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

G.W.P. 6930-10-00, SITE No. 45-260/C

Geocres Number: 52B-13

Report to

Hatch Mott MacDonald

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TABLE OF CONTENTS

Appendices

Appendix A	Record of Borehole Sheets
Appendix B	Laboratory Test Results
Appendix C	Site Photographs
Appendix D	Foundation Comparison
Appendix E	List of SPs and OPSS, and Suggested Text for NSSP
Appendix F	Borehole Locations and Soil Strata Drawings

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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 Caribus 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 written descriptions 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 Caribus Lake culvert is located on Highway 11, approximately 5 km west of the intersection of Highway 622 and Highway 11. This site is located in the Rainy River District of Ontario.

The existing Caribus Lake culvert is a 2-cell timber culvert. The cells are 2.1 m wide each and the total length of the culvert is 28 m. As observed in the field, an extension has been added to the south end (inlet) of the existing culvert. The extension consists of a 2-m diameter CSP including a shaped beaver and debris trap. The water flows through the culvert from south to north.

The road embankment is approximately 3.0 m to 4.0 m high above Caribus Lake. The surrounding lands are undeveloped, heavily treed and relatively flat. Bedrock outcroppings and small creeks/water bodies are visible along the existing Highway 11 right-of-way.

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

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

3 SITE INVESTIGATION AND FIELD TESTING

The site investigation and field testing for this project was carried out on January 27 and 28, 2011 and February 16, 2011 and consisted of drilling and sampling two boreholes (identified as CAR11-01 and CAR11-02) in the area of the existing culvert, through the existing highway embankment. Borehole CAR11-01 was located on the Highway 11 eastbound shoulder, northwest of the existing culvert and Borehole CAR11-02 was located on the Highway 11 westbound shoulder, southeast of the existing culvert. Boreholes were advanced within the overburden to 2.9 m and 3.7 m depth (Elevations 406.6 and 406.3) where the drill rig encountered refusal. Bedrock was proved in both boreholes by NQ size diamond coring. Borehole CAR11-01 was advanced 2.5 m into bedrock and terminated at 6.2 m depth (Elevation 403.7). Borehole CAR11-02 was advanced 0.5 m into bedrock and terminated at 3.4 m depth (Elevation 406.1).

The boreholes were supplemented by three Dynamic Cone Penetration Tests (DCPTs), identified as DCPT1, DCPT2, and DCPT3. DCPT1 and DCPT2 were conducted near the culvert outlet and DCPT3 was conducted near the culvert inlet. TheDCPTs were terminated at 0.3 m and 1.1 m depth (Elevations 404.7 to 407.8), upon refusal on probable bedrock or boulder.

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 was carried out using a truck-mounted CME 75 drill rig and the boreholes were advanced with hollow-stem augers and NQ coring techniques. Overburden samples were obtained at selected intervals using a split spoon sampler in conjunction with Standard Penetration Testing (SPT). The DCPTs were conducted using portable equipment.

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

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

Groundwater conditions in the open boreholes were observed throughout the drilling operations. Upon completion of drilling, boreholes were backfilled with sand and auger cuttings to 0.12 m, then asphalt 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). 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 overlying sand and gravel fill. Granitic gneiss bedrock was encountered directly below the fill at depths ranging from 2.9 m to 3.7 m. More detailed descriptions of the individual strata are presented below.

5.1 Pavement structure

Pavement structure was encountered in the two boreholes drilled at this site. Both boreholes were drilled through the existing Highway 11 shoulders. The pavement structure consists of approximately 75 mm to 100 mm of asphalt overlying granular fill.

5.2 Fill

Brown sand and gravel fill containing some silt and trace clay was encountered below the pavement structure in both boreholes. The thickness of the fill ranged from 2.8 m to 3.6 m.

The depth to the base of the fill was 3.7 m and 2.9 m (Elevations 406.3 and 406.6) in Boreholes CAR11-01 and CAR11-02, respectively.

SPT N-values recorded in the sand and gravel fill were quite variable and ranged from 6 blows for 0.3 m penetration to 50 blows for less than 0.3 m penetration, indicating a loose to very dense condition. However, frost was observed in the ground to an approximate depth of 1.4 m and is therefore likely the cause of the high SPT N-values. Though not encountered in the boreholes, the fill may contain cobbles and boulders. The moisture content of the fill samples ranged from 3% and 16%.

Selected samples of the fill underwent laboratory grain size analysis testing, the results of which are summarized below. These results are also presented on the Record of Borehole sheets included in Appendix A. The grain size distribution curves for these samples are shown on Figure B1 of Appendix B.

Soil Particles	Percentage (%)
Gravel	24 to 36
Sand	38 to 57
Silt	11 to 22
Clay	4 to 7

5.3 Bedrock

The overburden soils described above are underlain by dark grey granitic gneiss bedrock. The bedrock is slightly weathered near the surface. Occasional mechanical breaks and sub-vertical fractures were observed in the rock cores.

Bedrock was proved by coring at each borehole. Table 5.1 summarizes depths and elevations to the top of bedrock in the boreholes.

Table 5.1 – Depths and Elevations of Top of Bedrock

Borehole	Top of Bedrock	
	Depth (m)	Elevation (m)
CAR11-01	3.7	406.3
CAR11-02	2.9	406.6

Core recovery in the bedrock was 100% in all cores except in Borehole CAR11-01 Run 2 where recovery was only 42%. The RQD values were quite variable and ranged from 0% to 83%, indicating very poor to good rock quality. The Fracture Index (FI) of the rock, expressed as fractures per 0.3 m of core, generally ranged from 0 to 5.

The estimated unconfined compressive strength of the rock cores ranged from 192 MPa to 201 MPa, indicating a very strong to extremely strong rock. These estimated rock strength values are interpreted from point load tests that were conducted on rock cores recovered from the boreholes. A summary of the Point Load Test Results is presented in Appendix B.

5.4 Water Levels

Water levels were observed in the open boreholes upon completion of drilling operations. The water levels observed in the open boreholes are summarized in Table 5.2.

Table 5.2 – Water Level Measurements

Borehole	Date	Water Level		Comment
		Depth (m)	Elev. (m)	
CAR11-01	January 27, 2011	2.8	407.2	Open borehole
DCPT1	February 16, 2011	0.3	405.6	Open DCPT

The water level in the Caribus Lake culvert was measured at Elevation 406.5 on July 6, 2006.

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. Portable DCPT equipment was also supplied and operated by Eastern Ontario Diamond Drilling Ltd. The drilling operations were supervised by Mr. Ryan Kromer, of Thurber.

Overall supervision of the field program was conducted by Mr. Tony Harte. Interpretation of the data and preparation of the report were carried out by Ms. Lindsey Blaine, E.I.T. and 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 Caribus Lake in the District of Rainy River, Ontario.

The existing culvert is a two-cell timber structure underlying the Highway 11 embankment. Each cell is 2.1 m high. The total length of the culvert is 28.0 m. The south end of the culvert was previously extended with a 2-m diameter CSP which includes a shaped beaver and debris trap.

The proposed replacement culvert (as shown on the Preliminary General Arrangement dated July 27, 2011) consists of two parallel sheet pile walls supporting a precast cap consisting of concrete panels. The new structure will have a span of 9.6 m, and a length of 26.7 m of which 24.2 m will be capped by precast panels.

The approximate original ground level is between Elevations 406.0 and 406.6. The finished road grades over the culvert will be at Elevation 409.8 m, resulting in a maximum embankment height of about 3.8 m.

The proposed culvert design was selected to avoid or minimize any disturbance or environmental impact on the stream 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 soils encountered above the existing culvert consist of pavement structure and embankment fill consisting of loose to very dense sand and gravel. Granitic gneiss bedrock was encountered directly below the fill at 2.9 m and 3.7 m depth.

Water level was observed at 2.8 m (Elevation 407.2) below ground surface in Borehole CAR11-01 upon completion of drilling.

The water level in the Caribus Lake culvert was measured at Elevation 406.5 on July 6, 2006.

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

For the open footing culvert option, use of deep foundations such as piles and caissons is not considered practical or cost effective at this site due to the presence of shallow bedrock. Consequently, deep foundations are not recommended and detailed design recommendations for these foundations types were not developed.

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 driven to achieve resistance on bedrock.

However, due to the shallow bedrock encountered at this site, the embedment of the sheet piles will be shallow and additional lateral support will be required for lateral stability of the sheet piles.

Sheet piles must be driven to bedrock 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 for piles driven to bedrock or refusal (m)
West wall	CAR11-01 DCPT2 DCPT3	406.3* 407.8 405.3
East wall	CAR11-02 DCPT1	406.6* 404.7

*Bedrock was proven by coring in these two boreholes. The DCPT's were terminated on a layer of refusal which may be boulders or bedrock.

The factored Geotechnical Resistance at ULS (per metre width) for sheet piles driven to bedrock has been assumed to be 30% of the structural capacity of the sheet pile selected.

The factored Geotechnical Resistances at ULS (per metre width) recommended for three sheet pile sections driven to bedrock 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 provided with sheet tip protector to minimize tip damage.

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".

Due to the presence of shallow bedrock and proposed underside elevation of the concrete panel cap, the length of the sheet piles will be about 1.4 m to 1.7 m.

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

However, the lateral resistance is not expected to be sufficient for such short piles to provide lateral support of the sheet piles. In this regard, 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 Bedrock

Based on the subsurface stratigraphy encountered at this site, the proposed culvert panel could be supported on concrete abutment walls supported on spread footings bearing on bedrock.

Spread footings bearing on undisturbed bedrock at or below elevations presented in Table 8.1 may be designed for the following geotechnical resistance:

- Factored geotechnical resistance of 2,000 kPa at Ultimate Limit States (ULS)

The SLS condition will not govern design of footings founded on bedrock.

This resistance value is for vertical, concentric loads. Where eccentric or inclined loads are applied, the resistance used in design must be reduced in accordance with the CHBDC Clause 6.7.3 and Clause 6.7.4.

The bearing surface should be prepared by removing all loose/disturbed material and shattered/loosened rock fragments. Areas requiring subexcavation beneath the underside of footing should be backfilled with the same class of concrete as used in the footing. The same value of resistance as the bedrock may be used where concrete of suitable strength is poured in neat contact with clean, sound bedrock surface.

8.3 Lateral Resistance on Bedrock

Initial calculations of the horizontal resistance may be carried out using a value of 0.7 for the ultimate friction factor of concrete poured on rock.

If the frictional component is insufficient, the horizontal resistance may be increased by dowelling the footing into the rock mass. Dowels are considered to be comparatively short steel bars that may be assumed to provide only shear resistance. If vertical resistance in tension is required, rock anchors must be included in the design.

A 1.0 m long 35M dowel bar grouted into a 75 mm hole is expected to provide 2.0 MN ultimate geotechnical shear resistance. The depth of embedment is measured below the bedrock surface.

Rock anchors must be installed in accordance with OPSS 942, November 2009.

The shearing resistance of the selected dowel must be checked structurally.

8.4 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 shallow along the culvert alignment. Sheet piles will meet refusal at varying depths and sheet piles may not have enough penetration.

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 into bedrock in front of the sheet pile.

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

8.5 Frost Cover

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

However, frost penetration is not an issue for sheet piles or footings bearing on bedrock or concrete fill placed on bedrock.

9 CULVERT BACKFILL AND LATERAL EARTH PRESSURES

Culvert backfill should consist of granular material conforming to OPSS Granular A or Granular B specifications. A subdrain should be provided at the base of the granular backfill.

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 similar 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, OPSS Granular B Type I or Type III $\phi = 32°, \gamma = 21.2$ kN/m^3
Active (Unrestrained Wall)	0.27	0.31
At rest (Restrained Wall)	0.43	0.47
Passive (Movement Towards Soil Mass)	3.7	3.3

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, foundation settlement 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.

The near surface foundation soils at the inlet and outlet areas consist of a relative thin layer of sand with some gravel over bedrock.

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

Earth excavation and possible rock excavation will be required at this site.

11.1 Earth Excavation

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 may be classified as Type 3 soils. This classification is based on the lack of cohesion in the soils. The sand and gravel fill below the water table is a Type 4 soil.

Water level was observed at 2.8 m (Elevation 407.2) below ground surface in one borehole upon completion of drilling. The water level in the steam was reported to be at elevation 406.5 on July 6, 2006. Excavation below the groundwater level without prior dewatering is not recommended.

Based on the preliminary culvert design, excavation below the groundwater level to construct the new sheet pile culvert will not be required. However, excavation below the creek level may be required to remove the existing culvert foundations, or if an alternate culvert design is selected.

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

Any excavation below the water level would require prior dewatering. The Contractor must be prepared to control the groundwater and surface water to permit construction in the dry. The Contract Documents should contain a NSSP alerting the Contractor to the risks associated with excavation of cohesionless soils submerged below the groundwater level without prior dewatering. Suggested wording is included in Appendix E.

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 orType III** $\phi = 32°, \gamma = 21.2$ kN/m^3
Active (K_{AE})*	0.28	0.32
Passive (K_{PE})	3.7	3.2
At Rest (K_{OE})**	0.45	0.50

* 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) method1. Using this method, it is estimated that under the existing conditions the foundation soils are not prone to liquefaction.

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.

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

γ	=	20 kN/m^3	(bulk unit weight)
γ_w	=	10 kN/m^3	(submerged unit weight under groundwater table)
K_a	=	0.31	(Active pressure coefficient for road embankment fill)
K_p	=	3.3	(Passive pressure coefficient for road embankment 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)

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.

[1] Seed, H.B. and Idriss, I.M. 1971, "Simplified Procedure for Evaluating Soil Liquefaction Potential" *Journal of Soil Mechanics and Foundations Division,* ASCE, Vol. 101, No. SM9, September, pp. 1249-1273.

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:

- Excavation below the water level, if required, will involve lowering of the groundwater level below the excavation base to maintain a reasonably dry excavation.
- 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 surface of the bedrock has been shown in the investigation to be variable. Since the elevation of the bedrock surface was only established at discrete points, it is possible that higher or lower elevations will be encountered during construction.

The successful performance of the culvert will depend largely upon good workmanship and quality control during construction. Sheet pile driving supervision, subgrade examination and field density testing 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.

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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

Ministry of Transportation Ontario

RECORD OF BOREHOLE No CAR11-01

METRIC

W.P. 19-1605-121	LOCATION Caribus Lake Culvert		ORIGINATED BY RK
HWY 11	BOREHOLE TYPE Hollow Stem Augers/NQ Coring		COMPILED BY AN
DATUM Geodetic	DATE 2011.01.27 - 2011.01.27		CHECKED BY TJH

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
410.0 / 0.0	ASPHALT: (100mm)					410			
0.1	SAND and GRAVEL, some silt to silty, trace clay, Dense to Compact, Brown, Moist to Wet (FILL)	1	AS				o		24 57 13 6
		~~1~~	~~SS~~	~~50~~			o		
						409			
		2	SS	35			o		
						408			
		3	SS	26	∇		o		33 38 22 7
						407			
	Loose	4	SS	7			o	FI	
406.3 / 3.7	BEDROCK, granitic gneiss, slightly weathered, dark grey, some joints, filled with quartz, occasional mechanical breaks	1	RUN			406		3, 1	RUN #1 TCR=100% SCR=60% RQD=60%
		2	RUN			405		1, 1	RUN #2 TCR=42% SCR=24% RQD=14% UCS=201MPa
		3	RUN			404		1, 0, 2	RUN #3 TCR=100% SCR=83% RQD=83% UCS=197MPa (Average)
403.7 / 6.2	END OF BOREHOLE AT 6.2m. WATER LEVEL AT 2.8m UPON COMPLETION. BOREHOLE BACKFILLED WITH CUTTINGS AND SAND TO 0.12m, THEN ASPHALT COLD PATCH TO SURFACE.								

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

ONTMT4S 5121.GPJ 7/14/11

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

Ministry of Transportation Ontario

RECORD OF BOREHOLE No CAR11-02

METRIC

W.P.	19-1605-121	LOCATION	Caribus Lake Culvert	ORIGINATED BY	RK
HWY	11	BOREHOLE TYPE	Hollow Stem Augers/NQ Coring	COMPILED BY	AN
DATUM	Geodetic	DATE	2011.01.28 - 2011.01.28	CHECKED BY	TJH

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
409.5								
0.0 / 0.1	**ASPHALT:** (75mm)	~~1~~	~~SS~~	~~50/~~				
	SAND and GRAVEL, some silt to silty, trace clay Loose to Very Dense Brown Moist (FILL)	1	AS	0.025		o		36 49 11 4
		~~2~~	~~SS~~	~~50/~~ 0.075	409	o		
		3	SS	6	408	o		
		4	SS	75	407	o	FI	
406.6 / 2.9	**BEDROCK**, granitic gneiss, slightly weathered, dark grey, occasional sub-vertical fractures, quartz infilling	1	RUN				5 2	RUN #1 TCR=100% SCR=38% RQD=0%
406.1 / 3.4	END OF BOREHOLE AT 3.3m. BOREHOLE DRY UPON COMPLETION. BOREHOLE BACKFILLED WITH CUTINGS AND SAND TO 0.12m, THEN ASPHALT COLD PATCH TO SURFACE.							

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60.

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

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

Ministry of Transportation
Ontario

RECORD OF BOREHOLE No DCPT1

METRIC

W.P.	19-1605-121	LOCATION	Caribus Lake Culvert	ORIGINATED BY	RK
HWY	11	BOREHOLE TYPE	Dynamic Cone Penetration Test	COMPILED BY	AN
DATUM	Geodetic	DATE	2011.02.16 - 2011.02.16	CHECKED BY	LRB

ELEV / DEPTH	DESCRIPTION (SOIL PROFILE)	STRAT PLOT	NUMBER (SAMPLES)	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
405.9											
0.0	**SAND**, some gravel, some organics,					∇					
405.4 / 0.4	Start DCPT at 0.4m						405				
404.7 / 1.1	END OF DCPT AT 1.1m. WATER OBSERVED AT 0.3m.										

ONTMT4S 5121.GPJ 7/26/11

+ ³, × ³: Numbers refer to Sensitivity

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

Ministry of Transportation
Ontario

RECORD OF BOREHOLE No DCPT2

METRIC

W.P. 19-1605-121 | LOCATION Caribus Lake Culvert | ORIGINATED BY RK

HWY 11 | BOREHOLE TYPE Dynamic Cone Penetration Test | COMPILED BY AN

DATUM Geodetic | DATE 2011.02.16 - 2011.02.16 | CHECKED BY LRB

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
408.1											
0.0	**SAND**										
407.8	Brown						408				
0.3	BEDROCK AT 0.3m.										

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

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

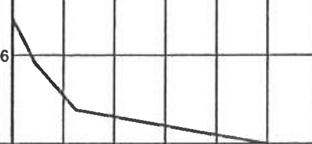
ONTMT4S 5121.GPJ 7/14/11

Ministry of Transportation
Ontario

RECORD OF BOREHOLE No DCPT3

METRIC

W.P.	19-1605-121	LOCATION	Caribus Lake Culvert	ORIGINATED BY	RK
HWY	11	BOREHOLE TYPE	Dynamic Cone Penetration Test	COMPILED BY	AN
DATUM	Geodetic	DATE	2011.02.16 - 2011.02.16	CHECKED BY	LRB

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	STRAT PLOT	SAMPLES – NUMBER	SAMPLES – 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/m^3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
406.4											
0.0	**SAND**, some gravel, occasional cobbles & boulders										
406.1 / 0.3	Brown										
	Start DCPT at 0.3m						406				
405.3 / 1.1	END OF DCPT AT 1.1m.										

ONTMT4S 5121.GPJ 6/24/11

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

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

Appendix B

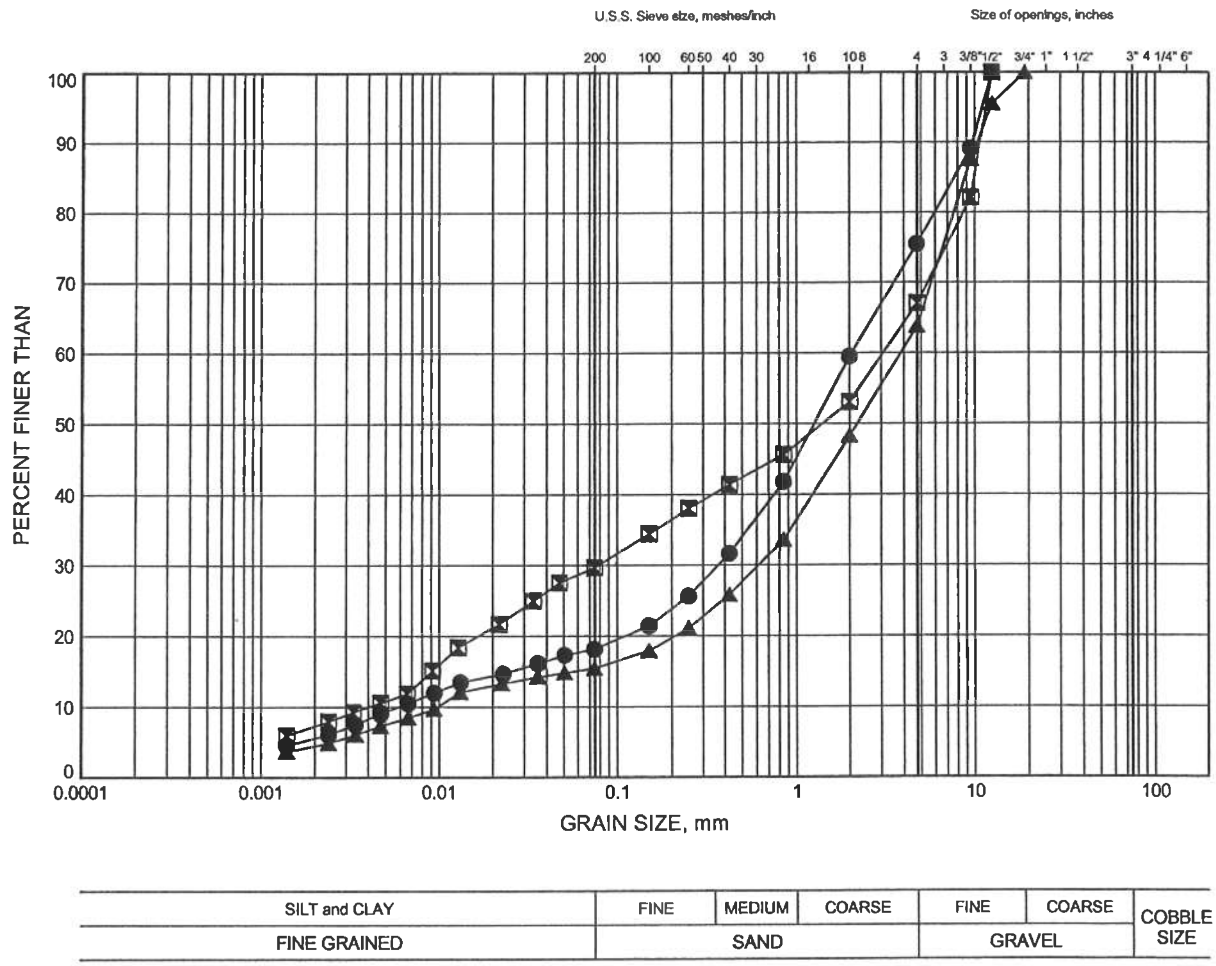
Laboratory Test Results

6010-E-0010 Bridge and Culvert Rehabs NWR

GRAIN SIZE DISTRIBUTION

FIGURE B1

SAND & GRAVEL FILL



LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	CAR11-01	0.35	409.64
⊠	CAR11-01	2.59	407.40
▲	CAR11-02	0.30	409.20

W.P.# 19-1605-121

Prepared By AN

Checked By LRB

THURBER

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POINT LOAD TEST SHEET

Job No :	19-1605-121	**Client :**	HATCH
		Date Drilled :	1/28/2011
Project Name :	Caribus Lake Culvert	**Date Tested :**	6/24/2011
Core Size : NQ	**BH No :** CAR11-01	**Tester :**	DB

Test No.	Run No.	Depth (m)	Axial or Diametral	Force (kN)	Diameter (mm)	Length (mm)	UCS (MPa)	Rock Type	Notes
1	2		D	19.3	47.3	124.2	201.5		Very Strong
2	3		D	18.4	47.3	50.9	192.6		Very Strong
3	3		D	19.4	47.3	73.7	202.5		Very Strong
4									
5									
6									
7									
8									
9									
10									
11									
12									
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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.

Appendix C

Site Photographs

Photograph 1 – Highway 11 and Caribus Lake Culvert crossing

Photograph 2 – Existing conditions of north end of Caribus Lake Culvert (looking south)

Beaver and debris trap

Photograph 3– Existing conditions of south end of Caribus Lake Culvert (looking west)

Appendix D

Foundation Comparison

COMPARISON OF FOUNDATION ALTERNATIVES FOR EACH FOUNDATION ELEMENT

Driven Sheet Piles	Footings on Bedrock
Advantages: i. Ease of construction. ii. Provides shoring and foundation elements in one operation. iii. Installation of piles could continue in freezing weather. iv. Potentially minimizes volume of excavation and roadway protection requirements. v. Minimizes potential for disturbance of streambed. ***Disadvantages:*** i. Unconventional design. ii. Cost of sheet piles. iii. Additional lateral support required for the sheet piles. **RECOMMENDED**	***Advantages:*** i. High values of geotechnical resistance are available on the bedrock. ii. Lower cost than deep foundations ***Disadvantages:*** i. High cost of excavation to bedrock. ii. Mass concrete fill required to create a level founding surface. iii. Groundwater control and streambed dewatering will be required. iv. Disturbance of streambed v. Additional lateral resistance using dowels may be required for the footing option. **FEASIBLE**

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Appendix E

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

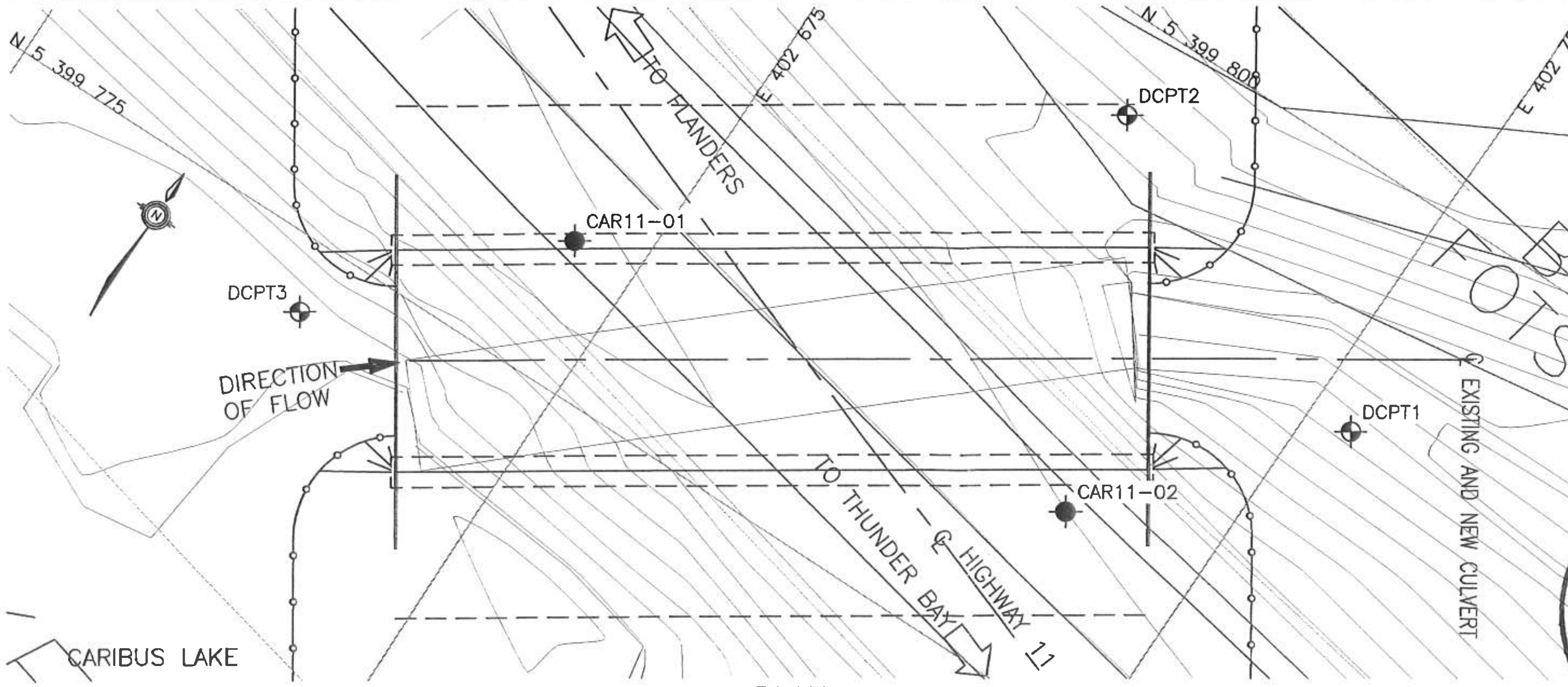
- OPSS 501 dated November 2010
- OPSS 804, November 2010
- OPSS 902, November 2010
- OPSS 539

2. Suggested Text for NSSP on Dewatering

The soils underlying this site are cohesionless in nature and the observed groundwater table is approximately 2.8 m below ground surface. Excavation below the groundwater level is expected to lead to instability and slough of the sides of the excavation and boiling of the base. If excavation is required to be carried out below the groundwater level prevailing at the time of construction, appropriate means of dewatering must be implemented to depress the groundwater level to prevent any instability, sloughing, or boiling and so as to preserve the stability of the excavation and to allow the work to proceed in the dry.

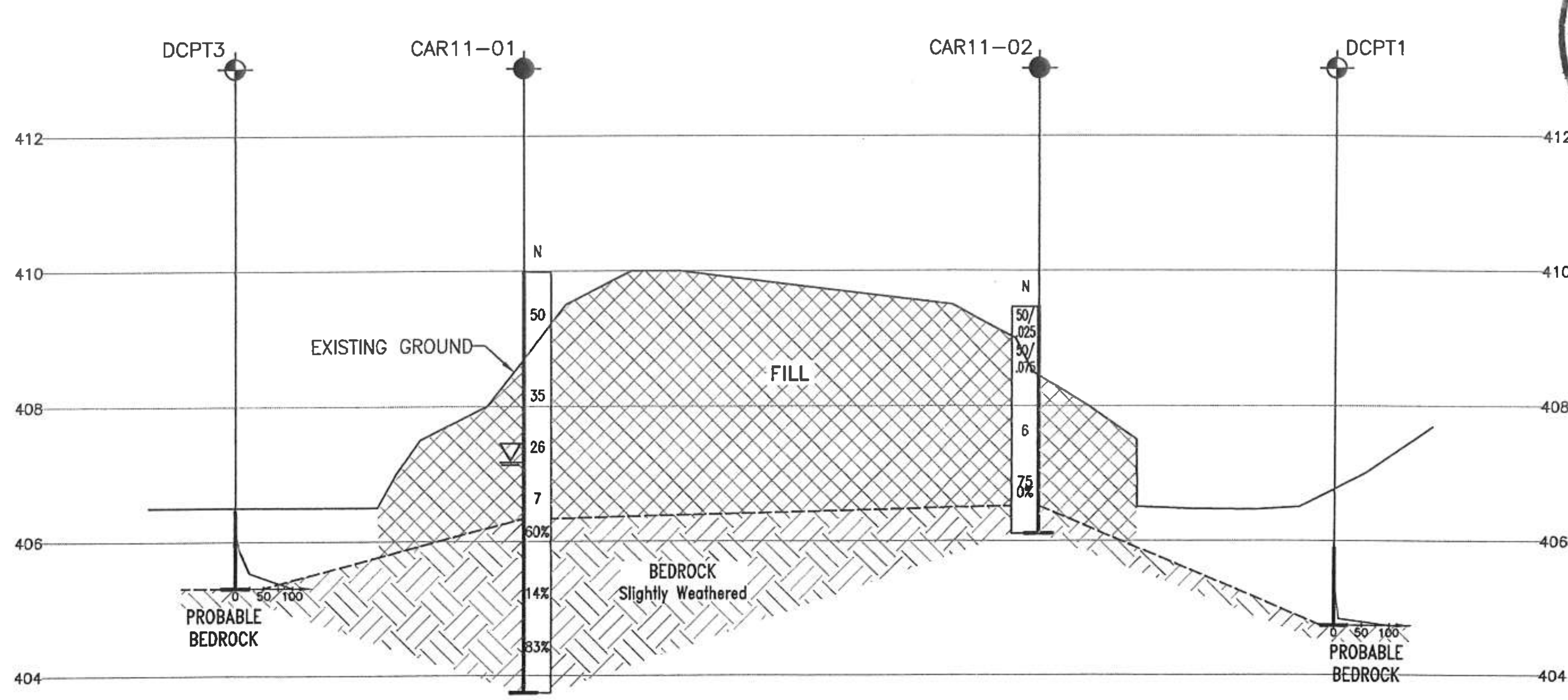
Appendix F

Drawing
Borehole Locations and Soil Strata



PLAN

SCALE 1:250



PROFILE

H 1:250

V 1:100

METRIC

DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

CONT No

GWP No 6930-10-00

SHEET

HIGHWAY 11
CARIBUS LAKE CULVERT
STRUCTURAL REPLACEMENT
BOREHOLE LOCATIONS AND SOIL STRATA

Hatch Mott MacDonald

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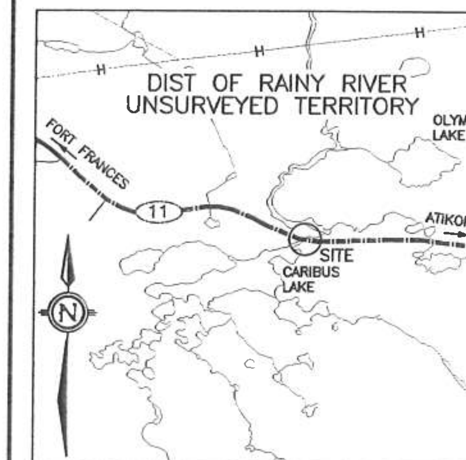

KEYPLAN

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
CAR11-01	410.0	5 399 780.5	402 672.3
CAR11-02	409.5	5 399 782.1	402 694.3
DCPT1	405.9	5 399 790.8	402 701.8
DCPT2	408.1	5 399 796.5	402 687.6
DCPT3	406.4	5 399 772.3	402 664.8

-NOTES-

1) The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.

2) This drawing is for subsurface information only. Surface details and features are for conceptual illustration.

GEOCRES No. 52-B13

REVISIONS	DATE	BY	DESCRIPTION		
DESIGN TJH	CHK	CODE	LOAD	DATE NOV. 2011	
DRAWN MFA	CHK AEG	SITE	STRUCT	DWG 2	

FILENAME: H:\Drafting\19\1605\121\ted5121-Bridge&Culvert\Boreholes.dwg
PLOTDATE: 11/28/2011 2:00 PM