FOUNDATION INVESTIGATION AND DESIGN REPORT
MUNICIPAL CLASS ENVIRONMENTAL
ASSESSMENT STUDY
RIDGEWAY DRIVE/HIGHWAY 403 GRADE SEPARATION
MISSISSAUGA, ONTARIO
PROCUREMENT NO: FA.49.333-05

Submitted to:

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1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by Philips Engineering Ltd. (Philips) on behalf of the Corporation of the City of Mississauga to provide foundation and geotechnical (pavement) engineering services as part of the Municipal Class Environmental Assessment study and subsequent preliminary and detail design for the Ridgeway Drive / Highway 403 grade separation in Mississauga, Ontario. The proposed bridge structure will connect Ridgeway Drive south and north of Highway 403 between Unity Drive and Angel Pass Drive. Foundation engineering services were also required for the preliminary design of the proposed Ridgeway Drive crossing of the Transit Way, located to the north of Highway 403, for the Bus Rapid Transit (BRT) proposed in the Highway 403 / Eglinton Avenue Corridor.

This report addresses the foundation investigation carried out for the Ridgeway Drive crossing of Highway 403 and the Highway 407 ramps. The results of the geotechnical (pavement) investigation carried out for the Ridgeway Drive tie-in locations at the north and south limits of the project are presented in a separate report, a copy of which is included in Appendix B. The results of the foundation investigation for the Ridgeway Drive crossing of the Transit Way are presented in a separate report.

The terms of reference and scope of work for the foundation and geotechnical investigations are outlined in The Corporation of the City of Mississauga's Request for Proposal (RFP) document for Procurement No. FA.49.333-05 and in Golder's Proposal No. P51-1837 dated January 6, 2006.

This report should be read in conjunction with the "Important Information and Limitations of this Report" following the text of the report. The reader's attention is specifically drawn to this information, as it is essential for the proper use and interpretation of this report.

2.0 SITE DESCRIPTION

The site of the proposed Ridgeway Drive crossing of Highway 403/Highway 407 ramps is located between Winston Churchill Boulevard and Ninth Line, approximately one kilometre west of Winston Churchill Boulevard in Mississauga, Ontario. The key plan on Drawings 1 to 4 provides an overview of the site location. The proposed Ridgeway Drive overpass structure will connect the existing Ridgeway Drive from the intersection of Ridgeway Drive and Unity Drive south of Highway 403 to the intersection of Ridgeway Drive and Angel Pass Drive north of the highway.

The terrain in this area is generally flat-lying with the exception of two drainage ditches that run along the north and south sides of Highway 403 and an embankment / berm, approximately 5 m high relative to the surface of the highway, located directly south of the Highway 403 Eastbound lanes. A natural water course cuts through the relatively flat-lying field north of Highway 403/Highway 407 ramps and flows to the south through a box culvert, about 100 m east of the site. Fill materials have been locally placed along the north and south sides of Highway 403 and the grade across the site varies between approximately Elevations 176.6 m and 181.2 m, while the Highway 403/Highway 407 ramp grades vary from about Elevation 179.2 m to Elevation 179.5 m at the proposed bridge, based on the topographic plan provided by Philips.

3.0 INVESTIGATION PROCEDURES

A subsurface investigation was carried out at the site of the proposed Ridgeway Drive Bridge crossing Highway 403/Highway 407 ramps between January 22 and February 2, 2007. At this time, sixteen (16) boreholes (numbered BH1 to BH13, BH18, BH20 and BH21) were advanced at the site using a track-mounted CME 55 drill rig supplied and operated by Geo-Environmental Drilling Ltd. of Milton, Ontario.

The boreholes were advanced using 108 mm inside diameter (I.D.) continuous flight hollow stem augers and 102 mm outside diameter (O.D.) solid stem augers, to depths ranging from 6.5 m to 18.3 m below the existing ground surface. Soil samples were obtained at 0.75 m and 1.5 m intervals of depth, using 50 mm O.D. split-spoon samplers in accordance with the Standard Penetration Test (SPT) procedure. Samples of the bedrock were obtained using an ‘NQ’ size core barrel.

The groundwater conditions in the open boreholes were observed during the drilling operations and three piezometers were installed; one in each of Boreholes BH5, BH12 and BH18 to permit monitoring of the groundwater level at the site. The piezometers consist of 50 mm diameter PVC pipe with 3 m long screens surrounded by a sand pack, sealed with bentonite from the top of the sand pack to the ground surface. The installation details and water level readings are described on the Record of Borehole sheets that follow the text of this report. Upon completion of the drilling operations, the boreholes were backfilled to the ground surface using bentonite pellets, as per Ontario Regulation 128 (amendment to O.Reg. 903).

The field work was monitored on a full-time basis by a member of Golder’s engineering staff who arranged for the clearance of underground utility services, directed the 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 boreholes were located in the field and surveyed by Philips based on the borehole location plan prepared by Golder. Where boreholes were shifted at the time of drilling, the northings, eastings and elevations of the as-drilled boreholes were measured in the field by a member of our engineering staff relative to the locations staked by Philips.

The borehole locations (including UTM NAD83 northing and easting coordinates) and ground surface elevations (referenced to geodetic datum) are shown on Drawings 1 to 4 and summarized below.

Borehole Number	***Borehole Locations***	***UTM NAD83 Northing (m)***	***UTM NAD83 Easting (m)***	***Ground Surface Elevation (m)***
BH-1	South Approach	4820789.2	603605.4	180.5
BH-2	South Approach	4820831.8	603579.3	180.2
BH-3	South Approach/ High Fill	4820871.1	603548.2	180.2
BH-4	South Approach/ High Fill	4820905.2	603511.4	181.2
BH-5	South Abutment	4820914.0	603487.7	178.4
BH-6	South Abutment	4820927.5	603502.1	183.0
BH-7	South Pier	4820934.8	603457.6	179.3
BH-8	South Pier	4820952.8	603469.6	179.3
BH-9	North Pier	4820951.1	603449.2	179.0
BH-10	North Pier	4820965.9	603459.5	179.3
BH-11	North Abutment	4820973.6	603423.8	176.7
BH-12	North Abutment	4820987.6	603433.6	176.6
BH-13	North Approach/High Fill	4820997.5	603410.1	179.1
BH-18	North Approach/High Fill	4821028.4	603386.5	178.6
BH-20	North Approach/High Fill	4821058.6	603342.9	176.9
BH-21	North Approach	4821098.4	603299.1	179.9

4.0 SITE GEOLOGY AND STRATIGRAPHY

4.1 Regional Geology

The site is located on the border of the physiographic regions known as the Peel Plain and the Trafalgar Moraine portion of the South Slope. This area slopes gradually downward towards Lake Ontario. The overburden generally consists of silty clay to clayey silt till with significant shale content. The till in turn overlies shale bedrock of the Queenston Formation, with interbedded grey limestone / siltstone layers (Chapman, L.J. and Putnam, D.F. "The Physiography of Southern Ontario", 3rd Edition, 1984).

4.2 Site Stratigraphy

Sixteen boreholes were advanced at the site of the proposed Ridgeway Drive overpass at the locations shown on Drawings 1 to 4. Four boreholes were drilled at the locations of the proposed south and north abutments, four boreholes were advanced at the locations of the proposed south and north piers, and eight boreholes were advanced along the south and north approach embankments and high fills.

The detailed subsurface soil, bedrock, 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 the results of the laboratory tests are also presented on Figures 1 to 12. The stratigraphic boundaries shown on the Record of Borehole sheets are inferred from non-continuous sampling, observations of drilling progress and the results of Standard Penetration Tests (SPTs). These boundaries, therefore, represent transitions between soil types rather than exact planes of geological change. The subsoil conditions will vary between and beyond the borehole locations. The inferred soil stratigraphies based on the results of the boreholes are shown on Drawings 1 to 4.

In summary, the boreholes advanced to the south of Highway 403/Highway 407 ramps (BH1 to BH6) encountered fill materials beneath a thin layer of topsoil, except at the location of Borehole BH-5. The fill materials vary in composition from sandy silt to clayey silt and are in turn underlain by a deposit of clayey silt till. At the location of Boreholes BH1 to BH4, a layer of very dense grey sandy silt or a layer of hard clayey silt to silty clay underlies the till. In Boreholes BH5 and BH6, the clayey silt till, which was encountered directly below the topsoil in Borehole BH5, is underlain by clayey silt to silty clay, silty sand to sandy silt till, and/or a layer of sandy silt residual soil layer overlying shale bedrock.

The boreholes drilled within the paved and unpaved highway shoulders (BH7 to BH10) encountered asphalt and/or granular sand and gravel road base materials underlain by a 'probable' fill layer consisting of brown clayey silt with some sand; these 'probable' fill materials appear to consist of on-site borrow materials potentially used to backfill a pre-existing valley associated with the water course located about 100 m north of the site. The fill materials are in turn underlain by a deposit of clayey silt till, which overlies a layer of clayey silt to silty clay and/or silty sand to sandy silt till, overlying a layer of silty sand residual soil and/or shale bedrock.

The subsoils encountered in the boreholes drilled to the north of Highway 403/Highway 407 ramps (BH-11 to BH-13, BH-18, BH-20, and BH-21) typically consist of a thin layer of topsoil overlying a deposit of very stiff to hard clayey silt till. In Borehole 21, a surficial layer of clayey silt fill was first encountered at the ground surface. The clayey silt till is underlain by a deposit of very dense silty sand to sandy silt till, underlain by a layer of hard clayey silt to silty clay and very dense silty sand residual soil at the location of Boreholes BH-11, BH-12, and BH-18. Boreholes BH-13, BH-20 and BH-21 were terminated within the deposit of silty sand to sandy silt till. Shale bedrock was encountered in Boreholes BH-11 and BH-12.

The bedrock at the site consists of the red-brown shale of the Queenston Formation and contains interbeds of grey siltstone and occasional limestone.

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

4.2.1 Topsoil / Asphalt

A surficial layer of topsoil or asphalt was encountered at the ground surface in all but three boreholes – BH9, BH10, and BH21; Boreholes BH7 and BH8 which were advanced on the south shoulder of the ramp to Highway 407 West encountered a 300 mm and 200 mm thick layer of asphalt, respectively. The layer of topsoil encountered in the boreholes ranges in thickness between 200 mm and 400 mm.

4.2.2 Fill

Fill materials were encountered in Boreholes BH1 to BH4, BH6 to BH10, and BH21. The fill in Boreholes BH1 to BH4 and BH6, advanced south of Highway 403 /Highway 407 ramps consist of sandy silt and clayey silt with trace to some gravel and occasional boulders both within the fill and on the ground surface. In this area, the fill extends to depths ranging from 0.9 m in Borehole BH1 to 5.2 m in Borehole BH-6, and from Elevation 179.6 m to Elevation 177.6 m. It is noted that the fill thickness is greatest at the location of Borehole BH-6 which is located at the top of the soil embankment / berm just south of Highway 403 Eastbound lanes. Standard Penetration Tests (SPT) ‘N’ values measured within the fill materials in Boreholes BH1 to BH4 and BH6, range from 4 to 15 blows per 0.3 m of penetration, indicating a generally firm to stiff consistency. One SPT ‘N’ value measured within the fill in Borehole BH2 was 50 blows per 0.05 m of penetration indicating the presence of a boulder within the fill materials; this borehole was subsequently moved about 3 m north of the original borehole location due to refusal to further advance of the augers in the original borehole. Occasional cobbles and boulders were noted within the fill materials in the boreholes located to the south of Highway 403/Highway 407 ramps. The water content measured on six samples of these fill materials range from about 14 percent to 21 percent.

Fill materials encountered beneath the surficial asphalt in Boreholes BH7 and BH8 and at the ground surface in Boreholes BH9 and BH10 in the area of the central piers consist of sand and gravel forming the granular road base materials underlain by a layer of 'probable' fill comprised of clayey silt with sand. This clayey silt fill appear to consist of on-site borrow materials potentially used to backfill a pre-existing valley associated with the water course located about 100 m north of the site. The thickness of the granular base varies between 0.5 m and 1.8 m and the thickness of the clayey silt fill varies from about 2.7 m in Borehole BH7 to 3.7 m in Borehole BH9, corresponding to between Elevation 176.0 m and Elevation 174.6 m at the base of the fill layer.

Standard Penetration Tests (SPT) 'N' values measured within the granular fill range between 17 and 31 blows per 0.3 m of penetration, indicating a compact to dense relative density, while SPT 'N' values measured within the clayey silt fill range between 6 and 20 blows per 0.3 m of penetration, indicating a firm to very stiff consistency. Water contents measured on eight samples of these fill materials range from 13 percent to 28 percent. The results of a grain size distribution test and an Atterberg limits test carried out on a selected sample of the clayey silt fill materials are shown on Figures 1 and 2, respectively; the results of the Atterberg limits test indicated the fill to consist of a clayey silt of low plasticity.

North of Highway 403/Highway 407 ramps, borehole BH21 encountered a surficial layer of fill materials consisting of clayey silt with trace sand and organic matter, extending from the ground surface to a depth of about 2.5 m (Elevation 177.3 m). The fill encountered at this borehole location appears to have been placed directly on top of the original topsoil. Standard Penetration Tests (SPT) 'N' values measured within this fill material were both 1 blow per 0.3 m of penetration, indicating a very soft consistency. Water contents measured on the two samples of this fill are about 27 percent and 29 percent.

4.2.3 Clayey Silt with Sand (Till)

A deposit of brown to grey clayey silt with sand till was encountered below the topsoil or asphalt and/or fill materials in all the boreholes. The top of the clayey silt till deposit was encountered between Elevation 174.6 m and Elevation 179.6 m and its thickness varies from 1.4 m to 6.1 m. Standard Penetration Testing (SPT) 'N' values measured within the till deposit typically range from 19 blows per 0.3 m of penetration to 50 blows per 0.15 m of penetration, indicating a very stiff to hard consistency. One SPT 'N' value of 6 blows per 0.3 m of penetration was however measured in the upper portion of the till deposit in Borehole BH20, which is adjacent to the existing water course; this value indicates a firm consistency within the upper 1.2 m of the till deposit.

The results of the four grain size distribution tests carried out on selected samples of the till deposit are provided on Figure 3. Results of four Atterberg limit tests on selected samples of the till are shown on Figure 4 and summarized below; these tests indicate that the till deposit is classified as a clayey silt of low plasticity.

Borehole No.	***Sample No.***	***Borehole Elevation (m)***	***Liquid Limit (%)***	***Plastic Limit (%)***	***Plasticity Index (%)***
BH-6	7	176.6	25	16	9
BH-10	5	174.4	21	14	7
BH-11	3	174.1	17	12	5
BH-20	3	174.3	24	15	9

The natural water content measured on selected samples of the clayey silt till deposit typically varies from about 6 percent to 13 percent, with an average of about 10 percent.

4.2.4 Sandy Silt

A deposit of grey sandy silt some clay, was encountered immediately below the clayey silt till deposit in Boreholes BH1, BH2 and BH3 to the south of Highway 403/Highway 407 Ramps. The top of the sandy silt deposit was encountered between Elevation 173.9 m and Elevation 175.5 m and all three boreholes were terminated within this deposit. Standard Penetration Testing (SPT) ‘N’ values measured within the sandy silt layer range between 55 and greater than 100 blows per 0.3 m of penetration, indicating a very dense relative density.

The results of one grain size distribution test are provided on Figure 5. The results of three natural water contents measured on samples of the sandy silt deposit range from about 11 percent to 17 percent with an average moisture content of 14 percent.

4.2.5 Clayey Silt to Silty Clay

A deposit of grey clayey silt to silty clay was encountered in eight boreholes advanced at the site (BH4 to BH9, BH11, and BH18). This deposit was found underlying the clayey silt till in Boreholes BH4, BH5, BH7 and BH9 at depths ranging from 4.4 m to 7 m below ground surface, with the top of the deposit ranging from Elevation 174.0 m to Elevation 172.0 m. In Boreholes BH6, BH8, BH11 and BH18, the clayey silt to silty clay layer was encountered underlying a deposit of silty sand to sandy silt till (see Section 4.2.6); the surface of the clay silt deposit in these boreholes range between Elevation 169.6 m and Elevation 166.7 m. Where fully penetrated (i.e. in Boreholes BH-6, BH-8, and BH-11), the clayey silt to silty clay deposit is about 1.6 m thick. Borehole BH-18 was terminated within this deposit; rock fragments were noted within the deposit at this location.

Standard Penetration Testing (SPT) 'N' values measured within the clayey silt to silty clay deposit range from 37 to greater than 100 blows per 0.3 m of penetration, indicating a hard consistency.

The results of four grain size distribution tests carried out on selected samples of the clayey silt deposit are provided on Figure 6. The results of Atterberg limits tests performed on two selected samples from this deposit are summarized below and presented on Figure 7; these results indicate that the deposit is a clayey silt to silty clay of low plasticity.

Borehole	***Sample***	***Elevation (m)***	***Liquid Limit (%)***	***Plastic Limit (%)***	***Plasticity Index (%)***
5	6	173.5	35	18	17
11	9	167.5	31	17	14

It is noted that a thin reddish brown interlayer of clayey silt with some sand, approximately 0.5 m thick, was encountered within the grey clayey silt to silty clay deposit in Borehole BH7; the results of a grain size distribution test carried out on a sample of this interlayer are shown on Figure 8.

The natural water content measured on selected samples of the clayey silt to silty clay deposit ranged between 12 percent and 17 percent, with an average moisture content of 14 percent.

4.2.6 Silty Sand to Sandy Silt (Till)

A deposit of grey to reddish brown silty sand to sandy silt till with trace to some clay was encountered directly below the clayey silt till deposit in Boreholes BH5 to BH8, BH10 to BH13, BH18, BH20 and BH21 and immediately below the clayey silt to silty clay layer in Boreholes BH5 and BH7.

The surface of the sandy silt to silty sand till across the site was encountered between approximately Elevation 173.8 m and Elevation 171.1 m with a thickness, where fully penetrated, varying between about 1.6 m and 7.3 m.

Standard Penetration Test (SPT) 'N' values measured within the sandy silt to silty sand till deposit were greater than 100 blows per 0.3 m of penetration, indicating a very dense relative density. Occasional rock fragments and boulders were encountered within the till at some borehole locations.

The results of three grain size distribution tests carried out on samples of the sandy silt to silty sand till are shown on Figure 9. Natural water contents measured on selected samples of this till range between 6 percent and 10 percent, with an average of about 8 percent.

4.2.7 Residual Soil

A thin deposit of reddish brown silty sand with some gravel and clay and with a “till-like” texture was encountered in Boreholes BH6 and BH8 to BH12 directly above the shale bedrock. This deposit is classified as residual soil (derived from weathering of the underlying shale bedrock) and contains varying amounts of siltstone/limestone and shale fragments. The surface of this deposit was encountered between Elevation 169.0 m and Elevation 166.1 m in the boreholes and the deposit was found to be between 0.3 m and 1.8 m thick.

Standard Penetration Test (SPT) ‘N’ values measured within the residual soil were greater than 100 blows per 0.3 m of penetration, indicating a very dense state of packing. The results of one grain size distribution analysis performed on a selected sample of this deposit is provided on Figure 10. Three natural water contents measured on selected samples of the residual soil vary between about 5 percent and 9 percent.

4.2.8 Bedrock

Shale bedrock of the Queenston Formation was encountered in Boreholes BH5 to BH12 and confirmed by split spoon sampling and/or rock coring. The surface of the bedrock was encountered at these boreholes between Elevation 170.2 m and Elevation 164.7 m, as summarized below.

Borehole	*Ground Surface Elevation (m)*	*Depth to Bedrock (m)*	*Elevation of Top of Bedrock (m)*
BH5	178.4	9.0	169.4
BH6	183.0	16.8	166.2
BH7	179.3	9.1	170.2
BH8	179.3	13.9	165.4
BH9	179.0	10.8	168.2
BH10	179.3	13.9	165.5
BH11	176.7	10.7	166.0
BH12	176.6	11.9	164.7

Based on the borehole data, the bedrock surface at the site generally slopes downward in a northerly and easterly direction: bedrock surface elevations on the west side of the proposed roadway crossing typically range between Elevation 166 m to the north and 170.2 m to the south; and bedrock surface elevations on the east side of the proposed roadway crossing range between Elevation 164.7 m to the north and 166.2 m to the south.

Bedrock was cored for a length of 3.0 m in each of Boreholes BH5 and BH12 located at the proposed south and north abutments, respectively. Bedrock samples obtained consist of red-brown, typically highly to moderately weathered, thinly layered, very fine grained, very weak to weak calcareous shale bedrock of the Queenston Formation. Occasional layers of weathered, grey, siltstone and medium strong to strong grey limestone were present within the calcareous shale bedrock. The Total Core Recovery was between 90 percent and 100 percent for all core runs greater than 0.15 m in length. The Rock Quality Designation (RQD) measured on the core samples from these two boreholes ranged from about 18 percent to 63 percent, with the lower values encountered below Elevation 163.9 m in Borehole BH12. This indicates a rock mass of very poor to fair quality. Fifteen SPT 'N' values obtained within the red shale bedrock all measured greater than 100 blows per 0.3 m of penetration.

4.2.9 Groundwater Conditions

The water levels were observed in the open boreholes during and after the drilling and coring operations and piezometers were installed in Boreholes BH5, BH12 and BH18. The piezometers installed in Boreholes BH5 and BH12 were sealed in the bedrock and immediately above the contact between the bedrock and overlying residual soil, while the piezometer installed in Borehole BH18 was sealed within the silty sand to sandy silt till deposit. Details of the piezometer installations are shown on the Record of Borehole Sheets following the text of this report. The water levels in the piezometers are summarized below:

Borehole No.	***Ground Surface Elevation (m)***	***Water Level Depth (m)***		***Water Level Elevation (m)***	
		February 21, 2007	***April 3, 2007***	***February 21, 2007***	***April 3, 2007***
BH5	178.4	1.1	0.3	177.3	178.1
BH12	176.6	0.1*	0.0	176.5*	176.6
BH18	178.6	1.8	1.4	176.8	177.2

* On February 21, 2007 the piezometer in BH12 was frozen

It should be noted that the groundwater levels are subject to seasonal fluctuations.

5.0 CLOSURE

This Foundation Investigation Report was prepared by Mr. Brian Lapos, E.I.T., and reviewed by Ms. Anne S, Poschmann, P.Eng., a Principal of Golder. Mr. Jorge Costa, P.Eng., a Designated MTO Contact for Golder, carried out an independent review of the report.

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PART B

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6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides foundation design recommendations for the proposed Ridgeway Drive Grade Separation structure over Highway 403 and Highway 407 ramps. 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 only to provide the designers with sufficient information to assess the feasible foundation alternatives and to design the proposed structure foundations. As such, where comments are made on construction they are provided only in order to highlight those aspects which could affect the design of the project. 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.

It is understood that the proposed bridge will connect Ridgeway Drive from Unity Drive (located south of Highway 403/407) to Angel Pass Drive (located north of Highway 403/407) and extend over the Highway 403/Highway 407 ramp corridor. The proposed structure is a three span bridge with span lengths of about 34 m, 20 m and 34 m from the south to the north, respectively. The proposed approach embankments and high fill embankments will be up to about 8 m to 9 m in height; varying relative to the existing ground surface. The revised General Arrangement drawings for the Ridgeway Drive structure was provided by Philips in electronic format on July 10, 2007 and has been incorporated into the attached Drawings 1 to 4.

6.1 General

Various alternatives for the abutment and pier foundations were considered and a summary comparison of these alternatives is presented in Table 1. Shallow foundations are not recommended for support of the bridge abutments or the piers at this site because the upper clayey silt till and earth fill under the existing Ridgeway Drive pavement and Highway 403/Highway 407 ramps is variable and not suitable for the support of the shallow foundation elements. Further, to ensure that a suitable founding strata is obtained for support of spread footings, excavations up to a depth of 5 m below ground surface would be required and these could impact adjacent highway traffic.

Caissons and steel H piles extending to bedrock are considered feasible for support of the bridge abutments and piers; however, due to the presence of very dense silty sand to sandy silt till and potential presence of cobbles and boulders it would be unlikely that the piles could be driven to bedrock through the till without first pre-augering the pile locations. Similar difficulties could be experienced in advancing the caisson through about 7 m of till. Consideration could be given to the use of caissons founded within the very dense silty sand to sandy silt till although there is potential for difficulties in augering if cobbles and boulders are encountered and for loosening of the founding soils due to groundwater inflow unless some form of groundwater control is implemented. It is therefore considered that steel H-piles driven to within the very dense silty sand to sandy silt till or very dense silty sand residual soil is the most feasible option for support of the abutments and piers from a geotechnical/foundation perspective.

It is assumed that any wing walls associated with this bridge structure will be supported by the pile cap foundation elements and therefore have not been addressed in this report.

6.2 Steel H-Pile Foundations

Steel H-piles driven to within the very dense silty sand to sandy silt till and/or very dense silty sand residual soil are considered to be the most practical option for support of the bridge piers and abutments, as noted above.

It is assumed that the abutment pile caps will be constructed at about the presently existing ground surface elevation, while the pier pile caps will be constructed about 2 m below the presently existing ground surface (i.e. Highway 403). For design, the following pile tip elevations have been assumed for piles founded within the very dense silty sand to sandy silt till or residual soil; it has been assumed that the piles would penetrate about 1 m into the very dense deposit. These tip elevations may be assumed for determining pile lengths; however, provision should be made in the contract to deal with greater pile lengths in the event that the piles penetrate deeper into the founding strata.

Foundation Location	*Relevant Boreholes*	*Overburden Design Pile Tip Elevation (m)*	
		West of Centreline	*East of Centreline*
South Abutment	5 and 6	170.0	171.5
South Pier	7 and 8	171.0	171.0
North Pier	9 and 10	169.0	172.0
North Abutment	11 and 12	171.5	171.5

There should be provision made in the contract for dealing with pile lengths varying from these design tip elevations.

It may also be feasible to found the piles on the shale bedrock; however, the very dense nature of the silty sand to sandy silt till deposit overlying the bedrock and the potential presence of cobbles and boulders within the till deposit will pose difficulties in advancing the piles to the required depth by driving. Therefore, it would be necessary to pre-auger through the till deposit. For the option of steel H piles driven to bedrock (with provision for pre-augering in the event that the piles “hang up” within the till), the following pile tip levels may be assumed for design.

Foundation Location	*Relevant Boreholes*	*Bedrock Design Pile Tip Elevation (m)*	
		West of Centreline	*East of Centreline*
South Abutment	5 and 6	168.0	166.0
South Pier	7 and 8	169.0	165.0
North Pier	9 and 10	167.0	165.0
North Abutment	11 and 12	165.0	164.5

There should be provision made in the contract for dealing with pile lengths varying from this design table.

6.2.1 Axial Geotechnical Resistance

For HP 310 x 110 piles driven to within the very dense silty sand to sandy silt till or residual soil deposits, a factored axial resistance at Ultimate Limit States (ULS) of 1400 kN may be assumed for design at the abutment locations and ULS of 1000 kN for design at the pier locations. These values represent the geotechnical limitation of the soil. The geotechnical resistance at Serviceability Limit States (SLS) for 25 mm of settlement is greater than the factored axial resistance at ULS, since the very dense silty sand to sandy silt till or residual soil are considered to be non-yielding; as such, ULS conditions will govern for steel H piles terminated within the silty sand to sandy silt till or residual soil. In this case, the pile capacity must be verified in the field by the use of the Hiley formula (Standard Structural Drawing SS-103-11) during the final stages of driving to achieve an ultimate capacity of 2800 kN. The following note should be shown on the Contract drawing assuming that a resistance factor of 0.5 (in accordance with MTO Foundations requirements) is applied to the use of the Hiley:

- "Piles to be driven in accordance with Standard SS-103-11 using an ultimate capacity of 2800 kN per pile but must be driven to the elevations given above."

For HP 310 x 110 piles driven to bedrock, a factored axial resistance at Ultimate Limit States (ULS) of 2,000 kN may be assumed for design. This value represents the structural limitation for the pile rather than the geotechnical limitation. The geotechnical resistance at Serviceability Limit States (SLS) for 25 mm of settlement will be greater than the factored axial resistance at ULS, since the bedrock is non-yielding; as such, ULS conditions will govern. It is highly likely that H piles will "hang up" within the very dense silty sand to sandy silt till deposit, and therefore pre-augering will likely be required to reach the bedrock surface. In this case, pre-augering should be to within about 1 m of the design tip elevation with the assumption that it would be possible to drive the piles at least 1 m beyond the pre-augered depth.

Pile installation should be in accordance with 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. The piles should be stiffened with "driving shoes" and MTO flange plates for protection during driving in accordance with OPSD 3300.100, OPSD 3300.200 and OPSS 903.07.05.04. The following notes should be included in the contract drawings for piles driven within the overburden:

- "At the South Abutment, piles to be driven below Elev. 170.5 m west of the centreline and below Elev. 172 m east of the centreline";
- "At the South Pier, piles to be driven below Elev. 171.5 m";
- "At the North Pier, piles to be driven below Elev. 169.0 m west of the centreline and below Elev. 173 m east of the centreline"; and
- "At the North Abutment, piles to be driven below Elev. 173.0 m".

The following note is considered appropriate for piles driven to bedrock:

- "Piles to be driven to bedrock; pre-augering to within 1 m of the design tip level is required".

6.2.2 Downdrag Load (Negative Skin Friction)

Loading from the approach embankments will cause consolidation settlement of the underlying clayey silt and clayey silt till deposits. The consolidation settlement is time-dependent and will not occur completely during the construction period; post-construction settlement of the clayey deposits will take place. As a consequence, negative skin friction or downdrag loads will need to be taken into account in the design of the piles supporting the south and north abutments. The structural design of the piles should be based on the full downdrag load acting on the piles using an unfactored downdrag load of 200 kN per pile assuming HP 310x110 piles.

The load calculated in this manner is an unfactored load. 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 Canadian Highway Design Code (2001) (*CHBDC)* for ULS conditions. Downdrag loads could be reduced by preloading of the abutment areas, the use of lightweight fill or surcharging the areas as discussed in Section 6.4.4.

6.2.3 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 (i.e. if an integral abutment structure is adopted), 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, k_h, is based on the equation given below, as described by Terzaghi (1955) and Davisson (1970) (expressed in Canadian Foundation Engineering Manual, 3rd Edition).

For cohesionless soils:

$$k_h = \frac{n_h z}{B}$$

where

k_h is the coefficient of horizontal subgrade reaction (kPa/m);
n_h is the constant of horizontal subgrade reaction, as given below (kPa/m);
z is the depth (m); and
B is the pile diameter/width (m).

For cohesive soils:

$$k_h = \frac{67 s_u}{B}$$

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. The range in values reflects the variability in the subsurface conditions as well as the two extremes of design: the requirement for flexibility in the case of integral abutments and the requirement for lateral support in the case of no-integral abutments.

Soil Unit	***Elevation***	***n_h***	***s_u***
South abutment; 10 m west of centreline (BH-5):			
Very stiff to hard clayey silt till	178.1 m – 174.0 m	–	150 kPa
Hard clayey silt to silty clay	174.0 m – 171.1 m	–	200 kPa
Very dense silty sand to sandy silt till	171.1 m – 169.4 m	11,000 kPa/m	–
South abutment; 10 m east of centreline (BH-6):			
Embankment fill	183.0 m – 177.8 m	–	–
Very stiff clayey silt till	177.8 m – 173.5 m	–	100 kPa
Very dense silty sand to sandy silt till	173.5 m – 169.6 m	11,000 kPa/m	–
Hard clayey silt to silty clay	169.6 m – 168.0 m	–	200 kPa
Residual Soil	168.0 m – 166.2 m	11,000 kPa/m	–
North abutment; 10 m west of centreline (BH-11):			
Very stiff to hard clayey silt till	176.4 m – 173.0 m	–	150 kPa
Very dense silty sand to sandy silt till	173.0 m – 167.8 m	11,000 kPa/m	–
Hard clayey silt to silty clay	167.8 m – 166.3 m	–	200 kPa
Residual Soil	166.3 m – 166.0 m	11,000 kPa/m	–
North abutment; 10 m east of centreline (BH-12):			
Hard clayey silt till	176.2 m – 173.5 m	–	150 kPa
Very dense silty sand to sandy silt till	173.5 m – 166.2 m	11,000 kPa/m	–
Residual Soil	166.2 m – 164.7 m	11,000 kPa/m	–

A maximum lateral resistance of 160 kN at ULS, and a maximum lateral resistance of 65 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:

Pile Spacing in Direction of Loading (d = Pile Diameter)	***Subgrade Reaction Reduction Factor***
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).

The subgrade reaction reduction factor should be interpolated for pile spacing between those provided in the above table.

6.2.4 Frost Protection

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

6.3 Caissons

Consideration could be given to the use of caissons socketted into the very dense silty sand to sandy silt till or residual soil for support of the bridge. It may also be feasible to found the caissons on shale bedrock; however, difficulties are anticipated with extending the caissons to/into the bedrock as a result of the very dense silty sand to sandy silt till and the highly variable condition of the bedrock. If consideration is being given to the use of caissons, the design base levels as provided in Section 6.2 may be used.

6.3.1 Axial Geotechnical Resistance

Caissons founded within the silty sand to sandy silt till or residual soil will derive their axial resistance in part from end-bearing and in part from shaft friction. The performance of caissons will depend to a large degree upon the final cleaning and verification of the condition of the subgrade soils at the base. Each caisson excavation must be carefully cleaned to remove all loose cuttings to ensure that the concrete is in intimate contact with the competent bearing stratum. The base of the caissons will need to be inspected immediately prior to placing concrete to ensure that the above procedures have been followed. A temporary liner will be required to support the sides of the caisson excavations during cleaning and inspection. The liner must be maintained tight to the sides of the soil to minimize seepage of water into the drilled excavation. In addition, the Ontario Occupational Health and Safety Act (2007) outlines appropriate safety procedures and requirements that must be implemented prior to entry of personnel into the caissons and which will make the base cleaning and inspection tasks more onerous. Therefore, cleaning and inspection of the caissons is expected to involve specialized construction methods which have not typically been associated with caisson installation projects in Ontario in the past.

The factored axial geotechnical resistance at ULS for caissons founded within the silty sand to sandy silt till or residual soil at the elevations given in Section 6.2 at the abutment and pier locations are given below. The SLS resistance required to achieve 25 mm of settlement is greater than the factored axial geotechnical resistance at ULS, except as noted below for the 1.5 m diameter caissons at the abutments.

Caisson Location	***Caisson Diameter (m)***	***Silty Sand to Sandy Silt or Residual Soil***	
		ULS	***SLS***
Abutments	0.9	2,250 kN	n/a
	1.5	6,200 kN	5,500 kN
Piers	0.9	1,800 kN	n/a
	1.5	5,000 kN	n/a

Caissons socketted approximately 1.0 m into the shale bedrock should be designed based on end-bearing resistance using a factored axial resistance at ULS of 5.5 MPa; for a 0.9 m and a 1.5 m diameter caisson, this equates to a factored axial resistance at ULS of 3,500 kN and 9,600 kN, respectively. Serviceability Limit States (SLS) resistances do not apply to caissons founded on the shale bedrock, since the SLS resistance for 25 mm of settlement is greater than the factored axial geotechnical resistance at ULS.

6.3.2 Downdrag Load (Negative Skin Friction)

Loading from the approach embankments will cause consolidation settlement of the underlying clayey silt and clayey silt till deposits. The consolidation settlement is time-dependent and will not completely occur during the construction period; post-construction settlement of the clayey deposits will take place. As a consequence, negative skin friction or downdrag loads will need to be taken into account during design of the caissons supporting the south and north abutments. The structural design of the caissons should be based on the full downdrag load acting on a 0.9 m and 1.5 m diameter caisson of 300 kN and 500 kN per caisson, respectively.

The load calculated in this manner is an unfactored load. 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* for ULS conditions. Downdrag loads could be reduced by preloading of the abutment areas, the use of lightweight fill or surcharging the areas as discussed in Section 6.4.4.

6.3.3 Resistance to Lateral Loads

The resistance to lateral loading for the caissons should be in accordance with Section 6.2.3.

6.3.4 Frost Protection

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

6.4 Approach and High Fill Embankment Design and Construction

The proposed Ridgeway Drive bridge structure grade varies from about Elevation 186.5 m to Elevation 187.0 m. The existing ground surface at the south approach varies from about Elevation 178.4 m (offset 10 m west of the bridge centreline) to Elevation 183.0 m (offset 10 m east of the bridge centreline). The difference in elevation is the result of previous earth fill placed at the site during the construction of a soil berm that parallels the south drainage ditch along Highway 403/Highway 407 ramps. The proposed south approach embankment will be about 8.1 m high assuming that the firm silty clay to clayey silt fill used to construct the soil berm will be replaced and/or stripped and re-compacted to provide competent embankment fill.

The existing ground surface at the north approach varies from about Elevation 176.6 m to 179.1 m. The boreholes at the abutment location were advanced at the base of the north drainage ditch that parallels Highway 403/Highway 407 ramps, while the borehole at the approach was advanced at an elevation more representative of the natural ground surface in the area (Elevation 179.1 m). The proposed north approach embankment will be between about 7.5 m and 10 m high at the approach location and at the bottom of the drainage ditch, respectively.

In addition to the approach embankments immediately adjacent to the bridge structure, high fill embankments will also be required along the proposed Ridgeway Drive alignment. Based on profiles and cross-sections of the proposed Ridgeway Drive alignment provided by Phillips, the road construction will require fill embankments up to about 8 m in height. All embankments greater than 4.5 m in height are addressed in this report as high fill embankments.

Where earth fill embankments are greater than 8 m in height, a 2 m wide berm constructed at mid-height is required in accordance with MTO guidelines. The presence of a mid-height berm will increase the internal and surficial stability of the embankment and aid in surface water control on the slope.

6.4.1 Subgrade Preparation and Embankment Construction

All topsoil, organic matter and softened / loosened soils should be stripped from below the approach embankments and high fill areas; topsoil thickness ranges between 200 mm and 400 mm at the location of the boreholes. In addition, previously placed fill at the site will require stripping. It is recommended that an Operational Constraint (OC) be included in the Contract Documents in-order to limit disturbance to the clayey subgrade soils that will be exposed within the embankment footprints at the site. A sample OC is provided in Appendix A. The approximate existing fill thicknesses and the approximate embankment height that will be required following the removal of topsoil and existing fill at the site is summarized below.

Station	Alignment	Remarks/Location	Approximate Thickness of Existing Fill (m)	Proposed Embankment Height (m)
10+020 to 10+200	Centreline	Fill thickness increases with increased chainage	0.5 - 2.5	0.0 – 6.0
10+200 to 10+230	Centreline	No existing fill based on visual observations	0	7.0
10+230 to 10+260	o/s left of centreline	South approach embankment	3	8.0
10+230 to 10+260	Centreline	South approach embankment	3	8.0
10+230 to 10+260	o/s right of centreline	South approach embankment	5	8.0
10+260 to 10+270	o/s left of centreline	South abutment	0	8.1
10+260 to 10+270	Centreline	South abutment	5	8.1
10+260 to 10+270	o/s right of centreline	South abutment	5	8.1
10+350 to 10+360	Centreline	North abutment	0	8.5
10+360 to 10+390	Centreline	North approach embankment	0	7.5
10+390 to 10+470	Centreline	No existing fill based on visual observations	0	7.0
10+470 to 10+550	Centreline	Extent of existing fill unknown near Angel Pass Drive	2.5	0.0 – 6.0

The existing silty clay to clayey silt fill after being stripped from the site could be reused to construct the high fill embankments provided that the material is placed with proper compaction. Once the topsoil and fill is stripped at the site, all subgrade soils should be proof-rolled prior to fill placement in accordance with OPSS 206. Embankment (earth) fill should be placed in regular lifts with loose thickness not exceeding 300 mm, and be compacted to at least 95 per cent of the material's Standard Proctor maximum dry density. The final lift prior to placement of the granular subbase course should be compacted to 100 per cent of the Standard Proctor maximum dry density. Inspection and field density testing should be carried out by qualified personnel during placement operations to ensure that appropriate materials are used and that adequate levels of compaction have been achieved.

The approach and high fill embankments are greater than 8 m in height in some areas (i.e. from about Sta. 10+260 to Sta. 10+360); therefore mid-height benches must be provided to ensure that there is no "unbroken" slope height greater than 8 m. The mid height benches should be at least 2 m in width. To protect the embankment side slopes and reduce surface water erosion, placement of topsoil and seeding or pegged sod is recommended immediately after completion of embankment construction.

6.4.2 Approach and High Fill Embankment Stability

Stability analyses using the limit equilibrium method were performed on the critical sections of the proposed approach and high fill embankments. For this report, critical sections are assumed to correspond to the greatest new embankment height within a stretch of embankment. In all cases, side slopes of 2 horizontal to 1 vertical (2H:1V) were assumed.

All slope stability analyses were performed using the commercially available program SLOPE/W (Version 6.20), produced by Geo-Slope International Ltd., employing the Morgenstern-Price method of analysis. For all analyses, the factor of safety of numerous potential failure surfaces were computed in order to establish the minimum factor of safety. 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 for the design of embankment slopes under static conditions. This factor of safety is considered adequate for the embankments at these sites considering the design requirements and the field data available. The stability analyses were performed to check that the target minimum factor of safety was achieved for the various embankment heights and geometries.

The subsoils encountered in the areas of the proposed embankments are composed of a combination of cohesive and cohesionless soils. For cohesionless layers, effective stress parameters were employed in the analysis assuming drained conditions for the soils. The effective stress parameters for the cohesionless soils were estimated from empirical correlations using the results of in situ Standard Penetration Tests (SPT). For cohesive layers, total stress parameters were employed in the analysis. The total stress parameters (i.e. the mobilized undrained shear strength, s_u) for the cohesive soils were estimated from correlations with the SPT results and other laboratory test data.

Static slope stability analyses that examine the global stability of the approach and high fill embankments were carried out using the following parameters based on field and laboratory test data and accepted correlations:

Soil Deposit	**γ**	**ϕ'**	**S_u**
Embankment Fill (assumed earth fill)	21 kN/m^3	32°	–
Very Stiff Brown Clayey Silt Till	19.5 kN/m^3	30° *	50 - 80 kPa
Hard Grey Clayey Silt to Silty Clay	20 kN/m^3	32° *	120 kPa
Very Dense Silty Sand to Sandy Silt Till	21 kN/m^3	34°	–

Where: γ bulk unit weight (kN/m^3)
ϕ' effective friction angle (degrees)
S_u undrained shear strength (kPa)
* effective friction angle (degrees) used for seismic stability analysis

The analyses assume that all topsoil, organic matter and softened / loosened soils are stripped from below the embankment footprint prior to construction of the new embankments. The piezometric conditions used in the analyses were based on the groundwater levels noted during drilling and measured in the standpipe piezometer installations. In general, the groundwater level is located between 1.0 m and 2.0 m below the existing ground surface. For the stability analyses the groundwater level was assumed to be at the interface of the fill and clayey silt till (about 0.5 m below the ground surface).

The clayey silt till and underlying silty sand to sandy silt till and clayey silt to silty clay subsoils encountered at the site are suitable founding strata for the support of the approach and high fill embankments. A generic model assuming the subsurface soil conditions noted above for the lower bound S_u value of 50 kPa of the clayey silt till underlying an 8.5 m high embankment with 2H:1V side slopes and no mid height berm was analysed. This worst case (conservative) scenario indicates a slope stability Factor of Safety (F.S) of 1.6, which is greater than the minimum criteria of 1.3. The results of the stability analysis are provide on Figure 11, which shows the potential slip surface and the corresponding factor of safety for the embankment fill being placed on the properly prepared clayey silt till subgrade.

6.4.3 Approach and High Fill Embankment Settlement

Settlement of the approach and high fill embankments will occur as a result of compression of the new embankment fill itself, as well as consolidation of the clayey soils underlying the new embankments. The settlement analyses assume that all topsoil and organics have been removed prior to construction of the new embankments and that the new embankments will be constructed with 2H:1V earth fill side slopes.

Provided that the embankment material consists of select subgrade material or clean earth fill, the settlement of up to 8.5 m of fill itself is expected to be up to about 10 mm. The use of granular fill for the new embankment construction will reduce this magnitude of settlement since the majority of settlement of granular fills will occur during construction, whereas the majority of the settlement of cohesive fill, if used, would occur shortly after construction.

In order to estimate the magnitude of settlement, analyses were carried out using the commercially-available program Unisettle (version 3.21) distributed by Unisoft Ltd. It is assumed that all embankment fills at the site will be founded on the very stiff to hard clayey silt till deposit where some time dependant consolidation settlement is expected. The magnitude of settlement will be variable and will depend on the thickness of the compressible stratum as well as the embankment height. The settlement of the founding soils has been estimated using consolidation parameters and elastic deformation moduli based on correlations with the Atterberg limit testing and SPT "N" values as given below.

Soil Type	**γ'**	**E**	**P_c'**	**e_o**	**C_c**	**C_r**
Embankment fill (assumed for earth fill)	21 kN/m^3	–	–	–	–	–
Firm to very stiff clayey silt till	9.5 kN/m^3*	–	450 kPa	0.6	0.1	0.01
Hard clayey silt to silty clay	10 kN/m^3*	50 MPa	–	–	–	–
Very dense silty sand to sandy silt till	11 kN/m^3*	100 MPa	–	–	–	–

Where:
γ unit weight (*effective unit weight used below the water table)
E elastic modulus (estimated base on SPT 'N' values and correlations proposed by Bowles (1984) and Kulhawy and Mayne (1990))
P_c' preconsolidation pressure ($s_u = 0.22\sigma_p$' (Mesri, 1975))
e_o initial void ratio
C_c compression index ($C_c = 0.009(w_L\text{-}10)$ (Skempton, 1944) and Cc = PI/74 (Kulhawy and Mayne, 1990))
C_r recompression index (C_r=C_c/10 (Kulhawy and Mayne, 1990))

Based on the groundwater levels measured in the standpipe installations, the groundwater levels at the site are between 1 m and 2 m below the existing ground surface. For design, the groundwater level is assumed to be at the interface of the fill and clayey silt till (about 0.5 m below the ground surface).

The results of the embankment settlement analysis at the site is summarized below.

Station	**Alignment**	**Location**	**Embankment Height (m)**	**Settlement (mm)**
10+020 to 10+230	Centreline	High fills south of Highway 403/407	0.0 – 8.0	25
10+230 to 10+270	o/s left of centreline	South approach embankment	8.1	25
10+230 to 10+270	Centreline	South approach embankment	8.1	10
10+230 to 10+270	o/s right of centreline	South approach embankment	8.1	5
10+350 to 10+390	Centreline	North approach embankment	8.5	25
10+390 to 10+550	Centreline	High fills north of Highway 403/407	0.0 – 7.5	25

Based on Terzaghi (1925) one-dimensional consolidation theory as defined in Holtz and Kovacs (1981), and using a coefficient of consolidation (c_v) equal to 0.01 cm^2/s (c_v was established using the results of the correlation with liquid limit (NAVFAX, 1971). It is estimated that 90 percent of the primary consolidation settlement of the clayey subsoils will be completed within approximately five months following completion of embankment construction.

6.4.4 Mitigation of Time Dependent Settlement

Time dependent, post-construction settlements of the new embankments are expected as a result of consolidation of the underlying stiff to hard clayey silt till and clayey silt to silty clay. In these areas, consideration could be given to preloading, surcharging, use of lightweight fill and/or EPS to limit the post-construction settlements and subsequent maintenance on the new roadway pavement structure. Each of these methods will reduce the settlement associated with the construction of the abutment and high fill embankments; however, given the magnitude of settlement anticipated, mitigation may not be essential but if it is, preloading is considered to be the most cost effective method for settlement mitigation at this site.

6.4.4.1 Preloading

If the embankment areas are pre-loaded for a minimum of five months, it is estimated that 90 percent of consolidation settlement will occur. As discussed in Section 6.4.2, stability at this site is not an issue that would prevent the maximum embankment height of 8.5 m with sides slopes of 2H:1V to be constructed at the south and north approach locations. It is also understood that there is sufficient room to construct all embankments at the site with 2H:1V side slopes during the pre-load stage.

Following the preload period, it is estimated there will be less than 5 mm of post-construction settlement. Based on the total post-construction settlement and the potential elastic compression of the long piles proposed at this site, downdrag loads (as given in Sections 6.2.2 and 6.3.2) do not have to be considered in the structural design of the piles, if preloading of the approach and high fill embankment areas is carried out prior to pile installation.

6.5 Liquefaction Potential and Seismic Analysis

6.5.1 Analysis Methods

The liquefaction potential of granular soils under seismic loading is assessed using the empirical method outlined in Section C.4.6.2 of the *CHBDC Commentary* (2001) based on papers by Seed and Idriss (1971) and Seed et al. (1984), which compares the cyclic resistance ratio (CRR) of the soils to the cyclic stress ratio (CSR) caused by an earthquake. The CRR is determined based on correlations with the normalized penetration resistance and fines content of soil together with the characteristic earthquake magnitude for liquefaction assessment (that is indirectly related to the number of significant stress cycles or duration of strong shaking). The CRR is corrected for earthquake magnitude and overburden stress effects. The CSR at a given depth is related to the peak ground acceleration, the ratio of the total to effective overburden stress at that depth, and soil flexibility. A factor of 0.65 is used to convert the maximum CSR to an equivalent CSR of uniform cycles (Section C4.6.2 of the CHBDC *Commentary*).

In general, geologically young, loose, saturated deposits of sand, silty sands, and non-plastic silt are susceptible to liquefaction.

6.5.1.1 Liquefaction-Induced Settlements and Lateral Movements

Where liquefaction is identified to be a problem using the methods described above, vertical settlements of the soil under the earthquake loading may occur due to the contraction of the sand deposit. The anticipated post-earthquake settlements are estimated using a relationship developed by Tokimatsu and Seed (1987) where the anticipated post-earthquake volume change is related to the SPT ‘N’ values and CSR.

The lateral movements can be estimated using relationships proposed by Makdisi and Seed (1978). If unacceptable lateral movements are anticipated, soil improvement methods should be considered and could include densification, removal and re-compaction, grouting, or permanent drainage so that the pore water pressure rise necessary to trigger liquefaction is controlled.

6.5.1.2 Embankment Stability under Seismic Conditions

If liquefaction of the subsoils under an embankment loading is not anticipated, the stability of the embankment slope may be assessed using conventional pseudo-static methods of slope stability analysis under earthquake-induced peak ground acceleration. A calculated factor of safety of 1.0 is considered appropriate; however, a factor of safety less than 1.0 does not indicate full-scale failure of the embankment slope due to the application of the peak ground acceleration in one direction for a short period of time. In this case, other methods, such as the Newmark sliding block method may be used to assess the magnitude of the ground movement.

Where liquefaction is triggered in the underlying soil deposit, the stability of the embankment is analyzed using post-liquefaction, residual shear strength parameters in the liquefied layers using the correlation proposed by Seed and Harder (1990) which is correlated to SPT ‘N’ values. If under these conditions, the embankment is estimated to have a factor of safety less than 1.0 under static conditions (i.e. without inertia effects), the embankment is considered to be susceptible to a flow slide. Flow slides are characterized by very large lateral and vertical displacements of the embankment. If under residual strength conditions, the static factor of safety is greater than 1.0, lateral displacements may still occur, and these are estimated using the Newmark method, which relates the horizontal acceleration necessary to induce a factor of safety equal to 1.0 in the embankment (i.e. yield acceleration) to the anticipated displacements. If the yield acceleration is greater than the maximum acceleration for the site, then no remedial measures are required. If the yield acceleration is less than the maximum acceleration and the computed movements are unacceptable, soil improvement methods may be necessary to improve soil conditions.

6.5.2 Results of Analyses

The liquefaction susceptibility of the soil deposits underlying the proposed roadway embankments and the consequent stability of the embankment under seismic loading conditions for the Ridgeway Drive site has been assessed. The peak zonal acceleration used for the Ridgeway Drive site (Mississauga) is 0.065 g, which is based on a zonal acceleration of 0.05 g multiplied by an amplification factor of 1.3 for the types of soils found at the site. This amplification factor was estimated in accordance with Section 4.1.8.4 and Table 4.1.8.4 B of the NBC (2006) for Class D soils and 5 % percent damped spectral acceleration Sa (0.2) less than or equal to 0.25. Typically, for free-draining soils, the seismic loading is applied to the long-term (drained) conditions.

Using the methods outlined in Section 6.5.1, the soils at this site have a very low risk of liquefaction. This assessment corresponds to a characteristic earthquake of magnitude 7 representing approximately 10 to 15 effective cycles of loading and has been established based on historical earthquake data and de-aggregation of seismic risk carried out for other projects in the general region, and taking into consideration that smaller magnitude events (i.e. $\leq$ M5) do not contribute to liquefaction damage.

A factor of safety greater than 1.0 against embankment instability under seismic conditions is obtained. The results of the embankment slope stability under an earthquake-induced peak ground acceleration equal to 0.065 g using the commercially available program SLOPE/W (Version 6.20), produced by Geo-Slope International Ltd., employing the Morgenstern-Price method of analysis are shown on Figure 12.

6.6 Lateral Earth Pressures for Design

The lateral earth pressures acting on the abutment stems and associated wing wallsl/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. Seismic (earthquake) loading must also be taken into account in the design as subsequently discussed in Section 6.6.1.

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) 1010 Granular 'A' or Granular 'B' Type II should be used as backfill behind the walls. The Granular 'A' or Granular 'B' Type II fill should be compacted in loose lifts not greater than 200 mm in thickness to 95 per cent of the material's Standard Proctor maximum dry density in accordance with OPSS 501. Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the granular backfill requirements with respect to sub-drains and frost taper should be in accordance with OPSD 3101.150 and 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, 2001,* Section 6.9.3 and Figure 6.9.3. 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 width equal to at least 1.2 m behind the back of the wall stem (Case I in Figure C6.9.1(l) of the *Commentary to the CHBDC, 2001*) 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(l) of the *Commentary to the CHBDC, 2001).*
- For Case I, the pressures are based on the proposed embankment fill materials and the following parameters (unfactored) may be used:

	EARTH FILL
Soil unit weight:	21 kN/m^3
Coefficients of static lateral earth pressure:	
Active, K_a	0.31
At rest, K_o	0.47

- 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. The movement to allow active pressures to develop within the backfill, and thereby assume an unrestrained structure, may be taken as follows in accordance with Section C6.9.1.(a) of the CHBDC *Commentary*:

- rotation (i.e. ratio of wall movement to wall height) of approximately 0.002 about the base of a vertical wall;
- horizontal translation of 0.001 times the height of the wall; or
- a combination of both.

A restrained structure is typically culverts or rigid frame bridge where the rotational or horizontal movement is not sufficient to mobilize an active earth pressure condition. For this condition, an at-rest pressure plus any compaction surcharge should be included in the design of the structure.

6.6.1 Seismic Considerations

Seismic (earthquake) loading must be considered in the design in accordance with Section 4.6 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, which is to be added to the static earth 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:

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

Where
- K = either the static active earth pressure coefficient (K_a) or the static at rest earth pressure coefficient (K_o);
- K_{AE} = the seismic active earth pressure coefficient determined in accordance with Sections 4.6.4 and C.4.6.4 of the CHBDC and its *Commentary*;
- γ' = the effective unit weight of the soil (kN/m^3)
  - taken as the soil unit weight given above for the fill materials;
  - taken as 21 kN/m^3 for the till deposit and 20 KN/ m^3 for the existing fill, where encountered;
- d = the depth below the top of the wall (m); and
- H = the height of the wall above the toe (m).

Using the amplified zonal acceleration ratio of 0.065g obtained for this site (refer to Section 6.5), the seismic lateral earth pressure coefficients (K_{AE}) for both yielding and non-yielding walls considering earth or granular fills were determined in accordance with Sections 4.6.4 and C 4.6.4 of the CHBDC and its *Commentary* and are presented in the table below. It should be noted that these seismic earth pressure coefficients assume that the back of the wall is vertical and the ground surface behind the wall is flat:

SEISMIC ACTIVE PRESSURE COEFFICIENTS, K_{AE}

	Case I	Case II	
	Earth Fill	Granular A	Granular B Type II
Yielding wall [1]	0.30	0.26	0.26
Non-yielding wall	0.34	0.30	0.30

[1] The above K_{AE} values for yielding walls are applicable provided that the wall can move up to 250A (mm), where A is the design zonal acceleration ratio of 0.065.

The CHBDC K_{AE} values include the effect of wall friction ($\delta = \phi'/2$) and are less than the static values of K_a and K_o reported above for the very low zonal acceleration ratio for this site. Therefore the contribution of the dynamic component in the active lateral earth pressures acting on the abutment stem or retaining walls at this site is not significant and the static lateral earth pressures are adequate for the design.

6.7 Design and Construction Considerations

6.7.1 Open-Cut Excavations

It is assumed that at the pier locations the base of the pile cap may extend to approximately 2 m below the existing Highway 403/Highway 407 ramps grade. In addition of the existing earth fill previously placed in various thickness across the site excavation will be required for subgrade preparation and for proper embankment construction. As outlined in Section 6.4.1, the fill is thickest at the south approach embankment location where approximately 5.0 m of excavation will be required. Where subexcavation is required, excavations must be carried out in accordance with the guidelines outlined in the Occupational Health and Safety Act (OHSA) for Construction Activities. The existing fill, and the upper clayey silt till which for the most part lies below the groundwater level, are classified as Type 3 soil, according to the OHSA. Temporary excavations (i.e. those which are open for a relatively short time period) should be made with side slopes not steeper than 1 horizontal to 1 vertical (1H:1V).

6.7.2 Temporary Roadway Protection

Depending on construction staging, temporary roadway protection may be required along Highway 403/Highway 407 ramps to facilitate construction of the piers and abutments. The temporary excavation support system should be designed and constructed in accordance with MTO's Special Provision SP105S19. The lateral movement of the temporary shoring system should meet Performance Level 2 as specified in SP105S19, provided that any buried utilities that may be present adjacent to the excavation(s) can tolerate this magnitude of deformation.

6.7.3 Groundwater Control

The groundwater level at this site is relatively high, typically between 0.3 m and 1.8 m below ground surface and may be at the invert level of the ditches paralleling the existing Ridgeway Drive right-of-way. Minor seepage into excavations made through the clayey silt till deposit should be expected. Existing earth fill should be expected to be water-bearing, with water “perched” on top of the low permeability clayey silt till, particularly during wet periods of the year. It is anticipated that the groundwater seepage into the foundation excavations can be adequately controlled by pumping from properly filtered sumps.

6.7.4 Obstructions During Pile Driving and Protection System Installation

It is noted that occasional boulders were encountered within the fill material in the boreholes located to the south of Highway 403/Highway 407 ramps. Occasional boulders and rock fragments were also noted within the glacially derived silty sand to sandy silt till in some of the boreholes. It is recommended that a Non-Standard Special Provision (NSSP) be included in the Contract Documents to warn the Contractor of the presence of cobbles and boulders within the overburden soils, which are glacially derived, as such obstructions may affect the installation of steel H-piles and/or caissons. A sample NSSP is provided in Appendix A.

7.0 CLOSURE

This Design Report was prepared by Mr. Brian Lapos, E.I.T., and reviewed by Ms. Anne Poschmann, P.Eng., a Principal of Golder. Mr. Jorge Costa, P.Eng., a Designated MTO Contact for Golder, carried out an independent review of the report.

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U.S. Navy, 1982. Soil Mechanics, Foundations and Earth Structures. NAVFAC Design Manual DM-7, Washington, D.C.

IMPORTANT INFORMATION AND LIMITATIONS OF THIS REPORT

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Unless otherwise stated, the suggestions, recommendations and opinions given in this report are intended only for the guidance of the Client in the design of the specific project. The extent and detail of investigations, including the number of test holes, necessary to determine all of the relevant conditions which may affect construction costs would normally be greater than has been carried out for design purposes. Contractors bidding on, or undertaking the work, should rely on their own investigations, as well as their own interpretations of the factual data presented in the report, as to how subsurface conditions may affect their work, including but not limited to proposed construction techniques, schedule, safety and equipment capabilities.

Soil, Rock and Groundwater Conditions: Classification and identification of soils, rocks, and geologic units have been based on commonly accepted methods employed in the practice of geotechnical engineering and related disciplines. Classification and identification of the type and condition of these materials or units involves judgment, and boundaries between different soil, rock or geologic types or units may be transitional rather than abrupt. Accordingly, Golder does not warrant or guarantee the exactness of the descriptions.

IMPORTANT INFORMATION AND LIMITATIONS OF THIS REPORT (cont'd)

Special risks occur whenever engineering or related disciplines are applied to identify subsurface conditions and even a comprehensive investigation, sampling and testing program may fail to detect all or certain subsurface conditions. The environmental, geologic, geotechnical, geochemical and hydrogeologic conditions that Golder interprets to exist between and beyond sampling points may differ from those that actually exist. In addition to soil variability, fill of variable physical and chemical composition can be present over portions of the site or on adjacent properties. **The professional services retained for this project include only the geotechnical aspects of the subsurface conditions at the site, unless otherwise specifically stated and identified in the report.** The presence or implication(s) of possible surface and/or subsurface contamination resulting from previous activities or uses of the site and/or resulting from the introduction onto the site of materials from off-site sources are outside the terms of reference for this project and have not been investigated or addressed.

Soil and groundwater conditions shown in the factual data and described in the report are the observed conditions at the time of their determination or measurement. Unless otherwise noted, those conditions form the basis of the recommendations in the report. Groundwater conditions may vary between and beyond reported locations and can be affected by annual, seasonal and meteorological conditions. The condition of the soil, rock and groundwater may be significantly altered by construction activities (traffic, excavation, groundwater level lowering, pile driving, blasting, etc.) on the site or on adjacent sites. Excavation may expose the soils to changes due to wetting, drying or frost. Unless otherwise indicated the soil must be protected from these changes during construction.

Sample Disposal: Golder will dispose of all uncontaminated soil and/or rock samples 90 days following issue of this report or, upon written request of the Client, will store uncontaminated samples and materials at the Client's expense. In the event that actual contaminated soils, fills or groundwater are encountered or are inferred to be present, all contaminated samples shall remain the property and responsibility of the Client for proper disposal.

Follow-Up and Construction Services: All details of the design were not known at the time of submission of Golder's report. Golder should be retained to review the final design, project plans and documents prior to construction, to confirm that they are consistent with the intent of Golder's report.

During construction, Golder should be retained to perform sufficient and timely observations of encountered conditions to confirm and document that the subsurface conditions do not materially differ from those interpreted conditions considered in the preparation of Golder's report and to confirm and document that construction activities do not adversely affect the suggestions, recommendations and opinions contained in Golder's report. Adequate field review, observation and testing during construction are necessary for Golder to be able to provide letters of assurance, in accordance with the requirements of many regulatory authorities. In cases where this recommendation is not followed, Golder's responsibility is limited to interpreting accurately the information encountered at the borehole locations, at the time of their initial determination or measurement during the preparation of the Report.

Changed Conditions and Drainage: Where conditions encountered at the site differ significantly from those anticipated in this report, either due to natural variability of subsurface conditions or construction activities, it is a condition of this report that Golder be notified of any changes and be provided with an opportunity to review or revise the recommendations within this report. Recognition of changed soil and rock conditions requires experience and it is recommended that Golder be employed to visit the site with sufficient frequency to detect if conditions have changed significantly.

Drainage of subsurface water is commonly required either for temporary or permanent installations for the project. Improper design or construction of drainage or dewatering can have serious consequences. Golder takes no responsibility for the effects of drainage unless specifically involved in the detailed design and construction monitoring of the system.

TABLE 1
EVALUATION OF FOUNDATION ALTERNATIVES
RIDGEWAY DRIVE/HIGHWAY 403 GRADE SEPARATION STRUCTURE

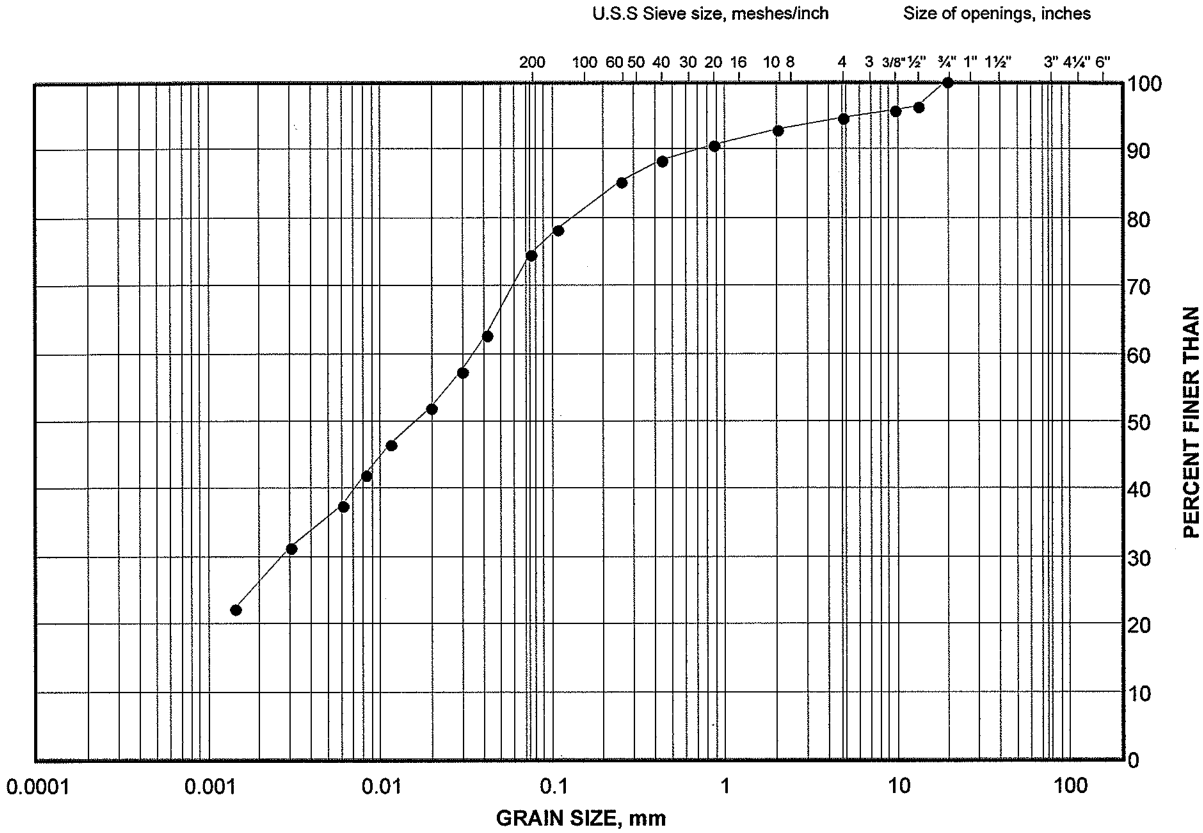
Footing Option		*Advantages*	*Disadvantages*	*Relative Costs*	*Risks/Consequences*
Spread footings on stiff to very stiff clayey silt till.	NF	N/A	Low geotechnical resistance near surface; therefore will require excavations to extend down to 5 m to the very dense silty sand to sandy silt till Groundwater control may be required.	May be very expensive since deep excavations and large earthworks are required to reach a competent founding stratum.	Large work area required and therefore more disruption to highway traffic.
Steel H Piles driven to within the overburden.		Minimized hard driving through very dense silty sand to sandy silt till deposit.	Lower capacity than piles driven to bedrock.	Approximately $40,000 assuming five piles at the abutment locations and nine piles at the pier locations.	
Steel H Piles driven to shale bedrock.		Increased capacity over piles terminated in overburden.	- Difficulties anticipated driving through very dense silty sand to sandy silt/ till deposit; pre-augering likely required to reach bedrock. - Potential increased costs for pre-drilling if piles "hang-up".	Approximately $62,000 assuming five piles at the abutment locations and nine piles at the pier locations.	High likelihood that piles could "hang-up" in the very dense silty sand to sandy silt till deposit. Pile locations would require pre-augering to permit pile installation.
Caissons founded within very dense silty sand to sandy silt till and/or hard clayey silt to silty clay		Minimized difficult augering through very dense silty sand to sandy silt till deposit.	- Lower capacity than caissons socketted into bedrock. Temporary liners required for groundwater control and to provide support. - Require combination of caissons for the piers and piles for abutments	Approximately $170,000 assuming five piles at the abutment locations and eight caissons at the pier locations.	Groundwater inflow through the base of excavation may impact the integrity of the founding soils. Unable to inspect base of caisson to confirm suitability of founding strata due to Health and Safety considerations.
Caissons socketted into shale bedrock		Higher bearing capacity than Steel H piles driven to bedrock.	- Difficulty may be encountered augering through very dense silty sand to sandy silt till deposit. Temporary liners required for groundwater control. - Require combination of caissons for the piers and piles for abutments - Increased cost of augering through very dense silty sand to sandy silt till deposit and socketting into bedrock.	Approximately $240,000 assuming five piles at the abutment locations and eight caissons at the pier locations.	Difficulty in advancing caisson excavation through 7 m of very dense till.

NF: Indicates that the founding option is considered not feasible.

GRAIN SIZE DISTRIBUTION

Clayey Silt some Sand (Probable Fill)

FIGURE 1



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	7	4	176.7

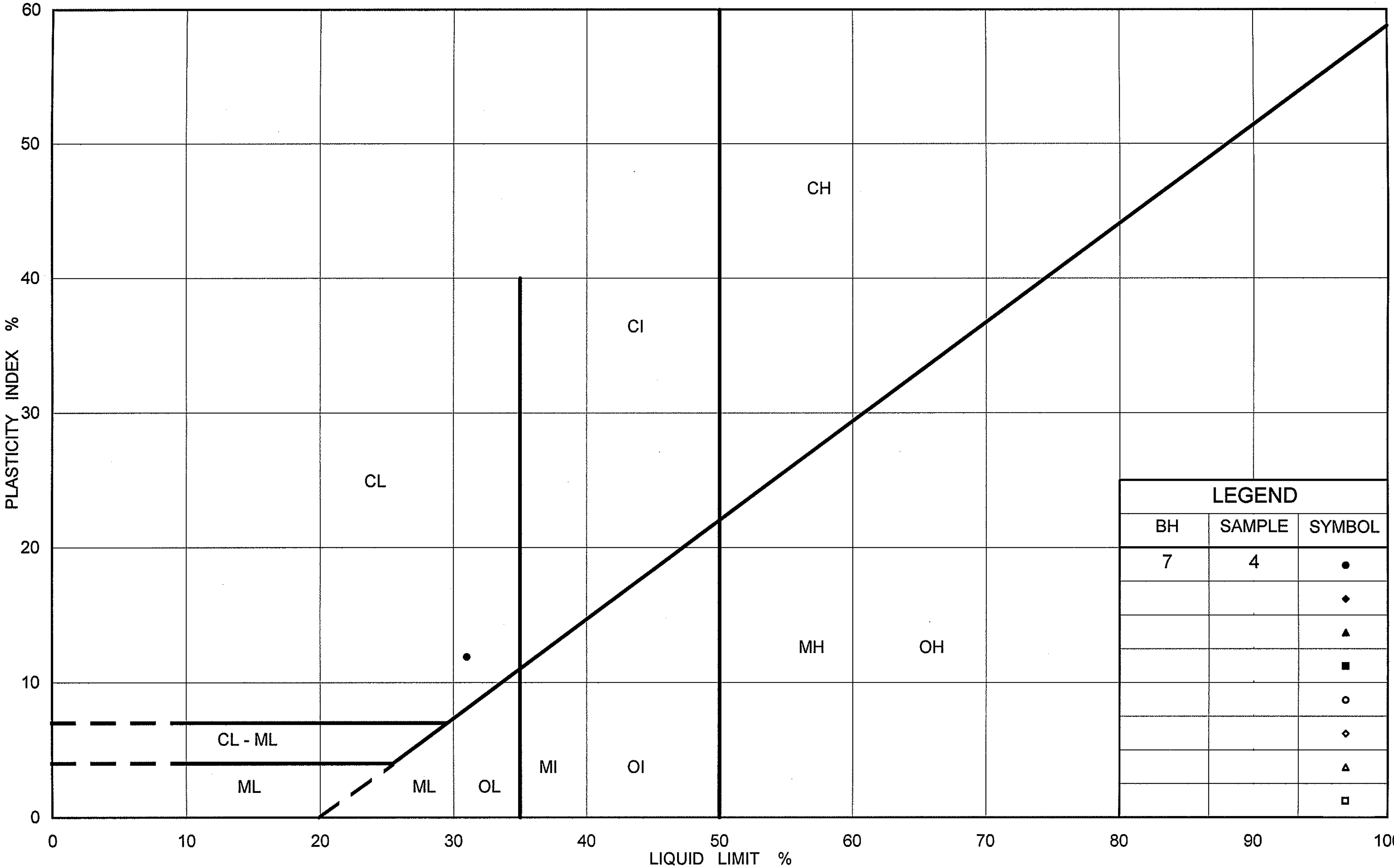
Project Number: 06-1111-021

Checked By: ____________________

Golder Associates

Date: 11-Apr-07

Oct 75, FF-S-21



Ministry of Transportation

Ontario

PLASTICITY CHART
Clayey Silt (Probable Fill)

Figure No. 2

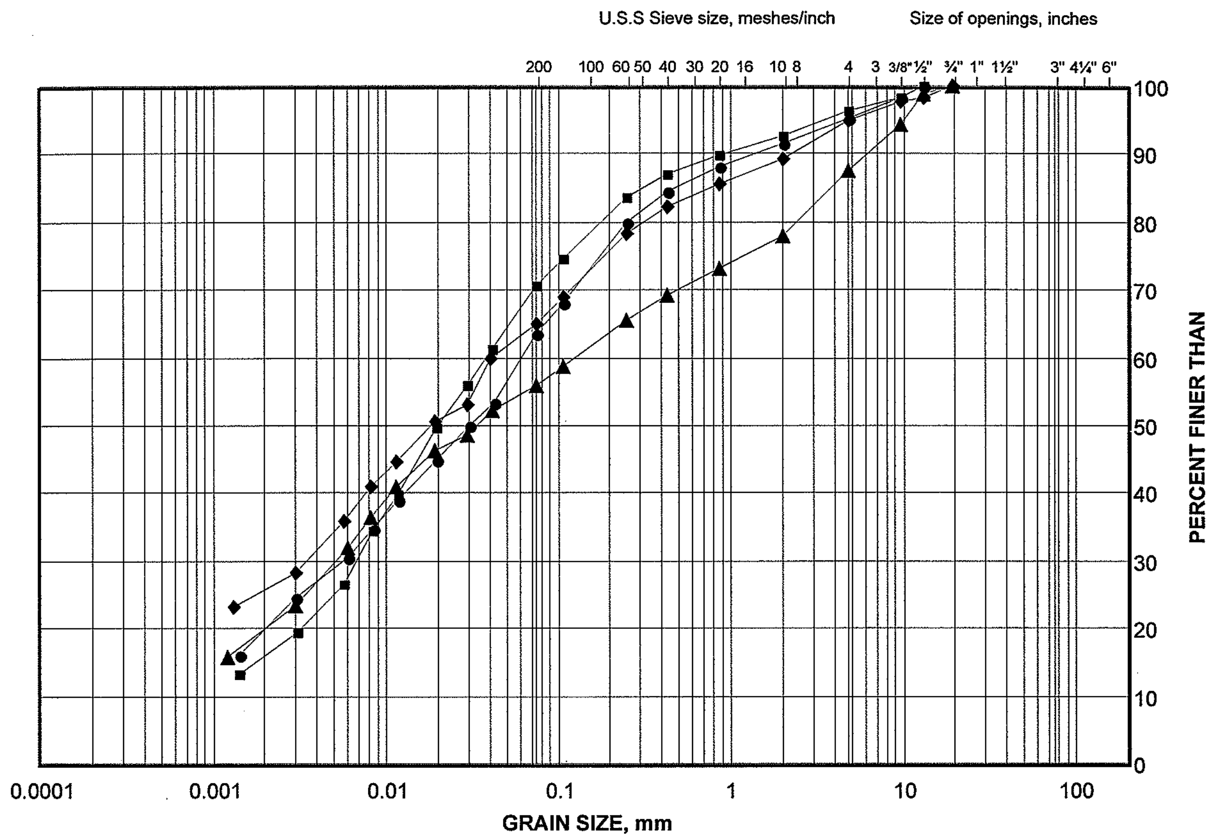
Project No. 06-1111-021

Checked By:

GRAIN SIZE DISTRIBUTION

Clayey Silt with Sand (Till)

FIGURE 3



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	10	5	174.4
■	9	6	172.8
◆	2	6	176.1
▲	6	7	176.6

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Date: 11-Apr-07

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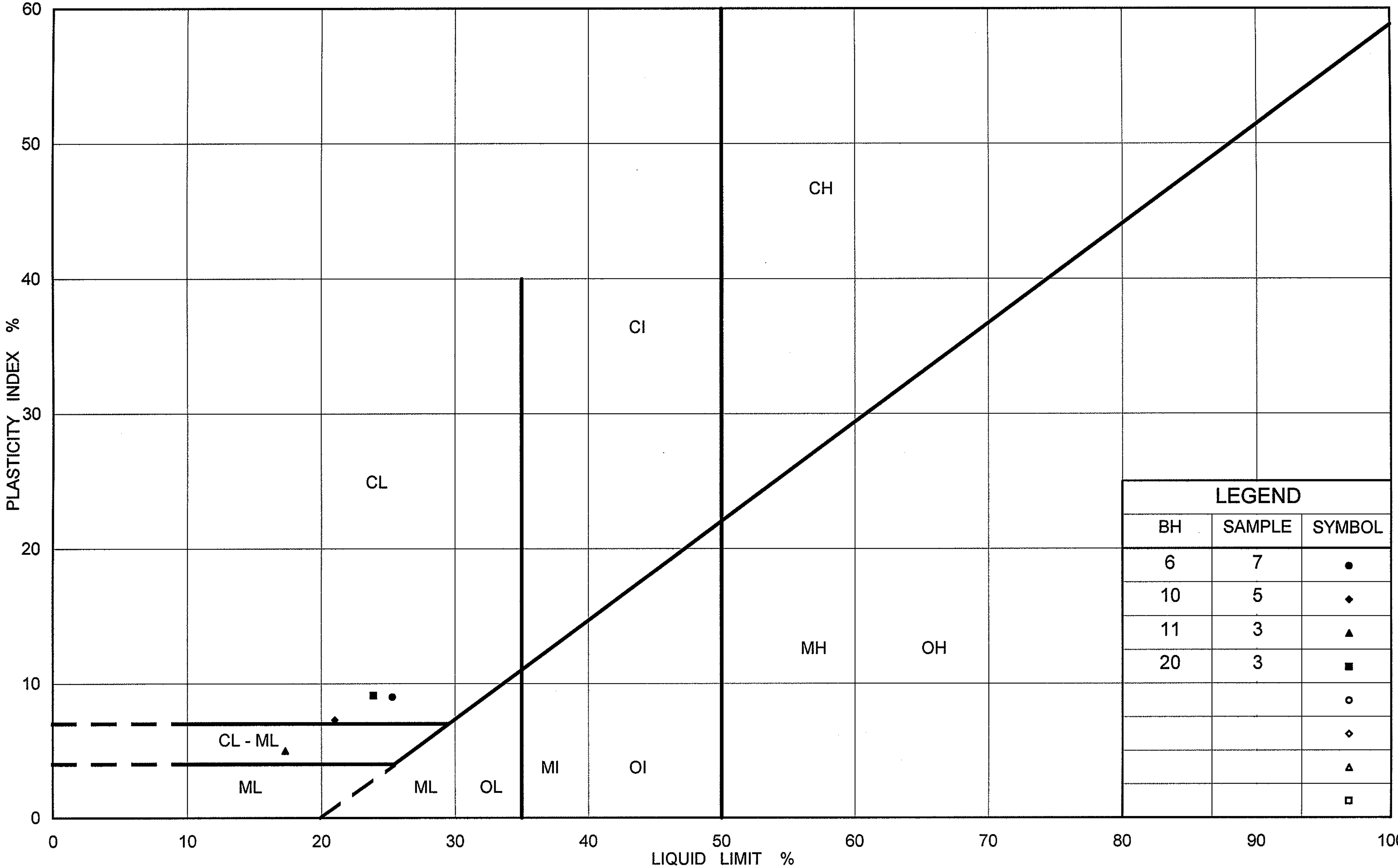


BH	SAMPLE	SYMBOL
6	7	●
10	5	◆
11	3	▲
20	3	■
		○
		◇
		△
		□

Ministry of Transportation

Ontario

PLASTICITY CHART
Clayey Silt with Sand (Till)

Figure No. 4

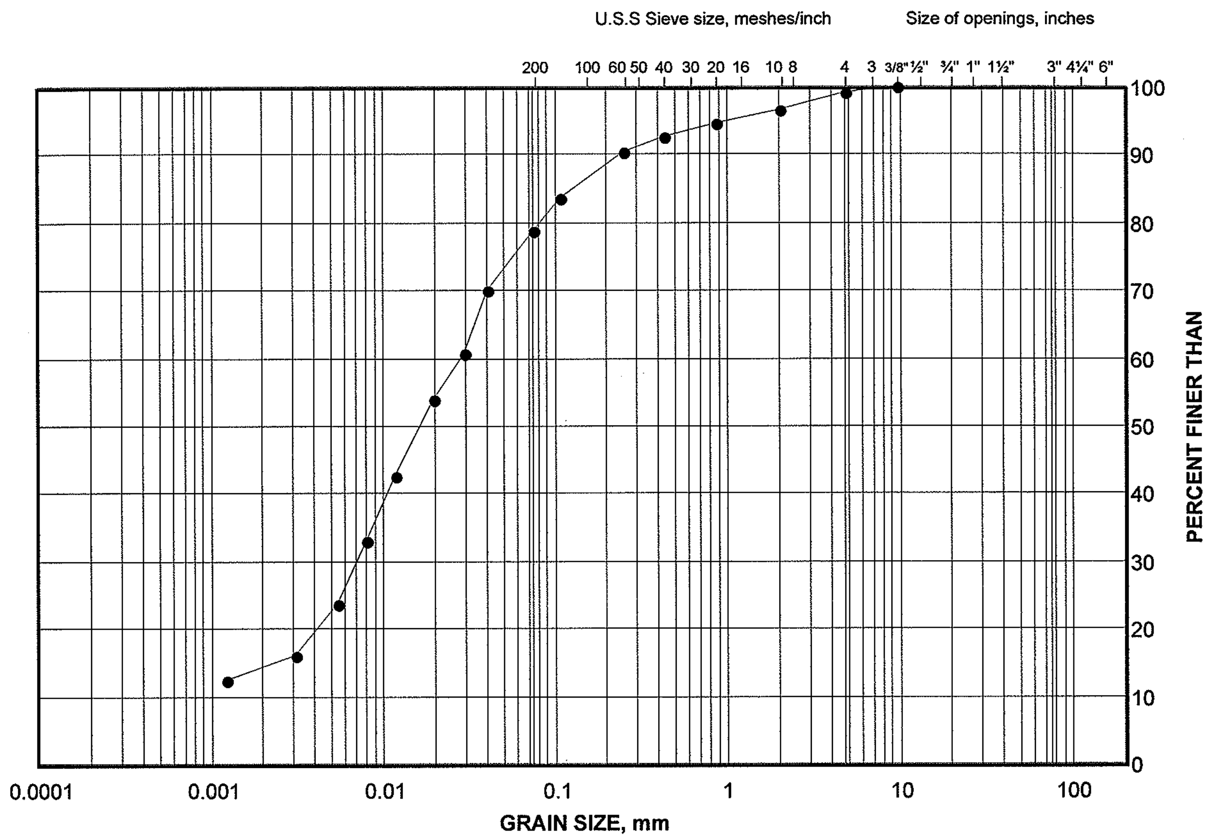
Project No. 06-1111-021

Checked By:

GRAIN SIZE DISTRIBUTION

Sandy Silt, some Clay

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)
●	2	9	172.2

Project Number: 06-1111-021

Checked By: ____________________

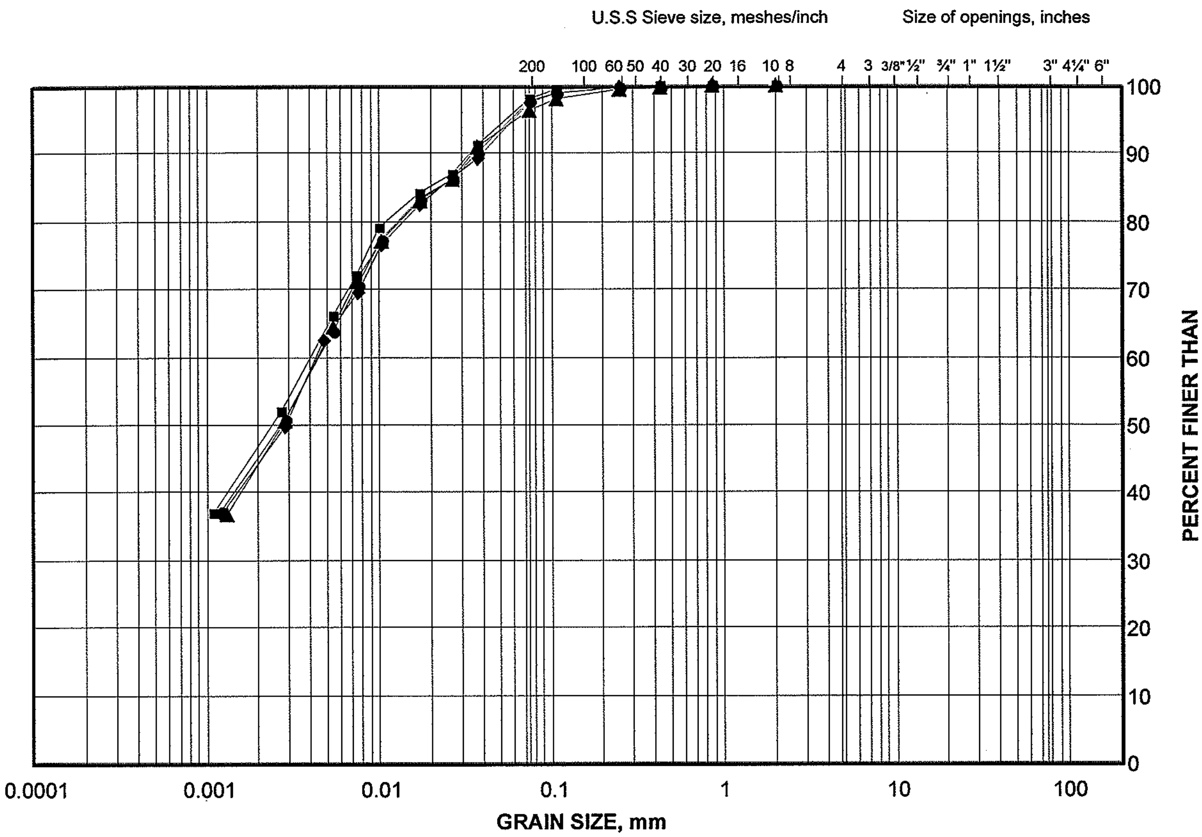
Golder Associates

Date: 11-Apr-07

GRAIN SIZE DISTRIBUTION

Clayey Silt to Silty Clay

FIGURE 6



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	6	12	169.0
■	5	6	173.5
◆	4	8	173.3
▲	11	9	167.5

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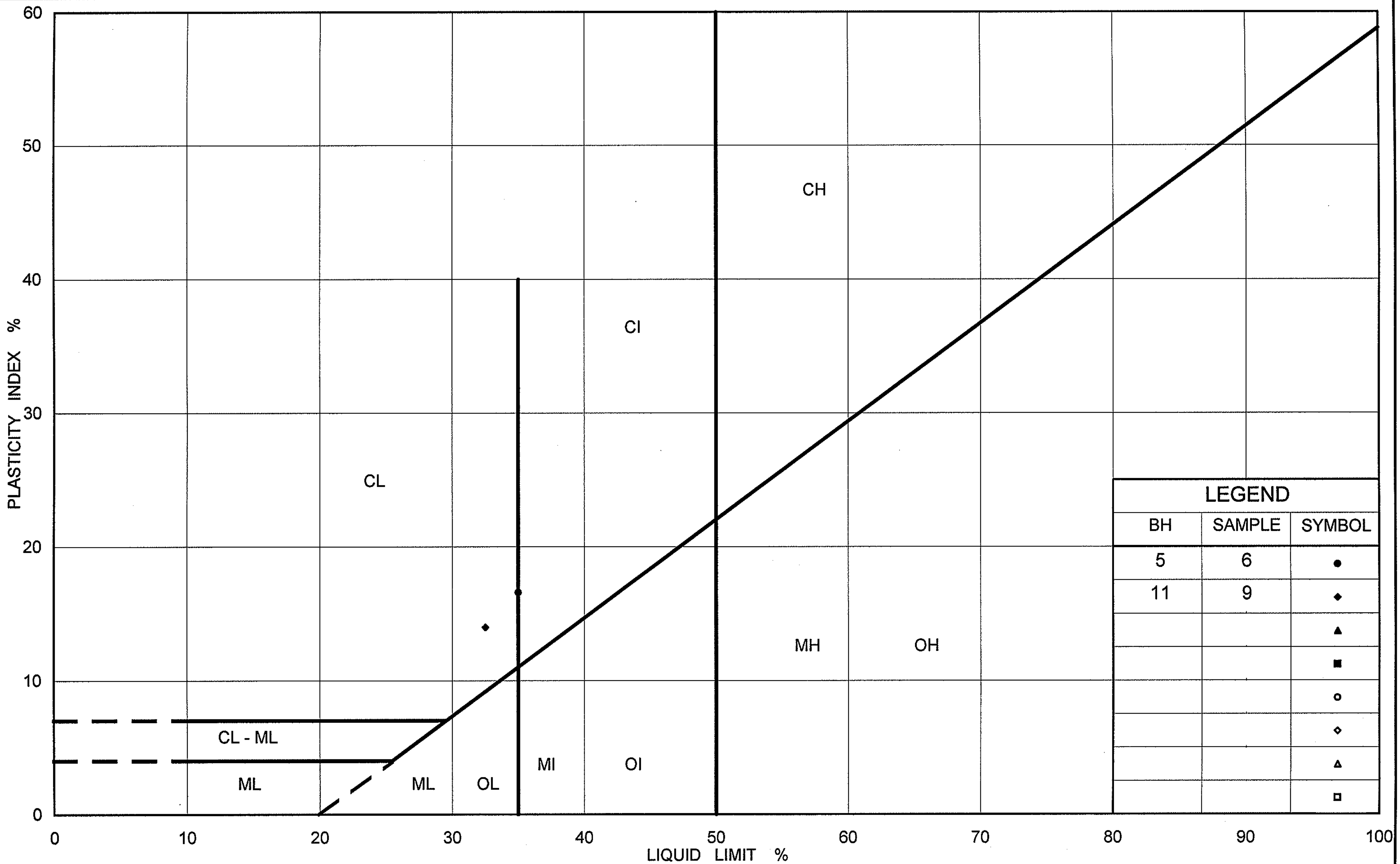
Checked By: ____________________

Golder Associates

Date: 11-Apr-07

Oct 75, FF-S-21



LEGEND		
BH	SAMPLE	SYMBOL
5	6	●
11	9	◆
		▲
		■
		○
		◇
		△
		□

Ministry of Transportation

Ontario

PLASTICITY CHART
Clayey Silt to Silty Clay

Figure No. 7

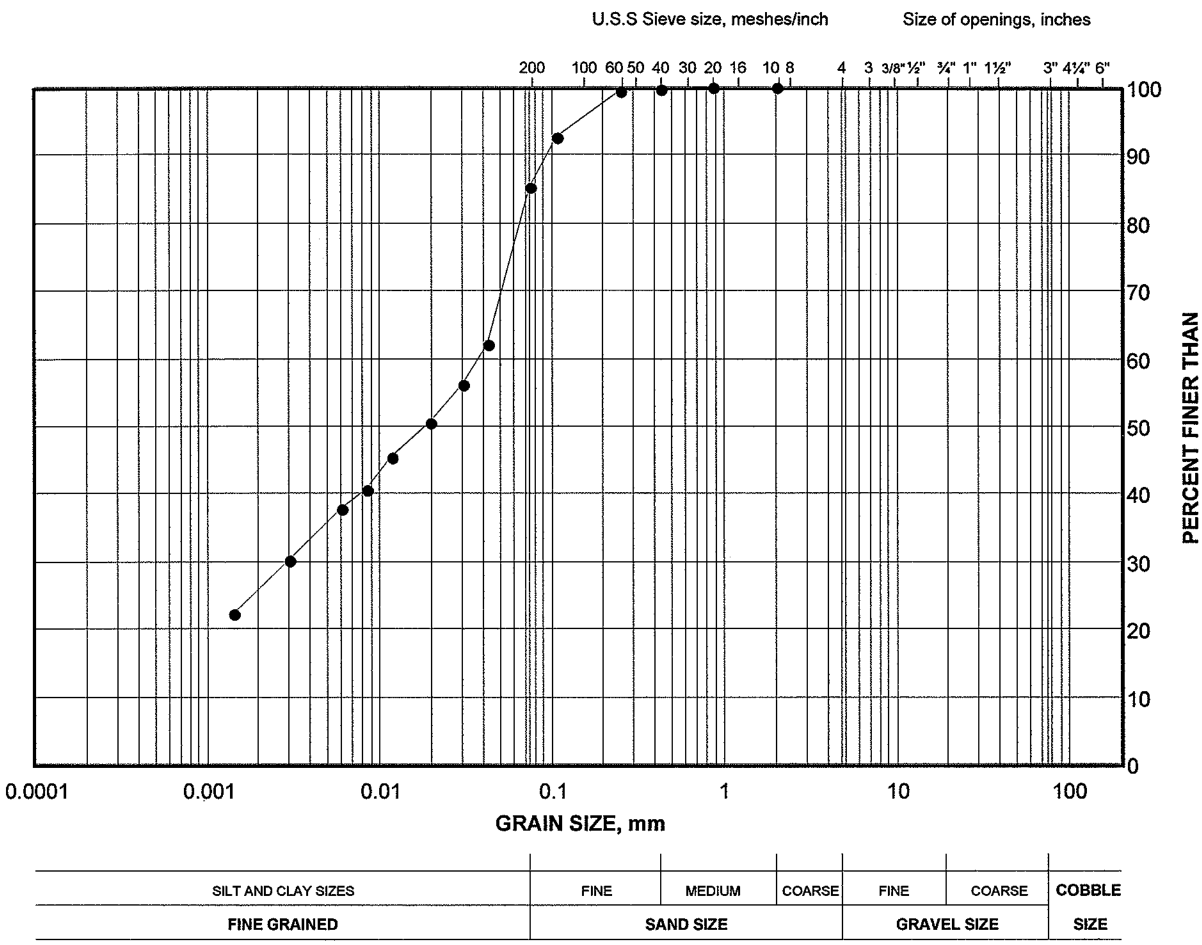
Project No. 06-1111-021

Checked By:

GRAIN SIZE DISTRIBUTION

Clayey Silt, some Sand

FIGURE 8



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	7	9	172.9

Project Number: 06-1111-021

Checked By: ____________________

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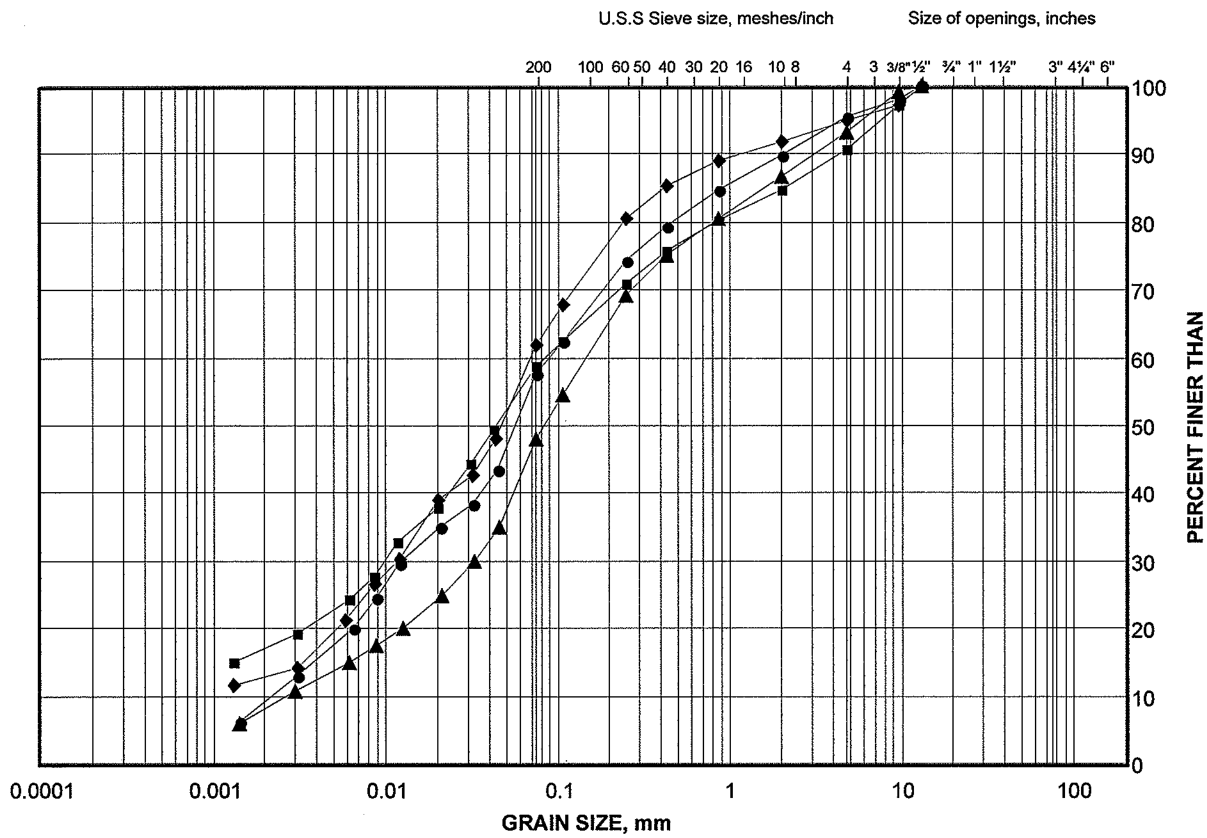
Date: 11-Apr-07

GRAIN SIZE DISTRIBUTION Silty Sand to Silty Sand (Till)	FIGURE 9



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	10	10	167.1
■	20	5	172.1
◆	5	8	170.7
▲	18	9	169.2

Project Number: 06-1111-021

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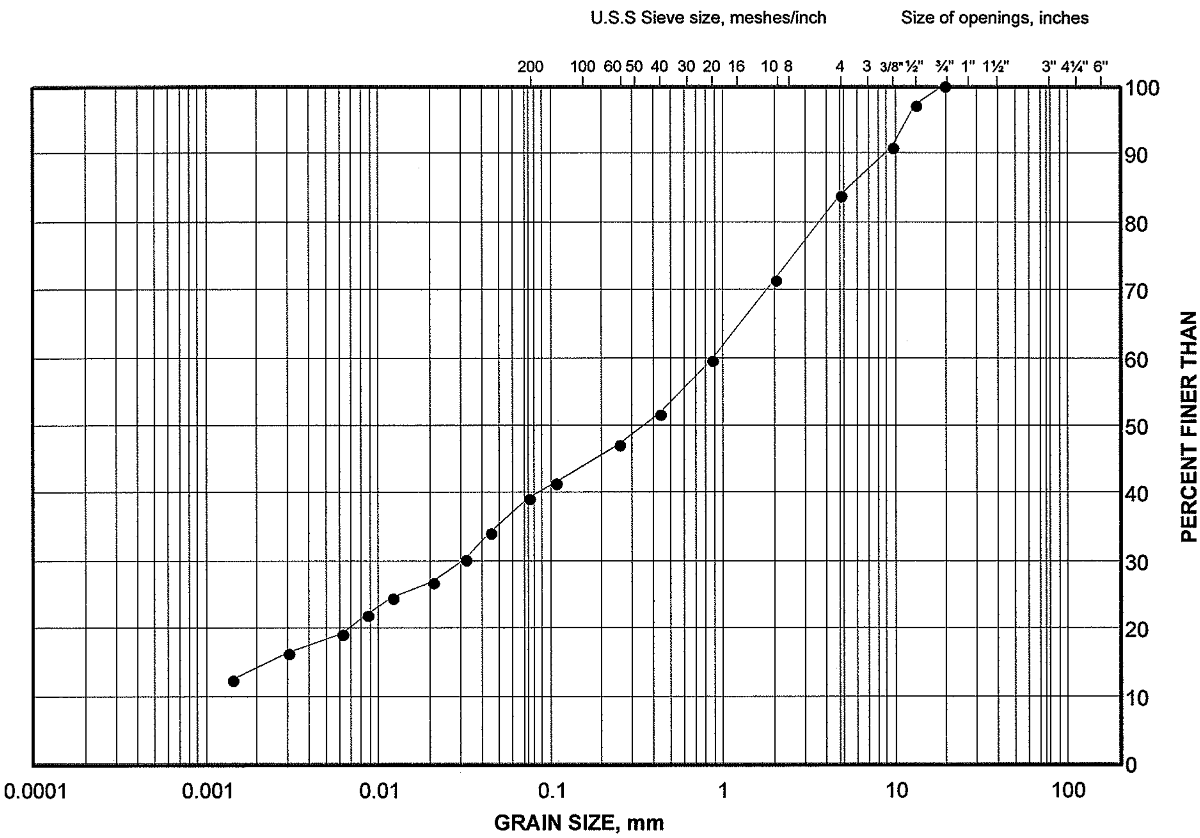
Date: 11-Apr-07

GRAIN SIZE DISTRIBUTION

Silty Sand (Residual Soil)

FIGURE 10



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	8	12	165.5

Project Number: 06-1111-021

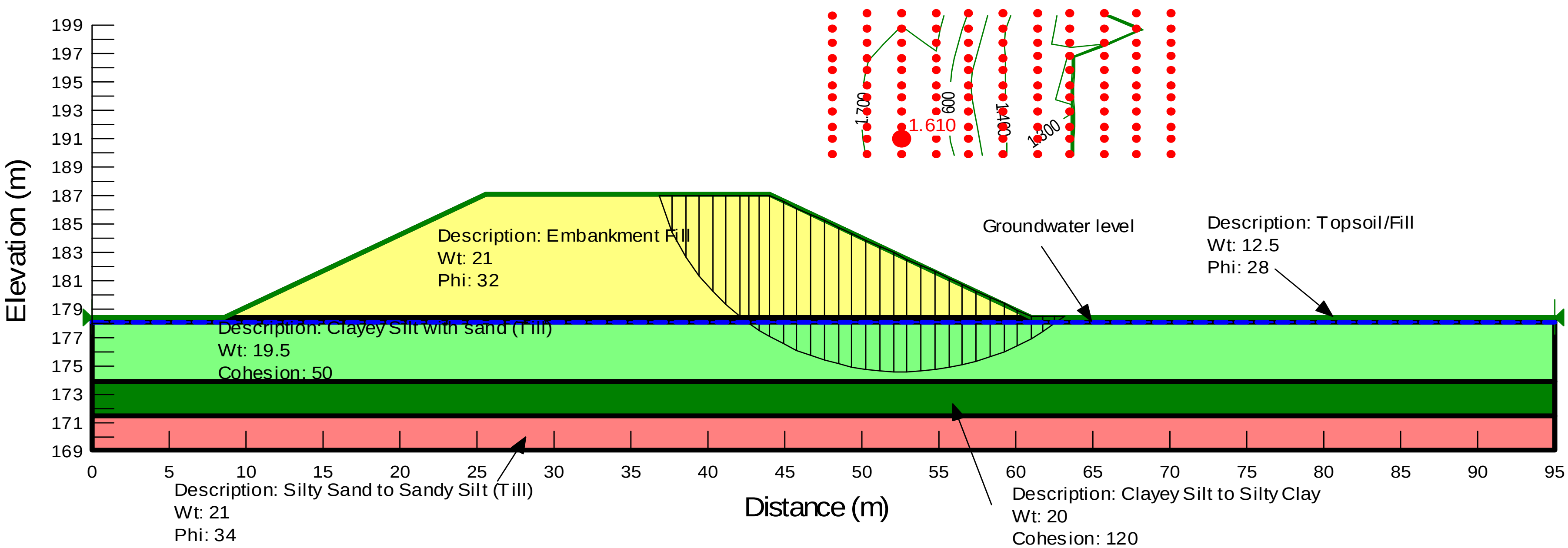
Checked By: ____________________

Golder Associates

Date: 11-Apr-07

APPROACH AND HIGH FILL EMBANKMENT STABILITY ANALYSIS

FIGURE 11



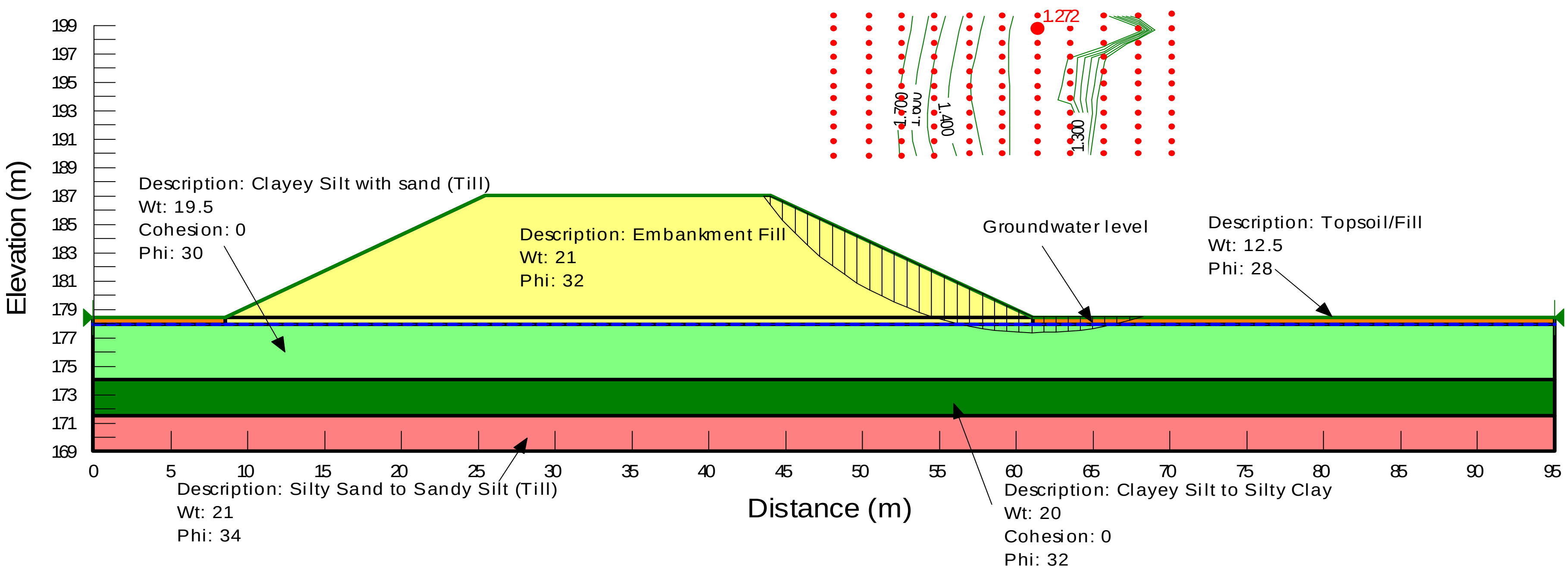
Date: April 2007
Project: 06-1111-021

Golder Associates

Drawn: BML
Checked: ASP

APPROACH AND HIGH FILL EMBANKMENT
STABILITY ANALYSIS UNDER SEISMIC LOADING (PGA = 0.065g)

FIGURE 12



Date: April 2007
Project: 06-1111-021

Golder Associates

Drawn: BML
Checked: ASP

LIST OF ABBREVIATIONS

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

I. SAMPLE TYPE

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

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

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

Dynamic Cone Penetration Resistance; N_d:

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

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

Piezo-Cone Penetration Test (CPT)

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

III. SOIL DESCRIPTION

(a) Cohesionless Soils

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

(b) Cohesive Soils

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

IV. SOIL TESTS

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

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

S:\FINALDAT\ABBREV\2000\LOFA-D00.DOC

LIST OF SYMBOLS

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

I. General

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

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. in stress: $\Delta\ \sigma$
ε	linear strain
ε_v	volumetric strain
η	coefficient of viscosity
v	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

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

(a) Index Properties (continued)

w	water content
w_l	liquid limit
w_p	plastic limit
l_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

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 δ
c'	effective cohesion
c_u, s_u	undrained shear strength (ϕ = 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

LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

WEATHERING STATE

Fresh: no visible sign of weathering.

Faintly weathered: weathering limited to the surface of major discontinuities.

Slightly weathered: penetrative weathering developed on open discontinuity surfaces but only slight weathering of rock material.

Moderately weathered: weathering extends throughout the rock mass but the rock material is not friable.

Highly weathered: weathering extends throughout rock mass and the rock material is partly friable.

Completely weathered: rock is wholly decomposed and in a friable condition but the rock texture and structure are preserved.

BEDDING THICKNESS

Description	Bedding Plane Spacing
Very thickly bedded	> 2 m
Thickly bedded	0.6 m to 2m
Medium bedded	0.2 m to 0.6 m
Thinly bedded	60 mm to 0.2 m
Very thinly bedded	20 mm to 60 mm
Laminated	6 mm to 20 mm
Thinly laminated	< 6 mm

JOINT OR FOLIATION SPACING

Description	Spacing
Very wide	> 3 m
Wide	1 - 3 m
Moderately close	0.3 - 1 m
Close	50 - 300 mm
Very close	< 50 mm

GRAIN SIZE

Term	Size*
Very Coarse Grained	> 60 mm
Coarse Grained	2 - 60 mm
Medium Grained	60 microns - 2 mm
Fine Grained	2 - 60 microns
Very Fine Grained	< 2 microns

Note: * Grains >60 microns diameter are visible to the naked eye.

CORE CONDITION

Total Core Recovery

The percentage of solid drill core recovered regardless of quality or length, measured relative to the length of the total core run.

Solid Core Recovery (SCR)

The percentage of solid drill core, regardless of length, recovered at full diameter, measured relative to the length of the total core run.

Rock Quality Designation (RQD)

The percentage of solid drill core, greater than 100 mm length, recovered at full diameter, measured relative to the length of the total core run. RQD varies from 0% for completely broken core to 100% for core in solid sticks.

DISCONTINUITY DATA

Fracture Index

A count of the number of discontinuities (physical separations) in the rock core, including both naturally occurring fractures and mechanically induced breaks caused by drilling.

Dip with Respect to (W.R.T.) Core Axis

The angle of the discontinuity relative to the axis (length) of the core. In a vertical borehole a discontinuity with a 90° angle is horizontal.

Description and Notes

An abbreviated description of the discontinuities, whether naturally occurring separations such as fractures, bedding planes and foliation planes or mechanically induced features caused by drilling such as ground or shattered core and mechanically separated bedding or foliation surfaces. Additional information concerning the nature of fracture surfaces and infillings are also noted.

Abbreviations

B - Bedding	P - Polished
FO - Foliation/Schistosity	S - Slickensided
CL - Cleavage	SM - Smooth
SH - Shear Plane/Zone	R - Ridged/Rough
VN - Vein	ST - Stepped
F - Fault	PL - Planar
CO - Contact	FL - Flexured
J - Joint	UE - Uneven
FR - Fracture	W - Wavy
MF - Mechanical Fracture	C - Curved
$\parallel$ - Parallel To	
$\perp$ - Perpendicular To	

Golder Associates

RECORD OF BOREHOLE No BH1

 METRIC

PROJECT 06-1111-021
W.P.
LOCATION N 4820789.2 ;E 603605.4
ORIGINATED BY BML
DIST
HWY Ridgeway Dr
BOREHOLE TYPE CME 75 Track Mount, 102 mm Solid Stem Augers
COMPILED BY BML
DATUM Geodetic
DATE January 24, 2007
CHECKED BY HJ

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)
180.5	GROUND SURFACE					
0.0 / 180.1	Topsoil	1	SS	4	180	
179.6 / 0.9	Sandy Silt, some gravel, trace clay (FILL) Loose Brown Moist	2	SS	17		~14
	Clayey Silt with Sand (TILL) Very stiff to hard Brown Moist	3	SS	28	179	
		4	SS	22	178	~12
		5	SS	29	177	
		6	SS	31	176	~10
175.5 / 5.0		7	SS	21		
	Sandy Silt, some clay Very dense Grey Moist to wet				175	
174.1		~~8~~	~~SS~~	~~50/0.07~~		~14
6.5	End of Borehole					

Notes:

1. Open borehole dry upon completion of drilling.

2. Borehole open to 6.5 m depth upon completion of drilling.

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

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

Golder Associates

RECORD OF BOREHOLE No BH2

 METRIC

PROJECT 06-1111-021
W.P.
LOCATION N 4820831.8 ;E 603579.3
ORIGINATED BY BML
DIST HWY Ridgeway Dr
BOREHOLE TYPE CME 75 Track Mount, 102 mm Solid Stem Augers
COMPILED BY BML
DATUM Geodetic
DATE January 24, 2007
CHECKED BY HJ

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%) w	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
180.2	GROUND SURFACE					180		
0.0	Topsoil	1	SS	3				
0.3	Clayey Silt, trace gravel (FILL), occasional boulders, Stiff to hard, Reddish brown, Moist	2	SS	14		179	o	
		~~3~~	~~SS~~	~~58/0.05~~			o	
177.6 / 2.6	Clayey Silt with Sand (TILL), Very stiff to hard, Brown, Moist	4	SS	12		178		
		5	SS	20		177		
		6	SS	43		176	o	5 29 40 26
175.2 / 5.0	Sandy Silt, some clay, Very dense, Grey, Moist	7	SS	46	▽	175		
		8	SS	60		174		
		9	SS	55		173	o	0 21 65 14
172.0 / 8.2	End of Borehole					172		

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

Notes:

1. Water level in open borehole at 5.1 m depth (Elev. 175.1 m) upon completion of drilling.

2. Borehole open to 7.0 m depth upon completion of drilling.

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

Golder Associates

RECORD OF BOREHOLE No BH3

 METRIC

PROJECT 06-1111-021

W.P.

LOCATION N 4820871.1 ;E 603548.2

ORIGINATED BY BML

DIST HWY Ridgeway Dr

BOREHOLE TYPE CME 75 Track Mount, 102 mm Solid Stem Augers

COMPILED BY BML

DATUM Geodetic

DATE January 24, 2007

CHECKED BY HJ

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
180.2	GROUND SURFACE								
0.0	Topsoil								
0.2	Clayey Silt, some sand and gravel, occasional boulders (FILL) Stiff to soft Reddish brown Moist	1	SS	15		180			
		2	SS	12		179			
		3	SS	4					
177.8 2.4	Clayey Silt with Sand (TILL) Very stiff to hard Brown Moist to dry	4	SS	26		178			
		5	SS	24		177			
		6	SS	28		176			
		7	SS	60		175			
173.9	Sandy Silt, some clay Very dense Grey Dry	8	SS	64		174			
173.5 6.7	End of Borehole								

Notes:

1. Open borehole dry upon completion of drilling.

2. Borehole open to 6.7 m depth upon completion of drilling.

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100; PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30

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

Golder Associates

RECORD OF BOREHOLE No BH4

 METRIC

PROJECT 06-1111-021
W.P.
LOCATION N 4820905.2 ;E 603511.4
ORIGINATED BY BML
DIST
HWY Ridgeway Dr
BOREHOLE TYPE CME 75 Track Mount, 102 mm Solid Stem Augers
COMPILED BY BML
DATUM Geodetic
DATE January 24, 2007
CHECKED BY HJ

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
181.2	GROUND SURFACE						
0.0	Topsoil				181		
0.3	Clayey Silt, trace sand and gravel, frequent rootlets (FILL) Stiff to firm Brown to dark brown Moist	1	SS	14	180		
		2	SS	16	179		
		3	SS	8		~21	
177.8 3.4	Clayey Silt with Sand (TILL) Very stiff to hard Brown Moist	4	SS	14	178		
		5	SS	25	177	~12	
		6	SS	35	176		
		7	SS	65	175	~9	
174.0 7.2	Clayey Silt to Silty Clay, trace sand Hard Grey Moist				174		
173.0		8	SS	37	173	~14	0 3 52 45
8.2	End of Borehole						

Notes:

1. Open borehole dry upon completion of drilling.

2. Borehole open to 8.2 m depth upon completion of drilling.

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

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

RECORD OF BOREHOLE No BH5

 METRIC

PROJECT 06-1111-021			
W.P.	LOCATION N 4820914.0 ;E 603487.7		ORIGINATED BY BML
DIST	HWY Ridgeway Dr	BOREHOLE TYPE CME 75 Track Mount, 102 mm Solid Stem Augers	COMPILED BY BML
DATUM Geodetic		DATE Started on Jan. 25, 2007; Completed on Feb. 1, 2007	CHECKED BY HJ

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

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	REMARKS (GR SA SI CL)
178.4	GROUND SURFACE				
0.0	Topsoil				
0.3	Clayey Silt with Sand (TILL) Very stiff to hard Brown Moist	1	SS	20	
		2	SS	33	
		3	SS	27	
		4	SS	35	
	Becoming grey below 3.5 m depth (Elev. 174.9 m)	5	SS	28	
174.0 4.4	Clayey Silt to Silty Clay, trace sand Hard Grey Dry	6	SS	49	0 2 51 47
		7	SS	46	
171.1 7.3	Sandy Silt, some clay (TILL) Very dense Reddish brown Dry	8	SS	90/0.15	5 33 49 13
169.4 9.0	Highly to moderately weathered, red, calcareous SHALE BEDROCK (Queenston Formation) with occasional grey siltstone and limestone layers up to 100 mm thick	9	SS	50/0.05	
		10	SS	50/0.10	
	NQ Coring from 10.3 m depth (Elev. 168.1 m)	1	NQ RC	REC 96%	RQD = 33%
		2	NQ RC	REC 100%	RQD = 36%
	For coring details see Record of Drillhole BH5	3	NQ RC	REC 100%	RQD = 53%
165.4		4	NQ RC	REC 93%	RQD = 48%
13.0	End of Borehole				

Elevation scale: 178, 177, 176, 175, 174, 173, 172, 171, 170, 169, 168, 167, 166

Notes:

1. Water level in open borehole on Feb. 1, 2007 before resuming drilling at 1.5 m depth (Elev. 176.9 m).

2. Water level in piezometer on April 3, 2007 at 0.3 m depth (Elev. 178.1 m).

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

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

RECORD OF DRILLHOLE: BH5

PROJECT: 06-1111-021
SHEET 1 OF 1

LOCATION: N 4820914.0 ;E 603487.7
DRILLING DATE: Started on Jan. 25, 2007; Completed on Feb. 1, 2007
DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---
DRILL RIG: CME 75
DRILLING CONTRACTOR: Geo-Environmental Ltd.

JN - Joint, FLT - Fault, SHR- Shear, VN - Vein, CJ - Conjugate, BD- Bedding, FO- Foliation, CO- Contact, OR- Orthogonal, CL - Cleavage, PL - Planar, CU- Curved, UN- Undulating, ST - Stepped, IR - Irregular, PO- Polished, K - Slickensided, SM- Smooth, Ro - Rough, MB- Mechanical Break, BR - Broken Rock

NOTE: For additional abbreviations refer to list of abbreviations & symbols.

DEPTH SCALE METRES	DESCRIPTION	ELEV. DEPTH (m)	RUN No.	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	CAVITIES OBSERVED IN BOREHOLE VIDEO
	GROUND SURFACE	168.21			
	Highly to moderately weathered, thinly layered, red, very fine grained, very weak to weak, calcareous SHALE BEDROCK (Queenston Formation)	10.16	1	,BC,,	
				,JN, BD,PL,Ro	
			2		
	Occasional interbeds of weathered, grey siltstone and limestone		3	,BC,,	Screen
12				,JN, BD,PL,Ro	
	Elevation (m) Thickness (mm)			,JN, BD,PL,Ro	
	166.5 50			,JN, BD,PL,Ro	
	160.0 125		4	,JN, BD,PL,Ro	
	165.6 25	165.34			
		13.03			
	All fractures are rough bedding				
14	End of Drillhole				
16					
18					
20					
22					
24					
26					
28					
30					

DEPTH SCALE
1 : 100

Golder Associates

LOGGED: BML
CHECKED: HJ

Golder Associates

RECORD OF BOREHOLE No BH6

 METRIC

PROJECT 06-1111-021

W.P.

LOCATION N 4820927.5 ;E 603502.1

ORIGINATED BY BML

DIST HWY Ridgeway Dr

BOREHOLE TYPE CME 75 Track Mount, 102 mm Solid Stem Augers

COMPILED BY BML

DATUM Geodetic

DATE January 25, 2007

CHECKED BY HJ

ELEV DEPTH (m)	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
183.0	GROUND SURFACE					
0.0	Topsoil					
0.2	Clayey Silt, trace to some sand and gravel (FILL), occasional rootlets. Firm to stiff. Reddish brown. Moist	1	SS	5		
		2	SS	8		
		3	SS	5		
		4	SS	5		
		5	SS	5		
		6	SS	14		
177.8 / 5.2	Clayey Silt with Sand (TILL). Very stiff. Brown. Moist	7	SS	19		13 31 36 20
		8	SS	25		
173.5 / 9.4	Sandy Silt to Silty Sand, trace to some clay (TILL), contains rock fragments. Very dense. Reddish brown. Moist	9	SS	43		
		10	SS	50/0.25		
		~~11~~	~~SS~~	~~70/0.10~~		
169.6 / 13.4	Clayey Silt to Silty Clay, trace sand. Hard. Grey. Moist	12	SS	60		0 2 53 45
168.0 / 14.9	Silty Sand, some gravel and clay (Residual Soil). Very dense. Reddish brown. Moist	~~13~~	~~SS~~	~~50/0.08~~	∇ (approx. El. 168.1)	
166.2	Red SHALE BEDROCK (Queenston Formation) with interlayers of grey siltstone	14	SS	118		
165.7 / 17.2	End of Borehole					

Note:

1. Water level in open borehole at 14.9 m depth (Elev. 168.1 m) upon completion of drilling.

ELEVATION SCALE: 182, 181, 180, 179, 178, 177, 176, 175, 174, 173, 172, 171, 170, 169, 168, 167, 166

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED; 20 40 60 80 100

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L

WATER CONTENT (%): 10 20 30

UNIT WEIGHT γ kN/m^3

+ 3, × 3: Numbers refer to Sensitivity

O 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No BH7

 METRIC

PROJECT 06-1111-021			
W.P.	LOCATION N 4820934.8 ;E 603457.6		ORIGINATED BY BML
DIST	HWY Ridgeway Dr	BOREHOLE TYPE CME 75 Track Mount, 102 mm Solid Stem Augers	COMPILED BY BML
DATUM Geodetic		DATE January 29, 2007	CHECKED BY HJ

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
179.3	GROUND SURFACE					
0.0	Asphalt					
	Sand and Gravel (FILL)					
178.5 / 0.8	Clayey Silt, some sand (Probable FILL) Stiff to firm Brown Moist Occasional dark brown organics/rootlets between 2.2 m and 3.1 m depth (Elev. 177.1 m to 176.2 m)	2	SS	14		
		3	SS	10		
		4	SS	6		6 18 50 26
176.0 / 3.4	Clayey Silt with Sand (TILL) Hard Brown Moist	5	SS	18		
		6	SS	42		
		7	SS	50/0.15		
173.7 / 5.6	Clayey Silt to Silty Clay, some sand Hard Grey Moist Reddish brown Clayey Silt, some sand from 6.1 m to 6.6 m depth	8	SS	44		
		9	SS	56		0 13 62 25
		10	SS	52		
171.8 / 7.5	Sandy Silt to Silty Sand, trace to some clay (TILL) Very dense Reddish brown Moist	11	SS	50/0.13		
170.2 / 9.1	Red SHALE BEDROCK (Queenston Formation) with interlayers of grey siltstone	12	SS	50/0.07		
		13	SS	50/0.07		
		14	SS	108		
		15	SS	100/0.10	▽	
164.0 / 15.3	End of Borehole	16	SS	50/0.08		

Notes:

1. Water level in open borehole at 14.0 m depth (Elev. 153.2 m) upon completion of drilling.

2. Borehole open to 15.3 m depth upon completion of drilling.

Natural moisture content (%) plotted: sample 2 ≈ 15; sample 4 ≈ 26 (plastic limit ≈ 19, liquid limit ≈ 31); sample 6 ≈ 10; sample 8 ≈ 17; sample 9 ≈ 11; sample 11 ≈ 9.

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

Golder Associates

RECORD OF BOREHOLE No BH8

 METRIC

PROJECT 06-1111-021			
W.P.	LOCATION N 4820952.8 ;E 603469.6	ORIGINATED BY BML	
DIST	HWY Ridgeway Dr	BOREHOLE TYPE CME 75 Track Mount, 102 mm Solid Stem Augers	COMPILED BY BML
DATUM Geodetic	DATE January 30, 2007	CHECKED BY HJ	

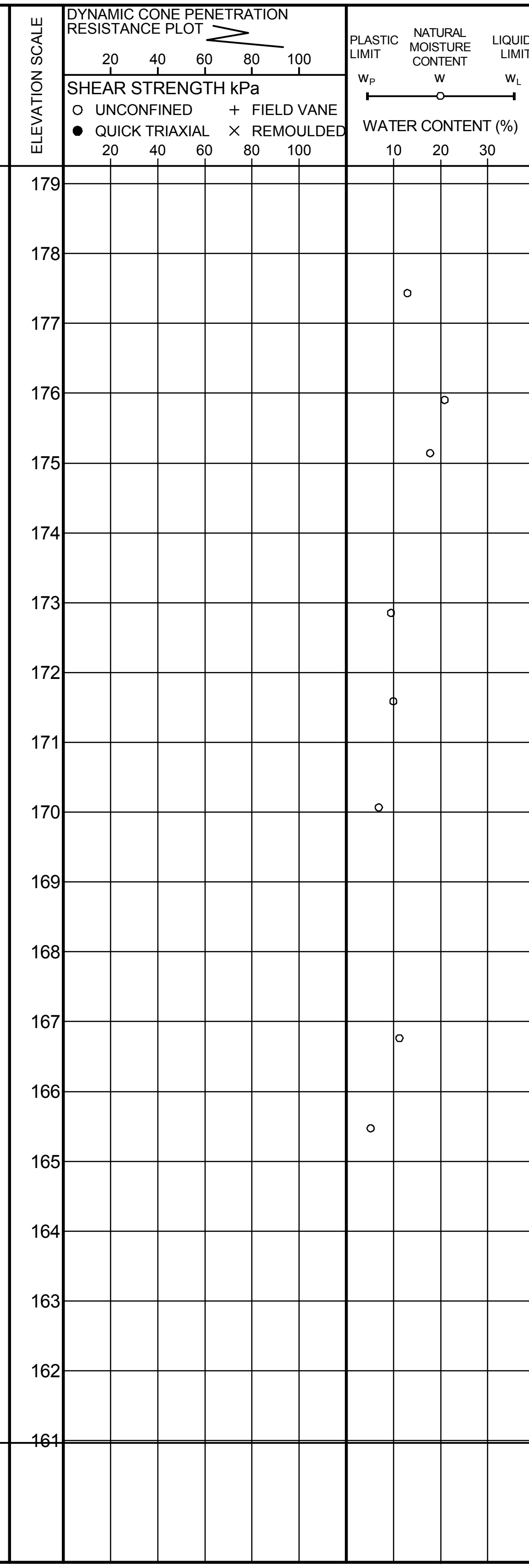
SOIL PROFILE

ELEV DEPTH	DESCRIPTION
179.3	GROUND SURFACE
0.0	Asphalt
0.2	Sand and Gravel (FILL) Compact Light brown Moist
178.1 1.2	Clayey Silt, some sand (Probable FILL) Stiff to very stiff Brown Moist
174.9 4.3	Clayey Silt with Sand (TILL) Very stiff Brown Moist
173.0 6.3	Silty Sand to Sandy Silt (TILL), trace to some gravel Very dense Grey to reddish brown Moist
167.7 11.6	Clayey Silt to Silty Clay, trace sand Hard Grey Moist
166.1 13.1	Silty Sand, some gravel and clay (Residual Soil) Very dense Reddish brown Moist
165.4 13.9	Red SHALE BEDROCK (Queenston Formation) with interlayers of grey siltstone
161.0 18.3	End of Borehole

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	17	
2	SS	10	
3	SS	20	
4	SS	9	
5	SS	15	
6	SS	30	
7	SS	90	
~~8~~	~~SS~~	~~50/.10~~	
~~9~~	~~SS~~	~~90/0.10~~	
~~10~~	~~SS~~	~~100/0.10~~	
11	SS	100	
~~12~~	~~SS~~	~~100/0.13~~	17 44 25 14
~~13~~	~~SS~~	~~100/0.13~~	

GROUND WATER CONDITIONS: ∇ at approx. Elev. 166.5



UNIT WEIGHT γ kN/m³

Notes:

1. Water level in open borehole at 12.8 m depth (Elev. 166.5 m) upon completion of drilling.

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

Golder Associates

RECORD OF BOREHOLE No BH9

 METRIC

PROJECT 06-1111-021
W.P.
LOCATION N 4820951.1 ;E 603449.2
ORIGINATED BY BML
DIST
HWY Ridgeway Dr
BOREHOLE TYPE CME 75 Track Mount, 102 mm Solid Stem Augers
COMPILED BY BML
DATUM Geodetic
DATE January 31, 2007
CHECKED BY HJ

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
179.0	GROUND SURFACE											
0.0	Sand and Gravel (FILL) Compact Brown Moist											
178.4 / 0.6	Clayey Silt, some sand (Probable FILL) Stiff Brown Moist	1	SS	9		178						
		2	SS	12		177	o					
		3	SS	11								
		4	SS	10		176	o					
174.6 / 4.4	Clayey Silt with Sand, trace gravel (TILL) Very stiff to hard Brown Moist Becoming grey below 5.5 m depth (Elev. 173.5 m)	5	SS	30		174						
		6	SS	50/0.07		173	o		4	25	56	15
172.0 / 7.0	Clayey Silt to Silty Clay, trace sand Hard Grey Moist	7	SS	50/.13		171	o					
		8	SS	79		170						
169.0 / 10.1	Silty Sand, some gravel and clay (Residual Soil) Very dense Reddish brown Moist					169						
168.2 / 10.8	Red SHALE BEDROCK (Queenston Formation) with interlayers of grey siltstone	9	SS	50/0.13		168	o					
		10	SS	50/0.07		167						
166.6 / 12.4	End of Borehole											

Notes:

1. Open borehole dry upon completion of drilling.

2. Borehole open to 12.4 m depth upon completion of drilling.

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

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

RECORD OF BOREHOLE No BH10

 METRIC

PROJECT 06-1111-021

W.P.

DIST HWY Ridgeway Dr

DATUM Geodetic

LOCATION N 4820965.9 ;E 603459.5

BOREHOLE TYPE CME 75 Track Mount, 102 mm Solid Stem Augers

DATE January 31, 2007

ORIGINATED BY BML

COMPILED BY BML

CHECKED BY HJ

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	WATER CONTENT (%) w	PLASTIC LIMIT – LIQUID LIMIT	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
179.3	GROUND SURFACE							
0.0	Sand and Gravel, trace asphalt (FILL) Dense Light brown Moist	1	SS	31				
177.5 1.8	Clayey Silt, some sand (Probable FILL) Stiff to very stiff Brown Moist	2	SS	26		o		
		3	SS	11		o		
	Sand seam, compact, dark brown to grey, moist between 3.7 to 4.1 m depth	4	SS	15				
174.7 4.6	Clayey Silt with Sand (TILL) Very stiff Brown Moist	5	SS	24		o	├─┤	5 32 43 20
173.4 5.9	Silty Sand to Sandy Silt, trace to some clay (TILL), trace to some gravel, occasional boulders Very dense Grey Moist	~~6~~	~~SS~~	~~50/0.05~~		o		
		~~7~~	~~SS~~	~~60/0.15~~		o		
		~~8~~	~~SS~~	~~80/0.10~~		o		
		~~9~~	~~SS~~	~~80/0.13~~				
		~~10~~	~~SS~~	~~80/0.13~~	∇	o		5 36 50 9
166.4 12.9	Silty Sand, some gravel and clay (Residual Soil) Very dense Reddish brown Moist	~~11~~	~~SS~~	~~80/0.13~~				
165.5 13.9	Red SHALE BEDROCK (Queenston Formation) with interlayers of grey siltstone	~~12~~	~~SS~~	~~50/0.05~~				
163.9 15.4	End of Borehole							

Elevation scale: 179, 178, 177, 176, 175, 174, 173, 172, 171, 170, 169, 168, 167, 166, 165, 164

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

SHEAR STRENGTH kPa: O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100

PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L; WATER CONTENT (%) 10 20 30

UNIT WEIGHT γ kN/m³

Notes:

1. Water level in open borehole at 12.2 m depth (Elev. 167.1 m) upon completion of drilling.

2. Borehole open to 15.4 m depth upon completion of drilling.

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

Golder Associates

RECORD OF BOREHOLE No BH11

 METRIC

PROJECT	06-1111-021		
W.P.		LOCATION	N 4820973.6 ;E 603423.8
ORIGINATED BY	BML		
DIST		HWY	Ridgeway Dr
BOREHOLE TYPE	CME 75 Track Mount, 102 mm Solid Stem Augers	COMPILED BY	BML
DATUM	Geodetic	DATE	January 29, 2007
CHECKED BY	HJ		

SOIL PROFILE

ELEV DEPTH	DESCRIPTION
176.7	GROUND SURFACE
0.0	Topsoil
0.3	Clayey Silt with Sand (TILL) Very stiff to hard Brown Moist Becoming grey below 2.1 m (Elev. 174.6 m)
173.0 3.7	Silty Sand to Sandy Silt, trace to some clay (TILL) Very dense Grey Moist
167.8 8.8	Clayey Silt to Silty Clay, trace sand Hard Grey Moist
166.3	Silty Sand, some gravel and clay (Residual Soil) Very dense Reddish brown Moist
10.7	Red SHALE BEDROCK (Queenston Formation) with interlayers of grey siltstone
162.8 13.9	End of Borehole

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	28	
2	SS	37	
3	SS	26	
4	SS	58	
5	SS	56/0.15	
6	SS	50/0.10	
7	SS	84/0.07	
8	SS	75/0.10	
9	SS	50/0.10	0 4 51 45
10	SS	50/0.07	
11	SS	50/0.07	
12	SS	90/0.15	

GROUND WATER CONDITIONS: ∇ at Elev. 166.0

ELEVATION SCALE: 176, 175, 174, 173, 172, 171, 170, 169, 168, 167, 166, 165, 164, 163

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: 20 40 60 80 100
○ UNCONFINED + FIELD VANE
● QUICK TRIAXIAL × REMOULDED

PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L

WATER CONTENT (%): 10 20 30

UNIT WEIGHT γ kN/m³

Notes:

1. Water level in open borehole at 10.7 m depth (Elev. 166.0 m) upon completion of drilling.

2. Borehole open to 12.5 m depth upon completion of drilling.

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

MIS-MTO 001 06-1111-021.GPJ GAL-MISS.GDT 7/20/07 DD

RECORD OF BOREHOLE No BH12

 METRIC

PROJECT 06-1111-021			
W.P.	LOCATION N 4820987.6 ;E 603433.6		ORIGINATED BY BML
DIST	HWY Ridgeway Dr	BOREHOLE TYPE CME 75 Track Mount, 110 mm I.D. Solid Stem Augers	COMPILED BY BML
DATUM Geodetic	DATE February 1, 2007		CHECKED BY HJ

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%)
176.6	GROUND SURFACE				
176.2 / 0.0	Topsoil				
0.4	Clayey Silt with Sand (TILL) Hard Brown Moist	1	SS	36	
		2	SS	72	
	Becoming grey below 2.3 m depth (Elev. 174.3 m)				
173.5 / 3.1	Silty Sand to Sandy Silt, trace to some clay (TILL) Very dense Grey	3	SS	50/0.10	
		4	SS	50/0.07	
	Some gravel in auger cuttings, augers grinding on possible cobbles/boulders at 5.2 m depth	5	SS	50/0.13	
		6	SS	50/0.10	
		7	SS	50/0.07	
166.2 / 10.4	Silty Sand, some gravel and clay (Residual Soil) Very dense Reddish brown Moist	8	SS	50/0.13	
164.7 / 11.9	Red SHALE BEDROCK (Queenston Formation) with interlayers of grey siltstone and limestone	9	SS	50/0.07	
		1	NQ RC	REC 0.33%	RQD = 0%
	NQ Coring from 12.0 m depth to 14.9 m depth (Elev. 164.2 m to 161.7 m)	2	NQ RC	REC 90%	RQD = 63%
	For coring details see Record of Drillhole BH12	3	NQ RC	REC 91%	RQD = 18%
161.6 / 14.9	End of Borehole Note: 1. Water level in piezometer on April 3, 2007 at 0.0 m depth (Elev. 176.6 m).				

Column headings: STRAT PLOT; GROUND WATER CONDITIONS; ELEVATION SCALE (176 – 162); DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100); SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED (20, 40, 60, 80, 100); PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) (10, 20, 30); UNIT WEIGHT γ kN/m^3; GR SA SI CL.

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

PROJECT: 06-1111-021

RECORD OF DRILLHOLE: BH12

SHEET 1 OF 1

LOCATION: N 4820987.6 ;E 603433.6

DRILLING DATE: February 1, 2007

DATUM: Geodetic

DRILL RIG: CME 75

INCLINATION: -90° AZIMUTH: ---

DRILLING CONTRACTOR: Geo-Environmental Ltd.

JN - Joint, FLT - Fault, SHR - Shear, VN - Vein, CJ - Conjugate
BD - Bedding, FO - Foliation, CO - Contact, OR - Orthogonal, CL - Cleavage
PL - Planar, CU - Curved, UN - Undulating, ST - Stepped, IR - Irregular
PO - Polished, K - Slickensided, SM - Smooth, Ro - Rough, MB - Mechanical Break
BR - Broken Rock
NOTE: For additional abbreviations refer to list of abbreviations & symbols.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	DISCONTINUITY DATA - TYPE AND SURFACE DESCRIPTION	CAVITIES OBSERVED IN BOREHOLE VIDEO
		GROUND SURFACE		164.53			
		Moderately to highly weathered, thinly layered, red, very fine grained, very weak to weak, calcareous SHALE BEDROCK (Queenston Formation)		152.51 12.02	1	,BC,,	
		Occasional interbeds of weathered, grey siltstone and limestone			2	,JN, BD,PL,Ro ,JN, BD,PL,Ro ,BC,,	Screen
14		Elevation (m) Thickness (mm) 161.9 25			3	,JN, BD,PL,Ro ,JN, BD,PL,Ro ,JN, BD,PL,Ro ,BC,,	
		All fractures are rough bedding		161.64 14.91			
		End of Drillhole					

Other columns: PENETRATION RATE min/(m); COLOUR % RETURN; FLUSH; RECOVERY TOTAL CORE % / SOLID CORE % (80 60 40 20); R.Q.D. % (80 60 40 20); FRACT. INDEX PER 0.3 m (5 10 15 20); B Angle (0 90 180 270); DIP w.r.t. CORE AXIS (0 30 60 90); Jr Ja Jn; HYDRAULIC CONDUCTIVITY K, cm/sec (10^{-6} 10^{-5} 10^{-4} 10^{-3}); Diametral Point Load Index (MPa) (2 4 6); RMC -Q' AVG.

Depth scale: 14, 16, 18, 20, 22, 24, 26, 28, 30, 32

DEPTH SCALE
1 : 100

Golder Associates

LOGGED: BML
CHECKED: HJ

Golder Associates

RECORD OF BOREHOLE No BH13

 METRIC

PROJECT 06-1111-021
W.P.
DIST HWY Ridgeway Dr
DATUM Geodetic
LOCATION N 4820997.5 ;E 603410.1
BOREHOLE TYPE CME 75 Track Mount, 102 mm Solid Stem Augers
DATE January 30, 2007
ORIGINATED BY BML
COMPILED BY BML
CHECKED BY HJ

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%) (w)
179.1	GROUND SURFACE				179	
0.0	Topsoil					
0.3	Clayey Silt with Sand (TILL) Very stiff to hard Brown Moist	1	SS	23	178	~12
		2	SS	28	177	
		3	SS	33		~12
		4	SS	43	176	
	Becoming grey below 4.3 m depth (Elev. 174.8 m)	5	SS	28	175 / 174	~8
172.7 / 6.4	Silty Sand to Sandy Silt, trace to some clay (TILL), contains rock fragments Very dense Moist	6	SS	65	173 / 172	~8
		7	SS	50/0.13		~8
171.1 / 8.1	End of Borehole Notes: 1. Open borehole dry upon completion of drilling. 2. Borehole open to 8.1 m depth upon completion of drilling.					

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

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

RECORD OF BOREHOLE No BH18

 METRIC

PROJECT 06-1111-021
W.P.
LOCATION N 4821028.4 ;E 603386.5
ORIGINATED BY BML
DIST HWY Ridgeway Dr
BOREHOLE TYPE CME 75 Track Mount, 110 mm I.D. Solid Stem Augers
COMPILED BY BML
DATUM Geodetic
DATE January 22, 2007
CHECKED BY HJ

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	WATER CONTENT (%)	REMARKS GR SA SI CL
178.6	GROUND SURFACE					
0.0	Topsoil					
0.3	Clayey Silt with Sand, trace gravel (TILL) Very stiff to hard Brown Moist	1	SS	19		
		2	SS	28		
		3	SS	38		
		4	SS	23		
	Becoming grey below 4.0 m depth (Elev. 174.6 m)	5	SS	30		
		6	SS	25		
172.8 / 5.8	Silty Sand to Sandy Silt, trace clay, trace gravel (TILL) Very dense Reddish brown to grey Dry	7	SS	50/0.10		
		8	SS	50/0.10		
		9	SS	50/0.13		8 43 41 8
		10	SS	55/0.13		
166.7	Clayey Silt to Silty Clay, containing shale fragments Hard Grey Moist	11	SS	50/0.07		
166.2 / 12.4	End of Borehole					

Elevation scale: 178, 177, 176, 175, 174, 173, 172, 171, 170, 169, 168, 167

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED: 20 40 60 80 100

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

UNIT WEIGHT γ kN/m³

Note:

1. Water level in piezometer on April 3, 2007 at 1.4 m depth (Elev. 177.2 m).

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

MIS-MTO 001 06-1111-021.GPJ GAL-MISS.GDT 7/20/07 DD

Golder Associates

RECORD OF BOREHOLE No BH20

 METRIC

PROJECT 06-1111-021
W.P.
LOCATION N 4821058.6 ;E 603342.9
ORIGINATED BY BML
DIST
HWY Ridgeway Dr
BOREHOLE TYPE CME 75 Track Mount, 102 mm Solid Stem Augers
COMPILED BY BML
DATUM Geodetic
DATE January 22, 2007
CHECKED BY HJ

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
176.9	GROUND SURFACE					
0.0	Topsoil					
0.3	Clayey Silt with Sand (TILL) Firm to hard Brown Moist					
		1	SS	6	176	
		2	SS	17	175	
		3	SS	45	174	
173.3		4	SS	32		
3.6	Silty Sand to Sandy Silt, trace to some clay (TILL), occasional cobbles Very dense Grey Moist Augers grinding from 3.6 m to 3.8 m depth				173	
		5	SS	92	172	10 31 42 17
					171	
170.2		6	SS	98		
6.7	End of Borehole					

Notes:

1. Open borehole dry upon completion of drilling.

2. Borehole open to 5.5 m depth upon completion of drilling.

Column headings: SOIL PROFILE; SAMPLES; STRAT PLOT; GROUND WATER CONDITIONS; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m^3

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

Golder Associates

RECORD OF BOREHOLE No BH21

 METRIC

PROJECT 06-1111-021
W.P.
LOCATION N 4821098.4 ;E 603299.1
ORIGINATED BY BML
DIST
HWY Ridgeway Dr
BOREHOLE TYPE CME 75 Track Mount, 102 mm Solid Stem Augers
COMPILED BY BML
DATUM Geodetic
DATE January 22, 2007
CHECKED BY HJ

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)
179.9	GROUND SURFACE					
0.0	Clayey silt, trace to some sand, organics and rootlets (FILL) Very soft Dark brown Moist	1	SS	1	179	○
		2	SS	1	178	○
177.3 2.5	Clayey Silt with Sand (TILL) Very stiff to hard Brown Moist	3	SS	8	177	
		4	SS	20		○
		5	SS	33	176	○
		6	SS	41	175	
					174	
173.7	Silty Sand to Sandy Silt, trace to some clay (TILL), containing rock fragments Very dense Reddish brown Moist	7	SS	72		○
173.2 6.7	End of Borehole					

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

Notes:

1. Open borehole dry upon completion of drilling.

2. Borehole open to 5.8 m depth upon completion of drilling.

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

APPENDIX A

NON-STANDARD SPECIAL PROVISION
AND
OPERATIONAL CONSTRAINT

BOULDERS/OBSTRUCTIONS DURING PILE INSTALLATION - Item No.

Non-Standard Special Provision

The soils at the site are comprised of fill materials underlain by glacially-derived tills, and contain occasional cobbles and boulders. Such soils should be expected to contain cobbles, boulders and rock fragments and appropriate equipment and procedures may be required to penetrate such obstructions 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

SUBGRADE PROTECTION FOR SUBEXCAVATION AREA

Operational Constraint

In order to limit disturbance to the clayey subgrade soils that will be exposed within the embankment footprints at the Ridgeway Drive/Highway 403 grade separation site, following stripping of the surficial topsoil deposit:

- The Contractor shall minimize travel over the clayey subgrade soils.
- The embankment fill shall be placed as soon as possible following the stripping of the topsoil deposit.

APPENDIX B

TECHNICAL MEMORANDUM
PAVEMENT INVESTIGATION AND DESIGN
MUNICIPAL CLASS ENVIRONMENTAL ASSESSEMENT STUDY
RIDGEWAY DRIVE/HIGHWAY 403 GRADE SEPARATION
MISSISSAUGA, ONTARIO