

October 2014

## REPORT ON

**Preliminary Foundation Investigation and Design  
Proposed Culvert Replacement  
Site No. 28-7C  
Highway 62 Culvert  
Pearsall Creek, Ontario  
W.P. 4119-09-01**

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REPORT

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**PRELIMINARY FOUNDATION REPORT  
HIGHWAY 62 CULVERT REPLACEMENT - SITE NO. 28-7C**

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**PART A**

**PRELIMINARY FOUNDATION INVESTIGATION  
PROPOSED CULVERT REPLACEMENT  
SITE NO. 28-7C  
PEARSALL CREEK, HIGHWAY 62  
ONTARIO  
W.P. 4119-09-01**



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### 1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by Dillon Consulting Limited on behalf of the Ministry of Transportation, Ontario (MTO) to carry out foundation investigations associated with numerous culvert and bridge replacements or 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 28-7c, which is located at the crossing of Pearsall Creek and Highway 62, south of Highway 401, in the Township of Prince Edward County, Ontario (WP 4119-09-01).

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 soil 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.



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## **2.0 SITE DESCRIPTION**

The existing culvert (Site no. 28-7c) is located at the crossing of Highway 62 over Pearsall Creek about 21 km south of Belleville and 12 km west of Picton in Prince Edward County, Ontario. The existing culvert location is shown on Drawing 1.

The existing culvert is a two-cell, open-bottom concrete frame structure which is about 20 m in length. Each culvert cell is approximately 2.3 m high by 6.1 m wide. The culvert was constructed around 1959. The base of the creek channel at the culvert location is at about Elevation 88 m and the flow in the culvert is from east to west. The depth of water within the culvert was about 1 m in August 2013.

The existing pavement grade at the culvert location is at about Elevation 91 to 92 m with a cross-fall generally from east to west across the road. In this area, Highway 62 is one lane wide in each direction (i.e., a 2-lane highway).

Based on information provided by Dillon, the areas of the culvert concrete visible during a May 2013 inspection were in fair to poor condition. Large areas of delamination and spalling, with reinforcing steel exposed in the soffit and barrel walls, were identified. Active leaking and efflorescence was also observed throughout the culvert. For these reasons, replacement of the existing culvert is recommended.

It is understood that a new two-cell culvert with similar dimensions to the existing culvert will be constructed at approximately the same general location as the existing culvert but the new culvert will be offset approximately 1 m to the south of the existing structure. It is further understood that there will not be any highway grade raise as part of the culvert replacement project; however, modifications to the retaining wall configuration and the highway embankment sideslope geometry are planned with the new embankment sideslopes constructed at 2 horizontal to 1 vertical (2H:1V).



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### 3.0 INVESTIGATION PROCEDURES

The subsurface investigation was carried out for the culvert replacement in June 2013, at which time four boreholes (numbered 13-1 to 13-4, inclusive) were advanced in the shoulders of the highway at the locations shown on Drawing 1. The boreholes 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 ranging from about 3.4 to 7.6 m below the existing ground surface.

Soil samples in the boreholes were obtained at vertical intervals of about 0.8 m of depth, using a 50 mm outer diameter (O.D.) split-spoon sampler in accordance with Standard Penetration Test (SPT) procedures, until auger refusal was encountered.

In Boreholes 13-2 and 13-3, the bedrock surface was then proven at depths of about 4.1 and 3.5 m, respectively, by rotary core drilling in NQ size, and the bedrock was penetrated for additional depths of about 3.5 and 3.1 m, respectively.

A standpipe piezometer was installed within the overburden soils at the location of Borehole 13-3 to allow for monitoring of the groundwater level at the site. The standpipe consists of a 25 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. The site conditions were restored following completion of the work.

The field work was supervised throughout by a member of Golder's technical staff, who located the boreholes, supervised the drilling, sampling and in-situ testing operations, logged the subsurface conditions encountered in the boreholes, and examined and cared for the soil and bedrock core samples. The samples were identified in the field, placed in appropriate containers, labelled, and transported to Golder's laboratories in Ottawa and Mississauga for further examination. Index and classification tests consisting of grain size distribution, water content, organic content and Atterberg limit testing were carried out on selected soil samples at Golder's Ottawa laboratory. Unconfined compression tests were carried out on two samples of the bedrock, one from each of Boreholes 13-2 and 13-3. This testing was carried out at Golder's Mississauga laboratory. 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 Ltd. using a Trimble R8 GPS unit. The borehole 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	South of Creek, West of Road	4877038.1	241861.0	91.39
13-2	South of Creek, East of Road	4877045.2	241871.2	92.13
13-3	North of Creek, West of Road	4877065.8	241855.9	91.49
13-4	North of Creek, East of Road	4877076.0	241864.9	92.29



## 4.0 SITE GEOLOGY AND STRATIGRAPHY

### 4.1 Regional Geological Conditions

The study area for this assignment is located within the physiographic region known as the *Prince Edward Peninsula*, as delineated in *The Physiography of Southern Ontario*<sup>1</sup>, which lies within the major physiographic region of Prince Edward County.

The Prince Edward Peninsula lies between the Napanee Plains and the Iroquios Plains. It is a flat lying area, with the highest point reaching slightly more than 76.2 m above the surface of Lake Ontario.<sup>1</sup> More than half of the Prince Edward Peninsula region has shallow soils over bedrock. Some parts of the county contain clay loam, till, clay deposits as well as marsh and other organic soils. This region is underlain by sedimentary rock of the Lindsay formation, consisting of limestone interbedded with shale. One small section of Precambrian granite is present near Ameliasburgh.

### 4.2 Site Stratigraphy

As part of the subsurface investigation at this site, four boreholes were advanced in the vicinity 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 displayed on the attached Record of Borehole sheets and on Figures 1 to 3.

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 crushed stone fill up to about 0.1 to 0.2 m in thickness (where present) overlying about 1.3 to 2.6 m of predominantly granular fill typically comprised of gravelly sand to sand and gravel. The granular fill materials are typically underlain by deposits of very loose to compact silty sand/sandy silt materials (probable fill); a piece of asphalt was encountered within these materials at the location of Borehole 13-3. The fill/probable fill materials typically extend to depths ranging from about 2.9 to 3.4 below the existing ground surface. Below this depth, deposits containing organic matter ranging in composition from silty peat to organic silty clay to silty sand containing trace to some peat were encountered to depths of 3.4 to 3.9 m at the locations of Boreholes 13-1, 13-2 and 13-4. A thin layer of sand and gravel was present beneath the peat in Borehole 13-2 and a silty sand till deposit was encountered below the organic soils in borehole 13-4. Limestone bedrock and/or auger refusal (inferred to be a result of encountering bedrock) was encountered at depths of 3.4 to 4.3 m corresponding to an elevation of about 88 m at all borehole locations.

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

<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.



#### **4.2.1 Pavement Structure and Granular Fill Materials**

Crushed stone fill with a thickness ranging from 100 to 200 mm was encountered at ground surface at all boreholes with the exception of Borehole 13-4 which was located north of the creek and east of the road on the roadway shoulder. The crushed stone is underlain by about 1.3 to 2.6 m of predominantly granular embankment fill typically consisting of silty sand and gravel to gravelly sand. Embankment fill of similar composition was encountered at surface at Borehole 13-4 and extended to 1.6 m below the existing ground surface at that location.

The granular fill was encountered to depths ranging from 1.3 to 2.6 m below the ground surface (corresponding to Elevations of about 89.5 to 90.7 m) at all borehole locations.

Standard Penetration Test (SPT) "N" resistance values measured in the granular fill ranged from 11 to 62 blows per 0.3 m of penetration indicating these materials are compact to very dense.

The results of grain size distribution testing carried out on three samples of the granular fill material are provided on Figure 1. The measured water contents of three samples of the fill ranged from approximately 4.5 to 6.0 percent.

#### **4.2.2 Silty Sand/Sandy Silt to Silty Sand and Gravel (Probable Fill)**

The predominantly granular embankment fill materials are underlain by materials varying in composition from silty sand/sandy silt containing trace to some clay and gravel to silty sand and gravel at all borehole locations. These materials were generally brown in colour but had a mottled/mixed appearance in several locations. A piece of asphalt was encountered within a sample collected from a depth of about 2.5 m below ground surface in Borehole 13-3.

Based on the presence of asphalt in Borehole 13-3, the mixed colour/appearance of the materials and the presence of underlying native organic deposits at the majority of the boreholes, the silty sand/sandy silt and silty sand and gravel soils are inferred to be fill. These deposits extend to depths of about 2.9 to 3.5 m below the existing ground surface (corresponding to Elevations of about 88.1 to 89.0 m).

SPT N values measured within the silty sand to sandy silt ranged from 3 to 11 blows per 0.3 m of penetration indicating the material is very loose to compact. A SPT 'N' resistance value of 28 blows per 0.3 m of penetration was measured within the silty sand and gravel materials.

The results of grain size distribution testing carried out on three samples of the silty sand (probable fill) material are provided on Figure 2. The measured water contents of three samples of the granular deposits ranged from approximately 13 to 19 percent. The results of Atterberg limit testing on the fine-grained portion of a sample of these materials from Borehole 13-3 measured a plastic limit value of about 12 percent and a liquid limit value of about 20 percent indicating a soil of low plasticity.

#### **4.2.3 Organic Deposits**

Deposits comprised of, or containing, organic soils were encountered beneath the inferred fill at the locations of Boreholes 13-1, 13-2 and 13-4. The organic deposits are highly variable in composition ranging from sandy silt/silty sand containing peat (Borehole 13-1) to silty peat (Borehole 13-2) to clayey peat and organic silty clay (Borehole 13-4). The organic deposits are about 0.5 to 0.6 m thick and were encountered to depths of 3.4 to 3.9 m below ground surface corresponding to Elevations of about 88 to 88.5 m.



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SPT "N" resistance values of 5 blows per 0.3 m of penetration were measured within the organic soils at Boreholes 13-2 and 13-4.

The moisture contents of the organic soils varied from 43 to 147 percent. The organic contents of three samples of the organic deposits were measured to vary from about 7 to 48 percent.

### 4.2.4 Sand and Gravel

A sand and gravel deposit was encountered beneath the silty peat deposit in Borehole 13-2 at a depth of 3.9 m below ground surface. The sand and gravel layer was approximately 0.2 m thick and extended to the underlying bedrock surface at an elevation of about 88 m.

A SPT "N" resistance value of greater than 50 blows was measured within the sand and gravel indicating these materials are very dense.

### 4.2.5 Glacial Till

Glacial till was encountered beneath the organic soils at Borehole 13-4 on the northern side of Pearsall Creek. The deposit is considered to consist of a heterogeneous mixture of gravel, cobbles, and boulders in a silty sand matrix.

A SPT "N" resistance value of 33 blows per 0.2 m of penetration was measured within the glacial till indicating these materials are dense.

### 4.2.6 Bedrock

Bedrock or auger refusal on the inferred bedrock surface was encountered beneath the fill in Borehole 13-3 and below the native overburden soils at all other borehole locations. The bedrock was cored in Boreholes 13-2 and 13-3 for lengths of 3.5 and 3.1 m, respectively.

The following table summarizes the bedrock surface depths and elevations as encountered at the two borehole locations.

Borehole Number	Existing Ground Surface Elevation (m)	Depth to Bedrock (m)	Bedrock Surface Elevation (m)
13-1	91.39	3.4	88.0 <sup>R</sup>
13-2	92.13	4.1	88.0
13-3	91.49	3.5	88.0
13-4	92.29	4.3	88.0 <sup>R</sup>

**Note:** <sup>R</sup> Inferred based on auger refusal

The bedrock encountered in the boreholes consists of limestone bedrock. The bedrock is typically fresh and strong to very strong.

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The Rock Quality Designation (RQD) values measured on the recovered bedrock core samples typically ranged from about 52 to 93 percent, indicating good to excellent quality rock; however, RQD values of 0 were measured over two small (~0.2 m thick) sections of the bedrock core at Boreholes 13-2 and 13-3 each located within the uppermost metre of the bedrock. Further details on discontinuities/joints in the rock mass are identified on the Record of Drillhole Sheets in Appendix A. The discontinuities are typically sub-horizontal and associated with bedding.

The results of UCS testing carried out on two selected core samples indicate a compressive strength of 68.1 and 82.3 MPa for each individual core sample.

#### **4.2.7 Groundwater Conditions**

The groundwater level in the piezometer in Borehole 13-3 was measured on July 10, 2013. The piezometer was sealed into the fill materials.

The groundwater level in the piezometer is summarized in the table below.

<b>Borehole</b>	<b>Ground Surface Elevation (m)</b>	<b>Water Level Depth (m)</b>	<b>Water Level Elevation (m)</b>	<b>Date</b>
13-3	91.5	2.4	89.1	July 7, 2013

The water level in the creek is understood to be at an elevation of 88.9 m in August 2103.

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

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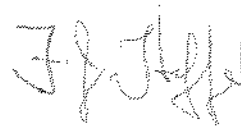
## **5.0 CLOSURE**


This preliminary report was prepared by Ms. Katya Edney and Mr. Kevin Nelson, P.Eng., and was reviewed by Mr. Fintan Heffernan, P.Eng., the designated MTO contact for this project.

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**PART B**

**PRELIMINARY FOUNDATION DESIGN  
PROPOSED CULVERT REPLACEMENT  
SITE NO. 28-7C  
PEARSALL CREEK, HIGHWAY 62  
ONTARIO  
W.P. 4119-09-01**

## 6.0 DISCUSSION AND PRELIMINARY ENGINEERING RECOMMENDATIONS

### 6.1 General

This section of the report provides preliminary foundation design recommendations for the design-build assignment for the proposed replacement of the existing Pearsall Creek culvert on Highway 62. 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 preliminary design of the project. 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.

A new two-cell replacement culvert, with cell openings of approximately 2.3 by 6.1 m, is planned to be constructed along essentially the same alignment as, but offset approximately 1 m to the south of, the existing culvert alignment which is shown on Drawing 1. The proposed foundations will also be offset from the existing foundations which are planned to be removed.

The proposed invert level is expected to be similar to the existing creek invert at about Elevation 88 m. It is further understood that the road grade will not be raised as part of the culvert replacement project; however, modifications to the retaining wall configuration and the highway embankment sideslope geometry are planned. The new retaining walls are planned to be constructed parallel to the highway. The highway embankments adjacent to the retaining walls will be constructed at sideslopes of 2 horizontal to 1 vertical (2H:1V) and provided with a surficial layer of rock protection.

The culvert replacement is planned to be carried out in stages with traffic shifted to one-side of the highway during reconstruction of the culvert on the other side. Temporary protection systems (Performance Level 2) would be provided adjacent to the active traffic lanes during this staged construction.

### 6.2 Foundation Options

The existing Pearsall Creek two-cell culvert was built circa 1959. The existing culvert is understood to be founded on shallow foundations founded slightly below Elevation 88 m and, based on the bedrock elevations identified during the foundation investigation, the existing foundations are expected to be bear on the bedrock.

Based on the subsurface conditions, only shallow foundation options have been considered for the replacement of the Pearsall Creek culvert. Deep foundations are not required or recommended as shallow foundations supported on the bedrock will provide sufficient bearing resistance and acceptable settlement performance for the proposed culvert replacement.

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



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**Cast-In-Place/Precast Rigid Frame, Open Footing Culvert Founded on the Bedrock:** The bedrock surface was encountered at an elevation of approximately 88 m in all boreholes advanced at this site which is at or near to the elevation of the base of the creek channel. For this configuration, it is considered feasible to replace the culvert with a rigid frame, open bottom culvert founded on shallow foundations bearing on the limestone bedrock. Foundations for this type of structure could be founded near the surface of the bedrock which would reduce bedrock excavation in comparison to a concrete box structure. High geotechnical resistances can be achieved for foundations bearing on the bedrock with negligible settlement of the footings. Cast-in-place culvert foundations would be preferred over the use of precast culvert foundations for this option in order to account for potential variations in the bedrock surface in the immediate vicinity of the creek channel. A precast open footing culvert supported on cast-in-place footings could also be considered as a viable foundation option for this site; however, it is understood that, for a two-cell culvert, a cast-in-place superstructure is preferred from a construction perspective. Temporary protection systems and cofferdams would be required during excavation and construction of the culvert foundations.

**Precast Concrete Box Culvert Founded on a Granular Pad on the Bedrock:** A box culvert could be considered for the culvert replacement provided it is founded on or within the bedrock. A working/levelling pad of granular materials or lean concrete would be required for the box culvert installation. Therefore, if the culvert/creek invert level is maintained, increased bedrock removal (in comparison to an open culvert/footing configuration) would be required across the width of the culvert opening in order to accommodate the working pad and the box culvert base slab. Drill and blast procedures could be required given the strong to very strong limestone bedrock present at this site. Relatively lower geotechnical resistances will also apply for the pad of granular fill on the bedrock as opposed to founding on the bedrock itself with potential for some settlement of the culvert. As above, it is expected that temporary protection systems and/or cofferdams would be required during excavation and construction. 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 resulting in a shorter construction period and less disruption to traffic.

Based on the above considerations, the preferred option from a geotechnical/foundations perspective is to replace the culvert with a rigid frame concrete, open bottom culvert supported on shallow foundations bearing on the bedrock.

### 6.3 Culvert Foundation Options

#### 6.3.1 Cast-in-Place/Precast Rigid Frame Open Footing Culvert Founded on Bedrock

##### 6.3.1.1 *Founding Level and Frost Protection Requirements*

Strip footings for an open footing culvert replacement should be founded on the limestone bedrock. Similarly, any associated concrete retaining walls should also be founded on the limestone bedrock to reduce the potential for differential settlement between these walls and the culvert.

As per Ontario Provincial Standard Drawing (OPSD) 3090.101 (*Foundation Frost Depths for Southern Ontario*) frost penetration depth in the area is 1.4 m. Typically this would require footings to be provided with a minimum of 1.4 m of earth cover to provide adequate protection against frost penetration. However, this requirement can be waived where the founding level on the bedrock is above the frost depth as the bedrock at this site does not appear to contain any seams of frost susceptible soil.



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Bedrock was encountered at an elevation of approximately 88 m in all boreholes. The preliminary General Arrangement drawing for the culvert replacement indicates that shallow foundations for an open footing culvert would be founded at an elevation of approximately 87.4 m. If acceptable from structural, scour protection and resistance to sliding perspectives, the footings could also be founded near the surface of the bedrock (following removal of any loose or highly fractured bedrock) in order to minimize bedrock removal. However, the design should include provision for localized lowering of the foundations in the event that the bedrock surface is variable and/or that bedrock of suitable quality for foundation support is lower than encountered at the borehole locations.

The footing subgrade should be inspected in accordance with OPSS 902 (*Construction Specification for Excavating and Backfilling – Structures*). Further discussion regarding subgrade preparation and protection is provided in Section 6.7.1.

### 6.3.1.2 Geotechnical Resistances

For footings founded on the bedrock at the elevations provided in Section 6.3.2.1, a factored geotechnical resistance at ULS of 5 MPa may be used for design purposes. SLS resistances do not apply to the design of footings on the strong to very strong limestone bedrock, because the SLS resistance for 25 mm of settlement is greater than the factored axial geotechnical resistance at ULS.

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.

### 6.3.2 Precast Concrete Box Culvert

#### 6.3.2.1 Founding Level and Bedding

For a box culvert, 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 precast concrete box culverts. It is recommended that the box culvert segments be placed on a pad of granular bedding material meeting the requirements of OPSS 1010 Granular A.

Bedrock was encountered at an elevation of approximately 88 m in all boreholes which is near to the existing creek invert level. Assuming a base slab thickness of 300 mm and the installation of a bedding layer as described above, bedrock removal would be required to reach the base of the excavation for the granular leveling pad. All organic materials and creek bed sediments should be removed from beneath the plan area of the box culvert prior to placement of the working/leveling pad.

The foundation subgrade should be inspected in accordance with OPSS 902 (*Construction Specification for Excavating and Backfilling – Structures*).





### 6.3.2.2 Geotechnical Resistances

For a box culvert founded on a thin granular levelling pad above the bedrock, a geotechnical resistance at Serviceability Limit States (SLS) for 25 mm of settlement of 500 kPa may be used for preliminary design purposes. The factored geotechnical resistance at ULS would exceed 1 MPa and would not govern the design.

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.

## 6.4 Seismic Design Considerations

The site is located about 12 km west of Picton, Ontario. According to Table A.3.1.1. of the Canadian Highway Bridge Design Code (CHBDC), the zonal acceleration,  $A$ , applicable to this site is 0.05 and the corresponding acceleration related seismic zone,  $Z_a$ , is 1. The importance category of the culvert is understood to be "other" based on the current version of the CHBDC. Therefore, the corresponding seismic performance zone (SPZ) for this importance category is 1.

The culvert will be founded on bedrock. Based these conditions, the soil profile type for this site is categorized as Type I with a seismic site response coefficient,  $S$ , of 1.0 based on CHBDC criteria.

## 6.5 Settlement

The replacement culvert and retaining walls should be founded on the bedrock or a thin levelling pad overlying the bedrock. Provided the SLS geotechnical resistance for the culvert are limited to the values provided in Section 6.3, then the total and differential culvert settlements should be minimal (i.e., less than about 25 and 15 mm, respectively) with the majority of the settlement occurring during construction.

It is understood that the embankment sideslopes adjacent to the new retaining walls will be widened slightly and flattened to final slopes of 2H:1V. For an open footing culvert replacement, the culvert and wing-wall footings will be founded on the bedrock; therefore, settlements of these foundations should be negligible even with the proposed widening and future grade raise.

## 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 902 (*Construction Specification for Excavating and Backfilling – Structures*) and/or OPSD 803.010 (*Backfill and Cover for Concrete Culverts*).

Backfill to culvert walls and retaining 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.

For the box culvert option, where the culvert is founded on a pad of granular fill on the bedrock, a concrete cut-off wall/apron or clay seal should be provided at the upstream end of the culvert replacement 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). Where the culvert is founded on the bedrock (i.e., for the rigid frame open footing culvert option) and incorporates wingwalls that are also founded on bedrock, it is not considered necessary to provide a clay seal or concrete cut-off wall at the upstream end of the culvert.

If the flow velocities are sufficiently high, a 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 OPSP 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 reconstruction of the highway embankment sideslopes in the vicinity of the culvert and retaining walls is planned. The embankments will be about 3 to 3.5 m in height relative to the original ground surface and are planned to be sloped at about 2H:1V for granular fill which may require some minor widening near the toe of the existing embankment slopes. The subsurface conditions beneath the embankment reconstruction areas are expected to typically consist of fill and native organic soils overlying shallow bedrock. Any topsoil, peat/organic matter or softened/loosened soils should be stripped from below the embankment widening areas prior to placement of any new fill materials in the embankment toe/sideslope areas.

The fill for the embankment widening areas adjacent to the culvert should be placed and compacted in accordance with OPSS.PROV 206 and OPSS501. Benching of the existing embankment side slopes should be carried out to "key in" the new fill materials for the widening, in accordance with OPSP 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; any fill containing organic matter should be wasted. Additional fill for construction of the embankment widening above the level of the original ground surface could consist of clean earth fill, granular fill or rock fill.

Replacement of topsoil and seeding or pegged sod is recommended to be provided on the surface of the reconstructed embankment slopes to reduce surface water erosion.

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., greater than 1.3 under static conditions, and 1.1 under seismic conditions).

### 6.7.1 Embankment Settlement

Layers of peat/organic soils were encountered beneath the existing embankment fill materials at several of the borehole locations. Ongoing settlements of these organic soils resulting from the existing embankments, which are understood to have been in place for over 50 years, are expected to be nominal. However, the application of new loads above these organic soils (e.g., as a result of grade raises) would result in compression of the organic soils, increased settlement of the roadway surface and differential settlement between the highway



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embankment and a culvert founded on bedrock. In this regard, it is recommended that the highway/road grade is not raised; it is understood that no grade raise is planned in the plan area or immediate vicinity of the travelled surface of the roadway. Replacement of existing embankment fill with heavier fill materials (e.g., with Type I or Type II materials within the culvert and wing-wall backfill zones) could also lead to compression and settlement of the peat/organic materials which could result in differential settlement between the culvert/retaining walls founded on bedrock and the adjacent portions of the highway embankment. Therefore, it is recommended that the existing organic soils located beneath the highway surface within the extents of the planned retaining walls (i.e., within a zone extending back 6 m from the culvert) be subexcavated and replaced with engineered fill to limit the potential for such differential settlement to occur.

It is understood that the sideslopes of the highway embankments adjacent to the new culvert retaining walls will be reconstructed with sideslopes of approximately 2H:1V. Minor amounts of new fill materials (i.e., sliver fills) will be placed over the existing embankment sideslopes to facilitate the regrading in this area. As noted above, removal of any organic soils is recommended in areas of new fill placement for stability purposes.

Settlement of the reconstructed/regraded portions of the embankment sideslopes will also occur as a result of compression of the new embankment fill. 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.

Where rock fill is used, 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 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 planning and preliminary design as well as the future design stages.

### 6.8.1 Subgrade Preparation

Subgrade preparation should be performed and monitored in accordance with OPSS 902 (*Construction Specification for Excavating and Backfilling – Structures*).

Footings, or leveling pads for box culverts, must be constructed on undisturbed bedrock that is relatively free of defects. All embankment fill, topsoil, organics and soft or loose soils should be removed from below the proposed founding elevations to expose the underlying bedrock and wasted or reused as landscaping fill, as required. The bedrock bearing surface should also be cleared of all loose and broken rock. The cleaned excavation base should be inspected prior to placing concrete for the footings for the rigid frame open footing culvert or granular bedding for the box culvert.



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As an alternative to the placement of a Granular A levelling pad for a box culvert, a 100 mm thick concrete working slab could be placed on the subgrade within the culvert footprint, to protect the subgrade from degradation. In this case, a 75 mm thick layer of OPSS 1010 Granular A or concrete fine aggregate meeting the gradation requirements set out in OPSS 1002 (*Material Specification for Aggregates – Concrete*) should be placed on top of the concrete mat to provide a "levelling pad" for the box culvert replacement. The working slab should be placed within four hours after inspection and approval of the subgrade.

### 6.8.2 Groundwater and Surface Water Control

Control of the surface water and groundwater will be necessary for the construction of the culvert replacement, to allow excavation, foundation construction and fill placement to be carried out in dry conditions.

Cofferdams are planned to be constructed at the upstream and downstream ends of the existing culvert to direct creek water into one cell of the existing culvert at a time in order to allow for foundation construction and removal of the existing foundations to be carried out under 'dry' conditions. Once construction activities are completed on side of the culvert, the cofferdams will be relocated to direct water into the other culvert cell.

The water level measured at the site was above the base of the existing embankment fill materials. Therefore, some groundwater inflow into the excavations through the embankment fill materials will occur. Groundwater seepage will also occur through discontinuities in the bedrock. It should be possible to handle the groundwater inflow by pumping from well-filtered sumps established in the floor of the excavations, provided that an appropriate cut-off/cofferdam is in place between the culvert foundation excavations and the creek.

Surface water should be directed away from the excavation area, to prevent ponding of water that could result in disturbance and weakening of the foundation subgrade.

### 6.8.3 Excavation and Temporary Protection Systems

Temporary excavations for the culvert, up to a depth of about 3.5 m below current road grades, will be made through the existing fill, organic soils, silty sand and till. Localized excavations into the bedrock will be required for foundation construction. 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 material, organic soils, and granular soils 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. If the above open cut excavation side slopes cannot be accommodated, then temporary protection systems (i.e., temporary excavation shoring) will be required.

Bedrock removal is expected to be required, at least locally, for foundation construction. Bedrock excavation could be carried out using line drilling and pre-shearing or blasting techniques, given the strong to very strong limestone bedrock at this site.

Based on the proposed, staged culvert construction, temporary protection systems will be required adjacent to the active highway lanes. These support systems should be designed and constructed by the contractor in accordance with OPSS 539. The lateral movement of the temporary shoring system should meet Performance Level 2 as specified in OPSS 539 provided that any utilities that may be present in the area can tolerate this magnitude of deformation.



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## PRELIMINARY FOUNDATION REPORT HIGHWAY 62 CULVERT REPLACEMENT - SITE NO. 28-7C

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A conventional shoring system for these conditions could consist of soldier piling and lagging or interlocking steel sheet piling supported against lateral movement using walers, tie backs and/or internal struts/braces. For a soldier pile and lagging system, it would be necessary to control seepage or include measures to mitigate loss of soil particles through the lagging boards. The toes of the soldier piles would also need to be socketed into the bedrock. Interlocking steel sheet piling would contribute to both ground and groundwater control but the sheet piles would not penetrate significantly into the bedrock and, therefore, support/pinning of the toes of the sheet piles is expected to be required. The selection and design of the shoring system is the responsibility of the contractor.

### 6.8.4 Existing Foundations

The culvert is understood to be founded on strip footings bearing on the bedrock. The new culvert foundations will be offset slightly from the existing foundations. The existing foundations are planned to be removed as part of the new culvert construction. If removal of the existing foundations results in excavations that extend below the founding level of the new footings, all materials disturbed as a result of should be subexcavated to expose the underlying bedrock and replaced with lean mix concrete up to the founding level of the new footings.

### 6.8.5 Obstructions

The existing embankment fill materials were noted to contain cobbles. Similarly the till deposits are expected to contain cobbles and/or boulders. These materials could affect the installation of the protection systems.

## 6.9 Recommendations for Further Work in Detail Design

The design-build proponent will be responsible for the detail design and assessing additional requirements for investigations to suit the final design and mitigating any identified construction risks. However, at this functional design stage, it is anticipated that additional investigation will be required during the design-build 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 the bedrock surface to confirm the founding elevations in the immediate vicinity of the culvert/creek channel area.
- Assessment of the presence, thickness and properties of organic soil layers in areas where the highway embankment sideslope geometry will be modified/new embankment fill materials will be placed.

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HIGHWAY 62 CULVERT REPLACEMENT - SITE NO. 28-7C

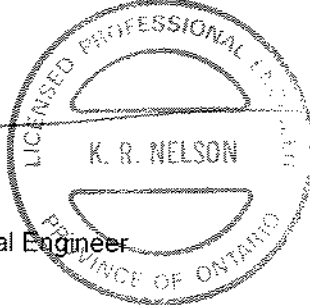
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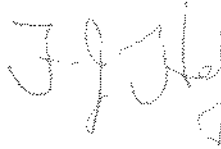
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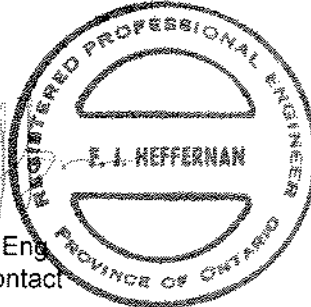
This preliminary report was prepared by Mr. Kevin Nelson, P.Eng., and was reviewed by Mr. Fintan Heffernan, P.Eng., the designated MTO contact for this project.

### GOLDER ASSOCIATES LTD.

  
Kevin Nelson, P.Eng.  
Associate, Geotechnical Engineer



  
Fintan Heffernan, P.Eng.  
MTO Designated Contact



KE/KN/FJH/bg

n:\active\2012\1121 - geotechnical\12-1121-0193 dillon mega 3 eastern region\foundations\5 - reports\contract c - highway 62 site 28-7c\12-1121-0193-1115 site 28-7c pearsall creek final october 2014.docx

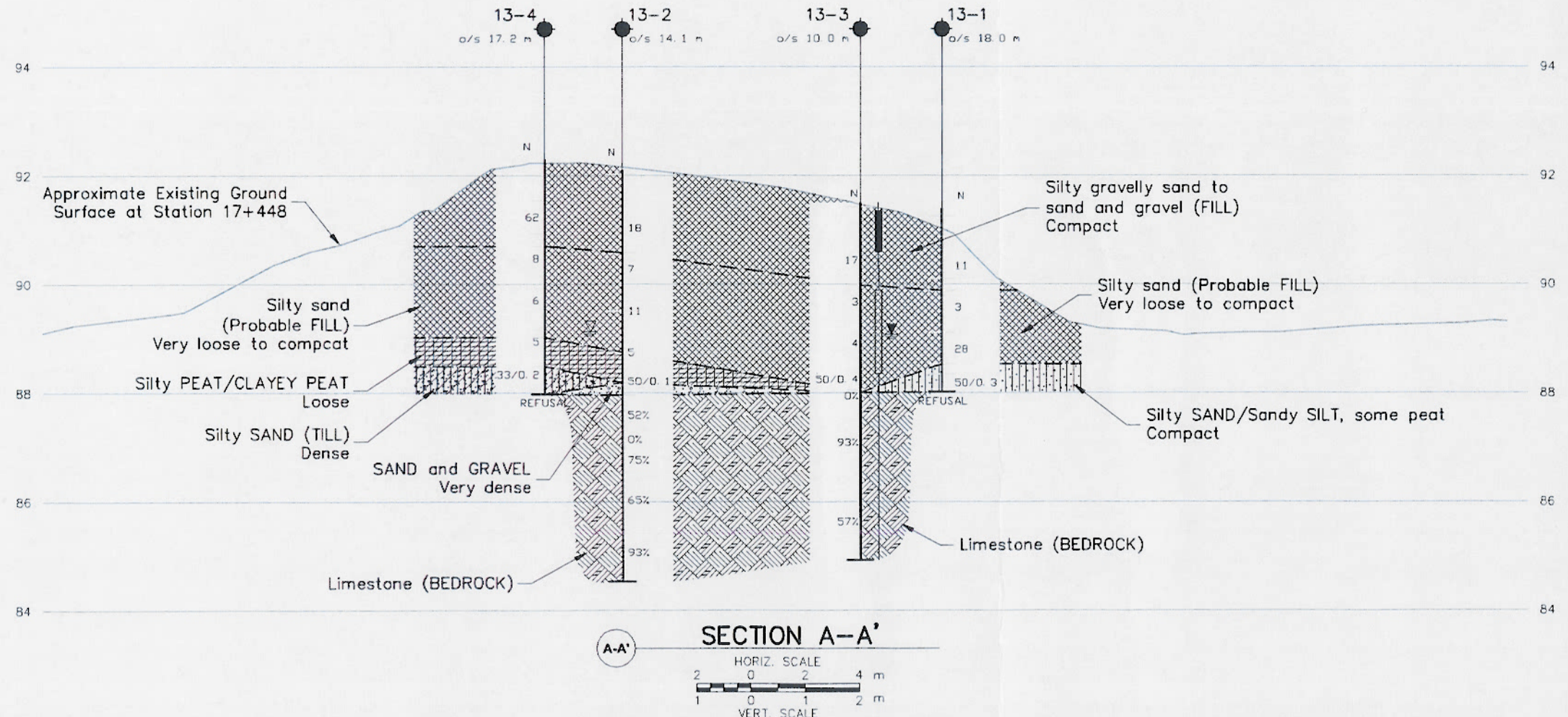
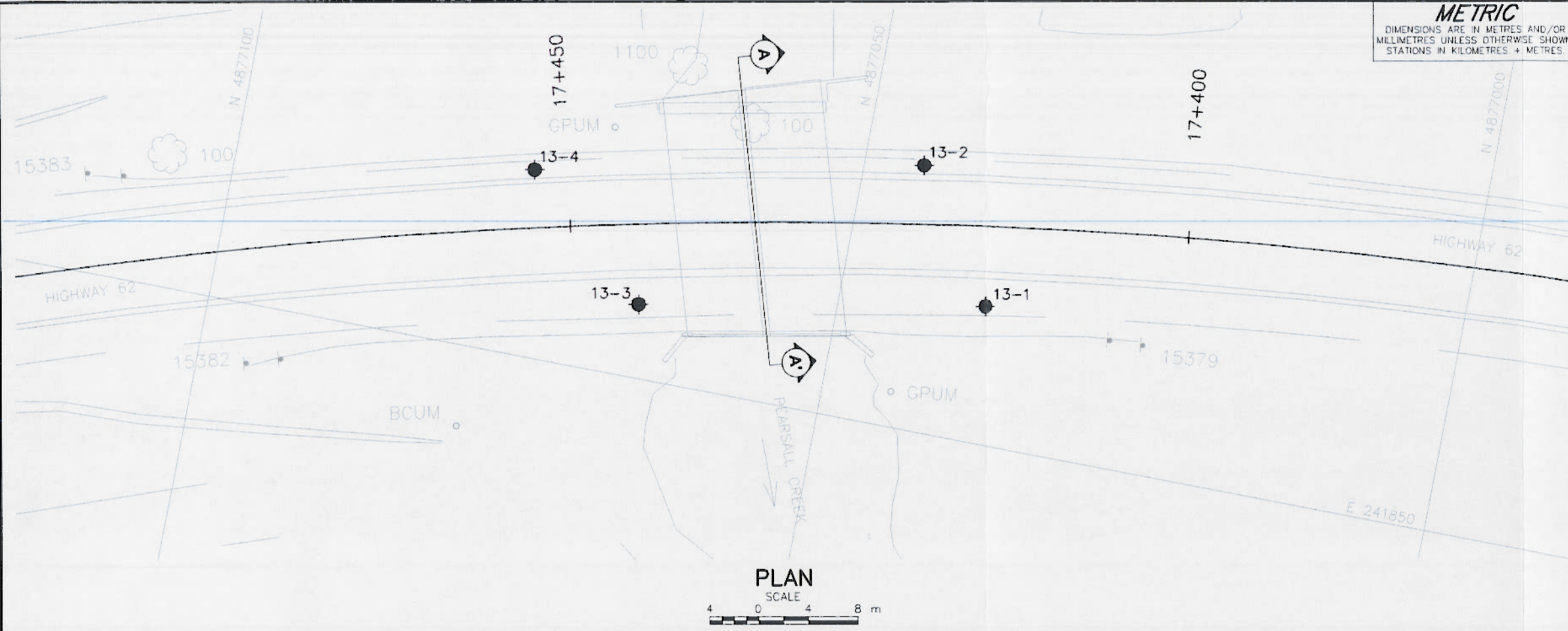
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**PRELIMINARY FOUNDATION REPORT  
HIGHWAY 62 CULVERT REPLACEMENT - SITE NO. 28-7C**

**Table 1  
Comparison of Foundation Alternatives  
Pearsall Creek Culvert Replacement  
W.P. 4119-09-01**

Foundation Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risks/Consequences
<b>Option 1</b> Rigid Frame Open Footing	<ul style="list-style-type: none"> <li>Feasible, preferred option</li> </ul>	<ul style="list-style-type: none"> <li>High geotechnical resistances</li> <li>Minimal settlement</li> <li>Minimizes amount of bedrock excavation</li> </ul>	<ul style="list-style-type: none"> <li>Groundwater and creek water control systems required.</li> <li>Bedrock present at, or above, foundation excavation level. Lower portions of temporary protection systems will need to either be drilled/socketed into bedrock (for soldier piles) or provided with pins/dowels (for sheet piles) into the bedrock to provide sufficient lateral support.</li> </ul>	<ul style="list-style-type: none"> <li>Low to Moderate cost</li> </ul>	<ul style="list-style-type: none"> <li>Generally low risk option (except for shoring)</li> <li>Risk of variations in bedrock surface near creek channel affecting founding depths</li> </ul>
<b>Option 2</b> Concrete Box	<ul style="list-style-type: none"> <li>Feasible</li> </ul>	<ul style="list-style-type: none"> <li>High geotechnical resistances</li> <li>Small culvert settlement</li> <li>Use of pre-cast members could reduce construction time</li> </ul>	<ul style="list-style-type: none"> <li>Deeper founding levels required for hydraulic design will result in increased requirement for bedrock excavation in comparison to open bottom culvert. Drill and blast methods may be needed for bedrock removal.</li> <li>Also requires roadway protection system (same comments as for Option 1).</li> </ul>	<ul style="list-style-type: none"> <li>Moderate cost</li> </ul>	<ul style="list-style-type: none"> <li>Generally low risk option (except for shoring)</li> <li>Risk of increased groundwater seepage being encountered due to larger bedrock excavation</li> </ul>
<b>Option 3</b> Deep Foundations	<ul style="list-style-type: none"> <li>Feasible but not required/practical</li> </ul>	<ul style="list-style-type: none"> <li>Minimal culvert settlement</li> </ul>	<ul style="list-style-type: none"> <li>Pre-drilling into bedrock would be required for deep foundation installations. Potential for encountering increased groundwater flows within highly fractured bedrock zones.</li> <li>Also requires roadway protection system (same comments as for Option 1).</li> </ul>	<ul style="list-style-type: none"> <li>Most expensive option</li> </ul>	<ul style="list-style-type: none"> <li>Low risk option</li> </ul>





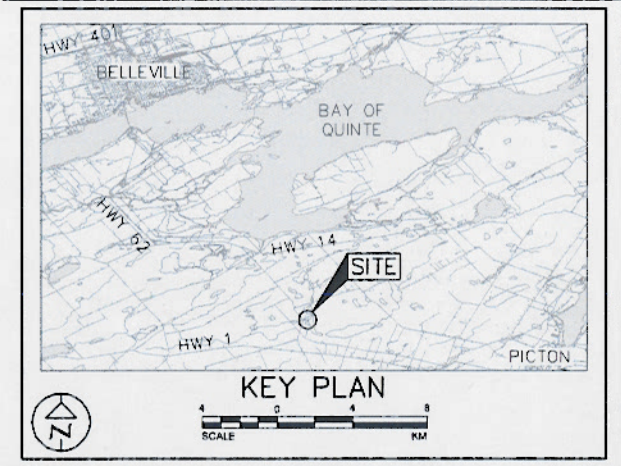
**METRIC**  
 DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

**CONT No.**  
WP No. 4119-09-01

**SHEET**

**PEARSCALL CREEK CULVERT  
BOREHOLE LOCATIONS AND SOIL STRATA**

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



- LEGEND**
- Borehole - Current Investigation
  - 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 10, 2013
  - WL upon completion of drilling
  - Seal
  - Piezometer

BOREHOLE CO-ORDINATES			
No.	ELEVATION	NORTHING	EASTING
13-1	91.4	4877036.1	241861.0
13-2	92.1	4877045.2	241871.2
13-3	91.5	4877065.8	241855.9
13-4	92.3	4877076.0	241864.9

**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.

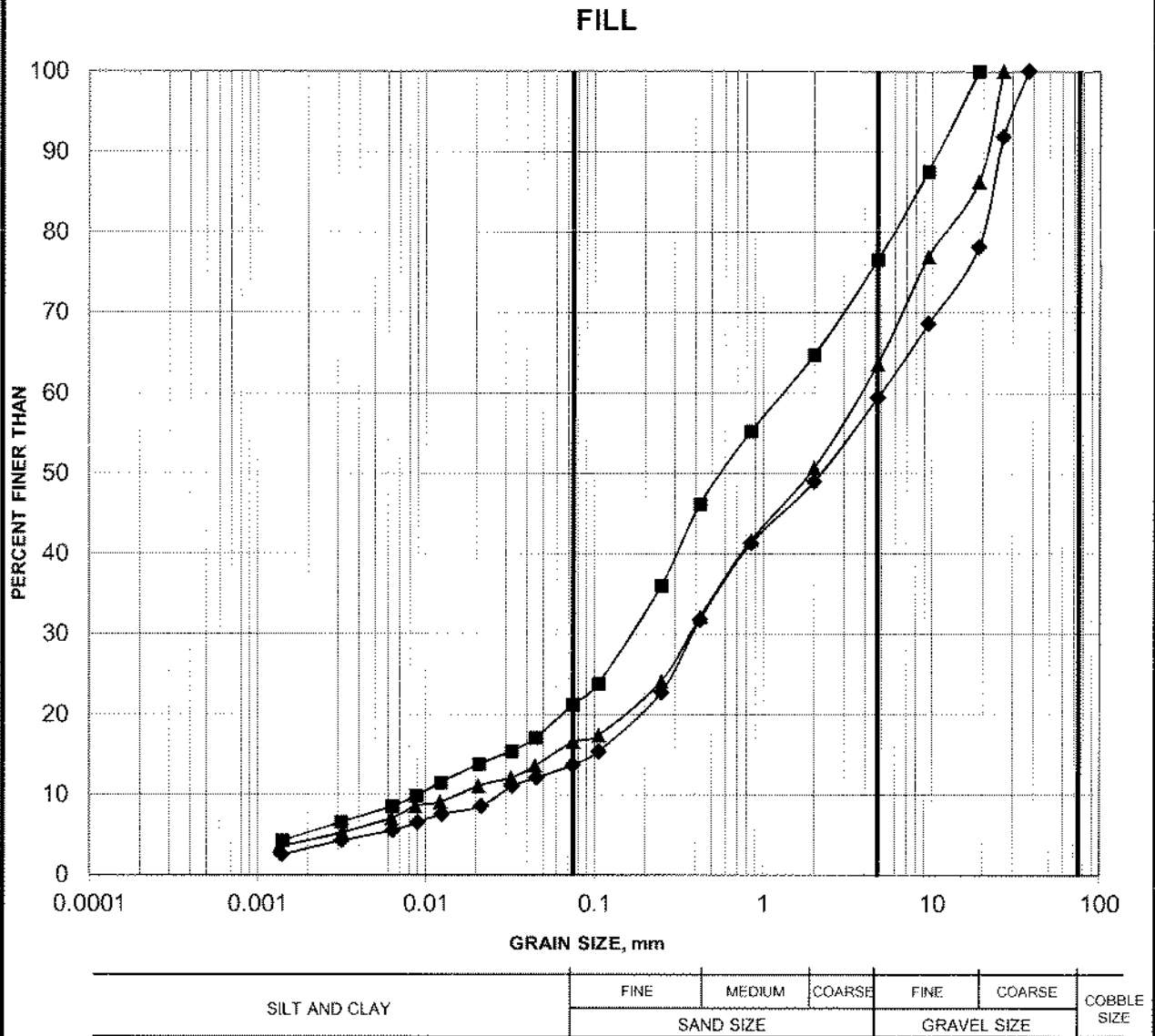
The complete Foundation Investigation and Design Report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with Section GC 2.01 of OPS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

REFERENCE			
Base plan provided in digital format by Dillon, drawing file no. 4119-Base.dwg and 4119-Photogrammetry.dwg, received October 11, 2013.			
NO.	DATE	BY	REVISION
1			
Geocres No. 31c-226			
HWY. 62	PROJECT NO. 12-1121-0193		DIST.
SUBM'D. KN	CHKD. KE	DATE: 10/1/2014	SITE: 28-7C
DRAWN: JM	CHKD. KN	APPD. FJH	DWG. 1



# GRAIN SIZE DISTRIBUTION

FIGURE 1

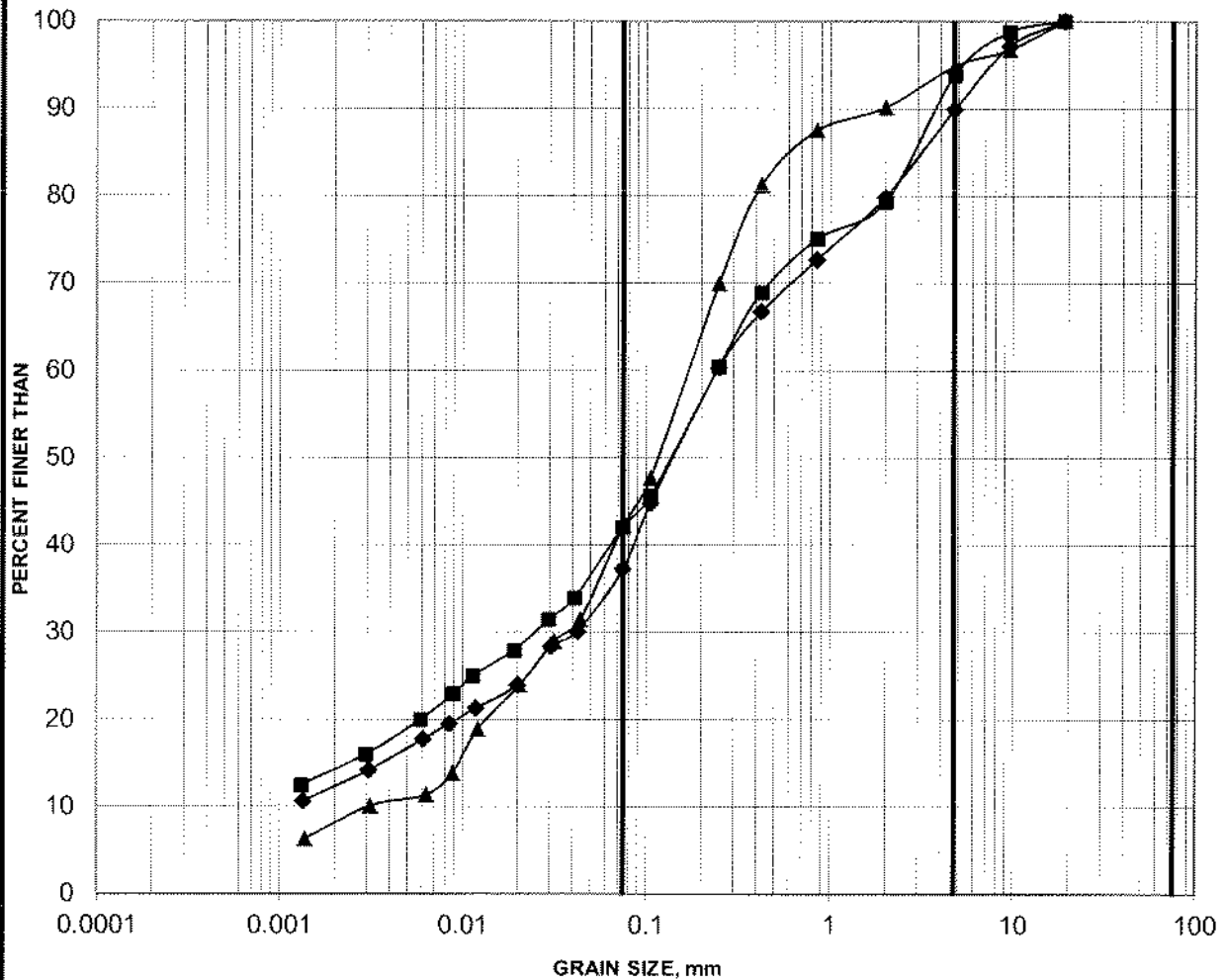


Borehole	Sample	Depth (m)
13-1	1	0.76-1.37
13-2	1	0.76-1.37
13-4	1	0.76-1.37

# GRAIN SIZE DISTRIBUTION

FIGURE 2

## SILTY SAND (PROBABLE FILL)



SILT AND CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
	SAND SIZE			GRAVEL SIZE		

Borehole	Sample	Depth (m)
13-1	2	1.52-2.13
13-3	2	1.52-2.13
13-4	3	2.29-2.90



# APPENDIX A

List of Abbreviations and Symbols  
Record of Borehole and Drillhole Sheets

## LIST OF ABBREVIATIONS

The abbreviations commonly employed on Records of Boreholes, on figures, and in the text of the report are as follows:

### I. SAMPLE TYPE

AS	Auger sample
BS	Block sample
CS	Chunk sample
DO or DP	Seamless open-ended, driven or pushed tube samplers
DS	Denison type sample
FS	Foil sample
RC	Rock core
SC	Soil core
SS	Split spoon sampler
ST	Slotted tube
TO	Thin-walled, open
TP	Thin-walled, piston
WS	Wash sample
DT	Dual tube sample
DD	Diamond drilling

### II. PENETRATION RESISTANCE

#### Standard Penetration Resistance (SPT), N:

The number of blows by a 63.5 kg. (140 lb.) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) split spoon sampler for a distance of 300 mm (12 in.).

#### Dynamic Cone Penetration Resistance (DCPT); $N_d$ :

The number of blows by a 63.5 kg (140 lb.) hammer dropped 760 mm (30 in.) to drive an uncased 50 mm (2 in.) diameter, 60° cone attached to "A" size drill rods for a distance of 300 mm (12 in.).

<b>PH:</b>	Sampler advanced by hydraulic pressure
<b>PM:</b>	Sampler advanced by manual pressure
<b>WH:</b>	Sampler advanced by static weight of hammer
<b>WR:</b>	Sampler advanced by weight of sampler and rod

#### Cone Penetration Test (CPT):

An electronic cone penetrometer with a 60° conical tip and a projected end area of 10 cm<sup>2</sup> pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance ( $q_t$ ), porewater pressure ( $u$ ) and friction along a sleeve are recorded electronically at 25 mm penetration intervals.

### III. SOIL DESCRIPTION

#### (a) Cohesionless Soils

Density Index (Relative Density)	N Blows/300 mm Or Blows/ft.
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	over 50

#### (b) Cohesive Soils $C_u$ or $S_u$

Consistency	kPa	Psf
Very soft	0 to 12	0 to 250
Soft	12 to 25	250 to 500
Firm	25 to 50	500 to 1,000
Stiff	50 to 100	1,000 to 2,000
Very stiff	100 to 200	2,000 to 4,000
Hard	Over 200	Over 4,000

### IV. SOIL TESTS

w	Water content
$w_p$ or PL	Plastic limited
$w_l$ or LL	Liquid limit
C	Consolidation (oedometer) test
CHEM	Chemical analysis (refer to text)
CID	Consolidated isotropically drained triaxial test <sup>1</sup>
CIU	Consolidated isotropically undrained triaxial test with porewater pressure measurement <sup>1</sup>
$D_R$	Relative density
DS	Direct shear test
Gs	Specific gravity
M	Sieve analysis for particle size
MH	Combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	Organic content test
SO <sub>4</sub>	Concentration of water-soluble sulphates
UC	Unconfined compression test
UU	Unconsolidated undrained triaxial test
V	Field vane test (L.V.-laboratory vane test)
$\gamma$	Unit weight

Note: <sup>1</sup> Tests which are anisotropically consolidated prior shear are shown as CAD, CAU.

## 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: $\Delta \sigma'$
$\epsilon$	linear strain
$\epsilon_v$	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 $(\sigma_1 + \sigma_2) / 2$
$p'$	mean effective stress $(\sigma'_1 + \sigma'_2) / 2$
$q$	$(\sigma_1 - \sigma_3) / 2$ or $(\sigma'_1 - \sigma'_3) / 2$
$q_u$	compressive strength $(\sigma_1 - \sigma_3)$
$S_t$	sensitivity

Notes:

<sup>1</sup>  $\tau = c' + \sigma' \tan \phi'$

<sup>2</sup> shear strength  $= (\text{compressive strength}) / 2$

# LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

## WEATHERING STATE

**Fresh:** no visible sign of rock material weathering

**Faintly Weathered:** weathering limited to the surface of major discontinuities.

**Slightly weathered:** penetrative weathering developed on open discontinuity surfaces but only slight weathering of rock material.

**Moderately weathered:** weathering extends throughout the rock mass but the rock material is not friable

**Highly weathered:** weathering extends throughout rock mass and the rock material is partly friable.

**Completely weathered:** rock is wholly decomposed and in a friable condition but the rock texture and structure are preserved.

## BEDDING THICKNESS

<u>Description</u>	<u>Bedding Plane Spacing</u>
Very Thickly Bedded	> 2 m
Thickly Bedded	0.6 m to 2m
Medium Bedded	0.2 m to 0.6 m
Thinly Bedded	60 mm to 0.2 m
Very Thinly Bedded	20 mm to 60 mm
Laminated	6 mm to 20 mm
Thinly Laminated	< 6 mm

## JOINT OR FOLIATION SPACING

<u>Description</u>	<u>Spacing</u>
Very Wide	> 3 m
Wide	1 – 3 m
Moderately Close	0.3 – 1 m
Close	50 – 300 mm
Very Close	< 50 mm

## GRAIN SIZE

<u>Term</u>	<u>Size*</u>
Very Coarse Grained	> 60 mm
Coarse Grained	2 – 60 mm
Medium Grained	60 microns – 2mm
Fine Grained	2 – 60 microns
Very Fine Grained	< 2 microns

Note: \*Grains > 60 microns diameter are visible to the naked eye.

## CORE CONDITION

### Total Core Recovery

The percentage of solid drill core recovered regardless of quality or length, measured relative to the length of the total core run.

### Solid Core Recovery (SCR)

The percentage of solid drill core, regardless of length, recovered at full diameter, measured relative to the length of the total core run.

### Rock Quality Designation (RQD)

The percentage of solid drill core, greater than 100 mm length, recovered at full diameter, measured relative to the length of the total core run. RQD varies from 0% for completely broken core 100% for core in solid sticks.

## DISCONTINUITY DATA

### Fracture Index

A count of the number of discontinuities (physical separations) in the rock core, including naturally occurring fractures but not including mechanically induced breaks caused by drilling.

### Dip with Respect to (W.R.T.) Core Axis

The angle of the discontinuity relative to the axis (length) of the core. In a vertical borehole a discontinuity with a 90° angle is horizontal.

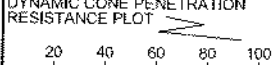
### Description and Notes

An abbreviated description of the discontinuities, whether naturally occurring separations such as fractures, bedding planes and foliation ground or shattered core and mechanically separated bedding or foliation surfaces. Additional information concerning the nature of fracture surfaces and infillings are also noted.

### Abbreviations

BD -	Bedding	PY -	Pyrite
FO -	Foliation/Schistosity	Ca -	Calcite
CL -	Clean	PO -	Polished
SH -	Shear Plane/Zone	K -	Slickensided
VN -	Vein	SM -	Smooth
FLT -	Fault	RO -	Ridged/Rough
CO -	Contact	ST -	Stepped
JN -	Joint	PL -	Planar
FR -	Fracture	IR -	Irregular
MB -	Mechanical Break	UN -	Undulating
BR -	Broken Rock	CU -	Curved
BL -	Blast Induced	TCA -	To Core Axis
II -	Parallel To	STR -	Stress Induced
OR -	Orthogonal		

PROJECT 12-1121-0193-1115		<b>RECORD OF BOREHOLE No 13-1</b>		SHEET 1 OF 1		<b>METRIC</b>	
G.W.P. 4119-09-01		LOCATION N 4877038.1, E 241861.0		ORIGINATED BY HEC			
DIST _____ HWY 62		BOREHOLE TYPE Power Auger 200 mm Diam. (Hollow Stem)		COMPILED BY JM			
DATUM Geodetic		DATE June 20, 2013		CHECKED BY KE			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT  SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED	PLASTIC LIMIT w <sub>p</sub> NATURAL MOISTURE CONTENT w LIQUID LIMIT w <sub>L</sub> WATER CONTENT (%) w	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
91.4	GROUND SURFACE										
0.0	Crushed stone (FILL)										
0.2	Grey Silty gravelly sand to sand and gravel (FILL)										
	Compact Brown Moist										
90.1			1	SS	11						23 55 16 6
89.9	Silty sand and gravel (FILL)										
1.5	Loose Brown Moist										
	Silty sand, some clay, trace gravel (Probable FILL)		2	SS	3						6 52 28 14
89.1	Very loose Brown Moist										
2.3	Silty sand and gravel, trace clay (Probable FILL)		3	SS	28						
88.5	Compact Brown Moist to wet										
88.4	Sandy SILT, some peat										
3.1	Dense Brown Wet		4	SS	50/0.3						GC = 7.0%
88.0											
3.4	Silty SAND, trace to some peat, trace rootlets										
	Compact Brown Wet										
	END OF BOREHOLE AUGER REFUSAL										

**RECORD OF BOREHOLE No 13-2**

SHEET 1 OF 1

**METRIC**

PROJECT 12-1121-0193-1115

G.W.P. 4119-09-01

LOCATION N 4877045.2 E 241871.2

ORIGINATED BY HEC

DIST HWY 62

BOREHOLE TYPE Power Auger 200 mm Diam. (Hollow Stem) Rotary Drill, NQ Core

COMPILED BY JM

DATUM Geodetic

DATE June 19, 2013

CHECKED BY KE

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT $W_p$	NATURAL MOISTURE CONTENT $w$	LIQUID LIMIT $W_L$	UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100					
92.1 0.0 0.1	GROUND SURFACE Crushed stone (FILL) Grey Silty sand and gravel, trace cobbles (FILL) Compact Brown Moist						92							
90.6 1.5	Sand, trace gravel (FILL) Loose to compact Light brown to grey Moist		1	SS	18		91							41 44 11 4
89.5 2.6	Sandy silt, trace clay and rootlets (Probable FILL) Compact Grey-brown Moist to wet		2	SS	7		90							
88.8 3.4	Silty PEAT, trace sand Loose Dark brown Moist to wet		3	SS	11		89							
88.2 4.1	SAND and GRAVEL Very dense Brown Wet Limestone (BEDROCK)  Bedrock cored from depths of 4.1 m to 7.6 m  For bedrock coring details refer to Record of Drillhole 13-2		4	SS	50/0.1		88						UCS = 68.1 MPa	ROD = 52%
			1	RC	REC 100%									ROD = 0%
			2	RC	REC 67%		87							ROD = 75%
			3	RC	REC 100%									ROD = 65%
			4	RC	REC 100%		86							ROD = 93%
			5	RC	REC 100%		85							
84.5 7.6	END OF BOREHOLE													

GTA-MTO 001 1211210193-1115.GPJ GAL-GTA.GDT 10/01/14 JM



PROJECT: 12-1121-0193-1115

## RECORD OF DRILLHOLE: 13-2

SHEET 1 OF 1

LOCATION: N 4877045.2 E 241871.2

DRILLING DATE: June 19, 2013

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 75

DRILLING CONTRACTOR: Marathon Drilling

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	NOTE: For abbreviations, symbols and descriptions refer to LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY										FEATURES
						RECOVERY		R.G.D. %	FRAC. INDEX PER	SP. Wt. CORE AVG	DISCONTINUITY DATA		HYDRAULIC CONDUCTIVITY K, cm/sec			WEATH- ERING INDEX
						TOTAL CORE %	SOLID CORE %				TYPE AND SURFACE DESCRIPTION	10	10	10	W1	
		GROUND SURFACE		87.95 4.14		100	100	100	100	100						
		LIMESTONE									BD, PL, RO					
		Fresh, thinly to medium bedded, fine to medium grained, non-porous, medium strong, grey, with some black shale nodules									BD, UN, RO					
		- Broken/lost core from 4.1 m to 4.3 m									BD, UN, RO					
		- Broken core from 4.8 m to 4.9 m									BD, UN, SM					
											BD, PL, RO					
											UN, UN, RO					
											BD, PL, RO					
											BD, PL, RO					
											BD, PL, RO					
											BD, PL, RO					
											BD, UN, SM					
											BD, PL, RO, Ca < 1 mm					
											BD, PL, RO					
											BD, PL, SM					
											BD, PL, RO					
											BD, PL, SM					
											BD, PL, RO, Ca < 1 mm					
											BD, UN, SM, Ca < 1 mm					
											BD, PL, SM, Ca < 1 mm					
											BD, UN, SM					
											BD, UN, SM					
											BD, PL, RO					
		END OF DRILLHOLE		84.53 7.57												

DEPTH SCALE

1 : 50

LOGGED: HEC

CHECKED: KE

**RECORD OF BOREHOLE No 13-3**

SHEET 1 OF 1

**METRIC**

PROJECT 12-1121-0193-1115

G.W.P. 4119-09-01

LOCATION N 4877065.8 E 241555.9

ORIGINATED BY HEC

DIST HWY 62

BOREHOLE TYPE Power Auger 200 mm Diam. (Hollow Stem)/Rotary Drill, NQ Core

COMPILED BY JM

DATUM Geodetic

DATE June 20, 2013

CHECKED BY KE

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT LIMIT			UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	W <sub>p</sub>	W	W <sub>L</sub>		
91.5 0.0 0.1	GROUND SURFACE Crushed stone (FILL) Grey Silty sand and gravel (FILL) Compact Brown Moist													
90.0 1.5	Sandy silt, some clay, trace gravel, trace asphalt in Sa #3 (FILL) Very loose to loose Grey-brown Wet		1	SS	17		91							10 53 25 12
			2	SS	3		90							
			3	SS	4		89							
			4	SS	50/0.4		88							
88.0 3.5	Limestone (BEDROCK)  Bedrock cored from depths of 3.5 m to 6.6 m  For bedrock coring details refer to Record of Drillhole 13-3		1	RC	REC 100%		88							RQD = 0%
			2	RC	REC 100%		87							RQD = 93%
			3	RC	REC 100%		86							RQD = 57%
84.9 6.6	END OF BOREHOLE  NOTES:  1. Water level in piezometer at 2.4 m below ground surface (Elev. 89.1m), measured on July 7, 2013.						85							

GTA-MTO 001 12-1121-0193-1115.GPJ GAL-GTA.GDT 10/01/14 JM

PROJECT: 12-1121-0193-1115

## RECORD OF DRILLHOLE: 13-3

SHEET 1 OF 1

LOCATION: N 4977065.8 :E 241855.9

DRILLING DATE: June 20, 2013

DATUM: Geodetic

INCLINATION: -90°

AZIMUTH: --

DRILL RIG: CME 75

DRILLING CONTRACTOR: Marathon Drilling

DEPTH SCALE METRES	DRILLING RECORD	SYMBOLIC LOG	DESCRIPTION	ELEV. DEPTH (m)	RUN No. FLUSH RETURN	NOTE: For abbreviations, symbols and descriptions refer to LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY										FEATURES		
						RECOVERY		R.Q.D. %	FRACT. INDEX PER CENT	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	HYDRAULIC CONDUCTIVITY K, cm/sec			WEATHERING INDEX				
						TOTAL CORE %	SOLID CORE %				10 <sup>1</sup> m	10 <sup>2</sup> m	10 <sup>3</sup> m	W1	W2		W3	W4
			GROUND SURFACE	88.00														
			LIMESTONE	3.50	1	100					JN, UN, RO, Ca <1 cm							
4			Fresh, thinly to medium bedded, fine to medium grained, non-porous, medium strong, grey, with minor black shale nodules							BD, PL, RO								
			- Lost core from 4.6 m to 4.7 m		2	100				BD, PL, RO BD, PL, RO BD, UN, SM BD, PL, RO BD, PL, RO								
			- Broken core from 5.5 m to 5.8 m							BD, UN, SM BD, UN, SM								
6			- Broken core from 6.2 m to 6.3 m							BD, UN, SM JN, UN, SM								
			END OF DRILLHOLE	84.92						BD, UN, SM BD, UN, RO BD, PL, RO BD, PL, SM BD, PL, SM								
?				5.50														
															</			

DEPTH SCALE

1:50

LOGGED: HEC

CHECKED: KE

SHEET 1 OF 1

METRIC

ORIGINATED BY HEC

COMPILED BY JM

CHECKED BY KE

