

**FOUNDATION INVESTIGATION AND DESIGN REPORT
HIGHWAY 11 SBL OVER STIRLING CREEK TRIBUTARY
HIGHWAY 11, BURK'S FALLS TO SOUTH RIVER
ONTARIO
G.W.P. 742-93-00, W.P. 5100-06-01, SITE: 44-438/2**

Geocres Number:31E-276

Report to

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April 28, 2008
File: 19-1423-39

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PART 1: FACTUAL INFORMATION

1 INTRODUCTION

This report presents the factual findings obtained from a foundation investigation conducted at the the location of a proposed bridge carrying Highway 11 SBL over Stirling Creek Tributary north of Burk's Falls, Ontario.

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 present investigation.

Thurber carried out the investigation as a sub-consultant to MMM Group Limited (MMM), under the Ministry of Transportation Ontario (MTO) Agreement Number 5005-A-000188.

2 SITE DESCRIPTION

The site lies on the east side of existing Highway 11 approximately 500 m north of the intersection with Pevensey Road.

At present, the creek flows from north to south along the east toe of the Highway 11 embankment and turns westward into a culvert under the existing highway at the proposed bridge site. A smaller creek flows east to west to join the main creek as it turns westward into the culvert. This smaller creek forms a small pond to the east of the proposed bridge, due to what may be an old, overgrown beaver dam.

The footprint of the new bridge partially overlaps the existing Highway 11 embankment.

A Hydro One pole line traverses the site in a generally north-south direction, parallel to existing Highway 11.

The site lies within the Canadian Shield, characterized by low, rounded hills of Pre-Cambrian bedrock mantled by varying thicknesses of overburden. At this site, the bedrock is mantled by cohesionless sand deposits that probably originated from glacial outwash and are typical of the soils encountered in this stretch of the Highway 11 corridor.

There is no development within the immediate vicinity of the site.

Photographs in Appendix G show:

1. A view looking north over the general site of the two structures with the small beaver pond in the middle ground and the Hydro One lines visible.
2. A view of the point where the two streams meet and flow westward under the existing highway.
3. A view looking south on existing Highway 11 over the south approaches to the future structures.
4. A view north along existing Highway 11 over the future SBL structure site. The north approaches are behind the trees to the right.

3 SITE INVESTIGATION AND FIELD TESTING

The site investigation and field testing for this project were carried out between June 24 and July 31, 2007 and consisted of drilling and sampling four boreholes at the foundation elements to depths ranging from 18.4 m to 20.9 m (elevations 292.3 m to 298.6 m) and two boreholes at the approach embankments to depths of 8.2 m and 10.1 m (elevations 310.8 m and 302.5 m). The boreholes were numbered SCS-1 to SCS-6 and their approximate locations are shown on the attached Borehole Locations and Soil Strata Drawing in Appendix H.

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

Drilling was carried out using a track mounted CME 75 drill rig. A combination of hollow-stem auger drilling techniques and rotary coring methods were used to advance the boreholes and samples were obtained at selected intervals using a split spoon sampler in conjunction with Standard Penetration Testing (SPT) in the overburden soils. Boreholes SCS-2, SCS-4 and SCS-5 were also advanced 3.2 m, 1.0 and 2.4 m, respectively, into bedrock by NQ size diamond coring.

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.

All rock cores were logged, and the Total Core Recovery (TCR), Rock Quality Designation (RQD) and the Fracture Indices (FI) were determined.

Groundwater conditions in the open boreholes were observed throughout the drilling operations. Standpipe piezometers consisting of 19 mm PVC pipe with slotted screens were installed and enclosed in filter sand in two boreholes (one at each foundation element) to permit longer term groundwater level monitoring. The locations and completion details of the piezometers are shown in Table 3.1.

Table 3.1 – Borehole Completion Details

Foundation Unit	Borehole	Piezometer Tip Depth/Elevation (m)	Completion Details
South Approach	SCS-1	None installed	Borehole grouted to surface using Aquagrout bentonite.
South Abutment			
West	SCS-2	None installed	Borehole grouted using bentonite grout to 2.4 m, auger cuttings to 1.5 m, holeplug to 0.6 m, auger cuttings to 0.15 m then asphalt to road surface.
East	SCS-3	19.4/295.6	Sand from 19.4 m to 16.2 m, holeplug from 16.2 m to 15.4 m, Aquagrout from 15.4 m to 0.5 m, holeplug to surface.
North Abutment			
West	SCS-4	None installed	Borehole grouted using bentonite grout to 1.2 m, holeplug to 0.6 m, auger cuttings to 0.15 m then asphalt to road surface.
East	SCS-5	12.2/300.8	Borehole drilled to 20.7 m, elevation 292.3 m but piezometer could not be advanced below 12.2 m, boreholes assumed to then collapse below that depth. Sand from 12.2 m to 10.4 m, holeplug from 10.4 m to 9.8 m, Aquagrout from 9.8 m to 0.9 m and holeplug to surface.
North Approach	SCS-6	None installed	Borehole grouted with bentonite to the surface.

4 LABORATORY TESTING

The recovered soil samples were subjected to Visual Identification (VI) and to natural moisture content determination. The results of this testing are shown on the Record of Borehole sheets in Appendix A. Selected samples were also subjected to gradation analysis and the results of this testing program are shown on the Record of Borehole sheets in Appendix A and on the figures contained in Appendix B.

Point load tests were carried out on selected samples of intact bedrock upon arrival at the laboratory to assist in evaluation of the compressive strength of the bedrock. Results of point load tests on the selected rock core samples are shown in Table 1 immediately following the text and on the Record of Borehole sheets in Appendix A.

5 DESCRIPTION OF SUBSURFACE CONDITIONS

Reference is made to the Record of Borehole sheets in Appendix A. Details of the encountered soil and rock stratigraphy are presented in these sheets and on the "Borehole Locations and Soil Strata" and "Stratigraphic Sections" drawings in Appendix H. 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.

In general terms, the soil stratigraphy encountered at this site consists of topsoil overlying about 17 m to 20 m of cohesionless soils consisting of fill, native sand, occasional layers of silt, and layers of sand with gravel, cobbles and boulders. Migmatitic gneiss bedrock was contacted below the native soils. More detailed descriptions of the individual strata are presented below.

5.1 Pavement Structure

Pavement structure consisting of approximately 125 mm of asphalt overlying granular (sand and gravel fill) road base was encountered in Boreholes SCS-1, SCS-2 and SCS-4 drilled on the Highway 11 SBL existing lanes and shoulders.

5.2 Topsoil

Topsoil was identified at ground surface in Borehole SCS-3. The topsoil thickness generally was 300 mm. The topsoil thickness may vary between and beyond the borehole locations and the data is not intended for the purpose of estimating quantities.

5.3 Fill

Fill was contacted below the pavement structure in Boreholes SCS-1, SCS-2 and SCS-4, and surficially in Borehole SCS-6. The fill generally consists of cohesionless layers of brown sand containing some gravel, trace of silt and clay and occasional cobbles and rootlets. Although not directly encountered in the boreholes, the existing fill may contain boulders.

Fill extended to depths ranging from 2.2 m to 2.4 m (elevation 316.6 m) at the locations of the south abutment and south approach (Boreholes SCS-2 and SCS-1). At the north abutment and north approach (Boreholes SCS-4 and SCS-6), fill extended to depths of 7.7 m and 3.0 m (elevations 309.3 m and 309.5 m), respectively.

SPT N-values recorded in the sand fill ranged from 2 to 40 blows per 0.3 m penetration indicating a very loose to dense relative density. Higher 'N' values (more than 50 blows

for under 0.3 m penetration) were also observed within the fill at various depths, indicating a very dense relative density. These high values may be attributed to the presence of cobbles within the fill. The moisture content of samples collected ranged from 2% to 21%.

The grain size distribution of the cohesionless fill is represented by the data plotted in Figures B1 Appendix B.

The results of gradation conducted on selected samples of fill are summarized below:

Soil	(%)
Gravel	0
Sand	91 – 98
Silt & Clay	2 – 9

Despite no gravel being found in the sieve analysis, field observations and occasional SPT 'N' values exceeding 100 blows for 0.3 m of penetration, indicate scattered gravel and cobble sizes.

5.4 Sand

An extensive deposit of native brown sand containing some gravel to gravelly, trace to some silt and trace of clay was generally contacted below the fill and topsoil, and surficially in Borehole SCS-05. Occasional cobbles and boulders were also noted within the native sand deposit.

Deeper layers of brown and grey sand with occasional cobbles and boulders were also contacted in Boreholes SCS-4 and SCS-6 at 13.0 m and 9.1 m depth (elevations 304.0 m and 303.4 m), respectively.

Layers of cobbles and boulders were also encountered within the native sand as follows:

Location	Borehole	Depth/Elevation (m)	Thickness (mm)
South Abutment	SCS-3	2.4/312.5	600
North Abutment	SCS-5	15.7/297.3	500

Boreholes SCS-1 and SCS-6 were terminated within the sand deposit at 8.2 m and 10.1 m depth (elevations 310.8 m and 302.5 m).

Thickness of the sand layer, determined from Boreholes SCS-2 to SCS-5, generally ranged from 4.4 m to 18.3 m. The depth to the base of the sand deposit ranged from Elevations 304.3 m to 305.1 m at the south abutment and from 294.7 m to 299.6 m at the north abutment.

SPT 'N' values ranged from 11 to 75 blows for 0.3 m penetration in this stratum indicating a compact to very dense relative density. Higher 'N' values (more than 50 blows for under 0.3 m penetration) were observed in Boreholes SCS-2 to SCS-5 at various depths. These high values may be due to the presence of cobbles and boulders within the deposit.

The moisture content of samples from this deposit ranged from 2% to 23%; moisture contents higher than 12% were generally observed below elevation 308.0 m.

Grain size distribution curves for the samples tested are presented on the Record of Borehole sheets and on Figures B2 and B3. The results of laboratory tests carried out on samples of the sand were as follows:

Soil Particles	(%)
Gravel	0 to 32
Sand	66 to 98
Silt & Clay	1 to 9

5.5 Silt

Native brown and grey silt containing some sand to sandy, some clay and trace of rootlets were observed in Boreholes SCS-3 and SCS-4 at 19.4 m and 7.7 m depths (elevations 295.6 m and 309.3 m), respectively.

SPT N-value measured in the silt layer was 8 blows for 0.3 m of penetration in Borehole SCS-4, indicating a loose density. Higher 'N' value (more than 50 blows for under 0.3 m penetration) was measured in Borehole SCS-3, indicating a very dense relative density.

The natural moisture content of one sample recovered from the silt layer was 40%.

5.6 Sand and Gravel

Layers of brown to grey sand and gravel containing occasional cobbles and boulders were observed in Boreholes SCS-2 to SCS-4 and SCS-6 at depths ranging from 6.1 m to 14.5 m (elevations 304.3 m to 308.3 m).

Standard Penetration tests in this deposit gave 'N' values ranging from 32 blows per 0.3 m of penetration to greater than 100 blows for 0.075 m of penetration, indicating a compact to very dense density. Higher 'N' values (more than 50 blows for under 0.3 m penetration) may be due to the probable presence of cobbles and boulders within the deposit.

The moisture content of samples from this deposit varies between 11% and 17%.

Grain size distribution curves for the samples tested are presented on the Record of Borehole sheet and on Figure B4. The results of laboratory tests carried out on samples of the sand and gravel are summarized as follows:

Soil Particles	(%)
Gravel	37 to 50
Sand	45 to 54
Silt & Clay	2 to 11

5.7 Bedrock

The overburden soils described above are underlain by Pre-Cambrian migmatitic gneiss bedrock. Bedrock was proved by coring at both abutments. The migmatitic gneiss bedrock is described as fresh to slightly weathered. Its colour is dark grey to black with occasional pink bands visible in most cores.

Table 5.2 summarizes the bedrock depths and the elevations to the top of bedrock.

Effective refusal, defined as an SPT value exceeding 100 blows for 0.3 m of penetration (or 50 blows for less than 150 mm penetration), was encountered in the sand with cobbles and boulders. The depths at which effective refusal was encountered are also shown in Table 5.2.

Table 5.2 – Depths and Elevations of Refusal and Top of Bedrock

Foundation Unit	Borehole	Refusal		Top of Bedrock	
		Depth (m)	Elevation (m)	Depth (m)	Elevation (m)
South Approach	SCS-1	-	-	-	-
South Abutment					
West	SCS-2	13.6	305.2	17.7	301.1
East	SCS-3	15.2	299.8	19.6*	295.4*
North Abutment					
West	SCS-4	17.0	300.0	17.4	299.6
East	SCS-5	16.2	296.8	18.3	294.7
North Approach	SCS-6	-	-	-	-

*Possible bedrock inferred from refusal to sampling

Core recovery in the bedrock was 100%. The RQD values generally ranged from 63% to 100% indicating fair to excellent rock quality.

The Fracture Index (FI) of the rock, expressed as fractures per 0.3 m of core, was generally low ranging from 0 to 6 although zones of >10 fractures per 0.3 m of core were recorded in

Borehole SCS-5. Horizontal and vertical joints were encountered within the rock mass in all the cores.

The unconfined compressive strength of the rock cores is approximately 120 to 150 MPa indicating a very strong rock. The estimated rock strength value is based on one point load tests conducted on rock cores recovered from Borehole SCS-5. A summary of the Point Load Test Results is presented in Table 1 immediately following the text of this report.

5.8 Water Levels

Water levels were observed in the boreholes during and upon completion of drilling. Standpipe piezometers were installed in two boreholes to monitor water levels after completion of drilling. The water levels measured in the piezometers are summarized in Table 5.3, along with the measurements in the boreholes upon completion of drilling.

Table 5.3 – Water Level Measurements

Foundation Unit	Borehole	Date (2007)	Water Level (m)		Comment
			Depth	Elevation	
South Approach	SCS-1	July 31	-	-	Open borehole
South Abutment					
West	SCS-2	July 30	-	-	Open borehole
East	SCS-3	June 27	4.1	310.9	In piezometer
		June 28	4.1	310.9	
		July 6	4.3	310.7	
		July 23	5.0	310.0	
North Abutment					
West	SCS-4	July 31	-	-	Open borehole
East	SCS-5	July 5	3.1	309.9	Open borehole
		July 23	3.4	309.6	In piezometer
North Approach	SCS-6	July 5	3.1	309.5	Open borehole

The piezometric readings indicate that the groundwater level is near Elevation 310.0 m.

GA Drawing indicates that the water level of the Stirling creek was 311.8 m in December, 1997.

The above values are short-term readings and seasonal fluctuations of the groundwater level are to be expected. In particular, the groundwater level may be at a higher elevation after the spring snowmelt or after periods of heavy rainfall.

6 MISCELLANEOUS

Borehole locations were selected by Thurber Engineering Ltd. Surveyors from MMM Group Limited staked these locations in the field, confirmed the co-ordinates and obtained the ground surface elevations.

Thurber obtained utility clearances for the borehole locations prior to drilling.

George Downing Estate Drilling Ltd. supplied a track mounted CME 75 drill rig and conducted the drilling, sampling and in-situ testing operations.

The field program was supervised on a full time basis by Mr. George Azzopardi, Mr. Stephane Loranger and Ms. Jessica Lee of Thurber.

Routine laboratory testing was carried out by Thurber Engineering Ltd.

Overall supervision of the field program was conducted by Mr. Alastair E. Gorman, P.Eng. Interpretation of the data and preparation of the report were carried out by Mr. Alastair E. Gorman, P.Eng and Ms. R. Palomeque Reyna, P.Eng.

The report was reviewed by 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 design recommendations to assist the design team to select and design a suitable foundation system and approach embankments for the proposed structure.

It is understood that the proposed bridge is to carry the new Highway 11 SBL over Stirling Creek Tributary, located north of Burk's Falls, Ontario.

Based on the preliminary General Arrangement (GA) drawing provided by MMM Group Limited, a one-span structure supported on integral abutments is proposed. The span will be 57 m long. The proposed finished grade at the structure will be about Elevation 325.6 m at the north abutment and the original ground surface is near Elevation 313 m, resulting in an approach embankment of 12.6 m high. At the south abutment, the finished grade will be at Elevation 326.4 m and the original ground surface is at average Elevation 315.0 m, resulting in an approach embankment up to 11.4 m high.

The discussion and recommendations presented in this report are based on our understanding of the project and on the factual data obtained in the course of the investigations.

8 STRUCTURE FOUNDATIONS

The proposed structure is a one-span bridge with two abutments.

The stratigraphy encountered at the abutment locations consist of pavement structure and fill overlying extensive deposits of cohesionless native sand and sand and gravel containing cobbles and boulders. Thickness of the cohesionless soils varies from 17.0 m to 20.0 m. Migmatitic gneiss bedrock was contacted below the sand deposits at depths ranging from 17.4 m to 18.3 m (Elevations 294.7 m to 301.1 m). The groundwater level is high and lies near elevation Elevation 310.0 m and as a result most of the overburden sands and silts are below the groundwater level.

Initial consideration was given to the following foundation types:

- Spread footings on native soils
- Spread footings on engineered fill
- Augered Caissons (drilled shafts)
- Driven piles

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

8.1 Spread Footings on Native Ground

At the north abutment, the near surface soils consist of 7.7 m of sand fill overlying native loose to compact sand and silt layers. Footings must not be founded on the fill and loose native soils and footings should bear on the underlying competent native soils.

At the south abutment, spread footings can be founded on the compact native undisturbed sand, contacted below the 2.2 m of fill (Borehole SCS-2).

Footings at the north and south abutments should not be placed in the existing embankment fills.

The highest permitted founding elevations for spread footings are given in Table 8.1.

Table 8.1 – Highest Permitted Founding Elevations

Foundation Unit	Borehole	Footing on Native Undisturbed Soil	
		Depth below existing ground surface (m)	Founding Elevation (m)
South Abutment			
West	SCS-2	3.0	315.8
East	SCS-3	1.0	314.0
North Abutment			
West	SCS-4	9.0	308.0
East	SCS-5	3.0	310.0

Provided a minimum footing width of 2 m is maintained abutment footings founded on the compact sand at or below the elevations shown in Table 8.1 above, may be designed for the following resistance values:

- Factored geotechnical resistance of 300 kPa at Ultimate Limit States (ULS)
- Geotechnical resistance of 200 kPa at Serviceability Limit States (SLS)

The geotechnical resistances quoted above are for concentric, vertical loads only. In the case of eccentric or inclined loading, the geotechnical resistance must be calculated as illustrated in the CHBDC Clause 6.7.3 and Clause 6.7.4.

Elevations indicated in Table 8.1 will be in cohesionless soils at, or possible below the groundwater table, making spread footings a less desirable alternative at the site.

The geotechnical SLS resistance values given above are based on an estimated total settlement not exceeding 25 mm. This settlement is expected to be substantially complete by the end of construction. Differential settlement is not expected to exceed 20 mm.

The sliding resistance of mass concrete poured on the native cohesionless soil may be computed on the basis of an ultimate coefficient of friction of 0.55. This is an “ultimate” value and requires a degree of sliding movement to occur to fully mobilize the resistance.

The bases of the foundation excavations should be inspected by a geotechnical engineer to confirm that the exposed surface conforms to the design requirements and has been adequately prepared to receive concrete. Where subexcavation is required to remove unsuitable material from below the design founding level, the founding surface should be re-established using engineered fill. The engineered fill must consist of OPSS Granular “A” placed in 150 mm lifts, compacted to 100% of its SPMDD at $\pm 2\%$ of optimum moisture content.

All footings should be provided with a minimum of 1.9 m of earth cover over the footing base (founding elevation) as protection against frost action.

8.2 Spread Footings on Engineered Fill

Consideration was also given to placing spread footings on an engineered fill pad if higher founding levels are required. At this site, the engineered fill will bear on compact to dense native sand. The permitted founding/base elevations at which engineered fill should be placed, are given in Table 8.1.

If an engineered fill pad is used at this site, all topsoil, fill or other deleterious materials must be stripped from the footprint of the engineered fill to expose competent material.

The engineered fill must consist of OPSS Granular “A” placed in 150 mm lifts and compacted to 100% of its SPMDD at $\pm 2\%$ of optimum moisture content and generally conforming to the geometry illustrated in Figure 1 in Appendix D. The thickness of engineered fill must be a minimum of 2.0 m, to facilitate compaction, and adequate protection must be provided to prevent erosion or scour of the engineered fill if it is exposed to the creek.

Provided a minimum footing width of 2 m is maintained footings bearing on the well compacted engineered fill may be designed for the following values:

- Factored geotechnical resistance of 900 kPa at Ultimate Limit States (ULS)
- Geotechnical resistance of 350 kPa at Serviceability Limit States (SLS)

These resistance values are for concentric, vertical loads only. In the case of eccentric or inclined loading, the geotechnical resistance must be calculated as illustrated in the CHBDC Clause 6.7.3 and Clause 6.7.4.

For footings designed on the basis of the geotechnical resistance values given above, total settlement under a footing is expected to not exceed 25 mm. Differential settlements are not expected to exceed 20 mm across the width of the structure.

The lateral resistance of the footings founded on engineered fill may be computed using an unfactored friction of 0.7. This is an “ultimate” value and requires a degree of sliding movement to occur to fully mobilize the resistance.

If temporary excavations required to construct the engineered fill pads extends below the water table, dewatering prior to excavation must be conducted to construct the fill pad and the footing in the dry and to prevent sloughing of the sides or disturbance of the base of the excavation due to the inflow of groundwater. The dewatering method adopted must depress the groundwater level to at least 0.5 m below the base of excavation and must maintain a stable, unwatered excavation throughout the duration of the fill pad and footing construction. Dewatering must remain operational and effective until the footing is constructed and backfilled or until the engineered fill pad is completed to a level at least 0.5 m above the groundwater level.

The option of spread footings on an engineered fill pad is feasible for the support of the structure at this site.

8.3 Augered Caissons (Drilled Shafts)

Augered caisson foundations were also considered for the support of the structure. However, the cohesionless overburden is not considered suitable for caisson support and the caissons must be founded on the bedrock at depths in the order of 17.4 m to 19.6 m below original ground surface. The base of the caissons would be about 15 m below the groundwater level, resulting in high hydrostatic heads at the base.

The permeable nature of the overburden soil and the presence of boulders above the bedrock would make it difficult to seal the bottom of the caisson liner into the founding stratum to exclude groundwater. Unwatering of the caissons would be impractical and attempts to do so might result in continued flow of fines into the caisson excavation.

Installation of deep caissons to bedrock is also expected to be a more expensive option than driven piles.

For these reasons, the use of a caisson foundation is not recommended.

8.4 Driven Piles

The subsurface conditions at the site are considered suitable for the design of foundations supported on steel H-piles driven to achieve resistance in the very dense soil or on bedrock.

At both abutments, the bedrock is mantled by 2.5 m to 4.0 m of very dense sand and gravel, possibly containing cobbles and boulders. The presence of boulders and cobbles makes it difficult to predict the depth at which piles will achieve the required resistance. Resistance might be developed in the very dense soil or piles may fully penetrate the soil layer and achieve resistance on the underlying bedrock. Consequently, it is recommended that the foundations be designed on the basis of the geotechnical resistance achieved by piles driven into the very dense soil.

The elevations at which the piles are expected to develop the required resistance are given in Table 8.2.

Table 8.2 – Estimated Pile Tip Elevation

Foundation Unit	Borehole	Anticipated Pile Tip Elevation To Develop Required Resistance	Anticipated Pile Length below original ground (m)	Anticipated Founding Material
South Abutment				
West	SCS-2	304.5	14.3	Very dense sand and gravel
East	SCS-3	300.0	15.0	Very dense sand and gravel
North Abutment				
West	SCS-4	302.0	15.0	Very dense sand
East	SCS-5	301.0	12.0	Very dense gravelly sand

The pile tip elevations shown in Table 8.3 should be used for estimating purposes only. The actual pile tip elevations will be controlled as described in Section 8.4.4 Pile Driving.

8.4.1 Axial Resistance

The vertical, axial geotechnical resistances at Serviceability Limit States (SLS) for three pile sections when driven into very dense soil are presented in Tables 8.3.

In the event that the piles do not develop the required resistance in the dense to very dense soil, they will develop resistance on the underlying bedrock and the resistance values computed for native very dense sand will be conservative.

Table 8.3 – Axial Resistance of Three Pile Sections Founded on Very Dense Soils

Soil Strata	Pile Section					
	HP 310 x 110		HP 310 x 152		HP 360 x 174	
	SLS (kN)	ULS (Factored) (kN)	SLS (kN)	ULS (Factored) (kN)	SLS (kN)	ULS (Factored) (kN)
Very dense sand	1,600	1,800	1,700	1,900	2,200	2,400

8.4.2 Pile Tips

Due to the presence of cobbles and boulders in the expected founding layer, the tips of all piles should be fitted with cast steel, H-section rock points from an approved manufacturer such as Titus Steel (Standard H-point) or approved equivalent.

The use of rock points is recommended for the following reasons:

- Some piles will be driven into soil containing cobbles and boulders, which requires a higher level of protection than driving into soils containing only smaller particle sizes
- Some piles may achieve refusal on large boulders, which will require the same pile tip protection and reinforcement as founding on bedrock

8.4.3 Pile Installation

Pile installation should be in accordance with Special Provision No. 903S01.

The Contract Documents should contain a NSSP alerting the Bidders to:

- The presence of cobbles and boulders in the expected bearing stratum.
- The possibility of piles within a group achieving the specified resistance at different elevations.
- The possibility of some piles meeting refusal on a large boulder.

Suggested texts for the NSSP's are included in Appendix E.

8.4.4 Pile Driving

Pile driving must be controlled by the Hiley Formula and an ultimate pile resistance to be specified by the designer in accordance with Clause 3.3.2 (b) Construction Stage of the Structural Manual. The Hiley formula need not be used until the piles are approaching the bearing stratum below Elevation 304.0 m at the north abutment and 306.0 m at the south abutment. The appropriate pile driving note is "Piles to be driven in accordance with Standard SS 103-11 using an ultimate resistance of "R" kN per pile". "R" must have the minimum values shown in Table 8.4.

Table 8.4 – Ultimate Geotechnical Resistance of Piles

Pile	Ultimate Resistance (R) (kN)
HP 310x110	3,600
HP 310x152	3,800
HP 360x174	4,800

The NSSP should require the QVE to terminate driving before the pile is damaged by overdriving.

To facilitate pile installation, embankment fill through which piles will be driven must not contain oversize material, i.e. no particles exceeding 75 mm in size.

8.4.5 Downdrag

The soils at this site are non-cohesive and settlements induced in the foundation soils by construction of the approach fills will be substantially complete as construction of the embankment is completed and downdrag on the piles is not considered to be an issue at this site. However, it is recommended that the approach embankments be constructed at least up to the level of the base of the CSPs prior to pile driving.

8.4.6 Integral Abutment Considerations

The ground conditions at this site are considered suitable for an integral abutment design. The use of H-piles at the abutments allows for the design of an integral abutment structure.

The integral abutment design requires that the piles possess flexibility in the upper 3 m of the pile length. The near surface, native soils at this site are loose to compact and the lateral resistance of a pile in this soil might provide sufficient flexibility. However, the upper 3 m of the pile may lie partially within the compacted fill of the approach embankment and partially in the underlying native soils, which may become densified under the embankment loading. Accordingly, to provide the required flexibility in the

piles, the upper 3 m of the piles should be surrounded by a 600 mm diameter CSP as specified by the integral abutment design procedures.

After the pile is driven, the space between the pile and the CSP should be filled with sand. An NSSP should be included in the contract drawings specifying the gradation of the sand according to Table 8.5 and SP included in Appendix E.

Table 8.5 – Integral Abutment Sand Grading

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

8.4.7 Lateral Resistance

The lateral resistance of the pile 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 in metres

n_h = value from Table 8.6

γ = unit weight (Table 8.6)

K_p = passive earth pressure coefficient (Table 8.6)

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, $K = k_s \cdot L \cdot D$ (kN/m), where k_s is the coefficient of horizontal subgrade reaction (kN/m³), D is the pile width (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, $P_{ult} = p_{ult} \cdot L \cdot D$. This represents the ultimate load at which the pile fails and will not support any additional load at greater displacements. It is recommended, however,

that the total lateral resistance in one pile be limited to no more than 150 kN at ULS and 50 kN at SLS. Parameters for lateral pile resistance are shown in Table 8.6.

Table 8.6 – Parameters for Lateral Pile Resistance

Location	Elevation	n_h (kN/m ³)	K_p	Unit Weight (kN/m ³)	Soil Conditions
South Abutment	OGI to 310.0	3,500	3.3	21	Sand, very loose to dense
	310.0 to 304.0	6,000	3.3	11*	Sand, loose to very dense
	Below 304.0	8,000	4.0	11*	Sand and gravel, cobbles, very dense
North Abutment	OGI to 312.0	3,500	3.0	21	Sand, very loose to compact
	312.0 to 308.0	2,500	3.0	11*	Sand, very loose to compact
	308.0 to 297.0	6,000	3.0	11*	Sand, compact to very dense
	Below 297.0	8,000	4.0	11*	Sand and gravel, cobbles, very dense

*Buoyant unit weight below the water table.

Pile interaction should be considered with reference to CHBDC Clause 6.8.9.2.

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

For conventional abutments, the lateral resistance may be provided by battered piles.

8.5 Recommended Foundation

From a geotechnical point of view, it is recommended that all foundations for the bridge structure (abutments) be supported on steel H-piles driven into the very dense soil.

8.6 Frost Cover

The design depth of frost penetration at this site is 1.9 m.

Frost protection should be provided for the undersides of all foundation elements and should consist of a minimum of 1.9 m of soil cover.

9 EXCAVATION

All excavation must be carried out in accordance with 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 soils below the water table. This classification is based on the lack of cohesion in the soils and the resulting possibility that excavation slopes will slough if excavated vertically for the lower 1.2 m. Excavation slopes should not exceed 1H:1V above the groundwater level.

The excavation and backfilling for foundations must be carried out in accordance with SP 902S01.

Excavation below the groundwater level without prior dewatering is not recommended since the inflow of groundwater will cause boiling and sloughing of the soil below the water table making it difficult to maintain a dry, sound base on which to work.

Prior to excavation below the natural groundwater level, the groundwater must be depressed to a level below the deepest excavation level sufficient to maintain a stable base and prevent soil disturbance by construction traffic.

Bidders must be alerted to the fact that excavation must be carried out through cohesionless soils, which may include cobbles and boulders.

10 UNWATERING

Piezometers installed in boreholes revealed that groundwater level is near Elevation 310.0 m.

If spread footings are the selected design for foundations, water may be encountered during excavation. Dewatering to lower the groundwater level below the footing excavation base will be required prior to the start of footing excavation. The Contractor should also be prepared to pump from sumps to remove any remaining seepage water or surface water collecting in an excavation. Placement of concrete or compacting granular engineered fill must be done in the dry. Unwatering must remain operational and effective until the footing is constructed and backfilled.

The Contract Documents should also contain a NSSP alerting the Contractor to the risks associated with excavation of cohesionless soils submerged below the groundwater level without prior dewatering. Suggested wording is included in Appendix E.

The design of the dewatering system that may be required is the responsibility of the Contractor and the Contract Documents must alert him to this responsibility and the need to engage a dewatering specialist. While the responsibility for dewatering remains with the Contractor, suitable systems that might be employed include pumping from filtered sumps for penetration of no more than 0.5 m below the groundwater level and the use of vacuum wellpoints for deeper penetration below the groundwater level. Vacuum wellpoints in conjunction with sheetpiled cofferdam may be required due to the proximity of the creek.

11 APPROACH EMBANKMENTS

Approach embankment construction using either earth fill or rock fill is feasible on the foundation soils encountered at this site. At the north abutment, settlement in the order of 60 to 70 mm is estimated in the foundation soils under a 12.6 m high approach fill. Similarly at the south abutment, 30 to 40 mm of settlement is estimated in the foundation soils under an 11.4 m high approach fill. Due to the non cohesive nature of the foundation soils, these settlements will be immediate and essentially completed when construction of the fill is completed.

Post construction settlement of the fill mass is estimated to be 60 mm at both the north and south approaches. Based on the above settlement estimates, it is considered prudent to overbuild the approach embankment to account for a total settlement of 100 mm.

Rock fill embankments should be overbuilt in accordance with current Northeastern Region policies and guidelines.

The global, internal and surficial stability of the approach embankment fills will depend on the slope geometry and also to a large degree on the material used to construct the embankments. If the embankments are constructed of blast rock fill, it may be assumed that the side slopes will be stable at inclinations up to 1.25H:1V. Embankments constructed using granular material, select subgrade material or non-cohesive earth fill will have stable side slopes at inclinations of up to 2H:1V.

For the purpose of embankment stability analyses, the commercially available slope stability program GSLOPE developed by Mitre Software Inc. was used. The Bishop's simplified method for stability analysis was employed.

Global stability analyses were conducted for 2H:1V SSM or earth fill embankments and for 1.25H:1V rock fill embankments. The stability of the embankments was also checked under seismic loading assuming an acceleration of 0.08g. The computed factors of safety are as shown in Table 11.1. Slope stability computation outputs are included in Appendix F.

Table 11.1 Computed Factors of Safety

Location / Material	Condition	Factor of Safety	Figure (Appendix F)
South Approach			
Rock Fill	Normal	1.7	1
Rock Fill	Seismic = 0.08g	1.4	2
Earth Fill	Normal	1.6	3
Earth Fill	Seismic = 0.08g	1.3	4
North Approach			
Rock Fill	Normal	1.7	5
Rock Fill	Seismic = 0.08g	1.4	6
Earth Fill	Normal	1.5	7
Earth Fill	Seismic = 0.08g	1.3	8

In each case of normal loading, the factor of safety against global failure was greater than 1.4. Under the assumed seismic loading, the minimum factor of safety calculated was 1.3. These factors of safety are considered to be acceptable for the proposed embankment bearing on non-cohesive soil.

It is recommended that all topsoil and peat be stripped prior to constructing the approach fills. Embankment construction should be in accordance with OPSS 206, as amended by Special Provision "Amendment to OPSS 206, December 1993", dated November 2002.

The approach fills should be constructed in advance of pile driving operations.

Where earth fill embankments are higher than 8 m, mid-height berms should be incorporated in the design. The berms should:

- extend for the length through which the embankment height exceeds 8 m
- be at least 2 m wide
- have 2% positive grade to shed run-off water.

Where rock fill embankments are higher than 10 m, mid-height berms should be incorporated in the design. The berms should:

- extend for the length through which the embankment height exceeds 10 m
- be at least 2 m wide

Earth fill embankment slopes must be provided with erosion protection in accordance with OPSS 572.

12 BACKFILL TO ABUTMENTS

In the case of integral or semi-integral abutments, backfill to the abutment should be granular material.

In the case of a conventional abutment, granular backfill is recommended but rock backfill can be permitted. A NSSP is required to specify grading limits for the rock fill. The rock fill used as backfill to the abutment should be limited to fragments no greater than 75 mm.

In all cases where the approach embankment consists of rock fill and granular backfill to the abutment wall is used, the granular backfill must consist of OPSS Granular "B" Type II.

The backfill to the abutment walls must be in accordance with OPSS 902 as amended by Special Provision 902S01. Granular backfill must be placed to the extents shown in OPSD 3101.150, and rock backfill must be placed to the extents shown in OPSD 3101.200. All granular material should meet the requirements of SP 110F13 Amendment to OPSS 1010, March 1993.

Compaction equipment to be used adjacent to retaining structures must be restricted in accordance with SSP 105S10.

The design of the abutment must incorporate a subdrain as shown in OPSD 3101.150 or OPSD 3101.200, as applicable.

13 EARTH PRESSURE

Earth pressures acting on the structure may be assumed to be triangular and to be governed by the characteristics of the abutment backfill. 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)

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.

Earth pressure coefficients for backfill to the abutment wall are dependent on the material used as backfill. Typical values are shown in Table 13.1.

In conventional design, the use of a material with a high friction angle and low active pressure coefficient (e.g. Granular A, Granular B Type II) might be preferred as it results in lower earth pressures acting on the wall. In the case of integral abutments, material with a lower passive pressure coefficient (e.g. Granular B Type I) might be preferred as it results in lower forces acting on the ballast wall as the wall moves toward the soil mass.

The factors in Table 13.1 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 C6.9.1 (a) in the Commentary to the Canadian Highway Bridge Design Code.

Table 13.1 – Earth Pressure Coefficient (K)

Condition	Earth Pressure Coefficient (K)					
	OPSS Granular A or OPSS Granular B Type II $\phi = 35^\circ, \gamma = 22.8 \text{ kN/m}^3$		OPSS Granular B Type I $\phi = 32^\circ, \gamma = 21.2 \text{ kN/m}^3$		Rock Fill (Limited to 300 mm size) $\phi = 42^\circ, \gamma = 19 \text{ kN/m}^3$	
	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)
Active (Unrestrained Wall)	0.27	0.40*	0.31	0.48*	0.2	0.28*
At rest (Restrained Wall)	0.43	-	0.47	-	0.33	-
Passive (Movement Towards Soil Mass)	3.7	-	3.3	-	5.0	-

* For wing walls.

14 SEISMIC CONSIDERATIONS

14.1 Seismic Design Parameters

The site is treated as lying in Seismic Zone 2. The following seismic parameters should be used for design:

- Velocity Related Seismic Zone 2
- Zonal Velocity Ratio 0.1
- Acceleration Related Seismic Zone 2
- Zonal Acceleration Ratio 0.1
- Peak Horizontal Acceleration 0.11

The soil profile type at this site has been classified as Type I. Therefore, according to Table 4.4.6.1 of the CHBDC, a Site Coefficient “S” (ground motion amplification factor) of 1.0 should be used in seismic design.

14.2 Liquefaction Potential

The potential for liquefaction of the foundations soils was assessed using the Seed and Idriss (1971) method¹.

Using this method, it is estimated that under the existing conditions the foundation soils at both abutments are not prone to liquefaction. At the abutments, the approach embankments will increase the effective stress on the soil under the embankment and around the piles and as a result, liquefaction at the foundation is not considered to be likely.

If the structure is supported on steel piles, the foundation loads will be transferred by the steel piles to very dense sand with cobbles and boulders, or possibly to bedrock. In either case, it is not considered likely that the vertical geotechnical resistance of the piles will be compromised.

The embankments themselves will be constructed above the groundwater level and are not considered to be in danger of undergoing liquefaction. Some toe failure may occur but it is expected to be of limited nature and readily repairable.

14.3 Retaining Wall Dynamic Earth Pressures

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.

In calculating the active, passive and at rest earth pressure coefficients the angle of friction between the wall and backfill material is assumed to be 0.5ϕ . For the design of retaining walls, the coefficients of horizontal earth pressure in Table 14.1 may be used:

¹ Seed, H.B. and Idriss, I.M. 1971, “Simplified Procedure for Evaluating Soil Liquefaction Potential” *Journal of Soil Mechanics and Foundations Division*, ASCE, Vol. 101, No. SM9, September, pp. 1249-1273.

Table 14.1 – Earth Pressure Coefficient for Earthquake Loading

Earth Pressure Coefficient (K) for Earthquake Loading						
Wall Condition	Granular A or Granular B Type II $\phi = 35^\circ$; $\delta = 17.5^\circ$ $\gamma = 22.8 \text{ kN/m}^3$		OPSS Granular B Type I $\phi = 32^\circ$; $\delta = 16^\circ$ $\gamma = 21.2 \text{ kN/m}^3$		Rock Fill $\phi = 42^\circ$; $\delta = 21^\circ$ $\gamma = 19.0 \text{ kN/m}^3$	
	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)
Active (K_{AE})*	0.3	0.45	0.33	0.54	0.23	0.31
Passive (K_{PE})	6.3	6.3	5.4	5.4	12.0	12.0
At Rest (K_{OE})**	0.59		0.63		0.33	

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

** After Woods

15 CONSTRUCTION CONCERNS

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

- The possibility of piles reaching refusal on large boulders. In this case, the Hiley formula is not appropriate and site staff must make a decision regarding pile resistance and the appropriateness of continued driving.
- The potential variability of pile lengths at refusal.
- Unwatering in the case of excavations that must penetrate below the groundwater level.

16 CLOSURE

Engineering analysis and preparation of the report were carried out by Mr. Alastair E. Gorman, P.Eng and Ms. R. Palomeque Reyna, P.Eng.

The report was reviewed by Dr. P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations Projects.

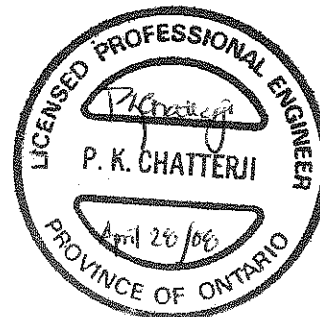
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Report reviewed by:
P.K. Chatterji, P.Eng., Ph.D.
Review Principal

STIRLING CREEK TRIBUTARY SBL
HIGHWAY 11, BURK'S FALLS TO SOUTH RIVER

[illegible]

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

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

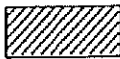

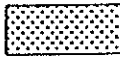


 Water Level
 Shear Strength Determination by Pocket Penetrometer

- (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

ROCK WEATHERING CLASSIFICATION		SYMBOLS	
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)

DISCONTINUITY SPACING		STRENGTH CLASSIFICATION			
Bedding	Bedding Plane Spacing	Rock Strength	Approximate Uniaxial Compressive Strength (MPa) (psi)	Field Estimation of Hardness*	
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.

TERMS		Weak	5.0 to 25.0	750 to 3,500	Can be peeled by a pocket knife with difficulty
Total Core Recovery: (TCR)	Core recovered as a percentage of total core run length.	Very Weak	1.0 to 5.0	150 to 750	Can be peeled by a pocket knife, crumbles under firm blows of geological pick.
Solid Core Recovery: (SCR)	Percent Ratio of solid core of full cylindrical shape recovered. Expressed with respect to the total length of core run.	Extremely Weak (Rock)	0.25 to 1.0	35 to 150	Indented by thumbnail
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 SCS-1

1 OF 1

METRIC

G.W.P. 742-93-00 LOCATION Stirling Creek Tributary SBL N 5 059 675.41 E 310 644.51 ORIGINATED BY SLL
 HWY 11 BOREHOLE TYPE Hollow Stem Augers COMPILED BY MFA
 DATUM Geodetic DATE 2007.07.31 - 2007.07.31 CHECKED BY AEG

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			20 40 60 80 100	20 40 60 80 100	PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	
319.0													
0.0	ASPHALT: (125mm)						319						
0.1	Gravelly SAND		1	AS									
318.4	Loose												
0.6	Brown												
	Moist (FILL)		1	SS	50/								
	SAND, trace silt, occasional cobbles				150		318						
	Very Dense												
	Brown												
	Moist (FILL)												
	becoming Compact		2	SS	20		317						
	Mottled Brown												
316.6													
2.4	SAND, trace to some gravel, trace silt, trace rootlets		3	SS	17								
	Compact												
	Dark Brown to Brown												
	Moist		4	SS	18		316						
							315						
	Dense		5	SS	31		314						
	occasional cobbles						313						
	Very Dense		6	SS	53								
							312						
	becoming gravelly		7	SS	73		311						
310.8													
8.2	END OF BOREHOLE AT 8.2m. BOREHOLE OPEN TO 8.1m, AND WATER LEVEL DRY. BOREHOLE GROUTED WITH AQUAGROUT BENTONITE TO SURFACE.												

ONTM14S 2339.GPJ 5/6/08

RECORD OF BOREHOLE No SCS-2

1 OF 3

METRIC

G.W.P. 742-93-00 LOCATION Stirling Creek Tributary SBL N 5 059 690.88 E 310 636.85 ORIGINATED BY SLL
 HWY 11 BOREHOLE TYPE Hollow Stem Augers/NW Casing/NQ Casing COMPILED BY MFA
 DATUM Geodetic DATE 2007.07.30 - 2007.07.30 CHECKED BY AEG

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			20 40 60 80 100	PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	
318.8												
0.0	ASPHALT: (125mm)											
0.1	SAND, trace to some gravel Compact to Very Dense Brown Moist (FILL)		1	AS			318					
			1	SS	74/ 275							
			2	SS	29		317					
316.6												
2.2	SAND, trace silt and clay Compact to Very Dense Brown Moist		3	SS	15		316					
			4	SS	57							
							315					
			5	SS	13		314					
			6	SS	29		313					
							312					
	occasional cobbles		7	SS	66		311					
	becoming gravelly		8	SS	75		310					
							309					

Continued Next Page

+ 3, X 3: Numbers refer to
Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No SCS-2

2 OF 3

METRIC

G.W.P. 742-93-00 LOCATION Stirling Creek Tributary SBL N 5 059 690.88 E 310 636.85 ORIGINATED BY SLL
 HWY 11 BOREHOLE TYPE Hollow Stem Augers/NW Casing/NQ Casing COMPILED BY MFA
 DATUM Geodetic DATE 2007.07.30 - 2007.07.30 CHECKED BY AEG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				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 x LAB VANE										
Continued From Previous Page							20 40 60 80 100				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT W _P W W _L				WATER CONTENT (%) 20 40 60			
	SAND, some gravel to gravelly Dense to Very Dense Brown Wet		9	SS	46		308											
								307										
								306										
					10	SS	46											
	cobbles and boulders		11	SS	100 .025		305											
								304										
304.3																		
14.5	SAND and GRAVEL, occasional cobbles and boulders Very Dense Grey Wet		12	SS	100 .075		304											
								303										
					13	SS	74		302									
301.2																		
17.7	MIGMATITIC GNEISS BEDROCK, slightly weathered, dark grey Coring started at 17.7m Horizontal joints at 17.8 and 18.0m		1	RUN			301											
					2	RUN			300									
							299											

Continued Next Page

+³, x³: Numbers refer to
Sensitivity

20
15
10


(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No SCS-2

3 OF 3

METRIC

G.W.P. 742-93-00 LOCATION Stirling Creek Tributary SBL N 5 059 690.88 E 310 636.85 ORIGINATED BY SLL
 HWY 11 BOREHOLE TYPE Hollow Stem Augers/NQ Casing/NQ Casing COMPILED BY MFA
 DATUM Geodetic DATE 2007.07.30 - 2007.07.30 CHECKED BY AEG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)		
								<div><div>20406080100</div><div>○ UNCONFINED + FIELD VANE</div><div>● QUICK TRIAXIAL × LAB VANE</div></div>										<div><div>204060</div><div>○</div></div>		
	Continued From Previous Page														kn/m ³	GR SA SI CL				
297.9	MIGMATITIC GNEISS BEDROCK, slightly weathered, dark grey Horizontal joint at 18.3m Sub-vertical joint at 18.5m		3	RUN											1 0 3					
20.9	END OF BOREHOLE AT 20.9m. BOREHOLE BACKFILLED WITH BENTONITE GROUT TO 2.4m, THEN AUGER CUTTINGS TO 1.5m, THEN HOLEPLUG TO 0.6m, THEN CUTTINGS TO 0.15m, AND ASPHALT TO SURFACE.						296													

+ 3 × 3 : Numbers refer to
Sensitivity

20
15
10
5
0
(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No SCS-3

1 OF 3

METRIC

G.W.P. 742-93-00 LOCATION Stirling Creek Tributary SBL N 5 059 694.86 E 310 651.95 ORIGINATED BY JHL
 HWY 11 BOREHOLE TYPE Hollow Stem Augers COMPILED BY MFA
 DATUM Geodetic DATE 2007.06.24 - 2007.06.26 CHECKED BY AEG

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				
								<div><div>○ UNCONFINED</div><div>● QUICK TRIAXIAL</div></div> <div><div>+ FIELD VANE</div><div>× LAB VANE</div></div>	<div><div>20 40 60 80 100</div><div>WATER CONTENT (%)</div><div><div>PLASTIC LIMIT</div><div>NATURAL MOISTURE CONTENT</div><div>LIQUID LIMIT</div></div></div>			
315.0							315					
0.0	TOPSOIL: (300mm)											
314.7												
0.3	Gravelly SAND, trace silt Compact Brown Moist		1	SS	18		314					
			2	SS	18		313					32 66 2 (SI+CL)
312.5			3	SS	100/ 300							
2.4	BOULDERS and COBBLES											
311.9			4	SS	22		312					
3.0							311					
	occasional cobbles Wet		5	SS	24		310					
309.5												
5.5	SAND, some gravel, trace silt and clay Very Dense Wet		6	SS	70		309					15 82 3 (SI+CL)
							308					
			7	SS	52		307					
							306					
			8	SS	78							
305.0												

Continued Next Page

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

RECORD OF BOREHOLE No SCS-3

2 OF 3

METRIC

G.W.P. 742-93-00 LOCATION Stirling Creek Tributary SBL N 5 059 694.86 E 310 851.95 ORIGINATED BY JHL
 HWY 11 BOREHOLE TYPE Hollow Stem Augers COMPILED BY MFA
 DATUM Geodetic DATE 2007.06.24 - 2007.06.26 CHECKED BY AEG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								WATER CONTENT (%)	
								○ UNCONFINED	+ FIELD VANE	● QUICK TRIAXIAL							× LAB VANE
						20	40	60	80	100	20	40	60	kn/m ³	GR SA SI CL		
10.0	Continued From Previous Page SAND and GRAVEL, trace silt and clay Compact to Very Dense Grey Wet																
			9	SS	79												
			10	SS	32												
	occasional cobbles		11	SS	75										37 52 11 (SI+CL)		
	occasional cobbles and boulders		12	SS	100/ .150												
	occasional cobbles and boulders		13	SS	100/ .100												
295.6																	
19.4																	
295.4	SILT, some clay, trace sand Very Dense Grey Moist		14	SS	100/												
19.6																	

Continued Next Page

+ 3 × 3 Numbers refer to
Sensitivity

20
15-5
10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No SCS-3

3 OF 3

METRIC

G.W.P. 742-93-00 LOCATION Stirling Creek Tributary SBL N 5 059 694.86 E 310 651.95 ORIGINATED BY JHL
 HWY 11 BOREHOLE TYPE Hollow Stem Augers COMPILED BY MFA
 DATUM Geodetic DATE 2007.06.24 - 2007.06.26 CHECKED BY AEG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa	WATER CONTENT (%)					
	Continued From Previous Page													
	END OF BOREHOLE AT 19.61m ON POSSIBLE BEDROCK. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen. WATER LEVEL READINGS: DATE DEPTH(m) ELEV.(m) 06/27/07 4.1 310.9 06/28/07 4.1 310.9 07/06/07 4.3 310.7 07/23/07 5.0 310.0				100									

ONTMT4S 2339.GPJ 3/27/08

RECORD OF BOREHOLE No SCS-4

1 OF 2

METRIC

G.W.P. 742-93-00 LOCATION Stirling Creek Tributary SBL N 5 059 745.00 E 310 618.54 ORIGINATED BY SLL
 HWY 11 BOREHOLE TYPE Hollow Stem Augers/NW Casing COMPILED BY MFA
 DATUM Geodetic DATE 2007.07.31 - 2007.07.31 CHECKED BY AEG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT Y 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
								○ UNCONFINED + FIELD VANE	● QUICK TRIAXIAL × LAB VANE										
317.0								20	40	60	80	100							
0.0	ASPHALT: (125mm)																		
0.1	SAND, trace gravel, trace silt Compact to Very Dense Brown Moist (FILL)		1	AS															
			1	SS	28														
			2	SS	40														
			3	SS	21														
			4	SS	87/ 275														
			5	SS	24														
			6	SS	2														
	Very Loose Grey Wet																		
309.3			7	SS	8														
7.7	SILT, some sand to sandy, trace rootlets Loose Dark Brown Moist to Wet																		
308.3			8	SS	52														
8.7	SAND and GRAVEL, trace silt Very Dense Dark Brown Wet																		

Continued Next Page

+ 3, x 3: Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE


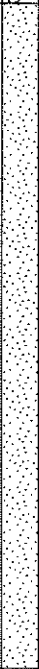

ONTMT4S 2339.GPJ 3/27/08

RECORD OF BOREHOLE No SCS-4

2 OF 2

METRIC

G.W.P. 742-93-00 LOCATION Stirling Creek Tributary SBL N 5 059 745.00 E 310 618.54 ORIGINATED BY SLL
 HWY 11 BOREHOLE TYPE Hollow Stem Augers/NW Casing COMPILED BY MFA
 DATUM Geodetic DATE 2007.07.31 - 2007.07.31 CHECKED BY AEG

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				
	Continued From Previous Page							20 40 60 80 100				
	SAND and GRAVEL , trace silt Very Dense Dark Brown Wet		9	SS	65		307					
	occasional cobbles Grey		10	SS	100/ .250		306					
304.0							305					
13.0	SAND , trace gravel, trace silt Dense to Very Dense Grey Wet		11	SS	42		304					
							303					
			12	SS	72		302					
							301					
299.6			13	SS	100/ .225		300					
17.4	Coring started at 17.4m MIGMATITIC GNEISS BEDROCK , grey, weathered		1	RUN			299					
298.6												
18.4	END OF BOREHOLE AT 18.39m. BOREHOLE BACKFILLED WITH AQUAGROUT TO 1.2m, THEN HOLEPLUG TO 0.6m, THEN CUTTINGS TO 0.15m, AND ASPHALT TO SURFACE.											

ONTMT4S 2339.GPJ 3/27/08

+³ ×³: Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

METRIC

+³, ×³: Numbers refer to Sensitivity

METRIC

+³, ×³: Numbers refer to Sensitivity

RECORD OF BOREHOLE No SCS-5

3 OF 3

METRIC

G.W.P. 742-93-00 LOCATION Stirling Creek Tributary SBL N 5 059 749.98 E 310 637.31 ORIGINATED BY GA
HWY 11 BOREHOLE TYPE Hollow Stem Augers/NW Casing/NQ Casing COMPILED BY MFA
DATUM Geodetic DATE 2007.07.05 - 2007.07.06 CHECKED BY AEG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	WATER CONTENT (%)		
	Continued From Previous Page													
292.3	MIGMATITIC GNEISS BEDROCK , fresh to slightly weathered, massive, grey, pink bands Horizontal joint at 19.89m Sub-horizontal joint at 19.93m Rubble zone from 20.54 to 20.73m		2	RUN			293						2	RUN 2#
20.7	END OF BOREHOLE AT 20.73m. BOREHOLE OPEN TO 19.81m AND WATER LEVEL AT 3.05m 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) 07/23/07 3.4 309.6												0	TCR=100%, SCR=100%, RQD=86%, UCS=122MPa

+ 3, × 3: Numbers refer to
Sensitivity

20
15
10





(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No SCS-6

1 OF 2

METRIC

G.W.P. 742-93-00 LOCATION Stirling Creek Tributary SBL N 5 059 765.31 E 310 622.93 ORIGINATED BY GA
 HWY 11 BOREHOLE TYPE Hollow Stem Augers COMPILED BY MFA
 DATUM Geodetic DATE 2007.07.05 - 2007.07.05 CHECKED BY AEG

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								
312.6							20 40 60 80 100	PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT								
0.0	SAND, trace silt, occasional cobbles Loose to Compact Brown Moist (FILL)		1	SS	9											
			2	SS	10											
			3	SS	5											
			4	SS	15											
309.5																
3.0	SAND, fine to medium grained, trace silt Compact Brown Wet		5	SS	17											
			6	SS	27											
306.5																
6.1	SAND and GRAVEL, trace silt Very Dense Brown Wet		7	SS	55											
			8	SS	85											
303.4																
9.1	SAND, trace gravel Very Dense Brown Wet		9	SS	120											

Continued Next Page

+³, ×³: Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No SCS-6

2 OF 2

METRIC

G.W.P. 742-93-00 LOCATION Stirling Creek Tributary SBL N 5 059 765.31 E 310 622.93 ORIGINATED BY GA
 HWY 11 BOREHOLE TYPE Hollow Stem Augers COMPILED BY MFA
 DATUM Geodetic DATE 2007.07.05 - 2007.07.05 CHECKED BY AEG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100					
302.5 10.1	Continued From Previous Page END OF BOREHOLE AT 10.06m. BOREHOLE OPEN AND WATER LEVEL AT 3.05m UPON COMPLETION. BOREHOLE GROUTED WITH BENTONITE TO SURFACE.						302							

+ 3 . × 3 : Numbers refer to
Sensitivity

20
15 10 5
(%) STRAIN AT FAILURE

Appendix B

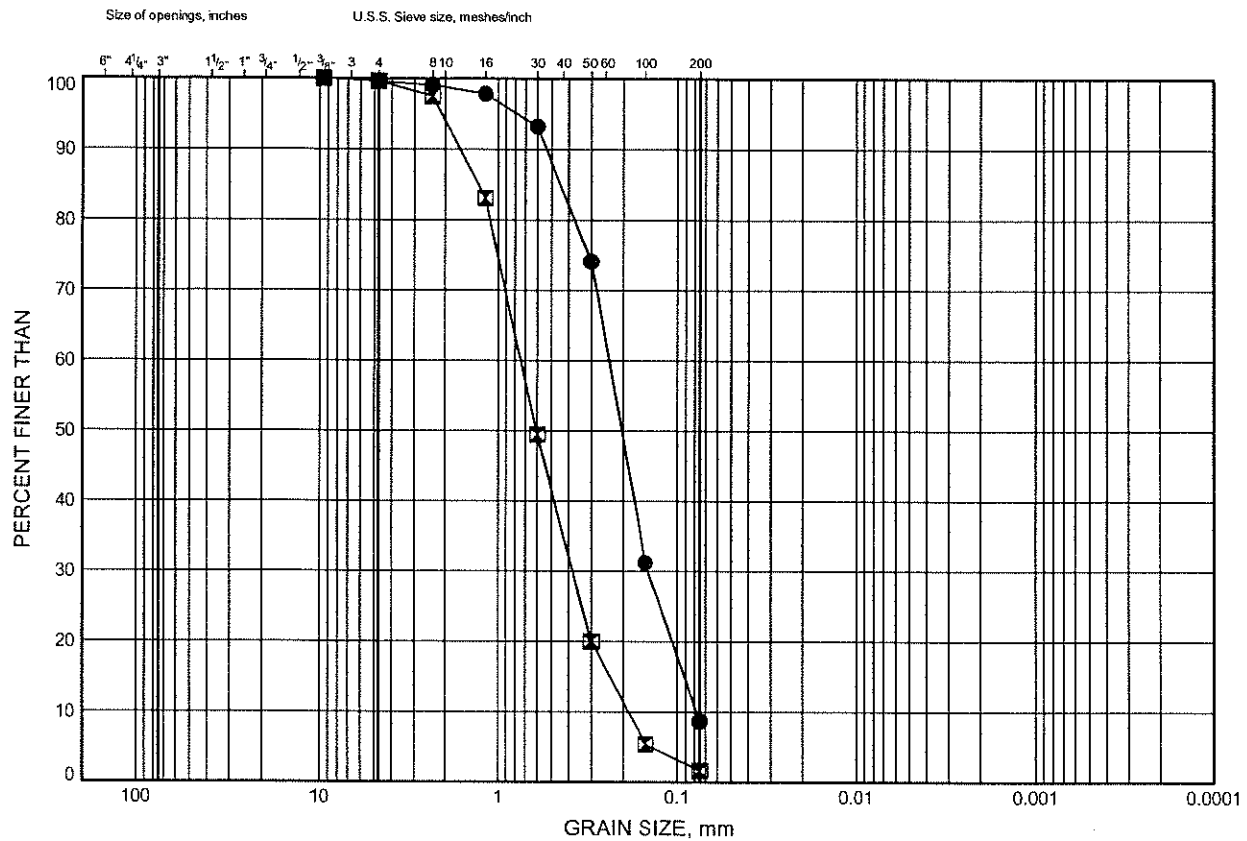
Laboratory Test Results

Stirling Creek Tributary Bridges

GRAIN SIZE DISTRIBUTION

FIGURE B1

SAND FILL



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

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	SCS-4	2.59	314.37
⊠	SCS-6	2.59	310.00

Date March 2008
Project 742-93-00



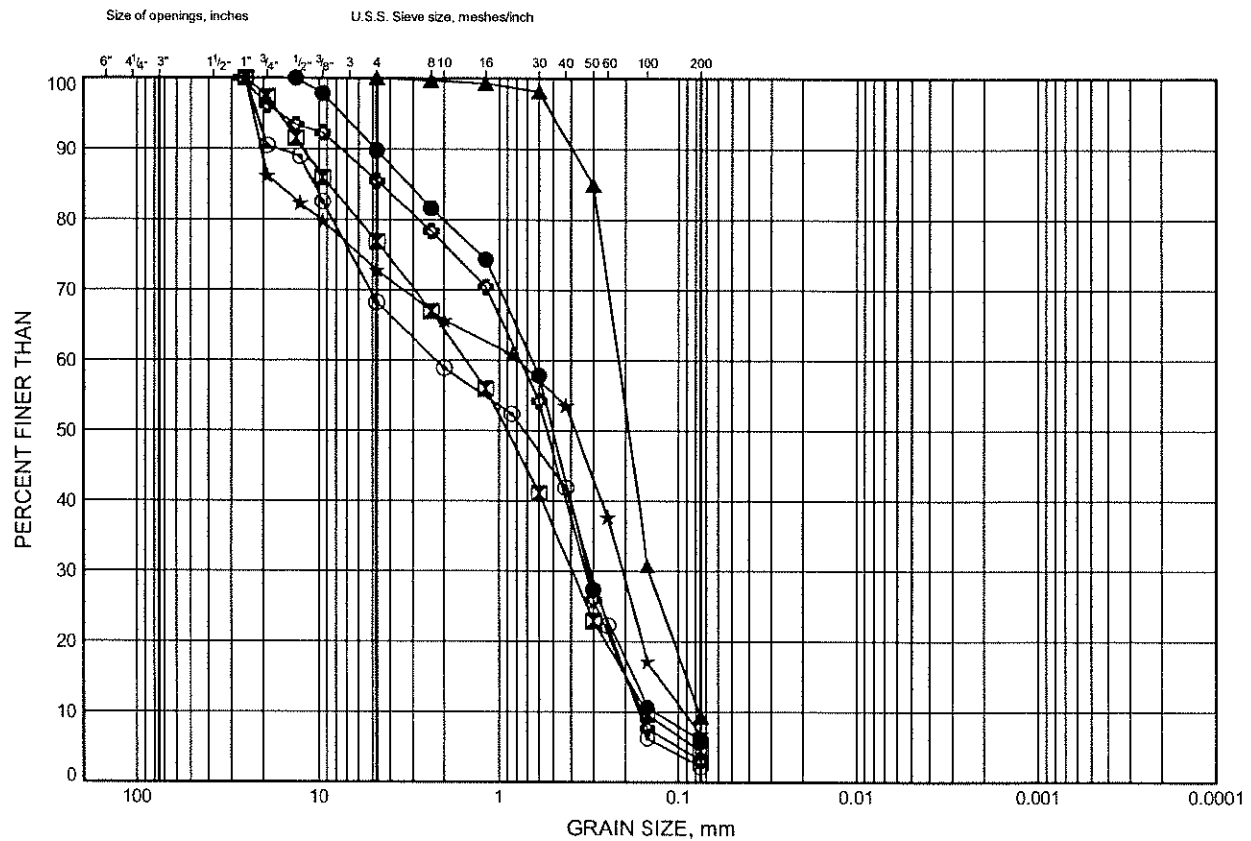
Prep'd MFA
Chkd. RPR

Stirling Creek Tributary Bridges

GRAIN SIZE DISTRIBUTION

FIGURE B2

SAND (SOME GRAVEL TO GRAVELLY)



SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	SCS-1	3.35	315.67
⊠	SCS-1	7.92	311.10
▲	SCS-2	3.35	315.49
★	SCS-2	9.45	309.40
⊙	SCS-3	1.83	313.15
⊗	SCS-3	6.40	308.58

Date March 2008
Project 742-93-00



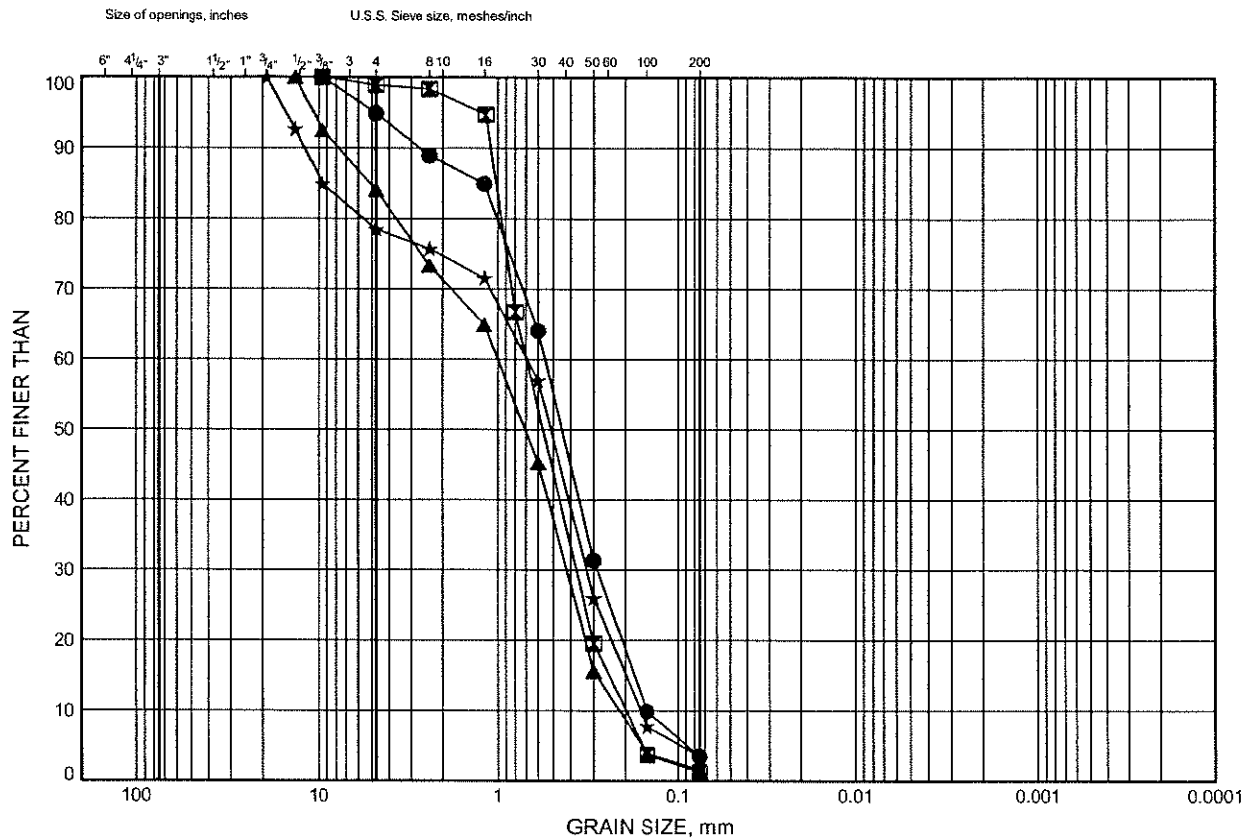
Prep'd MFA
Chkd. RPR

Stirling Creek Tributary Bridges

GRAIN SIZE DISTRIBUTION

FIGURE B3

SAND (SOME GRAVEL TO GRAVELLY)



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

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	SCS-4	14.02	302.94
■	SCS-5	2.59	310.39
▲	SCS-5	10.36	302.62
★	SCS-5	16.46	296.53

Date March 2008
Project 742-93-00



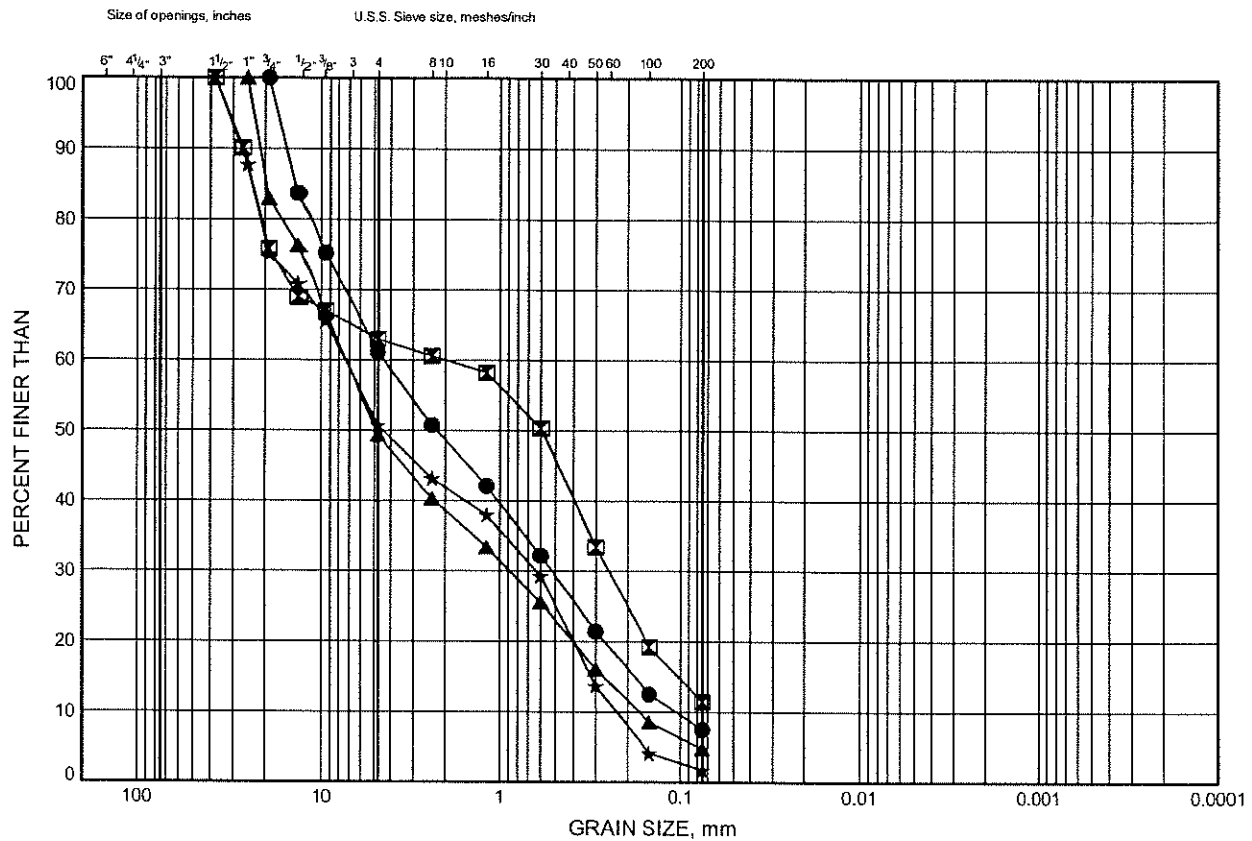
Prep'd MFA
Chkd. RPR

Stirling Creek Tributary Bridges

GRAIN SIZE DISTRIBUTION

FIGURE B4

SAND AND GRAVEL



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

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	SCS-2	17.07	301.78
⊠	SCS-3	14.02	300.96
▲	SCS-4	9.45	307.51
★	SCS-6	7.92	304.67

Appendix C

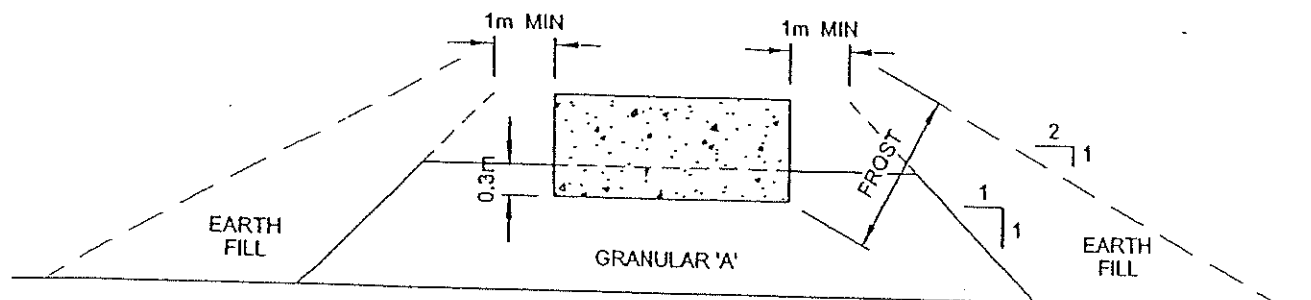
Foundation Comparison

COMPARISON OF FOUNDATION ALTERNATIVES FOR EACH FOUNDATION ELEMENT

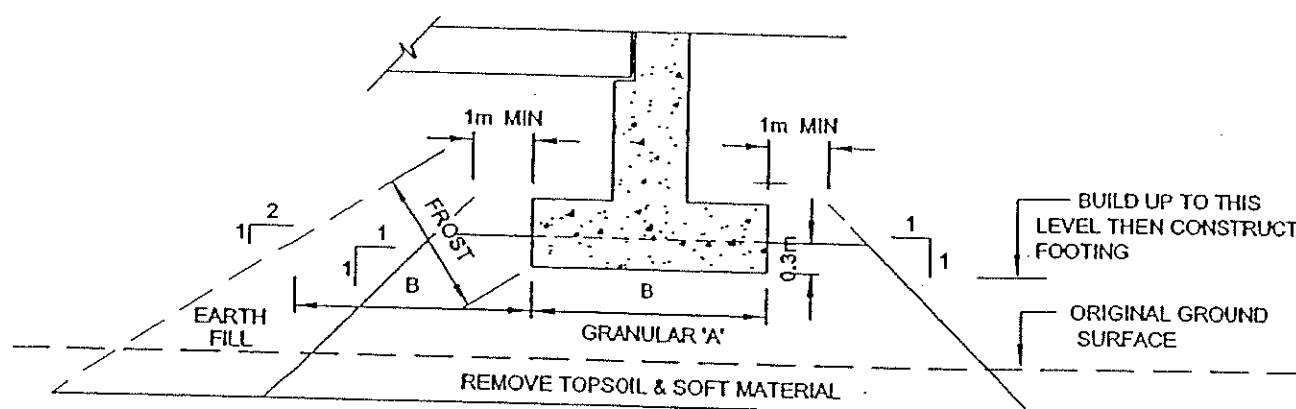
Driven Piles	Augered Caissons	Footing on Native Soil	Footing on Engineered Fill
<p>Advantages:</p> <ul style="list-style-type: none"> i. High geotechnical resistance available by driving piles to achieve resistance in the very dense soil overlying bedrock. ii. Allows choice of conventional, integral or semi-integral abutment design. iii. Readily installed. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Higher unit cost compared to footings. ii. Construction concerns related to the possibility of piles being obstructed by a boulder during driving. <p>RECOMMENDED</p>	<p>Advantages:</p> <ul style="list-style-type: none"> i. High geotechnical resistance available for units founded in very dense soil or on bedrock. ii. Construction of caissons could continue in freezing weather. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Higher unit cost compared to other foundation options such as footings or driven piles. ii. High risks associated with inflow of groundwater and soil fines. iii. Soil conditions encountered at this site are considered to be unsuitable. <p>NOT RECOMMENDED</p>	<p>Advantages:</p> <ul style="list-style-type: none"> i. Economical to install. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Comparatively longer abutment stem. ii. Deep excavation and dewatering required. iii. Potential for settlements. <p>FEASIBLE BUT NOT RECOMMENDED</p>	<p>Advantages:</p> <ul style="list-style-type: none"> i. Possibility of shortening the abutment height. ii. Higher geotechnical resistance than is available on native soil. iii. Lower cost compared to deep foundations. iv. Allows use of perched abutments. v. Allows choice of semi-integral abutment. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Lower geotechnical resistance than piles. ii. High cost of constructing engineered fill. iii. Potential settlements. <p>NOT RECOMMENDED</p>

Appendix D

Figure



CROSS-SECTION



LONGITUDINAL SECTION

NOT TO SCALE

NOTES:

1. REMOVE TOPSOIL AND OR SOFT SUBSOIL UNDER AREA OF COMPACTED GRANULAR 'A' AND EARTH FILL.
2. PLACE GRANULAR 'A' AND EARTH FILL TO BOTTOM OF FOOTING LEVEL, COMPACTED ACCORDING TO O.P.S.S. 501.
3. CONSTRUCT CONCRETE FOOTING.
4. PLACE REMAINDER OF GRANULAR 'A' AND EARTH FILL AS REQUIRED.
5. SOURCE M.T.C..1982.

ENGINEER	AEG
DRAWN	SS
DATE	
APPROVED	PKC
SCALE	NTS

ABUTMENT ON COMPACTED FILL SHOWING
GRANULAR A CORE



THURBER

DWG. NO.

FIGURE 1

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 572
- OPSS 902 as amended by Special Provision 902S01.
- SP 110F13 Amendment to OPSS 1010, March 1993
- SSP 105S10
- OPSD 3501.000
- OPSD 3505.000
- OPSD 3501.000
- OPSD 3505.000

OPSS 206, as amended by Special Provision "Amendment to OPSS 206, December 1993", dated November 2002.

2. Suggested text for a NSSP on Pile Installation

The soil overlying the bedrock contains cobbles and boulders, particularly below Elevation 300.0 m. The presence of cobbles and boulders will potentially have an impact on the installation of piles at the site. Some possible impacts that must be taken into consideration include, but are not necessarily limited to:

- The need to provide protection to the pile tips in the form of rock points.
- The cobbles and boulders may impede the driving of the piles resulting in more arduous driving to reach bedrock.
- Some piles may meet refusal on boulders that are large enough not to be dislodged or broken by the pile driving.
- As a result of the presence of boulders, piles may meet refusal at varying depths.
- Pile driving must be controlled according to the criteria specified for the site.
- If a pile meets refusal at a depth less than the anticipated depth, the QVE must terminate driving before the pile is damaged due to over-driving.

3. Suggested text for a NSSP on Dewatering

The soils underlying this site are cohesionless in nature and the observed groundwater table lies close to the surface. Excavation below the groundwater level is expected to lead to instability and slough of the sides of the excavation and boiling of the base, accompanied by loss in geotechnical resistance of the soils. If excavation is required to be carried out below the groundwater level prevailing at the time of construction, appropriate means of dewatering must be implemented to depress the groundwater level sufficiently far below the base of the excavation to prevent any instability, sloughing, or boiling and so as to preserve the stability of the excavation and to allow the work to proceed in the dry.

CSP FOR INTEGRAL ABUTMENT - Item No.

Special Provision

Scope

This specification covers the requirements for the installation of the CSP's, including sand fill and polystyrene sheets, at the integral abutments.

References

This specification refers to the following standards, specifications or publications:

Ontario Provincial Standard Specifications, Construction:

OPSS 906	Structural Steel
OPSS 909	Prestressed Concrete - Precast Members

Ontario Provincial Standard Specifications, General:

OPSS 180	Management and Disposal of Excess Materials
----------	---

Ontario Provincial Standard Specifications, Material:

OPSS 1605	Expanded Extruded Polystyrene
OPSS 1801	Corrugated Steel Pipe Products

Canadian Standards Association Standards:

CSA G164-M	Galvanizing of Irregularly-Shaped Articles
------------	--

Ministry of Transportation Publications

MTO Manual of Designated Sources of Materials

Definitions

For the purposes of this specification, the following definitions apply:

Abutment Stem: means the cast-in-place concrete component of the abutment placed over the top of the piles and forming the bearing seat for the girders.

CSP: means helical corrugated steel pipe.

Design Engineer: means the Engineer who produces the design and/or working drawings, and who has a minimum of five (5) years in the design and/or construction of bridges.

Submission and Design Requirements

Submissions

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

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

The Contractor shall have a copy of the submitted working drawings on site at all times.

Working Drawing Requirements

Working drawings shall include at least the following:

1. Layout and Elevations of the CSP's;
2. Source of the sand fill, and description of placing method and equipment;
3. Location and details of all temporary bracing, including permanent and temporary spacers, for the piles, CSP's and abutment stems;
4. Detailed construction sequence for the work, including installation and removal of the temporary bracing.

Design Requirements

The Contractor shall be responsible for the complete detailed design of the construction sequence for the work, including the installation and removal of all temporary bracing. The general sequence of construction shall be as shown on the Contract drawings.

The Contractor shall be responsible for the complete detailed design of all temporary bracing, including temporary and permanent spacers, required to maintain the piles, CSP's, abutment stems and girders in their specified positions through all stages of construction until concrete in deck has reached a compressive strength of 25 MPa. All temporary bracing, except spacers identified as permanent on the Contract drawings, shall be removed.

Temporary bracing for prestressed, precast girders shall meet the requirements of OPSS 909. Temporary bracing for structural steel girders shall meet the requirements of OPSS 906.

Material

Corrugated Steel Pipe

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

Permanent Spacers and Associated Hardware

Permanent spacers and associated hardware left in place shall not consist of wood and corrodible material.

Sand Fill

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

Table 1 - Sand Fill Gradation Requirements

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

Expanded Extruded Polystyrene

Expanded extruded polystyrene shall be in accordance with OPSS 1605, and shall be from a supplier listed under DSM # 3.30.30.

Construction

General

The sequence of construction for installing the concrete pads, CSP's, sand fill and abutment stems, including the installation and removal of the temporary bracing, shall be in accordance with the working drawings.

The Contractor shall not proceed with the abutment backfill above the level of the bottom of the CSP's without written permission from the Contract Administrator.

Corrugated Steel Pipe

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

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

The Contractor shall set the inner and outer CSP over each pile in the abutment into the concrete pad, following the batter of the pile, while the concrete in the concrete pad is still plastic. The CSP's shall extend at least 150 mm into the concrete pad.

The Contractor shall ensure the full perimeter of the tops of all CSP's at each abutment are at the elevation shown on the working drawings.

After the CSP's have been set, the Contractor shall take all measures necessary to prevent the ingress of water, backfill and debris into the CSP's.

Sand Fill

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

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

After the sand fill has been placed to the top of each inner CSP, the Contractor shall take all measures necessary to prevent the ingress of water and other liquids into the sand fill until after the concrete in the abutment stem has been placed and cured.

Expanded Extruded Polystyrene

The expanded extruded polystyrene sheets shall completely cover the area under the abutment stem as shown on the Contract drawings. The sheets shall be placed in one piece for the width of the abutment stem, with butt joints perpendicular to the centre-line of abutment bearings. The minimum length of sheet shall be 500 mm.

Joints between sheets within 500 mm of a pile centre-line will not be permitted. At each pile location, a minimum 1000 mm long sheet shall be centred on the pile and a 500 mm diameter hole neatly cut in the sheet so as to fit over the pile in one piece, fully spanning the annular space between the double CSP's.

The Contractor shall adjust the backfill to ensure full and uniform contact of the sheets with the backfill and the full perimeter of the tops of the CSP's. The vertical step at joints between sheets shall not exceed 5 mm.

The Contractor shall protect the sheets from damage during installation of the reinforcing for the abutment stem, and shall secure the sheets from "floating" during placing of the concrete in the abutment stem. Only hardware approved by the Owner shall be used to secure the sheets. All hardware used to secure the sheets shall be installed so as not to project above the top surface of the sheets into the abutment stem.

Temporary Bracing

Temporary bracing shall be installed and removed in accordance with the working drawings.

The temporary bracing shall not distort, nor pierce the walls of, the CSP's. Welding to the CSP's will not be permitted.

Concrete anchors shall be removed and the holes filled with non-shrink grout.

Tolerances

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

<u>Criteria</u>	<u>Tolerance</u>
Maximum deviation of inner and outer CSP from pile centroid.	± 25 mm
Maximum deviation from specified spacing between inner and outer CSP's.	± 25 mm
Maximum deviation of any point on the top perimeter of the CSP's from the specified Elevation.	± 10 mm

Quality Assurance

Prior to placing the CSP's, the Contractor shall establish reference points at each abutment and determine the location of the centroid of each pile in the abutment with respect to these reference points. The Contractor shall maintain the reference points until written permission to proceed with the backfill above the level of the bottom of the CSP's has been given by the Contract Administrator.

Measurement for Payment

There will be no measurement for this item.

Basis of Payment

Payment at the contract price for the above items shall be full compensation for all labour, equipment and material required to do the work.

Appendix F

Slope Stability Output

	Gamma	C	Phi	Piezo
	kN/m ³	kPa	deg	Surf.
Rock Fill	20	0	45	1
Sand	21	0	30	1

Thurber Engineering Ltd. - Toronto
 19-1423-39 Hwy 11 Burk's Falls
 Stirling Creek, SBL
 April 3, 2008
 South Approach Rock fill

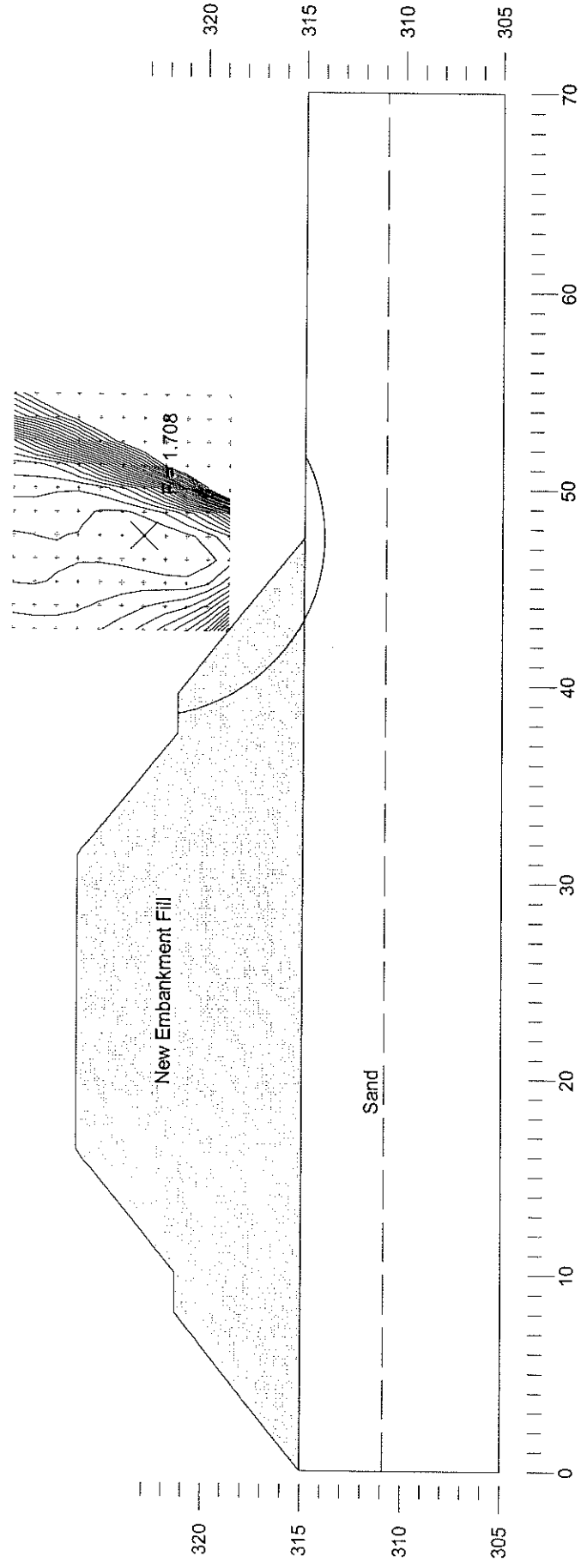


FIGURE 1

Thurber Engineering Ltd. - Toronto
 19-1423-39 Hwy 11 Burk's Falls
 Stirling Creek, SBL
 April 3, 2008
 South Approach Rock fill

	Gamma C	Phi	Piezo
	kN/m3	deg	Surf.
Rock Fill	20	45	1
Sand	21	30	1

Seismic coefficient = 0.08

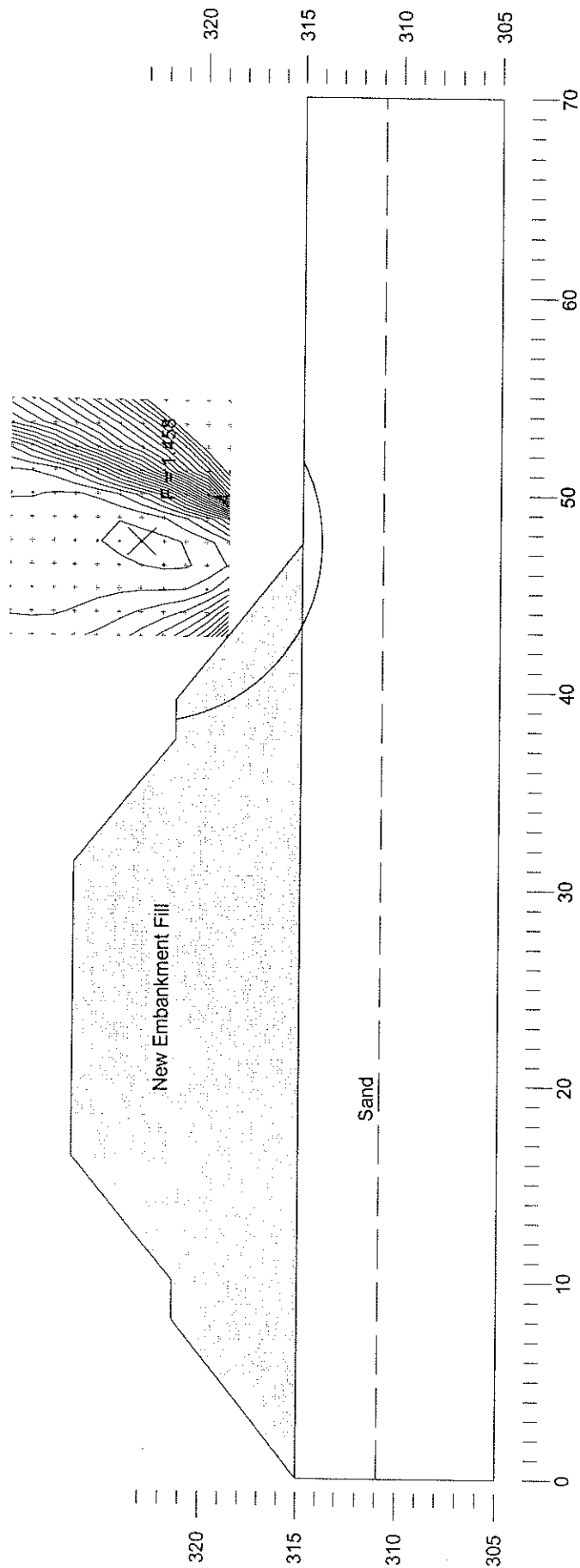


FIGURE 2

Thurber Engineering Ltd. - Toronto
 19-1423-39 Hwy 11 Burk's Falls
 Stirling Creek, SBL
 April 3, 2008
 South Approach Earth fill

Earth Fill	Gamma C	Phi	Piezo
Sand	kN/m3	deg	Surf.
	21	30	1
	21	30	1

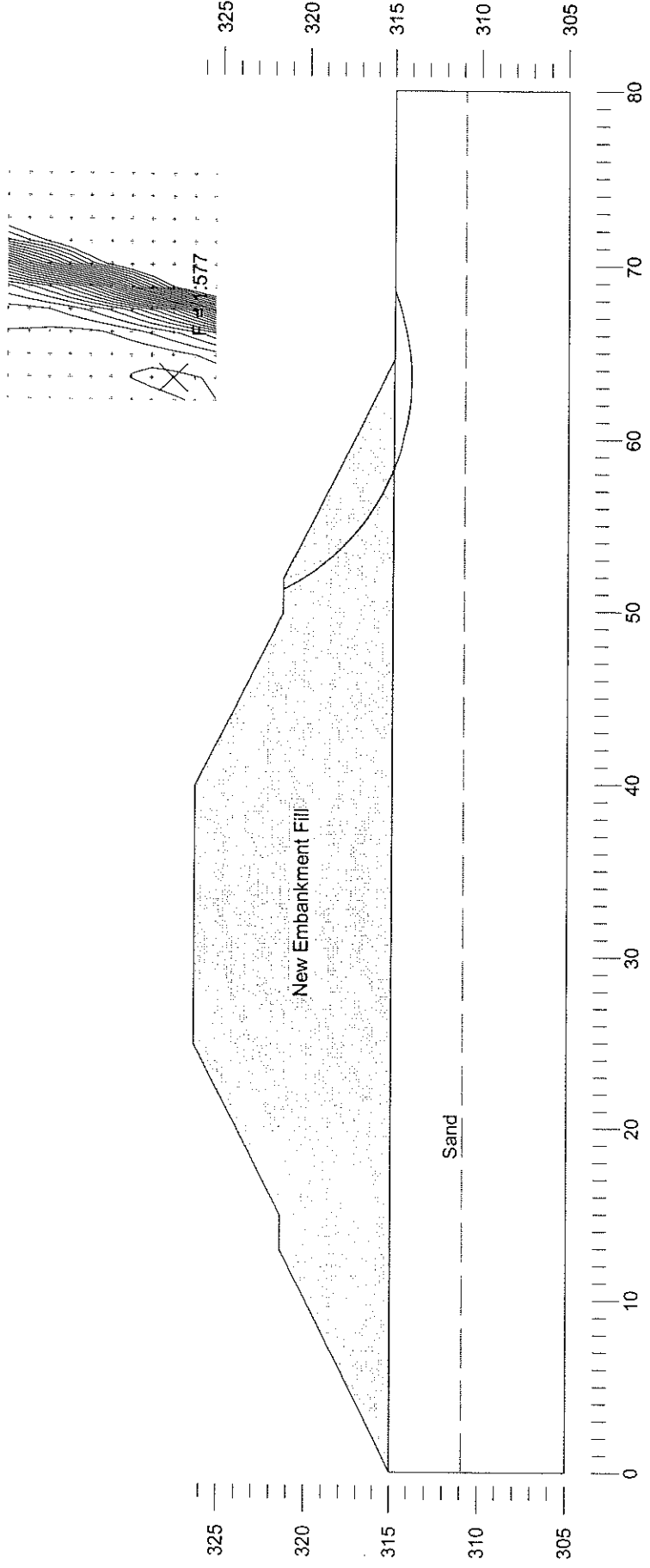


FIGURE 3

Thurber Engineering Ltd. - Toronto
 19-1423-39 Hwy 11 Burk's Falls
 Stirling Creek, SBL
 April 3, 2008
 South Approach Earth fill

	Gamma C	Phi	Piezo
	kN/m ³	deg	Surf.
Earth Fill	21	30	1
Sand	21	30	1

Seismic coefficient = 0.08

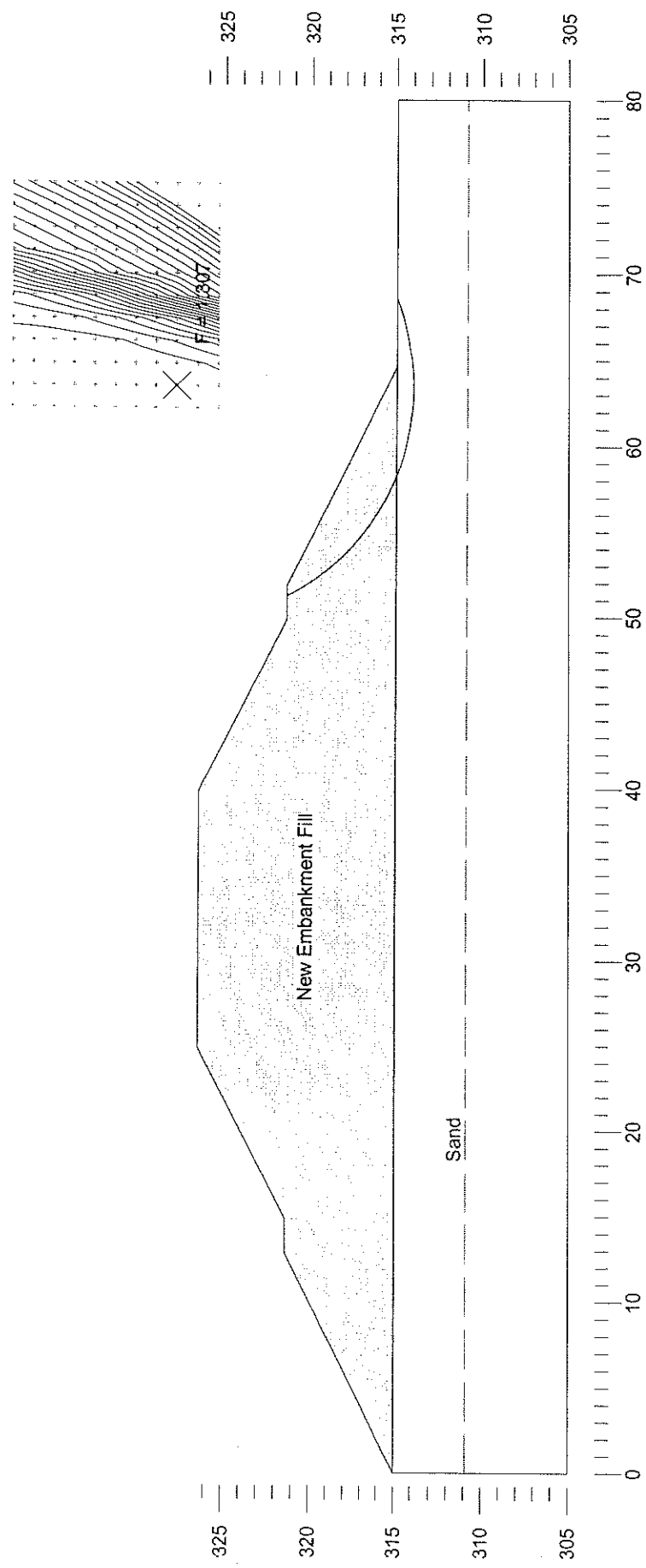


FIGURE 4

Thurber Engineering Ltd. - Toronto
 19-1423-39 Hwy 11 Burk's Falls
 Stirling Creek, SBL
 April 3, 2008
 North Approach Rock fill

	Gamma	C	Phi	Piezo
	kN/m3	kPa	deg	Surf.
Rock Fill	20	0	45	1
Sand	21	0	30	1

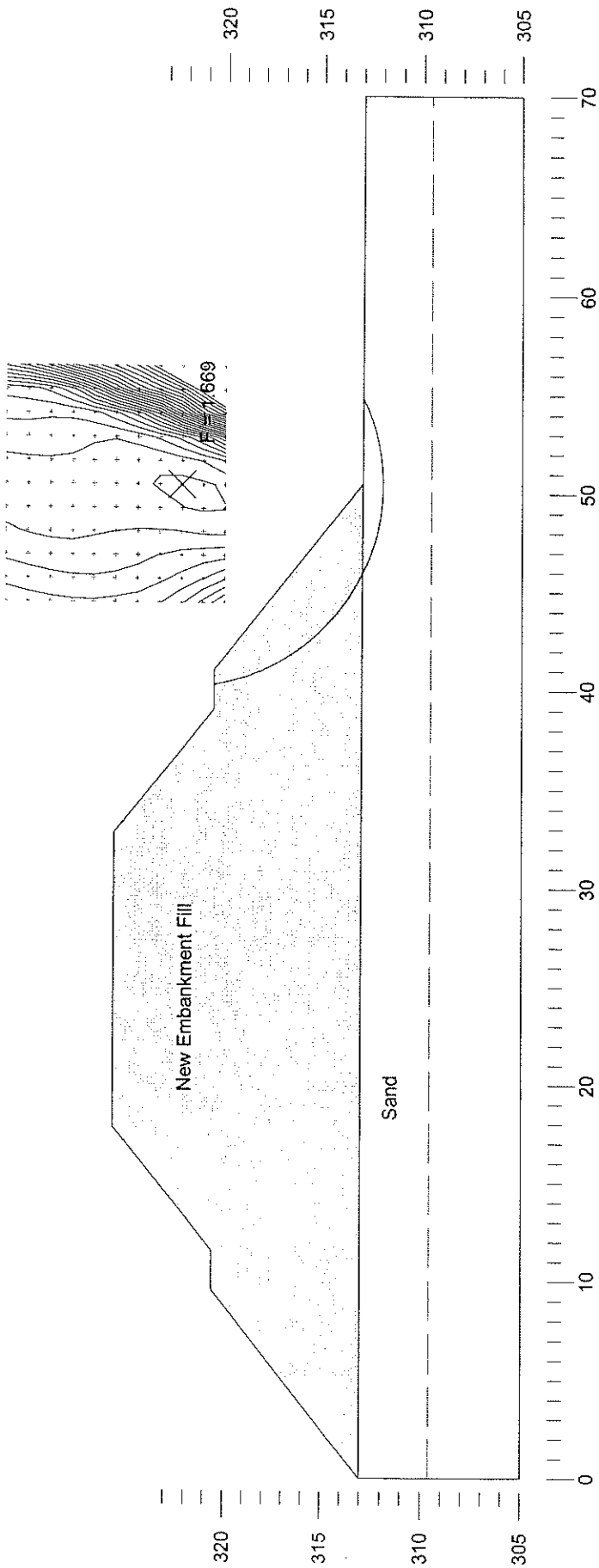


FIGURE 5

Thurber Engineering Ltd. - Toronto
 19-1423-39 Hwy 11 Burk's Falls
 Stirling Creek, SBL
 April 3, 2008
 North Approach Rock fill

	Gamma	C	Phi	Piezo
	kN/m ³	kPa	deg	Surf.
Rock Fill	20	0	45	1
Sand	21	0	30	1

Seismic coefficient = 0.08

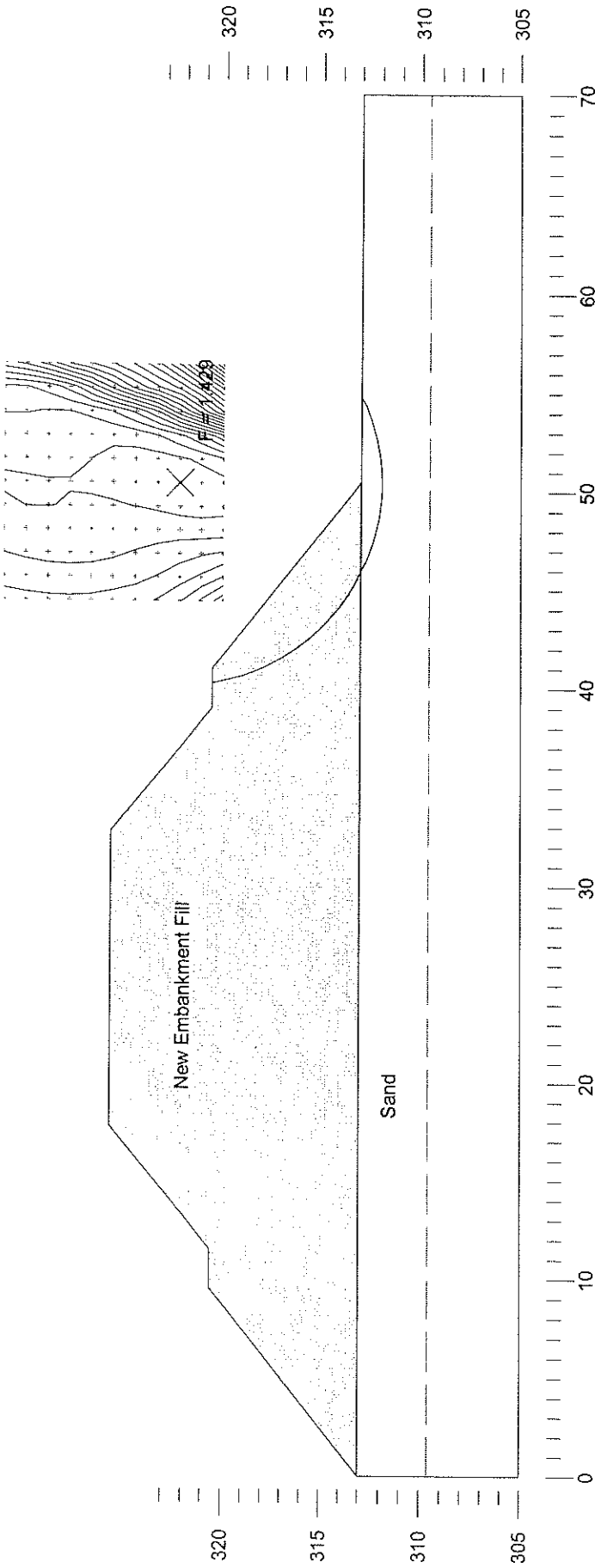


FIGURE 6

	Gamma C	Phi	Piezo
	kN/m3	deg	Surf.
Earth Fill	21	30	1
Sand	21	30	1

Thurber Engineering Ltd. - Toronto
 19-1423-39 Hwy 11 Burk's Falls
 Stirling Creek, SBL
 April 3, 2008
 North Approach Earth fill

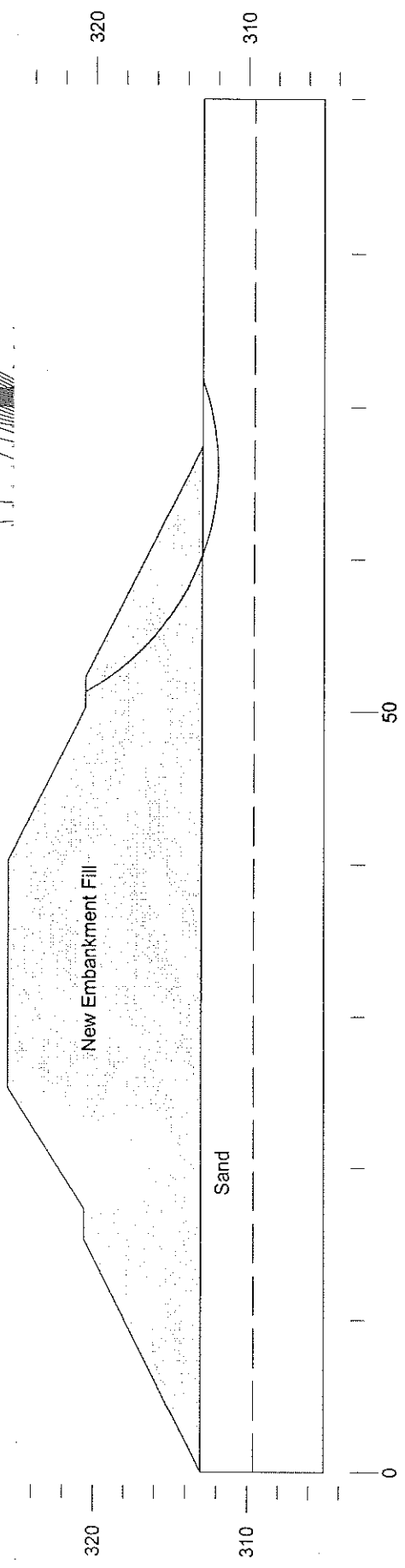
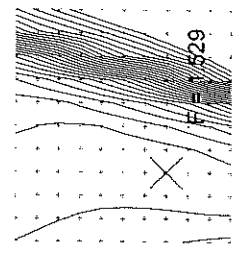


FIGURE 7

Thurber Engineering Ltd. - Toronto
 19-1423-39 Hwy 11 Burk's Falls
 Stirling Creek, SBL
 April 3, 2008
 North Approach Earth fill

	Gamma C	Phi	Piezo
	kN/m ³	deg	Surf.
Earth Fill	21	30	1
Sand	21	30	1

Seismic coefficient = 0.08

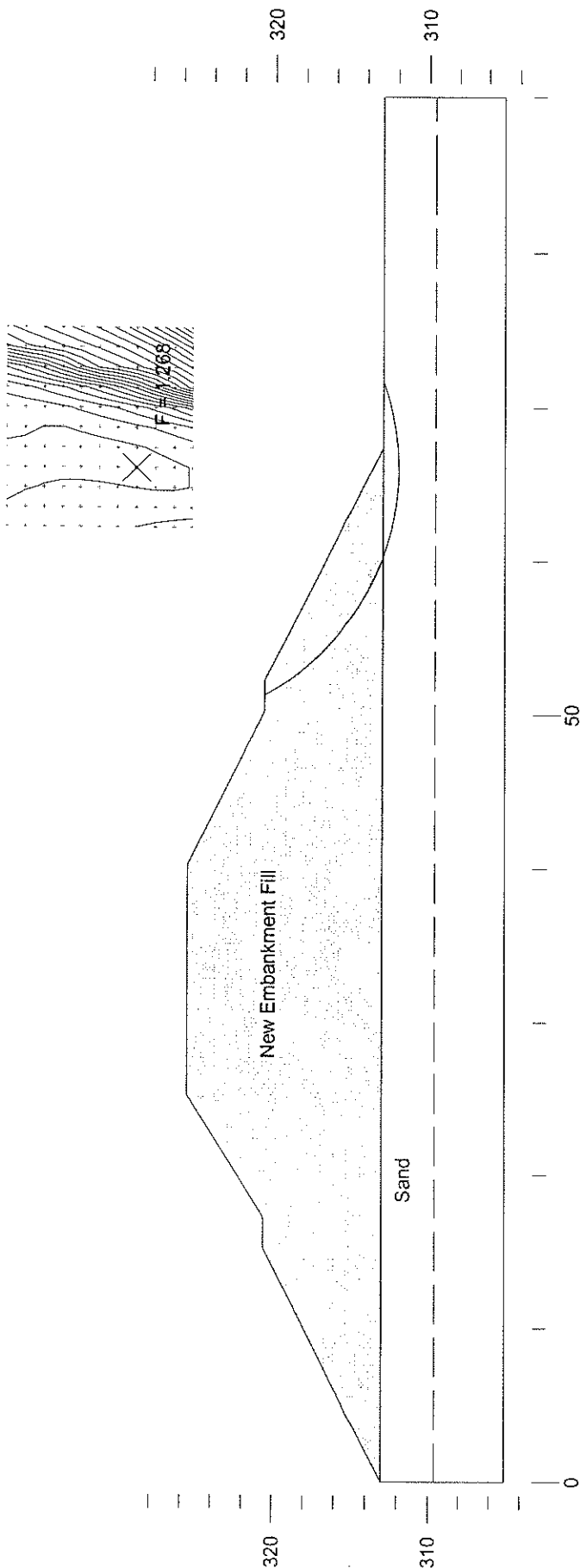


FIGURE 8

Highway 11 SBL over Stirling Creek Tributary
Highway 11 Burk's Falls to South River

Appendix G

Site Photographs

Highway 11 SBL over Stirling Creek Tributary
Highway 11 Burk's Falls to South River



Photograph 1 – General view of the site, looking northwards, pond in middle ground.



Photograph 2 – Confluence of the two streams at mouth of culvert.

Highway 11 SBL over Stirling Creek Tributary
Highway 11 Burk's Falls to South River



Photograph 3 – South approach looking south along Highway 11.



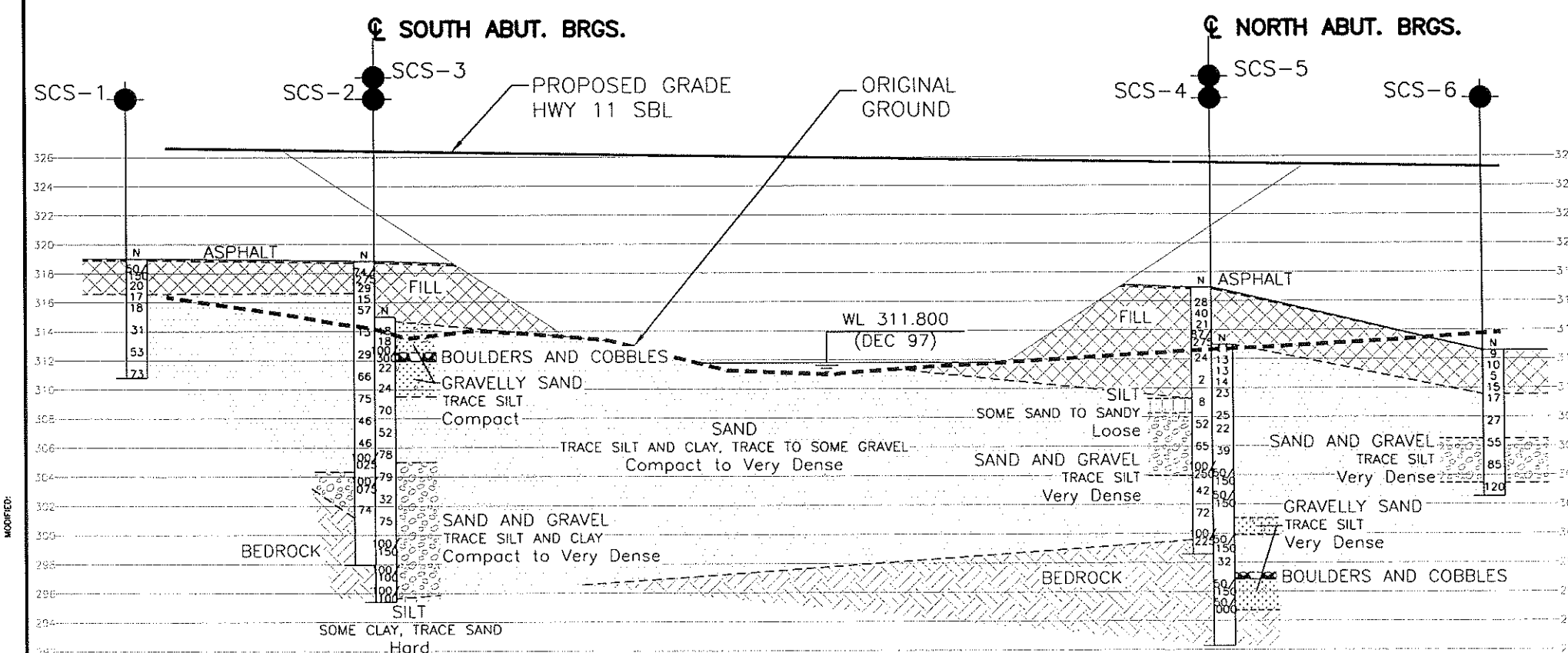
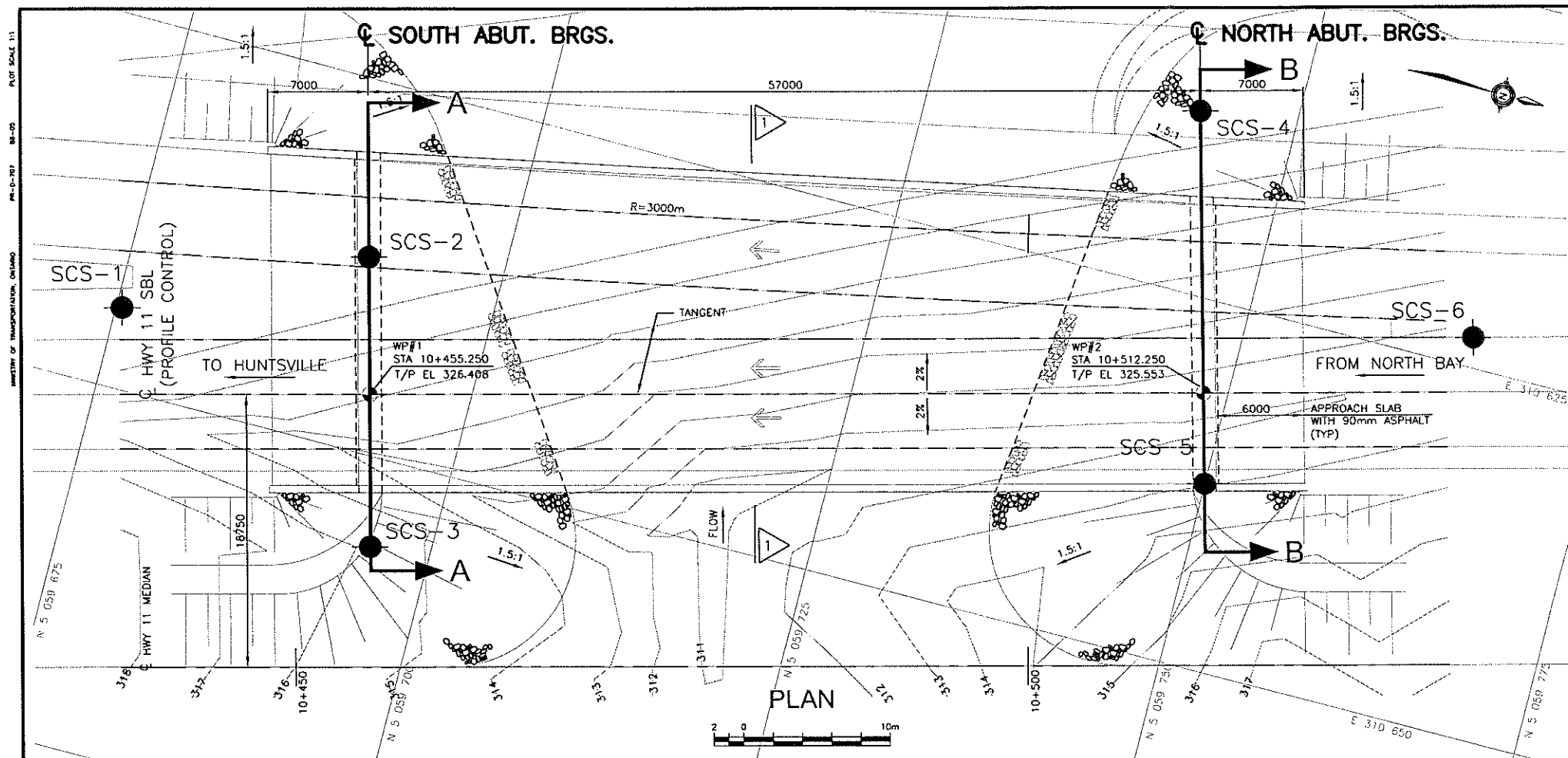
Photograph 4 – Looking north along Highway 11 over future SBL structure site.

Highway 11 SBL over Stirling Creek Tributary
Highway 11 Burk's Falls to South River

Appendix H

Drawing

Borehole Locations and Soil Strata

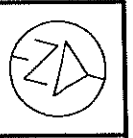


DHO BENCHMARK 343-67
ELEVATION 329.137
TABLET IN ROCK OUTCROP
49.337m RL 24+795.574
TWP OF ARMOUR

PROFILE HWY 11 SBL

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

CONT No 2008-5113
WP No 5100-06-01



STIRLING CK TRIBUTARY SBL
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET
293



KEYPLAN

LEGEND

- Borehole by THURBER
- N Blows /0.3m (Std Pen Test, 475J/blow)
- CONE Blows /0.3m (60' Cone, 475J/blow)
- PH Pressure, Hydraulic
- Water Level
- Head Artesian Water
- Piezometer
- 90% Rock Quality Designation (RQD)
- A/R Auger Refusal

NO	ELEVATION	NORTHING	EASTING
SCS-1	319.0	5 059 675.41	310 644.51
SCS-2	318.8	5 059 690.88	310 636.85
SCS-3	315.0	5 059 695.94	310 656.03
SCS-4	317.0	5 059 743.44	310 612.69
SCS-5	313.0	5 059 749.98	310 637.31
SCS-6	312.6	5 059 765.31	310 622.93

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. 31E-276



Refer to DWG 2 for Sections A-A and B-B.

DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

DATE	BY	DESCRIPTION
DESIGN	AEF	CHK AEG
DRAWN	MFA	CHK AEG
		SITE
		STRUCT
		IScheme
		DWG 2

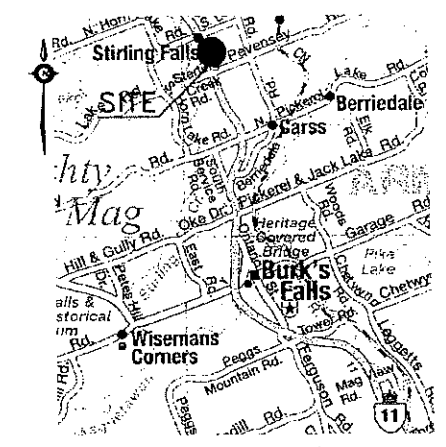
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PLOTDATE: May 06, 2008 - 9:08am

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

CONT No 2008-5113
WP No 5100-06-01

STIRLING CK TRIBUTARY SBL
STRATIGRAPHIC SECTIONS

SHEET
294



KEYPLAN

LEGEND

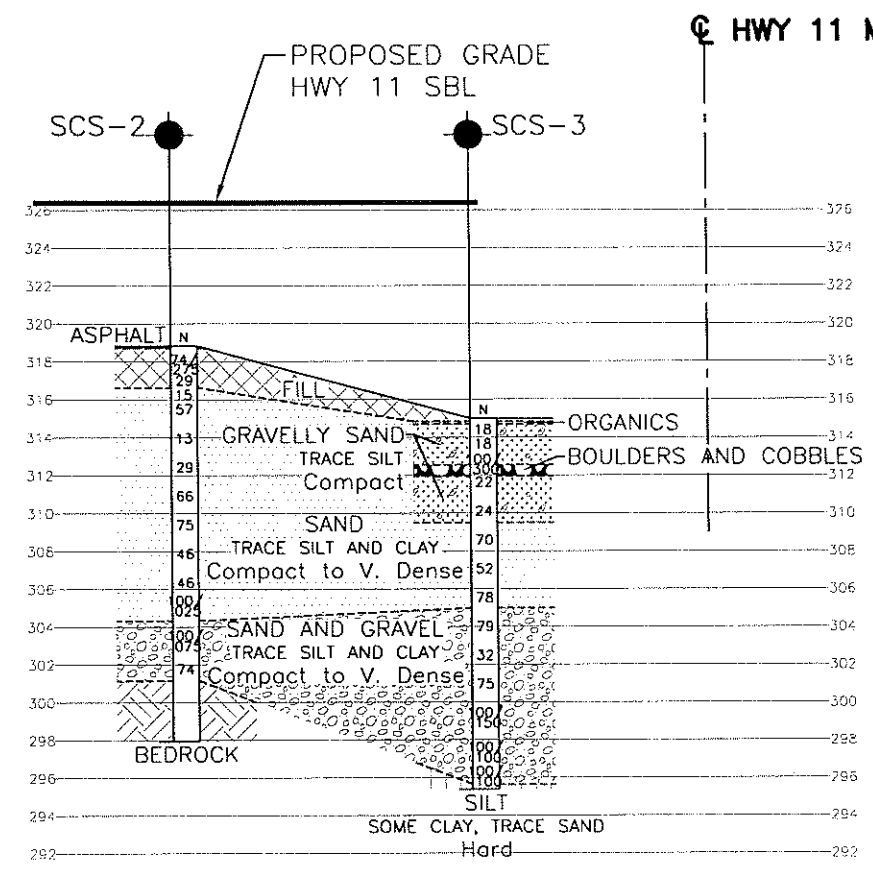
- Borehole by THURBER
- N Blows /0.3m (Std Pen Test, 475J/blow)
- CONE Blows /0.3m (60° Cone, 475J/blow)
- PH Pressure, Hydraulic
- Water Level
- Head Artesian Water
- Piezometer
- 90% Rock Quality Designation (RQD)
- A/R Auger Refusal

NO	ELEVATION	NORTHING	EASTING
SCS-1	319.0	5 059 675.41	310 644.51
SCS-2	318.8	5 059 690.88	310 636.85
SCS-3	315.0	5 059 694.86	310 651.95
SCS-4	317.0	5 059 745.00	310 618.54
SCS-5	313.0	5 059 749.98	310 637.31
SCS-6	312.6	5 059 765.31	310 622.93

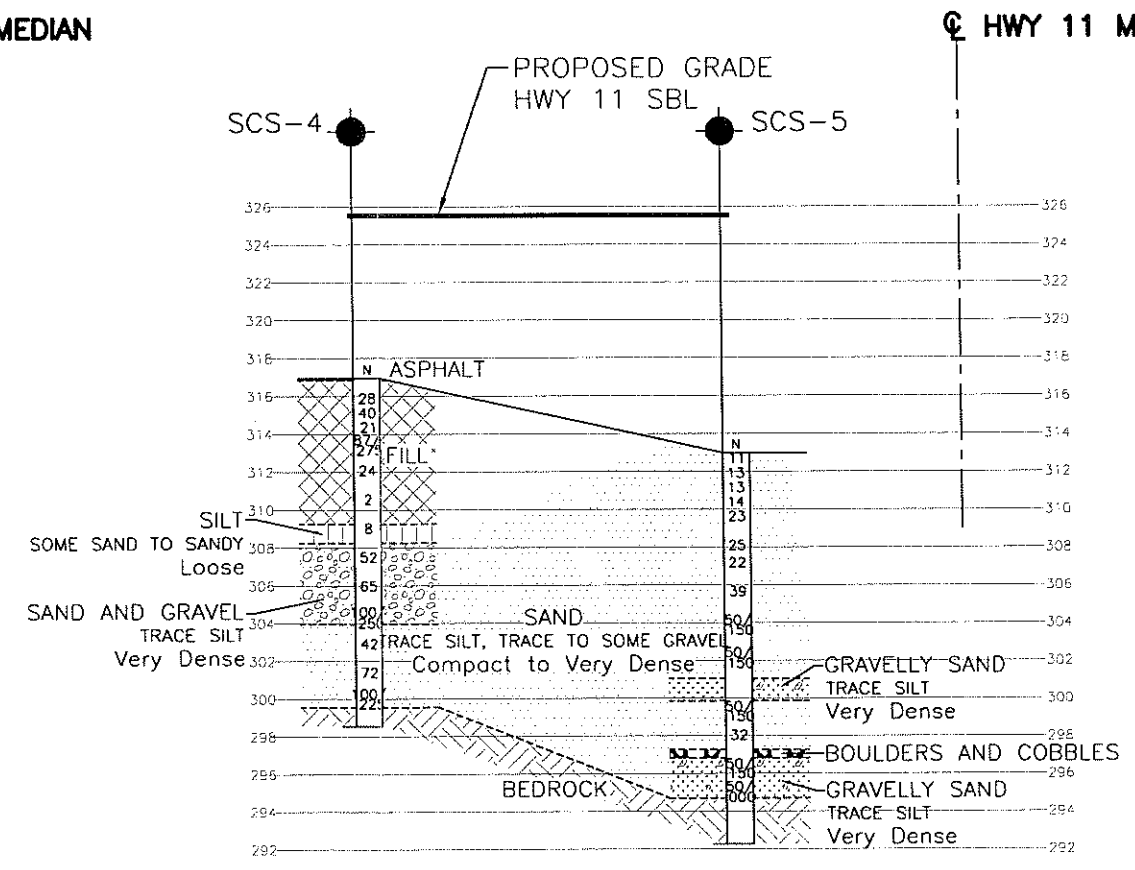
-NOTES-

- 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.

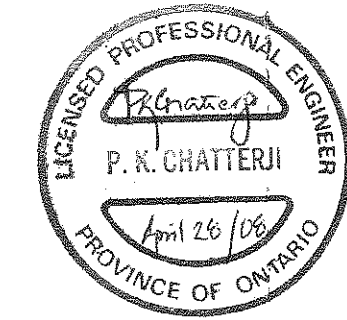
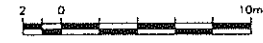
GEOCRES No. 31E-276



SECTION A-A



SECTION B-B



DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

REVISIONS	DATE	BY	DESCRIPTION
DESIGN	AEG	CHK	AEG
DRAWN	MFA	CHK	AEG
CODE	SITE	STRUCT	SCHEME
LOAD	DATE	APR 2008	DWG 3

DRAWING NAME:
CREATED:

DHO BENCHMARK 343-67
ELEVATION 329.137
TABLET IN ROCK OUTCROP
49.337m RL 24+795.574
TWP OF ARMOUR