FOUNDATION INVESTIGATION AND DESIGN REPORT
DEER CREEK (FROOD) BRIDGE REPLACEMENT
HIGHWAY 539, TOWNSHIP OF CRERAR
W.P. 5236-05-01, SITE: 43-012

Geocres Number: 41I-265

Report to

MMM Group

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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 the bridge that carries Highway 539 over Deer Creek (Frood) in the Township of Crerar, Ontario. It is proposed that this bridge will be replaced on or close to the existing alignment.

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, records of boreholes, stratigraphic profile and cross-sections, laboratory test results and a written description of the subsurface conditions. A model of the subsurface conditions was developed from the data obtained in the course of the investigation.

Thurber carried out the investigation as a sub-consultant to MMM Group, under the Ministry of Transportation Ontario (MTO) Agreement Number 5008-E-0013.

2 SITE DESCRIPTION

The site is located approximately 13.6 km north of the intersection of Highway 17 (Trans-Canada Highway) and Highway 539 in Warren, Ontario. At the site, Deer Creek flows towards the southwest on a relatively gentle gradient. The channel is approximately 9.5 m wide and the water level in the creek was recorded as Elevation 235.65 in February 2010. The banks of the creek are approximately 3.5 m high at the site. The creek banks are heavily vegetated with shrubs and small trees. Selected photographs of the site are included in Appendix D.

Geologically, the site lies within the Canadian Shield, which is characterized by Pre-Cambrian bedrock. Locally, however, Deer Creek flows across post-glacial deposits of silt and sand, and sand and gravel with cobbles and boulders. There is a single private residence/farm located northeast of the existing bridge. No other buildings or other developments are located within the immediate vicinity of the site.

3 SITE INVESTIGATION AND FIELD TESTING

The site investigation and field-testing for this project was carried out from May 17 to 19, 2010 and consisted of drilling six boreholes identified as DCR10-01 to DCR10-06. The approximate locations of the six (6) boreholes are shown on the attached Borehole Locations and Soil Strata Drawing in Appendix E. Two boreholes (DCR10-04 and DCR10-05) were drilled at the approximate location of the proposed east abutment and two boreholes (DCR10-02 and DCR10-03) were drilled at the approximate location of the proposed west abutment. One borehole (DCR10-01) was drilled along the west approach and one borehole (DRC10-06) was drilled along the east approach. The depths of the boreholes ranged from 8.2 m to 18.5 m. The Record of Borehole sheets are included in Appendix A.

Prior to commencing the site investigation, clearance was obtained from utility companies having buried plant in the area.

A combination of hollow-stem auger drilling and NQ-sized coring techniques was 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.

At least 3 m of bedrock was cored in Boreholes DCR10-02 to DCR10-05 at the proposed abutment locations. The rock cores were logged and the total core recovery (TCR), solid core recovery (SCR) and Rock Quality Designation (RQD) were determined for each core.

Groundwater conditions in the open boreholes were observed throughout the drilling operations. At each abutment one standpipe piezometer consisting of 19 mm PVC pipe with a slotted screen was installed and enclosed in filter sand to permit groundwater level monitoring. The locations and completion details of the piezometers and boreholes are shown in Table 3.1.

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 for further examination and testing.

Table 3.1 – Borehole Completion Details

Borehole Location	Borehole ID	Piezometer Tip Depth/ Elevation (m)	Completion Details
West Approach	DCR10-01	None Installed	Borehole backfilled with bentonite to 2.6 m, then drill cuttings to 0.1 m, then asphalt to ground surface.
West Abutment	DCR10-02	9.1 / 230.0	Piezometer with 1.5 m slotted screen installed with sand filter to 6.9 m, bentonite seal from 6.9 m to ground surface.
	DCR10-03	None Installed	Borehole backfilled with bentonite to 1.8 m, then drill cuttings to 0.1 m, then asphalt to ground surface.
East Abutment	DCR10-04	None Installed	Borehole backfilled with bentonite to 1.8 m, then drill cuttings to 0.1 m, then asphalt to ground surface.
	DCR10-05	15.2 / 224.0	Piezometer with 1.5 m slotted screen installed with sand filter to 13.0 m, bentonite seal from 13.0 m to 12.3 m, then drill cuttings to 11.8 m, then bentonite to 1.6 m, then cuttings to 0.1 m, the asphalt to ground surface.
East Approach	DCR10-06	None Installed	Borehole backfilled with bentonite to 2.0 m, then drill cuttings to ground surface.

4 LABORATORY TESTING

All of the recovered soil samples were subjected to Visual Identification (VI) and to natural moisture content determination in the laboratory. Selected samples were also subjected to gradation analysis (hydrometer and sieve) and Atterberg Limits testing where appropriate, the results of which are summarized on the Record of Borehole sheets in Appendix A and on the figures contained in Appendix B.

Point load tests were carried out in the laboratory on selected samples of intact bedrock to assist in evaluation of the compressive strength of the bedrock. The results of the point load tests are tabulated in Table 1 in Appendix B and on the Record of Borehole sheets in Appendix A.

5 DESCRIPTION OF SUBSURFACE CONDITIONS

A detailed description of the soil stratigraphy encountered at each borehole location is presented in Appendix A and on the "Borehole Locations and Soil Strata" drawings 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.

In general, the site is underlain by sand fill overlying silty clay over cohesionless deposits of silt and sand with some gravel, cobbles and boulders, overlying granite bedrock.

5.1 Asphalt

Asphalt was encountered at the surface in all of the boreholes and the thickness ranged from 50 to 100 mm. Borehole DCR10-02 was drilled through the existing bridge deck and encountered 150 mm of concrete and 150 mm of wood underlying 100 mm of asphalt.

5.2 Sand Fill (Road Base Material)

Sand fill was encountered below the asphalt pavement in all of the boreholes, with the exception of Borehole DCR10-02, which was drilled through the existing bridge deck. The sand fill contained trace to some gravel and trace to some silt. The thickness of the sand fill ranged from 0.6 to 2.1 m and the elevation of the underside of the sand fill layer ranged from 236.9 to 239.2 m.

SPT N-values recorded in the sand fill ranged from 5 to 12 blows per 0.3 m of penetration, indicating a loose to compact relative density. Natural moisture contents of samples from the sand fill ranged from 3 to 13%.

A grain size distribution curve for a sample of the sand fill is shown on Figure B1 in Appendix B. The results of this test are summarized on the appropriate Record of Borehole sheet included in Appendix A and are presented below.

Soil Particles	Percentage
Gravel	4
Sand	83
Silt and Clay	13

5.3 Silty Clay

A layer of silty clay with trace gravel and sand seams was encountered below the sand fill in Boreholes DCR10-01 and DCR10-03 to DCR10-06 and at the ground surface under the bridge at the location of Borehole DCR10-02. The thickness of the silty clay layer ranged from 0.8 to 3.4 m with underside elevations of 234.8 to 236.2 m.

SPT N-values recorded in the silty clay layer ranged from 1 to 19 blows per 0.3 m of penetration, indicating a very soft to very stiff condition. Natural moisture contents of samples collected from the silty clay layer ranged from 13 to 44%.

Selected samples of the silty clay material were subjected to gradation analysis and Atterberg Limits testing where appropriate. The results are summarized below:

Soil Particles / Index Property	Percentage
Gravel	0
Sand	1 to 6
Silt	47 to 72
Clay	22 to 52
Liquid Limit	42 to 44
Plastic Limit	21 to 22

The grain size distribution curves for these samples are presented in Figure B2 of Appendix B and the results of the Atterberg Limits tests are plotted on Figure B6 of Appendix B. The Atterberg Limits tests indicate that the material is classified as a medium plasticity clay (CI). The results are also summarized on the appropriate Record of Borehole sheet in Appendix A.

5.4 Silt

Silt containing trace to some clay and trace to some sand was encountered below the silty clay layer in Boreholes DCR10-04 to DCR10-06. The thickness of the silt layer ranged from 3.4 to 5.7 m with an underside elevation of 230.5 to 232.4 m.

SPT N-values recorded in the silt layer ranged from 2 to 8 blows per 0.3 m of penetration, indicating a very loose to loose relative density. Natural moisture contents of the silt samples ranged from 21 to 26%.

Selected samples of the silt were subjected to gradation analysis, the results of which are summarized below. The grain size distribution curves for these samples are presented in Figure B3 of Appendix B and the results are summarized on the appropriate Record of Borehole sheets in Appendix A.

Soil Particles	Percentage
Gravel	0
Sand	1 to 14
Silt	67 to 90
Clay	6 to 19

5.5 Sandy Silt to Silty Sand

A layer of sandy silt to silty sand with varying proportions of silt, sand, and clay was encountered underlying the silty clay and silt layers in all of the boreholes. Where the layer was fully penetrated, the thickness of the deposits ranged from 2.1 m to 4.5 m with underside elevations of 227.2 to 231.9 m.

SPT N-values recorded in the sandy silt to silty sand generally ranged from 0 to 9 blows per 0.3 m penetration, indicating a very loose to loose condition. Natural moisture contents of the sandy silt to silty sand samples ranged from 18 to 43%.

Selected samples of the sandy silt to silty sand were subjected to gradation analysis (hydrometer and sieve), the results of which are summarized below. The grain size distribution curves for these samples are presented in Figure B4 of Appendix B and the results are summarized on the appropriate Record of Borehole sheet in Appendix A.

Soil Particles	Percentage
Gravel	0 to 3
Sand	27 to 77
Silt and Clay	20
Silt	23 to 69
Clay	2 to 5

5.6 Sand to Sand and Gravel

A deposit of sand to sand and gravel was encountered in Boreholes DCR10-01 to DCR10-05 below the silty sand to sandy silt layer. The sand to sand and gravel layer also contains some cobbles and boulders. Where the deposit was fully penetrated, the thickness ranged from 2.0 to 4.6 m, with an underside elevations of 223.8 to 229.9 m.

SPT N-values recorded in the sand and gravel layer ranged from 11 blows per 0.3 m penetration to 100 blows for less than 0.3 m of penetration indicating a compact to very dense relative density. The N-values of 100 blows for less than 0.3 m penetration are indicative of the presence of cobbles and boulders. Rock coring methods were required in Boreholes DCR10-03 and DCR10-05 to penetrate this dense layer containing cobbles and boulders. The natural moisture contents of the sand and gravel samples ranged from 11 to 24%.

One sand and gravel sample was subjected to laboratory gradation analysis, the results of which are summarized below. The grain size distribution curve for this sample is presented in Figure B5 of Appendix B and the results are summarized on the appropriate Record of Borehole sheet in Appendix A.

Soil Particles	Percentage
Gravel	53
Sand	37
Silt and Clay	10

5.7 Bedrock

The overburden soils described above are underlain by granitic bedrock. The bedrock was generally light grey with occasional pink and white bands visible in most cores. Occasional mechanical breaks and sub-vertical fractures were observed in the rock cores.

Approximately 3.1 to 4.3 m of bedrock core was collected from Boreholes DCR10-02 to DCR10-05.

Bedrock was encountered at various depths and it was proved by coring at the abutment boreholes. Table 5.1 summarizes the depths and elevations to the top of bedrock in the boreholes.

Table 5.1 – Depths and Elevations of Top of Bedrock

Borehole	Location	Top of Bedrock	
		Depth (m)	Elevation (m)
DCR10-02	STA. 11+857.5 2.8 m LT	9.2	229.9
DCR10-03	STA. 11+856.0 3.0 m RT	10.8	228.3
DCR10-04	STA. 11+831.1 2.3 m RT	14.1	225.0
DCR10-05	STA. 11+834.0 3.2 m LT	15.4	223.8

Core recovery in the bedrock generally ranged from 96% to 100%. The RQD values generally ranged from 68% to 100%, indicating fair to excellent rock quality. A RQD value of 33%, indicating poor rock quality, was noted for Borehole DCR10-05 Run 4. The Fracture Index (FI) of the rock, expressed as fractures per 0.3 m of core, generally ranged from 0 to 10.

The estimated unconfined compressive strength of the rock cores generally ranges from 158 MPa to 271 MPa, indicating a very strong to extremely strong rock. These estimated rock strength values are interpreted from point load tests that were conducted on rock cores recovered from the boreholes. A summary of the Point Load Test Results is presented in Table 1 in Appendix B.

5.8 Groundwater Conditions

Two 19 mm diameter standpipe piezometers were installed in selected boreholes, one at each abutment. Water levels were measured after completion of drilling and are presented in Table 5.2.

Table 5.2 – Groundwater Levels and Elevations

Borehole	Location	Date	Groundwater	
			Depth (m)	Elevation (m)
DCR10-01	Open borehole	May 19, 2010	5.6	233.6
DCR10-02	Piezometer	May 20, 2010	2.4	236.7
DCR10-05	Piezometer	May 19, 2010 May 20, 2010	2.4 2.3	236.8 236.9
DCR10-06	Open borehole	Open borehole	4.0	235.9

The water table will fluctuate seasonally and will be strongly influenced by the level of the river.

6 MISCELLANEOUS

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

The drilling and sampling operations in the field were supervised on a full time basis by Mr. Stephane Loranger of Thurber, under the direction of Mr. Tony Harte, M.Sc..

The borehole locations were recorded in the field as Station and Offset and coordinates and elevations are based on AutoCad drawings provided by MMM Group Ltd.

Ms. Lindsey Blaine, E.I.T. and Mr. Alastair E. Gorman, P.Eng prepared the Foundation Investigation 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

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 new structure proposed for this site.

The GA Drawing provided by MMM Group Limited shows that Highway 539 will cross Deer Creek on a single span structure that will lie essentially on the same alignment as the existing structure.

The highway will cross the new structure at Elevation 240.0 at the west abutment and Elevation 240.4 at the east abutment. These elevations represent a grade raise of less than 1.0 m above the existing highway profile. The resulting approach embankments will be no more than 2 m above the original ground level.

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

8 STRUCTURE FOUNDATIONS

The stratigraphy identified in the investigation consisted of 9.2 to 15.4 m of overburden overlying bedrock. The upper strata consist of silty clay and silt, while the lower strata consisted of silty sand, grading to sand with gravel, cobbles and boulders with increasing depth.

In the preparation of the geotechnical design recommendations, consideration was given to the following foundation types:

- Spread footings bearing on native soil
- Spread footings on engineered fill
- Steel pipe piles or H-piles
- Drilled shafts

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 bearing on native soil generally are the least expensive form of foundation.

However, at this site, spread footings are not recommended for the following reasons:

1. The near surface soils are very soft and very loose. SLS bearing resistances would be less than 50 kPa and the settlement expected to occur under these footings is assessed to be greater than 100 mm.
2. Shallow foundations placed near the edge of the creek would be at risk from erosion during the design life of the structure. Adding scour protection may be possible but will increase the complexity and cost of construction.
3. Founding spread footings at sufficient depth to resist erosion and develop higher bearing resistance will require deep excavation in permeable, cohesionless soils below the water table. Such an excavation would require extensive dewatering and yet would remain at risk of becoming destabilized due to the inflow of unbalanced groundwater heads.

Spread footings bearing on native soil are not considered to be a feasible solution at this site and are not recommended.

8.2 Spread Footings on Engineered Fill

The available geotechnical resistance could be improved, including improvement of the SLS condition, by founding the footings on a pad of Granular “A” engineered fill. Typically, spread footings on pads of engineered granular fill may be designed for the following geotechnical resistances:

Factored ULS	900 kPa
SLS	350 kPa

However, the use of spread footings on engineered fill at this site is not recommended for the following reasons:

1. The engineered fill pad could be subject to erosion during the design life of the structure. Adding scour protection may be possible but will increase the complexity and cost of construction.
2. Founding the engineered fill on suitably dense, uniform subgrade soils will require deep excavation in permeable, cohesionless soils below the water table. Such an excavation would require extensive dewatering and yet would remain at risk of becoming destabilized due to the inflow of unbalanced groundwater heads.

Spread footings bearing on native soil are not considered to be a feasible solution at this site and are not recommended.

8.3 Steel Pipe Piles

Based on the stratigraphy encountered at this site, piles must be driven to bedrock or into the sand and gravel layer containing cobbles and boulders overlying the bedrock. Even with reinforced tips, pipe piles are considered to be susceptible to damage when driven into bouldery soil and are not recommended at this site.

The pipe pile, having a larger soil displacement, is considered to have a higher susceptibility to damage than an H-pile.

8.4 Steel H-Piles

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

At the west abutment, the driven H-piles are expected to develop resistance on the bedrock. At the east abutment, based on the greater frequency of cobbles and boulders in the sand and gravel layer just above the bedrock, the piles may achieve refusal in this layer a short distance above bedrock. Accordingly, design recommendations have been developed on the assumption that the piles will develop resistance in the overburden. This is a conservative assumption and if the piles do reach bedrock the resistance recommendations will remain valid.

8.4.1 Axial Resistance

The following geotechnical resistances can be used for piles founded in the very dense native soils, a short distance above bedrock.

Pile Section	**Geotechnical Resistance (kN)**	
	Factored ULS	**SLS (25 mm)**
HP 310 X 110	1 600	1 400
HP 360 X 132	1 800	1 600

The structural resistance of the pile must be checked by the structural designer.

The highest recommended tip elevations for the H-piles are as follows:

Location		**Elevation**
West Abutment	BH DCR10-02	229.9
	BH DCR10-03	230.0
East Abutment	BH DCR10-04	226.5
	BH DCR10-05	227.0

The above elevations are for use in design and for estimating purposes. The actual pile tip will be controlled as described elsewhere in this report.

8.4.2 Pile Installation

Pile installation must be in accordance with OPSS 903.

Care must be taken to avoid overdriving and damaging the pile tips, a condition that could develop if a pile develops refusal on bedrock or a boulder. The contract documents must include an NSSP requiring the QVE to observe the pile driving operations and to ensure that the piles are not damaged by overdriving.

There is evidence of sloping bedrock at the site. The tips of the piles should be fitted with cast-steel H-section rock points from an approved manufacturer such as Titus Steel (Standard H-points) or approved equivalent to provide protection while driving into bouldery soil or to bedrock.

Suggested text requiring the use of rock points is included in Appendix F.

8.4.3 Pile Driving

Pile driving must be controlled by the Hiley Formula and an ultimate pile resistance to be specified by the designer in accordance with Clause 3.3.2 (b) Construction Stage of the Structural Manual. The Hiley Formula need not be used until the pile tips are within 1 m of the bearing stratum.

The appropriate pile driving note is "Piles to be driven in accordance with Standard SS 103-11 using an ultimate resistance of 3,200 kN for HP 310 X 110 piles and 3600 kN for HP 360 X 132 piles.

8.5 Drilled Shafts

Initial consideration was given to the use of drilled shafts to support the structure.

Drilled shafts at this site must be founded on the bedrock. Based on the stratigraphy and groundwater conditions, the caissons must be constructed using permanent steel liners, which would have to be advanced to a seal in the bedrock to exclude soil and groundwater. The drilled shafts are also likely to encounter obstructions in the bouldery granular layer just above the bedrock.

In view of the anticipated difficulties related to installation of the liners, removing obstructions such as boulders, and the typically higher costs of drilled shafts for a small bridge, the use of drilled shafts is not recommended.

8.6 Downdrag

Consolidation of the silty clay layer encountered at this site could induce some downdrag forces in the piles. Based on the 2.9 m maximum thickness of silty clay at the east abutment, a maximum, unfactored downdrag force of 110 kN must be applied to the pile design.

8.7 Frost Depth

The design depth of frost penetration at this site is 2.0 m.

The pile caps at the abutments must be provided with 2.0 m of earth cover as frost protection.

8.8 Abutment Design Considerations

From a geotechnical perspective, the conditions at this site are considered to be suitable for the design of conventional or semi-integral abutments with horizontal loads taken by battered piles.

The site is also suitable for the design of integral abutments provided a sufficiently long pile can be developed. The recommended minimum pile length at this site, from a geotechnical perspective, is 6 m for a pile driven to refusal. Based on the GA provided by MMM, this minimum pile length will be achieved

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 must be surrounded by a 600 mm diameter CSP as specified by the integral abutment design procedures.

After the pile is driven, the space between the pile and the CSP must be filled with sand. An NSSP should be included in the contract drawings specifying the gradation of the sand according to Table 8.1.

Table 8.1 – Integral Abutment Sand Backfill 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%

Design of the abutment must take account of the CHBDC requirements for scour and erosion protection.

8.9 Pile Lateral Resistance

The lateral resistance of the piles may be calculated using a value for the coefficient of horizontal subgrade reaction (k_s) and ultimate lateral resistance (p_{ult}) as follows:

Non-cohesive

$k_s = n_h \cdot z / D$ (kN/m^3)

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

Cohesive

$k_s = 67 * S_u / D$ (kN/m^3)

p_{ult} = $9 * S_u$ (kPa) at a depth of $3 * D$ (m) reduce to zero at the ground surface

where z = depth of embedment of pile in metres

D = pile width in metres

n_h = value from Table 8.2

S_u = undrained soil shear strength (kPa)

$\cdot\gamma$ = unit weight (Table 8.2)

K_p = passive earth pressure coefficient (Table 8.2)

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

The spring constant, K, for analysis may be obtained by the expression, $K = k_s * L * D$ (kN/m), where k_s is the coefficient of horizontal subgrade reaction (kN/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 on any one segment of pile, P_{ult}, may be obtained from the expression, $P_{ult} = p_{ult} * L * D$. This represents the ultimate load at which that particular element of 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 120 kN at ULS and 50 kN as SLS.

Table 8.2 – Parameters for Lateral Pile Resistance

Location	Elevation (m)	n_h (kN/m^3)	K_p	S_u (kPa)	Unit Weight* (kN/m^3)	Soil Conditions
West Abutment	OGL to 237	2,000	3.0	-	21	Sand fill, loose
	237 to 235	-	2.7	30	10	Silty clay
	235 to 232	1,300	3.0	-	11	Silty sand
	232 to 228	8,000	3.3	-	12	Sand and gravel with cobbles and boulders
East Abutment	OGL to 238	2,000	3.0	-	21	Sand fill, loose to compact
	238 to 235	-	2.7	45	10	Silty clay
	235 to 228	1,300	3.0	-	11	Silt and sand
	228 to 224	10,000	3.3	-	12	Sand and gravel with cobbles and boulders

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, the lateral resistance may be provided by battered piles.

8.10 Recommended Foundation

From a geotechnical and foundation cost perspective, the recommended foundation consists of steel H-piles driven to the specified resistance for a conventional or semi-integral abutment.

However, if other considerations favour integral abutments, a design based on H-piles in rock sockets could be developed. If this approach is adopted, it will be necessary for the structural and geotechnical engineers to work together to develop a solution.

9 EXCAVATION AND BACKFILL

9.1 General

All excavations must be carried out in accordance with the Occupational Health and Safety Act (OHSA) and in accordance with OPSS 902, November 2009. For the purposes of the OHSA, the native soils and the fill in the existing approach embankments at this site may be classified as Type 3 soils. 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 OPSS 902, November 2009.

9.3 Abutments

Backfill to the abutment must be granular material placed to the extents shown in OPSD 3101.150.

In the case of a conventional abutment, granular backfill is recommended but rock backfill can be permitted. An NSSP is required to limit rock fill used as abutment backfill to fragments no greater than 300 mm.

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

The backfill to the abutment walls must be in accordance with OPSS 902, November 2009. All granular material should meet the requirements of SP 110S13 Amendment to OPSS 1010, April 2004.

Compaction equipment to be used adjacent to the abutment walls must be restricted in accordance with SSP 105S10.

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

10 GROUNDWATER AND SURFACE WATER CONTROL

Based on the GA, the abutment bases will be slightly above to slightly below the 2 year flood stage of the creek. Depending on the creek level and groundwater level at the time of construction, there may be a need for dewatering which may also include a need for protection from inundation by the creek. The pervious nature of the soils below the surficial silty clay and the proximity of the creek will make unwatering of excavations difficult if construction is carried out at a time when the water level is above the base of the excavation.

Since it will not be possible to control the time of construction or the weather at that time, it is recommended that the contract require that the abutment excavation be protected from inundation by creek water. Typically a combination of a sheet pile cut-off and vacuum well points may be required. The contract documents should provide a high water level in the creek against which the contractor is required to provide protection.

Steps should be taken to divert surface run off away from the excavation.

Suggested wording for an NSSP on unwatering is included in Appendix F.

11 BRIDGE APPROACHS AND EMBANKMENTS

The existing approach fills are generally less than 2 m high above the original ground and the grade raise shown on the GA is in the order of 1 m.

The approach embankments will be stable if constructed with side slopes not exceeding 2H:1V.

The grade raise may induce settlements in the very soft to very stiff foundation clay that may be in the order of 30 to 40 mm and that will be time dependent.

These settlements could be avoided by excavating the silty clay and backfilling with granular fill. Excavation to Elevation 235 would be required at the west abutment and Elevation 235 at the east abutment. However, the costs of this work, especially within a staged construction environment, are expected to outweigh the benefits.

It is recommended that the replacement structure and the associated 1 m+ grade raise be constructed and the performance of the pavement in the approaches be monitored for a period of years, perhaps 5 years. It is recommended that, in conjunction with this approach, the Ministry make allowance to carry out partial depth milling and repaving if the magnitude of settlement is found to exceed acceptable limits.

12 ROADWAY PROTECTION

Roadway protection will be required to facilitate staging of removals and new construction at this site. Sheet-piles or soldier pile & lagging walls are considered appropriate for roadway protection at this site. The contractor should select the wall type and undertake design taking into account the earth pressure parameters given in Section 13 of this report.

The temporary excavation support system should be designed and constructed in accordance with OPSS 539, November 2009. In general, the lateral movement of the temporary shoring system should meet Performance Level 2 as specified in OPSS 539, November 2009.

13 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 lateral 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 lateral yielding (restrained system), at-rest horizontal earth pressures should be used.

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

For fully drained conditions, 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:

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

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

K = earth pressure coefficient (see Table13.1)

γ = unit weight of retained soil (see Table 13.1)

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 13.1.

In conventional design, the use of a material with a high friction angle and low active pressure coefficient (e.g. Granular A, Granular B Type II) would result 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) would result 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 coefficients in the Table 13.1 are ultimate values and require certain movements for the respective conditions to be mobilized. The values to use in design can be estimated from Figure C6.9.1 (a) in the Commentary to the CHBDC, 2006.

Table 13.1 – Earth Pressure Coefficients

Wall Condition	**Earth Pressure Coefficient (K)**					
	OPSS Granular A or OPSS Granular B Type II $\phi = 35°$; $\gamma = 22.8$ kN/m^3		OPSS Granular B Type I $\phi = 32°$; $\gamma = 21.2$ kN/m^3		Rock Fill (Max. size: 300 mm) $\phi = 42°$; $\gamma = 19.0$ 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	0.38*	0.31	0.46*	0.20	0.26*
At rest (Restrained Wall)	0.43	-	0.47	-	0.33	-
Passive (Movement Towards Soil Mass)	3.70	-	3.30	-	5.0	-

* For wing walls

14 SEISMIC CONSIDERATIONS

14.1 Seismic Design Parameters

The bridge is located in an area where the overburden is underlain by Pre-Cambrian rocks of very low activity.

The following seismic parameters apply to this site:

Velocity Related Seismic Zone	1.0
Zonal Velocity Ratio	0.05
Acceleration Related Seismic Zone	1.0
Zonal Acceleration Ratio	0.05

14.2 Liquefaction Potential

The soil profile type at this site has been classified as Type I. Therefore, according to Table 4.4 of the CHBDC, a Site Coefficient "S" (ground motion amplification factor) of 1.0 should be used in seismic design.

As the structure is supported on steel piles, the foundation loads will be transferred by the steel piles to dense soils or bedrock. It is not considered likely that the vertical geotechnical resistance of the piles will be compromised due to seismic loading.

The embankments themselves will be constructed above the groundwater level and are not considered to be in danger of undergoing liquefaction.

14.3 Retaining Wall Dynamic Earth Pressures

In accordance with Clause 4.6.4 of the CHBDC, retaining structures should be designed using active (KAE) and passive (KPE) earth pressure coefficients that incorporate the effects of earthquake loading.

For the design of retaining walls, the coefficient of horizontal earth pressure in Table 14.1 may be used.

Table 14.1 – Earth Pressure Coefficient (K) for the Earthquake Loading

Wall Condition	**Earth Pressure Coefficient (K)**					
	OPSS Granular A or OPSS Granular B Type II $\phi = 35^\circ$; $\gamma = 22.8$ kN/m^3		OPSS Granular B Type I $\phi = 32^\circ$; $\gamma = 21.2$ kN/m^3		Rock Fill (Max. size: 300 mm) $\phi = 42^\circ$; $\gamma = 19.0$ 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)	0.30	0.47*	0.34	0.58*	0.22	0.31*
At rest (Restrained Wall)	0.53	-	0.58	-	0.44	-
Passive (Movement Towards Soil Mass)	3.58	-	3.15	-	4.92	-

*For wing walls

15 CONSTRUCTION CONCERNS

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

1. Pile Installation

The presence of cobbles and boulders in a very dense matrix of sand and gravel just above the bedrock may present difficulties in installing the piles to the specified depth and to the specified tolerances for location and verticality.

2. Excavation

Hydraulic equipment is expected to be capable of excavating to the depths required for abutments on piles. If excavations advance below the existing groundwater level, groundwater control measures may have to be implemented in order to maintain stables sides and base in the excavation.

3. Unwatering

The pervious nature of the soils encountered at this site and the proximity to the creek will make unwatering of excavations difficult if the creek level is high at the time of construction. Depending on the locations of the abutments, steps may have to be taken to control the river and exclude it from the excavations. Typically, a combination of a sheet pile cutoff and vacuum well-points may be required.

16 CLOSURE

Engineering analysis and preparation of the Foundation Design Report were carried out by Mr. Alastair E. Gorman, P.Eng.

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

Thurber Engineering Ltd.

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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

C_{pen} Shear Strength Determination by Pocket Penetrometer

(1) SPT 'N' Value Standard Penetration Test 'N' Value – refers to the number of blows from a 63.5kg hammer free falling a height of 0.76m to advance a standard 50 mm outside diameter split spoon sampler for 0.3 m depth into undisturbed ground.

(2) DCPT Dynamic Cone Penetration Test – Continuous penetration of a 50 mm outside diameter, 60° conical steel point attached to "A" size rods driven by a 63.5 kg hammer free falling a height of 0.76 m. The resistance to cone penetration is the number of hammer blows required for each 0.3 m advance of the conical point into undisturbed ground.

UNIFIED SOILS CLASSIFICATION

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

EXPLANATION OF ROCK LOGGING TERMS

ROCK WEATHERING CLASSIFICATION

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

SYMBOLS

▨	CLAYSTONE
▤	SILTSTONE
░	SANDSTONE
■	COAL
▦	Bedrock (general)

DISCONTINUITY SPACING

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

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.

STRENGTH CLASSIFICATION

Rock Strength	Approximate Uniaxial Compressive Strength		Field Estimation of Hardness*
	(MPa)	(psi)	
Extremely Strong	Greater than 250	Greater than 36,000	Specimen can only be chipped with a geological hammer
Very Strong	100-250	15,000 to 36,000	Requires many blows of geological hammer to break
Strong	50-100	7,500 to 15,000	Requires more than one blow of geological hammer to break
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

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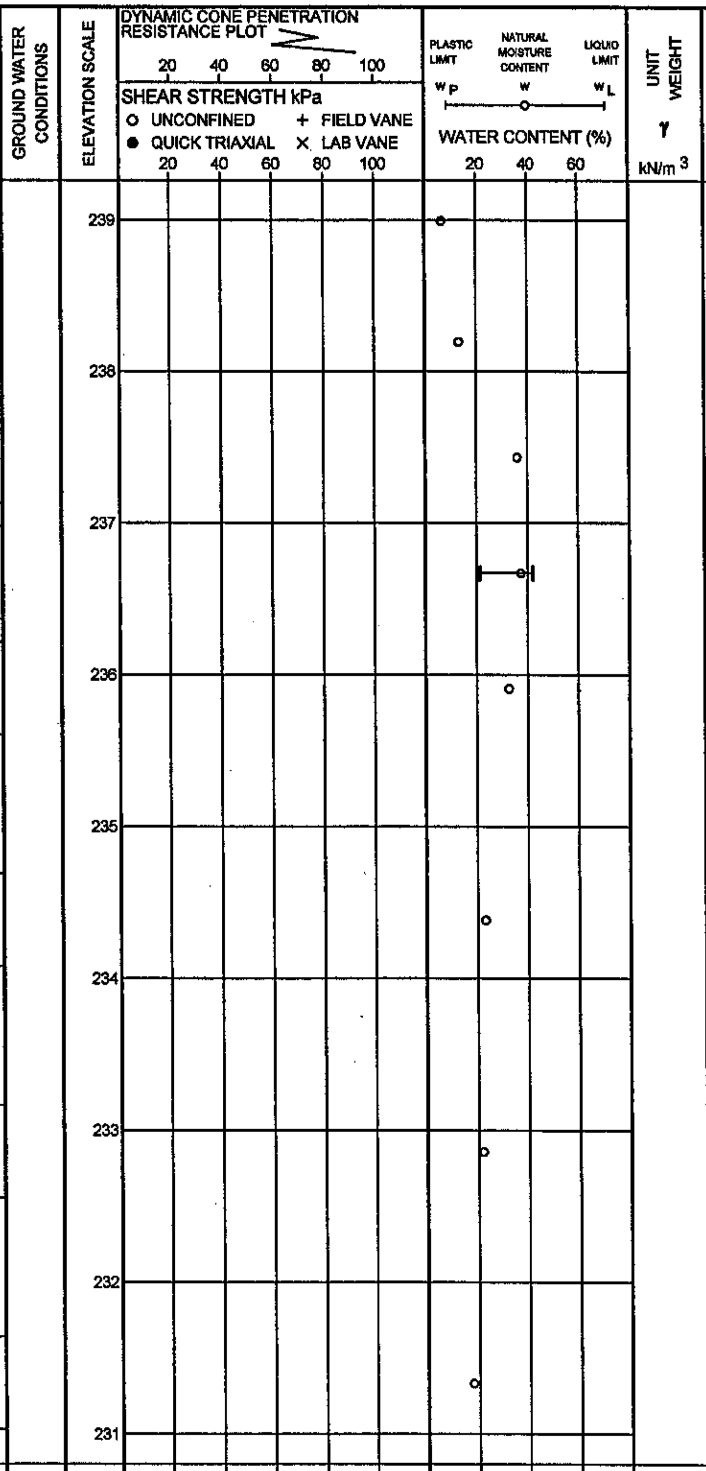
RECORD OF BOREHOLE No DCR10-01

1 OF 1

METRIC

W.P. 5236-05-01	LOCATION STA 11+815.4, 3.0m LT		ORIGINATED BY SLL
HWY 539	BOREHOLE TYPE Hollow Stem Augers		COMPILED BY MFA
DATUM Geodetic	DATE 2010.05.19 - 2010.05.19		CHECKED BY LRB

ELEV DEPTH	DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
239.2								
0.0	**ASPHALT:** (100mm)							
0.1	**SAND**, trace to some gravel, trace to some silt Compact Brown Moist (FILL)	1	GS					
		1	SS	10				
237.8 1.4	Silty **CLAY**, trace sand Firm to Very Stiff Brown (CI)	2	SS	19				
		3	SS	5	0	1	58	41
	Some sand seams	4	SS	7				
235.1 4.1	Sandy **SILT**, trace clay Very Loose to Loose Grey Wet to Saturated	5	SS	3				
		6	SS	5	0	31	64	5
231.9 7.3	**SAND**, some silt, some cobbles and boulders Dense Grey Wet	7	SS	45				
230.7 8.5	END OF BOREHOLE AT 8.5m UPON AUGER REFUSAL ON PROBABLE BEDROCK. BOREHOLE OPEN TO 6.2m AND WATER LEVEL AT 5.6m UPON COMPLETION. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG TO 2.6m, THEN CUTTINGS TO 0.1m, THEN ASPHALT TO SURFACE.							



+³, X³: Numbers refer to Sensitivity

20 15 5 10 (%) STRAIN AT FAILURE

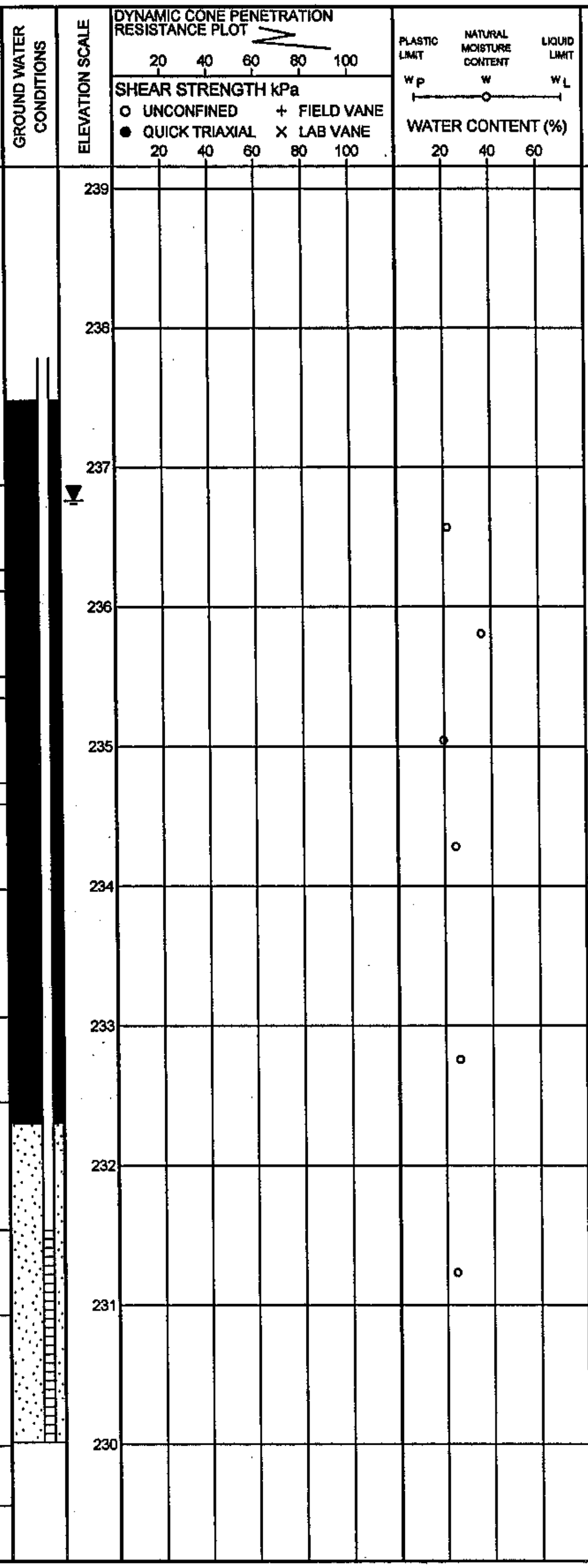
ONTMT4S 6158 (DC FROOD).GPJ 11/19/10

RECORD OF BOREHOLE No DCR10-02

1 OF 2

METRIC

W.P. 5236-05-01	LOCATION STA 11+834.0, 3.2m LT	ORIGINATED BY SLL
HWY 539	BOREHOLE TYPE Hollow Stem Augers/NQ Coring	COMPILED BY MFA
DATUM Geodetic	DATE 2010.05.19 - 2010.05.19	CHECKED BY LRB



ELEV DEPTH	DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
239.1						
0.0	**ASPHALT:** (100mm)					
238.8	**CONCRETE:** (150mm)					
238.7 0.4	**WOOD:** (150mm) Open space under bridge deck.					
237.3 1.8	Silty **CLAY**, mixed with sand, trace roots and rootlets Very Soft to Soft Brown	1	SS	1		
		2	SS	3		
235.2 3.9	Silty **SAND**, trace gravel Very Loose to Loose Grey Wet	3	SS	6		
		4	SS	2		3 76 20 (SI+CL)
	Timber at 5.2m.					
233.0 6.1	Sandy **SILT**, trace clay Loose Grey Moist to Wet	5	SS	7		0 27 69 4
231.9 7.2	**SAND**, some silt, trace cobbles Compact Grey Wet	6	SS	11		
229.9 9.2	**GRANITE BEDROCK**, with micaceous seams, very strong to extremely strong	1	RUN		FI 0 0	RUN 1# TCR=100%, SCR=100%, RQD=100% UCS=257MPa

Continued Next Page

$+^3, \times^3$: Numbers refer to Sensitivity

$15\overset{20}{\underset{10}{\oplus}}5$ (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No DCR10-02

2 OF 2

METRIC

W.P.	5236-05-01	LOCATION	STA 11+834.0, 3.2m LT	ORIGINATED BY	SLL
HWY	539	BOREHOLE TYPE	Hollow Stem Augers/NQ Coring	COMPILED BY	MFA
DATUM	Geodetic	DATE	2010.05.19 - 2010.05.19	CHECKED BY	LRB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m ^{3}	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION Continued From Previous Page	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	Sub-vertical joints at 9.96 and 10.06m.						229			2 0	RUN 2# TCR=100%, SCR=95%, RQD=86% UCS=271MPa
	Sub-vertical joints at 10.42, 10.57, 10.62, 10.72, and 10.97m.		2	RUN						4 3	
							228			3	RUN 3# TCR=100%, SCR=95%, RQD=78% UCS=172MPa
	Vertical joint at 11.38 to 11.58m. Sub-vertical joints at 11.58, 11.66, 11.73, 11.91, and 12.04m.		3	RUN						7 4	
226.8							227			2	
12.3	END OF BOREHOLE AT 12.3m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen.										

WATER LEVEL READINGS:

DATE	DEPTH (m)	ELEV. (m)
2010.05.20	2.4	236.7

$+^3, \times^3$: Numbers refer to Sensitivity

$15 \overset{20}{\underset{10}{\oplus}} 5$ (%) STRAIN AT FAILURE

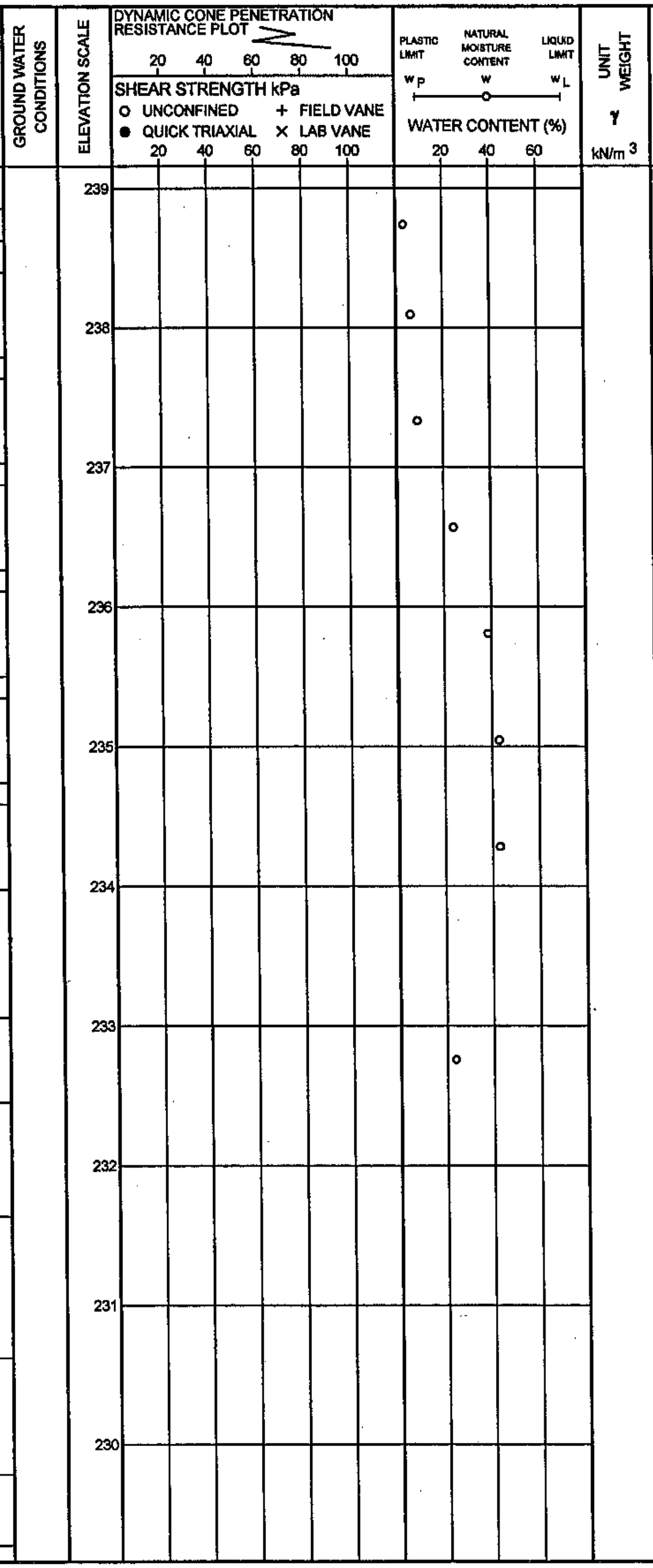
Ministry of Transportation
Ontario

RECORD OF BOREHOLE No DCR10-03

1 OF 2 **METRIC**

W.P. 5236-05-01
LOCATION STA 11+831.1, 2.3m RT
ORIGINATED BY SLL
HWY 539
BOREHOLE TYPE Hollow Stem Augers/NQ Coring
COMPILED BY MFA
DATUM Geodetic
DATE 2010.05.17 - 2010.05.17
CHECKED BY LRB

SOIL PROFILE		SAMPLES			REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
239.1								
0.0	**ASPHALT:** (100mm)							
0.1	**SAND**, trace to some gravel, trace silt Loose Brown Moist (FILL)	1	GS					
		1	SS	9				
		2	SS	5	4	82	13 (SI+CL)	
236.9 2.2	Silty **CLAY**, trace gravel, with sand seams Soft Brown	3	SS	3				
236.1 3.0	Silty **SAND**, trace gravel, trace clay, mixed with organics and wood fibers Very Loose to Loose Brown Moist to Wet	4	SS	0				
		5	SS	1				
		6	SS	2				
	Becoming grey, saturated							
		7	SS	5	1	74	23	2
231.6 7.5	**SAND**, some gravel, some cobbles and boulders Very Dense Grey Moist	1	RUN					
		2	RUN					
		3	RUN					



Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity

15-⊕-5 (20 above, 10 below) (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No DCR10-03

2 OF 2

METRIC

W.P. 5236-05-01
LOCATION STA 11+831.1, 2.3m RT
ORIGINATED BY SLL
HWY 539
BOREHOLE TYPE Hollow Stem Augers/NQ Coring
COMPILED BY MFA
DATUM Geodetic
DATE 2010.05.17 - 2010.05.17
CHECKED BY LRB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION Continued From Previous Page	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	SAND, some gravel, some cobbles and boulders Very Dense Grey Moist		4 8	RUN SS	 100/		229				
228.3 10.8	**GRANITE BEDROCK**, fresh, strong to very strong Sub-vertical joint at 11.34 to 11.42m. Rubble zone at 11.73 to 12.17m.		5	RUN	.050		228			FI 1 1 2 8	RUN 5# TCR=96%, SCR=82%, RQD=68% UCS=240MPa
	Sub-vertical joint at 12.17 to 12.24, and 13.49 to 13.64m.		6	RUN			227 226			3 1 4 0 4	RUN 6# TCR=100%, SCR=100%, RQD=79% UCS=227MPa
	Sub-vertical joint at 13.64 to 13.72, and 14.20 to 14.35m. Mechanical break at 15.14m.		7	RUN			225			1 3 2 1 0	RUN 7# TCR=100%, SCR=100%, RQD=98% UCS=158MPa
224.0 15.1	END OF BOREHOLE AT 15.1m. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG TO 1.8m, THEN CUTTINGS TO 0.1m, THEN ASPHALT TO SURFACE.										

$+^3, \times^3$: Numbers refer to Sensitivity

$\overset{20}{\underset{10}{15 \text{-}\oplus\text{-} 5}}$ (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

RECORD OF BOREHOLE No DCR10-04

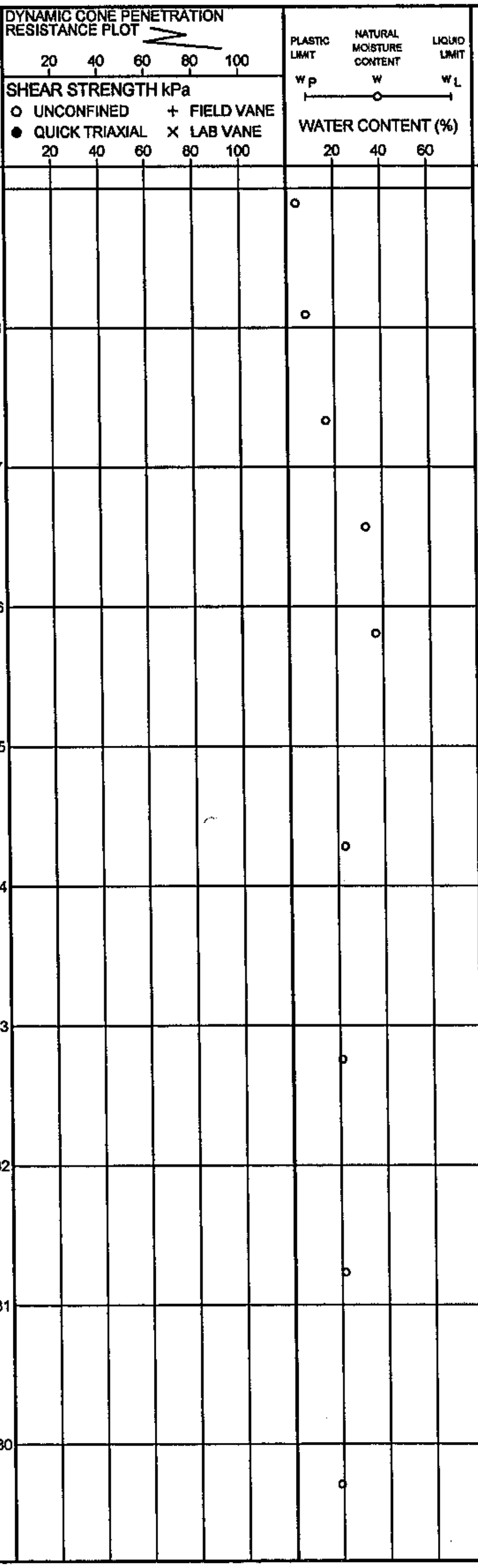
1 OF 2

METRIC

W.P. 5236-05-01
LOCATION STA 11+856.0, 3.0m RT
ORIGINATED BY SLL
HWY 539
BOREHOLE TYPE Hollow Stem Augers/NW Casing/NQ Coring
COMPILED BY MFA
DATUM Geodetic
DATE 2010.05.18 - 2010.05.19
CHECKED BY LRB

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
239.1								
0.0	**ASPHALT:** (88mm)							
0.1	**SAND**, some gravel, trace silt Loose Brown Moist (FILL)	1	GS					
		1	SS	6				
237.7 1.4	Silty **CLAY**, trace sand, trace roots and rootlets Firm Brown	2	SS	5				
		3	SS	5	0	6	72	22
		4	SS	4				
234.8 4.3	**SILT**, trace to some clay, trace rootlets Very Loose to Loose Grey Wet	5	SS	5				
	Some sand seams	6	SS	6	0	1	90	9
		7	SS	3				
230.5 8.6	Sandy **SILT**, trace clay, some cobbles Loose Grey to Brown Moist to Wet	8	SS	9	0	27	68	5

ELEVATION SCALE: 239, 238, 237, 236, 235, 234, 233, 232, 231, 230



UNIT WEIGHT γ kN/m³

REMARKS & GRAIN SIZE DISTRIBUTION (%)

Continued Next Page

$+^3, \times^3$: Numbers refer to Sensitivity

20 15 5 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No DCR10-04

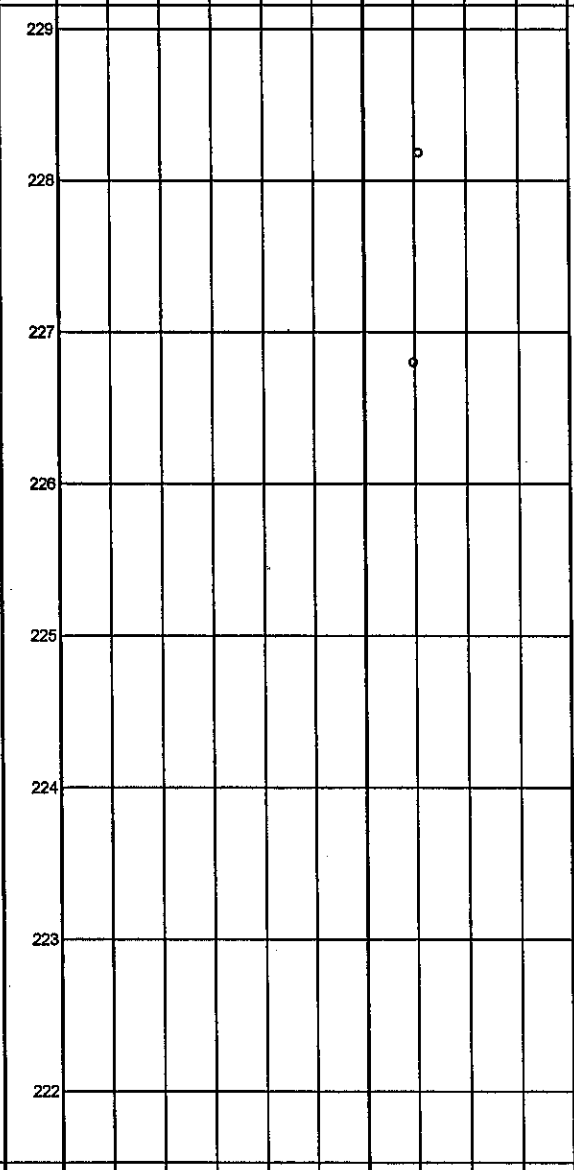
2 OF 2

METRIC

W.P.	5236-05-01	LOCATION	STA 11+856.0, 3.0m RT	ORIGINATED BY	SLL
HWY	539	BOREHOLE TYPE	Hollow Stem Augers/NW Casing/NQ Coring	COMPILED BY	MFA
DATUM	Geodetic	DATE	2010.05.18 - 2010.05.19	CHECKED BY	LRB

SOIL PROFILE · SAMPLES · GROUND WATER CONDITIONS · ELEVATION SCALE · DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 · SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE 20 40 60 80 100 · PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L · WATER CONTENT (%) 20 40 60 · UNIT WEIGHT γ kN/m³ · REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	UNIT WEIGHT	REMARKS
	Continued From Previous Page							
	Sandy SILT, trace clay, some cobbles Loose to Very Dense Grey to Brown Moist to Wet		9	SS	6			
227.2 / 11.9	**SAND**, some silt, some cobbles Very Dense Brown Wet		10	SS	100/ .175			
225.0 / 14.1	**GRANITE BEDROCK**, very strong to extremely strong		1	RUN			FI 5 5 0	RUN 1# TCR=100%, SCR=100%, RQD=74% UCS=264MPa
			2	RUN			0 0 2	RUN 2# TCR=100%, SCR=100%, RQD=94% UCS=218MPa
	Sub-vertical joint at 16.13 to 16.21m. Sub-vertical joints at 16.54, 16.71, 16.79, and 17.12 to 17.21m.		3	RUN			0 1 4 1 0	RUN 3# TCR=100%, SCR=100%, RQD=85% UCS=209MPa
221.5 / 17.6	END OF BOREHOLE AT 17.6m. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG TO 1.8m, THEN SAND CUTTINGS TO 0.1m, THEN ASPHALT TO SURFACE.							



ONTMT4S 6158 (DC FROOD).GPJ 11/19/10

+³, ×³: Numbers refer to Sensitivity

20 / 15 ◯ 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No DCR10-05

1 OF 2

METRIC

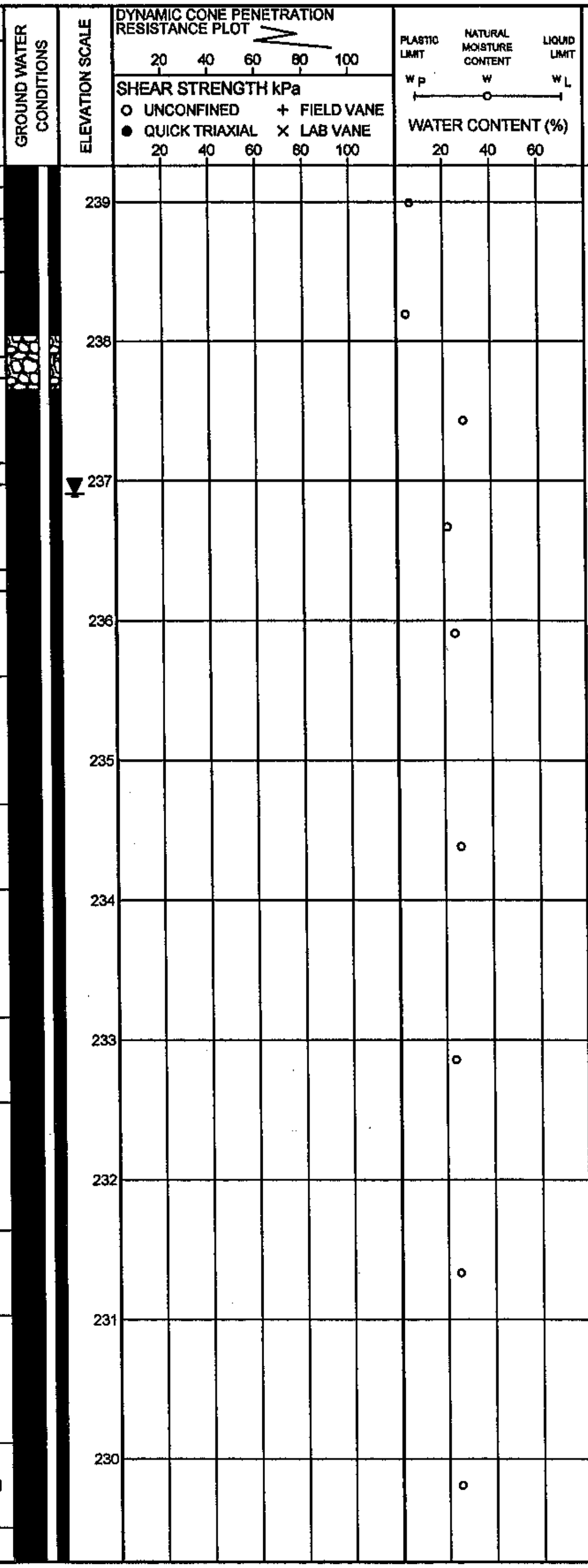
W.P.	5236-05-01	LOCATION	STA 11+857.5, 2.8m LT	ORIGINATED BY	SLL
HWY	539	BOREHOLE TYPE	Hollow Stem Augers/NQ Coring	COMPILED BY	MFA
DATUM	Geodetic	DATE	2010.05.18 - 2010.05.18	CHECKED BY	LRB

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
239.2	
0.0	**ASPHALT:** (100mm)
0.1	**SAND**, some gravel, trace silt Compact Brown Moist (FILL)
237.8 1.4	Silty **CLAY**, some sand seams Firm to Very Stiff Brown to Grey
236.2 3.0	**SILT**, some clay, some sand, trace rootlets Very Loose to Loose Dark Brown Moist Becoming grey, wet
230.5 8.7	Silty **SAND**, trace clay Very Loose Brown Saturated

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	GS		
1	SS	12	
2	SS	4	
3	SS	16	
4	SS	6	0 14 67 19
5	SS	2	
6	SS	5	
7	SS	8	
8	SS	WH	0 65 33 3



UNIT WEIGHT γ kN/m³

Continued Next Page

+ ³, × ³: Numbers refer to Sensitivity

20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

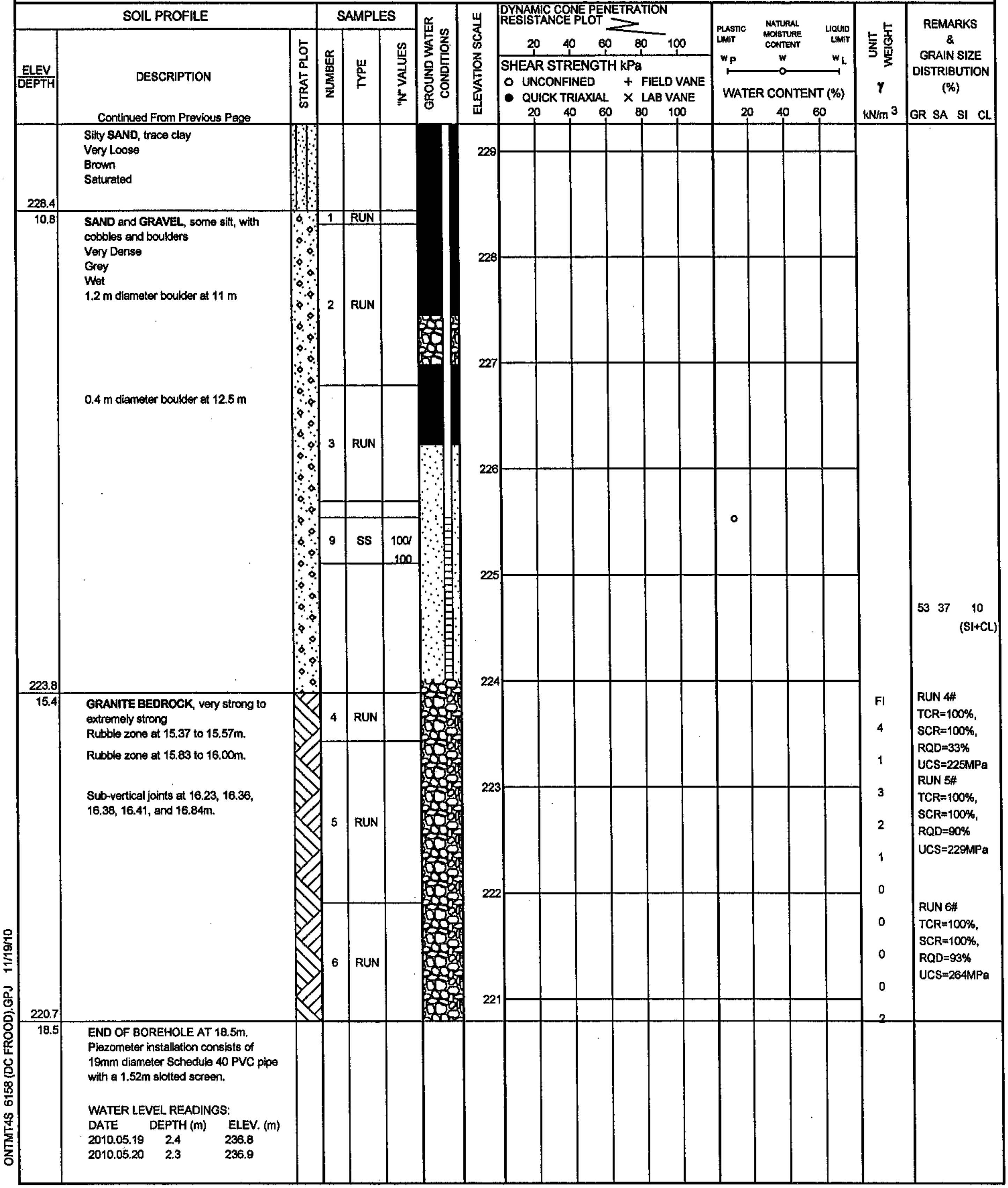
THURBER

RECORD OF BOREHOLE No DCR10-05

2 OF 2

METRIC

W.P. 5236-05-01
LOCATION STA 11+857.5, 2.8m LT
ORIGINATED BY SLL
HWY 539
BOREHOLE TYPE Hollow Stem Augers/NQ Coring
COMPILED BY MFA
DATUM Geodetic
DATE 2010.05.18 - 2010.05.18
CHECKED BY LRB



Continued From Previous Page

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
228.4	Silty **SAND**, trace clay Very Loose Brown Saturated					
10.8	**SAND** and **GRAVEL**, some silt, with cobbles and boulders Very Dense Grey Wet 1.2 m diameter boulder at 11 m	1	RUN			
		2	RUN			
	0.4 m diameter boulder at 12.5 m	3	RUN			
		9	SS	100/100		53 37 10 (SI+CL)
223.8 15.4	**GRANITE BEDROCK**, very strong to extremely strong Rubble zone at 15.37 to 15.57m. Rubble zone at 15.83 to 16.00m. Sub-vertical joints at 16.23, 16.36, 16.38, 16.41, and 16.84m.	4	RUN		FI 4 1	RUN 4# TCR=100%, SCR=100%, RQD=33% UCS=225MPa
		5	RUN		3 2 1 0	RUN 5# TCR=100%, SCR=100%, RQD=90% UCS=229MPa
		6	RUN		0 0 0 2	RUN 6# TCR=100%, SCR=100%, RQD=93% UCS=264MPa
220.7 18.5	END OF BOREHOLE AT 18.5m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen.					

WATER LEVEL READINGS:

DATE	DEPTH (m)	ELEV. (m)
2010.05.19	2.4	236.8
2010.05.20	2.3	236.9

ONTMT4S 6158 (DC FROOD).GPJ 11/19/10

+ ³, × ³: Numbers refer to Sensitivity

$15\overset{20}{\underset{10}{\oplus}}5$ (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

RECORD OF BOREHOLE No DCR10-06

1 OF 1 **METRIC**

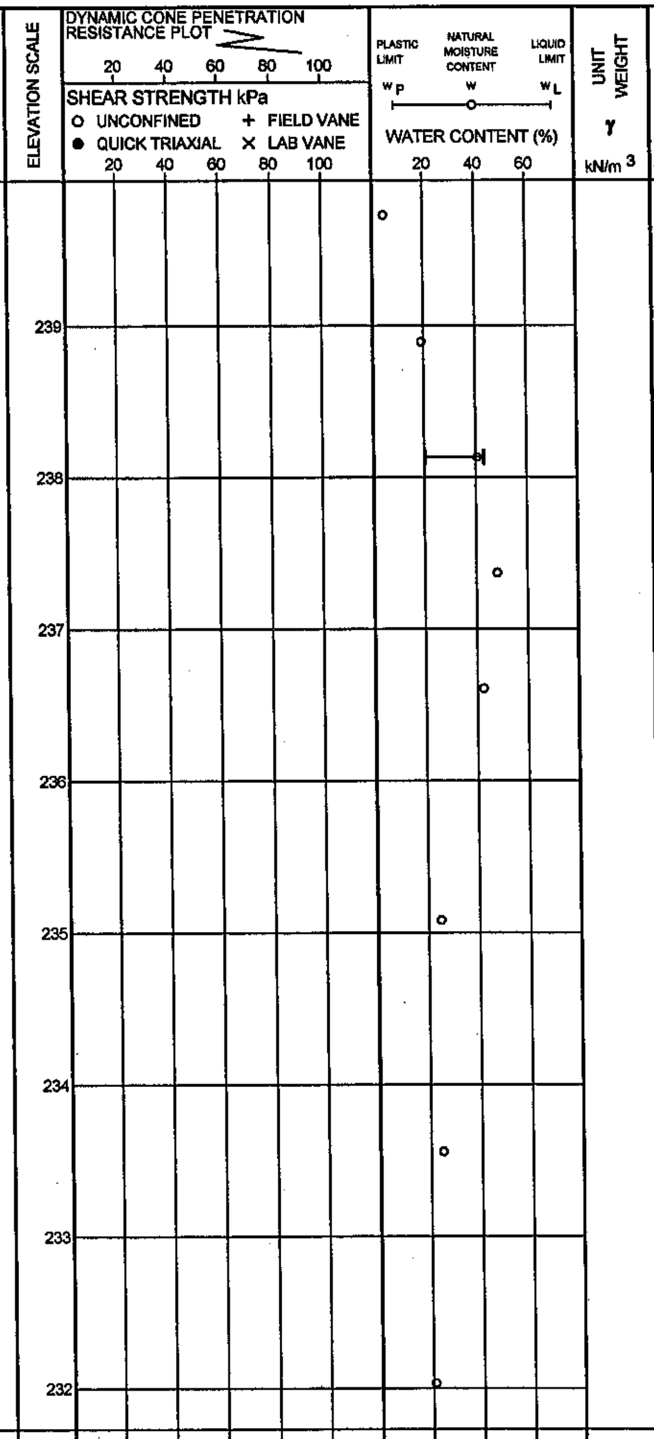
W.P.	5236-05-01	LOCATION	STA 11+876.0, 2.6m RT	ORIGINATED BY	SLL
HWY	539	BOREHOLE TYPE	Hollow Stem Augers	COMPILED BY	MFA
DATUM	Geodetic	DATE	2010.05.19 - 2010.05.19	CHECKED BY	LRB

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION	STRAT PLOT
239.9		
0.0 0.1	**ASPHALT:** (50mm)	■
239.2 0.7	**SAND**, some gravel, trace silt Brown Moist (FILL)	☒
235.8 4.1	Silty **CLAY**, trace sand Soft to Firm Brown (CI)	▨
232.4 7.5	**SILT**, trace sand, trace clay Loose Grey to Brown Moist to Wet	‖
231.7 8.2	Silty **SAND** Loose Brown Moist to Wet	⁘
	END OF BOREHOLE AT 8.2m. BOREHOLE OPEN TO 8.2m AND WATER LEVEL AT 4.0m UPON COMPLETION. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG TO 2.0m, THEN CUTTINGS TO SURFACE.	

SAMPLES

NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
1	GS						
1	SS	6					
2	SS	6		0	1	47	51
3	SS	3					
4	SS	3					
5	SS	4					
6	SS	7		0	5	89	6
7	SS	9					



+3, ×3: Numbers refer to Sensitivity

15-⊕-5 (20 above, 10 below) (%) STRAIN AT FAILURE

Appendix B

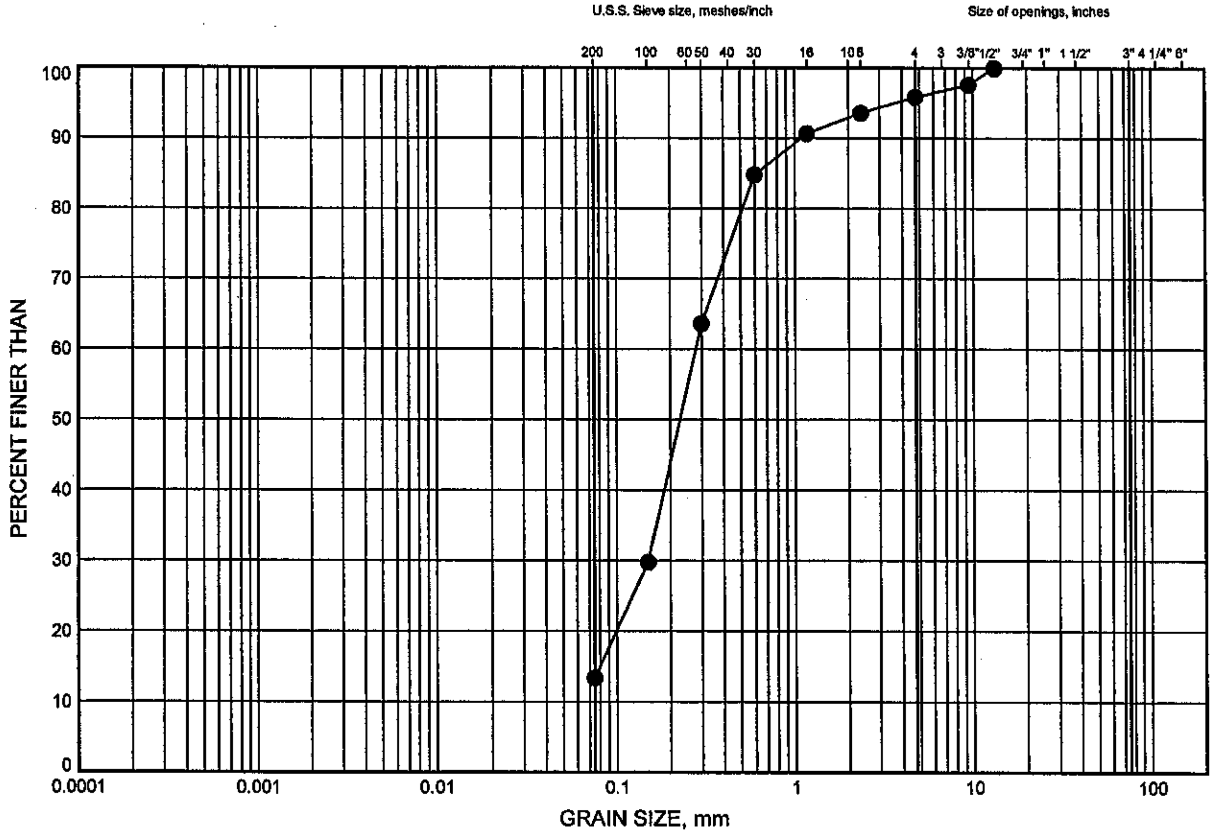
Laboratory Test Results

DEER CREEK (FROOD) BRIDGE

GRAIN SIZE DISTRIBUTION

FIGURE B1

SAND FILL



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	DCR10-03	1.83	237.27

W.P.# 5236-05-01

Prepared By AN

Checked By LRB

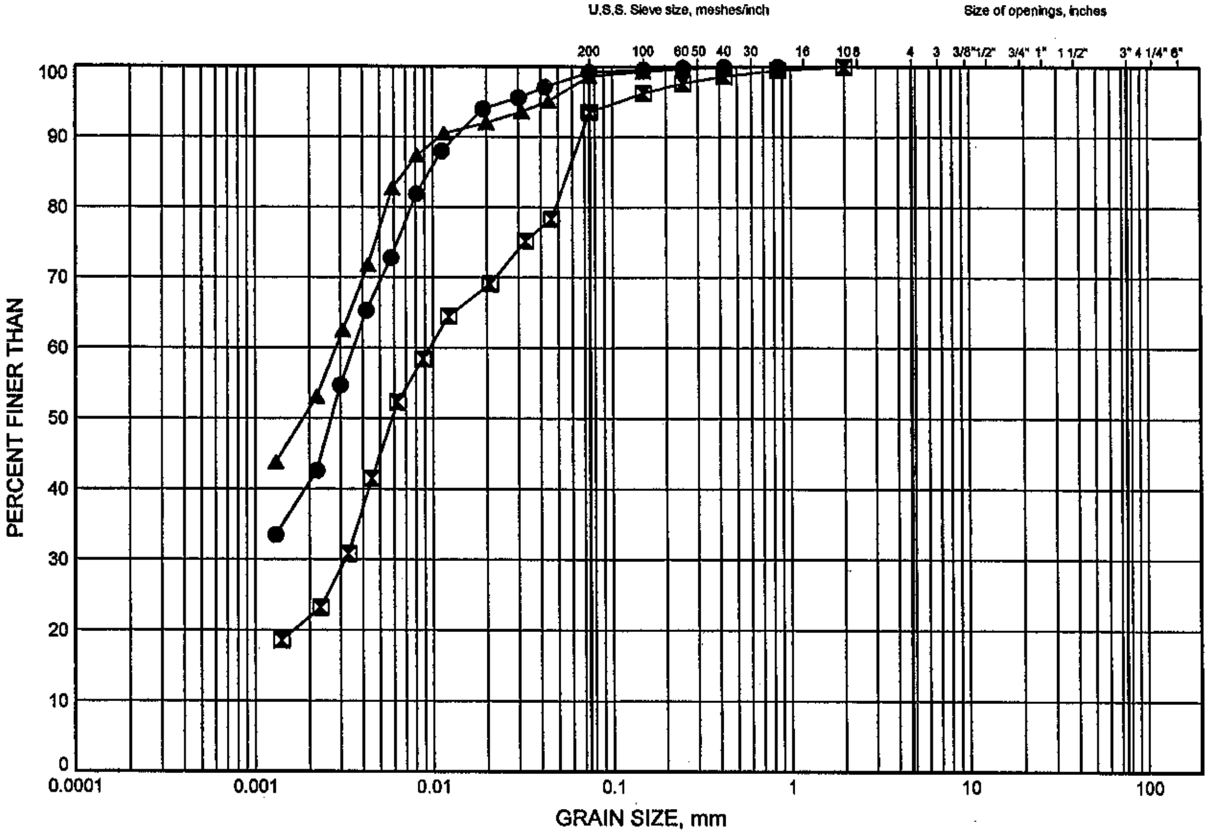
THURBER

GRAIN SIZE DISTRIBUTION - THURBER 6158.GPJ 11/19/10

DEER CREEK (FROOD) BRIDGE
GRAIN SIZE DISTRIBUTION

FIGURE B2

SILTY CLAY



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	DCR10-01	2.59	236.61
☒	DCR10-04	2.59	236.51
▲	DCR10-06	1.83	238.07

GRAIN SIZE DISTRIBUTION - THURBER 6158.GPJ 11/19/10

W.P.# 5236-05-01
Prepared By AN
Checked By LRB

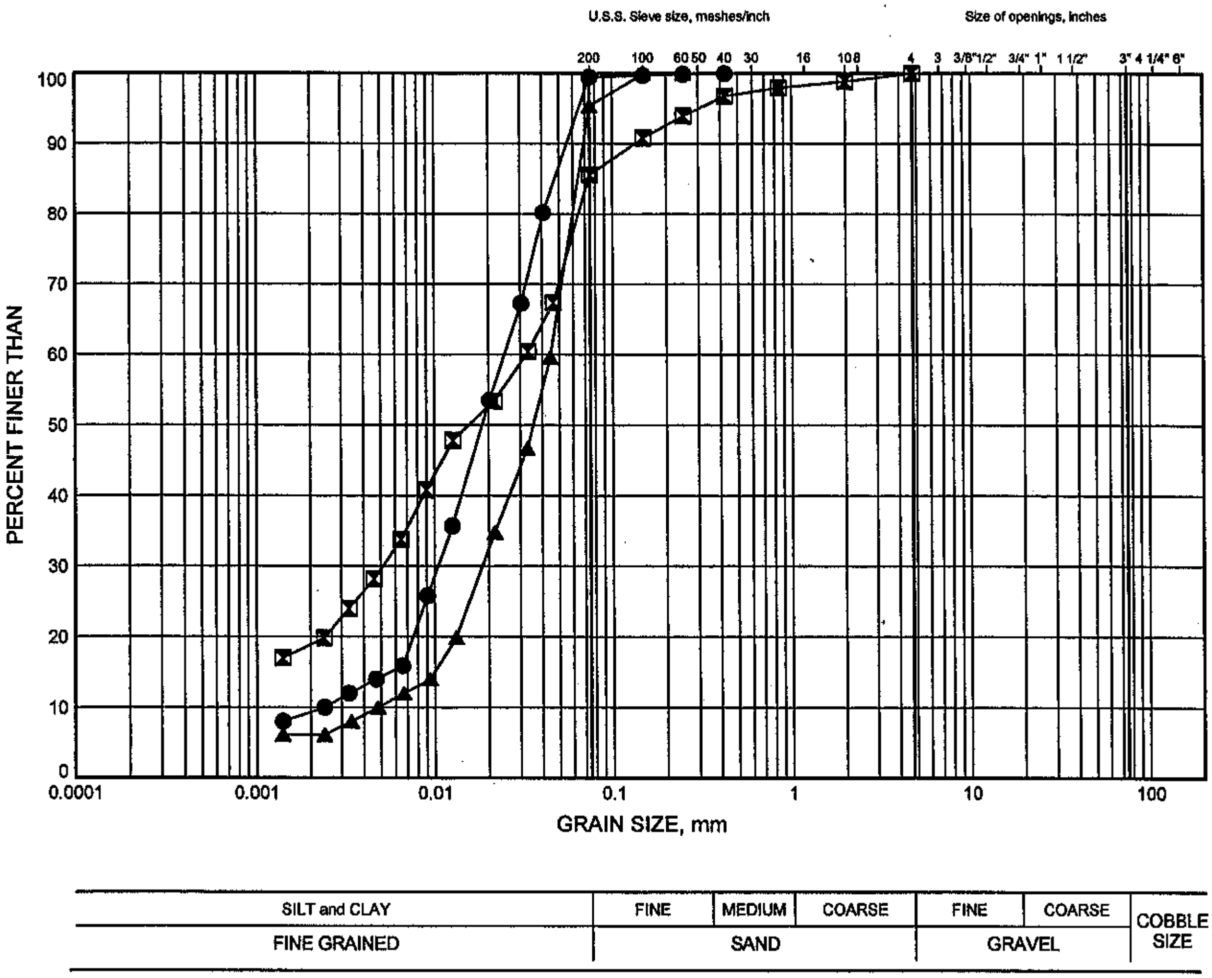
THURBER

DEER CREEK (FROOD) BRIDGE

GRAIN SIZE DISTRIBUTION

FIGURE B3

SILT



LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	DCR10-04	6.40	232.70
☒	DCR10-05	3.35	235.85
▲	DCR10-06	6.40	233.50

GRAIN SIZE DISTRIBUTION - THURBER 6158.GPJ 11/19/10

W.P.# 5236-05-01
Prepared By AN
Checked By LRB

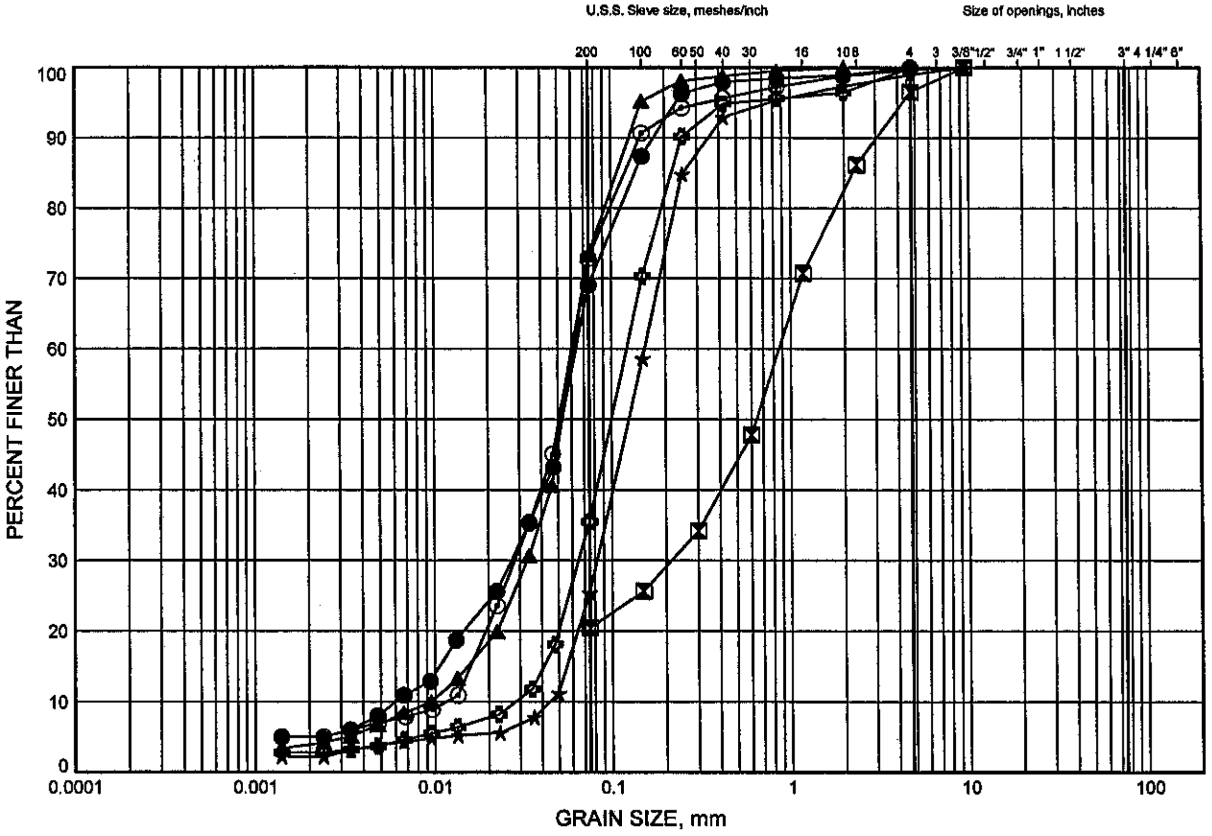
THURBER

DEER CREEK (FROOD) BRIDGE

GRAIN SIZE DISTRIBUTION

FIGURE B4

SANDY SILT to SILTY SAND



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	DCR10-01	6.40	232.80
⊠	DCR10-02	4.88	234.22
▲	DCR10-02	6.40	232.70
★	DCR10-03	6.40	232.70
⊙	DCR10-04	9.45	229.65
✿	DCR10-05	9.45	229.75

W.P.# 5236-05-01

Prepared By AN

Checked By LRB

THURBER

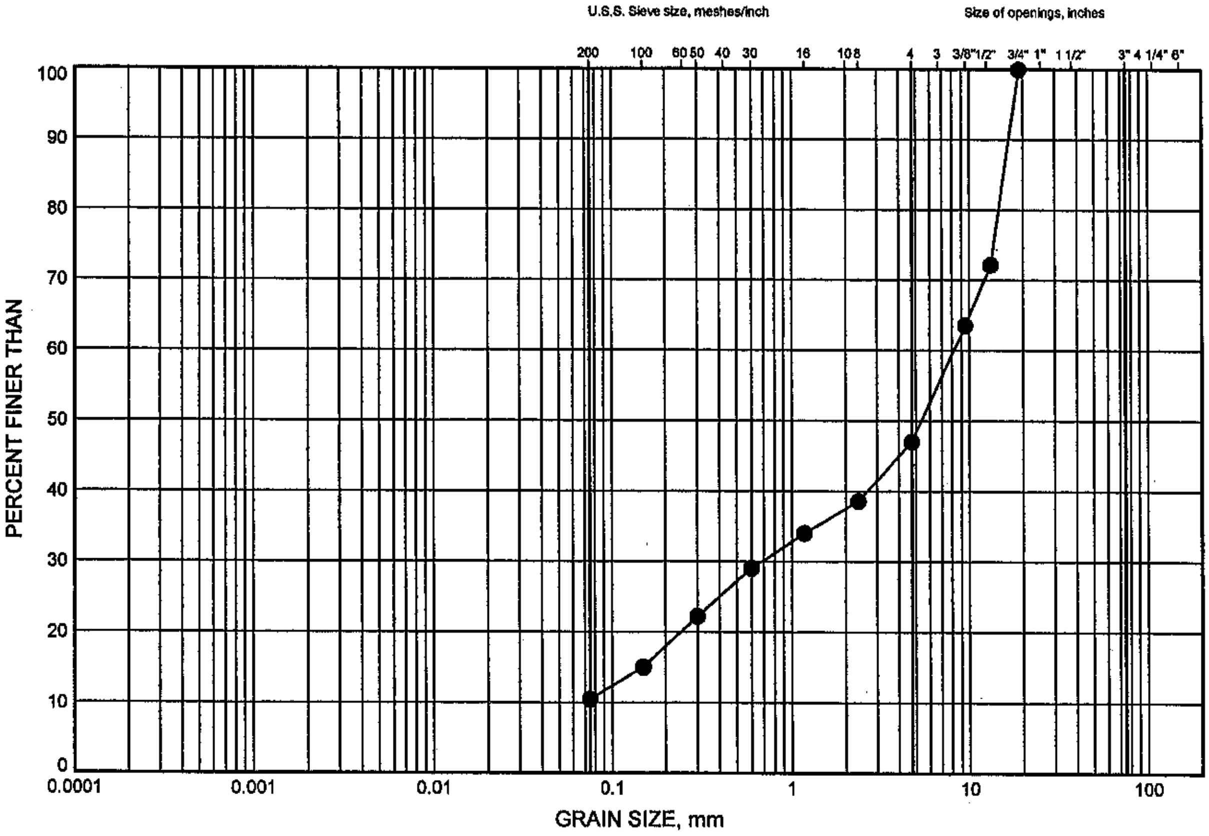
GRAIN SIZE DISTRIBUTION - THURBER 6158.GPJ 11/19/10

DEER CREEK (FROOD) BRIDGE

GRAIN SIZE DISTRIBUTION

FIGURE B5

SAND & GRAVEL



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	DCR10-05	14.55	224.65

W.P.# 5236-05-01

Prepared By AN

Checked By LRB

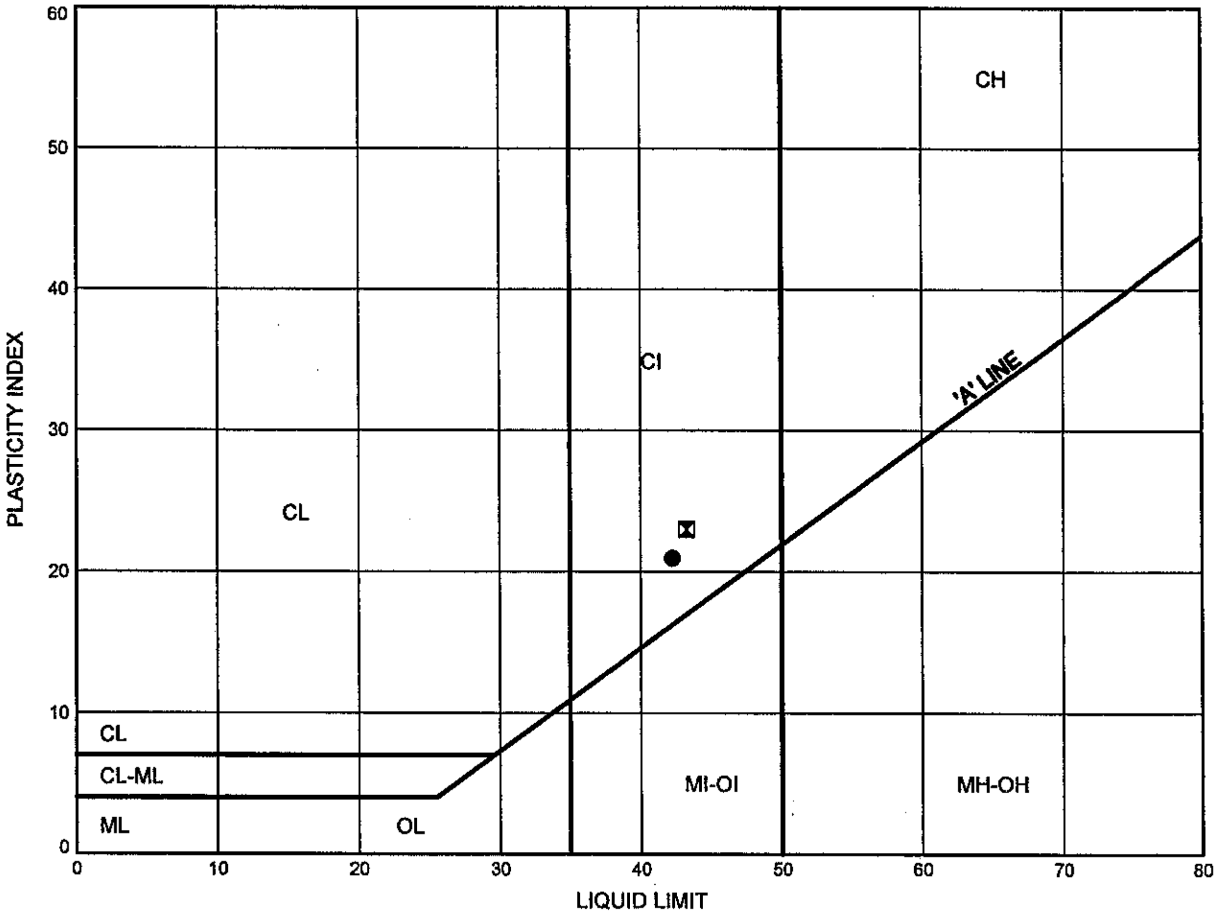
THURBER

DEER CREEK (FROOD) BRIDGE

ATTERBERG LIMITS TEST RESULTS

FIGURE B6

SILTY CLAY



SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	DCR10-01	2.59	236.61
☒	DCR10-06	1.83	238.07

THURBALT 5158.GPJ 11/19/10

Date November 2010

Project 5236-05-01

THURBER

Prep'd AN

Chkd. LRB

TABLE 1 - Point Load Test Results

MMM GROUP LIMITED

DEER CREEK (FROOD) BRIDGE

19-5161-58

9-Jun-10

DCR 10-2	DEPTH			FORCE (kN)	AXIAL / DIAMETRIC	DIAMETER (mm)	LENGTH (mm)	Is (MPa)	Is50 (MPa)	BREAK	UCS (Mpa)	ROCK TYPE
	FT.	IN.	METERS									
RUN #1	32	2	9.80	24.2	D	46.88	176.00	11.011	10.697	ok	256.72	Granite
RUN # 2	34	5	10.49	25.7	D	47.06	138.74	11.605	11.292	ok	271.02	Granite
RUN #3	38	11	11.86	16.2	D	46.90	164.00	7.365	7.156	ok	171.74	Granite

CONCLUSIONS

	AVERAGE	MAX	MIN
RUN #1:	257	257	257
RUN #2:	271	271	271
RUN #3:	172	172	172

DCR 10-3	DEPTH			FORCE (kN)	AXIAL / DIAMETRIC	DIAMETER (mm)	LENGTH (mm)	Is (MPa)	Is50 (MPa)	BREAK	UCS (Mpa)	ROCK TYPE
	FT.	IN.	METERS									
RUN #5	35	9	10.90	22.8	D	47.16	122.08	10.251	9.985	ok	239.65	Granite
RUN # 6	40	10	12.45	21.5	D	47.03	130.70	9.720	9.456	ok	226.95	Granite
RUN #7	46	0	14.02	8.2	D	47.08	142.60	3.699	3.601	ok	86.42	Granite
Re-Test	46	5	14.15	21.8	D	47.13	155.00	9.814	9.557	ok	229.36	Granite

CONCLUSIONS

	AVERAGE	MAX	MIN
RUN #5:	240	240	240
RUN #6:	227	227	227
RUN #7:	158	229	86

DCR 10-4	DEPTH			FORCE (kN)	AXIAL / DIAMETRIC	DIAMETER (mm)	LENGTH (mm)	Is (MPa)	Is50 (MPa)	BREAK	UCS (Mpa)	ROCK TYPE
	FT.	IN.	METERS									
RUN #1	47	0	14.33	24.6	D	46.58	168.00	11.338	10.982	ok	263.57	Granite
RUN # 2	52	9	16.08	20.6	D	46.92	146.74	9.357	9.093	ok	218.24	Granite
RUN #3	54	0	16.46	19.8	D	46.99	138.92	8.967	8.720	ok	209.28	Granite

CONCLUSIONS

	AVERAGE	MAX	MIN
RUN #1:	264	264	264
RUN #2:	218	218	218
RUN #3:	209	209	209

DCR 10-5	DEPTH			FORCE (kN)	AXIAL / DIAMETRIC	DIAMETER (mm)	LENGTH (mm)	Is (MPa)	Is50 (MPa)	BREAK	UCS (Mpa)	ROCK TYPE
	FT.	IN.	METERS									
RUN #3	43	4	13.21	17.9	D	46.99	127.20	8.107	7.883	ok	189.20	Granite
RUN # 4	50	11	15.52	21.2	D	46.83	154.00	9.667	9.386	ok	225.27	Granite
RUN #5	54	2	16.51	21.5	D	46.79	164.00	9.820	9.532	ok	228.76	Granite
RUN # 6	57	8	17.58	24.9	D	46.85	176.00	11.344	11.017	ok	264.41	Granite

CONCLUSIONS

	AVERAGE	MAX	MIN
RUN #3:	189	189	189
RUN #4:	225	225	225
RUN #5:	229	229	229
RUN #6:	264	264	264

Appendix C

Foundation Comparison

COMPARISON OF FOUNDATION ALTERNATIVES FOR EACH FOUNDATION ELEMENT

Foundation Element	**Spread Footings**	**Spread Footings on Engineered Fill**	**Piles**	**Drilled Shafts**
Abutments	***Advantages:*** i. Generally less costly construction than deep foundation elements. ***Disadvantages:*** i. Low geotechnical resistance available due to variable and soft soils at the surface. ii. Shallow foundations near the edge of the river may be subject to scour. iii. Deeper excavations required as a result of scour protection will be difficult to dewater and maintain in an undisturbed condition. **NOT RECOMMENDED**	***Advantages:*** i. Generally less costly construction than deep foundation elements. ***Disadvantages:*** i. Better geotechnical resistance than spread footings on native, but still influenced by the variable and soft soils at the surface. ii. Shallow foundations on engineered fill pads near the edge of the river may be subject to scour. iii. Deeper excavations required as a result of scour protection will be difficult to dewater and maintain in an undisturbed condition. **NOT RECOMMENDED**	***Advantages:*** i. High geotechnical resistance may be developed by driving the piles into very dense soil or to bedrock. ii. Comparatively short abutment stem possible iii. Permits integral abutment design iv. Readily permits founding below the scour depth ***Disadvantages:*** i. Higher unit cost compared to footings. **RECOMMENDED**	***Advantages:*** i. High geotechnical resistance is potentially available using deep shafts ***Disadvantages:*** i.Higher costs compared to spread footings. Probably higher costs compared to driven piles. ii. Difficulties advancing through very dense sand and gravel containing cobbles and boulders. iii. High risk of not being able to maintain undisturbed shaft walls and base below the groundwater level. iv. An integral abutment design is not an available option v. It may be necessary to place concrete by tremie methods. vi. Base inspection is very difficult. **NOT RECOMMENDED**

Appendix D

Site Photographs

Photo 1. Looking west across Deer Creek (Frood) Bridge

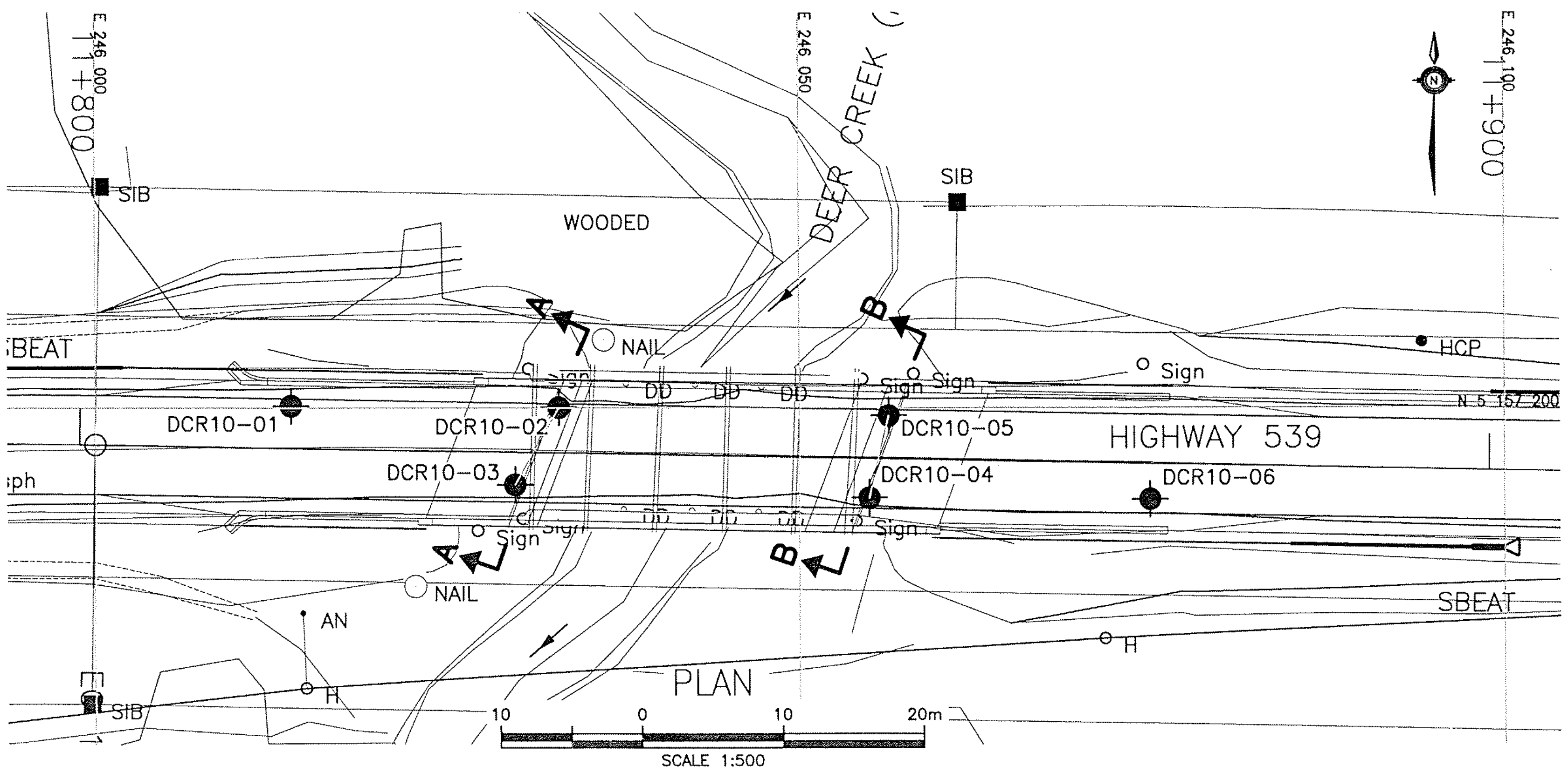
Photo 2. East approach to the Deer Creek (Frood) Bridge (looking west)

Photo 3. West approach to the Deer Creek (Frood) Bridge (looking west)

Appendix E

Drawings

MINISTRY OF TRANSPORTATION, ONTARIO



METRIC

DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

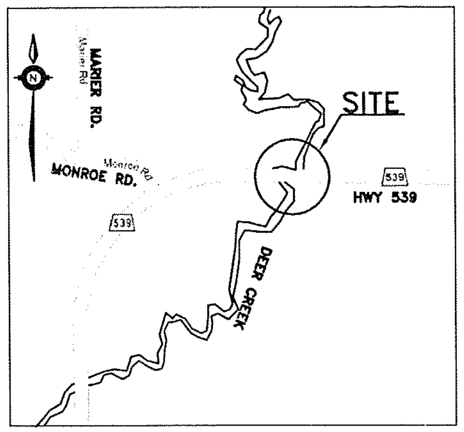
CONT No
WP No 5236-05-01

HIGHWAY 539
DEER CREEK
(FROOD) BRIDGE
BOREHOLE LOCATIONS AND SOIL STRATA

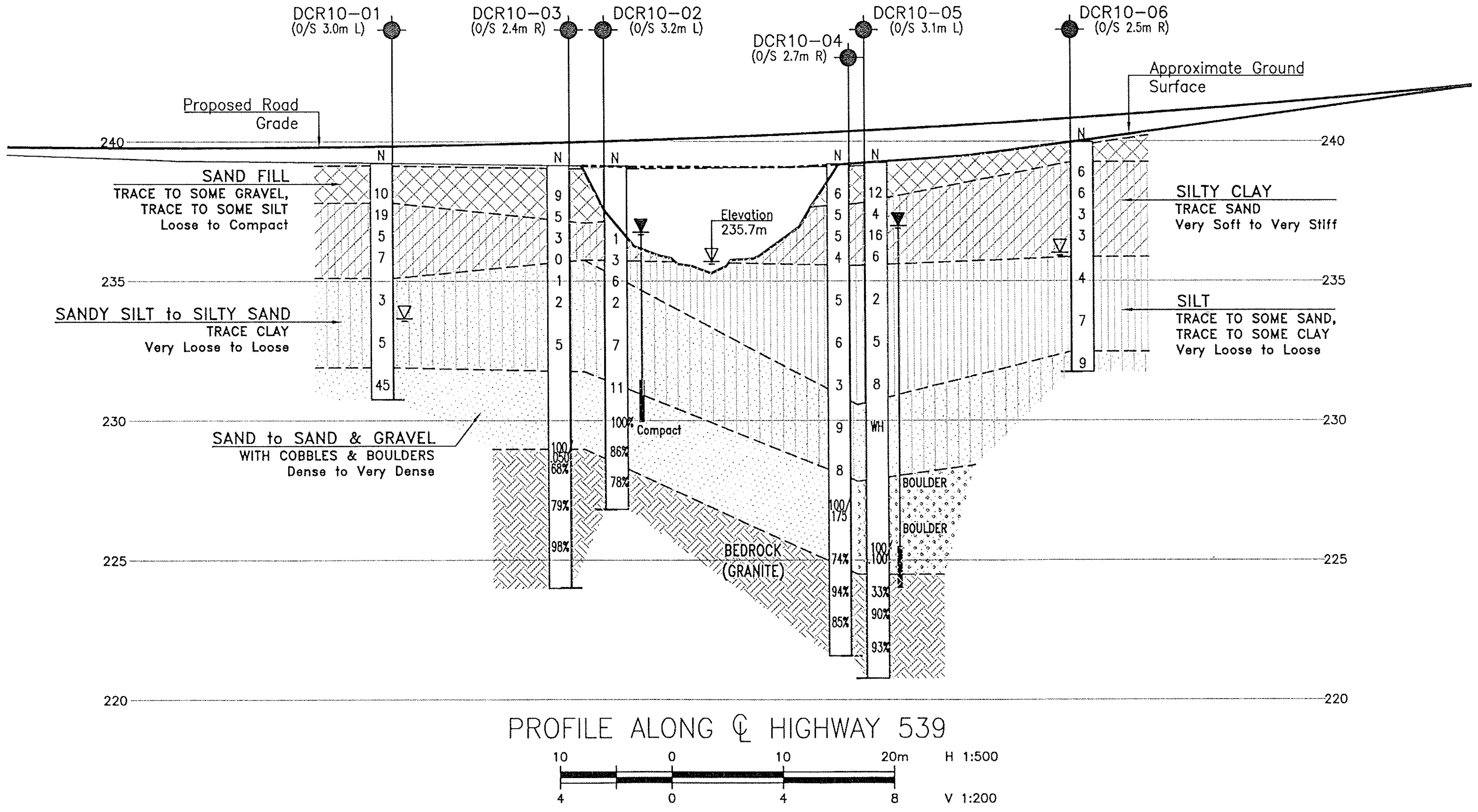
SHEET

MMM GROUP

THURBER ENGINEERING LTD.
GEOTECHNICAL • ENVIRONMENTAL • MATERIALS



KEYPLAN



A. E. GORMAN
APR 27/11

P. K. CHATTERJI
April 27/2011

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
▽	Water Level
▼	Head Artesian Water
┃	Piezometer
90%	Rock Quality Designation (RQD)
A/R	Auger Refusal

NO	ELEVATION	NORTHING	EASTING
DCR10-01	239.2	5 157 200.1	246 013.9
DCR10-02	239.1	5 157 200.0	246 032.9
DCR10-03	239.1	5 157 194.6	246 029.8
DCR10-04	239.1	5 157 199.5	246 054.9
DCR10-05	239.2	5 157 193.6	246 056.3
DCR10-06	239.9	5 157 193.5	246 074.8

-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. 41I-265

REVISIONS			
DATE	BY	DESCRIPTION	

DESIGN LRB	CHK	CODE	LOAD	DATE APR. 2011
DRAWN AN	CHK	SITE	STRUCT	DWG

FILENAME: H:\Drafting\19\5181\58\ted8158-DeerCreekBridge-Plan&Profile.dwg
PLOTDATE: Apr 26, 2011 - 3:08pm

METRIC

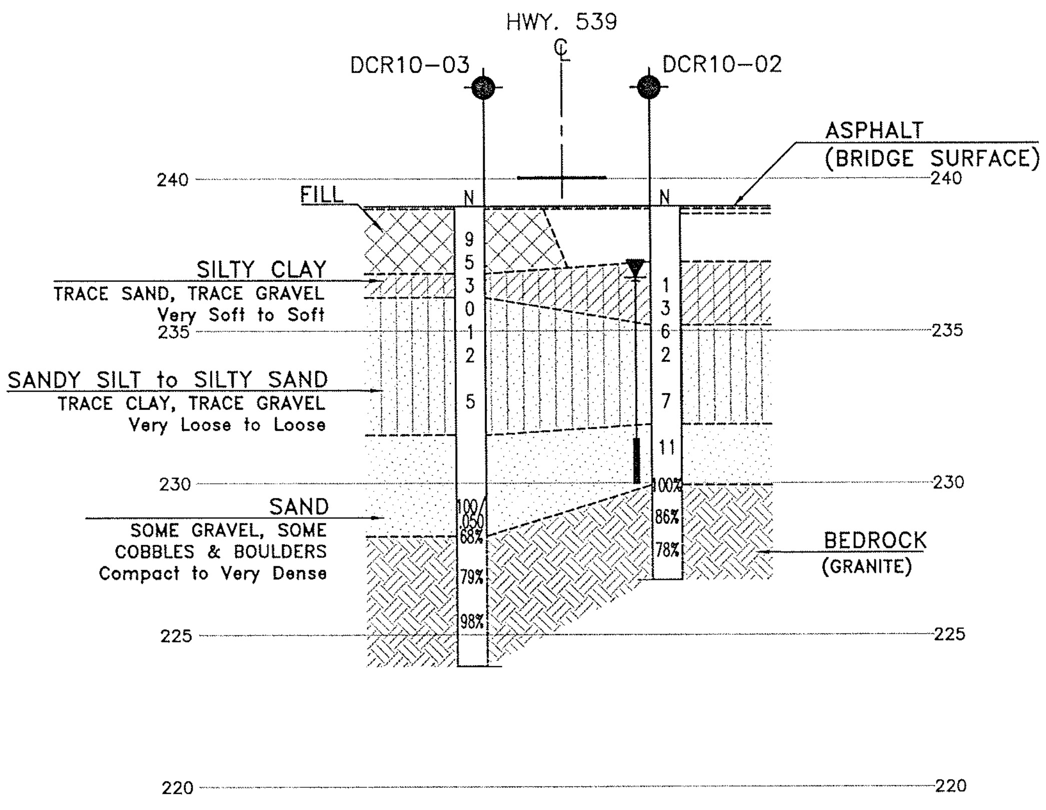
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

CONT No
WP No 5236-05-01

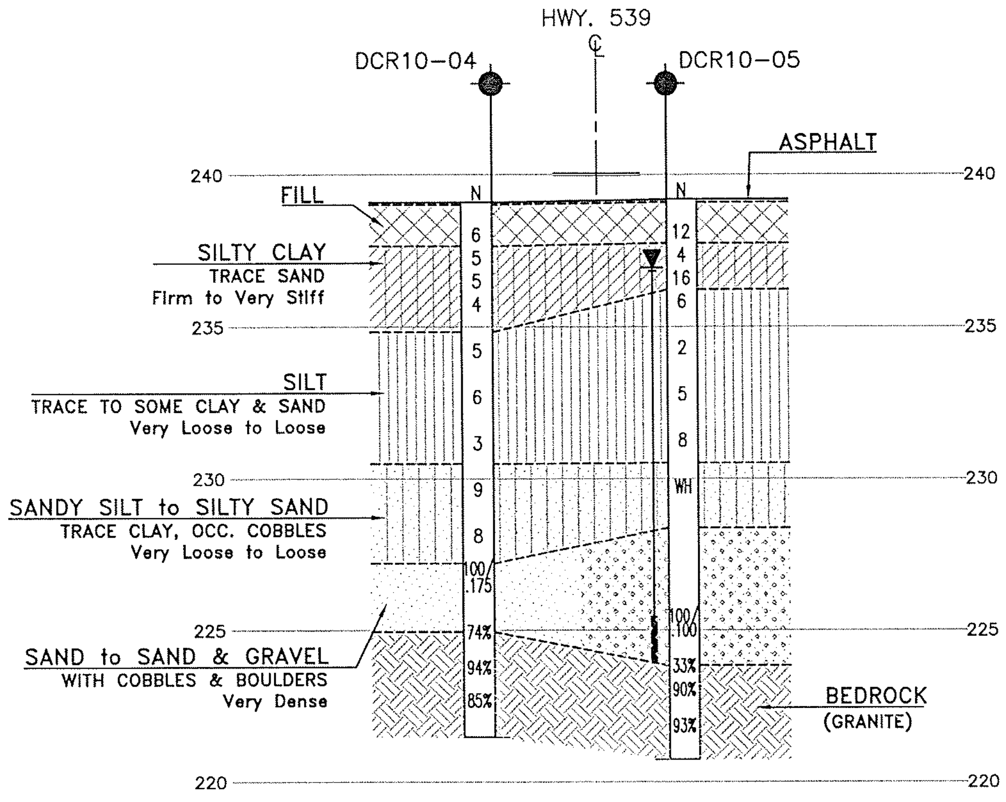
HIGHWAY 539
DEER CREEK
(FROOD) BRIDGE
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET

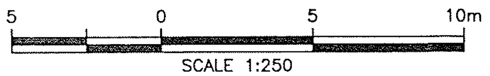
THURBER ENGINEERING LTD.
GEOTECHNICAL • ENVIRONMENTAL • MATERIALS



SECTION A–A



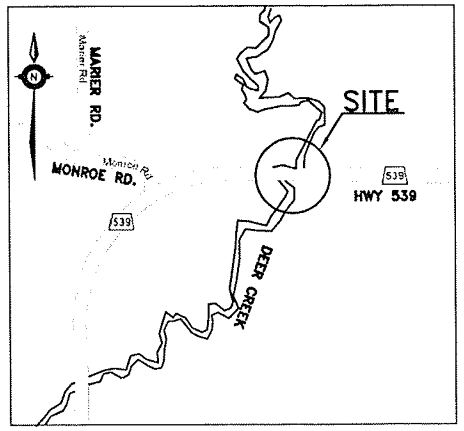
SECTION B–B



SCALE 1:250

A. E. GORMAN
APRIL 27/11

P. K. CHATTERJI
April 27/2011



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
∇	Water Level
▼	Head Artesian Water
┬	Piezometer
90%	Rock Quality Designation (RQD)
A/R	Auger Refusal

NO	ELEVATION	NORTHING	EASTING
DCR10-01	239.2	5 157 200.1	246 013.9
DCR10-02	239.1	5 157 200.0	246 032.9
DCR10-03	239.1	5 157 194.6	246 029.8
DCR10-04	239.1	5 157 199.5	246 054.9
DCR10-05	239.2	5 157 193.6	246 056.3
DCR10-06	239.9	5 157 193.5	246 074.8

-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. 41I-265

REVISIONS	DATE	BY	DESCRIPTION		
DESIGN LRB	CHK	CODE	LOAD	DATE APR. 2011	
DRAWN AN	CHK	SITE	STRUCT	DWG	

FILENAME: H:\Drafting\19\5161\58\ted8158-DeerCreekBridge-Plan&Profile.dwg
PLOTDATE: Apr 28, 2011 - 3:08pm

Appendix F

Technical References and Suggested Text for Selected NSSP

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

- OPSS 902
- OPSS 903
- OPSS 572
- OPSD 3101.150
- OPSD 3101.200
- SP 110F13 Amendment to OPSS 1010, March 1993
- SP 105S10

2. List of Canadian Highway Building Design Code References in this Report

- Clause 6.7.3
- Clause 6.7.4
- Clause 6.8.9.2
- Clause 6.9.3
- Clause 4.6.4
- Figure C6.9.1(a) – CHBDC Commentary
- Table 4.4.6.1

3. Suggested text for a NSSP on Unwatering

It must be assumed that excavations at this site may penetrate below the groundwater level.

The soils overlying the bedrock at this site are predominantly cohesionless and will be readily disturbed by unbalanced water heads or by flow of water.

The Contractor shall design, install and operate systems that shall:

i. Unwater the excavations

ii. Control the flow of groundwater, surface water and river water into the excavations

iii. Prevent the disturbance of the base of the excavation

iv. Prevent the sloughing of soil into the excavations.

Particular attention must be paid to the design of unwatering systems and shoring systems at the pier locations due to the proximity of the river and the cohesionless nature of the overburden.

The selection and design of suitable unwatering and shoring systems shall remain the responsibility of the Contractor. However, factors that might influence the selection and design of the unwatering system and the shoring system include, but are by no means limited to the probable level of the river during construction. The selected systems must prevent flooding of the work area due to rising river levels. It is recommended that the designs allow for a river level that will rise to Elevation xxx.xx.

4. Suggested text for a NSSP on Pile Driving

Steel H-piles driven at this site will meet refusal on bedrock or in soil containing cobbles and boulders. All driven piles shall be fitted with cast steel, H-section rock points from an approved manufacturer such as Titus Steel (Standard H-point) or approved equivalent.