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**REPORT ON**

**FOUNDATION INVESTIGATION AND DESIGN  
DETAIL DESIGN  
HIGHWAY 400 NBL BRIDGE WIDENING  
OVER CP RAIL  
HIGHWAY 400 FROM HIGHWAY 11 TO 93  
SIMCOE COUNTY  
G.W.P. 167-99-00**

Submitted to:

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GEOCREs No.

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## **PART A**

**FOUNDATION INVESTIGATION REPORT  
HIGHWAY 400 NBL BRIDGE WIDENING OVER CP RAIL  
HIGHWAY 400 FROM HIGHWAY 11 TO 93  
G.W.P. 167-99-00**

**Golder Associates**

## 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 widening 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 foundation investigation carried out for the widening of the existing Highway 400 NBL structure over the CP Rail lines. The investigation was supplemented with the 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.

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 April 7, 2006, entitled "Proposal for Foundations Engineering Services, Highway 400 NBL Widening, Bridge Over CP Rail".

## 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 structure was constructed in 1959, and was widened to the east in 1979; the existing Highway 400 SBL structure was also constructed in 1979. Both the Highway 400 NBL and SBL structures are three spans, and are 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 CP Rail tracks have been constructed at or near the original ground surface, at about Elevation 250 m under the Highway 400 structures. 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 north and south approach embankment widening, respectively.

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 Highway 400 NBL bridge widening was carried out between March 21 and 29, 2006, at which time four boreholes (Boreholes 06-1 to 06-4) were advanced at the site using a track-mounted drill rig. The locations of the boreholes advanced as part of the current investigation, as well as two boreholes advanced during the previous (1957) investigation at this site, are shown on Drawing 1. Boreholes 06-1 and 06-3, which were advanced within the widening limits for the north and south abutments, were advanced to 40.1 m and 44.5 m, respectively, in order to extend into material having Standard Penetration Test (SPT) "N" values greater than 100 blows per 0.3 m of penetration. Boreholes 06-2 and 06-4, which were advanced within the widening areas for the north and south approach embankments, were drilled to a depth of 18.9 m in order to extend through the existing embankment fill, surficial silty sand to silt and clay, and into the extensive sand deposit at the site.

The field investigation was carried out using a track-mounted CME-55 drill rig supplied and operated by Marathon Drilling Co. Ltd. of Ottawa, Ontario. The boreholes were advanced using 108 mm inner diameter hollow stem augers to between about 18 m and 24 m depth; below this depth, due to the wet sandy conditions, the boreholes were advanced using 75 mm outer diameter "N" casing in conjunction with a tricone. A "head" of water was maintained within the auger column or 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 about 5 m depth, at 1.5 m intervals between approximately 5 m and 20 m depth, and at 3 m intervals below approximately 20 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). In situ vane shear strength testing was carried out where cohesive soils were encountered. The water level in the open boreholes was observed and recorded throughout the drilling operations.

The field work was supervised on a full-time basis by a member of Golder's staff who located the boreholes 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 classification testing (natural water contents, Atterberg limits, and grain size distributions) on selected samples. All of the laboratory tests were carried out to MTO and/or ASTM Standards as appropriate.

The as-drilled borehole locations and ground surface elevations were surveyed by Transenco. The following table summarizes the borehole locations (northing and easting coordinates referenced to the NAD83 MTM coordinate system) and ground surface elevations (referenced to the geodetic datum).

<i><b>Borehole Number</b></i>	<i><b>Borehole Location</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	North Abutment	4,931,044.9	286,036.4	258.0
06-2	North Approach	4,931,062.1	286,026.0	257.9
06-3	South Abutment	4,930,984.1	286,079.0	258.5
06-4	South Approach	4,930,967.3	286,089.4	258.4



## 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 results of the in situ and laboratory testing are given on the Record of Borehole sheets and on Figures 1 to 5 following the text of this report. The Record of Borehole sheets from the 1957 investigation are included in Appendix A.

The stratigraphic boundaries shown on the Record of Borehole sheets 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 borehole locations.

The interpreted soil 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 (comprising the approach embankments for the existing Highway 400 structures), overlying a layer of loose to compact, surficial silty sand to silt and a layer of generally stiff to very stiff clay. These surficial soils are underlain by an extensive deposit of sand, which grades to a gravelly sand at depth.

A more detailed description of the subsurface conditions encountered in the boreholes 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 Topsoil**

A 100 mm thick layer of topsoil was encountered in Borehole 06-2, which is located within the limits of the north approach embankment.

Approximately 300 mm of topsoil was also encountered at the original ground surface in Boreholes 57-1 and 57-4, which were advanced as part of the 1957 investigation. Since 1957, embankment fill has been placed at these locations as part of the construction of the Highway 400 NBL and SBL approach embankments.

#### **4.2.2 Fill**

Between 7.6 m and 9.1 m of 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 median area. 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 state of packing; 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.

The water content measured on selected samples of the sand fill ranged from 7 to 18 per cent.

#### **4.2.3 Surficial Silty Sand to Silt**

A 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 silty sand to silt are shown on Figure 2.

The measured SPT "N" values within the surficial silty sand to silt generally range from 5 to 27 blows per 0.3 m of penetration, indicating 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 some 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 drilling to attempt to prevent such heave.

The water contents measured on selected samples of the surficial silty sand to silt range from 14 to 22 per cent.

#### 4.2.4 Silty Clay to Clay

A 1.8 m to 2.7 m thick layer of silty clay to clay, containing trace sand and gravel, 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 1957 boreholes between Elevations 247.3 m and 248.8 m.

Atterberg limits testing was carried out on three selected samples of the silty clay to clay obtained from the current investigation, and on three samples obtained from the 1957 investigation. All of the test results are summarized in the following table, and the results from the current investigation are plotted on a plasticity chart on Figure 3. 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>
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. In addition, 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>.

#### 4.2.5 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 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 current ground surface, while Boreholes 57-1 and 57-4 were terminated within this deposit at depths of 15.5 m and 18.0 m below the original ground surface at the time of the 1957 investigation.

The sand deposit contains trace to some silt and, in some locations, trace to some gravel; interlayers of silty clay were observed within samples recovered from below about Elevation 232.1 m in Borehole 06-1, and below about Elevation 235.7 m in Borehole 06-3. Trace quantities of organics were noted within the upper zone of the sand in Borehole 57-1. The results of grain size distribution testing conducted on four selected samples of the sand deposit are shown on Figure 4.

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 the 1957 boreholes are higher, ranging from 12 to 40 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), the sand is typically compact above approximately Elevation 232 m, and dense to very dense below this level. At the south abutment and south pier (Boreholes 06-3, 06-4 and 57-4), the sand is typically compact above approximately Elevation 236 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).

#### **4.2.6 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 (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 5. 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

Details of the water levels observed in the open boreholes at the time of drilling are summarized on the records for Boreholes 06-1 to 06-4 following the text of this report, and on the records of boreholes advanced during the 1957 investigation in Appendix A. 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 hole dropped approximately 3 m to approximately Elevation 245.5 m.

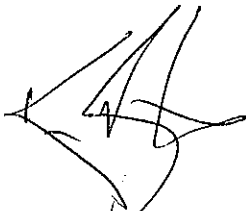
As shown in the above table, the water level associated with the extensive sand deposit at the site varies from about Elevation 244.7 m to 245.5 m, typically about 4 m to 5 m below the original ground surface at the site. The surficial silty sand to silt deposit is also water-bearing, with groundwater "perched" on top of the underlying, less permeable silty clay to clay deposit.

The water level(s) 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.

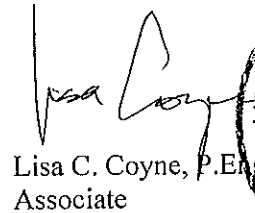
## 5.0 CLOSURE

The borehole investigation program documented in this report was supervised by Mr. Suresh Bainey, a senior geotechnical technician with Golder. This report was prepared by Mr. Peter Maki, EIT, and Mr. Kevin Bentley, P.Eng., a geotechnical engineer, 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.

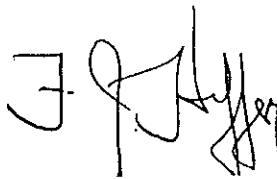
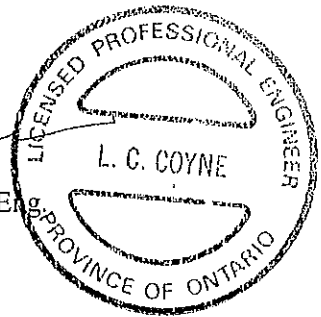
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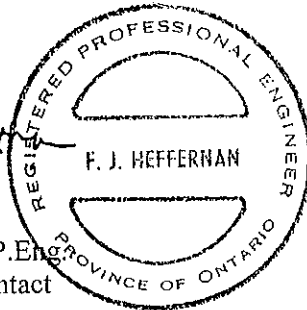
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## **PART B**

**FOUNDATION DESIGN REPORT  
HIGHWAY 400 NBL BRIDGE WIDENING OVER CP RAIL  
HIGHWAY 400 FROM HIGHWAY 11 TO 93  
G.W.P. 167-99-00**

## **6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS**

This section of the report provides foundation recommendations for the design of the proposed median widening of the existing Highway 400 NBL 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 widening. Where comments are made on construction, they are provided in order to highlight those aspects which 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.

The reader is referred to the "Important Information and Limitations of This Report" which follows the text of this report, but forms an integral part of this document.

### **6.1 General**

The proposed Highway 400 NBL bridge widening will be a three-span structure, constructed to match the existing NBL structure configuration, which has 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 structure is to be widened by approximately 10 m on the west side (i.e. within the median separating the existing Highway 400 northbound and southbound lanes). The finished grade of Highway 400 at the site will be maintained at approximately Elevation 258.5 m at the north abutment, and 259.5 m at the south abutment location. This will require placement of up to about 1.5 m of additional fill in the median area within the approach embankment limits.

The existing Highway 400 NBL structure was originally constructed in 1959, and was widened to the east in 1979. Both the original Highway 400 NBL structure and its eastward widening are three spans, and are supported on driven steel H-pile foundations; according to the available information, the foundations for both the original structure and for the eastward widening were designed the same way. The following information regarding the foundations for the existing Highway 400 NBL structure is based on available drawings (specifically 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):



- The existing north and south abutments for the Highway 400 NBL structure are supported on steel HP 12 x 53 (HP 310 x 79) piles with a design tip elevation of Elevation 800 feet (approximately 243.8 m). Some of the piles are vertical, and some are battered at 1H:3V. The design drawing notes indicate that the abutment piles 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 350 kN).
- The existing north and south piers for the Highway 400 NBL structure are supported on steel HP 12 x 89 (HP 310 x 132) piles with a design tip elevation of Elevation 780 feet (approximately 237.7 m). These piles are shown as battered at 1H:8V. The design drawing notes indicate that pier piles 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).

## 6.2 Foundation Options for Structure Widening

Based on the site conditions and the existing structure founding conditions, the following foundation options have been considered for support of the Highway 400 NBL structure widening:

- **Spread footings, either perched within the approach embankments for the abutments, or founded in the foreslope fill or native soils at the piers:** This option is considered undesirable for the following reasons: (1) the thickness and variability (i.e. very loose to loose portions) of the fill at the abutment locations; (2) the potential for differential settlement between the widening and the existing pile-supported structure; and (3) the requirement for temporary shoring/sheeting of the embankment foreslope and adjacent to the rail tracks, possibly in conjunction with dewatering of the surficial silty sand to silt deposit, for construction of the footings.
- **Driven steel H-pile foundations:** This foundation option is considered the most practical and appropriate option for support of the widened abutments and piers, since there will be minimal settlement of the new foundation elements relative to the existing structure, which is also pile-supported. Also, if the pier pile caps are configured to match the existing structure (i.e. pile cap perched above the ground surface immediately below the structure deck), there will be no requirement for pier pile cap excavation; this will minimize impacts on the relatively heavily travelled rail lines, and avoid requirements for temporary excavation support (to protect both the rail tracks and the abutment foreslopes) and for dewatering. Consideration has been given to both end-bearing piles founded at depth within the "100-blow" material, and to friction piles driven to a more moderate depth. Friction piles may have a slight advantage over end-bearing piles since the friction piles are expected to induce less vibration (which could affect the NBL and SBL structures) during driving, and since friction piles would be most consistent with the existing structure foundations (i.e. friction piles). However, the costs for both friction and end-bearing pile options are similar, and the choice can be made on the basis of structural requirements.

- **Caisson foundations:** Given the difficulties associated with drilling the caissons to depths of greater than 30 m in the saturated sands, liner installation and removal, and the potential for basal heave/disturbance, this option is less practical/feasible and more expensive than driven steel H-pile foundations.

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. Recommendations for the two most feasible and practical options – friction piles and deep end-bearing piles – are provided in the following sections.

### 6.3 Steel H-Pile Foundations

The abutments and piers may be supported on steel H-piles driven to a moderate depth, into the dense sand deposit (i.e. friction piles). 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, boulders and debris within the fills, and cobbles and boulders within the extensive sand and lower gravelly sand deposits at this site. The piles should be stiffened with MTO flange plates for protection during driving, in accordance with MTO’s Special Provision 903S01. Vibration monitoring should be carried out during pile installation to ensure that the vibration levels at the existing Highway 400 NBL and SBL structures are maintained within tolerable ranges. The pile driving criteria may have to be adjusted depending on the results of the vibration monitoring.

#### 6.3.1 Friction Steel H-Piles – Axial Geotechnical Resistance

At the north abutment and north pier location (Boreholes 06-1, 06-2, and 57-1), the sand was typically compact above Elevation 232 m, and dense to very dense below this level. At the south abutment and south pier location (Boreholes 06-3, 06-4, and 57-4), the sand was typically compact above Elevation 236 m, and dense to very dense below this level. The following table provides pile tip levels for design of friction steel H-piles (assuming steel HP 310 x 110 piles). Provision should be made in the Contract Documents to deal with varying pile lengths.

<i>Foundation Element</i>	<i>Relevant Boreholes</i>	<i>Minimum Embedment Length</i>	<i>Estimated Pile Tip Elevation</i>
North Abutment	06-1, 06-2	20 m	231 m
North Pier	06-1, 57-1	15 m	231 m
South Pier	06-3, 57-4	15 m	234 m
South Abutment	06-3, 06-4	20 m	234 m

For design, the factored axial resistance at Ultimate Limit States (ULS) for steel HP 310 x 110 piles driven into the dense sand to the design pile tip elevations given above may be taken as 850 kN. The axial resistance at Serviceability Limit States (SLS) for 25 mm of settlement may be taken as 700 kN.

Pile installation should be in accordance with MTO's Special Provision 903S01. 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 (SS 103-11). For friction piles driven into the dense sand, the following note is considered appropriate for the design and site conditions assuming 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."*

### 6.3.2 End Bearing Steel H-Piles – Axial Geotechnical Resistance

At the north abutment/pier, the surface of the "100-blow" soil was encountered at about Elevation 222 m in Borehole 06-1. At the south abutment/pier, the surface of the "100-blow" soil was encountered at about Elevation 216 m in Borehole 06-3. For design of end-bearing piles, the following pile tip levels may be assumed, based on approximately a minimum embedment of 1.5 m within the "100-blow" soil. There should be provision made in the contract for dealing with varying pile lengths.

<i>Foundation Element</i>	<i>Relevant Boreholes</i>	<i>Estimated Elevation of 100-Blow Soil</i>	<i>Estimated Pile Tip Elevation</i>	<i>Estimated Pile Length *</i>
North Abutment	06-1, 06-2	222 m	220.5 m	35 m
North Pier	06-1, 57-1	222 m	220.5 m	36 m
South Pier	06-3, 57-4	216 m	214.5 m	42 m
South Abutment	06-3, 06-4	216 m	214.5 m	42 m

\* Assuming the pile cap underside is at Elevation 255.4 m at the north abutment, Elevations 256.4 m and 256.6 m (perched above the ground surface, below the bridge deck) at the north and south piers, respectively, and Elevation 255.9 m at the south abutment, consistent with the existing structure.

For design, the factored axial resistance at ULS for steel HP 310 x 110 piles founded in the very dense sand to gravelly sand (using the pile tip elevations given in the table above) may be taken as 1,750 kN. The axial resistance at SLS for 25 mm of settlement may be taken as 1,400 kN per pile.

Pile installation should be in accordance with MTO's Special Provision 903S01. 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 (SS 103-11). For end-bearing piles driven into the "100-blow" sand and/or gravelly sand, the following note is considered appropriate for the design and site conditions assuming 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 3,500 kN per pile."*

As discussed in Section 4.3 (Groundwater Conditions), the water level associated with the extensive sand deposit at the site varies from about Elevation 244.7 m to 245.5 m, as measured during the 1957 and 2006 investigations. This is typically about 4 m to 5 m below the original ground surface at the site and about 5 m to 6 m below the existing ground surface at the pier locations. However, if end-bearing piles are adopted, and if the piles for the pier widenings need to be driven deeper than the design tip elevations given above, there is a possibility that artesian groundwater conditions could be encountered. In this case, water flow along the piles could carry fine soil particles to the ground surface. It is recommended that a 0.5 m thick filter blanket of Granular "A" fill be placed at ground surface around the pier pile groups. The Granular "A" blanket should extend a minimum distance of 1 m outside of the pier pile group on all sides, and should be graded to provide drainage to nearby ditches.

### 6.3.3 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:

$$k_h = \frac{n_h z}{B} \quad \text{where } n_h \text{ is the constant of horizontal subgrade reaction (MPa/m);}$$

$z$  is the depth (m); and

$B$  is the pile diameter (m).

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.

Soil Unit	$\tau_u$ (kPa)	$n_h$ (MPa/m)
North Abutment and North Pier:		
Fill above Elev. 250 m	—	5 to 10
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
South Abutment and South Pier:		
Fill above Elev. 250 m	—	5 to 10
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:

Pile Spacing in direction of Loading ( $d$ = Pile Diameter)	Subgrade Reaction Reduction Factor, $R$
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.

### 6.3.4 Frost Protection

The abutment pile caps should be provided with a minimum of 1.6 m of soil cover (see OPSD 3090.101) for frost protection. It is assumed that the pier pile caps will be perched above the ground surface immediately below the structure deck, as for the existing structures, and so frost protection will not be applicable to the pier pile caps.

## 6.4 Lateral Earth Pressures for Design

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 the walls. It should be noted that 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:

	Granular "A"	Granular "B" Type II
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.
- If this structure is considered a "lifeline" structure, then 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,  $K_{AE}$** 

	<b>Case I</b>	<b>Case II</b>
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).

## **6.5 Approach Embankment Design and Construction**

Based on the Highway 400 cross-sections and the information provided on the General Arrangement drawing for the widening, up to about 1.5 m of fill will be placed in the median area on top of the existing Highway 400 NBL embankment side slope, as part of the new approaches for the bridge widening.

### **6.5.1 Subgrade Preparation and Embankment Construction**

In order to minimize differential settlement between the proposed widened portions and the existing Highway 400 approach embankments, it is recommended that all topsoil and surficial softened / loosened soils be stripped from below the widened approach embankment areas. The use of granular fill is recommended over the use of cohesive fill, since the majority of settlement of granular fills will occur during construction whereas some settlement of cohesive fills, if used, would occur post-construction. The new embankment fills should be benched into the existing



embankment in accordance with OPSD 208.010. The fill for the new or widened embankments should be placed and compacted in accordance with MTO's Special Provision 105S10, with inspection and field density testing by qualified personnel during placement operations to ensure appropriate materials are used and that adequate levels of compaction are achieved.

To reduce surface water erosion on the embankment side slopes, placement of topsoil and seeding or pegged sod is recommended.

### 6.5.2 Approach Embankment Stability

With appropriate subgrade preparation and proper placement and compaction of embankment fill materials, the raised abutment foreslopes (which will be up to approximately 9 m in height) will have a factor of safety of greater than 1.3 against deep-seated slope instability, provided that the foreslopes are maintained at 2 horizontal to 1 vertical.

The static slope stability analyses for this embankment configuration were carried out based on the following parameters, derived from field and laboratory testing and accepted correlations, using the commercially available program SLOPE/W produced by Geo-Slope International Ltd., employing the Morgenstern-Price method of analysis.

<i>Soil Type</i>	<i>Bulk Unit Weight</i>	<i>Effective Angle of Friction</i>	<i>Undrained Shear Strength</i>
Embankment fill (range of parameters assumed for existing fill and proposed granular fill)	20 – 22 kN/m <sup>3</sup>	32° to 35°	–
Surficial silty sand to silt	20 kN/m <sup>3</sup>	28°	–
Surficial clay	19 kN/m <sup>3</sup>	28°	100 kPa
Sand / Gravelly Sand	20 kN/m <sup>3</sup>	30° to 35°	–

NOTE: Both undrained and effective stress analyses were carried out for the surficial clay layer.

### 6.5.3 Approach Embankment Settlement

Settlement of the widened/raised approach embankments will occur as a result of short-term compression of the existing cohesionless embankment fill and the native cohesionless soils; some post-construction settlement of the stiff to very stiff silty clay to clay stratum will also occur. The settlement of the widening areas under the grade raise will be differential with respect to the existing embankment. As noted in Section 6.5.1 above, the new embankment fill should be keyed into the existing embankment side slopes in accordance with OPSD 208.010, and this will help to reduce the impact of differential settlement. In addition, the differential settlement between the existing and widened portions of the Highway 400 embankment can be minimized by the use of granular fill for the widenings.

Settlement analyses for the foundation soils were carried out using hand calculations and the commercially available computer program Unisettle. The compression of the existing fill and native soils has been estimated using the elastic deformation moduli given in the table below, based on correlations with the SPT "N" values.

<i>Soil Type</i>	<i>Bulk Unit Weight</i>	<i>Elastic Modulus</i>
Existing embankment fill	21 kN/m <sup>3</sup>	20 MPa
Loose to compact surficial silty sand to silt	20 kN/m <sup>3</sup>	10 MPa
Stiff to very stiff surficial silty clay to clay	19 kN/m <sup>3</sup>	—
Compact sand	20 kN/m <sup>3</sup>	30 MPa
Dense to very dense sand	21 kN/m <sup>3</sup>	>50 MPa

Based on the above, and provided that proper subgrade preparation is carried out, the settlement under the maximum 1.5 m high widening/grade raise will be less than 15 mm. The majority of this is associated with elastic compression of the existing embankment fill and the underlying surficial silty sand to silt. However, it is estimated that approximately 5 mm of the predicted settlement will occur in the stiff to very stiff silty clay to clay stratum, within approximately three to six months following completion of construction.

## 6.6 Construction Considerations

### 6.6.1 Excavation and Groundwater Control

Excavations for the abutment pile caps will extend through the existing embankment fill; these excavations will be maintained above the water level at the site, assuming that the underside of the abutment pile caps is maintained at approximately Elevation 256 m. If space permits at the abutment widening pile cap locations, open-cut excavations into these materials should be carried out in accordance with the guidelines outlined in the Occupational Health and Safety Act (OHSA) for Construction Activities. The existing fill is classified as Type 3 soil, according to the OHSA. Temporary excavations (i.e. those that are open for a relatively short time period) should be made with side slopes no steeper than 1 horizontal to 1 vertical.

Provided that the pile caps for the widened piers are constructed similar to the existing structure (i.e. perched above ground surface, immediately below the deck), no excavation should be required at the piers. If excavation is required at the pier locations, then such excavation would extend into the water-bearing surficial silty sand to silt deposit; dewatering of the surficial deposit would be necessary. It is noted that open-cut excavation would not be feasible at the pier locations; temporary excavation support would be required, as discussed further in Section 6.6.2. Further details related to dewatering of the surficial silty sand to silt (including a Non-Standard Special Provision) will be provided if it is determined that excavation will be required for widening of the pier pile caps.

### **6.6.2 Temporary Roadway / Track Protection**

Temporary roadway protection will be required along the west side of the Highway 400 northbound lanes and along the east side of the Highway 400 southbound lanes, to facilitate construction of the temporary access roadway and working area. The temporary excavation support system 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.

Provided that the widened piers have the same configuration as the existing piers (i.e., pier pile cap perched above the ground surface, immediately below the bridge deck), no excavation will be required for the pier widenings; therefore, temporary track protection will not be required. If the widening configuration changes from the existing configuration and excavation is required for the pier pile caps, then it is anticipated that temporary excavation support will be required to protect both the rail tracks and the embankment foreslopes. At this stage, it is recommended that the lateral movement of the temporary shoring system in this area should meet Performance Level 1b as specified in MTO's SP 105S19; however, this will be reviewed further if temporary track protection is determined to be necessary.

### **6.6.3 Obstructions During Pile Installation**

It is recommended that a Non-Standard Special Provision (NSSP) be included in the Contract Documents to warn the contractor of the presence of boulders within the overburden soils, which are glacially derived, as such obstructions may affect the installation of steel H-piles for abutment widenings. An NSSP for this purpose is provided in Appendix B.

### **6.6.4 Vibration Monitoring During Pile Installation**

Vibration monitoring should be carried out during pile installation to ensure that the vibration levels at the existing northbound and southbound structures are maintained below tolerable levels. An NSSP is provided in Appendix B, for inclusion in the Contract Documents.

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.

### **6.6.5 Artesian Groundwater Conditions for Pier Piles**

The water level associated with the extensive sand deposit at the site varies from about Elevation 244.7 m to 245.5 m, as measured during the 1957 and 2006 investigations. This is typically about 4 m to 5 m below the original ground surface at the site. However, if end-bearing piles are adopted, and if the piles for the pier widenings need to be driven deeper than the design tip elevations given in Section 6.3.2, there is a possibility that artesian groundwater conditions could be encountered. Construction of a granular filter blanket around the pier pile group is recommended, as discussed in Section 6.3.2, if end-bearing piles are adopted for support of the pier widenings.

## 7.0 CLOSURE

This Foundation Design Report was prepared by Mr. Kevin Bentley, P.Eng., and reviewed by Ms. Lisa Coyne, P.Eng., an Associate and geotechnical engineer with Golder, with technical input from Mr. Murty Devata, P.Eng., a Specialist Foundations Consultant to Golder. Mr. Fintan Heffernan, P.Eng., a Designated MTO Contact for Golder, carried out an independent review of the report.

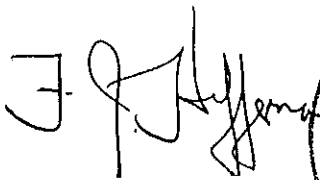
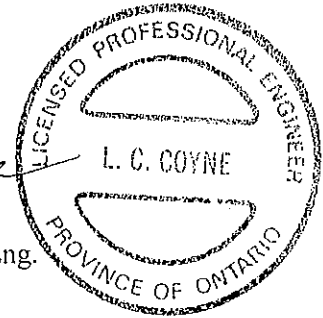
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**TABLE 1**  
**COMPARISON OF FEASIBLE FOUNDATION ALTERNATIVES**  
**WIDENING OF HIGHWAY 400 NBL BRIDGE OVER CP RAIL, G.W.P. 167-99-00**

<i>Foundation Option</i>	<i>Feasibility</i>	<i>Advantages</i>	<i>Disadvantages</i>	<i>Relative Costs</i>
Spread footings	<ul style="list-style-type: none"> <li>Not practical for support of abutments or piers</li> </ul>	<ul style="list-style-type: none"> <li>Likely lower vibration impacts on existing structure than for any of the deep foundation options</li> </ul>	<ul style="list-style-type: none"> <li>Existing embankment fill is variable (SPT "N" values varying from 2 to 38 blows per 0.3 m of penetration); differential settlement would occur between existing pile-supported structure and widening</li> <li>Excavations for footings would require temporary excavation support to protect rail tracks and abutment fore-slopes; groundwater control also required</li> </ul>	<ul style="list-style-type: none"> <li>Least expensive foundation option, but must add costs for temporary excavation support and dewatering</li> </ul>
Friction steel H-piles, driven to found within dense sand	<ul style="list-style-type: none"> <li>Feasible and practical for support of abutments and piers</li> </ul>	<ul style="list-style-type: none"> <li>Pier pile cap can be perched above ground surface, immediately below bridge deck (similar to existing), thus avoiding excavation, shoring and dewatering requirements for pier pile caps</li> <li>Minimal differential settlement between existing and widened portions of structure, and between foundation elements</li> </ul>	<ul style="list-style-type: none"> <li>Some vibration impacts on existing Highway 400 NBL and SBL structures</li> <li>Potential for encountering obstructions (cobbles and boulders) during driving</li> </ul>	<ul style="list-style-type: none"> <li>Cost for friction steel H-piles and for end-bearing H-piles are similar; they are more expensive than spread footings but less expensive than caissons</li> </ul>
End-bearing steel H-piles driven to found within 100-blow soil	<ul style="list-style-type: none"> <li>Feasible and practical for support of abutments and piers</li> </ul>	<ul style="list-style-type: none"> <li>Pier pile cap can be perched above ground surface, immediately below bridge deck (similar to existing), thus avoiding excavation, shoring and dewatering requirements for pier pile caps</li> <li>Minimal differential settlement between existing and widened portions of structure, and between foundation elements</li> </ul>	<ul style="list-style-type: none"> <li>Compared with friction piles, potentially higher vibration impacts on existing Highway 400 NBL and SBL structures (particularly during driving through dense/very dense lower soils)</li> <li>Compared with friction piles, greater potential for encountering obstructions (cobbles and boulders) during driving into dense/very dense lower soils</li> <li>Possibility of encountering artesian groundwater pressures during driving of piles for the pier widenings, if these piles need to be driven deeper than the design elevations; can be mitigated by drainage blanket</li> </ul>	<ul style="list-style-type: none"> <li>Cost for end-bearing H-piles and friction piles are similar; they are more expensive than spread footings but less expensive than caissons</li> </ul>
Caissons bored to found within 100-blow lower till or bedrock	<ul style="list-style-type: none"> <li>Feasible but less practical than driven piles for support of abutments and pier</li> </ul>	<ul style="list-style-type: none"> <li>Minimal differential settlement between existing and widened portions of structure, and between foundation elements</li> <li>Caissons can be extended to the underside of the bridge deck, thus avoiding excavation, shoring and dewatering requirements for pier pile caps</li> </ul>	<ul style="list-style-type: none"> <li>Possibility of basal heave; temporary liners required during installation</li> <li>Due to difficulties associated with extraction of temporary liners following installation into "100-blow" gravelly sand materials, permanent liners are recommended and this would add to the cost of this foundation option</li> <li>Potential for encountering obstructions (cobbles and boulders) during installation</li> <li>Excavation likely required for pier pile caps, and this would necessitate temporary excavation support and groundwater control</li> </ul>	<ul style="list-style-type: none"> <li>Most expensive foundation option</li> </ul>

## IMPORTANT INFORMATION AND LIMITATIONS OF THIS REPORT

**Standard of Care:** Golder Associates Ltd. (Golder) has prepared this report in a manner consistent with that level of care and skill ordinarily exercised by members of the engineering and science professions currently practising under similar conditions in the jurisdiction in which the services are provided, subject to the time limits and physical constraints applicable to this report. No other warranty, expressed or implied is made.

**Basis and Use of the Report:** This report has been prepared for the specific site, design objective, development and purpose described to Golder by the Client. The factual data, interpretations and recommendations pertain to a specific project as described in this report and are not applicable to any other project or site location. Any change of site conditions, purpose, development plans or if the project is not initiated within eighteen months of the date of the report may alter the validity of the report. Golder can not be responsible for use of this report, or portions thereof, unless Golder is requested to review and, if necessary, revise the report.

The information, recommendations and opinions expressed in this report are for the sole benefit of the Client. No other party may use or rely on this report or any portion thereof without Golder's express written consent. If the report was prepared to be included for a specific permit application process, then upon the reasonable request of the client, Golder may authorize in writing the use of this report by the regulatory agency as an Approved User for the specific and identified purpose of the applicable permit review process. Any other use of this report by others is prohibited and is without responsibility to Golder. The report, all plans, data, drawings and other documents as well as all electronic media prepared by Golder are considered its professional work product and shall remain the copyright property of Golder, who authorizes only the Client and Approved Users to make copies of the report, but only in such quantities as are reasonably necessary for the use of the report by those parties. The Client and Approved Users may not give, lend, sell, or otherwise make available the report or any portion thereof to any other party without the express written permission of Golder. The Client acknowledges that electronic media is susceptible to unauthorized modification, deterioration and incompatibility and therefore the Client can not rely upon the electronic media versions of Golder's report or other work products.

The report is of a summary nature and is not intended to stand alone without reference to the instructions given to Golder by the Client, communications between Golder and the Client, and to any other reports prepared by Golder for the Client relative to the specific site described in the report. In order to properly understand the suggestions, recommendations and opinions expressed in this report, reference must be made to the whole of the report. Golder can not be responsible for use of portions of the report without reference to the entire report.

Unless otherwise stated, the suggestions, recommendations and opinions given in this report are intended only for the guidance of the Client in the design of the specific project. The extent and detail of investigations, including the number of test holes, necessary to determine all of the relevant conditions which may affect construction costs would normally be greater than has been carried out for design purposes. Contractors bidding on, or undertaking the work, should rely on their own investigations, as well as their own interpretations of the factual data presented in the report, as to how subsurface conditions may affect their work, including but not limited to proposed construction techniques, schedule, safety and equipment capabilities.

**Soil, Rock and Groundwater Conditions:** Classification and identification of soils, rocks, and geologic units have been based on commonly accepted methods employed in the practice of geotechnical engineering and related disciplines. Classification and identification of the type and condition of these materials or units involves judgment, and boundaries between different soil, rock or geologic types or units may be transitional rather than abrupt. Accordingly, Golder does not warrant or guarantee the exactness of the descriptions.

## IMPORTANT INFORMATION AND LIMITATIONS OF THIS REPORT (cont'd)

Special risks occur whenever engineering or related disciplines are applied to identify subsurface conditions and even a comprehensive investigation, sampling and testing program may fail to detect all or certain subsurface conditions. The environmental, geologic, geotechnical, geochemical and hydrogeologic conditions that Golder interprets to exist between and beyond sampling points may differ from those that actually exist. In addition to soil variability, fill of variable physical and chemical composition can be present over portions of the site or on adjacent properties. **The professional services retained for this project include only the geotechnical aspects of the subsurface conditions at the site, unless otherwise specifically stated and identified in the report.** The presence or implication(s) of possible surface and/or subsurface contamination resulting from previous activities or uses of the site and/or resulting from the introduction onto the site of materials from off-site sources are outside the terms of reference for this project and have not been investigated or addressed.

Soil and groundwater conditions shown in the factual data and described in the report are the observed conditions at the time of their determination or measurement. Unless otherwise noted, those conditions form the basis of the recommendations in the report. Groundwater conditions may vary between and beyond reported locations and can be affected by annual, seasonal and meteorological conditions. The condition of the soil, rock and groundwater may be significantly altered by construction activities (traffic, excavation, groundwater level lowering, pile driving, blasting, etc.) on the site or on adjacent sites. Excavation may expose the soils to changes due to wetting, drying or frost. Unless otherwise indicated the soil must be protected from these changes during construction.

**Sample Disposal:** Golder will dispose of all uncontaminated soil and/or rock samples 90 days following issue of this report or, upon written request of the Client, will store uncontaminated samples and materials at the Client's expense. In the event that actual contaminated soils, fills or groundwater are encountered or are inferred to be present, all contaminated samples shall remain the property and responsibility of the Client for proper disposal.

**Follow-Up and Construction Services:** All details of the design were not known at the time of submission of Golder's report. Golder should be retained to review the final design, project plans and documents prior to construction, to confirm that they are consistent with the intent of Golder's report.

During construction, Golder should be retained to perform sufficient and timely observations of encountered conditions to confirm and document that the subsurface conditions do not materially differ from those interpreted conditions considered in the preparation of Golder's report and to confirm and document that construction activities do not adversely affect the suggestions, recommendations and opinions contained in Golder's report. Adequate field review, observation and testing during construction are necessary for Golder to be able to provide letters of assurance, in accordance with the requirements of many regulatory authorities. In cases where this recommendation is not followed, Golder's responsibility is limited to interpreting accurately the information encountered at the borehole locations, at the time of their initial determination or measurement during the preparation of the Report.

**Changed Conditions and Drainage:** Where conditions encountered at the site differ significantly from those anticipated in this report, either due to natural variability of subsurface conditions or construction activities, it is a condition of this report that Golder be notified of any changes and be provided with an opportunity to review or revise the recommendations within this report. Recognition of changed soil and rock conditions requires experience and it is recommended that Golder be employed to visit the site with sufficient frequency to detect if conditions have changed significantly.

Drainage of subsurface water is commonly required either for temporary or permanent installations for the project. Improper design or construction of drainage or dewatering can have serious consequences. Golder takes no responsibility for the effects of drainage unless specifically involved in the detailed design and construction monitoring of the system.



## 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.)

#### (b) Cohesive Soils

Consistency	$c_u, s_u$	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
$SO_4$	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V	field vane (LV-laboratory vane test)
$\gamma$	unit weight

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

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

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

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

#### (b) Hydraulic Properties

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

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

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

#### (d) Shear Strength

$\tau_p, \tau_r$	peak and residual shear strength
$\phi'$	effective angle of internal friction
$\delta$	angle of interface friction
$\mu$	coefficient of friction $= \tan \delta$
$c'$	effective cohesion
$c_u, s_u$	undrained shear strength ( $\phi = 0$ analysis)
$p$	mean total stress $(\sigma_1 + \sigma_3)/2$
$p'$	mean effective stress $(\sigma'_1 + \sigma'_3)/2$
$q$	$(\sigma_1 + \sigma_3)/2$ or $(\sigma'_1 + \sigma'_3)/2$
$q_u$	compressive strength $(\sigma_1 + \sigma_3)$
$S_t$	sensitivity

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

**RECORD OF BOREHOLE No 06-1**

1 OF 3 **METRIC**

PROJECT 06-1111-011

W.P. 167-99-00

LOCATION N 4931044.9 :E 286036.4

ORIGINATED BY SB

DIST 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 (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							
							20 40 60 80 100								
							○ UNCONFINED + FIELD VANE								
							● QUICK TRIAXIAL × REMOULDED								
							20 40 60 80 100			10 20 30					
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		248								
247.3			11	TO	PH		247								
10.7	Silty Clay to Clay, trace sand and gravel Stiff to very stiff Brown Moist		12	SS	7		246								
244.6							245								
13.4	Sand, trace silt Loose to compact Brown Wet						244								

Continued Next Page

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

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**RECORD OF BOREHOLE No 06-1**

2 OF 3 **METRIC**

PROJECT 06-1111-011

W.P. 167-99-00

LOCATION N 4931044.9 ; E 286036.4

ORIGINATED BY SB

DIST            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	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	20					
... CONTINUED FROM PREVIOUS PAGE ...													
	Sand, trace silt Loose to compact Brown Wet		13	SS	9								
						242							
			14	SS	6								
						241							
						240							
			15	SS	9								
						239							
						238							
						237							
			16	SS	21								
						236							
						235							
						234							
			17	SS	7								
						233							
						232							
						231							
			18	SS	56								
						230							
						229							
232.1 25.9	Sand, trace silt, containing silty clay interlayers Dense to very dense Brown Moist to wet												

MIS-MTO 001 06-1111-011.GPJ GAL-MISS.CDT 1/5/06

Continued Next Page

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

**RECORD OF BOREHOLE No 06-1**

3 OF 3 **METRIC**

PROJECT 06-1111-011

W.P. 167-99-00

LOCATION N 4931044.9 ; E 286036.4

ORIGINATED BY SB

DIST 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 (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								WATER CONTENT (%)
								○ UNCONFINED	+ FIELD VANE	×						
								● QUICK TRIAXIAL	×	REMOULDED						
	--- CONTINUED FROM PREVIOUS PAGE ---						20 40 60 80 100				10 20 30					
	Sand, trace silt, containing silty clay interlayers Dense to very dense Brown Moist to wet		19	SS	39		227									
							226									
							225									
			20	SS	55		224									
							223									
							222									
			21	SS	137		221									
							220									
			22	SS	144		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.															

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PROJECT 06-1111-011				RECORD OF BOREHOLE No 06-2				1 OF 2 METRIC						
W.P. 167-99-00		LOCATION N 4931062.1; E 286026.0		ORIGINATED BY SB										
DIST _____ 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			20 40 60 80 100	20 40 60 80 100					
257.9	GROUND SURFACE													
8.9	Topsoil Sand, trace silt, gravel and organics (FILL) Loose to dense Brown Moist		1	SS	28	257								
			2	SS	9	256								
			3	SS	22	255								
			4	SS	38	254								
			5	SS	36	253								
			6	SS	28	252								
			7	SS	16	251								
250.3	Silty Sand, trace gravel Loose Brown Moist		8	SS	5	250								
248.8	Silty Clay to clay, trace sand and gravel Firm to stiff Brown Moist		9	SS	7	249								
9.1			10	SS	13	248								
			11	SS	18	247								
246.3	Sand, trace silt Loose to dense Brown Moist to wet		12	SS	4	246								
11.6						245								
						244								
					243									

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Continued Next Page

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

<div style="display: flex; justify-content: space-between;"> <div>PROJECT <u>06-1111-011</u></div> <div><b>RECORD OF BOREHOLE No 06-2</b></div> <div>2 OF 2 <b>METRIC</b></div> </div>															
W.P. <u>167-99-00</u>		LOCATION <u>N 4931062.1 ; E 286026.0</u>		ORIGINATED BY <u>SB</u>											
DIST <u>HWY 400</u>		BOREHOLE TYPE <u>Continuous Flight Hollow Stem Augers</u>		COMPILED BY <u>KB</u>											
DATUM <u>Geodetic</u>		DATE <u>March 22, 2006</u>		CHECKED BY <u>LCC</u>											
SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT $W_p$	NATURAL MOISTURE CONTENT $W$	LIQUID LIMIT $W_L$	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 <div style="display: flex; justify-content: space-between;"> <span>20 40 60 80 100</span> <span>20 40 60 80 100</span> </div>							
... CONTINUED FROM PREVIOUS PAGE ...															
239.0 18.9	Sand, trace silt Loose to dense Brown Moist to wet		13	SS	42									0 92 (8)	
			14	SS	25										
			15	SS	11										
END OF BOREHOLE  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 1/5/06

**RECORD OF BOREHOLE No 06-3**

1 OF 4 **METRIC**

PROJECT 06-1111-011  
W.P. 167-99-00  
DIST HWY 400  
DATUM Geodetic

LOCATION N 4930984.1 E 286079.0  
BOREHOLE TYPE Continuous Flight Hollow Stem Augers and 'N' Casing/Tricone  
DATE March 23, 2006

ORIGINATED BY SB  
COMPILED BY KB  
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									WATER CONTENT (%)		
								○ UNCONFINED	+ FIELD VANE	● QUICK TRIAXIAL	× REMOULDED						20	40	60
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														
			2	SS	5														
			3	SS	10														
			4	SS	17														
			5	SS	23														
			6	SS	37														
			7	SS	31														
			8	SS	2														
249.8																			
8.7	Silty Sand to Silt, some sand, trace to some clay Very loose Brown Wet		9	SS	WR														
			10	SS	WR														
			11	SS	WR														
245.7																			
12.8	Sand, trace silt Very loose to compact Brown Moist to wet		12	SS	4														

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Continued Next Page

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



**RECORD OF BOREHOLE No 06-3**

2 OF 4 **METRIC**

PROJECT 06-1111-011

W.P. 167-99-00

LOCATION N 4930984.1 ; E 286079.0

ORIGINATED BY SB

DIST 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 $w_p$	NATURAL MOISTURE CONTENT $w$	LIQUID LIMIT $w_L$	UNIT WEIGHT $\gamma$  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								WATER CONTENT (%)		
								20	40	60							80	100
						○ UNCONFINED + FIELD VANE												
						● QUICK TRIAXIAL × REMOULDED												
							20	40	60	80	100	10	20	30				
15.0	Sand, trace silt Very loose to compact Brown Moist to wet		13	SS	8		243											
							242											
			14	SS	3		241											
							240											
			15	SS	1		239											
							238											
							237											
			16	SS	27		236											
							235											
							234											
235.7 22.9	Sand, trace to some silt, trace clay, containing silty clay interlayers Dense to very dense Brown Wet		17	SS	72		233											
							232											
							231											
			18	SS	41		230											
							229											

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0 85 12 3

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Continued Next Page

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

# RECORD OF BOREHOLE No 06-3

3 OF 4 **METRIC**

PROJECT 06-1111-011  
W.P. 167-99-00 LOCATION N 4930984.1; E 286079.0 ORIGINATED BY SB  
DIST 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 (%) GR SA SI CL					
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)				
--- CONTINUED FROM PREVIOUS PAGE ---							20 40 60 80 100	20 40 60 80 100	10 20 30										
	Sand, trace to some silt, trace clay, containing silty clay interlayers Dense to very dense Brown Wet		19	SS	58	228													
						227													
						226													
225.0						225													
33.6	Silty Clay Grey		20	SS	35	224													
	Sand, trace silt, trace to some gravel Dense to very dense Brown Wet					223													
						222													
			21	SS	53	221													
						220													
						219													
		22	SS	76	218														
					217														
216.8					216														
41.7	Gravelly Sand, trace to some silt, trace clay Very dense Brown Wet				215														
		23	SS	91	214								25 64 9 2						
214.0		24	SS	100/0.15															
44.5																			

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Continued Next Page

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

PROJECT 06-1111-011										RECORD OF BOREHOLE No 06-3										4 OF 4 METRIC									
W.P. 167-99-00										LOCATION N 4930984.1 ; E 286079.0										ORIGINATED BY SB									
DIST 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 NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT $\gamma$	REMARKS & GRAIN SIZE DISTRIBUTION (%)										
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES	SHEAR STRENGTH kPa					WATER CONTENT (%)																		
	--- CONTINUED FROM PREVIOUS PAGE ---					20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL X REMOULDED					10 20 30 W <sub>p</sub> W W <sub>L</sub>				GR SA SI CL														
	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 1/5/06

**RECORD OF BOREHOLE No 06-4**

1 OF 2 **METRIC**

PROJECT 06-1111-011 LOCATION N 4930967.3 ; E 286089.4 ORIGINATED BY SB  
 W.P. 167-99-00 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY KB  
 DIST HWY 400 DATE March 29, 2006 CHECKED BY LCC  
 DATUM Geodetic

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 (%)
								○ UNCONFINED	+ FIELD VANE						
								● QUICK TRIAXIAL	× REMOULDED						
258.4	GROUND SURFACE					20	40	60	80	100	10	20	30	GR SA SI CL	
0.0	Sand, trace to some silt and gravel, trace organics, containing cobbles In upper 1.5m (FILL) Compact to dense Brown Moist		1	SS	29										
			2	SS	26										
			3	SS	39										
			4	SS	33										
			5	SS	27										
			6	SS	24										
			7	SS	27										
			8	SS	28										
249.3			9	SS	27										
9.1	Silt, trace to some sand, trace to some clay Compact Brown Moist		10	SS	10									0 10 80 100	
247.7			11	SS	30										
10.7	Silty Clay to clay, trace sand and gravel Stiff to very stiff Brown Moist		12	SS	24										
245.9															
12.5	Sand, trace silt Loose to compact Brown Wet														

Continued Next Page

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

**RECORD OF BOREHOLE No 06-4**

2 OF 2 **METRIC**

PROJECT 06-1111-011

W.P. 167-99-00

LOCATION N 4930967.3 ; E 286089.4

ORIGINATED BY SB

DIST HWY 400

BOREHOLE TYPE Continuous Flight Hollow Stem Augers

COMPILED BY KB

DATUM Geodetic

DATE March 29, 2006

CHECKED BY LCC

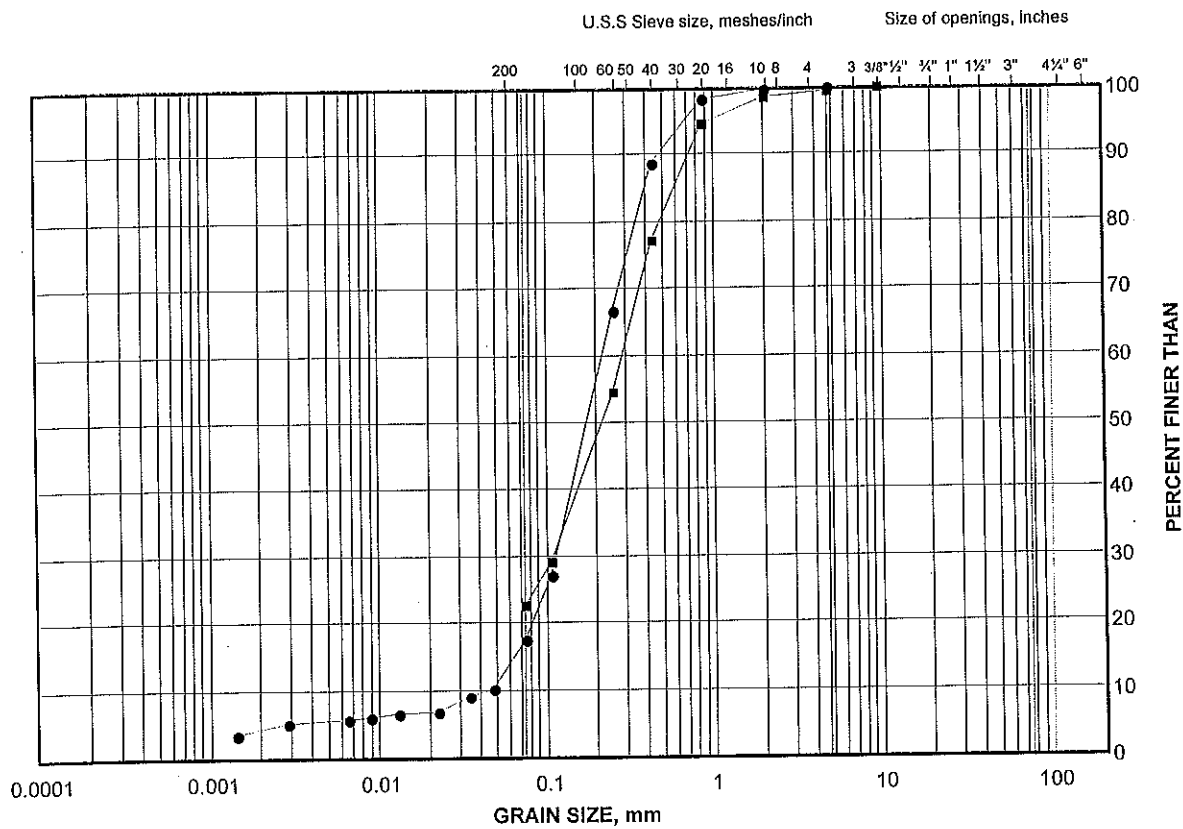
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa	WATER CONTENT (%)					
	... 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													
	Notes:  1. Water level inside hollow stem augers at 13.7m depth below ground surface upon completion of drilling.													

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# GRAIN SIZE DISTRIBUTION TEST RESULTS

Sand to Silty Sand (Fill)

FIGURE 1



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

## LEGEND

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

Project Number: 06-1111-011

Checked By: ll

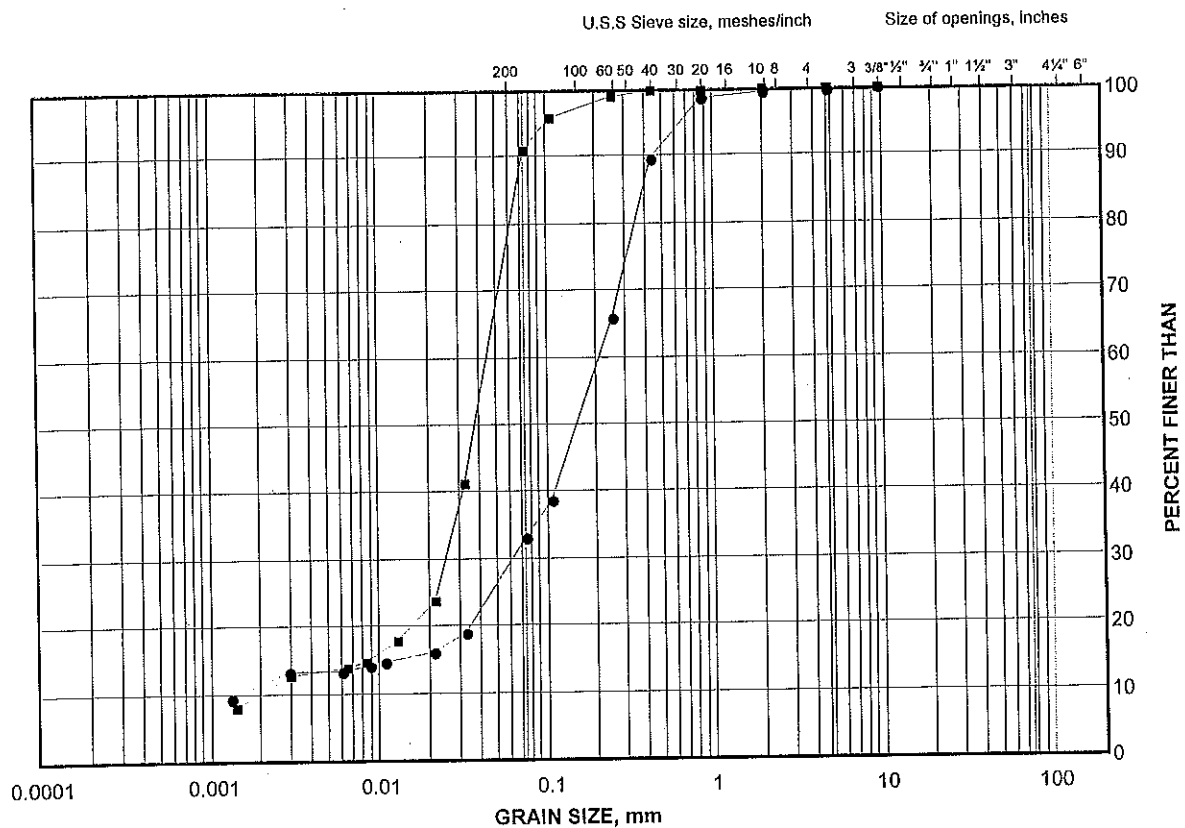
Golder Associates

Date: 02-May-06

# GRAIN SIZE DISTRIBUTION TEST RESULTS

Surficial Silty Sand to Silt

FIGURE 2



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	11	246.0
■	06-4	9	249.0

Project Number: 06-1111-011

Checked By: *[Signature]*

Golder Associates

Date: 02-May-06

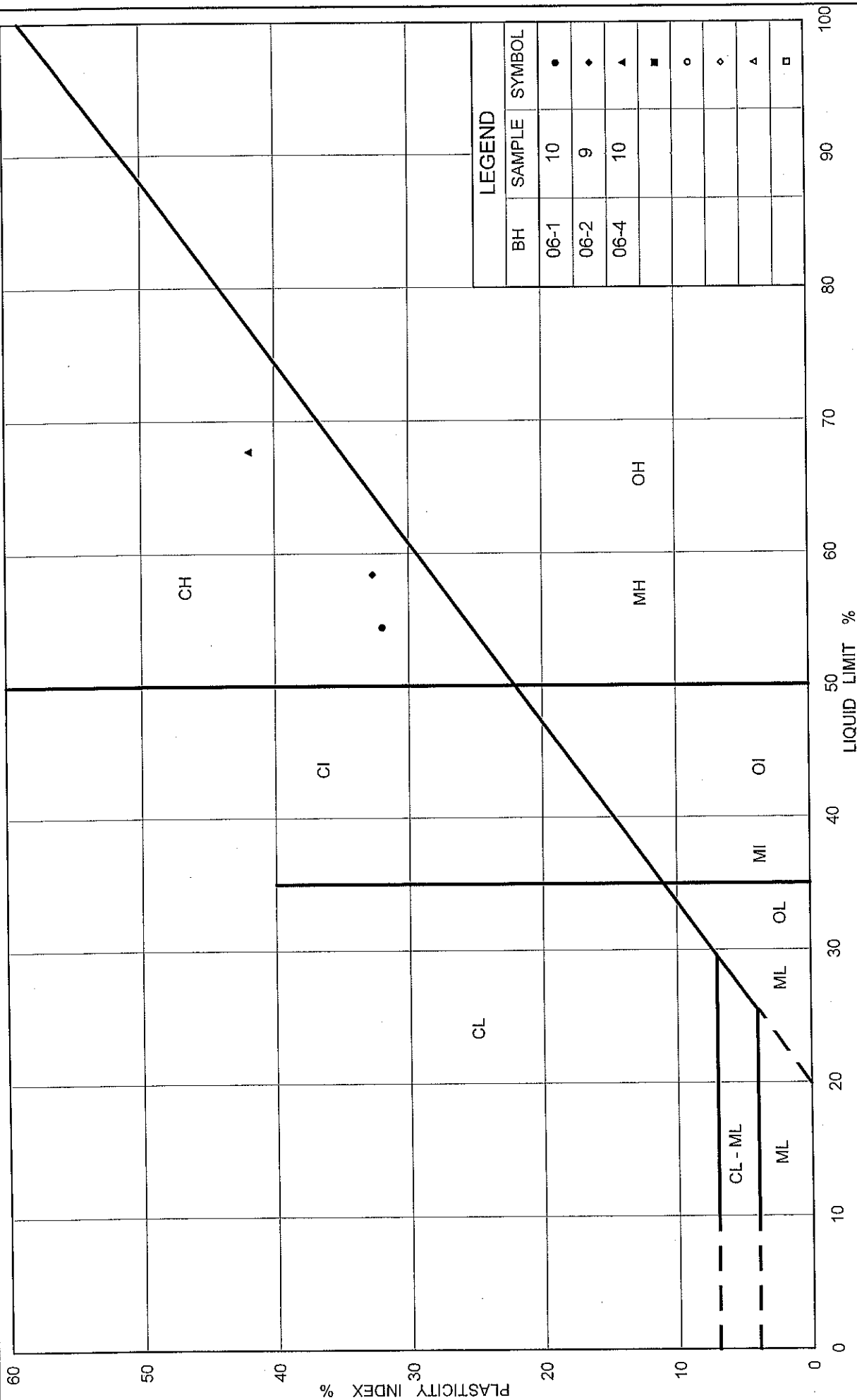


FIG No. 3

# PLASTICITY CHART

Clay

Ministry of Transportation



Ontario

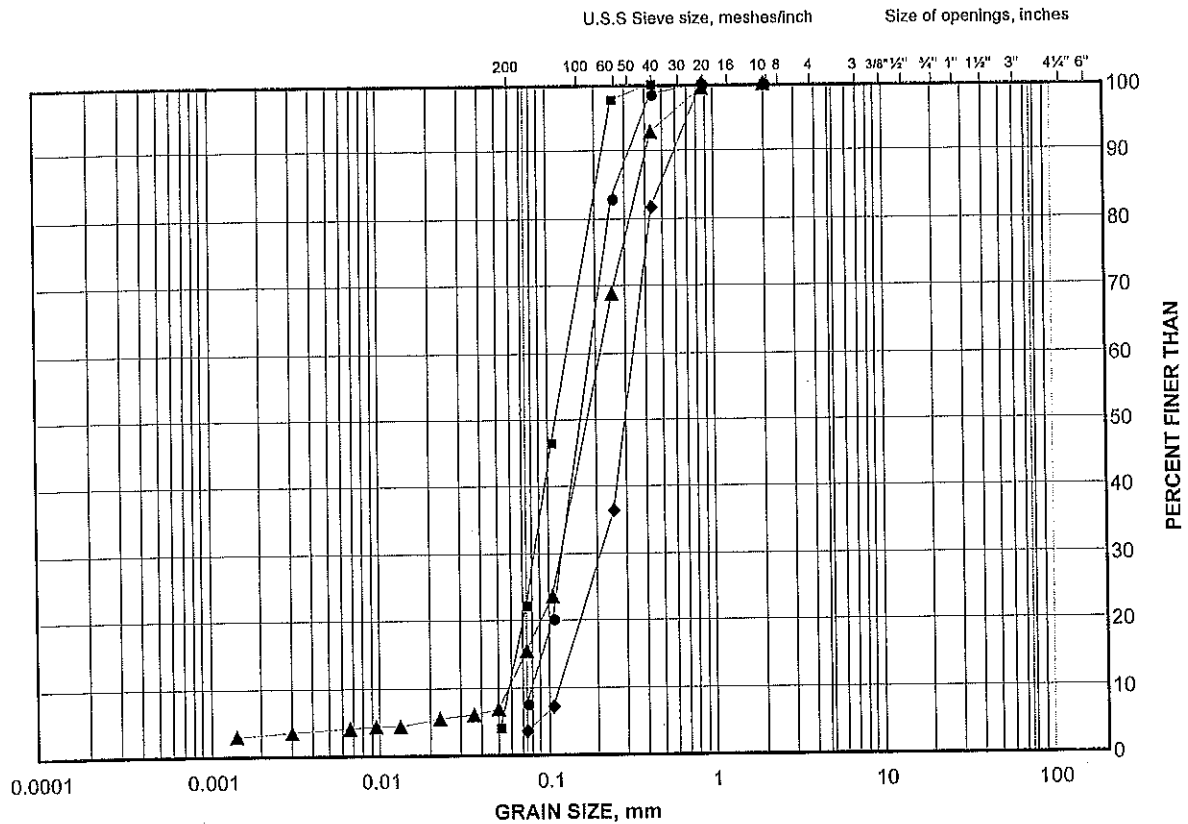
Project No. 06-1111-011



# GRAIN SIZE DISTRIBUTION TEST RESULTS

Sand

FIGURE 4



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

## LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	06-2	13	242.4
■	06-1	22	219.8
◆	06-4	14	241.3
▲	06-3	17	233.8

Project Number: 06-1111-011

Checked By: *[Signature]*

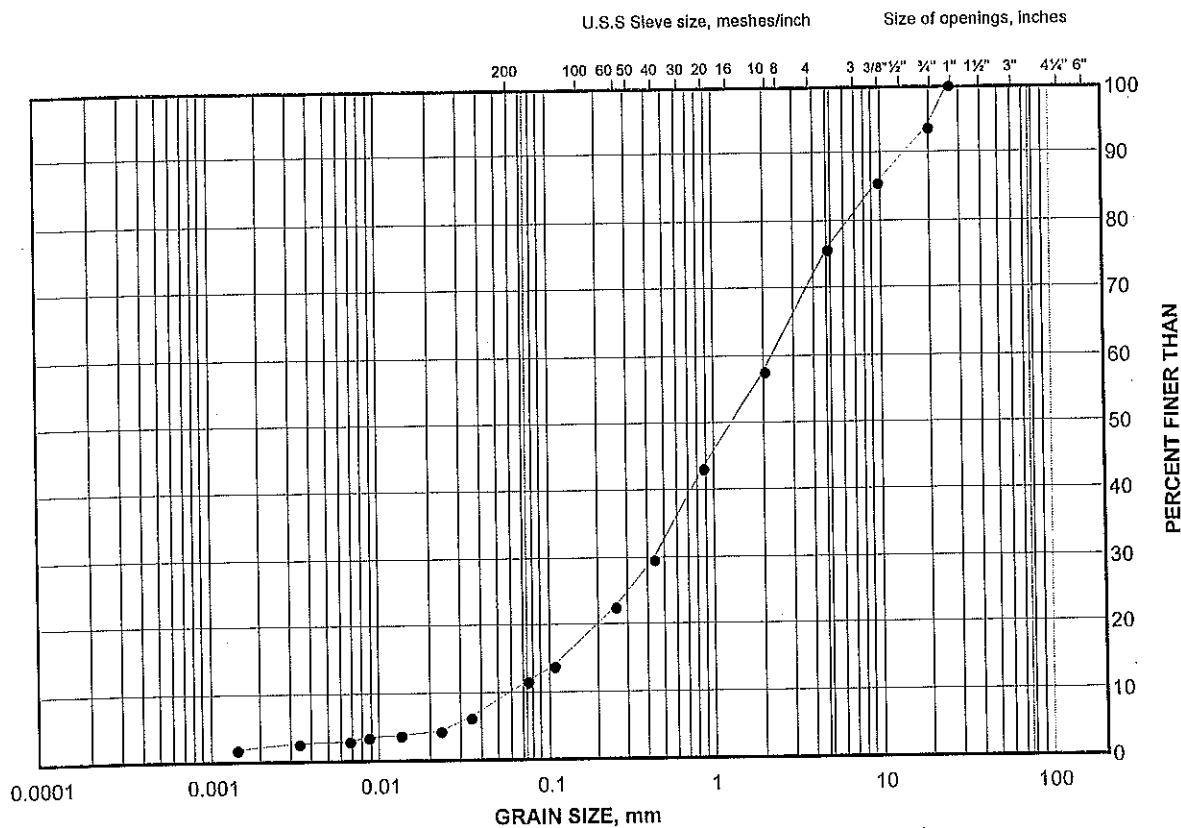
Golder Associates

Date: 02-May-06

# GRAIN SIZE DISTRIBUTION TEST RESULT

Gravelly Sand

FIGURE 5



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: *ll*

Golder Associates

Date: 02-May-06

June 2006

06-1111-011

## **APPENDIX A**

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

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

W.P. 99-75-18 LOCATION Co-ords N 16,177,217; E 938,456 ORIGINATED BY U.G.L.<sup>a</sup>  
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 LIMIT NATURAL MOISTURE CONTENT			UNIT WEIGHT γ	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>		
249.0m	Ground Level															
816.9	0.0															
247.3	Silty Sand occasional clay layers		1	SS	8											
811.4	Loose		2	SS	18											
5.5	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	End of Borehole															
51.0	NOTE: BH. #1A sampled to depth 13.5' and two TW samples were obtained															
	* U.G.L.: Universal Geotechnique Limited															

3, x 5: Numbers refer to  
Sensitivity

20  
15 5 (%) STRAIN AT FAILURE  
10

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					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			20	40	60	80	100					
818.3	Ground Level															
0.0	Silty Sand		1	SS	23											
24 7.3m	Occ. Clay Layers		2	SS	15											
811.3	Compact		3	SS	20											
7.0	Silty Clay to Clay		4	SS	20											
24 5.2m	(Stratified)		5	SS	20											
804.3	Traces of Sand and		6	SS	30											
14.0	Gravel															
	Stiff to Very Stiff															
	Silty Sand to Sand		7	SS	40											
	Occasional Silty Clay		8	SS	25											
	Layers		9	SS	24											
	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																

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

June 2006

06-1111-011

## **APPENDIX B**

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

**BOULDERS/OBSTRUCTIONS DURING PILE INSTALLATION - Item No.**

---

**Special Provision**

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The soils at the site are glacially-derived and should be expected to contain cobbles and boulders; in addition, obstructions (such as debris, cobbles or boulders) should be anticipated within the existing Highway 400 embankment fill. Appropriate equipment and procedures will be required to penetrate obstructions that are encountered during pile driving.

***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

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

---

### **Special Provision**

---

#### ***Scope***

This special provision describes requirements for vibration monitoring during the piling installation works for the widening of the existing Highway 400 NBL structure over the CP Rail tracks.

#### ***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, Highway 400 NBL Bridge Widening Over CP Rail 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 existing Highway 400 NBL and SBL structures.
- Proposed frequency of readings.
- Proposed methods for adjusting piling methods if readings show vibrations exceeding tolerable levels.

#### ***Monitoring***

The vibration monitoring equipment shall be placed on the existing Highway 400 NBL and SBL 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 for each widening area.

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