FOUNDATION INVESTIGATION AND DESIGN REPORT
MOOSEHORN LAKE CULVERT REPLACEMENT
HIGHWAY 11
WATTEN TOWNSHIP
DISTRICT OF RAINY RIVER, ONTARIO

G.W.P. 6931-10-00, SITE No. 45-276/C

Geocres Number: 52C-24

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 location of a culvert at the Highway 11 crossing of the Moosehorn Lake in the District of Rainy River, 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, a stratigraphic profile, 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 Moosehorn Lake culvert is located on Highway 11, approximately 6.7 km west of the intersection of Highway 502 and Highway 11. This site is located in Watten Township in the Rainy River District of Ontario.

The existing Moosehorn Lake culvert is a 2-cell timber culvert. Each cell is 1.3 m wide and the total length of the culvert is 27.2 m. The water flows through the culvert from northwest to southeast.

The road embankment is approximately 4.6 m to 5.8 m high above Moosehorn Lake. The surrounding lands are undeveloped and heavily treed. Bedrock outcroppings and small creeks/water bodies are visible along the existing Highway 11 right-of-way. Some rockfill is present near the toe of the embankment slope.

Photographs in Appendix C show the general nature of the site and the existing structure.

The region is characterized by Precambrian meta-volcanic and meta-sedimentary rocks intruded by later stage diabase dykes. The bedrock is mantled by glaciolacustrine clays and sand and gravel deposits.

3 SITE INVESTIGATION AND FIELD TESTING

The site investigation and field testing for this project was carried out on June 16 and 20 and July 19 to 21, 2011 and consisted of drilling and sampling six boreholes (identified as MHLC-01 to MHLC-04, MHLC-01A and MHLC-04A) in the area of the existing culvert. Boreholes MHLC-02 and MHLC-03 were drilled on the highway eastbound and westbound shoulders, through the existing highway embankment. Boreholes MHLC-01, MHLC-04, MHLC-01A and MHLC-04A were drilled at the toe of the highway embankment, near each corner of the existing culvert.

Boreholes were advanced to depths ranging from 3.9 m to 11.6 m (Elevations 336.7 to 342.9) where the drill rig encountered refusal on probable bedrock or boulders, except in Boreholes MHLC-02 and MHLC-03 where rock was cored. Coring through cobbles and boulders was required to advance Boreholes MHLC-02 and MHLC-03 drilled from the top of the highway embankment. Bedrock was proved by NQ size diamond coring in Boreholes MHLC-02 and MHLC-03.

A Dynamic Cone Penetration Test (DCPT) was conducted adjacent to each borehole to supplement the data/information collected from the boreholes. The DCPTs were terminated at various depths ranging from 3.8 m to 8.0 m (Elevations 336.6 to 344.1), upon refusal on probable bedrock or boulders. Boreholes and DCPTs refusal depths were relatively consistent.

The approximate borehole and DCPTs locations are shown on the attached Borehole Locations and Soil Strata Drawing in Appendix F.

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

Drilling on the highway shoulders was carried out using a truck-mounted CME 75 drill rig and the boreholes were advanced with hollow-stem augers and NQ coring techniques. The drilling at the toe of the embankment was carried out using wash-boring methods with casing and tripod. Portable split spoon sampling equipment driven with a Standard SPT hammer was used for penetration testing. In general, samples were obtained at selected intervals using a split spoon sampler in conjunction with Standard Penetration Testing (SPT) in the overburden soils.

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.

Rock cores were logged, and the Total Core Recovery (TCR) and Rock Quality Designation (RQD) were determined.

Groundwater conditions in the open boreholes were observed throughout the drilling operations. A standpipe piezometer consisting of 19 mm PVC pipe with slotted screen was installed in Borehole MHLC-02 and enclosed in filter sand to permit longer term groundwater level monitoring. The location and completion details of the piezometer and boreholes are shown in Table 3.1.

Table 3.1 – Borehole Completion Details

Foundation Unit	Borehole	Piezometer Tip Depth/ Elevation (m)	Completion Details
South wall	MHLC-01	None installed	Borehole backfilled with holeplug to surface.
	MHLC-02	11.3/338.8	Sand from 11.3 m to 7.8 m, holeplug from 7.8 m to 2.1 m, sand and gravel from 2.1 m to 0.1 m, then asphalt to surface.
	MHLC-04	None installed	Borehole backfilled with holeplug to surface.
	MHLC-01A	None installed	Borehole backfilled with holeplug to surface.
North wall	MHLC-03	None installed	Borehole backfilled with holeplug to 3.0 m, auger cuttings to 0.1 m, then asphalt to surface.
	MHLC-04A	None installed	Borehole backfilled with holeplug to surface.

4 LABORATORY TESTING

All recovered soil samples were subjected to Visual Identification (VI) and moisture content determination and rock samples to geological logging. 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 included in Appendix A and on the figures presented 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 shown in Table 1 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 included in Appendix A. Details of the encountered soil and rock stratigraphy are presented in these sheets and on the "Borehole Locations and Soil Strata" drawing included 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.

In general terms, the overburden encountered at this site consists of pavement structure or topsoil overlying sand and gravel fill and silty clay fill. Cobbles and boulders were contacted within the embankment fill. Native silty clay was encountered below the fill layers. Occasional cobbles and boulders were also encountered in the native silty clay deposit. A thin layer of sand was noted underlying the silty clay in the boreholes drilled at the toe of the embankment. Mafic bedrock was inferred directly below the silty clay or sand at depths ranging from 3.9 m to 10.0 m. Bedrock was confirmed in Boreholes MHLC-02 and MHLC-03 at 10.0 m and 5.5 m depth. More detailed descriptions of the individual strata are presented below.

5.1 Pavement structure

Pavement structure was encountered in the two boreholes drilled through the existing Highway 11 shoulders. The pavement structure consists of approximately 100 mm to 110 mm of asphalt overlying granular fill.

5.2 Topsoil

A thin layer of topsoil was encountered in Boreholes MHLC-01, MHLC-04 and MHLC-04A drilled at the toe of the highway embankment. The thickness of the topsoil layer ranged from 50 mm to 200 mm.

The thickness of the topsoil may vary between and beyond the borehole locations and the data is not intended for the purpose of estimating quantities.

5.3 Silty Clay Fill

Brown to dark brown silty clay fill containing trace to some sand, trace gravel and occasional roots and organics was encountered below the topsoil in Borehole MHLC-01 and surficially in Borehole MHLC-01A. The thickness of the silty clay fill was 1.0 m and 1.1 m.

Boulders were contacted within the silty clay fill at 1.0 m and 0.6 m depth in Boreholes MHLC-01 and MHLC-01A, respectively. The boulder layer was 0.2 m to 0.5 m thick.

The depth to the base of the silty clay fill was 1.2 m and 1.1 m (elevations 343.3 and 343.2) in Boreholes MHLC-01 and MHLC-01A, respectively.

SPT N-values recorded in the silty clay fill were 3 and 8 blows per 0.3 m of penetration, indicating a soft to firm consistency. High SPT N-values of 57 blows per 0.2 m of penetration and 50 blows without any penetration, were measured within the layer of boulders.

The moisture content of the fill samples ranged from 21% to 37%.

5.4 Sand and Gravel Fill and Cobbles and Boulders

Brown sand and gravel fill containing trace to some silt and clay and occasional cobbles was contacted below the pavement structure in Boreholes MHLC-02 and MHLC-03 drilled through the highway embankment. The thickness of the fill was 6.7 m and 4.5 m in Boreholes MHLC-02 and MHLC-3, respectively.

The depths to the base of the sand and gravel fill were 6.8 m and 4.6 m (elevations 343.3 and 345.4).

Cobbles and boulders were contacted within the sand and gravel fill at various depths, generally below 5.2 m and 1.3 m, in Boreholes MHLC-02 and MHLC-03, respectively. Coring through boulders was required to advance both boreholes. The diameters of the cobbles and boulders ranged from 0.13 m to 0.55 m.

SPT N-values recorded in the sand and gravel fill ranged from 8 to 40 blows per 0.3 m of penetration, indicating a loose to dense relative density. An SPT N-value of 65 blows per 0.3 m of penetration, indicating very dense relative density, was contacted in Borehole MHLC-03 above the layer of cobbles and boulders.

The moisture content of the sand and gravel fill ranged from 2% to 8%.

Grain size distribution curves for samples of sand and gravel fill tested are presented on the Record of Borehole sheet and on Figure 1 of Appendix B. The results of the laboratory test are summarized as follows:

Soil Particles	(%)
Gravel	31 to 48
Sand	38 to 62
Silt and Clay	7 to 15

5.5 Silty Clay

Native brown to grey silty clay containing trace sand to sandy was encountered below the silty clay fill at 1.2 m and 1.1 m depth (elevations 343.3 and 343.2) in Boreholes MHLC-01 and MHLC-01A, respectively. In Borehole MHLC-02, the silty clay was contacted below the sand and gravel fill at 6.8 m depth (elevation 343.3). A layer of silty clay was contacted in Borehole MHLC-03 from 4.6 m to 5.2 m depth. Dark brown to reddish brown silty clay was encountered below the topsoil in Boreholes MHLC-04 and MHLC-04A. Occasional cobbles and boulders were encountered in the silty clay layer. The thickness of the silty clay ranged from 0.6 m to 6.5 m.

The depths to the base of the silty clay ranged from 3.8 m to 10.0 m (elevation 336.9 to 344.8).

SPT 'N' values recorded in the silty clay ranged from 0 to 29 blows for 0.3 m of penetration, indicating a very soft to very stiff consistency.

The moisture content of samples collected from the silty clay layer generally varies between 18% and 75%.

Grain size distribution curves for selected silty clay samples are presented in Appendix B, Figures 2 to 4. The results are also summarized on the Record of Borehole sheets included in Appendix A. Atterberg Limits test results are presented in Figures 5 and 6 of Appendix B. The results of the laboratory tests are summarized as follows:

Soil Particles	Percentage (%)
Gravel	0
Sand	1 to 26
Silt	29 to 60
Clay	35 to 70

Index Property	Percentage (%)
Liquid Limit	53 to 78
Plastic Limit	19 to 25

The above results show that the silty clay is of high plasticity with a group symbol of CH.

5.6 Sand

A thin layer of dense sand was contacted below the silty clay at depths ranging from 3.8 m to 7.7 m (elevation 336.9 to 341.6) in Boreholes MHLC-01, MHLC-01A, MHLC-04 and MHLC-04A, drilled at the toe of the embankment. Thickness of the sand layer ranged from 100 mm to 200 mm. The moisture content of the sand layer is in the range of 23% to 27%.

All four boreholes were terminated within the sand layer at depths ranging from 3.9 m to 7.9 m upon auger refusal on probable bedrock or boulders.

5.7 Bedrock

The overburden soils described above are underlain by grey mafic bedrock. The bedrock is moderately weathered. Occasional mechanical breaks and sub-vertical fractures were observed in the rock cores. The bedrock surface is sloping and appears to dip to the southwest as well as towards both sides of the roadway.

Bedrock was proved by coring in Boreholes MHLC-02 and MHLC-03. Table 5.1 summarizes depths and elevations to the top of bedrock or auger refusal in the boreholes and DCPTs.

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

Foundation Unit	Borehole/DCPT	Top of Bedrock or Auger Refusal	
		Depth (m)	Elevation (m)
South wall	MHLC-01	7.9	336.7
	MHLC-02*	10.0	340.1
	MHLC-04	6.1	339.3
North wall	MHLC-01A	4.7	339.5
	MHLC-03*	5.2	344.8
	MHLC-04A	3.9	341.5

*Proved by coring

Core recovery in the bedrock was 100%. The RQD value in one core was 97% and 100%, indicating an excellent rock quality.

The estimated unconfined compressive strength of the rock core ranged from 134 MPa to 282 MPa, indicating a very strong to extremely strong rock. These estimated rock strength values are interpreted from point load tests that were conducted on rock cores recovered from one borehole. A summary of the Point Load Test Results is presented in Appendix B.

5.8 Water Levels

Water levels were observed in the boreholes during and upon completion of drilling. A standpipe piezometer was installed in Borehole MHLC-02 to monitor water levels after completion of drilling. The water levels measured in the piezometer are summarized in Table 5.2, along with the measurements in the boreholes upon completion of drilling.

Table 5.2 – Water Level Measurements

Borehole	Date	Water Level Depth (m)	Water Level Elev. (m)	Comment
MHLC-01	July 20, 2011	0.3	344.3	Open borehole
MHLC-01A	July 19, 2011	0.7	343.5	Open borehole
MHLC-02	August 26, 2011	7.2	342.9	Piezometer
MHLC-03	June 16, 2011	5.4	344.6	Open borehole
MHLC-04	July 20, 2011	0.5	344.9	Open borehole
MHLC-04A	July 21, 2011	1.5	343.9	Open borehole

Groundwater level measured in the boreholes upon completion of drilling varies from elevation 343.5 to 344.9. The piezometric reading of the current investigation indicates that the groundwater level is at 7.2 m depth (elevation 342.9).

GA indicates that water level in the Moosehorn Lake is variable. However, the water level is shown in the drawing at an approximate elevation of 344.0.

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. Surveyors retained by Hatch Mott MacDonald provided data and drawings to obtain the co-ordinates and the ground surface elevations for the boreholes and DCPTs.

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.

OGS Drilling Inc. of Almonte, Ontario supplied the portable drilling/coring equipment to drill and core boreholes that were not accessible using a truck mounted rig.

The field program was supervised by Ms. Eckie Siu of Thurber.

Overall 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 culvert to replace the existing culvert at the Highway 11 crossing of Moosehorn Lake in the District of Rainy River, Ontario.

The existing culvert is a two-cell timber structure. Each cell is 1.3 m wide. The total length of the culvert is 27.2 m.

The proposed replacement culvert (as shown on the Preliminary General Arrangement dated June 2011) will consist of two parallel sheet pile walls supporting a precast cap consisting of concrete panels. The new structure will have a span of approximately 9.0 m, and a length of 33.4 m of which 18.3 m will be capped by precast panels. The clearance between the bottom of the concrete panels and the lake bed will be about 4.8 m. The underside of the cap panel is at approximate elevation 348.2.

The approximate original ground surface ranges from elevation 344.2 to 345.4. The finished road grades over the culvert will be at Elevation 350.0, resulting in a maximum embankment height of about 4.6 m to 5.8 m.

The proposed culvert design was selected to avoid or minimize any disturbance or environmental impact on the lake bed. The design also minimizes use of cast-in-place concrete which increases the cost of construction significantly.

The discussions and recommendations presented in this report are based on the factual data obtained during the course of the investigation. The plans and profiles used for preparation of this report were provided by Hatch Mott MacDonald.

8 STRUCTURE FOUNDATIONS

In general terms, the overburden encountered at this site consists of pavement structure or topsoil overlying loose to dense sand and gravel fill and soft to firm silty clay fill. Cobbles and boulders were contacted within the sand and gravel fill at various depths. Native very soft to very stiff silty clay was encountered below the fill. A thin layer of sand (100 mm to 200 mm thick) was noted underlying the silty clay in boreholes drilled at the toe of the highway embankment. Mafic bedrock was encountered directly below the silty clay or sand at depths ranging from 3.9 m to 10.0 m. The bedrock surface is sloping and generally dips steeply to the southwest as well as toward both sides of the road.

Measurement in the piezometer installed in Borehole MHLC-02 during the current investigation, indicate that the groundwater level is at 7.2 m depth (elevation 342.9).

GA indicates that water level in the Moosehorn Lake is variable. However, the water level is shown at an approximate elevation 344.0.

Recommendations are provided for a sheet pile foundation supporting the precast cap panels.

Consideration was also given to the option of open footing culvert supported on:

- Spread footings on native soils or bedrock
- Augered Caissons (drilled shafts)
- Driven piles

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

8.1 Steel Sheet Pile Walls

The preferred culvert replacement structure at this site consists of precast cap panels supported on steel sheet piles. The depth to bedrock is variable along the culvert alignment and sheet pile depth will vary along the culvert alignment. Furthermore as noted in Borehole MHLC-03, the bedrock is 5.2 m below the road surface and only 0.6 m below the original ground. The embedment of the sheet piles will be very shallow near Borehole MHLC-03 and additional lateral support will be required for lateral stability of the sheet piles at this location.

Sheet piles must be driven to bedrock or to refusal at or below elevations given in Table 8.1.

Table 8.1 – Recommended Sheet Pile Tip Elevation

Foundation Unit	Borehole/DCPT	Estimated Pile Tip Elevation to Bedrock or Auger Refusal (m)
South wall	MHLC-01	336.7
	MHLC-02	340.1
	MHLC-04	339.3
North wall	MHLC-01A	339.5
	MHLC-03	344.8
	MHLC-04A	341.5

Most of the sheet piles are expected to reach bedrock or a layer of refusal. In the area of Boreholes MHLC-03 and MHLC-02 drilled through the existing roadway embankment, cobbles and boulders were encountered at depths of 1.3 m and 5.2 m. However, the DCPTs driven next to these boreholes advanced through this layer, indicating that sheet piles should be capable of advancing through the fill.

The Contract Documents should contain a NSSP alerting the Bidders to the possibility of some sheet piles meeting refusal on a large boulder or shallow bedrock. Suggested texts for NSSP's are included in Appendix E. Sheet piles should be provided with sheet pile tip protector to minimize any tip damage.

Existing rock fill along the sides of the embankment should be removed prior to driving the sheet piles.

The factored Geotechnical Resistance at ULS (per metre width) has been assumed to be 30% of the structural capacity of the sheet pile selected. This reflects the possibility that in light of the variable depth to bedrock or refusal layer along the culvert alignment, the sheet piles may not rest uniformly on bedrock and some sheet piles driven through the existing road embankment may encounter cobbles and boulders.

The factored Geotechnical Resistances at ULS (per metre width) recommended for three sheet pile sections driven to bedrock or layer of refusal are as follows:

Table 8.2 – Recommended Axial Resistances of Steel Sheet Piles

Sheet Pile Section	Factored ULS Resistance per meter width (kN)
EZ-88	1,000
XZ-100	1,400
JZ-127	1,850

The SLS condition will not govern for steel sheet piles driven to bedrock.

Steel sheet pile installation should be in accordance with OPSS 903, November 2009.

Sheet piles should be driven to the specified elevation noted in Table 8.1. The appropriate pile driving note is "Sheet piles to be driven to bedrock or refusal".

The lateral resistance of sheet piles may be computed using the lateral earth pressure distribution and parameters presented in Section 9.

As indicated earlier, the sheet piles along the north culvert wall are expected to encounter bedrock immediately below the embankment fill, resulting in shorter piles. If the lateral resistance is not sufficient for these short piles, additional lateral resistance may be provided by installing suitably designed dowels or pins into bedrock in front of the sheet piles.

It is understood that all exposed sheet piles will be coated with Hycote 151 to 300 mm below the finished grade.

For open footing culverts, the following foundation options were considered:

8.2 Spread Footings on native soils or bedrock

Spread footings on native soils are not recommended at this site due to the relatively low geotechnical resistance available in the overburden and potential settlement in the near surface soils.

Spread footings on bedrock are neither recommended as they will involve a deep excavation to the inferred top of bedrock ranging from 3.9 m to 10.0 m below ground surface, and extending also below the water level.

Additionally, both options may encounter dewatering difficulties during temporary excavation for footings and potential disturbance to the lake will be caused by footing excavation.

8.3 Caissons

Augered caisson foundations were also considered for the support of the open footing culvert option. The caissons must be founded on the inferred bedrock at depths in the order of 3.9 m to 10.0 m below original ground surface or roadway embankment. The base of the caissons would be about 1.4 m to 6.2 m below the groundwater level, resulting in high hydrostatic heads at the base.

Unwatering of the caisson would be impractical and attempts to do so might result in continued flow of fines into the caisson excavation. Installation of caissons through the boulders noted in Boreholes MHLC-02 and MHLC-03 may also be difficult.

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

For these reasons, the use of a caisson foundation is not recommended.

8.4 Driven Steel H-Piles

The subsurface conditions at the site are considered suitable for the design of open footing culvert supported on steel H-piles driven to achieve resistance on bedrock or a layer of refusal. It should be noted that the top of bedrock is sloping at the north and south culvert walls.

The elevations at which the piles are expected to develop the required resistance are given in Table 8.1.

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 8.4.4 Pile Driving.

8.4.1 Axial Resistance

The axial, factored Geotechnical Resistances at Ultimate Limit States (ULS_f) and Geotechnical Resistance at Serviceability Limit States (SLS) recommended for an HP 310x110 pile section when driven to refusal are:

- Factored ULS Resistance per pile 1,400 kN
- SLS Resistance 1,200 kN

Along the north wall of the culvert near Borehole MHLC-03, the H-piles may have to be socketed into rock to achieve sufficient penetration. This would involve coring the bedrock to set and grout the pile into the bedrock.

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

8.4.2 Pile Tips

If driven piles are selected, the tips of all piles should be fitted with cast steel, H-section rock points from an approved manufacturer such as Titus Steel (Rock injector) or approved equivalent. Some piles may achieve refusal on large boulders, which will require the same pile tip protection and reinforcement as founding on bedrock.

In areas of steeply sloping bedrock, the driving energy should be reduced as required to seat the pile in bedrock and avoid sliding of the pile tip.

8.4.3 Pile Installation

Pile installation should be in accordance with OPSS 903, November 2009.

The Contract Documents should contain a NSSP alerting the Bidders to:

- The presence of cobbles and boulders within the highway embankment and in the native silty clay.
- Sloping bedrock surface was revealed at the location of the proposed culvert walls. It is expected that piles within a group will achieve the specified resistance at varying elevations.
- The possibility of some piles meeting refusal on a large boulder. When boulders are encountered within a shallow depth, pre-drilling through the boulders may be required to advance and install the piles to the required pile tip elevation. Rock coring and rock breaking equipment may be required in addition to augering equipment.

Suggested texts for NSSP's are included in Appendix E.

8.4.4 Pile Driving

Piles should be driven to elevations shown in Table 8.1.

Pile driving must be controlled by the Hiley Formula and an ultimate pile resistance to be specified by the designer in accordance with Clause 3.3.2 (b) Construction Stage of the Structural Manual. The Hiley Formula need not to be used until the pile tips are within 1.0 m of the bearing stratum. The appropriate pile driving note is "Piles driven in accordance with Standard SS103-11 using an ultimate resistance of 2,800 kN for HP310x110 pile".

8.4.5 Downdrag

Downdrag on the piles is not considered to be an issue at this site, since no highway grade raise is proposed.

8.4.6 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	=	$n_h . z / D$ (kN/m^3)
	p_{ult}	=	$3 . \gamma . z . K_p$ (kPa)
where	z	=	depth of embedment of pile in metres
	D	=	pile width in metres
	n_h	=	value from Table 8.4
	γ	=	unit weight (Table 8.4)
	K_p	=	passive earth pressure coefficient (Table 8.4)

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 50 kN at SLS. Parameters for lateral pile resistance are shown in Table 8.4.

Table 8.4 – Parameters for Lateral Pile Resistance

Location	Elevation	n_h (kN/m^3)	S_u kPa	K_p	Unit Weight (kN/m^3)	Soil Conditions
South wall	350.0 to 343.3	6,000	-	3.3	21	Sand and gravel, compact to dense (FILL)
	343.3 to Bedrock	-	75	2.7	10*	Silty clay, very soft to very stiff
North wall	350.0 to 345.4	8,000	-	3.3	21	Sand and gravel, very dense (FILL)
	345.4 to Bedrock	-	85	2.7	10*	Silty clay, soft to very stiff

*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.

8.5 Proposed Foundation

It is understood that based on environmental considerations and cost of cast-in-place concrete, the preferred solution for culvert replacement at this site is precast cap panels founded on sheet piles. As indicated earlier, the depth to bedrock is variable along the culvert alignment and along the north wall bedrock is shallow under the highway. Sheet piles will meet refusal at varying depths and sheet piles may not have enough penetration in the mid culvert area.

When the bedrock is at shallow depth, additional lateral resistance may have to be provided for the sheet pile wall by installing pins or dowels drilled into bedrock in front of the sheet pile.

The Contract must address these issues with the sheet pile foundation.

8.6 Frost Cover

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

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

9 CULVERT BACKFILL AND LATERAL EARTH PRESSURES

Culvert backfill should consist of granular material conforming to OPSS Granular A or Granular B specifications. The existing highway embankment fill consists of sand and gravel and is not considered susceptible to frost action.

Heavy compaction equipment should not be used adjacent to the sheet pile walls and roof of the culvert. Compaction should be carried out in accordance with OPSS 501 dated November 2010. Backfill for the culvert should be placed and compacted in simultaneous equal lifts on both sides of the culvert, and the top of backfill elevation should be within 400 mm on both sides of the culvert at all times.

In general, earth pressures acting on the culvert walls may be assumed to impose a triangular distribution governed by the characteristics of the backfill. For a fully drained condition, the pressures should be computed in accordance with the CHBDC but generally are given by the expression:

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

where:
- p = horizontal pressure on the wall at depth h (kPa)
- K = earth pressure coefficient (see Table 9.1)
- γ = bulk 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 culvert are dependent on the material used as backfill and the inclination of the ground surface behind the wall. Recommended unfactored values for a level ground surface are shown in Tables 9.1. The at-rest coefficients should be employed for restrained culvert walls. For the at-rest condition, all soil above a horizontal surface behind the wall should be treated as a surcharge load. Active pressures shall be used for any wingwalls or unrestrained walls.

If the ground surface behind the sheet pile walls is sloping, the earth pressure parameters will increase. Thurber should be contacted to provide revised earth pressure parameters for this condition.

The parameters in the table correspond to full mobilization of active and passive earth pressures, and require certain relative movements between the wall and adjacent soil to produce these conditions. The values to be used in design can be assessed from Figure C6.9.1 (a) of the Commentary to the CHBDC.

Table 9.1 – Earth Pressure Coefficients (K) for Horizontal Ground Surface

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 or OPSS Granular B Type I or Type III** $\phi = 32°$, $\gamma = 21.2\ kN/m^3$	**Silty Clay and Silty Clay Fill** $\phi = 27°$ $\gamma = 18\ kN/m^3$
Active (Unrestrained Wall)	0.27	0.30	0.37
At rest (Restrained Wall)	0.43	0.47	0.55
Passive (Movement Towards Soil Mass)	3.7	3.3	2.7

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

The design of the culvert must incorporate measures such as weepholes or subdrains to permit drainage of the culvert backfill, or alternatively the culvert walls should be designed to withstand the potential build-up of hydrostatic pressures behind the walls.

Since no grade change is proposed at this site and the culvert will be founded on bedrock, settlement of the embankment is not an issue.

10 EROSION CONTROL

Erosion protection should be provided along any section of embankment slope that may be in contact with stream flow. We understand that the exiting stream channel is not to be disturbed by culvert replacement work.

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.

11 EXCAVATION AND GROUNDWATER CONTROL

All excavation must be carried out in accordance with the Occupational Health and Safety Act (OHSA). For the purposes of the OHSA, the sand and gravel fill forming the existing embankment and the silty clay fill at the toe of the embankment may be classified as Type 3 soils. The sand and gravel fill below the water table is a Type 4 soil.

The piezometric reading from the current investigation indicates that the groundwater level is at 7.2 m depth (elevation 342.9).

GA indicates that water level in the Moosehorn Lake is at approximate elevation 344.0.

We understand that measures such as stream diversions will not be permitted to avoid disturbance of the channel. The water level in the stream may be lower in the dry seasons and construction should be conducted during these periods.

Based on the preliminary culvert design, excavation below the groundwater level to construct the new sheet pile culvert will not be required.

For any temporary excavation, the Contractor must be prepared to control the groundwater and surface water to permit construction in the dry.

The design of the dewatering system that may be required is the responsibility of the Contractor and the Contract Documents must alert him to this responsibility and the need to engage a dewatering specialist.

The Contractor should also be prepared to pump from sumps to remove any remaining seepage water or surface water collecting in an excavation. Placement of concrete (if required) must be done in the dry. Unwatering must remain operational and effective until the culvert is installed and backfilled.

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

12 SEISMIC CONSIDERATIONS

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 I. Therefore, according to Table 4.4 of the CHBDC, a Site Coefficient "S" (ground motion amplification factor) of 1.0 should be used in seismic design.

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 12.1 may be used:

Table 12.1 – 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 or OPSS Granular B Type I or Type III** $\phi = 32°$, $\gamma = 21.2$ kN/m^3	**Silty Clay and Silty Clay Fill** $\phi = 27°$ $\gamma = 18$ kN/m^3
Active (K_{AE})*	0.28	0.32	0.38
Passive (K_{PE})	3.7	3.2	2.6
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 site overlies very soft to very stiff cohesive soils deposits over bedrock and a high water table. A review of the subsurface conditions indicates the site is not susceptible for liquefaction under current conditions.

The existing embankments are above the groundwater level and are not considered to be in danger of undergoing liquefaction. Some toe failure may occur but it is expected to be of limited nature and readily repairable.

13 ROADWAY PROTECTION

During the new culvert construction, temporary excavation of existing embankments will be required. The culvert 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.

One option is to use continuous sheet pile wall with lateral support such as dowels or pins installed in front of the wall to provide temporary support to the soils during excavation. These sheet piles may encounter cobbles and boulders in the highway embankment fill as noted in Boreholes MHLC-02 and MHLC-03. Pre-drilling may be required at some locations to install the sheet piles to the desired depth.

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	=	0.30	(Active pressure coefficient for road embankment fill)
	=	0.37	(Active pressure coefficient for silty clay/silty clay fill)
K_p	=	3.3	(Passive pressure coefficient for road embankment fill)
	=	2.7	(Passive pressure coefficient for silty clay/silty clay fill)
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, soldier pile and lagging)
h_w	=	344.0	(elevation for hydrostatic pressure build-up behind sheet piles)

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 will 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, who will determine an appropriate support system.

14 CONSTRUCTION CONCERNS

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

- The surface of the bedrock is variable at this site. Since the elevation of the bedrock surface was only established at two boreholes, it is possible that higher or lower elevations will be encountered during construction. Sheet piles/H-piles will encounter refusal at varying depth along the culvert alignment and this must be addressed in the Contract document.
- Evidence of cobbles and boulders was noted during drilling. It is possible that a sheet pile or H-pile will achieve refusal at a higher elevation than anticipated due to encountering a

boulder. If it is suspected that this is happening, the QVE must immediately bring it to the attention of the CA. If the CA cannot resolve the issue, it must be referred to the design team for resolution.

- 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.
- Erosion protection should be provided to the embankment surfaces after construction.

The successful performance of the culvert will depend largely upon good workmanship and quality control during construction. Sheet pile or H-pile driving supervision should be carried out by qualified geotechnical personnel during construction to confirm that foundation recommendations are correctly implemented and material specifications are met.

15 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.

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Review Principal

LICENSED PROFESSIONAL ENGINEER
P. K. CHATTERJI
Dec 20/11
PROVINCE OF ONTARIO

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.

<u>UNIFIED SOILS CLASSIFICATION</u>

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

THURBER

RECORD OF BOREHOLE No MHLC-01

METRIC

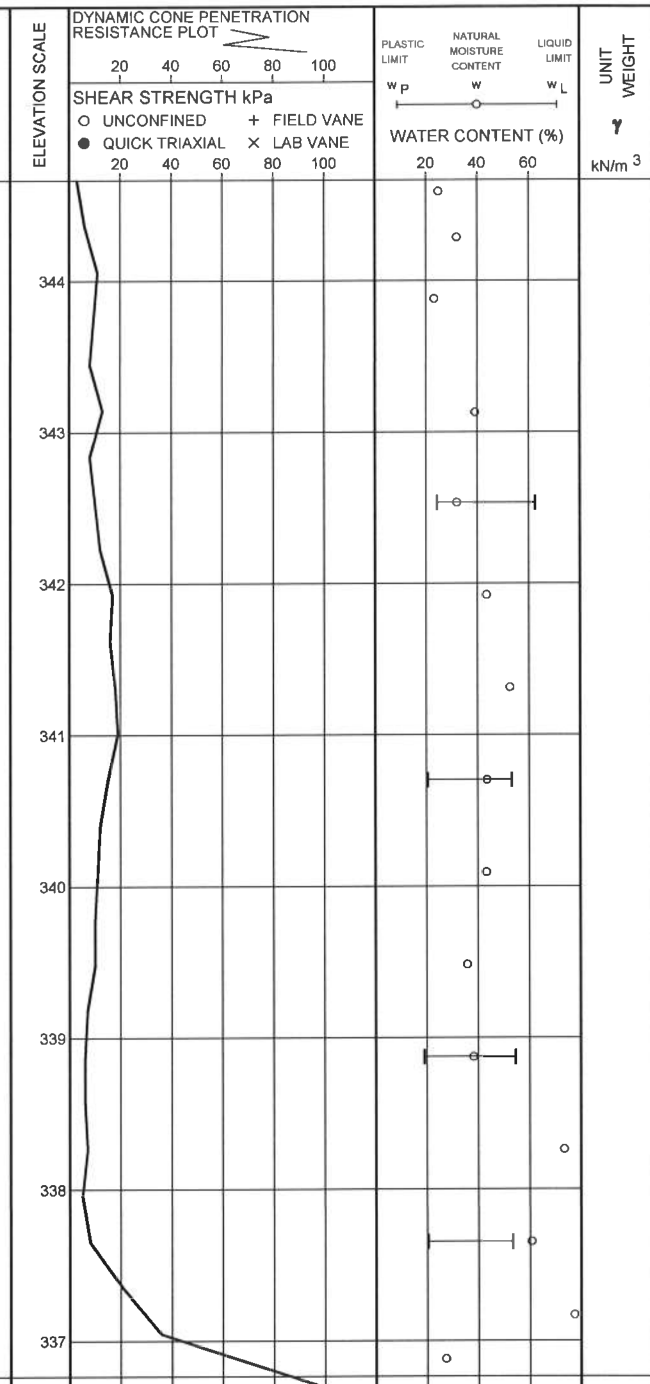
W.P. 6931-10-00 | LOCATION N 5 395 470.9 E 294 209.8 Moosehorn Lake Culvert | ORIGINATED BY ES

HWY 17 | BOREHOLE TYPE Mud Rotary and Tripod | COMPILED BY AN

DATUM Geodetic | DATE 2011.07.19 - 2011.07.20 | CHECKED BY RPR

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	GR	SA	SI	CL
344.6									
0.0	**TOPSOIL**, occasional roots and rootlets: (200mm)								
0.2	Silty **CLAY**, mixed with organics, trace sand, trace gravel, occasional roots. Loose. Dark Brown. Damp. Firm. Brown. (FILL)	1	SS	8	▽				
		2	SS	57/ ~~0.200~~					
343.3 / 1.2	Boulder (200mm) at 1.0m	3	SS	11					
	Silty **CLAY**, trace to some sand, occasional roots and rootlets. Stiff to Very Stiff. Brown to Dark Grey. Sand pockets	4	SS	9		0	12	39	49
		5	SS	17					
	Reddish Brown	6	SS	13					
		7	SS	12		0	1	46	53
		8	SS	8					
	Firm to Soft	9	SS	5					
		10	SS	5		0	1	56	43
	Very Soft	11	SS	4					
		12	SS	0		0	4	47	49
		13	SS	7					
336.9 / 7.7	**SAND**, trace silt. Loose. Grey. Wet								
336.7 / 7.9									

END OF BOREHOLE AT 7.9m UPON REFUSAL ON PROBABLE BEDROCK.
WATER LEVEL AT 0.35m UPON COMPLETION.
BOREHOLE BACKFILLED WITH HOLEPLUG TO SURFACE.



+ ³, × ³: Numbers refer to Sensitivity

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

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No MHLC-01A

METRIC

W.P. 6931-10-00
LOCATION N 5 395 478.4 E 294 214.8 Moosehorn Lake Culvert
ORIGINATED BY ES

HWY 17
BOREHOLE TYPE Mud Rotary and Tripod
COMPILED BY AN

DATUM Geodetic
DATE 2011.07.19 - 2011.07.19
CHECKED BY RPR

SOIL PROFILE / SAMPLES

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
344.2						
0.0	Silty **CLAY**, some sand, trace gravel, occasional organics Soft Dark Brown (FILL) Boulders from 0.6m to 1.1m	1	SS	3		
		2	SS	50/ 0.00	∇	
343.2 1.1	Silty **CLAY**, trace sand, occasional roots and rootlets Firm to Very Stiff Grey	3	SS	7		0 1 44 55
	Reddish Brown to Grey	4	SS	18		
	Grey	5	SS	21		
		6	SS	10		
	Soft	7	SS	4		0 1 48 51
		8	SS	52/ 0.275		
339.6 4.6 339.5 4.7	**SAND**, trace silt Grey Wet					

END OF BOREHOLE AT 4.7m UPON REFUSAL ON PROBABLE BEDROCK.
WATER LEVEL AT 0.7m UPON COMPLETION.
BOREHOLE BACKFILLED WITH HOLEPLUG TO SURFACE.

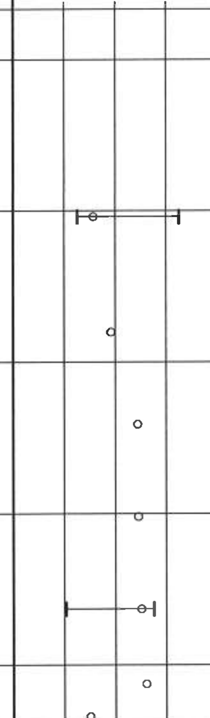
ELEVATION SCALE: 344, 343, 342, 341, 340

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

+ ³, × ³: Numbers refer to Sensitivity

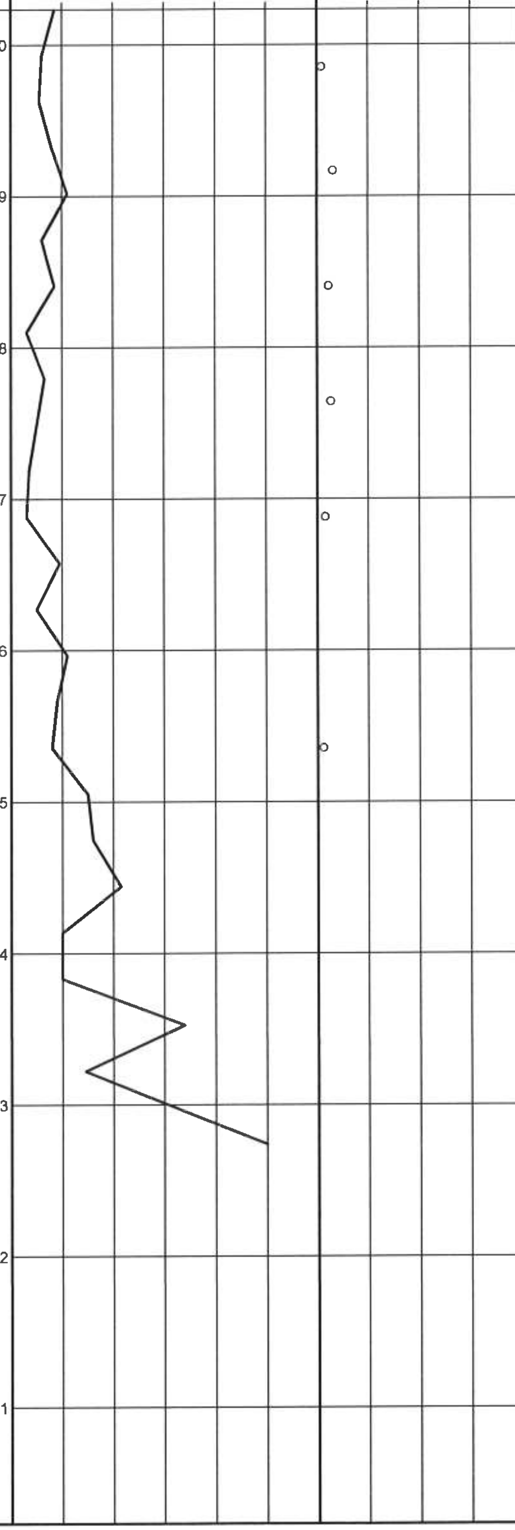
15-⊕-5 (20 above, 10 below) (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No MHLC-02

METRIC

W.P.	6931-10-00	LOCATION	N 5 395 478.3 E 294 196.7 Moosehorn Lake Culvert	ORIGINATED BY	ES
HWY	17	BOREHOLE TYPE	Hollow Stem Augers/Coring	COMPILED BY	AN
DATUM	Geodetic	DATE	2011.06.16 - 2011.06.16	CHECKED BY	RPR

ELEV / DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
350.1					
0.0	**ASPHALT:** (100mm)				
0.1	**SAND** and **GRAVEL**, trace to some silt and clay Compact Brown Damp (FILL) Occasional cobbles	1	GS		
		1	SS	24	
	Dark Brown	2	SS	26	47 38 15 (SI+CL)
		3	SS	15	
	Loose	4	SS	8	49 38 14 (SI+CL)
	Compact Trace gravel Auger refusal at 5.2m and start coring cobbles and boulders from 5.2m to 6.1m	5	SS	16	
		1	RUN		RUN #1 TCR=100% SCR=100% RQD=100%
		6	SS	17	
343.3 6.8	Silty **CLAY**, some sand to sandy, occasional organics Very Stiff Dark Brown to Reddish Brown Occasional cobbles and boulders at 7.2m and 8.7m	7	SS	12	0 26 39 35
		8	SS	10	
		9	SS	13	0 1 37 62
340.1		10	SS	50/	



UNIT WEIGHT γ kN/m³

Continued Next Page

+ ³, × ³: Numbers refer to Sensitivity

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

THURBER

RECORD OF BOREHOLE No MHLC-02

METRIC

W.P. 6931-10-00	LOCATION N 5 395 478.3 E 294 196.7 Moosehorn Lake Culvert	ORIGINATED BY ES
HWY 17	BOREHOLE TYPE Hollow Stem Augers/Coring	COMPILED BY AN
DATUM Geodetic	DATE 2011.06.16 - 2011.06.16	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										
10.0	**BEDROCK**, mafic, moderately weathered, grey, occasional mechanical breaks		2	RUN	0.05		340 339				RUN #2 TCR=100% SCR=100% RQD=100%
338.5 11.6	END OF BOREHOLE AT 11.6m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen. WATER LEVEL READINGS: DATE DEPTH (m) ELEV. (m) Aug.26/11 7.2 342.9 Sep.18/11 6.3 343.8										

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

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

Ministry of Transportation
Ontario

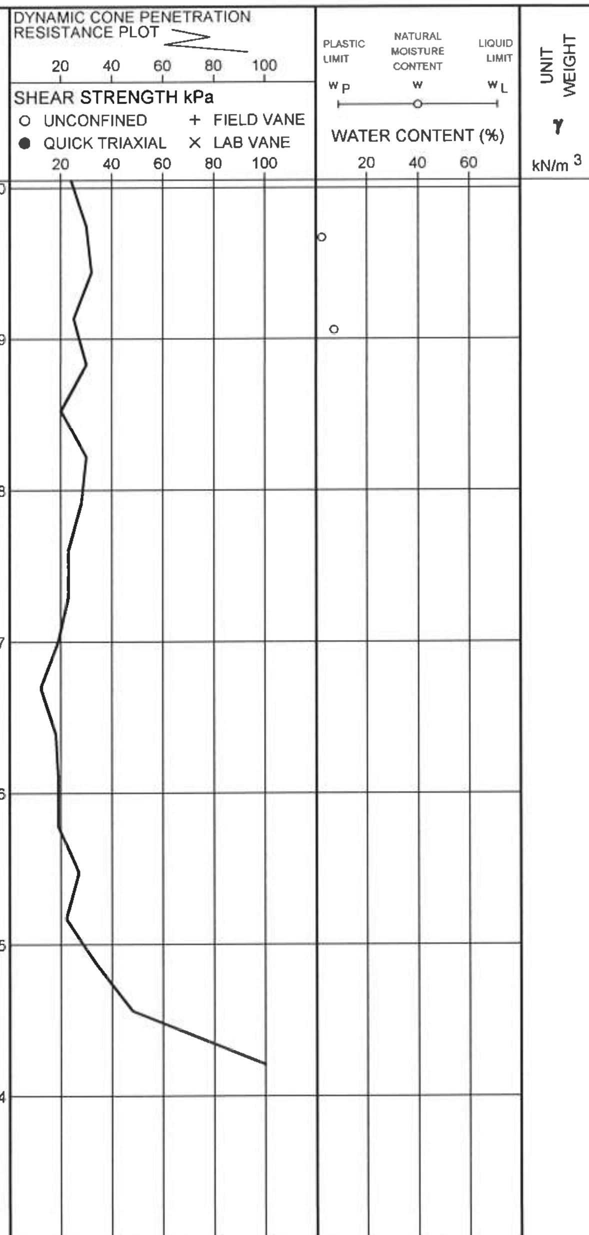
THURBER

RECORD OF BOREHOLE No MHLC-03

METRIC

W.P. 6931-10-00	LOCATION N 5 395 491.5 E 294 198.8 Moosehorn Lake Culvert		ORIGINATED BY ES
HWY 17	BOREHOLE TYPE Hollow Stem Augers/ NQCoring		COMPILED BY AN
DATUM Geodetic	DATE 2011.06.16 - 2011.06.16		CHECKED BY RPR

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
350.0						
0.0	**ASPHALT:** (110mm)					
0.1	**SAND** and **GRAVEL**, trace silt and clay, occasional cobbles Brown Damp (FILL) Very Dense	1	GS			31 62 7 (SI+CL)
		1	SS	65		
	Coring through cobbles and boulders started at 1.3m (diameter: 0.13m, 0.26m, 0.20m, 0.24m and 0.55m)	1	RUN			RUN #1 TCR=38% SCR=36% RQD=36%
		2	RUN			RUN #2 TCR=35% SCR=30% RQD=30%
345.4 4.6	Silty **CLAY**, trace sand Brown	3	RUN			RUN #3 TCR=60% SCR=22% RQD=22%
344.8 5.2	**BEDROCK**, mafic, moderately weathered, grey, occasional mechanical breaks				▽	
		4	RUN			RUN #4 TCR=100% SCR=100% RQD=97%
342.9 7.0	END OF BOREHOLE AT 7.0m. WATER LEVEL AT 5.4m UPON COMPLETION. BOREHOLE BACKFILLED WITH HOLEPLUG TO 3.0m, AUGER CUTTINGS TO 0.1m, THEN ASPHALT TO SURFACE.					



+ 3, × 3: Numbers refer to Sensitivity

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

ONTMT4S 5121.GPJ 12/20/11

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No MHLC-04

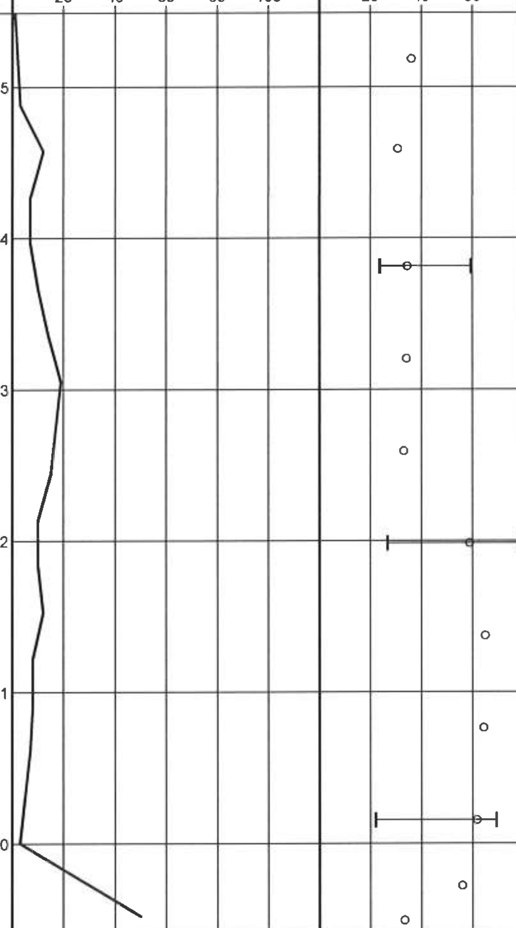
METRIC

W.P. 6931-10-00 LOCATION N 5 395 486.6 E 294 179.9 Moosehorn Lake Culvert ORIGINATED BY ES

HWY 17 BOREHOLE TYPE Mud Rotary and Tripod COMPILED BY AN

DATUM Geodetic DATE 2011.07.20 - 2011.07.20 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
345.4							
0.0	**TOPSOIL**, occasional roots: (50mm) Dark Brown	1	SS	4			
	Silty **CLAY**, trace sand, occasional roots Soft Brown to Grey				∇	345	
		2	SS	4			
	Boulder at 1.2m Firm to Very Stiff					344	
		3	SS	7			0 2 43 55
		4	SS	16		343	
		5	SS	13			
		6	SS	11		342	0 1 29 70
	Soft to Very Soft	7	SS	4			
		8	SS	2		341	
		9	SS	1		340	0 1 45 54
339.5 / 5.9		10	SS	55/ 0.300			
339.3 / 6.1	**SAND**, trace silt Dense Grey Wet						



END OF BOREHOLE AT 6.1m UPON REFUSAL ON PROBABLE BEDROCK.
WATER LEVEL AT 0.5m UPON COMPLETION.
BOREHOLE BACKFILLED WITH HOLEPLUG TO SURFACE.

ONTMT4S 5121.GPJ 12/20/11

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

Ministry of Transportation
Ontario

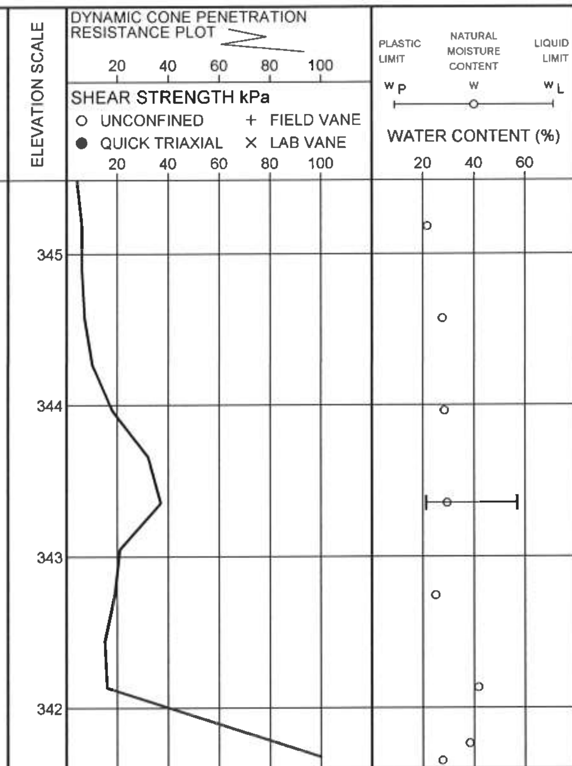
THURBER

RECORD OF BOREHOLE No MHLC-04A

METRIC

W.P. 6931-10-00	LOCATION N 5 395 496.9 E 294 185.9 Moosehorn Lake Culvert	ORIGINATED BY ES
HWY 17	BOREHOLE TYPE Mud Rotary and Tripod	COMPILED BY AN
DATUM Geodetic	DATE 2011.07.21 - 2011.07.21	CHECKED BY RPR

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
345.4 / 0.0	**TOPSOIL:** (50mm)					
	Silty **CLAY**, trace sand, occasional roots and rootlets; Soft to Firm; Dark Brown to Reddish Brown	1	SS	4		
		2	SS	6		
	Stiff to Very Stiff	3	SS	15	∇	
		4	SS	29		0 1 46 54
		5	SS	18		
		6	SS	11		
341.6		7	SS	50/0.075		0 2 60 38
3.8	**SAND**, some silt; Grey; Wet					
3.9	END OF BOREHOLE AT 3.9m UPON REFUSAL ON PROBABLE BEDROCK. WATER LEVEL AT 1.5m UPON COMPLETION. BOREHOLE BACKFILLED WITH HOLEPLUG TO SURFACE.					



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

Appendix B

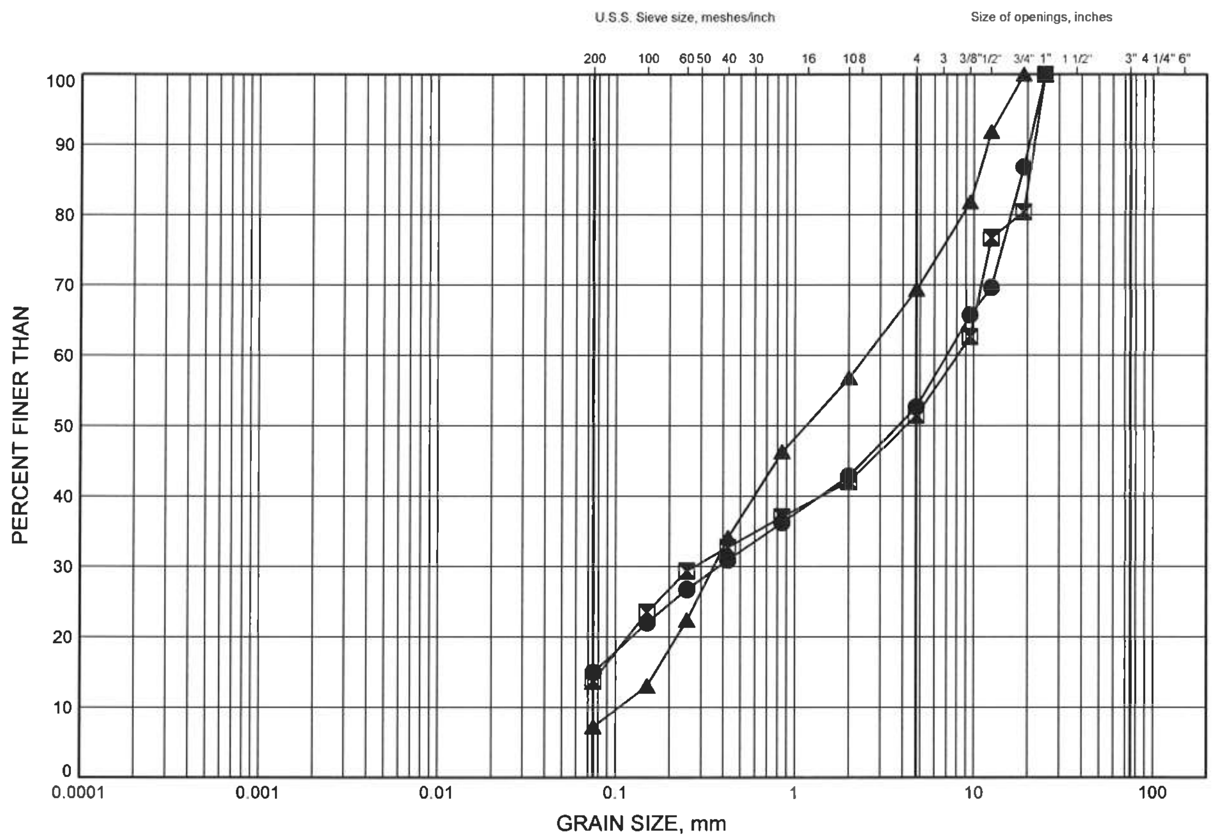
Laboratory Test Results

6010-E-0010 Bridge and Culvert Rehabs NWR

GRAIN SIZE DISTRIBUTION

FIGURE 1

SAND & GRAVEL FILL



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	MHLC-02	1.83	348.32
⊠	MHLC-02	3.35	346.79
▲	MHLC-03	0.38	349.58

W.P.# 6931-10-00

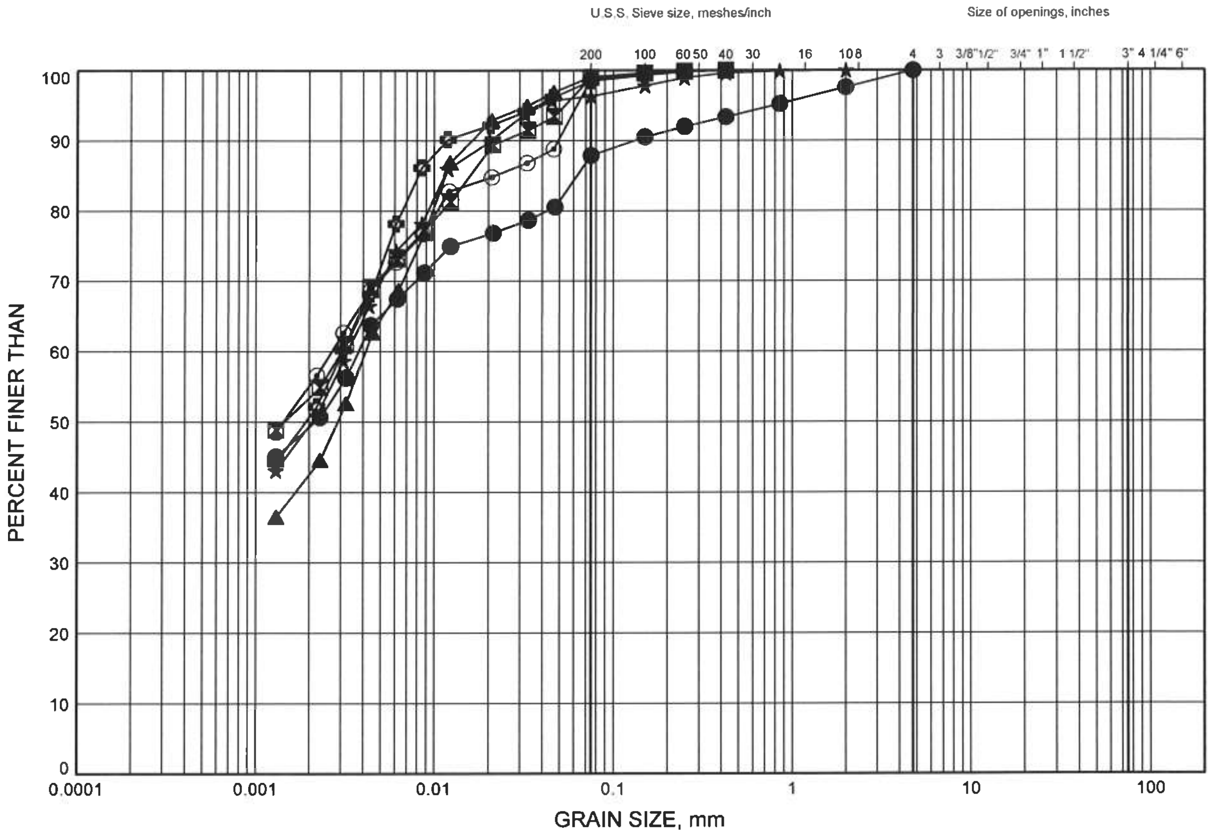
Prepared By AN

Checked By RPR

THURBER

6010-E-0010 Bridge and Culvert Rehabs NWR **GRAIN SIZE DISTRIBUTION**	FIGURE 2

SILTY CLAY



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

LEGEND

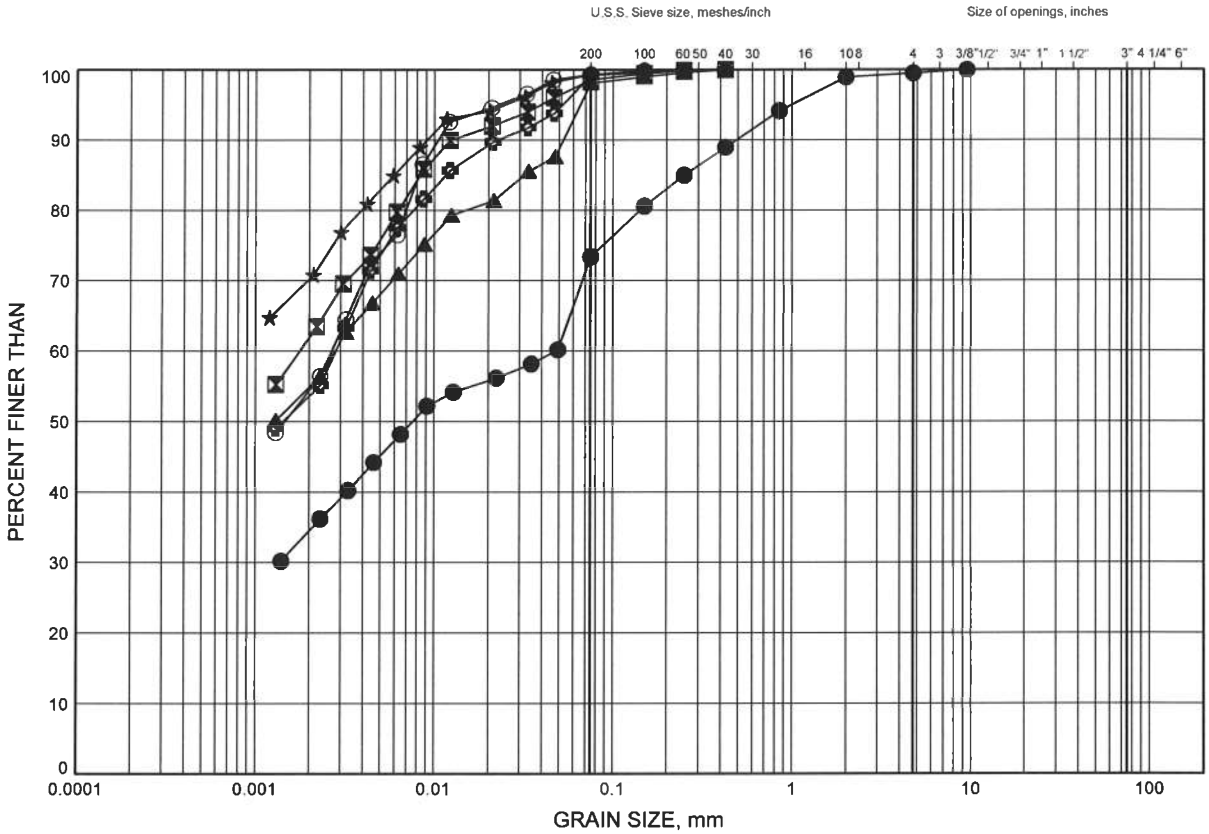
SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	MHLC-01	2.13	342.45
⊠	MHLC-01	3.96	340.62
▲	MHLC-01	5.79	338.79
★	MHLC-01	7.01	337.57
⊙	MHLC-01A	1.37	342.87
✿	MHLC-01A	3.96	340.28

W.P.# 6931-10-00
Prepared By AN
Checked By RPR

THURBER

6010-E-0010 Bridge and Culvert Rehabs NWR **GRAIN SIZE DISTRIBUTION**	FIGURE 3

SILTY CLAY



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	MHLC-02	7.24	342.91
⊠	MHLC-02	8.69	341.46
▲	MHLC-04	1.68	343.73
★	MHLC-04	3.51	341.90
⊙	MHLC-04	5.33	340.07
✿	MHLC-04A	2.13	343.27

W.P.# 6931-10-00

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Checked By RPR

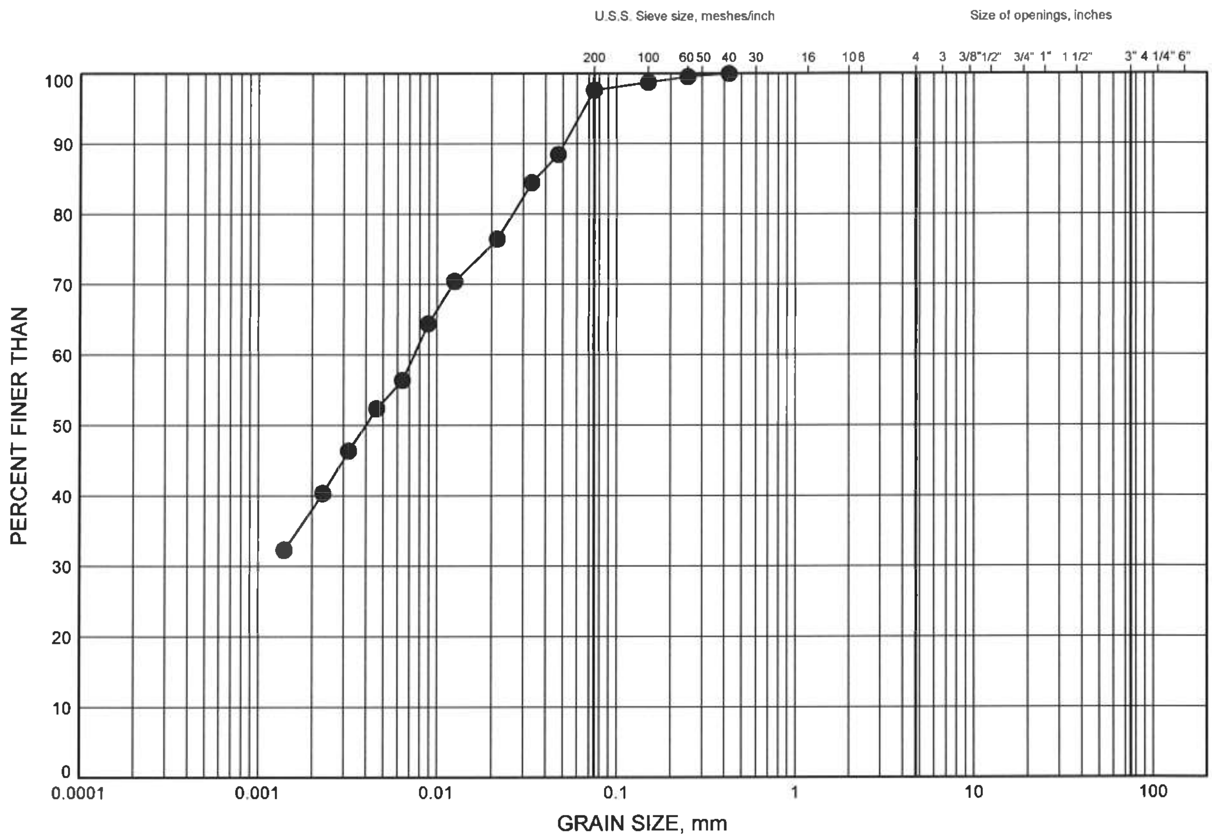
THURBER

6010-E-0010 Bridge and Culvert Rehabs NWR

GRAIN SIZE DISTRIBUTION

FIGURE 4

SILTY CLAY



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	MHLC-04A	3.72	341.68

W.P.# 6931-10-00

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Checked By RPR

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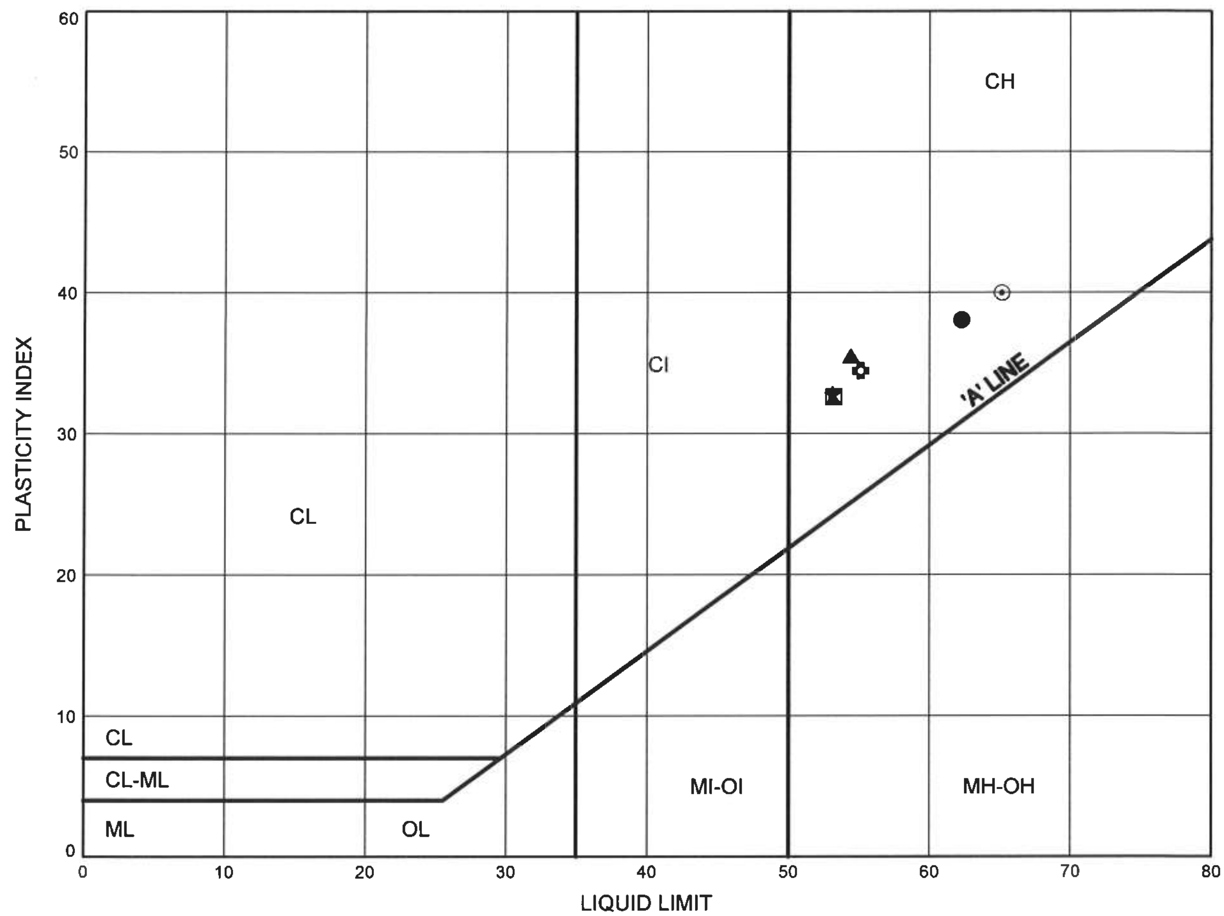
6010-E-0010 Bridge and Culvert Rehabs NWR

ATTERBERG LIMITS TEST RESULTS

FIGURE 5

SILTY CLAY



SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	MHLC-01	2.13	342.45
⊠	MHLC-01	3.96	340.62
▲	MHLC-01	5.79	338.79
★	MHLC-01	7.01	337.57
⊙	MHLC-01A	1.37	342.87
✿	MHLC-01A	3.96	340.28

Date December 2011

Project 6931-10-00

THURBER

Prep'd AN

Chkd. RPR

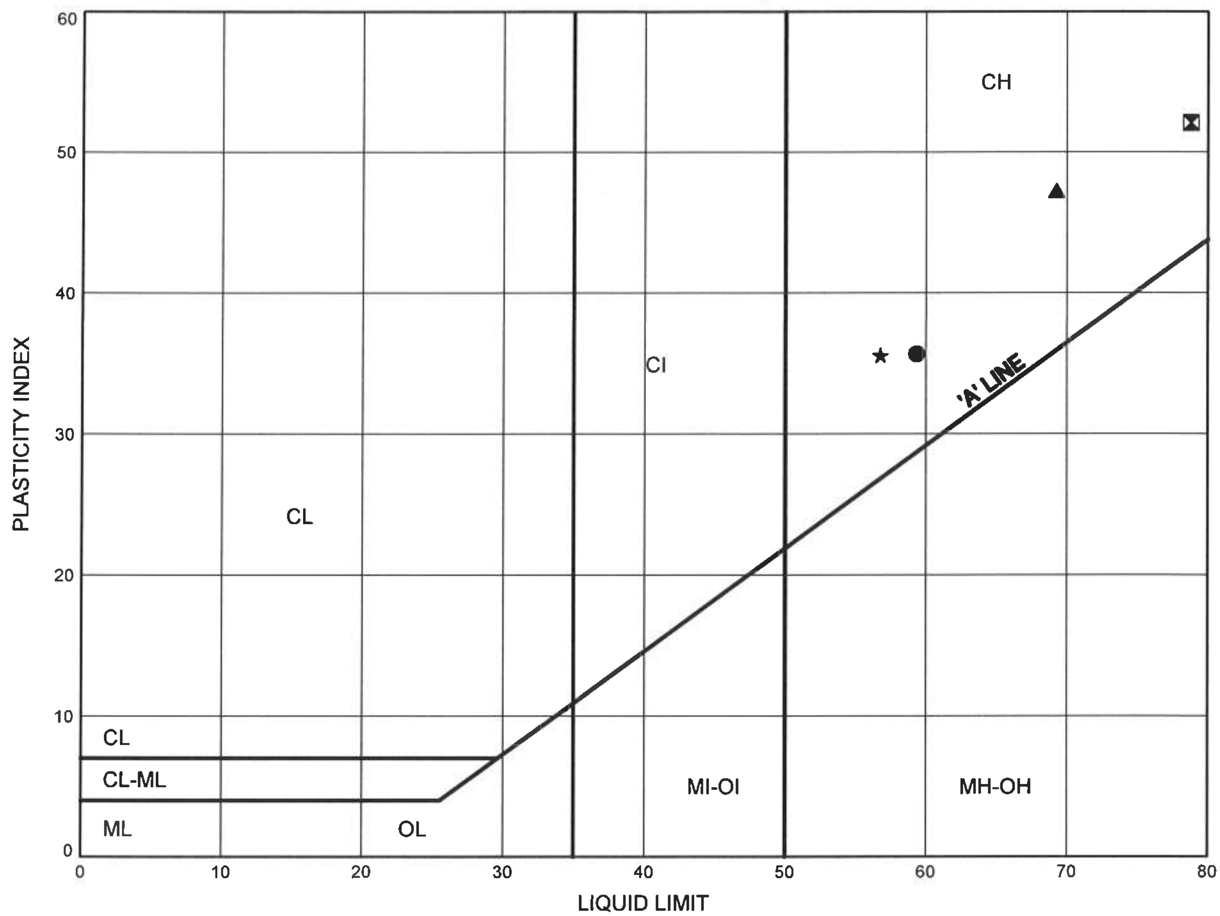
THURBALT 5121.GPJ 12/20/11

6010-E-0010 Bridge and Culvert Rehabs NWR

ATTERBERG LIMITS TEST RESULTS

FIGURE 6

SILTY CLAY



SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	MHLC-04	1.68	343.73
⊠	MHLC-04	3.51	341.90
▲	MHLC-04	5.33	340.07
★	MHLC-04A	2.13	343.27

Date December 2011

Project 6931-10-00

THURBER

Prep'd AN

Chkd. RPR

THURBALT 5121.GPJ 12/20/11

THURBER ENGINEERING LTD.
GEOTECHNICAL • ENVIRONMENTAL • MATERIALS

POINT LOAD TEST SHEET

Job No : 19-1605-121 **Client :** Hatch Mott MacDonald

Date Drilled : 6/16/2011

Project Name : 6010-E-0010 Bridge and Culvert Rehabs NWR **Date Tested :** 7/5/2011

Core Size : NQ **BH No :** MHLC-02 **Tester :** DB

Test No.	Run No.	Depth (m)	Axial or Diametral	Gauge (kPa)	Diameter (mm)	Length (mm)	UCS (MPa)	Rock Type	Notes
4	5	10.2	D	13560	47.4	90.4	134.3	Mafic	Very Strong
5	5	10.8	D	19400	47.3	77.8	192.4	Mafic	Very Strong
6	5	11.4	D	25360	47.4	69.0	251.1	Mafic	Extremely Strong
7	5	11.8	D	18200	47.4	96.1	180.1	Mafic	Very Strong
8									
9									
10									
11									
12									
13									
14									
15									
16									
17									
18									
19									
20									
21									
22									
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24									
25									
26									
27									
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.

THURBER ENGINEERING LTD.
GEOTECHNICAL • ENVIRONMENTAL • MATERIALS

POINT LOAD TEST SHEET

Job No :	19-1605-121	**Client :**	Hatch Mott MacDonald
		Date Drilled :	6/16/2011
Project Name :	6010-E-0010 Bridge and Culvert Rehabs NWR	**Date Tested :**	7/5/2011
Core Size : NQ	**BH No :** MHLC-03	**Tester :**	DB

Test No.	Run No.	Depth (m)	Axial or Diametral	Gauge (kPa)	Diameter (mm)	Length (mm)	UCS (MPa)	Rock Type	Notes
7	3	5.4	D	33700	47.2	88.7	335.7	Mafic	Extremely Strong
8	4	5.6	D	20180	47.2	69.8	200.6	Mafic	Very Strong
9	4	5.9	D	23460	47.1	78.0	233.9	Mafic	Very Strong
10	4	6.3	D	28420	47.2	81.9	282.9	Mafic	Extremely Strong
11	4	6.6	D	25900	47.2	61.1	258.0	Mafic	Extremely Strong
12	4	6.9	D	23520	47.2	81.2	233.8	Mafic	Very Strong
13									
14									
15									
16									
17									
18									
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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.

Appendix C

Site Photographs

Photograph 1 – Highway 11 and Moosehorn Lake Culvert crossing

Photograph 2 – Highway 11 and Moosehorn Lake Culvert crossing

Photograph 3 – Existing conditions of the embankment and side slope at Highway 11 and Moosehorn Lake Culvert crossing

Photograph 4 – Existing two-cell timber culvert at Highway 11 and Moosehorn Lake Culvert crossing

Appendix D

Foundation Comparison

COMPARISON OF FOUNDATION ALTERNATIVES FOR EACH FOUNDATION ELEMENT

Precast Panel on Sheet Piles	Open Footings Culvert Founded on			
	Footing on Native Soil	**Footings on Bedrock**	**Augered Caissons (drilled shafts)**	**Driven H-Piles**
Advantages: i. Minimizes potential for disturbance of streambed. ii. Ease of construction. iii. Provides shoring and foundation elements in one operation. iv. Installation of piles could continue in freezing weather. v. Potentially minimizes volume of excavation and roadway protection requirements. ***Disadvantages:*** i. Unconventional design. ii. Cost of sheet piles.	***Advantages:*** i. Economical to install. ***Disadvantages:*** i. Low available geotechnical resistance in overburden soils. ii. High groundwater levels. Dewatering will be required. iii. Deep excavation extending below the groundwater level is required.	***Advantages:*** i. High values of geotechnical resistance are available on the bedrock. ii. Lower cost than deep foundations ***Disadvantages:*** i. Sloping bedrock surface. ii. High cost of excavation to bedrock. iii. Mass concrete fill required to create a level founding surface. iv. Control of groundwater will be required.	***Advantages:*** i. High geotechnical resistance available for caissons founded on bedrock. ii. Installation less influenced by weather and groundwater than spread footings. ***Disadvantages:*** i. Difficulty in unwatering, cleaning and inspecting bases. Requires placement of concrete by tremie methods. ii. More costly than conventional footing construction in normal situations.	***Advantages:*** i. High geotechnical resistance available by driving piles to achieve resistance on bedrock. ii. Installation of piles could continue in freezing weather iii. Foundation construction may require less volume of excavation than footings. iv. Readily installed. ***Disadvantages:*** i. Higher unit costs than footings. ii. Pile lengths required to achieve design resistance may vary.
FEASIBLE	**NOT RECOMMENDED**	**NOT RECOMMENDED**	**NOT RECOMMENDED**	**FEASIBLE**

Appendix E

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, November 2009
- OPSD 803.010
- OPSS 501 dated November 2010
- OPSS 804, November 2010
- OPSS 902, November 2010
- OPSS 539

2. Suggested text for a NSSP on Steel Sheet Pile installation

The existing embankment fill contains cobbles and boulders. Steel sheet piles may encounter refusal on these cobbles and boulders or shallow bedrock.

Pre-drilling may be required at some locations to extend the sheet piles to the design depth.

The sheet piles should be provided with sheet pile tip protectors to minimize tip damage.

If sheet piles meet refusal at a depth less than the anticipated depth, the QVE must terminate driving before the pile is damaged due to over-driving. The QVE must immediately bring it to the attention of the CA. If the CA cannot resolve the issue, it must be referred to the design team for resolution.

3. Suggested text for a NSSP on H-Pile Installation

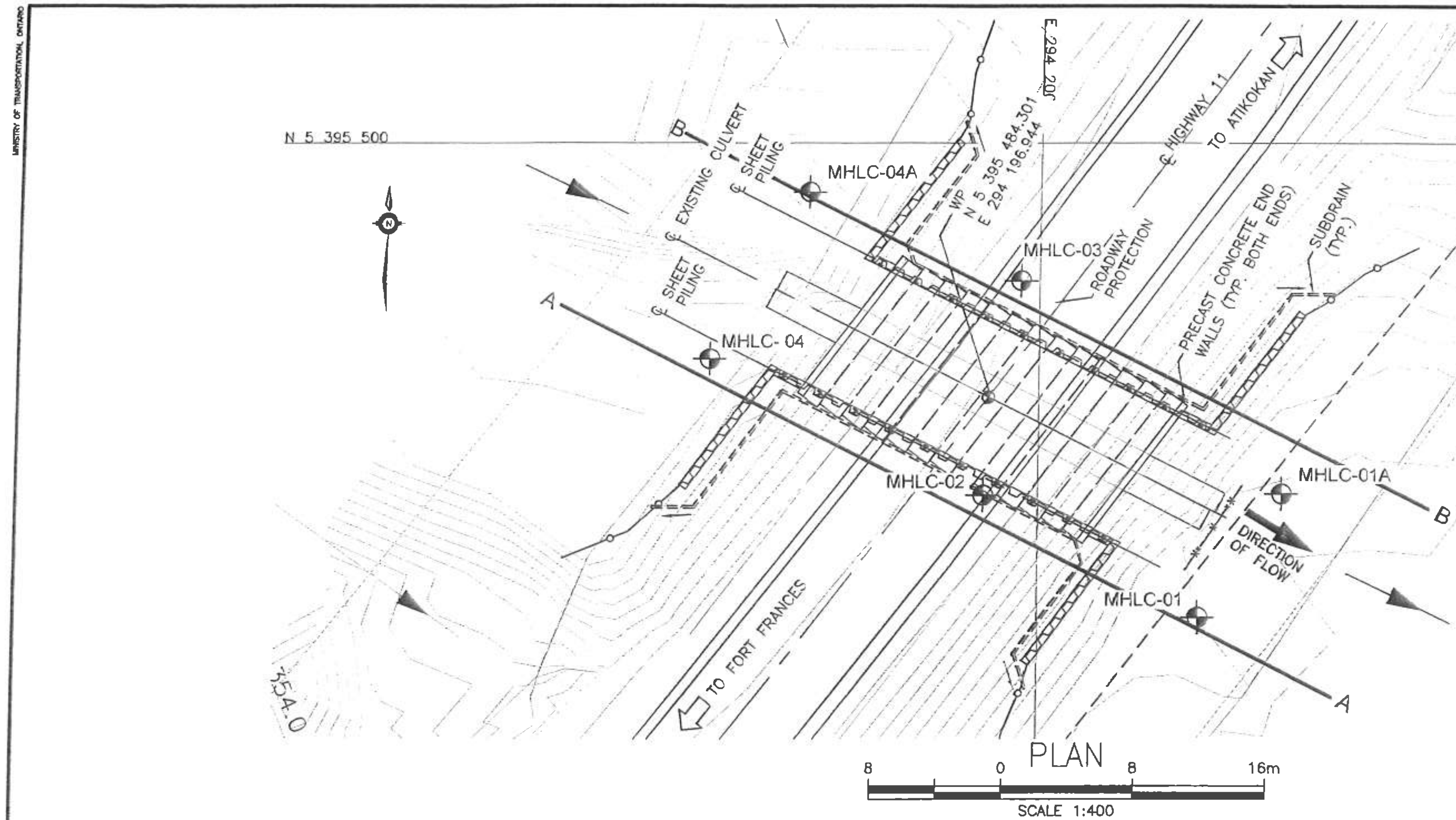
The existing embankment fill contains cobbles and boulders, which may potentially have an impact on the installation of piles at the site. Some possible impacts that must be taken into consideration include, but are not necessarily limited to:

- The need to provide protection to the H-pile tips in the form of rock points.
- The cobbles may impede the driving of the piles resulting in more arduous driving to reach bedrock.
- Pre-drilling might be required at some locations to extend the pile to the design depth.
- As a result of the presence of boulders, piles may meet refusal at varying depths.
- At some locations, piles will be driven to sloping bedrock conditions.
- If a pile meets refusal at a depth less than the anticipated depth, the QVE must terminate driving before the pile is damaged due to over-driving. the QVE must immediately bring it to the attention of the CA. If the CA cannot resolve the issue, it must be referred to the design team for resolution.

Appendix F

Drawing
Borehole Locations and Soil Strata

MINISTRY OF TRANSPORTATION, ONTARIO



PLAN

8 0 8 16m

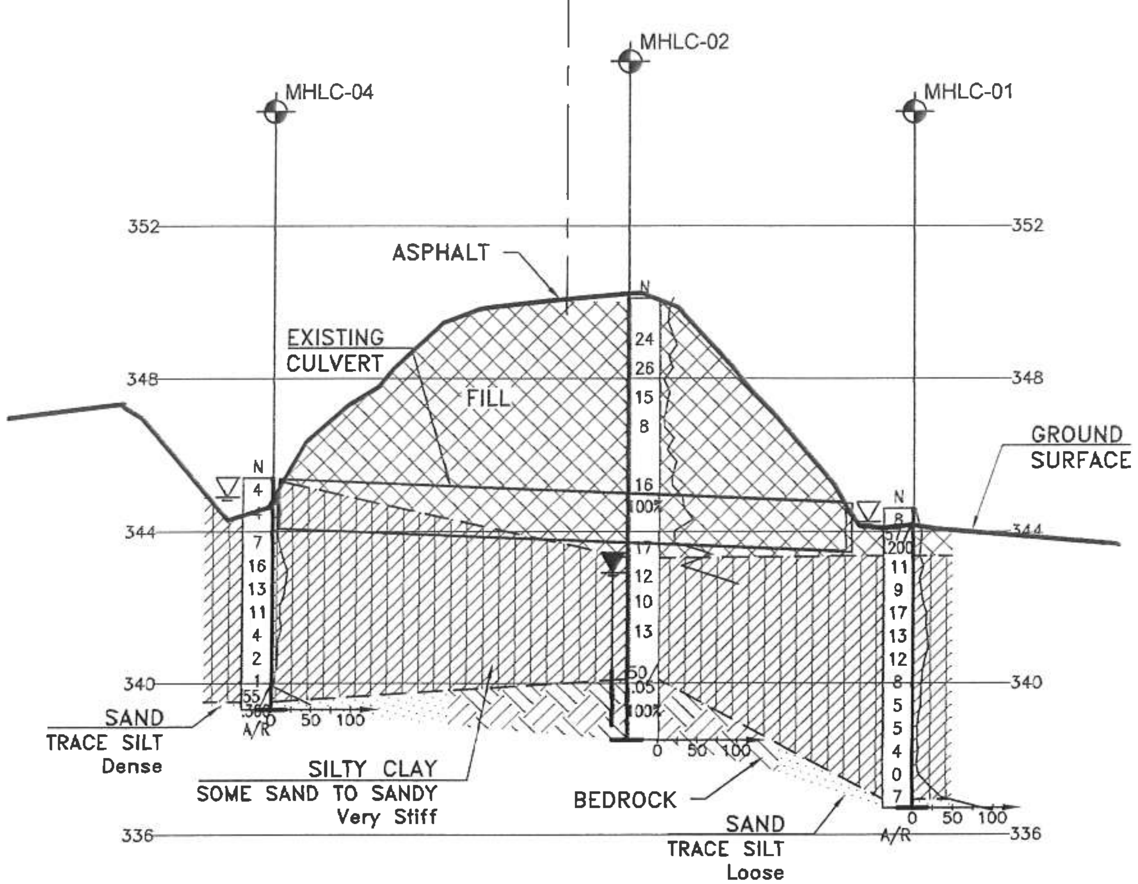
SCALE 1:400

METRIC

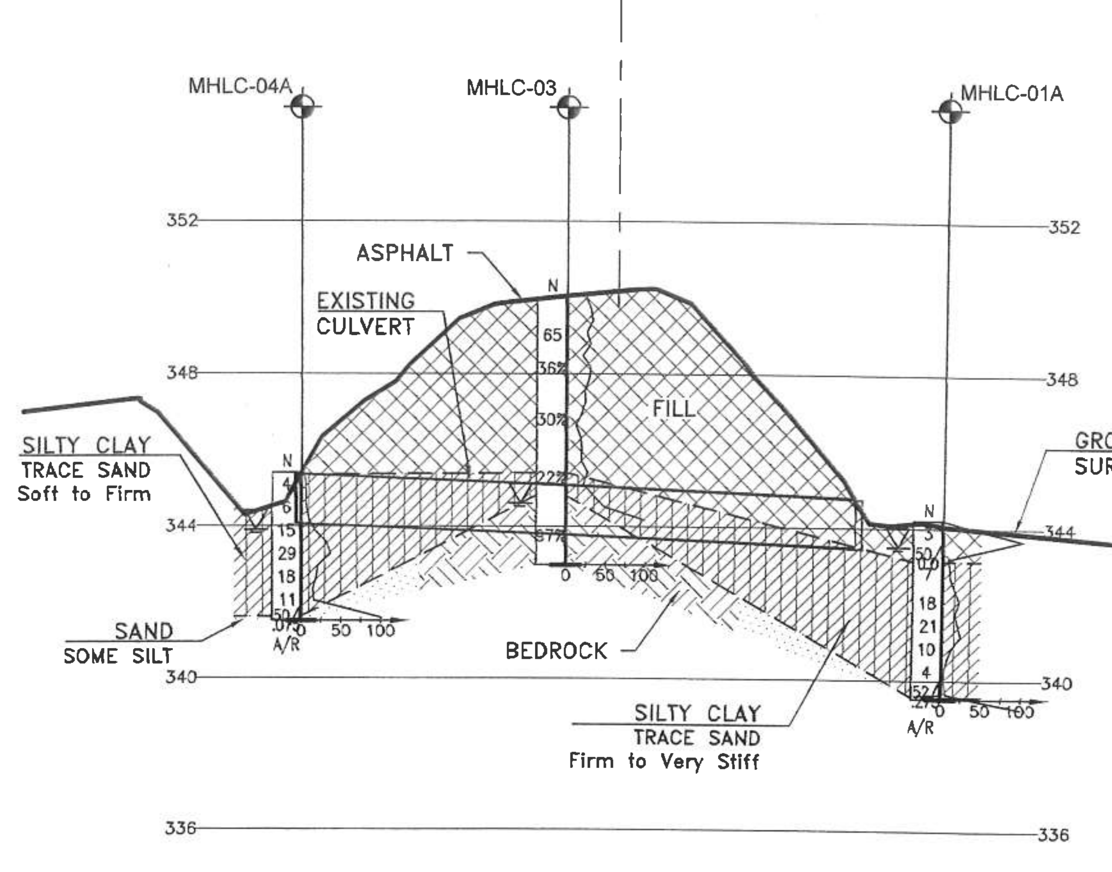
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

LICENSED PROFESSIONAL ENGINEER R. Palomeque Reyna 100083209 Dec/20/2011 PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER P. K. CHATTERJI Dec 20/11 PROVINCE OF ONTARIO



SECTION ALONG A–A



SECTION ALONG B–B

8 0 8 16m H 1:400

4 0 4 8 V 1:200

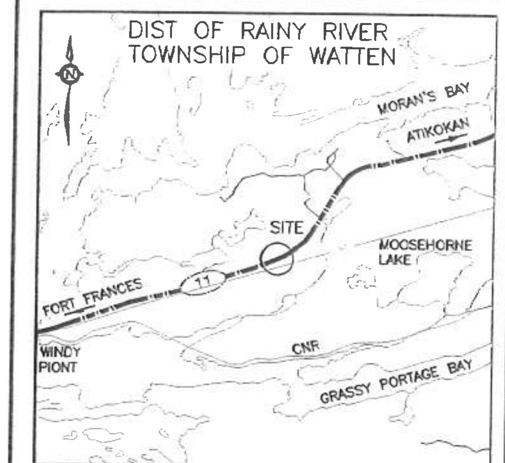
CONT No 2011–6034
WP No 6931–10–00

HIGHWAY 11
BRIDGE & CULVERT REHABS
MOOSEHORN LAKE
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET 8

Hatch Mott MacDonald

THURBER ENGINEERING LTD.



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
MHLC–01	344.5	5 395 470.9	294 209.8
MHLC–01A	344.2	5 395 478.4	294 214.8
MHLC–02	350.1	5 395 478.3	294 196.7
MHLC–03	350.0	5 395 491.5	294 198.8
MHLC–04	345.4	5 395 486.5	294 179.9
MHLC–04A	345.4	5 395 496.9	294 185.9

-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. 52C-24

REVISIONS	DATE	BY	DESCRIPTION

DESIGN RPR	CHK RPR	CODE CAN/CSA S6-06	LOAD CL-625-ONT	DATE	DEC. 2011		
DRAWN AN	CHK RPR	SITE 45-276/C	STRUCT	DWG 2			

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