

**FOUNDATION INVESTIGATION  
AND DESIGN REPORT  
REHABILITATION OF HIGHWAY 400 NBL  
CP RAIL OVERHEAD STRUCTURE  
HIGHWAY 400 FROM HIGHWAY 11 TO 93  
SIMCOE COUNTY  
W.P. 2168-06-00**

Submitted to:

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GEOCRES No.

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August 2008

06-1111-011-2



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06-1111-011-2

## **PART A**

**FOUNDATION INVESTIGATION REPORT  
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W.P. 2168-06-00**

**LIST OF TABLES**

Table 1	Comparison of Feasible Foundation Alternatives, Rehabilitation of Existing Highway 400 NBL-CP Rail Overhead Structure, G.W.P. 2168-06-00
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**LIST OF DRAWINGS**

Drawing 1	Highway 400 NBL, Rehabilitation of Existing CP Rail Overhead Structure, Borehole Locations and Soil Strata
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**LIST OF FIGURES**

Figure 1	Grain Size Distribution Test Results – Sand to Silty Sand Fill
Figure 2	Grain Size Distribution Test Results – Surficial Silty Sand to Silt
Figure 3	Grain Size Distribution Test Result – Surficial Silty Clay
Figure 4	Plasticity Chart – Surficial Silty Clay
Figure 5	Grain Size Distribution Test Results – Sand
Figure 6	Grain Size Distribution Test Result – Gravelly Sand

**LIST OF APPENDICES**

Appendix A	Records of Boreholes from 1957 Investigation by Department of Highways, Ontario
Appendix B	Records of Boreholes from 2006 Investigation by Golder Associates Ltd.
Appendix C	Non-Standard Special Provisions

## TABLE OF CONTENTS

<b><u>SECTION</u></b>	<b><u>PAGE</u></b>
<b>PART A - FOUNDATION INVESTIGATION REPORT</b>	
1.0 INTRODUCTION.....	1
2.0 SITE DESCRIPTION .....	2
3.0 INVESTIGATION PROCEDURES .....	3
4.0 SITE GEOLOGY AND STRATIGRAPHY .....	5
4.1 Regional Geological Conditions .....	5
4.2 Site Stratigraphy .....	5
4.2.1 Embankment Fill .....	6
4.2.2 Surficial Silty Sand to Silt .....	6
4.2.3 Silty Clay to Clay .....	6
4.2.4 Sand .....	7
4.2.5 Gravelly Sand .....	8
4.3 Groundwater Conditions .....	9
5.0 CLOSURE .....	10
<b>PART B - FOUNDATION DESIGN REPORT</b>	
6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS .....	11
6.1 General .....	11
6.2 Assessment of Existing H-Pile Foundations .....	11
6.2.1 Abutment Piles .....	12
6.2.2 Pier Piles .....	13
6.3 Design Requirements and Constructability Issues .....	14
6.4 Foundation Options for Pier Reconstruction .....	15
6.5 New Steel H-Pile Foundations for Piers .....	16
6.5.1 Geotechnical Resistance .....	16
6.5.2 Resistance to Lateral Loads .....	17
6.5.3 Frost Protection .....	18
6.6 Lateral Earth Pressures .....	18
6.7 Construction Considerations .....	21
6.7.1 Excavation and Groundwater Control .....	21
6.7.2 Temporary Roadway / Track Protection .....	21
6.7.2.1 Lateral Earth Pressures for Shoring Design .....	21
6.7.3 Vibration Monitoring During Pile Installation .....	23
7.0 CLOSURE .....	24

In Order  
Following  
Page 24

Table 1  
Lists of Abbreviations and Symbols  
Record of Borehole 07-1  
Drawing 1  
Figures 1 to 6  
Appendices A to C

## **1.0 INTRODUCTION**

Golder Associates Ltd. (Golder) has been retained by Transenco Limited (Transenco) on behalf of the Ministry of Transportation, Ontario (MTO) to provide foundation engineering services associated with the rehabilitation of the existing Highway 400 Northbound Lane (NBL) structure over CP Rail, about 0.6 km south of the Highway 400-Horseshoe Valley Road interchange, in the City of Barrie (former Township of Vespra), in the County of Simcoe, Ontario.

This report addresses the results of the current foundation investigation carried out for the rehabilitation of the existing Highway 400 NBL-CP Rail overhead structure. The current investigation was supplemented with information from previous investigations and widenings at this structure site, as follows:

- MTO GEOCRES No. 31D-251: Report titled “Foundation Investigation Report for C.P.R. Overhead S.B.L. and C.P.R. Overhead Widening N.B.L., Highway 400, District 5, Owen Sound, W.P. 99-75-03/18, Site No. 30-89A/B”, Highway Engineering Division – Engineering Materials Office – Soil Mechanics Section, dated 1976. This report contains borehole logs from an original investigation dated 1957.
- CPR Overhead Widening (North Bound Lanes), N.B.L. Bridge, Highway 400, Design Drawings Nos. 1 to 14 (Sheet Nos. 136 to 149), Morrison, Hershfield, Burgess & Huggins, Limited, dated September and November 1977.
- Golder’s report titled “Foundation Investigation and Design Report for Highway 400 NBL Bridge Widening Over CP Rail, G.W.P. 167-99-00”, dated June 2006. This report contains borehole logs from the field investigation carried out for the median widening of the existing Highway 400 NBL-CP Rail overhead structure in March 2006.

The terms of reference and scope of work for the foundation investigation are outlined in MTO’s Terms of Reference for Foundations Engineering Services, and in Golder’s revised proposal letter, dated May 4, 2007, entitled “Additional Foundation Engineering Services for Highway 400 NBL-CP Rail Overhead Rehabilitation”.

## **2.0 SITE DESCRIPTION**

The existing Highway 400 structures over the CP Rail tracks are located in the City of Barrie, about 600 m south of the Highway 400-Horseshoe Valley Road (County Road 22) interchange and about 4.3 km south of the Highway 400-Highway 93 interchange. The existing Highway 400 NBL overhead structure was constructed in 1959, and was widened to the east in 1979; the existing Highway 400 SBL overhead structure was also constructed in 1979. Both structures consist of three spans, with the abutments and piers supported on driven steel H-pile foundations.

The natural ground surface in the vicinity of the structure site varies from about Elevation 254 m to 249 m, generally declining from the southeast toward the north and west. The existing Highway 400 northbound and southbound lanes have been constructed on embankment fill that is up to about 8 m to 9 m in height in the immediate vicinity of the structures; the embankment foreslopes and side slopes are oriented at approximately 2 horizontal to 1 vertical (2H:1V). The existing Highway 400 NBL grade varies from about Elevation 258.5 m to 259.5 m at the north and south abutment locations, respectively. The ground surface in the centre median area varies from about Elevation 257.5 m to 258.5 m within the limits of the existing approach embankments. The CP Rail tracks have been constructed at or near the original ground surface, at about Elevation 250 m under the Highway 400 structures.

This is a rural area, with the majority of the surrounding land occupied by farm fields and trees. A commercial operation is present approximately 150 m to the northeast of the structure site.

### **3.0 INVESTIGATION PROCEDURES**

The borehole investigation for the rehabilitation of the existing Highway 400 NBL-CP Rail overhead structure was carried out between October 16 and 18, 2007, at which time one borehole (Borehole 07-1) was advanced near the CP Rail track, approximately 22 m east of the existing Highway 400 NBL structure. The purpose of this borehole was to compare and corroborate the Standard Penetration Test (SPT) “N” values obtained during the 1957 and 2006 investigations at the site. The location of Borehole 07-1, as well as the boreholes advanced during the previous investigations at this site, are shown on Drawing 1.

Initially, a dynamic cone was driven to a depth of 26 m to assess the variation in the density of the soil. Subsequently, Borehole 07-1 was drilled to a total depth of 28 m using a D-50 turbo drill rig, supplied and operated by Walker Drilling Ltd. of Utopia, Ontario. The borehole was advanced using 200 mm inner diameter hollow stem augers to a depth of about 9 m (Elevation 241 m); below this depth, the borehole was advanced using 75 mm outer diameter N-casing driven manually with a rope-pull hammer, to simulate the 1957 drilling procedure. At each 1.5 m sampling interval, the N-casing was washed out to the casing bottom using a side discharge chopping bit, to minimize loosening of the water-bearing cohesionless soil at the sampling depth. A “head” of water was maintained within the auger column and N-casing to minimize disturbance of the soils due to water inflow to the borehole during sampling.

Samples of the overburden were obtained at 0.75 m intervals to a depth of approximately 3.5 m, and at 1.5 m intervals between approximately 3.5 m and 28 m depth. The samples were obtained using a 50 mm outside diameter split-spoon sampler driven with an automatic hammer, in accordance with the Standard Penetration Test procedure (ASTM D1586-99).

The water level in the open Borehole 07-1 was observed and recorded throughout the drilling operations. Two standpipe piezometers were installed: one at a depth of 1.5 m in a separate borehole, and one at a depth of 25.8 m, to monitor the groundwater level(s) at the site. The piezometers consist of a 1.5 m long slotted screen installed within a filter sand pack, then backfilled to ground surface using bentonite pellets; the details of the piezometer installation are shown on the borehole record.

The field work was supervised on a full-time basis by a member of Golder’s staff who located the borehole in the field, arranged for clearance of underground utilities, supervised the drilling, sampling, and in situ testing operations, logged the boreholes, and examined and cared for the soil samples. The soil samples were identified in the field, placed in labelled containers, and transported to Golder’s geotechnical laboratory in Mississauga for further examination and testing. Index and classification tests consisting of natural water content determinations, Atterberg limits testing and grain size distribution analyses were carried out on selected soil

samples. All of the laboratory tests were carried out to MTO and/or ASTM Standards as appropriate.

The as-drilled location for Borehole 07-1, as well as Boreholes 06-1 to 06-4, were measured in the field by Golder relative to existing site features, and the ground surface elevation at the borehole locations was determined from the digital terrain model for the site. The following table summarizes the borehole locations (northing and easting coordinates referenced to the MTM NAD83 coordinate system) and ground surface elevations (referenced to the geodetic datum).

<i><b>Borehole Number</b></i>	<i><b>MTM NAD83 Northing (m)</b></i>	<i><b>MTM NAD83 Easting (m)</b></i>	<i><b>Ground Surface Elevation (m)</b></i>
06-1	4,931,044.9	286,036.4	258.0
06-2	4,931,062.1	286,026.0	257.9
06-3	4,930,984.1	286,079.0	258.5
06-4	4,930,967.3	286,089.4	258.4
07-1	4,931,075.4	286,061.5	250.0



## **4.0 SITE GEOLOGY AND STRATIGRAPHY**

### **4.1 Regional Geological Conditions**

This section of Highway 400 is located within the physiographic region known as the Simcoe Uplands, according to *The Physiography of Southern Ontario*<sup>1</sup>. This region extends from Barrie northerly to beyond the structure site.

The general topography within the Simcoe Uplands consists of sloping till (moraine) plains, separated by steep-walled valleys. The surficial soils in this region are primarily sandy till deposits that are known to contain boulders; low-lying valley areas may be infilled with sand and gravel deposits, which represent shoreline and stream deposits of a former glacial lake that once flooded the area. To the northwest of Lake Simcoe, a broad belt of sand hills extends from near Midhurst through to Bass Lake, and the Highway 400-CP Rail structure site is located in this area.

### **4.2 Site Stratigraphy**

The detailed subsurface soil and groundwater conditions encountered in the boreholes and the result of the in situ and laboratory testing are given on the Record of Borehole sheets and on Figures 1 to 6 following the text of this report. The borehole records from the 1957 and 2006 investigations are included in Appendices A and B respectively. The stratigraphic boundaries shown on the borehole records are inferred from non-continuous sampling, observations of drilling progress and SPT results. These boundaries, therefore, represent transitions between soil types rather than exact planes of geological change. Further, subsurface conditions will vary between and beyond the location of the borehole.

The interpreted stratigraphy at the site, based on the results of the boreholes from the current and previous investigations, is shown on Drawing 1. In general, the soils encountered in the boreholes consist of sand to silty sand fill (associated with the Highway 400 embankments), overlying relatively thin surficial deposits of loose to compact silty sand to silt and stiff to very stiff silty clay. These surficial deposits are underlain by an extensive deposit of sand, which grades to a stratified sand containing silty clay layers at depth, and then to a gravelly sand.

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

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

#### **4.2.1 Embankment Fill**

Between 7.6 m and 9.1 m of embankment fill was encountered immediately below the ground surface in Boreholes 06-1 to 06-4, which were all advanced through the existing highway embankment fill in the centre median area. Borehole 07-1, advanced near the east toe of the Highway 400 NBL embankment, encountered approximately 0.9 m of fill. The fill consists of sand to silty sand, containing trace gravel and clay, and trace quantities of organic material; cobbles were encountered within the upper 1.5 m of the fill in Borehole 06-4. The results of grain size distribution testing conducted on two selected samples of the fill are shown on Figure 1.

The measured SPT “N” values range from 2 to 39 blows per 0.3 m of penetration, indicating that the fill has a very loose to dense relative density; however, the SPT “N” values are generally greater than 10 blows per 0.3 of penetration, indicating that these soils typically have a compact to dense relative density.

#### **4.2.2 Surficial Silty Sand to Silt**

A surficial layer of silty sand to silt was encountered below the embankment fill in Boreholes 06-1 to 06-4, and below the topsoil in Boreholes 57-1 and 57-4; the surface of this layer was encountered between Elevations 248.7 m and 250.3 m. It is typically 1.4 m to 2.5 m in thickness, except in Borehole 06-3 where the deposit is approximately 4.1 m in thickness; at this location, no clay layer was encountered underlying the surficial silt deposit.

The surficial silty sand to silt contains trace to some clay and trace gravel; the results of grain size distribution tests carried out on two selected samples of the surficial silty sand to silt are shown on Figure 2.

The measured SPT “N” values within the surficial silty sand to silt deposit generally range from 5 to 27 blows per 0.3 m of penetration, indicative of a loose to compact relative density; however, SPT “N” values equal to the weight of the rods were recorded within the wet, silty surficial soils in Borehole 06-3. The lower measured SPT “N” values are attributed to heaving/disturbance of the saturated silty sand to silt soils at the base of the boreholes during sampling operations, although a head of water was used during the 2006 drilling to attempt to try to control such heave.

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

A 1.8 m to 2.7 m thick layer of silty clay to clay was encountered below the surficial silty sand to silt layer in all of the boreholes except Borehole 06-3. The surface of this silty clay to clay deposit was encountered in the current and previous (1957 and 2006) boreholes between Elevations 247.3 m and 249.1 m.

The silty clay to clay contains trace sand and gravel; the result of a grain size distribution test completed on one selected sample of the silty clay to clay deposit is shown on Figure 3. Atterberg limits testing was carried out on five selected samples of the cohesive deposit obtained from the 2006 and 2007 investigations, and three samples obtained from the 1957 investigation. All of the test results are summarized in the following table, and the results from the 2006 and 2007 investigations are plotted on a plasticity chart on Figure 4. These Atterberg limits test results confirm that this deposit varies from an intermediate plasticity silty clay to a high plasticity clay.

<i><b>Borehole Number</b></i>	<i><b>Sample Number</b></i>	<i><b>Sample Elevation</b></i>	<i><b>Liquid Limit (%)</b></i>	<i><b>Plastic Limit (%)</b></i>	<i><b>Plasticity Index (%)</b></i>
07-1	2	249.2 m	49	20	29
07-1	3	248.6 m	49	20	29
06-1	10	247.0 m	54	22	32
06-2	9	248.5 m	58	26	32
06-4	10	247.5 m	68	26	42
57-1	3	246.5 m	59	25	34
57-1	5	245.0 m	44	23	21
57-4	4	247.0 m	56	31	25

The measured SPT “N” values within the silty clay to clay range from 7 to 20 blows per 0.3 m of penetration, while in situ field vane testing carried out during the current investigation measured undrained shear strengths exceeding 100 kPa. The 1957 borehole records (contained in Appendix A) show that unconfined compression testing on selected samples of the silty clay to clay measured undrained shear strengths ranging from approximately 62 kPa to 120 kPa. These results indicate that the silty clay to clay typically has a stiff to very stiff consistency. The measured bulk unit weight of the silty clay to clay (from the 1957 investigation) ranges from approximately 18.4 kN/m<sup>3</sup> to 19.6 kN/m<sup>3</sup>.

#### **4.2.4 Sand**

An extensive sand deposit was encountered below the surficial deposits of silty sand to silt and silty clay to clay (where present) in all of the boreholes. The surface of the sand deposit was encountered between Elevations 244.6 m and 246.3 m. Borehole 06-3 fully penetrated the sand deposit, extending into an underlying gravelly sand deposit; at this location, the sand deposit is approximately 28.9 m in thickness, extending down to approximately Elevation 216.8 m. The remaining boreholes were terminated within this deposit: Boreholes 06-1, 06-2, and 06-4 were terminated at depths ranging from 18.9 m to 40.1 m below the embankment ground surface, while Boreholes 07-1, 57-1 and 57-4 were terminated within this deposit at depths of 28.0 m (Elevation 222.0 m), 15.5 m and 18.0 m below the original ground surface, respectively.

The sand deposit contains trace to some silt and, in some locations, trace to some gravel; interlayers of silty clay up to 50 mm thickness were observed within samples recovered from below about Elevation 233.3 m in Borehole 07-1, and below about Elevations 232.1 m and

235.7 m in Boreholes 06-1 and 06-3. Trace quantities of organic material were noted within the upper zone of the sand in Borehole 57-1. The results of grain size distribution testing conducted on eight selected samples of the sand deposit are shown on Figure 5.

The measured SPT “N” values within the sand layer vary from 1 to greater than 100 blows per 0.3 m of penetration. The lower measured SPT “N” values of 1 to 9 blows per 0.3 m of penetration were generally measured within the upper 10 m in Boreholes 06-1 and 06-3 and, to a lesser extent, in Boreholes 06-2 and 06-4; these lower SPT “N” values are considered to result from some loosening/disturbance of the saturated sand at the base of the boreholes during sampling, despite attempts to maintain a balanced water pressure head at the sampling level. The SPT “N” values from the same horizon in Borehole 07-1 and in the 1957 boreholes are higher, ranging from 12 to 51 blows per 0.3 m of penetration, and it is therefore considered that the upper portion of the sand deposit has a compact to dense (but typically compact) relative density.

At the north abutment and north pier (Boreholes 06-1, 06-2 and 57-1), and at Borehole 07-1, the sand is typically compact above approximately Elevation 232 m, and dense to very dense below this elevation. At the south abutment and south pier (Boreholes 06-3, 06-4 and 57-4), the sand is typically compact above approximately Elevation 234 m, becoming dense to very dense below this level. Measured SPT “N” values of greater than 100 blows per 0.3 m of penetration were obtained within the sand in Borehole 06-1, below approximately Elevation 222 m (about 36 m below the Highway 400 median grade at this location) and greater than 70 blows per 0.3 m of penetration were obtained in Borehole 07-1 below Elevation 226 m.

The dynamic cone penetration test (DCPT) results for a test adjacent to Borehole 07-1 are provided on the borehole record. The DCPT in this deposit measured values consistent with a compact sand material down to about Elevation 241 m, a compact to dense material down to approximately Elevation 228 m, and dense to very dense material below about Elevation 228 m.

#### **4.2.5 Gravelly Sand**

A deposit of gravelly sand was encountered in Borehole 06-3 below the extensive sand deposit; the surface of this layer was encountered at Elevation 216.8 m (at a depth of approximately 41.7 m below the Highway 400 median grade at this location).

The gravelly sand layer contains trace to some silt and trace clay; the result of a grain size distribution test carried out on one selected sample of this stratum is shown on Figure 6.

The measured SPT “N” values within the gravelly sand were 91 and greater than 100 blows per 0.3 m of penetration, indicating that this material has a very dense relative density.

### 4.3 Groundwater Conditions

The details of the water levels observed in the open boreholes during and upon completion of drilling are summarized on the record of Borehole 07-1 following the text of this report, and on the records of boreholes advanced during the 1957 and 2006 investigations in Appendices A and B, respectively. The water levels measured within the open boreholes upon completion of drilling are summarized in the table below:

<i>Borehole Number</i>	<i>Ground Surface Elevation (m)</i>	<i>Depth to Water Level (m)</i>	<i>Water Level Elevation (m)</i>	<i>Date</i>
06-1	258.0	12.5	245.5	March 21, 2006
06-2	257.9	12.8	245.1	March 22, 2006
06-3	258.5	13.3	245.2	March 26, 2006
06-4	258.4	13.7	244.7	March 29, 2006
57-1	249.0	4.3	244.7	June 26, 1957
57-4	249.4	*0.9	248.5*	June 26, 1957

\* Depth and elevation at which groundwater was first encountered in Borehole 57-4. However, when this borehole penetrated through the silty clay deposit, the water level in the open borehole dropped approximately 3 m to about Elevation 245.5 m.

Two standpipe piezometers were installed in Borehole 07-1 to monitor the groundwater level(s) at the site: one shallow piezometer within the surficial sand, and one deeper piezometer installed within the stratified sand deposit. The following table summarized the water levels measured in the piezometers:

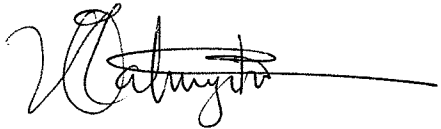
<i>Borehole Number</i>	<i>Piezometer Installation</i>	<i>October 18, 2007</i>		<i>November 14, 2007</i>	
		<i>Depth</i>	<i>Elevation</i>	<i>Depth</i>	<i>Elevation</i>
07-1	Shallow	1.1	248.9	1.2	248.8
	Deep	5.2	244.8	5.3	244.7

The groundwater levels at the site should be expected to fluctuate with variations in precipitation, and should be expected to be higher during wet periods of the year. In addition, the surficial silty sand to silt deposit is water-bearing, with groundwater “perched” on top of the underlying, less permeable silty clay to clay deposit.

## 5.0 CLOSURE

The borehole investigation program documented in this report was supervised by Mr. Pat Speirs, a senior geotechnical technician with Golder. This report was prepared by Ms. Veronica Olatunji, and reviewed by Ms. Lisa Coyne, P.Eng., an Associate and geotechnical engineer with Golder. Mr. Fintan Heffernan, P.Eng., a Designated MTO Contact for Golder, carried out an independent quality control review of the report.

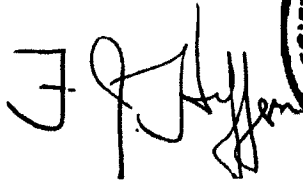
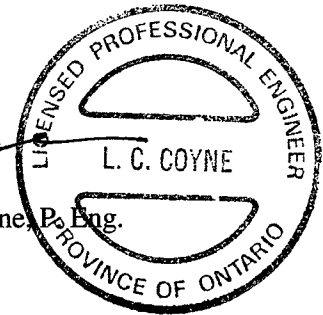
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August 2008

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## **PART B**

**FOUNDATION DESIGN REPORT  
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CP RAIL OVERHEAD STRUCTURE  
HIGHWAY 400 FROM HIGHWAY 11 TO 93  
W.P. 2168-06-00**

## **6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS**

### **6.1 General**

This section of the report provides foundation recommendations for the design of the proposed rehabilitation of the existing Highway 400 NBL-CP Rail overhead structure. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the current and previous subsurface investigations at this site. The interpretation and recommendations contained in this report are intended to provide the designers with sufficient information to assess the feasible foundation alternatives and to carry out detail design of the foundations for the proposed structure rehabilitation. Where comments are made on construction, they are provided in order to highlight those aspects that could affect the design of the project, and for which special provisions or operational constraints may be required in the Contract Documents. Those requiring information on aspects of construction should make their own interpretation of the factual information provided as it may affect equipment selection, proposed construction methods, scheduling and the like.

Golder's Foundation Investigation and Design Report for the westward widening of the existing Highway 400 NBL-CP Rail overhead structure was submitted previously (June 2006) as part of this project. Construction of the westward (median) widening of the Highway 400 NBL-CP Rail overhead structure is currently in progress, to match the three-span configuration of the existing NBL structure.

During the 2006 investigation for the westward widening project, questions were raised about the adequacy of the existing piles for support of the proposed rehabilitation of the existing Highway 400 NBL-CP Rail overhead structure. As per the RFP Terms of Reference prepared by the MTO Foundations Section, foundation engineering services for the detailed design of the rehabilitation of the existing Highway 400 NBL-CP Rail overhead structure are required for the following items:

- Evaluation of the adequacy of the existing H-pile foundations for the bridge piers for the proposed rehabilitation, including recommendations for necessary modifications, and consideration of constructability aspects such as clearance requirements and construction staging.
- Assessment of foundation adequacy for the existing abutments and recommendations for retrofitting measures, if necessary.

### **6.2 Assessment of Existing H-Pile Foundations**

The existing Highway 400 NBL-CP Rail overhead structure was originally constructed in 1959, and was widened toward the east in 1979. Both the original Highway 400 NBL structure and its



eastward widening are three spans, with span lengths of approximately 19 m between the abutments and piers, and a span length of about 24 m between the north and south piers. The abutments and piers are supported on driven steel H-pile foundations.

The following design drawings are available for the original Highway 400 NBL structure and its eastward widening:

- Drawing D3944-1, “Craighurst C.P.R. Overhead – General Arrangement” and Drawing D3944-2, “Craighurst C.P.R. Overhead – Layout of Piles and Piers and Details of Boreholes”, both prepared by T. O. Lazarides and Associates Limited, dated July 31, 1957, for the original Highway 400 NBL structure; and
- Sheet 138, Design Drawing No. 3, “C.P.R. Overhead Widening – Northbound Lanes, N.B.L. Bridge, Layout of Piles and Piers”, prepared by Morrison Hershfield, dated September 28, 1977, for the eastward widening.

The following sub-sections provide discussion on the existing abutment and pier piles, based on information from the design drawings as referenced above.

### **6.2.1 Abutment Piles**

The existing north and south abutments for the original Highway 400 NBL-CP Rail overhead structure are supported on steel 12BP53 (HP 310x79) piles. Some of the piles are vertical, and some are battered at 1H:3V. Based on a memorandum dated February 16, 1977, prepared by Mr. Ken Selby of the MTO Foundations Section, it is inferred that the design load (working stress) for the abutment piles for the original structure was 40 tons (approximately 355 kN); this corresponds to an SLS value, and the factored ULS value would be approximately 535 kN. The design pile tip elevation(s) for the original abutment piles are not shown on the available design drawings.

The existing north and south abutments for the eastward widening of the Highway 400 NBL-CP Rail overhead structure are supported on steel 12BP53 (HP 310x79) piles with a design pile tip at approximately Elevation 243.8 m (Elevation 800 ft.). Some of the piles are vertical, and some are battered at 1H:3V. The design drawing notes indicate that the abutment piles for the eastward widening were to be driven in accordance with Standard SS3-11, Drawing 13 (i.e. the Hiley formula) using a design working load of 40 tons per pile (approximately 355 kN) for SLS; again, this corresponds to an SLS value, and the factored ULS value would be approximately 535 kN.

Golder has researched the pile driving records for the eastward widening of the Highway 400 NBL-CP Rail overhead structure. According to these records, the abutment piles for the eastward widening were driven approximately 10 m deeper than the design pile tip elevation, to between Elevation 232 m and 234 m, to satisfy the design capacity as determined by the Hiley formula. The piles were driven using a D12 diesel hammer (which is a light hammer by present day

standards), with a rated capacity of 33,500 Joules. The recorded set was 6 to 8 blows for the last 25 mm of driving, and the recorded rebounds were on the order of 6 mm to 10 mm.

The pile driving results from the 1979 widening work are consistent with the recent abutment pile driving experience for the new westward widening, in which an HP 310x110 pile was driven at the south abutment to Elevation 234 m, and achieved an ultimate resistance of 1,260 kN as measured by the Hiley formula, corresponding to a factored geotechnical resistance at ULS of 630 kN. This pile for the new westward widening was driven with a heavier hammer (D19-42 with a rated capacity of about 57,000 Joules), and the set was 1 blow for 15 mm.

The abutment piles for the original 1957 structure were probably driven with a hammer similar to the D12, and to a set and the 40 ton capacity as likely determined by the Engineering News Record formula, which was in use at that time. Golder considers that, based on the standards of the time and the satisfactory performance of the abutments for the original structure to date, the required capacity of 40 tons (working load) would have been satisfied. With the widening of the bridge to the east and now to the west, the wing walls have been/will be removed and the loading on the abutment piles has been/will be reduced, potentially sufficiently to offset the heavier loading on the bridge with current vehicle loads. As such, it is understood that the existing 1957 and 1979 abutment piles with a 40 ton working load (corresponding to a factored geotechnical resistance at ULS of 535 kPa, and a geotechnical resistance at SLS of 355 kPa) will be sufficient for support of the rehabilitated structure; therefore, the existing abutment piles and pile cap can be left in place and used for support of the rehabilitation.

## **6.2.2 Pier Piles**

The existing north and south piers for the original Highway 400 NBL-CP Rail overhead structure are supported on steel 14BP73 (HP 360x108) piles, which are battered at 1H:8V. According to a memorandum dated February 16, 1977 that was prepared by Mr. Ken Selby of the MTO Foundations Section, it is inferred that the design load for the pier piles was 75 tons (corresponding to a geotechnical resistance at SLS of 677 kN, and a factored geotechnical resistance at ULS of approximately 850 kN to 1,000 kN). The design pile tip elevation(s) for the existing piers are not shown on the available design drawings.

The north and south piers for the existing eastward widening of the Highway 400 NBL-CP Rail overhead structure are supported on steel 12BP89 (HP 310x132) piles with a design pile tip at approximately Elevation 237.7 m (Elevation 780 ft.). These piles are shown in the design drawings to be battered at 1H:8V. The design drawing notes indicate that pier piles for the eastward widening were to be driven in accordance with Standard SS3-11, Drawing.13 (i.e. the Hiley formula), using a design working load of 80 tons per pile (approximately 700 kN).

From the available 1979 pile driving records, it is now known that the pier piles for the eastward widening were driven approximately 7 m deeper than the design elevation, to approximately Elevation 231 m, to satisfy the design capacity as determined by the Hiley formula. The piles were driven using a D12 diesel hammer, with a rated capacity of 33,500 Joules. The recorded set was approximately 11 blows for the last 25 mm and the rebound was about 3 mm.

The pile driving results from the 1979 widening work are consistent with the recent pier pile driving experience for the new westward widening, where HP 310x110 piles have been driven to about Elevation 229 m and achieved an ultimate resistance of greater than 1,700 kN as measured by the Hiley formula, corresponding to a factored geotechnical resistance at ULS of greater than 850 kN. These new pier piles for the westward widening were driven with a heavier hammer (D19-42 with a rated capacity of about 57,000 Joules), and the set was 3 blows for 25 mm.

### **6.3 Design Requirements and Constructability Issues**

It is understood that, based on the assessment of the existing abutment pile capacity as presented in Section 6.2.1, together with the satisfactory foundation performance to date, the existing abutment piles will be sufficient for support of the rehabilitated structure.

Due to CP Rail's crash protection standards, the rehabilitated piers must incorporate a crash wall (similar to that for the westward structure widening), and therefore new pier foundations will be required. The constructability of the piers has been considered in some depth, as there will be limited access and working room particularly during the first stage, based on the required traffic staging sequence.

Under the current construction staging sequence, the Highway 400 northbound traffic will be transferred onto the new westward widening structure; however, the eastern portion of the existing Highway 400 NBL-CP Rail overhead structure must remain in place during the first stage of the pier replacement, to allow the Horseshoe Valley Road off-ramp to remain open. Based on these traffic staging requirements and the resulting structural requirements related to the existing NBL structure, the first stage of pier reconstruction will take place within an approximately 8 m width (between the east edge of the westward widening structure, and the west edge of that portion of the existing NBL structure that must remain in place during the first stage of construction). Once the first stage of pier reconstruction and bridge deck replacement is completed, the ramp traffic will be shifted onto this new structure, and the remaining eastern portion of the bridge deck for the existing NBL structure will be demolished to allow reconstruction of the piers and bridge deck.

Access to the pier level for the first and subsequent stages of reconstruction is planned to be through the median and under the westward widening structure, with extension of the "working

platform” at the pier level. It is understood that the vertical clearance under the westward widening structure will be approximately 4 m.

Golder has reviewed the constructability of the piers with representatives from MTO, Transenco, Ellis Engineering and a pile driving / shoring contractor, based on the staging sequence and access conditions as described above. Based on this review, it is considered that conventional pile driving equipment can be mobilized into position through the median access and working platform under the westward widening structure. Since the existing bridge deck will be removed for both the first and subsequent phases of the rehabilitation, sufficient headroom will be available for operation of conventional pile driving equipment.

As an alternative to the above approach, temporary bridging or temporary shoring could be provided between the existing abutments and piers to provide access for a pile driving rig to operate at or near the bridge deck level.

#### **6.4 Foundation Options for Pier Reconstruction**

Based on the subsoil conditions, access conditions and consideration of constructability, the following foundation options have been considered for the pier reconstruction that is required as part of the rehabilitation of the Highway 400 NBL-CP Rail overhead structure:

- **Driven steel H-pile foundations:** This foundation option is considered the most practical and appropriate option for support of the replacement piers, since there will be minimal settlement of the new foundation elements relative to the existing abutments and to the widened structure. Consideration can be given to either end-bearing piles founded at depth within the “100-blow” soil, or to friction piles driven to a more moderate depth. Friction piles have a slight advantage over end-bearing piles since the friction piles are expected to induce less vibration (which could affect the existing NBL and SBL structures) during driving, and since friction piles would be most consistent with the existing structure foundations.
- **Micropile foundations:** This technology has been applied successfully in highway bridge projects in North America for many years. Based on the site conditions, this option is expected to be feasible to be carried out within the limited working space between the existing rail lines and the bridge foundation elements, and is expected to induce less vibration during construction; however, the actual resistance of each micropile will be lower than for a conventional driven steel H-pile. Micropiles are typically used to increase the capacity of existing foundations and may be of use in increasing the lateral capacity of the existing abutment piles, if this is necessary; in this case, a feasibility study and detailed design for micropile foundations would be required. However, in the case of the new piers, as discussed above, new steel H-piles are constructable and will provide the required capacity.

A summary comparison of the advantages, disadvantages, approximate costs, and risks associated with the above foundation options is presented in Table 1 following the text of this report. From a

foundations perspective, new steel H-piles are considered to be the most appropriate option since they are constructable and will provide the required capacity cost effectively. Recommendations for the most feasible and practical options (friction piles and end-bearing piles) are provided in the following sections.

## **6.5 New Steel H-Pile Foundations for Piers**

The new piers for the structure rehabilitation may be supported on friction steel H-piles driven into the dense sand deposit. Alternatively, and to achieve higher axial resistances, steel H-piles driven to found within the “100-blow” sand and/or gravelly sand deposit (i.e. deep-seated end-bearing piles) could be used.

For the installation of steel H-piles, consideration must be given to the potential presence of cobbles and boulders within the extensive sand and lower gravelly sand deposits at this site. The H-piles should be stiffened with MTO flange plates for protection during driving, in accordance with MTO’s Special Provision SP903S01.

Pile installation should be in accordance with MTO’s Special Provision SP903S01. The pile termination or set criteria will be dependent on the pile driving hammer type, helmet, selected pile and length of pile. The criteria must therefore be established at the time of construction after the piling equipment is known. The driving in this granular soil should be controlled by the Hiley Formula as per MTO Standards (SS103-11).

### **6.5.1 Geotechnical Resistance**

#### **Friction Piles**

For design, based on pile capacity calculations together with the results of pile driving for both the westward and eastward widening of the Highway 400 NBL structure, the factored axial resistance at ULS for new steel HP 310x110 piles driven to a design pile tip elevation of 228 m may be taken as 850 kN. The axial resistance at SLS for 25 mm of settlement may be taken as 700 kN. For friction piles driven into the dense sand, the following note is considered appropriate for the design and site conditions assuming that a resistance factor of 0.5 (as suggested by MTO Foundations Section) is applied to the use of the Hiley formula:

*“Piles to be driven in accordance with Standard SS 103-11 using an ultimate capacity of 1,700 kN per pile.”*

#### **“End-Bearing” Piles**

Alternatively, the steel H-piles could be driven into the very dense sand that exists below about Elevation 217 m at the south pier/abutment area and Elevation 222 m at the north pier/abutment

area. Some piles were driven to Elevation 222 m at the south abutment for the westward structure widening, and developed an ultimate resistance of 1,800 kN to 2,200 kN (as determined by the Hiley formula), corresponding to a factored geotechnical resistance at ULS of 900 kN to 1,100 kN. It may be necessary at some locations to drive below the target pile tip elevations given above to develop higher capacities for the piles. For piles driven into the dense sand, the following note is considered appropriate for the design and site conditions assuming that a resistance factor of 0.5 (as suggested by MTO Foundations Section) is applied to the use of the Hiley formula:

*“Piles to be driven in accordance with Standard SS 103-11 using an ultimate capacity of 2,200 kN per pile.”*

### 6.5.2 Resistance to Lateral Loads

Resistance to lateral loading can be derived using vertical piles, with enhanced support offered by battered piles, if required. If vertical piles are used, the resistance to lateral loading will have to be derived from the soil in front of the piles.

The resistance to lateral loading in front of the pile may be calculated using subgrade reaction theory, where the coefficient of horizontal subgrade reaction,  $k_h$  (MPa/m), is based on the equations given below.

For cohesive soils:

$$k_h = \frac{67\tau_u}{B} \quad \text{where} \quad \begin{array}{l} B \text{ is the pile diameter (m); and} \\ \tau_u \text{ is the undrained shear strength of the soil (kPa).} \end{array}$$

For cohesionless soils:

$$k_h = \frac{n_h z}{B} \quad \text{where} \quad \begin{array}{l} n_h \text{ is the constant of horizontal subgrade reaction (MPa/m);} \\ z \text{ is the depth (m); and} \\ B \text{ is the pile diameter (m).} \end{array}$$

The following ranges for the value of  $\tau_u$  and  $n_h$  may be used in the structural analysis. The range in values reflects the variability in the subsurface conditions.

<i>Soil Unit</i>	<i><math>\tau_u</math> (kPa)</i>	<i><math>n_h</math> (MPa/m)</i>
<b>North Pier</b>		
Surficial silty sand between Elev. 250 m and 247 m	—	4 to 8
Stiff to very stiff clay between Elev. 247 m and 244.5 m	100	—
Compact water-bearing sand between Elev. 244.5 m and 232 m	—	4 to 8
Dense to very dense water-bearing sand below Elev. 232 m	—	8 to 12

<i>Soil Unit</i>	<i><math>\tau_u</math> (kPa)</i>	<i><math>n_h</math> (MPa/m)</i>
<b>South Pier</b>		
Surficial silty sand between Elev. 250 m and 246 m	–	4 to 8
Compact water-bearing sand between Elev. 246 m and 236 m	–	4 to 8
Dense to very dense water-bearing sand below Elev. 236 m	–	8 to 12

A maximum factored lateral resistance of 110 kN at ULS, and a maximum lateral resistance of 50 kN at SLS (for 10 mm of horizontal deflection at pile cap level) is recommended for HP 310 x 110 piles. These values are based on the “Assessed Horizontal Passive Resistance Values for Various Pile Types” provided in Table C6.8.7.1(a) of the *Commentary* to the *CHBDC*.

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

<i>Pile Spacing in direction of Loading (<math>d</math> = Pile Diameter)</i>	<i>Subgrade Reaction Reduction Factor, <math>R</math></i>
8d	1.00
6d	0.70
4d	0.40
3d	0.25

Reference: Foundations and Earth Structures – Design Manual 7.2, NAVFAC DM-7.2. Department of the Navy, Naval Facilities Engineering Command (1982).

The subgrade reaction reduction factor should be interpolated for pile spacings in between those provided in the above table.

### 6.5.3 Frost Protection

Pile caps for the new piers should be provided with a minimum of 1.6 m of soil cover (see OPSD 3090.101) for frost protection purposes.

## 6.6 Lateral Earth Pressures

The lateral earth pressures acting on the abutment stems and any associated wing walls / retaining walls will depend on the type and method of placement of the backfill materials, on the nature of the soils behind the backfill, on the magnitude of surcharge including construction loadings, on the freedom of lateral movement of the structure, and on the drainage conditions behind the walls.

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

- Select free-draining granular fill meeting the specifications of Ontario Provincial Standard Specifications (OPSS) Granular “A” or Granular “B” Type II but with less than 5 per cent passing the 200 sieve should be used as backfill behind the walls. This fill should be placed and compacted in accordance with MTO’s Special Provision SP105S10. Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the granular backfill requirements with respect to sub-drains and frost taper should be in accordance with OPSD 3501.00 and 3504.00.
- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the wall stem, in accordance with *CHBDC* Section 6.9.3 and Figure 6.9.3. Compaction equipment should be used in accordance with MTO’s SP105S10. Other surcharge loadings should be accounted for in the design, as required.
- The granular fill may be placed either in a zone with width equal to at least 1.6 m behind the back of the wall stem (Case I in Figure C6.9.1(l)(i) of the *Commentary to the CHBDC*) or within the wedge-shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical (1.5H:1V) extending up and back from the rear face of the footing (Case II in Figure C6.9.1(l)(ii) of the *Commentary to the CHBDC*).
- For Case I, the pressures are based on the existing and new embankment fill materials and the following parameters (unfactored) may be used:

Soil unit weight:	20 kN/m <sup>3</sup>
Coefficients of static lateral earth pressure:	
Active, $K_a$	0.33
At rest, $K_o$	0.50

- For Case II, the pressures are based on the granular fill placed for the widening, and the following parameters (unfactored) may be assumed:

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



- If the wall support and superstructure allow lateral yielding of the stem, active earth pressures may be used in the geotechnical design of the structure. If the abutment support does not allow lateral yielding, at-rest earth pressures should be assumed for geotechnical design.
- MTO has confirmed that this structure is an “Emergency Route Bridge” and seismic loading must be taken into account. Seismic loading will result in increased lateral earth pressures acting on the abutment stem. The walls should be designed to withstand the combined lateral loading for the appropriate static pressure conditions given above, plus the earthquake-induced dynamic earth pressure. The zonal acceleration ratio (A) for the Barrie area is 0.05. Based on the subsurface conditions at the site, a 10 to 20 per cent amplification of the ground motion may occur, resulting in an increase in the ground surface acceleration from 0.05g to between 0.055g and 0.06g. The seismic lateral earth pressure coefficients given below have been derived based on a design zonal acceleration ratio of  $A = 0.06$ .
- In accordance with Sections 4.6.4 and C.4.6.4 of the *CHBDC* and its *Commentary*, for structures which allow lateral yielding, the horizontal seismic coefficient,  $k_h$ , used in the calculation of the seismic active pressure coefficient, is taken as 0.5 times the zonal acceleration ratio (i.e.  $k_h = 0.03$ ). For structures that do not allow lateral yielding,  $k_h$  is taken as 1.5 times the zonal acceleration ratio (i.e.  $k_h = 0.09$ ). The seismic active earth pressure coefficient is also dependent on the vertical component of the earthquake acceleration,  $k_v$ . Three discrete values of vertical acceleration are typically selected for analysis, corresponding to  $k_v = +2.3k_h$ ,  $k_v = 0$ , and  $k_v = -2/3$ .
- The following seismic active pressure coefficients ( $K_{AE}$ ) for the two cases (Case I and Case II) may be used in design; these coefficients reflect the maximum  $K_{AE}$  obtained using the  $k_h$  and three values of  $k_v$  as described above. It should be noted that these seismic earth pressure coefficients assume that the back of the wall is vertical and the ground surface behind the wall is flat.

#### SEISMIC ACTIVE PRESSURE COEFFICIENTS, $K_{AE}$

	Case I	Case II
Yielding wall	0.32	0.26
Non-yielding wall	0.37	0.30

Note : These CHBDC seismic  $K_{AE}$  values include the effect of wall friction ( $\delta = \phi'/2$ ) and are less than the static values of  $K_a$  and  $K_o$  reported above for the very low zonal acceleration ratio for this site.

- The above  $K_{AE}$  values for yielding walls are applicable provided that the wall can move up to  $250A$  (mm), where  $A$  is the design zonal acceleration ratio of 0.06. This corresponds to displacements of up to 15 mm at this site.

- The earthquake-induced dynamic pressure distribution, which is to be added to the static earth pressure distribution, is a linear distribution with maximum pressure at the top of the wall and minimum pressure at its toe (i.e. an inverted triangular pressure distribution). The total pressure distribution (static plus seismic) may be determined as follows:

$$P = K \gamma' d + (K_{AE} - K) \gamma' H$$

where:  $K$  is either the static active earth pressure coefficient ( $K_a$ ) or the static at-rest earth pressure coefficient ( $K_o$ );  
 $K_{AE}$  is the seismic active earth pressure coefficient;  
 $\gamma'$  is the effective unit weight of the soil ( $\text{kN/m}^3$ ), which can be taken as  $21 \text{ kN/m}^3$  within the existing embankment fill;  
 $d$  is the depth below the top of the wall (m); and  
 $H$  is the height of the wall above the toe (m)

## **6.7 Construction Considerations**

### **6.7.1 Excavation and Groundwater Control**

At the pier locations, the depth of excavation should be minimized consistent with frost depth requirements. These excavations would extend into the surficial silty sand to silt deposit, where the water level was measured at Elevation 248.8 m (i.e., at a depth of 1.2 m) in November 2007. Based on the experience during the excavation and construction of the piers for the westward structure widening, groundwater inflow is expected to be relatively minor during drier periods of the year. Some groundwater control should be anticipated for the pier excavations, particularly during wet periods of the year; however, as the seepage volumes are expected to be relatively small it is not anticipated that a Permit to Take Water will be required.

Open-cut excavation will not be feasible at the pier replacement locations, due to space constraints and proximity to the CP Rail tracks; temporary excavation support and/or a cut-off will be required, as discussed further in Section 6.7.2.

### **6.7.2 Temporary Roadway / Track Protection**

Temporary roadway protection will be required along the west or east side of the Highway 400 northbound lanes to facilitate construction for the structure rehabilitation. The temporary excavation support system adjacent to the highway should be designed and constructed in accordance with MTO's Special Provision 105S19. The lateral movement of the temporary shoring system should meet Performance Level 2 as specified in SP 105S19.

Temporary track protection will be required during excavation for the pier pile caps. It is anticipated that the temporary excavation support could consist of sheet piling. Temporary slopes in the embankment fill adjacent to the pier excavation may be cut at 1.5 horizontal to 1 vertical.

It is recommended that the lateral movement of the temporary shoring system in this area meet Performance Level 1b as specified in MTO's SP 105S19. The shoring design should also meet the requirements set out in the American Railway Engineering and Maintenance of Way (AREMA) *Manual for Railway Engineering*.

Based on the subsoil conditions, the location of the pier excavations relative to the rail tracks, and case studies of ground deformation adjacent to shored excavations, it is anticipated that deformation at the rail line will be limited to less than 25 per cent of the displacement at the shoring line for an appropriately designed and constructed shoring system.

### 6.7.2.1 Lateral Earth Pressures for Shoring Design

Strutted shoring walls should be designed to resist a rectangular earth pressure distribution. The unfactored rectangular earth pressure distribution ( $p$  in  $\text{kN/m}^2$ ; constant with depth), can be calculated as follows:

$$p = K \gamma H$$

where

- $K$  = ratio of horizontal to vertical effective stresses, using 0.25 for level ground behind excavation
- $\gamma$  = soil unit weight, as given in the following table
- $H$  = the total height of the excavation

Shoring walls that are not laterally supported, or that are supported using soil anchors or rakers, should be designed to resist a triangular earth pressure distribution. The unfactored triangular earth pressure distribution ( $p$  in  $\text{kN/m}^2$ ; increasing with depth), can be calculated as follows:

$$P = K_a [ \gamma (H - h_w) + (\gamma - \gamma_w) h_w ]$$

where

- $K_a$  = 0.36 for level ground behind excavation wall;  $K_a$  must be adjusted if there is sloping ground behind the excavation wall
- $\gamma$  = soil bulk unit weight, as given in the table below
- $H$  = the height of the excavation at any point in metres
- $h_w$  = the height of the groundwater level above the base of the excavation

The following table provides bulk and effective unit weights to be used in the above lateral earth pressure equations. If an interlocking sheetpile wall is adopted and dewatering of the surficial silty sand to silt deposit is not required, the shoring wall design should include the water pressure distribution, with the design groundwater level taken at a depth of 1 m below the ground surface. However, adequacy of the shoring design should be checked for a water level at the ground surface, in the event of a failure of the dewatering system during a period of high groundwater.

<i>Soil Unit</i>	<i>Bulk Unit Weight, <math>\gamma</math></i>	<i>Effective Unit Weight, <math>\gamma'</math></i>
Surficial silty sand to silt above Elevation 249 m	20 kN/m <sup>3</sup>	10 kN/m <sup>3</sup>
Surficial silty clay to clay between Elevations 249 m and 245 m	19 kN/m <sup>3</sup>	9 kN/m <sup>3</sup>

Passive toe restraint to the protection system should be determined using a triangular pressure distribution. The coefficient of passive lateral earth pressure,  $K_p$ , and the effective unit weight,  $\gamma'$ , for the soil in front of the piles may be taken as follows:

<i>Soil Deposit</i>	<i><math>K_p</math></i>	<i><math>\gamma'</math></i>
Surficial silty clay to clay above Elevation 245 m	2.8	9 kN/m <sup>3</sup>
Lower sand below Elevation 245 m	3.0	10 kN/m <sup>3</sup>


### 6.7.3 Vibration Monitoring During Pile Installation

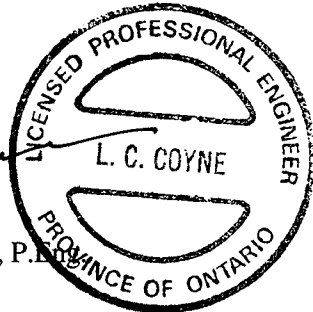
Vibration monitoring should be carried out during pier pile installation for the structure rehabilitation, to ensure that the vibration levels at the existing northbound and southbound structures are maintained below tolerable levels. A maximum peak particle velocity (PPV) of 50 mm/s is recommended at the existing structures. The piles further from the existing Highway 400 NBL structure should be driven first, in order to check the vibration level at the existing structure and if necessary, alter the pile driving criteria for the remaining piles. A sample NSSP is provided in Appendix B, for inclusion in the Contract Documents.


## 7.0 CLOSURE

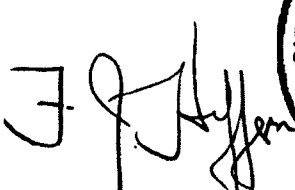
This Foundation Design Report was prepared by Mr. Sen Hu and reviewed by Ms. Lisa Coyne, P.Eng., an Associate and geotechnical engineer with Golder. Technical input and review were provided by Mr. Murty Devata, P.Eng., a Specialist Foundations Consultant to Golder, and Mr. Fin Heffernan, P.Eng., a Designated MTO Contact for Golder.

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**TABLE 1**  
**COMPARISON OF FEASIBLE FOUNDATION ALTERNATIVES**  
**REHABILITATION OF EXISTING HIGHWAY 400 NBL-CP RAIL OVERHEAD STRUCTURE**  
**W.P. 2168-06-00**

<i>Foundation Option</i>	<i>Feasibility</i>	<i>Advantages</i>	<i>Disadvantages</i>	<i>Relative Costs</i>
Friction steel H-Piles driven with conventional pile driving equipment	<ul style="list-style-type: none"> <li>Feasible and practical for support of new piers</li> </ul>	<ul style="list-style-type: none"> <li>Minimal differential settlement between existing and rehabilitation portions of structures, and between foundation elements</li> <li>Pile capacities similar to existing structures</li> </ul>	<ul style="list-style-type: none"> <li>Some vibration impacts on existing Highway 400 NBL and SBL structures</li> </ul>	<ul style="list-style-type: none"> <li>Cost for friction H-piles driven with conventional equipment is comparable to end-bearing piles (in relation to design capacity) and much less expensive than micropiles</li> </ul>
End bearing steel H-piles driven with conventional pile driving equipment.	<ul style="list-style-type: none"> <li>Feasible and practical for support of new piers</li> </ul>	<ul style="list-style-type: none"> <li>Minimal differential settlement between existing and rehabilitation portions of structures</li> <li>Pile capacities larger than those for existing structure</li> </ul>	<ul style="list-style-type: none"> <li>Vibration impacts on existing Highway 400 NBL and SBL structures; vibration monitoring will be required</li> </ul>	<ul style="list-style-type: none"> <li>Cost for end-bearing piles comparable to friction piles (in relation to design capacity)</li> </ul>
Micropiles bored to found within dense sand	<ul style="list-style-type: none"> <li>Feasible and practical for support of rehabilitated abutments (if required) and new piers</li> </ul>	<ul style="list-style-type: none"> <li>Small micropile rig suitable for difficult access or limited headroom</li> <li>Locations and directions of micropiles can be adjusted according to site conditions</li> <li>Minimal vibration impacts on existing Highway 400 NBL and SBL structures</li> <li>Minimal differential settlement between existing and rehabilitated portions of structure, and between foundation elements</li> <li>Lateral resistance of foundation can be increased by adding sufficient numbers of micropiles or adding shell thickness</li> </ul>	<ul style="list-style-type: none"> <li>Temporary liners may be required during installation due to existence of high groundwater level</li> <li>Permanent liners may be required for structural loading requirements and this would add to the cost of this foundation option</li> </ul>	<ul style="list-style-type: none"> <li>Cost dependent on configuration of foundation elements, but more expensive than driven piles</li> </ul>

## 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

### III. SOIL DESCRIPTION

#### (a) Cohesionless Soils

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

### 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.)

#### 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

#### 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.).

**PH:** Sampler advanced by hydraulic pressure

**PM:** Sampler advanced by manual pressure

**WH:** Sampler advanced by static weight of hammer

**WR:** 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.

### IV. SOIL TESTS

w	water content
$w_p$	plastic limit
$w_l$	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test <sup>1</sup>
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement <sup>1</sup>
$D_R$	relative density (specific gravity, $G_s$ )
DS	direct shear test
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
SO <sub>4</sub>	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V	field vane (LV-laboratory vane test)
$\gamma$	unit weight

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

## LIST OF SYMBOLS

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

### I. General

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

### II. STRESS AND STRAIN

$\gamma$	shear strain
$\Delta$	change in, e.g. in stress: $\Delta \sigma$
$\epsilon$	linear strain
$\epsilon_v$	volumetric strain
$\eta$	coefficient of viscosity
$\nu$	poisson's ratio
$\sigma$	total stress
$\sigma'$	effective stress ( $\sigma' = \sigma - u$ )
$\sigma'_{vo}$	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stress (major, intermediate, minor)
$\sigma_{oct}$	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
$\tau$	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

### III. SOIL PROPERTIES

#### (a) Index Properties

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

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

w	water content
$w_l$	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  $= (\text{compressive strength})/2$
  - \* density symbol is  $\rho$ . Unit weight symbol is  $\gamma$  where  $\gamma = \rho g$  (i.e. mass density x acceleration due to gravity)



PROJECT 06-1111-011			RECORD OF BOREHOLE No 07-1			1 OF 3 METRIC		
W.P. 2168-06-00			LOCATION N 4931075.4 ; E 286061.5			ORIGINATED BY PKS		
DIST Central HWY 400			BOREHOLE TYPE Power Auger, 200 mm Hollow Stem Augers and 'N' Casing			COMPILED BY VO		
DATUM Geodetic			DATE October 15-18, 2007			CHECKED BY LCC		
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100
250.0	GROUND SURFACE							PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT W <sub>p</sub> W W <sub>L</sub> WATER CONTENT (%)
0.0	Silty sand, trace topsoil, clay and gravel (FILL) Compact Brown Moist		1	SS	22			UNIT WEIGHT γ kN/m <sup>3</sup>
249.1								
0.9	SILTY CLAY, trace to some sand Stiff to very stiff Mottled brown and grey Moist		2	SS	10		249	49
			3	SS	13		248	49
			4	SS	16		247	
			5	SS	29		246	
245.7							245	
4.3	SAND, trace to some silt, trace clay Compact to very dense Brown Moist to wet		6	SS	15		244	
							243	
			7	SS	14		242	
							241	
			8	SS	17		240	
							239	
			9	SS	14		238	
							237	
			10	SS	25		236	
			11	SS	51			
			12	SS	50			

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

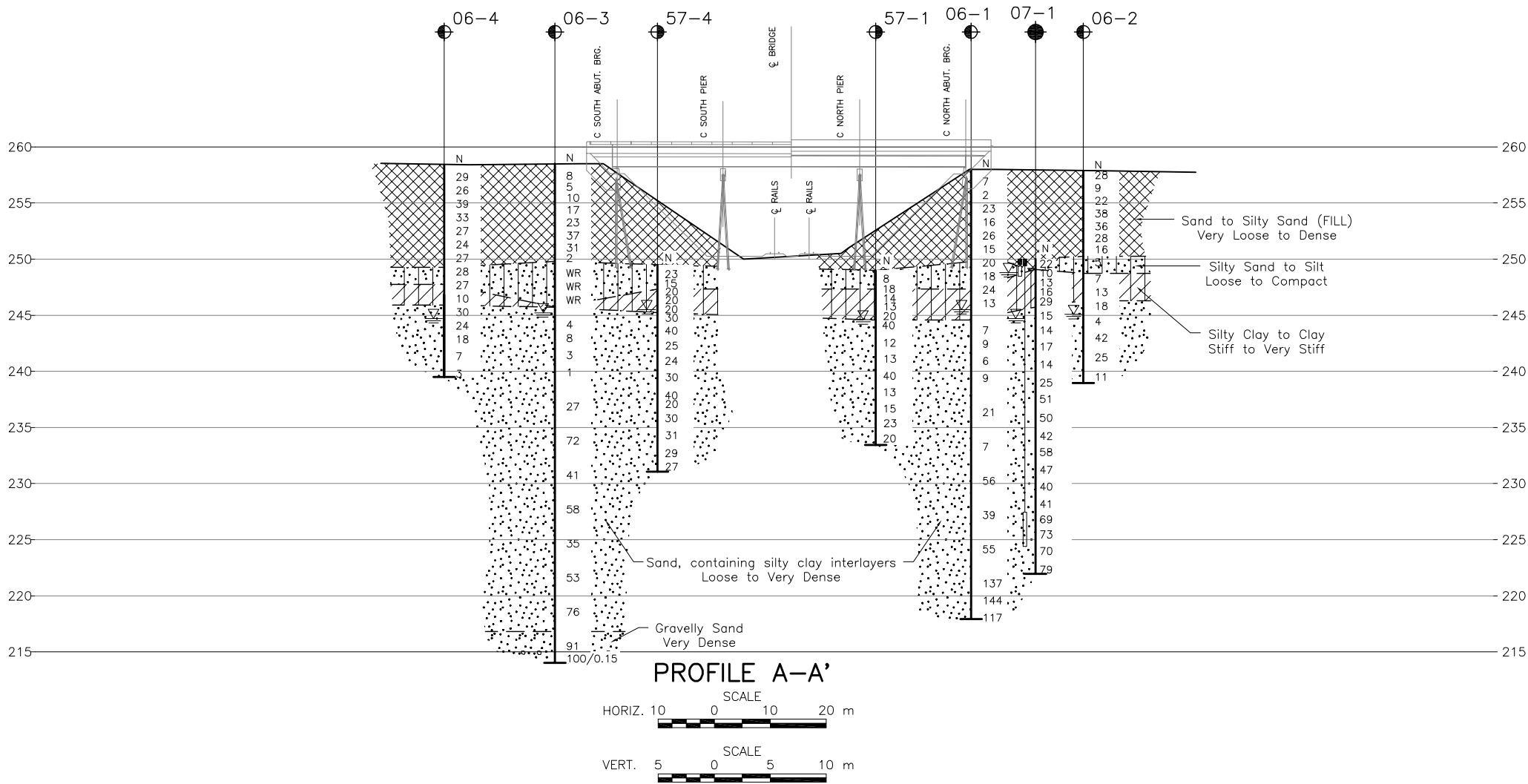
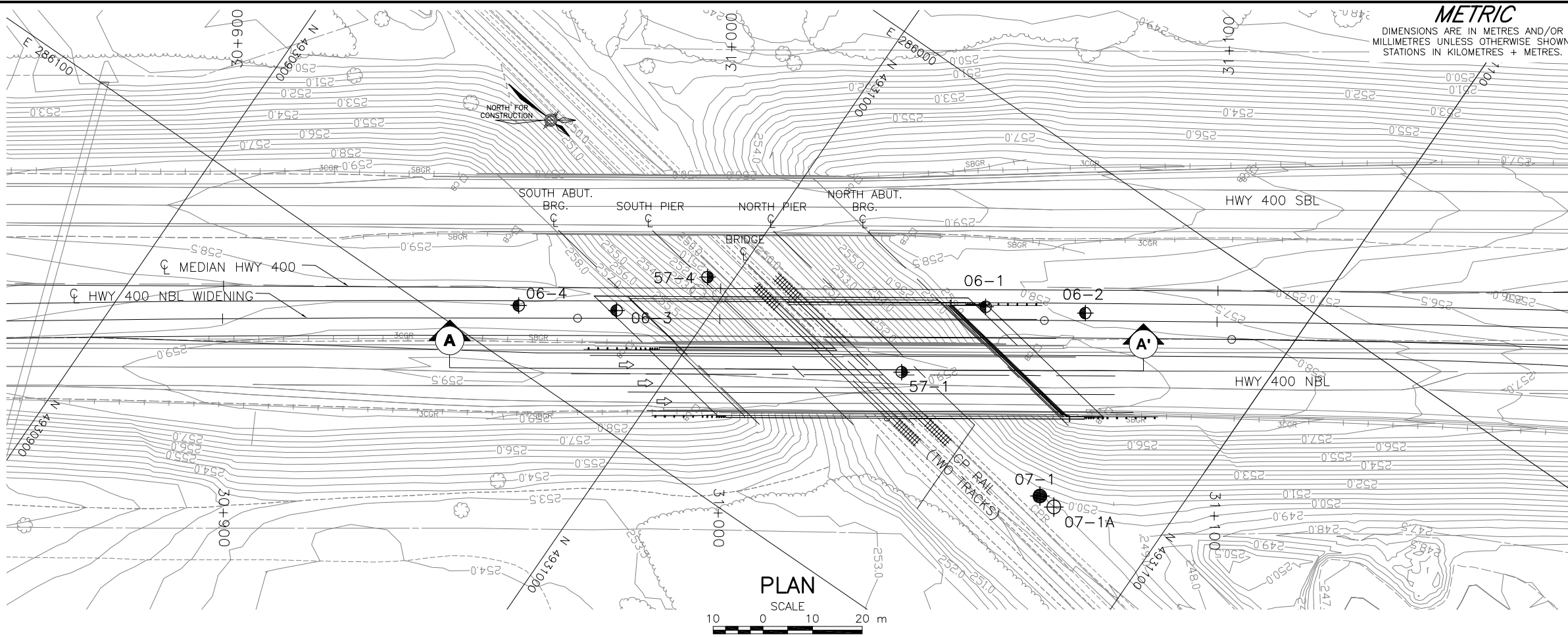
PROJECT 06-1111-011			RECORD OF BOREHOLE No 07-1			2 OF 3 METRIC						
W.P. 2168-06-00			LOCATION N 4931075.4 ; E 286061.5			ORIGINATED BY PKS						
DIST Central HWY 400			BOREHOLE TYPE Power Auger, 200 mm Hollow Stem Augers and 'N' Casing			COMPILED BY VO						
DATUM Geodetic			DATE October 15-18, 2007			CHECKED BY LCC						
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	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	ELEVATION SCALE					
	--- CONTINUED FROM PREVIOUS PAGE ---						20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					
233.3	SAND, trace to some silt, trace clay Compact to very dense Brown Moist to wet		13	SS	42							0 82 17 1
16.7	Stratified SAND, some silt, occasional silty clay lenses and random silty clay layer up to 50 mm thick Dense to very dense Brown Wet		14	SS	58							
			15	SS	47							
			16	SS	40							
			17	SS	41							
			18	SS	69							0 85 14 1
			19	SS	73							
			20	SS	70							
			21	SS	79							
222.0												
28.0												

Continued Next Page

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

MIS-MTO 001 06-1111-011.GPJ GAL-MISS.GDT 8/15/08 DD

PROJECT 06-1111-011		RECORD OF BOREHOLE No 07-1				3 OF 3 METRIC										
W.P. 2168-06-00		LOCATION N 4931075.4 ;E 286061.5				ORIGINATED BY PKS										
DIST Central HWY 400		BOREHOLE TYPE Power Auger, 200 mm Hollow Stem Augers and 'N' Casing				COMPILED BY VO										
DATUM Geodetic		DATE October 15-18, 2007				CHECKED BY LCC										
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	SHEAR STRENGTH kPa					W <sub>p</sub>	W		
--- CONTINUED FROM PREVIOUS PAGE ---							20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100					10 20 30 WATER CONTENT (%)				
	END OF BOREHOLE  NOTES:  1. Water level in shallow piezometer at 1.2 m depth on November 14, 2007.  2. Water level in deep piezometer at 5.2 m depth on October 18, 2007.  3. Water level in deep piezometer at 5.3 m depth on November 14, 2007.  4. Drilling technique switched to using 'N' casing below 9 m depth															



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

CONT No.  
WP No. 2168-06-00

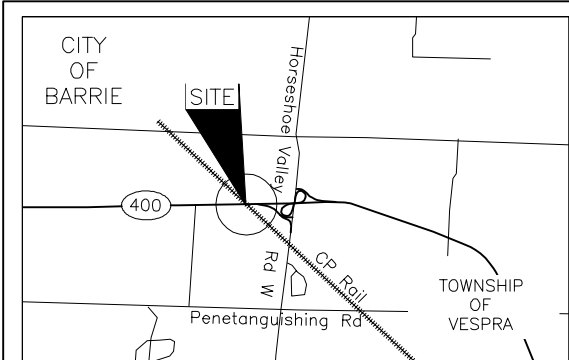


HWY 400 NBL BRIDGE REHABILITATION  
OVER CP RAIL  
BOREHOLE LOCATIONS AND  
SOIL STRATA

SHEET



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



**KEY PLAN**  
SCALE  
1 0 1 km

### LEGEND

- Borehole - Current Investigation
- Borehole - Previous Investigation (GOLDER, 2006)
- Borehole - Previous Investigation (MTO, 1957)
- ⊕ Dynamic Cone Penetration Test
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- ≡ WL upon completion of drilling

No.	ELEVATION	CO-ORDINATES	
		NORTHING	EASTING
07-1	250.0	4931044.9	286061.5
06-1	258.0	4931044.9	286036.4
06-2	257.9	4931062.1	286026.0
06-3	258.5	4930984.1	286079.0
06-4	258.4	4930967.3	286089.4
57-1	249.0	4930816	286041
57-4	249.4	4930773	286049

### NOTES

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

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

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

### REFERENCE

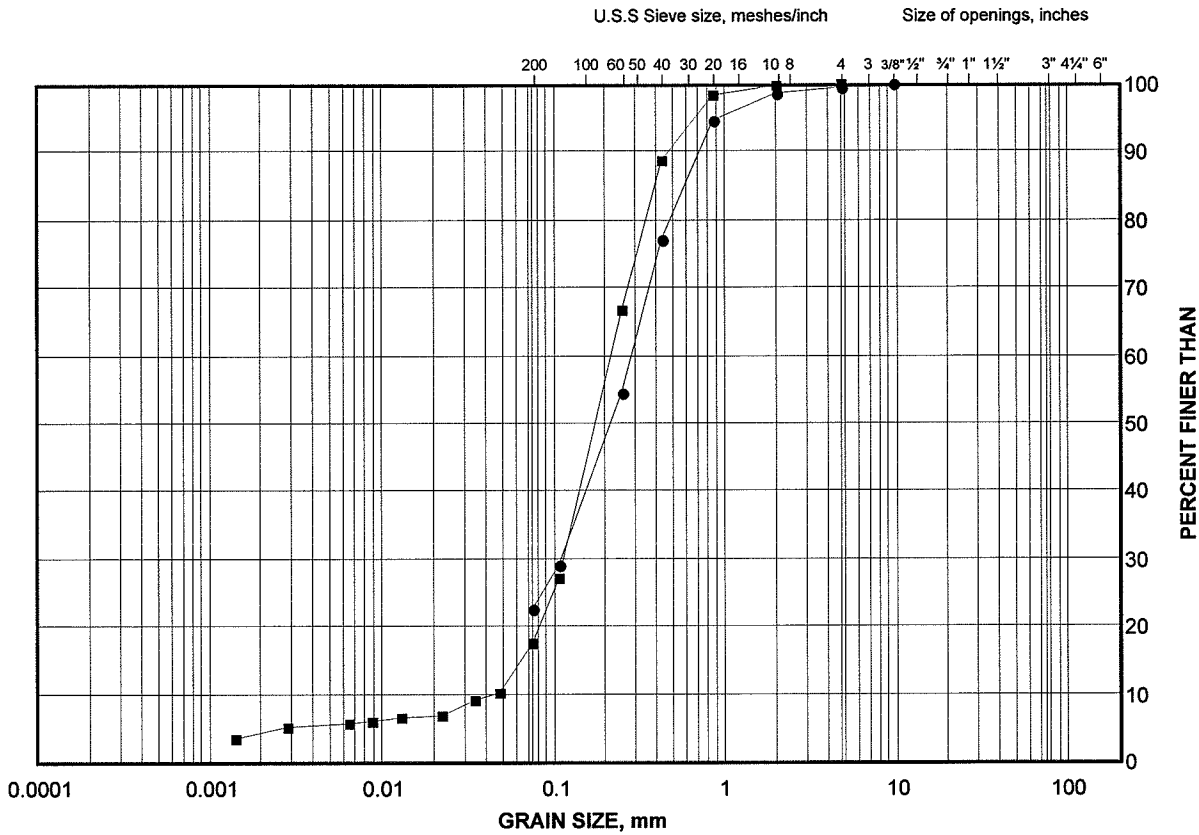
Base plans provided in digital format by Transenco Limited, (Drawing Files Hwy 400 Base Plan\_2.dwg and Hwy 400 Base Plan\_3.dwg, received March 24, 2006, and Drawing File Hwy 400 PD Plan - Detour Staging.dwg, received March 27, 2006) and by Ellis Engineering (Drawing File 351Ga CPR N.B.L.Revised.dwg, received April 18, 2006).

NO.	DATE	BY	REVISION
Geocres No.			
HWY. 400	PROJECT NO. 06-1111-011		DIST.
SUBM'D.VO	CHKD. FJH	DATE: Aug. 15, 08	SITE:
DRAWN: MSM	CHKD. VO	APPD. LCC	DWG. 1

# GRAIN SIZE DISTRIBUTION TEST RESULTS

Sand to Silty Sand Fill

FIGURE 1



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

## LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	06-3	3	255.9
■	06-1	6	253.1

Project Number: 06-1111-011

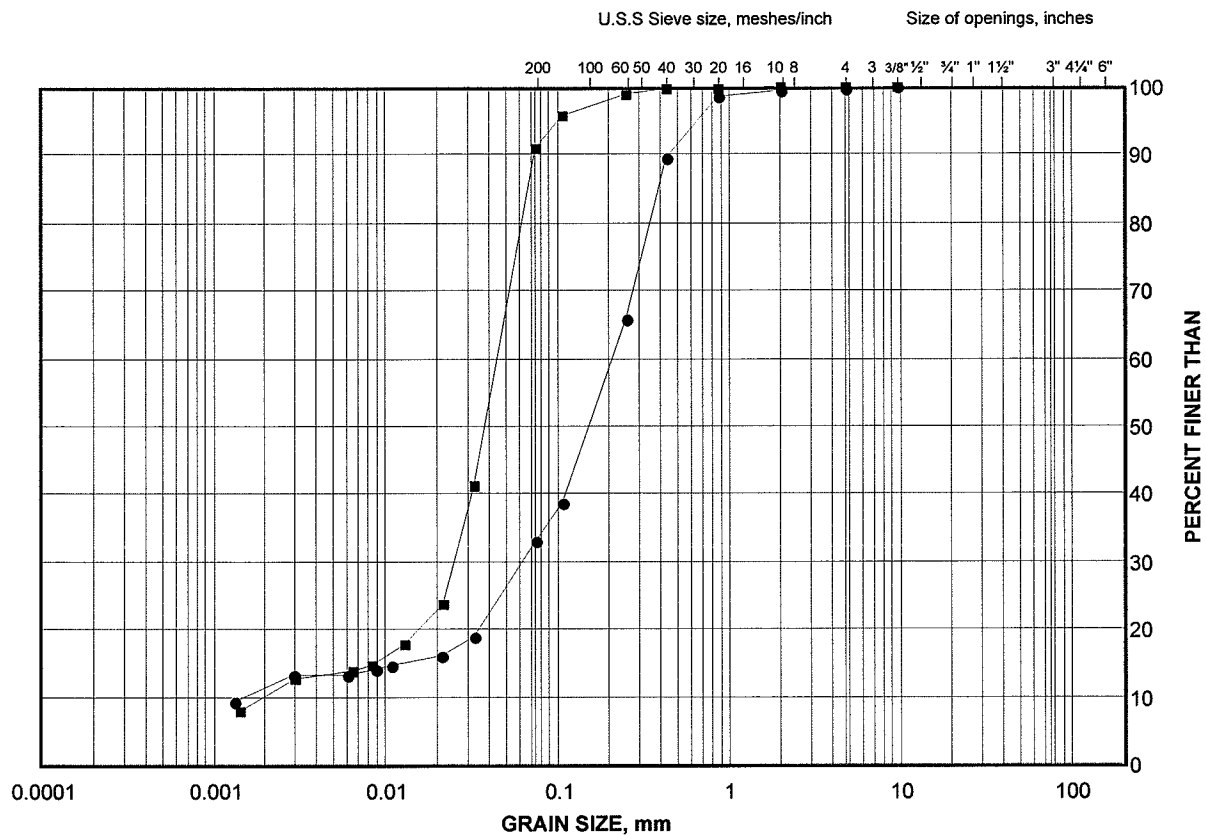
Checked By: *[Signature]*

Golder Associates

Date: 15-Feb-08

### Surficial Silty Sand to Silt

FIGURE 2



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

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	06-3	11	246.0
■	06-4	9	249.0

Checked By: Moyle

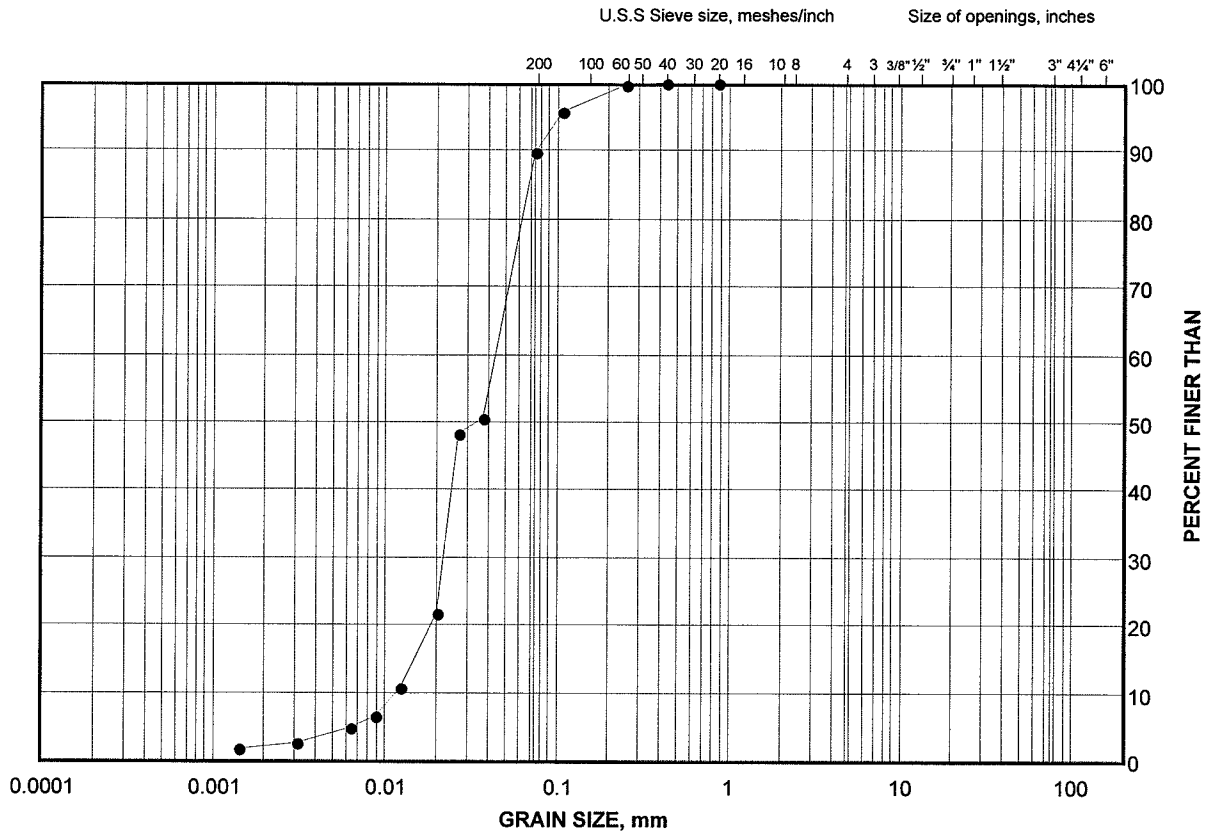
## Golder Associates

Date: 15-Feb-08

# GRAIN SIZE DISTRIBUTION TEST RESULT

Surficial Silty Clay

FIGURE 3



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

## LEGEND

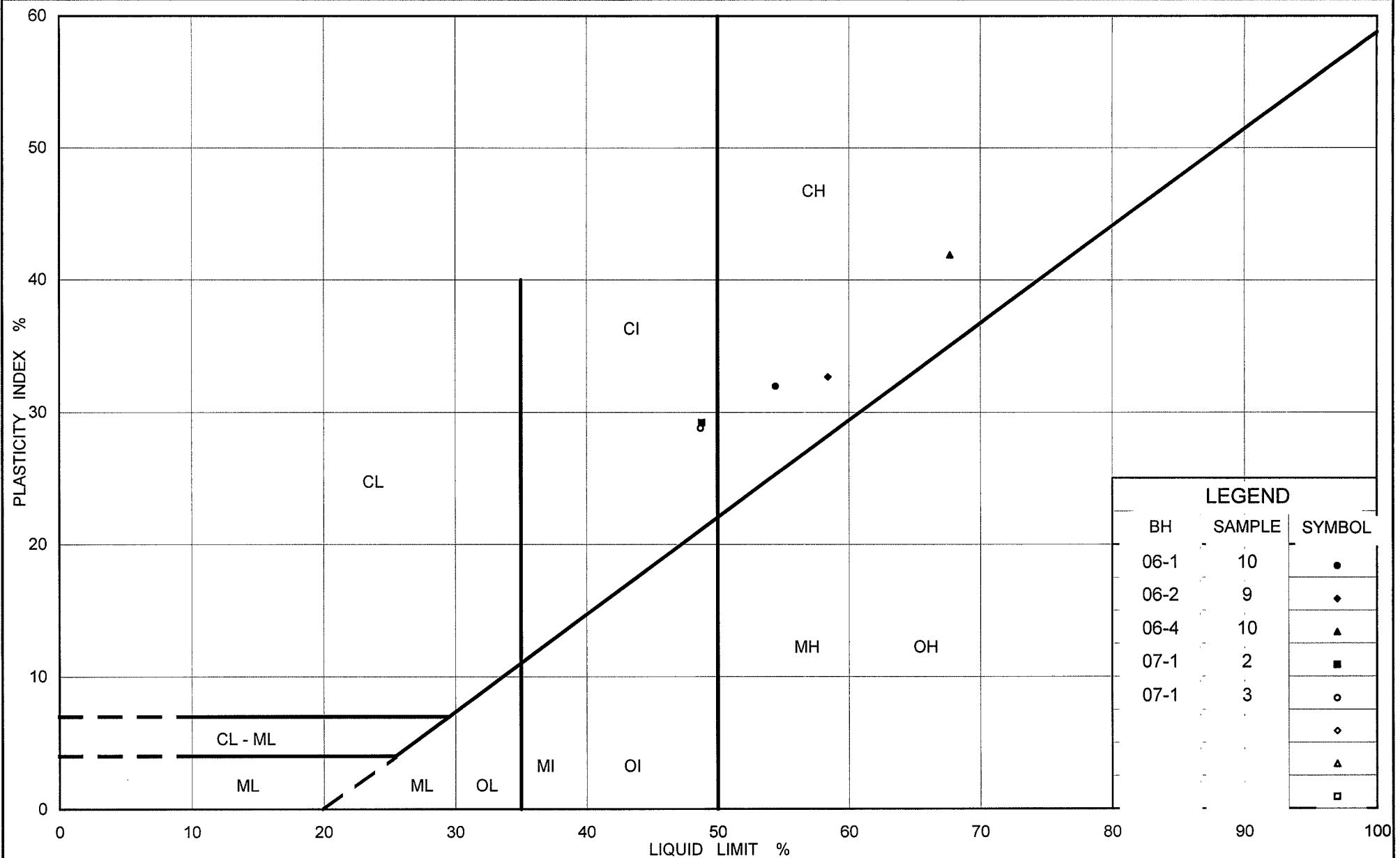
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
•	07-1	4	247.7

Project Number: 06-1111-011

Checked By: *[Signature]*

Golder Associates

Date: 15-Feb-08



Ministry of Transportation

Ontario

# PLASTICITY CHART Silty Clay to Clay

Figure 4

Project No. 06-1111-011

Checked By: *Woy*



**Sand**

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

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	07-1	11	237.8
■	07-1	13	234.8
◆	06-2	13	242.4
▲	06-4	14	241.3
▽	06-3	17	233.8
○	07-1	18	227.1
□	06-1	22	219.8
△	07-1	9	240.9

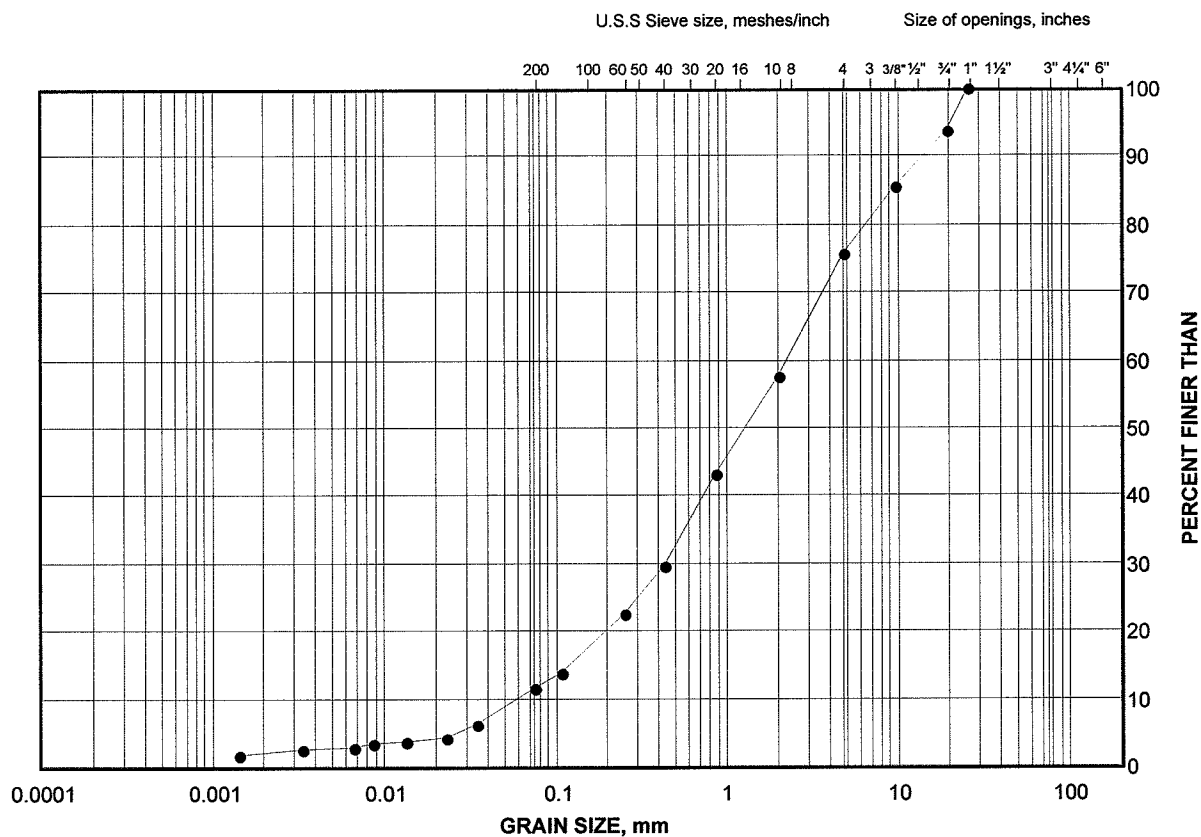
Checked By:

Date: 15-Feb-08

# GRAIN SIZE DISTRIBUTION TEST RESULT

Gravelly Sand

FIGURE 6



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

## LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
•	06-3	23	215.5

Project Number: 06-1111-011

Checked By: *[Signature]*

Golder Associates

Date: 15-Feb-08

## **APPENDIX A**

**RECORDS OF BOREHOLES  
FROM 1957 INVESTIGATION  
BY DEPARTMENT OF HIGHWAYS, ONTARIO**



Ministry of  
Transportation and  
Communications

HIGHWAY ENGINEERING DIVISION-ENGINEERING MATERIALS OFFICE-SOIL MECHANICS SECTION

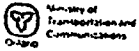
RECORD OF BOREHOLE No 57-1 (Formerly H.P. 572-56)

W P 99-75-18 LOCATION Co-ords N 16, 177, 217; E 938, 456 ORIGINATED BY U.G.L.  
DIST 5 HWY 400 BOREHOLE TYPE Washbore- BX Casing COMPILED BY PF  
DATUM Geodetic DATE June 16-26, 1957 CHECKED BY RS

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT			UNIT WEIGHT $\gamma$	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PICT	NUMBER	TYPE	VALUES		20	40	60	80	100	W <sub>p</sub>	W	W <sub>L</sub>		
816.9	Ground Level															
247.3	Silty Sand occasional clay layers loose		1	SS	8											
811.4			2	SS	18											
5.3	Silty Clay to Clay (Stratified)		3	SS	14											
244.6m	Traces of Sand and Gravel		4	SS	13											
802.4	Stiff to Very Stiff		5	SS	20											
14.5	Silty Sand to Sand Occasional Silty Clay Layers and Oxidized Zones		6	SS	40											
	Traces of Organics		7	SS	12											
	Compact to Dense		8	SS	13											
			9	SS	40											
			10	SS	13											
			11	SS	15											
			12	SS	23											
233.4m			13	SS	20											
765.9																
51.0	End of Borehole															
	NOTE: BR. #1A sampled to depth 13.5' and two TW samples were obtained															
	* U.G.L: Universal Geotechnique Limited															

\*3, \*5: Numbers refer to  
Sensitivity

20  
15-20 (%) STRAIN AT FAILURE  
10



## HIGHWAY ENGINEERING DIVISION-ENGINEERING MATERIALS OFFICE-SOIL MECHANICS SECTION

## RECORD OF BOREHOLE No 57-4 (Formerly W.P. 572-56)

W P 99-75-03 LOCATION Co-ords N 16,177,078; E 938,477 ORIGINATED BY U.G.L. #  
DIST 5 HWY 400 BOREHOLE TYPE Washbore- BX Casing COMPILED BY PP  
DATUM Geodetic DATE June 16-26, 1957 CHECKED BY RS

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				NATURAL MOISTURE CONTENT			UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			20	40	60	80	100	W <sub>p</sub>	W	W <sub>L</sub>	
818.3	Ground Level														
0.0	Silty Sand		1	SS	23										
247.3m	Some Gravel		2	SS	15										
811.3	Occ. Clay Layers		3	SS	20										
7.0	Compact		4	SS	20										
245.2m	Silty Clay to Clay (Stratified)		5	SS	20										
804.3	Traces of Sand and Gravel		6	SS	30										
14.0	Stiff to Very Stiff		7	SS	40										
	Silty Sand to Sand		8	SS	25										
	Occasional Silty Clay Layers		9	SS	26										
	Compact to Dense		10	SS	30										
			11	SS	40										
			12	SS	20										
			13	SS	30										
			14	SS	31										
231.4m			15	SS	29										
759.3			16	SS	27										
59.0	End of Borehole														
NOTE: B.H. 4A Sampled to depth 13.0' and two TW Samples were Obtained															
*U.G.L.: Universal Geotechnique Limited															

\*3, \*5: Numbers refer to Sensitivity  
20  
15-5 (%) STRAIN AT FAILURE  
10

## **APPENDIX B**

**RECORDS OF BOREHOLES  
FROM 2006 INVESTIGATION  
BY GOLDER ASSOCIATES LTD.**



PROJECT 06-1111-011		RECORD OF BOREHOLE No 06-1		1 OF 3 METRIC	
W.P. 167-99-00		LOCATION N 4931044.9 ; E 286036.4		ORIGINATED BY SB	
DIST Central HWY 400		BOREHOLE TYPE Continuous Flight Hollow Stem Augers and 'N' Casing/Tricone		COMPILED BY KB	
DATUM Geodetic		DATE March 21, 2006		CHECKED BY LCC	

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 (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)		
258.0	GROUND SURFACE																
0.0	Sand, trace silt (FILL) Very loose to compact Brown Moist		1	SS	7		257										
			2	SS	2		256										
255.7			3	SS	23		255										
2.3	Sand, trace to some silt, trace clay, trace gravel and organics (FILL) Compact Brown Moist		4	SS	16		254										
			5	SS	26		253										
			6	SS	15		252										
			7	SS	20		251										
			8	SS	18		250										
249.8			9	SS	24		249										
8.2	Silty Sand Compact Brown Moist to Wet		10	SS	13		247										
247.3			11	TO	PH		246										
10.7	Silty Clay to Clay, trace sand and gravel Stiff to very stiff Brown Moist		12	SS	7		244										
244.6																	
13.4	Sand, trace silt Loose to compact Brown Wet																


Continued Next Page

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

MIS-MTO 001 06-1111-011.GPJ GAL-MISS.GDT 2/25/08 DD






PROJECT 06-1111-011			RECORD OF BOREHOLE No 06-1				3 OF 3 METRIC										
W.P. 167-99-00		LOCATION N 4931044.9 ; E 286036.4				ORIGINATED BY SB											
DIST Central HWY 400		BOREHOLE TYPE Continuous Flight Hollow Stem Augers and 'N' Casing/Tricone				COMPILED BY KB											
DATUM Geodetic		DATE March 21, 2006				CHECKED BY LCC											
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									
— CONTINUED FROM PREVIOUS PAGE —								20	40	60	80	100					
	Sand, trace silt, containing silty clay interlayers Dense to very dense Brown Moist to wet		19	SS	39		227										
								226									
								225									
								224									
								223									
								222									
								221									
								220									
								219									
								218									
217.9 40.1	END OF BOREHOLE		23	SS	117												
Notes: 1. Water level in borehole at 12.5m depth below ground surface upon completion of drilling. 2. Drilling technique switched to using 'N' Casing below 24m depth.																	

MIS-MTO 001 06-1111-011.GPJ GAL-MISS.GDT 2/25/08 DD



PROJECT 06-1111-011				RECORD OF BOREHOLE No 06-2				2 OF 2 METRIC									
W.P. 167-99-00		LOCATION N 4931062.1 ; E 286026.0				ORIGINATED BY SB											
DIST Central HWY 400		BOREHOLE TYPE Continuous Flight Hollow Stem Augers				COMPILED BY KB											
DATUM Geodetic		DATE March 22, 2006				CHECKED BY LCC											
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa									WATER CONTENT (%)
	— CONTINUED FROM PREVIOUS PAGE —						○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED										
	Sand, trace silt Loose to dense Brown Moist to wet		13	SS	42												0 92 (8)
			14	SS	25												
239.0 18.9	END OF BOREHOLE		15	SS	11												
	Notes: 1. Water level inside hollow stem augers at 12.8m below ground surface upon completion of drilling.																

MIS-MTO 001 06-1111-011.GPJ GAL-MISS.GDT 2/25/08 DD

PROJECT 06-1111-011

**RECORD OF BOREHOLE No 06-3**

1 OF 4 **METRIC**

W.P. 167-99-00

LOCATION N 4930984.1 ; E 286079.0

ORIGINATED BY SB

DIST Central HWY 400

BOREHOLE TYPE Continuous Flight Hollow Stem Augers and 'N' Casing/Tricone

COMPILED BY KB

DATUM Geodetic

DATE March 23, 2006

CHECKED BY LCC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100		
258.5	GROUND SURFACE													
0.0	Sand to Silty Sand, trace to some clay, gravel and organics (FILL) Very loose to dense Brown Moist		1	SS	8		258							
			2	SS	5		257							
			3	SS	10		256							
			4	SS	17		255							
			5	SS	23		254							
			6	SS	37		253							
			7	SS	31		252							
			8	SS	2		251							
249.8							250							
8.7	Silty Sand to Silt, some sand, trace to some clay Very loose Brown Wet		9	SS	WR		249							
			10	SS	WR		248							
			11	SS	WR		247							
245.7							246							
12.8	Sand, trace silt Very loose to compact Brown Moist to wet		12	SS	4		245							
							244							
243.5														

Continued Next Page

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

MIS-MTO 001 06-1111-011.GPJ GAL-MISS.GDT 2/25/08 DD



PROJECT 06-1111-011			RECORD OF BOREHOLE No 06-3			2 OF 4 METRIC							
W.P. 167-99-00			LOCATION N 4930984.1; E 286079.0			ORIGINATED BY SB							
DIST Central HWY 400			BOREHOLE TYPE Continuous Flight Hollow Stem Augers and 'N' Casing/Tricone			COMPILED BY KB							
DATUM Geodetic			DATE March 23, 2006			CHECKED BY LCC							
SOIL PROFILE		STRAT PLOT	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 (%)
ELEV. DEPTH	DESCRIPTION		NUMBER	TYPE			"N" VALUES	20 40 60 80 100					
— CONTINUED FROM PREVIOUS PAGE —													
15.0	Sand, trace silt Very loose to compact Brown Moist to wet		13	SS	8	243							
						242							
						241							
						240							
						239							
						238							
						237							
						236							
						235							
						234							
235.7 22.9	Sand, trace to some silt, trace clay, containing silty clay interlayers Dense to very dense Brown Wet		16	SS	27	237							0 85 12 3
						236							
						235							
						234							
						233							
						232							
						231							
						230							
					229								
			17	SS	72	234							
						233							
						232							
						231							
						230							
			18	SS	41	231							
						230							
						229							

Continued Next Page

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

MIS-MTO 001 06-1111-011.GPJ GAL-MISS.GDT 2/25/08 DD



PROJECT 06-1111-011

## RECORD OF BOREHOLE No 06-3

3 OF 4 METRIC

W.P. 167-99-00

LOCATION N 4930984.1 ; E 286079.0

ORIGINATED BY SB

DIST Central HWY 400

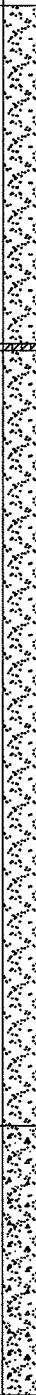
BOREHOLE TYPE Continuous Flight Hollow Stem Augers and 'N' Casing/Tricone

COMPILED BY KB

DATUM Geodetic

DATE March 23, 2006

CHECKED BY LCC

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 (%)						
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100						20	40	60	80	100	10
— CONTINUED FROM PREVIOUS PAGE —																							
	Sand, trace to some silt, trace clay, containing silty clay interlayers Dense to very dense Brown Wet		19	SS	58																		
225.0																							
33.6	Silty Clay Grey			20	SS	35																	
	Sand, trace silt, trace to some gravel Dense to very dense Brown Wet																						
216.8																							
41.7	Gravelly Sand, trace to some silt, trace clay Very dense Brown Wet																						

Continued Next Page

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

MIS-MTO 001 06-1111-011.GPJ GAL-MISS.GDT 2/25/08 DD

PROJECT <u>06-1111-011</u>										RECORD OF BOREHOLE No 06-3										4 OF 4 METRIC									
W.P. <u>167-99-00</u>					LOCATION <u>N 4930984.1 ; E 286079.0</u>					ORIGINATED BY <u>SB</u>																			
DIST <u>Central</u> HWY <u>400</u>					BOREHOLE TYPE <u>Continuous Flight Hollow Stem Augers and 'N' Casing/Tricone</u>					COMPILED BY <u>KB</u>																			
DATUM <u>Geodetic</u>					DATE <u>March 23, 2006</u>					CHECKED BY <u>LCC</u>																			
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	W <sub>p</sub>	W			W <sub>L</sub>												
— CONTINUED FROM PREVIOUS PAGE —																													
	END OF BOREHOLE  Notes: 1. Water level in borehole maintained at 9.5m depth below ground surface during drilling. 2. Drilling technique switched to using 'N' Casing and tricone below 18.3m depth. 3. Water level inside casing at 13.3m depth below ground surface upon completion of drilling on March 26, 2006.																												

MIS-MTO 001 06-1111-011.GPJ GAL-MISS.GDT 2/25/08 DD

PROJECT 06-1111-011

**RECORD OF BOREHOLE No 06-4**

1 OF 2 **METRIC**

W.P. 167-99-00

LOCATION N 4930967.3 ; E 286089.4

ORIGINATED BY SB

DIST Central HWY 400

BOREHOLE TYPE Continuous Flight Hollow Stem Augers

COMPILED BY KB

DATUM Geodetic

DATE March 29, 2006

CHECKED BY LCC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT   NATURAL MOISTURE CONTENT   LIQUID LIMIT			UNIT WEIGHT  γ  kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		w <sub>p</sub>	w	w <sub>L</sub>		
								○ UNCONFINED	+ FIELD VANE					
								● QUICK TRIAXIAL	× REMOULDED					
								WATER CONTENT (%)			GR   SA   SI   CL			
258.4 0.0	GROUND SURFACE Sand, trace to some silt and gravel, trace organics, containing cobbles in upper 1.5m (FILL) Compact to dense Brown Moist					▽	258							0   10   80   10
			1	SS	29		257							
			2	SS	26		256							
			3	SS	39		255							
			4	SS	33		254							
			5	SS	27		253							
			6	SS	24		252							
			7	SS	27		251							
		8	SS	28	250									
249.3 9.1	Silt, trace to some sand, trace to some clay Compact Brown Moist		9	SS	27		249							
							248							
247.7 10.7	Silty Clay to clay, trace sand and gravel Stiff to very stiff Brown Moist		10	SS	10		247							
						246								
245.9 12.5	Sand, trace silt Loose to compact Brown Wet		11	SS	30	245								
						244								
			12	SS	24									

Continued Next Page

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

MIS-MTO 001 06-1111-011.GPJ GAL-MISS.GDT 2/25/08 DD





PROJECT 06-1111-011		RECORD OF BOREHOLE No 06-4				2 OF 2		METRIC								
W.P. 167-99-00		LOCATION N 4930967.3 ; E 286089.4		ORIGINATED BY SB												
DIST Central HWY 400		BOREHOLE TYPE Continuous Flight Hollow Stem Augers		COMPILED BY KB												
DATUM Geodetic		DATE March 29, 2006		CHECKED BY LCC												
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								
	— CONTINUED FROM PREVIOUS PAGE —															
	Sand, trace silt Loose to compact Brown Wet		13	SS	18		243									
							242									
			14	SS	7		241									0 97 (3)
							240									
239.5 18.9	END OF BOREHOLE		15	SS	3											
	Notes: 1. Water level inside hollow stem augers at 13.7m depth below ground surface upon completion of drilling.															

MIS-MTO 001 06-1111-011.GPJ GAL-MISS.GDT 2/25/08 DD

## **APPENDIX C**

### **NON-STANDARD SPECIAL PROVISIONS**

## **VIBRATION MONITORING - Item No.**

---

### **Special Provision**

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#### ***Scope***

This special provision describes requirements for vibration monitoring during the piling installation works for the rehabilitation of the existing Highway 400 NBL-CP Rail overhead structure.

#### ***References***

The subsurface conditions at the site are described in the following Foundation Investigation Report for G.W.P 167-99-00:

- Foundation Investigation Report, Rehabilitation of Highway 400 NBL-CP Rail Overhead Structure, Highway 400 from Highway 11 to 93, Simcoe County, G.W.P. 167-99-00.

#### ***Definitions***

Quality Verification Engineer (QVE): An Engineer with a minimum of five (5) years experience in the field of installation of piling and vibration monitoring or, alternatively, with expertise demonstrated by providing satisfactory quality verification services for a minimum of two (2) projects of similar scope to the contract. The QVE shall be retained by the Contractor to ensure general conformance with the contract documents and issue certificates of conformance.

#### ***Submission Requirements***

The Contractor/QVE shall submit details of the vibration monitoring plan to the Contract Administrator for review. The submittals shall satisfy the specifications and at a minimum contain the following specific information:

- Qualifications of vibration monitoring specialist.
- Details regarding proposed instrumentation.
- Proposed location of instruments on the existing Highway 400 NBL structures.
- Proposed frequency of readings.
- Proposed methods for adjusting piling methods if readings show vibrations exceeding tolerable levels.

#### ***Monitoring During Pile Driving***

The vibration monitoring equipment shall be placed on the existing Highway 400 NBL structures, as close as possible to the piling works. The Contractor/QVE shall take readings on the existing structures during driving of each pile, starting with the pile furthest away from the existing Highway 400 NBL structure(s) for each foundation element.

The vibrations measured on the existing structure shall not exceed 50 mm/s (peak particle velocity).

The results shall be submitted to the Contract Administrator after each pile has been driven, prior to continuing with the subsequent piles. As a minimum, the pile number, location, set criteria and driving log must be submitted with vibration monitoring results.

If the vibration monitoring results are acceptable, the Contractor may continue with the next pile(s) with readings taken during driving of each pile. The results of subsequent piles should be submitted to the Contract Administrator after each pile has been driven.

If the readings are not within the limits stated above, the Contractor must alter the driving procedures until the vibrations at the existing structures are within acceptable levels. The above process must be repeated for each pile.

#### **Basis of Payment**

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

END OF SECTION