



**THURBER** ENGINEERING LTD.

**FINAL  
FOUNDATION INVESTIGATION AND DESIGN REPORT  
HIGHWAY 60 COCHRANE CREEK CULVERT REPLACEMENT  
4.5 KM WEST OF GOLDEN LAKE, RENFREW COUNTY  
SITE NO.: 29-238C**

**DB 2014-4039**

Geocres No.: 31F-190

Report to:

**Ainley & Associates Limited**

March 3, 2016  
File: 19-6748-5

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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 replacement of the Cochrane Creek Culvert crossing Highway 60, approximately 4.5 km west of the community of Golden Lake, in the Township of North Algona/Wilberforce, Renfrew County. The proposed culvert replacement is being constructed under a Ministry of Transportation Design-Build procurement. Thurber carried out the current investigation as a sub-consultant to Ainley and Associates Limited (Ainley) under Contract No. DB 2014-4039.

A model of the subsurface conditions was developed from the data obtained from a previous foundation investigation report that has been provided to Thurber Engineering Ltd. (Thurber) as follows:

“Preliminary Foundation Investigation and Design, Proposed Culvert Replacement, Site No. 29-238c, Cochrane Creek Culvert, Highway 60, Renfrew County, Ontario, W.P. 4170-11-01 (GEOCRE 31F-182), dated March 2014, prepared by Golder Associates Ltd. (Golder)

This report in its entirety is included in Appendix B. The Record of Borehole sheets for the four boreholes (13-1 through 13-4) from this report have been incorporated into the Borehole Location and Soil Strata drawing included in Appendix A.

**2 SITE DESCRIPTION**

The existing culvert is located at Sta. 14+391 on Highway 60 and consists of a 3.810 m wide by 2.565 m high Structural Plate Corrugated Steel Pipe Arch (SPCSPA) culvert in an approximate northeast to southwest alignment. At the location of the culvert, Highway 60 is a two-lane highway with a rural cross-section travelling around the north shore of Mundts Bay between the communities of Golden Lake and Deacon. The flow through the culvert is from north to south and Cochrane Creek ultimately drains into Golden Lake to the south. The road surface of the Highway 60 embankment is approximately 2 m above the top of the culvert at an elevation of 173.0 m with embankment toe ditches at approximate elevation 170.4 and 169.8 m for the north and south ditch lines, respectively. Steel beam guiderails are present on both sides of Highway 60 at the culvert site.

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A single dwelling with outbuildings is located directly to the west of the culvert. The land to the south and east of the culvert is heavily vegetated with trees and open farm land exists north of Highway 60 right-of-way.

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

### **3 SITE INVESTIGATION AND FIELD TESTING**

The site investigation and field testing program was previously carried out by others in May 2013 and consisted of drilling and sampling four boreholes (identified as 13-01 and 13-04). Boreholes 13-01 and 02 were advanced with a truck mounted drill rig from the pavement level of Highway 60 and borehole 13-03 and 04 were advanced with portable drilling equipment near the culvert inlet and outlet, respectively. The boreholes were extended to depths ranging from 10.4 to 14.3 m below the ground surface. A single standpipe piezometer was installed in Borehole 13-03. The description of the drilling equipment and procedure, sampling, in-situ testing and standpipe piezometer installation along with laboratory testing and the Record of Borehole sheets are included in the preliminary foundation investigation report noted above and included in Appendix B.

The approximate borehole locations are shown on the Borehole Locations and Soil Strata Drawing included in Appendix A. The coordinates and elevation of the boreholes are provided on the drawing and on the individual Record of Borehole sheets.

### **4 LABORATORY TESTING**

The recovered soil samples were subjected to visual identification and to natural moisture content determination. Selected samples were also subjected to Atterberg Limit testing and gradation analysis (hydrometer and/or sieve). The results of these tests are summarized in the preliminary foundation investigation report noted above and 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 B and the Borehole Location and Soil Strata drawing included in Appendix A. A general description of the stratigraphy, based on the conditions encountered in the boreholes, is given in the preliminary foundation investigation report noted above. However, the factual data presented on the Record of Borehole sheets takes precedence over this general description for interpretation of the site conditions. It must be recognized that the soil and groundwater conditions may vary between and beyond borehole locations.

In general terms, the site was found to be underlain by a pavement structure and embankment fill overlying a deposit of predominantly silt. Adjacent to the embankment footprint, an silt and sand layer with organics was encountered surficially which was underlain by a localized layer of sand over silt.

The pavement structure consisted of a layer of asphaltic concrete ranging from 100 to 200 mm in thickness underlain by a 400 to 500 mm thick layer of sand base/subbase. The pavement structure was further underlain by a 4.5 to 4.6 m thick layer of granular fill ranging

in composition from sand and gravel to gravelly sand to sand some gravel to sand some silt varying from very loose to dense consistency. The measured moisture content of the fill ranged from 3 to 33%. The granular fill was noted to contain cobbles and boulders as inferred from drilling observations and SPT refusals. A 100 mm thick organic layer was encountered directly below the granular fill in Borehole 13-1.

The boreholes located adjacent to the embankment alignment encountered a very loose 600 mm thick organic layer varying from clayey sand with silt to sandy silt. The measured moisture content of the organic layer ranged from 32 to 40%. The organic layer was underlain by a 0.5 to 1.7 m thick deposit of very loose to compact silty sand to sand with a recorded moisture content of 68%. The sand layer contained trace wood and organic matter.

Underlying all the soil layers described above were cohesionless deposits varying from sandy silt to silt some sand to silt to silt some clay. The predominantly silt layers ranged in consistency from very loose to compact to the maximum depth of investigation (Elev. 158.6 m). The recorded moisture content of these layers typically ranged from 20 to 48%.

A single standpipe piezometer was installed by others in Borehole 13-3 to monitor groundwater levels after drilling as part of the preliminary foundation investigation report. The measured groundwater levels are summarized in the table below.

**Table 5-1. Measured Groundwater Levels**

Borehole	Date	Groundwater Level		Comment
		Depth (m)	Elevation (m)	
13-3	Jul. 7, 2013	- 0.4 <sup>(*)</sup>	170.8	Piezometer
	Aug. 13, 2015	0.5	169.9	Piezometer

*Note: (\*) Groundwater level in the piezometer was measured above the existing ground surface (artesian flow conditions) as reported in Geocres 31F-182*

It should be noted that the values shown may not reflect groundwater levels at the time of construction 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.

## 6 MISCELLANEOUS

Borehole locations, ground surface elevations, drilling, sampling, in-situ testing, standpipe piezometer installation, field supervision and geotechnical laboratory testing were completed by Golder as part of the preliminary foundation investigation report.

A site inspection was completed by Mr. Stephen Peters, P.Eng. Interpretation of the factual data and preparation of this report were carried out by Mr. Stephen Peters, P.Eng. The report was reviewed by Dr. Fred Griffiths, P.Eng and Dr. P.K. Chatterji, P.Eng a Designated Principal Contact for MTO Foundation Projects.

Thurber Engineering Ltd.  
Report Prepared By:



Stephen Peters, P.Eng.  
Geotechnical Engineer



Fred Griffiths, P.Eng, Ph.D.  
Associate, Senior Geotechnical Engineer



P.K. Chatterji, P.Eng., Ph.D.  
Review Principal, Senior Engineer

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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 also presents geotechnical recommendations for the replacement of the existing Cochrane Creek culvert crossing Highway 60, located approximately 4.5 km west of the community of Golden Lake, in the Township of North Algona/Wilberforce, Renfrew County. The plans and profiles used for preparation of this report were provided by Ainley and Associates Limited.

The existing culvert, conveying Cochrane Creek under Highway 60, is a Structural Plate Corrugated Steel Pipe Arch (SPCSPA) culvert which is 3.810 m wide by 2.565 m high and approximately 29 m long. The embankment fill height above the culvert is in the order of 2 m high. The creek flows in a north to south direction at this site and drains into Golden Lake.

The discussion and recommendations presented in this report are based on the factual data obtained from a preliminary foundation investigation report completed by others. Select photographs showing the existing conditions of the culvert area are included in Appendix C for reference.

**8 CULVERT FOUNDATIONS**

**8.1 General**

The proposed culvert structure, as shown on the General Arrangement (GA) drawing dated July 2015, is a precast segmental box culvert with a span of 6 m and a rise of 3 m. The inverts of the culvert are shown at Elev. 168.45 and 168.15 m for the inlet and outlet, respectively and the finished road grade is shown at Elev. 173.0 m.

The replacement culvert is proposed to be constructed along the same alignment of the existing culvert and a temporary flow passage, shown to be constructed to the east of the existing culvert alignment, will be utilized during the staged culvert replacement. Widening of the existing embankment to the north of the existing Highway 60 alignment is also proposed to accommodate the construction staging requirements.

**8.2 Foundation Alternative**

In general terms, the site was found to be underlain by pavement structure and embankment fill overlying a deposit of predominantly silt. Adjacent to the embankment footprint, an organic layer was encountered surficially which was further underlain by a

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localized layer of sand. The most recent groundwater level measurement in the standpipe piezometer was recorded at Elev. 169.9 m.

Selection of the culvert type must consider the proposed construction procedures, staging requirement, geotechnical resistance available in the foundation soils, the depth to suitable bearing stratum and post-construction settlement criteria. From a geotechnical perspective, the following culvert types were considered:

- Concrete Open Footing Culvert  
Concrete, open footing culverts are not recommended for this site from a foundation engineering perspective since the available geotechnical resistance will be low and the post construction settlement from this type of culvert would be greater than alternative options and would also require greater dewatering efforts during construction to place the foundation in the dry.
- Circular Pipes (Concrete, HDPE)  
From a foundation engineering perspective, pipe culverts are a technically feasible alternative, provided that other design issues including flow capacity, hydraulic properties and durability can be satisfied.
- Concrete Box (Closed) Culvert  
Given the subsurface conditions and the anticipated construction sequencing, the proposed precast segmental box culvert is the preferred option from a foundation engineering standpoint. Precast sections, rather than cast-in-place construction, can be installed rapidly with less potential for disturbance of the founding soils during installation.

A comparison of these alternatives, based on their respective advantages and disadvantages, is included in Appendix D. It is not considered to be economical or practical to support a culvert on deep foundations at this site and therefore this option is not presented in this report. This report will focus on providing foundation recommendations on the design and construction of a box culvert.

### **8.3 Foundation Design for Box Culverts**

Foundation design aspects for the replacement culvert includes subgrade conditions, geotechnical resistances, settlements of founding soils, lateral earth pressures, erosion control, protection system design and groundwater control, staged excavation and stability of widening detour embankment.

#### **8.3.1 Geotechnical Resistances**

The culvert must be designed to resist loadings including frost forces, 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.

Since the replacement culvert will be constructed on the same alignment as the existing culvert and there is negligible grade raise proposed, it is anticipated that the subgrade soils within the culvert footprint will not be subjected to any significant additional loading.

In order to provide a more uniform foundation subgrade condition, a minimum 300 mm thick layer of bedding material conforming to OPSS 1010 Granular A requirements must be provided under the base of the box culvert as per OPSS 422 and OPSD 803.010.

The recommended geotechnical resistances along the existing culvert alignment at the founding elevation are as follows:

- Factored Geotechnical Resistance at ULS of 175 kPa
- Geotechnical Resistance as SLS of 100 kPa

The bearing resistance values are for vertical, concentric loading. In the case of eccentric or inclined loading, the bearing resistance must be reduced in accordance with CHBDC Clause 6.7.3 and Clause 6.7.4.

Resistance to lateral forces/sliding resistance between the precast concrete and the underlying Granular A should be evaluated in accordance with the CHBDC assuming an ultimate coefficient of friction of 0.45.

### 8.3.2 Settlement

It is understood that there is no grade raise at this site and the existing culvert is to be replaced with the new culvert along the same alignment. Taking into consideration the proposed conceptual construction sequencing for this site, it is anticipated that rebound of the subgrade after removal of the existing culvert and the surrounding fill will also be minimal. The foundation settlement based on the supplied SLS resistance is expected to be less than 25 mm and is anticipated to be substantially completed by the end of construction.

### 8.3.3 Subgrade Preparation

After excavation and removal of the existing culvert and existing fill, all organics, soft or loose creek bed deposits, disturbed soils and deleterious materials must be stripped from the footprint of the foundation to expose competent native subgrade material at or below the desired founding elevations. The exposed subgrade must be inspected to confirm that the subgrade is suitable and uniformly competent. Any soft areas should be sub-excavated and backfilled with well compacted granular soil.

Given the loose conditions anticipated at the founding level of the replacement culvert, construction equipment should not be permitted to travel on the exposed subgrade. In addition, without dewatering prior to excavating to the founding elevation the compaction of granular bedding directly above the subgrade is likely to result in disturbance of the material with pumping of fines into the granular bedding and difficulty achieving the specified degree of compaction. Where saturated subgrade soils are exposed, protection of the subgrade should include over excavation to allow placement of a mud slab 100 mm thick beneath the 300 mm thick Granular A bedding layer.

Culvert construction and subgrade preparation must be carried out in the dry.

#### 8.3.4 Frost Depth

The depth of frost penetration at this site is 1.8 m. It is not necessary to found a box culvert at a depth below frost penetration however, frost treatment for a box culvert should be as per OPSD 803.010.

### 9 CULVERT BACKFILL AND EARTH PRESSURE

It is recommended that backfill to the culvert consist of free-draining, non-frost susceptible granular materials such as Granular A or Granular B Type II material meeting the requirements of OPSS.PROV 1010. The backfill must be in accordance with OPSS 902 and placed to the extent shown on OPSD 3101.150.

The backfill should be compacted in regular lifts. Heavy compaction equipment, used adjacent to structure, must be restricted in accordance with OPSS 501. The top of the backfill elevation should be within 400 mm on both sides of the culvert at all times. Care must be exercised when compacting the fill above the culvert in order not to damage the culvert.

Earth pressures acting on the culvert may be assumed to be triangular and to be governed by the characteristics of the backfill. For a fully drained condition, the pressures should be computed in accordance with the CHBDC but generally are given by the following expression:

$$p_h = K (\gamma h + q) \quad (\text{kN/m}^2)$$

where:

$p_h$	=	horizontal pressure on the wall at depth h (kPa)
$K$	=	earth pressure coefficient (see table below)
$\gamma$	=	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)

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 the fill and decreasing to 0 kPa at a depth of 2.0 m for Granular B Type I or 1.7 m for Granular A or Granular B Type II.

Earth pressure coefficients for backfill to the culvert wall are dependent on the material used as backfill and the inclination of the ground surface behind the wall. Typical values 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 $\phi = 35^\circ, \gamma = 22.8 \text{ kN/m}^3$		Existing Fill or 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)	Horizontal Surface Behind Wall	Slope Surface Behind Wall (2H:1V)
Active, $K_A$ (Unrestrained Wall)	0.27	0.40	0.31	0.48
At Rest, $K_o$ (Restrained Wall)	0.43	-	0.47	-
Passive, $K_P$ (Movement towards Soil Mass)	3.7	-	3.3	-
Soil Group(*)	"medium dense sand"		"loose to medium dense 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 pressures should be used for any wingwalls 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.

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.

## 10 SEISMIC CONSIDERATIONS

The following seismic parameters should be used for design:

Velocity Related Seismic Zone	1
Zonal Velocity Ratio	0.05
Acceleration Related Seismic Zone	3
Zonal Acceleration Ratio	0.15
Peak Horizontal Acceleration	0.1g

The soil profile type at this site has been classified as Type IV. Therefore, according to Clause 4.4.6 of the CHBDC, a Site Coefficient "S" (ground motion amplification factor) of 2.0 should be used in seismic design.

In accordance with Clause 4.6.4 of the CHBDC, retaining structures should be designed using active ( $K_{AE}$ ) and passive ( $K_{PE}$ ) earth pressure coefficients that incorporate the effects

of earthquake loading. The coefficients of horizontal earth pressure for seismic loading presented in Table 10-1 may be used.

**Table 10-1. Earth Pressure Coefficients for Earthquake Loading**

Condition	Earth Pressure Coefficient (K)			
	OPSS Granular A or OPSS Granular B Type II $\phi = 35^\circ, \gamma = 22.8 \text{ kN/m}^3$		Existing Fill or 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)	Horizontal Surface Behind Wall	Slope Surface Behind Wall (2H:1V)
Active, $K_{AE}$	0.30	0.47	0.34	0.58
At Rest, $K_{OE}$	0.53	-	0.58	-
Passive, $K_{PE}$	3.6	-	3.2	-

The potential for liquefaction of the foundation soils has been assessed comparing Cyclic Stress Ratio to Cyclic Resistance Ratio generated from the SPT N-values. Using this method the results indicate that an adequate Factor of Safety against liquefaction under earthquake loading exists for this site using the site specific PGA value. Some toe failure may occur but it is expected to be of limited nature and readily repairable.

## 11 SCOUR PROTECTION AND EROSION CONTROL

Scour and erosion protection should be provided for the culvert inlet and outlet areas. 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.

Typically, rock protection should be provided over all earth surfaces with which stream flow is likely to be in contact. Treatment at the outlet should be in accordance with OPSS 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.

It is recommended that a clay seal or a concrete cut-off wall be used to minimize the potential for piping and erosion around the inlet of the culvert. The clay seal must extend to the order of 300 mm above the high water level and laterally for the width of the granular material, and have a minimum thickness of 500 mm. The material requirements should be in accordance with OPSS 1205. Geosynthetic clay liner may be used as a clay seal.

## 12 EXCAVATION AND GROUNDWATER CONTROL

### 12.1 Excavations

All excavation must be carried out in accordance with the Occupational Health and Safety Act (OHSA). For the purposes of OHSA, the fill and native soils above the water table may be classified as Type 3 soil. The organics soils, alluvial deposits as well as the native soils below the water table are classified as 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 sand and silt deposits. The sides of temporary excavations must be sloped in accordance with the requirement of the OHSA. At locations where there is space restrictions or where a slope has to be retained, the excavations will need to be carried out within a protection system. Any protection system must be designed by a licensed Professional Engineer experienced in such design. Further discussion is presented in Section 14 below.

## 12.2 Surface and Groundwater Control

Culvert construction and subgrade preparation must be carried out in the dry. A temporary flow passage must be constructed adjacent to the proposed culvert alignment to convey creek flow around the construction site. Construction of a cofferdams will be required to divert the creek flow away from the culvert foundation.

Based on the preliminary GA, excavation below the groundwater level to construct the culvert foundation will be required. The culvert subgrade will be formed in the native loose to compact silt under a head of 2 to 3 m. Excavation below the groundwater level without prior dewatering is not recommended since the inflow of groundwater will cause boiling and sloughing of the soil below the water level, making it difficult to maintain a dry, sound base on which to work. Temporary groundwater and surface water control measures will be required to remain operational during construction until the culvert is installed and backfilled. Dewatering systems must be designed by a dewatering specialist.

Based on the groundwater and soil conditions, special attention must be paid to construction dewatering. It is recommended that the excavation be enclosed within a water tight sheet pile enclosure driven to cut off flow through the sand and silt layers. The groundwater level within the enclosure should be lowered by pumping from vacuum well points prior to excavation to a minimum of 500 mm below the underside of the final subgrade. As indicated in Section 8.3.3, a mud slab can be poured with lean mix concrete to protect the exposed saturated subgrade surface from disturbance.

A Permit to Take Water (PTTW) is required where pumping rates exceed 50,000 L/day. Further assessment of dewatering requirements and the need for a PTTW should be carried out by specialists experienced in this field. Hydraulic conductivities that can be used in analysis for dewatering requirements are given below:

Material	Type of Analysis	Hydraulic Conductivity (m/s)
Existing Fill	Particle size	$1 \times 10^{-5}$
Native Sand to Silty Sand	Particle size	$1 \times 10^{-4}$
Native Silts and Sandy Silt	Particle size	$1 \times 10^{-6}$

## **13 EMBANKMENT DESIGN AND CONSTRUCTION**

### **13.1 Culvert Replacement**

Embankment reconstruction after culvert replacement should be carried out in accordance with OPSS 206. The embankments should be reinstated at side slopes of 2H:1V (or flatter) if constructed using Select Subgrade Material (SSM) or Granular B Type I. Where new embankment fill is placed against existing embankment slopes or on a sloping ground surface, benching of the existing fill should be carried out in accordance with OPSD 208.010. Embankment slopes must be provided with erosion protection in accordance with OPSS 804. Provided proper construction methods are used, no long term or global stability issues are anticipated for approach embankments built at this site.

The magnitude of the embankment compression in embankments with granular materials due to compression of the compacted fill is in the order of 0.5% of the embankment height and is expected to be completed after completion of fill placement.

### **13.2 Embankment Widening for Detour**

Widening of the existing highway embankment on the north side to accommodate a temporary traffic detour lane will require placement and compaction of granular fill. In embankment widening areas, all organic, soft and other deleterious materials must be stripped from the footprint of the foundation. Where applicable, benching of the existing earth slope surface should be carried out as per OPSD 208.010.

As the new fill is placed on the existing embankment slope, it is anticipated that the settlement due to elastic compression of the underlying native soils will take place. It is anticipated that the wedge of new fill would induce foundation settlement in the order of 15 to 25 mm. This settlement is considered short term settlement and is expected to be completed by the end of construction.

## **14 ROADWAY PROTECTION**

Roadway protection will be required during various stages of construction. Roadway protection must be implemented in accordance with OPSS 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.

Lateral earth pressure coefficients that can be used in design for the embankment fill and culvert backfill are provided in Table 9-1. The lateral earth pressure coefficients for the native silty soils are given below:

$\gamma_w$	=	10	(kN/m <sup>3</sup> , unit weight of water)
$\gamma$	=	19	(kN/m <sup>3</sup> , bulk unit weight of soil)
$K_A$	=	0.36	
$K_P$	=	2.8	

The designer of the roadway protection system should ensure the penetration depth is sufficient to provide base fixity. All shoring should be designed by a licensed Professional Engineer experienced in such designs.

## **15 CONSTRUCTION CONCERNS**

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

- Excavation below the water level, where required, will involve lowering the groundwater level below the excavation base to maintain a reasonably dry excavation and stable side slopes. The dewatering scheme will be critical for culvert construction at this site.
- Disturbance of the soil subgrade within the culvert foundation footprint
- Cobbles or other buried obstructions may be encountered during excavation in the existing embankment fill or interfere with driving of sheet piles.
- Water levels in the creek may fluctuate
- The Contractor's selection of construction equipment and methodology must include assessment of the capability of the existing embankment to support the proposed construction equipment and any temporary structure fill (i.e., as a pad for crane support).

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



## 16 CLOSURE

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

Thurber Engineering Ltd.  
Report Prepared By:



Stephen Peters, P.Eng.  
Geotechnical Engineer



Fred Griffiths, P.Eng, Ph.D.  
Associate, Senior Geotechnical Engineer



P.K. Chatterji, P.Eng., Ph.D.  
Review Principal, Senior Engineer

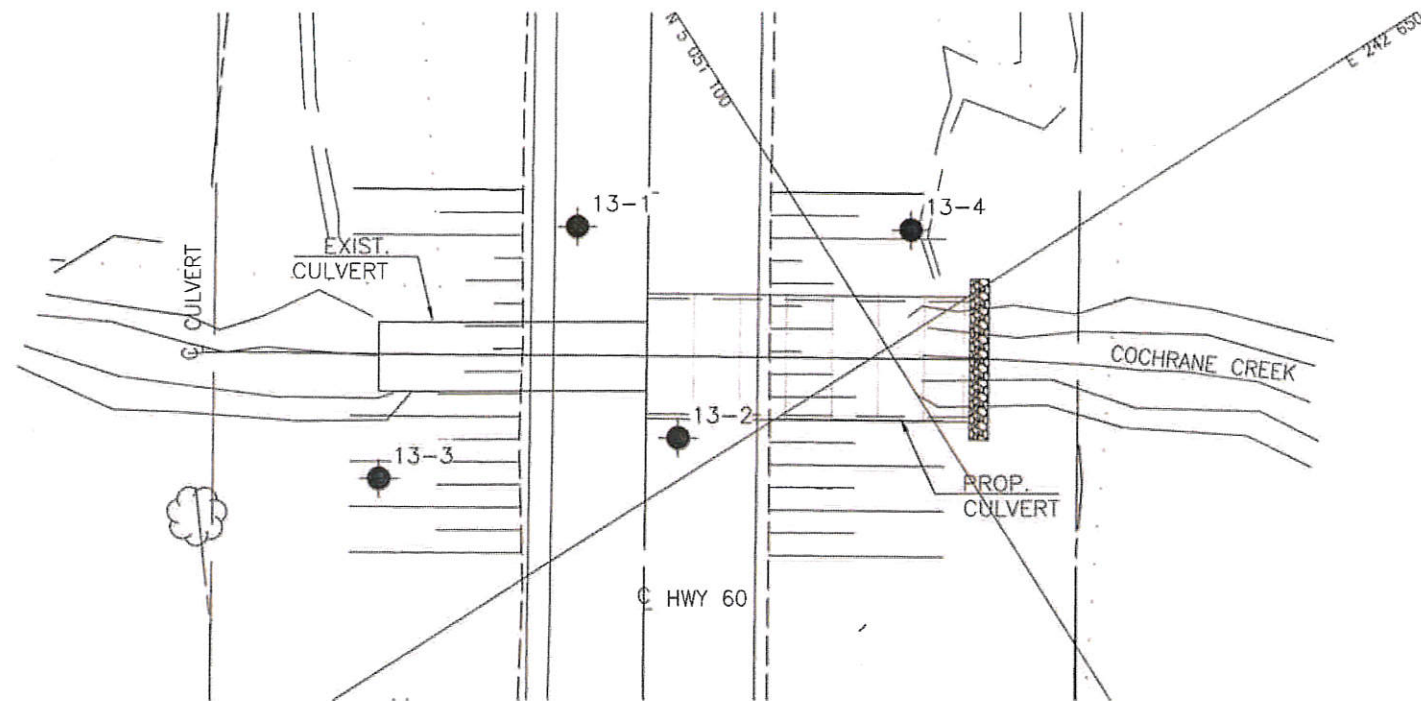
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**Appendix A.**

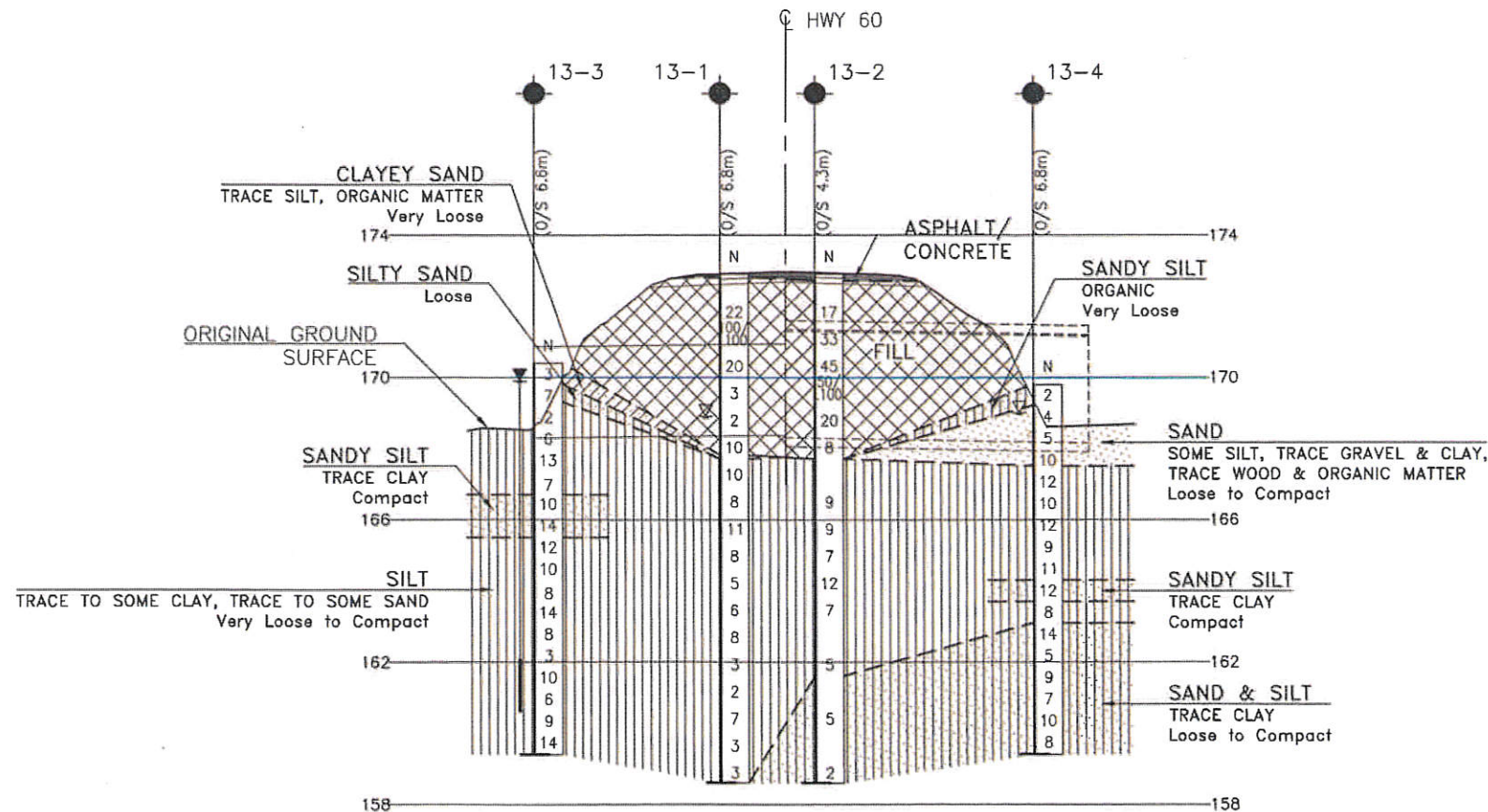
**Borehole Location Plan and Stratigraphic Drawings**



14+400



PLAN



PROFILE ALONG CULVERT



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

CONT No  
WP No 4170-11-01

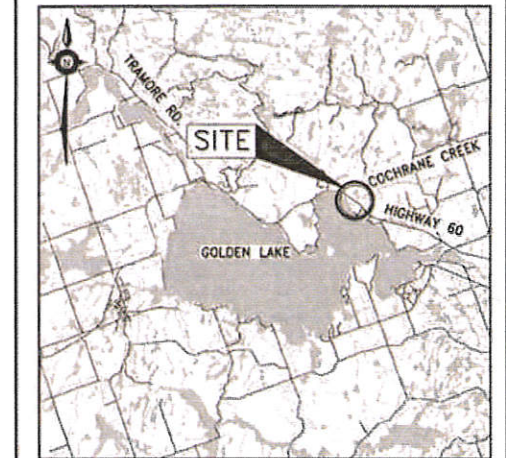
CULVERT REPLACEMENT  
COCHRANE CREEK CULVERT  
AT HIGHWAY 60  
BOREHOLE LOCATIONS AND SOIL STRATA



SHEET



THURBER ENGINEERING LTD.



KEYPLAN

LEGEND

●	Borehole (Geocres 31F-182)
⊕	Borehole and Cone
N	Blows /0.3m (Std Pen Test, 475J/blow)
CONE	Blows /0.3m (60° Cone, 475J/blow)
PH	Pressure, Hydraulic
W	Water Level
HA	Head Artesian Water
P	Piezometer
90%	Rock Quality Designation (RQD)
A/R	Auger Refusal

NO	ELEVATION	NORTHING	EASTING
13-1	172.9	5 051 110.4	242 664.2
13-2	172.9	5 051 111.9	242 651.9
13-3	170.4	5 051 126.4	242 658.6
13-4	169.8	5 051 095.6	242 654.6

-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. 31F-190



REVISIONS	DATE	BY	DESCRIPTION
DESIGN	SP	CHK	SP
DRAWN	AN	CHK	SITE
			LOAD
			STRUCT
			DWG 1
			DATE MAR 2016

**Appendix B.**

**Preliminary Foundation Investigation Report**

March 2014

REPORT ON

Preliminary Foundation Investigation and Design  
Proposed Culvert Replacement  
Site No. 29-238c, Cochrane Creek Culvert  
Highway 60, Renfrew County, Ontario  
W.P. 4170-11-01

Submitted to:  
Dillon Consulting Limited  
130 Dufferin Avenue, Suite 1400  
London, Ontario  
N6A 5R2

Geocres Number: 31F-182

Report Number: 12-1121-0193-1315

Distribution:

- 3 copies - Ministry of Transportation, Ontario, Kingston
- 1 copy - Ministry of Transportation, Ontario, Downsview
- 2 copies - Dillon Consulting Limited
- 2 copies - Golder Associates Ltd.



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Record of Borehole Sheets



**PART A**

**PRELIMINARY FOUNDATION INVESTIGATION  
PROPOSED CULVERT REPLACEMENT  
SITE NO. 29-238C  
COCHRANE CREEK, HIGHWAY 60  
RENFREW COUNTY, ONTARIO  
W.P. 4170-11-01**



## 1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by Dillon Consulting Limited (Dillon) on behalf of the Ministry of Transportation, Ontario (MTO) to carry out foundation investigations associated with numerous culvert replacements and rehabilitations and bridge rehabilitations at various locations in the Eastern Region of Ontario as part of the 23 Structures MEGA 3 project.

This report presents the results of the preliminary foundation investigation conducted for the replacement of a structural culvert located at Site No 29-238c, which is constructed at the crossing of Cochrane Creek and Highway 60, in Renfrew County, Ontario (WP 4170-11-01).

The purpose of the preliminary foundation investigation was to assess the subsurface conditions for the proposed culvert replacement by drilling 4 boreholes and carrying out in-situ testing and laboratory testing on selected samples.

The terms of reference for the original scope of work are outlined in the MTO's Request for Proposal (RFP) dated August 2012.

The work was carried out in accordance with Golder's Quality Control Plan dated December 2012.



## 2.0 SITE DESCRIPTION

The existing culvert (Site No. 29-238c) is located at the crossing of Cochrane Creek and Highway 60, about 1.4 km west of Berndt Road in Renfrew County, Ontario. The existing culvert is located at about Station 14+390.

The existing culvert is a 3.660 m wide by 2.4 m high structural plate corrugated steel pipe arch culvert which is about 29 m in length. The date of construction of the culvert is unknown. The existing culvert invert is at about Elevation 168.5 m and Cochrane Creek ultimately drains into Golden Lake to the south. The depth of water within the culvert was about 1.2 m at the time of the field investigation.

The existing pavement grade at the culvert location is at about Elevation 173 m. In this area, Highway 60 is one lane wide in each direction (i.e., 2-lane highway). The existing embankment slopes at the culvert locations are up to about 4 m in height and are sloped at about 3H:1V. Based on visual observations at the time of the field investigation, the embankment side slopes are in good condition with no evidence of instability.

It is understood that the new culvert will consist of a one-cell concrete box culvert with an internal span opening of 6.0 m. The rise will be 2.9 m with approximately 0.3 m of the culvert embedded below the stream bed. The invert level will be at about Elevation 168.5 m at the centreline. The new culvert will be slightly longer than the existing culvert, and measure about 34 m in length. Widening of the existing shoulder/rounding will also be required at the culvert site to accommodate detour traffic. The embankments will be constructed with side slopes of 3H:1V for the upper portion of the embankment, and 2H:1V for the lower portions of the side slopes.

It is understood that the culvert replacement will be carried out in stages. It is understood that the Stage 1 will involve shifting a single lane of traffic to the north (partially onto the shoulder) and removing/replacing the south half of the culvert. Stage 2 will involve shifting a single-lane of traffic to the south onto the reconstructed half and removing/replacing the north half of the culvert. A sheet pile enclosure will be provided around the perimeter of the excavation area(s).

A pavement condition survey undertaken along Highway 60 at the culvert site by Golder in June 2013 indicated a fairly good pavement condition, with no obvious distortions or distresses that require immediate rehabilitation.



### 3.0 INVESTIGATION PROCEDURES

The subsurface investigation was carried out for the culvert replacement in May 2013, at which time four boreholes (numbered 13-1 to 13-4, inclusive) were advanced at the locations shown on Drawing 1. The boreholes were advanced as follows:

- Boreholes 13-1 and 13-2 were advanced using 108 mm inner diameter (I.D.) continuous-flight hollow-stem augers on a truck-mounted drill rig, supplied and operated by Marathon Drilling Ltd. of Ottawa, Ontario. The boreholes were advanced to depths of about 14.3 m below the existing ground surface in the overburden. Borehole 13-1 initially encountered auger refusal at a depth of approximately 1.6 m; this borehole was relocated approximately 1 m and drilled to its intended depth.
- Boreholes 13-2 and 13-4 were advanced using portable drilling equipment, supplied and operated by OGS Inc. of Almonte, Ontario. The boreholes were advanced to depths of about 11.0 and 10.4 m, respectively, below the existing ground surface in the overburden.

Soil samples in the boreholes were obtained at vertical intervals ranging from about 0.60 m to 1.52 m, using a 50 mm outer diameter (O.D.) split-spoon sampler in accordance with Standard Penetration Test (SPT) procedures.

A standpipe piezometer was installed in Borehole 13-3 to monitor the groundwater level at the site. The standpipe consists of a 32-mm diameter rigid PVC pipe with a 1.5 m long slotted screen section, installed within silica sand backfill and sealed by a section of bentonite pellet backfill. The water level in the standpipe piezometer was measured on July 7, 2013.

The boreholes were backfilled with bentonite pellets, mixed with native soils in the overburden and bentonite pellets in the bedrock. The site conditions were restored following completion of the work. The standpipe piezometer will be decommissioned following construction, unless instructed otherwise by the Ministry.

The field work was supervised throughout by members of Golder's technical staff, who located the boreholes, supervised the drilling, sampling and in-situ testing operations, logged the boreholes, and examined and cared for the soil samples. The samples were identified in the field, placed in appropriate containers, labelled, and transported to Golder's laboratory in Ottawa for further examination. Index and classification tests consisting of grain size distribution, Atterberg limit and water content testing were carried out on selected soil samples. All of the laboratory tests were carried out to MTO and/or ASTM standards as appropriate.

The borehole locations and ground surface elevations were surveyed by Golder Associates. The boreholes locations, including MTM NAD83 northing and easting coordinates, and ground surface elevations referenced to Geodetic datum are summarized in the following table and are shown on Drawing 1.

Borehole Number	Borehole Location	MTM NAD83 Northing (m)	MTM NAD83 Easting (m)	Ground Surface Elevation (m)
13-1	Centre of the culvert alignment	5051110.4	242664.2	172.9
13-2	Centre of the culvert alignment	5051111.9	242651.9	172.9
13-3	North end of the culvert	5051126.4	242658.6	170.4
13-4	South end of the culvert	5051095.6	242654.6	169.8



### 4.0 SITE GEOLOGY AND STRATIGRAPHY

#### 4.1 Regional Geological Conditions

The site is located in the physiographic region known as the Algonquin Highlands, as delineated in The Physiography of Southern Ontario<sup>1</sup>.

The overall topography of the region is gently rolling to moderately rugged with bare rock ridges and shallow till soils which vary greatly over short distances. The vast majority of soil in this region is forested, being non-agricultural mainly because of the rock outcrops and associated shallow soil, rough topography, stones and swamps. Groups of sands and gravel hills with associated outwash sand flats are found in the northern part of Algonquin Park, Round Lake and Golden Lake.

#### 4.2 Site Stratigraphy

As part of the subsurface investigation at this site, four boreholes were advanced along the alignment of the existing culvert. The borehole locations, ground surface elevations and an interpreted stratigraphic profile are shown on Drawing 1.

The detailed subsurface soil and groundwater conditions as encountered in the boreholes advanced during this investigation, together with the results of the in-situ and laboratory tests carried out on selected soil samples, are given on the attached Record of Borehole sheets and on Figures 1 to 5.

The stratigraphic boundaries shown on the Record of Borehole sheets are inferred from non-continuous sampling and, therefore, represent transitions between soil types rather than exact planes of geological change. The subsoil conditions will vary between and beyond the borehole locations.

In general, the subsurface conditions at the locations of the proposed culvert replacement consist of the pavement structure and embankment fill up to 5.2 m depth (under the roadway) overlying a deposit of silt.

A more detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

##### 4.2.1 Pavement Structure and Embankment Fill

The pavement structure of the travelled lanes of the highway was penetrated within the westbound lane at Borehole 13-1 and the eastbound lane at Borehole 13-2. At the borehole locations, the pavement structure consists of about 0.1 m to 0.2 m of asphaltic concrete over 0.4 to 0.5 m of sand base.

The measured water content of one sample of the sand base was approximately 4 percent.

The sand base is underlain by about 4.5 to 4.6 m of embankment fill. The embankment fill generally consists of sand with varying amounts of gravel and silt, and with the presence of cobbles and boulders inferred from drilling observations (e.g., difficult drilling, grinding of augers) as well as split spoon and auger refusal.

The embankment fill was fully penetrated to depths of about 5.1 and 5.2 m below the ground surface (Elevations 167.8 and 167.7 m) at Boreholes 13-1 and 13-2, respectively.

<sup>1</sup> Chapman, L.J. and D.F. Putnam. *The Physiography of Southern Ontario*. Ontario Geological Survey Special Volume 2, Third Edition, 1984. Accompanied by Map P.2715, Scale 1:600,000.





Standard penetration test N values for the upper 3 to 4 m of embankment fill range from 17 blows per 0.3 m of penetration to greater than 50 blows per 0.1 m of penetration, indicating a compact to dense state of packing. Standard penetration test N values for the lower 1 to 2 m of embankment fill range from 2 to 10 blows per 0.3 m of penetration, indicating a very loose to loose state of packing.

The results of grain size distribution testing carried out on four samples of the embankment fill are provided on Figure 1. The measured water contents of seven samples of the embankment fill ranged from approximately 3 to 33 percent.

#### 4.2.2 Topsoil and Peat

A clayey sand to sandy silt topsoil layer was encountered at the ground surface at Boreholes 13-3 and 13-4, with a thickness of about 0.6 m.

Standard penetration test 'N' values for the topsoil were measured to be about 2 and 3 blows per 0.3 m of penetration indicating a very loose state of packing.

The measured water contents of two samples of the topsoil were approximately 32 and 40 percent.

A buried peat layer was also encountered beneath the embankment fill at borehole 13-1, with a thickness of about 0.1 m.

#### 4.2.3 Sand and Silty Sand

A deposit of sand to silty sand containing organic matter was encountered beneath the topsoil at Boreholes 13-3 and 13-4. The sand and silty sand in these boreholes has a thickness of about 0.5 and 1.7 m, respectively, and extends to elevations of 169.3 and 167.5 m, respectively.

Standard penetration test 'N' values measured for this material range from about 4 to 10 blows per 0.3 m of penetration indicating a very loose to compact state of packing.

The results of grain size distribution testing carried out on one sample of the sand are provided on Figure 2. The measured natural water content of one sample of the sand was about 68 percent.

#### 4.2.4 Silt

The embankment fill in Boreholes 13-1 and 13-2 and the sand/silty sand deposit in Boreholes 13-3 and 13-4 are underlain by a thick deposit of silt with varying amounts of sand and clay. Occasional layers of sandy silt were also encountered within the silt deposit in Boreholes 13-3 and 13-4. The silt deposit was not fully penetrated within the boreholes but was proven to a depth of about 14.3 m below the ground surface (Elevation of 158.6 m).

The measured SPT 'N' values within the silt and sandy silt range from about 2 to 14 blows per 0.3 m of penetration which indicate a very loose to compact state of packing.

The results of grain size distribution testing carried out on 12 samples of the silt are provided on Figure 3. The measured natural water content of the silt ranges from 20 to 48 percent.

The results of grain size distribution testing carried out on one sample of the sandy silt are provided on Figure 4. The measured natural water content of one sample of the sandy silt was 24 percent.

The results of Atterberg Limits testing on one sample of the silt gave a plasticity index of about 1 percent and a liquid limit of about 21 percent, as shown on Figure 5, which indicates a silt of non-plastic to low plasticity.



#### 4.2.5 Groundwater Conditions

The water level in the creek in July 2013 was indicated to be at about Elevation 169.6 m. The groundwater level in the piezometer in Borehole 13-3 was measured on July 7, 2013 and is given in the table below. The piezometer was sealed into the silt deposit.

Borehole	Ground Surface Elevation (m)	Water Level Depth (m)	Water Level Elevation (m)	Date
13-3	170.4	+0.4 <sup>1</sup>	170.8	July 7, 2013

**Note:** <sup>1</sup> The groundwater level in the standpipe was measured above the existing ground surface (i.e., artesian flow condition).

It should be noted that groundwater levels in the area are subject to fluctuations both seasonally and with precipitation events.



## PRELIMINARY FOUNDATION REPORT HIGHWAY 60 CULVERT REPLACEMENT - SITE NO. 29-238C

### 5.0 CLOSURE

This preliminary report was prepared by Ms. Kim Lesage, P.Eng. and reviewed by Kevin Nelson, P.Eng., an Associate and geotechnical engineer with Golder. Mr. Fintan Heffernan P.Eng., the designated MTO contact for this project.

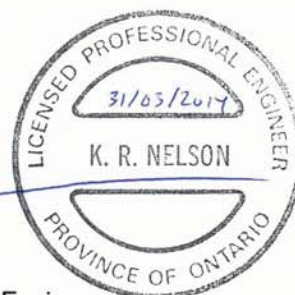
**GOLDER ASSOCIATES LTD.**

Kim Lesage, P.Eng.  
Geotechnical Engineer

Fin Heffernan, P.Eng.  
MTO Designated Contact



Kevin Nelson, P.Eng.  
Associate, Geotechnical Engineer



WAM/KSL/KN/FJH/bg

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## PRELIMINARY FOUNDATION REPORT HIGHWAY 60 CULVERT REPLACEMENT - SITE NO. 29-238C

### PART B

PRELIMINARY FOUNDATION DESIGN  
PROPOSED CULVERT REPLACEMENT  
SITE NO. 29-238C  
COCHRANE CREEK, HIGHWAY 60  
RENFREW COUNTY, ONTARIO  
W.P. 4170-11-01



## 6.0 DISCUSSION AND PRELIMINARY ENGINEERING RECOMMENDATIONS

### 6.1 General

This section of the report provides preliminary foundation design recommendations for the proposed replacement of the existing Cochrane creek culvert on Highway 60. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during this preliminary subsurface investigation. The discussion and preliminary recommendations presented are intended to provide the designers with sufficient information to assess the feasible foundation alternatives and to carry out the preliminary design of the foundations for the replacement structure. Further investigation and analysis will be required during detail design.

Where comments are made on construction, they are provided to highlight those aspects that could affect the future detail design of the project, and for which special provisions may be required in the Contract Documents. Those requiring information on aspects of construction should make their own interpretation of the factual information provided as such interpretation may affect equipment selection, proposed construction methods, scheduling and the like.

The replacement of the culvert will be along the existing highway and culvert alignment as shown on Drawing 1. It is understood that the proposed invert level of the new culvert will be about the same as the existing, at about Elevation 168.0 to 168.5 m, and the existing grades of Highway 60 will be maintained. Widening of the existing shoulder/rounding will also be required at the culvert site to accommodate the construction staging, however, the existing pavement grades will be maintained. The embankments will be constructed with side slopes of 3H:1V for the upper portion of the embankment, and 2H:1V for the lower portions of the side slopes.

### 6.2 Foundation Options

The existing Cochrane creek culvert is a 3.66 m wide by 2.4 m high structural plate corrugated steel pipe arch culvert which is about 29 m in length. The date of construction of the culvert is unknown. The existing culvert invert is at about Elevation 168.5 m and Cochrane Creek ultimately drains into Golden Lake. The depth of water within the culvert was about 1.2 m at the time of the field investigation.

The existing pavement grade at the culvert location is at an Elevation of about 173 m. In this area, Highway 60 is one lane wide in each direction (i.e., a 2-lane highway). The existing embankment slopes at the culvert locations are up to about 4 m in height and are sloped at about 1.5H to 3H:1V.

Based on the subsurface conditions, only shallow foundation options have been considered in sufficient detail for preliminary design for the replacement of the existing Cochrane Creek culvert. It is not considered to be a practical or economical option to support the culvert on deep foundations because the shallow subsoils will provide adequate bearing resistance and settlement performance given that the Highway 60 grade will not be raised and only a minor widening/rounding will be required.

A summary of the advantages and disadvantages associated with each shallow foundation option is provided below, and a comparison of the alternative foundation options based on advantages, disadvantages, constructability and relative costs is provided in Table 1 following the text of this report.



- **Concrete box culvert founded on the native sand and silt:** A box culvert could be considered for the culvert replacement provided it is founded on or within the native sand and silt. It is noted that the base of the existing embankment fill extends below the proposed founding level of the box culvert replacement. The existing fill and thin peat layer encountered in Borehole 13-1 would need to be removed and replaced with compacted granular fill. Dewatering methods will be required for this foundation option. A precast culvert would be preferred over a cast-in-place culvert for this option because it would likely be easier and quicker to install, and require less construction time and disruption to traffic.
- **Rigid frame open footing culvert founded on the native sand and silt:** A rigid frame open footing culvert could also be considered for the culvert replacement, provided it is founded on the native sand and silt. The settlements for this type of culvert would be larger than those for a closed box culvert due to the higher concentration of foundation stresses below the footings. As above, it is expected that any existing fill below the proposed foundation level would need to be removed and replaced with compacted granular fill. More extensive dewatering would be required for this foundation option. A precast culvert would be preferred over a cast-in-place culvert for this option because it would likely be easier and quicker to install, and require less construction time and disruption to traffic.

Based on the above considerations, the preferred option from a geotechnical/foundations perspective is to replace the culvert with a precast concrete box culvert founded on the native sand and silt. This option would be preferred over a rigid frame open footing culvert, which would require higher bearing capacities and more extensive dewatering during excavation and construction.

### 6.3 Liquefaction Considerations

The layered sand and silt deposit which underlies the site is in a very loose to compact state of packing. The potential for seismic liquefaction of this deposit has therefore been evaluated.

Seismic liquefaction occurs when earthquake induced vibrations cause an increase in pore water pressure within the soil. The presence of excess pore water pressures reduces the effective stress between the soil particles and the soil's frictional resistance to shearing. This phenomenon, which leads to a temporary reduction in the shear strength of the soil, may cause:

- Large lateral movements of even gently sloping ground, referred to as 'lateral spreading'. This strength loss can also result in instability of slopes, approach embankments, and retaining structures (i.e., deep-seated shear failure through the underlying soil);
- Reduced shear resistance (i.e., bearing capacity) of soils which support foundations, as well as reduced resistance to sliding of shallow foundations; and,
- Reduced shaft resistance for deep foundations as well as reduced resistance to lateral loading.

In addition, 'seismic settlements' may occur once the vibrations and shear stresses have ceased. Seismic settlement is the process whereby the soils stabilize into a denser arrangement after an earthquake, causing potentially large surface settlements.





The following conditions are more prone to experiencing seismic liquefaction:

- Coarse grained soils (i.e., more probable for sands than for silts);
- Soils having a loose state of packing; and,
- Soils located below the groundwater level.

An assessment of the liquefaction potential of the layered sand and silt deposit was carried out using the Seed and Idriss (1971) simplified procedure based on SPT  $N_{60}$  values from the boreholes. The SPT N values reported on the borehole records were corrected for the overburden stress, rod length during sampling, and hammer energy efficiencies.

The results of this assessment suggest that a large portion of the native submerged sands and silts, particularly at depth, could be classified as liquefiable under an earthquake with a magnitude of 6.2 (Ottawa area specified design value) and a peak ground acceleration of 0.2 g (recognizing that the 'design' earthquake has a return period of 1 in 500 years). The liquefaction potential and induced settlements/slope movements should therefore be reviewed during detail design.

## 6.4 Concrete Box Culvert Foundations

### 6.4.1 Founding Level and Bedding

It is not necessary to found the box culvert at the standard depth for frost protection purposes as box structures are tolerant of small magnitude movements related to freeze-thaw cycles should these occur. The box culvert should, however, be founded below any existing fill and surficial soils containing organic matter.

The bedding and/or leveling pad requirements for a box culvert replacement should be in accordance with Ontario Provincial Standard Specification (OPSS) 422 (*Construction Specification for Precast Reinforced Concrete Box Culverts and Box Sewers in Open Cut*) for concrete box culverts. It is recommended that the box culvert segments be placed on a minimum thickness of 300 mm of granular bedding material meeting OPSS 1010 Granular A. A geotextile should also be placed below the Granular A bedding to protect it from disturbance during placement of the granular fill and to provide a proper filter to the silt subgrade. The geotextile should consist of a woven Class II geotextile as per OPSS 1860.

It is understood that the proposed invert level will be at about Elevation 168.5 m, and therefore the box culvert replacement will be typically within 1 m of the native silt. If the existing foundations extend beneath the proposed subgrade levels as provided above (which is expected based on the borehole information), the existing foundations and any disturbed or softened material and fill should be removed and replaced with compacted granular material meeting OPSS 1010 Granular B Type II.

The footing subgrade should be inspected by a Quality Verification Engineer (QVE) in accordance with OPSS 902 (*Construction Specification for Excavating and Backfilling – Structures*). Further discussion regarding subgrade preparation and protection is provided in Section 6.8.3.

### 6.4.2 Geotechnical Resistances

For a box culvert founded at the elevations provided in Section 6.4.1 a factored geotechnical resistance at Ultimate Limit States (ULS) of 150 kPa and a geotechnical resistance at Serviceability Limit States (SLS) for 25 mm of settlement of 100 kPa may be used for preliminary design purposes (for non-seismic loading conditions).



These preliminary geotechnical resistances are provided for loads applied perpendicular to the surface of the footings; where applicable, inclination of the load should be taken into account in accordance with Section 6.7.4 of the *Canadian Highway Bridge Design Code (CHBDC 2006)* and its *Commentary*.

The preliminary geotechnical resistance values provided above will have to be re-evaluated and modified as necessary during detail design. The residual strength of the liquefied soil during an earthquake would have to be assessed during detail design in the determination of the factored ULS geotechnical resistance for the culvert under liquefaction loading conditions.

## 6.5 Settlement

It is understood that widening of the existing shoulder/rounding will also be required at the culvert site to accommodate the construction staging, however, the existing pavement grades will be maintained. This will require placement of up to 1 m thickness of new fill on top of the existing side slope near the culvert inlet and outlet, at the toe of the embankments. Provided the SLS geotechnical resistance for the culvert is limited to the values provided in Section 6.4.2, then the culvert settlements should be minor (i.e., less than about 25 mm). Most of this settlement will consist of the recompression of the layered sand and silt and will occur during construction.

## 6.6 Culvert Backfill and Erosion Protection

Backfill, cover and construction of the frost taper (backfill transition) for concrete culverts should be completed in accordance with OPSS 422 (*Construction Specification for Precast Reinforced Concrete Box Culverts and Box Sewers in Open Cut*) and/or OPSD 803.010 (*Backfill and Cover for Concrete Culverts*).

Backfill to culvert walls should consist of granular fill meeting the requirements of OPSS 1010 Granular A or Granular B Type II, but with less than 5 per cent passing the No. 200 sieve. The backfill should be placed and compacted in accordance with MTO's Special Provision SP105S21 (Amendment to OPSS 501). The fill depth during placement should be maintained equal on both sides of the culvert walls, with one side not exceeding the other by more than 500 mm. The culvert should be designed for the full overburden pressure and live load assuming that the embankment fill has a unit weight of 22 kN/m<sup>3</sup> for Granular A and 21 kN/m<sup>3</sup> for Granular B Type II or select earth fill above and/or surrounding the culvert.

To prevent surface water from flowing either beneath the culvert (potentially causing undermining and scouring) or around the culvert (creating seepage through the embankment fill, and potentially causing erosion and loss of fine soil particles), a clay seal or concrete cut-off wall should be provided at the upstream and downstream ends of the culvert replacement.

If the flow velocities are sufficiently high, provision should be made for scour and erosion protection (suitable non-woven geotextiles and/or rip-rap) at the culvert inlet and outlet. The requirements for and design of erosion protection measures for the culvert inlet should be assessed by the hydraulic design engineer. As a minimum, rip-rap treatment for the culvert outlet should be consistent with the standard Treatment Type A presented in OPSD 810.010 (*Rip-Rap Treatment for Sewer and Culvert Outlets*), with the rip-rap placed up to the toe of slope level, in combination with the cut-off measures noted above.



## 6.7 Embankment Construction and Stability

It is understood that widening of the existing shoulder/rounding will also be required at the culvert site to accommodate the construction staging, however, the existing pavement grades will be maintained. This will require placement of up to 1 m thickness of new fill on top of the existing side slope near the culvert inlet and outlet, at the toe of the embankments. It is further understood that the proposed embankments will be constructed with side slopes of 3H:1V for the upper portion of the embankment, and 2H:1V for the lower portions of the side slopes.

The subsurface conditions in the area of the widening consist of layered sand and silt. In preparation for the widening, any existing fill, topsoil, organic matter or softened/loosened soils should be stripped from below the embankment area.

The embankment fill for the widening areas of the culvert should be placed and compacted in accordance with MTO's Special Provisions 206S03 and 105S10. Benching of the existing embankment side slopes should be carried out to "key in" the new fill materials for the widening, in accordance with OPSD 208.010. Commonly in embankment widening construction, the fill material cut from the existing embankment side slope for creation of these benches is re-used for the embankment widening below/adjacent to each bench area. Additional fill for construction of the embankment widening above the level of the original ground surface (i.e., above the groundwater level) could consist of clean earth fill or granular fill; rock fill may also be used but is considered to be impractical for the minor widening required at this site.

For the soil conditions at the culvert and the embankment height, the embankment will have an adequate factor of safety against both static and seismic slope instability (i.e., at or greater than 1.3 and 1.1 under static and seismic conditions, respectively).

Settlement of the Highway 60 embankment will occur as a result of compression of the new embankment fill placed in the immediate vicinity of the culvert replacement. Provided that the embankment material consists of Select Subgrade Material or clean earth fill, the settlement of the embankment fill itself is expected to be less than 25 mm. The use of granular fill for the new embankment construction would reduce this magnitude of post-construction settlement (likely to less than half that value) since the majority of settlement of these fills will occur during construction.

If rock fill is used, which could be considered if a more significant widening is required for construction staging purposes, settlement of the rock fill itself will depend on the type of rock and on the method and sequence of placement and compaction of the fill. Assuming that the rock fill is placed in accordance with the requirements outlined in the SP206S03, the settlement of the rock fill in embankments is estimated to be about 1 percent of the embankment height and it is anticipated that the majority of this settlement will occur during the first year following construction.

## 6.8 Construction Considerations

The following sections identify future construction issues that should be considered at this stage as they may impact the preliminary design as well as the future detail design. Where applicable, Non-Standard Special Provisions (NSSP) should be developed during detail design for incorporation into the Contract Documents.



## 6.8.1 Groundwater and Surface Water Control

Control of the surface water and groundwater will be necessary for the construction of the replacement culvert, to allow excavation and foundation construction to be carried out in stable, undisturbed, dry conditions. It is expected that coffer dams will need to be constructed to bypass the flows around the culvert areas during construction and it is understood that consideration is being given to pumping water through a temporary bypass pipe or pipes installed outside of the main excavation area (i.e., providing a bypass pipe(s) through the embankment outside of the excavation area instead of trying to convey flows through the culvert replacement area).

Groundwater flow into the excavations should be expected from the native silts and sands encountered at the site. Excavations for installation of the replacement culvert will be required to extend to depths of approximately 3 m to 3.5 m below the water level measured within a piezometer installed at depth within the silty site soils. These conditions could result in basal instability (i.e., piping) and/or disturbance of the foundation subgrade materials unless adequate dewatering is provided.

Pumping from filtered sumps established in the floor of the excavations is unlikely to effectively dewater the excavation due to the fine grained nature of the silt. It is therefore considered that pre-pumping from a series of sanded-in vacuum well-points or eductor wells installed into the silty subgrade soils would be required to lower the piezometric level below the excavation level in advance of culvert construction. Due to the fine-grained nature of the silt deposit, difficulties may be encountered effectively dewatering these soils even using such vacuum assisted dewatering systems. In this regard, it is recommended that further investigation including a small scale pump/response test be carried out during or prior to the detail design stage to further assess the requirements for dewatering systems to adequately dewater the silt deposit.

Consideration should be given to including an NSSP in the contract documents that alerts the contractor to potential groundwater inflow and the related potential disturbance of the silt/sand subgrade soils. It is recommended that coffer dams be constructed around the excavations (on all sides) to bypass the flows around the culvert areas during construction and reduce groundwater flow into the excavation area. The coffer dams should be comprised of interlocking steel sheet piling that penetrates into the silt/sand deposits and extends either into an underlying, less permeable stratum or to sufficient depth to address both basal instability and lateral restraint considerations.

Surface water should be directed away from the excavation area, to prevent ponding of water that could result in disturbance and weakening of the sensitive sand and silt subgrade soils; further discussion on this aspect is provided in Section 6.8.3.

## 6.8.2 Excavation and Temporary Protection Systems

The contractor should be aware that trafficking over the wet silt and sand material may not be possible and an Operational Constraint or a non-standard Special Provision should be included in the contract in this regard, which directs the contractor to not travel on the subgrade surface with equipment.

Temporary excavations for the culvert will be made through the existing fill and layered sand and silt. Excavation works must be carried out in accordance with the guidelines outlined in the Occupational Health and Safety Act (OHSA) and Regulations for Construction Projects. The existing fill above the water table would be classified as Type 3 soil based on the OHSA. According to OHSA, excavations that extend to, or into, Type 3 soils should be made with side slopes no steeper than 1 horizontal to 1 vertical (1H:1V). The fill and layered sand and silt below the water table would be classified as Type 4 soil based on OSHA, and excavations in these materials should be sloped no steeper than 3H:1V.





## PRELIMINARY FOUNDATION REPORT HIGHWAY 60 CULVERT REPLACEMENT - SITE NO. 29-238C

If 3H:1V open cut excavation side slopes cannot be accommodated, then a temporary protection system (i.e., temporary excavation shoring) will be required. Where shoring is required, the support system should be designed and constructed by the contractor in accordance with SP105S19. The lateral movement of the temporary shoring system should meet Performance Level 2 as specified in SP105S19.

A conventional shoring system for these conditions could consist of soldier piling and lagging or interlocking steel sheet piling supported against lateral movement using tie backs and/or internal struts/braces. The design of that system is the responsibility of the contractor.

### 6.8.3 Existing Foundations and Subgrade Protection

All embankment fill, topsoil, organics, and soft or loose soils should be removed from below the proposed founding elevations and wasted or reused as landscaping fill, as required. Subgrade preparation should be performed and monitored in accordance with OPSS 902 (*Construction Specification for Excavating and Backfilling – Structures*). The cleaned excavation base should be inspected by a QVE qualified in geotechnical engineering prior to placing granular bedding for the box culvert.

All existing foundations and any disturbed and softened material and fill below the founding level should be removed and replaced with compacted granular material meeting OPSS 1010 Granular A placed to above the water level and compacted in accordance with the requirements of MTO's Special Provision SP105S10.

The sensitive layered sand and silt subgrade will be susceptible to disturbance and degradation on exposure to water and construction traffic. It is recommended that a minimum 300 mm thick layer of Ontario Provincial Standard Specification 1010 (*Material Specification for Aggregates – Base, Subbase, Select Subgrade and Backfill Material*) Granular A be placed below the base slab on the subgrade to form a bedding layer for the box culvert segments, and to limit the degradation of the sand and silt subgrade. The bedding should be placed within four hours after inspection and approval of the subgrade to limit such degradation. A Class II woven geotextile (per OPSS 1860) should also be placed on the subgrade to protect it from disturbance during placement of the granular fill and to provide a proper filter to the silt subgrade.

### 6.9 Recommendations for Further Work in Detail Design

Additional boreholes will be required during the future detail design stage of investigation, to further assess and/or confirm the subsurface conditions and the preliminary recommendations provided in this report, as follows:

- Assessment of the variability of any existing fill and surficial soils to confirm the founding elevations within the culvert area;
- Assessment of the liquefaction potential of the layered sand and silt deposit and related impacts (e.g., settlements, stability issues); and,
- Due to the fine-grained nature of the site soils, further investigation and assessment should be completed to determine dewatering feasibility and requirements.



## PRELIMINARY FOUNDATION REPORT HIGHWAY 60 CULVERT REPLACEMENT - SITE NO. 29-238C

### 7.0 CLOSURE

This preliminary report was prepared by Ms. Kim Lesage, P.Eng. and reviewed by Mr. Kevin Nelson, P.Eng., an Associate and geotechnical engineer with Golder. Mr. Fin Heffernan, P.Eng., Golder's Designated MTO Foundations Contact for this project, conducted an independent quality review of the report.

#### GOLDER ASSOCIATES LTD.

Kim Lesage, P.Eng.  
Geotechnical Engineer

Kevin Nelson, P.Eng.  
Associate, Geotechnical Engineer

Fin Heffernan, P.Eng.  
MTO Designated Contact



WAM/KSL/KN/FJH/bg

n:\active\2012\1121 - geotechnical\12-1121-0193 dillon mega 3 eastern region\foundations\5 - reports\contract b - highway 60 site 29-238c\12-1121-0193-1315 site 29-238c hwy 60 cochrane creek final march 2014.docx

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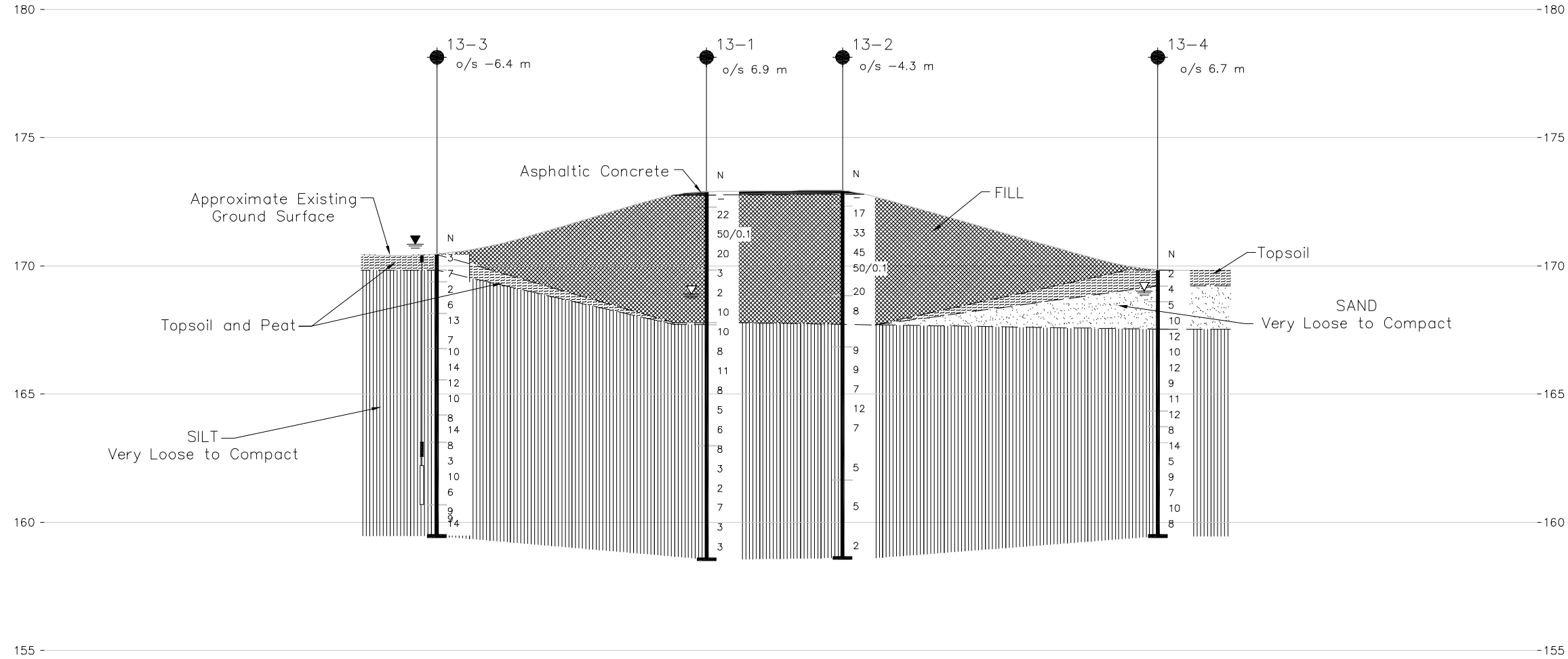
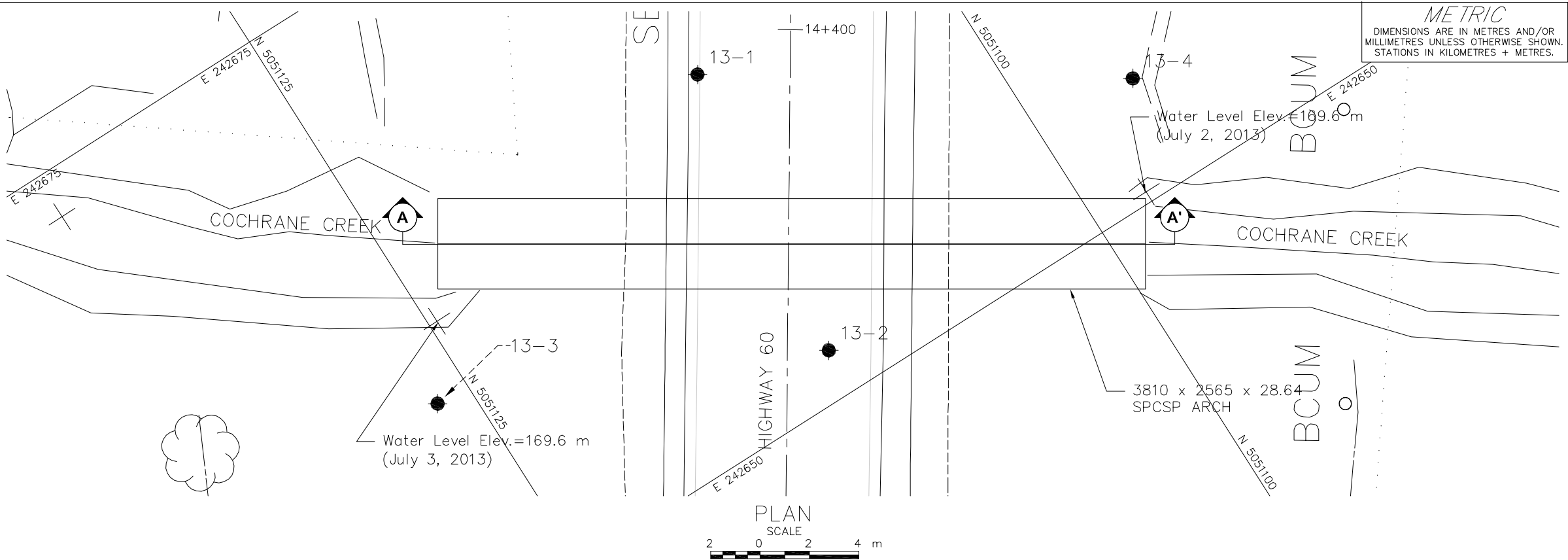


PRELIMINARY FOUNDATION REPORT  
HIGHWAY 60 CULVERT REPLACEMENT - SITE NO. 29-238C

Table 1  
Comparison of Foundation Alternatives  
W.P. 4170-11-01

Foundation Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risks/Consequences
<b>Option 1</b> Box Culvert founded on the layered sand and silt	<ul style="list-style-type: none"><li>Feasible, preferred option</li></ul>	<ul style="list-style-type: none"><li>Potentially shallower excavation depths.</li><li>Foundation loads distributed over a larger area, therefore reducing settlement magnitudes.</li><li>Precast sections would be quicker and easier to install (i.e., less disruption to traffic).</li></ul>	<ul style="list-style-type: none"><li>Pumping from sumps will not completely lower the groundwater level.</li><li>Interlocking sheet pile enclosure would be required to control groundwater.</li></ul>	<ul style="list-style-type: none"><li>Moderate cost</li><li>Least expensive option</li></ul>	<ul style="list-style-type: none"><li>Low risk option</li></ul>
<b>Option 2</b> Rigid Frame Open Footing Culvert founded on the layered sand and silt	<ul style="list-style-type: none"><li>Feasible</li></ul>	<ul style="list-style-type: none"><li>Not applicable</li></ul>	<ul style="list-style-type: none"><li>Deeper excavation depth.</li><li>Larger settlement than those of a box culvert due to the higher concentration of foundation stresses.</li><li>Interlocking sheet pile enclosure would be required to control groundwater. Groundwater control requirements would be more extensive.</li><li>Probably insufficient geotechnical resistances for this option.</li></ul>	<ul style="list-style-type: none"><li>Higher cost</li></ul>	<ul style="list-style-type: none"><li>High risk option</li></ul>
<b>Option 3</b> Deep Foundations	<ul style="list-style-type: none"><li>Feasible but not practical</li></ul>	<ul style="list-style-type: none"><li>Would not result in culvert settlement.</li></ul>	<ul style="list-style-type: none"><li>Would require deep piles.</li><li>Culvert settlement would not conform to ground settlements. Could result in roadway distortion.</li><li>Also requires roadway protection system (same comments as for Option 1).</li></ul>	<ul style="list-style-type: none"><li>High cost</li><li>Most expensive option</li></ul>	<ul style="list-style-type: none"><li>Low risk option</li></ul>





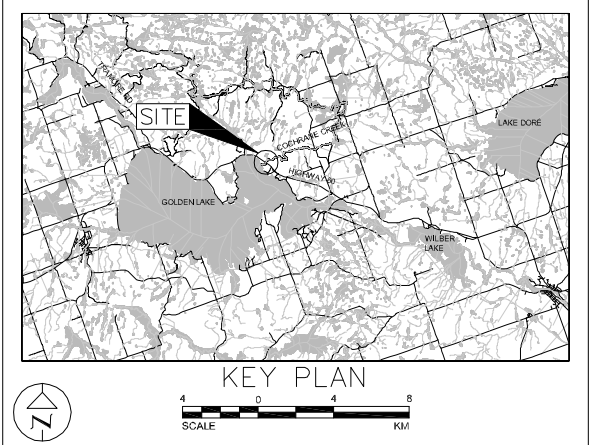
A-A' CULVERT SECTION  
SCALE  
2 0 2 4 m

METRIC  
DIMENSIONS ARE IN METRES AND/OR  
MILLIMETRES UNLESS OTHERWISE SHOWN.  
STATIONS IN KILOMETRES + METRES.

CONT No.  
WP No. 4170-11-01

CULVERT REPLACEMENT  
COCHRANE CREEK CULVERT AT HIGHWAY 60  
BOREHOLE LOCATIONS AND SOIL  
STRATA

**Golder Associates Ltd.**  
MISSISSAUGA, ONTARIO, CANADA



LEGEND

Borehole - Current Investigation  
 Seal  
 Piezometer  
N Standard Penetration Test Value  
16 Blows/0.3m unless otherwise stated  
(Std. Pen. Test, 475 j/blow)  
100% Rock Quality Designation (RQD)  
 WL in piezometer, measured on July 7, 2013  
 WL upon completion of drilling

BOREHOLE CO-ORDINATES			
No.	ELEVATION	NORTHING	EASTING
13-1	172.9	5051110.4	242664.2
13-2	172.9	5051111.9	242651.9
13-3	170.4	5051126.4	242658.6
13-4	169.8	5051095.6	242654.6

NOTES

This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contracts Documents.

The boundaries between soil strata have been established only at borehole locations. Between boreholes the boundaries are assumed from geological evidence.

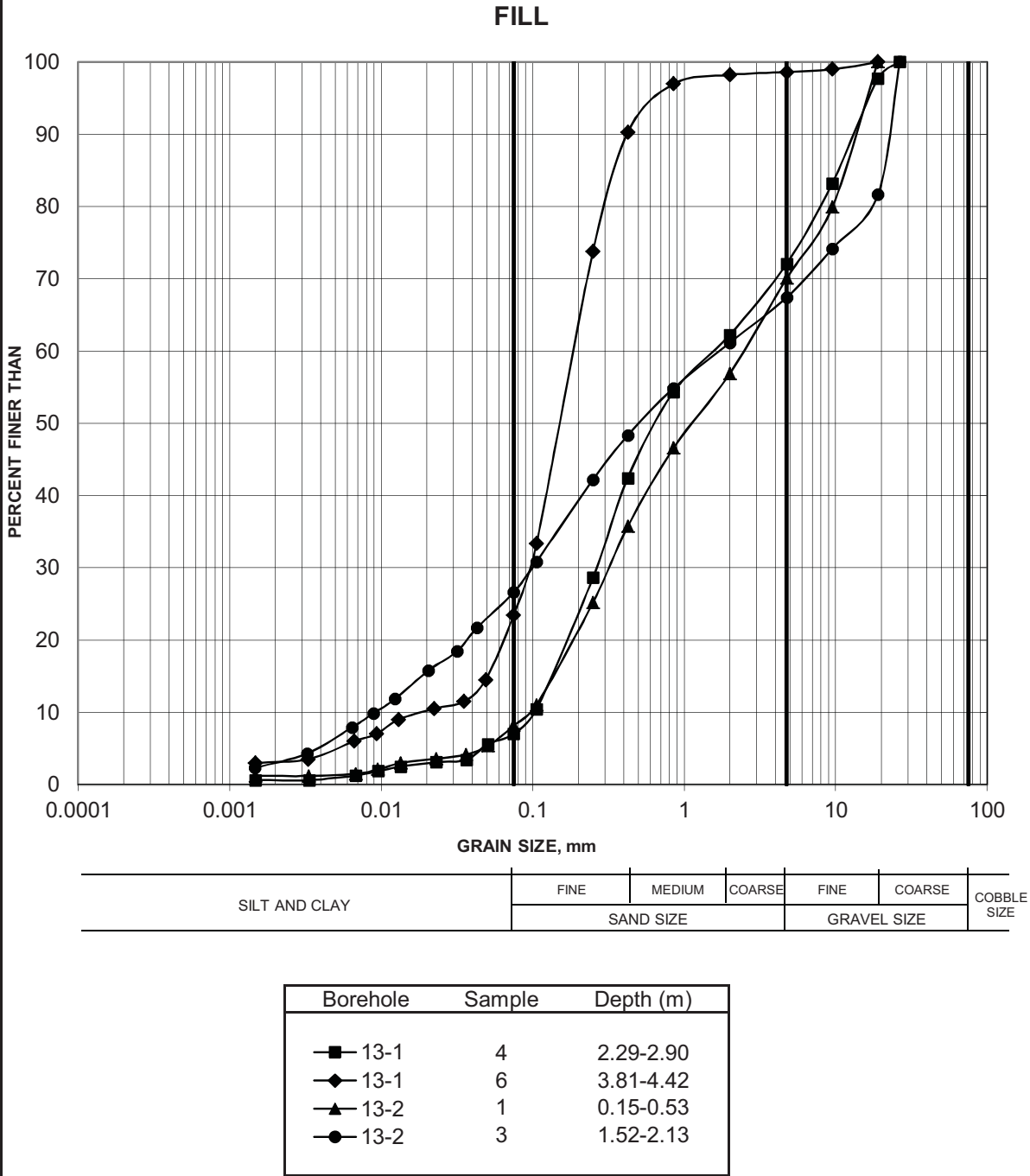
The complete Foundation Investigation and Design Report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with Section GC 2.01 of OPS General Conditions.

REFERENCE			
Base plan provided in digital format by Dillon, drawing file no. Contract B - Hwy 60 - B Plan.dwg, received August 6, 2013.			
Geocres No. 31F-182			
NO.	DATE	BY	REVISION
Hwy. 60 PROJECT NO. 12-1121-0193 DIST. EASTERN			
SUBM'D. KSL	CHKD. KSL	DATE: 3/28/2014	SITE: 29-238C
DRAWN: JFC	CHKD. KSL	APPD. FJH	DWG. 1



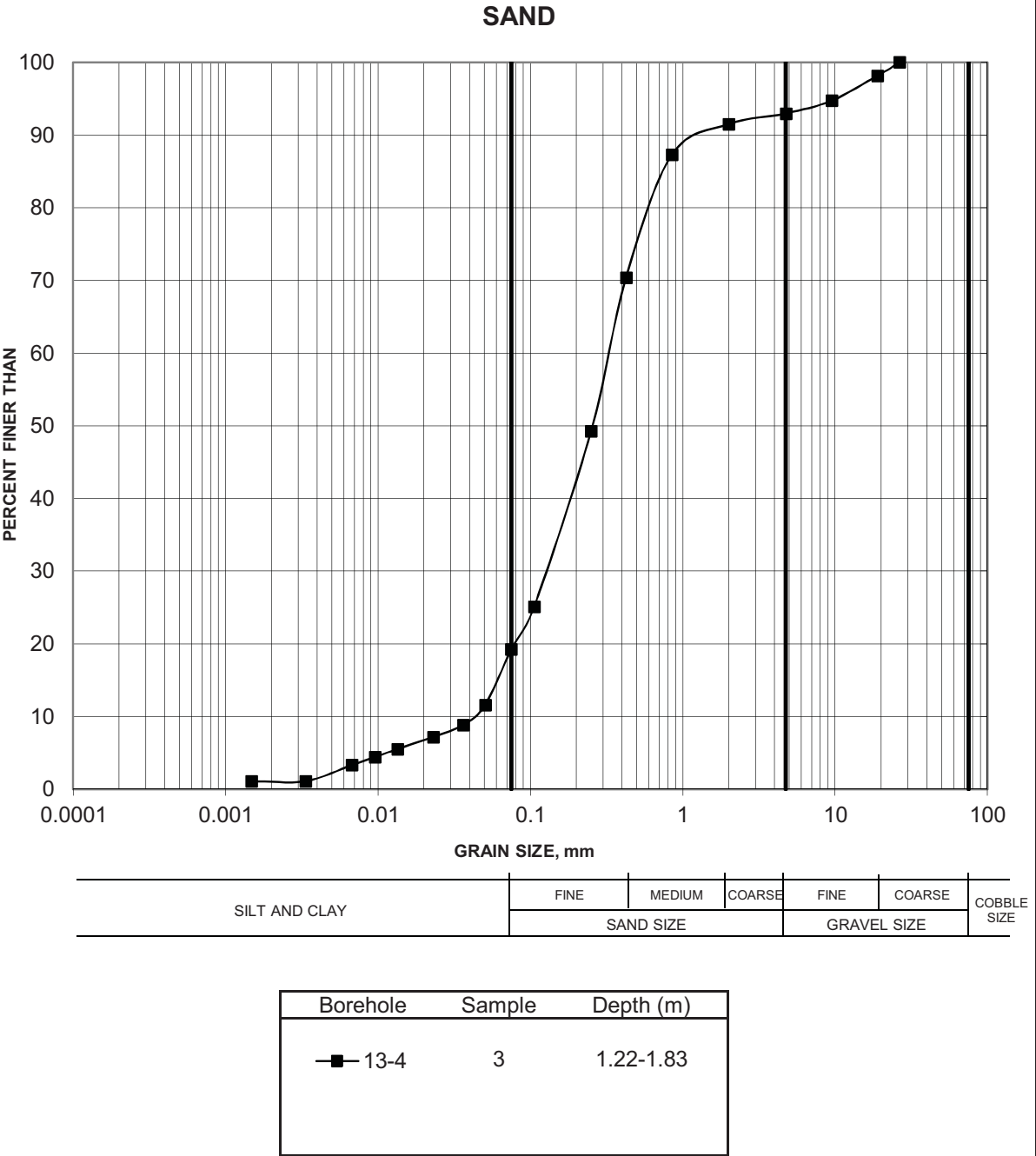
GRAIN SIZE DISTRIBUTION

FIGURE 1



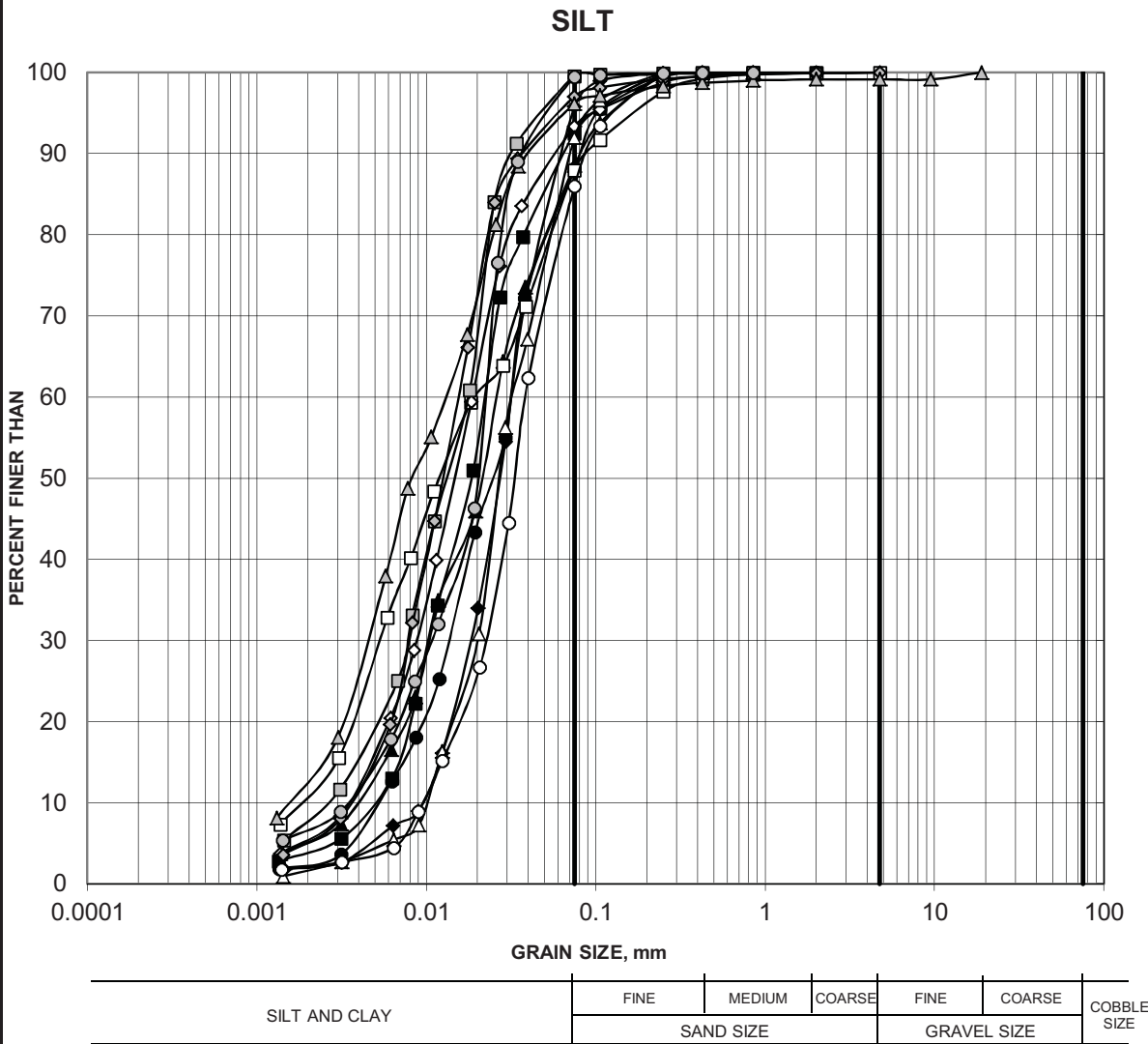
GRAIN SIZE DISTRIBUTION

FIGURE 2



GRAIN SIZE DISTRIBUTION

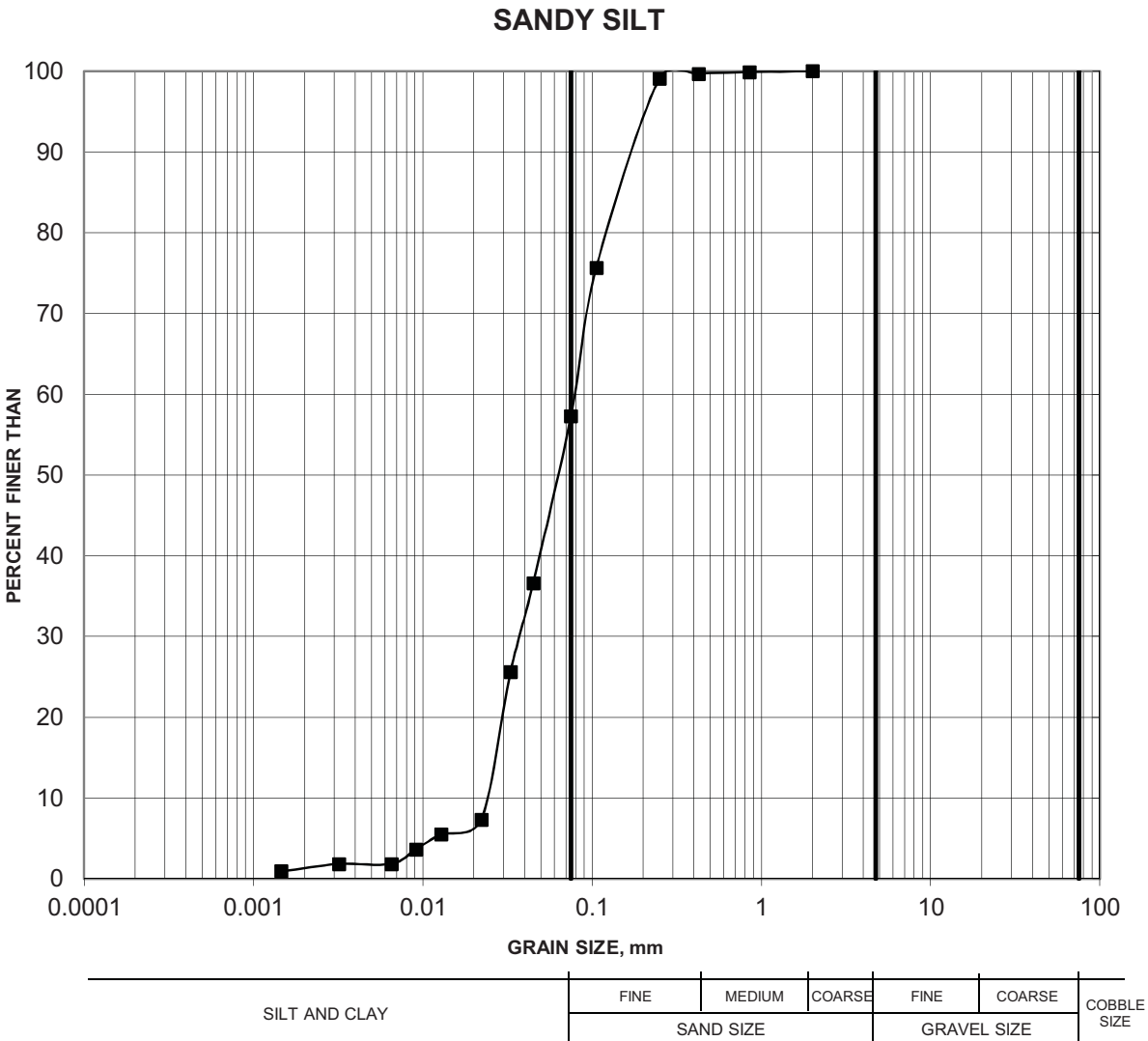
FIGURE 3



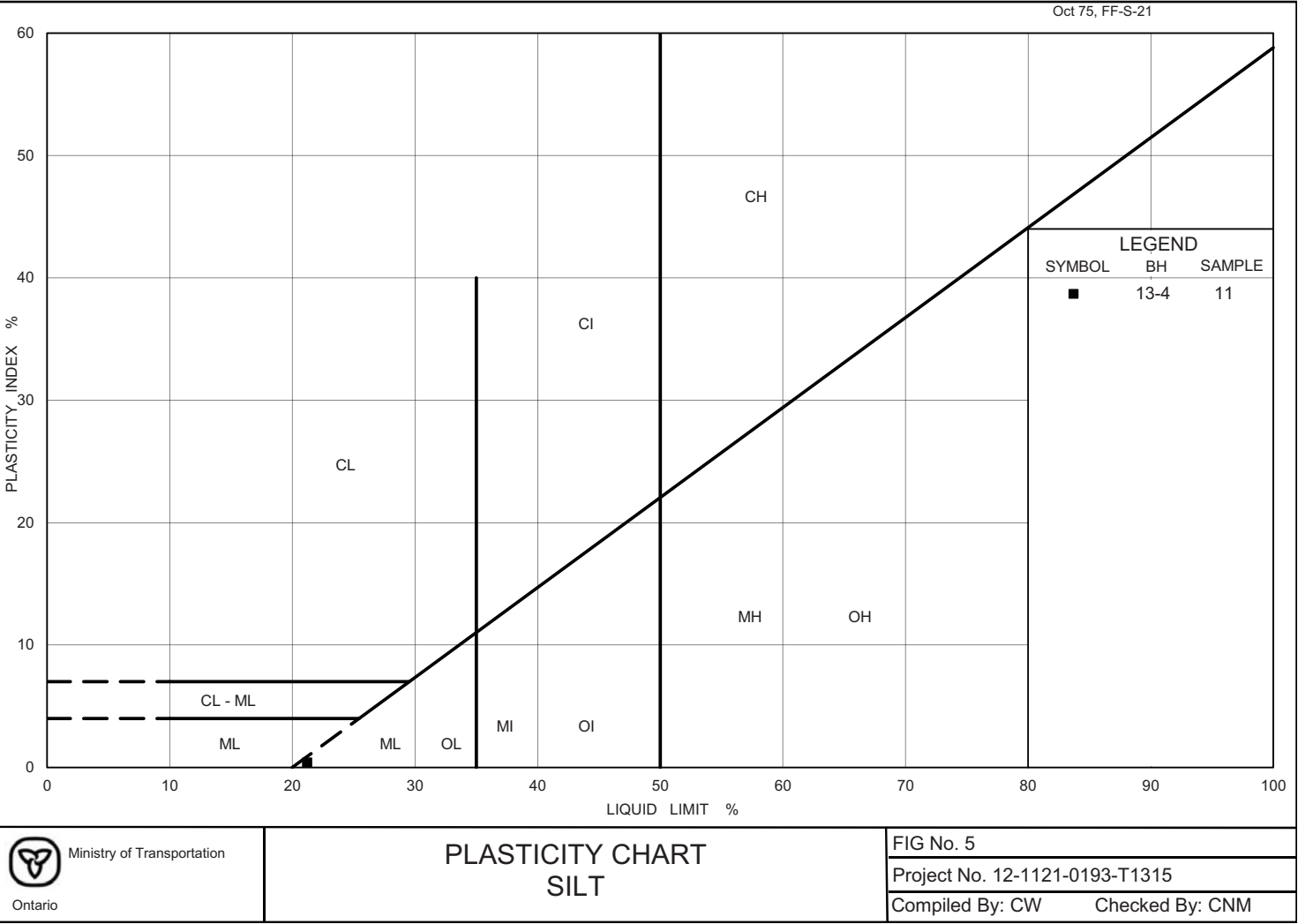
Borehole	Sample	Depth (m)
■ 13-1	9	6.10-6.71
◆ 13-1	14	9.91-10.52
▲ 13-1	18	12.96-13.57
● 13-2	10	7.62-8.23
□ 13-3	3	1.22-1.83
◇ 13-3	5	2.44-3.05
△ 13-3	9	4.88-5.49
○ 13-3	13	7.32-7.93
▣ 13-3	17	9.76-10.37
◇ 13-4	5	2.44-3.05
△ 13-4	11	6.10-6.71
○ 13-4	15	8.54-9.15

GRAIN SIZE DISTRIBUTION

FIGURE 4



Borehole	Sample	Depth (m)
■ 13-2	15	13.72-14.33



# APPENDIX A

List of Abbreviations and Symbols  
Record of Borehole Sheets

## LIST OF SYMBOLS

Unless otherwise stated, the symbols employed in the report are as follows:

I.

GENERAL

$\pi$

3.1416

$\ln x$

natural logarithm of x

$\log_{10} x$  or  $\log x$

logarithm of x to base 10

$g$

acceleration due to gravity

$t$

time

FOS

factor of safety

V

volume

W

weight

II.

STRESS AND STRAIN

$\gamma$

shear strain

$\Delta$

change in, e.g. in stress:  $\Delta \sigma'$

$\varepsilon$

linear strain

$\varepsilon_v$

volumetric strain

$\eta$

coefficient of viscosity

$\nu$

Poisson's ratio

$\sigma$

total stress

$\sigma'$

effective stress ( $\sigma' = \sigma - u$ )

$\sigma'_{vo}$

initial vertical effective overburden stress

$\sigma_1 \sigma_2 \sigma_3$

principal stresses (major, intermediate, minor)

$\sigma_{oct}$

mean stress or octahedral stress

$= (\sigma_1 + \sigma_2 + \sigma_3) / 3$

$\tau$

shear stress

$u$

porewater pressure

E

modulus of deformation

G

shear modulus of deformation

K

bulk modulus of compressibility

III.

SOIL PROPERTIES

(a) Index Properties

$\rho(\gamma)$

bulk density (bulk unit weight)\*

$\rho_d(\gamma_d)$

dry density (dry unit weight)

$\rho_w(\gamma_w)$

density (unit weight) of water

$\rho_s(\gamma_s)$

density (unit weight) of solid particles

$\gamma'$

unit weight of submerged soil ( $\gamma' = \gamma - \gamma_w$ )

$D_R$

relative density (specific gravity) of solid particles ( $D_R = \rho_s / \rho_w$ ) formerly ( $G_s$ )

$e$

void ratio

$n$

porosity

$S$

degree of saturation

$*$

Density symbol is  $\rho$ . Unit weight symbol is  $\gamma$  where  $\gamma = \rho g$  (i.e. mass density multiplied by acceleration due to gravity)

(a) Index Properties (continued)

$w$

water content

$w_L$  or LL

liquid limit

$w_p$  or PL

plastic limit

$I_p$  or PI

plasticity Index =  $(w_L - w_p)$

$w_s$

shrinkage limit

$I_L$

liquidity index =  $(w - w_p) / I_p$

$I_c$

consistency index =  $(w_L - w) / I_p$

$e_{max}$

void ratio in loosest state

$e_{min}$

void ratio in densest state

$I_D$

density index =  $(e_{max} - e) / (e_{max} - e_{min})$  (formerly relative density)

(b) Hydraulic Properties

$h$

hydraulic head or potential

$q$

rate of flow

$v$

velocity of flow

$i$

hydraulic gradient

$k$

hydraulic conductivity (coefficient of permeability)

$j$

seepage force per unit volume

(c) Consolidation (one-dimensional)

$C_c$

compression index (normally consolidated range)

$C_r$

recompression index (overconsolidated range)

$C_s$

swelling index

$C_\alpha$

coefficient of secondary consolidation

$m_v$

coefficient of volume change

$c_v$

coefficient of consolidation (vertical direction)

$T_v$

time factor (vertical direction)

$U$

degree of consolidation

$\sigma'_p$

pre-consolidation stress

OCR

overconsolidation ratio =  $\sigma'_p / \sigma'_{vo}$

(d) Shear Strength

$\tau_p$  or  $\tau_r$

peak and residual shear strength

$\phi'$

effective angle of internal friction

$\delta$

angle of interface friction

$\mu$

coefficient of friction =  $\tan \delta$

$c'$

effective cohesion

$c_u$  or  $s_u$

undrained shear strength ( $\phi = 0$  analysis)

$p$

mean total stress  $(\sigma_1 + \sigma_3) / 2$

$p'$

mean effective stress  $(\sigma'_1 + \sigma'_3) / 2$

$q$

$(\sigma_1 - \sigma_3) / 2$  or  $(\sigma'_1 - \sigma'_3) / 2$

$q_u$

compressive strength  $(\sigma_1 - \sigma_3)$

$S_t$

sensitivity

Notes:

$\tau = c' + \sigma' \tan \phi'$

shear strength = (compressive strength) / 2



PROJECT 12-1121-0193-1315										RECORD OF BOREHOLE No 13-1										SHEET 1 OF 2										METRIC									
G.W.P. 4170-11-01										LOCATION N 5051110.4 :E 242664.2										ORIGINATED BY DG																			
DIST Eastern HWY 60										BOREHOLE TYPE Power Auger 200 mm Diam. (Hollow Stem)										COMPILED BY JM																			
DATUM Geodetic										DATE May 28-30, 2013										CHECKED BY KSL																			
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT			UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)																							
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa				WATER CONTENT (%)				GR	SA	SI	CL																				
								UNCONFINED		FIELD VANE		W <sub>p</sub>	W	W <sub>L</sub>																									
								○	+	○	×																												
172.9	GROUND SURFACE					20	40	60	80	100	25	50	75																										
0.0	ASPHALTIC CONCRETE																																						
0.1	Sand (BASE) Brown Moist		1	GRAB	-						○																												
172.3																																							
0.6	Sand, some gravel, with cobbles and boulders (FILL) Compact Brown Moist		2	SS	22						○																												
			3	SS	50/0.1																																		
			4	SS	20						○																												
169.9																																							
3.1	Sand, some silt, trace clay (FILL) Very loose to loose Grey Wet		5	SS	3																																		
			6	SS	2						○																												
167.8																																							
5.2	Organic matter (PEAT) SILT, trace to some sand, trace clay Loose to compact Grey Wet		8	SS	10																																		
			9	SS	8						○																												
			10	SS	11																																		
			11	SS	8																																		
			12	SS	5						○																												
			13	SS	6																																		
163.0																																							

GTA-MTO 001 1211210193-1315.GPJ GAL-GTA.GDT 03/28/14 JM

Continued Next Page

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

PROJECT 12-1121-0193-1315										RECORD OF BOREHOLE No 13-1										SHEET 2 OF 2										METRIC									
G.W.P. 4170-11-01										LOCATION N 5051110.4 :E 242664.2										ORIGINATED BY DG																			
DIST Eastern HWY 60										BOREHOLE TYPE Power Auger 200 mm Diam. (Hollow Stem)										COMPILED BY JM																			
DATUM Geodetic										DATE May 28-30, 2013										CHECKED BY KSL																			
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT			UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)																							
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa				WATER CONTENT (%)				GR	SA	SI	CL																				
								UNCONFINED		FIELD VANE		W <sub>p</sub>	W	W <sub>L</sub>																									
								○	+	○	×																												
9.9	SILT, trace to some sand, trace clay Very loose to loose Grey Wet		14	SS	8																																		
			15	SS	3																																		
			16	SS	2																																		
			17	SS	7																																		
			18	SS	3																																		
			19	SS	3																																		
158.6	END OF BOREHOLE																																						
14.3	NOTES:  1. Water level in open borehole at a depth of 4.0 m below ground surface (Elev. 168.9 m), measured during drilling.  2. Borehole 13-1 initially encountered auger refusal at about 1.6 m depth; borehole was relocated ~ 1 m to the southeast and redrilled.																																						

GTA-MTO 001 1211210193-1315.GPJ GAL-GTA.GDT 03/28/14 JM

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE



PROJECT 12-1121-0193-1315		RECORD OF BOREHOLE No 13-2		SHEET 2 OF 2		METRIC	
G.W.P. 4170-11-01		LOCATION N 5051111.9 :E 242651.9		ORIGINATED BY		DG	
DIST Eastern HWY 60		BOREHOLE TYPE Power Auger 200 mm Diam. (Hollow Stem)		COMPILED BY		JM	
DATUM Geodetic		DATE May 22-24, 2013		CHECKED BY		KSL	

[illegible]

GTA-MTO 001 1211210193-1315.GPJ GAL-GTA.GDT 03/28/14 JM

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity      ○<sup>3%</sup> STRAIN AT FAILURE

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity      ○<sup>3%</sup> STRAIN AT FAILURE



PROJECT12-1121-0193-1315

G.W.P.4170-11-01

DISTEastern

DATUMGeodetic

LOCATIONN 5051126.4 ; E 242658.6

BOREHOLE TYPEPortable Drill

DATEMay 21, 2013

ORIGINATED BYHEC

COMPILED BYJM

CHECKED BYKSL

RECORD OF BOREHOLE No 13-3

SHEET 1 OF 2

METRIC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT			UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				
								20	40	60	80	100	W <sub>p</sub>	W	W <sub>L</sub>		
170.4	GROUND SURFACE																
0.0	CLAYEY SAND, trace silt, with organic matter Very loose Dark brown Moist		1	SS	3												
169.8																	
0.6	Silty SAND Loose Brown Moist to wet		2	SS	7												
169.3																	
1.1	SILT, some sand, trace clay Very loose to loose Grey Wet		3	SS	2												
168.1			4	SS	6												
2.3	SILT, trace clay, occasional sand seams Compact to loose Grey Wet		5	SS	13												
166.7			6	SS	7												
3.7	Sandy SILT, trace clay Compact Grey Wet		7	SS	10												
165.5			8	SS	14												
4.9	SILT, trace sand and clay Compact Grey Wet		9	SS	12												
164.2			10	SS	10												
6.3	SILT, some clay, trace to some sand Loose Grey Wet		11	SS	8												
163.1			12	SS	14												
7.3	SILT, some sand, trace clay Very loose to loose Grey Wet		13	SS	8												
160.7			14	SS	3												
9.8			15	SS	10												
			16	SS	6												
			17	SS	9												

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Continued Next Page

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity  
○ 3% STRAIN AT FAILURE

PROJECT12-1121-0193-1315

G.W.P.4170-11-01

DISTEastern

DATUMGeodetic

LOCATIONN 5051126.4 ; E 242658.6

BOREHOLE TYPEPortable Drill

DATEMay 21, 2013

ORIGINATED BYHEC

COMPILED BYJM

CHECKED BYKSL

RECORD OF BOREHOLE No 13-3

SHEET 2 OF 2

METRIC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT			UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				
								20	40	60	80	100	W <sub>p</sub>	W	W <sub>L</sub>		
	--- CONTINUED FROM PREVIOUS PAGE ---																
	SILT, trace clay Loose to compact Grey Wet		17	SS	9												
			18	SS	14												
159.4	END OF BOREHOLE																
11.0	NOTES:  1. Water level in piezometer at Elev. 170.8 m (0.4 m above ground surface) on July 7, 2013.																

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+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity  
○ 3% STRAIN AT FAILURE



## Foundation Design

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+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity      ○<sup>3%</sup> STRAIN AT FAILURE

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+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity      ○<sup>3%</sup> STRAIN AT FAILURE



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[solutions@golder.com](mailto:solutions@golder.com)  
[www.golder.com](http://www.golder.com)

**Golder Associates Ltd.**  
**32 Steacie Drive**  
**Kanata, Ontario, K2K 2A9**  
**Canada**  
**T: +1 (613) 592 9600**



**Appendix C.**

**Site Photographs**

HIGHWAY 60 COCHRANE CREEK CULVERT REPLACEMENT  
4.5 KM WEST OF GOLDEN LAKE, RENFREW COUNTY

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**Photo 1. Looking southeast from the eastbound shoulder of Highway 60**



**Photo 2. Looking northwest from the westbound shoulder of Highway 60**



HIGHWAY 60 COCHRANE CREEK CULVERT REPLACEMENT  
4.5 KM WEST OF GOLDEN LAKE, RENFREW COUNTY

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**Photo 3. Looking west toward culvert outlet.**



**Photo 4. Looking east along westbound Highway 60 ditch line towards culvert inlet**

**Appendix D.**

**Foundation Comparison**

**COMPARISON OF ALTERNATIVE FOUNDATION TYPES**

<b><i>Concrete Rigid Box Culvert</i></b>	<b><i>Circular Pipe Culvert</i></b>	<b><i>Concrete Open Footing Culvert</i></b>
<p><i>Advantages:</i></p> <ul style="list-style-type: none"> <li>i. Smaller magnitude of settlement than open footing culvert due to lower bearing stress on subgrade.</li> <li>ii. Relatively expedient installation if precast units.</li> </ul>	<p><i>Advantages:</i></p> <ul style="list-style-type: none"> <li>i. Can tolerate larger magnitude of settlement than concrete (rigid frame) culverts).</li> <li>ii. Lower cost than concrete (rigid frame) culverts.</li> </ul>	<p><i>Advantages:</i></p> <ul style="list-style-type: none"> <li>i. Relatively expedient installation if precast units are used.</li> </ul>
<p><i>Disadvantages:</i></p> <ul style="list-style-type: none"> <li>i. Requires compacted granular pad on subgrade.</li> </ul>	<p><i>Disadvantages:</i></p> <ul style="list-style-type: none"> <li>i. CSP and HDPE pipes not as durable as concrete culverts.</li> <li>ii. Feasibility also depends on flow capacity and other hydraulic properties.</li> </ul>	<p><i>Disadvantages:</i></p> <ul style="list-style-type: none"> <li>i. Compressible founding subgrade will provide low geotechnical resistances.</li> <li>ii. Potential for post construction settlement.</li> </ul>
<b>Recommended</b>	<b>Generally Feasible</b>	<b>Not Recommended</b>

## **Appendix E.**

### **List of Special Provisions and OPSS Documents Referenced in this Report**

1. The following Special Provisions and OPSS Documents are referenced in this report:

- OPSS 206
- OPSS 422
- OPSS 501
- OPSS 539
- OPSS 804
- OPSS 902
- OPSS 1010
- OPSS 1205
  
- OPSD 208.010
- OPSD 803.010
- OPSD 3101.150