FOUNDATION INVESTIGATION AND DESIGN REPORT
NUGGET CREEK BRIDGE REPLACEMENT
HIGHWAY 17
BETWEEN DRYDEN AND IGNACE ONTARIO
TOWNSHIP OF ZEALAND, ONTARIO

W.P. 6145-04-00, Site No. 41S-62

Geocres Number: 52F-38

Report to

Hatch Mott MacDonald

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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 17 over Nugget Creek, in the Town of Wabigoon in the Township of Zealand, 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 Nugget Creek Bridge is located on Highway 17 at the east end of the Town of Wabigoon in the Township of Zealand, Ontario (between Dryden and Ignace) in the District of Thunder Bay.

At present, the highway crosses Nugget Creek on a four-span structure supported on timber piles. The two end spans are 5.94 m and 4.5 m in length, while the two interior spans are 6.1 m in length. The total length of the bridge is 25.8 m and the width is 13.03 m. There is a pedestrian walkway that crosses the creek along the north side of the bridge. Nugget Creek flows from the south to the north. Wabigoon Lake, from where Nugget Creek originates, in located approximately 250 m south of the bridge.

The area surrounding the bridge site is relatively flat. The areas to the north and southwest of the site are treed and include individual residential dwellings, while the area to the southeast consists of low lying marsh vegetation. The Town of Wabigoon and its associated infrastructure are located just west of the site.

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

The site lies within the physiographic region known as the Wabigoon Subprovince of the Superior Province of the Canadian Shield, which is underlain by Archean rocks. The region is characterized by mafic to intermediate metavolcanic rocks consisting of basaltic and andesitic flows, tuffs and breccias, chert, iron formation, minor metasedimentary and intrusive rocks and related migmatites. Locally, the bedrock is mantled by glaciolacustrine varved silty clay and silt deposits.

3 SITE INVESTIGATION AND FIELD TESTING

The site investigation and field testing for this project were carried out during the period of July 22 to 27, 2011 and consisted of drilling and sampling six boreholes (numbered NGT-01 to NGT-06) through the highway embankment in the area of the existing west and east approaches and abutments. Boreholes NGT-01 and NGT-06 were drilled at the west and east approaches, respectively and were both terminated at a depth of 11.3 m (elevation 359.5). Boreholes NGT-02 and NGT-03 were drilled near the west abutment to depths of 21.2 m and 16.2 m, respectively (elevations 349.6 and 354.7). Boreholes NGT-04 and NGT-05 were drilled near the east abutment to depths of 16.8 m and 21.3 m, respectively (elevations 354.1 and 349.6).

Bedrock was proved in Boreholes NGT-02 and NGT-05 by NQ size diamond coring. Boreholes NGT-02 and NGT-05 were advanced 2.9 m and 3.0 m into bedrock.

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

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

For Boreholes NGT-01 to NGT-05, the drilling was carried out from the highway grade using a CME 75 truck-mounted drill rig. For Borehole NGT-06, the drilling was again carried out from highway grade, however a Hilty drill rig was used. NW casing was used to advance the boreholes through the overburden deposits and NQ coring methods were used to advance selected boreholes through the bedrock. 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 in selected boreholes 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.

One standpipe piezometer, consisting of 19 mm diameter PVC pipe with slotted screen and enclosed in filter sand, was installed at this site to permit longer term groundwater level monitoring. Upon completion, the boreholes were backfilled with bentonite holeplug in general accordance with O.Reg. 903. The location and completion details of the piezometer and boreholes are presented in Table 3.1.

Table 3.1 – Borehole Abandonment Details

Location	Borehole	Piezometer Tip Depth/ Elevation (m)	Abandonment Details
West Approach	NGT-01	None installed	Borehole backfilled with bentonite holeplug from 11.3 m to 0.3 m, concrete from 0.3 m to 0.1 m, then asphalt cold patch to surface.
West Abutment	NGT-02	None installed	Borehole backfilled with bentonite holeplug from 21.2 m to 0.3 m, concrete from 0.3 m to 0.1 m, then asphalt cold patch to surface.
	NGT-03	None installed	Borehole backfilled with bentonite holeplug from 16.2 m to 0.3 m, concrete from 0.3 m to 0.1 m, then asphalt cold patch to surface.
East Abutment	NGT-04	16.8 / 354.0	Sand from 16.8 m to 14.9 m, bentonite holeplug from 14.9 m to 0.3 m, concrete from 0.3 m to 0.1 m, then asphalt cold patch to surface.
	NGT-05	None installed	Borehole backfilled with bentonite holeplug from 21.3 m to 0.3 m, concrete from 0.3 m to 0.1 m, then asphalt to surface.
East Approach	NGT-06	None installed	Borehole backfilled with sand to 0.1 m, then asphalt cold patch to surface.

The piezometers will be decommissioned in accordance with O. Reg. 903 prior to the end of 2012.

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 summarized on the Record of Borehole sheets in Appendix A and shown on the figures included in Appendix B.

Point load tests were carried out on selected samples of intact bedrock upon arrival at the laboratory to assist in evaluation of the compressive strength of the bedrock. Results of point load tests on the 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" drawings in Appendix F. 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 asphalt and at some locations asphalt over concrete overlying granular fill, which is underlain by native sand to sand and gravel layers. Native silty clay was encountered below the sand and gravel layer. The silty clay was underlain by silty sand/sandy silt layers. Slightly weathered to fresh, grey, mafic to intermediate metavolcanic bedrock was contacted below the silty sand/sandy silt at depths ranging from 16.2 m to 18.3m.

More detailed descriptions of the individual strata are presented below.

5.1 Asphalt and Concrete

Asphalt was encountered surficially in all the boreholes drilled through the existing Highway 17 roadway. The thickness of the asphalt ranged from 75 mm on the west side of the bridge to 150 mm on the east side of the bridge.

Concrete was encountered below the asphalt in Boreholes NGT-02 to NGT-05, which were drilled through the existing bridge approach slabs. The concrete was 200 mm to 250 mm thick.

5.2 Sand to Sand and Gravel Fill

Granular fill consisting of sand to sand and gravel was encountered below the asphalt in Boreholes NGT-01 and NGT-06 and below the concrete in Boreholes NGT-02 to NGT-05. The granular fill is brown to grey and contains trace silt and clay and cobbles and boulders. Wood fragments and fibres were encountered in some of the samples of the granular fill. Coring through boulders encountered within the fill, was required in Boreholes NGT-01 and NGT-02 to advance the boreholes. It must be recognized that embankment fills are heterogeneous in nature and may contain rockfill in areas where no boreholes were drilled.

The thickness of the granular fill ranged from 2.0 m to 3.5 m.

The depth to the base of the granular fill ranged from 2.1 m to 3.8 m (elevations 367.0 to 368.7).

SPT 'N' values recorded in the granular fill generally ranged from 17 to 47 blows for 0.3 m penetration, indicating a compact to dense relative density. SPT 'N' values of 50 blows for less than 0.3 m were also recorded in the granular fill at depths where cobbles or boulders were encountered.

The moisture content of the samples of the granular fill generally ranged from 5% to 20%.

Three samples of the granular fill underwent laboratory gradation analysis, the results of which are presented below. These results are also summarized on the Record of Borehole sheets in Appendix A and presented in Figure B1, Appendix B.

Soil Particles	Percentage (%)
Gravel	52 to 88
Sand	9 to 42
Silt and Clay	2 to 6

5.3 Sand to Sand and Gravel

Native grey sand to sand and gravel layers were encountered below the granular fill in all the boreholes.

Cobbles and boulders were encountered within this granular layer in Boreholes NGT-01 and NGT-05. Coring through cobbles and boulders was required to advance Borehole NGT-05.

The thickness of this layer ranged from 0.9 m to 2.2 m, with a lower boundary at a depth of 4.1 m to 5.3 m (elevations 365.5 to 366.7).

SPT 'N' values recorded in the sand to sand and gravel layer typically ranged from 15 to 26 blows for 0.3 m penetration, indicating a compact relative density. Higher 'N' values of 50 blows for less than 0.3 m penetration were recorded where cobbles or boulders were encountered.

The moisture content of samples of the native sand to sand and gravel layers typically ranged from 8% to 12%. A moisture content of 30% was measured in Borehole NGT-06 at a depth of 3.9 m.

A sample of the sand and gravel was selected for laboratory gradation analysis. The results of this test are summarized on the Record of Borehole sheets in Appendix A and the grain size distribution curve for this sample is plotted on Figure B2, Appendix B. The results of this test are as follows:

Soil Particles	Percentage (%)
Gravel	58
Sand	37
Silt and Clay	5

5.4 Silty Clay

A deposit of silty clay was encountered below this native sand to sand and gravel layers in all the boreholes. The silty clay is typically brown to grey and contains trace sand and occasional wood fibres. The silty clay was described as varved.

Where fully penetrated, the thickness of the silty clay layer ranged from 10.2 m to 11.7 m.

The depth to the base of the silty clay ranged from 15.5 m to 16.3 m (elevations 354.5 to 355.3).

Boreholes NGT-01 and NGT-06 were terminated at a depth of 11.3 m (elevation 359.5) within the silty clay layer.

SPT 'N' values recorded in the silty clay typically ranged from 0 to 8 blows for 0.3 m penetration, indicating a very soft to firm consistency. A SPT 'N' value of 26 blows for 0.3 m penetration was recorded in Borehole NGT-06 at a depth of approximately 5.0 m, near the surface of this layer.

In-situ vane shear tests were carried out to assess the undrained shear strength of soft to firm cohesive deposits. Typically, shear strengths of 8 kPa to 20 kPa were measured in the silty clay layer. A shear strength of 32 kPa was measured in Borehole NGT-04 at a depth of approximately 5.5 m. Based on remoulded shear vane tests, the silty clay had a Sensitivity Value ranging from 3 to 10.

The moisture content of samples of the silty clay ranged from 26% to 88%, typically greater than 50%.

Selected samples of the silty clay underwent laboratory grain size analysis testing and Atterberg Limits testing, the results of which are summarized below. These results are also summarized on the Record of Borehole sheets in Appendix A. The grain size distribution curves for tested samples of the silty clay are included in Figures B3 to B5, Appendix B. The results of the Atterberg Limits tests are also presented in Figures B7 and B8.

Soil Particles	Percentage (%)
Gravel	0
Sand	0 to 3
Silt	18 to 56
Clay	43 to 82

Index	Percentage (%)
Plastic Limit	19 to 22
Liquid Limit	42 to 56

The above results indicate that the silty clay is of medium to high plasticity with group symbols of CI-CH.

5.5 Sandy Silt to Silty Sand

A layer of native grey sandy silt to silty sand was encountered below the silty clay in Boreholes NGT-02 to NGT-05. This sandy silt to silty sand layer contains trace gravel, trace clay and occasional cobbles.

This layer was fully penetrated in Boreholes NGT-02 and NGT-05 and the thickness was 2.0 m and 2.1 m. In both boreholes, the lower boundary of the sandy silt/silty sand was encountered at a depth of 18.3 m (elevation 352.5 and 352.6).

Boreholes NGT-03 and NGT-04 penetrated 0.7 m to 1.0 m into this layer and were terminated upon auger refusal on probable bedrock or boulder at 16.2 m and 16.8 m depth, respectively (elevations 354.7 and 354.1).

SPT 'N' values recorded in the sandy silt/silty sand layer on the west side of the bridge ranged from 7 to 8 blows for 0.3 m penetration, indicating a loose relative density. An SPT 'N' value of 42 blows for 0.3 m penetration was recorded in the silty sand layer on the east side of the bridge, indicating a dense relative density.

The moisture content of samples of the sandy silt to silty sand layer ranged from 8% and 48%.

One sample of the sandy silt was selected for gradation analysis, the results of which are summarized below. These results are also presented on the Record of Borehole sheets in Appendix A and on Figure B6, Appendix B.

Soil Particles	Percentage (%)
Gravel	2
Sand	20
Silt	74
Clay	4

5.6 Bedrock and Refusal

The overburden soils described above are underlain by mafic to intermediate metavolcanic bedrock. The bedrock was grey with occasional white bands and slightly weathered to fresh. Occasional mechanical breaks and horizontal joints were noted throughout the bedrock cores.

Bedrock was proved by coring in Boreholes NGT-02 and NGT-05. Boreholes NGT-03 and NGT-04 were terminated upon auger refusal on probable bedrock. Table 5.1 summarizes depths and elevations to the top of bedrock and auger refusal.

Table 5.1 – Depths and Elevations of Top of Bedrock and Auger Refusal

Foundation Element	Borehole	Top of Bedrock	
		Depth (m)	Elevation (m)
West abutment	NGT-02	18.3*	352.5*
	NGT-03	16.2	354.7
East abutment	NGT-04	16.8	354.1
	NGT-05	18.3*	352.6*

*Bedrock proved by coring

Core recovery in the bedrock was 100%. The RQD values ranged from 78% to 100%, indicating good to excellent rock quality.

The Fracture Index (FI) of the rock, expressed as fractures per 0.3 m of core, was generally less than 2. A FI greater than 5 was noted in Run 2 of Borehole NGT-02.

The estimated unconfined compressive strength of the rock cores ranged generally from 138 MPa to 219 MPa, indicating a very strong rock. Only one point load test result from Borehole NGT-05 Run 1, was 86 MPa, which indicates a strong rock. These estimated rock strength values are interpreted from point load tests that were conducted on rock cores recovered from the boreholes. The results of these tests are summarized on the Record of Borehole sheets included in Appendix A and in Point Load Test Sheets in Appendix B.

5.7 Water Levels

Water levels were monitored in the open boreholes upon completion of drilling. One standpipe piezometer was installed at this site, in Borehole NGT-04, to monitor water levels after completion of drilling. The water levels measured in the piezometer 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	NGT-01	27-Jul-2011	2.6	368.2	Open borehole
West Abutment	NGT-02	23-Jul-2011	0.0	370.8	Open borehole
	NGT-03	25-Jul-2011	2.4	368.4	Open borehole
East Abutment	NGT-04	24-Jul-2011 25-Jul-2011 17-Aug-2011 15-Sep-2011	1.5 0.8 0.5 0.8	369.3 370.0 370.3 370.0	Piezometer
	NGT-05	27-Jul-2011	0.6	370.3	Open borehole
East Approach	NGT-06	25-Jul-2011	2.4	368.4	Open borehole

Piezometric readings indicate that the water level is near elevation 370.1.

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.

Preliminary GA drawings indicate that the water level in Nugget Creek was near Elevation 368.7 on April 28, 2011.

During drilling of Borehole NGT-02, an artesian head of 0.3 m above ground surface was noted in the sandy silt layer at 18.2 m depth (elevation 352.6).

6 MISCELLANEOUS

Borehole locations were selected and established in the field by Thurber Engineering Ltd. Hatch Mott MacDonald surveyed the borehole locations and 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 for Boreholes NGT-01 to NGT-05. Ohlmann Geotechnical Services (OGS) Inc., of Almonte, Ontario supplied a Hilty drill rig and conducted the drilling, sampling and in-situ testing operations for Borehole NGT-06.

The drilling and sampling operations in the field were supervised on a full time basis by Ms. Eckie Siu and Mr. George Azzopardi, both 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. Lindsey Blaine, E.I.T. and Ms. Rocio 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 at the crossing of Highway 17 over Nugget Creek in the Town of Wabigoon, Township of Zealand, Ontario.

The Nugget Creek bridge was constructed in 1951 and underwent rehabilitation in 2001. At present, Highway 17 crosses Nugget Creek on a four-span structure supported on timber piles. The length of the bridge is approximately 25.8 m. The highway grade is near elevation 370.8. The existing embankment heights are approximately 3.0 m to 4.0 m. It is understood 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 with a sheet pile wall driven just behind the H-piles. Precast pre-stressed box girders will then be spanning the abutment pile caps to support the deck finishing. The total length of the structure will be 22.0 m. The proposed structure will be approximately 14.3 m wide. The existing structure will be replaced maintaining the same alignment for the new structure. Hatch Mott MacDonald has indicated that the highway grade will be raised 400 mm.

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 a pavement structure, overlying compact to dense sand and gravel embankment fill. The thickness of the fill varies from 2.0 m to 3.5 m. Below the fill, native layers of compact to very dense sand and gravel and sand were contacted. Cobbles and boulders were encountered within the fill and the native sand and gravel layers. Coring through the cobbles and boulders was required at some locations to advance the boreholes. Native very soft to firm silty clay was contacted below the cohesionless layers. The thickness of the silty clay ranged from 10.2 m to 11.7 m. Loose sandy silt to silty sand with occasional cobbles and boulders was contacted below the silty clay. Bedrock was contacted at 18.3 m depth (elevations 352.5 and 352.6) and was proved by coring in Boreholes NGT-02 and NGT-05. Auger refusal on probable bedrock or boulders was encountered at 16.2 m and 16.8 m depth (elevations 354.7 and 354.1) in Boreholes NGT-03 and NGT-04, respectively.

Piezometric readings indicate that the water level is near elevation 370.1. Preliminary GA drawings indicate that the water level in Nugget Creek was near Elevation 368.7 on April 28, 2011. During drilling of Borehole NGT-02, an artesian head of 0.3 m above ground surface was noted in the sandy silt layer at 18.2 m depth (elevation 352.6).

Based on the existing site conditions, initial consideration was given to the following foundation types:

- Spread footings on native soils
- Augered Caissons (drilled shafts)
- Steel H-piles driven to bedrock

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

8.1 Spread Footings on Native Soils

Consideration was given to supporting the structure on spread footings founded on native soils, however this option is not recommended due to the following reasons:

- Low geotechnical capacities are present at this site in the native soils below the existing fill. An extensive deposit of very soft to firm clay exists below the upper granular soils which will be subjected to settlement under the footing loads.

- High water levels were encountered at this site during drilling. Any footing excavation will require prior dewatering.
- Spread footings could be subject to erosion or undermining/scour during high river flows.
- Footing excavation may adversely impact the adjacent creek.

In light of the above factors, the spread footings option was not further developed

8.2 Augered Caissons (drilled shafts)

Augered caisson foundations bearing on bedrock were also considered for supporting the structure at this site. The bedrock was contacted at 18.3 m depth below ground surface and approximately 17.5 below groundwater level.

The permeable nature of the granular soils immediately above the bedrock would make it difficult to seal the bottom of the caisson liner into the founding stratum to exclude groundwater and sloughing soils. Unwatering of the caissons may result in continued flow of fines into the caisson excavation.

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

8.3 Driven Piles to Bedrock

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

The elevations at which bedrock was contacted are given in Table 8.1.

Table 8.1– Estimated Pile Tip Elevation

Foundation Unit	Borehole	Anticipated Pile Tip Elevation for piles driven to bedrock
West abutment	NGT-02	352.5[(1)]
East abutment	NGT-05	352.6[(1)]

[(1)]Bedrock proved by coring. Depth to bedrock from existing Highway 17 grade was 18.3 m.

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 8.3.5 Pile Driving.

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.

8.3.1 Pile Tips

For H-piles driven to bedrock, 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.

Rockfill, cobbles and boulders were encountered within the sand and gravel fill and native sand/sand and gravel layers. The same pile tip protection and reinforcement as founding on bedrock will be required at locations were boulders are contacted.

8.3.2 Pile Installation

Pile installation should be in accordance with OPSS 903.

The embankment fill is noted to contain cobbles, boulders and rockfill. Coring was required to get through these obstructions. Pre-augering in the fill may be required if the piles meet refusal on such obstructions.

The location of new piles should be carefully chosen so that the piles of the existing structure do not interfere with driving of new piles.

8.3.3 Pile Driving

We understand that the proposed bridge design may require that the deviation at the top of the pile be limited to 12 mm. Use of a driving template or other means may be required to achieve this high tolerance.

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".

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."

8.3.4 Downdrag

Hatch Mott MacDonald indicated that the highway grade will be raised 400 mm. Downdrag forces will therefore develop along the length of the pile embedded in the silty clay layer. For design purposes, an unfactored downdrag load of 220 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.

8.3.5 Artesian

An artesian head was noted in the sandy silt layer in Borehole NGT-02 at 18.2 m depth (elevation 352.6).

Artesian pressure has the potential to cause flow up the pile shaft, with accompanying loss of fines. However, since there is a considerable thick layer of clay above the sandy silt layer, it is expected that this clay layer will seal around the pile, minimizing the potential for upward flow around a pile shaft.

8.3.6 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:

$$k_s = n_h \, . \, z / D \qquad (kN/m^3)$$

$$p_{ult} = 3 \, . \, \gamma \, . \, z \, . \, K_p \qquad (kPa)$$

where
z = depth of embedment of pile in metres
D = pile width in metres
n_h = value from Table 8.2
γ = 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.

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 367.8	5,000	-	3.3	11*	Sand and gravel, dense to compact (FILL)
	367.8 to 366.2	4,000	-	3.0	11*	Sand and gravel, compact
	366.2 to 354.5	-	25	2.7	10*	Silty clay, very soft to firm
	354.5 to 352.5	3,500	-	3.0	11*	Silty sand/sandy silt, loose

*Buoyant unit weight below the water table.

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.

8.4 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 2.5 m.

Frost protection should be provided for pile caps, if used, and should consist of a minimum of 2.5 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.

The existing approach fill contains cobbles, boulders and possibly some rockfill.

Driving of the sheet piles through the existing approach fill (compact to dense sand and gravel) may encounter these obstructions. If such obstructions are encountered, they will to be removed to facilitate driving sheet piles.

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 native soils. 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)

γ = 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 abutment wall are dependent on the material used as backfill. Typical values 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 $\phi = 35°$, $\gamma = 22.8\ kN/m^3$		Existing Sand and gravel (fill and native), and OPSS Granular B Type I $\phi = 32°$, $\gamma = 21.2\ kN/m^3$		Native Silty Clay $\phi = 27°$, $\gamma = 20\ kN/m^3$	
	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)
Active (Unrestrained Wall)	0.27	0.40*	0.31	0.48*	0.38	0.75*
At rest (Restrained Wall)	0.43	-	0.47	-	0.55	-
Passive (Movement Towards Soil Mass)	3.7	-	3.3	-	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

Any temporary earth excavation must be carried out in accordance with the Occupational Health and Safety Act (OHSA). For the purposes of the OHSA, the native soils within the probable depth of excavation at this site may be classed as Type 3 soils above the water table and Type 4 soils below the water table.

The excavation must be carried out in accordance with OPSS 902.

Piezometric readings indicate that the water level is near elevation 370.1. Preliminary GA drawings indicate that the water level in Nugget Creek was near Elevation 368.7 on April 28, 2011.

Based on the preliminary GA for the bridge structure and the use of pile foundations, it is not expected that work at the abutments will require excavation below the river/groundwater level.

It is recommended that any excavation for removal of existing structures be maintained above the water level in the river. Any excavation below the groundwater level/river level without dewatering is not recommended since the inflow of groundwater will make difficult to maintain a dry, sound base on which to work.

The Contract Documents should contain a NSSP alerting the Contractor to the risks associated with excavation of soils submerged below the groundwater level without dewatering.

In general, the design of the dewatering system should be the responsibility of the Contractor and the Contract Documents should alert him to this responsibility.

11 APPROACH EMBANKMENTS

Based on site observations, borehole logs and the GA drawing provided by Hatch Mott MacDonald, it is estimated that the existing approach embankments are 3.0 m to 4.0 m in heights. The foundation soils governing stability of the approach embankments consist generally of existing layers of native compact to very dense sand and sand/gravel with cobbles and boulders andvery soft to firm silty clay.

Communication with Hatch Mott MacDonald indicates that the existing Highway 17 grade will be raised 400 mm.

For placement of new fill adjacent to the existing embankment, the existing slope surfaces should be appropriately benched, as per OPSD 208.010, after stripping of vegetation, topsoil, organics, soft soils or otherwise unsuitable overburden materials.

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

11.1 Slope stability

The global, internal and surficial stability of the approach embankment fills depends on the slope geometry, nature of the foundation soils and on the material used to construct the embankment.

The existing embankment at this site has performed well under the existing conditions. After the bridge replacement is completed, it is expected that the embankment will continue to perform satisfactorily if it is constructed with similar materials and the existing embankment slopes are maintained. The grade raise of 400 mm is not anticipated to adversely impact the stability of the approach embankment particularly, since the grade raise is contained within a sheet pile cofferdam.

11.2 Settlement

A grade raise of 400 mm will induce immediate (elastic) settlement in the non-cohesive foundation soils 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 total settlement is estimated to be about 23 mm, for a highway grade raise of 400 mm at this site.

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.

12 EROSION PROTECTION

Erosion and scour protection should be provided along the lower parts of any slopes that may be in contact with the river flow.

A vegetation cover should be establish

hed on all other exposed earth surfaces to protect against surficial erosion, in general accordance with OPSS 804.

13 BACKFILL TO ABUTMENT

Backfill to the abutment if required, must consist of granular material.

Backfill to the abutments should consist of Granular A or Granular B Type II material meeting the requirements of Special Provision 110S13 "Amendment to OPSS 1010". The backfill must be in accordance with OPSS 902, and placed to the extents shown in OPSD 3101.150.

All new embankment earth fill should be placed in uniform lifts and be compacted in accordance with OPSS 501. Also, compaction equipment to be used adjacent to retaining structures must be restricted in accordance OPSS 501.

14 ROADWAY PROTECTION

If roadway protection is required, such protection should be provided in accordance with OPSS 539 and designed for Performance Level 2.

Conventional steel soldier pile and timber lagging walls is one option 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. If the soldier piles meet obstructions in the highway embankment fill, these obstructions must be removed to facilitate pile installation.

The following parameters apply for design of the temporary shoring system:

γ	=	21 kN/m^3	(bulk unit weight)
γ_w	=	11 kN/m^3	(submerged unit weight under groundwater table)
K_a			(Active pressure coefficient for:
	=	0.31	road embankment sand and gravel fill and native sand and gravel
	=	0.38	native silty clay
K_p			(Passive pressure coefficient for:
	=	3.3	road embankment sand fill and native sand and gravel
	=	2.7	native silty clay
h_w	=	0	(assuming that the groundwater is maintained below the base of the excavation and that there is no hydrostatic pressure build-up behind a presumably permeable wall)
h_w	=	370.1	(elevation for hydrostatic pressure build-up behind wall)

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 may be required during construction.

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.

15 SEISMIC CONSIDERATIONS

15.1 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 and gravel (fill and native), and OPSS Granular B Type I** $\phi = 32°$, $\gamma = 21.2$ kN/m^3	**Native Silty Clay** $\phi = 27°$ $\gamma = 20$ kN/m^3
Active (K_{AE})*	0.28	0.32	0.39
Passive (K_{PE})	3.7	3.2	2.7
At Rest (K_{OE})**	0.45	0.50	0.57

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

The potential for liquefaction of the foundations soils was assessed using the Seed and Idriss (1971) method for cohesionless soils and the Bray et al. (2004) criteria, based on Figure 6.15 of CFEM, for fine-grained soils (cohesive soils).

The results of these methods indicate that the site is not susceptible for liquefaction under current conditions.

16 CONSTRUCTION CONCERNS

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

- Cobbles and boulders may be encountered within the highway embankment fill and native sand/sand and gravel during pile driving operations. These obstructions, if encountered, may have to be removed to facilitate driving of the H-piles and sheet piles.
- Excavation, if required, should be maintained above the water level in the creek.

- Roadway protection must be provided to maintain traffic during construction. Temporary shoring systems should be properly designed by a Professional Engineer experienced in such designs.
- The side embankment slopes should be inspected after construction for surficial disturbance. Where necessary, erosion control measures must be implemented

17 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.

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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

RECORD OF BOREHOLE No NGT-01

METRIC

W.P. 6936-10-00	LOCATION N 5 509 138.5 E 333 625.7 Nugget River Bridge	ORIGINATED BY GA
HWY 17	BOREHOLE TYPE Casing	COMPILED BY AN
DATUM Geodetic	DATE 2011.07.27 - 2011.07.27	CHECKED BY RPR

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
370.8 / 0.0 / 0.1	**ASPHALT:** (75mm)							
	SAND, trace gravel Dense to Compact Brown Wet (FILL) Cored through boulder at 0.6m	1	SS	39			○	
		2	SS	39		370	○	
		3	SS	21		369	○	
368.7 / 2.1	**SAND** and **GRAVEL**, with cobbles and boulders Very Dense Brown	4	SS		▽			
		5	SS	50/ 0.0		368		
366.5 / 4.3	Silty **CLAY**, trace sand Firm to Very Soft Grey					367		
		6	SS	7		366	○	0 2 35 63
						365		
		7	SS	2		364	○	
							Field vane + (4)	
		8	SS	1		363	○	
						362		
		9	SS	1		361	├──○┤	0 0 32 68

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

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

RECORD OF BOREHOLE No NGT-01

 METRIC

W.P. 6936-10-00 | LOCATION N 5 509 138.5 E 333 625.7 Nugget River Bridge | ORIGINATED BY GA

HWY 17 | BOREHOLE TYPE Casing | COMPILED BY AN

DATUM Geodetic | DATE 2011.07.27 - 2011.07.27 | CHECKED BY RPR

ELEV DEPTH	DESCRIPTION	STRAT PLOT	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
	Continued From Previous Page										
	Silty **CLAY** Very Soft Grey							$+^{6}$			
			10	SS	1		360		o		
359.5 11.3	END OF BOREHOLE AT 11.3m. WATER LEVEL AT 2.6m UPON COMPLETION OF DRILLING. BOREHOLE BACKFILLED WITH HOLEPLUG FROM 11.3m TO 0.3m, CONCRETE TO 0.3m 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

THURBER

RECORD OF BOREHOLE No NGT-02

METRIC

W.P. 6936-10-00 | LOCATION N 5 509 130.3 E 333 630.7 Nugget River Bridge | ORIGINATED BY GA

HWY 17 | BOREHOLE TYPE NW Casing | COMPILED BY AN

DATUM Geodetic | DATE 2011.07.22 - 2011.07.23 | CHECKED BY RPR

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%) / SHEAR STRENGTH	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
370.8							
0.0	**ASPHALT:** (75mm)						
370.5 0.1 / 0.3	**CONCRETE:** (200mm)						
	SAND and GRAVEL Dense to Compact Brown Damp (FILL)	1	SS	45		o	
		2	SS	47	370	o	
		3	SS	19	369	o	52 42 6 (SI+CL)
	Coring through rockfill at 2.1m	~~4~~	~~SS~~	~~50/~~ 0.0			
	Wood fragments from 2.5m to 2.8m				368		
367.8 3.0	**SAND and GRAVEL** Compact Grey Wet	5	SS	18		o	
					367		
366.2 4.6	Silty **CLAY**, varved Very Soft Grey	6	SS	2	366	o	
						+ 10 (field vane)	
					365		
		7	SS	2		o	
					364		
		8	SS	1	363	wp–wL with o	0 0 29 71
						+ 4 (field vane)	
					362		
		9	SS	1		o	
					361		

Legend: 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/m^3; GROUND WATER CONDITIONS; STRAT PLOT.

Continued Next Page

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

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

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No NGT-02

METRIC

W.P.	6936-10-00	LOCATION	N 5 509 130.3 E 333 630.7 Nugget River Bridge	ORIGINATED BY	GA
HWY	17	BOREHOLE TYPE	NW Casing	COMPILED BY	AN
DATUM	Geodetic	DATE	2011.07.22 - 2011.07.23	CHECKED BY	RPR

Continued From Previous Page

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	DCPT / SHEAR STRENGTH	WATER CONTENT	UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Silty **CLAY**, varved Very Soft Grey								
		10	SS	2	360				0 0 37 63
						5			
					359				
		11	SS	1					
					358				
		12	SS	1	357				
						+4			
					356				
		13	SS	1					
					355				
354.5 16.3	Sandy **SILT**, trace gravel, trace clay Loose Grey								
					354				
		14	SS	7					2 20 74 4
	Occasional cobbles at 17.4m								
					353				
352.5	Artesian pressure at 18.2m							FI	
18.3	**BEDROCK**, mafic to intermediate metavolcanic, slightly weathered to fresh, grey, occasional white bands, occasional mechanical breaks Coring started at 18.3m							2 0	
					352				
	Horizontal joint at 19.1m	1	RUN					1	RUN #1 TCR=100% SCR=77% RQD=93% UCS=190MPa (Average)
	Sub-horizontal joints at 18.3m, 18.4m and 19.2m							0 0	
					351				

Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity

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

ONTMT4S 5121.GPJ 10/5/12

RECORD OF BOREHOLE No NGT-02

METRIC

W.P.	6936-10-00	LOCATION	N 5 509 130.3 E 333 630.7 Nugget River Bridge	ORIGINATED BY	GA
HWY	17	BOREHOLE TYPE	NW Casing	COMPILED BY	AN
DATUM	Geodetic	DATE	2011.07.22 - 2011.07.23	CHECKED BY	RPR

ELEV DEPTH	DESCRIPTION (SOIL PROFILE)	STRAT PLOT	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, 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										
349.6 21.2	**BEDROCK**, mafic to intermediate metavolcanic, slightly weathered to fresh, grey, occasional white bands, occasional mechanical breaks Highly broken zone at: 125mm at 20.0m 175mm at 20.4m Horizontal joints at 20.0m, 20.1m, 20.4m and 20.6m		2	RUN			350			>5 0 >5 0 0	RUN #2 TCR=100% SCR=78% RQD=78% UCS=185MPa ~~(Average)~~
	END OF BOREHOLE AT 21.2m. WATER LEVEL AT SURFACE UPON COMPLETION OF DRILLING. BOREHOLE BACKFILLED WITH HOLEPLUG FROM 21.2m TO 0.3m, CONCRETE FROM 0.3m TO 0.1m, THEN ASPHALT PATCH TO SURFACE.										

ONTMT4S 5121.GPJ 7/11/12

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

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

RECORD OF BOREHOLE No NGT-03

METRIC

W.P. 6145-04-00 LOCATION N 5 509 134.9 E 333 633.3 Nugget River Bridge ORIGINATED BY GA

HWY 17 BOREHOLE TYPE Casing COMPILED BY AN

DATUM Geodetic DATE 2011.07.25 - 2011.07.25 CHECKED BY RPR

ELEV / DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
370.8							
0.0	**ASPHALT:** (75mm)						
370.5 / 0.1 / 0.3	**CONCRETE:** (225mm)						
	SAND and **GRAVEL**, trace silt, trace clay, occasional cobbles; Dense to Compact; Brown; Wet; (FILL)	1	SS	32			
		2	SS	37		370	
		3	SS	32		369	
	Grey; Boulders and cobbles from 2.2m to 2.8m	4	SS	45	∇		76 22 2 (SI+CL)
		5	SS	21		368	
367.0 / 3.8	**SAND**, some gravel, occasional wood fibres; Compact; Grey; Wet					367	
		6	SS	15		366	
365.5 / 5.3	Silty **CLAY**; Soft; Grey					365	
		7	SS	3			0 0 18 82
						364	
		8	SS	1		363	
						362	
		9	SS	1		361	0 0 27 73

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

Field vane at approx. elev. 363.8: +4

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

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

RECORD OF BOREHOLE No NGT-03

METRIC

W.P. 6936-10-00 | LOCATION N 5 509 134.9 E 333 633.3 Nugget River Bridge | ORIGINATED BY GA
HWY 17 | BOREHOLE TYPE Casing | COMPILED BY AN
DATUM Geodetic | DATE 2011.07.25 - 2011.07.25 | CHECKED BY RPR

Continued From Previous Page

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	SHEAR STRENGTH (kPa)	WATER CONTENT (%)	GR	SA	SI	CL
	Silty **CLAY** Very Soft Grey					+ (field vane) 6					
		10	SS	1	360		w				
					359						
		11	SS	0			w				
					358	+ (field vane) 4					
		12	SS	1	357		w_P–w, w_L	0	0	47	53
					356						
355.3 15.5	Sandy **SILT** Loose Grey Wet	13	SS	8			w				
					355						
354.7 16.2	END OF BOREHOLE AT 16.2m UPON REFUSAL. WATER LEVEL AT 2.4m UPON COMPLETION OF DRILLING. BOREHOLE BACKFILLED WITH HOLEPLUG FROM 16.2m TO 0.3m, CONCRETE FROM 0.3m TO 0.1m, THEN ASPHALT TO SURFACE.										

Chart headings: 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 (%); GROUND WATER CONDITIONS; STRAT PLOT.

+ ³, × ³ : Numbers refer to Sensitivity

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

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No NGT-04

METRIC

W.P. 6936-10-00	LOCATION N 5 509 117.5 E 333 658.1 Nugget River Bridge	ORIGINATED BY GA
HWY 17	BOREHOLE TYPE	COMPILED BY AN
DATUM Geodetic	DATE 2011.07.24 - 2011.07.24	CHECKED BY RPR

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
370.8						
0.0	**ASPHALT:** (100mm)					
370.6 / 0.3	**CONCRETE:** (250mm)	~~1~~	~~SS~~			
	SAND and **GRAVEL** Compact Brown Damp (FILL) Boulder at 0.6m	2	SS	24	o	
		3	SS	20	o	
	Cobbles at 2.1m Boulders (200mm) from 2.3m to 2.5m Cobbles and boulders at 3.0m	~~4~~	~~SS~~			
367.8 / 3.0	**SAND** and **GRAVEL**, trace silt and clay Compact Wet	5	SS	26	o	58 37 5 (SI+CL)
366.6 / 4.3	Silty **CLAY**, trace sand Firm to Very Stiff Brown	6	SS	6	Wp–w–WL	0 3 50 47
					Field vane + (sensitivity 3)	
		7	SS	1	o	
		1	TW	PH		
					Field vane + (sensitivity 4)	
		8	SS	1	o	

Elevation scale: 370, 369, 368, 367, 366, 365, 364, 363, 362, 361

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/m^3

Continued Next Page

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

THURBER

RECORD OF BOREHOLE No NGT-04

METRIC

W.P. 6936-10-00 | LOCATION N 5 509 117.5 E 333 658.1 Nugget River Bridge | ORIGINATED BY GA

HWY 17 | BOREHOLE TYPE | COMPILED BY AN

DATUM Geodetic | DATE 2011.07.24 - 2011.07.24 | CHECKED BY RPR

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/m³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	SHEAR STRENGTH (kPa)	GR	SA	SI	CL
	Continued From Previous Page									
	Silty **CLAY**, trace sand Very Soft Reddish Brown to Grey									
		9	SS	1	360					
					359	+ 4				
		10	SS	0			0	0	41	59
					358					
		11	SS	1	357					
					356	+ 6				
355.0		12	SS	6						
15.8	Sandy **SILT** Grey Cobbles from 15.9m to 16.6m				355					
354.1 16.8	END OF BOREHOLE AT 16.8m UPON REFUSAL ON PROBABLE BEDROCK. BOREHOLE OPEN TO 16.8m AND WATER LEVEL AT 1.5m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen.									

WATER LEVEL READINGS:

DATE	DEPTH (m)	ELEV. (m)
Jul.25/11	0.8	370.0
Aug.17/11	0.5	370.3
Sep.15/11	0.8	370.0

+ 3 , × 3 : Numbers refer to Sensitivity

20 15 5 10 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No NGT-05

METRIC

W.P. 6936-10-00 LOCATION N 5 509 122.5 E 333 660.6 Nugget River Bridge ORIGINATED BY GA

HWY 17 BOREHOLE TYPE Casing COMPILED BY AN

DATUM Geodetic DATE 2011.07.26 - 2011.07.27 CHECKED BY RPR

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
370.9							
0.0	**ASPHALT:** (100mm)						
370.8 0.1	**CONCRETE:** (200mm)						
0.3	**SAND** and **GRAVEL** Dense to Compact Brown Wet (FILL)	1	SS	30	▽		
		2	SS	32		370	
		3	SS	28		369	88 9 3 (SI+CL)
	Occasional cobbles	4	SS	38		368	
367.8		5	SS				
3.1	**SAND** and **GRAVEL**, with cobbles and boulders Cored through cobbles and boulders from 2.9m to 4.3m					367	
366.3							
4.6	Silty **CLAY**, varved Very Soft Grey	6	SS	2		366	
						365	
		7	SS	1		364	0 0 43 57
		8	SS	1		363	
						362	
		9	SS	1		361	

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

Field vane: +4 (≈365.3), +3 (≈362.3)

PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 20 40 60

UNIT WEIGHT γ kN/m^3

Continued Next Page

+ ^{3}, × ^{3}: Numbers refer to Sensitivity

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

RECORD OF BOREHOLE No NGT-05

METRIC

W.P.	6936-10-00	LOCATION	N 5 509 122.5 E 333 660.6 Nugget River Bridge
HWY	17	BOREHOLE TYPE	Casing
DATUM	Geodetic	DATE	2011.07.26 - 2011.07.27

ORIGINATED BY GA
COMPILED BY AN
CHECKED BY RPR

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 (○ 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)

Continued From Previous Page

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	SHEAR STRENGTH (Field Vane)	WATER CONTENT	UNIT WEIGHT	REMARKS / GRAIN SIZE (GR SA SI CL)
	Silty **CLAY**, varved Very Soft Grey								
		10	SS	1	360				0 0 46 54
						$+^3$			
					359				
		11	SS	0					
					358				
		12	SS	2	357				
						$+^3$			
					356				
	Soft	13	SS	4					
354.7 16.2	Silty **SAND**, trace gravel, occasional cobbles Dense Grey Wet				355				
		14	SS	42	354				
					353			FI	
352.6 18.3	**BEDROCK**, mafic to intermediate metavolcanic, slightly weathered to fresh, grey, occasional white bands, occasional mechanical breaks Coring started at 18.3m							0 0	
		1	RUN		352			0 0 0	RUN #1 TCR=100% SCR=100% RQD=100% UCS=145MPa (Average)
					351				

Continued Next Page

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

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

RECORD OF BOREHOLE No NGT-05

METRIC

W.P. 6936-10-00 | LOCATION N 5 509 122.5 E 333 660.6 Nugget River Bridge | ORIGINATED BY GA
HWY 17 | BOREHOLE TYPE Casing | COMPILED BY AN
DATUM Geodetic | DATE 2011.07.26 - 2011.07.27 | CHECKED BY RPR

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
	Continued From Previous Page										
	BEDROCK, mafic to intermediate metavolcanic, slightly weathered to fresh, grey, occasional white bands, occasional mechanical breaks Quartz seam at 20.2m		2	RUN			350			0 0 0 0 0	RUN #2 TCR=100% SCR=100% RQD=100% UCS=205MPa (Average)
349.5 21.3	END OF BOREHOLE AT 21.3m. WATER LEVEL AT 0.6m UPON COMPLETION OF DRILLING. BOREHOLE BACKFILLED WITH HOLEPLUG FROM 21.3m TO 0.3m, CONCRETE FROM 0.3m TO 0.1m, THEN ASPHALT TO SURFACE.										

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

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

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No NGT-06

METRIC

W.P. 6936-10-00	LOCATION N 5 509 118.4 E 333 669.9 Nugget River Bridge	ORIGINATED BY ES	
HWY 61	BOREHOLE TYPE Casing	COMPILED BY AN	
DATUM	DATE 2011.07.25 - 2011.07.25	CHECKED BY RPR	

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
370.8					
0.0	**ASPHALT:** (150mm)				
0.2	**SAND** and **GRAVEL** Brown Moist (FILL)	1	GS		
	Occasional cobbles	2	GS		
	Compact	1	SS	25	
		2	SS	17	
		3	SS	25	
367.6	Occasional wood fibres				
3.2	**SAND**, some gravel Compact Moist	4	SS	16	
366.7					
4.1	Silty **CLAY**, trace sand, occasional wood pieces Very Stiff Grey	5	SS	26	0 1 56 43
	Very Soft	6	SS	2	
		7	SS	3	
		8	SS	6	0 0 38 62

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE (370, 369, 368, 367, 366, 365, 364, 363, 362, 361); 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

Continued Next Page

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

20 15 5 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No NGT-06

METRIC

W.P. 6936-10-00	LOCATION N 5 509 118.4 E 333 669.9 Nugget River Bridge		ORIGINATED BY ES
HWY 17	BOREHOLE TYPE Casing		COMPILED BY AN
DATUM Geodetic	DATE 2011.07.25 - 2011.07.25		CHECKED BY RPR

ELEV DEPTH	SOIL PROFILE 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
	Continued From Previous Page										
	Silty **CLAY**, varved Firm Reddish Brown to Grey		9	SS	8		360		>>O		
359.5 11.3	END OF BOREHOLE AT 11.3m. WATER LEVEL AT 2.4m UPON COMPLETION OF DRILLING. BOREHOLE BACKFILLED WITH SAND TO 0.05m, THEN ASPHALT TO SURFACE.										

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

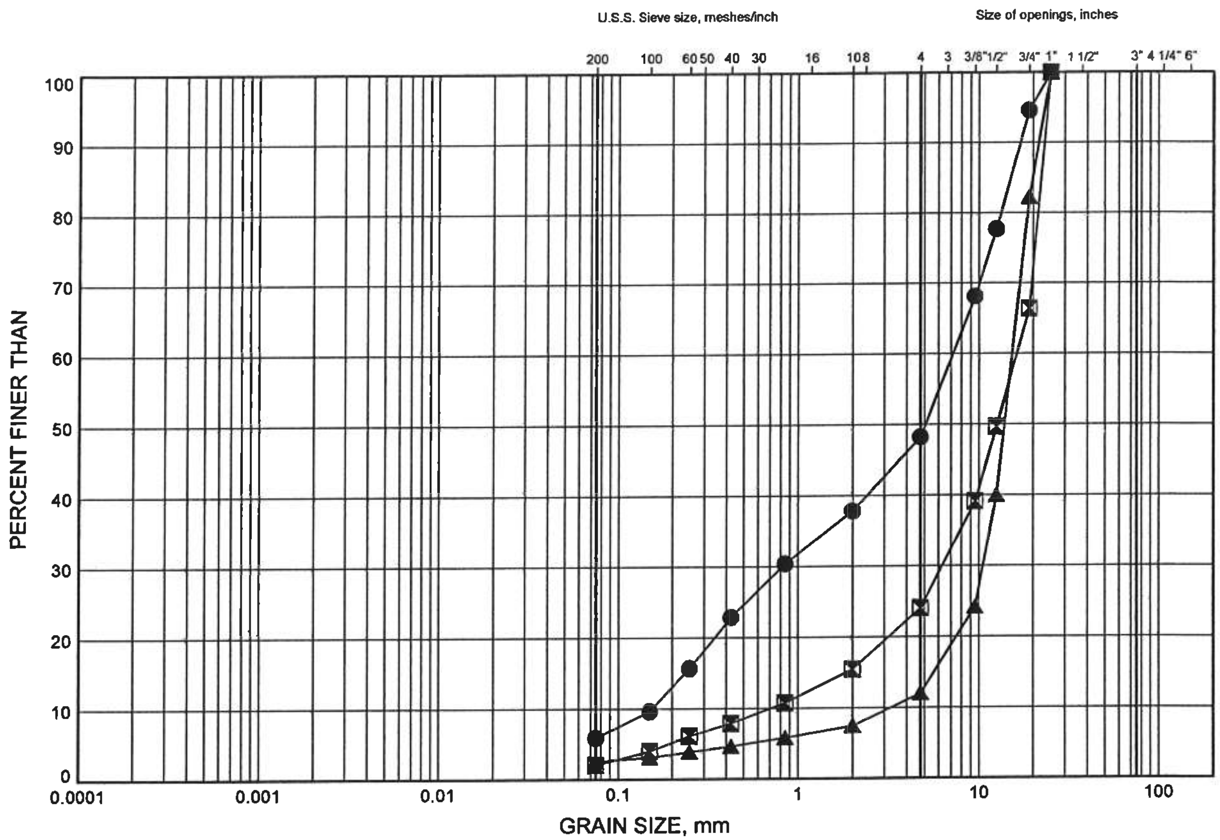
15-⊕-5 (20 above, 10 below) (%) STRAIN AT FAILURE

Appendix B

Laboratory Test Results

6010-E-0010 Bridge and Culvert Rehabs NWR GRAIN SIZE DISTRIBUTION	FIGURE B1

SAND & GRAVEL FILL



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	NGT-02	1.83	368.98
⊠	NGT-03	2.59	368.24
▲	NGT-05	1.83	369.02

W.P.# 6145-04-00

Prepared By AN

Checked By LRB

THURBER

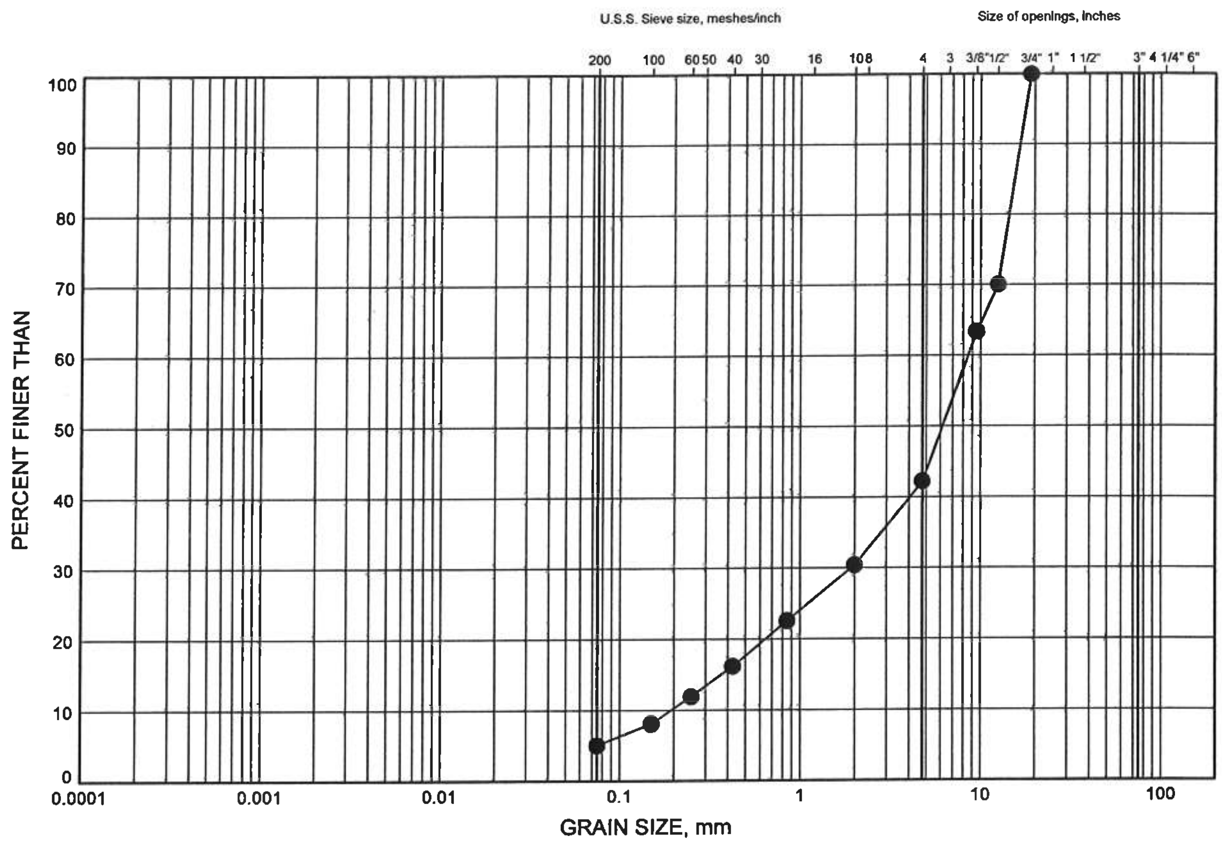
GRAIN SIZE DISTRIBUTION - THURBER 5121.GPJ 11/23/11

6010-E-0010 Bridge and Culvert Rehabs NWR

GRAIN SIZE DISTRIBUTION

FIGURE B2

SAND & GRAVEL



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	NGT-04	3.35	367.49

W.P.# 6145-04-00
Prepared By AN
Checked By LRB

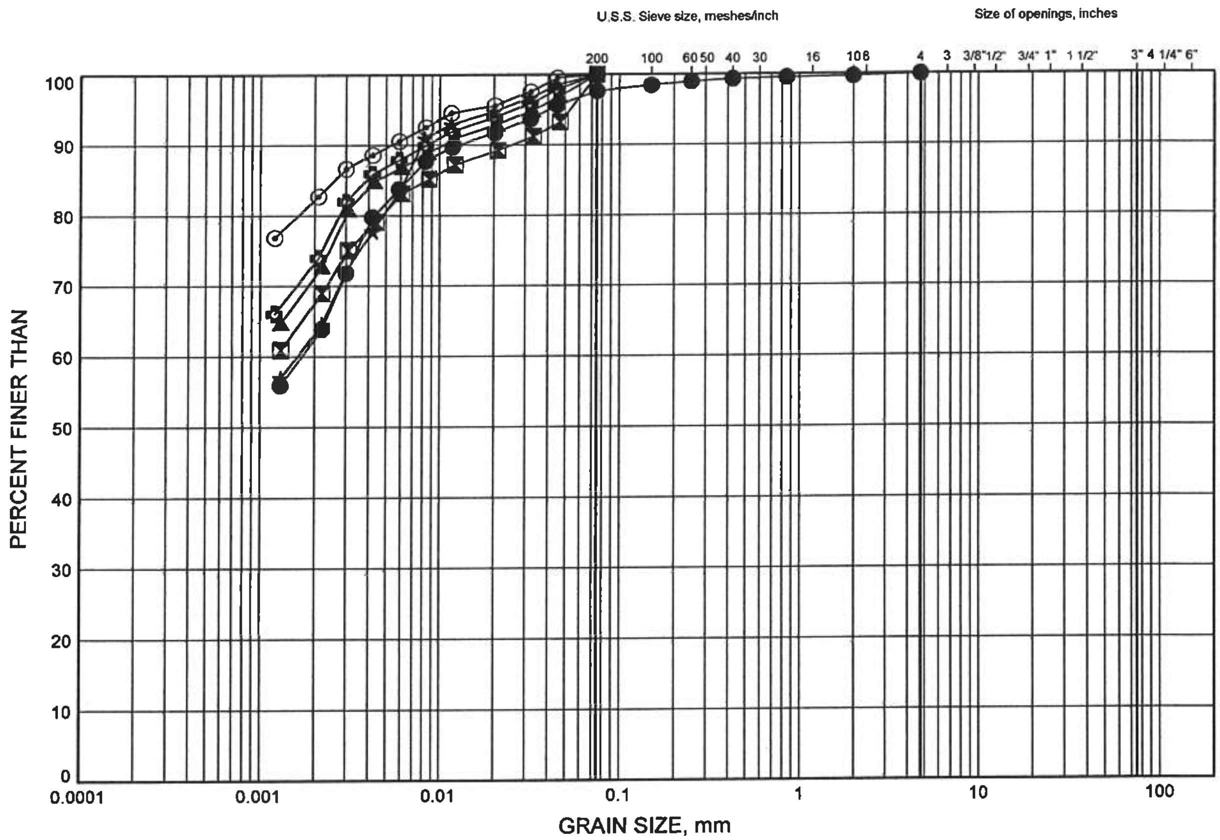
THURBER

6010-E-0010 Bridge and Culvert Rehabs NWR

GRAIN SIZE DISTRIBUTION

FIGURE B3

SILTY CLAY



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	NGT-01	4.88	365.92
⊠	NGT-01	9.45	361.34
▲	NGT-02	7.92	362.88
★	NGT-02	10.97	359.83
⊙	NGT-03	6.40	364.43
✿	NGT-03	9.45	361.39

W.P.# 6145-04-00

Prepared By AN

Checked By LRB

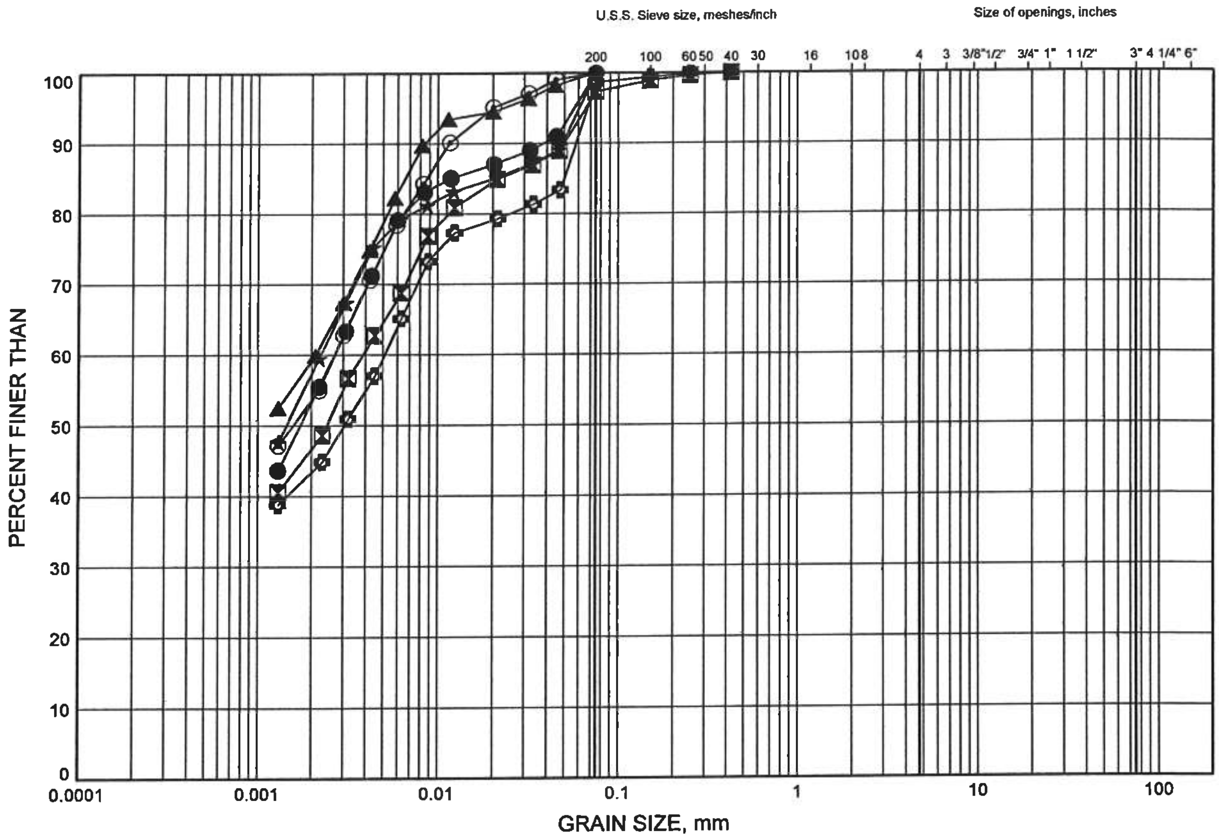
THURBER

6010-E-0010 Bridge and Culvert Rehabs NWR

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)
●	NGT-03	14.02	356.81
⊠	NGT-04	4.88	365.97
▲	NGT-04	12.50	358.35
★	NGT-05	6.40	364.45
⊙	NGT-05	10.97	359.88
✿	NGT-06	4.88	365.94

W.P.# 6145-04-00

Prepared By AN

Checked By LRB

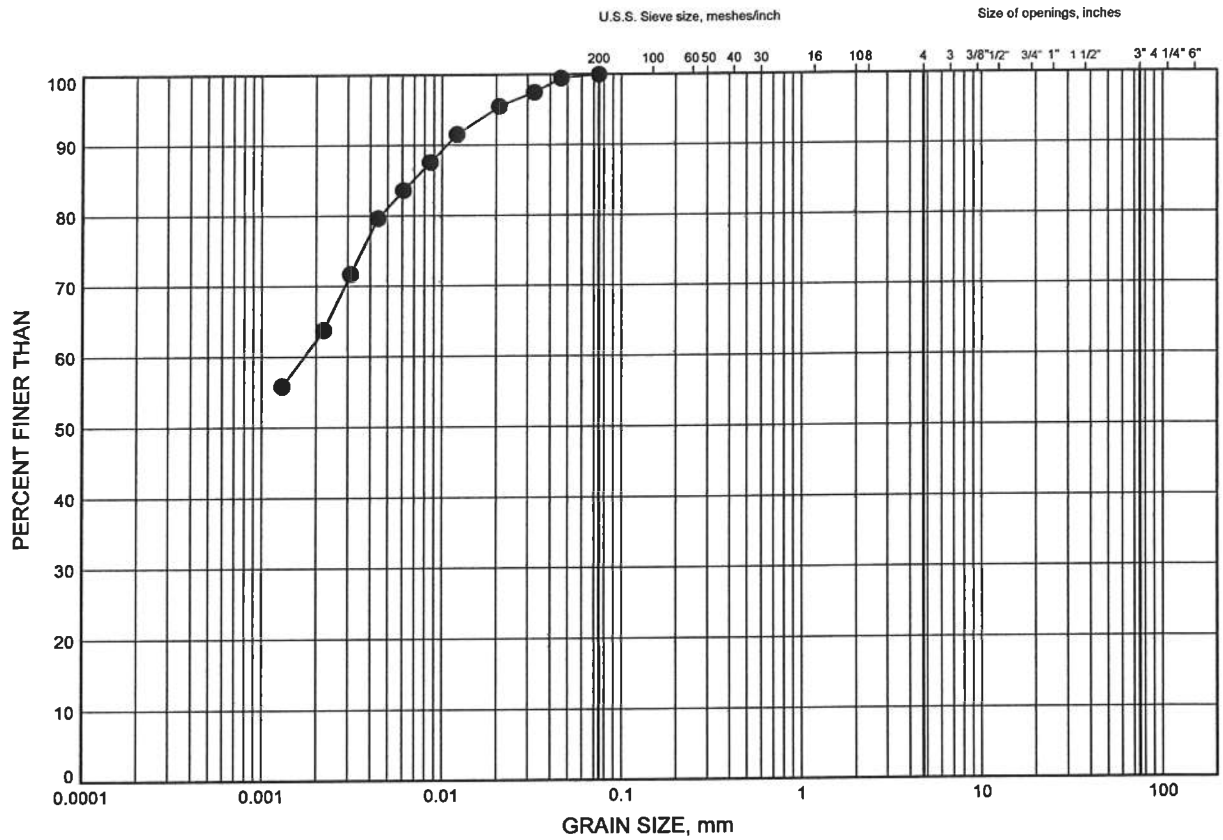
THURBER

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)
●	NGT-06	9.45	361.37

GRAIN SIZE DISTRIBUTION - THURBER 5121.GPJ 11/23/11

W.P.# 6145-04-00

Prepared By AN

Checked By LRB

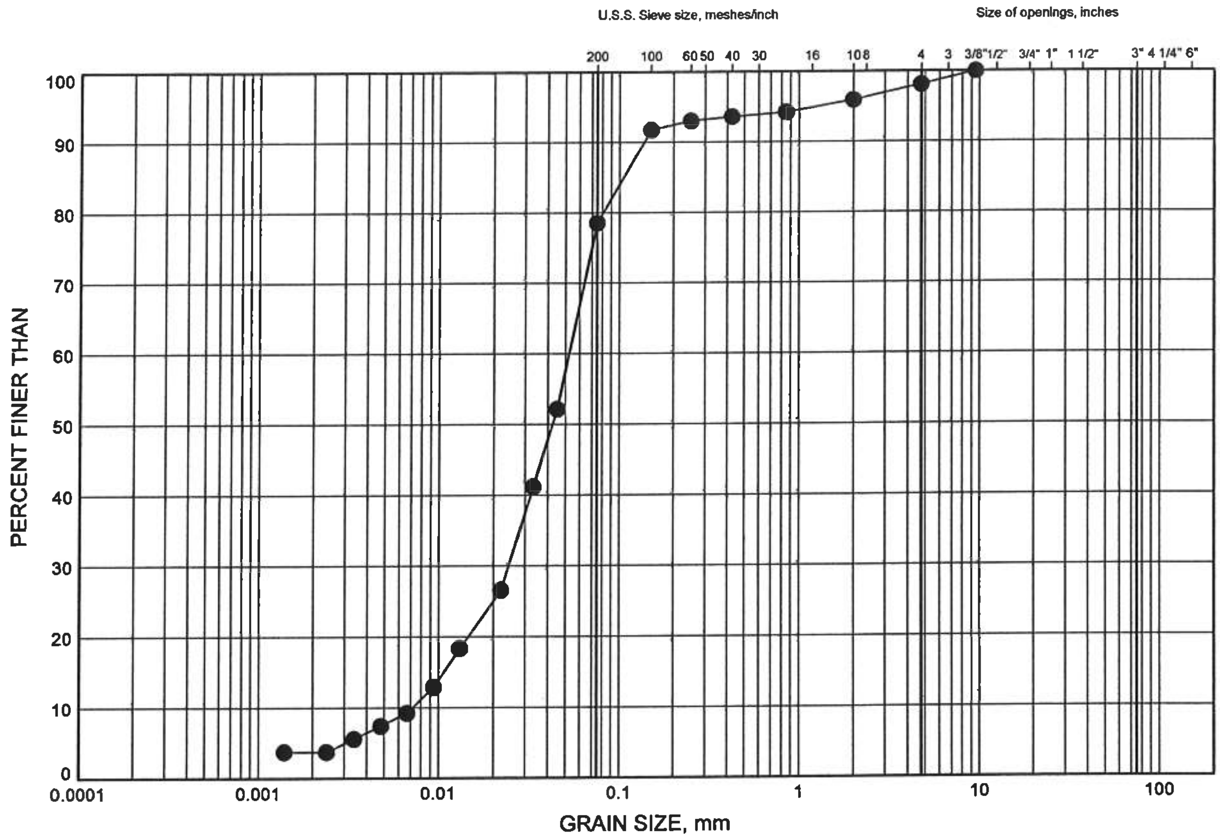
THURBER

6010-E-0010 Bridge and Culvert Rehabs NWR

GRAIN SIZE DISTRIBUTION

FIGURE B6

SANDY SILT



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	NGT-02	17.07	353.74

W.P.# 6145-04-00
Prepared By AN
Checked By LRB

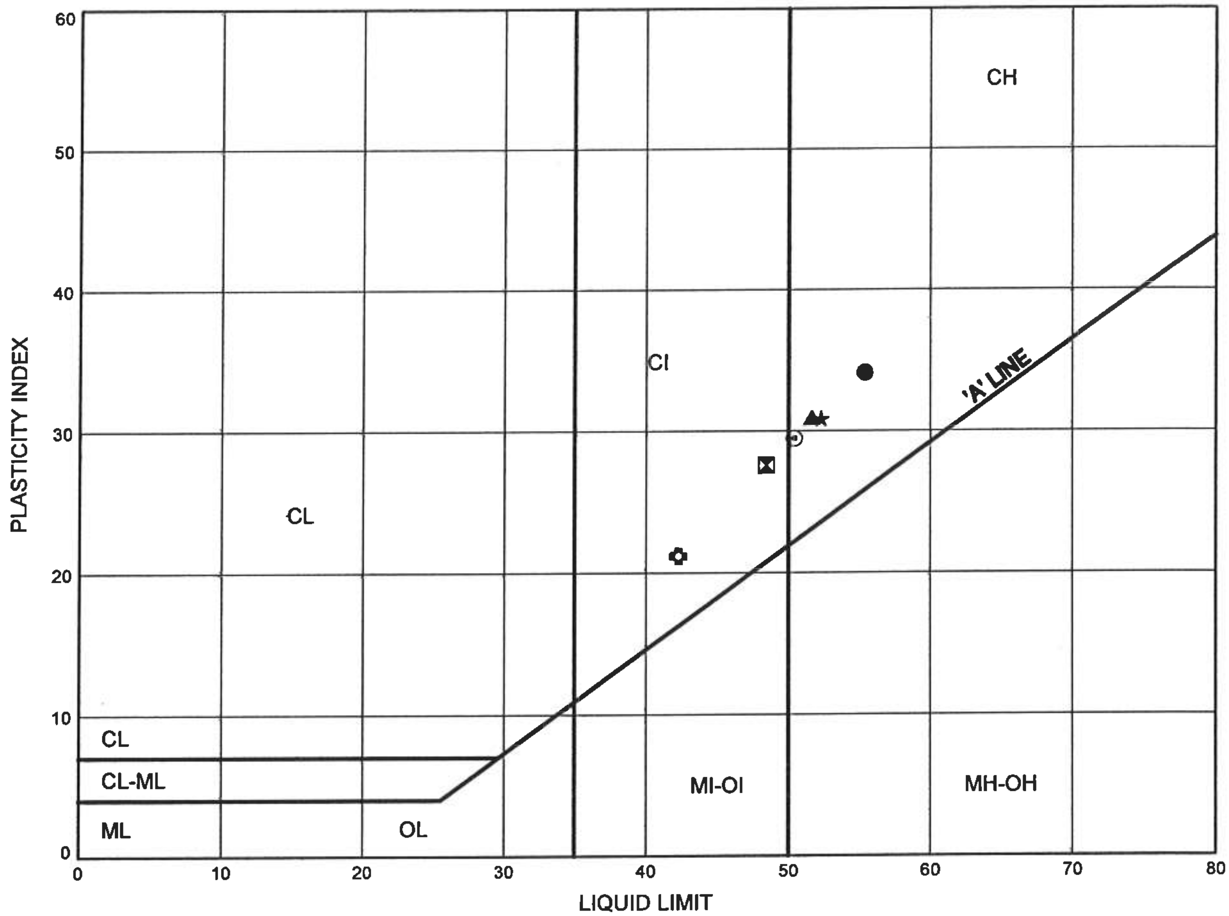
THURBER

6010-E-0010 Bridge and Culvert Rehabs NWR

ATTERBERG LIMITS TEST RESULTS

FIGURE B7

SILTY CLAY



SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	NGT-01	9.45	361.34
⊠	NGT-02	7.92	362.88
▲	NGT-02	10.97	359.83
★	NGT-03	6.40	364.43
⊙	NGT-03	9.45	361.39
✿	NGT-03	14.02	356.81

Date November 2011

Project 6145-04-00

THURBER

Prep'd AN

Chkd. LRB

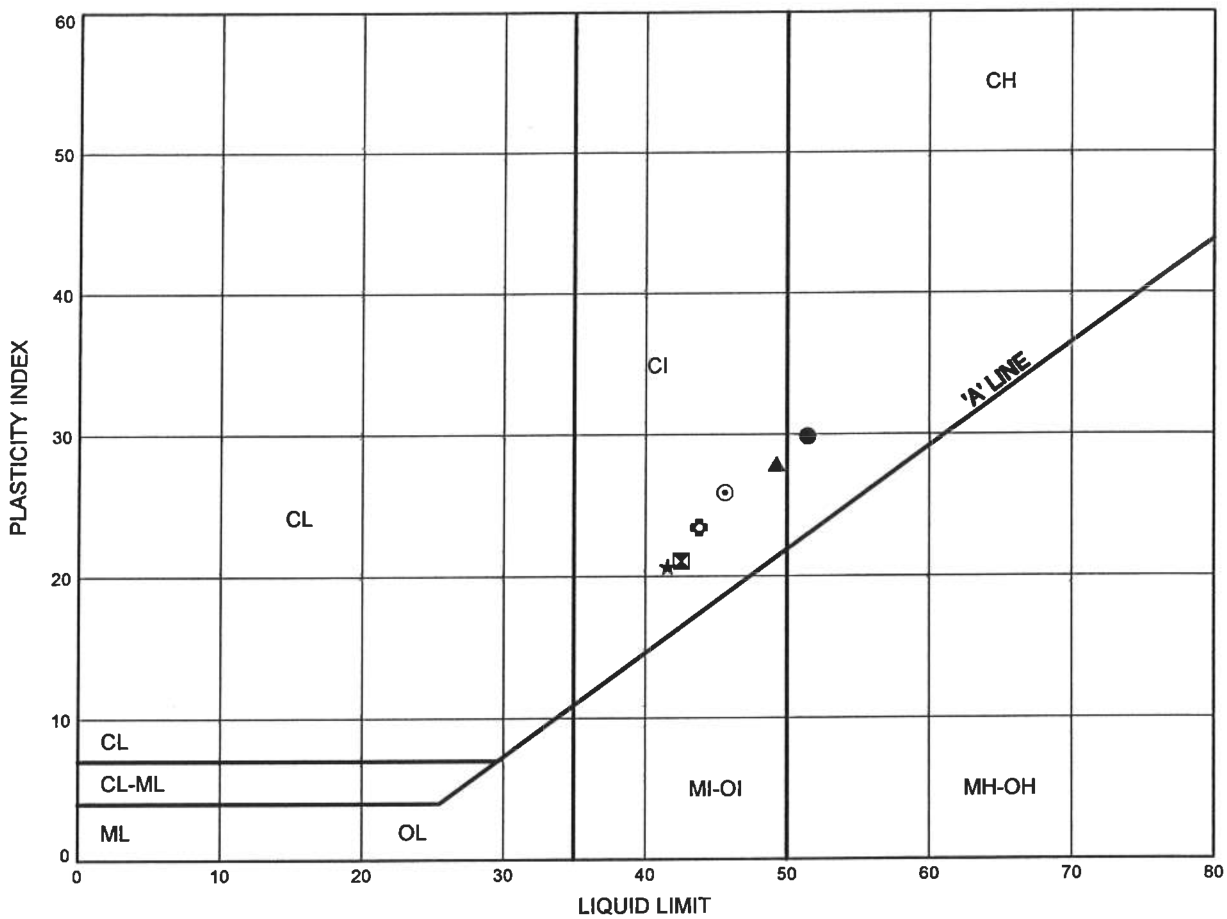
THURBALT 5121.GPJ 11/23/11

6010-E-0010 Bridge and Culvert Rehabs NWR

ATTERBERG LIMITS TEST RESULTS

FIGURE B8

SILTY CLAY



SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	NGT-04	4.88	365.97
⊠	NGT-04	12.50	358.35
▲	NGT-05	6.40	364.45
★	NGT-05	10.97	359.88
⊙	NGT-06	4.88	365.94
✿	NGT-06	9.45	361.37

THURBALT 5121.GPJ 11/23/11

Date November 2011

Project 6145-04-00

THURBER

Prep'd AN

Chkd. LRB

THURBER ENGINEERING LTD.
GEOTECHNICAL • ENVIRONMENTAL • MATERIALS

POINT LOAD TEST SHEET

Job No :	19-1605-121	**Client :**	HMM
		Date Drilled :	July 23,2011
Project Name :	Nugget Creek Bridge	**Date Tested :**	July 29,2011
Core Size : NQ	**BH No :** NGT-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	18.5	D	20.0	47.3	104.6	209.2	Mafic	Very Strong
2	1	18.8	D	19.7	47.3	108.0	206.3	Mafic	Very Strong
3	1	19.5	D	19.8	47.3	112.7	206.9	Mafic	Very Strong
4	1	20.9	D	13.3	47.4	95.9	138.6	Mafic	Very Strong
5	2	20.0	D	13.5	47.4	97.2	140.8	Mafic	Very Strong
6	2	20.8	D	20.0	47.4	114.7	208.3	Mafic	Very Strong
7	2	21.0	D	20.0	47.4	90.7	208.4	Mafic	Very Strong
8									
9									
10									
11									
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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.

THURBER ENGINEERING LTD.
GEOTECHNICAL • ENVIRONMENTAL • MATERIALS

POINT LOAD TEST SHEET

Job No : 19-1605-121
Client : HMM
Date Drilled : July 27,2011
Project Name : Nugget Creek Bridge
Date Tested : August 05,2011
Core Size : NQ **BH No :** NGT-05
Tester : JM

Test No.	Run No.	Depth (m)	Axial or Diametral	Force (kN)	Diameter (mm)	Length (mm)	UCS (MPa)	Rock Type	Notes
1	1	18.4	D	18.4	47.2	151.4	192.9	Mafic	Very Strong
2	1	18.7	D	17.1	47.1	137.1	180.1	Mafic	Very Strong
3	1	19.1	D	9.5	47.1	189.5	100.0	Mafic	Strong
4	1	19.3	D	15.7	47.3	132.4	164.1	Mafic	Very Strong
5	1	19.8	A	9.8	47.3	46.2	86.6	Mafic	Strong
6	2	20.0	D	17.2	47.3	87.4	180.3	Mafic	Very Strong
7	2	20.3	D	18.1	47.4	113.2	189.1	Mafic	Very Strong
8	2	20.5	D	21.0	47.4	116.4	219.4	Mafic	Very Strong
9	2	20.9	D	21.0	47.4	127.3	219.4	Mafic	Very Strong
10	2	21.3	D	21.0	47.4	90.5	219.4	Mafic	Very Strong
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* 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: i. Generally less costly construction than deep foundation elements.	***Advantages:*** i. High geotechnical resistance available for caissons founded on bedrock ii. Deeper foundation system is less susceptible to slope erosion processes. iii. Construction of caissons could continue in freezing weather. iv. Subexcavation of fill not required.	***Advantages:*** i. High geotechnical resistance available by driving piles to achieve resistance on the bedrock. ii. Installation of piles could continue in freezing weather. iii. May require less volume of excavation than footings. iv. Readily installed.
Disadvantages: i. Low available geotechnical resistance in native soils. ii. Potential for settlements. iii. Excavation extending below the groundwater level is required. iv. Foundations close to river flow would be at risk due to scour and erosion v. River disturbance during excavation	***Disadvantages:*** i. Higher unit cost compared to other foundation options such as footings or driven piles. ii. Top of bedrock is deep (18 m) iii. Specialized installation measures such as temporary liners and drilling mud will be required to install caissons under the water table. iv. Potential difficulty in dewatering, cleaning and inspecting bases.	***Disadvantages:*** i. Higher unit costs than footings. ii. Pile lengths required to achieve design resistance may vary.
NOT RECOMMENDED	**NOT RECOMMENDED**	**RECOMMENDED**

Appendix D

List of SPs and OPSS, and Suggested Text for Selected NSSP

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

- OPSS 903
- OPSS 902
- OPSS 804
- OPSD 208.010
- OPSD 3101.150
- OPSS 539
- Special Provision 110S13 “Amendment to OPSS 1010

2. Pile Installation

Cobbles, boulders and rockfill were encountered in the embankment fill. Some possible impacts that must be taken into consideration include, but are not necessarily limited to:

- The need to provide protection to the pile tips.
- Pre-augering through cobbles, boulders and rockfill might be required to install the piles if any of these obstructions is encountered.

3. Sheet Pile Installation

Cobbles, boulders and rockfill were encountered in the existing embankment fill. Removal of these obstructions will be required if encountered during sheet pile wall installation.

Photograph 1– Nugget Creek Bridge - Looking West

Photograph 2 – Nugget Creek Bridge, Looking East

Photograph 3 – Nugget Creek, Looking North from Bridge

Photograph 4 – South Side of Nugget Creek Bridge

Appendix F

Drawing titled "Borehole Locations and Soil Strata"

METRIC

DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

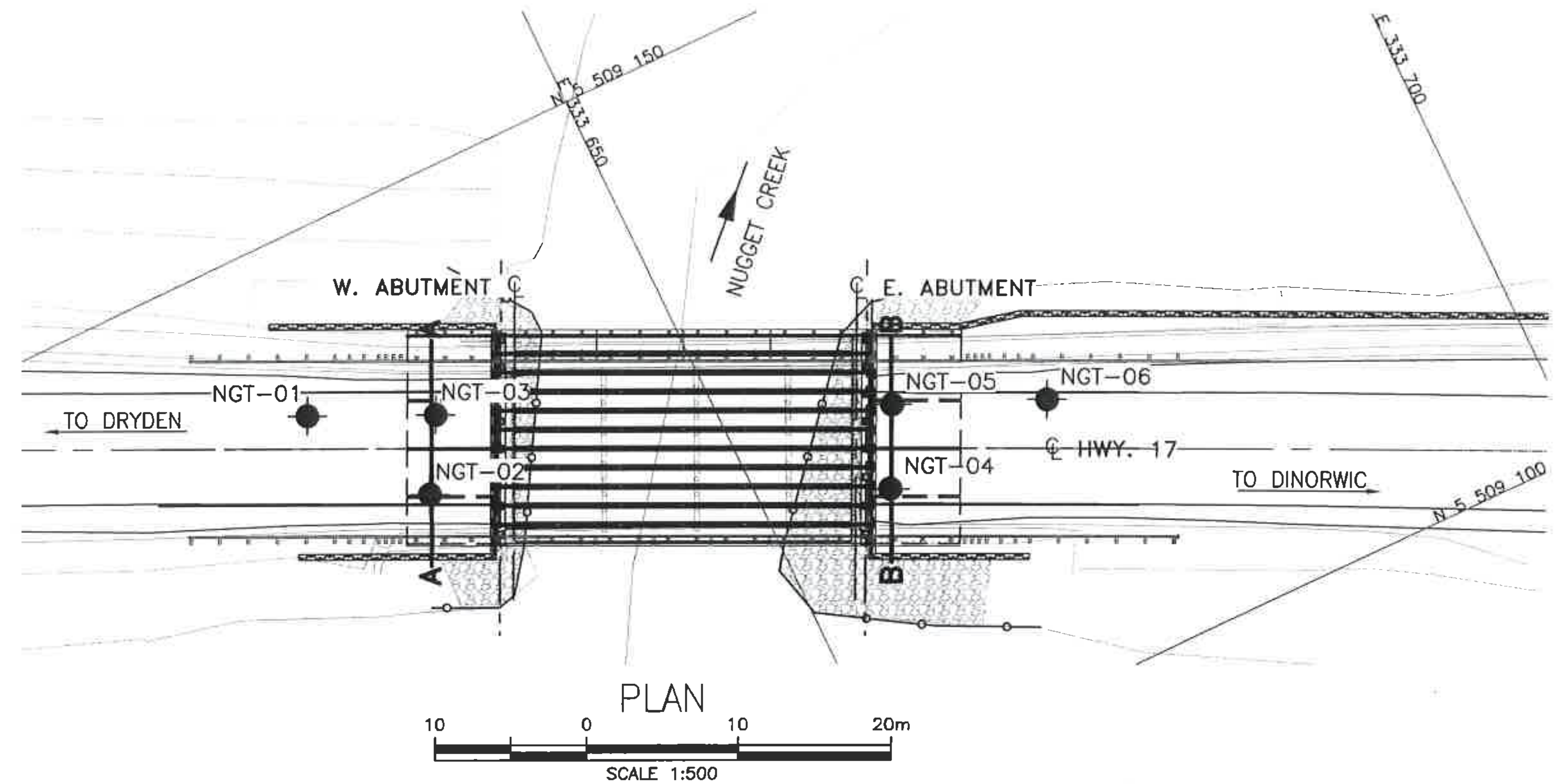
CONT No 2012–6014
WP No 6056–08–02

HIGHWAY 17
NUGGET CREEK
BRIDGE REPLACEMENT
BOREHOLE LOCATIONS AND SOIL STRATA

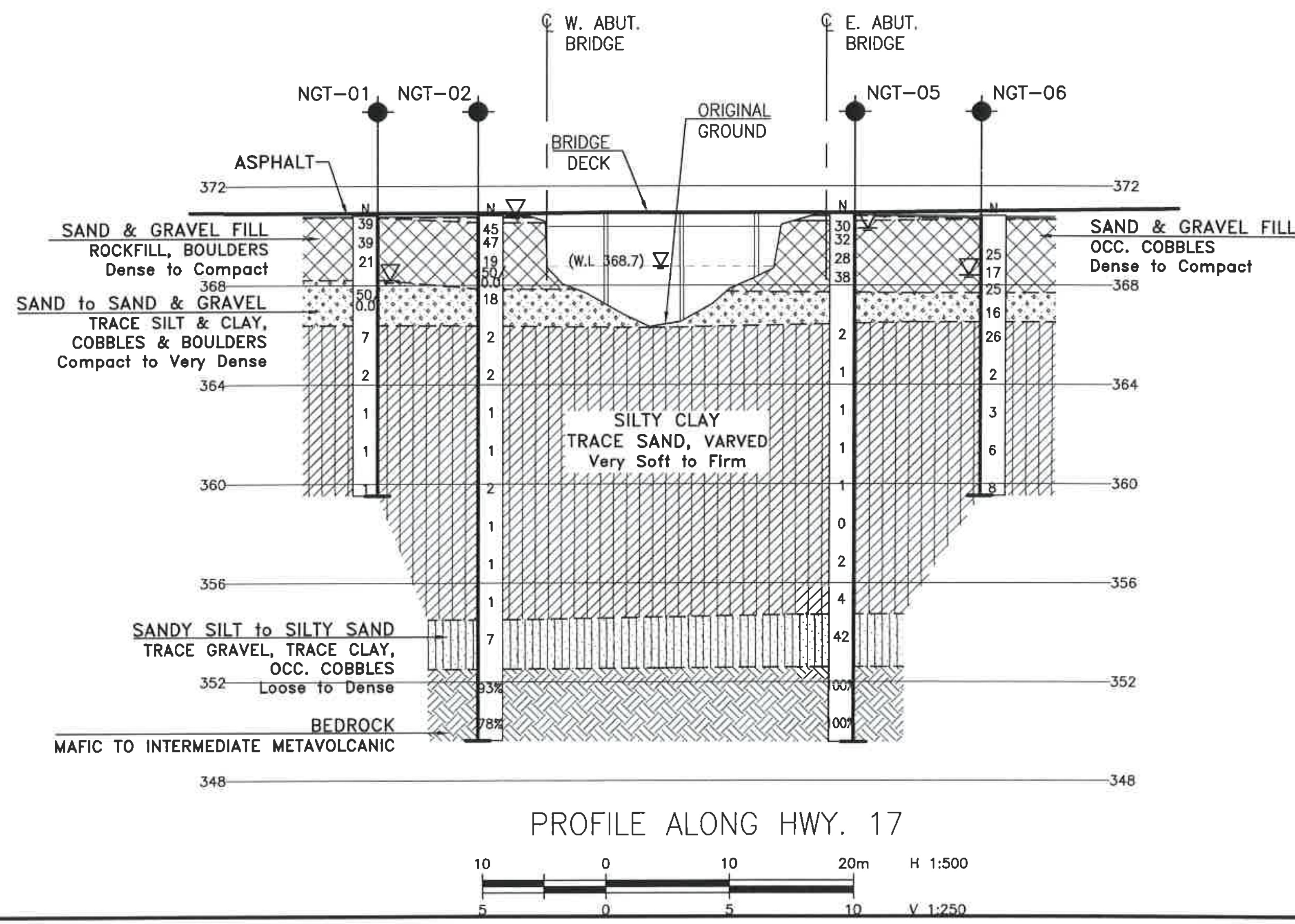
SHEET 9

Hatch Mott MacDonald

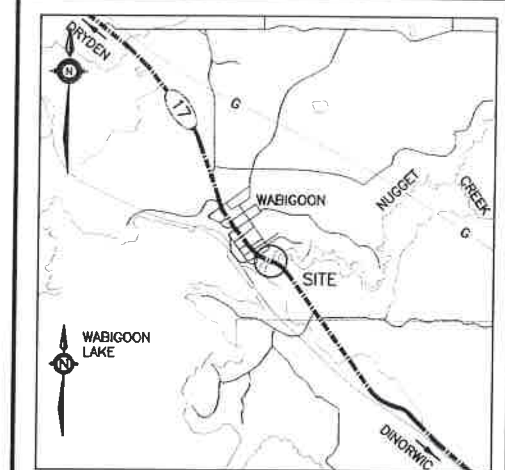
THURBER ENGINEERING LTD.



PLAN



PROFILE ALONG HWY. 17



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
NGT–01	370.8	5 509 138.5	333 625.7
NGT–02	370.8	5 509 130.3	333 630.9
NGT–03	370.8	5 509 134.9	333 633.3
NGT–04	370.9	5 509 117.5	333 658.1
NGT–05	370.9	5 509 122.5	333 660.6
NGT–06	370.8	5 509 118.4	333 669.9

LICENSED PROFESSIONAL ENGINEER
R. Palomeque Reyna
100083209
Oct/9/2012
PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER
P. K. CHATTERJI
Oct 9/12
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. 52F-38

REVISIONS | DATE | BY | DESCRIPTION

DESIGN LRB | CHK LRB | CODE CAN/CSA S6–06 | LOAD CL–625–ONT | DATE OCT. 2012
DRAWN AN | CHK RPR | SITE 41S–62 | STRUCT | DWG 2

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PLOTDATE: 10/12/2012 2:25 PM

METRIC

DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

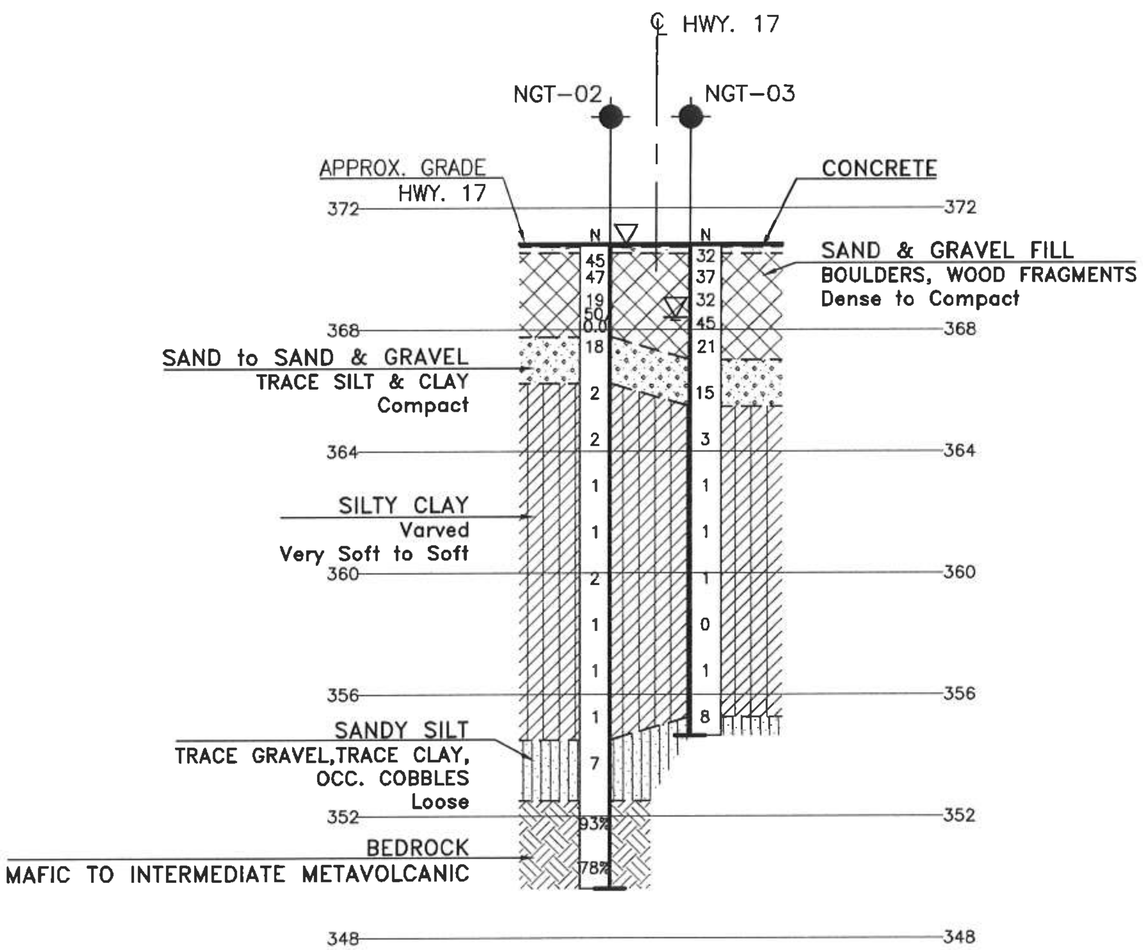
CONT No 2012–6014
WP No 6056–08–02

HIGHWAY 17
NUGGET CREEK
BRIDGE REPLACEMENT
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET 10

Hatch Mott MacDonald

THURBER ENGINEERING LTD.



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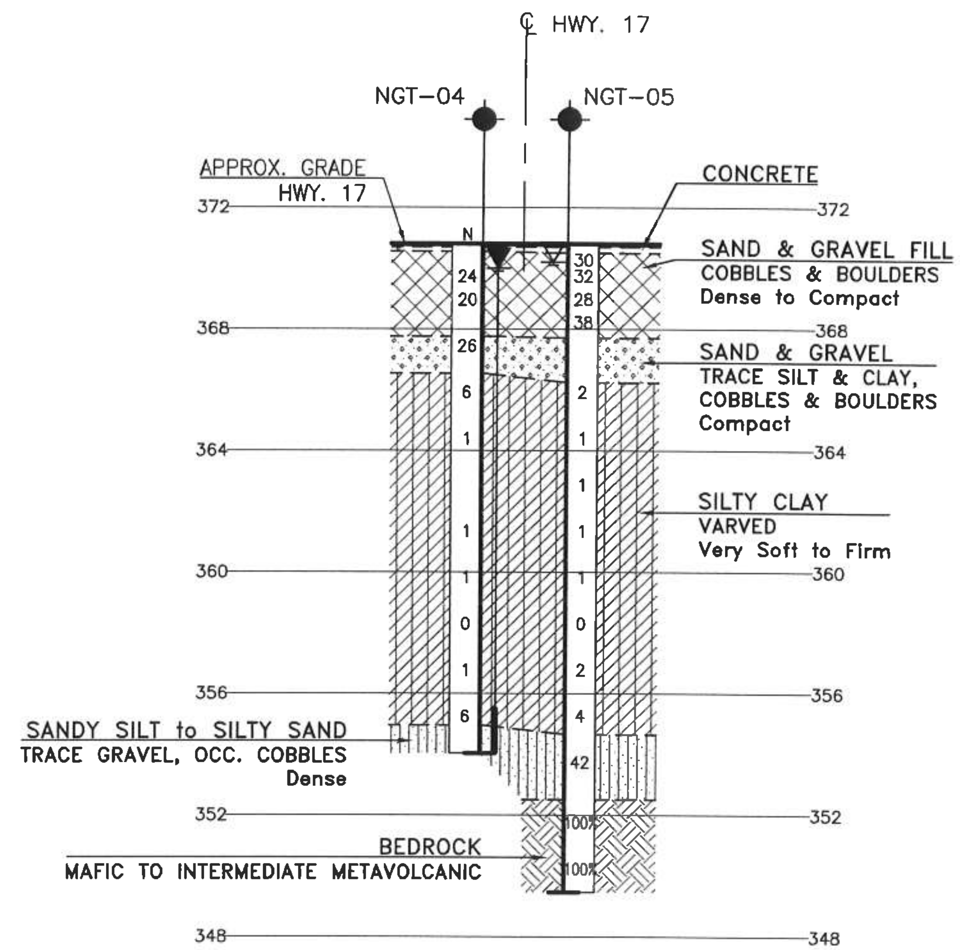

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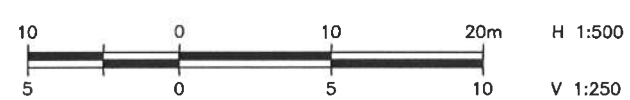

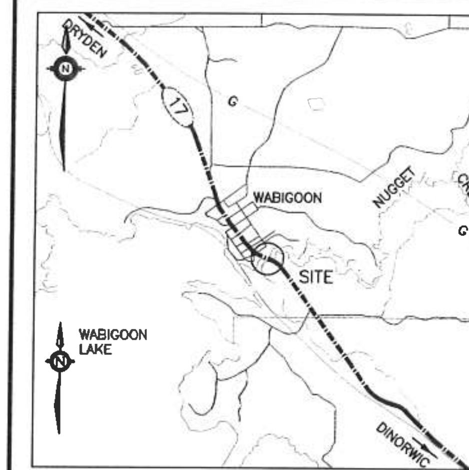

KEYPLAN

L E G E N D

	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
NGT–01	370.8	5 509 138.5	333 625.7
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NGT–03	370.8	5 509 134.9	333 633.3
NGT–04	370.9	5 509 117.5	333 658.1
NGT–05	370.9	5 509 122.5	333 660.6
NGT–06	370.8	5 509 118.4	333 669.9

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GEOCRES No. 52F-38

REVISIONS	DATE	BY	DESCRIPTION

DESIGN LRB	CHK LRB	CODE CAN/CSA S6–06	LOAD CI–625–ONT	DATE OCT. 2012
DRAWN AN	CHK RPR	SITE 41S–82	STRUCT	DWG 3