

**FOUNDATION INVESTIGATION AND DESIGN REPORT
HWY 71 STURGEON CREEK BRIDGE REHABILITATION
G.W.P. 121-97-00, SITE: 45-046**

GEOCREs No. 52C-19

Report to

Cook Engineering

Thurber Engineering Ltd.
2010 Winston Park Drive, Suite 103
Oakville, Ontario
L6H 5R7
Phone: (905) 829 8666
Fax: (905) 829 1166

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PART 1: FACTUAL INFORMATION

1 INTRODUCTION

This report presents the factual findings obtained from a foundation investigation conducted at the site of Highway 71 crossing of Sturgeon Creek, located south of Kenora, Ontario.

A foundation investigation program was conducted at this site between June 5th & 14th, 2006. The factual data from this investigation has been used in preparing this report.

The purpose of the investigation was to explore the subsurface conditions at the site and, based on the data obtained, to provide a borehole location plan, borehole logs, stratigraphic profile, and a written description of the subsurface conditions. A model of the subsurface conditions was developed, through interpretation of the data obtained from the present investigation.

Thurber carried out the investigation as a sub-consultant to Cook Engineering, under the Ministry of Transportation Ontario (MTO) Agreement Number 6005-E-0006.

2 SITE DESCRIPTION

The site is located on Highway 71, 14.2 km north of the junction between Highways 11 and 71. Highway 71 crosses Sturgeon Creek on a multi span timber pile supported bridge. The existing structure carries both the north and southbound lanes. Sturgeon Creek is approximately 6 m in width with a water level elevation of approximately 361.1m (December 5, 2004). The banks are gently inclined and covered in grass vegetation. Flow direction is to the west. There are no residential or commercial properties in the immediate vicinity of the bridge.

Geologically, the site area is located within the physiographic region known as the Canadian Shield, characterized by Pre-Cambrian igneous bedrock typically occurring as rounded protrusions and ridges where exposed. The Quaternary geology of the site area consists primarily of low energy fluvial-glacial overburden deposits of peat, sand, silt and clay.

Photographs of the site are included in Appendix F. Plate 1 is taken from the south side of Hwy 71 looking northward across Sturgeon Creek. Plate 2 illustrates the existing timber piles that support the bridge.

3 SITE INVESTIGATION AND FIELD TESTING

Four boreholes numbered ST-1 to ST-4 were drilled and sampled to depths ranging from 11.1 m to 37.8 m. Borehole locations are shown on the attached Borehole Location and Soil Strata Drawing in Appendix E.

Thurber selected the borehole locations and obtained utility clearances prior to drilling. Cook Engineering provided coordinates and geodetic elevations.

A combination of hollow-stem auger drilling techniques and casing and wash boring methods were used to advance the boreholes. Samples were obtained at selected intervals using a split spoon sampler in conjunction with Standard Penetration Testing (SPT) in the overburden soils.

Boreholes ST-1 and ST-4 were drilled in the approach fills, sampling was undertaken to depths of 11.1 m and Dynamic Cone Penetration Testing was undertaken from a depth of 11.1 m to refusal.

One borehole adjacent to the south abutment foundation (ST-2) was advanced 3.1 m into bedrock with NQ size diamond coring.

Groundwater conditions in the open boreholes were observed throughout the drilling operations. Two standpipe piezometers were installed and enclosed in filter sand to permit longer term groundwater level monitoring. The location, installation and borehole completion details are shown in Table 3.1. The installed piezometers were subsequently decommissioned and grouted with bentonite, and capped with concrete and asphalt at the completion of the field investigation.

Table 3.1 – Borehole Completion Details

Location	Piezometer Tip Depth/ Elevation (m)	Completion Details
ST-1	No Installation	Bentonite grout from 15.0 m to 1.2 m, hole plug from 1.2 m to 0.6 m, sand and gravel from 0.60 m to 0.30 m, concrete from 0.30m to 0.15m and asphalt from 0.15 to ground level.
ST-2	28.9 / 334.5	Piezometer - 19 mm PVC pipe with 1.5 m slotted screen installed at 29.0 m to 27.5 m with sand filter from 29.0 m to 27.1 m, hole plug from 27.1 m to 26.2 m, bentonite grout from 26.2 m to 0.9 m, hole plug from 0.9 m to 0.3 m and concrete seal from 0.3 m to ground level with a flush mounted protective cover. Artesian pressure observed above bedrock. Artesian conditions through the piezometer pipe persisted for 5 days post drilling until the installation was grouted.

ST-3	4.5 / 358.6	Bentonite grout from 30 m to 6.0 m, hole plug from 6.0 m to 4.5 m, Piezometer - 19 mm PVC pipe with 1.5 m slotted screen installed at 4.5 m to 3.0 m with sand filter from 4.5 m to 2.7 m, hole plug from 2.7 m to 0.3 m and concrete seal from 0.3 m to ground level with a flush mounted protective cover.
ST-4	No Installation	Bentonite grout 15.2 m to 0.9 m, hole plug from 0.9 m to 0.5 m, concrete from 0.5 m to 0.2 m and asphalt from 0.15 to ground level.

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

All rock cores were logged with Total Core Recovery (TCR), Solid Core Recovery (SCR), Rock Quality Designation (RQD) and the Fracture Indices (FI) determined on site prior to transportation to Thurber's laboratory.

4 LABORATORY TESTING

Visual identification (VI) and natural moisture content (MC) determination was undertaken on all recovered soil samples returned to the laboratory. The results of this testing are shown on the Record of Borehole sheets in Appendix A. Selected granular soils were subjected to gradation analysis. Selected cohesive soils underwent gradation and Atterberg Limit tests. The results of the testing program are shown on the Record of Borehole sheets and are presented graphically in Appendix B. Rock core strengths estimated from point load tests are recorded on the borehole logs 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 this appendix and on the "Borehole Locations and Soil Strata" drawing in Appendix E. 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.

The boreholes drilled indicate a deep deposit of overburden, overlying bedrock. The overburden deposits generally consist of granular fill, silty clay and silt, sand and gravel.

5.1 Asphalt

Thicknesses of 150mm to 170 mm of asphalt were encountered at the borehole location.

5.2 Granular Fill

Underlying the asphalt pavement, all boreholes encountered a layer of granular fill consisting of sand, gravel and trace silt with cobbles. The underside of the fill layer was

recorded at elevations between 360.8 m and 361.3 m. The thickness of the granular fill ranges from 1.9 m to 2.1 m.

'N' values ranging from 19 to 58, derived from Standard Penetration Tests conducted in the sand and gravel fill, indicate a compact to very dense relative density.

Moisture content of samples from the granular fill ranged from approximately 3% to 16%.

Gradation of the granular fill is presented in Figure B-1 in Appendix B.

5.3 Peat

An inclusion of peat was encountered below the fill in the south approach. The peat thickness was 0.1 m (ST-1). A moisture content of 72 % was recorded.

5.4 Silty Clay

A relatively deep deposit of grey silty clay with trace to some sand, trace gravel was encountered underlying the granular fill in all boreholes. This deposit extends to approximately elevation 339 m. Thicknesses of 21.7 m to 22.1 m were recorded at the north and south abutments.

SPT 'N' values range from 4 to 13 blows per 0.3 m penetration, indicating the consistency of the silty clay to be soft to stiff.

Ten grain size distribution tests were undertaken on this material, the results of which are presented in Appendix B and summarised in Table 5.1.

Table 5.1 – Summary of Grain Size Distribution tests undertaken on silty clay

	Sand (%)	Silt (%)	Clay (%)
Range	1 to 24	18 to 38	35 to 81

The moisture content of this material ranges from 21 % to 46 %. The silty clay has a Liquid Limit ranging from 37 % to 77 % and a Plasticity Index of 22 to 52, indicative of inorganic, medium to high plasticity clay.

5.5 Sand

A layer of dark brown to grey wet sand with trace clay, 0.7 m to 0.9 m in thickness, was encountered interbedded in the clay at the north abutment and approach at depths of 3.1 m and 2.1 m, respectively.

Two SPT N values of 7 and 18 were recorded in this stratum indicating a loose to compact relative density. The moisture content of the sand ranges from 18 % to 32 %

5.6 Sand and Silt

A layer of grey wet sand and silt with trace clay was encountered underlying the silty clay at approximately 24 m below ground level at the north and south abutments (ST-2, ST-3). The sand and silt layer ranged in thickness from 6.1 m to more than 7.1 m. The top elevation of this layer is at about 339.0 m and the base of layer is deeper than 332.2 m.

SPT N values ranging between 8 and 22 were recorded in this stratum indicating a loose to compact relative density.

Moisture content of this layer ranges from 18 to 22 %. Gradation of this material is presented in Figure B4 in Appendix B.

5.7 Sand and Gravel

A layer of wet grey sand and gravel with cobbles and boulders was encountered below the sand and silt deposit at the south abutment (ST-2), the base of which was recorded at 328.6 m, overlying bedrock. This layer is 4.2 m in thickness.

An SPT N value of 23 was recorded near the top of this stratum indicating a compact relative density. An SPT N value of 100 blows for 25 mm was recorded deeper in the deposit, indicating refusal on a cobble or boulder. The moisture content of this layer is about 19 %. Based on the observations in the borehole and in the piezometer, this layer may be the source of artesian water pressure.

5.8 Bedrock

The overburden soils described above are underlain by mafic and ultra mafic, granitic rock. Rotary coring at the south abutment identified bedrock. Table 5.2 summarizes depth to bedrock, rock mass classification, and unconfined compressive strength estimated from point load testing.

TABLE 5.2 – Bedrock Quality at Foundation Elements

Borehole Location	NQ Coring	Depth to Bedrock (m)	Elevation of Top of Bedrock	NQ Core recovery (m)	Rock Quality Designation (RQD)	Estimated Rock Strength (MPa)
ST- 1	N	-	-	-	-	-
ST - 2	Y	34.7	328.6	3.1	Very Poor to Fair 15 % - 68%	173 286
ST - 3	N	-	-	-	-	-
ST - 4	N	-	-	-	-	-

100 % total core recovery (TCR) was achieved in rock coring at the south abutment (ST-2). The Fracture Index (FI) of the rock, expressed as fractures per 0.3 m of core, was

between 0 and greater than 5. The rock mass classification based on RQD values ranged from 15% to 68% indicating poor to fair quality rock. The RQD value increases with depth indicating rock mass quality increasing with depth. The bedrock is visually described as fresh to slightly weathered, grey, and crystalline.

Based on laboratory Point Load testing conducted on 2 rock core samples, unconfined compressive strength values were estimated to be between 173 MPa and 286 MPa indicating very strong to extremely strong bedrock.

5.9 Groundwater Conditions

Two standpipe piezometers were installed in Boreholes ST-2 and ST-3 to assess groundwater levels and to allow for long term monitoring. The piezometer readings are presented in Table 5.3.

Table 5.3 – Water Level Measurements

Date	BH ST-1*		BH ST-2		BH ST-3		BH ST-4*	
	Depth (m)	Elev. (m)	Depth (m)	Elev. (m)	Depth (m)	Elev. (m)	Depth (m)	Elev. (m)
June 5, 2006	-	-	-	-	-	-	2.3	360.7
June 7, 2006	5.8	357.8	-	-	-	-	-	-
June 9, 2006	-	-	+ 0.7	364.1	-	-	-	-
June 10, 2006	-	-	-	-	2.3	360.8	-	-
June 12, 2006	-	-	-	-	2.3	360.8	-	-
June 14, 2006	-	-	+ 1.1	364.5	2.3	360.8	-	-
June 15, 2006	-	-	-	-	2.3	360.8	-	-

* Water levels in open boreholes

+ Denotes artesian head of water above ground surface

Artesian groundwater flow was encountered in Borehole ST-2 at the south abutment when the borehole reached a depth of 31 m (elevation 332.1 m) on June 8, 2006. The initial flow rate from the borehole was estimated to be in the range of 200 to 400 l/min. The drill casing was extended to a height of 9 m above ground level in order to contain the flow. By the following day, June 9, 2006, the pressure and flow had reduced and the water level had stabilized at approximately 0.7 m above the highway grade. A piezometer was sealed into this borehole.

On June 14, 2007, a new borehole was drilled adjacent to the original Borehole ST-2 and was advanced to bedrock and cored 3 m into bedrock. Artesian flow was again encountered immediately upon rock coring, and a head of 9 m above highway grade was required to stop the flow. The casing was then pulled back to approximately 6 m above the

bedrock contact and the flow rapidly diminished to a trickle at highway grade. By June 15, 2007, the groundwater level had stabilized at approximately 500 mm above highway grade.

It is concluded that the sand and gravel with cobbles and boulders overlying the bedrock forms a confined aquifer in which artesian pressure develops to the equivalent of a 9 m head above the highway grade. The behaviour in the exploratory boreholes indicates that the pressure dissipates with time, suggesting that the static pressure in the aquifer is high but the recharge rate is low.

The standpipe piezometer and the second borehole were successfully decommissioned on June 15, 2007, by grouting with bentonite and cement grouts.

The general groundwater level at the site is high and likely reflects the creek level. All groundwater observations at this site are short term and the levels are expected to fluctuate seasonally and after severe weather events.

6 MISCELLANEOUS

Paddock Drilling Ltd of Brandon, Manitoba supplied a MPS-T Acker™ truck mounted drill rig and conducted the drilling, sampling and in-situ testing operations.

Mr. George Azzopardi of Thurber Engineering supervised the drilling and sampling operations in the field on a full time basis.

Mr. Alastair E. Gorman, P.Eng. and Mr. Mark E. Farrant, P.Eng. directed field operations.

Mr. Alastair E. Gorman, P.Eng. and Mr Tony Harte prepared the report.

Dr. P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations projects, reviewed the report.

Thurber Engineering Ltd.

Alastair E. Gorman, P.Eng.,
Senior Foundations Engineer



Report Reviewed by:
P.K. Chatterji, P.Eng.,
Review Principal, Designated MTO Contact



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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 bridge replacement.

The existing structure will be rehabilitated and the process is expected to result in a 0.2 m grade raise above the existing approach embankments.

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 this investigation.

8 STRUCTURE FOUNDATIONS

Based on the boreholes drilled at the foundation elements, the stratigraphy at the site consists of granular fill, cohesive and cohesionless soils overlying bedrock. A synopsis of the soils at each foundation element is presented in Table 8.1. For a detailed description of the soil stratigraphy, refer to Section 5 of the report.

Table 8.1 – Summarized Soil Conditions

Borehole Location	Geodetic Elevation (m)	Stratigraphy	Groundwater (m)
South Approach ST-1	363.6 to 361.2	Sand & Gravel Fill, Peat	357.81
	361.2 to 352.4	Silty Clay	
	352.4 to 348.2	Unknown overburden. DCPT Refusal @ 348.2 m	
South Abutment ST-2	363.4 to 361.1	Sand & Gravel Fill	Groundwater 364.5 Artesian groundwater from sand and gravel
	361.1 to 339.0	Silty Clay	
	339.0 to 332.9	Silt	

	332.9 to 328.6	Sand & Gravel	layer at 332.1 m
	328.6 to 325.6	Granite bedrock	
North Abutment ST-3	363.1 to 361.0	Sand & Gravel Fill	360.8
	361.0 to 360.0	Silty Clay	
	360.0 to 359.3	Sand	
	359.3 to 339.3	Silty Clay	
	339.3 to 332.2	Sand	
North Approach ST-4	363.0 to 360.8	Sand & Gravel Fill	360.7
	360.8 to 359.9	Sand	
	359.9 to 351.8	Silty Clay	
	351.8 to 347.6	Unknown – DCPT Refusal @ 347.6 m	

Initial consideration was given to the following foundation types:

- Spread footings (on native soil or engineered fill)
- Augered Caissons (drilled shafts)
- Driven piles – end bearing
- Driven piles - friction

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

8.1 Spread Footings on Native Soil

Spread footings founded on the native soil are not considered to be suitable at this site for the following reasons:

- The low geotechnical resistance available in the native soil
- The risk of unacceptably large settlements under footings
- The potential for footings being undermined by hydraulic erosion

Accordingly, spread footings on native soil were not analysed further.

8.2 Spread Footings on Engineered Fill

Spread footings on engineered fill are not recommended and are not considered to be suitable at this site for the same reasons as above.

8.3 Caissons Founded in Soil

Caissons in soil are not considered to be feasible on account of the low geotechnical resistance that could be developed. Caissons socketted into bedrock at or below elevation 328.6 m are not considered to be economically feasible. It would also be difficult to achieve a good seal in the artesian bearing sand and gravel layer, just above the bedrock.

8.4 Driven Steel H Piles in End Bearing

The soil stratigraphy at the site is considered to be suitable for the support of foundations on driven steel H-piles.

The stratigraphy encountered at the abutments consists of stiff to firm cohesive and loose to compact cohesionless soils to elevation 328.6 m at the south abutment and deeper than 332.2 m at the north abutment. Bedrock was not encountered at 30.9 m depth (elev. 332.2 m) at the north abutment (ST-3). At the south abutment a layer of sand and gravel with cobbles and boulders was encountered below elevation 332.9 m. Bedrock was encountered at elevation 328.6 m. It is expected that driven steel H piles will achieve resistance in this sand and gravel stratum below elevation 332.9 m, and the geotechnical resistance values are based on this assumption. Estimated pile lengths for driven steel H piles are presented in Table 8.2.

Table 8.2 – Estimated Pile Lengths

Location	Borehole No.	Top of Sand and Gravel or Sand (m)	Top of Bedrock Elevation	Estimated Length of Pile (m)	Estimated Pile Tip Elevation (m)
South Abutment	ST-2	332.9	328.6	32.0*	331.4
North Abutment	ST-3	339.3	Deeper than 332.2	32.0*	331.1

*Assume for design and costing purposes

8.4.1 Axial Resistance

The factored, vertical, concentric, geotechnical resistances at ULS and SLS for two pile sections driven to the tip elevation estimated above are as follows:

Table 8.3 – Factored Geotechnical Resistances at ULS & SLS

Pile Type	Factored ULS (kN)	SLS (kN) (25 mm Settlement)
HP 310 x 79	1200	1000
HP 310 x 110	1600	1400

The structural designer must check the structural resistance of the pile.

Oversize materials (e.g. greater than 75 mm nominal diameter) must not be used in the fills through which the piles will be driven.

8.4.2 Pile Tips

Cobbles and boulders were encountered overlying the bedrock at the south abutment and should be anticipated at the north abutment. The tips of all piles should be fitted with H-section rock points from an approved manufacturer such as Titus Steel (Standard H-point) or approved equivalent.

In the case of partial bearing on bedrock, the cast steel point will provide better stress redistribution without failure than would be achieved in a pile tip reinforced with a driving shoe.

8.4.3 Pile Installation

Pile installation should be in accordance with Special Provision No. 903S01 but modified in accordance with the NSSP attached in Appendix D. This NSSP addresses the provision of a sand pad covering the tops of piles to drain any possible artesian seepage along the pile shaft.

8.4.4 Pile Driving

Pile driving must be controlled by the Hiley Formula and an ultimate pile resistance should be specified by the designer in accordance with Clause 3.3.2 (b) Construction Stage of the Structural Manual. 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 value shown in Table 8.3.

Table 8.4 – Ultimate Geotechnical Resistance of Piles

Pile	Ultimate Resistance (R) (kN)
HP 310 x 79	2400
HP 310 x 110	3200

8.5 Driven Friction Piles

Structural loads could also be supported on friction piles.

Analysis indicates that the frictional resistance developed by an HP 310 X 110 pile driven to a depth of 20 m below the existing road grade (approximate Elevation 343.2) is as follows:

Factored ULS	380 kN
SLS	250 kN

8.5.1 Pile Tips

In the case of friction piles, the piles will be driven in the clay stratum and protection of the pile tips is not considered to be necessary.

It is recommended that neither driving shoes nor pile tips be used in the case of friction piles at this site.

8.6 Downdrag

Downdrag on the piles will not be an issue at this site, provided the grade is not raised more than 200mm.

8.7 Pile Lateral Resistance

The geotechnical lateral resistance acting on a 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 = coefficient of horizontal subgrade reaction (Table 8.4)

γ = unit weight (Table 8.4)

K_p = passive earth pressure coefficient (Table 8.4)

Table 8.5 – Recommended Soil Parameters

Location	Elevation	n_h (kN/m^3)	K_p	Unit Weight* (kN/m^3)	Soil Conditions
South Abutment	363.4 to 361.1	10 000	3.0	22	Compacted fill
	361.1 to 339.0	3 000	2.8	9	Silty Clay firm to stiff
	339.0 to 332.9	5 000	3.0	10	Sand and Silt, compact
	332.9 to 328.6 (bedrock)	10 000	3.3	12	Sand and Gravel with cobbles and boulders
North Abutment	363.1 to 361.0	10,000	3.0	22	Compacted fill
	361.0 to 339.3	3 000	2.8	9	Silty Clay firm to stiff
	339.3 to 332.2	5 000	3.0	10	Sand and Silt, compact

*Buoyant unit weight below the water table.

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 must not exceed the ultimate lateral resistance.

The spring constant, K , for analysis may be obtained by the expression, $K = k_s \times L \times D$ (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, P_{ult} may be obtained from the expression, $P_{ult} = p_{ult} \times L \times D$. 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 assumed in one pile be limited to no more than 150 kN at ULS and 50 kN at SLS.

Since the piles are end bearing on dense sand and gravel, the vertical resistance will not be significantly affected by the pile spacing. Pile interaction should be considered with reference to CHBDC Clause 6.8.9.2.

For lateral soil/pile group interaction analysis, the equation for k_s and p_{ult} quoted above may be used in conjunction with appropriate reduction factors.

Where a pile group is oriented *perpendicular* to the direction of loading, group action may be considered by reducing values for k_s and p_{ult} 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, battered piles may provide the lateral resistance.

8.8 Integral Abutment Considerations

From a geotechnical perspective, the subsurface conditions at this site are considered to be suitable for the construction of conventional, semi-integral or integral abutments. An H-pile foundation is required for an integral abutment design.

The integral abutment design requires that the piles possess flexibility in the upper 3 m of the pile length. 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 filled with sand.

The sand must be placed in the CSP after the pile has been driven to avoid the danger of the sand being densified by pile driving and the possibility of the CSP being dragged down by the pile.

Backfill sand should meet the gradation shown in Table 8.6.

Table 8.6 – 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%

8.9 Frost Cover

Pile caps must be provided with a minimum of 2.3 m of earth cover over the pile cap base (founding elevation).

8.10 Erosion Protection

It is recommended that foundations are protected from erosion by the Creek. Placement of suitably designed riprap on the watercourse side of the piles should be considered.

9 EXCAVATION AND BACKFILL

9.1 General

All excavations must be carried out in accordance with the Occupational Health and Safety Act (OHSA). For the purposes of the OHSA, the native soils at this site may be classified as Type 3 soils above the water table and Type 4 soils below the water table. Excavation below the groundwater level is not recommended without prior dewatering. Provided dewatering is carried out as described below, temporary excavations may be sloped at 1H:1V.

9.2 Foundations

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

Bidders must be alerted to the fact that excavation must be carried out through cohesionless soils, which may include man-made fill or obstructions, cobbles and boulders.

10 TEMPORARY GROUNDWATER AND FLOOD CONTROL

At the time of investigation, local artesian groundwater was encountered at the south abutment borehole. The source of the artesian water is believed to be the sand and gravel strata at about 30 m depth, directly above bedrock. The general water level at this site is also high and within 1 to 2 m of the ground surface, and likely reflects the creek water level.

The groundwater level will vary and may be higher at the time of construction. At this site, the design of dewatering and protection systems must also take account of the possibility of the Sturgeon Creek water level rising rapidly due to flood conditions. The groundwater and surface (flood) water must be controlled during construction to maintain a stable foundation excavation and to allow concrete to be placed in a dry excavation.

The design of the groundwater control system is the responsibility of the Contractor. However, suitable systems that might be considered include pumping from filtered sumps for very nominal penetration below the groundwater level or the use of a sheet pile enclosure for deeper penetration below the water level.

Any accumulation of water from the base of the excavation should be removed prior to placing concrete or compacting granular fill. Placement of concrete or compacting engineered fill must be done in the dry.

11 LONG TERM SEEPAGE CONTROL

If piles are driven to bedrock, they will penetrate the zone in which groundwater under an artesian head was encountered. Since the piles will be driven through a thick deposit of plastic clay, it is considered likely that the clay will form a seal against the piles and the risk of long term upward seepage along the pile shaft is low, although there may be an initial flow of water when the first pile is driven.

As a precaution against possible long term seepage removing soil particles, a sand filter should be constructed around the upper portion of the pile shafts. Suggested wording for a NSSP is included in Appendix D.

Friction piles driven entirely in the clay and not penetrating below the base of the clay are not expected to encounter any seepage problems.

12 APPROACH EMBANKMENTS

It is understood that the highway will be maintained on the present horizontal alignment and that only a 200 mm grade raise is contemplated. This magnitude of grade raise is not expected to result in significant settlement of the embankment.

13 BACKFILL TO ABUTMENTS

The backfill to the abutment walls should be in accordance with OPSS 902 as amended by Special Provision 902S01. Granular backfill should be placed to the extents shown in OPSD 3101.150.

All granular material should meet the specifications of Special Provision 110F13 "Amendment to OPSS 1010, March 1993". Compaction equipment to be used adjacent to retaining structures should be restricted in accordance with SP 105S10.

Some settlement will occur within the mass of the approach fill after the fill has been completed. For design purposes, the settlement at final grade should be assumed to equal 0.5% of the height of the fill.

The design of the abutment should incorporate a subdrain as shown in OPSD 3101.150 and weeping drains as shown in OPSD 3190.100.

14 ROADWAY PROTECTION

If the bridgework is staged within the existing platform, it is anticipated that roadway protection may be required during construction. In that case, an item titled "Protection System" as per SP 105S19 should be included in the contract documents. It is recommended that Performance Level 2, as per Clause 539.04.02.01, be specified for this site and that the alignment of the roadway protection be shown on the contract drawings.

The design of roadway protection is the responsibility of the Contractor. However, one option that is considered to be suitable for use as temporary shoring at this site is a soldier pile and lagging wall.

The geotechnical parameters for the design of the roadway protection are provided in Table 14.1. All shoring systems should be designed by a Professional Engineer experienced in such designs.

Table 14.1 – Geotechnical Parameter for Roadway Protection Design

Soil Type	Bulk Unit Weight	Submerged Unit Weight	Angle of Internal Friction	Undrained Shear Strength
Granular Fill	21 kN/m ³	11 kN/m ³	32°	-
Silty Clay	19 kN/m ³	9 kN/m ³	-	75 kPa

15 EARTH PRESSURE

For cases where backfill to the abutment is placed in accordance with OPSD 3101.150 or OPSD 3101.200, as recommended, the lateral earth pressure will be governed by the properties of the material within the backfill limits shown in the respective OPSD, i.e. a line projected up at 1.5H:1V for granular backfill and 1.25H:1V for rock backfill.

If the support system allows yielding of the wall (unrestrained system), active horizontal earth pressure may be used in the geotechnical design of the structure. If the support system does not allow yielding (restrained system), at-rest horizontal earth pressures should be used.

Earth pressures acting on the structure should be computed in accordance with Clause 6.9 of the CHBDC but generally are given by the expression for a fully drained condition:

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

P_h = horizontal pressure on the wall (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)

Table 15.1 – Earth Pressure Coefficients

Wall 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 $\phi = 42^\circ; \gamma = 19.0 \text{ kN/m}^3$	
	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 (Unrestrained Wall)	0.27	0.40*	0.31	0.48*	0.20	0.28*
At rest (Restrained Wall)	0.43	-	0.47	-	0.33	-
Passive (Movement Towards Soil Mass)	3.70	-	3.30	-	5.0	-

* For wing walls.

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 at a depth of 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 given in Table 15.1.

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 or semi-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. However, the use of Granular "B" Type I may be restricted if the approach embankment consists of rock fill.

The factors in the Table 15.1 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 CHBDC, 2000.

16 SEISMIC CONSIDERATIONS

16.1 Seismic Design Parameters

The site is treated as lying in Seismic Zone 0. The following seismic parameters should be used for design:

- | | |
|-------------------------------------|------|
| • Velocity Related Seismic Zone | 0 |
| • Zonal Velocity Ratio | 0.00 |
| • Acceleration Related Seismic Zone | 0 |
| • Zonal Acceleration Ratio | 0.00 |
| • Peak Horizontal Acceleration | 0.00 |

17 CONSTRUCTION CONCERNS

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

- Driving H-Piles through obstructions such as cobbles and boulders and other obstructions in the approach fill, remnants of buried timber or timber piles.
- It is envisaged that the bridge rehabilitation will be staged, so that one lane of traffic is maintained. The existing bridge foundations should be monitored for settlement during driving of the adjacent new piles and corrective action taken if settlement is detected.

- Piles driven below the base of the clay layer may encounter artesian water conditions, in which case the sand pad described in Appendix D will be required
- Adequate dewatering of excavation for sand bed and pile cap construction

18 CLOSURE

Mr. T Harte, M.Sc., FGS and Mr. Alastair E. Gorman, P.Eng, carried out engineering analysis and preparation of the report.

Dr. P.K. Chatterji, P.Eng, a designated Principal Contact for MTO Foundations Projects reviewed the report

Thurber Engineering Ltd.

Alastair E. Gorman, P.Eng.
Senior Foundations Engineer



P. K. Chatterji, P.Eng.
Review Principal



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

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


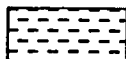



 Water Level
 Shear Strength Determination by Pocket Penetrometer

- (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
CLAY SHALE			
SANDSTONE			
SILTSTONE			
CLAYSTONE			
COAL			

EXPLANATION OF ROCK LOGGING TERMS

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

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

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

RECORD OF BOREHOLE No ST-1

1 OF 2

METRIC

G.W.P. 121-97-00 LOCATION Sturgeon River N 5 404 220.68 E 237 361.87 ORIGINATED BY GA
 HWY 71 BOREHOLE TYPE Hollow Stem Augers/Dynamic Cone Penetration Test COMPILED BY WM
 DATUM Geodetic DATE 07.06.06 - 07.06.06 CHECKED BY MEF/TH

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					WATER CONTENT (%)		
								20 40 60 80 100					20 40 60		
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE					w _p — w — w _L		
363.6															
0.0	ASPHALT: (150 mm)														
0.2	SAND and GRAVEL, trace silt, occasional cobbles Dense to Compact Brown Dry (FILL)		1	SS	32		363					32 57 11 (SI+CL)			
			2	SS	23		362								
			3	SS	19										
361.3															
361.3	PEAT, fibrous Black		4	SS	10		361								
2.4	Silty CLAY, some sand, trace rootlets, occasional wood fibers Stiff to Firm Grey		5	SS	6										
			6	SS	12		360					1 15 33 51			
359.7							359								
3.8	Silty CLAY, trace to some sand, trace gravel Stiff to Firm Grey (CI-CH)		7	SS	9		358								
			8	SS	10		357								
							356								
			9	SS	7		355								
							354					1 9 26 64			
			10	SS	9										

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Sensitivity

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(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No ST-1

2 OF 2

METRIC

G.W.P. 121-97-00 LOCATION Sturgeon River N 5 404 220.68 E 237 361.87 ORIGINATED BY GA
 HWY 71 BOREHOLE TYPE Hollow Stem Augers/Dynamic Cone Penetration Test COMPILED BY WM
 DATUM Geodetic DATE 07.06.06 - 07.06.06 CHECKED BY MEF/TH

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT			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	W _p W W _L				
	Continued From Previous Page													
352.4	Silty CLAY, trace to some sand, trace gravel Stiff to Firm Grey (CI-CH)		11	SS	7									
11.1	END OF SOIL SAMPLING AT 11.13m. Dynamic Cone Penetration Test (DCPT) Started at 11.13 m.													
348.2														
15.4	END OF DCPT AT 15.39 m. BOREHOLE OPEN TO 15.39 m AND WATER LEVEL AT 5.79 m UPON COMPLETION. BOREHOLE GROUTED WITH BENTONITE TO 0.6m AND PATCHED WITH CONCRETE AND ASPHALT TO SURFACE. WATER LEVEL READINGS: DATE DEPTH(m) ELEV.(m) 07/6/06 5.79 357.81													

RECORD OF BOREHOLE No ST-2

1 OF 4

METRIC

G.W.P. 121-97-00 LOCATION Sturgeon River N 5 404 230.73 E 237 354.08 ORIGINATED BY GA
 HWY 71 BOREHOLE TYPE Hollow Stem Augers/NW Casing/NQ Coring COMPILED BY WM
 DATUM Geodetic DATE 06.06.06 - 14.06.06 CHECKED BY MEF/TH

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					WATER CONTENT (%)
								○ UNCONFINED ● QUICK TRIAXIAL	+ FIELD VANE × LAB VANE				
363.4							20 40 60 80 100					GR SA SI CL	
0.0	ASPHALT: (150 mm)						+0.68					*Artesian	
0.2	SAND and GRAVEL, trace silt, occasional cobbles Compact to Dense Brown Dry (FILL)		1	SS	28		363						
			2	SS	32								
			3	SS	22		362						
361.1													
2.3	Silty CLAY, some sand, trace rootlets, occasional wood fibers Firm Grey		4	SS	7		361						
			5	SS	6		360						
359.6													
3.8	Silty CLAY, some sand to sandy, trace gravel Stiff to Firm Grey (CI-CH)		6	SS	9		359					3 24 38 35	
			7	SS	7		358						
			8	SS	11		357						
			9	SS	8		356					0 13 30 57	
			10	SS	7		355						
							354						

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Sensitivity 20
15 5
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RECORD OF BOREHOLE No ST-2

4 OF 4

METRIC

G.W.P. 121-97-00 LOCATION Sturgeon River N 5 404 230.73 E 237 354.08 ORIGINATED BY GA
 HWY 71 BOREHOLE TYPE Hollow Stem Augers/NW Casing/NQ Coring COMPILED BY WM
 DATUM Geodetic DATE 06.06.06 - 14.06.06 CHECKED BY MEF/TH

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		
	Continued From Previous Page							SHEAR STRENGTH kPa						
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE						
								WATER CONTENT (%)						
								20	40	60	80	100		
								PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT				
								W _P	W	W _L				
332.9	SILT AND SAND, trace clay Loose to Compact Grey Wet						333							0 82 18 (SI+CL)
30.5	SAND and GRAVEL, with cobbles and boulders Compact to Very Dense Grey Wet		21	SS	23									
							332							
							331							
							330							
							329							
328.6														
34.7	GRANITE BEDROCK Slightly weathered to Fresh, coarse grained, very strong		1	RUN			328							RUN 1# TCR=100%, SCR=37%, RQD=15%, UCS=173MPa
							327							RUN 2# TCR=100%, SCR=88%, RQD=68%, UCS=286MPa
			2	RUN			326							
325.6														
37.8	END OF BOREHOLE AT 37.80 m. BOREHOLE OPEN TO 37.80 m. Artesian pressure 9.1 m above ground. BOREHOLE GROUTED WITH BENTONITE TO 0.6m AND PATCHED WITH CONCRETE AND ASPHALT TO SURFACE. WATER LEVEL READINGS: DATE DEPTH(m) ELEV.(m) 9/6/06 0.67 Above G.S. 14/6/06 1.50 psi* * Pressure gauge reading													

ONTMT4S 0782.GPJ 16/03/07

RECORD OF BOREHOLE No ST-3

1 OF 4

METRIC

G.W.P. 121-97-00 LOCATION Sturgeon River N 5 404 245.56 E 237 362.08 ORIGINATED BY GA
 HWY 71 BOREHOLE TYPE NW Casing COMPILED BY WM
 DATUM Geodetic DATE 08.06.06 - 09.06.06 CHECKED BY MEF/TH

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		W _p	W	W _L		
363.1								20 40 60 80 100						
0.0	ASPHALT (150 mm)							20 40 60 80 100						
0.2	SAND and GRAVEL, trace silt, occasional cobbles Compact to Dense Brown Moist to Wet (FILL)		1	SS	26		363							
			2	SS	25		362							
			3	SS	30									
361.0														6 91 3 (SI+CL)
2.1	Silty CLAY, trace sand, trace wood fibers, trace rootlets Firm Grey		4	SS	6		361							
360.0														
3.1	SAND, fine to medium grained Compact Grey Wet		5	SS	18		360							
359.3														
3.8	Silty CLAY, trace sand, trace rootlets Stiff Grey (CH)		6	SS	8		359							0 1 18 81
358.5														
4.6	Silty CLAY, trace to some sand, trace gravel Firm to Stiff Grey (CH)		7	SS	4		358							
			8	SS	9		357							
			9	SS	6		356							
							355							
			10	SS	6		354							

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Sensitivity 20 15 10 5 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No ST-3

2 OF 4

METRIC

G.W.P. 121-97-00 LOCATION Sturgeon River N 5 404 245.56 E 237 362.08 ORIGINATED BY GA
 HWY 71 BOREHOLE TYPE NW Casing COMPILED BY WM
 DATUM Geodetic DATE 08.06.06 - 09.06.06 CHECKED BY MEF/TH

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT	PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES								
Continued From Previous Page													
	Silty CLAY, trace to some sand, trace gravel Firm to Stiff Grey (CH)		11	SS	7		353						1 15 24 60
			12	SS	4		352						
			13	SS	8		351						
			14	SS	8		350						
			15	SS	4		349						
			16	SS	8		348						
							347						
							346						0 8 23 69
							345						
							344						

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METRIC

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(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No ST-3

4 OF 4

METRIC

G.W.P. 121-97-00 LOCATION Sturgeon River N 5 404 245.56 E 237 362.08 ORIGINATED BY GA
 HWY 71 BOREHOLE TYPE NW Casing COMPILED BY WM
 DATUM Geodetic DATE 2006-06-08 - 2006-06-09 CHECKED BY MEF/TH

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 (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100	W _P	W	W _L		
	Continued From Previous Page																
332.2	SAND and silt, trace clay Loose to Compact Grey Wet		21	SS	22		333										
30.9	END OF BOREHOLE AT 30.94 m. BOREHOLE OPEN TO 30.48 m. BOREHOLE GROUTED WITH BENTONITE TO 6.1m Piezometer installation consists of 19 mm diameter Schedule 40 PVC pipe with a 1.52 m slotted screen. WATER LEVEL READINGS: DATE DEPTH(m) ELEV.(m) 10/6/06 2.30(m) 360.80 12/6/06 2.29(m) 360.81 14/6/06 2.30(m) 360.80 15/6/06 2.31(m) 360.79																

+³ × 3³ Numbers refer to
Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No ST-4

1 OF 2

METRIC

G.W.P. 121-97-00 LOCATION Sturgeon River N 5 404 255.61 E 237 354.29 ORIGINATED BY GA
 HWY 71 BOREHOLE TYPE Hollow Stem Augers/Dynamic Cone Penetration Test COMPILED BY WM
 DATUM Geodetic DATE 05.06.06 - 05.06.06 CHECKED BY MEF/TH

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						
								20 40 60 80 100						
								20 40 60 80 100						
							WATER CONTENT (%)							
							W _p — W — W _L							
							○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE							
							20 40 60 80 100							
							20 40 60							
363.0							363							
0.0	ASPHALT: (175 mm)													
0.2	SAND and GRAVEL, trace silt, occasional cobbles Very Dense to Compact Brown Dry (FILL)		1	SS	58		362							
			2	SS	22									
			3	SS	26									
360.8							361							
2.1	SAND, trace clay, trace rootlets Loose Dark Brown to Grey		4	SS	7									
359.9							360							
3.0	Silty CLAY, trace sand Stiff Grey		5	SS	8									
359.2							359							
3.8	Silty CLAY, trace to some sand, trace gravel Firm to Stiff Grey (Cl to CH)		6	SS	7									
			7	SS	4		358							
							357							
			8	SS	13		356							
							355							
			9	SS	7									
							354							
			10	SS	9									

Continued Next Page

+ 3 × 3 Numbers refer to
Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No ST-4

2 OF 2

METRIC

G.W.P. 121-97-00 LOCATION Sturgeon River N 5 404 255.61 E 237 354.29 ORIGINATED BY GA
 HWY 71 BOREHOLE TYPE Hollow Stem Augers/Dynamic Cone Penetration Test COMPILED BY WM
 DATUM Geodetic DATE 05.06.06 - 05.06.06 CHECKED BY MEF/TH

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT	PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES								
	Continued From Previous Page												
351.8	Silty CLAY, trace to some sand, trace gravel Firm to Stiff Grey (CI to CH)		11	SS	7								
11.1	END OF SOIL SAMPLING AT 11.13 m. Dynamic Cone Penetration Test (DCPT) started at 11.13 m.												
347.6													
15.4	END OF DCPT AT 15.39 m. BOREHOLE OPEN TO 15.39 m AND WATER LEVEL AT 2.29 m UPON COMPLETION. BOREHOLE GROUTED WITH BENTONITE TO 0.45m AND PATCHED WITH CONCRETE AND ASPHALT TO SURFACE. WATER LEVEL READINGS: DATE DEPTH(m) ELEV.(m) 05/6/06 2.29 360.71												

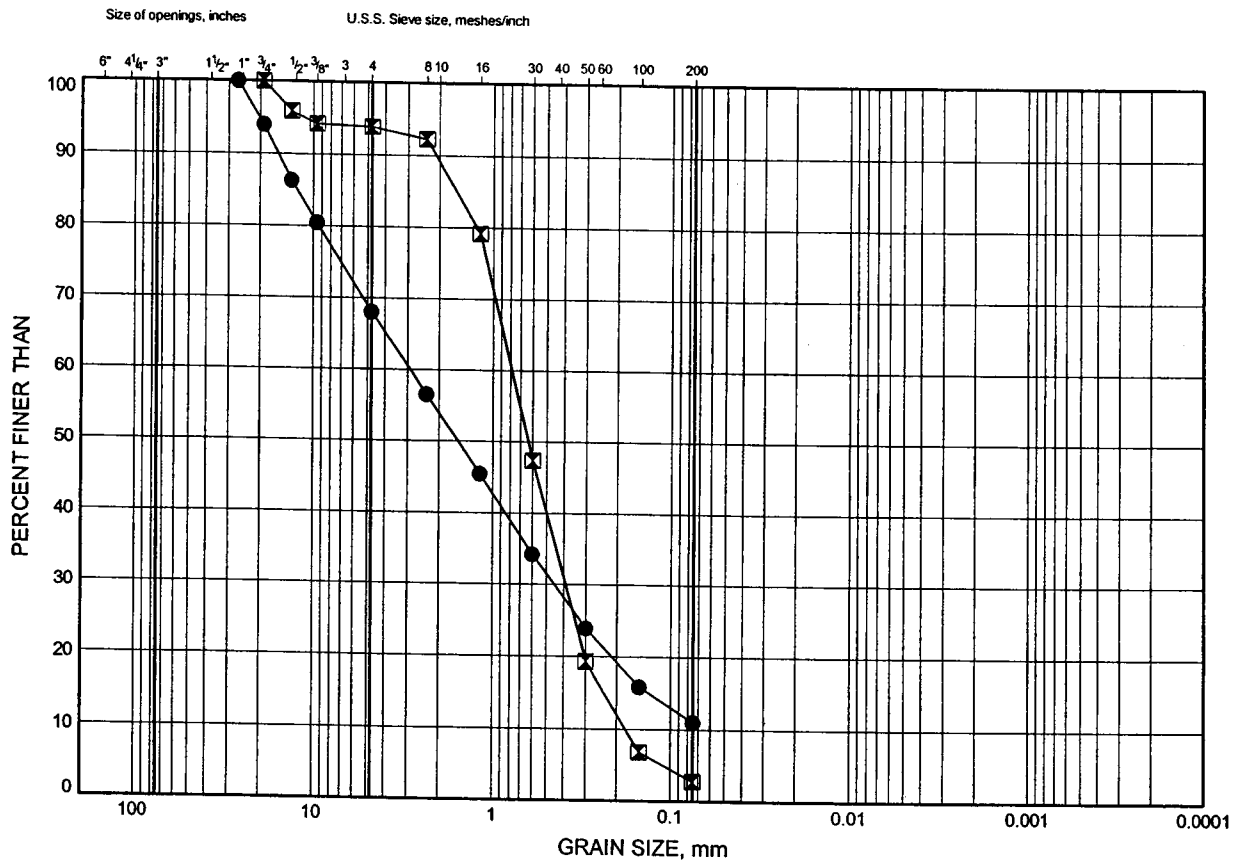
Appendix B

Laboratory Test Results

HWY 71 GRAIN SIZE DISTRIBUTION

FIGURE B1

FILL



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

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	ST-1	1.07	362.49
⊠	ST-3	1.83	361.29

Date March 2007
Project 121-97-00

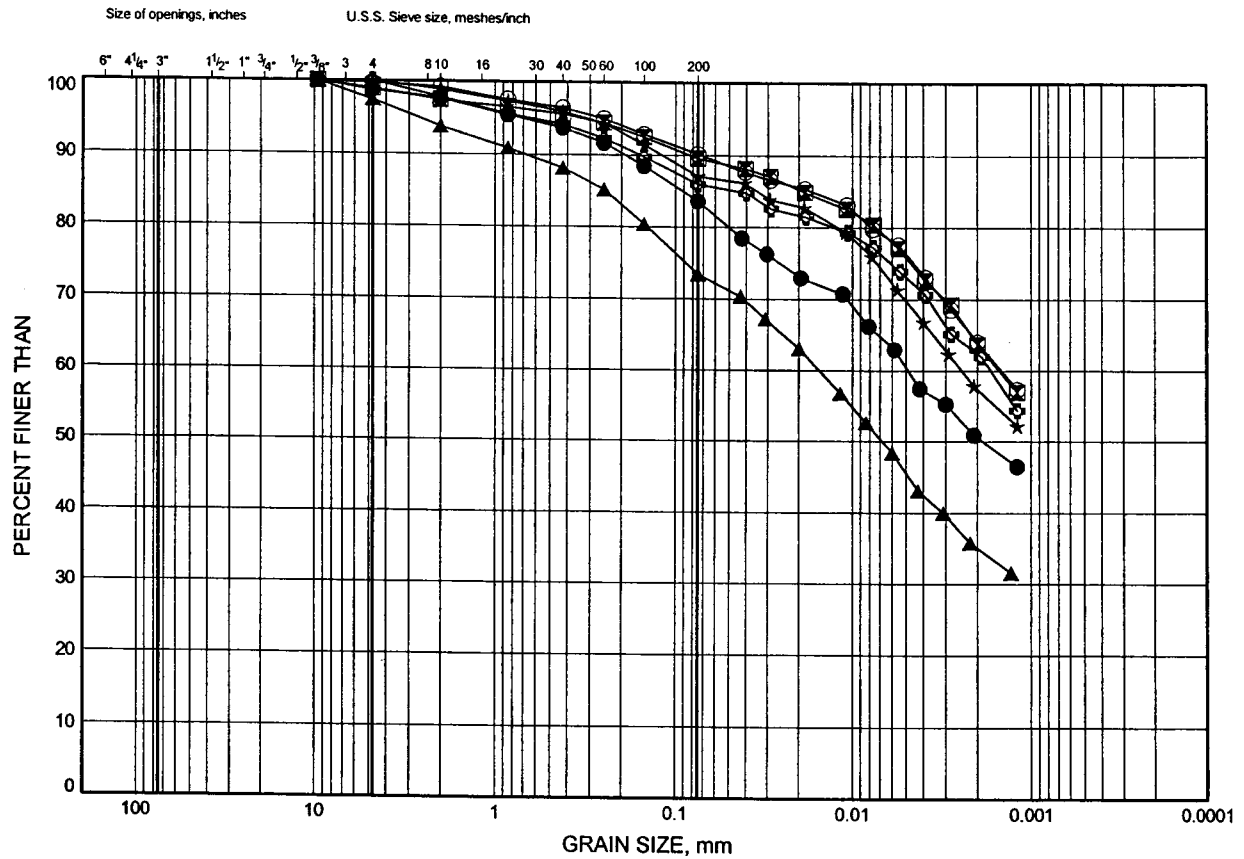


Prep'd MFA
Chkd. TJH

HWY 71 GRAIN SIZE DISTRIBUTION

FIGURE B2

SILTY CLAY



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

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	ST-1	4.11	359.44
⊠	ST-1	9.37	354.19
▲	ST-2	4.72	358.66
★	ST-2	7.85	355.53
⊙	ST-2	12.42	350.96
⊗	ST-2	21.49	341.89

Date March 2007
Project 121-97-00

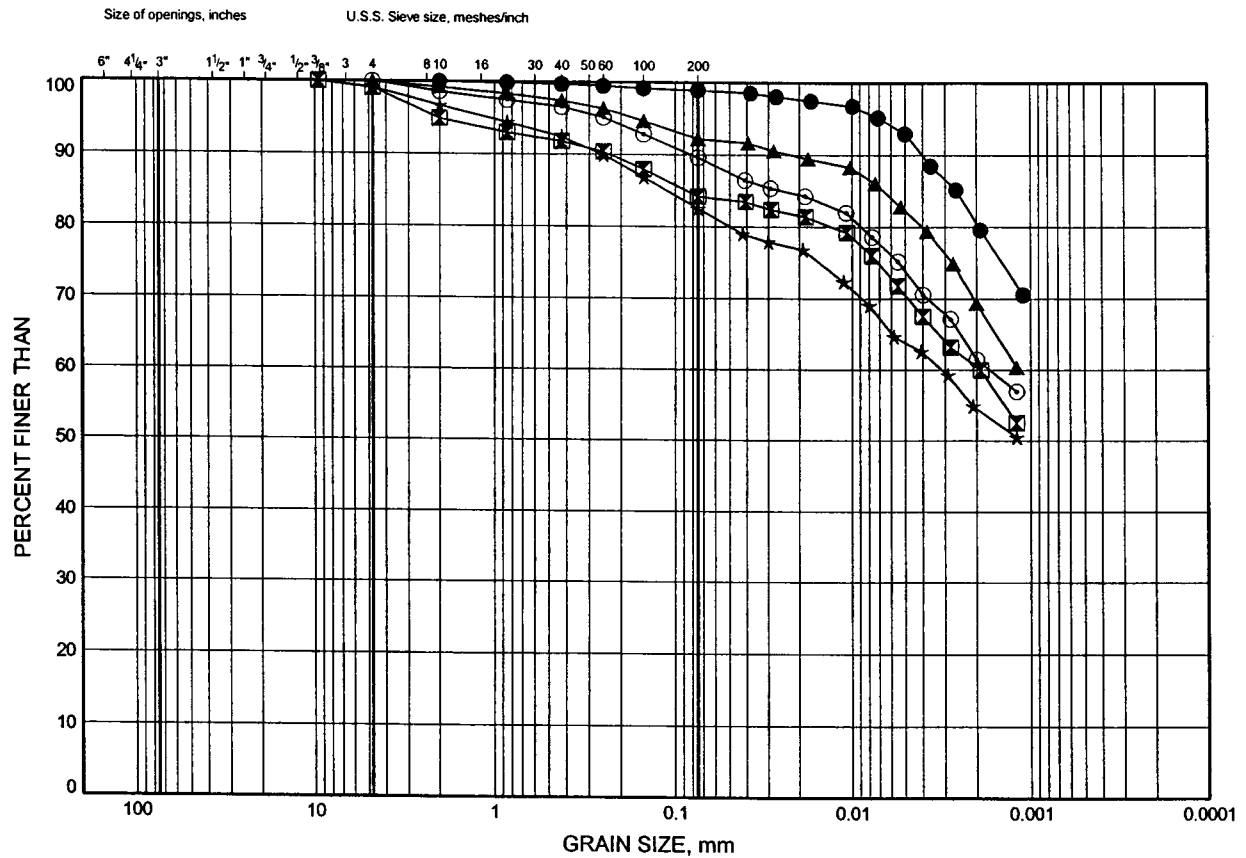


Prep'd MFA
Chkd. TJH

HWY 71 GRAIN SIZE DISTRIBUTION

FIGURE B3

SILTY CLAY



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

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	ST-3	4.11	359.01
☒	ST-3	10.90	352.22
▲	ST-3	16.99	346.13
★	ST-4	4.11	358.86
⊙	ST-4	9.37	353.60

Date March 2007
Project 121-97-00

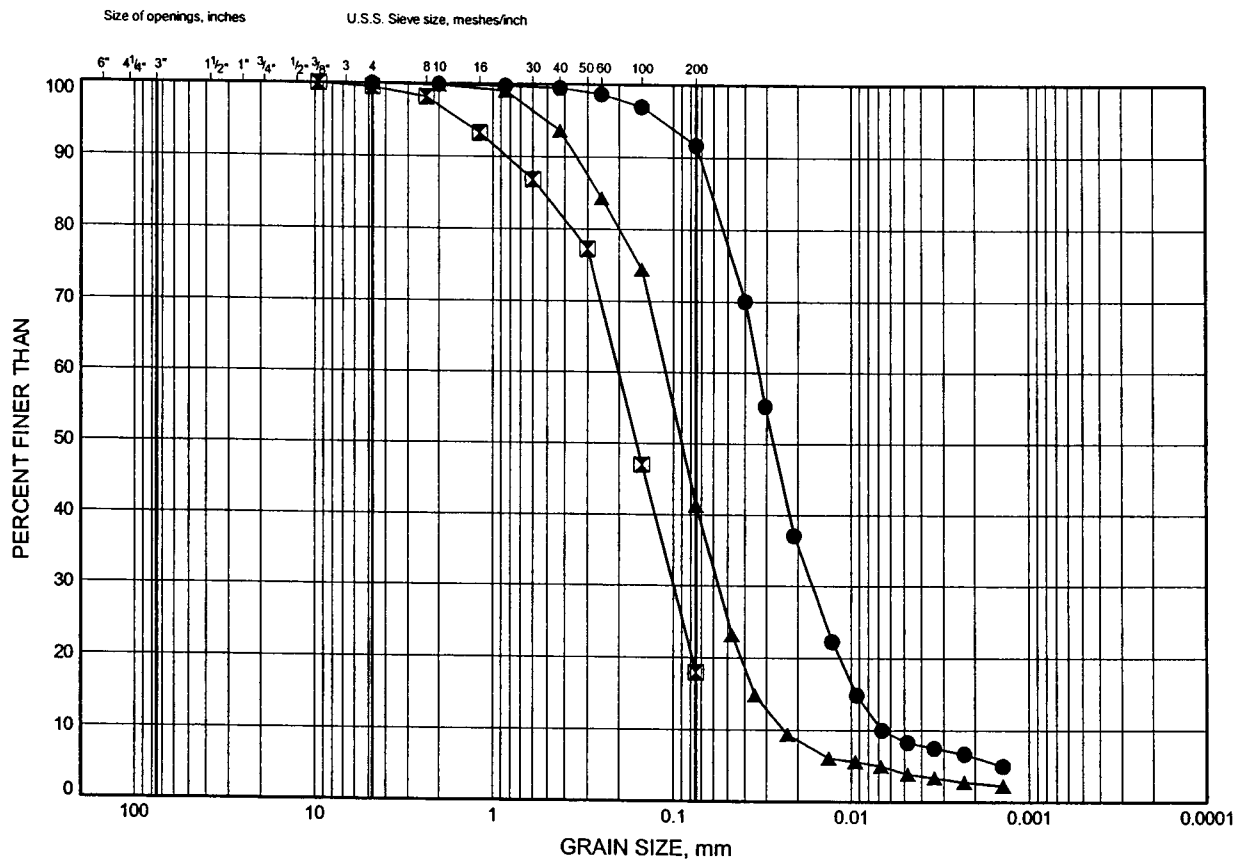


Prep'd MFA
Chkd. TJH

HWY 71 GRAIN SIZE DISTRIBUTION

FIGURE B4

SAND AND SILT



SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	ST-2	24.61	338.77
☒	ST-2	30.63	332.75
▲	ST-3	27.66	335.46

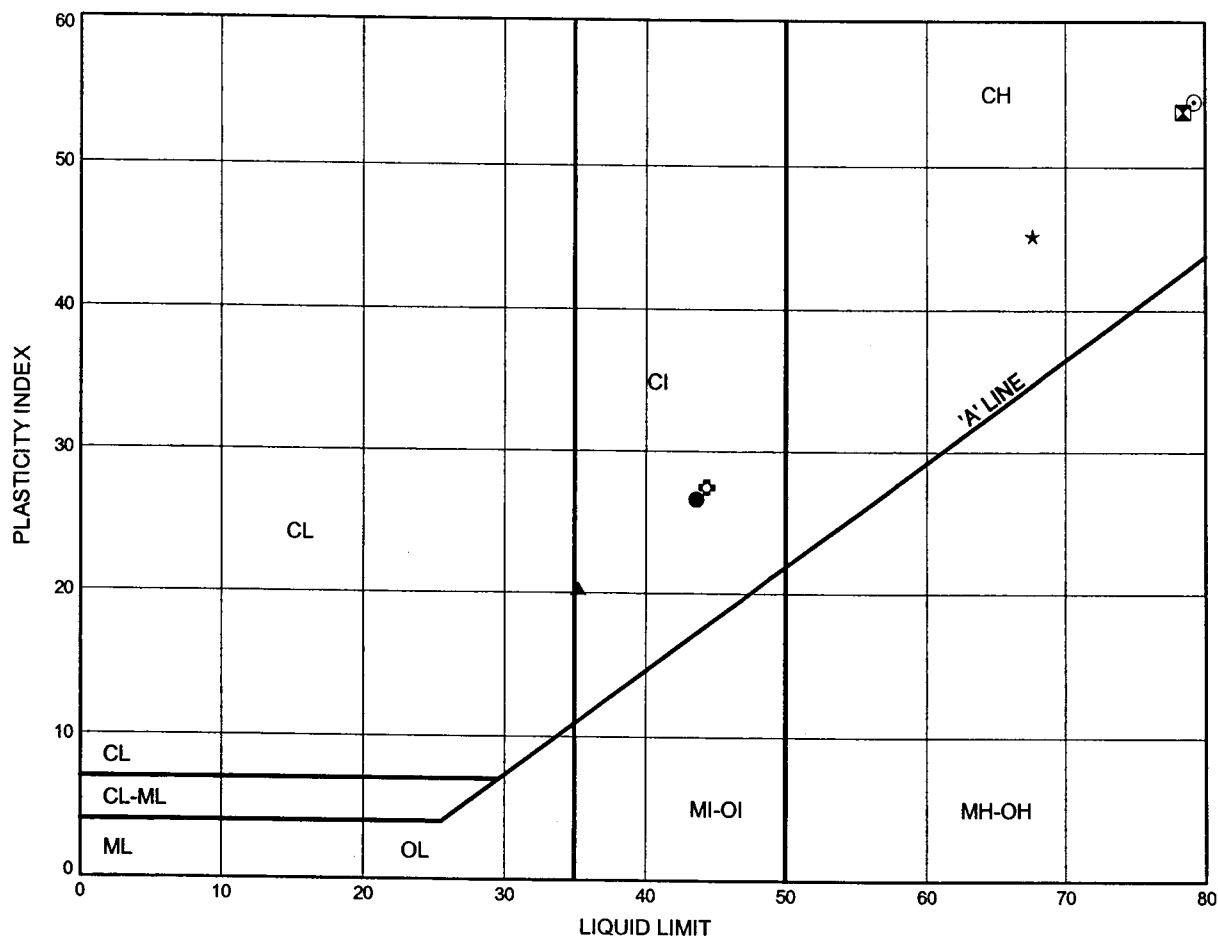
Date March 2007
Project 121-97-00



Prep'd MFA
Chkd. TJH

HWY 71 ATTERBERG LIMITS TEST RESULTS

FIGURE B5



SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	ST-1	4.11	359.44
⊠	ST-1	9.37	354.19
▲	ST-2	4.80	358.58
★	ST-2	7.85	355.53
⊙	ST-2	12.42	350.96
⊗	ST-2	21.49	341.89

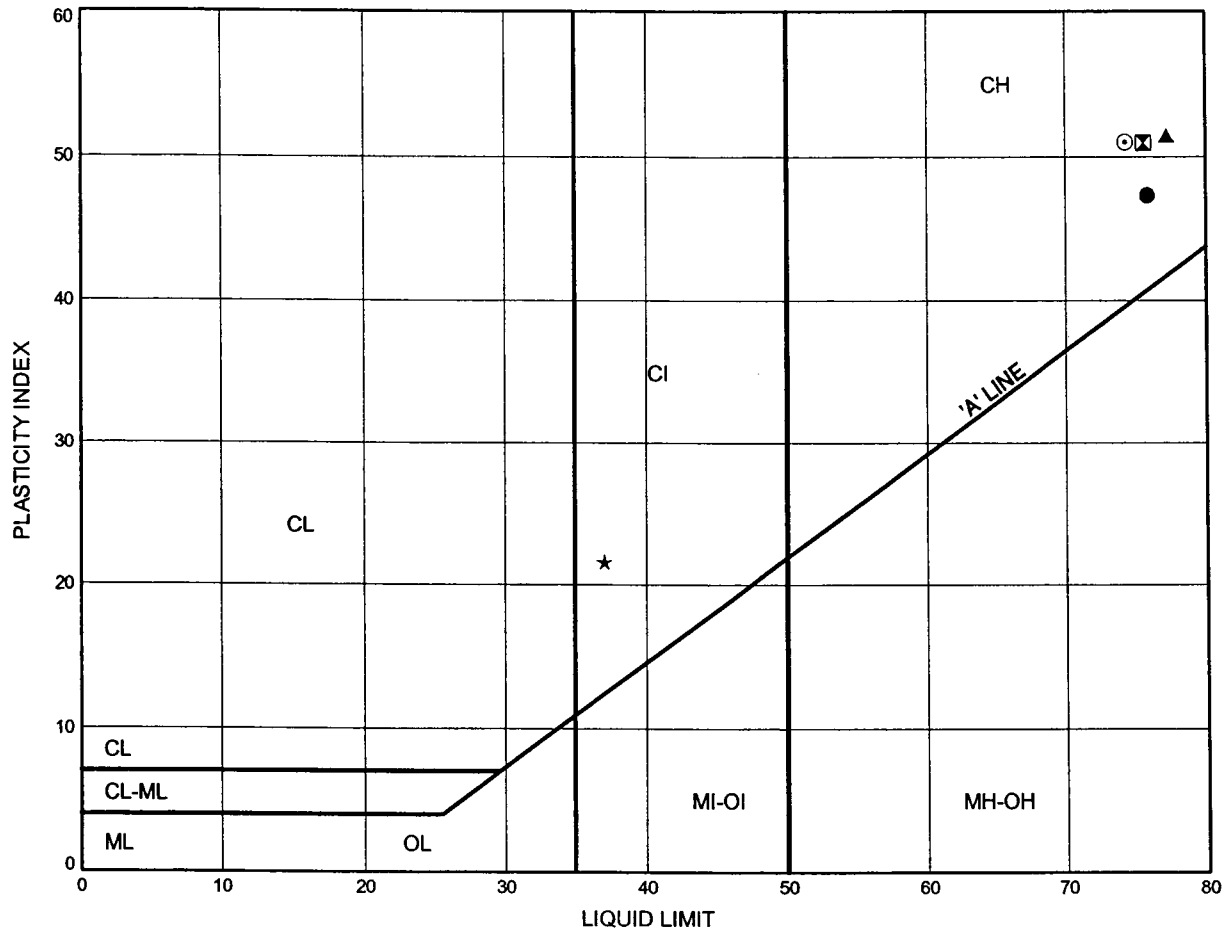
Date March 2007
 Project 121-97-00



Prep'd MFA
 Chkd. TJH

HWY 71 ATTERBERG LIMITS TEST RESULTS

FIGURE B6



SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	ST-3	4.11	359.01
⊠	ST-3	10.90	352.22
▲	ST-3	16.99	346.13
★	ST-4	4.11	358.86
⊙	ST-4	9.37	353.60

Date March 2007
 Project 121-97-00



Prep'd MFA
 Chkd. TJH

Appendix C

Foundation Comparison



COMPARISON OF FOUNDATION ALTERNATIVES FOR EACH FOUNDATION ELEMENT

Foundation Element	H-Piles (End Bearing)	H-Piles (Friction)	Caissons	Footings on Native Soil	Footings on Engineered Fill
All	<p>Advantages:</p> <ul style="list-style-type: none">i. High geotechnical resistance available by driving piles to dense soil or bedrock.ii. Will allow for the construction of an integral abutment structure.iii. Independent of groundwater conditions.iv. Comparatively short abutment stem <p>Disadvantages:</p> <ul style="list-style-type: none">i. Higher unit cost compared to footings.ii. Need a sand pad covering the piles to allow drainage of possible artesian seepage <p>RECOMMENDED</p>	<p>Advantages:</p> <ul style="list-style-type: none">i. Moderate geotechnical resistance available by driving piles in the clay layer.ii. Will allow for the construction of an integral abutment structure.iii. Independent of groundwater conditions.iv. Comparatively short abutment stem <p>Disadvantages:</p> <ul style="list-style-type: none">i. Higher unit cost compared to footings.ii. Lower resistance than piles driven to end bearing. <p>RECOMMENDED AS ALTERNATIVE SYSTEM</p>	<p>Advantages:</p> <ul style="list-style-type: none">i. High bearing resistances available on bedrock. <p>Disadvantages</p> <ul style="list-style-type: none">i. Difficulties in obtaining a seal below the liner to pour concrete in dry conditions.ii. Higher cost than all other systems. <p>NOT RECOMMEND</p>	<p>Advantages:</p> <ul style="list-style-type: none">i. Lower unit cost compared to pile foundations. <p>Disadvantages:</p> <ul style="list-style-type: none">i. Low bearing resistance at this siteii. An integral abutment design is not an available optioniii. Comparatively longer abutment stem.iv. Possible dewatering requirements <p>NOT RECOMMENDED</p>	<p>Advantages</p> <ul style="list-style-type: none">i. Lower unit cost compared to pilesii. Shorter abutment stem possible. <p>Disadvantages:</p> <ul style="list-style-type: none">i. An integral abutment design is not an available optionii. Cost of constructing engineered filliii. Possible dewatering requirements. <p>NOT RCOMMENDED</p>

Appendix D

Special Provisions

The following Special Provisions are referenced in this report:

110F13

105S10

105S19

Amendment to OPSS 206, December 1993

902S01

903S01

Suggested text for a NSSP on Pile Installation should contain the following:

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

- *The pile tips must be protected through the use of rock points*
- *The cobbles and boulders may impede the driving of the piles resulting in more arduous driving*
- *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"*

Suggested text for NSSP on Sand Pad Covering the Piles

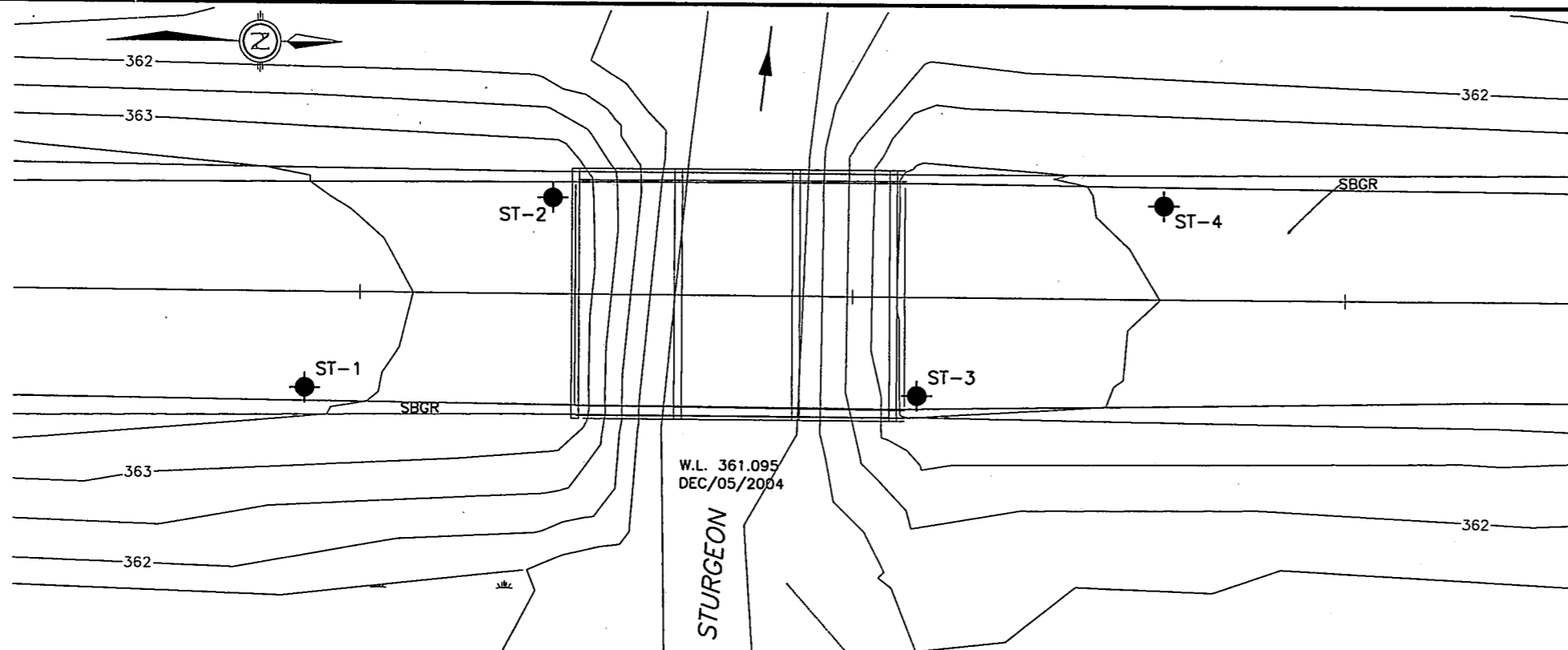
- *Work to be completed prior to driving any piles.*
- *Excavate for a width of 2.0 m, centered on the line of the bearing piles and extending 1.0 m beyond the end piles. The base of the excavation must lie at least 1.0 m below the level of the subdrain behind the abutment but not below the level of the creek.*
- *Maintain the base of the excavation in an unwatered condition and free of disturbed soil.*
- *Backfill the excavation to the underside of the subdrain using material meeting OPSS 1002 Table 1 "Gradation Requirements for Fine Aggregates". Place the fill in accordance with 105S01.*

Payment

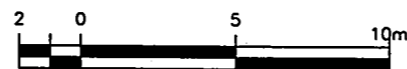
Payment at the contract price for the above tender item shall include full compensation for labour, equipment and materials to do the work.

Appendix E

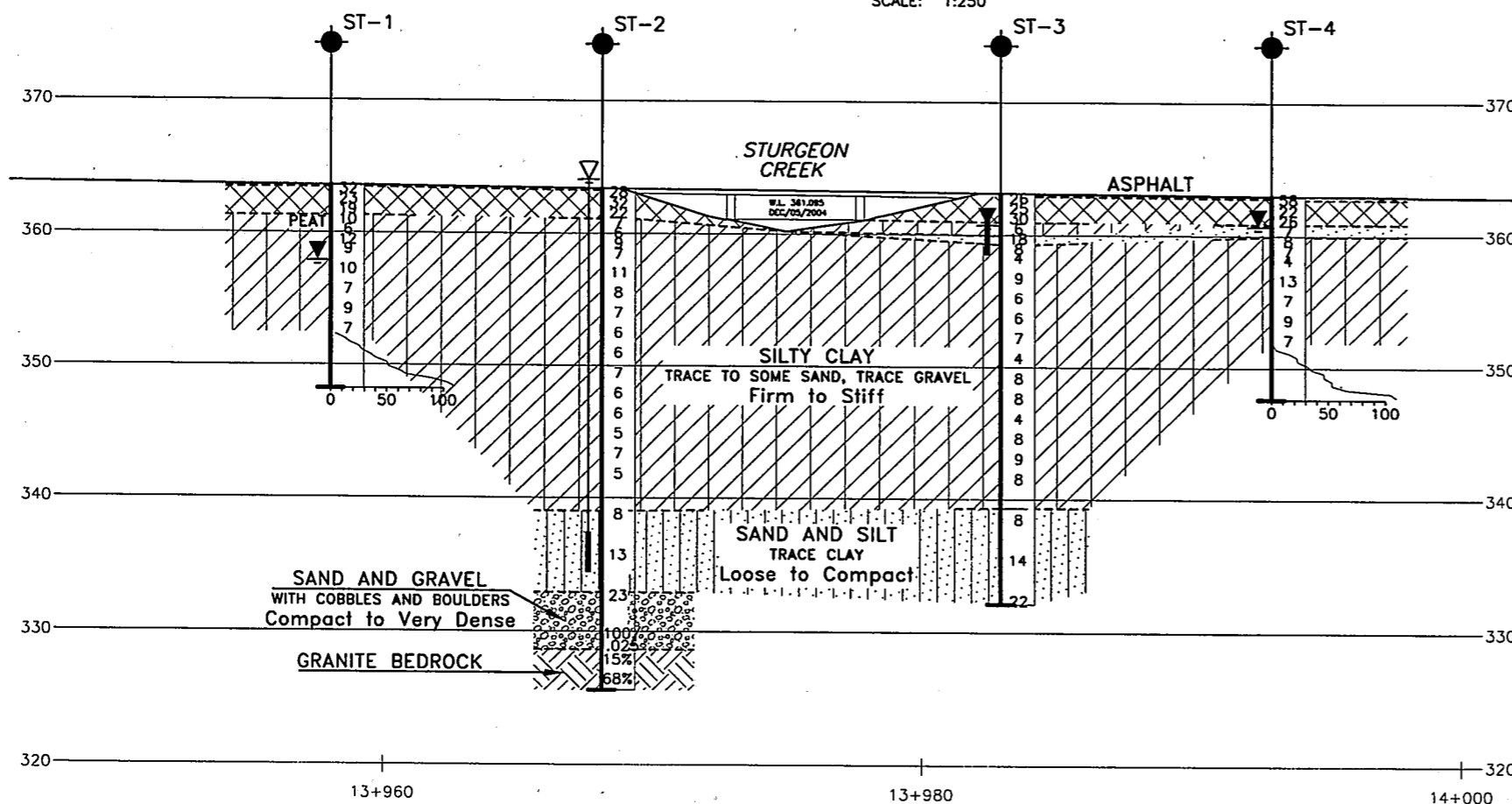
Drawings



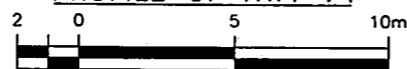
PLAN



SCALE: 1:250



PROFILE OF HWY 71



HOR 1:250

VERT 1:125

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

CONT No
 GWP No. 121-97-00

STURGEON CREEK BRIDGE

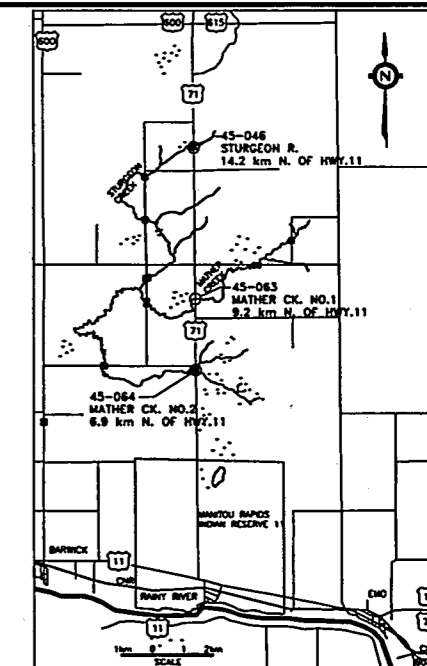
BOREHOLE LOCATIONS AND
 SOIL STRATA



SHEET
 21

cookengineering
 Thunder Bay, Ontario

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KEYPLAN

LEGEND

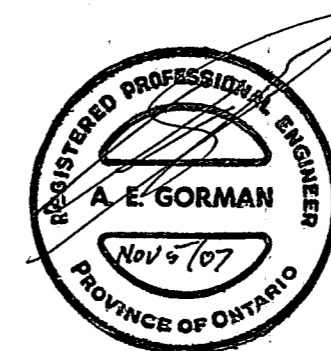
●	Borehole
⊙	Borehole and Cone
N	Blows /0.3m (Std Pen Test, 475J/blow)
CONE	Blows /0.3m (60° Cone, 475J/blow)
PH	Pressure, Hydraulic
W	Water Level
↑	Head Artesian Water
⊥	Piezometer
90%	Rock Quality Designation (RQD)
A/R	Auger Refusal

NO	ELEVATION	NORTHING	EASTING
ST-1	363.6	5 404 220.68	237 361.87
ST-2	363.4	5 404 230.73	237 354.08
ST-3	363.1	5 404 245.56	237 362.08
ST-4	363.0	5 404 255.61	237 354.29

NOTES

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

GEOCRES No. 520-19



DRAWING NOT TO BE SCALED
 100 mm ON ORIGINAL DRAWING

REVISIONS	DATE	BY	DESCRIPTION
DESIGN	CHK TJH	CODE	LOAD
DRAWN	MFA	CHK AEG	SITE 45-046
			STRUCT
			DWG 2
			DATE JAN 2007

Appendix F

Site Photographs

Hwy 71 Sturgeon Creek Bridge Rehabilitation



Plate 1 Existing Bridge on Hwy 71 crossing Sturgeon Creek. Note sign on right of plate is in incorrect.



Plate 2 Existing Sturgeon Creek timber piles.

