AUGUST 2009

PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT

CPR/McGILLIVRAY ROAD OVERPASSES (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
L5K 2P8

REPORT

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PART A

PRELIMINARY FOUNDATION INVESTIGATION REPORT
CPR / McGILLIVRAY ROAD OVERPASSES
HIGHWAY 427 EXTENSION
FROM HIGHWAY 7 TO MAJOR MACKENZIE DRIVE
W.O. 05-20012

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) overpasses at McGillivary Road and the Canadian Pacific Railway (CPR) tracks, and the immediate approach embankments to these overpass structures. 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 CPR / McGillivray Road overpass structures are located approximately 300 m south of Major Mackenzie Drive and about 250 m east of Huntington Road in the City of Vaughan, Ontario (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.

McGillivray Road is a two-lane road with ditches on the north and south side of the road. The double CPR tracks are located about 10 m north of the ditch line north of McGillivray Road. South of McGillivray Road and north of the tracks, the land is currently used for agricultural purposes. The CPR tracks are bordered to the north and south by a fence. The ground surface across the site is generally flat (with the exception of the ditches), varying from about Elevation 201 m to 202 m.

3.0 INVESTIGATION PROCEDURES

The borehole investigation for the CPR / McGillivray Road overpasses was carried out in March and April 2009, during which time a total of six boreholes were advanced. The boreholes, designated as Boreholes S25 to S30, were advanced at the locations shown on Drawing 1.

The field investigation for the boreholes was carried out using a truck-mounted D-90 drill rig and a track-mounted D-120 drill rig, both supplied 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), or using a 76 mm O.D. thin walled 'Shelby' tube (ASTM D1587-00) for relatively undisturbed samples in cohesive soils. Field vane shear tests were carried out firm to stiff cohesive soils in some of the boreholes. An "N" and a "B" size vane were used and the appropriate conversion factors were applied to the field measurements to take into account the vane size for determination of undrained shear strengths (ASTM D2573 01).

The boreholes were terminated after penetrating at least 3 m into hard or very dense soil having SPT 'N' values of greater than 100 blows per 0.3 m of penetration. Boreholes S26, S28, S29 and S30 were drilled to depths of between 15.6 m and 19.7 m. Borehole S25, was drilled to a depth of 22.0 m, and Borehole S27, was drilled to a depth of 34.1 m.

The groundwater conditions in the open boreholes were observed during the drilling operations, and a standpipe piezometer was installed in Borehole S28 to permit monitoring of the groundwater level at the site. The piezometer 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 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 indicated on the Record of Borehole Sheet 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 and a Global Positioning System unit (GPS). 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

[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.

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 CPR/McGillivray Road overpass structures are shown on Drawings 2 and 3. 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 near-surface conditions north and south of McGillivray Road consist of a surficial layer of topsoil underlain by up to about 1.5 m of surficial silty clay. In the boreholes drilled through McGillivray Road, the near-surface conditions consist of up to about 0.5 m of sand and gravel fill. Both the surficial silty clay and the fill are underlain by a silty clay to clayey silt till deposit that grades with depth to a sand and silt till and then back to a clayey silt till. Based on one borehole drilled to a depth of 34 m, the till deposit is underlain by a cohesionless deposit that grades from a sand and silt to sandy silt to silt.

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

4.2.1 Topsoil

Approximately 0.3 m of topsoil was encountered immediately below ground surface in Boreholes S25, S26, S29 and S30, which were drilled outside of McGillivray Road.

4.2.2 Asphalt and Sand and Gravel Fill

Approximately 0.1 m of asphalt was encountered immediately below the ground surface in Boreholes S27 and S28 that were drilled through the north edge of McGillivray Road at this site.

In Boreholes S27 and S28 the asphalt is underlain by a layer of sand and gravel fill that extends to a depth of 0.5 and 0.3 m, respectively. The base of the fill layer was encountered at Elevation 200.6 m and 200.5 m in Boreholes S27 and 28, respectively.

4.2.3 Surficial Silty Clay

The topsoil in Boreholes S25, S26, S29 and S30 is underlain by a surficial silty clay deposit that extends to depths of about 0.8 m to 1.5 m below ground surface. The surficial silty clay contains trace to some sand and gravel, rootlets and organics. On the borehole records in Appendix A, the upper 0.8 m of the surficial silty clay is also described as reworked as it appears that this material has been disturbed by previous agricultural activities.

The Standard Penetration Test (SPT) 'N' values in the surficial silty clay ranged from 8 to 15 blows per 0.3 m of penetration, indicating that the surficial soil has a stiff consistency. Measured water contents on selected samples of the surficial silty clay range from 28 to 32 percent.

4.2.4 Till Deposits

In all boreholes drilled at this site, the fill and surficial cohesive soil are underlain by an upper silty clay to clayey silt till deposit that grades with depth to a cohesionless till; in Boreholes S28 and S29, the cohesionless till deposit grades back to a cohesive till.

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
S25	10.7 to 11.3 m	191.1 to 190.5 m
	12.2 to 12.8 m	189.6 to 189.0 m
	13.1 to 14.3 m	188.7 to 187.5 m
	16.5 to16.8 m	185.3 to 185.0 m
	17.4 to 17.8 m	184.4 to 184.0 m
S29	3.8 to 4.4 m	198.2 to 197.6 m
S30	13.4 m	188.9 m
	14.6 m	187.7 m

4.2.4.1 Silty Clay to Clayey Silt Till (Upper Cohesive Till)

In all of the boreholes at this site the upper cohesive till deposit extends to depths of between 11.9 m and 20.0 m; the base of the cohesive till was encountered in the boreholes between approximately Elevations 181.8 m and 189.6 m.

The upper cohesive till consists of silty clay to clayey silt containing trace to some sand and gravel. Within the clayey silt till deposit in Borehole S30 a 0.8 m thick layer of silty sand was encountered at a depth of 10.4 m and extended to Elevation 191.1 m. Grain size distribution tests were completed on eight selected samples of the upper clayey silt till deposit and the results are presented on Figures B1 and B2 in Appendix B. Atterberg limits testing was carried out on eleven samples of the upper clayey silt portion of the till deposit, and the plastic limits varied from 12 to 16 percent, the liquid limits varied from 21 to 33 percent, and the plasticity indices varied from 9 to 17 percent. These results, which are plotted on a plasticity chart on Figure B3 in Appendix B, confirm that this portion of the till deposit is a clayey silt of low plasticity. Atterberg limits testing was carried out on six samples of the silty clay portion of the till deposit, and measured plastic limits of 18 to 21 percent, liquid limits of 37 to 48 percent, and plasticity indices of 19 to 28 percent. These results, which are plotted on a plasticity chart on Figure B4 in Appendix B, confirm that this portion of the till deposit is a silty clay of medium plasticity. Measured water contents on samples of the cohesive till deposit ranged from about 13 to 24 percent.

Generally the SPT ‘N’ values indicate that there is an upper stiffer crust underlain by a stiff zone which is in turn underlain by very stiff to hard till. The measured SPT ‘N’ values within 3 m to 4 m depth below ground surface vary from 11 to 29 blows per 0.3 m of penetration, indicating a stiff to very stiff consistency. Between approximately Elevation 197 m and 192.5 m, the SPT ‘N’ values vary from 7 to 15 blows per 0.3 m of penetration, and in situ field vane tests measured undrained shear strengths typically greater than 100 kPa with the exception of three measured values ranging from about 65 kPa to 97 kPa at depths of between about 7.0 m and 8.5 m (Elevations 193 m to 195 m) in Boreholes S28 to S30. The field vane test results together with the SPT ‘N’ values indicate that this approximately 4.5 m thick middle zone of the cohesive till has a generally stiff consistency. Below Elevation 192.5 m the SPT ‘N’ values increase and vary from 18 to greater than 100 blows per 0.3 m of penetration, indicating a very stiff to hard consistency.

4.2.4.2 Sand and Silt Till (Cohesionless Till)

The upper silty clay to clayey silt till grades with depth to a cohesionless till in Boreholes S26, S28, S29 and S30; the surface of the sand and silt till was encountered between Elevations 186.3 m to 189.6 m. Boreholes S28 and S29 fully penetrated the cohesionless portion of the till deposit, which was found to have a thickness of approximately 1.5 m and 4.6 m, respectively. The base of the cohesionless portion of the till was encountered in Boreholes S28 and S29 at Elevations 184.8 m and 183.7 m, although this deposit may be higher or lower than this in the other boreholes where it was not fully penetrated.

The cohesionless portion of the till consists of sand and silt containing trace gravel and trace clay. The results of grain size distribution tests completed on two samples of the sand and silt till are provided on Figure B5 in Appendix B. Atterberg limit testing was carried out on two samples of the sand and silt till, and measured plastic limits of 13 and 17 percent, liquid limits of 16 and 21 percent and plasticity indices of 3 and 4 percent. These results, which are plotted on a plasticity chart on Figure B6 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 two samples of the lower cohesive till were 11 and 21 percent.

During drilling within the sand and silt till deposit in Borehole S29, "blowing" sands was encountered in which the sand penetrated up inside the hollow stem augers. In Borehole S29 the sand came 1.0 m up inside the hollow stem augers when the borehole was at a depth of 15.2 m below ground surface (Elevation 186.8 m). "Blowing" sand was not encountered in any of the other boreholes drilled at this site.

The SPT 'N' values measured within the cohesionless till typically ranged from 80 to greater than 100 blows per 0.3 m of penetration, indicating that the cohesionless till has a very dense relative density.

4.2.4.3 Clayey Silt Till (Lower Cohesive Till)

A lower cohesive till deposit was encountered underlying the cohesionless portion of the till in Boreholes S28 and S29. The surface of the lower cohesive till was encountered at a depth of about 16.0 m and 18.4 m (Elevation 184.8 m and 183.7 m, respectively) in these boreholes, and it was penetrated for a depth of 1.2 m and 0.3 m. Neither borehole fully penetrated this deposit.

The lower cohesive till deposit consists of clayey silt containing trace to some sand and trace gravel. The result of a grain size distribution test completed on one selected sample of the lower clayey silt till is presented on Figure B2 in Appendix B. Atterberg limit testing was carried out on one sample of the lower clayey silt till deposit and measured a plastic limit of 14 percent, a liquid limit of 23 percent and a plasticity index of 9 percent. This result, which is plotted on a plasticity chart on Figure B3 in Appendix B, confirms that the lower cohesive till is a clayey silt of low plasticity. Measured water contents on two samples of the lower cohesive till were 11 and 13 percent.

The SPT 'N' values measured within the lower cohesive till typically greater than 100 blows per 0.3 m of penetration, indicative of a hard consistency.

4.2.5 Sand and Silt to Sandy Silt

In Boreholes S25 and S27 the cohesive till is underlain by a cohesionless deposit. In Borehole S25 the cohesionless deposit consists of sand and silt, and in Borehole 27 the cohesionless deposit grades with depth from sand and silt to sandy silt. Borehole S25 terminated within the sand and silt deposit at a depth of 22.0 m (Elevation 179.9 m); however Borehole S27 fully penetrated the sand and silt to sandy silt deposit which was found to have a thickness of about 11.0 m. The base of the sand and silt to sandy silt deposit was encountered in Borehole S27 at Elevation 172.1 m, although the deposit base may be lower or higher than this in Borehole S25 where it was not fully penetrated.

The sand and silt to sandy silt portions of the cohesionless deposit contain trace clay. The result of a grain size distribution test completed on a sample of the sandy silt is provided on Figure B7 in Appendix B.

The SPT 'N' values measured within the cohesionless deposit typically ranged from 27 blows to 55 blows per 0.3 m of penetration, indicating that the cohesionless deposit has a compact to very dense relative density.

4.2.6 Silt

Beneath the sandy silt deposit in Borehole S27, a silt deposit was encountered at a depth of 29.0 m (Elevation 172.1 m); Borehole S27 terminated within the silt deposit at a depth of 34.1 m (Elevation 167.0 m).

The silt deposit contains trace to some clay and trace sand. The result of a grain size distribution test completed on a sample of the silt is provided on Figure B8 in Appendix B. A measured water content on a sample of the silt was about 30 percent.

The SPT 'N' values measured within the silt deposit were 13 and 28 blows per 0.3 m of penetration, indicating that the silt deposit has a compact relative density.

Below the last sample recovered within this deposit (end of the borehole) a dynamic cone penetration test (DCPT) was conducted to a depth of 38.4 m (Elevation 162.7 m). The DCPT blows per 0.3 m ranged from 15 to 258 and were generally greater than 100 blows below a depth of 36.5 m. It is noted that the deeper the DCPT is advanced the more unreliable the data becomes since the soil is collapsing around the rods and adding friction, thereby resulting in elevated blows. The data is more reliable if there is a sudden increase in the number of blows required to advance the rods 0.3 m; as seen on the Record of Borehole sheet S27 there was a steady increase. The DCPT merely provides an indication as to where more competent material is encountered.

4.3 Groundwater Conditions

The water level in the boreholes as noted during and upon completion of drilling operations was typically between about Elevation 185.2 m and Elevation 191.6 m (typically at depths varying from 10.7 m to 16.6 m) in all the boreholes drilled at this site, with the exception of Borehole S27; this borehole, which was open to 25 m depth, was dry upon completion of drilling.

A standpipe piezometer was installed in Borehole S28 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 approximately nine weeks following borehole completion are summarised below.

Borehole No.	Ground Surface Elevation	Depth to Groundwater Level	Groundwater Elevation	Date of Measurement
S28	200.8 m	8.5 m 8.5 m 8.6 m 9.1 m 9.1 m	192.3 m 192.3 m 192.2 m 191.7 m 191.7 m	April 27, 2009 May 13, 2009 May 25, 2009 June 15, 2009 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. Chris Radway, Suresh Bainey and Jordan Black. 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.

LICENSED PROFESSIONAL ENGINEER
S.M. McGAGHRAN
PROVINCE OF ONTARIO

Sandra McGaghran, P. Eng.
Geotechnical Engineer

Lisa Coyne, P. Eng.
Associate, Geotechnical Engineer

REGISTERED PROFESSIONAL ENGINEER
F. J. HEFFERNAN
PROVINCE OF ONTARIO

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

SMM/LCC/FJH/jl

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PART B

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CPR / McGILLIVRAY ROAD OVERPASSES
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6.0 ENGINEERING RECOMMENDATIONS FOR PRELIMINARY DESIGN

This section of the report provides foundation design recommendations for the preliminary design of the proposed Canadian Pacific Railway (CPR) / McGillivray Road overpass structures 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 overpass is finalized, to confirm and expand on the preliminary foundation recommendations provided in this report.

6.1 General

The CPR / McGillivray Road overpasses are proposed to consist of three-span structures. Based on the preliminary General Arrangement (GA) Drawing provided by MRC on May 15, 2009, the south abutments will be located south of McGillivray Road, the south piers will be located between McGillivray Road and CPR tracks, and the north piers and the north abutments will be located in a field north of the CPR tracks. Between the piers and the abutments the proposed span lengths are approximately 29 m, and between the north piers and south piers the proposed span lengths are about 34 m.

According to the preliminary GA Drawing, the finished grade of Highway 427 NBL and SBL over CPR / McGillivray Road varies from approximately Elevation 210.6 m to 213.0 m, rising northward. The natural ground surface across the site varies from about Elevation 201 m to 202 m. The south approach embankments are proposed to be about 8.5 m high relative to the existing ground surface, and the north approach embankments are proposed to be about 11.3 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 have been considered for the proposed CPR / McGillivray Road overpass structures:

- **Spread footings founded on the very stiff silty clay to clayey silt till**: This option is feasible at the north and south piers, where footings would have to extend below any “reworked” or stiff surficial silty clay to be founded on the very stiff clayey silt to silty clay till; very stiff till was encountered at depths of between 0.5 m and 1.4 m in the boreholes in the vicinity of the proposed piers. Considering that the grade at the north and south abutments is to be raised by about 11.3 m and 8.5 m, respectively, this option is considered neither economical nor feasible at the abutments, given both the resulting height of abutment walls and the predicted settlement of the foundation soils under the approach embankment loading.

- **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 would be necessary to subexcavate the upper 0.8 m to 1.5 m of reworked and stiff surficial clayey silt to expose the very stiff to hard clayey silt till at the north and south abutments, prior to construction of the new approach embankments. For this option, the loading from the new approach embankments would still result in some settlement in the stiff portion of the silty clay to clayey silt till deposit, which could result in differential settlement between the abutments and piers.

- **Steel H-piles driven to found within the till deposit:** This option could be adopted to support the abutments and piers in either a conventional or an integral abutment-type structure. 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:** This option could be adopted to support the abutments and 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 can be founded on very stiff silty clay to clayey silt till at a depth of 1.4 m, with no additional subexcavation required beyond that needed for frost protection. Therefore spread footings on very stiff silty clay to clayey silt till are 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 deep foundations perspective over caissons for support of the abutments and piers, as the caissons would terminate in the water-bearing sand and silt till at most of the foundation units, which would be susceptible to disturbance and which would require special construction procedures.

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 at the piers founded on very stiff silty clay to clayey silt till.

6.2.2.1 Founding Elevations

The piers may be supported on spread footing placed below the stiff surficial silty clay to clayey silt on very stiff silty clay to clayey silt till. 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; these depths are given relative to lowest surrounding grade. Maximum (highest) founding elevations are also given in the following table (in the event that the grade surrounding the piers is to be raised), to ensure that the footings are supported on the very stiff till deposit.

Foundation Element	Founding Stratum	Highway 427 NBL	Highway 427 SBL
South Pier	Very Stiff Silty Clay to Clayey Silt Till	1.4 m depth (Elevation 200.0 m)	1.4 m depth (Elevation 200.0 m)
North Pier	Very Stiff Silty Clay to Clayey Silt Till	1.4 m depth (Elevation 200.0 m)	1.4 m depth (Elevation 200.0 m)

6.2.2.2 Geotechnical Resistances

A factored geotechnical resistance at Ultimate Limit States (ULS) of 300 kPa and a geotechnical resistance at Serviceability Limit States (SLS) of 200 kPa (for 25 mm of settlement) may be used for preliminary design purposes, assuming 3 m wide footings. This assessment was based on the information obtained at Boreholes S27 and S28, drilled at the south piers (NBL and SBL), where the upper very stiff zone is only about 3 m thick and is underlain by a less competent stiff soil. In Boreholes S29 and S30 drilled at the north abutments, “stiff” soil was encountered at depth as well; however the upper “crust” was thicker. Based on the subsoil conditions encountered in Boreholes S29 and S30, higher geotechnical resistances may be considered at detail design subject to the results of additional drilling and in-situ field vane testing at each of the piers. Based on the other boreholes drilled at this site, as an upper limit it is suggested that a factored geotechnical resistance at Ultimate Limit States (ULS) of 350 kPa and a geotechnical resistance at Serviceability Limit States (SLS) of 250 kPa (for 25 mm of settlement) may be feasible 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 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 native silty clay to clayey silt till should be calculated in accordance with Section 6.7.5 of the *CHBDC*. A coefficient of friction, tan φ', of 0.55 can be used for cast-in-place concrete footings on the properly prepared silty clay to 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 (OPSD 3090.101).

For this option, subexcavation will be required of the reworked/stiff surficial silty clay 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 up to 1.4 m of soil below ground surface would be required as both the north and south abutments are located within an agricultural field. 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

At the proposed north and south abutment area it is estimated that approximately 80 mm to 110 mm of settlement will occur under the loading from the proposed approach embankments, primarily in the stiff zone of the clayey silt till between about 3 m and 9 m depth. If "perched" spread footings are adopted for support of the north and south abutments, it will be necessary to preload the approach embankment area before construction of the footings and overpass structure, to mitigate settlement at the abutments and to minimize differential settlement between the abutments and centre pier.

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, provided that preloading of the approach embankment area or other settlement mitigation measures have been completed. 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 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 φ', 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 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.

6.2.4.1 Founding Elevations

Steel H-piles driven to found within the very dense sand and silt till or hard clayey silt till deposit may be used for support of the abutments and piers. "Refusal" (i.e. soil having SPT 'N' values of greater than 100 blows per 0.3 m of penetration) was encountered in the boreholes between approximately Elevation 187.3 m to 188.7 m, with the exception of Borehole S28 (located near the south pier of the NBL bridge) where it was encountered at Elevation 185.7 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.

Foundation Unit	Borehole No.	Founding Stratum	Estimated Pile Tip Elevation
South Abutment SBL Bridge	S25	Hard Clayey Silt Till	188.5 m
South Abutment NBL Bridge	S26	Very Dense Sand and Silt Till	188.0 m
South and North Pier SBL Bridge	S27	Hard Clayey Silt Till	187.5 m
South and North Pier NBL Bridge	S28	Very Dense Sand and Silt Till	185.7 m
North Abutment SBL Bridge	S29	Very Dense Sand and Silt Till	186.8 m
North Abutment NBL Bridge	S30	Very Dense Sand and Silt Till	188.7 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 or hard clayey sit 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.

<table>
<tr><th>Foundation Unit</th><th>Founding Stratum</th><th>Factored Geotechnical Resistance at ULS</th><th>Geotechnical Resistance at SLS</th></tr>
<tr><td>North and South Abutments</td><td rowspan="2">Very Dense Sand and Silt Till / Hard Clayey Silt Till</td><td>1,600 kN</td><td>1,400 kN</td></tr>
<tr><td>North and South Piers</td><td>1,300 kN</td><td>1,100 kN</td></tr>
</table>

At the proposed north and south abutment areas it is estimated that up to about 80 and 110 mm of settlement will occur, primarily in the stiff clayey silt till between 3 m and 9 m depth under the proposed loading from the approach embankment. For preliminary design purposes it is recommended that a downdrag load of 250 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 very stiff 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 sand and silt till or hard clayey silt till for support of the foundation elements for the overpass structures. 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 or hard clayey silt till at the elevations given in Section 6.2.4.1.

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 / Hard Clayey Silt Till	0.9 m	3,900 kN	3,200 kN
		1.2 m	6,900 kN	5,700 kN
		1.5 m	10,800 kN	9,000 kN
North and South Piers	Very Dense Sand and Silt Till / Hard Clayey Silt Till	0.9 m	2,500 kN	2,100 kN
		1.2 m	4,400 kN	3,700 kN
		1.5 m	6,800 kN	5,700 kN

At the proposed north and south abutment areas, it is estimated that about 80 mm to 110 mm of settlement will occur, primarily in the stiff clayey silt till between 3 m and 9 m depth, under the proposed loading from the approach embankments. For preliminary design purposes it is recommended that the following downdrag loads be included in the design for caissons supporting the abutments:

Caisson Diameter	Downdrag Load
0.9 m	500 kN
1.2 m	750 kN
1.5 m	1,000 kN

Further investigation and assessment will be required during the 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*.

6.2.5.3 Resistances to Lateral Loads

For preliminary design purposes, a maximum factored lateral resistance at ULS of 400 kN and a maximum lateral resistance 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* and correlation with lateral pile load tests. 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 :

	Earth Fill
Soil unit weight:	20 kN/m^3
Coefficients of static lateral earth pressure: Active, K_a At rest, K_o	 0.33 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 At rest, K_o	 0.27 0.43	 0.27 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:

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

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)

- 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 amplication 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 (δ=Φ'/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 CPR / McGillivray Road overpass structures will require placement of up to about 8.5 m of fill within the limits of the south approach and up to about 11.3 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 very stiff silty clay to clayey silt till at the north and south approach.

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 (as encountered beneath the road surface on McGillivray Road) and any softened or loosened soil should be stripped from below the approach embankment areas. If spread footings "perched" within the approach embankments are adopted for support of the abutments, then it is recommended that subexcavation of the reworked/stiff surficial clayey silt be conducted within the loading footprint for the compacted Granular A pad, as discussed in Section 6.2.3.1.

Embankment fill should be placed and compacted in accordance with MTO's Special Provision SP 206S03 and SP 105S10. In accordance with MTO's standard practice, a minimum 2 m wide bench should be provided where embankment slopes are greater than 8 m in height, such that the uninterrupted slope height does not exceed 8 m.

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 appropriate 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 Elevation 192.3 m in the analyses.

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
Stiff Silty Clay	20	50 kPa	--	30
Stiff to Very Stiff Silty Clay to Clayey Silt Till	21	100 kPa	--	30
Stiff Silty Clay to Clayey Silt Till	21	65 kPa[1]	--	28
Very Stiff to Hard Silty Clay to Clayey Silt Till	21	150 kPa	--	34
Very Dense Sand and Silt Till	21	--	--	34

1. Based on field vane testing (minimum values).

With appropriate subgrade preparation and proper placement and compaction of embankment fill materials, the proposed 8.5 m to 11.3 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, for both short-term (undrained) and long-term (effective stress) conditions. The results of an example static stability analysis are provided on Figures 2 and 3.

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 4.

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 8.5 m to 11.3 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 be about 80 mm to 110 mm, the majority of which will occur within the "stiff" zone of the clayey silt till deposit. It is estimated that it would take about three to six months to complete 90 percent of this predicted 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 stiff portion of the clayey silt till deposit, consolidation parameters have been estimated based on correlation with Atterberg limits and undrained vane shear strength data, and experience with similar soil types in the Peel Plain.

Soil Deposit	Bulk Unit Weight kN/m^3	Elastic Modulus	Consolidation Parameters
Embankment fill (range of parameters assumed for earth fill and granular fill)	20 – 22	--	--
Very Stiff Silty Clay to Clayey Silt Till	21	35 MPa	
Stiff Silty Clay to Clayey Silt Till	20	15 MPa	Cc = 0.25 Cr = 0.025
Very Stiff to Hard Silty Clay to Clayey Silt Till	21	100 MPa	--
Very Dense Sand and Silt Till	21	150 MPa	--

Settlement mitigation measures will be required to accommodate the predicted 80 mm to 110 mm of settlement of the founding soils, particularly if spread footings "perched" in the approach embankments are adopted for support of the abutments, but also to address post-construction settlement that could impact the new Highway 427 pavement. Provided that there is sufficient time in the construction schedule, the simplest and most economical mitigation measure would be preloading the approach embankment areas for a period of about six months. If there is insufficient time available, the 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 overpass 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 stiff portion of the clayey silt till between depths of 3 m and 9 m 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.

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 compact sand fill and stiff silty clay to clayey silt and into very stiff to hard silty clay to clayey silt till, which are 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 was measured in a standpipe piezometer at the site at about 8.5 m below ground surface. It is expected that excavations for the piers and north abutment foundations will be above the groundwater level.

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 piles to facilitate driving into the very dense sand and silt till and the hard clayey 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 to warn the contractor of the possible presence of cobbles and/or boulders within the overburden soils.

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.

LICENSED PROFESSIONAL ENGINEER
S.M. McGAGHRAN
PROVINCE OF ONTARIO

Sandra McGaghran, P. Eng.
Geotechnical Engineer

Lisa Coyne, P. Eng.
Associate, Geotechnical Engineer

REGISTERED PROFESSIONAL ENGINEER
F. J. HEFFERNAN
PROVINCE OF ONTARIO

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

SMM/LCC/FJH/jl

n:\active\2006\1111\06-1111-012 mrc hwy 427 extension\5 - reports\foundation\final reports\06-1111-012-8 final rpt 09aug highway 427 cpr-mcgillivray overpass.docx

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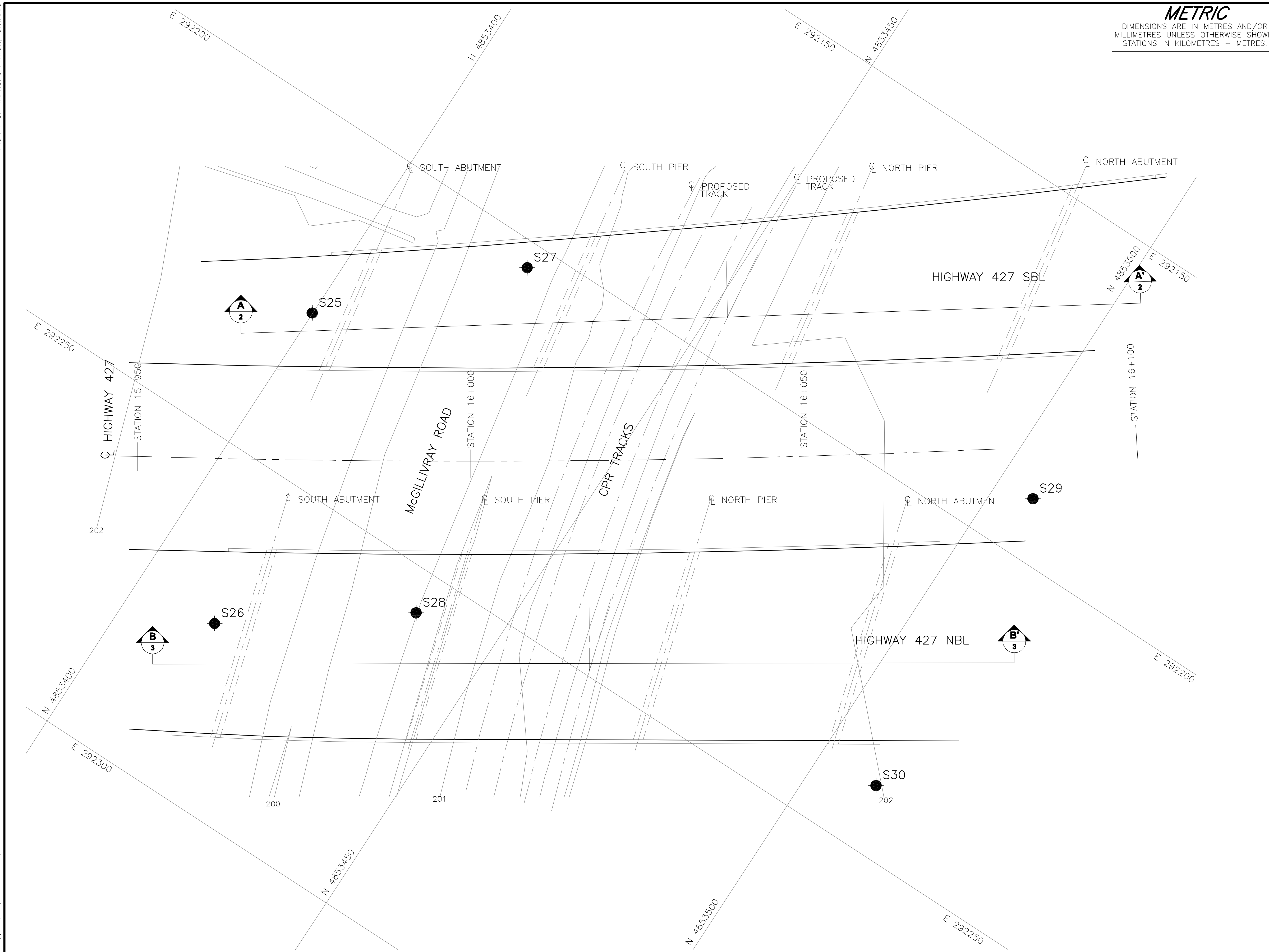
TABLE 1
COMPARISON OF FOUNDATION ALTERNATIVES
CPR / McGILLIVRAY ROAD OVERPASSES – HIGHWAY 427 (NBL AND SBL) EXTENSION
W.O. 05-20012

Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Spread Footings on very stiff silty clay to clayey silt till	Feasible for support of piers	• Relative ease of construction • Negligible post-construction settlement	• Lowest bearing capacities of the four options	• Lower relative cost than piled foundations •	• Disturbance of subgrade soil due to ponded water
Spread Footings "perched" in approach embankment fill	May be feasible for support of abutments if preloading completed	• Negligible post-construction settlement provided that preloading of approach embankment areas is completed • Construction maintained above groundwater level	• Embankment preloading must be taken into account in construction schedule	• Low cost option • Subexcavation of 1.4 m of surficial soils required within loading footprint for compacted Granular A pad	• Low to moderate risk that preloading period will extend beyond six months, impacting construction schedule for overpass • Low to moderate risk of some differential settlement between abutments and piers; and, • Must ensure proper compaction of Granular 'A' pad to minimize post-construction settlement
Steel H-pile foundations driven to found within very dense sand and silt till or hard clayey silt till	Feasible for support of abutments and piers	• Sub-excavation is not required • Higher bearing capacity compared to spread footings • Negligible post-construction settlement • Can be used for support of conventional or integral abutments	• Downdrag loading must be taken into account in design, unless embankment areas are fully preloaded prior to construction of the overpass structures; and, • Piles may encounter obstructions (cobbles and boulders) during driving	• More costly than spread footings; and, • Installation cost could be impacted by presence of obstructions	• Negligible risk of post-construction settlement of overpass structure, or of differential settlement of foundation elements, • Low to moderate risk of encountering obstructions that could impact pile installation
Caisson foundations founded within very dense sand and silt till or hard clayey silt till	Feasible for support of abutments and piers	• Sub-excavation is not required • Highest bearing capacity • Negligible post-construction settlement • Can be used for support of conventional or semi-integral abutments	• Need for temporary or permanent liners during installation through water-bearing sand and silt till, • Cleaning and inspection of the base in cohesionless till below the water table could be difficult; and, • Caissons may encounter obstructions (cobbles and boulders) and/or "blowing" sand during installation; and, • Downdrag loading must be taken into account in design, unless embankment areas are fully preloaded prior to construction of the overpass structures.	• Additional cost associated with specialised drilling equipment and temporary or permanent liners • More costly option that steel H-piles	• Negligible risk of post-construction settlement of overpass structure, or of differential settlement of foundation elements, • 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 pile installation

MINISTRY OF TRANSPORTATION, ONTARIO



PLAN

SCALE

6 0 6 12 m

METRIC

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

CONT No.
WO No. 05–20012

SHEET

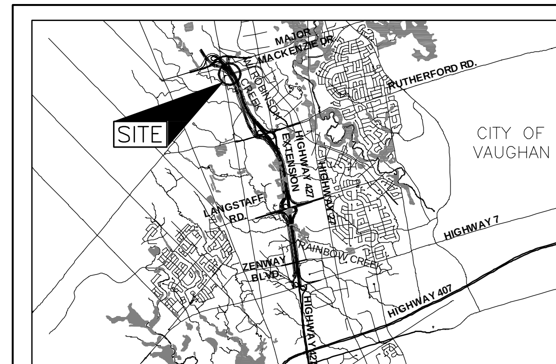
HIGHWAY 427 EXTENSION

CPR/McGILLIVRAY ROAD OVERPASSES

BOREHOLE LOCATIONS

Golder Associates

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



KEY PLAN

SCALE

2 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 25, 2009
- WL upon completion of drilling

No.	ELEVATION	CO-ORDINATES	
		NORTHING	EASTING
S25	201.8	4853399.9	292226.9
S26	201.5	4853413.1	292274.0
S27	201.1	4853423.2	292203.6
S28	200.8	4853435.3	292253.3
S29	202.0	4853505.8	292191.2
S30	202.3	4853513.2	292239.8

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 "cpr_ga.dwg", received May 15, 2009).

REGISTERED PROFESSIONAL ENGINEER
F. J. HEFFERNAN
Aug. 5, 2009
PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER
S.M. McGAGHRAN
Aug. 5, 2009
PROVINCE OF ONTARIO

NO.	DATE	BY	REVISION

Geocres No. 30M13–173

HWY. 427		PROJECT NO. 06–1111–012–8	DIST.
SUBM'D. SB/JEB	CHKD. SMM	DATE: 4–Aug–2009	SITE:
DRAWN: JFC	CHKD. SMM	APPD. LCC	DWG. 1

PLOT DATE: August 4, 2009

MINISTRY OF TRANSPORTATION, ONTARIO

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

CONT No.
WO No. 05–20012

HIGHWAY 427 EXTENSION
CPR/McGILLIVRAY ROAD OVERPASSES
SOIL STRATA

SHEET

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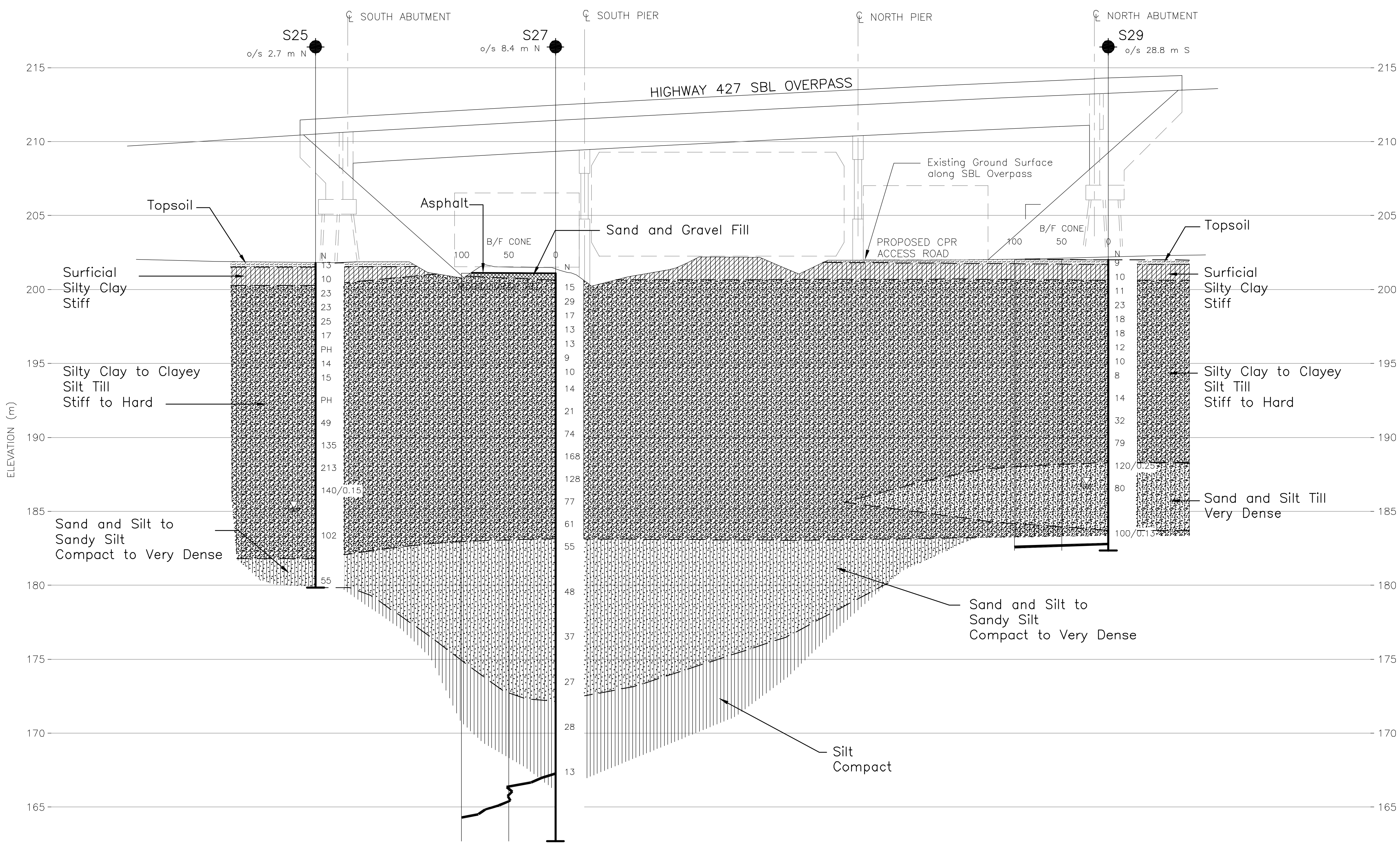
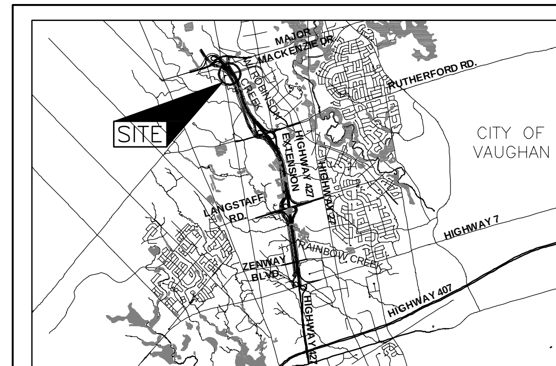
MISSISSAUGA, ONTARIO, CANADA



PROFILE A–A' CPR/McGILLIVRAY ROAD SBL OVERPASS

HORIZONTAL SCALE 6 0 6 12 m
3 0 3 6 m VERTICAL SCALE



KEY PLAN
SCALE 2 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 25, 2009
- WL upon completion of drilling

No.	ELEVATION	CO–ORDINATES	
		NORTHING	EASTING
S25	201.8	4853399.9	292226.9
S26	201.5	4853413.1	292274.0
S27	201.1	4853423.2	292203.6
S28	200.8	4853435.3	292253.3
S29	202.0	4853505.8	292191.2
S30	202.3	4853513.2	292239.8

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REGISTERED PROFESSIONAL ENGINEER F. J. HEFFERNAN Aug. 5, 2009 PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER S.M. McGAGHRAN Aug. 5, 2009 PROVINCE OF ONTARIO

NO.	DATE	BY	REVISION

Geocres No. 30M13–173

HWY. 427		PROJECT NO. 06–1111–012–8	DIST.
SUBM'D. SMM	CHKD. SMM	DATE: 5–Aug–2009	SITE:
DRAWN: JFC	CHKD. SMM	APPD. LCC	DWG. 2

PLOT DATE: August 5, 2009

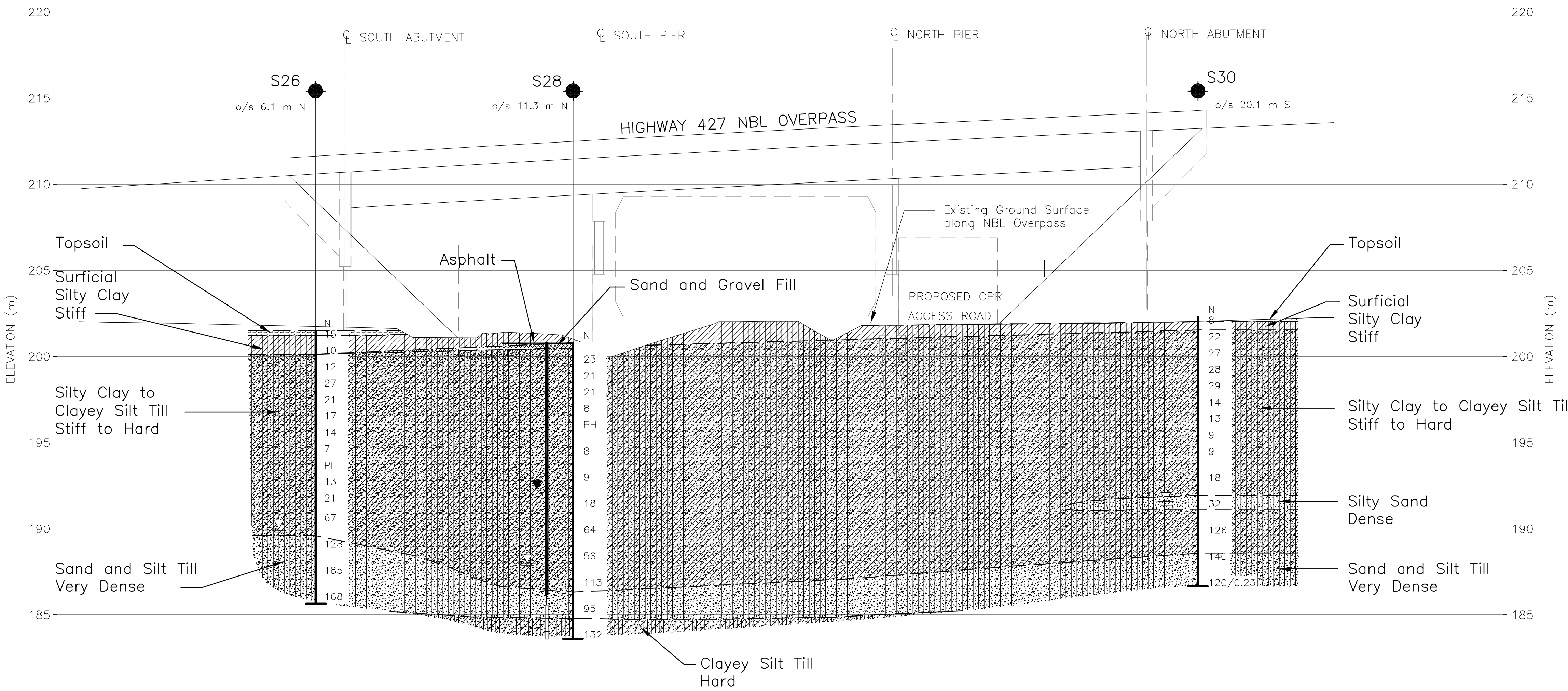
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

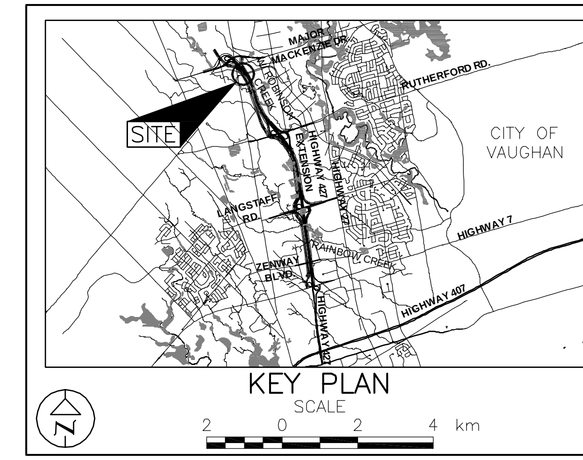
CONT No.
WO No. 05–20012

HIGHWAY 427 EXTENSION
CPR/McGILLIVRAY ROAD OVERPASSES
SOIL STRATA

SHEET

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA





KEY PLAN
SCALE
2 0 2 4 km

LEGEND

Symbol	Description
●	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 25, 2009
	WL upon completion of drilling

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REFERENCE

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PROFILE B–B' CPR/McGILLIVRAY ROAD NBL OVERPASS

HORIZONTAL SCALE
6 0 6 12 m
3 0 3 6 m
VERTICAL SCALE

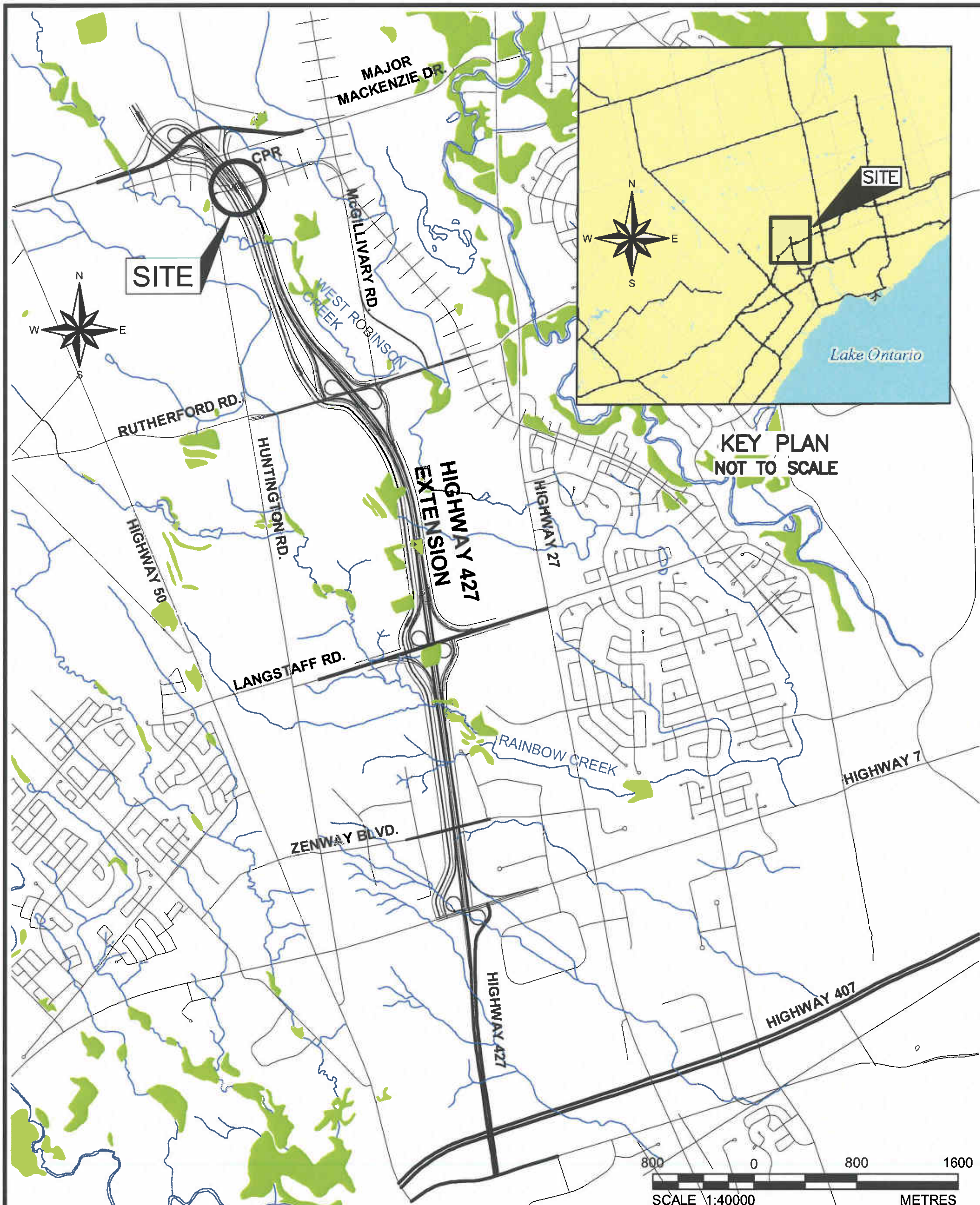
REGISTERED PROFESSIONAL ENGINEER F. J. HEFFERNAN Aug. 5, 2009 PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER S.M. McGAGHRAN Aug. 5, 2009 PROVINCE OF ONTARIO

NO.	DATE	BY	REVISION

Geocres No. 30M13–173

HWY. 427		PROJECT NO. 06–1111–012–8	DIST.
SUBM'D. SB/JEB	CHKD. SMM	DATE: 5–Aug–2009	SITE:
DRAWN: JFC	CHKD. SMM	APPD. LCC	DWG. 3



Golder Associates
Mississauga, Ontario, Canada

SCALE	AS SHOWN	TITLE	SITE LOCATION PLAN CPR/McGILLIVRAY ROAD OVERPASSES
DATE	Aug. 5, 2009		
DESIGN	PKS		
CAD	JFC		
CHECK	SMM	HIGHWAY 427 EXTENSION	FIGURE 1
REVIEW	LCC		

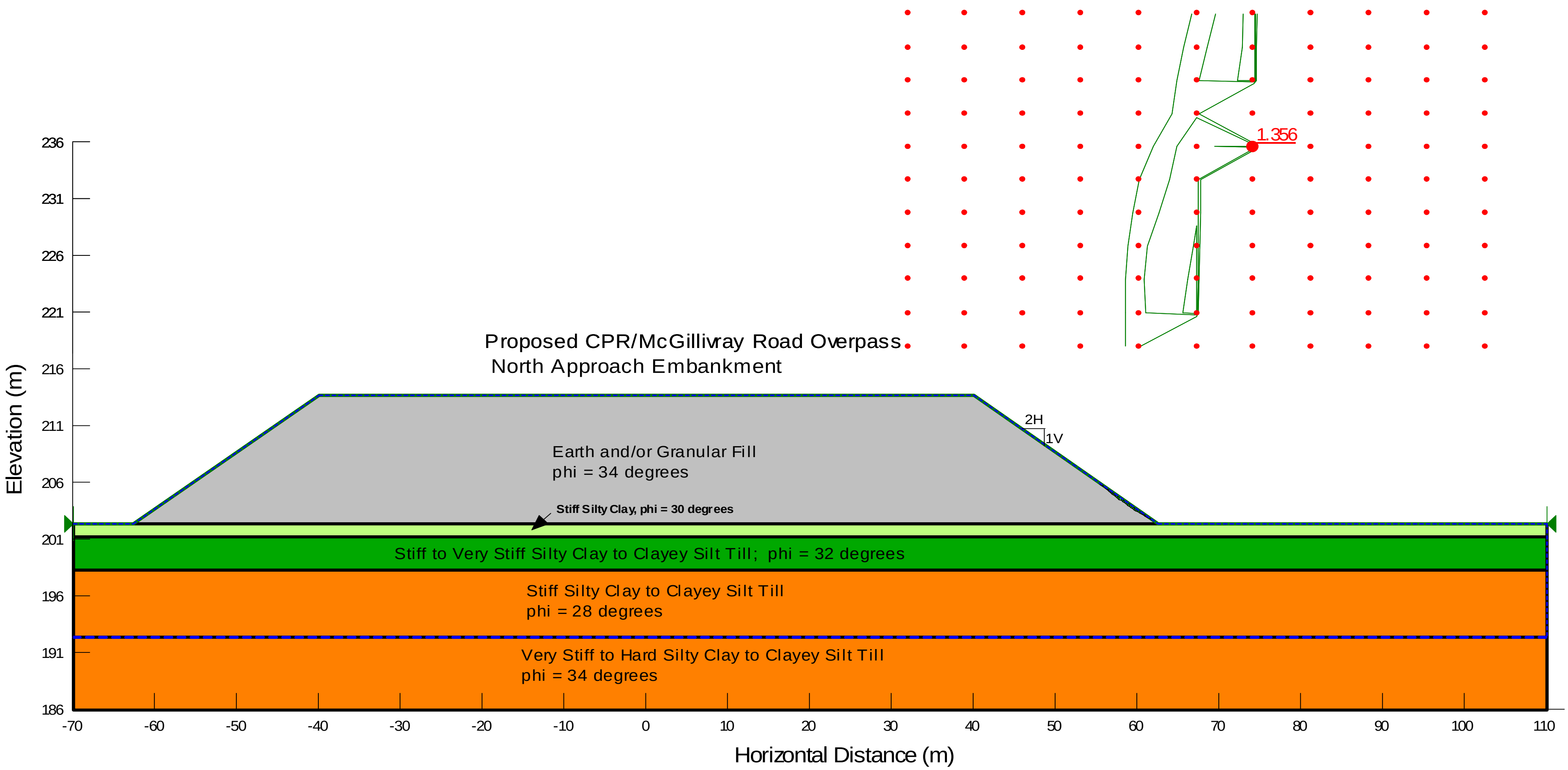
FILE No. 061111012IB0F1.dwg

PROJECT No. 06-1111-012-8 REV. B

PLOT DATE: August 5, 2009
FILENAME: T:\Projects\2006\06-1111-012 (MRC, Vaughan)\-IB- (CPR)\061111012IB0F1.dwg

HIGHWAY 427 EXTENSION - CPR/McGILLIVRAY ROAD OVERPASS
NORTH APPROACH EMBANKMENT- STATIC GLOBAL STABILITY

FIGURE 2

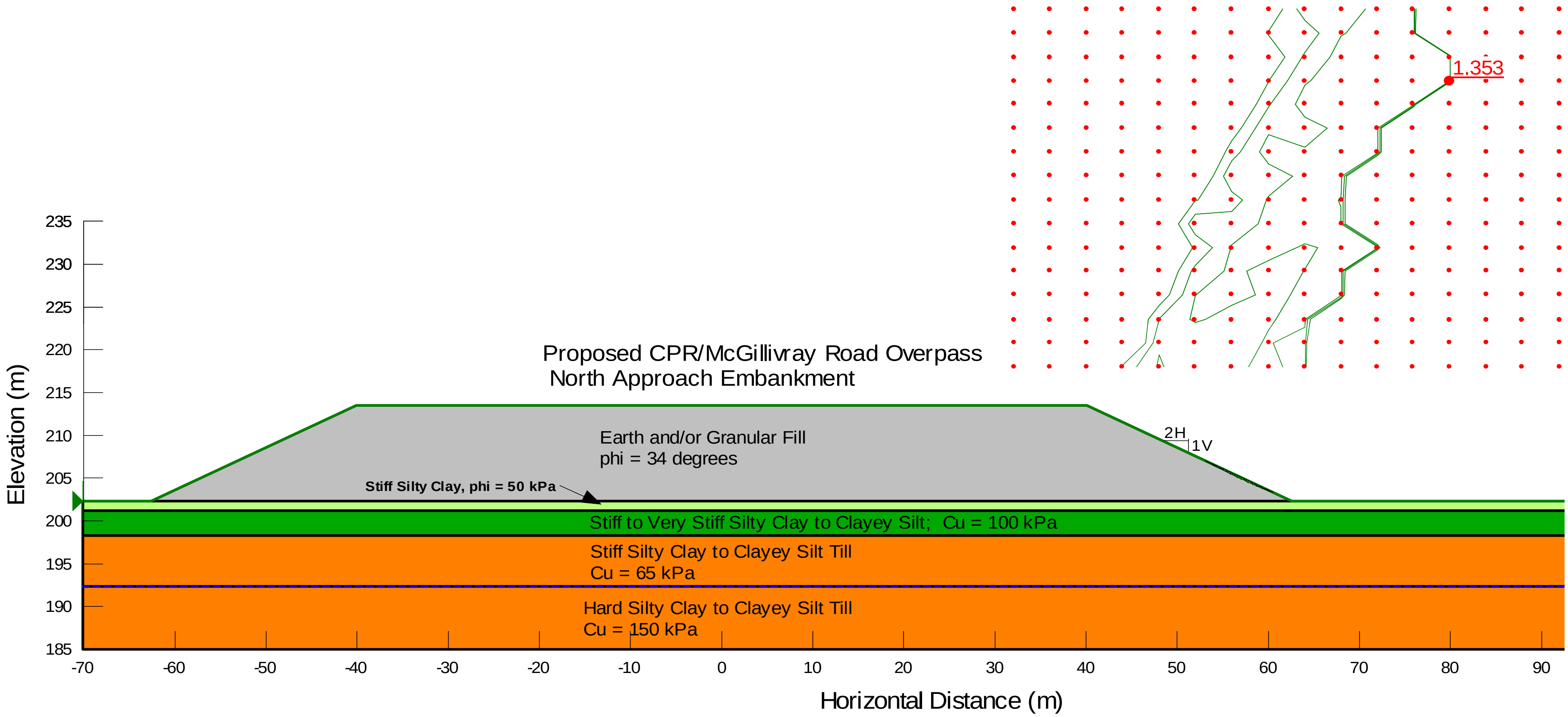


Date: June 2009
Project: 06-1111-012 - 8

Golder Associates

Drawn: SMM
Checked: LCC

HIGHWAY 427 EXTENSION - CPR/McGILLIVRAY ROAD OVERPASS NORTH APPROACH EMBANKMENT- UNDRAINED STATIC GLOBAL STABILITY	FIGURE 3



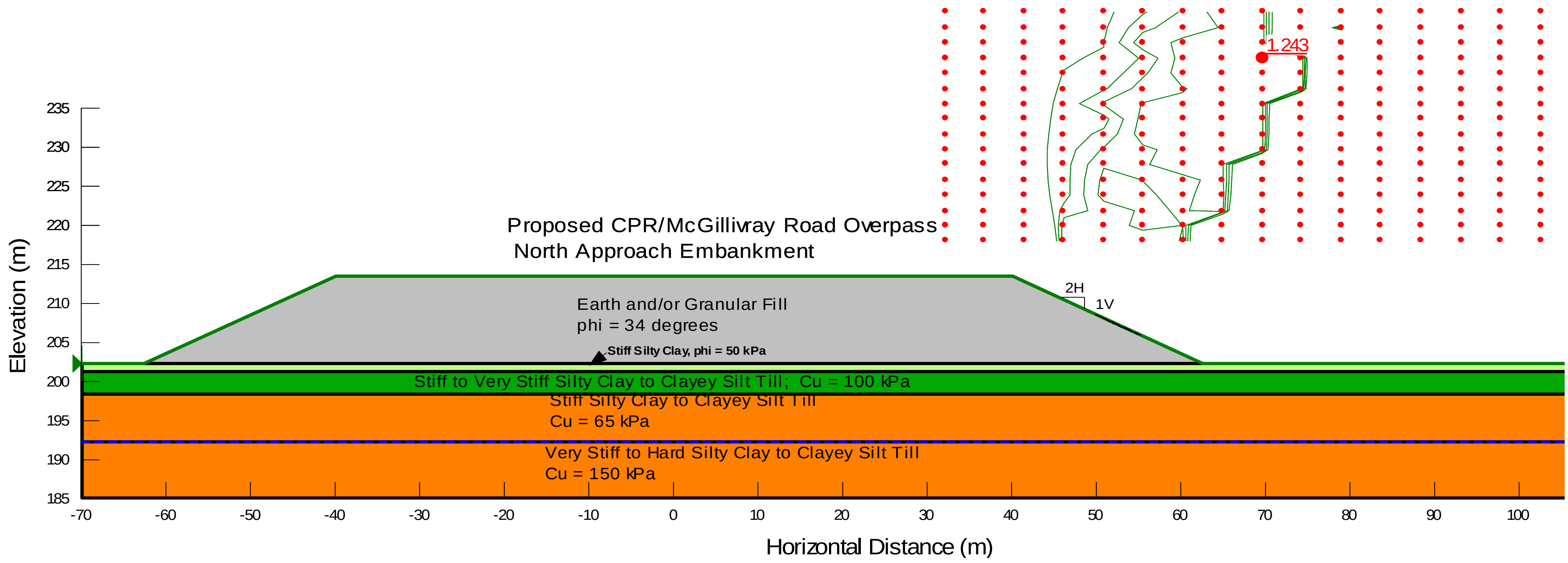
Date: June 2009
Project: 06-1111-012 - 8

Golder Associates

Drawn: SMM
Checked: LCC

HIGHWAY 427 EXTENSION - CPR/McGILLIVRAY ROAD OVERPASS
NORTH APPROACH EMBANKMENT- SEISMIC GLOBAL STABILITY

FIGURE 4



Date:	June 2009		Drawn:	SMM
Project:	06-1111-012 - 8	**Golder Associates**	Checked:	LCC

APPENDIX A

Borehole Records

LIST OF SYMBOLS

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

I. GENERAL

π	3.1416
in 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
υ	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_1	liquid limit
w_p	plastic limit
I_p	plasticity index $= (w_1 - w_p)$
w_s	shrinkage limit
I_L	liquidity index $= (w - w_p) / I_p$
I_C	consistency index $= (w_1 - w) / I_p$
e_{max}	void ratio in loosest state
e_{min}	void ratio in densest state
I_D	density index $= (e_{max} - e) / (e_{max} - e_{min})$ (formerly relative density)

(b) Hydraulic Properties

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

(c) Consolidation (one-dimensional)

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

(d) Shear Strength

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

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

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, 60º 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^2 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	c_u, s_u psf
Very soft	0 to 12	0 to 250
Soft	12 to 25	250 to 500
Firm	25 to 50	500 to 1,000
Stiff	50 to 100	1,000 to 2,000
Very stiff	100 to 200	2,000 to 4,000
Hard	over 200	over 4,000

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.

Golder Associates

RECORD OF BOREHOLE No S25

 METRIC

PROJECT 06-1111-012			
W.O. 05-20012	LOCATION N 4853399.9 ;E 292226.9	ORIGINATED BY CR	
DIST Central HWY 427	BOREHOLE TYPE 200 mm Outside Diameter Hollow Stem Augers	COMPILED BY PKS/VA	
DATUM Geodetic	DATE March 12, 13 & 16, 2009	CHECKED BY SMM	

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	SHEAR STRENGTH / DCPT	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
201.8	GROUND SURFACE						
201.5 0.0	TOPSOIL	1	SS	13			
0.3	SILTY CLAY, some sand, trace gravel (Reworked to a depth of 0.8 m) Stiff Brown to grey Moist	2	SS	10			
200.3 1.5	SILTY CLAY, trace sand, trace gravel (TILL) Very stiff Brown to grey Moist	3	SS	23		44	
		4	SS	23			
		5	SS	25			
198.1 3.7	CLAYEY SILT, trace to some sand, trace to some gravel (TILL) Stiff to hard Grey Moist to wet	6	SS	17			
		7	TO	PH	+ >100		
		8	SS	14			4 12 56 28
		9	SS	15			
		10	TO	PH			
	Auger grinding between a depth of 10.7 m to 11.3 m	11	SS	49			
	Auger grinding between a depth of 12.2 m to 12.8 m	12	SS	135			
	Auger grinding between a depth of 13.1 m to 14.3 m						
		13	SS	213			

Elevation scale: 201, 200, 199, 198, 197, 196, 195, 194, 193, 192, 191, 190, 189, 188, 187

Continued Next Page

$+^3, \times^3$: Numbers refer to Sensitivity O $^{3\%}$ STRAIN AT FAILURE

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

Foundation Design

Golder Associates

RECORD OF BOREHOLE No S25

 METRIC

PROJECT 06-1111-012			ORIGINATED BY CR
W.O. 05-20012	LOCATION N 4853399.9 ;E 292226.9		COMPILED BY PKS/VA
DIST Central	HWY 427	BOREHOLE TYPE 200 mm Outside Diameter Hollow Stem Augers	CHECKED BY SMM
DATUM Geodetic		DATE March 12, 13 & 16, 2009	

— CONTINUED FROM PREVIOUS PAGE —

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	CLAYEY SILT, trace to some sand, trace to some gravel (TILL) Stiff to hard Grey Moist to wet	14	SS	40/0.15		186		
	Auger grinding between a depth of 16.5 m to 16.8 m				∇	185		
	Auger grinding between a depth of 17.4 m to 17.8 m					184		
		15	SS	102		183	o	23 16 44 17
181.8 20.0	SAND and SILT, trace gravel Very dense Grey Wet					182, 181		
		16	SS	55		180		
179.9 22.0	END OF BOREHOLE							

NOTES:

1. Water level in open borehole at a depth of 16.6 m below ground surface (Elev. 185.2 m) upon completion of drilling.
2. Borehole backfilled with bentonite.

Legend: DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m³

$+^3, \times^3$: Numbers refer to Sensitivity O $^{3\%}$ STRAIN AT FAILURE

Golder Associates

RECORD OF BOREHOLE No S26

 METRIC

PROJECT 06-1111-012			ORIGINATED BY CR
W.O. 05-20012	LOCATION N 4853413.1 ;E 292274.0		COMPILED BY VA
DIST Central HWY 427	BOREHOLE TYPE 200 mm Outside Diameter Hollow Stem Augers		CHECKED BY SMM
DATUM Geodetic	DATE March 11 & 12, 2009		

SOIL PROFILE / SAMPLES / GROUND WATER CONDITIONS / ELEVATION SCALE / DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED; 20 40 60 80 100) / PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L — WATER CONTENT (%) 10 20 30 / UNIT WEIGHT γ kN/m³ / REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

ELEV DEPTH	DESCRIPTION
201.5	GROUND SURFACE
201.2 / 0.0	TOPSOIL
0.3	SILTY CLAY, some gravel, some sand, containing organics and rootlets (Reworked to a depth of 0.8 m) Stiff Brown Moist
200.1 / 1.4	CLAYEY SILT, some sand, trace to some gravel (TILL), containing oxidation zones Stiff to very stiff Brown Moist
197.8 / 3.7	SILTY CLAY, trace to some sand, trace gravel (TILL) Stiff to very stiff Grey Moist to wet Wet below a depth of 6.1 m
192.8 / 8.7	CLAYEY SILT, some sand, trace gravel, containing cobbles and boulders (TILL) Very stiff to hard Grey Moist
189.6 / 11.9	SAND and SILT, trace clay, trace gravel (TILL) Very dense Grey Moist

NUMBER	TYPE	"N" VALUES	REMARKS (GR SA SI CL)
1	SS	15	
2	SS	10	
3	SS	12	
4	SS	27	
5	SS	21	
6	SS	17	
7	SS	14	
8	SS	7	
9	TO	PH	
10	SS	13	
11	SS	21	0 17 57 26
12	SS	67	
13	SS	128	
14	SS	185	

Elevation scale: 201, 200, 199, 198, 197, 196, 195, 194, 193, 192, 191, 190, 189, 188, 187

43

>100

Continued Next Page

$+^3, \times^3$: Numbers refer to Sensitivity $\circ^{3\%}$ STRAIN AT FAILURE

Golder Associates

RECORD OF BOREHOLE No S26

 METRIC

PROJECT 06-1111-012
W.O. 05-20012 LOCATION N 4853413.1 ;E 292274.0 ORIGINATED BY CR
DIST Central HWY 427 BOREHOLE TYPE 200 mm Outside Diameter Hollow Stem Augers COMPILED BY VA
DATUM Geodetic DATE March 11 & 12, 2009 CHECKED BY SMM

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED) (20 40 60 80 100)	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L — WATER CONTENT (%) (10 20 30)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	— CONTINUED FROM PREVIOUS PAGE —										
185.7 15.9	SAND and SILT, trace clay, trace gravel (TILL) Very dense Grey Moist		15	SS	168		186				
	END OF BOREHOLE										

NOTES:

1. Water level in open borehole at a depth of 11.5 m below ground surface (Elev. 190.0 m) upon completion of drilling.

2. Borehole caved to a depth of 13.0 m below ground surface (Elev. 188.5 m) upon removal of augers and backfilled with bentonite.

$+^3, \times^3$: Numbers refer to Sensitivity O $^{3\%}$ STRAIN AT FAILURE

Golder Associates

RECORD OF BOREHOLE No S27

 METRIC

PROJECT 06-1111-012			ORIGINATED BY SB
W.O. 05-20012	LOCATION N 4853423.2 ;E 292203.6		
DIST Central HWY 427	BOREHOLE TYPE 200 mm Outside Diameter Hollow Stem Augers		COMPILED BY JB/VA
DATUM Geodetic	DATE March 13, 2009		CHECKED BY SMM

SOIL PROFILE

ELEV DEPTH	DESCRIPTION
201.1	GROUND SURFACE
0.0	Asphalt
200.6 0.5	Sand and gravel (FILL) Compact Brown Moist
	SILTY CLAY, trace gravel, trace sand (TILL) Stiff to very stiff Brown Moist Becoming grey at a depth of 2.2 m
196.5 4.6	CLAYEY SILT, with sand, trace to some gravel (TILL) Stiff to hard Grey Moist

SAMPLES

NUMBER	TYPE	"N" VALUES	Remarks & Grain Size Distribution (%) GR SA SI CL
1	SS	15	
2	SS	29	
3	SS	17	
4	SS	13	
5	SS	13	
6	SS	9	
7	SS	10	
8	SS	14	
9	SS	21	
10	SS	74	
11	SS	168	24 24 44 8
12	SS	128	

Column headings: GROUND WATER CONDITIONS; ELEVATION SCALE (201 to 187); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa — O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED — 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L — WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

Plot annotations: 44; >145; >145

Continued Next Page

$+^3, \times^3$: Numbers refer to Sensitivity O $^{3\%}$ STRAIN AT FAILURE

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

Golder Associates

RECORD OF BOREHOLE No S27

 METRIC

PROJECT 06-1111-012
W.O. 05-20012
LOCATION N 4853423.2 ;E 292203.6
ORIGINATED BY SB
DIST Central HWY 427
BOREHOLE TYPE 200 mm Outside Diameter Hollow Stem Augers
COMPILED BY JB/VA
DATUM Geodetic
DATE March 13, 2009
CHECKED BY SMM

— CONTINUED FROM PREVIOUS PAGE —

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	GR	SA	SI	CL
	CLAYEY SILT, with sand, trace to some gravel (TILL) Stiff to hard Grey Moist	13	SS	77	186		4	29	47	20
		14	SS	61	184					
183.1 18.0	SAND and SILT, trace clay Dense to very dense Grey Moist to wet	15	SS	55	183					
		16	SS	48	180					
177.4 23.7	Sandy SILT, trace clay Compact to dense Grey Moist to wet	17	SS	37	177		0	28	68	4
		18	SS	27	173					
172.1 29.0	SILT, trace clay Compact Grey Moist to wet				172					

Legend: DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100); SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED (20, 40, 60, 80, 100); PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

Continued Next Page

$+^3, \times^3$: Numbers refer to Sensitivity $\circ^{3\%}$ STRAIN AT FAILURE

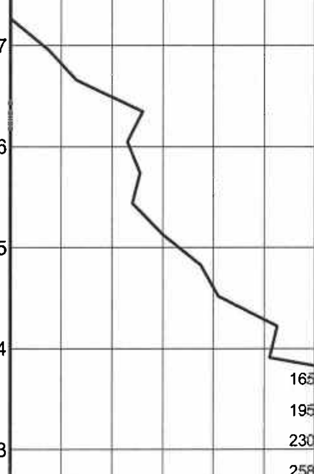
RECORD OF BOREHOLE No S27

 METRIC

PROJECT 06-1111-012			
W.O. 05-20012	LOCATION N 4853423.2 ;E 292203.6		ORIGINATED BY SB
DIST Central HWY 427	BOREHOLE TYPE 200 mm Outside Diameter Hollow Stem Augers		COMPILED BY JB/VA
DATUM Geodetic	DATE March 13, 2009		CHECKED BY SMM

ELEV DEPTH	DESCRIPTION (— CONTINUED FROM PREVIOUS PAGE —)	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%) w	GR SA SI CL
	SILT, trace clay Compact Grey Moist to wet				171		
		19	SS	28		o (about 21)	0 1 90 9
					170		
					169		
					168		
		20	SS	13			
167.0 34.1	END OF BOREHOLE Dynamic Cone Penetration Test (DCPT) was performed between depths of 33.8 m and 38.4 m				167		
					166		
					165		
					164		
					163		
162.7 38.4	END OF DCPT NOTES: 1. Borehole open to 25 m and dry upon completion of drilling. 2. A Dynamic Cone Penetration Test was carried out between depths of 33.8 m and 38.4 m . 3. Borehole backfilled with bentonite.						

Column headings: SOIL PROFILE (ELEV DEPTH, DESCRIPTION, STRAT PLOT); SAMPLES (NUMBER, TYPE, "N" VALUES); GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED; 20 40 60 80 100); PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL



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

Golder Associates

Foundation Design

RECORD OF BOREHOLE No S28

 METRIC

PROJECT 06-1111-012
W.O. 05-20012
DIST Central HWY 427
DATUM Geodetic

LOCATION N 4853435.3 ;E 292253.3
BOREHOLE TYPE 200 mm Outside Diameter Hollow Stem Augers
DATE March 17, 2009

ORIGINATED BY SB
COMPILED BY JB/VA
CHECKED BY SMM

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	SHEAR STRENGTH kPa / DCPT	WATER CONTENT (%)	REMARKS GR SA SI CL
200.8	GROUND SURFACE							
0.0	Asphalt							
0.3	Sand and gravel, trace silt (FILL) Brown Moist							
	CLAYEY SILT, trace gravel, trace sand (TILL) Very stiff Brown and grey Moist	1	SS	23	200			
		2	SS	21	199			
		3	SS	21	198			
197.8 / 3.1	SILTY CLAY, trace sand, trace gravel (TILL) Stiff Grey Moist	4	SS	8				
					197	>145		
		5	TO	PH	196			
					195	>145		
		6	SS	8	194			
						>145		
		7	SS	9	193			
						>145		
192.0 / 8.8	CLAYEY SILT, with sand, trace gravel (TILL) Very stiff to hard Grey Moist				192	>145		
		8	SS	18	191			
		9	SS	64	190			3 30 55 12
					189			
		10	SS	56	188			
		11	SS	113	187			
186.3 / 14.5					186			

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED; 20 40 60 80 100
PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30
UNIT WEIGHT γ kN/m³

Continued Next Page

$+^3, \times^3$: Numbers refer to Sensitivity $O^{3\%}$ STRAIN AT FAILURE

Golder Associates

Foundation Design

RECORD OF BOREHOLE No S28

 METRIC

PROJECT 06-1111-012
W.O. 05-20012
DIST Central HWY 427
DATUM Geodetic

LOCATION N 4853435.3 ;E 292253.3
BOREHOLE TYPE 200 mm Outside Diameter Hollow Stem Augers
DATE March 17, 2009

ORIGINATED BY SB
COMPILED BY JB/VA
CHECKED BY SMM SM

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	GR	SA	SI	CL
	— CONTINUED FROM PREVIOUS PAGE —									
184.8 16.0	SAND and SILT, trace gravel, trace clay (TILL) Very dense Grey Moist	12	SS	95	185	H				
183.6 17.2	CLAYEY SILT, some sand, trace gravel (TILL) Hard Grey Moist	13	SS	132	184	o	2	12	59	27

END OF BOREHOLE

NOTES:

1. A 50 mm diameter monitoring well was installed at a depth of 16.8 m (Elev. 184.0 m).

Water level measurements

Date	Depth	Elev.
On Completion	12.8 m	188.0 m
April 27, 2009	8.5 m	192.3 m
May 13, 2009	8.5 m	192.3 m
May 25, 2009	8.6 m	192.2 m
June 15, 2009	9.1 m	191.7 m
July 09, 2009	9.1 m	191.7 m

2. Borehole backfilled with bentonite.

Column headings: DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%); STRAT PLOT; GROUND WATER CONDITIONS.

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

Golder Associates

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RECORD OF BOREHOLE No S29

 METRIC

PROJECT 06-1111-012

W.O. 05-20012

LOCATION N 4853505.8 ;E 292191.2

ORIGINATED BY JEB

DIST Central HWY 427

BOREHOLE TYPE 200 mm Outside Diameter Hollow Stem Augers

COMPILED BY PKS/VA

DATUM Geodetic

DATE April 27, 2009

CHECKED BY SMM

ELEV DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
202.0	GROUND SURFACE					
201.7 / 0.0	TOPSOIL					
0.3	SILTY CLAY, trace sand, trace gravel (Reworked to a depth of 0.8 m) Stiff Brown Moist	1	SS	9		
		2	SS	10	201	
200.6 / 1.4	CLAYEY SILT, trace to some sand, trace gravel (TILL) Stiff to hard Brown becoming grey below a depth of 2.3 m Moist	3	SS	11	200	
		4	SS	23	199	
		5	SS	18		
	Auger grinding between a depth of 3.8 m to 4.4 m	6	SS	18	198	
		7	SS	12	197	0 5 41 54
		8	SS	10	196	
		9	SS	8	195 / 194	
		10	SS	14	193	
		11	SS	32	192 / 191	
		12	SS	79	190 / 189	
188.3 / 13.7	SAND and SILT, trace gravel, trace clay (TILL) Very dense Grey Moist	13	SS	20/0.25	188	1 29 62 8

Column headings: STRAT PLOT; GROUND WATER CONDITIONS; DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED (20 40 60 80 100); PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m³

Continued Next Page

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

Golder Associates

Foundation Design

RECORD OF BOREHOLE No S29

 METRIC

PROJECT 06-1111-012

W.O. 05-20012 LOCATION N 4853505.8 ;E 292191.2 ORIGINATED BY JEB

DIST Central HWY 427 BOREHOLE TYPE 200 mm Outside Diameter Hollow Stem Augers COMPILED BY PKS/VA

DATUM Geodetic DATE April 27, 2009 CHECKED BY SMM

— CONTINUED FROM PREVIOUS PAGE —

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	SAND and SILT, trace gravel, trace clay (TILL) Very dense Grey Moist Becoming wet below a depth of 15.2 m	14	SS	80	∇	186, 185, 184	
183.7	CLAYEY SILT, trace sand, trace gravel (TILL) Hard Grey Wet	15	SS	100/0.13		183	
183.4 / 18.7	END OF BOREHOLE						
182.4 / 19.7	End of DCPT Dynamic Cone Penetration Test (DCPT) below a depth of 18.7 m						

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 140, 240

NOTES:

1. At 15.2 m depth (Elev. 186.8 m) 1.0 m of sand was up inside the augers during drilling due to "blowing" sands.

2. Water level in open borehole at a depth of 15.2 m below ground surface (Elev. 186.8 m) upon completion of drilling.

3. Borehole backfilled with bentonite.

$+^3, \times^3$: Numbers refer to Sensitivity   O $^{3\%}$ STRAIN AT FAILURE

Golder Associates

RECORD OF BOREHOLE No S30

 METRIC

PROJECT	06-1111-012		
W.O.	05-20012	LOCATION	N 4853513.2 ;E 292239.8
DIST	Central HWY 427	BOREHOLE TYPE	200 mm Outside Diameter Hollow Stem Augers
DATUM	Geodetic	DATE	April 27, 2009

ORIGINATED BY JEB
COMPILED BY PKS/VA
CHECKED BY SMM

SOIL PROFILE

ELEV DEPTH	DESCRIPTION
202.3	GROUND SURFACE
0.0	TOPSOIL
202.0 / 0.3	CLAYEY SILT, some sand, trace gravel, containing organics (Reworked) Stiff Brown Moist
201.5 / 0.8	SILTY CLAY, trace sand, trace gravel (TILL) Stiff to very stiff Brown grey Moist. Becoming grey below a depth of 4.6 m
196.7 / 5.6	CLAYEY SILT, some sand, trace gravel (TILL) Stiff to very stiff Grey Moist
191.9 / 10.4	Silty SAND, trace gravel, trace clay Dense Grey Wet
191.1 / 11.2	CLAYEY SILT, with sand, trace gravel (TILL) Hard Grey Moist. Auger grinding at a depth of 13.4 m
188.6 / 13.7	SAND and SILT, some gravel, trace clay (TILL) Very dense Grey Wet

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	8	
2	SS	22	
3	SS	27	
4	SS	28	
5	SS	29	
6	SS	14	
7	SS	13	
8	SS	9	
9	SS	9	
10	SS	18	
11	SS	32	
12	SS	126	3 26 59 12
13	SS	140	13 31 52 4

ELEVATION SCALE: 202, 201, 200, 199, 198, 197, 196, 195, 194, 193, 192, 191, 190, 189, 188

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED: 20 40 60 80 100

PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L
WATER CONTENT (%): 10 20 30
48

UNIT WEIGHT γ kN/m³

GROUND WATER CONDITIONS

Continued Next Page

$+^3, \times^3$: Numbers refer to Sensitivity $O^{3\%}$ STRAIN AT FAILURE

Golder Associates

RECORD OF BOREHOLE No S30

 METRIC

PROJECT 06-1111-012

W.O. 05-20012

DIST Central HWY 427

DATUM Geodetic

LOCATION N 4853513.2 ;E 292239.8

BOREHOLE TYPE 200 mm Outside Diameter Hollow Stem Augers

DATE April 27, 2009

ORIGINATED BY JEB

COMPILED BY PKS/VA

CHECKED BY SMM SM

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED) 20 40 60 80 100	PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L / WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	— CONTINUED FROM PREVIOUS PAGE —										
	SAND and SILT, some gravel, trace clay (TILL) Very dense Grey Wet		14	SS	20/0.25		187				
186.7 15.6	Auger grinding at a depth of 14.6 m										
	END OF BOREHOLE										

NOTES:

1. Water level in open borehole at a depth of 10.7 m below ground surface (Elev. 191.6 m) upon completion of drilling.

2. Borehole backfilled with bentonite.

$+^3, \times^3$: Numbers refer to Sensitivity

$\circ^{3\%}$ STRAIN AT FAILURE

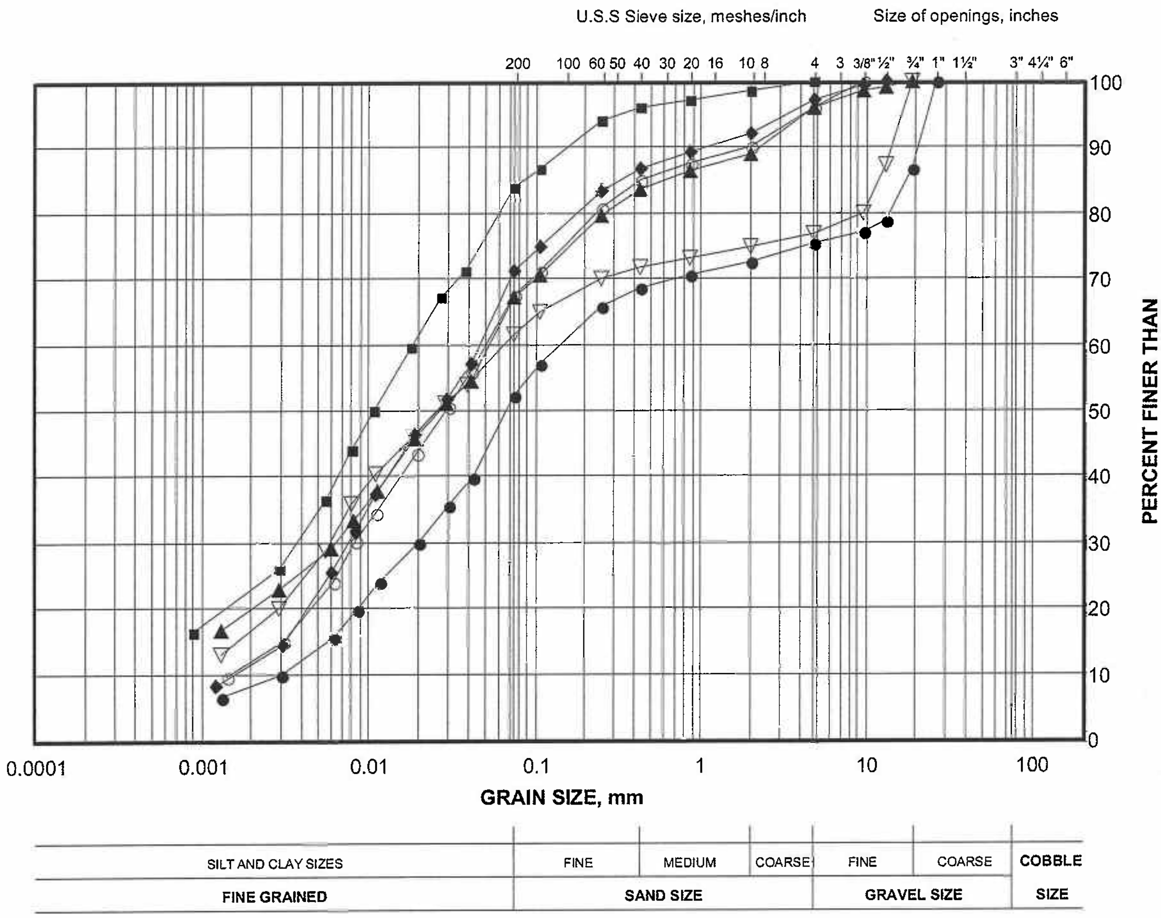
APPENDIX B

Laboratory Test Results

GRAIN SIZE DISTRIBUTION TEST RESULTS

Clayey Silt Till

FIGURE B1



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	S27	11	188.7
■	S26	12	190.5
◆	S30	12	190.0
▲	S27	13	185.6
▽	S25	15	183.2
○	S28	9	189.8

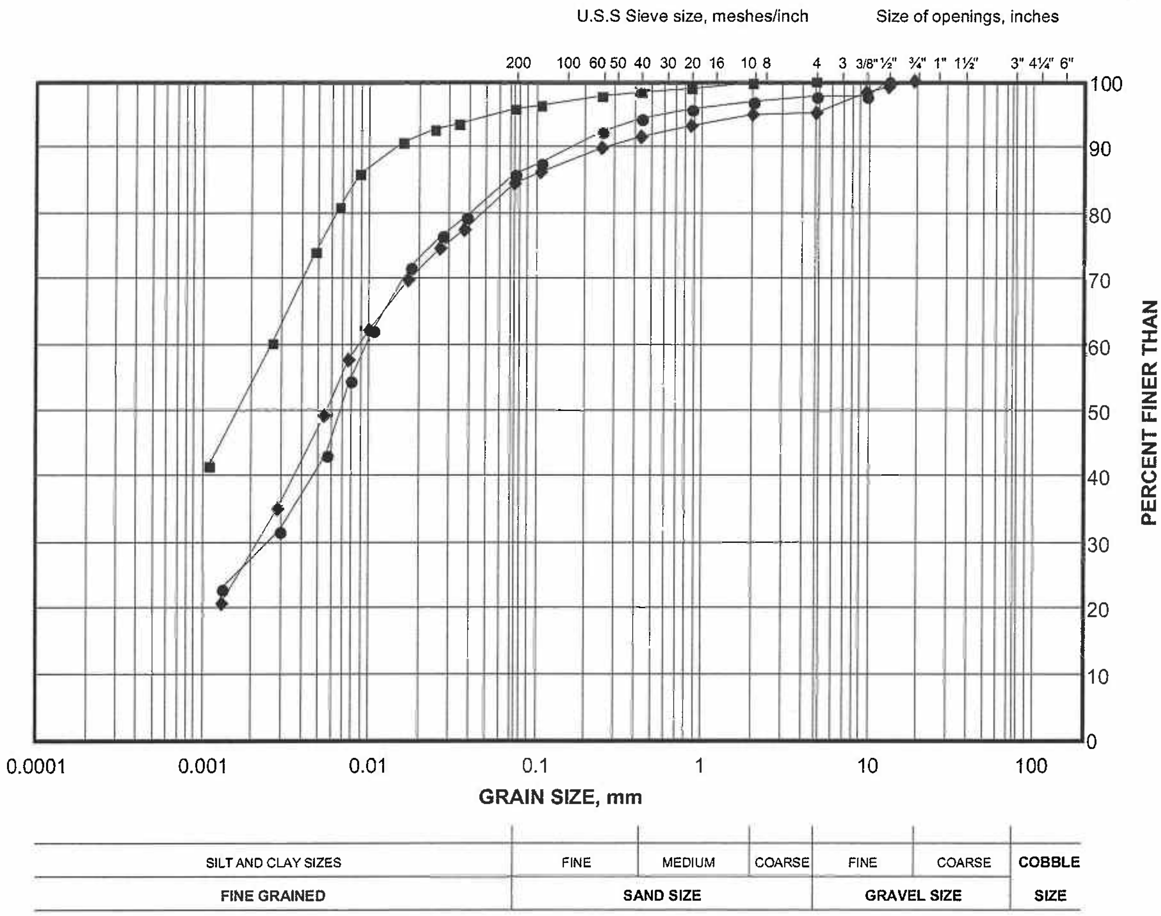
Project Number: 06-1111-012-8

Checked By: SM

Golder Associates

Date: 16-Jun-09

GRAIN SIZE DISTRIBUTION TEST RESULTS Clayey Silt Till	FIGURE B2



LEGEND

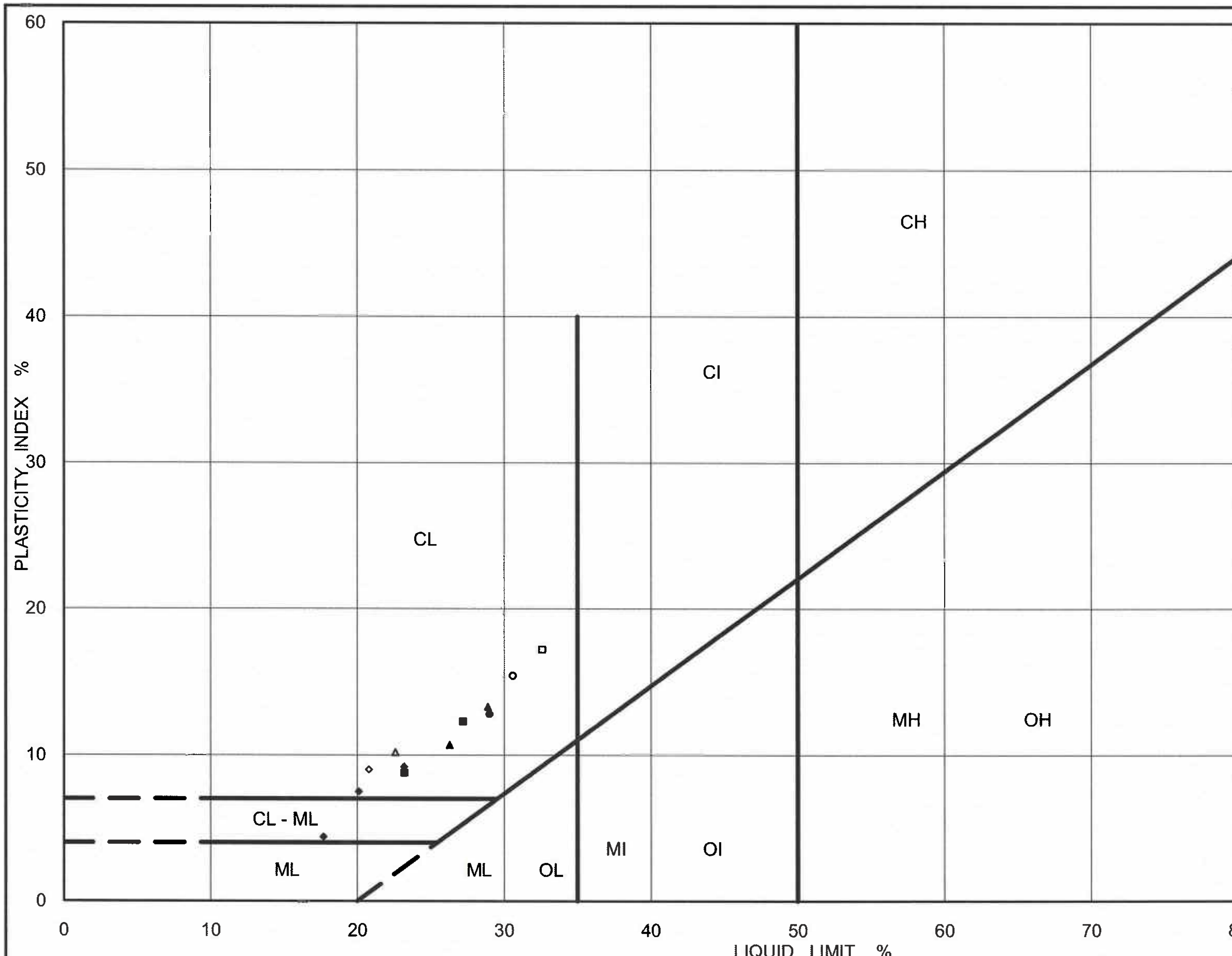
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	S28	13	183.8
■	S29	7	197.1
◆	S25	8	195.4

Project Number: 06-1111-012-8

Checked By: SM

Golder Associates

Date: 16-Jun-09



LEGEND

BH	SAMPLE	SYMBOL
S25	8	●
S25	11	◆
S26	4	▲
S26	12	■
S27	6	○
S27	13	◇
S28	8	△
S29	5	□
S29	9	■
S29	15	◆
S30	8	▲
S30	12	◆
		×
		●
		×
		□
		◇
		△
		○
		×

Ministry of Transportation

Ontario

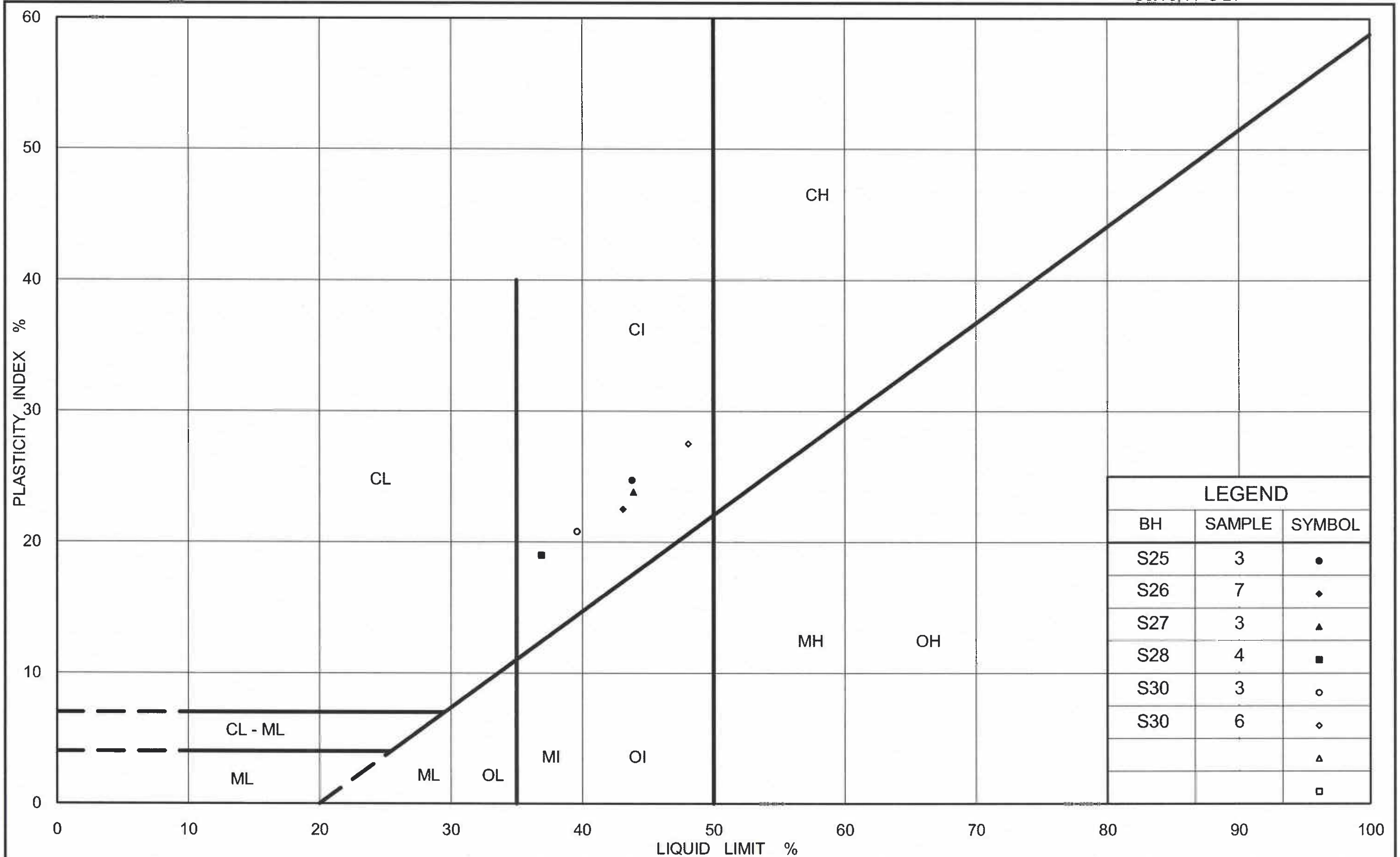
PLASTICITY CHART
Clayey Silt Till

Figure No. B3

Project No. 06-1111-012-8

Checked By: SM

Oct 75, FF-S-21



Ministry of Transportation

Ontario

PLASTICITY CHART
Silty Clay Till

Figure No. B4

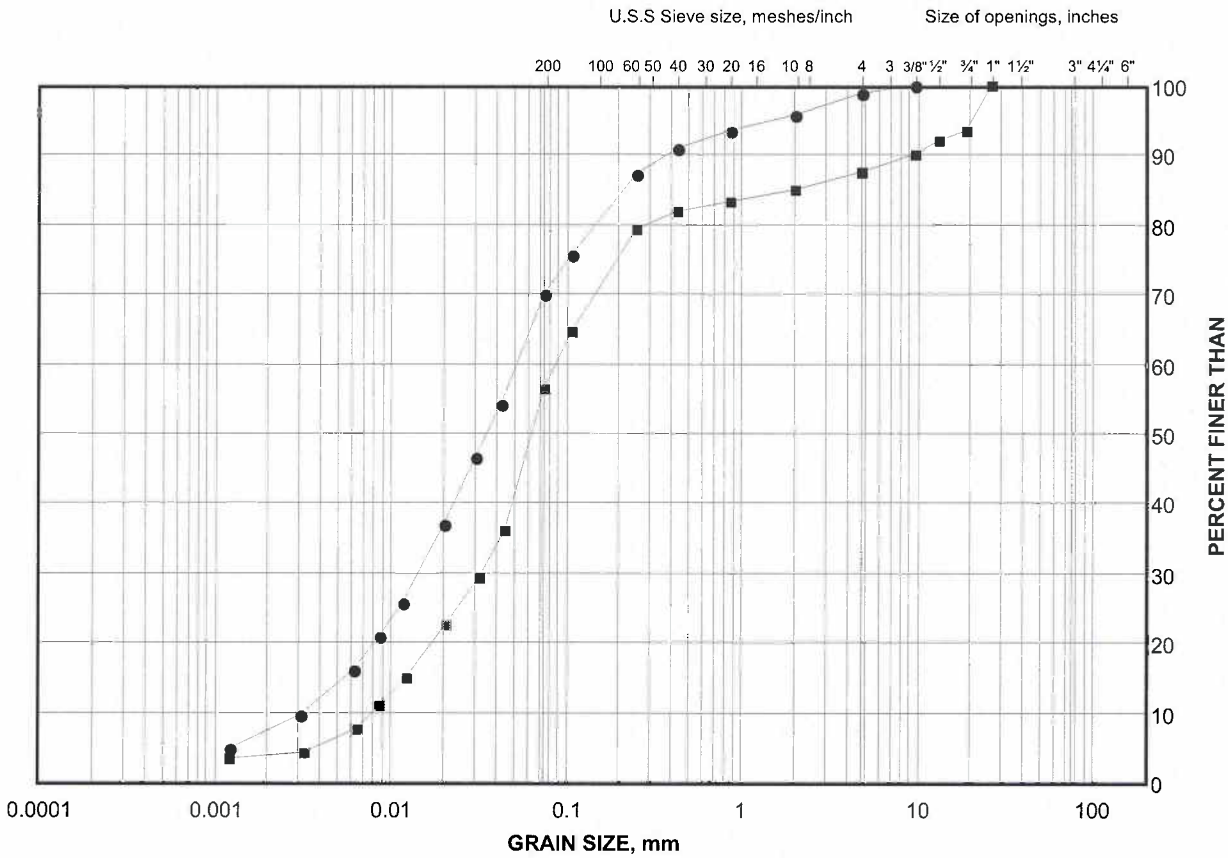
Project No. 06-1111-012-8

Checked By: SM

GRAIN SIZE DISTRIBUTION TEST RESULTS

Sand and 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)
●	S29	13	188.0
■	S30	13	188.3

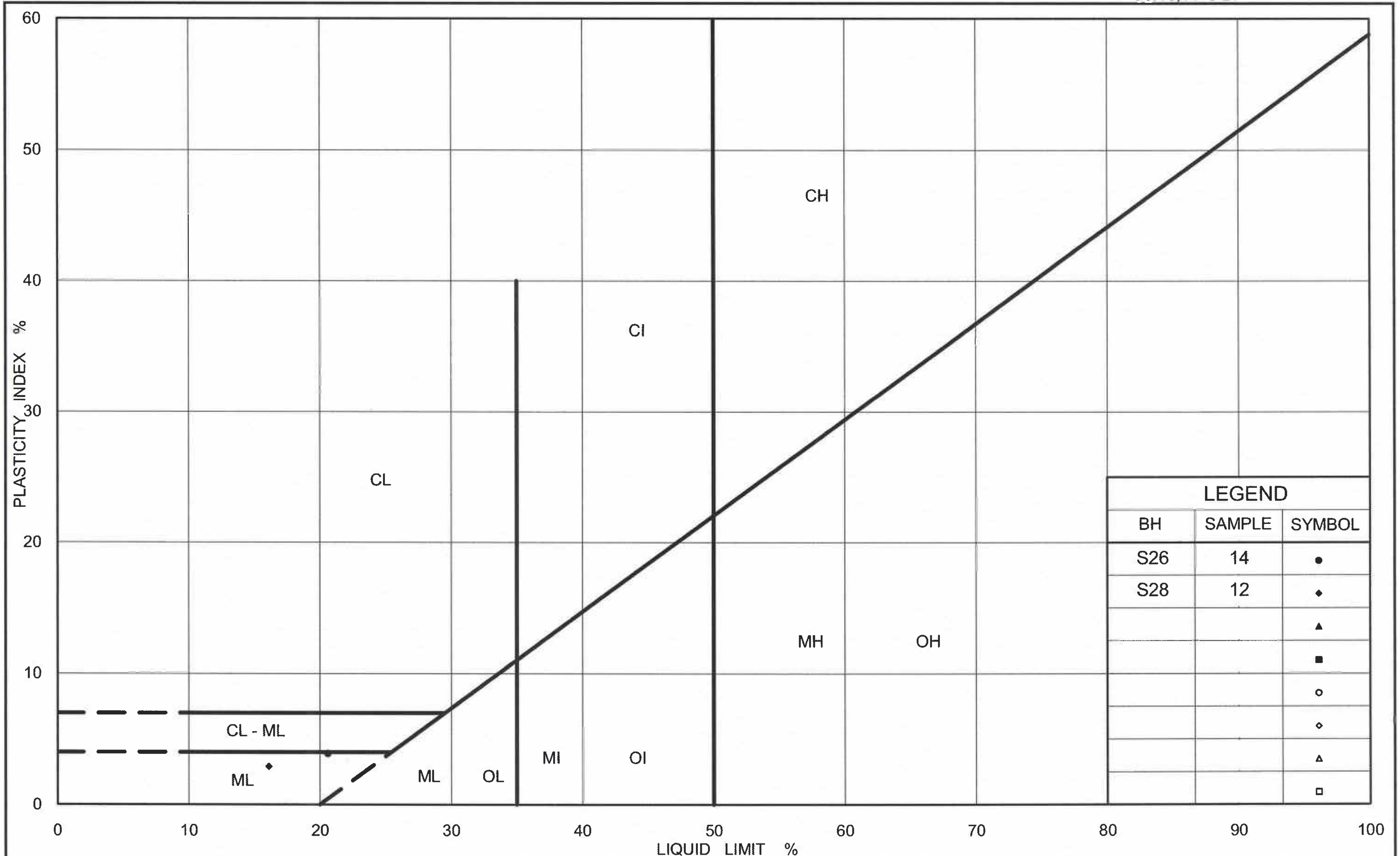
Project Number: 06-1111-012-8

Checked By:

Golder Associates

Date: 16-Jun-09

Oct 75, FF-S-21

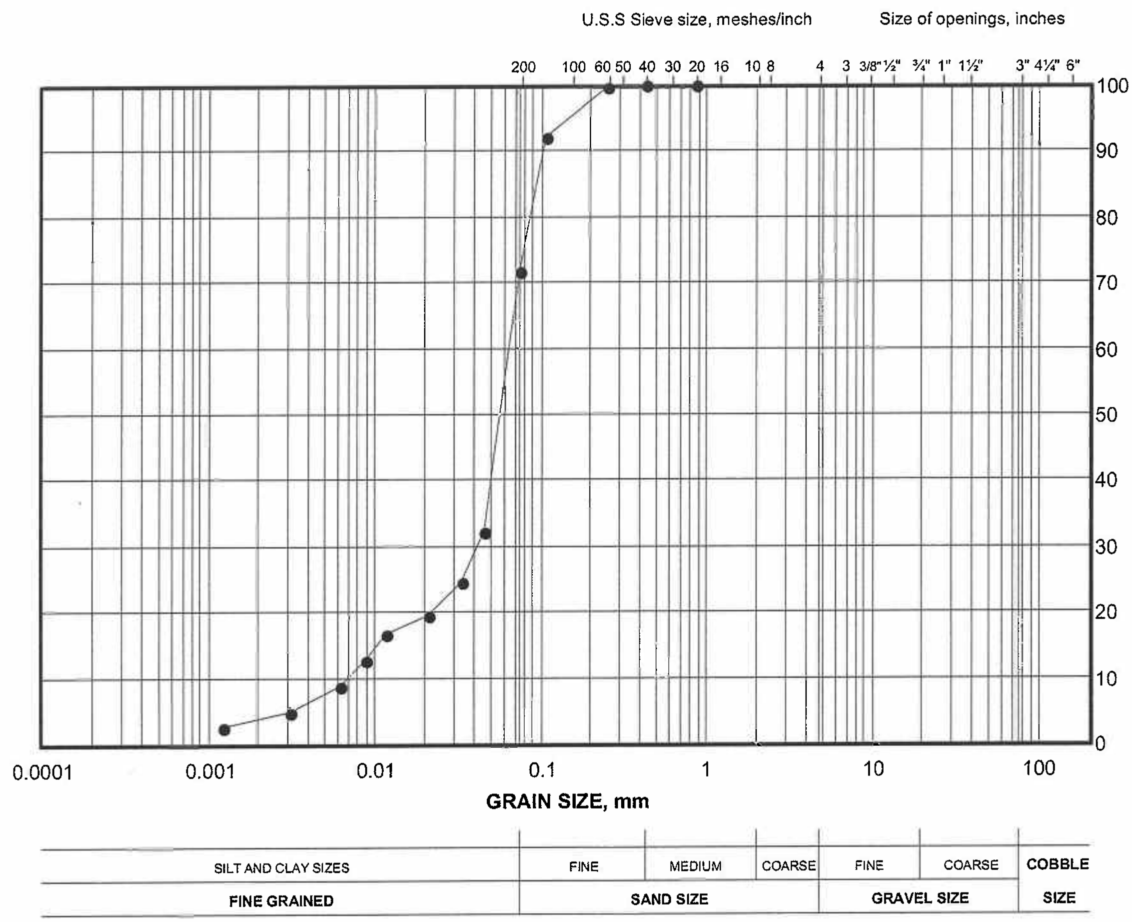


Ministry of Transportation Ontario	**PLASTICITY CHART** **Sand and Silt Till**	Figure No. B6 Project No. 06-1111-012-8 Checked By: SM

GRAIN SIZE DISTRIBUTION TEST RESULT

Sandy Silt

FIGURE B7



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	S27	17	176.4

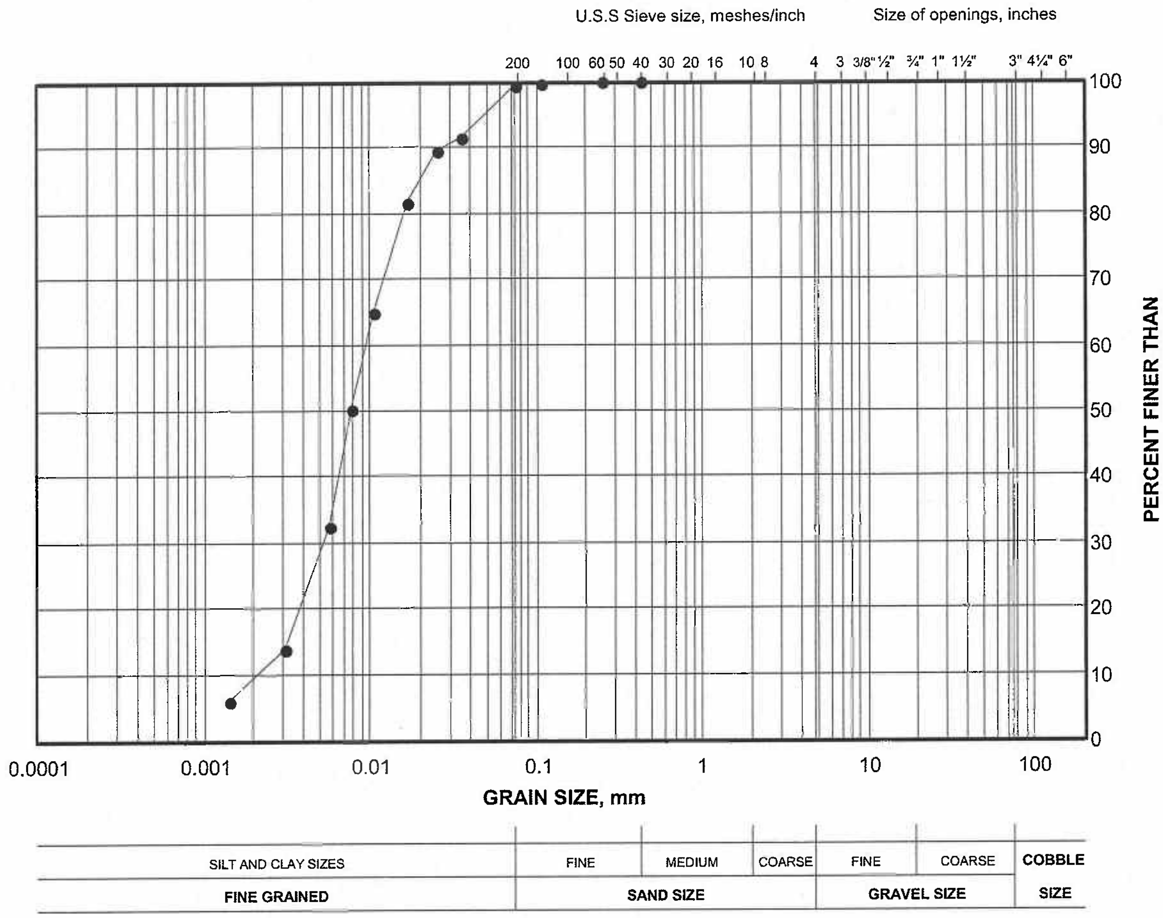
Project Number: 06-1111-012-8

Checked By: SM

Golder Associates

Date: 16-Jun-09

GRAIN SIZE DISTRIBUTION TEST RESULT Silt	FIGURE B8



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	S27	19	170.3

Project Number: 06-1111-012-8

Checked By: SM

Golder Associates

Date: 16-Jun-09

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.

Africa	+ 27 11 254 4800
Asia	+ 852 2562 3658
Australasia	+ 61 3 8862 3500
Europe	+ 356 21 42 30 20
North America	+ 1 800 275 3281
South America	+ 55 21 3095 9500

solutions@golder.com
www.golder.com

Golder Associates Ltd.
2390 Argentia Road
Mississauga, Ontario, L5N 5Z7
Canada
T: +1 (905) 567 4444