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REPORT ON

DETAIL DESIGN
FOUNDATION INVESTIGATION AND DESIGN
RHCE OVER E-S RAMP OVER CENTENNIAL PARKWAY
BRIDGE 10
RED HILL CREEK EXPRESSWAY INTERCHANGE
G.W.P. 441-97-00
MINISTRY OF TRANSPORTATION, ONTARIO
HAMILTON, ONTARIO

Submitted to:

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GEOCRES NO.

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June 2004 021-1162-BR10

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1.0 INTRODUCTION

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

The terms of reference for the scope of work are outlined in Golder's proposal P21-1334, dated November 2002, that forms part of the Consultant's Agreement (Number P.O.2005-A-000482) for this project. This report addresses Bridge 10 carrying the Red Hill Creek Expressway E-S Ramp over Centennial Parkway as part of the interchange project. The work was carried out in accordance with the Quality Control Plan for this project dated November 2002. 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 terrain in this area is generally flat-lying with the exception of a high fill embankment that exists to the east of the proposed bridge site. The Centennial Parkway grade at the bridge site is at about Elevation 80 m in the area of the proposed works. Minor undulations across the site mainly involve fill embankments at the existing interchange, the highway right-of-ways, as well as regrading and landscaping on adjacent lands.

3.0 INVESTIGATION PROCEDURES

3.1 Foundation Investigation

The field work at the Bridge 10 site was carried out between August 20 and August 21, 2003 at which time two (2) boreholes, numbered BR10-1 and BR10-2 were advanced. Boreholes 5, 6 and RESR-7 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 8.1 m to 24.4 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 two of the boreholes were 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) was carried out on selected samples. Point load testing was carried out on samples of the rock core.

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

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

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

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

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

4.2 Subsoil Conditions

The detailed subsurface soil, bedrock and groundwater conditions as encountered in the boreholes advanced during this investigation, together with the results of the laboratory tests carried out on selected soil samples, are given on the attached Record of Borehole sheets and in Appendix A following the text of this report. The stratigraphic boundaries shown on the Record of Borehole sheets are inferred from non-continuous sampling, observations of drilling progress and the results of Standard Penetration Tests (SPTs). These boundaries, therefore, represent transitions between soil types rather than exact planes of geological change. Further, subsurface conditions will vary between and beyond the borehole locations. The inferred soil stratigraphy based on the results of the boreholes at the bridge location are shown on Drawing 1.

In general, the subsoils at the site consist of a thin layer of topsoil underlain by a surficial layer of fill which is underlain by thin layers of silty sand to sandy silt and clayey silt. The surficial deposits are underlain by a thick deposit of grey clayey silt till. The upper portion of the clayey silt till deposit is typically soft to firm 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 in the deeper boreholes. The clayey tills are underlain by a thin deposit of sandy silt till containing shale and limestone fragments 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 20.4 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 boreholes. The existing ground surface ranged between Elevation 79.2 m to 80.6 m and the topsoil ranged from 0.1 m to 0.3 m thick.

Standard Penetration Testing (SPT) measured 'N' values within the topsoil ranged between 2 and 23 blows per 0.3 m of penetration, indicating very loose to compact state of packing.

4.2.2 Fill

Fill materials were encountered below the topsoil in Borehole 5, located near the proposed west abutment. The fill is composed of sand, trace silt and gravel and extends to 1.2 m below ground surface at this borehole location.

One Standard Penetration Test (SPT) measured 'N' value within the non-cohesive fill was 23 blows per 0.3 m of penetration, indicating a compact relative density.

4.2.3 Sandy Silt to Silty Sand

A deposit of brown to grey, sandy silt to silty sand containing trace to some clay and gravel was encountered below the topsoil in all boreholes except Borehole 5. Occasional pockets of silty clay were noted in Boreholes 6 and BR10-1. The sandy silt to silty sand layer ranged from 1.9 to 3.6 m in thickness, and the surface was encountered between Elevations 79.0 and 80.4 m at the borehole locations. Trace organics were noted in the silty sand to sandy silt layer in Borehole 6.

Standard Penetration Testing (SPT) measured 'N' values ranged between 15 and 50 blows per 0.3 m of penetration within this deposit, indicating a compact to dense state of packing. A grain size distribution curve on one sample of this deposit is shown on Figure A1 in Appendix A. In this sample, the soil is classified as a silt containing some sand and clay.

The natural water content measured on samples of the sandy silt to silty sand ranges from 13 to 17 percent.

4.2.4 Clayey Silt

A deposit of mottled grey and brown clayey silt containing trace sand was encountered below the fill or the sandy silt to silty sand layer in Boreholes 5, 6 and RESR-7, which are located to the north of the proposed bridge. This deposit was not encountered in the Boreholes BR10-1 and BR10-2, which are located at the proposed bridge abutments. The clayey silt ranged from 0.8 to

1.7 m in thickness, and the surface was encountered between Elevations 78.4 and 78.8 m at the borehole locations. Occasional rootlets were noted in Borehole 5.

Standard Penetration Testing (SPT) measured 'N' values ranged between 11 and 44 blows per 0.3 m of penetration, indicating a stiff to hard consistency. A grain size distribution curve for one selected sample from this deposit is shown on Figure A2 in Appendix A.

Atterberg Limits testing was carried out on one sample of the clayey silt deposit. The liquid limit was 28 percent and the plastic limit was 14 percent giving a plasticity index of 14. The results of the testing indicate that the soil is a clayey silt of low plasticity. The results of the Atterberg limits tests are plotted on the plasticity chart on Figure A3 Appendix A.

The natural water contents measured on samples of the clayey silt were between 13 and 27 percent.

4.2.5 Clayey Silt Till

A deposit of clayey silt till was encountered underlying the surficial deposits in all boreholes at the site. The clayey silt till contains trace to some sand and gravel and occasional silt and sandy silt seams were noted in Borehole 5, 6, RESR-7 and BR10-2. The clayey silt was 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 75.4 and 78.2 m in all boreholes and the thickness varied from 14.5 m and 16.0 m in Boreholes BR10-1 and BR10-2 where the deposit was fully penetrated. This deposit was not fully penetrated in Boreholes 5, 6 and RESR-7.

The upper 2.9 m to 5.7 m of the clayey silt till deposit is described as a 'softened' till zone. The SPT measured 'N' values within the upper portion of the clayey silt ranged between 0 (weight of hammer) and 5 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 stiff consistency. In general, the results of the field vane tests indicate a soft to firm consistency. Grain size distribution curves for selected samples from this portion of the deposit are shown on Figure A4 in Appendix A. The results of the vane testing are summarized in the following table:

Borehole	*Location*	*Sample Depth/Elevation (m)*	*Undisturbed Shear Strength(kPa)*	*Remoulded Shear Strength (kPa)*	*Sensitivity*
RESR-7	West approach	4.5 / 76	24.0	8.0	3.0
		5.3 / 75.2	22.0	9.2	2.4
		6.3 / 74.2	40.0	20.0	2.0
5	West abutment	3.3 / 76.6	54.0	23.0	2.3
		4.0 / 75.9	18.0	8.0	2.2
		6.3 / 73.6	26.0	9.0	3.2
		6.9 / 73	23.0	12.0	1.9
6	East abutment	5.0 / 75.6	15.0	10.0	1.5
		8.0 / 72.6	40.0	22.0	1.8
BR10-1	West abutment	4.4 / 74.8	44.0	18.2	2.4
		4.7 / 74.5	72.0	37.3	1.9
		5.6 / 73.6	44.0	18.2	2.4
		5.9 / 73.3	72.8	37.3	1.9
		6.7 / 72.5	> 100	-	-
		8.2 / 71	> 100	-	
BR10-2	East abutment	2.9 / 77.6	36.4	14.4	2.5
		3.7 / 76.8	24.9	10.5	2.4
		4.0 / 76.5	36.4	22	1.7

It should be noted that the higher vane strengths are typically associated with the surface of the ‘softened’ till or the surface of the underlying stiff to hard clayey silt till.

The lower portion of clayey silt till deposit had measured SPT ‘N’ values ranging from 8 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 72.0 m to 74.4 m. In Boreholes 5, 6, BR10-1 and BR10-2, this portion of the stratum was penetrated and was found to range between 9.4 m and 12.2 m in thickness. Grain size distribution curves for selected samples from this portion of the deposit are shown on Figure A4 in Appendix A.

Atterberg limits testing was carried out on four samples of the softened clayey silt till deposit and on one sample 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 A5 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 RESR-7, sample 5 indicates a silty clay of intermediate plasticity.

Borehole	*Sample*	*Elevation (m)*	*Liquid Limit (%)*	*Plastic Limit (%)*	*Plasticity Index (%)*
RESR-7	5	75.9 – 76.4	36	18	18
5	7	75.3 – 75.8	29	18	11
6	6	76.8 – 77.2	31	18	13
10-1	11	68.5 – 67.0	30	16	14
10-2	6	75.9 – 76.4	28	13	15
Average	-	-	31	17	14

The natural water content measured on selected samples of the clayey silt till deposit ranged between 8 percent and 31 percent, with an average of 21 percent.

Laboratory oedometer (consolidation) testing was carried out on two specimens of the upper 'softened' portion of the clayey silt till obtained from the 1998 Boreholes 6 and RESR-7. Details of the test results are shown on Figures A6 and A7 in Appendix A and the results summarized in the table below:

Borehole and Sample No.	*Elevation (m)*	σ_{vo}' *(kPa)*	σ_p' *(kPa)*	*OCR*	e_o	C_r	C_c	c_v^* (cm^2/s)
RESR-7, Sa 5	75.9 – 76.4	70	100	1.4	0.90	0.045	0.27	3.2×10^{-2}
6, Sa 6	76.3 – 76.7	83	160	1.9	0.85	0.028	0.23	1.5×10^{-3}

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 cm^2/s

Laboratory consolidated undrained (CIU) triaxial compression test were carried out on carefully trimmed specimen of the clayey silt till obtained from Borehole RESR-7. The test results indicate effective angle of shearing resistance of 15 degrees and effective shear resistance of 20 kPa. Details of the test results are shown on Figure A8 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 5 and 6, located to the north of the proposed bridge site.

Occasional weathered shale and limestone fragments were observed in the deposit. The top of this layer was encountered between Elevation 61.9 m and 62.6 m. Boreholes 5 and 6 were terminated in this layer proving a thickness of 0.4 and 1.3 m.

The measured SPT 'N' value for samples of this lower portion of the till deposit were greater than 100 blows per 0.3 m of penetration, indicating a hard consistency. A grain size distribution curve for one sample of this material is shown on Figure A10 in Appendix A.

The natural water content measured on one selected sample of this hard clayey silt till layer was 9 percent.

4.2.6 Sandy Silt Till

In Boreholes BR10-1 and BR10-2 a deposit of reddish-brown sandy silt till, was encountered below the reddish-grey clayey silt till. The deposit contains varying amounts of clay and gravel. This deposit is considered to be the 'Wentworth" till sheet. Although not encountered in the boreholes at this site, it should be noted that the sandy silt till in this area is known to contain numerous cobbles and boulders. The surface of the deposit was encountered between Elevation 60.9 m and 62.2 m and the deposit was between 1.2 and 2.1 m in thickness.

The measured SPT 'N' value for samples of this lower portion of the till deposit were greater than 100 blows per 0.3 m of penetration, indicating a very dense state of packing.

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

4.2.7 Bedrock

In Boreholes BR10-1 and BR10-2, the bedrock surface was encountered between Elevation 59.7 to 60.1 m and bedrock was cored for a depth of 4 m.

The bedrock samples obtained consist of reddish-grey to light grey, moderately to highly weathered, thinly layered, 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 78 percent and 100 percent. The Rock Quality Designation (RQD) measured on the core samples in Borehole BR10-1 and BR10-2 ranged from about 53 to 100 percent, with the lower values encountered near the surface of the bedrock. This indicates a rock mass of fair to excellent quality; typically good.

Point load strength tests were performed on selected samples of the rock core from Boreholes BR10-1 and BR10-2. Diametral point load strength index values are shown on the Record of

Drillhole Sheets. Diametral point load index values on core samples of the shale range from 0.06 to 1.2 MPa which corresponds to an estimated unconfined compressive strength (UCS) ranging from 1 to 27 MPa. The axial point load index values on core samples of the shale range from 0.06 to 2.4 MPa corresponding to approximate UCS values between 1 and 55 MPa. Using the Intact Rock Strength Classification table, these values indicate that the shale is classified very weak to strong; however, the shale is typically weak. On the limestone core samples, the axial and diametral point load index values range from 1.2 to 4.8 MPa corresponding to approximate UCS values between 28 and 107 MPa. This indicates that the limestone is classified as medium strong to very strong.

4.2.8 Groundwater Conditions

The water levels were noted during and after the drilling and coring operations in the boreholes. Piezometers were installed in Boreholes BR10-1 and 6. The piezometer in Borehole BR10-1 was sealed into the bedrock and the piezometer in Borehole 6 was sealed in the clayey silt till deposit, just above 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 and open holes upon completion of drilling are summarized in the table below:

Borehole	*Installations*	*Ground Surface Elevation (m)*	*Ground Water Level Depth (m)*	*Ground Water Level Elevation (m)*	*Date*
BR10-1	Piezometer	79.2	2.6	76.6	October 22, 2003
RESR-7	Open borehole	80.5	6.7	73.8	September 10, 1998
5	Open borehole	79.9	3.7	76.2	February 17, 1998
6	Piezometer	80.6	4.3	76.2	October 22, 2003

The groundwater level at the borehole locations is typically within 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.

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PART B
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5.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides foundation design recommendations for the proposed Bridge 10 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 single 45 m span bridge will carry the Red Hill Creek Expressway E-S Ramp over Centennial Parkway. The proposed grade of the Expressway varies from about Elevation 88.0 m to 88.5 m at the east and west approaches, respectively. The existing ground surface at the bridge varies from about Elevation 80.5 m to 81 m resulting in approach embankments between 7 m and 8 m in height. The revised General Arrangement drawing for Bridge 10 was provided by MRC in electronic format in June 2004 and incorporated into Drawing 1.

5.1 General

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

5.2 Shallow Foundations

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

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

5.2.1 Geotechnical Resistance

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

5.2.2 Resistance to Lateral Loads

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

5.2.3 Frost Protection

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

5.3 Steel H-Pile Foundations

Steel H-piles driven to found in the hard clayey silt till or very dense sandy silt till (where SPT ‘N’ values are greater than 100 blows per 0.3 m of penetration) may be used for support of the abutments and/or wing walls. Alternatively, steel H-piles could be driven into the shale bedrock at the site; however, although the hard/very dense till deposits are relatively thin, it is likely that practical refusal for the piles may be met within the hard till deposits prior to reaching the bedrock surface.

It is assumed that the abutment pile caps will be perched within the embankment. For design, the following pile tip levels may be assumed for piles terminated within the till (assumed minimum 2 m penetration) or just into the bedrock (assumed 0.5 m penetration). There should be provision made in the contract for dealing with varying pile lengths.

Foundation Location	***Relevant Boreholes***	***Design Pile Tip Elevation (m)***	
		Very Dense/Hard Till	Bedrock
East Abutment	BR10-2	62.0	59.5
West Abutment	BR10-1	60.0	59.0

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 Elevation 62 m.

Pile installation should be in accordance with SP903S01. 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 pile termination or set criteria will be dependent on the pile driving hammer type, helmet, selected pile and length of pile. The criteria must therefore be established at the time of construction after the piling equipment is known. For piles driven into the hard / very dense till, the following note is considered appropriate for the design and site conditions assuming a resistance factor of 0.4 is applied to the used of the Hiley:

- "Piles to be driven in accordance with Standard SS 103-11 using an ultimate capacity of 3,500 kN per pile but must be driven below EL 62 m (East Abutment) and below EL 60 m (West Abutment)."

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

– "Piles to be driven to bedrock."

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

5.3.2 Downdrag Load (Negative Skin Friction)

The embankment loading will cause consolidation settlement of the underlying softened clayey silt till deposit. The consolidation settlement is time-dependent and will not completely occur during the construction period. That is, post-construction settlement of the clayey silt deposit will take place. Negative skin friction or downdrag loads will need to be taken into account during design of the piles supporting the abutments as a consequence of settlement of the ground with respect to the pile. The abutment pile structural design should be based on the full downdrag load acting on the piles within and above the soft to firm till zone. The estimated unfactored downdrag load acting on the HP 310x110 piles may be taken as 150 kN per pile at the abutment locations.

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, preloading of the embankments, or by sub-excavation of softened material as discussed in Section 5.6.4.

5.3.3 Resistance to Lateral Loads

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

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

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

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

The following range for the value of s_u (based on stratigraphy anticipated at the east abutment) may be assumed in the structural analysis:

Soil Unit	***Elevation***	***s_u (kPa)***
Soft clayey silt till	Between Elev. 77.5 and 74.0 m	20
Firm clayey silt till	Between Elev. 74.0 and 72.0 m	40
Stiff to hard clayey silt till	Below Elev. 72.0 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:

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

5.3.4 Frost Protection

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

5.4 Caissons

Consideration could be given to the use of caissons socketted into the hard clayey silt till / very dense sandy silt till deposits or into the shale bedrock for support of the bridge. It should be noted that although the sandy silt till overlying the bedrock is relatively thin, this deposit is known to contain numerous cobbles and boulders which may pose difficulties in advancing the caissons / temporary liners through to the bedrock surface. The following design base elevations may be used at the bridge abutments for caissons founded on the surface or just into the sandy silt till and for caissons socketted at least 2 m into the bedrock:

Caisson Diameter (m)	*Unfactored Downdrag Load (kN)*
0.9	350
1.5	550

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

5.4.3 Resistance to Lateral Loads

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

5.4.4 Frost Protection

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

5.5 Lateral Earth Pressures for Design

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

The following recommendations are made concerning the design of the walls. It should be noted that these design recommendations and parameters assume level backfill and ground surface behind the walls. Where there is sloping ground behind the walls, the coefficient of lateral earth pressure must be adjusted to account for the slope.

- Select free-draining granular fill meeting the specifications of Ontario Provincial Standard Specifications (OPSS) Granular 'A' or Granular 'B' should be used as backfill behind the walls. This fill should be compacted in loose lifts not greater than 200 mm in thickness to 95 per cent of the material's Standard Proctor maximum dry density in accordance with OPSS 501. Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the granular backfill requirements with respect to sub-drains and frost taper should be in accordance with OPSD 3501.00 and 3504.00.

Foundation Location	*Design Caisson Founding Elevation (m)*	
	Very Dense Till	Bedrock
East Abutment	63.0	58.0
West Abutment	61.0	57.5

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

5.4.1 Axial Geotechnical Resistance

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

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

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

5.4.2 Downdrag Load (Negative Skin Friction)

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

- 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(l) 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(l) of the *Commentary to the CHBDC)*.

- For Case I, the pressures are based on the proposed embankment fill materials and the following parameters (unfactored) may be used:

	Earth Fill
Soil unit weight:	21 kN/m^3
Coefficients of static lateral earth pressure:	
Active, K_a	0.33
At rest, K_o	0.50

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

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

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

5.6 Approach Embankment Design and Construction

The proposed grade of the Expressway varies from about Elevation 88.0 m to 88.5 m at the east and west approaches, respectively. The existing ground surface at the bridge varies from about

Elevation 80.5 m to 81 m resulting in approach embankments between 7 m and 8 m in height. The design of the embankments beyond the limits of the bridge approaches will be addressed in a separate report.

5.6.1 Subgrade Preparation and Embankment Construction

Where the embankments are greater than 8 m in height, a mid-height, 2 m wide berm is required in accordance with MTO guidelines for surficial stability. Although embankments 8 m high or less are anticipated at this site, provisions for a mid-height berm should be included in the design if higher embankments are required.

It is our understanding that it is not normal practice to carry out topsoil stripping from below embankments which are greater than 1.2 m in height. At this site, however, given the 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 fill should be placed in regular lifts with loose thickness not exceeding 300 mm, and be compacted to at least 95 per cent of the material's Standard Proctor maximum dry density. The final lift prior to placement of the granular subbase and base courses should be compacted to 100 per cent of the Standard Proctor maximum dry density. Inspection and field density testing should be carried out by qualified personnel during placement operations to ensure that appropriate materials are used and that adequate levels of compaction have been achieved.

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

5.6.2 Approach Embankment Stability

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

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

Soil Deposit	***Bulk Unit Weight***	***Effective Friction Angle***	***Undrained Shear Strength***
Embankment Fill	21 kN/m^3	32°	–
Surficial Silty Sand	20 kN/m^3	32°	–
Soft Clayey Silt Till (N. Abut)	19 kN/m^3	–	40 - 20 kPa
Firm Clayey Silt Till (N.& S. Abut)	19 kN/m^3	–	40 kPa
Stiff to Hard Clayey Silt Till	20 kN/m^3	–	100 kPa
Sandy Silt Till	21 kN/m^3	32°	–

The undrained shear strength data interpreted 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 shear strength calculated from the oedometer test results is an average mobilized undrained shear strength based on the formula $s_u = 0.22 \times \sigma_p'$ (in kPa). The undrained shear strength data from the boreholes at the Bridge 10 site is summarized on Figure 2 and formed the basis for the design strength profiles shown in the table above for the east and west abutments.

The analyses indicate that a factor of safety of less than 1.3 for a deep-seated failure surface is obtained for the approach embankments with side slopes at the proposed 2 horizontal to 1 vertical (2H:1V) profile. In order to achieve the target factor of safety of 1.3, toe berms (4 m wide and 3 m high) would be required (see Figure 3).

Alternatively, consideration could be given to the use of EPS lightweight fill in a portion of the embankments in order to reduce driving forces and satisfy the target factor of safety for the front slope without the need for toe berms. In order to achieve a factor of safety of 1.3, a minimum thickness of 1.5 m of EPS would be required beneath the pavement structure. In this case, the effective height of the embankment fill would be 6.5 m (see Figure 4). 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.

Other alternatives which could be considered include sub-excavation of the soft to firm clayey silt till (to a depth of about 8.5 m) or the use of lightweight or ultra-lightweight fill in combination with earth fill.

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 in the area of the approach embankments. In order to estimate the magnitude and rate of settlement, analyses were carried out in part using the commercially available computer program Unisettle 3.0 in conjunction with hand calculations.

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 (i.e. long-term, post construction) settlement of the up to 5.7 m thick soft to firm clayey silt till deposit underlying the 8 m high embankments is estimated to be about 275 mm for the earth fill option (including the influence of toe berms). If the EPS alternative is chosen, up to about 200 mm of post-construction settlement is estimated to occur. If sub-excavation of the soft to firm clayey silt till is carried out below the approach embankments to Elevation 72.0 m, the settlement of the underlying stiff to hard clayey silt till is estimated to be less than about 60 mm and the majority of this settlement will occur during and immediately following construction. Depending on the alternative chosen, pre-loading of the embankments would be necessary in order to limit post-construction settlements.

5.6.4 Mitigation of Stability Issues / Time Dependant Settlement

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.

5.6.4.1 Preloading

The maximum preload embankment height with side slopes of 2H:1V (without toe berms) is 6.5 m. If space permits, the embankment can be preloaded to a height of 8 m using toe berms 4 m wide and 3 m high (side slopes at 2H:1V). The embankment height would have to be tapered from a height of 6.5 m at the abutment area to a height of 8 m in the area where the required toe berms can be provided. The embankment preload should be left in place for a period of at least 12 months and it is estimated that 90 percent of the consolidation settlement will occur during this time. If space does not permit the embankment preload to be sufficient to cover to fully encompass the proposed abutment, the use of EPS would enable the final embankment to be constructed without additional load being applied. In this regard, details of the preload embankment should be reviewed to establish what additional EPS may be required.

If the approach embankment areas are preloaded, the total post-construction settlement beneath the wing walls will be about 25 mm. 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 limiting space for preload embankments, it will not be possible to place a surcharge load in the area of the approaches.

Consideration could be given to the use of wick drains in combination with preloading in order to reduce the time required for 90 percent consolidation to occur. In this case, 90 percent of consolidation will occur in about 3 months with a wick drain spacing of 1 m. 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 (and ability to interpret the monitoring results).

If the preloading and preloading/wick drain option (without surcharge) is chosen as the preferred alternative, there will be about 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.

5.6.4.2 Subexcavation

Alternatively, sub-excavation of the 2.9 m to 5.7 m of soft to firm clayey silt till could be considered to eliminate the post-construction settlement and eliminate the requirement for EPS fill and/or toe berms. The compressible material would have to be excavated to a minimum of Elevation 72.0 m which would require excavations up to 8.5 m deep. The requirements with respect to maximum cut slope profiles and groundwater control should be in accordance with Section 5.6. Due to the depth of the required excavation at this site and the proximity to the existing road (possible roadway protection), sub-excavation is not considered to be an economical option.

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

It is anticipated that the abutment pile caps will be constructed within the newly constructed embankment fill. If sub-excavation is being considered as part of the settlement mitigation scheme, excavations will extend through surficial silty sand, sandy silt and clayey silt soils and into the soft to firm clayey silt till. This soft to firm clayey silt till deposit is very susceptible to disturbance from ponded water and construction traffic. If the base of the excavation is within the soft to firm clayey silt till, special precautions may be required to provide suitable working conditions. 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.

It is anticipated that the bulk of the excavations at the site can be made in open cut. Excavations should be carried out in accordance with the guidelines outlined in the latest edition of the Occupational Health and Safety Act (OHSA) for Construction Activities. The surficial fills and the upper 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) are classified as Type 4 soil. Temporary excavations (i.e. those which are only open for a relatively short period) extended into the soft to firm clayey silt till should be made with side slopes no steeper than 1.5 horizontal to 1 vertical (1.5H:1V) through these materials. For excavations terminated above the "softened" till, side slopes at 1H:1V are suitable.

The ground water level at the site is typically at about Elevation 76.5 m and is between 2.5 m and 4.5 m below the ground surface. In general, the boreholes were typically dry upon completion of drilling and it is anticipated that for the open-cut excavations, the groundwater can be adequately controlled by sumping from properly filtered sumps.

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

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TABLE

TABLE 1
EVALUATION OF FOUNDATION ALTERNATIVES
BRIDGE 10 - RED HILL CREEK EXPRESSWAY INTERCHANGE

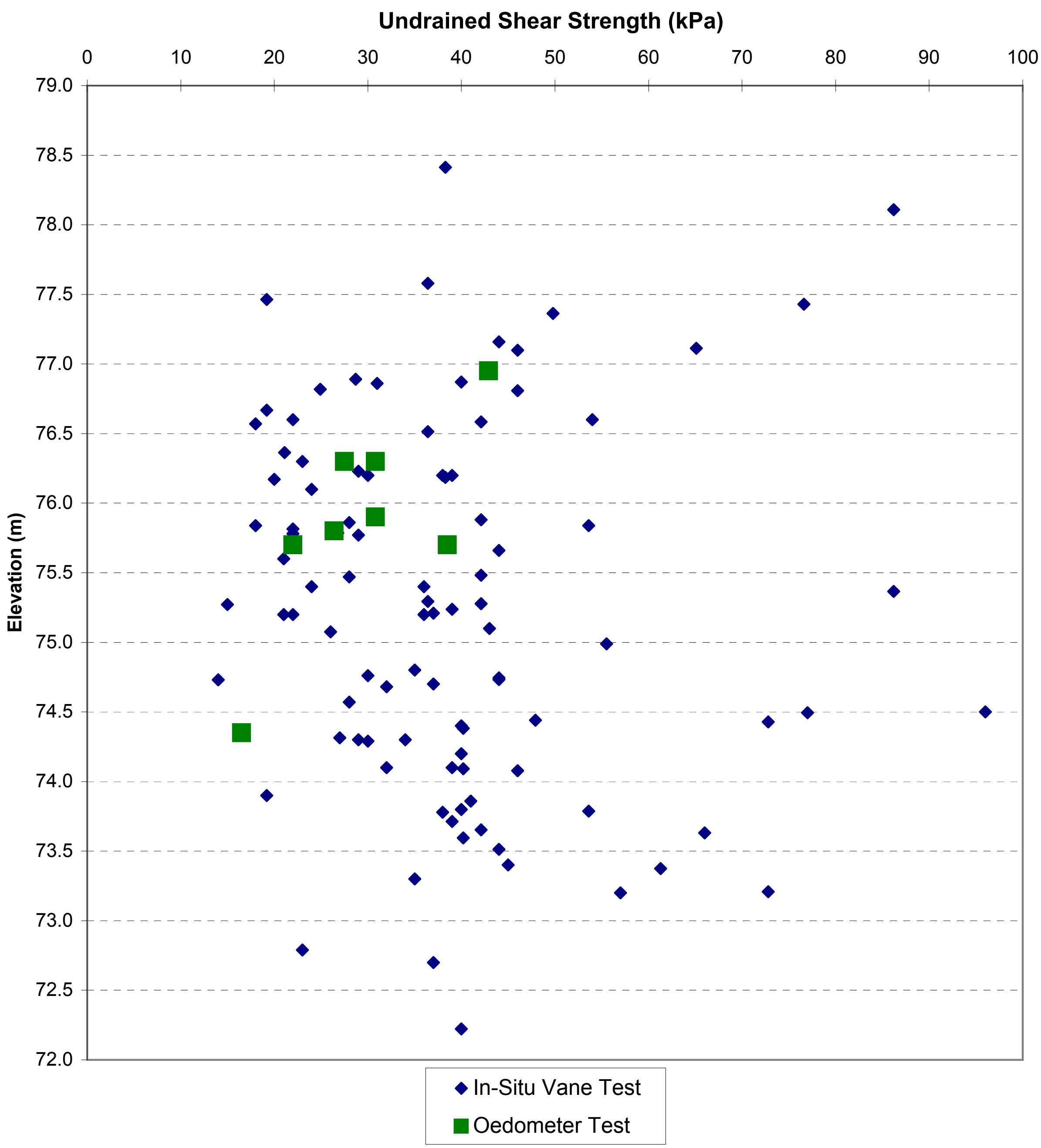
Footing Option	***NF***	***Advantages***	***Disadvantages***	***Relative Costs***	***Risks/Consequences***
Spread Footings	NF		Low geotechnical resistance. Potential differential settlement; consolidation of soft to firm clayey silt due to embankment loading. Groundwater control may be required.		Differential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

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

FIGURES

SHEAR STRENGTH SUMMARY
RED HILL CREEK EXPRESSWAY INTERCHANGE

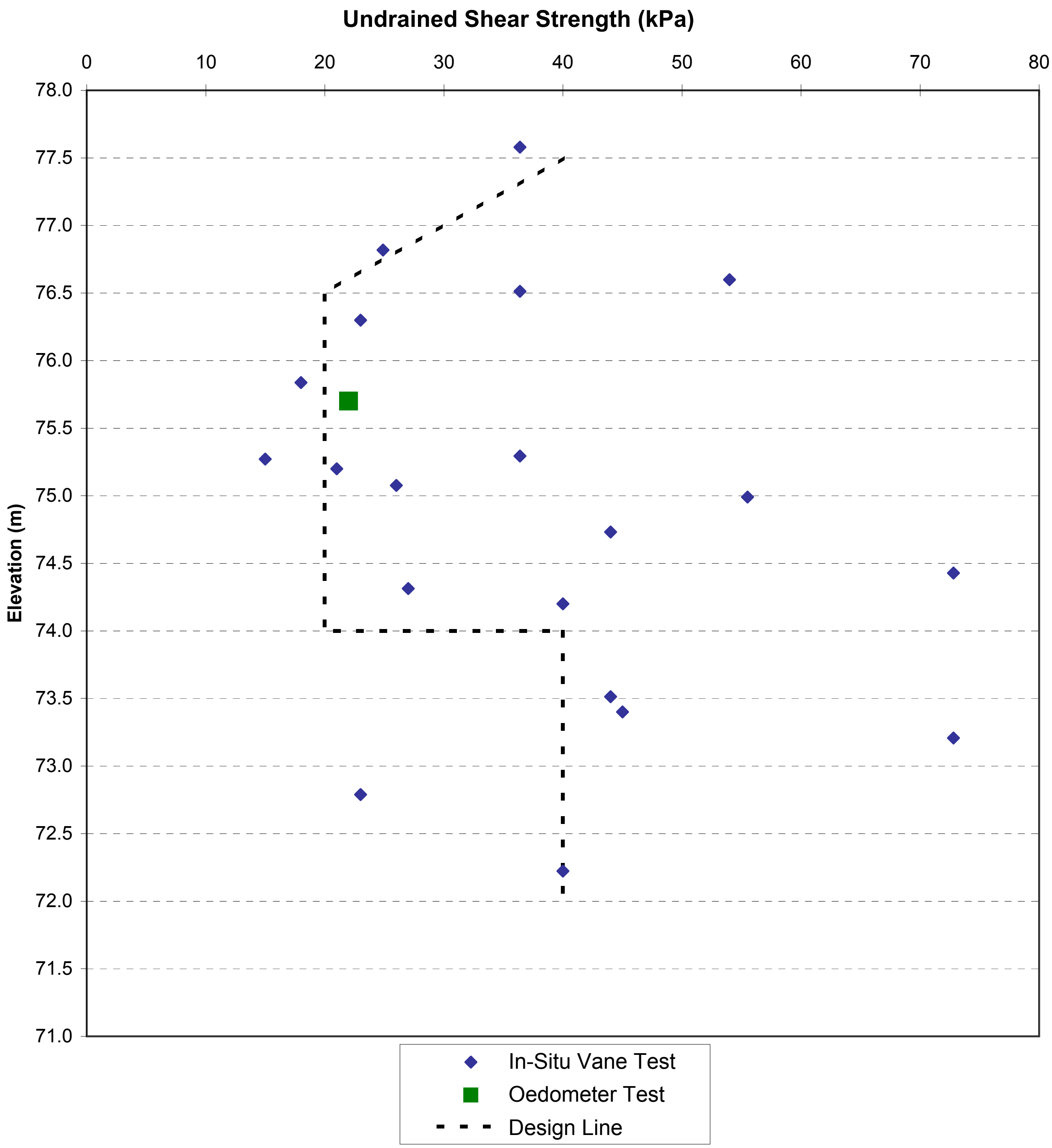
FIGURE 1



Date:	June 2004		Drawn:	SEP
Project:	021-1162-BR10	**Golder Associates**	Checked:	JPD

SHEAR STRENGTH SUMMARY AND DESIGN LINE
BRIDGE 10

FIGURE 2

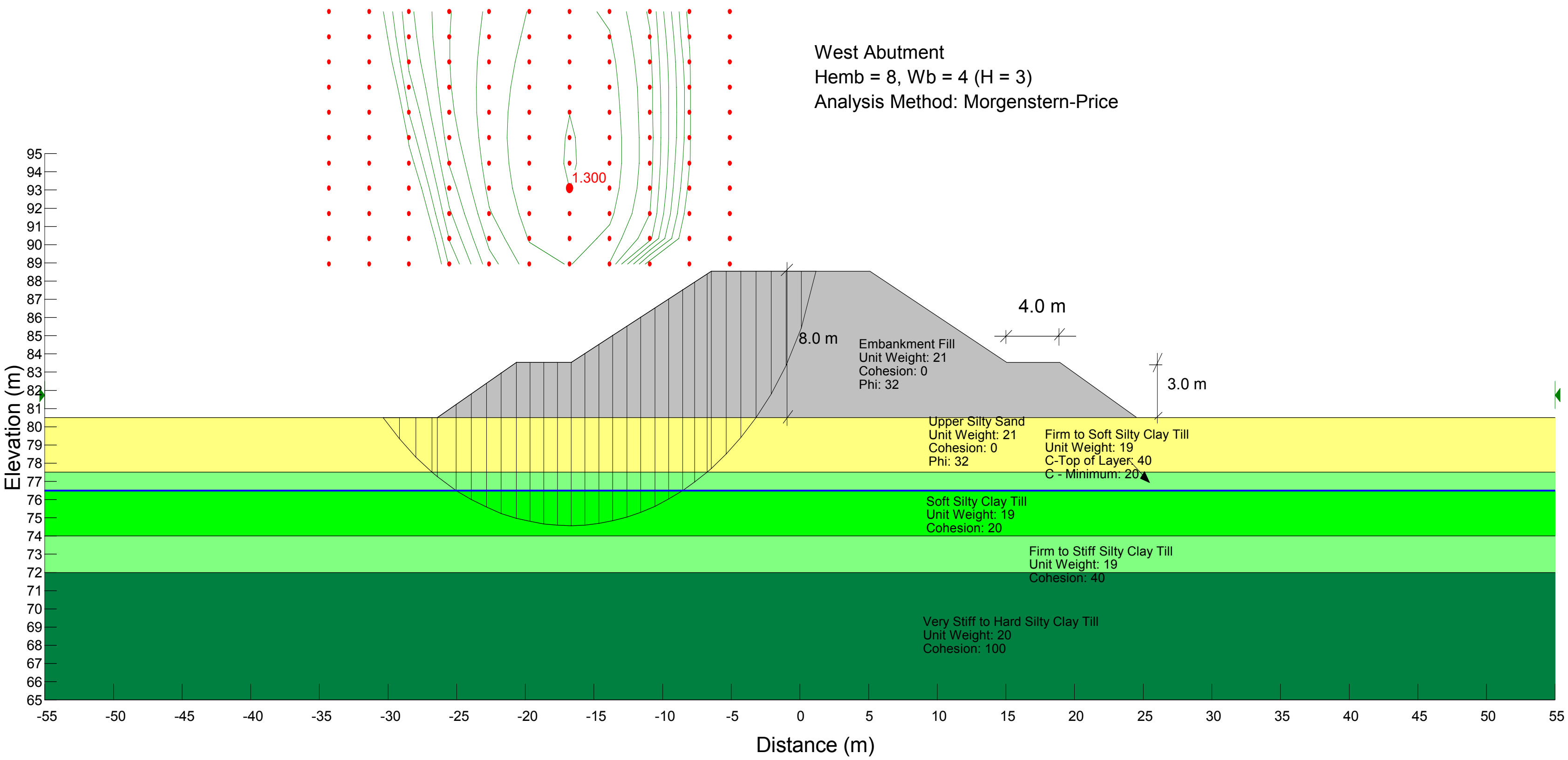


Date: June 2004
Project: 021-1162-BR10

Golder Associates

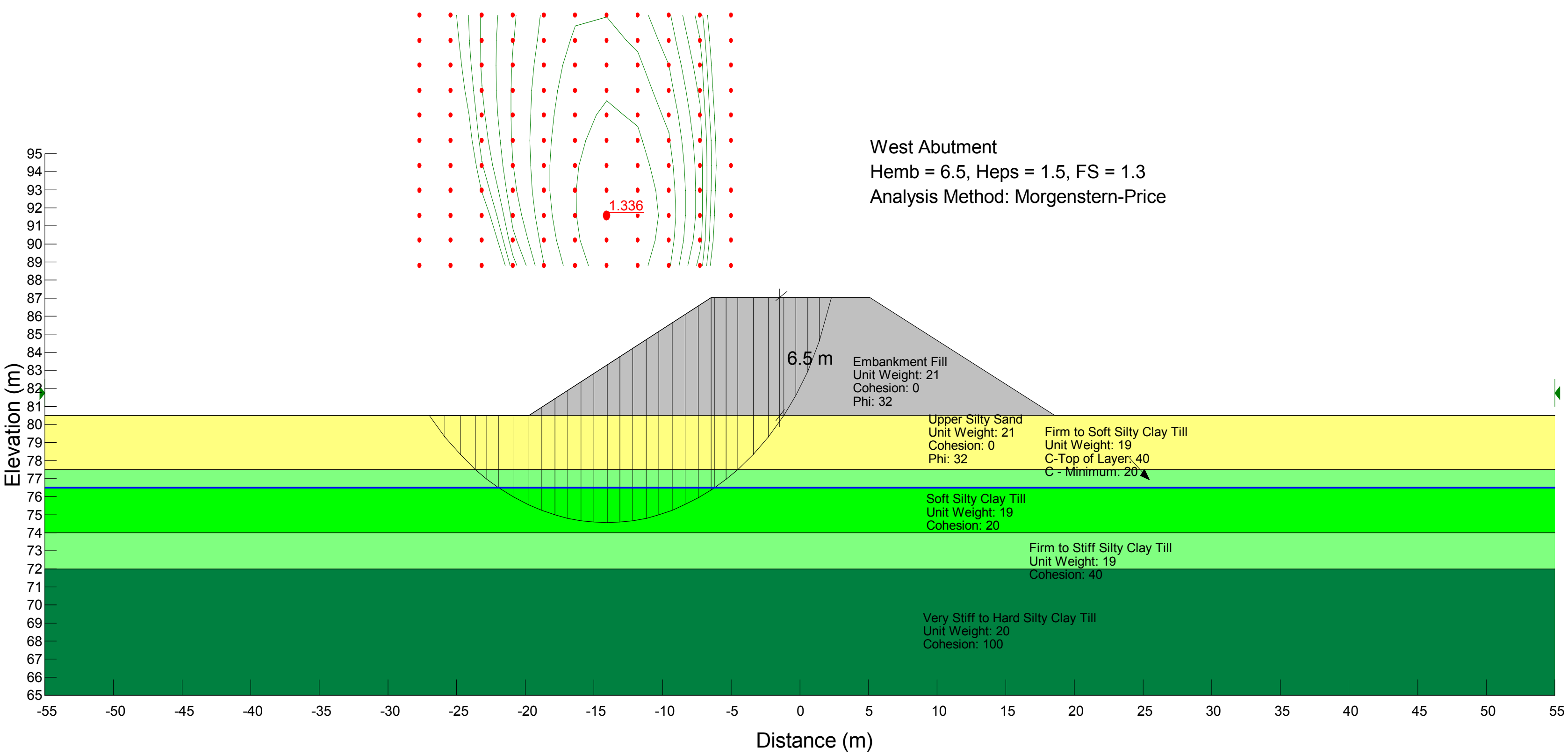
Drawn: SEP
Checked: JPD

APPROACH EMBANKMENT STABILITY ANALYSIS EARTH FILL OPTION	FIGURE 3



Date:	June 2004		Drawn:	SEP
Project:	021-1162-BR10	**Golder Associates**	Checked:	JPD

APPROACH EMBANKMENT STABILITY ANALYSIS EPS AND EARTH FILL OPTION	FIGURE 4



Date:	June 2004		Drawn:	SEP
Project:	021-1162-BR10	**Golder Associates**	Checked:	JPD

DRAWING

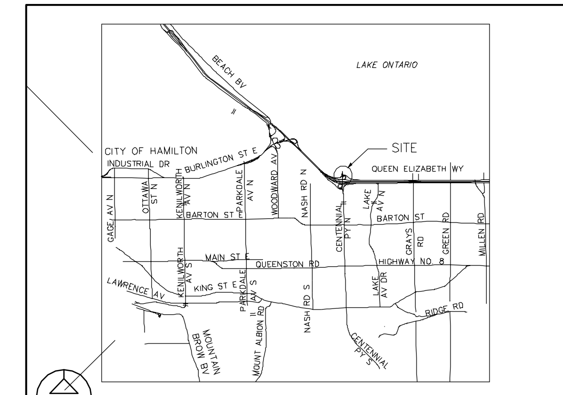
DIST. HWY. QEW CONT No. WP No.	SHEET
REDHILL CREEK EXPRESSWAY E–S RAMP OVER CENTENNIAL PARKWAY (BRIDGE 10) BOREHOLE LOCATION AND SOIL STRATA	

Golder Associates

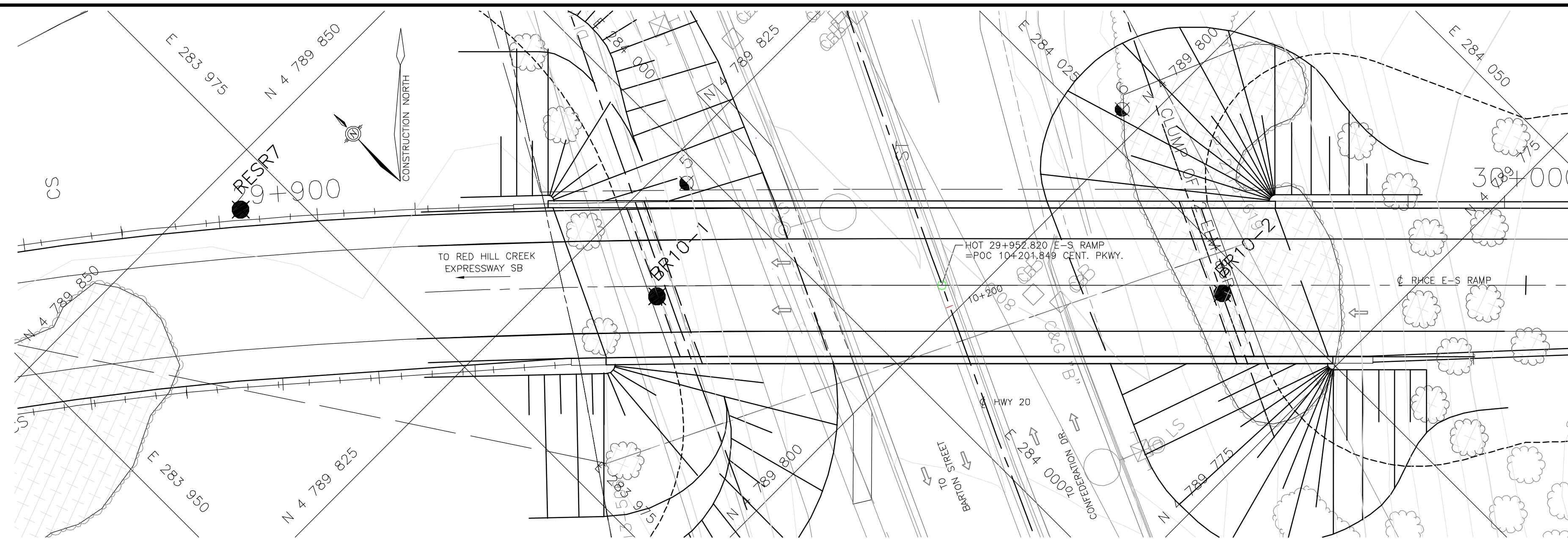
Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA

METRIC

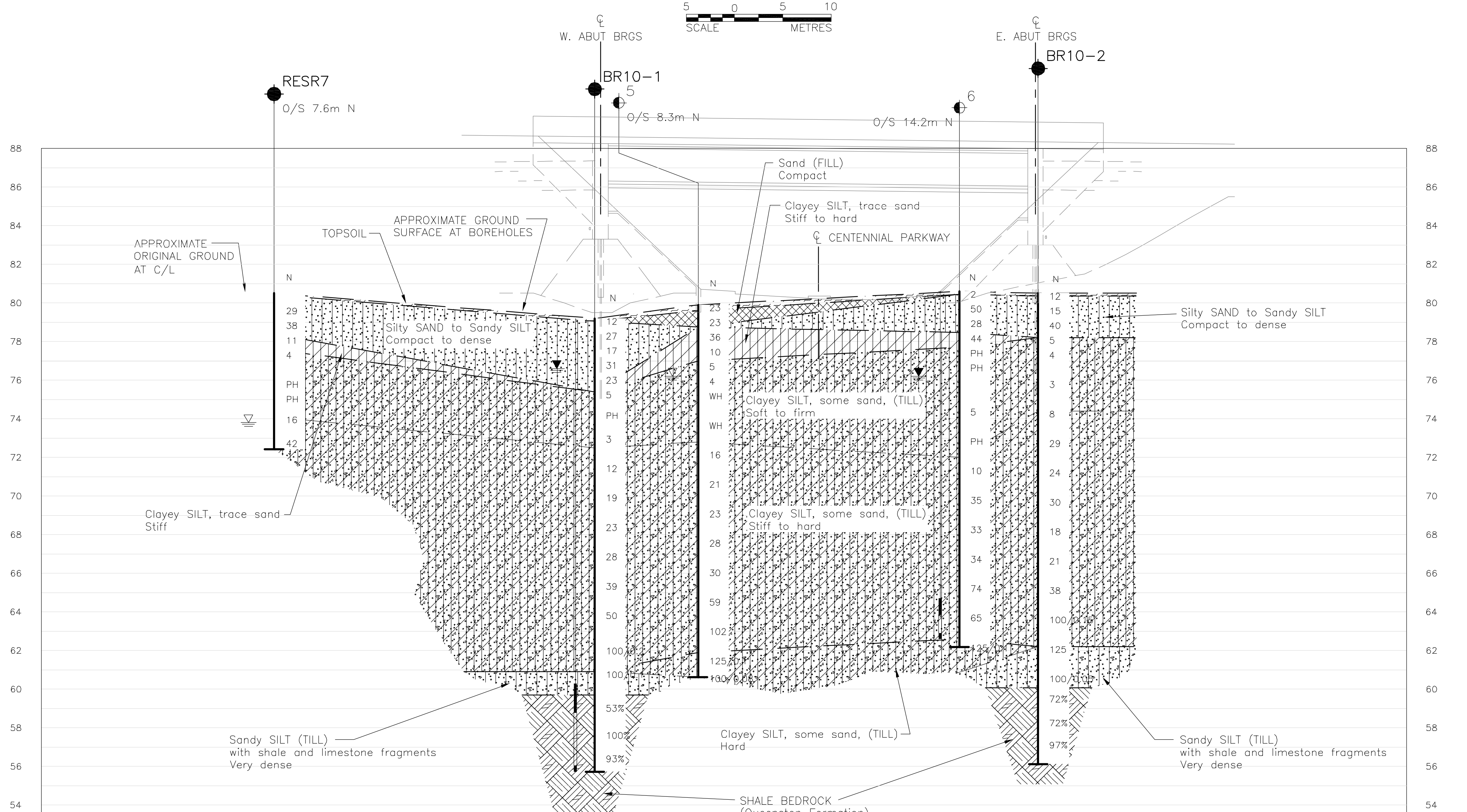
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN



KEY PLAN



PLAN



SECTION/PROFILE

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
- WL upon completion of drilling

No.	ELEVATION	CO–ORDINATES NORTHING	CO–ORDINATES EASTING
5	79.9	4789822.8	283998.3
6	80.6	4789802.0	284027.5
BR10–1	79.2	4789818.0	283990.1
BR10–2	80.5	4789785.7	284022.7
RESR7	80.5	4789846.9	283971.2

NOTES

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

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

REFERENCE

General Arrangement plan provided by McCormick Rankin Corporation, S5132–312–001.dwg, received June 2004.

NO.	DATE	BY	REVISION

Geocres No.

HWY. QEW		PROJECT NO. 021–1162	DIST.
SUBM'D.	CHKD. SEP	DATE: DEC. 2003	SITE:
DRAWN: JDR	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

II. PENETRATION RESISTANCE

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

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

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

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

III. SOIL DESCRIPTION

(a) Cohesionless Soils

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

(b) Cohesive Soils

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

IV. SOIL TESTS

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

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

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LIST OF SYMBOLS

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

I. General

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

II. STRESS AND STRAIN

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

III. SOIL PROPERTIES

(a) Index Properties

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

(a) Index Properties (continued)

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

(b) Hydraulic Properties

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

(c) Consolidation (one-dimensional)

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

(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 δ
c'	effective cohesion
c_u, s_u	undrained shear strength ($\phi = 0$ analysis)
p	mean total stress $(\sigma_1 + \sigma_3)/2$
p'	mean effective stress $(\sigma'_1 + \sigma'_3)/2$
q	$(\sigma_1 + \sigma_3)/2$ or $(\sigma'_1 + \sigma'_3)/2$
q_u	compressive strength $(\sigma_1 + \sigma_3)$
S_t	sensitivity

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

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

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
II - Parallel To	
$\perp$ - Perpendicular To	

Form...G.A.-R-3

Golder Associates

RECORD OF BOREHOLE No 5

METRIC

PROJECT 021-1162
W.P. 441-97-00
LOCATION N 4789822.8 ;E 283998.3
ORIGINATED BY GD
DIST HWY QEW
BOREHOLE TYPE Power Auger 108mm I.D. Hollow Stem Auger
COMPILED BY KG
DATUM Geodetic
DATE Febuary 17, 1998
CHECKED BY SEP

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
79.9	GROUND SURFACE
0.0	Topsoil
79.6 / 0.3	Sand, trace silt and gravel (FILL) Compact Brown Moist
78.8 / 1.2	Clayey SILT, trace sand, occasional rootlets Hard to stiff Mottled brown Moist
77.0 / 2.9	Clayey SILT, trace sand, occasional gravel; silt seams and parting throughout (TILL) Firm to very soft Grey Moist
72.8 / 7.1	Clayey SILT, trace sand and gravel (TILL) Very stiff to hard Grey becoming reddish-grey with depth Moist

SAMPLES

NUMBER	TYPE	"N" VALUES
1	SS	23
2	SS	23
3	SS	36
4	SS	10
5	SS	5
6	SS	4
7	SS	WH
8	SS	WH
9	SS	16
10	SS	21
11	SS	23
12	SS	28
13	SS	30

GROUND WATER CONDITIONS: ∇ (approx. elev. 76.2)

ELEVATION SCALE: 79, 78, 77, 76, 75, 74, 73, 72, 71, 70, 69, 68, 67, 66, 65

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20, 40, 60, 80, 100

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

Field vane sensitivity values: 2.3, 2.2, 2.9, 2.1, 1.9

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

WATER CONTENT (%): 10, 20, 30

UNIT WEIGHT γ kN/m³

REMARKS & GRAIN SIZE DISTRIBUTION (%)

GR	SA	SI	CL
1	21	48	30
3	15	42	40

Continued Next Page

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

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

RECORD OF BOREHOLE No 5

METRIC

PROJECT	021-1162		
W.P.	441-97-00	LOCATION	N 4789822.8 ;E 283998.3
DIST		HWY	QEW
BOREHOLE TYPE	Power Auger 108mm I.D. Hollow Stem Auger		
DATUM	Geodetic	DATE	Febuary 17, 1998
ORIGINATED BY	GD	COMPILED BY	KG
CHECKED BY	SEP		

SOIL PROFILE		SAMPLES									
ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED, ● QUICK TRIAXIAL, + FIELD VANE, × REMOULDED; 20 40 60 80 100)	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	STRAT PLOT
	--- CONTINUED FROM PREVIOUS PAGE ---										
	Clayey SILT, trace sand and gravel (TILL) Very stiff to hard Grey becoming reddish-grey with depth Moist	14	SS	59		64					
		15	SS	102		63		○			
61.9 18.0	Clayey SILT, some sand, trace gravel, occasional shale and limestone fragments (TILL) Hard Red Dry					62					
		16	SS	125/0.1		61					
60.6 19.3	End of Borehole	17	SS	[illegible]				○			

Notes:
1. Auger and spoon refusal at 19.3 m depth
2. Water level in open hole at 3.7 m depth upon completion of drilling
3. For samples 5, 6 and 7, the in-situ vane was pushed prior to SPT sampling and as such the 'N' values may not be representative

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

○ 3% STRAIN AT FAILURE

Golder Associates

RECORD OF BOREHOLE No 6 — 1 OF 2 — METRIC

PROJECT 021-1162
W.P. 441-97-00 — LOCATION N 4789802.0 ;E 284027.5 — ORIGINATED BY GD
DIST — HWY QEW — BOREHOLE TYPE Power Auger 108mm I.D. Hollow Stem Auger — COMPILED BY KG
DATUM Geodetic — DATE February 18, 1998 — CHECKED BY SEP

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	SHEAR STRENGTH kPa	WATER CONTENT (%)
80.6	GROUND SURFACE					
0.0	Topsoil					
0.2	Silty SAND, trace clay; Dense to compact; Brown; Moist to wet	1	SS	2		
		2	SS	50		○
		3	SS	28		
78.5 / 2.1	Clayey SILT, trace sand; Hard; Grey; Moist	4	SS	44		○
77.7 / 2.9	Clayey SILT, trace sand, occasional gravel, silt seams and parting throughout (TILL); Soft to firm; Grey; Moist	5	TO	PH		
		6	TO	PH		├─○┤
					×+ 1.5	
		7	SS	5		○
		8	TO	PH	× + 1.8	
72.0 / 8.6	Clayey SILT, trace sand and gravel, occasional shale fragments (TILL); Stiff to hard; Grey becoming reddish-grey with depth; Moist	9	SS	10		○
		10	SS	35		
		11	SS	33		○
		12	SS	34		

Elevation scale: 80, 79, 78, 77, 76, 75, 74, 73, 72, 71, 70, 69, 68, 67, 66

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

Continued Next Page

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

Golder Associates

RECORD OF BOREHOLE No 6

METRIC

PROJECT 021-1162
W.P. 441-97-00
LOCATION N 4789802.0 ;E 284027.5
ORIGINATED BY GD
DIST
HWY QEW
BOREHOLE TYPE Power Auger 108mm I.D. Hollow Stem Auger
COMPILED BY KG
DATUM Geodetic
DATE February 18, 1998
CHECKED BY SEP

ELEV / DEPTH	DESCRIPTION (SOIL PROFILE)	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, X REMOULDED; 20 40 60 80 100)	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	--- CONTINUED FROM PREVIOUS PAGE ---										
	Clayey SILT, trace sand and gravel, occasional shale fragments (TILL) Stiff to hard Grey becoming reddish-grey with depth Moist		13	SS	74		65		o		
							64				
			14	SS	65						
							63				
62.6 18.0	Clayey SILT, some sand, trace gravel, occasional shale and limestone pieces (TILL) Red Hard Dry										
62.2 18.4	End of Borehole		15	SS	25/0						

Notes:
1. Auger and spoon refusal at 18.4 m depth
2. Borehole dry upon completion of drilling
3. Water level in piezometer at Elev. 76.3 m on March 25, 1998 and at Elev. 76.2 m on Oct. 22, 2003

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

Golder Associates

RECORD OF BOREHOLE No BR10-1

METRIC

PROJECT 021-1162
W.P. 441-97-00
DIST ___ HWY QEW
DATUM Geodetic

LOCATION N 4789818.0 ;E 283990.1
BOREHOLE TYPE Power Auger 108mm I.D. Hollow Stem Auger
DATE August 20, 2003

ORIGINATED BY PKS
COMPILED BY KG
CHECKED BY SEP

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
79.2	GROUND SURFACE				
0.0	Sandy Topsoil				
0.2	Sandy SILT to Silty SAND, some clay and gravel, occasional pockets of silty clay Compact to dense Brown to grey Moist to wet	1	SS	12	
		2	SS	27	
		3	SS	17	
		4	SS	31	
		5	SS	23	
75.4 / 3.8	Clayey SILT, some sand, trace gravel (TILL) Firm Grey Wet	6	SS	5	
		7	TO	PH	
		8	SS	3	
72.5 / 6.7	Clayey SILT, some sand, trace gravel (TILL) Stiff to hard Grey to reddish-grey Moist	9	SS	12	
		10	SS	19	
		11	SS	23	3 15 49 33
		12	SS	28	
		13	SS	39	

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

Field vane sensitivity values: 2.4, 1.9, 2.4, 1.9

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

UNIT WEIGHT γ kN/m³

ELEVATION SCALE: 79, 78, 77, 76, 75, 74, 73, 72, 71, 70, 69, 68, 67, 66, 65

Continued Next Page

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

Golder Associates

RECORD OF BOREHOLE No BR10-1

METRIC

PROJECT 021-1162
W.P. 441-97-00
DIST ____ HWY QEW
DATUM Geodetic

LOCATION N 4789818.0 ;E 283990.1
BOREHOLE TYPE Power Auger 108mm I.D. Hollow Stem Auger
DATE August 20, 2003

ORIGINATED BY PKS
COMPILED BY KG
CHECKED BY SEP

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES
	Clayey SILT, some sand, trace gravel (TILL) Stiff to hard Grey to reddish-grey Moist Shale and limestone pieces below 15.2m depth	14	SS	50
		15	SS	100/0.2
60.9 18.3	Sandy SILT, trace clay and gravel with shale and limestone fragments (TILL) Very dense Red Moist	16	SS	100/0
59.7 19.5	Moderately to highly weathered, reddish-grey, calcareous SHALE BEDROCK (Queenston Formation) with occasional grey limestone seams/layers Bedrock cored from 19.5m to 23.4m For details of bedrock coring see Record of Drillhole BR10-1			
55.7 23.5	End of Borehole			

Notes:
1. Auger refusal at 19.5m depth (Elev. 59.7m)
2. Water level in piezometer at Elev. 75.7 m on Aug. 22, 1998 and at Elev. 76.6 m on Oct. 22, 2003

ELEVATION SCALE: 64, 63, 62, 61, 60, 59, 58, 57, 56

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

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

RECORD OF DRILLHOLE: BR10-1

PROJECT: 021-1162

LOCATION: N 4789818.0; E 283990.1

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: August 20, 2003

DRILL RIG: Bomb CME 75

DRILLING CONTRACTOR: Geo-Environmental Drilling Ltd.

DATUM: Geodetic

FR/FX-FRACTURE, F-FAULT, SM-SMOOTH, FL-FLEXURED, BC-BROKEN CORE
CL-CLEAVAGE, J-JOINT, R-ROUGH, UE-UNEVEN, MB-MECH. BREAK
SH-SHEAR, P-POLISHED, ST-STEPPED, W-WAVY, B-BEDDING
VN-VEIN, S-SLICKENSIDED, PL-PLANAR, C-CURVED

DEPTH SCALE METRES	DESCRIPTION	ELEV. DEPTH (m)	RUN No.	NOTES WATER LEVELS INSTRUMENTATION
	Refer to previous page	59.70		
20–23	Moderately to highly weathered, thinly layered, reddish-grey to light grey, fine grained, very weak to medium strong, calcareous SHALE BEDROCK (Queenston Formation) Occasional seams/layers of light grey, medium to very strong limestone Run No. 1: 21% limestone Run No. 3: 13% limestone All fractures are bedding, smooth	19.50	1, 2, 3	
		55.70 23.50		
	End of Drillhole			

Column headings: DRILLING RECORD; SYMBOLIC LOG; PENETRATION RATE (m/min); COLOUR % RETURN; FLUSH; RECOVERY: TOTAL CORE % (80 60 40 20), SOLID CORE % (80 60 40 20); R.Q.D. % (80 60 40 20); FRACT. INDEX PER 0.3 (5 10 15 20); DISCONTINUITY DATA: DIP w.r.t. CORE AXIS (0 30 60 90), TYPE AND SURFACE DESCRIPTION; HYDRAULIC CONDUCTIVITY K, cm/sec (10^{-6} 10^{-5} 10^{-4} 10^{-3}); DIAMETRAL POINT LOAD INDEX (MPa) (2 4 6)

Depth scale: 20, 21, 22, 23, 24, 25, 26, 27, 28, 29

MISS_ROCK 0211162EARCK.GPJ GLDR_CAN.GDT 18/6/04 TM

DEPTH SCALE
1 : 50

Golder Associates

LOGGED: PS

CHECKED: SEP

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RECORD OF BOREHOLE No BR10-2

METRIC

PROJECT 021-1162
W.P. 441-97-00
LOCATION N 4789785.7 ;E 284022.7
ORIGINATED BY PKS
DIST HWY QEW
BOREHOLE TYPE Power Auger 108mm I.D. Hollow Stem Auger
COMPILED BY KG
DATUM Geodetic
DATE August 21, 2003
CHECKED BY SEP

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES
80.5	GROUND SURFACE			
0.0	Sandy Topsoil			
0.2	Sandy SILT, trace to some clay, trace gravel, trace organics, occasional pockets of silty clay Compact to dense Brown Moist	1	SS	12
		2	SS	15
		3	SS	40
78.2 2.3	Clayey SILT, trace to some sand, trace gravel, occasional sandy silt seam (TILL) Firm to stiff Grey Moist to wet	4	SS	5
		5	SS	4
	Becoming wet below 4.6m depth	6	SS	3
74.4 6.1	Clayey SILT, some sand, trace gravel (TILL) Stiff to hard Grey to reddish-grey Wet to moist	7	SS	8
	Shale and limestone fragments below 7.6m depth	8	SS	29
		9	SS	24
		10	SS	30
		11	SS	18
		12	SS	21

Column headings: GROUND WATER CONDITIONS; ELEVATION SCALE (80, 79, 78, 77, 76, 75, 74, 73, 72, 71, 70, 69, 68, 67, 66); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

Field vane sensitivity values: 2.5, 2.4, 1.7, 1.7, 1.5

Continued Next Page

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

RECORD OF BOREHOLE No BR10-2 METRIC

PROJECT 021-1162			
W.P. 441-97-00	LOCATION N 4789785.7 ;E 284022.7		ORIGINATED BY PKS
DIST	HWY QEW	BOREHOLE TYPE Power Auger 108mm I.D. Hollow Stem Auger	COMPILED BY KG
DATUM Geodetic	DATE August 21, 2003		CHECKED BY SEP

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Clayey SILT, some sand, trace gravel (TILL) Stiff to hard Grey to reddish-grey Wet to moist	13	SS	38	65	
					64	
		14	SS	100/0.15		13 17 50 20
					63	
62.2 18.3	Sandy SILT, trace clay, trace gravel with shale and limestone pieces (TILL) Very dense Red Moist	15	SS	125	62	
					61	
		~~16~~	~~SS~~	~~100/0.05~~		
60.1 20.4	Moderately weathered, reddish-grey, calcareous SHALE BEDROCK (Queenston Formation) with occasional grey limestone layers Bedrock cored from 20.4m to 24.4m For details of bedrock coring see Record of Drillhole BR10-2				60	
					59	
					58	
					57	
56.1 24.4	End of Borehole Notes: 1. Auger and spoon refusal at 20.4m depth (Elev. 60.1m)					

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

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

RECORD OF DRILLHOLE: BR10-2

PROJECT: 021-1162

SHEET 1 OF 1

LOCATION: 4789785.7; E 280422.7

DRILLING DATE: August 22, 2003

DATUM: Geodetic

DRILL RIG: Bomb CME 75

INCLINATION: -90° AZIMUTH: ---

DRILLING CONTRACTOR: Geo-Environmental Drilling Ltd.

FR/FX-FRACTURE, F-FAULT, SM-SMOOTH, FL-FLEXURED, BC-BROKEN CORE
CL-CLEAVAGE, J-JOINT, R-ROUGH, UE-UNEVEN, MB-MECH. BREAK
SH-SHEAR, P-POLISHED, ST-STEPPED, W-WAVY, B-BEDDING
VN-VEIN, S-SLICKENSIDED, PL-PLANAR, C-CURVED

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (m/min)	FLUSH COLOUR % RETURN	RECOVERY TOTAL CORE %	RECOVERY SOLID CORE %	R.Q.D. %	FRACT. INDEX PER 0.3	DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION	HYDRAULIC CONDUCTIVITY K, cm/sec	DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
		Refer to previous page														
21–24	NQ CORING	Moderately weathered, thinly layered, reddish-grey to light grey, fine grained, very weak to medium strong, calcareous SHALE BEDROCK (Queenston Formation) Occasional seams/layers of light grey, medium to very strong limestone Run No. 1: 11% limestone Run No. 2: 10% limestone Run No. 3: 12% limestone All fractures are bedding, smooth to rough		60.10 20.40	1 2 3											
		End of Drillhole		56.12 24.38												

Depth scale: 21, 22, 23, 24, 25, 26, 27, 28, 29, 30

MISS_ROCK 0211162EARCK.GPJ GLDR_CAN.GDT 18/6/04 TM

DEPTH SCALE

1 : 50

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LOGGED: PS

CHECKED: SEP

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Foundation Design

RECORD OF BOREHOLE No RESR-7

METRIC

PROJECT 021-1162

W.P. 441-97-00

LOCATION N 4789846.9 ;E 283971.2

ORIGINATED BY PKS

DIST ___ HWY QEW

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

COMPILED BY KG

DATUM Geodetic

DATE September 10, 1998

CHECKED BY SEP

ELEV / DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	UNIT WEIGHT γ kN/m³	GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
80.5	GROUND SURFACE						
80.4 / 0.1	Topsoil						
	Sandy SILT to SILT, trace to some clay, trace gravel, Compact to dense, Brown, Moist	1	SS	29			
		2	SS	38			0 16 73 11
78.4 / 2.1	Clayey SILT, trace sand, Stiff, Grey, Moist to wet	3	SS	11			0 1 65 34
77.6 / 2.9	Clayey SILT, trace sand, occasional gravel, occasional silt seams (TILL), Soft to firm, Grey, Moist to wet	4	SS	4			
						18.8	0 1 52 47
		5	TO	PH			0 0 56 44
		6	TO	PH			
74.1 / 6.4	Clayey SILT, trace sand and gravel (TILL), Very stiff to hard, Grey, Moist	7	SS	16	∇		
		8	SS	42			
72.4 / 8.1	End of Borehole						

Notes:
1. Water level in open hole at 6.7 m depth upon completion of drilling

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

Field vane sensitivity values: 3 (at approx. El. 76), 2.4 (at approx. El. 75.2), 2 (at approx. El. 74.2)

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

WATER CONTENT (%): 10 20 30

ELEVATION SCALE: 80, 79, 78, 77, 76, 75, 74, 73

+3, ×3: Numbers refer to Sensitivity

O 3% STRAIN AT FAILURE

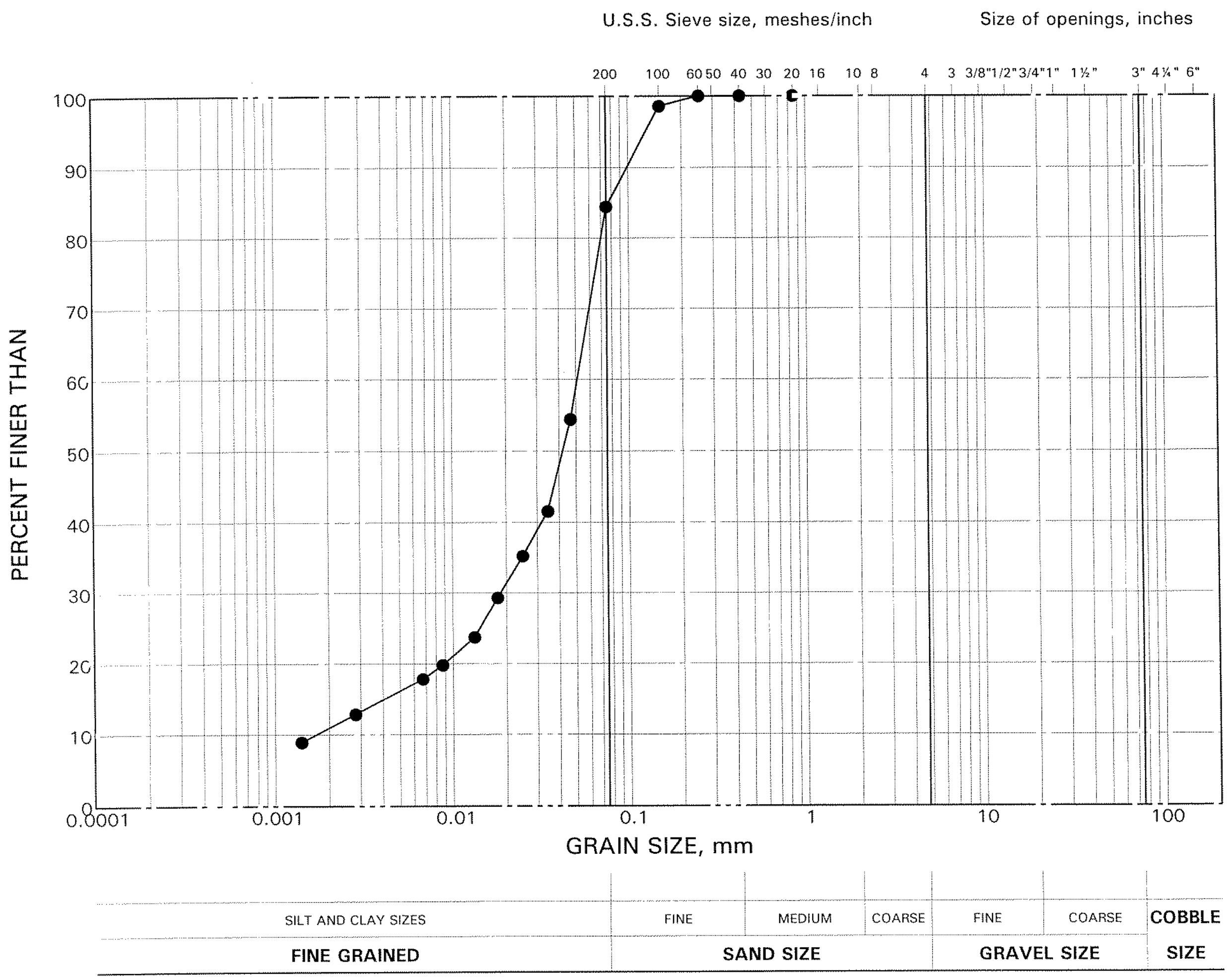
MISS_MTO 0211162EAMTO.GPJ ON_MOT.GDT 18/6/04

APPENDIX A

LABORATORY TEST DATA

GRAIN SIZE DISTRIBUTION
Silt, some sand and clay

FIGURE A1



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
●	RESR-7	2	78.5

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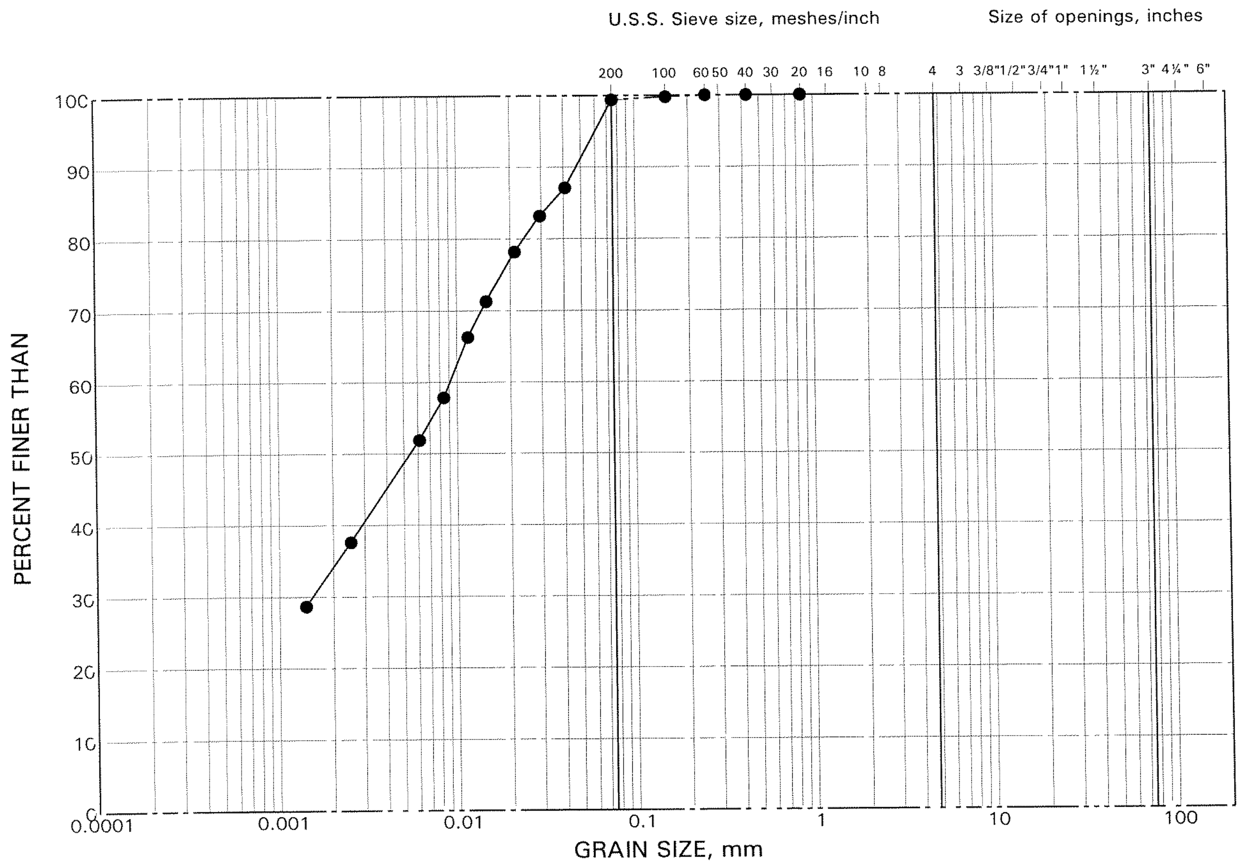
GRAIN SIZE DISTRIBUTION
Clayey Silt, trace sand

FIGURE A2

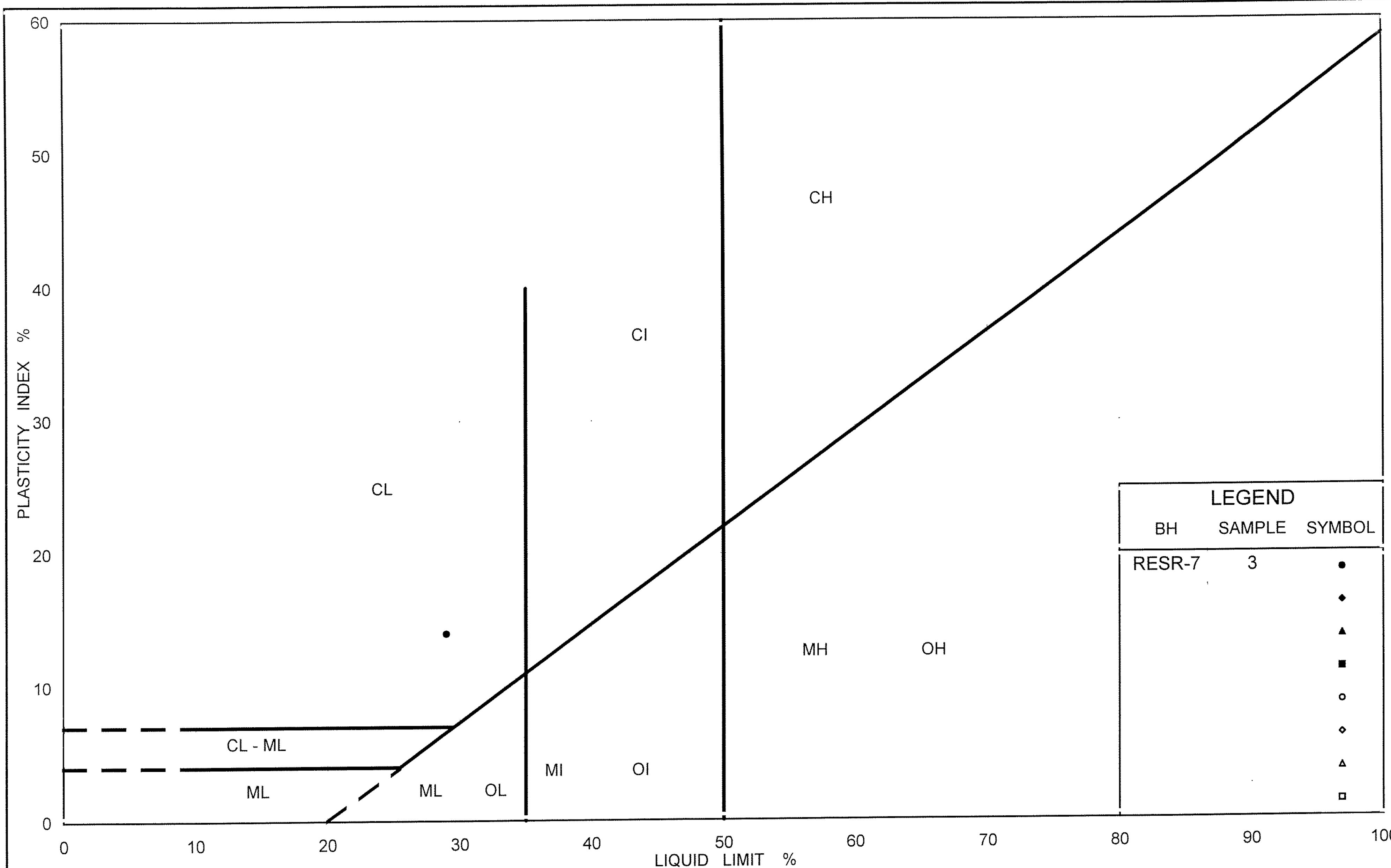


SILT AND CLAY SIZES	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE
FINE GRAINED	**SAND SIZE**			**GRAVEL SIZE**		**SIZE**

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
●	RESR-7	3	77.8

Oct 75, FF-S-21



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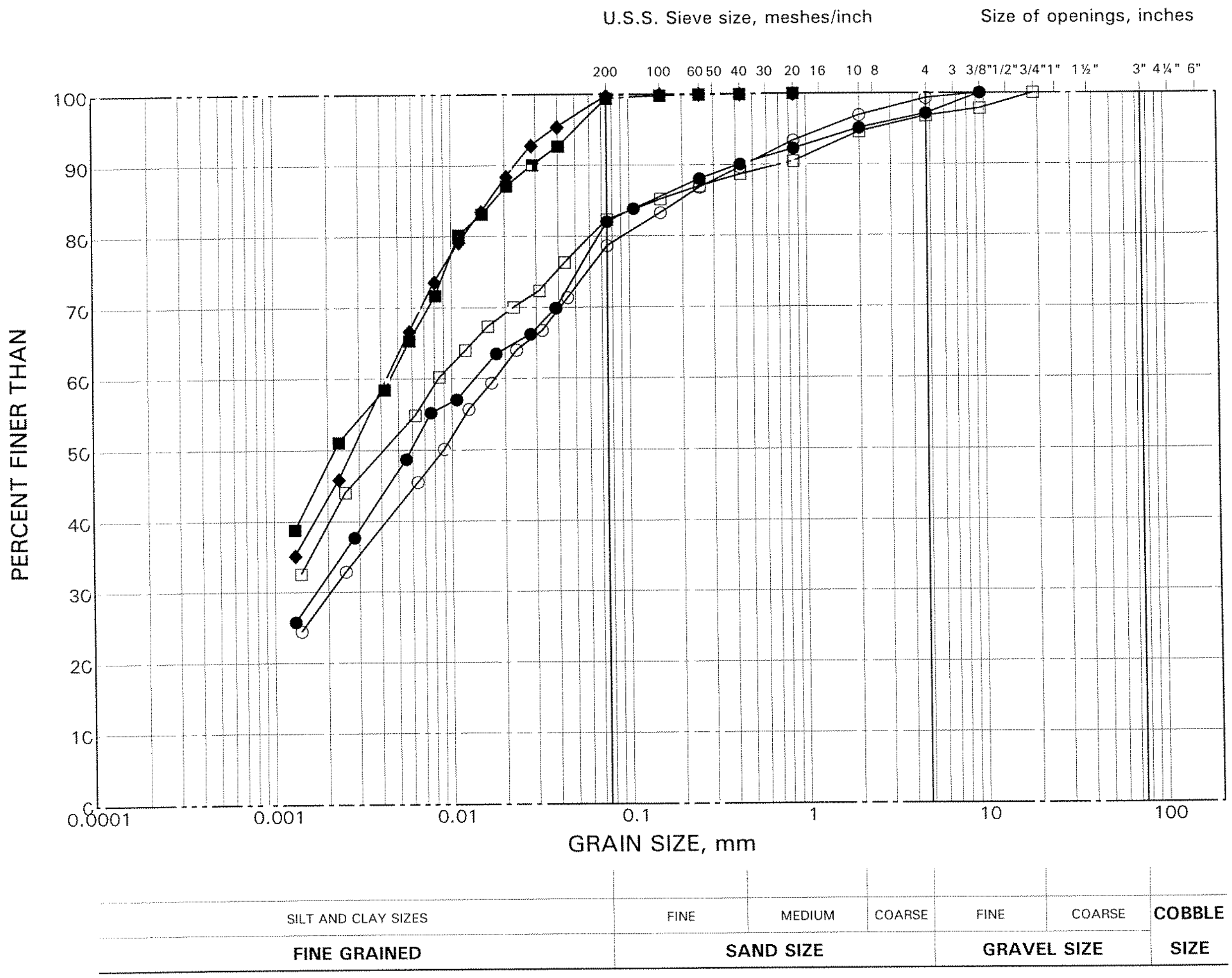
PLASTICITY CHART
Clayey Silt, trace sand

FIG No. A3

Project No. 021-1162-BR10

GRAIN SIZE DISTRIBUTION
Clayey Silt (Till)

FIGURE A4



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
●	BR10-1	11	68.1
■	RESR-7	5	75.6
◆	RESR-7	5	76.0
○	5	7	74.9
□	5	13	65.7

Project 021-1162-BR10

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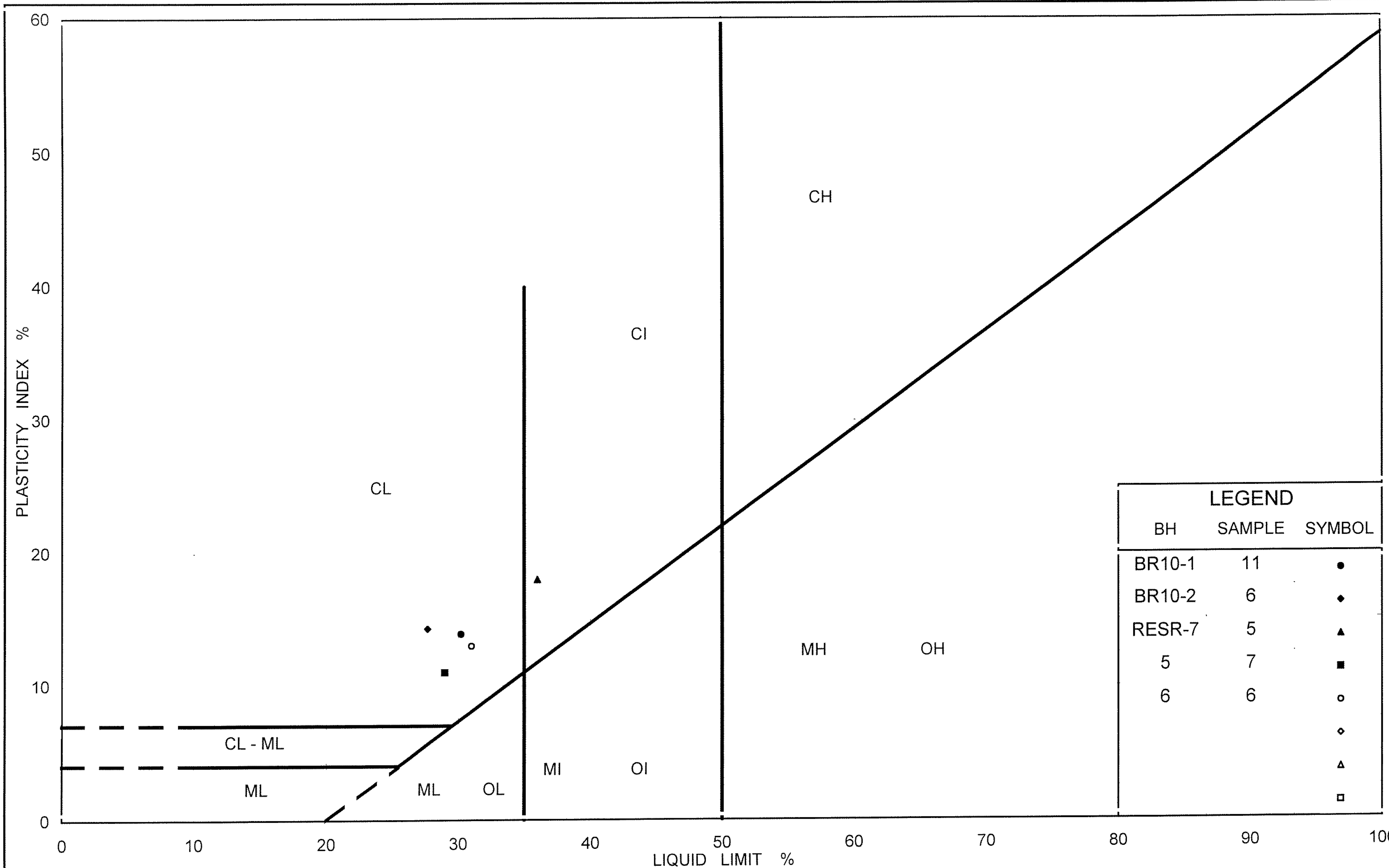
Oct 75, FF-S-21



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PLASTICITY CHART
Clayey Silt (Till)

FIG No. A5

Project No. 021-1162-BR10

CONSOLIDATION TEST DATA SUMMARY
BOREHOLE RESR-7 SAMPLE 5

FIGURE A6

PROJECT	981-8033	SPECIFIC GRAVITY	2.70	measured	DATE STARTED	98-10-16
BOREHOLE	RESR-7	AREA(mm^2)	3154.96		DATE COMPLETED	98-10-31
SAMPLE	5	SOLIDS HT.2HS	10.002			
DEPTH, m		DRY WEIGHT, g	85.04			

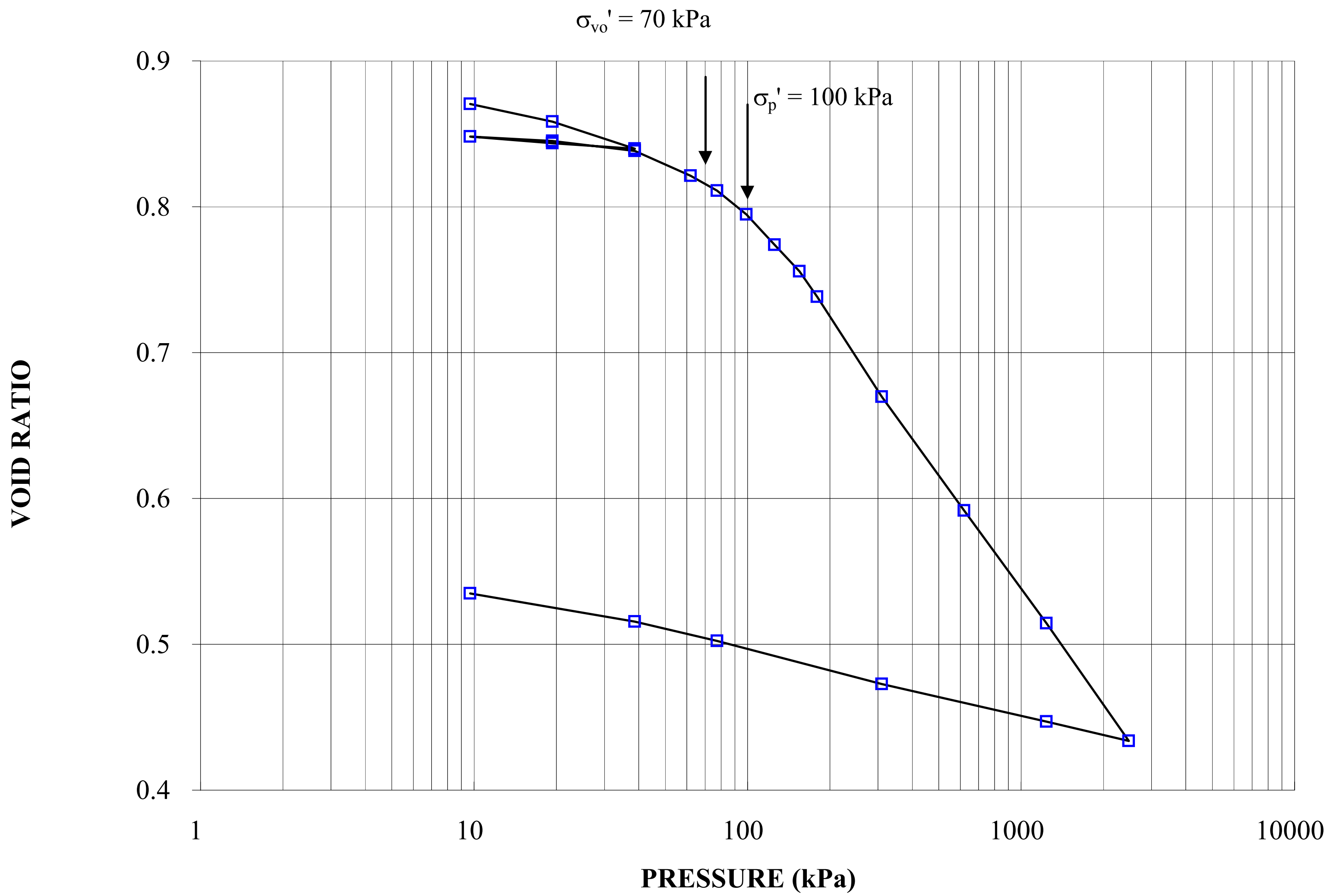
Load kPa	Corr. Height mm	Void Ratio	Average Height mm	t_{90} sec	t_{50} sec	cv. t_{90} cm^2/s	k cm/s	mv m^2/kN
0.00	19.050	0.905	19.050					
9.67	18.707	0.870	18.879	27		2.80E-02	5.11E-06	1.86E-03
19.39	18.588	0.858	18.648	8		9.21E-02	5.81E-06	6.43E-04
38.77	18.399	0.840	18.494	18		4.03E-02	2.02E-06	5.12E-04
19.39	18.438	0.844	18.419					1.06E-04
9.69	18.484	0.848	18.461					2.49E-04
19.39	18.452	0.845	18.468	45		1.61E-02	2.73E-07	1.73E-04
38.77	18.385	0.838	18.419	15		4.79E-02	8.52E-07	1.81E-04
62.03	18.215	0.821	18.300	8		8.87E-02	3.34E-06	3.84E-04
77.54	18.113	0.811	18.164	51		1.37E-02	4.64E-07	3.45E-04
99.25	17.948	0.795	18.031	29		2.38E-02	9.29E-07	3.99E-04
125.62	17.742	0.774	17.845	26		2.60E-02	1.04E-06	4.10E-04
155.09	17.560	0.756	17.651	55		1.20E-02	3.82E-07	3.24E-04
179.90	17.385	0.738	17.473	11		5.88E-02	2.13E-06	3.70E-04
310.17	16.700	0.670	17.043	585		1.05E-03	2.85E-08	2.76E-04
620.34	15.918	0.592	16.309	315		1.79E-03	2.32E-08	1.32E-04
1240.68	15.145	0.514	15.532	165		3.10E-03	1.99E-08	6.54E-05
2481.36	14.338	0.434	14.742	52		8.86E-03	2.96E-08	3.41E-05
1240.68	14.471	0.447	14.405					
310.17	14.729	0.473	14.600					
77.54	15.026	0.502	14.878					
38.77	15.156	0.515	15.091					
9.69	15.352	0.535	15.254					

Notes:
k calculated using Cv based on t90 values.

Water Content %, initial	35.4
Water Content %, final	23.5
Original Volume, cc	60.10
Volume of Solids, cc	31.55
Volume of Voids, cc	28.55
Degree of Saturation, %	105.5

Unit Weight, kN/m^3	18.79
Dry Unit Weight, kN/m^3	13.88

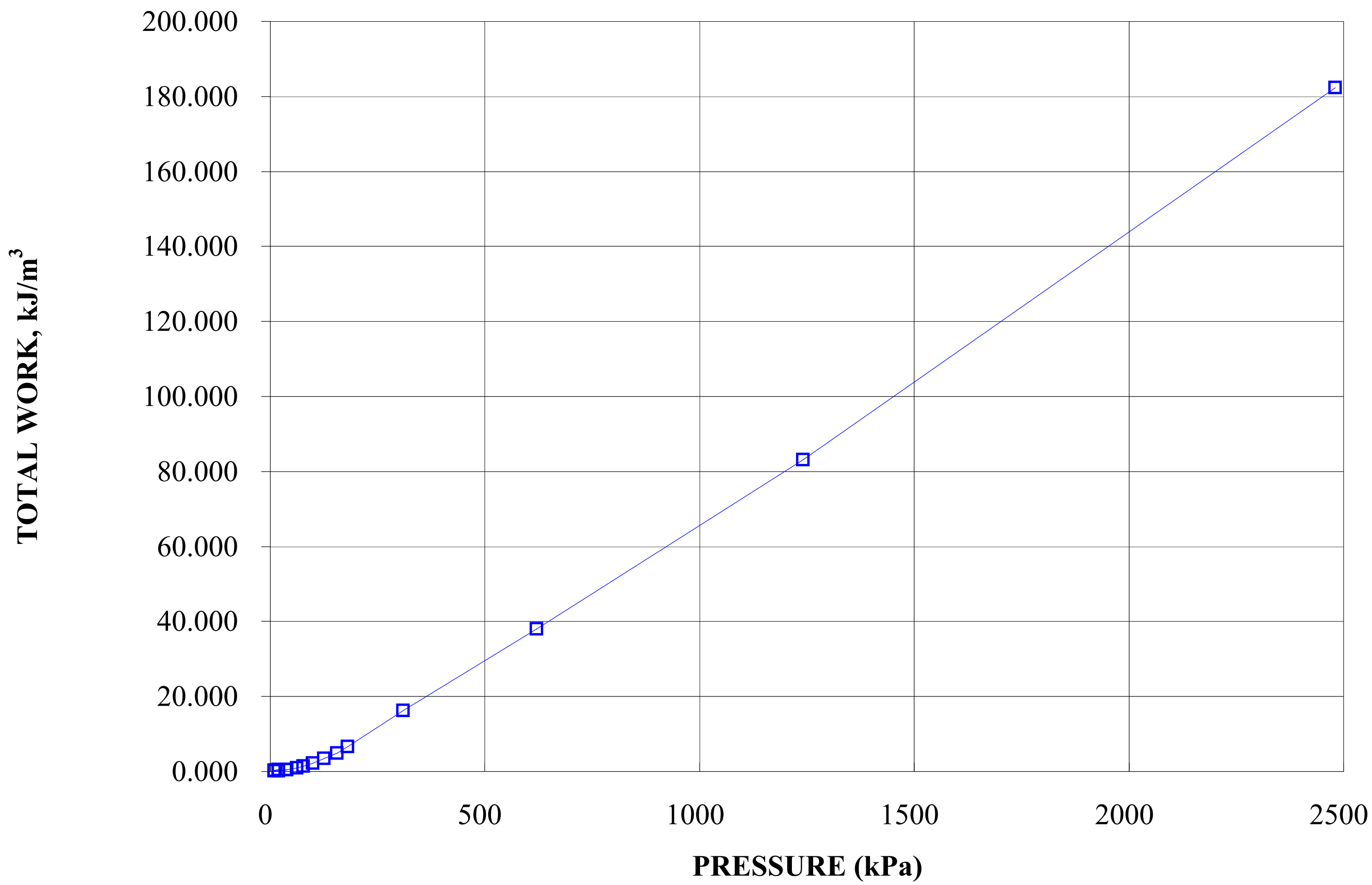
CONSOLIDATION TEST
VOID RATIO vs LOG. PRESSURE



CONSOLIDATION TEST
VOID RATIO VS. LOG PRESSURE
BOREHOLE RESR-7 SAMPLE 5

FIGURE A6
Page 2 of 3

CONSOLIDATION TEST
TOTAL WORK, kJ/m^3 vs PRESSURE



CONSOLIDATION TEST
TOTAL WORK, kJ/m^3 vs PRESSURE
BOREHOLE RESR-7 SAMPLE 5

FIGURE A6
Page 3 of 3

CONSOLIDATION SUMMARY

FIGURE A7

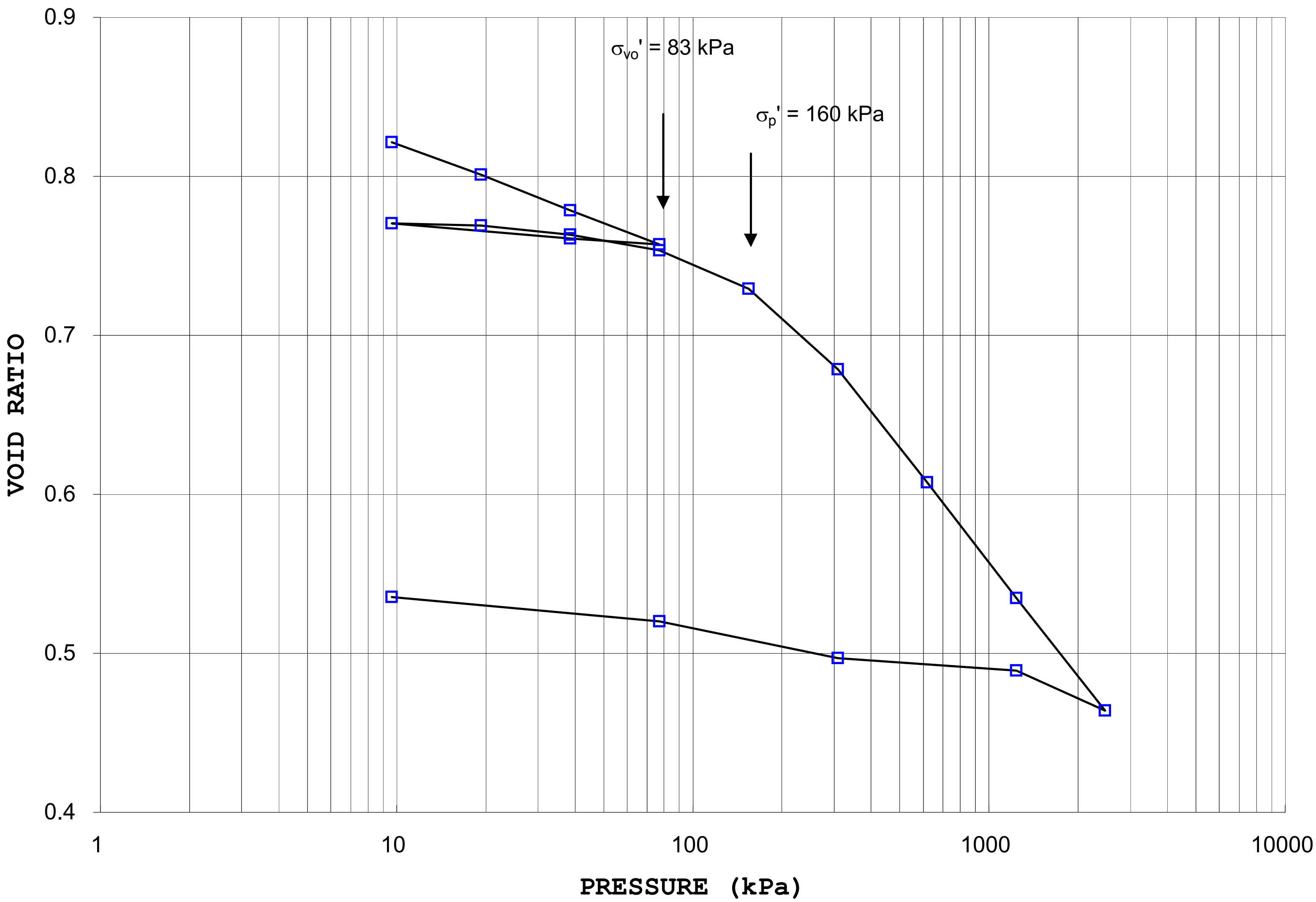
PROJECT	971-1108	SPECIFIC GRAVITY	2.73 measured	DATE STARTED	97-03-10
SAMPLE	BH 6 SA 6	AREA(mm2)	3166.9	DATE COMPLETED	97-03-10
DEPTH,m	4.1	SOLIDS HT.2HS	10.162		
		DRY WEIGHT,gm	87.86		

Load kPa	Corr. Height mm	Void Ratio	Average Height mm	t90 sec	t50 sec	cv. t90 cm2/s	k cm/S	mv m2/kN
0.00	18.830	0.853	18.830					
9.65	18.510	0.821	18.670	15		4.93E-02	8.50E-06	1.76E-03
19.30	18.301	0.801	18.406	5		1.44E-01	1.62E-05	1.15E-03
38.61	18.074	0.779	18.188	7		1.00E-01	6.13E-06	6.24E-04
77.21	17.856	0.757	17.965	5		1.37E-01	4.02E-06	3.00E-04
38.61	17.893	0.761	17.875					5.09E-05
9.65	17.992	0.770	17.943					1.82E-04
19.30	17.978	0.769	17.985	3		2.29E-01	1.73E-06	7.70E-05
38.61	17.919	0.763	17.949	17		4.02E-02	6.39E-07	1.62E-04
77.21	17.819	0.753	17.869	2		3.38E-01	4.56E-06	1.38E-04
154.43	17.573	0.729	17.696	3		2.21E-01	3.67E-06	1.69E-04
308.86	17.058	0.679	17.316	459		1.38E-03	2.40E-08	1.77E-04
617.71	16.336	0.608	16.697	385		1.54E-03	1.87E-08	1.24E-04
1235.42	15.597	0.535	15.967	254		2.13E-03	1.32E-08	6.35E-05
2470.85	14.877	0.464	15.237	16		3.08E-02	9.33E-08	3.10E-05
1235.42	15.132	0.489	15.005					
308.86	15.212	0.497	15.172					
77.21	15.447	0.520	15.330					
9.65	15.602	0.535	15.525					

Notes:
k calculated using Cv based on t90 values.

Water Content % ,initial	29.0	Liquid Limit %	31.2
Water Content % , final	20.6	Plastic Limit %	18.3
		Plastic Index %	12.9
Original Volume,cc	59.63	Liquidity Index	0.8
Volume of Solids,cc	32.18		
Volume of Voids,cc	27.45	Unit Weight,kN/m3	19.13
Degree of Saturation %	92.9	Dry Unit Weight,kN/m3	14.83

CONSOLIDATION TEST
VOID RATIO vs LOG. PRESSURE



CONSOLIDATION TEST
VOID RATIO VS. LOG PRESSURE

FIGURE A7
Page 2 of 3

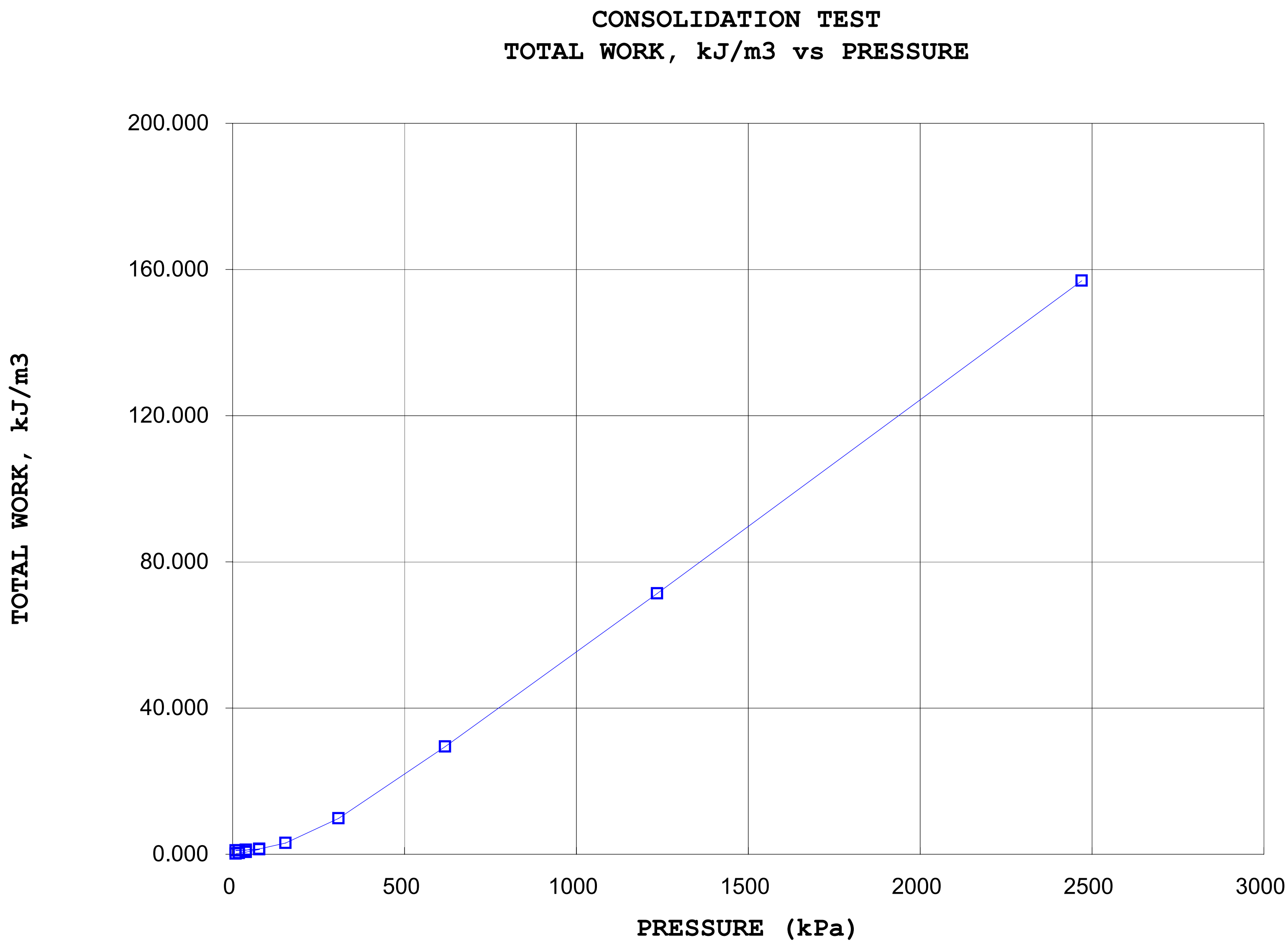


FIGURE A7
Page 3 of 3

CONSOLIDATED UNDRAINED TRIAXIAL WITH PORE PRESSURE MEASUREMENTS SHEET 1 OF 3			FIGURE A8
TEST STAGE	A	B	
BOREHOLE NUMBER	RESR-7	RESR-7	
SAMPLE NUMBER	5	5	
SPECIMEN DIAMETER, cm	4.96	4.95	
SPECIMEN HEIGHT, cm	10.09	10.06	
WATER CONTENT BEFORE CONSOLIDATION, %	36.5	37.8	
CELL PRESSURE, δ_3, kPa	255.0	355.0	
BACK PRESSURE, kPa	205.0	205.0	
PORE PRESSURE PARAMETER "B"	0.99	0.99	
CONSOLIDATION PRESSURE, δc, kPa	50.0	150.0	
VOLUMETRIC STRAIN DURING CONSOLIDATION, %	2.9	6.0	
WATER CONTENT AFTER CONSOLIDATION, %	34.4	33.5	
AVERAGE RATE OF STRAIN, %/hr	0.5	0.5	
TIME TO FAILURE, DAYS	2	2	
WATER CONTENT AFTER TEST, %	33.9	32.9	
MAX. DEVIATOR STRESS, $(\delta_1-\delta_3)$, kPa	72.1	105.8	
AXIAL STRAIN AT $(\delta_1-\delta_3)$ MAXIMUM, %	2.7	5.2	
MAX EFFECTIVE PRINCIPAL STRESS RATIO, (δ_1/δ_3) MAXIMUM	3.5	2.6	
DEVIATOR STRESS AT (δ_1/δ_3) MAXIMUM, kPa	72.1	94.5	
AXIAL STRAIN AT (δ_1/δ_3) MAXIMUM, %	2.7	8.1	
PORE PRESSURE PARAMETER, Af, AT $(\delta_1-\delta_3)$ MAXIMUM	0.29	0.69	
PORE PRESSURE PARAMETER, Af, AT (δ_1/δ_3) MAXIMUM	0.29	0.96	
NATURAL WATER CONTENT, w, %	33.9	33.5	
DRY DENSITY, mg/m^3	1.39	1.37	
FILTER DRAINS USED, y/n	y	y	
TEST NOTES:			
CHANGED RATE OF STRAIN, %/hr			
AXIAL STRAIN WHERE RATE OF STRAIN WAS CHANGED, %			
FAILURE PLANE NUMBER	2	1	
ANGLE OF FAILURE, DEGREES	50	65	

DATE: September, 1998

PROJECT NUMBER: 981-8033

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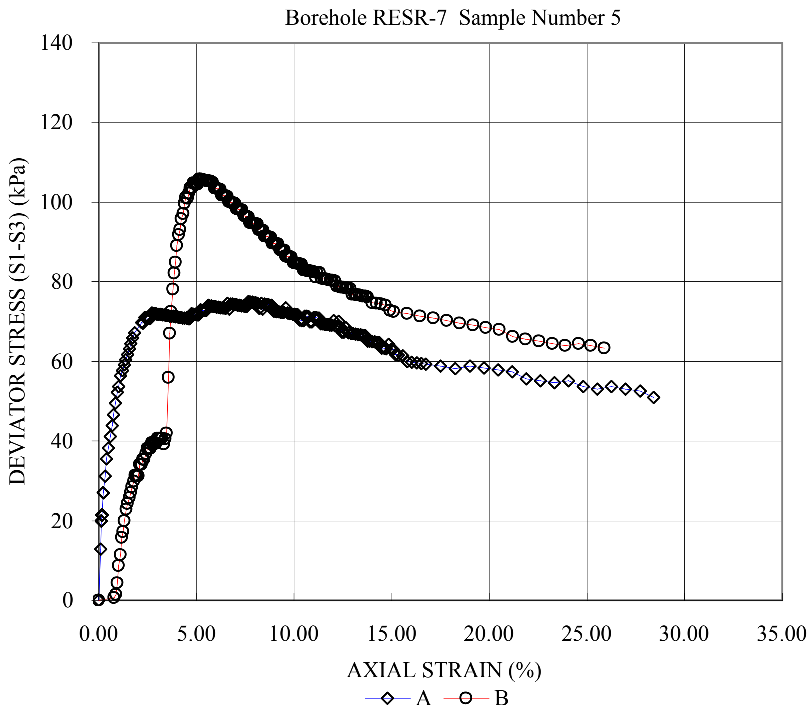
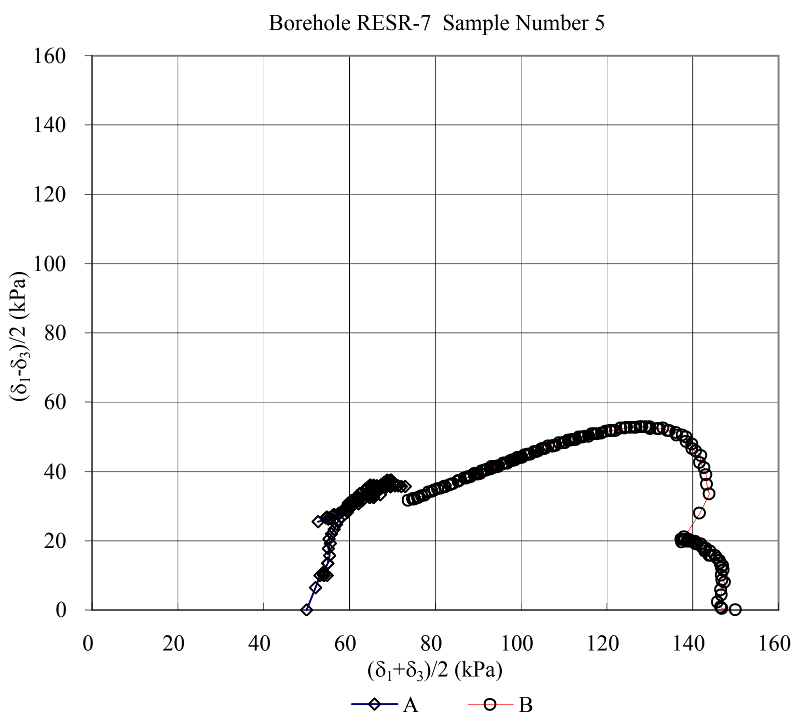
CONSOLIDATED UNDRAINED TRIAXIAL WITH PORE PRESSURE MEASUREMENTS SHEET 2 OF 3	FIGURE A8





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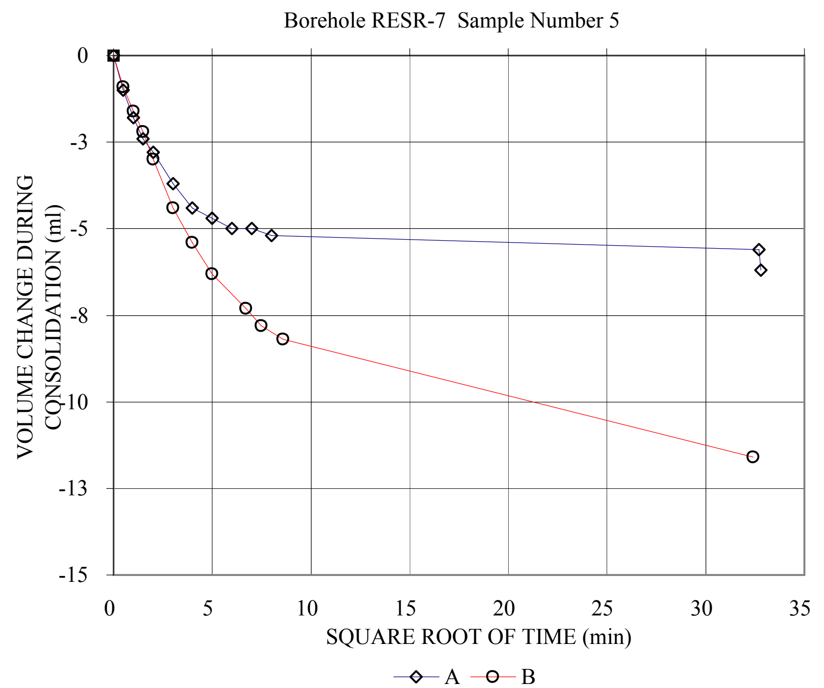
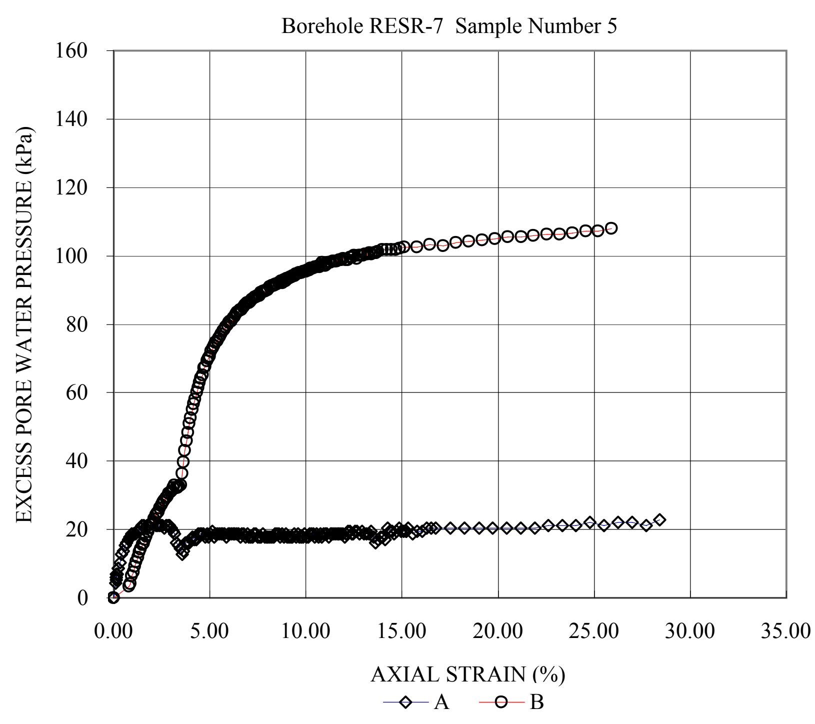
CONSOLIDATED UNDRAINED TRIAXIAL WITH PORE PRESSURE MEASUREMENTS SHEET 3 OF 3	FIGURE A8





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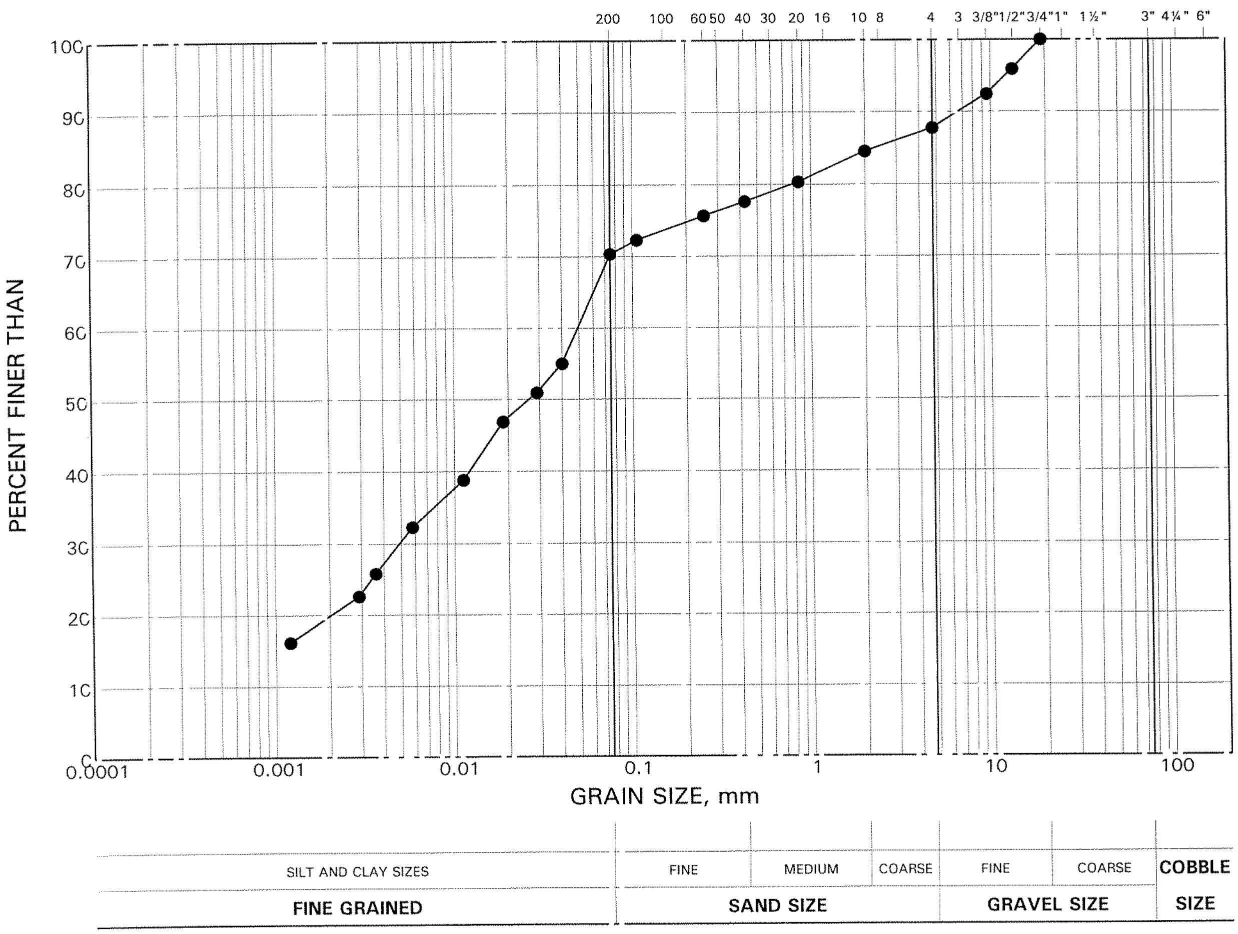
GRAIN SIZE DISTRIBUTION
Hard Clayey Silt (Till)

FIGURE A9



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
●	BR10-2	14	63.6