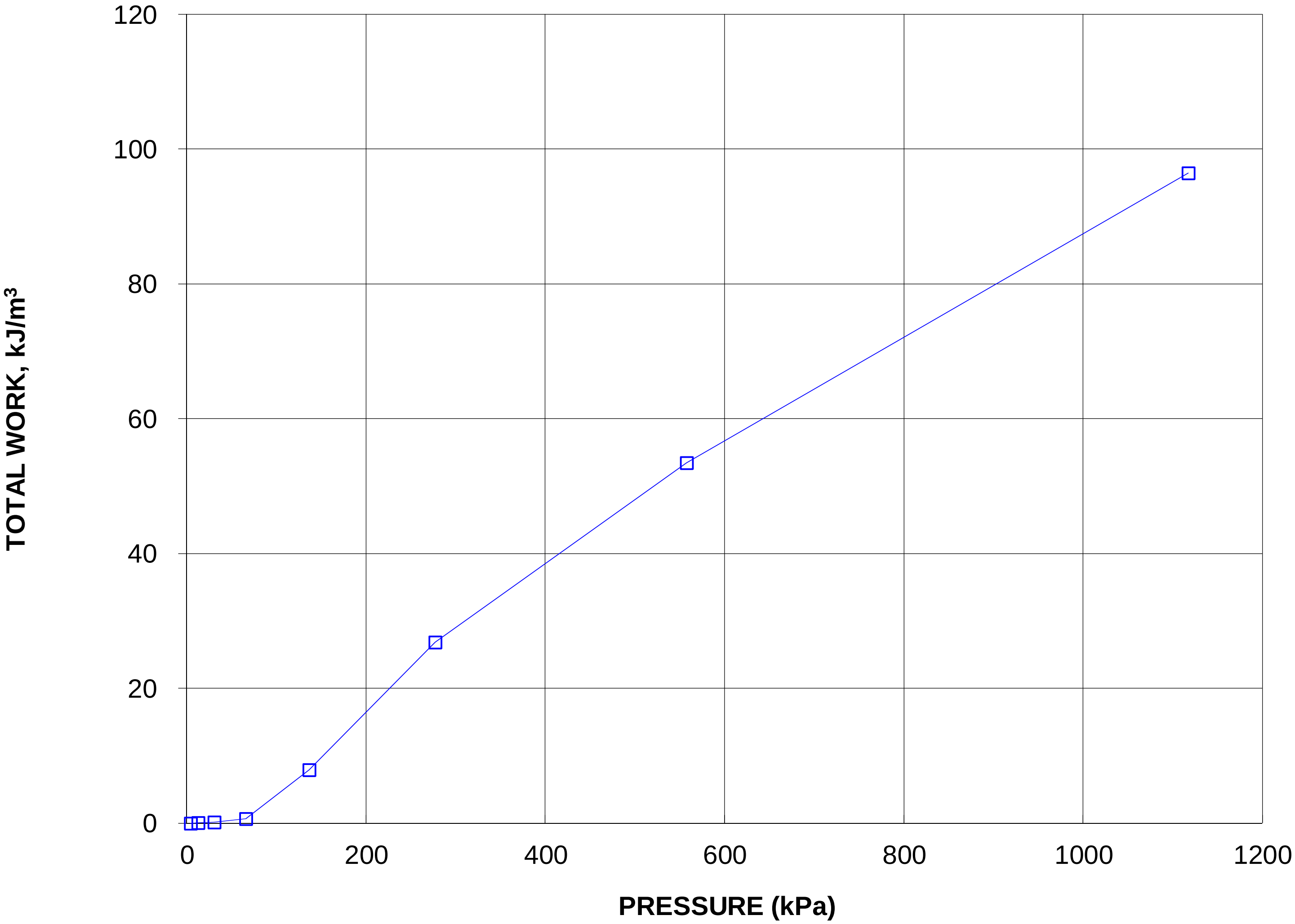
FIGURE B5
Pg. 3 of 4

Project No: 11-1191-0008
Prepared By: TG

Golder Associates

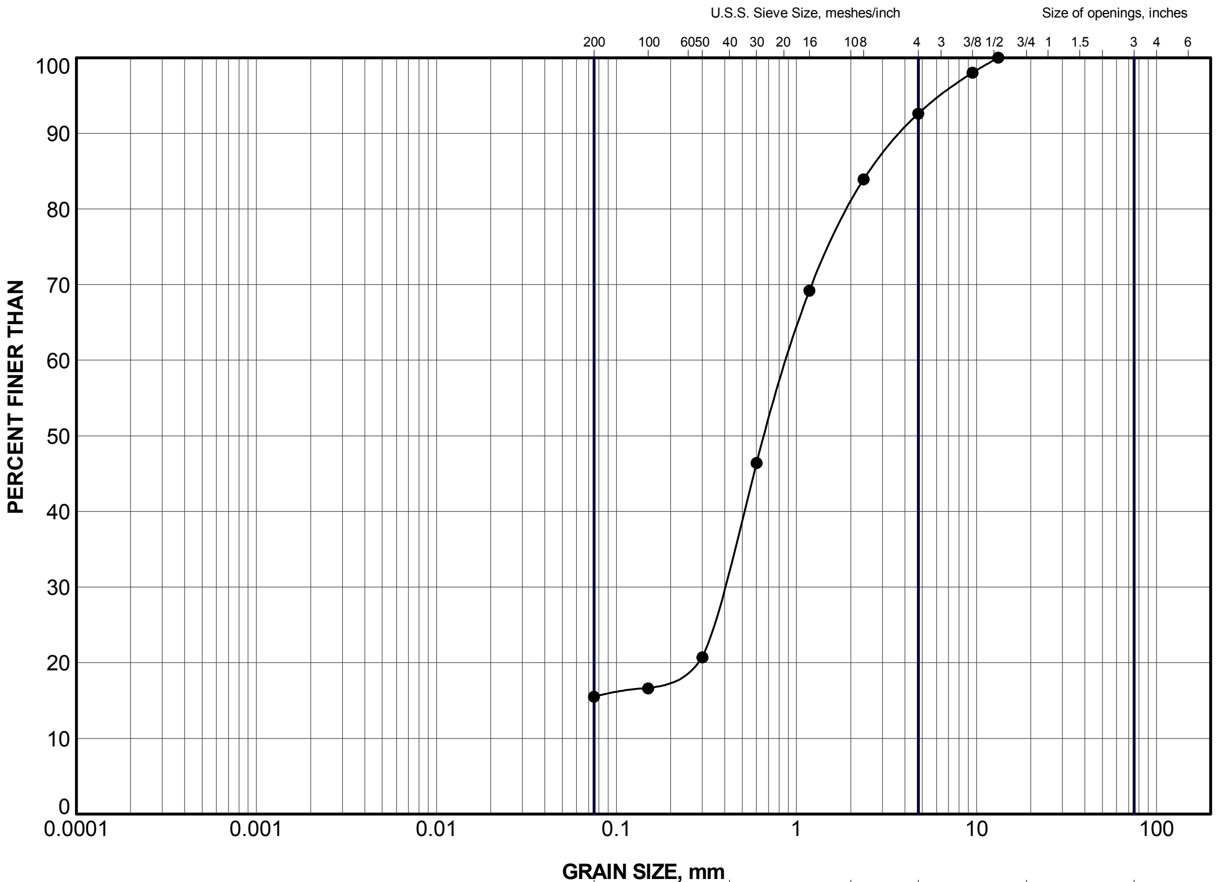
Checked By: SL

CONSOLIDATION TEST
TOTAL WORK, kJ/m³ vs PRESSURE
BH 7Sa 4



CONSOLIDATION TEST
TOTAL WORK VS PRESSURE

FIGURE B5
Pg. 4 of 4



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	P7	7	241.6

PROJECT

HIGHWAY 583
PRUNE CREEK BRIDGE

TITLE

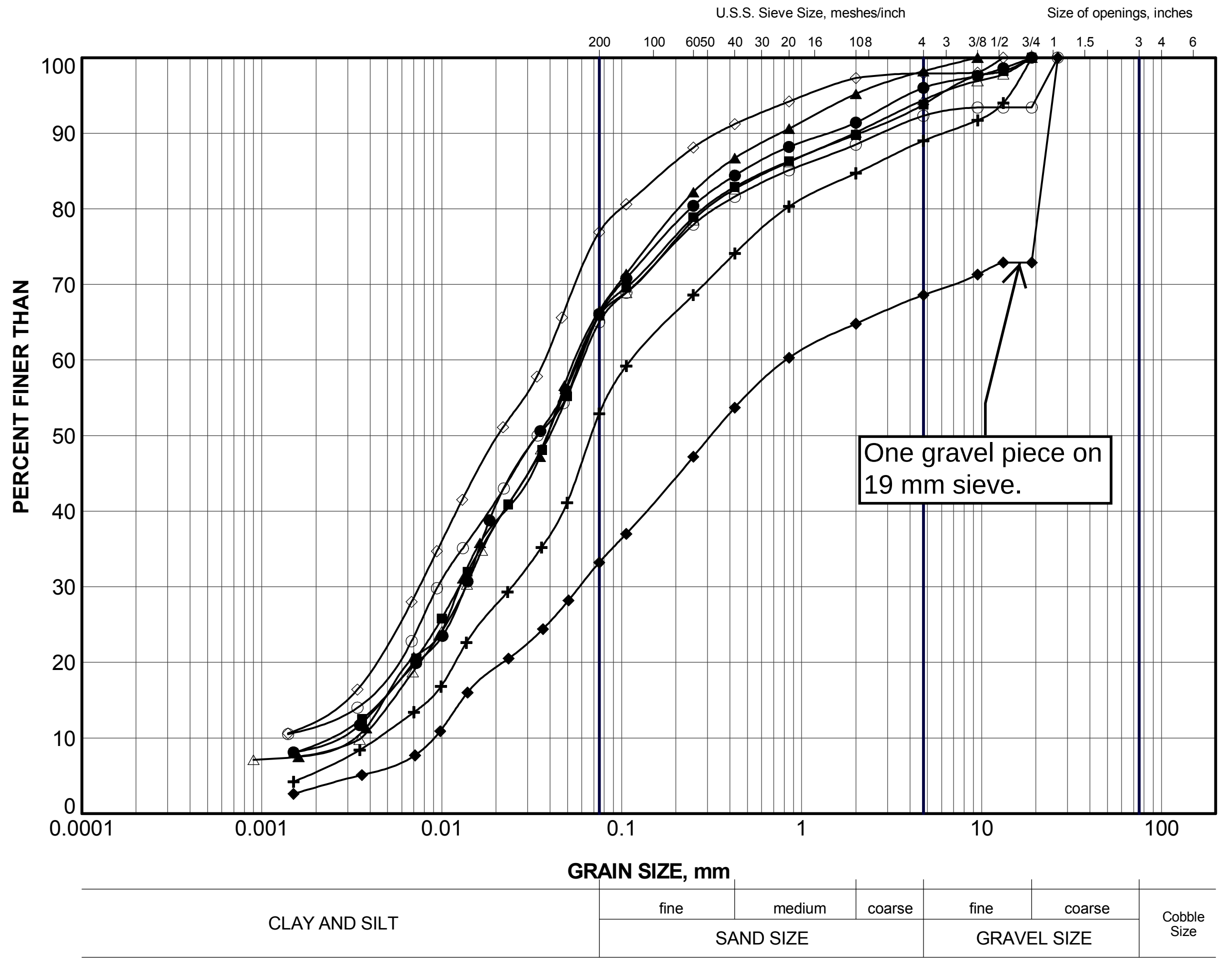
GRAIN SIZE DISTRIBUTION

SAND

Golder Associates
SUDBURY, ONTARIO

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

SUD-MTO GSD (NEW) GLDR_LDN.GDT



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	P1	9	241.2
■	P2	7	240.1
▲	P3	8	240.3
✚	P4	8	239.2
◆	P5	7	240.0
◇	P6	7	242.7
○	P6	10	238.1
△	P8/P8A	10	240.9

PROJECT

HIGHWAY 583
PRUNE CREEK BRIDGE

TITLE

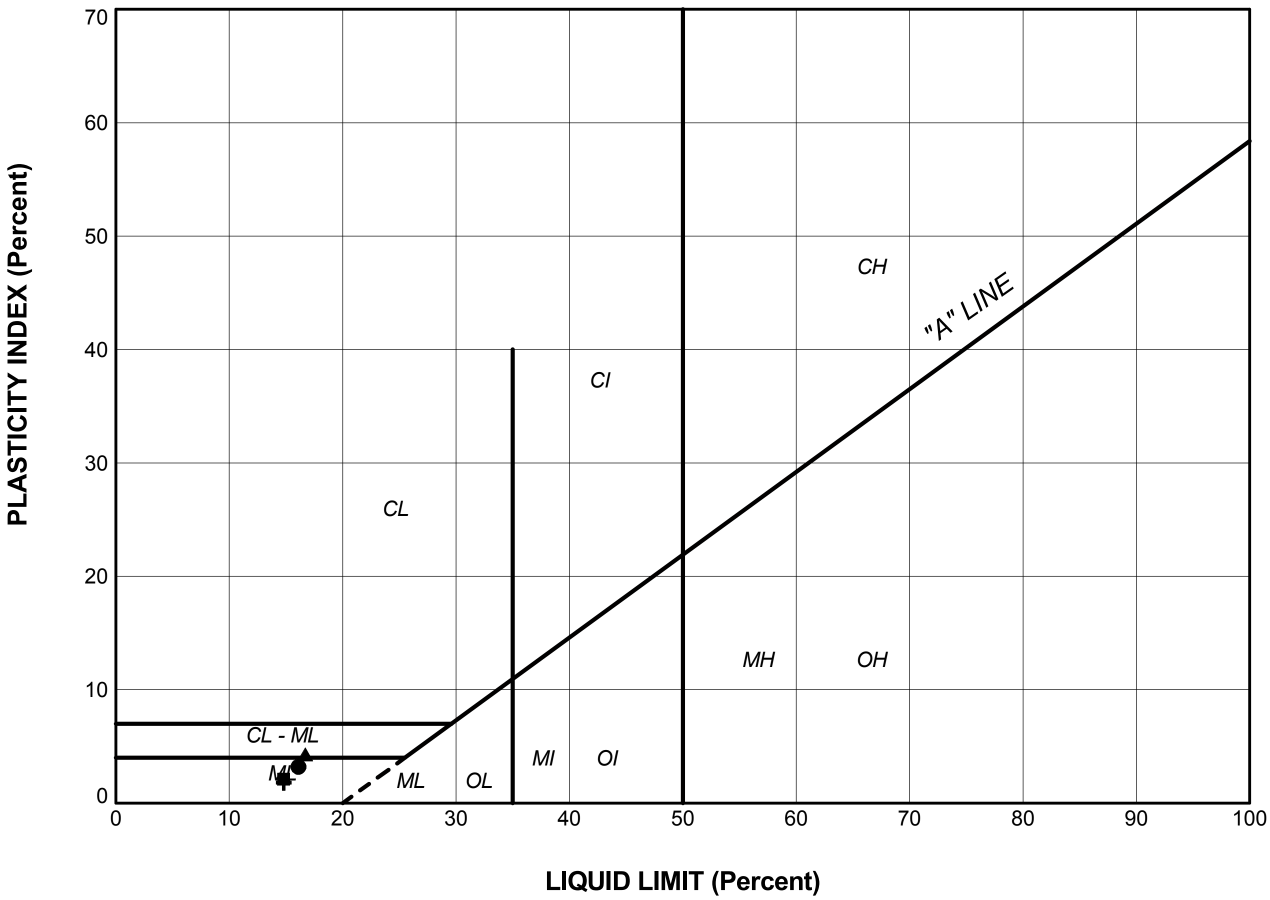
GRAIN SIZE DISTRIBUTION

SANDY SILT to SAND AND SILT (TILL)

Golder Associates
SUDBURY, ONTARIO

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

SUD-MTO GSD (NEW) GLDR_LDN.GDT



SOIL TYPE
C = Clay
M = Silt
O = Organic

PLASTICITY
L = Low
I = Intermediate
H = High

LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	P6	7	16.1	12.9	3.2
■	P6	8	14.8	12.7	2.1
▲	P6	10	16.7	12.4	4.3
+	P8/P8A	10	14.8	13.0	1.8

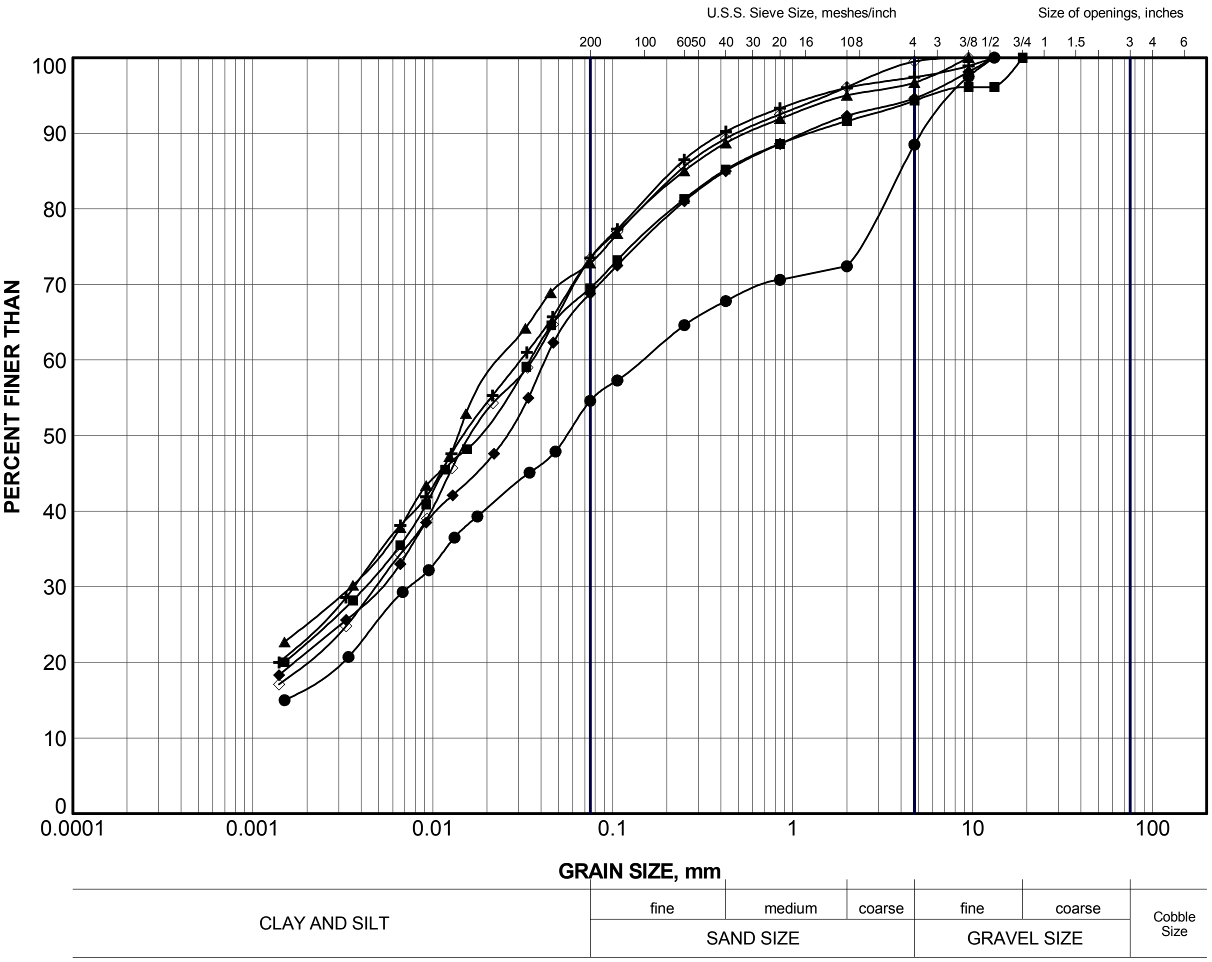
PROJECT
HIGHWAY 583
PRUNE CREEK BRIDGE

TITLE
PLASTICITY CHART
SANDY SILT to SAND AND SILT (TILL)

Golder Associates
SUDBURY, ONTARIO

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

SUD-MTO PL (NEW) GLDR_LDN.GDT



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	P1	12	236.7
■	P2	10	236.3
▲	P3	10	238.8
+	P7	10	237.0
◆	P7	12	234.1
◇	P8/P8A	12	237.8

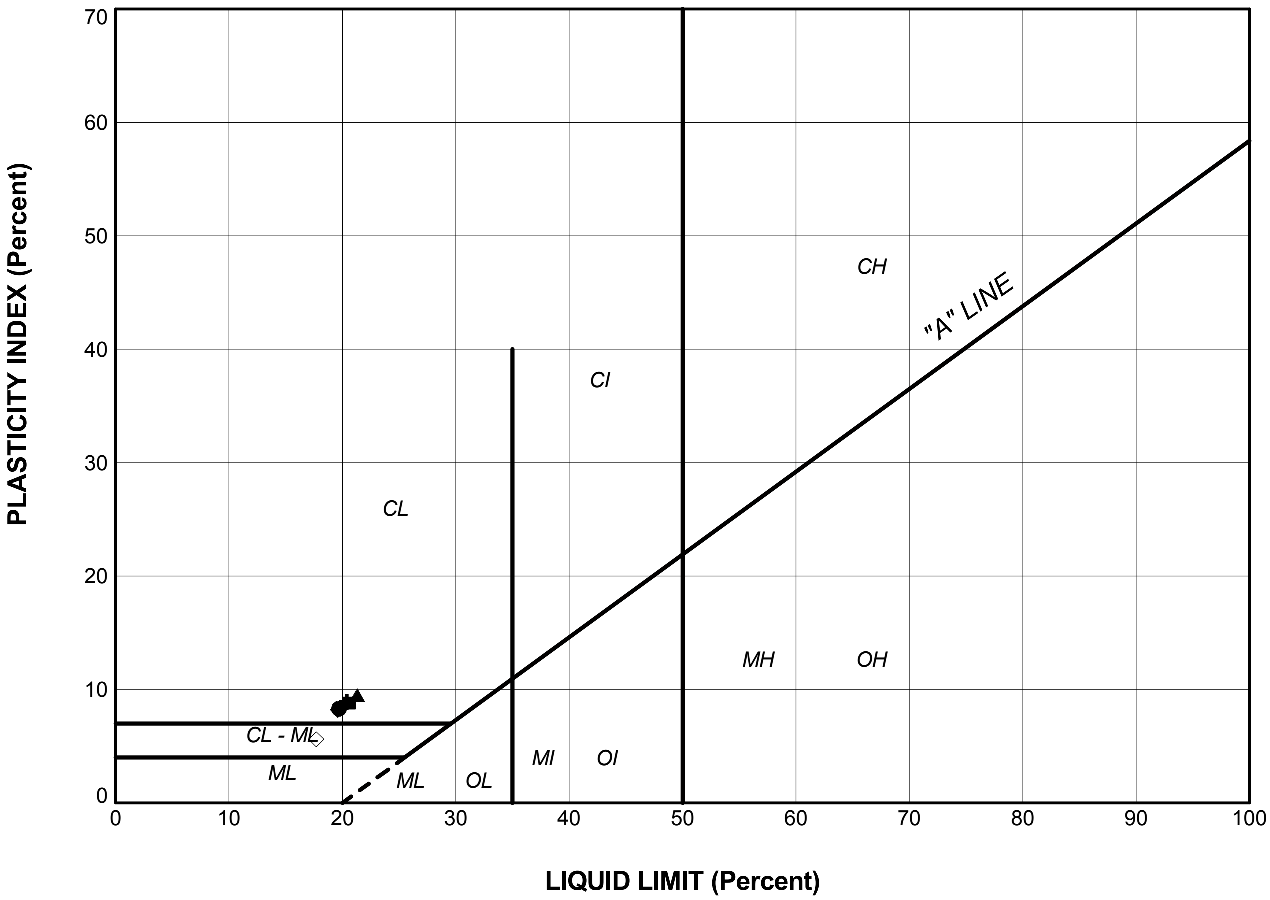
PROJECT
HIGHWAY 583
PRUNE CREEK BRIDGE

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

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

Golder Associates
SUDBURY, ONTARIO

SUD-MTO GSD (NEW) GLDR_LDN.GDT



SOIL TYPE
C = Clay
M = Silt
O = Organic

PLASTICITY
L = Low
I = Intermediate
H = High

LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	P1	12	19.7	11.4	8.3
■	P2	10	20.6	11.8	8.8
▲	P3	10	21.3	11.8	9.5
+	P7	10	20.4	11.5	8.9
◆	P7	12	19.6	11.4	8.2
◇	P8/P8A	12	17.7	12.1	5.6

PROJECT

HIGHWAY 583
PRUNE CREEK BRIDGE

TITLE

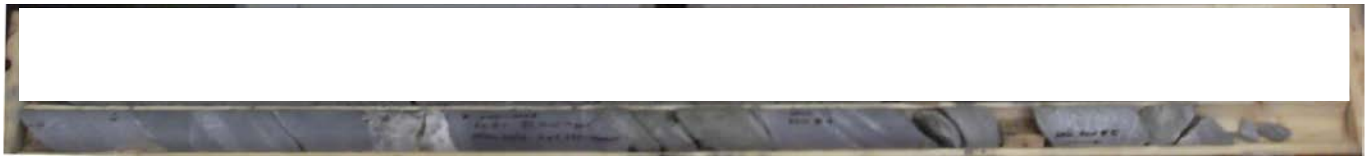
PLASTICITY CHART

SANDY CLAYEY SILT to CLAYEY SILT (TILL)

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

Golder Associates
SUDBURY, ONTARIO

SUD-MTO PL (NEW) GLDR_LDN.GDT



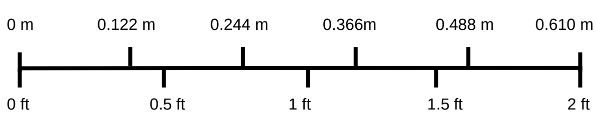
Borehole P2 (Box 1 of 2)
Elevation 230.4 m to 228.8 m

Borehole P2 (Box 2 of 2)
Elevation 228.8 m to 227.4 m

(Note: tape used to prevent loss of core during transport)

Borehole P8/8A (Box 1 of 1)
Elevation 232.2 m to 229.0 m

(Note: tape used to prevent loss of core during transport)



PROJECT
HIGHWAY 583
PRUNE CREEK BRIDGE

TITLE
BEDROCK CORE PHOTOGRAPHS

Golder Associates

PROJECT No.	11-1191-0008		FILE No. ----
DESIGN	MT	Oct 2013	SCALE AS SHOWN REV.
CADD	--		**FIGURE B11**
CHECK	SEMC	Oct 2013	
REVIEW	JMAC	Oct 2013	

APPENDIX C

Analytical Laboratory Test Results

Table C1 - Summary of Analytical Testing of Creek Water

Parameter	Units	Result
Resistivity	ohm-cm	14,000
Conductivity	µmho/cm	73
pH	pH	7.20
Sulphate	mg/L	80
Chloride	mg/L	41

Prepared by: DAM
Reviewed by: SEMC

Notes:
1. Sample obtained March 23, 2012
2. Analytical testing carried out by Maxxam Analytics Inc.

Your Project #: 11-1191-0008
Site#: 11-1191-0008
Site Location: PRUNE CREEK BRIDGE, HEARST, ONTARIO
Your C.O.C. #: EO863712

Attention: David Muldowney
Golder Associates Ltd
1010 Lorne St
Sudbury, ON
P3C 4R9

Report Date: 2013/10/04

CERTIFICATE OF ANALYSIS

MAXXAM JOB #: B3G4937
Received: 2013/09/28, 09:40

Sample Matrix: Soil
Samples Received: 2

Analyses	Quantity	Date Extracted	Date Analyzed	Laboratory Method	Method Reference
Chloride (20:1 extract)	2	N/A	2013/10/04	CAM SOP-00463	EPA 325.2
Cyanide (WAD) in Leachates	2	N/A	2013/10/03	CAM SOP-00457	Ontario MOE CN-3015
Petroleum Hydro. CCME F1 & BTEX in Soil	2	2013/10/01	2013/10/03	CAM SOP-00315	CCME CWS
Petroleum Hydrocarbons F2-F4 in Soil	2	2013/10/03	2013/10/03	CAM SOP-00316	CCME CWS
Fluoride by ISE in Leachates	2	2013/10/03	2013/10/03	CAM SOP-00449	SM 4500FC
Mercury (TCLP Leachable) (mg/L)	2	N/A	2013/10/03	CAM SOP-00453	EPA 7470
Total Metals Analysis by ICP	2	2013/10/03	2013/10/03	CAM SOP-00408	SW-846 6010C
Total Metals in TCLP Leachate by ICPMS	2	2013/10/03	2013/10/03	CAM SOP-00447	EPA 6020
Moisture	2	N/A	2013/10/02	CAM SOP-00445	R.Carter,1993
Nitrate(NO3) + Nitrite(NO2) in Leachate	2	N/A	2013/10/03	CAM SOP-00440	SM 4500 NO3I/NO2B
PAH Compounds in Leachate by GC/MS (SIM)	2	2013/10/02	2013/10/03	CAM SOP-00318	EPA 8270
TCLP - % Solids	2	2013/10/01	2013/10/02	CAM SOP-00401	EPA 1311 modified
TCLP - Extraction Fluid	2	N/A	2013/10/02	CAM SOP-00401	EPA 1311 modified
TCLP - Initial and final pH	2	N/A	2013/10/02	CAM SOP-00401	EPA 1311 modified

Remarks:

Maxxam Analytics has performed all analytical testing herein in accordance with ISO 17025 and the Protocol for Analytical Methods Used in the Assessment of Properties under Part XV.1 of the Environmental Protection Act. All methodologies comply with this document and are validated for use in the laboratory. The methods and techniques employed in this analysis conform to the performance criteria (detection limits, accuracy and precision) as outlined in the Protocol for Analytical Methods Used in the Assessment of Properties under Part XV.1 of the Environmental Protection Act. Reporting results to two significant figures at the RDL is to permit statistical evaluation and is not intended to be an indication of analytical precision.

The CWS PHC methods employed by Maxxam conform to all prescribed elements of the reference method and performance based elements have been validated. All modifications have been validated and proven equivalent following the 'Alberta Environment Draft Addenda to the CWS-PHC, Appendix 6, Validation of Alternate Methods'. Documentation is available upon request. Maxxam has made the following improvements to the CWS-PHC reference benchmark method: (i) Headspace for F1; and, (ii) Mechanical extraction for F2-F4. Note: F4G cannot be added to the C6 to C50 hydrocarbons. The extraction date for samples field preserved with methanol for F1 and Volatile Organic Compounds is considered to be the date sampled.

Maxxam Analytics is accredited for all specific parameters as required by Ontario Regulation 153/04. Maxxam Analytics is limited in liability to the actual cost of analysis unless otherwise agreed in writing. There is no other warranty expressed or implied. Samples will be retained at Maxxam Analytics for three weeks from receipt of data or as per contract.

* RPDs calculated using raw data. The rounding of final results may result in the apparent difference.
* Results relate only to the items tested.

Maxxam Job #: B3G4937
Report Date: 2013/10/04

Golder Associates Ltd
Client Project #: 11-1191-0008
Site Location: PRUNE CREEK BRIDGE, HEARST, ONTARIO
Sampler Initials: SA

Encryption Key

Please direct all questions regarding this Certificate of Analysis to your Project Manager.

Antonella Brasil, Project Manager
Email: ABrasil@maxxam.ca
Phone# (905) 817-5817

==
Maxxam has procedures in place to guard against improper use of the electronic signature and have the required "signatories", as per section 5.10.2 of ISO/IEC 17025:2005(E), signing the reports. For Service Group specific validation please refer to the Validation Signature Page.

Total cover pages: 2

Maxxam Job #: B3G4937
Report Date: 2013/10/04

Golder Associates Ltd
Client Project #: 11-1191-0008
Site Location: PRUNE CREEK BRIDGE, HEARST, ONTARIO
Sampler Initials: SA

RESULTS OF ANALYSES OF SOIL

Maxxam ID		TG5607		TG5608		
Sampling Date		2013/09/25 16:00		2013/09/25 16:30		
	Units	**TP1**	**RDL**	**TP2**	**RDL**	**QC Batch**
Inorganics						
Soluble (20:1) Chloride (Cl)	ug/g	ND	20	ND	20	3374274
Final pH	pH	6.17		6.16		3368820
Leachable Fluoride (F-)	mg/L	0.7	0.1	0.3	0.1	3372492
Leachable Free Cyanide	mg/L	ND	0.002	ND	0.002	3372489
Initial pH	pH	9.01		8.74		3368820
Moisture	%	40	1.0	28	1.0	3370850
TCLP - % Solids	%	100	0.2	100	0.2	3368817
TCLP Extraction Fluid	N/A	FLUID 1		FLUID 1		3368819
Leachable Nitrite (N)	mg/L	ND	0.1	ND	0.1	3372490
Leachable Nitrate (N)	mg/L	260	5	ND	1	3372490
Leachable Nitrate + Nitrite	mg/L	260	5	ND	1	3372490
Metals						
Leachable Mercury (Hg)	mg/L	ND	0.001	ND	0.001	3371492

N/A = Not Applicable
ND = Not detected
RDL = Reportable Detection Limit
QC Batch = Quality Control Batch

Maxxam Analytics International Corporation o/a Maxxam Analytics 6740 Campobello Road, Mississauga, Ontario, L5N 2L8 Tel: (905) 817-5700 Toll-Free: 800-563-6266 Fax: (905) 817-5777 www.maxxam.ca

Maxxam Job #: B3G4937
Report Date: 2013/10/04

Golder Associates Ltd
Client Project #: 11-1191-0008
Site Location: PRUNE CREEK BRIDGE, HEARST, ONTARIO
Sampler Initials: SA

ELEMENTS BY ATOMIC SPECTROSCOPY (SOIL)

Maxxam ID		TG5607	TG5608		
Sampling Date		2013/09/25 16:00	2013/09/25 16:30		
	Units	**TP1**	**TP2**	**RDL**	**QC Batch**
Metals					
Leachable Arsenic (As)	mg/L	ND	ND	0.2	3372683
Leachable Barium (Ba)	mg/L	1.1	0.7	0.2	3372683
Leachable Boron (B)	mg/L	0.5	0.2	0.1	3372683
Leachable Cadmium (Cd)	mg/L	ND	ND	0.05	3372683
Leachable Chromium (Cr)	mg/L	ND	ND	0.1	3372683
Leachable Lead (Pb)	mg/L	ND	ND	0.1	3372683
Leachable Selenium (Se)	mg/L	ND	ND	0.1	3372683
Leachable Silver (Ag)	mg/L	ND	ND	0.01	3372683
Acid Extractable Sodium (Na)	ug/g	420	270	100	3372939
Leachable Uranium (U)	mg/L	ND	ND	0.01	3372683

SEMI-VOLATILE ORGANICS BY GC-MS (SOIL)

Maxxam ID		TG5607	TG5608		
Sampling Date		2013/09/25 16:00	2013/09/25 16:30		
	Units	**TP1**	**TP2**	**RDL**	**QC Batch**
Polyaromatic Hydrocarbons					
Leachable Benzo(a)pyrene	ug/L	ND	ND	0.04	3371892
Surrogate Recovery (%)					
Leachable D10-Anthracene	%	106	103		3371892
Leachable D14-Terphenyl (FS)	%	94	92		3371892
Leachable D8-Acenaphthylene	%	94	91		3371892

ND = Not detected
RDL = Reportable Detection Limit
QC Batch = Quality Control Batch

Maxxam Job #: B3G4937
Report Date: 2013/10/04

Golder Associates Ltd
Client Project #: 11-1191-0008
Site Location: PRUNE CREEK BRIDGE, HEARST, ONTARIO
Sampler Initials: SA

PETROLEUM HYDROCARBONS (CCME)

Maxxam ID		TG5607		TG5608		
Sampling Date		2013/09/25 16:00		2013/09/25 16:30		
	Units	**TP1**	**RDL**	**TP2**	**RDL**	**QC Batch**
BTEX & F1 Hydrocarbons						
Benzene	ug/g	ND	0.020	ND	0.020	3373478
Toluene	ug/g	ND	0.020	ND	0.020	3373478
Ethylbenzene	ug/g	ND	0.020	ND	0.020	3373478
o-Xylene	ug/g	ND	0.020	ND	0.020	3373478
p+m-Xylene	ug/g	ND	0.040	ND	0.040	3373478
Total Xylenes	ug/g	ND	0.040	ND	0.040	3373478
F1 (C6-C10)	ug/g	ND	10	ND	10	3373478
F1 (C6-C10) - BTEX	ug/g	ND	10	ND	10	3373478
F2-F4 Hydrocarbons						
F2 (C10-C16 Hydrocarbons)	ug/g	ND	20	ND	10	3372378
F3 (C16-C34 Hydrocarbons)	ug/g	ND	100	ND	50	3372378
F4 (C34-C50 Hydrocarbons)	ug/g	ND	100	ND	50	3372378
Reached Baseline at C50	ug/g	YES		YES		3372378
Surrogate Recovery (%)						
1,4-Difluorobenzene	%	103		103		3373478
4-Bromofluorobenzene	%	87		95		3373478
D10-Ethylbenzene	%	96		94		3373478
D4-1,2-Dichloroethane	%	123		125		3373478
o-Terphenyl	%	88		101		3372378

ND = Not detected
RDL = Reportable Detection Limit
QC Batch = Quality Control Batch

Maxxam Job #: B3G4937
Report Date: 2013/10/04

Golder Associates Ltd
Client Project #: 11-1191-0008
Site Location: PRUNE CREEK BRIDGE, HEARST, ONTARIO
Sampler Initials: SA

Test Summary

Maxxam ID TG5607
Sample ID TP1
Matrix Soil

Collected 2013/09/25
Shipped
Received 2013/09/28

Test Description	Instrumentation	Batch	Extracted	Analyzed	Analyst
Chloride (20:1 extract)	AC/EC	3374274	N/A	2013/10/04	Alina Dobreanu
Cyanide (WAD) in Leachates	TECH/CN	3372489	N/A	2013/10/03	Louise Harding
Petroleum Hydro. CCME F1 & BTEX in Soil	HSGC/MSFD	3373478	2013/10/01	2013/10/03	Domnica Andronescu
Petroleum Hydrocarbons F2-F4 in Soil	GC/FID	3372378	2013/10/03	2013/10/03	Jeevaraj Jeevaratrnam
Fluoride by ISE in Leachates	ISE	3372492	2013/10/03	2013/10/03	Surinder Rai
Mercury (TCLP Leachable) (mg/L)	CVAA	3371492	N/A	2013/10/03	Magdalena Carlos
Total Metals Analysis by ICP	ICP	3372939	2013/10/03	2013/10/03	Suban Kanapathippllai
Total Metals in TCLP Leachate by ICPMS	ICP1/MS	3372683	2013/10/03	2013/10/03	Hua Ren
Moisture	BAL	3370850	N/A	2013/10/02	Valentina Kaftani
Nitrate(NO3) + Nitrite(NO2) in Leachate	LACH	3372490	N/A	2013/10/03	Sandeep Singh
PAH Compounds in Leachate by GC/MS (SI	GC/MS	3371892	2013/10/02	2013/10/03	Darryl Tiller
TCLP - % Solids	BAL	3368817	2013/10/01	2013/10/02	Jian (Ken) Wang
TCLP - Extraction Fluid		3368819	N/A	2013/10/02	Jian (Ken) Wang
TCLP - Initial and final pH	PH	3368820	N/A	2013/10/02	Jian (Ken) Wang

Maxxam ID TG5608
Sample ID TP2
Matrix Soil

Collected 2013/09/25
Shipped
Received 2013/09/28

Test Description	Instrumentation	Batch	Extracted	Analyzed	Analyst
Chloride (20:1 extract)	AC/EC	3374274	N/A	2013/10/04	Alina Dobreanu
Cyanide (WAD) in Leachates	TECH/CN	3372489	N/A	2013/10/03	Louise Harding
Petroleum Hydro. CCME F1 & BTEX in Soil	HSGC/MSFD	3373478	2013/10/01	2013/10/03	Domnica Andronescu
Petroleum Hydrocarbons F2-F4 in Soil	GC/FID	3372378	2013/10/03	2013/10/03	Jeevaraj Jeevaratrnam
Fluoride by ISE in Leachates	ISE	3372492	2013/10/03	2013/10/03	Surinder Rai
Mercury (TCLP Leachable) (mg/L)	CVAA	3371492	N/A	2013/10/03	Magdalena Carlos
Total Metals Analysis by ICP	ICP	3372939	2013/10/03	2013/10/03	Suban Kanapathippllai
Total Metals in TCLP Leachate by ICPMS	ICP1/MS	3372683	2013/10/03	2013/10/03	Hua Ren
Moisture	BAL	3370850	N/A	2013/10/02	Valentina Kaftani
Nitrate(NO3) + Nitrite(NO2) in Leachate	LACH	3372490	N/A	2013/10/03	Sandeep Singh
PAH Compounds in Leachate by GC/MS (SI	GC/MS	3371892	2013/10/02	2013/10/03	Darryl Tiller
TCLP - % Solids	BAL	3368817	2013/10/01	2013/10/02	Jian (Ken) Wang
TCLP - Extraction Fluid		3368819	N/A	2013/10/02	Jian (Ken) Wang

Maxxam Job #: B3G4937
Report Date: 2013/10/04

Golder Associates Ltd
Client Project #: 11-1191-0008
Site Location: PRUNE CREEK BRIDGE, HEARST, ONTARIO
Sampler Initials: SA

Test Summary

TCLP - Initial and final pH	PH	3368820	N/A	2013/10/02	Jian (Ken) Wang

Package 1	6.3°C

Each temperature is the average of up to three cooler temperatures taken at receipt

GENERAL COMMENTS

Sample TG5607-01: F2-F4 Analysis.
Detection limits were adjusted for high moisture content.

Maxxam Job #: B3G4937
Report Date: 2013/10/04

Golder Associates Ltd
Client Project #: 11-1191-0008
Site Location: PRUNE CREEK BRIDGE, HEARST, ONTARIO
Sampler Initials: SA

QUALITY ASSURANCE REPORT

			Matrix Spike		Spiked Blank		Method Blank		RPD		Leachate Blank	
QC Batch	Parameter	Date	% Recovery	QC Limits	% Recovery	QC Limits	Value	Units	Value (%)	QC Limits	Value	Units
3370850	Moisture	2013/10/02							0	20		
3371492	Leachable Mercury (Hg)	2013/10/03	98	80 - 120	99	80 - 120	ND, RDL=0.001	mg/L	NC	25	ND	mg/L
3371892	Leachable D10-Anthracene	2013/10/03	103	50 - 130	103	50 - 130	103	%				
3371892	Leachable D14-Terphenyl (FS)	2013/10/03	89	50 - 130	90	50 - 130	89	%				
3371892	Leachable D8-Acenaphthylene	2013/10/03	96	50 - 130	98	50 - 130	95	%				
3371892	Leachable Benzo(a)pyrene	2013/10/03	102	50 - 130	101	50 - 130	0.06, RDL=0.04	ug/L				
3372378	o-Terphenyl	2013/10/03	100	50 - 130	98	50 - 130	100	%				
3372378	F2 (C10-C16 Hydrocarbons)	2013/10/03	101	50 - 130	100	80 - 120	ND, RDL=10	ug/g	NC	30		
3372378	F3 (C16-C34 Hydrocarbons)	2013/10/03	104	50 - 130	101	80 - 120	ND, RDL=50	ug/g	NC	30		
3372378	F4 (C34-C50 Hydrocarbons)	2013/10/03	102	50 - 130	100	80 - 120	ND, RDL=50	ug/g	NC	30		
3372489	Leachable Free Cyanide	2013/10/03	88	80 - 120	101	80 - 120	ND, RDL=0.002	mg/L	NC	20	ND	mg/L
3372490	Leachable Nitrite (N)	2013/10/03	100	80 - 120	100	85 - 115	ND, RDL=0.1	mg/L	NC	25	ND	mg/L
3372490	Leachable Nitrate (N)	2013/10/03	100	80 - 120	97	85 - 115	ND, RDL=1	mg/L	NC	25	ND	mg/L
3372490	Leachable Nitrate + Nitrite	2013/10/03	100	80 - 120	98	85 - 115	ND, RDL=1	mg/L	NC	25	ND	mg/L
3372492	Leachable Fluoride (F-)	2013/10/03	113	80 - 120	100	80 - 120	ND, RDL=0.1	mg/L	NC	25	ND	mg/L
3372683	Leachable Arsenic (As)	2013/10/03	105	75 - 125	105	75 - 125			NC	35	ND	mg/L
3372683	Leachable Barium (Ba)	2013/10/03	108	75 - 125	108	75 - 125			NC	35	ND	mg/L
3372683	Leachable Boron (B)	2013/10/03	105	75 - 125	120	75 - 125			NC	35	ND	mg/L
3372683	Leachable Cadmium (Cd)	2013/10/03	105	75 - 125	105	75 - 125			NC	35	ND	mg/L
3372683	Leachable Chromium (Cr)	2013/10/03	105	75 - 125	105	75 - 125			NC	35	ND	mg/L
3372683	Leachable Lead (Pb)	2013/10/03	105	75 - 125	108	75 - 125			NC	35	ND	mg/L
3372683	Leachable Selenium (Se)	2013/10/03	107	75 - 125	108	75 - 125			NC	35	ND	mg/L
3372683	Leachable Silver (Ag)	2013/10/03	102	75 - 125	103	75 - 125			NC	35	ND	mg/L
3372683	Leachable Uranium (U)	2013/10/03	107	75 - 125	107	75 - 125			NC	35	ND	mg/L
3372939	Acid Extractable Sodium (Na)	2013/10/03	103	75 - 125	100	80 - 120	ND, RDL=100	ug/g				
3373478	1,4-Difluorobenzene	2013/10/03	106	60 - 140	106	60 - 140	106	%				
3373478	4-Bromofluorobenzene	2013/10/03	110	60 - 140	109	60 - 140	102	%				
3373478	D10-Ethylbenzene	2013/10/03	97	60 - 140	90	60 - 140	89	%				
3373478	D4-1,2-Dichloroethane	2013/10/03	125	60 - 140	122	60 - 140	124	%				
3373478	Benzene	2013/10/03	99	60 - 140	100	60 - 130	ND, RDL=0.020	ug/g	NC	50		
3373478	Toluene	2013/10/03	97	60 - 140	99	60 - 130	ND, RDL=0.020	ug/g	NC	50		
3373478	Ethylbenzene	2013/10/03	115	60 - 140	121	60 - 130	ND, RDL=0.020	ug/g	NC	50		
3373478	o-Xylene	2013/10/03	119	60 - 140	124	60 - 130	ND, RDL=0.020	ug/g	NC	50		
3373478	p+m-Xylene	2013/10/03	107	60 - 140	112	60 - 130	ND, RDL=0.040	ug/g	NC	50		
3373478	F1 (C6-C10)	2013/10/03	90	60 - 140	91	80 - 120	ND, RDL=10	ug/g	NC	50		
3373478	Total Xylenes	2013/10/03					ND, RDL=0.040	ug/g	NC	50		

Maxxam Analytics International Corporation o/a Maxxam Analytics 6740 Campobello Road, Mississauga, Ontario, L5N 2L8 Tel: (905) 817-5700 Toll-Free: 800-563-6266 Fax: (905) 817-5777 www.maxxam.ca

Maxxam Job #: B3G4937
Report Date: 2013/10/04

Golder Associates Ltd
Client Project #: 11-1191-0008
Site Location: PRUNE CREEK BRIDGE, HEARST, ONTARIO
Sampler Initials: SA

QUALITY ASSURANCE REPORT

			Matrix Spike		Spiked Blank		Method Blank		RPD		Leachate Blank	
QC Batch	**Parameter**	**Date**	**% Recovery**	**QC Limits**	**% Recovery**	**QC Limits**	**Value**	**Units**	**Value (%)**	**QC Limits**	**Value**	**Units**
3373478	F1 (C6-C10) - BTEX	2013/10/03					ND, RDL=10	ug/g	NC	50		
3374274	Soluble (20:1) Chloride (Cl)	2013/10/04	107	75 - 125	100	75 - 125	ND, RDL=20	ug/g	NC	35		

N/A = Not Applicable
RDL = Reportable Detection Limit
RPD = Relative Percent Difference
Duplicate: Paired analysis of a separate portion of the same sample. Used to evaluate the variance in the measurement.
Matrix Spike: A sample to which a known amount of the analyte of interest has been added. Used to evaluate sample matrix interference.
Leachate Blank: A blank matrix containing all reagents used in the leaching procedure. Used to determine any process contamination.
Spiked Blank: A blank matrix sample to which a known amount of the analyte, usually from a second source, has been added. Used to evaluate method accuracy.
Method Blank: A blank matrix containing all reagents used in the analytical procedure. Used to identify laboratory contamination.
Surrogate: A pure or isotopically labeled compound whose behavior mirrors the analytes of interest. Used to evaluate extraction efficiency.
NC (RPD): The RPD was not calculated. The level of analyte detected in the parent sample and its duplicate was not sufficiently significant to permit a reliable calculation.

Validation Signature Page

Maxxam Job #: B3G4937

The analytical data and all QC contained in this report were reviewed and validated by the following individual(s).

Cristina Carriere

Cristina Carriere, Scientific Services

M. Riskallah

Medhat Riskallah, Manager, Hydrocarbon Department

Ming Wang

Michael Wang, Senior Analyst

Suzana Popovic

Suzana Popovic, Supervisor, Hydrocarbons

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Your Project #: 11-1191-0008
Site#: 11-1191-0008
Site Location: PRUNE CREEK BRIDGE, HEARST, ONTARIO
Your C.O.C. #: na

Attention: David Muldowney
Golder Associates Ltd
1010 Lorne St
Sudbury, ON
P3C 4R9

Report Date: 2013/10/17

CERTIFICATE OF ANALYSIS

MAXXAM JOB #: B3H2928
Received: 2013/10/10, 13:11

Sample Matrix: Soil
Samples Received: 2

Analyses	Quantity	Date Extracted	Date Analyzed	Laboratory Method	Method Reference
Acid Extr. Metals (aqua regia) by ICPMS	2	2013/10/17	2013/10/17	CAM SOP-00447	EPA 6020
Polychlorinated Biphenyl in Leachate	2	2013/10/16	2013/10/16	CAM SOP-00309	SW846 8082
TCLP - % Solids	2	2013/10/15	2013/10/16	CAM SOP-00401	EPA 1311 modified
TCLP - Extraction Fluid	2	N/A	2013/10/16	CAM SOP-00401	EPA 1311 modified
TCLP - Initial and final pH	2	N/A	2013/10/16	CAM SOP-00401	EPA 1311 modified
TCLP Zero Headspace Extraction	2	2013/10/11	2013/10/11	CAM SOP-00430	EPA 1311 modified
VOCs in ZHE Leachates	2	2013/10/15	2013/10/15	CAM SOP 00226	EPA 8260 modified

Remarks:

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Maxxam Analytics is accredited for all specific parameters as required by Ontario Regulation 153/04. Maxxam Analytics is limited in liability to the actual cost of analysis unless otherwise agreed in writing. There is no other warranty expressed or implied. Samples will be retained at Maxxam Analytics for three weeks from receipt of data or as per contract.

* RPDs calculated using raw data. The rounding of final results may result in the apparent difference.
* Results relate only to the items tested.

Maxxam Job #: B3H2928
Report Date: 2013/10/17

Golder Associates Ltd
Client Project #: 11-1191-0008
Site Location: PRUNE CREEK BRIDGE, HEARST, ONTARIO
Sampler Initials: DM

Encryption Key

Please direct all questions regarding this Certificate of Analysis to your Project Manager.

Antonella Brasil, Project Manager
Email: ABrasil@maxxam.ca
Phone# (905) 817-5817

==
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Total cover pages: 2

Maxxam Job #: B3H2928
Report Date: 2013/10/17

Golder Associates Ltd
Client Project #: 11-1191-0008
Site Location: PRUNE CREEK BRIDGE, HEARST, ONTARIO
Sampler Initials: DM

RESULTS OF ANALYSES OF SOIL

Maxxam ID		TK7235	TK7236		
Sampling Date		2013/09/25 16:00	2013/09/25 16:30		
	Units	**TP 1**	**TP 2**	**RDL**	**QC Batch**
Charge/Prep Analysis					
Amount Extracted (Wet Weight) (g)	N/A	25	25	N/A	3382849
Inorganics					
Final pH	pH	6.18	7.36		3387186
Initial pH	pH	9.07	9.13		3387186
TCLP - % Solids	%	100	100	0.2	3387180
TCLP Extraction Fluid	N/A	FLUID 1	FLUID 1		3387185

N/A = Not Applicable
RDL = Reportable Detection Limit
QC Batch = Quality Control Batch

Maxxam Job #: B3H2928
Report Date: 2013/10/17

Golder Associates Ltd
Client Project #: 11-1191-0008
Site Location: PRUNE CREEK BRIDGE, HEARST, ONTARIO
Sampler Initials: DM

ELEMENTS BY ATOMIC SPECTROSCOPY (SOIL)

Maxxam ID		TK7235	TK7236		
Sampling Date		2013/09/25 16:00	2013/09/25 16:30		
	Units	**TP 1**	**TP 2**	**RDL**	**QC Batch**
Metals					
Acid Extractable Aluminum (Al)	ug/g	18000	9900	50	3388232
Acid Extractable Antimony (Sb)	ug/g	0.24	ND	0.20	3388232
Acid Extractable Arsenic (As)	ug/g	4.0	2.1	1.0	3388232
Acid Extractable Barium (Ba)	ug/g	120	57	0.50	3388232
Acid Extractable Beryllium (Be)	ug/g	0.85	0.49	0.20	3388232
Acid Extractable Bismuth (Bi)	ug/g	ND	ND	1.0	3388232
Acid Extractable Boron (B)	ug/g	12	9.4	5.0	3388232
Acid Extractable Cadmium (Cd)	ug/g	0.13	0.10	0.10	3388232
Acid Extractable Calcium (Ca)	ug/g	94000	130000	50	3388232
Acid Extractable Chromium (Cr)	ug/g	52	32	1.0	3388232
Acid Extractable Cobalt (Co)	ug/g	14	8.1	0.10	3388232
Acid Extractable Copper (Cu)	ug/g	30	17	0.50	3388232
Acid Extractable Iron (Fe)	ug/g	31000	18000	50	3388232
Acid Extractable Lead (Pb)	ug/g	12	6.6	1.0	3388232
Acid Extractable Magnesium (Mg)	ug/g	23000	29000	50	3388232
Acid Extractable Manganese (Mn)	ug/g	520	470	1.0	3388232
Acid Extractable Molybdenum (Mo)	ug/g	0.60	ND	0.50	3388232
Acid Extractable Nickel (Ni)	ug/g	36	21	0.50	3388232
Acid Extractable Phosphorus (P)	ug/g	540	490	50	3388232
Acid Extractable Potassium (K)	ug/g	4000	2100	200	3388232
Acid Extractable Selenium (Se)	ug/g	ND	ND	0.50	3388232
Acid Extractable Silver (Ag)	ug/g	ND	ND	0.20	3388232
Acid Extractable Sodium (Na)	ug/g	320	210	100	3388232
Acid Extractable Strontium (Sr)	ug/g	100	98	1.0	3388232
Acid Extractable Thallium (Tl)	ug/g	0.21	0.12	0.050	3388232
Acid Extractable Tin (Sn)	ug/g	ND	ND	5.0	3388232
Acid Extractable Uranium (U)	ug/g	1.4	1.0	0.050	3388232
Acid Extractable Vanadium (V)	ug/g	48	30	5.0	3388232
Acid Extractable Zinc (Zn)	ug/g	74	42	5.0	3388232
Acid Extractable Mercury (Hg)	ug/g	ND	ND	0.050	3388232

ND = Not detected
RDL = Reportable Detection Limit
QC Batch = Quality Control Batch

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Maxxam Job #: B3H2928
Report Date: 2013/10/17

Golder Associates Ltd
Client Project #: 11-1191-0008
Site Location: PRUNE CREEK BRIDGE, HEARST, ONTARIO
Sampler Initials: DM

VOLATILE ORGANICS BY GC/MS (SOIL)

Maxxam ID		TK7235	TK7236		
Sampling Date		2013/09/25 16:00	2013/09/25 16:30		
	Units	**TP 1**	**TP 2**	**RDL**	**QC Batch**
Volatile Organics					
Leachable Benzene	mg/L	ND	ND	0.020	3385488
Leachable Carbon Tetrachloride	mg/L	ND	ND	0.020	3385488
Leachable Chlorobenzene	mg/L	ND	ND	0.020	3385488
Leachable Chloroform	mg/L	ND	ND	0.020	3385488
Leachable 1,2-Dichlorobenzene	mg/L	ND	ND	0.050	3385488
Leachable 1,4-Dichlorobenzene	mg/L	ND	ND	0.050	3385488
Leachable 1,2-Dichloroethane	mg/L	ND	ND	0.050	3385488
Leachable 1,1-Dichloroethylene	mg/L	ND	ND	0.020	3385488
Leachable Methylene Chloride(Dichloromethane)	mg/L	ND	ND	0.20	3385488
Leachable Methyl Ethyl Ketone (2-Butanone)	mg/L	ND	ND	1.0	3385488
Leachable Tetrachloroethylene	mg/L	ND	ND	0.020	3385488
Leachable Trichloroethylene	mg/L	ND	ND	0.020	3385488
Leachable Vinyl Chloride	mg/L	ND	ND	0.020	3385488
Surrogate Recovery (%)					
Leachable 4-Bromofluorobenzene	%	96	95		3385488
Leachable D4-1,2-Dichloroethane	%	79	77		3385488
Leachable D8-Toluene	%	104	104		3385488

POLYCHLORINATED BIPHENYLS BY GC-ECD (SOIL)

Maxxam ID		TK7235	TK7236		
Sampling Date		2013/09/25 16:00	2013/09/25 16:30		
	Units	**TP 1**	**TP 2**	**RDL**	**QC Batch**
PCBs					
Leachable Total PCB	ug/L	ND	ND	3	3387210
Surrogate Recovery (%)					
Leachable Decachlorobiphenyl	%	126	120		3387210

ND = Not detected
RDL = Reportable Detection Limit
QC Batch = Quality Control Batch

Maxxam Job #: B3H2928
Report Date: 2013/10/17

Golder Associates Ltd
Client Project #: 11-1191-0008
Site Location: PRUNE CREEK BRIDGE, HEARST, ONTARIO
Sampler Initials: DM

Test Summary

Maxxam ID TK7235
Sample ID TP 1
Matrix Soil

Collected 2013/09/25
Shipped
Received 2013/10/10

Test Description	Instrumentation	Batch	Extracted	Analyzed	Analyst
Acid Extr. Metals (aqua regia) by ICPMS	ICP/MS	3388232	2013/10/17	2013/10/17	John Bowman
Polychlorinated Biphenyl in Leachate	GC/ECD	3387210	2013/10/16	2013/10/16	Sarah Huang
TCLP - % Solids	BAL	3387180	2013/10/15	2013/10/16	Jian (Ken) Wang
TCLP - Extraction Fluid		3387185	N/A	2013/10/16	Jian (Ken) Wang
TCLP - Initial and final pH	PH	3387186	N/A	2013/10/16	Jian (Ken) Wang
TCLP Zero Headspace Extraction		3382849	2013/10/11	2013/10/11	Walt Wang
VOCs in ZHE Leachates	GC/MS	3385488	2013/10/15	2013/10/15	Edwin Ayala

Maxxam ID TK7236
Sample ID TP 2
Matrix Soil

Collected 2013/09/25
Shipped
Received 2013/10/10

Test Description	Instrumentation	Batch	Extracted	Analyzed	Analyst
Acid Extr. Metals (aqua regia) by ICPMS	ICP/MS	3388232	2013/10/17	2013/10/17	John Bowman
Polychlorinated Biphenyl in Leachate	GC/ECD	3387210	2013/10/16	2013/10/16	Sarah Huang
TCLP - % Solids	BAL	3387180	2013/10/15	2013/10/16	Jian (Ken) Wang
TCLP - Extraction Fluid		3387185	N/A	2013/10/16	Jian (Ken) Wang
TCLP - Initial and final pH	PH	3387186	N/A	2013/10/16	Jian (Ken) Wang
TCLP Zero Headspace Extraction		3382849	2013/10/11	2013/10/11	Walt Wang
VOCs in ZHE Leachates	GC/MS	3385488	2013/10/15	2013/10/15	Edwin Ayala

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GENERAL COMMENTS

Sample TK7235-01: TCLP VOCs Extraction: Sample(s) analyzed past hold time. Analysis performed with client's consent.

Sample TK7236-01: TCLP VOCs Extraction: Sample(s) analyzed past hold time. Analysis performed with client's consent.

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Maxxam

Maxxam Job #: B3H2928
Report Date: 2013/10/17

Golder Associates Ltd
Client Project #: 11-1191-0008
Site Location: PRUNE CREEK BRIDGE, HEARST, ONTARIO
Sampler Initials: DM

QUALITY ASSURANCE REPORT

			Matrix Spike		Spiked Blank		Method Blank		RPD	
QC Batch	Parameter	Date	% Recovery	QC Limits	% Recovery	QC Limits	Value	Units	Value (%)	QC Limits
3385488	Leachable 4-Bromofluorobenzene	2013/10/15	100	70 - 130	104	70 - 130	96	%		
3385488	Leachable D4-1,2-Dichloroethane	2013/10/15	72	70 - 130	81	70 - 130	79	%		
3385488	Leachable D8-Toluene	2013/10/15	110	70 - 130	107	70 - 130	106	%		
3385488	Leachable Benzene	2013/10/15	87	70 - 130	96	70 - 130	ND, RDL=0.020	mg/L	NC	30
3385488	Leachable Carbon Tetrachloride	2013/10/15	97	70 - 130	105	70 - 130	ND, RDL=0.020	mg/L	NC	30
3385488	Leachable Chlorobenzene	2013/10/15	101	70 - 130	105	70 - 130	ND, RDL=0.020	mg/L	NC	30
3385488	Leachable Chloroform	2013/10/15	82	70 - 130	91	70 - 130	ND, RDL=0.020	mg/L	NC	30
3385488	Leachable 1,2-Dichlorobenzene	2013/10/15	97	70 - 130	99	70 - 130	ND, RDL=0.050	mg/L	NC	30
3385488	Leachable 1,4-Dichlorobenzene	2013/10/15	98	70 - 130	101	70 - 130	ND, RDL=0.050	mg/L	NC	30
3385488	Leachable 1,2-Dichloroethane	2013/10/15	81	70 - 130	93	70 - 130	ND, RDL=0.050	mg/L	NC	30
3385488	Leachable 1,1-Dichloroethylene	2013/10/15	99	70 - 130	107	70 - 130	ND, RDL=0.020	mg/L	NC	30
3385488	Leachable Methylene Chloride(Dichloromethane)	2013/10/15	84	70 - 130	92	70 - 130	ND, RDL=0.20	mg/L	NC	30
3385488	Leachable Methyl Ethyl Ketone (2-Butanone)	2013/10/15	68	60 - 140	80	60 - 140	ND, RDL=1.0	mg/L	NC	30
3385488	Leachable Tetrachloroethylene	2013/10/15	112	70 - 130	113	70 - 130	ND, RDL=0.020	mg/L	NC	30
3385488	Leachable Trichloroethylene	2013/10/15	91	70 - 130	99	70 - 130	ND, RDL=0.020	mg/L	NC	30
3385488	Leachable Vinyl Chloride	2013/10/15	90	70 - 130	95	70 - 130	ND, RDL=0.020	mg/L	NC	30
3387210	Leachable Decachlorobiphenyl	2013/10/16	126	60 - 130	121	60 - 130	124	%		
3387210	Leachable Total PCB	2013/10/16	106	60 - 130	101	60 - 130	ND, RDL=3	ug/L	NC	40
3388232	Acid Extractable Aluminum (Al)	2013/10/17	NC	75 - 125	101	80 - 120	ND, RDL=50	ug/g		
3388232	Acid Extractable Antimony (Sb)	2013/10/17	102	75 - 125	99	80 - 120	ND, RDL=0.20	ug/g	NC	30
3388232	Acid Extractable Arsenic (As)	2013/10/17	103	75 - 125	98	80 - 120	ND, RDL=1.0	ug/g	5.0	30
3388232	Acid Extractable Barium (Ba)	2013/10/17	97	75 - 125	96	80 - 120	ND, RDL=0.50	ug/g	2.7	30
3388232	Acid Extractable Beryllium (Be)	2013/10/17	105	75 - 125	95	80 - 120	ND, RDL=0.20	ug/g	NC	30
3388232	Acid Extractable Bismuth (Bi)	2013/10/17	102	75 - 125	102	80 - 120	ND, RDL=1.0	ug/g		
3388232	Acid Extractable Boron (B)	2013/10/17	101	75 - 125	93	80 - 120	ND, RDL=5.0	ug/g	NC	30
3388232	Acid Extractable Cadmium (Cd)	2013/10/17	102	75 - 125	101	80 - 120	ND, RDL=0.10	ug/g	NC	30
3388232	Acid Extractable Calcium (Ca)	2013/10/17	NC	75 - 125	102	80 - 120	ND, RDL=50	ug/g		
3388232	Acid Extractable Chromium (Cr)	2013/10/17	105	75 - 125	100	80 - 120	ND, RDL=1.0	ug/g	0.06	30
3388232	Acid Extractable Cobalt (Co)	2013/10/17	104	75 - 125	101	80 - 120	ND, RDL=0.10	ug/g	8.7	30
3388232	Acid Extractable Copper (Cu)	2013/10/17	103	75 - 125	99	80 - 120	ND, RDL=0.50	ug/g	4.9	30
3388232	Acid Extractable Iron (Fe)	2013/10/17	NC	75 - 125	105	80 - 120	ND, RDL=50	ug/g		
3388232	Acid Extractable Lead (Pb)	2013/10/17	NC(1)	75 - 125	102	80 - 120	ND, RDL=1.0	ug/g	5.6	30
3388232	Acid Extractable Magnesium (Mg)	2013/10/17	NC	75 - 125	95	80 - 120	ND, RDL=50	ug/g		
3388232	Acid Extractable Manganese (Mn)	2013/10/17	NC	75 - 125	99	80 - 120	ND, RDL=1.0	ug/g		
3388232	Acid Extractable Molybdenum (Mo)	2013/10/17	109	75 - 125	104	80 - 120	ND, RDL=0.50	ug/g	NC	30
3388232	Acid Extractable Nickel (Ni)	2013/10/17	103	75 - 125	102	80 - 120	ND, RDL=0.50	ug/g	6.4	30
3388232	Acid Extractable Phosphorus (P)	2013/10/17	NC	75 - 125	89	80 - 120	ND, RDL=50	ug/g		
3388232	Acid Extractable Potassium (K)	2013/10/17	104	75 - 125	96	80 - 120	ND, RDL=200	ug/g		
3388232	Acid Extractable Selenium (Se)	2013/10/17	105	75 - 125	100	80 - 120	ND, RDL=0.50	ug/g	NC	30
3388232	Acid Extractable Silver (Ag)	2013/10/17	103	75 - 125	102	80 - 120	ND, RDL=0.20	ug/g	NC	30

Maxxam Job #: B3H2928
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Golder Associates Ltd
Client Project #: 11-1191-0008
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QUALITY ASSURANCE REPORT

			Matrix Spike		Spiked Blank		Method Blank		RPD	
QC Batch	Parameter	Date	% Recovery	QC Limits	% Recovery	QC Limits	Value	Units	Value (%)	QC Limits
3388232	Acid Extractable Sodium (Na)	2013/10/17	104	75 - 125	94	80 - 120	ND, RDL=100	ug/g		
3388232	Acid Extractable Strontium (Sr)	2013/10/17	NC	75 - 125	100	80 - 120	ND, RDL=1.0	ug/g		
3388232	Acid Extractable Thallium (Tl)	2013/10/17	88	75 - 125	93	80 - 120	ND, RDL=0.050	ug/g	NC	30
3388232	Acid Extractable Tin (Sn)	2013/10/17	105	75 - 125	100	80 - 120	ND, RDL=5.0	ug/g		
3388232	Acid Extractable Uranium (U)	2013/10/17	107	75 - 125	104	80 - 120	ND, RDL=0.050	ug/g	2.6	30
3388232	Acid Extractable Vanadium (V)	2013/10/17	105	75 - 125	98	80 - 120	ND, RDL=5.0	ug/g	NC	30
3388232	Acid Extractable Zinc (Zn)	2013/10/17	NC(1)	75 - 125	101	80 - 120	ND, RDL=5.0	ug/g	1.2	30
3388232	Acid Extractable Mercury (Hg)	2013/10/17	99	75 - 125	111	80 - 120	ND, RDL=0.050	ug/g	NC	30

N/A = Not Applicable
RDL = Reportable Detection Limit
RPD = Relative Percent Difference
Duplicate: Paired analysis of a separate portion of the same sample. Used to evaluate the variance in the measurement.
Matrix Spike: A sample to which a known amount of the analyte of interest has been added. Used to evaluate sample matrix interference.
Spiked Blank: A blank matrix sample to which a known amount of the analyte, usually from a second source, has been added. Used to evaluate method accuracy.
Method Blank: A blank matrix containing all reagents used in the analytical procedure. Used to identify laboratory contamination.
Surrogate: A pure or isotopically labeled compound whose behavior mirrors the analytes of interest. Used to evaluate extraction efficiency.
NC (Matrix Spike): The recovery in the matrix spike was not calculated. The relative difference between the concentration in the parent sample and the spiked amount was not sufficiently significant to permit a reliable recovery calculation.
NC (RPD): The RPD was not calculated. The level of analyte detected in the parent sample and its duplicate was not sufficiently significant to permit a reliable calculation.
(1) - The recovery in the matrix spike was not calculated (NC). Spiked concentration was less than 2x that native to the sample.

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Validation Signature Page

Maxxam Job #: B3H2928

The analytical data and all QC contained in this report were reviewed and validated by the following individual(s).

Charles Ancker, B.Sc., M.Sc., C.Chem, Senior Analyst

Ewa Pranjic, M.Sc., C.Chem, Scientific Specialist

===

Maxxam has procedures in place to guard against improper use of the electronic signature and have the required "signatories", as per section 5.10.2 of ISO/IEC 17025:2005(E), signing the reports. For Service Group specific validation please refer to the Validation Signature Page.

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APPENDIX D

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:

Criteria	**Tolerance**
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 SS103-11) once the pile reaches a depth of 3.0 m above the design pile tip elevation shown in the Contract Drawings and at subsequent 0.5 m intervals of depth until the ultimate axial resistance is achieved. If the ultimate axial resistance as determined by the Hiley Formula is not achieved within the 3.0 m interval down to the design pile tip elevation the Contractor shall stop pile driving and notify the Contract Administrator. At this depth the pile should be allowed to rest for 48 hours, and the Hiley Formula shall then be applied immediately upon re-striking of the pile. If the ultimate axial resistance is still not achieved after the 48 hour wait period, the Contract Administrator shall be notified and authorization given prior to driving the pile below the design pile tip elevation.

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

OBSTRUCTIONS

Non-Standard Special Provision

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

Basis of Payment

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

END OF SECTION

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

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