

PRELIMINARY
FOUNDATION INVESTIGATION AND DESIGN REPORT
SARAFIANS SIDE ROAD CULVERT REPLACEMENT
TOWNSHIP OF LIMERICK, MUNICIPALITY OF HASTINGS
SITE # 11-416C, OFF HIGHWAY 62
W.P. 4009-13-01
Geocres Number: 31C-237

Report to
AECOM

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PART 1: FACTUAL INFORMATION

1 INTRODUCTION

This report presents the factual findings obtained during a foundation investigation completed at the site of a proposed culvert replacement at Sarafians Side Road in the Township of Limerick, Municipality of Hastings, 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, record of borehole sheets, laboratory test results and a written description of the subsurface conditions. A model of the subsurface conditions was developed from the data obtained in the course of the investigation.

Thurber Engineering Ltd. (Thurber) carried out the investigation as a sub-consultant to AECOM under the Ministry of Transportation (MTO) Agreement Number 4013-E-0015.

2 SITE DESCRIPTION

The culvert at Site 11-416/C is located on Sarafians Side Road, off Highway 62 approximately 43 km north of the town of Madoc, in the Township of Limerick, Ontario. The existing structure is a single span, sectional plate, corrugated steel pipe arch, 3.1 m x 2 m in size and 21 m long.

The surrounding land is treed and undulating. Outcropping bedrock is visible at most of the high points of the landscape and swamps are present at most of the low points. The site is located in a swampy area. The waters generated from the swamps forms an unnamed creek, which flows along the east side of Highway 62 and through the Sarafians Side Road culvert, eventually discharging into Limerick Lake. Photographs of the culvert and surrounding area are presented in Appendix C.

The site is situated within the physiographic region known as the Algonquin Highlands, which is comprised of shallow till deposits and rock ridges. At the higher elevations, Precambrian, metasedimentary bedrock is often exposed or is overlain by a thin layer of overburden (likely till or muskeg).

3 SITE INVESTIGATION AND FIELD TESTING

The field investigation and testing at this site was carried out on December 4, 2014, and consisted of drilling and sampling of two boreholes identified as 14-06 and 14-07. The boreholes were extended to depths ranging between 3.6 and 4.0 m below the existing ground surface. The approximate borehole locations are shown on the attached Borehole Location and Soil Strata Drawing included in Appendix D.

Prior to commencement of drilling, utility clearances were obtained for all borehole locations. Drilling was carried out using a truck mounted D90 drill rig with solid stem augers. Soil samples were obtained at selected intervals using a split spoon sampler in conjunction with Standard Penetration Testing (SPT).

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 samples for transport to Thurber's laboratory in Oakville, Ontario for further examination and testing.

The elevations of the boreholes are referenced to a geodetic datum (Elev. 316.5), assigned to the top of the north end of the culvert. The geodetic datum was provided by AECOM.

Groundwater conditions were observed in the open boreholes during and upon completion of the drilling operations. A standpipe piezometer, consisting of 19 mm Schedule 40 PVC pipe with a 1.5 m long slotted screen, was installed in borehole 14-06. The piezometer screen was enclosed in filter sand to permit groundwater level monitoring. The boreholes were backfilled in general accordance with Ontario Regulation 903 amended by Ontario Regulation 372, as summarized in Table 3-1.

Table 3-1. Piezometer Installation and Backfilling Details

Borehole	Borehole Depth/ Elevation (m)	Borehole Backfilling Details
14-06	4.0 / 313.0	Piezometer with 1.5 m slotted screen installed, sand filter from 4.0 to 2.1 m, bentonite holeplug and cuttings from 2.1 m to ground surface
14-07	3.6 / 313.7	Bentonite holeplug and cuttings from 3.6 m to ground surface

Following the final water level reading, the piezometer was decommissioned in general accordance with Ontario Regulation 903.

4 LABORATORY TESTING

The recovered soil samples were subjected to Visual Identification (VI) and to natural moisture content determination. Selected samples were also subjected to gradation analysis (hydrometer and/or sieve) where appropriate. The results of these tests are summarized on the Record of Borehole sheets included in Appendix A and are presented on the figures included in Appendix B.

5 DESCRIPTION OF SUBSURFACE CONDITIONS

Details of the encountered soil stratigraphy are presented on the Record of Borehole sheets included in Appendix A. A general description of the stratigraphy, based on the conditions encountered in the boreholes, is given in the following paragraphs. However, the factual data presented on the Record of Borehole sheets takes precedence over this general description and interpretation of the site conditions.

It should be recognized that soil conditions may vary between and beyond the borehole locations. The stratigraphic boundaries shown on the Record of Borehole sheets are inferred from non-continuous sampling and represent transition between soil types rather than exact geological change.

In general, the soils encountered at this site consisted of a pavement/embankment fill either extending to the refusal stratum or underlain by a layer of fibrous peat. The refusal stratum was interpreted as probable bedrock. More detailed descriptions of the individual strata are presented below.

5.1 Pavement Structure/Road Embankment Fill

Both boreholes were drilled through the Sarafians Side Road pavement structure in proximity to the existing culvert and encountered approximately 75 mm of asphalt overlying granular road base and embankment fill.

The granular road base consisted of silty sand fill with trace clay, trace gravel and trace organic matter. Where fully penetrated in borehole 14-07, the thickness of the granular fill was 2.2 m with an underside depth of 2.3 m (Elev. 314.7). Borehole 14-06 was terminated within this layer upon auger refusal on probable bedrock at a depth of 4.0 m (Elev. 313.0). SPT tests performed in the granular fill gave N-values between 2 and 38 blows per 0.3 m of penetration, indicating a very loose to dense relative density. One value of 60 blows per 0.275 m of penetration obtained in Borehole 14-06 was probably indicative of the presence of rock fragments or cobbles. Moisture contents within the granular fill varied from 6 to 41%.

A grain size distribution tests were completed on three samples of the granular fill. The results are summarized on the Record of Borehole sheets in Appendix A and the grain size distribution curves are included in Figure B1 of Appendix B. The results of the laboratory tests are summarized as follows:

Soil Particles	Percentage (%)
Gravel	3 to 6
Sand	59 to 73
Silt	20 to 31
Clay	4 to 6

5.2 Peat

A layer of fibrous peat with some sand was encountered below the granular fill in Borehole 14-07. The borehole was terminated within this layer upon auger refusal on probable bedrock at a depth of 3.6 m (Elev. 313.7).

SPT tests performed in this layer gave N-values between 2 and 15 blows per 0.3 m of penetration, indicating a soft to stiff consistency. Moisture contents of 102 and 188% were measured in this deposit.

5.3 Groundwater Levels

Water levels were observed in the open boreholes upon completion of the drilling. As outlined in Table 3-1, a standpipe piezometer was installed in borehole 14-06 to monitor groundwater levels after drilling. The measured groundwater levels are summarized below in Table 5-1.

Table 5-1. Measured Groundwater Levels

Borehole	Date	Groundwater Level		Comment
		Depth (m)	Elevation (m)	
14-06	Dec. 4, 2014	2.0	315.0	Piezometer
	Dec. 18, 2014	2.5	314.5	
14-07	Dec. 4, 2014	2.4	314.9	Open Borehole

The values shown 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 periods of significant or prolonged precipitation events and will respond to the water level in the creek.

6 MISCELLANEOUS

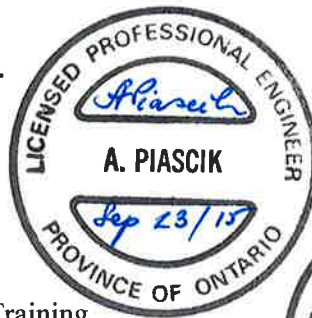
Borehole locations were selected and marked in the field by an experienced Thurber staff member and were established with a Trimble Pathfinder ProXRT differential GPS unit.

Walker Drilling Ltd. from Utopia, Ontario supplied and operated the drilling, sampling and in-situ testing equipment for the field program. The field investigation was supervised on a full time basis by Mr. Sean Petrus, EIT of Thurber. Overall supervision of the investigation program was conducted by Mr. Weiss Mehdawi, P.Eng.

Routine laboratory testing was carried out by Thurber's geotechnical laboratory in Oakville, Ontario. Interpretation of the data and preparation of this report were carried out by Mr Michael Eastman, EIT and Ms. Anna Piascik, P.Eng. The report was reviewed by Mr. Alastair Gorman, P.Eng. and Mr. P.K. Chatterji, P.Eng. who is a Designated Principal Contact for MTO Foundations Projects.

Thurber Engineering Ltd.

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PART 2: ENGINEERING DISCUSSION AND RECOMMENDATIONS

7 INTRODUCTION

This section of the report provides an interpretation of the factual data and presents geotechnical discussion and preliminary recommendations for planning of the proposed culvert replacement at Site #11-416/C at Sarafians Side Road in the Township of Limerick, Ontario.

The existing culvert is a single span, sectional plate, corrugated steel pipe arch, 3.1 m x 2 m in size and 21 m long. An unnamed creek flows in a northwest to southeast direction at the site and eventually flows into Limerick Lake.

No details of the proposed culvert replacement were available at the time of writing of this report.

The discussions presented in this preliminary report are for planning purposes only. The Design Builder shall satisfy himself as to the sufficiency of the information needed to meet the requirements for detailed design. The Design Builder is solely responsible for selecting the appropriate foundation alternatives for replacement of the culvert and detailed design of the culvert.

8 CULVERT FOUNDATION

8.1 General

In summary, the subsurface conditions encountered at the site consist of a pavement structure and embankment fill underlain by peat. Both boreholes were terminated at depths between 3.6 to 4.0 m upon auger refusal on probable bedrock. The boreholes were located in the vicinity of the existing culvert, and the subsurface conditions encountered at the culvert may vary from those encountered in the boreholes.

Groundwater levels measured in a standpipe piezometer installed at the site and in an open boreholes ranged from 2.0 to 2.5 m below the ground surface. The groundwater levels will correspond to the water level in the creek.

The site is located in a generally swampy area and the original site conditions can be described as peat over bedrock. Fill has been placed over this subsurface condition to construct Highway 62 and to construct Sarafians Side Road. As a result of these swampy site conditions, creation of a dry work space may be difficult. Sheet pile cofferdams may

not be effective as they cannot penetrate below the culvert invert and bedrock appears to be at shallow depth. An enlarged excavation accompanied by sand-bagging may be at least partially effective in controlling water inflow.

Consideration should also be given to controlling/diverting the flow of water in the creek during construction. All dewatering operations should be conducted in accordance with OPSS 518.

8.2 Culvert Alternatives

This section presents discussions on alternatives for culvert replacement, and on a feasible/preferred foundation option. Several common culvert and foundation types are listed below:

- Circular Pipes (Concrete, CSP, HDPE) or pipe Arch
- Box Culvert
- Open Footing Culvert

All three culvert types are considered to be possible candidates for this site. However, from a constructability point of view, pipe or pre-cast box culvert options are preferred. The two boreholes drilled at the site show the presence of probable bedrock at depths of 3.6 to 4.0 m below the grade of Sarafians Road. In Borehole 14-06, 4.0 m of sand fill overlies probable bedrock. In Borehole 14-07, 2.3 m of sand fill overlies peat, with probable bedrock at a depth of 3.6 m. This suggest that the original peat was stripped to bedrock when the existing culvert was constructed.

Considering that the height of the existing culvert is 2.0 m, the underside of this culvert, and presumably of the replacement, is interpreted to lie at approximately Elevation 313.5, or a maximum of 500 mm above bedrock

8.3 Circular Pipe or Pipe Arch

Replacement using a circular pipe or pipe arch should be designed and constructed in accordance with the appropriate OPSD and OPSS requirements.

Excavation should be carried out to the level of bedrock at Elevations 313.0 to 313.7 as shown on the borehole logs. The extent of excavation should accommodate the minimum extent of bedding and backfill shown in OPSD 802.010 or 802.020.

The factored geotechnical resistance at Ultimate Limit State (ULS) and the geotechnical reaction at Serviceability Limit State (SLS) for a pipe culvert founded on clear stone bedding placed on bedrock can be assumed as follows:

- Factored Geotechnical Resistance at ULS of 500 kPa
- Geotechnical Resistance at SLS (less than 25 mm settlement) of 300 kPa.

8.4 Box Culvert

8.4.1 Foundation Design

Excavation should be carried down to the level of bedrock at Elevations 313.0 to 313.7 as shown on the borehole logs. The extent of excavation should accommodate the minimum extent of bedding and backfill shown in OPSD 803.010 or MTOD 803.021.

Since it will be difficult to maintain an unwatered excavation, bedding could consist of a layer of 53 mm clear stone (OPSS.PROV 1004) placed from the base of excavation in bedrock to the underside of culvert.

8.4.2 Geotechnical Resistance

The factored geotechnical resistance at the Ultimate Limit State (ULS) and the geotechnical reaction at Serviceability Limit State (SLS) for a box culvert founded on clear stone bedding placed on bedrock as described above can be assumed as follows:

- Factored Geotechnical Resistance at ULS of 500 kPa
- Geotechnical Resistance at SLS (less than 25 mm settlement) of 300 kPa.

The geotechnical resistance and reaction provided above are based on a footing subjected to vertical concentric loading. Where eccentric or inclined loads are applied, the resistance used in the design should be reduced in accordance with the CHBDC clause 6.7.3 and 6.7.4.

Resistance to lateral forces/sliding between precast concrete and the underlying bedding material should be evaluated in accordance with the CHBDC assuming an ultimate/unfactored coefficient of friction of 0.4.

The box culvert should be designed to resist external loadings, including lateral earth pressure, hydrostatic pressure, weight of embankment fill, traffic loadings and surcharge due to construction equipment and activities. The box structure composed of pre-cast segments are, in general, tolerant of small magnitudes of movement related to freeze-thaw cycles; therefore, founding the box culvert below the standard depth for frost penetration is not necessary.

8.5 Open Footing Culvert

8.5.1 Foundation Design

If an open footing culvert design is adopted, it is recommended that the site be excavated to probable bedrock encountered at the local datum Elevations 313.0 to 313.7 as shown on the borehole logs.

Footings should be founded on the bedrock and the factored geotechnical resistance at ULS of 500 kPa could be used in design.

The geotechnical reaction at SLS will be much higher than the geotechnical resistance; therefore, the geotechnical resistance at ULS will govern the design.

The resistance provided above is based on a footing subjected to vertical concentric loading. Where eccentric or inclined loads are applied, the resistance used in the design should be reduced in accordance with the CHBDC clause 6.7.3 and 6.7.4.

Resistance to lateral forces/sliding between concrete and the underlying bedding material should be evaluated in accordance with the CHBDC assuming an ultimate/unfactored coefficient of friction of 0.5.

The culvert should be designed to resist external loadings, including lateral earth pressure, hydrostatic pressure, weight of embankment fill, traffic loadings and surcharge due to construction equipment and activities. For culvert footings founded on bedrock, frost action is not a concern.

8.6 Settlement

All existing material will be removed to bedrock and will be replaced using granular bedding and backfill. No grade raise is anticipated. Accordingly, no significant post-construction settlements are anticipated.

9 BACKFILL AND LATERAL EARTH PRESSURES

For a box culvert replacement, reference should be made to the backfill arrangements stipulated in OPSS 422 for pre-cast rigid frame culverts. Backfill for a concrete culvert should be completed in accordance with OPSD 803.010. As indicated earlier, the bedding and backfill below the water level should be 53 mm clear stone. Backfill above the water level should consist of Granular A or Granular B Type II.

All fills above the water level should be placed in regular lifts and be compacted in accordance with OPSS 501. The backfill should be placed and compacted in simultaneous lifts on both sides of the culvert, and the top of the backfill elevation should be the same on both sides of the culvert at all times. Heavy compaction equipment should not be used adjacent to the walls and roofs of the culvert.

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

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

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

If full drainage is not achievable, the culvert walls should be designed to withstand full hydrostatic pressure assuming a water level at least equal to the design creek water level.

Earth pressure coefficients for backfill are dependent on the material used as backfill. Unfactored coefficients that could be used in design are shown below in Table 9-1.

Table 9-1. Earth Pressure Coefficients

Wall 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 (modified) $\phi = 32^\circ$; $\gamma = 21.2 \text{ kN/m}^3$		Embankment Fill $\phi = 30^\circ$; $\gamma = 20.0 \text{ kN/m}^3$	
	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)
Active (Unrestrained Wall)	0.27	0.40	0.31	0.48	0.33	0.54
At rest (Restrained Wall)	0.43	0.62	0.47	0.70	0.50	0.76
Passive (Movement Towards Soil Mass)	3.70	-	3.30	-	3.00	-

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 will result in lower earth pressures acting on the culvert.

The parameters in the tables 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. Active pressures should be used for any wingwalls or unrestrained walls.

For rigid structures, such as a concrete box culvert, at-rest horizontal earth pressures should be used for design.

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 B Type II.

10 EMBANKMENT DESIGN AND CONSTRUCTION

Embankment reconstruction, after culvert replacement, should be carried out in accordance with OPSS 206. The embankment material should consist of imported Granular A or B Type II material. Excavated granular fill may also be reused as backfill provided the following conditions are satisfied:

- There is sufficient space to stockpile on site and control moisture content within acceptable limits for compaction
- No peat or organics are included in the fill
- Gradation and compaction characteristics meet the requirements prior to reuse as backfill

Where applicable, benching the existing slope surface should be carried out in accordance with OPSD 208.010 in order to enhance the keying in of the newly placed fill. Provided that all existing fill and peat are excavated to bedrock level and granular fill is placed as indicated in Section 9, it is anticipated that an embankment slope inclination of 2H:1V, or flatter, should remain stable.

11 EROSION CONTROL

Erosion protection should be provided at the culvert inlet and outlet areas. Design of the erosion protection measures should consider hydrologic and hydraulic factors and should be carried out by specialists experienced in this field.

Typically, rock protection should be provided over all surfaces with which creek water is likely to be in contact. Treatment at the outlets 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 804.

A clay seal or a concrete cut-off wall could be used to minimize the potential for erosion near the inlet area. The clay seal should extend a minimum of 0.3 m above the high water level and laterally for the width of the granular material, and have a minimum thickness of 0.5 m. The material requirements should be in accordance with OPSS 1205. A geosynthetic clay liner may be used as a clay seal.

12 EXCAVATION AND GROUNDWATER CONTROL

12.1 General

All excavations should be carried out in accordance with the Occupational Health and Safety Act (OHSA). For the purposes of the OHSA, the embankment fill and native peat at this site are classified as Type 3 soils above the water level and Type 4 soils below the water level.

12.2 Excavations

The excavation for the culvert replacement will typically be carried out through the existing embankment fill. At locations where there is space restriction or where a slope has to be retained, the excavation will need to be carried out in conjunction with a protection system. Any protection system should be designed by a licensed Professional Engineer experienced in such designs. OPSS 539 "Construction Specifications for Protection Systems" will have to be included in the contract documents. The protection system should be designed for Performance Level 2 (maximum 25 mm horizontal deflection).

12.3 Groundwater Control

It is expected that groundwater and surface water will accumulate in the excavation during culvert construction. Creek diversion will be required for construction of the replacement culvert. The groundwater level is expected to be largely governed by the water level in the creek and seasonal weather patterns. The Contractor should recognize the challenges presented by the creek and groundwater conditions and should adopt construction practices that will ensure a finished product that will meet the contract requirements. All dewatering operations should be carried out in accordance with OPSS 518.

13 CLOSURE

Engineering analysis and preparation of the report was carried out by Ms. Anna Piascik, P.Eng.

The report was reviewed by Mr. Alastair Gorman, P.Eng. and Dr. P.K. Chatterji, P.Eng. who is a Designated Principal Contact for MTO Foundations Projects.

Thurber Engineering Ltd.

Anna Piascik, P.Eng.
Senior Geotechnical Engineer



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P. K. Chatterji, P.Eng.
Review Principal



Appendix A

Record of Borehole Sheets

SYMBOLS, ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES

1. TEXTURAL CLASSIFICATION OF SOILS

CLASSIFICATION	PARTICLE SIZE	VISUAL IDENTIFICATION
Boulders	Greater than 200mm	same
Cobbles	75 to 200mm	same
Gravel	4.75 to 75mm	5 to 75mm
Sand	0.075 to 4.75mm	Not visible particles to 5mm
Silt	0.002 to 0.075mm	Non-plastic particles, not visible to the naked eye
Clay	Less than 0.002mm	Plastic particles, not visible to the naked eye

2. COARSE GRAIN SOIL DESCRIPTION (50% greater than 0.075mm)

TERMINOLOGY	PROPORTION
Trace or Occasional	Less than 10%
Some	10 to 20%
Adjective (e.g. silty or sandy)	20 to 35%
And (e.g. sand and gravel)	35 to 50%

3. TERMS DESCRIBING CONSISTENCY (COHESIVE SOILS ONLY)

DESCRIPTIVE TERM	UNDRAINED SHEAR STRENGTH (kPa)	APPROXIMATE SPT ⁽¹⁾ 'N' VALUE
Very Soft	12 or less	Less than 2
Soft	12 to 25	2 to 4
Firm	25 to 50	4 to 8
Stiff	50 to 100	8 to 15
Very Stiff	100 to 200	15 to 30
Hard	Greater than 200	Greater than 30

NOTE: Hierarchy of Soil Strength Prediction

- 1) Laboratory Triaxial Testing
- 2) Field Insitu Vane Testing
- 3) Laboratory Vane Testing
- 4) SPT value
- 5) Pocket Penetrometer


4. TERMS DESCRIBING DENSITY (COHESIONLESS SOILS ONLY)

DESCRIPTIVE TERM	SPT "N" VALUE
Very Loose	Less than 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very Dense	Greater than 50

5. LEGEND FOR RECORDS OF BOREHOLES

SYMBOLS AND ABBREVIATIONS FOR SAMPLE TYPE	SS Split Spoon Sample	WS Wash Sample	AS Auger (Grab) Sample
	TW Thin Wall Shelby Tube Sample	TP Thin Wall Piston Sample	
	PH Sampler Advanced by Hydraulic Pressure	PM Sampler Advanced by Manual Pressure	
	WH Sampler Advanced by Self Static Weight	RC Rock Core	SC Soil Core

$$\text{Sensitivity} = \frac{\text{Undisturbed Shear Strength}}{\text{Remoulded Shear Strength}}$$

 Water Level
 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.

EXPLANATION OF ROCK LOGGING TERMS


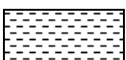

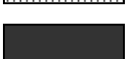

ROCK WEATHERING CLASSIFICATION

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.

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

SYMBOLS

	CLAYSTONE
	SILTSTONE
	SANDSTONE
	COAL
	BEDROCK

STRENGTH CLASSIFICATION

Rock Strength	Approximate Uniaxial Compressive Strength		Field Estimation of Hardness*
	(MPa)	(psi)	
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

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 % 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.

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			

RECORD OF BOREHOLE No 14-06

1 OF 1

METRIC

W.P. 4009-13-01 LOCATION Site 11-416/C, Sarafians Side Rd. N 4 968 901.7 E 212 301.5 ORIGINATED BY SDP
 HWY 62 BOREHOLE TYPE Solid Stem Augers COMPILED BY AN
 DATUM Geodetic DATE 2014.12.04 - 2014.12.04 CHECKED BY MW

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									
								20	40	60	80	100					
317.0	GROUND SURFACE																
0.0 0.1	ASPHALT: (75mm)																
	SAND, silty, trace clay, trace gravel Very Loose to Compact Brown Moist to Wet (FILL)		1	SS	21												
			2	SS	8		316										
	trace organics		3	SS	2		315									3 60 31 6	
			4	SS	9		314										
	rock fragments/cobbles		5	SS	60/ 0.275											6 59 31 4	
313.0 4.0	END OF BOREHOLE AT 4.0m UPON AUGER REFUSAL ON PROBABLE BEDROCK. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen. WATER LEVEL READINGS: DATE DEPTH (m) ELEV. (m) Dec 04/14 2.0 315.0 Dec 18/14 2.5 314.5																

ONTMT4S 0620.GPJ 2015TEMPLATE(MTO).GDT 9/22/15

RECORD OF BOREHOLE No 14-07

1 OF 1

METRIC

W.P. 4009-13-01 LOCATION Site 11-416/C, Sarafians Side Rd. N 4 968 892.9 E 212 295.6 ORIGINATED BY SDP
 HWY 62 BOREHOLE TYPE Solid Stem Augers COMPILED BY AN
 DATUM Geodetic DATE 2014.12.04 - 2014.12.04 CHECKED BY MW

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						
317.3	GROUND SURFACE							20 40 60 80 100						
0.0 0.1	ASPHALT: (75mm)													
	SAND, silty, trace clay, trace gravel Loose to Dense Brown Moist (FILL)		1	SS	38		317							
			2	SS	12									
							316							
			3	SS	6									
315.0														
2.3	PEAT, fibrous, some sand Soft to Stiff Black Wet		4	SS	2		315							
	rock fragments/cobbles		5	SS	15		314							
313.7														
3.6	END OF BOREHOLE AT 3.6m UPON AUGER REFUSAL UPON PROBABLE BEDROCK. WATER LEVEL AT 2.4m. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG AND CUTTINGS TO 0.5m, CONCRETE TO 0.2m, THEN ASPHALT PATCH TO SURFACE.													

+³, ×³: Numbers refer to
Sensitivity

20
15
10
(%) STRAIN AT FAILURE

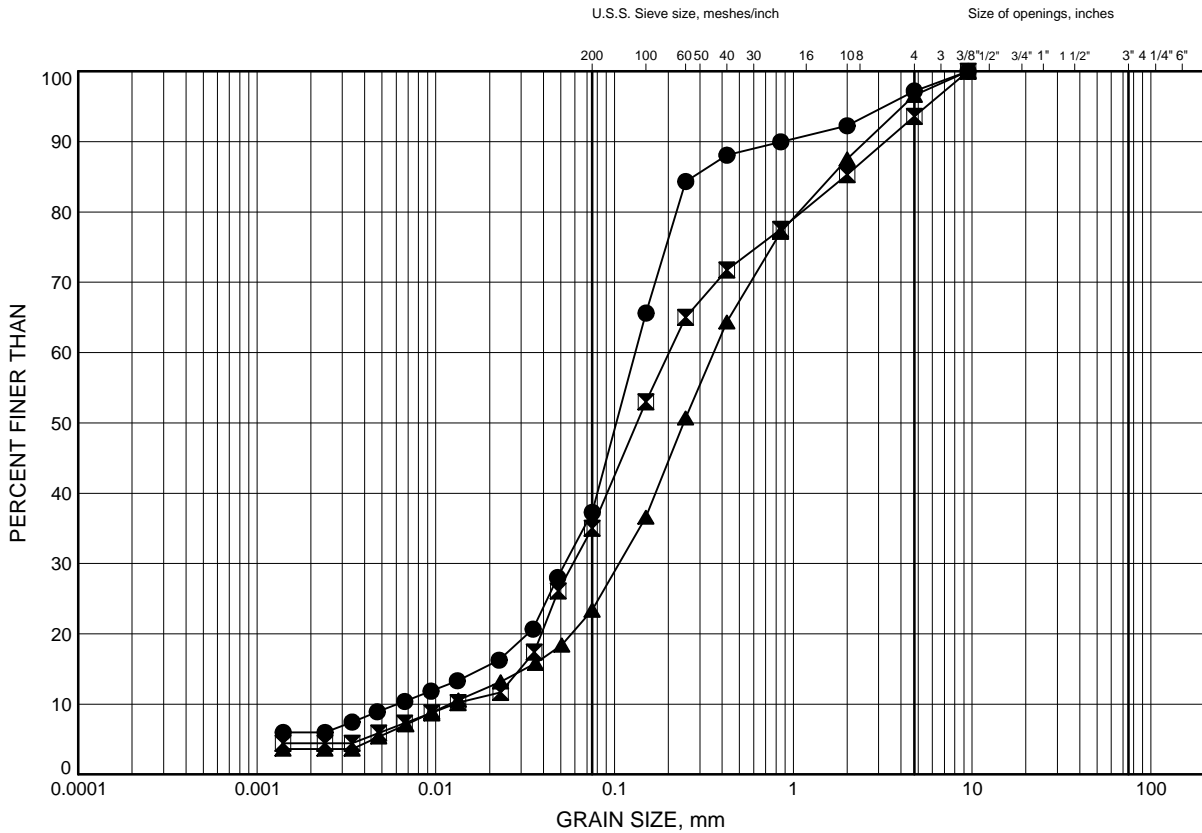
Appendix B

Laboratory Test Results

Site 11-416/C, Sarafians Side Rd.
GRAIN SIZE DISTRIBUTION

FIGURE B1

Silty SAND FILL



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	14-06	1.83	315.17
⊠	14-06	3.26	313.74
▲	14-07	1.07	316.23

Date .. September 2015 ..
W.P. .. 4009-13-01 ..



Prep'd .. AN ..
Chkd. .. AMP ..

Appendix C

Site Photographs



Photograph 1 - Outlet of Culvert on South Side of Sarafians Side Road.



Photograph 2 – Inlet of Culvert on North Side of Sarafians Side Road.



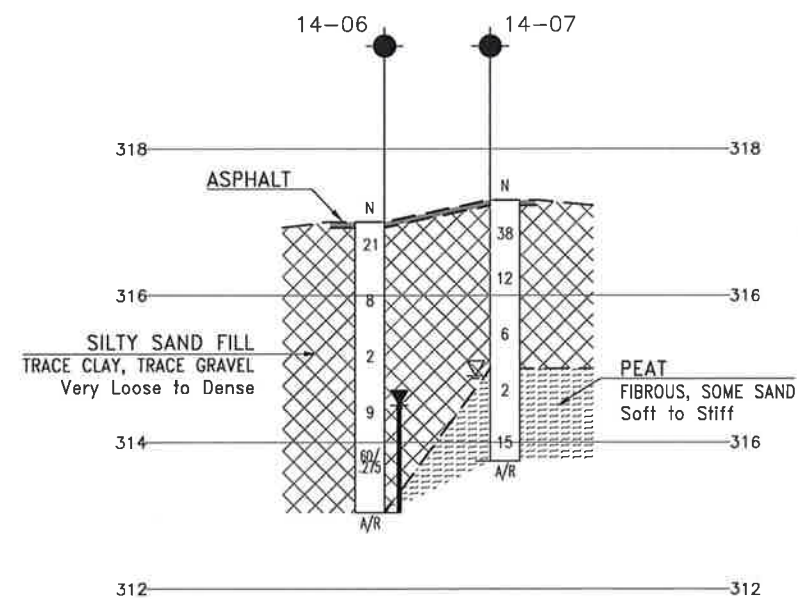
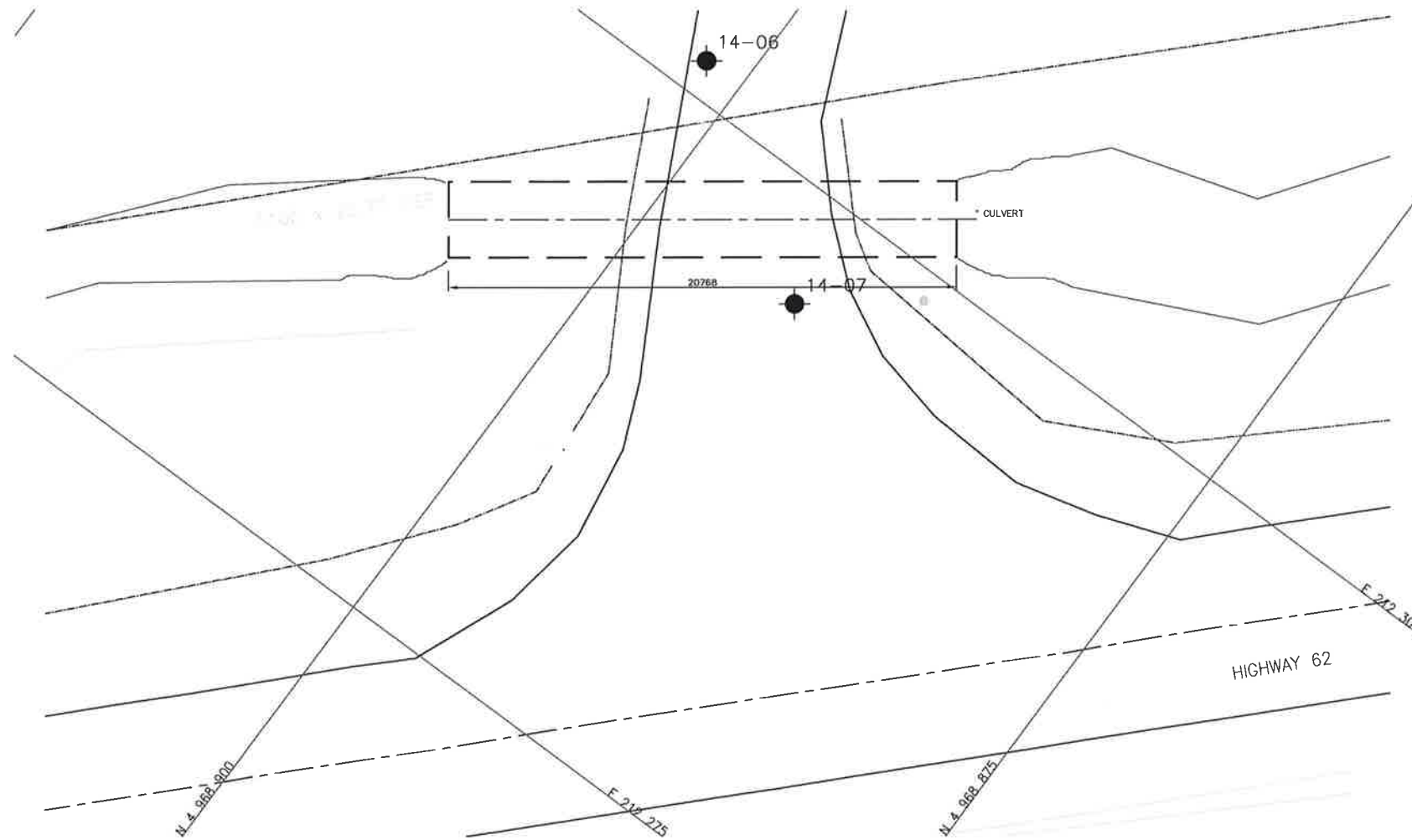
Photograph 3 – Borehole 14-06 on North Side of Sarafians Side Road.



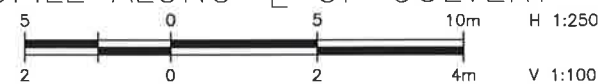
Photograph 4 – Borehole 14-07 on South Side of Sarafians Side Road.

Appendix D

Borehole Location and Soil Strata Drawing



PROFILE ALONG CL OF CULVERT



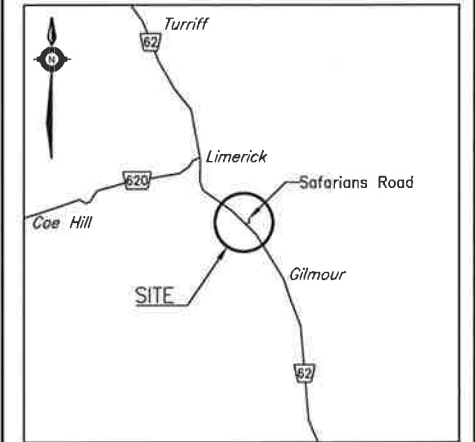
METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

CONT No
WP No 4009-13-01

SARAFIANS SIDE ROAD
CULVERT
BOREHOLE LOCATIONS AND SOIL STRATA

AECOM

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
14-06	317.0	4 968 901.7	212 301.5
14-07	317.3	4 968 892.9	212 295.6

-NOTES-

- The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.
- This drawing is for subsurface information only. Surface details and features are for conceptual illustration.

GEOCRES No. 31C-237



REVISIONS	DATE	BY	DESCRIPTION
DESIGN	AEG	CHK PKC	CODE
DRAWN	MFA	CHK AEG	SITE 11-416/C/STRUCT
DATE	SEP 2015	DATE	SEP 2015
DWG	1		

Appendix E

Comparison of Culvert Foundation Alternatives

COMPARISON OF CULVERT FOUNDATION ALTERNATIVES

Pipe Culvert - Concrete	Pipe Culvert - CSP or HDPE	Concrete Rigid Box Culvert	Concrete Open Footing Culvert
<p>Advantages:</p> <ul style="list-style-type: none"> i. Relatively simple installation/construction ii. Lower cost than a concrete rigid frame culvert <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Feasibility depends on flow capacity and other hydraulic properties ii. May require deeper excavation than for box culvert <p>FEASIBLE</p>	<p>Advantages:</p> <ul style="list-style-type: none"> i. Relatively simple installation/construction ii. Can tolerate larger magnitude of settlement than a concrete pipe or rigid frame culvert iii. Lower cost than a concrete pipe or rigid frame culvert <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Feasibility depends on flow capacity and other hydraulic properties ii. CSP and HDPE pipes not as durable as a concrete culvert iii. May require deeper excavation than for box culvert <p>FEASIBLE</p>	<p>Advantages:</p> <ul style="list-style-type: none"> i. Relatively rapid installation when precast units are used ii. Minimizes depth of excavation and duration of construction/dewatering iii. Can tolerate larger magnitude of differential settlements than open footing culvert, when precast units are used iv. Smaller magnitude of settlement than an open footing culvert due to lower stress on subgrade <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Some risk of disturbance of the founding stratum during construction. <p>PREFERRED</p>	<p>Advantages:</p> <ul style="list-style-type: none"> i. Relatively rapid installation if precast units used ii. Might better satisfy fisheries requirements, if applicable <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Greater risk of disturbance of the founding stratum during construction than for box culvert. ii. May require longer construction duration. iii. Potential for more elaborate dewatering system than for the box culvert. <p>FEASIBLE</p>