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REPORT ON

**DETAIL DESIGN
FOUNDATION INVESTIGATION AND DESIGN
RHCE E-S RAMP OVER NSR-W RAMP AND E-NSR RAMP
BRIDGE 11
RED HILL CREEK EXPRESSWAY INTERCHANGE
G.W.P. 441-97-00
MINISTRY OF TRANSPORTATION, ONTARIO
HAMILTON, ONTARIO**

Submitted to:

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PART A

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1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by McCormick Rankin Corporation (MRC) on behalf of the Ministry of Transportation, Ontario (MTO) to carry out a foundation investigation as part of the detailed design for the Queen Elizabeth Way / Red Hill Creek Expressway Interchange in Hamilton, Ontario.

The terms of reference for the scope of work are outlined in Golder's proposal P21-1334, dated November 2002, that forms part of the Consultant's Agreement (Number P.O.2005-A-000482) for this project. This report addresses Bridge 11 along the Red Hill Creek Expressway E-S Ramp and the Queen Elizabeth Way Ramps as part of the interchange project. The work was carried out in accordance with the Quality Control Plan for this project dated 2003. A digital file of the General Arrangement was provided to Golder by MRC in June 2004.

The investigation was supplemented with information contained in the following reports:

- Preliminary Foundation Investigation Report No. 981-1108, Queen Elizabeth Way / Red Hill Creek Expressway Interchange, Stoney Creek, Ontario, dated April 1998;
- Foundation Investigation and Design, Embankments, Queen Elizabeth Way / Red Hill Creek Expressway and Burlington Street Interchanges, Agreement No. 9820-7411-2805, Hamilton, Ontario, dated January 1999.

2.0 SITE DESCRIPTION

The site is located in the vicinity of the existing interchange between the Queen Elizabeth Way (QEW) and Centennial Parkway (Highway 20). The south shore of Lake Ontario is less than 1 km north of the site (see key plan on Drawing 1). The existing QEW lies within the limits of the proposed bridge alignment.

The terrain in this ramp area is generally flat-lying and a swampy area exists to the west of the bridge site. The existing ramp grades at the bridge site are at about Elevation 78 m in the area of the proposed works; however, the ground surface to the east of the site rises to about Elevation 81 m. Minor undulations across the site mainly involve fill embankments at the existing interchange, the right-of-way of the QEW, as well as regrading and landscaping on adjacent lands.

3.0 INVESTIGATION PROCEDURES

3.1 Foundation Investigation

The field work at the Bridge 11 site was carried out between August 25 and August 27, 2003 at which time two (2) boreholes, numbered BR11-1 and BR11-2 were advanced. Boreholes, RESR10, 7 and 8 were advanced at the site as part of the investigations carried out by Golder in 1998. All of these boreholes are shown on Drawing 1.

The current field investigation was carried out 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. Soil samples were obtained at intervals ranging from 0.75 m to 1.5 m in depth, using a 50 mm outer diameter (O.D.) split-spoon sampler in accordance with Standard Penetration Test (SPT) procedures. In-situ vane testing (N vanes) was carried out at regular intervals of depth through the soft stratum. Shelby tubes were obtained within the soft stratum by advancing a second borehole adjacent to the first borehole and drilling to the required depth. Samples of the bedrock were obtained using an 'NQ' size rock core barrel.

The boreholes were advanced to depths ranging from 9.6 m to 22.0 m below the existing ground surface (including rock coring). All of the boreholes were advanced to refusal within the till deposits or on the bedrock and one of the boreholes was extended into the bedrock by coring. The groundwater conditions in the open boreholes were observed during the drilling operations and piezometers were installed in selected boreholes to permit monitoring of the groundwater level at these locations. The piezometers consist of a 25 mm outside diameter rigid PVC tubing with a 0.3 m long slotted tip that is sealed at a selected depth within the boreholes. The holes were backfilled with bentonite mixed with soil cuttings; typically one bag of bentonite was used per 3 m of hole backfilled. The installation details and water level readings are described on the Record of Borehole sheets that follow the text of this report.

The field work was supervised throughout by members of our engineering and technical staff, who located the boreholes, arranged for the clearance of underground service locations, supervised the drilling, sampling and in-situ testing operations, logged the boreholes, and examined and cared for the soil and rock samples. The samples were identified in the field, placed in appropriate containers, labelled and transported to our Mississauga geotechnical laboratory where the samples underwent further detailed visual examination and laboratory testing. All of the laboratory tests were carried out to MTO and/or ASTM Standards as appropriate. Classification testing (water content, Atterberg Limits and grain size distribution) as

well as specialized oedometer testing was carried out on selected samples. Point load testing was carried out on samples of the rock core.

The boreholes were laid out in the field by J.D. Barnes Surveying Ltd. using the NAD 83 MTM (Zone 12) co-ordinate system and the geodetic datum for elevation. Where the boreholes were shifted at the time of drilling, the northings, eastings and elevations of the as-drilled boreholes were measured in the field relative to the staked locations by members of our engineering staff.

The Record of Borehole logs for the boreholes from the 1998 investigations have been modified from their original format using the current accepted MTO logging program. In addition, some of the strata descriptions have been updated accordingly and based on the results of the recent investigation.

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

The QEW in this area follows the shoreline of Lake Ontario and lies mainly in the Iroquois Plain physiographic region. The Iroquois Plain is generally composed of shallow sandy materials deposited on the bed of the glacial Lake Iroquois. The area is also referred to as the Niagara Fruit Belt (Chapman and Putnam, "The Physiography of Southern Ontario", 3rd Edition, 1984). The bedrock in the area of the site is shale of the Queenston Formation, the bedrock is typically at depths of 10 m or deeper below ground surface. There are infilled bedrock valleys known to exist in the general area; in particular at the Burlington Skyway.

The overburden at the site consists predominantly of two main till sheets laid during two distinct glacial events; the Halton till and the Wentworth till. The Wentworth till is predominantly sandy silt till and is the lower till sheet at the site. The Halton till is present over the lower till and is predominantly clayey silt to silty clay with low plasticity.

4.2 Subsoil Conditions

The detailed subsurface soil, bedrock and groundwater conditions as encountered in the boreholes advanced during this investigation, together with the results of the laboratory tests carried out on selected soil samples, are given on the attached Record of Borehole sheets and in Appendix A following the text of this report. 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. Further, subsurface conditions will vary between and beyond the borehole locations. The inferred soil stratigraphy based on the results of the boreholes at the bridge location are shown on Drawing 1.

In general, the subsoils at the site consist of a thin layer of topsoil underlain by a surficial deposit of sandy silt to clayey silt fill or by surficial deposits of sandy silt to clayey silt. The surficial deposits are in turn underlain by a thick deposit of clayey silt till. At the east side of the proposed bridge, the upper portion of the clayey silt till deposit is typically firm to very stiff. Elsewhere, the deposit is very stiff to hard. Reddish brown clayey silt till containing shale fragments was encountered below the upper clayey silt till deposit in one borehole. The clayey tills are underlain by a sandy silt till deposit in turn underlain by shale bedrock of the Queenston Formation. The total overburden thickness is up to 18.3 m as encountered in the deepest borehole at the site where bedrock was proven by coring. A more detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

4.2.1 Topsoil

Topsoil was encountered at the existing ground surface in all boreholes. The surface of the topsoil ranged between Elevation 77.8 m to 81.1 m and ranged from 0.1 m to 0.3 m thick.

A Standard Penetration Testing (SPT) measured 'N' value within the topsoil was 8 blows per 0.3 m of penetration, indicating a loose relative density.

4.2.2 Fill

Fill materials were encountered below the topsoil at the west end of the site in Borehole BR11-1, RESR10 and 7. The fill ranges in composition from clayey silt to silty clay and sandy silt to silty sand and extends to between 1.1 m and 2.8 m depth below ground surface. Trace organics and rootlets were noted within the fill.

Measured Standard Penetration Testing (SPT) measured 'N' values within the non-cohesive fill ranged between 14 and 25 blows per 0.3 m of penetration, indicating a compact relative density. Measured SPT 'N' values within the cohesive fill ranged between 7 and 13 blows per 0.3 m of penetration, indicating a firm to stiff consistency.

The natural water content measured on samples of the fill range from 17 to 29 percent.

4.2.3 Sandy Silt to Silty Sand

A surficial deposit of brown sandy silt to silty sand containing trace to some clay and trace gravel was encountered below the topsoil in Borehole BR11-2, located in the vicinity of the east approach area. Trace organics were noted in the deposit in Borehole BR11-2. This deposit was approximately 1.3 metres thick, and the surface was encountered at Elevation 79.9 m.

One measured Standard Penetration Testing (SPT) 'N' value was 15 blows per 0.3 m of penetration, indicating a compact relative density.

The natural water content measured on one sample of this deposit was 10 percent.

4.2.4 Silty Clay

A surficial deposit of silty clay was encountered in all boreholes across the site immediately below the topsoil or fill in Boreholes BR11-1, 7 and 8 and below the surficial sandy silt to silty sand deposit in Borehole BR11-2. The mottled brown and grey silty clay deposit contained trace to some sand and trace gravel. Occasional sandy silt seams were noted in Boreholes BR11-1 and

BR11-2. Trace organics were noted within the deposit in Borehole 8. This surficial silty clay deposit ranged from 2.0 to 4.2 metres thick, being thickest near the west abutment. The surface of the deposit was encountered between Elevations 76.4 and 79.6 m.

At the borehole locations, Standard Penetration Testing (SPT) measured 'N' values ranged between 5 and 36 blows per 0.3 m of penetration, indicating a firm to hard consistency. A grain size distribution curve for one sample of the silty clay is shown on Figure A1 in Appendix A.

Atterberg limits testing was carried out on two samples of surficial silty clay deposit. The results of the Atterberg limits tests are plotted on the plasticity chart on Figure A2 Appendix A. The test results indicate that the deposit is a silty clay of intermediate elasticity. The test results are summarized in the following table.

<i>Borehole</i>	<i>Sample</i>	<i>Elevation (m)</i>	<i>Liquid Limit (%)</i>	<i>Plastic Limit (%)</i>	<i>Plasticity Index (%)</i>
BR11-1	5	75.1 – 75.5	38	18	20
7	5	74.3 – 74.8	45	27	18

It should be noted that the test result from Borehole 7, sample 5 plots below the 'A line' on the plasticity chart suggesting the presence of organic material; however, the test result is suspect.

The natural water content measured on samples of the silty clay range from 22 to 27 percent.

4.2.5 Clayey Silt Till

A deposit of clayey silt till was encountered underlying the surficial deposits in all boreholes. The clayey silt till contains trace to some sand and gravel. The clayey silt was typically grey in color becoming reddish-grey to reddish-brown near the base of the deposit. This till deposit is considered to be the 'Halton' till sheet. The top of the clayey silt till deposit was encountered between Elevations 72.2 and 77.6 m in the boreholes. This deposit was not fully penetrated in all the boreholes except Borehole BR11-1, where it was 10 m in thickness.

The measured SPT 'N' values within the clayey silt till deposit ranged between 2 and greater than 100 blows per 0.3 m of penetration. A grain size distribution curve for a selected sample from this deposit is shown on Figure A3 in Appendix A. The lowest 'N' values were measured in Boreholes BR11-2 and 8, located at the east end of the site and the results of in situ field vane tests in these two boreholes indicate that the upper 2.5 to 4.0 m of the deposit in this area has a soft to firm consistency. Elsewhere, the results of the vane testing indicates that the deposit has a firm to hard consistency. The results of the vane testing are summarized in the following table:

<i>Borehole</i>	<i>Location</i>	<i>Sample Depth/Elevation (m)</i>	<i>Undisturbed Shear Strength(kPa)</i>	<i>Remoulded Shear Strength (kPa)</i>	<i>Sensitivity</i>
BR11-1	Center pier	5.2 / 73.4	>100	-	-
8	East abutment	4.0 / 74.0	39	20	1.9
		5.5 / 72.5	39	22	1.8
BR11-2	East approach	3.7 / 77.4	76.7	33.5	2.3
		4.4 / 76.7	19.2	5.7	3.3
		4.7 / 76.4	21.1	9.6	2.2
		6.7 / 74.4	40.2	24.9	2.9
		7.0 / 74.1	46	30.6	1.5

Atterberg limits testing was carried out on three samples of the clayey silt till deposit. The results of the Atterberg limits tests are plotted on the plasticity chart on Figure A4 Appendix A. The test results are summarized in the following table. The test results indicate that the deposit is classified as a clayey silt of low plasticity; however the test result from Borehole BR11-2, sample 8 indicates a silty clay of intermediate plasticity.

<i>Borehole</i>	<i>Sample</i>	<i>Elevation (m)</i>	<i>Liquid Limit (%)</i>	<i>Plastic Limit (%)</i>	<i>Plasticity Index (%)</i>
BR11-1	11	67.5 – 67.9	32	16	16
BR11-2	7	75.8 – 76.2	24	13	11
BR11-2	8	74.5 – 75.0	36	18	18
Average	-	-	31	16	15

The natural water content measured on selected samples of the till deposit ranged between 9 percent and 34 percent, with an average of about 20 percent.

Laboratory oedometer (consolidation) testing was carried out on one specimen of the upper soft to firm portion of the clayey silt till obtained from Borehole BR11-2. Details of the test results are shown on Figures A5 and A6 in Appendix A and the results summarized in the table below:

<i>Borehole and Sample No.</i>	<i>Elevation (m)</i>	<i>σ_{vo}' (kPa)</i>	<i>σ_p' (kPa)</i>	<i>OCR</i>	<i>e_o</i>	<i>C_r</i>	<i>C_c</i>	<i>c_v^* (cm²/s)</i>
BR11-2	76.2 – 75.6	104	140	1.3	0.91	0.058	0.325	0.005

Note: * For stress range of $20 \leq \sigma_v' \leq 300$ kPa

where: σ_{vo}' is the effective overburden pressure in kPa
 σ_p' is the preconsolidation pressure in kPa
OCR is overconsolidation ratio
 e_o is initial void ratio
 C_c is the compression index (based on void ratio)
 C_r is the recompression index
 c_v is the coefficient of consolidation in cm²/s

The bottom 2.1 m of the till deposit in Borehole 8 is described as reddish-brown clayey silt containing some sand and gravel and occasional weathered shale and limestone fragments were observed in the deposit.

One measured SPT 'N' value for this portion of the till deposit was greater than 100 blows per 0.3 m of penetration, indicating a hard consistency.

4.2.6 Sandy Silt Till

In Borehole BR11-1, where the clayey silt till was penetrated, a deposit of reddish-brown sandy silt till, was encountered below the reddish-grey to reddish-brown clayey silt till. The deposit contains some clay and trace gravel along with shale and limestone fragments. This deposit is considered to be the 'Wentworth' till sheet. The surface of the sandy silt deposit was encountered at Elevation 64.0 m and the stratum was 3.7 m in thickness.

The measured SPT 'N' values for samples of this sandy silt till deposit were greater than 100 blows per 0.3 m of penetration, indicating a very dense state of packing. A grain size distribution curve for one sample of this deposit is shown on Figure A7 in Appendix A.

The natural water content measured on one selected sample of this sandy silt deposit was approximately 10 percent.

4.2.7 Bedrock

Bedrock was encountered in Borehole BR11-1, where the overburden was fully penetrated. The bedrock surface was encountered at a depth of 18.3 m below ground surface at Elevation 60.3 m.

The bedrock samples obtained consist of reddish-grey, moderately to highly weathered, thinly layered, very fine grained, calcareous shale of the Queenston Formation. Thin layers of the fresh

to slightly weathered limestone were present within the shale bedrock. The Total Core Recovery was between 70 percent and 100 percent. The Rock Quality Designation (RQD) measured on the core samples in Borehole BR11-1 ranged from about 43 to 71 percent, with the lower values encountered near the surface of the bedrock. This indicates a rock mass of poor to fair quality.

Point load strength tests were performed on selected samples of the rock core from Borehole BR11-1. Diametral point load strength index values are shown on the Record of Drillhole Sheet. Diametral point load index values on samples of the shale range from 0 to 1.03 MPa which corresponds to an estimated unconfined compressive strength (UCS) ranging from 0 to 23 MPa. The axial point load index values on the samples of the shale range from 0.38 to 0.81 MPa corresponding to approximate UCS values of 9 and 18 MPa, respectively. Using the Intact Rock Strength Classification table, these values indicate that the shale is classified very weak to weak. In the limestone, diametral and axial point load index values of 1.09 and 1.77 MPa corresponding to an approximate UCS values of 24 and 40 MPa, respectively. This indicates that the limestone is classified as medium strong; however, the limestone typically has a range of strength from strong to very strong.

4.2.8 Groundwater Conditions

The water levels were noted during and after the drilling and coring operations in the boreholes. Typically, the open boreholes were dry upon completion of drilling. Piezometers were installed in Boreholes BR11-1, RESR10 and 7. The piezometer in Borehole BR11-1 was sealed in the sandy silt till deposit and the piezometers in Boreholes RESR10 and 7 were sealed within the clayey silt till deposit. Details of the piezometer installations are shown in the Record of Borehole Sheets following the text of this report. The water levels in the piezometers are summarized in the table below:

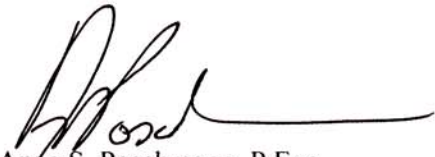
<i>Borehole</i>	<i>Ground Surface Elevation (m)</i>	<i>Ground Water Level Depth (m)</i>	<i>Ground Water Level Elevation (m)</i>	<i>Date</i>
BR11-1	78.6	2.8	75.8	October 22, 2003
RESR10	78.1	3.0	75.1	November 10, 1998
7	77.6	3.2	74.4	March 25, 1998

The groundwater level at the borehole locations is typically at the surface of the clayey silt till. The groundwater table is likely controlled by the water level in Lake Ontario and is expected to slope slightly downwards towards the lake. It should be noted that groundwater levels in the area are subject to seasonal fluctuations.

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SEP/ASP/FJH/sm

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5.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides foundation design recommendations for the proposed Bridge 11 as part of the QEW/Red Hill Creek Expressway Interchange. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during three phases of 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 carry the Red Hill Creek Expressway E-S Ramp over the existing NSR-W and E-NSR ramps (North Service Road). The proposed bridge has two spans which are 26.5 m and 25.5 m at the west and east abutments, respectively. The proposed embankments will be up to between 7.5 and 9.0 m in height. The revised General Arrangement drawing for Bridge 11 was provided by MRC in electronic format in June 2004 and incorporated into Drawing 1.

5.1 General

Various alternatives for the abutment foundations were considered and a summary of these alternatives is presented in Table 1, following the text of this report. Shallow foundations are not recommended for support of the bridge due to the variable surficial deposits and the anticipated settlements as a result of the presence of the firm till deposit. It is considered that steel H-piles driven to found within the very dense sandy silt till for support of the pier and abutments is the most feasible option from a geotechnical / foundation perspective. The wing walls could be supported on spread footings within the embankment fill or on steel H-piles driven to refusal into the very dense sandy silt till.

5.2 Shallow Foundations

If consideration is being given to founding the concrete wing walls on the embankment fill, it is recommended that the embankment fill below the wall footings consist of Granular A or Granular B Type II placed in regular lifts not greater than 200 mm in loose thickness and compacted to at least 95 percent of the materials Standard Proctor maximum dry density. Consolidation of the native founding soils below the wall footings/granular will occur causing settlement of the wall and alternatives for mitigation of this settlement will be addressed in Section 5.6.4.

If spread footings are chosen for founding of the wing walls, the approach embankment area will have to be preloaded and there may be a differential settlement of up to 25 mm between the wing wall and the abutment. In this regard, an articulation joint between the wall and the abutment should be provided to take into account this settlement.

5.2.1 Geotechnical Resistance

For spread footings founded on a compacted Granular 'B' pad, a factored axial geotechnical resistance of 300 kPa at Ultimate Limit States (ULS) may be used for design. An axial geotechnical resistance of 200 kPa at Serviceability Limit States (SLS) for 25 mm of settlement may be used for design. These values assume a footing width of 1 m.

5.2.2 Resistance to Lateral Loads

The resistance to lateral forces / sliding resistance between the compacted granular fill and the subgrade should be calculated in accordance with Section 6.7.5 of the CHBDC. The coefficient of friction, $\tan \phi'$, between the concrete and the compacted Granular 'B' may be taken as 0.55. This represents an unfactored value; in accordance with the CHBDC, a factor of 0.8 is to be applied in calculating the horizontal resistance.

5.2.3 Frost Protection

The wing wall footings should be provided with a minimum of 1.2 m of soil cover for frost protection.

5.3 Steel H-Pile Foundations

Steel H-piles driven to found in the hard clayey silt till or very dense sandy silt till (where SPT 'N' values are greater than 100 blows per 0.3 m of penetration) may be used for support of the abutments and/or wing walls. Alternatively, steel H-piles could be driven to the shale bedrock at the site which was proven at the centre pier borehole location. The thickness of the hard/very dense till deposits overlying the bedrock are expected to be 6 m or less; however, it is likely that practical refusal for the piles may be met within the hard till deposits prior to reaching the bedrock surface.

It is assumed that the abutment pile caps will be perched within the embankment. For design, the following pile tip levels may be assumed for piles terminated within the hard/very dense till (assumed minimum 2 m penetration) or just into the bedrock. There should be provision made in the contract for dealing with varying pile lengths.

<i>Foundation Location</i>	<i>Relevant Boreholes</i>	<i>Design Pile Tip Elevation (m)</i>	
		Very Dense/Hard Till	Bedrock
East Abutment	8	64.0	59.5
Central Pier	BR11-1	62.0	
West Abutment	7	64.0	

5.3.1 Axial Geotechnical Resistance

For HP 310 x 110 piles driven to found within into the hard clayey silt / very dense sandy silt till deposits at or below the above elevations, a factored axial geotechnical resistance at Ultimate Limit States (ULS) of 1,400 kN may be assumed for design. The axial geotechnical resistance at Serviceability Limit States (SLS) for 25 mm of settlement will be at about the factored axial resistance at ULS.

The piles should be stiffened with MTO flange plates for protection during driving in accordance with OPSD 3301.00 and OPSS 903.07.05.04. 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. For piles driven into the hard / very dense till, the following note is considered appropriate for the design and site conditions assuming a resistance factor of 0.4 is applied to the used of the Hiley:

- “Piles to be driven in accordance with Standard SS 103-11 using an ultimate capacity of 3,500 kN per pile but must be driven below EL 62 m at the pier and below EL 64 at the abutments.”

If additional pile capacity is required, consideration could be given to HP 310 x 110 piles driven to found on/in the shale bedrock at the site, a factored axial resistance at Ultimate Limit States (ULS) of 2,000 kN may be assumed for design. The axial geotechnical resistance at Serviceability Limit States (SLS) may be taken as 1,600 kN. It should be noted that pre-augering to the pile tip elevation would be required to ensure that the piles can be driven into the bedrock to the elevations given above. This full depth pre-augering is not considered to be practical for

the limited increase in pile capacity. For piles driven into the bedrock, the following note should be used for the drawings:

- “Piles to be driven to bedrock.”

If pre-augering is not to be included / specified in the contract, then the pile capacities given for piles terminating within the hard / very dense till should be used for design

5.3.2 Downdrag Load (Negative Skin Friction)

The embankment loading will cause consolidation settlement of the underlying softened clayey silt till deposit. The consolidation settlement is time-dependent and will not completely occur during the construction period. That is, post-construction settlement of the clayey silt deposit will take place. Negative skin friction or downdrag loads will need to be taken into account during design of the piles supporting the abutments as a consequence of settlement of the ground with respect to the pile. The abutment pile structural design should be based on the full downdrag load acting on the piles within and above the soft to firm till zone. The estimated unfactored downdrag load acting on the HP 310x110 piles may be taken as 100 kN per pile at the east abutment location only. Downdrag loads can be ignored at the west abutment.

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 *CHBDC* for ULS conditions. Downdrag loads could be reduced by preloading of the abutment area as discussed in Section 5.6.4.

5.3.3 Resistance to Lateral Loads

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

The evaluation of the existing piles subjected to lateral loads should take into account such factors as the relative rigidity of the pile to the surrounding soil, the fixity condition at the head of the pile (pile cap level), the structural capacity of the pile to withstand bending moment, the soil resistance that can be mobilized, the tolerable lateral deflection at the head of the pile and the pile group effects.

The resistance to lateral loading in front of the pile within the clayey silt till may be calculated using subgrade reaction theory where the coefficient of horizontal subgrade reaction, k_h , is based on the following equation:

$k_h = \frac{67s_u}{B}$	where	k_h is the coefficient of horizontal subgrade reaction (kPa/m); s_u is the undrained shear strength of the soil (kPa), as given below; and B is the pile diameter (m).
-------------------------	-------	--

The following range for the value of s_u may be assumed in the structural analysis:

<i>Location</i>	<i>Soil Unit</i>	<i>Elevation</i>	<i>s_u (kPa)</i>
East Abutment	Firm to very stiff clayey silt	Ground surface to Elev. 75.5 m	100
	Firm clayey silt till	Between Elev. 75.5 and 72.5 m	40
	Stiff to hard clayey silt till	Below Elev. 72.5 m	100 to 200
Central Pier	Firm to stiff clayey silt	Ground surface to Elev. 74.0 m	50 to 80
	Firm clayey silt till	Between Elev. 74.0 and 73.0 m	40
	Stiff to hard clayey silt till	Below Elev. 73.0 m	80 to 100
West Abutment	Very stiff to hard clayey silt/ clayey silt till	Below Elev. 76.0 m	100 to 200

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

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

5.3.4 Frost Protection

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

5.4 Caissons

Consideration could be given to the use of caissons socketted into the hard clayey silt till / very dense sandy silt till deposits or into the shale bedrock for support of the bridge. It should be noted that although the sandy silt till overlying the bedrock is relatively thin (less than 4 m), this

deposit is known to contain numerous cobbles and boulders which may pose difficulties in advancing the caissons / temporary liners through to the bedrock surface. It should also be noted that the sandy silt till and bedrock surface were confirmed only at the pier borehole location. If consideration is being given to the use of caissons socketted into the bedrock at the abutments, it is recommended that further borehole investigation at the abutments be carried out to obtain sufficient information for construction. For design purposes, the following design base elevations may be used at the bridge abutments for caissons founded on the surface or just into the sandy silt till and for caissons socketted at least 2 m into the bedrock:

<i>Foundation Location</i>	<i>Design Caisson Founding Elevation (m)</i>	
	Very Dense Till	Bedrock
East Abutment	63.5	58.0
Central Pier	63.5	
West Abutment	65.5	

The caisson excavations must be inspected by qualified geotechnical personnel to ensure that the founding stratum has been reached and is consistent with the design assumptions and that the base has been properly cleaned and is dry. In this regard, temporary liners will be required to permit downhole inspection.

5.4.1 Axial Geotechnical Resistance

The caissons will derive their axial resistance in part from end-bearing and in part from shaft friction. For this site, the majority of the resistance will be derived from base resistance. It is also assumed that there would be only nominal socketting (less than 1 m) into the sandy silt till. For caissons in bedrock, the caissons should be socketted at least 2 m into the bedrock. For these assumptions, and assuming that all caisson excavations are inspected prior to pouring concrete, the factored axial geotechnical resistance at ULS and axial geotechnical resistance at SLS that may be used for design are given in the table below:

<i>Caisson Diameter(m)</i>	<i>Axial Resistance</i>			
	<i>Hard/Very Dense Till</i>		<i>Bedrock</i>	
	ULS	SLS	ULS	SLS
0.9	3,200 kN	2,800 kN	4,000 kN	n/a
1.5	6,600 kN	4,500 kN	8,000 kN	n/a

For caissons founded in the shale bedrock, the resistance required to achieve 25 mm of settlement is greater than that given for ULS and therefore SLS conditions do not apply.

5.4.2 Downdrag Load (Negative Skin Friction)

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

<i>Caisson Diameter (m)</i>	<i>Unfactored Downdrag Load (kN)</i>
0.9	250
1.5	400

Other requirements for structural design with respect to downdrag load on the caissons should be in accordance with Section 5.3.2.

5.4.3 Resistance to Lateral Loads

The resistance to lateral loading for the caissons should be in accordance with Section 5.3.3.

5.4.4 Frost Protection

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

5.5 Lateral Earth Pressures for Design

The lateral earth pressures acting on the abutment stems and any associated wing walls / retaining walls will depend on the type and method of placement of the backfill materials, on the nature of the soils behind the backfill, on the magnitude of surcharge including construction loadings, on the freedom of lateral movement of the structure, and on the drainage conditions behind the walls. For this site location, the geotechnical seismic considerations do not impact on the design since it is within the lowest seismic zone given in CHDBC.

The following recommendations are made concerning the design of the walls. It should be noted that these design recommendations and parameters assume level backfill and ground surface

behind the walls. Where there is sloping ground behind the walls, the coefficient of lateral earth pressure must be adjusted to account for the slope.

- Select free-draining granular fill meeting the specifications of Ontario Provincial Standard Specifications (OPSS) Granular 'A' or Granular 'B' should be used as backfill behind the walls. This 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 3501.00 and 3504.00.
- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the wall stem, in accordance with *CHBDC* Section 6.9.3 and Figure 6.9.3. Compaction equipment should be used in accordance with 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(I) of the *Commentary to the CHBDC*) or within the wedge-shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical (1.5H:1V) extending up and back from the rear face of the footing (Case II in Figure C6.9.1(I) of the *Commentary to the CHBDC*).
- For Case I, the pressures are based on the 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	0.33
At rest, K_o	0.50

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

	GRANULAR 'A'	GRANULAR 'B' TYPE II
Soil unit weight:	22 kN/m ³	21 kN/m ³
Coefficients of static lateral earth pressure:		
Active, K_a	0.27	0.31
At rest, K_o	0.43	0.47

- 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.

5.6 Approach Embankment Design and Construction

The proposed grade of the Expressway varies from about Elevation 86 m to 87 m at the east and west approaches, respectively. The existing ground surface at the bridge varies from about Elevation 77.5 m to 78.0 m resulting in approach embankments between 7.5 m and 9 m in height. It should be noted that to the east of the site, the elevation of the ground surface rises to about Elevation 81 m, reducing the overall embankment height in this area. The design of the embankments beyond the limits of the bridge approaches will be addressed in a separate report.

5.6.1 Subgrade Preparation and Embankment Construction

Where the embankments are greater than 8 m in height, a mid-height, 2 m wide berm is required in accordance with MTO guidelines for surficial stability.

It is our understanding that it is not normal practice to carry out topsoil stripping from below embankments which are greater than 1.2 m in height. At this site, however, given the variability of the surficial soils, it is recommended that all topsoil, organic matter and softened / loosened soils are stripped from below the approach embankment areas. For quantity estimation purposes at this site, a topsoil thickness of 0.3 m should be assumed.

All subgrade soils should be proof-rolled prior to fill placement in accordance with OPSS 206. Embankment 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 and base courses 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.

To reduce surface water erosion on the embankment side slopes, placement of topsoil and seeding or pegged sod is recommended.

5.6.2 Approach Embankment Stability

Limit equilibrium slope stability analyses were performed using the commercially available program SLOPE/W, produced by Geo-Slope International Ltd., employing the Morgenstern-Price method of analysis, to check that a minimum factor of safety of 1.3 is achieved for the proposed approach embankment height and geometry under static conditions. This minimum factor of safety is considered appropriate for the embankments at this site considering the design requirements and the available field and laboratory testing data.

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

<i>Soil Deposit</i>	<i>Bulk Unit Weight</i>	<i>Effective Friction Angle</i>	<i>Undrained Shear Strength</i>
Embankment Fill	21 kN/m ³	32°	—
Surficial Clayey Silt	20 kN/m ³	—	75 kPa
Firm Clayey Silt Till (East Approach)	19 kN/m ³	30°	40 kPa
Stiff to Hard Clayey Silt Till	20 kN/m ³	30°	100 kPa
Sandy Silt Till	21 kN/m ³	32°	—

The undrained shear strength data from in situ vane and consolidation tests for all the boreholes drilled to date at the Red Hill Creek Interchange are shown on Figure 1. It should be noted that undrained shear strengths as low as 20 kPa were measured in the softened clayey silt till in Borehole BR11-2, located about 30 m east of the east abutment where the embankment height is less than 4 m. As such, the undrained shear strength data from BR11-2 was not used in the above analysis for assessing the stability of the proposed east abutment.

The analyses indicate that a factor of safety of greater than 1.3 against deep-seated slope instability is obtained for the proposed 7.5 m to 9 m high approach embankments with side slopes at 2 horizontal to 1 vertical (2H:1V). The results of the analysis at the east abutment are shown in Figure 2. In this regard, toe berms for stability will not be required for the proposed embankment heights.

It should be noted that while the overall embankment height decreases with increasing grade to the east of the east abutment, the undrained shear strength of the soil also decreases. Comments with regards to the embankments east of the site will be addressed under separate cover.

5.6.3 Approach Embankment Settlement

Settlement of the approach embankment subgrade can be expected mainly due to consolidation of the surficial clayey silt and the firm clayey silt till deposit encountered at the central pier and east abutment and approach embankment. Settlement analyses were carried out in part using the commercially available computer program Unisettle 3.0.

Provided that the embankment fill material consists of earth fill or granular fill, the settlement of the new embankment fill itself is expected to be less than 25 mm. If granular fill is used, the majority of settlement will occur during construction.

The settlement of the embankments as a consequence of consolidation settlement of the clayey silt till deposit underlying the earth fill embankments is calculated to be about 75 mm at the east abutment due to the 7.5 m high embankment. At the west abutment, where no softened clay was found in Boreholes 7 and RESR-10, the settlement as a result of the loading from the 9 m high embankment would be less than 35 mm.

5.6.4 Mitigation of Stability Issues / Time Dependent Settlement

Time dependent settlements of the new embankments are expected as a result of consolidation of the underlying clayey silt till. In these areas, consideration could be given to preloading of the embankment, placement of a surcharge, wick drains or sub-excavation of the soft to firm clayey silt till in order to limit the post-construction settlements and subsequent maintenance on the new roadway pavement structure.

5.6.4.1 Preloading

The most feasible option is to construct preloaded embankments the proposed embankment height with side slopes of 2H:1V. If it is desirable to use 1H:1V slopes for temporary preloading, the maximum preload embankment height will be reduced to 8.5 m at the west abutment; the full embankment height may be constructed at the east abutment using 1H:1V side slopes. The preload for the west approach embankment should be left in place for 6 months and for the east approach embankment for 12 months for 90 percent of the settlement to occur. At the east approach embankment, which is underlain by softened soil, consideration should be given to employing a 1.5 m surcharge in order to obtain sufficient settlement in 6 months.

Alternatively, consideration could be given to the use of EPS fill within a portion of the embankment in order to reduce the magnitude of post-construction settlement. For the east approach embankment constructed with 2.5 m of EPS beneath the pavement structure, a total

settlement of less than 35 mm will be induced by the remaining earth fill. Preloading of the embankment should still be carried out as discussed earlier. The EPS should be placed between the wing walls and extended away from the abutments. The EPS thickness should be tapered at a slope of 5H:1V under the road to minimize abrupt differential settlement. The EPS should be provided with a minimum of 1.0 m of cover and not more than 1.8 m.

If the approach embankment areas are preloaded (and surcharged for the east approach embankment), the total post-construction settlement beneath the wing walls will be reduced to below 25 mm at the east and west abutments.

In order to reduce the preload period at the east approach, consideration could be given to the use of wick drains at this site. The wick drains should be at 1 m spacing to be fully effective and 90 percent of the settlement would occur in 3 months.

If the preloading and preloading/surcharging option is chosen as the preferred alternative, there will be less than 25 mm of post-construction settlement after the preload period is complete. 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 5.3.2 and 5.4.2) will not have to be considered in the structural design of the piles.

5.6.4.2 Subexcavation

Alternatively, consideration could be given to sub-excavation of the 2.5 m of firm clayey silt till at the east approach embankment to eliminate the post-construction settlement and eliminate any requirement for EPS fill/surcharging/wick drains. The compressible material would have to be excavated to a minimum of Elevation 72.5 m which would require excavations up to 5.5 m deep. Subexcavation is not considered to be a feasible option at the west abutment since the settlements are anticipated be less than about 35 mm under the embankment loading.

The requirements with respect to maximum cut slope profiles and groundwater control should be in accordance with Section 5.6. Due to the depth of the required excavation at this site and the proximity to the existing road (possible roadway protection), sub-excavation is not considered to be an economical option.

If subexcavation is chosen as the preferred alternative for mitigating settlement and stability at the east abutment, the post-construction settlement will be eliminated and thus the downdrag loads do not have to be considered in the structural design of the piles as given in Sections 5.3.2 and 5.4.2.

5.7 Excavations and Temporary Cut Slopes

The abutment pile caps will be constructed within the approach embankments. Excavations for construction of the pier pile cap at the abutments will typically extend through fills and surficial clayey silt soils. It is anticipated that the bulk of the excavations at the site can be made in open cut. Excavations should be carried out in accordance with the guidelines outlined in the latest edition of the Occupational Health and Safety Act (OHSA) for Construction Activities. The surficial fills and the upper clayey silt soils at the site are classified as Type 3 soil, according to the OHSA. Temporary excavations (i.e. those which are only open for a relatively short period) through these overburden soils should be made with side slopes no steeper than 1 horizontal to 1 vertical (1H:1V).

The water levels at the site are typically between about Elevation 74.4 m and 75.8 m and are about 3 m below the ground surface. In general, the boreholes were typically dry upon completion of drilling and it is anticipated for the open-cut excavations at this site, the groundwater can be adequately controlled by sumping from properly filtered sumps.

Excavation support for roadway protection may be required at this site. Where required, the temporary excavation support system should be designed and constructed in accordance with MTO's Special Provision 539S01. The lateral movement of the temporary shoring system should meet Performance Level 2 as specified in SP 539S01.

GOLDER ASSOCIATES LTD.



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SEP/ASP/FJH/sd

N:\Active\2002\1100\021-1162 QEW-Red Hill Creek\Phase 5000 Reports\Red Hill Creek Interchange Bridge 11\Final\021-1162 RPT 04Jun Final Report Bridge 11.doc

TABLE

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TABLE 1
EVALUATION OF FOUNDATION ALTERNATIVES
BRIDGE 11 - RED HILL CREEK EXPRESSWAY INTERCHANGE

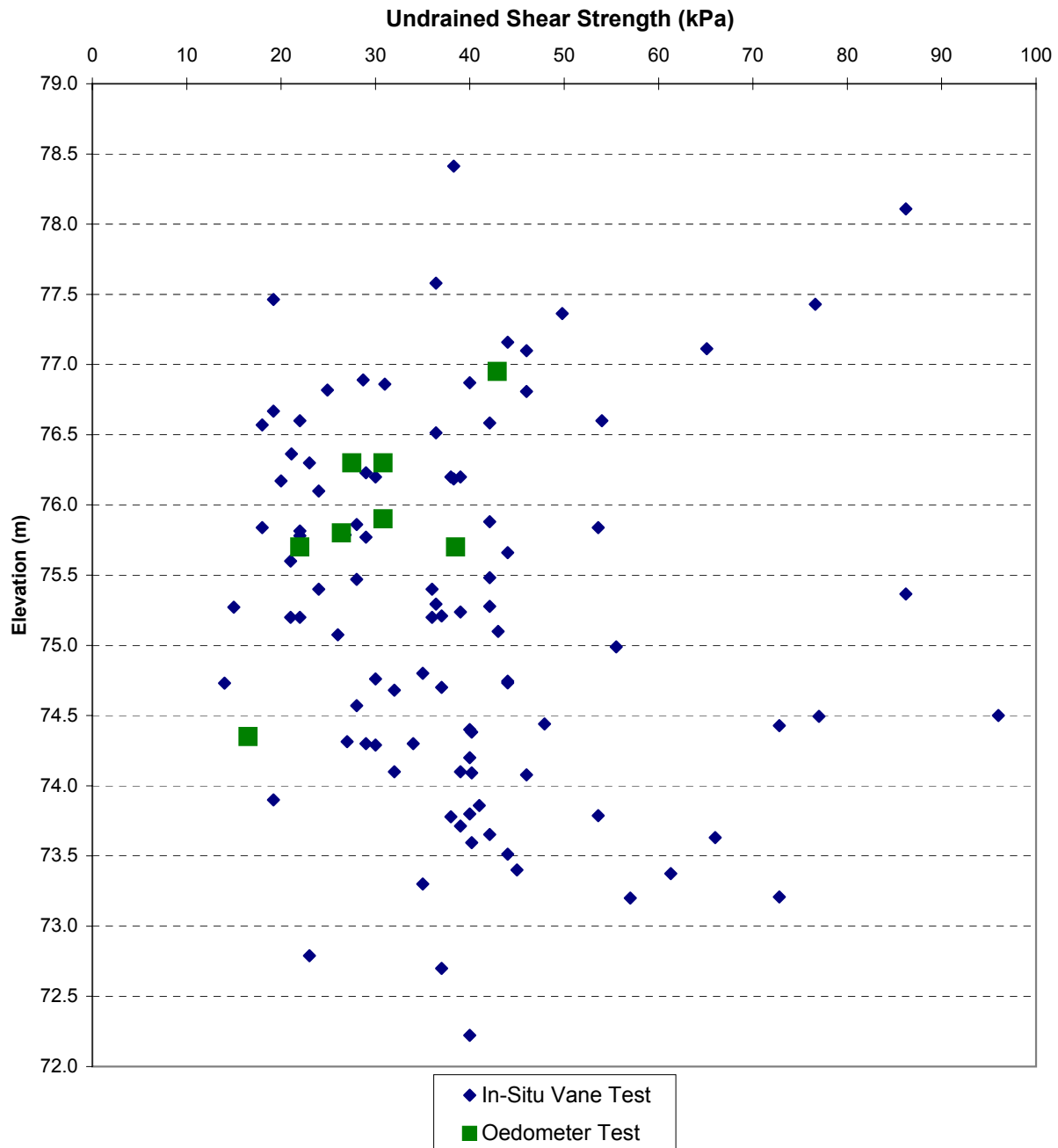
<i>Footing Option</i>	<i>NF</i>	<i>Advantages</i>	<i>Disadvantages</i>	<i>Relative Costs</i>	<i>Risks/Consequences</i>
Spread Footings	NF		Low geotechnical resistance. Potential differential settlement; consolidation of soft to firm clayey silt due to embankment loading. Groundwater control may be required.		Differential settlement between abutments.
Steel H Piles driven to practical refusal within hard clayey silt till/very dense sandy silt till		Minimize hard driving through hard/very dense bouldery till deposits.	Lower capacity than piles driven to found on bedrock.	Lower relative costs than piles driven to bedrock.	Low risk
Steel H Piles driven to shale bedrock		Increased capacity over piles terminated in overburden.	Longer pile lengths. Hard driving through till deposits; piles may "hang-up" on boulders or within hard/dense till deposits.	Relative costs of driving piles through bouldery deposit less than augering for caissons.	Low risk
Caissons socketted into very dense sandy silt till just below stiff to hard clayey silt till		Minimize caisson length and extent of difficult augering through bouldery till deposit.	Lower capacity than caissons socketted into bedrock. Temporary liners required for side support. Groundwater flow into excavation through sandy silt till could be encountered.		May be disturbance of base within sandy silt till; potential groundwater inflow loosening founding soils and requiring subexcavation
Caissons socketted into shale bedrock			Although till deposit is relatively thin, may encounter difficulties in advancing caissons through bouldery till. Temporary liners extended into bedrock required for groundwater control.	Increased cost of socketting into bedrock. Extra costs associated with liners and inspection.	Difficulty may be encountered in extending liner through till deposit to seal off groundwater inflow; downhole inspection may not be possible.

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

FIGURES

**SHEAR STRENGTH SUMMARY
RED HILL CREEK EXPRESSWAY INTERCHANGE**

FIGURE 1



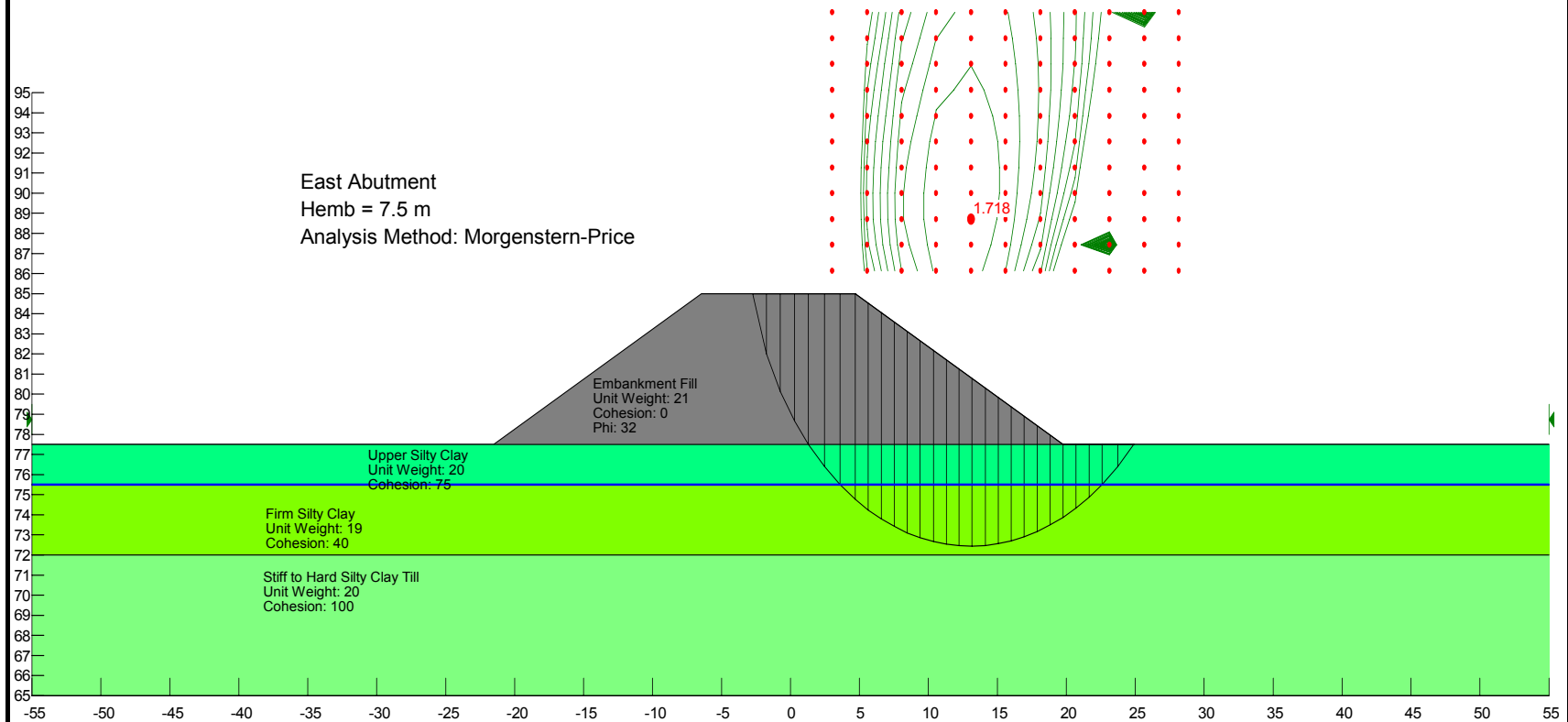
Date: June 2004
Project: 021-1162-BR11

Golder Associates

Drawn: SEP
Checked: JPD

**APPROACH EMBANKMENT STABILITY ANALYSIS
EARTH FILL OPTION**

FIGURE 2

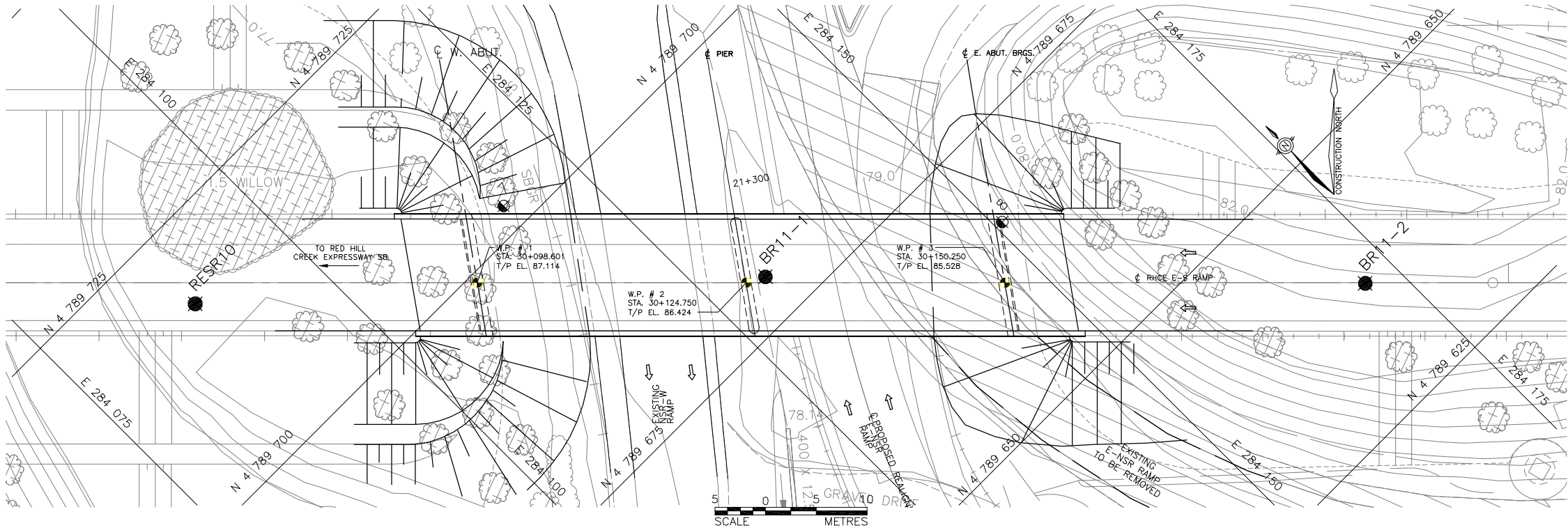


Date: June 2004
Project: 021-1162-BR11

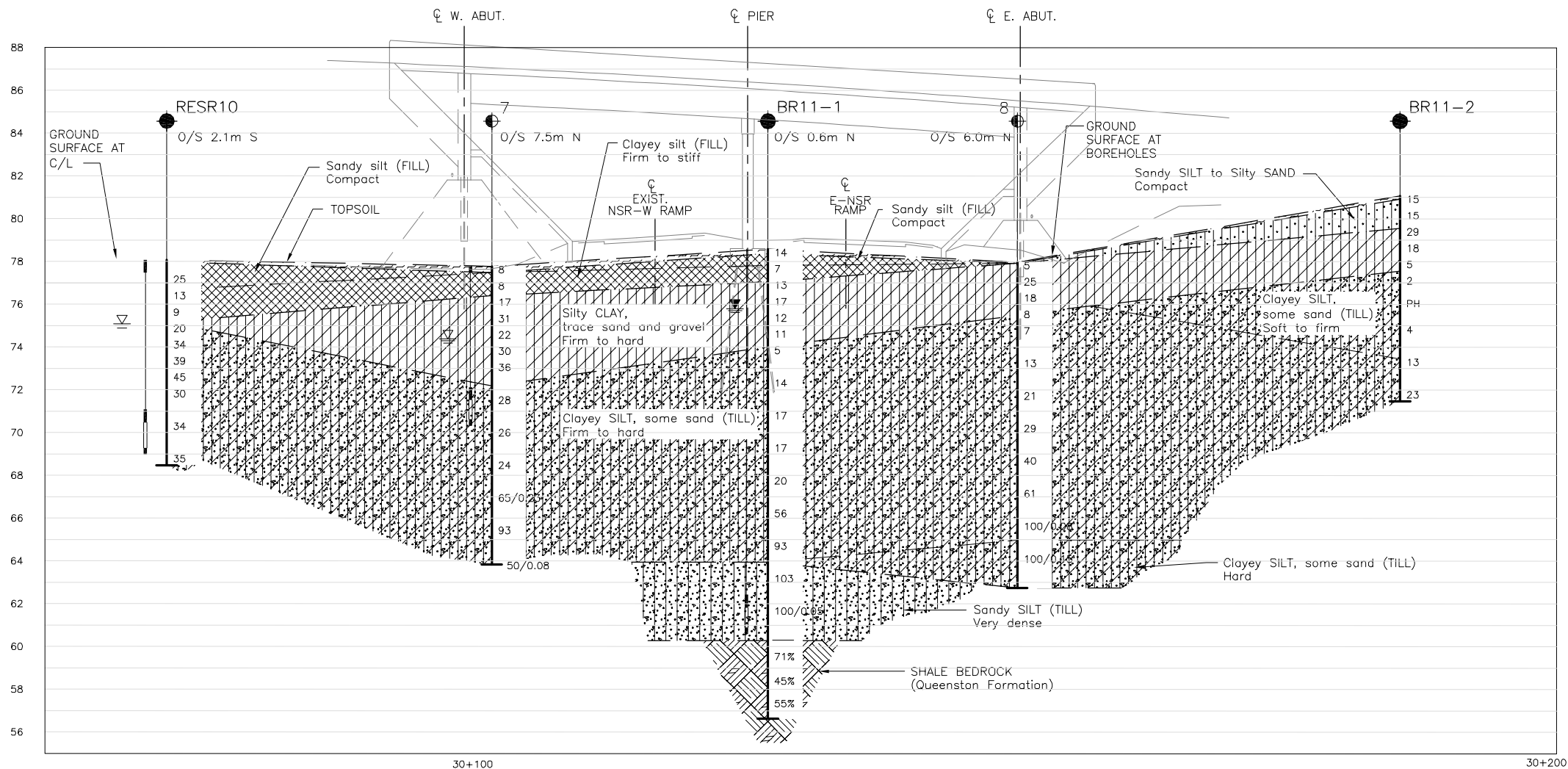
Golder Associates

Drawn: SEP
Checked: JPD

DRAWING



PLAN



SECTION/PROFILE

DIST.
CONT No.
WP No.

HWY. QEW

REDHILL CREEK EXPRESSWAY
E-S RAMP OVER NSR-W RAMP
AND E-NSR RAMP (BRIDGE 11)
BOREHOLE LOCATION AND SOIL STRATA

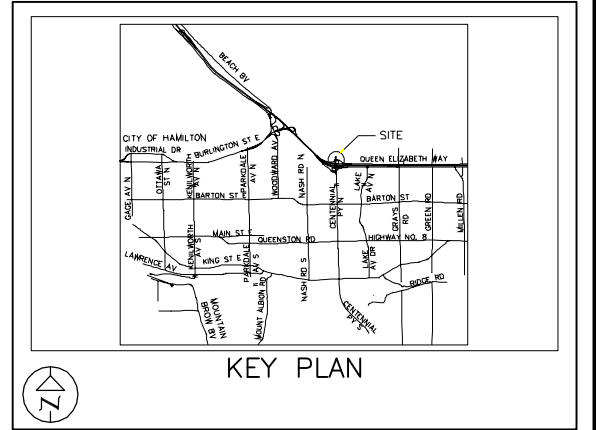
SHEET



Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA

METRIC

DIMENSIONS ARE IN METRES AND/OR
MILLIMETRES UNLESS OTHERWISE SHOWN



KEY PLAN

LEGEND

- Borehole - Current Investigation
- Borehole - Previous Investigation
- Seal
- Piezometer
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated
(Std. Pen. Test, 475 j/blow)
- 100% Rock Quality Designation (RQD)
- WL in piezometer, measured on October 22, 2003

No.	ELEVATION	CO-ORDINATES	
		NORTHING	EASTING
7	77.8	4789702.6	284117.2
8	78.0	4789666.7	284150.8
BR11-1	78.6	4789679.4	284130.5
BR11-2	81.1	4789637.1	284171.8
RESR10	78.1	4789717.3	284088.9

NOTES

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

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

REFERENCE

General Arrangement plan provided by McCormick Rankin Corporation, E-plan Bridge 11 S5132-313-001.dwg, received June 2004.

NO.	DATE	BY	REVISION

Geocres No.	PROJECT NO. Q21-1162	DIST.
HWY. QEW	DATE:	SITE:
SUBM'D.	CHKD. SEP	APPD. FJH
DRAWN: JDR	CHKD. ASP	DWG. 1

LIST OF ABBREVIATIONS

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

I. SAMPLE TYPE

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

III. SOIL DESCRIPTION

(a) Cohesionless Soils

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

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

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

Consistency

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

(b) Cohesive Soils

c_u, s_u

Dynamic Cone Penetration Resistance; N_d :

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

PH: Sampler advanced by hydraulic pressure

PM: Sampler advanced by manual pressure

WH: Sampler advanced by static weight of hammer

WR: Sampler advanced by weight of sampler and rod

Piezo-Cone Penetration Test (CPT)

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

IV. SOIL TESTS

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

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

LIST OF SYMBOLS

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

I. General

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

II. STRESS AND STRAIN

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

III. SOIL PROPERTIES

(a) Index Properties

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

(a) Index Properties (continued)

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

(b) Hydraulic Properties

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

(c) Consolidation (one-dimensional)

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

(d) Shear Strength

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

- Notes:**
- 1 $\tau = c' + \sigma' \tan \phi'$
 - 2 shear strength $= (\text{compressive strength})/2$
 - * density symbol is ρ . Unit weight symbol is γ where $\gamma = \rho g$ (i.e. mass density x acceleration due to gravity)

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 To	
⊥ - Perpendicular To	

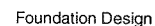
PROJECT 021-1162		RECORD OF BOREHOLE No 7		1 OF 2	METRIC
W.P. 441-97-00		LOCATION N 4789702.6 :E 284117.2		ORIGINATED BY PKS	
DIST _____ HWY QEW		BOREHOLE TYPE Power Auger 108mm I.D. Hollow Stem Auger		COMPILED BY KG	
DATUM Geodetic		DATE February 20, 1998		CHECKED BY SEP	

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa					
77.8	GROUND SURFACE						20 40 60 80 100						
0.0	Topsoil		1	SS	8								
77.5													
0.3	Clayey Silt, trace sand, some gravel (FILL) Firm to stiff Brown Moist		2	SS	8								
76.4													
1.4	Clayey SILT, trace sand and gravel Very stiff to hard Mottled grey and brown to brown Moist		3	SS	17								
			4	SS	31								
			5	SS	22								
			6	SS	30								
			7	SS	36								
72.2	Clayey SILT, some sand, trace gravel (TILL) Very stiff to hard Grey Moist		8	SS	28								
5.6													
			9	SS	26								
			10	SS	24								
	Becoming reddish-grey and occasional shale fragments below 10 m depth		11	SS	65/0.23								
			12	SS	93								
63.9			13	SS	100/0.07								
14.0													

Continued Next Page

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

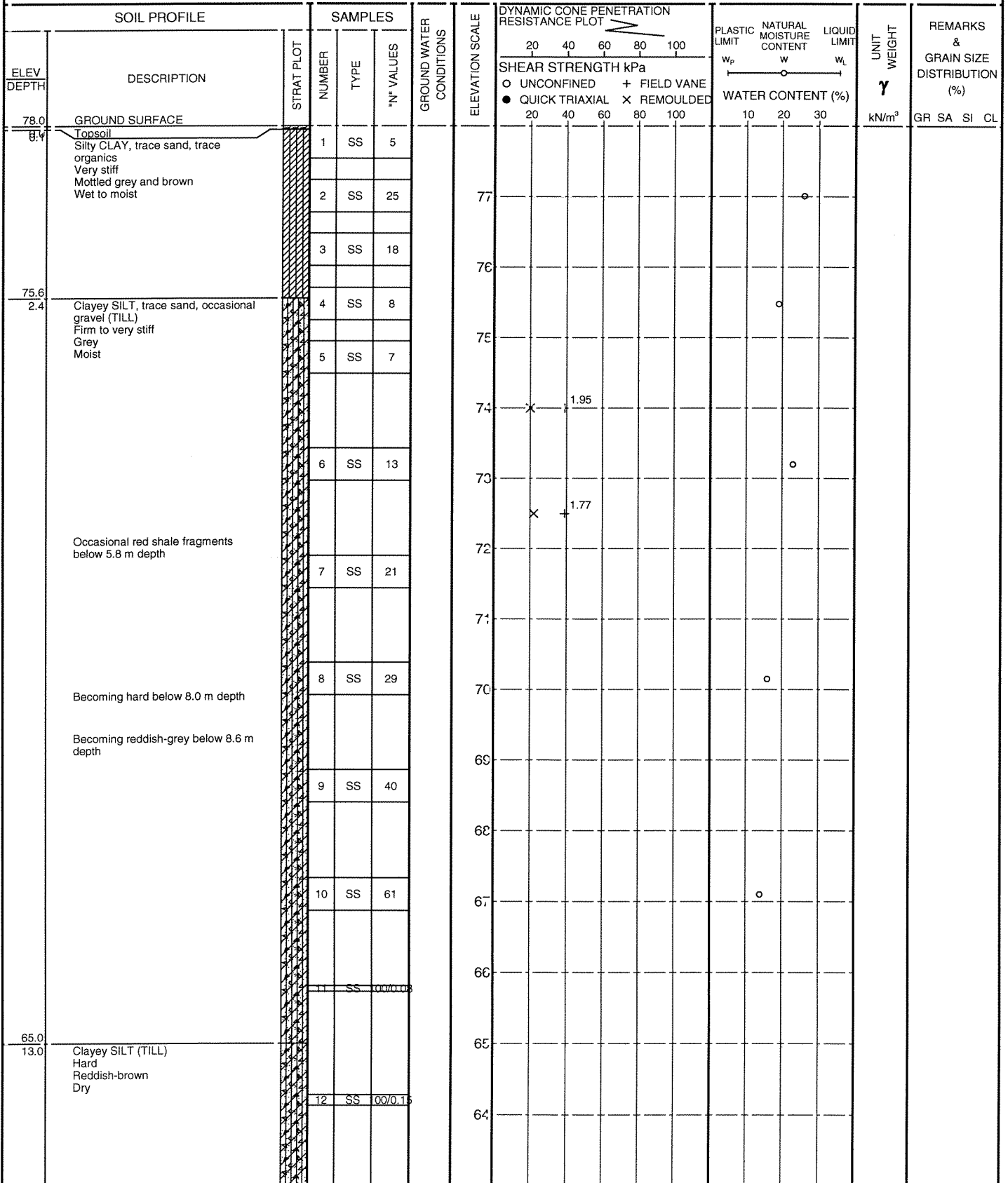
MISS_MTO_0211162EAMTO.GPJ ON_MOT.GDT 18/6/04



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

MISS_MTO 0211162EAMTO.GPJ ON_MOT.GDT 18/6/04

PROJECT <u>021-1162</u>		RECORD OF BOREHOLE No 8		1 OF 2	METRIC
W.P. <u>441-97-00</u>	LOCATION <u>N 4789666.7 ; E 284150.8</u>	ORIGINATED BY <u>GD</u>			
DIST <u> </u> HWY <u>QEW</u>	BOREHOLE TYPE <u>Power Auger 108mm I.D. Hollow Stem Auger</u>	COMPILED BY <u>KG</u>			
DATUM <u>Geodetic</u>	DATE <u>February 23, 1998</u>	CHECKED BY <u>SEP</u>			



MISS_MTO_0211162EAMTO.GPJ ON_MOT.GDT 18/6/04

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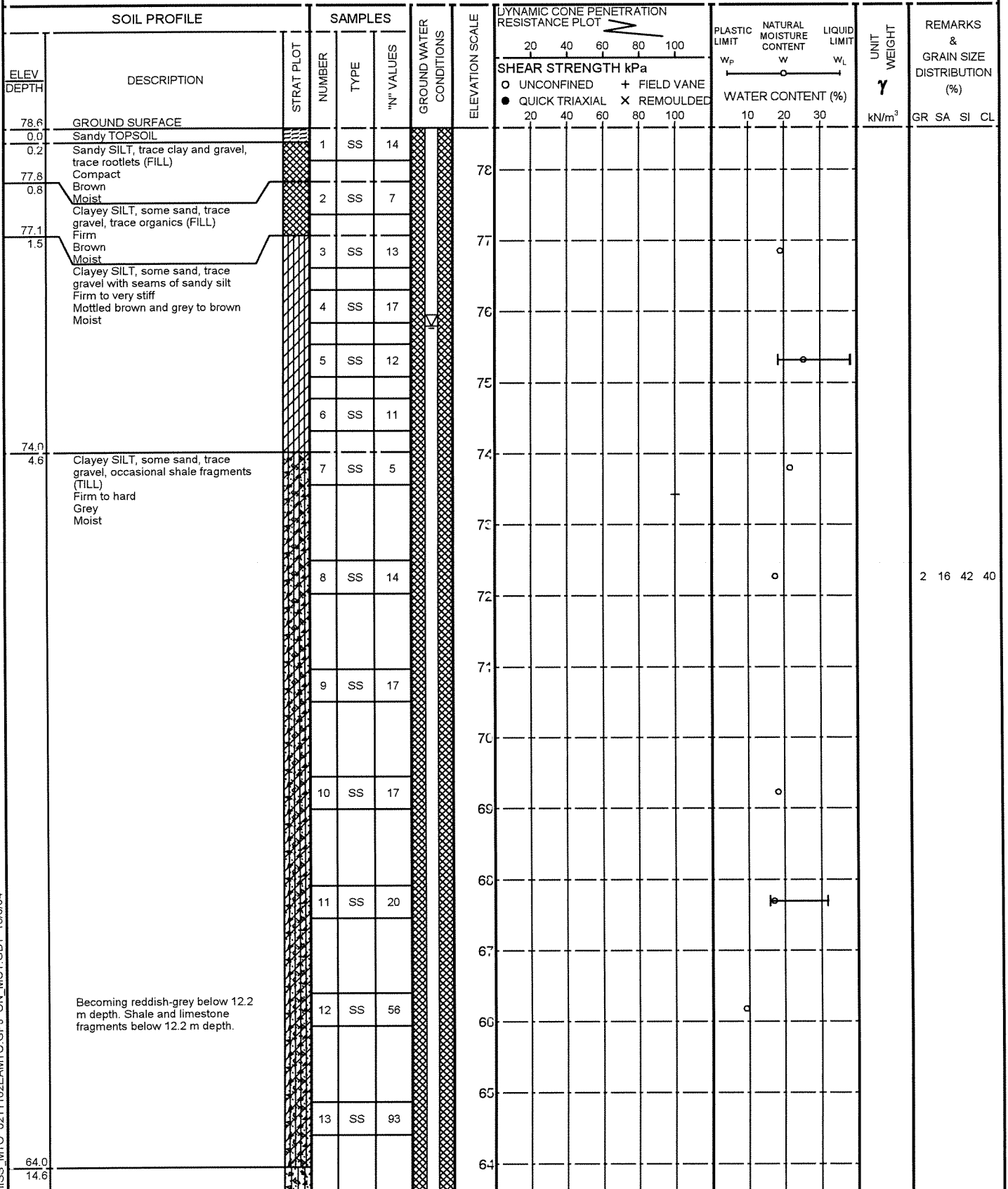
+ 3, X 3: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

PROJECT <u>021-1162</u>		RECORD OF BOREHOLE No 8		2 OF 2	METRIC
W.P. <u>441-97-00</u>	LOCATION <u>N 4789666.7 ; E 284150.8</u>	ORIGINATED BY <u>GD</u>			
DIST <u> </u> HWY <u>QEW</u>	BOREHOLE TYPE <u>Power Auger 108mm I.D. Hollow Stem Auger</u>	COMPILED BY <u>KG</u>			
DATUM <u>Geodetic</u>	DATE <u>February 23, 1998</u>	CHECKED BY <u>SEP</u>			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x REMOULDED		WATER CONTENT (%) w _p w w _L				
62.8 15.2	--- CONTINUED FROM PREVIOUS PAGE --- End of Borehole Notes: 1. Auger and spoon refusal at 15.2 m depth 2. Borehole dry upon completion of drilling	10												

MISS_MTO_0211162EAMTO.GPJ ON MOT_GDT 18/6/04




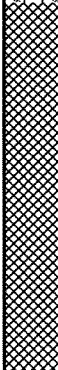

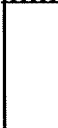
PROJECT 021-1162		RECORD OF BOREHOLE No BR11-1		1 OF 2	METRIC
W.P. 441-97-00		LOCATION N 4789679.4; E 284130.5		ORIGINATED BY PKS	
DIST _____ HWY QEW		BOREHOLE TYPE CME 75 Bombardier; 210mm O.D. Hollow Stem Auger		COMPILED BY KG	
DATUM Geodetic		DATE August 25, 2003		CHECKED BY SEP	



MISS_MTO_0211162EAMTO.GPJ ON_MOT_GDT 18/6/04

Continued Next Page

+³, X³: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

PROJECT		RECORD OF BOREHOLE		No BR11-1		2 OF 2		METRIC															
W.P. 441-97-00		LOCATION		N 4789679.4 ; E 284130.5		ORIGINATED BY		PKS															
DIST _____ HWY QEW		BOREHOLE TYPE		CME 75 Bombardier, 210mm O.D. Hollow Stem Auger		COMPILED BY		KG															
DATUM Geodetic		DATE		August 25, 2003		CHECKED BY		SEP															
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			SHEAR STRENGTH kPa			WATER CONTENT (%)			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100	20 40 60 80 100	W _p	W	W _L	20 40 60 80 100	10 20 30	γ	kN/m ³	GR	SA	SI	CL			
--- CONTINUED FROM PREVIOUS PAGE ---																							
60.3	Sandy SILT, some clay, trace gravel, shale and limestone fragments (TILL) Very dense Reddish-brown Moist		14	SS	103		63																
18.3																							
60.3	Moderately to highly weathered, reddish-grey calcareous SHALE BEDROCK (Queenston Formation) with occasional grey limestone/siltstone layers Bedrock cored from 18.3m to 21.9m depth. For details of bedrock coring refer to Record of Drillhole BR11-1		15	SS	00/0.0		62																
18.3																							
56.7	End of Borehole Notes: 1. Spoon refusal at 18.3m depth. 2. Water level in piezometer at 2.88m depth (Elev. 75.9m) on October 22, 2003.						57																
22.0																							

PROJECT: 021-1162

RECORD OF DRILLHOLE: BR11-1

SHEET 1 OF 1

LOCATION: N 4789679.4; E 284130.5

DRILLING DATE: August 25, 2003

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: Bomb CME 75

DRILLING CONTRACTOR: Geo-Environmental Drilling Ltd.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (m/min)	COLOUR % RETURN	FR/FX-FRACTURE F-FAULT												SM-SMOOTH				FL-FLEXURED				BC-BROKEN CORE				NOTES WATER LEVELS INSTRUMENTATION			
								CL-CLEAVAGE				J-JOINT				R-ROUGH				UE-UNEVEN				MB-MECH. BREAK				B-BEDDING							
								SH-SHEAR				P-POLISHED				ST-STEPPED				W-WAVY															
								VN-VEIN				S-SLICKENSIDED				PL-PLANAR				C-CURVED															
								RECOVERY				R.Q.D. %		FRACT. INDEX PER 0.3		DISCONTINUITY DATA				HYDRAULIC CONDUCTIVITY K, cm/sec				DIAMETRAL POINT LOAD INDEX (MPa)											
								TOTAL CORE %				SOLID CORE %								DIP w.r.t. CORE AXIS				TYPE AND SURFACE DESCRIPTION											
								80 60 40 20				80 60 40 20				80 60 40 20		5 10 15 20		0 20 40 60 80															
								1 2 3 4				1 2 3 4				1 2 3 4		1 2 3 4		10 ⁻⁶ 10 ⁻⁵ 10 ⁻⁴ 10 ⁻³				2 4 6											
19		Refer to previous page		60.30																															
		Moderately to highly weathered, thinly layered, reddish-grey to light grey, fine grained, very weak to medium strong, calcareous SHALE BEDROCK (Queenston Formation)		18.30	1																														
		Occasional seams/layers of light grey, strong to very strong limestone																																	
		All fractures are bedding, smooth			2																														
20																																			
21																																			
22		End of Drillhole		56.60																															
				22.00																															
23																																			
24																																			
25																																			
26																																			
27																																			
28																																			

DEPTH SCALE

1 : 50



LOGGED: PS

CHECKED: SEP

MISS ROCK 021162EARCK GPJ GAL-CANADA GDT 18/6/04 TM

PROJECT 021-1162		RECORD OF BOREHOLE No BR11-2				1 OF 1		METRIC					
W.P. 441-97-00		LOCATION N 4789637.1 E 284171.8				ORIGINATED BY PKS							
DIST _____ HWY QEW		BOREHOLE TYPE CME 75 Bombardier, 108mm O.D. Solid Stem Auger				COMPILED BY KG							
DATUM Geodetic		DATE August 27, 2003				CHECKED BY SEP							
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa					
81.1	GROUND SURFACE												
0.0	Sandy TOPSOIL												
0.2	Sandy SILT to Silty SAND, trace to some clay, trace gravel, trace organics		1	SS	15								
	Compact Brown Moist		2	SS	15								
79.6	Clayey SILT, some sand, trace gravel with seams of sandy silt		3	SS	29								
1.5	Very stiff to firm Brown to mottled brown and grey Moist		4	SS	18								
			5	SS	5								
77.6	Clayey SILT, some sand, trace gravel, (TILL)		6	SS	2								
3.5	Soft to firm Moist Brown to mottled brown and grey		7	TO	PH								
			8	SS	4								
73.5	Clayey SILT, some sand, trace gravel, (TILL)		9	SS	13								
7.6	Stiff to very stiff Moist Brown to mottled brown and grey												
	Red shale fragments below 9.1m depth		10	SS	23								
71.5	End of Borehole												
9.6	Notes: 1. Open borehole dry upon completion of drilling												

MISS MTO 0211162EAMTO.GPJ ON MOT.GDT 18/6/04

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

RECORD OF BOREHOLE No RESR-10

1 OF 1

METRIC

PROJECT 021-1162

W.P. 441-97-00

LOCATION N 4789717.3 : E 284088.9

ORIGINATED BY PKS

DIST HWY QEW

BOREHOLE TYPE Power Auger 108mm I.D. Hollow Stem Auger

COMPILED BY KG

DATUM Geodetic

DATE September 16, 1998

CHECKED BY SEP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED							WATER CONTENT (%)
78.1 0.9	GROUND SURFACE Topsoil Silty Sand, trace clay and gravel (FILL) Compact Brown Moist		1	SS	25										
76.7 1.4	Silty Clay, some sand, trace gravel, trace organics and rootlets (FILL) Stiff Brown to grey Moist to wet		2	SS	13										
			3	SS	9										
75.2 2.9	Clayey SILT, trace sand and gravel (TILL) Very stiff to hard Mottled brown and grey to grey Moist		4	SS	20										
			5	SS	34										
			6	SS	39										
			7	SS	45										
			8	SS	30										
			9	SS	34										
			10	SS	35										
68.5 9.6	End of Borehole														
Notes: 1. Water level in piezometer at Elev. 76.4 m on Oct. 19, 1998 2. Water level in piezometer at Elev. 75.1 m on Nov. 10, 1998															

Notes:

1. Water level in piezometer at Elev. 76.4 m on Oct. 19, 1998
2. Water level in piezometer at Elev. 75.1 m on Nov. 10, 1998

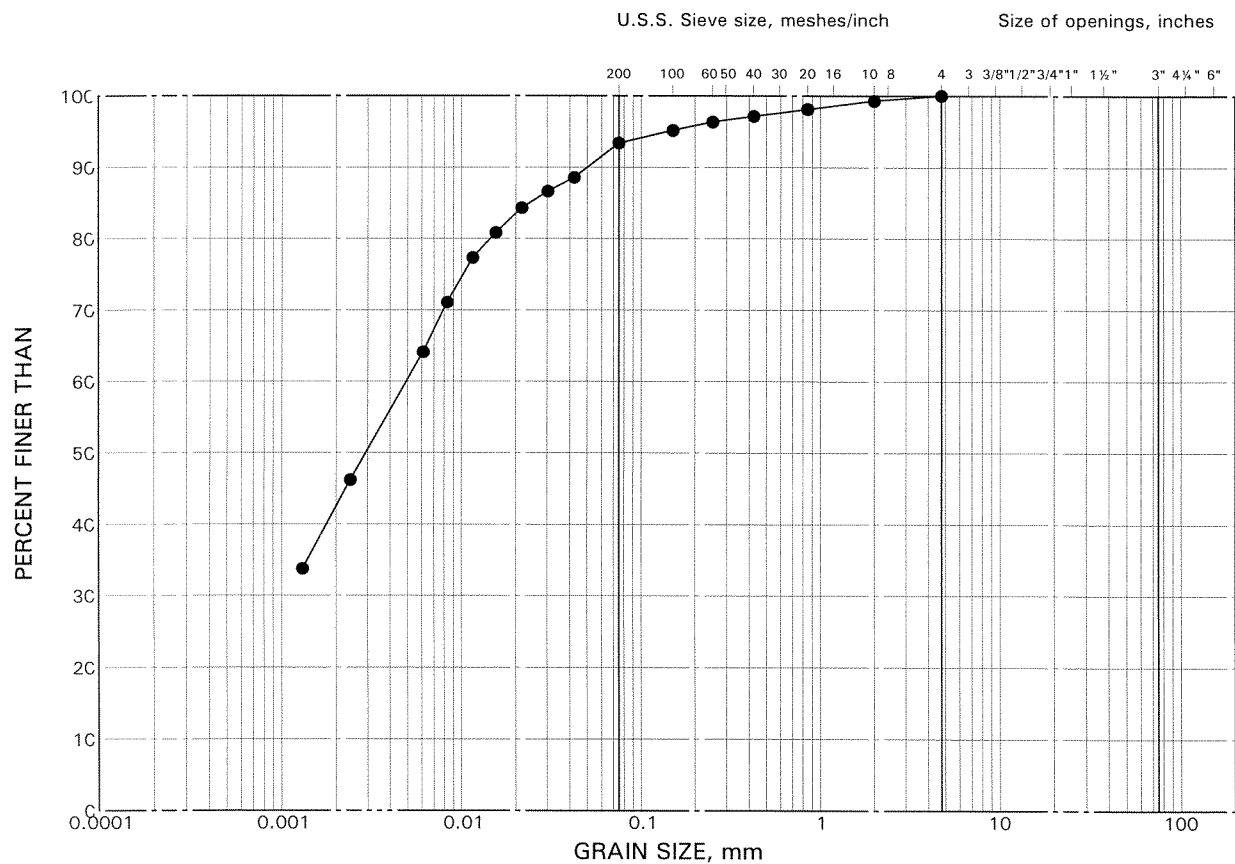
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APPENDIX A
LABORATORY TEST DATA

GRAIN SIZE DISTRIBUTION

Silty Clay

FIGURE A1



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
•	7	5	74.3

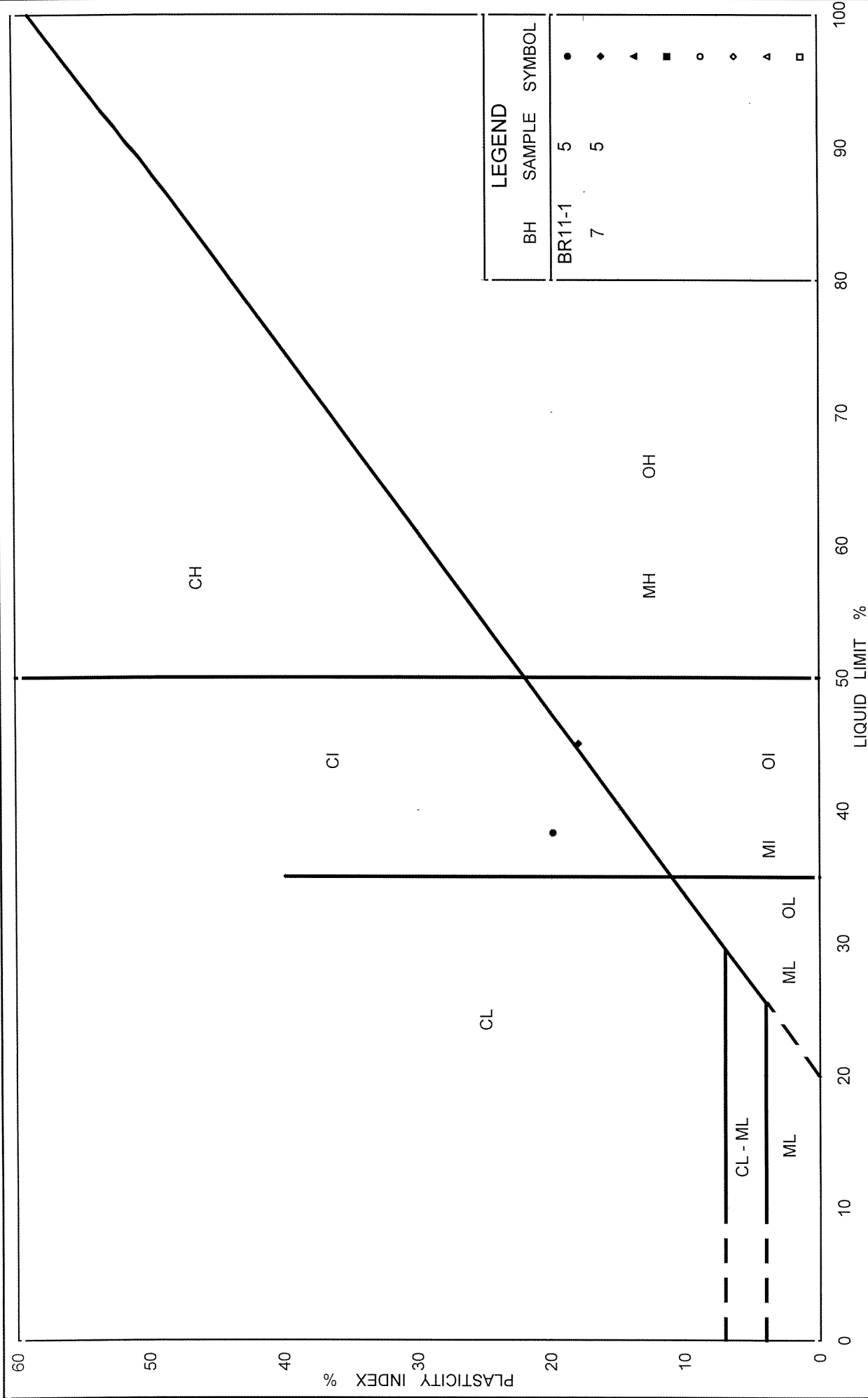


FIG No. A2

PLASTICITY CHART

Silty Clay

Ministry of Transportation



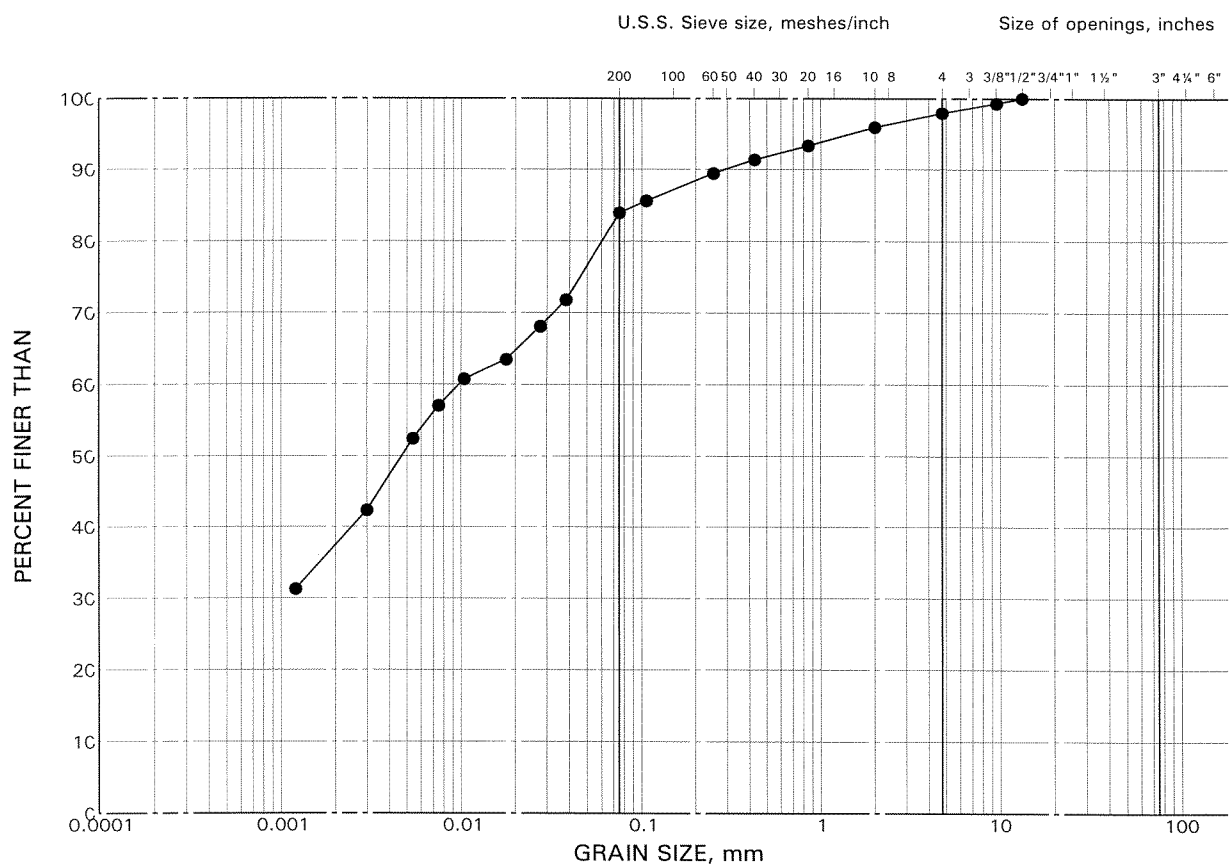
Ontario

Project No. 021-1162-BR11

GRAIN SIZE DISTRIBUTION

Clayey Silt (Till)

FIGURE A3



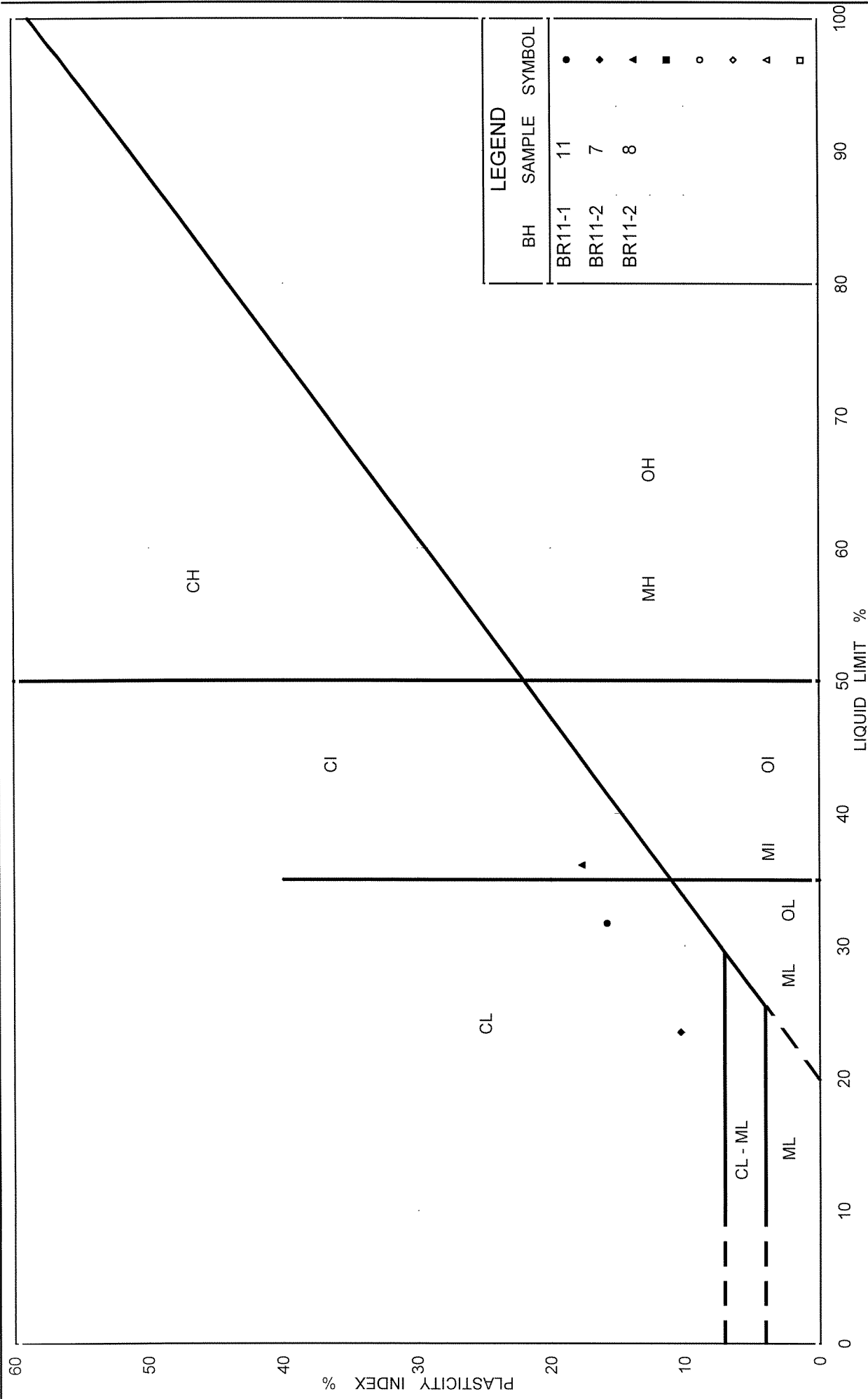


FIG No. A4

PLASTICITY CHART
Clayey Silt (Till)

Ministry of Transportation



Ontario

OEDOMETER CONSOLIDATION SUMMARY

SAMPLE IDENTIFICATION

Project Number	021-1162	Sample Number	7
Borehole Number	BR11-2	Sample Depth, m	4.9-5.5

TEST CONDITIONS

Test Type	Standard	Load Duration, hr	24
Oedometer Number	8		
Date Started	10/11/2003		
Date Completed	10/23/2003		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	1.91	Unit Weight, kN/m ³	18.87
Sample Diameter, cm	6.34	Dry Unit Weight, kN/m ³	14.01
Area, cm ²	31.57	Specific Gravity, measured	2.73
Volume, cm ³	60.30	Solids Height, cm	0.999
Water Content, %	34.68	Volume of Solids, cm ³	31.55
Wet Mass, g	116.01	Volume of Voids, cm ³	28.74
Dry Mass, g	86.14	Degree of Saturation, %	103.9

TEST COMPUTATIONS

Pressure kPa	Corr. Height cm	Void Ratio	Average Height cm	t ₉₀ sec	cv. cm ² /s	mv m ² /kN	k cm/s
0.00	1.910	0.911	1.910				
4.71	1.898	0.899	1.904	658	1.17E-03	1.32E-03	1.51E-07
9.56	1.892	0.893	1.895	366	2.08E-03	6.69E-04	1.36E-07
19.34	1.881	0.882	1.886	330	2.29E-03	5.89E-04	1.32E-07
38.81	1.865	0.866	1.873	146	5.09E-03	4.28E-04	2.13E-07
77.64	1.842	0.843	1.853	141	5.16E-03	3.13E-04	1.58E-07
155.06	1.802	0.803	1.822	146	4.82E-03	2.66E-04	1.26E-07
309.90	1.718	0.719	1.760	211	3.11E-03	2.85E-04	8.71E-08
620.75	1.629	0.630	1.673	211	2.81E-03	1.50E-04	4.14E-08
1240.94	1.548	0.548	1.588	146	3.66E-03	6.86E-05	2.46E-08
2481.89	1.470	0.471	1.509	129	3.74E-03	3.27E-05	1.20E-08
1240.94	1.505	0.505	1.487				
309.90	1.544	0.545	1.524				
77.64	1.603	0.604	1.574				
9.56	1.609	0.610	1.606				
4.71	1.620	0.621	1.614				

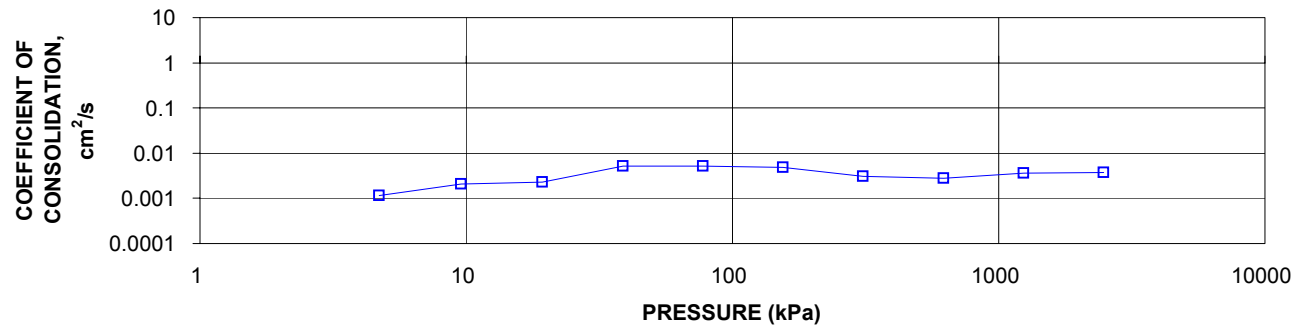
Notes:
k calculated using cv based on t₉₀ values.

SAMPLE DIMENSIONS AND PROPERTIES - FINAL

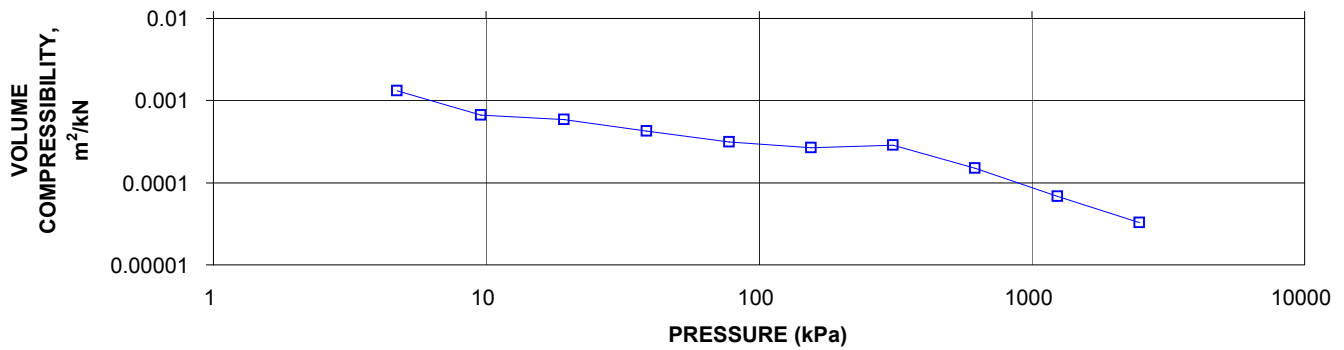
Sample Height, cm	1.62	Unit Weight, kN/m ³	20.64
Sample Diameter, cm	6.34	Dry Unit Weight, kN/m ³	16.52
Area, cm ²	31.57	Specific Gravity, measured	2.73
Volume, cm ³	51.13	Solids Height, cm	0.999
Water Content, %	24.92	Volume of Solids, cm ³	31.55
Wet Mass, g	107.61	Volume of Voids, cm ³	19.58
Dry Mass, g	86.14		

OEDOMETER CONSOLIDATION SUMMARY

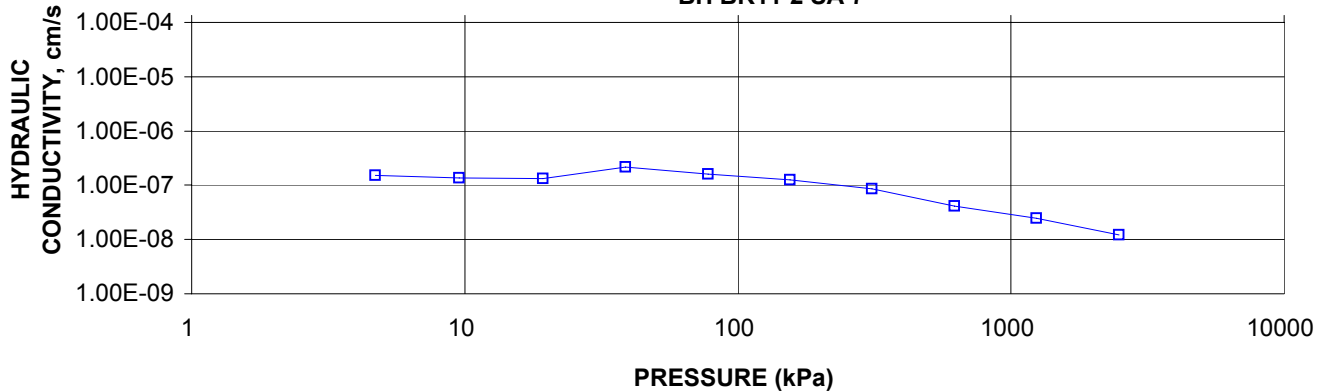
CONSOLIDATION TEST
CV cm²/s VS PRESSURE (kPa)
BH BR11-2 SA 7

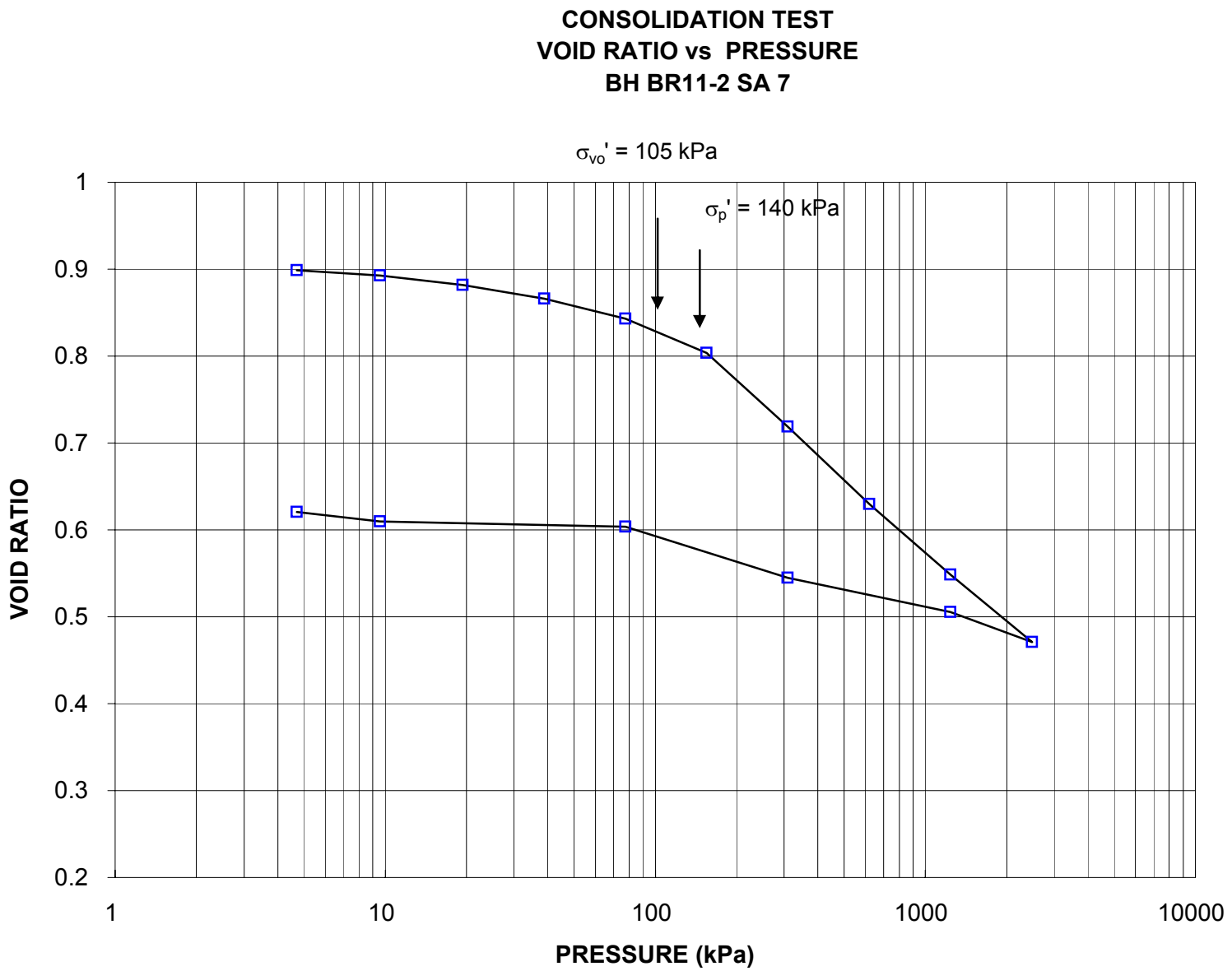


CONSOLIDATION TEST
MV m²/kN vs PRESSURE (kPa)
BH BR11-2 SA 7



CONSOLIDATION TEST
HYDRAULIC CONDUCTIVITY vs PRESSURE
BH BR11-2 SA 7





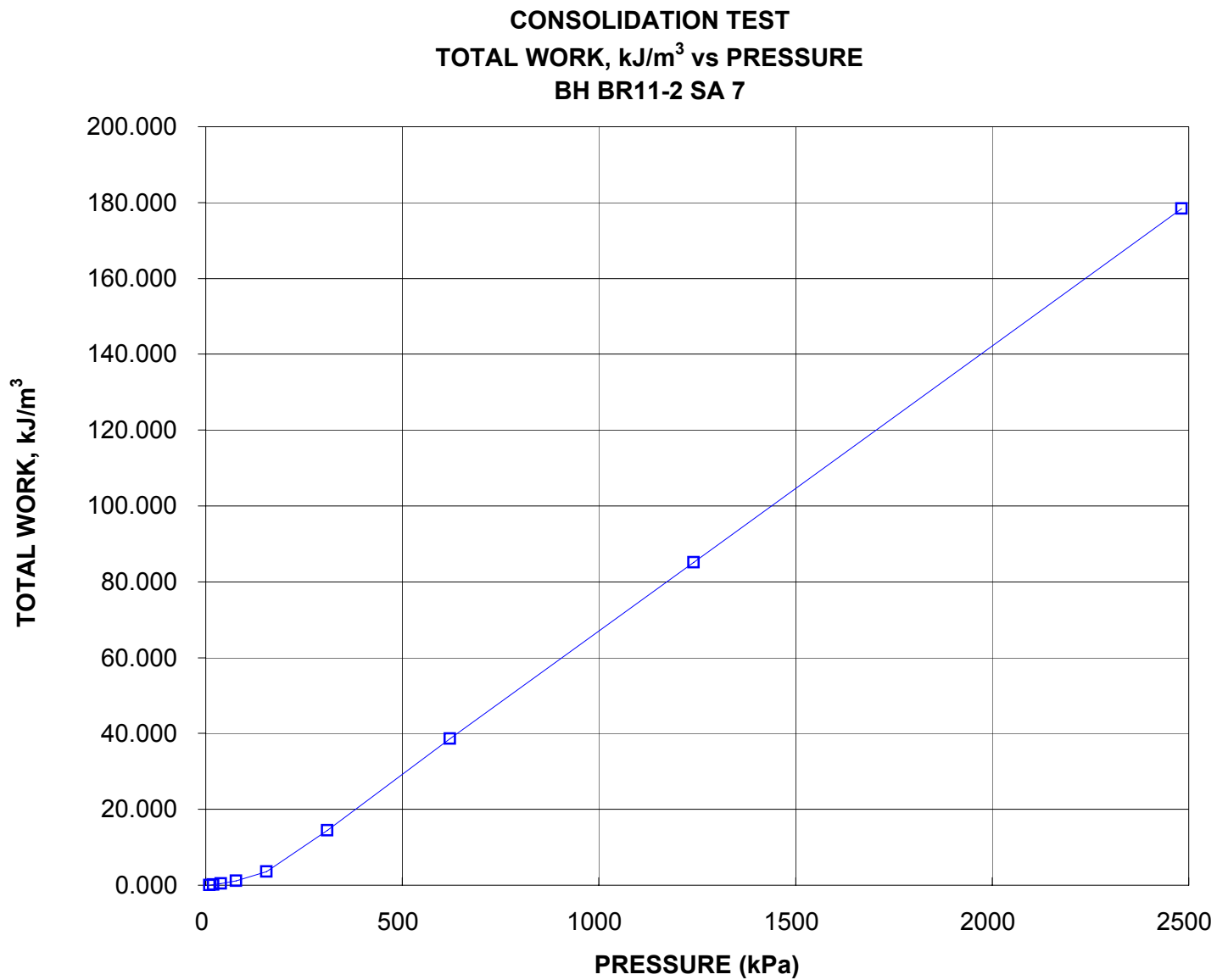
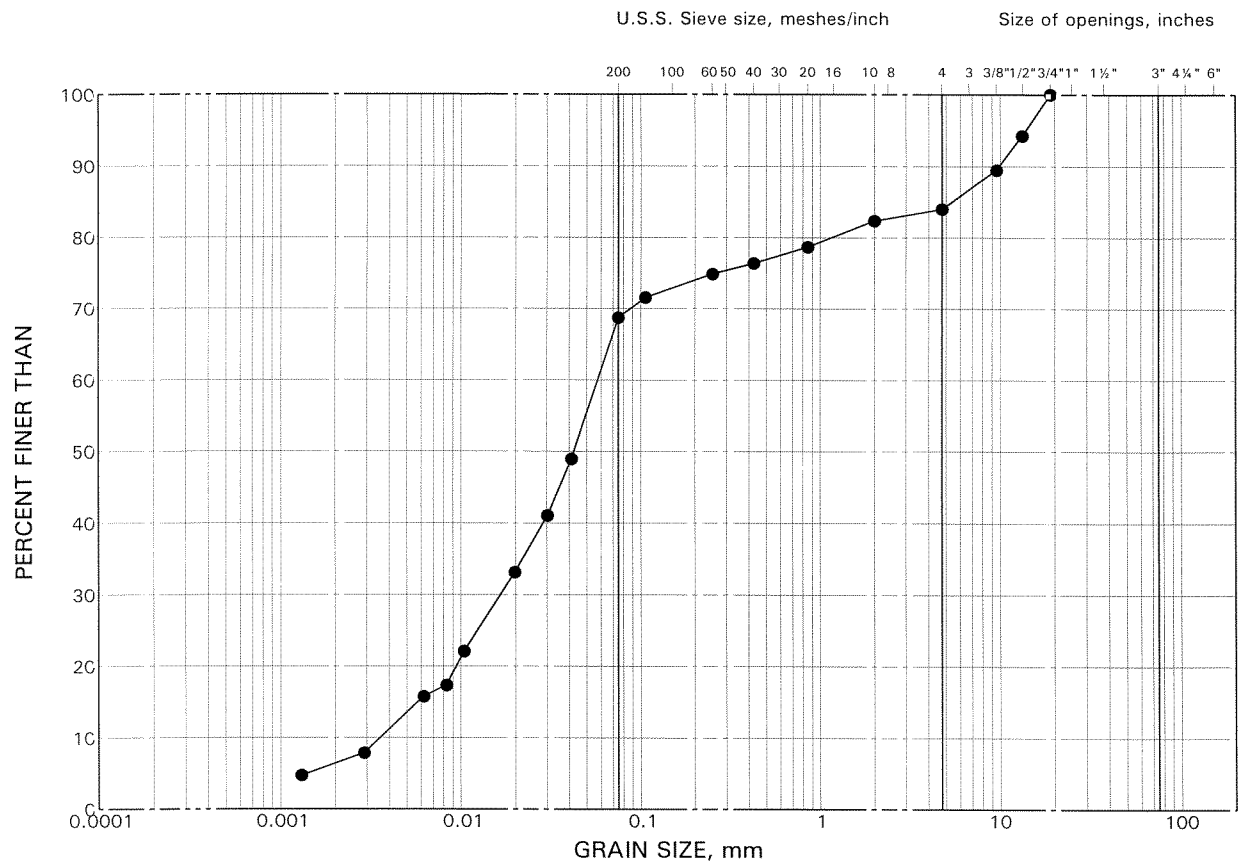


FIGURE A6

GRAIN SIZE DISTRIBUTION
Sandy Silt (Till)

FIGURE A7



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
•	BR11-1	14	63.1