



January 30, 2013

## FOUNDATION INVESTIGATION AND DESIGN REPORT

NBL AND SBL UNGULATE CULVERTS AT STA 11+198, SITE 44-594/C1-C2  
HIGHWAY 69 FOUR-LANING FROM 0.4 KM NORTH OF HIGHWAY 7182  
(SHEBESHEKONG ROAD) NORTHERLY 11 KM  
MINISTRY OF TRANSPORTATION, ONTARIO  
GWP 5403-05-00, WP 5133-12-20 & -21

**Submitted to:**

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REPORT

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# **PART A**

**FOUNDATION INVESTIGATION REPORT**  
**NBL AND SBL UNGULATE CULVERTS AT STA 11+198**  
**HIGHWAY 69 FOUR-LANING FROM 0.4 KM**  
**NORTH OF HIGHWAY 7182 (SHEBESHEKONG ROAD)**  
**NORTHERLY 11 KM**  
**MINISTRY OF TRANSPORTATION, ONTARIO**  
**GWP 5403-05-00**



## **1.0 INTRODUCTION**

Golder Associates Ltd. (Golder) has been retained by MMM Group (MMM) on behalf of Ministry of Transportation, Ontario (MTO) to provide foundation engineering services for the proposed Highway 69 Northbound Lane (NBL) and Southbound Lane (SBL) Ungulate Culverts. This project is part of the detail design for the four-laning of Highway 69 from 0.4 km north of Highway 7182 (Shebeshekong Road) northerly for 11 km. The general location of this section of the Highway 69 four-laning alignment is shown on the Key Plan on Drawing 1, following the text of this report.

The Terms of Reference and Scope of Work for the foundation investigation are outlined in MTO's Request for Proposal (RFP), dated March 28, 2007, and Change Request 9, dated December 19, 2011. The work was carried out in accordance with Golder's Supplemental Specialty Quality Control Plan for this project dated September 2007. The General Arrangement (GA) Drawing for the proposed Ungulate Culverts was provided to Golder by MMM.

This report addresses the investigation carried out for the Highway 69 NBL and SBL Ungulate Culverts and associated Retained Soil System (RSS) walls at Station 11+198 in Phase 1 of the project limits. Separate reports have been submitted detailing the foundation investigations for other culverts, bridges and embankments crossing swamps for the project.

The purpose of this investigation is to establish the subsurface conditions at the proposed culverts and RSS walls, by borehole drilling, rock coring and laboratory testing on selected soil and bedrock core samples. The investigated areas are shown on Drawing 1, following the text of this report. The details of the culverts are presented in Table 1.

## **2.0 SITE DESCRIPTION**

The proposed Highway 69 NBL and SBL Ungulate Culverts are located in the Township of Harrison, about 3 km south of South Shore Road. The NBL culvert will extend across the proposed NBL embankment, which is generally within, or in proximity to, the existing Highway 69 roadway embankment in this area. The proposed NBL embankment will be up to about 8.0 m high above existing grade at the west toe of the embankment, and about 4.5 m high above the existing embankment roadway grade. The culvert will extend across the proposed SBL embankment, which will be up to about 9.0 m high above existing grade.

In general, the topography in the area of the overall project limits consists of rolling terrain including densely treed areas and numerous bedrock outcrops separated by low-lying swamps. The topography in the immediate NBL culvert area is comprised of a generally flat and low-lying swamp on the west side of the existing embankment and exposed bedrock on the east side of the existing embankment. The proposed SBL culvert is located within a generally flat and low-lying swamp.



### **3.0 INVESTIGATION PROCEDURES**

The fieldwork for the investigation associated with the two (2) 7 m wide by 5 m high ungulate crossing culverts was carried out between February 21 and March 6, 2012, during which time a total of three (3) boreholes (LG-1 to LG-3) and two (2) dynamic cone penetration tests (DCPTs) (LG-DC1 to LG-DC2) were advanced along the alignment of the culverts. In addition, pertinent boreholes and DCPTs from the field investigation carried out by Golder for the Phase 1 Swamp and Pond Crossings were utilized to supplement this current investigation. The locations of the boreholes and details of the proposed culverts are summarized in Table 1, and the locations of the boreholes and culverts are shown on Drawing 1 (attached).

The field investigation was carried out using a truck- and a track-mounted CME 55 and portable equipment, supplied and operated by Landcore Drilling of Sudbury, Ontario. The boreholes were advanced through the overburden using 108 mm inside diameter (I.D.) hollow stem augers or NW casing with wash boring. Soil samples were obtained continuously or at intervals of depths of about 0.75 m and 1.5 m, using a 50 mm outer diameter (O.D.) split-spoon sampler (operated by automatic hammers, or 1/3 weight hammers on the portable equipment swamp boreholes), performed in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586). Field vane shear tests were carried out in cohesive soils for determination of undrained shear strength (ASTM D2573). Samples of bedrock were obtained in Borehole LG-3 using an 'NQ' size rock core barrel. All boreholes were backfilled with bentonite upon completion in accordance with Ontario Reg. 903 (as amended).

The boreholes drilled during the previous and current investigation at the culvert sites were advanced to depths up to 12.3 m below existing ground surface, generally penetrating 3 m into competent material, which is defined as material that will provide resistance to settlement or instability of the embankments, or to refusal. Most boreholes were terminated on refusal to further auger, casing and/or split-spoon advancement and bedrock coring was carried out at one borehole location. The depths to refusal in boreholes and in the DCPTs where bedrock was not cored do not confirm bedrock surface elevations, but may be inferred to indicate potential proximity to the bedrock surface.

The groundwater conditions and water levels in the open boreholes were observed during the drilling operations and are described on the Record of Borehole sheets (Appendix A). It should be noted that groundwater elevations, as encountered in the boreholes, may not be representative of static groundwater levels since the groundwater levels may not have stabilized on completion of drilling. Furthermore, groundwater elevations will vary depending on seasonal fluctuations, precipitation and local soil permeability.

The fieldwork was monitored by members of our engineering and technical staff, who located the boreholes, arranged for the clearance of underground services, observed the drilling, sampling and in situ testing operations, logged the boreholes, and examined and cared for the soil and rock core samples. The samples were identified in the field, placed in appropriate containers, labelled and transported to our Sudbury geotechnical laboratory, where the samples underwent further visual examination and laboratory testing. All of the laboratory tests were carried out to MTO and/or ASTM Standards, as appropriate. Classification testing (water content, Atterberg limits and grain size distribution) was carried out on selected representative soil samples. Strength testing (uniaxial compression) was also carried out on one selected specimen of the bedrock core.



The centreline of the highway was surveyed and staked in the field by MMM prior to drilling, and wooden stakes were also installed near the northeast and northwest corners of the NBL culvert for reference. The as-drilled borehole locations and ground surface elevations were measured/surveyed by members of our technical staff, referenced to the survey stakes. The borehole locations, presented on the Record of Borehole sheets and shown on Drawing 1, are positioned relative to MTM NAD 83 northing and easting coordinates and the ground surface elevations are referenced to Geodetic datum. The as-drilled borehole locations, ground surface elevations and drilled depths for all the boreholes and DCPTs associated with this component of the culvert investigation in Phase 1 are summarized below.

Borehole/ DCPT	Location (m)		Ground Surface Elevation (m)	Drilled Depth (m)
	Northing	Easting		
LG-1	5048584.9	238900.8	198.1	11.4
LG-2	5048570.3	238892.2	194.5	9.0
LG-3	5048594.9	238906.8	198.1	5.9
S6-2	5048541.8	238862.7	194.3	5.9
S6-3	5048553.4	238884.1	194.3	12.3
S6-4	5048560.9	238903.1	194.5	10.6
LG-DC1	5048606.7	238905.6	196.7	0.7
LG-DC2	5048599.5	238917.6	197.6	1.1
S6-DC1	5048573.7	238881.6	194.7	3.7
S6-DC2	5048529.1	238884.3	194.3	3.5

## 4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

### 4.1 Regional Geology

As delineated in The Physiography of Southern Ontario (Chapman and Putnam, 1984)<sup>1</sup>, this section of Highway 69 lies within the physiographic region known as the Georgian Bay Fringe which extends along the east side of Georgian Bay through the Parry Sound and Muskoka areas, then eastward from Muskoka in patches into the area north of the Kawartha Lakes.

This part of the Georgian Bay Fringe physiographic region was never submerged during periods of glacial recession. As a result, the surficial soils in this area consist of very shallow deposits of sand, silt and clay overlying metamorphic bedrock and numerous bare knobs and ridges of bedrock are present throughout the area. Localized low-lying swampy areas, containing peat and/or organic soils overlying soft/loose native soils, are present in valleys between the bedrock knobs and ridges.

The bedrock in the area consists typically of gneisses of the Britt Domain of the Central Gneiss Belt, a subdivision of the Grenville Structural Province, as described in Geology of Ontario (OGS, 1991)<sup>2</sup>. Deposition of Paleozoic strata initially covered the bedrock and later erosion during glaciation exposed these Precambrian rocks.

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

<sup>2</sup> Ontario Geological Society, 1991. *Geology of Ontario Volume 4, Part 2*. Ministry of Northern Development and Mines, Ontario.





## **4.2 Subsurface Conditions**

The detailed subsurface soil and groundwater conditions, as encountered in the boreholes advanced during this investigation, together with the results of the laboratory tests carried out on selected soil and bedrock samples, are presented on the Record of Borehole and Drillhole sheets in Appendix A. The results of the laboratory tests carried out on selected soil samples are presented in Appendix B. The stratigraphic boundaries shown on the Record of Borehole sheets are inferred from non-continuous sampling and observations of drilling progress and the results of SPT measurements. These boundaries, therefore, represent transitions between soil types rather than exact planes of geological change. It should be noted that the interpreted stratigraphy shown on Drawing 1 is a simplification of the subsurface conditions. Variation in the stratigraphic boundaries between and beyond the boreholes will exist and is to be expected.

In general, the subsoils in the area of the ungulate crossing culvert consist of embankment fill or peat underlain by cohesive deposits of silty clay to clay and/or cohesionless deposits of silty sand to sand and gravel. The total thickness of overburden is variable at the site, ranging from about 12.3 m at the midpoint of the SBL culvert to no overburden at the east side of the NBL culvert where bedrock is exposed.

A detailed description of the subsurface conditions at each investigated culvert alignment is provided in the following sections of this report.

### **4.2.1 Highway 69 NBL – STA 11+198**

The plan and profile along the culvert centreline showing the borehole locations and interpreted stratigraphy at approximately STA 11+198 in Harrison Township are shown on Drawing 1. A total of three boreholes and two DCPTs were completed to investigate the subsurface conditions at this culvert location. Two boreholes (Boreholes LG-2 and LG-3) were advanced near the ends of the culvert and one borehole (Borehole LG-1) was advanced near the midpoint of the culvert. In addition, two DCPTs (LG-DC1 and LG-DC2) were advanced near the footprint of the east head walls to confirm the depth to refusal at these locations.

#### ***Fill***

Boreholes LG-1 and LG-3 were advanced through the existing Highway 69 embankment and penetrated a layer of asphalt about 65 mm and 135 mm thick respectively, underlain by a deposit of granular fill comprised of sand and gravel to sand, with cobbles inferred from the split-spoon sampling operations. The top of the embankment (surface of the asphalt) is at Elevation 198.1 m and the thickness of the fill in Boreholes LG-1 and LG-3 is 4.3 m and 1.8 m respectively.

SPT 'N'-values recorded within the granular fill range between 1 blow and 23 blows per 0.3 m of penetration, indicating a very loose to compact relative density. A SPT 'N'-value of 50 blows per 0.1 m of penetration was noted, indicating the presence of cobbles.

The grain size distribution of one sample of the sand fill is shown on Figure B1.

The natural water content measured on a sample of the fill is about 9 per cent.



### ***Peat***

A deposit of black fibrous and/or amorphous peat was encountered at ground surface in Borehole LG-2 at Elevation 194.5 m and the thickness of the deposit is 3.2 m.

SPT 'N'-values recorded within this deposit are 0 blows (weight of hammer/rods) per 0.3 m of penetration, suggesting a very soft consistency.

The natural water content measured on a sample of this deposit is about 134 per cent.

### ***Silty Clay to Clay***

A deposit of grey silty clay to clay was encountered underlying the embankment fill in Borehole LG-1 and the peat in Borehole LG-2. The top of the deposit was encountered at 4.4 m and 3.2 m below ground surface, corresponding to Elevation 193.7 m and 191.3 m, and the thickness of the cohesive deposit is 1.2 m and 2.5 m in the respective boreholes.

The SPT 'N'-values measured within this deposit are 0 blows (weight of hammer/rods) per 0.3 m of penetration, suggesting a very soft consistency.

Atterberg limits testing was carried out on two samples of the silty clay to clay and the test results indicate liquid limits of about 36 per cent and 55 per cent, plastic limits of about 17 per cent and 23 per cent and plasticity indices of about 19 per cent and 32 per cent. The results of the Atterberg limits tests are shown on the plasticity chart on Figure B2 and indicate that the material is classified as a silty clay of medium plasticity to a clay of high plasticity.

Measured water content on two samples of this deposit are 44 per cent and 51 per cent.

### ***Silty Sand to Sand and Gravel***

A deposit of brown to grey silty sand to sand and gravel, containing trace clay was encountered underlying the clay in Boreholes LG-1 and LG-2 and underlying the fill in Borehole LG-3. Cobbles were noted at depths within the deposit in Boreholes LG-1 and LG-2, as indicated on the borehole records. The top of this deposit was encountered between 1.9 m and 5.7 m below ground surface, ranging between Elevation 196.2 m and 188.8 m and the thickness of the deposit in Borehole LG-3 is 1.0 m. The deposit was not likely fully penetrated in Boreholes LG-1 and LG-2 after drilling 3.3 m and 5.8 m, respectively, into the deposit.

SPT 'N'-values recorded within this deposit range between 4 blows and 64 blows per 0.3 m of penetration, indicating a very loose to very dense relative density. SPT 'N'-values of 50 blows and 87 blows per 0.1 m of penetration were also measured near the bottom of the deposit in Borehole LG-2, likely as a result of cobbles present within the deposit.

The grain size distributions of two samples of this deposit are shown on Figure B3.

The natural water content measured on samples of this deposit range between about 9 per cent and 15 per cent.



### ***Bedrock/Refusal***

Bedrock was encountered underlying the sand in Borehole LG-3 at a depth of 2.9 m below ground surface, corresponding to Elevation 195.2 m, and 3.0 m of bedrock core was recovered. Based on a review of the bedrock core sample, the bedrock consists of fine to coarse grained, fresh, pink and grey gneiss.

The Total Core Recovery (TCR) for the core samples in Borehole LG-3 is 100 per cent and the Solid Core Recovery (SCR) ranges from about 7 per cent to about 87 per cent due to the presence of vertical joints encountered between 3.2 m and 5.0 m depths. The RQD measured on the recovered bedrock core sample ranges from 94 per cent to 100 per cent, indicating that the rock is of excellent quality according to Table 3.10 of the Canadian Foundation Engineering Manual (CFEM, 2006)<sup>3</sup>.

A Uniaxial Compressive Strength (UCS) test was carried out on a representative sample of the rock core taken from Borehole LG-3 and the measured UCS is 67 MPa, indicating that the bedrock is strong ( $R_4$ ,  $50 \text{ MPa} < \text{UCS} < 100 \text{ MPa}$ ) according to Table 3.5 of CFEM (2006)<sup>3</sup>. Visual examination of the broken core sample after the UCS test revealed the presence of a healed joint within the tested sample.

In Borehole LG-2, refusal to further split-spoon penetration was encountered at a depth of 9.0 m below ground surface (Elevation 185.5 m). In DCPTs LG-DC1 and LG-DC2 refusal to further penetration was encountered at depths of 0.7 m and 1.1 m below ground surface, corresponding to Elevation 196.0 m and 196.5 m at the respective boreholes.

### ***Groundwater Conditions***

The unstabilized water level in Boreholes LG-1 to LG-3 was measured at depths between 0.2 m and 1.8 m below ground surface, corresponding to between Elevations 194.3 m and 196.9 m. Groundwater/surface water levels in the area are subject to seasonal fluctuations and variations due to precipitation events.

## **4.2.2 Highway 69 SBL – STA 11+198**

The plan and profile along the culvert centreline showing the borehole locations and interpreted stratigraphy at approximately STA 11+198 in Harrison Township are shown on Drawing 1. The culvert will extend across the proposed SBL embankment, which will be up to about 9.0 m above existing grade. One borehole was completed to investigate the subsurface conditions at this culvert location, supplemented with three swamp boreholes. One borehole (Borehole LG-2) was advanced near the end of the culvert and one borehole (Borehole S6-3) was advanced near the midpoint of the culvert. In addition, one borehole (S6-4) and one DCPT (S6-DC1) were advanced near the ends of the east wing walls and one borehole (S6-2) and one DCPT (S6-DC2) were advanced near the end of the west wing walls. The topography of this section of proposed highway is a generally flat and low-lying swamp.

<sup>3</sup> Canadian Geotechnical Society, 2006. Canadian Foundation Engineering Manual, 9<sup>th</sup> Edition. The Canadian Geotechnical Society ??BiTech Publisher Ltd., British Columbia



### **Peat**

A deposit of black fibrous and amorphous peat was encountered at ground surface in Boreholes LG-2 and S6-2, to S6-4. The top of the deposit was encountered between Elevation 194.5 m and 194.3 m and the thickness of the peat deposit ranges from 2.9 m to 3.7 m.

The SPT 'N'-values recorded within this deposit are between 0 blows (weight of hammer/rods) and 2 blows per 0.3 m of penetration, suggesting a very soft consistency.

Measured water content on samples of this deposit range between about 115 per cent and 626 per cent.

### **Silty Clay to Clay**

A deposit of grey silty clay to clay was encountered underlying the peat in Boreholes LG-2 and S6-2 to S6-4. The overall thickness of the cohesive deposit ranges from 1.4 m to 4.0 m and the top of the deposit was encountered between 2.9 m and 3.7 m below ground surface, ranging between Elevation 191.4 m and 190.6 m. In Boreholes S6-3 and S6-4 the silty clay to clay deposit is inter-layered with a 0.8 m and 0.3 m thick layer of grey sand to silty sand at a depth of 4.4 m (Elevation 189.9 m) and 5.5 m (Elevation 189.0 m), respectively.

The SPT 'N'-values measured within this cohesive deposit range are 0 blows (weight of hammer/rods) and 1 blow per 0.3 m of penetration. In situ field vane testing carried out within this stratum measured undrained shear strengths ranging from about 17 kPa to 29 kPa. The SPT 'N'-values together with the in situ field vane tests suggest the cohesive deposit has a very soft to firm consistency. An SPT 'N'-value of 1 blow per 0.3 m of penetration was measured within the sand seam in Borehole S6-3, indicating a very loose relative density.

Atterberg limits testing was carried out on four samples of the silty clay to clay and the test results indicate liquid limits ranging from about 36 per cent to 57 per cent, plastic limits ranging from about 17 per cent to 24 per cent and plasticity indices ranging from about 17 per cent to 32 per cent. The results of the Atterberg limits tests are shown on the plasticity chart on Figure B2 and indicate that the material is classified as a silty clay of medium plasticity to a clay of high plasticity.

The grain size distribution of one sample of the silty clay portion of the deposit is shown on Figure B4. The grain size distribution of two samples of the sand seam are shown on Figure B3.

Measured water content on samples of this deposit range between about 44 per cent and 87 per cent.

### **Silty Sand to Sand and Gravel**

A deposit of brown to grey silty sand to sand and gravel containing trace clay was encountered below the silty clay to clay deposit in Boreholes LG-2 and S6-2 to S6-4. The top of this deposit was encountered between about 4.3 m and 7.7 m below ground surface, between about Elevation 190.0 m and 186.6 m. The deposit is between 1.6 m and 4.0 m thick in Boreholes LG-2, S6-2 and S6-4 where penetrated to refusal to further split-spoon advancement, and was not fully penetrated in Boreholes S6-3 after drilling 4.6 m into the deposit.

The SPT 'N'-values measured within this deposit range between 3 blows and 37 blows per 0.3 m of penetration, indicating a very loose to very dense relative density. In Boreholes LG-2 and S6-2, SPT 'N'-values of 50 blows



and 87 blows per 0.1 m of penetration and 60 blows per 0.15 m of penetration were measured near the bottom of the boreholes indicating a very dense relative density, likely as a result of cobbles within the deposit.

The natural water content measured on samples of this deposit range between about 9 per cent and 15 per cent.

The grain size distributions of three samples of this deposit are shown on Figure B3.

An Atterberg limits test carried out on a sample of the gravelly sand deposit from Borehole S6-3 indicates non-plastic fines.

### ***Refusal***

In Borehole S6-2 and LG-2, refusal to further split-spoon penetration was encountered at a depth of about 5.9 m and 9.0 m below ground surface respectively, corresponding to Elevations 188.4 m and 185.5 m.

### ***Groundwater Conditions***

The unstabilized water level in Boreholes S6-2, S6-3, S6-4 and LG-2 was measured at ground surface to a depth of 0.2 m below ground surface, corresponding to Elevation 194.3 m and 194.5 m. Groundwater/surface water levels in the area are subject to seasonal fluctuations and variations due to precipitation events.

## **5.0 CLOSURE**

The field personnel supervising the drilling program were Mr. Ed Savard and Mr. Shane Albert. This report was prepared by Mr. Evan Childerhose, P.Eng. and Mr. André Bom, P.Eng. The technical aspects were reviewed by Mr. Jorge M. A. Costa, P.Eng., Golder's Designated MTO Contact for this project, who also carried out a quality control review of the report.

## Report Signature Page

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N:\Active\2007\1190 Sudbury\1191\07-1191-0020 MMM Hwy 69 Twinning\7000 Reporting\Final\Ungulate Culvert\07-1191-0020-UC FINAL RPT 13Jan30 FIR Ungulate Culvert.Docx



# **PART B**

## **FOUNDATION DESIGN REPORT**

**NBL AND SBL UNGULATE CULVERTS AT STA 11+198**

**HIGHWAY 69 FOUR-LANING FROM 0.4 KM**

**NORTH OF HIGHWAY 7182 (SHEBESHEKONG ROAD)**

**NORTHERLY 11 KM**

**MINISTRY OF TRANSPORTATION, ONTARIO**

**GWP 5403-05-00**



## **6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS**

This section of the report provides interpretation of the factual geotechnical data obtained during the investigation and recommendations on the foundation aspects of design of the proposed works. The recommendations provided are intended for the guidance of the design engineer. Where comments are made on construction, they are provided in order to highlight those aspects which could affect the design of the project. Those requiring information on the aspects of construction should make their own interpretation of the subsurface information provided as it affects their proposed construction methods, costs, equipment selection, scheduling and the like.

### **6.1 General**

The new Highway 69 NBL and SBL Ungulate Culverts will each be 7 m wide by 5 m high and 19.2 m long. The new NBL embankment at the proposed culvert location crosses the existing Highway 69 at a skew and the proposed SBL culvert is located in a swamp. RSS walls are proposed for each side of the two culverts at both ends of the culverts.

In general, the subsoils in the area of the culverts consist of embankment fill or peat underlain by cohesive deposits of silty clay to clay and/or cohesionless deposits of silty sand to sand and gravel. The total thickness of overburden is variable at the site, ranging from at least 12.3 m at the midpoint of the SBL culvert to about 2.9 m of overburden at the east end of the NBL culvert, thinning further to the east where bedrock is exposed.

During construction of the NBL and SBL embankments and culverts in this area, the traffic from the existing highway will be routed to the new Site 9 Road embankment west of the existing/proposed highway.

### **6.2 Culvert Type**

The analysis and recommendations presented in this report assume that the NBL and SBL Ungulate Culvert will consist of open footing cast-in-place concrete culverts. The culvert dimensions and invert elevations are summarized in Table 1. Due to the relatively large size of the proposed culverts for this site, pre-cast culverts are not considered feasible.

#### **6.2.1 Frost Protection**

The estimated frost penetration depth for the Pointe au Baril Station area is 1.8 m, as per OPSD 3090.101 (Foundation Frost Penetration Depths for Southern Ontario). Spread footings for an open footing concrete culvert should be provided with a minimum of 1.8 m of conventional soil cover for frost protection. For spread footings founded directly on the bedrock, frost susceptibility is not an issue.





## **6.3 Culvert Construction Options**

In general, the foundation strata at the culvert crossings will undergo settlement as a result of loading from new embankment construction and/or existing embankment grade raises or widenings. Therefore, the timing of culvert construction is an essential factor in determining the preferred settlement mitigation option. The following alternatives for culvert construction can be considered (where applicable, giving due consideration to the recommended foundation mitigation option for the adjacent embankment):

- concurrent with embankment construction;
- following the embankment construction preload period; or
- following full sub-excavation along the culvert alignment and concurrent with embankment construction.

Where relatively small settlements are estimated to occur as a result of culverts being constructed within existing highway embankments or due to relatively thin, compressible foundation strata at the culvert locations, culvert construction can commence immediately following excavation of the existing embankment fill concurrently with construction of the proposed new embankments so long as any requirements for maintaining embankment stability are addressed. If required, the culvert design could include a camber.

Where relatively large settlements are estimated to occur, it is recommended that the culverts be constructed subsequent to the embankment preload period or following full sub-excavation of soft compressible deposits to provide for adequate long-term performance of the culvert and the associated overlying and adjacent roadway. The following sections provide a more detailed discussion on the possible alternatives for culvert construction to mitigate settlements and improve long-term performance.

### **6.3.1 Culvert Construction Concurrent with Embankment Construction**

Culverts which are constructed concurrently with the new embankments will experience settlement (both short-term and long-term) as well as lateral spreading (or horizontal strain in the longitudinal direction) as a result of the embankment loading. The analyses of settlement and horizontal strain are discussed in Section 6.4.2 and Section 6.4.3, respectively. If the culvert is capable of tolerating the estimated total and differential settlements and associated strains, the culvert could be constructed with a camber (if necessary), such that once the settlement has occurred, the invert grade will be maintained as originally designed. However, culvert designs which include a camber may have a relatively high risk of poor performance, such as resulting in unfavourable drainage/surface flow conditions in the case of drainage culverts. It is important to note that it is inherently difficult to predict settlements for the variable subsurface conditions along the culvert alignments with such a degree of accuracy to allow a successful camber design. If the actual settlements are smaller than predicted, the culvert may not achieve the design grade or slope, whereas if actual settlements are larger than expected, the culvert may sag below the design invert elevation. Expansion joints should also be included along the length of the culvert to accommodate horizontal strain which will occur in conjunction with the vertical settlement. If the culvert cannot tolerate the estimated settlement and horizontal strain, consideration should be given to constructing the culvert following the preload period of the embankment (see Section 6.3.2) or following full sub-excavation of the compressible, cohesive deposits (see Section 6.3.3).



It should be noted that if this option is adopted as the preferred alternative for construction of any of the culverts, it is still necessary that all existing organic material be sub-excavated prior to placement of any fill material due to the highly compressible nature of organic soils which can undergo significant secondary (creep) settlement.

### **6.3.2 Culvert Construction Following Embankment Preload Period**

At locations where the magnitudes of estimated total and differential settlements and horizontal strains cannot be tolerated and/or where removal of localized clayey deposits and replacement with rock fill or granular fill is not considered practical, the culverts should be constructed after a preload period. Preloading refers to the placement of fill to the proposed height of the embankment (possibly in stages), in advance of construction of the permanent culvert, in order to consolidate the underlying compressible soils. If preloading of the embankment at the culvert location is completed prior to construction of the permanent culvert, the magnitude of total and differential settlement beneath the culvert and horizontal strain along the culvert will be reduced. However, this mitigation option requires excavation through the new embankment fill to the culvert founding elevation at the end of the preload period in order to construct the culvert. Provided that the final fill above the culvert is properly placed and compacted, the magnitude of differential settlement between the fill embankment (that has been compressed under its self-weight for the entire preload period) and the final backfill above the culvert should be acceptable.

In addition, it may be more practical to construct a temporary granular fill core within the embankment in the proximity of the culvert location to allow for ease of sub-excavation of the embankment fill in this area following the preload period. Details regarding the temporary granular fill core are provided in Section 6.9.1.

It should be noted that with preloading, it is still required that all existing organic material be sub-excavated prior to placement of any fill material due to the highly compressible nature of organic soils which undergo significant secondary (creep) settlement.

### **6.3.3 Culvert Construction Following Full Sub-Excavation**

Depending on the depth and thickness of the soft, compressible deposit(s), the magnitude of total and differential settlement and horizontal strain could also be reduced by means of full sub-excavation and replacement with rock fill or granular fill along the culvert alignment to allow for permanent culvert construction prior to embankment loading (i.e. concurrent with embankment construction). At culvert locations where the compressible deposits are thick, the resulting magnitude of settlements as well as the associated horizontal strains, even with full sub-excavation, may still be too large, as a result of compression of the underlying fill itself, to accommodate standard culvert construction. However, where there is a limited thickness of soft, compressible soils underlying the proposed culvert, full sub-excavation and replacement is a feasible option to reduce the settlement and allow for culvert construction in conjunction with the new embankment. The costs of full sub-excavation and backfilling would have to be assessed in the cost/benefit analysis when choosing the preferred mitigation option.

Although full sub-excavation will improve the settlement performance of the culverts and embankments in close proximity of the sub-excavation, adjacent areas of the embankment may not experience the same improvements



in settlement performance depending on the mitigation measures adopted for the adjacent embankment(s) crossing the swamp. As a result, the overlying embankment may experience some differential settlements depending on the timing of embankment construction/culvert construction, type of backfill and timing of final earthwork and paving.

It should also be noted that settlement of the replacement rock fill beneath the culvert base will occur, primarily during construction, and could constitute a significant portion of the expected settlements, depending on the depth of sub-excavation required.

Where full sub-excavation is adopted, the additional rock fill below the base of the culvert should be constructed with the same side slope profile as that of the above-grade embankment (i.e. 1.25H:1V) since this is the natural slope of rock fill and should not be affected by underwater placement. In addition, the necessity to develop stable side slopes and back slopes within the excavation may result in cut slope geometries ranging from 1H:1V to as flat as 3H:1V, especially where excavations are carried out 'in-the-dry'. Recommendations with respect to excavations and unwatering are given in Section 6.8. It should be noted that full sub-excavation at the culvert locations will produce extra volumes of spoil material requiring disposal.

## **6.4 Stability, Settlement and Horizontal Strain**

The following sections summarize the methods utilized to carry out analyses of stability and settlement of the culverts and methods utilized to evaluate horizontal strains along the culverts beneath the zone of influence of the proposed embankment loading.

### **6.4.1 Stability**

The methodology used to evaluate embankment stability at the various culvert locations is described below. In addition, the parameters used in the analyses for each culvert location are also presented. The results of the analyses for each culvert location are discussed in Section 6.6.

#### **6.4.1.1 Methodology**

Embankment stability analyses were carried out at each culvert location. In all areas where cohesive deposits were encountered in the subsoils, the stability of the proposed new embankment section(s) was analyzed using limit equilibrium methods. In areas where the subsoils consist of cohesionless soils and/or rock fill only, the stability of the proposed embankment section was assessed based on precedent experience in similar soil conditions.

All limit equilibrium slope stability analyses were carried out using the commercially available program Geostudio 2007 (Version 7.19), produced by Geo-Slope International Ltd., employing the Morgenstern-Price method of analysis. For all analyses, the factor of safety of numerous potential failure surfaces was computed in order to establish the minimum Factor of Safety. The Factor of Safety is defined as the ratio of the forces tending to resist failure to the driving forces tending to cause failure. A target minimum Factor of Safety of 1.3 is normally adopted for the design of embankment slopes under static conditions. This Factor of Safety is considered



adequate for the embankments at these sites considering the design requirements and the field data available and is based on deep-seated, global failure surfaces that would affect the operation of the roadway. The stability analyses were carried out to check that the target minimum Factor of Safety was achieved for the various embankment heights and geometries at the culvert locations.

The stability analyses assume that all organic soils beneath the culvert alignment will be removed prior to construction and that rock fill will be used for replacement of sub-excavated material (as discussed in Section 6.8.1.1). The piezometric conditions required in the analyses were based on the groundwater levels observed during drilling which were generally located at about the level of the natural ground surface.

#### 6.4.1.2 Parameter Selection

The simplified stratigraphy together with the associated strength and unit weight values assigned to the different native soil types at the culvert locations are summarized in Table 2. The rock fill modeled in the analyses is assumed to have a unit weight of 19 kN/m<sup>3</sup> (the analysis was also carried out using a unit weight of 18 kN/m<sup>3</sup>) and an effective friction angle of 40° and the embankments constructed with 1.25 Horizontal to 1 Vertical (1.25H:1V) side slopes.

The subsoils encountered in the various culvert areas are comprised of granular soils (silty sand to sand and gravel) or cohesive soil (silty clay to clay). For granular soils, effective stress parameters were employed in the analyses assuming drained conditions. The effective stress parameters (effective friction angle and effective cohesion) for the granular soils were estimated from empirical correlations using the results of the in situ Standard Penetration Tests (SPT), in conjunction with engineering judgement based on experience in similar soil conditions.

For cohesive deposits, total stress parameters were employed in the analyses assuming undrained conditions. The total stress parameters (i.e. average mobilized undrained shear strength –  $s_u$ ) for the cohesive soils were assessed based primarily on the results of in situ field vane shear tests and also inferred from the laboratory consolidation tests results (where available), and estimated from correlations with the SPT results and other laboratory test data (natural water content). From the consolidation tests completed on samples of the cohesive deposit in Swamp 6, the following correlation proposed by Mesri (1975) was employed to estimate the undrained shear strength:

$$s_u = 0.22\sigma_p'$$

where:

$s_u$	=	average mobilized undrained shear strength (kPa)
$\sigma_p'$	=	preconsolidation stress (kPa)

Where appropriate, Bjerrum's correction factor was employed to estimate the average mobilized undrained shear strength from the results of the in situ field vane tests as follows:

$$s_{u(mob)} = \mu s_{u(FV)} \quad (\text{after Bjerrum, 1973})$$

where:

$s_{u(mob)}$	=	average mobilized undrained shear strength (kPa)
$s_{u(FV)}$	=	undrained shear strength from field vane test (kPa)
$\mu$	=	Bjerrum's correction factor based on Plasticity Index



When developing the culvert area-specific correlations of engineering parameters based on laboratory or field test data, the results from the two culvert crossings were correlated with the adjacent swamp crossings. It is considered that the culvert and swamp crossings in this area exhibit sufficiently similar soil mineralogy and geology that correlations based on all of the data are justified. Having developed the area-specific correlations, the test results for each individual culvert area were examined and the design lines developed accordingly.

## **6.4.2 Settlement**

The following sections outline the methods used to carry out the settlement analyses at the various culvert locations. The results of the analyses for each culvert location are discussed in Section 6.6.

### **6.4.2.1 Methodology**

To estimate the magnitude of the expected settlements, analyses were carried out along the individual culvert alignments using the commercially available program Settle3D (Version 2.0) produced by Rocscience Inc. and/or hand/spreadsheet calculations. The rate of settlement/consolidation of the cohesive foundation soils was assessed using Terzaghi's one-dimensional consolidation theory.

The sources of settlement were considered to include:

- primary time-dependent consolidation of the cohesive deposits;
- secondary time-dependent (creep) consolidation of the cohesive deposits (long-term);
- immediate settlement of the native granular soils; and
- self-weight compression of the embankment fill materials beneath the culvert.

The thickness of the compressible foundation soils and the height of the embankment vary along the proposed culvert crossings and as such the settlements along the length of a given culvert will similarly vary. As such, settlements have been assessed at the inlet, mid-point, and outlet of each culvert alignment.

The settlement analyses assume that all organic soils beneath the culvert alignment will be removed prior to construction and that rock fill will be used for replacement of sub-excavated material (as discussed in Section 6.8.1.1). The piezometric conditions required in the analyses are based on the groundwater levels observed during drilling and are generally located at about the level of the natural ground surface.

### **6.4.2.2 Parameter Selection**

The simplified stratigraphy together with the associated deformation and time-rate consolidation parameters employed for the different native soil types at the culvert locations in the swamp crossing are given in Table 2.

The immediate compression of the silty sand to sand and gravel layers was modeled by estimating an elastic modulus of deformation based on the SPT 'N'-values and using correlations proposed by Bowles (1984) and Kulhawy and Mayne (1990). These estimated values were compared with the typical range of expected values



for similar soil types, as outlined in *Canadian Highway Bridge Design Code, CHBDC* (2006) and adjusted, if necessary.

The consolidation settlement of the cohesive deposits was assessed using the results of the laboratory consolidation tests and in situ field vane tests to estimate the stress history and deformation parameters for the cohesive deposits. In addition, the results of the consolidation tests in the adjacent swamp were supplemented with estimates of deformation parameters (i.e. recompression and compression indices) using empirical correlations proposed in literature by Koppula (1986), Terzaghi and Peck (1967), Kulhawy and Mayne (1990) and Azzouz et al. (1976).

The following correlation relating in situ undrained shear strength to preconsolidation stress proposed by Mesri (1975) was employed:

$$\sigma_p' = \frac{S_{u(mob)}}{0.22}$$

where :

$S_{u(mob)}$	=	$\mu S_{u(FV)}$	(after Bjerrum, 1973)
$\sigma_p'$	=	preconsolidation stress (kPa)	
$S_{u(mob)}$	=	average mobilized undrained shear strength (kPa)	
$S_{u(FV)}$	=	undrained shear strength from field vane test (kPa)	
$\mu$	=	Bjerrum's correction factor based on Plasticity Index	

The coefficient of consolidation,  $c_v$  (cm<sup>2</sup>/s), required in the time-rate analysis was established using the results of the consolidation tests (based on  $t_{90}$ ) and/or estimated from the U.S. Navy (1986) correlation with liquid limits assuming normally-consolidated soils.

In addition to primary consolidation within clays, secondary compression may also occur. Secondary compression is referred to as creep settlement and occurs over a long period of time, after full dissipation of excess pore pressure under a constant stress. The following relationships have been employed for estimating the magnitude of creep settlement over the life of the embankment following the completion of primary settlement at each location.

$$S_c = HC_{\alpha\epsilon} \log\left(\frac{t}{t_{EoP}}\right)$$

$$C_{\alpha\epsilon} \approx \frac{w_n}{10,000} \quad (\text{after Mesri, 1973})$$

where :

$S_c$	=	secondary (creep) settlement (mm)
$C_{\alpha\epsilon}$	=	modified secondary compression index
$H$	=	initial thickness of normally consolidated portion of compressible clay deposit (mm)
$t$	=	post-construction period of interest (10 years for this project)
$t_{EoP}$	=	time to reach end of primary consolidation (years)
$w_n$	=	natural water content (%)

The values of modified secondary compression index ( $C_{\alpha\epsilon}$ ) from the correlation noted above were also compared with the values of  $C_{\alpha\epsilon}$  calculated from the results of the laboratory consolidation tests, where necessary.





### **6.4.2.3 Settlement of Rock Fill Beneath Culvert**

Where rock fill is to be used as replacement fill below the culvert alignment in sub-excavation areas or the construction of the proposed embankments, there will be settlement due to compression of the rock fill itself under self-weight, in addition to the settlement of the underlying foundation soils as described above. The magnitude of settlement of the rock fill depends on the following factors:

- type of rock/strength of particles;
- size and shape of rock particles;
- gradation of rock fill;
- total height/thickness of rock fill (stress level); and,
- method of construction and sequence of placement (including lift thickness, compactive effort and state of packing).

The settlement of rock fill occurs as a result of re-arrangement of rock particles under load and wetting and as a result of localized crushing of rock particles at point contacts. The magnitude of both the short-term and long-term post-construction settlement of the rock fill is a function of the height of fill as well as the method of fill placement (i.e. compacted versus dumped rock fill) as outlined in MTO's Guideline for Rock Fill Settlement and Rock Fill Quantity Estimates, dated September 2010.

Rock fill should be placed, whenever possible, in a controlled manner (i.e. not end dumped) in accordance with Special Provision (SP) 206S03. Blading, dozing and 'chinking' the rock fill to form a dense, compact mass is required to minimize voids and bridging and reduce settlements and should be used to construct rock fill embankments above the existing groundwater table. Where rock fill cannot be placed in a controlled manner (i.e. below the groundwater table), the post-construction settlement of the rock fill is expected to be greater.

### **Short-Term Rock Fill Settlement**

The magnitude of short-term post-construction settlement associated with compacted and end-dumped rock fill may be estimated in accordance with the MTO's Guideline (September 2010), as follows:

<b>Total Height of Rock Fill, H</b>	<b>Short-Term Rock Fill Settlement (m)</b>	
	<b>Compacted Rock Fill</b>	<b>Dumped Rock Fill</b>
Up to 5 m	0.5% H	1.0% H
>5 m to 10 m	0.75% H	1.5% H
>10 m to 15 m	1.0% H	2.0% H

It should be noted that approximately 90 percent of the short-term settlement may be expected to occur within the first six (6) months following completion of rock fill placement and construction of the embankment to full height. The short-term settlement is expected to be fully completed within one (1) year following the completion of rock fill placement and embankment construction to full height.



## Long-Term Rock Fill Settlement

The magnitude of long-term post-construction settlement for compacted and end-dumped rock fill may be estimated in accordance with the MTO Foundations Guideline (September 2010), as follows:

Total Height of Rock Fill, H	Long-Term Rock Fill Settlement (m)	
	Compacted Rock Fill	Dumped Rock Fill
Up to 15 m	0.1% H	0.2% H

The long-term rock fill settlement is expected to occur from one (1) year following the completion of construction to over the life of the embankment.

### 6.4.3 Horizontal Strain

The following sections outline the method used to estimate the horizontal strain along the culvert at the various locations.

#### 6.4.3.1 Parameter Selection

As a result of the two-dimensional nature of the proposed embankment geometry, shear stresses will be mobilized in the foundation soils (upon completion of preload embankment construction and during the preload period) causing lateral spreading of the foundation soils and new embankment fill. This, in conjunction with the non-uniform vertical settlement of the foundation soils along the proposed culvert alignments will generate horizontal straining along the newly constructed culverts. In order to maintain structural integrity of the culverts, the culvert design must incorporate a suitable allowance for extension at the joints/couplings of the culvert segments to prevent the culvert from cracking and/or failing in tension.

The research work by Rutledge and Gould (1973) on the movements on articulated conduits under earth dams on compressible foundations can be used to estimate the magnitude of the horizontal strain likely to occur as a result of the proposed embankment construction at the culvert sites. The following equations have been used to obtain a relationship between vertical settlement, vertical strain, horizontal strain and maximum joint opening as a result of settlement of the foundation soils:

$$\epsilon_v = \frac{\delta_v}{d}$$

$$\epsilon_h = \epsilon_v \frac{\epsilon_h}{\epsilon_v}$$

$$\Delta L = \epsilon_h L$$





where :

$\Delta L$	=	maximum joint opening (m)
$\epsilon_v$	=	maximum vertical strain
$\epsilon_h$	=	maximum horizontal strain
$\frac{\epsilon_h}{\epsilon_v}$	=	estimated ratio of maximum horizontal strain to maximum vertical strain (from Rutledge and Gould, 1973 – Figure 2)
L	=	length of culvert (m)
$\delta_v$	=	maximum vertical settlement of culvert as a result of immediate and post-construction settlement of foundation soils and rock fill material (m)
d	=	thickness of compressible foundation strata at culvert location (m)

## 6.5 Geotechnical Axial Resistance

Section 6.6 outlines the recommended factored geotechnical axial resistance at Ultimate Limit States (ULS) and geotechnical reaction at Serviceability Limit States (SLS) for 25 mm of settlement for design of each culvert founded on a properly prepared granular subgrade (as discussed in Section 6.8.1). For culverts founded on bedrock, Serviceability Limit States (SLS) conditions do not apply. The geotechnical resistances provided are for loads applied perpendicular to the surface of the base of the culverts. Where loads are not applied perpendicular to the base of the culvert, inclination of the loads should be taken into account in accordance with Section 6.7.4 and Section C6.7.4 of the *Canadian Highway Bridge Design Code (CHBDC)* and its *Commentary*.

The loading on the foundation soils below the culverts and the associated total settlement at the culvert locations will be governed by the design height of the overlying and adjacent embankment fills. As such, it is recommended that the structural engineer exercise caution when utilizing the values of the geotechnical axial resistance at SLS in the design of the culverts. Where culverts are constructed following completion of all foundation soil settlement due to construction of embankment fills, the SLS values as provided may be used for the culvert design for settlement of 25 mm.

### 6.5.1 Resistance to Lateral Loads/Sliding Resistance

Resistance to lateral forces/sliding resistance between the base of the cast-in-place concrete footings and the granular fill/bedding placed following sub-excavation should be calculated in accordance with Section 6.7.5 of the *CHBDC*. The following summarizes the coefficient of friction,  $\tan \Phi$ , for the interface materials.

Interface Materials	Coefficient of Friction ( $\tan \Phi$ )
Cast-in-Place Concrete Footings on Compacted Granular 'B' Type II	0.58

## 6.6 Results of Analysis

The results of the stability and settlement analysis, estimated maximum vertical and horizontal strains, factored geotechnical axial resistance at Ultimate Limit States (ULS) and geotechnical reaction at Serviceability Limit States (SLS) for 25 mm of settlement are provided for each of the culvert sites in the following sections. In



addition, the options and recommendations for achieving the target factor of safety for the stability of the required embankment geometry at the culvert location, if necessary, and for minimizing the time dependent, post-construction settlements are also discussed. These options take into consideration the foundation mitigation recommendations for the embankment construction at the swamp area in which the SBL culvert is located as provided in the Foundation Investigation and Design Report (Golder, 2011). The results of analysis/foundation recommendations for each culvert are summarized in Table 3 and Table 4.

If the expected settlements, vertical strain and horizontal strain are relatively small, the preferred option is typically to construct the culvert concurrently with the embankment construction. Due to variations in the subsurface conditions along the length of the culverts, the settlements and horizontal strains may differ at different points along the culvert and this should be considered when choosing an appropriate design and construction methodology to be employed.

Where the expected settlements, vertical strain and horizontal strain are relatively large, the preferred option is to either install a temporary culvert (if necessary) and then construct the permanent culvert following the embankment preload period or to construct the culvert following full sub-excavation of compressible deposit(s) along the culvert alignment.

### **6.6.1 Highway 69 NBL – STA 11+198**

The culvert will extend across the existing Highway 69 roadway embankment (new NBL embankment) at STA 11+198 in Harrison Township. The existing embankment is oriented on a skew to the proposed NBL embankment as shown on Drawing 1. The new embankment will require about a 4.5 m grade raise above the existing Highway 69 embankment, resulting in the new embankment being about 8.0 m high above the existing swamp (relative to the west toe of the embankment). The topography of this section of proposed highway is generally flat and low-lying, with the swamp on the west side of the highway and bedrock outcrop on the east side of the highway.

The subsoils along the culvert alignment generally consist of embankment fill on the east side of the culvert and peat on the west side of the culvert, underlain by a deposit of silty clay to clay (west side of the culvert) and silty sand to sand and gravel, underlain by bedrock at the east end. The bedrock surface slopes steeply down from east to west beneath the culvert footprint. The bedrock surface was encountered at about Elevation 195.2 m at Borehole LG-3 advanced near the east end of the culvert and refusal to further split-spoon advancement was encountered at Elevation 185.5 m at Borehole LG-2 near the west end of the culvert. Due to the presence of the high bedrock surface on the east side of the culvert, bedrock removal will be required as discussed further below. Further, depending on the founding elevation of the RSS walls at the east end of the culvert, bedrock removal will likely be required as refusal was encountered in DCPTs LG-DC1 and LG-DC2 at Elevation 196.0 m and 196.5 m, respectively, consistent with but slightly higher than the bedrock surface at the culvert inlet location.

Details of the subsurface conditions along this culvert are presented in the respective Record of Borehole sheets and shown on Drawing 1.



Prior to the start of construction of the NBL embankment and culvert, the traffic will be routed from the existing highway to the new Site 9 Road embankment west of the existing/new highway alignment and therefore staged construction of the culvert will not be required.

As discussed in Section 6.6.2, the preferred stability and settlement mitigation option for the new SBL embankment crossing the swamp area is full sub-excavation of the cohesive deposit and replacement with rock fill and preloading the embankment for 6 months, as discussed in the Foundation Investigation and Design Report (Golder, 2011) for the adjacent swamp crossing by the roadway embankment. Based on the recommendations for full sub-excavation of the cohesive deposit under the SBL culvert, we recommend that the cohesive deposit below the NBL culvert also be fully sub-excavated and replaced with rock fill.

The stability analysis carried out for the west half of the NBL embankment indicates that after sub-excavation of the cohesive deposit and replacement with rock fill, the embankment adjacent to the culvert will have a Factor of Safety (FoS) equal to or greater than 1.3 (for rock fill unit weights of  $18 \text{ kN/m}^3$  and  $19 \text{ kN/m}^3$ ) for a deep seated global failure surface. Figure 1 presents the results of the stability analysis for the west end of the SBL culvert, with generally consistent subsoils at the west end of the NBL culvert.

As discussed above, at the east end of the culvert, the bedrock is present approximately 0.7 m below the invert and slopes down to the west. Where the full 1.8 m thickness of soil cover for frost protection of the bottom of the footing is not available due to the high bedrock surface elevation at the east end, the bedrock surface should be benched/excavated to accommodate the required frost cover and granular leveling pad below the culvert footings. Constructing the east end footings over a 0.6 m granular leveling pad and with 1.8 m frost cover instead of directly founding the footings on the bedrock surface as a mitigation measure to reduce the potential for differential settlement between the east end and the west end of the culvert. It is recommended that an approximately 5 m long section of the bedrock along/at the east end of the culvert be excavated to a depth of 2.4 m below the culvert invert and replaced with a 0.6 m thick granular leveling pad below the footings consisting of compacted SP110S13 (Aggregates) Granular 'B' Type II material.

At the west portion of the NBL culvert, due to the need for unwatering in order to construct the culvert footings in the dry, as discussed further in Section 6.8.3, Granular 'B' Type II should be placed immediately below the bottom of the footings over the rock fill. We recommend that a 600 mm thick transition layer consisting of 300 mm-minus crushed rock be placed between the underlying rock fill and Granular 'B' Type II. Alternatively, consideration could be given to replacement of the sub-excavated cohesive deposit with Granular 'B' Type II from the surface of the native cohesionless soils to the underside of the footing.

The factored geotechnical axial resistance at Ultimate Limit States (ULS) for an assumed 1.5 m wide strip footing founded at a depth of 1.8 m below final ground surface on a properly prepared Granular 'B' Type II subgrade/bedding underlain by rock fill (i.e. backfill within the area of sub-excavation of the cohesive deposit), bedrock and the native cohesionless soils at this location is estimated to be 750 kPa. The geotechnical resistance at SLS (for 25 mm settlement) for a 1.5 m strip footing constructed on a properly prepared subgrade as noted above may be taken as 350 kPa, as noted in Table 4.

As the cohesive deposit will be sub-excavated and replaced with rock fill, and given the presence of native cohesionless deposits underlying the rock fill or granular bedding over bedrock, the total settlement along the alignment of the new culvert after full embankment construction is estimated to be up to about 100 mm at the west end of the culvert, 50 mm at the centre of the culvert and less than 5 mm at the east end of the culvert. As



discussed above, the bedrock along/near the east end of the culvert should be excavated to 2.4 m below culvert invert and replaced with compacted Granular 'B' Type II to provide for both a granular bedding layer to reduce the potential for differential settlement and soil cover for frost protection. Due to the relatively large settlements and expected horizontal strain due to the thickness of rock fill and cohesionless deposit underlying the west side of the embankment, it is recommended that the permanent NBL culvert be constructed following the 6-month NBL embankment preload period. Following the 6-month preload period, the total settlement along the alignment of the new NBL culvert after full embankment construction is estimated to be less than 5 mm at the east end of the culvert, 10 mm at the centre of the culvert and up to about 20 mm at the west end of the culvert, as noted in Table 4. The settlement is a result of the remaining short-term and total long-term settlement of the rock fill. The maximum horizontal strain along the 25 m long NBL culvert associated with the total settlement described above after a 6-month preload period is estimated to be up to about 0.14 percent of the culvert length. As such, the permanent culvert can be constructed after the six-month preload period concurrent with final construction of the embankment without the need for any additional special foundation mitigation measures, provided that the structural design of the culvert can accommodate the estimated settlement and horizontal strain.

### **6.6.2 Highway 69 SBL – STA 11+198**

The culvert will extend across the proposed Highway 69 SBL and a swamp (Swamp 6) at about STA 11+198 in Harrison Township. The proposed embankment at the culvert location is about 9.0 m high above existing grade. The topography of this section of proposed highway is generally a flat and low-lying swamp.

The subsoils along the culvert alignment generally consist of peat underlain by silty clay to clay and silty sand to sand and gravel.

Details of the subsurface conditions along this culvert are presented in the respective Record of Borehole sheets and shown on Drawing 1.

The preferred stability and settlement mitigation option for the new SBL embankment crossing this swamp area is full sub-excavation of the cohesive deposit and replacement with rock fill and preloading the embankment for 6 months, as discussed in the Foundation Investigation and Design Report for the adjacent swamp crossing by the roadway embankment (Golder, 2011). Figure 1 presents the results of the stability analysis for the west end of the SBL culvert.

As discussed in Section 6.6.1 and further in Section 6.8.3, due to the need for unwatering in order to construct the culvert footings in the dry, SP110S13 (Aggregates) Granular 'B' Type II should be placed below the bottom of the footings over the rock fill. We recommend that a 600 mm thick transition layer consisting of 300 mm-minus crushed rock be placed between the underlying rock fill and Granular 'B' Type II. Alternatively, consideration could be given to replacement of the sub-excavated cohesive deposit with Granular 'B' Type II from the surface of the native cohesionless soils to the underside of the footing.

The factored geotechnical axial resistance at Ultimate Limit States (ULS) for an assumed 1.5 m wide strip footing founded at a depth of 1.8 m below final ground surface on a properly prepared Granular 'B' Type II subgrade underlain by rock fill (i.e. backfill within the area of sub-excavation of the cohesive deposit) and the native cohesionless soils at this location is estimated to be 750 kPa. The geotechnical resistance at SLS (for 25 mm



settlement) for a 1.5 m strip footing constructed on a properly prepared subgrade as noted above may be taken as 350 kPa, as noted in Table 4.

As the cohesive deposit is to be fully sub-excavated and replaced with rock fill and given the presence of native cohesionless deposits underlying the rock fill, the total settlement along the alignment of the new culvert is estimated to be up to about 130 mm at the east end of the culvert, 180 mm at the centre of the culvert and 70 mm at the west end of the culvert. Due to the relatively large settlements and expected horizontal strain due to the thickness of rock fill and cohesionless deposit underlying the embankment, it is recommended that the permanent SBL culvert be constructed following the 6-month SBL embankment preload period. Following the 6-month preload period, the total settlement along the alignment of the permanent SBL culvert after full embankment construction is estimated to be up to about 20 mm at the east end of the culvert, 25 mm at the centre of the culvert and 15 mm at the west end of the culvert, as noted in Table 4. The settlement is a result of the remaining short-term and total long-term settlement of the rock fill. The maximum horizontal strain along the 25 m long culvert associated with the total settlement described above is negligible. As such, the permanent culvert can be constructed after the 6-month preload period concurrent with final construction of the embankment without the need for any additional special foundation mitigation measures.

## **6.7 Retained Soil System (RSS) Walls**

RSS walls are required at the ends of each side of the culvert to maximize the culvert openness ratio by minimizing the culvert length. As shown on Drawing 1, the walls at the east end of the NBL culvert will be at a skew to the culvert and will be about 9 m in length and the walls at the west end of the SBL culvert will also be at a skew to the culvert and will be about 17 m in length. Between the two culverts, RSS walls up to about 8 m high will be installed in line between the NBL and SBL culverts.

An RSS wall consists generally of granular fill placed and compacted in layers and reinforced with fabric strips or grids. A facing material, typically pre-cast concrete panels mechanically fastened to the reinforcing strips or grids, is used to form the vertical face of the reinforced soil structure and to prevent loss of fill material. A typical RSS wall has the front facing supported on a strip footing placed at shallow depth below the ground surface in front of the wall. For design, a minimum founding depth of 0.8 m is recommended for the facing footing.

The facing footing and soil mass should be placed on a 150 mm thick granular fill levelling pad comprised of compacted SP 110S13 (Aggregates) Granular 'A' or Granular 'B' Type II.

As discussed in Section 6.6.1 and 6.6.2, the cohesive deposit below the culverts will be sub-excavated and replaced with rock fill and SP110S13 Granular 'B' Type II placed over the rock fill to facilitate dewatering. We recommend that for consistency of founding soils, the sub-excavation and backfilling within the footprint of the SBL RSS walls and the west NBL RSS walls be similar to the adjacent culvert in these areas. The backfilling with rock fill and Granular 'B' Type II over which the 150 mm thick granular levelling pad will be placed should extend a minimum of 2 m beyond the edge of the RSS wall soil mass and front facing footing.

Based on the results of Borehole LG-3 and DCPTs LG-DC1 and LG-DC2, at the east side of the NBL culvert, depending on the founding elevation of the facing footing and soil mass, the subgrade below the 150 mm thick granular levelling pad for the facing footing and soil mass will likely transition from a relatively thin layer of granular fill (i.e. culvert bedding) over bedrock to bedrock. Where bedrock surface is either exposed or located



below a relatively thin layer of overburden and is noted to be sloping, we recommend that the bedrock be benched/levelled prior to placement of the 150 mm thick granular fill levelling pad to increase the frictional resistance between the granular pad and the bedrock surface (i.e. to reduce the opportunity for sliding of the granular pad on the bedrock surface) and to avoid the need for rock dowels for the facing footing.

Assuming that the RSS wall acts as a unit and utilizes the full width of the reinforced soil mass, which has been taken as 0.8 times the height of the wall, the factored geotechnical axial resistance at ULS and the geotechnical resistance at SLS (for 25 mm of settlement) given below may be used for assessment of the reinforced mass founded on the properly prepared granular fill or rock fill at the SBL culvert and the bedrock at the east side of the NBL culvert.

<b>Culvert RSS Wall</b>	<b>Wall Height (m)</b>	<b>Assumed Reinforced Width* (m)</b>	<b>Factored Geotechnical Axial Resistance at ULS (kPa)</b>	<b>Geotechnical Resistance at SLS (for 25 mm of settlement) (kPa)</b>
NBL east side	6	5	10,000	N/A
NBL west side	7	6	750	350
SBL	8	6	750	350

\* Assumed equivalent to 80 % of the wall height.

The resistance to lateral forces/sliding resistance between the compacted fill of the RSS wall and the subgrade should be calculated in accordance with Section 6.7.5 of the CHBDC. The coefficient of friction,  $\tan \phi'$ , between the compacted granular fills of the RSS wall and the properly prepared granular fill subgrade may be taken as 0.6. The coefficient of friction between the compacted granular fills of the RSS wall for the NBL and the properly prepared bedrock surface may be taken as 0.7.

The static global stability of the RSS wall at the west end of the NBL and SBL culverts has been analyzed and the results of the analysis are discussed in Section 6.6 (see Figure 1). As the wall will be constructed essentially on shallow bedrock at the east side of the NBL, stability is not a concern at this location. The internal stability of the RSS wall should be checked by the RSS supplier/designer. As discussed in Section 6.6, full sub-excavation of the cohesive deposit is recommended below the NBL and SBL culverts. Provided the sub-excavation is also carried out below the RSS walls and the rock fill below the RSS walls is preloaded for 6 months, settlement below the RSS wall is anticipated to be negligible.

## **6.8 Culverts – Construction Considerations**

### **6.8.1 Subgrade Preparation, Excavation and Replacement**

The following sections discuss general aspects of subgrade preparation and embankment construction at the culvert and RSS walls, including: removal of surficial organic soils; and excavation and replacement of soft subsoils.





All excavations must be carried out in accordance with Ontario Regulation 213 Ontario Occupational Health and Safety Act for Construction Projects (as amended by Ontario Regulation 443). Sub-excavation of the cohesive deposit below the NBL and east half of the SBL culverts should be carried out only after the existing Highway 69 traffic has been routed to the new Site 9 Road embankment. However, as discussed in Golder (2011), we recommend that the cohesive deposit below the west half of the new SBL embankment be sub-excavated at the same time as the sub-excavation for the new Site 9 Road embankment, in order to minimize disturbance to the new Site 9 Road embankment once traffic is routed to this embankment.

### **6.8.1.1      *Removal of Organic Materials and Soft Soils***

Based on the information from the boreholes advanced during the field investigation, the thickness of organic deposits (i.e. peat) and underlying cohesive soils at the culvert and RSS wall locations is up to 7.7 m. It is anticipated that conventional equipment will be suitable for the excavation of the soft subsoils in these areas.

As the sub-excavation will be carried out in the swamp area away from or adjacent to existing highway embankments, as recommended above, embankment construction should be carried out as per OPSP 203.010 (Embankments Over Swamp-New Construction) and OPSS 209 (Embankments Over Swamp and Compressible Soils).

### **6.8.1.2      *Replacement Backfill Below Base of Culverts and RSS walls***

For replacement of sub-excavated material along the culverts and RSS walls, it is assumed that rock fill will be used to backfill the excavations. Where sub-excavation of soft subsoils is being carried out as a foundation mitigation option within culvert areas, it will not likely be possible to place rock fill in accordance with SP 206S03, as discussed in Section 6.4.2.3. In these instances (i.e. typically backfill below the water table), the rock fill will likely to be end-dumped concurrently as the excavation advances.

As discussed in Section 6.6.1 and 6.6.2, we recommend that a 600 mm thick transition layer be placed between the underlying rock fill and the SP110S18 (Aggregates) Granular 'B' Type II material immediately below the west side of the NBL culvert and associated west RSS walls as well as the SBL culvert and associated east and west RSS walls. The Granular 'B' Type II below the culvert will likely be placed in the wet and when nominally compacted, should achieve a density of 90 percent of the Standard Proctor Maximum Dry Density (SPMDD).

### **6.8.1.3      *Excavation of Bedrock***

Along/near the east end of the NBL culvert, where rock excavation is recommended to allow for placement of a granular bedding layer as the foundation stratum for the footings to reduce the potential for differential settlement, consideration could be given to blasting of the bedrock as the method of rock excavation.

Based on laboratory test result on rock core samples, the bedrock at this site is generally classified as strong (R4) and as such, bedrock excavation along the culvert alignment should be carried out in such a manner as to minimize blast damage on the rock (i.e. shattering and over-break) and provide better control over the configuration of the founding surface. Further, following excavation, it will be necessary to scale and remove all



loose, shattered and/or fractured rock within the footprint of the culvert foundation to provide for a prepared surface upon which the granular bedding can be properly placed and compacted.

### **6.8.2 Culvert Backfill**

The backfill requirements for the proposed cast-in-place culverts should be in accordance with OPSS 902 (Excavating and Backfilling – Structures).

Backfill behind the culvert walls should consist of granular fill meeting the specifications of SP 110S13 (Aggregates) Granular 'A' or Granular 'B' Type II, but with less than 5 percent passing the No. 200 (0.075 mm) sieve. The backfill should be placed and compacted in accordance with OPSS 501 (Compacting). The fill should also be placed concurrently on both sides of the culvert walls, ensuring that the backfill depth on one side does not exceed the other side by more than 400 mm.

Where preloading is used to reduce post-construction settlements, and temporary culverts are incorporated into the works and are subsequently removed, the backfill above the permanent culverts should consist of SP110S13 (Aggregates) Granular 'A' or Granular 'B' Type II or rock fill with a maximum particle size less than or equal to 250 mm, to minimize differential settlements along the highway embankments.

The culverts should be designed for the full overburden stress and appropriate live loads, assuming a fill unit weight of 22 kN/m<sup>3</sup> for Granular 'A' and 21 kN/m<sup>3</sup> for Granular 'B' Type II backfill above and surrounding the culvert.

Inspection and field density testing should be carried out by qualified geotechnical personnel during all engineered fill placement operations to ensure that appropriate materials are used, and that adequate levels of compaction have been achieved.

### **6.8.3 Control of Groundwater and Surface Water**

Excavation within the plan limits of the proposed culvert alignments will be required to remove organic and/or soft deposits prior to placement of backfill/embankment fill, bedding material and the actual culvert structure. As a result of the excavation, groundwater flow into the excavation can be expected to occur due to the relatively permeable subsoils and high groundwater levels observed at the culvert locations. Therefore, control of surface water and groundwater will be necessary at the culvert locations to allow for construction to be carried out in dry conditions.

Depending on surface flows and groundwater levels at the time of construction, water flow could be passed through the area by means of a temporary culvert (as discussed in Section 6.9, if required), or diverted by pumping from behind a temporary cofferdam. Surface water should be directed away from the excavations areas to prevent ponding of water.

Unwatering is required to minimize groundwater inflow into the excavation and construction area and to allow construction of the cast-in-place footings and foundation walls in dry conditions. Seepage into the excavation should be adequately controlled by a suitable pumping system and/or diversion system and/or a cut-off system such as a sheet pile cofferdam. In the case of a sheet pile cofferdam, the area for the construction of the





cast-in-place elements should be fully enclosed within the temporary sheet pile cut-off wall which should be driven into a minimum 3 m thick SP110S13 Granular 'B' Type II "blanket" extending downwards from the lowest elevation of the cast-in-place elements and constructed over the entire bottom of the excavation area. The Granular 'B' Type II blanket should extend laterally for a minimum of 6 m beyond the limits of the sheet pile cofferdam. Alternatively, a minimum 0.5 m thick concrete plug could be placed in the bottom of the cofferdam to facilitate dewatering of groundwater inflow into the excavation area. It is recommended that OPSS 517 (Dewatering) be referenced in the Contract Documents to address unwatering requirements.

## **6.9 Temporary Culverts**

As the permanent culverts will be constructed subsequent to a preload period to mitigate settlements of the embankments/sub-excavation backfill, temporary culverts may be required during the preload period. It should be noted that the estimated settlements are expected to be relatively large (as outlined in Table 3) and as a result the temporary culverts should be sized such that they may still perform the intended function for the duration of the preload period. Bedding recommendations should be in accordance with the corresponding OPSS and/or OPSD depending on the type of the temporary culvert chosen.

The location of the temporary culverts could be offset from the actual alignment of the permanent culverts. The temporary culverts should also be constructed within a temporary granular core. It is recommended that these culverts be removed following the permanent culvert construction. If it is not desirable to remove the temporary culvert, consideration could be given to backfilling the temporary culvert with 'unshrinkable' fill material.

### **6.9.1 Temporary Granular Fill Core**

As the permanent culverts will likely be constructed subsequent to the preload period, excavation through the new embankment highway fill will be required to reach the culvert founding level. Therefore, to provide for an easier excavation operation, it may be preferable to construct a 'temporary core' at the culvert locations using granular fill or SP110S13 Granular 'B' Type II material, as these materials may be excavated more easily than rock fill. The material comprising the temporary core should be placed in accordance with SP 206S03 (Earth Excavation – Grading).

If constructed, the granular core should encompass the entire width of each culvert and should extend upwards at a slope no steeper than 1.25H:1V from the base of the core to the crest of the embankment. However, the outer (exterior) side slopes of the temporary granular fill core should be no steeper than 2H:1V. This requires that the length of the temporary culverts be greater than the length of the permanent culverts, as the temporary culverts will need to extend beyond the temporary granular side slopes of the embankment to allow for drainage. A temporary geotextile separator, such as OPSS1860 (Geotextile) Class II non-woven with FoS between 45 µm and 150 µm, should be placed between the rock fill and granular fill core to prevent mitigation of fines into the rock fill. A schematic of the temporary core construction details is presented on Figure 2.

Upon completion of the preload period and construction of the permanent culverts, the temporary core material may be reused as backfill material. However, the outer side slopes of the embankment where comprised of the permanent granular fill would need to be maintained at no steeper than 2H:1V, resulting in a longer permanent



culverts. If this option is not desired, the temporary core material should be completely removed and replaced with rock fill with maximum particle size less than or equal to 250 mm to minimize differential settlement.

## **7.0 CLOSURE**

This report was prepared by Mr. André Bom, P.Eng. Mr. Jorge M. A. Costa, P.Eng., Golder's Designated MTO Contact for this project and a Principal with Golder, reviewed the technical aspects of and conducted an independent quality control review of the report.

## Report Signature Page

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**COMMERCIAL SOFTWARE:**

Geoslope (version 7.19) by Geo-Studio International Ltd.

Settle 3D (version 2.0) by Rocscience Inc.

**STANDARDS:**

ASTM International:

ASTM D1586                      Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils

ASTM D2573                      Standard Test Method for Field Vane Shear Test in Cohesive Soil

Contract Design Estimating and Documentation (CDED):

Special Provision 110S13      Material Specification for Aggregates – Base, Subbase, Select Subgrade and Backfill Material.

Special Provision 206S03      Earth Excavation, Grading.

Ministry of Transportation Ontario:

Guideline for Rock Fill Settlement and Rock Fill Quantity Estimates. September 2010.

Ontario Occupational Health and Safety Act:

Ontario Regulation 213/91      Construction Projects

Ontario Regulation 443/09      Amendment to Ontario Regulation 213

Ontario Provincial Standard Drawing:

OPSD 203.010                      Embankments Over Swamp – New Construction.

OPSD 802.010                      Flexible Pipe Embedment and Backfill Earth Excavation.

OPSD 3090.101                      Foundation Frost Penetration Depths for Southern Ontario.

Ontario Provincial Standard Specification:

OPSS 209                              Construction Specification For Embankments Over Swamps and Compressible Soils.

OPSS 501                              Construction Specification for Compacting.

OPSS 517                              Construction Specification for Dewatering of Pipeline, Utility and Associated Structure Excavation.

OPSS 902                              Construction Specification for Excavating and Backfilling – Structures.

Ontario Water Resources Act:

Ontario Regulation 903          Wells (as Amended by O.Reg. 468/10)



**FOUNDATION REPORT - HIGHWAY 69 UNGULATE CULVERTS**  
**GWP 5403-05-00, WP 5133-12-20 & -21**

**Table 1: Summary of Culvert Details**

Culvert Location (Associated Swamp)	Approximate Proposed Embankment Height (m)	Invert Elevations <sup>1</sup>		Culvert Dimensions <sup>1</sup>			Walls	Boreholes/DCPTs
		East End of Culvert (m)	West End of Culvert (m)	Width (m)	Height (m)	Length (m)		
Highway 69 NBL STA 11+198	8.0	195.9	195.1	7.0	5.0	19.2	RSS	3 Boreholes (LG-2 that is shared with SBL culvert, LG-1 and LG-3) 2 DCPTs (LG-DC1 and LG-DC2)
Highway 69 SBL STA 11+198 (Swamp 6)	9.0	194.2	194.0	7.0	5.0	19.2	RSS	1 Borehole (LG-2 that is shared with NBL culvert) 3 Swamp Boreholes (S6-2 to S6-4) 2 Swamp DCPTs (S6-DC1 and S6-DC2)

Note: <sup>1</sup> Invert elevations and culvert dimensions provided by MMM on April 17, 2012.

Prepared by: EC  
 Checked by: AB/JMAC



**FOUNDATION REPORT - HIGHWAY 69 UNGULATE CULVERTS**  
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**Table 2: Summary of Foundation Engineering Parameters**

Culvert Location (Associated Swamp)	Stratigraphic Unit <sup>1</sup>	Top Elevation (m)	Thickness (m)	$\gamma'$ (kN/m <sup>3</sup> )	$\phi'$ (°)	$c'$ (kPa)	$S_u$ (kPa)	$\sigma_p'^2$ (kPa)	$e_o^2$	$C_c^2$	$C_r^2$	$m_v$ (kPa <sup>-1</sup> )	$E'$ (MPa)	$c_v$ (cm <sup>2</sup> /s)
Highway 69 NBL STA 11+198	Peat	194.5	3.2	12	27	1	-	-	-	-	-	-	-	-
	Silty Clay to Clay	193.7 to 191.3	1.2 to 2.5	16	-	-	10	45	2.0	0.81	0.07	-	-	$1.6 \times 10^{-3}$
	Silty Sand to Sand and Gravel	196.2 to 188.8	1.0 to > 5.8	18.5	30	0	-	-	-	-	-	-	15	-
Highway 69 SBL STA 11+198 (Swamp 6)	Peat	194.5 to 194.3	2.9 to 3.7	12	27	1	-	-	-	-	-	-	-	-
	Silty Clay to Clay	191.4 to 190.6	0.9 to 2.5	16	-	-	10	45	2.0	0.81	0.07	-	-	$1.6 \times 10^{-3}$
	Silty Sand to Sand and Gravel	190.0 to 186.6	1.6 to >4.6	19	30	0	-	-	-	-	-	-	15	-

Notes: <sup>1</sup> Stratigraphic units exclude Fill deposit associated with existing roadway embankment.

<sup>2</sup> Engineering parameters for cohesive deposits are based on correlations from laboratory test data in Swamp 6.

Prepared by: EC  
 Checked by: AB/JMAC



**FOUNDATION REPORT - HIGHWAY 69 UNGULATE CULVERTS**  
**GWP 5403-05-00, WP 5133-12-20 & -21**

**Table 3: Summary of Settlement Analysis and Geotechnical Axial Resistance**

Culvert Location (Associated Swamp)	Approximate Proposed Embankment Height (m)	Expected Total Settlement (mm) <sup>1</sup>			Preferred Mitigation Option for Culvert Construction <sup>2</sup>	Geotechnical Axial Resistance <sup>4</sup>	
		Culvert Construction Concurrent with Embankment Construction	Culvert Construction Following Preload Period	Culvert Construction Following Full Sub-Excavation		Founding Soil <sup>3</sup>	ULS/SLS (kPa)
Highway 69 NBL STA 11+198	8.0	$\delta_{East} < 5 \text{ mm}$ $\delta_{Centre} = 50 \text{ mm}$ $\delta_{West} = 100 \text{ mm}$	N/A	$\delta_{East} < 5 \text{ mm}$ $\delta_{Centre} = 10 \text{ mm}$ $\delta_{West} = 20 \text{ mm}$	Full Sub-Excavation (up to about 5.7 m deep) with Pre-loading of Rock Fill (6 months)	Granular 'B' Type II over Rock Fill over Silty Sand to Sand and Gravel	750/350
Highway 69 SBL STA 11+198 (Swamp 6)	9.0	$\delta_{East} = 130 \text{ mm}$ $\delta_{Centre} = 180 \text{ mm}$ $\delta_{West} = 70 \text{ mm}$	N/A	$\delta_{East} = 20 \text{ mm}$ $\delta_{Centre} = 25 \text{ mm}$ $\delta_{West} = 15 \text{ mm}$	Full Sub-Excavation (up to about 7.7 m deep) with Pre-loading of Rock Fill (6 months)	Granular 'B' Type II over Rock Fill over Silty Sand to Sand and Gravel	750/350

Notes: <sup>1</sup> Total settlement refers to the sum of immediate, primary and secondary settlement as well as settlement of rock fill below the base of the culvert.

<sup>2</sup> All peat/organic deposits to be removed prior to culvert construction and excavation backfilled with rock fill.

<sup>3</sup> Bedding for the culverts should be at least 300 mm thick and consist of Granular 'B' Type II, likely placed in the wet and nominally compacted.

<sup>4</sup> Geotechnical axial resistance at ULS is a factored value; geotechnical reaction value at SLS is for 25 mm of settlement.

Prepared by: EC

Checked by: AB/JMAC





## FOUNDATION REPORT - HIGHWAY 69 UNGULATE CULVERTS

### GWP 5403-05-00, WP 5133-12-20 & -21

**Table 4: Summary of Preferred Mitigation Options for Ungulate Culvert Construction and Estimated Settlement and Strains**

Culvert Location (Associated Swamp)	Approximate Proposed Embankment Height (m)	Preferred Mitigation Option for Culvert Construction <sup>1</sup>	Expected Total Settlement <sup>2</sup> (mm)	Geotechnical Axial Resistance <sup>4</sup>		Culvert Strain				
				Founding Soil <sup>3</sup>	ULS/SLS (kPa)	Estimated Vertical Strain (%)	Estimated Ratio of Horizontal Strain to Vertical Strain	Estimated Horizontal Strain (%)	Culvert Length (m)	Estimated Maximum Joint Opening (mm)
Highway 69 NBL STA 11+198	8.0	Full Sub-Excavation (up to about 5.7 m deep) with Pre-loading of Rock Fill Embankment (6 months)	$\delta_{East} < 5 \text{ mm}$ $\delta_{Centre} = 10 \text{ mm}$ $\delta_{West} = 20 \text{ mm}$	Granular 'B' Type II over Rock Fill over Silty Sand to Sand and Gravel	750/350	0.3	0.47	0.14	19.2	30
Highway 69 SBL STA 11+198 (Swamp 6)	9.0	Full Sub-Excavation (up to about 7.7 m deep) with Pre-loading of Rock Fill Embankment (6 months)	$\delta_{East} = 20 \text{ mm}$ $\delta_{Centre} = 25 \text{ mm}$ $\delta_{West} = 15 \text{ mm}$	Granular 'B' Type II over Rock Fill over Silty Sand to Sand and Gravel	750/350	Negligible	Negligible	Negligible	19.2	Negligible

Notes <sup>1</sup> All peat/organic deposits to be removed prior to culvert construction and excavation backfilled with rock fill.

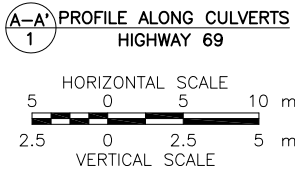
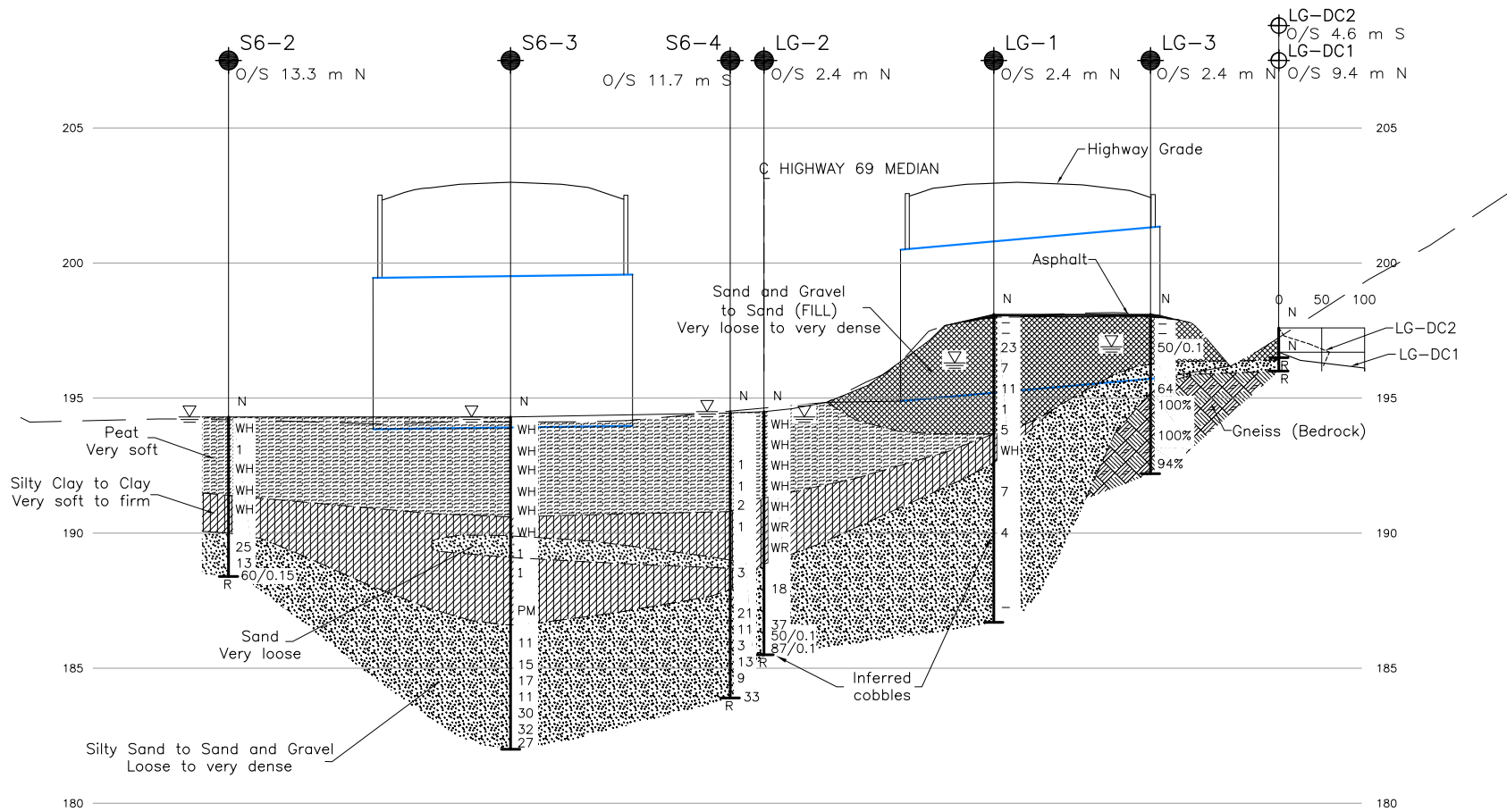
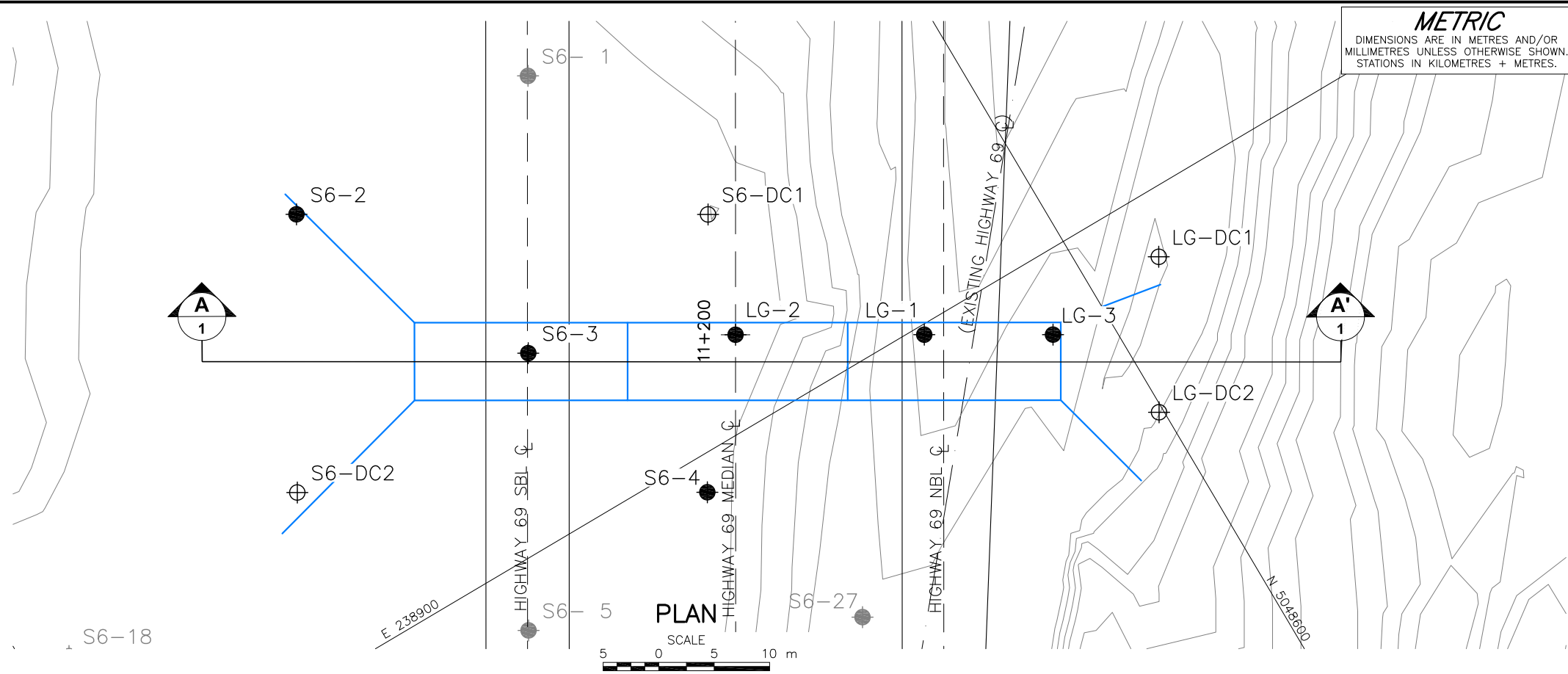
<sup>2</sup> Total settlement refers to the sum of immediate, primary and secondary settlement, as well as settlement of rock fill below the base of the culvert.

<sup>3</sup> Bedding for the culverts should be at least 300 mm thick and consist of Granular 'B' Type II, likely placed in the wet and nominally compacted.

<sup>4</sup> Geotechnical axial resistance at ULS is a factored value; geotechnical reaction value at SLS is for 25 mm of settlement.

Prepared by: EC

Checked by: AB/JMAC



**REFERENCE**

Base plan provided in digital format by MMM Group, drawing file no. Hwy 69 Design - Rollplan - Golder Foundation.dwg (received Dec. 2007) and key plan, drawing file no. Hwy 69-529-Project key plan (received Apr. 2008). Culvert locations provided by MMM in drawing file no. Animal Crossing CV-Harrison-11+200x5.dwg (received April 17, 2012)

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

CONT No.  
WP No.5133-12-20 & 21

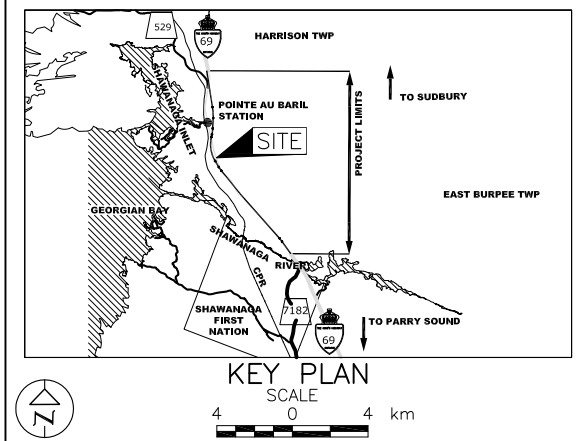


HIGHWAY 69  
CULVERTS AT STA. 11+198  
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET



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



**LEGEND**

●	Borehole
⊕	Dynamic Cone Penetration Test
N	Standard Penetration Test Value
16	Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
R	Refusal
100%	Rock Quality Designation (RQD)
≡	WL upon completion of drilling

BOREHOLE CO-ORDINATES			
No.	ELEVATION	NORTHING	EASTING
LG-1	198.1	5048584.9	238900.8
LG-2	194.5	5048570.3	238892.2
LG-3	198.1	5048594.9	238906.8
LG-DC1	196.7	5048606.7	238905.6
LG-DC2	197.6	5048599.6	238917.6
S6-2	194.3	5048541.8	238862.7
S6-3	194.3	5048553.4	238884.1
S6-4	194.5	5048560.9	238903.1
S6-DC1	194.7	5048573.7	238881.6
S6-DC2	194.3	5048529.1	238884.3

**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.

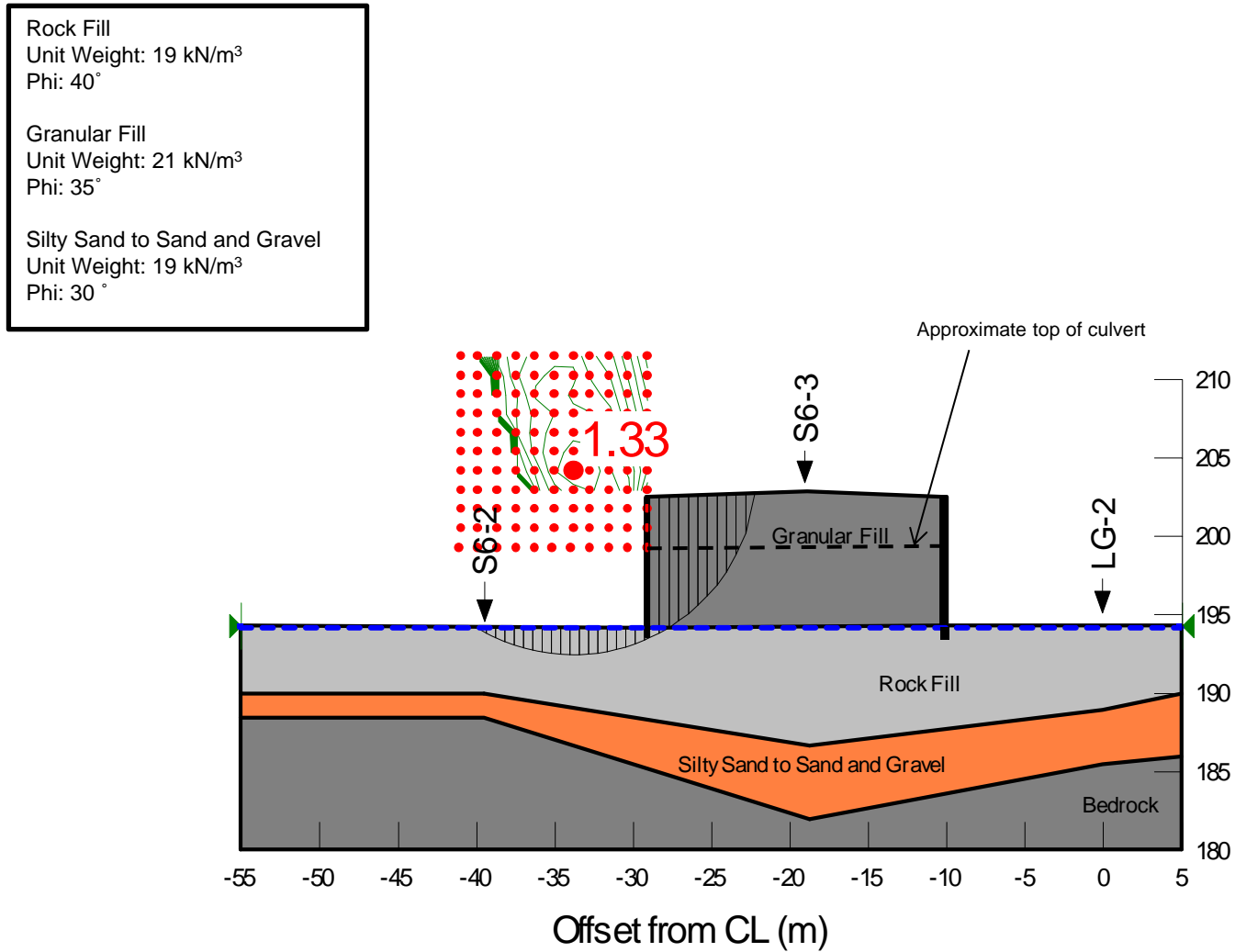
The complete Foundation Investigation and Design Report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with Section GC 2.01 of OPS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

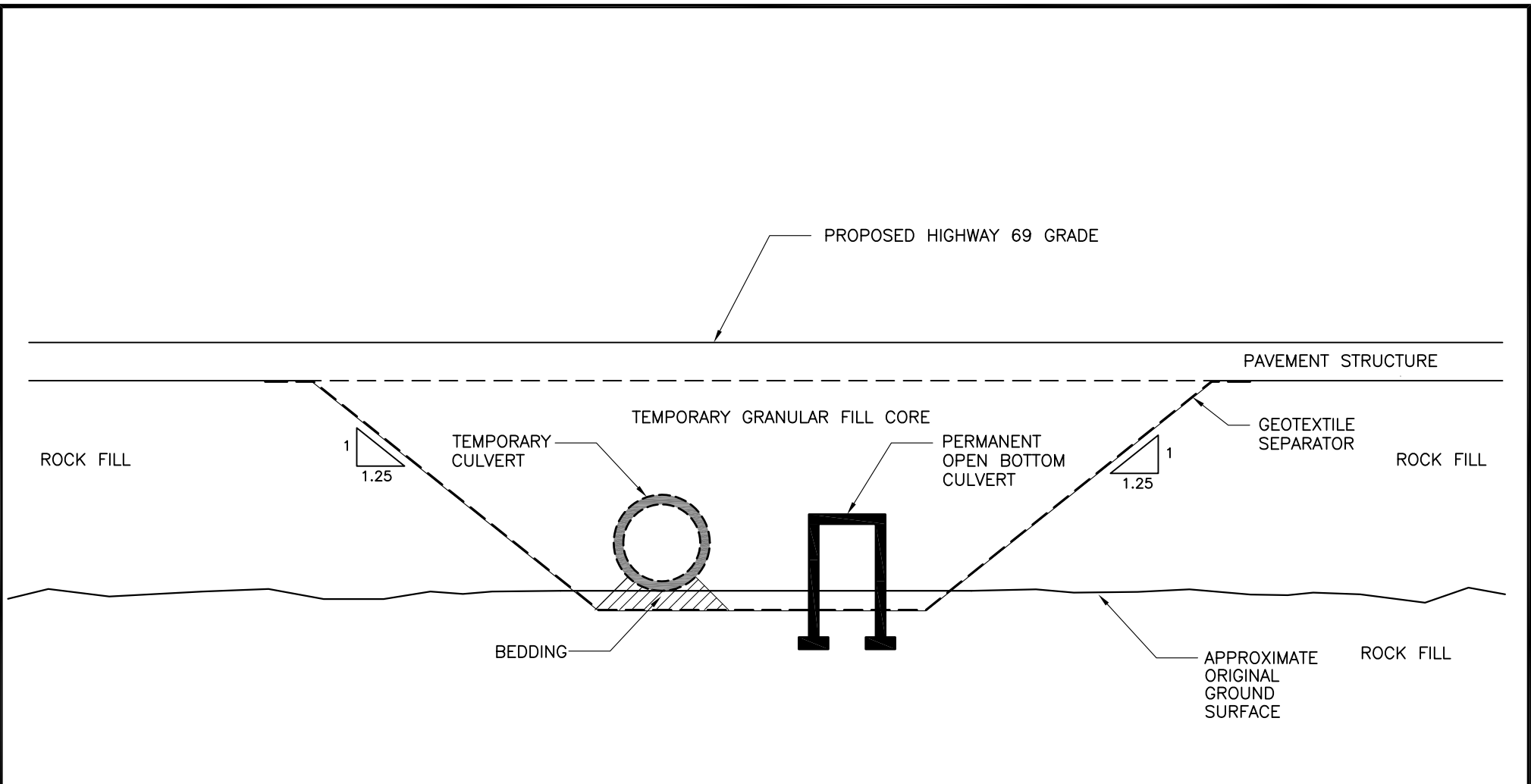
NO.	DATE	BY	REVISION
Geocres No. 41H-116			
HWY: 69		PROJECT NO. 07-1191-0020	
SUBM'D. EC		SITE:44-594/C1-C2	
DRAWN: TB		DWG: 1	




## Highway 69 Ungulate Culvert at STA 11+198 SBL West Side Slope Stability

Figure 1





**NOTE:**  
1. OUTER SIDE SLOPES OF TEMPORARY GRANULAR FILL CORE SHOULD BE NO STEEPER THAN 2H:1V.

 Sudbury, Ontario, Canada	SCALE	N.T.S.	<b>HIGHWAY 69 FOUR - LANING - CULVERTS FROM 0.4 KM NORTH OF HIGHWAY 7182 (SHEBESHEKONG ROAD) NORTHERLY 11 KM</b>
	DATE	January 13	
	DESIGN		
	CAD	TB	
FILE No. 07-1191-0020 Figure 2 Ungulate.dwg	CHECK	AB	<b>TEMPORARY GRANULAR FILL CORE</b>
PROJECT No. 07-1191-0020	REV.	JMAC	
			FIGURE <b>2</b>



# **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

### II. STRESS AND STRAIN

$\gamma$	shear strain
$\Delta$	change in, e.g. in stress: $\Delta \sigma$
$\varepsilon$	linear strain
$\varepsilon_v$	volumetric strain
$\eta$	coefficient of viscosity
$\nu$	Poisson's ratio
$\sigma$	total stress
$\sigma'$	effective stress ( $\sigma' = \sigma - u$ )
$\sigma'_{vo}$	initial 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
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

### III. SOIL PROPERTIES

#### (a) Index Properties

$\rho(\gamma)$	bulk density (bulk unit weight)*
$\rho_d(\gamma_d)$	dry density (dry unit weight)
$\rho_w(\gamma_w)$	density (unit weight) of water
$\rho_s(\gamma_s)$	density (unit weight) of solid particles
$\gamma'$	unit weight of submerged soil ( $\gamma' = \gamma - \gamma_w$ )
$D_R$	relative density (specific gravity) of solid particles ( $D_R = \rho_s / \rho_w$ ) (formerly $G_s$ )
e	void ratio
n	porosity
S	degree of saturation

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

w	water content
$w_l$ or LL	liquid limit
$w_p$ or PL	plastic limit
$I_p$ or PI	plasticity index = $(w_l - w_p)$
$w_s$	shrinkage limit
$I_L$	liquidity index = $(w - w_p) / I_p$
$I_C$	consistency index = $(w_l - w) / I_p$
$e_{max}$	void ratio in loosest state
$e_{min}$	void ratio in densest state
$I_D$	density index = $(e_{max} - e) / (e_{max} - e_{min})$ (formerly relative density)

#### (b) Hydraulic Properties

h	hydraulic head or potential
q	rate of flow
v	velocity of flow
i	hydraulic gradient
k	hydraulic conductivity (coefficient of permeability)
j	seepage force per unit volume

#### (c) Consolidation (one-dimensional)

$C_c$	compression index (normally consolidated range)
$C_r$	recompression index (over-consolidated range)
$C_s$	swelling index
$C_\alpha$	secondary compression index
$m_v$	coefficient of volume change
$C_v$	coefficient of consolidation (vertical direction)
$C_h$	coefficient of consolidation (horizontal direction)
$T_v$	time factor (vertical direction)
U	degree of consolidation
$\sigma'_p$	pre-consolidation stress
OCR	over-consolidation ratio = $\sigma'_p / \sigma'_{vo}$

#### (d) Shear Strength

$\tau_p, \tau_r$	peak and residual shear strength
$\phi'$	effective angle of internal friction
$\delta$	angle of interface friction
$\mu$	coefficient of friction = $\tan \delta$
$c'$	effective cohesion
$C_u, S_u$	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  
2

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

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





## 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	

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

PROJECT		7-1191-0020		RECORD OF BOREHOLE No LG-2				1 OF 1 METRIC									
W.P.		5403-05-00		LOCATION		N 5048570.3; E 238892.2		ORIGINATED BY SA									
DIST		HWY 69		BOREHOLE TYPE		Portable Equipment, NW Casing, Wash Boring		COMPILED BY EC									
DATUM		Geodetic		DATE		March 5, 2012		CHECKED BY AB									
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									WATER CONTENT (%)
194.5	GROUND SURFACE																
0.0	PEAT (Fibrous) Very soft Black Wet		1	SS	WH	▽	194										
			2	SS	WH		193										
193.1	PEAT (Amorphous) Very soft Black Wet		3	SS	WH		192										
1.4			4	SS	WH		191										
191.3	SILTY CLAY Very soft Grey Wet		5	SS	WH		191										
3.2			6	SS	WR		190										
			7	SS	WR		189										
							188										
188.8	Silty SAND to SAND and GRAVEL, trace clay Compact to very dense Brown Wet		8	SS	18		188										
5.7							187										
			9	SS	37		186										
			10	SS	50/0.1												
185.5	Cobbles inferred from spoon sampling and casing grinding below 8.5 m depth.		11	SS	87/0.1												
9.0	END OF BOREHOLE SPOON REFUSAL																
	Note:  1. Water level at a depth of 0.2 m below ground surface (Elev. 194.3 m) upon completion of drilling.																

SUD-MTO 001 07-1191-0020-9100 CULVERTS BH LOGS.GPJ GAL-MISS.GDT 18/12/12 DATA INPUT:

PROJECT		07-1191-0020		<b>RECORD OF BOREHOLE No LG-3</b>		1 OF 1 <b>METRIC</b>										
W.P.		5403-05-00		LOCATION		N 5048594.9; E 238906.8										
DIST		HWY 69		BOREHOLE TYPE		108mm I.D. Continuous Flight Hollow Stem Augers, NW Casing, Wash Boring										
DATUM		Geodetic		DATE		March 6, 2011										
				ORIGINATED BY		EHS										
				COMPILED BY		EC										
				CHECKED BY		AB										
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT $\gamma$	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa				WATER CONTENT (%)				
198.1	ROAD SURFACE															
0.9	ASPHALT		1	AS	-											
	Sand and gravel to sand, trace to some silt, cobbles below 0.8 m depth (FILL)		2	AS	-											
	Very Dense		3	SS	50/10.1											
	Brown															
	Moist to wet															
196.2																
1.9	SAND, some silt, trace gravel		4	SS	64											
	Very dense															
	Grey															
	Wet															
195.2																
2.9	GNEISS (BEDROCK)		1	RC	REC 100%											RQD = 100%
	Bedrock cored from 2.9 m depth to 5.9 m depth.		2	RC	REC 100%											RQD = 100%
	For coring details see Record of Drillhole LG-3.		3	RC	REC 100%											RQD = 94%
192.2																
5.9	END OF BOREHOLE															
	Note:															
	1. Water level at a depth of 1.2 m below ground surface (Elev. 196.9 m) upon completion of drilling.															

SUD-MTO 001 07-1191-0020-9100 CULVERTS BH LOGS.GPJ GAL-MISS.GDT 18/12/12 DATA INPUT:

PROJECT: 07-1191-0020

**RECORD OF DRILLHOLE: LG-3**

SHEET 1 OF 1

LOCATION: N 5048594.9 ; E 238906.8

DRILLING DATE: March 6, 2011

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME-55

DRILLING CONTRACTOR: Landcore Drilling

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	COLOUR FLUSH % 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 METRES	DISCONTINUITY DATA										HYDRAULIC CONDUCTIVITY		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	k, cm/s																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																
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DEPTH SCALE




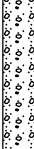
1 : 50



LOGGED: EHS

CHECKED: AB

SUD-RCK 07-1191-0020-9100 CULVERTS BH LOGS.GPJ GAL-MISS.GDT 18/12/12 DATA INPUT:

PROJECT 07-1191-0020			RECORD OF BOREHOLE No S6-2			1 OF 1 METRIC												
W.P. 5403-05-00			LOCATION N 5048541.8; E 238862.7			ORIGINATED BY MR												
DIST HWY 69			BOREHOLE TYPE Portable Equipment, NW Casing Wash Boring			COMPILED BY MM												
DATUM Geodetic			DATE February 27, 2008			CHECKED BY AB												
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 (%)			γ	GR SA SI CL	
								20 40 60 80 100	20 40 60 80 100	W <sub>p</sub> W W <sub>L</sub>	10 20 30							
194.3 0.0	GROUND SURFACE PEAT (Amorphous) Very soft Black Wet		1	SS	WH		194											
			2	SS	1		193											
			3	SS	WH		192											
			4	SS	WH		191											
191.4 2.9	SILTY CLAY Very soft to soft Grey Wet		5	SS	WH		190											
							189											
190.0 4.3	SAND and GRAVEL Compact Grey Wet		6	SS	25													
188.4 5.9	End of Borehole Spoon Refusal  Notes:  1. Water level at ground surface upon completion of drilling.  2. Split spoon samples obtained by driving with a 1/3 weight hammer; SPT 'N' values have been adjusted to the inferred values that would be obtained using a standard weight hammer.		8	SS	60/0.15													

SUD-MTO 001 07-1191-0020 S6 (SBL SITE 9 RD\_C76)METRIC.GPJ GAL-MISS.GDT 2608/11 DATA INPUT:

PROJECT		07-1191-0020		<b>RECORD OF BOREHOLE No S6-3</b>				1 OF 1 <b>METRIC</b>					
W.P.		5403-05-00		LOCATION		N 5048553.4; E 238884.1		ORIGINATED BY		MR			
DIST		HWY 69		BOREHOLE TYPE		Portable Equipment, NW Casing Wash Boring		COMPILED BY		MM			
DATUM		Geodetic		DATE		February 26, 2008		CHECKED BY		AB			
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT		UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		WATER CONTENT (%)			
194.3	GROUND SURFACE												
0.0	PEAT (Fibrous) Very soft Black Wet		1	SS	WH							248.7	
193.5	PEAT (Amorphous) Very soft Black Wet		2	SS	WH							626.1	
0.8			3	SS	WH								
			4	SS	WH								
			5	SS	WH							190.8	
190.6	SILTY CLAY Very soft Grey Wet		6	SS	WH								
3.7													
189.9	SAND, trace to some silt, trace clay, trace gravel Very loose Grey Wet		7	SS	1								
4.4													
189.1	SILTY CLAY to CLAY Very soft to soft Grey Wet		8	SS	1							50.87	
5.2													
			9	TO	PM								
186.6	Gravelly SAND, some silt Compact to dense Grey Wet		10	SS	11								
7.7			11	SS	15								
			12	SS	17								
			13	SS	11								
			14	SS	30								
			15	SS	32								
			16	SS	27								
182.0	End of Borehole												
12.3	Notes:  1. Water level at ground surface upon completion of drilling.  2. Split spoon samples obtained by driving with a 1/3 weight hammer; SPT 'N' values have been adjusted to the inferred values that would be obtained using a standard weight hammer.												

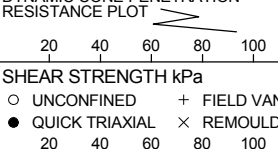
SUD-MTO 001 07-1191-0020 S6 (SBL SITE 9 RD\_C76)METRIC.GPJ GAL-MISS.GDT 2608/11 DATA INPUT:



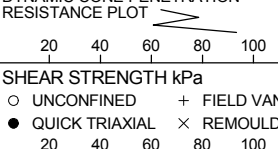
PROJECT 07-1191-0020			RECORD OF BOREHOLE No S6-4			1 OF 1 METRIC															
W.P. 5403-05-00			LOCATION N 5048560.9; E 238903.1			ORIGINATED BY MR															
DIST _____ HWY 69			BOREHOLE TYPE Portable Equipment, NW Casing Wash Boring			COMPILED BY MM															
DATUM Geodetic			DATE February 28, 2008			CHECKED BY AB															
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 (%)			γ			GR SA SI CL		
194.5	GROUND SURFACE							20 40 60 80 100	20 40 60 80 100	W <sub>p</sub> W W <sub>L</sub>	10 20 30										
0.0	PEAT (Fibrous) Very soft to soft Black Wet		1	SS	2		194														
193.7	PEAT (Amorphous) Very soft to soft Black Wet		2	SS	1		193														
0.8			3	SS	1		192														
			4	SS	1		191														
			5	SS	2		190														
190.8	SILTY CLAY Very soft to firm Grey Wet		6	SS	1		189														
3.7							188														
							187														
							186														
							185														
							184														
189.0	Silty SAND, trace clay, trace gravel Very loose Grey Wet		7a	SS	3																
188.7	CLAY Soft Grey Wet		7b	SS	3																
5.8																					
187.9	Silty SAND, some gravel, trace clay Very loose to dense Brown to grey Wet		8	SS	21																
6.6			9	SS	11																
			10	SS	3																
			11	SS	13																
			12	SS	9																
			13	SS	33																
183.9	End of Borehole Spoon Refusal																				
10.6	Notes:  1. Water level at ground surface upon completion of drilling.  2. Split spoon samples obtained by driving with a 1/3 weight hammer; SPT 'N' values have been adjusted to the inferred values that would be obtained using a standard weight hammer.																				

SUD-MTO 001 07-1191-0020 S6 (SBL SITE 9 RD\_C76)METRIC.GPJ GAL-MISS.GDT 2608/11 DATA INPUT:

PROJECT <u>07-1191-0020</u>		<b>RECORD OF PENETRATION TEST No LG-DC1</b>		1 OF 1 <b>METRIC</b>	
W.P. <u>5403-05-00</u>		LOCATION <u>N 5048606.7 ;E 238905.6</u>		ORIGINATED BY <u>EHS</u>	
DIST <u>          </u> HWY <u>69</u>		BOREHOLE TYPE <u>DYNAMIC CONE PENETRATION TEST</u>		COMPILED BY <u>EC</u>	
DATUM <u>Geodetic</u>		DATE <u>March 5, 2012</u>		CHECKED BY <u>AB</u>	

SOIL PROFILE		SAMPLES				GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 	PLASTIC LIMIT $w_p$ NATURAL MOISTURE CONTENT $w$ LIQUID LIMIT $w_L$ WATER CONTENT (%)	UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
196.7 0.0	GROUND SURFACE START OF DCPT						196				
196.0 0.7	END OF DCPT REFUSAL TO FURTHER PENETRATION (HAMMER BOUNCING)										

PROJECT <u>07-1191-0020</u>		<b>RECORD OF PENETRATION TEST No LG-DC2</b>		1 OF 1 <b>METRIC</b>	
W.P. <u>5403-05-00</u>		LOCATION <u>N 5048599.6 ;E 238917.6</u>		ORIGINATED BY <u>EHS</u>	
DIST <u>          </u> HWY <u>69</u>		BOREHOLE TYPE <u>DYNAMIC CONE PENETRATION TEST</u>		COMPILED BY <u>EC</u>	
DATUM <u>Geodetic</u>		DATE <u>March 5, 2012</u>		CHECKED BY <u>AB</u>	


SOIL PROFILE		SAMPLES				GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 	PLASTIC LIMIT $w_p$ NATURAL MOISTURE CONTENT $w$ LIQUID LIMIT $w_L$ WATER CONTENT (%)	UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
197.6 0.0	GROUND SURFACE START OF DCPT						197				
196.5 1.1	END OF DCPT REFUSAL TO FURTHER PENETRATION (HAMMER BOUNCING)										

SUD-MTO 002 07-1191-0020-9100 CULVERTS BH LOGS.GPJ GAL-MISS.GDT 27/03/12 DATA INPUT:

PROJECT <u>07-1191-0020</u>		<b>RECORD OF PENETRATION TEST No S6-DC1</b>		1 OF 1 <b>METRIC</b>	
W.P. <u>5403-05-00</u>		LOCATION <u>N 5048573.7; E 238881.6</u>		ORIGINATED BY <u>MR</u>	
DIST <u>          </u> HWY <u>69</u>		BOREHOLE TYPE <u>Dynamic Cone Penetration Test</u>		COMPILED BY <u>MM</u>	
DATUM <u>Geodetic</u>		DATE <u>February 26, 2008</u>		CHECKED BY <u>AB</u>	

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  kN/m <sup>3</sup>	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		
194.7 0.0	GROUND SURFACE																	
191.0 3.7	End of DCPT Refusal to Further Penetration (109 Blows/0.08 m)																	

SUD-MTO 001 07-1191-0020 S6 (SBL\_SITE 9 RD\_C76)METRIC.GPJ GAL-MISS.GDT 2608/11 DATA INPUT:

PROJECT <u>07-1191-0020</u>		<b>RECORD OF PENETRATION TEST No S6-DC2</b>				1 OF 1 <b>METRIC</b>				
W.P. <u>5403-05-00</u>		LOCATION <u>N 5048529.1; E 238884.3</u>				ORIGINATED BY <u>MR</u>				
DIST <u>          </u> HWY <u>69</u>		BOREHOLE TYPE <u>Dynamic Cone Penetration Test</u>				COMPILED BY <u>MM</u>				
DATUM <u>Geodetic</u>		DATE <u>February 27, 2008</u>				CHECKED BY <u>AB</u>				
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT  SHEAR STRENGTH kPa ○ UNCONFINED    + FIELD VANE ● QUICK TRIAXIAL    × REMOULDED	PLASTIC LIMIT W <sub>p</sub> NATURAL MOISTURE CONTENT W LIQUID LIMIT W <sub>L</sub> WATER CONTENT (%)	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE						
194.3 0.0	GROUND SURFACE									
190.8 3.5	End of DCPT Refusal to Further Penetration (105 Blows/0.18 m)									

SUD-MTO 001 07-1191-0020 S6 (SBL\_SITE 9 RD\_C76)METRIC.GPJ GAL-MISS.GDT 2608/11 DATA INPUT:



# **APPENDIX B**

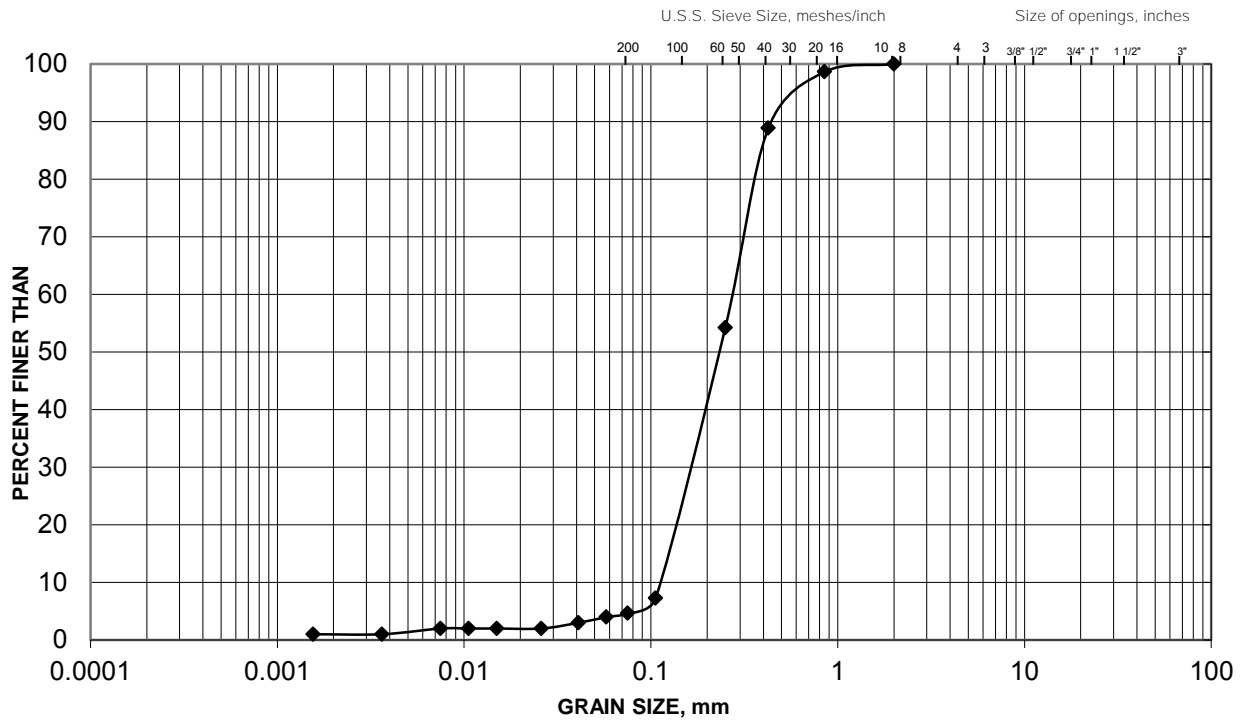
## **Laboratory Test Results**

# GRAIN SIZE DISTRIBUTION

**Sand (FILL)**  
**Highway 69 NBL and SBL STA 11+198**

**FIGURE**

**B1**



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

## LEGEND

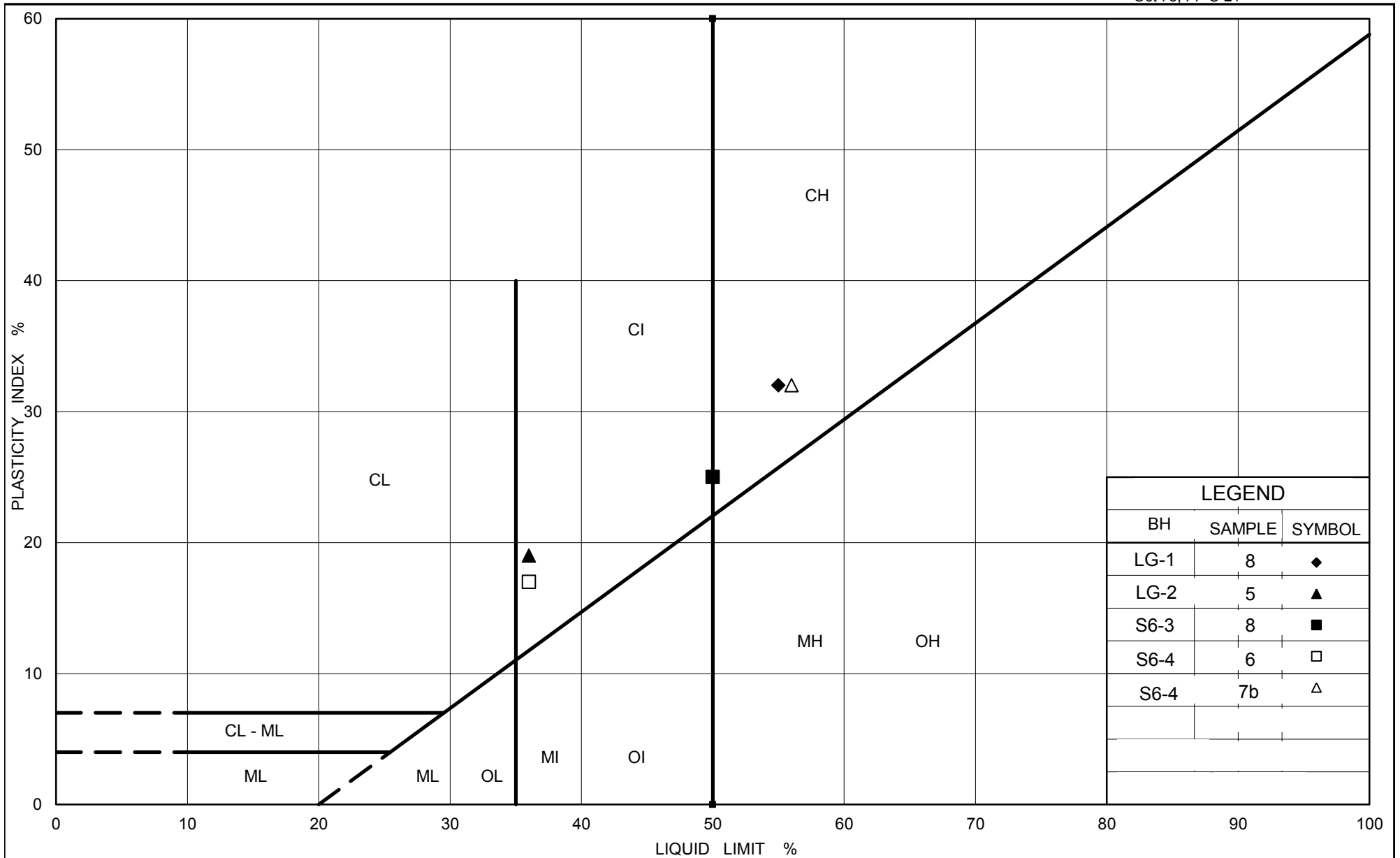
SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
—◆—	LG-1	4	196.3

Project Number: 07-1191-0020

Checked By: EC/AB

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Date: January 2013



Ministry of Transportation  
Ontario

**PLASTICITY CHART**  
Silty Clay to Clay  
Highway 69 NBL and SBL STA 11+198

Figure B2

Project No. 07-1191-0020

Checked By: EC/AB

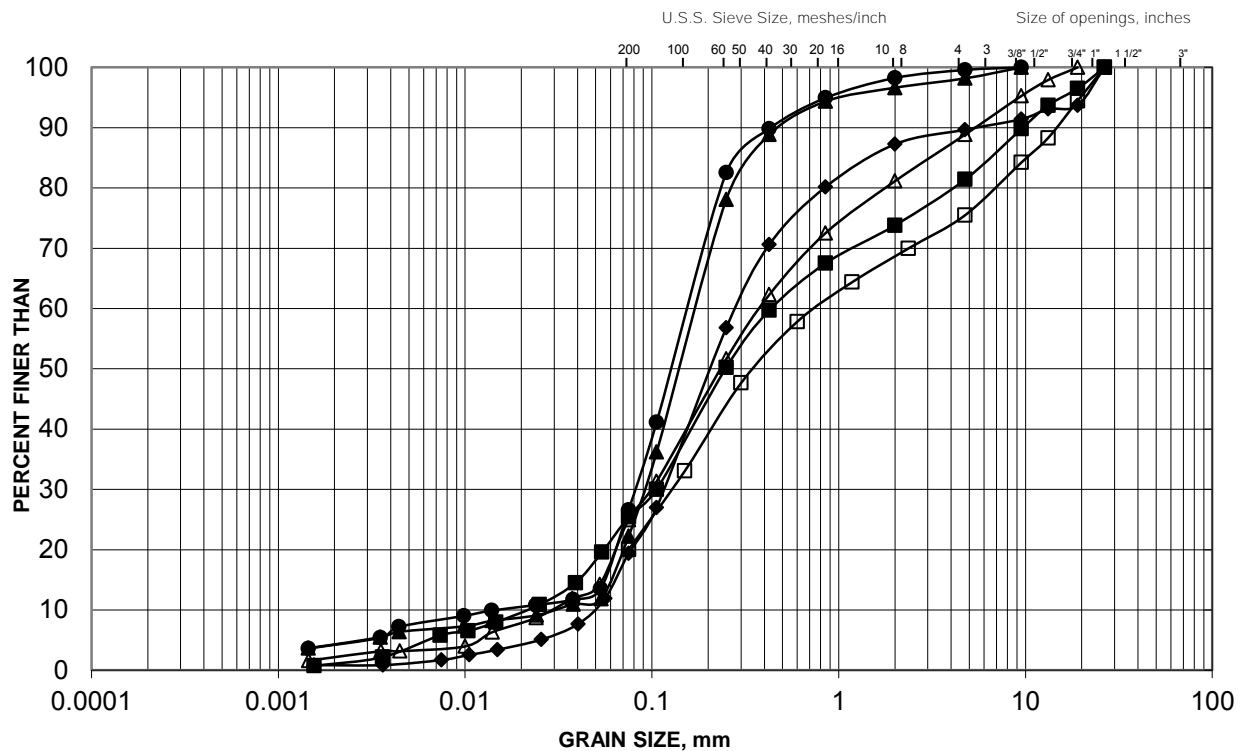


# GRAIN SIZE DISTRIBUTION

Silty Sand to Gravelly Sand  
Highway 69 NBL and SBL STA 11+198

FIGURE

B3



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

## LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
◆	LG-1	10	190.2
■	LG-2	8	188.1
▲	S6-3	7	189.4
●	S6-4	7a	188.8
△	S6-4	10	186.0
□	S6-3	13	184.1

Project Number: 07-1191-0020

Checked By: EC/AB

**Golder Associates**

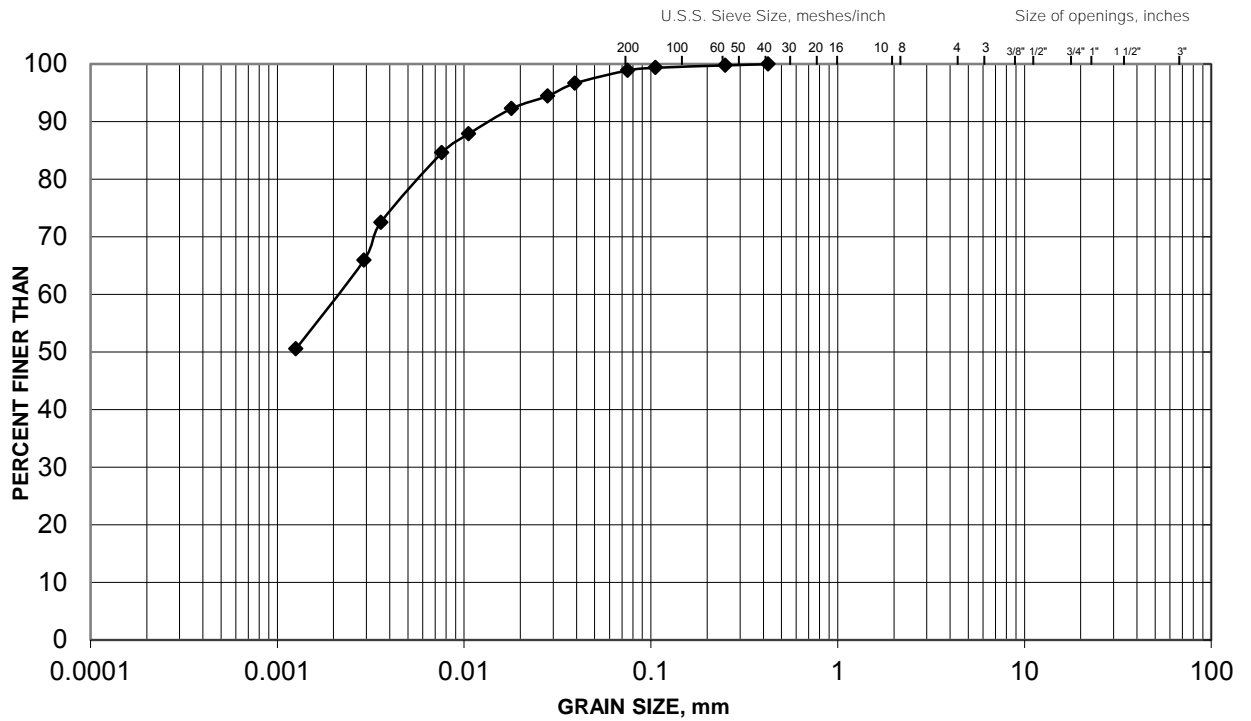
Date: January 2013

# GRAIN SIZE DISTRIBUTION

**Silty Clay**  
**Highway 69 NBL and SBL STA 11+198**

**FIGURE**

**B4**



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

## LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
—◆—	S6-3	6	190.2

Project Number: 07-1191-0020

Checked By: EC/AB

**Golder Associates**

Date: January 2013

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