AUGUST 2009

PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT

RUTHERFORD 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:
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REPORT

GEOCRES No: 30M13-171

Report Number: 06-1111-012-6

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PRELIMINARY FOUNDATION INVESTIGATION REPORT

RUTHERFORD 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 Rutherford Road, 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 Rutherford Road overpass structure is located approximately 800 m east of Huntington Road and 200 m west of McGillivray Road in the City of Vaughan, Ontario (see Drawing 1). The proposed structure site is north of Langstaff Road.

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

The proposed overpass structures and associated approach embankments are to be situated within agricultural property located north and south of Rutherford Road. Rutherford Road generally slopes downwards from west to east. A hydro corridor running in a north-south direction is located west of the proposed structure site. The ground surface at the site typically varies from about Elevation 194.0 m to 195.0 m.

3.0 INVESTIGATION PROCEDURES

The field work for the Rutherford Road overpass structure investigation was carried out in March, 2009 during which time a total of four boreholes were advanced. The boreholes, designated as Boreholes S15 to S18, were advanced at the locations shown on Drawing 1.

The field investigation for the boreholes was carried out using a track-mounted CME 55, drill rig 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).

Boreholes S15 and S17 were drilled to a depth of approximately 19 m below existing ground surface. Borehole S16 was advanced to a depth of 33.6 m, and Borehole S18 was advanced to a depth of 32.1 m. The boreholes were terminated after penetrating at least 3 m into hard or very dense soil having Standard Penetration Test (SPT) 'N' values of greater than 100 blows per 0.3 m of penetration or when the borehole encountered shale bedrock.

The groundwater conditions in the open boreholes were observed during the drilling operations and a standpipe piezometer was installed in Boreholes S17 to permit monitoring of the groundwater level at the site. The piezometer consisted of 51 mm diameter PVC pipe, with a slotted screen sealed at a select depth within the borehole. A sand filter pack surrounds the screen and above the screen the borehole and annulus surrounding the piezometer pipe were backfilled to the surface with bentonite pellets/grout. The piezometer installation details and water level readings are described on the Record of Borehole Sheets in Appendix A. All 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 content, 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 (GPS) unit. The as-drilled borehole locations and ground surface elevations were surveyed by MRC. The borehole locations shown on Drawing 1 and on the borehole records are given relative to MTM NAD 83 northing and easting coordinates, and the ground surface elevations are referenced to Geodetic datum.

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

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

4.2 Subsurface Conditions

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

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[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.

The interpreted stratigraphic conditions along the Highway 427 NBL and SBL mainline alignment at the Rutherford overpass structures are shown on Drawings 1 and 2. These stratigraphic profiles represent a simplification of the subsurface conditions as encountered in the boreholes. Variation in the stratigraphic boundaries and properties of the soil deposits will occur between and beyond the borehole locations.

In general, the subsurface conditions in the area of the proposed overpass structures consist of a surficial layer of topsoil and up to about 0.8 m of surficial clayey silt in Boreholes S15 and S16, up to 0.8 m of sand and gravel fill in Boreholes S17 and S18. The surficial clayey silt and fill are underlain by a till deposit that grades from a clayey silt / silty clay to sand and silt and then to clayey silt. In Boreholes S16 and S18 the till deposit is underlain by a silt deposit, which in turn is underlain by shale bedrock at Borehole S18. In Borehole S16 there is a layer of clayey silt till between the silt and the bedrock.

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

4.2.1 Topsoil

Approximately 0.1 m of topsoil was encountered immediately below ground surface in Boreholes S18. This borehole was located north of the shoulder, north of Rutherford Road.

4.2.2 Fill

Fill consisting of silty sand to sand and gravel was encountered underlying the topsoil in Borehole S18 and immediately below ground surface in Borehole S17. The fill extended to a depth of about 0.8 m (between Elevation 193.5 m and 193.8 m).

4.2.3 Surficial Clayey Silt

Underlying the fill in Borehole S17 and immediately below the ground surface in the Boreholes S15 and S16, a surficial clayey silt deposit was encountered. This deposit extended to depths of between 0.6 and 1.5 m below ground surface (between Elevation 193.2 m and 194.0 m). The surficial clayey silt contains trace to some sand, trace gravel and contains rootlets and organics. On the borehole records in Appendix A, the surficial clayey silt 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 clayey silt deposit were 6, 7 and 20 blows per 0.3 m of penetration, indicating that the clayey silt has a firm to very stiff consistency. A measured water content on a sample of the surficial clayey silt was 23 percent.

4.2.4 Surficial Sand

Underlying the fill in Borehole S18 a layer of sand containing some gravel, trace silt and trace clay was encountered at 0.8 m below existing grade. The layer of sand was approximately 1.0 m thick and the base of the sand extended to Elevation 192.5 m. Measured SPT ‘N’ values in the sand were 10 and 36 blows per 0.3 m of penetration, indicating that the sand has a compact to dense relative density. Measured water contents on two samples of sand were 4 and 10 percent.

4.2.5 Till Deposit

In all boreholes drilled at this site the topsoil or surficial clayey silt and surficial sand deposits are underlain by a clayey silt till deposit that grades with depth to a cohesionless till deposit. In Boreholes S16, S17 and S18 the cohesionless till deposit grades with depth to a cohesive till deposit.

Till deposits in southern Ontario typically contain cobbles and/or boulders. Although there was no evidence of cobbles and/or boulders during drilling, cobbles and / or boulders should be expected within the till deposit.

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

The upper cohesive till extends to depths of between 11.6 m and 13.4 m; the base of the cohesive till was encountered in the boreholes between approximately Elevation 181.2 m and 183.0 m. The upper cohesive till predominately consists of clayey silt with some sand and trace gravel; however in Borehole S17 the upper cohesive till consists of silty clay with some sand and trace gravel to a depth of 7.3 m (Elevation 187.3 m).

Grain size distribution tests were carried out on six selected samples of the silty clay to clayey silt till deposit and the results are presented on Figure B1 in Appendix B. Atterberg limits testing was carried out on six samples of the clayey silt till and one sample of the silty clay till. The measured plastic limits of the clayey silt till varied from 11 to 17 percent, the liquid limits varied from 18 to 33 percent, and the plasticity indices varied from 6 to 16 percent. These results, which are plotted on a plasticity chart on Figure B2 in Appendix B, confirm that this portion of the till deposit is a clayey silt of low plasticity. The measured plastic limit of the silty clay till was 18 percent, the liquid limit was 40 percent and the plasticity index was 22 percent. This result is also plotted on Figure B2 and confirms that this portion of the till is a silty clay of medium plasticity. Measured water contents on samples of the clayey silt till ranged from about 7 to 26 percent.

The SPT ‘N’ values measured within the upper cohesive till deposit typically ranged from 12 to 49 blows per 0.3 m of penetration, indicating a stiff to hard consistency.

4.2.5.2 Sand and Silt Till (Cohesionless Till)

The upper clayey silt till grades with depth to a cohesionless till, the surface of which was encountered between Elevation 181.2 m and 183.0 m. The cohesionless till was found to have a thickness of approximately 0.9 m to 4.4 m. The base of the cohesionless till was encountered in the boreholes between Elevations 177.7 m and 180.9 m.

The cohesionless portion of the till consists of sand and silt and contains trace to some gravel and trace clay. The results of grain size distribution tests completed on three selected samples of the sand and silt till is provided on Figure B3 in Appendix B. Atterberg limit testing was carried out on one sample of the sand and silt till and measured a plastic limit 12 percent, a liquid limit of 15 percent and a plasticity index of 2 percent. These results, which are plotted on a plasticity chart on Figure B4 in Appendix B, confirms that this material is a sand and silt till that is non-plastic or has low plasticity. Measured water contents on samples of the sand and silt till ranged from about 4 to 12 percent.

Within the cohesionless till the SPT ‘N’ values typically ranged from 31 to greater than 100 blows per 0.3 m of penetration, indicative of sand and silt till with a dense to very dense relative density.

4.2.5.3 Clayey Silt Till (Lower Cohesive Till)

As discussed above, the cohesionless till grades with depth to a cohesive till in Boreholes S16, S17 and S18. The cohesive till was encountered at depths between 13.4 m and 18.0 m (between Elevation 176.0 m and 180.9 m). Borehole S15 terminated within the clayey silt till deposit at a depth of 18.9 m (Elevation 175.1 m); however Boreholes S16, S17 and S18 fully penetrated the cohesive till deposit, which was found to have thicknesses of approximately 3.3 m to 4.6 m. The base of the cohesive till was encountered in the boreholes at between Elevation 175.1 m and 176.5 m, although the deposit base may be lower or higher than this in Borehole S15 where it was not fully penetrated. The cohesive till consists of clayey silt and contains trace to some sand and trace gravel. In Borehole S16 a lower cohesive till deposit was encountered underlying the silt deposit (see Section 4.2.6 for details) at a depth of 27.1 m (Elevation 167.5 m). The lower cohesive till consists of clayey silt and contains some sand and gravel and extends to about Elevation 165.0 m and overlies the shale bedrock.

Atterberg limits testing was carried out on four samples of the clayey silt till deposit. The measured plastic limits of the clayey silt till varied from 11 to 15 percent, the liquid limits varied from 21 to 30 percent, and the plasticity indices varied from 10 to 15 percent. These results, which are plotted on a plasticity chart on Figure B2 in Appendix B, confirm that this portion of the till deposit is a clayey silt of low plasticity. Measured water contents on samples of the clayey silt till ranged from about 9 to 22 percent.

Atterberg limit testing was carried out on one sample of the lower clayey silt till and measured a plastic limit of 12 percent, a liquid limit of 18 percent and a plasticity index of 6 percent. These results, which are plotted on a plasticity chart on Figure B2 in Appendix B, confirms that this portion of the till deposit is a clayey silt of low plasticity. A measured water content on a sample of the clayey silt till was 7 percent.

The SPT ‘N’ values measured within the cohesive till that underlies the cohesionless till, varied from 63 to greater than 100 blows per 0.3 m of penetration, indicative of hard consistency. The exception to this was a measured SPT ‘N’ value of 17 blows per 0.3 m of penetration at a depth of 14 m (Elevation 180.3 m) in Borehole S18, indicating that the clayey silt till at that depth has a very stiff consistency.

4.2.6 Silt

A deposit of silt was encountered in only Boreholes S16 and S18.. The surface of the silt deposit was encountered in Boreholes S16 and S18 at depths of 19.5 m and 17.8 m, (Elevations 175.1 m and 176.5 m), respectively. In Borehole S16 the silt deposit was 7.6 m thick and extended to Elevation 167.5 m. In Borehole S18 the base of the silt deposit was encountered at Elevation 165.7 m; corresponding to a thickness of 10.9 m. The silt deposit in Borehole S18 directly overlies the shale bedrock, whereas in Borehole S16 there is a 2.5 m thick layer of lower clayey silt till between the base of the silt and the surface of the shale bedrock.

The silt deposit contains trace to some clay. Grain size analyses were carried out on three selected samples of the silt deposit and are provided on Figure B5 in Appendix B. Atterberg limit testing was completed on three samples of the silt deposit. The measured plastic limits varied from 13 to 20 percent, the liquid limits varied from 22 to 25 percent, and the plasticity indices varied from 3 to 4 percent. These results, which are plotted on a plasticity chart on Figure B6 in Appendix B, confirm that this material is a silt that is non-plastic or has low plasticity. Measured water contents on samples of the silt deposit ranged from about 19 percent to 23 percent.

Measured SPT ‘N’ values typically ranged from 20 to greater than 100 blows per 0.3 m of penetration, indicative of a silt deposit with a compact to very dense relative density.

4.2.7 Clayey Silt

Underlying the sand and silt till in Borehole S15 a layer of clayey silt encountered between 16.3 m and 18.0 m depth (Elevation 177.7 m and 176.0 m).

The clayey silt contains trace sand. Grain size distribution test was carried out on one selected sample of the clayey silt deposit and the result is presented on Figure B7 in Appendix B. Atterberg limits testing was carried out on one sample of the clayey silt deposit and measured a plastic limit of 15 percent, a liquid limit of 24 percent, and a plasticity index of 8 percent. These results, which are plotted on a plasticity chart on Figure B8 in Appendix B, confirms that this material is a clayey silt of low plasticity. A measured water content on a sample of the clayey silt was 16 percent.

Within the clayey silt layer SPT ‘N’ values were greater than 100 blows per 0.3 m of penetration, indicative of a hard consistency.

4.3 Shale Bedrock

Bedrock was encountered and split spoon samples were recovered from Boreholes S16 and S18.. The depth of the surface of the bedrock was encountered at the following depths and elevations:

Borehole No.	Depth to Bedrock Surface	Bedrock Surface Elevation
S16	29.6 m	165.0 m
S18	28.7 m	165.7 m

The bedrock samples consisted of light grey to dark grey shale. Based on available bedrock geology maps, the bedrock at this site is understood to be part of the Georgian Bay Formation.

4.4 Groundwater Conditions

The water level in the boreholes as noted during and upon completion of drilling operations was between about Elevation 183.4 m and Elevation 188.6 m (at a depths of between 6.0 m and 11.2 m) in the four boreholes drilled for this site, although the level had not yet stabilized. In general, the clayey silt till samples taken in the boreholes drilled were noted to be moist, the sand and silt till samples were wet and the silt samples were moist.

A standpipe piezometer was installed in Borehole S17 to permit monitoring of the groundwater level at this site. Details of the piezometer installations are shown the borehole records in Appendix A. The groundwater level measured in the piezometer installation, some eight weeks following borehole completion, is summarised below:

Borehole No.	Ground Surface Elevation	Depth to Groundwater	Groundwater Elevation	Date of Measurement
S17	194.6 m	4.0 m	190.6 m	April 24, 2009
		4.1 m	190.5 m	May 25, 2009
		4.0 m	190.5 m	June 15, 2009
		3.8 m	190.7 m	July 9, 2009

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

5.0 CLOSURE

The field technicians directing the drilling program were Messrs. Suresh Bainey and Jordan Black. This report was prepared by Ms. Sandra McGaghran, P.Eng. with input from Ms. Lisa Coyne, P.Eng. a geotechnical engineer and Associate with Golder Associates and Mr. Fin Heffernan, P.Eng., Golder's Designated MTO Contact for this project, conducted an independent quality control review of the report.

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PART B

PRELIMINARY FOUNDATION DESIGN REPORT

RUTHERFORD ROAD OVERPASSES

HIGHWAY 427 EXTENSION

FROM HIGHWAY 7 TO MAJOR MACKENZIE DRIVE

W.O. 05-20012

6.0 ENGINEERING RECOMMENDATIONS FOR PRELIMINARY DESIGN

This section of the report provides foundation design recommendations for the preliminary design of the proposed Rutherford Road overpass structures on the Highway 427 Extension 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 Rutherford Road overpasses are proposed to consist of two-span structures with centre piers in the median of Rutherford Road. Based on the preliminary General Arrangement (GA) Drawing provided by MRC on May 15, 2009, the span length between each abutment and the pier is approximately 33 m.

According to the preliminary GA Drawing, the finished grade of Highway 427 NBL and SBL over Rutherford Road will be at approximately Elevation 202.5 m, which is approximately 8.5 m above the proposed Rutherford Road grade. Therefore, the north and south approach embankments will be about 8.5 m high relative to the existing ground surface.

6.2 Foundation Recommendations

6.2.1 Foundation Options

Based on the proposed vertical elevations and subsurface soil conditions, the following foundation options are considered feasible for the Rutherford Road overpass:

- **Spread footings founded on the very stiff to hard silty clay to clayey silt till**: This option is feasible at the piers; where the footings would have to extend below any “reworked” or surficial clayey silt or surficial sand to be founded on the very stiff to hard silty clay to clayey silt till. The very stiff to hard till was encountered at depths of between 1.5 m to 1.8 m in the boreholes in vicinity of the proposed piers. Considering that the grade at the north and south abutments are to be raised by about 8.5 m, this option is may not be economical at the abutments given the resulting height of abutment walls.

- **Spread footings “perched” on a granular pad within the approach embankment fill**: This option could be adopted to support the abutments for an open structure, with 2 horizontal to 1 vertical (2H:1V) foreslopes in front of the abutment footings. In order to minimize potential settlements, it would be necessary to subexcavate the upper 0.8 m of reworked surficial clayey silt and the surficial sand which extended to a depth of about 1.5 m to expose the very stiff to hard silty clay to clayey silt till at the south abutments, prior to construction of the new approach embankments. Although boreholes were not drilled at the north abutments, it is anticipated that similar soil conditions will be encountered as the north abutments are located within an agricultural field.

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

- **Caissons founded within the glacial till deposit:** This option could be adopted to support the abutments 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 would require only minor additional subexcavation of about 0.4 m below the frost depth in order to found the spread footings on very stiff to hard silty clay to clayey silt till, and these are therefore preferred if sufficient geotechnical resistance can be achieved; otherwise, support of the piers on deep foundations will be required to achieve a higher capacity. The use of piles is preferred from a foundations perspective over caissons for support of the abutments and piers, as the caissons would extend through the water-bearing sand and silt till, which would be susceptible to disturbance and which would require special construction procedures. Higher capacities can be achieved by driving the piles to bedrock, however considering multiple construction techniques may be required this option may not be considered practical or economical.

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

6.2.2 Spread Footings on Native Soils

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

6.2.2.1 Founding Elevations

The abutments and piers may be supported on spread footings placed below the upper firm to stiff clayey silt, on very stiff to hard clayey silt till (depth varies from approximately 0.8 m at the south abutments for the NBL and SBL overpass to between 1.5 m and 1.8 m at the piers). A minimum founding depth of 1.4 m is required for frost protection purposes (OPSD 3090.101). Preliminary recommendations for minimum (highest) founding depths are provided in the following table, based on both frost protection and subexcavation requirements; these depths are given relative to lowest surrounding grade.

Foundation Element	Borehole	Founding Stratum	Depth
South Abutment SBL Overpass	S15	Very Stiff Clayey Silt Till	1.4 m depth
South Abutment NBL Overpass	S16	Very Stiff Clayey Silt Till	1.4 m depth
Pier SBL Overpass	S17	Very Stiff to Hard Silt Clay Till	1.5 m depth
Pier NBL Overpass	S18	Very Stiff to Hard Clayey Silt Till	1.8 m depth

6.2.2.2 Geotechnical Resistances

A factored geotechnical resistance at Ultimate Limit States (ULS) of 450 kPa and a geotechnical resistance at Serviceability Limit States (SLS) of 300 kPa (for 25 mm of settlement) may be used for preliminary design purposes, assuming 3 m wide footings.

The ULS and SLS resistances and settlement are dependent on the footing size, configuration and applied loads. The geotechnical resistances should, therefore, be reviewed during detail design, once further drilling has been carried out at the foundation elements to 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 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 very stiff to hard 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 overpass 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 clayey silt material (based on Boreholes S15 and S16) that is present within the embankment footprint below the perched abutment, to minimize settlement due to the embankment loading. It is expected that subexcavation of the upper 0.8 m of soil would be required at the abutments. Although boreholes were not drilled at the north abutments, considering they are within a field, it is anticipated that subexcavation of the upper 0.8 m will also be required at the north abutments to remove the soil disturbed by agricultural activities. The area to be subexcavated should be defined by a line extending from the toe of the OPSS 1010 Granular 'A' pad, outward and downward at 1 horizontal to 1 vertical (1H:1V). The subexcavation should be replaced with compacted OPSS 1010 Granular 'B'. The Granular 'A' pad should be a minimum of 2 m thick and should extend at least 1 m beyond the plan limits of the footing. The Granular 'A' pad should be constructed in accordance with MTO Special Provision SP105S10.

6.2.3.2 Geotechnical Resistances

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

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

6.2.3.3 Resistances to Lateral Loads

The resistance to lateral forces/sliding resistance between the concrete footings and the compacted Granular 'A' pad should be calculated in accordance with Section 6.7.5 of the *CHBDC*. The coefficient of friction, tan 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 driven to found within the hard clayey till deposits or hard clayey silt are provided in the subsections that follow.

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

6.2.4.1 Founding Elevations

Steel H-piles driven to found within the hard clayey silt till at Boreholes S16, S17 and S18 and within the hard clayey silt deposit in Borehole S15, may be used for support of the abutments and piers. "Refusal" (i.e. soil having SPT 'N' values greater than 100 blows per 0.3 m of penetration) was encountered in the boreholes between approximately Elevation 178.6 m and 180.0 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 Overpass	S15	Hard Clayey Silt	177.1 m
South Abutment NBL Overpass	S16	Hard Clayey Silt Till	178.0 m
Pier SBL Overpass	S17	Hard Clayey Silt Till	178.5 m
Pier NBL Overpass	S18	Hard Clayey Silt Till	178.0 m

The till deposit encountered in the boreholes for this structure site are underlain by very dense silt that becomes compact with depth. The thickness of the clayey silt till deposit having SPT 'N' values greater than 100 blows per 0.3 m of penetration varied from about 2 m in Borehole S18 to about 5 m in Boreholes S17. It is preferable to terminate the piles a bit shallower than the conventional 1.5 m into the refusal material so that the pile isn't bearing on the less competent silt. It is recommended at detail design stage that the sampling interval within the till deposit be reduced in order to more accurately define the thickness of the clayey silt till having a thickness of greater than 100 blows per 0.3 m penetration deposit. Depending on the thickness of the soil having SPT 'N' values greater than 100 blows per 0.3 m of penetration in the boreholes at the detail design stage consideration may be given to reduced geotechnical resistances for piles and/or abutments where the this material is thinner.

6.2.4.2 Geotechnical Axial Resistances

The proposed abutments and piers can be supported on steel H-piles driven to found within the hard clayey silt and hard clayey silt till. For HP 310x110 piles driven about 1.5 m below the surface of the soil having SPT 'N' values greater than 100 blows per 0.3 m of penetration to the estimated tip elevations provided in Section 6.2.4.1

Founding Material	Foundation Unit	Factored Geotechnical Resistance at ULS	Geotechnical Resistance at SLS
Hard Clayey Silt Till and Hard Clayey Silt	Piers	1,400 kN	1,200 kN
	Abutments	1,700 kN	1,400 kN

At the proposed north and south abutment area it is estimated that up to about 25 mm of settlement will occur, under the proposed loading from the approach embankment. For preliminary design purposes it is recommended that a downdrag load of 100 kN be included, although further investigation and assessment will be required during detail design stage. The structural capacity of the piles must be checked for the factored dead and downdrag loads in accordance with Section 6.8.4 of the *CHBDC*.

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

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

6.2.4.3 Resistances to Lateral Loads

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

The resistance to lateral loading in front of the pile, as well as pile group action for lateral loading if the pile spacing in the direction of loading is less than six to eight pile diameters, should be accounted for and assessed during the detail design phase of the project. For preliminary design, a factored lateral geotechnical resistance at ULS of 200 kN may be used and a lateral geotechnical resistance at SLS of 110 kN (for 10 mm of lateral displacement at the pile cap level) may be used for a single vertical HP 310x110 pile embedded in hard 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 hard clayey silt till having SPT 'N' values greater than 100 blows per 0.3 m of penetration for support of the foundation elements for the overpasses. 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 near 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 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
Piers	Hard Clayey Silt Till	0.9 m	3,000 kN	2,500 kN
		1.2 m	5,300 kN	4,400 kN
		1.5 m	8,300 kN	6,900 kN
Abutments	Hard Clayey Silt / Hard Clayey Silt Till	0.9 m	4,300 kN	3,600 kN
		1.2 m	7,800 kN	6,600 kN
		1.5 m	11,500 kN	10,000 kN

6.2.5.3 Resistances to Lateral Loads

For preliminary design purposes, a maximum factored lateral resistances at ULS of 400 kN and a maximum lateral resistances at SLS (for 10 mm of horizontal deflection at pile cap level) of 250 kN are recommended for 0.9 m diameter caissons, based on the "Assessed Horizontal Passive Resistance and Geotechnical Reaction at SLS" provided under Clause C6.8.7.1, Table C6.4 of the *Commentary on* CHBDC and correlation with lateral pile load test. 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 Rutherford Road overpass structure will require placement of up to about 8.5 m of fill within the limits of the north and south approach embankments.

Based on the results of the boreholes drilled at this site, the approach embankments will be founded on firm to very stiff surficial clayey silt, underlain by very stiff to hard silty clay to clayey silt till.

6.4.1 Subgrade Preparation and Embankment Construction

The existing native subsoils are considered to be an appropriate subgrade for the proposed approach embankments; however, to improve the embankment performance, it is recommended that prior to the placement of any fill, all topsoil, organic matter and existing fill and any softened/loosened native soils should be stripped from below the approach embankment areas. Embankment fill should be placed and compacted in accordance with MTO's SP 206S03 and SP 105S10. 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 adequate for the embankments at this site.

The soil parameters used in the analysis, as given in the following table, were estimated from empirical correlations using the results of in situ Standard Penetration Tests (SPT) and geotechnical classification testing. The groundwater table was taken at Elevation 190.6 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
Firm to Very Stiff Surficial Clayey Silt	19	50 kPa	--	28
Stiff to Hard Clayey Silt Till	21	100 kPa	--	34
Very Dense Sand and Silt Till	21	--	--	34

With appropriate subgrade preparation and proper placement and compaction of embankment fill materials, the proposed 8.5 m high approach embankments with side slopes maintained at 2H:1V will have a factor of safety of greater than 1.3 against deep-seated slope instability. The results of an example static stability analysis are provided on Figure 2.

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

6.4.3 Approach Embankment Settlement

Settlement of the approach embankments at the site will occur due to compression of the new embankment fill itself, as well as compression of the underlying native soils. Provided that the embankment material consists of clean earth fill or granular fill, the settlement of the 8.5 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 approximately 25 mm; the majority of this settlement will occur during or immediately following construction of the approach embankments. This compression has been estimated using the elastic deformation moduli given in the table below, based on correlations with the measured SPT 'N' values. For the firm portion of the surficial clayey silt, where present, consolidation parameters have been estimated based on correlation with Atterberg limits and experience with similar soil types in the Peel Plain.

Soil Deposit	Bulk Unit Weight	Elastic Modulus	Consolidation Parameters
Embankment fill (range of parameters assumed for earth fill and granular fill)	20 – 22 kN/m^3	--	--
Very Stiff to Hard Clayey Silt Till	21 kN/m^3	75 MPa	--
Very Dense Sand and Silt Till	21 kN/m^3	150 MPa	--

6.5 Detail Design and Construction Considerations

6.5.1 Additional Investigation Requirements

As noted previously, additional borehole investigation, laboratory testing and analysis will be required during detail design, once the layout of the proposed 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, in Borehole S18, which was drilled between the pier and the north abutment for the NBL, the soil having SPT 'N' values of greater than 100 blows was only about 2 m thick and is underlain by silt where the relative density decreases with depth from very dense to compact. There is concern with founding piles driven into this material at foundation units in vicinity of Borehole S18. At detail design stage it is recommended that further investigation be completed to determine the thickness of the this material by obtaining samples at 0.75 m interval within the lower portion of the till deposit until the silt is encountered.

6.5.2 Excavation

Depending on the foundation option adopted, excavations for the overpass foundations are expected to extend to depths of up to 1.8 m below existing ground surface and will be made through compact to dense sand/firm to very stiff clayey silt and into very stiff to hard clayey silt till, which are considered Type 3 soil according to Occupational Health and Safety Act and Regulation for Construction Projects (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 4.0 m below ground surface. It is expected that excavations for the piers and north abutment foundations will be above the groundwater level. Some water inflow into the excavation should be expected perched in the fill; however, it is anticipated that water inflow can be handled by pumping from filtered sump pumps placed at the base of the excavation.

6.5.4 Subgrade Preparation

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

6.5.5 Obstructions During Pile Driving / Caisson Installation

It is anticipated that cobbles and/or boulders will be encountered within the till deposits, as noted in several boreholes at this site, and may affect the installation of steel H-piles and/or caissons. It is recommended that flange plate reinforcement or driving shoes be used on all steel H-Piles to facilitate driving into the 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 in the contract documents 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., with input from Ms. Lisa Coyne, P.Eng. and Mr. Fintan Heffernan, P.Eng., Golder's Designated MTO Contact for this project, conducted an independent quality control review of the report.

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TABLE 1
COMPARISON OF FOUNDATION ALTERNATIVES
RUTHERFORD ROAD OVERPASS – HIGHWAY 427 (NBL AND SBL) EXTENSION W.O. 05-20012

Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Spread Footings on very stiff to hard clayey silt till	Feasible for support of piers	• Relative ease of construction; and, • Negligible post-construction settlement.	• Approximately between 0.8 m and 2.0 m sub-excavation required, • Any groundwater control required (can be controlled by pumping from sumps depending on the time of year); and, • Lowest bearing capacities of the four options.	• Lower relative cost than piled foundations; and, • Subexcavation of between 0.8 m and 2.05 m of fill and surficial soils required within footing footprint.	• Loosening of subgrade soil due to ponded water.
Spread Footings "perched" in Approach Embankment Fill	Feasible for support of abutments	• Negligible post-construction settlement.	• Footing subgrade will not be disturbed by groundwater.	• Subexcavation of 0.8 m of surficial soils required within footing footprint; and, • Low cost option	• Must ensure proper compaction of Granular 'A' pad to minimise post-construction settlement.
Steel H-pile Foundations driven to found within hard clayey silt till/hard clayey silt	Feasible for support of abutments and piers	• Sub-excavation is not required, • Higher bearing capacity, compared to spread footings • Negligible post-construction settlement; and, • Can be used for support of conventional or integral abutments.		• More costly than spread footings.	
Caissons Foundations founded on hard clayey silt till/hard clayey silt	Feasible at the piers and abutments	• Sub-excavation is not required • Highest bearing compared to piles driven to hard clayey silt till, • Negligible post-construction settlement; and, • Can be used for support of conventional or semi-integral abutments.	• Need for liners; and, • Cleaning of the base below the water table could be difficult.	• More costly option that Steel H-piles.	

MINISTRY OF TRANSPORTATION, ONTARIO

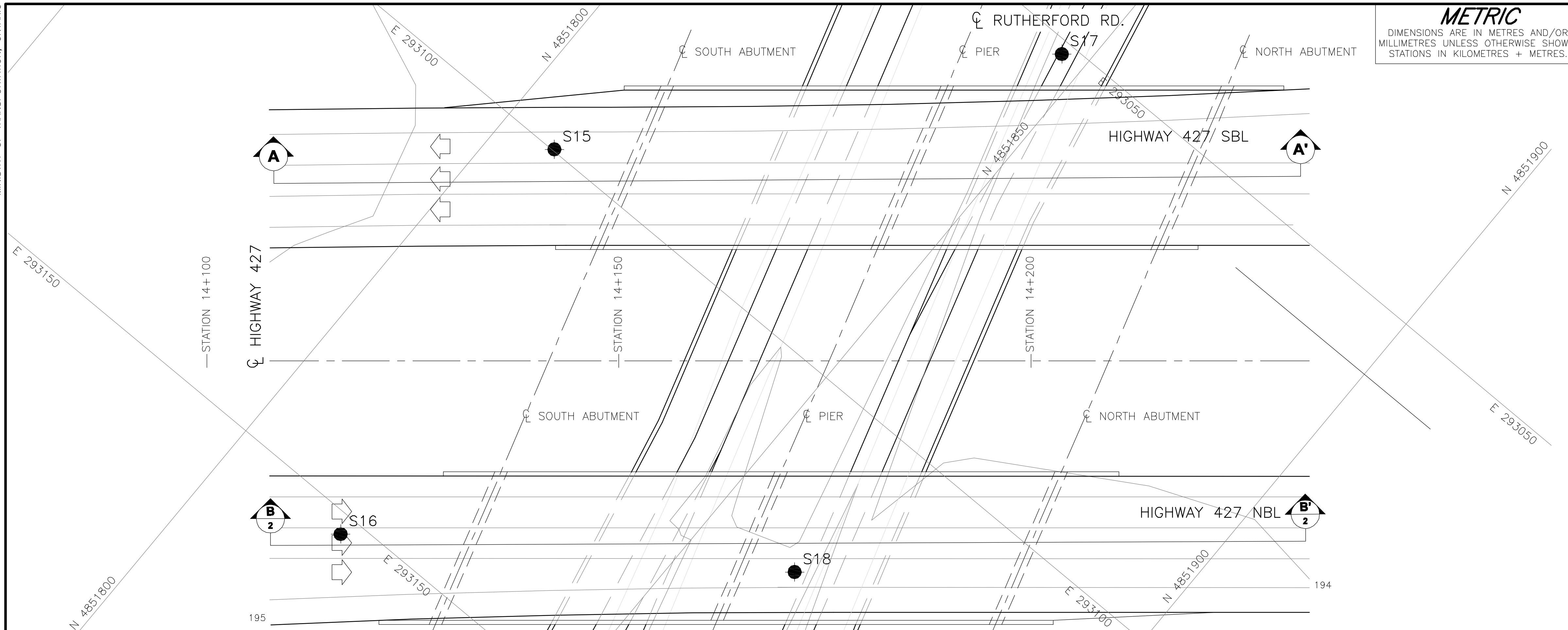
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.
WO No. 05–20012

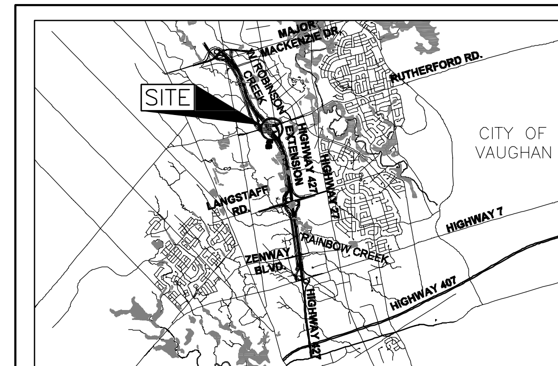
SHEET

HIGHWAY 427 EXTENSION
RUTHERFORD ROAD OVERPASS
BOREHOLE LOCATIONS AND SOIL STRATA

Golder Associates
Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



PLAN
SCALE
6 0 6 12 m



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

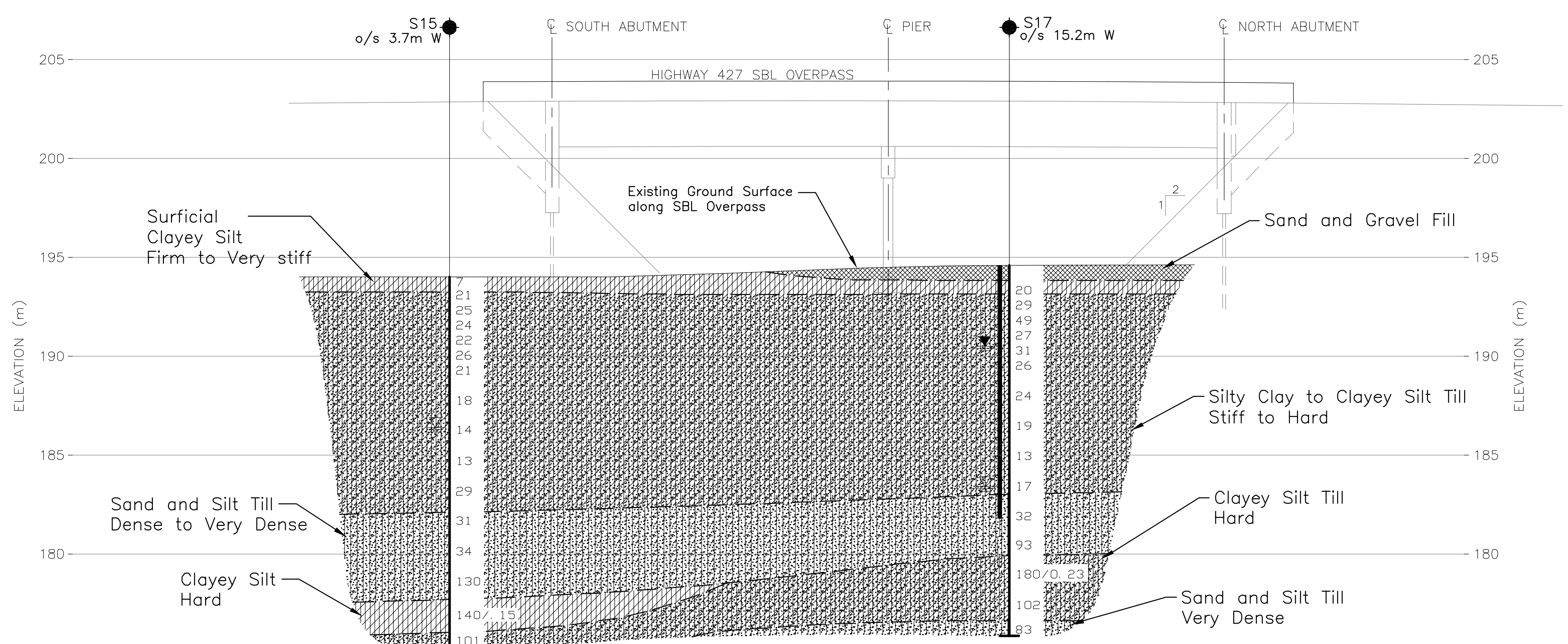
No.	ELEVATION	CO–ORDINATES	
		NORTHING	EASTING
S15	194.0	4851810.8	293098.7
S16	194.6	4851817.3	293152.1
S17	194.6	4851849.1	293063.4
S18	194.3	4851863.9	293113.0

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

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

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

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



PROFILE A–A' RUTHERFORD ROAD SBL OVERPASS

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

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

NO.	DATE	BY	REVISION

Geocres No. 30M13–171

HWY. 427	PROJECT NO. 06–1111–0012–6		DIST.
SUBM'D. JEB/CR	CHKD. SMM	DATE: 5–Aug–2009	SITE:
DRAWN: JFC/JM	CHKD. SMM	APPD. LCC	DWG. 1

PLOT DATE: August 5, 2009

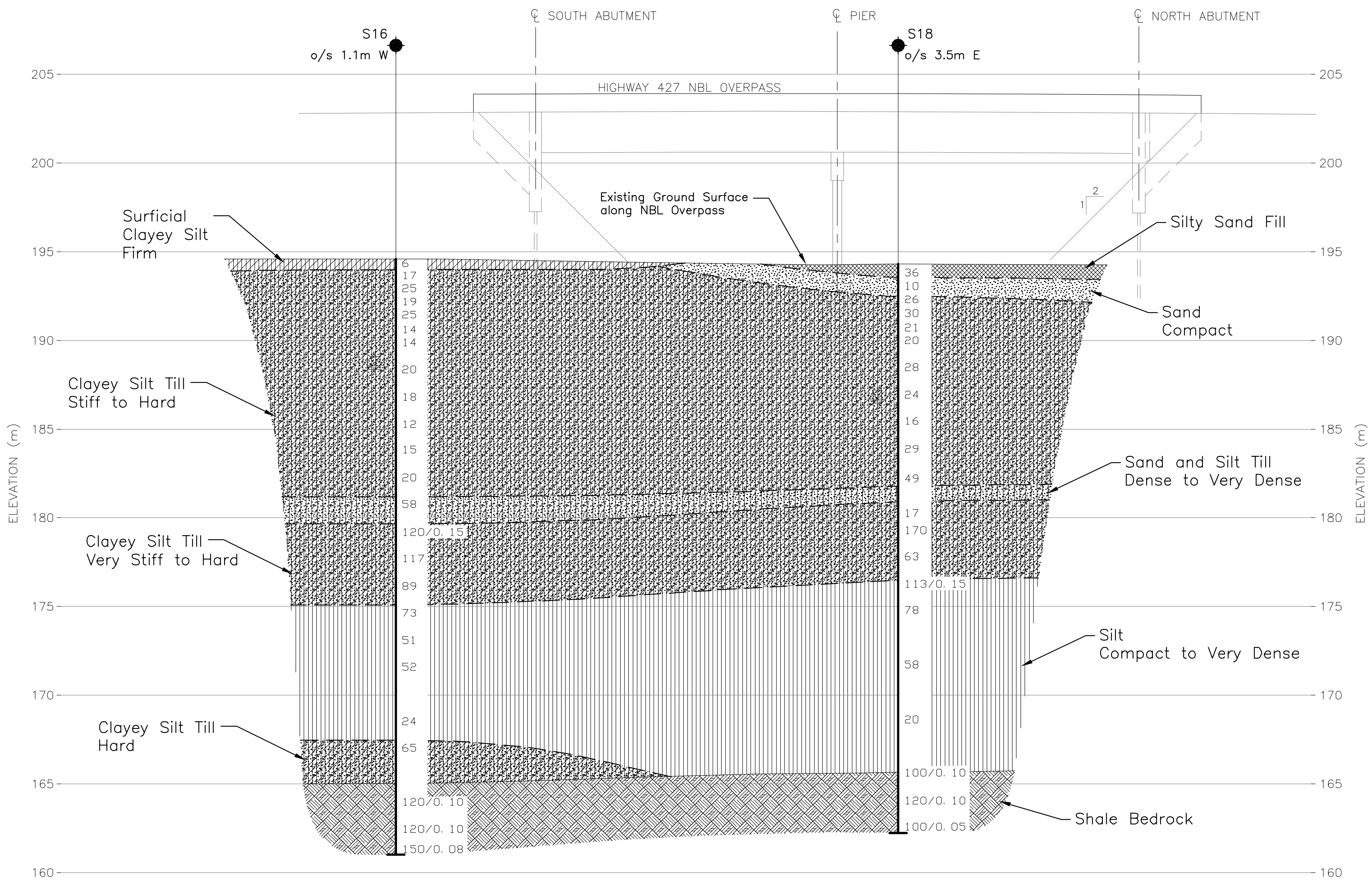
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.
WO No. 05–20012

SHEET

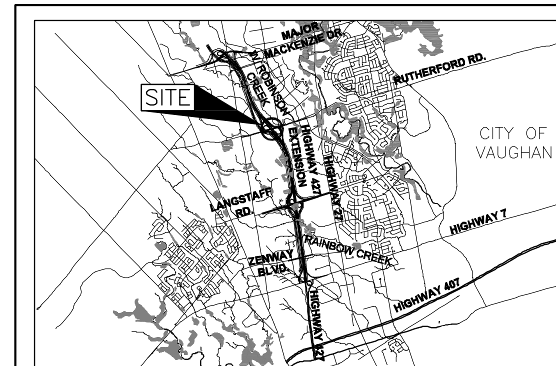
HIGHWAY 427 EXTENSION
RUTHERFORD ROAD OVERPASS
SOIL STRATA

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



PROFILE B–B' RUTHERFORD ROAD NBL 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
S15	194.0	4851810.8	293098.7
S16	194.6	4851817.3	293152.1
S17	194.6	4851849.1	293063.4
S18	194.3	4851863.9	293113.0

NOTES

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

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

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

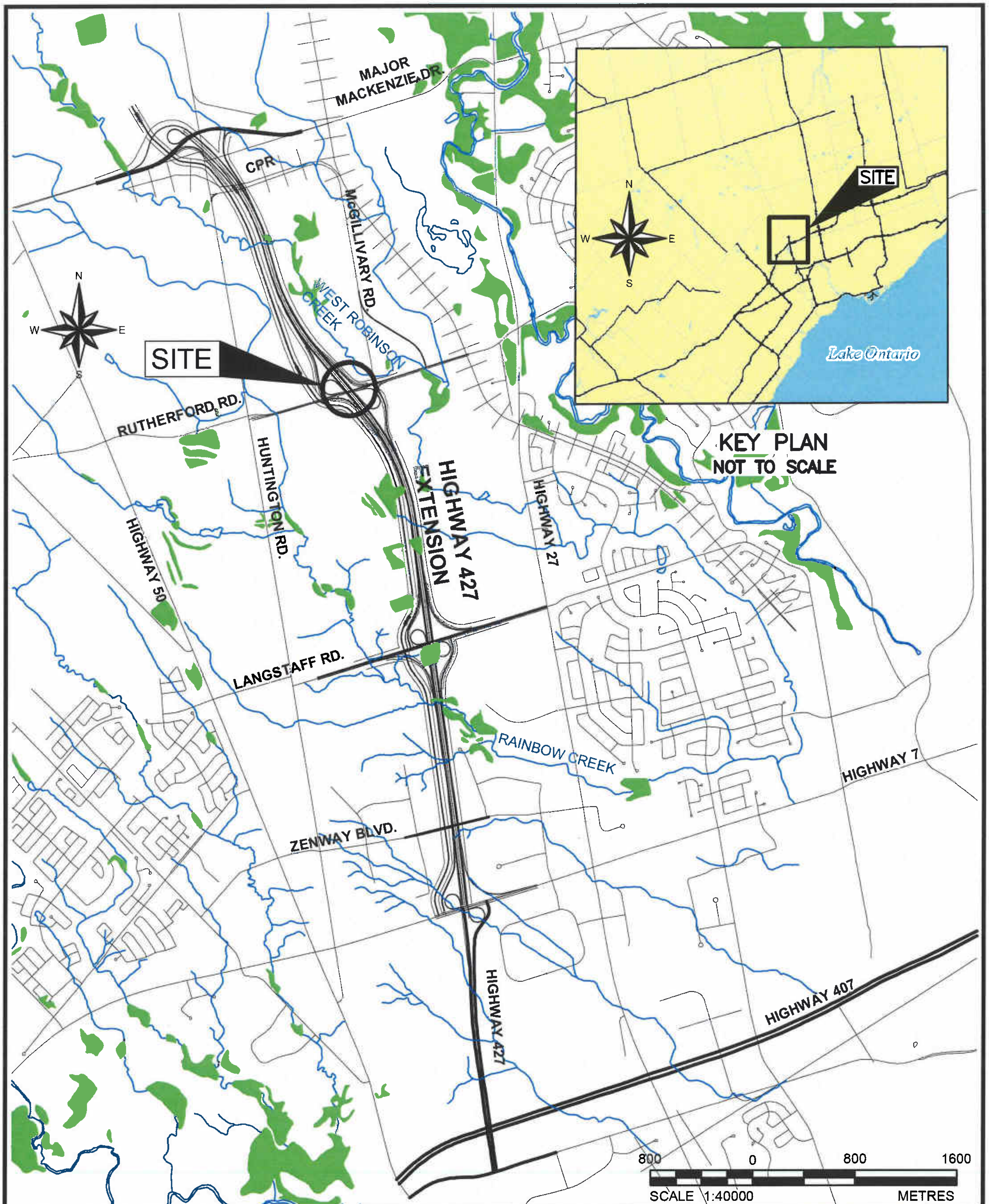
REFERENCE

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

NO.	DATE	BY	REVISION

Geocres No. 30M13–171

HWY. 427		PROJECT NO. 06–1111–0012–6	DIST.
SUBM'D. JEB/CR	CHKD. SMM	DATE: 5–Aug–2009	SITE:
DRAWN: JFC/JM	CHKD. SMM	APPD. LCC	DWG. 2



Golder Associates
Mississauga, Ontario, Canada

SCALE	AS SHOWN	TITLE	
DATE	Aug. 5, 2009	**SITE LOCATION PLAN RUTHERFORD ROAD OVERPASS**	
DESIGN	PKS		
CAD	JFC		
CHECK	JEB	HIGHWAY 427 EXTENSION	FIGURE 1
REVIEW SM	SMM		

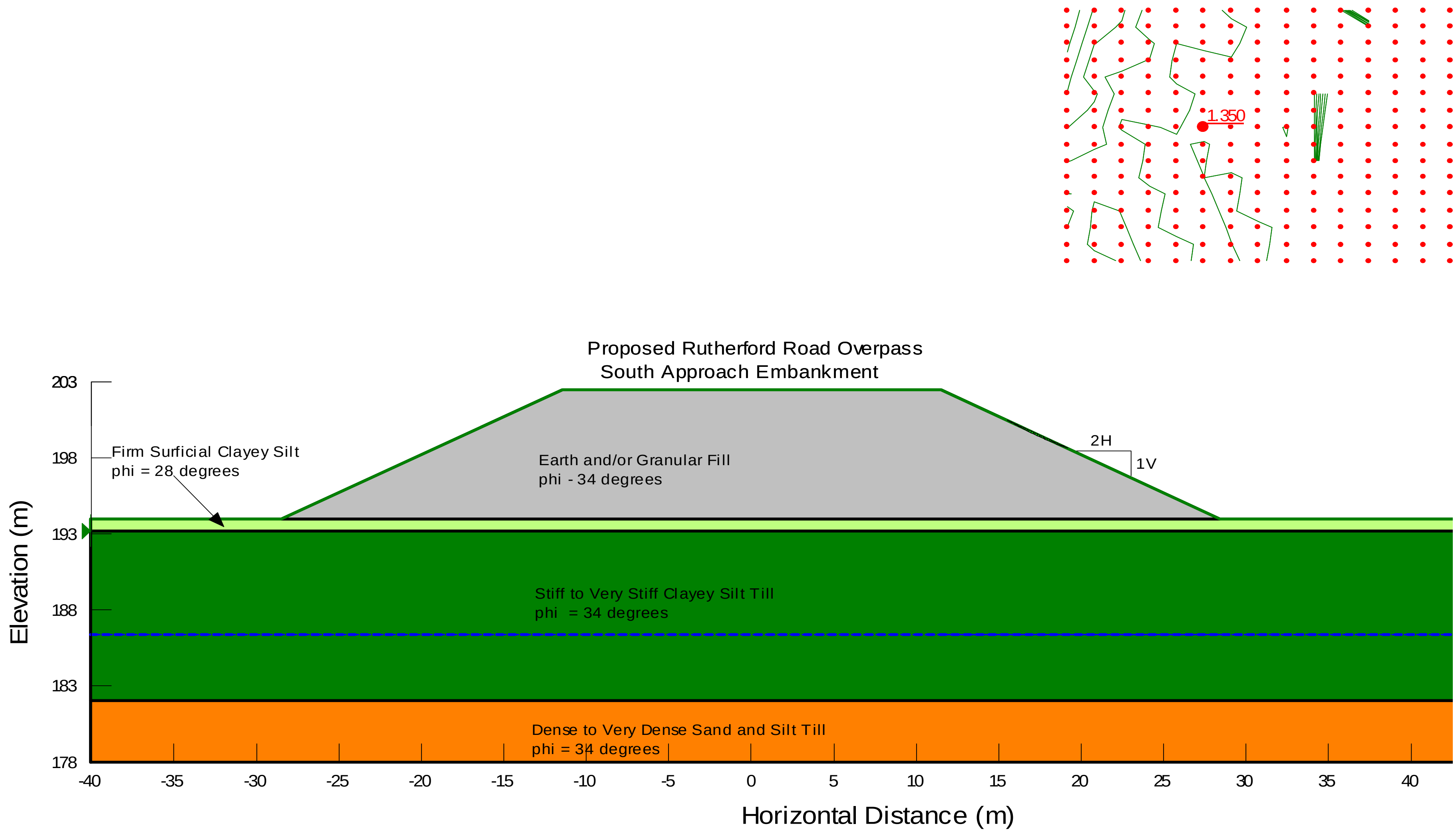
FILE No. 061111012HB0F1.dwg

PROJECT No. 06-1111-012-6 REV. B

PLOT DATE: August 5, 2009
FILENAME: T:\Projects\2006\06-1111-012 (MRC, Vaughan)\-HB- (RUTHERFORD ROAD)\061111012HB0F1.dwg

HIGHWAY 427 EXTENSION - RUTHERFORD ROAD OVERPASS
SOUTH APPROACH EMBANKMENT - STATIC GLOBAL STABILITY

FIGURE 2

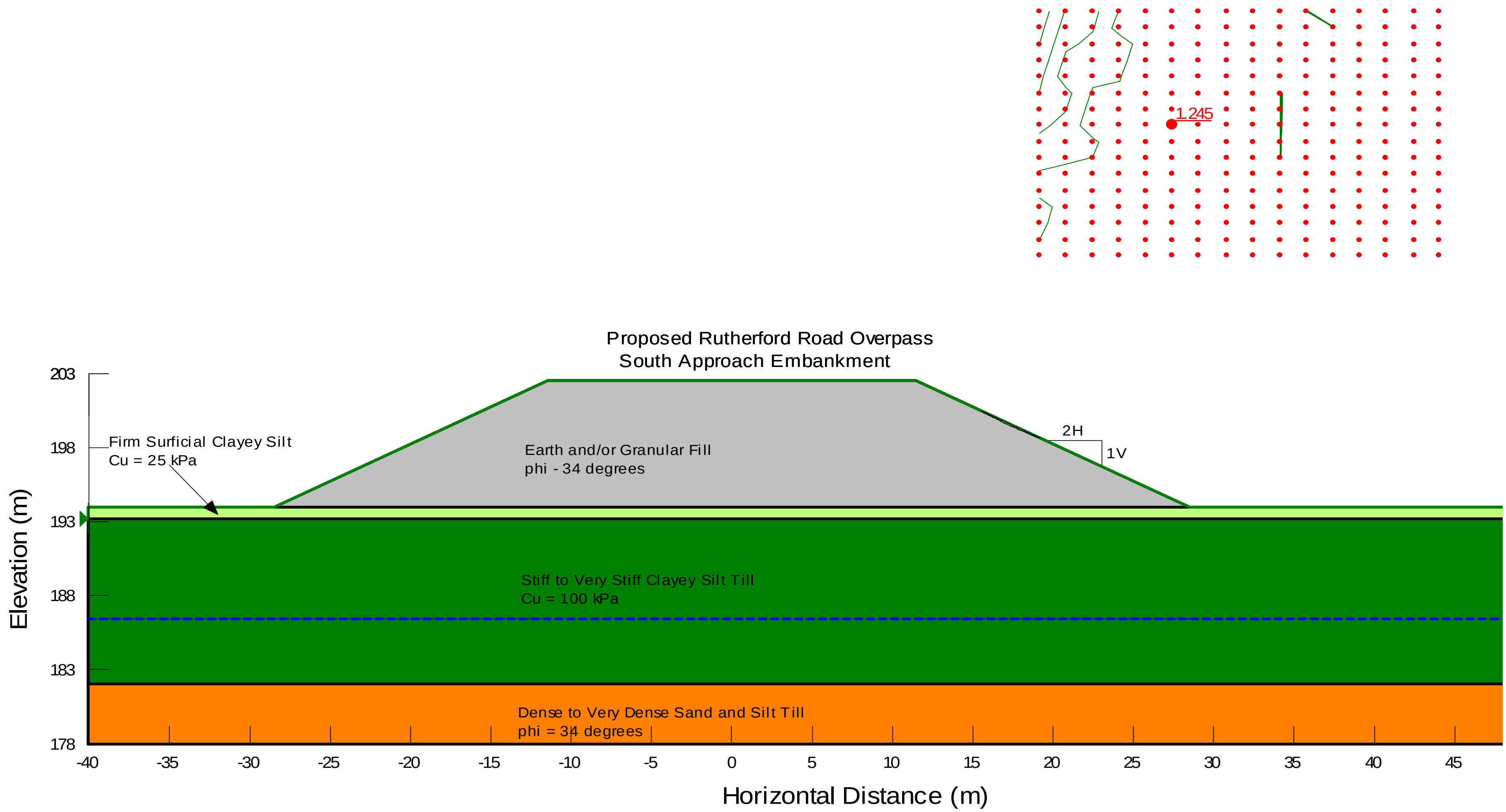


Date: June 2009
Project: 06-1111-012-6

Golder Associates

Drawn: SMM
Checked: LCC

HIGHWAY 427 EXTENSION - RUTHERFORD ROAD OVERPASS SOUTH APPROACH EMBANKMENT - SEISMIC GLOBAL STABILITY	FIGURE 3



Date: June 2009
Project: 06-1111-012-6

Golder Associates

Drawn: SMM
Checked: LCC

APPENDIX A

Record of Boreholes

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

LIST OF ABBREVIATIONS

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

I. SAMPLE TYPE

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

II. PENETRATION RESISTANCE

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

Dynamic Cone Penetration Resistance; N_d:
The number of blows by a 63.5 kg (140 lb.) hammer dropped 760 mm (30 in.) to drive uncased a 50 mm (2 in.) diameter, 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.

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RECORD OF BOREHOLE No S15

 METRIC

PROJECT 06-1111-012			
W.O. 05-20012	LOCATION N 4851810.8 ;E 293098.7		ORIGINATED BY JEB
DIST Central	HWY 427	BOREHOLE TYPE 200 mm Outside Diameter Hollow Stem Augers	COMPILED BY TB/VA
DATUM Geodetic		DATE March 25, 2009	CHECKED BY SMM

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
194.0	GROUND SURFACE					
0.0	CLAYEY SILT, trace gravel, trace sand, containing organics (REWORKED) Firm Brown Moist	1	SS	7		
193.2 / 0.8	CLAYEY SILT, some sand, trace gravel (TILL) Stiff to very stiff Brown Moist Containing sand seam at depths of 1.2 m and 1.8 m	2	SS	21		
		3	SS	25		
		4	SS	24		
		5	SS	22		1 17 49 33
		6	SS	26		
		7	SS	21		
		8	SS	18		
		9	SS	14	▽ (approx. El. 186.5)	
		10	SS	13		
	Containing about 50 mm thick sandy silt layer at a depth of 11.4 m	11	SS	29		
182.1 / 11.9	SAND and SILT, trace to some gravel, some clay (TILL) Dense to very dense Grey Wet	12	SS	31		5 32 49 14
		13	SS	34		

ELEVATION SCALE: 193, 192, 191, 190, 189, 188, 187, 186, 185, 184, 183, 182, 181, 180

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

Golder Associates

RECORD OF BOREHOLE No S15

 METRIC

PROJECT 06-1111-012
W.O. 05-20012
LOCATION N 4851810.8 ;E 293098.7
ORIGINATED BY JEB
DIST Central HWY 427
BOREHOLE TYPE 200 mm Outside Diameter Hollow Stem Augers
COMPILED BY TB/VA
DATUM Geodetic
DATE March 25, 2009
CHECKED BY SMM

— CONTINUED FROM PREVIOUS PAGE —

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	SAND and SILT, trace to some gravel, some clay (TILL) Dense to very dense Grey Wet	14	SS	130	178	
177.7 16.3	CLAYEY SILT, trace sand Hard Grey Moist	15	SS	40/0.15	177	0 2 80 18
176.0 18.0	CLAYEY SILT, some sand, trace gravel (TILL) Hard Grey Moist	16	SS	101	176	
175.1 18.9	END OF BOREHOLE					

NOTES:

1. Water level in open borehole at a depth of 7.6 m below ground surface (Elev. 186.4 m) upon completion of drilling.

2. Borehole backfilled with bentonite.

Chart 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; STRAT PLOT; GROUND WATER CONDITIONS.

+³, ×³: Numbers refer to Sensitivity   O³% STRAIN AT FAILURE

Golder Associates

RECORD OF BOREHOLE No S16

 METRIC

PROJECT 06-1111-012
W.O. 05-20012
DIST Central HWY 427
DATUM Geodetic
LOCATION N 4851817.3 ;E 293152.1
BOREHOLE TYPE 200 mm Outside Diameter Hollow Stem Augers
DATE March 20, 2009
ORIGINATED BY JEB
COMPILED BY TB/VA
CHECKED BY SMM

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
194.6	GROUND SURFACE					
0.0	CLAYEY SILT, some sand, containing rootlets (REWORKED) Firm Brown Moist	1	SS	6		
194.0 0.6	CLAYEY SILT, some sand, trace gravel (TILL) Stiff to very stiff Brown Moist	2	SS	17		
		3	SS	25		
		4	SS	19		
		5	SS	25		
		6	SS	14		
		7	SS	14		
					∇	
		8	SS	20		2 18 56 24
		9	SS	18		
		10	SS	12		
		11	SS	15		
		12	SS	20		
181.2 13.4	SAND and SILT, some gravel, trace clay (TILL) Very dense Grey Wet	13	SS	58		19 30 46 5
179.7						

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; ELEVATION SCALE (194–180); 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

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PROJECT 06-1111-012	RECORD OF BOREHOLE No S16	2 OF 3	METRIC
W.O. 05-20012	LOCATION N 4851817.3 ;E 293152.1		ORIGINATED BY JEB
DIST Central HWY 427	BOREHOLE TYPE 200 mm Outside Diameter Hollow Stem Augers		COMPILED BY TB/VA
DATUM Geodetic	DATE March 20, 2009		CHECKED BY SMM

— CONTINUED FROM PREVIOUS PAGE —

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
14.9	CLAYEY SILT, trace to some sand, trace gravel, containing cobbles (TILL) Hard Grey Wet to moist	14	SS	120/0.15	179	
		15	SS	117	178 / 177	
		16	SS	89	176	
175.1 19.5	SILT, trace to some sand, trace clay Compact to very dense Grey Moist	17	SS	73	175 / 174	
		18	SS	51	173	0 0 90 10
		19	SS	52	172 / 171	
		20	SS	24	170 / 169 / 168	
167.5 27.1	CLAYEY SILT, some sand, some gravel (TILL) Hard Grey Moist	21	SS	65	167 / 166	
165.0 29.6	SHALE (BEDROCK) Grey				165	

Chart 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; GROUND WATER CONDITIONS; STRAT PLOT.

Continued Next Page

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

RECORD OF BOREHOLE No S16

 METRIC

PROJECT 06-1111-012

W.O. 05-20012

DIST Central HWY 427

DATUM Geodetic

LOCATION N 4851817.3 ;E 293152.1

BOREHOLE TYPE 200 mm Outside Diameter Hollow Stem Augers

DATE March 20. 2009

ORIGINATED BY JEB

COMPILED BY TB/VA

CHECKED BY SMM SM

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
	— CONTINUED FROM PREVIOUS PAGE —										
	SHALE (BEDROCK) Grey										
			22	SS	20/0.10		164				
							163				
			23	SS	20/0.10						
							162				
161.0			24	SS	50/0.08						
33.6	END OF BOREHOLE										

NOTES:

1. Water level in open borehole at a depth of 6.0 m below ground surface (Elev. 188.6 m) upon completion of drilling.

2. Borehole backfilled with bentonite.

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

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

PROJECT 06-1111-012

RECORD OF BOREHOLE No S17

 METRIC

W.O.	05-20012
LOCATION	N 4851849.1 ;E 293063.4
ORIGINATED BY	SB
DIST	Central
HWY	427
BOREHOLE TYPE	200 mm Outside Diameter Hollow Stem Augers
COMPILED BY	VA
DATUM	Geodetic
DATE	March 25, 2009
CHECKED BY	SMM

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
194.6	GROUND SURFACE
0.0	Sand and gravel (FILL) Brown Moist
193.8 / 0.8	CLAYEY SILT, trace gravel, trace sand (Reworked) Very stiff Brown and grey to brown Moist
193.2 / 1.5	SILTY CLAY, some sand, trace gravel (TILL) Very stiff to hard Grey Moist
187.3 / 7.3	CLAYEY SILT, some sand, trace gravel (TILL) Stiff to very stiff Grey Moist
183.0 / 11.6	SAND and SILT, trace to some gravel, trace clay (TILL) Dense to very dense Grey Moist to wet
179.9 / 14.7	

Continued Next Page

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	20	
2	SS	29	
3	SS	49	
4	SS	27	5 13 44 38
5	SS	31	
6	SS	26	
7	SS	24	
8	SS	19	
9	SS	13	1 16 58 25
10	SS	17	
11	SS	32	
12	SS	93	7 35 49 9

Column headings: STRAT PLOT; GROUND WATER CONDITIONS; ELEVATION SCALE (194, 193, 192, 191, 190, 189, 188, 187, 186, 185, 184, 183, 182, 181, 180); 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

RECORD OF BOREHOLE No S17

 METRIC

PROJECT 06-1111-012
W.O. 05-20012 LOCATION N 4851849.1 ;E 293063.4 ORIGINATED BY SB
DIST Central HWY 427 BOREHOLE TYPE 200 mm Outside Diameter Hollow Stem Augers COMPILED BY VA
DATUM Geodetic DATE March 25, 2009 CHECKED BY SMM

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE
	— CONTINUED FROM PREVIOUS PAGE —				
	CLAYEY SILT, trace gravel, trace sand (TILL) Hard Grey Moist	13	SS	80/0.2	179
					178
		14	SS	102	
					177
176.6 18.0	SAND and SILT, trace gravel, trace clay (TILL) Very dense Moist Grey	15	SS	83	176
175.9 18.8	END OF BOREHOLE				

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; 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

NOTES:

1. A 50 mm diameter monitoring well was installed at a depth of 15.2 m (Elev. 179.4 m).

Water level measurements

Date	Depth	Elev.
On Completion	11.2 m	183.4 m
April 24, 2009	4.0 m	190.6 m
May 25, 2009	4.1 m	190.4 m
June 15, 2009	4.0 m	190.5 m
July 09, 2009	3.8 m	190.7 m

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

RECORD OF BOREHOLE No S18

 METRIC

PROJECT 06-1111-012

W.O. 05-20012 | LOCATION N 4851863.9 ;E 293113.0 | ORIGINATED BY SB

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

DATUM Geodetic | DATE March 23, 2009 | CHECKED BY SMM SM

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
194.3	GROUND SURFACE					
0.0	TOPSOIL					
	Silty sand, some gravel (FILL) Brown Moist					
193.5 / 0.8	SAND, some gravel, trace silt Compact to dense Brown Moist	1	SS	36		
192.5 / 1.8	CLAYEY SILT, some sand, trace gravel (TILL) Stiff to hard Brown to grey Moist	2	SS	10		
		3	SS	26		
		4	SS	30		
	Becoming grey below a depth of 3.8 m	5	SS	21		1 23 48 28
		6	SS	20		
		7	SS	28		
					∇ (approx. El. 186.8)	
		8	SS	24		
		9	SS	16		
		10	SS	29		2 11 55 31
181.8 / 12.5	SAND and SILT, some gravel, trace clay (TILL) Dense Grey Wet	11	SS	49		
180.9 / 13.4	CLAYEY SILT, some sand, trace gravel (TILL) Very stiff to hard Grey Moist	12	SS	17		

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; ELEVATION SCALE (194 to 180); 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/m^3.

Continued Next Page

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

Golder Associates

RECORD OF BOREHOLE No S18

 METRIC

PROJECT 06-1111-012			
W.O. 05-20012	LOCATION N 4851863.9 ;E 293113.0		ORIGINATED BY SB
DIST Central HWY 427	BOREHOLE TYPE 200 mm Outside Diameter Hollow Stem Augers		COMPILED BY VA
DATUM Geodetic	DATE March 23, 2009		CHECKED BY SMM

— CONTINUED FROM PREVIOUS PAGE —

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	GR	SA	SI	CL
	CLAYEY SILT, some sand, trace gravel (TILL) Very stiff to hard Grey Moist	13	SS	170	179				
					178				
		14	SS	63	177				
176.5 17.8	SILT, trace to some clay Compact to very dense Grey Moist	15	SS	13/0.15	176				
					175				
		16	SS	78	174	0	0	84	16
					173				
					172				
		17	SS	58	171				
					170				
					169				
		18	SS	20	168	0	6	89	5
					167				
					166				
165.7 28.7	SHALE (BEDROCK) Grey	19	SS	100/0.10	165				

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

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 S18

 METRIC

PROJECT 06-1111-012
W.O. 05-20012
DIST Central HWY 427
DATUM Geodetic
LOCATION N 4851863.9 ;E 293113.0
BOREHOLE TYPE 200 mm Outside Diameter Hollow Stem Augers
DATE March 23, 2009
ORIGINATED BY SB
COMPILED BY VA
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 —										
	SHALE (BEDROCK) Grey						164				
			20	SS	120/0.10						
							163				
162.2 / 32.1			21	SS	100/0.05						
	END OF BOREHOLE										

NOTES:

1. Water level in open borehole at a depth of 7.6 m below ground surface (Elev. 186.7 m) upon completion of drilling.

2. Borehole backfilled with bentonite.

+ 3, × 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

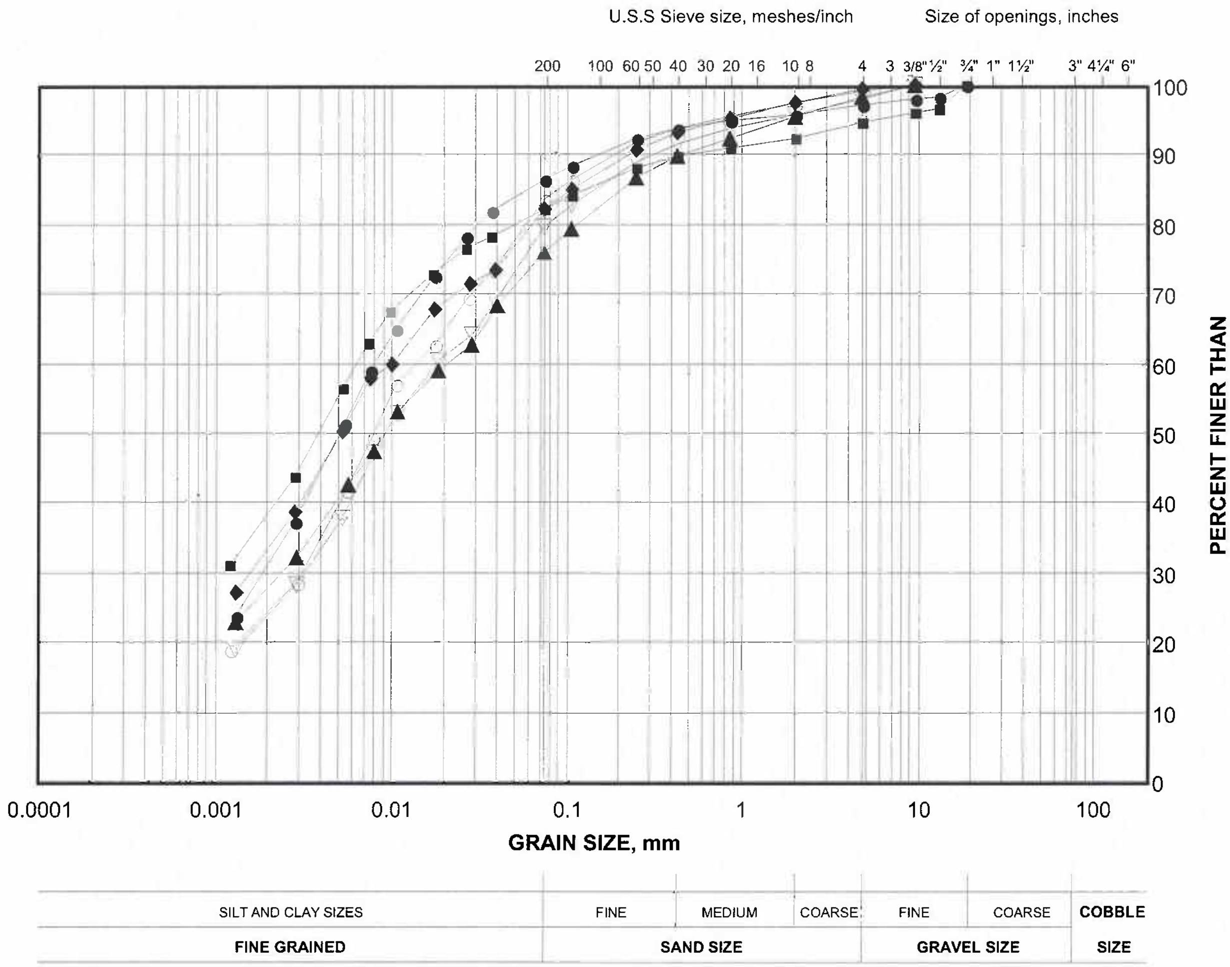
APPENDIX B

Laboratory Test Results

GRAIN SIZE DISTRIBUTION TEST RESULTS

Silty Clay to Clayey Silt Till

FIGURE B1



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	S18	10	183.3
■	S17	4	191.2
◆	S15	5	190.6
▲	S18	5	188.3
▽	S16	8	188.2
○	S17	9	185.2

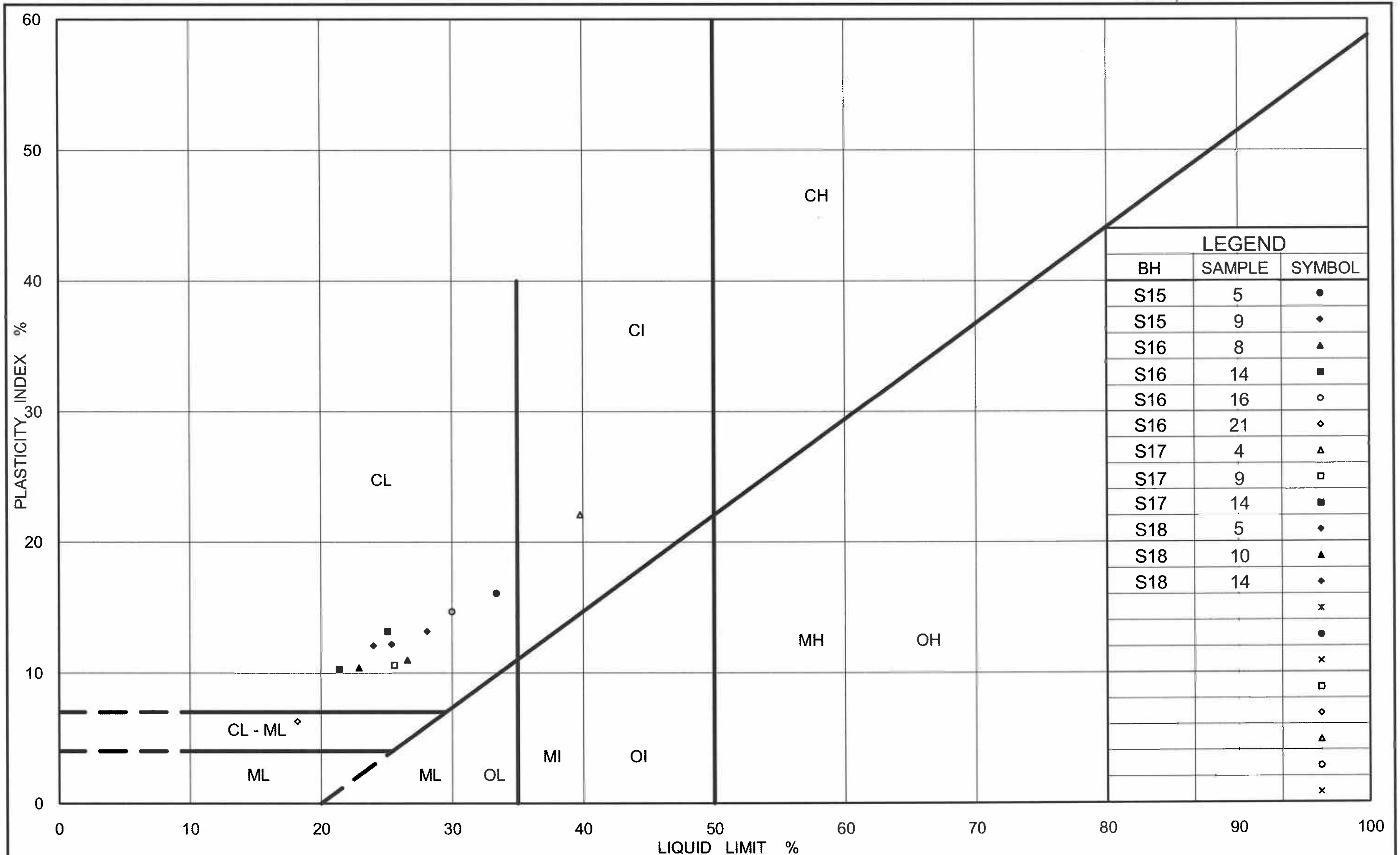
Project Number: 06-1111-012-6

Checked By: SM

Golder Associates

Date: 08-Jun-09

Oct 75, FF-S-21



Ministry of Transportation

Ontario

PLASTICITY CHART
Clayey Silt to Silty Clay Till

Figure No. B2

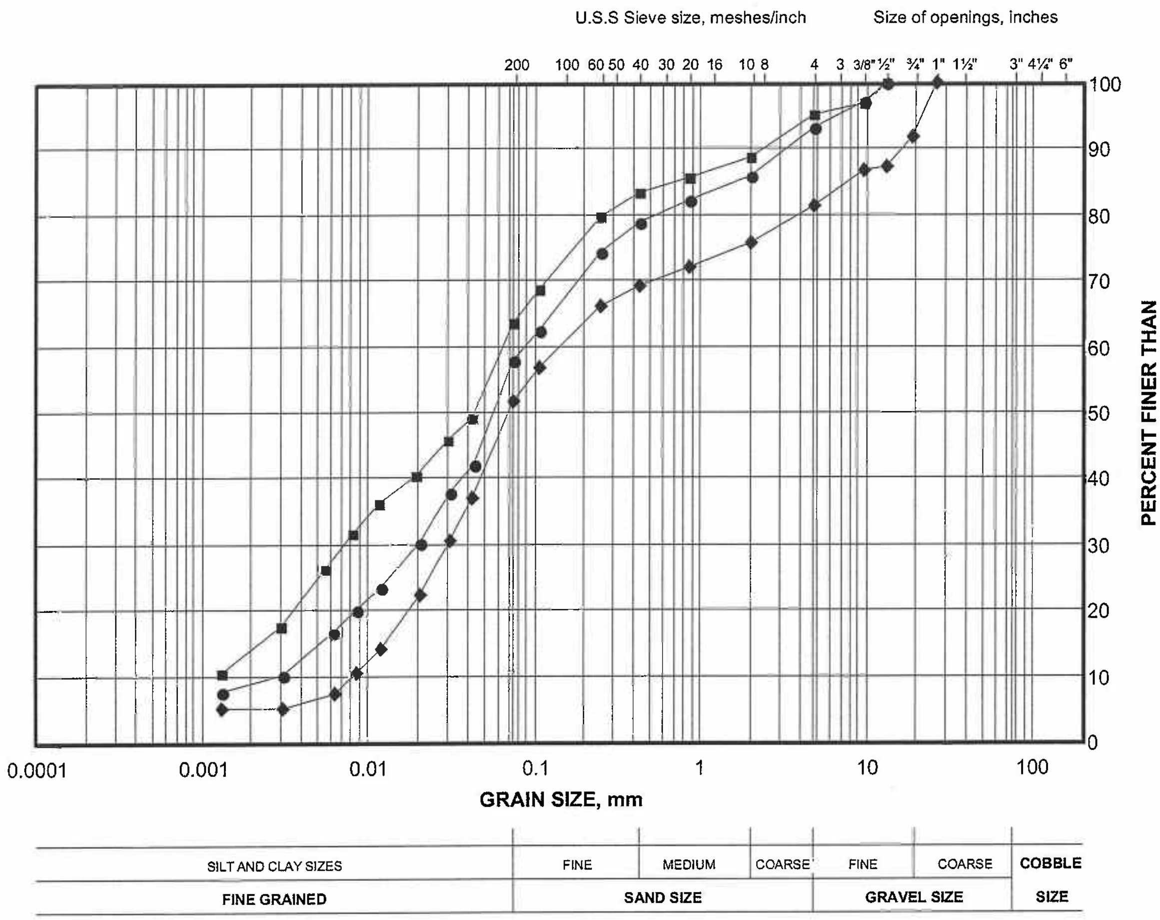
Project No. 06-1111-012-6

Checked By: SM

GRAIN SIZE DISTRIBUTION TEST RESULTS

Sand and Silt Till

FIGURE B3



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	S17	12	180.7
■	S15	12	181.5
◆	S16	13	180.6

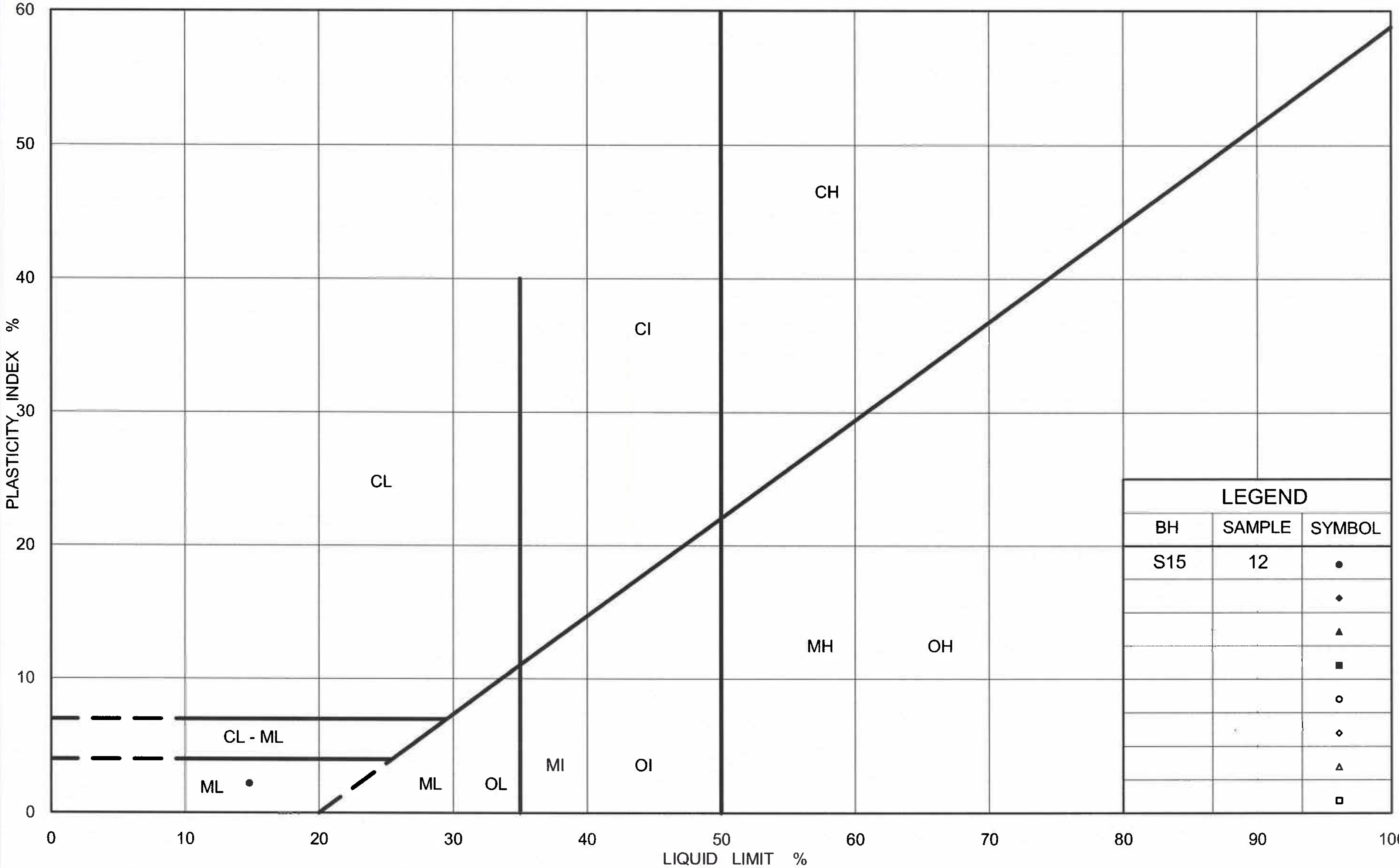
Project Number: 06-1111-012-6

Checked By: SMU

Golder Associates

Date: 08-Jun-09

Oct 75, FF-S-21



LEGEND		
BH	SAMPLE	SYMBOL
S15	12	●
		◆
		▲
		■
		○
		◇
		△
		□

Ministry of Transportation

Ontario

PLASTICITY CHART
Sand and Silt Till

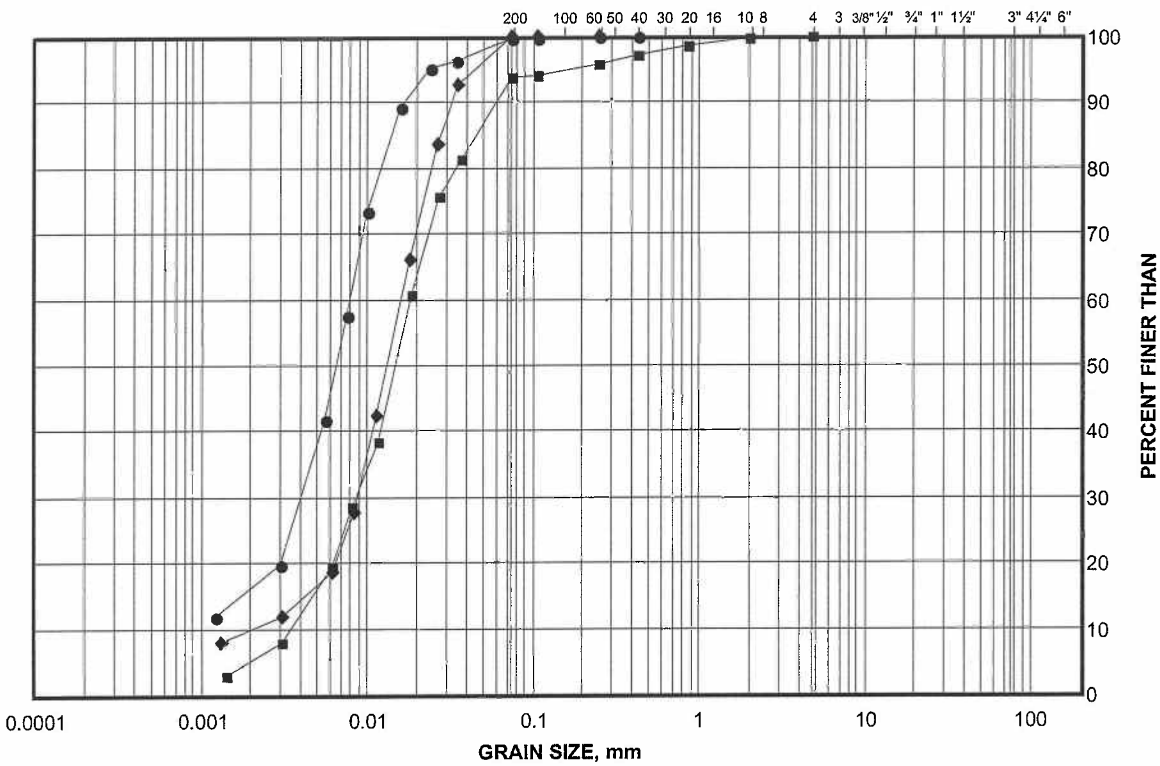
Figure No. B4

Project No. 06-1111-012-6

Checked By: SM

GRAIN SIZE DISTRIBUTION TEST RESULTS
Silt

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)
●	S18	16	174.6
■	S18	18	168.1
◆	S16	18	173.0

Project Number: 06-1111-012-6

Checked By: SM

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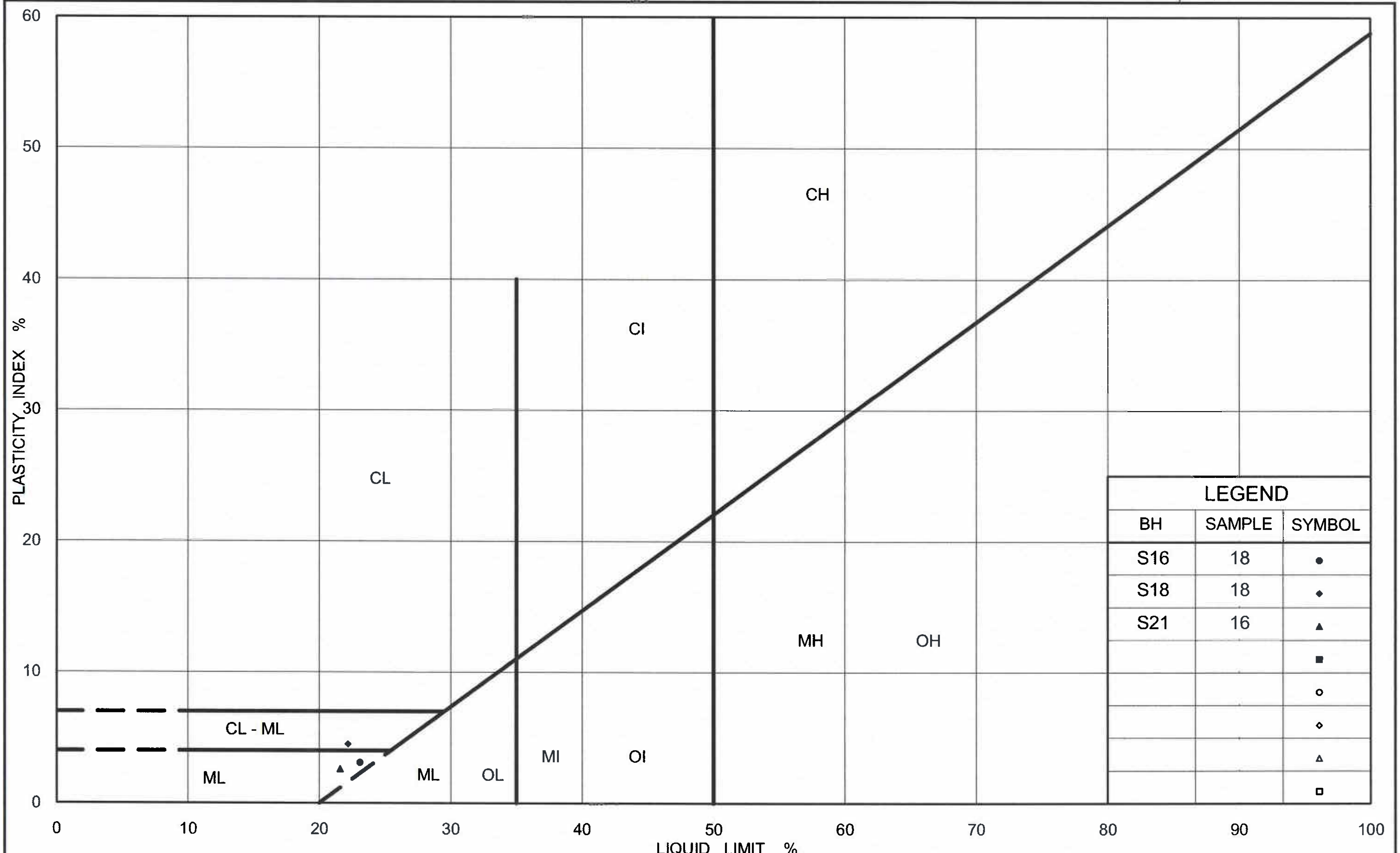
Date: 03-Jun-09

Oct 75, FF-S-21



LEGEND

BH	SAMPLE	SYMBOL
S16	18	●
S18	18	◆
S21	16	▲
		■
		○
		◇
		△
		□

Ministry of Transportation

Ontario

PLASTICITY CHART
Silt

Figure No. B6

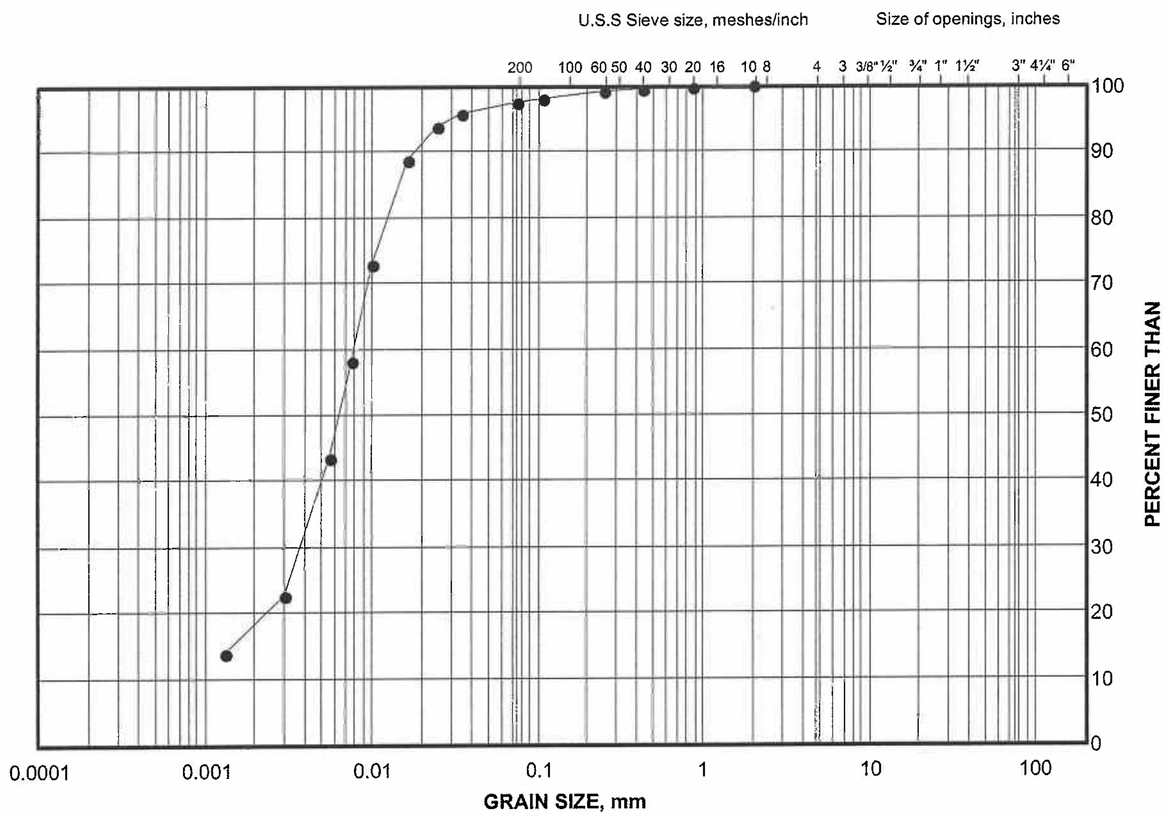
Project No. 06-1111-012-6

Checked By: SM

GRAIN SIZE DISTRIBUTION TEST RESULT

Clayey Silt

FIGURE B7



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	S15	15	176.9

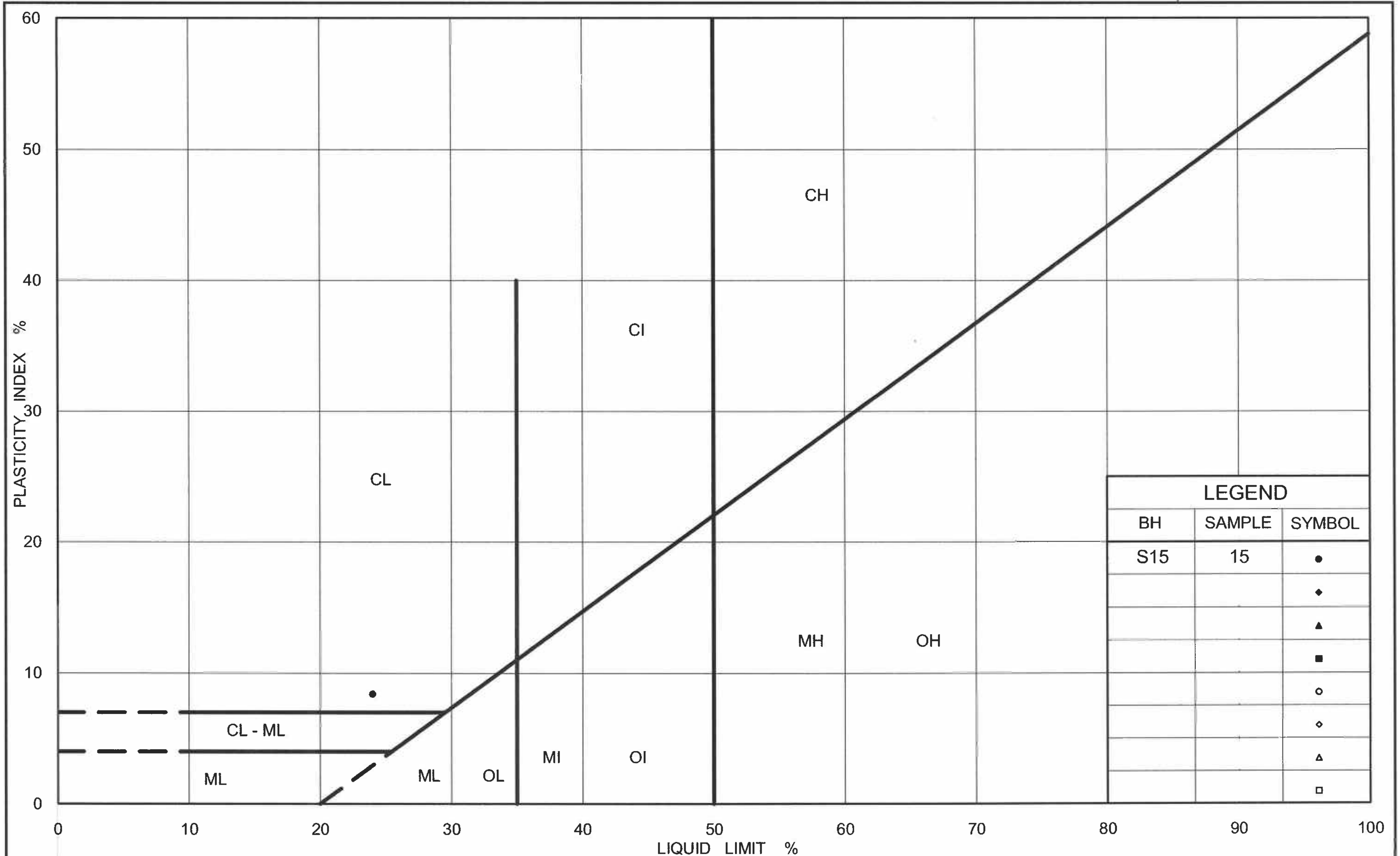
Project Number: 06-1111-012-6

Checked By: SM

Golder Associates

Date: 03-Jun-09

Oct 75, FF-S-21



Ministry of Transportation Ontario	PLASTICITY CHART Clayey Silt	Figure No. B8 Project No. 06-1111-012-6 Checked By: SM

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