# FOUNDATION INVESTIGATION AND DESIGN REPORT
# McCAULEY CREEK BRIDGE REPLACEMENT
# HIGHWAY 11
# WEST OF THE TOWNSHIP OF ATIKOKAN
# DISTRICT OF RAINY RIVER, ONTARIO

**W.P. 6043-08-00, Site No. 45-106**

**Geocres Number: 52B-14**

**Report to**

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**PART 1: FACTUAL INFORMATION**

## 1 INTRODUCTION

This report presents the factual findings obtained from a foundation investigation conducted at the site of a proposed replacement of the existing bridge structure which carries Highway 11 over McCauley Creek, west of The Township of Atikokan, Ontario.

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

Thurber carried out the investigation as a sub-consultant to Hatch Mott MacDonald, under the Ministry of Transportation Ontario (MTO) Agreement Number 6010-E-0010.

## 2 SITE DESCRIPTION

The McCauley Creek Bridge is located on Highway 11 approximately 22 km west of The Township of Atikokan, Ontario and approximately 15 km east of Flanders, Ontario in the Rainy River District.

At present, the highway crosses the McCauley Creek on a seven-span structure supported on timber piles. The end spans are 4.0 m each and the interior spans are 5.0 m each. The total length of the bridge is 33 m and the width is 9.75 m. The McCauley Creek flows to the north.

The surrounding area near the site is relatively flat. The areas to the east and west of the site are treed.

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

The site lies within the physiographic region known as the Quetico Subprovince of the Superior Province of the Canadian Shield. The region is characterized by Precambrian meta-volcanic and meta-sedimentary rocks intruded by later stage diabase dykes. In some areas the Precambrian rocks are covered by sedimentary rocks of the Huronian Supergroup. The bedrock is mantled by glaciolacustrine varved clays and sand and gravel deposits.

## 3 SITE INVESTIGATION AND FIELD TESTING

The site investigation and field testing for this project was carried out from July 13 to 16, 2011 and consisted of drilling and sampling six boreholes (numbered MCB-01 to MCB-06) through the highway embankment in the area of the existing west and east approaches and abutments. Boreholes MCB-02 to MCB-05 were drilled at the two abutments and advanced within the overburden to depths ranging from 11.6 m to 13.5 m (elevations 371.2 to 373.2) where the drill rig encountered refusal. Boreholes MCB-01 and MCB-06 were drilled at the west and east approaches, respectively, and terminated at 11.4 m and 9.6 m depths (elevations 373.3 and 375.1) upon refusal. Bedrock was proved in Boreholes MCB-02 and MCB-05 by NQ size diamond coring. Boreholes MCB-02 and MCB-05 were advanced 3.1 m and 3.4 m into bedrock and terminated at 14.7 m and 15.0 m depths (elevations 370.1 and 369.8).

Boreholes were supplemented by dynamic cone penetration testing (DCPT) conducted adjacent to each borehole. The depths to the DCPT ranged from 9.7 m to 13.4 m (elevations 371.3 to 375.1).

The approximate locations of the boreholes are shown on the attached Borehole Locations and Soil Strata Drawing in Appendix G.

The borehole locations were marked in the field and utility clearances were obtained prior to drilling. Road occupancy permits were obtained for boreholes drilled on the existing Highway 11 platform.

The drilling was carried out from the highway grade using a CME75 truck-mounted drill rig. A combination of hollow-stem auger drilling techniques, casing and coring methods were used to advance the boreholes. Overburden samples were obtained at selected intervals using a split spoon sampler in conjunction with Standard Penetration Testing (SPT). In situ vane shear testing was carried out to assess the undrained shear strength of soft to firm cohesive deposits.

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

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

Groundwater conditions in the open boreholes were observed throughout the drilling operations. Two standpipe piezometers consisting of 19 mm PVC pipe with slotted screen were installed in Boreholes MCB-03 and MCB-04, and enclosed in filter sand to permit longer term groundwater level monitoring. The boreholes were backfilled with bentonite holeplug in general accordance with O.Reg. 903 upon completion. The location and completion details of the piezometers and boreholes are shown in Table 3.1.

**Table 3.1 – Borehole Abandonment Details**

| Location | Borehole | Piezometer Tip Depth/ Elevation (m) | Abandonment Details |
|---|---|---|---|
| West approach | MCB-01 | None installed | Borehole backfilled with auger cuttings to 0.1 m, then asphalt to surface. |
| West abutment | MCB-02 | None installed | Borehole backfilled with holeplug from 14.7 m to 1.5 m, sand and gravel from 1.5 m to 0.1 m, then asphalt to surface. |
| | MCB-03 | 13.5/371.2 | Sand from 13.5 m to 11.3 m, holeplug from 11.3 m to 10.5 m, auger cuttings from 10.5 m to 0.2 m, then asphalt to surface. |
| East abutment | MCB-04 | 12.3/372.5 | Sand from 12.3 m to 9.8 m, holeplug from 9.8 m to 0.2 m, then asphalt to surface. |
| | MCB-05 | None installed | Borehole backfilled with holeplug from 15.0 m to 9.1 m, auger cuttings to 0.6 m, holeplug to 0.1 m, then asphalt to surface. |
| East approach | MCB-06 | None installed | Borehole backfilled with holeplug from 9.6 m to 1.5 m, sand and gravel from 1.5 m to 0.9 m, concrete from 0.9 m to 0.1 m, then asphalt to surface. |

## 4 LABORATORY TESTING

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

Point load tests were carried out on selected samples of intact bedrock to assist in evaluation of the compressive strength of the bedrock. Results of point load tests on the rock core samples are included in Appendix B and on the Record of Borehole sheets in Appendix A.

## 5 DESCRIPTION OF SUBSURFACE CONDITIONS

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

In general terms, the stratigraphy encountered at this site consists of pavement structure overlying granular fill (embankment fill). Peat was encountered below the fill in one borehole location at the east approach. Layers of native silt and sand were contacted below the fill and peat. The native silt and sand are underlain by silty clay and sand and silt layers. Slightly weathered to fresh, grey, arkose/sandstone bedrock was contacted below the sand and silt layers at depths ranging from 9.6 m to 13.5 m.

More detailed descriptions of the individual strata are presented below.

### 5.1 Pavement structure

Pavement structure was encountered in all the boreholes drilled at this site. The boreholes were drilled through the existing Highway 11 shoulders. The pavement structure consists of approximately 50 mm to 150 mm of asphalt overlying granular fill.

### 5.2 Fill

Fill was contacted below the asphalt pavement in all the boreholes. The fill generally consists of brown sand containing some gravel, trace to some silt and clay. In Borehole MCB-06, drilled at the east approach, the fill consisted of brown sand and gravel. Cobbles and boulders were encountered within the fill in Boreholes MCB-01 and MCB-06, drilled at the west and east approaches, respectively. A layer of gravel fill was contacted at 1.5 m depth (elevation 383.2) in Borehole MCB-01. The thickness of the fill ranged from 1.4 m to 4.1m.

The depth to the base of the fill varied from 1.4 m to 4.1m (elevations 380.6 to 383.4).

SPT 'N' values recorded in the cohesionless fill ranged from 14 to 54 blows per 0.3 m of penetration, indicating a compact to very dense relative density. A low SPT 'N' value of 7 blows per 0.3 m of penetration, indicating a loose relative density, was recorded in Borehole MCB-02 near elevation 383.0. In Borehole MCB-01, SPT 'N' values of 50 blows per 0.1 m of penetration were measured within the upper 1.0 m of sand fill. These values indicate a very dense relative density and might reflect the presence of cobbles.

The moisture content of the fill ranged from 3% to 18%.

Grain size distribution curves for samples of the fill tested are presented on the Record of Borehole sheet and on Figures B1 and B2 of Appendix B. The results of the laboratory test are summarized as follows:

| Soil Particles | Sand fill (%) | Gravel fill (%) |
|---|---|---|
| Gravel | 9 | 81 |
| Sand | 71 | 17 |
| Silt and Clay | 20 | 2 |

### 5.3 Peat

Dark brown peat containing some roots and trace of gravel was contacted below the sand fill at 3.4 m depth (elevation 381.4) in Borehole MCB-06, drilled at the east approach. The thickness of the peat was 700 mm.

The depth to the base of the peat was 4.1 m (elevation 380.6).

SPT 'N' value recorded in the peat was 5 blows per 0.3 m penetration indicating a loose relative density.

The moisture content of the peat was 169%.

### 5.4 Sand

Native brown to grey sand containing trace gravel, trace to some silt and clay and occasional cobbles was contacted in Boreholes MCB-02 to MCB-05, drilled at the abutments, at depths ranging from 1.4 m to 2.3 m (elevations 382.5 to 383.4). A 900-mm thick layer of sand mixed with organics was encountered at 3.7 m depth (elevation 381.1) in Borehole MCB-04. The thickness of the native sand layer ranged from 1.8 m to 3.3 m.

The depth to the base of the sand ranged from 3.5 m to 4.7 m (elevations 380.0 to 381.2).

SPT 'N' values recorded in the native sand layer ranged from 4 to 17, indicating a loose to compact relative density. A SPT 'N' value of 1 blow per 0.3 m of penetration was measured in Borehole MCB-02 near elevation 381.5, indicting a very loose relative density.

The moisture contents of samples from the sand generally vary between 15% and 22%.

Grain size distribution curves for the sand samples tested are presented in Figure B3 in Appendix B. The results of the laboratory test are summarized as follows:

| Soil Particles | (%) |
|---|---|
| Gravel | 2 to 6 |
| Sand | 81 to 88 |
| Silt and Clay | 10 to 13 |

### 5.5 Silt

Grey silt containing some clay and trace to some sand was contacted below the fill at 4.1m depth (elevation 380.6) in Borehole MCB-01 and below the native sand at 3.5 m depth (elevation 381.2) in Borehole MCB-03. The thickness of the silt was 0.6 m and 1.2 m.

The depths to the base of the silt were 4.7 m and 4.9 m (elevations 380.0 and 379.8), in Boreholes MCB-01 and MCB-03, respectively.

Standard Penetration tests in the silt layer gave SPT 'N' values of 5 and 8 blows per 0.3 m of penetration, indicating a loose relative density.

The moisture contents of samples from the silt layer were 38% and 39%.

### 5.6 Silty Clay

Native reddish brown to grey silty clay containing trace sand and occasional wood fibres was contacted below the silt in Boreholes MCB-01 and MCB-03, below the sand in Boreholes MCB-02, MCB-04 and MCB-05 and below the peat in Borehole MCB-06. The native silty clay was generally encountered at depths ranging from 4.1 m to 4.9 m (elevations 379.8 to 380.7). The thickness of the silty clay ranged from 4.6 m to 7.5 m.

The depth to the base of the silty clay was from 8.7 m to 12.4 m (elevations 372.3 to 376.1).

SPT 'N' values recorded in the silty clay ranged from 0 to 11 blows for 0.3 m of penetration, indicating a very soft to stiff consistency. Typically, N-values in the native silty clay were 0 to 2 blows for 0.3 m penetration. In situ vane shear tests performed during drilling indicated undrained shear strengths ranging from 16 to 32 kPa. Locally in Boreholes MCB-01 and MCB-02, the undrained shear strength was 40 kPa and 48 kPa near elevation 377.8.

The moisture content of samples collected from the silty clay layer generally varies between 28% and 82%.

Grain size distribution curves for selected silty clay samples are presented in Appendix B, Figures B4 and B5. The results are also summarized on the Record of Borehole sheets included in Appendix A. Atterberg Limits test results are presented in Figures B7 and B8 of Appendix B. The results of the laboratory tests are summarized as follows:

| Soil Particles | Percentage (%) |
|---|---|
| Gravel | 0 |
| Sand | 0 to 3 |
| Silt | 44 to 70 |
| Clay | 28 to 56 |

| Index Property | Percentage (%) |
|---|---|
| Liquid Limit | 30 to 60 |
| Plastic Limit | 22 to 23 |

The above results show that the silty clay is of medium to high plasticity with group symbols of CI-CH. One sample from Borehole MCB-04 revealed low plasticity with a group symbol of CL.

### 5.7 Sand and silt

A layer of grey sand and silt containing trace to some gravel and trace clay was contacted in all the boreholes below the silty clay at depths ranging from 8.7 m to 12.4 m (elevations 372.3 to 376.1). The thickness of the sand and silt layer varied from 0.4 m to 1.8 m.

Boreholes were terminated within sand and silt layer upon refusal on bedrock at depths ranging from 9.6 m to 13.5 m (elevations 371.2 to 375.1).

SPT 'N' values recorded in the sand and silt layer ranged from 5 to 20 blows per 0.3 m of penetration, indicating a loose to compact relative density. SPT 'N' values of 33 and 47 blows per 0.3 m of penetration, indicating dense to very dense relative density, were measured in Borehole MCB-01 and MCB-06 drilled at the approaches.

The moisture contents of samples from the sand generally vary between 19% and 24%. A high moisture content of 52% was measured in Borehole MCB-02, near elevation 374.2.

Grain size distribution curves for the sand samples tested are presented in Figure B6 in Appendix B. The results of the laboratory test are summarized as follows:

| Soil Particles | (%) |
|---|---|
| Gravel | 0 |
| Sand | 42 |
| Silt | 56 |
| Clay | 2 |

### 5.8 Bedrock

The overburden soils described above are underlain by grey metasedimentary bedrock described as arkose/sandstone. The bedrock was slightly weathered to fresh. Occasional mechanical breaks and sub-vertical fractures were noted throughout the bedrock cores.

Bedrock was proved by coring at two boreholes. Table 5.1 summarizes depths and elevations to the top of bedrock and auger refusal on probable bedrock in the boreholes.

**Table 5.1 – Depths and Elevations of Top of Bedrock and auger refusal on probable bedrock**

| Borehole | Top of Bedrock/Auger refusal | |
|---|---|---|
| | Depth (m) | Elevation (m) |
| MCB-01 | 11.4 | 373.3 |
| MCB-02* | 11.6 | 373.2 |
| MCB-03 | 13.5 | 371.2 |
| MCB-04 | 12.3 | 372.5 |
| MCB-05* | 11.6 | 373.2 |
| MCB-06 | 9.6 | 375.1 |

* Bedrock proved by coring.

Core recovery in the bedrock was 97% in one core and 100% in the remaining cores. The RQD values ranged from 75% to 100%, indicating fair to excellent rock quality. In Borehole MCB-02 Run 1 the RQD value was 7%, indicating a very poor rock quality.

The Fracture Index (FI) of the rock, expressed as fractures per 0.3 m of core, was generally less than 4.

The estimated unconfined compressive strength of the rock cores ranged from 59 MPa to 207 MPa, indicating a strong to very 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 Appendix B.

### 5.9 Water Levels

Water levels were monitored in the boreholes during and upon completion of drilling. Two standpipe piezometers were installed in Boreholes MCB-03 and MCB-04, drilled at the west and east abutments to monitor water levels after completion of drilling. The water levels measured in the piezometers and open boreholes are summarized in Table 5.2.

**Table 5.2 – Water Level Measurements**

| Foundation Unit | Borehole | Date | Water Level (m) | | Comments |
|---|---|---|---|---|---|
| | | | Depth | Elevation | |
| West approach | MCB-01 | July 14, 2011 | 1.8 | 382.9 | Open borehole |
| West abutment | MCB-02 | July 15, 2011 | 1.9 | 382.9 | Open borehole |
| | MCB-03 | September 18, 2011 | 1.9 | 382.8 | In piezometer |
| East abutment | MCB-04 | August 27, 2011<br>September 18, 2011 | 2.0<br>1.9 | 382.8<br>382.9 | In piezometer |
| East approach | MCB-06 | July 16, 2011 | 3.0 | 381.8 | Open borehole |

Piezometric readings indicate that water level is near elevation 382.9.

Preliminary GA drawing indicates that water level in the McCauley Creek was near Elevation 382.5 on October 23, 2009.

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

## 6 MISCELLANEOUS

Borehole locations were selected and established in the field by Thurber Engineering Ltd. Hatch Mott MacDonald provided the co-ordinates and the ground surface elevations.

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

Eastern Ontario Diamond Drilling Ltd. from 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 Ms. Eckie Siu of Thurber.

Routine laboratory testing was carried out by Thurber Engineering Ltd.

Overall planning and supervision of the field program was conducted by Mr. Mark Farrant, P. Eng.

Interpretation of the data and preparation of the report were carried out by Ms. R. Palomeque Reyna, P.Eng.

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

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**PART 2: ENGINEERING DISCUSSION AND RECOMMENDATIONS**

## 7 GENERAL

This report presents interpretation of the geotechnical data in the factual report and presents geotechnical recommendations for design of a new bridge to replace the existing bridge located at the crossing of Highway 11 over McCauley Creek, west of The Township of Atikokan, Ontario.

The McCauley Creek bridge was constructed in 1962 and underwent rehabilitation in 1986. At present, Highway 11 crosses McCauley Creek on a seven-span structure supported on timber piles. The length of the bridge is approximately 34.0 m. The highway grade is near elevation 384.8. The existing embankment heights are approximately 2.0 m to 4.0 m. It is understood that the existing structure will be removed.

Based on the preliminary General Arrangement (GA) drawing provided by Hatch Mott MacDonald, a single-span structure supported on two abutments is proposed. GA drawings show that the abutments are proposed to be founded on driven steel H-piles. A sheet pile wall will be driven just behind the H-piles to retain the approach fill. Precast pre-stressed box girders will be spanning the abutment pile caps to support the deck finishing. The total length of the structure will be reduced from 34.0 m to 21.0 m. The proposed structure will be approximately 10.25 m wide. The existing structure will be replaced maintaining the same alignment for the new structure. The highway grade will be raised approximately 400 mm.

As the length of the new bridge will be shortened, placement of new fill behind the sheet pile wall will be required at each abutment.

The discussion and recommendations presented in this report are based on the information provided by Hatch Mott MacDonald and on the factual data obtained in the course of the investigations.

## 8 STRUCTURE FOUNDATIONS

In general terms, the stratigraphy encountered at this site consists of pavement structure (asphalt) over compact to very dense embankment fill (sand, sand/gravel) overlying native loose to compact sand and silt layers. A 700-mm thick layer of peat was encountered below the fill in one borehole drilled near the east approach. A deposit of very soft to stiff silty clay was encountered below the sand, silt and peat layers. The thickness of the silty clay ranged from 4.6 m to 7.5 m. Loose to very dense sand and silt were contacted below the silty clay. The overburden soils are underlain by slightly weathered to fresh, grey arkose/sandstone bedrock. Bedrock and auger refusal were encountered at 9.6 m to 13.5 m depth across the area of investigation.

Piezometric reading indicates that water level is near elevation 382.9. Preliminary GA drawing indicates that water level in the McCauley Creek was near Elevation 382.5 on October 23, 2009.

Geotechnical recommendations for design of the proposed H-pile foundation system are presented in the following sections. Foundation alternatives together with corresponding geotechnical design parameters for feasible options are also presented in the event that the foundation concept changes.

A comparison of the technical advantages and disadvantages of alternative foundation schemes (driven steel H-piles, spread footings on native soil, and caissons/drilled shafts) is presented in Appendix C. A foundation scheme preferred from a foundations perspective is recommended.

### 8.1 Steel H-Pile Foundations

The subsurface conditions at the north and south abutments are considered suitable for the design of foundations supported on steel H-piles driven to achieve resistance on bedrock.

The elevations at which the piles are expected to encounter bedrock are given in Table 8.1.

**Table 8.1– Estimated Pile Tip Elevation**

| Foundation Unit | Borehole | Anticipated Pile Length below base of deck (1) (m) | Anticipated Pile Tip Elevation on Bedrock |
|---|---|---|---|
| West abutment | MCB-02 | 11.6 | 373.2 |
| | MCB-03 | 13.6 | 371.2 |
| East abutment | MCB-04 | 12.3 | 372.5 |
| | MCB-05 | 11.6 | 373.2 |

(1) Base of deck is approximately at elevation 384.0.

The pile tip elevations shown in Table 8.1 should be used for estimating purposes only. The actual pile tip elevations will be controlled as described in Section 8.1.3 Pile Installation.

### 8.1.1 Axial Resistance

The axial, factored geotechnical resistance at Ultimate Limit States ($ULS_f$) for an H-Pile section 310x110 driven to refusal on bedrock is 2,000 kN.

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

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

### 8.1.2 Pile Tips

The tips of all piles should be fitted with cast steel, H-section rock points from an approved manufacturer such as Titus Steel (Standard H-point) or approved equivalent.

### 8.1.3 Pile Installation

Pile installation should be in accordance with OPSS 903, November 2009. For piles installed for the tolerances shown in Clause 903.07.05.01 of the Specification, the foundation drawing should include the note "Piles to be driven to bedrock".

We understand that the proposed bridge design may require that the deviation at the top of the pile be limited to 12 mm. To reduce the potential for misalignment resulting from hard driving to confirm bedrock, it is recommended that the pile driving note on the foundation drawing be modified as follows:

"Piles to be driven to bedrock. Upon initial contact with the bedrock:

1. Apply 10 blows at 10% of the hammer energy. Record the penetration.
2. Apply 10 blows at 50% of the hammer energy. If the penetration under 10 blows is less than 12.5 mm, the pile is set.
3. If the penetration under 10 blows is greater than 12.5 mm, refer the issue to the design team for resolution."

Use of a driving template or other means may also be required to achieve the specified maximum deviation.

As the existing bridge is supported on timber piles, the new abutments should be positioned to avoid interference between the new steel piles and the existing timber piles during installation. The existing timber piles should not be extracted near the new foundation area.

#### 8.1.4 Downdrag

Since the highway grade will be raised by 400 mm, downdrag forces will develop along the length of the pile embedded in the silty clay layer. For design purposes, an unfactored downdrag load of 250 kN per pile is recommended to evaluate the impact of downdrag.

This downdrag load should be multiplied by a load factor of 1.25 as per CHBDC Commentary Clause C8.6.4 to obtain a factored downdrag load.

In accordance with Section 6.8.4 of the CHBDC and clause C6.8.4 of the Commentary to CHBDC, in the structural design of a pile, the factored downdrag load should be added to the factored permanent loads to assess the effects of downdrag.

In geotechnical analysis of downdrag, live load effects should not be considered. The location of the neutral plane for a pile or groups of piles should be determined by using unfactored loads and unfactored geotechnical parameters.

Factored dead and downdrag load should not exceed the factored structural resistance of a pile.

#### 8.1.5 Lateral Resistance

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

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

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

where

$z$ = depth of embedment of pile in metres

$D$ = pile width in metres

$n_h$ = value from Table 8.2

$\gamma$ = unit weight (Table 8.2)

$K_p$ = passive earth pressure coefficient (Table 8.2)

For cohesive soils, the lateral resistance of the piles may be calculated as follows:

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

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

where

$D$ = pile width in metres

$S_u$ = undrained shear strength (kPa)

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 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 40 kN at SLS. Parameters for lateral pile resistance are shown in Table 8.2.

**Table 8.2 – Parameters for Lateral Pile Resistance**

| Location | Elevation | $n_h$ ($kN/m^3$) | $S_u$ kPa | $K_p$ | Unit Weight ($kN/m^3$) | Soil Conditions |
|---|---|---|---|---|---|---|
| West and east Abutment | OGL to 383.0 | 6,000 | - | 3.0 | 21 | Sand, very dense to compact (FILL) |
| | 383.0 to 380 | 2,000 | - | 3.0 | 11* | Sand and silt, loose to compact |
| | 380.0 to 374.0 | - | 25 | 2.7 | 10* | Silty clay, very soft to stiff |
| | 374.0 to 372.5 | 3,000 | - | 3.0 | 11* | Sand, silt, loose to compact |

*Buoyant unit weight below the water table.

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

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

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

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

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

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

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

Intermediate values may be obtained by interpolation.

### 8.2 Spread Footings on Native Soils

Spread footings founded on native soils are not recommended at this site due to the following reasons:

- Very low geotechnical capacities are present at this site in much of the overburden soils (native sand and silty clay) below the existing fill.
- Relatively large settlements under footing loads will occur if footings are placed on the native soils.
- Groundwater level is high at this site. Unwatering/groundwater control will be difficult for construction of the footings.
- Spread footings could be subject to erosion or undermining/scour during high creek flows.

### 8.3 Augered Caissons (drilled shafts)

Augered caisson foundations were also considered for supporting the structure at this site.

However, caissons will have to be extended to 9.6 m to 13.5 m depth to reach the top of bedrock. Construction of caissons will require the use of a liner or slurry methods to control groundwater. Since the base of the caisson will be well below the groundwater level, there will be difficulties in dewatering, base cleaning and base inspection.

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

Due to the above issues, the use of augered caissons is not recommended at this site.

### 8.4 Recommended Foundation

From a geotechnical perspective and based on the subsurface conditions steel H-pile foundations driven to refusal on bedrock are considered the most cost effective foundation option for supporting the bridge at this site.

### 8.5 Frost Cover

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

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

## 9 SHEET PILE WALLS

Steel sheet pile walls will be driven adjacent to the H-pile foundations at each abutment. The sheet piles will provide containment and resistance to lateral earth pressures from the approach fill.

Driving of the sheet piles through the existing approach fill and into the underlying loose to compact sands and silts and soft clay is considered feasible based on the borehole data.

Backfill to the sheet pile walls should be in accordance with OPSS 902. Granular backfill should be placed to the extents shown in OPSD 3101.150. All granular material should meet the specifications of OPSS 1010 as amended by Special Provision 110S13. Compaction equipment to be used adjacent to retaining structures should be restricted in accordance with OPSS 501.

Earth pressures acting on the sheet pile walls may be assumed to be triangular and to be governed by the characteristics of the abutment backfill and the underlying existing native silts, sands and clay. For a fully drained condition, the pressures should be computed in accordance with the CHBDC but generally are given by the expression:

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

Where:

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

K = earth pressure coefficient (see Table 9.1)

$\gamma$ = unit weight of retained soil (see Table 9.1)

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

q = value of any surcharge (kPa)

Earth pressure coefficients for backfill to the wall are dependent on the material used as backfill. Typical values for the backfill as well as the underlying native soils are shown in Table 9.1.

**Table 9.1 – Earth Pressure Coefficient (K)**

| Condition | Earth Pressure Coefficient (K) | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | OPSS Granular A or OPSS Granular B Type II $\phi = 35°, \gamma = 22.8$ kN/m³ | | Existing Sand Fill or OPSS Granular B Type I $\phi = 32°, \gamma = 21.2$ kN/m3 | | Native Sand/Silt $\phi = 29°$ $\gamma = 20$ kN/m3 | | Silty Clay $\phi = 27°, \gamma = 20$ kN/m³ | |
| | 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) | Horizontal Surface Behind Wall | Sloping Surface Behind Wall (2H:1V) |
| Active (Unrestrained Wall) | 0.27 | 0.40* | 0.31 | 0.48* | 0.35 | 0.58* | 0.38 | 0.75* |
| At rest (Restrained Wall) | 0.43 | - | 0.47 | - | 0.51 | - | 0.55 | - |
| Passive (Movement Towards Soil Mass) | 3.7 | - | 3.3 | - | 2.9 | - | 2.7 | - |

* 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.

The factors in Table 9.1 are "ultimate" values and require certain movements for the respective conditions to be mobilized. The values to use in design can be estimated from Figure C6.16 in the Commentary to the Canadian Highway Bridge Design Code.

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

## 10 EXCAVATION AND GROUNDWATER CONTROL

Excavation for removal of the existing abutment is expected to be limited to the existing sand approach fill, above the groundwater and creek water level.

All excavation must be carried out in accordance with the Occupational Health and Safety Act (OHSA). For the purposes of the OHSA, the sand fill forming the existing approach embankment and the underlying native silt and sand may be classified as Type 3 soil above the water table and Type 4 below the water table.

The excavation and backfilling must be carried out in accordance with OPSS 902, November 2010.

The selection of the method of excavation is the responsibility of the contractor and must be based on his equipment, experience and interpretation of the site conditions. Excavations should be inspected regularly for evidence of instability.

The Contractor should be prepared to pump from sumps to remove any seepage water or surface water collecting in an excavation. Excavation below the creek level, if required for an alternate foundation system, would require dewatering within a cofferdam to lower the water level below the base of the excavation.

## 11 APPROACH EMBANKMENTS

Based on a contour drawing provided by Hatch Mott MacDonald, it was estimated that the existing approach embankments are up to 3.5 m high. The foundation soils governing stability of the approach embankments consist generally of existing native loose to compact sand and silt overlying very soft to firm silty clay.

Communication with Hatch Mott MacDonald indicates that the existing Highway 11 grade will be raised approximately 400 mm. Further, the new bridge will be shortened by approximately 13.0 m and therefore additional fill will be required between the new sheet pile wall and the existing abutment. This new fill is expected to have a maximum thickness of about 2 m and extend for a length of up to 8 m behind the new sheet pile abutment wall. The sides of the new approach fill will be contained by sheet pile walls installed along each edge of the road.

Embankment construction and widening should be carried out in accordance with OPSS 206. Prior to placement of new fill, existing sloped embankment surfaces should be appropriately benched as per OPSD 208.010, after stripping of vegetation/organics, soft soils or otherwise unsuitable materials.

Comments regarding stability of embankment slopes and settlement of the foundations soils are provided in the following sections.

### 11.1 Slope stability

The existing embankments bearing on the foundation soils at this site appear to be performing satisfactorily under the existing conditions. Placement of an additional 400 mm of new fill to raise the road grade is expected to have minimal impact on the stability of the embankments.

The additional approach fill to be placed behind the new abutment will be supported within a sheet pile enclosure and therefore the stability of the new approach will be governed by the sheet pile wall design. A global slope stability analysis was conducted to assess the embedment requirements for a sheet pile supporting the new approach fill including the 400 mm grade raise. The analyses were carried out using the Morgenstern-Price method of slope stability analysis.

The results of the analyses indicate that an adequate factor of safety for the long term conditions of 1.5 is achieved if the sheet pile is driven to elevation 379.0.

The slope stability computation outputs are included in Appendix D.

The stability of the embankments was not checked under seismic loading as the zonal acceleration at this site is 0.0g.

### 11.2 Settlement

The placement of approximately 2.0 m of new fill behind the sheet pile abutments and 0.4 m of granular fill to raise the existing highway grade will induce immediate (elastic) settlement in the existing non-cohesive fill and sand layers as well as time dependent (consolidation) settlement in the underlying silty clay.

The total immediate and consolidation settlements were assessed using elastic methods and one-dimensional consolidation theory. Based on these analyses, the anticipated immediate and consolidation settlements at the bridge approaches are in the order of 25 and 75 mm, respectively. The H-piles should be designed for a lateral displacement of 10 mm.

The primary consolidation settlement is expected to be completed within 6 to 10 months of completion of fill placement. Secondary consolidation will continue for several years afterward. Inspection of the roadway surface and padding of the asphalt at the approaches to re-establish grades as necessary should be implemented during and after construction.

If post-construction settlement and maintenance is not acceptable, consideration should be given to increasing the bridge span or constructing a three span structure to avoid the need for fill placement in the approaches (in excess of the 0.4 m grade raise).

Several other alternatives were considered to reduce the magnitude of settlement in the approaches. However, these are generally considered unsuitable for this site, based on the following:

- Subexcavation of the clay is not practical due to the significant depth of excavation required.
- Preloading is not feasible as highway traffic must be maintained and bridge replacement will be carried out in two stages.
- For an EPS alternative, buoyancy effects could develop on EPS due to its low density and the high water levels at this site.

- Use of lightweight fill adjacent to the waterway may not be acceptable to environmental authorities.
- Wick drain installation is not appropriate at an existing roadway bridge site.

## 12 EROSION PROTECTION

Erosion protection should be provided along the toe of any slopes that may be in contact with the creek flow.

A vegetation cover should be established on all other exposed earth surfaces to protect against surficial erosion, in general accordance with OPSS 804, November 2010.

## 13 ROADWAY PROTECTION

During the new bridge construction, temporary excavation within existing embankments will be required. The bridge construction will be done in stages in order to keep at least one highway lane operational. Roadway protection will be required to facilitate staging of removals and support the existing Highway 11 adjacent to the excavation.

Roadway protection should be provided in accordance with OPSS 539 and designed for Performance Level 2.

Conventional steel soldier pile and timber lagging walls or continuous sheet pile wall are two options to provide temporary support to the roadway during excavation. Timber lagging boards should be installed as soon as the soil face is exposed and properly prepared. The following parameters apply for design of the temporary shoring system:

| | | | |
|---|---|---|---|
| $\gamma$ | = | 21 $kN/m^3$ | (bulk unit weight) |
| $\gamma_w$ | = | 11 $kN/m^3$ | (submerged unit weight under groundwater table) |
| $K_a$ | = | 0.31 | (Active pressure coefficient for road embankment fill) |
| | = | 0.35 | (Active pressure coefficient for sand/silt) |
| $K_p$ | = | 3.3 | (Passive pressure coefficient for road embankment fill) |
| | = | 2.9 | (Passive pressure coefficient for sand/silt) |
| $h_w$ | = | 382.5 | (groundwater elevation) |

The actual pressure distribution acting on the shoring system is a function of the construction sequence and the relative flexibility of the wall and these factors must be considered when designing the shoring system.

Temporary groundwater and surface water control measures might be required to keep the temporary excavation dry behind the roadway protection.

The design of roadway protection should be the responsibility of the Contractor. All shoring systems should be designed by a Professional Engineer experienced in such designs.

## 14 SEISMIC CONSIDERATIONS

### 14.1 Seismic Design Parameters

The following seismic parameters should be used for design:

- Velocity Related Seismic Zone 0
- Zonal Velocity Ratio 0.0
- Acceleration Related Seismic Zone 0
- Zonal Acceleration Ratio 0.0
- Peak Horizontal Acceleration 0.02

The soil profile type at this site has been classified as Type III. Therefore, according to Table 4.4.6.1 of the CHBDC, a Site Coefficient "S" (ground motion amplification factor) of 1.5 should be used in seismic design.

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

**Table 15.1 – Earth Pressure Coefficients for Earthquake Loading**

| Condition | Earth Pressure Coefficient (K) | | | |
|---|---|---|---|---|
| | **OPSS Granular A or OPSS Granular B Type II** $\phi = 35°$ $\gamma = 22.8$ kN/m$^3$ | **Existing Sand Fill or OPSS Granular B Type I** $\phi = 32°$ $\gamma = 21.2$ kN/m$^3$ | **Native Sand/Silt** $\phi = 29°$ $\gamma = 20$ kN/m$^3$ | **Silty Clay** $\phi = 27°$ $\gamma = 20$ kN/m$^3$ |
| Active ($K_{AE}$)* | 0.28 | 0.32 | 0.35 | 0.39 |
| Passive ($K_{PE}$) | 3.7 | 3.2 | 2.8 | 2.7 |
| At Rest ($K_{OE}$)** | 0.45 | 0.50 | 0.54 | 0.57 |

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

The foundation soils at the abutments are not in danger of liquefaction under earthquake loading.

## 15 CONSTRUCTION CONCERNS

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

- The potential variability of pile lengths driven to bedrock. Bedrock and auger refusal on probable bedrock were contacted at depths ranging from 9.6 m to 13.5 m (elevations 371.2 to 375.1).
- Pile tips must be protected with H-section rock points and driving must be terminated before pile is damaged.
- Excavation, if required, should be maintained above the water level in the creek
- The embankment side slopes should be inspected after construction for surficial disturbance. Where necessary, erosion control measures must be implemented.

## 16 CLOSURE

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

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

Thurber Engineering Ltd.


Rocío Palomeque Reyna, P.Eng., M.Eng.
Geotechnical Engineer


Report reviewed by:
P.K. Chatterji, P.Eng., Ph.D.
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 (MPa)** | **(psi)** | **Field Estimation of Hardness*** |
|---|---|---|---|
| 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 |

THURBER

Ministry of Transportation Ontario

# RECORD OF BOREHOLE No MCB-01



**METRIC**

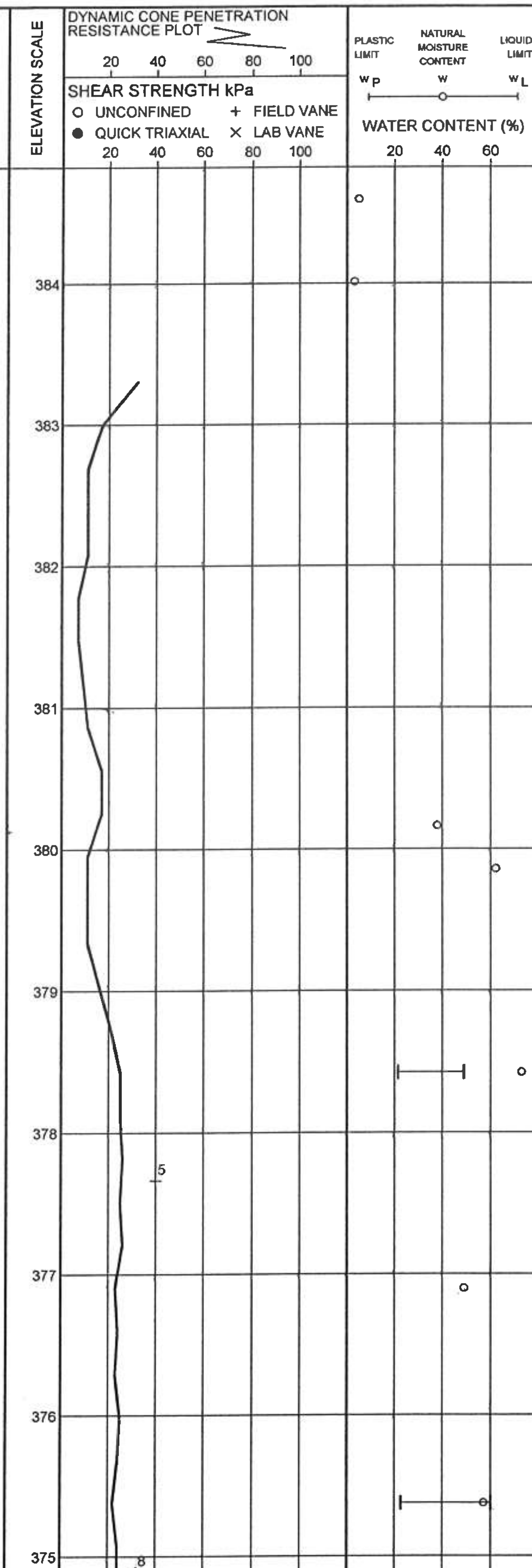
| | | | | | |
|---|---|---|---|---|---|
| W.P. | 6043-08-00 | LOCATION | N 5 398 543.1 E 385 455.7 McCauley Creek Bridge | ORIGINATED BY | ES |
| HWY | 11 | BOREHOLE TYPE | Hollow Stem Augers | COMPILED BY | AN |
| DATUM | Geodetic | DATE | 2011.07.14 - 2011.07.14 | CHECKED BY | RPR |

## SOIL PROFILE

| ELEV DEPTH | DESCRIPTION |
|---|---|
| 384.7 / 0.0 | **ASPHALT:** (50mm) |
| | **SAND**, some gravel, trace silt and clay<br>Very Dense to Dense<br>Brown<br>Damp<br>(FILL)<br>Occasional cobbles<br>Wet<br>Layer of gravel at 1.5m<br>Auger refusal at 2.1m<br>Boulder from 2.0m to 2.7m<br>Compact |
| 380.6 / 4.1 | **SILT**, some clay, trace sand<br>Loose<br>Grey<br>Moist |
| 380.0 / 4.7 | Silty **CLAY**, trace sand<br>Firm<br>Reddish Brown<br>Very Stiff<br>Very Soft<br>Grey |

## SAMPLES

| NUMBER | TYPE | "N" VALUES | GROUND WATER CONDITIONS | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
|---|---|---|---|---|
| 1 | SS | 50/ 0.150 | | |
| 2 | SS | 50/ 0.100 | | |
| 3 | SS | 32 | ∇ | 81 17 2 (SI+CL) |
| 4 | SS | 16 | | |
| 5 | SS | 5 | | |
| 6 | SS | 18 | | 0 0 44 56 |
| 7 | SS | 0 | | |
| 8 | SS | 0 | | 0 0 45 55 |




Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity

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

ONTMT4S 5121.GPJ 9/28/11

Ministry of Transportation
Ontario

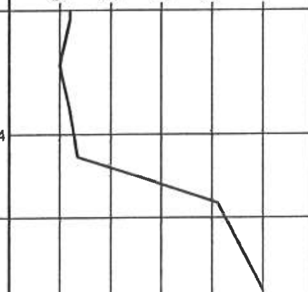
# RECORD OF BOREHOLE No MCB-01



**METRIC**

| | | | |
|---|---|---|---|
| W.P. 6043-08-00 | LOCATION N 5 398 543.1 E 385 455.7 McCauley Creek Bridge | | ORIGINATED BY ES |
| HWY 11 | BOREHOLE TYPE Hollow Stem Augers | | COMPILED BY AN |
| DATUM Geodetic | DATE 2011.07.14 - 2011.07.14 | | CHECKED BY RPR |

| ELEV DEPTH | SOIL PROFILE: DESCRIPTION | STRAT PLOT | SAMPLES: NUMBER | SAMPLES: TYPE | SAMPLES: "N" VALUES | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, X 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 |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | Continued From Previous Page | | | | | | | | | | |
| | Silty **CLAY**, trace sand<br>Hard<br>Grey | | | | | | 374 | | | | |
| 373.7<br>11.0 | | | 9 | SS | 33 | | | | o | | |
| 373.3<br>11.4 | **SAND** and **SILT**, trace clay<br>Dense<br>Grey<br>Wet | | | | | | | | | | |
| | END OF BOREHOLE AT 11.4m UPON CASING REFUSAL ON PROBABLE BEDROCK.<br>WATER LEVEL AT 1.8m UPON COMPLETION.<br>BOREHOLE BACKFILLED WITH AUGER CUTTINGS TO 0.1m, THEN ASPHALT TO SURFACE. | | | | | | | | | | |



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

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

Ministry of Transportation
Ontario

# RECORD OF BOREHOLE No MCB-02



**METRIC**

W.P. 6043-08-00
LOCATION N 5 398 537.1 E 385 462.0 McCauley Creek Bridge
ORIGINATED BY JM
HWY 11
BOREHOLE TYPE NQ Casing
COMPILED BY AN
DATUM Geodetic
DATE 2011.07.15 - 2011.07.15
CHECKED BY RPR

## SOIL PROFILE

| ELEV / DEPTH | DESCRIPTION |
|---|---|
| 384.8 | |
| 0.0 | **ASPHALT:** (150mm) |
| 0.2 | **SAND**, some gravel, trace to some silt and clay<br>Very Dense to Loose<br>Brown<br>Moist<br>(FILL) |
| 382.5 / 2.3 | **SAND**, trace gravel, some silt and clay<br>Loose to Very Loose<br>Brown to Grey<br>Wet |
| 380.7 / 4.1 | Silty **CLAY**, trace sand<br>Firm<br>Grey<br><br>Very Soft |

## SAMPLES

| NUMBER | TYPE | "N" VALUES | GROUND WATER CONDITIONS | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
|---|---|---|---|---|
| 1 | SS | 51 | | |
| 2 | SS | 24 | | |
| 3 | SS | 7 | ∇ | 9 71 20 (SI+CL) |
| 4 | SS | 5 | | |
| 5 | SS | 1 | | |
| 6 | SS | 7 | | |
| 7 | SS | 5 | | 0 1 44 55 |
| 8 | SS | 1 | | |
| 9 | SS | 1 | | 0 1 59 40 |

ELEVATION SCALE: 384, 383, 382, 381, 380, 379, 378, 377, 376, 375

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/m³

+5

Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity

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

ONTMT4S 5121.GPJ 9/28/11

Ministry of Transportation Ontario

# RECORD OF BOREHOLE No MCB-02



**METRIC**

| | | | |
|---|---|---|---|
| W.P. 6043-08-00 | LOCATION N 5 398 537.1 E 385 462.0 McCauley Creek Bridge | | ORIGINATED BY JM |
| HWY 11 | BOREHOLE TYPE NQ Casing | | COMPILED BY AN |
| DATUM Geodetic | DATE 2011.07.15 - 2011.07.15 | | CHECKED BY RPR |

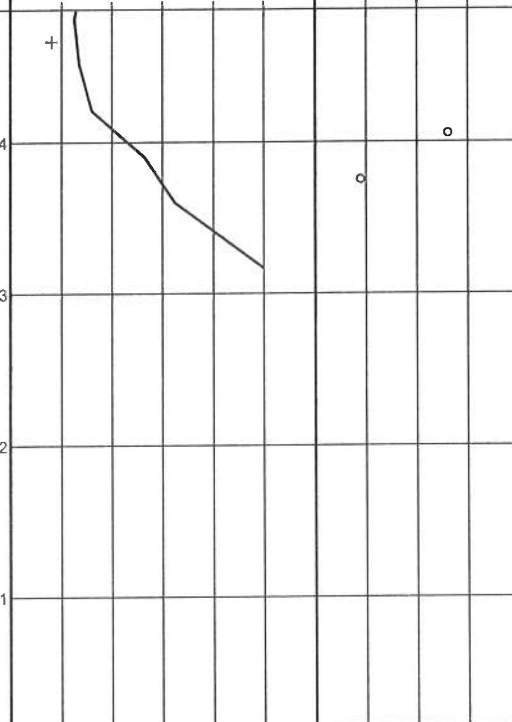
| ELEV DEPTH | SOIL PROFILE DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | GROUND WATER CONDITIONS | ELEVATION SCALE | UNIT WEIGHT γ kN/m³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
|---|---|---|---|---|---|---|---|---|---|
| | Continued From Previous Page | | | | | | | | |
| 374.2 | Silty **CLAY**, trace sand Very Soft Grey | | | | | | | | |
| 10.6 / 373.2 | **SAND** and **SILT**, trace gravel Compact Grey Wet | | 10 | SS | 10 | | 374 | | |
| 11.6 | **BEDROCK ARKOSE/SANDSTONE**, slightly weathered to fresh, grey, occasional mechanical breaks Coring started at 11.6m Sub-vertical breaks at 11.9m and 12.3m Horizontal breaks at 12.2m, 12.3m, 12.5m, 12.6m, 12.8m | | 1 | RUN | | | 373, 372 | FI 0 1 4 3 3 | RUN #1 TCR=97% SCR=97% RQD=7% UCS=158MPa (Average) |
| 370.1 | Sub-vertical breaks (25mm to 75mm thick) at 13.5m, 13.8m and 14.1m Fresh Horizontal breaks at 13.2m, 13.3m, 13.5m and 13.6m | | 2 | RUN | | | 371 | 4 3 3 4 0 | RUN #2 TCR=100% SCR=95% RQD=75% UCS=113MPa (Average) |
| 14.7 | END OF BOREHOLE AT 14.7m. BOREHOLE OPEN TO 14.7m AND WATER LEVEL AT 1.9m. BOREHOLE BACKFILLED WITH HOLEPLUG FROM 14.7m TO 1.5m, SAND AND GRAVEL FROM 1.5m TO 0.1m, THEN ASPHALT TO SURFACE. | | | | | | | | |

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



+³, ×³: Numbers refer to Sensitivity

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

ONTMT4S 5121.GPJ 9/28/11

Ministry of Transportation Ontario

# RECORD OF BOREHOLE No MCB-03



**METRIC**

| | | | |
|---|---|---|---|
| W.P. 6145-04-00 | LOCATION N 5 398 542.0 E 385 463.3 McCauley Creek Bridge | | ORIGINATED BY ES |
| HWY 17 | BOREHOLE TYPE Hollow Stem Augers | | COMPILED BY AN |
| DATUM Geodetic | DATE 2011.07.14 - 2011.07.14 | | CHECKED BY RPR |

| ELEV DEPTH | SOIL PROFILE – DESCRIPTION | SAMPLE NUMBER | TYPE | "N" VALUES | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
|---|---|---|---|---|---|
| 384.7 | | | | | |
| 0.0 / 0.1 | **ASPHALT:** (75mm) | | | | |
| | **SAND**, some gravel, trace to some silt and clay<br>Very Dense to Compact<br>Brown<br>Damp<br>(FILL) | 1 | SS | 52 | |
| | | 2 | SS | 28 | |
| 383.0 / 1.7 | **SAND**, fine grained, trace gravel, some silt and clay<br>Compact to Loose<br>Brown to Grey<br>Moist | 3 | SS | 17 | |
| | | 4 | SS | 12 | 6 81 13 (SI+CL) |
| | | 5 | SS | 5 | |
| 381.2 / 3.5 | **SILT**, some sand, some clay<br>Loose<br>Grey<br>Moist | | | | |
| 379.8 / 4.9 | Silty **CLAY**, trace sand<br>Firm<br>Reddish Brown | 6 | SS | 6 | |
| | | 7 | SS | 8 | |
| | Very Soft | 8 | SS | 0 | 0 1 45 54 |
| | | 9 | SS | 1 | |

Column headings: SOIL PROFILE (ELEV DEPTH, DESCRIPTION, STRAT PLOT); SAMPLES (NUMBER, TYPE, "N" VALUES); 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

Elevation scale: 384, 383, 382, 381, 380, 379, 378, 377, 376, 375

Field vane sensitivity values plotted: +3 (approx. elev. 379.2), +4 (approx. elev. 376.1)

Continued Next Page

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

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

Ministry of Transportation
Ontario

# RECORD OF BOREHOLE No MCB-03



**METRIC**

| | | | |
|---|---|---|---|
| W.P. 6145-04-00 | LOCATION N 5 398 542.0 E 385 463.3 McCauley Creek Bridge | ORIGINATED BY ES |
| HWY 17 | BOREHOLE TYPE Hollow Stem Augers | COMPILED BY AN |
| DATUM Geodetic | DATE 2011.07.14 - 2011.07.14 | CHECKED BY RPR |

Continued From Previous Page

| ELEV DEPTH | DESCRIPTION | SAMPLE NUMBER | TYPE | "N" VALUES | ELEVATION SCALE | PLASTIC LIMIT $w_P$ / NATURAL MOISTURE CONTENT $w$ / LIQUID LIMIT $w_L$ | GR | SA | SI | CL |
|---|---|---|---|---|---|---|---|---|---|---|
| | Silty **CLAY**, varved, trace sand<br>Very Soft<br>Grey | | | | 374 | | | | | |
| | | 10 | SS | 0 | | | 0 | 2 | 70 | 28 |
| | | | | | 373 | | | | | |
| 372.3<br>12.4 | **SAND** and **SILT**, fine grained, trace clay<br>Compact<br>Grey<br>Wet | 11 | SS | 11 | 372 | | | | | |
| 371.2<br>13.5 | END OF BOREOLE AT 13.5m UPON AUGER REFUSAL ON PROBABLE BEDROCK.<br>WATER LEVEL AT 1.9m UPON COMPLETION.<br>Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen. | | | | | | | | | |

WATER LEVEL READINGS:

| DATE | DEPTH (m) | ELEV. (m) |
|---|---|---|
| Sep.18/11 | 1.9 | 382.8 |

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; 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); WATER CONTENT (%) (20 40 60); UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%)

ONTMT4S 5121.GPJ 10/3/11

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

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

Ministry of Transportation
Ontario

# RECORD OF BOREHOLE No MCB-04



**METRIC**

| | | | |
|---|---|---|---|
| W.P. 6145-04-00 | LOCATION N 5 398 529.8 E 385 499.2 McCauley Creek Bridge | | ORIGINATED BY JM |
| HWY 17 | BOREHOLE TYPE Hollow Stem Augers | | COMPILED BY AN |
| DATUM Geodetic | DATE 2011.07.16 - 2011.07.16 | | CHECKED BY RPR |

| ELEV / DEPTH | SOIL PROFILE: DESCRIPTION | SAMPLE NUMBER | TYPE | "N" VALUES | ELEVATION SCALE | WATER CONTENT (%) | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
|---|---|---|---|---|---|---|---|
| 384.8 | | | | | | | |
| 0.0 | **ASPHALT:** (135mm) | | | | | | |
| 0.1 | **SAND**, some gravel, trace to some silt and clay<br>Very Dense to Compact<br>Brown<br>Moist<br>(FILL) | 1 | SS | 54 | | | |
| | | 2 | SS | 19 | 384 | | |
| | | 3 | SS | 14 | 383 | | |
| 382.7 / 2.1 | **SAND**, trace gravel, trace to some silt and clay<br>Loose<br>Grey<br>Wet<br>Occasional cobbles | 4 | SS | 9 | 382 | | |
| | | 5 | SS | 4 | | | 2 86 12 (SI+CL) |
| | Layer of dark brown sand mixed with organics at 3.7m (900mm) | 1 | AS | | 381 | | |
| 380.2 / 4.6 | Silty **CLAY**, trace sand<br>Stiff<br>Grey | 6 | SS | 9 | 380 | | 0 3 64 33 |
| | | | | | 379 | | |
| | Very Soft | 7 | SS | 0 | | 81 | |
| | | | | | 378 | | |
| | | 8 | SS | 0 | 377 | | |
| | | | | | 376 | | |
| | | 9 | SS | 0 | 375 | | |

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/m³

Continued Next Page

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

Ministry of Transportation
Ontario

# RECORD OF BOREHOLE No MCB-04



**METRIC**

| | | | |
|---|---|---|---|
| W.P. | 6145-04-00 | LOCATION | N 5 398 529.8 E 385 499.2 McCauley Creek Bridge |
| HWY | 17 | BOREHOLE TYPE | Hollow Stem Augers |
| DATUM | Geodetic | DATE | 2011.07.16 - 2011.07.16 |

ORIGINATED BY JM
COMPILED BY AN
CHECKED BY RPR

| ELEV DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | GROUND WATER CONDITIONS | ELEVATION SCALE | PLASTIC LIMIT $w_P$ / NATURAL MOISTURE CONTENT $w$ / LIQUID LIMIT $w_L$ — WATER CONTENT (%) | UNIT WEIGHT $\gamma$ kN/m$^3$ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
|---|---|---|---|---|---|---|---|---|---|---|
| | Continued From Previous Page | | | | | | | | | |
| | Silty **CLAY**, trace sand<br>Soft<br>Grey | | | | | | | | | |
| 374.2<br>10.5 | **SAND** and **SILT**, trace clay<br>Compact<br>Grey<br>Wet | | 10 | SS | 20 | | 374 | o | | 0 42 56 2 |
| | | | | | | | 373 | | | |
| 372.5<br>12.3 | END OF BOREHOLE AT 12.3m UPON AUGER REFUSAL ON PROBABLE BEDROCK.<br>Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen. | | 11 | SS | 100/<br>0.100 | | | o | | |

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
WATER CONTENT (%): 20 40 60

WATER LEVEL READINGS:

| DATE | DEPTH (m) | ELEV. (m) |
|---|---|---|
| Aug.27/11 | 2.0 | 382.8 |
| Sep.18/11 | 1.9 | 382.9 |

ONTMT4S 5121.GPJ 10/3/11

$+^3, \times^3$: Numbers refer to Sensitivity    15-⊕-5 (20 above, 10 below) (%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No MCB-05

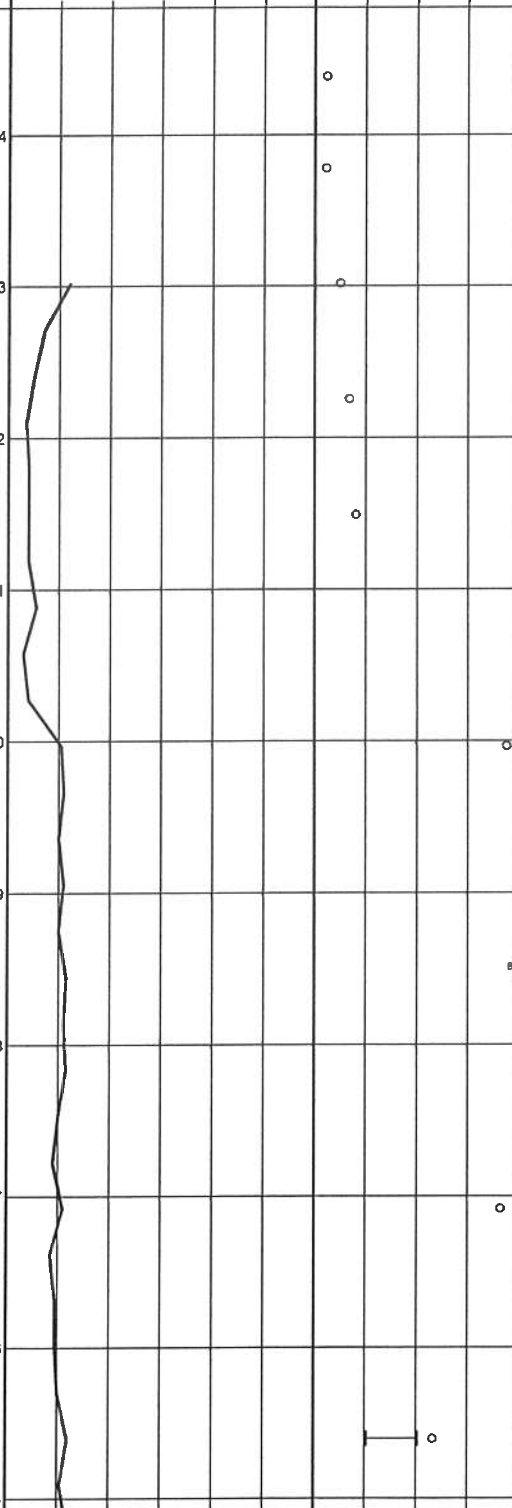


**METRIC**

| | | | |
|---|---|---|---|
| W.P. 6043-08-00 | LOCATION N 5 398 534.4 E 385 500.0 McCauley Creek Bridge | | ORIGINATED BY ES |
| HWY 11 | BOREHOLE TYPE Hollow Stem Augers/NQ Coring | | COMPILED BY AN |
| DATUM Geodetic | DATE 2011.07.13 - 2011.07.13 | | CHECKED BY RPR |

| ELEV / DEPTH | DESCRIPTION | SAMPLE NUMBER | TYPE | "N" VALUES | ELEVATION SCALE | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
|---|---|---|---|---|---|---|
| 384.7 / 0.0 | **ASPHALT:** (50mm) | | | | | |
| | **SAND**, some gravel, trace to some silt and clay<br>Dense to Compact<br>Brown<br>Damp<br>(FILL) | 1 | SS | 37 | | |
| | | 2 | SS | 18 | 384 | |
| 383.4 / 1.4 | **SAND**, fine grained, trace gravel, trace to some silt and clay, occasional oxide staining<br>Compact to Loose<br>Brown to Grey<br>Damp | 3 | SS | 10 | 383 | |
| | | 4 | SS | 5 | 382 | |
| | | 5 | SS | 6 | 381 | 2 88 10 (SI+CL) |
| 380.0 / 4.7 | Silty **CLAY**, trace sand, occasional wood fibres<br>Stiff<br>Grey | 6 | SS | 11 | 380 | 0 3 69 28 |
| | | | | | 379 | |
| | Very Soft | 7 | SS | 0 | 378 | |
| | | 8 | SS | 0 | 377 | |
| | | | | | 376 | |
| | | 9 | SS | 0 | 375 | 0 2 69 29 |

DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, X 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 $\gamma$ kN/m³




Continued Next Page

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

Ministry of Transportation
Ontario

# RECORD OF BOREHOLE No MCB-05



**METRIC**

W.P. 6043-08-00 LOCATION N 5 398 534.4 E 385 500.0 McCauley Creek Bridge ORIGINATED BY ES

HWY 11 BOREHOLE TYPE Hollow Stem Augers/NQ Coring COMPILED BY AN

DATUM Geodetic DATE 2011.07.13 - 2011.07.13 CHECKED BY RPR

Continued From Previous Page

| ELEV DEPTH | DESCRIPTION | SAMPLE NUMBER | TYPE | "N" VALUES | ELEVATION SCALE | UNIT WEIGHT kN/m³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
|---|---|---|---|---|---|---|---|
| | Silty **CLAY**, trace sand<br>Very Soft<br>Grey | | | | | | |
| 373.8<br>10.9 | **SAND** and **SILT**, trace clay<br>Loose<br>Grey<br>Wet | 10 | SS | 5 | 374 | | |
| 373.2<br>11.6 | Casing refusal | ~~11~~ | ~~SS~~ | ~~50/~~<br>0.025 | | FI<br>4 | |
| | **BEDROCK ARKOSE/SANDSTONE**, slightly weathered, occasional mechanical breaks, grey, quartz interbeds<br>Start coring at 11.5m | 1 | RUN | | 373 | 0<br>1<br>1 | RUN #1<br>TCR=100%<br>SCR=95%<br>RQD=95%<br>UCS=87MPa<br>(Average) |
| | 50mm thick vertical breaks at 11.6m<br>Sub-vertical breaks at 12.2m, 12.7m, 12.9m | 2 | RUN | | 372 | 0<br>2<br>0 | RUN #2<br>TCR=100%<br>SCR=100%<br>RQD=100% |
| | Quartz interbeds (between 25mm to 75mm) at 12.6m, 12.7m, 13.1m, 13.5m, 13.6m and 13.7m<br>Sub-vertical breaks (between 25mm to 75mm) at 13.1m, 13.3m, 13.4m, 13.5m 200mm at 13.1m | 3 | RUN | | 371<br>370 | 0<br>0<br>0<br>0 | RUN #3<br>TCR=100%<br>SCR=100%<br>RQD=100%<br>UCS=207MPa<br>(Average) |
| 369.8<br>15.0 | END OF BOREHOLE AT 15.0m. BOREHOLE BACKFILLED WITH HOLEPLUG TO 9.1m, AUGER CUTTINGS TO 0.6m, HOLEPLUG TO 0.1m, THEN ASPHALT TO SURFACE. | | | | | | |

Column headings: SOIL PROFILE (ELEV DEPTH, DESCRIPTION, STRAT PLOT); SAMPLES (NUMBER, TYPE, "N" VALUES); 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



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

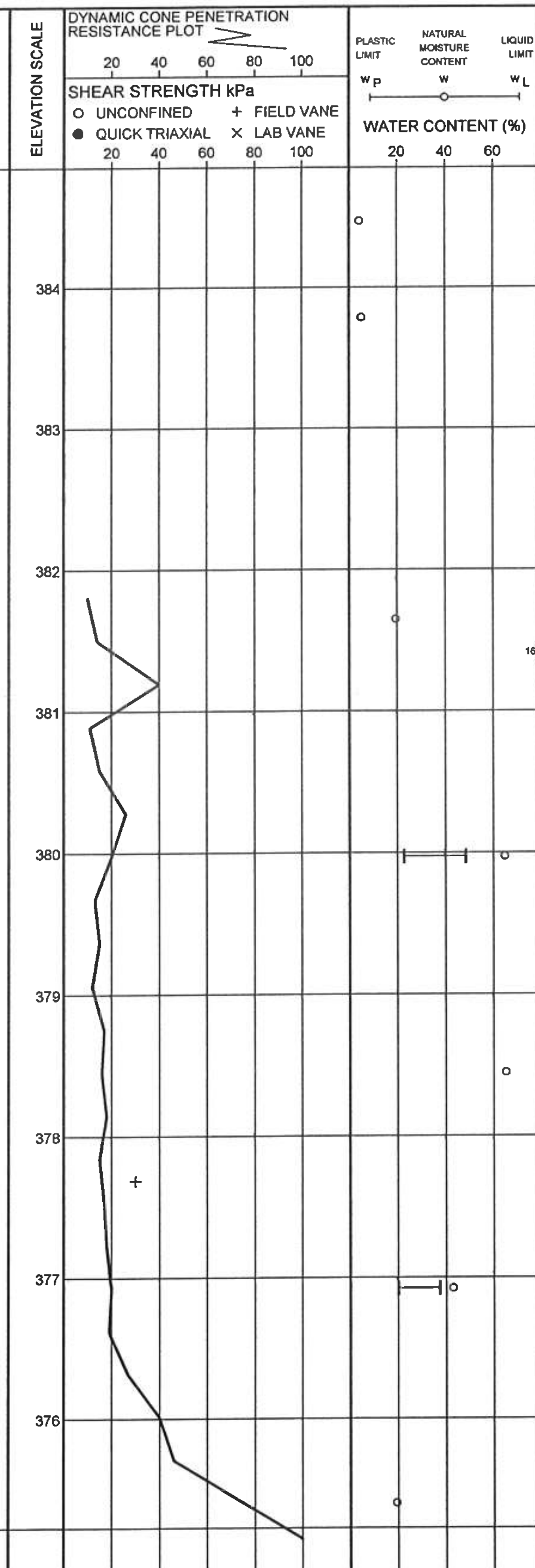
# RECORD OF BOREHOLE No MCB-06



**METRIC**

| | | | |
|---|---|---|---|
| W.P. 6043-08-00 | LOCATION N 5 398 528.2 E 385 507.8 McCauley Creek Bridge | ORIGINATED BY JM |
| HWY 11 | BOREHOLE TYPE Hollow Stem Augers | COMPILED BY AN |
| DATUM Geodetic | DATE 2011.07.16 - 2011.07.16 | CHECKED BY RPR |

| ELEV / DEPTH | DESCRIPTION | SAMPLE NUMBER | TYPE | "N" VALUES | GROUND WATER CONDITIONS | WATER CONTENT (%) / REMARKS | GR | SA | SI | CL |
|---|---|---|---|---|---|---|---|---|---|---|
| 384.8 | | | | | | | | | | |
| 0.0 | **ASPHALT:** (100mm) | | | | | | | | | |
| 0.1 | **SAND** and **GRAVEL** Dense Brown Moist (FILL) | 1 | SS | 41 | | | | | | |
| | | 2 | SS | 38 | | | | | | |
| | Cobbles and boulders from 1.4m to 2.9m | 1 | AS | | | | | | | |
| 381.8 / 2.9 | **SAND**, some gravel Loose Grey Wet (FILL) | 3 | SS | 5 | ∇ | 16 | | | | |
| 381.4 / 3.4 | **PEAT**, some roots, trace gravel Loose Dark Brown Wet | | | | | | | | | |
| 380.6 / 4.1 | Silty **CLAY**, trace sand Soft to Very Soft Grey | 4 | SS | 2 | | | 0 | 2 | 47 | 51 |
| | | 5 | SS | 0 | | | | | | |
| | | 6 | SS | 0 | | | 0 | 1 | 66 | 33 |
| 376.1 / 8.7 | **SAND and SILT**, some gravel, trace clay Compact to Dense Grey Wet | 7 | SS | 47 | | | | | | |
| 375.1 / 9.6 | END OF BOREHOLE AT 9.6m ON PROBABLE BEDROCK. | | | | | | | | | |




Continued Next Page

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

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

Ministry of Transportation
Ontario

# RECORD OF BOREHOLE No MCB-06



**METRIC**

| | | | |
|---|---|---|---|
| W.P. 6043-08-00 | LOCATION N 5 398 528.2 E 385 507.8 McCauley Creek Bridge | ORIGINATED BY JM |
| HWY 11 | BOREHOLE TYPE Hollow Stem Augers | COMPILED BY AN |
| DATUM Geodetic | DATE 2011.07.16 - 2011.07.16 | CHECKED BY RPR |

| 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 | | | | | | |
| | Continued From Previous Page | | | | | | | | | | |
| | BOREHOLE OPEN TO 9.6m AND WATER LEVEL AT 3.0m UPON COMPLETION OF DRILLING. BOREHOLE BACKFILLED WITH HOLEPLUG FROM 9.6m TO 1.5m, SAND AND GRAVEL FROM 1.5m TO 0.9m, CONCRETE FROM 0.9m TO 0.1m, THEN ASPHALT TO SURFACE. | | | | | | | | | | |

ONTMT4S 5121.GPJ 9/28/11

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

15 20 5 10 (%) STRAIN AT FAILURE

**Appendix B**

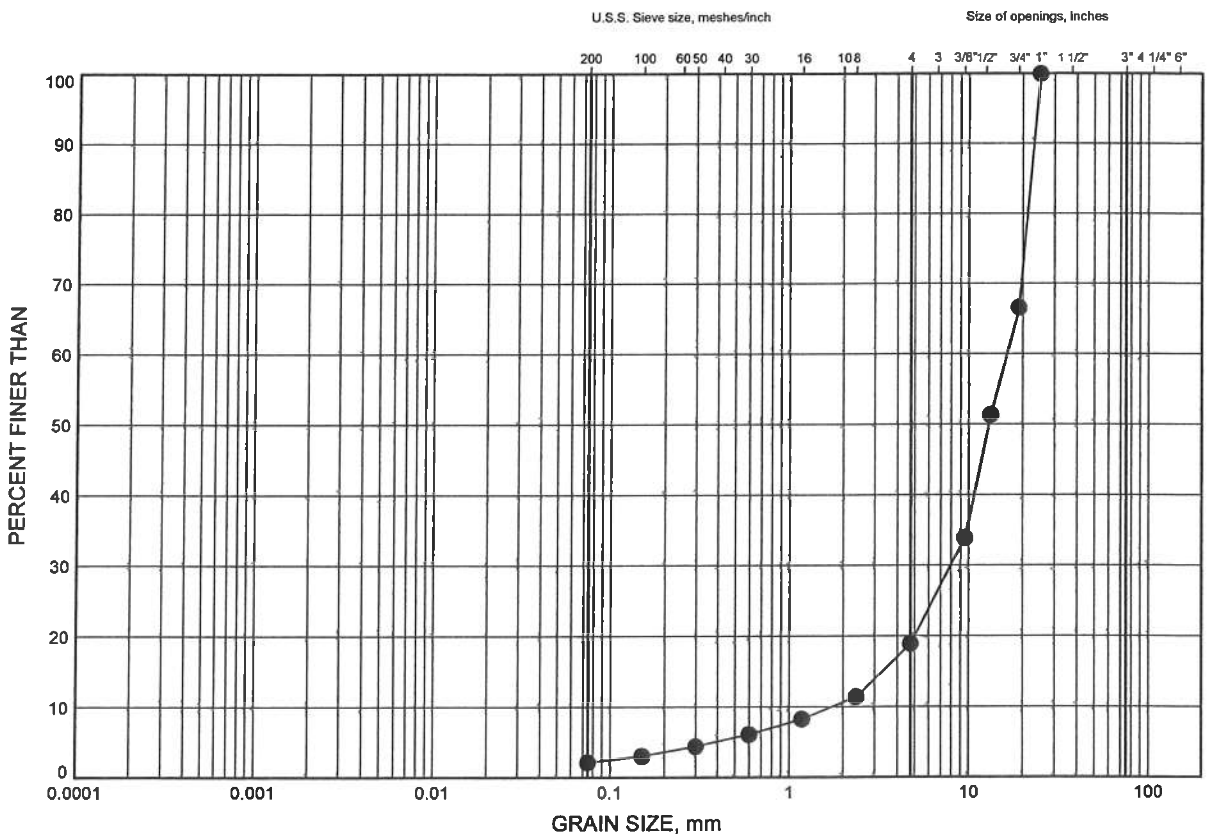
**Laboratory Test Results**

6010-E-0010 Bridge and Culvert Rehabs NWR

# GRAIN SIZE DISTRIBUTION

FIGURE B1

## GRAVEL FILL




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

## LEGEND

| SYMBOL | BOREHOLE | DEPTH (m) | ELEV. (m) |
|---|---|---|---|
| ● | MCB-01 | 1.75 | 382.98 |

THURBER

W.P.# 6043-08-00
Prepared By AN
Checked By RPR

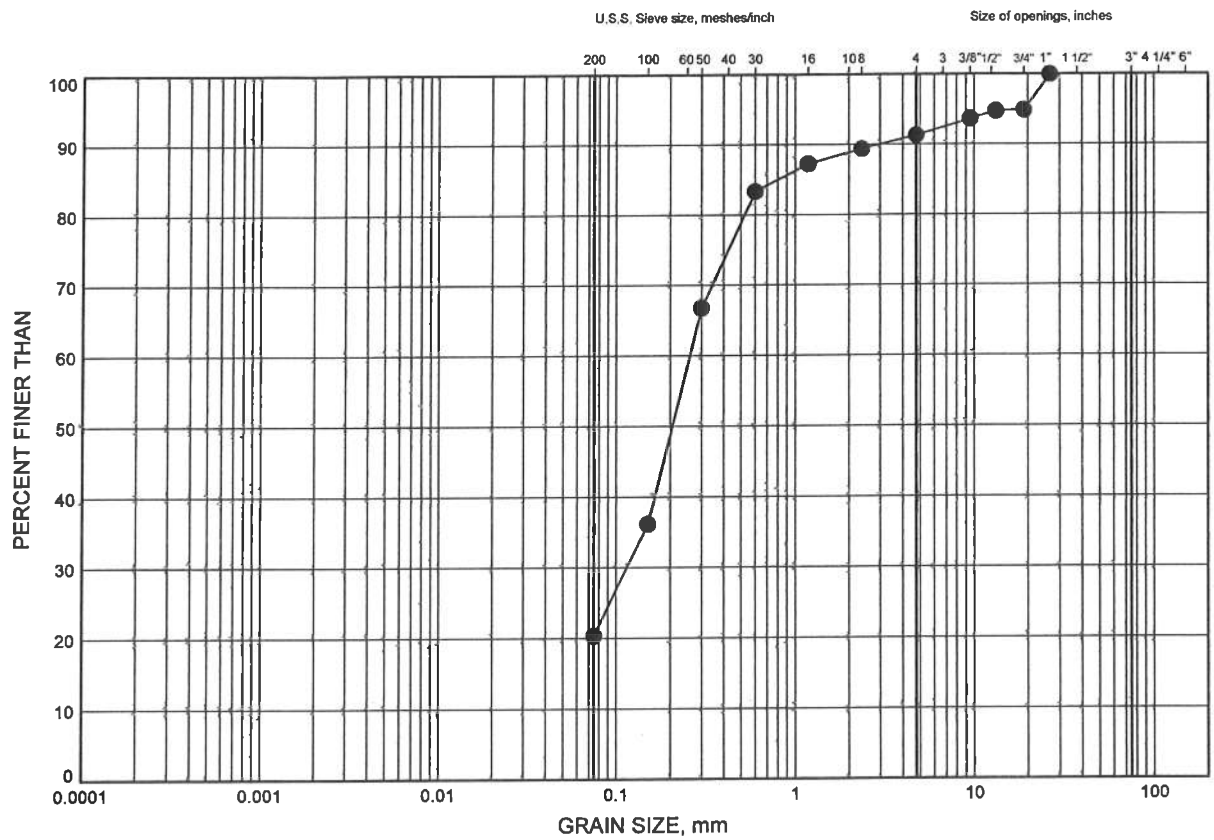
GRAIN SIZE DISTRIBUTION - THURBER 5121.GPJ 9/26/11

6010-E-0010 Bridge and Culvert Rehabs NWR

# GRAIN SIZE DISTRIBUTION

FIGURE B2

## SAND FILL




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

## LEGEND

| SYMBOL | BOREHOLE | DEPTH (m) | ELEV. (m) |
|---|---|---|---|
| ● | MCB-02 | 1.83 | 382.95 |

W.P.# 6043-08-00

Prepared By AN

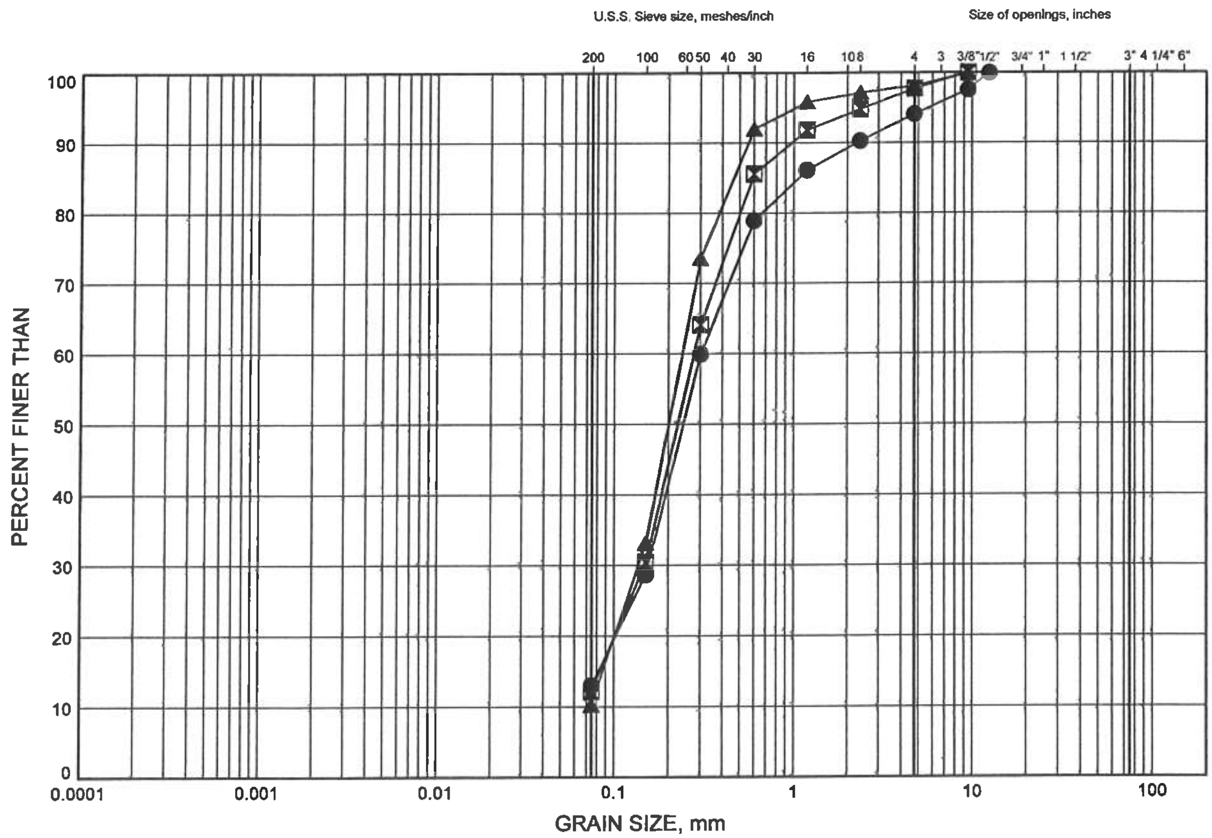
Checked By RPR

THURBER

GRAIN SIZE DISTRIBUTION - THURBER 5121.GPJ 9/26/11

| 6010-E-0010 Bridge and Culvert Rehabs NWR<br>GRAIN SIZE DISTRIBUTION | FIGURE B3 |
|---|---|

## SAND




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

### LEGEND

| SYMBOL | BOREHOLE | DEPTH (m) | ELEV. (m) |
|---|---|---|---|
| ● | MCB-03 | 2.59 | 382.11 |
| ⊠ | MCB-04 | 3.35 | 381.41 |
| ▲ | MCB-05 | 3.35 | 381.40 |

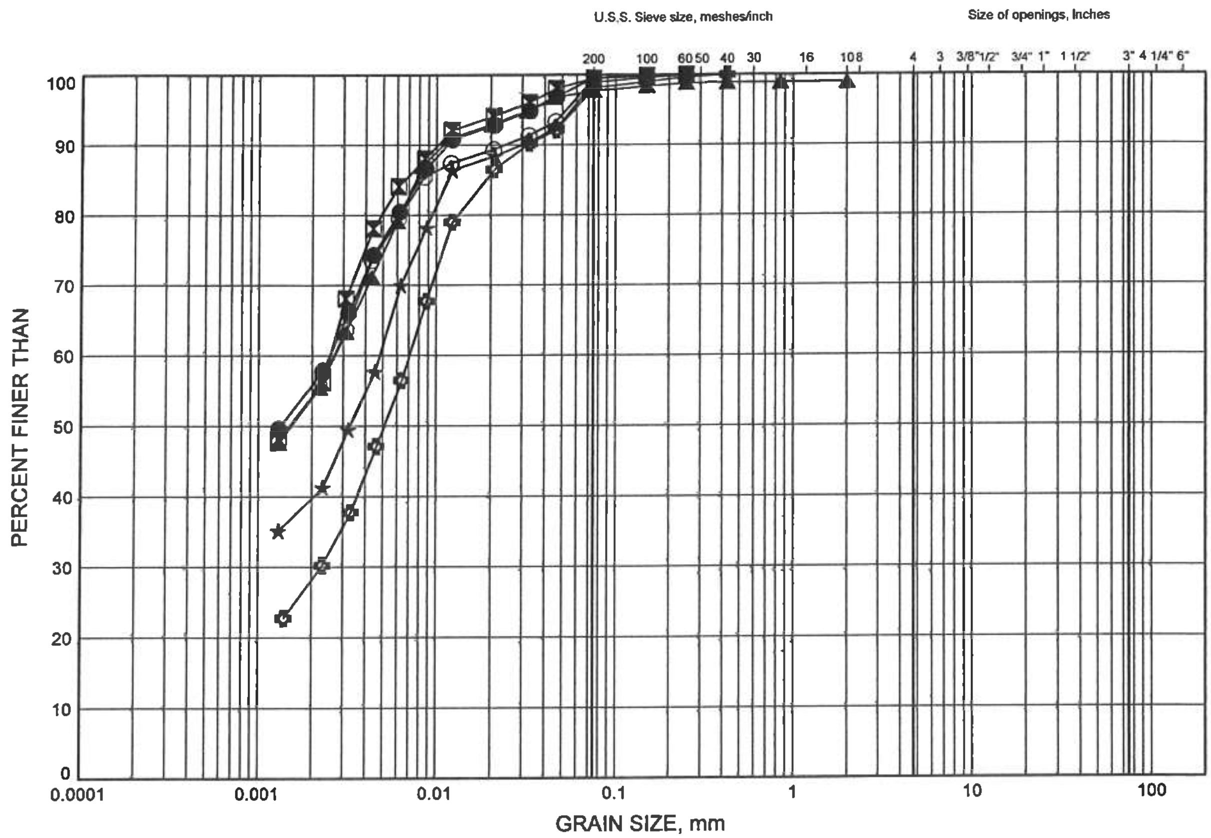
THURBER

W.P.# 6043-08-00
Prepared By AN
Checked By RPR

GRAIN SIZE DISTRIBUTION - THURBER 5121.GPJ 9/26/11

| 6010-E-0010 Bridge and Culvert Rehabs NWR<br>GRAIN SIZE DISTRIBUTION | FIGURE B4 |
|---|---|

## SILTY CLAY




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

## LEGEND

| SYMBOL | BOREHOLE | DEPTH (m) | ELEV. (m) |
|---|---|---|---|
| ● | MCB-01 | 6.40 | 378.33 |
| ⊠ | MCB-01 | 9.45 | 375.28 |
| ▲ | MCB-02 | 6.40 | 378.38 |
| ★ | MCB-02 | 9.45 | 375.33 |
| ⊙ | MCB-03 | 7.92 | 376.78 |
| ✿ | MCB-03 | 10.97 | 373.73 |

W.P.# 6043-08-00
Prepared By AN
Checked By RPR

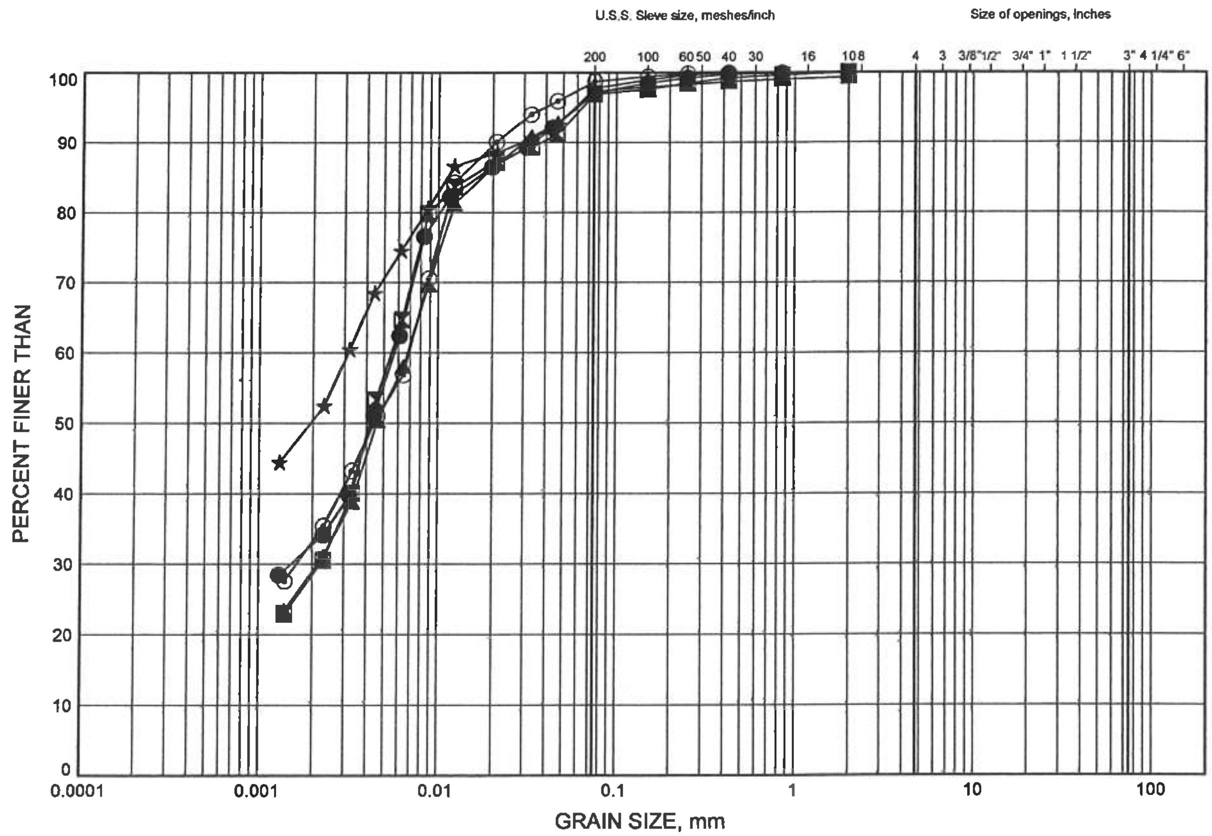
THURBER

6010-E-0010 Bridge and Culvert Rehabs NWR

# GRAIN SIZE DISTRIBUTION

FIGURE B5

## SILTY CLAY




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

### LEGEND

| SYMBOL | BOREHOLE | DEPTH (m) | ELEV. (m) |
|---|---|---|---|
| ● | MCB-04 | 4.88 | 379.88 |
| ⊠ | MCB-05 | 4.88 | 379.87 |
| ▲ | MCB-05 | 9.45 | 375.30 |
| ★ | MCB-06 | 4.88 | 379.88 |
| ⊙ | MCB-06 | 7.92 | 376.83 |

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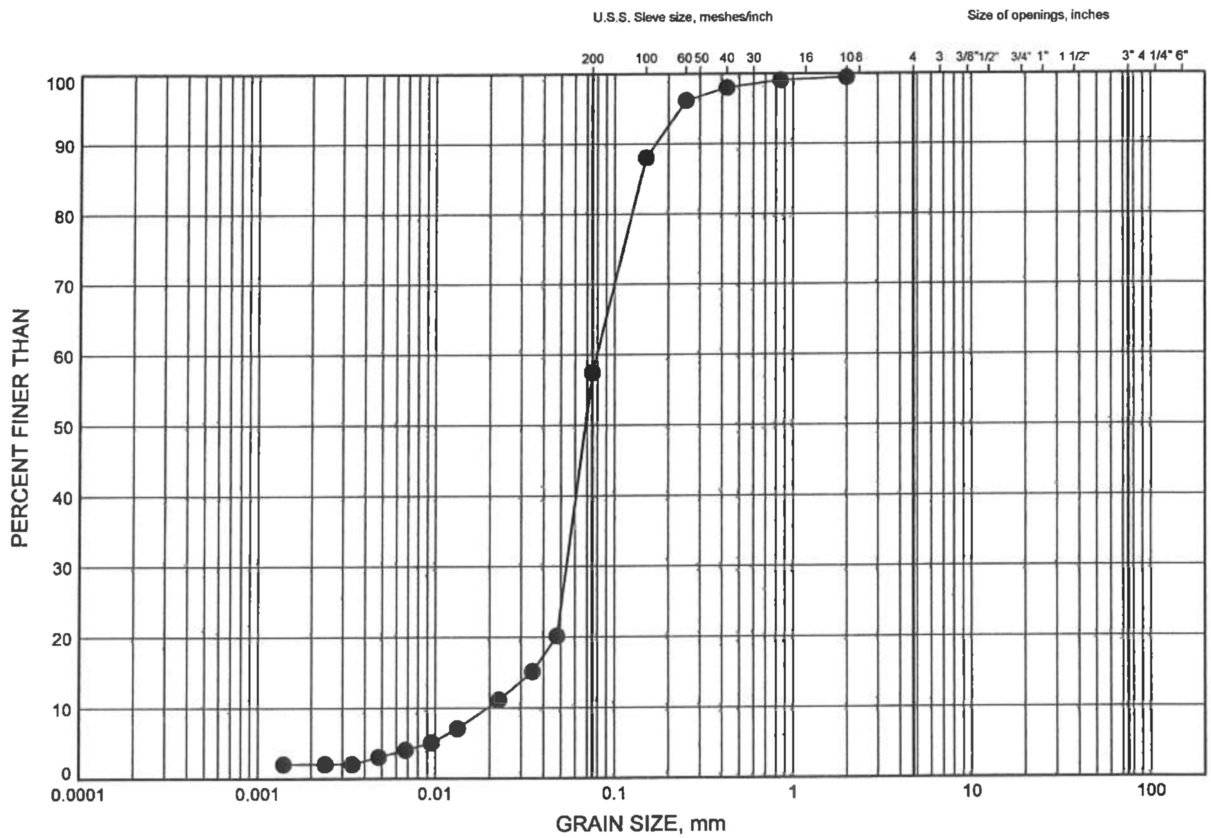
# GRAIN SIZE DISTRIBUTION

FIGURE B6

## SAND & SILT




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

### LEGEND

| SYMBOL | BOREHOLE | DEPTH (m) | ELEV. (m) |
|---|---|---|---|
| ● | MCB-04 | 10.97 | 373.79 |

W.P.# 6043-08-00

Prepared By AN

Checked By RPR

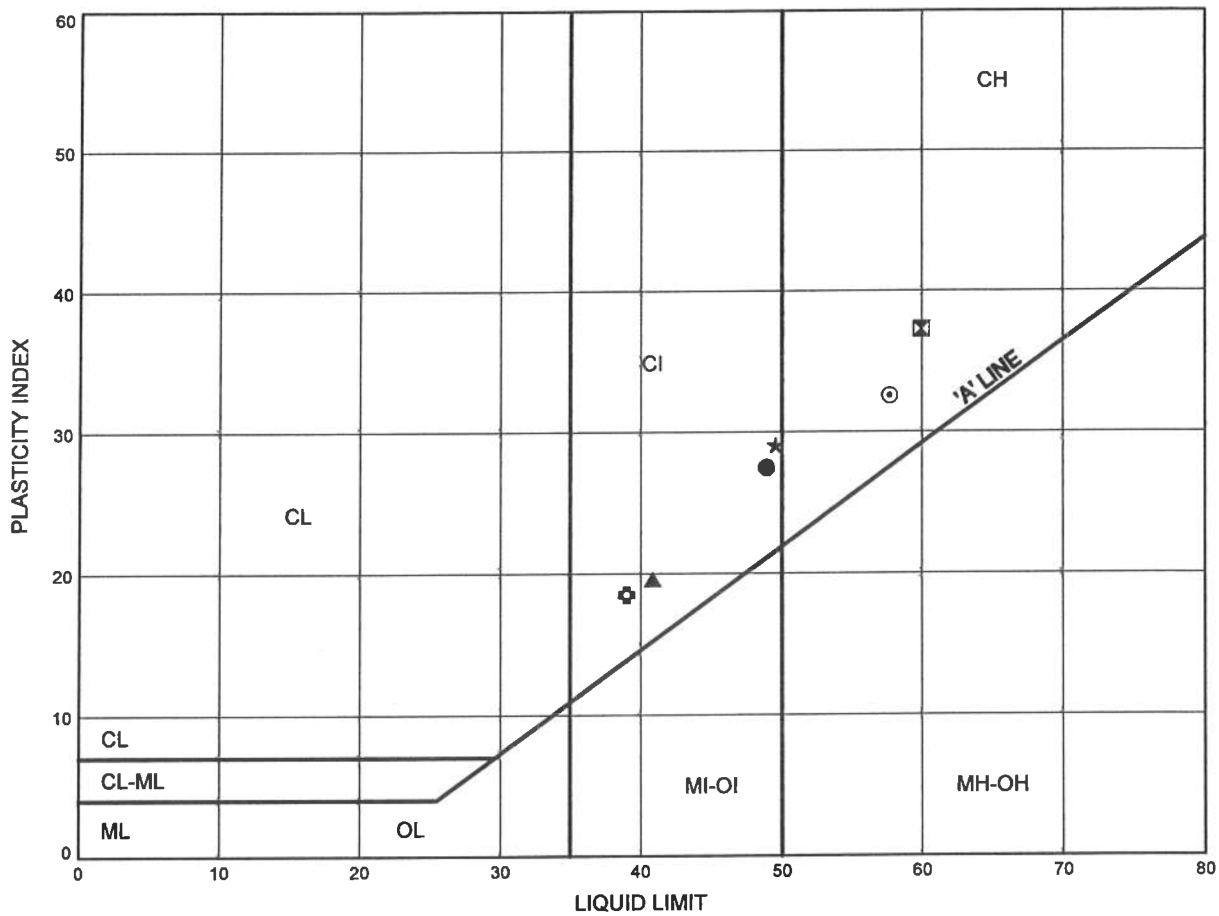
THURBER

6010-E-0010 Bridge and Culvert Rehabs NWR

# ATTERBERG LIMITS TEST RESULTS

FIGURE B7

## SILTY CLAY




| SYMBOL | BH | DEPTH (m) | ELEV. (m) |
|---|---|---|---|
| ● | MCB-01 | 6.40 | 378.33 |
| ⊠ | MCB-01 | 9.45 | 375.28 |
| ▲ | MCB-02 | 6.40 | 378.38 |
| ★ | MCB-02 | 9.45 | 375.33 |
| ⊙ | MCB-03 | 7.92 | 376.78 |
| ✪ | MCB-03 | 10.97 | 373.73 |

Date September 2011

Project 6043-08-00

THURBER

Prep'd AN

Chkd. RPR

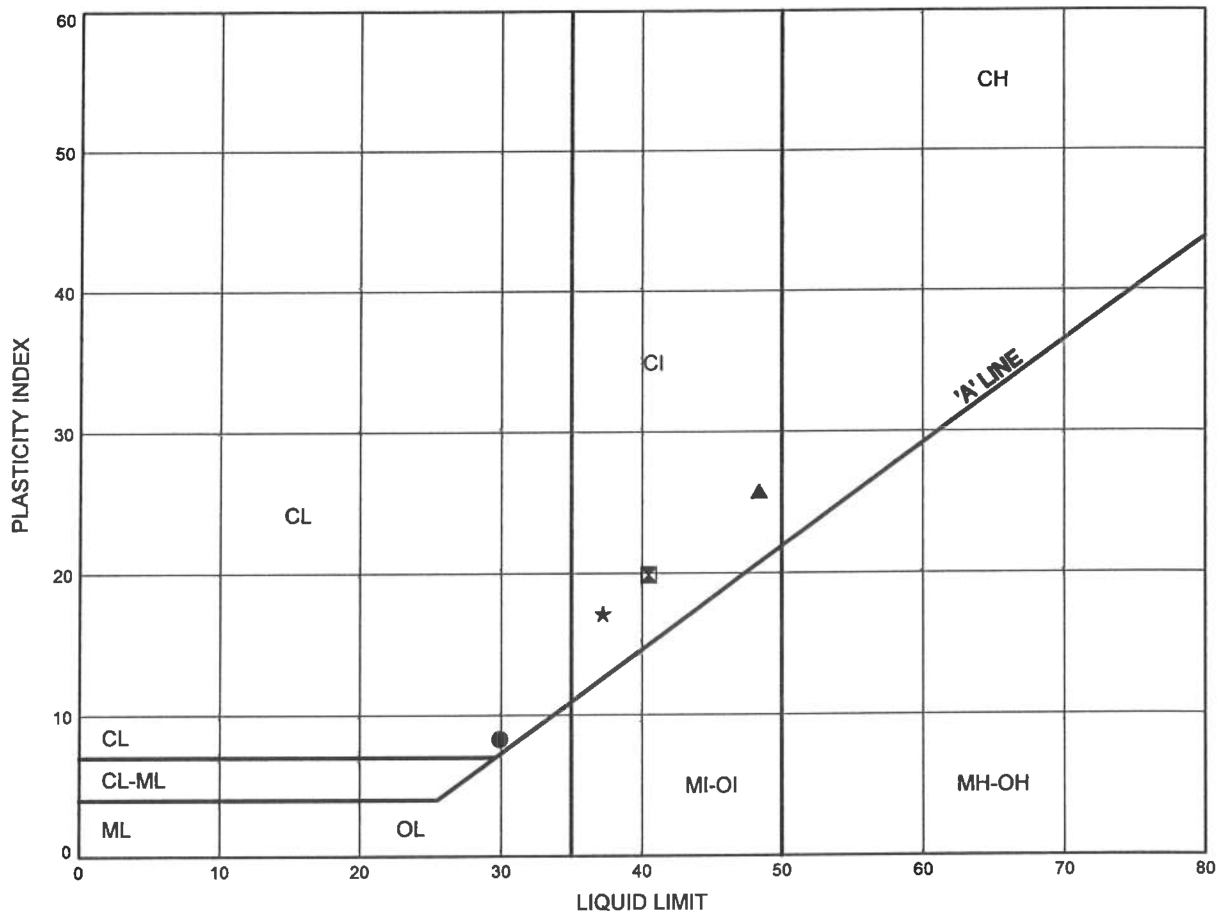
THURBALT 5121.GPJ 9/26/11

6010-E-0010 Bridge and Culvert Rehabs NWR

# ATTERBERG LIMITS TEST RESULTS

FIGURE B8

## SILTY CLAY




| SYMBOL | BH | DEPTH (m) | ELEV. (m) |
|---|---|---|---|
| ● | MCB-04 | 4.88 | 379.88 |
| ⊠ | MCB-05 | 9.45 | 375.30 |
| ▲ | MCB-06 | 4.88 | 379.88 |
| ★ | MCB-06 | 7.92 | 376.83 |

Date September 2011

Project 6043-08-00

THURBER

Prep'd AN

Chkd. RPR

THURBALT 5121.GPJ 9/26/11

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GEOTECHNICAL • ENVIRONMENTAL • MATERIALS

# POINT LOAD TEST SHEET

| | | | |
|---|---|---|---|
| **Job No :** | 19-1605-121 | **Client :** | HMM |
| | | **Date Drilled :** | 15/7/2011 |
| **Project Name :** | McCauley Bridge Creek | **Date Tested :** | 2/8/2011 |
| **Core Size :** | NQ **BH No :** MCB-02 | **Tester :** | MAT |

| Test No. | Run No. | Depth (m) | Axial or Diametral | Force (kN) | Diameter (mm) | Length (mm) | UCS (MPa) | Rock Type | Notes |
|---|---|---|---|---|---|---|---|---|---|
| 1 | 1 | 11.8 | D | 12.0 | 47.5 | 74.3 | 125.1 | Arkose/sandstone | Very Strong |
| 2 | 1 | 12.2 | D | 18.5 | 47.5 | 83.6 | 191.9 | Arkose/sandstone | Very Strong |
| 3 | 1 | 25.3 | D | 15.0 | 47.5 | 85.8 | 156.1 | Arkose/sandstone | Very Strong |
| 4 | 2 | 13.4 | D | 19.4 | 47.5 | 86.1 | 201.3 | Arkose/sandstone | Very Strong |
| 5 | 2 | 13.7 | D | 16.0 | 47.5 | 90.6 | 166.2 | Arkose/sandstone | Very Strong |
| 6 | 2 | 14.1 | D | 8.6 | 47.5 | 93.0 | 89.1 | Arkose/sandstone | Strong |
| 7 | 2 | 13.9 | A | 7.7 | 47.5 | 55.8 | 58.4 | Arkose/sandstone | Strong |
| 8 | 2 | 14.7 | D | 5.0 | 47.5 | 102.6 | 52.0 | Arkose/sandstone | Strong |
| 9 | | | | | | | | | |
| 10 | | | | | | | | | |
| 11 | | | | | | | | | |
| 12 | | | | | | | | | |
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| 29 | | | | | | | | | |
| 30 | | | | | | | | | |

* It is ideal to perform axial test on core specimens with D/L ratio of 1.1 ± 0.1
Long pieces of core can be tested diametrically to produce suitable lengths for axial testing

* Diametral Test should have 0.7 x D on either side of test point.

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# POINT LOAD TEST SHEET

**Job No :** 19-1605-121 **Client :** HMM

**Date Drilled :** 15/7/2011

**Project Name :** McCauley Bridge Creek **Date Tested :** 2/8/2011

**Core Size :** NQ **BH No :** MCB-05 **Tester :** MAT

| Test No. | Run No. | Depth (m) | Axial or Diametral | Force (kN) | Diameter (mm) | Length (mm) | UCS (MPa) | Rock Type | Notes |
|---|---|---|---|---|---|---|---|---|---|
| 1 | 1 | 11.8 | D | 7.0 | 47.6 | 83.0 | 72.8 | Arkose/sandstone | Strong |
| 2 | 1 | 12.0 | D | 12.3 | 47.6 | 87.4 | 127.6 | Arkose/sandstone | Very Strong |
| 3 | 1 | 12.7 | D | 5.8 | 47.6 | 104.7 | 59.8 | Arkose/sandstone | Strong |
| 4 | 3 | 13.8 | D | 20.0 | 47.6 | 86.1 | 207.2 | Arkose/sandstone | Very Strong |
| 5 | 3 | 14.0 | D | 20.0 | 47.6 | 90.6 | 207.3 | Arkose/sandstone | Very Strong |
| 6 | 3 | 14.4 | D | 20.0 | 47.6 | 93.0 | 207.3 | Arkose/sandstone | Very Strong |
| 7 | 3 | 14.8 | D | 20.0 | 47.6 | 55.8 | 207.0 | Arkose/sandstone | Very Strong |
| 8 | | | | | | | | | |
| 9 | | | | | | | | | |
| 10 | | | | | | | | | |
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| 12 | | | | | | | | | |
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| 28 | | | | | | | | | |
| 29 | | | | | | | | | |
| 30 | | | | | | | | | |

* It is ideal to perform axial test on core specimens with D/L ratio of 1.1 ± 0.1
Long pieces of core can be tested diametrically to produce suitable lengths for axial testing

* Diametral Test should have 0.7 x D on either side of test point.

# Appendix C

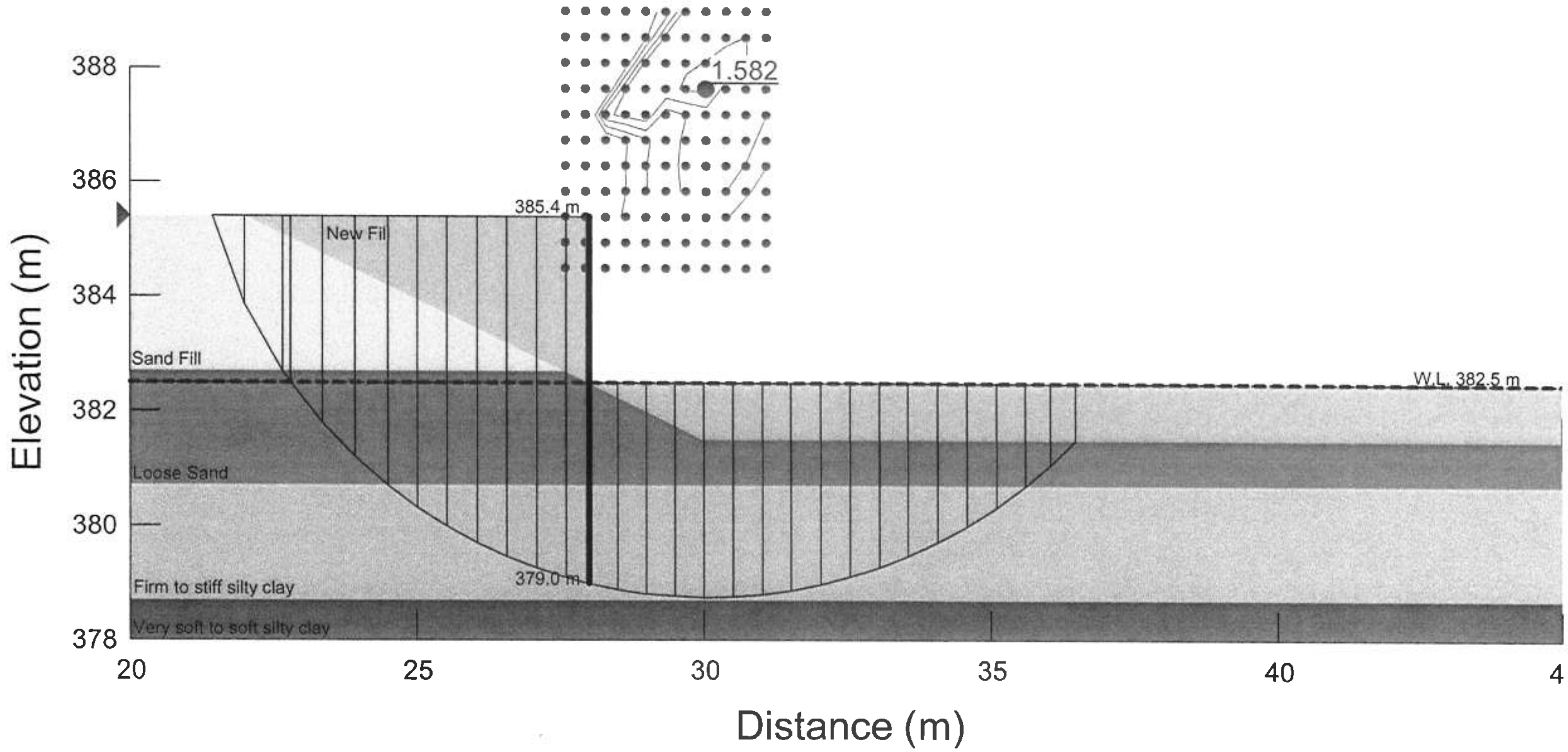
## Foundation Comparison

## COMPARISON OF FOUNDATION ALTERNATIVES FOR EACH FOUNDATION ELEMENT

| Footing on Native Soil | Augered Caissons (drilled shafts) | Driven Piles |
|---|---|---|
| ***Advantages:***<br>i. Generally less costly construction than deep foundation elements. | ***Advantages:***<br>i. High geotechnical resistance available for caissons founded on bedrock<br>ii. Construction of caissons could continue in freezing weather. | ***Advantages:***<br>i. High geotechnical resistance available by driving piles to achieve resistance on the bedrock.<br>ii. Installation of piles could continue in freezing weather.<br>iii. May require less volume of excavation than footings.<br>iv. Readily installed. |
| ***Disadvantages:***<br>i. Low available geotechnical resistance in native soils.<br>ii. Potential for settlements.<br>iii. Excavation extending below the groundwater level is required.<br>iv. Foundations close to creek flow would be at risk due to scour and erosion.<br>v. Potential disturbance of creek during excavation. | ***Disadvantages:***<br>i. Higher unit cost compared to other foundation options such as footings or driven piles<br>ii. Specialized installation measures such as temporary liners and drilling mud will be required to install caissons under the water table.<br>iii. Potential difficulty in cleaning and inspecting bases. | ***Disadvantages:***<br>i. Higher unit costs than footings.<br>ii. Pile lengths required to achieve design resistance may vary. |
| **NOT RECOMMENDED** | **NOT RECOMMENDED** | **RECOMMENDED** |

# Appendix D

## Slope Stability Output

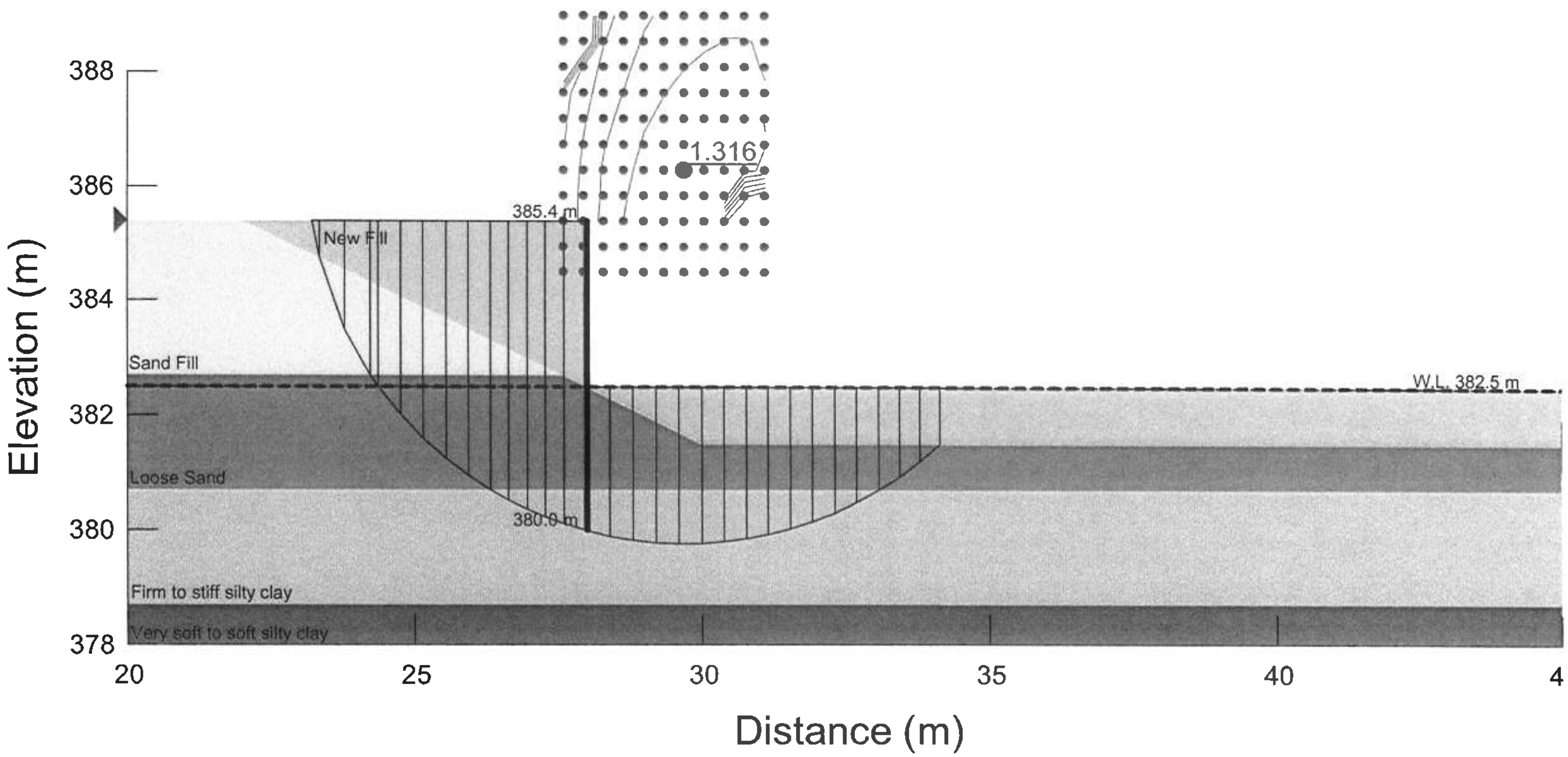



McCauley Creek
19-1605-121
Highway 11
McCauleyCreek_01.gsz
1/20/2012
Height = 3.9 m, Slope = 2H:1V

Factor of Safety: 1.563 (Bishop)
Factor of Safety: 1.582 (Morgenstern-Price)

Material #: 1 Description: Water Wt: 9.807 Piezometric Line: 1
Material #: 2 Description: New Fill Wt: 22 Cohesion: 0 Phi: 32 Piezometric Line: 1
Material #: 3 Description: Sand Fill Wt: 21 Cohesion: 0 Phi: 32 Piezometric Line: 1
Material #: 4 Description: Sand Wt: 20 Cohesion: 0 Phi: 30 Piezometric Line: 1
Material #: 5 Description: Silty Clay Wt: 18 Cohesion: 0 Phi: 28 Piezometric Line:
Material #: 6 Description: Silty Clay Wt: 18 Cohesion: 0 Phi: 26 Piezometric Line:

Figure 1




McCauley Creek
19-1605-121
Highway 11
McCauleyCreek_02.gsz
1/20/2012
Height = 3.9 m, Slope = 2H:1V

Factor of Safety: 1.294 (Bishop)
Factor of Safety: 1.316 (Morgenstern-Price)

| | | | | | | |
|---|---|---|---|---|---|---|
| Material #: 1 | Description: Water | Wt: 9.807 | Piezometric Line: 1 | | | |
| Material #: 2 | Description: New Fill | Wt: 22 | Cohesion: 0 | Phi: 32 | Piezometric Line: 1 |
| Material #: 3 | Description: Sand Fill | Wt: 21 | Cohesion: 0 | Phi: 32 | Piezometric Line: 1 |
| Material #: 4 | Description: Sand | Wt: 20 | Cohesion: 0 | Phi: 30 | Piezometric Line: 1 |
| Material #: 5 | Description: Silty Clay | Wt: 18 | Cohesion: 0 | Phi: 28 | Piezometric Line: |
| Material #: 6 | Description: Silty Clay | Wt: 18 | Cohesion: 0 | Phi: 26 | Piezometric Line: |

Figure 2

# Appendix E

## List of SPs and OPSS

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

- OPSS 903, November 2009.
- OPSS 902, November 2010.
- OPSS 804, November 2010.
- OPSD 208.010
- OPSD 3101.150
- Special Provision 110S13 “Amendment to OPSS 1010, April 2004”.
- OPSS 501

**Appendix F**

**Site Photographs**

**Photograph 1**– McCauley Creek Bridge - Looking West


**Photograph 2** – McCauley Creek Bridge- Looking East

**Photograph 3** – North side of the McCauley Creek Bridge

**Photograph 4** – North side of the McCauley Creek Bridge

**Photograph 5** – South side of the McCauley Creek Bridge

**Appendix G**

**Drawing titled "Borehole Locations and Soil Strata"**

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

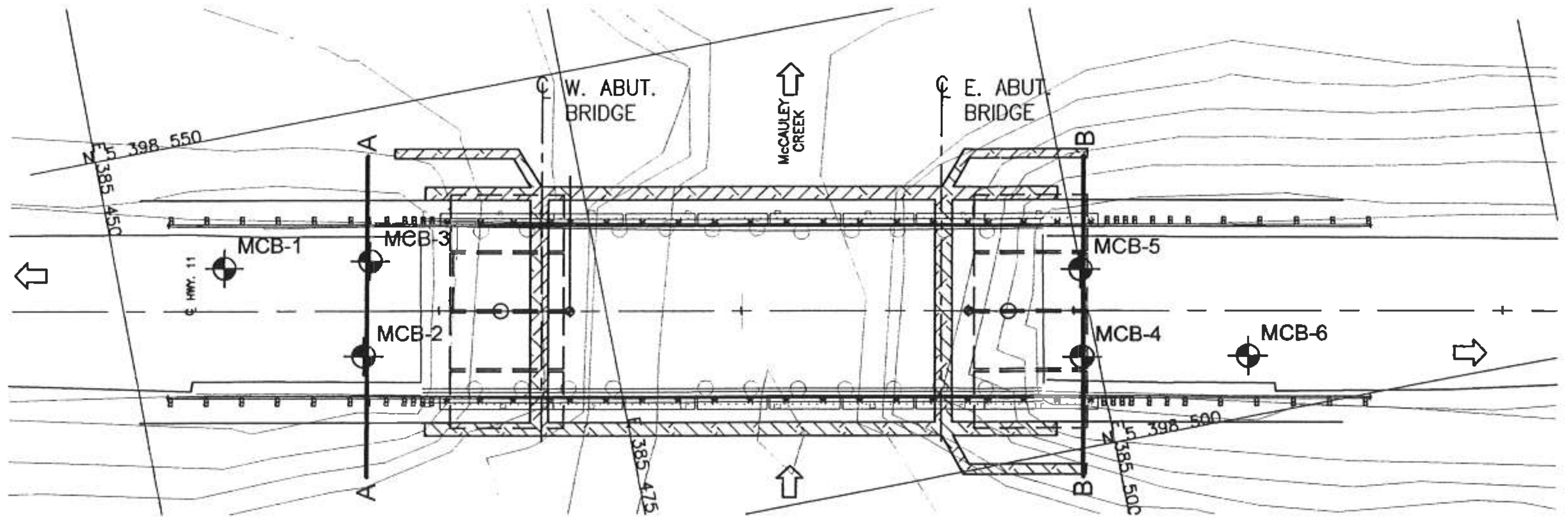
CONT No
WP No 6043–08–00

SHEET

HIGHWAY 11
BRIDGE & CULVERT REHABS
McCAULEY CREEK BRIDGE
BOREHOLE LOCATIONS AND SOIL STRATA

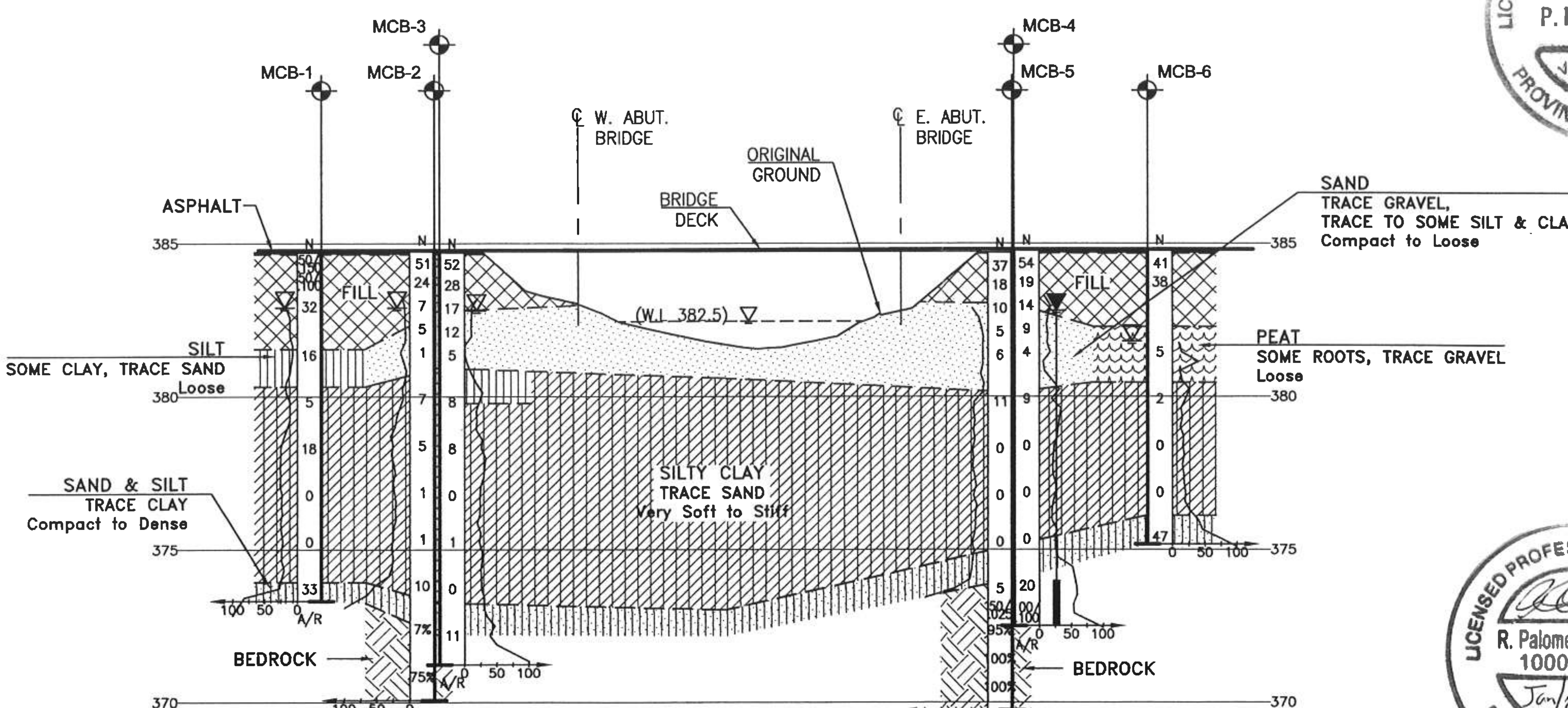
Hatch Mott MacDonald

THURBER ENGINEERING LTD.




PLAN

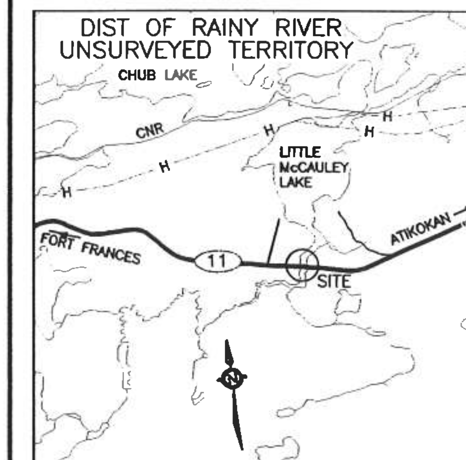
SCALE 1:400




PROFILE ALONG HWY. 11

H 1:400

V 1:200




## 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 |
|---|---|---|---|
| MCB–01 | 384.7 | 5 398 543.1 | 385 455.7 |
| MCB–02 | 384.8 | 5 398 537.1 | 385 462.0 |
| MCB–03 | 384.7 | 5 398 542.0 | 385 463.3 |
| MCB–04 | 384.8 | 5 398 529.8 | 385 499.2 |
| MCB–05 | 384.7 | 5 398 534.4 | 385 500.0 |
| MCB–06 | 384.8 | 5 398 528.2 | 385 507.8 |

LICENSED PROFESSIONAL ENGINEER
P. K. CHATTERJI
Jan 23/2012
PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER
R. Palomeque Reyna
100083209
Jan/23/2012
PROVINCE OF ONTARIO

### -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. 52B-14**

| REVISIONS | DATE | BY | DESCRIPTION |
|---|---|---|---|

| | | | | |
|---|---|---|---|---|
| DESIGN RPR | CHK RPR | CODE | LOAD | DATE JAN. 2012 |
| DRAWN AN | CHK | SITE | STRUCT | DWG 1 |

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PLOTDATE: 1/18/2012 11:47 AM

# METRIC

DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

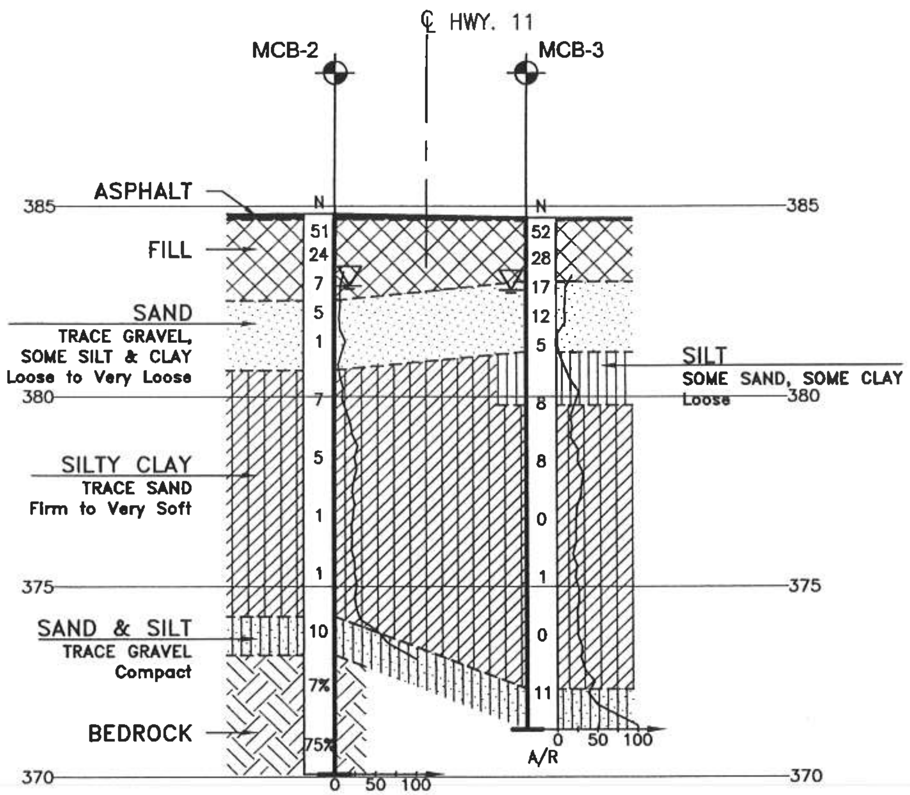
CONT No
WP No 6043-08-00

SHEET

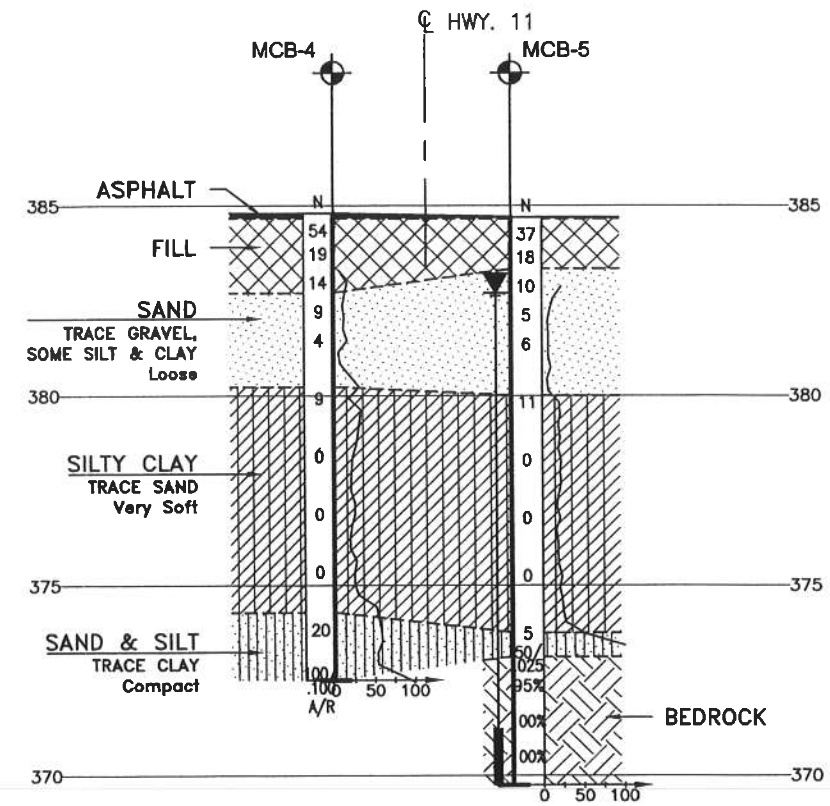
HIGHWAY 11
BRIDGE & CULVERT REHABS
McCAULEY CREEK BRIDGE
BOREHOLE LOCATIONS AND SOIL STRATA

Hatch Mott MacDonald

THURBER ENGINEERING LTD.

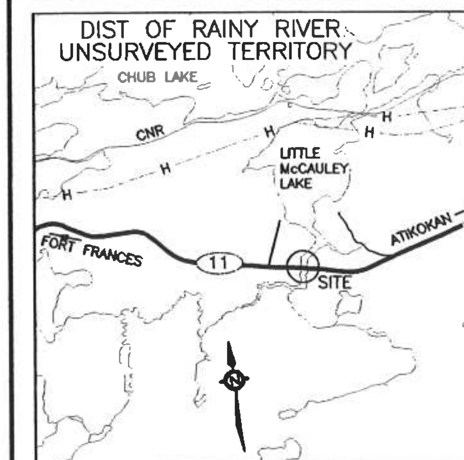



SECTION ALONG A–A




SECTION ALONG B–B

SCALE 1:200




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 |
|---|---|---|---|
| MCB–01 | 384.7 | 5 398 543.1 | 385 455.7 |
| MCB–02 | 384.8 | 5 398 537.1 | 385 462.0 |
| MCB–03 | 384.7 | 5 398 542.0 | 385 463.3 |
| MCB–04 | 384.8 | 5 398 529.8 | 385 499.2 |
| MCB–05 | 384.7 | 5 398 534.4 | 385 500.0 |
| MCB–06 | 384.8 | 5 398 528.2 | 385 507.8 |

-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. 52B-14

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100083209
Jan/23/2012
PROVINCE OF ONTARIO

| REVISIONS | DATE | BY | DESCRIPTION | | |
|---|---|---|---|---|---|
| DESIGN RPR | CHK RPR | CODE | LOAD | DATE JAN. 2012 | |
| DRAWN AN | CHK | SITE | STRUCT | DWG 2 | |

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MINISTRY OF TRANSPORTATION, ONTARIO