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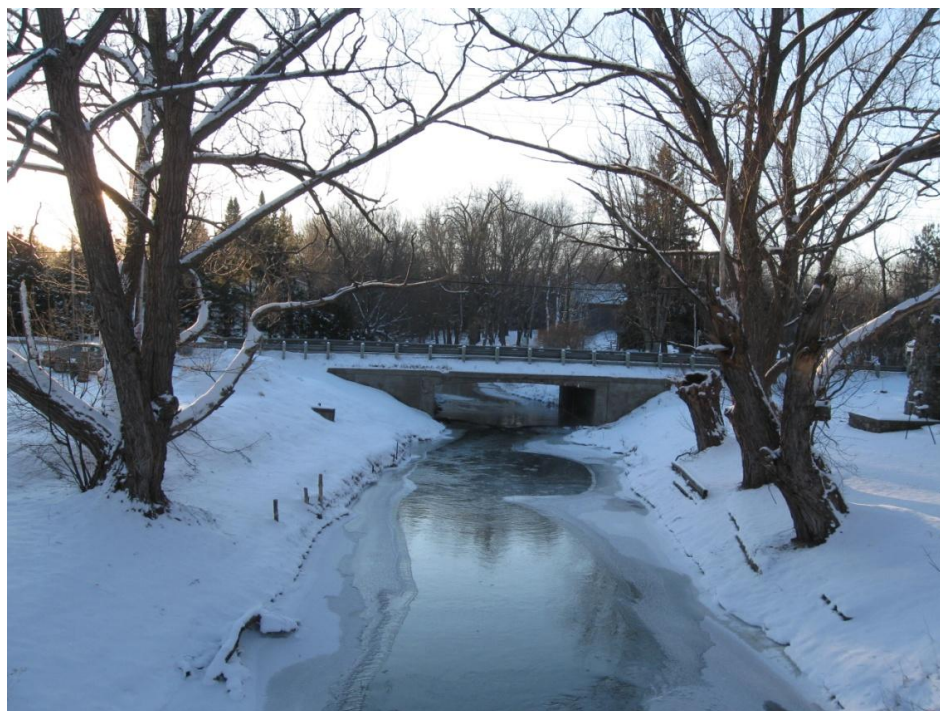
# REPORT



## FOUNDATION INVESTIGATION AND DESIGN REPORT

### Sturgeon River Bridge (Site No. 30-22) Replacement Highway 12, District of Midland, Ontario G.W.P. 2004-08-00

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# **PART A**

**FOUNDATION INVESTIGATION REPORT  
STURGEON RIVER BRIDGE (Site No. 30-22) REPLACEMENT  
HIGHWAY 12, DISTRICT OF MIDLAND, ONTARIO  
G.W.P. 2004-08-00**



### 1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by Morrison Hershfield (MH) on behalf of the Ministry of Transportation, Ontario (MTO) to provide detail foundation engineering services for the replacement of the existing Highway 12 Sturgeon River Bridge in the District of Midland, Ontario.

This report addresses the results of the subsurface investigation carried out for the proposed replacement of the existing Sturgeon River Bridge structure.

The terms of reference and scope of work for the foundation engineering services are outlined in MTO's Request for Proposal (RFP) and clarifications for Assignment No. 2009-E-0100 issued in December 2010, and in Section 6.8 of the *Technical Proposal* for this assignment.

### 2.0 SITE DESCRIPTION

#### 2.1 General

The existing Sturgeon River Bridge structure is located approximately 130 m west of Rosemount Road along Highway 12 in the District of Midland, Ontario. According to a drawing provided by MH titled "Standard Rigid Frame Bridge – Waubesaushene Diversion", prepared by the Department of Highways Ontario, dated August 31, 1937, the existing bridge is a 9.1 m long single-span rigid frame concrete structure supported on shallow foundations.

In general, the terrain in this area is relatively flat, with the natural ground surface in the immediate vicinity of the structure site at about Elevation 180 m. The existing bridge crosses the Sturgeon River valley and the valley walls slope down at about 2 Horizontal to 1 Vertical (2H:1V) from the crest of the highway shoulder at about Elevation 179.5 m to the bottom of the valley bank at about Elevation 176 m. The Sturgeon River water level is shown to be at Elevation 176.1 m (measured June 1, 2011) on the preliminary General Arrangement (GA) drawing provided by MH.

### 3.0 INVESTIGATION PROCEDURES

The field work for the subsurface investigation was carried out in July and August 2011, during which time seven (7) boreholes were advanced at the site, at the locations shown on Drawing 1. Track-mounted drilling rigs (CME-55 and D90 models) were supplied and operated by Walker Drilling Ltd. of Utopia, Ontario and Geo-Environmental Drilling Inc. of Milton, Ontario.

The boreholes were drilled using 108 mm inner diameter (194 mm outer diameter) hollow stem augers and NQ and HQ core barrels to depths ranging from 12.7 m to 22.7 m below ground surface. Soil samples were obtained or field vane testing performed at 0.75 m and 1.5 m intervals of depth in the boreholes, using a 50 mm outside diameter split-spoon sampler in accordance with the Standard Penetration Test (SPT) procedure (ASTM D1586) or using a standard MTO 'N' vane, respectively. Bedrock was cored for lengths of 3.2 m and 4.4 m in two boreholes using NQ and HQ-size core barrels.

The groundwater conditions were observed in the open boreholes during and immediately following the drilling operations. Two piezometers (one shallow installation in the near surface overburden and one deep installation



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near the overburden/bedrock contact) were installed in one of the boreholes to permit monitoring the groundwater level at this site.

The piezometers consist of 50 mm diameter PVC pipe, with a slotted screen sealed within a sand filter pack at a selected depth interval within the borehole. Above and below the sand filter pack and piezometer screen, the annulus surrounding the piezometer pipe was backfilled with bentonite pellets up to the ground surface. The details of the piezometer installation and water level readings are indicated on the borehole records contained in Appendix A.

The field work was monitored on a full-time basis by a member of Golder's staff who located the boreholes in the field, directed the drilling, sampling, and in situ testing operations, and logged the boreholes. The soil and rock samples were identified in the field, placed in labelled containers and transported to Golder's laboratory in Mississauga for further examination and laboratory testing. Index and classification tests consisting of water content determinations, Atterberg limits and grain size distribution were carried out on selected soil samples. Point load tests and unconfined compressive strength tests were carried out on selected rock samples. The geotechnical laboratory testing was carried out in accordance with ASTM and MTO LS standards, as applicable.

The borehole locations were measured in the field by Golder personnel relative to site features and the ground surface elevation and coordinates at each borehole were determined from the digital terrain model supplied by MH. The borehole locations (referenced to the MTM NAD83 coordinate system) and ground surface elevations (referenced to Geodetic datum) are summarized below and are shown on Drawing 1.

Borehole Number	MTM NAD83 Northing (m)	MTM NAD83 Easting (m)	Ground Surface Elevation (m)	Borehole Depth (m)
1	4954691.0	286335.5	179.5	12.8
2	4954693.5	286340.5	179.5	12.8
3	4954703.5	286356.5	179.5	22.6
4	4954687.5	286343.5	179.5	19.3
4A	4954686.5	286342.0	179.5	22.7
5	4954697.0	286360.0	179.5	12.8
6	4954700.0	286364.5	179.5	12.7

## 4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

### 4.1 Regional Geology

The Sturgeon River Bridge site is located within the "Simcoe Uplands" physiographic region, as delineated in *The Physiography of Southern Ontario* (Chapman and Putnam, 1984)<sup>1</sup>. The predominant overburden stratum

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



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consists of a glacial deposit comprised of sandy silt to silty sand till. Surficial deposits of boulders, sands and silts from the glacial Lake Algonquin overly the till materials.

According to Barnett (1997)<sup>2</sup>, the quaternary geology of the study area is characterized by lacustrine deposits of sand, silts and clays and till units. A transition between units is mapped in the vicinity of the site. To the south, the geology is characterized by lacustrine coarse grained sediments of very fine to medium grained sand, silt and minor clay. The unit is described by Barnett (1997) as being deposited near to the shore or sediment source in a nonglacial lake. To the north of the transition, the geology is described as lacustrine fine grained sediments of silt and clay. Barnett states these features as being deposited in a basinal area of a non-glacial lake.

Limestone, dolostone and shale of the Simcoe Group typically underlie the overburden deposits. Precambrian crystalline (granite) bedrock of the Grenville Province underlies the Simcoe Group bedrock or underlies the overburden deposits directly in some areas.

The subsurface conditions encountered at the site is consistent with the description provided by Barnett (1997).

### 4.2 Subsurface Conditions

The subsurface investigation consisted of the advancement of seven boreholes (Boreholes 1 to 4, 4A, 5 and 6) on Highway 12 near the north and south abutments of the existing Sturgeon River Bridge. The locations of the boreholes, ground surface elevations and interpreted stratigraphic conditions are shown on Drawings 1 and 2. The detailed subsurface soil, rock and groundwater conditions encountered in the boreholes and the results of in situ and laboratory testing are given on the Record of Borehole and Record of Drillhole sheets contained in Appendix A. The results of geotechnical laboratory testing are also presented on Figures B1 to B7 contained in Appendix B. The stratigraphic boundaries shown on the borehole records and on the interpreted stratigraphic sections 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 soils encountered at the site consist of embankment fill underlain by a deposit of very soft to stiff clayey silt to silty clay, deposits of very loose to loose sand and silt to sandy silt/silty sand, in turn underlain by a stratum of clayey silt. The clayey silt deposit is underlain by granite gneiss bedrock, encountered at depths of 18.3 m and 19.4 m below ground surface or top of pavement. A more detailed description of the major soil deposits encountered in the boreholes is provided in the following sections.

#### 4.2.1 Asphalt

A 100 mm thick surficial layer of asphalt was encountered in Boreholes 4 and 5.

#### 4.2.2 Fill

A fill deposit comprising predominantly of sandy silt to sand and gravel was encountered in all boreholes at the ground surface or below the asphalt. The cohesionless fill typically contains cobbles, as inferred from auger grinding during advancement, and trace organics. A layer of clayey silt fill containing organics was encountered in Borehole 5 underlying the cohesionless fill deposit.

<sup>2</sup> Barnett, P.J., 1997. Quaternary Geology, Eastern Half of the Barrie and Elmvalle Areas; Ontario Geological Survey, Map 2645, scale 1:50,000.





The elevation of the top of the cohesionless fill deposit was encountered at about Elevation 179.5 m and 179.4 m and the thickness of the deposit ranges from 2.3 m to 4.2 m.

The measured SPT “N” values within the cohesionless fill deposit range from 2 blows to 35 blows per 0.3 m of penetration, indicating a very loose to dense relative density, although the deposit is generally very loose to compact according to the measured SPT “N” values.

The measured natural water content of samples of the cohesionless fill range from 2 percent to 27 percent. The results of grain size distribution tests completed on three samples of the cohesionless fill deposits are shown on Figure B1 in Appendix B.

The elevation of the top of the clayey silt fill deposit in Borehole 5 is at Elevation 176.5 m and the deposit is 0.7 m thick. A SPT “N” value of 2 blows per 0.3 m penetration was measured within this deposit, suggesting a soft consistency. A measured natural water content of one sample of this deposit is 48 per cent.

### 4.2.3 Clayey Silt to Silty Clay

A deposit of cohesive soil comprised of an upper stratum of brown to grey clayey silt to silty clay and a lower stratum of grey clayey silt was encountered below the fill deposit or below an upper deposit of sandy silt in all boreholes. The elevation of the top of this deposit ranges from Elevation 177.2 m to 175.3 m and overall thickness of the deposit is 14.7 m and 15.2 m, including interlayers of cohesionless soil, where it was fully penetrated in Boreholes 3 and 4, respectively. Sand and silt to silty sand seams are present within the cohesive deposit and organic seams are present within the upper 1.5 m of the deposit. The sand and silt to silty sand layers were encountered at depths ranging from 2.7 m to 7.3 m below ground surface as described in more detail in the next section.

Boreholes 1, 2, 5 and 6 were terminated within the clayey silt to silty clay deposit at depths of 12.7 m and 12.8 m below ground surface (Elevation 166.7 m and 166.8 m).

The measured SPT “N” values within the clayey silt to silty clay range from 0 blows (weight of the hammer) to 7 blows per 0.3 m of penetration. In situ field vane tests carried out within this clayey silt to silty clay deposit measured undrained shear strengths ranging from 17 kPa to greater than 97 kPa indicating that the clayey silt to silty clay is generally soft to stiff.

Natural water content values ranging from about 20 per cent to 50 per cent were measured on samples of the clayey silt to silty clay material. One water content value of 70 per cent was measured in a clay interlayer in Borehole 4.

Atterberg limits testing was carried out on eleven selected samples of this deposit and measured plastic limits between 21 per cent and 30 per cent, liquid limits between 20 per cent and 50 per cent, and plasticity indices between 3 per cent and 24 per cent. These test results, which are plotted on a plasticity chart on Figure B2, confirm that the material consists of predominantly clayey silt to silty clay of low to medium plasticity in the upper portion of the deposit and clayey silt of low plasticity in the lower portion of the deposit. One Atterberg limits test carried out on a clay interlayer within the clayey silt to silty clay deposit in Borehole 4 measured a plastic limit of 26 per cent, a liquid limit of 75 per cent and a plasticity index of 49 per cent indicating that this material has a high plasticity.



The results of grain size distribution tests completed on four samples of the clayey silt portion of the deposit are shown on Figure B3. Two laboratory organic content tests carried out on samples of the upper 1.5 m of the clayey silt to silty clay deposit measured 5.5 per cent and 5.2 per cent organic content.

One laboratory consolidation (oedometer) test was carried out on a specimen of the clayey silt lower portion of the deposit obtained from Borehole 3 and the test results are shown on Figure B4. The pre-consolidation pressure was estimated from the Void Ratio versus logarithmic Pressure plots using the Casagrande method as well as from the Total Work versus Pressure plots. The relevant consolidation test results are summarized below.

Borehole / Sample Number	Elevation (m)	$\sigma_{vo}'$ (kPa)	$\sigma_p'$ (kPa)	$\sigma_p' - \sigma_{vo}'$ (kPa)	OCR	$e_o$	$C_r$	$C_c$	$c_v^*$ (cm <sup>2</sup> /s)
3/Sa 11	165.5	155	155	0	1	0.89	0.04	0.28	$3.2 \times 10^{-3}$

Note: \* For  $\sigma_v' \leq 2500$  kPa

where:  $\sigma_{vo}'$  effective overburden pressure in kPa  
 $\sigma_p'$  preconsolidation pressure in kPa  
OCR over consolidation ratio  
 $e_o$  initial void ratio  
 $C_c$  compression index (based on void ratio)  
 $C_r$  recompression index (based on void ratio)  
 $c_v$  coefficient of consolidation in cm<sup>2</sup>/s in the normally consolidated range

### 4.2.4 Sandy Silt to Silty Sand/Sand and Silt

Interlayered deposits of brown sandy silt to silty sand /sand and silt were encountered below the fill in Borehole 2 at a depth of 2.7 m below ground surface (Elevation 176.8 m) and within the clayey silt to silty clay deposit at depths ranging from 5.2 m to 6.4 m below ground surface (Elevation 174.3 m to 173.1 m) in Boreholes 1, 3, 4 and 6. The thickness of the sandy silt to silty sand/sand and silt layer ranges from 1.2 m to 4.4 m. Trace amounts of organics were encountered in the sandy silt to silty sand layer directly below the fill in Borehole 2.

The measured SPT "N" values within the silty sand to sandy silt/sand and silt layers range from 2 blows to 7 blows per 0.3 m of penetration, indicating a very loose to loose relative density.

Natural water content values ranging from 19 per cent to 38 per cent were measured on samples of the cohesionless layers. The results of grain size distribution tests completed on six samples of this cohesionless layers are shown on Figure B5.

### 4.2.5 Sand

A deposit of grey sand was encountered below the sandy silt to silty sand layer in Borehole 2 and within the clayey silt deposit in Borehole 5, at a depth of 6.7 m and 7.3 m below ground surface corresponding to Elevation 172.8 and 172.2 m, respectively. The deposit is 1.2 m thick.

SPT "N" values of 3 blows and 15 blows per 0.3 m penetration were measured in this deposit, indicating a very loose and compact relative density.



Natural water content values of 20 per cent and 22 per cent were measured on two samples of the deposit. The results of a grain size distribution test completed on one sample of the sand deposit is shown on Figure B6.

### 4.2.6 Silty Sand (Lower)

A deposit of brown silty sand was encountered below the clayey silt deposit in Borehole 4. The top of this layer was encountered at a depth of 17.8 m below ground surface, corresponding to Elevation 161.7 m and the deposit is 1.5 m thick.

A SPT “N” value of 16 blows per 0.3 m penetration was measured in this deposit, indicating a compact relative density.

A laboratory natural water content value of 19 per cent was measured on one sample of the silty sand deposit. The result of a grain size distribution test completed on one sample of the layer is shown on Figure B7.

### 4.2.7 Sand and Gravel

A deposit of grey to brown sand and gravel was encountered below the clayey silt deposit in Borehole 3 at a depth of at 18.9 m below ground surface, corresponding to Elevation 160.6 m, and the thickness of the deposit is 0.5 m.

A SPT “N” value of 100 blows per 0.05 m penetration was measured in this deposit, indicating a very dense relative density.

A laboratory natural water content value of 13 percent was measured on one sample of the sand and gravel.

### 4.2.8 Bedrock

Bedrock was encountered and cored in Boreholes 3 and 4A and inferred from auger refusal in Borehole 4 at the depths and elevations shown below.

Borehole	Depth to Refusal/Bedrock Surface (m)	Refusal/Bedrock Surface Elevation (m)	Comments
3	19.4	160.1	Bedrock cored
4	19.3	160.2	Auger refusal
4A	18.3	161.2	Bedrock cored

It is noted that the bedrock surface elevation is generally consistent at the east and west abutment locations; however, the inferred bedrock surface at the west abutment location ranged from about 160.2 m to 161.2 m (i.e. one metre difference) indicating sloping bedrock may be present.

The bedrock consisted of granite/gneiss in the cored boreholes and the bedrock core samples are described as slightly weathered, banded pink, white and black, coarse grained biotitic granitic gneiss in Borehole 3 and fresh, pink, very coarse, medium strong feldspathic granite in Borehole 4A.

The Rock Quality Designation (RQD) measured on the core samples ranges from 50 percent to 96 percent indicating a rock mass of fair to excellent quality as per Table 3.10 of the Canadian Foundation Engineering Manual (CFEM, 2006). The Total Core Recovery (TCR) was measured between 75 and 99 percent.



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Laboratory Uniaxial Compressive Strength (UCS) testing was carried out on two core samples of the granite/gneiss bedrock. The UCS values are presented in Appendix A and summarised below and the test results indicate the bedrock is strong (i.e. Grade R4) as per Table 3.5 in CFEM (2006).

Borehole	Elevation (m)	UCS (MPa)
3	158.7	82
4A	158.5	89

Diametral and axial point load strength tests were carried out on selected samples of the granite/gneiss bedrock and the values are shown on the Record of Drillhole Sheets. The point load index ( $I_{s50}$ ) values range from about 3.9 MPa to 10.2 MPa. These index values correspond to estimated UCS values ranging between 82 MPa and 250 MPa, based on a relationship between  $I_{s50}$  and UCS which is given by a correlation factor (K) in accordance with ASTM D5731, which varies depending on the size of the core samples and the strength of the rock. These values have been given for comparison only and should be interpreted together with the results of the laboratory UCS tests.

### 4.3 Groundwater Conditions

Details of the water levels observed in the open boreholes at the time of drilling are summarized on the borehole records contained in Appendix A. Two nested piezometers (one shallow installation screened and sealed within the clayey silt to silty clay deposit containing cohesionless interlayers and one deep installation screened and sealed at the overburden/bedrock contact) were installed in Borehole 4A to permit monitoring of the groundwater level at the site.

The water levels measured within the open boreholes upon completion of drilling and in the piezometers are summarized below:

Borehole Number	Piezometer Installation	Depth to Water Level (m)	Depth to Water Elevation (m)	Date
1	-	6.1	173.4	August 2, 2011
2	-	5.3	174.2	August 2, 2011
3	-	6.1	173.4	July 28, 2011
4	-	7.1	172.4	August 4, 2011
4A	Shallow	2.9	176.6	September 30, 2011
	Deep	2.7	176.8	September 30, 2011
5	-	5.2	174.3	July 21, 2011
6	-	6.1	173.4	July 19, 2011

The groundwater levels observed in the open boreholes may not represent stabilized groundwater conditions. In addition, groundwater levels are expected to fluctuate seasonally and are expected to rise during wet periods of the year, consistent with the Sturgeon River water level. It is noted that the groundwater levels measured in the piezometers are expected to be at or near the Sturgeon River water level, measured at Elevation 176.1 m in June 2011.

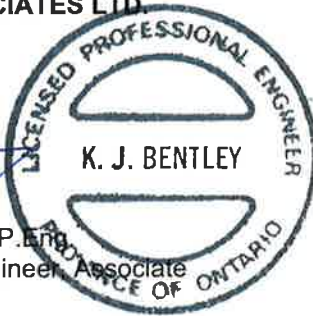


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### 5.0 CLOSURE

This Foundation Investigation Report was prepared by Mr. Gilberto Alexandre and reviewed by Mr. Kevin Bentley, P.Eng., a senior geotechnical engineer with Golder. Mr. Jorge M.A. Costa, P.Eng., a Designated MTO Foundations Contact and Principal of Golder, conducted an independent quality control review of this report.

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# **PART B**

**FOUNDATION DESIGN REPORT  
STURGEON RIVER BRIDGE (Site No. 30-22) REPLACEMENT  
HIGHWAY 12, DISTRICT OF MIDLAND, ONTARIO  
G.W.P. 2004-08-00**



## 6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

### 6.1 General

This section of the report provides geotechnical foundation recommendations for detail design of the proposed replacement of the Sturgeon River Bridge structure located on Highway 12, as shown on Drawing 1. The recommendations are based on interpretation of the factual data obtained from the boreholes and drillholes advanced during this subsurface investigation. The discussion and recommendations presented are intended to provide the designers with sufficient information to assess the feasible foundation alternatives and to carry out the detail design of the replacement structure foundations.

Where comments are made on construction, they are provided to highlight those aspects that could affect the design of the project, and for which special provisions may be required in the Contract Documents. Those requiring information on the 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.

### 6.2 Foundation Options

According to the original design drawing provided by MH titled “Standard Rigid Frame Bridge - Waubauskene Diversion”, Drawing No. 02489-1, dated August 31, 1937, the existing bridge is a 9.1 m long by 15.2 m wide single-span rigid frame concrete structure supported on spread/strip footings founded approximately 4.7 m below the pavement surface (Elevation 174.8 m). A note on the August 1937 drawing indicates that the depth of the (existing) foundations were subject to revision and that the use of piles for the support of the structure was to be considered depending on the soil conditions encountered at the time of excavation for the foundations. Golder is not aware of any as-built drawings showing that the original design was altered and it is assumed that the existing bridge is supported on spread footings as indicated above.

According to the preliminary General Arrangement drawing by MH titled “Sturgeon River Bridge Structure Replacement – General Arrangement”, dated October 27, 2011, it is proposed to replace the existing bridge with an 18m long single-span structure. Based on the GA drawing, the proposed bridge abutments will be located approximately 4.3 m behind the existing abutments as it is intended that the existing bridge abutment walls and wing walls be used as temporary shoring for the construction of the proposed bridge and partial groundwater control features. We also understand that the top surface of the new bridge is proposed to be at approximately the same grade elevation as the existing bridge (i.e. there will be no grade raise) and bridge widening is not proposed.

Based on the subsurface conditions encountered during the current investigation, the following foundation options are considered feasible for the support of the proposed new abutments and wing walls. A summary of the advantages, disadvantages, relative costs and risks associated with each option is provided in Table 1 following the text of this report.

- **Shallow foundations:** Shallow spread/strip footings founded on firm to stiff clayey silt to silty clay deposit are feasible for the proposed new bridge considering that the existing bridge is understood to be supported on this type of foundation and does not show any visual signs of major distress. However, considering that the founding soils below the proposed bridge foundations consist of soft to stiff clayey silt to silty clay





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underlain by very loose to loose sand and silt, the geotechnical resistance of these founding strata are low compared to deep foundation options and dewatering/sensitive soil conditions may result in challenges maintaining a competent founding subgrade during construction. In addition, due to the variable thickness and consistency of the cohesive deposits below the foundation footprint there is a risk that differential settlement and tilting of the proposed new bridge structure could occur; although the existing structure appears to have performed satisfactorily. Further, higher resistance values are not readily available at greater depths.

- **Driven steel H-piles/Tube piles:** Steel H-piles or tube piles driven to refusal on bedrock are feasible and suitable for foundation support at this site. Steel H-pile foundations would also allow for the construction of integral abutments; although steel tube piles could also be considered. The piles should be provided with suitable rock points to assist in seating the pile on bedrock and avoid damage to the pile tip during driving.
- **Caissons:** Caissons socketted into the bedrock are feasible for foundation support of the new structure. From a foundations perspective and due to the high rigidity (in comparison with H-piles), caissons are more suitable for a rigid frame bridge design. It should be noted, however, that there may be difficulty in socketting large diameter caissons within the strong crystalline granite gneiss bedrock and achieving an adequate seal for concrete placement. In addition, difficulties in cleaning and inspecting the base of the sockets, which will be well below the water level, should be expected. Temporary liners would be required to advance the caissons through the cohesionless soil deposits below the water table and caisson construction could result in environmental impact on the river.

Based on the above considerations, the preferred option from a geotechnical/foundations perspective is to support the proposed bridge on H-piles driven to bedrock. Design recommendations for each foundation option are provided below.

### 6.3 Shallow Foundations

#### 6.3.1 Founding Elevation and Geotechnical Resistance

For support of the abutments and associated wing walls of the proposed bridge, shallow strip/spread footings could be founded below the existing bridge foundations which are understood to be founded at about Elevation 174.8 m. New footings should be founded below the fill and any very soft to soft or very loose soils, on the firm to stiff clayey silt to silty clay deposit. Based on the results the investigation, subexcavation depths ranging from 5.5 m to 11 m bgs (and 2 m to 7 m below the measured water level) would be required to remove the unsuitable materials resulting in foundation design levels as follows:

	West Abutment Subexcavation Elevation (m)	East Abutment Subexcavation Elevation (m)
North Side	169.0 m	171.0 m
South Side	174.5 m	174.5 m





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As shown above, the subexcavation levels required to remove the very soft to soft and very loose soils ranges significantly across the footprint of the proposed abutments. Consideration could be given to replacing the subexcavated material with engineered fill for foundation support; however, preloading would be required to reduce post construction settlement to tolerable levels. This option is not considered practical at this location due mainly to traffic disruption during the preload period which could likely extend at least over six months.

If this option is considered, the footing subgrade should be inspected by a Quality Verification Engineer (QVE) following excavation, in accordance with OPSS 902 (*Excavating and Backfilling Structures*) to verify that all existing fill, very soft to soft or very loose soils, or other unsuitable material have been removed. The founding soils will be susceptible to disturbance and should be protected with a concrete working slab (100 mm thick concrete slab with a compressive strength of 20 MPa) if the concrete for the footing or granular backfill is not placed within four hours of the inspection and approval of the subgrade. An example NSSP for the concrete working slab is provided in Appendix C.

### 6.3.2 Geotechnical Resistance

Shallow foundations placed on the properly prepared firm to stiff clayey silt to silty clay at or below the design elevations given in Section 6.3.1 or on engineered fill at higher elevation, should be designed based on the factored geotechnical resistance at Ultimate Limit State (ULS) and geotechnical reaction at Serviceability Limit State (SLS) given below.

Maximum Footing Width	Factored Geotechnical Resistance at ULS	Geotechnical Resistance at SLS (for 25 mm of settlement)
3 m	75 kPa	50 kPa

Although the existing rigid frame bridge appears to have performed satisfactorily, the results of the subsurface investigation indicate the subsurface conditions are variable. Given the range in design founding elevations across each proposed abutment footprint, or variable thickness of engineered fill, and the variable thickness of cohesive deposits below the proposed abutment foundation, differential settlements (up to 25 mm) should be expected to occur between abutments and between the north and south limits of each abutment footprint.

The geotechnical resistances should be reviewed if the selected footing width or founding elevations differ from those given above. These geotechnical resistances are provided for loads applied perpendicular to the surface of the footings and 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.

Footings should be founded at a minimum depth of 1.6 m below the lowest surrounding grade to provide frost protection, as per OPSD 3090.101 (Foundation Frost Penetration Depths for Southern Ontario).

### 6.3.3 Resistance to Lateral Loads

Resistance to lateral forces/sliding resistance between the concrete footing and the subgrade should be calculated in accordance with Section 6.7.5 of the CHBDC. For cast-in-place concrete footings constructed on



the firm to stiff clayey silt to silty clay deposit or on engineered backfill, the coefficient of friction,  $\tan \Phi'$  can be taken as 0.4.

### 6.4 Deep Foundations

#### 6.4.1 Steel H-Pile/Steel Tube Foundations

Steel H-piles or steel tube piles driven to bedrock may be used for support of the proposed abutments and wing walls. For the installation of the piles, consideration must be given to the presence of cobbles in the existing fill soils as inferred from auger grinding during advancement of the boreholes. Steel H-piles are preferred over steel tube piles given that H-piles are more conventional for integral abutment design, are likely easier to drive and displace/squeeze a lesser volume of soil and less likely to “hang up” or be deflected away from vertical during driving.

For design, the estimated pile tip elevations and lengths for the piles terminating on the bedrock surface are presented below. The elevations and lengths are based on the depth to/elevation of the bedrock surface encountered in the boreholes advanced adjacent to proposed abutments and the proposed underside of pile cap elevations provided on the preliminary GA drawing provided by MH. Consideration should be given to raising the pile cap level as high as possible (while maintaining the minimum frost depth cover) to reduce the amount of subexcavation required. There should be a provision made in the Contract for dealing with varying pile lengths.

Foundation Unit	Borehole Number	Proposed Underside of Pile Cap (m)	Bedrock Surface Elevation (m)	Approximate Design Pile Length (m)
West Abutment	4/4A	175.4	160.2	15.2
East Abutment	3	174.6	160.1	14.5

#### 6.4.2 Geotechnical Axial Resistance

For HP310x110 piles or 324 mm x 6.4 mm (12 ¾ in. O.D. and ¼ in. wall thickness) tube (pipe) piles driven to practical refusal on bedrock, a factored geotechnical axial resistance at Ultimate Limit States (ULS) of 2,000 kN may be assumed for design. This value represents a structural limitation for the pile rather than a geotechnical limitation. The geotechnical resistance at Serviceability Limit States (SLS) for 25 mm of settlement (for the length of piles required at this site) will be greater than the factored axial resistance at ULS, since the bedrock is considered to be an unyielding material and, as such, ULS conditions will govern for this foundation type.

All pile installation/driving should be in accordance with OPSS 903 (Deep Foundations). The piles should be provided with driving shoes to reduce the potential for damage to the piles in the event that cobbles are encountered in the fill soils and to assist in seating the pile (i.e. if sloping bedrock is present) and avoid damage to the pile tip on the bedrock. The steel H-piles should be reinforced with driving shoes such as per OPSD 3000.201 (HP310 Oslo Point), Titus Injector Bearing Pile Point design or equivalent. Similarly, if steel tube piles are considered, driving shoes should be in accordance with OPSD 3001.100 Type II (Steel Tube Pile Driving Shoe). The requirement for driving shoes should be included in the Contract Drawings. An NSSP should be



included in the Contract Documents to address the potential for obstructions with the existing fill, an example is included in Appendix C.

If corrugated steel pipes (CSPs) are installed as part of the integral abutment design, the CSP's should be backfilled with loose, fine to medium sand. A Non Standard Special Provision (NSSP) detailing the installation method and gradation of this sand should be included in the Contract Documents; an example is included in Appendix C.

### 6.4.3 Downdrag Loads

As a grade raise is not proposed at this site and no significant dewatering is anticipated below the pile cap for the proposed abutments and wing walls, no significant downdrag loads are expected on the piles.

### 6.4.4 Pile Driving Note

The pile driving note to be added to the drawings for this project is Note 5 in Clause 3.3.3 of the Structural Manual (MTO, 2008):

- "Piles to be driven to bedrock".

### 6.4.5 Resistance to Lateral Loads

Resistance to lateral loading can be derived using vertical piles, with enhanced support offered by battered piles, if required. The evaluation of the piles subjected to lateral loads should take into account such factors as the relative rigidity of the pile to the surrounding soil, the fixity condition at the head of the pile (pile cap level), the structural capacity of the pile to withstand bending moment, the soil resistance that can be mobilized, the tolerable lateral deflection at the head of the pile and the pile group effects.

The lateral load response of a single pile may be calculated using subgrade reaction theory where the coefficient of horizontal subgrade reaction,  $k_h$  (kPa/m), is determined in accordance with Section C6.8.7 in the Commentary to the CHBDC based on the equation for cohesionless and cohesive soils given below (CFEM, 1992).

For cohesionless soils:

$$k_h = \frac{n_h z}{B} \quad \text{where}$$

$n_h$  is the constant of horizontal subgrade reaction (kPa/m);

$z$  is the depth (m) below the underside of the pile cap; and

$B$  is the pile diameter or width (m).

And for cohesive soils:

$$k_h = \frac{67 s_u}{B} \quad \text{where}$$

$s_u$  is the undrained shear strength of the soil (kPa); and

$B$  is the pile diameter or width (m).

The values of  $n_h$  and  $s_u$  to be used to calculate the coefficient of horizontal subgrade reaction ( $k_h$ ) to be utilized in the structural analysis for the piles at this location are given below.



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Soil Unit	$n_h$ (kPa/m)	$s_u$ (kPa)
Firm Clayey Silt to Silty Clay	-	35
Very loose to compact Sand and Silt, Silty Sand to Sandy Silt	4,400	-

For a single HP310x110 vertical pile extending to bedrock, an estimated factored lateral resistance of 80 kN at ULS and an estimated lateral resistance of 50 kN at SLS (for 10 mm of horizontal deflection at the pile cap) may be used for design. These values are based on analysis carried out using the Broms' (1964) method as outlined in the CFEM (2006) and the commercially available program LPILE Plus (Version 5.0), produced by Ensoft Inc. Similar values can be assumed for integral abutments where a 3 m long CSP liner (with the annular space between the pile and the liner filled with sand) is installed. The structural capacity of the pile should be checked and verified by the structural engineer.

The upper zone of soil (down to a depth below the pile cap equal to about  $1.5 \times B$  after Broms' (1964), where  $B$  = pile diameter) should be neglected in the calculation of lateral resistance of the pile to account for disturbance effects during installation.

Group action for lateral loading should also be considered when the pile spacing in the direction of the loading is less than six to eight pile diameters. Group action can be evaluated by reducing the coefficient of horizontal subgrade reaction (NAVFAC, 1982) in the direction of loading by a reduction factor,  $R$ , as follows:

Pile Spacing in Direction of Loading $d$ = Pile Diameter	Horizontal Subgrade Reaction Reduction Factor, $R$
8 $d$	1.00
6 $d$	0.70
4 $d$	0.40
3 $d$	0.25

Where a pile group is oriented perpendicular to the direction of loading, group action may be considered by reducing the coefficient of horizontal subgrade reaction (NAVFAC, 1982) by a reduction factor,  $R$ , as follows:

Pile Spacing Perpendicular to Direction of Loading $d$ = Pile Diameter	Horizontal Subgrade Reaction Reduction Factor, $R$
4 $d$	1.00
1 $d$	0.50



The horizontal subgrade reaction reduction factor should be interpolated for pile spacings in between those listed above.

### 6.4.6 Frost Protection

The pile caps should be provided with a minimum of 1.6 m of conventional soil cover for frost protection, as per OPSD 3090.101 (Foundation Frost Penetration Depths for Southern Ontario).

## 6.5 Caissons

For rigid frame bridge design, caissons socketted into bedrock may be considered for support of the abutments and wing walls. It should be noted, however, that there may be difficulty in socketting large diameter caissons within the strong crystalline granite gneiss bedrock and achieving an adequate seal to prevent soil infiltration. A temporary liner and tremie concrete placement, possibly in combination with dewatering, will likely be required to install caissons at this site.

### 6.5.1 Geotechnical Axial Resistance

For caissons socketted at least 1 m into bedrock, a factored shaft resistance of 1,300 kPa at ULS may be used for design. The contribution from end-bearing has been neglected due to the anticipated difficulties in cleaning and inspecting the base of the sockets which will be below the water level at great depth. The factored geotechnical axial resistance at ULS for two different caisson diameters socketted a minimum of 1 m into the bedrock is given below.

Caisson Diameter (m)	Granite/Gneiss Bedrock (minimum 1 m socket)	
	ULS (kN)	SLS for 25 mm
0.6	2,400	n/a
0.9	3,600	n/a

The resistance required to achieve 25 mm settlement is greater than the value given for ULS for caissons socketted into the bedrock and, therefore, SLS conditions do not apply.

It should be noted that running or flowing water-bearing cohesionless soils could occur during or after drilling the caisson (if a proper seal is not provided). If caisson foundations are adopted, a temporary or permanent liner sealed into the bedrock would be required to support the soils during installation and to prevent the movement of the cohesionless soils from coming up into the liner. A sufficient head of water should be maintained inside the liner (or groundwater control provided) at all times to balance the hydrostatic pressures.

### 6.5.2 Downdrag Loads on Caissons

As discussed in Section 6.4.3, no significant downdrag is expected on the caissons after installation.



### 6.5.3 Resistance to Lateral Loads

The geotechnical resistance to lateral loading for the caissons should be calculated using the horizontal subgrade reaction formulas and soil parameters provided in Section 6.4.5. Lateral capacities for the caisson can be provided as the details of the reinforcement become available, if applicable.

### 6.5.4 Frost Protection

The pile caps for the caissons at the abutments should be provided with a minimum of 1.6 m of conventional soil cover for frost protection as described in Section 6.4.6.

## 6.6 Approach Embankments

As described in Section 6.2, the footprint of the proposed bridge is similar to the existing bridge and a grade raise or widening is not proposed. As a result, no stability or settlement issues at the approach embankments are anticipated at this site.

The existing side-slopes are angled at about 2 Horizontal to 1 Vertical (2H:1V) and it is assumed the new side-slopes/front slopes will be maintained at 2H:1V or shallower.

The new abutments are located about 4.3 m behind the existing closed abutments and as a result, existing fill materials will need to be removed/regraded accordingly, resulting in a net unloading of embankment fill in the immediate area of the abutments.

Any new embankment fill required as part of re-grading the abutment areas should consist of suitable earth fill placed and compacted in accordance with OPSS 501 (Compacting) and MTO's Special Provision 206S03 (*Earth Excavation and Grading*). To reduce the potential for erosion of the embankment side slopes/front slope due to surface water run-off and to establish vegetation on the slopes, placement of topsoil and seeding or pegged sod is recommended as soon as practicable after re-grading/construction of the new embankments. Topsoil should be placed on exposed granular fill slopes in accordance with OPSS 802 (Topsoil) and covered with erosion protection in accordance with OPSS 804 (Seed and Cover) or pegged sod in accordance with OPSS 803 (Sodding).

## 6.7 Seismic Considerations

### 6.7.1 Site Coefficient

The soil profile type at this site has been classified as Type III according to the CHBDC. Therefore, according to Table 4.4 of the CHBDC, a Site Coefficient "S" (ground motion amplification factor) of 1.5 should be used in seismic design, if required.

### 6.7.2 Seismic Analysis Coefficient

The potential for seismic (earthquake) loading must also be considered for the design of abutment stems/retaining walls in accordance with Section 4.6 of the CHBDC. According to Table A3.1.1 of the CHBDC, this site is located in Seismic Performance Zone 1. In accordance with Section 4.4.5.1 of the CHBDC, seismic analysis is not required for structures located in Seismic Performance Zone 1.



### 6.8 Lateral Earth Pressures for Design

The lateral earth pressures acting on the abutment walls and associated wing walls will depend on the type and method of placement of the backfill materials, the nature of the soils behind the backfill, the magnitude of surcharge including construction loadings, the freedom of lateral movement of the structure, and the drainage conditions behind the walls. As discussed in Section 6.7.2, seismic (earthquake) loadings are not anticipated to be required for this structure.

The following recommendations are made concerning the design of the abutment walls and wing walls. These design recommendations and parameters assume level backfill and ground surface behind the walls. Where there is sloping ground behind the walls, the coefficient of lateral earth pressure must be adjusted to account for the slope.

- Select free-draining granular fill meeting the specifications of SP 110S13 (Aggregates) Granular “A” or Granular “B” Type II but containing less than 5 percent passing the No. 200 sieve size should be used as backfill behind abutments and wing walls. Compaction (including type of equipment, target densities, etc.) should be carried out in accordance with OPSS 501 (Compacting). Longitudinal drains and weep holes should be installed consistent with OPSD 3102.100 to provide positive drainage of the granular backfill. Other aspects of the granular backfill requirements with respect to sub-drains and frost taper should be in accordance with OPSD 3101.150 (Walls, Abutment, Backfill) and 3121.150 (Walls, Retaining, Backfill).
- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the walls, in accordance with CHBDC Section 6.9.3 and Figure 6.6. Other surcharge loadings should be accounted for in the design, as required.
- For restrained structures, the granular fill should be placed in a zone with width equal to at least 1.6 m behind the back of the walls (in accordance with Figure C6.20(a) of the Commentary to the CHBDC).

For unrestrained structures, granular fill should be placed within the wedge shaped zone defined by a line drawn at no steeper than 1.5H:1V extending up and back from the rear face of the base of the footing (in accordance with Figure C6.20(b) of the Commentary to the CHBDC).

- For restrained or unrestrained structures, the pressures are based on the proposed embankment fill materials and the existing overburden soils and the following parameters (unfactored) may be used assuming the use of granular or earth fill:

	Granular “A”	Granular “B” Type II	Earth Fill
Soil unit weight:	22 kN/m <sup>3</sup>	21 kN/m <sup>3</sup>	21 kN/m <sup>3</sup>
Coefficients of static lateral earth pressure:			
Active, $K_a$	0.27	0.27	0.33
At Rest, $K_o$	0.43	0.43	0.50





The movement to allow active pressures to develop within the backfill, and thereby assume an unrestrained structure for design, should be calculated in accordance with Section C6.9.1 and Table C6.6 of the Commentary to the CHBDC.

## 6.9 Construction Considerations

### 6.9.1 Excavation and Temporary Protection Systems

Excavations for the construction of the foundations or pile cap at the abutments and wing walls will extend through the existing fill and possibly into the very soft to stiff clayey silt to silty clay and the very loose to compact sand/silt deposits. Depending on the Sturgeon River water level at the time of construction, the excavations may also extend below the river water level (measured at Elevation 186 m in June 2011) and temporary dewatering may be necessary as discussed in Section 6.9.2.

Where space permits, open-cut excavations into these materials should be carried out in accordance with the guidelines outlined in the Occupational Health and Safety Act (OHSA) for Construction Activities. The existing fill, the firm to stiff clayey silt to silty clay and the loose to compact sand and silt soils above the groundwater level would be classified as Type 3 soils, and the very loose to loose sand and silt below the groundwater level and very soft to soft clayey silt to silty clay soils would be classified as Type 4 soil according to the OHSA. Temporary excavation (i.e. those that are open for a relatively short time period) should be made with side slopes no steeper than 1H:1V in Type 3 soils and with side slopes no steeper than 3H:1V in Type 4 soils. If Type 4 soils are expected to be encountered within a subexcavation (near the ground surface or at depth), the sides of the excavation should be sloped at 3H:1V from ground surface down to the bottom of the Type 4 soil.

Based on discussions with MH, we understand that the existing bridge abutments and wing walls will be used as temporary shoring and partial groundwater control system for the construction of the proposed bridge foundations and walls. However, it is anticipated that temporary roadway protection will be required to facilitate construction staging, the removal of the existing wing walls/abutments and excavation for construction of the new abutment foundations and associated wing walls. The temporary excavation support system should be designed and constructed in accordance with OPSS 539 (Temporary Protection Systems). The lateral movement of the temporary shoring systems should meet Performance Level 2 as specified in OPSS 539. It is considered that an interlocking steel sheet pile system or a soldier pile and timber lagging system would be suitable for the temporary excavation support at this site based on the subsurface soils and groundwater conditions. If deep subexcavation for the shallow foundation options is considered, a more elaborate excavation support system will be required.

### 6.9.2 Groundwater Control

Depending on the Sturgeon River water level and the founding level of the pile cap, dewatering may be necessary during the period of construction of the pile cap. The groundwater level was measured at Elevation 176.8 m on September 30, 2011 in Borehole 4. The anticipated base of the pile cap is at Elevation 175.4 m as shown on the preliminary GA drawing. As a result, excavations up to 1.4 m below the groundwater level are anticipated. If CSPs are to be installed as part of the integral abutment design, the base of the CSPs could be up to 4.4 m below groundwater level. For excavations up to about 1.5 m below groundwater level, it is anticipated that water inflow can be handled by pumping from filtered sump pumps placed at the base of the excavations. If dewatering is required for placement of the CSPs (up to 4.4 m below groundwater level), or





subexcavation/replacement options for shallow foundation (up to 7 m below groundwater level) the design of a dewatering system by a dewatering specialist retained by the Contractor would likely be required. The dewatering system must be effective and the groundwater level must be depressed to not less than 0.5 m below the base of the deepest excavation to maintain a stable base and prevent soil disturbance by construction traffic/equipment.

The volume of water that will be removed by dewatering/unwatering may trigger the need for a Permit to Take Water (PTTW). It is recommended that an application for a PTTW be submitted to MOE in time to have the PTTW in place at the time of contract award to avoid delays in the construction schedule.

Water discharged from dewatering operations or displaced during concrete placement may not be suitable for direct discharge to the Sturgeon River. The Contract Documents must alert the Contractor to this fact and include an item for treatment of the water to the satisfaction of MOE, MNR, DFO or other agencies having jurisdiction, prior to discharge to the river.

All surface water should be diverted away from any temporary excavation.

### 6.9.3 Settlement Monitoring During Pile Installation

Depending on the chosen replacement option and construction sequence, settlement monitoring is recommended at the existing bridge structure to ensure that the differential settlement / structural integrity of the existing bridge is maintained within tolerable levels during live traffic conditions. Pile driving operations will create vibrations which may induce settlement of the fill and silty sand soils adjacent to and/or below the existing bridge foundations. From a foundations perspective, a differential settlement of 25 mm is considered tolerable between abutments during construction, however, this value needs to be confirmed and/or determined by a structural engineer prior to construction operations to ensure that a safe structure is maintained during live traffic conditions as part of the staging process. The allowable/tolerable differential settlement within each abutment footprint must also be determined by the structural design prior to construction operations. A series of monitoring points along each bridge abutment stem is recommended; however, the structural engineer should determine the location of these settlement monitoring points in order to properly assess the structural integrity of the existing bridge during construction should settlements occur.



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### 7.0 CLOSURE

This Foundation Design report was prepared by Mr. Gilberto Alexandre and reviewed by Mr. Kevin J. Bentley, P.Eng., a senior geotechnical engineer with Golder. Mr. Jorge M.A. Costa, P.Eng., the Designated MTO Foundations Contact and Principal with Golder, conducted an independent quality control review of this report.

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### Ontario Provincial Standard Specifications (OPSS)

OPSS 501	Construction Specification For Compacting
OPSS 539	Construction Specification for Temporary Protection Systems
OPSS 802	Construction Specification for Topsoil
OPSS 803	Construction Specification for Sodding
OPSS 804	Construction Specification for Seed and Cover
OPSS 902	Construction Specification for Excavating and Backfilling Structures
OPSS 903	Construction Specification for Deep Foundations

### Ontario Provincial Standard Drawings (OPSD)

OPSD 3000.201	Foundation Piles, Steel HP 310 Oslo Point
OPSD 3001.100	Foundation Piles, Steel Tube Pile Driving Shoe
OPSD 3090.101	Foundation Frost Penetration Depths for Southern Ontario
OPSD 3101.150	Walls, Abutment, Backfill, Minimum Granular Requirement
OPSD 3102.100	Walls, Abutment, Backfill Drain
OPSD 3121.150	Walls, Retaining, Backfill, Minimum Granular Requirement



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### **Construction Design Estimating and Documentation (CDED) Special Provisions (SP)**

SP 110S13	Material Specification for Aggregates - Base, Subbase, Select Subgrade and Backfill Material
SP 206S03	Earth Excavation, Grading

### **ASTM International**

ASTM D1586	Standard Test Method for Standard Penetration Test (SPT) and Split Barrel Sampling of Soils
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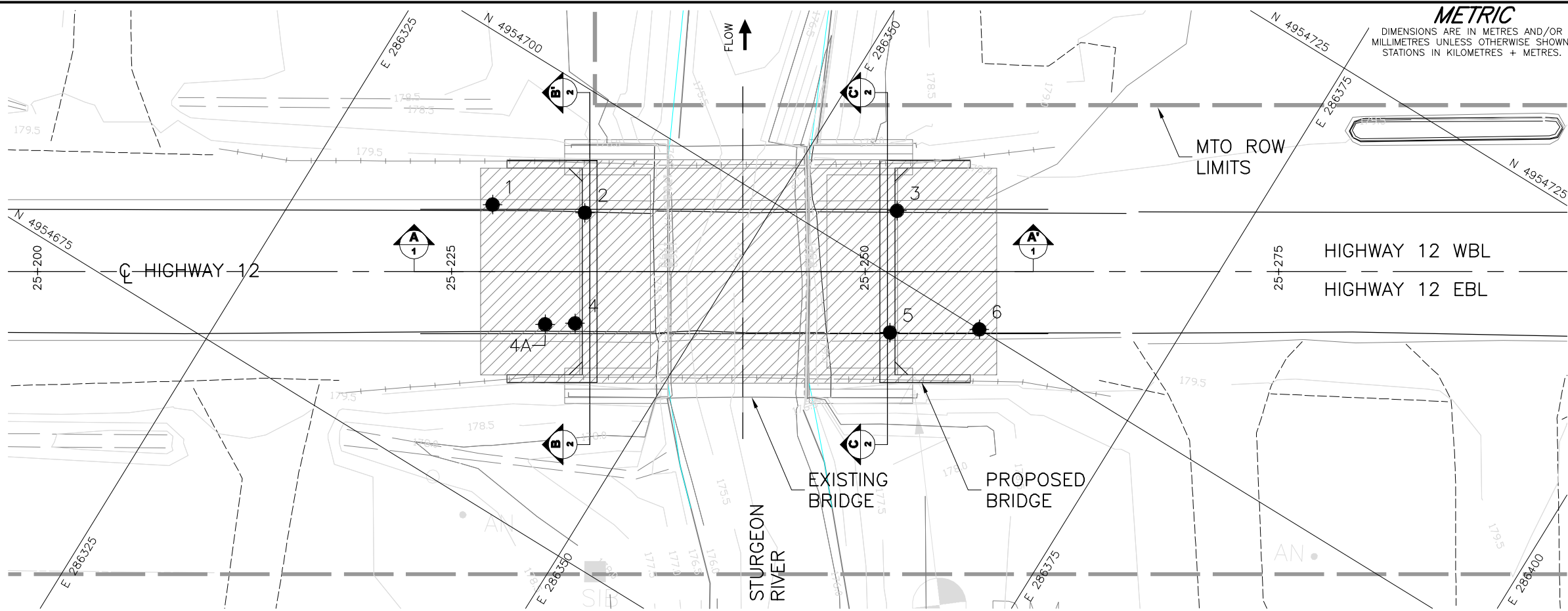
**Table 1 – Comparison of Foundation Alternatives  
Highway 12 / Sturgeon River Bridge Replacement  
W.P. 2004-08-00**

Foundation Option	Advantages	Disadvantages	Relative Costs	Risks
Steel H-piles or steel tube piles driven to refusal on the bedrock	<ul style="list-style-type: none"> <li>Allows for integral abutment construction (H-piles more conventional)</li> <li>Conventional deep foundations construction procedure</li> <li>Higher resistances than shallow foundations</li> </ul>	<ul style="list-style-type: none"> <li>Driving of piles may induce settlement of existing foundations (depending on the type of foundation supporting the structure)</li> <li>Pile cap below groundwater level requires dewatering of excavation</li> </ul>	<ul style="list-style-type: none"> <li>Typically \$250/m for steel H-piles (HP310x110) (20 pilesx2x 20 m @ \$250/m ≈ \$200,000)</li> </ul>	<ul style="list-style-type: none"> <li>Driving of piles may induce settlement of existing foundations;</li> <li>Dewatering for pile cap may induce settlement of the existing foundations</li> <li>Potential for Significant traffic disruption.</li> </ul>
Caissons socketted into bedrock	<ul style="list-style-type: none"> <li>Higher capacity than steel piles, so reduced number of deep foundation elements compared to steel piles</li> <li>Rigid foundation system in comparison to H-piles (preferred option for a rigid frame bridge).</li> <li>Allows for pile cap to be constructed at underside of superstructure (no excavation).</li> </ul>	<ul style="list-style-type: none"> <li>Will require temporary or permanent steel liners;</li> <li>Will need to be socketted into hard bedrock and will require good seal at soil/bedrock interface to prevent soil infiltration during construction</li> <li>Does not allow for integral abutment design</li> <li>Pile cap below groundwater level requires dewatering of excavation if caissons cannot be extended to underside of superstructure</li> </ul>	<ul style="list-style-type: none"> <li>Higher costs than piles but higher capacity will result in fewer caissons</li> <li>Added cost for liners</li> <li>Typically \$2,500/m (for 0.9 m diameter caisson) (6 caissons x 2 x 22 m @ \$2,500/m ≈ \$660,000)</li> </ul>	<ul style="list-style-type: none"> <li>Water-bearing cohesionless deposits could contribute to loss of ground during or after drilling; likely not possible to inspect caisson base or rely on base resistance.</li> <li>Dewatering (if required for cap and/or installation of caissons) may induce settlement of the existing foundations.</li> <li>Difficulties achieving tight seal in granite gneiss bedrock resulting in greater dewatering operation</li> <li>Environmental controls required during construction/tremie concrete procedures</li> <li>Larger equipment may pose difficulties for staging operations and potential for significant traffic disruption</li> </ul>

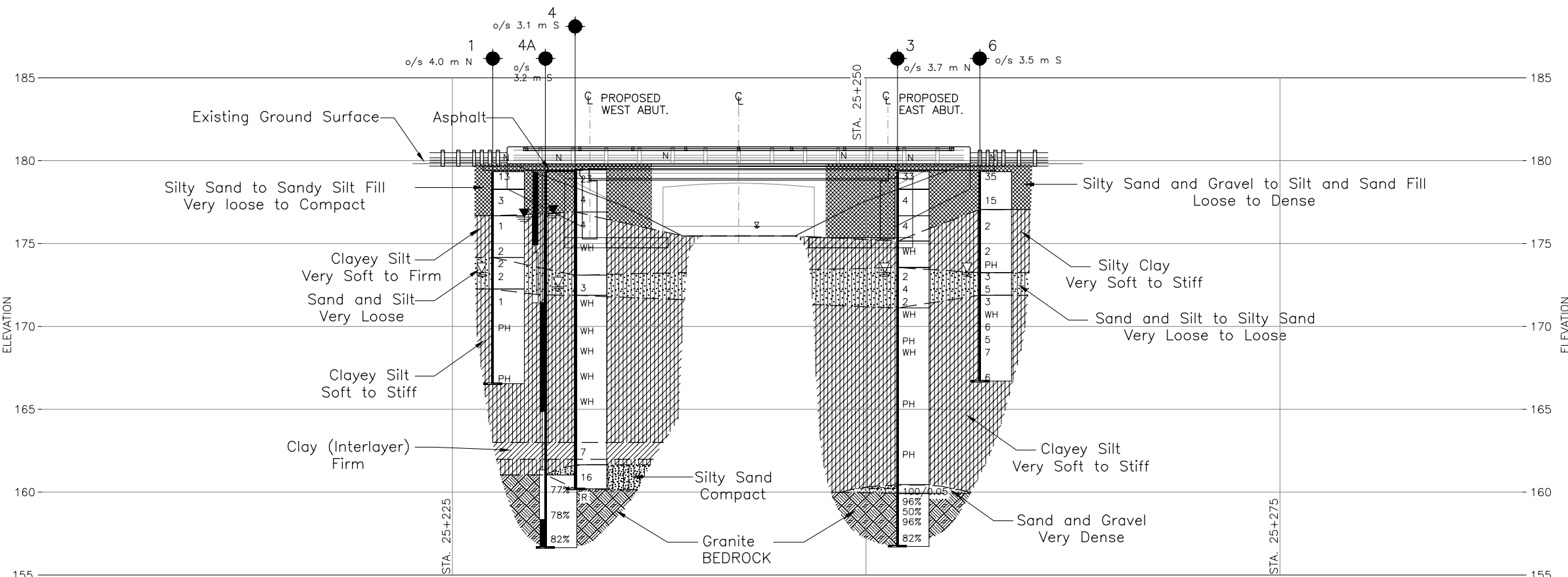


## FOUNDATION REPORT - STURGEON RIVER BRIDGE

Foundation Option	Advantages	Disadvantages	Relative Costs	Risks
Spread/Strip footings founded on the firm to stiff clayey silt to silty clay	<ul style="list-style-type: none"><li>• Shoring and Dewatering requirements are somewhat standard construction procedures and local competent contractors likely available</li></ul>	<ul style="list-style-type: none"><li>• Much lower (and possibly not practical) geotechnical resistances based on results of investigation compared to deep foundation options;</li><li>• High potential for differential settlement and tilting due to the presence of variable thickness and consistency of soft cohesive materials and very loose cohesionless soils at the site;</li><li>• Dewatering system required for construction of the footings and/or excavation/replacement with engineered fill options in dry conditions</li><li>• Scour protection required</li><li>• Does not allow for integral abutment design.</li><li>• Excavation support system (shoring) will be required</li><li>• Generates excavation spoil requiring proper disposal</li></ul>	<ul style="list-style-type: none"><li>• Excavation support system and dewatering system will result in much higher costs than for conventional shallow foundation option.</li><li>• Likely less costly than deep foundations but cost likely to increase depending on dewatering/shoring requirements.</li></ul>	<ul style="list-style-type: none"><li>• Potential for tilting/differential settlement due to the presence of soft cohesive and very loose soils at the site;</li><li>• Dewatering may induce settlement of the existing foundations and surrounding grade;</li><li>• Significant traffic disruption and requirement for more elaborate temporary excavation support systems.</li><li>• Existing foundation subgrade level not clear on design drawings and risk of undermining existing foundation during construction</li></ul>



PLAN



CENTRELINE PROFILE

1



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

CONT No.  
WP No. 2004-08-00

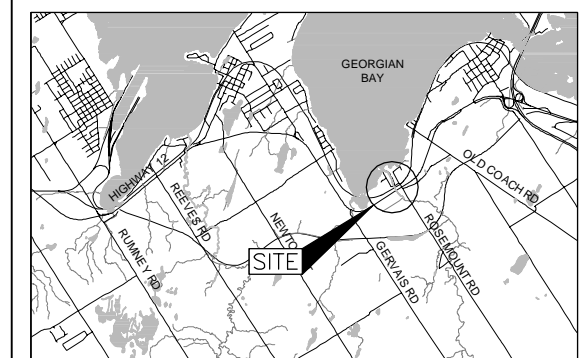


HIGHWAY 12  
STURGEON RIVER BRIDGE REPLACEMENT  
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET



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



KEY PLAN

SCALE  
1.5 0 1.5 3 km

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 Sept. 30, 2011
- ≡ WL upon completion of drilling

BOREHOLE CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
1	179.5	4954691.0	286335.5
2	179.5	4954693.5	286340.5
3	179.5	4954703.5	286356.5
4	179.5	4954687.5	286343.5
4A	179.5	4954686.5	286342.0
5	179.5	4954697.0	286360.0
6	179.5	4954700.0	286364.5

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.

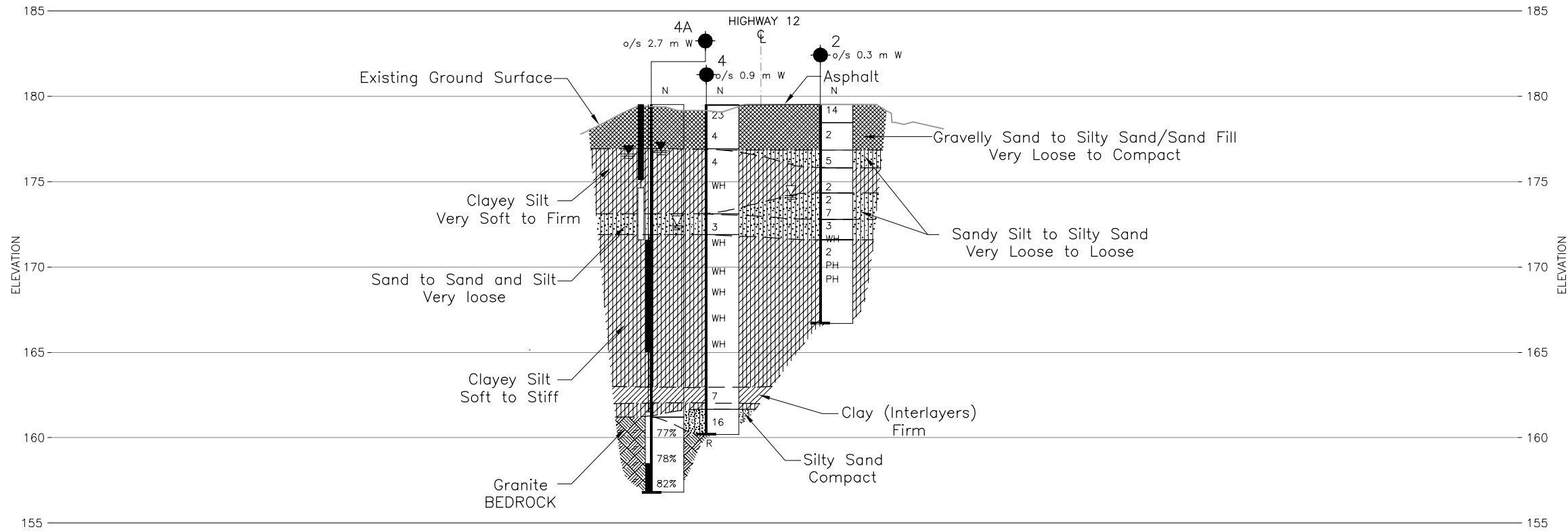
REFERENCE

Base plans provided in digital format by Morrison Hershfield, drawing file nos. 3022-01.dwg, Sturgeon River-Contour.dwg, x104178Align-Sturgeon.dwg and x104178Base-Sturgeon.dwg, received October 24, 2011Y.

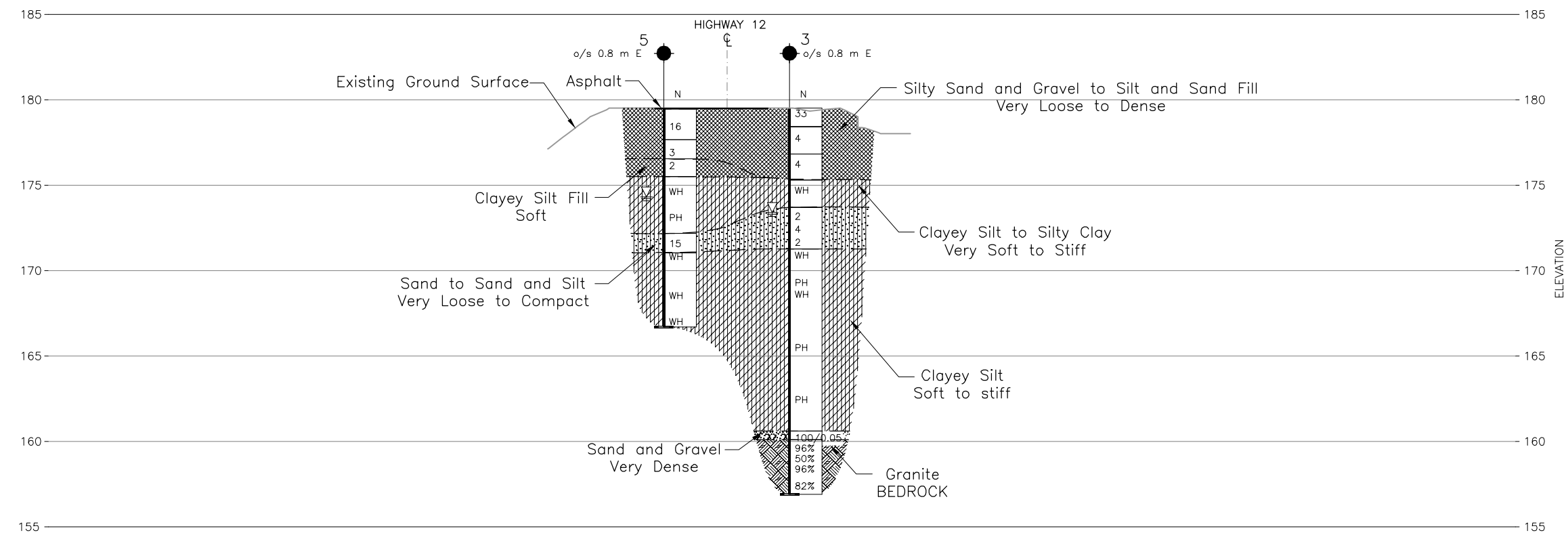
NO.	DATE	BY	REVISION
Geocres No. 31D-546			
HWY. 12		PROJECT NO. 11-1111-0077	DIST.
SUBM'D. GA	CHKD. KJB	DATE: 8/30/2012	SITE: 30-22
DRAWN: CD	CHKD.	APPD. JMAC	DWG. 1



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



**CROSS-SECTION AT WEST ABUTMENT**



**CROSS-SECTION AT EAST ABUTMENT**



CONT No.  
WP No. 2004-08-00

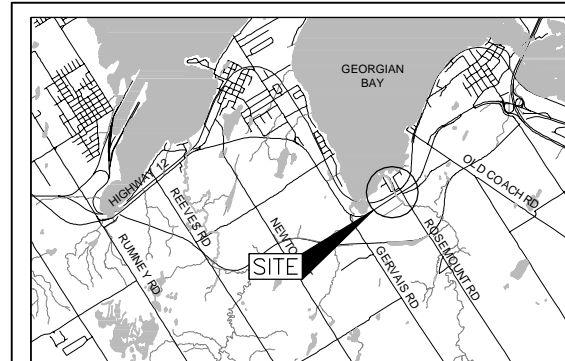
HIGHWAY 12  
STURGEON RIVER BRIDGE REPLACEMENT  
SOIL STRATA



SHEET



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



KEY PLAN  
SCALE 1.5 0 1.5 3 km

**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 Sept. 30, 2011
- WL upon completion of drilling

**BOREHOLE CO-ORDINATES**

No.	ELEVATION	NORTHING	EASTING
2	179.5	4954693.5	286340.5
3	179.5	4954703.5	286356.5
4	179.5	4954687.5	286343.5
4A	179.5	4954686.5	286342.0
5	179.5	4954697.0	286360.0

**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 plans provided in digital format by Morrison Hershfield, drawing file nos. 3022-01.dwg, Sturgeon River-Contour.dwg, x104178Align-Sturgeon.dwg and x104178Base-Sturgeon.dwg, received October 24, 2011Y.

NO.	DATE	BY	REVISION
Geocres No. 31D-546			
HWY. 12		PROJECT NO. 11-1111-0077	DIST.
SUBM'D. GA	CHKD. KJB	DATE: 8/30/2012	SITE: 30-22
DRAWN: CD	CHKD.	APPD. JMAC	DWG. 2





# **APPENDIX A**

## **Record of Boreholes and Drillholes**



## 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
SS	Split-spoon
DS	Denison type sample
FS	Foil sample
RC	Rock core
SC	Soil core
ST	Slotted tube
TO	Thin-walled, open
TP	Thin-walled, piston
WS	Wash sample

### 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.) drive open sampler for a distance of 300 mm (12 in.)

#### Dynamic Cone Penetration Resistance; $N_d$ :

The number of blows by a 63.5 kg (140 lb.) hammer dropped 760 mm (30 in.) to drive uncased a 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

#### Piezo-Cone Penetration Test (CPT)

A electronic cone penetrometer with a 60° conical tip and a project 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 (PWP) and friction along a sleeve are recorded electronically at 25 mm penetration intervals.

### III. SOIL DESCRIPTION

#### (a) Cohesionless Soils

Density Index	N
Relative Density	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 Consistency

	$C_u, S_u$	
	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
$w_p$	plastic limit
$w_l$	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 (specific gravity, $G_s$ )
DS	direct shear test
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 (LV-laboratory vane test)
$\gamma$	unit weight

**Note:** 1 Tests which are anisotropically consolidated prior to shear are shown as CAD, CAU.

### V. MINOR SOIL CONSTITUENTS

Percent by Weight	Modifier	Example
0 to 5	Trace	Trace sand
5 to 12	Trace to Some (or Little)	Trace to some sand
12 to 20	Some	Some sand
20 to 30	(ey) or (y)	Sandy
over 30	And (cohesionless) or With (cohesive)	Sand and Gravel Silty Clay with sand / Clayey Silt with sand



## 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
F	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 ( $\sigma' = \sigma - \mu$ )
$\sigma'_{vo}$	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stress (major, intermediate, minor)
$\sigma_{oct}$	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
$\tau$	shear stress
$\mu$	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

#### (a) Index Properties (continued)

w	water content
$w_l$	liquid limit
$w_p$	plastic limit
$I_p$	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 (over-consolidated range)
$C_s$	swelling index
$C_a$	coefficient of secondary consolidation
$m_v$	coefficient of volume change
$c_v$	coefficient of consolidation
$T_v$	time factor (vertical direction)
U	degree of consolidation
$\sigma'_p$	pre-consolidation pressure
OCR	over-consolidation ratio $= \sigma'_p / \sigma'_{vo}$

#### (d) Shear Strength

$T_p, T_r$	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
$c_u, s_u$	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

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

Notes: 1  $\tau = c' + \sigma' \tan \phi'$   
2 shear strength = (compressive strength)/2



## LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

### WEATHERINGS STATE

**Fresh:** no visible sign of 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 and structure are preserved.

### BEDDING THICKNESS

<u>Description</u>	<u>Bedding Plane Spacing</u>
Very thickly bedded	Greater than 2 m
Thickly bedded	0.6 m to 2 m
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	Less than 6 mm

### JOINT OR FOLIATION SPACING

<u>Description</u>	<u>Spacing</u>
Very wide	Greater than 3 m
Wide	1 m to 3 m
Moderately close	0.3 m to 1 m
Close	50 mm to 300 mm
Very close	Less than 50 mm

### GRAIN SIZE

<u>Term</u>	<u>Size*</u>
Very Coarse Grained	Greater than 60 mm
Coarse Grained	2 mm to 60 mm
Medium Grained	60 microns to 2 mm
Fine Grained	2 microns to 60 microns
Very Fine Grained	Less than 2 microns

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

### CORE CONDITION

#### Total Core Recovery (TCR)

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 varied from 0% for completely broken core to 100% for core in solid sticks.

### DISCONTINUITY DATA

#### Fracture Index

A count of the number of discontinuities (physical separations) in the rock core, including both naturally occurring fractures and mechanically induced breaks caused by drilling.

#### Dip with Respect to Core Axis

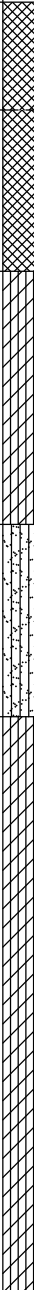
The angle of the discontinuity relative to the axis (length) of the core. In a vertical borehole a discontinuity with a 90° angle is horizontal.

#### Description and Notes

An abbreviation description of the discontinuities, whether naturally occurring separations such as fractures, bedding planes and foliation planes or mechanically induced features caused by drilling such as 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

JN Joint	PL Planar
FLT Fault	CU Curved
SH Shear	UN Undulating
VN Vein	IR Irregular
FR Fracture	K Slickensided
SY Stylolite	PO Polished
BD Bedding	SM Smooth
FO Foliation	SR Slightly Rough
CO Contact	RO Rough
AXJ Axial Joint	VR Very Rough
KV Karstic Void	
MB Mechanical Break	


PROJECT 11-1111-0077			RECORD OF BOREHOLE No 1			SHEET 1 OF 1			METRIC								
G.W.P. 2004-08-00			LOCATION N 4954691.0 ; E 286335.5			ORIGINATED BY DD											
DIST Midland HWY 12			BOREHOLE TYPE 108mm I.D. Continuous Flight Hollow Stem Auger			COMPILED BY CS											
DATUM Geodetic			DATE August 2, 2011			CHECKED BY KJB											
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
179.5	GROUND SURFACE							20	40	60	80	100					
0.0	Silty sand, some gravel, trace clay (FILL) Compact Brown Moist		1	SS	13												
178.4	Sandy silt, trace gravel, trace clay (FILL) Very loose Brown Moist		2	SS	3												
176.8	CLAYEY SILT, trace sand Very soft Dark brown Moist Containing organics to a depth 3.6 m		3	SS	1												
174.3	SAND and SILT, trace to some clay Very loose Brown Wet		4	SS	2												
172.4	CLAYEY SILT, trace sand Soft to stiff Grey Moist		5	SS	2												
			6	SS	2												
			7	SS	1												
			8	TO	PH												
			9	TO	PH												
166.7	END OF BOREHOLE																
12.8	NOTE: 1. Water level in open borehole at a depth of 6.1 m below ground surface (Elev. 173.4 m) upon completion of drilling.																

+ 3, × 3: Numbers refer to Sensitivity      ○ 3% STRAIN AT FAILURE

GTA-MTO 001 11-1111-0077.GPJ GAL-GTA.GDT 8/30/12 CD

PROJECT 11-1111-0077		RECORD OF BOREHOLE No 2		SHEET 1 OF 1		METRIC						
G.W.P. 2004-08-00		LOCATION N 4954693.5 ; E 286340.5		ORIGINATED BY DD								
DIST Midland HWY 12		BOREHOLE TYPE 108mm I.D. Continuous Flight Hollow Stem Auger		COMPILED BY CS								
DATUM Geodetic		DATE August 2, 2011		CHECKED BY KJB								
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC NATURAL LIQUID UNIT REMARKS			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100	W <sub>p</sub> W W <sub>L</sub>	WATER CONTENT (%)	γ	GR SA SI CL
179.5	GROUND SURFACE											
0.0	Gravelly sand, some silt, trace clay (FILL) Compact Brown Moist		1	SS	14		179					26 55 15 4
178.4	Silty sand, containing organics (FILL) Very loose Brown Moist		2	SS	2		178					
176.8	Sandy SILT, trace to some clay, trace gravel Loose Dark grey to grey Moist Containing organics to a depth of 3.7 m.		3	SS	5		177					4 31 55 10
175.8	CLAYEY SILT, trace to some sand Very soft Grey Moist		4	SS	2		176					
174.3	Sandy SILT to silty SAND, trace clay Loose to very loose Grey Wet		5	SS	2		175					
172.8	SAND, trace silt, trace to some gravel Very loose Grey Wet		6	SS	7		174					
171.6	CLAYEY SILT, trace to some sand Soft to stiff Grey Wet		7	SS	3		173					
166.7	NOTES: 1. Water level in open borehole at a depth of 5.3 m below ground surface (Elev. 174.2 m) upon completion of drilling. 2. Attempted Shelby Tube samples at depths of 9.1 m and 9.9 m below ground surface resulted in zero recovery.		8	SS	WH		172					
12.8	END OF BOREHOLE		9	SS	2		171					0 7 67 26
			-	TO	PH		170					
			-	TO	PH		169					
							168					
							167					



PROJECT 11-1111-0077			RECORD OF BOREHOLE No 3			SHEET 2 OF 2			METRIC																
G.W.P. 2004-08-00			LOCATION N 4954703.5 ; E 286356.5			ORIGINATED BY DD																			
DIST Midland HWY 12			BOREHOLE TYPE 108mm I.D. Continuous Flight Hollow Stem Auger-HQ Coring			COMPILED BY CS																			
DATUM Geodetic			DATE July 25 and 28, 2011			CHECKED BY KJB																			
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS			ELEVATION SCALE			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES																				
--- CONTINUED FROM PREVIOUS PAGE ---																									
160.6	CLAYEY SILT, trace sand Soft to stiff Grey Wet																								
163			12	TO	PH																				
162																									
161																									
18.9	SAND and GRAVEL, trace silt Very dense Grey to brown Wet		13	SS	100/0.05																				
19.4	Granitic GNEISS (BEDROCK)		1	HQ RC	REC 96%																			RQD = 96%	
	Bedrock cored from depths of 19.4 m to 22.6 m		2	HQ RC	REC 75%																			RQD = 50%	
	For bedrock coring detail, refer to Record of Drillhole 3		3	HQ RC	REC 98%																			RQD = 96%	
			4	HQ RC	REC 96%																			RQD = 82%	
156.9	END OF BOREHOLE																								
22.6	NOTE:  1. Water level in open borehole at a depth of 6.1 m below ground surface (Elev. 173.4 m) upon completion of drilling.																								



PROJECT: 11-1111-0077

**RECORD OF DRILLHOLE: 3**

SHEET 1 OF 1

LOCATION: N 4954703.5 ;E 286356.5

DRILLING DATE: July 25 and 28, 2011

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: D90

DRILLING CONTRACTOR: Walker Drilling Ltd.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No. PENETRATION RATE min/(m)	FLUSH COLOUR % RETURN	JN - Joint FLT - Fault SHR- Shear VN - Vein CJ - Conjugate BD- Bedding FO- Foliation CO- Contact OR- Orthogonal CL - Cleavage PL - Planar CU- Curved UN- Undulating ST - Stepped IR - Irregular PO- Polished K - Slickensided SM- Smooth Ro - Rough MB- Mechanical Break BR - Broken Rock NOTE: For additional abbreviations refer to list of abbreviations & symbols.														NOTES WATER LEVELS INSTRUMENTATION
							RECOVERY		R.Q.D. %	FRACT. INDEX PER 0.3 m	DISCONTINUITY DATA						HYDRAULIC CONDUCTIVITY K, cm/sec		Diametral Point Load Index (MPa)	RMC -Q AVG.	
							TOTAL CORE %	SOLID CORE %			B Angle	DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jn	10 <sup>-6</sup>	10 <sup>-5</sup>			
							000000	000000	000000	000000	000000	000000	000000	000000	000000	000000	000000	000000	000000	000000	
		Continued from Record of Borehole 3		160.09																	
	Uncased HQ Coring	Slightly weathered, banded pink, white and black, coarse grained, metamorphic, biotitic Granitic GNEISS (BEDROCK)		19.41	1																
20				2															0.2 (Axial)		
21				3																UCS = 82 MPa (Axial)	
22				4																	
		END OF DRILLHOLE		156.89																	
23				22.61																	
24																					
25																					
26																					
27																					
28																					
29																					

DEPTH SCALE

1 : 50



LOGGED: DD

CHECKED: KJB

GTA-RCK 037 11-1111-0077.GPJ GAL-MISS.GDT 9/6/12 CD



PROJECT <u>11-1111-0077</u>		<b>RECORD OF BOREHOLE No 4</b>		SHEET 1 OF 2 <b>METRIC</b>	
G.W.P. <u>2004-08-00</u>		LOCATION <u>N 4954687.5 ; E 286343.5</u>		ORIGINATED BY <u>JC</u>	
DIST <u>Midland</u> HWY <u>12</u>		BOREHOLE TYPE <u>108mm I.D. Continuous Flight Hollow Stem Auger-NQ Coring</u>		COMPILED BY <u>CS</u>	
DATUM <u>Geodetic</u>		DATE <u>August 4, 2011</u>		CHECKED BY <u>KJB</u>	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT			UNIT WEIGHT  γ  kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)		
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED							w <sub>p</sub> w w <sub>L</sub>		
								20 40 60 80 100	20 40 60								
179.5	GROUND SURFACE																
0.0	ASPHALT																
	Sand, some gravel, some silt, inferred cobbles at a depth of 2.6 m (FILL) Loose to compact Brown Moist		1	SS	23												
			2	SS	4												
176.9																	
2.6	CLAYEY SILT, trace sand Firm Grey Wet Containing organics to a depth 3.7 m.		3	SS	4												
			4	SS	WH												
173.1																	
6.4	SAND and SILT, trace clay Very loose Grey Wet		5	SS	3												
171.9																	
7.6	CLAYEY SILT, trace sand Firm to stiff Grey Wet		6	SS	WH												
			7	SS	WH												
			8	SS	WH												
			9	SS	WH												
			10	SS	WH												

Continued Next Page

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

GTA-MTO 001 11-1111-0077.GPJ GAL-GTA.GDT 8/30/12 CD

PROJECT 11-1111-0077			RECORD OF BOREHOLE No 4			SHEET 2 OF 2			METRIC															
G.W.P. 2004-08-00			LOCATION N 4954687.5 ; E 286343.5			ORIGINATED BY JC																		
DIST Midland HWY 12			BOREHOLE TYPE 108mm I.D. Continuous Flight Hollow Stem Auger-NQ Coring			COMPILED BY CS																		
DATUM Geodetic			DATE August 4, 2011			CHECKED BY KJB																		
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS			ELEVATION SCALE			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES																			
--- CONTINUED FROM PREVIOUS PAGE ---																								
161.7	CLAYEY SILT, trace sand Firm to stiff Grey Wet		11	SS	7																			
	Clay interlayers below 16.5 m depth (Elev. 163.0 m)																							
17.8	Silty SAND, trace clay, trace gravel Compact Brown Wet		12	SS	16																			
160.2	END OF BOREHOLE																							
19.3	AUGER REFUSAL																							
	NOTES:  1. Attempted to seat casing for coring operations; however 3.1 m of NW casing was lost at the bottom of borehole. Borehole backfilled and abandoned: moved 2.0 m west and drilled BH 4A.  2. Water level in open borehole at a depth of 7.1 m below ground surface (Elev. 172.4m) upon completion of drilling.																							

Continued Next Page

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

○ 3% STRAIN AT FAILURE

PROJECT		11-1111-0077		RECORD OF BOREHOLE No 4A		SHEET 2 OF 2		METRIC							
G.W.P.		2004-08-00		LOCATION		N 4954686.5 ; E 286342.0		ORIGINATED BY							
DIST		Midland HWY 12		BOREHOLE TYPE		108mm I.D. Continuous Flight Hollow Stem Auger-NW Casing		COMPILED BY							
DATUM		Geodetic		DATE		August 15, 2011		CHECKED BY							
KJB															
SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS		ELEVATION SCALE		DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT		UNIT WEIGHT		REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	W <sub>p</sub>	W	W <sub>L</sub>	γ	GR	SA	SI	CL		
161.2	Not sampled. Refer to Borehole 4 for soil profile														
18.3	Auger Refusal  Felspathic GRANITE (BEDROCK)  Bedrock cored from 18.3 m to 22.7 m  For Bedrock coring details, refer to Record of Drillhole 4A		1	NQ RC	REC 91%								RQD = 77%		
			2	NQ RC	REC 98%								RQD = 78%		
			3	NQ RC	REC 99%								RQD = 82%		
156.8	END OF BOREHOLE														
22.7	NOTES:  1. Drilled through overburden (unsampled) to bedrock surface. For overburden information refer to Record of Borehole 4.  2. Water level in deep piezometer at a depth of 2.7 m below ground surface (Elev. 176.8 m) on September 30, 2011.  3. Water level in shallow piezometer at a depth of 2.9 m below ground surface (Elev. 176.6 m) on September 30, 2011.														

GTA-MTO 001 11-1111-0077.GPJ GAL-GTA.GDT 8/30/12 CD

SHEET 1 OF 1

DATUM: Geodetic

DRILLING CONTRACTOR: Geo-Environmental Drilling Inc.

LOGGED: JC  
CHECKED: KJB

GTA-RCK 037 11-1111-0077.GPJ GAL-MISS.GDT 9/6/12 CD



PROJECT 11-1111-0077		RECORD OF BOREHOLE No 6		SHEET 1 OF 1		METRIC												
G.W.P. 2004-08-00		LOCATION N 4954700.0 ; E 286364.5		ORIGINATED BY DD														
DIST Midland HWY 12		BOREHOLE TYPE 108mm I.D. Continuous Flight Hollow Stem Auger		COMPILED BY CS														
DATUM Geodetic		DATE July 19, 2011		CHECKED BY KJB														
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa					WATER CONTENT (%)					
179.5	GROUND SURFACE							20 40 60 80 100	20 40 60 80 100	W <sub>p</sub>	W	W <sub>L</sub>						
0.0	Silty sand with gravel, inferred cobbles at 0.9 m depth (FILL) Compact to dense Brown Moist		1	SS	35		179											
			2	SS	15		178											
177.2	SILTY CLAY, trace sand, containing organics to a depth of 3.7 m Soft Dark brown to dark grey Moist		3	SS	2		177											
2.3			4	SS	2		176											
			5	TO	PH		175											
173.4	Silty SAND, trace gravel, trace clay Very loose to loose Brown Wet		6	SH	3		174											
6.1			7	SS	5		173											
172.0	CLAYEY SILT, some sand Very soft to firm Grey Wet		8	SS	3		172											
7.5			9	SS	WH		171											
	Containing seams of sandy silt between depths of 9.1 m and 11.0 m		10	SS	6		170											
			11	SS	5		169											
			12	SS	7		168											
166.8			13	SS	6		167											
12.7	END OF BOREHOLE																	
	NOTE: 1. Water level in open borehole at a depth of 6.1 m below ground surface (Elev. 173.4 m) upon completion of drilling.																	





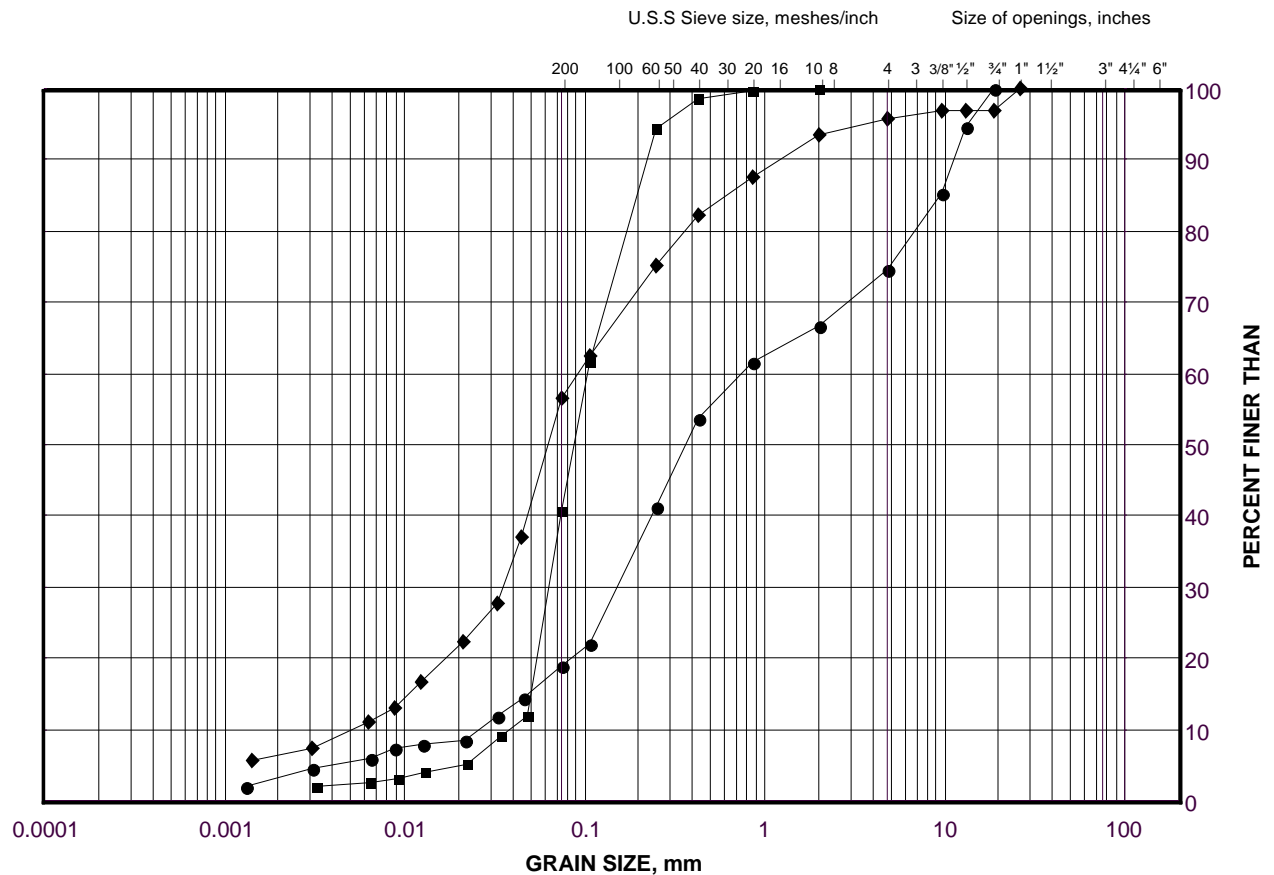
# **APPENDIX B**

## **Laboratory Test Results**

# GRAIN SIZE DISTRIBUTION

Sand and Silt to Gravelly Sand Fill

FIGURE B1



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

## LEGEND

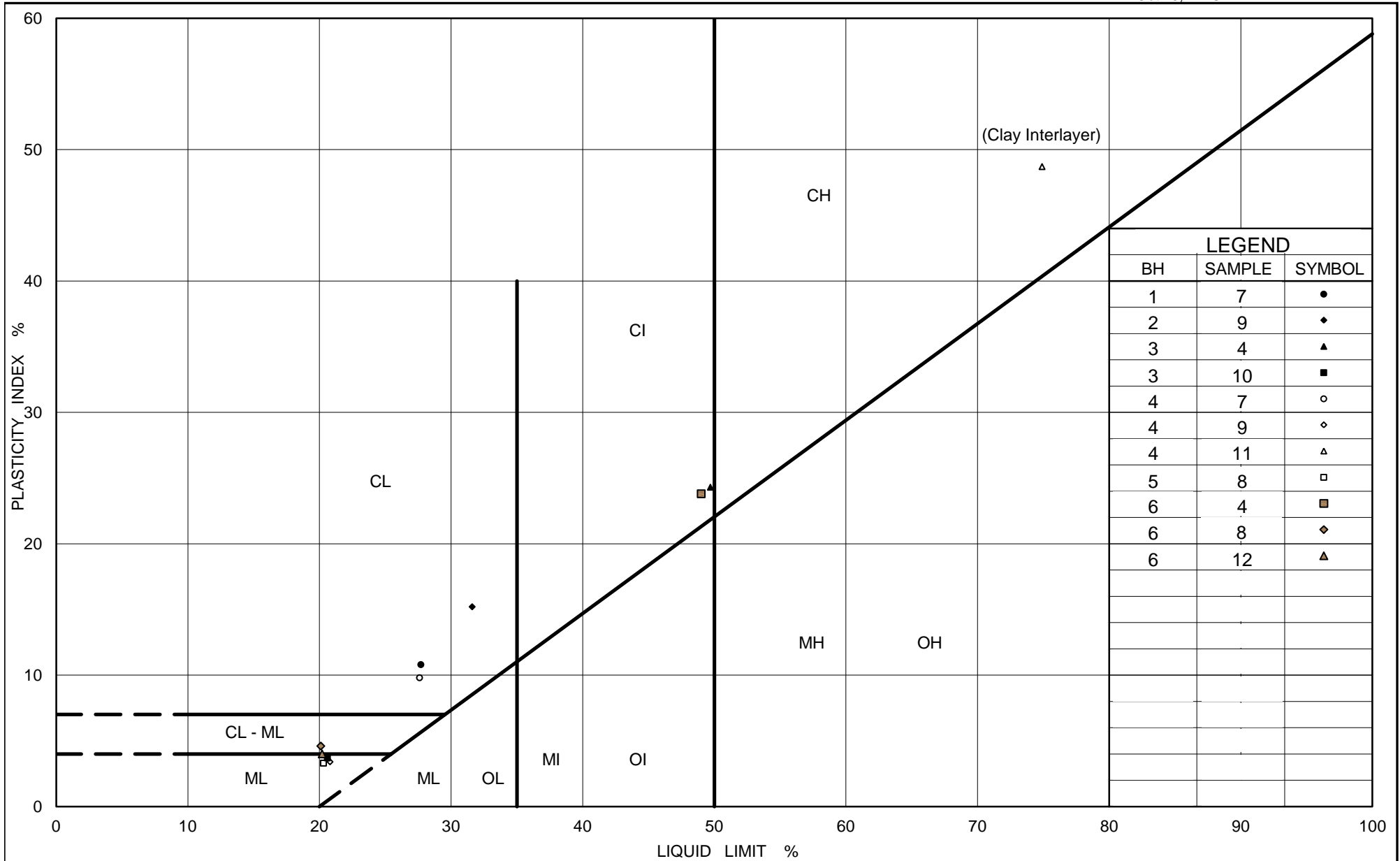
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	2	1	179.3
■	5	2	176.9
◆	3	2	177.8

Project Number: 11-1111-0077

Checked By: KJB

**Golder Associates**

Date: 08-Mar-12



Ontario

Ministry of  
Transportation

# PLASTICITY CHART Clayey Silt to Silty Clay

Figure No. B2

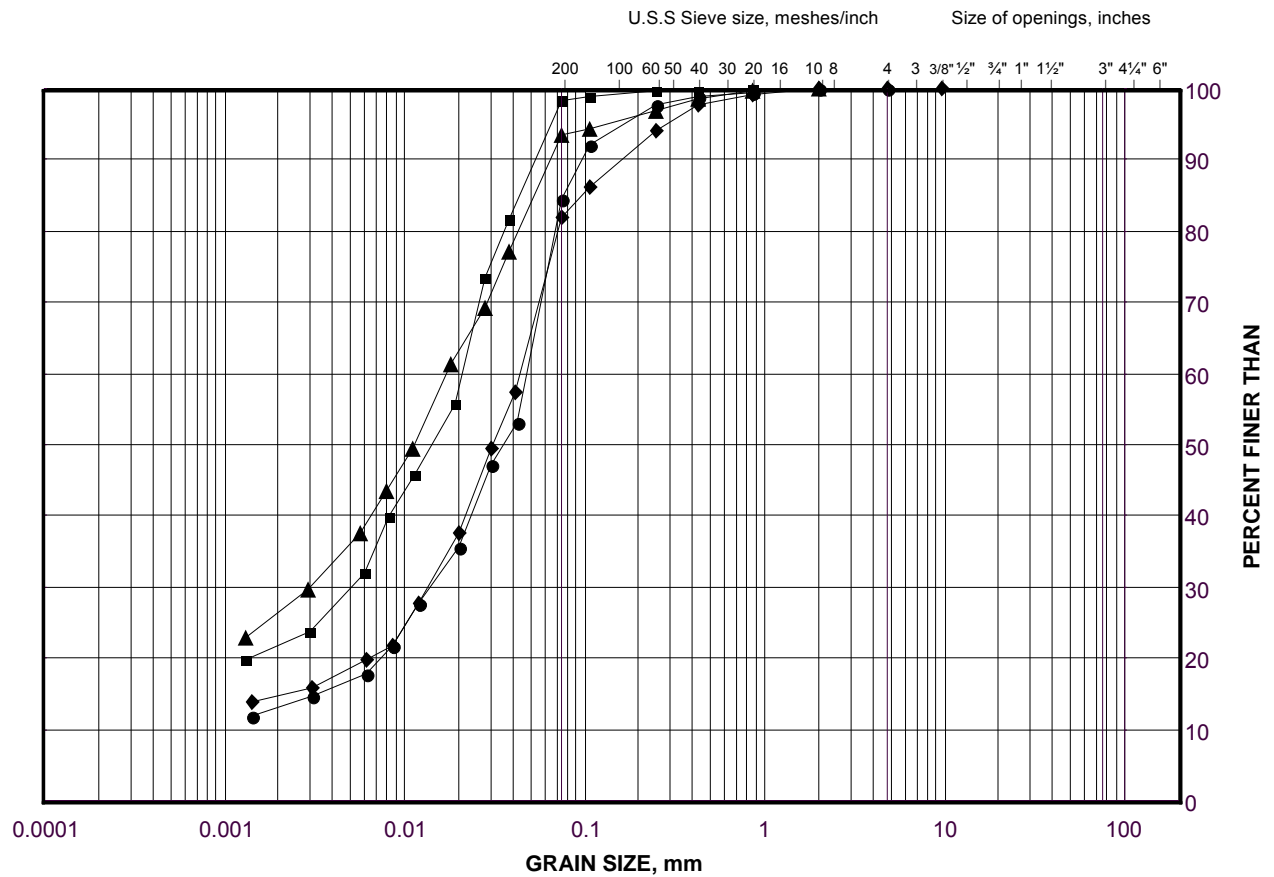
Project No. 11-1111-0077

Checked By: KJB

# GRAIN SIZE DISTRIBUTION

Clayey Silt

FIGURE B3



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

## LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	6	13	167.0
■	4	7	169.8
◆	6	8	171.7
▲	2	9	170.9

Project Number: 11-1111-0077

Checked By: KJB

Golder Associates

Date: 08-Mar-12

**CONSOLIDATION TEST SUMMARY****FIGURE B4**

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**SAMPLE IDENTIFICATION**

Project Number	11-1111-0077	Sample Number	11
Borehole Number	3	Sample Depth, m	13.72-14.33

**TEST CONDITIONS**

Test Type	Standard	Load Duration, hr	24
Oedometer Number	3		
Date Started	9/09/2011		
Date Completed	9/23/2010		

**SAMPLE DIMENSIONS AND PROPERTIES - INITIAL**

Sample Height, cm	2.54	Unit Weight, kN/m <sup>3</sup>	19.04
Sample Diameter, cm	6.32	Dry Unit Weight, kN/m <sup>3</sup>	14.30
Area, cm <sup>2</sup>	31.35	Specific Gravity, measured	2.76
Volume, cm <sup>3</sup>	79.51	Solids Height, cm	1.340
Water Content, %	33.08	Volume of Solids, cm <sup>3</sup>	42.02
Wet Mass, g	154.33	Volume of Voids, cm <sup>3</sup>	37.49
Dry Mass, g	115.97	Degree of Saturation, %	102.3

**TEST COMPUTATIONS**

Pressure	Corr. Height	Void Ratio	Average Height	t <sub>90</sub>	cv.	mv	k
kPa	cm		cm	sec	cm <sup>2</sup> /s	m <sup>2</sup> /kN	cm/s
0.00	2.536	0.892	2.536				
6.16	2.536	0.892	2.536	566	2.41E-03	3.20E-05	7.55E-09
11.04	2.529	0.887	2.532	375	3.62E-03	5.66E-04	2.01E-07
20.89	2.517	0.878	2.523	778	1.73E-03	4.80E-04	8.16E-08
40.50	2.494	0.861	2.505	463	2.87E-03	4.54E-04	1.28E-07
79.60	2.462	0.837	2.478	228	5.71E-03	3.22E-04	1.80E-07
157.56	2.412	0.800	2.437	346	3.64E-03	2.53E-04	9.04E-08
313.74	2.285	0.705	2.348	445	2.63E-03	3.21E-04	8.27E-08
626.61	2.167	0.617	2.226	463	2.27E-03	1.48E-04	3.30E-08
1251.21	2.061	0.537	2.114	346	2.74E-03	6.70E-05	1.80E-08
2500.77	1.967	0.467	2.014	190	4.52E-03	2.97E-05	1.32E-08
1251.21	1.971	0.470	1.969				
313.74	2.002	0.494	1.986				
79.60	2.039	0.521	2.020				
20.89	2.078	0.551	2.059				
6.16	2.107	0.572	2.093				

Note:  
k calculated using cv based on t<sub>90</sub> values.

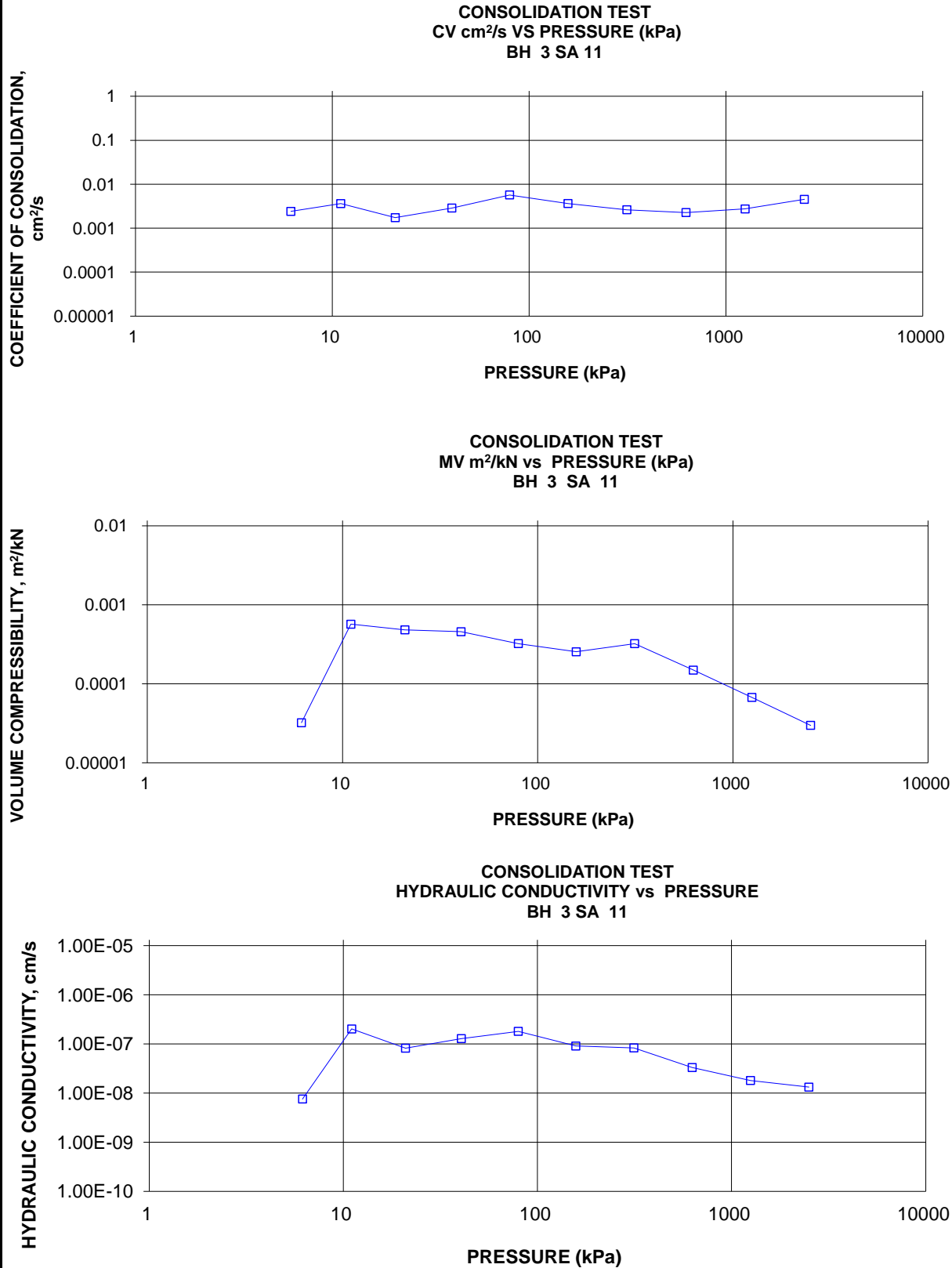
**SAMPLE DIMENSIONS AND PROPERTIES - FINAL**

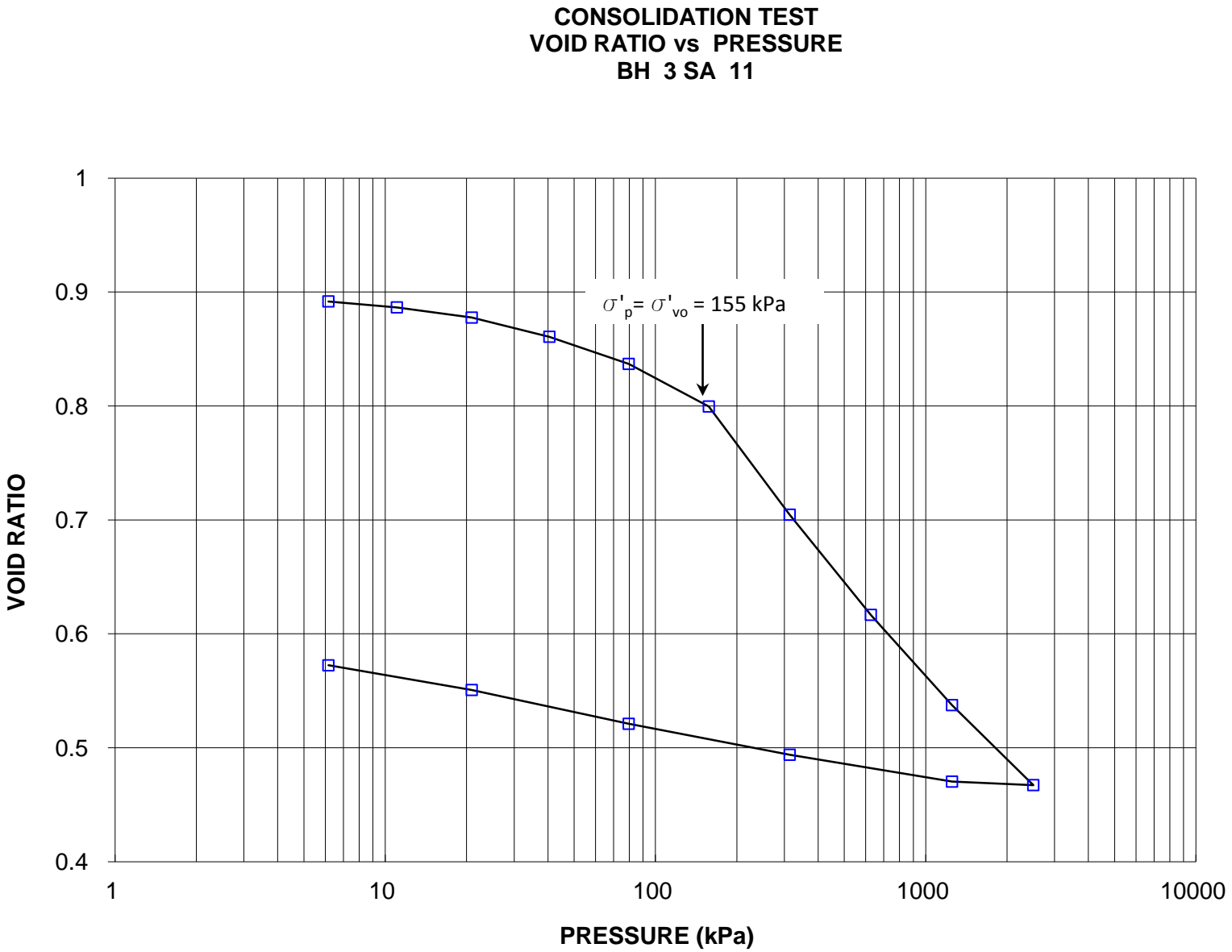
Sample Height, cm	2.11	Unit Weight, kN/m <sup>3</sup>	21.06
Sample Diameter, cm	6.32	Dry Unit Weight, kN/m <sup>3</sup>	17.21
Area, cm <sup>2</sup>	31.35	Specific Gravity, measured	2.76
Volume, cm <sup>3</sup>	66.07	Solids Height, cm	1.340
Water Content, %	22.36	Volume of Solids, cm <sup>3</sup>	42.02
Wet Mass, g	141.90	Volume of Voids, cm <sup>3</sup>	24.05
Dry Mass, g	115.97		

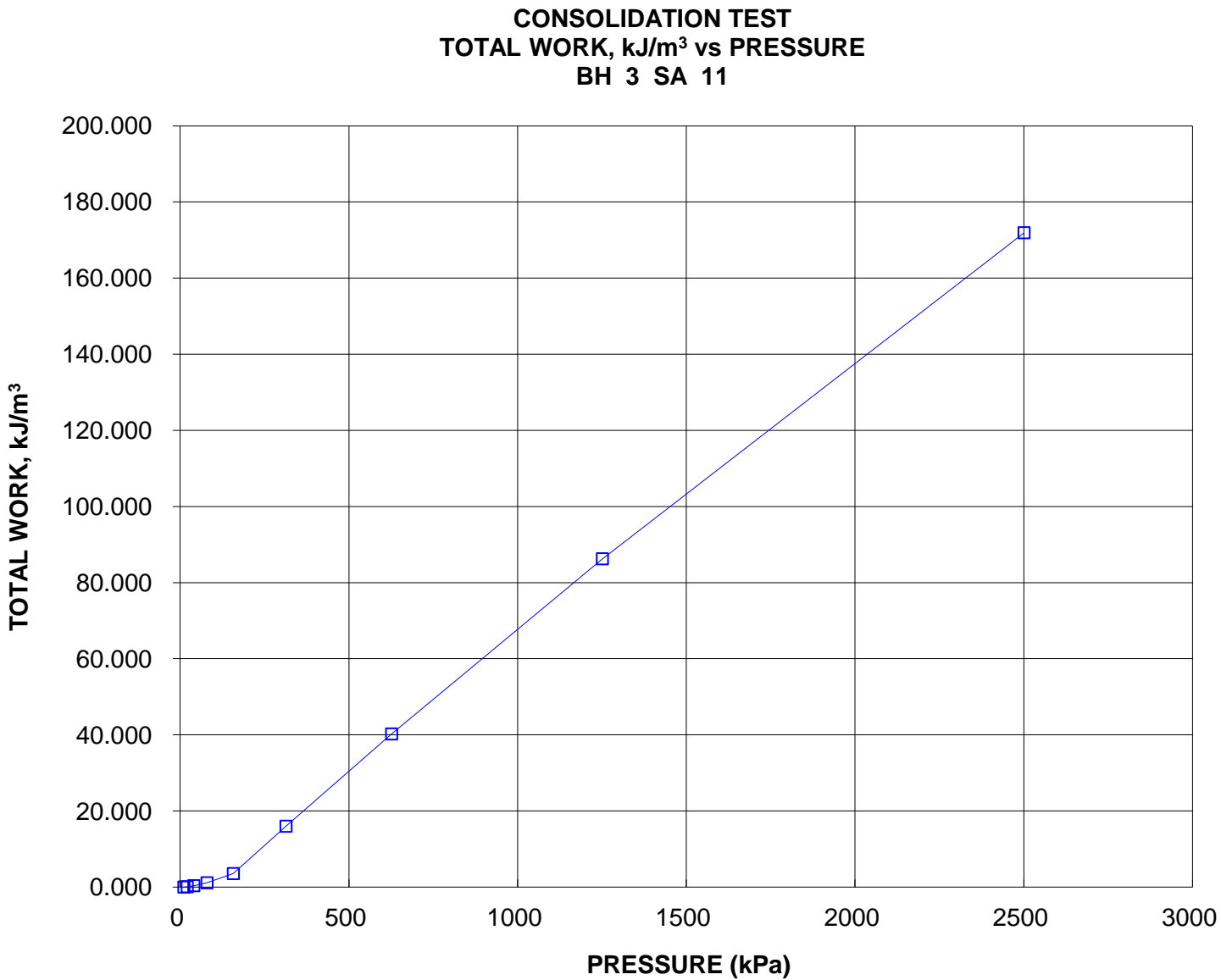
# CONSOLIDATION TEST SUMMARY

## FIGURE B4

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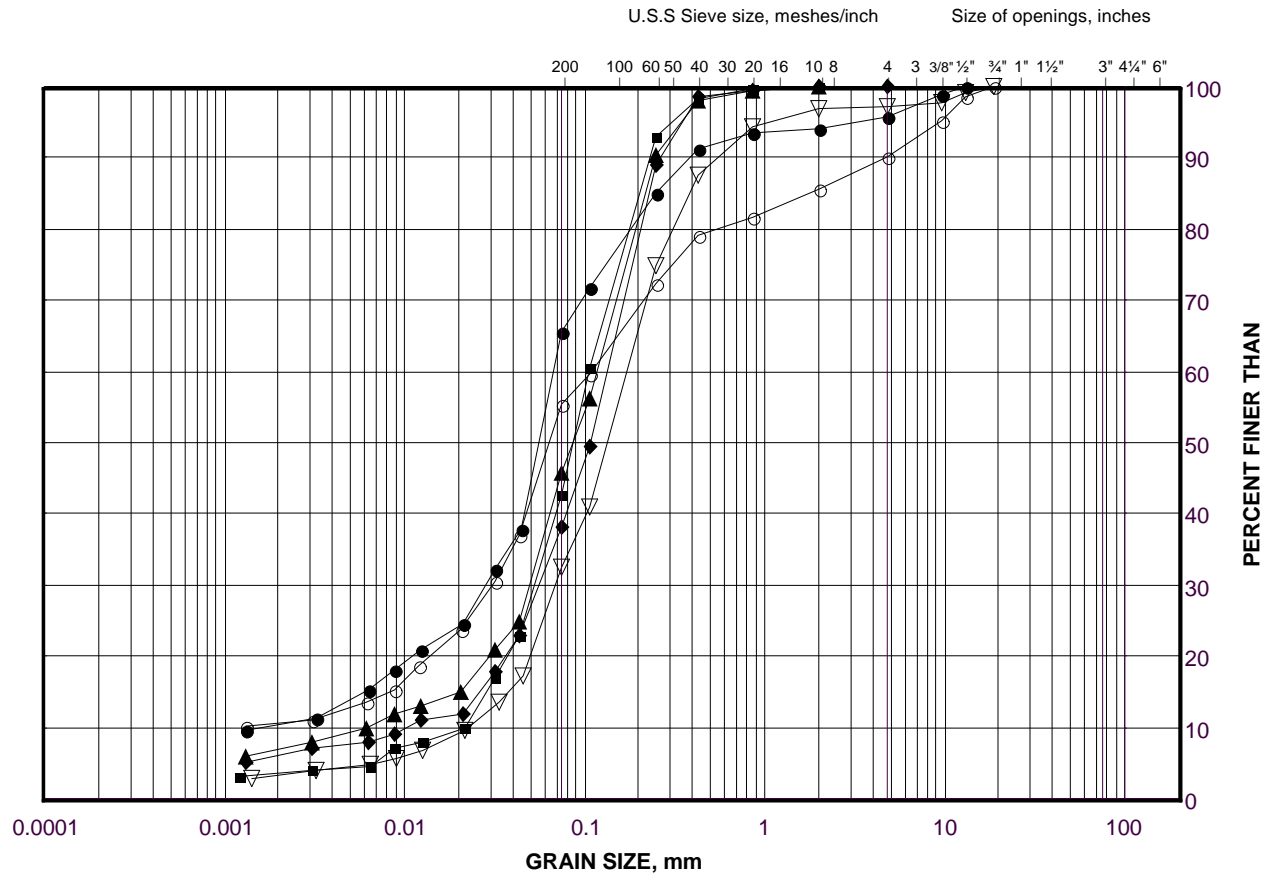




# GRAIN SIZE DISTRIBUTION

Sandy Silt to Silty Sand

FIGURE B5



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

## LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	2	3	176.2
■	4	5	172.3
◆	3	5	173.2
▲	1	5	173.9
▽	6	7	172.5
○	3	7	171.6

Project Number: 11-1111-0077

Checked By: KJB

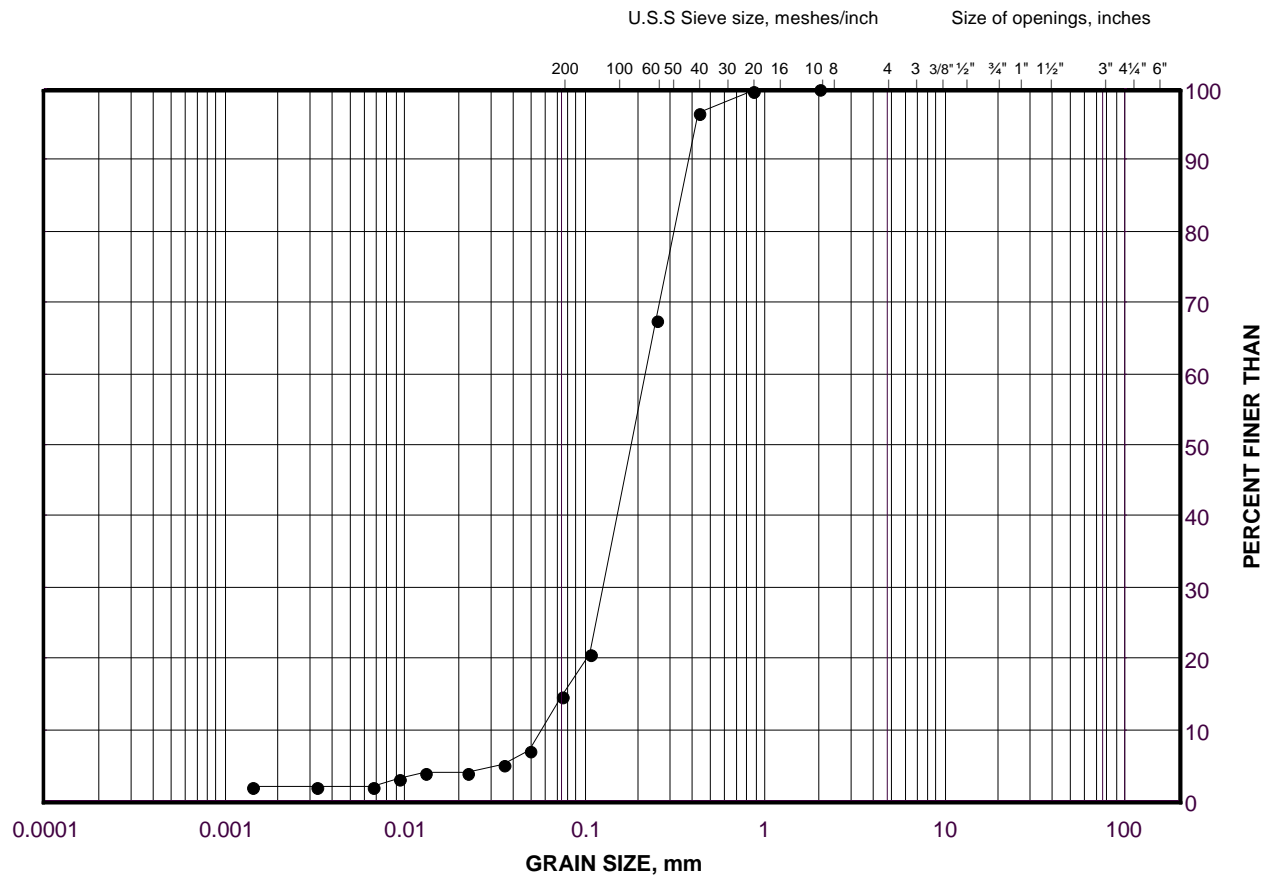
**Golder Associates**

Date: 08-Mar-12

# GRAIN SIZE DISTRIBUTION

Sand

FIGURE B6



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

## LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
•	5	6	171.6

Project Number: 11-1111-0077

Checked By: KJB

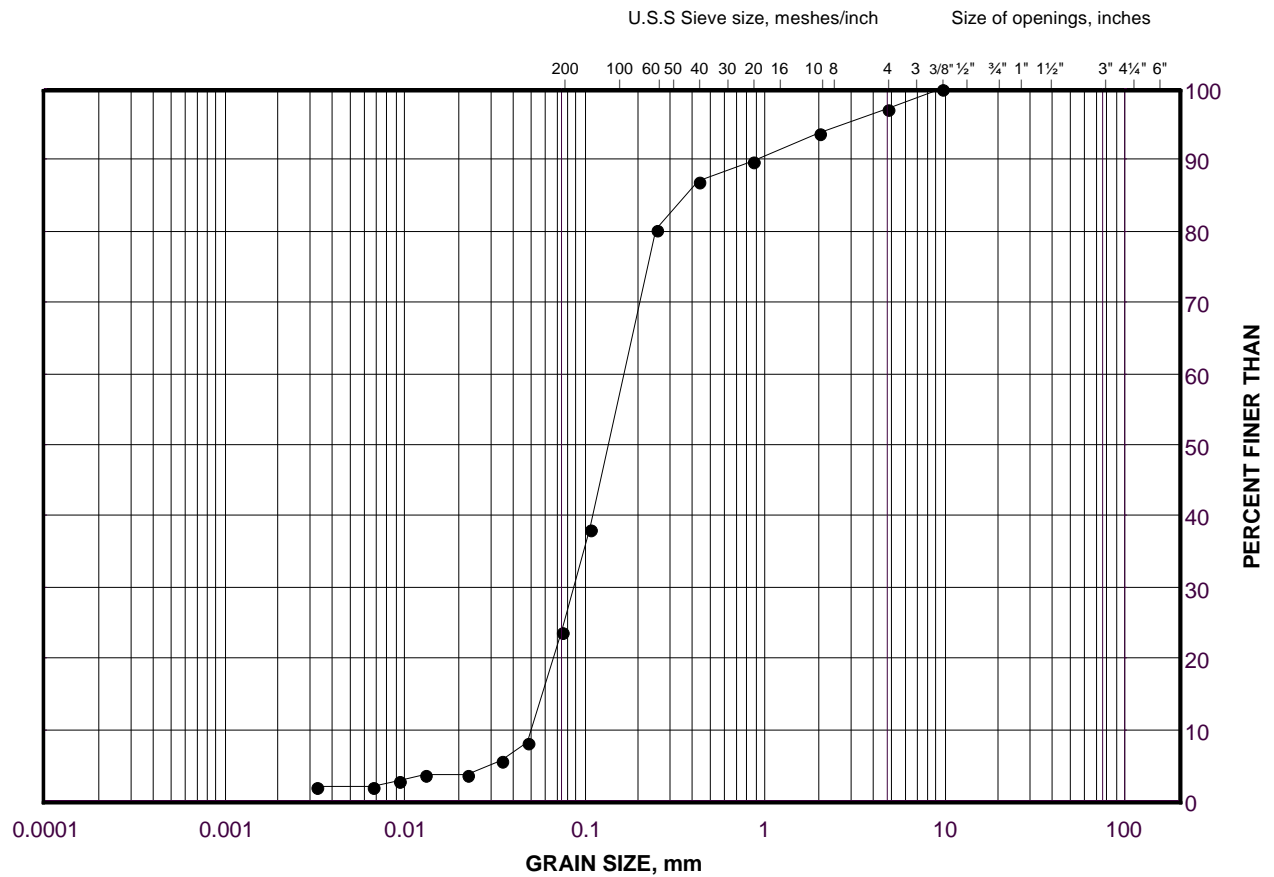
**Golder Associates**

Date: 08-Mar-12

# GRAIN SIZE DISTRIBUTION

Silty Sand (Lower)

FIGURE B7



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

## LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
•	4	12	160.9

Project Number: 11-1111-0077

Checked By: KJB

**Golder Associates**

Date: 08-Mar-12



# **APPENDIX C**

## **Non-Standard Special Provisions**



### WORKING SLAB - Item No.

---

Special Provision

---

#### 1.0 SCOPE

This Special Provision covers the requirements for the supply and placement of a concrete working slab under structure foundations.

#### 2.0 REFERENCES

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

##### **Ontario Provincial Standard Specifications, Construction**

OPSS 902      Excavating and Backfilling - Structures

#### 3.0 DEFINITIONS - Not Used

#### 4.0 DESIGN AND SUBMISSION REQUIREMENTS - Not Used

#### 5.0 MATERIALS

Concrete for working slabs shall have a minimum 28 day strength of 20 MPa.

#### 6.0 EQUIPMENT - Not Used

#### 7.0 CONSTRUCTION

##### 7.01              Excavation

Excavation for the working slab shall be according to OPSS 902.

##### 7.02              Protection of Founding Soil

Following inspection and approval of the prepared subgrade, a working slab with a minimum thickness of 100 mm shall be placed on the foundation subgrade as specified in the Contract Documents.



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## **FOUNDATION REPORT - STURGEON RIVER BRIDGE**

---

### **7.03 Protection of Founding Bedrock**

The surface of the footing founding rock shall be exposed, cleaned and any loose or fractured parts removed so that sound rock is exposed. The working slab shall be placed on the exposed cleaned sound founding rock surface as specified in the Contract Documents.

Thickness of the mass concrete pad shall depend on the slope and irregularities in the exposed founding rock surface. A nominal thickness and a footprint plan view area has been specified on the Contract Documents

### **7.04 Dewatering**

Dewatering shall be carried out according to OPSS 902.

### **8.0 QUALITY ASSURANCE - Not Used**

### **9.0 MEASUREMENT FOR PAYMENT - NOT USED**

### **10.0 BASIS OF PAYMENT**

#### **10.01 Working Slab - Item**

Payment at the Contract price for the above tender item shall be full compensation for all labour, Equipment and Material to do the work.

**END OF SECTION**



---

## FOUNDATION REPORT - STURGEON RIVER BRIDGE

---

### OBSTRUCTIONS - Item No.

---

Non-Standard Special Provision

---

The existing embankment fill at the Sturgeon River Bridge site contains cobbles (and possibly boulders) as indicated in the Record of Borehole sheets. Consideration of the presence of these obstructions must be made in the selection of appropriate equipment and procedures for sub-excavation for pile caps and pre-augering for deep foundations.

### BASIS OF PAYMENT

Payment at the lump sum contract price for this tender item shall be full compensation for all labour, equipment and materials for completion of the work.

### END OF SECTION



### CSP FOR INTEGRAL ABUTMENTS – Item No

Special Provision

#### ***SCOPE***

This specification covers the requirements for the installation of the corrugated steel pipes (CSPs) at the integral abutments.

#### ***SUBMISSION AND DESIGN REQUIREMENTS***

All submissions shall bear the seal and signature of an Engineer.

At least two weeks prior to commencement of installation of the abutment piles, the Contractor shall submit to the Contract Administrator, for information purposes only, three (3) sets of the working drawings.

The Contractor shall have a copy of the submitted working drawings on site at all times. Working drawings shall include at least the following:

1. Layout and elevations of the CSPs;
2. Location of reference points, and location of the centroid of each pile with respect to the reference points;
3. Construction sequence and details;
4. Source of the sand fill, and description of placing methods and equipment;
5. Location and details of all temporary bracing and spacers for the piles and CSPs;
6. Method for preventing water and debris from entering the CSP prior to placing sand; and
7. Method for preventing concrete from abutment pours from entering the CSPs during placement.

The Contractor shall be responsible for the complete detailed design of all temporary bracing, including spacers required to maintain the piles, CSP spacing and abutment stems in their specified positions through all stages of construction until the CSPs have been backfilled. All temporary bracing shall be removed.

#### ***MATERIAL***

##### ***Corrugated Steel Pipe***

CSP shall be in accordance with OPSS 1801, and shall be from a supplier listed under DSM#4.60.80. The CSP shall be of the diameter and wall thickness specified on the Contract drawings, and shall be galvanized in accordance with CSA G164-M.

CSPs shall be supplied in the lengths and with the end treatments, either square or skew, as specified on the Contract drawings; field cutting and splicing of CSPs will not be permitted. Cut ends shall be neat and free of burrs. The planes defined by the end treatments of each CSP shall be parallel to each other.

Handling and storage of CSPs shall be in accordance with the manufacturer's recommendations. Damaged CSPs shall be rejected. Localized areas of damaged galvanizing on otherwise acceptable CSPs shall be repaired with two coats of zinc-rich paint.





## FOUNDATION REPORT - STURGEON RIVER BRIDGE

### *Sand Fill*

The sand fill for backfilling the CSP shall meet the gradation requirements of Table 1 below:

**Table 1 – Sand Fill Gradation Requirements**

MTO Sieve Designation		Percentage Passing by Mass
2 mm	#10	100%
600 $\mu\text{m}$	#30	80% to 100%
425 $\mu\text{m}$	#40	40% to 80%
250 $\mu\text{m}$	#60	5% to 25%
150 $\mu\text{m}$	#100	0% to 6%

### **CONSTRUCTION**

The sequence of construction shall be in accordance with the working drawings and as follows, unless otherwise approved:

1. Construct levelling pad and place CSPs and spacers.
2. Install piles by driving to design criteria.
3. Place loose sand into 600 diameter CSP.
4. Remove temporary spacers.

The CSP shall be positioned such that the piles are centrally positioned within the CSP. Temporary blocking and bracing shall be used to hold the CSP in position.

The Contractor shall ensure the full perimeters of the tops of all CSPs at each abutment are at the elevation and orientation shown on the working drawings.

The CSP at each pile shall be constructed to the following tolerances:

Criteria	Tolerance
Maximum deviation of CSP from pile centroid	+/- 50 mm
Maximum deviation of any point on the top perimeter of the CSP from the specified elevation	+/- 10 mm



## FOUNDATION REPORT - STURGEON RIVER BRIDGE

---

The sand fill shall be placed dry of optimum and free-flowing, completely filling the volume between the CSP and pile. No additional compaction effort other than the action of placing the sand itself shall be applied to the sand fill.

The placing of the sand fill shall be carried out in a manner such as to not damage and displace the CSP.

### ***BASIS OF PAYMENT***

Payment at the contract price for the above tender item shall include all labour, equipment and material required to do the work.

**END OF SECTION**

At Golder Associates we strive to be the most respected global group of companies specializing in ground engineering and environmental services. Employee owned since our formation in 1960, we have created a unique culture with pride in ownership, resulting in long-term organizational stability. Golder professionals take the time to build an understanding of client needs and of the specific environments in which they operate. We continue to expand our technical capabilities and have experienced steady growth with employees now operating from offices located throughout Africa, Asia, Australasia, Europe, North America and South America.

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Australasia	+ 61 3 8862 3500
Europe	+ 356 21 42 30 20
North America	+ 1 800 275 3281
South America	+ 55 21 3095 9500

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[www.golder.com](http://www.golder.com)

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