

Golder Associates Ltd.

2390 Argentia Road
Mississauga, Ontario, Canada L5N 5Z7
Telephone: (905) 567-4444
Fax: (905) 567-6561



**FOUNDATION INVESTIGATION
AND DESIGN REPORT
GENEVA STREET OVERPASS
QEW WIDENING FROM HIGHWAY 406
TO GARDEN CITY SKYWAY
ST CATHARINES, ONTARIO
G.W.P. 607-00-00**

Submitted to:

Morrison Hershfield
235 Yorkland Boulevard, Suite 600
Toronto, Ontario
M2J 1T1

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PART A

**FOUNDATION INVESTIGATION REPORT
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1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by Morrison Hershfield (MH) on behalf of the Ministry of Transportation, Ontario (MTO) to provide foundation engineering services associated with the widening of the Queen Elizabeth Way (QEW) between Highway 406 and the Garden City Skyway in the City of St. Catharines, in the Region of Niagara. Foundation engineering services are required for the widening or replacement of five structures (Third Street overpass, Martindale Road underpass, Lake Street underpass, Geneva Street overpass, and Welland Avenue overpass), new retaining walls and noise barrier walls, culvert extensions, and high mast light poles.

This report addresses the foundation investigation carried out for the Geneva Street overpass structure.

The terms of reference and scope of work for the foundation investigation are outlined in MTO's Request for Proposal for Agreement No. 2005-A-000564, issued in July 2002, and in Section 6.8 of MH's *Technical Proposal* for G.W.P. 607-00-00.

2.0 SITE DESCRIPTION

The existing single-span Geneva Street overpass structure is located approximately 880 m east of the QEW – Lake Street interchange and 770 m west of the QEW – Niagara Street interchange, in the City of St. Catharines in the Region of Niagara.

The overall surface topography in the area is relatively flat-lying, with a gentle slope downward to the north towards Lake Ontario. The natural ground surface at the structure site is at Elevation 99 m to 100 m, the Geneva Street grade is at about Elevation 100 m, and the existing QEW grade has a maximum elevation of approximately 106 m at the existing overpass. The existing QEW approach embankments are approximately 6 m high, and are sloped at approximately 2 horizontal to 1 vertical (2H:1V) along the north and south sides of the QEW; short retaining walls are present along both sides of the QEW, separating the QEW embankment from the adjacent local roads.

3.0 INVESTIGATION PROCEDURES

A subsurface investigation was carried out at the Geneva Street structure site in June and July 2005, at which time six boreholes (Boreholes 401 to 406) were advanced at the site using a track-mounted drill rig supplied and operated by Walker Drilling Ltd. of Utopia, Ontario.

The boreholes were advanced using solid stem augers to depths ranging from 9.8 m to 36.7 m below the existing ground surface. Samples of the overburden were obtained at 0.75 m, 1.5 m and 3 m intervals of depth, using 50 mm outside diameter split-spoon samplers driven by an automatic hammer in accordance with the Standard Penetration Test (SPT) procedure. The water level in the open boreholes was observed throughout the drilling operations, and a 50 mm diameter standpipe piezometer was installed in Borehole 405 to monitor the groundwater level at the site. The piezometer consists of a 1.5 m long slotted screen installed within a filter sand pack, then backfilled to ground surface using bentonite pellets, as depicted on the borehole record. Where no piezometer was installed, the boreholes were backfilled to ground surface upon completion using bentonite pellets.

The field work was supervised on a full-time basis by members of Golder's staff who located the boreholes in the field, directed the drilling, sampling, and in situ testing operations, and logged the boreholes. The soil samples were identified in the field, placed in labelled containers and transported to Golder's laboratory in Mississauga for further examination and testing. Index and classification tests consisting of water content determinations, Atterberg limits testing, and grain size distribution analyses were carried out on selected soil samples.

The borehole locations were measured by Golder relative to site features, and the ground surface elevations at the borehole locations were determined from the digital terrain model (DTM) for this project. The borehole locations (including MTM NAD83 northing and easting coordinates) and ground surface elevations (referenced to geodetic datum) are summarized in the following table and are shown on Drawing 1.

<i>Borehole Number</i>	<i>Borehole Location</i>	<i>MTM NAD83 Northing (m)</i>	<i>MTM NAD83 Easting (m)</i>	<i>Ground Surface Elevation (m)</i>
401	West Abutment	4,781,742.3	325,860.8	105.7
402	East Abutment	4,781,732.1	325,899.5	106.0
403	West Abutment	4,781,714.1	325,863.9	105.8
404	East Abutment	4,781,705.4	325,898.0	106.1
405	East Approach	4,781,733.9	325,941.6	100.0
406	West Approach	4,781,719.6	325,844.8	105.6

4.0 SITE GEOLOGY AND STRATIGRAPHY

4.1 Regional Geological Conditions

This area of the QEW lies within the Iroquois Plain physiographic region, as delineated in *The Physiography of Southern Ontario*¹ and *Urban Geology of Canadian Cities*².

The Iroquois Plain extends around the western shores of Lake Ontario; on the south side of the lake, in the St. Catharines area, the Plain is located between the present Lake Ontario shorebluffs and the foot of the Niagara Escarpment. The Plain is comprised of the flat to undulating lake bed and beaches of the former glacial Lake Iroquois, which occupied this area during the last glacial recession.

The surficial soils in the Iroquois Plain are typically comprised of glaciolacustrine clays and silts. However, in the St. Catharines area, surficial deposits of beach sand and gravel are present. The surficial sands, silts and clays are underlain by an extensive till deposit; portions of the till are considered to be “water-lain” (that is, formed by sediment rain-out either from a floating ice margin or from iceberg dumping), resulting in a predominantly massive, matrix-supported structure, as well as relatively thin sand to silt stringers or interlayers. This extensive till deposit may be underlain by or interlayered with a lower glaciolacustrine clay deposit, although this glaciolacustrine layer is absent in some portions of the Iroquois Plain in the St. Catharines area. Finally, the till and/or glaciolacustrine layer may be underlain by a lower till unit, that typically has increasing gravel content with proximity to the underlying bedrock (Menzies and Taylor, 1998).

The overburden soils are underlain by red shale bedrock of the Queenston Formation. This shale formation contains siltstone interlayers as well as “occasional patches of gypsum” (Menzies and Taylor, 1998).

4.2 Site Stratigraphy

As part of the subsurface investigation at this site, six boreholes (401 to 406) were advanced in the vicinity of the proposed foundation elements for the new overpass structure. The borehole locations and ground surface elevations are shown on Drawing 1.

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.

² J. Menzies and E.M. Taylor. “Urban Geology of St. Catharines-Niagara Falls, Region Niagara”. In *Urban Geology of Canadian Cities*, Geological Association of Canada Special Paper 42, Ed. P.F. Karrow and O.L. White, 1998.

The detailed subsurface soil and groundwater conditions encountered in the boreholes and the results of in situ and laboratory testing are given on the Record of Borehole sheets and on Figures 1 to 6. The stratigraphic boundaries shown on the borehole records are inferred from non-continuous sampling and, therefore, represent transitions between soil types rather than exact planes of geological change. The subsoil conditions will vary between and beyond the borehole locations.

All of the boreholes except Borehole 405 were advanced from the QEW grade, and so encountered asphalt overlying between 6.4 m and 8.9 m of existing QEW embankment fill. The native soils at the site consist of a thin, localized surficial sandy silt to clayey silt deposit underlain by an extensive stiff to hard clayey silt till to silty clay till deposit, which is in turn underlain by a hard till/residual soil deposit. Red shale bedrock of the Queenston Formation was encountered below the till/residual soil deposit in one borehole, at approximately Elevation 70.8 m (at about 35.1 m depth below the QEW grade).

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

4.2.1 Topsoil, Asphalt, and Fill Materials

Approximately 150 mm to 200 mm of asphalt was encountered in Boreholes 401, 402, 403, 404, and 406, all of which were drilled from the QEW grade. Borehole 405, which was drilled at the toe of the QEW embankment, encountered about 100 mm of topsoil at the ground surface.

The existing QEW embankment fill was encountered in Boreholes 401 to 404 and 406. The fill thickness varies from 6.4 m to 7.4 m thick at the west approach/west abutment, and from 7.4 m to 8.9 m thick at the east approach/east abutment, as encountered in the boreholes. The fill generally consists of an upper, 0.9 m to 3.6 m thick layer of sand and gravel, generally underlain by silty sand, sand and gravel or foundry sand fill materials. The results of grain size distribution tests carried out on four selected samples of the fill are shown on Figure 1.

The measured Standard Penetration Test (SPT) “N” values within the existing embankment fill range from 4 to 107 blows per 0.3 m of penetration. The lower SPT “N” values, those below approximately 20 to 25 blows per 0.3 m of penetration, were generally measured within the fill in Boreholes 402 and 403; further comments on the relative density of the various embankment fill materials are provided as follows:

- In the upper sand and gravel layer, the measured SPT “N” values range from 8 to 53 blows per 0.3 m of penetration, but are typically between about 20 and 40 blows per 0.3 m of penetration; these results indicate that this layer has a loose to very dense, but typically compact to dense, relative density. One measured SPT “N” value of 107 blows per 0.3 m of penetration at the base of the sand and gravel layer in

Borehole 406 is considered to be attributable to the presence of gravel or cobbles within the fill.

- In the foundry sand, the measured SPT “N” values range from 10 to 87 blows per 0.3 m of penetration, but are typically greater than 25 blows per 0.3 m of penetration; based on this, the foundry sand typically has a compact to very dense relative density. It is noted that the foundry sand is between 2.8 m and 5.3 m thick in Boreholes 401, 403 and 406 at the west approach/west abutment; at the east approach/east abutment, a 1.2 m thick layer of foundry sand was encountered at the base of the embankment fill in Borehole 404.
- In the silty sand to sand and gravel fill (below the upper sand and gravel or “road base” fill), the measured SPT “N” values range from 4 to 55 blows per 0.3 m of penetration, but are typically greater than 20 blows per 0.3 m of penetration. This portion of the fill has a typically compact to very dense relative density.

4.2.2 Surficial Sandy Silt and Clayey Silt

Boreholes 403 and 406 encountered an approximately 1 m to 1.8 m thick layer of sandy silt below the fill materials, and atop the till deposit. One measured SPT “N” value of 16 blows per 0.3 m of penetration indicates that this surficial deposit has a compact relative density. The result of a grain size distribution test carried out on a selected sample of the surficial sandy silt is shown on Figure 2.

In Borehole 405, a 0.7 m thick layer of clayey silt is present immediately below the topsoil, atop the till deposit. One measured SPT “N” value of 6 blows per 0.3 m of penetration indicates that this surficial clayey silt has a firm consistency.

4.2.3 Clayey Silt to Silty Clay Till

A thick till deposit was encountered beneath the fill and surficial layers at all of the borehole locations. Where fully penetrated in Boreholes 401 to 404, the till deposit is between 19.2 m and 25.9 m in thickness and extends to depths ranging from 28.3 m to 33.5 m below the QEW grade, with the deposit base encountered between Elevations 72.3 m and 77.7 m. The deposit is thicker and its base was encountered lower in Boreholes 403 and 404, which were advanced on the south side of the QEW.

The till deposit is comprised predominantly of clayey silt to silty clay containing trace to some sand and trace gravel; the results of six grain size distribution tests are shown on Figure 3. Atterberg limit testing was carried out on eighteen samples of this deposit, and measured plastic limits of 15 to 22 per cent, liquid limits of 28 to 50 per cent, and plasticity indices of 11 to 28 per cent. The results, plotted on Figure 4, confirm that the till material grades from a low plasticity

clayey silt to an intermediate plasticity silty clay. The natural water contents measured on selected samples of the till deposit range from 8 to 35 per cent.

The measured SPT “N” values within the clayey silt to silty clay till deposit vary from 8 to 57 blows per 0.3 m of penetration. The higher SPT “N” values, those above approximately 30 blows per 0.3 m of penetration, were measured in the upper few metres or at the base of the till deposit in most of the boreholes, indicating that these portions of the till are generally hard. The majority of the deposit is stiff to very stiff, with SPT “N” values varying from about 12 to 30 blows per 0.3 m of penetration, and undrained shear strengths in excess of 100 kPa. However, a zone of softer clayey silt to silty clay, ranging in thickness between 1.5 m and 8.5 m, is present within the till below approximately Elevation 88.2 m in Borehole 402, below approximately Elevations 82.2 m to 84.2 m in Boreholes 403 and 404, and below approximately Elevation 95.5 m in Borehole 405. The measured SPT “N” values within this softened zone vary from 8 to 11 blows per 0.3 m of penetration, and field vane tests in this zone measured undrained shear strengths of approximately 75 kPa to 80 kPa with sensitivities of about 1.6 to 1.7. These results indicate that the softened zone within the till has a firm to stiff consistency.

4.2.4 Clayey Silt to Sand and Silt Till / Residual Soil

A till / residual soil deposit that varies in composition from clayey silt with sand to sand and silt, containing trace gravel, shale and limestone fragments, was encountered below the till deposit. The surface of this till / residual soil deposit was encountered between Elevations 72.3 m and 77.7 m (about 28.3 m to 33.5 m below the QEW grade) in Boreholes 401 to 404; the surface of the deposit is lower in Boreholes 403 and 404, which are located on the south side of the QEW. Based on its composition, consistency/relative density, and its position atop the shale bedrock (as encountered in Borehole 403), this deposit is interpreted to be a residual soil derived from weathering of the underlying bedrock.

The results of grain size distribution tests conducted on two selected samples of this deposit are shown on Figure 5. Atterberg limit testing was carried out on four samples of the cohesive portions of this deposit, and measured plastic limits of 13 to 14 per cent, liquid limits of 20 to 22 per cent, and plasticity indices of 6 to 8 per cent. The results, plotted on Figure 6, confirm that the cohesive portion of the till/residual soil is a clayey silt of low plasticity.

The measured SPT “N” values within the residual soil range from 81 to greater than 100 blows, but are generally greater than 100 blows per 0.3 m of penetration, indicating that this deposit has a hard consistency.

4.2.5 Shale Bedrock

Bedrock was encountered in Borehole 403, where it was observed in a split-spoon sample. The surface of the bedrock was encountered in this borehole at about Elevation 70.8 m (at about 35.1 m depth below the QEW grade).

The bedrock observed in the sample consists of red shale of the Queenston Formation. Although not noted in the split-spoon sample collected, interlayers of strong to very strong limestone and siltstone are anticipated to be present within the Queenston Formation shale bedrock.

4.3 Groundwater Conditions

One piezometer was installed within the till deposit in Borehole 405 to monitor the groundwater level at the site; the piezometer installation details are shown on the borehole record. The water level measurements obtained to date are shown in the table below. The groundwater level should be expected to fluctuate seasonally.

<i>Date</i>	<i>Water Level Depth</i>	<i>Water Level Elevation</i>
August 8, 2005	2.8 m	97.2 m
December 6, 2005	0.6 m	99.4 m

In addition, “perched” groundwater should be expected within the surficial sandy silt deposit, especially during the wetter months of the year.

5.0 CLOSURE

This Foundation Investigation Report was prepared by Ms. Houda Jadi, Ph.D., P.Eng., and reviewed by Ms. Lisa Coyne, P.Eng., an Associate and geotechnical engineer with Golder, with technical input from Mr. Murty Devata, P.Eng., a Specialist Foundations Consultant to Golder. Mr. Fintan Heffernan, P.Eng., a Designated MTO Contact for Golder, carried out an independent review of the report.

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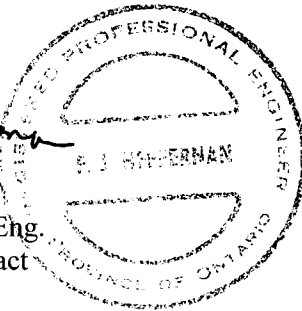
Houda Jadi, P.Eng.
Geotechnical Engineer



Lisa C. Coyne, P.Eng.
Associate



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



HJ/LCC/MSD/FJH/hj/lcc

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PART B

**FOUNDATION DESIGN REPORT
GENEVA STREET OVERRPASS
QEW WIDENING FROM HIGHWAY 406
TO GARDEN CITY SKYWAY
ST CATHARINES, ONTARIO
G.W.P. 607-00-00**

6.0 ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides geotechnical recommendations for the design of the foundations for the proposed Geneva Street overpass replacement structure. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the subsurface investigation at this site. The interpretation and recommendations provided are intended to provide the designers with sufficient information to assess the feasible foundation alternatives and to carry out design of the proposed replacement structure foundations. Where comments are made on construction, they are provided in order to highlight those aspects which could affect the design and construction of the project, and for which special provisions or operational constraints may be required in the Contract Documents. Those requiring information on aspects of construction should make their own interpretation of the factual information provided as it may affect equipment selection, proposed construction methods, scheduling and the like.

6.2 Bridge Foundation Options

The existing single-span, rigid frame Geneva Street overpass is to be replaced with a new, wider single-span structure, and new retaining walls will be constructed along the north and south sides of the QEW to accommodate the approach embankment widening within the available space. The widened portions of the QEW will be maintained at approximately Elevation 106 m within the limits of the structure and its immediate approach embankments, requiring placement of a maximum of 2.1 m to 2.8 m of new fill atop the existing embankment side slopes based on the proposed embankment cross-sections (dated October 28, 2005).

The existing overpass and the QEW embankment have been in place for over forty years, and the soils at the site have essentially completed their consolidation settlement as a result of the existing embankment loadings. The main concern for the design and construction of the Geneva Street overpass replacement structure is the settlement that could result from placement of the widening fill. As discussed further in Section 6.7, between 25 mm and 65 mm of settlement will occur at this site as a result of the combined structure and widened approach embankment loadings, assuming the use of spread footings for support of the structure and conventional earth or granular fill behind the abutments. Although up to half of this settlement will occur relatively quickly following construction (within three to six months), it is estimated that it will take approximately two to three years to complete the majority of the remaining settlement.

Based on the magnitude of settlement as noted above, the use of shallow foundations is not feasible for support of the replacement structure at this site if used in conjunction with conventional earth or granular fill for the embankment widenings. However, if EPS fill (having a bulk unit weight of approximately 1 kN/m^3) is used for the embankment widening so that there is no net increase in the embankment loading immediately behind the abutments, it may be feasible to support the replacement structure on spread footings. Assuming the use of EPS fill, it is possible to limit the settlement under 3 m wide spread footings to less than 25 mm, based on Serviceability Limit States (SLS) loadings of 225 kPa to 275 kPa. If this option is structurally feasible, it will be necessary to examine the effect of 25 mm of settlement on the 900 mm diameter sewer that passes under the north end of the proposed west abutment.

Based on the settlement estimates presented above and considering the proposed road grades and the subsurface conditions, the following foundation options are available:

- **Spread footings founded on the stiff to hard till deposit, with the use of a “wedge” of EPS fill behind the abutments in the widened sections:** In order to adopt spread footings at this site, a “zero net loading” approach (i.e. no additional loading on the foundation soils) would be required for the embankment widening to ensure that the maximum settlements as a result of foundation loading only are restricted to 25 mm or less. In this regard, in order to supply a minimum of 1.2 m of soil cover above the EPS fill, it would be necessary to subexcavate at least 1.2 m deep into the existing embankment side slopes to key in the EPS fill. It is noted that there would still be some differential settlement along the length of each abutment footing due to the foundation loading, owing to the variable subsoil conditions; however, the maximum differential settlement along each footing is expected to be 15 mm or less. In considering this option, particular attention must be given to the presence of the 900 mm combined sewer that runs under the north end of the west abutment; if this utility cannot tolerate 25 mm of settlement, alternatives may need to be adopted. For example, the “imperfect trench” method, in which loose backfill is placed above the utility, could be considered to reduce loading on and settlement of the utility.
- **Steel H-piles driven to found within the “100-blow” lower till/residual soil deposit:** Driven steel H-piles are suitable for support of the abutments, in either a conventional or integral abutment configuration. The loading imposed by the widening of the approach embankments will cause settlement of the silty clay till and existing embankment fills, which will in turn impart downdrag loads on the northern piles for the east abutment. For the southern portion of the east abutment and for the west abutment, the settlement resulting from the embankment widening is predicted to be on the order of 10 mm, as discussed in Section 6.7.3, and therefore downdrag loads will not apply to the piles supporting the southern portion of the east abutment, or the west abutment. In order to eliminate the downdrag loads and/or to minimize post-construction settlement of the embankment adjacent to the pile-supported east abutment, consideration could be given to preloading the northeast approach embankment widening area (although it is understood that this is not an option under the current construction staging schedule) or to the use of ultra-lightweight slag fill or EPS fill behind the east abutment as discussed further in Sections 6.3 and 6.7.

- **Caissons founded within the 100-blow till/residual soil deposit:** As an alternative to steel H-pile foundations, caissons supported on the 100-blow till could also be used for support of the new structure. However, caissons are considered to be less practicable and cost-effective than driven steel H-piles, particularly since the use of a permanent liner would be recommended for caissons given the soil conditions at the site.

Geotechnical recommendations for the design of foundations for the bridge abutments and associated wing walls are presented in the following sections. A summary comparison of the advantages, disadvantages, relative costs, and risks associated with the foundation options is presented in Table 1 following the text of this report.

6.3 Spread Footings

If adopted in conjunction with the use of EPS fill for the embankment widening, shallow footings for support of the overpass structure and associated wing walls should be founded on the stiff to hard till deposit below the existing embankment fill; design founding elevations are provided below. Excavation through the existing embankment fill and some subexcavation of the surficial sandy silt and upper portion of the till deposit will be required. The design founding level should be checked to ensure that there is adequate soil cover to provide protection against frost penetration, as discussed in Section 6.3.3.

In order to restrict settlements resulting from the shallow foundation loading to 25 mm or less, the widening of the embankments must result in “zero net loading”. Further discussion on the EPS configuration to address this requirement is provided in Section 6.7.

6.3.1 Geotechnical Resistance

The subsoils at the east abutment include a thicker zone of compressible, firm to stiff clayey silt to silty clay till (8.5 m thick in Borehole 402, and less than 2.5 m thick in the remaining boreholes). As a result, a spread footing for support of the east abutment will have a lower geotechnical resistance at SLS. The following founding elevations and geotechnical resistances at Ultimate Limit States (ULS) and Serviceability Limit States (SLS) may be used for the design of 3 m wide spread footings placed on the properly prepared clayey silt to silty clay till:

<i>Foundation Element</i>	<i>Founding Elevation</i>	<i>Factored Geotechnical Resistance at ULS</i>	<i>Geotechnical Resistance At SLS*</i>
East abutment	96.5 m	350 kPa	225 kPa
West abutment	97.5 m	400 kPa	275 kPa

* For 25 mm of settlement, assuming a 3 m wide footing.

As highlighted in Section 6.2, differential settlement will occur along the length of the abutment footings due to the variable consistency of the till deposit and the variation in the thickness of the firm to stiff till zone across the site. Foundation loading at the SLS resistances provided above will result in approximately 25 mm of settlement at the north end of both footings, about 20 mm of settlement at the south end of the west abutment footing, and about 10 mm of settlement at the south end of the east abutment footing. The maximum differential settlement along each footing and between the two abutment footings is therefore expected to be approximately 15 mm.

It is noted that the ULS resistance and the magnitude of settlement are dependent on the footing size, configuration and applied loads. Therefore, if this option is feasible and is pursued, the geotechnical resistances for spread footings should be reviewed as the detailed design progresses, once the final geometry of the foundations has been established.

The geotechnical resistances provided above are given under the assumption that the loads are 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.

6.3.2 Resistance to Lateral Loads

Resistance to lateral forces / sliding resistance between the concrete footings and the subgrade should be calculated in accordance with Section 6.7.5 of the *CHBDC*. The coefficient of friction, $\tan \phi'$, between cast-in-place concrete footings and the undisturbed, properly prepared clayey silt to silty clay till may be taken as 0.45. This represents an unfactored value; in accordance with the *CHBDC*, a factor of 0.8 is to be applied in calculating the horizontal resistance.

6.3.3 Frost Protection

The footings should be provided with a minimum of 1.2 m of soil cover, or equivalent, for frost protection purposes.

6.4 Steel H- Pile Foundations

The abutments may be supported on steel H-piles driven to found within the “100-blow” lower till/residual soil deposit. The site soils are suitable for the use of conventional, integral, or semi-integral abutment configurations.

The surface of the “100-blow” till/residual soil was encountered between Elevation 75.5 m and 72.5 m in the boreholes, as summarized in the table below. For design, the following pile tip levels may be assumed based on 1.5 m of penetration into the “100-blow” till.

<i>Foundation Element</i>	<i>Relevant Boreholes</i>	<i>Estimated Elevation Of “100-Blow” Till</i>	<i>Estimated Pile Tip Elevation</i>
East Abutment	402, 404	75.5 m	74 m
West Abutment	401, 403	75 m to 72.5m	73.5 m to 71 m

Although not encountered in the boreholes advanced at this site, consideration must be given to the potential presence of cobbles and boulders within the till deposits at this site; a sample Non-Standard Special Provision (NSSP) is provided in Appendix A to warn the contractor of the presence of cobbles and boulders within the till that could affect the installation of the piles. Steel H-piles should be stiffened with MTO flange plates for protection during driving, in accordance with OPSS 903.07.05.04.

6.4.1 Axial Geotechnical Resistance

For HP 310 x 110 piles driven to practical refusal within the 100-blow lower clayey silt till, a factored axial resistance at ULS of 1,800 kN may be assumed for design. The axial geotechnical resistance at SLS may be taken as 1,600 kN.

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 and length of pile. The criteria must therefore be established at the time of construction after the piling equipment is known. For piles driven into the 100-blow till/residual soil, the following note is considered appropriate for the design and site conditions assuming a resistance factor of 0.5 is applied to the use of the Hiley formula:

“Piles to be driven in accordance with Standard SS 103-11 using an ultimate capacity of 3,600 kN per pile.”

6.4.2 Downdrag Load (Negative Skin Friction)

Assuming the use of conventional earth or granular fill, the widened embankment loading will cause consolidation settlement of the firm to stiff zone within the clayey silt to silty clay deposit at this site. As discussed further in Section 6.7, approximately 40 mm of settlement will occur under the northern widening of the east approach embankment, and about 10 mm of settlement will occur as a result of the approach embankment widening in other areas. Based on the predicted magnitude of settlement relative to the elastic shortening of the pile, negative skin friction or downdrag loads will need to be taken into account in the design of the piles for support of the northern portion of the east abutment (where the compressible clayey silt to silty clay

stratum is thickest), unless EPS or ultra-lightweight slag fill is used for the widening of the north-east approach embankment as discussed in Section 6.7.

In calculating the magnitude of the downdrag force, the methods described in both the Canadian Foundation Engineering Manual as well as the US Transportation Research Board's report, "Design and Construction Manual For Downdrag on Uncoated and Bitumen-Coated Piles" [Briaud and Tucker (1994)] were considered. Considering the larger predicted settlement of the clayey silt to silty clay deposit versus the elastic shortening of the pile, the neutral plane used in those analyses was assumed to be at the underside of the clayey silt to silty clay deposit.

Based on the above, the unfactored downdrag load acting on a single HP 310 x 110 pile in the northern portion of the east abutment, over the length of pile within the native soils, is estimated to be 450 kN. 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. Downdrag loads could be eliminated with the use of EPS fill as backfill behind the abutments; further discussion on the use of EPS fill is provided in Section 6.7.

6.4.3 Resistance to Lateral Loads

Lateral loading could be resisted fully or partially by the use of battered steel H-piles. If vertical piles are used, the resistance to lateral loading will have to be derived from the soil in front of the piles.

The resistance to lateral loading in front of the pile may be calculated using subgrade reaction theory where the coefficient of horizontal subgrade reaction is determined based on the equations given below:

For cohesionless soils:

$$k_h = \frac{n_h z}{B} \quad \text{where}$$

k_h is the coefficient of horizontal subgrade reaction (MPa/m);
 n_h is the constant of subgrade reaction (MPa/m);
 z is the depth (m); and
 B is the pile diameter (m).

For cohesive soils:

$$k_h = \frac{67s_u}{B} \quad \text{where}$$

k_h is the coefficient of horizontal subgrade reaction (kPa/m);
 s_u is the undrained shear strength of the soil (kPa); and
 B is the pile diameter (m).

The following ranges for the value of n_h and s_u may be assumed in the structural analyses. Approximate elevation intervals are given in this table for the deposits at each of the foundation elements; however, the deposit boundaries vary from north to south at each of the foundation elements, and reference should also be made to the interpreted stratigraphic sections shown on Drawing 2.

<i>Soil Unit</i>	<i>Elevation</i>	<i>n_h</i>	<i>s_u</i>
East abutment:			
Existing compact to dense embankment fill	Above 97 m	15 MPa/m	–
Stiff to hard clayey silt to silty clay till	97 m to 88 m	–	150 kPa
Firm to stiff silty clay to clayey silt	88 m to 80 m	–	75 kPa
Very stiff to hard clayey silt till	80 m to 76 m	–	200 kPa
Hard/very dense (100-blow) lower till/ residual soil	Below 76 m	–	500 kPa
West abutment:			
Existing compact to dense embankment fill	Above 98 m	15 MPa/m	–
Very stiff to hard clayey silt till	98 m to 95 m	–	200 kPa
Stiff to very stiff clayey silt till	95 m to 82.5 m	–	150 kPa
Firm to stiff clayey silt till	82.5 m to 79.5 m	–	75 kPa
Very stiff to hard clayey silt till	79.5 m to 74 m	–	200 kPa
Hard/very dense (100-blow) lower till/ residual soil	Below 74 m	–	500 kPa

A maximum factored lateral resistance of 200 kN at ULS, and a maximum lateral resistance of 110 kN at SLS (for 10 mm of horizontal deflection at pile cap level) is recommended for HP 310 x 110 piles. These values are based on the “Assessed Horizontal Passive Resistance Values for Various Pile Types” provided in Table C6.8.7.1(a) of the *Commentary* to the *CHBDC*.

Group action for lateral loading should also be considered when the pile spacing in the direction of the loading is less than six to eight pile diameters. Group action can be evaluated by reducing the coefficient of horizontal subgrade reaction in the direction of loading by a reduction factor, R , as follows:

<i>Pile Spacing in Direction of Loading (d = Pile Diameter)</i>	<i>Subgrade Reaction Reduction Factor (R)</i>
8d	1.00
6d	0.70
4d	0.40
3d	0.25

Reference: Foundations and Earth Structures – Design Manual 7.2, NAVFAC DM-7.2. Department of the Navy, Naval Facilities Engineering Command (1982).

6.4.4 Frost Protection

The pile caps should be provided with a minimum of 1.2 m of soil cover for frost protection.

6.5 Caissons

Consideration could be given to the use of caissons founded within the “100-blow” till/residual soil deposit or shale bedrock for support of the new Geneva Street overpass structure. The surface of the “100-blow” till/residual soil was encountered between Elevation 75.5 m and 72.5 m in the boreholes, as summarized in the table below. The following design base elevations may be used at the abutments, based on approximately 2 m of embedment within the “100-blow” till or shale (as encountered in Borehole 403):

<i>Foundation Element</i>	<i>Relevant Boreholes</i>	<i>Estimated Elevation Of “100-Blow” Till</i>	<i>Estimated Caisson Base Elevation</i>
East Abutment	402, 404	75.5 m	73.5 m
West Abutment	401, 403	75 m to 72.5m	73 m to 70.5 m

The lower till/residual soil should be expected to contain cobbles and boulders which may pose difficulties in advancing caissons / liners.

Running or flowing of water-bearing cohesionless soil strata could occur during or after drilling of the caissons, and basal heave could occur where more pervious sand and silt till soils are present at/near the caisson base. If caisson foundations are adopted for this site, temporary or permanent caisson liners would be required to support the soils during construction and permit inspection and cleaning of the caisson base. However, construction experience in similar soil conditions has demonstrated that temporary liners can be difficult to withdraw, owing to the length of the liners and the hard/very dense nature of the 100-blow material, and that such difficulties can result in “necking” of the caisson. As such, permanent liners would be preferred for the construction of the caissons in these soil conditions.

If caisson foundations are adopted for this site, an NSSP will be developed to address the potential presence of cobbles and boulders and the need for control of the ground and groundwater during caisson construction.

6.5.1 Axial Geotechnical Resistance

The caissons will derive the majority of their capacity from base resistance, although some shaft friction has also been taken into account based on “socketting” approximately 2 m into the “100-blow” till/residual soil deposit. Using the design elevations given above, and assuming that all caisson excavations are inspected prior to pouring concrete, the factored axial geotechnical resistance at ULS and the axial resistance at SLS are given below for various caisson diameters:

<i>Caisson Diameter</i>	<i>Axial Geotechnical Resistance</i>	
	ULS	SLS
0.9 m	4,000 kN	3,200 kN
1.2 m	7,000 kN	5,600 kN
1.5 m	11,000 kN	8,800 kN

If permanent liners are used for construction of the caissons, the geotechnical resistances provided above would have to be reduced to neglect the component of shaft friction over the “socket” within the 100-blow soil.

6.5.2 Downdrag Load (Negative Skin Friction)

The estimated unfactored downdrag load acting on the caissons supporting the northern portion the east abutment may be taken as shown in the table below:

<i>Caisson Diameter</i>	<i>Unfactored Downdrag Load</i>
0.9 m	1,250 kN
1.2 m	1,650 kN
1.5 m	2,250 kN

Other requirements for structural design with respect to downdrag load on the caissons are discussed in Section 6.4.2.

The downdrag loads provided in the table above are relatively large and may render the use of caisson foundations for the east abutment impractical. If caisson foundations are preferred, measures to eliminate the downdrag load include the use of ultra-lightweight slag fill or EPS fill for the widening of the north side of the east approach embankment (as discussed in Section 6.7). Alternatively, it may also be feasible to construct the caissons with a permanent lining and bentonite slurry “slip” layer; however, such construction may also prove costly. Recommendations for this type of construction can be developed if caisson foundations are determined to be the preferred option from a structural perspective.

6.5.3 Resistance to Lateral Loads

The resistance to lateral loading developed by the soils in front of the caissons (based on subgrade reaction theory), and the reductions due to group effects, may be determined as per Section 6.4.3.

A maximum factored lateral resistance of 400 kN at ULS, and a maximum lateral resistance of 250 kN at SLS (for 10 mm of horizontal deflection at pile cap level) are recommended for 0.9 m diameter caissons. Values for alternative caisson diameters can be provided if larger diameter caisson foundations are adopted at this site.

6.5.4 Frost Protection

The pile caps should be provided with a minimum of 1.2 m of soil cover for frost protection.

6.6 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, on the nature of the soils behind the backfill, on the magnitude of surcharge including construction loadings, on the freedom of lateral movement of the structure, and on the drainage conditions behind the walls.

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 per cent passing the 200 sieve should be used as backfill behind the walls. This fill should be placed and compacted in accordance with MTO's Special Provision 105S10. 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 3501.00 and 3504.00.
- 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 6.9.3. Compaction equipment should be used in accordance with MTO's Special Provision 105S10. Other surcharge loadings should be accounted for in the design, as required.
- The granular fill may be placed either in a zone with width equal to at least 1.2 m behind the back of the wall stem (Case I in Figure C6.9.1(I) 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 (Case II in Figure C6.9.1(I) of the *Commentary to the CHBDC*).
- For Case I, the pressures are based on the existing and new embankment fill materials and the following parameters (unfactored) may be used, assuming the use of Select Subgrade material for the new portions of the approach embankments:

Soil unit weight:	20 kN/m ³
Coefficients of static lateral earth pressure:	
Active, K_a	0.35
At rest, K_o	0.50

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

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

- If the wall support and superstructure allow lateral yielding of the stem, active earth pressures may be used in the geotechnical design of the structure. If the abutment support does not allow lateral yielding, at-rest earth pressures should be assumed for geotechnical design.

6.7 Approach Embankment Design and Construction

The widening of the existing QEW embankment will require placement of “wedges” of fill, having a maximum thickness of 2.1 m to 2.8 m, on top of the existing embankment side slopes.

6.7.1 Subgrade Preparation and Embankment Construction

In order to minimize differential settlement between the existing and widened portions of the approach embankments, it is recommended that all topsoil and softened / loosened soils be stripped from the existing embankment side slopes below the widening areas. All subgrade soils should be proof-rolled prior to fill placement in accordance with OPSS 206. Embankment fill should be placed and compacted in accordance with MTO's Special Provision 105S10.

In order to minimize differential settlement between the widened portions of the approach embankments due to settlement of the fill itself, the use of granular fill is recommended over the use of cohesive fill, since the majority of settlement of granular fills will occur during construction whereas some settlement of cohesive fills, if used, would occur post-construction. The new embankment fills should be benched into the existing embankment in accordance with OPSD 208.010.

6.7.2 Approach Embankment Stability

Due to the limited footprint for widening of the QEW (at this location, the North Service Road and Dunlop Drive run immediately adjacent to the toe of the existing embankment side slopes), new retaining walls will be constructed along both the north and south sides of the widened QEW. The factor of safety related to global stability for properly designed and constructed retaining walls at this site will be greater than 1.3; detailed discussion regarding the retaining

walls is provided in Golder's Foundation Investigation and Design Report for the retaining walls on this project.

6.7.3 Approach Embankment Settlement

Settlement of the 1.5 m to 8.5 m thick, firm to stiff silty clay to clayey silt till layer as well as the overlying stiffer till will occur as a result of the widening of the existing approach embankments, assuming the use of conventional earth or granular fill. In order to estimate the magnitude of settlement, analyses were carried out using the commercially-available computer program *Unisettle* as well as hand calculations. The settlement of the founding soils has been estimated using the consolidation parameters and elastic deformation moduli given in the table below, based on correlations with the undrained shear strength, Atterberg limits, and SPT "N" values:

<i>Soil Unit</i>	<i>Bulk Unit Weight (kN/m³)</i>	<i>Preconsolidation Pressure (kPa)</i>	<i>C_c</i>	<i>C_r</i>	<i>Elastic Modulus (MPa)</i>
Embankment fill (range of parameters assumed for earth fill and granular fill)	19-21	—	—	—	—
Surficial sandy silt / clayey silt	19-20	—	—	—	25-35
Stiff to hard clayey silt to silty clay till	20	—	—	—	40-50
Firm to stiff clayey silt to silty clay till	19	275-400	0.16-0.35	0.030-0.044	—
Hard/very dense (100-blow) till/residual soil	21	—	—	—	100

Based on placement of a "wedge" of fill, having a maximum thickness of 2.1 m to 2.8 m, on the existing embankment side slopes, the following magnitudes of settlement have been estimated below the widening footprint and the outside portion of the existing embankment:

<i>Location</i>	<i>Borehole No.</i>	<i>Thickness / Depth of Soft Zone</i>	<i>Max. Additional Fill Thickness</i>	<i>Predicted Settlement*</i>
East Approach North Side	402	8.5 / 18 m	2.5 m	40 mm
East Approach South Side	404	3 / 22 m	2.1 m	10 mm
West Approach North Side	401	N/A	2.8 m	10 mm
West Approach South Side	403	2.2 / 24 m	2.1 m	10 mm

* Settlement in foundation soils due to loading under embankment widening only (i.e. excluding any loading from shallow foundations, if adopted).

Approximately one-third to one-half of the predicted settlement is associated with elastic compression of the upper, stiff to hard clayey silt to silty clay till, and this settlement will occur during and immediately following embankment construction. For the north side of the east

approach embankment widening, approximately 25 mm of consolidation settlement will remain following completion of construction and the immediate elastic settlement; it is estimated that this consolidation settlement will be completed within approximately fifteen months following completion of the northeast embankment widening. This magnitude of settlement would affect the widening area, including the Toronto-bound QEW pavement/shoulder behind the east abutment, and the concrete retaining wall at the QEW embankment toe.

If the short concrete retaining wall at the QEW embankment toe can accommodate 25 mm to 40 mm of post-construction settlement with the use of construction slip joints, or if this concrete retaining wall is supported on deep foundations, then no settlement mitigation measures may be necessary in regard to the wall. If the construction staging schedule permits, the post-construction settlement of the Toronto-bound QEW pavement/shoulder behind the east abutment could be mitigated by means of padding prior to final paving; no other settlement mitigation measure would be required in this regard.

Due to the construction staging and the proximity of the local roads to the QEW embankment toe, there is no opportunity for preloading or surcharging of the northeast embankment widening area prior to construction of the new Geneva Street overpass structure and associated concrete retaining wall in this area. Therefore, if it is desired to reduce the magnitude of post-construction settlement under the northeastern embankment widening to less than 25 mm, the most practical mitigation option is to use lightweight fill. Consideration could be given to the use of ultra-lightweight slag fill (having a bulk unit weight of 11.5 kN/m³) for the widening at this location. With the use of ultra-lightweight slag fill, the post-construction consolidation settlement under this portion of the embankment widening would be reduced to approximately 15 mm. In terms of constructability, ultra-lightweight slag fill would require careful placement and compaction in order to maintain its composition and desired bulk unit weight; the slag fill would have to be tapered into the conventional earth/granular fill used for the QEW embankment widening toward the east.

If spread footings are adopted for support of the replacement structure, it would be necessary to use EPS fill to achieve “zero net embankment loading” behind the abutments in the widening areas in order to limit the settlement due to foundation loading to 25 mm or less. However, there is concern regarding the durability of EPS blocks in the QEW embankment, based on the high traffic volume and heavy truck loading in this area; as a result, the use of spread footings with EPS fill is not a preferred option for this structure site. In terms of constructability, placement of EPS fill would require subexcavation at least 1.2 m deep into the existing embankment side slopes to key in the EPS fill (in order to supply a minimum of 1.2 m of soil cover above the EPS fill to minimize freezing/icing on the road surface). The EPS blocks would also have to be wrapped in polyethylene sheeting for protection, and covered with a concrete slab.

6.8 Excavation and Groundwater Control

6.8.1 Open-Cut Excavations

Excavations for the foundation elements will extend through the existing fill, and may extend into the underlying surficial sandy silt or clayey silt to silty clay till deposit if spread footings are adopted. Open-cut excavations into these materials should be carried out in accordance with the guidelines outlined in the Occupational Health and Safety Act (OHSA) for Construction Activities. The existing fill, the surficial and the upper silty sand to sand and silt soils are classified as Type 3 soil, according to the OHSA. Where space and construction staging layouts permit, temporary excavations (i.e. those which are open for a relatively short time period) should be made with side slopes no steeper than 1 horizontal to 1 vertical.

6.8.2 Temporary Roadway Protection

It is expected that excavation support will be required in some areas for temporary roadway protection to facilitate excavation for the new foundation elements for the replacement structure. Where required, the temporary excavation support system should be designed and constructed in accordance with MTO's Special Provision 105S19. The lateral movement of the temporary shoring system should meet Performance Level 2 as specified in SP 105S19.

6.8.3 Groundwater Control

The surficial sandy silt deposit, which is present in localized areas atop the till deposit, and the base of the existing embankment fill should be expected to be water-bearing, particularly during wet periods of the year, with groundwater "perched" atop the underlying, less permeable clayey silt to silty clay till deposit. It is anticipated that the groundwater in the excavations can be adequately controlled by pumping from properly filtered sumps.

7.0 CLOSURE

This Foundation Design Report was prepared by Ms. Houda Jadi, Ph.D., P.Eng, and reviewed by Ms. Lisa Coyne, P.Eng., an Associate and geotechnical engineer with Golder, with technical input from Mr. Murty Devata, P.Eng., a Specialist Foundations Consultant to Golder. Mr. Fintan Heffernan, P.Eng., a Designated MTO Contact for Golder, carried out an independent review of the report.

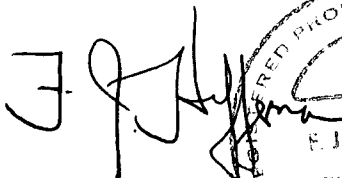
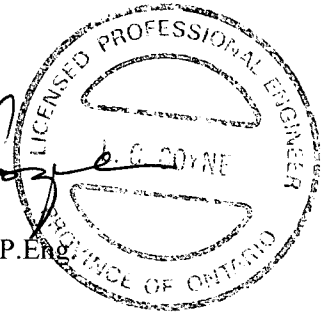
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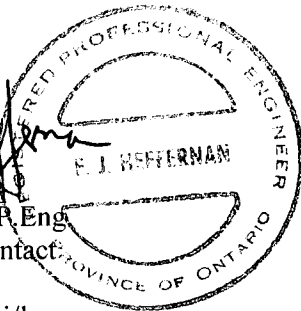
Houda Jadi, P.Eng
Geotechnical Engineer



Lisa C. Coyne, P.Eng
Associate



Fintan J. Heffernan, P.Eng
Designated MTO Contact



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TABLE 1
COMPARISON OF FEASIBLE FOUNDATION ALTERNATIVES
GENEVA STREET OVERPASS, G.W.P. 607-00-00

<i>Foundation Option</i>	<i>Feasibility</i>	<i>Advantages</i>	<i>Disadvantages</i>	<i>Constructability / Practicality</i>	<i>Relative Costs</i>
Spread footings on native till, with a wedge of EPS fill behind abutments	<ul style="list-style-type: none"> • Feasible for support of abutments 	<ul style="list-style-type: none"> • Potentially easier and faster construction 	<ul style="list-style-type: none"> • Lower geotechnical resistance • Potential for up to about 15 mm of differential settlement along abutments. • Significant excavation (through existing embankment fill) required. • Shallow foundations may not be compatible with 900 mm diameter combined sewer under west abutment footing. • Concern re: long-term durability of EPS fill on this heavily loaded and traveled section of QEW. 	<ul style="list-style-type: none"> • Conventional excavation and construction techniques, except for placement of EPS fill if adopted. • EPS fill installation would require more time and skill than conventional fill placement. As noted under disadvantages, there is concern regarding the long-term durability of the EPS fill under the high volume and heavy traffic loads on the QEW in this area, which makes this option less preferred. 	<ul style="list-style-type: none"> • Foundation costs less expensive than deep foundations. • However, higher costs related to lightweight fill and subexcavation.
Steel H-piles driven to found within 100-blow lower till or bedrock	<ul style="list-style-type: none"> • Feasible for support of abutment foundations 	<ul style="list-style-type: none"> • Minimize differential settlement across abutments and between foundation elements • Higher geotechnical resistance than for shallow foundations • Readily installed 	<ul style="list-style-type: none"> • Piles may “hang up” on boulders within lower till/residual soil deposit. • East abutment piles will have to accommodate downdrag loads, unless ultra-lightweight fill used for northward widening of east approach embankment. 	<ul style="list-style-type: none"> • Conventional construction methods for H-pile foundations. • Ultra-lightweight fill could be considered for the north part of the east abutment to reduce downdrag forces. Minor constructability issues related to placement and compaction of slag fill to preserve its structure and unit weight. 	<ul style="list-style-type: none"> • Less expensive than caissons.
Caissons bored to found within 100-blow lower till or bedrock	<ul style="list-style-type: none"> • Feasible for support of abutment foundations 	<ul style="list-style-type: none"> • Minimize differential settlement across abutments and between foundation elements • Higher bearing resistances than for steel H-piles 	<ul style="list-style-type: none"> • Possibility of basal heave. Temporary or permanent liner required due to soil conditions. • Difficulty with cobbles and boulders within lower till/residual soil deposit. • East abutment piles will have to accommodate downdrag loads, unless ultra-lightweight fill used for northward widening of east approach embankment. 	<ul style="list-style-type: none"> • Permanent liners recommended over temporary liners, to avoid difficulties with withdrawal of temporary liner due to length of caissons and presence of hard/very dense soils near caisson base, and to avoid “necking” of the caissons which occur as a result of the temporary liner withdrawal. 	<ul style="list-style-type: none"> • More expensive than steel H-piles, plus the cost of permanent liners.

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
DO	Drive open
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

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

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.)

Consistency

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

(b) Cohesive Soils

c_u, s_u

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.

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.

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 ($\sigma' = \sigma - u$)
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
u	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

(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 $= \sigma'_p / \sigma'_{vo}$

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

PROJECT <u>04-1111-002</u>		RECORD OF BOREHOLE No 401		1 OF 3 METRIC	
W.P. <u>607-00-00</u>		LOCATION <u>N 4781742.3 ; E 325860.8</u>		ORIGINATED BY <u>PKS</u>	
DIST <u>Central</u> HWY <u>QEW</u>		BOREHOLE TYPE <u>108 mm Diameter Solid Stem Augers</u>		COMPILED BY <u>HJ</u>	
DATUM <u>Geodetic</u>		DATE <u>June 29, 2005</u>		CHECKED BY <u>LCC</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	+ FIELD VANE						
								● QUICK TRIAXIAL	× REMOULDED						
105.7	GROUND SURFACE														
0.0	ASPHALT														
0.2	Sand and gravel (FILL) Compact to dense Red Moist		1	SS	40										
			2	SS	26										
103.4	Foundry sand (FILL) Dense to very dense Black Moist		3	SS	61										
2.3			4	SS	40										
			5	SS	47										
			6	SS	71										
			7	SS	87										
98.1	SILTY CLAY, some sand, trace gravel (TILL) Very stiff to hard Brown Moist Grey below 10.7 m depth		8	SS	40										
7.6			9	SS	30										
95.0	CLAYEY SILT, some sand, trace gravel and shale pieces (TILL) Stiff to very stiff Grey Moist		10	SS	21										
10.7			11	SS	12										
			12	SS	15										

Continued Next Page

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

MIS-MTO 001 041111002AAMTO.GPJ GAL-MISS.GDT 16/10/06


PROJECT <u>04-1111-002</u>		RECORD OF BOREHOLE No 401		2 OF 3 METRIC	
W.P. <u>607-00-00</u>		LOCATION <u>N 4781742.3 ; E 325860.8</u>		ORIGINATED BY <u>PKS</u>	
DIST <u>Central</u> HWY <u>QEW</u>		BOREHOLE TYPE <u>108 mm Diameter Solid Stem Augers</u>		COMPILED BY <u>HJ</u>	
DATUM <u>Geodetic</u>		DATE <u>June 29, 2005</u>		CHECKED BY <u>LCC</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/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)
								○ UNCONFINED	+ FIELD VANE						
								● QUICK TRIAXIAL	× REMOULDED						
	--- CONTINUED FROM PREVIOUS PAGE ---														
	CLAYEY SILT, some sand, trace gravel and shale pieces (TILL) Stiff to very stiff Grey Moist		13	SS	14										
			14	SS	14										
			15	SS	17										
			16	SS	20										
			17	SS	17										
	Becoming red below 25.0 m depth														
	Containing shale pieces below 27.4 m depth														
77.4			18	SS	25										
28.3	CLAYEY SILT, some sand, trace gravel and shale pieces (TILL/RESIDUAL SOIL) Hard Red Moist		19	SS	81										

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

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PROJECT <u>04-1111-002</u>		RECORD OF BOREHOLE No 401				3 OF 3 METRIC											
W.P. <u>607-00-00</u>		LOCATION <u>N 4781742.3 ; E 325860.8</u>				ORIGINATED BY <u>PKS</u>											
DIST <u>Central</u> HWY <u>QEW</u>		BOREHOLE TYPE <u>108 mm Diameter Solid Stem Augers</u>				COMPILED BY <u>HJ</u>											
DATUM <u>Geodetic</u>		DATE <u>June 29, 2005</u>				CHECKED BY <u>LCC</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 ---																
	CLAYEY SILT, some sand, trace gravel and shale pieces (TILL/RESIDUAL SOIL) Hard Red Moist		20	SS	153		75										11 21 55 13
								74									
			21	SS	100/0.13												
									73								
72.0 33.7	END OF BOREHOLE		22	SS	100/0.15												
	Note: 1.) Borehole dry upon completion of drilling operations.																

PROJECT <u>04-1111-002</u>		RECORD OF BOREHOLE No 402		1 OF 3 METRIC	
W.P. <u>607-00-00</u>		LOCATION <u>N 4781732.1 ; E 325899.5</u>		ORIGINATED BY <u>PKS</u>	
DIST <u>Central</u> HWY <u>QEW</u>		BOREHOLE TYPE <u>108 mm Diameter Solid Stem Augers</u>		COMPILED BY <u>HJ</u>	
DATUM <u>Geodetic</u>		DATE <u>June 27, 2005</u>		CHECKED BY <u>LCC</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/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								WATER CONTENT (%)
								○ UNCONFINED	+ FIELD VANE							
								● QUICK TRIAXIAL	× REMOULDED							
106.0	GROUND SURFACE						20 40 60 80 100									
0.0	ASPHALT															
0.2	Sand and gravel (FILL) Compact to very dense Red Moist		1	SS	53											
			2	SS	24											
			3	SS	13											
103.0																
3.1	Silty sand to sand, some silt, trace to some gravel (FILL) Loose to compact Red Moist		4	SS	7											
			5	SS	12											
			6	SS	4											
99.6			7	SS	30											
6.4	Sand and gravel (FILL) Compact to very dense Grey/brown Moist															
	Wet below 7.6 m depth		8	SS	50											
96.9																
9.1	CLAYEY SILT, some sand, trace gravel and shale pieces (TILL) Stiff to very stiff Grey Moist/wet		9	SS	16											
			10	SS	15											
			11	SS	15											
			12	SS	13											

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

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PROJECT <u>04-1111-002</u>		RECORD OF BOREHOLE No 402		2 OF 3 METRIC	
W.P. <u>607-00-00</u>		LOCATION <u>N 4781732.1 ; E 325899.5</u>		ORIGINATED BY <u>PKS</u>	
DIST <u>Central</u> HWY <u>QEW</u>		BOREHOLE TYPE <u>108 mm Diameter Solid Stem Augers</u>		COMPILED BY <u>HJ</u>	
DATUM <u>Geodetic</u>		DATE <u>June 27, 2005</u>		CHECKED BY <u>LCC</u>	

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 (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	W _p	W	W _L		
	--- CONTINUED FROM PREVIOUS PAGE ---													
88.2	CLAYEY SILT, some sand, trace gravel and shale pieces (TILL) Stiff to very stiff Grey Moist/wet		13	SS	11		90							
17.8	SILTY CLAY, some sand, trace gravel and shale fragments (TILL) Firm to Stiff Grey Wet		14	SS	10		89							
			15	SS	8		88							
			16	SS	8		86							
			17	SS	9		85							
			18	SS	10		83							
	Containing sand seams below 24.4 m depth		19	SS	11		81							
79.8	CLAYEY SILT, some sand, trace gravel, shale and limestone pieces (TILL) Hard Grey to red Wet		20	SS	43		79							
26.2			21	SS	93		77							
77.7	CLAYEY SILT, some sand, trace gravel, shale and limestone pieces (TILL/RESIDUAL SOIL) Hard Red Wet													

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

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

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PROJECT		04-1111-002		RECORD OF BOREHOLE No 403		1 OF 3 METRIC								
W.P.		607-00-00		LOCATION		N 4781714.1 ; E 325863.9								
DIST		Central HWY QEW		BOREHOLE TYPE		108 mm Diameter Solid Stem Augers								
DATUM		Geodetic		DATE		June 19, 2005								
				ORIGINATED BY		PKS								
				COMPILED BY		HJ								
				CHECKED BY		LCC								
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
105.8	GROUND SURFACE													
0.0	ASPHALT													
0.2	Sand and gravel (FILL) Loose to dense Reddish brown to red Moist		1	SS	38									
			2	SS	25									
			3	SS	8									
			4	SS	8									
102.0	Foundry sand (FILL) Compact to very dense Black Moist		5	SS	10									
3.8			6	SS	25									
			7	SS	82									
99.3	Sandy SILT Very dense Brown Moist													
6.6														
98.2	CLAYEY SILT to SILTY CLAY, trace sand, trace gravel (TILL) Stiff to very stiff Grey-brown Moist		8	SS	24									
7.6														
			9	SS	28									
	Becoming grey below 10.7 m depth		10	SS	14									
			11	SS	12									
			12	SS	14									
90.8														

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

MIS-MTO 001 041111002AAMTO.GPJ GAL-MISS.GDT 16/10/06


2 OF 3 METRIC

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

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PROJECT <u>04-1111-002</u>		RECORD OF BOREHOLE No 403				3 OF 3 METRIC							
W.P. <u>607-00-00</u>		LOCATION <u>N 4781714.1 ; E 325863.9</u>				ORIGINATED BY <u>PKS</u>							
DIST <u>Central</u> HWY <u>QEW</u>		BOREHOLE TYPE <u>108 mm Diameter Solid Stem Augers</u>				COMPILED BY <u>HJ</u>							
DATUM <u>Geodetic</u>		DATE <u>June 19, 2005</u>				CHECKED BY <u>LCC</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					
	--- CONTINUED FROM PREVIOUS PAGE ---						20 40 60 80 100						
72.3	CLAYEY SILT, trace sand, trace gravel (TILL) Hard Grey Moist Containing shale pieces and red in color below 30.5 m depth		21	SS	57		75						
33.5	CLAYEY SILT, some sand, containing shale pieces (TILL/RESIDUAL SOIL) Hard Red Moist		22	SS	100/20		72						
70.8	Red SHALE (BEDROCK)						71						
35.1							70						
69.2	END OF BOREHOLE		23	SS	100/07								
36.7	Note: 1.) Borehole dry upon completion of drilling operations.												

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PROJECT 04-1111-002			RECORD OF BOREHOLE No 404			1 OF 3 METRIC		
W.P. 607-00-00			LOCATION N 4781705.4 ; E 325898.0			ORIGINATED BY PKS		
DIST Central HWY QEW			BOREHOLE TYPE 108 mm Diameter Solid Stem Augers			COMPILED BY HJ		
DATUM Geodetic			DATE June 22, 2005			CHECKED BY LCC		
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	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 (%)
106.1	GROUND SURFACE							
0.0	ASPHALT						106	
0.2	Sand and gravel (FILL) Compact Red Moist		1	SS	24		105	
104.6								
1.5	Silty sand to sand, some silt, trace gravel (FILL) Compact to dense Reddish brown Moist		2	SS	25		104	
			3	SS	27		103	
			4	SS	24		102	
			5	SS	21		101	
			6	SS	22		100	
99.7			7	SS	40		99	
6.4	Foundry sand (FILL) Dense Black Moist						98	
98.5			8	SS	41		97	
7.6	CLAYEY SILT, trace sand, trace gravel (TILL) Very stiff to hard Brown Moist		9	SS	28		96	
	Grey below 9.0 m depth		10	SS	18		95	
	Wet below 10.7 m depth		11	SS	13		94	
94.5			12	SS	13		93	
11.6	CLAYEY SILT, trace sand and gravel (TILL) Stiff to very stiff Grey Wet						92	

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

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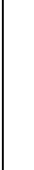
PROJECT <u>04-1111-002</u>		RECORD OF BOREHOLE No 404		2 OF 3 METRIC	
W.P. <u>607-00-00</u>		LOCATION <u>N 4781705.4 ; E 325898.0</u>		ORIGINATED BY <u>PKS</u>	
DIST <u>Central</u> HWY <u>QEW</u>		BOREHOLE TYPE <u>108 mm Diameter Solid Stem Augers</u>		COMPILED BY <u>HJ</u>	
DATUM <u>Geodetic</u>		DATE <u>June 22, 2005</u>		CHECKED BY <u>LCC</u>	

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 (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		WATER CONTENT (%)			
								20 40 60 80 100	20 40 60 80 100	W _p W W _L			
--- CONTINUED FROM PREVIOUS PAGE ---								○ UNCONFINED + FIELD VANE					
							● QUICK TRIAXIAL × REMOULDED						
							20 40 60 80 100			10 20 30			
											</		

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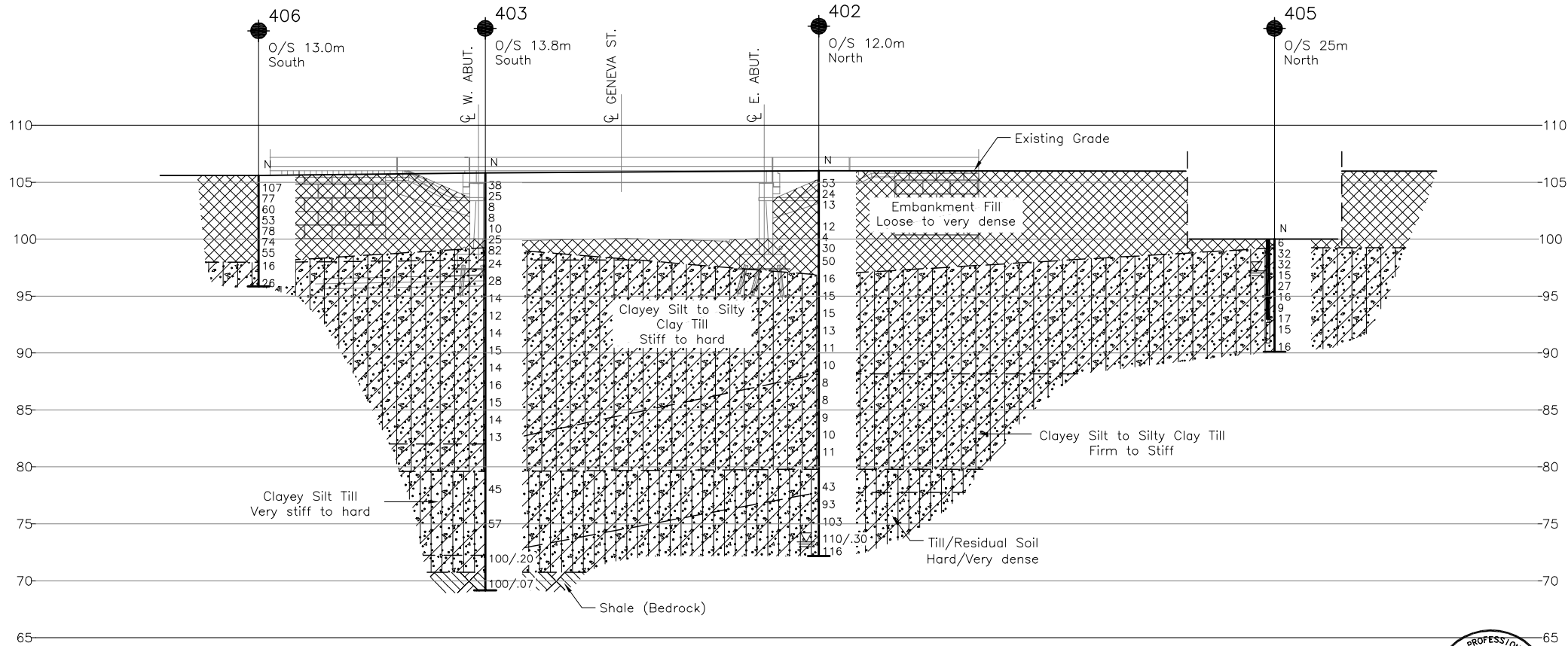
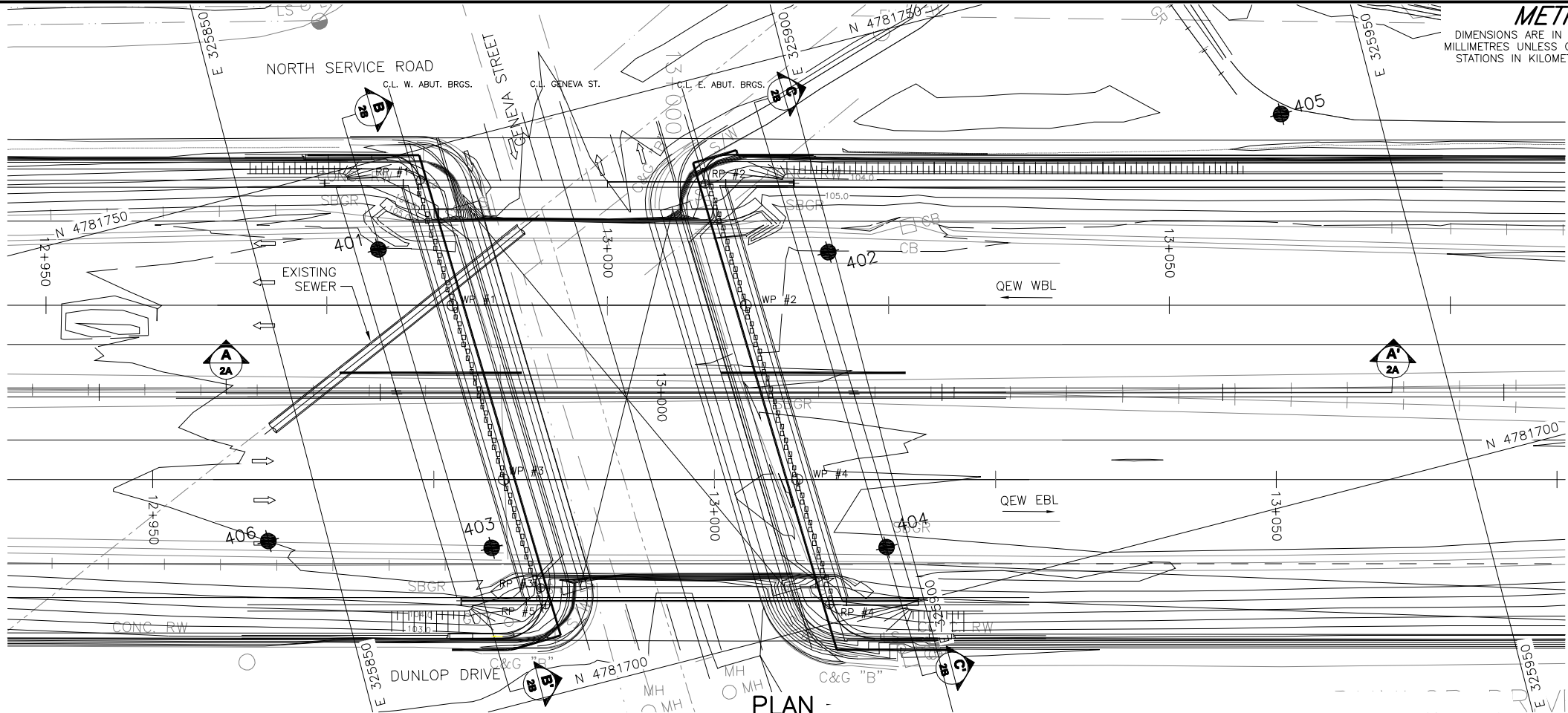
PROJECT		RECORD OF BOREHOLE No 404				3 OF 3 METRIC										
W.P. 607-00-00		LOCATION N 4781705.4 ;E 325898.0				ORIGINATED BY PKS										
DIST Central HWY QEW		BOREHOLE TYPE 108 mm Diameter Solid Stem Augers				COMPILED BY HJ										
DATUM Geodetic		DATE June 22, 2005				CHECKED BY LCC										
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa								WATER CONTENT (%)
	--- CONTINUED FROM PREVIOUS PAGE ---															
75.6	SAND and SILT, trace to some gravel and shale fragments, trace clay (TILL/RESIDUAL SOIL) Very dense Grey to red Moist to wet														5 36 53 6	
30.5																
72.5	CLAYEY SILT, some sand trace gravel and shale pieces (TILL/RESIDUAL SOIL) Hard Red Wet															
33.6																
70.9	END OF BOREHOLE															
35.2																
	Note: 1.) Open borehole wet below 10.7m depth upon completion of drilling operations.															

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PROJECT 04-1111-002			RECORD OF BOREHOLE No 405			1 OF 1 METRIC		
W.P. 607-00-00			LOCATION N 4781733.9; E 325941.6			ORIGINATED BY PKS		
DIST Central HWY QEW			BOREHOLE TYPE 108 mm Diameter Solid Stem Augers			COMPILED BY HJ		
DATUM Geodetic			DATE July 28, 2005			CHECKED BY LCC		
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100
100.0	GROUND SURFACE							PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT W _p W W _L WATER CONTENT (%)
0.0	TOPSOIL		1	SS	6			
99.2	CLAYEY SILT, some sand, trace to some gravel, trace organics Firm Dark brown Moist		2	SS	32		99	
0.8	CLAYEY SILT to SILTY CLAY, some sand, trace gravel (TILL) Stiff to hard Brown Moist		3	SS	32		98	
	Wet below 2.1 m depth		4	SS	15		97	
			5	SS	27		96	
	Becoming grey below 4.5 m depth		6	SS	16		95	
			7	SS	9		94	
			8	SS	17		93	
			9	SS	15		92	
			10	SS	16		91	
90.1	END OF BOREHOLE							
9.9	Note: 1. Water level measured in open borehole at 8.2 m depth upon completion of drilling operations. 2. Water level measured in piezometer at 2.8 m depth (Elevation 97.2 m) on August 8, 2005. 3. Water level measure in piezometer at 0.6 m depth (Elevation 99.4 m) on December 6, 2005.							

MIS-MTO 001 041111002AAMTO.GPJ GAL-MISS.GDT 16/10/06

PROJECT		04-1111-002		RECORD OF BOREHOLE No 406		1 OF 1 METRIC											
W.P.		607-00-00		LOCATION		N 4781719.6 ; E 325844.8											
DIST		Central HWY QEW		BOREHOLE TYPE		108 mm Diameter Solid Stem Augers											
DATUM		Geodetic		DATE		June 17, 2005											
				ORIGINATED BY		PKS											
				COMPILED BY		HJ											
				CHECKED BY		LCC											
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa					WATER CONTENT (%)			γ kN/m³	GR SA SI CL
								20 40 60 80 100	20 40 60 80 100	W _p	W	W _L					
105.6	GROUND SURFACE																
0.0	ASPHALT																
0.2	Sand and gravel (FILL) Very dense Red Moist						105										
104.5			1	SS	107												
1.1	Foundry sand (FILL) Very dense Black Moist						104										
			2	SS	77												
							103										
			3	SS	60												
							102										
			4	SS	53												
							101										
			5	SS	78												
							100										
			6	SS	74												
99.2							99										
6.4	Sand and gravel (FILL) Very dense Grey Moist		7	SS	55												
98.0							98										
7.6	Sandy SILT Compact Brown Wet		8	SS	16												
							97										
96.2							96										
95.9	CLAYEY SILT, some sand, trace gravel (TILL) Very stiff Grey Wet		9	SS	26												
9.8	END OF BOREHOLE																
	Note: 1. Bottom of borehole wet upon completion of drilling operations.																



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

CONT No.
WP No.607-00-00

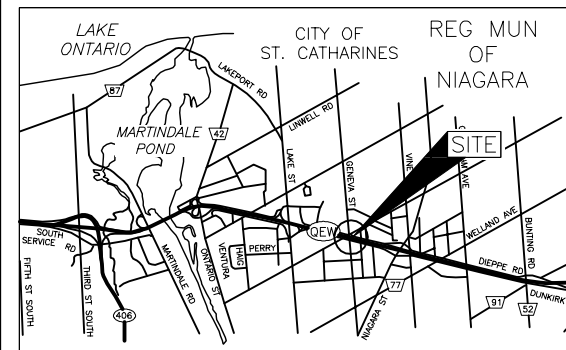
GENEVA STREET OVERPASS
QEW WIDENING, ST. CATHARINES
BOREHOLE LOCATION AND
SOIL STRATA



SHEET



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)
- 100% Rock Quality Designation (RQD)
- WL upon completion of drilling

No.	ELEVATION	CO-ORDINATES	
		NORTHING	EASTING
401	105.7	4781742.3	325860.8
402	106.0	4781732.1	325899.5
403	105.8	4781714.1	325863.9
404	106.1	4781705.4	325898.0
405	100.0	4781733.9	325941.6
406	105.6	4781719.6	325844.8

NOTES

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

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

The complete 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 Morrison Hershfield Limited, drawing file nos. 18107-01.dwg, received October 16, 2006; x4026design.dwg and x4026baseplan.dwg, received March 21, 2005.



NO.	DATE	BY	REVISION
Geocres No.			
HWY. QEW		PROJECT NO. 04-1111-002	DIST.
SUBM'D. PKS	CHKD. HJ	DATE: OCT 2006	SITE:
DRAWN: MSM	CHKD. HJ	APPD. LCC	DWG. 1

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

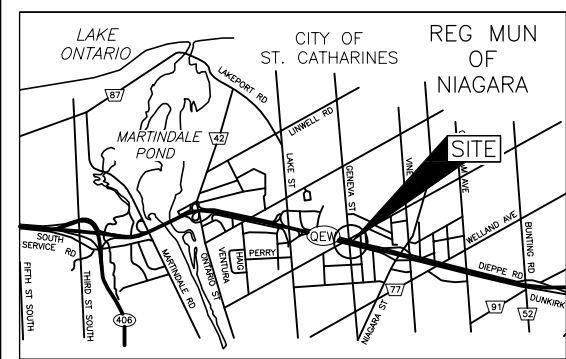
CONT No.
WP No.607-00-00

GENEVA STREET OVERPASS
QEW WIDENING, ST. CATHARINES
SOIL STRATA

SHEET



Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



KEY PLAN

SCALE
0 1 km

LEGEND

- Borehole - Current Investigation
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- 100% Rock Quality Designation (RQD)
- WL upon completion of drilling

No.	ELEVATION	CO-ORDINATES	
		NORTHING	EASTING
401	105.7	4781742.3	325860.8
402	106.0	4781732.1	325899.5
403	105.8	4781714.1	325863.9
404	106.1	4781705.4	325898.0

NOTES

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

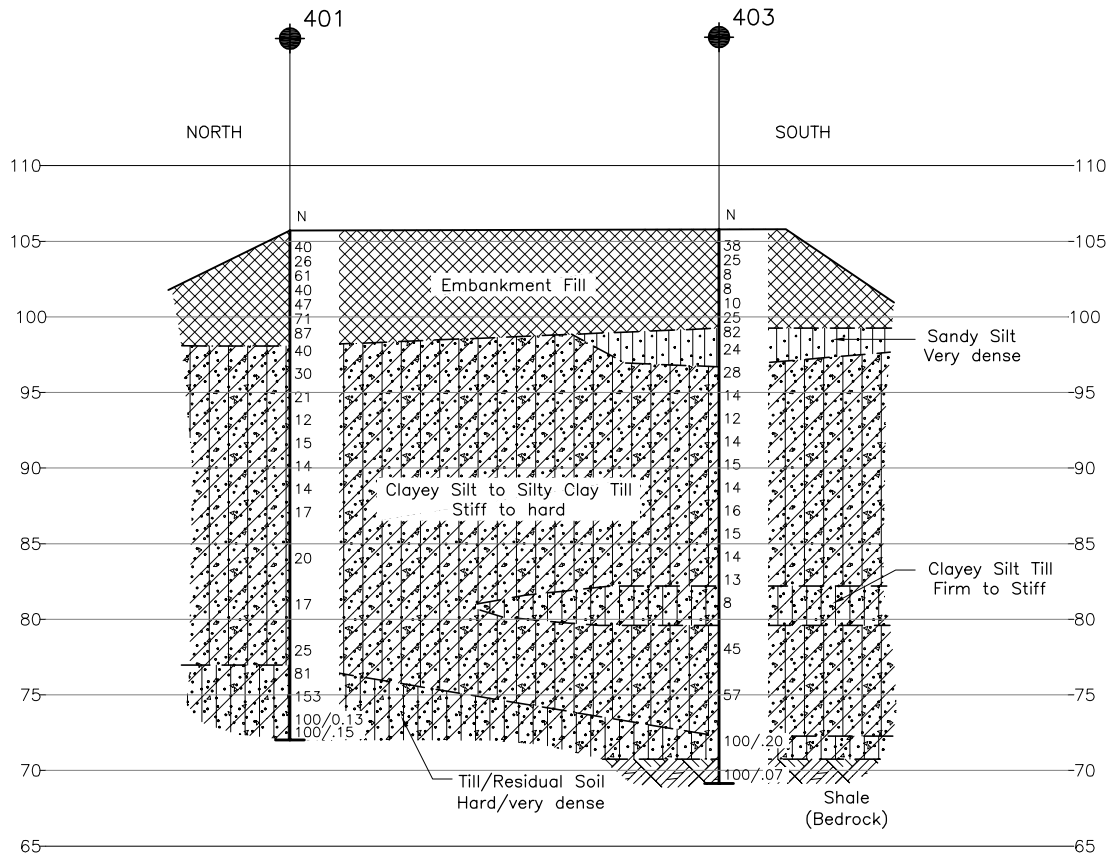
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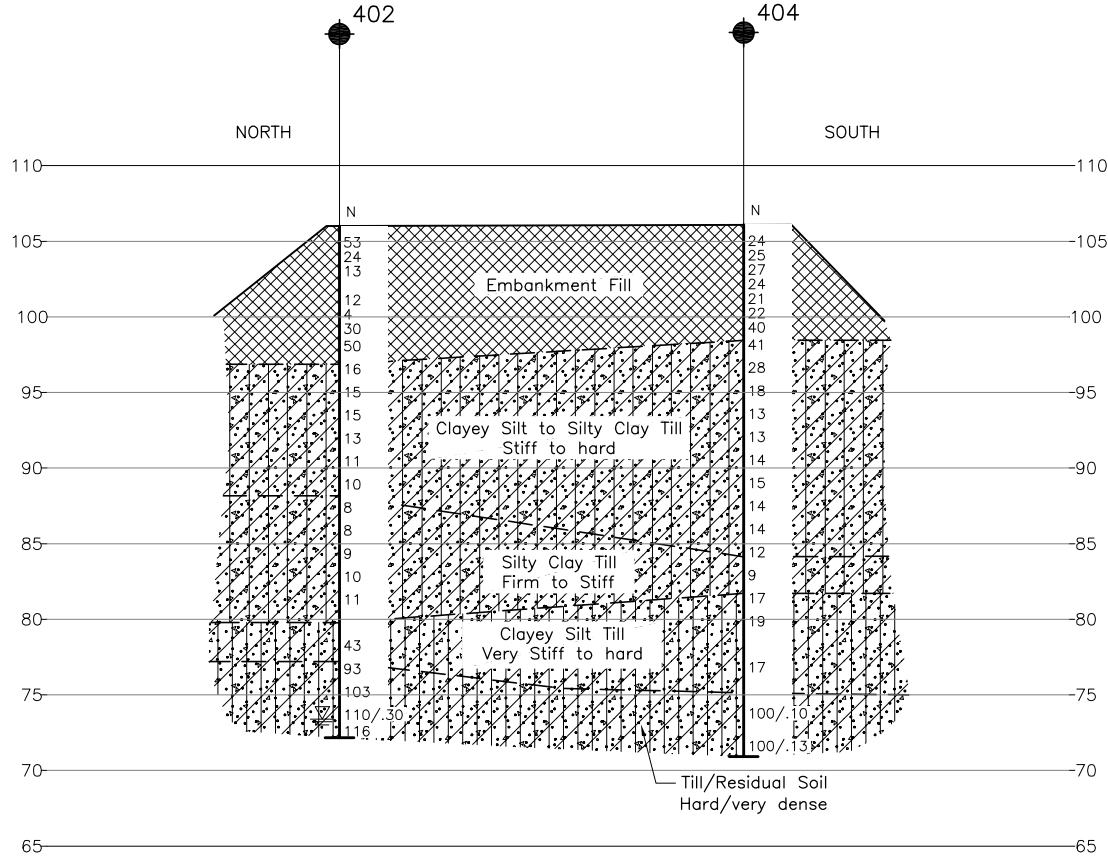
REFERENCE

Base plans provided in digital format by Morrison Hershfield Limited, drawing file nos. 18107-01.dwg, received October 16, 2006; x4026design.dwg and x4026baseplan.dwg, received March 21, 2005.

NO.	DATE	BY	REVISION
Geocres No.			
HWY. QEW	PROJECT NO. 04-1111-002		DIST.
SUBM'D. PKS	CHKD. HJ	DATE: OCT 2006	SITE:
DRAWN: MSM	CHKD. HJ	APPD. LCC	DWG. 2



B-B'
WEST ABUTMENT



C-C'
EAST ABUTMENT

SECTIONS

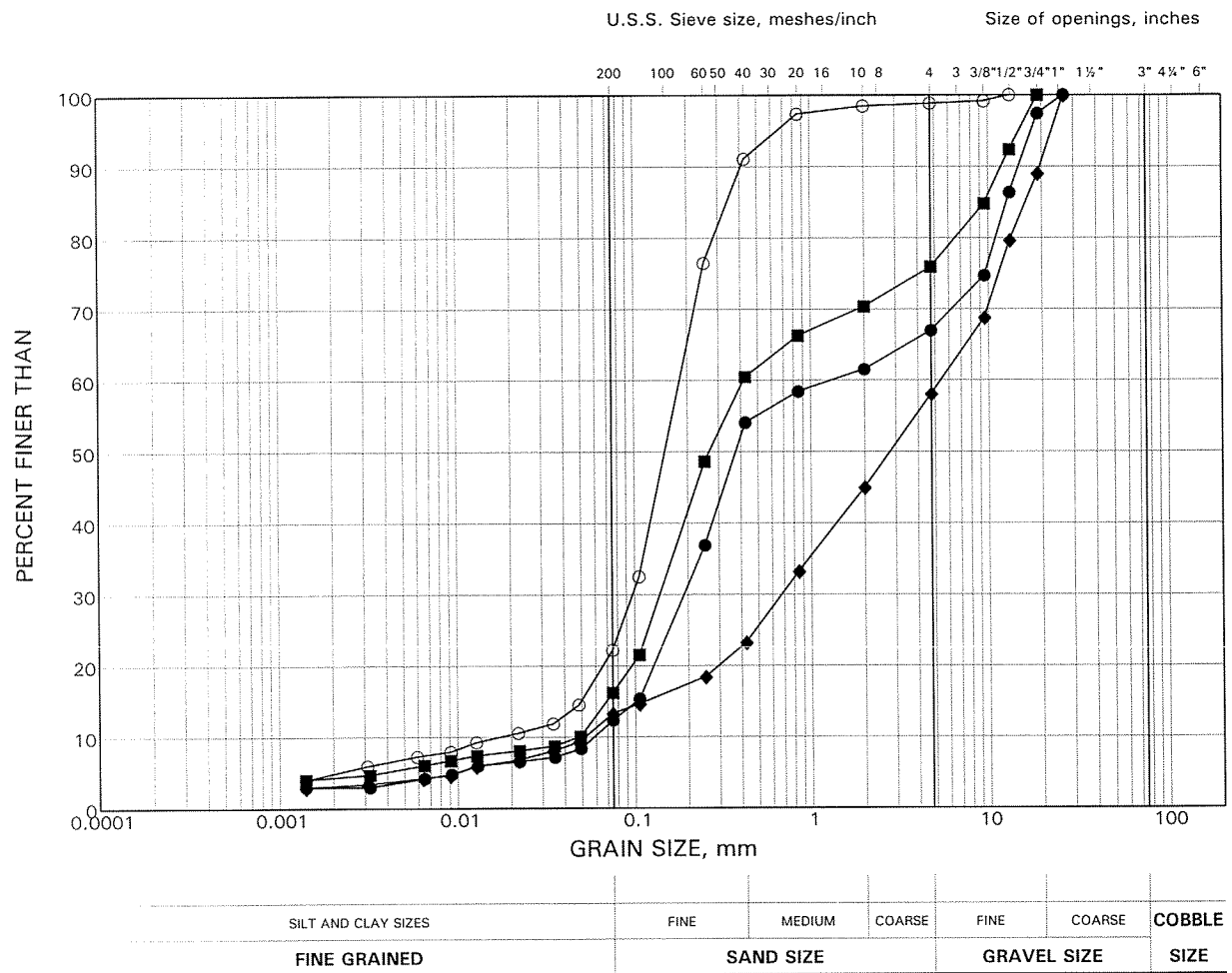
SCALE
5 0 5 10 m



GRAIN SIZE DISTRIBUTION TEST RESULTS

Fill

FIGURE 1



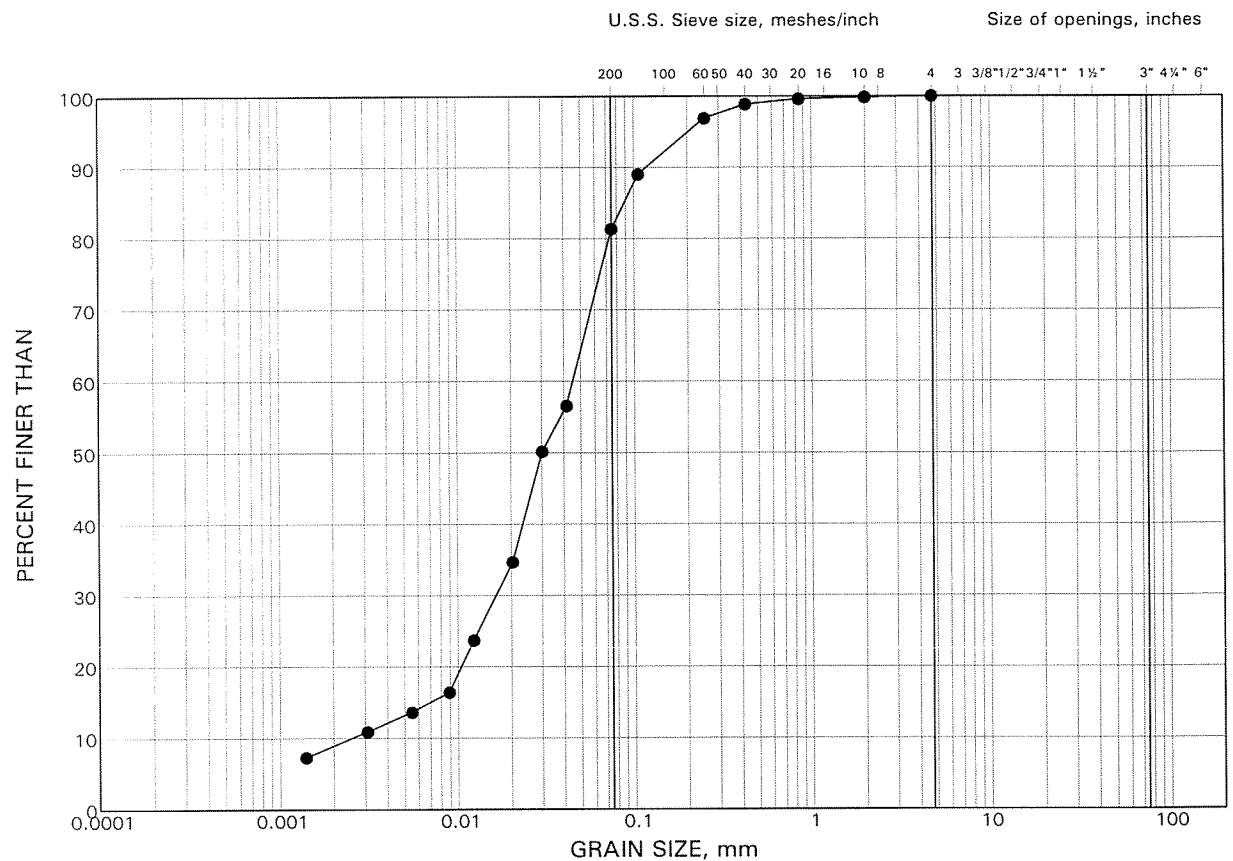
LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
●	401	7	99.3
■	402	4	102.7
◆	403	4	102.4
○	404	4	102.8

GRAIN SIZE DISTRIBUTION

Surficial Sandy Silt

FIGURE 2



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

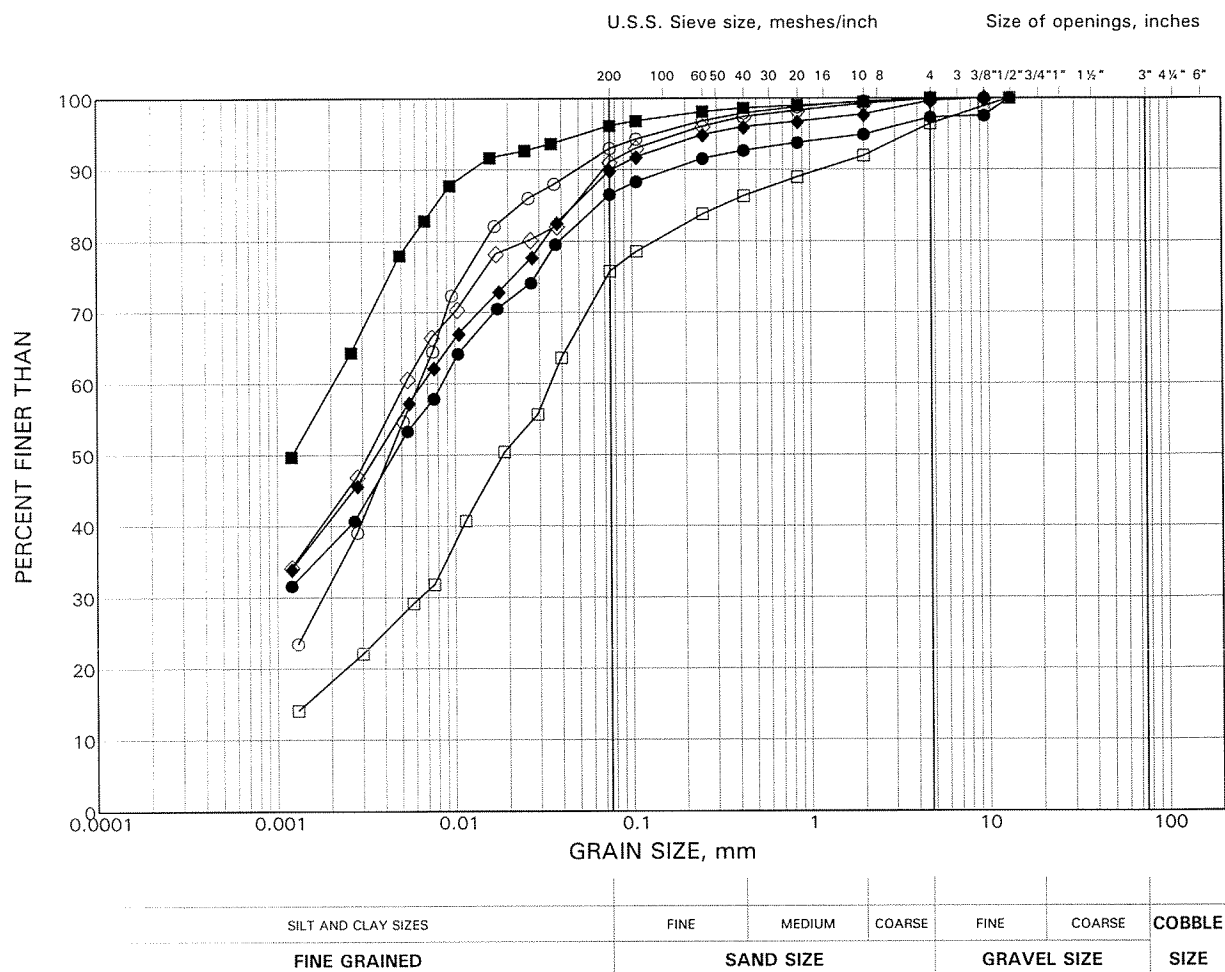
LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
•	406	8	97.7

GRAIN SIZE DISTRIBUTION TEST RESULTS

Clayey Silt to Silty Clay Till

FIGURE 3



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
●	401	11	93.2
■	402	16	85.9
◆	403	9	96.3
○	403	19	81.1
□	404	20	79.9
◇	405	7	95.1

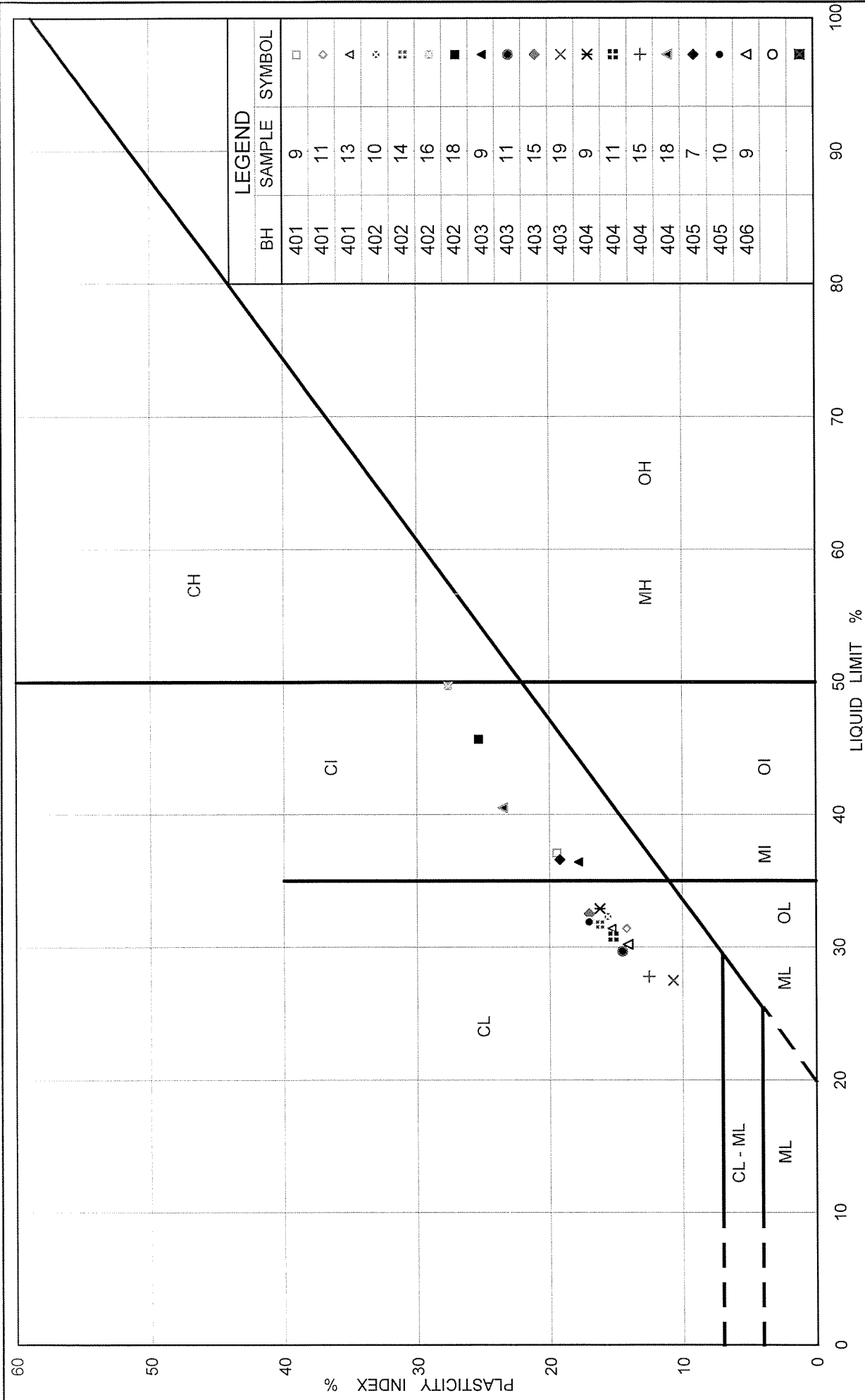


FIG No. 4

PLASTICITY CHART
Clayey Silt to Silty Clay Till

Ministry of Transportation



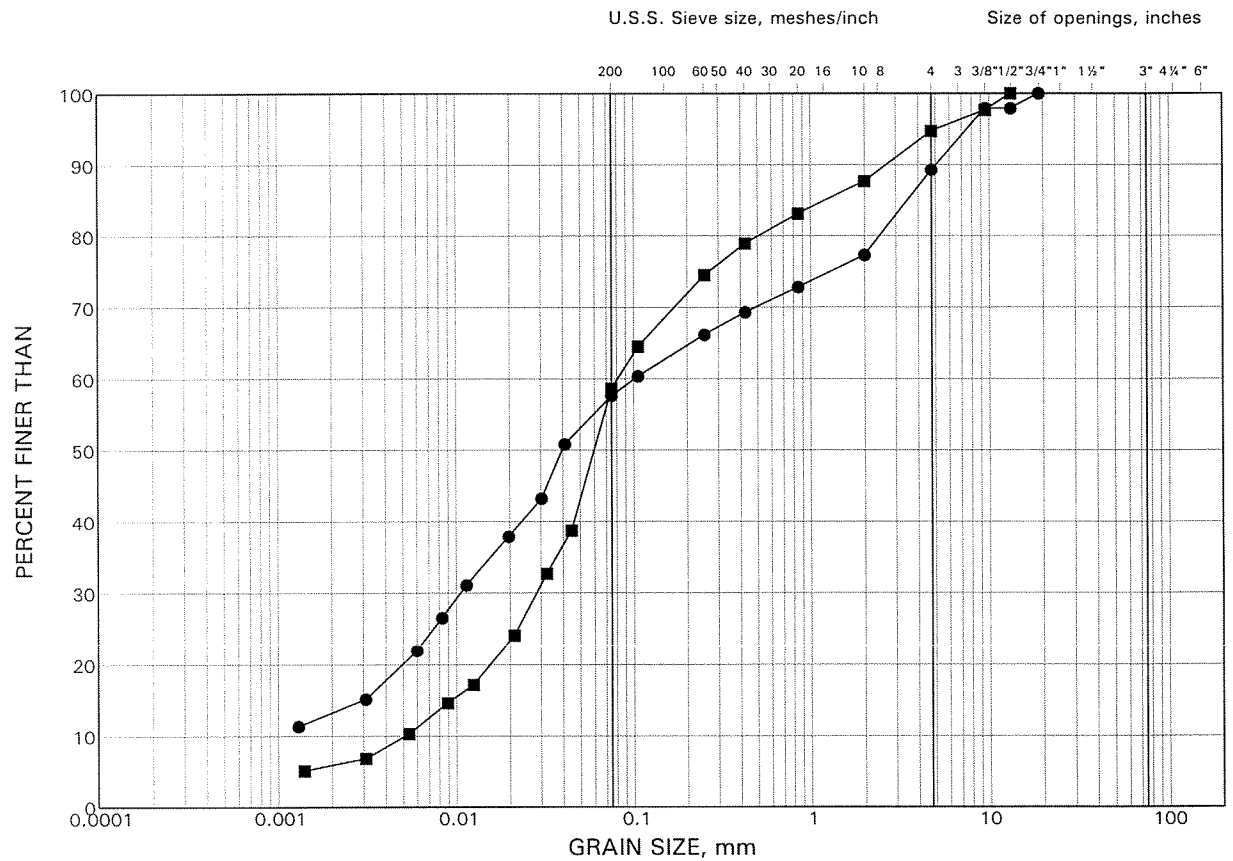
Ontario

Project No. 04-1111-002

GRAIN SIZE DISTRIBUTION TEST RESULTS

Residual Soil

FIGURE 5



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
●	401	21	73.6
■	404	22	74.1

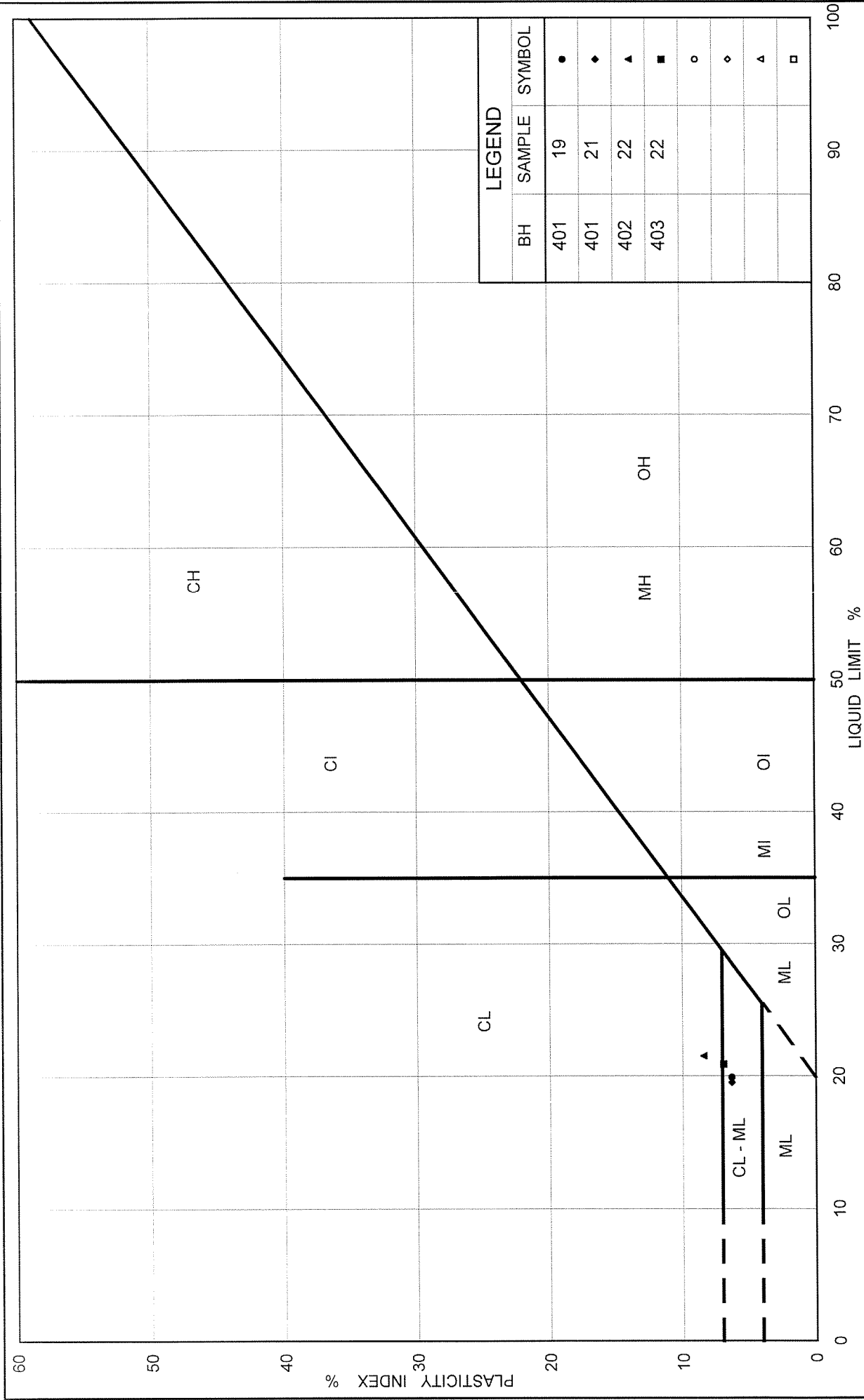


FIG No. 6

PLASTICITY CHART Clayey Silt Till / Residual Soil

Ministry of Transportation



Ontario

Project No. 04-1111-002

APPENDIX A

NON-STANDARD SPECIAL PROVISIONS

BOULDERS/OBSTRUCTIONS DURING PILE INSTALLATION - Item No.

Special Provision

The soils at the site are glacially-derived and should be expected to contain cobbles and boulders. Appropriate equipment and procedures will be required to penetrate obstructions (cobbles and boulders) that are encountered during pile driving.

Basis of Payment

Payment at the lump sum contract price for this tender item shall be full compensation for all labour, equipment and materials for completion of the work.

END OF SECTION