



AUGUST 2009

PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT

**RAINBOW CREEK BRIDGE ON LANGSTAFF ROAD
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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PART A

**PRELIMINARY FOUNDATION INVESTIGATION REPORT
RAINBOW CREEK BRIDGE ON LANGSTAFF ROAD
HIGHWAY 427 EXTENSION
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PRELIMINARY FOUNDATION REPORT RAINBOW CREEK BRIDGE ON LANGSTAFF ROAD

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 Langstaff Road bridge over Rainbow Creek, and the immediate approach embankments to this bridge. 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 West Rainbow Creek bridge is located approximately 2.1 km south of Rutherford Road and approximately 1.6 km north of Zenway Boulevard in the City of Vaughan, Ontario. The proposed structure site is approximately 1.5 km west of Highway 27 and 600 m 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 bridge and associated approach embankments are to be situated within the Rainbow Creek west valley and floodplain area east of the creek. Currently Rainbow Creek flows through a corrugated steel pipe (CSP). Rainbow Creek, at the site of the proposed bridge, is approximately 3 m to 4.5 m wide and approximately 0.6 m to 1.0 m deep. The west valley slope is about 6.8 m high relative to the creek level; which is at about Elevation 178.2 m. The west valley slope, which is moderately to heavily treed, slopes from about Elevation 185.0 m down to the creek level at a gradient of 1.1 horizontal to 1 vertical (1.1 H:1V). East of the creek, the floodplain is about 68 m wide and the ground surface gradually slopes up from about Elevation 179 m to 181 m.

No evidence of surficial or deep-seated slope instability was observed on the west valley slope at the time of the borehole investigation at this site.

3.0 INVESTIGATION PROCEDURES

The field work for the Rainbow Creek bridge structure site was carried out in March 2009, during which time a total of two boreholes were advanced at the locations shown on Drawing 1.

The field investigation for the boreholes was carried out along the north and south sides of existing Langstaff Road using a truck-mounted D-90 drill rig supplied and operated by Walker Drilling Ltd. of Utopia, Ontario. These boreholes were advanced using 200 mm outside diameter hollow-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 typically terminated after penetrating at least 3 m into hard soil having Standard Penetration Test (SPT) 'N' values of greater than 100 blows per 0.3 m of penetration or into shale bedrock where



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encountered. Borehole S10 located approximately near the crest of the west valley slope was drilled to a depth of 8.1 m and Borehole S11 located within the area of the east floodplain was advanced to a depth of 13.8 m.

The groundwater conditions in the open boreholes were observed during the drilling operations and a standpipe piezometer was installed in Borehole S11 to permit monitoring of the groundwater level at the site. The piezometer consisted of 51 mm diameter PVC pipe, with a slotted screen sealed at a select depth within the borehole. A sand filter pack surrounds the screen and above the screen the borehole and annulus surrounding the piezometer pipe were backfilled to the surface with bentonite pellets/grout. The piezometer installation details and water level readings are described on the Record of Borehole Sheets in Appendix A. Borehole S10 in which no standpipe piezometer was installed was 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 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 relative to Rainbow creek. The as-drilled borehole locations and ground surface elevations were surveyed by MRC. 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.

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*¹. 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.

¹ 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.



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These boundaries, therefore, represent transitions between soil types rather than exact planes of geological change.

The interpreted stratigraphic conditions along Langstaff Road at the proposed Rainbow Creek bridge site are shown on Drawing 1. This stratigraphic profile represents 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 in the area of the proposed Rainbow Creek bridge consist of asphalt and up to about 0.8 m of fill, overlying an upper clayey silt deposit (where present). The fill and the upper clayey silt are underlain by a clayey silt till deposit that grades to sand and silt till. The sand and silt till grades with depth to a lower clayey silt till deposit in the borehole drilled within the west valley slope. In the borehole drilled east of Rainbow Creek the sand and silt till was underlain by hard clayey silt. Shale bedrock was encountered underlying the clayey silt till in Borehole S11.

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

4.2.1 Asphalt

Approximately 0.1 m of asphalt was encountered immediately below ground surface in both boreholes, which were drilled through the existing Langstaff Road pavement at this site.

4.2.2 Sand Fill

In Boreholes S10 and S11, the asphalt is underlain by a layer of sand fill containing trace gravel. The sand fill extends to a depth of 0.8 m (Elevations 182.6 m and 180.1 m in Boreholes S10 and S11, respectively).

4.2.3 Upper Clayey Silt

An upper deposit of clayey silt was encountered below the sand fill in Borehole S11 drilled within the floodplain area. The cohesive deposit extends to a depth of 3.0 m below ground surface, corresponding to Elevation 177.9 m.

The clayey silt contains trace to with sand, trace gravel; and the upper portion of this deposit is slightly organic and contains rootlets. The result of one grain size distribution test is presented on Figure B1 in Appendix B. An Atterberg limits test was carried out on one sample of the clayey silt deposit and a measured plastic limit of 20 percent, a liquid limit of 35 percent and a plasticity index of 15 percent. These results, which are plotted on a plasticity chart on Figure B2 in Appendix B, confirm that the material is a clayey silt of low to medium plasticity. Measured water contents on selected samples of the clayey silt are typically about 21 percent but a higher water content of about 179 percent was measured within the upper portion of the deposit containing organic material.

Measured SPT 'N' values within the upper clayey silt were 6 and 8 blows per 0.3 m of penetration, indicating that the clayey silt has a firm to stiff consistency.

4.2.4 Till Deposits

In the boreholes drilled at this site, the fill and the clayey silt deposit are underlain by a clayey silt till deposit that grades with depth to a cohesionless till deposit; in Borehole S11, the cohesionless till deposit grades back into a



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cohesive till deposit. Till deposits in southern Ontario typically contain cobbles and/or boulders. Cobbles and/or boulders were encountered within the upper clayey silt till deposit in the boreholes drilled at this site.

4.2.4.1 Clayey Silt Till (Upper Till)

The surface of the clayey silt till was encountered at depths of 0.8 m and 3.0 m in Boreholes S10 and S11, respectively. The clayey silt portion of the till deposit extended to depths of 3.9 m and 4.5 m in Boreholes S11 and S10, respectively. The base of the cohesive till was encountered in the Boreholes S11 and S10 at approximately Elevations 177.7 m and 178.9 m, respectively.

The upper cohesive till consists of clayey silt some sand, trace gravel and cobbles. An Atterberg limits test was carried out on one sample of the clayey silt till deposit and the measured plastic limit was 15 percent, the liquid limit was 31 percent, and the corresponding plasticity index was 16 percent. These results, which are plotted on a plasticity chart on Figure B3 in Appendix B, confirm that this till deposit is clayey silt of low plasticity. Measured water contents on selected samples of the clayey silt till were about 11 and 12 percent.

The SPT 'N' values measured within the upper clayey silt till typically ranged from 11 blows to 88 blows per 0.3 m of penetration, indicating a stiff to 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 at about Elevations 178.9 m and 177.0 m. The cohesionless till extended to depths of 5.9 m and 7.0 m in Boreholes S10 and S11, respectively. The base of the cohesionless till was encountered in the boreholes at Elevations 177.5 m and 173.9 m, corresponding to thickness of about 1.4 m and 3.1 m.

The cohesionless portion of the till deposit consists of sand and silt containing trace to some gravel and trace clay. A grain size distribution test was carried out on one selected sample of the sand and silt till deposit and the results are presented on Figure B4 in Appendix B. Measured water contents on samples of the sand and silt till were about 6 percent and 7 percent.

Within the cohesionless till the SPT 'N' values measured ranged from 86 blows to greater than 100 blows per 0.3 m of penetration, indicative of till with a very dense relative density.

4.2.4.3 Clayey Silt Till (Lower Till)

A lower cohesive till deposit was encountered underlying the cohesionless portion of the till in Borehole S11 at a depth of 7.0 m (Elevation 173.9 m). This portion of the deposit was 5.2 m thick and extended to a depth of 12.2 m (Elevation 168.7 m). The lower cohesive till consists of clayey silt with sand to some sand, trace to some gravel and contains shale fragments in the lower 1.8 m of the deposit.

A grain size distribution test was carried out on one selected sample of the clayey silt till deposit and the results are presented on Figure B5 in Appendix B. Atterberg limits testing was carried out on two samples of the lower clayey silt till deposit and measured plastic limits of 15 and 17 percent, liquid limits of 25 and 33 percent, and plasticity indices of 10 and 16 percent. These results, which are plotted on a plasticity chart on Figure B3 in Appendix B, confirm that this portion of the till is a clayey silt of low plasticity. The measured water contents on samples of the lower clayey silt till ranged from about 9 to 12 percent.

The SPT 'N' values measured within the lower clayey silt till varied from 55 blows to greater than 100 blows per 0.3 m of penetration, indicating that the lower clayey silt till has a hard consistency.



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4.2.5 Lower Clayey Silt

In Borehole S10, a cohesive deposit of clayey silt was encountered underlying the sand and silt till deposit. Borehole S10 terminated in this deposit at a depth of 8.1 m below ground surface, (Elevation 175.3 m). A grain size distribution test was carried out on one selected sample of the clayey silt deposit and the results are presented on Figure B6 in Appendix B. An Atterberg limits test was carried out on one sample of the clayey silt deposit and a measured plastic limit of 17 percent, a liquid limit of 24 percent and a plasticity index of 7 percent. These results, which are plotted on a plasticity chart on Figure B7 in Appendix B, confirm that the material is a clayey silt of low plasticity. A measured water content on one sample of the clayey silt deposit was about 11 percent.

The SPT 'N' values measured within the clayey silt deposit were 101 blows and 171 blows per 0.3 m of penetration, indicating that the clayey silt has a hard consistency.

4.3 Shale Bedrock

Bedrock was encountered and split spoon samples were recovered from Borehole S11 at 12.2 m depth (Elevation 168.7 m). The bedrock samples consisted of light grey to dark grey shale. Based on bedrock geology mapping, the bedrock at this site is understood to be part of the Georgian Bay Formation.

4.4 Groundwater Conditions

The water level in Borehole S11 as noted during and upon completion of drilling operations was at about Elevation 173.1 m, though the level had not stabilized. Borehole S10 was dry upon completion of drilling. In general, the samples taken in the boreholes were wet.

A standpipe piezometer was installed in Borehole S11 to permit monitoring of the groundwater level at this site. Details of the piezometer installation are shown on the borehole record in Appendix A. The groundwater level measured in the piezometer installation up to approximately seven weeks following completion of drilling are summarised below.

Borehole No.	Ground Surface Elevation	Depth to Groundwater Level	Groundwater Elevation	Date of Measurement
S11 (floodplain)	180.9 m	0.0 m (at ground surface)	180.9 m	April 24, 2009
		0.0 m (at ground surface)	180.9 m	May 13, 2009
		0.0 m (at ground surface)	180.9 m	June 15, 2009
		0.0 m (at ground surface)	180.9 m	July 9, 2009

The normal water level in the creek is understood to be at about Elevation 178.0 m and the high water level is at about Elevation 181.0 m. The groundwater level 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.



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5.0 CLOSURE

The field investigation program at this site was arranged and supervised by Mr. Suresh Bainey. This report was prepared by Ms. Veronica Ayetan, E.I.T., and reviewed by Ms. Sandra McGaghran, P.Eng., a geotechnical engineer 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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RAINBOW CREEK BRIDGE ON LANGSTAFF ROAD**

PART B

**PRELIMINARY FOUNDATION DESIGN REPORT
RAINBOW CREEK BRIDGE ON LANGSTAFF ROAD
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 bridge structure on the Langstaff Road 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 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 bridge is proposed to consist of a two-span structure. Based on the preliminary General Arrangement (GA) Drawing provided by MRC on May 15, 2009, the west abutment will be located near the crest of the west valley slope and the east abutment and the central pier are proposed to be located within the floodplain area (east of the creek). Between the centre pier and the abutments the proposed span lengths are approximately 36 m. The as-drilled locations of the boreholes were laid based on the originally proposed borehole locations.

According to the preliminary GA Drawing, the finished grade of Langstaff Road over Rainbow Creek varies from approximately Elevation 186.2 m to 188.0 m, rising eastward. At the proposed west abutment the ground surface is currently at about Elevation 185 m and slopes down to Rainbow Creek. Based on the preliminary GA a 1 m to 2 m cut is proposed along the west valley slope. The height of the proposed west approach embankment will vary from about 1 m to 3 m. At the proposed east abutment the current grade is at about Elevation 180.4 m; considering that Langstaff Road will be raised to approximately Elevation 187.8 m, this results in a 7.2 m high approach embankment.

6.2 Foundation Recommendations

6.2.1 Foundation Options

Based on the proposed vertical elevations and subsurface soil conditions, the following foundation options have been considered for the proposed Rainbow Creek bridge:

- **Spread footings founded on the hard clayey silt till:** This option is feasible at the centre pier and west abutment. At the centre pier the existing ground surface south of Langstaff Road is at approximately Elevation 179.3 m. At the centre pier locations footings would have to extend through the firm to stiff clayey silt to be founded on the hard clayey silt till at the centre pier location; hard clayey silt till was encountered at approximately Elevation 177.9 m, which is approximately 1.4 m below the existing ground surface south of Langstaff Road (therefore no additional subexcavation is required other than that required for frost protection). At the west abutment the ground surface along the west slope is proposed to be lowered to



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approximately Elevation 183.4 m; footings would have to extend through the fill and the surficial stiff portion of the clayey silt till to found on the hard clayey silt till. Hard clayey silt was encountered at approximately Elevation 182.0 m, therefore for a proposed grade at Elevation 183.4 m excavations would be required to extend to a depth of about 1.4 m below the future grade (therefore no additional subexcavation is required other than for frost). Considering that the grade at the east abutment is to be raised by about 7.2 m this option is considered neither economical nor feasible at the east abutment given the resulting height of abutment walls and the predicted settlement of the foundations soils under the east approach embankment.

- **Spread footings “perched” on a granular pad within the approach embankment fill:** This option could be adopted to support the east abutment for an open structure, with 2 horizontal to 1 vertical (2H:1V) foreslopes in front of the abutment footings. The loading from the new east approach embankment would result in some settlement in the upper, stiff portion of the clayey silt deposit, which could result in differential settlement between the east abutment and the centre pier. The time required for preloading should be considered at the construction stage if traffic along Langstaff Road is to be maintained. At the west abutment, the ground surface along the existing slope is proposed to be lowered, therefore the difference in elevation between the proposed ground surface at the west abutment and the surface of the hard clayey silt till is only about 1.4 m; for “perched” abutments a minimum of 2 m of Granular ‘A’ is required below the underside of the “perched” footing. Therefore this option is not practical for support of the west abutment.
- **Steel H-piles driven to found within the till deposit:** This option could be adopted to support the abutments and centre pier 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 till deposit or bedrock:** This option could be adopted to support the abutments and centre pier in either a conventional or a semi-integral abutment-type structure.

At the east abutment, either “perched” footing or steel H-piles are preferred over spread footing founded on the native soils due to the resulting height of the abutment wall; while at the west abutment, spread footings on hard clayey silt till or steel H-piles are preferred over “perched” footings since “perched” footings at the west abutment are not practical as there is only about 1.4 m between the proposed ground surface and the surface of the hard clayey silt till. At the centre pier, spread footings on hard clayey silt till are preferred if sufficient geotechnical resistance can be achieved; otherwise, support of the centre pier on deep foundations will be required to achieve a higher capacity. The use of piles is preferred from a deep foundation perspective over caissons for support of the centre pier, as the caissons would have to go through the water-bearing sand and silt till to terminate within the clayey silt till, which would require special construction procedures.

Recommendations for preliminary design of spread footing, 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 the hard portion of the clayey silt till deposit.



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6.2.2.1 Founding Elevations

The west abutment and centre pier may be supported on spread footings placed below the firm to stiff clayey silt and stiff clayey silt till, on hard clayey silt till. A minimum founding depth of 1.4 m is required for frost protection purposes (OPSD 3090.101). Preliminary recommendations for maximum (highest) founding elevations are provided in the following table.

Foundation Element	Borehole	Founding Stratum	Maximum Founding Elevation
West Abutment	S10	Stiff to Hard Clayey Silt Till	182.0 m
Centre pier	S11	Hard Clayey Silt Till	177.9 m

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 upper 1 m to 3 m of the 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 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 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 east abutment wall, spread footings for the east bridge abutment may be placed on a compacted Granular ‘A’ pad constructed within the approach embankment fill. The following sections provide geotechnical resistances for spread footing at the east abutment that are “perched” within the approach embankment fill on a compacted granular pad.



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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 (OPSD 3090.101).

6.2.3.2 *Geotechnical Resistances*

At the proposed east abutment area it is estimated that approximately 50 mm of settlement will occur under the loading from the proposed approach embankment, primarily in the firm to stiff clayey silt between Elevation 180 m and 178 m. If “perched” spread footings are adopted for the support of the east abutment, it will be necessary to preload the approach embankment area before construction of the footings and bridge structure to mitigate settlement at the abutments and to minimize differential settlement between the abutment and the centre pier.

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.

Assuming the above preloading 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, 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 footing. 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 driven to found within the till deposit are provided in the subsections that follow.

For the installation of steel H-piles, consideration will have to be given to the potential 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.



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6.2.4.1 Founding Elevations

Steel H-piles driven to found within the hard clayey silt and hard clayey silt till deposit may be used for support of the abutments and centre pier. "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 3.9 m and 4.5 m depth below ground surface. For integral abutments the pile length is required to be a minimum of 5 m below the underside of the pile cap. Therefore for integral abutments the pile tip should be extended to Elevation 177.1 m at the west abutment and Elevation 173.1 m at the centre pier and east abutment. Therefore, to reach these minimum elevations the piles would be required to drive through 1.8 m of very dense sand and silt till and hard clayey silt at the west abutment and through 3.9 m of very dense sand and silt till and hard clayey silt till at the centre pier and east abutment. Given the length of "100-blow" material that the piles would have to drive through it may be necessary to pre-auger the holes. The table below summarizes the recommended 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.

Foundation Unit	Borehole No.	Founding Stratum	Recommended Pile Tip Elevation Based on Minimum Pile Length of 5 m
West Abutment	S10	Hard Clayey Silt	177.1 m
Centre pier and East Abutment	S11	Hard Clayey Silt Till	173.1 m

6.2.4.2 Axial Geotechnical Resistances

For HP 310x110 piles driven a minimum of 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 recommended tip elevations provided in Section 6.2.4.1, or

Founding Material	Foundation Unit	Factored Geotechnical Resistance at ULS	Geotechnical Resistance at SLS
Hard Clayey Silt	West Abutment	1,000 kN	800 kN
Hard Clayey Silt Till	Centre pier and East Abutment	900 kN	700 kN

The pile capacity values provided above are not high due to the high water table present at this site and the shallow depth to refusal. These 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, particularly at the west abutment where it is necessary to determine the depth to shale bedrock.

At the proposed west approach embankment, it is estimated that less than 25 mm of settlement will occur. At the proposed east abutment area, it is estimated that about 50 mm of settlement will occur, primarily in the firm to stiff clayey silt, under the proposed loading from the approach embankment. For preliminary design purposes it is recommended that a downdrag load of 150 kN be included for the piles supporting the east abutment, 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*.



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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 hard clayey silt/clayey 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 clayey silt or clayey silt till having SPT 'N' values greater than 100 blows per 0.3 m of penetration for support of the foundation elements for the bridges. Consideration could also be given to socketting the caissons into shale bedrock, if higher geotechnical resistances are required. 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 along the caisson length or 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

Caissons founded within the hard clayey silt or clayey silt till may be used for support of the abutments and centre pier. It is not recommended to found the caissons higher as the base would be within the sand and silt till which may experience disturbance at the base due to the high water table and the lack of cohesion. At the east abutment and centre pier, caissons may be found within the lower hard clayey silt till material at approximately Elevation 173.0 m; and at the west abutment, caissons could be found within the hard clayey silt material at approximately Elevation 176.0 m. Alternatively, the east abutment and centre pier could be supported on



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caissons founded on the shale bedrock at approximately Elevation 168.7 m. The table below summarizes the depth to competent soil to found the caissons (as encountered in the boreholes) and the associated elevations.

Foundation Unit	Borehole No.	Founding Stratum	Estimated Caisson Elevation
West Abutment	S10	Hard Clayey Silt	176.0 m
Centre pier and East Abutment	S11	Hard Clayey Silt Till	173.0 m

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 hard clayey silt/clayey silt till at the elevations given in Section 6.2.5.1 at the abutment and centre pier locations.

Foundation Unit	Founding Material	Caisson Diameter	Factored Geotechnical Resistance at ULS	Geotechnical Resistance at SLS
West Abutment	Clayey Silt	0.9 m	1,400 kN	1,200 kN
		1.2 m	2,600 kN	2,200 kN
		1.5 m	4,100 kN	3,400 kN
Centre pier and East Abutment	Clayey Silt Till	0.9 m	1,200 kN	1,000 kN
		1.2 m	2,200 kN	1,800 kN
		1.5 m	3,400 kN	2,800 kN
Centre pier and East Abutment	Shale Bedrock	0.9 m	5,000 kN	N/A
		1.2 m	9,000 kN	
		1.5 m	14,000 kN	

At the proposed west approach embankment, it is estimated that less than 25 mm of settlement will occur. At the proposed east abutment areas, it is estimated that about 50 mm of settlement will occur, primarily in the firm to stiff clayey silt, under the proposed loading from the east approach embankment. For preliminary design purposes it is recommended that the following downdrag loads be included in the design for caissons supporting the east abutment:

Caisson Diameter	Downdrag Load
0.9 m	350 kN
1.2 m	550 kN
1.5 m	750 kN

Further investigation and assessment will be required during detail design stage. The structural capacity of the caissons must be checked for the factored dead and downdrag loads in accordance with Section 6.8.4 of the *CHBDC*.



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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.
- 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:



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	Earth Fill
Soil unit weight:	20 kN/m ³
Coefficients of static lateral earth pressure:	
Active, K_a	0.33
At rest, K_o	0.50

- For Case 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 ³	21 kN/m ³
Coefficients of static lateral earth pressure:		
Active, K_a	0.27	0.27
At rest, K_o	0.43	0.43

If the wall support and superstructure allow lateral yielding of the stem, active earth pressures 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:



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

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;
	γ'	is the effective unit weight of the soil (kN/m^3)
		<ul style="list-style-type: none"> taken as soil unit weights given above for fill materials taken as 20 kN/m^3 for the native materials
	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 bridge structure will require placement of up to about 3.3 m of fill within the limits of the west approach and up to about 7.2 m of fill within the limits of the east approach embankment.

Based on the results of the boreholes drilled at this site, the approach embankments will be founded on 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 all topsoil, organic matter, existing fill and any softened/loosened native soils should be stripped from below the approach embankment areas. Embankment fill should be placed and compacted in accordance with MTO's SP 206S03 and SP 105S10. To reduce erosion of the embankment side slopes due to surface



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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 Slide (version 5.035), produced by Rocscience Inc., employing the Morgenstern-Price method of analysis. The stability analyses were performed 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 approximately Elevations 180.9 m.

Soil Type	Unit Weight (kN/m ³)	Undrained Shear Strength (kPa)	Cohesion, c' (kPa)	Angle of Internal Friction, ϕ' (degrees)
New Earth or Granular Fill	21	--	--	34
Sand Fill	20	--	--	32
Firm to Stiff Clayey Silt	19	50	--	28
Hard Clayey Silt Till	21	150	--	34
Hard Clayey Silt Till	21	--	--	34
Very Dense Sand and Silt Till	21	--	--	34
Hard Clayey Silt Till	21	--	--	34

With appropriate subgrade preparation and proper placement and compaction of embankment fill materials, the proposed 3.3 m to 7.2 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 result of an example static stability analysis at the east embankment is 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. The total settlement within the founding soils has been estimated based on the existing site-specific subsoil conditions for preliminary design using the commercially available program Settle 3D, 2008 (Version 2.0) produced by Rocscience Inc. Provided that the embankment material consists of clean earth fill or granular fill, the settlement of the 3.3 m to 7.2 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.



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The settlement of the foundations soils under the west approach embankment is anticipated to be less than 25 mm. The settlement of the foundation soils under the east approach embankment loading is anticipated to be about 50 mm. It is estimated that it would take about three to six months to complete 90 percent of the settlement. 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 to stiff portion of the clayey silt deposit, consolidation parameters have been estimated based on correlation with Atterberg limits and experience with similar soil types in the Peel Plain.

Soil Deposit	Bulk Unit Weight (kN/m ³)	Elastic Modulus (MPa)	Consolidation Parameters
Embankment fill (range of parameters assumed for earth fill and granular fill)	20 – 22	--	--
Sand Fill	20	20	--
Firm to Stiff Clayey Silt	19	10	Cc = 0.25 Cr = 0.025
Very Stiff to Hard Clayey Silt Till	21	75	--
Hard Clayey Silt	21	100	--
Very Dense Sand and Silt Till	21	150	--

Settlement mitigation measures will be required to accommodate the predicted 50 mm of settlement of the founding soils, particularly if spread footings “perched” in the east approach embankments are adopted for support of the abutments, but also to address post-construction settlement that could impact the new Rainbow Creek bridge. Provided that there is sufficient time in the construction schedule, the simplest and most economical mitigation measure would be preloading the east approach embankment areas for a period of three to six months. If there is insufficient time available, the east approach embankment areas could be preloaded and surcharged with an additional 1 m to 2 m of fill, to shorten the preloading period.

Further examination of the predicted magnitude and time rate of settlement and the proposed mitigation measures will be required during detail design.

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 the depth to shale bedrock be determined at the foundation units if piles driven to bedrock is the preferred option. Further investigation should be completed to determine the extent and thickness of the firm to stiff portion of the clayey silt 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



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correlation purposes. Depending on the area 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.

6.5.2 Excavation

Depending on the foundation option adopted, excavations for the bridge foundations are expected to extend to depths of about 1.4 m below the proposed Rainbow Creek bridge cut grade and will be made through firm to stiff clayey silt and firm clayey silt till into hard clayey silt till. The firm to stiff clayey silt is considered Type 3 soil according to Occupational Health and Safety Act and Regulation for Construction Projects (OHSA). The excavation work should be carried out in accordance with the requirements of the OHSA, with side slopes no steeper than 1 horizontal to 1 vertical (1H:1V).

6.5.3 Groundwater and Surface Water Control for Foundation Excavation

The groundwater level measured in a standpipe piezometer at the site was at ground surface in April and May of this year, which may be lower during dry periods. Based on this observed high water level it is anticipated that some form of dewatering will be required during excavation, probably pumping from well filtered sumps at the perimeter of the excavation.

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 some of the 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.



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7.0 CLOSURE

This report was prepared by Ms. Veronica Ayetan, E.I.T., and reviewed by Ms Sandra McGaghran, P.Eng. a geotechnical engineer with Golder. Mr. Fintan Heffernan, P.Eng., Golder's Designated MTO Contact for this project, conducted an independent quality control review of the report.

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TABLE 1
COMPARISON OF FOUNDATION ALTERNATIVES – RAINBOW CREEK BRIDGE ON LANGSTAFF ROAD
HIGHWAY 427 EXTENSION
W.O. 05-20012

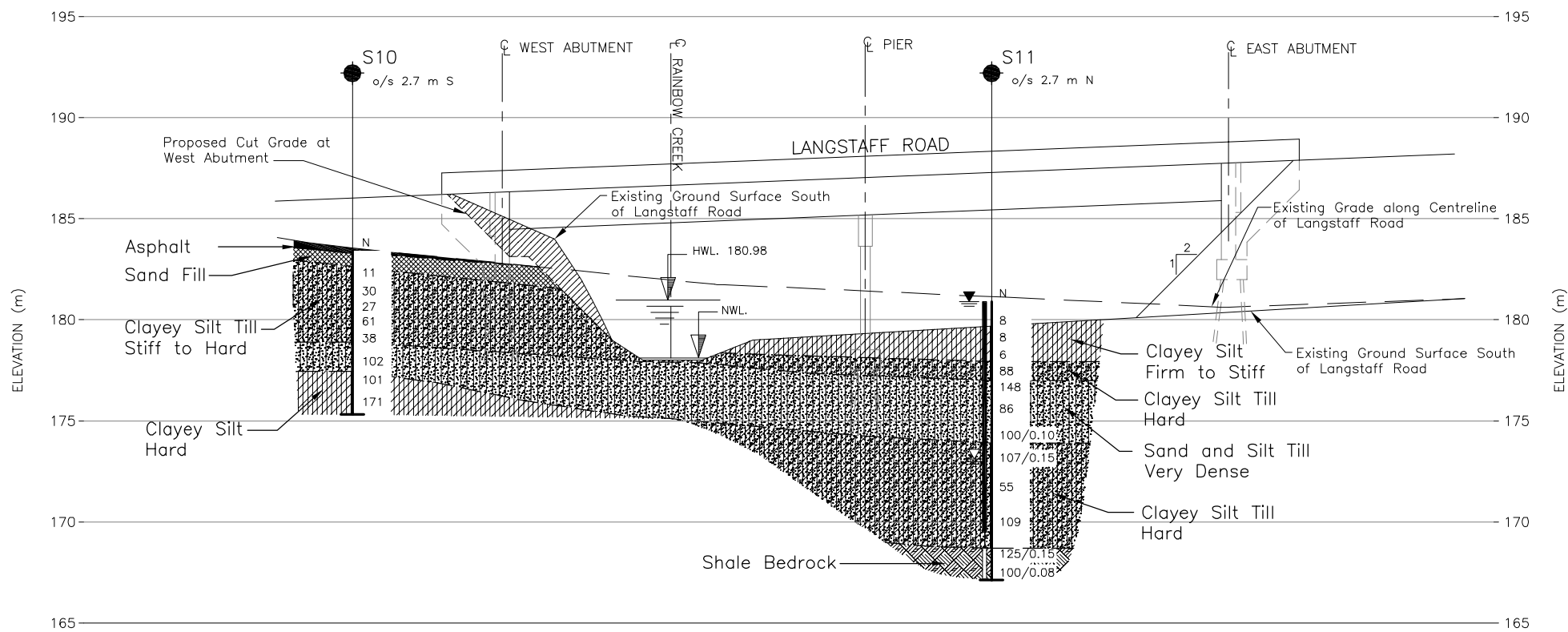
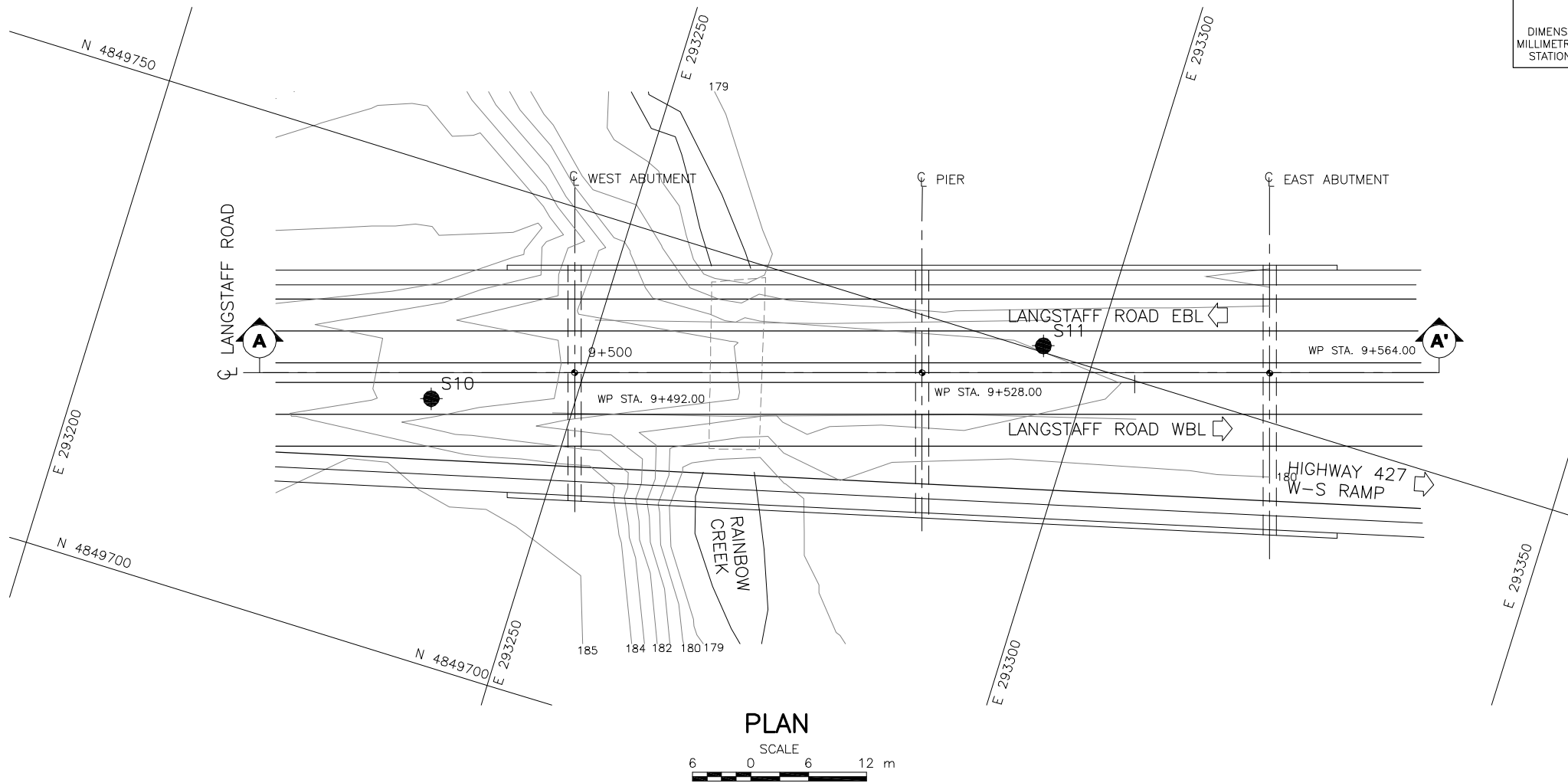
Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Spread footings on hard clayey silt till.	Feasible for support of centre pier and west abutment.	<ul style="list-style-type: none"> Relative ease of construction. 	<ul style="list-style-type: none"> Groundwater control required (can be controlled by pumping from well filtered sumps at the perimeter of the excavation depending on the time of year); and, Lowest bearing capacities of the four options. 	<ul style="list-style-type: none"> Lower relative cost than piled foundations. 	<ul style="list-style-type: none"> Low to moderate risk of some differential settlement between centre pier and abutments; and, Disturbance of subgrade soil due to ponded water.
Spread footings “perched” in approach embankment fill.	Feasible for support of east abutment.	<ul style="list-style-type: none"> Negligible post-construction settlement provided that preloading of approach embankment areas is completed; and, Construction maintained above groundwater level. 	<ul style="list-style-type: none"> Embankment preloading must be taken into account in construction schedule. 	<ul style="list-style-type: none"> Low cost option. 	<ul style="list-style-type: none"> Low to moderate risk that preloading period will extend beyond six months, impacting construction schedule for bridge Low to moderate risk of some differential settlement between abutments and pier; and, Must ensure proper compaction of Granular ‘A’ pad to minimize post-construction settlement
Steel H-pile foundations driven to found within hard clayey silt till and hard clayey silt.	Feasible for support of abutments and centre pier.	<ul style="list-style-type: none"> Sub-excavation is not required, Negligible post-construction settlement; and, Can be used for support of conventional or integral abutments. 	<ul style="list-style-type: none"> Downdrag loading must be taken into account in design, unless embankment areas are fully preloaded prior to construction of bridge structure, Piles may encounter obstructions (cobble and/or boulders) during driving; and, Monitoring of pile installation required to ensure presence of hard stratum at pile founding elevation. 	<ul style="list-style-type: none"> More costly than spread footings; and, Installation cost could be impacted by presence of obstructions. 	<ul style="list-style-type: none"> Negligible risk of post-construction settlement of bridge structure, or of differential settlement of foundation elements; and, Low to moderate risk of encountering obstructions that could impact pile installation.



PRELIMINARY FOUNDATION REPORT RAINBOW CREEK BRIDGE ON LANGSTAFF ROAD

TABLE 1 (Continued)
COMPARISON OF FOUNDATION ALTERNATIVES – RAINBOW CREEK BRIDGE ON LANGSTAFF ROAD
W.O. 05-20012

Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Caissons founded on hard clayey silt or hard clayey silt till.	Feasible at the centre pier and abutments.	<ul style="list-style-type: none"> Sub-excavation is not required; Negligible post-construction settlement; and, Can be used for support of conventional or semi-integral abutments. 	<ul style="list-style-type: none"> Downdrag loading must be taken into account in design, unless embankment areas are fully preloaded prior to construction of bridge structure; Need for temporary or permanent liners during installation through water-bearing sand and silt till to hard clayey silt/clayey silt till; and, Caissons may encounter obstructions (cobbles and boulders) during installation. 	<ul style="list-style-type: none"> More costly than spread footings or steel H-piles; and, Additional cost associated with specialised drilling equipment and temporary or permanent liners. 	<ul style="list-style-type: none"> Negligible risk of post-construction settlement of bridge structure, or of differential settlement of foundation elements, Low to moderate risk of disturbance of water-bearing sand and silt till soils, requiring special construction procedures including use of temporary or permanent liners; and, Low to moderate risk of encountering obstructions that could impact caisson installation.
Caissons founded on shale bedrock.	Feasible at the centre pier and abutments.	<ul style="list-style-type: none"> Sub-excavation is not required; Highest bearing capacity compared with caissons founded in hard clayey silt till; Negligible post-construction settlement; Can be used for support of conventional or semi-integral abutments. 	<ul style="list-style-type: none"> Downdrag loading must be taken into account in design, unless embankment areas are fully preloaded prior to construction of bridge structure; Need for temporary or permanent liners during installation through water-bearing sand and silt till to shale bedrock; and, Caissons may encounter obstructions (cobbles and boulders) during installation. 	<ul style="list-style-type: none"> More costly than spread footings or steel H-piles; and, Additional cost associated with specialised drilling equipment and temporary or permanent liners. 	<ul style="list-style-type: none"> Negligible risk of post-construction settlement of bridge structure, or of differential settlement of foundation elements, Low to moderate risk of disturbance of water-bearing sand and silt till soils, requiring special construction procedures including use of temporary or permanent liners; and, Low to moderate risk of encountering obstructions that could impact caisson installation.



PROFILE A-A' RAINBOW CREEK BRIDGE ON LANGSTAFF ROAD

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

CONT No.
WO No. 05-20012

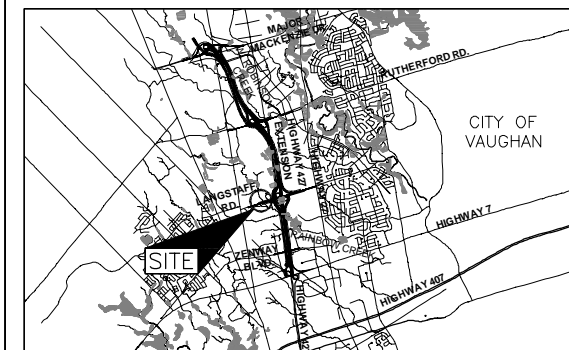
HIGHWAY 427 EXTENSION
RAINBOW CREEK BRIDGE ON LANGSTAFF ROAD
BOREHOLE LOCATIONS AND SOIL STRATA



SHEET



Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



KEY PLAN
SCALE
0 2 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
S10	183.4	4849726.6	293235.7
S11	180.9	4849750.6	293294.6

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.

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.

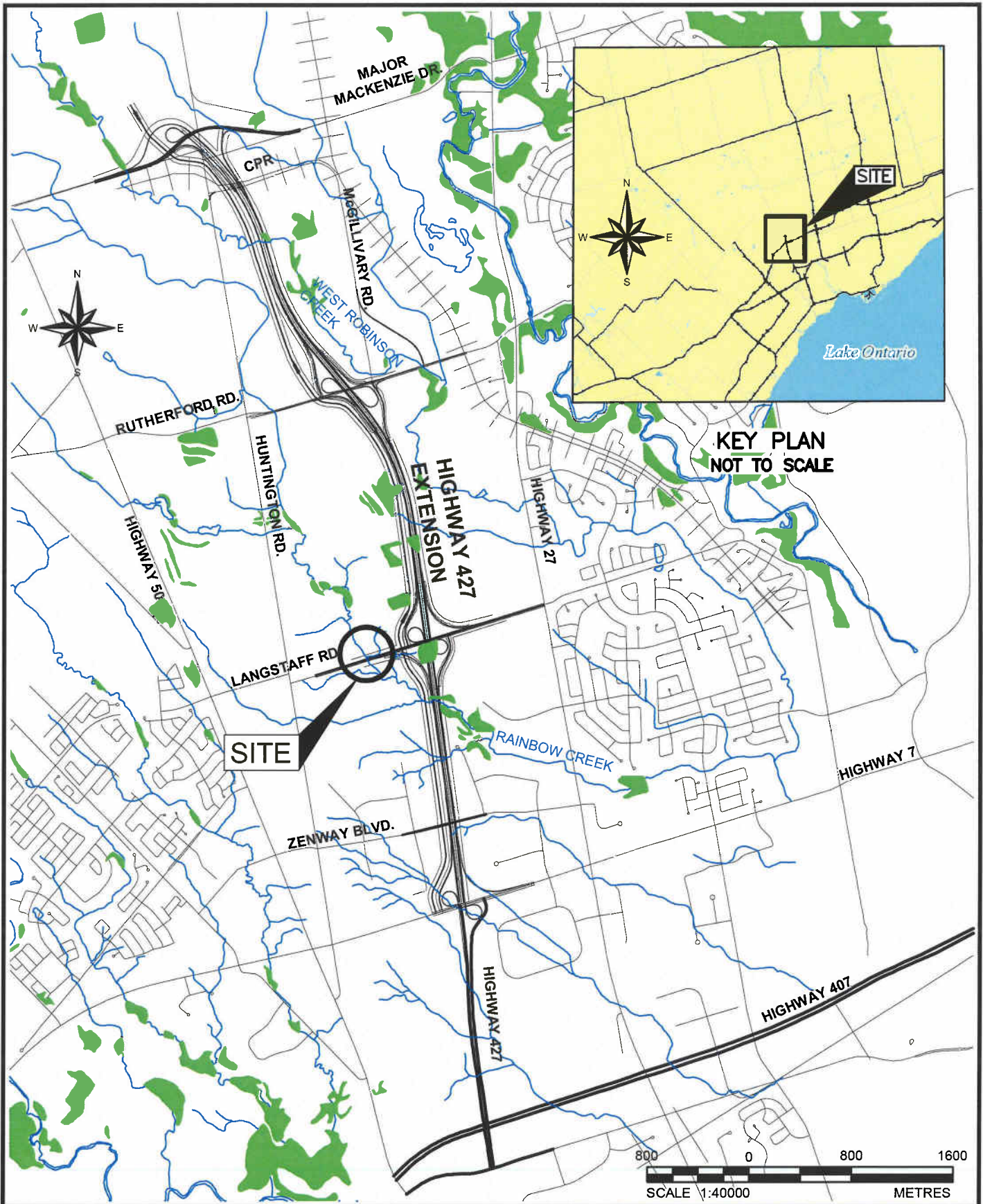
REFERENCE

Base plans provided in digital format by MRC (Drawing file no. "rainbow_ga2.dwg", received May 15, 2009).



NO.	DATE	BY	REVISION
Geocres No. 30M13-170			
HWY. 427	PROJECT NO. 06-1111-012-9		
SUBM'D. SB	CHKD. SMM	DATE: 4-Aug-2009	SITE:
DRAWN: JFC	CHKD. VA	APPD. LCC	DWG. 1

PLOT DATE: August 5, 2009
 FILENAME: T:\Projects\2006\06-1111-012 (MRC, Vaughan)\-JB- (RAINBOW CREEK on LANGSTAFF)\061111012JB0F1.dwg



SCALE	AS SHOWN
DATE	Aug. 5, 2009
DESIGN	PKS
CAD	JFC
CHECK	VA
REVIEW	<i>[Signature]</i> SMM

TITLE

SITE LOCATION PLAN RAINBOW CREEK AT LANGSTAFF ROAD

FILE No. 061111012JB0F1.dwg

PROJECT No. 06-1111-012-9 REV. B

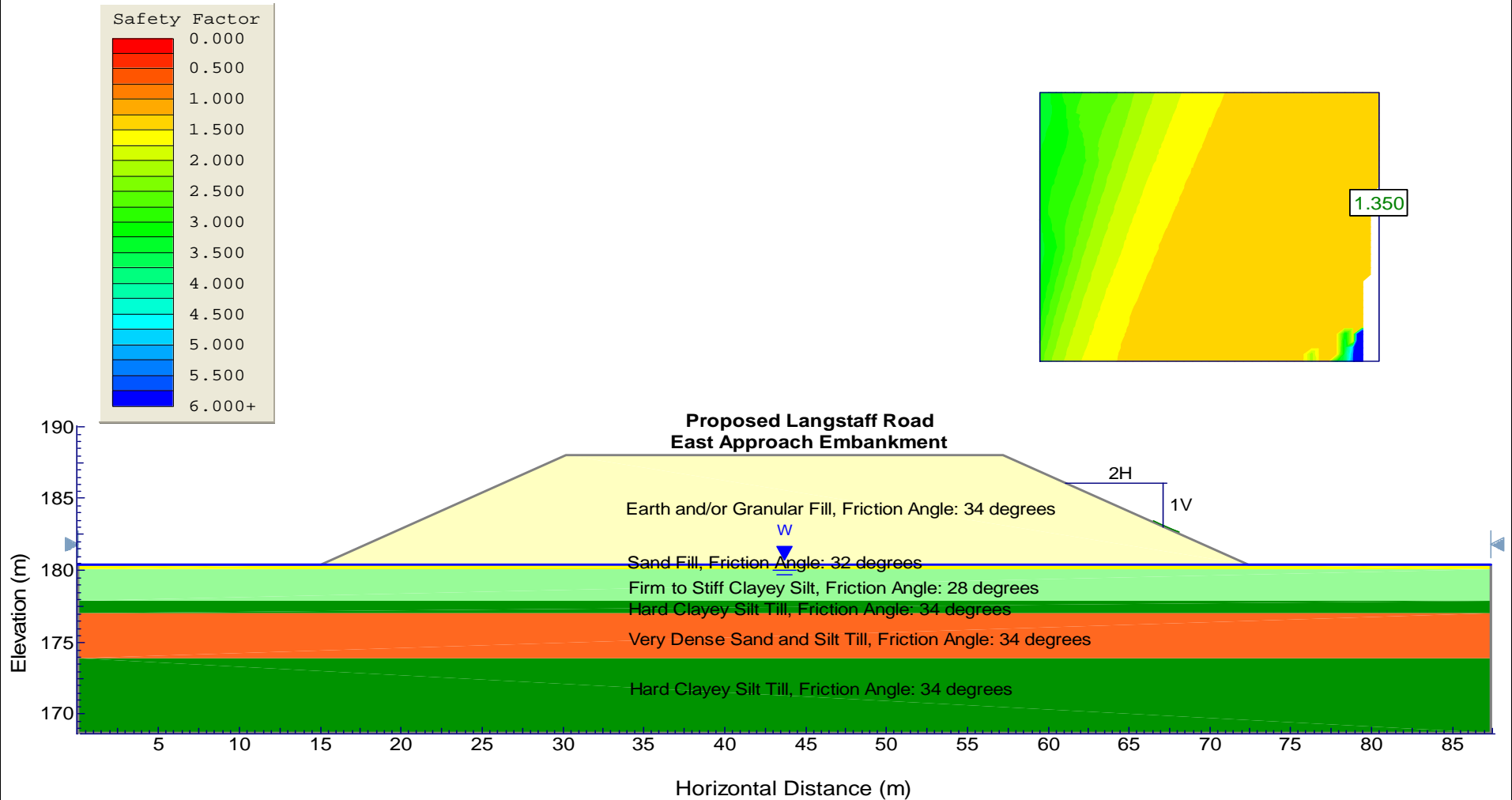
HIGHWAY 427 EXTENSION

FIGURE

1

**HIGHWAY 427 EXTENSION - RAINBOW CREEK BRIDGE ON LANGSTAFF ROAD
EAST APPROACH EMBANKMENT- STATIC GLOBAL STABILITY**

FIGURE 2



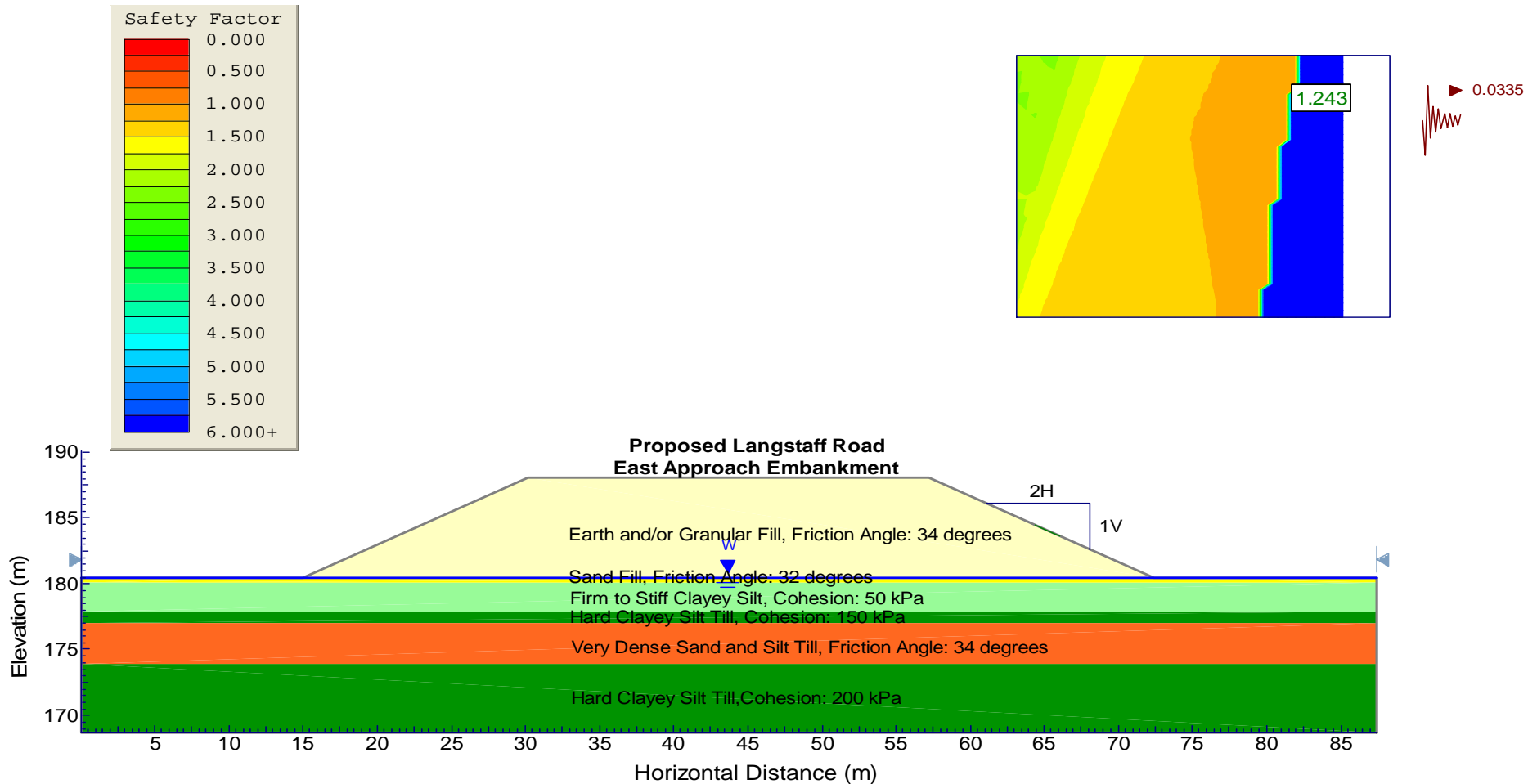
Date: July 2009
Project: 06-1111-012-9

Golder Associates

Drawn: VA
Checked: SMM

HIGHWAY 427 EXTENSION - RAINBOW CREEK BRIDGE ON LANGSTAFF ROAD EAST APPROACH EMBANKMENT- SEISMIC GLOBAL STABILITY

FIGURE 3



Date: July 2009
Project: 06-1111-012-9

Golder Associates

Drawn: VA
Checked: SMM



**PRELIMINARY FOUNDATION REPORT
RAINBOW CREEK BRIDGE ON LANGSTAFF ROAD**

APPENDIX A

Records of Boreholes



LIST OF SYMBOLS

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

I. GENERAL

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

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. in stress: $\Delta \sigma$
ε	linear strain
ε_v	volumetric strain
η	coefficient of viscosity
ν	poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma - \mu$)
σ'_{vo}	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stress (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ	shear stress
μ	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

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

* Density symbol is ρ . Unit weight symbol is γ where $\gamma = \rho g$ (i.e. mass density 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
σ'_p	pre-consolidation pressure
OCR	over-consolidation ratio = σ'_p / σ'_{vo}

(d) Shear Strength

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

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



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 cm² pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (Q_t), porewater pressure (PWP) and friction along a sleeve are recorded electronically at 25 mm penetration intervals.

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 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

IV. SOIL TESTS

w	water content
w_p	plastic limit
w_l	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test ¹
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement ¹
D_R	relative density (specific gravity, G_s)
DS	direct shear test
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
SO_4	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V	field vane (LV-laboratory vane test)
γ	unit weight

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

PROJECT <u>06-1111-012</u>		RECORD OF BOREHOLE No S10		1 OF 1 METRIC	
W.O. <u>05-20012</u>		LOCATION <u>N 4849726.6 : E 293235.7</u>		ORIGINATED BY <u>SB</u>	
DIST <u>Central</u> HWY <u>427</u>		BOREHOLE TYPE <u>200 mm Outside Diameter Hollow Stem Augers</u>		COMPILED BY <u>VA</u>	
DATUM <u>Geodetic</u>		DATE <u>March 20, 2009</u>		CHECKED BY <u>SMM</u>	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED						
183.4	GROUND SURFACE													
0.0	Asphalt													
0.2	Sand, trace gravel (FILL)													
182.6	Brown Moist													
0.8	CLAYEY SILT, some sand, trace gravel, containing cobbles (TILL) Stiff to hard Brown Moist		1	SS	11									
			2	SS	30									
			3	SS	27									
	Becoming grey at a depth of 2.3 m		4	SS	61									
			5	SS	38									
178.9	SAND and SILT, trace gravel, trace clay (TILL) Very dense Grey Moist		6	SS	102									
177.5	CLAYEY SILT Hard Grey Moist		7	SS	101									
175.3	END OF BOREHOLE		8	SS	171									
8.1	NOTES: 1. Open borehole dry 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 <u>06-1111-012</u>		RECORD OF BOREHOLE No S11		1 OF 2 METRIC	
W.O. <u>05-20012</u>	LOCATION <u>N 4849750.6 ; E 293294.6</u>	ORIGINATED BY <u>SB</u>			
DIST <u>Central</u> HWY <u>427</u>	BOREHOLE TYPE <u>200 mm Outside Diameter Hollow Stem Augers</u>	COMPILED BY <u>VA</u>			
DATUM <u>Geodetic</u>	DATE <u>March 20, 2009</u>	CHECKED BY <u>SMM</u>			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa	WATER CONTENT (%)					
180.9	GROUND SURFACE													
0.0	Asphalt													
0.2	Sand, trace gravel (FILL)													
180.1	Brown Moist													
0.8	CLAYEY SILT, some sand, trace gravel, slightly organic, containing rootlets		1	SS	8									
	Stiff													
	Dark grey													
	Moist		2	SS	8									
178.7	CLAYEY SILT with sand, trace gravel		3	SS	6									
2.2	Firm													
	Brown													
177.9	Moist		4	SS	88									
3.0	CLAYEY SILT, some sand, trace gravel, containing cobbles (TILL)													
	Hard													
	Grey													
177.0	Moist		5	SS	148									
3.9	SAND and SILT, trace gravel, trace clay (TILL)													
	Very dense													
	Grey													
	Moist		6	SS	86									
			7	SS	100/0.1									
173.9	CLAYEY SILT, some sand, trace gravel (TILL)		8	SS	07/0.1									
7.0	Hard													
	Grey and brown													
	Moist													
			9	SS	55									
170.5	CLAYEY SILT with sand, trace to some gravel, containing shale fragments (TILL)		10	SS	109									
10.4	Hard													
	Grey													
	Moist													
168.7	SHALE (BEDROCK)		11	SS	125/0.1									
12.2	Grey													
167.1	END OF BOREHOLE		12	SS	00/0.0									
13.8														

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

PROJECT <u>06-1111-012</u>		RECORD OF BOREHOLE No S11		2 OF 2 METRIC	
W.O. <u>05-20012</u>		LOCATION <u>N 4849750.6 ; E 293294.6</u>		ORIGINATED BY <u>SB</u>	
DIST <u>Central</u> HWY <u>427</u>		BOREHOLE TYPE <u>200 mm Outside Diameter Hollow Stem Augers</u>		COMPILED BY <u>VA</u>	
DATUM <u>Geodetic</u>		DATE <u>March 20, 2009</u>		CHECKED BY <u>SM</u>	

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL																	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED																						
	<p style="text-align: center;">— CONTINUED FROM PREVIOUS PAGE —</p> <p style="text-align: center;">END OF BOREHOLE</p> <p>NOTE:</p> <p>1. A 50 mm diameter monitoring well was installed at a depth of 13.7 m (Elev. 167.2 m).</p> <p>Water level measurements</p> <table style="width:100%; border-collapse: collapse;"> <tr> <td style="text-align: left;">Date</td> <td style="text-align: left;">Depth</td> <td style="text-align: left;">Elev.</td> </tr> <tr> <td>On Completion</td> <td>7.8 m</td> <td>173.1 m</td> </tr> <tr> <td>April 24/09</td> <td>0.0 m</td> <td>180.9 m</td> </tr> <tr> <td>May 13/09</td> <td>0.0 m</td> <td>180.9 m</td> </tr> <tr> <td>June 15/09</td> <td>0.0 m</td> <td>180.9 m</td> </tr> <tr> <td>July 09/ 09</td> <td>0.0 m</td> <td>180.9 m</td> </tr> </table>	Date	Depth	Elev.	On Completion	7.8 m	173.1 m	April 24/09	0.0 m	180.9 m	May 13/09	0.0 m	180.9 m	June 15/09	0.0 m	180.9 m	July 09/ 09	0.0 m	180.9 m											
Date	Depth	Elev.																												
On Completion	7.8 m	173.1 m																												
April 24/09	0.0 m	180.9 m																												
May 13/09	0.0 m	180.9 m																												
June 15/09	0.0 m	180.9 m																												
July 09/ 09	0.0 m	180.9 m																												



**PRELIMINARY FOUNDATION REPORT
RAINBOW CREEK BRIDGE ON LANGSTAFF ROAD**

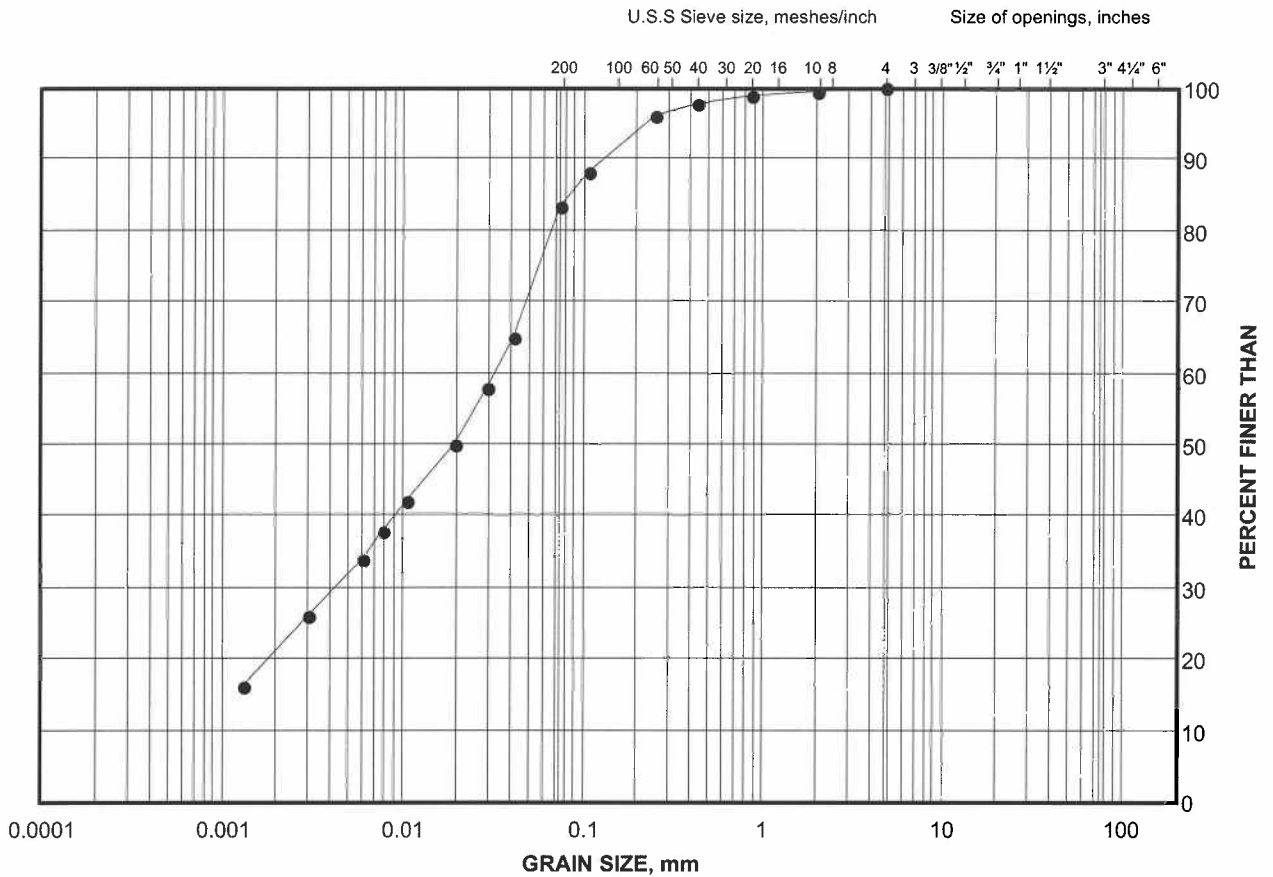
APPENDIX B

Laboratory Test Results

GRAIN SIZE DISTRIBUTION TEST RESULT

Surficial Clayey Silt

FIGURE B1



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

LEGEND

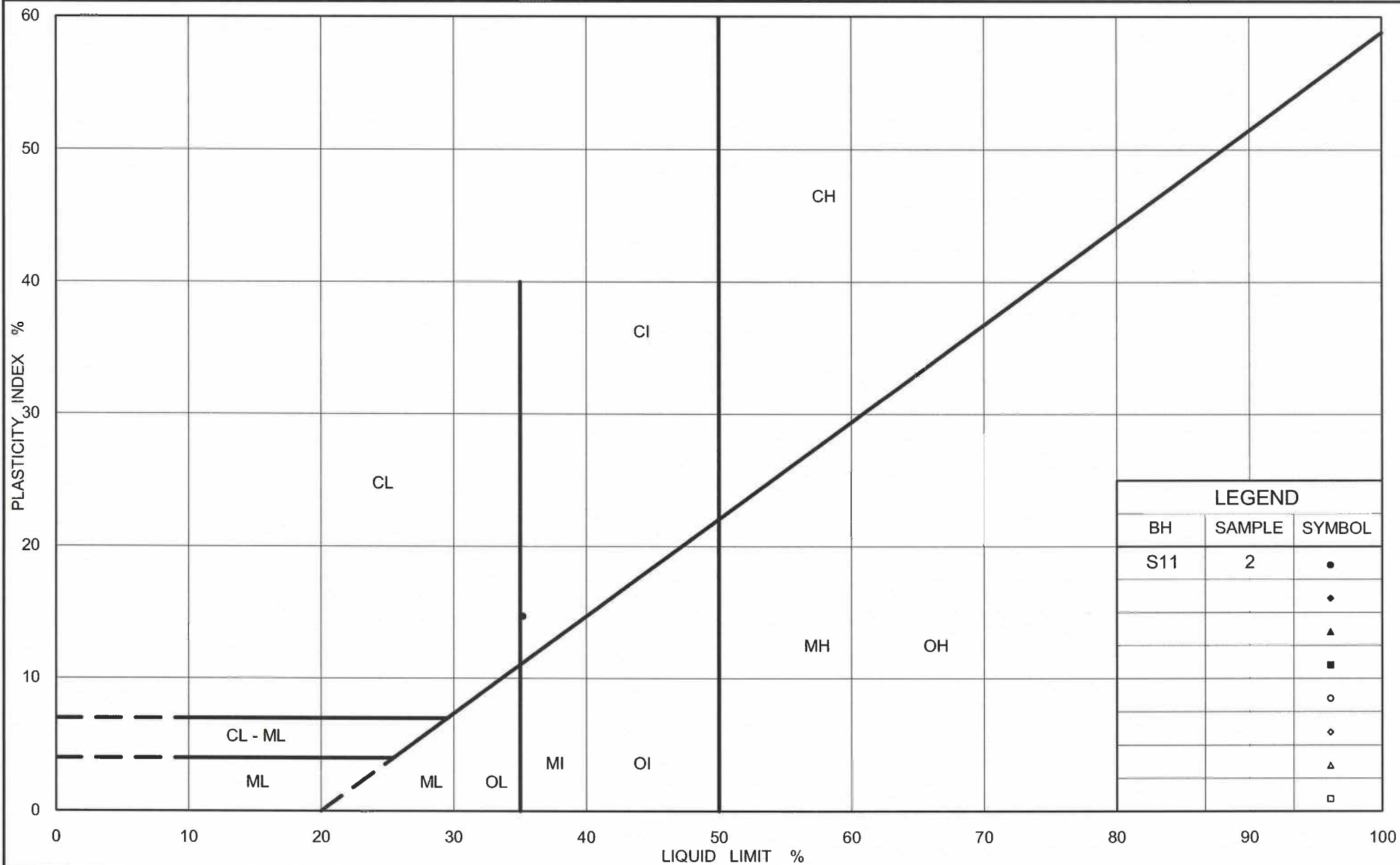
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
•	S11	2	179.1

Project Number: 06-1111-012-9

Checked By: SM

Golder Associates

Date: 04-Aug-09



Ministry of Transportation

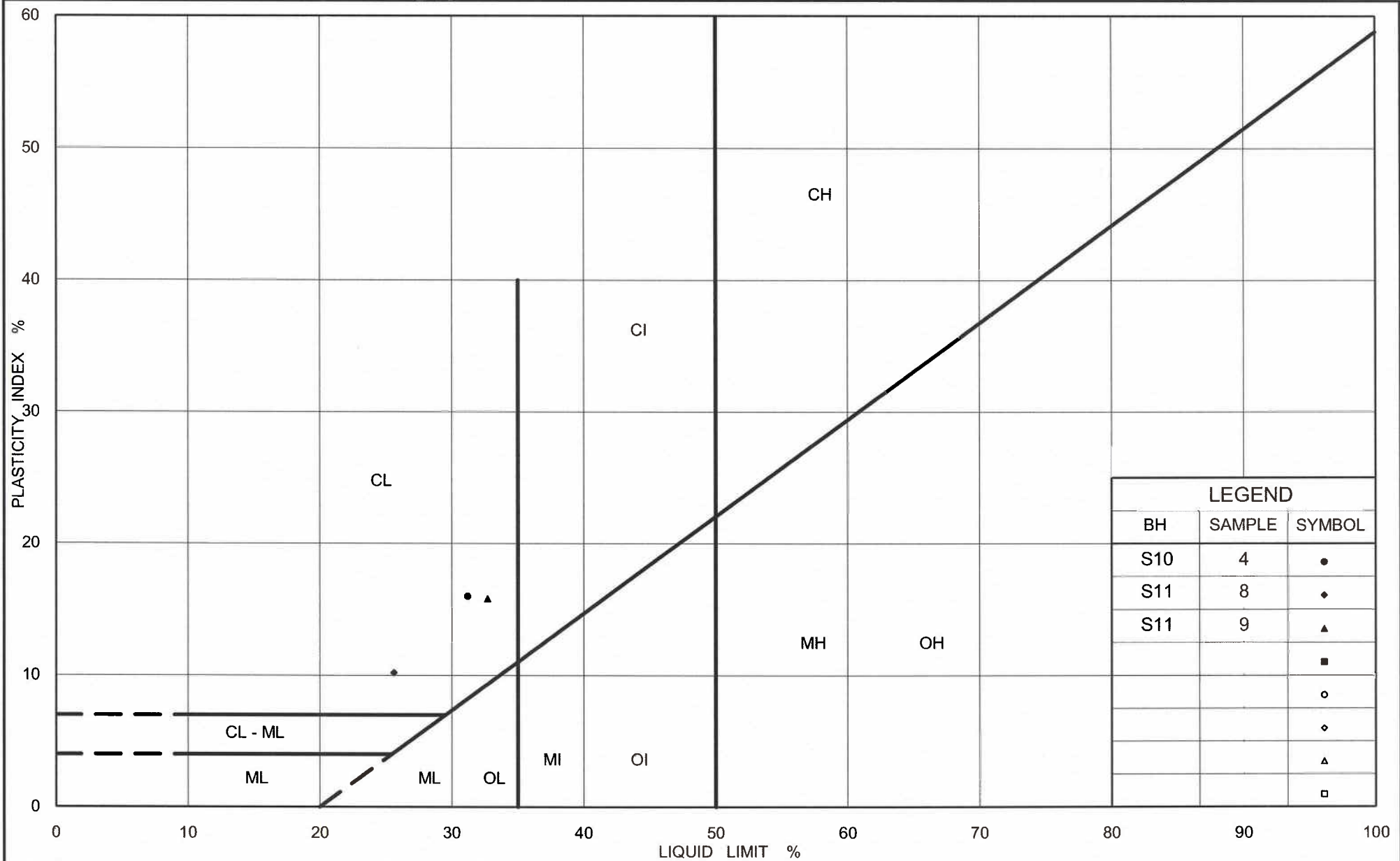
Ontario

PLASTICITY CHART Surficial Clayey Silt

Figure No. B2

Project No. 06-1111-012-9

Checked By: *SM*



Ministry of Transportation

Ontario

PLASTICITY CHART Upper Clayey Silt Till

Figure No. B3

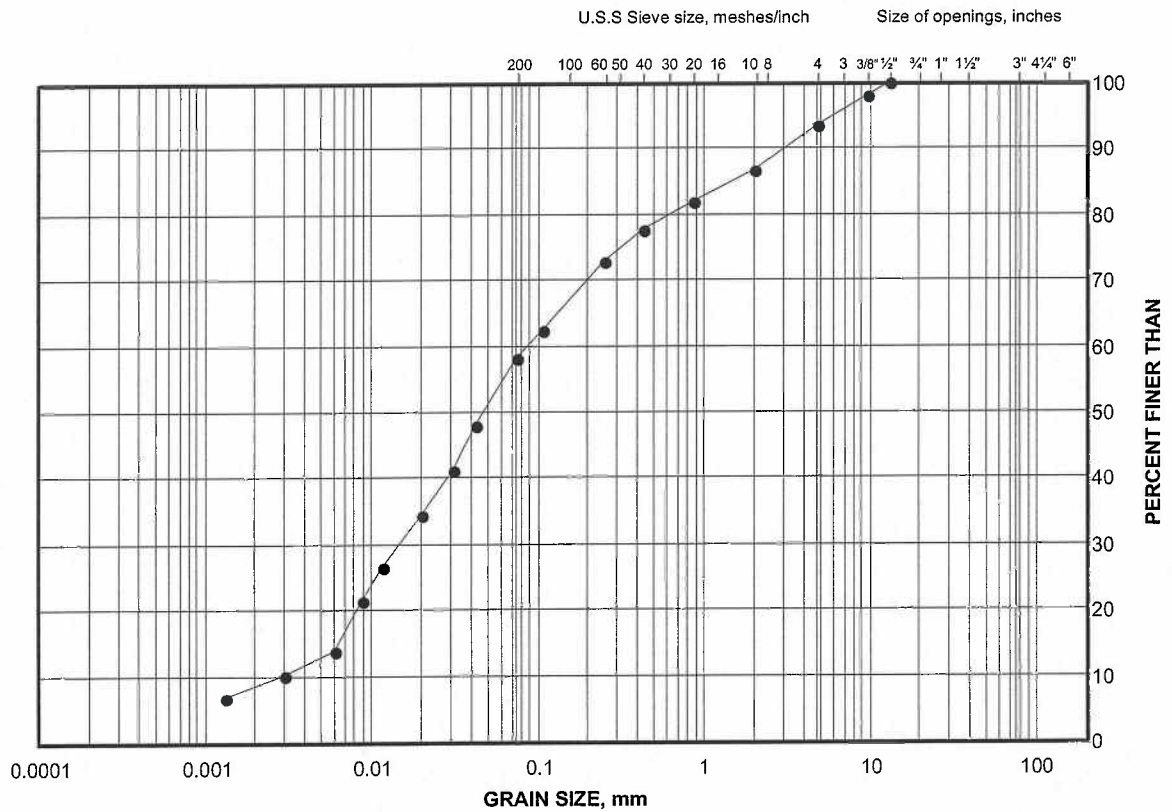
Project No. 06-1111-012-9

Checked By: *SMU*

GRAIN SIZE DISTRIBUTION TEST RESULT

Sand and Silt Till

FIGURE B4



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
•	S11	5	176.9

Project Number: 06-1111-012-9

Checked By: *SM*

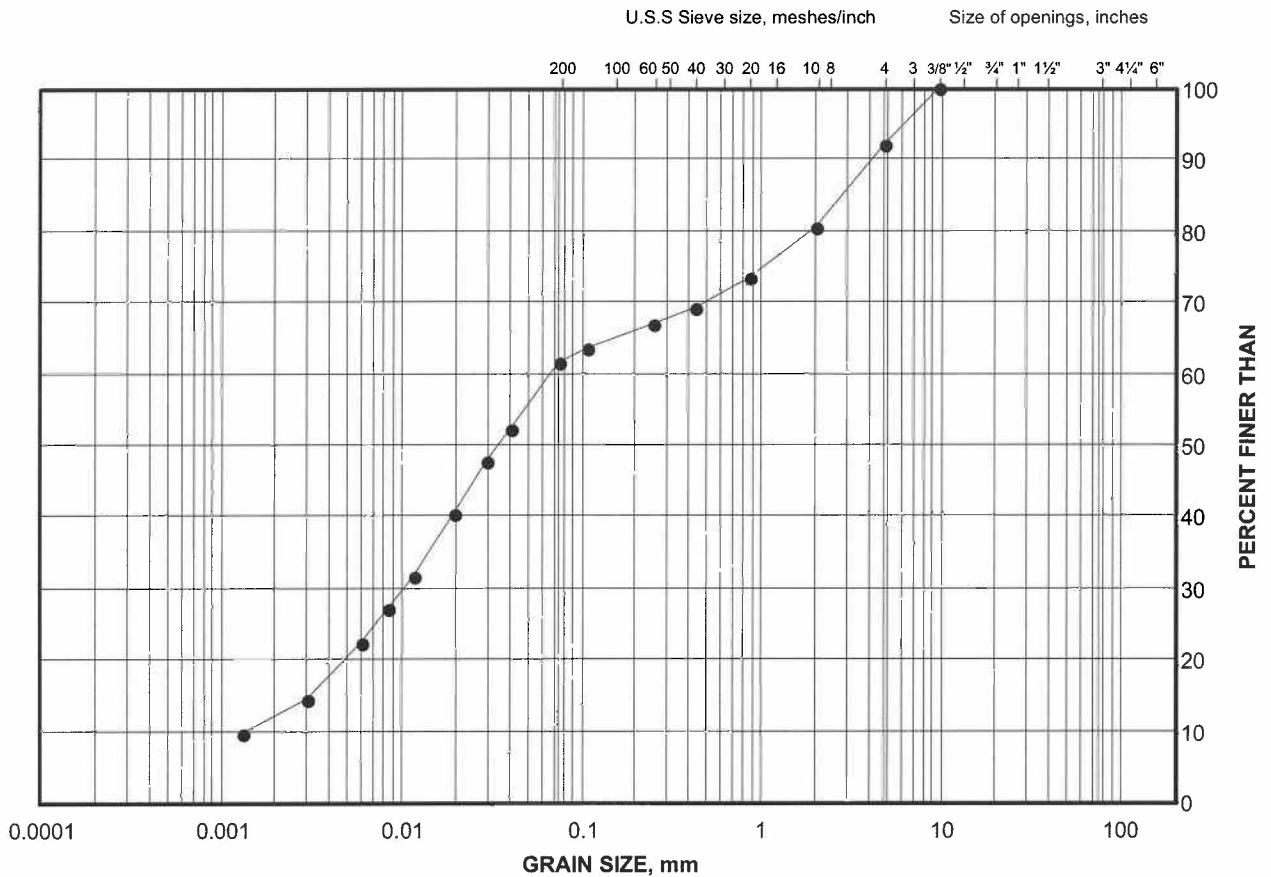
Golder Associates

Date: 04-Jul-09

GRAIN SIZE DISTRIBUTION TEST RESULT

Lower Clayey Silt Till

FIGURE B5



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
•	S11	10	169.9

Project Number: 06-1111-012-9

Checked By: SM

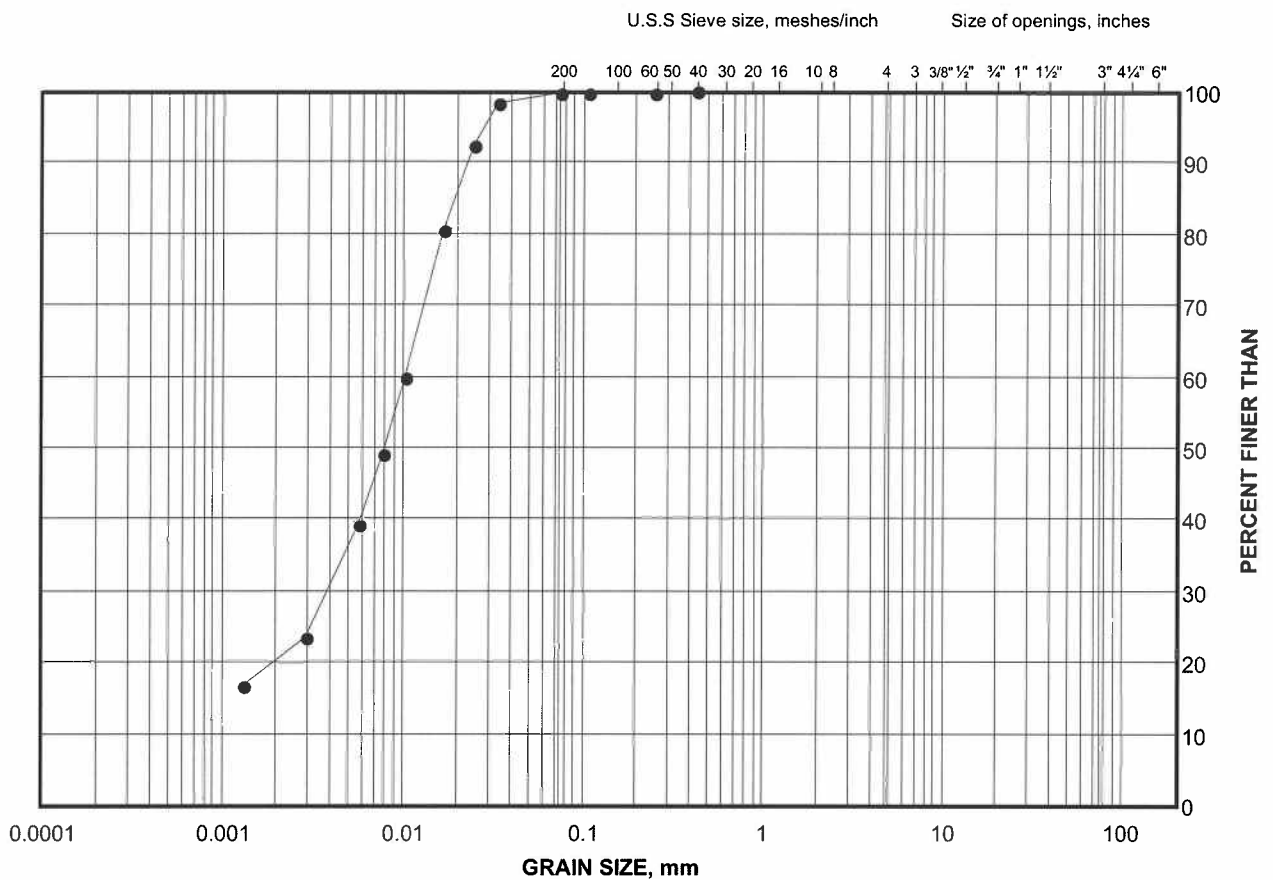
Golder Associates

Date: 04-Aug-09

GRAIN SIZE DISTRIBUTION TEST RESULT

Lower Clayey Silt

FIGURE B6



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

LEGEND

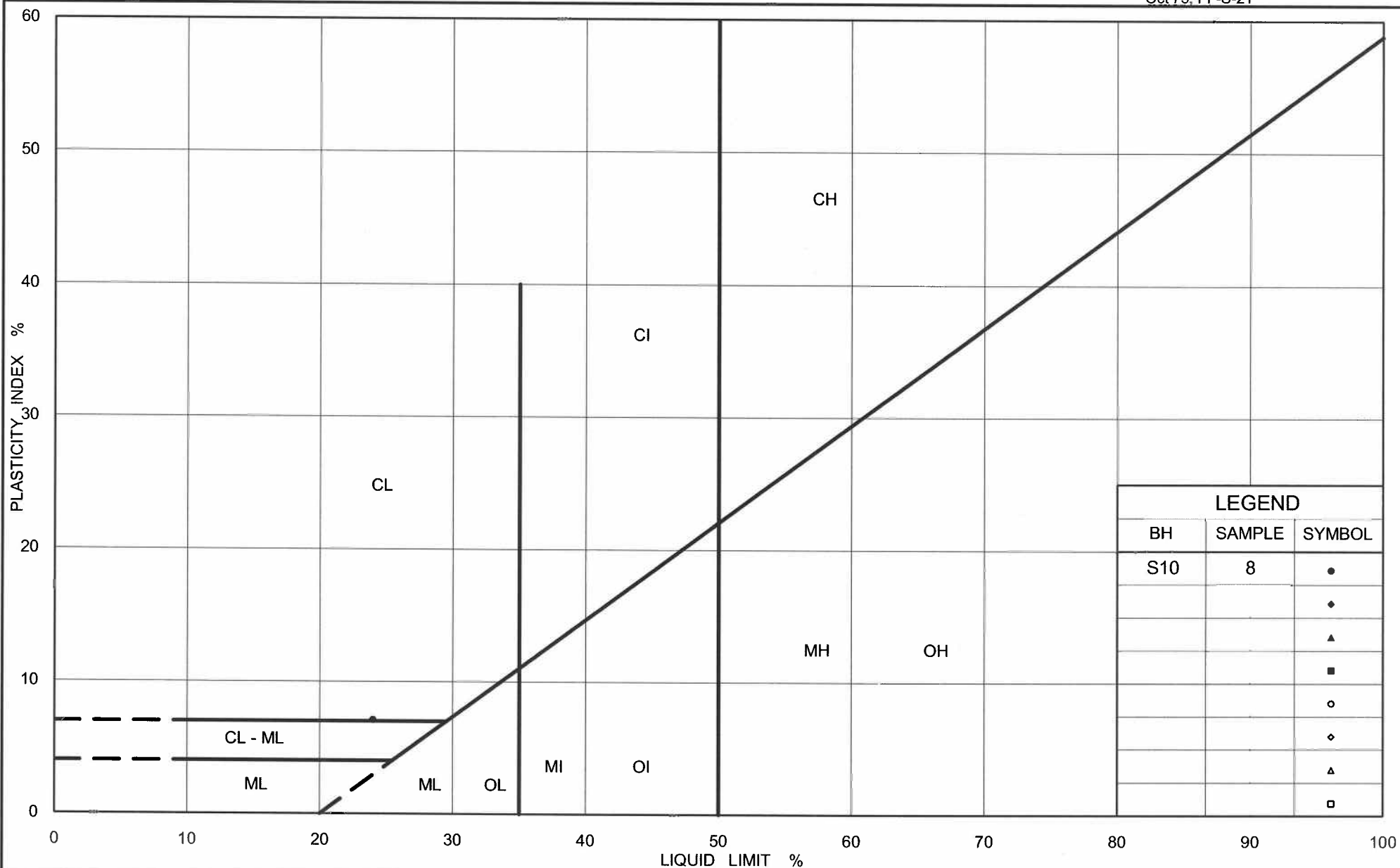
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
•	S10	8	175.6

Project Number: 06-1111-012-9

Checked By: *SM*

Golder Associates

Date: 04-Aug-09



Ministry of Transportation

Ontario

PLASTICITY CHART Lower Clayey Silt

Figure No. B7

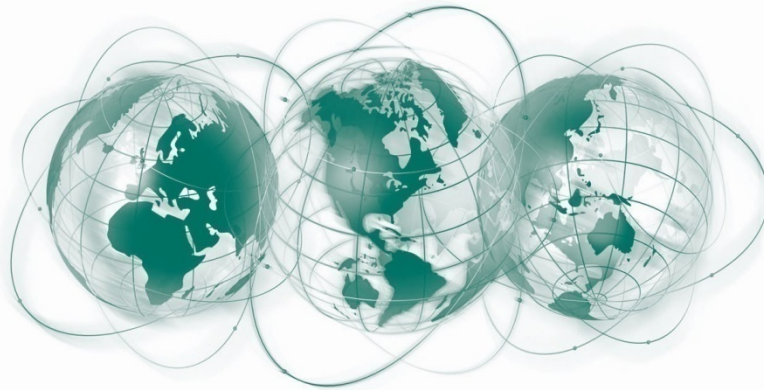
Project No. 06-1111-012-9

Checked By: *SM*

At Golder Associates we strive to be the most respected global group of companies specializing in ground engineering and environmental services. Employee owned since our formation in 1960, we have created a unique culture with pride in ownership, resulting in long-term organizational stability. Golder professionals take the time to build an understanding of client needs and of the specific environments in which they operate. We continue to expand our technical capabilities and have experienced steady growth with employees now operating from offices located throughout Africa, Asia, Australasia, Europe, North America and South America.

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