



**August 2, 2011**

## **FOUNDATION INVESTIGATION AND DESIGN REPORT**

**POND CROSSING - PHASE 2  
HIGHWAY 69 FOUR-LANING  
FROM 0.4 KM NORTH OF HIGHWAY 7182  
(SHEBESHEKONG ROAD)  
NORTHERLY 11 KM  
MINISTRY OF TRANSPORTATION, ONTARIO  
GWP 5005-08-00**

**Submitted to:**  
MMM Group  
100 Commerce Valley Drive West  
Thornhill, Ontario  
L3T 0A1

**REPORT**



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**GEOCRES No.: 41H-82**

**Report Number:** 07-1191-0020-2

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## Table of Contents

### **PART A – FOUNDATION INVESTIGATION REPORT**

<b>1.0 INTRODUCTION.....</b>	<b>1</b>
<b>2.0 SITE DESCRIPTION.....</b>	<b>1</b>
<b>3.0 INVESTIGATION PROCEDURES .....</b>	<b>2</b>
<b>4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS .....</b>	<b>3</b>
4.1 Regional Geology .....	3
4.2 Subsurface Conditions.....	4
<b>5.0 CLOSURE .....</b>	<b>6</b>

### **PART B – FOUNDATION DESIGN REPORT**

<b>6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS.....</b>	<b>9</b>
6.1 General.....	9
6.2 Embankment over the Pond .....	9
6.2.1 Embankment Fill Types and Benching Requirements .....	10
6.2.2 Stability .....	11
6.2.2.1 Methodology .....	11
6.2.2.2 Parameter Selection .....	11
6.2.3 Settlement.....	12
6.2.3.1 Methodology .....	12
6.2.3.2 Parameter Selection .....	12
6.2.3.3 Settlement of Embankment Fill.....	14
6.3 Stability and Settlement Mitigation Options .....	15
6.3.1 Full Sub-Excavation .....	15
6.3.2 Preloading (with Toe Berms and/or Staged Construction) .....	16
6.3.3 Surcharging (with Toe Berms and/or Staged Construction).....	17
6.3.4 Wick Drains.....	17
6.3.5 Lightweight Fill .....	18
6.3.6 Instrumentation and Monitoring.....	18
6.4 Results of Analysis .....	18



## FOUNDATION REPORT - POND CROSSING - PHASE 2 HIGHWAY 69 GWP 5005-08-00

6.4.1.1	Stability .....	19
6.4.1.2	Settlement .....	19
6.4.1.3	Mitigation of Stability Issues and/or Time Dependent Settlements .....	20
6.5	Subgrade Preparation and Embankment Construction.....	21
6.5.1	Removal of Organics .....	22
6.5.2	Excavation and Replacement of Soft Subsoils.....	22
6.5.3	Groundwater and Surface Water Control .....	22
6.5.4	Backfilling.....	22
6.5.5	Embankment Fill Placement .....	23
6.5.6	Embankment Platform Widening.....	23
7.0	CLOSURE .....	23

### TABLES

Table 1	Evaluation of Stability/Settlement Mitigation Options
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### DRAWINGS

Drawing 1	Borehole Location and Soil Strata
Drawing 2	Soil Strata

### FIGURES

Figure 1	Pond 1 – Hwy 69 NBL, Slope Stability (Toe Berm)
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### APPENDICES

#### Appendix A Record of Boreholes

List of Symbols and Abbreviations

Record of Boreholes	P1-1, P1-3 to P1-5 and P1-7 to P1-15
Record of Penetration Test	P1-DC1 and P1-DC3

#### Appendix B Laboratory Test Results

Figure B-1	Grain Size Distribution – Silty Sand
Figure B-2	Plasticity Chart- Clayey Silt to Silty Clay
Figure B-3	Grain Size Distribution –Silty Clay
Figure B-4	Oedometer Consolidation Summary
Figure B-5	Grain Size Distribution – Sand and Gravel



# **PART A**

**FOUNDATION INVESTIGATION REPORT**  
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## **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 one (1) pond crossing for the proposed Highway 69 northbound lane (NBL) road embankment. 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 for the scope of work for the foundation investigation are outlined in MTO's Request for Proposal dated March 28, 2007. Golder's proposal (P7-1191-0020, dated April 24, 2007) for foundation engineering services associated with the pond and swamp crossings is contained in Section 6.8 of MMM's Technical Proposal that forms part of the Consultant's Agreement (Purchase Order Number 5006-E-0031) for this project. 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 new alignment of Highway 69 was provided to Golder by MMM in October 2008.

This report addresses the investigation carried out for the Phase 2 pond crossing. Separate reports will be submitted for the foundation investigations for Phase 1 swamp and pond crossings, culverts and bridge structures.

The purpose of this investigation is to establish the subsurface conditions along the proposed roadway alignment for the Phase 2 pond crossing by borehole drilling, in situ testing and laboratory testing on selected samples. The centreline of the proposed highway was staked in the field by MMM and the limits of the pond were reviewed by Golder in the field.

## **2.0 SITE DESCRIPTION**

The proposed Highway 69 NBL embankment crossing the pond is located between Stations 14+450 and 14+550 in the Township of Harrison, approximately 600 m south of Sucker Creek and about 250 m east of the existing Highway 69 alignment. The proposed NBL embankment will be up to about 2.6 m above existing shoreline grade at this pond crossing.

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. At the pond crossing location, the shoreline is about 1 m to 2 m above the pond water level and is relatively flat inland towards the area of the SBL. The ice surface at the borehole locations at the time of drilling ranged between Elevation 199.8 m and 200.4 m, while the ground surface at the boreholes (including below ice/water) ranges between 198.8 m and 202.0 m.



### **3.0 INVESTIGATION PROCEDURES**

The investigation for the pond crossing was carried out on February 22 and 25, 2008 and February 9 and 10, 2009 during which time a total of 13 boreholes (Boreholes P1-1, P1-3 to P1-5 and P1-7 to P1-15) and 2 Dynamic Cone Penetration Tests (DCPTs) (DCPTs P1-DC1 and P1-DC3) were advanced generally along or near the centreline and the toes of the proposed embankment. The field investigation was carried out using Portable Equipment supplied and operated by Walker Drilling Ltd. of Utopia, Ontario and OGS Inc. of Almonte, Ontario. The locations of the boreholes and DCPTs are shown in plan on Drawing 1 following the text of this report

The boreholes were advanced through the overburden using NW or BW casing. Hand excavation/sampling methods were used, as appropriate, depending on the terrain and anticipated thickness of overburden. Soil samples were obtained continuously or at intervals of depth of about 0.75 m and 1.5 m, using a 50 mm outer diameter (O.D.) split-spoon sampler, performed in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586-08a), employing  $\frac{1}{2}$  weight hammers lifted manually to the SPT height. The 'N'-values recorded were then corrected for the lower energy drive. A sample of the cohesive soils was obtained using a 48 mm O.D. Shelby tube (ASTM D1587) where BW casing was used to advance the boreholes. Field vane shear tests were conducted in cohesive soils for determination of undrained shear strengths (ASTM D2573-08) using MTO Standard 'N' size and 'B' size vanes. All boreholes were backfilled with bentonite upon completion in accordance with Ontario Reg. 903 (as amended by Ontario Reg. 372).

The boreholes and DCPTs were advanced to depths up to 6.6 m below existing ground surface or pond bottom (up to 8.1 m below ice/water surface) and were terminated on refusal to further casing and/or split-spoon advancement or cone penetration. These depths to refusal do not confirm bedrock surface elevations, but may be inferred to indicate potential proximity to the bedrock surface. At various borehole locations where refusal was encountered at shallow depth, the bedrock was exposed by hand shovel excavation to confirm the refusal condition.

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 in Appendix A. It should be noted that groundwater elevations as encountered in the boreholes may not be representative of static groundwater levels since most boreholes were advanced through ice/water columns and because the groundwater levels in the boreholes may not have stabilized on completion of drilling. Furthermore, groundwater elevations will vary depending on ponded water conditions and seasonal fluctuations, precipitation and local soil permeability.

The fieldwork was observed 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 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 soil samples. In addition, one (1) one-dimensional consolidation (oedometer) test was carried out on a sample of the cohesive deposit.

The centreline of the highway was surveyed and staked in the field by MMM prior to drilling. 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 given on the Record of Borehole and DCPT 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.



## FOUNDATION REPORT - POND CROSSING - PHASE 2 HIGHWAY 69 GWP 5005-08-00

The borehole and dynamic cone penetration test locations and ground surface elevations are as follows:

BH/DCPT	Location		Ground Surface Elevation (m)	Drilled Depth (m)
	Northing	Easting		
P1-1	5051063.2	236953.5	200.0	0.9
P1-3	5051039.5	236961.3	200.0	0.2
P1-4	5051031.6	236977.6	199.8	1.7
P1-5	5051015.7	236969.2	202.0	0.1
P1-7	5050991.9	236976.8	199.8	0.6
P1-8	5050984.4	236994.2	199.8	0.5
P1-9	5050968.2	236984.5	199.8	0.9
P1-10	5051045.5	236973.1	199.8	4.3
P1-11	5051073.0	236982.7	200.3	1.6
P1-12	5051048.4	236988.2	200.3	8.1
P1-13	5051024.6	236993.5	200.3	7.2
P1-14	5050999.7	236999.1	200.3	2.3
P1-15	5050974.9	237004.7	200.4	0.8
P1-DC1	5051055.4	236969.8	199.8	3.5
P1-DC3	5051007.9	236985.6	199.8	1.1

## 4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

### 4.1 Regional Geology

As delineated in *The Physiography of Southern Ontario*<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. Localised 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 Special Volume 4<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> *Geology of Ontario*, 1991. Ontario Geological Society Special 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 (including hand shovel excavated overburden to expose the bedrock where boreholes encountered refusal at shallow depth, typically less than 0.3 m), together with the results of the laboratory tests carried out on selected soil samples, are presented on the Record of Borehole sheets included in Appendix A and the laboratory test sheets included in Appendix B. The stratigraphic boundaries shown on the Record of Borehole sheets are inferred from non continuous sampling, observations of drilling progress and the results of SPTs and in situ testing. These boundaries, therefore, represent transitions between soil types rather than exact planes of geological change. Further, subsurface conditions will vary between and beyond the borehole locations. The thickness of the overburden in the investigated areas as inferred from the resistance to DCPT results are shown on the Record of Penetration Test sheets in Appendix A, as applicable.

The inferred soil stratigraphy as encountered in the boreholes and DCPTs advanced in the pond crossing area is shown in profile on Drawings 1 and 2. It should be noted that the orientation (i.e. north, south, east, west) stated in the text of the report is typically referenced to project north (and up-chainage along the proposed Highway 69 alignment) and therefore may differ from the Magnetic North shown on the drawing.

In general, the subsurface soils along the NBL alignment in this pond area consist of a surficial layer of peat underlain by deposits of silty sand, sand and gravel and clayey silt to silty clay, to the refusal depths encountered. A detailed description of the surface conditions encountered in the boreholes is provided in the following sections.

### **Ice/Water**

Boreholes P1-4, P1-7, P1-8 and P1-10 to P1-15 were advanced from the ice surface and penetrated an ice/water layer to depths between 0.3 m and 1.5 m.

### **Peat**

A deposit of black, fibrous/amorphous peat was encountered in all boreholes except Borehole P1-15. The deposit was encountered below ice/water in Boreholes P1-4, P1-7, P1-8 and P1-10 to P1-14 and from ground surface in Boreholes P1-1, P1-3, P1-5 and P1-9. The thickness of the peat deposit is between 0.1 m and 4.2 m and the top of the deposit was encountered between Elevation 202.0 m and 198.8 m.

The SPT 'N'-values measured within the peat are between 0 blows (i.e. weight of hammer) and 6 blows per 0.3 m of penetration suggesting a very soft to firm consistency.

The measured water content on samples of the peat ranges between about 68 percent and 1175 percent.

### **Silty Sand**

A deposit of grey to brown silty sand was encountered underlying the peat in Boreholes P1-1, P1-9 and P1-10. The top of the deposit ranges between about Elevation 199.8 m and 198.6 m in these boreholes and the thickness of the deposit ranges from 0.6 m to 1.1 m. In Boreholes P1-1 and P1-9, the bottom of this deposit was defined by refusal to further split-spoon advancement.





The SPT 'N'-values measured within this deposit are 1 blow per 0.3 m of penetration and 1 and 23 blows per 0.15 m of penetration, indicating a very loose to dense relative density.

The grain size distributions of three samples of this deposit are presented on Figure B-1 in Appendix B. An Atterberg limits test on one (1) sample of this deposit indicates that the material is non-plastic.

The measured water content on samples of this deposit ranges between about 22 percent and 56 percent.

### **Clayey Silt to Silty Clay**

A deposit of grey clayey silt to silty clay, trace to some sand was encountered in Boreholes P1-10 to P1-13. The top of the deposit ranges from Elevation 199.1 m to 194.6 m and the thickness of the deposit ranges from 0.4 m to 2.2 m.

The SPT 'N'-values measured within the cohesive deposit range from 0 blows (i.e. weight of hammer) to 1 blow per 0.3 m of penetration and 1 blow per 0.15 m of penetration. In situ field vane testing carried out within this stratum measured undrained shear strengths ranging from about 9 kPa to 26 kPa. The results of the SPT 'N'-values together with the in situ field vanes indicate that the cohesive deposit has a very soft to firm consistency.

Atterberg limits tests were carried out on six (6) samples of the clayey silt to silty clay deposit. The test results indicate liquid limits ranging from 31 percent to 46 percent, plastic limits ranging from 15 percent to 20 percent and plasticity indices ranging from 12 percent to 24 percent. The results of the Atterberg limits tests are shown on the plasticity chart on Figure B-2 in Appendix B and indicate that the material is classified as a clayey silt of low plasticity to a silty clay of medium plasticity.

The grain size distribution of one sample from this deposit is shown on Figure B-3 in Appendix B.

The measured water content on samples of this deposit ranges between about 54 percent and 81 percent.

One laboratory consolidation test was carried out on a specimen of the silty clay obtained from Borehole P1-13 and the test results are shown on Figure B-4. The pre-consolidation pressure was estimated from the Void Ratio versus logarithmic Pressure plots using the Casagrande method as well as from the Total Work versus Pressure plots. The relevant consolidation test results are summarized below.

<b>Borehole / Sample Number</b>	<b>Elevation (m)</b>	<b><math>\sigma_{vo}'</math> (kPa)</b>	<b><math>\sigma_p'</math> (kPa)</b>	<b><math>\sigma_p' - \sigma_{vo}'</math> (kPa)</b>	<b>OCR</b>	<b><math>e_o</math></b>	<b><math>C_r</math></b>	<b><math>C_c</math></b>	<b><math>c_v^*</math> (cm<sup>2</sup>/s)</b>
P1-13 / 6	193.9	20	30	10	1.5	1.66	0.08	0.44	$3.9 \times 10^{-3}$

Note: <sup>\*</sup> For approximate stress range of  $15 \leq \sigma_v' \leq 113$  kPa  
where:  $\sigma_{vo}'$  effective overburden pressure in kPa  
 $\sigma_p'$  preconsolidation pressure in kPa  
OCR overconsolidation ratio  
 $e_o$  initial void ratio  
 $C_c$  compression index (based on void ratio)  
 $C_r$  recompression index (based on void ratio)  
 $c_v$  coefficient of consolidation in cm<sup>2</sup>/s in the normally consolidated range



### ***Sand and Gravel***

A deposit of grey sand and gravel was encountered underlying the peat in Borehole P1-14 and overlying the clayey silt in Borehole P1-12. The top of the sand and gravel deposit was encountered at Elevation 198.2 m and 192.7 m in the respective boreholes and the thickness of the deposit is 0.2 m and 0.5 m, respectively. The bottom of this deposit was defined by refusal to further split-spoon advancement in both boreholes.

The SPT 'N'-values measured within this deposit are 50 blows per 0.3 m of penetration and 23 blows per 0.3 m of penetration in Boreholes P1-14 and P1-12, respectively, indicating a compact to very dense relative density.

The grain size distribution of one sample of this deposit is presented on Figure B-5 in Appendix B.

The measured water content on one sample of this deposit is about 26 percent.

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

In all boreholes and in DCPTs P1-DC1 and P1-DC3, refusal to further split-spoon or casing advancement or cone penetration was encountered at depths between about 0.1 m and 8.1 m below ground/ice surface, corresponding to between Elevation 201.9 m and 192.2 m.

### ***Groundwater Conditions***

In general, the samples taken in the boreholes were wet with free water noted in select non-cohesive samples. Water levels observed in the boreholes upon completion of drilling were essentially at ground/ice surface, ranging from Elevation 200.4 m to 199.8 m. Boreholes P1-3, P1-5 and P1-9 were found to be dry upon completion of drilling. It should also be noted that the groundwater levels in the area are subject to seasonal fluctuations.

## **5.0 CLOSURE**

The field drilling program was carried out under the supervision of Mr. Indulis Dumpis and Mr. Trevor Moxam, field technicians with Golder, under the overall direction of Mr. André Bom, P.Eng. This report was prepared by Mr. Adam Wissink, EIT, and under the direction and review of Mr. André Bom, P.Eng. Mr. Jorge M. A. Costa, P.Eng., Golder's Designated MTO Contact for this project and Principal with Golder, conducted an independent quality control review of the report.



## Report Signature Page

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# PART B

FOUNDATION DESIGN REPORT  
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## **6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS**

This section of the report provides an interpretation of the geotechnical data obtained during the investigation and recommendations on the foundation aspects of design of the proposed works. The recommendations are intended for the guidance of the design engineer. Where comments are made on construction, they are provided to highlight aspects of construction that could affect the design of the project. Those requiring information on aspects of construction must 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**

Golder was retained by MMM to provide recommendations on foundation aspects for the detail design and construction of the new Highway 69 NBL embankment over the pond between Stations 14+450 and 14+550 in the Township of Harrison, approximately 600 m south of Sucker Creek and about 250 m east of the existing Highway 69 alignment. This report presents the results of embankment stability and settlement analyses and provides recommendations for stable embankment geometry and embankment fill materials and implementation of mitigation alternatives that may be required as a means to reduce settlements and to improve stability. The work also includes addressing specialized construction concerns and potential geotechnical problems associated with embankment construction, sub-excavating organic/soft materials and placement of new fill materials.

The overall project involves the design of an 11 km section of the new Highway 69 four-laning including six (6) structures (Site 9 Road Underpass structure, Highway 529 Interchange structure, Highway 529 Overpass structures and Sucker Creek structures) and several culverts and overhead signs.

### **6.2 Embankment over the Pond**

Based on the profiles provided by MMM, the new highway (NBL) embankment over the pond will be up to about 2.6 m high above existing grade (bottom of the pond) at the pond crossing. Sections 6.2.2 and 6.2.3 of this report summarize the methods used to analyze the stability and settlement of the embankment across the pond. Section 6.3 provides discussions on potential alternatives for mitigating stability and settlement. The results of the analyses and recommendations for mitigating stability and time-dependent settlements in the pond crossing area are presented in Section 6.4.

The subsoils encountered from ground surface or below (1.5 m) water generally consist of an organic deposit (peat) between 0.4 m and 4.2 m thick, underlain by deposits of very loose to compact silty sand between 0.6 m and 1.1 m thick and/or very soft to firm clayey silt to silty clay between 0.4 m and 2.2 m thick were encountered in places underlain by 0.2 m to 0.5 m of compact to very dense sand and gravel. The cohesive stratum is thickest and deepest at the east toe of the embankment (in the pond) at Station 14+525 with the bottom of the deposit encountered up to 6.1 m below the bottom of the pond (i.e. 7.6 m below the ice surface at the time of drilling).



Refusal to borehole advancement or dynamic cone penetration was encountered between depths of about 0.1 m and 8.1 m below ground/ice surface and at a greater depth at about STA 14+525, decreasing in depth to the north and south limits of the pond.

Details of the subsurface conditions for this pond crossing are presented in Section 4.2 and shown on Drawing 1.

The stability and settlement analysis assumes that the peat under the water and the organic soils (encountered at the ground surface) will be removed prior to construction of the new embankment. For design purposes, the groundwater level is based on the conditions observed during drilling. In general, the groundwater level used in the analysis is at about Elevation 200 m.

### **6.2.1 Embankment Fill Types and Benching Requirements**

Different embankment fill alternatives (i.e. rock fill and granular fill) provide relative advantages and disadvantages in terms of availability, weight (i.e. driving force and applied load to founding subsoils), construction cost and time, ease of construction and post-construction performance.

Rock fill is the preferred embankment fill material for this project due to the availability from rock blasting required elsewhere on the project and due to ease of placement of such material in sub-excavated areas under water. In this regard, the stability and settlement analyses discussed in Section 6.4 have been carried out on the basis that the majority of the roadway embankment specifically at this pond crossing will be constructed of rock fill.

#### **Rock Fill**

The main advantage of constructing an embankment using rock fill is the ability to achieve steeper side slopes (1.25H:1V), which is required in areas with limited right-of-way, reducing the overall quantity of material required for the project and placement of material in sub-excavated areas under water. Rock fill will also likely be available locally, either from excavations in deep cuts through existing bedrock outcrops within other phases of the project alignment or from rock borrow areas close to the project limits. The disadvantage of using rock fill for the construction of high embankments is that some post-construction settlement of the embankment fill itself will occur, although mostly within about the first year of construction. Settlement of the rock fill is discussed further in Section 6.2.3.3.

For rock fill embankments, the incorporation of 2 m wide successive bench(es) into the uniform side slope profile is required wherever the embankment will exceed a height of 10 m (in accordance to MTO Northeastern Region Guidelines). Given that the proposed new embankment in Phase 2 pond is less than 10 m high, 2 m wide benches are not required.



### **Granular Fill**

The main advantage of using granular fill for embankment construction is the ease of construction and negligible post-construction settlement within the embankment fill itself. However, this option will require a larger volume of fill and wider right-of-way because the side slopes of a granular fill embankment (2H:1V) are flatter than those of rock fill. For this project, acceptable granular fill is considered to be well-graded, locally available and/or imported sand and gravel material. Should granular fill be considered, a constraint limiting the fines content should be included in the Contract to reduce the potential for segregation of fines, if placed in sub-excavated areas under water. On this project, the embankment is not likely to be constructed of granular fill.

## **6.2.2 Stability**

The following sections outline the methodology and the parameters used in the analyses to evaluate embankment stability at the pond crossing. The results of the analyses are presented in Section 6.4 where they are discussed in combination with the results of the settlement analyses and recommendations regarding possible design and construction mitigation alternatives.

### **6.2.2.1 Methodology**

Stability analyses were performed for the critical section of the proposed fill embankment in the pond crossing. Critical sections correspond to the greatest new embankment height and/or the maximum thickness of soft, compressible cohesive soils. At this site, Station 14+525 is considered to be the critical section.

Limit equilibrium slope stability analyses were performed using the commercially available program GeoStudio 2007 (Version 7.13), produced by Geo-Slope International Ltd., employing the Morgenstern-Price method of analysis. For all analyses, the Factor of Safety (FoS) of numerous potential failure surfaces was computed in order to establish the minimum FoS. The FoS is defined as the ratio of the forces tending to resist failure to the driving forces tending to cause failure. A target minimum FoS of 1.3 is normally adopted for the design of embankment slopes under static conditions. This FoS is considered adequate for the embankment at this site 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 performed to check that the target minimum FoS was achieved for the critical section, but is equally applicable for the various embankment heights at the site.

### **6.2.2.2 Parameter Selection**

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 in situ 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 on the results of in situ field vane shear tests, inferred from the laboratory consolidation tests results, and estimated from correlations with the SPT results and other laboratory test data (natural water content).



Summarized below are the simplified stratigraphy and the associated strengths and unit weights employed for the different soil types in the pond area.

<b>Soil Type (West Embankment Area)</b>	<b>Unit Weight (kN/m<sup>3</sup>)</b>	<b>Undrained Shear Strength (kPa)</b>	<b>Angle of Internal Friction</b>
Rock Fill	19	--	40°
Peat	12	--	27°
Clayey Silt to Silty Clay	16	10	--
Silty Sand and Sand and Gravel	19	--	28°

### **6.2.3 Settlement**

The following sections outline the methods used to conduct the settlement analyses at the site. In addition, the parameters used in the analyses for the critical section are also presented. The results of the analyses are presented in Section 6.4 for the pond crossing where they are discussed in combination with the results of the stability analyses and possible design and construction mitigation alternatives.

#### **6.2.3.1 Methodology**

To estimate the magnitude of the expected settlements, analyses were carried out on the critical sections of the proposed fill embankments using the commercially available program Settle3D (Version 2.0) produced by Rocscience Inc. and hand calculations. The critical section corresponds to the greatest new embankment height and/or the maximum thickness of soft, compressible cohesive soils. The rate of settlement/consolidation of the cohesive foundation soils was assessed using Terzaghi's one-dimensional consolidation theory (Terzaghi and Peck, 1967).

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.

The thickness of the compressible foundation soils and the height of the embankment vary along the proposed alignment within the pond crossing and as such the settlements along the length of a given alignment will similarly vary. Given that the analyses were carried out in the critical section of the pond crossing, the settlements estimated will generally represent the maximum value along a given alignment.

#### **6.2.3.2 Parameter Selection**

The immediate compression of the silty sand and sand and gravel layers was modelled 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 CHBDC (2006) and adjusted, if necessary.





The consolidation settlement of the cohesive deposit was assessed using the results of the laboratory consolidation test and/or in situ field vane tests to estimate the deformation parameters for the clayey foundation soils.

The coefficient of consolidation,  $c_v$  ( $\text{cm}^2/\text{s}$ ), required in the time-rate analysis was established using the results of the consolidation test.

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 for the design life following the completion of primary settlement at the pond crossing.

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

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

where :

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

The following simplified stratigraphy (assuming the peat has been removed) and deformation parameters have been developed for and employed in the settlement analysis for the pond.

Material	Maximum Thickness (m)	Unit Weight ( $\text{kN/m}^3$ )	Estimated Deformation Properties
Rock Fill		19	Refer to Section 6.2.3.3
Clayey Silt to Silty Clay	2.2	16	(see below)
Silty Sand to Sand and Gravel	1.1	19	$E' = 5 \text{ MPa}$

The following consolidation parameters were estimated for the clayey silt to silty clay deposit based primarily on the results of a laboratory consolidation test performed on a specimen of the silty clay obtained from Borehole P1-13. These results were compared with values estimated from empirical correlations using the results of the in situ tests and laboratory index testing as described above.

Location	Elevation (m)	$\sigma_{v0}'$ (kPa)	$\sigma_p'$ (kPa)	OCR	$e_o$	$C_r$	$C_c$	$c_v$ ( $\text{cm}^2/\text{s}$ )	$C_{\alpha\epsilon}^*$
East Embankment Toe at Sta. 14+525 (BH P1-13)	195.3 to 193.1	20	30	1.5	1.66	0.08	0.44	$2.7 \times 10^{-3}$	0.0068

\* Based on average water content of deposit.



### 6.2.3.3 Settlement of Embankment Fill Rock Fill

Where rock fill is to be used for 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 Foundations Guideline "Post-Construction Rock Fill Settlement and Guidelines For Estimating Rock Fill Quantity", dated April 2010.

Rock fill should be placed, whenever possible, in a controlled manner (i.e. not end-dumped) in accordance with OPSS 206 (Grading). 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 Foundations Guideline (April 2010), as follows:

Total Height of Rock Fill, H	Short-Term Rock Fill Settlement	
	Compacted Rock Fill	Dumped Rock Fill
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

Approximately 90 percent of the short-term settlement may be expected to occur within the first six (6) months following 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 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 (April 2010), as follows:

Total Height of Rock Fill, H	Long-Term Rock Fill Settlement	
	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 over the life of the embankment.

## **6.3 Stability and Settlement Mitigation Options**

At the pond crossing, stability and settlement have been assessed based on existing subsurface conditions and proposed embankment fill height. The presence of weak/soft, compressible soils underlying a proposed embankment can lead to the potential for instability or unacceptably large settlements with the placement of high rock fills. There are a number of options for mitigating the potential for settlements and/or instability. A brief general discussion on these alternatives is given below.

Details of the stability/settlement mitigation options for the pond crossing are provided in Section 6.4. The advantages, disadvantages, relative costs and risks/consequences for the pond are summarized in the evaluation of stability/settlement mitigation options in Table 1.

### **6.3.1 Full Sub-Excavation**

Sub-excavation of the weak/soft and compressible (i.e. clayey) soils underlying the footprint of the proposed embankment in advance of the placement of rock fill is a viable option for improving the stability and controlling long-term settlement of the proposed embankments at this site. The removal of the soft, compressible cohesive soils would result in improved stability and significantly reduce settlements within the areas underlain by clayey deposits. The additional below-grade rock fill embankment should be constructed with the same side slope profile as that of the above-grade embankment (i.e. 1.25H:1V for rock fill) since this is the natural slope of rock fill and should not be affected by underwater placement. This option has the advantage that construction of the above-grade embankment could proceed upon completion of sub-excavation and replacement without concerns of instability. However, full sub-excavation will produce a larger volume of spoil material for disposal and will require a larger volume of rock fill replacement. The necessity to develop stable side slopes or back slopes within the excavation may result in slope geometries ranging from 1H:1V to as flat as 3H:1V. Flatter slopes would increase the lateral extent of the excavation and may require a wider right-of-way. Further, the increase in thickness would result in additional post-construction settlement of the embankment rock fill itself (see Section 6.2.3.3). For purposes of property requirements, a 3H:1V back slope should be assumed in the pond area where sub-excavation is required.



We understand that based on MTO field experience on similar highway construction projects, the practical maximum depths that can be reached with conventional and long-stick excavator equipment is about 6 m and 12 m, respectively, which should be suitable to sub-excavate the organic and underlying cohesive deposit in this pond area.

This option is most suited for areas where there is a limited thickness of organics (peat) and weak/soft compressible soils underlying the proposed embankment, making their removal feasible where there are no requirements for setbacks and adequate right-of-way is available, and where there are no conflicts with encroachment on existing adjacent features.

### **6.3.2 Preloading (with Toe Berms and/or Staged Construction)**

As an alternative to sub-excavation and replacement of the weak/soft, compressible foundation soils, preloading may be considered for improving the stability and reducing post-construction settlements of the proposed embankment. Preloading refers to the placement of fill to the proposed profile grade of the embankment (in one or more stages) in advance of pavement construction in order to preconsolidate the underlying compressible soils. The fill placed should be rock fill up to roadway subbase level overlain by Granular 'B' Type II (Special Provision 110S13) for the roadway profile grade. Preloading reduces the magnitude of long-term, post-construction settlements by promoting such settlements to occur under embankment fill loads in advance of final grading of the embankment. It also increases the strength of the clayey subsoils underlying the embankment footprint, thereby improving stability.

Preloading requires placement of embankment fill and monitoring of settlements, and possibly pore pressures, for a period of time corresponding to approximately the 'End of Primary' (EoP) consolidation of clayey subsoils. EoP consolidation times will vary depending on the properties of the clayey subsoils, the thickness of the clayey deposits, and the height of the embankment. Once the estimated EoP consolidation has occurred, final grading for construction can proceed. Long-term secondary (creep) settlements will still continue to occur over the design life of the embankment; however, such settlements would be less than primary consolidation settlements. Where secondary (creep) settlements are considered to be large enough to affect the long-term performance of the roadway, these can be reduced by surcharging, as discussed in Section 6.3.3.

In areas where clayey deposits are thick and/or very soft, and where such conditions coincide with proposed high embankment fills, it may be necessary to construct stability berms along the embankment toes or to place the embankment fill in stages in layers of limited thickness to ensure that the stability of the embankment is maintained. Toe berms consist of rock fill buttresses placed against the toe of the proposed embankment fill, producing a stepped embankment cross-section geometry. This stepped configuration produces a similar effect (i.e. increased stability) as using flatter embankment slopes but often requires less fill material. Depending on the subsurface conditions and the proposed embankment height, toe berms will typically be on the order of about one third to one half of the height of the final embankment. The lateral extent (width) of toe berms will vary depending on the results of the stability analyses, but could range from half to one times the highway embankment height or greater. Where staged construction is required, the individual layers of fill would have limited thickness and each construction phase would be separated by a suitable time interval to allow pore pressures to dissipate and strength gain to occur in the underlying clayey soils while limiting the potential for instability of the embankment.



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

This option is most suited for areas where removal of clayey soils and their replacement with rock fill is not considered practical, where the thickness of the existing compressible soils is nominal (less than about 4 m) and where a delay in the construction schedule is acceptable or can be accommodated.

### **6.3.3     Surcharging (with Toe Berms and/or Staged Construction)**

Similar to preloading, surcharging refers to the placement of embankment fill in advance of final pavement construction to reduce long-term, post-construction settlements (including creep). The difference between preloading and surcharging is the amount of fill placed and the time required for consolidation to be achieved. With surcharging, the preload is placed as described above, followed by an additional lift of fill (the surcharge) above that required to construct the final embankment geometry. This additional lift of fill applies greater stress to the underlying clayey soils and increases the rate of primary consolidation over that achieved by preloading only, resulting in over-consolidation of the underlying compressible foundations soils. At the EoP consolidation, the portion of the surcharge fill remaining above the required embankment height (sub-base level) is removed. The surcharge fill can also be left in place for a longer duration to reduce the long-term, secondary (creep) settlements.

As with preloading alone, it may be necessary to construct toe berms or stage the placement of preload and surcharge to limit the potential for instability. Upon completion of primary consolidation, the removed surcharge may be re-used on other parts of the site.

Surcharging is most suited to those areas considered appropriate for preloading, where the stability of the higher surcharged embankment can be practically maintained by reasonably sized toe berms or staged construction, but where sufficient time for primary consolidation settlements to occur under preload fill loads alone is not available. Surcharging is also best suited for areas where large secondary (creep) settlements are expected.

### **6.3.4     Wick Drains**

Where sub-excavation is not practical (i.e. due to the thickness of or depth to the compressible soil deposits), but it is considered feasible to preload or surcharge the foundation subsoils, consideration may be given to installing wick drains in conjunction with preloading or surcharging to further accelerate the rate of primary consolidation. Wick drains are pre-fabricated geotextile drains installed vertically from ground surface into or through the soft, compressible foundation soils in order to increase the rate of excess pore pressure dissipation. Typically, wick drains are installed on a 1 m to 3 m triangular grid spacing over the embankment footprint.

Use of wick drains are most suited to areas with thick (i.e. greater than about 5 m) deposits of soft, compressible foundation soils and high proposed embankment fills where primary consolidation times are large even under surcharge conditions.

It would still be necessary to sub-excavate and remove surficial organics and place a granular drainage blanket at ground surface level prior to the installation of the wick drains.



### **6.3.5 Lightweight Fill**

Another alternative for reducing the magnitude of long-term settlement and improving stability in areas of soft, compressible foundation soils is to use lightweight fill, such as expanded polystyrene (EPS), for embankment construction.

The use of lightweight fill reduces the load applied to the foundation soils due to the low density of the fill materials. This in turn reduces the magnitude of post-construction settlement and reduces the potential for instability.

Lightweight fill is not considered a practical option for general use over large areas due to the expense and/or shipping costs for the supply of these types of fills. Rather, lightweight fill is most suited for areas underlain by deep compressible subsoil conditions, where sub-excavation is not practical or feasible, and where there is no available time in the construction schedule for a preload or surcharge period (typically near bridge structures).

### **6.3.6 Instrumentation and Monitoring**

If the preloading, or preload with a surcharge, option is adopted, the magnitude and time rate of settlement as well as dissipation of pore pressures during and after construction of the embankment over the pond should be assessed with monitoring instrumentation. Such monitoring would consist of installing settlement pins (SPs), settlement rods (SRs) and vibrating wire piezometers (VWPs) below the embankment and taking regular measurements/readings at given intervals of time during and after construction of the embankment for the duration of the preloading/surcharging period. In addition, standpipe piezometers (SPPs) may be required and are usually installed to provide background pore pressure readings for the vibrating wire piezometers. This monitoring instrumentation is particularly important where it is considered necessary to carefully monitor the stability of the subsoils during staged placement of fill, and also to provide confirmation of the time for surcharge removal.

The extent of instrumentation and the frequency of monitoring required will depend on the foundation treatment alternative chosen for a given site and the height of the proposed embankment fill. Specifications for the type, number and layout of the instrumentation, together with the supply, installation, protection and monitoring should be included as Special Provisions in the Contract.

## **6.4 Results of Analysis**

The results of the stability and settlement analyses for the pond crossing are provided in the following sections. In addition, the options and recommendations for achieving the target FoS for the required embankment geometry and for minimizing the time-dependent, post-construction settlements are also discussed.

The area extending from about STA 14+450 to 14+550 along the proposed Highway 69 NBL alignment through a pond requires a new embankment up to 2.6 m high (relative to the bottom of the pond) to achieve the proposed vertical profile. The topography of this section of proposed highway is generally flat and low-lying, with the pond generally located along the east side/toe of the proposed NBL embankment.





As indicated in Section 6.2 on embankment fill types, the new embankment was analyzed assuming a rock fill composition and 1.25H:1V side slopes. The stability and settlement analysis assumes that the up to 4.2 m of organic soils encountered at the site under the embankment footprint have been removed (in accordance with OPSD 203.010) prior to construction of the new embankment.

#### **6.4.1.1 Stability**

As indicated in previous sections, based on the results of the subsurface investigation and review of the profile drawings, the critical section (i.e. greatest embankment height and/or maximum thickness of soft, compressible foundation soils) for this pond crossing is located at about STA 14+525. The stability analysis performed on the critical section indicates that after the completion of embankment construction (including removal and replacement of the very soft organic deposits), the embankment will have a FoS less than 1.3 for a deep-seated, global failure surface that would impact the operation of the roadway. The stability analysis performed also indicates that a FoS 1.3 or greater cannot be achieved by staged construction methods.

To achieve a FoS equal to or greater than 1.3 for the proposed 2.6 m high embankment will require mitigation measures consisting of either full sub-excavation of the cohesive deposit or the construction of rock fill berms along the toes of the embankment.

#### **6.4.1.2 Settlement**

To estimate the magnitude of the expected settlements due to new construction, analyses were carried out on the critical section representative of the subsurface conditions within the pond crossing at STA 14+525. For the condition where the organic materials are sub-excavated and replaced with rock fill but the cohesive deposits are left in place, it is estimated that the settlement of the foundation soils in the critical section will be about 220 mm. This total settlement is estimated to comprise about 20 mm of immediate settlement due to compression of the cohesionless deposits and about 200 mm of primary consolidation within the cohesive deposit.

Based on an average coefficient of consolidation ( $c_v$ ) of about  $2.7 \times 10^{-3} \text{ cm}^2/\text{s}$  estimated for the cohesive deposit based on the laboratory consolidation test for a sample of similar material, the imposed loading conditions for an approximately 3 m high embankment plus up to 4.2 m of replacement backfill, and assuming two-way drainage of the cohesive deposit, it is estimated that about 90 percent of the primary consolidation settlement will be completed in about 1 month.

The magnitude of secondary (creep) settlement for the cohesive deposit is expected to be about 15 mm per log-cycle of time for this area, generally corresponding to two log cycles for a period of 10 years after completion of embankment construction.

Based on a 2.6 m high embankment plus about 4.2 m of additional rock fill required after removal of the organic deposit, the total rock fill embankment settlement is estimated to be up about 80 mm of which 60 mm of settlement will occur in the first 6 months after embankment construction and 20 mm will occur thereafter.

The post-construction settlement for up to 10 years after completion of embankment construction is about 300 mm and, therefore, mitigation of the post-construction settlement is required.



### **6.4.1.3 Mitigation of Stability Issues and/or Time Dependent Settlements**

The presence of an up to 2.2 m thick clayey silt to silty clay deposit influences both the stability and the settlement for the proposed 2.6 m high embankment. In order to construct the embankment to achieve a FoS equal to or greater than 1.3, and to minimize post-construction settlements, the preferred alternative to mitigate both settlement and stability is to sub-excavate the clayey silt to silty clay deposit to a depth of up to 7.6 m below pond surface (i.e. 6.1 m below the bottom of the pond). However, from a foundation perspective, preloading of the clayey silt to silty clay deposit by the embankment constructed with toe berms to mitigate stability issues is also feasible. The feasible and non-feasible alternatives described below have been evaluated and ranked on the basis of the advantages, disadvantages, relative costs and risk/consequences and are summarised in Table 1. The recommended alternative to mitigate stability and settlement for the pond is full sub-excavation of the cohesive deposit.

#### **Full Sub-Excavation**

The bottom of the cohesive deposit is up to 6.1 m below the pond bottom (up to 7.6 m below ice/water surface) at the time of drilling within the proposed embankment footprint. Full sub-excavation of the cohesive deposit to this depth is considered feasible and would be the best technical solution in terms of the long-term performance of the roadway. Stability analysis indicates that full sub-excavation of the cohesive deposit would be required for the entire length of the pond crossing.

Since the groundwater table is at or near the pond shoreline ground surface, the majority of the sub-excavation would have to be carried out 'in-the-wet' (i.e. below the water level). Excavation 'in-the-wet' results in less risk of instability and base heave than under dry conditions but will create more uncertainty regarding full removal of the cohesive deposits. Excavation 'in-the-wet' to remove the cohesive deposit in this area should be carried out with side slopes no steeper than 1H:1V to limit the risk of instability. Complete removal of the cohesive deposit should extend to a horizontal distance beyond the toe of the proposed embankment equal to the horizontal component of the side slope profile (i.e. 1.25 for rock fill) multiplied by the depth to the bottom of the cohesive deposit below the ground surface (in accordance with OPSD 203.010, Embankments Over Swamp, New Construction).

It should be noted, however, that full sub-excavation of the cohesive deposit would increase the effective thickness of the new embankment fill by up to an additional approximately 2 m in the critical section (i.e. for a total below-grade rock fill thickness of about 6.2 m), requiring a larger volume of rock fill material. The additional below-grade fill would need to be constructed with the same side slope profile as that used for the above-grade embankment (OPSD 203.010). Based on a 2.6 m high embankment plus about 6.2 m of additional rock fill required after full sub-excavation, the total rock fill embankment settlement is estimated to be about 120 mm at the critical section, of which approximately 100 mm of settlement will occur in the first 6 months following embankment construction and approximately 20 mm thereafter.

Provided the embankment will be preloaded for 6 months, the post-construction settlement for this alternative (after roadway paving) is estimated to be about 20 mm.





### ***Preloading with Toe Berms***

The results of the stability analyses carried out for the option comprised of preloading the cohesive deposits and incorporating berms along the east toe of the embankment are presented on Figure 1. For this site, toe berms 12 m wide at crest Elevation 200.5 m (top of berm) would be required along the full length of the pond crossing.

Based on the estimated coefficient of consolidation for the cohesive deposit, it is estimated that 90 percent of primary consolidation settlement will be completed in about 1 month. However, if preloading is adopted as the foundation mitigation option at this location, to eliminate the need for instrumentation and settlement monitoring during and after the construction of the embankment, it is recommended that a preload period of 2 months be included in the construction schedule. If the construction schedule can accommodate this preload period, by constructing the embankment as early as possible, preloading the foundation soils is considered a technically feasible option.

The total post-construction settlement of the foundation and rock fill embankment after a 2-month preload will be about 80 mm over a 10-year period. If the embankment can be preloaded for 6 months, the post-construction settlement is reduced to about 45 mm.

### ***Surcharging with Toe Berms***

Due to the relatively short time period for preloading and the additional material required for surcharging, including larger toe berms, this option is not considered practical for this site.

### ***Wick Drains***

Due to the limited lateral extent and the thickness (i.e. about 2 m) of the cohesive deposit, the use of wick drains to reduce the length of the time period required for primary consolidation settlement is not considered to be practical for this area.

### ***Lightweight Fill***

Given the low embankment height, high water table and relatively short length of pond, the use of lightweight fill is not considered to be practical for this area.

## **6.5 Subgrade Preparation and Embankment Construction**

As discussed in Section 6.1, the new Highway 69 four-laning for the section from Shawanaga River northerly 11 km and the development of the Site 9 Road and Highway 529 Interchange (including the associated ramps and roadways) will require the construction of numerous embankments over swamps and ponds. The following sections discuss general aspects of subgrade preparation and embankment construction for the Highway 69 NBL embankment crossing the pond between Station 14+450 and 14+550, including: removal of surficial and near surface organic materials; excavation and replacement of soft subsoils; groundwater control where required; and embankment fill placement.



### **6.5.1 Removal of Organics**

Based on the information from the boreholes advanced during the field investigation, the thickness of organic deposits (i.e. peat) in the pond was up to about 4.2 m below the bottom of the pond. After clearing and grubbing of the pond shoreline area and prior to the placement of any fill for the new construction, all surficial and near surface layers of organic deposits should be stripped from the plan limits of the proposed works. The organic deposit should be removed using construction procedures in accordance with OPSS 209 (Embankment Over Swamps and Compressible Soils).

As the new embankment will be constructed away from existing highway embankments, the excavation limits should be consistent with OPSD 203.010 (Embankments Over Swamps, New Construction) modified to remove the restrictions on the height of the embankment and the depth of excavation (i.e. Note A).

### **6.5.2 Excavation and Replacement of Soft Subsoils**

Excavations up to about 6.1 m below the bottom of the pond (7.6 m below the top of ice/water at the time of drilling) are anticipated in this pond area where sub-excavation and replacement of soft materials is recommended as the preferred mitigation option. For this pond crossing, conventional equipment is considered suitable for the excavation of the soft subsoils.

All excavations must be carried out in accordance with the latest edition of the Ontario Occupational Health and Safety Act and Regulations for Construction Projects.

### **6.5.3 Groundwater and Surface Water Control**

Excavation within the plan limits of the proposed works will be required to remove organic and soft deposits prior to embankment fill placement. The water table is represented by the pond water surface and therefore the excavation will be carried out underwater. In addition, groundwater flow into the excavations may occur due to the relatively permeable subsoils under the clayey silt to silty clay deposit and high groundwater levels observed in the pond shoreline boreholes. Dewatering is not required for the excavation and backfilling in the pond area. Surface water should be directed away from the excavations at all times.

Conventional (or long-stick type) excavators should be suitable for all of the excavating operations through the pond crossing.

### **6.5.4 Backfilling**

For replacement of the sub-excavated material, it is assumed that rock fill will be used. Where sub-excavation of soft subsoils is being carried out as a foundation mitigation option, it will not be possible to place rock fill in accordance with OPSS 206 (Grading), and the embankment under water should be constructed in accordance with OPSS 209 (Embankments Over Swamps and Compressible Soils), with the rock fill end dumped (below the water table) as the excavation advances.



### **6.5.5 Embankment Fill Placement**

Placement of all rock fill material above the water table for construction of new embankments should be carried out in accordance with the requirements as outlined in OPSS 206 (Grading). Voids and bridging should be minimized by blading, dozing and 'chinking' the rock to form a dense, compacted mass. Side slopes for rock fill embankments should be no steeper than 1.25H:1V.

### **6.5.6 Embankment Platform Widening**

In accordance with the requirements of MTO NRE 98-200 (Embankment Design Guidelines), the construction of the embankment should include an allowance for platform widening (in 500 mm increments) to accommodate post-construction settlements so that the minimum standard shoulder widths are maintained if future grade raises on the embankments are required. According to NRE 98-200, the need for future raises in road grade could occur due to settlement of the embankment fill, settlement of the foundation soils and to accommodate future pavement overlays up to 200 mm thick. It is understood that this policy applies to all rock fill embankments as well as for earth fill embankments where widening restrictions are present (i.e. due to space/property issues, presence of a sensitive body of water and so on). It is further understood that the minimum platform widening for all swamp/pond crossings on major highways (i.e. including Highway 69) is 2 m per side, unless the preferred mitigation option eliminates uncertainty regarding embankment settlement/performance (i.e. full sub-excavation to bedrock and backfill with a granular material).

The minimum required embankment platform width (per embankment side) is calculated based on the estimated total primary and secondary (creep) settlement of cohesive material left in place plus the post-construction settlement of rock fill plus the 200 mm pavement overlay multiplied by the horizontal component of the pavement structure granular (i.e. 4H:1V) (but cannot be less than 2 m as described above). For this site, the minimum required embankment widening to account for the estimated settlements and for structure overlays is 2 m.

## **7.0 CLOSURE**

This report was prepared by Mr. André Bom, P.Eng., an intermediate engineer with Golder. Mr. Jorge M. A. Costa, P.Eng., Golder's Designated MTO Contact for this project and a Principal with Golder, conducted an independent 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\C2 - Pond\07-1191-0020-2 Rpt 11Aug3 Pond.Docx



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- Terzaghi, K. and Peck, R.B., 1967. Soil Mechanics in Engineering Practice, 2nd Edition, John Wiley and Sons, New York.

## STANDARDS:

### ASTM International:

ASTM D1586-08a	Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils
ASTM D2573-08	Standard Test Method for Field Vane Shear Test in Cohesive Soil

### Ministry of Transportation:

Foundation Section	Post-Construction Rock Fill Settlement and Guidelines for Estimating Rock Fill Quantity, April 2010
NRE 98.200	Northern Region Embankment Design Guidelines, Northern Region Engineering Directive, October 28, 1998

### Ontario Provincial Standard Specification:

OPSS 206	Construction Specification for Grading. November 2009
OPSS 501	Construction Specification for Compacting. November 2005
OPSS 209	Construction Specification For Embankments Over Swamps And Compressible Soils, April 2009



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**FOUNDATION REPORT - POND CROSSING - PHASE 2**  
**HIGHWAY 69 GWP 5005-08-00**

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Ontario Provincial Standard Drawings

OPSD 203.010                      Embankments Over Swamps, New Construction, November 2005

Ontario Water Resources Act:

Ontario Regulation 372/97    Amendment to Ontario Regulation 903

Ontario Regulation 903/90    Wells



## FOUNDATION REPORT - POND CROSSING - PHASE 2

### HIGHWAY 69 GWP 5005-08-00

**Table 1: Evaluation of Stability/Settlement Mitigation Options**  
**Highway 69 NBL - STA 14+450 to 14+550 (Pond 1)**

Stability/Settlement Mitigation Option	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Full Sub-Excavation (up to 6.2 m deep below bottom of pond)	1	<ul style="list-style-type: none"> <li>Improved stability.</li> <li>Reduced total settlement.</li> <li>No delay in construction.</li> <li>Eliminates need for stability berms.</li> </ul>	<ul style="list-style-type: none"> <li>Additional effort required for sub-excavation and replacement below water level.</li> <li>Generation of large volume of excess excavation spoil (could be used for slope flattening).</li> <li>Greater quantity of rock fill required.</li> </ul>	<ul style="list-style-type: none"> <li>Additional costs associated with sub-excavation, disposal and replacement of weak/soft, compressible deposits and overlying or interlayers of cohesionless materials.</li> </ul>	<ul style="list-style-type: none"> <li>Incomplete removal of organics and loose/soft materials could result in post-construction settlement.</li> </ul>
Preloading (6 months) and Toe Berms (no instrumentation and monitoring)	2	<ul style="list-style-type: none"> <li>Standard construction operation.</li> <li>Reduced quantity of material(s) removed and larger quantity of rock fill required.</li> </ul>	<ul style="list-style-type: none"> <li>Short delay in construction to allow for at least 90% primary consolidation settlement to be completed.</li> <li>Re-grading is required to account for settlement prior to final pavement structure construction.</li> <li>Toe berms 0.5 m high (above water) by 12 m wide required for stability; therefore, additional right-of-way may be required.</li> </ul>	<ul style="list-style-type: none"> <li>Slightly increased cost for toe berms compared to full sub-excavation.</li> </ul>	<ul style="list-style-type: none"> <li>Some secondary consolidation (creep) will occur.</li> <li>Some risk with respect to maintaining stability of fill on weak foundation soils.</li> </ul>
Surcharging and Toe Berms	NF	<ul style="list-style-type: none"> <li>Standard construction operation.</li> <li>Reduced secondary (creep) consolidation settlement.</li> </ul>	<ul style="list-style-type: none"> <li>Reduced delay in construction to allow for at least 90% primary consolidation to be completed.</li> <li>Increased handling of fill to remove surcharge.</li> <li>May need to acquire additional right-of-way due to the width of toe berms.</li> </ul>	<ul style="list-style-type: none"> <li>Increased costs associated with construction and materials for 2 m high surcharge and larger toe berms as compared with both full sub-excavation and preload only options.</li> </ul>	<ul style="list-style-type: none"> <li>Some risk with respect to maintaining stability of higher (surcharged) fills on weak/soft foundation soils.</li> </ul>



## FOUNDATION REPORT - POND CROSSING - PHASE 2

### HIGHWAY 69 GWP 5005-08-00

**Table 1: Evaluation of Stability/Settlement Mitigation Options**  
**Highway 69 NBL - STA 14+450 to 14+550 (Pond 1)**

Stability/Settlement Mitigation Option	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Wick Drains (with or without surcharge)	NF	<ul style="list-style-type: none"> <li>Potential to decrease consolidation time under preload or surcharge.</li> <li>Potential faster rate of construction possible.</li> </ul>	<ul style="list-style-type: none"> <li>Limited thickness (up to 1.8 m) of the cohesive deposit will not reduce drainage path enough to reduce time for 90% consolidation.</li> <li>Increased time for installation of wick drains.</li> <li>Instrumentation and monitoring program required to monitor staged construction and to assess when end of primary consolidation is reached.</li> <li>Wick drain design required.</li> </ul>	<ul style="list-style-type: none"> <li>Additional costs associated with toe berms, wick drain design, instrumentation and monitoring program.</li> </ul>	<ul style="list-style-type: none"> <li>Increased secondary consolidation (creep) may occur if surcharge is not applied.</li> </ul>
Lightweight Fill (EPS)	NF	<ul style="list-style-type: none"> <li>Improved stability.</li> <li>Reduced post-construction settlements.</li> <li>No delay in construction.</li> </ul>	<ul style="list-style-type: none"> <li>High cost of construction materials.</li> <li>Restricted use within the embankment cross-section to between anticipated high water table level and ballast cap.</li> <li>Additional design required to assess extent of EPS practical for the embankment configuration.</li> </ul>	<ul style="list-style-type: none"> <li>Reduced costs for disposal/management of excavation spoil as compared with full sub-excavation option.</li> <li>Relative cost of EPS fill is at least an order of magnitude higher than fill required for the other options.</li> </ul>	<ul style="list-style-type: none"> <li>Very low risk with respect to stability and long-term settlement of foundation soils.</li> </ul>

NF indicates that the alternative has been considered but is not feasible.





# **APPENDIX A**

## **Record of Boreholes**



## LIST OF SYMBOLS

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

### 1. GENERAL

$\pi$	3.1416
$\ln x$ ,	natural logarithm of x
$\log_{10}$	x or log x, logarithm of x to base 10
g	acceleration due to gravity
t	time
FoS	Factor of Safety
V	volume
W	weight

### II. STRESS AND STRAIN

$\gamma$	shear strain
$\Delta$	change in, e.g. stress: $\Delta\sigma$
$\epsilon$	linear strain
$\epsilon_v$	volumetric strain
$\eta$	coefficient of viscosity
$\nu$	Poisson's ratio
$\sigma$	total stress
$\sigma'$	effective stress ( $\sigma' = \sigma - u$ )
$\sigma_{vo}$	initial 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

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

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

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

#### (b) Hydraulic Properties

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

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

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

#### (d) Shear Strength

$\tau_p, \tau_r$	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

**Notes:** 1  $\tau = c' + \sigma' \tan \phi'$   
2 Shear strength = (Compressive strength)/2



## 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
$SO_4$	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

PROJECT <u>07-1191-0020</u>		<b>RECORD OF BOREHOLE No P1- 1</b>		1 OF 1 <b>METRIC</b>	
W.P. <u>5005-08-00</u>		LOCATION <u>N 5051063.2 ;E 236953.5</u>		ORIGINATED BY <u>ID</u>	
DIST <u>          </u> HWY <u>69</u>		BOREHOLE TYPE <u>Hand Sampling Equipment</u>		COMPILED BY <u>MM</u>	
DATUM <u>Geodetic</u>		DATE <u>February 25, 2008</u>		CHECKED BY <u>AB</u>	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			UNIT WEIGHT  <b>γ</b>  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									
200.0	GROUND SURFACE																
0.0	PEAT (Fibrous)		1	SS	1												
0.2	Very soft Black Wet																
199.1	Silty SAND, trace gravel		2	SS	1/0.15												
0.9	Very loose Brown Wet																
	End of Borehole Spoon Refusal (Hammer Bouncing)																
	Notes:  1. Water level at ground surface (Elev. 200.0 m) upon completion of drilling.  2. Split spoon samples obtained by driving with a 1/2 weight hammer; SPT 'N' values have been adjusted to the inferred values that would be obtained using a standard weight hammer.																

PROJECT <u>07-1191-0020</u>		<b>RECORD OF BOREHOLE No P1- 3</b>		1 OF 1 <b>METRIC</b>	
W.P. <u>5005-08-00</u>		LOCATION <u>N 5051039.5 ;E 236961.3</u>		ORIGINATED BY <u>ID</u>	
DIST <u>          </u> HWY <u>69</u>		BOREHOLE TYPE <u>Hand Sampling Equipment</u>		COMPILED BY <u>MM</u>	
DATUM <u>Geodetic</u>		DATE <u>February 25, 2008</u>		CHECKED BY <u>AB</u>	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			UNIT WEIGHT  <b>γ</b>  kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED									
200.0	GROUND SURFACE																
0.0	PEAT (Fibrous)		1	SS	2/0.05												
0.2	Very Soft Black Wet  End of Borehole Spoon Refusal (Hammer Bouncing)  Notes:  1. Borehole dry upon completion of drilling.  2. Bedrock exposed at borehole location with hand shovel upon completion of drilling.  3. Split spoon sample obtained by driving with a 1/2 weight hammer; SPT 'N' value has been adjusted to the inferred value that would be obtained using a standard weight hammer.																

MIS-MTO 002 07-1191-0020 P1 METRIC.GPJ GAL-MISS.GDT 03/02/11 DATA INPUT:

PROJECT		RECORD OF BOREHOLE No P1- 4				1 OF 1		METRIC				
W.P. 5005-08-00		LOCATION N 5051031.6 ;E 236977.6				ORIGINATED BY ID						
DIST HWY 69		BOREHOLE TYPE Portable Equipment, NW Casing Wash Boring				COMPILED BY MM						
DATUM Geodetic		DATE February 25, 2008				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	20 40 60 80 100	20 40 60	W <sub>p</sub> W W <sub>L</sub>		
199.8	ICE SURFACE											
199.5	ICE											
0.3	WATER											
198.9	0.9											
	PEAT (Fibrous) Very soft Black Wet		1	SS	WR						190.1	
198.1	1.7											
	End of Borehole Casing Refusal											
	Notes:											
	1. Water level at ice surface (Elev. 199.8 m) upon completion of drilling.											
	2. Split spoon sample obtained by driving with a 1/2 weight hammer; SPT 'N' value has been adjusted to the inferred value that would be obtained using a standard weight hammer.											

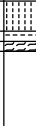
PROJECT		RECORD OF BOREHOLE No P1- 5				1 OF 1		METRIC				
W.P. 5005-08-00		LOCATION N 5051015.7 ;E 236969.2				ORIGINATED BY ID						
DIST HWY 69		BOREHOLE TYPE Hand Sampling Equipment				COMPILED BY MM						
DATUM Geodetic		DATE February 25, 2008				CHECKED BY AB						
SOIL PROFILE		SAMPLES			GROUND 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	20 40 60 80 100	20 40 60	W <sub>p</sub> W W <sub>L</sub>		
202.0	GROUND SURFACE											
0.1	PEAT (Fibrous) Soft Black Wet		1	SS	4/0.08							
	End of Borehole Spoon Refusal (Hammer Bouncing)											
	Notes:											
	1. Borehole dry upon completion of drilling.											
	2. Bedrock exposed at borehole location with hand shovel upon completion of drilling.											
	3. Split spoon sample obtained by driving with a 1/2 weight hammer; SPT 'N' value has been adjusted to the inferred value that would be obtained using a standard weight hammer.											

MIS-MTO 002 07-1191-0020 P1 METRIC.GPJ GAL-MISS.GDT 03/02/11 DATA INPUT:



PROJECT <u>07-1191-0020</u>		<b>RECORD OF BOREHOLE No P1-7</b>		1 OF 1 <b>METRIC</b>	
W.P. <u>5005-08-00</u>		LOCATION <u>N 5050991.9 ;E 236976.8</u>		ORIGINATED BY <u>ID</u>	
DIST <u>          </u> HWY <u>69</u>		BOREHOLE TYPE <u>Portable Equipment, NW Casing Wash Boring</u>		COMPILED BY <u>MM</u>	
DATUM <u>Geodetic</u>		DATE <u>February 25, 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									
199.8	ICE SURFACE							○ UNCONFINED + FIELD VANE									
199.5	ICE		1	SS	WR			● QUICK TRIAXIAL × REMOULDED									
199.2	PEAT (Fibrous) Very Soft Black Wet																
0.6	End of Borehole Spoon Refusal (Hammer Bouncing)						199										
Notes:  1. Water level at ice surface (Elev. 199.8 m) upon completion of drilling.  2. Split spoon sample obtained by driving with a 1/2 weight hammer; SPT 'N' value has been adjusted to the inferred value that would be obtained using a standard weight hammer.																	

PROJECT <u>07-1191-0020</u>		<b>RECORD OF BOREHOLE No P1-8</b>		1 OF 1 <b>METRIC</b>	
W.P. <u>5005-08-00</u>		LOCATION <u>N 5050984.4 ;E 236994.2</u>		ORIGINATED BY <u>ID</u>	
DIST <u>          </u> HWY <u>69</u>		BOREHOLE TYPE <u>Portable Equipment, NW Casing Wash Boring</u>		COMPILED BY <u>MM</u>	
DATUM <u>Geodetic</u>		DATE <u>February 25, 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	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
								○ UNCONFINED + FIELD VANE									
								● QUICK TRIAXIAL × REMOULDED									
199.8	ICE SURFACE							20	40	60	80	100					
199.5	ICE																
	WATER			1	SS	1/0.15											
0.5	PEAT (Fibrous) Very soft Black Wet  End of Borehole Spoon Refusal (Hammer Bouncing)  Notes:  1. Water level at ice surface (Elev. 199.8 m) upon completion of drilling.  2. Split spoon sample obtained by driving with a 1/2 weight hammer; SPT 'N' value has been adjusted to the inferred value that would be obtained using a standard weight hammer.						199										

MIS-MTO 002 07-1191-0020 P1 METRIC.GPJ GAL-MISS.GDT 03/02/11 DATA INPUT:

PROJECT 07-1191-0020		<b>RECORD OF BOREHOLE No P1-9</b>				1 OF 1 <b>METRIC</b>							
W.P. 5005-08-00		LOCATION N 5050968.2; E 236984.5				ORIGINATED BY ID							
DIST HWY 69		BOREHOLE TYPE Hand Sampling Equipment				COMPILED BY MM							
DATUM Geodetic		DATE February 25, 2008				CHECKED BY AB							
SOIL PROFILE			SAMPLES			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	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa				W <sub>p</sub> W W <sub>L</sub>	
199.8	GROUND SURFACE							20 40 60 80 100					
0.0 199.5 0.3	PEAT (Fibrous) Very soft Black Wet		1	SS	2								
198.9	Silty SAND, trace gravel Very loose to dense Brown Moist		2	SS	23/0.15		199						
0.9	End of Borehole Spoon Refusal (Hammer Bouncing)												
Notes: 1. Borehole dry upon completion of drilling. 2. Split spoon samples obtained by driving with a 1/2 weight hammer; SPT 'N' values have been adjusted to the inferred values that would be obtained using a standard weight hammer.													

PROJECT 07-1191-0020		<b>RECORD OF BOREHOLE No P1-10</b>				1 OF 1 <b>METRIC</b>								
W.P. 5005-08-00		LOCATION N 5051045.5; E 236973.1				ORIGINATED BY ID								
DIST HWY 69		BOREHOLE TYPE Portable Equipment, NW Casing Wash Boring				COMPILED BY MM								
DATUM Geodetic		DATE February 25, 2008				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 $\gamma$ 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						
199.8	ICE SURFACE													
0.0	ICE													
199.5														
0.3	WATER													
198.9							199							
198.6	PEAT (Fibrous) Very soft Black Wet		1	SS	1								87.5	
1.2	Silty SAND, some clay, trace gravel Very loose Grey Wet		2	SS	1		198							2 48 24 26
197.5			3	SS	1		197						80.7	0 10 43 47
2.3	SILTY CLAY to CLAYEY SILT, trace to some sand Soft Grey Wet						196							
195.5			4	SS	1/0.15									
4.3	End of Borehole Spoon Refusal (Hammer Bouncing)													
Notes:  1. Water level at ice surface (Elev. 199.8 m) upon completion of drilling.  2. Split spoon samples obtained by driving with a 1/2 weight hammer; SPT 'N' values have been adjusted to the inferred values that would be obtained using a standard weight hammer.														

MIS-MTO001 07-1191-0020 P1 METRIC.GPJ GAL-MISS.GDT 03/02/11 DATA INPUT:



PROJECT 07-1191-0020				RECORD OF BOREHOLE No P1-11				1 OF 1 METRIC									
W.P. 5005-08-00				LOCATION N 5051073.0; E 236982.7				ORIGINATED BY TDM									
DIST HWY 69				BOREHOLE TYPE Portable Equipment, BW Casing Wash Boring				COMPILED BY MM									
DATUM Geodetic				DATE February 10, 2009				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									
200.3	ICE SURFACE							20	40	60	80	100					
0.0	ICE																
200.0																	
0.3	WATER																
199.5																	
199.1	PEAT (Fibrous) Very Soft Firm Black Wet		1	SS	6												
198.7																	
1.6	CLAYEY SILT Stiff Grey Wet		2	SS	50/0.1												
	End of Borehole Spoon Refusal (Hammer Bouncing)																
	Notes:																
	1. Water level at ice surface (Elev. 200.3 m) upon completion of drilling.																
	2. Split spoon samples obtained by driving with a 1/2 weight hammer; SPT 'N' values have been adjusted to the inferred values that would be obtained using a standard weight hammer.																

PROJECT 07-1191-0020			RECORD OF BOREHOLE No P1-12			1 OF 1 METRIC								
W.P. 5005-08-00			LOCATION N 5051048.4; E 236988.2			ORIGINATED BY TDM								
DIST _____ HWY 69			BOREHOLE TYPE Portable Equipment, BW Casing Wash Boring			COMPILED BY MM								
DATUM Geodetic			DATE February 10, 2009			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 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						
200.3	ICE SURFACE													
0.0	ICE													
200.0														
0.3	WATER													
198.8														
1.5	PEAT (Amorphous) Very soft Black Wet		1	SS	WH									
			2	SS	1								504.2	
			3	SS	WH									
			4	SS	WH								503.7	
			5	SS	WH									
194.6														
5.7	CLAYEY SILT Soft Grey Wet		6	SS	WH									
192.7														
7.6	SAND and GRAVEL, some silt, trace to some clay		7	SS	23									31 49 14 6
192.2	Compact Grey Wet													
8.1	End of Borehole Spoon Refusal (Hammer Bouncing)													
	Notes:  1. Water level at ice surface (Elev. 200.3 m) upon completion of drilling.  2. Split spoon samples obtained by driving with a 1/2 weight hammer; SPT 'N' values have been adjusted to the inferred values that would be obtained using a standard weight hammer.													

MIS-MTO001 07-1191-0020 P1 METRIC.GPJ GAL-MISS.GDT 03/02/11 DATA INPUT:

PROJECT 07-1191-0020			RECORD OF BOREHOLE No P1-13			1 OF 1 METRIC								
W.P. 5005-08-00			LOCATION N 5051024.6; E 236993.5			ORIGINATED BY TDM								
DIST HWY 69			BOREHOLE TYPE Portable Equipment, BW Casing Wash Boring			COMPILED BY MM								
DATUM Geodetic			DATE February 9, 2009			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 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						
200.3	ICE SURFACE													
0.0	ICE													
200.0														
0.3	WATER													
198.8														
1.5	PEAT (Amorphous) Very soft to soft Black Wet		1	SS	WH									
	Sand seam 0.3 m thick at a depth of 2.6 m		2	SS	3								495.6	
			3	SS	WH									
			4	SS	WH								1174.5	
195.3			5A	SS	WH									
5.0	SILTY CLAY Very soft Grey Wet		5B											
			6	TO	WR									
193.1														
7.2	End of Borehole Casing Refusal													
	Notes:  1. Water level at ice surface (Elev. 200.3 m) upon completion of drilling.  2. Split spoon samples obtained by driving with a 1/2 weight hammer; SPT 'N' values have been adjusted to the inferred values that would be obtained using a standard weight hammer.													

MIS-MTO 001 07-1191-0020 P1 METRIC.GPJ GAL-MISS.GDT 03/02/11 DATA INPUT:

PROJECT <u>07-1191-0020</u>		<b>RECORD OF BOREHOLE No P1-14</b>		1 OF 1 <b>METRIC</b>	
W.P. <u>5005-08-00</u>		LOCATION <u>N 5050999.7 ; E 236999.1</u>		ORIGINATED BY <u>TDM</u>	
DIST <u>          </u> HWY <u>69</u>		BOREHOLE TYPE <u>Portable Equipment, BW Casing Wash Boring</u>		COMPILED BY <u>MM</u>	
DATUM <u>Geodetic</u>		DATE <u>February 9, 2009</u>		CHECKED BY <u>AB</u>	

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			20 40 60 80 100	20 40 60 80 100	W <sub>p</sub> W W <sub>L</sub>					
								SHEAR STRENGTH kPa			WATER CONTENT (%)				
							○ UNCONFINED + FIELD VANE								
							● QUICK TRIAXIAL × REMOULDED								
200.3	ICE SURFACE						200								
0.0	ICE														
200.0															
0.3	WATER														
198.8							199								
1.5	PEAT (Amorphous) Very soft Black Wet		1	SS	WH								351.5		
198.2															
2.3	SAND and GRAVEL Very dense Grey Wet  End of Borehole Spoon Refusal  Notes:  1. Water level at ice surface (Elev. 200.3 m) upon completion of drilling.  2. Split spoon samples obtained by driving with a 1/2 weight hammer; SPT 'N' values have been adjusted to the inferred values that would be obtained using a standard weight hammer.		2	SS	50/0.03		198								

PROJECT <u>07-1191-0020</u>		<b>RECORD OF BOREHOLE No P1-15</b>		1 OF 1 <b>METRIC</b>	
W.P. <u>5005-08-00</u>		LOCATION <u>N 5050974.9 ; E 237004.7</u>		ORIGINATED BY <u>TDM</u>	
DIST <u>          </u> HWY <u>69</u>		BOREHOLE TYPE <u>Portable Equipment, BW Casing Wash Boring</u>		COMPILED BY <u>MM</u>	
DATUM <u>Geodetic</u>		DATE <u>February 10, 2009</u>		CHECKED BY <u>AB</u>	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT CONTENT			UNIT WEIGHT  <b>γ</b>  kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED									
200.4	ICE SURFACE					200											
0.0	ICE																
200.1																	
0.3	WATER																
199.6																	
0.8	End of Borehole Casing Refusal																
	Note:  1. Water level at ice surface (Elev. 200.4 m) upon completion of drilling.																

MIS-MTO 002 07-1191-0020 P1 METRIC.GPJ GAL-MISS.GDT 03/02/11 DATA INPUT:

PROJECT <u>07-1191-0020</u>		<b>RECORD OF PENETRATION TEST No P1-DC1</b>		1 OF 1 <b>METRIC</b>	
W.P. <u>5005-08-00</u>		LOCATION <u>N 5051055.4 ;E 236969.8</u>		ORIGINATED BY <u>ID</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 22, 2008</u>		CHECKED BY <u>AB</u>	

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			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	20 40 60 80 100	20 40 60 80 100	W <sub>p</sub> W W <sub>L</sub>	WATER CONTENT (%)		
199.8	ICE SURFACE												
196.3	End of DCPT Refusal to Further Penetration (40 Blows/0.0 m)												
3.5													

PROJECT <u>07-1191-0020</u>		<b>RECORD OF PENETRATION TEST No P1-DC3</b>		1 OF 1 <b>METRIC</b>	
W.P. <u>5005-08-00</u>		LOCATION <u>N 5051007.9 ;E 236985.6</u>		ORIGINATED BY <u>ID</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 22, 2008</u>		CHECKED BY <u>AB</u>	

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			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	20 40 60 80 100	20 40 60 80 100	W <sub>p</sub> W W <sub>L</sub>	WATER CONTENT (%)		
199.8	ICE SURFACE												
198.7	End of DCPT Refusal to Further Penetration (Hammer Bouncing)												
1.1													

MIS-MTO 002 07-1191-0020 P1 METRIC.GPJ GAL-MISS.GDT 03/02/11 DATA INPUT:



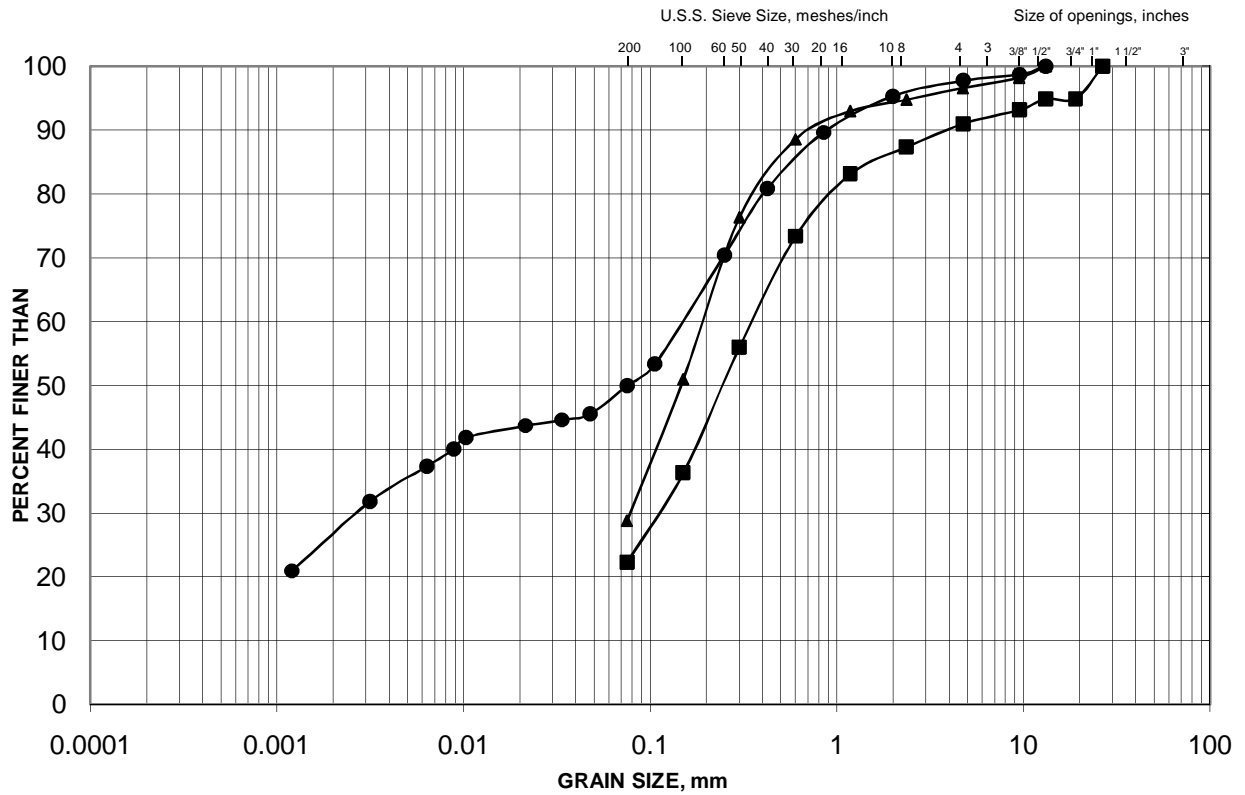
# APPENDIX B

## Laboratory Test Results

# GRAIN SIZE DISTRIBUTION

Silty Sand

FIGURE  
B-1



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

## LEGEND

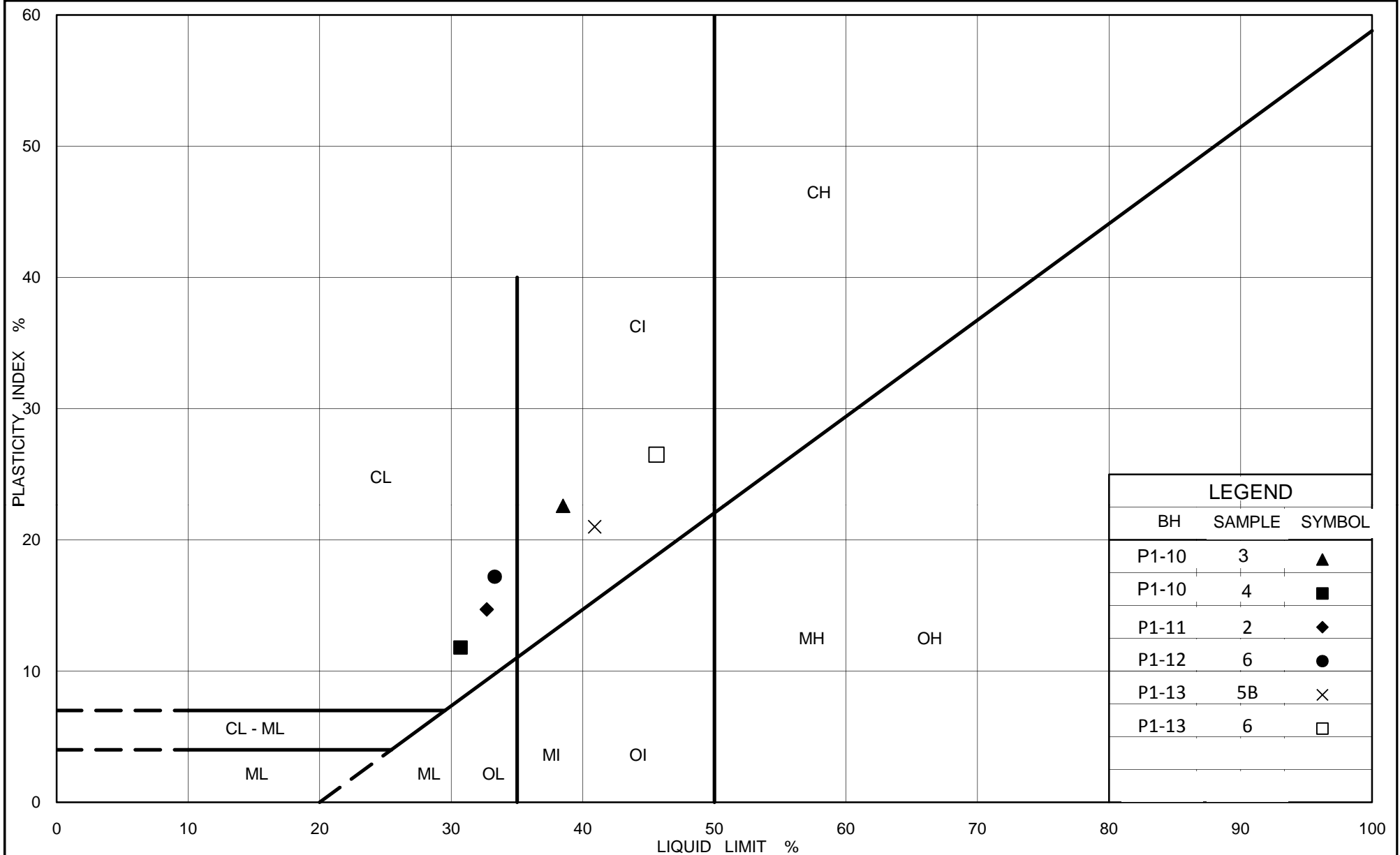
SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
▲	P1-1	2	199.2
■	P1-9	2	199.0
●	P1-10	2	197.8

Project Number: 07-1191-0020-2

Checked By: AB

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Date: August 2011



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## PLASTICITY CHART Clayey Silt to Silty Clay

Figure B-2

Project No. 07-1191-0020-2

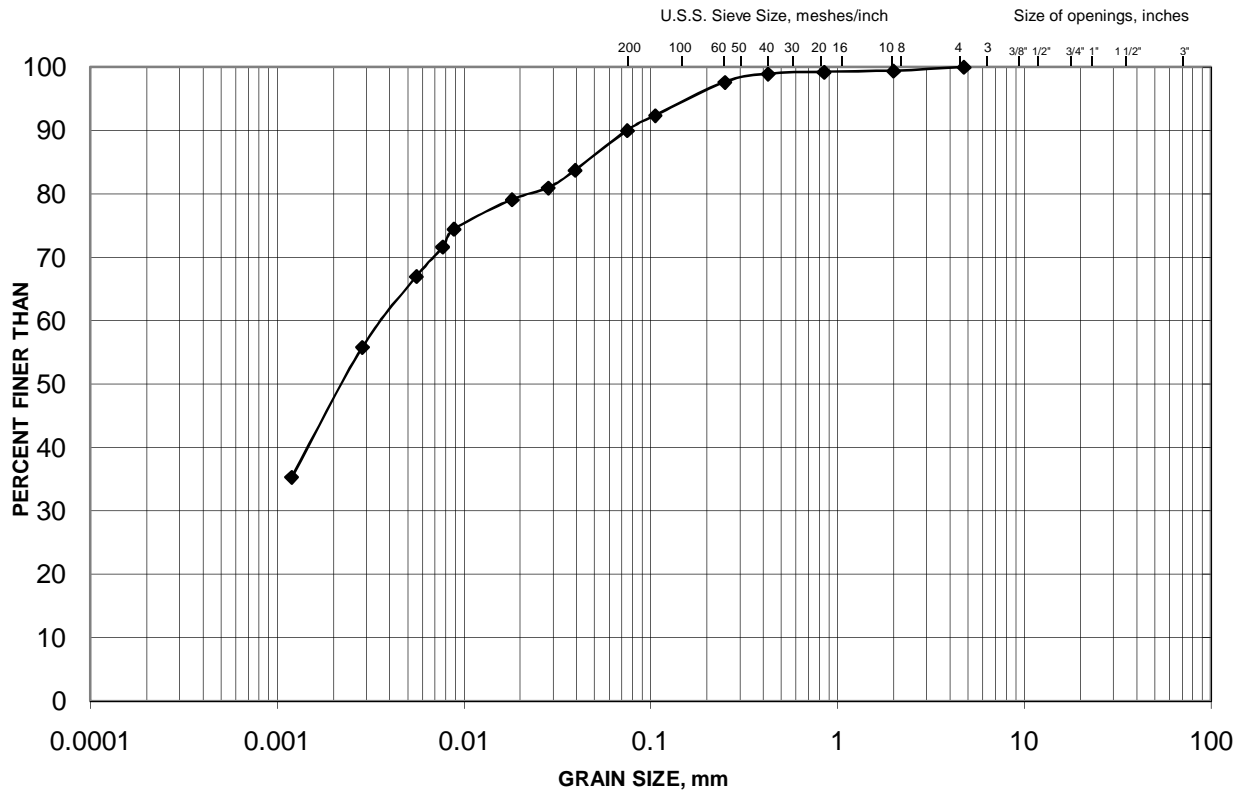
Checked By: AB



# GRAIN SIZE DISTRIBUTION

Silty Clay

FIGURE  
B-3



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

## LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
—◆—	P1-10	3	197.1

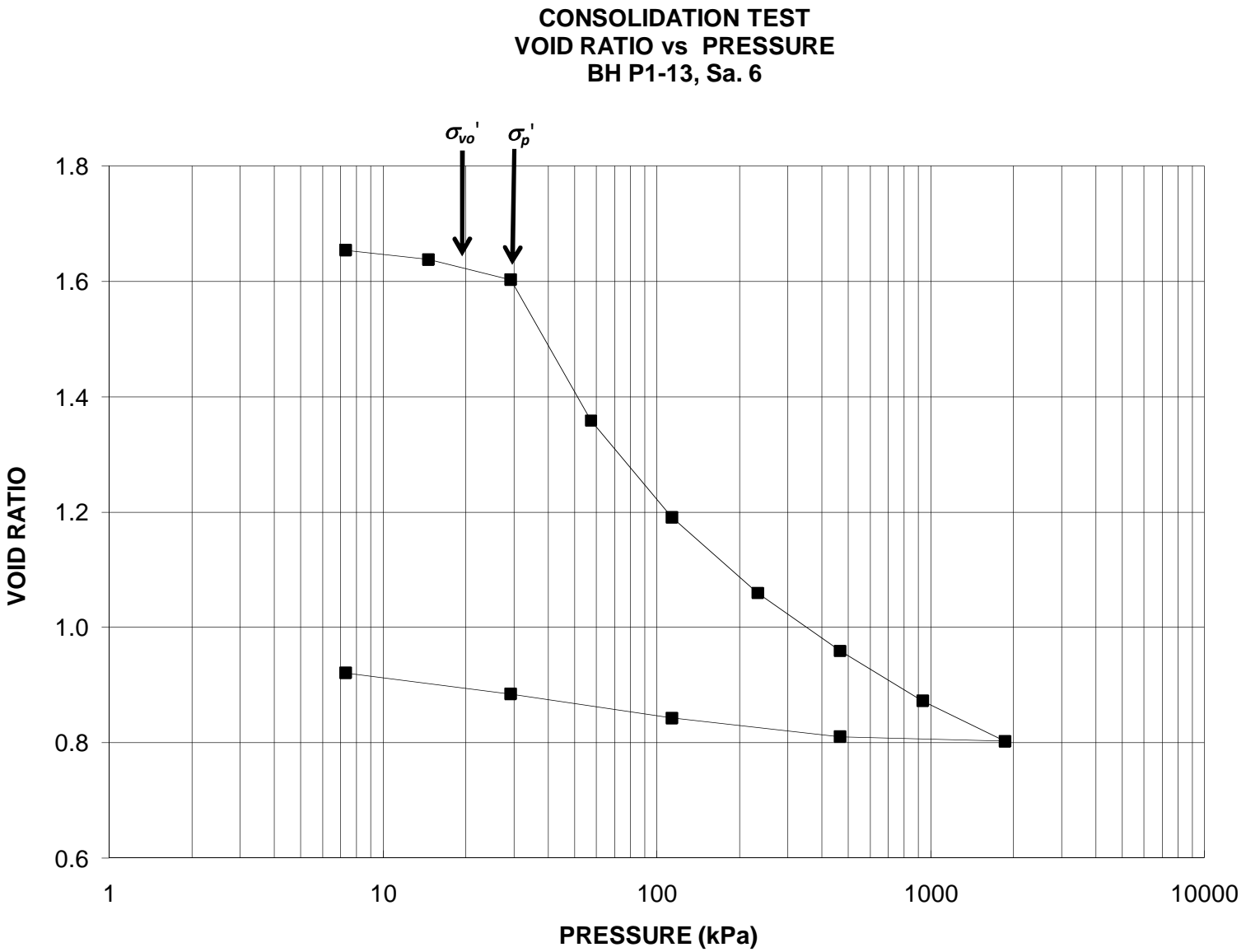
Project Number: 07-1191-0020-2

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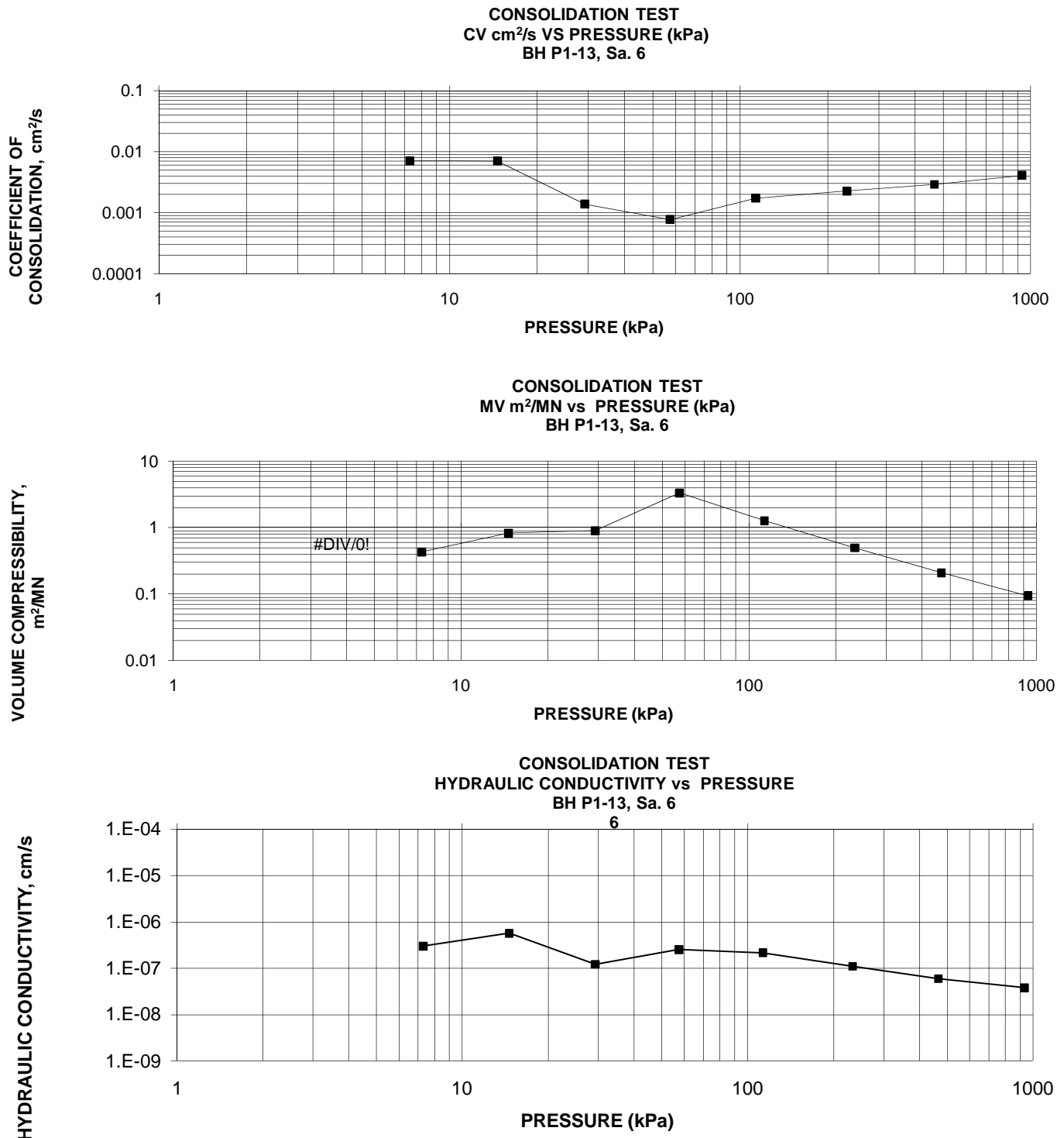
OEDOMETER CONSOLIDATION SUMMARY						FIGURE B-4 Page 1 of 4		
SAMPLE IDENTIFICATION								
Project Number		07-1191-0020-2		Borehole, Sample		P1-13, 6		
Pond:		1		Sample Depth, (m)		6.4		
TEST CONDITIONS								
Test Type		Standard		Load Duration, hr		24		
Oedometer Number		1						
Date Started		June 23/09						
Date Completed		July 9/09						
SAMPLE DIMENSIONS AND PROPERTIES - INITIAL								
Sample Height, cm		1.906		Unit Weight, kN/m <sup>3</sup>		15.9		
Sample Diameter, cm		5.000		Dry Unit Weight, kN/m <sup>3</sup>		9.9		
Area, cm <sup>2</sup>		19.63		Specific Gravity, assumed		2.7		
Volume, cm <sup>3</sup>		37.42		Solids Height, cm		0.716		
Water Content, %		59.8		Volume of Solids, cm <sup>3</sup>		14.06		
Wet Mass, g		60.64		Volume of Voids, cm <sup>3</sup>		23.37		
Dry Mass, g		37.95		Degree of Saturation, %		97.1		
TEST COMPUTATIONS								
Pressure	Primary Consolidation	Corr. Height	Void	Average Height	t <sub>50</sub>	cv.	m <sub>v</sub>	k
kPa	mm	cm	Ratio	cm	s	cm <sup>2</sup> /s	m <sup>2</sup> /MN	cm/s
0	0.00	1.906	1.663	1.906				
7.3	0.06	1.900	1.654	1.903	100	0.00710	0.431	3.003E-07
14.6	0.12	1.889	1.638	1.894	100	0.00703	0.828	5.713E-07
29.2	0.25	1.864	1.603	1.876	500	0.00138	0.905	1.225E-07
57.4	1.75	1.689	1.359	1.776	800	0.00077	3.336	2.529E-07
113.1	1.20	1.569	1.191	1.629	300	0.00173	1.275	2.167E-07
233.3	0.94	1.475	1.060	1.522	200	0.00227	0.499	1.110E-07
466.1	0.72	1.403	0.959	1.439	140	0.00290	0.210	5.962E-08
933.2	0.62	1.341	0.873	1.372	90	0.00410	0.095	3.803E-08
1864.2	0.50	1.291	0.803	1.316	50	0.00678	0.040	2.666E-08
466.1	-0.05	1.296	0.810	1.293				
113.1	-0.23	1.319	0.843	1.308				
29.2	-0.30	1.349	0.884	1.334				
7.3	-0.26	1.375	0.921	1.362				
Notes: k calculated using cv based on t <sub>50</sub> values.								
SAMPLE DIMENSIONS AND PROPERTIES - FINAL								
Sample Height, cm		1.375		Unit Weight, kN/m <sup>3</sup>		17.6		
Sample Diameter, cm		5.000		Dry Unit Weight, kN/m <sup>3</sup>		13.8		
Area, cm <sup>2</sup>		19.63		Specific Gravity, assumed		2.7		
Volume, cm <sup>3</sup>		27.00		Solids Height, cm		0.716		
Water Content, %		27.7		Volume of Solids, cm <sup>3</sup>		14.06		
Wet Mass, (after test) g		48.461		Volume of Voids, cm <sup>3</sup>		12.95		
Dry Mass, g (oven)		37.95		Degree of Saturation, %		84.6		
Prepared By: SL			Golder Associates			Checked By: AB		

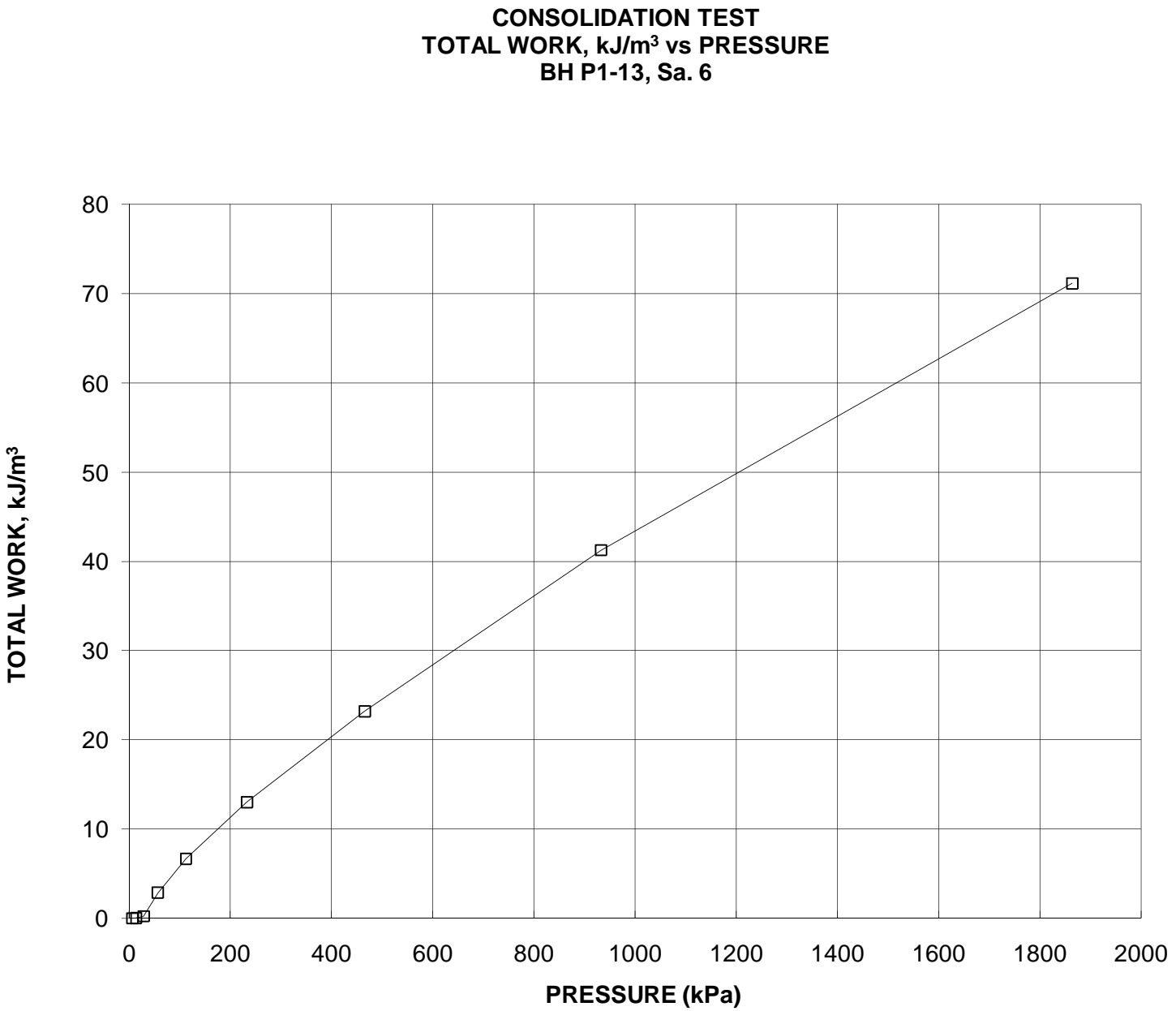


# OEDOMETER CONSOLIDATION SUMMARY

FIGURE B-4

Page 3 of 4

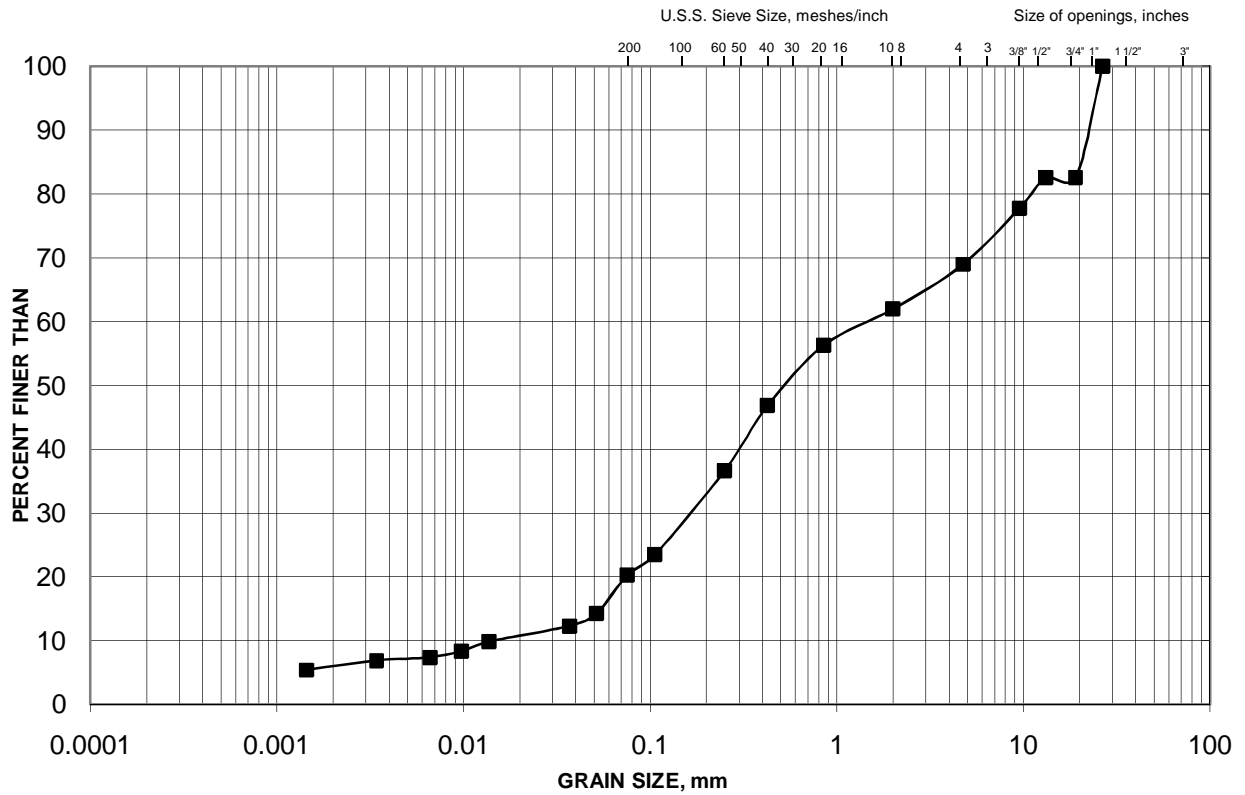




# GRAIN SIZE DISTRIBUTION

## Sand and Gravel

**FIGURE  
B-5**



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

### LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
—■—	P1-12	7	192.5

Project Number: 07-1191-0020-2

Checked By: AB

**Golder Associates**

Date: August 2011

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

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North America	+ 1 800 275 3281
South America	+ 55 21 3095 9500

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