

**FOUNDATION INVESTIGATION AND DESIGN REPORT
STILLWATER CREEK BRIDGE WBL
HIGHWAY 11/17 RED ROCK TO NIPIGON
FROM 4.8 KM WEST OF HWY 628 TO 1.5 KM WEST OF HWY 585
DISTRICT OF THUNDER BAY, ONTARIO**

G.W.P. 647-89-00, Site No. 48C-096

Geocres Number: 52H-20

Report to

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PART 1: FACTUAL INFORMATION

1 INTRODUCTION

This report presents the factual findings obtained from a foundation investigation conducted at the location of a proposed bridge which will carry the new westbound lanes (WBL) of Highway 11/17 over Stillwater Creek in the Township of Nipigon. The proposed bridge is part of the Highway 11/17 four-laning project, involving construction of a divided highway from 4.8 km west of Highway 628 to 1.5 km west of Highway 585 in the District of Thunder Bay.

The purpose of this investigation was to explore the subsurface conditions at the site and, based on the data obtained, to provide a borehole location plan, records of boreholes, stratigraphic profile and cross-sections, laboratory test results and a written description of the subsurface conditions. A model of the subsurface conditions was developed from the data obtained in the course of the investigation.

Thurber carried out the investigation as a sub-consultant to Hatch Mott MacDonald (HMM), under the Ministry of Transportation Ontario (MTO) Agreement Number 6010-E-0006.

2 SITE DESCRIPTION

The site is located approximately 4 km west of Nipigon, Ontario, and approximately 2 km south-west of the intersection of Highway 11/17 and Highway 585. The proposed four-laning of Highway 11/17 will include construction of a new bridge to carry the westbound lanes of the highway over Stillwater Creek. The existing Highway 11/17 bridge, located immediately to the south, will carry the eastbound lanes.

Stillwater Creek at the site meanders within a flood plain, generally flowing towards the southeast into the Nipigon River. The surrounding area is typically heavily treed. A tent and trailer park is located on the north side of the east approach.

The existing Highway 11/17 bridge is a three-span steel girder structure supported on concrete abutments and piers. The bridge approaches consist of fill embankments placed within the creek floodplain. The embankment height is in the order of 4 m on the north side of the east approach and 7 to 8 m in the remaining quadrants.

Photographs in Appendix C show the general nature of the site.

The site lies within the physiographic region known as the Quetico Subprovince of the Superior Province of the Canadian Shield. The region is characterized by early Precambrian felsic igneous (granite) and metamorphic (granitic gneiss) bedrock. The surficial deposits at the site typically comprise glaciofluvial sand deposits.

3 SITE INVESTIGATION AND FIELD TESTING

The site investigation and field testing for this project were carried out during the period June 20 to June 27, 2012 and consisted of drilling and sampling ten boreholes identified as SCW-01 to SCW-10. The approximate locations of the boreholes are shown on the attached Borehole Locations and Soil Strata Drawing in Appendix G.

The borehole locations were selected on the basis of a conceptual three-span structure being considered at the time of investigation. Boreholes SCW-01 and SCW-10 were drilled at the proposed west and east approaches to auger refusal at depths of 3.0 m and 12.7 m (Elev. 263.3 m and 255.8 m), respectively. Boreholes SCW-02 to SCW-05 were drilled at the proposed west abutment and piers, and Boreholes SCW-06 to SCW-09 were drilled at the east abutment and piers. These boreholes were terminated at depths of 7.6 m to 12.1 m (Elev. 257.3 to 255.3 m), and included recovery of 2.8 to 3.4 m of rock core from each borehole.

The borehole locations were marked in the field and utility clearances were obtained prior to drilling.

A track-mounted CME-45 drill rig was used to drill the boreholes. A combination of hollow-stem augers, NW casing, and NQ coring methods were used to advance the boreholes. Soil samples were obtained at selected intervals using a split spoon sampler in conjunction with Standard Penetration Testing (SPT). All rock cores were logged, and the Total Core Recovery (TCR), Rock Quality Designation (RQD) and the Fracture Indices (FI) were determined.

The drilling and sampling operations were supervised on a full time basis by a member of Thurber's technical staff. The supervisor logged the boreholes and processed the recovered soil and rock samples for transport to Thurber's laboratory for further examination and testing.

Groundwater conditions in the open boreholes were observed throughout the drilling operations. Standpipe piezometers consisting of 19 mm PVC pipe with slotted screen enclosed in filter sand

were installed in four boreholes to monitor groundwater levels. The piezometers were subsequently decommissioned in general accordance with MOE Regulation 903. Boreholes without piezometers were backfilled in general accordance with Regulation 903. The installation and completion details of the boreholes and piezometers are summarized in Table 3.1.

Table 3.1 – Boreholes and Piezometers Completion Details

Borehole	Piezometer Tip Depth/ Elevation (m)	Borehole Completion Details
SCW-01	None installed	Borehole backfilled with cuttings from 3.0 m to surface.
SCW-02	4.8 / 259.8	Sand filter from 7.8 m to 2.7 m, bentonite holeplug from 2.7 m to surface.
SCW-03	None installed	Borehole backfilled with bentonite holeplug from 7.8 m to surface.
SCW-04	None installed	Borehole backfilled with bentonite holeplug from 7.6 m to 2.8 m, cuttings from 2.8 m to surface.
SCW-05	7.6 / 256.1	Sand filter from 7.6 m to 2.4 m, bentonite holeplug from 2.4 m to surface.
SCW-06	None installed	Borehole backfilled with bentonite holeplug and cuttings from 10.0 m to surface.
SCW-07	7.6 / 259.0	Sand filter from 10.7 m to 5.5 m, bentonite holeplug from 5.5 m to surface.
SCW-08	7.6 / 259.5	Sand filter from 10.6 m to 5.6 m, bentonite holeplug from 5.6 m to surface.
SCW-09	None installed	Borehole backfilled with bentonite holeplug and cuttings from 12.1 m to surface.
SCW-10	None installed	Borehole caved from 12.7 m to 3.7 m upon completion, then backfilled with cuttings from 3.7 m to surface.

4 LABORATORY TESTING

The recovered soil samples were subjected to Visual Identification (VI) and to natural moisture content determinations. Selected samples were also subjected to grain size distribution analyses (sieve and hydrometer) and Atterberg Limits testing, where appropriate. The results of this testing program are summarized on the Record of Borehole Sheets in Appendix A and shown on the figures in Appendix B.

Point load tests were carried out on selected samples of intact bedrock to evaluate the unconfined compressive strength (UCS) of the bedrock. The UCS values assessed from the point load data are reported on the Record of Borehole Sheets in Appendix A as an average UCS value per core run.

5 DESCRIPTION OF SUBSURFACE CONDITIONS

Reference is made to the Record of Borehole Sheets in Appendix A and the Borehole Locations and Soil Strata drawings in Appendix G. An overall description of the stratigraphy is given in the following paragraphs. However, the factual data presented in the Record of Borehole Sheets governs any interpretation of the site conditions. It must be recognized that soil conditions may vary between and beyond borehole locations.

The subsurface stratigraphy at this site varies, consisting of discontinuous layers of surficial topsoil, fill and/or peat, overlying successive layers of sand to silty sand, sand and gravel, silt to sandy silt, and silty clay of variable thickness and continuity. These deposits are typically underlain by a layer of sand, gravel and cobbles overlying bedrock. More detailed descriptions of the individual strata encountered along the proposed bridge alignment are presented below.

5.1 Topsoil

Topsoil was encountered at the ground surface in all boreholes except for Boreholes SCW-06 and SCW-08. The topsoil layer was between 25mm and 200mm thick. The topsoil thickness may vary between the borehole locations and in other areas of the site.

5.2 Sand Fill

Brown sand to silty sand fill containing trace to some gravel was encountered beneath the topsoil in Boreholes SCW-02 and SCW-03. The thickness of the fill layer was 0.6 m and 1.3 m, and the base of the fill was at depths of 0.8 m and 1.5 m (Elev. 263.8 and 263.6).

SPT 'N' values recorded in the sand fill ranged from 8 to 21 blows per 0.3 m of penetration, indicating a loose to compact relative density. The moisture content of the fill samples ranged from 11% to 21%.

A grain size distribution analysis was carried out on one sample. The results of the test are plotted on Figure B1 of Appendix B, and are summarized below:

Gravel %	17
Sand %	78
Silt and Clay %	5

5.3 Peat

A layer of black to dark brown sandy peat was encountered beneath the silty sand fill in Boreholes SCW-02 and SCW-03. The thickness of peat layer was 0.3 m and 0.8 m, with a lower boundary at depths of 1.1 m and 2.3 m (Elev. 263.5 m and 262.8 m).

An SPT 'N'-value of 7 blows per 0.3 m of penetration was recorded, and moisture contents of 49% and 78% were measured.

5.4 Sand to Silty Sand

Sand to silty sand was encountered below the peat in Borehole SCW-02 and below the topsoil in all other boreholes except Borehole SCW-03. On the west side of the creek, the sand layer ranged from 0.4 m to 2.5 m in thickness, with a lower boundary at depths of 0.6 m to 2.6 m (Elev. 262.5 m to 263.7 m). On the east side, the sand layer was 4.1 m to 7.1 m thick (9.0 m in Borehole SCW-10 but interrupted by a 1.7 m sand and gravel layer), and the lower boundary was at depths of 4.1 to 9.1 m (Elev. 262.5 to 259.4).

The sand to silty sand locally contained trace to some gravel, and contained organics and wood fragments in Boreholes SCW-01, SCW-02 and SCW-04.

SPT 'N' values recorded in the sand typically ranged from 4 to 26 blows per 0.3 m penetration, indicating a loose to compact relative density. "N" values of 34 blows per 0.3 m to 100 blows per 0.125 m penetration were recorded in isolated tests. Natural moisture contents of the sand deposit varied from 4% to 24%, locally up to 31% in three samples containing organics.

Grain size distribution analyses were carried out on eight samples of the sand deposit. Results of the tests are plotted on Figures B2 and B3 of Appendix B, and are summarized below:

Gravel %	0 to 4
Sand %	65 to 92
Silt and Clay %	8 to 35

5.5 Sand and Gravel

A layer of sand and gravel was encountered beneath the peat and sand in Boreholes SCW-02 to SCW-05 and within the sand deposit in Borehole SCW-10. The sand and gravel layer was 0.4 to 1.2 m thick in Boreholes SCW-02 to SCW-05 drilled on the west side of the creek, with a lower boundary at depths of 1.8 to 2.8 m (Elev. 262.3 to 261.9 m). In Borehole SCW-10, the sand and gravel layer was 1.7 m thick and encountered between 0.6 and 2.3 m depth (Elev. 267.9 and 266.2 m), within the sand deposit. Cobbles were encountered within this layer, and coring was used to penetrate these obstructions in Borehole SCW-05.

SPT 'N' values recorded in the sand and gravel ranged from 10 to 20 blows per 0.3 m penetration, indicating a compact relative density. Natural moisture contents varied from 4% to 20%.

Grain size distribution analyses were carried out on two samples of the sand and gravel deposit. The results are plotted on Figure B4 of Appendix B, and are summarized below:

Gravel %	18 to 41
Sand %	54 to 80
Silt and Clay %	2 to 5

5.6 Silt to Sandy Silt

Brown to grey silt with trace to some sand and clay was encountered beneath the sand to sand and gravel deposits in Boreholes SCW-01, SCW-04, SCW-05, SCW-09 and SCW-10. Sandy silt was encountered below the sand in Borehole SCW-07.

Borehole SCW-01 was terminated upon auger refusal at the probable base of the silt at 3.0 m depth (Elev. 263.3 m), indicating a thickness of 0.4 m. The silt layer was 0.5 m thick in Boreholes SCW-04 and SCW-05, with a lower boundary at 2.3 m depth (Elev. 261.4 to 261.6 m). In Boreholes SCW-07, SCW-09 and SCW-10, the thickness of silt to sandy silt was 1.7 to 3.6 m, with the lower boundary at depths of 7.7 m to 11.7 m (Elev. 258.9 to 256.8 m).

SPT 'N' values recorded in the silt typically ranged from 17 to 42 blows per 0.3 m penetration, indicating a compact to dense relative density. One 'N' value of 1 blow per 0.3 m penetration was recorded in Borehole SCW-10, possibly reflecting hydraulic disturbance. A value of 50 blows per 0.125 m penetration was recorded at the boundary of the underlying bedrock in Borehole SCW-07. Natural moisture contents of the silt layer ranged from 12% to 24%.

Grain size distribution analyses were carried out on five samples of the silt deposit. The results are plotted on Figure B5 of Appendix B, and are summarized below:

Gravel %	0
Sand %	3 to 21
Silt %	75 to 88
Clay %	4 to 12

5.7 Silty Clay

Grey silty clay with trace sand and trace to some gravel was encountered beneath the sand, gravel and silt layers in Boreholes SCW-02, SCW-04 to SCW-06, SCW-08 and SCW-10. The thickness of the silty clay layer varied from 0.2 m to 1.4 m. The lower boundary was at depths of 2.8 m to 3.7 m (Elev. 261.3 to 260.0 m) on the west side of the creek, and 6.6 to 12.5 m (Elev. 260.5 to 256.0 m) on the east side.

SPT 'N' values recorded in the clay typically ranged from 13 to 44 blows per 0.3m penetration, indicating a stiff to hard consistency. A value of 50 blows per 0.125 m penetration was recorded at the boundary with the underlying sand/gravel/cobble deposit in Borehole SCW-02. In Borehole SCW-10, the sampler sank 300 mm under self-weight in the clay before encountering an underlying sand and gravel deposit, indicating a very soft consistency at this location only.

Moisture contents of the silty clay ranged from 10% to 34%.

Grain size distribution curves from two silty clay samples are presented on the Record of Borehole Sheets and on Figure B6 of Appendix B. Atterberg Limits test results for three samples are presented on Figure B7 of Appendix B. The results are summarized as follows:

Gravel %	0 to 1
Sand %	1 to 9
Silt %	54 to 57
Clay %	33 to 45
Liquid Limits %	26 to 33
Plastic Limits %	13 to 15
Plastic Index %	12 to 19

The results indicate that the silty clay is of low plasticity with a group symbol of CL.

5.8 Sand, Gravel and Cobbles

A layer of sand, gravel and cobbles with occasional boulders was encountered beneath the silt and silty clay deposits, locally beneath the sand and gravel layer, in all boreholes except Boreholes SCW-01, SCW-06 and SCW-07. This layer directly overlies bedrock.

Borehole SCW-10 was terminated upon auger refusal within this deposit at a depth of 12.7 m (Elev. 255.8). In each of the other boreholes, coring was required to advance through this layer, and a thickness of 0.4 to 1.9 m was recorded. The lower boundary of the layer was encountered at depths of 4.3 m to 9.3 m (Elev. 260.4 to 258.1 m).

5.9 Bedrock

Pink and grey granitic bedrock was proved by coring in Boreholes SCW-02 to SCW-09. The depths and elevations of the bedrock surface encountered in the boreholes are summarized in Table 4.1. Boreholes SCW-01 and SCW-10 were terminated upon auger refusal at depths of 3.0 and 12.7 m (Elev. 263.3 and 255.8).

Table 4.1 – Depth and Elevation of Bedrock Surface

Borehole	Bedrock Surface Proved by Coring	
	Depth (m)	Elevation (m)
SCW-02	4.4	260.2
SCW-03	4.7	260.4
SCW-04	4.3	259.6
SCW-05	4.6	259.1
SCW-06	6.8	259.7
SCW-07	7.7	258.9
SCW-08	7.2	259.9
SCW-09	9.3	258.1

Rock core lengths of 2.8 m to 3.4 m were recovered from the boreholes. Total core recovery was generally between 89% and 100%, typically 100%. Lower recovery of 82% was obtained in the initial sample in Borehole SCW-08 which was partially advanced within the overlying sand, gravel and cobbles, and a recovery of 65% was obtained in a highly fractured zone in Borehole SCW-05.

RQD values measured in the bedrock cores recovered from boreholes on the west side of the creek ranged from 0% to 67%, indicating very poor to fair rock quality. In rock cores recovered from the east side of the creek, the RQD values ranged from 55% to 95%, indicating fair to excellent rock quality.

The Fracture Index (FI) of the rock, expressed as fractures per 0.3 m of core, ranged from 0 to 7. A Fracture Index of 12 was noted in Borehole SCW-06 Run 2.

The unconfined compressive strength of the rock cores estimated from the results of point load tests ranged from 53 MPa to 149 MPa, indicating a strong to very strong rock. The results are presented on the Record of Borehole Sheets in Appendix A (as average per run).

5.10 Water Levels

Water levels were monitored in the open boreholes during and upon completion of drilling. Four standpipe piezometers were installed to monitor water levels after completion of drilling. The water levels measured in the piezometers and open boreholes are summarized in Table 4.2.

Table 4.2 – Measured Groundwater Levels

Borehole	Date	Water Level (m)		Comments
		Depth	Elevation	
SCW-01	June 24, 2012	2.0	264.3	Open Borehole
SCW-02	July 15, 2012	0.9	263.7	Piezometer
	March 2, 2013	1.3	263.3	Piezometer
SCW-03	June 23, 2012	0.8	264.3	Open Borehole
SCW-04	June 22, 2012	0.9	263.0	Open Borehole
SCW-05	June 22, 2012	0.6	263.1	Open Borehole
	July 15, 2012	0.3	263.4	Piezometer
SCW-07	June 24, 2012	3.0	263.6	Open Borehole
	July 15, 2012	3.0	263.6	Piezometer
	March 2, 2013	3.5	263.1	Piezometer
SCW-08	July 15, 2012	2.9	264.2	Piezometer
	March 2, 2013	3.8	263.3	Piezometer
SCW-09	June 20, 2012	4.3	263.1	Open Borehole
SCW-10	June 27, 2012	3.5	265.0	Open Borehole

The above values are short-term readings and seasonal fluctuations of the groundwater level are to be expected. In general, the groundwater level is expected to be at or slightly above the water level in Stillwater Creek, which is shown to be near Elev. 263.1 on the contour plan provided by HMM (Appendix G). The creek and groundwater levels may be higher at the time of construction, and in particular after the spring snowmelt or periods of heavy rainfall.

6 MISCELLANEOUS

Borehole locations were selected and established in the field by Thurber Engineering Ltd. The coordinates and the ground surface elevations at the boreholes were surveyed by TBT Engineering Limited. Thurber obtained utility clearances for the borehole locations prior to drilling.

Eastern Ontario Diamond Drilling Ltd. from Hawkesbury, Ontario supplied a track-mounted CME 45 drill rig and conducted the drilling, sampling and in-situ testing operations.

The drilling and sampling operations were supervised on a full time basis by Mr. Ryan Kromer E.I.T. of Thurber Engineering Ltd.

Routine laboratory testing was carried out by Thurber Engineering Ltd.

Overall planning and supervision of the field program was conducted by Mr. Mark Farrant, P.Eng. Interpretation of the data and preparation of the report were carried out by Ms. Mei Cheong, M.Phil. The report was reviewed by Mr. Murray Anderson, P.Eng., and Dr. P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations Projects.

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PART 2: ENGINEERING DISCUSSION AND RECOMMENDATIONS

7 GENERAL

This report presents interpretation of the geotechnical data in the factual report and presents geotechnical recommendations for selection and design of a suitable foundation system for the new bridge planned to carry the westbound lanes of Highway 11/17 over Stillwater Creek in the Township of Nipigon, District of Thunder Bay.

The new bridge will be located immediately to the north of the existing Highway 11/17 bridge, which will carry the eastbound lanes. The preliminary General Arrangement drawing provided by HMM indicates that the new WBL bridge will comprise a two-span structure with a total length of 37.0 m and a width of approximately 15 m. The structure will consist of a hollow precast girder deck carried on cast-in-place concrete header beams, supported on concrete caissons or steel piles. RSS walls with maximum exposed heights of approximately 6.3 m (west) and 4.5 m (east) will be installed immediately behind the abutment foundations to retain the approach fill, in lieu of a conventional abutment.

The existing Highway 11/17 bridge is a three-span steel girder structure supported on concrete abutments and piers. Contract drawings from a 2004 rehabilitation assignment indicate that the abutments are supported on piles and the piers are supported on footings.

The discussion and recommendations presented in this report are based on the information provided by HMM and on the factual data obtained in the course of the investigation.

8 STRUCTURE FOUNDATIONS

The subsurface stratigraphy at this site varies, consisting of discontinuous layers of surficial topsoil, fill and/or peat, overlying successive layers of sand to silty sand, sand and gravel, silt to sandy silt, and silty clay of variable thickness and continuity. These deposits are typically underlain by a layer of sand, gravel and cobbles, which overlies bedrock proven at depths of 4.3 to 4.7 m on the west side of the creek and 6.8 to 9.3 m on the east side.

Groundwater levels measured in piezometers ranged from 0.3 to 3.8 m below the ground surface, at Elev. 263.1 to 264.2. In general, the groundwater level is expected to be at or slightly above the water level in Stillwater Creek, which is shown to be near Elev. 263.1 on the contour plan provided by HMM.

Based on the existing site conditions, initial consideration was given to the following foundation types:

- Spread footings on native soils or bedrock
- Augered Caissons (drilled shafts) socketed into bedrock
- Drilled-in pipe piles socketed into bedrock
- Steel H-piles

A comparison of the foundation alternatives based on advantages and disadvantages of each is included in Appendix D.

8.1 Spread Footings on Native Soils or Bedrock

Consideration was given to supporting the structure on spread footings founded on native soils or bedrock. However, this option is not recommended due to the following concerns:

- The native soils at this site are variable in both stratigraphy and support capability.
- Footings could be constructed on native compact to dense sands at elevations 262.3 to 263.1. However, the geotechnical resistance at these levels are relatively low and footing sizes would be large.
- To achieve higher resistance values, footings would need to be extended down to the sand, gravel and cobbles layer at elevations 260.0 to 262.3, or to bedrock at elevations 258.9 to 260.4.
- Construction of footings on the native soils or bedrock at the levels noted above would require excavation of fill and cohesionless soils below the groundwater level and the water level in the creek. Temporary shoring and dewatering would be required.
- Installation of temporary shoring (i.e. sheet pile cofferdam) will be difficult due to the presence of shallow bedrock and the overlying deposit containing cobbles and occasional boulders, particularly at the pier and west abutment.

- Temporary excavation for footing construction may have environmental impact on the creek.
- Scour protection will be required for the footings.

In light of the above factors, the spread footing option was not further developed.

8.2 Rock Socketed Drilled Shafts/Caissons

Caissons socketed into bedrock are considered a feasible foundation option to support the bridge structure. Bedrock was encountered and proved by coring at elevations given in Table 8.1.

Table 8.1 – Depth and Elevation of Bedrock Surface

Foundation Unit	Borehole	Top of Bedrock	
		Depth (m)	Elevation (m)
West Abutment	SCW-02	4.4	260.2
	SCW-03	4.7	260.4
Pier	SCW-04	4.3	259.6
	SCW-05	4.6	259.1
East Abutment	SCW-06	6.8	259.7
	SCW-07	7.7	258.9

The factored vertical geotechnical resistance computed for 0.76 m, 0.91 m, 1.2 m and 1.5 m diameter sockets for various socket depths below the bedrock surface are presented in Table 8.2. The SLS condition will not govern for caissons socketed into the rock.

Table 8.2 – Recommended Resistance Values for Caisson Design

Caisson Diameter (m)	Socket Length below bedrock surface (m)	Factored Geotechnical Resistance at ULS (kN)
0.76	1.5	2,500
	2.0	3,250
	3.0	5,000
0.91	1.5	3,000
	2.0	4,000
	3.0	6,000
1.2	1.5	3,500
	2.0	5,000
	3.0	7,500
1.5	1.5	4,500
	2.0	6,500
	3.0	9,500

The vertical geotechnical resistances were computed using the method outlined in the Canadian Foundation Engineering Manual, 4th Edition, Section 18.6.4. The resistance values are based on shaft resistance within the bedrock socket only; end-bearing resistance has been ignored in anticipation of difficulties cleaning and inspecting the caisson base below the water level.

The selection of a suitable socket depth will be governed by axial loads, lateral load and maximum shear and moment demand on each caisson. The depth of rock socket should not be less than 1.5 m and the axial load, shear and moment demands may require a deeper depth of rock socket.

The assessment of rock socket depth allows for the presence of some weathered and broken up rock just below the bedrock surface. Since the elevation of the bedrock surface is variable across the site and there is evidence of cobbles and boulders above the bedrock, it is critical to determine in the field during inspection of caisson installation that the entire depth of socket is formed in bedrock and not partly in cobbles and boulders and partly in bedrock. This issue is addressed in an NSSP included in Appendix E.

8.2.1 Caisson Socket Lateral Resistance

The ultimate passive force that can be mobilized by the embedded portion of a socket within rock is constant with depth and is given by:

$$P_p = 6 \cdot c \cdot D \cdot L$$

Where

c = 2,000 kPa (equivalent Mohr-Coulomb cohesion based on Hoek and Brown rock mass classification)

L = Depth of socket in rock, m

D = Socket diameter, m

8.2.2 Caisson Socket Installation

Caisson installation must be in accordance with OPSS 903.

Caisson installation will require excavation through cohesionless deposits of sand, gravel, cobbles and silt below the groundwater table, and construction of sockets in the underlying bedrock. The installation of caissons at this site must consider the following issues:

- The installation method must prevent collapse of the caisson sidewalls and washing-in of cohesionless soils into the rock socket.
- The caisson installation equipment must be able to advance through the layer of sand, gravel and cobbles with occasional boulders overlying the bedrock.

- The strength and hardness of the bedrock at this site must be considered when selecting equipment to excavate the rock socket. Blasting to facilitate rock removal is not permitted.

Selection of the methods and equipment employed to address the above issues is the responsibility of the Contractor. The contract documents must contain a statement to alert bidders of the above conditions. The wording for an NSSP addressing this issue is included in Appendix E.

8.3 Drilled-in Pipe Piles Socketed into Bedrock

An alternative foundation option is to support the bridge on drilled-in steel pipe piles socketed into bedrock and filled with concrete. Due to the presence of shallow bedrock as well as cobbles and occasional boulders above the bedrock, driven pipe piles are not recommended.

For drilled in pipe piles socketed into bedrock, the vertical geotechnical resistance estimated for 305 mm, 455 mm and 610 mm diameter pipe piles installed in a 1.5 m deep bedrock socket are presented in Table 8.3.

Table 8.3 – Recommended Resistance Values for Drilled in Pipe Piles

Pipe Pile Diameter (mm)	Socket Length Below Bedrock Surface (m)	Factored Geotechnical Resistance at ULS (kN)
305	1.5	1,400
455	1.5	2,600
610	1.5	4,000

The factored Geotechnical Resistance at ULS is based on the structural resistance of the pile section, with the end-bearing resistance reduced to account for the following factors:

- The tip of the pipe pile will not be in direct contact with the bedrock. The area of contact between the bedrock and the teeth of the cutting shoe at the tip of a pipe pile will be less than the full contact area between a shoeless pipe pile and the bedrock.
- Cleaning the rock socket from debris and water will be difficult.

The above resistances are for pipe wall thickness of 12.5 mm, steel yield strength of 245 MPa and concrete strength of 30 MPa. The depth of the socket will be governed by the lateral resistance requirement, base fixity requirement and shear and moment demand for each pile. The structural resistance of the pipe pile must be checked by the structural engineer.

8.3.1 Pipe Pile Installation

Installation of pipe piles must follow OPSS 903 specifications.

The method of installation of the pipe piles is the responsibility of the Contractor. One option for installing pipe piles is to drill them in using Rotary Duplex Drilling Methods. The Contractor's drilling method must be capable of dislodging, removing or penetrating obstructions such as cobbles, boulders or large rock fragments in the overburden soils. Care must be exercised while drilling the socket within the bedrock; the drilling methodology must be capable of excavating the bedrock to the specified socket dimensions without disturbing or fracturing the bedrock forming the sidewalls and base of the socket. Blasting to facilitate rock removal is not permitted.

Since the rock cutting shoe at the tip of a pipe pile will be slightly larger in diameter than the outside diameter of a pipe pile, there will be a small gap between the rock socket wall and the pipe pile. It is recommended that the annular space between the pipe pile and socket wall be grouted to the bedrock surface to achieve fixity.

During and subsequent to installation, the pipe pile will be partially filled with water and it may not be practical to dewater the pipe prior to concreting. Tremie concreting will be required for concreting these pipe piles.

A NSSP addressing the above issues is included in Appendix E.

8.3.2 Lateral Resistance for Pipe Pile Sockets

The ultimate passive force that can be mobilized by the embedded portion of a pipe pile socketed within rock is constant with depth and is given by the lateral resistance formula presented in Section 8.2.1 of this report.

8.4 Steel H-piles

The use of driven steel H-piles may be considered to support the bridge. At the east abutment, driven piles are expected to encounter refusal on bedrock at depths of 6.8 to 7.7 m. At the pier and west abutment, the H-piles will encounter refusal on the bedrock at depths of 4.3 to 4.7 m or potentially above the bedrock in the overlying sand, gravel and cobble layer encountered at depths of 2.8 to 3.7 m. The short pile length may not provide sufficient lateral stability or fixity, and the feasibility of driven piles at these foundation units must be assessed.

The anticipated pile tip elevations and geotechnical resistance values recommended for HP 310 X 110 steel H-piles driven to refusal on bedrock or the overlying sand/gravel/cobble layer are presented in Table 8.4.

Table 8.4 –Recommended Pile Resistance Values for Driven H-Piles

Foundation Unit	Borehole	Probable Refusal Stratum	Estimated Pile Tip Elevation	Pile Section HP 310 x 110	
				Factored Geotechnical Resistance at ULS _f	Geotechnical Reaction at SLS
West abutment	SCW-02	Bedrock, possibly cobbles	260.2	1,600	1,400
	SCW-03		260.4		
Pier	SCW-04	Bedrock, possibly sand, gravel and cobbles	259.6	1,600	1,400
	SCW-05		259.1		
East abutment	SCW-06	Bedrock	259.7	2,000	Does not govern
	SCW-07		258.9		

The resistance values given for driven piles at the pier and west abutment have been reduced to account for the possibility that some piles may encounter refusal in the sand, gravel and cobbles layer above the bedrock. An NSSP alerting the Contractor to the possibility of piles encountering refusal at varying depths above the bedrock is provided in Appendix E.

The length of driven H-piles, particularly at the proposed west abutment and pier will be short and may not provide sufficient lateral stability or fixity. At these locations, rock socketed H-piles are recommended. Pile installation should involve advancing a socket at least 1.5 m below the bedrock surface, inserting the pile to the base of the socket, and then backfilling around the pile with concrete. For a HP 310 x110 steel H-pile, a rock socket diameter of 610mm is required.

An axial, factored geotechnical resistance at ULS of 2,000 kN per pile is recommended for HP 310 x 110 piles socketed into bedrock.

The SLS condition will not govern for piles socketed into bedrock.

The recommended rock socket depth allows for the presence of some weathered and broken up rock just below the bedrock surface. The socket depth may need to be greater than 1.5 m to satisfy structural requirements such as lateral loads and maximum shear and moment demand on each pile.

Since the elevation of the bedrock surface is variable across the site and there is evidence of cobbles and boulders immediately above the bedrock, it is critical to determine in the field during inspection of rock socket installation that the entire depth of socket is formed in sound bedrock and not partly in cobbles and boulders and partly in bedrock. This issue is addressed in an NSSP included in Appendix E.

8.4.1 Pile Tips

For all driven H-piles, the tips of the piles must be fitted with pile tip protection from an approved manufacturer such as Titus Steel (Standard H-point) or approved equivalent. Pile tip protection is recommended to prevent pile damage when driving through the cobble layer and setting the piles on bedrock.

8.4.2 Pile Installation

Pile installation should be in accordance with OPSS 903.

For piles installed to the tolerances shown in Clause 903.07.05.01 of the Specification, the foundation drawing should include the note “Piles to be driven to bedrock”.

If the proposed bridge design requires that the deviation at the top of the pile be limited to tight tolerances, a driving template or other means may be required to achieve the specified maximum deviation.

For rock socketed piles, the method of installation of the piles is the responsibility of the Contractor. The Contractor’s drilling method must be capable of dislodging, removing or penetrating obstructions such as cobbles and boulders in the overburden soils. Care must be exercised while drilling the socket within the bedrock; the drilling methodology must be capable of excavating the bedrock to the specified socket dimensions without disturbing or fracturing the bedrock forming the sidewalls and base of the socket. Blasting to facilitate rock removal is not permitted.

The drilling method must also maintain sidewall stability of the drilled hole and allow cleaning of the socket without cohesionless soils running into the socket. The drilling method must not undermine the existing bridge foundation.

During and subsequent to installation, the drilled hole and socket will be partially filled with water and it may not be practical to dewater the socket prior to concreting. Tremie concreting will be required for concreting these piles.

A NSSP addressing these issues is included in Appendix E. This NSSP must be included in the tender documents.

8.4.3 Downdrag

Downdrag on the piles is not considered to be an issue at this site.

8.4.4 Lateral Resistance for H-piles

For the predominant cohesionless soils at the site, the lateral resistance of the piles may be calculated using a value for the coefficient of horizontal subgrade reaction (k_s) and ultimate lateral resistance (p_{ult}) as follows:

$$k_s = n_h \cdot z / D \quad (\text{kN/m}^3)$$

$$p_{ult} = 3 \cdot \gamma \cdot z \cdot K_p \quad (\text{kPa})$$

where

z = depth of embedment of pile in metres

D = pile width/diameter in metres

n_h = coefficient related to soil density (see table below)

γ = unit weight (see table below)

K_p = passive earth pressure coefficient (see table below)

The recommended parameters for use in the above equations are presented in Table 8.5. In all cases, lateral resistance to the piles above elevation 262.0 should be ignored in the design, noting that the soils at the pier and west abutment are highly variable, with fill, peat and loose sand encountered in the upper 1.4 to 2.3 m, and the east abutment is situated relatively close to a steep creek bank.

Table 8.5 – Parameters for Lateral Pile Resistance

Location	Elevation	C_u (kPa)	n_h (kN/m ³)	K_p	Unit Weight (kN/m ³)	Soil Conditions
West Abutment (BHs SCW-02 and SCW-03)	262.0 to 260.2	-	7,500	3.8	11*	Sand, Gravel and Cobbles
	Below 260.2	-	-	-	-	Bedrock
Pier (BHs SCW-04 and SCW-05)	262.0 to 261.5	-	2,500	3.0	10*	Silt
	261.5 to 260.0	100	-	2.7	10*	Silty Clay
	260.0 to 259.1	-	7,500	3.8	11*	Sand, Gravel and Cobbles
	Below 259.1	-	-	-	-	Bedrock
East Abutment (BHs SCW-06 and SCW-07)	262.0 to 258.9	-	3,500	3.1	10.5*	Sand to Sandy Silt
	Below 258.9	-	-	-	-	Bedrock

*Buoyant unit weight below the water table.

The ultimate passive force that can be mobilized by the embedded portion of a H-pile socketed within rock is constant with depth and is given by the lateral resistance formula presented in Section 8.2.1 of this report.

The above equations and recommended parameters may be used to analyze the interaction between a pile and the surrounding soil. The lateral pressures obtained from the analysis should not exceed the ultimate lateral resistance.

The spring constant, K , for analysis may be obtained by the expression, $K = k_s * L * D$ (kN/m), where k_s is the coefficient of horizontal subgrade reaction (kN/m³), D is the pile width/diameter (m) and L is the length (m) of the pile segment or element used in the analysis. The ultimate lateral resistance on any one segment of pile, P_{ult} , may be obtained from the expression, $P_{ult} = p_{ult} * L * D$. This represents the ultimate load at which the soil fails and will not support any additional load at greater displacements.

For lateral soil/pile group interaction analysis, the modulus of subgrade reaction (k_s) may have to be reduced based on pile spacing.

Where a pile group is oriented *perpendicular* to the direction of loading, group action may be considered by reducing values for k_s by a reduction factor R as follows:

Pile Spacing Perpendicular to Direction of Loading	Horizontal Subgrade Reaction Reduction Factor, R
4 D^*	1.00
1 D^*	0.50

* D is the width of the pile, and spacing is measured centre to centre

Where a pile group is oriented *parallel* to the direction of loading, group action may be considered by reducing values for k_s by a reduction factor R as follows:

Pile Spacing Parallel to Direction of Loading	Horizontal Subgrade Reaction Reduction Factor, R
8 D	1.00
6 D	0.70
4 D	0.40
3 D	0.25

Intermediate values may be obtained by interpolation.

8.5 Recommended Foundation

From a geotechnical perspective, the recommended foundation to support the bridge abutments at this site would be driven H-piles at the east abutment and H-piles socketed into bedrock at the pier and west abutment. However, drilled shafts/caissons socketed into bedrock or drilled-in pipe piles socketed into bedrock are considered feasible alternatives.

8.6 Frost Cover

The design depth of frost penetration at this site is 2.3 m.

Frost protection should be provided for buried pile caps, if used, and should consist of a minimum of 2.3m of soil cover.

9 RETAINED SOIL SYSTEMS

The current design concept calls for the abutments to be constructed as Retained Soil Systems (RSS). Based on the preliminary GA drawing, the maximum exposed RSS height is expected to be approximately 6.3 m at the west abutment face and 4.5 m at the east abutment face.

The RSS wing walls will extend back parallel to the roadway, with a length of approximately 13.5 m at the west approach and 9.0 m at the east approach. At the west approach, the proposed base level of the wall will step up from Elev. 264.0 at the abutment to Elev. 267.0 at the westerly extent. At the east approach, the proposed wall base level will step up from about Elev. 265.8 at the abutment to Elev. 268.0 at the easterly extent.

The foundation of the entire RSS mass (i.e. from the face of the wall to the furthest extent of the reinforcement) must be founded on competent native soils or engineered fill. At this site, measures will be required to provide a competent foundation for the RSS walls. The recommended foundation treatment for the RSS walls at the west and east abutments are outlined below.

9.1 West Abutment (Boreholes SCW-02 and SCW-03)

At the west abutment, loose to compact sand fill and peat are present below the proposed RSS base level. It is recommended that foundation preparation involve subexcavation of the existing sand fill, peat and loose silty sand layers, followed by replacement with Granular A engineered fill. The base of the engineered fill must be constructed on the compact sand and gravel layer encountered at depths of 1.5 and 2.3 m (Elev. 263.1 and 262.8) in Boreholes SCW-02 and SCW-03.

In general, engineered fill placed under the RSS mass to achieve the design founding level should consist of OPSS Granular A material placed in 150 mm lifts and compacted to 100% of its SPMDD at a moisture content within 2% of optimum. The engineered pad must extend at least 1.0 m beyond the limits of the RSS mass and levelling strip.

Excavation for removal of the peat and loose silty sand is expected to extend about 0.5 m below the water level measured in the piezometer in Borehole SCW-02 in March 2013. Dewatering to lower the groundwater level below the excavation depth is likely to be impractical at this location due to the proximity to the creek, the high permeability of the soils, and the presence of cobbles and boulders resisting sheet pile installation. Therefore subexcavation and granular placement may need to be carried out in wet conditions, and a thicker initial lift of granular material will need to be placed to raise the surface above the groundwater prior to compaction.

RSS walls founded on granular engineered fill constructed as outlined above may be designed for the following geotechnical resistances:

Factored Geotechnical Resistance at ULS	350 kPa
Geotechnical Resistance at SLS	200 kPa

The geotechnical resistances provided above are for concentric, vertical loading. The effects of load inclination and eccentricity need to be taken into account according to the CHBDC 2006 Section 6.7.

Total settlement of an RSS wall constructed as outlined above is expected to be less than 25 mm.

Analysis of the global stability of the RSS at the west abutment was conducted to assess stability of a maximum 6.5 m high wall founded on the improved subgrade. The stability analysis was carried out using the commercially available slope stability program GEO-SLOPE, applying the Morgenstern-Price method. The slope model and the analysis results are shown in Figure F1 of Appendix F. The computed factor of safety for the proposed RSS geometry was 2.2, which exceeds the minimum value of 1.3 normally accepted for this type of analysis. Stability of the RSS abutment is therefore not considered to be an issue.

9.2 East Abutment (Boreholes SCW-06 and SCW-07)

At the east abutment, sand to silty sand was encountered at the base level of the proposed RSS wall. The sand is typically compact at the proposed base level, however a loose zone was identified in Borehole SCW-06 between approximate Elev. 262.5 and 264.2, about 1.6 m below the lowest level of the wall.

To minimize the potential for deleterious differential settlement of the wall, it is recommended that foundation preparation at the east abutment include subexcavation of the existing sand to silty sand down to Elev. 264.5, compaction of the exposed subgrade at this level, and re-establishment of the base level using Granular A engineered fill.

Engineered fill placed under the RSS mass to achieve the design founding level should consist of OPSS Granular A material placed in 150 mm lifts and compacted to 100% of its SPMDD at a moisture content within 2% of optimum. The engineered pad must extend at least 1.0 m beyond the limits of the RSS mass and levelling strip.

RSS walls founded on a minimum 1.3 m thick pad of granular engineered fill constructed as outlined above may be designed for the following geotechnical resistances:

Factored Geotechnical Resistance at ULS	350 kPa
Geotechnical Resistance at SLS	200 kPa

As an alternative to subexcavation and placement of an engineered fill pad, consideration may be given to founding the wall on the native sand at the design level and using lightweight cellular concrete as RSS backfill in order to reduce the potential for differential settlement. In this case, the geotechnical resistance values recommended for design of an RSS wall founded on the native soils are as follows:

Factored Geotechnical Resistance at ULS	250 kPa
Geotechnical Resistance at SLS	125 kPa

The geotechnical resistances provided above are for concentric, vertical loading. The effects of load inclination and eccentricity need to be taken into account according to the CHBDC 2006 Section 6.7.

Settlement of the foundation soils under the weight of the RSS wall constructed using granular backfill is estimated to be in the order of 25 to 30mm. For a wall with cellular concrete backfill, the settlement is expected to be less than 25mm. As the foundations soils are typically non-cohesive, this settlement is expected to occur essentially as the RSS mass is constructed.

A preliminary analysis of the global stability of the RSS at the east abutment was conducted to assess stability of a maximum 4.5 m high wall founded at the proposed level. The stability analysis was carried out using the commercially available slope stability program GEO-SLOPE, applying the Morgenstern-Price method. Analyses were carried out for an RSS wall founded on the native sand at the design level, a wall constructed on granular engineered fill as outlined above, and a wall with cellular concrete backfill founded on the native sand. The slope model and the analysis results are shown on Figures F2 to F4 in Appendix F.

The computed factor of safety for the proposed RSS on native sand was 1.3, which is a value normally accepted for this type of analysis. The factors of safety for a wall on engineered fill and for a wall backfilled with cellular concrete were 1.3 and 1.4, respectively. Stability of the RSS abutment is therefore not considered to be an issue provided the wall is constructed as recommended in the preceding paragraphs.

9.3 General

In general, RSS walls used in conjunction with the new abutments must be “High Performance” and “High Appearance”. The contract drawings should include information on the longitudinal alignment of the wall in plan, the top and base elevations of the wall in profile, cross-sectional space constraints and an NSSP for the RSS wall.

The entire block of RSS mass must be designed against various modes of failure including sliding and overturning. Sliding resistance along the base of the wall on compact sand and gravel or on granular engineered fill may be estimated using an ultimate friction coefficient of 0.55. For a wall founded on the sand to silty sand at the east abutment, an ultimate friction coefficient of 0.4 is recommended.

The RSS walls must be protected from undermining and reduction in the global stability due to erosion and scour of the creek banks and soils below the base of the wall. Design must consider the impacts of potentially fluctuating water levels in the creek.

10 SHEET PILE WALLS

Installation of steel sheet pile walls adjacent to the abutment foundations could be considered in lieu of RSS abutments. The sheet piles will provide containment and resistance to lateral earth pressures from the approach fill.

Bedrock is present at relatively shallow depths at the west abutment, and the bedrock is overlain by a layer of sand, gravel and cobbles. Further, cobbles, boulders and rock protection are exposed on the sides and forward slopes of the existing approach embankments. These conditions will impede the driving of the sheet piles resulting in more arduous driving, limited depth of penetration, and refusal at varying depths.

Sheet piles should be provided with sheet pile tip protection to minimize any tip damage. Any visible boulders along the sides of the embankment should be removed prior to driving the sheet piles.

Design of permanent sheet pile walls must consider environmental conditions such as road salts or fluctuating water levels that may cause corrosion and reduce the service life of the structure.

11 LATERAL EARTH PRESSURES

Backfill to retaining walls or sheet pile walls if employed should be in accordance with OPSS 902 and consist of Granular A or Granular B Type II material. All granular material should meet the specifications of OPSS 1010 as amended by Special Provision 110S13. Compaction equipment to be used adjacent to retaining structures should be restricted in accordance with OPSS 501 and SP 105S21.

Lateral earth pressures acting on the abutments/retaining walls may be assumed to be distributed triangularly and to be governed by the characteristics of the wall backfill and the underlying native soils. For a fully drained condition, the pressures should be computed in accordance with the CHBDC but generally are given by the expression:

$$p_h = K (\gamma h + q)$$

Where:

p_h = horizontal pressure on the wall at depth h (kPa)

K = earth pressure coefficient (see table below)

γ = unit weight of retained soil (see table below)

h = depth below top of fill where pressure is computed (m)

q = value of any surcharge (kPa)

Earth pressure coefficients for backfill to the abutment wall are dependent on the material used as backfill. Typical values are shown in Table 11.1.

Table 11.1 – Earth Pressure Coefficient (K) for Horizontal Surface Behind Wall

Condition	Earth Pressure Coefficient (K)		
	OPSS Granular A or Granular B Type II $\phi = 35^\circ, \gamma = 22.8 \text{ kN/m}^3$	OPSS Granular B Type I $\phi = 32^\circ, \gamma = 21.2 \text{ kN/m}^3$	Existing Sand Fill or Native Sand to Silt $\phi = 30^\circ, \gamma = 20 \text{ kN/m}^3$
Active (Unrestrained system)	0.27	0.31	0.33
At rest (Restrained system)	0.43	0.47	0.50
Passive (Movement Towards Soil Mass)	3.7	3.3	3.0

The use of a material with a high friction angle and low active pressure coefficient (Granular A, Granular B Type II) is preferred as it results in lower earth pressures acting on the wall.

The factors in Table 11.1 are “ultimate” values and require certain movements for the respective conditions to be mobilized. The values to use in design can be estimated from Figure C6.16 in the Commentary to the Canadian Highway Bridge Design Code.

In accordance with Clause 6.9.3 of the CHBDC, a compaction surcharge should be added. The magnitude should be 12 kPa at the top of fill and decreasing to 0 kPa at a depth of 2.0 m for Granular B Type I or 1.7 m for Granular A or Granular B Type II.

12 APPROACH EMBANKMENTS

The approach embankments beyond the extents of the RSS walls will have a maximum height of approximately 5.5 m at the west approach and 3.5 m at the east approach. The foundation soils governing the stability and settlement of the approach embankments consist of loose to compact sands and silts over bedrock or cobbles and boulders at the west approach, and compact sand to sand and gravel at the east approach.

In view of the embankment foundation conditions and the relatively low embankment height, stability of the approach embankments is not expected to be a concern.

The total settlement of the embankment foundation soils under the weight of the approach fill is estimated to be less than 25 mm. The foundations soils are cohesionless and therefore these settlements will be immediate and essentially completed during construction.

Embankment construction should be carried out in accordance with SP 206S03. If new fill is placed against existing sloped embankments, the existing surfaces should be appropriately benched as per OPSD 208.010, after stripping of vegetation/organics, soft soils or otherwise unsuitable materials.

13 EROSION PROTECTION

The native sands and silts at this site are susceptible to erosion. The potential for erosion along the creek channel, particularly on the east bank under the bridge, must be assessed and erosion

protection measures such as rock protection provided where required. In particular, erosion protection must be provided to prevent undermining of RSS walls at the abutments.

A vegetative cover should be established on all other exposed earth surfaces to protect against surficial erosion, in general accordance with OPSS 804.

14 EXCAVATION AND GROUNDWATER CONTROL

In general, excavation for construction of the new bridge is expected to be limited to foundation preparation and construction of the RSS walls.

All excavation must be carried out in accordance with OPSS 902 and the Occupational Health and Safety Act (OHSA). For the purposes of the OHSA, the native soils within the probable depth of excavation at this site may be classed as Type 3 soils above the water table and Type 4 below the water level.

Piezometer readings at the abutments indicated groundwater level between Elev. 263.1 and 264.2 m. At the west abutment, excavation for removal of the peat is expected to extend about 0.5 m below the water level measured at this location. Dewatering to lower the groundwater level below the excavation depth is unlikely to be practical due to the proximity to the creek, the high permeability of the soils, and the presence of cobbles and boulders resisting sheet pile installation. Therefore subexcavation and granular placement may need to be carried out in wet conditions.

Excavation for construction of the RSS walls at the east abutment is expected to be maintained above the groundwater level. Sump pumping should be suitable to handle any seepage entering the temporary excavation.

Roadway protection must be provided where the excavation encroaches into the toe of the existing Highway 11/17 embankment. Roadway protection should be provided in accordance with OPSS 539 and designed for Performance Level 2. All shoring systems should be designed by a Professional Engineer experienced in such designs.

The design of any dewatering system and any road protection that may be required on the project is the responsibility of the Contractor and the Contract Documents must alert him to this responsibility.

15 SEISMIC CONSIDERATIONS

The following seismic parameters should be used for design:

- Velocity Related Seismic Zone 0
- Zonal Velocity Ratio 0.0
- Acceleration Related Seismic Zone 0
- Zonal Acceleration Ratio 0.0
- Peak Horizontal Acceleration 0.011g

The soil profile type at this site has been classified as Type I. Therefore, according to Clause 4.4.6 of the CHBDC, a Site Coefficient “S” (ground motion amplification factor) of 1.0 should be used in seismic design.

In accordance with Clause 4.6.4 of the CHBDC, retaining structures should be designed using active (K_{AE}) and passive (K_{PE}) earth pressure coefficients that incorporate the effects of earthquake loading. The coefficients of horizontal earth pressure for seismic loading presented in Table 15.1 may be used:

Table 15.1 – Earth Pressure Coefficients for Earthquake Loading

Condition	Earth Pressure Coefficient (K)		
	OPSS Granular A or Granular B Type II $\phi = 35^\circ, \gamma = 22.8 \text{ kN/m}^3$	OPSS Granular B Type I $\phi = 32^\circ, \gamma = 21.2 \text{ kN/m}^3$	Existing Sand Fill or Native Sand to Silt $\phi = 30^\circ, \gamma = 20 \text{ kN/m}^3$
Active (K_{AE})*	0.28	0.32	0.33
At Rest (K_{OE})**	3.7	3.2	3.0
Passive (K_{PE})	0.45	0.50	0.50

* After Mononobe and Okabe, passive case assumes a horizontal surface in front of the wall.

** After Woods

The potential for liquefaction of the foundations soils was assessed using the Seed and Idriss (1971) method for cohesionless soils. It is estimated that under the existing conditions, the foundation soils at the site are not prone to liquefaction.

16 CONSTRUCTION CONCERNS

Potential construction concerns include, but are not necessarily limited to:


1. Bedrock is present at relatively shallow depths at the west abutment, and the bedrock is overlain by a layer of sand, gravel and cobbles. The Contractor must be prepared to remove, drill through or otherwise penetrate these obstructions and form a socket within the bedrock for installation of H-piles, drilled-in pipe piles or caissons. The drilling methodology must be capable of excavating the hard bedrock to the specified socket dimensions without disturbing or fracturing the bedrock.
2. Caisson installation and excavation of rock sockets must consider the potential for sloughing of cohesionless soils into the socket, particularly below the groundwater table. The drilling method must maintain sidewall stability of the drilled hole.
3. Excavation of the peat for RSS construction at the west abutment may extend below the groundwater level, and may need to be carried out in the wet. Sloughing of the excavation sidewalls should be anticipated.
4. Excavation for new bridge construction must not undermine the existing Highway 11/17 bridge or embankment slopes. Roadway protection must be provided where required to maintain the stability of the existing embankment.

17 CLOSURE

Engineering analysis and preparation of the foundation design report were carried out by Ms. Mei Cheong, M.Phil. The report was reviewed by Mr. Murray R. Anderson, P.Eng. and Dr. P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations Projects.

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May 23, 2013

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Appendix A

Record of Borehole Sheets

SYMBOLS, ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES

1. TEXTURAL CLASSIFICATION OF SOILS

CLASSIFICATION	PARTICLE SIZE	VISUAL IDENTIFICATION
Boulders	Greater than 200mm	same
Cobbles	75 to 200mm	same
Gravel	4.75 to 75mm	5 to 75mm
Sand	0.075 to 4.75mm	Not visible particles to 5mm
Silt	0.002 to 0.075mm	Non-plastic particles, not visible to the naked eye
Clay	Less than 0.002mm	Plastic particles, not visible to the naked eye

2. COARSE GRAIN SOIL DESCRIPTION (50% greater than 0.075mm)

TERMINOLOGY	PROPORTION
Trace or Occasional	Less than 10%
Some	10 to 20%
Adjective (e.g. silty or sandy)	20 to 35%
And (e.g. sand and gravel)	35 to 50%

3. TERMS DESCRIBING CONSISTENCY (COHESIVE SOILS ONLY)

DESCRIPTIVE TERM	UNDRAINED SHEAR STRENGTH (kPa)	APPROXIMATE SPT ⁽¹⁾ 'N' VALUE
Very Soft	12 or less	Less than 2
Soft	12 to 25	2 to 4
Firm	25 to 50	4 to 8
Stiff	50 to 100	8 to 15
Very Stiff	100 to 200	15 to 30
Hard	Greater than 200	Greater than 30

NOTE: Hierarchy of Soil Strength Prediction

- 1) Laboratory Triaxial Testing
- 2) Field Insitu Vane Testing
- 3) Laboratory Vane Testing
- 4) SPT value
- 5) Pocket Penetrometer


4. TERMS DESCRIBING DENSITY (COHESIONLESS SOILS ONLY)

DESCRIPTIVE TERM	SPT "N" VALUE
Very Loose	Less than 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very Dense	Greater than 50

5. LEGEND FOR RECORDS OF BOREHOLES

SYMBOLS AND ABBREVIATIONS FOR SAMPLE TYPE	SS Split Spoon Sample	WS Wash Sample	AS Auger (Grab) Sample
	TW Thin Wall Shelby Tube Sample	TP Thin Wall Piston Sample	
	PH Sampler Advanced by Hydraulic Pressure	PM Sampler Advanced by Manual Pressure	
	WH Sampler Advanced by Self Static Weight	RC Rock Core	SC Soil Core

$$\text{Sensitivity} = \frac{\text{Undisturbed Shear Strength}}{\text{Remoulded Shear Strength}}$$

 Water Level


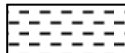



C_{pen} Shear Strength Determination by Pocket Penetrometer

- (1) SPT 'N' Value Standard Penetration Test 'N' Value – refers to the number of blows from a 63.5kg hammer free falling a height of 0.76m to advance a standard 50 mm outside diameter split spoon sampler for 0.3 m depth into undisturbed ground.
- (2) DCPT Dynamic Cone Penetration Test – Continuous penetration of a 50 mm outside diameter, 60° conical steel point attached to "A" size rods driven by a 63.5 kg hammer free falling a height of 0.76 m. The resistance to cone penetration is the number of hammer blows required for each 0.3 m advance of the conical point into undisturbed ground.

UNIFIED SOILS CLASSIFICATION

MAJOR DIVISIONS		GROUP SYMBOL	TYPICAL DESCRIPTION
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	GW	Well-graded gravels or gravel-sand mixtures, little or no fines.
		GP	Poorly-graded gravels or gravel-sand mixtures, little or no fines.
		GM	Silty gravels, gravel-sand-silt mixtures.
		GC	Clayey gravels, gravel-sand-clay mixtures.
	SAND AND SANDY SOILS	SW	Well-graded sands or gravelly sands, little or no fines.
		SP	Poorly-graded sands or gravelly sands, little or no fines.
		SM	Silty sands, sand-silt mixtures.
		SC	Clayey sands, sand-clay mixtures.
FINE GRAINED SOILS	SILTS AND CLAYS $W_L < 50\%$	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity.
		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays. ($W_L < 30\%$).
		CI	Inorganic clays of medium plasticity, silty clays. ($30\% < W_L < 50\%$).
		OL	Organic silts and organic silty-clays of low plasticity.
	SILTS AND CLAYS $W_L > 50\%$	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.
		CH	Inorganic clays of high plasticity, fat clays.
		OH	Organic clays of medium to high plasticity, organic silts.
HIGHLY ORGANIC SOILS		Pt	Peat and other highly organic soils.
CLAY SHALE			
SANDSTONE			
SILTSTONE			
CLAYSTONE			
COAL			

EXPLANATION OF ROCK LOGGING TERMS

<u>ROCK WEATHERING CLASSIFICATION</u>		<u>SYMBOLS</u>	
Fresh (FR)	No visible signs of weathering.		
Fresh Jointed (FJ)	Weathering limited to the surface of major discontinuities.		CLAYSTONE
Slightly Weathered (SW)	Penetrative weathering developed on open discontinuity surfaces, but only slight weathering of rock material.		SILTSTONE
Moderately Weathered (MW)	Weathering extends throughout the rock mass, but the rock material is not friable.		SANDSTONE
Highly Weathered (HW)	Weathering extends throughout the rock mass and the rock is partly friable.		COAL
Completely Weathered (CW)	Rock is wholly decomposed and in a friable condition, but the rock texture and structure are preserved.		Bedrock (general)

<u>DISCONTINUITY SPACING</u>		<u>STRENGTH CLASSIFICATION</u>			
Bedding	Bedding Plane Spacing	Rock Strength	Approximate Uniaxial Compressive Strength		Field Estimation of Hardness*
			(MPa)	(psi)	
Very thickly bedded	Greater than 2m	Extremely Strong	Greater than 250	Greater than 36,000	Specimen can only be chipped with a geological hammer
Thickly bedded	0.6 to 2m				
Medium bedded	0.2 to 0.6m	Very Strong	100-250	15,000 to 36,000	Requires many blows of geological hammer to break
Thinly bedded	60mm to 0.2m				
Very thinly bedded	20 to 60mm	Strong	50-100	7,500 to 15,000	Requires more than one blow of geological hammer to break
Laminated	6 to 20mm				
Thinly Laminated	Less than 6mm	Medium Strong	25.0 to 50.0	3,500 to 7,500	Breaks under single blow of geological hammer.
		Weak	5.0 to 25.0	750 to 3,500	Can be peeled by a pocket knife with difficulty
		Very Weak	1.0 to 5.0	150 to 750	Can be peeled by a pocket knife, crumbles under firm blows of geological pick.
		Extremely Weak (Rock)	0.25 to 1.0	35 to 150	Indented by thumbnail

<u>TERMS</u>	
Total Core Recovery: (TCR)	Core recovered as a percentage of total core run length.
Solid Core Recovery: (SCR)	Percent Ratio of solid core of full cylindrical shape recovered. Expressed with respect to the total length of core run.
Rock Quality Designation: (RQD)	Total length of sound core recovered in pieces 0.1m in length or larger as a percentage of total core run length.
Uniaxial Compressive Strength (UCS)	Axial stress required to break the specimen
Fracture Index: (FI)	Frequency of natural fractures per 0.3m of core run.

RECORD OF BOREHOLE No SCW-01

1 OF 1

METRIC

GWP# 647-89-00 LOCATION Stillwater Creek N 5 431 060.5 E 208 142.8 ORIGINATED BY RK
 HWY 11/17 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN
 DATUM Geodetic DATE 2012.06.24 - 2012.06.24 CHECKED BY MC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE									WATER CONTENT (%)		
266.3								20	40	60	80	100							
0.0	TOPSOIL Black Moist SAND , some silt and clay, trace gravel Loose to Compact Brown Moist With black organics, trace clay Some gravel		1	SS	9	▽	266								○				
0.1							265									○			
				2	SS		16										○		
																		○	
				3	SS		6	264										○	
263.7			4	SS	22											○			
2.6	SILT , some sand, trace gravel, trace clay															○			
263.3	Compact																		
3.0	Brown Wet																		
	END OF BOREHOLE AT 3.0m UPON AUGER REFUSAL. WATER LEVEL AT 2.0m UPON COMPLETION. BOREHOLE BACKFILLED WITH CUTTINGS TO SURFACE.																		

+³, ×³: Numbers refer to
Sensitivity

20
15
10
(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No SCW-02

1 OF 1

METRIC

GWP# 647-89-00 LOCATION Stillwater Creek N 5 431 079.0 E 208 153.6 ORIGINATED BY RK
 HWY 11/17 BOREHOLE TYPE NW Casing/NQ Coring COMPILED BY AN
 DATUM Geodetic DATE 2012.06.23 - 2012.06.23 CHECKED BY MC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
264.6								20	40	60	80	100		
0.0	TOPSOIL Black Moist		1	SS	11									
0.2	Silty SAND , medium grained, some gravel						264							
263.8	Compact													
263.5	Brown		2	SS	5									
1.1	Moist (FILL)													
263.1	PEAT , amorphous, sandy													
1.5	Loose Black Saturated		3	SS	16									
262.3	Silty SAND , trace gravel, trace organics													
2.3	Loose Grey Wet		4	SS	22									1 9 57 33
261.3	SAND and GRAVEL , medium to coarse													
3.3	Compact Brown Wet		5	SS	50/ .125									RUN #1 TCR=52%
260.2	Silty CLAY , trace sand and gravel Very Stiff Grey Wet (TILL-LIKE)		1	RUN			261							
4.4	SAND, GRAVEL and COBBLES													
	BEDROCK granite, very strong, pink/grey, medium grained, porphyritic texture		2	RUN			260							RUN #2 TCR=100% SCR=45% RQD=30% UCS=113MPa (Average)
							259							
							258							RUN #3 TCR=100% SCR=71% RQD=67% UCS=103MPa (Average)
	coarse grained		3	RUN										
256.8							257							
7.8	END OF BOREHOLE AT 7.8m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen. WATER LEVEL READINGS: DATE DEPTH (m) ELEV. (m) Jul. 15/12 0.9 263.7 Mar. 02/13 1.3 263.3													

ONTMT4S 05117.GPJ 2012TEMPLATE(MTO).GDT 4/25/13

+³, ×³: Numbers refer to
Sensitivity 20
15 5 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No SCW-03

1 OF 1

METRIC

GWP# 647-89-00 LOCATION Stillwater Creek N 5 431 073.6 E 208 159.5 ORIGINATED BY RK
 HWY 11/17 BOREHOLE TYPE NW/NQ Coring COMPILED BY AN
 DATUM Geodetic DATE 2012.06.23 - 2012.06.23 CHECKED BY MC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																							
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																													
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+³, ×³: Numbers refer to Sensitivity
 20
15 10 5 0
 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No SCW-04

1 OF 1

METRIC

GWP# 647-89-00 LOCATION Stillwater Creek N 5 431 087.0 E 208 162.5 ORIGINATED BY RK
 HWY 11/17 BOREHOLE TYPE Hollow Stem Augers/NW Coring COMPILED BY AN
 DATUM Geodetic DATE 2012.06.22 - 2012.06.22 CHECKED BY MC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							
263.9								20	40	60	80	100			
0.0	TOPSOIL , wood fragments, sandy Black Moist		1	SS	5		263								
0.2	Silty SAND , trace gravel, trace organics, wood fragments Loose Brown to Dark Brown Moist to Wet		2	SS	4										
262.5															
1.4	SAND and GRAVEL , coarse Dense Brown Wet		3	SS	39										
262.1															
1.8	SILT , trace to some sand, trace clay Compact to Dense Grey Wet		4	SS	17										
261.6															
2.3	Silty CLAY , trace sand Very Stiff Grey Wet (TILL-LIKE)		1	RUN											
261.1															
2.8	SAND, GRAVEL and COBBLES occasional boulders Red/Grey/Brown		2	RUN											
259.6															
4.3	BEDROCK granite, jointed, grey/pink, fine grained, porphyritic texture, strong		3	RUN											
	Vertical joint from 5.8m to 6.3m		4	RUN											
	Granite/granitic gneiss														

+³, ×³: Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No SCW-05

1 OF 1

METRIC

GWP# 647-89-00 LOCATION Stillwater Creek N 5 431 078.2 E 208 170.4 ORIGINATED BY RK
 HWY 11/17 BOREHOLE TYPE Hollow Stem Augers/NQ Coring COMPILED BY AN
 DATUM Geodetic DATE 2012.06.22 - 2012.06.22 CHECKED BY MC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
263.7								20	40	60	80	100		
0.0	TOPSOIL , sandy Black Moist		1	SS	4									
263.1	Silty SAND , trace gravel Loose Dark Brown Moist													
0.6														
	SAND and GRAVEL , occasional cobble Dense Brown Wet													
261.9	Cored from 0.6m to 1.5m		2	SS	30									
1.8	SILT , trace clay, trace sand Dense Grey Wet													0 3 88 9
261.4														
2.3	Silty CLAY , trace sand Hard to Stiff Grey Wet (TILL-LIKE) Silt seams		3	SS	38									0 1 54 45
			4	SS	13									
260.0														
3.7	COBBLES and GRAVEL , some sand Brown/Grey/Red		1	RUN										RUN #1 TCR=94%
259.1	BEDROCK granite, jointed, grey/pink, strong to very strong, porphyritic texture Rubble zone at 4.6m to 4.8m													RUN #2 TCR=95% SCR=70% RQD=38% UCS=81MPa (Average)
4.6			2	RUN										
	Highly fractured													RUN #3 TCR=65% SCR=18% RQD=0% UCS=149MPa (Average)
			3	RUN										
256.1														
7.6	END OF BOREHOLE AT 7.6m. WATER LEVEL AT 0.6m UPON COMPLETION. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen. WATER LEVEL READINGS: DATE DEPTH (m) ELEV. (m) Jul. 15/12 0.3 263.4													

+³, ×³: Numbers refer to
Sensitivity






20
15
10
(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No SCW-06

1 OF 2

METRIC

GWP# 647-89-00 LOCATION Stillwater Creek N 5 431 100.3 E 208 177.4 ORIGINATED BY RK
 HWY 11/17 BOREHOLE TYPE Hollow Stem Augers/NW/NQ Coring COMPILED BY AN
 DATUM Geodetic DATE 2012.06.26 - 2012.06.26 CHECKED BY MC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa												
266.5	Silty SAND Loose to Compact Brown Moist		1	SS	4		20	40	60	80	100						0 76 24 (SI+CL)			
0.0																				
	Trace silt		2	SS	16												0 91 9 (SI+CL)			
			3	SS	12															
			4	SS	6															
			5	SS	4															
262.5	SAND , medium grained, trace silt Very Dense Brown Wet																			
4.0																				
			6	SS	100/ .125															
260.3	Silty CLAY , some gravel Hard Grey Wet																			
6.2																				
			7	SS	44															
259.7	BEDROCK granite, coarse grained, strong to very strong, grey/pink, porphyritic texture																RUN #1 TCR=100% SCR=73% RQD=63% UCS=66MPa (Average)			
6.8																				
					1		RUN													
																	RUN #2 TCR=100% SCR=92% RQD=85% UCS=102MPa (Average)			
			2	RUN																
																	RUN #3 TCR=100% SCR=100% RQD=83% UCS=94MPa			
			3	RUN																
256.5																				

Continued Next Page

+³, ×³: Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No SCW-06

2 OF 2

METRIC

GWP# 647-89-00 LOCATION Stillwater Creek N 5 431 100.3 E 208 177.4 ORIGINATED BY RK
 HWY 11/17 BOREHOLE TYPE Hollow Stem Augers/NW/NQ Coring COMPILED BY AN
 DATUM Geodetic DATE 2012.06.26 - 2012.06.26 CHECKED BY MC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL (Average)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100					
10.0	Continued From Previous Page END OF BOREHOLE AT 10.0m. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG AND CUTTINGS TO SURFACE.																

RECORD OF BOREHOLE No SCW-07

1 OF 2

METRIC

GWP# 647-89-00 LOCATION Stillwater Creek N 5 431 094.1 E 208 186.3 ORIGINATED BY RK
 HWY 11/17 BOREHOLE TYPE Hollow Stem Augers/NQ Coring COMPILED BY AN
 DATUM Geodetic DATE 2012.06.24 - 2012.06.24 CHECKED BY MC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)						
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE					WATER CONTENT (%) w _p w w _L				GR	SA	SI	CL			
266.6								20	40	60	80	100											
0.0	TOPSOIL: (25mm) SAND, trace silt Loose to Compact Brown Moist to Wet		1	SS	5		266							○									
			2	SS	14									○									
			3	SS	16		265							○									
			4	SS	13		264							○						0	92	8	(SI+CL)
			5	SS	11		263							○									
262.5																							
4.1	Sandy SILT, trace clay Dense Brown to Grey Wet		6	SS	30		262							○						0	21	75	4
			7	SS	42		261																
			8	SS	50/		260							○									
258.9	Clayey						259							○									
7.7	BEDROCK granite, coarse grained, strong to very strong, pink/grey		1	RUN	.125		258																
	Frequent mechanical breaks at 8.1m to 9.1m						257																
	Faint weathering, porphyritic texture		2	RUN																			
		</																					

Continued Next Page

+³, ×³: Numbers refer to
Sensitivity

20
15
10
(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No SCW-07

2 OF 2

METRIC

GWP# 647-89-00 LOCATION Stillwater Creek N 5 431 094.1 E 208 186.3 ORIGINATED BY RK
 HWY 11/17 BOREHOLE TYPE Hollow Stem Augers/NQ Coring COMPILED BY AN
 DATUM Geodetic DATE 2012.06.24 - 2012.06.24 CHECKED BY MC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
	Continued From Previous Page							20	40	60	80	100					
255.9																	
10.7	END OF BOREHOLE AT 10.7m. WATER LEVEL AT 3.0m UPON COMPLETION. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen. WATER LEVEL READINGS: DATE DEPTH (m) ELEV. (m) Jul. 15/12 3.0 263.6 Mar. 02/13 3.5 263.1																

METRIC

[illegible]

+³, ×³: Numbers refer to Sensitivity

ONTMT4S 05117.GPJ 2012TEMPLATE(MTO).GDT 4/25/13

RECORD OF BOREHOLE No SCW-08

2 OF 2

METRIC

GWP# 647-89-00 LOCATION Stillwater Creek N 5 431 108.3 E 208 183.6 ORIGINATED BY RK
 HWY 11/17 BOREHOLE TYPE Hollow Stem Augers/NW/NQ Coring COMPILED BY AN
 DATUM Geodetic DATE 2012.06.27 - 2012.06.27 CHECKED BY MC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL (Average)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE					WATER CONTENT (%)				
							20	40	60	80	100	W _p	W	W _L			
	Continued From Previous Page		4	RUN			257										
256.5																	
10.6	END OF BOREHOLE AT 10.6m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen. WATER LEVEL READINGS: DATE DEPTH (m) ELEV. (m) Jul. 15/12 2.9 264.2 Mar. 02/13 3.8 263.3																

RECORD OF BOREHOLE No SCW-09

1 OF 2

METRIC

GWP# 647-89-00 LOCATION Stillwater Creek N 5 431 100.2 E 208 193.5 ORIGINATED BY RK
 HWY 11/17 BOREHOLE TYPE Hollow Stem Augers/NW Coring COMPILED BY AN
 DATUM Geodetic DATE 2012.06.20 - 2012.06.20 CHECKED BY MC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT LIMIT				UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa				WATER CONTENT (%)					GR	SA	SI	CL	
								20	40	60	80	100	20	40	60						
267.4																					
0.0																					
0.1	TOPSOIL: (75mm)																				
	SAND , trace to some silt, trace gravel Loose to Compact Brown Moist		1	SS	8									o							
			2	SS	8										o						
			3	SS	8										o						
			4	SS	26										o						
			5	SS	19											o					

Continued Next Page

+³, ×³: Numbers refer to Sensitivity
 20
 15
 10
 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No SCW-09

2 OF 2

METRIC

GWP# 647-89-00 LOCATION Stillwater Creek N 5 431 100.2 E 208 193.5 ORIGINATED BY RK
 HWY 11/17 BOREHOLE TYPE Hollow Stem Augers/NW Coring COMPILED BY AN
 DATUM Geodetic DATE 2012.06.20 - 2012.06.20 CHECKED BY MC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL (Average)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE					WATER CONTENT (%)				
							20	40	60	80	100	W _p	W	W _L			
	Continued From Previous Page																
							257									1	
																6	
																0	
			3	RUN												4	
							256									1	
																1	
255.3																1	
12.1	END OF BOREHOLE AT 12.1m. WATER LEVEL AT 4.3m UPON COMPLETION. BOREHOLE BACKFILLED WITH HOLEPUG AND CUTTINGS TO SURFACE.																

RECORD OF BOREHOLE No SCW-10

1 OF 2

METRIC

GWP# 647-89-00 LOCATION Stillwater Creek N 5 431 116.7 E 208 206.3 ORIGINATED BY RK
 HWY 11/17 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN
 DATUM Geodetic DATE 2012.06.27 - 2012.06.27 CHECKED BY MC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										
268.5								20	40	60	80	100						
0.0	TOPSOIL: (75mm)																	
0.1	Silty SAND , fine grained, trace roots		1	SS	4		268											
	Loose																	
267.9	Brown																	
0.6	Moist																	
	SAND , some gravel to SAND and GRAVEL , coarse grained, trace silt		2	SS	10													18 80 2
	Compact																	(SI+CL)
	Brown						267											
	Moist																	
			3	SS	20													41 54 5
																		(SI+CL)
266.2																		
2.3	SAND , trace to some silt		4	SS	24		266											
	Compact																	
	Light Brown to Brown																	
	Moist to Wet																	
			5	SS	20		265											
							264											
			6	SS	20													
263.2																		
5.3	Silty SAND						263											
	Compact																	
	Brown																	
	Moist to Wet																	
			7	SS	21		262											0 65 35
																		(SI+CL)
							261											
			8	SS	7													
							260											
259.4																		
9.1	SILT , trace sand, some clay		9	SS	1		259											
	Very Loose to Compact																	
	Grey																	
	Wet																	

Continued Next Page

+³, ×³: Numbers refer to Sensitivity
 20
 15
 10
 (%) STRAIN AT FAILURE

METRIC

[illegible]

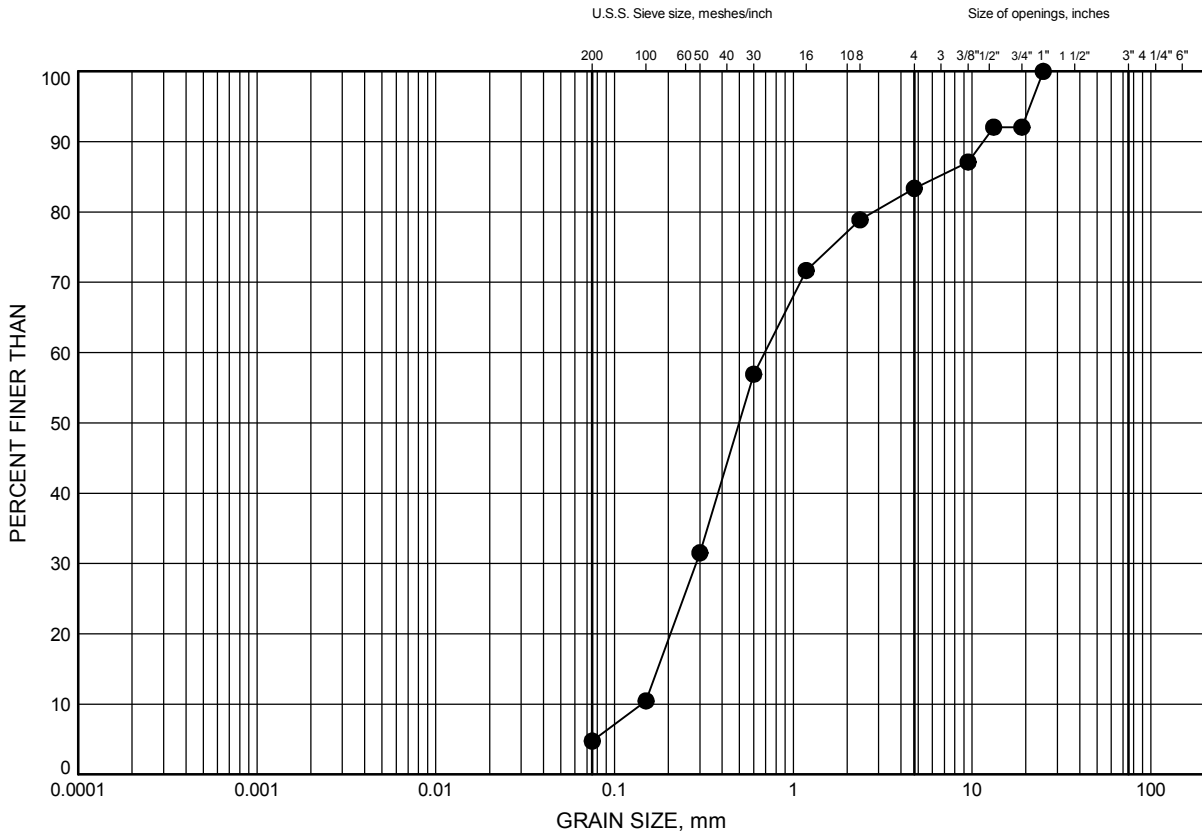
Appendix B

Laboratory Test Results

Hwy 11/17 Nipigon GRAIN SIZE DISTRIBUTION

FIGURE B1

SAND FILL



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	SCW-03	1.07	264.04

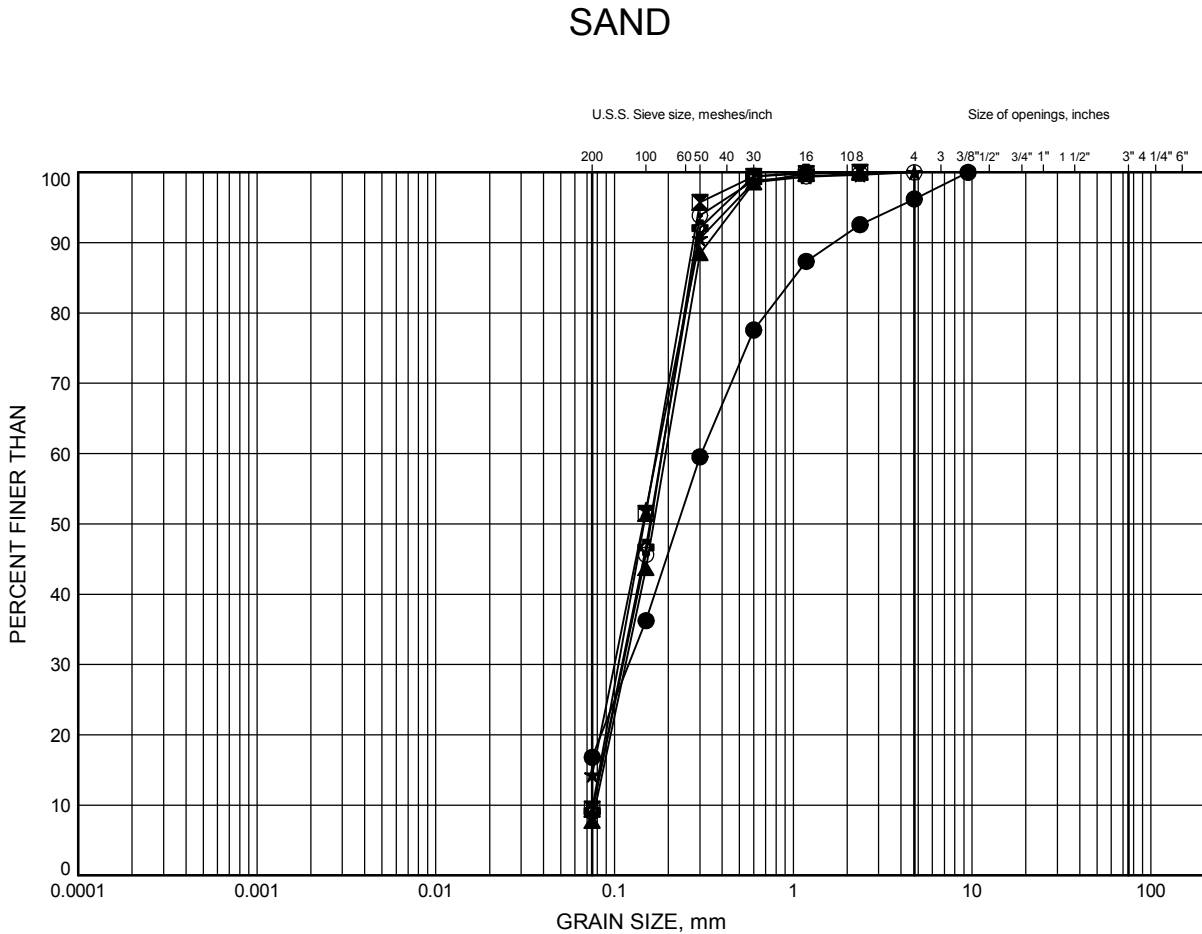
Date December 2012
W.P.# 647-89-00



Prep'd AN
Chkd. MC

Hwy 11/17 Nipigon GRAIN SIZE DISTRIBUTION

FIGURE B2



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	SCW-01	1.07	265.24
⊠	SCW-06	3.35	263.15
▲	SCW-07	2.59	264.01
★	SCW-08	1.83	265.28
⊙	SCW-08	3.35	263.76
⊕	SCW-09	3.35	264.05

Date March 2013
W.P. 647-89-00



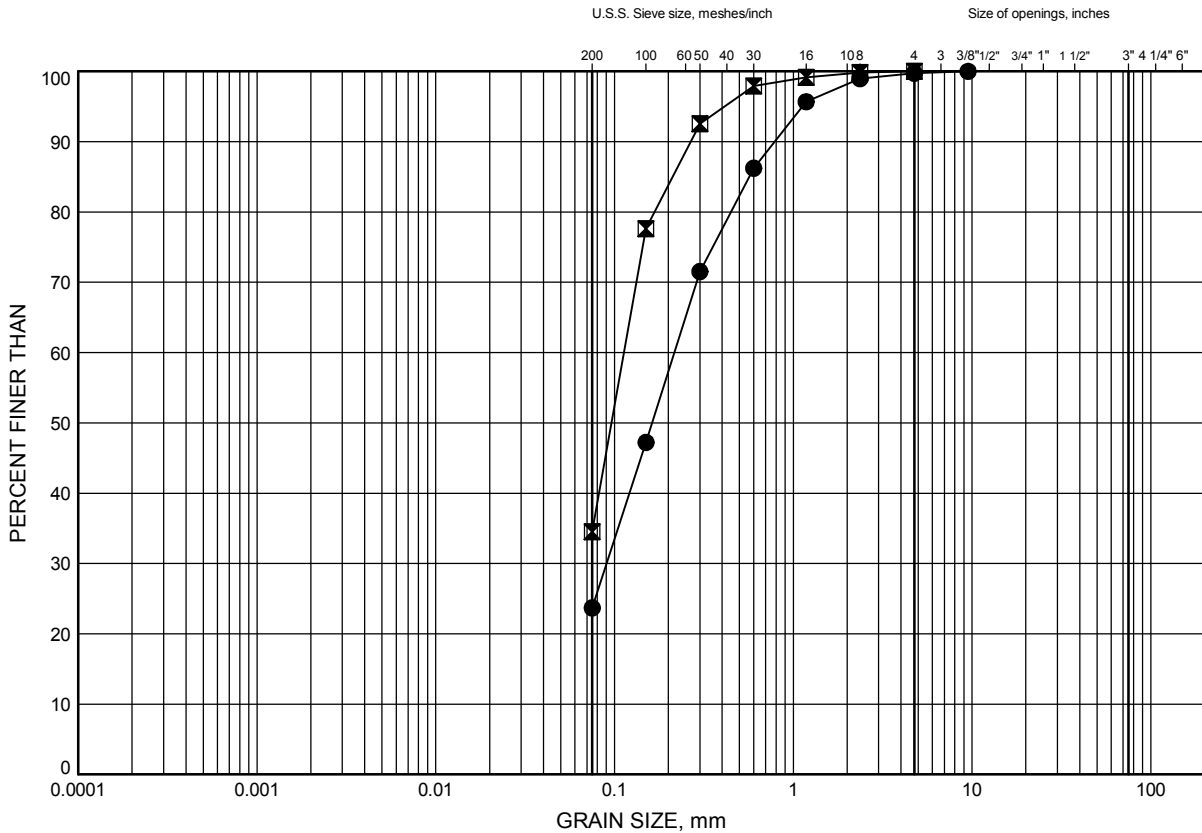
Prep'd AN
Chkd. MC

Hwy 11/17 Nipigon

GRAIN SIZE DISTRIBUTION

FIGURE B3

SILTY SAND



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	SCW-06	1.07	265.43
⊠	SCW-10	6.40	262.10

Date March 2013
W.P. 647-89-00

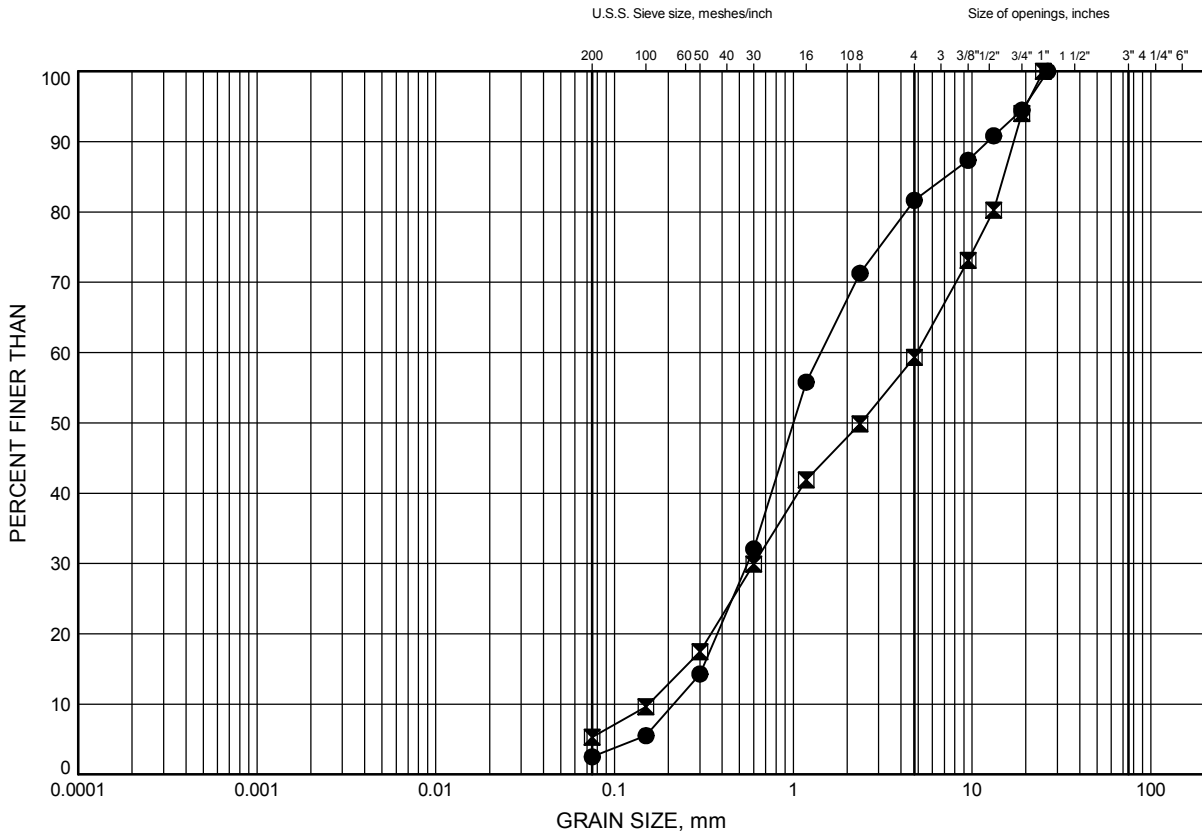


Prep'd AN
Chkd. MC

Hwy 11/17 Nipigon
GRAIN SIZE DISTRIBUTION

FIGURE B4

SAND AND GRAVEL



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	SCW-10	1.07	267.43
⊠	SCW-10	1.83	266.67

Date December 2012
W.P.# 647-89-00

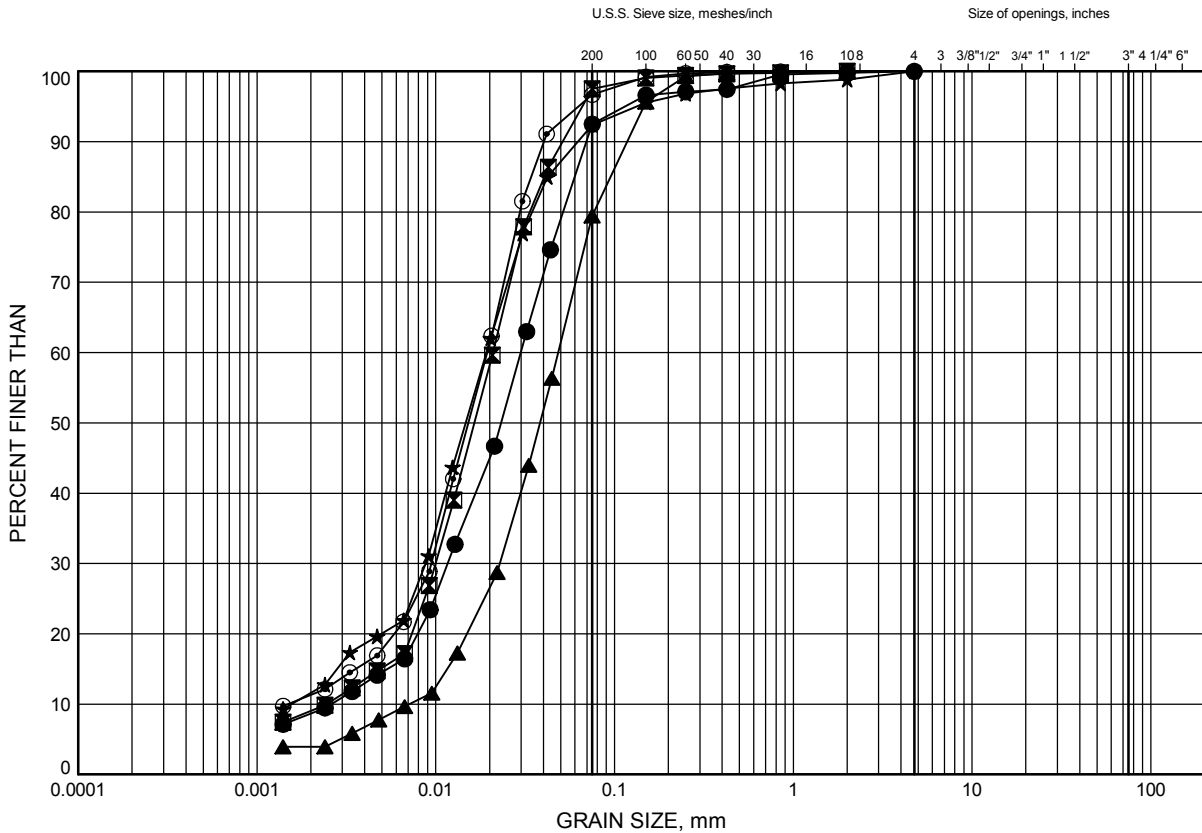


Prep'd AN
Chkd. MC

Hwy 11/17 Nipigon GRAIN SIZE DISTRIBUTION

FIGURE B5

SILT TO SANDY SILT



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	SCW-04	1.98	261.90
⊠	SCW-05	1.98	261.72
▲	SCW-07	4.88	261.04
★	SCW-09	7.92	259.77
⊙	SCW-10	10.97	257.53

Date December 2012
W.P.# 647-89-00

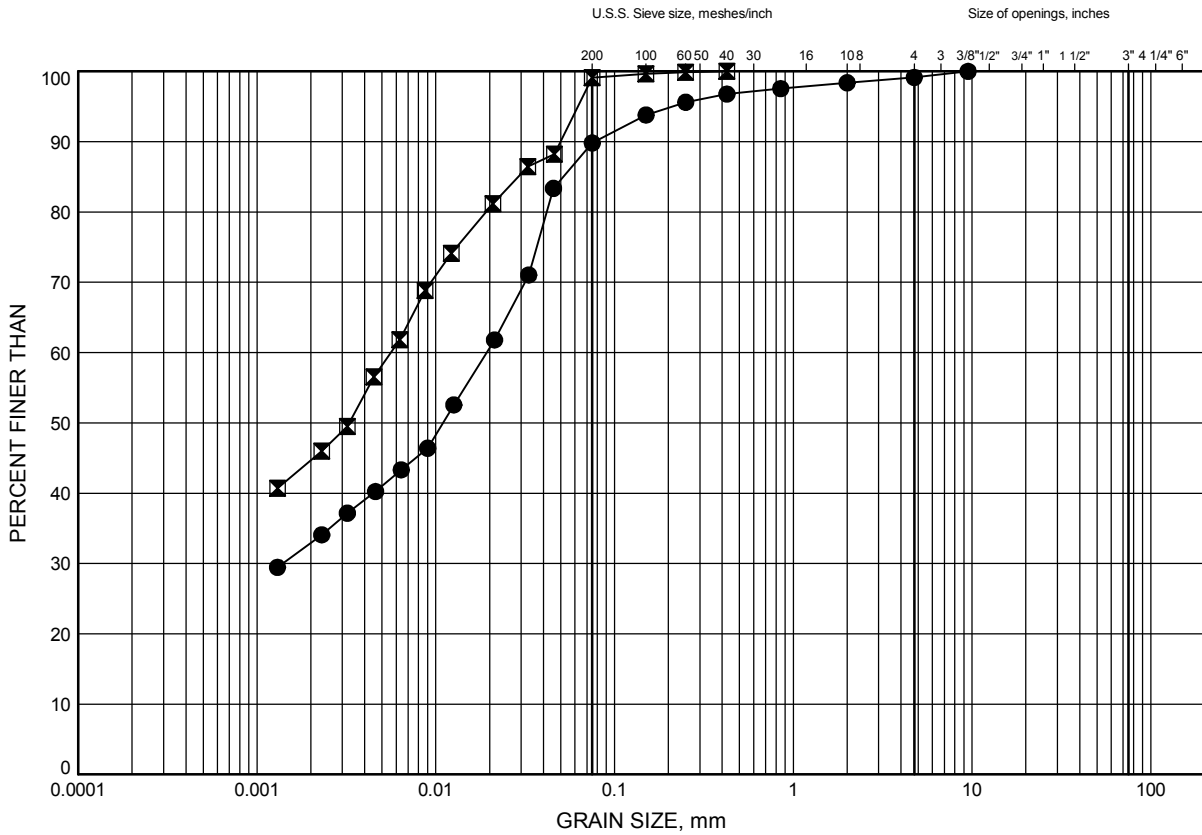


Prep'd AN
Chkd. MC

Hwy 11/17 Nipigon
GRAIN SIZE DISTRIBUTION

FIGURE B6

SILTY CLAY



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	SCW-02	2.59	262.01
⊠	SCW-05	2.59	261.11

Date December 2012
W.P.# 647-89-00



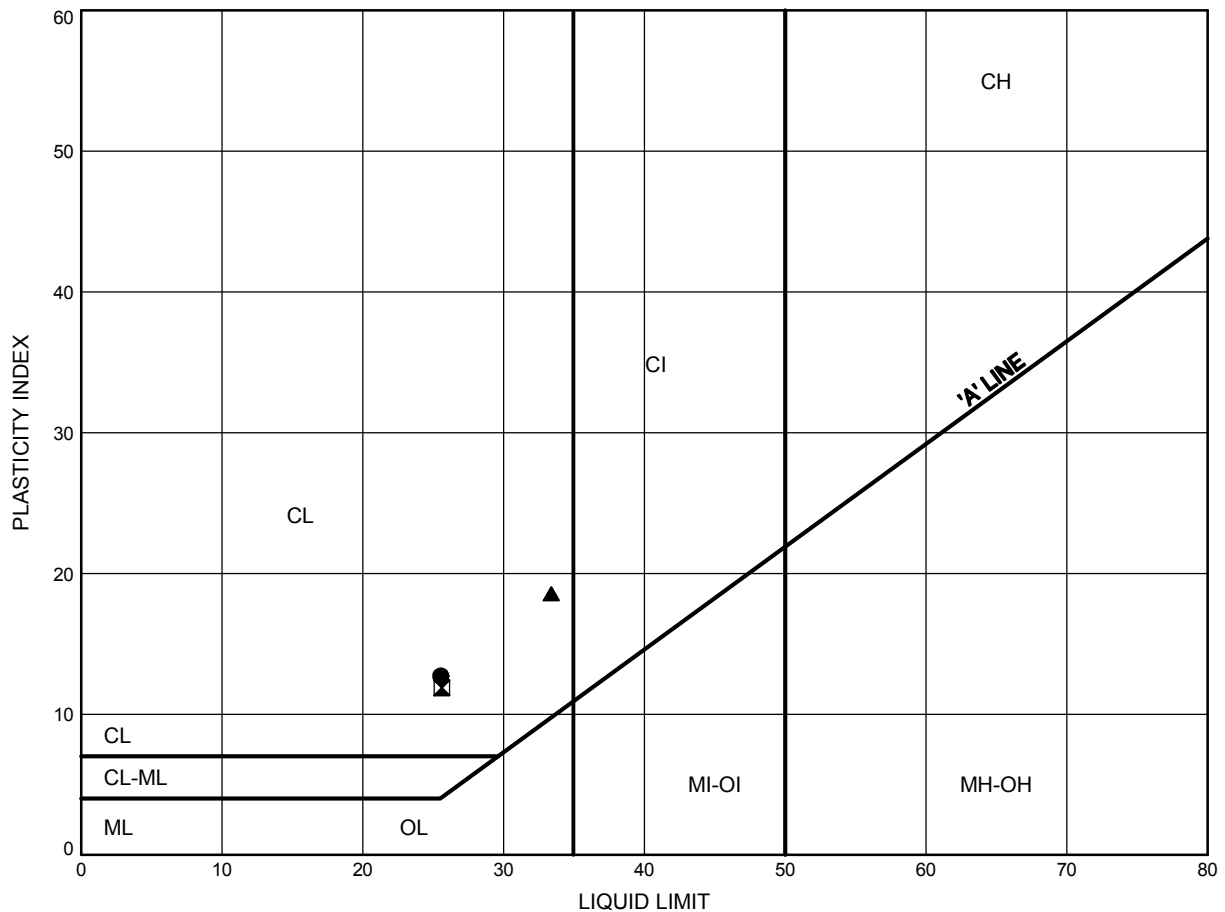
Prep'd AN
Chkd. MC

Hwy 11/17 Nipigon

ATTERBERG LIMITS TEST RESULTS

FIGURE B7

SILTY CLAY



LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	SCW-04	2.59	261.29
⊠	SCW-05	2.59	261.11
▲	SCW-06	6.52	259.98

Date December 2012
W.P.# 647-89-00



Prep'd AN
Chkd. MC

Appendix C

Site Photographs



**Photograph 1 – North side of existing Highway 11/17 Bridge over Stillwater Creek,
looking east**



**Photograph 2 – North side of existing Highway 11/17 Bridge over Stillwater Creek,
looking southwest**



Photograph 3 – Site of proposed Stillwater Creek WBL Bridge, looking north



Photograph 4 – Site of proposed Stillwater Creek WBL Bridge, looking northeast

Appendix D

Foundation Comparison

COMPARISON OF FOUNDATION ALTERNATIVES

Footings on Native Soil or Bedrock	Augered Caissons	Drilled in Pipe Piles	Steel H-Piles
<p>Advantages:</p> <ul style="list-style-type: none"> i. Generally less costly construction than deep foundation elements. ii. High values of geotechnical resistance are available on the bedrock 	<p>Advantages:</p> <ul style="list-style-type: none"> i. High geotechnical resistance available for caissons socketed into bedrock. ii. Construction of caissons could continue in freezing weather. iii. Excavation and dewatering requirements are minimized. 	<p>Advantages:</p> <ul style="list-style-type: none"> i. High geotechnical resistance available for pipe piles drilled into bedrock. ii. Liner is not required to support excavation sidewalls. iii. Excavation and dewatering requirements are minimized. iv. Cleaning and inspection of the socket base is not required. 	<p>Advantages:</p> <ul style="list-style-type: none"> i. High geotechnical resistance available for H-piles driven to or socketed in bedrock. ii. Installation of piles could continue in freezing weather. iii. Excavation and dewatering requirements are minimized.
<p>Disadvantages:</p> <ul style="list-style-type: none"> i. Relatively low and variable geotechnical resistance available in the native soils. ii. To achieve higher resistance values, footings would need to be extended down to the sand, gravel and cobbles layer or to bedrock. iii. Temporary shoring and dewatering would be required for construction of footings on native soils or bedrock. iv. Temporary excavation for footing construction may have environmental impact on the creek. v. Scour protection will be required for footings. 	<p>Disadvantages:</p> <ul style="list-style-type: none"> i. Higher unit costs than footings on bedrock. ii. Caissons must be socketed into very strong bedrock. iii. Penetration of cobbles and boulders above the bedrock may be difficult. iv. Measures will be required to provide sidewall support during drilling through cohesionless materials. v. Difficulties in obtaining a seal below the liner. Tremie concrete may be required. vi. Potential difficulty in cleaning and inspection of socket base. 	<p>Disadvantages:</p> <ul style="list-style-type: none"> i. Higher unit cost than footings on bedrock. ii. Piles must be socketed into very strong bedrock. iii. Penetration of cobbles and boulders above the bedrock may be difficult. iv. Concreting or grouting of the annular space within the pile socket is required. 	<p>Disadvantages:</p> <ul style="list-style-type: none"> i. Higher unit cost than footings on bedrock. ii. H-piles may encounter refusal on at varying depths on cobbles and boulders above the bedrock. iii. Socketing of the piles into hard bedrock will likely be needed to achieve structural requirements at west abutment. iv. Measures will be required to provide sidewall support during drilling through cohesionless soils. v. Difficulties in obtaining a seal below the liner. Tremie concrete may be required.
NOT RECOMMENDED	FEASIBLE	FEASIBLE	RECOMMENDED

Appendix E

List of SPs and OPSS, and Suggested Text for Selected NSSP

1. List of Special Provisions and OPSS Documents Referenced in this Report

- OPSS 501
- OPSS 539
- OPSS 804
- OPSS 902
- OPSS 903
- OPSS 1010
- SP 206S03
- SP 105S21
- SP 110S13
- OPSD 208.010

1. Suggested text for a NSSP on “Construction of Caissons”

Caisson installation shall be in accordance with OPSS 903 and the following.

Caisson installation at this site will require excavation through cohesionless soils below the groundwater table and construction of sockets in the underlying bedrock. Bedrock is present at relatively shallow depths and is overlain by a layer of sand, gravel and cobbles with boulders. The Contractor is advised of the following:

- The cohesionless soil above the bedrock is susceptible to disturbance under conditions of unbalanced hydrostatic head, and measures must be employed to maintain sidewall stability during installation of the caissons and prevent collapse/washing of cohesionless soils into the rock socket. Selection of the methods and equipment employed to achieve this is the responsibility of the Contractor.
- The installation methods and equipment must be capable of dislodging, removing or otherwise penetrating cobbles and boulders in the soils overlying the bedrock.
- The bedrock consists of strong to very strong granite. The strength and hardness of this rock must be taken into account when selecting equipment to advance the socket into rock. Equipment supplied to construct or drill the rock socket must be capable of excavating the bedrock to the specified socket dimensions without disturbing or

fracturing the bedrock forming the sidewalls and base of the socket. Blasting to facilitate the removal of bedrock is not permitted.

- The rock socket must be formed entirely within the bedrock below the level of any cobbles and boulders. Any length of caisson above the bedrock surface will not be considered part of the specified length of rock socket.

2. Suggested text for a N SSP on “Construction of Drilled-in Pipe Piles”

Installation of drilled-in pipe piles shall be in accordance with OPSS 903 and the following.

Drilled-in pipe pile installation at this site will require excavation through cohesionless soils below the groundwater table and advancing the pile into the underlying bedrock. Bedrock is present at relatively shallow depths and is overlain by a layer of sand, gravel and cobbles with boulders. The Contractor is advised of the following:

- The installation methods and equipment must be capable of dislodging, removing or otherwise penetrating cobbles and boulders in the soils overlying the bedrock.
- The bedrock consists of strong to very strong granite. The strength and hardness of this rock must be taken into account when selecting equipment to advance the pile into rock. Equipment supplied to advance the pile into rock must be capable of penetrating the bedrock without disturbing or fracturing the bedrock adjacent to the pile. Blasting to facilitate the removal of bedrock is not permitted.
- The rock embedment length must be formed entirely within the bedrock below the level of any cobbles and boulders. Any length of pile above the bedrock surface will not be considered part of the specified length of rock embedment.

3. Suggested text for a N SSP on “Construction of Driven H-piles”

Installation of H-piles shall be in accordance with OPSS 903 and the following.

The bedrock at the site is typically overlain by a layer of sand, gravel and cobbles with boulders. At some locations, H-piles may not be able to penetrate this layer to reach bedrock. The Contractor is alerted to the possibility of piles encountering refusal above the bedrock at varying depths.

The H-piles should be provided with pile tip protectors to minimize tip damage.

If the piles meet refusal at a depth less than the anticipated depth, the QVE must terminate driving before the pile is damaged due to over-driving. The QVE must immediately bring it

to the attention of the CA. If the CA cannot resolve the issue, it must be referred to the design team for resolution.

4. Suggested text for a NSSP on “Construction of H-Piles in Rock Sockets”

Installation of H-piles shall be in accordance with OPSS 903 and the following.

H-pile installation at this site will require excavation through cohesionless soils below the groundwater table and construction of sockets in the underlying bedrock. Bedrock is present at relatively shallow depths and is overlain by a layer of sand, gravel and cobbles with boulders. The Contractor is advised of the following:

- The cohesionless soil above the bedrock is susceptible to disturbance under conditions of unbalanced hydrostatic head, and measures must be employed to maintain sidewall stability during installation of the piles and prevent collapse/washing of cohesionless soils into the rock socket. Selection of the methods and equipment employed to achieve this is the responsibility of the Contractor.
- The installation methods and equipment must be capable of dislodging, removing or otherwise penetrating cobbles and boulders in the soils overlying the bedrock.
- The bedrock consists of strong to very strong granite. The strength and hardness of this rock must be taken into account when selecting equipment to advance the socket into rock. Equipment supplied to construct or drill the rock socket must be capable of excavating the bedrock to the specified socket dimensions without disturbing or fracturing the bedrock forming the sidewalls and base of the socket. Blasting to facilitate the removal of bedrock is not permitted.
- The rock socket must be formed entirely within the bedrock below the level of any cobbles and boulders. Any length of pile above the bedrock surface will not be considered part of the specified length of rock socket.
- H-piles shall be placed centred into the holes, bearing directly on the sound rock at the bottom of the hole. Piles shall be stabilized in place by temporary supports.
- The annular space between the rock socket wall and H-pile shall be filled with 30 MPa concrete to top of existing ground. The plumbness and alignment of the pile shall be maintained during concreting.

Appendix F

Slope Stability Output

File Name: SCW-RSS -1 - v1.gsz
 Directory: H:\19\1605\117 Hwy 11-17 Nipigon\Analysis\SCW\
 Created By: Mei Cheong
 Date: 4/17/13
 Description: Long-term
 Method: Morgenstern-Price
 Minimum Slip Surface Depth: 1.5 m

Name: Approach Fill	Unit Weight: 21 kN/m ³	Cohesion: 0 kPa	Phi: 32 °	Piezometric Line: 1
Name: Existing fill	Unit Weight: 20 kN/m ³	Cohesion: 0 kPa	Phi: 30 °	Piezometric Line: 1
Name: Sand	Unit Weight: 20 kN/m ³	Cohesion: 0 kPa	Phi: 32 °	Piezometric Line: 1
Name: Silt	Unit Weight: 20 kN/m ³	Cohesion: 0 kPa	Phi: 28 °	Piezometric Line: 1
Name: Silty Clay	Unit Weight: 19 kN/m ³	Cohesion: 0 kPa	Phi: 26 °	Piezometric Line: 1
Name: Cobbles Boulders	Unit Weight: 20 kN/m ³	Cohesion: 0 kPa	Phi: 38 °	Piezometric Line: 1
Name: Bedrock	Piezometric Line: 1			
Name: Peat	Unit Weight: 18 kN/m ³	Cohesion: 0 kPa	Phi: 21 °	Piezometric Line: 1
Name: RSS wall	Unit Weight: 19 kN/m ³	Cohesion: 200 kPa	Phi: 42 °	Piezometric Line: 1
Name: Granular A	Unit Weight: 21 kN/m ³	Cohesion: 0 kPa	Phi: 35 °	Piezometric Line: 1

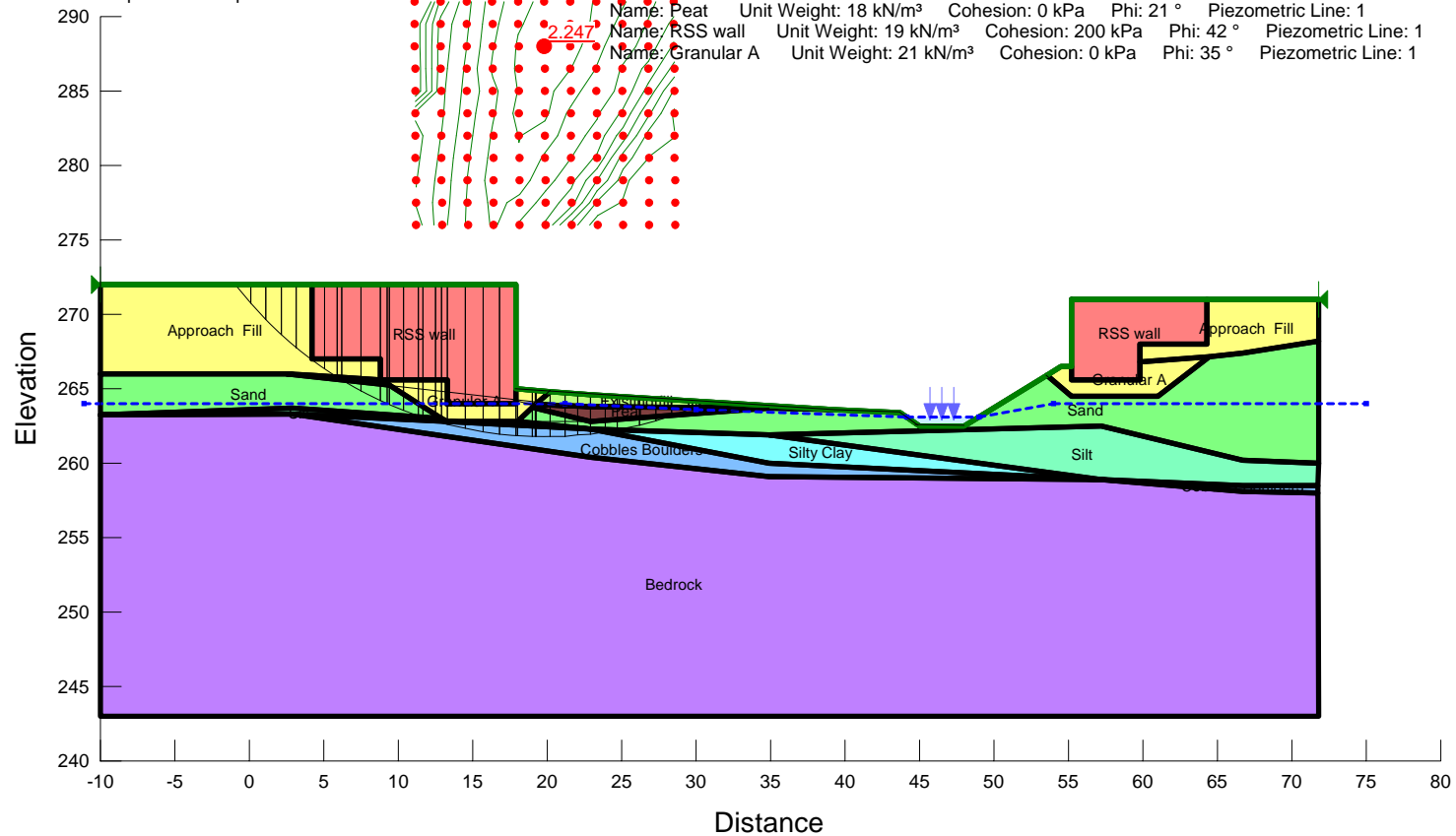


FIGURE F1

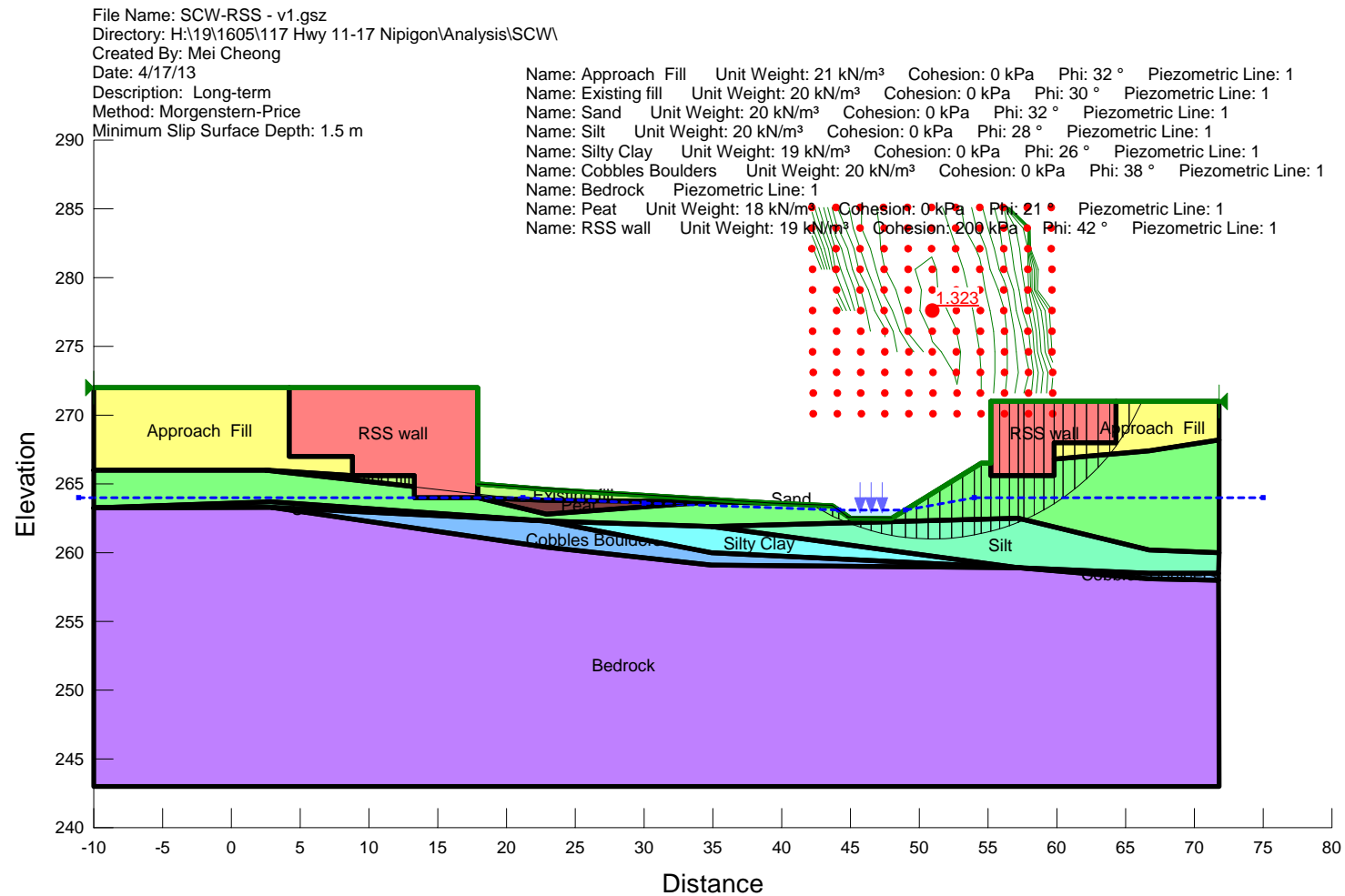


FIGURE F2

File Name: SCW-RSS -1 - v1.gsz
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 Date: 4/17/13
 Description: Long-term
 Method: Morgenstern-Price
 Minimum Slip Surface Depth: 1.5 m

Name: Approach Fill	Unit Weight: 21 kN/m ³	Cohesion: 0 kPa	Phi: 32 °	Piezometric Line: 1
Name: Existing fill	Unit Weight: 20 kN/m ³	Cohesion: 0 kPa	Phi: 30 °	Piezometric Line: 1
Name: Sand	Unit Weight: 20 kN/m ³	Cohesion: 0 kPa	Phi: 32 °	Piezometric Line: 1
Name: Silt	Unit Weight: 20 kN/m ³	Cohesion: 0 kPa	Phi: 28 °	Piezometric Line: 1
Name: Silty Clay	Unit Weight: 19 kN/m ³	Cohesion: 0 kPa	Phi: 26 °	Piezometric Line: 1
Name: Cobbles Boulders	Unit Weight: 20 kN/m ³	Cohesion: 0 kPa	Phi: 38 °	Piezometric Line: 1
Name: Bedrock	Piezometric Line: 1			
Name: Peat	Unit Weight: 18 kN/m ³	Cohesion: 0 kPa	Phi: 21 °	Piezometric Line: 1
Name: RSS wall	Unit Weight: 19 kN/m ³	Cohesion: 200 kPa	Phi: 42 °	Piezometric Line: 1
Name: Granular A	Unit Weight: 21 kN/m ³	Cohesion: 0 kPa	Phi: 35 °	Piezometric Line: 1

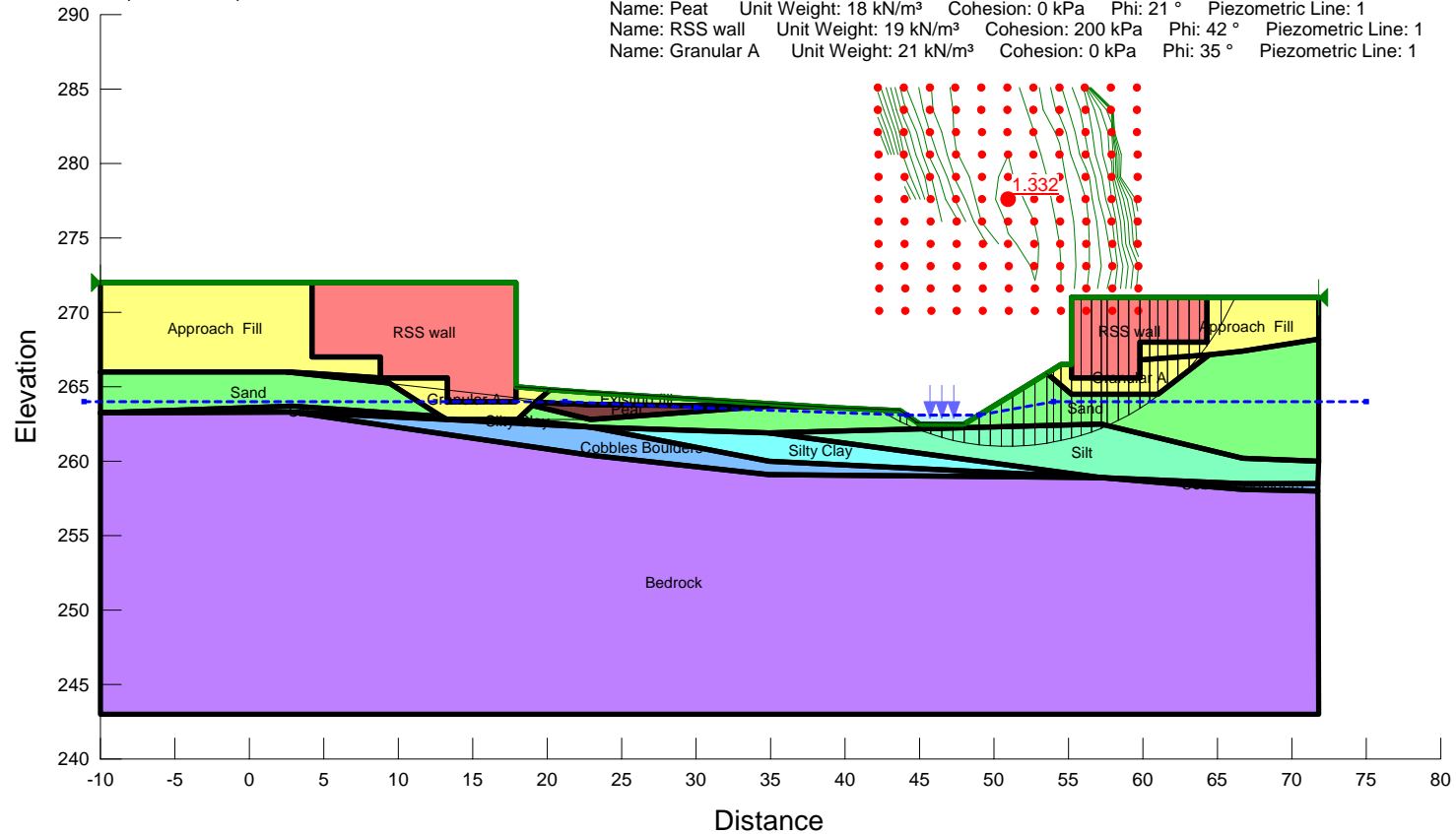


FIGURE F3

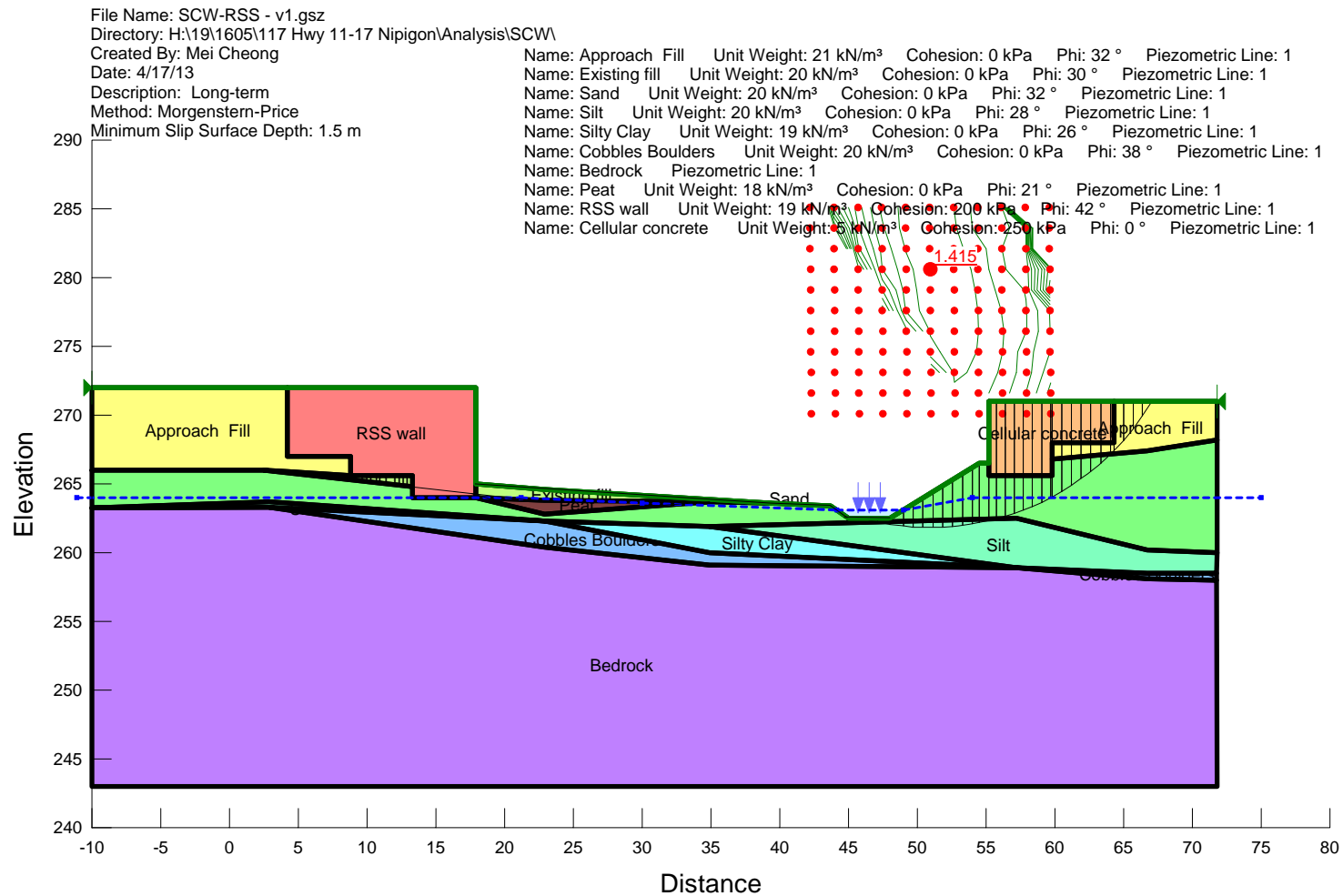
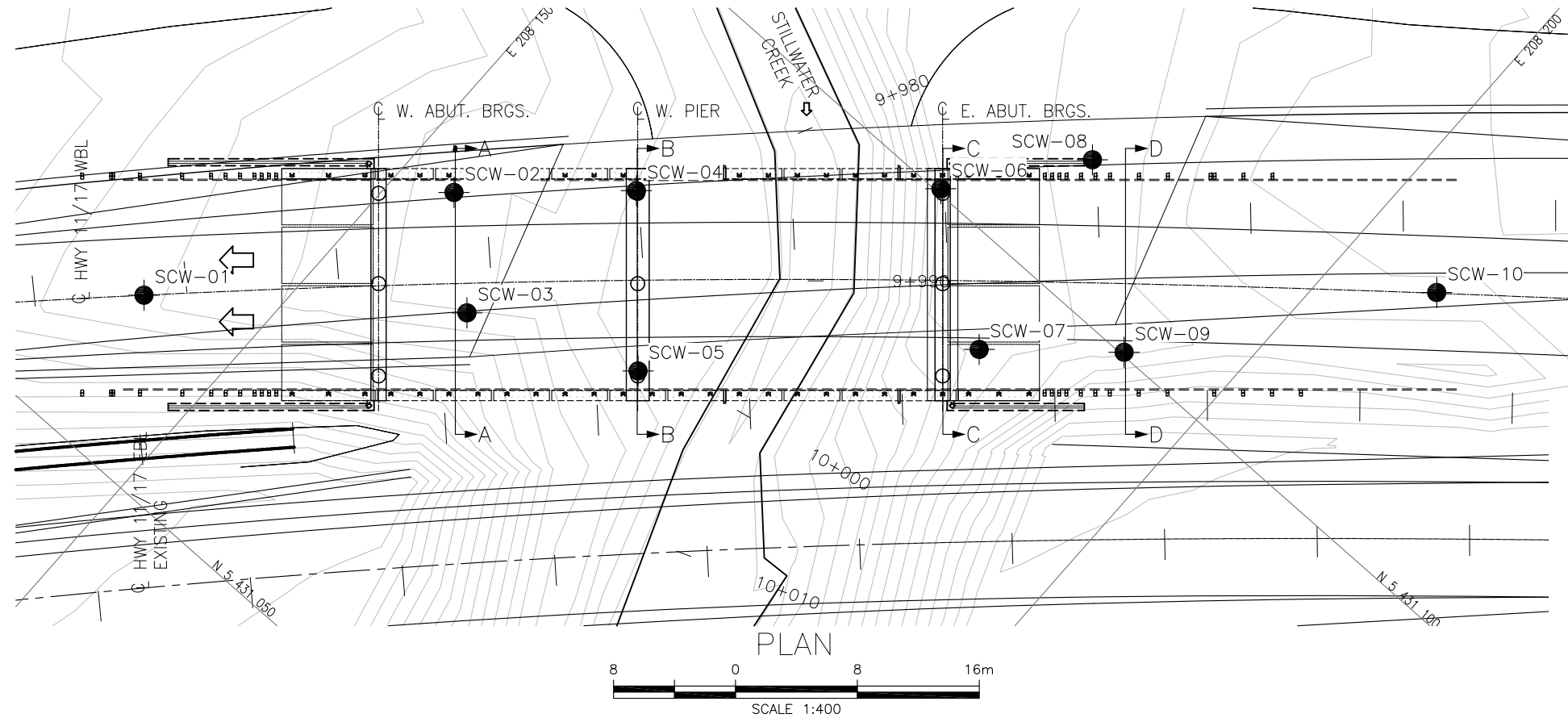


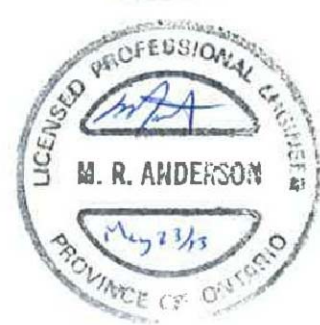
FIGURE F4

Appendix G

Drawing titled “Borehole Locations and Soil Strata”

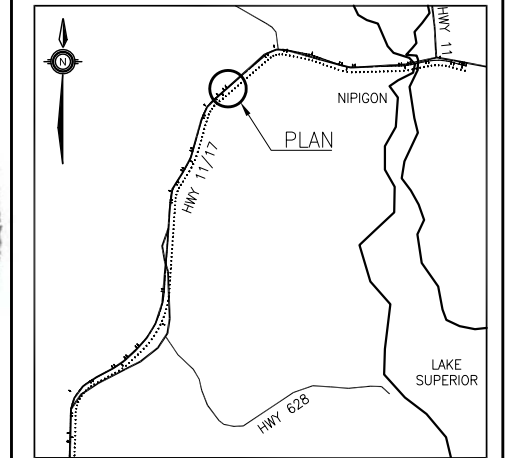


METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN



CONT No
WP No

HIGHWAY 11/17 FOUR LANING
STILLWATER CREEK BRIDGE
WESTBOUND LANES
BOREHOLE LOCATIONS AND SOIL STRATA



KEYPLAN

LEGEND

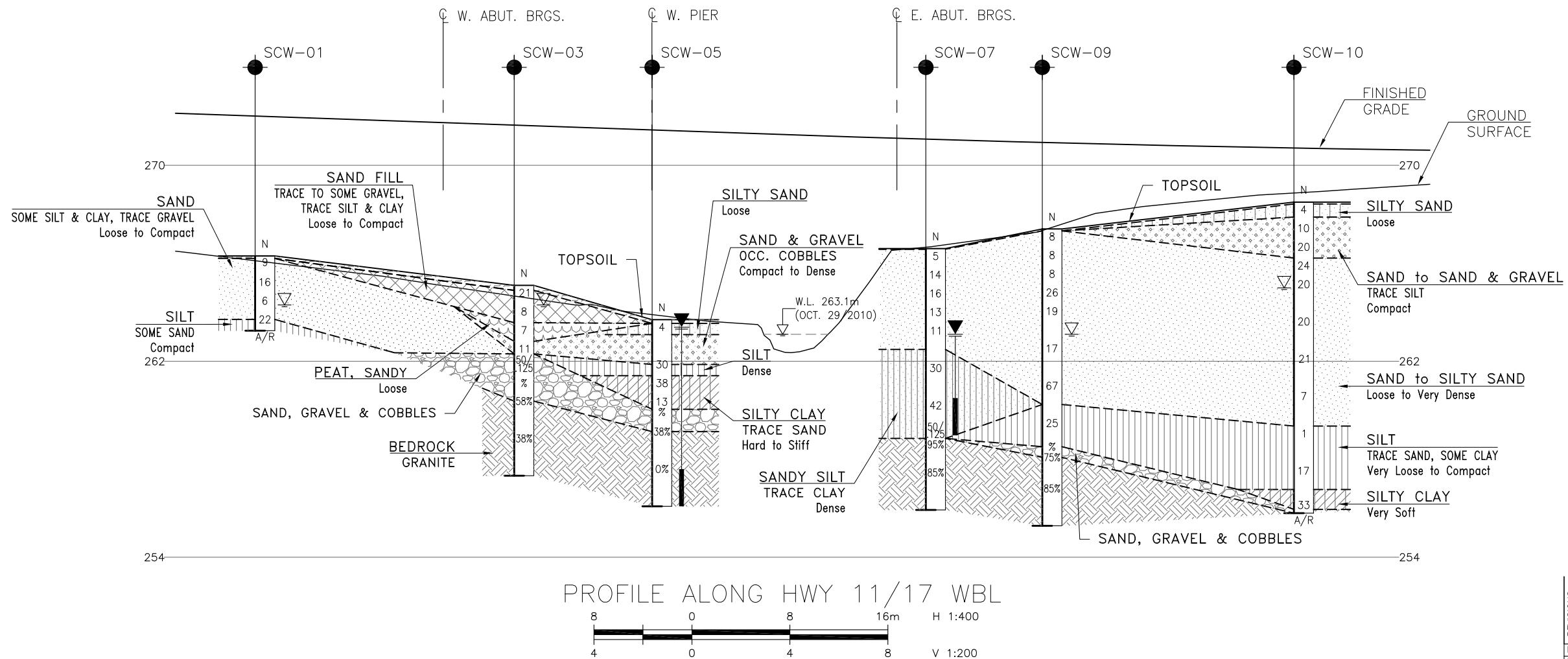
- Borehole
- Borehole and Cone
- N
- CONE
- PH
- Water Level During Drilling
- Water Level In Piezometer
- 90%
- A/R
- Rock Quality Designation (RQD)
- Auger Refusal

NO	ELEVATION	NORTHING	EASTING
SCW-01	266.3	5 431 060.5	208 142.8
SCW-02	264.6	5 431 079.0	208 153.6
SCW-03	265.1	5 431 073.6	208 159.5
SCW-04	263.9	5 431 087.0	208 162.5
SCW-05	263.7	5 431 078.2	208 170.4
SCW-06	266.5	5 431 100.3	208 177.4
SCW-07	266.6	5 431 094.1	208 186.3
SCW-08	267.1	5 431 108.3	208 183.6
SCW-09	267.4	5 431 100.2	208 193.5
SCW-10	268.5	5 431 116.7	208 206.3

-NOTES-

- The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.
- This drawing is for subsurface information only. Surface details and features are for conceptual illustration.

GEORES No. 52H-20



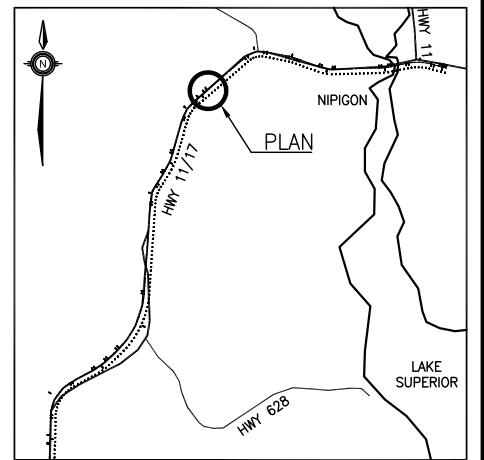
PROFILE ALONG HWY 11/17 WBL

REVISIONS	DATE	BY	DESCRIPTION
DESIGN	MC	CHK	MC
DRAWN	AN	CHK	
LOAD			
SITE	48C-096	STRUCT	DWG 1
DATE	MAY 2013		

METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

CONT No
WP No
HIGHWAY 11/17 FOUR LANE
STILLWATER CREEK BRIDGE
WESTBOUND LANES
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET



KEYPLAN

LEGEND

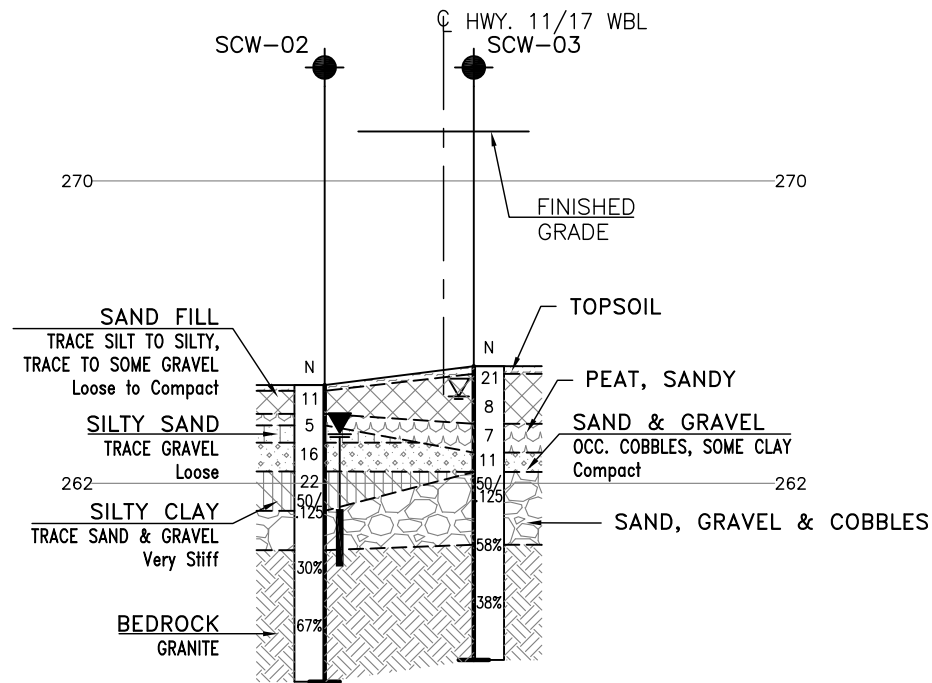
- Borehole
- Borehole and Cone
- N
- CONES
- PH
- Water Level During Drilling
- Water Level In Piezometer
- 90%
- A/R
- Rock Quality Designation (RQD)
- Auger Refusal

NO	ELEVATION	NORTHING	EASTING
SCW-01	266.3	5 431 060.5	208 142.8
SCW-02	264.6	5 431 079.0	208 153.6
SCW-03	265.1	5 431 073.6	208 159.5
SCW-04	263.9	5 431 087.0	208 162.5
SCW-05	263.7	5 431 078.2	208 170.4
SCW-06	266.5	5 431 100.3	208 177.4
SCW-07	266.6	5 431 094.1	208 186.3
SCW-08	267.1	5 431 108.3	208 183.6
SCW-09	267.4	5 431 100.2	208 193.5
SCW-10	268.5	5 431 116.7	208 206.3

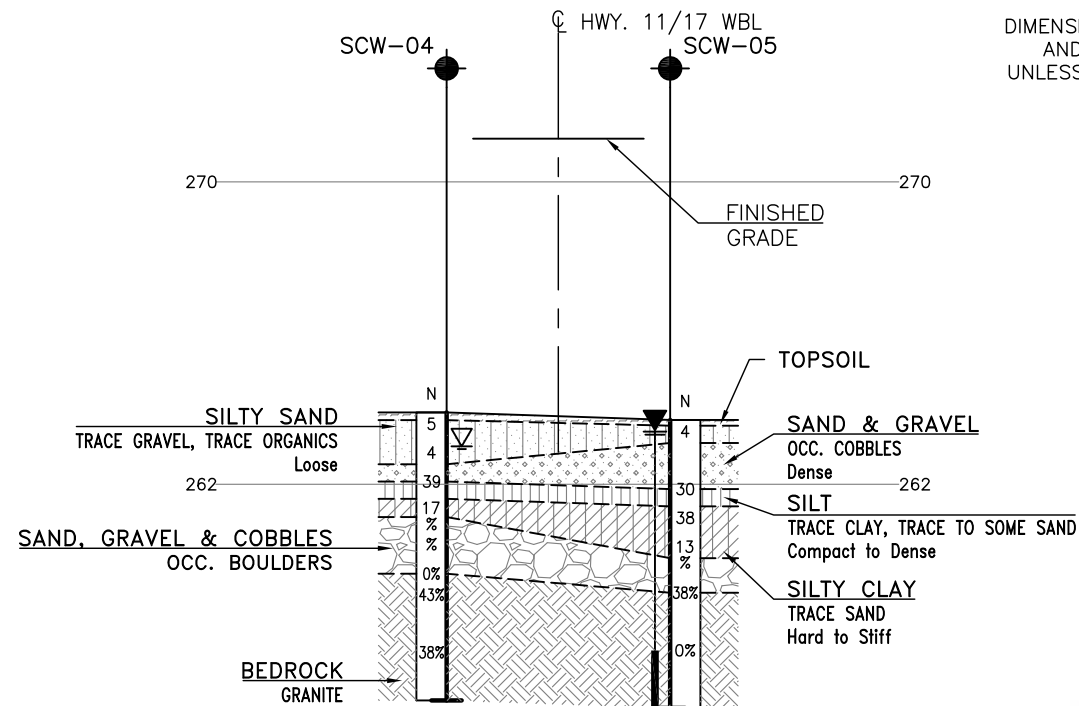
-NOTES-

- The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.
- This drawing is for subsurface information only. Surface details and features are for conceptual illustration.

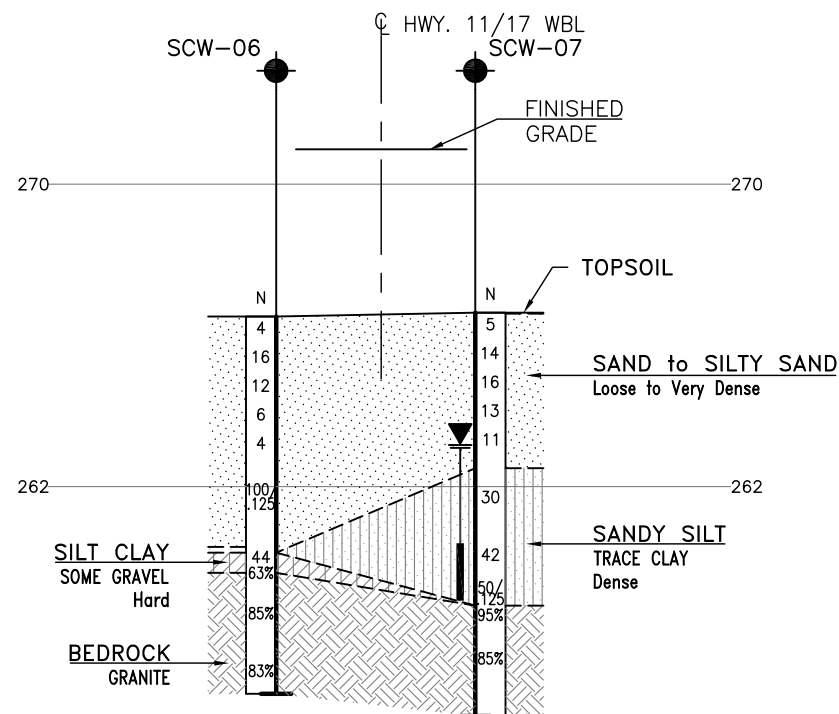
GEOCRES No. 52H-20



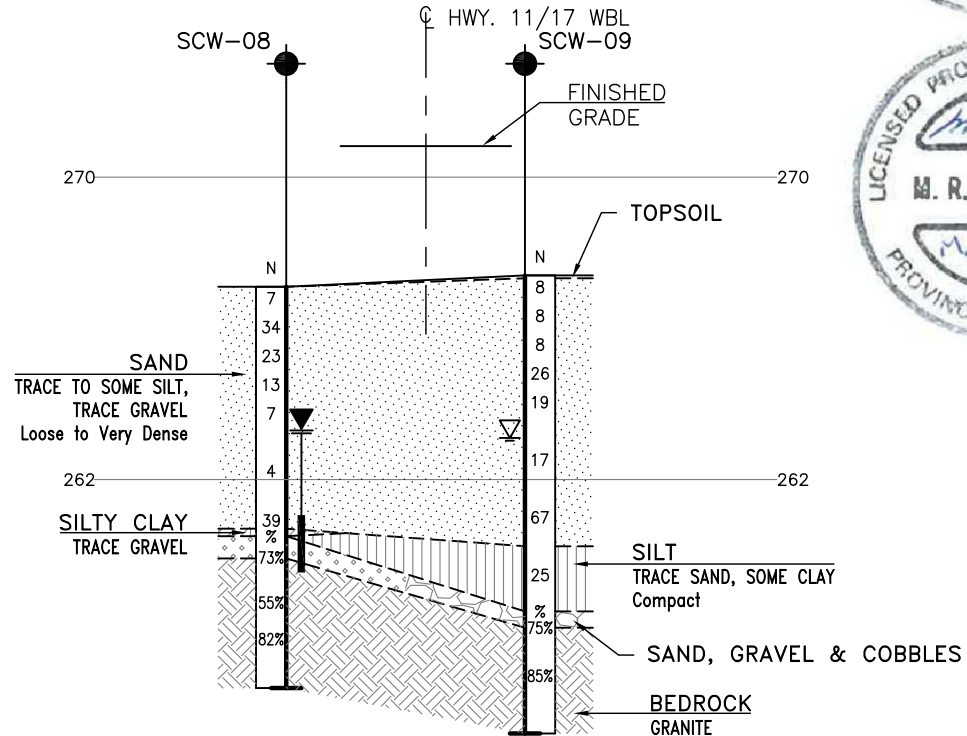
PROFILE ALONG A-A



PROFILE ALONG B-B



PROFILE ALONG C-C



PROFILE ALONG D-D



SCALE 1:400



REVISIONS	DATE	BY	DESCRIPTION
DESIGN	MC	CHK	MC
DRAWN	AN	CHK	