



THURBER ENGINEERING LTD.

**FINAL
FOUNDATION INVESTIGATION AND DESIGN REPORT
CULVERT REHABILITATION, STRUCTURE NO. 40-117/C
HIGHWAY 35 MINERS BAY CULVERT, LUTTERWORTH TOWNSHIP
AGREEMENT NO. 5015-E-0043**

G.W.P. 5087-11-00

Geocres No.: 31D-691

Report to:

McIntosh Perry Consulting Engineers Limited

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PART 1. FACTUAL INFORMATION

1 INTRODUCTION

This section of the report presents the factual findings obtained from a foundation investigation completed for the proposed rehabilitation of the Miners Bay culvert crossing of Highway 35. The culvert is located approximately 3.5 km north of Halliburton Road 2 (Deep Bay Road) within Lutterworth Township. Thurber Engineering Limited (Thurber) carried out the investigation as a sub-consultant to McIntosh Perry Consulting Engineers Ltd. (MPCE) under Agreement No. 5015-E-0043.

The purpose of this investigation was to explore the subsurface conditions at the site and, based on the data obtained, to provide a borehole location plan, records of boreholes, stratigraphic profile, laboratory test results and a written description of the subsurface conditions. A model of the subsurface conditions influencing design and construction was developed in the course of the current investigation. No previous foundation investigation information was available for the subject culvert site within the online Geocres Library.

2 SITE DESCRIPTION

Highway 35 is a north-south highway although at the location of the culvert the true alignment of Highway 35 is east-west. Based on the overall alignment of the highway, the highway will be described as north-south and the culvert will be described as oriented east (inlet) to west (outlet). The culvert is described within the RFP is an open footing culvert with a total length of 24 m. The east section of the culvert is noted to be a reinforced concrete rigid frame open footing culvert constructed in 1969; the length is 17 m, the span is 3.0 m and the height is 2.75 m. The east middle section of the culvert is noted to be a reinforced concrete non-rigid frame open footing culvert of unknown age. The west middle section is noted to be a stone masonry wall section with reinforced concrete top slab. The west end of the culvert is noted to be a reinforced concrete non-rigid frame open footing culvert with walls and top slab of different construction. The span for the three western sections is approximately 3.0 metres, the height is 3.1 m and the length is approximately 7 metres. Wingwalls are present at both ends of the culvert on the north and south sides of the culvert. The stream bed elevations at the culvert inlet and outlet are approximately 272.4 m and 271.2 m respectively. It was observed that the culvert is not in a straight alignment between the inlet and outlet.

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At the location of the culvert, Highway 35 is a two-lane highway with paved shoulders and a concrete curb at the west edge of pavement. Based on the RFP, there is approximately 0.6 m to 1.0 m of fill on the culvert with the centerline of Highway 35 having approximate elevation 276.2 m at the culvert. The embankment is retained by wingwalls at both culvert ends. Erosion was noted adjacent to the north east wingwall along Clear Lake Road at the time of the field investigation. Steel guiderails are present on both sides of the highway at the culvert. The land adjacent to the east side of the highway is developed with houses and the Miner's Bay Lodge. The west side of the highway is vegetated with trees and partially developed with houses. Bedrock outcrops are noted within close proximity to the culvert. Traffic volumes are understood to be 3150 AADT (2013).

Select photographs showing the existing conditions in the area of the culvert are included in Appendix D for reference.

3 SITE INVESTIGATION AND FIELD TESTING

Thurber contacted Ontario One Call in advance of the field investigation to obtain utility locate clearances in the vicinity of the proposed boreholes.

The site investigation and field testing program was carried out on May 8th, 2017 for the on-road drilling and August 10th, 2017 for the off-road drilling. The northing, easting and elevation of the boreholes are shown on the Borehole Location and Soil Strata Drawing No. 1 in Appendix A and are summarized in Table 3-1. The site is within MTM Zone 10.

Table 3-1: Borehole Summary

Borehole No.	Drilled Location	Northing (m)	Easting (m)	Ground Surface Elevation (m)	Borehole Termination Depth below Existing Ground Surface (m)
17-1	North of culvert – southbound lane	4 964 686.0	362 106.3	276.3	7.1
17-2	South of culvert – northbound lane	4 964 681.2	362 098.3	276.2	7.3
17-3	East end – culvert inlet	4 964 670.8	362 101.9	272.4	0.4
17-4	West end – culvert outlet	4 964 695.8	362 094.4	271.2	0.3

The drilling was carried out using portable equipment for off road boreholes 17-3 and 17-4, and a truck mounted drill rig equipped with NW casing for the on-road boreholes 17-1 and 17-2. The subsurface stratigraphy encountered in the boreholes was recorded in the field by Thurber personnel. Split spoon samples were collected at regular depth intervals in the boreholes during the completion of Standard Penetration Tests (SPT). Bedrock was cored and collected using NQ coring equipment in on-road boreholes 17-1 and 17-2. All soil and

rock core samples recovered from the boreholes were transported to Thurber's Ottawa geotechnical laboratory for further examination and testing.

All boreholes were backfilled with a low-permeability mixture of auger cuttings and bentonite pellets in accordance with Ontario MOE Regulation 903. Boreholes advanced within paved areas were capped with granular fill followed by 80 mm of cold patch asphalt to reinstate the travelling surface.

The approximate as-drilled locations and ground surface elevation of the boreholes were surveyed by Thurber in August 2017.

3.1 Laboratory Testing

Geotechnical laboratory testing consisted of natural moisture content determination and visual identification of all retained soil samples in accordance with the current MTO standards. Grain size distribution analyses testing was also carried out on selected samples to MTO and ASTM standards. All rock cores were photographed and their total core recovery (TCR), solid core recovery (SCR) and rock quality designation (RQD) were determined. Chemical analysis for determination of pH, conductivity, resistivity, soluble sulphate and chloride concentrations was carried out on one soil sample.

The results of the geotechnical tests are summarized on the Record of Borehole sheets included in Appendix B and all laboratory results are presented on the figures included in Appendix C.

4 DESCRIPTION OF SUBSURFACE CONDITIONS

4.1 Overview / General

Reference is made to the Record of Borehole sheets in Appendix B for details of the soil stratigraphy encountered in the boreholes. A stratigraphic profile is presented on Drawing No. 1 in Appendix A for illustrative purposes. An overall description of the stratigraphy is given in the following paragraphs; however, the factual data presented in the Record of Boreholes governs any interpretation of the site conditions. It must be recognized that soil and groundwater conditions may vary between and beyond sampled locations.

The stratigraphy encountered in the boreholes through the embankment near the culvert is generally characterized by the asphalt pavement structure underlain by granular fill embankment overlying bedrock.

4.2 Asphalt

Boreholes 17-1 and 17-2 were drilled through the existing Highway 35 lanes and encountered asphalt with a thickness of 80 mm.

4.3 Embankment Fill

Below the asphalt, within on-road Boreholes 17-1 and 17-2, was a layer of cohesionless embankment fill consisting of sand with silt and gravel to gravel with sand. Cobbles and boulders were noted within the embankment fill. The underside of the embankment fill was at a depth of 3.6 to 4.0 m (elev. 272.7 to 272.2 m) below the existing roadway surface.

SPT tests conducted in the fill gave N-values typically ranging from 11 to 88 blows indicating a relative density of compact to very dense. The higher N-values could be indicative of the presence of cobbles and boulders. Moisture contents ranged from 2 to 16%.

Gradation analyses were completed on three samples of the embankment fill layer. The grain size distribution curves for these samples are included in Figure C1 of Appendix C. The results of the tests are summarized below and are presented on the corresponding Record of Borehole sheets in Appendix B.

Table 4-1: Gradation Results for Embankment Fill

Soil Particle	Percentage (%)	
	Sand with silt and gravel	Gravel with sand
Gravel	27 - 36	62
Sand	54 - 67	32
Silt and Clay	6 - 10	6

4.4 Sand with Gravel to Gravel with Sand

A sand with gravel to gravel with sand layer was encountered from the surface of off-road Boreholes 17-3 and 17-4. Frequent cobbles and boulders were noted within the creek bed in close proximity to both boreholes and may have been reflected in SPT N-values exceeding 100 blows per 0.3 m. The thickness of this layer ranged from 0.3 to 0.4 m and was underlain by inferred bedrock at elevation 270.9 m to 272.0 m.

The recorded moisture contents ranged from 9 to 14%.

A gradation analysis was completed on a sample of the sand with gravel layer. The grain size distribution curve is included in Figure C2 of Appendix C. The results of the tests are summarized below and are presented on the corresponding Record of Borehole sheet in Appendix B.

Table 4-2: Gradation Results for Sand with Gravel

Soil Particle	Percentage (%)
Gravel	40
Sand	52
Silt and Clay	8

4.5 Bedrock

Bedrock was encountered below the fill in both of the on-road boreholes and was proven with coring techniques. The bedrock transitioned from quartzite to granite in both boreholes. The inlet and outlet boreholes were terminated on SPT refusal on inferred bedrock. The bedrock and inferred bedrock surface ranges from elevation 270.9 to 272.7 m and is summarized in the table below:

Table 4-3: Summary of Bedrock Elevation

Location	Borehole No.	Depth Below Existing Ground Surface (m)	Top of Bedrock Elevation (m)
North of culvert – southbound lane	17-1	3.6	272.7
South of culvert – northbound lane	17-2	4.0	272.2
East end – culvert inlet	17-3	0.4	272.0 ^(*)
West end – culvert outlet	17-4	0.3	270.9 ^(*)

^(*) – Inferred by SPT refusal

The Total Core Recovery (TCR) was 100%, the Solid Core Recovery (SCR) ranged from 92 to 100% and the Rock Quality Designation (RQD) ranged from 75 to 100%. Based on the RQD values the bedrock is classified as good to excellent quality. The granite bedrock is estimated to be strong to very strong.

4.6 Groundwater

The groundwater level was not measured in the embankment boreholes since water was introduced in the hole for coring operations. The water level in the creek was observed at 272.7 m and 271.2 m at the culvert inlet and outlet respectively on August 10th, 2017. It is expected that, based on the shallow bedrock surface encountered at this site, the groundwater level will largely be controlled by the water level in the creek. It should be noted that significant differences in the creek water level and flow rates were noted between the May and August field investigations. As the creek bed is bedrock, it should be expected that relatively quick changes in water level could occur.

It should be noted that the groundwater level at the time of construction may be higher and seasonal fluctuations of the groundwater level are to be expected. In particular, the groundwater level may be at a higher elevation after periods of significant and/or prolonged precipitation events.

4.7 Analytical Testing

One sample of the native soil was submitted to Paracel Laboratories in Ottawa, Ontario for analysis of pH, water soluble sulphate and chloride concentrations, resistivity and conductivity. The analysis results are provided in Appendix C and are summarized in the table below:

Table 4-4: Analytical Results Summary

Borehole	Sample	Depth (m)	Sulphate (µg/g)	pH	Resistivity (Ohm-cm)	Conductivity (µS/cm)	Chloride (µg/g)
17-3	SS1	0 – 0.4	23	7.85	4550	220	8

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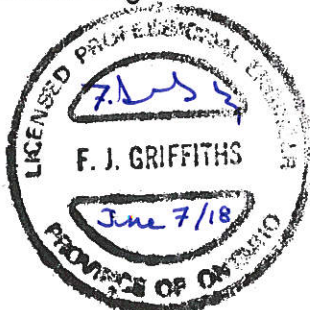
Borehole locations were selected by Thurber relative to the existing culvert alignment and the existing site features. The as-drilled locations and ground surface elevations for the on-road and off-road boreholes were surveyed by Thurber.

George Downing Estate Drilling Ltd. of Hawkesbury, Ontario and Forage M3 Drilling also of Hawkesbury, Ontario supplied and operated the drilling equipment to carry out the drilling, soil sampling, in-situ testing and borehole decommissioning. The field investigation was supervised on a full-time basis by Mr. Jeff Morrison, E.I.T. and Mr. Christopher Murray, P.Eng., of Thurber. Overall supervision of the investigation program was conducted by Mr. Stephen Peters, P.Eng.

Routine geotechnical laboratory testing was completed by Thurber's laboratory in Ottawa, Ontario. Analytical testing was completed by Paracel Laboratories in Ottawa, Ontario. Interpretation of the factual data and preparation of this report was carried out by Mr. Christopher Murray, M.Sc., P.Eng. and Dr. Fred Griffiths, P.Eng. The report was reviewed by Dr. P.K. Chatterji, P.Eng. a Designated Principal Contact for MTO Foundation Projects.



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PART 2. ENGINEERING DISCUSSION AND RECOMMENDATIONS

6 INTRODUCTION

This section of the report provides an interpretation of the factual data from Part 1 of this report and presents geotechnical recommendations to assist the design team in designing a suitable foundation for the proposed rehabilitation and partial replacement of the existing Miners Bay culvert crossing Highway 35. The discussion and recommendations presented in this report are based on the information provided by MPCE and on the factual data obtained during the course of the investigation.

This foundation investigation and design report with the interpretation and recommendations are intended for the use of the Ministry of Transportation, and shall not be used or relied upon for any other purposes or by any other parties including the construction or design-build contractor. The construction or design-build contractor must make their own interpretation based on the factual data in Part 1 of the report. Where comments are made on construction, they are provided only to highlight those aspects which could affect the design of the project. Contractors must make their own interpretation of the factual information provided as it may affect equipment selection, proposed construction methods and scheduling.

6.1 Proposed Structure

The proposed rehabilitation of the Miners Bay Culvert is indicated on the May 28th, 2018 60% Contract Drawing Package. The culvert rehabilitation includes the removal of an approximately 10.1 m length of the existing culvert at the outlet and reconstruction of a shorter 7.8 m culvert length on the same alignment (25 degree skew with the highway centerline). The section of the culvert being removed is understood to be constructed prior to 1969. The proposed culvert replacement section will have a span and height in the order of 3.0 and 3.65 m, respectively. The existing outlet wingwalls will also be removed and replaced as part of the rehabilitation. The new wingwalls are shown to be 2.5 and 5.0 m long with a skew of 30 and 65 degrees from the culvert alignment, respectively.

The proposed rehabilitation of the existing 14.6 m long culvert section and inlet wingwalls, that are all to remain in place, includes the repair of deteriorated concrete sections and infilling of cracks within the culvert top slab and walls.

The stream bed elevation of 272.7 m and 271.1 m at the culvert inlet and outlet, respectively, are understood to remain unchanged.

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6.2 Applicable Codes and Design Considerations

The geotechnical assessment presented below has been prepared based on the available data regarding the proposed foundations and existing ground conditions and in accordance with the Canadian Highway Bridge Design Code (CHBDC), version CSA S6-14.

It is understood that the proposed culvert structure has a consequence classification of *Typical Consequence*, in accordance with Section 6.5.1 of the CHBDC. Accordingly, a consequence factor (Ψ) of 1.0, as per Table 6.1 of the CHBDC, has been used in assessing factored geotechnical resistances. If the consequence classification changes, the geotechnical assessment will need to be reviewed and revised.

The frost penetration depth and associated recommendations are provided in Section 9.4.

7 SEISMIC CONSIDERATIONS

7.1 Spectral and Peak Acceleration Hazard Values

The seismic hazard data for the CHBDC is based on the fifth-generation seismic model developed by the Geological Survey of Canada (GSC). The seismic hazard for this site has been obtained from the GSC calculator. The data includes a peak ground acceleration (PGA), peak ground velocity (PGV) and the 5% spectral response acceleration values ($S_a(T)$) for the reference ground condition (Site Class C) for a range of periods (T) and for a range of return periods including 475-year, 975-year and 2475-year events. The GSC seismic hazard calculated data sheet for this site is included in Appendix E.

The site coefficients used to determine the design spectral acceleration and displacement values are a function of the Site Class and the peak ground acceleration (PGA), which is 0.071g at this site.

7.2 CHBDC Seismic Site Classification

In accordance with the CHBDC, the selection of the seismic site classification is based on the soil conditions encountered in the upper 30 m of the stratigraphy.

Based on the bedrock encountered at the culvert foundation elevation, the site has been classified as a Site Class B in accordance with Section 4.4.3.2 of the CHBDC (S6-14).

7.3 Seismic Liquefaction

Based on the shallow bedrock encountered in the drilled locations at this site, liquefaction beneath the foundations during a seismic event is not considered to be an issue.

8 DESIGN OPTIONS

8.1 Culvert Type

Since only the west 7.8 m of the culvert outlet is to be replaced, the culvert type must consider the connection with the section of the culvert that is to remain in place in addition to construction procedures, staging requirement and geotechnical resistance available. Based on these criteria, the most applicable replacement option is an open bottom culvert on footings constructed with similar geometry to the existing section of the culvert that is to

remain in place. For continuity of the culvert and ease of culvert connections, pipe and closed box culverts are not recommended.

8.2 Construction Methodology Alternatives

For the proposed culvert replacement, the following construction methods were considered.

- Open Cut with Full Road Closure and Temporary Detour
Installation of a new culvert using open cut techniques and a full road closure would allow for an expedited construction schedule and could reduce costs associated with requiring roadway protection. However, it is understood that an acceptable detour route is not available and therefore this option is not considered feasible.
- Open Cut with Temporary Widening
Widening of the existing highway and/or construction of a temporary detour embankment to the east to accommodate traffic passage during construction is considered feasible from a geotechnical perspective. However, a review of the requirement for property acquisition and alteration to highway geometry, intersection and entrance is also needed to assess this option.
- Open Cut with Temporary Protection System
The use of open cut techniques in conjunction with staged construction is a potentially feasible option from a geotechnical perspective. This option may require roadway protection, as discussed further in Section 10.2.

8.3 Recommended Approach for the Culvert Replacement

From a foundation engineering perspective, replacing the western section of the existing culvert using open cut techniques with staged construction is the recommended culvert replacement option. Temporary protection systems (TPS) will likely be needed to facilitate construction. It is understood that the platform width is wide enough that, depending on the construction stage, 1 or 2 lanes of traffic can be maintained during construction by providing a shift in the lanes without requiring an embankment widening.

9 FOUNDATION DESIGN RECOMMENDATIONS

Foundation design aspects for the partial culvert replacement includes subgrade conditions, geotechnical resistances, imposed loading pressures, erosion control, protection system design, groundwater control and stability of staged construction. The culvert must be designed to resist loading including, but not limited to, lateral earth pressures, hydrostatic pressure, weight of embankment fill, traffic loading and any surcharge due to construction equipment and activities under static and seismic conditions.

9.1 Spread Footings

The geotechnical bearing resistances provided in this report for spread footings include a resistance factor of 0.5 (ϕ_{gu}) and 0.8 (ϕ_{gs}) for the ULS and SLS values, respectively, as per Table 6.2 of the CHBDC (static analysis – typical understanding). The geotechnical resistances presented in the following subsections are for vertical concentric loading only

and will need to be adjusted for the effects of inclined or eccentric loadings, where applicable, in accordance with CHBDC Clause 6.10.3 and 6.10.4.

9.1.1 Open Bottom Culvert

The elevation of bedrock in the boreholes advanced at the site was noted to range from 272.7 m to 270.9 m in elevation. The existing overburden should be excavated and the cast-in-place culvert footing should be founded directly on sound bedrock. Where bedrock is exposed it should be inspected and loose pieces should be removed. The bedrock should be excavated to create a horizontal surface or alternatively, the founding elevation can be raised with the use of a concrete leveling course to reduce the excavation and dewatering efforts.

Culvert footings with a minimum width of 0.3 m founded on the bedrock can be designed with a factored geotechnical resistance at ULS of 1500 kPa. SLS will not govern design for a footing founded on bedrock. The horizontal resistance against sliding between cast-in-place concrete footings founded on bedrock can be computed using a friction factor of 0.70. Appropriate resistance factors should be applied for the design. If greater lateral resistance is required, rock anchors/dowels could be utilized (see Section 9.2).

9.1.2 Wingwalls

In conjunction with replacement of the outlet of the culvert, the outlet wingwalls will also be replaced. The design recommendation for wingwall footings founded on bedrock will be the same as those for the culvert foundations provided above.

9.2 Rock Anchors / Dowels

It is understood that vertical rock anchors and/or dowels may be required to provide additional capacity for the culvert replacement and wingwalls. Rock anchors grouted into the underlying bedrock are considered to be feasible at this site to provide additional vertical and/or lateral resistance. However, the additional vertical loading from the pre-tensioned rock anchors will need to be incorporated into the design of forces acting on the foundation. All overburden must be removed from above the bedrock surface. Resistance from weathered/fractured bedrock should be ignored and not included in the calculation of available anchor capacity. Based on a minimum grout strength of 30 MPa, a rock anchor or dowel installed within sound bedrock can be designed with an ultimate bond stress of 1000 kPa. A geotechnical resistance factor of 0.4 (ϕ_{gu}) as per Table 6.2 of the CHBDC (static analysis – typical understanding) for ground anchor pull-out is to be applied to the calculated value. The lower of the grout to anchor bond and grout to bedrock bond should be used in design. A minimum rock anchor length of 3 m into sound bedrock should be used in design irrespective of the calculated capacity. Rock anchor design, installation and proof testing should be in conformation with OPSS 942. Rock anchors should be provided with double corrosion protection.

A check should be completed to verify the calculated bond strength does not exceed the effective unit weight of rock encompassed within an inverted cone inclined at 45 degrees from vertical acting from the base of the bonded length of the anchor to the surface of the sound rock. Additionally, individual rock anchor capacity should be reviewed and reduced taking into consideration the proximity of other structural and foundation elements that encroach within the circumference of the inverted cone.

The lateral resistance for dowels socketed into sound bedrock with full contact can be assessed based on the following expression, $K_s = k_s * L$, where k_s is the coefficient of horizontal subgrade reaction (MN/m/m) and L is the length (m) of the dowel segment used in the analysis. The ultimate lateral capacity, P_{ult} , on any one segment of the dowel may be obtained from the following expression, $P_{ult} = p_{ult} * L$. This represents the ultimate load at which the dowel fails and will not support any additional load at greater displacement. For intact bedrock, a k_s value of 1,500 MN/m/m is recommended for analysis. The k_s value may be assumed to be constant with depth. A minimum dowel length of 1.5 m into sound bedrock should be used. An unfactored p_{ult} of 2.0 MN per dowel is recommended. A geotechnical resistance factor of 0.5 (ϕ_{gu}) as per Table 6.2 of the CHBDC (static analysis - typical understanding) for lateral resistance of deep foundations is to be applied to the calculated value. Where the lateral spacing between an adjacent dowel or another structural element embedded into the rock is less than 4 equivalent diameters of either structural element, the coefficient of horizontal subgrade reaction will need to be reduced based on the center-to-center spacing. The reduction factors to be used are provided in Figure C6.11.3(r) of the CHBDC. An NSSP on the supply, installation and testing of rock dowels is provided in Appendix F.

The Contractor's drilling equipment must be able to penetrate into the sound bedrock to achieve the design bond length. When installing the rock anchors/dowels, the pre-drilled holes shall be free of water, dust and debris prior to placement of the anchoring agent. The anchors shall be maintained in position during the setting of the anchoring agent and loss of anchoring agent from the holes shall be prevented.

9.3 Subgrade Preparation, Bedding and Backfilling

Subgrade preparation for the culvert replacement should include excavation and removal of the existing culvert and backfill materials. Care must be taken to avoid fracturing or disturbing the bedrock below existing and new footings. All loose bedrock, organics, existing fill, soft or loose deposits, disturbed soils, alluvial deposits and deleterious materials must be stripped from the footprint of the foundation to expose competent bedrock subgrade at or below the desired founding elevations.

The exposed bedrock subgrade must be inspected to confirm that the subgrade is suitable and uniformly competent. Mass concrete can be used to fill any uneven bedrock surfaces to achieve the design founding level for the culvert and/or wingwalls. The mass concrete should extend at least 300 mm beyond all edges of the footing.

It is noted that construction may extend below the creek elevation. Water diversion and dewatering will be required to prepare the subgrade in the dry. Please refer to Section 10.3 for additional comments on groundwater and surface water control.

It is recommended that culvert cover consist of free-draining, non-frost susceptible granular materials such as Granular A material meeting the requirements of OPSS.PROV 1010. The cover must be in accordance with OPSS 902.

Culvert backfill above the granular cover should be in accordance with OPSS 902 and consist of material meeting the requirements of OPSS Select Subgrade Material or better and should be compacted in regular lifts as per OPSS.PROV 501. Heavy compaction equipment, used adjacent to the structure, must be restricted in accordance with

OPSS.PROV 501. Care must be exercised when compacting the fill adjacent to and above the culvert in order not to damage the culvert.

9.4 Frost Depth

The depth of frost penetration at this site is 1.7 m however, foundations founded on bedrock are not required to be founded below frost depth. Frost taper treatment, if required, should be as per OPSD 803.010 and as directed within the Pavement Design Report.

9.5 Backfill and Earth Pressure

Lateral earth pressure provided in the equations in the sections below are based on the assumption that the backfill is fully drained so that there are no unbalanced hydrostatic pressures. If adequate drainage cannot be confirmed, the potential for buildup of hydrostatic pressures should be considered in design.

9.5.1 Static Lateral Earth Pressure Coefficients

Lateral earth pressures acting on structures should be computed in accordance with the CHBDC but under fully drained conditions are generally given by the following expression:

$$p_h = K * (\gamma h + q)$$

where:

p_h	=	horizontal pressure on the wall at depth h (kPa)
K	=	earth pressure coefficient (see table below)
γ	=	unit weight of retained soil (see table below), adjusted for water level
h	=	depth below top of fill where pressure is computed (m)
q	=	value of any surcharge (kPa)

A lateral earth pressure due to backfill compaction should be added to the calculated lateral earth pressure in accordance with Clause 6.12.3 of the CHBDC. Typical earth pressure coefficients for backfill are shown in Table 9-1.

Table 9-1. Earth Pressure Coefficients

Condition	Earth Pressure Coefficient (K)						
	OPSS Granular A or OPSS Granular B Type II $\phi = 35^\circ, \gamma = 22.8 \text{ kN/m}^3$			OPSS Granular B Type I $\phi = 32^\circ, \gamma = 21.2 \text{ kN/m}^3$		OPSS SSM and Existing Fill $\phi = 30^\circ, \gamma = 21.0 \text{ kN/m}^3$	
	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Sloping Surface Behind Wall (1.5H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)
Active, K_A (Yielding Wall)	0.27	0.40	0.53	0.31	0.48	0.33	0.54
At Rest, K_O (Non- Yielding Wall)	0.43	-	-	0.47	-	0.50	-
Passive, K_P (Movement towards Soil Mass)	3.7	-	-	3.3	-	3.0	-
CHBDC Soil Group ^(*)	"medium dense sand"			"loose to medium dense sand"		"loose sand"	

Note: (*) Figure C6.16 of the Commentary to the CHBDC.

The use of a material with a high friction angle and low active pressure coefficient (Granular A or Granular B Type II) is preferred as it results in lower earth pressures acting on the culvert.

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 C6.16 of the Commentary to the CHBDC using the soil group designation as outlined in Table 9-1. Active earth pressures should be used for any head walls or unrestrained walls. For rigid structures such as a concrete box culvert, it is recommended that at-rest horizontal earth pressures be used for design. Where ground surfaces are sloped behind the walls, the corresponding coefficients provided in the Table 9-1 should be used.

The culvert must be designed to withstand full hydrostatic pressure assuming a water level at least equal to the design creek water level. This is applicable when the water level behind the culvert is higher than the creek level.

9.5.2 Combined Static and Seismic Lateral Earth Pressure Parameters

In accordance with Clause 4.6.5 of the CHBDC (S6-14), retaining structures should be designed using dynamic earth pressure coefficients that incorporate the effects of earthquake loading. The following recommendations are per Section C4.6.5 of the Commentary of the CHBDC which states that seismically induced lateral soil pressures may be calculated using the Mononobe-Okabe Method with:

- $k_h = \frac{1}{2} * F(\text{PGA}) * \text{PGA}$, for structures that allow 25 to 50 mm of movement, and

- $k_h = F(PGA) * PGA$, for non-yielding walls

The ratio of wall movement to wall height required to mobilize the active conditions would be approximately 0.002 for a yielding structure with respect to the assessment of seismically induced lateral earth pressures.

The coefficients of horizontal earth pressure for seismic loading presented in Table 9-2 may be used. The earth pressure coefficients provided below are based on a Seismic Site Class B, PGA with a 2% probability of exceedance in 50 years of 0.071g (Geological Survey of Canada – Fifth Generation) and a $F(PGA)$ of 0.87 as per Table 4.8 of the CHBDC (S6-14 update No. 2, July 2017).

Table 9-2. Dynamic Earth Pressure Coefficients

Condition	Earth Pressure Coefficient (K)				
	OPSS Granular A or OPSS Granular B Type II $\phi = 35^\circ, \gamma = 22.8 \text{ kN/m}^3$			OPSS Granular B Type I $\phi = 32^\circ, \gamma = 21.2 \text{ kN/m}^3$	
	Horizontal Surface Behind Wall	Slope Surface Behind Wall (2H:1V)	Slope Surface Behind Wall (1.5H:1V)	Horizontal Surface Behind Wall	Slope Surface Behind Wall (1.5H:1V)
Active, K_{AE} Yielding Wall	0.29	0.43	0.51	0.32	0.52
Active, K_{AE} Non-Yielding Wall	0.30	0.48	0.55	0.34	0.60

The total pressure due to combined static and seismic loads acting at a specific depth below the top of the wall may be determined using the following equation that includes consideration of material properties and the soils profile.

$$\sigma_h = K * \gamma * d + (K_{AE} - K_A) * \gamma * (H - d)$$

where:

$$\begin{aligned} \sigma_h &= \text{lateral earth pressure at depth } d \text{ (kPa)} \\ d &= \text{depth below the top of the wall (m)} \\ K &= \text{static earth pressure coefficient} \\ &\quad (K_A \text{ for yielding walls, } K_o \text{ for non-yielding walls)} \\ \gamma &= \text{unit weight of retained soil, adjusted for water level} \\ K_{AE} &= \text{combined static and seismic earth pressure coefficient} \\ H &= \text{total height of the wall (m)} \end{aligned}$$

9.6 Embankment Design and Reinstatement

9.6.1 Embankment Reconstruction

Embankment reconstruction after culvert replacement should be carried out in accordance with OPSS.PROV 206. The embankment should be reinstated with side slopes of 2H:1V (or flatter) if constructed using Select Subgrade Material (SSM) or Granular B Type I. It is

understood that embankment slopes as steep as 1.5H:1V are proposed behind the new outlet wingwalls based on geometric constraints with property boundaries. The existing embankment slope behind the new wingwall must be benched as per OPSD 208.010 with steps formed at an effective inclination of 2H:1V (or flatter) starting behind the backside of the wall footing. The embankment slope/wall backfill can then be reconstructed with OPSS Granular B Type II (or better). The fill should be placed and compacted in accordance with OPSS.PROV 501.

Where newly placed embankment fill is placed against existing embankment slopes or on a sloping ground surface steeper than 3H:1V adjacent to the structure, benching of the existing slope should be carried out in accordance with OPSD 208.010.

9.6.2 Embankment Settlement and Stability

Provided the subgrade is prepared as outlined above and construction of the embankment is carried out in accordance with recommendations provided within this report, the embankment side slopes should remain stable and long-term stability issues are not anticipated.

It is understood that no grade raise is anticipated along the alignment of Highway 35 and therefore negligible foundation settlement is expected to occur.

The magnitude of the embankment compression constructed with granular materials is in the order of 0.5% of the embankment height and is expected to occur following fill placement.

9.7 Cement Type and Corrosion Potential

Analytical tests were completed to determine the potential for degradation of the concrete in the presence of soluble sulphates and the potential for corrosion of exposed steel. The concentration of soluble sulphate provides an indication of the degree of sulphate attack that is expected for concrete in contact with soil and groundwater at the site. Soluble sulphate concentrations less than 1000 µg/g generally indicate a low degree of sulphate attack is expected for concrete in contact with soil and groundwater. The class of concrete selected should consider the effects of road de-icing salts.

The pH, resistivity and chloride concentration provide an indication of the degree of corrosiveness of the sub-surface environment. The test results provided in Section 4.7 may be used to aid in the selection of coatings and corrosion protection systems for buried steel objects. The corrosion effects of road de-icing salts should also be considered.

10 CONSTRUCTION CONSIDERATIONS

10.1 Excavation

Care must be taken during excavation to not undermine the existing footings of the culvert section that is to remain in place. All excavation must be carried out in accordance with the Occupational Health and Safety Act (OHSA). For the purposes of OHSA, the fills above the design creek water level may be classified as Type 3 soil. All soils below the design creek water level are considered to be Type 4 soils.

Excavation for the culvert replacement must be carried out in accordance with OPSS 902 and will be carried out through the existing embankment fill and extend into the underlying native deposits and bedrock. Selection of the equipment and methodology to excavate and prepare the founding surface is the responsibility of the Contractor. Stockpiling or surface surcharge should not be allowed within a horizontal distance encompassed within 1H:1V inclination from the perimeter of the base of the excavation.

At locations where there are space restrictions or where a slope has to be retained, the excavations will need to be carried out within a protection system. Further discussion on temporary protection systems (TPS) is presented in Section 10.2.

10.2 Temporary Protection Systems

Temporary Protection Systems may be required during construction and must be implemented in accordance with OPSS.PROV 539 and designed for Performance Level 2 (maximum 25 mm horizontal deflection). 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.

The design of roadway protection is the responsibility of the Contractor. All protection systems should be designed by a licensed Professional Engineer experienced in such designs and retained by the Contractor. The design of the roadway protection system must incorporate traffic loading and surcharge loading due to construction equipment and operations.

It is recommended that an NSSP be included in the tender documents to alert the Contractor to the potential for cobbles and boulders and obstructions within the fill and native soils as well as the presence of shallow bedrock. The suggested wording for this NSSP has been provided in Appendix F of this report. Drilled in soldier piles socketed in rock, deadman tie-backs, struts and/or raker supports may be needed to achieve the specified performance level. An alternative temporary protection system may be required to retain soil over the section of culvert that is to remain in place.

The protection system should be installed at a sufficient distance away from the new culvert to limit the disturbance to the culvert associated with removal of the protection system following completion of construction. Alternatively, the protection system near the culvert could be left in place and cut off in accordance with OPSS.PROV 539.

Lateral earth pressure coefficients, under fully mobilized conditions, that can be used in design of the protection system installed through the embankment fill and culvert backfill are provided in Table 9-1.

10.3 Dewatering

Creek diversion will be required to ensure that culvert construction is carried out in the dry. The depth of excavation will extend below the creek level observed at the time of the investigation. The Contractor must be prepared to control the groundwater and surface water flow at the site to permit construction in a dry and stable excavation. Water from surface flow and/or groundwater must be diverted away from any excavation at all times. Groundwater perched within the embankment fill and, surface runoff will tend to seep into, and accumulate in excavations.

The design of dewatering systems is the responsibility of the Contractor. The Contract Documents must alert the Contractor to this responsibility and to design the system in accordance with SP No. FOUN0003 which amends OPSS 902. A preconstruction survey is not required, thus Designer Fill-In ** in the SP should be "N/A".

In accordance with SP FOUN0003, the dewatering system is to be designed in accordance with OPSS.PROV 517 and SP517F01. It is recommended that the design Engineer and design-checking Engineer have a minimum of 5 years of experience in designing systems of similar nature and scope to the required work, thus Designer Fill-In ***** in SP517F01 should be "Yes". A preconstruction survey is not required, thus Designer Fill-In ***** in this SP should be "N/A".

The groundwater level will fluctuate and the minimum groundwater elevation for the site at the time of the proposed work should be taken as defined by SP517F01 and SP FOUN0003.

Temporary groundwater and surface water control measures will be required to remain operational during construction until the culvert section and wingwalls are installed and backfilled.

10.4 Scour Protection and Erosion Control

The Contractor should provide silt fences and erosion control blankets as per OPSS 805 throughout the duration of construction to prevent transport of silt/sediment. Slope protection and drainage measures will be required to ensure the long-term surficial stability of the embankment slopes. Surface water should be directed away from unprotected slopes.

Scour and erosion protection should be reinstated at the culvert outlet. The existing embankment material consists of sand with silt and gravel and rockfill and is considered to have a low erosion potential. However, slopes reinstates with 1.5H:1V inclination, to match existing slopes, are considered to have high erosion potential. Design of the scour and erosion protection measures must consider hydrologic and hydraulic concerns and should be carried out by specialists experienced in this field. Significant erosion was noted adjacent to the north east wingwall along Clear Lake Road (see Photo 1 in Appendix D) and should be repaired with rock protection in accordance with OPSS 511 as part of this contract.

Typically, rock protection should be provided over all earth surfaces subjected to flowing water in accordance with OPSS 511. Treatment at the outlet should be in accordance with OPSD 810.010. A vegetation cover should be established on all other exposed earth surfaces to protect against surficial erosion in general accordance with OPSS.PROV 804.

11 CONSTRUCTION CONCERNS

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

- Cobbles, boulders and rockfill or other buried obstructions will be encountered in the existing approach embankments and native soils. Bedrock is present at shallow depth, an NSSP should be included in the contract alerting the Contractor to these conditions. Driving of sheet piles will be difficult.

- Seasonal fluctuations of the groundwater and creek level are to be expected which may impact the construction.

The successful performance of the culvert will depend largely upon good workmanship and quality control during construction. Subgrade examination should be carried out by qualified geotechnical personal during construction in accordance with SP109S12 to confirm that foundation recommendations are correctly implemented, and material specifications are met.

12 CLOSURE

Engineering analysis and preparation of this report were carried out by Mr. Christopher Murray, M.Sc., P.Eng. and Dr. Fred Griffiths, P.Eng. The report was reviewed by Dr. P.K. Chatterji, P.Eng a Designated Principal Contact for MTO Foundation Projects.

Thurber Engineering Ltd.
Report Prepared By:



Christopher Murray, M.Sc., P.Eng.
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Senior Associate
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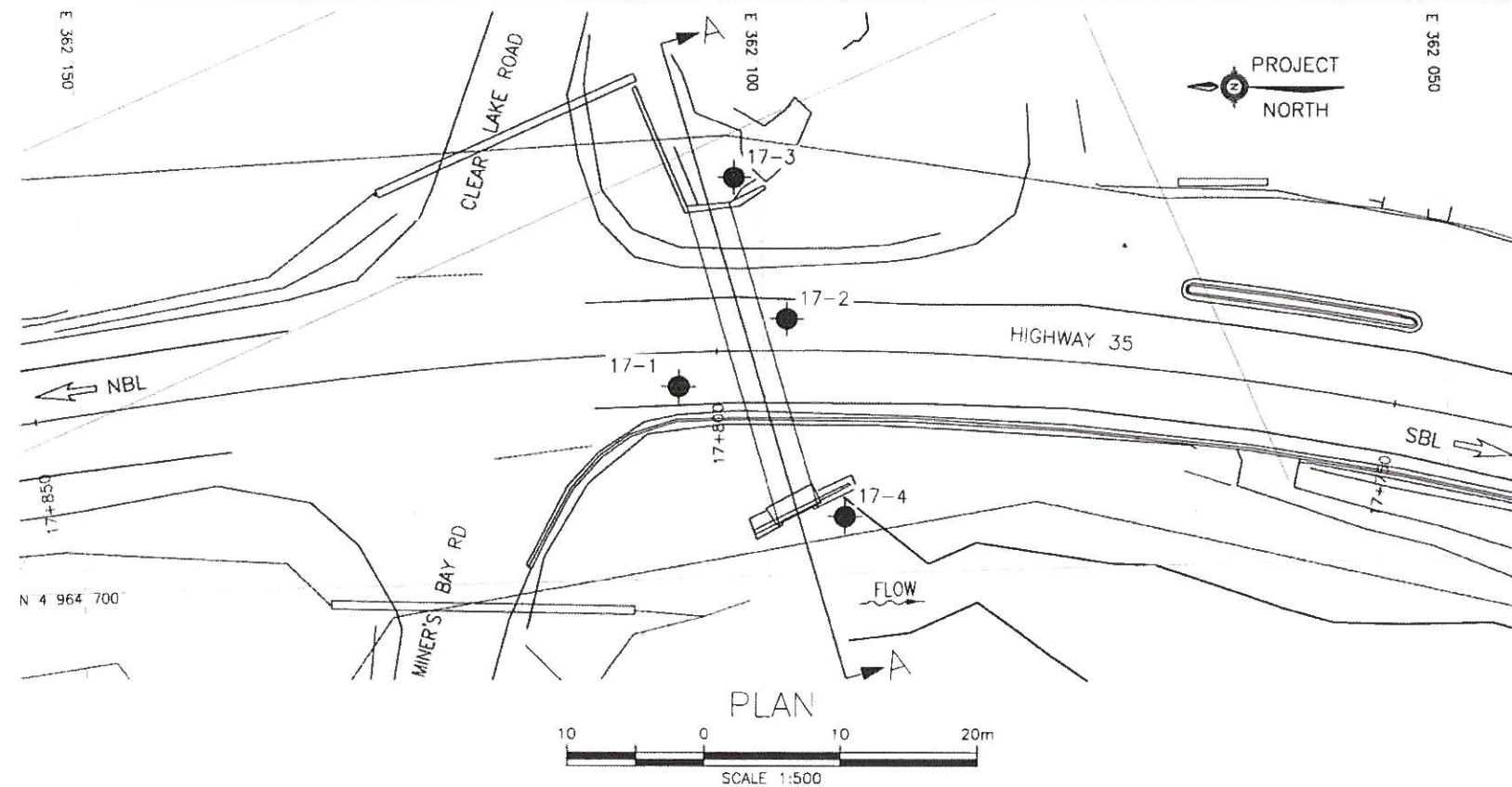


Dr. P.K. Chatterji, P.Eng.
Review Principal,
Senior Geotechnical Engineer

FINAL

Appendix A.

Borehole Location Plan and Stratigraphic Drawings



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

CONT No
GWP No 5087-11-00



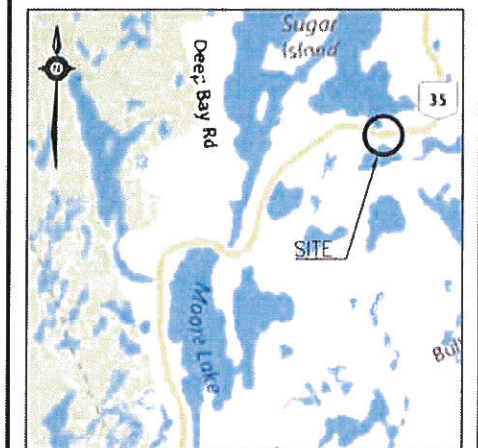
HIGHWAY 35
MINER'S BAY
CULVERT REHABILITATION
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET

**McINTOSH
PERRY**








THURBER ENGINEERING LTD.



KEYPLAN

LEGEND

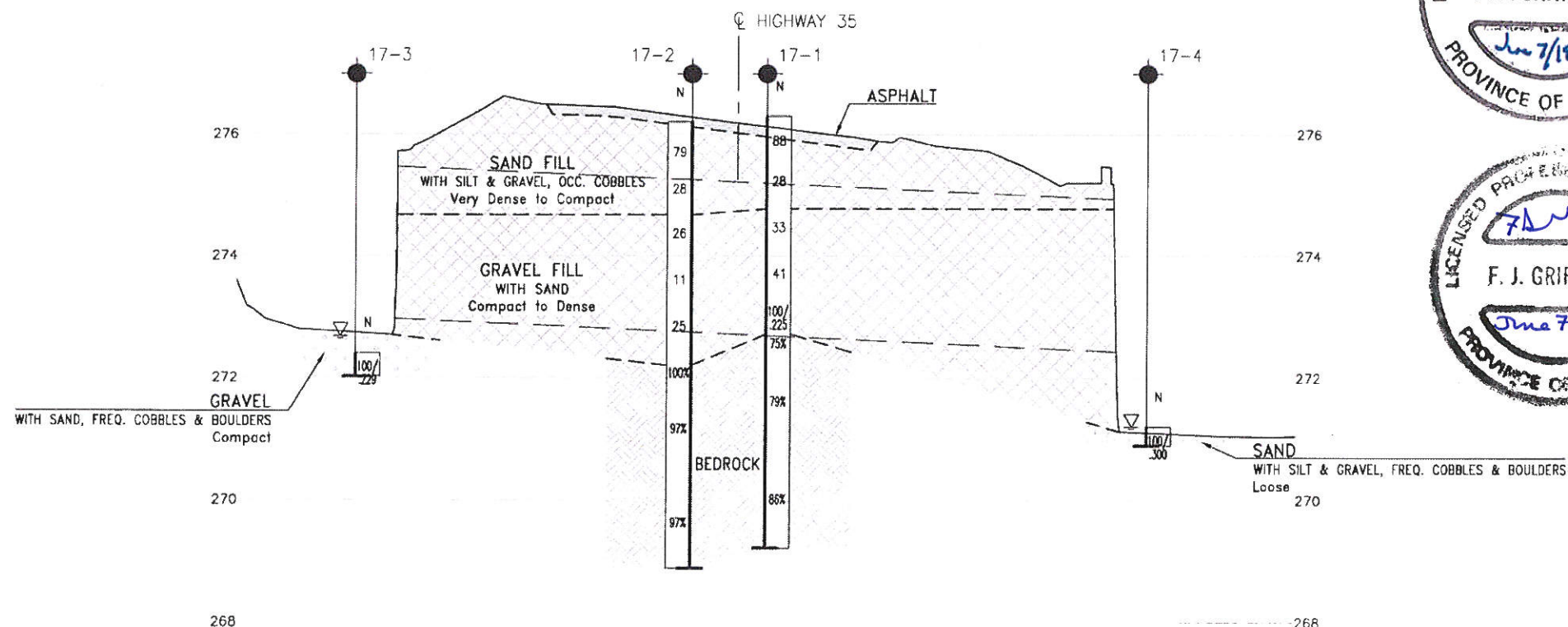
	Borehole
	Borehole and Cone
N	Blows /0.3m (Std Pen Test, 475J/blow)
CONE	Blows /0.3m (60° Cone, 475J/blow)
PH	Pressure, Hydraulic
	Water Level
	Head Artesian Water
	Piezometer
90%	Rock Quality Designation (RQD)
A/R	Auger Refusal

NO	ELEVATION	NORTHING	EASTING
17-1	276.3	4 964 686.0	362 106.3
17-2	276.2	4 964 681.2	362 098.3
17-3	272.4	4 964 670.8	362 101.9
17-4	271.2	4 964 695.8	362 094.4

-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. 31D-691



SECTION A-A



H 1:200

V 1:100



REVISIONS								
	DATE	BY	DESCRIPTION					
DESIGN	CM	CHK		CODE	LOAD	DATE	MAR 2018	
DRAWN	MFA	CHK	CM	SITE	STRUCT	DWG	1	

CULVERT REHABILITATION, STRUCTURE NO. 40-117/C
HIGHWAY 35 MINERS BAY CULVERT, LUTTERWORTH TOWNSHIP

Appendix B.

Record of Borehole Sheets



SYMBOLS, ABBREVIATIONS AND TERMS USED ON TEST HOLE RECORDS

TERMINOLOGY DESCRIBING COMMON SOIL GENESIS

Topsoil	mixture of soil and humus capable of supporting vegetative growth
Peat	mixture of fragments of decayed organic matter
Till	unstratified glacial deposit which may include particles ranging in sizes from clay to boulder
Fill	material below the surface identified as placed by humans (excluding buried services)

TERMINOLOGY DESCRIBING SOIL STRUCTURE:

Desiccated	having visible signs of weathering by oxidization of clay materials, shrinkage cracks, etc.
Fissured	having cracks, and hence a blocky structure
Varved	composed of alternating layers of silt and clay
Stratified	composed of alternating successions of different soil types, e.g. silt and sand
Layer	> 75 mm in thickness
Seam	2 mm to 75 mm in thickness
Parting	< 2 mm in thickness

RECOVERY:

For soil samples, the recovery is recorded as the length of the soil sample recovered.

N-VALUE:

Numbers in this column are the field results of the Standard Penetration Test: the number of blows of a 63.5 kg hammer falling 0.76 m, required to drive a 50 mm O.D. split spoon sampler 0.3 m into undisturbed soil. For samples where insufficient penetration was achieved and N-value cannot be presented, the number of blows are reported over the sampler penetration in millimetres (e.g. 50/75).

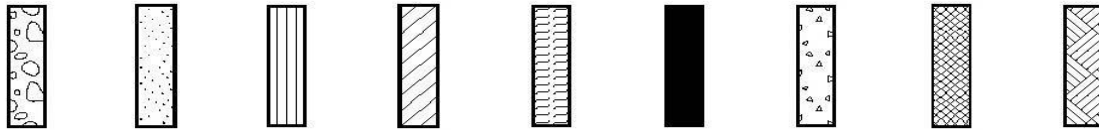
DYNAMIC CONE PENETRATION TEST (DCPT):

Dynamic cone penetration tests are performed using a standard 60 degree apex cone connected to an "A" size drill rods with the same standard fall height and weight as the Standard Penetration Test. The DCPT value is the number of blows of the hammer required to drive the cone 0.3 m into the soil. The DCPT is used as a probe to assess soil variability.



STRATA PLOT:

Strata plots symbolize the soil and bedrock description. They are combinations of the following basic symbols. The dimensions within the strata symbols are not indicative of the particle size, layer thickness, etc.



Boulders
Cobbles
Gravel Sand Silt Clay Organics Asphalt Concrete Fill Bedrock

TEXTURING CLASSIFICATION OF SOILS

Classification	Particle Size
Boulders	Greater than 200 mm
Cobbles	75 – 200 mm
Gravel	4.75 – 75 mm
Sand	0.075 – 4.75 mm
Silt	0.002 – 0.075 mm
Clay	Less than 0.002 mm

TERMS DESCRIBING CONSISTENCY (COHESIVE SOILS ONLY)

Descriptive Term	Undrained Shear Strength (kPa)
Very Soft	12 or less
Soft	12 – 25
Firm	25 – 50
Stiff	50 – 100
Very Stiff	100 – 200
Hard	Greater than 200

NOTE: Clay sensitivity is defined as the ratio of the undisturbed strength over the remolded strength.

SAMPLE TYPES

SS	Split spoon samples
ST	Shelby tube or thin wall tube
DP	Direct push sample
PS	Piston sample
BS	Bulk sample
WS	Wash sample
HQ, NQ, BQ etc.	Rock core sample obtained with the use of standard size diamond coring equipment

TERMS DESCRIBING CONSISTENCY (COHESIONLESS SOILS ONLY)

Descriptive Term	SPT "N" Value
Very Loose	Less than 4
Loose	4 – 10
Compact	10 – 30
Dense	30 – 50
Very Dense	Greater than 50

MODIFIED UNIFIED SOIL CLASSIFICATION

Major Divisions		Group Symbol	Typical Description
COARSE GRAINED SOIL	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	SILT AND CLAY SOILS $W_L < 35\%$	ML	Inorganic silts, 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.
		OL	Organic silts and organic silty-clays of low plasticity.
	SILT AND CLAY SOILS $35\% < W_L < 50\%$	MI	Inorganic compressible fine sandy silt with clay of medium plasticity, clayey silts.
		CI	Inorganic clays of medium plasticity, silty clays.
		OI	Organic silty clays of medium plasticity.
	SILT AND CLAY SOILS $W_L > 50\%$	MH	Inorganic silts, micaceous or diatomaceous fine sandy of silty soils, elastic silts.
		CH	Inorganic clays of high plasticity, fat clays.
		OH	Organic clays of high plasticity, organic silts.
HIGHLY ORGANIC SOILS		Pt	Peat and other organic soils.

Note - W_L = Liquid Limit



EXPLANATION OF ROCK LOGGING TERMS

ROCK WEATHERING CLASSIFICATION

Fresh (FR)	No visible signs of weathering.
Fresh Jointed (FJ)	Weathering limited to surface of major discontinuities.
Slightly Weathered (SW)	Penetrative weathering developed on open discontinuity surfaces, but only slight weathering of rock materials.
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 structures are preserved.

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 0.1 m in length or larger, as a percentage of total core length
Unconfined Compressive Strength: (UCS)	Axial stress required to break the specimen.
Fracture Index: (FI)	Frequency of natural fractures per 0.3 m of core run.

DISCONTINUITY SPACING

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

STRENGTH CLASSIFICATION

Rock Strength	Approximate Uniaxial Compressive Strength (MPa)
Extremely Strong	Greater than 250
Very Strong	100 – 250
Strong	50 – 100
Medium Strong	25 – 50
Weak	5 – 25
Very Weak	1 – 5
Extremely Weak	0.25 – 1

RECORD OF BOREHOLE No 17-1

1 OF 1

METRIC

GWP# 5087-11-00 LOCATION Miner's Bay Culvert, MTM z12: N 4 964 686.0 E 362 106.3 ORIGINATED BY JM
 HWY 35 BOREHOLE TYPE NW Casing / NQ Coring COMPILED BY JM
 DATUM Geodetic DATE 2017.08.05 - 2017.08.05 CHECKED BY CM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE									WATER CONTENT (%)
276.3								20	40	60	80	100					
0.0	80 mm ASPHALT																
0.2	SAND with Silt and Gravel, occasional Cobbles Very Dense to Compact Brown FILL		1	SS	88		276							○			
			2	SS	28									○			36 54 10 (SI+CL)
274.8							275										
1.5	GRAVEL with Sand Dense Grey FILL		3	SS	33									○			
			4	SS	41		274							○			
	-100 mm Cobble at 2.9 m		5	SS	100/ 225mm		273							○			
272.7																	RUN #1 TCR=100% SCR=92% RQD=75%
3.6	BEDROCK Quartzite to Granite Fresh to Slightly Weathered White to Grey		1	RUN			272										RUN #2 TCR=100% SCR=92% RQD=79%
			2	RUN			271										
			3	RUN			270										RUN #3 TCR=100% SCR=100% RQD=86%
269.2																	
7.1	End of Borehole																

ONTMT4S 16284 MINER'S BAY CULVERT.GPJ 2012TEMPLATE(MTO).GDT 16/3/18

RECORD OF BOREHOLE No 17-2

1 OF 1

METRIC

GWP# 5087-11-00 LOCATION Miner's Bay Culvert, MTM z12: N 4 964 681.2 E 362 098.3 ORIGINATED BY JM
 HWY 35 BOREHOLE TYPE NW Casing / NQ Coring COMPILED BY JM
 DATUM Geodetic DATE 2017.08.05 - 2017.08.05 CHECKED BY CM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					
								20 40 60 80 100					
								20 40 60 80 100					
276.2													
0.0	80 mm ASPHALT												
0.2	SAND with Silt and Gravel, occasional Cobbles Very Dense to Compact Brown FILL		1	SS	79								
			2	SS	28								
274.7													
1.5	GRAVEL with Sand Compact Grey FILL -75 mm Cobble at 2.2 m		3	SS	26								
			4	SS	11								
			5	SS	25								
	-200 mm Boulder at 3.7 m												
272.2													
4.0	BEDROCK Quartzite to Granite Fresh to Slightly Weathered White to Grey		1	RUN									
			2	RUN									
			3	RUN									
268.9													
7.3	End of Borehole												

ONTMT4S 16284 MINER'S BAY CULVERT.GPJ 2012TEMPLATE(MTO).GDT 16/3/18

RECORD OF BOREHOLE No 17-3

1 OF 1

METRIC

GWP# 5087-11-00 LOCATION Miner's Bay Culvert, MTM z12: N 4 964 670.8 E 362 101.9 ORIGINATED BY CM
 HWY 35 BOREHOLE TYPE Portable COMPILED BY KE
 DATUM Geodetic DATE 2017.10.08 - 2017.10.08 CHECKED BY CM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				
							20	40	60	80	100	W _p	W	W _L			
272.4																	
0.0																	
272.0	GRAVEL with Sand		1	SS	100/												
0.4	Frequent Cobbles and Boulders				229mm												
	Compact																
	Brown																
	End of Borehole on Probable Bedrock																
	Water 0.28 m above G.S. (Elev. 272.7																
	m) on 8/10/2017																

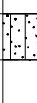
ONTMT4S 16284 MINER'S BAY CULVERT.GPJ 2012TEMPLATE(MTO).GDT 8/6/18

RECORD OF BOREHOLE No 17-4

1 OF 1

METRIC

GWP# 5087-11-00 LOCATION Miner's Bay Culvert, MTM z12: N 4 964 695.8 E 362 094.4 ORIGINATED BY CM
 HWY 35 BOREHOLE TYPE Portable COMPILED BY KE
 DATUM Geodetic DATE 2017.10.08 - 2017.10.08 CHECKED BY CM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								W P	W	W L	WATER CONTENT (%)
271.2								20	40	60	80	100							
0.0	SAND with Silt and Gravel Frequent Cobbles and Boulders Loose Brown End of Borehole on Probable Bedrock Water at G.S. (Elev. 271.2 m) on 8/10/2017		1	SS	100/		271												
270.9																			
0.3																			

ONTMT4S 16284 MINER'S BAY CULVERT.GPJ 2012TEMPLATE(MTO).GDT 16/3/18

Appendix C.

Laboratory Testing

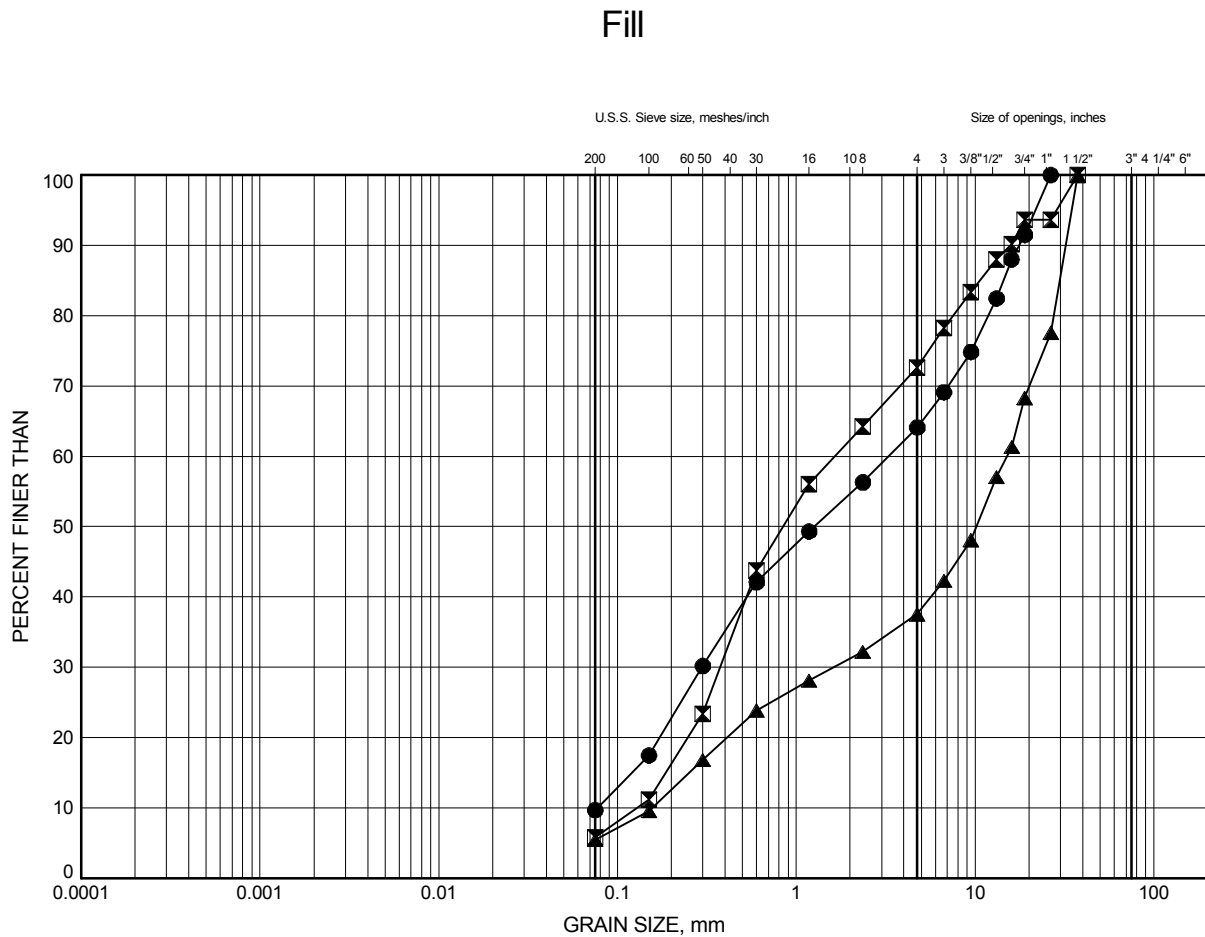
Appendix C.1

Particle Size Analysis Figures

Hwy's 35 and 523, 5 Structures

GRAIN SIZE DISTRIBUTION

FIGURE C1



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	17-1	1.07	275.23
⊠	17-2	1.09	275.13
▲	17-2	1.83	274.40

Date ..October 2017.....

GWP# ..5087-11-00.....



Prep'dCM.....

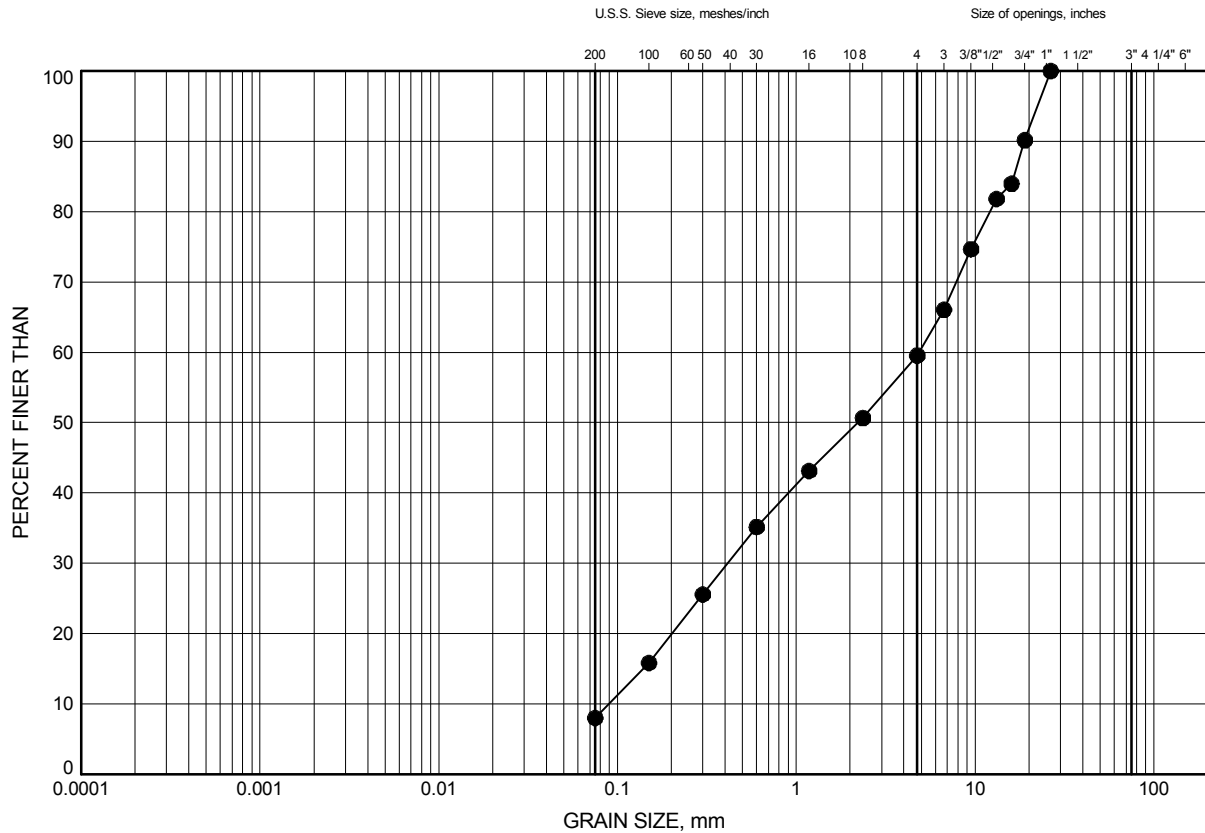
Chkd.SP.....

Hwy's 35 and 523, 5 Structures

GRAIN SIZE DISTRIBUTION

FIGURE C2

Sand with Silt and Gravel



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	17-4	0.15	271.06

Date ..October 2017.....
GWP# ..5087-11-00.....



Prep'dCM.....
Chkd.SP.....

Appendix C.2
Rock Core Photographs

Borehole 17-1
Runs 1 and 2 (of 3)
Elevation 272.7 m to 270.8 m



Borehole 17-1
Run 3 (of 3)
Elevation 270.8 m to 269.2 m



Borehole 17-2
Runs 1 and 2 (of 3)
Elevation 272.2 m to 270.4 m



Borehole 17-2
Run 3 (of 3)
Elevation 270.4 m to 268.9 m



Appendix C.3
Analytical Testing Results

Certificate of Analysis

Thurber Engineering Ltd.

2460 Lancaster Rd, Suite 104
Ottawa, ON K1B4S5
Attn: Stephen Peters

Client PO: 16284
Project: Hwy 35/523
Custody: 38404

Report Date: 29-Aug-2017
Order Date: 23-Aug-2017

Order #: 1734260

This Certificate of Analysis contains analytical data applicable to the following samples as submitted:

Parcel ID	Client ID
1734260-01	Black Creek 17-3 SS#2 7.83-9.83'
1734260-02	Black Creek 17-5 SS#3 10.17-12.17'
1734260-03	Miner's Bay 17-3 SS#1 0-1.25'
1734260-04	Bark Lake 17-3 SS#3 10-12'
1734260-05	Bark Lake 17-6 SS#2 15-17'

Approved By:



Dale Robertson, BSc
Laboratory Director

Certificate of Analysis
Client: Thurber Engineering Ltd.
Client PO: 16284

Report Date: 29-Aug-2017
Order Date: 23-Aug-2017
Project Description: Hwy 35/523

Analysis Summary Table

Analysis	Method Reference/Description	Extraction Date	Analysis Date
Anions	EPA 300.1 - IC, water extraction	25-Aug-17	25-Aug-17
Conductivity	MOE E3138 - probe @25 °C, water ext	29-Aug-17	29-Aug-17
pH, soil	EPA 150.1 - pH probe @ 25 °C, CaCl buffered ext.	28-Aug-17	28-Aug-17
Resistivity	EPA 120.1 - probe, water extraction	29-Aug-17	29-Aug-17
Solids, %	Gravimetric, calculation	26-Aug-17	26-Aug-17

Certificate of Analysis
 Client: Thurber Engineering Ltd.
 Client PO: 16284

Report Date: 29-Aug-2017
 Order Date: 23-Aug-2017
 Project Description: Hwy 35/523

		Client ID:	Black Creek 17-3 SS#2 7.83-9.83'	Black Creek 17-5 SS#3 10.17-12.17	Miner's Bay 17-3 SS#1 0-1.25'	Bark Lake 17-3 SS#3 10-12'
		Sample Date:	14-Aug-17	16-Aug-17	10-Aug-17	08-Aug-17
		Sample ID:	1734260-01	1734260-02	1734260-03	1734260-04
		MDL/Units	Soil	Soil	Soil	Soil
Physical Characteristics						
% Solids	0.1 % by Wt.		73.7	76.1	91.0	70.4
General Inorganics						
Conductivity	5 uS/cm		99	176	220	217
pH	0.05 pH Units		8.33	8.05	7.85	4.91
Resistivity	0.10 Ohm.m		101	56.8	45.5	46.1
Anions						
Chloride	5 ug/g dry		11	51	8	6
Sulphate	5 ug/g dry		23	25	23	176
		Client ID:	Bark Lake 17-6 SS#2 15-17'	-	-	-
		Sample Date:	09-Aug-17	-	-	-
		Sample ID:	1734260-05	-	-	-
		MDL/Units	Soil	-	-	-
Physical Characteristics						
% Solids	0.1 % by Wt.		88.8	-	-	-
General Inorganics						
Conductivity	5 uS/cm		63	-	-	-
pH	0.05 pH Units		5.70	-	-	-
Resistivity	0.10 Ohm.m		158	-	-	-
Anions						
Chloride	5 ug/g dry		7	-	-	-
Sulphate	5 ug/g dry		26	-	-	-

Certificate of Analysis
Client: Thurber Engineering Ltd.
Client PO: 16284

Report Date: 29-Aug-2017
Order Date: 23-Aug-2017
Project Description: Hwy 35/523

Method Quality Control: Blank

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Chloride	ND	5	ug/g						
Sulphate	ND	5	ug/g						
General Inorganics									
Conductivity	ND	5	uS/cm						
Resistivity	ND	0.10	Ohm.m						

Certificate of Analysis
Client: Thurber Engineering Ltd.
Client PO: 16284

Report Date: 29-Aug-2017
Order Date: 23-Aug-2017
Project Description: Hwy 35/523

Method Quality Control: Duplicate

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Chloride	10.5	5	ug/g dry	10.7			1.3	20	
Sulphate	22.3	5	ug/g dry	23.3			4.4	20	
General Inorganics									
Conductivity	844	5	uS/cm	841			0.4	6.2	
pH	8.36	0.05	pH Units	8.45			1.1	10	
Resistivity	11.8	0.10	Ohm.m	11.9			0.4	20	
Physical Characteristics									
% Solids	87.3	0.1	% by Wt.	87.2			0.0	25	

Certificate of Analysis
Client: Thurber Engineering Ltd.
Client PO: 16284

Report Date: 29-Aug-2017
Order Date: 23-Aug-2017
Project Description: Hwy 35/523

Method Quality Control: Spike

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Chloride	101	5	ug/g	10.7	90.4	78-113			
Sulphate	119	5	ug/g	23.3	96.2	78-111			

Certificate of Analysis
Client: Thurber Engineering Ltd.
Client PO: 16284

Report Date: 29-Aug-2017
Order Date: 23-Aug-2017
Project Description: Hwy 35/523

Qualifier Notes:

None

Sample Data Revisions

None

Work Order Revisions / Comments:

None

Other Report Notes:

n/a: not applicable
ND: Not Detected
MDL: Method Detection Limit
Source Result: Data used as source for matrix and duplicate samples
%REC: Percent recovery.
RPD: Relative percent difference.

Soil results are reported on a dry weight basis when the units are denoted with 'dry'.
Where %Solids is reported, moisture loss includes the loss of volatile hydrocarbons.

Appendix D.

Site Photographs

CULVERT REHABILITATION, STRUCTURE NO. 40-117/C
HIGHWAY 35 MINERS BAY CULVERT, LUTTERWORTH TOWNSHIP



Photo 1. Looking upstream from culvert inlet, note erosion at north wingwall



Photo 2. Looking northward from culvert site along Hwy 35

Photographs taken August 17, 2017

CULVERT REHABILITATION, STRUCTURE NO. 40-117/C
HIGHWAY 35 MINERS BAY CULVERT, LUTTERWORTH TOWNSHIP



Photo 3. Looking southward from culvert site along Hwy 35



Photo 4. Looking northward at culvert outlet and wingwalls

Photographs taken August 17, 2017

Appendix E.

GSC Seismic Hazard Calculation

2015 National Building Code Seismic Hazard Calculation

INFORMATION: Eastern Canada English (613) 995-5548 français (613) 995-0600 Facsimile (613) 992-8836
Western Canada English (250) 363-6500 Facsimile (250) 363-6565

October 20, 2017

Site: 44.8198 N, 78.7754 W User File Reference: Miners Bay Culvert

Requested by: , Thurber Engineering

National Building Code ground motions: 2% probability of exceedance in 50 years (0.000404 per annum)

Sa(0.05)	Sa(0.1)	Sa(0.2)	Sa(0.3)	Sa(0.5)	Sa(1.0)	Sa(2.0)	Sa(5.0)	Sa(10.0)	PGA (g)	PGV (m/s)
0.088	0.122	0.121	0.105	0.087	0.053	0.028	0.0071	0.0031	0.071	0.074

Notes. Spectral ($S_a(T)$, where T is the period in seconds) and peak ground acceleration (PGA) values are given in units of g (9.81 m/s^2). Peak ground velocity is given in m/s . Values are for "firm ground" (NBCC 2015 Site Class C, average shear wave velocity 450 m/s). NBCC2015 and CSAS6-14 values are specified in **bold** font. Three additional periods are provided - their use is discussed in the NBCC2015 Commentary. Only 2 significant figures are to be used. *These values have been interpolated from a 10-km-spaced grid of points. Depending on the gradient of the nearby points, values at this location calculated directly from the hazard program may vary. More than 95 percent of interpolated values are within 2 percent of the directly calculated values.*

Ground motions for other probabilities:

Probability of exceedance per annum	0.010	0.0021	0.001
Probability of exceedance in 50 years	40%	10%	5%
Sa(0.05)	0.013	0.037	0.056
Sa(0.1)	0.021	0.055	0.080
Sa(0.2)	0.023	0.057	0.082
Sa(0.3)	0.020	0.050	0.072
Sa(0.5)	0.016	0.041	0.059
Sa(1.0)	0.0078	0.024	0.036
Sa(2.0)	0.0033	0.012	0.018
Sa(5.0)	0.0007	0.0027	0.0042
Sa(10.0)	0.0005	0.0012	0.0018
PGA	0.012	0.031	0.046
PGV	0.0096	0.030	0.046

References

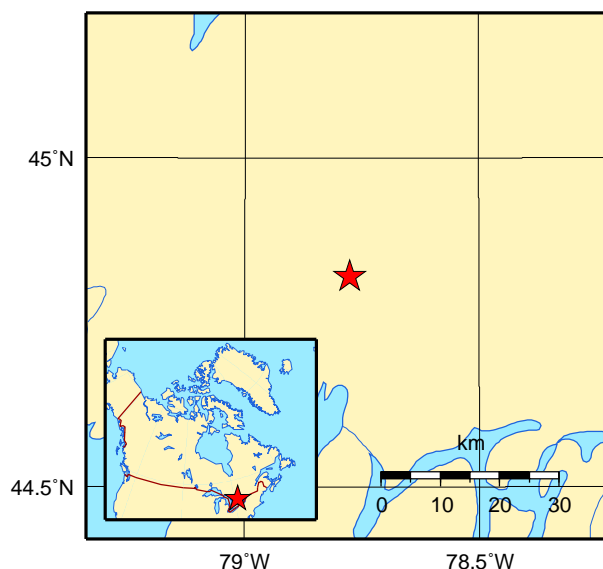
National Building Code of Canada 2015 NRCC no. 56190;
Appendix C: Table C-3, Seismic Design Data for Selected Locations in Canada

User's Guide - NBC 2015, Structural Commentaries NRCC no. xxxxxx (in preparation)
Commentary J: Design for Seismic Effects

Geological Survey of Canada Open File 7893 Fifth Generation Seismic Hazard Model for Canada: Grid values of mean hazard to be used with the 2015 National Building Code of Canada

See the websites www.EarthquakesCanada.ca and www.nationalcodes.ca for more information

Aussi disponible en français



Natural Resources
Canada

Ressources naturelles
Canada



Appendix F.

**List of Referenced Specifications
Suggested NSSP for Obstructions and Rock Dowels**

LIST OF REFERENCED SPECIFICATIONS

OPSD 208.010	Benching of Earth Slopes
OPSD 3101.150	Walls, Abutment, Backfill Minimum Granular Requirements
OPSS.PROV 206	Construction Specification for Grading
OPSS.PROV 501	Construction Specification for Compacting
OPSS 511	Construction Specification for Rip-Rap, Rock Protection, and Granular Sheeting
OPSS.PROV 539	Construction Specification for Temporary Protection Systems
OPSS.PROV 804	Construction Specification for Seed and Cover
OPSS 805	Construction Specification for Temporary Erosion and Sediment Control Measures
OPSS 902	Construction Specification for Excavating and Backfilling-Structures
OPSS.PROV 1010	Material Specification for Aggregates-Base, Subbase, Select Subgrade, and Backfill Material
SP 517F01	Standard Special Provision for Dewatering of Pipeline, Utility, and Associated Structure Excavation
SP 109S12	Amendment to OPSS 902, November 2010 - QVE, Backfilling Compaction, an Certificate of Conformance

SUGGESTED NON-STANDARD SPECIAL PROVISIONS

1. Suggested text for an NSSP on "Obstructions"

"The presence of cobbles, boulders and buried obstructions within the fill and native soils as well as the presence of shallow bedrock may have an impact on excavations as well as the installation of protection systems, rock dowels and/or coffer dams at this site."

DOWELS INTO ROCK – Item No.

Special Provision

CONSTRUCTION SPECIFICATION FOR THE SUPPLY, INSTALLATION AND TESTING OF DOWELS INTO ROCK FOR PIER FOOTINGS

1.0 SCOPE

The work for the above noted tender item shall be in accordance with OPSS 904, including all Special Provisions, except as extended herein. This document specifies additional requirements for the supply, installation and testing of Dowels into Rock for the pier footing.

2.0 REFERENCES

This Special Provision refers to the following standards, specifications, or publications:

ASTM International

D1143M Standard Test Methods for Deep Foundations Under Static Axial Compressive Load

3.0 DEFINITIONS

For the purpose of this Special Provision, the following definitions apply:

Dowels into Rock: reinforcing steel bar and non-shrink grout.

Design Engineer: An Engineer who has a minimum of five (5) years experience in all aspects associated with the installation of Dowels into Rock, including drilling, grouting and doweling work. The Design Engineer shall be retained by the Contractor to design various components for the installation and testing for the Dowels into Rock.

Quality Verification Engineer: An Engineer who has a minimum of five (5) years experience in all aspects associated with the installation of Dowels into Rock, including drilling, grouting and doweling work. The Quality Verification Engineer shall be retained by the Contractor to ensure conformance with the contract documents and issue certificate(s) of conformance.

4.0 DESIGN AND SUBMISSION REQUIREMENTS

4.01 Working Drawings

Working Drawings shall consist of drawings, testing and installation records, procedures and reports, and work plans.

The Contractor shall submit Working Drawings to the Contract Administrator as follows:

- a) All Working Drawings that include drawing, testing and installation procedures and reports, and work plans shall be sealed and signed by the Design Engineer.

- b) All Working Drawings that include testing and installation results and reports shall be signed and sealed by the Quality Verification Engineer.

Upon completion of testing or installation and testing for each component, the Contractor shall submit to the Contract Administrator a Certificate of Conformance sealed and signed by a Quality Verification Engineer. The Certificate shall state that the work has been carried out in conformance with the Working Drawings and in general conformance with the contract documents.

Working Drawings consisting of testing and installation records and reports shall be submitted four days after completion of testing and installation. All other Working Drawings shall be submitted two weeks prior to construction.

Working Drawings to be submitted include the following with further details outlined in the remainder of this specification:

- a) Design calculations, specifications and shop drawings covering all aspects of fabrication, installation and acceptance testing of Dowels into Rock.
- b) Test results verifying the 28 day strength of non-shrink grout.
- c) The method for constructing of the holes, maintaining the holes, and placing reinforcing steel bars, grout and other materials in the holes, including casing sizes, bit sizes and tremie grouting methods.
- d) The procedures to verify hole length. Records of measurements that verify the hole length.
- e) Records of all drilling procedures, rock conditions encountered, and installation times.
- f) Test procedures for Dowels into Rock.
- g) Drawings and design calculations for a suitable reaction system for the applied test loads.
- h) Records of vertical and horizontal movements of the reaction system, and elongation of the reinforcing steel bar.
- i) Drawings and details for reference system arrangement.
- j) Current calibration curves shall be provided for all gauges.
- k) Complete test records for all tests including plots of dowel movement versus dowel load, dowel load versus time, and dowel movement versus time.
- l) Remedial measures for unacceptable stressing results.

5.0 MATERIALS

5.01 Non-Shrink Grout

The non-shrink grout shall be an approved product from the MTO's Pre-Qualified Products List.

5.02 Anti-Washout Agent

The anti-washout agent shall be used with the non-shrink grout for the Dowels into Rock. The anti-washout agent shall be one of the following proprietary products:

- 1) Sikament 100 SC Anti-Washout Admixture
Sika Canada Inc.
6915 Davand Drive
Mississauga, ON, L5T 1L5
Toll Free Phone: 800-933-7452
- 2) Rheomac UW 450 Anti-Washout Admixture
BASF Construction Chemicals Canada Ltd (Master Builders)
1800 Clark Blvd
Brampton, ON, L6T 4M7
Toll Free Phone: 416-520-1392

5.03 Manufacturer Information

The Contractor shall provide the following information from the manufacturer for non-shrink grout and anti-washout agent:

- a) Data sheets for the non-shrink grout and anti-washout agent,
- b) Technical information that proves that the non-shrink grout and anti-washout agent are compatible, and
- c) installation procedures

6.0 EQUIPMENT

All equipment for the installation of the Dowels into Rock shall be suitable for the intended purposes and capable of working on the site under the prevailing access and clearance conditions.

The equipment shall not cause damage to the reinforcing steel bars.

7.0 CONSTRUCTION

7.01 Instructions to Contractor

These instructions are to be read in conjunction with the Contract Drawings.

A total of 2 test Dowels into Rock are required for the Dowels into Rock at the pier.

Dowels into rock at the pier shall be installed into sound bedrock to the specified embedment depth.

7.02 Responsibilities of the Contractor

The Contractor shall prove the allowable bond stress by tests of the Dowels into Rock on non-production Dowels into Rock.

The Contractor shall supply equipment, materials and skilled personnel to install production Dowels into Rock and conduct the specified acceptance tests. It shall be the responsibility of the Contractor to constantly monitor the acceptance tests, maintain specified test loads and record test measurements as specified by the Contract Administrator.

The Contractor is responsible for materials and workmanship. Any remedial measures, required because of defects in materials or workmanship, shall be completed by the Contractor at no cost to the Owner.

The Contractor shall submit 4 copies of all Working Drawings to the Contract Administrator as outlined in Section 4.0.

7.03 Subsurface Conditions

Rock and groundwater conditions are described in the Foundation Investigation Report for this Contract.

7.04 Construction of Holes

The sides and end of the hole shall not be disturbed. The Contractor shall submit Working Drawings to the Contract Administrator that include the method for constructing of the holes, maintaining the holes, and placing reinforcing steel bar, grout and other materials in the holes. All excavated material shall be removed from the site.

The hole diameters and hole length for this project are as specified on the Contract Drawings. Prior to commencing drilling operations, the Contractor shall submit Working Drawings to the Contract Administrator outlining devised procedures to verify hole length. The Contractor shall submit Working Drawings that include drilling operations records to the Contract Administrator that include the above noted records.

At all times, the Contractor shall keep a record of all drilling procedures, rock conditions encountered, and installation times. The Contractor shall submit Working Drawings to the Contract Administrator that include the above noted records.

7.05 Installation of Reinforcing Steel Bar

Reinforcing steel bar shall be installed in strict accordance with the Contract Drawings and installation procedures.

Centering devices shall be provided to ensure that the reinforcing steel bar is located centrally in the hole.

Dowels into Rock at the pier shall be installed into sound bedrock.

Reinforcing steel bar shall be installed after the dowel hole has been filled with non-shrink grout.

7.06 Grout and Anti-Washout Agent

The non-shrink grout shall entirely fill the annular space between the reinforcing steel bar and side for the dowel hole.

The placement of grout for the test Dowels into Rock shall be identical to the production Dowels into Rock.

Anti-washout agent shall be used in accordance with the specifications of the manufacturer.

Non-shrink grout shall be placed into the dowel hole using tremie placement methods.

8.0 QUALITY ASSURANCE

All work for the installation of Dowels into Rock shall be inspected by the Quality Verification Engineer.

8.01 Qualifications

8.01.01 Qualifications of Staff from Contractor or Sub-Contractor Completing Work for the Dowels into Rock

All work shall be performed under the direction of personnel experienced with all aspects associated with the installation of Dowels into Rock. Such experience shall have been obtained within the preceding five (5) years on projects of similar nature and scope to the work required for this project.

8.01.02 Qualifications of the Quality Verification Engineer

A resume of the work experience of the Quality Verification Engineer shall be submitted to the Contract Administrator for record purposes. The Quality Verification Engineer shall be a Professional Engineer licensed in the Province of Ontario having a minimum of five years of experience on projects of similar nature and scope to the work required for this project.

8.01.03 Qualifications of the Design Engineer

A resume of the work experience of the Design Engineer shall be submitted to the Contract Administrator for record purposes. The Design Engineer shall be a Professional Engineer licensed in the Province of Ontario having a minimum of five years of experience of projects of similar nature and scope to the work required for this project.

8.02 Testing Requirements

All work for the testing of Dowels into Rock shall be inspected by the Quality Verification Engineer.

8.02.01 General Testing Requirements

Refer to the attached Instructions to Contractor and the Contract Drawings for specific test details.

The Contractor shall install the number of Dowels into Rock specified in the contract documents for testing purposes. The purpose of the testing the Dowels into Rock is to prove the adequacy of the proposed anchor configuration and installation procedures under the site conditions, and to provide design parameters.

The equipment, labour and materials for test dowels shall be identical to Dowels into Rock at the pier. The Dowels into Rock for testing shall be M dowels grouted into mm diameter holes filled with an approved non-shrink grout with a minimum mm embedment into sound bedrock.

The Contractor shall submit Working Drawings that include proposed procedures for testing of the Dowels into Rock to the Contract Administrator. Such testing shall be executed in strict accordance with the proposed procedures of the Contractor.

The Quality Verification Engineer shall supervise the testing of the Dowels into Rock. The Contractor will notify the Contract Administrator of the testing schedule at least 10 days prior to commencement of the

testing program. Testing for Dowels into Rock shall be conducted concurrently, as scheduled by the Contract Administrator. The tests shall normally be conducted between 8:00 hrs and 20:00 hrs from Monday to Friday, unless otherwise directed by the Contract Administrator.

The Contractor shall supply materials and skilled personnel to conduct the tests for the Dowels into Rock. The equipment and materials shall be capable of stressing the Dowels into Rock to the specified loads. It shall be the responsibility of the Contractor to constantly monitor the test, maintain specified test loads and to record test measurements as specified by the Quality Verification Engineer.

The test site shall be restored to its pre-test condition. Reinforcing steel bars used in tests shall be cut down 25 mm below the top of the sound bedrock.

8.02.02 Testing Location

The Contractor shall remove all loose rock down to sound bedrock at the test location.

The test Dowels into Rock shall be constructed at locations specified by the Contract Administrator.

If site conditions dictate, changes to the test locations will be considered. The Contractor shall provide the Contract Administrator at least 2 days notice in writing of this operation.

8.02.03 Testing Equipment

The dowels into rock will be carried out generally in accordance with the prevailing requirements of ASTM International D1143M superseded where applicable by the procedures specified in this document.

The Contractor shall submit Working Drawings for a suitable reaction system for the applied test loads to the Contract Administrator. Jacks must be secured with chains to provide adequate protection for the personnel in the event of breakage of the reinforcing steel bar or stressing system.

The Contractor shall submit Working Drawings for the reference system arrangement to the Contract Administrator. All reference beams shall be as follows:

The beams shall be independently supported with the support firmly embedded in the ground.

The testing device shall not apply compression to the bedrock surrounding the test for the Dowels into Rock, within a circle concentric with the dowel hole and a diameter equal to 4.0 m.

Reference beams shall be sufficiently rigid to support instrumentation such that variations in readings do not occur.

The Contractor shall construct suitable enclosures to provide complete protection for equipment and instruments from variations in the weather conditions and disturbances during the test program. These provisions must meet the approval of the Quality Verification Engineer and will include that the test enclosures must be weather-proof and provide a consistent temperature in order to eliminate temperature variations that could affect instrumentation.

8.02.04 Testing for Dowels Into Rock, and Report

At all times, the Contractor shall keep records of vertical and horizontal movements of the reaction system, elongation of reinforcing steel bar, and the record of test enclosure temperature. The movements shall be recorded with respect to an independent fixed reference point. The Contractor shall submit Working Drawings that include the above noted records to the Contract Administrator.

Dial gauges shall have at least a 76.2 mm (3.0 in.) travel. Longer gauge stems or sufficient gauge blocks shall be provided to allow for greater travel if required. Gauges shall have precision of at least 0.025 mm (0.0001 in.). The dial gauges shall be placed on smooth bearing surfaces mounted perpendicular to the direction of movement. All gauges, scales or reference points attached to the test anchor shall be mounted so as to prevent movement relative to the test anchor during the test. The Contractor shall submit Working Drawings that include details for current calibration and curves for all gauges to the Contract Administrator.

Jacks used for reinforcing steel bars shall have a minimum ram dimension of 152.6 mm (6.0 in.). The Contractor shall submit Working Drawings that include details for current calibration and curves for all gauges to the Contract Administrator.

Requirements for Clauses 5.4.1 to 5.4.4 shall be repeated as required at different testing locations.

8.02.05 Testing Loading

The testing procedures shall safely load test the Dowels into Rock in tension at a rate of approximately 100kN per minute to the test load of [REDACTED] kN. The load shall be increased by an additional 50 kN beyond this level as directed by the Quality Verification Engineer.

Each load shall be maintained for a minimum time of 15 minutes and until the rate of displacement is not greater than 0.25 mm (0.01 inches) per hour.

8.03 Acceptance Criteria

The following acceptance criteria apply:

- a) The testing of dowels shall be carried out in advance of the instalment of Dowels into Rock at the pier footing.
- b) Tests for Dowels into Rock shall have a capacity of at least [REDACTED] kN. The Quality Verification Engineer shall report on the acceptance of the tests for Dowels into Rock. The Quality Verification Engineer shall report on the testing of the Dowels into Rock including recommendations for increasing embedment depth, if necessary.

9.0 MEASUREMENT FOR PAYMENT

For measurement purposes, a count shall be made of the number of dowels installed.

10.0 BASIS OF PAYMENT

Payment at the contract unit price for the above tender item shall include full compensation for all labour, equipment, and materials to do the work. No additional payment will be made for tests for Dowels into Rock which are deemed as included as part of the work for the above noted item.