

GEOCRES No:
52C-20

**FOUNDATION INVESTIGATION AND DESIGN REPORT
HWY 71 MATHER CREEK # 2 BRIDGE REHABILITATION
G.W.P. 121-97-00, SITE: 45-064**

Report to

Cook Engineering

GEOCRES Number 52C-20

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PART 1: FACTUAL INFORMATION

GEOCRES Number 52C-20

1 INTRODUCTION

This report presents the factual findings obtained from a foundation investigation conducted at the site of Mather Creek #2 crossing, located on Highway 71, 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, 6.9 km north of the junction between Highways 11 and 71. Highway 71 crosses Mather Creek #2 on a single span timber pile supported bridge. The existing structure carries both the north and southbound lanes. Mather Creek #2 is approximately 6 m in width with a water level elevation of approximately 350.2 m (December 3, 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, silty clay and sand.

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

3 SITE INVESTIGATION AND FIELD TESTING

Four boreholes numbered M2-1 to M2-4 were drilled and sampled to depths ranging from 11.1 m to 26.8 m. Approximate locations of the boreholes 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.

One borehole was drilled in each approach fill (M2-1 & M2-4), sampling was undertaken to depths of 11.1 m and Dynamic Cone Penetration Testing (DCPT) was undertaken from a depth of 11.1 m to refusal.

One borehole adjacent to each abutment foundation was advanced with NQ size diamond coring upon auger refusal to prove bedrock. Between 3.0 m and 3.1 m of rock core was recovered at each location.

Groundwater conditions in the open boreholes were observed throughout the drilling operations. A standpipe piezometer was installed in the north abutment (M2-3) 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 piezometer was 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
M2-1	No Installation	Bentonite grout from 11.1 m to 0.3 m, hole plug from 0.9m to 0.3 m, concrete from 0.30m to 0.15m and asphalt from 0.15 to ground level.
M2-2	No Installation	Bentonite grout 24.7 m to 1.2 m, hole plug from 1.2 m to 0.6 m, concrete from 0.6 m to 0.15m and asphalt from 0.15 to ground level.
M2-3	23.8 / 328.3	Piezometer - 19 mm PVC pipe with 1.5 m slotted screen installed at 23.8 m to 22.3 m with sand filter from 23.8 m to 22.0 m, hole plug from 22.0 m to 20.7 m, bentonite grout from 20.7 m to 1.2 m, hole plug from 1.2 m to 0.3 m and concrete seal from 0.3 m to 0.08 m and a flush mounted protective cover. Artesian pressure observed above bedrock. Artesian conditions through the piezometer pipe persisted for 4 days post drilling until the installation was grouted.
M2-4	No Installation	Bentonite grout 16.9 m to 0.9 m, hole plug from 0.9 m to 0.3 m, concrete from 0.3 m to 0.15 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 23.8 m of overburden, overlying Pre-Cambrian igneous bedrock. The overburden deposits generally consist of asphalt, granular fill, peat, silty clay and sands.

5.1 Asphalt

Thicknesses of 125 mm to 150 mm of asphalt were encountered at the borehole locations. Asphalt thickness increases toward the bridge on both approaches.

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 and boulders. The underside of the fill layer was recorded at elevations between 348.3 m and 349.9 m. The thickness of the granular fill ranges from 2.2 m to 3.8 m.

'N' values ranging from 16 to over 100, derived from Standard Penetration Tests conducted in the sand and gravel fill, indicate a compact to very dense relative density. The high blow counts may indicate tests on cobbles or boulders. The sand fill exhibits N values ranging from 5 and 28, indicating loose to compact relative density.

Moisture content of samples from the granular fill ranged from approximately 8% to 18%. Gradation of the granular fill is presented in Figure B1 in Appendix B.

5.3 Peat

A layer of wet fibrous peat was encountered under the granular fill in BH M2-2 and M2-4. The peat thicknesses ranged from 0.3 m to 0.4 m and the base of the peat is at elevation 348.0 to 349.3. SPT blow counts in the peat range from 2 to 4. Moisture content of the peat ranges from 100 % to 112 %.

5.4 Silty Clay

Dark brown to grey silty clay was encountered underlying the granular fill or peat in all boreholes. The soil is described as clay, silty, trace to some sand, trace gravel. Occasional layers of cobbles were encountered in this stratum (M2-3).

This deposit extends to depths ranging from 17.8 m to 23.5 m. A thickness of 21.3 m was recorded at the north abutment (M2-3). The elevation of the base of this layer ranges from 334.3 m to 328.6 m. Samples of this deposit were subjected to grain size distribution and Atterberg Limit tests; the results are presented in Figures B2 to B5 in Appendix B.

SPT 'N' values in the silty clay range from 4 to 13 blows per 0.3 m penetration, indicating the clay is firm to stiff.

Ten gradation tests were undertaken on this material the results of which are summarised in Table 5.1.

Table 5.1: Summary of Grain Size Distribution Tests undertaken on silty clay

	Sand (%)	Silt (%)	Clay (%)
Range	9 to 34	28 to 38	27 to 62

The Natural Moisture Content of this material ranged from 18 % to 68 %. This material has a Liquid Limit ranging from 32 % to 78 % and a Plasticity Index of 23 to 52, indicative of inorganic, low to high plasticity clay.

5.5 Sand

A layer of sand with some silt and trace gravel and occasional cobbles and boulders was encountered at the south and north abutments (M2-2 & M2-3) at depths of 17.8 m and 23.5 m respectively. The thickness of this stratum ranged from 0.3 m to 3.8 m. The base of the layer is at elevation 328.3 m to 330.5 m.

One SPT N value of 10 was recorded in this stratum indicating a loose to compact relative density. The moisture content of one sample recovered is 32 %.

The sand stratum appears to be a water-bearing layer within the predominantly silty clay overburden mass.

5.6 Bedrock

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

The bedrock is described as fresh to slightly weathered, grey, crystalline, very strong to extremely strong granite.

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)
M2 - 1	N	-	-	-	-	-
M2 - 2	Y	21.6	330.5	3.1	Poor to Excellent 40-96 %	303 245
M2 - 3	Y	23.8	328.3	3.0	Excellent 100 %	274 193
M2 - 4	N	-	-	-	-	-

The Fracture Index (FI) of the rock, expressed as fractures per 0.3 m of core, varied from 1 to 2 in the first core run to greater than 10 in the second core run in borehole M2-2. Lower fracture indices of between 1 and 3 were encountered in borehole M2-3. 100% total core recovery (TCR) was achieved in both coreholes. The rock mass classification based on RQD values ranged from 40% to 100% indicating some poor but overall excellent rock quality.

Based on laboratory Point Load testing conducted on 4 rock core samples, unconfined compressive strength values were estimated to be between 193 MPa and 303 MPa indicating very strong to extremely strong bedrock.

5.7 Groundwater Conditions

One standpipe piezometer was installed in borehole M2-3 to monitor the groundwater level. The water level readings at the foundation elements are presented in Table 5.3.

Table 5.3: Water Level Measurements

Date	BH M2-1		BH M2-2		BH M2-3		BH M2-4	
	Depth (m)	Elev. (m)	Depth (m)	Elev. (m)	Depth (m)	Elev. (m)	Depth (m)	Elev. (m)
June 11, 2006	2.44*	349.8	-	-	-	-	-	-
June 12, 2006	-	-	-	-	+1.05	353.2	2.74*	349.3
June 13, 2006	-	-	1.22*	351.0	-	-	-	-

* Water levels in open boreholes

+ Denotes artesian head of water above ground surface

Based on these observations, local artesian groundwater conditions exist. The source of the artesian groundwater in borehole M2-3 is a sand and gravel layer encountered at elevation 328.6 m. The standpipe installation at M2-3 was grouted at the end of the fieldwork period. 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.

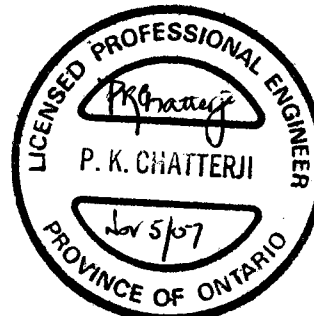
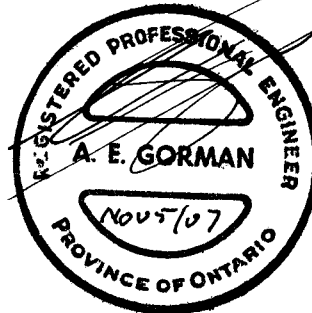
Mr. Alastair E. Gorman, P.Eng. and Mr Tony Harte prepared the report.

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

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PART 2: ENGINEERING DISCUSSION AND RECOMMENDATIONS

GEOCRES Number 52C-20

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 is supported on timber piles. A single span replacement structure is proposed along the existing alignment. At both north and south abutments the finished grade is at about elevation 352.4 m and 352.3 m respectively, indicating a small 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 near 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 this report.

Initial consideration was given to the following foundation types:

- Spread footings (on native soil or 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.

Table 8.1 – Summarized Soil Conditions

Borehole Location	Geodetic Elevation	Stratigraphy	Groundwater Elevation (m)
M2-1 South Approach	352.2 to 349.9	Sand & Gravel Fill, Cobbles	349.76
	349.9 to 341.0	Silty Clay	
	341.0 to 336.5	Unknown overburden. DCPT terminated @ 336.5m	
M2-2 South Abutment	352.2 to 348.3	Sand & Gravel Fill, Cobbles, Boulder	350.98
	348.3 to 348.0	Peat	
	348.0 to 334.3	Silty Clay	
	334.3 to 330.5	Sand, boulder	
	330.5 to 327.5	Granite bedrock	
M2-3 North Abutment	352.1 to 349.8	Sand & Gravel Fill, Cobbles	Groundwater level at 353.2 m Artesian groundwater from sand and gravel layer at 328.6 m
	349.8 to 328.6	Silty Clay	
	328.6 to 328.3	Sand and Gravel	
	328.3 to 325.2	Granite bedrock	
M2-4 North Approach	352.0 to 349.7	Sand & Gravel Fill	349.26
	349.7 to 349.3	Peat	
	349.3 to 340.9	Silty Clay	
	340.9 to 335.1	Unknown overburden. DCPT terminated @ 335.1m	

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 risk of unacceptably large settlements under footings
- The low geotechnical resistance available in the native soil
- There is a 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 socketed into bedrock at or below Elevation 330.5 at the south abutment and Elevation 328.3 at the north abutment could be considered. However, there could be difficulties in achieving a good seal in the moderate artesian conditions encountered in the sand layer, just above the bedrock. Caissons are not considered to be an economical alternative at this site and are not recommended.

8.4 Driven Steel H Piles

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

The stratigraphy at the abutments consists of mixed firm to stiff cohesive and loose cohesionless soils overlying bedrock. Cobbles and boulders were encountered in the roadway fill as well as in the overburden soils. With this stratigraphy, it is expected that steel H-piles can be driven to bedrock at the elevations shown in Table 8.2.

Table 8.2 – Estimated Pile Tip Elevation

Location	Borehole No.	Depth to Bedrock (m)	Top of Bedrock Elevation
South Abutment	M2-2	21.6	330.5
North Abutment	M2-3	23.8	328.3

8.4.1 Axial Resistance

The factored, vertical, concentric, geotechnical resistances at ULS for two pile sections, when driven to bedrock, are as follows:

- 2000 kN for HP 310 x 110
- 1400 kN for HP 310 x 79

The SLS condition will not govern for piles founded on bedrock.

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

The possible presence of cobbles and boulders above bedrock should not be overlooked. 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.

8.4.4 Pile Driving

The appropriate note for the foundation drawing is Note 5, i.e. "Piles to be driven to bedrock".

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 332.2) 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 8.3)

γ = unit weight (Table 8.3)

K_p = passive earth pressure coefficient (Table 8.3)

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.

Table 8.3 – Recommended Soil Parameters

Location	Elevation	n_h (kN/m^3)	K_p	Unit Weight* (kN/m^3)	Soil Conditions
South Abutment	352.2 to 348.3	10 000	3.0	22	Compacted fill
	348.3 to 334.3	3000	2.8	9	Silty Clay firm to stiff
	334.3 to 330.5	8 000	3.0	10	Sand Compact
North Abutment	352.1 to 349.8	10 000	3.0	22	Compacted fill
	349.8 to 328.3 (bedrock)	3 000	2.8	9	Silty Clay firm to stiff

*Buoyant unit weight below the water table.

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/m^3), 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 rock, 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.

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 the foundations should be 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 GROUNDWATER AND FLOOD CONTROL

At the time of investigation, local artesian groundwater was encountered at the north abutment borehole (M2-3). The source of the artesian water is believed to be water in the sand and gravel strata directly above bedrock. The general water level at the site is also high and within 1 to 3 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 Mather Creek water level rising rapidly due to flood conditions. The groundwater and surface (flood) water must be controlled during construction to maintain a stable excavation and to allow concrete to be placed in an unwatered 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 sheeted excavation 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 SSP 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)

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.

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 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:

- Difficulty in dewatering of temporary excavations for foundation construction, if any are required
- 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.

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.
Tony Harte, M.Sc. FGS



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

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


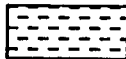
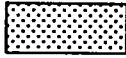


 Water Level
 Shear Strength Determination by Pocket Penetrometer

- (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 M2-1

1 OF 2

METRIC

G.W.P. 121-97-00 LOCATION Mather Creek #2 N 5 396 742.65 E 237 335.34 ORIGINATED BY GA
 HWY 71 BOREHOLE TYPE NW Casing/Dynamic Cone Penetration Test COMPILED BY WM
 DATUM Geodetic DATE 11.06.06 - 11.06.06 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 (%)					
								20 40 60 80 100										
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE										
						20 40 60 80 100					Wp — W — WL							
352.2																		
0.0	ASPHALT: (125 mm)																	
0.1	SAND and GRAVEL, trace silt, occasional cobbles Dense to Compact Brown Moist (FILL)						352											
			1	SS	32													
			2	SS	22													
350.6							351											28 68 4 (SI+CL)
1.5	SAND, fine to medium grained, trace silt Loose Brown Wet (FILL)																	
			3	SS	5													
349.9																		
2.3	Silty CLAY, trace sand, trace rootlets and organics Soft Dark Brown						350											
			4	SS	3													
349.1																		
3.0	Silty CLAY, trace to some sand, trace gravel Firm to Stiff Grey (CH)						349											
			5	SS	4													
			6	SS	5													
			7	SS	6													0 14 31 55
							347											
			8	SS	4													
							346											
							345											
			9	SS	8													
							344											
			10	SS	7		343											0 15 28 57

Continued Next Page

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

RECORD OF BOREHOLE No M2-1

2 OF 2

METRIC

G.W.P. 121-97-00 LOCATION Mather Creek #2 N 5 396 742.65 E 237 335.34 ORIGINATED BY GA
 HWY 71 BOREHOLE TYPE NW Casing/Dynamic Cone Penetration Test COMPILED BY WM
 DATUM Geodetic DATE 11.06.06 - 11.06.06 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			SHEAR STRENGTH kPa	WATER CONTENT (%)					
	Continued From Previous Page													
341.0	Silty CLAY, trace to some sand, trace gravel Firm to Stiff Grey (CH)		11	SS	11									
11.1	END OF SOIL SAMPLING AT 11.13 m. Dynamic Cone Penetration Test (DCPT) started at 11.13 m.													
336.5														
15.7	END OF DCPT AT 15.70 m. BOREHOLE OPEN TO 15.70m AND WATER LEVEL AT 2.44m UPON COMPLETION. BOREHOLE GROUTED WITH BENTONITE TO 0.3m AND PATCHED WITH CONCRETE AND ASPHALT TO SURFACE. WATER LEVEL READINGS: DATE DEPTH(m) ELEV.(m) 11/6/06 2.44 349.76													

RECORD OF BOREHOLE No M2-2

1 OF 3

METRIC

G.W.P. 121-97-00 LOCATION Mather Creek #2 N 5 396 752.75 E 237 328.13 ORIGINATED BY GA
 HWY 71 BOREHOLE TYPE NW Casing/NQ Coring COMPILED BY WM
 DATUM Geodetic DATE 2006-06-13 - 2006-06-13 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				
352.2							20 40 60 80 100	PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT		
0.0	ASPHALT: (150 mm)							W _P	W	W _L		
0.2	SAND, trace gravel, trace silt Dense to Compact Brown Wet (FILL)		1	SS	37			○ UNCONFINED	+ FIELD VANE			22 74 4 (SI+CL)
			2	SS	29			● QUICK TRIAXIAL	x LAB VANE			
350.5	Cobble: (150mm)											
1.7	SAND and GRAVEL, trace silt Brown (FILL)		3	SS	50/ .000							
349.8	Boulder: (200mm)											
2.3	SAND, fine to medium grained, trace silt Compact Brown Wet (FILL)		4	SS	50/ .000							
			5	SS	20							
348.3												
3.8	PEAT, fibrous, some rootlets											
348.0	Black		6	SS	4							
4.1	Wet											
	Silty CLAY, trace sand to sandy, trace gravel Firm to Stiff Grey (Cl to CH)		7	SS	9							0 25 38 37
			8	SS	8							
			9	SS	10							
			10	SS	9							

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+³ ×³: Numbers refer to
Sensitivity

20
15
10



(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No M2-2

2 OF 3

METRIC

G.W.P. 121-97-00 LOCATION Mather Creek #2 N 5 396 752.75 E 237 328.13 ORIGINATED BY GA
 HWY 71 BOREHOLE TYPE NW Casing/NQ Coring COMPILED BY WM
 DATUM Geodetic DATE 13.06.06 - 13.06.06 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 (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		WATER CONTENT (%)				
								○ UNCONFINED ● QUICK TRIAXIAL	+ FIELD VANE × LAB VANE	W _p	W	W _L		
								20 40 60 80 100						
Continued From Previous Page														
	Silty CLAY, trace to sandy, trace gravel Firm to Stiff Grey (CI to CH)		11	SS	9		342							0 13 27 60
							341							
			12	SS	9		340							
							339							
			13	SS	12		338							
							337							
			14	SS	10		336							
							335							0 9 29 62
			15	SS	10		334							
							333							
334.3														
17.8	SAND, fine grained, trace silt Grey													

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

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METRIC

ORIGINATED BY GA

COMPILED BY WM

CHECKED BY MEF/TH

+ 3, x 3; Numbers refer to Sensitivity

RECORD OF BOREHOLE No M2-3

1 OF 3

METRIC

G.W.P. 121-97-00 LOCATION Mather Creek #2 N 5 396 767.57 E 237 336.13 ORIGINATED BY GA
 HWY 71 BOREHOLE TYPE NW Casing/NQ Coring COMPILED BY WM
 DATUM Geodetic DATE 2006-06-11 - 2006-06-12 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			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE								
352.1							20	40	60	80	100	20	40	60		
0.0	ASPHALT: (150 mm)															
0.2	SAND and GRAVEL, trace silt, with cobbles Dense to Compact Brown Wet (FILL) Cobble		1	SS	36								○			
			2	SS	50/ .150								○			32 64 4 (SI+CL)
			3	SS	16								○			
349.8																
2.2 349.6	Silty CLAY, trace sand, trace wood fibers		4	SS	6										○	
2.4	Silty CLAY, trace sand, trace gravel Firm to Stiff Grey (CL-CH)		5	SS	6								○			
			6	SS	5								○			
			7	SS	6								┌─┐			2 34 37 27
			8	SS	13								○			
			9	SS	10								○			
			10	SS	9								○			

Continued Next Page

+ 3. x 3. Numbers refer to
Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No M2-3

2 OF 3

METRIC

G.W.P. 121-97-00 LOCATION Mather Creek #2 N 5 396 767.57 E 237 336.13 ORIGINATED BY GA
 HWY 71 BOREHOLE TYPE NW Casing/NQ Coring COMPILED BY WM
 DATUM Geodetic DATE 2006-06-11 - 2006-06-12 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			20	40					
	Continued From Previous Page													
	Silty CLAY, trace sand, trace gravel Firm to Stiff Grey (CH)		11	SS	5		342							0 14 30 56
							341							
			12	SS	9		340							
							339							
			13	SS	9		338							
							337							
			14	SS	8		336							
							335							
			15	SS	10		334							
							333							
	Cobble		16	SS	9									

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

RECORD OF BOREHOLE No M2-3

3 OF 3

METRIC

G.W.P. 121-97-00 LOCATION Mather Creek #2 N 5 396 767.57 E 237 336.13 ORIGINATED BY GA
 HWY 71 BOREHOLE TYPE NW Casing/NQ Coring COMPILED BY WM
 DATUM Geodetic DATE 2006-06-11 - 2006-06-12 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	PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	
	Continued From Previous Page							SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE	WATER CONTENT (%) 20 40 60			GR SA SI CL
330.7	Silty CLAY, trace sand, trace gravel Firm to Stiff Grey (TILL)		17	SS	8		332					2 17 27 54
21.3	Silty CLAY, occasional silt seams Varved Firm Grey		18	SS	5		331					
328.6							330					
23.5	SAND and GRAVEL, trace silt Grey						329					
23.8	GRANITE BEDROCK Fresh, coarse grained, very strong		1	RUN			328					RUN 1# TCR=100%, SCR=100%, RQD=100%, UCS=274MPa
			2	RUN			327					RUN 2# TCR=100%, SCR=100%, RQD=100%, UCS=193MPa
325.2							326					
26.8	END OF BOREHOLE AT 26.82 m. Piezometer installation consists of 19 mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen. 15/6/06 - FILLED PIEZO WITH CEMENT/GROUT AND ASPHALT TO STOP ARTESIAN WATER. WATER LEVEL READINGS: DATE DEPTH(m) ELEV.(m) 12/06/06 1.05 above G.S. 353.15											

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METRIC

ORIGINATED BY GA

COMPILED BY WM

CHECKED BY MEF/TH

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RECORD OF BOREHOLE No M2-4

2 OF 2

METRIC

G.W.P. 121-97-00 LOCATION Mather Creek #2 N 5 396 777.63 E 237 328.34 ORIGINATED BY GA
 HWY 71 BOREHOLE TYPE NW Casing/Dynamic Cone Penetration Test COMPILED BY WM
 DATUM Geodetic DATE 12.06.06 - 12.06.06 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	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40					
	Continued From Previous Page													
340.9	Silty CLAY, trace to some sand, trace gravel Firm to Stiff Grey (CH)		11	SS	9									
11.1	END OF SOIL SAMPLING AT 11.13m. Dynamic Cone Penetration Test (DCPT) started at 11.13 m.													
335.1														
16.9	END OF DCPT AT 16.92 m. BOREHOLE OPEN TO 16.92m AND WATER LEVEL AT 2.74m UPON COMPLETION. BOREHOLE GROUTED WITH BENTONITE AND PATCHED WITH CONCRETE AND ASPHALT TO SURFACE. WATER LEVEL READINGS: DATE DEPTH(m) ELEV.(m) 12/6/06 2.74 349.26													

ONTMT4S 0762.GPJ 19/03/07

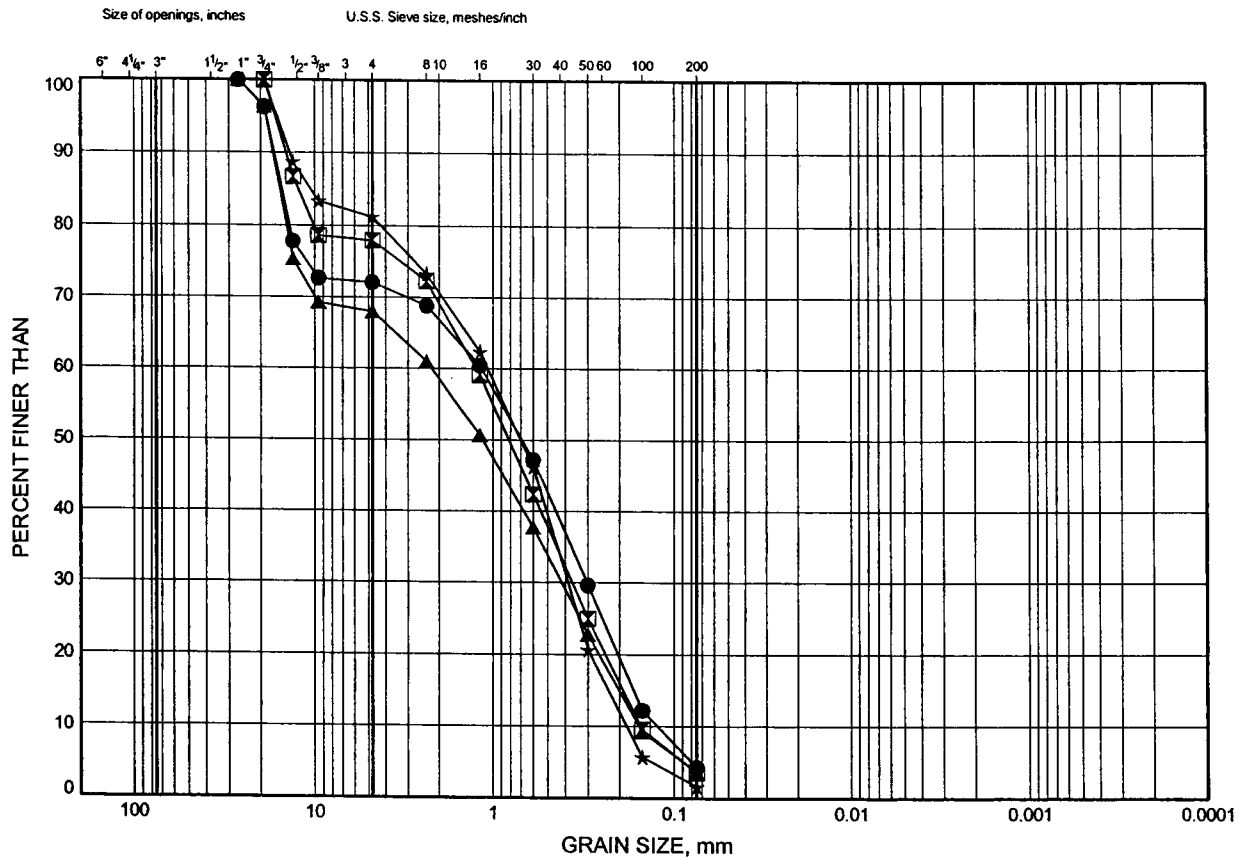
Appendix B

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)
●	M2-1	1.07	351.10
⊠	M2-2	0.46	351.69
▲	M2-3	1.07	350.98
★	M2-4	0.46	351.55

Date March 2007
Project 121-97-00

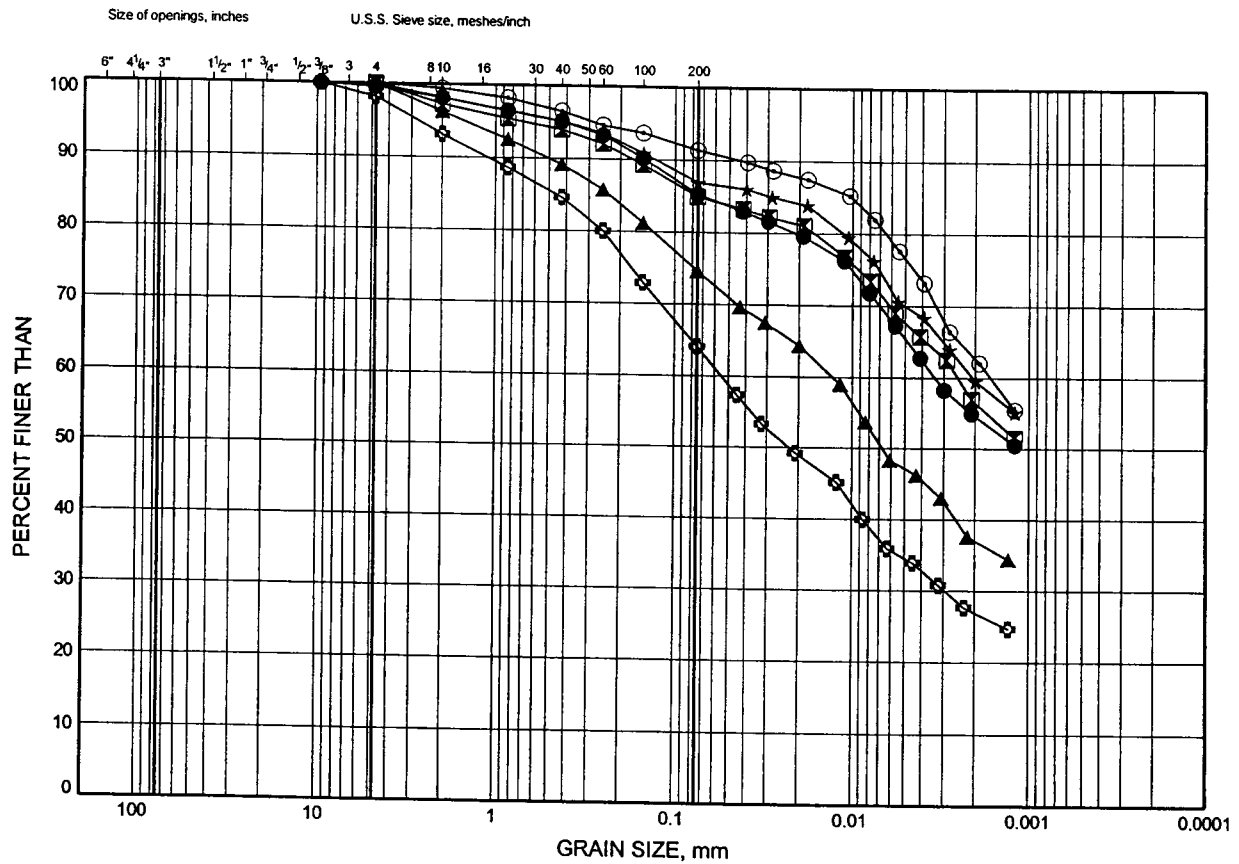


Prep'd MFA
Chkd. TJH

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)
●	M2-1	4.80	347.37
⊠	M2-1	9.37	342.80
▲	M2-2	4.80	347.35
★	M2-2	10.90	341.25
⊙	M2-2	16.99	335.16
⊗	M2-3	4.80	347.25

Date March 2007
Project 121-97-00

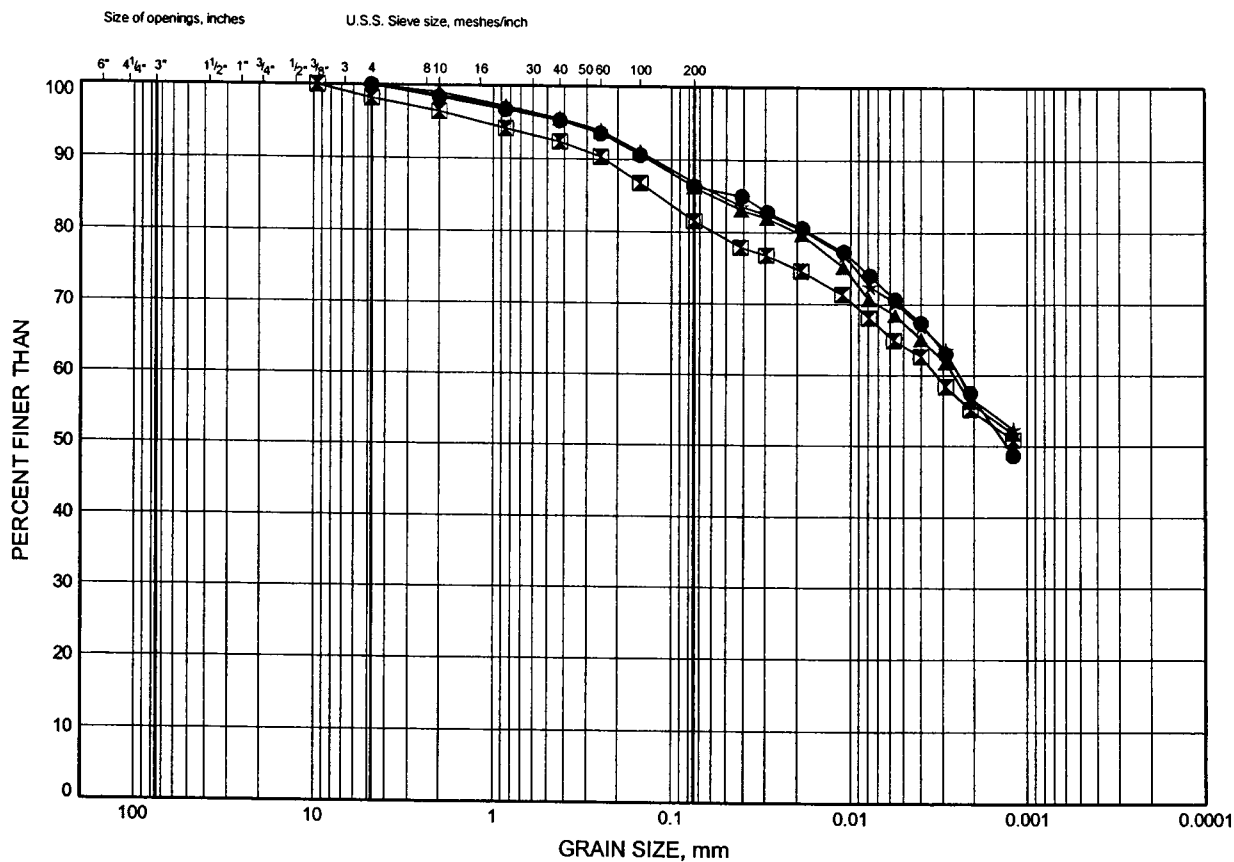


Prep'd MFA
Chkd. TJH

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)
●	M2-3	10.90	341.15
⊠	M2-3	20.04	332.01
▲	M2-4	4.80	347.21
★	M2-4	9.37	342.64

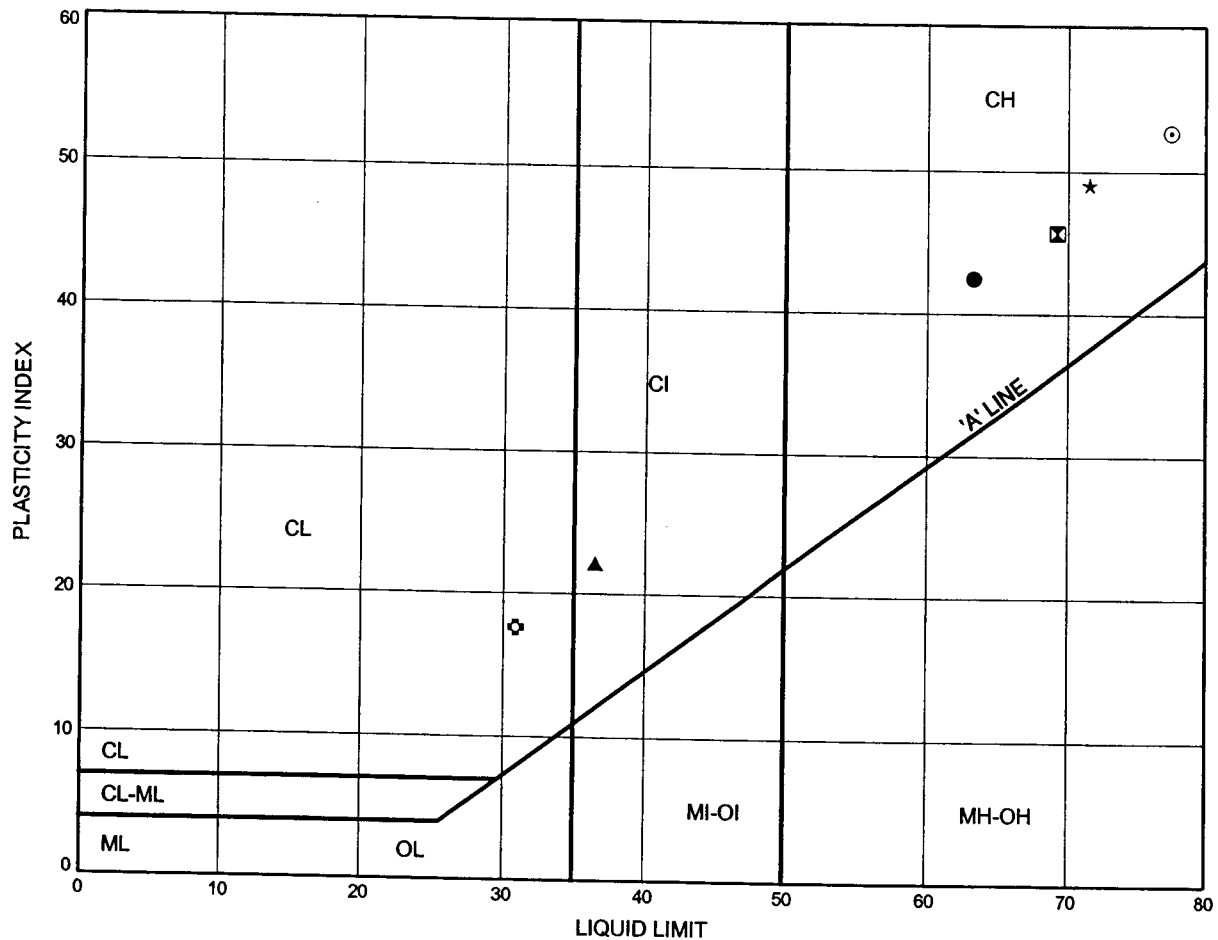
Date March 2007
Project 121-97-00



Prep'd MFA
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HWY 71 ATTERBERG LIMITS TEST RESULTS

FIGURE B4



SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	M2-1	4.80	347.37
⊠	M2-1	9.37	342.80
▲	M2-2	4.80	347.35
★	M2-2	10.90	341.25
⊙	M2-2	16.99	335.16
⊕	M2-3	4.80	347.25

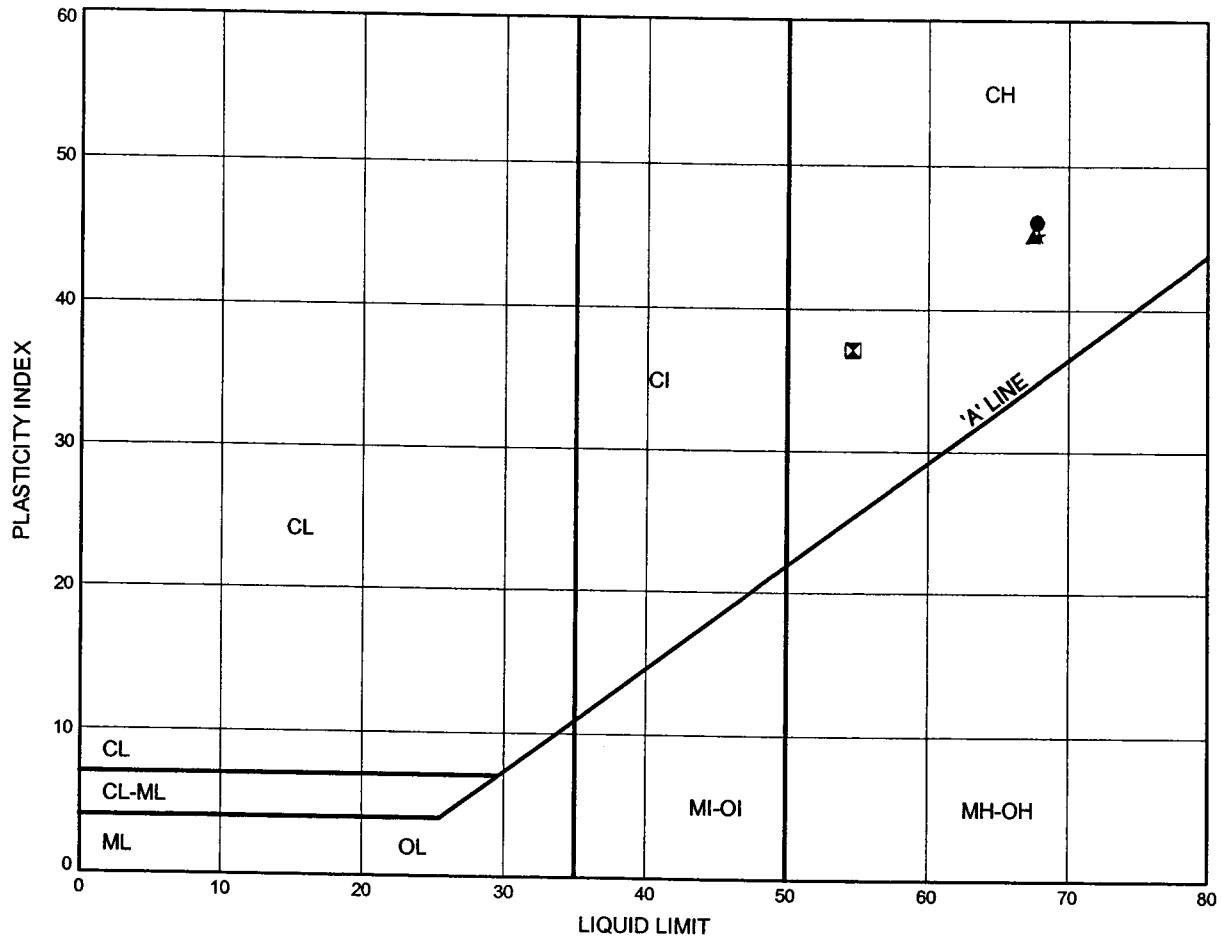
Date March 2007
 Project 121-97-00



Prep'd MFA
 Chkd. TJH

HWY 71 ATTERBERG LIMITS TEST RESULTS

FIGURE B5



SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	M2-3	10.90	341.15
⊠	M2-3	20.04	332.01
▲	M2-4	4.80	347.21
★	M2-4	9.37	342.64

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

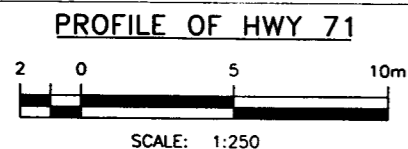
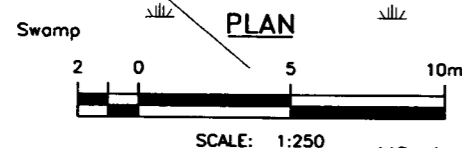
Suggested text for a NSSP on Pile Installation should contain the following:

"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*
- *Cobbles and boulders may impede the driving of the piles resulting in more arduous driving to reach bedrock*

Appendix E

Drawings



—+ .325
16+900

DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

Appendix F

Site Photographs





Plate 1 Existing bridge on Hwy 71 crossing Mather Creek # 2.



Plate 2 Existing Mather Creek # 2 timber piles.

