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REPORT ON

**FOUNDATION INVESTIGATION AND DESIGN
DETAIL DESIGN
CREDIT RIVER BRIDGE REPLACEMENT - NORTH BRANCH
HIGHWAY 10 WIDENING FROM 1 KM NORTH OF REGIONAL ROAD 24
NORTHERLY TO HIGHWAY 9
TOWN OF CALEDON, ONTARIO
W.P. 27-97-00, SITE NO. 24-09**

Submitted to:

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GEOCRES NO: 40P16-19

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PART A

**FOUNDATION INVESTIGATION REPORT
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1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by Morrison Hershfield Ltd. (Morrison Hershfield) on behalf of the Ministry of Transportation, Ontario (MTO) to carry out a foundation investigation as part of the detail design for the proposed widening of Highway 10 from 1 kilometre north of Regional Road 24 northerly to Highway 9 in the Town of Caledon, Ontario.

The terms of reference for the scope of work are outlined in Golder's proposal P31-1093, dated March 2003 and our supplemental letter "Revision to Borehole Drilling Program", dated November 20, 2003. The original terms of reference pertained to widening the existing bridge crossing at the North Branch of the Credit River (North Bridge) on Highway 10 as part of the project. However, after review of the options and site constraints with MTO and Morrison Hershfield, it was decided that the bridge widening was not practical and a new longer span bridge should be constructed and the old bridge removed. Structural design and permitting for the bridge replacement has been ongoing since 2004. The foundations work for the bridge replacement was carried out in accordance with the Quality Control Plan for this project dated July 2003 and our letter "Additional Work for Highway 10 Widening, Credit River Bridge Replacement – North Branch", submitted to Morrison Hershfield on August 19, 2004. A digital file of the site plan and replacement bridge general arrangement drawing was provided to Golder by Morrison Hershfield in October 2006.

The investigation was supplemented with information contained in the following reports:

- "Foundation Investigation for the Proposed Credit River Bridge, Highway 10, Ontario", GEOCREs No. 40P-16-01, prepared by Racey, MacCallum and Associates Limited, dated October 26, 1956;
- "MTO Preliminary Design Report", prepared by URS Cole, Sherman & Associates, dated July 2002.

2.0 SITE DESCRIPTION

The North Branch of the Credit River site is located about 1 km north of the intersection between Highway 10 and Highpoint Sideroad (see key plan on Drawing 1). The existing bridge is a single span reinforced concrete rigid frame structure, with a span of about 11.3 m between abutments.

The Credit River flows in a west to east direction at the site. The terrain at the site is generally flat, with the exception of the existing bridge approach embankments and elevated highway grade, at about Elevation 407 m. The ground surface adjacent to the embankment slope and within the surrounding low-lying swampy area is at about Elevation 404.5 m.

Drainage ditches are located along both sides of Highway 10, and discharge to the Credit River. At the time of the borehole investigation, the ground surface adjacent to the existing bridge abutments was below the Credit River water level.

There is a private entrance and driveway located to the northeast of the bridge. Vacant and generally flat low-lying land is present in the southeast, southwest, and northwest areas of the site. A pumping station that supplies water to the Town of Orangeville is located slightly northwest of the site, and utility lines are located within the vicinity of the site.

3.0 INVESTIGATION PROCEDURES

The field work at the North Bridge site was carried out between October 1 and December 8, 2003, at which time four (4) boreholes, numbered NB1, NB2, NB3 and NB4 were advanced at the locations shown on Drawing 1. One Dynamic Cone Penetration Test was performed adjacent to Borehole NB2. Two boreholes, numbered 56-1 and 56-2 were advanced as part of a previous investigation at the site by Racey, MacCallum, and Associates Ltd. in 1956. The locations of these boreholes are also shown on Drawing 1.

The current field investigation was carried out using track-mounted CME 55 drill rigs supplied and operated by Geo-Environmental Drilling Ltd. of Milton, Ontario (October 2003) and Groundwork Drilling Inc. of Etobicoke, Ontario (December 2003). The boreholes were advanced using 108 mm inside diameter (I.D.) continuous flight hollow stem augers, 108 mm outside diameter (O.D.) solid stem augers, and coring equipment using NW- and NQ-sized core barrels. Soil samples were obtained at depth intervals ranging from 0.75 m to 3.0 m using 50 mm outer diameter (O.D.) split-spoon samplers in accordance with Standard Penetration Test (SPT) procedures.

The boreholes were advanced to depths ranging from 9.6 m to 37.0 m below the existing ground surface. The groundwater conditions in the open boreholes were observed during the drilling operations and piezometers were installed in selected boreholes to permit monitoring of the groundwater level at these locations. The piezometers consist of a 25 mm outside diameter solid PVC pipe with a slotted screen sealed at a selected depth within the boreholes. The holes were backfilled with a bentonite slurry. The installation details and water level readings are described on the Record of Borehole sheets that follow the text of this report.

The field work was monitored throughout by members of our engineering and technical staff, who located the boreholes, arranged for the clearance of underground service locations, supervised the sampling and in-situ testing operations, logged the boreholes, and examined and cared for the soil samples. The samples were identified in the field, placed in appropriate containers, labelled and transported to our Mississauga geotechnical laboratory where the samples underwent further visual examination and laboratory testing. All of the laboratory tests were

carried out to MTO and/or ASTM Standards as appropriate. Classification testing (water content, Atterberg limits and grain size distribution) was carried out on selected samples.

The approximate borehole locations were staked in the field by Callon-Dietz personnel prior to drilling operations. Upon completion of the fieldwork, the locations of the completed boreholes were surveyed by Callon-Dietz Inc. using the NAD 83 MTM co-ordinate system and the geodetic datum for elevation.

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

The site is located within the intersection of two physiographic regions known as the Hillsburgh Sandhills and Guelph Drumlin Field physiographic regions (Chapman and Putnam, "The Physiography of Southern Ontario", 3rd Edition, 1984). The Hillsburgh Sandhills are described as having rough topography, sandy materials, and flat-bottomed swampy valleys running through the moraine from Orangeville to Hillsburgh. The Guelph Drumlin Field is predominantly composed of stony tills of the drumlins and deep gravel terraces of the old meltwater spillways; usually having a shallow overlying veneer of loam.

The ground conditions in the vicinity of the site are described to consist of kame moraines with spillways consisting of gravel terraces, and swamps. The Credit River runs through these regions and is described as following long swampy valleys in the Hillsburgh and Orangeville areas, where it has failed to cut deep channels.

4.2 Subsoil Conditions

The detailed subsurface soil and groundwater conditions as encountered in the boreholes advanced during this investigation, together with the results of the laboratory tests carried out on selected soil samples, are given on the attached Record of Borehole sheets and in Appendix A following the text of this report. The Record of Borehole logs for the boreholes from the 1956 investigation are included in Appendix B.

The stratigraphic boundaries shown on the Record of Borehole sheets are inferred from non-continuous sampling, observations of drilling progress and the results of Standard Penetration Tests (SPTs). 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 inferred soil stratigraphy based on the results of the boreholes at the Credit River Bridge – North Branch location are shown on Drawings 1 and 2.

The surficial soils at the site consist of either peat, topsoil or fill. The fill consists predominantly of sand to sand and gravel forming the existing highway embankment, and is typically underlain by topsoil, peat or silty clay. These deposits are typically underlain by a deposit of silty sand to sandy silt with organics and layers of silt, sand and clayey silt. These deposits are underlain by a compact to very dense layer of sand and gravel and by sandy gravel and gravel with interlayers of silty clay, sand, and silt at some locations. The sandy gravel and gravel layer is underlain by a stratum of interlayered silt, sandy silt, silty sand and sand which is underlain by hard clayey silt. The clayey silt is underlain at depth by a very dense sand and gravel deposit. A more detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

4.2.1 Fill

Fill was encountered at the ground surface in Boreholes NB3 and NB4, which were advanced along the existing Highway 10 embankment shoulder. The fill typically consists of sand and gravel, trace silt to sand with trace silt and gravel. The surface of the fill ranged between Elevation 406.3 m and 406.5 m and the thickness varied from 1.5 m to 2.3 m.

Standard Penetration Test (SPT) 'N' values recorded within the sand and gravel to sand fill ranged between 5 and 17 blows per 0.3 m of penetration, indicating a loose to compact relative density.

Natural water contents measured on samples of the fill ranged between 9 percent and 13 percent.

4.2.2 Topsoil / Peat / Sandy Silt to Silty Sand with Organics

Topsoil or peat was encountered at the existing ground surface in Boreholes NB1, NB2, 56-1 and 56-2. The peat and topsoil transitioned to a dark brown sandy silt to silty sand with organics, which was also encountered directly below the fill in Borehole NB4. The topsoil / peat and sandy silt to silty sand with organics layer typically contained roots and wood fragments. The surface of the topsoil/peat and sandy silt to silty sand with organic deposits ranged between Elevation 404.0 m and 404.9 m, with the thickness varying from 0.8 m to 2.3 m.

Standard Penetration Test (SPT) 'N' values recorded within the topsoil and peat layers ranged between 2 and 4 blows per 0.3 m of penetration, indicating a very soft to soft consistency. The SPT 'N' values within the sandy silt to silty sand with organics similarly ranged from 2 to 4 blows per 0.3 m of penetration, indicating a very loose to loose relative density.

The natural water content measured on samples taken within this organic layer ranged between 23 percent and 45 percent.

4.2.3 Upper Silt and Sand

A layer of predominantly silt and sand containing interlayers and seams of clayey silt and sandy silt was encountered in Boreholes NB1, NB2, NB4, 56-1, and 56-2 at a depth ranging between 1.2 m and 2.3 m below existing ground surface. The top of this layer ranged between Elevation 402.3 m and 403.9 m and the thickness recorded in the boreholes varied from 2.5 m to 4.1 m.

Standard Penetration Test (SPT) 'N' values recorded within the silt and sand layer generally ranged between 2 and 8 blows per 0.3 m of penetration, indicating a very loose to loose relative density.

The natural water content measured on samples of the silt and sand layer ranged between 20 percent and 25 percent. Two grain size distribution curves for selected samples of the silt and sand layer are shown on Figure A1.

4.2.4 Upper Clayey Silt / Silty Clay

A layer of silty clay and clayey silt was encountered immediately below the fill in Borehole NB3 and below the upper silt and sand layer in Boreholes NB2 and NB4. The mottled brown and grey clayey silt / silty clay layers contained trace to some sand and trace gravel. Occasional sand seams were noted in Borehole NB2. The top of this clayey silt / silty clay layer ranged from 2.3 m to 4.6 m below ground surface, between Elevation 401.7 m and 404.2 m, with the thickness varying between 1.1 m and 1.8 m.

Standard Penetration Test (SPT) 'N' values recorded within the clayey silt / silty clay layer generally ranged between 5 and 11 blows per 0.3 m of penetration, indicating a firm to stiff consistency.

Atterberg limits testing was carried out on two samples from this clayey silt deposit. The results of the Atterberg limits tests are plotted on the plasticity chart on Figure A4 in Appendix A. The test results, summarized below, indicate that the deposit is classified as clayey silt of low plasticity.

<i>Borehole</i>	<i>Sample</i>	<i>Elevation (m)</i>	<i>Liquid Limit (%)</i>	<i>Plastic Limit (%)</i>	<i>Plasticity Index (%)</i>
NB2	7	399.7-400.2	23	16	7
NB4	7	401.2 - 401.7	23	16	7

The natural water content measured on samples of the clayey silt / silty clay ranged between 18 percent and 23 percent.

4.2.5 Sand and Gravel / Sandy Gravel / Gravel

Layers of sand and gravel, sandy gravel, and gravel were encountered at all borehole locations. In general, an approximate 6 m thick layer of sand and gravel/gravel was encountered at the south abutment and south approach locations, at Boreholes NB1 and NB3. This deposit appears to have split into two thinner layers of sand and gravel/sandy gravel/gravel at the north abutment location (Borehole NB2), which were separated by interlayers of silt and sand and silty clay. The sand and gravel/sandy gravel/gravel (i.e., gravelly) layers typically contained trace silt and occasional silty clay and silty sand seams, and contained cobbles and boulders. The top of the gravelly layers was encountered at depths ranging between 3.4 m and 10.7 m below ground surface. The top of the gravelly layers ranged from Elevation 398.3 m to 403.1 m and the thickness where fully penetrated in Boreholes NB1 and NB2 ranged from 1.2 m to 6.1 m.

Boreholes 56-1 and 56-2 were terminated within the sand and gravel/gravel layer at a depth of 12.2 m (Elevation 392.4 m) and 10.7 m (Elevation 393.4 m) after penetrating into the layer for 3.1 m and 6.6 m respectively. Borehole NB3 was terminated within the gravel layer at a depth of 9.6 m (Elevation 396.9 m). A Dynamic Cone Penetration Test (DCPT) was performed adjacent to Borehole NB2 and effective refusal (i.e. 50 blows / 0 mm of cone penetration) was achieved at the top of the sandy gravel layer at a depth of 10.7 m (Elevation 394.1 m). Refusal may be attributed to a boulder which was encountered in the adjacent borehole at this depth.

Standard Penetration Test (SPT) 'N' values recorded within the gravelly layers ranged from 28 blows to over 100 blows per 0.3 m of penetration, indicating a compact to very dense relative density. The higher blow counts may be attributed to cobbles and boulders encountered within the deposit.

The natural water content measured on samples within these gravelly layers ranged between 4 percent and 11 percent. Three grain size distribution curves for selected samples within these gravelly layers are shown on Figures A2 and A3.

4.2.6 Sand and Silt

A layer of sand, trace gravel and a layer of silt with gravel was encountered in Boreholes NB2 and 56-1 respectively, within the gravelly layers noted above. The top of the sand and silt layers was encountered at depths of 7.6 m and 7.0 m below ground surface for Boreholes NB2 and 56-1, respectively, corresponding to Elevation 397.1 m and 397.6 m, with a thickness of 1.5 m and 2.1 m for Boreholes NB2 and 56-1, respectively.

Standard Penetration Test (SPT) 'N' values recorded within the sand and silt layers were 26 and 23 blows per 0.3 m of penetration, indicating a compact relative density.

The natural water content measured on a sample within the sand layer was 17 percent.

4.2.7 Silty Clay / Clayey Silt

A silty clay/clayey silt layer was encountered underlying the above noted sand layer in Borehole NB2, and underlying the gravelly layer in Borehole NB4. The silty clay/clayey silt layer typically contained trace to some sand, trace gravel. The top of the silty clay/clayey silt layer was encountered at a depth of 9.1 m below ground surface, corresponding to Elevations of 395.6 m and 397.2 m for Boreholes NB2 and NB4 respectively. The silty clay/clayey silt layer was 1.6 m thick in Borehole NB2, while Borehole NB4 terminated 0.5 m into the clayey silt at a depth of 9.6 m (Elevation 396.7 m).

Standard Penetration Test (SPT) 'N' values recorded within the clayey silt/silty clay layer were 9 and 20 blows per 0.3 m of penetration, indicating a stiff to very stiff consistency.

The natural water content measured on a sample of the clayey silt layer was 23 percent.

4.2.8 Lower Silt

Below the sand and gravel layers, a layer of silt was encountered in Boreholes NB1 and NB2. The silt contained trace to some sand and trace clay. The top of this silt layer was at 16.0 m and 16.8 m below ground surface in Boreholes NB1 and NB2, respectively, corresponding to Elevation 388.9 m and Elevation 388.0 m, and to a thickness of 5.3 m and 4.5 m, respectively.

Standard Penetration Test (SPT) 'N' values recorded within the lower silt layer ranged between 50 blows and 85 blows per 0.3 m of penetration, indicating a very dense state of packing.

The natural water content measured on two samples of the lower silt were 19 percent and 22 percent. Two grain size distribution curves for selected samples of the lower silt are shown on Figure A1.

4.2.9 Lower Sandy Silt / Silty Sand / Sand

Underlying the lower silt deposit, a layer of silty sand to sandy silt was encountered in Boreholes NB1 and NB2. This lower sandy silt / silty sand layer contained clayey silt to silty clay seams. The top of the sandy silt / silty sand layer was at a depth of 21.3 m in both boreholes, corresponding to Elevation 383.6 m and Elevation 383.4 m in Boreholes NB1 and NB2, respectively. This lower sandy silt / silty sand layer was 3.1 m thick in Boreholes NB1 and NB2.

Silty sand and sand layers were also encountered between the gravel layer and the lower silt layer in Borehole NB1. The top of the silty sand/sand layers was at 12.2 m depth (Elevation 392.7 m) and the combined layer thickness was 3.8 m.

Standard Penetration Test (SPT) 'N' values recorded within the lower sandy silt, silty sand, and sand layer ranged between 59 blows and 87 blows per 0.3 m of penetration, indicating a very dense state of packing.

The natural water content measured on samples of the lower sandy silt, silty sand, and sand ranged between 21 percent and 25 percent.

4.2.10 Lower Clayey Silt

Underlying the very dense lower silt and lower sandy layers, a layer of clayey silt was encountered at a depth of 24.4 m in Boreholes NB1 and NB2. The top of this clayey silt layer was at Elevation 380.5 m and 380.4 m in Boreholes NB1 and NB2 respectively. This lower clayey silt layer was found to be 9.1 m thick in Borehole NB1, while Borehole NB2 terminated 0.4 m into this clayey layer at a depth of 24.8 m (Elevation 379.9 m).

Standard Penetration Test (SPT) 'N' values recorded within the lower clayey silt layer ranged between 65 blows and 80 blows per 0.3 m of penetration, indicating a hard consistency.

The results of the Atterberg limits testing carried out on a sample of this lower clayey silt deposit are illustrated on the plasticity chart on Figure A4 in Appendix A and summarized below.

<i>Borehole</i>	<i>Sample</i>	<i>Elevation (m)</i>	<i>Liquid Limit (%)</i>	<i>Plastic Limit (%)</i>	<i>Plasticity Index (%)</i>
NB1	19	377.0 – 377.5	31	16	15

The test results on this sample indicate that the lower clayey silt is of low to medium plasticity.

The natural water content measured on selected samples of the lower clayey silt deposit ranged between 24 percent and 25 percent.

4.2.11 Lower Sand and Gravel

Underlying the lower clayey silt layer, a deposit of sand and gravel was encountered in Borehole NB1. The top of the sand and gravel deposit was encountered at 33.5 m below ground surface (Elevation 371.4 m) and the borehole was terminated within the deposit at a depth of 37 m (Elevation 367.9 m).

Standard Penetration Test (SPT) 'N' values recorded within this lower sand and gravel layer ranged between 105 blows per 0.3 m of penetration and 101 blows per 0.15 m of penetration, indicating a very dense state of packing.

The natural water content measured on a sample taken at the interface of the clayey silt and lower sand and gravel deposit was 14 percent.

4.2.12 Groundwater Conditions

Water levels were noted within the boreholes during and after the drilling operations. Piezometers were installed in Boreholes NB1 and NB2. The piezometer in Borehole NB1 was sealed in the upper silt layer and the piezometer in Borehole NB2 was sealed within the lower silty sand deposit. Details of the piezometer installations are shown in the Record of Borehole Sheets following the text of this report. The water levels in the piezometers are summarized below:

<i>Borehole</i>	<i>Ground Surface Elevation (m)</i>	<i>Ground Water Level Depth (m)</i>	<i>Ground Water Level Elevation (m)</i>	<i>Date</i>
NB1	404.9	0.75	404.2	January 7, 2004
NB2	404.7	0.50	404.2	January 7, 2004

The groundwater level in the open boreholes and piezometers was generally found to be at approximately the same level as the Credit River in the vicinity of the North Bridge structure. The river water level was measured at Elevation 404.7 m in December, 2003. It should be noted that rapid changes in the river water level were noticed at the site during our investigation as a result of heavy rains and snowmelt. It should also be noted that groundwater levels in the area are subject to seasonal fluctuations.

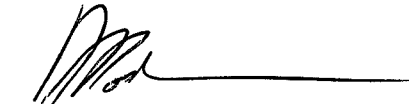
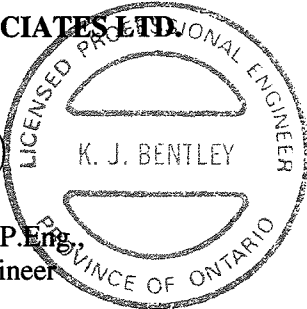
4.3 Closure

The field technician supervising the drilling program was Mr. Gerard Defreitas. This report was prepared by Mr. Kevin J. Bentley, P.Eng., a geotechnical engineer; the technical aspects were reviewed by Ms. Anne Poschmann, P.Eng, Principal of Golder. An independent quality control review was provided by Messrs. Fintan J. Heffernan, P.Eng, and Jorge Costa, P.Eng., both Designated MTO Contacts for Golder.

GOLDER ASSOCIATES LTD.



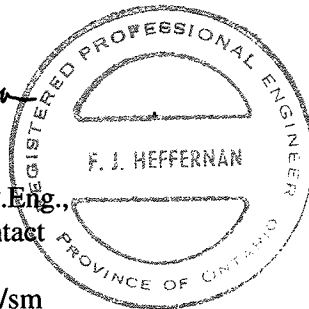
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KJB/ASP/JMAC/FJH/sm

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**PART B
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5.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides foundation design recommendations for the proposed replacement of the existing Credit River Bridge – North Branch as part of the Highway 10 widening project in the Town of Caledon. 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 provided are intended only to provide the designers with sufficient information to assess the feasible foundation alternatives and to design the proposed structure foundations. As such, where comments are made on construction they are provided only in order to highlight those aspects which could affect the design of the project. 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.

It is understood that the existing single span rigid frame bridge will be replaced by a single span reinforced concrete bridge. The span of the new bridge is about 20 m and its width is about 29 m. The available drawings indicate that the current bridge span is about 11 m, its width is about 15 m and it is supported on spread footings which are founded at Elevation 399.6 m. The dimensions of the existing spread footings are indicated to be approximately 2.1m wide x 15.8 m long x 0.9 m thick. The existing road grade at the North Bridge location is at about Elevation 407 m resulting in embankments up to 3 m in height above existing ground surface near the river bank. The General Arrangement drawing for the North Bridge replacement structure was provided by Morrison Hershfield in electronic format in October 2006.

5.1 Founding Options - General

Various alternatives for the abutment foundations were considered and a summary of these alternatives is presented in Table 1, following the text of this report. Based on the subsurface conditions and the high groundwater table, substantial site dewatering would be required in order to permit the use of spread footings for the bridge widening and protection works would be required to mitigate potential adverse impact on the founding soils of the existing spread footings. Given the proximity to the Credit River, the potential difficulties in achieving sufficient dewatering and the environmental sensitivity of the area, shallow foundations are not considered suitable or practical for the bridge replacement. Due to the presence of cobbles and boulders within the subsoils and the potential difficulties associated with groundwater inflow and increased drill depths, caissons are not considered a suitable option. Steel pipe piles founded within the upper very dense sand and gravel deposit were previously considered as one of the preferred foundation options when bridge widening was being considered and minimizing disturbance to the existing bridge foundations was considered critical in order for the existing bridge to remain in service. However, given that the existing bridge will now be removed and all traffic lanes will

ultimately be supported on the replacement structure, this option is no longer the preferred option given the potential variability of the founding stratum and the relatively low axial resistance and requirement for PDA testing to confirm the load carrying capacity of the pipe piles.

It is considered that steel H-piles driven below Elevation 385 m is the most practical option from a geotechnical / foundation perspective for support of the replacement structure. The pile cap should be maintained as high as possible in order to minimize dewatering requirements and the potential for disturbance to the existing spread footing founding soils during the excavation for pile installation and pile cap construction.

5.2 Steel H-Pile Foundations

Steel H-piles are recommended for support of the abutments. Steel H-piles are a non-displacement type pile and with varying depths of preaugering may be driven to found within either:

- i) the very dense sand and silt layer which underlies the sand and gravel/sandy gravel/gravel layers at variable depths;
- ii) the hard clayey silt layer encountered at about Elevation 380 m to 381 m; or
- iii) the very dense sand and gravel below about Elevation 370 m.

For design, the following pile tip levels may be assumed for piles terminated within the various soil strata options described above.

<i>Foundation Location</i>	<i>Design Pile Tip Elevation</i>		
	Very Dense Sand and Silt	Hard Clayey Silt	Very Dense Sand and Gravel
North & South Abutment	385 m	375 m	369 m

There should be provision made in the contract for dealing with varying pile lengths and pre-augering to achieve the design pile tip elevations.

5.2.1 Axial Geotechnical Resistance

For HP 310 x 110 piles, the deeper the piles can be driven (i.e. the lower the design pile tip level), the greater the design capacity will be. The following design capacities may be assumed for the three founding levels / strata listed in Section 5.2.

Pile Type	Design Pile Tip Elevation	Factored Axial Resistance at ULS	Axial Resistance at SLS
Steel H-Piles 310 x 110	385 m	800 kN	600 kN
	375 m	1,000 kN	700 kN
	369 m	1,250 kN	850 kN

Given the generally very dense/hard condition of the above founding strata, as well as the fact that the granular deposits contain cobbles and boulders, it is not possible to predict how deep the H-piles can be driven without encountering heavy driving and/or obstructions which may cause the piles to “hang up” or induce excessive vibrations during driving which may have adverse impact on the existing bridge foundations (see Section 5.6). If heavy driving is encountered during pile installation, there is potential for settlement or liquefaction of the silty/fine sandy deposits which may have an adverse impact on the existing bridge foundations and embankment which are to remain open to traffic during the initial stages of construction. In this regard, pre-augering would be required to minimize the vibrations from pile driving, as well as to ensure that the piles reach the design tip levels. Based on boreholes NB2 and 56-1, pre-augering at the north abutment should be carried out to about Elevation 388 m. Based on boreholes NB1 and 56-2, pre-augering at the south abutment should be carried out to Elevation 392 m.

Since the pile capacity at this site will be a combination of the shaft friction along the pile length and resistance at the pile toe, the capacity is extremely sensitive to the pile tip level which can be achieved.

The pile capacity must be verified in the field by the use of the Hiley formula (Standard Structural Drawing SS-103-11) during the final stages of driving to achieve an ultimate capacity equal to two times the design ULS value. A note, similar to the following example, should be shown on the Contract drawing (using a ULS value of 1,000 kN as an example):

- “Piles to be driven in accordance with Standard SS103-11 (Hiley method) using an ultimate capacity of 2,000 kN per pile, but must be driven to at least Elevation 375 m”

The pile termination or set criteria for the pile capacity selected 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.

It is generally recommended that the H-pile tips be stiffened when driving piles into soils which contain cobbles and boulders such as those found at this site. The piles should be stiffened with driving shoes (i.e. MTO flange plates in accordance with OPSD 3301.00 and OPSS 903.07.05.04, Titus “Standard” design, or equivalent). An NSSP should be included in the contract to address this issue; suggested wording is included in Appendix C for reference. Pile installation and driving shoes should be in accordance with SP903S01.

Vibration monitoring should be carried out during pile installation to ensure that the vibration levels at the existing bridge structure are maintained within tolerable ranges (see Section 5.7). The pile driving criteria may have to be adjusted depending on the results of the vibration monitoring.

In addition to vibration monitoring, it is recommended that a settlement monitoring program be established to monitor the existing bridge during piling operations.

5.2.2 Resistance to Lateral Loads

Lateral loading could be resisted fully or partially by the use of battered piles. If vertical piles are used, the resistance to lateral loading will have to be derived from the soil in front of the piles; however, the soils within the design scour depth should not be assumed to contribute to the lateral resistance.

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) for pile width B (m), is based on the equations given below (Canadian Foundation Engineering Manual, 3rd Edition).

For cohesive soils:

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

For cohesionless soils:

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

The recommended range for the values of n_h to be used in the structural analysis are presented below. The range in values reflects the variability in the subsurface conditions and will depend on the design elevation of the pile cap. While design values are provided for the full stratigraphic sequence at the site, this is likely more than what is needed for the design of the H-piles given the optional design pile tip elevations.

<i>Soil Unit</i>	<i>τ_u (MPa)</i>	<i>n_h (MPa/m)</i>
Embankment fill (assumed to be compacted granular fill)	–	5 to 10
North Abutment:		
Very loose to loose silts and sands above Elevation 399 m	–	1 to 3
Dense to very dense sands and gravels between Elevation 399 m and 388 m	–	8 to 12
Very dense sands and silts between Elevation 388 m and 380 m	–	10 to 15
Hard clayey silt between Elevation 380 m and 371 m	0.2	
Very dense sand and gravel between Elevation 371 m and 369 m	–	15
South Abutment:		
Very loose to loose silts and sands above Elevation 399 m	–	1 to 3
Very dense sands and gravels between Elevation 399 m and 393 m	–	10 to 15
Dense to very dense sands and silts between Elevation 393 m and 380 m	–	8 to 12
Hard clayey silt between Elevation 380 m and 371 m	0.2	
Very dense sand and gravel between Elevation 371 m and 369 m	–	15

Group action for lateral loading should 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 lateral subgrade reaction in the direction of loading by a reduction factor as follows:

<i>Pile Spacing in Direction of Loading (d = Pile Diameter)</i>	<i>Reduction Factor</i>
8d	1.0
6d	0.7
4d	0.4
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).

5.2.3 Frost Protection

The pile caps should be provided with a minimum of 1.4 m of soil cover for frost protection.

5.3 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. For this site location, the geotechnical seismic considerations do not impact on the design since it is within the lowest seismic zone given in the CHBDC.

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 percent passing the No. 200 sieve, should be used as backfill behind the walls. 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 3101.150 and 3121.150.
- 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 Special Provision 105S10. 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 1.4 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 proposed embankment fill materials and the following parameters (unfactored) may be used, assuming the use of Select Subgrade Material (SSM) for the new portions of the approach embankments:

	SSM
Soil unit weight:	20 kN/m ³
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 as placed and the following parameters (unfactored) may be assumed:

	Granular ‘A’	Granular ‘B’ (Type II)
Soil unit weight:	22 kN/m ³	21 kN/m ³
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.

5.4 Approach Embankment Design and Construction

The proposed top of pavement grade at the bridge replacement is at about Elevation 407.2, slightly higher than the existing grade which is at about Elevation 406.6 m at the east and west approaches. The existing ground surface at the proposed bridge and embankment widening varies from about Elevation 404.0 m in the low-lying swampy area to Elevation 406.8 m at the existing shoulder, resulting in approach embankments about 3 m in height. It should be noted that the elevation of the ground surface rises slightly to the north and south of the Credit River, reducing the overall embankment height in this area.

5.4.1 Subgrade Preparation and Embankment Construction

Based on the borehole results, the existing embankment fill is underlain by a layer of either peat, topsoil, or sandy silt to silty sand with organics that ranges in thickness from 0.7 m to 1.2 m. Topsoil, peat, and sandy silt to silty sand with organics are also present at ground surface extending to depths ranging from 0.8 m to 2.4 m beyond the limits of the existing fill embankment and within the footprint of the proposed embankment. These depths are based on borehole data from the current investigation as well as the pavement investigation performed by Golder.

Prior to the placement of any fill for the new approach embankment construction, grade raise and widening, all topsoil, peat, underlying soils containing organics, and softened / loosened soils should be stripped from below the proposed approach embankment areas in accordance with SP206S03 and wasted/reused for landscaping.

For the widening of the existing embankment, construction procedures should implement the guidelines of OPSD 203.020. These guidelines require that the temporary excavation be extended into the existing embankment with side slopes at 1H:1V which will allow removal of some extent of the organic material underlying the existing embankment; however, in order to ensure the integrity of the existing embankment, the procedures provided in Section 5.4.4 must be followed.

All subgrade soils above the water level should be proof-rolled prior to placement of the embankment fill, and any poorly performing areas should be subexcavated and replaced with approved engineered fill. Once the organic materials are removed and the subexcavation is backfilled, benching into the existing side slopes should be carried out as per OPSD 208.010 for construction of the embankment above the water level to ensure keying of the new fill into the existing fill.

Granular fill meeting the specification of Granular “A” or Granular “B” Type II should be used for backfilling of the subexcavation carried out for removal of the organic deposits. It is anticipated that sheetpiling and dewatering will be required for construction of the pile cap in the dry (see Section 5.5); however, it is likely to be impractical to dewater outside the area of the proposed sheetpile cofferdam. Clearstone or other coarse grained material should not be used as backfill to excavations below the groundwater table unless there is adequate filtration provided to prevent migration of fines from the underlying sands/silts into the coarse grained fill. The backfilling operations will be carried out partly under water unless groundwater control measures are implemented and as such the choice of material used will depend on whether there is option to carry out preloading as described in Section 5.5.5. If preloading is feasible, then the option of placing Granular “A” or “B” Type II is considered feasible. Construction of the embankment above the prepared subgrade may be carried out using clean earth fill meeting the specifications of OPSS 212, or Granular / Select Subgrade Material meeting the specifications of OPSS 1010, depending on material availability. Given that the existing embankment is composed of sand and gravel fill, it is preferable that granular fill be used for the widening to limit the potential for differential settlement.

Embankment fill should be placed in regular lifts with loose thickness not exceeding 300 mm, and be compacted to at least 95 per cent of the material’s standard Proctor maximum dry density. The final lift prior to placement of the granular subbase and base courses should be compacted to at least 100 per cent of the standard Proctor maximum dry density. Inspection and field density testing should be carried out by qualified geotechnical personnel during all engineered fill placement operations to ensure that appropriate materials are used and that adequate levels of compaction have been achieved.

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

5.4.2 Approach Embankment Stability

Limit equilibrium slope stability analyses were performed using the commercially available program SLOPE/W, produced by Geo-Slope International Ltd., employing the Morgenstern-Price method of analysis, to check that a minimum factor of safety of 1.3 is achieved for the proposed approach embankment height and geometry under static conditions. This minimum factor of safety is considered appropriate for the embankments at this site considering the design requirements and the available field and laboratory testing data.

The subsoils encountered in the area of the north and south approach embankments are composed primarily of cohesionless soils. For these soils, effective stress parameters were employed in the analysis assuming drained conditions and the parameters were estimated from empirical correlations using the results of in situ Standard Penetration Tests (SPT), visual classification, and laboratory data. The piezometric conditions used in the analysis is based on the groundwater levels noted during drilling and measured in the standpipe piezometers.

Static slope stability analyses were carried out using the following parameters based on field and laboratory test data and accepted correlations:

<i>Soil Deposit</i>	<i>Bulk Unit Weight</i>	<i>Effective Friction Angle</i>	<i>Undrained Shear Strength</i>
Embankment Fill	22 kN/m ³	32°	—
Peat/Topsoil/Native Soil with Organics	18 kN/m ³	27°	—
Upper Silt and Sand	20 kN/m ³	30°	—
Upper Clayey Silt (North Abutment)	19 kN/m ³	—	25 kPa
Sand and Gravel	21 kN/m ³	35°	—

The analyses were carried out for two conditions:

- 1) temporary condition for subexcavation of organics as shown on Figure 1, and
- 2) final embankment configuration, as shown on Figure 2.

For the temporary condition, the analysis assumes that the existing highway embankment is cut back at an overall slope of 1H:1V in order to remove as much as possible of the peat, topsoil, and native soils with organics underlying the existing embankment. The analysis also assumes that the peat, topsoil, and native soils with organics have been removed from the proposed new embankment footprint. The results of the analysis are shown in Figure 1 and indicate that a factor of safety of less than 1 is obtained for a failure surface that would impact the operation of the

existing roadway. Based on these results, it is recommended that the measures as described in Section 5.4.4 be implemented during subexcavation to ensure the integrity of the embankment.

For the final embankment configuration, assuming the grade of the widened highway is about Elevation 407 m and the new embankment side slopes are constructed at a 2 horizontal to 1 vertical (2H:1V) slope, a factor of safety greater than 1.3 is obtained (See Figure 2).

5.4.3 Settlement

Settlement analyses were performed for the conditions of constructing the proposed fill embankment widening with and without subexcavation of the organic deposits. For these analyses, the range in thickness of the organic deposits was based on the results of the boreholes put down at the site. It should be noted that the thickness of these deposits is expected to be variable given the nature of the site and the deposits themselves. The variability will have an impact on both the settlements which will occur under the embankment loading as well as on the subexcavation requirements.

5.4.3.1 Proposed Approach Embankment

Provided that the embankment fill material consists of properly placed and compacted earth fill or granular fill, the settlement of the new embankment fill itself is expected to be less than 25 mm. If granular fill is used, the majority of settlement will occur during construction.

Peat, topsoil and native soils with organics were encountered at ground surface in the boreholes at the toes of the existing embankment and these materials were also encountered below the existing embankment fill. If the peat/topsoil/organics were to be left in place under the widened embankment, settlement of the embankment would occur as a consequence of consolidation of these materials. For the settlement analyses completed, it is calculated that about 800 mm (0.8 m) of consolidation could occur under a 3 m high embankment. The majority of this settlement (up to about 90%) of the peat/topsoil/organics is expected to occur within about 3 to 4 months after completion of the embankment. The remainder of the settlement (up to about 150 mm) includes creep movement and is expected to continue over the life of the roadway.

If the organic deposits are removed, the results of the analyses indicate that settlement of the underlying very loose to loose silts and sands would be up to about 50 mm for the proposed embankment (assuming up to 2 m organics removal and engineered fill replacement). Settlement of the silts and sands is expected to occur rapidly, (i.e. during or shortly after construction) in response to the construction based on the relatively high permeability of the soils and presence of sand seams. In addition to the above foundation soil settlements, embankment settlement due to compression of the granular fill itself will occur as discussed previously in this section.

There will also be some consolidation of the existing organic deposits which are left in place under the existing embankment side slopes since it will not be possible to remove all of the existing organics. The proposed construction involves cutting back the existing side slope to 1H:1V but then adding fill to a higher grade level than is currently present – a maximum additional loading equivalent to about 1.5 m to 2 m of fill. Due to the variable thickness of the organic deposits (0.7 m to 1.2 m) underlying the existing embankment and the embankment construction/geometry, it is estimated that embankment settlement due to consolidation of the organic deposits could range up to 150 mm. The majority of this settlement (up to about 90% or 100 mm) is expected to occur within about 2 to 3 months following construction; however, there will also be continued creep settlement as well as natural decay of the organic material.

5.4.4 Measures for Improving Temporary Excavation Stability

As discussed in Section 5.4.2, assuming that the existing approach embankment is excavated back to a 1H:1V slope (from the existing approximate 2H:1V slope) in order to remove as much of the underlying peat/topsoil/native soil with organics, the resulting temporary slope is unstable. In order to complete the subexcavation without impacting the existing road embankment, a temporary support system would have to be installed to support the existing embankment. Alternatively, restrictions could be placed on the permitted length/width of open excavation.

The temporary support system should be designed to Performance Level 2 in accordance with MTO Special Provision 105S19. The system could consist of a driven steel sheet pile or soldier pile and lagging wall. Temporary support to the wall would probably have to be by tiebacks since the sands/silts underlying the organics would not provide suitable support to raker footings particularly given the groundwater level at the site.

Alternatively, excavation of the organics deposits could be carried out in stages as follows:

- Excavation in strips formed perpendicular to the highway alignment with the base of the excavation/trench no wider than 3 m;
- Excavation such that the base of the cut is maintained outside a zone defined by a line drawn downward at 1 horizontal : 1 vertical (1H:1V) from the crest of the existing roadway embankment to the base of the cut;
- Backfilling operations to original ground level completed for each strip prior to commencing excavation of the adjacent strip.

5.4.5 Mitigation of Time Dependant Settlement

As indicated in Section 5.4.3, there will still be some of the peat/topsoil/native soil with organics deposits remaining in place under the existing embankment and there will be consolidation settlement of these deposits due to the additional loading applied over the area of the temporary 1H:1V side slope. The majority of the settlement is anticipated to occur within 3 months of placing the embankment fill; however, there will also be ongoing settlement due to natural organic decay processes.

Consideration should be given to preloading the embankment to allow for as much of the settlement to occur as possible prior to constructing the approach slab or pavement structure and hence limit the subsequent maintenance and padding to raise the level the new roadway grade. It is recommended that the new embankment fill be constructed as early in the contract as possible.

5.5 Excavations and Temporary Cut Slopes

Excavations for pile installation and construction of the pile caps (i.e. bottom of abutment) at the abutments will typically extend through the existing sand and gravel embankment fill, the peat/topsoil/organic sandy silt to silty sand, and will be terminated within the underlying silts and sands. Based on the design drawings, the pile cap (i.e. base of abutment) is at about Elevation 402.7 m. The river water level at the North Bridge structure was measured to range between Elevation 404.7 m and 404.2 m in December 2003 and January 2004, respectively. Therefore, the excavations will extend up to about 2 m below the groundwater level which was generally found in the boreholes to be at the same level as the Credit River. It should be noted that rapid changes in the river water level were noticed at the site during our investigation as a result of heavy rains and snowmelt.

Due to the close proximity to the Credit River, adjacent existing road embankment and bridge structure, high water level, and presence of typically very loose to loose soils, it is anticipated that excavations for the pile caps will require extensive groundwater control together with temporary excavation support systems. An NSSP has been included in Appendix C to alert the contractor of the water-bearing soils and dewatering that is required.

In discussions with the bridge designer, it is understood that construction staging to maintain traffic along Highway 10 will require four separate work areas for the northeast, northwest, southeast, and southwest bridge widening area pile caps. Following installation of the piles and construction of the abutments at these four work areas, traffic will be diverted to the widened areas and the existing bridge removed, new bridge piles installed and remaining abutment constructed.

The following options could be considered for construction to allow placement of concrete and placement and compaction of backfill in the “dry

- A) Place a cofferdam around each work area to divert the river / surface water flow and restrict water seepage into the excavation. Install a well-point dewatering system and excavate in open cut to construct the pile cap. A temporary liner could be placed along the edge of the river and adjacent floodplain area to help control water infiltration into the excavation. Permits from the regulating authorities to allow for temporary diversion of the river and construction within the existing watercourse would need to be provided prior to construction.
- B) Install closed steel sheet-piling around each of the four work areas (northeast, northwest, southeast, and southwest pile caps) to form a cofferdam cutoff at each location. Sheet piles may be driven or vibrated to the design tip depths; the required tip depth will be dependent on the elevation of the base of the pile cap in relation to the river water level. Excavation within the sheet pile cofferdam will need to allow for installation of lateral support struts or reinforcement and allow control of water infiltration to mitigate “piping” of the underlying soils at the base of the excavation if design tip depths are not reached due to the cobbles and boulders present in the soils. Depending on the sheet pile toe depth achieved, water inflow may be controlled by continuous pumping using sump pumps located at the base of the excavation or well-points installed within the sheet pile box. It is expected that there could be difficulties in achieving a complete cut-off to water inflow along the side of the excavation adjacent to the temporary roadway protection system and special procedures would be required in order to minimize disturbance to the founding soils under the roadway. Prior to driving sheet-piling, the limits of any previous existing bridge temporary shoring should be confirmed.

Alternatively, the excavation could be carried out and the piles installed within the cofferdam maintaining a water level consistent or slightly lower than the river water level. A tremie plug would then be placed at the base of the excavation to allow dewatering and construction of the pile cap in “the dry”.

The detailed design of the sheet pile wall system is dependent on the final elevation of the pile cap. The higher the elevation of the pile cap, the less effort is required for dewatering. The detailed design of the sheet pile system should be performed by a specialist firm.

Temporary open cut slopes through the fill and native materials above the water level or in areas that have been dewatered may be made with side slopes no steeper than 1 horizontal to 1 vertical

(1H:1V). Excavations must be carried out in accordance with the latest edition of the Occupational Health and Safety Act (OHSA) for Construction Activities.

Excavation support for the existing roadway and bridge wing walls will be required at this site. Where required, 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 the Special Provision.

5.6 Obstructions During Pile Driving

It is recommended that a Non-Standard Special Provision (NSSP) be included in the Contract Documents to warn the contractor of the presence of cobbles and boulders within the overburden soils which will likely affect the installation (specifically pre-augering) of steel H-piles for abutment construction. A sample NSSP is provided in Appendix C.

5.7 Vibration Monitoring During Pile Installation

Vibration monitoring should be carried out during pile installation to ensure that the vibration levels at the existing overpass structure are maintained below tolerable levels. An NSSP should be included in the Contract Document for this purpose. A sample NSSP is provided in Appendix C.

A maximum peak particle velocity (PPV) of 50 mm/s is recommended at the existing bridge structure. The piles furthest from the existing 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.

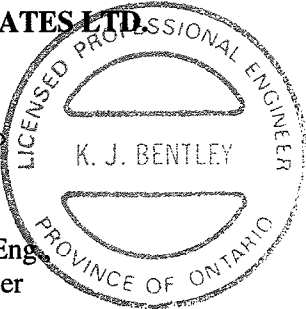
6.0 CLOSURE

This report was prepared by Mr. Kevin Bentley, P.Eng., a geotechnical engineer and the technical aspects were reviewed by Ms. Anne Poschmann, P.Eng., a Principal and senior geotechnical engineer. Messrs. Fintan J. Heffernan, P.Eng. and Jorge Costa, P.Eng., both Designated MTO Contacts for Golder, conducted an independent quality control review of the report.

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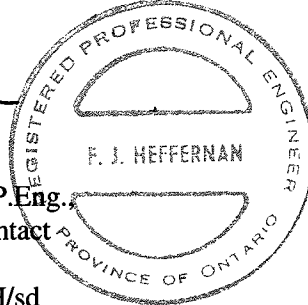
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KJB/ASP/JMAC/FJH/sd

TABLE 1
EVALUATION OF FOUNDATION ALTERNATIVES
CREDIT RIVER BRIDGE REPLACEMENT – NORTH BRANCH
HIGHWAY 10, TOWN OF CALEDON

<i>Footing Option</i>	<i>Option No.</i>	<i>Advantages</i>	<i>Disadvantages</i>	<i>Relative Costs</i>	<i>Risks/Consequences</i>
Steel H Piles driven into hard clayey silt (approximate pile tip Elevation 375 m)	1	Increased capacity over piles terminated at higher elevation (Options 2 and 3).	Longer pile lengths than Option 2 and 4; Hard driving likely to be encountered through gravelly deposits and piles may “hang-up” on boulders; May require pre-augering to minimize potential for adverse impact on existing spread footings.	Lower relative costs than piles driven to greater depths; Higher relative cost than piles terminated at shallower depths; however, less piles required due to higher capacities; Costs increase if pre-augering is required.	Potential for piles to “hang up” on cobbles and boulders or to be deflected away from vertical during driving ; Risk of disturbing existing spread footing with increased pile driving.
Steel H Piles driven into very dense sand and silt (approximate pile tip Elevation 385 m)	2	Higher capacity than pipe piles (Option 4).	Longer pile lengths than Option 4; Hard driving likely to be encountered through gravelly deposits and piles may “hang-up” on boulders; May require pre-augering to minimize potential for adverse impact on existing spread footing.	Lower relative costs than piles driven to greater depths; Costs increase if pre-augering is required.	Potential for piles to “hang up” on cobbles and boulders or to be deflected away from vertical during driving ; Risk of disturbing existing spread footing with increased pile driving.
Steel H Piles driven into very dense sand and gravel (approximate pile tip Elevation 369 m)	3	Increased capacity over piles terminated at higher elevation (Options 1 and 2).	Longest pile lengths compared to all options; Hard driving likely to be encountered through gravelly deposits and piles may “hang-up” on boulders; May require pre-augering to minimize potential for adverse impact on existing spread footing.	Increased construction costs due to longer lengths and installation costs; Costs increase if pre-augering is required.	Potential for piles to “hang up” on cobbles and boulders or to be deflected away from vertical during driving ; Risk of disturbing existing spread footing with increased pile driving.

Steel Pipe Piles driven into very dense sandy gravel/gravel (approximate pile tip Elevation 392 m to 394 m)	4	Relatively shallow pile tip depths minimize the potential for encountering hard driving; less potential for piles to “hang up” on cobbles and boulders.	Lower capacity relative to deeper H-Piles; PDA testing (accompanied by Hiley method) must be performed to verify design capacities are achieved.	Lower relative costs than piles driven to greater depths; Increased PDA testing costs.	Chance of pipe pile penetrating through founding elevation and into finer grained, slightly less dense material; Potential for heave/disturbance of existing spread footings since pipe piles are displacement piles.
Spread Footings	NF		Extensive measures for excavation and groundwater control required to minimize potential disturbance to existing foundation and differential settlement; Increased environmental impact on designated environmentally sensitive area.	Excavation and groundwater control cost higher than for pile cap construction.	Potential for difficulties in achieving adequate groundwater control/dewatering; Potential for loosening of the founding soils under the existing footing if groundwater control measures are inadequate; Disturbance/loosening of founding soils requiring further subexcavation could result in disturbance to the existing footings.
Caissons	NF		May encounter difficulties in advancing caissons through deposits containing cobbles and boulders; Caissons will need to be extended into hard clayey silt at depths greater than 25 m below existing ground surface in order to minimize requirements for groundwater control to ensure integrity of founding soils; Temporary liners required through the sands and silts and sealed into the clayey silt.	Higher costs relative to driven piles due to costs associated with liners and inspection.	Difficulty may be encountered in drilling and extending liner through cobbles and boulders; Difficulty may be encountered in sealing off groundwater inflow due to granular material interlayers; downhole inspection may not be possible.

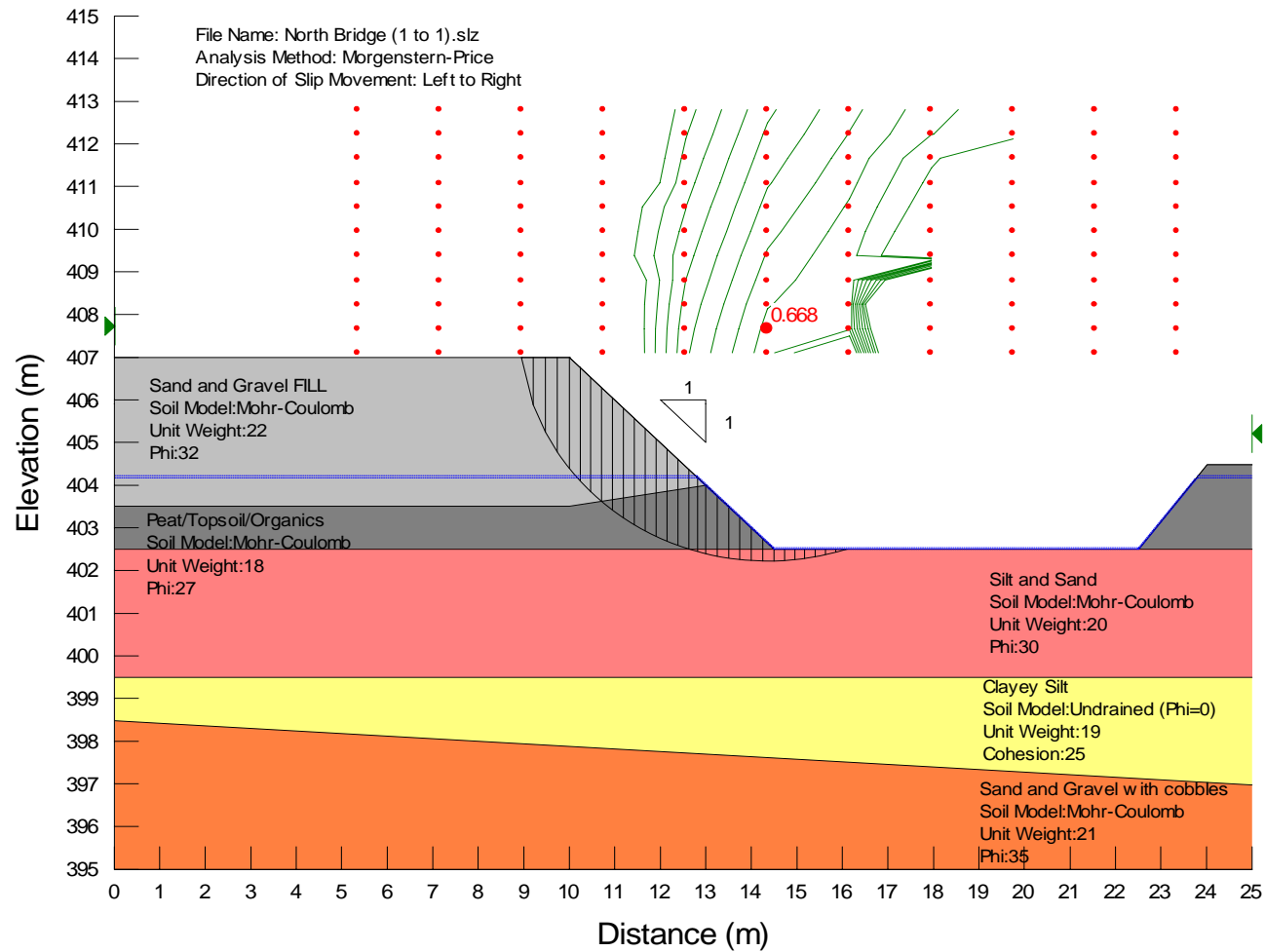
NF: Indicates that the founding option is considered not feasible.

Prepared by: KJB

Checked by: JMAC / FJH

TEMPORARY EXCAVATION CONFIGURATION

FIGURE 1



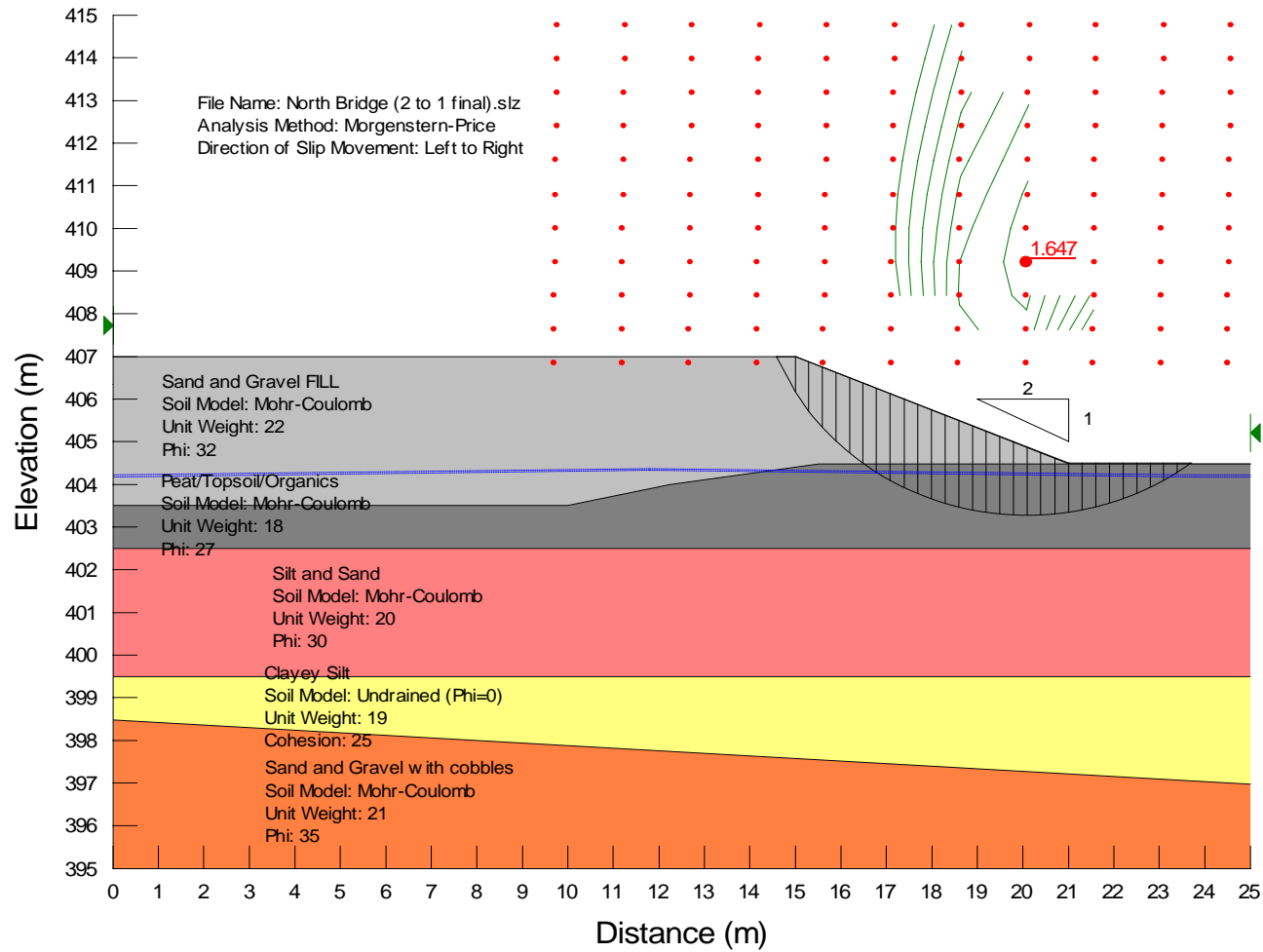
Date: November 2006
Project: 03-1111-023

Golder Associates

Drawn: KG
Checked: KJB

FINAL EMBANKMENT CONFIGURATION

FIGURE 2



Date: November 2006
 Project: 03-1111-023

Golder Associates

Drawn: KG
 Checked: KJB

LIST OF ABBREVIATIONS

The abbreviations commonly employed on Records of Boreholes, on figures and in the text of the report are as follows:

I. SAMPLE TYPE

AS	Auger sample
BS	Block sample
CS	Chunk sample
SS	Split-spoon
DS	Denison type sample
FS	Foil sample
RC	Rock core
SC	Soil core
ST	Slotted tube
TO	Thin-walled, open
TP	Thin-walled, piston
WS	Wash sample

III. SOIL DESCRIPTION

(a) Cohesionless Soils

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

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

The number of blows by a 63.5 kg. (140 lb.) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) drive open sampler for a distance of 300 mm (12 in.)

Consistency

	<u>kPa</u>	<u>psf</u>
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

(b) Cohesive Soils

c_u, s_u

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 cm² 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 ¹
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement ¹
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)
γ	unit weight

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

LIST OF SYMBOLS

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

I. General

π	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

γ	shear strain
Δ	change in, e.g. in stress: $\Delta \sigma$
ϵ	linear strain
ϵ_v	volumetric strain
η	coefficient of viscosity
ν	Poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma - u$)
σ'_{vo}	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stress (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ	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
γ'	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 ρ . Unit weight symbol is γ 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
σ'_p	pre-consolidation pressure
OCR	over-consolidation ratio $= \sigma'_p / \sigma'_{vo}$

(d) Shear Strength

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

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

[illegible]

Continued Next Page

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

MIS-MTO 001 031111023AAGDR.GPJ GAL-MISS.GDT 2/28/07


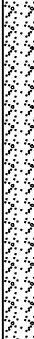
2 OF 3 **METRIC**

CHECKED BY KJB/JMAC





Continued Next Page

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

MIS-MTO 001 031111023AAGDR.GPJ GAL-MISS.GDT 2/28/07

PROJECT <u>03-1111-023</u>		RECORD OF BOREHOLE No NB1				3 OF 3 METRIC													
W.P. <u>27-97-00</u>		LOCATION <u>N 4863212.6 ; E 259542.4</u>				ORIGINATED BY <u>GD</u>													
DIST <u> </u> HWY <u>10</u>		BOREHOLE TYPE <u>POWER AUGERING USING 108 mm O.D. SOLID STEM AUGERS AND CASING</u>				COMPILED BY <u>KG</u>													
DATUM <u>Geodetic</u>		DATE <u>October 1, 2003</u>				CHECKED BY <u>KJB/JMAC</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 γ kN/m ³	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 ---						<div style="display: flex; justify-content: space-between;"> 20 40 60 80 100 20 40 60 80 100 </div> <div style="display: flex; justify-content: space-between;"> ○ UNCONFINED + FIELD VANE </div> <div style="display: flex; justify-content: space-between;"> ● QUICK TRIAXIAL × REMOULDED </div>					<div style="display: flex; justify-content: space-between;"> 10 20 30 </div>							
371.4	CLAYEY SILT, trace sand, contains silt seams Hard Grey Moist to wet		20	SS	65														
33.5	SAND and GRAVEL, trace silt Very dense Brown Wet		21	SS	101/0.15														
367.9			22	SS	105														
37.0	End of Borehole																		
	Note: 1. NW casing was used to advance the borehole below a depth of 6.1 m below ground surface 2. Water level in piezometer at 0.75m depth below ground surface (Elev. 404.2 m) on January 7, 2004																		

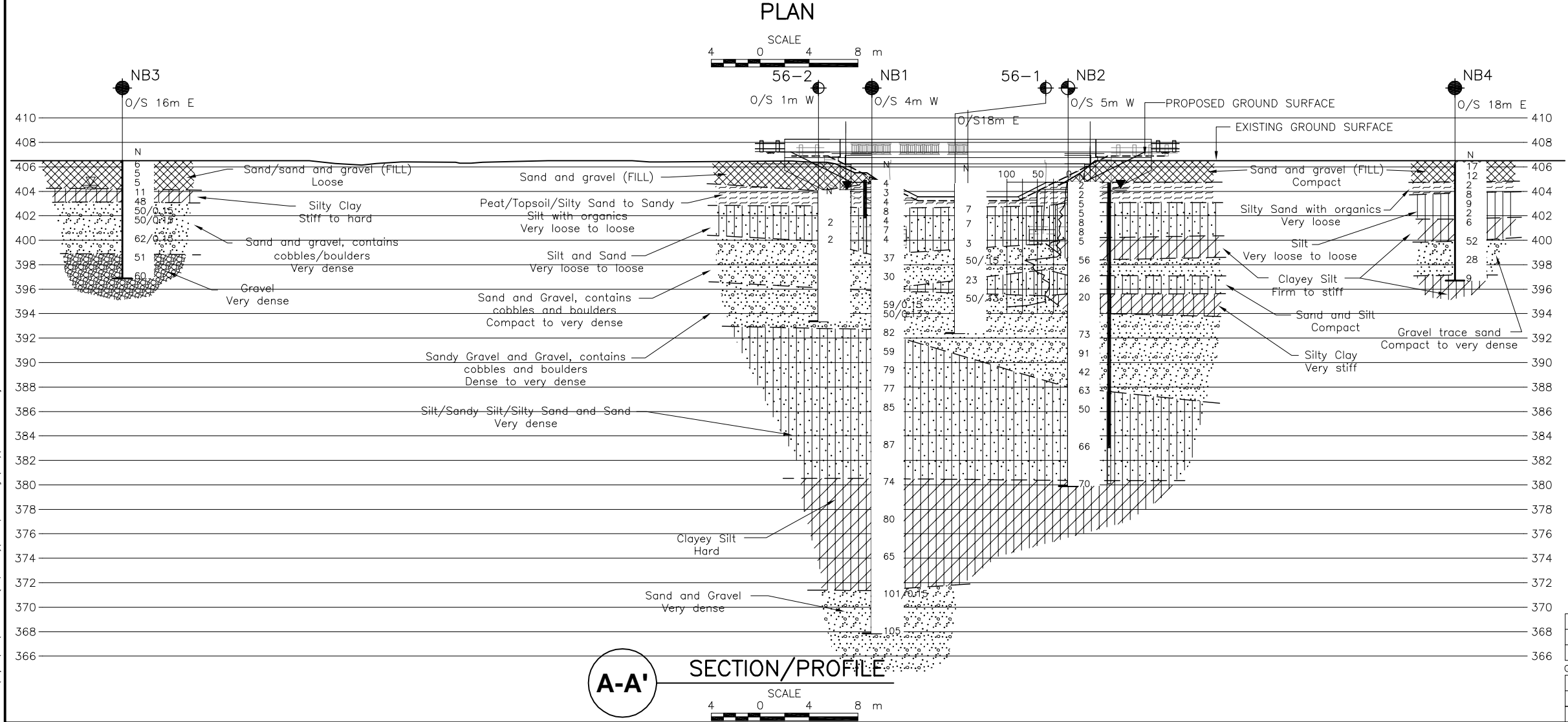
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PROJECT 03-1111-023			RECORD OF BOREHOLE No NB2			2 OF 2 METRIC											
W.P. 27-97-00			LOCATION N 4863223.4 ; E 259530.5			ORIGINATED BY GD											
DIST _____ HWY 10			BOREHOLE TYPE POWER AUGERING USING 108 mm I.D. HOLLOW STEM AUGERS AND CASING			COMPILED BY KG											
DATUM Geodetic			DATE December 8, 2003			CHECKED BY KJB/JMAC											
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa					WATER CONTENT (%)			γ kN/m ³	GR SA SI CL
							20 40 60 80 100	20 40 60 80 100	W _p	W	W _L	10 20 30					
	--- CONTINUED FROM PREVIOUS PAGE ---																
388.0	Sandy GRAVEL, trace to some silt, trace clay, occasional silty sand with clay seams, contains cobbles/boulders Very dense to dense Grey Wet		13	SS	42		389										70 23 6 1
16.8	SILT, some sand, trace clay Very dense to dense Grey Wet		14	SS	63		388										
							387										
			15	SS	50		386										0 13 82 5
							385										
							384										
383.4	Silty SAND, trace clay, occasional silty clay seams Very dense Grey Wet		16	SS	66		383										
							382										
							381										
380.4	CLAYEY SILT, trace sand, occasional silty sand seams Hard Grey Wet		17	SS	70		380										
24.8	End of Borehole																
Notes:																	
1. Water level encountered at 1.52m depth below ground surface during drilling.																	
2. Water level in piezometer at 0.5m depth below ground surface (Elev. 404.2 m) on January 7, 2004.																	
3. Cored through boulder (approximately 0.3m diameter) at depth of 10.7m.																	
4. NW casing was used to advance the borehole below a depth of 11.6m below ground surface.																	
5. Dynamic Cone Penetration Test performed 1.5m North of Borehole location.																	

PROJECT		03-1111-023		RECORD OF BOREHOLE No NB3		1 OF 1 METRIC												
W.P.		27-97-00		LOCATION		N 4863182.9 ; E 259599.7												
DIST		HWY 10		BOREHOLE TYPE		POWER AUGERING USING 108 mm I.D. HOLLOW STEM AUGERS												
DATUM		Geodetic		DATE		December 04, 2003												
				ORIGINATED BY		GD												
				COMPILED BY		JDR												
				CHECKED BY		KJB/JMAC												
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV	DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa					WATER CONTENT (%)			γ	GR SA SI CL
								20 40 60 80 100	20 40 60 80 100	W _p	W	W _L	10 20 30	kN/m ³				
406.5	0.0	GROUND SURFACE		1	SS	6		406										
		Sand and Gravel, trace silt (FILL) Loose Brown Moist																
405.7	0.8	Sand, trace silt and gravel (FILL) Loose Brown Moist		2	SS	5		405										
				3	SS	5												
404.2	2.3	Silty CLAY, some to trace sand Stiff to hard Brown Wet		4	SS	11		404										
				5	SS	48		403										
403.1	3.4	SAND and GRAVEL, trace silt, occasional silty clay seams, contains cobbles/boulders Dense to very dense Brown Wet		6	SS	50/0.15		402										
				7	SS	50/0.15		401										
				8	SS	62/0.15		400										
398.9	7.6	GRAVEL, trace sand and silt Very dense Grey Wet		9	SS	51		399										
								398										
396.9	9.6	End of Borehole		10	SS	60		397										
		Notes: 1. Water level at 2.0m depth (Elev. 404.5 m) upon completion of drilling. 2. Borehole caved below ground surface to 7.0m depth.																

PROJECT 03-1111-023			RECORD OF BOREHOLE No NB4			1 OF 1 METRIC		
W.P. 27-97-00			LOCATION N 4863262.1 ; E 259524.9			ORIGINATED BY GD		
DIST _____ HWY 10			BOREHOLE TYPE POWER AUGERING USING 108 mm I.D. HOLLOW STEM AUGERS			COMPILED BY KG		
DATUM Geodetic			DATE December 05, 2003			CHECKED BY KJB/JMAC		
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100 PLASTIC LIMIT W _p NATURAL MOISTURE CONTENT W LIQUID LIMIT W _L WATER CONTENT (%)
406.3	GROUND SURFACE							
0.0	Sand and Gravel, trace silt (FILL) Compact Brown Moist		1	SS	17		406	
			2	SS	12		405	
404.8								
1.5	Silty SAND with organics, occasional clayey silt seams Very loose Dark brown Moist		3	SS	2		404	
404.0								
2.4	SAND Loose Dark grey Wet		4	SS	8		403	
	SILT, some sand, trace to some clay, occasional sand seams Very loose to loose Brown Wet		5	SS	9		402	
			6	SS	2			
401.7								
4.6	CLAYEY SILT, trace to some sand Firm Greyish-brown Wet		7	SS	6		401	
399.9			8	SS	52		400	
6.4	GRAVEL, trace sand and silt Very dense to compact Grey Wet						399	
			9	SS	28		398	
397.2							397	
9.1	CLAYEY SILT, some sand, trace gravel, occasional silty sand seams Stiff Brown Wet		10	SS	9			
396.7								
9.6	End of Borehole							
Notes: 1. Borehole caved to 1.8m upon completion of drilling								

0 12 81 7



NO.	DATE	BY	REVISION	
Geocres No. 40P16-19				
HWY. 10		PROJECT NO. 03-1111-023		DIST.
SUBM'D. KJB	CHKD. KJB	DATE: 2/28/2007		SITE: 24-09
DRAWN: JDR/MSM		CHKD. FJH	APPD.	DWG. 1

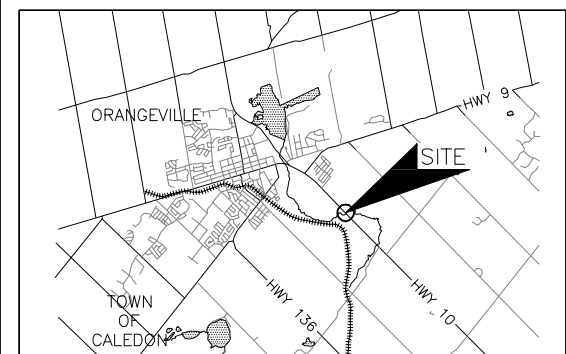
CONT No.
WP No. 27-97-00

HIGHWAY 10
CREDIT RIVER BRIDGE – NORTH BRANCH
SOIL STRATA

SHEET








Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



KEY PLAN

SCALE
0 2 km

LEGEND

- | | |
|---|--|
|  | Borehole — Racey, MacCallum and Associates Ltd.,
1956 |
|  | Borehole and Cone |
|  | Seal |
|  | Piezometer |
| N | Standard Penetration Test Value |
| 16 | Blows/0.3m unless otherwise stated
(Std. Pen. Test, 475 j/blow) |
|  | WL in piezometer, measured on January 07, 2004 |

No.	ELEVATION	CO-ORDINATES	
		NORTHING	EASTING
NB2	404.7	4863223.4	259530.5
56-1	404.6	4863238.3	259548.4

NOTES

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

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

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

REFERENCE

Base plans provided in digital format by Morrison Hershfield Ltd., drawing file no. 1034006 NB-S01.dwg, received October 16, 2006.

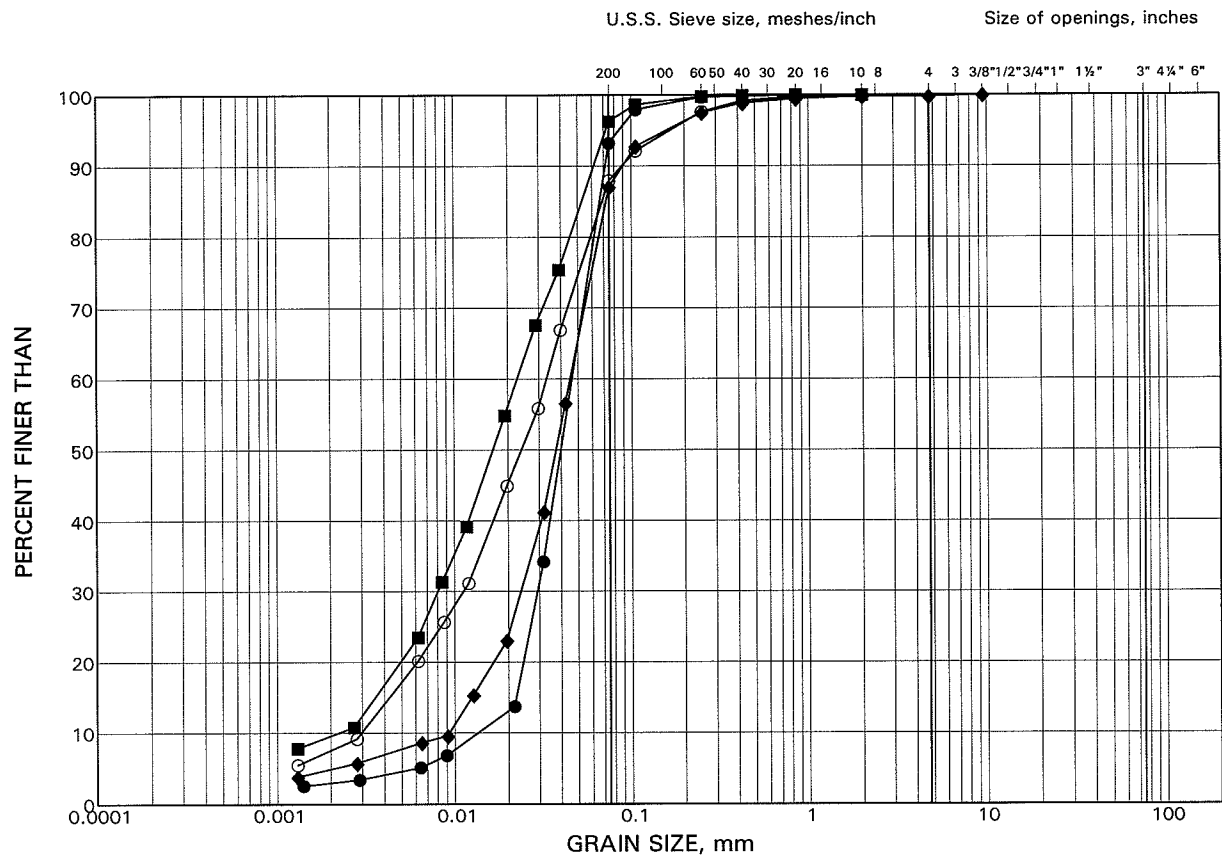
NO.	DATE	BY	REVISION	
Geocres No. 40P16-19				
HWY. 10		PROJECT NO. 03-1111-023		DIST.
SUBM'D. KJB		CHKD. KJB	DATE: 2/28/2007	SITE: 24-09
DRAWN: JDR/MSM		CHKD. FJH	APPD.	DWG. 2

APPENDIX A
LABORATORY TEST DATA

GRAIN SIZE DISTRIBUTION

Silt

FIGURE A1



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

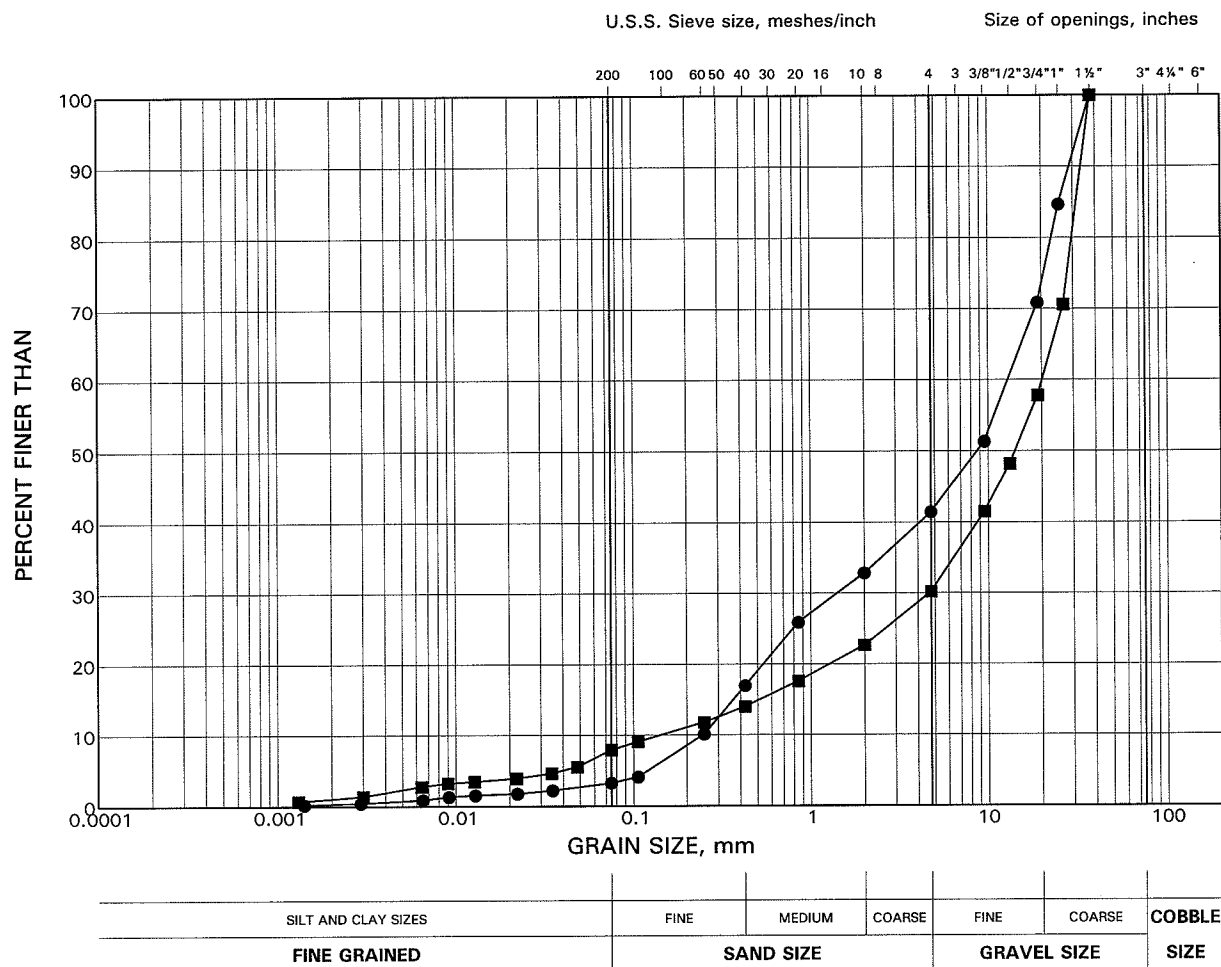
LEGEND

SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)
●	NB1	15	16.8-17.2
■	NB2	5	3.0-3.5
◆	NB2	15	18.3-18.7
○	NB4	5	3.0-3.5

GRAIN SIZE DISTRIBUTION

Sandy Gravel, Sand and Gravel

FIGURE A2



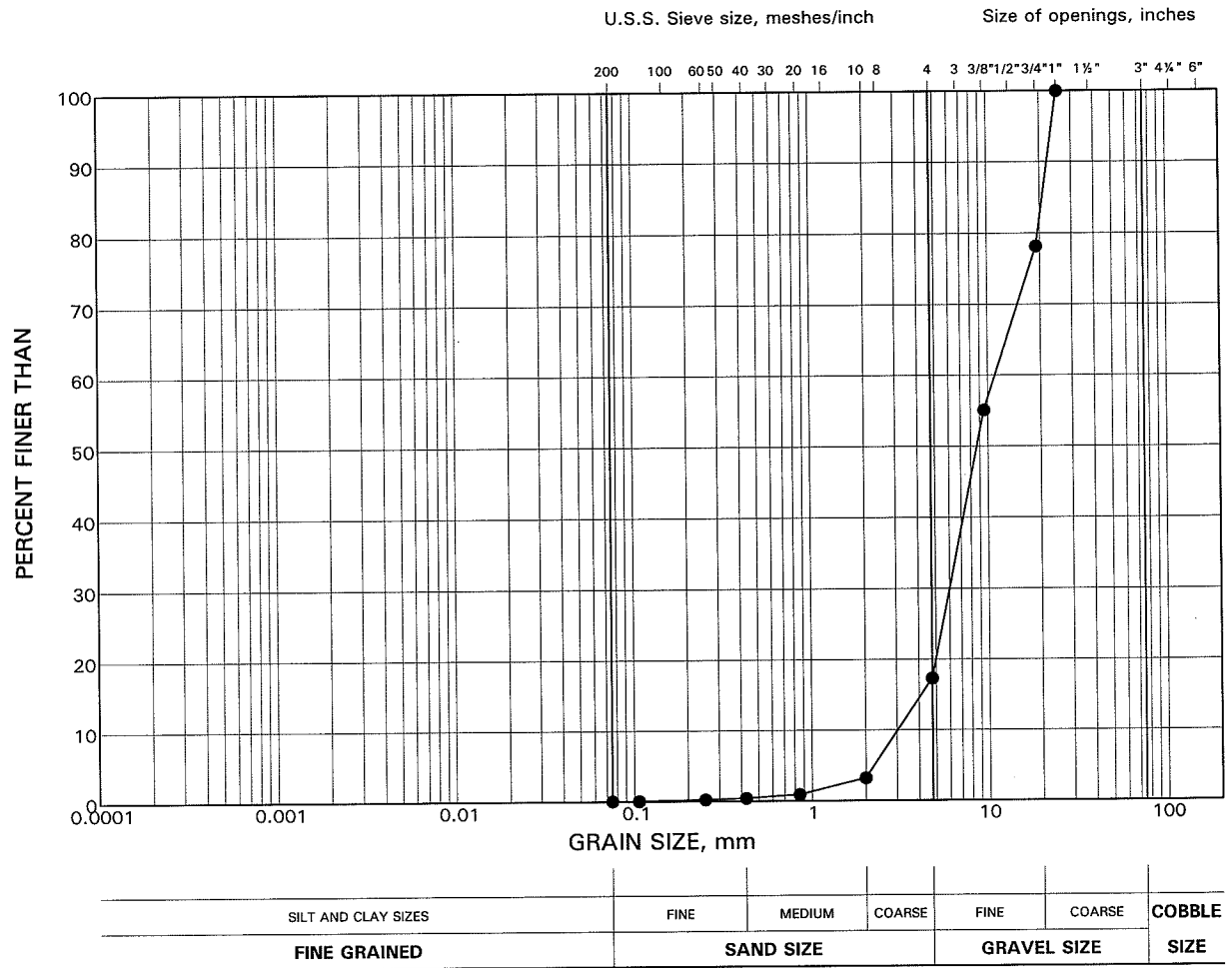
LEGEND

SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)
●	NB1	9	7.6-8.1
■	NB2	13	15.2-15.7

GRAIN SIZE DISTRIBUTION

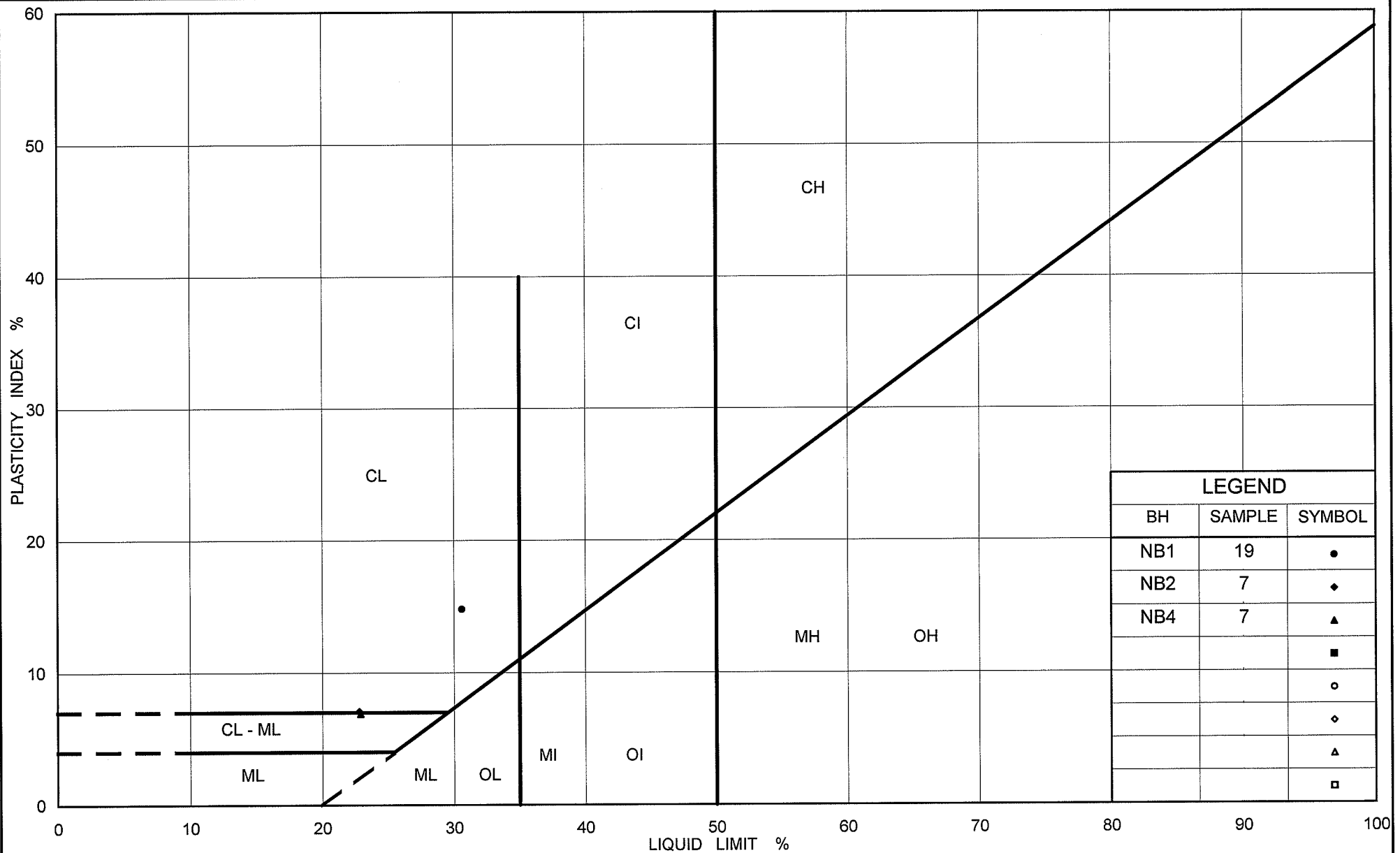
Gravel

FIGURE A3



LEGEND

SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)
•	NB1	11	10.7-10.8



Ministry of Transportation

Ontario

PLASTICITY CHART

Clayey Silt

FIG No. A4

Project No. 03-1111-023

Checked By *KTB*

APPENDIX B

RECORD OF BOREHOLES (56-1, 56-2)
PREVIOUS INVESTIGATION

Encl. No. 2.

Order No. 5500-629/1988 RACEY, MACCALLUM AND ASSOCIATES
LIMITED

Hole Begun _____

Foundation Engineering Division

Driller _____

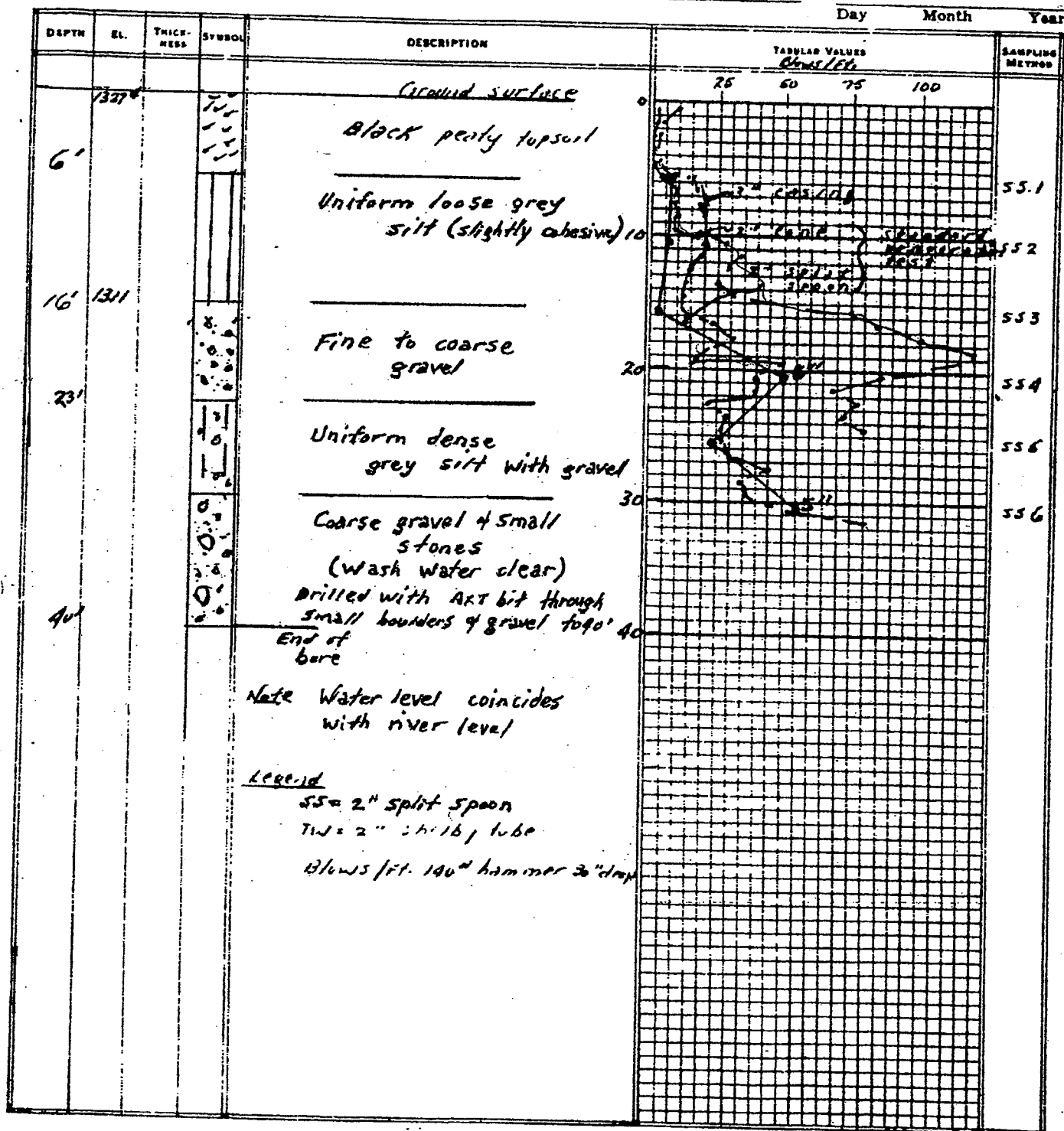
Hole Ended _____

Engineering Data Sheet for Borehole: 56-1

Helper _____

Job Name: Credit River Bridge Hwy No 10 Sta 555+70Job Located: Approx. 2 mi. South Orangethorpe OntHole Located: See Encl. No. 1

Checked by _____

Hole Elevation: 1327.4 Datum: Geodetic

Order No.: ~~5500-629/1380~~ RACEY, MACCALLUM AND ASSOCIATES
LIMITED

encl No 9

1.8.

Driller

Hole Begun _____ Foundation Engineering Division

Hole Ended _____ Engineering Data Sheet for Borehole: 56-2

Helper

Job Name: Credit River Bridge Hwy No 10 Sta 555 ± 70

Job Located: Approx 2 mi. South Orangeville Ont.

Checked by

Hole Located: See Encl. No. 1

Hole Elevation: 1325 ⁶ Datum: Geodetic

Day Month Year

DEPTH	EL.	THICK- NESS	SYMBOL	DESCRIPTION	TABULAR VALUES Blows/ft	SAMPLING METHOD
	1325 ⁶			Ground Surface	25 50 75 100	
				Black topsoil		
4				Loose medium sand		T.W. 1
6				Loose slightly cohesive silt		55.2
13 ⁵	1312			Dense gravel & boulders and coarse sand (sampling impossible)		T.W. 3
				some silt in evidence at 23 ft.		55.4
30				Drill with N.T. bit through dense gravel & boulders to 35'		133
35	1289			End of Bore		159
				Note: Water table coincides with river level		185
				Legend		186
				SS = 2" Split Spoon		120
				TW = 2" Shelby tube		120
				Blows/ft. 140# hammer 30" drop		145
						146
						205
						96
						160
						325
						815
						265
						230
						390
						Casing
						66W3

APPENDIX C
NON-STANDARD SPECIAL PROVISIONS

DRIVING SHOES - Item No.

Non-Standard Special Provision

Scope

As part of the work under the above tender item, the Contractor shall supply Titus “Standard” design driving shoes on HP 310 x 110 Piles for the Highway 10 Bridge Replacement over the Credit River – North Branch.

References

OPSS 906 – Structural Steel
SP903S01

Materials

The driving shoes shall be of the following:

Product

Manufacturer

HPP-S-12

Titus Steel Company Ltd.
6767 Invader Cr.
Mississauga, ON
Tel (905) 564-2446

(Or approved equivalent)

Basis of Payment

Payment at the Contract Price for the above tender items shall be full compensation for all labour, equipment and material to do the work.

END OF SECTION

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

Non-Standard Special Provision

The native soils at the site contain water-bearing silts, sands and gravels containing cobbles and boulders. The soils will be susceptible to cave-in, sloughing and boiling. In addition, obstructions (such as debris, cobbles or boulders) should be anticipated within the existing Highway 10 embankment fill. Appropriate equipment and procedures will be required to penetrate cobbles/boulders/obstructions that are encountered during pile driving / installation and to avoid excessive vibrations to the existing bridge.

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

UNWATERING FOR STRUCTURE EXCAVATION / SHEETPILE INSTALLATION -
Item No.

Non-Standard Special Provision

Scope

The contractor shall be alerted that the soils at the Highway 10 Replacement Bridge over the Credit River – North Branch site consist of water-bearing sand and gravel, peat and topsoil, silts and sands containing cobbles and boulders. Pile cap / abutment construction below the groundwater and/or river water levels must be carried out in the dry. The excavation shall be kept stable during the work.

Basis of Payment

Payment at the contract price for the above tender item shall be full compensation for all labour, equipment and materials required to do the work.

END OF SECTION

VIBRATION MONITORING - Item No.

Non-Standard Special Provision

Scope

This non-standard special provision describes requirements for vibration monitoring during the piling installation works for the replacement of the existing Highway 10 structure over the Credit River – North Branch.

References

The subsurface conditions at the site are described in the following Foundation Investigation Report for W.P 27-97-00:

- Foundation Investigation Report, Credit River Bridge Replacement – North Branch, Highway 10 Widening from 1 km North of Regional Road 24 Northerly to Highway 9, Town of Caledon, Ontario, W.P. 27-97-00.

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 10 structure (North Bridge) over the Credit River – North Branch.
- 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 10 North Bridge structure, as close as possible to the piling works. The Contractor/QVE shall take readings on the existing structure during driving of each pile, starting with the pile furthest away from the existing Highway 10 bridge 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

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