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REPORT ON

**DETAILED
FOUNDATION INVESTIGATION AND DESIGN
BRIDGE 8; RHCE OVER QEW
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

**DETAIL DESIGN
FOUNDATION INVESTIGATION REPORT
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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 detailed 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 8 carrying the Red Hill Creek Expressway over the Queen Elizabeth Way as part of the interchange project. The work was carried out in accordance with the Quality Control Plan for this project dated November 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 area is generally flat-lying with the exception of a tree-lined drainage ditch that crosses the northeast to the southwest corner of the proposed bridge. This drainage channel is in culvert under the QEW. The QEW grade at the bridge site is between about Elevation 80 m to 81 m in the area of the proposed works. 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 8 site was carried out between August 13 and August 25, 2003 at which time three (3) boreholes, numbered BR8-1, BR8-2 and RW8-1 were advanced. Boreholes 2 and 3 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 27.7 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 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 Soil Conditions and General Overview

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 surficial layers of sandy silt and clayey silt fill which is underlain by thin layers of silty sand to sandy silt, clayey silt to silty clay and by a silty sand deposit. The surficial deposits are in turn underlain by a thick deposit of grey clayey silt till. The upper portion of the clayey silt till deposit is typically firm to stiff while the lower portion of the deposit is stiff to hard. Reddish brown clayey silt till containing shale fragments was encountered below the upper clayey silt till deposit. The clayey tills are underlain by a sandy silt till deposit containing cobbles and boulders in turn underlain by shale bedrock of the Queenston Formation. In the deepest borehole at the site where bedrock was proven by coring, the total overburden thickness was 24.4 m. 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 of the boreholes at this site except Borehole 2, located at the south approach area. The ground surface ranges between Elevation 78.7 m to 80.8 m and the thickness of the topsoil ranged from 0.1 m to 0.3 m.

4.2.2 Fill

Fill materials were encountered below the topsoil or at ground surface in Borehole BR8-2 and 2, located at the south abutment and approach area. The fill ranges in composition from clayey silt, sandy silt and silty sand and extends to between 1.4 m and 2.2 m depth below ground surface at the borehole locations. Trace to some organics and rootlets were noted in the fill.

Standard Penetration Testing (SPT) measured 'N' values within the non-cohesive fill ranged between 4 and 12 blows per 0.3 m of penetration, indicating a very loose to compact relative density. SPT measured 'N' values within the cohesive fill ranged between 9 and 14 blows per 0.3 m of penetration, indicating a stiff consistency.

The natural water content measured on samples of the fill ranges from 18 to 20 percent.

4.2.3 Sandy Silt to Silty Sand

A deposit of brown sandy silt to silty sand containing trace to some clay and trace gravel and occasional clayey silt seams was encountered below the topsoil in Borehole RW8-1, located at the north approach area. The sandy silt to silty sand layer was 2.3 m thick, and the surface was encountered at Elevations 80.3 m.

Standard Penetration Testing (SPT) measured 'N' values ranged between 8 and 16 blows per 0.3 m of penetration within the deposit, indicating a loose to compact state of packing.

The natural water content measured on one sample of the sandy silt to silty sand was approximately 11 percent.

4.2.4 Clayey Silt

A deposit of clayey silt to silty clay was encountered immediately below the topsoil or fill layer in Boreholes 2 and 3 and below the surficial sandy silt to silty sand deposit in Borehole BR8-1. This deposit is discontinuous across the site. The brown and grey clayey silt deposit contained trace to some sand and gravel. Occasional sand and silt seams and trace oxidized stains were

noted in Boreholes 2 and 3. The clayey silt deposit ranged from 0.8 to 2.6 m in thickness, and the surface was encountered between Elevations 78.4 and 80.0 m at the borehole locations.

At the borehole locations, Standard Penetration Testing (SPT) measured 'N' values ranged between 13 and 67 blows per 0.3 m of penetration within the clayey silt deposit, indicating a stiff to hard consistency.

The natural water contents measured on samples of the clayey silt to silty clay range from 13 to 21 percent.

4.2.5 Silty Sand

A deposit of brown and grey silty sand was encountered underlying the surficial clayey silt to silty clay deposit in Boreholes BR8-1 and 2, located at the south abutment and approach area. Occasional silty clay seams were noted in Borehole BR8-1. The silty sand ranged from 1.5 to 2.7 m in thickness, and the surface was encountered between Elevations 77.5 and 77.6 m at the borehole locations.

Standard Penetration Testing (SPT) measured 'N' values ranged between 25 and 97 blows per 0.3 m of penetration within the silty sand deposit, indicating a compact to very dense state of packing.

The natural water contents measured on samples of the sandy silt to silty sand were between 12 and 17 percent.

4.2.6 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. Occasional silt seams were noted in Borehole 3. The clayey silt till is typically grey in color becoming reddish-grey 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 74.7 and 78.0 m in all boreholes and the thickness varied from 5.5 m to 10.8 m. This deposit was not fully penetrated in Borehole RW8-1.

The upper 2.0 m to 7.2 m of the clayey silt till deposit is described as a 'softened' till zone and was encountered in Borehole BR8-1, located at the south abutment and in Boreholes RW8-1 and 3, located at the north abutment and approach area. The SPT measured 'N' values within the upper portion of the clayey silt ranged between 3 and 14 blows per 0.3 m of penetration. In situ field vane testing was carried out within the upper 'softened' portion of this till deposit where encountered, using a standard MTO 'N' vane. The results of field vane tests indicate that the

upper softened portion of the deposit has a soft to firm consistency and the lower portion has a stiff to hard consistency. The results of the vane testing are summarized in the following table:

<i>Borehole</i>	<i>Location</i>	<i>Test Depth/Elevation (m)</i>	<i>Undisturbed Shear Strength(kPa)</i>	<i>Remoulded Shear Strength (kPa)</i>	<i>Sensitivity</i>
RW8-1	North approach	2.9 / 77.5	19	9	2.2
RW8-1	North approach	4.6 / 75.8	28	14	2.0
RW8-1	North approach	6.7 / 73.7	42	30	1.4
3	South abutment	4.0 / 74.7	38	18	2.1
BR8-1	South abutment	6.6 / 74.2	40	22	1.8

The lower portion of clayey silt till deposit had measured SPT 'N' values ranging from 10 to 100 blows per 0.3 m of penetration, suggesting a stiff to hard consistency. Typically the 'N' values increased with depth. The elevation of the top of the stiff to hard portion of the deposit varied from Elevation 72.3 m to 73.3 at the north abutment and approach area to between Elevation 72.7 m and 77.9 m at the south abutment and approach area. In Boreholes BR8-1, BR8-2, 2 and 3 the stratum was fully penetrated and was found to range between 3.5 m and 9.3 m in thickness. Grain size distribution curves for selected samples from this deposit are shown on Figure A1 in Appendix A.

Atterberg limits testing was carried out on three samples of the softened clayey silt till deposit and on two samples of the lower portion of the clayey silt till deposit. The results of the Atterberg limits tests are plotted on the plasticity chart on Figure A2 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 RW8-1, sample 6 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>
BR8-1	8	74.2 – 74.7	32	16	16
BR8-2	11	69.1 – 69.5	31	15	17
RW8-1	5	76.6 – 76.0	23	13	10
RW8-1	6	74.4 – 73.8	36	18	18
2	9	72.4 – 72.9	34	19	15
Average	-	-	31	16	15

The natural water content measured on selected samples of the till deposit ranged between 13 percent and 25 percent, with an average of 19 percent.

Laboratory oedometer (consolidation) testing was carried out on one specimen of the upper ‘softened’ portion of the clayey silt till obtained from Borehole RW8-1. Details of the test results are shown on Figures A3 and A4 in Appendix A and the results summarized in the table below:

<i>Borehole and Sample No.</i>	<i>Elevation (m)</i>	σ_{vo}' (kPa)	σ_p' (kPa)	<i>OCR</i>	e_o	C_r	C_c	c_v^* (cm ² /s)
RW8-1 Sa 5	76.6 – 76.0	66	140	2.1	0.534	0.026	0.142	0.0075

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

Laboratory consolidated undrained (CIU) triaxial compression test were carried out on one carefully trimmed specimen of the clayey silt till obtained from Borehole RW8-1. The test results indicate effective angle of shearing resistance of 30 degrees and effective shear resistance of 0 kPa. Details of the test results are shown on Figure A5 in Appendix A.

At the base of the till deposit, a thin layer of reddish-brown clayey silt containing some sand and gravel was encountered in Boreholes BR8-1, BR8-2, 2 and 3. Occasional weathered shale and limestone fragments were observed in the deposit. The top of this layer was encountered between Elevation 66.8 m and 69.5 m. This deposit was between 2.5 m and 3.0 m in thickness in Boreholes BR8-1, BR8-2 and 2. Borehole 3 was terminated in this layer and after penetrating the deposit by 1.9 m.

The measured SPT ‘N’ value within this lower portion of the till deposit were greater than 100 blows per 0.3 m of penetration, indicating a hard consistency.

The natural water contents measured on two selected samples of this hard clayey silt till layer were 6 percent.

4.2.7 Sandy Silt Till

In Boreholes BR8-1, BR8-2 and 2, where the clayey silt till was fully penetrated, a deposit of red sandy silt till, was encountered below the reddish-brown clayey silt till. The deposit contains

varying amounts of clay and gravel, cobbles and boulders. This deposit is considered to be the ‘Wentworth’ till sheet. The surface of the deposit was encountered between Elevation 65.6 m and 67.0 m. The deposit was penetrated in BR8-2 and the deposit was found to be approximately 9.8 m thick. Boreholes BR8-1 and 2 were terminated in this deposit after proving a thickness of 3.9 m and 7.9 m, respectively.

Borehole BR8-2 was advanced through this till deposit by augering and by rock coring where refusal to augering and spoon advance was encountered. ‘NQ’ coring was carried out between 16.3 and 19.8 m depths and between 22.6 and 24.4 m depths within the deposit. Limestone and siltstone boulders were recovered from the rock core between 16.3 m and 22.6 m depth and between 22.6 m and 24.4 m depth at the base of the deposit. A grain size distribution curve for samples of this deposit is shown on Figure A6 in Appendix A.

The measured SPT ‘N’ values within the till were greater than 100 blows per 0.3 m of penetration, indicating a very dense state of packing.

The natural water content measured on selected samples of this deposit varied between 6 and 11 percent.

4.2.8 Bedrock

In Borehole BR8-2, where the sandy silt till was fully penetrated, the bedrock surface was encountered at a depth of 24.4 m below ground surface at Elevation 55.8 m and the bedrock was cored for a depth of 3.3 m.

The bedrock samples obtained consist of reddish-grey, moderately to highly weathered, thinly layered, very fine grained, very weak to medium strong calcareous shale of the Queenston formation. Seams and layers of the fresh to slightly weathered limestone were present within the shale bedrock. The Total Core Recovery was between 50 percent and 100 percent. The Rock Quality Designation (RQD) measured on the core samples in Borehole BR8-2 ranged from about 0 to 73 percent, with the lower values encountered near the surface of the bedrock. This indicates a rock mass of very poor to fair quality; typically fair.

Point load strength tests were performed on selected samples of the rock core from Borehole BR8-2. Diametral point load strength index values are shown on the Record of Drillhole Sheet. Diametral point load index values on core samples of the shale range from 0.12 to 1.8 MPa which corresponds to an estimated unconfined compressive strength (UCS) ranging from 3 to 41 MPa. The axial point load index values on two core samples of the shale were 1.0 and 1.81 MPa corresponding to approximate UCS values of 23 and 41 MPa, respectively. Using the Intact Rock Strength Classification table, these values indicate that the shale is classified very weak to

medium strong. One diametral point load index of 7.87 MPa was measured on a core sample of the limestone corresponding to an approximate UCS of 177 MPa. This indicates that the limestone layer is classified as very strong.

4.2.9 Groundwater Conditions

The water levels were noted during and after the drilling and coring operations in the boreholes. Piezometers were installed in Boreholes BR8-1, BR8-2, 2 and 3. The piezometers in Boreholes BR8-1 and 2 were sealed within the sandy silt till deposit and piezometer in Borehole 3 was sealed in the clayey silt till deposit. The piezometer in BR8-2 was sealed into the bedrock. 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:

Borehole	Ground Surface Elevation (m)	Ground Water Level Depth (m)	Ground Water Level Elevation (m)	Date
BR8-1	80.8	8.7	72.1	October 22, 2003
BR8-2	80.2	4.9	75.3	October 22, 2003
2	80.5	1.9	78.7	October 22, 2003
3	78.7	Dry	n/a	March 25, 1998*

* The piezometer in Borehole 3 could not be found in 2003.

The groundwater level at the borehole location typically appears to be coincident with the base of the softened upper portion 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.

GOLDER ASSOCIATES LTD.



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SEP/ASP/FJH/sm

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**PART B
DETAIL DESIGN
FOUNDATION INVESTIGATION AND DESIGN
BRIDGE 8; RHCE OVER QEW
RED HILL CREEK EXPRESSWAY INTERCHANGE
G.W.P. 441-97-00
MINISTRY OF TRANSPORTATION, ONTARIO
HAMILTON, ONTARIO**

5.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides foundation design recommendations for the proposed Bridge 8 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 over the Queen Elizabeth Way. The existing QEW will be upgraded as part of the project. The proposed bridge is a 2-span bridge with span lengths of about 41 m. A retaining wall is proposed beyond the limits of the wing walls at the north approach; this wall will be addressed under separate cover. The proposed approach embankments will be up to about 9 m to 11 m in height; varying with existing ground surface. The revised General Arrangement drawing for Bridge 8 was provided by MRC in electronic format in June 2004 and incorporated into Drawing 1.

5.1 General

Various alternatives for the abutment and pier 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 anticipated settlements as a result of the presence and variability of the soft to firm till deposit. Caissons extended into the bedrock are not recommended given the extremely variable and bouldery till deposits which would have to be penetrated and variable bedrock conditions. In addition, the bedrock surface is at greater than 25 m depth and, in conjunction with the bouldery nature of the overlying till, caissons extended to bedrock may not be economical at this site. It is considered that steel H-piles driven to refusal just into the very dense sandy silt till for support of the piers and abutments is the most feasible option from a geotechnical / foundation perspective.

The associated wing walls could consist either of concrete cantilever walls or of RSS walls, depending on the bridge type chosen. For conventional bridge abutments, the associated wing walls are typically concrete, may be supported on spread footings within the embankment fill or on steel H-piles driven to refusal into the very dense sandy silt till. If integral abutments are

being considered in design of this bridge, the associated wing walls typically consist of Retained Soil System (RSS) type walls, founded on the embankment fill.

5.2 Shallow Foundations

For both concrete and RSS type wing walls founded within 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. Settlement of the founding soils below the embankment fill and wall footings will occur and alternatives for mitigation of this settlement will be addressed in Section 5.6.4.

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

5.2.1 Geotechnical Resistance

5.2.1.1 Concrete Cantilever Wing Walls

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.

5.2.1.2 RSS Wing Walls

A typical RSS wall is a mechanically-reinforced soil retaining wall system which consists of granular fill placed and compacted in layers, and reinforced with metal or fabric strips or grids. A facing material, typically pre-cast concrete panels mechanically fastened to the reinforcing strips or grids, is used to form the face of the reinforced soil structure and to prevent the loss of fill material and is supported on a strip footing. The facing footing must be founded below any topsoil, loose fill or unsuitable native soils. For an assumed width of 0.6 m for the facing footing and assuming the footing is placed on a granular levelling pad or properly prepared undisturbed subgrade, a factored geotechnical resistance at ULS of 150 kPa may be used for design of the facing footing.

The RSS wall acts as a unit and utilizes the full width of the reinforced soil mass, which is taken as two-thirds of the height of the wall. For an 8 m high wall, the width of the reinforced mass is about 5.3 m. For the facing footing and reinforced soil mass (assuming the facing footing is placed on a compacted Granular 'B' pad), a factored geotechnical resistance at ULS of 300 kPa and a geotechnical resistance at SLS of 200 kPa may be used for design.

5.2.2 Resistance to Lateral Loads

The resistance to lateral forces / sliding resistance between the concrete or 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. The coefficient of friction, $\tan \phi'$, between the compacted granular fill of the RSS wall and the compacted Granular 'B' embankment fill may be taken as 0.70. This represents an unfactored value; in accordance with the CHBDC, a factor of 0.8 is to be applied in calculating the horizontal resistance.

5.2.3 Frost Protection

The concrete wing wall footings should be provided with a minimum of 1.2 m of soil cover for frost protection. Frost protection is not required for the RSS walls.

5.3 Steel H-Pile Foundations

Steel H-piles driven to found in the hard clayey silt till or very dense sandy silt till may be used for support of the pier, abutments and/or concrete wing walls. It is not considered practical to drive the piles to the surface of the bedrock due to the density of and the presence of numerous cobbles and boulders within the sandy silt (Wentworth) till deposit overlying the bedrock.

It is assumed that the abutment pile caps will be constructed at or about the original ground surface or alternatively, perched within the embankment. The pier pile cap will be constructed below the original ground surface. For design, the following pile tip levels may be assumed for piles terminated within the till (assumed minimum 2 m penetration). There should be provision made in the contract for dealing with pile lengths varying from the design.

<i>Foundation Location</i>	<i>Relevant Boreholes</i>	<i>Design Pile Tip Elevation (m)</i>
South Abutment	BR8-2	68
Central Pier	(See Note 1)	66.5
North Abutment	3	65

1. Borehole drilling was not carried out at the central median; pile tip elevation estimated by interpolation between the north and south abutments.

5.3.1 Axial Geotechnical Resistance

For HP 310 x 110 piles driven to practical refusal into the hard clayey silt / very dense sandy silt till deposits, a factored axial resistance at Ultimate Limit States (ULS) of 1,400 kN may be assumed for design. The axial geotechnical resistance at Serviceability Limit States (SLS) may be taken as 1,100 kN. For the above pile capacities, the piles must be driven to at least the elevations given in the table above.

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 piles should be stiffened with MTO flange plates for protection during driving in accordance with OPSD 3301.00 and OPSS 903.07.05.04. The criteria must therefore be established at the time of construction after the piling equipment is known. The following note is considered appropriate for the design and site conditions assuming a resistance factor of 0.4 is applied to the use 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 68 m (south abutment), below EL 66.5 m (central pier) and below EL 65 m (north abutment).”

5.3.2 Downdrag Load (Negative Skin Friction)

The embankment loading will cause consolidation settlement of the underlying soft to firm 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 soft to firm 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. The estimated unfactored downdrag load acting on the HP 310x110 piles may be taken as 200 kN per pile at the north abutment location and 100 kN per pile at the south abutment location.

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 areas, the use of lightweight fill as backfill or by sub-excavation of soft to firm clayey silt till as discussed in Section 5.6.

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).
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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>
North Abutment (Boreholes 3, RW8-1)	Soft to firm clayey silt till	Between Elev. 78.0 and 74.0 m	20
		Between Elev. 74.0 and 73.0 m	40
	Stiff to hard clayey silt till	Below Elev. 73 m	100
South Abutment (Borehole BR8-1)	Firm clayey silt till	Between Elev. 75 and 73.0 m	40
	Stiff to hard clayey silt till	Below Elev. 73 m	100

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 $d = \text{Pile Diameter}$</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 or the very dense sandy silt till for support of the bridge. Due to the difficulties anticipated with extending the caissons to/into the bedrock as a result of the number of boulders within the sandy silt till and the highly variable condition of the shale bedrock, the use of caissons socketted into the bedrock is not recommended. If consideration is being given to the use of caissons, a design base level at Elevation 65 m (within the overlying till deposits) may be used.

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 these assumptions, including that the base of each caisson will be inspected 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>	
	ULS	SLS
0.9	3,200 kN	2,800 kN
1.5	6,600 kN	4,500 kN

5.4.2 Downdrag Load (Negative Skin Friction)

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

<i>Caisson Diameter (m)</i>	<i>Unfactored Downdrag Load (kN)</i>	
	<i>North Abutment</i>	<i>South Abutment</i>
0.9	450	250
1.5	750	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 (including lightweight fill) meeting the specifications of Ontario Provincial Standard Specifications (OPSS) Granular 'A' or Granular 'B'

should be used as backfill behind the walls. The Granular 'A' or Granular 'B' 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.

- The lightweight fill particles are subject to crushing if over compacted. Therefore, careful construction control is required to achieve adequate compaction without crushing. The slag backfill should be placed in loose lifts of 300 mm and compacted by eight passes of a manually guided tamper in accordance with OPSS 206.07.
- 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	LIGHTWEIGHT SLAG FILL
Soil unit weight:	21 kN/m ³	14.5 kN/m ³
Coefficients of static lateral earth pressure:		
Active, K_a	0.33	0.36
At rest, K_o	0.50	0.53

- 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	LIGHTWEIGHT SLAG FILL
Soil unit weight:	22 kN/m ³	21 kN/m ³	14.5 kN/m ³
Coefficients of static lateral earth pressure:	0.27	0.31	0.36
Active, K _a	0.43	0.47	0.53
At rest, K _o			

- 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 90 m to 90.5 m at the north and south approaches, respectively. The existing ground surface at the centreline of the bridge varies from about Elevation 79.5 m to 81 m resulting in approach embankments up to about 11 m in height. The ditch along the north side of the QEW has a base level as low as Elevation 78.5 and the ground surface to the northwest of the site dips to Elevation 77.5 m. RSS retaining walls / wing walls are proposed within the approach embankment 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 embankment are greater than 8 m in height, 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.

It is our understanding that for embankments greater than 1.2 m in height, it is not normal practice to remove any topsoil, organic matter and softened / loosened soils from below the approach embankment areas. However at this site, given the stability concerns discussed below, it will be necessary to ensure 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 (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 and base courses should be compacted to 100 per cent of the Standard Proctor maximum dry density. Lightweight fill should be placed and tested as described in Section 5.4. 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 - 100 kPa
Soft Clayey Silt Till (N. Abut)	19 kN/m ³	30°	20 – 35 kPa
Firm Clayey Silt Till (N.& S. Abut)	19 kN/m ³	30°	40 kPa
Stiff to Hard Clayey Silt Till	20 kN/m ³	30°	100 kPa
Hard Red Clayey Silt Till	22 kN/m ³	–	200 kPa
Sandy Silt Till	22 kN/m ³	35°	–

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. The undrained shear strength data from the boreholes at the Bridge 8 site is summarized on Figure 2 and formed the basis for the design profiles shown in the table above for the north and south abutments. At the south abutment, the clayey silt till has a firm consistency rather than soft to firm consistency as at the north abutment.

5.6.2.1 North Approach

The presence of the softened till at the north abutment and approach embankment poses stability concerns for the proposed embankment and retaining walls. In addition to a standard abutment with concrete wing walls, it is understood that consideration is being given to the use of a false integral abutment with RSS wall configuration.

The stability analyses indicate that a Factor of Safety of greater than 1.3 cannot be achieved for the proposed embankment heights (to Elevation 90.5 m) without the use of toe berms where earth fill is used for the embankment construction. It is understood that space restrictions preclude the use of toe berms at this site. In addition, a Factor of Safety of greater than 1.3 cannot be achieved for the proposed wall heights (to Elevation 90.5 m) at this site. The following table outlines the maximum embankment heights with 2 horizontal to 1 vertical (2H:1V) side slopes and maximum wall heights that are possible using earth fill or lightweight fill in order to achieve the target factor of safety of 1.3 for global stability (see Figures 3 and 4 for details).

<i>Fill Type</i>	<i>Unit Weight (kN/m³)</i>	<i>Maximum Embankment Height (above El. 80.m5)</i>		<i>Maximum Retaining Wall Height (above El. 80.5m)</i>	
		<i>Height (m)</i>	<i>Elevation (m)</i>	<i>Height (m)</i>	<i>Elevation (m)</i>
Earth Fill	21	5	85.5	5	85.5
Lightweight Fill	14.5	9	89.5	8	88.5

Since the appropriate factor of safety cannot be achieved for the full embankment/wall heights (to Elevation 90.5 m), consideration could be given to the use of EPS fill in a portion of the embankment. The following table gives the embankment fill types and the EPS thicknesses that would be required to achieve the target factor of safety of 1.3 for global stability.

<i>Fill Type</i>	<i>Unit Weight (kN/m³)</i>	<i>Required EPS Thickness (m)</i>
Earth Fill	21	5.0
Lightweight Fill	14.5	1.5

The EPS should be placed between the wing walls and extended away from the abutments until the full toe berm requirement (for side stability) is in place. 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 conventional fill / pavement structure cover and not more than 1.8 m in order to reduce the chance of freezing/icing on the road surface.

Alternatively, consideration could be given to full or partial sub-excavation of the soft to firm clayey silt till in order to achieve stability. Sub-excavation to Elevation 75 m would involve removal of the majority of the soft clayey silt till, leaving 2 m of the firm clayey silt till in place. In this case, the excavations would be up to about 6 m deep. Sub-excavation to Elevation 73 m would involve removal of all of the soft to firm clayey silt till. In this case, the excavations would be up to about 8 m deep. The following table outlines the maximum embankment heights with 2 horizontal to 1 vertical (2H:1V) side slopes and maximum wall heights that are possible using earth fill or lightweight fill in order to achieve the target factor of safety of 1.3 for global stability for the two subexcavation alternatives.

<i>Subexcavation Elevation (m)</i>	<i>Fill Type</i>	<i>Maximum Embankment Height (above El. 80.5m)</i>		<i>Maximum Retaining Wall Height (above El. 80.5m)</i>	
		<i>Height (m)</i>	<i>Elevation (m)</i>	<i>Height (m)</i>	<i>Elevation (m)</i>
75.0	Earth Fill	7	87.5	7	87.5
	Lightweight Fill	10	90.5	10	90.5
73.0	Earth Fill	10	90.5	10	90.5

It should be noted that if the subexcavation to Elevation 75 m is extend to the toe of the earth fill embankment, the embankment can be constructed higher. The subexcavation assumed for the analysis is extended only to just beyond the ultimate road shoulder.

For the subexcavation alternative, the excavated material would have to be replaced with compacted granular fill or compacted earth fill in conformance with the specifications given in the earlier sections. The limits of the subexcavation would have to cover an area delineated by a line extended from the base of the walls outwards at a 1H:1V slope. The excavation cut slopes should be made no steeper than 1.5H:1V. Requirements with respect to temporary excavations should be as described in Section 5.6.

5.6.2.2 South Approach

The analyses indicate that a factor of safety of greater than 1.3 against deep-seated slope instability is obtained for the proposed 10 m high approach embankments (see Figure 5) with side slopes at 2 horizontal to 1 vertical (2H:1V).

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 softened clayey silt till deposits encountered at the approach

embankments. 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, granular or lightweight slag fill, the settlement of the new embankment fill itself is expected to be less than 25 mm. 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 embankments is dependant on the thickness and the undrained shear strength of the clayey silt and is discussed in the following sections.

5.6.3.1 North Approach

The anticipated settlements for the feasible embankment / wall configurations described above are given in the table below:

<i>Embankment / Wall Option</i>	<i>Maximum Anticipated Settlement (mm)</i>
Subexcavation to El. 75 m, RSS Wall using lightweight fill	50
Subexcavation to El. 73 m, RSS Wall using lightweight fill	< 25
Subexcavation to El. 73 m, RSS Wall using earth fill	< 25
No subexcavation, 1.5 m EPS, lightweight fill	75
No subexcavation, 5 m EPS, earth fill	75

5.6.3.2 South Approach

At the south approach embankment, a thin layer of firm clayey silt till deposit was encountered in Borehole BR8-1. This firm portion of the deposit was not encountered in Boreholes 2 and BR8-2 which are also located in the area of the south approach. At the south approach embankment, it is calculated that the settlements induced by the embankment loading (embankment up to 10 m) would be up to 150 mm.

5.6.4 Mitigation of Time Dependant Settlement and Stability

Time dependent, post-construction settlements of the new embankments are expected as a result of consolidation of the underlying soft to firm 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. A summary of the options are given in Table 2.

5.6.4.1 North Approach

5.6.4.1.1 Preloading

In order to reduce the post-construction settlement, preloading of the north approach embankment could be considered; however, due to stability concerns as discussed above, the preload embankments constructed with 2H:1V side slopes and without toe berms are restricted to the heights given in the table below.

<i>Embankment / Wall Option</i>	<i>Maximum Preload Height (above El 80.5m)</i>
Subexcavation to El. 75 m, RSS Wall using lightweight fill	7 m (El. 87.5)
Subexcavation to El. 73 m, RSS Wall using lightweight fill	Not required
Subexcavation to El. 73 m, RSS Wall using earth fill	Not required
No subexcavation, 1.5 m EPS, lightweight fill	9 m (El. 89.5)
No subexcavation, 5 m EPS, earth fill	5 m (El 85.5)

After the preload period, the embankment height would have to be reduced such that the appropriate amount of EPS could be placed for stability considerations. The extent of embankment excavation and EPS placement would depend on the type of embankment fill used. Where RSS walls are proposed, the location/feasibility of EPS is extremely limited. It is expected that 90 percent of the settlement will occur within 12 months after loading.

The placement of a 2 m surcharge would reduce the magnitude of this post-construction settlement; however, due to the stability issues noted above and the space restrictions for preload embankments, it will not be possible to place a surcharge load in the area of the abutments.

For the preloading option, 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 limited elastic compression of the piles proposed at this site, the full downdrag loads (as given in Sections 5.3.2 and 5.4.2) will have to be considered in the structural design of the piles.

5.6.4.1.2 Wick Drains

Consideration could be given to the use of wick drains at the north approach embankment in combination with preloading in order to reduce the time required for 90 percent consolidation to occur. In this case, it is estimated that 90 percent of consolidation will occur in about 3 months.

In order to install wick drains at this site, pre-drilling through the upper compact to dense silty sand to sandy silt and stiff to hard clayey silt will have to be carried out. In addition to the regular monitoring requirements, additional instrumentation will have to be installed to permit monitoring of the effectiveness of the wick drains. The variable thickness of the soft clay impacts the effectiveness of the wick drains. If a 6 to 9 month period were available, staged construction in conjunction with wick drains could be considered to minimize the use of lightweight fill/EPS fill.

5.6.4.1.3 Subexcavation

Alternatively, consideration could be given to full or partial sub-excavation of the soft to firm clayey silt till under the north approach/abutment in order to achieve stability as outlined in Section 5.6.2.1. Sub-excavation could be carried out to Elevation 75 m (partial subexcavation) or 73 m (full subexcavation) resulting in excavations up to about 6 m and 8 m deep, respectively. The requirements with respect to maximum cut slope profiles and groundwater control should be in accordance with Section 5.7.

Given the uncertainties with respect to settlement and the complexity of the work (preload/subexcavation/replacement with EPS) it is considered that the subexcavation alternative is the most technically feasible option for dealing with both the stability and settlement concerns at the north approach. Which subexcavation option is most feasible (to El. 75 m or El. 73 m) will depend on space restrictions and excavation requirements.

If subexcavation is chosen as the preferred alternative for mitigating settlement and stability, 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.

5.6.4.2 South Approach

5.6.4.2.1 Preloading

Preloading of the south approach embankment area will be necessary in order to limit post-construction settlements as well as differential settlements across the width of the embankment. It is expected that 90 percent of this settlement will occur within 6 months after loading.

If the south approach embankment areas are preloaded, the total post-construction settlement beneath the wing walls will be reduced to below 25 mm at the south abutments. Therefore, placement of a surcharge would not be required to reduce the magnitude of this post-construction settlement any further.

If the preloading 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.

5.6.4.2.2 Wick Drains

In order to reduce the preload period, consideration could be given to the use of wick drains at the south abutment. However, due to the substantial variability in lateral extent and limited thickness of the soft to firm clayey silt till, wick drains may not be practical for the south approach. In addition, if the proposed construction schedule permits the full preload period of 6 months, then the use of wick drains to reduce the preload period may not be required.

5.6.4.2.3 Subexcavation

Alternatively, consideration could be given to sub-excavation of the firm clayey silt till at the south abutment to eliminate the post-construction settlement. The compressible material would have to be excavated to a minimum of Elevation 72.7 m which would require excavations up to 8 m deep. However, since the thickness and lateral extent of this material at the south abutment is extremely variable and due to the depth of the required excavation in close proximity to the existing road (probably requiring roadway protection), sub-excavation is not considered to be a feasible/economical option.

If subexcavation was chosen as the preferred alternative for mitigating settlement and stability at the south 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

Assuming that the abutments will be perched within the embankments, temporary excavations required for construction will be for the pier pile cap and the subexcavation at the north abutment. The excavations will typically be extended through fill, silty sand to sandy silt and into the soft to firm clayey silt till. If the base of the excavation is maintained within the soft to firm clayey silt till, measures may be required to provide suitable working conditions since the deposit is “softened” and highly susceptible to disturbance due to construction traffic, ponded water, etc. The contractor should be aware that trafficking over the exposed clayey material may not be possible and an Operational Constraint should be included in the contract in this regard.

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 sandy silt to silty sands and clayey silt at the site are classified as Type 3 soil, according to the OHSA. The soft to firm clayey silt till (may be encountered for the sub-excavation), classified as Type 4 soil. 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 1.5 horizontal to 1 vertical 1.5H:1V.

The water level in the piezometers ranged between about Elevations 72 m and 78.6 m, typically between about 2 m and 9 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 as well as space restrictions 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




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TABLES

TABLE 1
EVALUATION OF FOUNDATION ALTERNATIVES
BRIDGE 8, 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 on stiff to hard till	X		Low geotechnical resistance. Excavation through fills, surficial deposits and may increase depth of excavation required. Groundwater control may be required.		Differential settlements would be anticipated between abutments and piers.
Steel H Piles driven through stiff to hard clayey silt till just into very dense sandy silt till		Minimized hard driving through bouldery till deposit.	Lower capacity than piles on bedrock.	Lower relative costs than piles driven to bedrock.	Low risk
Steel H Piles driven to shale bedrock	X	Increased capacity over piles terminated in overburden. Differential settlement between abutment/pier foundations minimized.	Difficulties anticipated driving through bouldery till deposit; high likelihood that pre-drilling would be required.	Relative costs of driving piles through bouldery deposit less expensive than augering for caissons; however increased costs for pre-drilling if piles "hang-up".	High likelihood that piles could "hang-up" in bouldery till deposit. Pile locations would require pre-drilling to permit pile installation.
Caissons socketted into very dense sandy silt till just below stiff to hard clayey silt till		Minimized difficult augering through bouldery till deposit.	Lower capacity than caissons socketted into bedrock. Temporary liners may be required for groundwater control.		
Caissons socketted into shale bedrock	X	Differential settlement minimized. Higher bearing capacities than spread footings on native soils.	Extreme difficulty may be encountered augering through bouldery till deposit. Temporary liners required for groundwater control. Socketting into bedrock may require rock coring or churn drilling techniques.	Increased cost of augering through bouldery deposit and socketting into bedrock. Extra costs associated with liners and inspection.	Cost of advancing augers through bouldery till deposit may be prohibitive. Difficulty may be encountered socketting liner in till deposit to seal off water; downhole inspection may not be possible.

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

TABLE 2
MITIGATION OPTIONS FOR IMMEDIATE APPROACH EMBANKMENTS AT BRIDGE ABUTMENTS
RED HILL CREEK EXPRESSWAY INTERCHANGE
G.W.P. 441-97-00

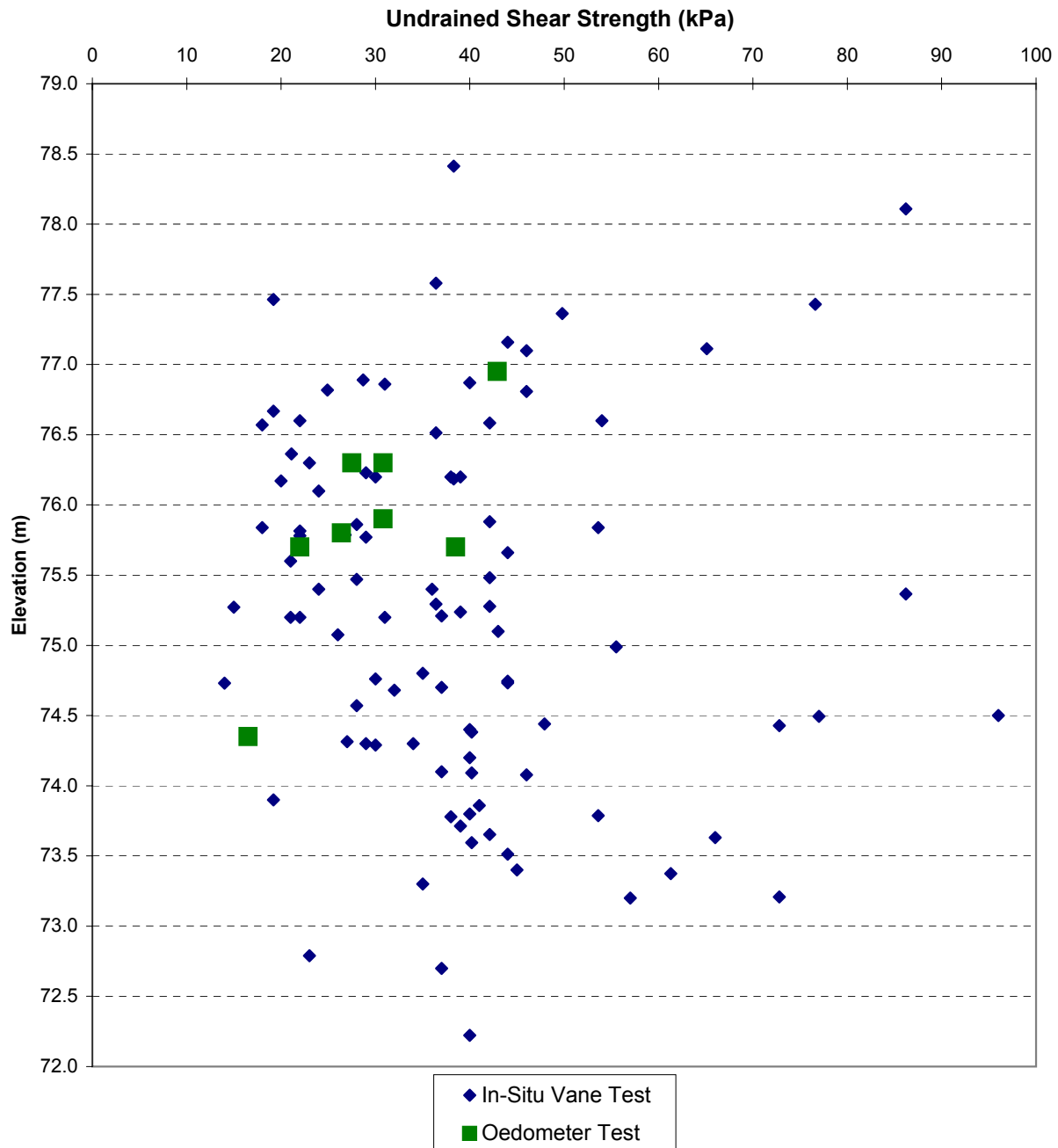
<i>Subsoils</i>	<i>Embankment Height</i>	<i>Preloading/Lightweight Fill</i>	<i>Subexcavation</i>	<i>Wick Drains</i>
North Abutment: 4 to 6 m thick deposit of soft to firm clayey silt till with base at depths of 6 to 8 m	13 m (effective height)	Site constraints and construction schedule may not permit required 12 month preload period. Lightweight fill required for stability and to reduce settlement but would not eliminate need for preload. Stabilizing toe berms required.	Would eliminate need for berms as well as minimize long term settlement, depending on depth of subexcavation. Deep excavations required.	Wick drains would accelerate settlement and reduce preloading period. Berms required for stability.
South Abutment: 0 to 2 m thick deposit of firm clayey silt till with base at depth of 8 m	10 m	Preloading 6 months recommended as a precaution since compressible layer is limited and variable in extent. Berms not required for stability.	Substantial excavation of competent soils overlying compressible deposit which is variable in thickness makes this option less desirable.	Variability of thickness and lateral extent of compressible layer makes wick drains less effective. Additional costs to penetrate overlying more competent soils.

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FIGURES

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

FIGURE 1



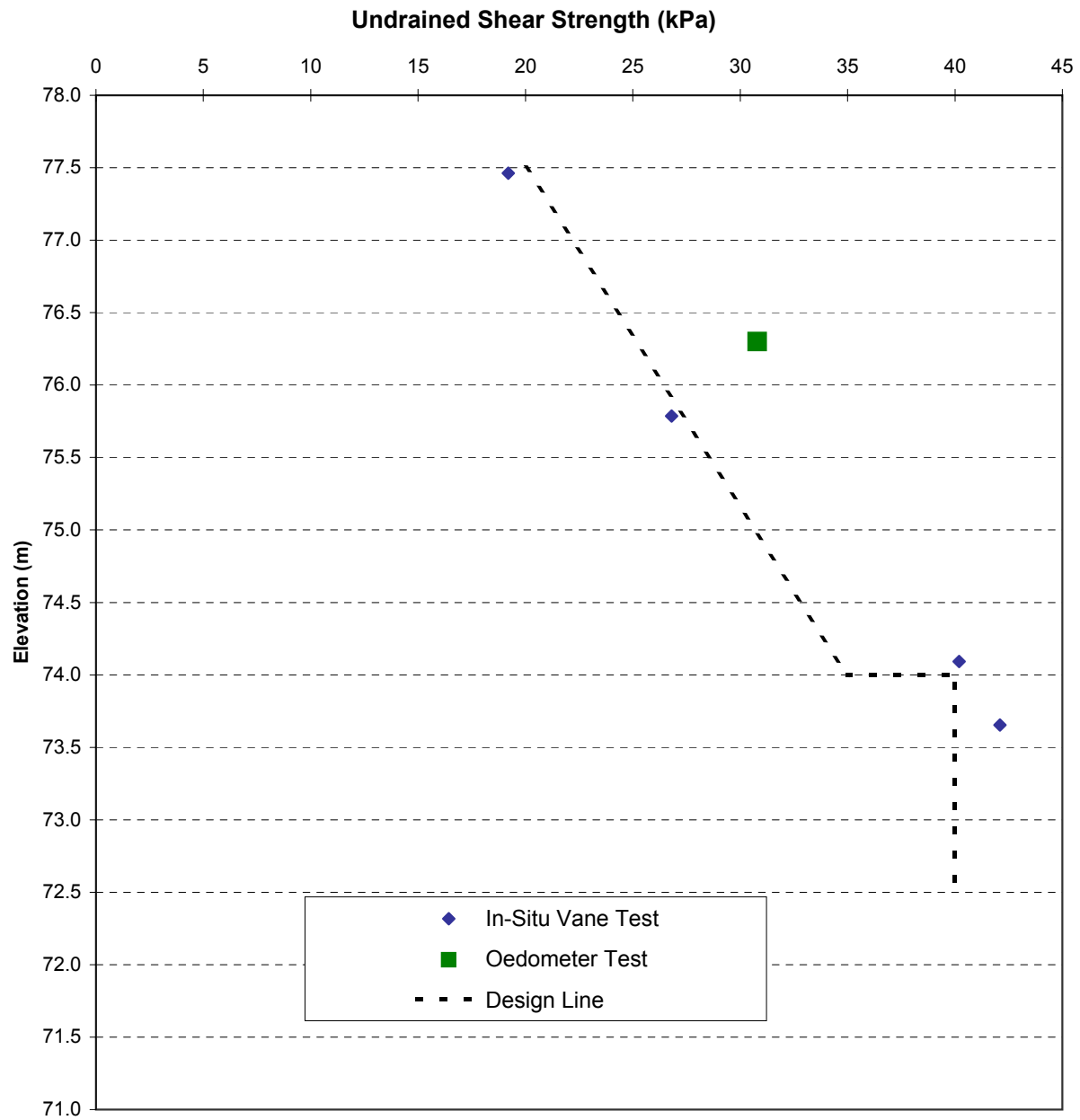
Date: June 2004
Project: 021-1162-BR8

Golder Associates

Drawn: SEP
Checked: JPD

**SHEAR STRENGTH SUMMARY AND DESIGN LINE
BRIDGE 8 - NORTH ABUTMENT**

FIGURE 2



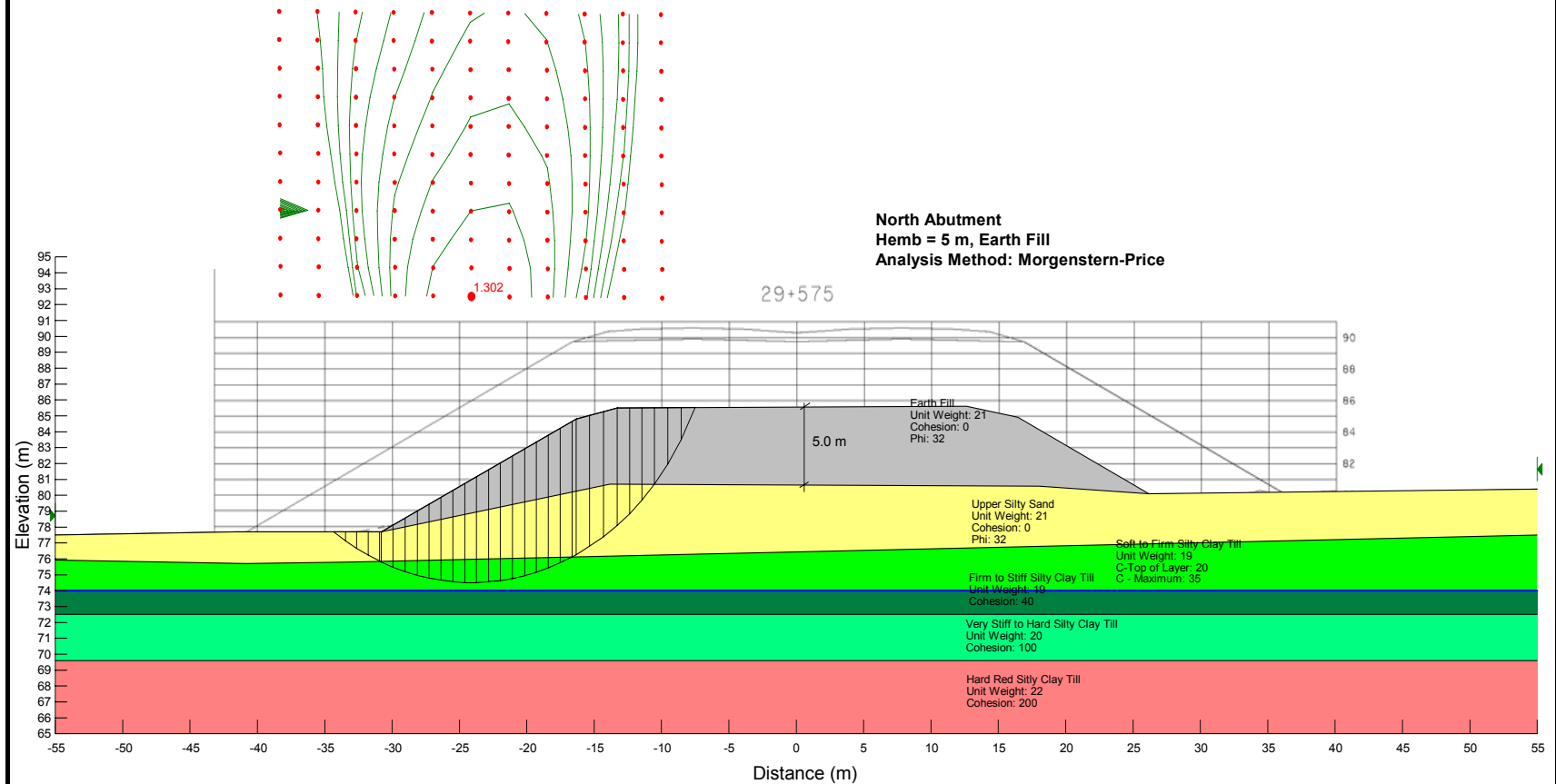
Date: June 2004
Project: 021-1162-BR8

Golder Associates

Drawn: SEP
Checked: JPD

APPROACH EMBANKMENT STABILITY ANALYSIS NORTH ABUTMENT - EARTH FILL OPTION

FIGURE 3



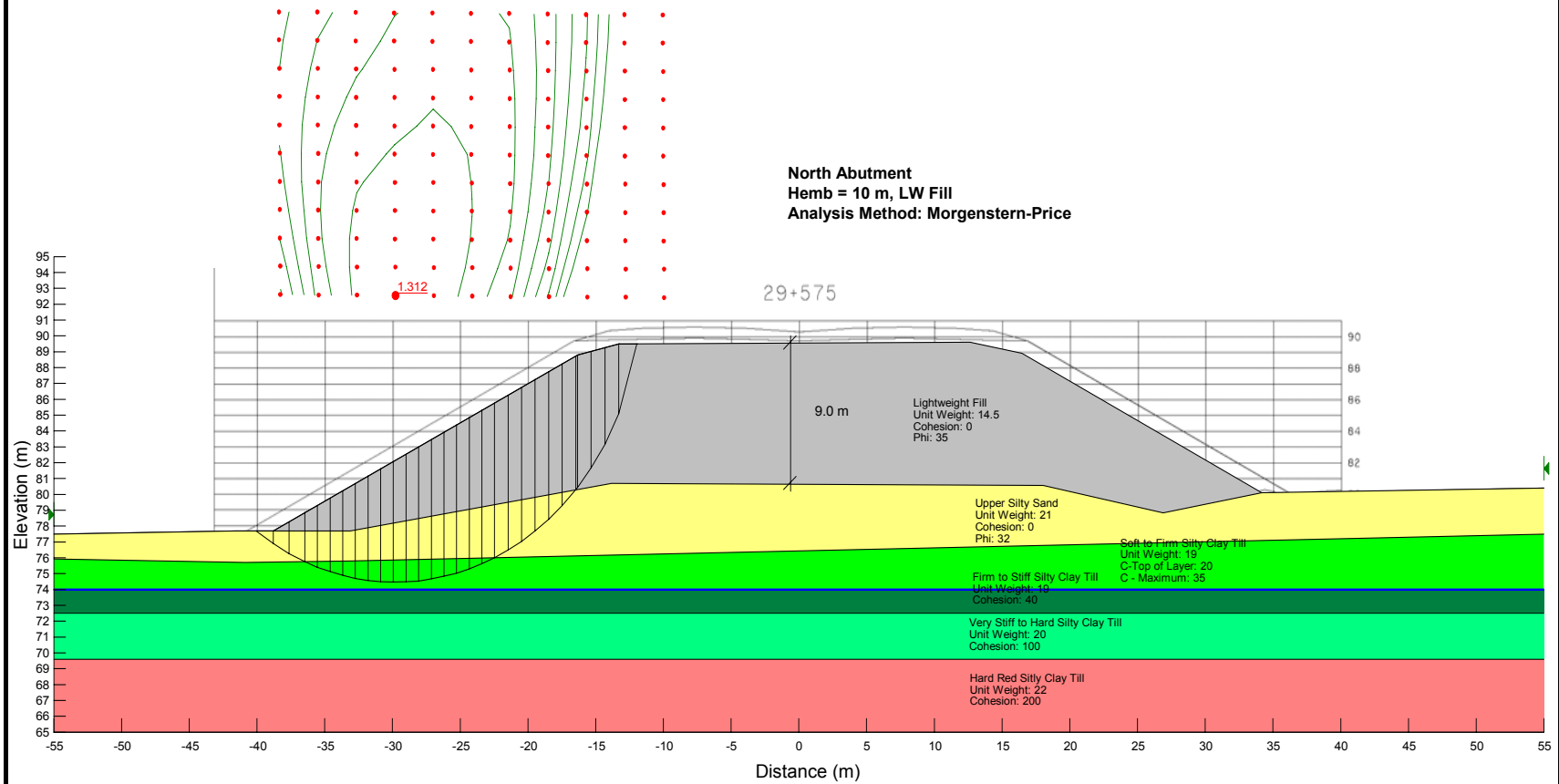
Date: June 2004
Project: 021-1162-BR8

Golder Associates

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APPROACH EMBANKMENT STABILITY ANALYSIS NORTH ABUTMENT - LIGHTWEIGHT FILL OPTION

FIGURE 4



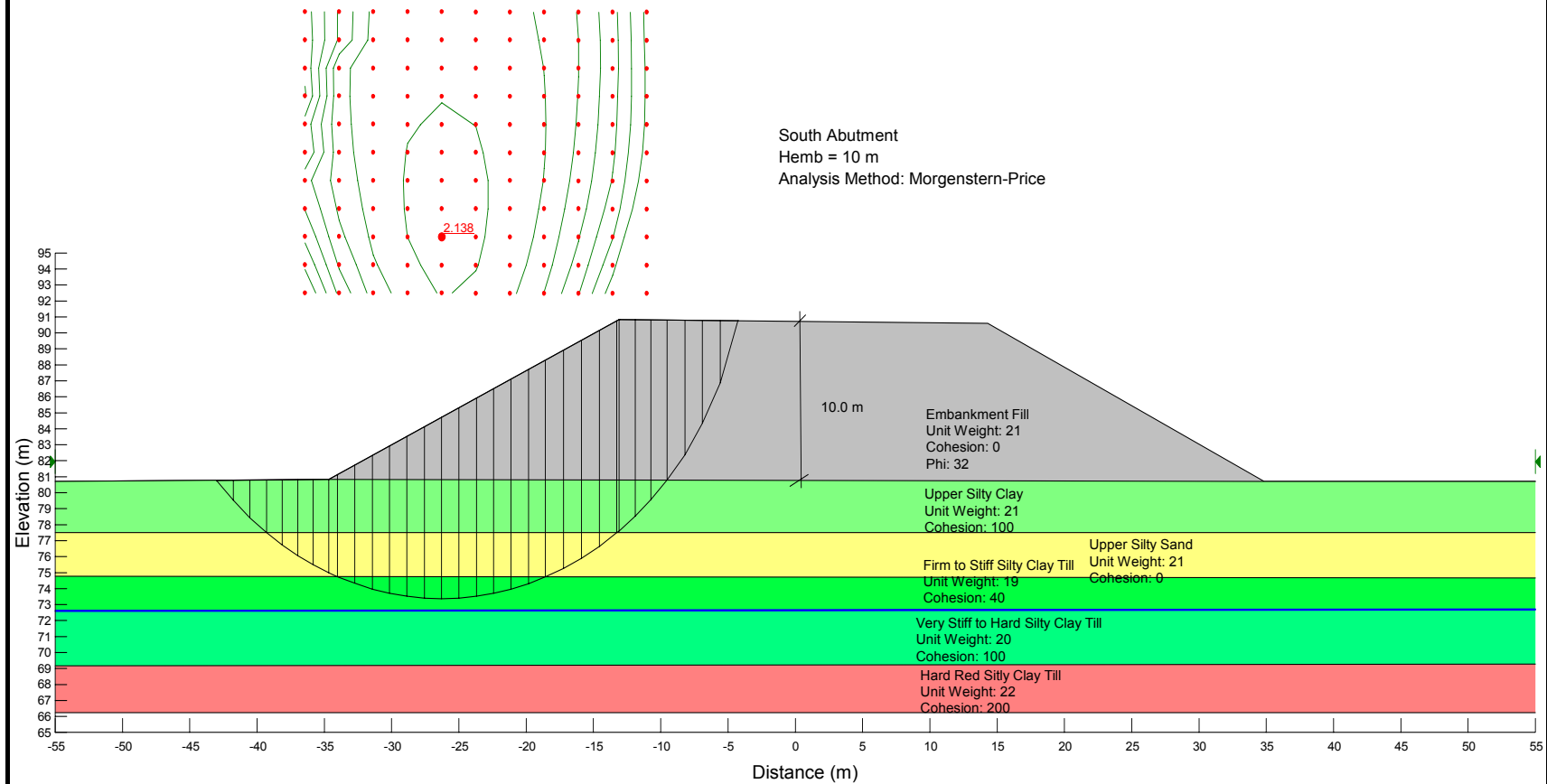
Date: June 2004
 Project: 021-1162-BR8

Golder Associates

Drawn: SEP
 Checked: JPD

APPROACH EMBANKMENT STABILITY ANALYSIS SOUTH ABUTMENT

FIGURE 5

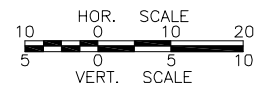
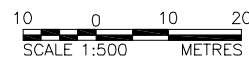


Date: June 2004
Project: 021-1162-BR8

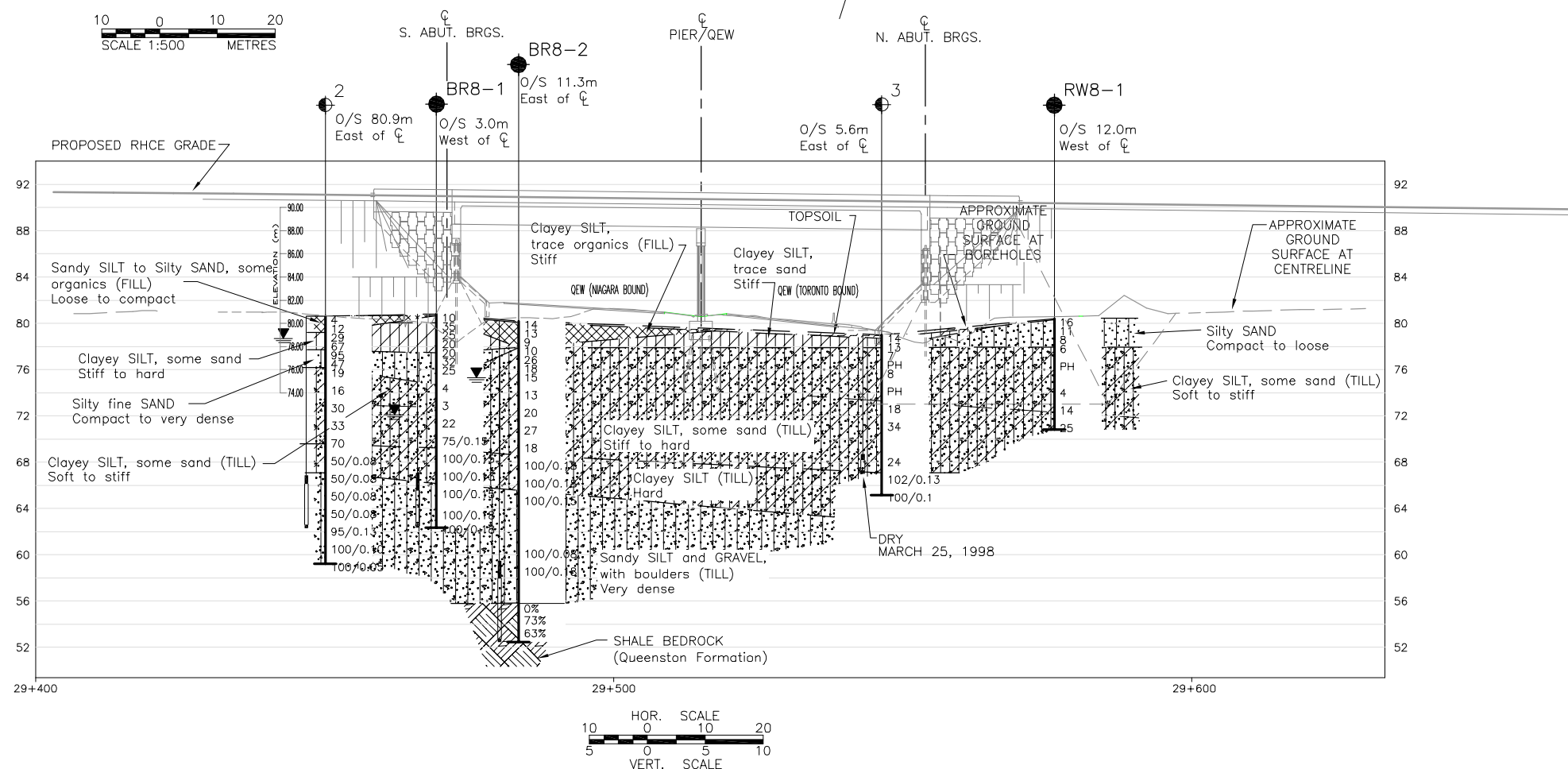
Golder Associates

Drawn: SEP
Checked: JPD

DRAWING



A-A' SOUTH ABUTMENT CENTERLINE



SECTION/PROFILE ALONG CENTRELINE






Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA

METRIC

DIMENSIONS ARE IN METRES AND/OR
MILLIMETRES UNLESS OTHERWISE SHOWN



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
2	80.6	4789861.9	283541.9
3	79.0	4789890.0	283632.9
BR8—1	80.8	4789877.6	283556.7
BR8—2	80.2	4789868.6	283574.6
RW8—1	80.4	4789912.4	283659.4

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 Delcan, rhce 8-GA.dwg, received June 2004.

NO.	DATE	BY	REVISION

Geocres No.			
HWY. QEW		PROJECT NO. 021-1162	
SUBM'D.		DIST.	
CHKD. SEP		DATE: JUNE 2004	
DRAWN: JDR		SITE:	
CHKD. ASP		APPD. FJH	
		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

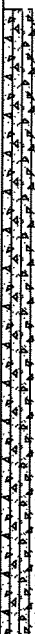
B - Bedding	P - Polished
FO - Foliation/Schistosity	S - Slickensided
CL - Cleavage	SM - Smooth
SH - Shear Plane/Zone	R - Ridged/Rough
VN - Vein	ST - Stepped
F - Fault	PL - Planar
CO - Contact	FL - Flexured
J - Joint	UE - Uneven
FR - Fracture	W - Wavy
MF - Mechanical Fracture	C - Curved
- Parallel To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

PROJECT 021-1162		RECORD OF BOREHOLE No 2		1 OF 2		METRIC				
W.P. 441-97-00		LOCATION N 4789861.9 ; E 283541.9		ORIGINATED BY GD						
DIST _____ HWY QEW		BOREHOLE TYPE Power Auger 108mm I.D. Hollow Stem Auger		COMPILED BY KG						
DATUM Geodetic		DATE February 12, 1998		CHECKED BY SEP						
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	PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT W _p — W — W _L WATER CONTENT (%)	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE						
80.6 0.0	GROUND SURFACE Sandy SILT to Silty SAND, some rootlets and organics (FILL) Loose to compact Dark brown to brown Wet to moist		1	SS	4					
79.2 1.4	Clayey SILT, some sand, trace gravel, trace oxidized stains, occasional sand seams Very stiff to hard Mottled grey and brown Moist		2	SS	12					
77.7 2.9	Silty fine SAND Very dense to dense Brown Dry to moist		3	SS	29					
76.2 4.4	Clayey SILT, some sand, trace to some gravel (TILL) Very stiff to hard Grey becoming reddish-grey Moist		4	SS	67					
69.6 11.0	Clayey SILT, some sand, trace gravel, with weathered shale and limestone fragments (TILL) Hard Reddish-brown Dry		5	SS	95					
67.1 13.5	Sandy SILT, some gravel, trace clay, shale and limestone cobbles and boulders (TILL) Very dense Reddish-brown Dry to moist		6	SS	47					
			7	SS	19					
			8	SS	16					
			9	SS	30					
			10	SS	33					
			11	SS	70					
			12	SS	50/10/0					
			13	SS	50/10/0					

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Continued Next Page

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

PROJECT <u>021-1162</u>		RECORD OF BOREHOLE No 2		2 OF 2		METRIC					
W.P. <u>441-97-00</u>		LOCATION <u>N 4789861.9; E 283541.9</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 12, 1998</u>		CHECKED BY <u>SEP</u>							
SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT LIMIT		UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER			TYPE	"N" VALUES	20 40 60 80 100	20 40 60 80 100		
--- CONTINUED FROM PREVIOUS PAGE ---											
	Sandy SILT, some gravel, trace clay; shale and limestone cobbles and boulders (TILL) Very dense Reddish-brown Dry to moist		14	SS	50/0/05						
65											
64											
63											
62											
			16	SS	95/0/13						
61											
			17	SS	00/0/10						
60											
59.2 21.4	End of Borehole		18	SS	00/0/0						
	Notes: 1. Water level in piezometer at Elev. 78.7 m immediately after installation (likely influenced by water used during drilling) 2. Water level in piezometer at Elev. 75.4 m on March 25, 1998 3. Water level in piezometer at Elev. 78.7 m on Oct. 22, 2003										

PROJECT 021-1162

RECORD OF BOREHOLE No 3

1 OF 2

METRIC

W.P. 441-97-00

LOCATION N 4789890.0 ; E 283632.9

ORIGINATED BY GD

DIST HWY QEW

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

COMPILED BY KG

DATUM Geodetic

DATE February 24, 1998

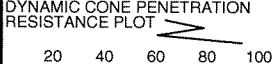
CHECKED BY SEP

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						
79.0	GROUND SURFACE							○ UNCONFINED + FIELD VANE						
0.0	Topsoil		1	SS	14			● QUICK TRIAXIAL × REMOULDED						
78.7														
0.3	Clayey SILT, trace sand Stiff Mottled brown Moist													
77.9			2	SS	13		70							
1.1	Clayey SILT, trace sand, occasional gravel, silt seams and parting throughout (TILL) Firm Grey Moist													
			3	SS	7		77							
			4	TO	PH									
			5	SS	8		70							
							75							
			6	TO	PH		74							
							73							
			7	SS	18									1 15 46 38
							72							
			8	SS	34		71							
							70							
							69							
			9	SS	24		60							
67.1							67							
11.9	Clayey SILT, some sand, trace gravel, with weathered shale and limestone fragments (TILL) Hard Reddish-brown Dry		10	SS	1020									
							66							
65.2			11	SS	1000									
13.8														

Continued Next Page

+³, X³: 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 3				2 OF 2		METRIC					
W.P. <u>441-97-00</u>		LOCATION <u>N 4789890.0 ; E 283632.9</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 24, 1998</u>				CHECKED BY <u>SEP</u>							
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa					
	--- CONTINUED FROM PREVIOUS PAGE ---												
	End of Borehole Notes: 1. Refusal to split spoon sampler advance 2. Borehole dry upon completion of drilling 3. Piezometer dry on March 25, 1998												

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

PROJECT 021-1162

RECORD OF BOREHOLE No BR8-1

1 OF 2

METRIC

W.P. 441-97-00

LOCATION N 4789877.6 :E 283556.7

ORIGINATED BY PKS

DIST HWY QEW

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

COMPILED BY KG

DATUM Geodetic

DATE August 18, 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 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						
80.8	GROUND SURFACE													
0.0	Sandy Topsoil													
0.2	Sandy SILT, some clay, trace gravel		1	SS	10									
80.0	Compact Brown Moist													
0.8	Clayey SILT, some sand and gravel, trace organics		2	SS	35									
	Very stiff to hard													
	Brown and grey		3	SS	25									
	Moist													
			4	SS	20									
77.5	Silty fine SAND, occasional silty clay seams		5	SS	20									
3.4	Compact to dense Grey Wet													
			6	SS	32									
			7	SS	25									
74.7	Clayey SILT, some sand, trace gravel (TILL)		8	SS	4									
6.1	Firm to stiff Grey Moist													
			9	SS	3									
72.7	Clayey SILT, some sand, trace gravel (TILL)													
8.1	Very stiff to hard Grey to reddish-grey Moist													
			10	SS	22									
			11	SS	75/0.15									
69.2	Clayey SILT, trace to some sand, trace gravel with shale and limestone pieces (TILL)													
11.6	Hard Reddish-brown Dry to Moist													
			12	SS	100/0.1									
			13	SS	100/0.1									
66.2														
14.6														

Continued Next Page

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

PROJECT <u>021-1162</u>		RECORD OF BOREHOLE No BR8-1		2 OF 2	METRIC
W.P. <u>441-97-00</u>		LOCATION <u>N 4789877.6 ; E 283556.7</u>		ORIGINATED BY <u>PKS</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>August 18, 2003</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/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa		W _p	W			W _L
	--- CONTINUED FROM PREVIOUS PAGE ---						<div><div>20406080100</div><div>○ UNCONFINED + FIELD VANE</div><div>● QUICK TRIAXIAL × REMOULDED</div></div>							
	Sandy SILT, some gravel, trace to some clay, trace gravel with shale and limestone fragments (TILL) Very Dense Red Moist		14	SS	00/0.15									
			15	SS	00/0.15									
			16	SS	00/0.15									
62.3 18.5	End of Borehole													
	Notes: 1. Open borehole dry upon completion of drilling 2. Borehole drilled 1 m west and shelly tube sample collected from 7.3 m to 7.9 m depth (Sample 8A) 3. Water level in piezometer at Elev. 70.8 m on Aug. 22, 2003 and at Elev. 72.1 m on Oct. 22, 2003													

PROJECT 021-1162		RECORD OF BOREHOLE No BR8-2		1 OF 2	METRIC
W.P. 441-97-00	LOCATION N 4789868.6 ; E 283574.6	ORIGINATED BY PKS			
DIST _____ HWY QEW	BOREHOLE TYPE Power Auger 108mm I.D. Hollow Stem Auger	COMPILED BY KG			
DATUM Geodetic	DATE August 13, 2003	CHECKED BY SEP			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	W _p W W _L				
80.2 0.0	GROUND SURFACE Sandy Topsoil Clayey SILT, some sand and gravel, trace to some organics (FILL) Stiff Brown Moist to wet		1	SS	14									
			2	SS	13									
			3	SS	9									
77.9 2.3	Clayey SILT, some sand, trace gravel (TILL) Stiff to very stiff Brown becoming grey at 6.1m depth Moist		4	SS	10									
			5	SS	26									
			6	SS	18									
			7	SS	15									
			8	SS	13									
			9	SS	20									
			10	SS	27									
68.6 11.6	Clayey SILT, some sand, trace gravel, with shale and limestone fragments (TILL) Hard Reddish-brown Moist		11	SS	18									
			12	SS	00/0.1									
		13	SS	00/0.1										
65.6 14.6														

MISS_MTO_0211162EAMTO.GPJ ON_MOT.GDT 18/6/04

Continued Next Page

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

PROJECT		021-1162		RECORD OF BOREHOLE		No BR8-2		2 OF 2		METRIC					
W.P.		441-97-00		LOCATION		N 4789868.6 ; E 283574.6		ORIGINATED BY		PKS					
DIST		HWY QEW		BOREHOLE TYPE		Power Auger 108mm I.D. Hollow Stem Auger		COMPILED BY		KG					
DATUM		Geodetic		DATE		August 13, 2003		CHECKED BY		SEP					
SOIL PROFILE		SAMPLES		GROUND WATER		ELEVATION SCALE		DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT		UNIT WEIGHT		REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	CONDITIONS	ELEVATION SCALE	20 40 60 80 100	20 40 60 80 100	Wp W WL	WATER CONTENT (%)	UNIT WEIGHT	GR SA SI CL		
--- CONTINUED FROM PREVIOUS PAGE ---															
	Sandy SILT, some gravel, trace clay; shale and limestone cobbles and boulders (TILL) Very dense Red Wet		14	SS	00/0.1		65								
	NQ Coring carried out between 16.3 m and 19.8 m depth.						64								
	Limestone and siltstone boulders throughout deposit based on core return.		15	RC			63								
			16	RC			62								
			17	RC			61								
			18	SS	00/0.1		60								
			19	SS	00/0.1		59								
	NQ Coring carried out between 22.6m and 24.4m depth.						58								
	Limestone and shale pieces within deposit based on core return. Fine gravel in wash return.		20	RC			57								
55.8 24.4	Moderately to highly weathered, reddish-grey, calcareous SHALE BEDROCK (Queenston Formation) with occasional grey limestone layers up to 100 mm thick						56								
	NQ Coring from 22.6m to 27.7m depth						55								
	For coring details see Record of Drillhole BR8-2						54								
52.5 27.7	End of Borehole						53								
Notes: 1. Auger and spoon refusal at 22.6m depth (Elev. 57.6m) 2. Water level in piezometer at Elev. 75.6 m on Aug. 22, 2003 and at Elev. 75.3 on Oct. 22, 2003															

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

PROJECT: 021-1162

RECORD OF DRILLHOLE: BR8-2

SHEET 1 OF 1

LOCATION: N 4789860.6; E 283582.6

DRILLING DATE: August 15, 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 (mm/min)	FLUSH % RETURN	COLOUR	FR/FX-FRACTURE F-FAULT CL-CLEAVAGE J-JOINT R-SMOOTH SH-SHEAR P-POLISHED R-ROUGH VN-VEIN S-SLICKENSIDED PL-PLANAR C-CURVED										FL-FLEXURED UE-UNEVEN W-WAVY C-CURVED				BC-BROKEN CORE MB-MECH. BREAK B-BEDDING				NOTES WATER LEVELS INSTRUMENTATION																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																
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8 8	8 8 8 8 8	8 8 8 8 8	8 8 8 8 8	8 8 8 8 8	8 8 8 8 8	8 8 8 8 8	8 8 8 8 8	8 8 8 8 8	8 8 8 8 8	8 8 8 8 8	8 8 8 8 8	8 8 8 8 8	8 8 8 8 8	8 8 8 8 8	8 8 8 8 8	8 8 8 8 8	8 8 8 8 8	8 8 8 8 8	8 8 8 8 8	8 8 8 8 8	8 8 8 8 8	8 8 8 8 8	8 8 8 8 8	8 8 8 8 8	8 8 8 8 8	8 8 8 8 8	8 8 8 8 8	8 8 8 8 8	8 8 8 8 8	8 8 8 8 8	8 8 8 8 8	8 8 8 8 8	8 8 8 8 8	8 8 8 8 8	8 8 8 8 8	8 8 8 8 8	8 8 8 8 8	8 8 8 8 8	8 8 8 8 8	8 8 8 8 8	8 8 8 8 8	8 8 8 8 8	8 8 8 8 8	8 8 8 8 8	8 8 8 8 8	8 8 8 8 8	8 8 8 8 8	8 8 8 8 8	8 8 8 8 8	8 8 8 8 8	8 8 8 8 8	8 8 8 8 8	8 8 8 8 8	8 8 8 8 8	8 8 8 8 8	8 8 8 8 8	8 8 8 8 8	8 8 8 8 8	8 8 8 8 8	8 8 8 8 8	8 8 8 8 8	8 8 8 8 8	8 8 8 8 8	8 8 8 8 8	8 8 8 8 8	8 8 8 8 8	8 8 8 8 8	8 8 8 8 8	8 8 8 8 8	8 8 8 8 8	8 8 8 8 8	8 8 8 8 8	8 8 8 8 8	8 8 8 8 8	8 8 8 8 8	8 8 8 8 8	8 8 8 8 8	8 8 8 8 8	8 8 8 8 8	8 8 8 8 8	8 8 8 8 8	8 8 8 8 8	8 8 8 8 8	8 8 8 8 8	8 8 8 8 8	8 8 8 8 8	8 8 8 8 8	8 8 8 8 8	8 8 8 8 8	8 8 8 8 8	8 8 8 8 8	8 8 8 8 8	8 8 8 8 8	8 8 8 8 8	8 8 8 8 8	8 8 8 8 8	8 8 8 8 8	8 8 8 8 8	8 8 8 8 8	8 8 8 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DEPTH SCALE

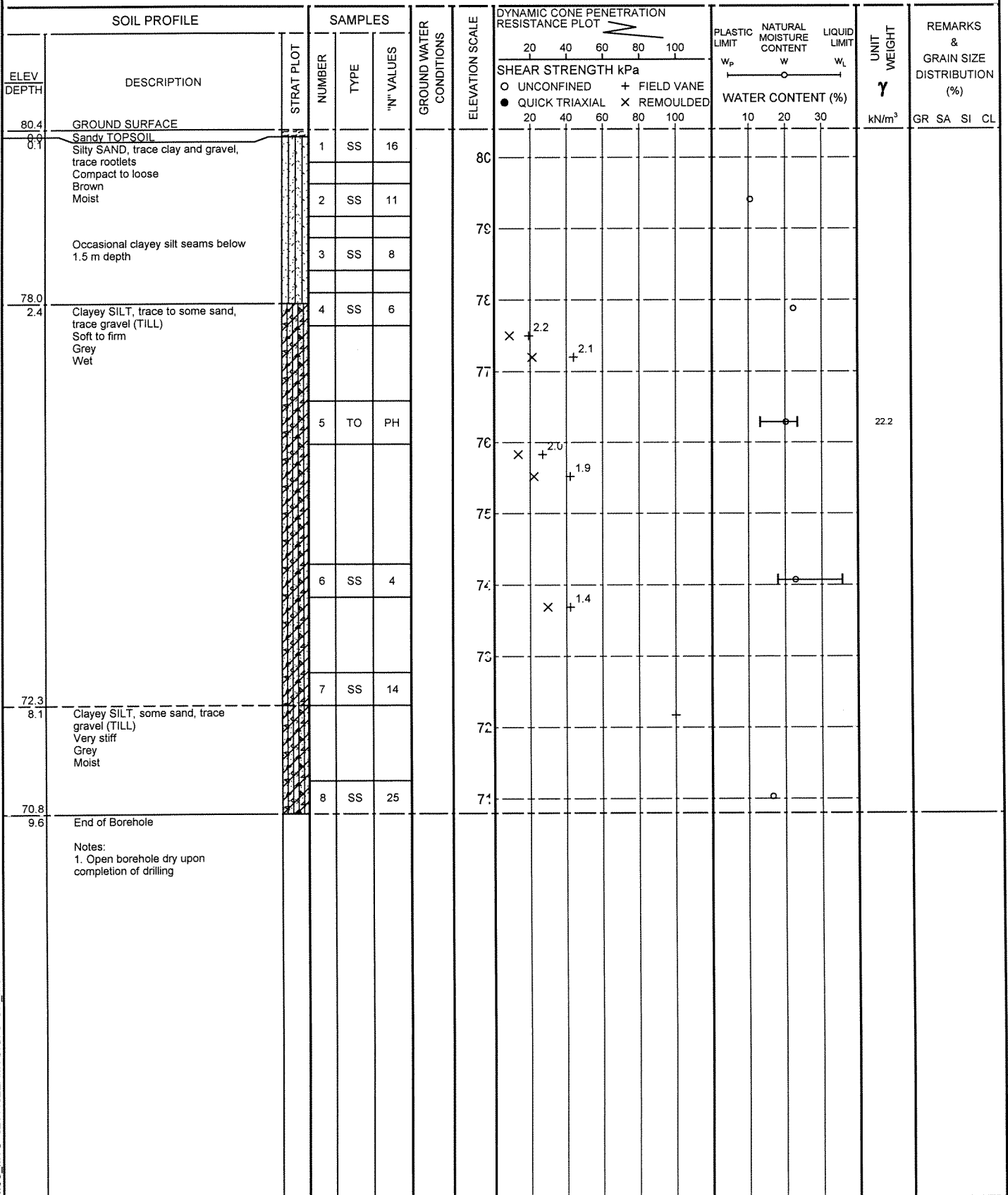
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LOGGED: PS

CHECKED: SEP

PROJECT <u>021-1162</u>		RECORD OF BOREHOLE No RW8-1		1 OF 1	METRIC
W.P. <u>441-97-00</u>	LOCATION <u>N 4789912.4 ; E 283659.4</u>	ORIGINATED BY <u>PKS</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>August 25, 2003</u>	CHECKED BY <u>SEP</u>			



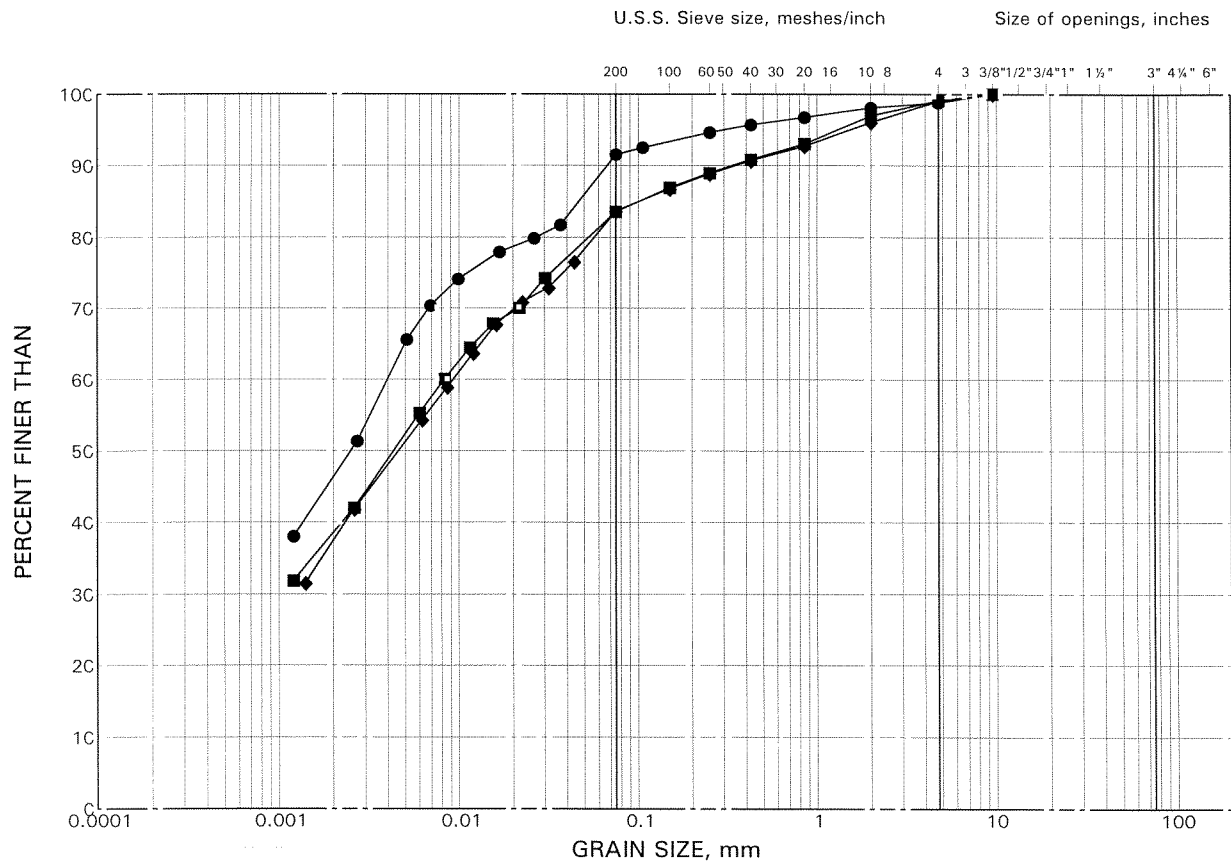
+³, X³ Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

APPENDIX A
LABORATORY TEST DATA

GRAIN SIZE DISTRIBUTION

Clayey Silt (Till)

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)
●	BR8-1	9	72.7
■	2	9	72.5
◆	3	7	72.4

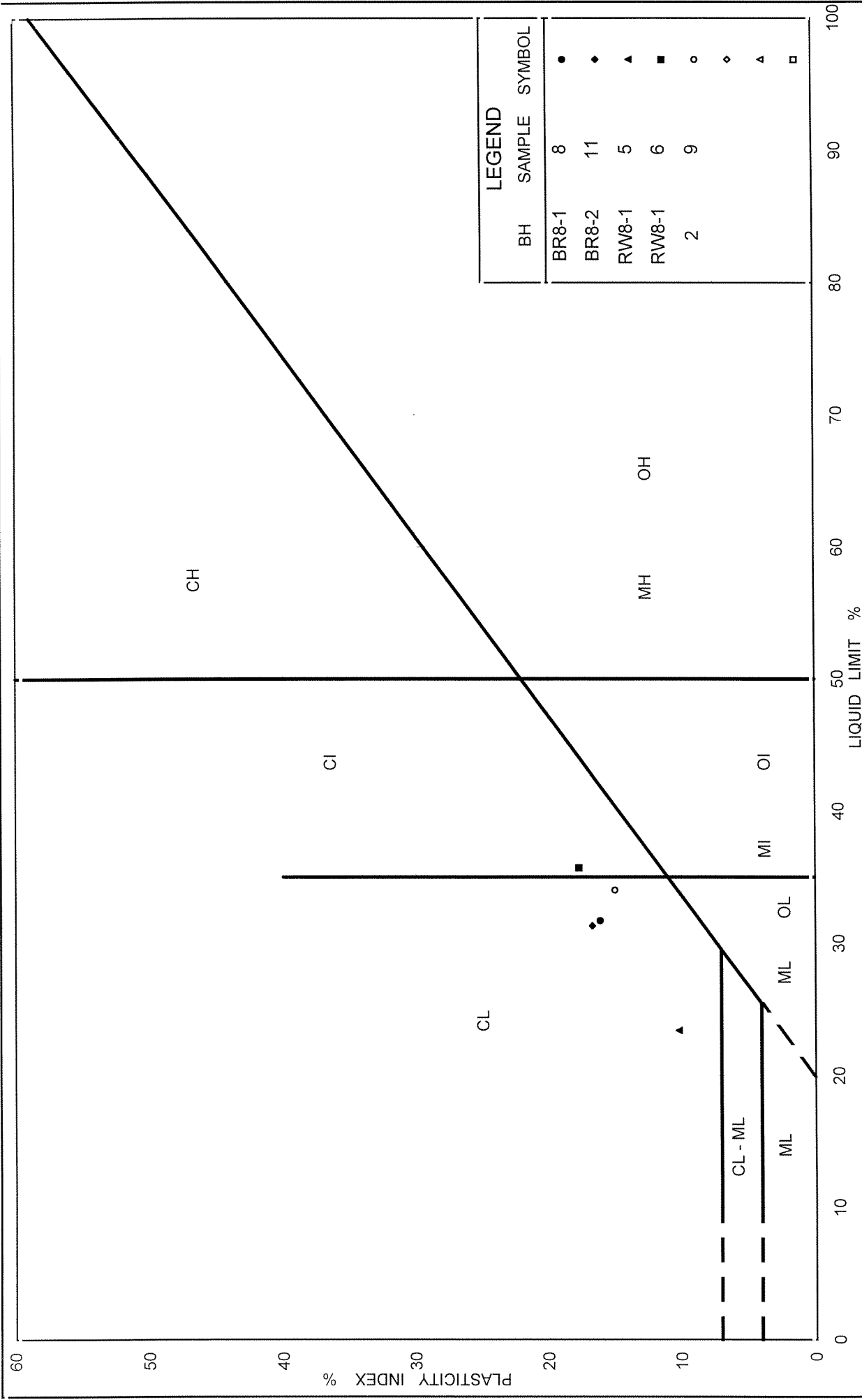


FIG No. A2

Project No. 021-1162-BR8

Ministry of Transportation



Ontario

OEDOMETER CONSOLIDATION SUMMARY

SAMPLE IDENTIFICATION

Project Number	021-1162	Sample Number	5
Borehole Number	RW8-1	Sample Depth, m	3.8 - 4.4 m

TEST CONDITIONS

Test Type	Standard	Load Duration, hr	24
Oedometer Number	6		
Date Started	10/17/2003		
Date Completed	10/28/2003		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	1.90	Unit Weight, kN/m ³	20.90
Sample Diameter, cm	6.34	Dry Unit Weight, kN/m ³	17.45
Area, cm ²	31.57	Specific Gravity, measured	2.73
Volume, cm ³	59.98	Solids Height, cm	1.238
Water Content, %	19.78	Volume of Solids, cm ³	39.09
Wet Mass, g	127.83	Volume of Voids, cm ³	20.89
Dry Mass, g	106.72	Degree of Saturation, %	101.1

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	1.900	0.534	1.900				
4.76	1.893	0.529	1.897	240	3.18E-03	7.52E-04	2.34E-07
9.58	1.888	0.525	1.891	271	2.80E-03	5.91E-04	1.62E-07
19.31	1.877	0.516	1.883	304	2.47E-03	5.73E-04	1.39E-07
38.80	1.864	0.505	1.870	197	3.76E-03	3.67E-04	1.35E-07
77.63	1.844	0.489	1.854	135	5.40E-03	2.64E-04	1.40E-07
155.17	1.817	0.468	1.831	94	7.56E-03	1.83E-04	1.35E-07
309.98	1.782	0.439	1.800	119	5.77E-03	1.19E-04	6.73E-08
620.68	1.740	0.406	1.761	103	6.39E-03	7.08E-05	4.43E-08
1240.11	1.695	0.369	1.718	68	9.20E-03	3.89E-05	3.51E-08
2479.33	1.643	0.327	1.669	68	8.68E-03	2.18E-05	1.85E-08
1240.11	1.653	0.335	1.648				
309.98	1.669	0.348	1.661				
77.63	1.687	0.363	1.678				
19.31	1.710	0.381	1.699				
4.77	1.729	0.396	1.719				

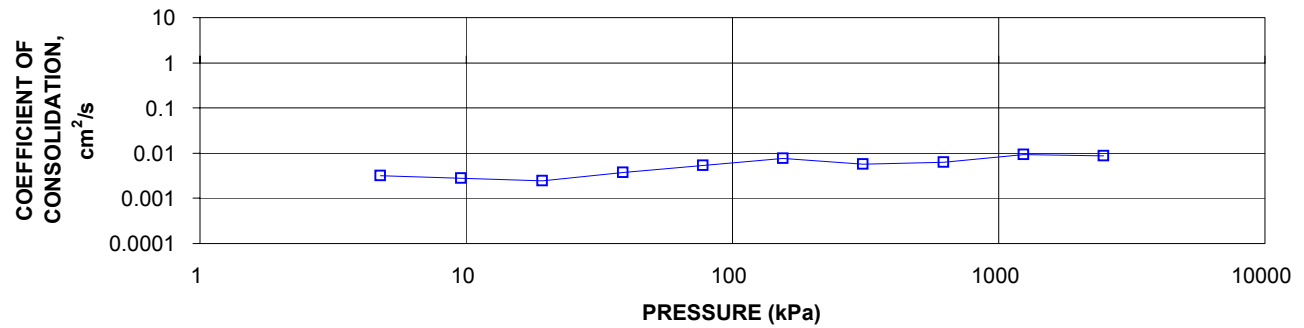
Notes:
k calculated using cv based on t_{90} values.

SAMPLE DIMENSIONS AND PROPERTIES - FINAL

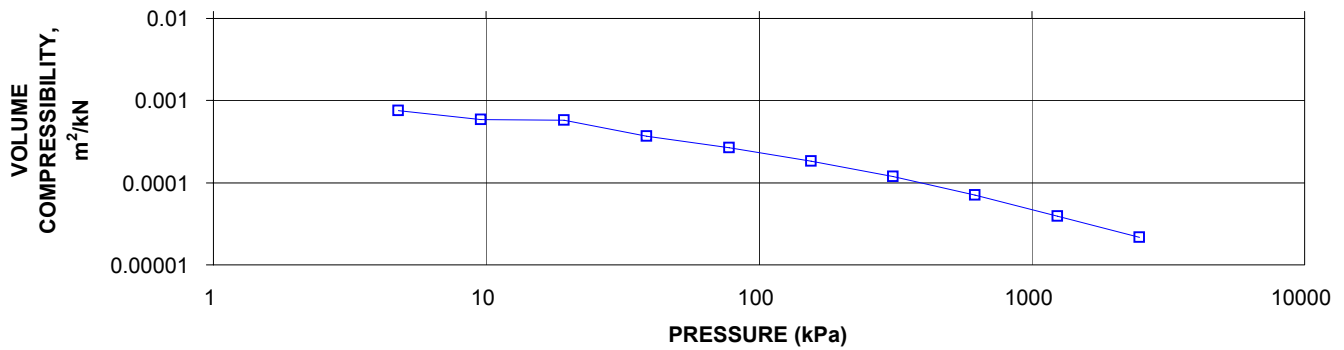
Sample Height, cm	1.73	Unit Weight, kN/m ³	22.20
Sample Diameter, cm	6.34	Dry Unit Weight, kN/m ³	19.18
Area, cm ²	31.57	Specific Gravity, measured	2.73
Volume, cm ³	54.57	Solids Height, cm	1.238
Water Content, %	15.73	Volume of Solids, cm ³	39.09
Wet Mass, g	123.51	Volume of Voids, cm ³	15.48
Dry Mass, g	106.72		

OEDOMETER CONSOLIDATION SUMMARY

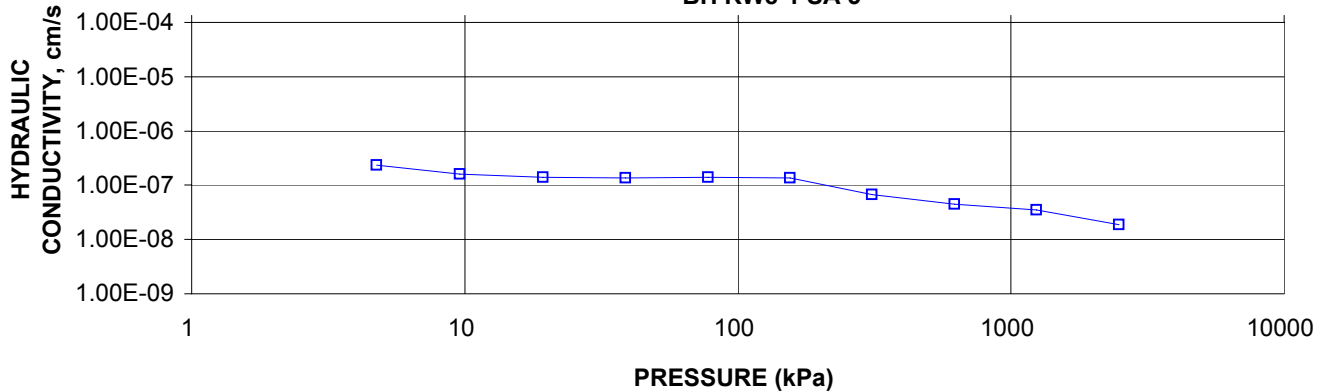
CONSOLIDATION TEST
CV cm²/s VS PRESSURE (kPa)
BH RW8-1 SA 5

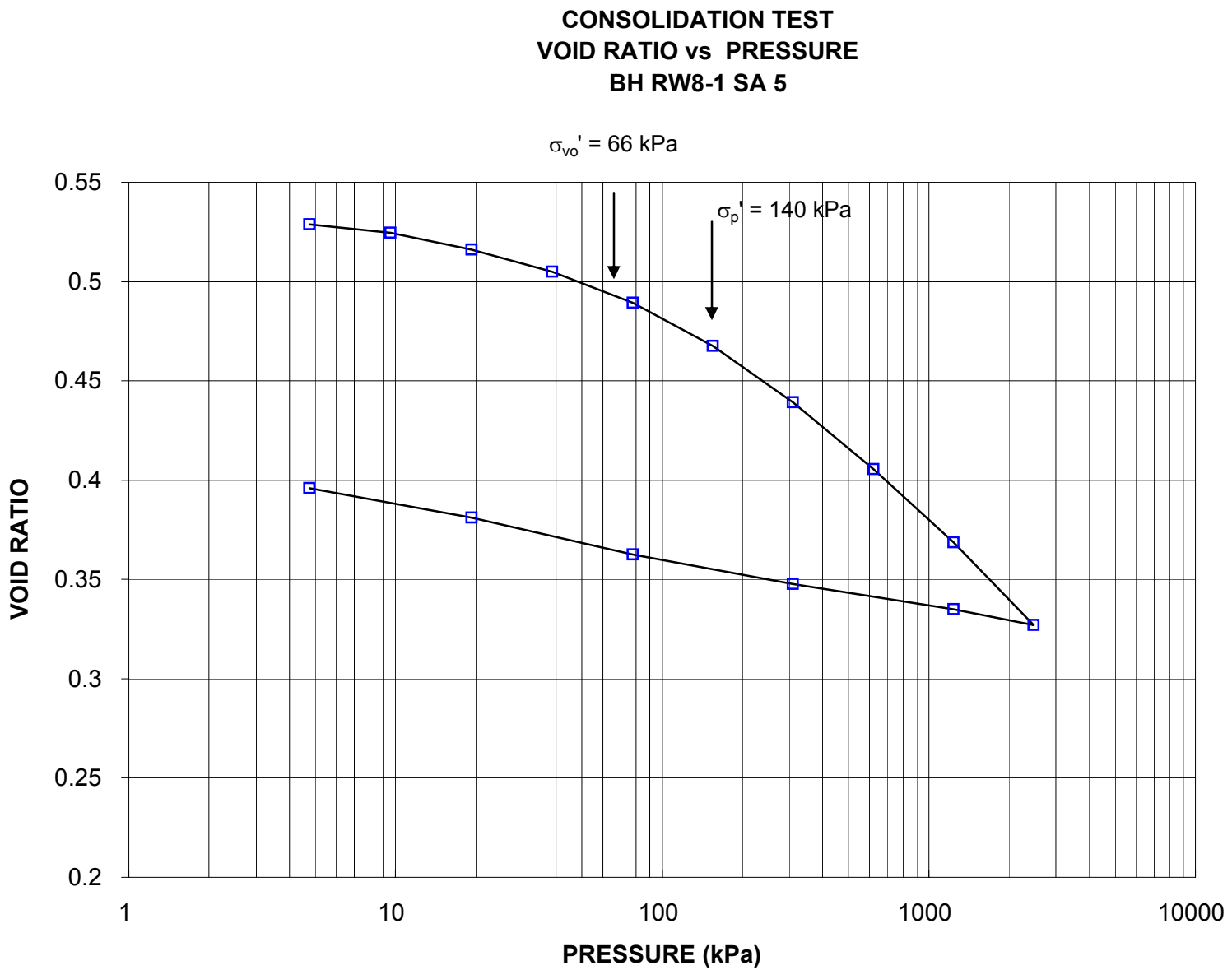


CONSOLIDATION TEST
MV m²/kN vs PRESSURE (kPa)
BH RW8-1 SA 5



CONSOLIDATION TEST
HYDRAULIC CONDUCTIVITY vs PRESSURE
BH RW8-1 SA 5





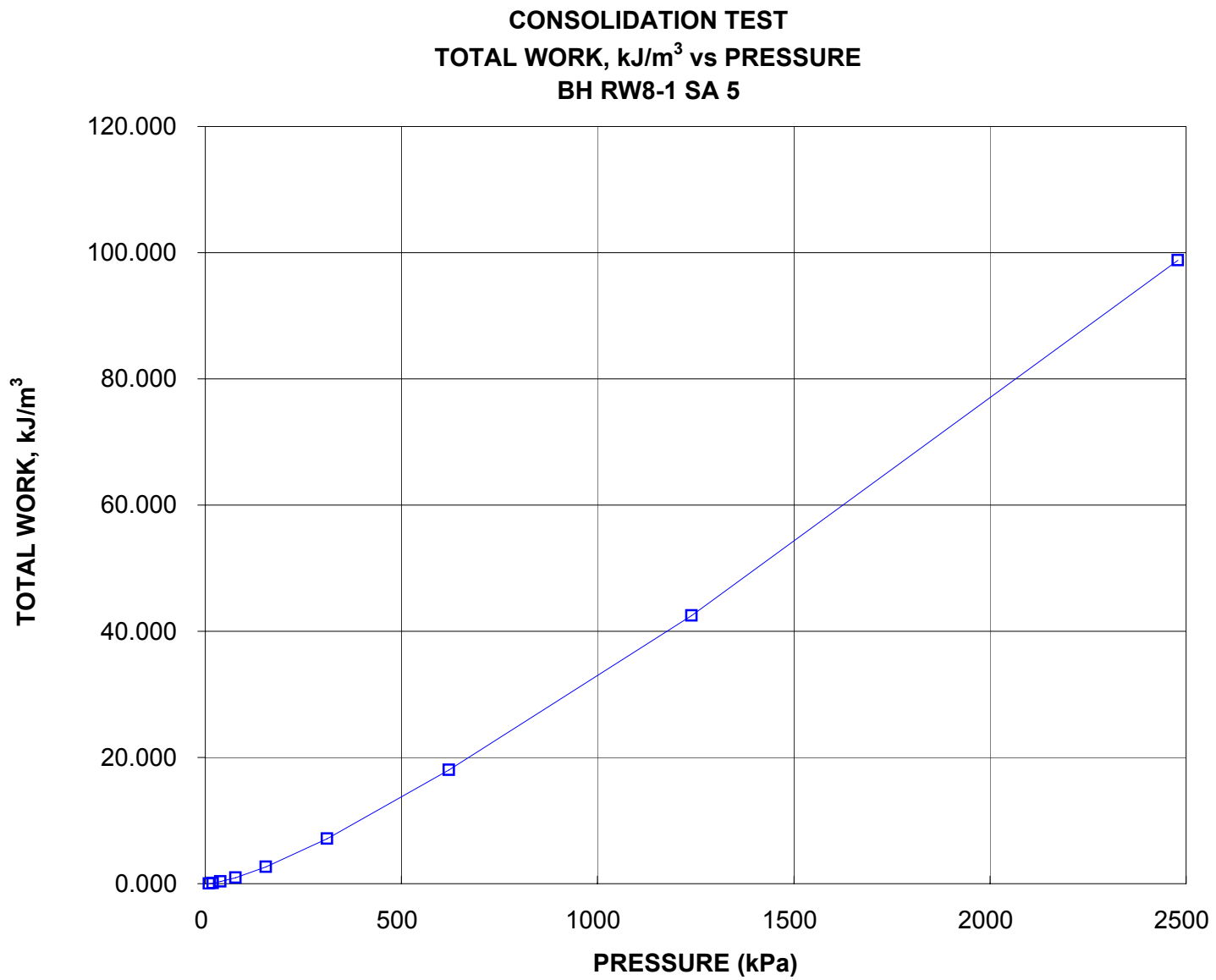
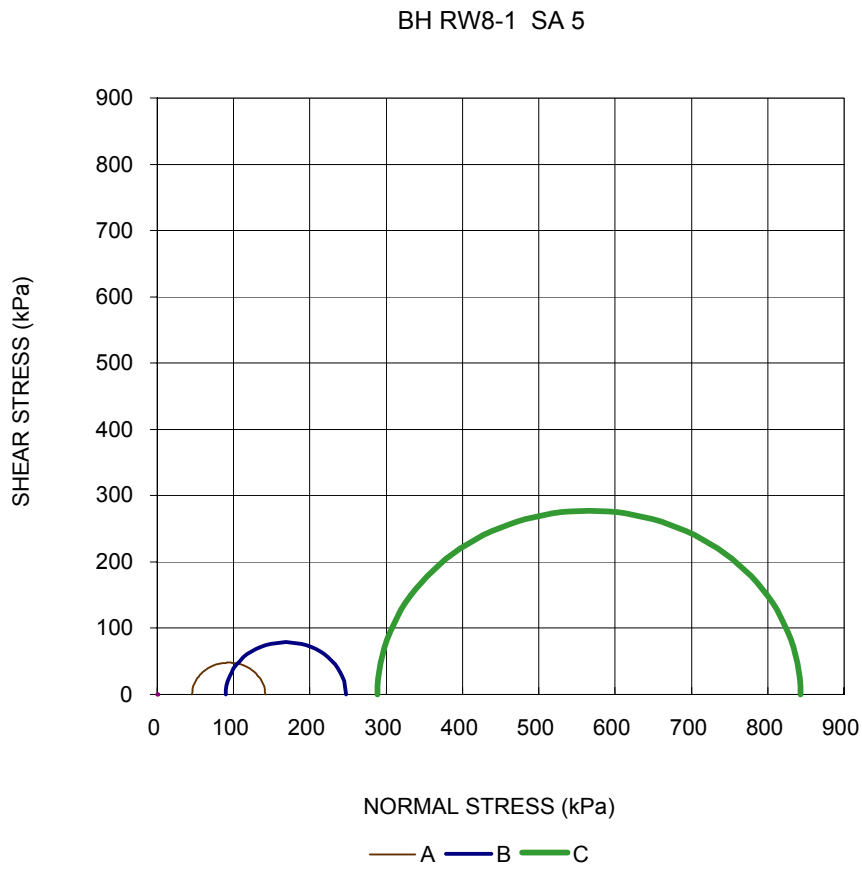


FIGURE A4

CONSOLIDATED UNDRAINED TRIAXIAL WITH PORE PRESSURE MEASUREMENTS SHEET 1 OF 4			FIGURE A5
TEST STAGE	A	B	C
BOREHOLE NUMBER	RW8-1	RW8-1	RW8-1
SAMPLE NUMBER	5	5	5
SPECIMEN DIAMETER, cm	5.04	5.01	5.01
SPECIMEN HEIGHT, cm	10.14	10.13	10.13
WATER CONTENT BEFORE CONSOLIDATION, %	25.4	20.1	22.5
CELL PRESSURE, σ_3 , kPa	280.0	425.0	435.0
BACK PRESSURE, kPa	205.0	275.0	135.0
PORE PRESSURE PARAMETER "B"	0.96	0.97	1.00
CONSOLIDATION PRESSURE, σ_c , kPa	75.0	150.0	300.0
VOLUMETRIC STRAIN DURING CONSOLIDATION, %	2.8	4.4	10.4
WATER CONTENT AFTER CONSOLIDATION, %	23.7	17.6	16.5
AVERAGE RATE OF STRAIN, %/hr	0.5	0.5	0.5
TIME TO FAILURE, DAYS	2	2	2
WATER CONTENT AFTER TEST, %	26.2	18.0	16.5
MAX. DEVIATOR STRESS, $(\sigma_1 - \sigma_3)$, kPa	95.6	157.7	554.5
AXIAL STRAIN AT $(\sigma_1 - \sigma_3)$ MAXIMUM, %	6.7	19.2	14.7
MAX EFFECTIVE PRINCIPAL STRESS	3.26	3.22	3.21
RATIO, (σ_1 / σ_3) MAXIMUM			
DEVIATOR STRESS AT (σ_1 / σ_3) MAXIMUM, kPa	95.5	145.0	553.2
AXIAL STRAIN AT (σ_1 / σ_3) MAXIMUM, %	10.6	6.3	13.4
PORE PRESSURE PARAMETER, A_f , AT $(\sigma_1 - \sigma_3)$ MAXIMUM	0.32	0.50	0.09
PORE PRESSURE PARAMETER, A_f , AT (σ_1 / σ_3) MAXIMUM	0.34	0.58	0.09
NATURAL WATER CONTENT, %	24.0	19.1	18.8
DRY DENSITY, Mg/m^3	1.61	1.79	1.72
FILTER DRAINS USED, y/n	y	y	y
TEST NOTES:			
Test C was partially drained due to the fact that back pressure valve was not completely closed.			
CHANGED RATE OF STRAIN, %/hr	-	-	-
AXIAL STRAIN WHERE RATE OF STRAIN WAS CHANGED, %	-	-	-
FAILURE PLANE NUMBER	1.0	1.0	-
ANGLE OF FAILURE, DEGREES	45.0	45.0	-
DATE: 10/28/2003			
Project No. 021-1162-BR8			Golder Associates

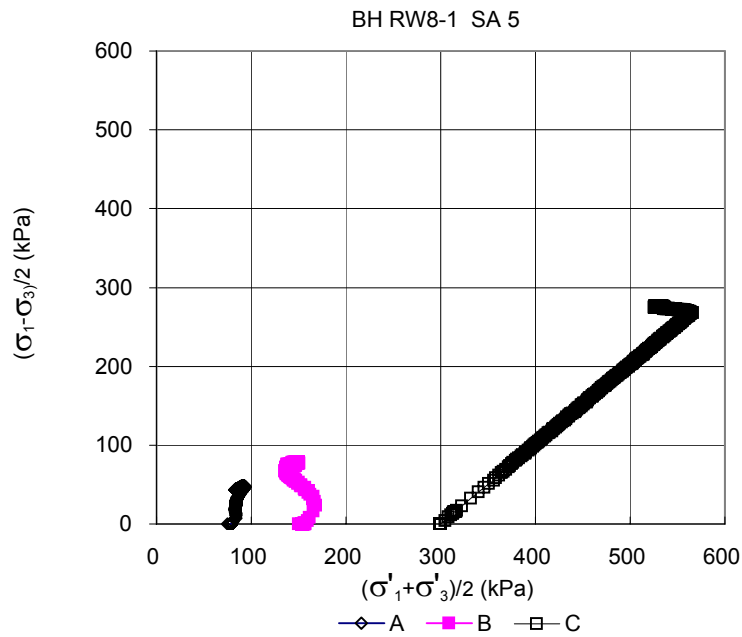
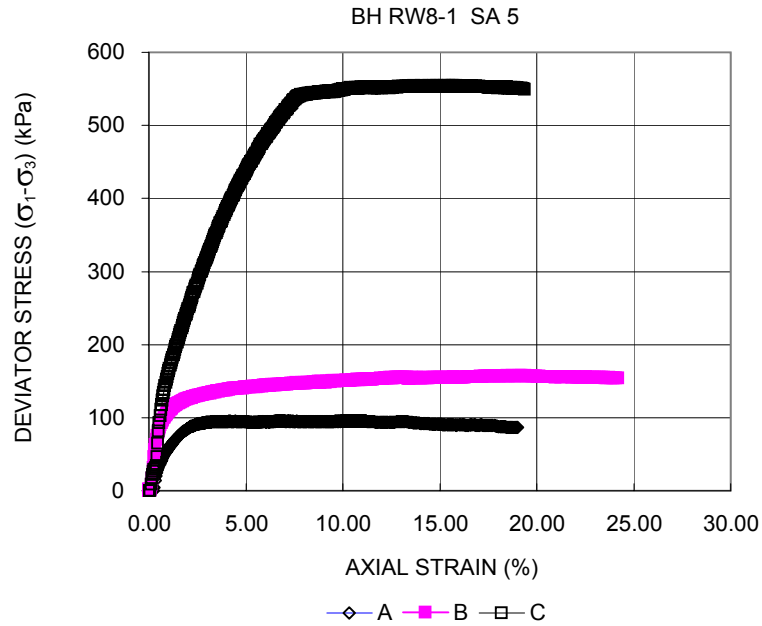
**CONSOLIDATED UNDRAINED TRIAXIAL
WITH PORE PRESSURE MEASUREMENTS
SHEET 2 OF 4**

FIGURE A5



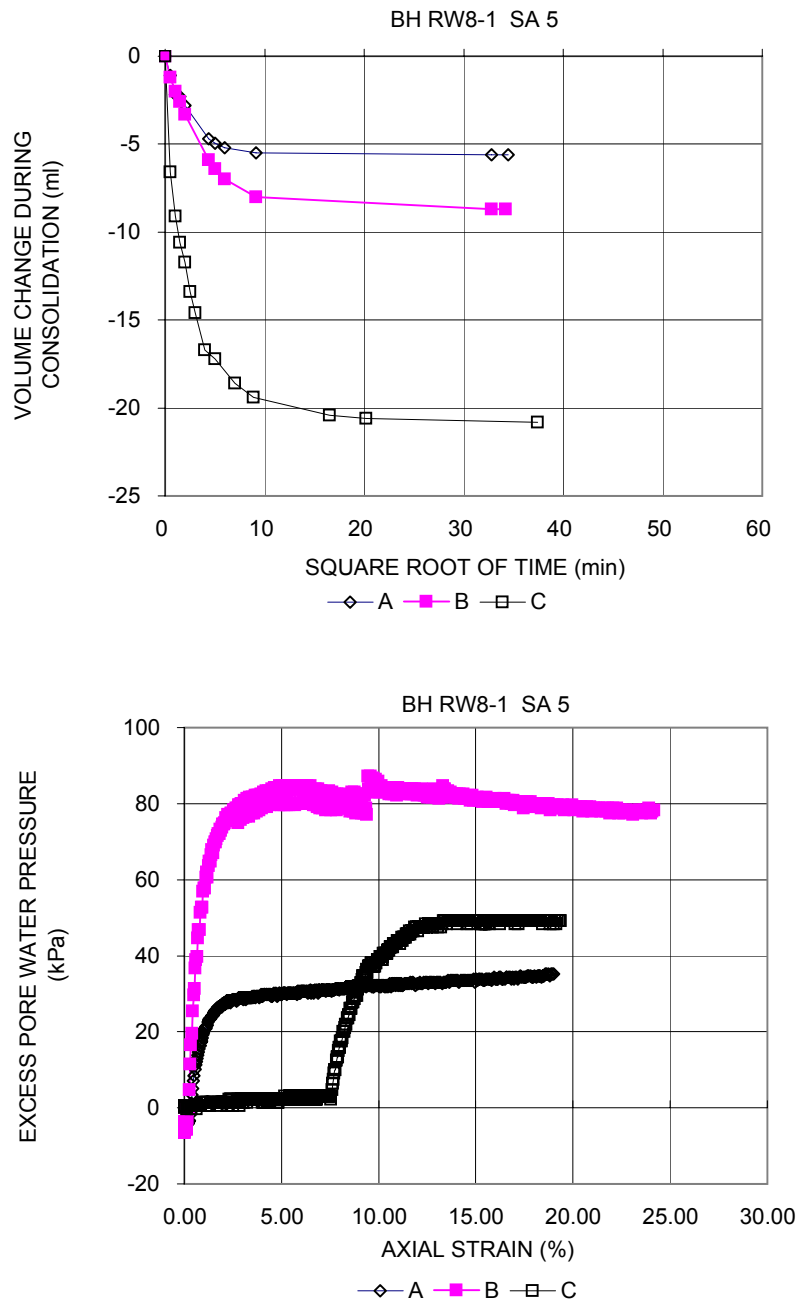
**CONSOLIDATED UNDRAINED TRIAXIAL
WITH PORE PRESSURE MEASUREMENTS
SHEET 3 OF 4**

FIGURE A5



CONSOLIDATED UNDRAINED TRIAXIAL
WITH PORE PRESSURE MEASUREMENTS
SHEET 4 OF 4

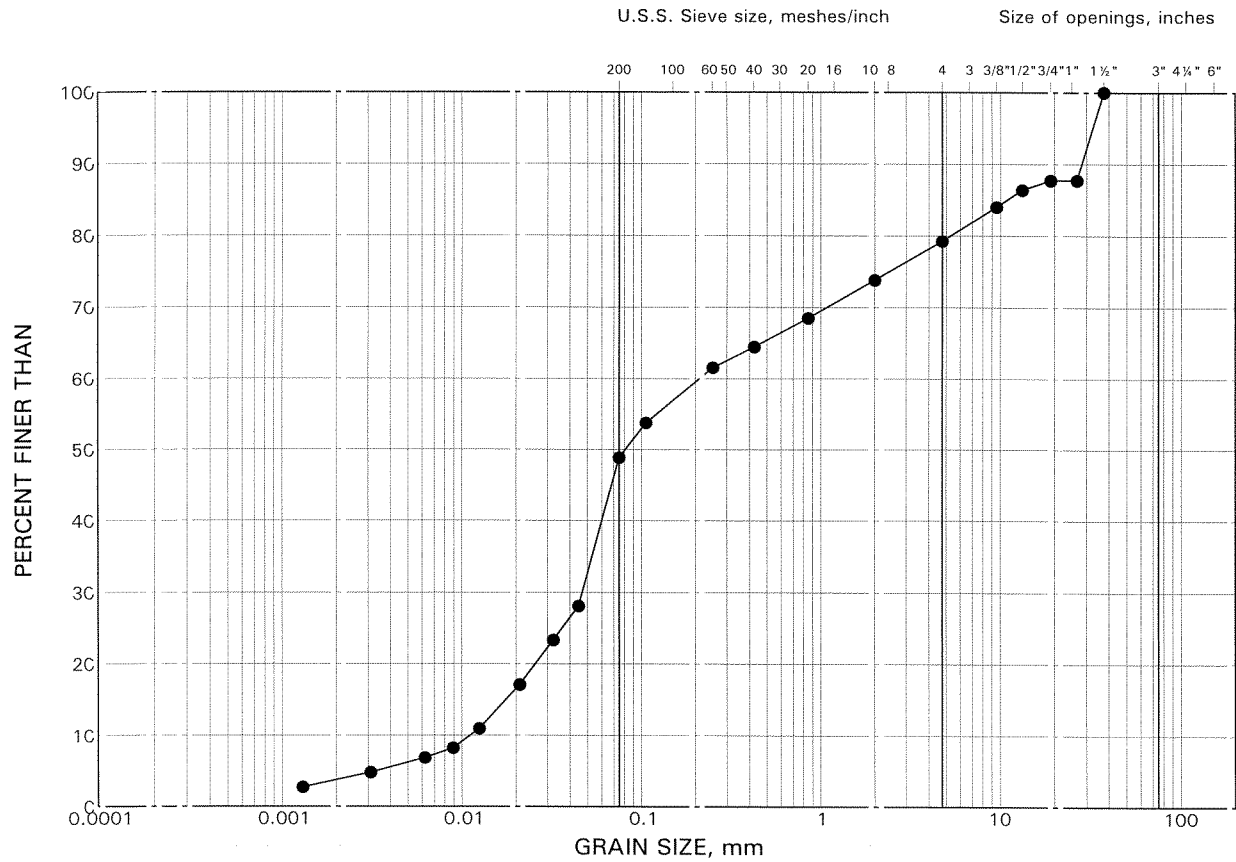
FIGURE A5



GRAIN SIZE DISTRIBUTION

Sandy Silt with Gravel (Till)

FIGURE A6



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
●	BR8-2	18 & 19	60.5-58.7