



AUGUST 2009

PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT

**MAJOR MACKENZIE DRIVE 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

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PART A

**PRELIMINARY FOUNDATION INVESTIGATION REPORT
MAJOR MACKENZIE DRIVE 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 the proposed realigned Major Mackenzie Drive, 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 Major Mackenzie Drive overpass structures are located approximately 250 m north of the existing Major Mackenzie Drive and are approximately 50 m west of Huntington Road in the City of Vaughan, Ontario (see Figure 1). It is understood that Major Mackenzie Drive is to be realigned approximately 260 m north of its current location in vicinity of the proposed Highway 427 Extension.

In general, the topography in the area of the overall project limits consists of flat-lying to gently sloping land which is associated with the two creek crossings; Rainbow Creek and West Robinson Creek. The terrain consists of densely treed areas and farmland. Along Zenway Boulevard, Langstaff Road and Rutherford Road there is some residential, commercial and/or light industrial development.

The proposed overpass structure and associated approach embankments are to be situated within an area of generally flat farmland with the ground surface at about Elevation 205 m.

3.0 INVESTIGATION PROCEDURES

The field work for the Major Mackenzie Drive overpass structures was carried out in March, 2009 during which time a total of two boreholes were advanced. These boreholes, designated as Boreholes S34 and S36, were advanced at the locations shown on Drawing 1.

The field investigation was carried out on the west shoulder of Huntington Road using a truck mounted D-90 drill rig supplied and operated by Walker Drilling Ltd. of Utopia, Ontario. The 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 (O.D.) 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 conducted in cohesive soils for determination of undrained shear strengths (ASTM D2573 01).

The MTO criteria for boreholes drilled at proposed structures is to terminate after penetrating at least 3 m of SPT 'N' values of greater than 100 blows per 0.3 m of penetration. Borehole S34 was drilled to a depth of 43.3 m below the existing ground surface into hard clayey silt till; however there was only one recorded SPT 'N' value of 100 blows at a depth of about 20 m, below which the SPT 'N' values ranged from about 16 to 62 blows per 0.3 m of penetration. A Dynamic Cone Penetration Test (DCPT) was advanced from the bottom of the borehole to a



depth of 47.6 m where refusal to further penetration was encountered. Borehole S36 was terminated at a depth of 15.9 m; however similar to Borehole S36 3 m of soil having SPT 'N' values of greater than 100 blows per 0.3 m of penetration was not encountered within the depth of the borehole.

The groundwater conditions in the open boreholes were observed during the drilling operations, and a standpipe piezometer was installed in Borehole S36 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 boreholes and annulus surrounding the piezometer pipe were backfilled to the surface with bentonite pellets/grout. The piezometer installation details and water level readings are described on the Record of Borehole Sheets in Appendix A. Borehole S34 was backfilled with bentonite upon completion, in accordance with Ontario Regulation 903 (as amended by Ontario Regulation 372).

The field work was observed by members of Golder's engineering and technical staff, who located the boreholes, arranged for the clearance of underground services, 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. 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 positioned relative to MTM NAD 83 northing and easting coordinates and the ground surface elevations are referenced to Geodetic datum.

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

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

4.2 Subsurface Conditions

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

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



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

The interpreted stratigraphic conditions along the Highway 427 Extension at the proposed realigned Major Mackenzie Drive overpasses are presented for the NBL alignment only as the boreholes were drilled between 35 m and 50 m east of the NBL alignment. This stratigraphic profile is shown on Drawing 2 represents a simplification of the subsurface conditions as encountered in the boreholes. Variation in the stratigraphic boundaries and properties of the soil deposits will occur between and beyond the borehole locations.

In general, the subsurface conditions in the investigated area consist of asphalt, underlain by up to 1.5 m of fill consisting of sand and gravel, to silty sand. The fill is underlain by a till deposit consisting of silty clay that grades with depth to clayey silt which in turn is underlain by a clayey silt and silt deposit.

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

4.2.1 Asphalt

Approximately 100 mm of asphalt was encountered immediately below the ground surface in all boreholes drilled for this site.

4.2.2 Sand and Gravel to Silty Sand Fill

In both boreholes drilled for this site, the asphalt is underlain by fill that extends to depths of between 1.1 m and 1.5 m below ground surface (between Elevations 203.7 m and 204.1 m). The composition of the fill varies; the upper 0.2 m to 0.7 m consists of sand and gravel. The sand and gravel fill is underlain by silty sand fill which varies in thickness from about 0.3 m to 1.3 m. The silty sand fill contains trace gravel and trace clay. The measured SPT 'N' values within the silty sand were 9 and 10 blows per 0.3 m of penetration, indicating that the silty sand fill has a loose to compact relative density.

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.

4.2.3 Silty Clay to Clayey Silt Till

In both boreholes drilled at this site, the fill is underlain by a till deposit that grades with depth from a silty clay to clayey silt. The surface of the silty clay till was encountered at Elevations of 204.1 m and 203.7 m and the base of the silty clay till deposit was encountered at Elevation 198.1 in both boreholes; equating to thicknesses of between 6.0 m and 5.6 m thick in Boreholes S34 and S36, respectively. The surface of the clayey silt till was encountered at Elevation 198.1 m in Borehole S34 and S36. Borehole S36 terminated within the clayey silt till at Elevation 189.4 m, however Borehole S34 fully penetrated the clayey silt till deposit, which was found to have a thickness of approximately 19.7 m. The base of the clayey silt till was encountered at Elevation 178.4 m, although the deposit base may be lower or higher than this in Borehole S36 where it was not fully penetrated.

The silty clay till deposit contains trace to some sand and trace gravel. The clayey silt till contains trace to some sand, and trace gravel; at depth the sand content increases and the clayey silt till contains with sand. The results of grain size distribution tests carried out on two selected samples of the silty clay till and six selected samples of the clayey silt till deposit are provided on Figures B1 and B2 in Appendix B, respectively. Atterberg limits testing was carried out on three samples of the silty clay till deposit and measured plastic limits ranging from 18 to 20 percent, liquid limits ranging from 36 to 41 percent, and plasticity indices ranging from 17 to 21



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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 silty clay of medium plasticity. Measured water contents on samples of the silty clay till ranged from 17 to 25 percent. Atterberg limits testing was carried out on five samples of the clayey silt till deposit and measured plastic limits ranging from 13 to 15 percent, liquid limits ranging from 21 to 27 percent, and plasticity indices ranging from 8 to 12 percent. These results, which are plotted on a plasticity chart on Figure B3 in Appendix B, confirm that this portion of the till is a clayey silt of low plasticity. Measured water contents on selected samples of the clayey silt till ranged from 12 to 28 percent.

A laboratory consolidation (oedometer) test was carried out on one specimen of the silty clay till obtained from a Shelby tube sample in Borehole S36 at a depth of 6.5 m below ground surface. A preconsolidation pressure of about 420 kPa was estimated from the void ratio versus logarithmic pressure plot and from the total work versus pressure plot. A bulk unit weight of about 20 kN/m³ and a specific gravity of about 2.8 were measured on the consolidation test specimen. Details of the test results are shown on Figure B4, and are summarized below.

Borehole Sample No.	Sample Depth / Elevation	σ_{vo}' (kPa)	σ_p' (kPa)	$\sigma_p' - \sigma_{vo}'$ (kPa)	OCR	C_c	C_r	e_o	c_v^* (cm ² /s)
Borehole S36 Sample 7	6.4 m / 198.8 m	130	422	292	3.2	0.215	.034	0.664	1.6×10^{-1}

Note: * For stress range of $150 \text{ kPa} \leq \sigma_v' \leq 300 \text{ kPa}$
 where: σ_{vo}' is the effective overburden pressure in kPa
 σ_p' is the preconsolidation pressure in kPa
 OCR is overconsolidation ratio
 e_o is initial void ratio
 C_c is the compression index
 C_r is the recompression index
 c_v is the coefficient of consolidation in cm²/s

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 4 m depth below ground surface vary from 10 to 19 blows per 0.3 m of penetration, indicating a stiff to very stiff consistency. Between approximately Elevation 201 m and 191.5 m, the SPT 'N' values vary from 6 to 13 blows per 0.3 m of penetration, and in situ field vane tests measured undrained shear strengths typically greater than 120 kPa with the exception of one measured values of about 83 kPa at a depth of 13.2 m (Elevation 192 m) in Boreholes S36. The field vane test results together with the SPT 'N' values indicate that this approximately 7 m thick middle zone of the cohesive till has a generally stiff consistency. Below Elevation 191.5 m the SPT 'N' values increase and vary from 30 to greater than 100 blows per 0.3 m of penetration, indicating a hard consistency.

4.2.4 Clayey Silt

Underlying the clayey silt till in Borehole S34, a deposit of clayey silt was encountered at 26.8 m depth below ground surface. This clayey silt deposit extends to about 33.0 m depth (Elevation 172.2 m); corresponding to a thickness of about 6.2 m.

The clayey silt contains trace sand. The results of grain size distribution tests were completed on two selected samples of the clayey silt are provided on Figure B5 in Appendix B. Atterberg limits testing was carried out on one sample of the clayey silt deposit and measured a plastic limit of 18 percent, a liquid limit of 24, and a plasticity index of 6 percent. These results, which are plotted on a plasticity chart on Figure B6 in Appendix B, confirm that this material is a clayey silt of low plasticity. Measured water contents on selected samples of the upper clayey silt were about 21 percent.



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SPT 'N' values measured within the clayey silt deposit were 13, 26 and 35 blows per 0.3 m of penetration, indicating a stiff to hard consistency.

4.2.5 Silt

Beneath the clayey silt deposit in Borehole S34, a silt deposit was encountered at a depth of 33.0 m (Elevation 172.2 m); Borehole S34 terminated within the silt deposit at a depth of 43.3 m (Elevation 161.9 m).

The silt contains trace sand and trace clay. The result of a grain size distribution test completed on a sample of the silt deposit is provided on Figure B7 in Appendix B. Atterberg limits testing was carried out on one sample of the silt deposit and measured a plastic limit of 19 percent, a liquid limit of 23, and a plasticity index of 4 percent as shown on Figure B8 in Appendix B, confirm that this material is a silt that is non-plastic or has a low plasticity. Measured water contents on two samples of the silt were 23 and 26 percent.

SPT 'N' values measured within the silt deposit were 0 blows (weight of rods), 16, 19 and 21 blows per 0.3 m of penetration. It is likely that measured SPT 'N' value of 0 is attributable to sample disturbance at the bottom of the borehole which may be the result of groundwater inflow into the borehole. Therefore it is likely that this material has a compact relative density.

Below the last sample recovered within this deposit (end of borehole) a dynamic cone penetration test (DCPT) was conducted to a depth of 47.6 m (Elevation 157.5 m). The DCPT blows per 0.3 m ranged from the weight of rods to 198 and were generally greater than 100 blows below a depth of 46.4 m (Elevation 158.8 m). It is noted that the deeper the DCPT is advanced the more unreliable the data becomes since the soil is collapsing around the rod 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. The DCPT merely provides an indication as to where more competent material is encountered.

4.3 Groundwater Conditions

During drilling the water level in Borehole S34 was at approximately 21.8 m depth (Elevation 183.4 m), however the borehole was dry upon completion of drilling. Borehole S36, which terminated within the clayey silt till, was dry during and upon completion of drilling. In general, the clayey silt till samples were moist, the clayey silt samples were moist to wet and the silt samples were wet.

A standpipe piezometer was installed in Borehole S36 to permit monitoring of the groundwater level at this site. Details of the piezometer installation are shown the borehole records in Appendix A. The groundwater levels measured in the piezometer installation are summarised below.

Borehole No.	Ground Surface Elevation (m)	Depth to Groundwater Level	Groundwater Elevation (m)	Date of Measurement
S36	205.2	7.3 m	197.9 m	April 27, 2009
		6.4 m	198.8 m	May 25, 2009
		6.2 m	199.0 m	June 15, 2009
		6.2 m	199.0 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.



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5.0 CLOSURE

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



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MAJOR MACKENZIE DR. OVERPASSES - HIGHWAY 427 EXTENSION**

PART B

**PRELIMINARY FOUNDATION DESIGN REPORT
MAJOR MACKENZIE DRIVE 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 Major Mackenzie Drive overpass north and southbound bridges. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the preliminary subsurface investigation. The discussion and 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 foundation and approach embankments. Where comments are made on construction, they are provided in order to highlight those aspects which could affect the design of the project. 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 realigned Major Mackenzie Drive overpasses are proposed to consist of two-span structures with centre piers located between the eastbound and westbound lanes of the realigned Major Mackenzie Drive. Based on the preliminary General Arrangement (GA) Drawing provided by MRC on May 15, 2009, the span length between the north abutment and the pier is about 36 m and between south abutment and the pier the span length is about 32 m.

According to the preliminary GA Drawing, the finished grade of Highway 427 NBL and SBL over realigned Major Mackenzie Drive varies from approximately Elevation 214.8 m to Elevation 213.3 m, rising southward. The natural ground surface across the site is generally at approximately Elevation 205.0 m. The south approach embankments are proposed to be about 9.6 m high relative to the existing ground surface, and the north approach embankments are proposed to be about 8.2 m high relative to the adjacent existing ground surface.

Due to restrictions on accessing the property where the structure is proposed the boreholes for this structure were drilled on Huntington Road, some 35 to 50 m from the proposed structure location, it is recommended at detail design stage that access be gained to the property west of Huntington Road and boreholes be drilled at the foundation units.

6.2 Foundation Recommendations

6.2.1 Foundation Options

Based on the proposed road configuration and subsurface soil conditions the following foundation options are considered feasible for the Major Mackenzie Road overpass structures:

- **Spread footings founded on the stiff to very stiff silty clay till:** This option is feasible at the piers; however considering that the grade at the north and south abutments is to be raised by about 8.2 m and 9.6 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. Considering that the proposed structure is located within an agricultural field it is anticipated that the fill encountered in the boreholes drilled through the road



will not be encountered in the field. However, as encountered at the boreholes drilled for the Canadian Pacific Railway (CPR) / McGillivray Road overpass structures it is anticipated that approximately the upper 1 m of soil may have been affected by agricultural activities and may need to be subexcavated to expose the stiff to very stiff silty clay till. At the piers, the footings would have to extend through any fill or disturbed soil present at the foundation units to be founded on the stiff to very stiff silty clay till. The surficial soil conditions should be determined at the proposed abutments at the detail design phase of the project.

- **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 is anticipated that subexcavation of the “reworked” soil would be required to expose the stiff to very stiff silty clay till at the north and south abutments, prior to construction of the new approach embankments. For this option, the loading from the new approach embankment would still result in some settlement in the stiff portion of the silty clay till at depth, which could result in differential settlement between the abutments and piers. The surficial soil conditions should be determined at the proposed abutments at the detail design phase of the project.
- **Steel H-piles driven to found within the clayey silt 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 stiff to very stiff silty clay till and it is anticipated that no additional subexcavation would be required beyond that needed for frost protection; however this must be confirmed at detail design stage by drilling boreholes at the foundations units. If sufficient geotechnical resistances can be achieved spread footings on stiff to very stiff silty clay till are therefore preferred; 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 piers, due to the relative ease of installation of driven piles.

Recommendations for preliminary design of spread footings, caissons and steel H-pile 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

This section provides preliminary foundation recommendations for spread footings at the piers on stiff to very stiff silty clay till.



6.2.2.1 *Founding Elevations*

The piers may be supported on spread footings placed on the stiff to very stiff silty clay till. A minimum founding depth of 1.4 m is required for frost protection purposes (OPSD 3090.101). The elevation of shallow spread footings founded on the stiff to very stiff silty clay till will have to be determined for each foundation element during detail design after further geotechnical investigation has been carried out.

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 S34 and S36, located between 35 m and 50 m from the NBL overpass. 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 foundation units. Based on the other boreholes drilled south of 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 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 stiff to very stiff native silty clay till should be calculated in accordance with Section 6.7.5 of the *CHBDC*. A coefficient of friction, $\tan \phi'$, of 0.55 can be used for cast-in-place concrete footings on the properly prepared silty clay 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).

As discussed in Section 6.2.1 the boreholes at this site were drilled through Huntington Road, some 35 m to 50 m east of the bridge structure; due to restrictions with accessing the property where the structures are proposed. The proposed structures are located in an existing agricultural field and based on the drilling carried out in the field east of Huntington Road and north of the Canadian Pacific Railway (CPR) tracks (for the CPR/McGillivray Road overpass structures) it is anticipated that at least the upper 1 m of soil has been “reworked” from agricultural activity. Therefore for the option “perched” spread footings option it is anticipated that subexcavation will be required of at least the upper 1 m of soil that has been “reworked” material which is anticipated to be present within the embankment footprint below the perched abutment, to minimize settlement due to the embankment loading. 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 110 mm to 160 mm of settlement will occur under the loading from the proposed approach embankments, primarily in the stiff zone of the silty clay to clayey silt till between about 4 m and 11 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. These geotechnical resistances will have to be reviewed during detail design, after 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 Canadian Highway Bridge Design Code (*CHBDC*) and its Commentary, using the curves for non-cohesive soils.

6.2.3.3 *Resistances to Lateral Loads*

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



6.2.4 Steel H-Piles

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

For the installation of steel H-piles, consideration will have to be given to the possible presence of cobbles and/or boulders within the till. It is recommended that the piles should 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

The proposed abutments and piers can be supported on steel H-piles driven to found within the clayey silt till having SPT 'N' values of greater than 100 blows per 0.3 m of penetration. In Borehole S34 only one value of 100 blows per 0.3 m of penetration was measured in the borehole at a depth of 17.2 m below ground surface (Elevation 185.3 m). The SPT 'N' values measured 1.5 m above and below the SPT 'N' value of 100 were 71 and 62 blows per 0.3 m of penetration, respectively. Below the SPT 'N' value of 62 blows the SPT 'N' values decrease with depth to 13 blows per 0.3 m of penetration at about 7.6 m below the a depth of 17.2 m. It is recommended that the piles terminate on the clayey silt till having a SPT 'N' value of 100 blows per 0.3 m of penetration at Elevation 187 m (depth of about 18.2 m) and not extend the typical 1.5 m into soil having SPT 'N' values greater than 100 blows per 0.3 m of penetration.

It is recommended that this founding depth be verified by further investigation at the actual foundation unit locations at detail design.

6.2.4.2 Geotechnical Axial Resistances

The proposed abutments and piers can be supported on steel HP 310x110 piles driven to found within the hard clayey silt till as encountered at Elevation 187 m in Borehole S34; the factored axial geotechnical resistance at ULS and the axial geotechnical resistance at SLS (for 25 mm of settlement) are given below:

Foundation Unit	Founding Stratum	Factored Geotechnical Resistance at ULS	Geotechnical Resistance at SLS
North and South Abutments	Hard Clayey Silt Till	1,000 kN	800 kN
North and South Piers	Hard Clayey Silt Till	800 kN	700 kN

Higher pile capacities may be achieved by driving the piles deeper to below Elevation 159 m, subject to the results of further investigation at the foundation elements during detail design. For preliminary design purposes, it is anticipated that a factored axial geotechnical resistance at ULS of about 1,800 kN and an axial geotechnical resistance at SLS (for 25 mm of settlement) of about 1,500 kN would apply at below that elevation.

At the proposed north and south abutment area it is estimated that between 110 mm and 150 mm of settlement will occur, primarily in the firm to stiff surficial clayey silt under the proposed loading from the approach embankment. For preliminary design purposes it is recommended that a downdrag load of 300 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 silty clay 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 for support of the foundation elements for the overpass structures. Preliminary geotechnical recommendations for caisson foundations are provided in the sub-sections that follow.

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.



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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 elevation given in Section 6.2.4.1.

Foundation Unit	Founding Stratum	Caisson Diameter	Factored Geotechnical Resistances at ULS	Geotechnical Resistances at SLS
Pier	Hard Clayey Silt Till	0.9 m	2,400 kN	1,900 kN
		1.2 m	4,100 kN	3,400 kN
		1.5 m	6,400 kN	5,300 kN
Abutments	Hard Clayey Silt Till	0.9 m	3,400 kN	2,800 kN
		1.2 m	6,000 kN	5,000 kN
		1.5 m	9,300 kN	7,700 kN

At the proposed north and south abutment areas, it is estimated that about 110 mm to 150 mm of settlement will occur, primarily in the stiff clayey silt till between 4 m and 11 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	750 kN
1.2 m	1,000 kN
1.5 m	1,250 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 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 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. It should be noted that these design recommendations and parameters assume level backfill and ground surface behind the walls. Where there is sloping ground behind the walls, the coefficient of lateral earth pressure must be adjusted to account for the slope.

- Select, free draining granular fill meeting the specifications of Ontario Provincial Standard Specifications (OPSS) Granular 'A' or Granular 'B' Type II but with less than 5 percent passing the 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 OPSS 3101.150 and OPSS 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 OPSS 501.06. 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 or rock fill:

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

- For Case B, the pressures are based on the rock fill as placed or on the granular fill as placed and the following parameters (unfactored) may be assumed:



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

If the wall support and superstructure allow lateral yielding of the stem, active earth pressures may 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)
		<ul style="list-style-type: none"> • taken as soil unit weights given above for fill materials • taken as 20 kN/m³ for the native materials
	d	is the depth below the top of the wall (m); and
	H	is the height of the wall above the toe (m).

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

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



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SEISMIC ACTIVE PRESSURE COEFFICIENTS, K_{AE}

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

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

6.4 Approach Embankments

The construction of the Major Mackenzie Drive overpass structure will require placement of up to about 9.6 m of fill within the limits of the south approach and up to about 8.2 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 stiff to very stiff silty clay 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 any softened and/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 anticipated "reworked" soil 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 adequate for the embankments at this site. The stability analyses



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were performed to check that the target minimum factor of safety was achieved for the proposed embankment heights and geometries.

A groundwater table elevation of approximately 198.8 m was used in the model for the approach embankment areas. 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.

Soil Type	Unit Weight (kN/m ³)	Undrained Shear Strength (kPa)	Cohesion, c' (kPa)	Angle of Internal Friction, ϕ' (degrees)
New Earth Fill (sand and gravel)	21	--	--	34
Stiff to Very Stiff Silty Clay Till	20	100 kPa	--	30
Stiff Silty Clay to Clayey Silt Till	20	80 kPa	--	28
Hard Clayey Silt Till	21	150 kPa	--	34

With appropriate subgrade preparation and proper placement and compaction of embankment fill materials, the proposed 8.2 m to 9.6 m high approach embankments with side slopes maintained at 2H:1V will have a factor of safety of greater than 1.3 against deep-seated slope instability 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 1.1. 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 8.2 m to 9.6 m high approach embankment fill itself is expected to be less than about 25 mm, and this settlement will occur relatively quickly during and immediately following construction.

The settlement of the foundation soils under the approach embankment loading is anticipated to be about 110 mm to 150 mm, the majority of which will occur within the "stiff" zone of the silty clay to 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 from the oedometer test have been used in conjunction with Atterberg limits, undrained vane shear strength data, and experience with similar soil types in the Peel Plain.



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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	--	--
Stiff Silty Clay to Clayey Silt Till	20	15 MPa	Cc = 0.215 Cr = 0.034
Hard Silty Clay to Clayey Silt Till	21	100 MPa	--

Settlement mitigation measures will be required to accommodate the predicted 110 mm to 150 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 bridge foundation elements is finalized and permission to enter has been obtained, 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 silty clay to clayey silt till between depths of 4 m and 11 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, excavation for the overpass structure foundations will extend to depths of up to 1.5 m below existing ground surface and it is anticipated that they will be made through stiff silty clay, which is considered Type 3 soil according to Occupational Health and Safety Act and Regulation for Construction Projects (OHSA). The excavation in the overburden should be carried out with side slopes no steeper than 1 horizontal to 1 vertical (1H:1V).

All excavations must be carried out in accordance with the latest edition of the OHSA.



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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 6.4 m below ground surface. It is therefore 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

Although there was no evidence of cobbles and/or boulders in the boreholes drilled at this site it should be anticipated that cobbles and/or boulders may be encountered within the till deposits, and this 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. a geotechnical engineer with Golder, with input by Ms. Lisa Coyne, P. Eng., a geotechnical engineer and Associate with Golder. Mr. Fin Heffernan, P. Eng., Golder's Designated MTO Contact for this project, conducted an independent quality control review of the report.



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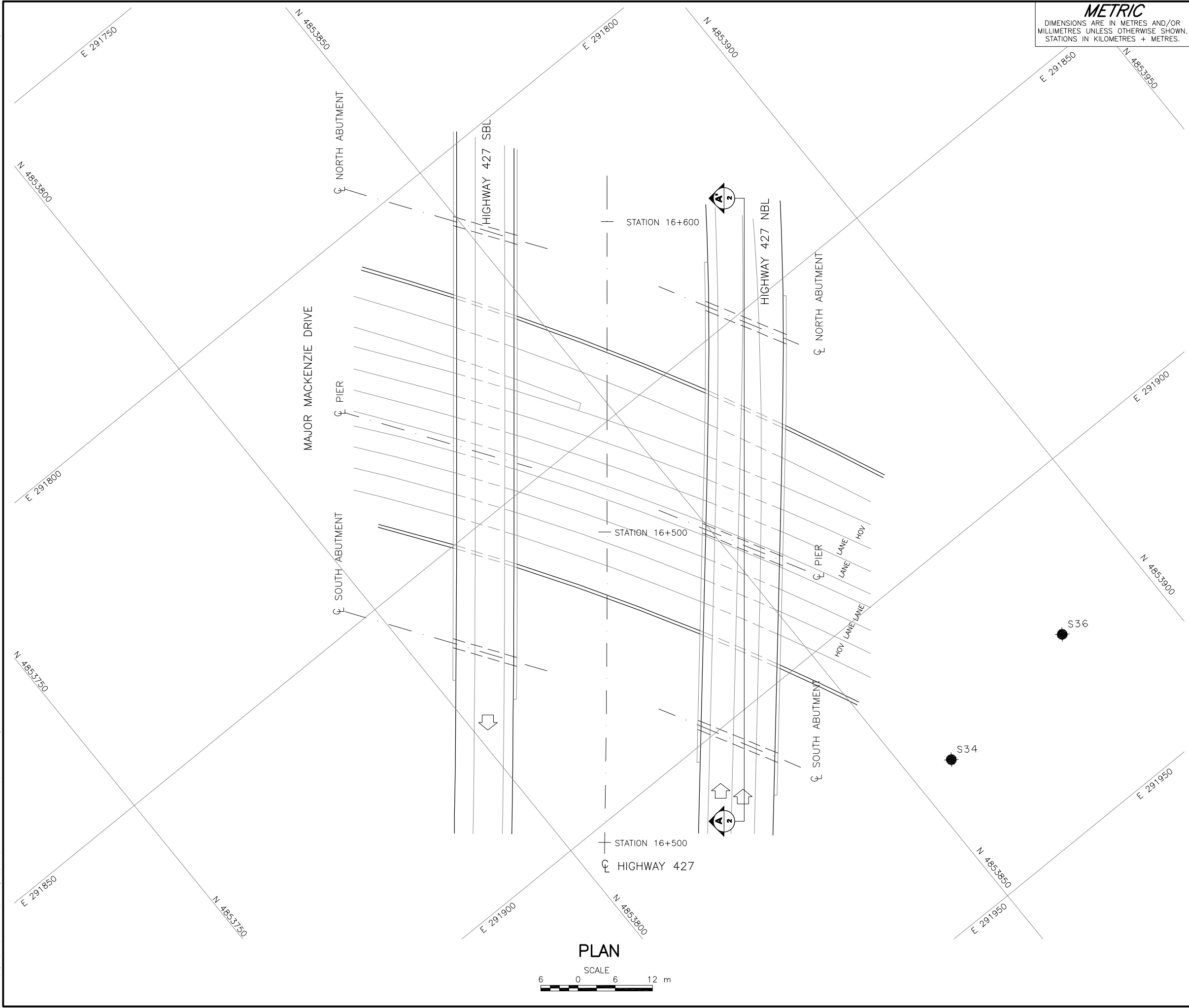
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TABLE 1
COMPARISON OF FOUNDATION ALTERNATIVES – MAJOR MACKENZIE DRIVE OVERPASS STRUCTURE
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Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Spread Footings on very stiff silty clay till	Feasible for support of piers	<ul style="list-style-type: none"> Relative ease of construction Negligible post-construction settlement 	<ul style="list-style-type: none"> Lowest bearing capacities of the four options 	<ul style="list-style-type: none"> Lower relative cost than piled foundations 	<ul style="list-style-type: none"> 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	<ul style="list-style-type: none"> Negligible post-construction settlement provided that preloading of approach embankment areas is completed; and, Construction maintained above groundwater level 	<ul style="list-style-type: none"> Embankment preloading must be taken into account in construction schedule 	<ul style="list-style-type: none"> Low cost option; and, Anticipated subexcavation of 1 m of surficial soils required within loading footprint for compacted Granular A pad 	<ul style="list-style-type: none"> 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 hard clayey silt till	Feasible for support of abutments and piers	<ul style="list-style-type: none"> 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 	<ul style="list-style-type: none"> Downdrag loading must be taken into account in design, unless embankment areas are fully preloaded prior to construction of the overpass structures; and, Cobbles /boulders were not encountered during drilling however given the nature of till it should be anticipated that they may be encountered during pile driving 	<ul style="list-style-type: none"> More costly than spread footings; and, Installation cost could be impacted by presence of obstructions, if encountered 	<ul style="list-style-type: none"> Low risk of post-construction settlement of overpass structure, or of differential settlement of foundation elements; and, Low risk of encountering obstructions that could impact pile installation
Caisson foundations founded within hard clayey silt till	Feasible for support of abutments and piers	<ul style="list-style-type: none"> Sub-excavation is not required, Highest bearing capacity, Negligible post-construction settlement; and, Can be used for support of conventional or semi-integral abutments 	<ul style="list-style-type: none"> Caissons may encounter obstructions (cobbles and boulders), see discussion above and, Downdrag loading must be taken into account in design, unless embankment areas are fully preloaded prior to construction of the overpass structures. 	<ul style="list-style-type: none"> Additional cost associated with specialised drilling equipment and temporary or permanent liners; and, More costly option than steel H-piles 	<ul style="list-style-type: none"> Low risk of post-construction settlement of overpass structure, or of differential settlement of foundation elements; and, Low risk of encountering obstructions that could impact caisson installation

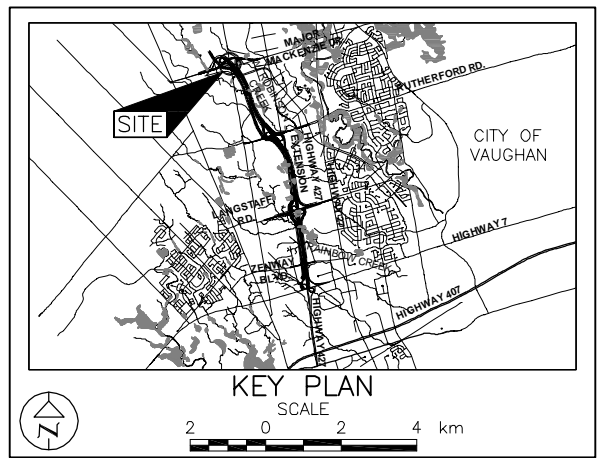


METRIC
DIMENSIONS ARE IN METRES AND/OR
MILLIMETRES UNLESS OTHERWISE SHOWN.
STATIONS IN KILOMETRES + METRES.

CONT No.
WO No. 05-20012

HIGHWAY 427 EXTENSION
MAJOR MACKENZIE DRIVE OVERPASSES
BOREHOLE LOCATIONS

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



LEGEND			
	Borehole - Current Investigation		
	Seal		
	Piezometer		
N	Standard Penetration Test Value		
16	Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)		
	WL in piezometer, measured on May 13, 2009		
	WL upon completion of drilling		
No.	ELEVATION	CO-ORDINATES	
		NORTHING	EASTING
S34	205.2	4853856.8	291927.3
S36	205.2	4853883.4	291922.9

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.

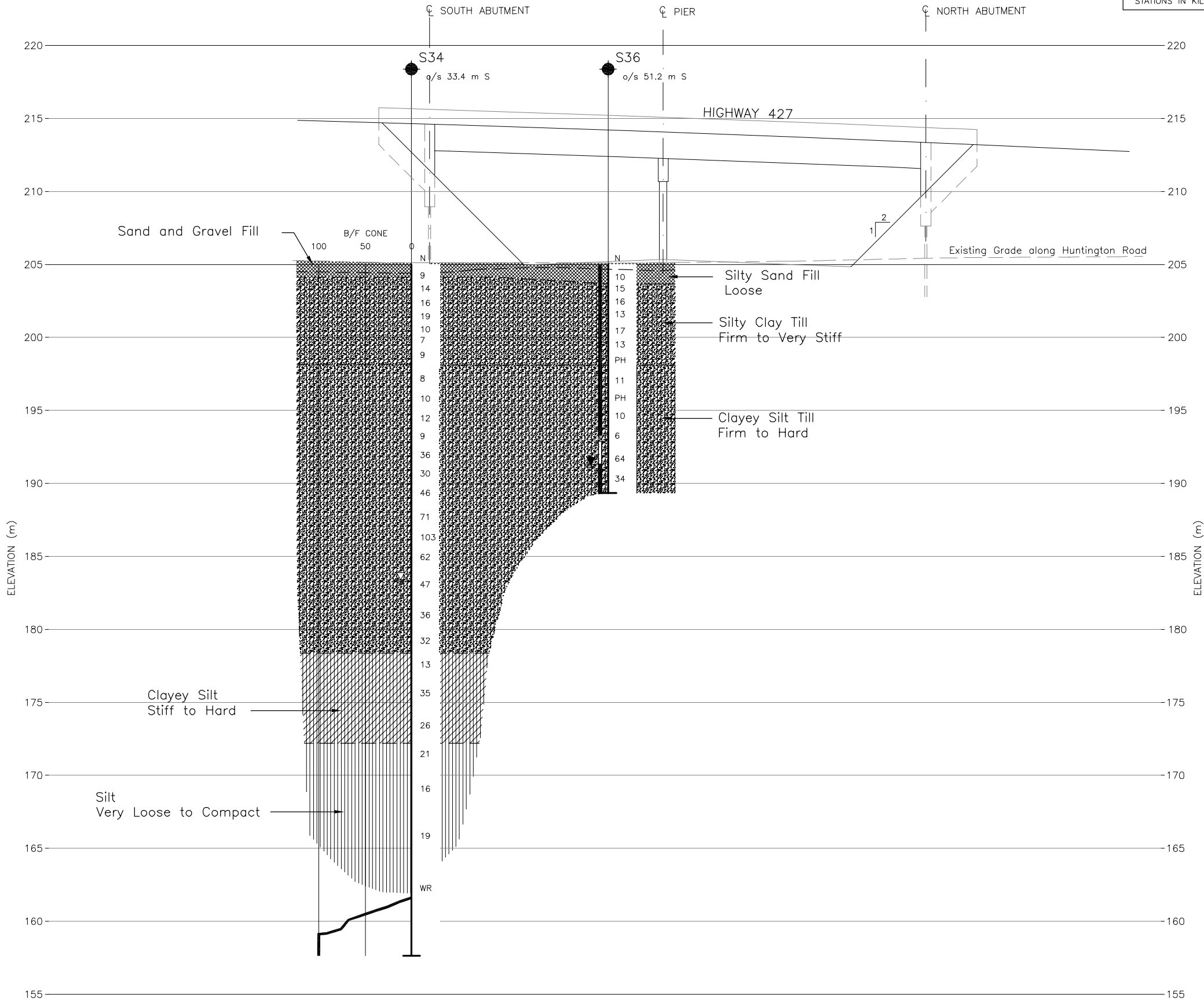
The complete Preliminary Foundation Investigation and Design Report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with Section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

REFERENCE

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



NO.	DATE	BY	REVISION
Geocres No. 30M13-174			
HWY. 427		PROJECT NO.06-1111-012-2	
SUBM'D. PKS	CHKD. SMM	DATE: 4-Aug-2009	SITE:
DRAWN: JFC	CHKD. MWK	APPD. LCC	DWG. 1



PROFILE A-A' MAJOR MACKENZIE DRIVE OVERPASSES

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

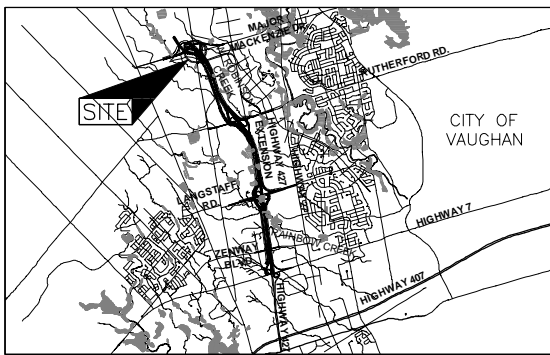
CONT No.
WO No. 05-20012

HIGHWAY 427 EXTENSION
MAJOR MACKENZIE DRIVE OVERPASSES
SOIL STRATA

SHEET



Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



KEY PLAN
SCALE
0 2 4 km

LEGEND

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

No.	ELEVATION	CO-ORDINATES	
		NORTHING	EASTING
S34	205.2	4853856.8	291927.3
S36	205.2	4853883.4	291922.9

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.

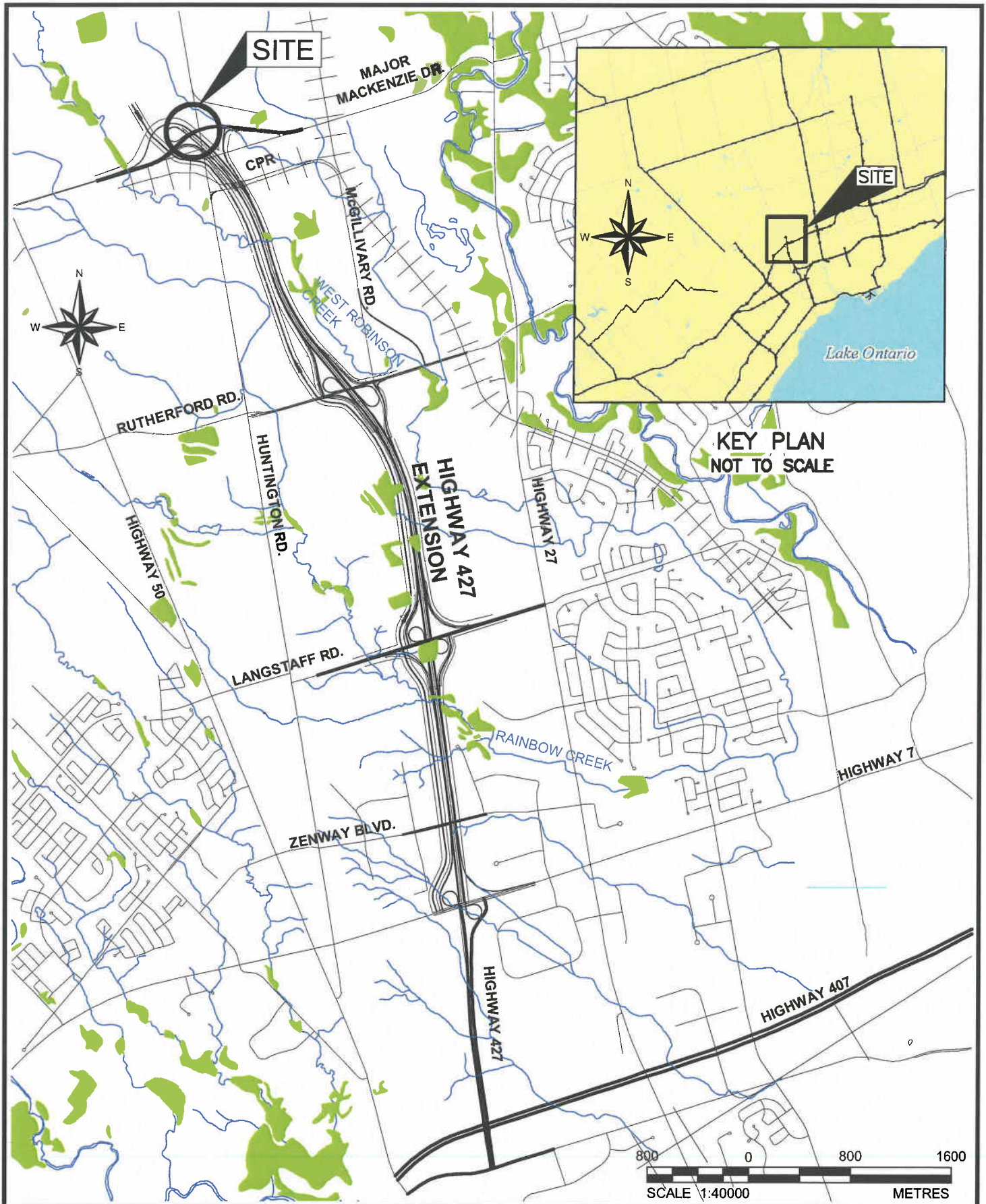
REFERENCE

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



NO.	DATE	BY	REVISION
Geocres No. 30M13-174			
HWY. 427	PROJECT NO.06-1111-012-2		
SUBM'D. PKS	CHKD. SMM	DATE: 5-Aug-2009	SITE:
DRAWN: JFC	CHKD. MWK	APPD. LCC	DWG. 2

PLOT DATE: August 5, 2009
 FILENAME: T:\Projects\2006\06-1111-012 (MRC, Vaughan)\-EB- (MAJOR MACKENZIE OVERPASSES)\061111012EB0F1.dwg



SCALE	AS SHOWN
DATE	Aug. 5, 2009
DESIGN	PKS
CAD	JFC
CHECK	MWK
REVIEW	<i>SM</i> SMM

TITLE

SITE LOCATION PLAN MAJOR MACKENZIE OVERPASS

FILE No. 061111012EB0F1.dwg

PROJECT No. 06-1111-012-2

REV. B

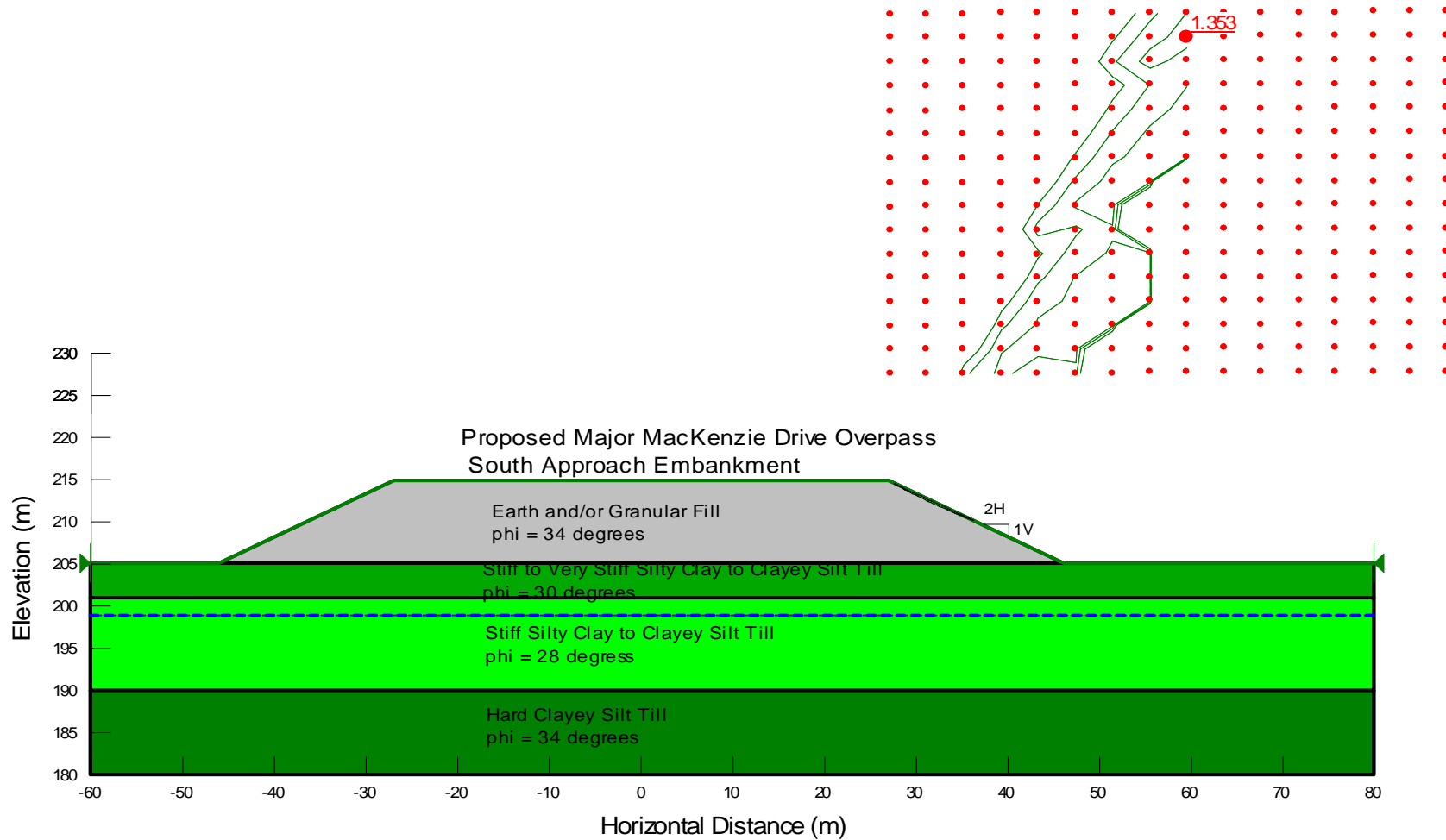
HIGHWAY 427 EXTENSION

FIGURE

1

HIGHWAY 427 EXTENSION - MAJOR MACKENZIE DRIVE OVERPASS SOUTH APPROACH EMBANKMENT- STATIC GLOBAL STABILITY - DRAINED ANALYSIS

FIGURE 2



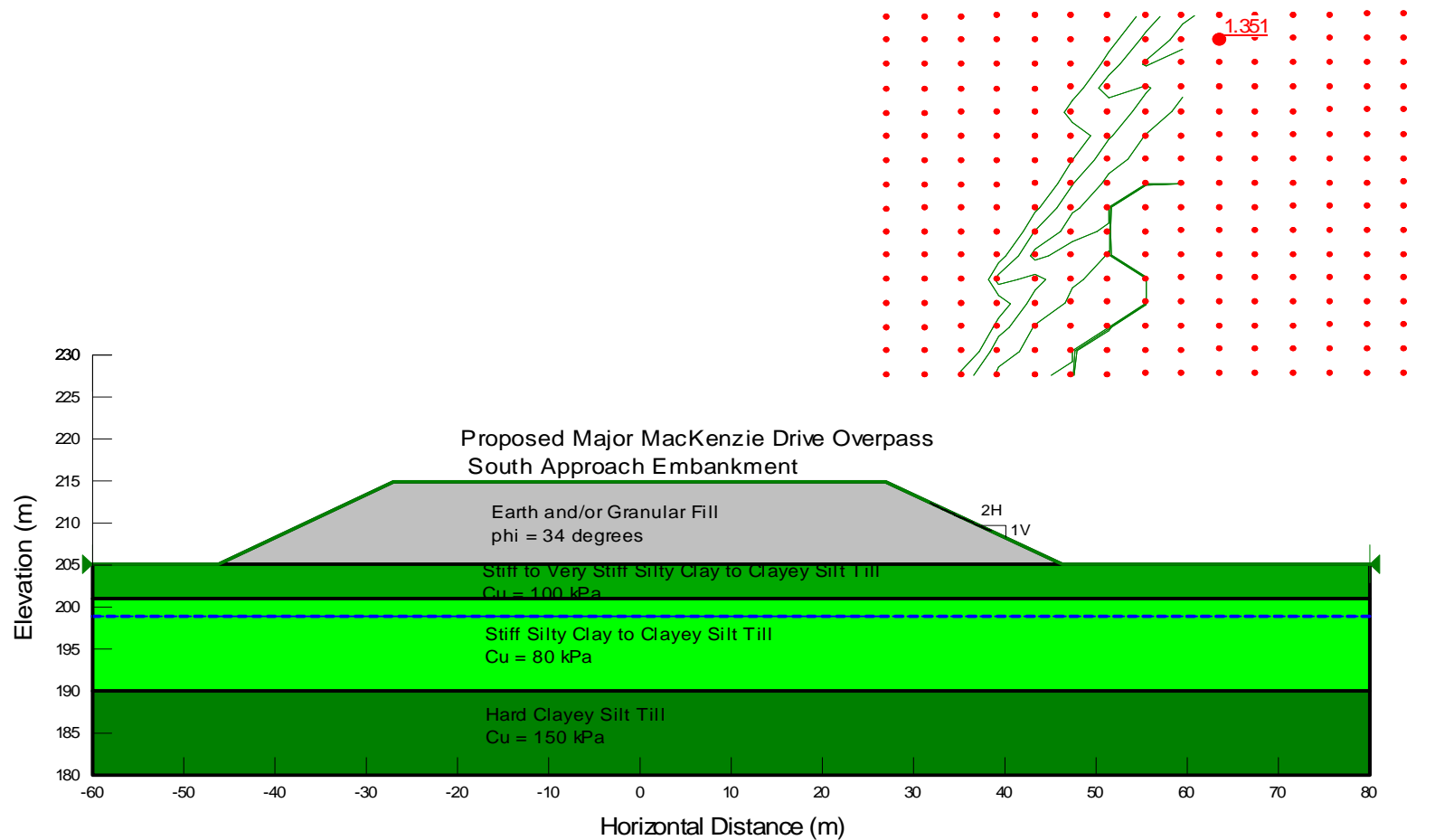
Date: June 2009
Project: 06-1111-012-2

Golder Associates

Drawn: SMM
Checked: LCC

**HIGHWAY 427 EXTENSION - MAJOR MACKENZIE DRIVE OVERPASS
SOUTH APPROACH EMBANKMENT- STATIC GLOBAL STABILITY - UNDRAINED ANALYSIS**

FIGURE 3



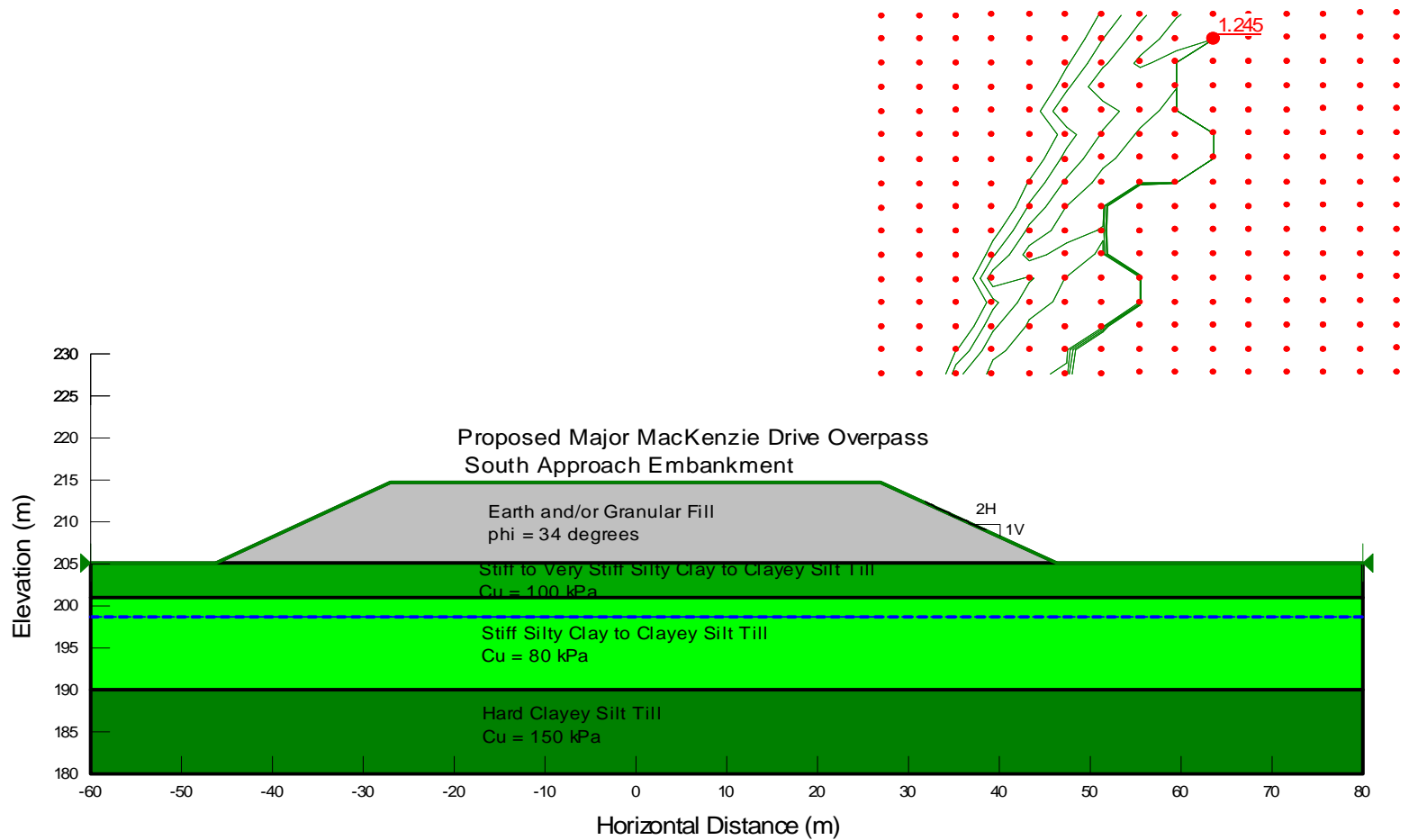
Date: June 2009
Project: 06-1111-012-2

Golder Associates

Drawn: SMM
Checked: LCC

HIGHWAY 427 EXTENSION - MAJOR MACKENZIE DRIVE OVERPASS SOUTH APPROACH EMBANKMENT- SEISMIC GLOBAL STABILITY

FIGURE 4



Date: June 2009
Project: 06-1111-012-2

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

II. STRESS AND STRAIN

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

III. SOIL PROPERTIES

(a) Index Properties

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

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

(a) Index Properties (continued)

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

(b) Hydraulic Properties

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

(c) Consolidation (one-dimensional)

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

(d) Shear Strength

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

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



LIST OF ABBREVIATIONS

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

I. SAMPLE TYPE

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

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

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

Dynamic Cone Penetration Resistance; N_d :

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

PH:	Sampler advanced by hydraulic pressure
PM:	Sampler advanced by manual pressure
WH:	Sampler advanced by static weight of hammer
WR:	Sampler advanced by weight of sampler and rod

Piezo-Cone Penetration Test (CPT)

A electronic cone penetrometer with a 60° conical tip and a project end area of 10 cm² pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (Q_t), porewater pressure (PWP) and friction along a sleeve are recorded electronically at 25 mm penetration intervals.

III. SOIL DESCRIPTION

(a) Cohesionless Soils

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

(b) Cohesive Soils Consistency

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

IV. SOIL TESTS

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

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

1 OF 4 METRIC

ORIGINATED BY SB

COMPILED BY VA

CHECKED BY _____ SM _____

Continued Next Page

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

PROJECT <u>06-1111-012</u>		RECORD OF BOREHOLE No S34		2 OF 4 METRIC	
W.O. <u>05-20012</u>		LOCATION <u>N 4853856.8 , E 291927.3</u>		ORIGINATED BY <u>SB</u>	
DIST <u>Central</u> HWY <u>427</u>		BOREHOLE TYPE <u>200 mm Outside Diameter Hollow Stem Augers</u>		COMPILED BY <u>VA</u>	
DATUM <u>Geodetic</u>		DATE <u>March 10 & 11, 2009</u>		CHECKED BY <u>SMM</u>	



SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)	
								○ UNCONFINED ● QUICK TRIAXIAL	+ FIELD VANE × REMOULDED						20 40 60 80 100	10 20 30
— CONTINUED FROM PREVIOUS PAGE —																
15.0	CLAYEY SILT, some to with sand, trace gravel (TILL) Hard Grey Moist		13	SS	30	▽	190									
							189									
			14	SS	46		188									
							187									
			15	SS	71		186									
							185								7 28 42 23	
			16	SS	103		184									
							183									
			17	SS	62		182									
							181									
		18	SS	47	180											
					179								0 3 72 25			
		19	SS	36	178											
					177											
		20	SS	32	176											
178.4	CLAYEY SILT, trace sand Stiff to hard Grey Moist to wet															
26.8			21	SS	13									0 3 83 17		

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

Continued Next Page

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

PROJECT <u>06-1111-012</u>		RECORD OF BOREHOLE No S34		3 OF 4 METRIC	
W.O. <u>05-20012</u>		LOCATION <u>N 4853856.8 E 291927.3</u>		ORIGINATED BY <u>SB</u>	
DIST <u>Central</u> HWY <u>427</u>		BOREHOLE TYPE <u>200 mm Outside Diameter Hollow Stem Augers</u>		COMPILED BY <u>VA</u>	
DATUM <u>Geodetic</u>		DATE <u>March 10 & 11, 2009</u>		CHECKED BY <u>SMM</u>	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)	
--- CONTINUED FROM PREVIOUS PAGE ---								20 40 60 80 100	20 40 60 80 100						10 20 30	
172.2 33.0	CLAYEY SILT, trace sand Stiff to hard Grey Moist to wet		22	SS	35		175							0 0 81 19		
							174									
			23	SS	26		173									
							172									
	SILT, trace clay Compact Grey Wet		24	SS	21		171									
								170								
							169									
			25	SS	16		168									
							167									
							166									
			26	SS	19		165							0 0 89 11		
							164									
							163									
161.9 43.3	END OF BOREHOLE		27	SS	WR		162									
160.2	Dynamic Cone Penetration Test (DCPT) below a depth of 43.3 m						161									

Continued Next Page

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

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

PROJECT <u>06-1111-012</u>		RECORD OF BOREHOLE No S34		4 OF 4 METRIC	
W.O. <u>05-20012</u>		LOCATION <u>N 4853856.8 ; E 291927.3</u>		ORIGINATED BY <u>SB</u>	
DIST <u>Central</u> HWY <u>427</u>		BOREHOLE TYPE <u>200 mm Outside Diameter Hollow Stem Augers</u>		COMPILED BY <u>VA</u>	
DATUM <u>Geodetic</u>		DATE <u>March 10 & 11, 2009</u>		CHECKED BY <u>SMM</u>	


SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT <div><div><div>20406080100</div><div>20406080100</div></div><div><div>○ UNCONFINED</div><div>● QUICK TRIAXIAL</div></div><div><div>+ FIELD VANE</div><div>× REMOULDED</div></div></div>	PLASTIC LIMIT W _p NATURAL MOISTURE CONTENT W LIQUID LIMIT W _L WATER CONTENT (%) <div><div>102030</div></div>	UNIT WEIGHT <div>γ</div> kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
45.0	Dynamic Cone Penetration Test (DCPT) below a depth of 43.3 m						160				
							159				
							158				
157.7	END OF DCPT										
47.6	Refusal to Further Penetration										
	NOTES: 1. Water level noted inside augers at a depth of 21.8 m below ground surface (Elev. 183.4 m) on March 10, 2009. 2. A Dynamic Cone Penetration Test was carried out below a depth of 43.3 m . 3. Open borehole dry upon completion of drilling on March 11, 2009. 4. Borehole backfilled with bentonite.										

PROJECT 06-1111-012		RECORD OF BOREHOLE No S36		1 OF 2 METRIC	
W.O. 05-20012		LOCATION N 4853883.4 E 291922.9		ORIGINATED BY SB	
DIST Central HWY 427		BOREHOLE TYPE 200 mm Outside Diameter Hollow Stem Augers		COMPILED BY VA	
DATUM Geodetic		DATE March 12, 2009		CHECKED BY SMM <i>[Signature]</i>	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT		UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	W _p W W _L	10 20 30		
205.2	GROUND SURFACE												
-0.0	Asphalt												
0.2	Sand with gravel (FILL)												
	Silty sand, trace gravel, trace clay (FILL)												
	Compact Brown Moist		1	SS	10								
203.7													
1.5	SILTY CLAY, trace sand, trace gravel (TILL)		2	SS	15								
	Stiff to very stiff												
	Brown becoming grey below a depth of 2.3 m		3	SS	16								
	Wet		4	SS	13								
			5	SS	17								
			6	SS	13								
			7	TO	PH								
198.1													
7.1	CLAYEY SILT, some sand, trace gravel (TILL)		8	SS	11								
	Stiff to hard												
	Grey		9	TO	PH								
	Wet		10	SS	10								
			11	SS	6								
			12	SS	64								

Continued Next Page

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

PROJECT 06-1111-012			RECORD OF BOREHOLE No S36				2 OF 2 METRIC						
W.O. 05-20012			LOCATION N 4853883.4 ; E 291922.9				ORIGINATED BY SB						
DIST Central HWY 427			BOREHOLE TYPE 200 mm Outside Diameter Hollow Stem Augers				COMPILED BY VA						
DATUM Geodetic			DATE March 12, 2009				CHECKED BY SMM						
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT	PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES								
— CONTINUED FROM PREVIOUS PAGE —													
189.4 15.9	CLAYEY SILT, some sand, trace gravel (TILL) Stiff to hard Grey Wet END OF BOREHOLE NOTES: 1. A 50 mm diameter monitoring well was installed at a depth of 13.9 m (Elev. 191.3 m). Water level measurements: Date Depth Elev. On Completion Dry April 27, 2009 7.3 m 197.9 m May 25, 2009 6.4 m 198.8 m June 15, 2009 6.2 m 199.0 m July 09, 2009 6.2 m 199.0 m		13	SS	34		190						0 8 68 24



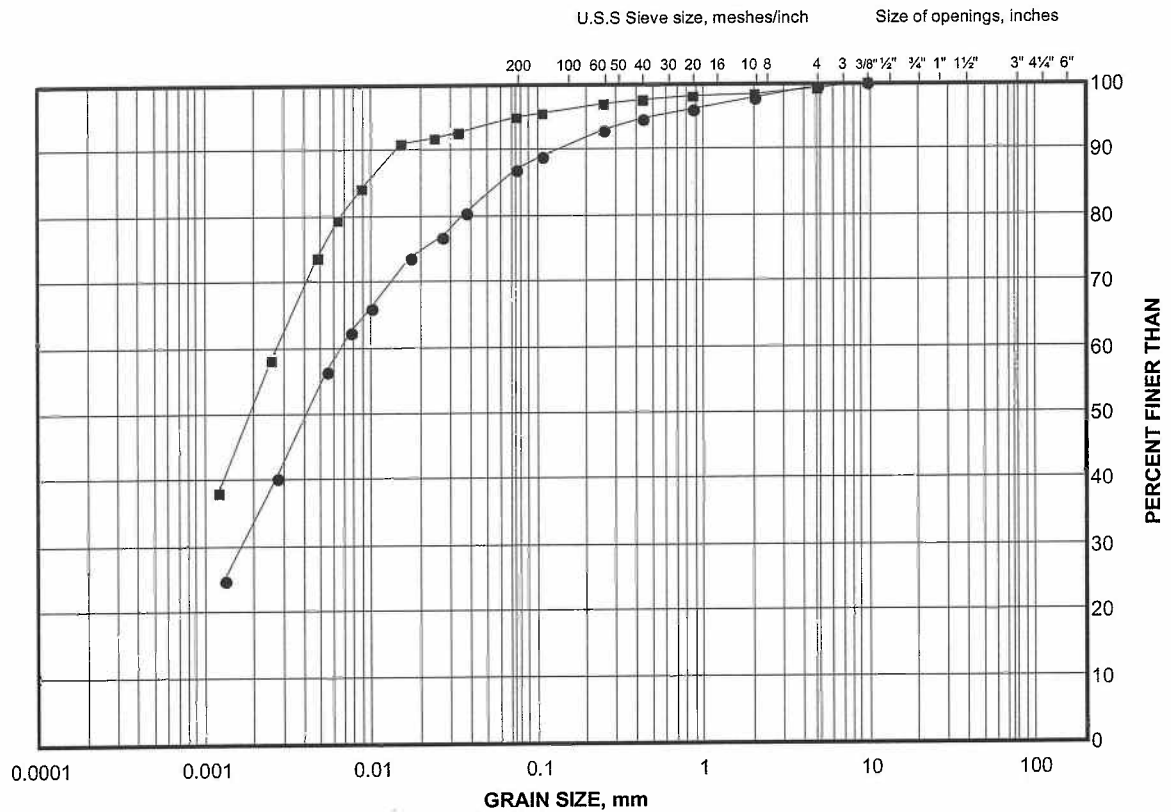
APPENDIX B

Laboratory Test Results

GRAIN SIZE DISTRIBUTION TEST RESULTS

Silty Clay Till

FIGURE B1



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	S34	3	202.6
■	S36	7	198.8

Project Number: 06-1111-012-2

Checked By: SMU

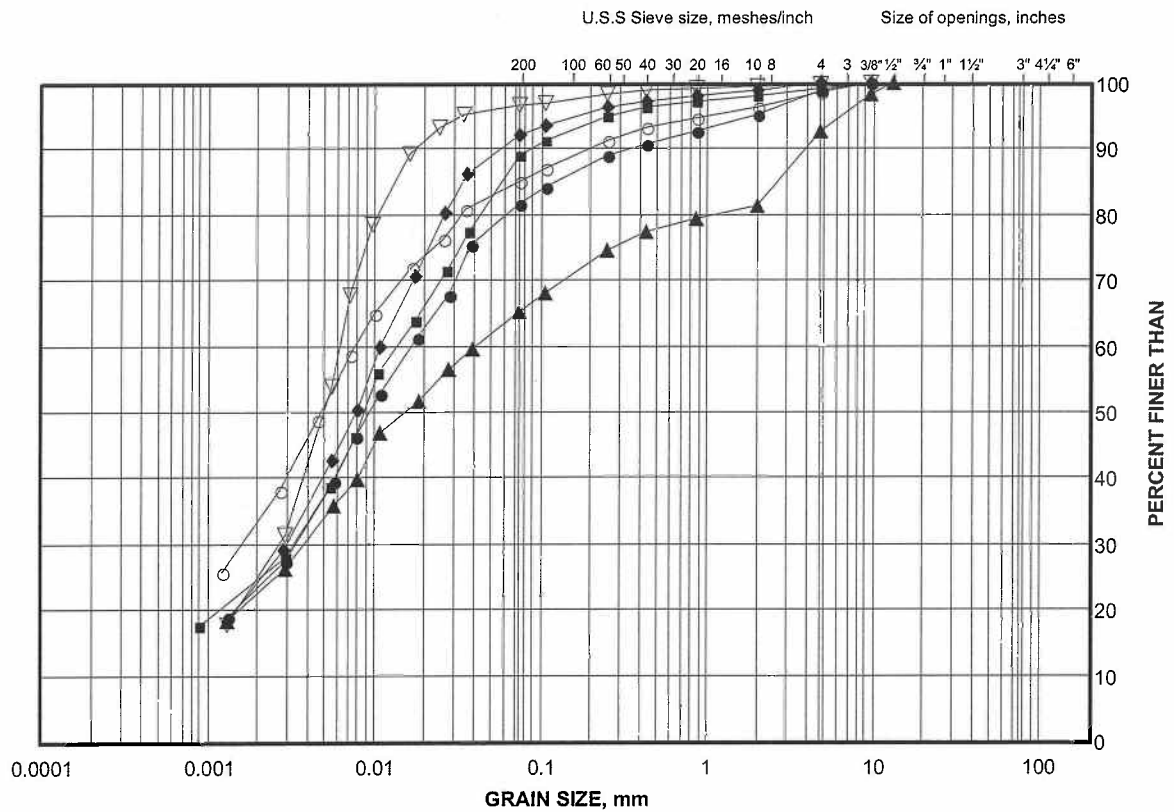
Golder Associates

Date: 29-May-09

GRAIN SIZE DISTRIBUTION TEST RESULTS

Clayey Silt Till

FIGURE B2



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

LEGEND

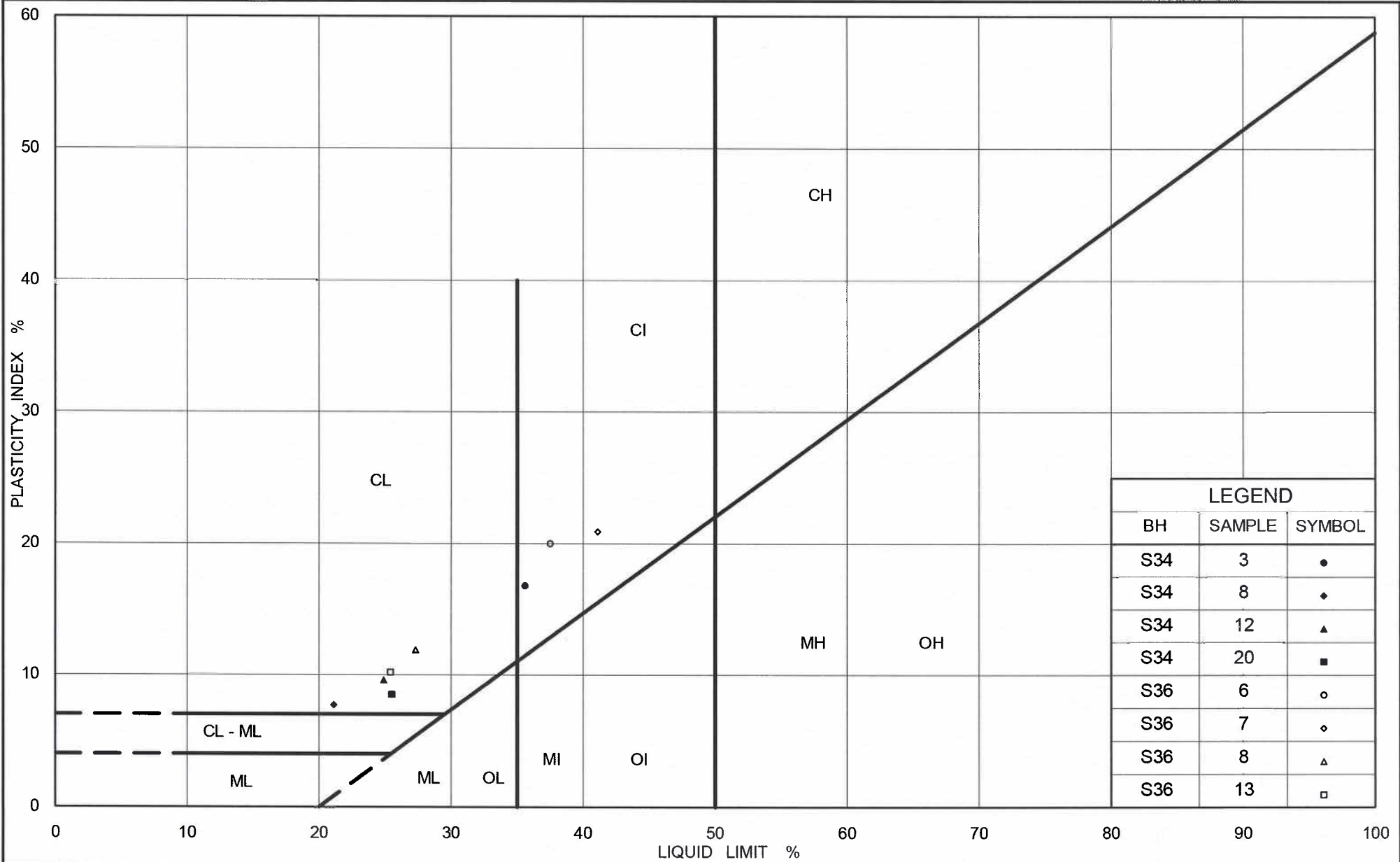
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	S36	12	191.2
■	S34	12	191.2
◆	S36	13	189.7
▲	S34	16	185.1
▽	S34	20	179.0
○	S36	8	197.3

Project Number: 06-1111-012-2

Checked By: *sm*

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PLASTICITY CHART Silty Clay to Clayey Silt Till

Figure No. B3

Project No. 06-1111-012-2

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OEDOMETER CONSOLIDATION SUMMARY**FIGURE B4****Silty Clay Till****(Sheet 1 of 4)****SAMPLE IDENTIFICATION**

Project Number	06-1111-012-2	Sample Number	7
Borehole Number	S36	Sample Depth, m	6.1-6.7

TEST CONDITIONS

Test Type	Standard	Load Duration, hr	24
Oedometer Number	3		
Date Started	03/27/2009		
Date Completed	04/10/2009		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	2.54	Unit Weight, kN/m ³	20.23
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m ³	16.33
Area, cm ²	31.62	Specific Gravity, measured	2.77
Volume, cm ³	80.28	Solids Height, cm	1.526
Water Content, %	23.94	Volume of Solids, cm ³	48.25
Wet Mass, g	165.65	Volume of Voids, cm ³	32.03
Dry Mass, g	133.65	Degree of Saturation, %	99.9

TEST COMPUTATIONS

Pressure kPa	Corr. Height cm	Void Ratio	Average Height cm	t ₉₀ sec	cv, cm ² /s	mv m ² /kN	k cm/s
0.00	2.539	0.664	2.539				
4.70	2.529	0.657	2.534	1	1.36E+00	8.38E-04	1.12E-04
9.55	2.515	0.648	2.522	2	6.74E-01	1.18E-03	7.78E-05
19.44	2.514	0.647	2.514	12	1.12E-01	3.98E-05	4.36E-07
38.70	2.508	0.644	2.511	9	1.48E-01	1.15E-04	1.67E-06
77.55	2.495	0.635	2.501	9	1.47E-01	1.31E-04	1.89E-06
154.93	2.476	0.623	2.485	8	1.64E-01	9.72E-05	1.56E-06
309.49	2.448	0.604	2.462	8	1.61E-01	7.21E-05	1.13E-06
619.14	2.403	0.575	2.425	23	5.42E-02	5.70E-05	3.03E-07
1239.29	2.323	0.522	2.363	60	1.97E-02	5.09E-05	9.85E-08
2500.36	2.229	0.460	2.276	29	3.79E-02	2.94E-05	1.09E-07
1239.29	2.237	0.466	2.233				
309.49	2.294	0.503	2.265				
77.50	2.352	0.541	2.323				
19.44	2.403	0.575	2.378				
4.76	2.436	0.597	2.420				

Note:

k calculated using cv based on t₉₀ values.**SAMPLE DIMENSIONS AND PROPERTIES - FINAL**

Sample Height, cm	2.44	Unit Weight, kN/m ³	21.02
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m ³	17.01
Area, cm ²	31.62	Specific Gravity, measured	2.77
Volume, cm ³	77.04	Solids Height, cm	1.526
Water Content, %	23.55	Volume of Solids, cm ³	48.25
Wet Mass, g	165.13	Volume of Voids, cm ³	28.79
Dry Mass, g	133.65		

Prepared By: LH

Golder Associates

Checked By: MM

OEDOMETER CONSOLIDATION SUMMARY

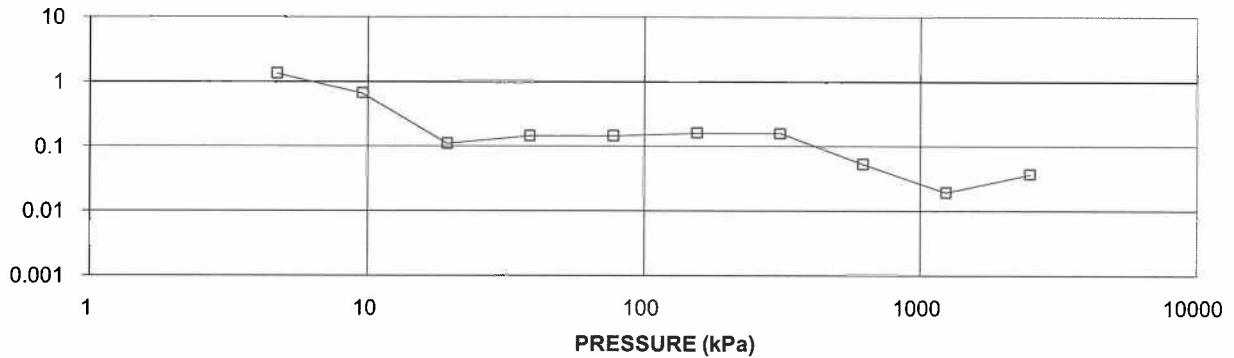
Silty Clay Till

FIGURE B4

(Sheet 2 of 4)

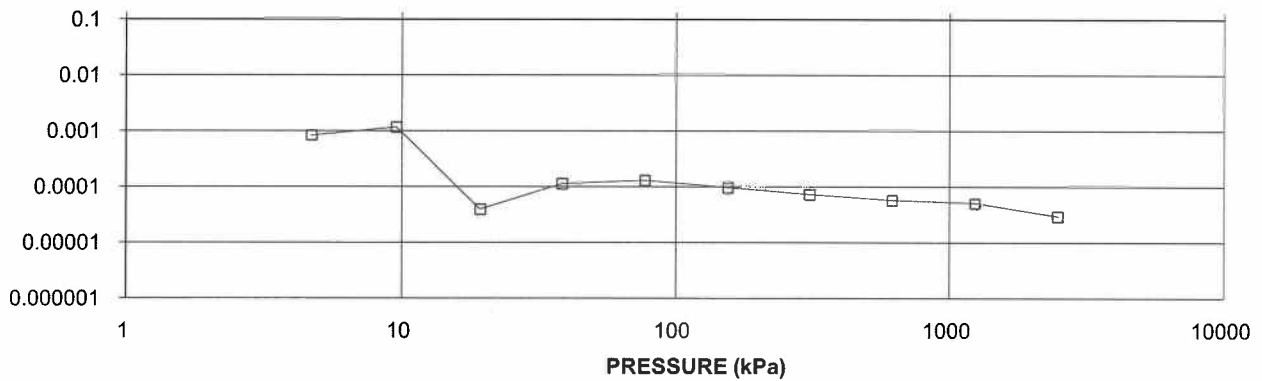
COEFFICIENT OF CONSOLIDATION,
cm²/s

CONSOLIDATION TEST
CV cm²/s VS PRESSURE (kPa)
BH S36 SA 7



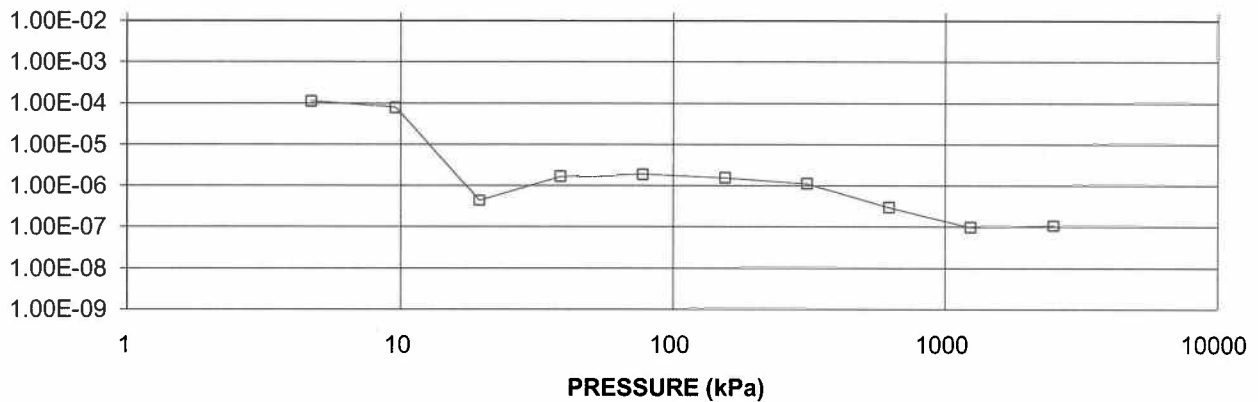
VOLUME COMPRESSIBILITY, m²/kN

CONSOLIDATION TEST
MV m²/kN vs PRESSURE (kPa)
BH S36 SA 7



HYDRAULIC CONDUCTIVITY,
cm/s

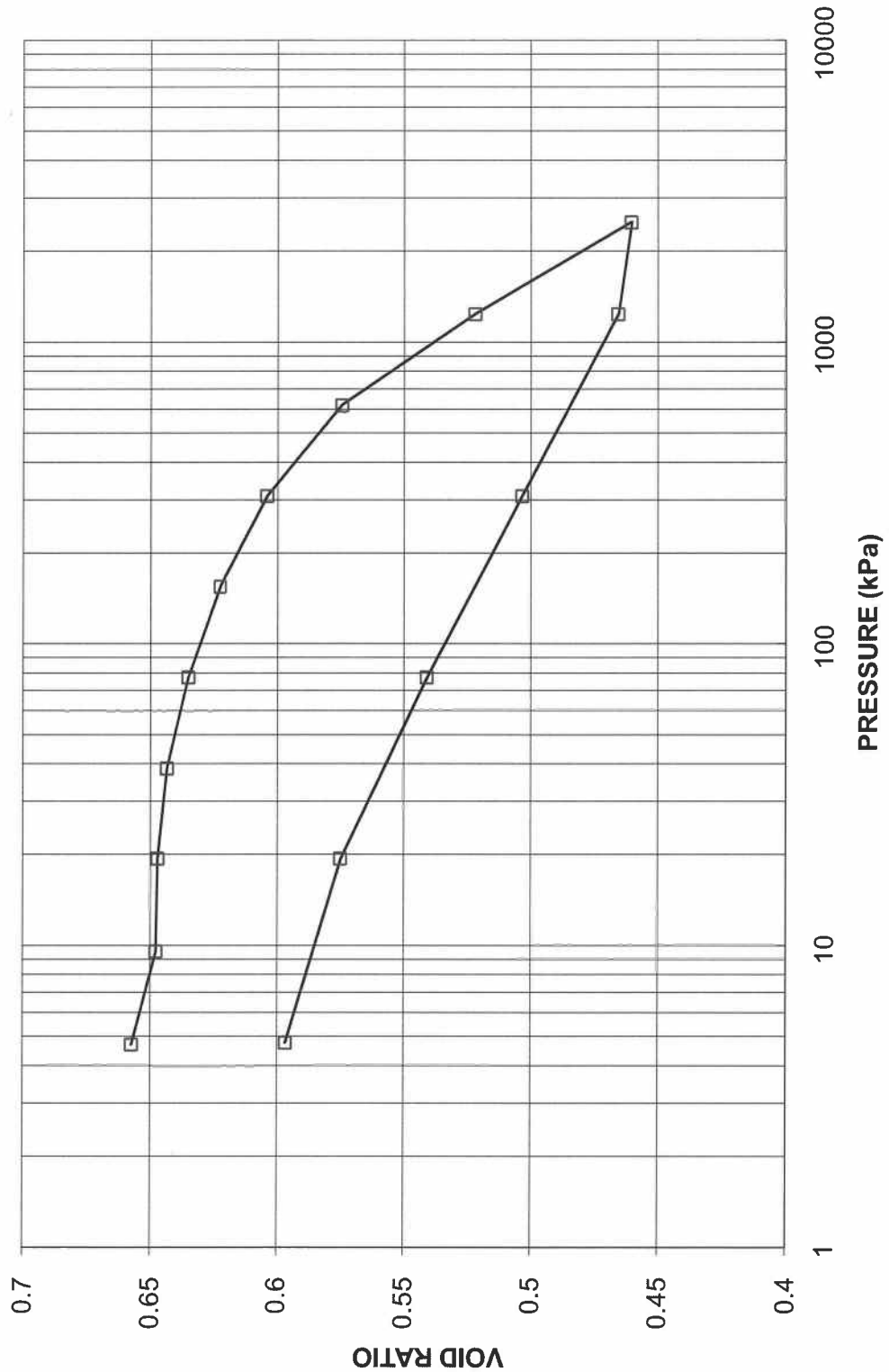
CONSOLIDATION TEST
HYDRAULIC CONDUCTIVITY vs PRESSURE
BH S36 SA 7



**CONSOLIDATION TEST
VOID RATIO VS. LOG PRESSURE
Silty Clay Till**

**FIGURE B4
(Sheet 3 of 4)**

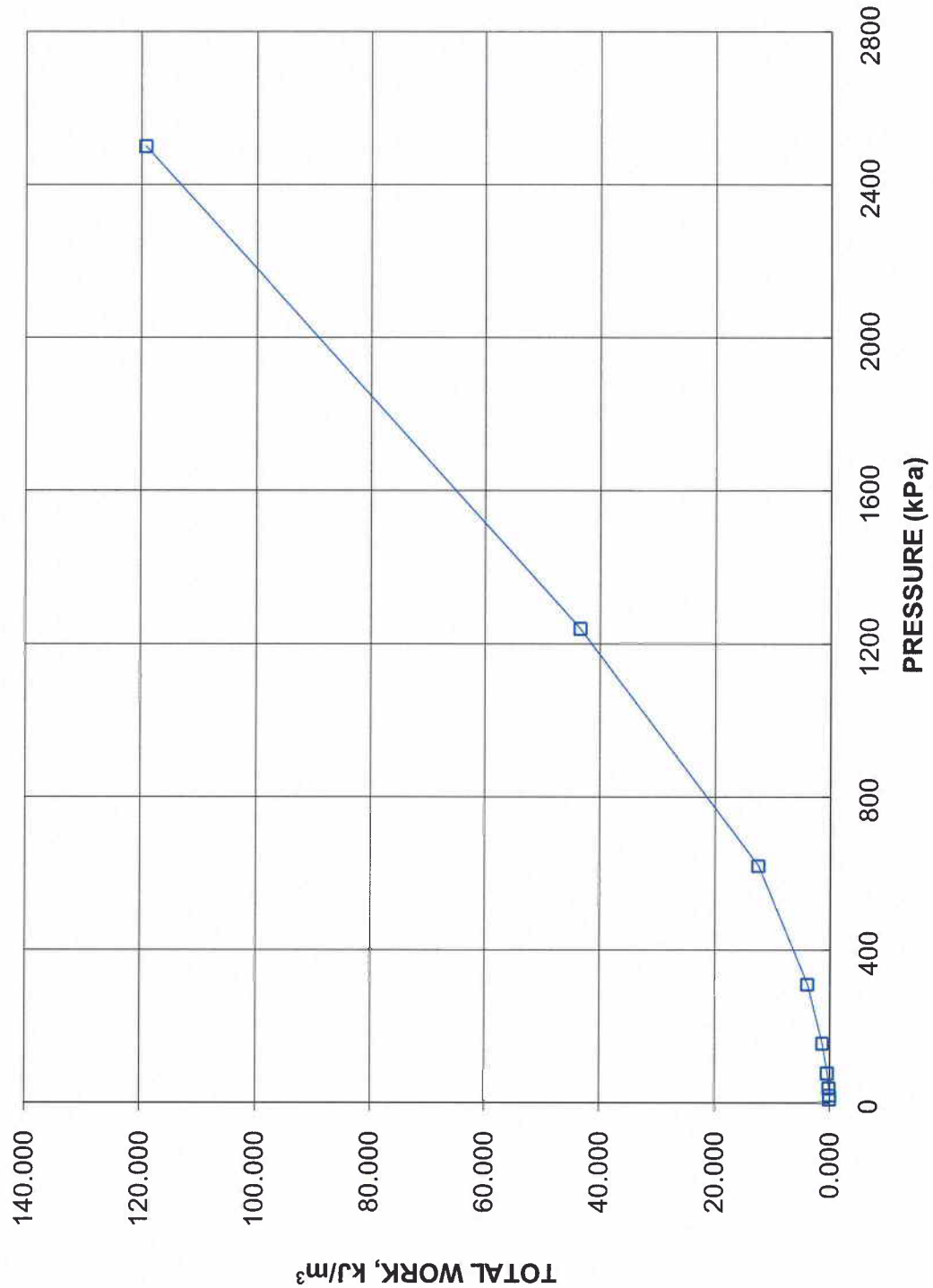
**CONSOLIDATION TEST
VOID RATIO vs PRESSURE
BH S36 SA 7**



**CONSOLIDATION TEST
TOTAL WORK VS. PRESSURE
Silty Clay Till**

**FIGURE B4
(Sheet 4 of 4)**

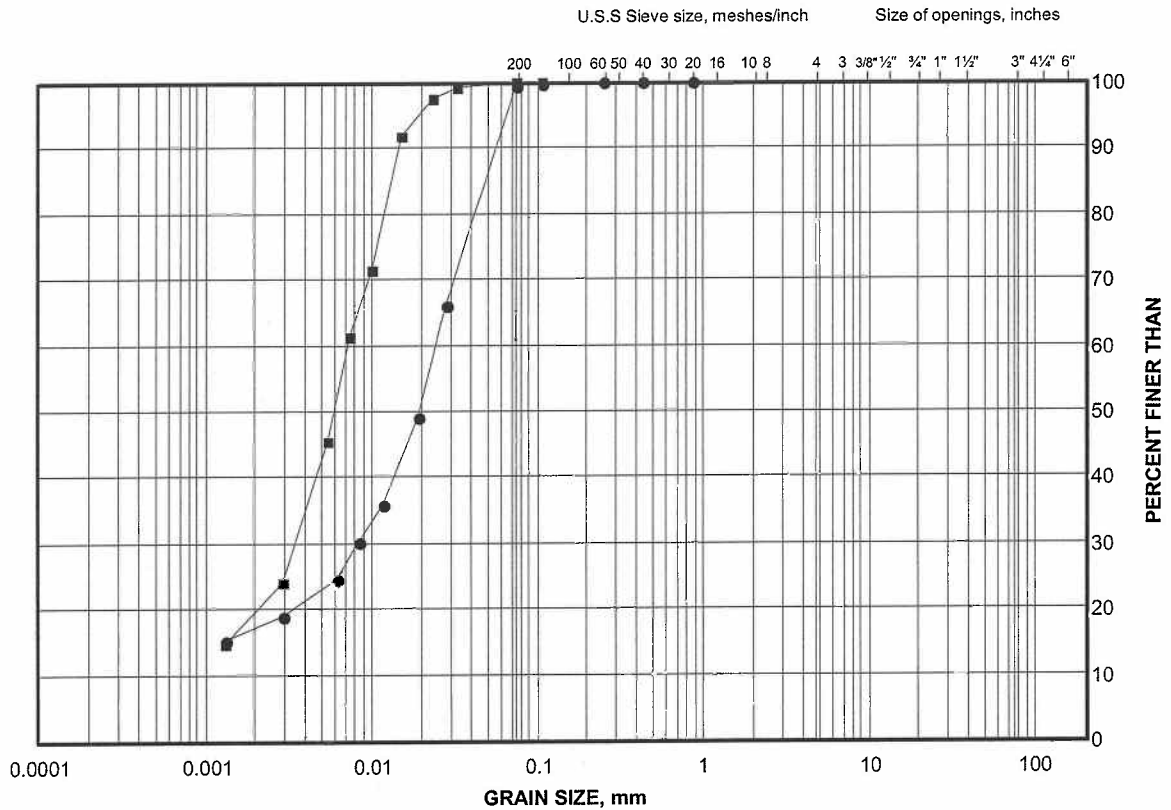
**CONSOLIDATION TEST
TOTAL WORK, kJ/m^3 vs PRESSURE
BH S36 SA 7**



GRAIN SIZE DISTRIBUTION TEST RESULTS

Clayey Silt

FIGURE B5



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

LEGEND

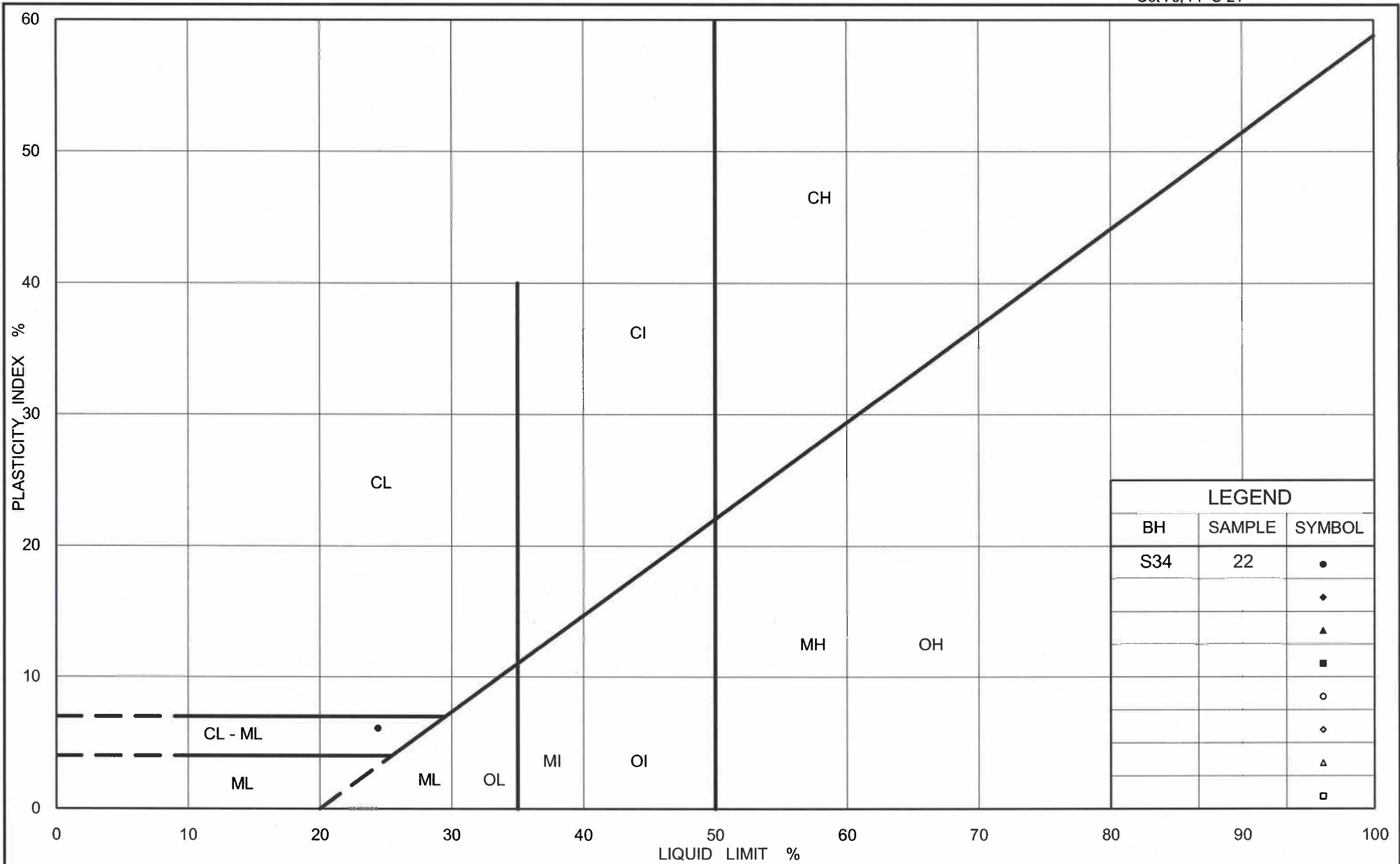
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	S34	21	177.5
■	S34	22	174.4

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Date: 29-May-09



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PLASTICITY CHART Clayey Silt

Figure No. B6

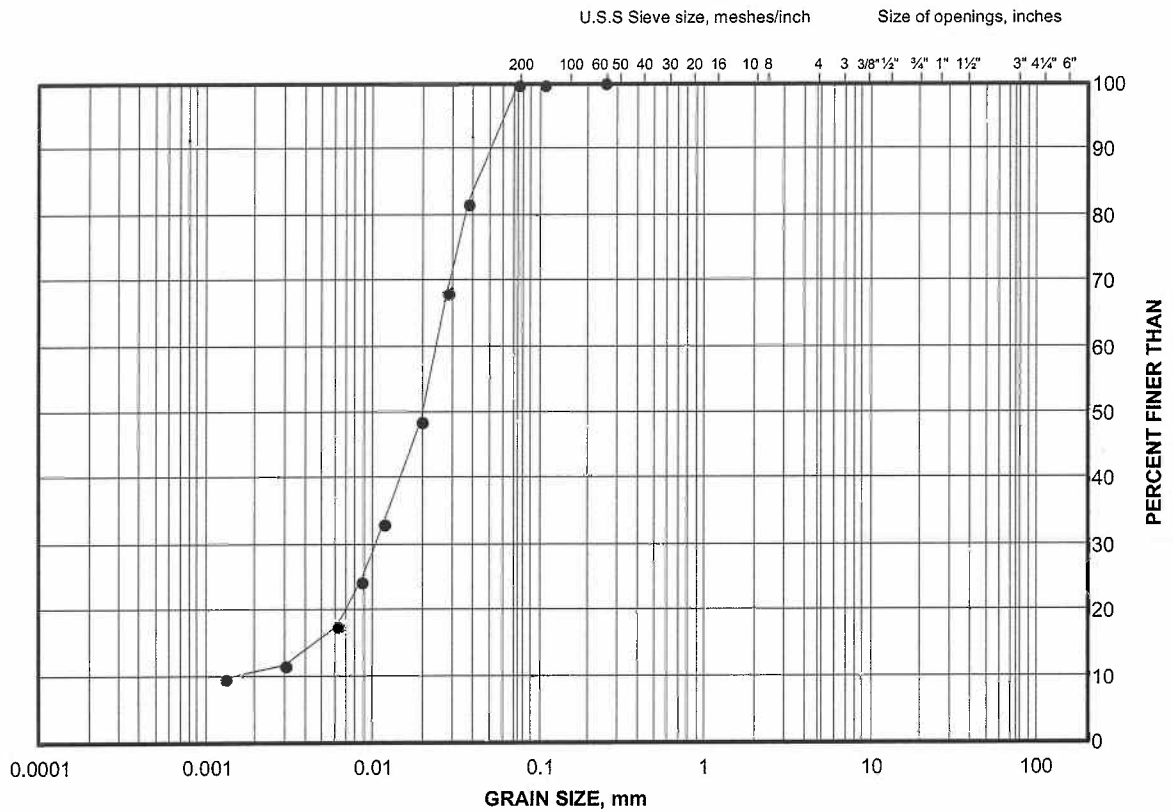
Project No. 06-1111-012-2

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GRAIN SIZE DISTRIBUTION TEST RESULT

Silt

FIGURE B7



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

LEGEND

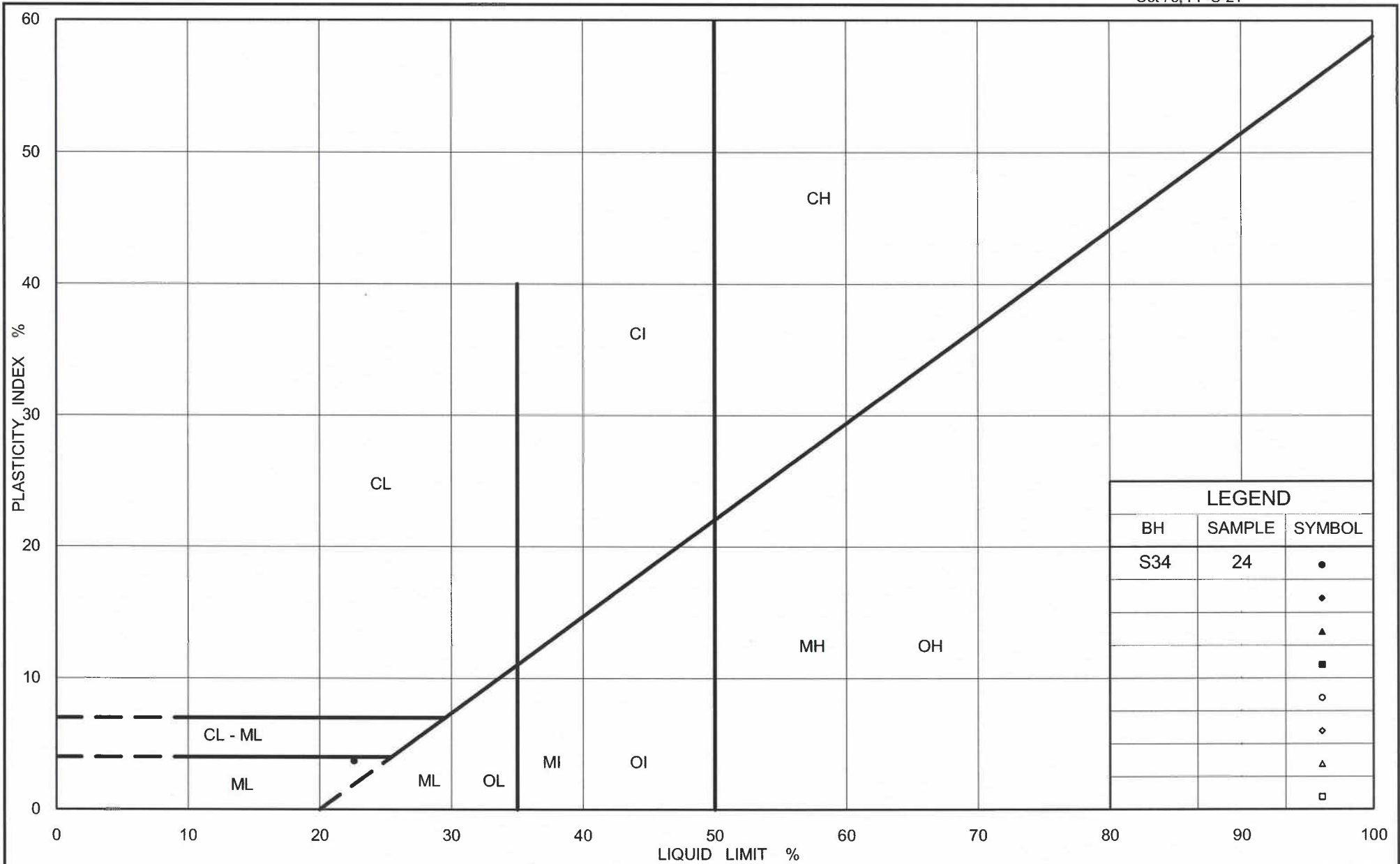
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
•	S34	26	165.3

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PLASTICITY CHART Silt

Figure No. B8

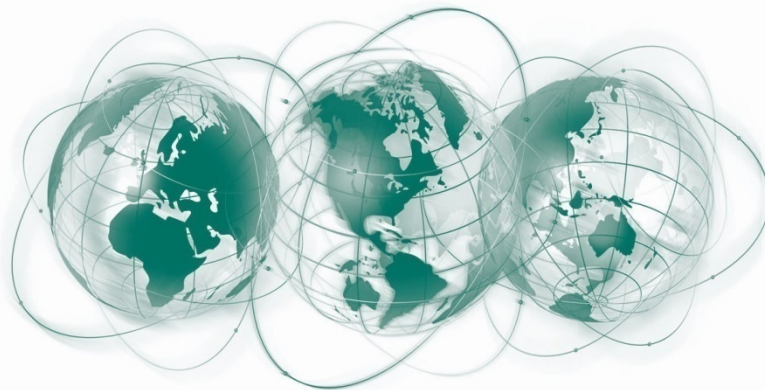
Project No. 06-1111-012-2

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