



**AUGUST 2009**

## **PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT**

**RAINBOW CREEK BRIDGES (NBL AND SBL)  
HIGHWAY 427 EXTENSION  
FROM HIGHWAY 7 TO MAJOR MACKENZIE DRIVE  
MINISTRY OF TRANSPORTATION, ONTARIO  
W.O. 05-20012**

**Submitted to:**  
McCormick Rankin Corporation  
2655 North Sheridan Way  
Mississauga, Ontario  
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**REPORT**



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# PRELIMINARY FOUNDATION REPORT RAINBOW CREEK BRIDGES - HIGHWAY 427 EXTENSION

## Table of Contents

### PART A – PRELIMINARY FOUNDATION INVESTIGATION REPORT

<b>1.0 INTRODUCTION .....</b>	<b>10</b>
<b>2.0 SITE DESCRIPTION .....</b>	<b>10</b>
<b>3.0 INVESTIGATION PROCEDURES .....</b>	<b>10</b>
<b>4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS .....</b>	<b>12</b>
4.1 Regional Geology .....	12
4.2 Subsurface Conditions .....	12
4.2.1 Topsoil .....	12
4.2.2 Surficial Clayey Silt .....	12
4.2.3 Surficial Silty Sand .....	13
4.2.4 Till Deposit .....	13
4.2.4.1 Clayey Silt Till .....	13
4.2.4.2 Sand and Silt Till .....	14
4.3 Groundwater Conditions .....	14
<b>5.0 CLOSURE .....</b>	<b>15</b>

### PART B – PRELIMINARY FOUNDATION DESIGN REPORT

<b>6.0 ENGINEERING RECOMMENDATIONS FOR PRELIMINARY DESIGN .....</b>	<b>7</b>
6.1 General .....	7
6.2 Foundation Recommendations .....	7
6.2.1 Foundation Options .....	7
6.2.2 Spread Footings on Native Soils .....	8
6.2.2.1 Founding Elevations .....	8
6.2.2.2 Geotechnical Resistances .....	9
6.2.2.3 Resistances to Lateral Loads .....	9
6.2.3 “Perched” Spread Footings .....	9
6.2.3.1 Founding Elevations .....	9
6.2.3.2 Geotechnical Resistances .....	10
6.2.3.3 Resistances to Lateral Loads .....	10



## PRELIMINARY FOUNDATION REPORT RAINBOW CREEK BRIDGES - HIGHWAY 427 EXTENSION

6.2.4	Steel H-Piles.....	10
6.2.4.1	Founding Elevations.....	10
6.2.4.2	Geotechnical Axial Resistances.....	11
6.2.4.3	Resistances to Lateral Loads.....	12
6.2.4.4	Frost Protection.....	12
6.2.5	Caissons.....	12
6.2.5.1	Founding Elevations.....	12
6.2.5.2	Geotechnical Resistances.....	12
6.2.5.3	Resistances to Lateral Loads.....	13
6.2.5.4	Frost Protection.....	13
6.3	Lateral Earth Pressures for Design.....	13
6.3.1	Seismic Considerations.....	14
6.4	Approach Embankments.....	15
6.4.1	Subgrade Preparation and Embankment Construction.....	15
6.4.2	Approach Embankment Stability.....	16
6.4.3	Approach Embankment Settlement.....	16
6.5	Detail Design and Construction Considerations.....	17
6.5.1	Additional Investigation Requirements.....	17
6.5.2	Excavation.....	17
6.5.3	Groundwater and Surface Water Control for Foundation Excavation.....	18
6.5.4	Subgrade Preparation.....	18
6.5.5	Obstructions During Pile Driving / Caisson Installation.....	18
7.0	CLOSURE.....	19

### REFERENCES

#### LIST OF TABLES

Table 1	Comparison of Foundation Alternatives – Rainbow Creek Bridges, Highway 427 (NBL and SBL) Extension, W.O. 05-20012
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#### LIST OF DRAWINGS

Drawing 1	Highway 427 Extension – Rainbow Creek Bridges – Borehole Locations
Drawing 2	Highway 427 Extension – Rainbow Creek Bridges – Soil Strata



## PRELIMINARY FOUNDATION REPORT RAINBOW CREEK BRIDGES - HIGHWAY 427 EXTENSION

### LIST OF FIGURES

- |          |   |
|----------|---|
| Figure 1 | Site Location Plan  |
| Figure 2 | Highway 427 Extension – Rainbow Creek Bridges – North Approach Embankment, Static Global Stability  |
| Figure 3 | Highway 427 Extension – Rainbow Creek Bridges – North Approach Embankment, Seismic Global Stability |

### LIST OF APPENDICES

#### **Appendix A      Borehole Records**

Lists of Symbols and Abbreviations  
Record of Borehole Sheets S4 to S9

#### **Appendix B      Laboratory Test Results**

- |           |   |
|-----------|---|
| Figure B1 | Grain Size Distribution Test Results – Clayey Silt Till   |
| Figure B2 | Plasticity Chart – Clayey Silt Till                       |
| Figure B3 | Grain Size Distribution Test Results – Sand and Silt Till |
| Figure B4 | Plasticity Chart – Sand and Silt Till                     |



**PRELIMINARY FOUNDATION REPORT  
RAINBOW CREEK BRIDGES - HIGHWAY 427 EXTENSION**

# **PART A**

**PRELIMINARY FOUNDATION INVESTIGATION REPORT  
RAINBOW CREEK BRIDGES  
HIGHWAY 427 EXTENSION  
FROM HIGHWAY 7 TO MAJOR MACKENZIE DRIVE  
W.O. 05-20012**



## PRELIMINARY FOUNDATION REPORT RAINBOW CREEK BRIDGES - HIGHWAY 427 EXTENSION

### 1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by McCormick Rankin Corporation (MRC) on behalf of the Ministry of Transportation, Ontario (MTO) to provide preliminary foundation engineering services for the proposed 6.6 km long extension of Highway 427 from Highway 7 northward to Major Mackenzie Drive in the City of Vaughan, Ontario. The terms of reference for the foundation engineering services are provided in the Request for Proposal for MTO Assignment No. 2005-E-0028, dated December 21, 2005.

This report addresses the preliminary foundation investigation carried out for the Highway 427 northbound lane (NBL) and southbound lane (SBL) bridges over Rainbow Creek, and the immediate approach embankments to these bridges. The approximate location of this site on the Highway 427 Extension alignment is shown on Figure 1.

The work was carried out in accordance with Golder's Supplemental Speciality Quality Control Plan for foundation engineering services for this project dated April 4, 2006.

### 2.0 SITE DESCRIPTION

The proposed Rainbow Creek bridges are located approximately 400 m north of the terminus of Rainbow Creek Drive and approximately 550 m south of Langstaff Road in the City of Vaughan, Ontario. The proposed structure site is located approximately 1 km west of Highway 27 and approximately 1.1 km east of Huntington Road (see Figure 1).

In general, the topography along the Highway 427 Extension alignment consists of flat-lying to gently sloping farm land and densely treed areas that are crossed by the valleys of Rainbow Creek and West Robinson Creek. Some residential, commercial and/or light industrial development is present along Zenway Boulevard, Langstaff Road and Rutherford Road.

The proposed bridges and associated approach embankments are to be situated within the Rainbow Creek valley. The valley slopes are about 8 m high relative to the creek level. South of the creek, the ground surface at the crest of the valley slope is at about Elevation 182 m. The ground surface in the approximately 20 m wide floodplain area south of Rainbow Creek is at about Elevation 175 m to 176 m. North of the creek, the floodplain is wider, extending to approximately 50 m to 80 m north the creek, with the ground surface varying from about Elevation 176 m to 178 m. The crest of the valley slope north of the creek is at about Elevation 183 m. Rainbow Creek at the site of the proposed bridges is up to about 4 m wide and the depth of the creek varies from about 1 m to 1.4 m.

The valley slopes, which are moderately to heavily treed, slope down to the floodplain level at a gradient of 3 to 4 horizontal to 1 vertical (3H:1V to 4H:1V) on the south slope and 3H:1V on the north slope. No evidence of surficial or deep-seated slope instability was observed on the valley slopes at the time of the borehole investigation at this site.

### 3.0 INVESTIGATION PROCEDURES

The borehole investigation for the Rainbow Creek bridges was carried out in February and March 2009, during which time a total of six boreholes were advanced. The boreholes, designated as Boreholes S4 to S9, were advanced at the locations shown on Drawing 1.



## PRELIMINARY FOUNDATION REPORT RAINBOW CREEK BRIDGES - HIGHWAY 427 EXTENSION

The boreholes near the crest of the south slope (Boreholes S4 and S5) are located within an open field, and the field work for these two boreholes was carried out using a track-mounted CME 55 drill rig supplied by Walker Drilling Ltd. of Utopia, Ontario. These two boreholes were advanced using 200 mm outside diameter hollow stem augers. Boreholes S6 to S9 are located within the Rainbow Creek floodplain, and access to these locations was via the moderately to heavily treed valley slopes; as the boreholes in the floodplain area are located within the jurisdiction of the Toronto Region Conservation Area (TRCA), no trees were permitted to be removed as part of Golder's investigation program. To access the floodplain boreholes, a specialized 1 m wide drill rig supplied by Kodiak Environmental Ltd. of Oakville, Ontario was used to manoeuvre between the trees. These boreholes were advanced using 108 mm diameter solid stem augers. Soil samples were obtained at 0.75 m and 1.5 m intervals of depth, using a 50 mm outer diameter split-spoon sampler driven by an automatic hammer in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586-99).

The boreholes were terminated after penetrating at least 3 m into hard or very dense soil having Standard Penetration Test (SPT) 'N' values of greater than 100 blows per 0.3 m of penetration. The boreholes near the crest of the south slope were drilled to a depth of 14.2 m. At the floodplain level the boreholes were advanced to depths ranging from 6.3 m to 9.3 m below existing ground surface.

The groundwater conditions in the open boreholes were observed during the drilling operations, and standpipe piezometers were installed in Boreholes S5 and S9 to permit monitoring of the groundwater level(s) at the site. The piezometers consisted of 51 mm diameter PVC pipe, with a slotted screen sealed within a sand filter pack at a selected depth interval within the borehole. Above the sand filter pack and piezometer screen, the boreholes and annulus surrounding the piezometer pipe were backfilled to the surface with bentonite pellets/grout. The piezometer installation details and water level readings are indicated on the Record of Borehole Sheets in Appendix A. The boreholes in which no standpipe piezometers were installed were backfilled with bentonite upon completion, in accordance with Ontario Regulation 903 (as amended by Ontario Regulation 372).

The field work was observed by members of Golder's engineering and technical staff, who located the boreholes, arranged for the clearance of underground services through both public utility companies and a private utility locator, observed the drilling, sampling and in situ testing operations, logged the boreholes, and examined and cared for the soil samples. The samples were identified in the field, placed in appropriate containers, labelled and transported to Golder's Mississauga geotechnical laboratory where the samples underwent further detailed visual examination and geotechnical classification testing (water contents, Atterberg limits, and grain size distribution tests). All of the laboratory tests were carried out to MTO and/or ASTM Standards, as appropriate.

Prior to drilling, the boreholes were located in the field using the Highway 427 Extension alignment centreline stakes installed by MRC. The as-drilled borehole locations and ground surface elevations were surveyed by MRC, with the exception of Borehole S8. The location of Borehole S8 shown on Drawing 1 is based on measurements relative to site features and, for the purposes of this draft report, the elevation of the borehole has been taken from the topographic contours shown on the drawing. The borehole locations shown on Drawing 1 and on the borehole records are given relative to MTM NAD 83 northing and easting coordinates, and the ground surface elevations are referenced to geodetic datum.



## PRELIMINARY FOUNDATION REPORT RAINBOW CREEK BRIDGES - HIGHWAY 427 EXTENSION

### 4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

#### 4.1 Regional Geology

The Highway 427 Extension area lies within the Peel Plain physiographic region, as delineated in *The Physiography of Southern Ontario*<sup>1</sup>. A surficial till sheet, which generally follows the surface topography, is present throughout much of this area. The till is typically comprised of clayey silt to silty clay, with occasional sand to silt zones; it is mapped in this area as the Halton Till. Shallow, localized deposits of loose sand and silt and/or soft clay can overlie this uppermost till sheet, and these represent relatively recent deposits, formed in small glacial meltwater ponds scattered throughout the Peel Plain and concentrated near river valleys. The recent sand, silt and clay and uppermost till deposits in this area overlie and are interbedded with stratified deposits of sand, silt and clay. The study area is underlain by Ordovician shales of the Georgian Bay Formation.

#### 4.2 Subsurface Conditions

The detailed subsurface soil and groundwater conditions as encountered in the boreholes advanced for this investigation and the results of the laboratory tests carried out on selected soil samples are provided in Appendices A and B, respectively. The stratigraphic boundaries shown on the borehole records are inferred from non-continuous sampling, observations of drilling progress and the results of Standard Penetration Tests. These boundaries, therefore, represent transitions between soil types rather than exact planes of geological change.

The interpreted stratigraphic conditions along the Highway 427 NBL and SBL mainline alignment at the Rainbow Creek bridge sites are shown on Drawing 2. These stratigraphic profiles represent a simplification of the subsurface conditions as encountered in the boreholes. Variation in the stratigraphic boundaries and properties of the soil deposits will occur between and beyond the borehole locations.

In general, the subsurface conditions at the site of the proposed Rainbow Creek bridges consist of a surficial layer of topsoil up to 0.1 m thick (where present), overlying a clayey silt till deposit underlain by a sand and silt till deposit. In the boreholes drilled in the floodplain area, surficial (alluvium) deposits of clayey silt and silty sand were encountered within the upper 1.5 m.

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

##### 4.2.1 Topsoil

Approximately 0.1 m of topsoil was encountered immediately below ground surface in Boreholes S4 and S5, which are located near the crest of the south valley slope.

##### 4.2.2 Surficial Clayey Silt

In the Boreholes S6, S7 and S9, drilled within the Rainbow Creek floodplain area, a surficial deposit of clayey silt containing some sand, trace gravel and containing rootlets was encountered, extending from ground surface to

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





## PRELIMINARY FOUNDATION REPORT RAINBOW CREEK BRIDGES - HIGHWAY 427 EXTENSION

depths of between 0.7 m and 0.9 m, with the base of this deposit encountered between approximately Elevation 174.9 m and 175.3 m.

The Standard Penetration Test (SPT) 'N' values in the clayey silt deposit were 4 and 6 blows per 0.3 m of penetration, indicating that the surficial floodplain deposit has a firm consistency. Measured water contents on samples of the clayey silt were 23 percent and 32 percent.

### 4.2.3 Surficial Silty Sand

In Borehole S8, drilled within the Rainbow Creek floodplain area, a surficial deposit of silty sand containing trace gravel, clay and rootlets was encountered, extending from ground surface to a depth of 1.5 m (Elevation 176.1 m).

The measured SPT 'N' values within the surficial silty sand deposit were 5 and 9 blows per 0.3 m of penetration, indicative of loose relative density.

### 4.2.4 Till Deposit

In all boreholes drilled at this site, the topsoil (at the south slope crest) and surficial silty sand and clayey silt deposits (in the floodplain) are underlain by a clayey silt till deposit that grades with depth to a cohesionless till deposit.

Till deposits in southern Ontario typically contain cobbles and/or boulders. Cobbles and/or boulders have been inferred to be present within the till deposits at this site, based on grinding of augers during borehole drilling, as summarized in the table below:

Borehole No.	Depth of Observed Auger Grinding	Elevation of Inferred Cobbles / Boulders
S4	5.2 m	177.3 m
	11.4 m	171.1 m
S5	8.5 m	173.1 m
	9.7 to 10.7 m	171.9 to 170.9 m
	11.9 to 12.5 m	169.7 to 169.1 m
	13.4 to 13.7 m	168.2 to 167.9 m
S6	5.5 m	172.1 m
S8	3.7 to 4.4 m	172.1 – 171.4 m
S9	5.8 to 8.8 m	170.2 to 167.2 m

#### 4.2.4.1 Clayey Silt Till

In the boreholes drilled near the crest of the south valley slope, the upper cohesive till extends to a depth of about 8.7 m, whereas in the boreholes drilled in the floodplain the cohesive till extends to depths of between 3.7 m and 5.8 m; the base of the cohesive till was encountered in the boreholes between approximately Elevations 170.0 m and 172.1 m.

The upper cohesive till consists of clayey silt with sand to some sand, trace gravel. Grain size analyses were carried out on six selected samples of the clayey silt till deposit and the results are presented on Figure B1 in



## PRELIMINARY FOUNDATION REPORT RAINBOW CREEK BRIDGES - HIGHWAY 427 EXTENSION

Appendix B. Atterberg limits testing was carried out on eleven samples of this portion of the till deposit, and measured plastic limits of 11 percent to 19 percent, liquid limits of 16 to 33 percent, and plasticity indices of 5 to 16 percent. These results, which are plotted on a plasticity chart on Figure B2 in Appendix B, confirm that this portion of the till deposit is a clayey silt of low plasticity. Measured water contents on samples of the clayey silt till ranged from about 7 percent to 23 percent.

A lower cohesive till deposit was encountered underlying the cohesionless portion of the till in Boreholes S4 and S7. The lower cohesive till deposit consists of clayey silt containing trace to with sand and trace gravel.

The SPT 'N' values measured within the upper clayey silt till typically ranged from 11 blows to greater than 100 blows per 0.3 m of penetration, indicating a stiff to hard consistency. SPT 'N' values as low as 5 blows to 9 blows per 0.3 m of penetration were recorded in the near-surface portions of the till deposit (from ground surface to a depth of about 0.7 m), indicating a firm to stiff consistency. The SPT 'N' values measured within the lower clayey silt till varied from 113 blows per 0.3 m of penetration to 100 blows per 0.13 m of penetration, indicating that the lower clayey silt till has a hard consistency.

### 4.2.4.2 Sand and Silt Till

The upper clayey silt till grades with depth to a cohesionless till, the surface of which was encountered between Elevations 170.0 m and 172.1 m. Boreholes S5, S6, S8 and S9 terminated in the cohesionless till deposit; however Boreholes S4 and S7 fully penetrated the cohesionless till deposit, which was found to have a thickness of approximately 2.9 m to at least 5.3 m. The base of the cohesionless till was encountered in the boreholes at Elevations 169.2 m and 167.1 m, although the deposit base may be lower or higher than this in the other boreholes where it was not fully penetrated.

The cohesionless portion of the till deposit consists of sand and silt containing trace to some gravel and trace clay. The results of grain size distribution tests completed on six selected samples of the sand and silt till are provided on Figure B3 in Appendix B. Atterberg limits testing was carried out on two samples of the sand and silt till, and measured plastic limits of 11 and 12 percent, liquid limits of 15 percent for both samples, and plasticity indices of 3 and 4 percent. These results, which are plotted on a plasticity chart on Figure B4 in Appendix B, confirm that this material is a sand and silt till that is non-plastic or has low plasticity. Measured water contents on samples of the sand and silt till ranged from about 4 percent to 12 percent.

Within the cohesionless till the SPT 'N' values ranged from 37 blows per 0.3 m of penetration to 100 blows per 0.13 m of penetration, indicative of till with a dense to very dense relative density.

## 4.3 Groundwater Conditions

The water level in the boreholes as noted during and upon completion of drilling operations is typically between about Elevation 175.6 m and Elevation 176.5 m (typically at a depth of 6.0 m) in the two boreholes drilled at the crest of the slope and between Elevation 174.8 m and Elevation 176.7 m (between 0.9 m and 3.0 m depth) in the boreholes drilled in the Rainbow Creek floodplain area. In general, the clayey silt till samples taken in the boreholes drilled near the crest of the south slope were noted to be moist to wet, and the sand and silt till samples were wet. In the boreholes drilled at the floodplain level all of the samples were wet.

Standpipe piezometers were installed in Boreholes S5 and S9 to permit monitoring of the water levels at this site. Details of the piezometer installations are shown on the borehole records in Appendix A. The groundwater levels measured in the piezometer installations are summarised below.



## PRELIMINARY FOUNDATION REPORT RAINBOW CREEK BRIDGES - HIGHWAY 427 EXTENSION

Borehole No.	Ground Surface Elevation	Depth to Groundwater Level	Groundwater Elevation	Date of Measurement
S5 (crest of south slope)	181.6 m	4.4 m	177.2 m	April 24, 2009
		4.4 m	177.2 m	May 13, 2009
		4.6 m	177.0 m	May 21, 2009
		4.7 m	176.9 m	June 15, 2009
		4.9 m	176.7 m	July 9, 2009
S9 (floodplain)	176.0 m	3.6 m	172.4 m	April 24, 2009
		1.2 m	174.8 m	May 13, 2009
		0.9 m	175.1 m	May 21, 2009
		0.5 m	175.5 m	June 15, 2009
		0.5 m	175.5 m	July 9, 2009

The groundwater levels in the area should be expected to be subject to seasonal fluctuations and precipitation events, and should be expected to be higher during wet periods of the year.

### 5.0 CLOSURE

The field investigation program at this site was arranged and supervised by Messrs. Pat Speirs and Dan Demmings. This report was prepared by Ms. Sandra McGaghran, P.Eng. a geotechnical engineer with Golder, and reviewed by Ms. Lisa Coyne, P.Eng., a geotechnical engineer and Associate with Golder. Mr. Fin Heffernan, P.Eng., Golder's Designated MTO Contact for this project, conducted an independent quality control review of the report.

  
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**PRELIMINARY FOUNDATION REPORT  
RAINBOW CREEK BRIDGES - HIGHWAY 427 EXTENSION**

# **PART B**

**PRELIMINARY FOUNDATION DESIGN REPORT  
RAINBOW CREEK BRIDGES  
HIGHWAY 427 EXTENSION  
FROM HIGHWAY 7 TO MAJOR MACKENZIE DRIVE  
W.O. 05-20012**



### 6.0 ENGINEERING RECOMMENDATIONS FOR PRELIMINARY DESIGN

This section of the report provides foundation design recommendations for the preliminary design of the proposed Rainbow Creek bridges on the Highway 427 NBL and SBL mainline alignment. The preliminary recommendations are based on interpretation of the factual data obtained from the boreholes advanced during this preliminary subsurface investigation. The discussion and preliminary recommendations presented are intended to provide the designers with sufficient information to assess the feasible foundation alternatives and to carry out the preliminary design of the structure foundations and approach embankments. Where comments are made on construction, they are provided in order to highlight those aspects that could affect the preliminary design of the project, and for which special provisions are expected to be required as the project proceeds through detail design and into contract preparation. Those requiring information on the aspects of construction should make their own interpretation of the factual information provided as such interpretation may affect equipment selection, proposed construction methods, scheduling and the like.

Further borehole investigation and analysis will be required during the detail design phase of the project, once the configuration of the proposed bridge is finalized, to confirm and expand on the preliminary foundation recommendations provided in this report.

#### 6.1 General

The Rainbow Creek bridges are proposed to consist of three-span structures. Based on the preliminary General Arrangement (GA) Drawing provided by MRC on May 15, 2009, the north abutments and the north and south piers for both the NBL and SBL bridges are proposed to be located within the floodplain area, and the south abutments are proposed to be located at approximately mid-slope on the south valley slope. Between the piers and the abutments the span lengths are approximately 24 m and between the north piers and south piers the span lengths are about 35 m.

According to the preliminary GA Drawing, the finished grade of Highway 427 NBL and SBL over Rainbow Creek will be at approximately Elevation 184 m, which is approximately 9 m above the lowest floodplain level. Therefore, the south approach embankments will be about 3 m high relative to the existing ground surface, and the north approach embankments will be about 6 m high relative to the adjacent existing ground surface.

#### 6.2 Foundation Recommendations

##### 6.2.1 Foundation Options

Based on the proposed vertical elevations and subsurface soil conditions, the following foundation options are considered feasible for the Rainbow Creek bridges:

- **Spread footings founded on the very stiff to hard clayey silt till:** This option is feasible at the piers; however the footings would have to extend through the firm to stiff clayey silt to a depth of about 1.5 m below ground surface (which is slightly deeper than the required depth of embedment for frost protection purposes). Considering that the grade at the north and south abutments is to be raised by about 6 m and 3 m, respectively, this option may not be economical at the abutments given the resulting height of abutment walls.
- **Spread footings “perched” on a granular pad within the approach embankment fill:** This option could be adopted to support the abutments for an open structure, with 2 horizontal to 1 vertical (2H:1V) foreslopes in front of the abutment footings. In order to minimize potential settlements, it





## PRELIMINARY FOUNDATION REPORT RAINBOW CREEK BRIDGES - HIGHWAY 427 EXTENSION

would be necessary to subexcavate the upper 0.8 m to 1.5 m of firm to stiff clayey silt to expose the very stiff to hard clayey silt till at the north and south abutments, respectively, prior to construction of the new approach embankments.

- **Steel H-piles driven to found within the glacial till deposit:** This option could be adopted to support the abutments and piers in either a conventional or an integral abutment-type structure. Given that the site soils will not present long-term settlement issues, the site is considered suitable for the use of integral abutments. Alternatively, an open bridge configuration could be adopted, in conjunction with 2H:1V foreslopes in front of the abutment pile caps.
- **Caissons founded within the glacial till deposit:** This option could be adopted to support the abutments or piers in either a conventional or a semi-integral abutment-type structure.

At the abutments, either “perched” footings or steel H-piles are preferred over spread footings founded on the native soils due the resulting height of the abutment walls. At the piers, spread footings would require only minor additional subexcavation of about 0.1 m below the frost depth in order to found the spread footings on very stiff to hard clayey silt till, and these are therefore preferred if sufficient geotechnical resistance can be achieved; otherwise, support of the piers on deep foundations will be required to achieve a higher capacity. The use of piles is preferred from a foundations perspective over caissons for support of the abutments and piers, as the caissons would terminate in the water-bearing sand and silt till, which would be susceptible to disturbance and which would require special construction procedures. However, support of the piers on caissons may be preferable considering that the depth to the material having SPT ‘N’ values greater than 100 blows per 0.3 m of penetration in the floodplain is typically within 4 m to 6 m below ground surface.

Recommendations for preliminary design of spread footings, steel H-pile and caisson foundations are presented in the following sections. A summary comparison of the advantages, disadvantages and relative costs associated with each of the feasible foundation options is presented in Table 1 following the text of this report.

### 6.2.2 Spread Footings on Native Soils

The following sections provide geotechnical resistances for spread footings founded on very stiff to hard clayey silt till.

#### 6.2.2.1 Founding Elevations

The abutments and piers may be supported on spread footing placed below the upper firm to stiff clayey silt, on very stiff to hard clayey silt till (depth varies from approximately 0.8 m at the south abutment to 1.5 m at the piers and north abutment). A minimum founding depth of 1.4 m is required for frost protection purposes (OPSD 3090.101). Preliminary recommendations for minimum (highest) founding depths are provided in the following table, based on both frost protection and subexcavation requirements; these depths are given relative to lowest surrounding grade.

Foundation Element	Highway 427 NBL	Highway 427 SBL	Founding Stratum
South Abutment	1.4 m depth	1.4 m depth	Clayey Silt Till
South Pier	1.5 m depth	1.5 m depth	
North Pier	1.5 m depth	1.5 m depth	
North Abutment	1.4 m depth	1.5 m depth	



## PRELIMINARY FOUNDATION REPORT RAINBOW CREEK BRIDGES - HIGHWAY 427 EXTENSION

### 6.2.2.2 Geotechnical Resistances

A factored geotechnical resistance at Ultimate Limit States (ULS) of 525 kPa and a geotechnical resistance at Serviceability Limit States (SLS) of 350 kPa (for 25 mm of settlement) may be used for preliminary design purposes, assuming 3 m wide footings.

The ULS and SLS resistances and settlement are dependent on the footing size, configuration and applied loads. The geotechnical resistances should, therefore, be reviewed during detail design, once further drilling has been carried out at the foundation elements to delineate the thickness and properties of the surficial clayey silt and confirm the founding level, and once the final geometry of the foundations has been established.

The geotechnical resistances provided above are given under the assumption that the loads will be applied perpendicular to the surface of the footings. Where the load is not applied perpendicular to the surface of the footing, inclination of the load should be taken into account in accordance with Section 6.7.4 of the *Canadian Highway Bridge Design Code (CHBDC)* and its Commentary, using the curves for non-cohesive soils.

### 6.2.2.3 Resistances to Lateral Loads

The resistance to lateral forces/sliding resistance between the concrete footings and the very stiff to hard native clayey silt till should be calculated in accordance with Section 6.7.5 of the *CHBDC*. A coefficient of friction,  $\tan \phi'$ , of 0.55 can be used for cast-in-place concrete footings on the properly prepared clayey silt till subgrade. This represents an unfactored value; in accordance with the *CHBDC*, a factor of 0.8 is to be applied in calculating horizontal resistance.

### 6.2.3 “Perched” Spread Footings

In order to minimize the height of the abutments walls, spread footings for the bridge abutments may be placed on a compacted Granular ‘A’ pad constructed within the approach embankment fill. The following sections provide geotechnical resistances for spread footings at the abutments that are “perched” within the approach embankment fill on a compacted granular pad.

#### 6.2.3.1 Founding Elevations

“Perched” abutment spread footings founded on Ontario Provincial Standard Specification (OPSS) 1010 Granular ‘A’ pads should be provided with a minimum of 1.4 m of soil cover for frost protection (OPSS 3090.101).

For this option, subexcavation will be required of the firm to stiff clayey silt material that is present within the embankment footprint below the perched abutment, to minimize settlement due to the embankment loading. It is expected that subexcavation of the upper 1.5 m of soil in the floodplain would be required at the north abutments, while at the south abutments the upper 0.8 m of soil would have to be subexcavated. The area to be subexcavated should be defined by a line extending from the toe of the OPSS 1010 Granular ‘A’ pad, outward and downward at 1 horizontal to 1 vertical (1H:1V). The subexcavation should be replaced with compacted OPSS 1010 Granular ‘B’. The Granular ‘A’ pad should be a minimum of 2 m thick and should extend at least 1 m beyond the plan limits of the footing. The Granular ‘A’ pad should be constructed in accordance with MTO Special Provision SP105S10.



### 6.2.3.2 Geotechnical Resistances

Assuming the above subexcavation depths and filling procedures, a factored geotechnical resistance at ULS of 850 kPa may be used for preliminary design. The geotechnical resistance at SLS may be taken as 350 kPa. These geotechnical resistances will have to be reviewed during detail design, after further drilling has been carried out at the foundation elements to confirm the extent of subexcavation that is required, and once the final geometry of the foundations and approach embankments has been established.

The geotechnical resistances provided above are given under the assumption that the loads will be applied perpendicular to the surface of the footings. Where the load is not applied perpendicular to the surface of the footing, inclination of the load should be taken into account in accordance with Section 6.7.4 of the *CHBDC* and its Commentary, using the curves for non-cohesive soils.

### 6.2.3.3 Resistances to Lateral Loads

The resistance to lateral forces/sliding resistance between the concrete footings and the compacted Granular 'A' pad should be calculated in accordance with Section 6.7.5 of the *CHBDC*. The coefficient of friction,  $\tan \phi$ , can be taken as 0.70. This represents an unfactored value; in accordance with the *CHBDC*, a factor of 0.8 is to be applied in calculating horizontal resistance.

### 6.2.4 Steel H-Piles

Preliminary geotechnical recommendations for steel H-pile foundations are provided in the subsections that follow.

For the installation of steel H-piles, consideration will have to be given to the possible presence of cobbles and/or boulders within the till. It is recommended that the piles be stiffened with driving shoes/flange plates for protection during driving, in accordance with OPSS 903.07.05.04 and OPSD 3000.100. Pile installation and driving shoes should be in accordance with Special Provision SP903S01.

#### 6.2.4.1 Founding Elevations

Steel H-piles driven to found within the very dense sand and silt till deposit may be used for support of the abutments and piers. "Refusal" (i.e. soil having SPT 'N' values greater than 100 blows per 0.3 m of penetration) was encountered in the boreholes between approximately Elevation 169.5 m to 171.5 m, with the exception of Borehole S8 (located near the south pier of the SBL bridge) where it was encountered at Elevation 174.3 m. The table below summarizes the estimated pile tip elevation for preliminary design purposes, based on assumed penetration of approximately 1.5 m into soil having SPT 'N' values of greater than 100 blows per 0.3 m of penetration.





## PRELIMINARY FOUNDATION REPORT RAINBOW CREEK BRIDGES - HIGHWAY 427 EXTENSION

Foundation Unit	Borehole No.	Founding Stratum	Estimated Pile Tip Elevation
South Abutment SBL Bridge	S4	Very Dense Sand and Silt Till	170.0 m
South Abutment NBL Bridge	S5		169.0 m
South Pier SBL Bridge	S8		170.9 m
North Pier NBL Bridge	S7		168.0 m
North Abutment SBL Bridge	S6		170.0 m
North Abutment NBL Bridge	S9		170.0 m

### 6.2.4.2 Geotechnical Axial Resistances

The proposed abutments and piers can be supported on steel H-piles driven to found within the very dense sand and silt till. For HP 310x110 piles driven about 1.5 m below the surface of the soil having SPT 'N' values greater than 100 blows per 0.3 m of penetration to the estimated tip elevations provided in Section 6.2.4.1 above, the factored axial geotechnical resistance at ULS and the axial geotechnical resistance at SLS (for 25 mm of settlement) are given below.

Foundation Unit	Founding Stratum	Factored Geotechnical Resistance at ULS	Geotechnical Resistance at SLS
North and South Abutments	Very Dense Sand and Silt Till	1,400 kN	1,200 kN
North and South Piers		1,000 kN	800 kN

At the proposed north abutment area it is estimated that up to about 30 mm of settlement will occur, primarily in the firm to stiff surficial clayey silt under the proposed loading from the approach embankment. For preliminary design purposes it is recommended that a downdrag load of 100 kN be included, although further investigation and assessment will be required during detail design stage. The structural capacity of the piles must be checked for the factored dead and downdrag loads in accordance with Section 6.8.4 of the *CHBDC*.

The pile capacity values provided above will have to be reviewed and modified if necessary during detail design, further to additional subsurface investigations at the locations of each bridge foundation element.

Pile installation should be in accordance with MTO's Special Provision SP903S01. The pile termination or set criteria will be dependent on the pile driving hammer type, helmet, selected pile size and length of pile. The pile capacity should then be verified in the field by the use of the Hiley formula (MTO Standard Structural Drawing SS-103-11) during the final stages of driving to achieve an ultimate capacity equal to the final recommended factored ULS capacity divided by a resistance factor of 0.5 applicable to the use of the Hiley formula.



### 6.2.4.3 Resistances to Lateral Loads

Resistance to lateral loading can be derived using vertical piles, with enhanced support offered by battered piles, if required. For vertical piles, the resistance to lateral loading will be derived solely from the soil in front of the piles, whereas battered piles derive lateral resistance from the soil in front of the piles as well as the horizontal component of the axial load present in the inclined pile.

The resistance to lateral loading in front of the pile, as well as pile group action for lateral loading if the pile spacing in the direction of loading is less than six to eight pile diameters, should be accounted for and assessed during the detail design phase of the project. For preliminary design, a factored lateral geotechnical resistance at ULS of 200 kN may be used and a lateral geotechnical resistance at SLS of 110 kN (for 10 mm of lateral displacement at the pile cap level) may be used for a single vertical HP 310x110 pile embedded in sand and silt till. These values are based on the "Assessed Horizontal Passive Resistance and Geotechnical Reaction at SLS" provided under Clause C6.8.7.1, Table C6.4 of the *Commentary on CHBDC*.

### 6.2.4.4 Frost Protection

All pile caps should be provided with a minimum of 1.4 m of soil cover for frost protection (OPSD 3090.101).

## 6.2.5 Caissons

Consideration could be given to the use of caissons socketted into the very dense sand and silt till for support of the foundation elements for the bridges. Preliminary geotechnical recommendations for caisson foundations are provided in the sub-sections that follow.

Running or flowing of water-bearing cohesionless soil strata could occur during or after drilling of the caissons and basal heave could occur in the water-bearing cohesionless soils that will be present at the caisson base. If caisson foundations are adopted for support of any of the foundation elements, a temporary or permanent liner would be required to support the soils during construction, and to permit inspection and cleaning of the caisson base.

### 6.2.5.1 Founding Elevations

The recommended pile tip elevations as given in Section 6.2.4.1 may also be used for preliminary design for the founding elevations for caissons.

### 6.2.5.2 Geotechnical Resistances

The following table provides preliminary recommendations for factored axial geotechnical resistance at ULS and axial geotechnical resistance at SLS (for 25 mm of settlement) for caissons founded within the very dense sand and silt till at the elevations given in Section 6.2.4.1.



## PRELIMINARY FOUNDATION REPORT RAINBOW CREEK BRIDGES - HIGHWAY 427 EXTENSION

Foundation Unit	Founding Stratum	Caisson Diameter	Factored Geotechnical Resistance at ULS	Geotechnical Resistance at SLS
South and North Abutments	Very Dense Sand and Silt Till	0.9 m	2,000 kN	1,700 kN
		1.2 m	3,600 kN	3,000 kN
		1.5 m	5,600 kN	4,600 kN
North and South Piers		0.9 m	1,250 kN	1,000 kN
		1.2 m	2,200 kN	1,800 kN
		1.5 m	3,400 kN	2,800 kN

### 6.2.5.3 Resistances to Lateral Loads

For preliminary design purposes, a maximum factored lateral resistances at ULS of 400 kN and a maximum lateral resistances at SLS (for 10 mm of horizontal deflection at pile cap level) of 250 kN are recommended for 0.9 m diameter caissons, based on the "Assessed Horizontal Passive Resistance and Geotechnical Reaction at SLS" provided under Clause C6.8.7.1, Table C6.4 of the *Commentary on CHBDC* together with lateral caisson load test data. Values for alternative caisson diameters can be developed if larger diameter caisson foundations are adopted for support of foundation elements at this site.

### 6.2.5.4 Frost Protection

The caisson caps should be provided with a minimum of 1.4 m of soil cover for frost protection (OPSD 3090.101).

## 6.3 Lateral Earth Pressures for Design

The lateral earth pressures acting on the abutment stems and any associated wing walls/retaining walls will depend on the type and method of placement of the backfill materials, the nature of the soils behind the backfill, the magnitude of surcharge including construction loadings, the freedom of lateral movement of the structure, and the drainage conditions behind the walls. Seismic (earthquake) loading must also be taken into account in the design.

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

- Select, free draining granular fill meeting the specifications of Ontario Provincial Standard Specifications (OPSS) 1010 Granular 'A' or Granular 'B' Type II but with less than 5 percent passing the 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 OPSD 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 6.6. Compaction equipment should be used in accordance with MTO's Special Provision SP105S10. Other surcharge loadings should be accounted for in the design as required.



## PRELIMINARY FOUNDATION REPORT RAINBOW CREEK BRIDGES - HIGHWAY 427 EXTENSION

- The granular fill may be placed either in a zone with the width equal to at least 1.4 m behind the back of the walls (see Case A in Figure C6.20(a) 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 (see Case B in Figure C6.20(b) of the *Commentary* to the CHBDC).
- For Case A, the pressures are based on the proposed embankment fill materials and the existing overburden soils and the following parameters (unfactored) may be used assuming the use of earth fill :

	Earth Fill
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 B, where the pressures are based on OPSS 1010 granular fill behind the wall, the following parameters (unfactored) may be assumed:

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

If the wall support and superstructure allow lateral yielding of the stem, active earth pressures should be used in the geotechnical design of the structure. If the abutment support does not allow lateral yielding (such as for a rigid frame structure), at-rest earth pressures should be assumed for geotechnical design. The movement required to allow active pressures to develop within the backfill, and thereby assume an unrestrained structure for design, should be calculated in accordance with Section C6.9.1 and Table C6.6 of the *Commentary* to the CHBDC.

### 6.3.1 Seismic Considerations

Seismic (earthquake) loading must also be taken into account in the design in accordance with Section 4.6 of the CHBDC. Seismic (earthquake) loading must be considered in the design in accordance with Section 4.6.4 of CHBDC, as significant seismic loading will result in increased lateral earth pressures acting on the abutment stem and retaining walls. The walls should be designed to withstand the combined lateral loading for the appropriate static pressure conditions given above, plus the applicable earthquake-induced dynamic earth pressure. The earthquake-induced dynamic pressure distribution is a linear distribution with maximum pressure at the top of the wall and minimum pressure at its toe (i.e. an inverted triangular pressure distribution). The total pressure distribution (static plus seismic) may be determined as follows:



## PRELIMINARY FOUNDATION REPORT RAINBOW CREEK BRIDGES - HIGHWAY 427 EXTENSION

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

Where	K	is either the static active earth pressure coefficient ( $K_a$ ) or the static at rest earth pressure coefficient ( $K_o$ );
	$K_{AE}$	is the seismic active earth pressure coefficient;
	$\gamma'$	is the effective unit weight of the soil ( $\text{kN/m}^3$ )
		<ul style="list-style-type: none"> <li>taken as soil unit weights given above for fill materials</li> <li>taken as <math>20 \text{ kN/m}^3</math> for the native materials</li> </ul>
	d	is the depth below the top of the wall (m); and
	H	is the height of the wall above the toe (m).

According to Table C4.2 of the *Commentary* to the *CHBDC*, this site is located in Seismic Zone 1, and the site specific zonal acceleration ratio for the Vaughan area is 0.05. For the thicknesses and type of competent overburden soils at this site, a site coefficient of 1.0 and) an amplification factor of 1.33 are recommended. Therefore, the recommended ground surface acceleration is 0.067g.

The seismic lateral earth pressure coefficients given below have been derived based on a design zonal acceleration ratio of  $A = 0.067$ . These coefficients have been determined in accordance with Sections 4.6.4 and C4.6.4 of the *CHBDC* and its *Commentary*, and assume that the back of the wall is vertical and the ground surface behind the wall is essentially flat.

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

	CASE A	CASE B	
	Earth Fill	Granular 'A'	Granular 'B' Type II
Yielding Wall	0.29	0.26	0.26
Non-Yielding Wall	0.33	0.29	0.29

Note : These *CHBDC* seismic  $K_{AE}$  values include the effect of wall friction ( $\delta=\Phi'/2$ ) and are not greater than the static values of  $K_a$  and  $K_o$  reported above for the very low zonal acceleration ratio for this site.

## 6.4 Approach Embankments

The construction of the Rainbow Creek bridges will require placement of up to about 3 m of fill within the limits of the south approach and up to about 6 m of fill within the limits of the north approach embankment.

Based on the results of the boreholes drilled at this site, the approach embankments will be founded on firm to hard clayey silt till at the south approach; at the north approach the embankments will be founded on an upper firm to stiff clayey silt or loose silty sand underlain by very stiff to hard clayey silt till.

### 6.4.1 Subgrade Preparation and Embankment Construction

The existing native subsoils are considered to be an appropriate subgrade for the proposed approach embankments; however, to improve the embankment performance, it is recommended that prior to the placement of any fill, all topsoil, organic matter, existing fill and any softened or loosened native soils should be stripped from below the approach embankment areas. Embankment fill should be placed and compacted in



## PRELIMINARY FOUNDATION REPORT RAINBOW CREEK BRIDGES - HIGHWAY 427 EXTENSION

accordance with MTO's SP 206S03 and SP 105S10. To reduce erosion of the embankment side slopes due to surface water runoff, placement of topsoil and seeding or pegged sod is recommended as soon as practicable after construction of the embankments. The erosion protection must be in accordance with OPSS 572.

### 6.4.2 Approach Embankment Stability

Static and seismic slope stability analyses of the proposed approach embankments were carried out with the commercially available program SLOPE-W (produced by Geo-Slope International Ltd.) to check that the target minimum factor of safety was achieved for the proposed embankment heights and geometries. The factor of safety is defined as the ratio of the forces tending to resist failure to the driving forces tending to cause failure. A target minimum factor of safety of 1.3 is normally used in the design of embankment slopes under static conditions. This factor of safety is considered adequate for the embankments at this site.

The soil parameters used in the analysis, as given in the following table, were estimated from empirical correlations using the results of in situ Standard Penetration Tests (SPT) and geotechnical classification testing. The groundwater table was taken at Elevations 177 m and 175 m in the model for the south and north approach embankment areas, respectively.

Soil Type	Unit Weight (kN/m <sup>3</sup> )	Undrained Shear Strength (kPa)	Cohesion, c' (kPa)	Angle of Internal Friction, $\phi'$ (degrees)
New Earth or Granular Fill	21	--	--	34
Firm Clayey Silt	19	25 kPa	--	30
Loose Silty Sand	19	--	--	30
Very Stiff to Hard Clayey Silt Till	21	100 kPa	--	34
Very Dense Silty Sand / Sand and Silt Till	21	--	--	34

With appropriate subgrade preparation and proper placement and compaction of embankment fill materials, the proposed 3 m to 6 m high approach embankments with side slopes maintained at 2H:1V will have a factor of safety of greater than 1.3 against deep-seated slope instability. The results of an example static stability analysis are provided on Figure 2.

Under seismic loading conditions with a horizontal peak ground acceleration (HPGA) equal to 0.067g, the factor of safety is greater than 1.2. The result of an example seismic slope stability analysis is shown on Figure 3.

### 6.4.3 Approach Embankment Settlement

Settlement of the approach embankments at the site will occur due to compression of the new embankment fill itself, as well as compression of the underlying native soils. Provided that the embankment material consists of clean earth fill or granular fill, the settlement of the 3 m to 6 m high approach embankment fill itself is expected to be less than about 25 mm, and this settlement will occur relatively quickly during and immediately following construction. The settlement of the foundation soils under the approach embankment loading is anticipated to





## PRELIMINARY FOUNDATION REPORT RAINBOW CREEK BRIDGES - HIGHWAY 427 EXTENSION

be approximately 30 mm; the majority of this settlement will occur during or immediately following construction of the approach embankments. This compression has been estimated using the elastic deformation moduli given in the table below, based on correlations with the measured SPT 'N' values. For the firm portion of the surficial clayey silt, where present, consolidation parameters have been estimated based correlation with Atterberg limits and on experience with similar soil types in the Peel Plain.

Soil Deposit	Bulk Unit Weight	Elastic Modulus	Consolidation Parameters
Embankment fill (range of parameters assumed for earth fill and granular fill)	20 – 22 kN/m <sup>3</sup>	--	--
Firm Clayey Silt	19 kN/m <sup>3</sup>	15 MPa	C <sub>c</sub> = 0.2 C <sub>r</sub> = 0.02
Loose Silty Sand	19 kN/m <sup>3</sup>	15 MPa	--
Very Stiff to Hard Clayey Silt Till	21 kN/m <sup>3</sup>	75 MPa	--
Very Dense Sand and Silt Till	21 kN/m <sup>3</sup>	150 MPa	--

## 6.5 Detail Design and Construction Considerations

### 6.5.1 Additional Investigation Requirements

As noted previously, additional borehole investigation, laboratory testing and analysis will be required during detail design, once the layout of the proposed bridge foundation elements is finalized, to confirm the preliminary foundation recommendations presented herein, including founding elevations and subexcavation requirements, geotechnical resistances, settlement, and dewatering.

In particular, it is recommended that further investigation be completed to determine the extent and thickness of the firm surficial clayey silt in the floodplain area and to further characterize this soil by carrying out field vane tests to measure the undrained shear strength of the soil and Atterberg limits tests for strength and settlement correlation purposes; depending on the areal extent, thickness and properties of this material as encountered in the detail stage of investigation, it is recommended that provision be made to conduct a consolidation test to determine the compressibility parameters.

Access to and drilling of additional boreholes at the east end of the south abutment for the Highway 427 NBL bridge may be difficult because the east limit of the abutment is located at approximately midway down the south valley slope, and this area is heavily treed. At the preliminary design stage, it was a condition of access to the TRCA Lands that no trees be cut down, and this condition may affect the detail investigation stage.

### 6.5.2 Excavation

Depending on the foundation option adopted, excavations for the bridge foundations are expected to extend to depths of up to 1.5 m below existing ground surface and will be made through loose silty sand/firm clayey silt and into very stiff to hard clayey silt till, which are considered Type 3 soil according to Occupational Health and Safety Act and Regulation for Construction Projects (OHSR). The excavation work should be carried out in accordance with the requirements of the OHSR, with side slopes no steeper than 1 horizontal to 1 vertical (1H:1V).



## PRELIMINARY FOUNDATION REPORT RAINBOW CREEK BRIDGES - HIGHWAY 427 EXTENSION

### 6.5.3 Groundwater and Surface Water Control for Foundation Excavation

The groundwater level was measured in a standpipe piezometer at the floodplain level at about 1.2 m below ground surface. It is expected that excavations for the piers and north abutment foundations will extend up to about 0.3 m below the groundwater level. Some water inflow into the excavation should be expected; however, it is anticipated that water inflow can be handled by pumping from filtered sump pumps placed at the base of the excavation.

The groundwater level in the piezometer in the borehole at the crest of the south valley slope was 4.5 m below ground surface and as such, excavations for spread footings or for subexcavation of the surficial material for placement of the granular pads will likely not encounter water.

### 6.5.4 Subgrade Preparation

The soils exposed at the footing or pile cap subgrade level will be susceptible to disturbance from construction traffic and/or ponded water. To limit this degradation, it is recommended that a working mat of mass concrete be placed on the subgrade within four hours after preparation, inspection and approval of the footing subgrade.

### 6.5.5 Obstructions During Pile Driving / Caisson Installation

It is anticipated that cobbles and/or boulders will be encountered within the till deposits, as noted in several boreholes at this site, and may affect the installation of steel H-piles and/or caissons. It is recommended that flange plate reinforcement or driving shoes be used on all steel H-Piles to facilitate driving into the very dense sand and silt till. In addition, as part of the detail design and contract preparation, it is recommended that consideration be given to including a Non-Standard Special Provision in the contract documents to warn the contractor of the possible presence of cobbles and/or boulders within the overburden soils.





## PRELIMINARY FOUNDATION REPORT RAINBOW CREEK BRIDGES - HIGHWAY 427 EXTENSION

### 7.0 CLOSURE

This report was prepared by Ms. Sandra McGaghran, P.Eng. and reviewed by Ms. Lisa Coyne, P.Eng., a geotechnical engineer and Associate with Golder. Mr. Fin Heffernan, P.Eng., Golder's Designated MTO Contact for this project, conducted an independent quality control review of the report.

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## PRELIMINARY FOUNDATION REPORT RAINBOW CREEK BRIDGES - HIGHWAY 427 EXTENSION

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## PRELIMINARY FOUNDATION REPORT RAINBOW CREEK BRIDGES - HIGHWAY 427 EXTENSION

**TABLE 1**  
**COMPARISON OF FOUNDATION ALTERNATIVES**  
**RAINBOW CREEK BRIDGES – HIGHWAY 427 (NBL AND SBL) EXTENSION**  
**W.O. 05-20012**

Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Spread Footings on very stiff to hard clayey silt till	Feasible for support of piers	<ul style="list-style-type: none"> <li>Relative ease of construction</li> <li>Negligible post-construction settlement</li> </ul>	<ul style="list-style-type: none"> <li>Approximately 1.5 m sub-excavation required (slightly more than standard foundation depth for frost protection),</li> <li>Groundwater control required (though expected to be adequately handled by pumping from sumps depending on the time of year);and</li> <li>Lowest bearing capacities of the four options</li> </ul>	<ul style="list-style-type: none"> <li>Lower relative cost than piled foundations; and,</li> <li>Subexcavation of 1.5 m of surficial soils required within footing footprint</li> </ul>	<ul style="list-style-type: none"> <li>Disturbance of subgrade soil due to ponded water</li> </ul>
Spread Footings "perched" in approach embankment fill	Feasible for support of abutments	<ul style="list-style-type: none"> <li>Negligible post-construction settlement;and,</li> <li>Construction maintained above groundwater level</li> </ul>	<ul style="list-style-type: none"> <li>Some subexcavation of firm clayey silt may be required below north approach embankment footprint to minimize settlement</li> </ul>	<ul style="list-style-type: none"> <li>Subexcavation of 1.5 m of surficial soils required within footing footprint; and,</li> <li>Low cost option</li> </ul>	<ul style="list-style-type: none"> <li>Some risk of higher post-construction settlements if firm surficial clayey silt is not subexcavated from below approach footprint; and,</li> <li>Must ensure proper compaction of Granular 'A' pad to minimize post-construction settlement</li> </ul>
Steel H-pile foundations driven to found within very dense sand and silt till	Feasible for support of abutments and piers	<ul style="list-style-type: none"> <li>Sub-excavation is not required,</li> <li>High bearing capacity,</li> <li>Negligible post-construction settlement; and,</li> <li>Can be used for support of conventional or integral abutments</li> </ul>	<ul style="list-style-type: none"> <li>Pre-augering may be required to penetrate cobbles and boulders in the overburden; and,</li> <li>Relatively short pile lengths may result at pier locations due to shallow depth to soil having SPT 'N' values greater than 100 blows per 0.3 m of penetration</li> </ul>	<ul style="list-style-type: none"> <li>Additional cost associated with specialised pre-augering equipment; and,</li> <li>More costly than spread footings</li> </ul>	<ul style="list-style-type: none"> <li>Specialized procedures may be required to penetrate and/or accommodate cobbles and boulders in the glacial till soils</li> </ul>
Caisson foundations founded within very dense sand and silt till	Feasible at piers and abutments	<ul style="list-style-type: none"> <li>Sub-excavation is not required,</li> <li>High bearing capacity,</li> <li>Negligible post-construction settlement; and,</li> <li>Can be used for support of conventional or semi-integral abutments</li> </ul>	<ul style="list-style-type: none"> <li>Caissons will be extended into and founded within water-bearing sand and silt till, with potential for soil disturbance at the caisson sides and base, resulting in requirement for temporary or permanent liners</li> <li>Preparation of the base in cohesionless till below the water table could be difficult</li> </ul>	<ul style="list-style-type: none"> <li>Additional cost associated with specialised drilling equipment and temporary or permanent liners; and,</li> <li>More costly option than steel H-piles</li> </ul>	<ul style="list-style-type: none"> <li>Specialised equipment and procedures may be required to penetrate cobbles and boulders in the glacial till soils</li> </ul>

CONT No.  
WO No. 05-20012



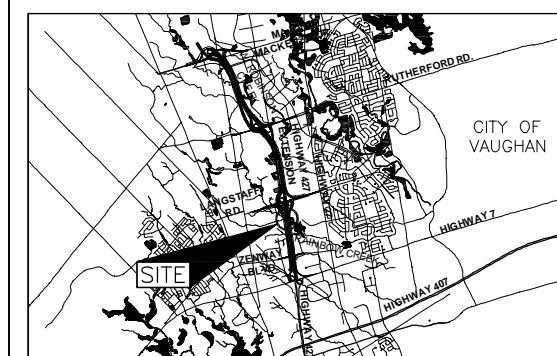
# HIGHWAY 427 EXTENSION

## RAINBOW CREEK BRIDGES

### BOREHOLE LOCATIONS

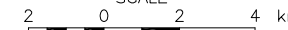


**Golder Associates Ltd.**  
MISSISSAUGA, ONTARIO, CANADA








## KEY PLAN

SCALE



### LEGEND

- |   |  |
|---|--|
|  | Borehole – Current Investigation                                   |
|  | Seal   |
|  | Piezometer   |
| N   | Standard Penetration Test Value                                    |
| 16  | Blows/0.3m unless otherwise stated<br>(Std. Pen. Test, 475 j/blow) |
|  | WL in piezometer, measured on May 13, 2009.                        |
|  | WL upon completion of drilling                                     |

No.	ELEVATION	CO—ORDINATES	
		NORTHING	EASTING
S4	182.5	4849365.1	293793.
S5	181.6	4849359.1	293821.
S6	177.6	4849472.0	293764.
S7	175.8	4849440.3	293806.
S8	175.8	4849407.4	293777.
S9	176.0	4849460.9	293829.

---

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 Preliminary Design Report.

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

The complete Preliminary 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

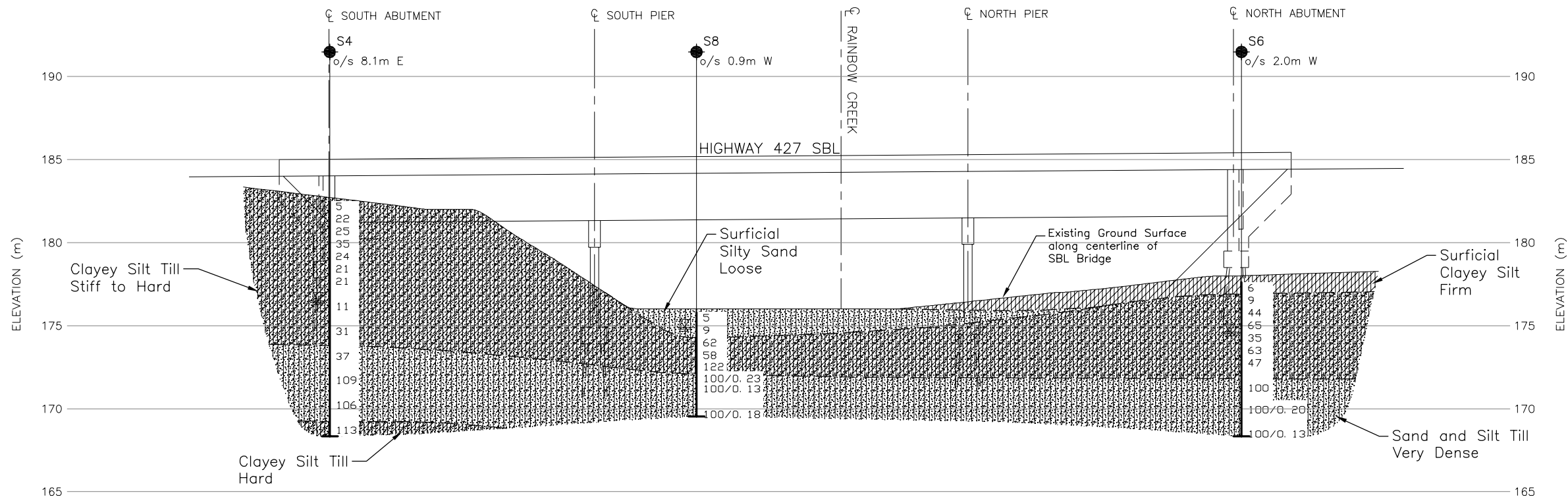
Base plans provided in digital format by MRC (Drawing "rainbow\_ga.dwg" received May 15, 2009).



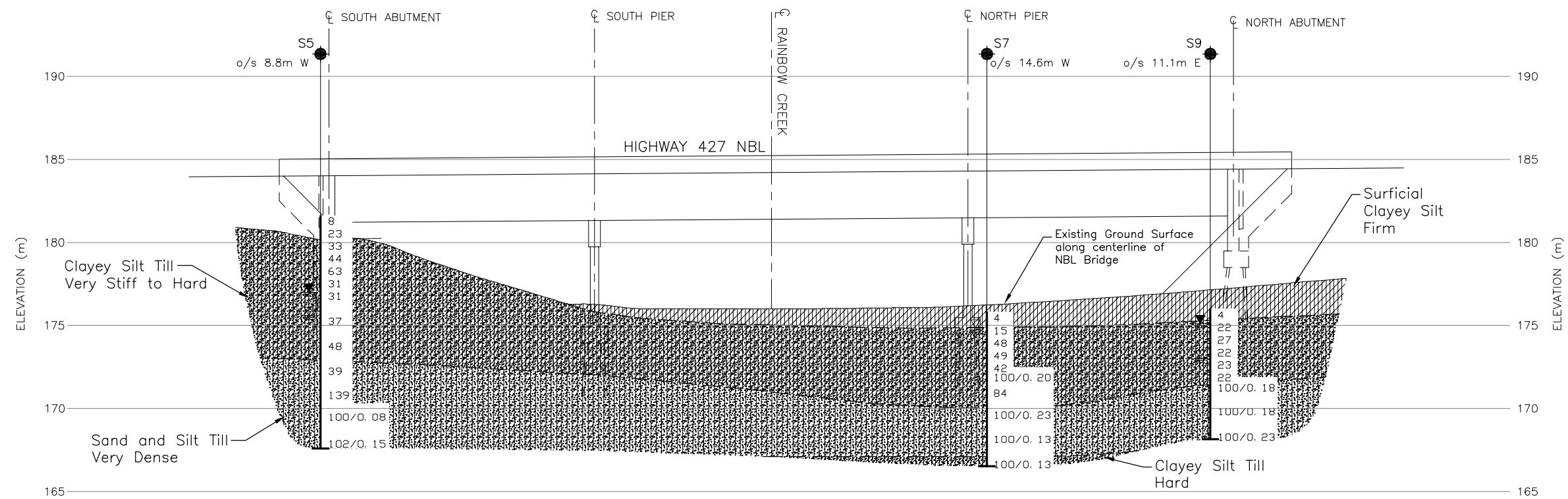
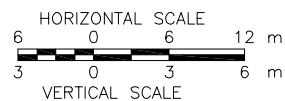
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Geocres No. 30M13-168			
HWY. 427		PROJECT NO. 06-1111-012-1	DIST.
SUBM'D. PKS	CHKD. SMM	DATE: 4-Aug-2009	SITE:
DRAWN: JM	CHKD. SMM	APPD. LCC	DWG. 1



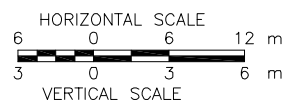
**METRIC**  
DIMENSIONS ARE IN METRES AND/OR  
MILLIMETRES UNLESS OTHERWISE SHOWN.  
STATIONS IN KILOMETRES + METRES.



**PROFILE A-A' RAINBOW CREEK SBL BRIDGE**



**PROFILE B-B' RAINBOW CREEK NBL BRIDGE**



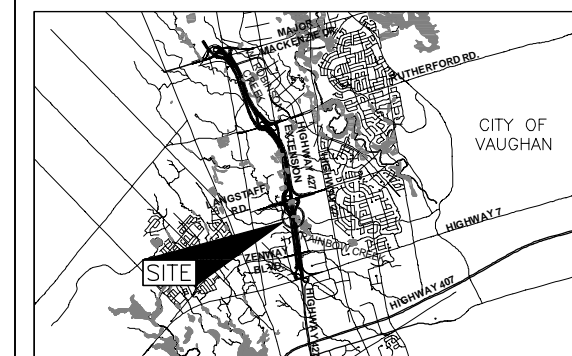
CONT No.  
WO No. 05-20012

HIGHWAY 427 EXTENSION  
RAINBOW CREEK BRIDGES  
SOIL STRATA

SHEET



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



**KEY PLAN**

SCALE: 0 to 4 km

**LEGEND**

- Borehole - 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 May 13, 2009.
- WL upon completion of drilling

No.	ELEVATION	CO-ORDINATES	
		NORTHING	EASTING
S4	182.5	4849365.1	293793.2
S5	181.6	4849359.1	293821.4
S6	177.6	4849472.0	293764.4
S7	175.8	4849440.3	293806.1
S8	175.8	4849407.4	293777.1
S9	176.0	4849460.9	293829.6

**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 Preliminary Design Report.

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**REFERENCE**

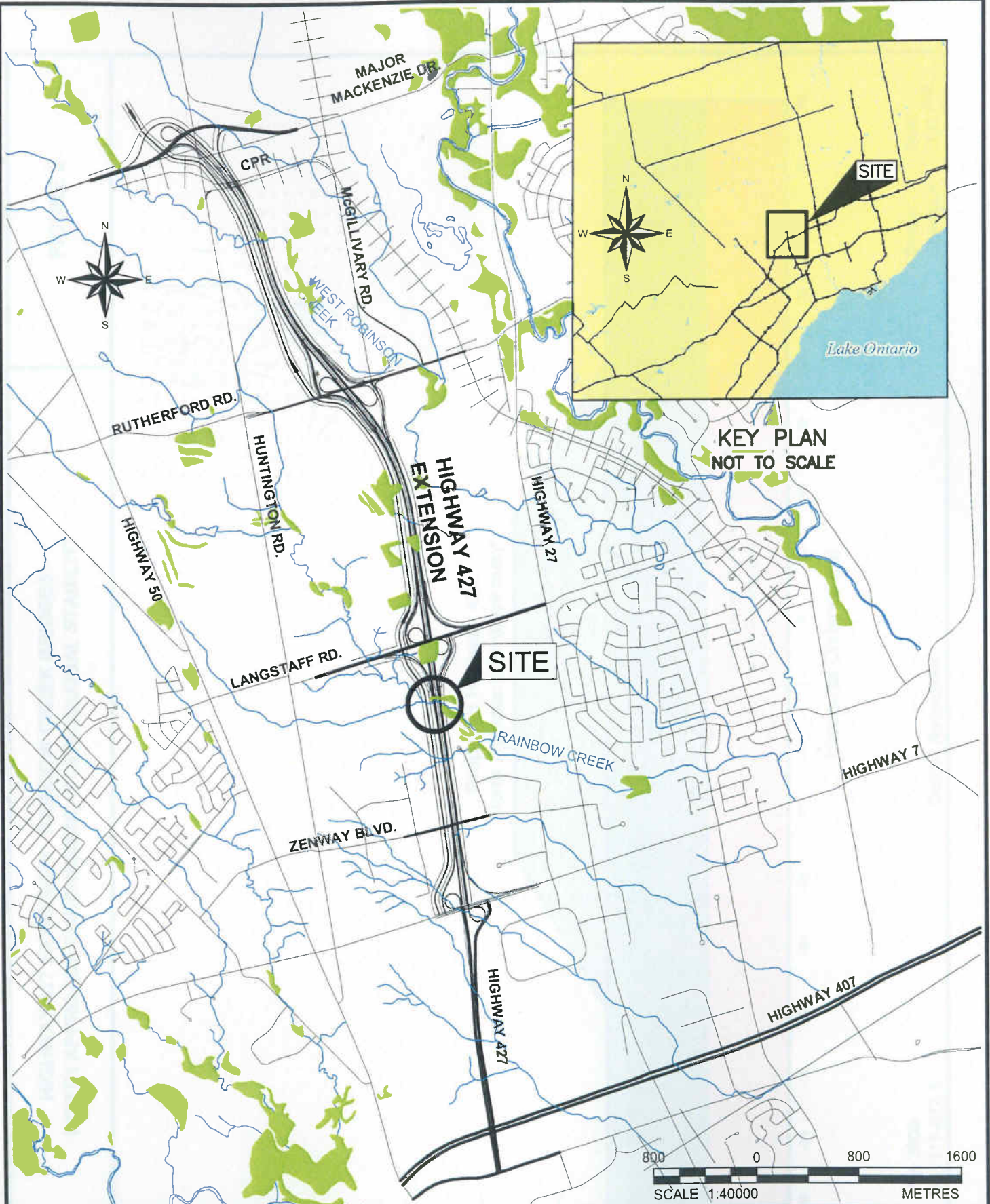
Base plans provided in digital format by MRC (Drawing "rainbow\_ga.dwg", received May 15, 2009).



NO.	DATE	BY	REVISION
1	4-Aug-2009	JM	1
Geocres No. 30M13-168			
HWY. 427		PROJECT NO. 06-1111-012-1	DIST.
SUBM'D. PKS	CHKD. SMM	DATE: 4-Aug-2009	SITE:
DRAWN: JM	CHKD. SMM	APPD. LCC	DWG. 2



PLOT DATE: August 5, 2009  
FILENAME: T:\Projects\2006\06-1111-012 (MRC, Vaughan)\-CB- (RAINBOW CREEK)\061111012CB0FG1.dwg



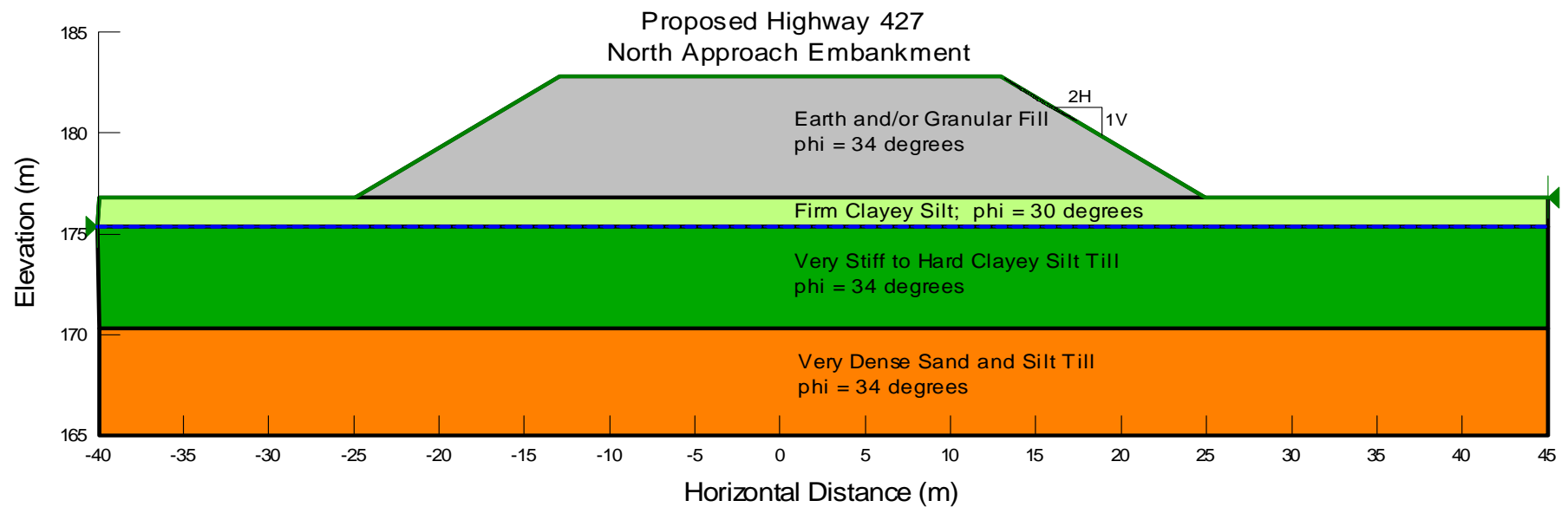
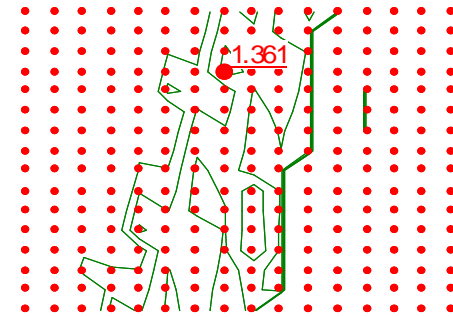
SCALE	AS SHOWN
DATE	Aug. 5, 2009
DESIGN	SB
CAD	JFC
CHECK	<i>SMM</i> SMM
REVIEW	LCC

TITLE	
<b>SITE LOCATION PLAN RAINBOW CREEK BRIDGES</b>	
FIGURE	<b>1</b>
<b>HIGHWAY 427 EXTENSION</b>	

FILE No.	061111012CB0FG1.dwg
PROJECT No.	06-1111-012-1
REV	B

# HIGHWAY 427 EXTENSION - RAINBOW CREEK BRIDGES NORTH APPROACH EMBANKMENT- STATIC GLOBAL STABILITY

FIGURE 2



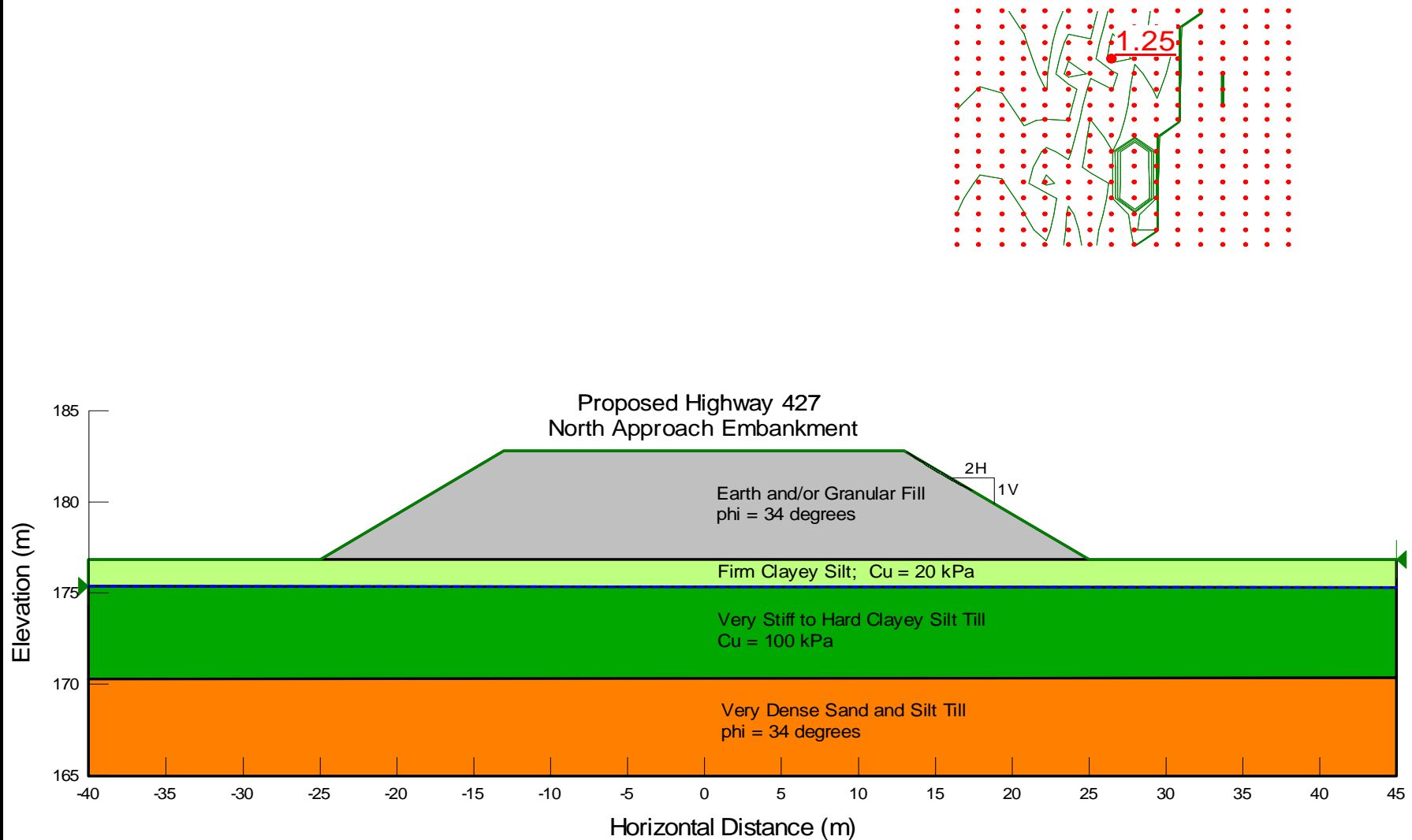
Date: May 2009  
Project: 06-1111-012-1

Golder Associates

Drawn: SMM  
Checked: LCC

**HIGHWAY 427 EXTENSION - RAINBOW CREEK BRIDGES  
NORTH APPROACH EMBANKMENT- SEISMIC GLOBAL STABILITY**

**FIGURE 3**



Date: May 2009  
Project: 06-1111-012-1

**Golder Associates**

Drawn: SMM  
Checked: LCC





**PRELIMINARY FOUNDATION REPORT  
RAINBOW CREEK BRIDGES - HIGHWAY 427 EXTENSION**

# **APPENDIX A**

## **Borehole Records**



## 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: $\Delta \sigma$
$\varepsilon$	linear strain
$\varepsilon_v$	volumetric strain
$\eta$	coefficient of viscosity
$\nu$	poisson's ratio
$\sigma$	total stress
$\sigma'$	effective stress ( $\sigma' = \sigma - \mu$ )
$\sigma'_{vo}$	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
$\mu$	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

### III. SOIL PROPERTIES

#### (a) Index Properties

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

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

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

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

#### (b) Hydraulic Properties

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

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

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

#### (d) Shear Strength

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

Notes: 1  $\tau = c' + \sigma' \tan \phi'$   
2 shear strength = (compressive strength)/2



## LIST OF ABBREVIATIONS

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

### I. SAMPLE TYPE

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

### II. PENETRATION RESISTANCE

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

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

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

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

**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.

### 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

#### (b) Cohesive Soils Consistency

$c_u, s_u$

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

### IV. SOIL TESTS

w water content

$w_p$  plastic limit

$w_l$  liquid limit

C consolidation (oedometer) test

CHEM chemical analysis (refer to text)

CID consolidated isotropically drained triaxial test<sup>1</sup>

CIU consolidated isotropically undrained triaxial test with porewater pressure measurement<sup>1</sup>

$D_R$  relative density (specific gravity,  $G_s$ )

DS direct shear test

M sieve analysis for particle size

MH combined sieve and hydrometer (H) analysis

MPC Modified Proctor compaction test

SPC Standard Proctor compaction test

OC organic content test

SO<sub>4</sub> concentration of water-soluble sulphates

UC unconfined compression test

UU unconsolidated undrained triaxial test

V field vane (LV-laboratory vane test)

$\gamma$  unit weight

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

PROJECT 06-1111-012		<b>RECORD OF BOREHOLE No S4</b>		1 OF 2 <b>METRIC</b>	
W.O. 05-20012		LOCATION N 4849365.1 ; E 293793.2		ORIGINATED BY DD	
DIST Central HWY 427		BOREHOLE TYPE 200 mm Outside Diameter Hollow Stem Augers		COMPILED BY VA	
DATUM Geodetic		DATE February 27, 2009		CHECKED BY SMM <i>[Signature]</i>	

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 × REMOULDED						

182.5	GROUND SURFACE													
8.9	TOPSOIL		1	SS	5		182							
	CLAYEY SILT, some sand, trace to some gravel (TILL), containing rootlets to a depth of 0.6 m and containing oxidation zones to a depth of 5.2 m Firm to hard Brown to grey Moist to wet		2	SS	22		181							
			3	SS	25		180							
			4	SS	35		179							6 12 62 20
			5	SS	24		178							
			6	SS	21		177							
			7	SS	21		176							
	Auger grinding at a depth of 5.2 m						175							
			8	SS	11		174							
							173							
							172							
							171							
							170							
173.8							169							
8.7	SAND and SILT, some gravel, trace clay, containing cobbles below 11.4 m depth (TILL) Dense to very dense Grey Wet		10	SS	37									
			11	SS	109									
	Auger grinding at a depth of 11.4 m													
			12	SS	106									
169.2														
13.3	CLAYEY SILT with sand, some gravel (TILL) Hard Grey Wet		13	SS	113									17 35 39 9
168.3														
14.2	END OF BOREHOLE													

MIS-MTO 001 06-1111-012.GPJ GAL-MISS.GDT 8/5/09 SAC/DD

Continued Next Page

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



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

PROJECT 06-1111-012			RECORD OF BOREHOLE No S5			1 OF 2 METRIC											
W.O. 05-20012			LOCATION N 4849359.7 ; E 293821.4			ORIGINATED BY DD											
DIST Central HWY 427			BOREHOLE TYPE 200 mm Outside Diameter Hollow Stem Augers			COMPILED BY VA											
DATUM Geodetic			DATE February 26, 2009			CHECKED BY SMM											
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100	20 40 60 80 100	W <sub>p</sub> W W <sub>L</sub>	WATER CONTENT (%)	10 20 30	γ	GR SA SI CL			
181.6	GROUND SURFACE																
8.9	TOPSOIL		1	SS	8		181										
	CLAYEY SILT, some sand, trace gravel (TILL), containing rootlets and oxidation zones to a depth of 2.1 m		2	SS	23		180										
	Stiff to hard		3	SS	33		179										
	Brown to grey		4	SS	44		178										
	Moist		5	SS	63		177										
	Grey below a depth of 3.8 m		6	SS	31		176										
			7	SS	31		175										
			8	SS	37		174										
			9	SS	48		173										
172.9	Auger grinding below a depth of 8.5m		10	SS	39		172										
8.7	SAND and SILT, some gravel, trace clay, containing cobbles (TILL)		11	SS	139		171										
	Dense to very dense		12	SS	00/0.0		170										
	Grey						169										
	Wet						168										
	Auger grinding from depth of 9.7 m to 10.7 m		13	SS	02/0.1												
	Auger grinding from depth of 11.9 m to 12.5 m																
	Auger grinding from depth of 13.4 m to 13.7 m																
167.6	END OF BOREHOLE																
14.0																	

MIS-MTO 001 06-1111-012.GPJ GAL-MISS.GDT 8/5/09 SAC/DD

Continued Next Page

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

**RECORD OF BOREHOLE No S5**

2 OF 2 **METRIC**

PROJECT 06-1111-012

W.O. 05-20012

LOCATION N 4849359.7 ; E 293821.4

ORIGINATED BY DD

DIST Central HWY 427

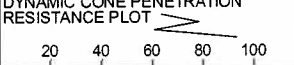
BOREHOLE TYPE 200 mm Outside Diameter Hollow Stem Augers


COMPILED BY VA





DATUM Geodetic

DATE February 26, 2009

CHECKED BY SMM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 	PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL																					
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES																													
<p>— CONTINUED FROM PREVIOUS PAGE —</p> <p>NOTES:</p> <p>1. A 50 mm diameter monitoring well was installed at a depth of 10.7 m (Elev. 170.9 m)</p> <p>Water level measurements</p> <table border="1"> <thead> <tr> <th>Date</th> <th>Depth</th> <th>Elev.</th> </tr> </thead> <tbody> <tr> <td>On Completion</td> <td>6.0 m</td> <td>175.6 m</td> </tr> <tr> <td>April 24, 2009</td> <td>4.4 m</td> <td>177.2 m</td> </tr> <tr> <td>May 13, 2009</td> <td>4.4 m</td> <td>177.2 m</td> </tr> <tr> <td>May 21, 2009</td> <td>4.6 m</td> <td>177.0 m</td> </tr> <tr> <td>June 15, 2009</td> <td>4.7 m</td> <td>176.9 m</td> </tr> <tr> <td>July 09, 2009</td> <td>4.9 m</td> <td>176.7 m</td> </tr> </tbody> </table>														Date	Depth	Elev.	On Completion	6.0 m	175.6 m	April 24, 2009	4.4 m	177.2 m	May 13, 2009	4.4 m	177.2 m	May 21, 2009	4.6 m	177.0 m	June 15, 2009	4.7 m	176.9 m	July 09, 2009	4.9 m	176.7 m
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<b>PROJECT</b> 06-1111-012		<b>RECORD OF BOREHOLE No S6</b>		1 OF 1 <b>METRIC</b>	
<b>W.O.</b> 05-20012		<b>LOCATION</b> N 4849472.0 ; E 293764.4		<b>ORIGINATED BY</b> PKS	
<b>DIST</b> Central HWY 427		<b>BOREHOLE TYPE</b> 108 mm Diameter Solid Stem Augers		<b>COMPILED BY</b> VA	
<b>DATUM</b> Geodetic		<b>DATE</b> March 13, 2009		<b>CHECKED BY</b> SMM 	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
177.6 0.0	GROUND SURFACE													
176.9 0.7	CLAYEY SILT, some sand, trace gravel Firm Brown Moist		1	SS	6									
	CLAYEY SILT, some sand, trace gravel (TILL) Stiff to hard Brown to grey Moist becoming wet below a depth of 1.2 m		2	SS	9									
			3	SS	44									
			4	SS	65									
	Grey below a depth of 3.0 m		5	SS	35									3 13 61 23
			6	SS	63									
			7	SS	47									
171.8 5.8	Augers grinding at a depth of 5.5 m SAND and SILT, some gravel, trace clay, containing cobbles (TILL) Very dense Grey Wet		8	SS	100									25 39 33 3
			9	SS	100/0.20									
168.3 9.3	END OF BOREHOLE		10	SS	100/0.13									3 33 55 9
<b>NOTES:</b> 1. Water level in open borehole at a depth of 3.0 m below ground surface (Elev. 174.7 m) upon completion of drilling. 2. Borehole backfilled with bentonite.														

MIS-MTO 001 06-1111-012.GPJ GAL-MISS.GDT 8/5/09 SAC/DD



PROJECT 06-1111-012

**RECORD OF BOREHOLE No S7**

1 OF 1 **METRIC**

W.O. 05-20012

LOCATION N 4849440.3 ; E 293806.1

ORIGINATED BY PKS

DIST Central HWY 427

BOREHOLE TYPE 108 mm Diameter Solid Stem Augers

COMPILED BY VA

DATUM Geodetic

DATE March 13, 2009

CHECKED BY SMM *SMM*

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						
175.8	GROUND SURFACE							20 40 60 80 100	20 40 60 80 100	10 20 30				
0.0	CLAYEY SILT with sand, trace gravel Firm Brown Moist to wet		1	SS	4	▽	175							
174.9	CLAYEY SILT, some sand, trace gravel (TILL) Very stiff to hard Brown to grey below a depth of 1.8 m Wet		2	SS	15		174							
0.9			3	SS	48		173							
			4	SS	49		172							
			5	SS	42		171							
			6	SS	100/0.2		170							
			7	SS	84		169							
170.0	SAND and SILT, trace to some gravel, trace clay (TILL) Very dense Grey Wet		8	SS	100/0.2		168							6 32 51 11
5.8			9	SS	100/0.1		167							
167.1	CLAYEY SILT, trace sand (TILL) Hard Grey Wet		10	SS	100/0.1									0 2 83 15
8.7														
166.5	END OF BOREHOLE													
9.3	NOTES:  1. Water level in open borehole at a depth of 0.9 m below ground surface (Elev.174.9 m) upon completion of drilling.  2. Borehole backfilled with bentonite.													

MIS-MTO 001 06-1111-012.GPJ GAL-MISS.GDT 8/5/09 SAC/DD

PROJECT 06-1111-012			RECORD OF BOREHOLE No S8			1 OF 1 METRIC											
W.O. 05-20012			LOCATION N 4849407.4 ; E 293777.1			ORIGINATED BY PKS											
DIST Central HWY 427			BOREHOLE TYPE 108 mm Diameter Solid Stem Augers			COMPILED BY VA											
DATUM Geodetic			DATE March 12, 2009			CHECKED BY SMM <i>SM</i>											
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100	20 40 60 80 100	W <sub>p</sub> W W <sub>L</sub>	WATER CONTENT (%)	10 20 30	γ	GR SA SI CL			
175.8	GROUND SURFACE																
0.0	Silty SAND, trace gravel, trace clay, containing rootlets Loose Brown Moist		1	SS	5		175										
			2	SS	9												
174.3																	
1.5	CLAYEY SILT with sand, trace gravel (TILL) Hard Grey Wet		3	SS	62		174										
			4	SS	58		173										
			5	SS	122												
172.1																	
3.7	SAND and SILT, some gravel, trace clay, containing cobbles (TILL) Very dense Grey Wet		6	SS	00/0.2		172										
			7	SS	00/0.1		171										
	Augers grinding at depths of 3.7 m and 4.4 m						170										
169.5			8	SS	100/0.1												
6.3	END OF BOREHOLE																
NOTES:																	
1. Water level in open borehole at a depth of 0.9 m below ground surface (Elev.176.7m) upon completion of drilling.																	
2. Borehole backfilled with bentonite.																	

MIS-MTO 001 06-1111-012.GPJ GAL-MISS.GDT 8/5/09 SAC/DD

PROJECT 06-1111-012

**RECORD OF BOREHOLE No S9**

1 OF 1 **METRIC**

W.O. 05-20012

LOCATION N 4849460.9 :E 293829.6

ORIGINATED BY PKS

DIST Central HWY 427

BOREHOLE TYPE 108 mm Diameter Solid Stem Augers

COMPILED BY VA

DATUM Geodetic

DATE March 16, 2009

CHECKED BY SMM *SMM*

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 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED						
176.0	GROUND SURFACE							20 40 60 80 100						
0.0	CLAYEY SILT, some sand, trace gravel, containing rootlets Firm Brown Moist		1	SS	4									
175.3														
0.7	CLAYEY SILT, trace to some sand, trace gravel (TILL) Very stiff Brown to grey Moist to wet below a depth of 1.5 m  Grey below a depth of 2.1 m		2	SS	22									
			3	SS	27									
			4	SS	22									
			5	SS	23									
			6	SS	22									
171.4	SAND and SILT, trace to some gravel, trace clay, containing cobbles (TILL) Very dense Grey Wet  Augers grinding at depths of 5.8 m and 8.8 m		7	SS	100/0.1									
4.6			8	SS	100/0.1									
168.2	END OF BOREHOLE		9	SS	100/0.2									
7.9	NOTES:  1. A 50 mm diameter monitoring well was installed at a depth of 7.6 m (Elev.168.4 m)  Water level measurements  Date            Depth        Elev.  On Completion 1.2 m    174.8 m April 24, 2009 3.6 m    172.4 m May 13, 2009 1.2 m    174.8 m May 21, 2009 0.9 m    175.1 m June 15, 2009 0.5 m    175.4 m July 09, 2009 0.5 m    175.5 m													

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



**PRELIMINARY FOUNDATION REPORT  
RAINBOW CREEK BRIDGES - HIGHWAY 427 EXTENSION**

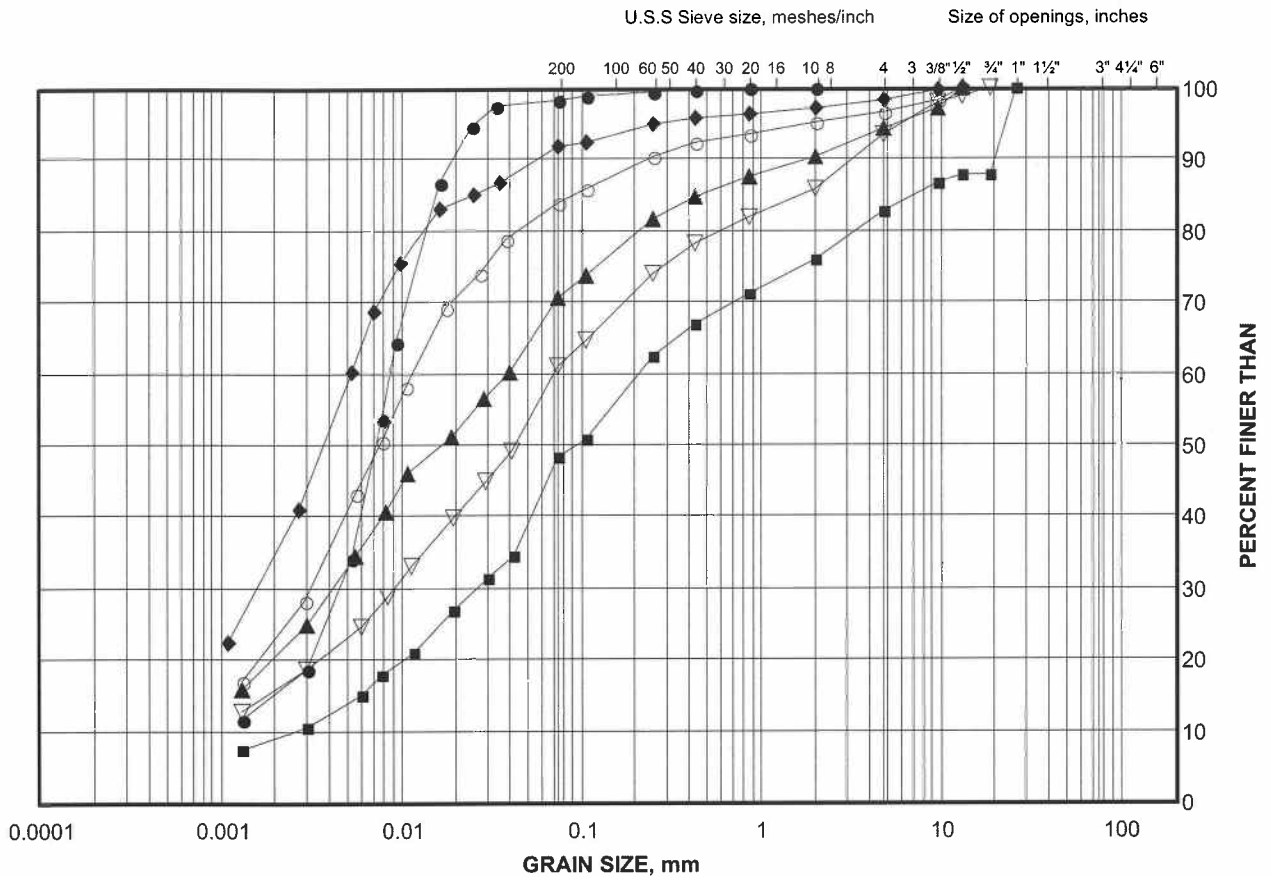
# **APPENDIX B**

## **Laboratory Test Results**

# GRAIN SIZE DISTRIBUTION TEST RESULTS

Clayey Silt Till

FIGURE B1



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

## LEGEND

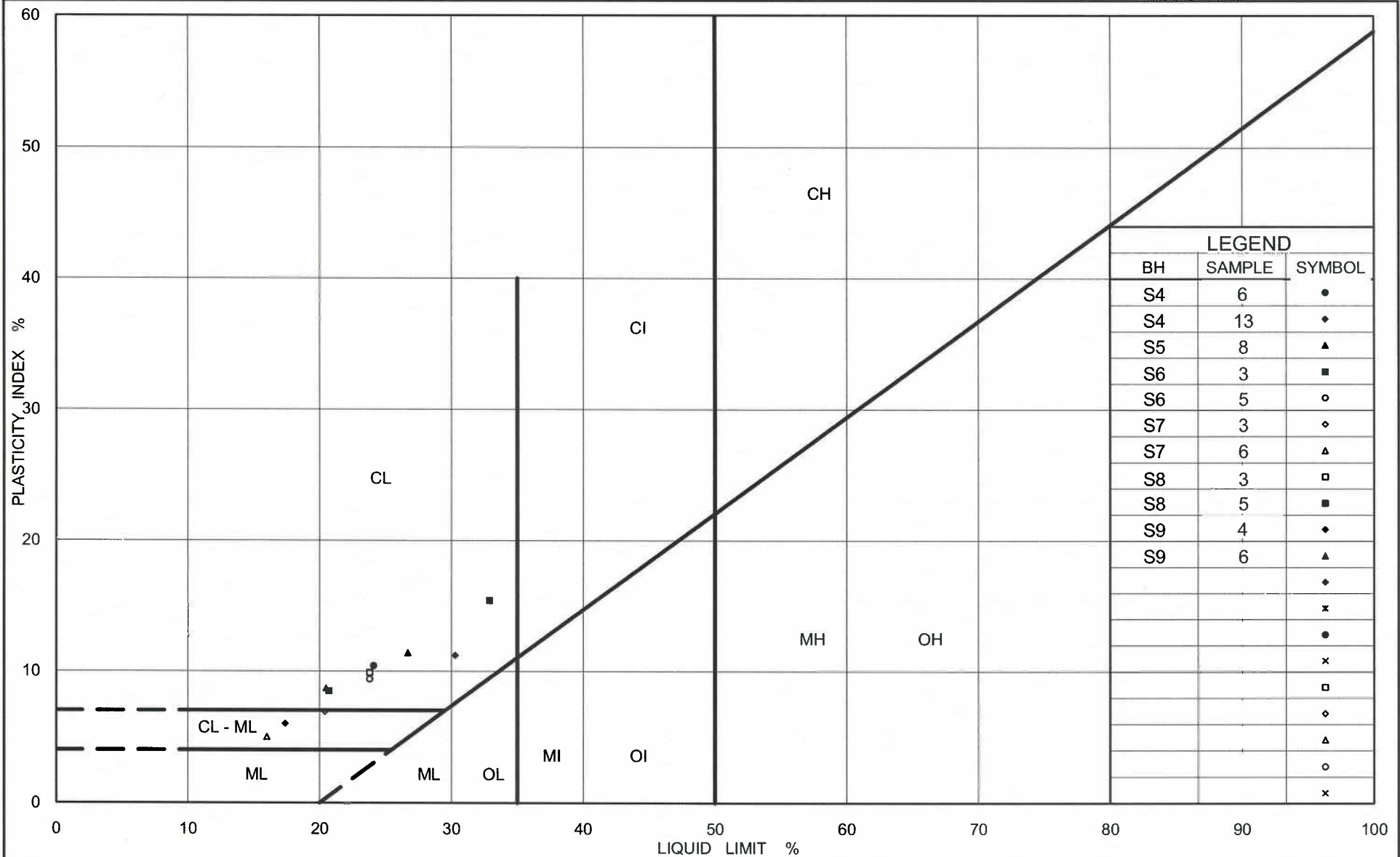
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	S7	10	166.7
■	S4	13	168.5
◆	S9	4	173.4
▲	S4	5	179.2
▽	S8	5	172.5
○	S6	5	174.2

Project Number: 06-1111-012-1

Checked By: *SM*

**Golder Associates**

Date: 04-Aug-09



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## PLASTICITY CHART

### Clayey Silt Till

Figure No. B2

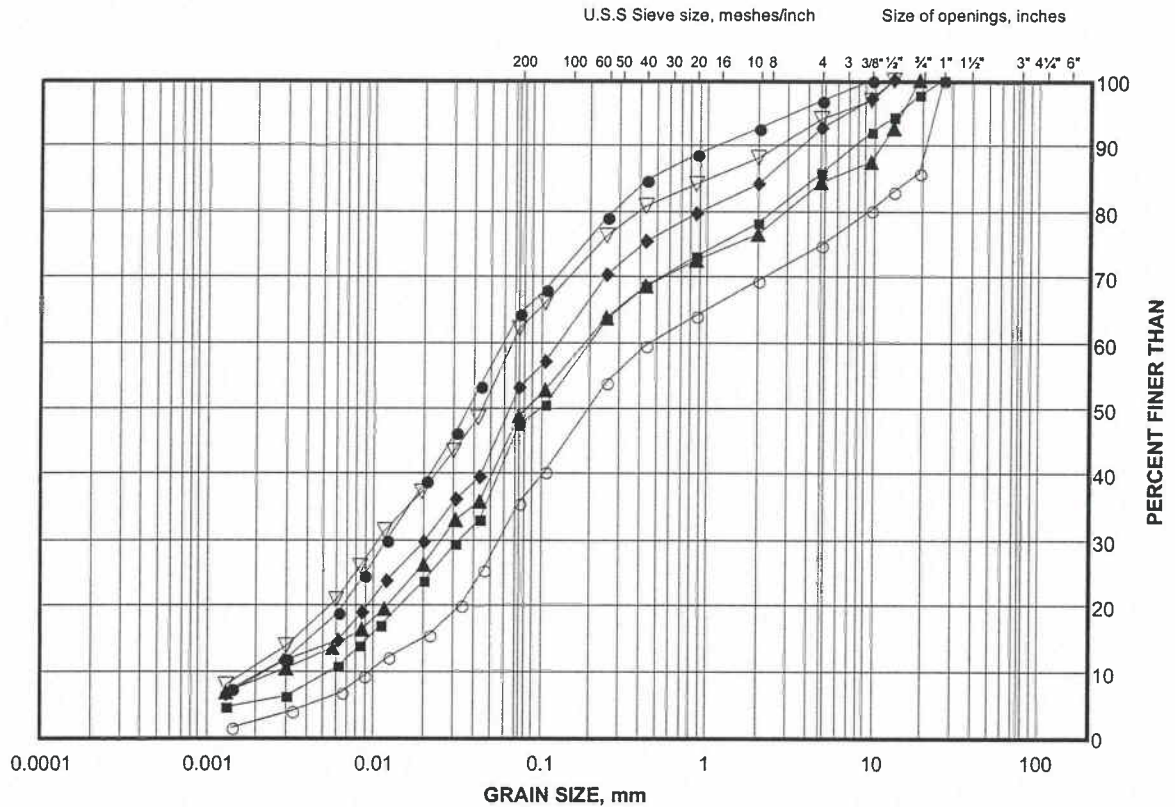
Project No. 06-1111-012-1

Checked By: *STM*

# GRAIN SIZE DISTRIBUTION TEST RESULTS

Sand and Silt Till

FIGURE B3



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

## LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	S6	10	168.3
■	S5	11	170.7
◆	S9	7	171.2
▲	S8	7	171.2
▽	S7	8	169.6
○	S6	8	171.2

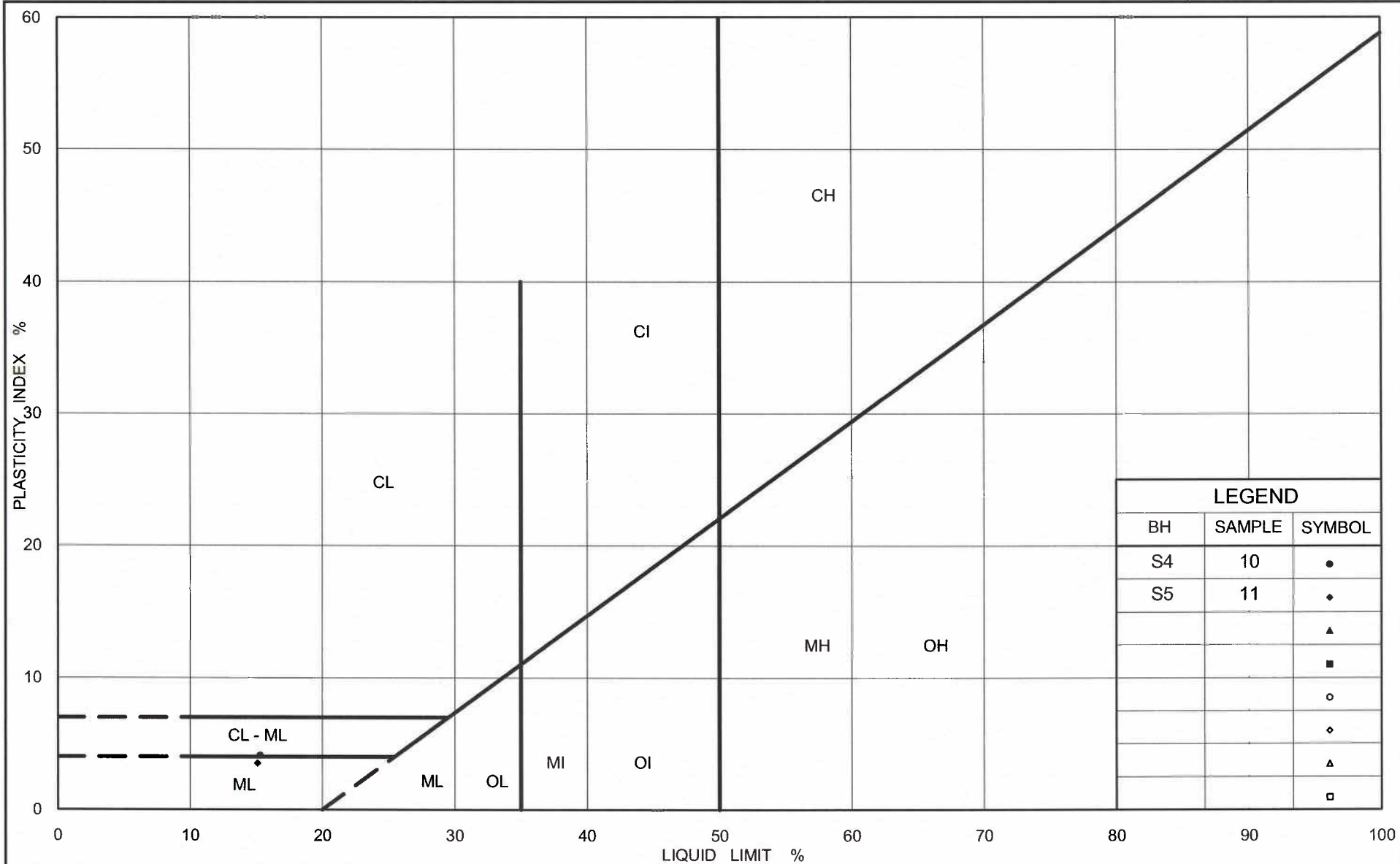
Project Number: 06-1111-012-1

Checked By: *SM*

Golder Associates

Date: 29-May-09





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# PLASTICITY CHART Sand and Silt Till

Figure No. B4

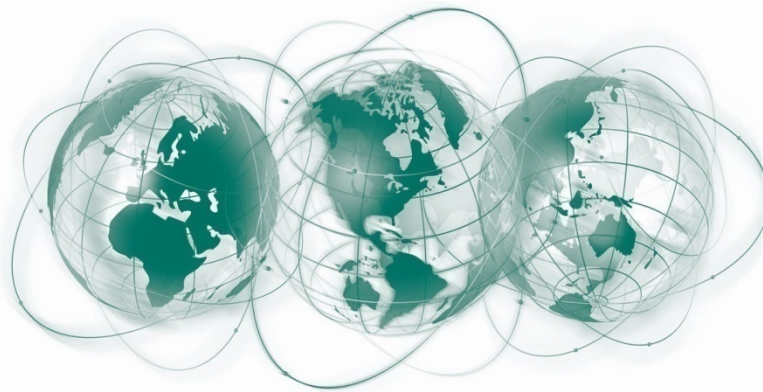
Project No. 06-1111-012-1

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