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**REPORT ON**

**FOUNDATION INVESTIGATION AND DESIGN  
DETAIL DESIGN  
CREDIT RIVER BRIDGE WIDENING - SOUTH 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. 24-10**

Submitted to:

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GEOCRES NO: 40P16-22

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**PART A**

**FOUNDATION INVESTIGATION REPORT  
DETAIL DESIGN  
CREDIT RIVER BRIDGE WIDENING - SOUTH BRANCH  
HIGHWAY 10 WIDENING FROM 1 KM NORTH OF REGIONAL ROAD 24  
NORTHERLY TO HIGHWAY 9  
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W.P. 27-97-00**

**Golder Associates**

## 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 detailed 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 supplemental letter "Revision to Borehole Drilling Program", dated November 20, 2003. This report addresses the widening of the existing bridge crossing the South Branch of the Credit River (South Bridge) on Highway 10 as part of the project. The work was carried out in accordance with the Quality Control Plan for this project dated July 2003. A digital file of the site plan and bridge general arrangement drawing was provided to Golder by Morrison Hershfield in January 2004.

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

- "Foundation Investigation for a Culvert, at the Credit River, Highway 10 Crossing, approximately four miles north of Caledon, Ontario", GEOCREs No. 40P-16-02, prepared by Racey, MacCallum and Associates Limited, dated November 5, 1956;
- "MTO Preliminary Design Report", prepared by URS Cole, Sherman & Associates, dated July 2002.

## 2.0 SITE DESCRIPTION

The South Branch Credit River site is located at the north side of the intersection between Highway 10 and Highpoint Sideroad North (see key plan on Drawing 1). The existing bridge is a single span reinforced concrete rigid frame structure, with a span of about 12.2 m between abutments.

The bridge spans over the Credit River which flows in a east to west direction near the site. The terrain at the site is generally flat, with the exception of the existing raised bridge approach embankments and elevated existing highway grade that ranges from about Elevation 406 m to 407 m. The ground surface adjacent to the existing bridge embankment within the surrounding low-lying swampy area is at about Elevation 402 m.

There is a private entrance and driveway located at the southeast corner of the site and the intersection of Highway 10 and Highpoint Sideroad North is located at the southwest corner of

the bridge site. Vacant and flat low-lying land is present in the northeast and northwest areas of the site. Primary utility lines are located within the vicinity of the site.

Drainage channels are located along both sides of Highway 10 which discharge to the Credit River. On the south side of the bridge, drainage culverts run beneath an existing driveway on the east side and under Highpoint Sideroad North on the west side that drain stormwater into the Credit River.

### **3.0 INVESTIGATION PROCEDURES**

#### **3.1 Foundation Investigation**

The field work at the South Bridge site was carried out between September 23 and December 4, 2003, at which time four (4) boreholes, numbering SB1, SB2, SB3, and SB4 were advanced at the locations shown on Drawing 1. Two boreholes, numbered 56-H2 and 56-H3 were advanced as part of a previous investigation at the site carried out by Racey, MacCallum, and Associates Ltd. in 1956. The locations of these boreholes are also shown in plan on Drawing 1.

The current field investigation was carried out using track-mounted CME 55 drill rigs supplied and operated by two companies, namely Geo-Environmental Drilling Ltd. of Milton, Ontario in October 2003 and Groundwork Drilling Inc. of Etobicoke, Ontario in 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 intervals ranging from 0.75 m to 3.0 m in depth, using a 50 mm (O.D.) split-spoon sampler in accordance with Standard Penetration Test (SPT) procedures.

The boreholes were advanced to depths ranging from 8.1 m to 24.5 m below the existing ground surface. The groundwater conditions in the open boreholes were observed during the drilling operations, and piezometers were installed in select boreholes to permit more long-term monitoring of the groundwater levels at these locations. The piezometers consist of a 25 mm outside diameter solid PVC pipe, with a slotted screen sealed at a select 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 supervised by members of our engineering and technical staff, who located the boreholes, arranged for the clearance of underground service locations, supervised the drilling, sampling and in-situ testing operations, logged the boreholes, and examined and cared for the soil samples. The samples were identified in the field, placed in appropriate containers, labelled and transported to our Mississauga geotechnical laboratory where the samples underwent

further detailed 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 personnel 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 (Chapman and Putnam, "The Physiography of Southern Ontario", 3<sup>rd</sup> 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 as consisting 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. Furthermore, subsurface conditions will vary between and beyond the borehole locations. The inferred soil stratigraphy based on the results of the boreholes within the vicinity of the Credit River Bridge – South Branch location are shown on Drawing 1.

The surficial soils at the South Bridge 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 underlain by peat and organic silty sand. The fill, topsoil, peat, and organic silty sand is underlain by a sand to sand and gravel layer, which is typically underlain by a deposit of silt with clayey silt interlayers. These deposits are underlain by a layer of sand and gravel to gravel containing cobbles/boulders at some locations. These deposits are underlain by interlayered sandy silt, silt, and clayey silt which is underlain by a dense to very dense interlayered silt and sand, and sand and gravel containing cobbles/boulders. These deposits are underlain at depth by



very dense silt with hard clayey silt interlayers. 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 SB1, SB2, and SB4, which were advanced near the top or toe of the bridge approach embankment. The fill typically consisted of silty sand, sand, to sand and gravel. The surface of the fill ranged between Elevations 402.4 m and 407.4 m and the thickness varied from 0.9 m to 4.9 m.

Standard Penetration Testing (SPT) 'N' values recorded within the fill ranged between 4 and 26 blows per 0.3 m of penetration, with all but two (2) of the recorded SPT 'N' values in the range of 4 to 14, indicating a loose to compact relative density.

Natural water contents measured on samples of the fill ranged between 9 and 20 percent. A grain size distribution curve for a selected sample of the fill deposit located near the interface of the underlying peat layer is shown on Figure A1.

#### **4.2.2 Topsoil / Peat / Organic Silty Sand**

Topsoil or peat was encountered at the existing ground surface in Boreholes SB3, 56-H2, and 56-H3 and underlying the fill in Boreholes SB1, SB2 and SB4. In Boreholes SB3 and SB4, the topsoil and peat transitioned to an organic silty sand containing rootlets. The top of the topsoil/peat and organic silty sand deposit ranged between Elevation 401.5 m to 403.4 m, with the thickness varying from 0.8 m to 3.7 m.

Standard Penetration Testing (SPT) 'N' values recorded within the topsoil and peat layers ranged from 2 blows to 10 blows per 0.3 m of penetration, indicating a very soft to stiff consistency. SPT 'N' values recorded within the organic silty sand were similarly between 2 and 4 blows per 0.3 m of penetration, indicating a very loose to loose relative density.

The natural water content measured on a samples of the topsoil/peat/organic silty sand varied between 24 and 48 percent.

#### **4.2.3 Upper Sand / Sand and Gravel**

Underlying the fill, peat, topsoil, and organic silty sand, a layer of brown sand to sand and gravel with trace silt was encountered in Boreholes SB1, SB2, SB3, SB4, and 56-H2. The top of the sand to sand and gravel layer was at a depth ranging between 2.0 m and 6.4 m. The top this layer ranged between Elevations 399.6 m and 401.0 m and the thickness varied from 0.7 m to 1.4 m.

Borehole SB2 encountered another sand deposit at a depth of 7.9 m (Elevation 397.7 m) and was terminated within the sand at a depth of 8.1 m (Elevation 397.5 m).

Standard Penetration Testing (SPT) 'N' values recorded within the upper sand to sand and gravel layer ranged between 6 blows and 9 blows per 0.3 m of penetration, indicating a loose state of packing.

The natural water content measured on two samples of the sand to sand and gravel layer was 15 percent.

#### **4.2.4 Upper Silt**

A layer of silt containing interlayers of predominantly clayey silt and sandy silt was encountered in Boreholes SB3, SB4, 56-H2 and 56-H3. The grey silt contained interlayers of gravel and cobbles in Borehole SB4. The top of this silt deposit was encountered at a depth ranging between 3.0 m and 3.7 m below existing ground surface. The top of this deposit ranged between Elevations 398.9 m and 399.7 m with the thickness varying from 2.0 m to 4.5 m.

Standard Penetration Testing (SPT) 'N' values recorded within the silt layer generally ranged from 4 to 18 blows per 0.3 m of penetration, indicating a very loose to compact relative density.

The natural water content measured on samples of the silt layer ranged from 22 to 23 percent, with one value of 11 percent measured within a sample containing gravel interlayers. A grain size distribution curve for a selected sample of the upper silt layer is shown on Figure A2.

#### **4.2.5 Sand and Gravel / Gravel**

Underlying the upper silt and upper sand deposits, a layer of sand and gravel to gravel was encountered at Boreholes SB2, SB3, 56-H2, and 56-H3. The sand and gravel/gravel layer typically contained cobbles and boulders, trace silt and clay. The top of the sand and gravel to gravel layer was encountered at a depth ranging from 5.0 m to 7.3 m below ground surface. The top of the sand and gravel to gravel layer ranged from Elevations 396.0 m to 399.5 m and the thickness varied between 2.0 m and 2.1 m.

Standard Penetration Testing (SPT) 'N' values recorded within the sand and gravel to gravel layer ranged between 11 blows and 80 blows per 0.3 m of penetration, indicating a compact to very dense state of packing. The higher blow counts may be attributed to cobbles and boulders encountered within the deposit.

The natural water content measured on samples of the sand and gravel to gravel layer ranged between 6 and 19 percent.

#### **4.2.6 Clayey Silt**

A deposit of brown clayey silt with trace sand was encountered below the upper sand layer in Borehole SB1. The top of this clayey silt deposit was 8.8 m below ground surface (Elevation 399.8 m) and drilling was terminated within this deposit at a depth of 8.8 m (Elevation 398.5). A Dynamic Cone Penetration Test (DCPT) was carried out from a depth of 8.8 m to 9.9 m (Elevation 397.5 m) at which depth effective refusal to advance the cone was achieved.

A Standard Penetration Test (SPT) 'N' value recorded within the clayey silt deposit was 3 blows per 0.3 m of penetration, indicating a very soft consistency. The DCPT values ranged from 34 blows per 0.3 m of penetration to 150 blows / 0.2 m of penetration of the cone tip. The increasing number of blows with depth from the DCPT results suggest the clayey silt deposit transitions to a very stiff to hard consistency.

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

#### **4.2.7 Interlayered Sandy Silt / Silt / Clayey Silt**

Interlayered sandy silt, silt, and clayey silt was encountered in Boreholes SB3, SB4, 56-H2, and 56-H3. The grey to brown interlayered silty deposit typically contained trace to some sand and gravel with cobbles/boulders near the interface with the overlying gravel layer. The top of this interlayered silty deposit was encountered at 7.0 to 9.6 m below ground surface. The top of this deposit ranged between Elevations 393.9 m and 394.9 m and the thickness was 6.1 m and 8.2 m for Boreholes SB4 and SB3 respectively. Boreholes 56-H2 and 56-H3 were terminated within the interlayered silty deposit at depths of 12.8 m (Elevation 389.6 m) and 12.2 m (Elevation 391.2 m) recording thicknesses of 5.8 m and 3.4 m respectively.

Standard Penetration Testing (SPT) 'N' values recorded within the interlayered silty deposit ranged between 10 blows and 53 blows per 0.3 m of penetration. SPT measured 'N' values within the sandy silt and silt interlayers ranged between 10 and 53 blows, indicating a compact to very dense state of packing. SPT measured 'N' values within the clayey silt interlayers ranged between 19 and 31 blows, indicating a very stiff to hard consistency.

The natural water content measured on samples of the sandy silt, silt, and clayey silt interlayers varied between 19 and 25 percent.

The results of Atterberg limits testing carried out on a sample of the clayey silt interlayer are illustrated on the plasticity chart on Figure A5 in Appendix A. The test results are summarized in the following table.

<i>Borehole</i>	<i>Sample</i>	<i>Elevation (m)</i>	<i>Liquid Limit (%)</i>	<i>Plastic Limit (%)</i>	<i>Plasticity Index (%)</i>
SB4	10	392.8-393.3	24	16	8

The test results on this sample within the clayey silt interlayer indicate it is of low plasticity.

#### **4.2.8 Interlayered Silt and Sand / Sand and Gravel**

A deposit of interlayered silt and sand to sand and gravel was encountered in the two deep boreholes, Boreholes SB3 and SB4, put down during the current investigation. The interlayers of brown silt and sand typically contained variable amounts of gravel, cobbles and boulders. The sand and gravel interlayers typically contained cobbles/boulders and trace silt.

The top of the interlayered silt and sand to sand and gravel deposit was at a depth of 13.7 m (Elevation 388.7 m) and 15.2 m (Elevation 386.7 m) in Boreholes SB4 and SB3 respectively. The interlayered silt and sand to sand and gravel deposit was 3.1 m and 6.1 m thick in Boreholes SB4 and SB3 respectively.

Standard Penetration Testing (SPT) 'N' values recorded within the interlayered silt and sand to sand and gravel typically ranged between 75 blows and over 100 blows per 0.3 m of penetration, indicating a very dense state of packing. One SPT 'N' value of 30 blows per 0.3 m of penetration was measured within an interlayer of silt and sand in Borehole SB4.

The natural water content measured on samples of the interlayered silt and sand to sand and gravel typically ranged between 8 and 12 percent, with one value measured at 23 percent. A grain size distribution curve for a selected sample of an interlayer of sand and gravel is shown on Figure A3.

#### **4.2.9 Lower Silt / Clayey Silt**

Below the interlayered silt and sand to sand and gravel, a deposit of interlayered silt and clayey silt was encountered in Boreholes SB3 and SB4. The grey silt and clayey silt deposit typically contained trace to some sand and contained seams of gravel and sand. The top of this silt and clayey silt layer was at 21.3 m and 16.8 m below ground surface in Boreholes SB3 and SB4 respectively. The top of this deposit was at Elevations 380.6 m and 385.6 m for Boreholes SB3 and SB4 respectively. Boreholes SB3 and SB4 were terminated within the silt deposit at depths of 24.5 m (Elevation 377.4 m) and 21.8 m (Elevation 380.6 m) respectively.

Standard Penetration Testing (SPT) 'N' values recorded within the lower silt/clayey silt layer ranged from 89 blows per 0.3 m of penetration to over 100 blows per 0.1 m of penetration, indicating a very dense to hard consistency.

The natural water content measured on samples of the lower silt to clayey silt deposits varied between 17 and 24 percent. A grain size distribution curve for a selected sample of the clayey silt is shown on Figure A4.

#### 4.2.10 Groundwater Conditions

Water levels were noted within the boreholes during and after the drilling operations. Piezometers were installed in Boreholes SB3 and SB4 to permit monitoring of water levels. Both piezometers were sealed within the lower interlayered silt/clayey silt deposit; the piezometer in Borehole SB3 was sealed in the interlayered silt and sand to sand and gravel deposit. The piezometer in Borehole SB4 was sealed within the lower silt and clayey silt 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 in the table below.

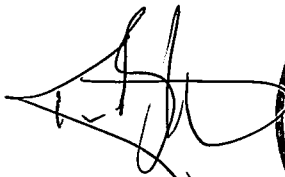
<i>Borehole</i>	<i>Ground Surface Elevation (m)</i>	<i>Water Level Depth (m)</i>	<i>Water Level Elevation (m)</i>	<i>Date</i>
SB3	401.9	0.20	401.7	January 7, 2004
SB4	402.4	3.75	398.7	January 7, 2004

The water level in Borehole SB3 was found to be at the same level as the Credit River in the vicinity of the South Bridge structure, which was measured to be at Elevation 401.7 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. The water level measured in the piezometer in Borehole SB4 may reflect a slower response time within the finer grained deposits or the level may not be stabilized. It should also be noted that groundwater levels in the area are subject to seasonal fluctuations.

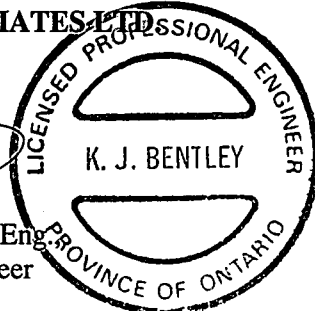
### 4.3 Closure

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 Mr. Fintan J. Heffernan, P.Eng, a Designated MTO Contact for Golder.

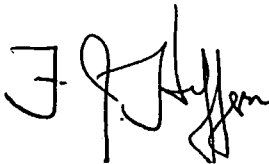
**GOLDER ASSOCIATES LTD.**



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KJBASP/FJH/al

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MINISTRY OF TRANSPORTATION, ONTARIO**

## **5.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS**

This section of the report provides foundation design recommendations for the proposed widening of the existing Credit River Bridge – South Branch as part of the Highway 10 widening project. 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 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 widened by about 5.4 m on the east side and 7.7 m on the west side to include the addition of a new southbound right-turning lane. The available drawings indicate that the current bridge is supported on timber piles which are illustrated to be about 4.6 m (15 feet) long. The top of the existing pile cap is indicated to be about Elevation 399.9 m. According to the Draft General Arrangement Drawing provided by Morrison Hershfield (January 2004), the top of the proposed new pile cap is to be at Elevation 400.5 m with a thickness of 1.2 m. The existing road grade at the South Bridge location ranges from about Elevation 407 m at the south side to about Elevation 406 m at the north side. The proposed bridge approach embankments may be up to 5 m in height above existing ground surface near the river bank.

### **5.1 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 and groundwater conditions, substantial site dewatering would be required to permit construction of shallow spread footings for the bridge widening. Such dewatering may however have an adverse effect on the existing pile supported foundations (i.e. negative skin friction inducing settlement). Considering the proximity to the Credit River, and the environmental sensitivity of the area, shallow foundations are not considered suitable for the bridge widening.

With respect to deep foundations, due to the presence of cobbles and boulders, and the potential difficulties associated with groundwater inflow and increased drill depths, caissons are not considered a suitable option. As a result of the variable thickness and apparent absence of the sand and gravel to gravel layer in some boreholes, pipe piles founded within the upper gravel layer are not considered a reliable option. Pipe piles driven to found within the underlying



interlayered deposits would have a relatively low carrying capacity and would likely be of variable lengths given the variability of these deposits. It is considered that the use of steel H-piles driven to found within the hard clayey silt or very dense interlayered sand and silt to sand and gravel for support of the abutments is the most feasible option from a geotechnical and foundation perspective. The pile cap should be maintained as high as possible in order to minimize the potential for disturbance of the soils surrounding the existing pile system during the excavation for the new pile installation and pile cap construction.

## **5.2 Steel H-Pile Foundations**

Based on the subsurface conditions, steel H-piles driven to found in the very dense interlayered silt and sand to sand and gravel or hard clayey silt encountered at or below Elevation 385 at both the north and south abutment locations may be used.

For design, a pile tip level at Elevation 380 m may be assumed for these piles. There should be provision made in the contract for dealing with varying pile lengths and possibly preaugering as there may be difficulties associated with the pile installation in the very dense sand and gravel, sandy silt, or silt deposits which contain cobbles and boulders.

The proposed pile cap elevation should be designed to be as high as possible to encourage ease of construction and construction in the dry, yet deep enough to provide sufficient frost and scour protection.

### **5.2.1 Axial Geotechnical Resistance**

For steel HP 310 x 110 piles driven to found within the very dense / hard deposits at about Elevation 380 m, the factored axial resistance at ULS may be taken as 1,000 kN at both abutment locations. The geotechnical resistance at SLS for 25 mm of settlement may be taken as 900 kN.

Alternatively, consideration could be given to friction piles driven to found within the compact to very dense sandy deposits at Elevation 388 m at the south abutment and Elevation 386 m at the north abutment. For these design tip levels, the factored axial resistance at ULS may be taken as 225 kN; the SLS resistance for 25 mm of settlement may be taken as 150 kN. Due to the anticipated loading on the piles and corresponding high number of piles needed, this option is not considered optimal and will not be discussed further herein. Further details for this option however can be provided upon request.

The piles founded at about Elevation 380 m will have to penetrate some distance into the hard to very dense interlayered deposits in order to develop the aforementioned capacities. Heavy

driving during pile installation within the very dense sand and gravel with cobbles and boulders may induce settlement or liquefaction of the silty to fine sandy deposits which may have an adverse impact on the existing bridge and embankment. In such cases, pre-augering to about Elevation 394 m may be required, depending on the driving conditions, to minimize the vibrations from the pile driving.

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 level which can be achieved and to what depth the preaugering is completed prior to driving.

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; an ultimate capacity equal to two times the design ULS value must be achieved. The following note should be shown on the Contract drawing:

- “Piles to be driven in accordance with Standard SS-103-11 (Hiley method) using an ultimate capacity of 2,000 kN per pile, but must be driven to at least Elevation 382 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.

If the Hiley method indicates that the design capacity may not have been achieved, pile dynamic analysis (PDA) testing should be carried out on some selected piles as a check to confirm the capacity and assess when the pile can be terminated. Provision should be made in the contract for carrying out the PDA testing.

It is generally recommended that the tips of the H-piles 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. 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

Resistance to lateral loading can be derived using vertical piles, with enhanced support offered by battered piles, if required. The resistance to lateral loading through vertical piles will be derived from the soil surrounding the piles; however, the soils located 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.

For cohesive soils:

$$k_h = \frac{k_{s1}}{5B} \quad \text{where} \quad \begin{array}{l} B \text{ is the pile diameter (m) and} \\ k_{s1} \text{ is the coefficient of horizontal subgrade reaction (MPa/m).} \end{array}$$

For cohesionless soils:

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

The following ranges for the value of  $k_{s1}$  and  $n_h$  may be assumed in the structural analysis. The range in values reflects the variability in the subsurface conditions and values used will depend on the design elevation of the pile cap. Design values are provided for the full stratigraphic sequence at the site, even though it is likely more than needed for the design of the H-piles.

Soil Unit	$k_{s1}$	$n_h$
Embankment fill (assumed to be compacted granular fill)	—	5 to 10 MPa/m
Abutments:		
Very loose to loose silts and sands above Elevation 396 m	—	1 to 3 MPa/m
Interlayered compact to very dense sandy silt/very stiff to hard clayey silt between Elevations 396 m and 388 m	—	4 to 10 MPa/m
Interlayered dense to very dense silt and sand/sand and gravel and silt below Elevation 388 m	—	10 to 15 MPa/m
Hard clayey silt below Elevation 386 m at south abutment and Elevation 380 m at north abutment	70 to 100 Mpa/m	

A maximum lateral resistance of 100 kN for ULS and 25 kN for SLS is recommended for vertical HP310x110 piles driven to the design pile tip elevation.

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 or retaining walls will depend on the type and method of placement of the backfill materials, the nature of the soils behind the backfill, the magnitude of surcharge including construction loadings, on the freedom of lateral movement of the structure, and 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 CHDBC.

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/pile cap (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 <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 as placed, 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 pressure coefficients may be used in the geotechnical design of the structure. If the abutment support does not allow lateral yielding, at-rest earth pressure coefficients should be assumed for geotechnical design.

#### 5.4 Approach Embankment Design and Construction

The proposed top of pavement grade at the bridge widening is expected to be raised about 90 mm from the existing grade which currently ranges from about Elevation 407 m at the south side to about Elevation 406 m at the north side. The existing ground surface at the proposed bridge and embankment widening area varies from about Elevation 402 m in the low-lying swamp area to Elevation 407 m at the existing shoulder, resulting in approach embankments up to about 5 m in height. It should be noted that the elevation of the ground surface profile near the toe of the existing embankment is generally flat at the proposed north approach embankment location; but rises from the north to the south away from the Credit River at the location of the proposed south approach embankment location.

#### **5.4.1 Subgrade Preparation and Embankment Construction**

Based on the borehole results, the existing embankment fill is underlain by a layer of peat and organic silty sand that ranges in thickness from 0.8 m to 1.5 m. Topsoil, peat, and organic silty sand is also present at ground surface extending to depths ranging from 1.4 m to 2.3 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 as outlined in the draft report entitled "Pavement Design Report, Highway 10 From 1 km North of Regional Road 24 Northerly to Highway 9, W.P. 27-97-00", dated February 2004.

Prior to the placement of any fill for the new approach embankment construction, all topsoil, peat, organic silty sand and any softened or otherwise loosened soils should be stripped from below the proposed approach embankment widening 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 for removal of some of the organic material underlying the existing embankment. In order to safeguard the integrity of the existing embankment, the procedures provided in Section 5.4.4 must be followed.

All subgrade soils should be proof-rolled prior to placement of the embankment fill, and any poorly performing areas should be subexcavated and replaced with approved backfill as discussed below.

Unless groundwater control measures are implemented, excavation and backfilling operations to remove and replace the organics, softened, or loosened soils will be carried out partly under water. As such, the choice of backfill material used will depend on whether the option to carry out preloading is available, as described in Section 5.4.5. If the construction schedule can accommodate a period of preloading, then the option of placing granular fill, meeting the specifications of Granular A or Granular B Type II, below the water level is considered feasible. If preloading is not an option, clear stone or other coarse grained material can be used as backfill below the water level. Clear stone should not be used, however, unless there is adequate filtration provided to prevent migration of fines from the underlying sands/silts into the coarse grained fill. If groundwater control measures are implemented, consideration can be given to backfilling with clean earth fill as described below.

Construction of the embankment fill above the water level 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. Given that the existing embankment is composed of sand to sand and gravel fill, it is preferable that granular fill is used for the widening to limit the potential for differential settlement. Once the organic materials are removed and the subexcavation is backfilled, benching into the existing embankment side slopes should be carried out as per OPSD 208.010 for construction of the embankment above the floodplain level to ensure keying of the new fill into the existing fill.

Embankment fill should be placed in regular lifts with the 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 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. A target minimum factor of safety of 1.3 was selected 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, with the parameters 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 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 <sup>3</sup>	32°	—
Peat/Topsoil/Organic Silty Sand	18 kN/m <sup>3</sup>	27°	—

Upper Sand / Sand and Gravel	22 kN/m <sup>3</sup>	32°	—
Upper Silt	20 kN/m <sup>3</sup>	30°	—
Interlayered Sandy Silt / Silt / Clayey Silt	22 kN/m <sup>3</sup>	32°	—

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 conditions, the analysis assumes that the existing highway embankment is cut back at an overall slope of 1H:1V in order to remove as much of the peat, topsoil, and organic soils as possible beneath the existing embankment. The analysis also assumes that the peat, topsoil and organics have been removed from the proposed new embankment footprint.

The results of the analysis for the temporary condition 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 to ensure the integrity of the embankment.

For the final embankment configuration, assuming the new embankment side slopes are constructed at a 2 horizontal to 1 vertical (2H:1V) slope and assuming the peat and organic soils are left in place, a factor of safety of 1.3 is obtained (See Figure 2). It is recommended that the new embankment side slope be flattened to 2.5H:1V if the peat and organic soils are left in place to increase the factor of safety above 1.3. Alternatively, if the peat and organic soils are removed and replaced with engineered fill according to the recommendations provided in Section 5.4.1, a new embankment side slope of 2H:1V can be constructed.

#### 5.4.3 Settlement

Settlement analyses were performed for situations 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 the ground surface in the boreholes at the toes of the existing embankment (up to 2.3 m thick) and these materials were also encountered below the existing embankment fill (up to 1.2 m thick).

If the peat, topsoil, or organics were to be left in place under the proposed widened embankment area, additional settlement of the embankment would occur as a result of consolidation settlement of these foundation materials. For the settlement analyses completed, it is calculated that about 1 m of consolidation could occur within a 2.3 m thick layer of the organic deposit under a 5 m high embankment. The majority of this settlement (up to about 90% or 900 mm) is expected to occur during construction and within about 3 to 4 months after completion of the embankment. The remainder of the settlement (up to about 100 mm) which includes creep movement, is expected to continue over the life of the roadway. Due to the magnitude of the anticipated settlement and variable nature of the peat, topsoil and organic soils, it is recommended that these materials be removed prior to constructing the new embankment.

If the organic deposits are removed from beneath the proposed widening embankment area, the results of the analysis indicate that settlement of the underlying very loose to loose silts and sands would be up to 150 mm for the proposed 5 m high embankment (assuming 2.3 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 relatively high permeability of the soils. 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.

Due to the fact that the organic deposits beneath the existing embankment will not be completely removed, the loading influence of the new widening of the embankment will likely induce additional consolidation beneath the existing embankment. 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. This construction will result in an additional load on the in situ organic deposit equivalent to about 2.5 m to 3 m of fill. Due to the variable thickness of the organic deposits (0.7 m to 1.2 m) located beneath the existing embankment and the proposed widening construction/geometry, it is estimated that embankment settlement due to consolidation of the organic deposits could range up to 200 mm. The majority of this settlement (up to about 90%) 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 deposits.

#### **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, or organics as possible, 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 or 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 piles or soldier piles with lagging wall. The temporary support to the wall would probably have to include tiebacks, since the sands and silts underlying the organics would not likely provide suitable support for 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 should be carried out in strips formed perpendicular to the highway alignment with the base of the excavation/trench no wider than 3 m;
- The excavation should be carried out such that the base of the excavation 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 excavation;
- Backfilling operations to original ground level should be completed for each strip prior to commencing excavation of the adjacent strip.

#### **5.4.5 Mitigation of Time Dependant Settlement**

As indicated previously, there will still be some peat, topsoil, or organics deposit remaining in place under the existing embankment. Consolidation settlement of these deposits will occur due to the additional loading applied over the area of the temporary 1H:1V side slope and increased road grade. The majority of the settlement is anticipated to occur within 3 months of placing the embankment fill; however, more long-term settlement should be expected due to creep and natural organic decay processes.

Consideration should be given to preloading (i.e., constructing the embankment to grade and allowing time for settlement) the embankment to induce as much of the settlement as possible, prior to constructing the approach slab and pavement structure. Such efforts should limit the subsequent maintenance time and expense to raise and level of 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 at the abutments will typically extend through the existing sand to sand and gravel embankment fill, the peat, topsoil and organic silty sand, and will be terminated within the underlying silts and sands. Excavations will be extended below the groundwater level which was generally found in the boreholes to be at the same level as the Credit River. The river water level at the South Bridge structure was measured to be at Elevation 401.7 m in January 2004. 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, the adjacent road embankment and bridge structure, high groundwater conditions, and generally very loose to loose surficial soils, it is anticipated that excavations for the pile caps will involve extensive groundwater control together with temporary excavation support systems, and may involve temporary diversion and/or constraint of the Credit River. An NSSP has been included in Appendix C to alert the contractor of the water-bearing soils and dewatering that is required. Four separate work areas for the northeast, northwest, southeast, and southwest pile caps will be required.

The following two main options could be considered for construction to allow placement of concrete in the "dry": it should be noted that both options assume that the base of the proposed pile cap excavation is maintained above the existing pile cap level.

- A) Place a cofferdam around one of the abutments to divert the river flow to the opposite side of the cofferdam. Difficulties associated with constructing the cofferdam beneath the existing bridge should be anticipated (i.e. there is less than 5 m clearance from the bottom of the bridge deck to the river bottom). Install a well-point dewatering system and excavate in a open cut to construct the pile cap. A temporary liner could be placed along the existing river bottom to help control water infiltration into the excavation. Upon completion of the construction at either the north or south abutment, the cofferdam would be repositioned to divert the river flow to the other side of the abutment under construction, and the process repeated. 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 toe 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 may need to allow for installation of lateral support struts or

reinforcement, especially should the design toe depths not be achieved due to refusal by cobbles/boulders. Control of water infiltration will need to be managed to mitigate "piping" of the underlying soils at the base of the excavation if design toe depths are not met due to the cobbles and boulders that may be 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 existing pile cap, and special procedures would be required in order to minimize disturbance to the founding soils under the existing pile caps surrounding the existing piles. Prior to driving sheet piling, the limits of the existing bridge pile caps should be confirmed.

As an alternative, 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.

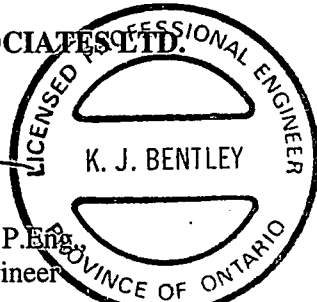
A maximum peak particle velocity (PPV) of 50 mm/s is recommended at the existing bridge structure. The piles furthest from the existing structure should be driven first, in order to check the vibration level at the existing structure and if necessary, alter the pile driving criteria for the remaining piles.


## 6.0 CLOSURE

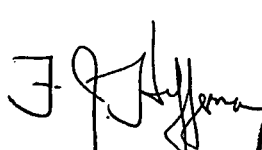
This report was prepared by Mr. Kevin Bentley, P.Eng., a geotechnical engineer and the technical aspects were reviewed by Ms. Anne Poschmann, P.Eng., a Principal and senior geotechnical engineer. Mr. Fintan J. Heffernan, P.Eng., a Designated MTO Contact for Golder, conducted an independent quality control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

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KJBASP/FJH/al

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**TABLE 1**  
**EVALUATION OF FOUNDATION ALTERNATIVES**  
**CREDIT RIVER CROSSING – SOUTH BRANCH**  
**HIGHWAY 10**

<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 the hard clayey silt or very dense interlayered silt and sand/sand and gravel (design pile tip Elevation 380 m)	1	Higher capacity relative to shallower H-piles; With pile cap maintained as high as possible, excavation and dewatering requirements are minimized resulting in low environmental impact on designated environmentally sensitive area.	Hard driving may be encountered through gravelly deposits and piles may “hang-up” on boulders; May require pre-augering to above Elevation 394 m to minimize potential for adverse impact on existing pile groups.	Lower relative costs than piles driven to greater depths;	Potential for piles to “hang up” on cobbles and boulders or to be deflected away from vertical during driving; Potential risk of disturbing existing timber pile groups with hard pile driving.
Steel H Piles driven into the interlayered sandy silt and silt/clayey silt (approximate pile tip Elevation 388 m at south abutment and 386 m at north abutment)	2	Reduced pile lengths compared to Option 1.	Lower pile capacity relative to deeper H-piles; Hard driving may be encountered through gravelly deposits and piles may “hang-up” on boulders; Larger number of piles required due to lower capacity; If pre-augering is required, shaft friction is reduced and pile capacities decrease.	Lower relative material costs than piles to greater depths; Costs increase if pre-augering is required due to larger number of piles required.	Potential for piles to “hang up” on cobbles and boulders or to be deflected away from vertical during driving; Potential risk of disturbing existing timber pile groups with hard pile driving;
Steel Pipe Piles	3	Reduced pile lengths compared to Option 1.	Shallow gravel layer is variable and not thick enough to provide design geotechnical resistance for short pipe piles; Pipe piles could be driven closed ended into underlying compact sands but would have limited carrying capacity.	Pipe piles driven to greater depths results in higher construction costs	Potential for piles to “hang up” on cobbles/boulders present within soil; Due to variable nature of the interlayered deposit, the required pile length for friction piles will have to be established during construction.

Spread Footings	NF	-	No suitable bearing stratum at reasonable founding depth.		
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.	Extra costs associated with liners and inspection. High costs of drilling through cobbles/boulders	Difficulty may be encountered in drilling and extending liner through cobbles and boulders; Difficulty may be encountered to seal off groundwater inflow due to many sand and silt interlayers; downhole inspection may not be possible.

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

## LIST OF ABBREVIATIONS

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

### I. SAMPLE TYPE

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

### III. SOIL DESCRIPTION

#### (a) Cohesionless Soils

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

### II. PENETRATION RESISTANCE

#### Standard Penetration Resistance (SPT), N:

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

#### Consistency

	$c_u, s_u$	$c_u, s_u$
	kPa	psf
Very soft	0 to 12	0 to 250
Soft	12 to 25	250 to 500
Firm	25 to 50	500 to 1,000
Stiff	50 to 100	1,000 to 2,000
Very stiff	100 to 200	2,000 to 4,000
Hard	over 200	over 4,000

#### Dynamic Cone Penetration Resistance; $N_d$ :

The number of blows by a 63.5 kg (140 lb.) hammer dropped 760 mm (30 in.) to drive uncased a 50 mm (2 in.) diameter, 60° cone attached to "A" size drill rods for a distance of 300 mm (12 in.).

**PH:** Sampler advanced by hydraulic pressure

**PM:** Sampler advanced by manual pressure

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

**WR:** Sampler advanced by weight of sampler and rod

#### Piezo-Cone Penetration Test (CPT)

A electronic cone penetrometer with a 60° conical tip and a project end area of 10 cm<sup>2</sup> pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance ( $Q_t$ ), porewater pressure (PWP) and friction along a sleeve are recorded electronically at 25 mm penetration intervals.

### IV. SOIL TESTS

w water content  
w<sub>p</sub> plastic limit  
w<sub>l</sub> 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<sub>R</sub> relative density (specific gravity, G<sub>s</sub>)  
DS direct shear test  
M sieve analysis for particle size  
MH combined sieve and hydrometer (H) analysis  
MPC Modified Proctor compaction test  
SPC Standard Proctor compaction test  
OC organic content test  
SO<sub>4</sub> concentration of water-soluble sulphates  
UC unconfined compression test  
UU unconsolidated undrained triaxial test  
V field vane (LV-laboratory vane test)  
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

$\pi$	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 = (Compressive strength)/2

PROJECT 03-1111-023

# RECORD OF BOREHOLE No SB1

1 OF 1 METRIC

W.P. 27-97-00

LOCATION N 4862338.9; E 260429.6

ORIGINATED BY GD

DIST HWY 10

BOREHOLE TYPE POWER AUGERING USING 108 mm O.D. SOLID STEM AUGERS

COMPILED BY KG

DATUM Geodetic

DATE December 04, 2003

CHECKED BY KB

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						
407.4	GROUND SURFACE					20	40	60	80	100	10	20	30		
0.0	Sand, trace to some gravel, trace silt and organics, occasional silty clay seams (FILL) Loose Brown Moist to wet		1	SS	4										
			2	SS	8										
			3	SS	5										
			4	SS	14										
			5	SS	9										
			6	SS	7										
402.5			7	SS	4										
4.9	PEAT Soft to firm Dark brown Moist														
401.0			8	SS	9										
6.4	SAND, some gravel, trace silt and organics Loose Brown Wet														
399.8			9	SS	3										
7.6	CLAYEY SILT, trace sand Soft Brown Wet														
398.5															
8.8	End of Drilling and beginning of Dynamic Cone Penetration Test														
397.5															
9.9	End of Penetration Test upon refusal at 9.91m depth  Notes:  1. Borehole caved to a depth of 3.81m below ground surface.														

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

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PROJECT 03-1111-023

# RECORD OF BOREHOLE No SB2

1 OF 1 METRIC

W.P. 27-97-00

LOCATION N 4862360.7 :E 260379.2

ORIGINATED BY GD

DIST HWY 10

BOREHOLE TYPE POWER AUGERING USING 108 mm O.D. SOLID STEM AUGERS

COMPILED BY KG

DATUM Geodetic

DATE December 04, 2003

CHECKED BY KB

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
405.6	GROUND SURFACE						20	40	60	80	100							
0.0	Sand and gravel, trace silt, occasional rootlets and sandy silt seams (FILL) Loose to compact Brown Moist		1	SS	7													
			2	SS	23													
404.1																		
1.5	Sand to silty sand, trace to some clay, trace gravel and organics (FILL) Compact to loose Brown Moist		3	SS	14													
			4	SS	9													
			5	SS	26													
401.5			6	SS	14													
4.1	PEAT, trace gravel and wood fragments Firm Dark brown Moist		7	SS	6													
400.7																		
4.9	SAND, trace silt Loose Grey Wet																	
399.5			8	SS	11													
6.1	SAND AND GRAVEL, trace silt Loose to compact Brown Wet																	
397.7			9	SS	14													
8.1	SAND, some silt, trace gravel Compact Brown Wet  End of Borehole  Notes:  1. Borehole caved to a depth of 5.2m below ground surface.																	

PROJECT 03-1111-023

**RECORD OF BOREHOLE No SB3**

1 OF 2 **METRIC**

W.P. 27-97-00

LOCATION N 4862370.9 ; E 260408.1

ORIGINATED BY GD

DIST HWY 10

BOREHOLE TYPE POWER AUGERING USING 108 mm O.D. SOLID STEM AUGERS AND CASING COMPILED BY KG

DATUM Geodetic

DATE September 23, 2003

CHECKED BY KJB

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					
401.9	GROUND SURFACE													
0.0	TOPSOIL Very loose Brown Wet		1	SS	3									
401.1														
0.8	ORGANIC SILTY SAND, occasional rootlets Very loose to loose Grey Wet		2	SS	4		401							
			3	SS	4		400							
399.6														
2.3	SAND, some gravel, trace silt Loose Grey Wet		4	SS	7		399							
398.9														
3.0	SILT, some sand, trace clay, contains clayey silt and sandy silt interlayers Loose to very loose Grey Wet		5	SS	9		398							
			6	SS	7									
396.9			7	SS	4		397							
5.0	SAND AND GRAVEL, trace silt and clay, contains cobbles and boulders Dense to very dense Grey Wet						396							
394.9			8	SS	47		395							
7.0	CLAYEY SILT, trace sand and gravel Hard Grey Wet						394							
393.4														
8.5	SILT, some sand Compact Brown Wet		9	SS	24		393							
391.2							392							
10.7	CLAYEY SILT, trace sand, occasional gravel interlayers Very stiff Brown Wet		10	SS	22		391							
389.7							390							
12.2	SANDY SILT, trace clay, contains interlayers of clayey silt Compact Brown Wet		11	SS	26		389							
			12	SS	26		388							
							387							

Continued Next Page

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

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PROJECT 03-1111-023

# RECORD OF BOREHOLE No SB3

2 OF 2 METRIC

W.P. 27-97-00

LOCATION N 4862370.9 ; E 260408.1

ORIGINATED BY GD

DIST HWY 10

BOREHOLE TYPE POWER AUGERING USING 108 mm O.D. SOLID STEM AUGERS AND CASING COMPILED BY KG

DATUM Geodetic

DATE September 23, 2003

CHECKED BY KJB

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 ● QUICK TRIAXIAL	+ FIELD VANE x REMOULDED						
— CONTINUED FROM PREVIOUS PAGE —															
386.7 15.2	Probable cobble/boulder encountered at 14.9 m SILT AND SAND with gravel, cobbles, and boulders Very dense Brown Wet		13	SS	75		386								50 40 9 1
							385								
							384								
384.2 17.7	SAND AND GRAVEL with cobbles/boulders Very dense brown Wet						383								
			14	SS	50/0.1		382								
							381								
380.6 21.3	SILT, trace to some sand, occasional clayey silt interlayers, contains sand and gravel seams. Very dense Grey Moist		15	SS	50/0.15		380								
							379								
							378								
377.4 24.5	End of Borehole		16	SS	100/0.15										
	Note: 1. Solid stem augers used to drill to a depth of 5 m. NW casing used to drill from 5.0m to a depth of 15.8 m. NQ sized core barrel was used to drill through a boulder from 15.8 m to 16.8 m. NW casing used to drill to the end of the borehole to a depth of 24.5 m.  2. Water level measured at 0.2m (EL. 401.7m) below ground surface on January 7, 2004.														

<b>PROJECT</b> 03-1111-023		<b>RECORD OF BOREHOLE No SB4</b>		1 OF 2 <b>METRIC</b>	
<b>W.P.</b> 27-97-00		<b>LOCATION</b> N 4862335.9 ; E 260392.5		<b>ORIGINATED BY</b> GD	
<b>DIST</b> HWY 10		<b>BOREHOLE TYPE</b> POWER AUGERING USING 108 mm I.D. HOLLOW STEM AUGERS		<b>COMPILED BY</b> KG	
<b>DATUM</b> Geodetic		<b>DATE</b> December 1, 2003		<b>CHECKED BY</b> KB	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT			UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x REMOULDED	W <sub>p</sub>	W	W <sub>L</sub>			
402.4	GROUND SURFACE													
0.0	Silt and sand, trace gravel and clay, contains rootlets (FILL) Very loose Brown Moist		1	SS	4									
401.5														
0.9	PEAT TO ORGANIC SILTY SAND, occasional sand seams Very soft to soft Dark brown Moist		2	SS	3									1 54 40 5
			3	SS	2									
400.1														
2.3	SAND AND GRAVEL, trace silt Loose Grey Wet		4	SS	6									
399.4														
3.1	SILT, trace to some clay and sand, occasional cobbles Loose Grey Wet		5	SS	7									
			6	SS	4									
			7	SS	4									
396.3														
6.1	SILT, some sand, clay, and gravel, contains gravel interlayers Very loose to loose Grey Wet		8	SS	4									17 12 58 13
394.8														
7.6	CLAYEY SILT, trace to some sand, trace gravel, occasional silt seams Very stiff to hard Grey Moist		9	SS	26									
			10	SS	31									
391.7														
10.7	SILT, some clay, trace sand Loose to compact Grey Wet		11	SS	10									
390.2														
12.2	CLAYEY SILT, some sand, trace gravel, occasional silt seams Very stiff Brown Wet		12	SS	19									
388.7														
13.7	SAND AND GRAVEL, trace silt, occasional boulders/cobbles Very dense Brown Wet		13	SS	50/0.15									

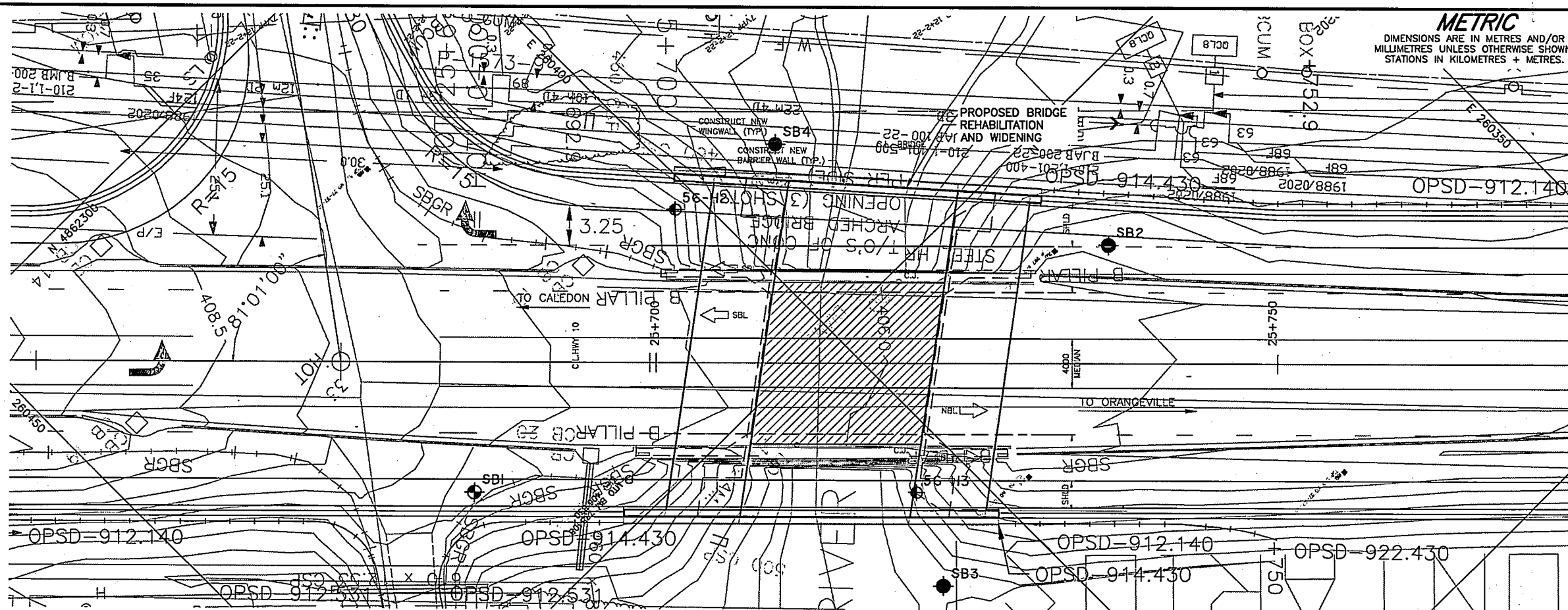
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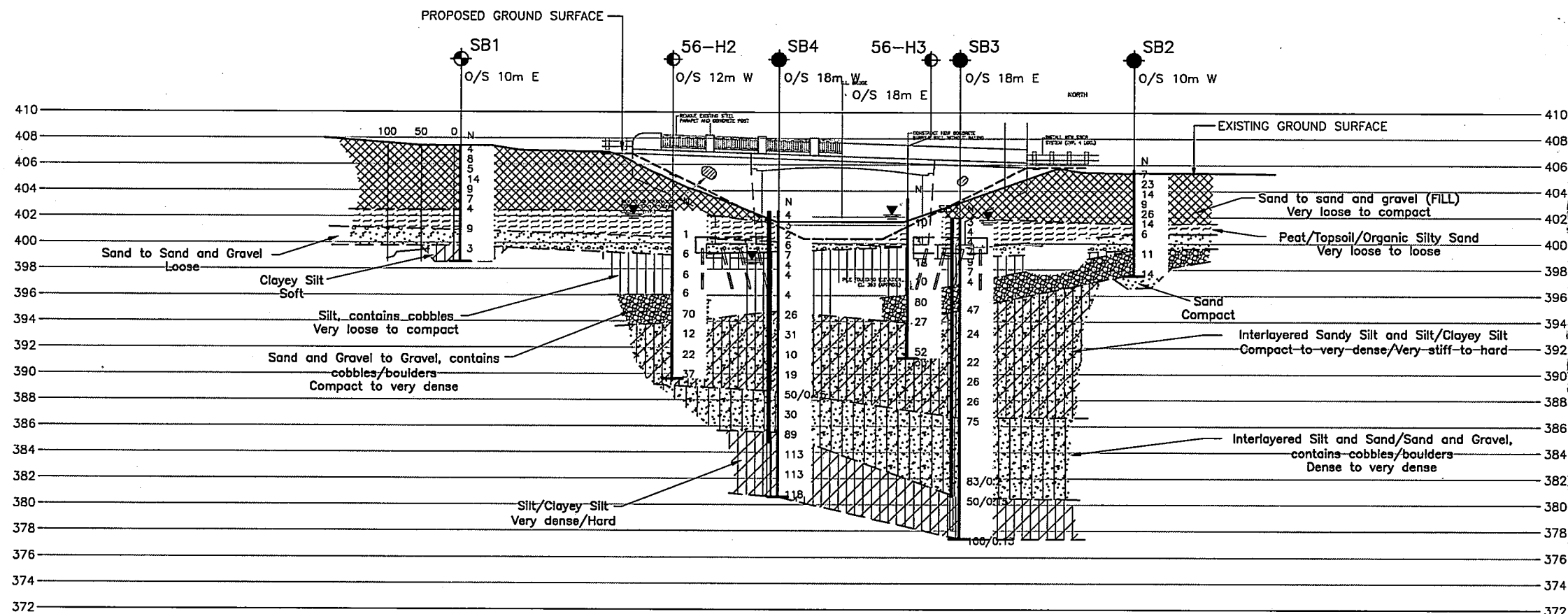
+ <sup>3</sup>, X <sup>3</sup>: Numbers refer to Sensitivity      ○ <sup>3</sup>% STRAIN AT FAILURE

PROJECT 03-1111-023		RECORD OF BOREHOLE No SB4				2 OF 2 METRIC							
W.P. 27-97-00		LOCATION N 4862335.9 E 260392.5				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 1, 2003				CHECKED BY KB							
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								
387.2	— CONTINUED FROM PREVIOUS PAGE —												
15.2	SAND AND SILT Dense Grey Wet		14	SS	30		387						
385.6							386						
16.8	SILT, trace to some sand, occasional clayey silt interlayers Very dense Grey Wet to moist		15	SS	89		385						
			16	SS	113		384						
382.4							383						
20.0	CLAYEY SILT, occasional silt seams Hard Grey Wet to moist		17	SS	113		382						
381.1							381						
21.3	SILT, trace sand Very dense Grey Moist		18	SS	118								
380.6													
21.8	End of Borehole												
Notes: 1. Water level in piezometer measured at 3.75m below ground surface (Elev. 398.7 m) on January 7, 2004.													

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PLAN  
SCALE  
0 4 8 m



SECTION/PROFILE ALONG CENTRE LINE  
SCALE  
0 4 8 m

CONT No.  
WP No. 27-97-00

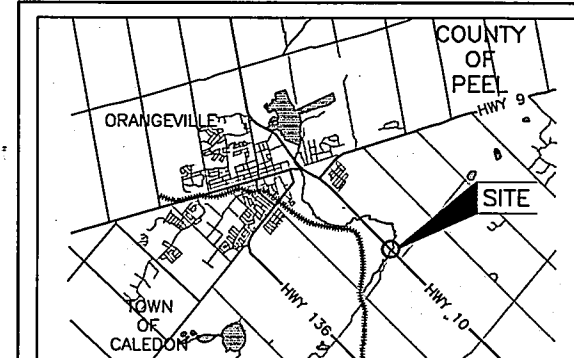
HIGHWAY 10  
CREDIT RIVER BRIDGE - SOUTH BRANCH  
BOREHOLE LOCATION AND SOIL STRATA



SHEET



Golder Associates Ltd.  
MISSISSAUGA, ONTARIO, CANADA



KEY PLAN  
SCALE  
0 2 km

# LEGEND

- Borehole - Current Investigation
- Borehole - Racey, MacCallum and Associates Ltd. (1956)
- Borehole and Cone - Current Investigation
- ⊥ 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
- ≡ WL upon completion of drilling

No.	ELEVATION	CO-ORDINATES	
		NORTHING	EASTING
SB1	407.4	4862338.9	260429.6
SB2	405.6	4862360.7	260379.2
SB3	401.9	4862370.9	260408.1
SB4	402.4	4862335.9	260392.5
56-H3	403.4	4862334.0	260402.0
56-H2	402.4	4862364.0	260404.3

# NOTES

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

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

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

# REFERENCE

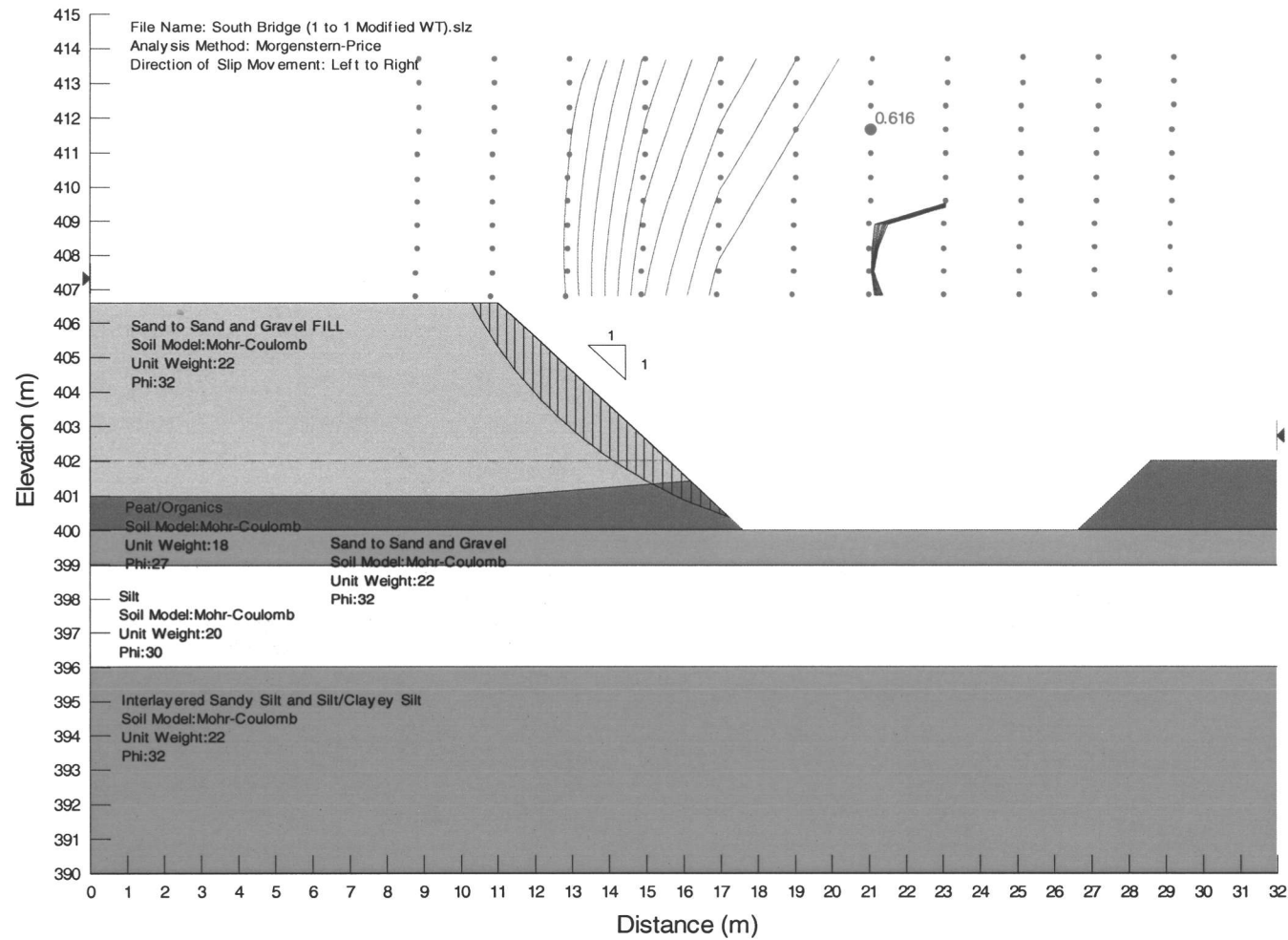
Base plans provided in digital format by Morrison Hershfield Ltd., drawing file nos. 1034006-S01(SB).dwg, dated Sept., 2003.

NO.	DATE	BY	REVISION
Geocres No. 40P16-22			
Hwy. 10			PROJECT NO. 03-1111-023 DIST.
SUBM'D. KJB		CHKD. KJB	DATE: Feb. 19, 07 SITE: 24-10
DRAWN: JDR/MSM		CHKD. ASP	APPD. DWG. 1



# TEMPORARY EXCAVATION CONFIGURATION

FIGURE 1



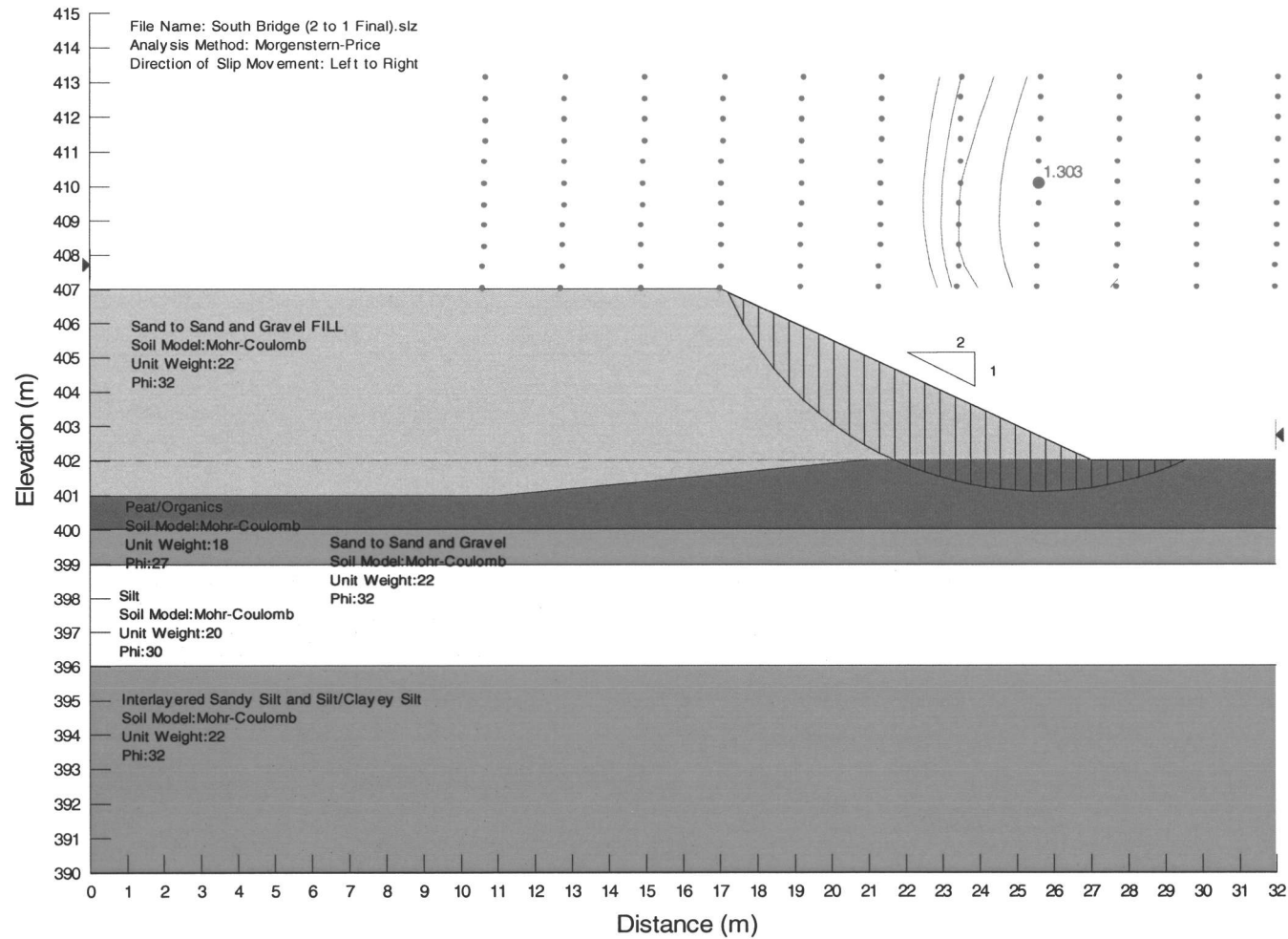
Date: March, 2004  
 Project: 03-1111-023

Golder Associates

Drawn: KG  
 Checked: KJB

# FINAL EMBANKMENT CONFIGURATION

FIGURE 2



Date: March, 2004  
 Project: 03-1111-023

Golder Associates

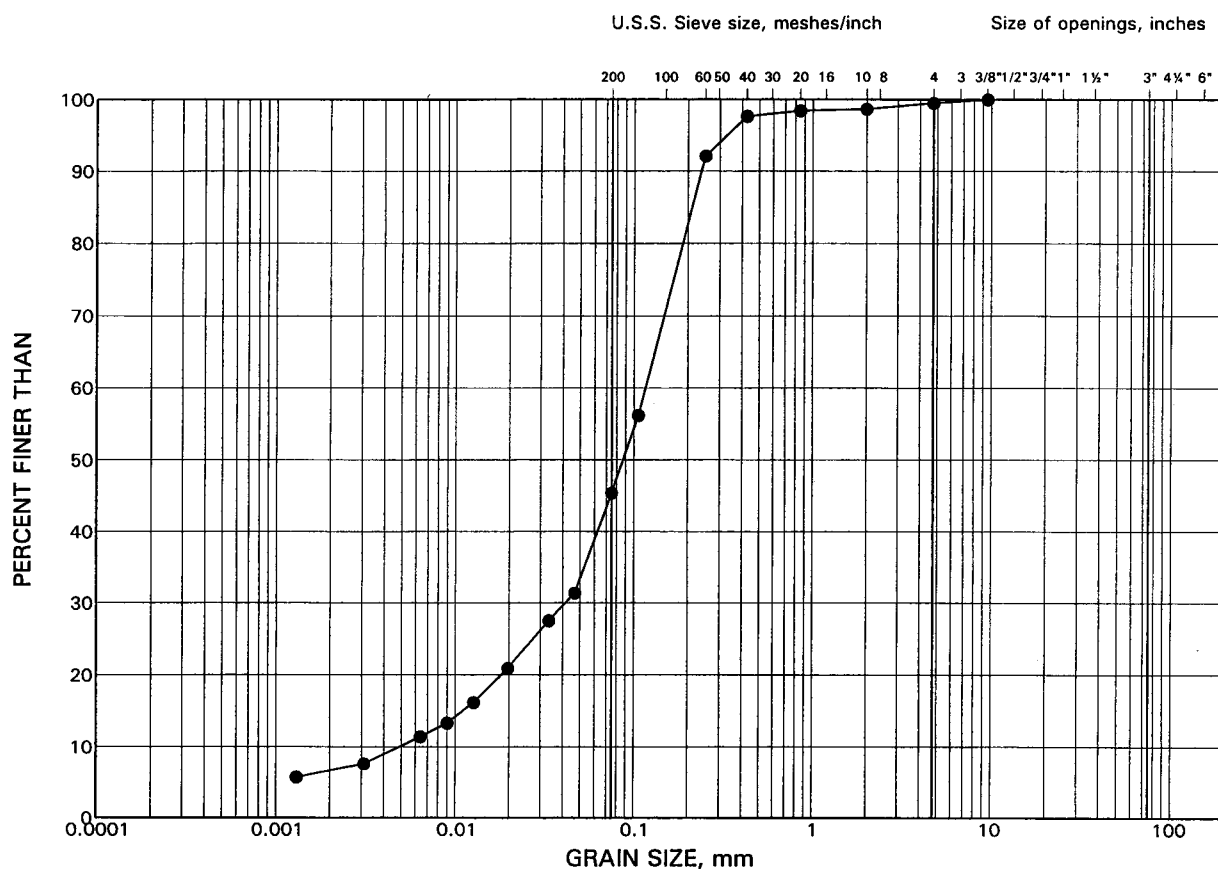
Drawn: KG  
 Checked: KJB

**APPENDIX A**  
**LABORATORY TEST DATA**

# GRAIN SIZE DISTRIBUTION

## Silt and Sand (Fill)

FIGURE A1



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

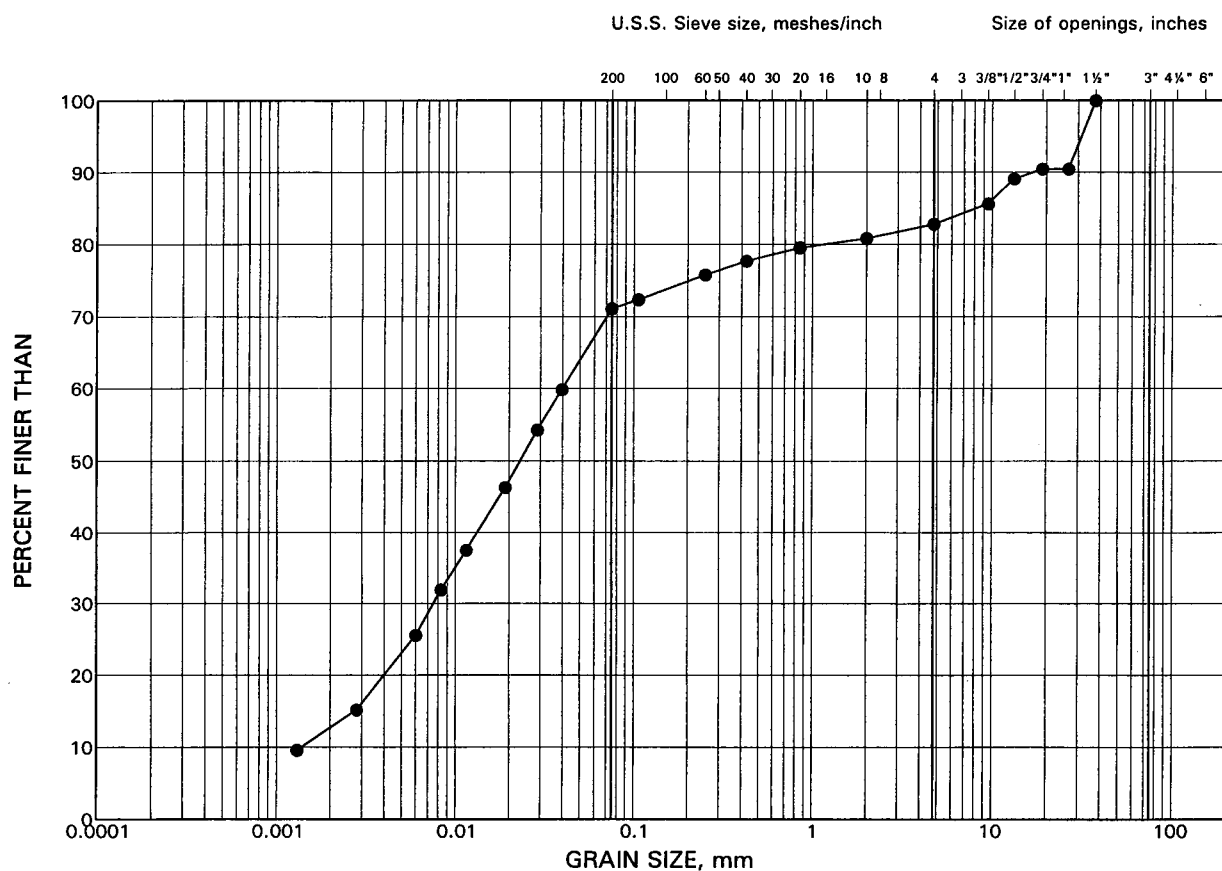
### LEGEND

SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)
•	SB4	2	0.8-0.9

# GRAIN SIZE DISTRIBUTION

Silt, some clay, sand and gravel

FIGURE A2



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

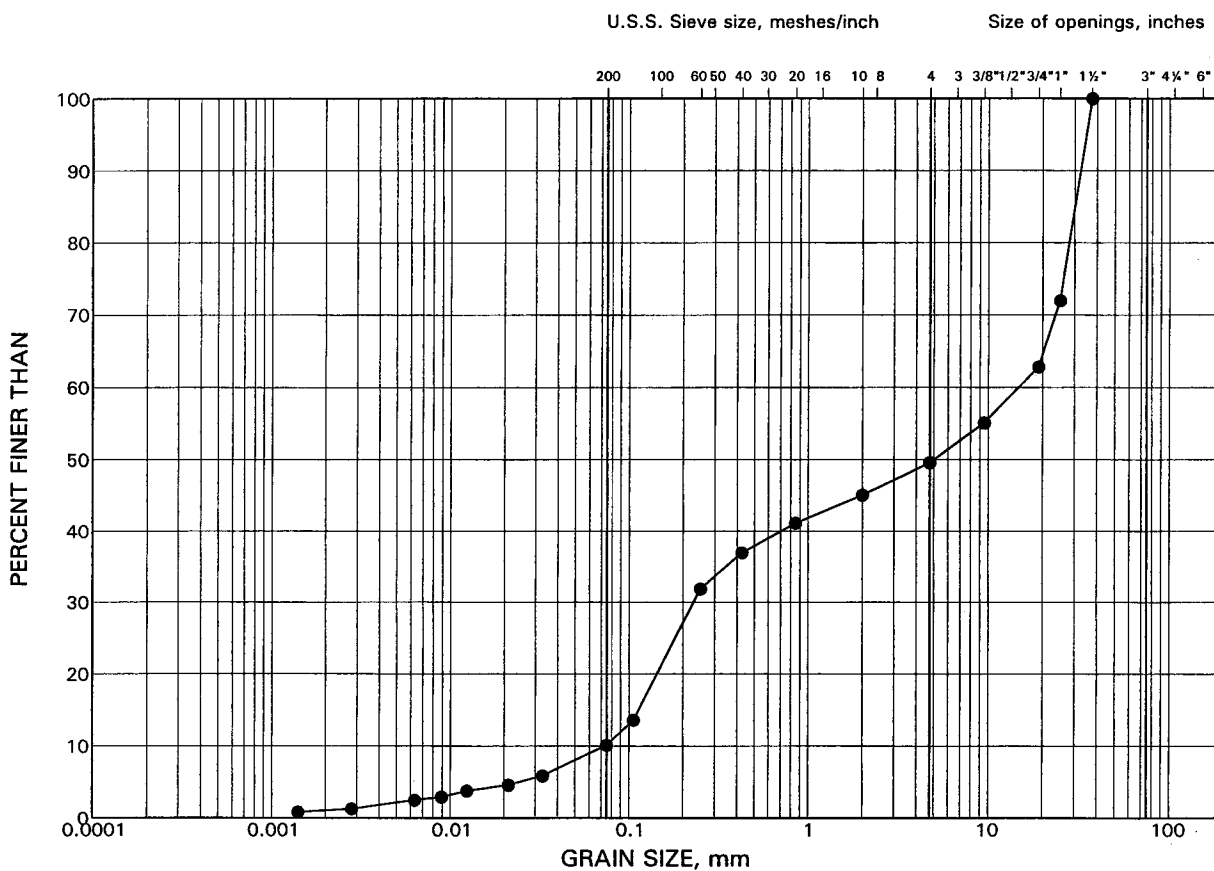
## LEGEND

SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)
•	SB4	8	6.1-6.6

# GRAIN SIZE DISTRIBUTION

## Sand and Gravel

FIGURE A3



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

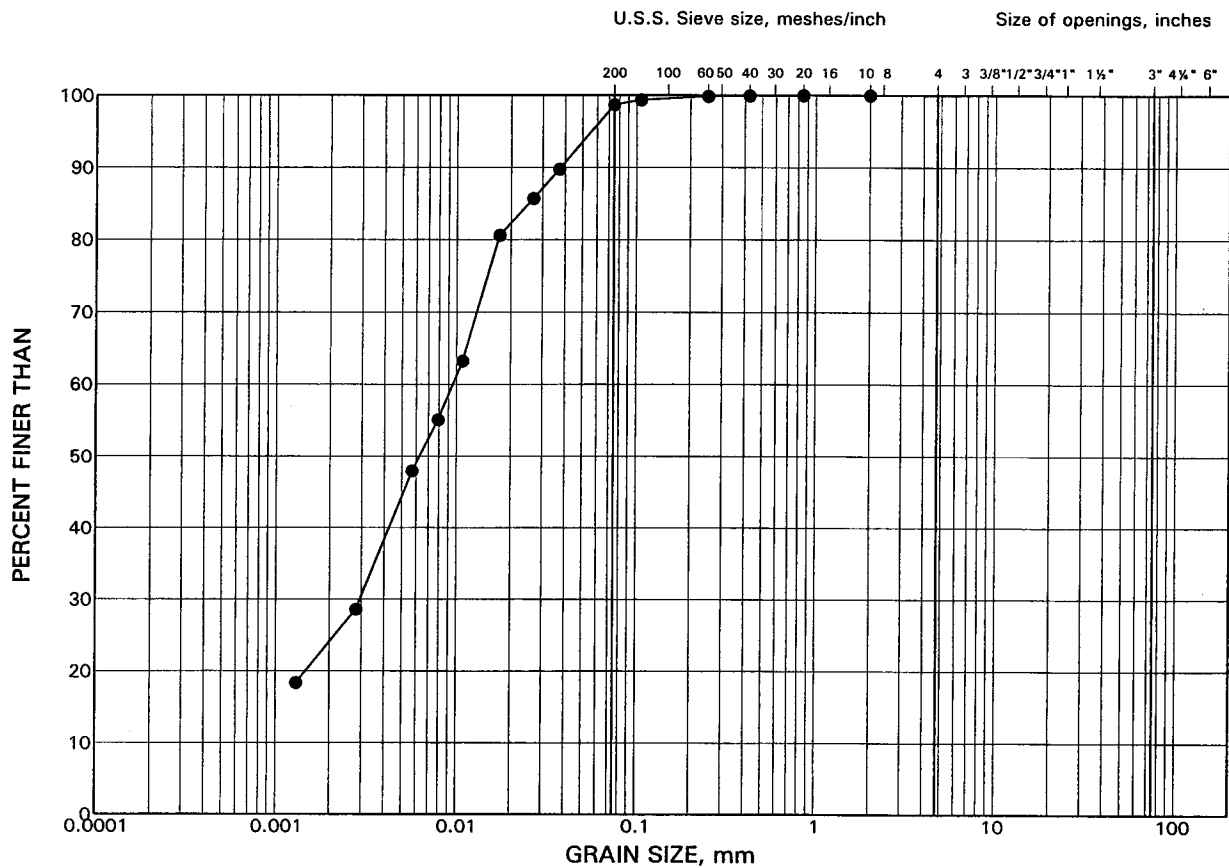
### LEGEND

SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)
•	SB3	14	19.8-20.0

# GRAIN SIZE DISTRIBUTION

Clayey Silt

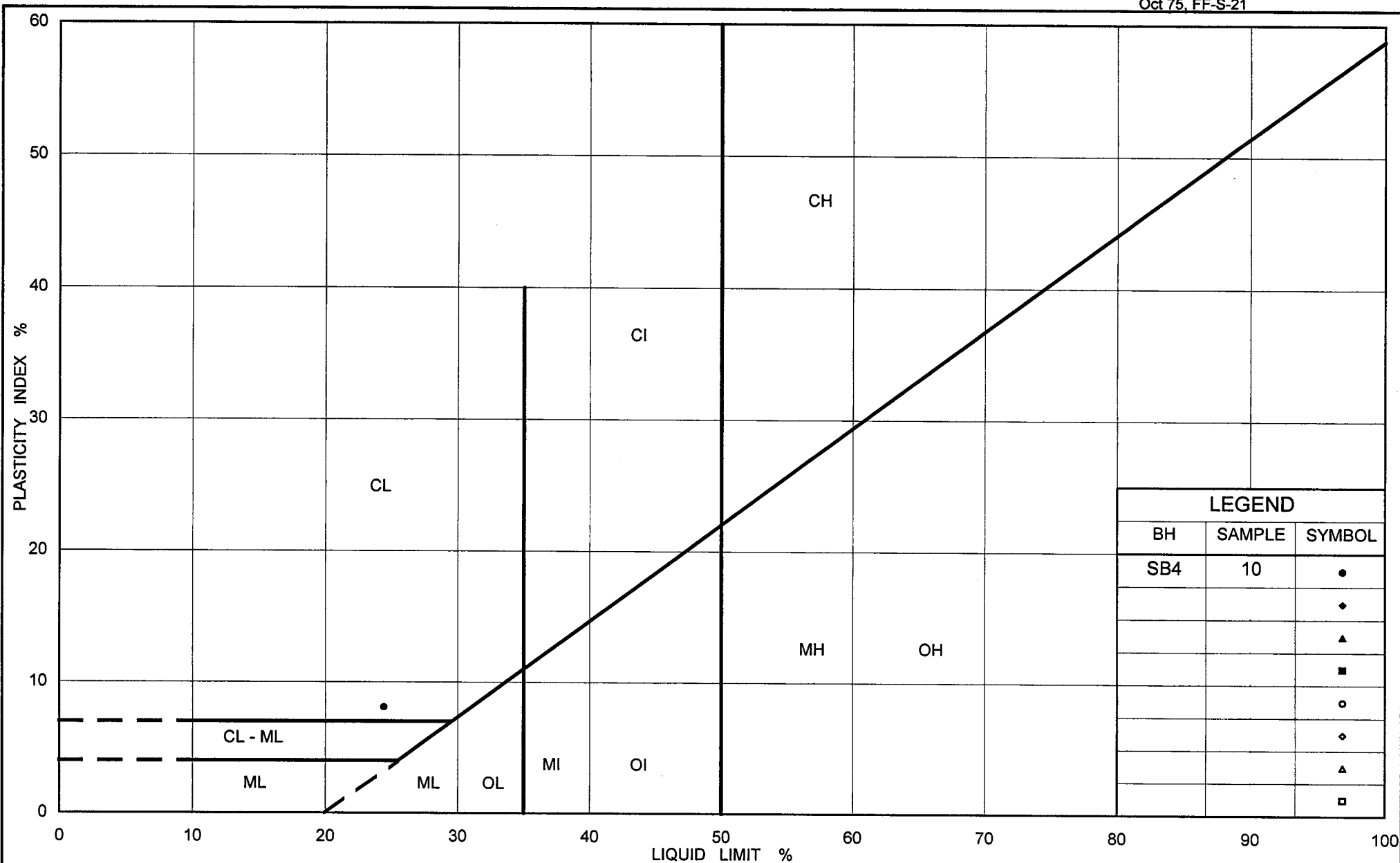
FIGURE A4



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

## LEGEND

SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)
•	SB4	17	19.8-20.3



Ministry of Transportation

Ontario

# PLASTICITY CHART Clayey Silt

FIG No. A5

Project No. 03-1111-023

Checked By: **KSB**



**APPENDIX B**

**RECORD OF BOREHOLES (56-H2, 56-H3)**  
**PREVIOUS INVESTIGATION**

Hole Begun \_\_\_\_\_ Foundation Engineering Division

Hole Ended \_\_\_\_\_ Engineering Data Sheet for Borehole: 56-H2

Helper \_\_\_\_\_

Job Name: Credit River Bridge ~ Sta 515+50 Hwy 10

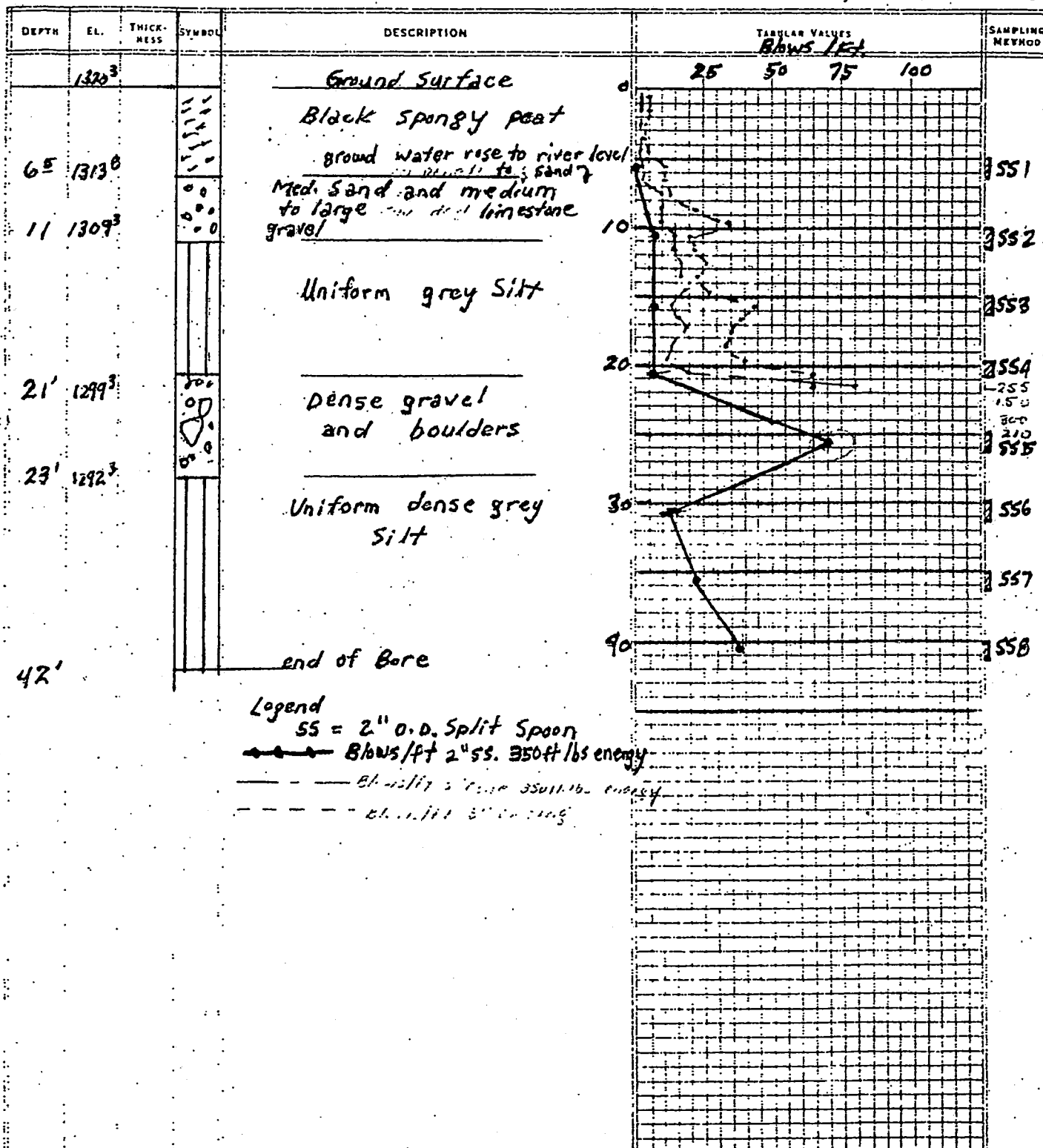
Job Located: Approx 4 miles North Caledon Ont

Checked by \_\_\_\_\_

Hole Located: See encl. No. 1

Hole Elevation: 1320<sup>3</sup> Datum: Geodetic E. 1331.9'

Day \_\_\_\_\_ Month \_\_\_\_\_ Year \_\_\_\_\_



Hole Begun \_\_\_\_\_ Foundation Engineering Division

Hole Ended \_\_\_\_\_ Engineering Data Sheet for Borehole: 56-H3

Job Name: Credit River Bridge ≈ Sta 515+50 Hwy. 10

Job Located: Approx 4 mi North Caledon, Ont

Hole Located: See encl. No. 1

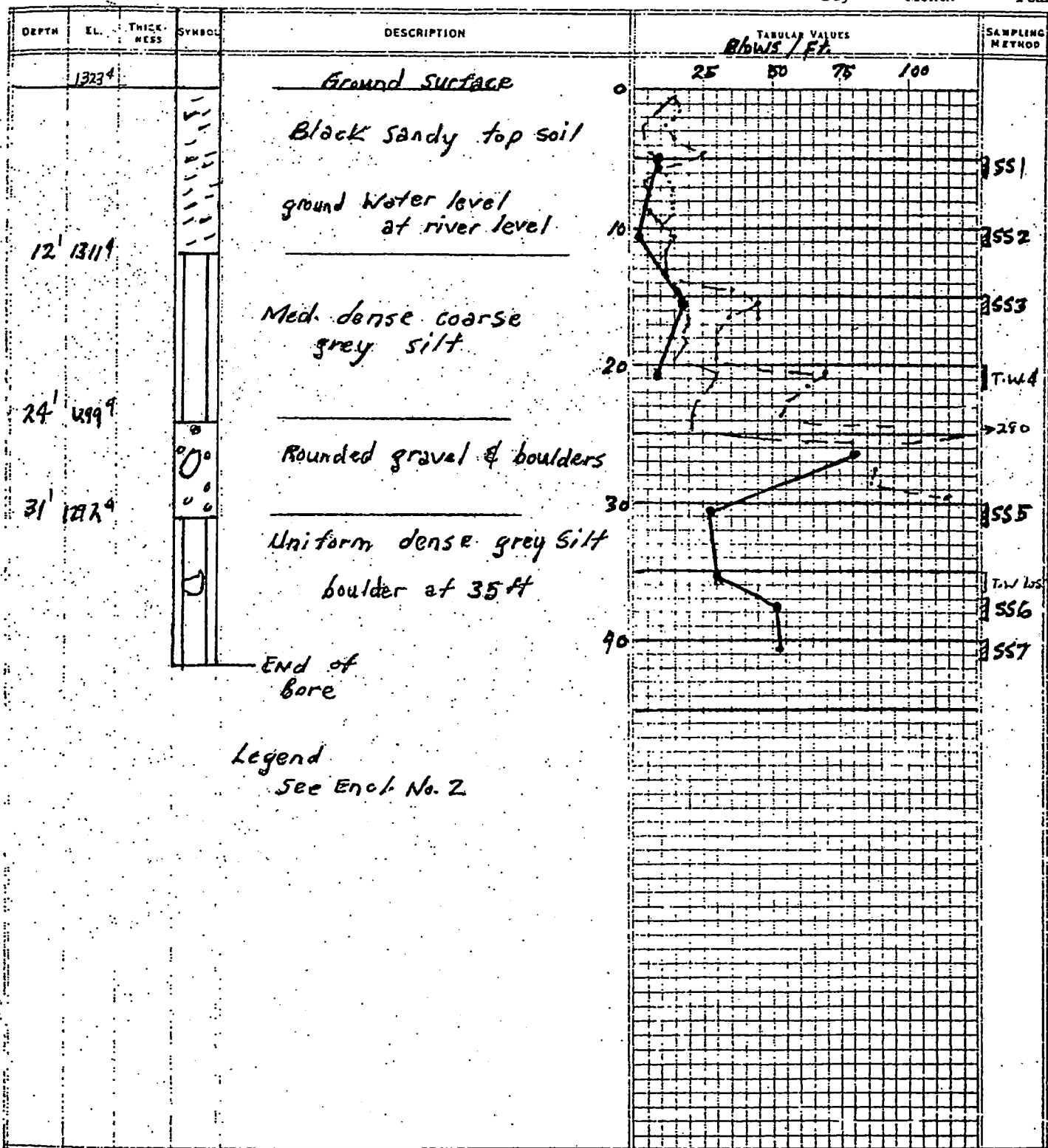
Hole Elevation: 1323.9 Datum: Geodetic F1 1331.91

Driller \_\_\_\_\_

Helper \_\_\_\_\_

Checked by \_\_\_\_\_

Day \_\_\_\_\_ Month \_\_\_\_\_ Year \_\_\_\_\_



**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 Widening over the Credit River – South 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

**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 Bridge Widening over the Credit River – South 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

## **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

## **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 widening of the existing Highway 10 structure over the Credit River – South 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 Widening – South 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. 24-10.

#### ***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 at least two weeks before pile driving commences. 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 (South Bridge) over the Credit River – South 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 South 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

n:\active\2003\1111\03-1111-023 mh hwy10\final reports\final fdr\03-1111-023b south bridge\03-1111-023 south bridge nssp vibration monitoring.doc



**Golder Associates Ltd.**

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Fax 905-567-6561



**REPORT ON**

**FOUNDATION INVESTIGATION  
CREDIT RIVER BRIDGE WIDENING - SOUTH 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. 24-10**

Submitted to:

Morrison Hershfield Ltd.  
Suite 600, 235 Yorkland Blvd.  
Toronto, Ontario  
M2J 1T1

GEOCRE NO: 40P16-22

**DISTRIBUTION**

- 1 Copy - Ministry of Transportation, Ontario,  
Downsview, Ontario (Central Region)
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- 1 Copy - Golder Associates Ltd.,  
Mississauga, Ontario

February 2007

03-1111-023B



February 2007

03-1111-023B

**PART A**

**FOUNDATION INVESTIGATION REPORT  
CREDIT RIVER BRIDGE WIDENING - SOUTH 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. 24-10**

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Figure A1	Grain Size Distribution – Silt and Sand (Fill)
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Figure A5	Plasticity Chart – Clayey Silt
Appendix B	Record of Boreholes (56-H2, 56-H3) – Previous Investigation

## 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 detailed 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 supplemental letter "Revision to Borehole Drilling Program", dated November 20, 2003. This report addresses the widening of the existing bridge crossing the South Branch of the Credit River (South Bridge) on Highway 10 as part of the project. The work was carried out in accordance with the Quality Control Plan for this project dated July 2003. A digital file of the site plan and bridge general arrangement drawing was provided to Golder by Morrison Hershfield in January 2004.

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

- "Foundation Investigation for a Culvert, at the Credit River, Highway 10 Crossing, approximately four miles north of Caledon, Ontario", GEOCRE No. 40P-16-02, prepared by Racey, MacCallum and Associates Limited, dated November 5, 1956;
- "MTO Preliminary Design Report", prepared by URS Cole, Sherman & Associates, dated July 2002.

## 2.0 SITE DESCRIPTION

The South Branch Credit River site is located at the north side of the intersection between Highway 10 and Highpoint Sideroad North (see key plan on Contract Drawings). The existing bridge is a single span reinforced concrete rigid frame structure, with a span of about 12.2 m between abutments.

The bridge spans over the Credit River which flows in a east to west direction near the site. The terrain at the site is generally flat, with the exception of the existing raised bridge approach embankments and elevated existing highway grade that ranges from about Elevation 406 m to 407 m. The ground surface adjacent to the existing bridge embankment within the surrounding low-lying swampy area is at about Elevation 402 m.

There is a private entrance and driveway located at the southeast corner of the site and the intersection of Highway 10 and Highpoint Sideroad North is located at the southwest corner of

the bridge site. Vacant and flat low-lying land is present in the northeast and northwest areas of the site. Primary utility lines are located within the vicinity of the site.

Drainage channels are located along both sides of Highway 10 which discharge to the Credit River. On the south side of the bridge, drainage culverts run beneath an existing driveway on the east side and under Highpoint Sideroad North on the west side that drain stormwater into the Credit River.

### **3.0 INVESTIGATION PROCEDURES**

#### **3.1 Foundation Investigation**

The field work at the South Bridge site was carried out between September 23 and December 4, 2003, at which time four (4) boreholes, numbering SB1, SB2, SB3, and SB4 were advanced at the locations shown on the Contract Drawings. Two boreholes, numbered 56-H2 and 56-H3 were advanced as part of a previous investigation at the site carried out by Racey, MacCallum, and Associates Ltd. in 1956. The locations of these boreholes are also shown in plan on the Contract Drawings.

The current field investigation was carried out using track-mounted CME 55 drill rigs supplied and operated by two companies, namely Geo-Environmental Drilling Ltd. of Milton, Ontario in October 2003 and Groundwork Drilling Inc. of Etobicoke, Ontario in 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 intervals ranging from 0.75 m to 3.0 m in depth, using a 50 mm (O.D.) split-spoon sampler in accordance with Standard Penetration Test (SPT) procedures.

The boreholes were advanced to depths ranging from 8.1 m to 24.5 m below the existing ground surface. The groundwater conditions in the open boreholes were observed during the drilling operations, and piezometers were installed in select boreholes to permit more long-term monitoring of the groundwater levels at these locations. The piezometers consist of a 25 mm outside diameter solid PVC pipe, with a slotted screen sealed at a select 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 supervised by members of our engineering and technical staff, who located the boreholes, arranged for the clearance of underground service locations, supervised the drilling, sampling and in-situ testing operations, logged the boreholes, and examined and cared for the soil samples. The samples were identified in the field, placed in appropriate containers,

labelled and transported to our Mississauga geotechnical laboratory where the samples underwent further detailed 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 personnel 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 (Chapman and Putnam, "The Physiography of Southern Ontario", 3<sup>rd</sup> 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 Hillburgh. 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 as consisting 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. Furthermore, subsurface conditions will vary between and beyond the borehole locations. The inferred soil stratigraphy based on the results of the boreholes within the vicinity of the Credit River Bridge – South Branch location are shown on the Contract Drawings.

The surficial soils at the South Bridge 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 underlain by peat and organic silty sand. The fill, topsoil, peat, and organic silty sand is underlain by a sand to sand and gravel layer, which is typically underlain by a deposit of silt with clayey silt interlayers. These deposits are underlain by a layer of sand and gravel to gravel containing cobbles/boulders at some locations. These deposits are underlain by interlayered sandy silt, silt, and clayey silt which is underlain by a dense to very dense interlayered silt and sand, and sand and gravel containing cobbles/boulders. These deposits are underlain at depth by

very dense silt with hard clayey silt interlayers. 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 SB1, SB2, and SB4, which were advanced near the top or toe of the bridge approach embankment. The fill typically consisted of silty sand, sand, to sand and gravel. The surface of the fill ranged between Elevations 402.4 m and 407.4 m and the thickness varied from 0.9 m to 4.9 m.

Standard Penetration Testing (SPT) 'N' values recorded within the fill ranged between 4 and 26 blows per 0.3 m of penetration, with all but two (2) of the recorded SPT 'N' values in the range of 4 to 14, indicating a loose to compact relative density.

Natural water contents measured on samples of the fill ranged between 9 and 20 percent. A grain size distribution curve for a selected sample of the fill deposit located near the interface of the underlying peat layer is shown on Figure A1.

#### **4.2.2 Topsoil / Peat / Organic Silty Sand**

Topsoil or peat was encountered at the existing ground surface in Boreholes SB3, 56-H2, and 56-H3 and underlying the fill in Boreholes SB1, SB2 and SB4. In Boreholes SB3 and SB4, the topsoil and peat transitioned to an organic silty sand containing rootlets. The top of the topsoil/peat and organic silty sand deposit ranged between Elevation 401.5 m to 403.4 m, with the thickness varying from 0.8 m to 3.7 m.

Standard Penetration Testing (SPT) 'N' values recorded within the topsoil and peat layers ranged from 2 blows to 10 blows per 0.3 m of penetration, indicating a very soft to stiff consistency. SPT 'N' values recorded within the organic silty sand were similarly between 2 and 4 blows per 0.3 m of penetration, indicating a very loose to loose relative density.

The natural water content measured on a samples of the topsoil/peat/organic silty sand varied between 24 and 48 percent.

#### **4.2.3 Upper Sand / Sand and Gravel**

Underlying the fill, peat, topsoil, and organic silty sand, a layer of brown sand to sand and gravel with trace silt was encountered in Boreholes SB1, SB2, SB3, SB4, and 56-H2. The top of the sand to sand and gravel layer was at a depth ranging between 2.0 m and 6.4 m. The top this layer ranged between Elevations 399.6 m and 401.0 m and the thickness varied from 0.7 m to 1.4 m.

Borehole SB2 encountered another sand deposit at a depth of 7.9 m (Elevation 397.7 m) and was terminated within the sand at a depth of 8.1 m (Elevation 397.5 m).

Standard Penetration Testing (SPT) 'N' values recorded within the upper sand to sand and gravel layer ranged between 6 blows and 9 blows per 0.3 m of penetration, indicating a loose state of packing.

The natural water content measured on two samples of the sand to sand and gravel layer was 15 percent.

#### **4.2.4 Upper Silt**

A layer of silt containing interlayers of predominantly clayey silt and sandy silt was encountered in Boreholes SB3, SB4, 56-H2 and 56-H3. The grey silt contained interlayers of gravel and cobbles in Borehole SB4. The top of this silt deposit was encountered at a depth ranging between 3.0 m and 3.7 m below existing ground surface. The top of this deposit ranged between Elevations 398.9 m and 399.7 m with the thickness varying from 2.0 m to 4.5 m.

Standard Penetration Testing (SPT) 'N' values recorded within the silt layer generally ranged from 4 to 18 blows per 0.3 m of penetration, indicating a very loose to compact relative density.

The natural water content measured on samples of the silt layer ranged from 22 to 23 percent, with one value of 11 percent measured within a sample containing gravel interlayers. A grain size distribution curve for a selected sample of the upper silt layer is shown on Figure A2.

#### **4.2.5 Sand and Gravel / Gravel**

Underlying the upper silt and upper sand deposits, a layer of sand and gravel to gravel was encountered at Boreholes SB2, SB3, 56-H2, and 56-H3. The sand and gravel/gravel layer typically contained cobbles and boulders, trace silt and clay. The top of the sand and gravel to gravel layer was encountered at a depth ranging from 5.0 m to 7.3 m below ground surface. The top of the sand and gravel to gravel layer ranged from Elevations 396.0 m to 399.5 m and the thickness varied between 2.0 m and 2.1 m.

Standard Penetration Testing (SPT) 'N' values recorded within the sand and gravel to gravel layer ranged between 11 blows and 80 blows per 0.3 m of penetration, indicating a compact to very dense state of packing. The higher blow counts may be attributed to cobbles and boulders encountered within the deposit.

The natural water content measured on samples of the sand and gravel to gravel layer ranged between 6 and 19 percent.

#### **4.2.6 Clayey Silt**

A deposit of brown clayey silt with trace sand was encountered below the upper sand layer in Borehole SB1. The top of this clayey silt deposit was 8.8 m below ground surface (Elevation 399.8 m) and drilling was terminated within this deposit at a depth of 8.8 m (Elevation 398.5). A Dynamic Cone Penetration Test (DCPT) was carried out from a depth of 8.8 m to 9.9 m (Elevation 397.5 m) at which depth effective refusal to advance the cone was achieved.

A Standard Penetration Test (SPT) 'N' value recorded within the clayey silt deposit was 3 blows per 0.3 m of penetration, indicating a very soft consistency. The DCPT values ranged from 34 blows per 0.3 m of penetration to 150 blows / 0.2 m of penetration of the cone tip. The increasing number of blows with depth from the DCPT results suggest the clayey silt deposit transitions to a very stiff to hard consistency.

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

#### **4.2.7 Interlayered Sandy Silt / Silt / Clayey Silt**

Interlayered sandy silt, silt, and clayey silt was encountered in Boreholes SB3, SB4, 56-H2, and 56-H3. The grey to brown interlayered silty deposit typically contained trace to some sand and gravel with cobbles/boulders near the interface with the overlying gravel layer. The top of this interlayered silty deposit was encountered at 7.0 to 9.6 m below ground surface. The top of this deposit ranged between Elevations 393.9 m and 394.9 m and the thickness was 6.1 m and 8.2 m for Boreholes SB4 and SB3 respectively. Boreholes 56-H2 and 56-H3 were terminated within the interlayered silty deposit at depths of 12.8 m (Elevation 389.6 m) and 12.2 m (Elevation 391.2 m) recording thicknesses of 5.8 m and 3.4 m respectively.

Standard Penetration Testing (SPT) 'N' values recorded within the interlayered silty deposit ranged between 10 blows and 53 blows per 0.3 m of penetration. SPT measured 'N' values within the sandy silt and silt interlayers ranged between 10 and 53 blows, indicating a compact to very dense state of packing. SPT measured 'N' values within the clayey silt interlayers ranged between 19 and 31 blows, indicating a very stiff to hard consistency.

The natural water content measured on samples of the sandy silt, silt, and clayey silt interlayers varied between 19 and 25 percent.

The results of Atterberg limits testing carried out on a sample of the clayey silt interlayer are illustrated on the plasticity chart on Figure A5 in Appendix A. The test results are summarized in the following table.

<i>Borehole</i>	<i>Sample</i>	<i>Elevation (m)</i>	<i>Liquid Limit (%)</i>	<i>Plastic Limit (%)</i>	<i>Plasticity Index (%)</i>
SB4	10	392.8-393.3	24	16	8

The test results on this sample within the clayey silt interlayer indicate it is of low plasticity.

#### **4.2.8 Interlayered Silt and Sand / Sand and Gravel**

A deposit of interlayered silt and sand to sand and gravel was encountered in the two deep boreholes, Boreholes SB3 and SB4, put down during the current investigation. The interlayers of brown silt and sand typically contained variable amounts of gravel, cobbles and boulders. The sand and gravel interlayers typically contained cobbles/boulders and trace silt.

The top of the interlayered silt and sand to sand and gravel deposit was at a depth of 13.7 m (Elevation 388.7 m) and 15.2 m (Elevation 386.7 m) in Boreholes SB4 and SB3 respectively. The interlayered silt and sand to sand and gravel deposit was 3.1 m and 6.1 m thick in Boreholes SB4 and SB3 respectively.

Standard Penetration Testing (SPT) 'N' values recorded within the interlayered silt and sand to sand and gravel typically ranged between 75 blows and over 100 blows per 0.3 m of penetration, indicating a very dense state of packing. One SPT 'N' value of 30 blows per 0.3 m of penetration was measured within an interlayer of silt and sand in Borehole SB4.

The natural water content measured on samples of the interlayered silt and sand to sand and gravel typically ranged between 8 and 12 percent, with one value measured at 23 percent. A grain size distribution curve for a selected sample of an interlayer of sand and gravel is shown on Figure A3.

#### **4.2.9 Lower Silt / Clayey Silt**

Below the interlayered silt and sand to sand and gravel, a deposit of interlayered silt and clayey silt was encountered in Boreholes SB3 and SB4. The grey silt and clayey silt deposit typically contained trace to some sand and contained seams of gravel and sand. The top of this silt and clayey silt layer was at 21.3 m and 16.8 m below ground surface in Boreholes SB3 and SB4 respectively. The top of this deposit was at Elevations 380.6 m and 385.6 m for Boreholes SB3 and SB4 respectively. Boreholes SB3 and SB4 were terminated within the silt deposit at depths of 24.5 m (Elevation 377.4 m) and 21.8 m (Elevation 380.6 m) respectively.

Standard Penetration Testing (SPT) 'N' values recorded within the lower silt/clayey silt layer ranged from 89 blows per 0.3 m of penetration to over 100 blows per 0.1 m of penetration, indicating a very dense to hard consistency.

The natural water content measured on samples of the lower silt to clayey silt deposits varied between 17 and 24 percent. A grain size distribution curve for a selected sample of the clayey silt is shown on Figure A4.

#### **4.2.10 Groundwater Conditions**

Water levels were noted within the boreholes during and after the drilling operations. Piezometers were installed in Boreholes SB3 and SB4 to permit monitoring of water levels. Both piezometers were sealed within the lower interlayered silt/clayey silt deposit; the piezometer in Borehole SB3 was sealed in the interlayered silt and sand to sand and gravel deposit. The piezometer in Borehole SB4 was sealed within the lower silt and clayey silt 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 in the table below.

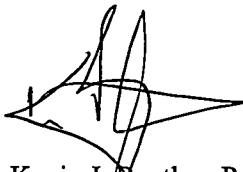
<i>Borehole</i>	<i>Ground Surface Elevation (m)</i>	<i>Water Level Depth (m)</i>	<i>Water Level Elevation (m)</i>	<i>Date</i>
SB3	401.9	0.20	401.7	January 7, 2004
SB4	402.4	3.75	398.7	January 7, 2004

The water level in Borehole SB3 was found to be at the same level as the Credit River in the vicinity of the South Bridge structure, which was measured to be at Elevation 401.7 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. The water level measured in the piezometer in Borehole SB4 may reflect a slower response time within the finer grained deposits or the level may not be stabilized. It should also be noted that groundwater levels in the area are subject to seasonal fluctuations.

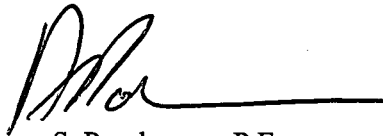
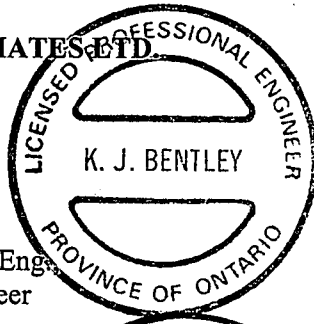
#### 4.3 Closure

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 Mr. Fintan J. Heffernan, P.Eng, a Designated MTO Contact for Golder.

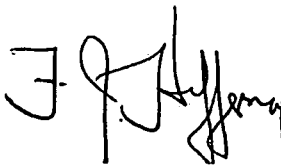
**GOLDER ASSOCIATES LTD.**



Kevin J. Bentley, P.Eng.  
Geotechnical Engineer



Anne S. Poschmann, P.Eng.,  
Principal



Fintan J. Heffernan, P.Eng.  
Designated MTO Contact



KJBASP/FJH/al

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## 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

	kPa	psf
Very soft	0 to 12	0 to 250
Soft	12 to 25	250 to 500
Firm	25 to 50	500 to 1,000
Stiff	50 to 100	1,000 to 2,000
Very stiff	100 to 200	2,000 to 4,000
Hard	over 200	over 4,000

#### Dynamic Cone Penetration Resistance; $N_d$ :

The number of blows by a 63.5 kg (140 lb.) hammer dropped 760 mm (30 in.) to drive uncased a 50 mm (2 in.) diameter, 60° cone attached to "A" size drill rods for a distance of 300 mm (12 in.).

**PH:** Sampler advanced by hydraulic pressure

**PM:** Sampler advanced by manual pressure

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

**WR:** Sampler advanced by weight of sampler and rod

#### Piezo-Cone Penetration Test (CPT)

A electronic cone penetrometer with a 60° conical tip and a project end area of 10 cm<sup>2</sup> pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance ( $Q_t$ ), porewater pressure (PWP) and friction along a sleeve are recorded electronically at 25 mm penetration intervals.

### IV. SOIL TESTS

w water content  
w<sub>p</sub> plastic limit  
w<sub>i</sub> 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<sub>R</sub> relative density (specific gravity, G<sub>s</sub>)  
DS direct shear test  
M sieve analysis for particle size  
MH combined sieve and hydrometer (H) analysis  
MPC Modified Proctor compaction test  
SPC Standard Proctor compaction test  
OC organic content test  
SO<sub>4</sub> concentration of water-soluble sulphates  
UC unconfined compression test  
UU unconsolidated undrained triaxial test  
V field vane (LV-laboratory vane test)  
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:

<b>I. General</b>		<b>(a) Index Properties (continued)</b>	
$\pi$	3.1416	w	water content
ln x,	natural logarithm of x	$w_L$	liquid limit
$\log_{10}$	x or log x, logarithm of x to base 10	$w_p$	plastic limit
g	acceleration due to gravity	$I_p$	plasticity index = $(w_L - w_p)$
t	time	$w_s$	shrinkage limit
F	factor of safety	$I_L$	liquidity index = $(w - w_p)/I_p$
V	volume	$I_C$	consistency index = $(w_L - w)/I_p$
W	weight	$e_{max}$	void ratio in loosest state
<b>II. STRESS AND STRAIN</b>		$e_{min}$	void ratio in densest state
$\gamma$	shear strain	$I_D$	density index = $(e_{max} - e)/(e_{max} - e_{min})$ (formerly relative density)
$\Delta$	change in, e.g. in stress: $\Delta \sigma$	<b>(b) Hydraulic Properties</b>	
$\epsilon$	linear strain	h	hydraulic head or potential
$\epsilon_v$	volumetric strain	q	rate of flow
$\eta$	coefficient of viscosity	v	velocity of flow
$\nu$	Poisson's ratio	i	hydraulic gradient
$\sigma$	total stress	k	hydraulic conductivity (coefficient of permeability)
$\sigma'$	effective stress ( $\sigma' = \sigma - u$ )	j	seepage force per unit volume
$\sigma'_{vo}$	initial effective overburden stress	<b>(c) Consolidation (one-dimensional)</b>	
$\sigma_1, \sigma_2, \sigma_3$	principal stress (major, intermediate, minor)	$C_c$	compression index (normally consolidated range)
$\sigma_{oct}$	mean stress or octahedral stress = $(\sigma_1 + \sigma_2 + \sigma_3)/3$	$C_r$	recompression index (over-consolidated range)
$\tau$	shear stress	$C_s$	swelling index
u	porewater pressure	$C_a$	coefficient of secondary consolidation
E	modulus of deformation	$m_v$	coefficient of volume change
G	shear modulus of deformation	$c_v$	coefficient of consolidation
K	bulk modulus of compressibility	$T_v$	time factor (vertical direction)
<b>III. SOIL PROPERTIES</b>		U	degree of consolidation
<b>(a) Index Properties</b>		$\sigma'_p$	pre-consolidation pressure
$\rho(\gamma)$	bulk density (bulk unit weight*)	OCR	over-consolidation ratio = $\sigma'_p/\sigma'_{vo}$
$\rho_d(\gamma_d)$	dry density (dry unit weight)	<b>(d) Shear Strength</b>	
$\rho_w(\gamma_w)$	density (unit weight) of water	$\tau_p, \tau_r$	peak and residual shear strength
$\rho_s(\gamma_s)$	density (unit weight) of solid particles	$\phi'$	effective angle of internal friction
$\gamma'$	unit weight of submerged soil ( $\gamma' = \gamma - \gamma_w$ )	$\delta$	angle of interface friction
$D_R$	relative density (specific gravity) of solid particles ( $D_R = \rho_s/\rho_w$ ) (formerly $G_s$ )	$\mu$	coefficient of friction = $\tan \delta$
e	void ratio	$c'$	effective cohesion
n	porosity	$c_{u, S_u}$	undrained shear strength ( $\phi = 0$ analysis)
S	degree of saturation	p	mean total stress $(\sigma_1 + \sigma_3)/2$
*	Density symbol is $\rho$ . Unit weight symbol is $\gamma$ where $\gamma = \rho g$ (i.e. mass density x acceleration due to gravity)	$p'$	mean effective stress $(\sigma'_1 + \sigma'_3)/2$
		q	$(\sigma_1 + \sigma_3)/2$ or $(\sigma'_1 + \sigma'_3)/2$
		$q_u$	compressive strength $(\sigma_1 + \sigma_3)$
		$S_i$	sensitivity

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

<b>PROJECT</b> 03-1111-023		<b>RECORD OF BOREHOLE No SB1</b>		1 OF 1 <b>METRIC</b>	
<b>W.P.</b> 27-97-00		<b>LOCATION</b> N 4862338.9; E 260429.6		<b>ORIGINATED BY</b> GD	
<b>DIST</b> HWY 10		<b>BOREHOLE TYPE</b> POWER AUGERING USING 108 mm O.D. SOLID STEM AUGERS		<b>COMPILED BY</b> KG	
<b>DATUM</b> Geodetic		<b>DATE</b> December 04, 2003		<b>CHECKED BY</b> KB	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE LIQUID LIMIT CONTENT			UNIT WEIGHT  γ  kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		WATER CONTENT (%)				
								○ UNCONFINED ● QUICK TRIAXIAL	+ FIELD VANE × REMOULDED	W <sub>p</sub>	W	W <sub>L</sub>		
407.4	GROUND SURFACE													
0.0	Sand, trace to some gravel, trace silt and organics, occasional silty clay seams (FILL) Loose Brown Moist to wet		1	SS	4									
			2	SS	8									
			3	SS	5									
			4	SS	14									
			5	SS	9									
			6	SS	7									
402.5			7	SS	4									
4.9	PEAT Soft to firm Dark brown Moist													
401.0			8	SS	9									
6.4	SAND, some gravel, trace silt and organics Loose Brown Wet													
399.8			9	SS	3									
7.6	CLAYEY SILT, trace sand Soft Brown Wet													
398.5														
8.8	End of Drilling and beginning of Dynamic Cone Penetration Test													
397.5														
9.9	End of Penetration Test upon refusal at 9.91m depth  Notes:  1. Borehole caved to a depth of 3.81m below ground surface.													

150 blows for last 150mm  
Refusal-50 blows/0mm

<b>PROJECT</b> 03-1111-023		<b>RECORD OF BOREHOLE No SB2</b>		1 OF 1 <b>METRIC</b>	
<b>W.P.</b> 27-97-00		<b>LOCATION</b> N 4862360.7 ; E 260379.2		<b>ORIGINATED BY</b> GD	
<b>DIST</b> _____ <b>HWY</b> 10		<b>BOREHOLE TYPE</b> POWER AUGERING USING 108 mm O.D. SOLID STEM AUGERS		<b>COMPILED BY</b> KG	
<b>DATUM</b> Geodetic		<b>DATE</b> December 04, 2003		<b>CHECKED BY</b> KB	

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									
								SHEAR STRENGTH kPa									
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED									
							WATER CONTENT (%)					kN/m <sup>3</sup>	GR SA SI CL				
405.6	GROUND SURFACE																
0.0	Sand and gravel, trace silt, occasional rootlets and sandy silt seams (FILL) Loose to compact Brown Moist		1	SS	7		405										
			2	SS	23												
404.1							404										
1.5	Sand to silty sand, trace to some clay, trace gravel and organics (FILL) Compact to loose Brown Moist		3	SS	14												
			4	SS	9		403										
			5	SS	26		402										
401.5			6	SS	14												
4.1	PEAT, trace gravel and wood fragments Firm Dark brown Moist		7	SS	6		401										
400.7																	
4.9	SAND, trace silt Loose Grey Wet						400										
399.5																	
6.1	SAND AND GRAVEL, trace silt Loose to compact Brown Wet		8	SS	11		399										
397.7			9	SS	14		398										
8.1	SAND, some silt, trace gravel Compact Brown Wet End of Borehole  Notes:  1. Borehole caved to a depth of 5.2m below ground surface.																

MIS-MTO 001 031111023AAGDR.GPJ GAL-MISS.GDT 3/5/07

**RECORD OF BOREHOLE No SB3**

1 OF 2 **METRIC**

PROJECT 03-1111-023

W.P. 27-97-00

LOCATION N 4862370.9 ; E 260408.1

ORIGINATED BY GD

DIST HWY 10

BOREHOLE TYPE POWER AUGERING USING 108 mm O.D. SOLID STEM AUGERS AND CASING COMPILED BY KG

DATUM Geodetic

DATE September 23, 2003

CHECKED BY KJB

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						
								SHEAR STRENGTH kPa						
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED						
							20 40 60 80 100		WATER CONTENT (%)			kN/m <sup>3</sup>	GR SA SI CL	
401.9	GROUND SURFACE													
0.0	TOPSOIL Very loose Brown Wet		1	SS	3									
401.1														
0.8	ORGANIC SILTY SAND, occasional rootlets Very loose to loose Grey Wet		2	SS	4		401							
			3	SS	4		400							
399.6														
2.3	SAND, some gravel, trace silt Loose Grey Wet		4	SS	7		399							
398.9														
3.0	SILT, some sand, trace clay, contains clayey silt and sandy silt interlayers Loose to very loose Grey Wet		5	SS	9		398							
			6	SS	7									
396.9			7	SS	4		397							
5.0	SAND AND GRAVEL, trace silt and clay, contains cobbles and boulders Dense to very dense Grey Wet						396							
394.9			8	SS	47		395							
7.0	CLAYEY SILT, trace sand and gravel Hard Grey Wet						394							
393.4														
8.5	SILT, some sand Compact Brown Wet		9	SS	24		393							
391.2							392							
10.7	CLAYEY SILT, trace sand, occasional gravel interlayers Very stiff Brown Wet		10	SS	22		391							
389.7							390							
12.2	SANDY SILT, trace clay, contains interlayers of clayey silt Compact Brown Wet		11	SS	26		389							
							388							
			12	SS	26		387							

Continued Next Page

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

MIS-MTO 001 031111023AAGDR.GPJ GAL-MISS.GDT 3/5/07

PROJECT <u>03-1111-023</u>		<b>RECORD OF BOREHOLE No SB3</b>		2 OF 2 <b>METRIC</b>	
W.P. <u>27-97-00</u>		LOCATION <u>N 4862370.9 ; E 260408.1</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 COMPILED BY</u>		KG	
DATUM <u>Geodetic</u>		DATE <u>September 23, 2003</u>		CHECKED BY <u>KJB</u>	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ KN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)
								○ UNCONFINED	+ FIELD VANE						
— CONTINUED FROM PREVIOUS PAGE —															
386.7 15.2	Probable cobble/boulder encountered at 14.9 m SILT AND SAND with gravel, cobbles, and boulders Very dense Brown Wet		13	SS	75		386								
								385							
								384							
384.2 17.7	SAND AND GRAVEL with cobbles/boulders Very dense brown Wet						383								
								382							
								381							
380.6 21.3	SILT, trace to some sand, occasional clayey silt interlayers, contains sand and gravel seams. Very dense Grey Moist		14	SS	50/0.1		380								
								379							
								378							
377.4 24.5	End of Borehole		15	SS	50/0.15										
	Note: 1. Solid stem augers used to drill to a depth of 5 m. NW casing used to drill from 5.0m to a depth of 15.8 m. NQ sized core barrel was used to drill through a boulder from 15.8 m to 16.8 m. NW casing used to drill to the end of the borehole to a depth of 24.5 m.  2. Water level measured at 0.2m (EL. 401.7m) below ground surface on January 7, 2004.														

**RECORD OF BOREHOLE No SB4**

1 OF 2 **METRIC**

PROJECT 03-1111-023

W.P. 27-97-00

LOCATION N 4862335.9 ; E 260392.5

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 1, 2003

CHECKED BY KB

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					
402.4	GROUND SURFACE						402							
0.0	Silt and sand, trace gravel and clay, contains rootlets (FILL) Very loose Brown Moist		1	SS	4									
401.5														
0.9	PEAT TO ORGANIC SILTY SAND, occasional sand seams Very soft to soft Dark brown Moist		2	SS	3		401							1 54 40 5
			3	SS	2									
400.1														
2.3	SAND AND GRAVEL, trace silt Loose Grey Wet		4	SS	6		400							
399.4														
3.1	SILT, trace to some clay and sand, occasional cobbles Loose Grey Wet		5	SS	7		399							
			6	SS	4		398							
			7	SS	4		397							
396.3														
6.1	SILT, some sand, clay, and gravel, contains gravel interlayers Very loose to loose Grey Wet		8	SS	4		396							17 12 58 13
394.8							395							
7.6	CLAYEY SILT, trace to some sand, trace gravel, occasional silt seams Very stiff to hard Grey Moist		9	SS	26		394							
			10	SS	31		393							
391.7							392							
10.7	SILT, some clay, trace sand Loose to compact Grey Wet		11	SS	10		391							
390.2														
12.2	CLAYEY SILT, some sand, trace gravel, occasional silt seams Very stiff Brown Wet		12	SS	19		390							
388.7							389							
13.7	SAND AND GRAVEL, trace silt, occasional boulders/cobbles Very dense Brown Wet		13	SS	50/0.15		388							

Continued Next Page

+<sup>3</sup>, ×<sup>3</sup>

Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

MIS-MTO 001 031111023AAGDR.GPJ GAL-MISS.GDT 3/5/07

PROJECT <u>03-1111-023</u>		<b>RECORD OF BOREHOLE No SB4</b>		2 OF 2 <b>METRIC</b>	
W.P. <u>27-97-00</u>		LOCATION <u>N 4862335.9 ; E 260392.5</u>		ORIGINATED BY <u>GD</u>	
DIST <u>HWY 10</u>		BOREHOLE TYPE <u>POWER AUGERING USING 108 mm I.D. HOLLOW STEM AUGERS</u>		COMPILED BY <u>KG</u>	
DATUM <u>Geodetic</u>		DATE <u>December 1, 2003</u>		CHECKED BY <u>KB</u>	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x REMOULDED		WATER CONTENT (%) W <sub>p</sub> — W — W <sub>L</sub>				
387.2	— CONTINUED FROM PREVIOUS PAGE —													
15.2	SAND AND SILT Dense Grey Wet		14	SS	30									
385.6														
16.8	SILT, trace to some sand, occasional clayey silt interlayers Very dense Grey Wet to moist		15	SS	89									
			16	SS	113									
382.4														
20.0	CLAYEY SILT, occasional silt seams Hard Grey Wet to moist		17	SS	113									0 2 75 23
381.1														
21.3	SILT, trace sand Very dense Grey		18	SS	118									
380.6	Moist													
21.8	End of Borehole													
	Notes:  1. Water level in piezometer measured at 3.75m below ground surface (Elev. 398.7 m) on January 7, 2004.													

MIS-MTO 001 031111023AAGDR.GPJ GAL-MISS.GDT 3/5/07

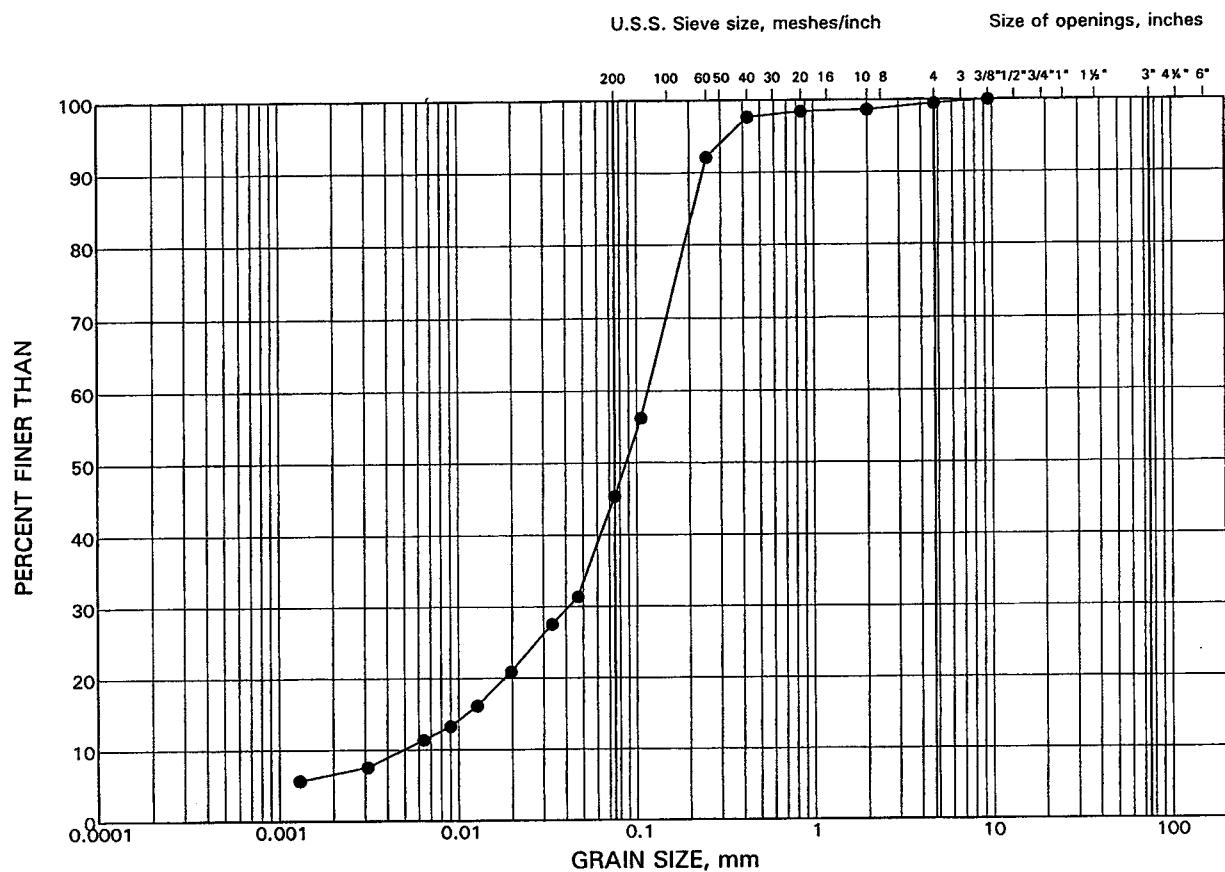
**APPENDIX A**  
**LABORATORY TEST DATA**



# GRAIN SIZE DISTRIBUTION

## Silt and Sand (Fill)

FIGURE A1



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

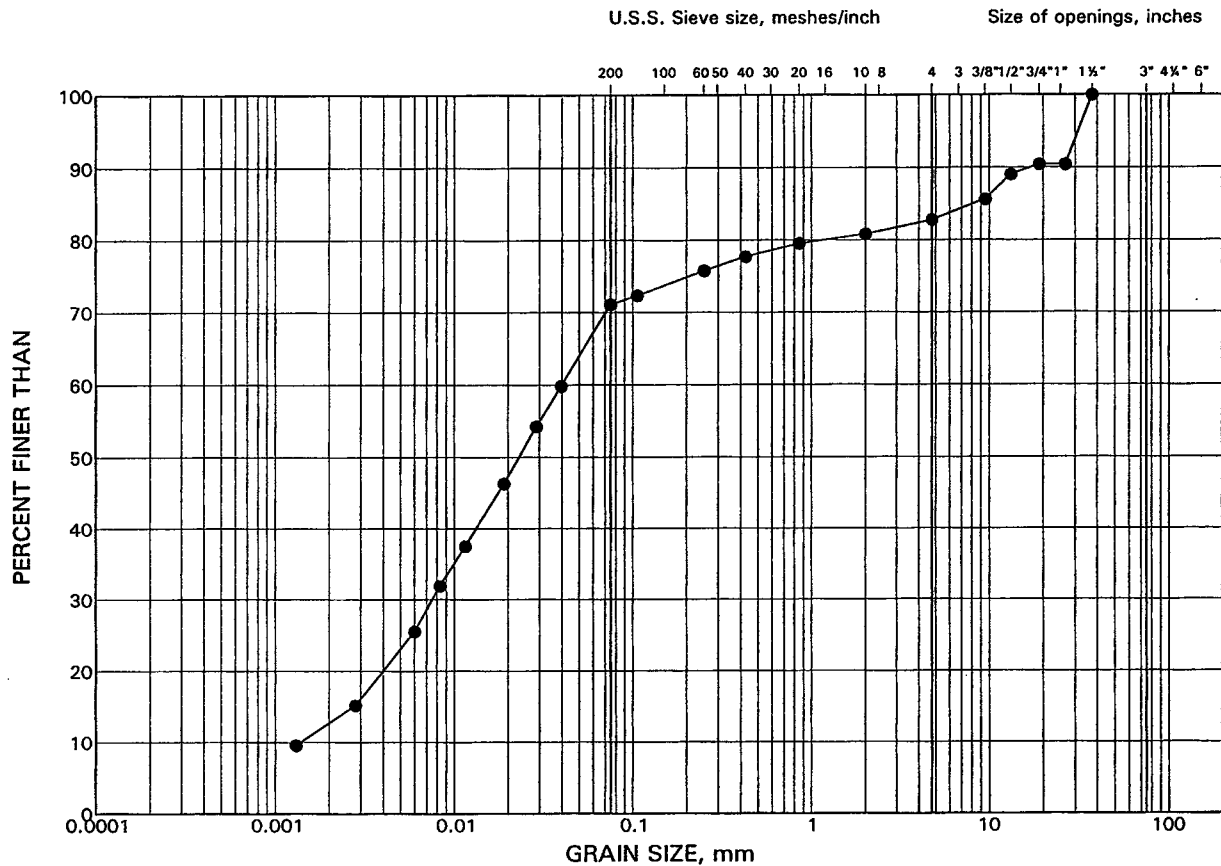
### LEGEND

SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)
•	SB4	2	0.8-0.9

# GRAIN SIZE DISTRIBUTION

Silt, some clay, sand and gravel

FIGURE A2



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

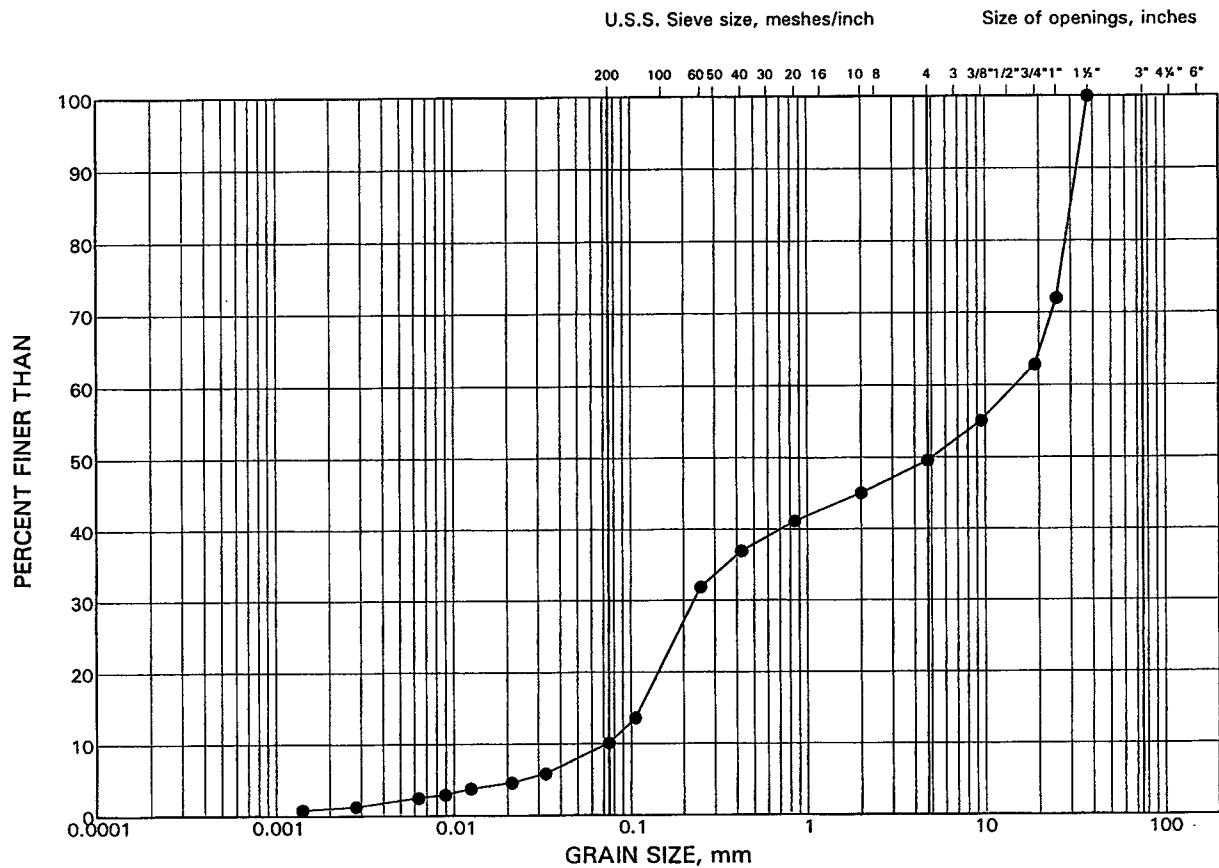
## LEGEND

SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)
•	SB4	8	6.1-6.6

# GRAIN SIZE DISTRIBUTION

## Sand and Gravel

FIGURE A3



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

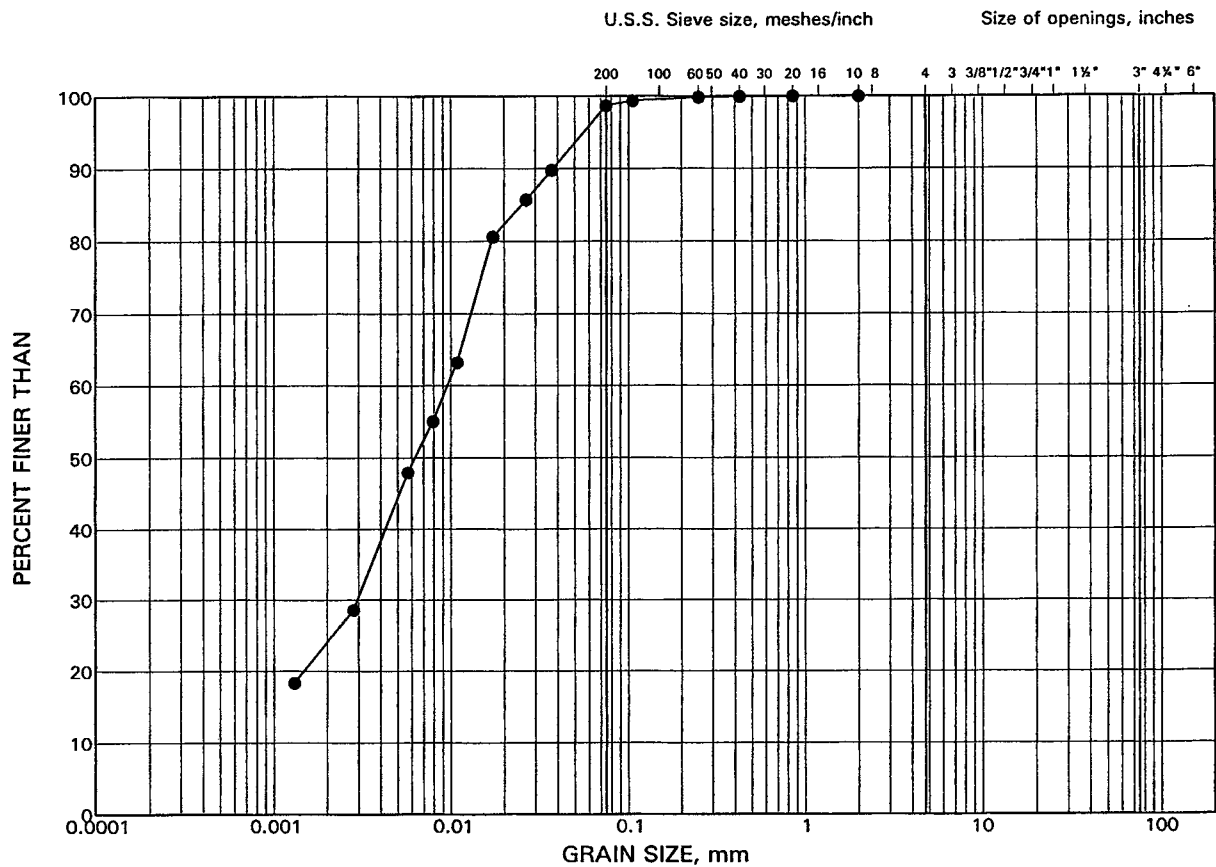
### LEGEND

SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)
•	SB3	14	19.8-20.0

# GRAIN SIZE DISTRIBUTION

Clayey Silt

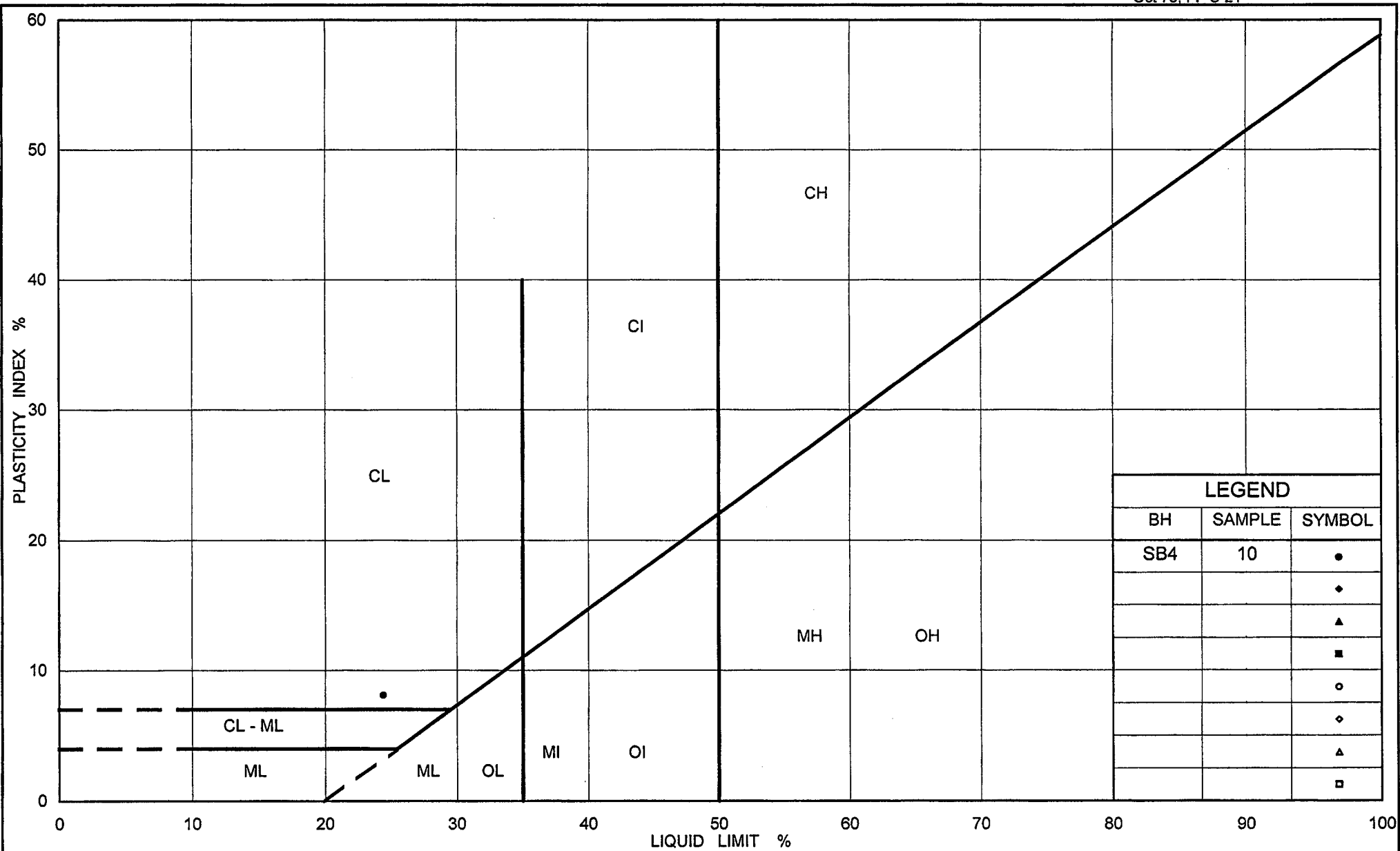
FIGURE A4



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

## LEGEND

SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)
•	SB4	17	19.8-20.3



Ministry of Transportation

Ontario

# PLASTICITY CHART Clayey Silt

FIG No. A5

Project No. 03-1111-023

Checked By: **KSB**

**APPENDIX B**  
**RECORD OF BOREHOLES (56-H2, 56-H3)**  
**PREVIOUS INVESTIGATION**

Hole Begun \_\_\_\_\_ Foundation Engineering Division

Hole Ended \_\_\_\_\_ Engineering Data Sheet for Borehole: 56-H2

Helper \_\_\_\_\_

Job Name: Credit River Bridge ~ Sta 515+50 Hwy 10

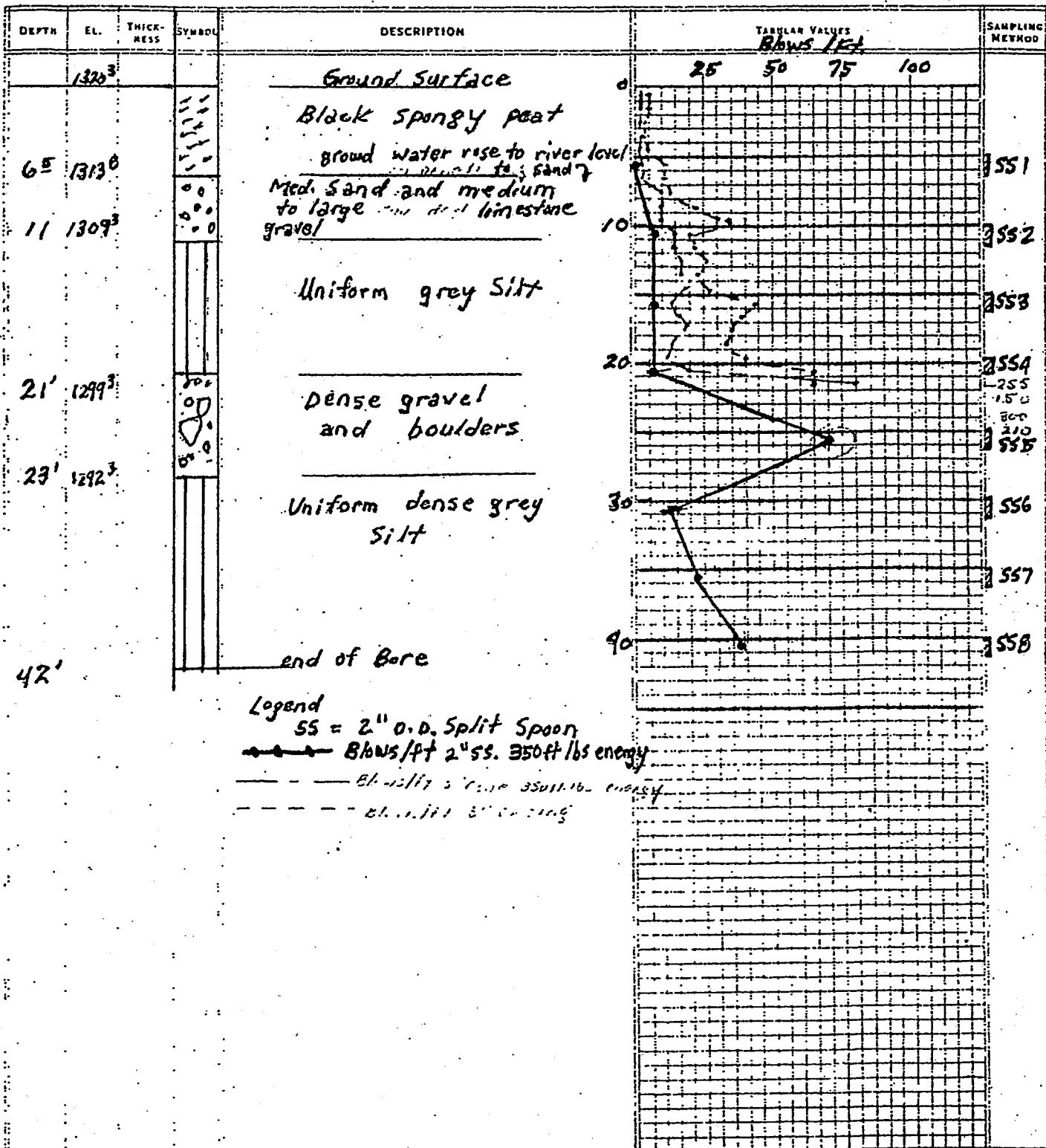
Job Located: Approx. 4 miles North Caledon Ont

Checked by \_\_\_\_\_

Hole Located: See encl. No. 1

Hole Elevation: 1320<sup>3</sup> Datum: Geodetic El. 1331.9'

Day \_\_\_\_\_ Month \_\_\_\_\_ Year \_\_\_\_\_



Order No.: S500/T487 RACEY, MACCALLUM AND ASSOCIATES

LIMITED

Driller

Hole Begun

Foundation Engineering Division

Hole Ended

Engineering Data Sheet for Borehole: 56-H3

Helper

Job Name: Credit River Bridge  $\approx$  Sta 515+50 Hwy. 10Job Located: Approx 4 mi North Galedon, Ont

Checked by

Hole Located: See encl. No. 1Hole Elevation: 1323.4 Datum: Geodetic E 1331.91

Day Month Year

