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REPORT ON

**DETAIL DESIGN
BRIDGE 9; RHCE S-E RAMP
FOUNDATION INVESTIGATION AND DESIGN
RED HILL CREEK EXPRESSWAY INTERCHANGE
G.W.P. 441-97-00
MINISTRY OF TRANSPORTATION, ONTARIO
HAMILTON, ONTARIO**

Submitted to:

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TABLE OF CONTENTS

<u>SECTION</u>	<u>PAGE</u>
PART A - FOUNDATION INVESTIGATION REPORT	
1.0 INTRODUCTION	1
2.0 SITE DESCRIPTION	2
3.0 INVESTIGATION PROCEDURES	3
3.1 Foundation Investigation	3
4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS	5
4.1 Regional Geology	5
4.2 Subsoil Conditions	5
4.2.1 Topsoil and Fill	6
4.2.2 Clayey Silt	6
4.2.3 Clayey Silt Till	7
4.2.4 Sandy Silt Till	9
4.2.5 Clayey Silt (Residual Soil)	10
4.2.6 Bedrock	10
4.2.7 Groundwater Conditions	11
PART B - FOUNDATION DESIGN REPORT	
5.0 ENGINEERING RECOMMENDATIONS	13
5.1 General	13
5.2 Shallow Foundations	13
5.2.1 Geotechnical Resistance	14
5.2.2 Resistance to Lateral Loads	14
5.2.3 Frost Protection	14
5.3 Steel H-Pile Foundations	14
5.3.1 Axial Geotechnical Resistance	15
5.3.2 Downdrag Load (Negative Skin Friction)	15
5.3.3 Resistance to Lateral Loads	16
5.3.4 Frost Protection	17
5.4 Caissons	17
5.4.1 Axial Geotechnical Resistance	17
5.4.2 Downdrag Load (Negative Skin Friction)	18
5.4.3 Resistance to Lateral Loads	18
5.4.4 Frost Protection	18
5.5 Lateral Earth Pressures for Design	18
5.6 Approach Embankment Design and Construction	20

5.6.1	Subgrade Preparation and Embankment Construction	20
5.6.2	Approach Embankment Stability	20
5.6.2.1	East Abutment Area.....	21
5.6.2.2	West Abutment Area	22
5.6.3	Approach Embankment and Settlement.....	22
5.6.4	Mitigation of Stability Issues / Time Dependant Settlement.....	23
5.6.4.1	Sub-excavation	23
5.6.4.2	Pre-loading.....	23
5.6.4.3	Surcharging.....	24
5.6.4.4	Wick Drains	24
5.7	Pre-load Embankment and Wick Drain Design.....	24
5.7.1	Summary of Design Parameters	25
5.7.1.1	Strength Parameters.....	25
5.7.1.2	Deformation Parameters.....	26
5.7.1.3	Rate of Consolidation	28
5.7.1.4	Pore Pressure Parameters	29
5.7.2	Performance Requirements	30
5.7.3	Method of Analysis.....	30
5.7.4	Results of Analysis.....	33
5.7.4.1	Settlement	33
5.7.4.2	Excess Pore Pressures	34
5.7.4.3	Stability	35
5.7.5	Design Recommendations for Embankments on Wick Drain Foundation	35
5.7.6	Specifications	37
5.7.7	Monitoring	37
5.8	Retaining Wall Foundations	38
5.8.1	Settlement and Stability	38
5.8.2	Geotechnical Resistance.....	38
5.9	Excavations and Temporary Cut Slopes	39

In Order
Following
Page 40

Tables 1 and 2

Figures 1 to 23

Drawing 1

Lists of Abbreviations and Symbols

Lithological and Geotechnical Rock Description Terminology

Record of Borehole Sheets (BR9-1 to BR9-9B, 9, RSER-6, RSER-7)

Cone Penetration Test Sheets CPT 04-01 and CPT 04-04

Appendix A

LIST OF TABLES

Table 1	Evaluation of Foundation Alternatives
Table 2	Evaluation of Alternatives for Mitigation of Stability and Settlement

LIST OF FIGURES

Figure 1	Water Content and Atterberg Limit
Figure 2	Shear Strength and Design Line
Figure 3	Preconsolidation Pressure and Design Line
Figure 4	CPT Pore Pressure Dissipation Tests
Figure 5	Approach Embankment Stability Analysis – East Embankment (Right to Left)
Figure 6	Approach Embankment Stability Analysis – West Embankment (Left to Right)
Figure 7	Approach Embankment Stability Analysis – East Embankment (Left to Right)
Figure 8	Approach Embankment Stability Analysis – West Embankment (Right to Left)
Figure 9	Approach Embankment Stability Analysis – West Pre-load Embankment (Right to Left)
Figure 10	Void Ratio and Design Line
Figure 11	Recompression Index and Design Line
Figure 12	Compression Index and Design Line
Figure 13	Time-Rate Settlement for Different Wick Drain Spacing – East Embankment
Figure 14	Time-Rate Settlement for Different Wick Drain Spacing – West Embankment
Figure 15	Percent Consolidation in 90 Days for Different Wick Drain Spacing
Figure 16	Time-Rate Settlement for 1.0 m Wick Drain Spacing
Figure 17	Excess Pore Pressure Response – East Embankment
Figure 18	Excess Pore Pressure Response – West Embankment
Figure 19	Approach Embankment Stability Analysis – Factor of Safety During Construction Period
Figure 20	Approach Embankment Stability Analysis – Factor of Safety with Time
Figure 21	Approach Embankment Stability Analysis – East Pre-load Embankment
Figure 22	Approach Embankment Stability Analysis – West Pre-load Embankment
Figure 23	‘N’ Values and Wick Drain Installation Limit

LIST OF DRAWINGS

Drawing 1	Borehole Locations and Soil Strata
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LIST OF APPENDICES

Appendix A	Laboratory Test Data
Table A1	Summary of Water Content Determination
Table A2	Specific Gravity Test Results
Table A3	Summary of Laboratory Vane Testing
Figure A1	Grain Size Distribution – Clayey Silt (Till)
Figure A2	Plasticity Chart – Clayey Silt (Till)
Figure A3	Consolidation Test – Void ratio vs Log Pressure (BR9-1 SA 5B)
Figure A4	Consolidation Test – Total Work vs Pressure (BR9-1 SA 5B)
Figure A5	Consolidation Test – Void ratio vs Log Pressure (BR9-3 SA 5B)
Figure A6	Consolidation Test – Total Work vs Pressure (BR9-3 SA 5B)

Figure A7	Consolidation Test – Void ratio vs Log Pressure (BR9-4 SA 7) (Horizontal Trim – HTO)
Figure A8	Consolidation Test – Total Work vs Pressure (BR9-4 SA 7) (Horizontal Trim – HTO)
Figure A9	Consolidation Test – Void ratio vs Log Pressure (BR9-4 SA 7) (Vertical Trim – VTO)
Figure A10	Consolidation Test – Total Work vs Pressure (BR9-4 SA 7) (Vertical Trim – VTO)
Figure A11	Consolidation Test – Void ratio vs Log Pressure (BR9-7 SA 8) (Horizontal Trim – HTO)
Figure A12	Consolidation Test – Total Work vs Pressure (BR9-7 SA 8) (Horizontal Trim – HTO)
Figure A13	Consolidation Test – Void ratio vs Log Pressure (BR9-7 SA 8) (Vertical Trim – VTO)
Figure A14	Consolidation Test – Total Work vs Pressure (BR9-7 SA 8) (Vertical Trim – VTO)
Figure A15	Consolidation Test – Void ratio vs Log Pressure (RSER-6 SA 6)
Figure A16	Consolidation Test – Total Work vs Pressure (RSER-6 SA 6)
Figure A17	Consolidated Undrained Triaxial Test (BR9-1 SA 5B)
Figure A18	Consolidated Undrained Triaxial Test (BR9-7 SA 6)
Figure A19	Grain Size Distribution – Sandy Silt and Gravel (Till)
Figure A20	Plasticity Chart – Sandy Silt and Gravel (Till)

PART A

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TABLE OF CONTENTS

<u>SECTION</u>	<u>PAGE</u>
PART A - FOUNDATION INVESTIGATION REPORT	
1.0 INTRODUCTION	1
2.0 SITE DESCRIPTION	2
3.0 INVESTIGATION PROCEDURES	3
3.1 Foundation Investigation	3
4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS	5
4.1 Regional Geology	5
4.2 Subsoil Conditions	5
4.2.1 Topsoil and Fill	6
4.2.2 Clayey Silt	6
4.2.3 Clayey Silt Till	7
4.2.4 Sandy Silt Till	9
4.2.5 Clayey Silt (Residual Soil)	10
4.2.6 Bedrock	10
4.2.7 Groundwater Conditions	11

In Order
Following
Page 12

Drawing 1
Lists of Abbreviations and Symbols
Lithological and Geotechnical Rock Description Terminology
Record of Borehole Sheets (BR9-1 to BR9-9B, 9, RSER-6, RSER-7)
Cone Penetration Test Sheets CPT 04-01 and CPT 04-04
Appendix A

LIST OF DRAWINGS

Drawing 1 Borehole Locations and Soil Strata

LIST OF APPENDICES

Appendix A	Laboratory Test Data
Table A1	Summary of Water Content Determination
Table A2	Specific Gravity Test Results
Table A3	Summary of Laboratory Vane Testing
Figure A1	Grain Size Distribution – Clayey Silt (Till)
Figure A2	Plasticity Chart – Clayey Silt (Till)

Figure A3	Consolidation Test – Void ratio vs Log Pressure (BR9-1 SA 5B)
Figure A4	Consolidation Test – Total Work vs Pressure (BR9-1 SA 5B)
Figure A5	Consolidation Test – Void ratio vs Log Pressure (BR9-3 SA 5B)
Figure A6	Consolidation Test – Total Work vs Pressure (BR9-3 SA 5B)
Figure A7	Consolidation Test – Void ratio vs Log Pressure (BR9-4 SA 7) (Horizontal Trim – HTO)
Figure A8	Consolidation Test – Total Work vs Pressure (BR9-4 SA 7) (Horizontal Trim – HTO)
Figure A9	Consolidation Test – Void ratio vs Log Pressure (BR9-4 SA 7) (Vertical Trim – VTO)
Figure A10	Consolidation Test – Total Work vs Pressure (BR9-4 SA 7) (Vertical Trim – VTO)
Figure A11	Consolidation Test – Void ratio vs Log Pressure (BR9-7 SA 8) (Horizontal Trim – HTO)
Figure A12	Consolidation Test – Total Work vs Pressure (BR9-7 SA 8) (Horizontal Trim – HTO)
Figure A13	Consolidation Test – Void ratio vs Log Pressure (BR9-7 SA 8) (Vertical Trim – VTO)
Figure A14	Consolidation Test – Total Work vs Pressure (BR9-7 SA 8) (Vertical Trim – VTO)
Figure A15	Consolidation Test – Void ratio vs Log Pressure (RSER-6 SA 6)
Figure A16	Consolidation Test – Total Work vs Pressure (RSER-6 SA 6)
Figure A17	Consolidated Undrained Triaxial Test (BR9-1 SA 5B)
Figure A18	Consolidated Undrained Triaxial Test (BR9-7 SA 6)
Figure A19	Grain Size Distribution – Sandy Silt and Gravel (Till)
Figure A20	Plasticity Chart – Sandy Silt and Gravel (Till)

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 9 along the S-E Ramp as part of the interchange project. The work was carried out in accordance with the Quality Control Plan for this project dated 2003. A digital file of the E-plan (Bridge Site Plan) showing the proposed Bridge 9 configuration was provided to Golder by MRC in October 2003.

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

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

2.0 SITE DESCRIPTION

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

The terrain in this area is generally flat-lying. The ground surface at the bridge site is at about Elevation 80 m to 81 m along the alignment of the proposed works. Minor undulations across the site mainly involve fill embankments at the existing interchange, the right-of-way of the QEW, as well as regrading and landscaping on adjacent lands.

3.0 INVESTIGATION PROCEDURES

3.1 Foundation Investigation

Subsurface investigation for the Bridge 9 structure were carried out in two phases. The first phases was carried out in 2003 between August 7 and August 13 and the second phase was carried out in 2004 between May 27 and June 2. During the investigations, three (3) boreholes (BR9-1 to BR9-3) were advanced in 2003 and seven (7) boreholes (BR9-4 to BR9-9B) and four (4) cone penetration tests (CPT04-01 to CPT04-04) were advanced in 2004. Boreholes 9, RSER-6 and RSER-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 2003/2004 subsurface 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 from the 2003 investigation were advanced using 108 mm inside diameter (I.D.) continuous flight hollow stem augers while boreholes from the 2004 investigation were advance using 102 mm outside diameter (O.D.) continuous flight solid 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. An automatic hammer was used for purposes of obtaining SPT 'N' values. In situ vane shear strength testing and Shelby tube samples were obtained at regular intervals, where appropriate, in clayey strata. Samples of the bedrock were obtained using an 'NQ' size rock core barrel.

The boreholes were advanced to depths ranging from 7.3 m to 31.1 m below the existing ground surface (including rock coring). All of the boreholes from the 2003 investigation were advanced to refusal within the till deposits or on bedrock and two of the boreholes were extended into the bedrock by coring. The boreholes from the 2004 investigation were terminated in the very stiff till deposit. The groundwater conditions in the open boreholes were observed during the drilling operations and one piezometer was installed in Borehole BR9-2 in the 2003/2004 investigation to permit monitoring of the groundwater level. The piezometer consists 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 borehole. The holes were backfilled with bentonite mixed with soil cuttings; typically one bag of bentonite was used per 3m of hole backfilled. The installation details and water level readings are described on the Record of Borehole sheets following the text of this report.

Additional boreholes were advanced through the fill and clayey silt/clayey silt till in order to facilitate the start of the CPTs. The CPT is a state-of-the-art, in situ technique for site characterisation studies. The CPT consists of a special cone tip equipped with electronic sensing elements to continuously measure tip resistance, local side friction on a sleeve and porewater

pressure. It is pushed at a constant rate into the ground using a drill rig (ASTM D5778-95). A continuous stratigraphic profile together with engineering properties, such as strength, stress history and density, can be interpreted from the results of the CPT.

The CPT equipment was advanced using the hydraulic ram system on the track-mounted drill rig. The four CPTs were advanced to refusal encountered at depths ranging from about 5.1 m to 8.0 m. Record of Cone Penetration Test sheets are included with the Record of Borehole sheets following the text of this report. Profiles of tip resistance, porewater pressure during pushing and friction are presented together with an interpreted profile of undrained shear strength (s_u) and classification index (I_c) that is used to infer soil type (stratigraphy).

In addition to the four CPTs that was advanced to refusal, six pore water pressure dissipation tests were carried out within the 'softened' clayey silt till using the CPT unit.

The field work was supervised throughout by members of our engineering and technical staff, who located the boreholes and CPTs, arranged for the clearance of underground service locations, supervised the drilling, sampling and in situ testing operations, logged the boreholes and CPTs, and examined and cared for the soil and rock samples. The samples were identified in the field, placed in appropriate containers, labelled and transported to our Mississauga geotechnical laboratory where the samples underwent further detailed visual examination and laboratory testing. All of the laboratory tests were carried out to MTO and/or ASTM Standards as appropriate. Classification testing (water content, Atterberg Limits and grain size distribution) as well as specialised oedometer (consolidation) and triaxial testing was carried out on selected samples. Point load testing were 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) coordinate 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 in keeping with the currently 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", 3^d 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 surficial layers of silty sand fill and clayey silt which is underlain by a thick deposit of grey clayey silt till. The upper portion of the clayey silt till deposit is typically firm to very stiff while the lower portion of the deposit is stiff to hard. Reddish brown clayey silt till containing shale fragments was encountered below the upper clayey silt till deposit at two borehole locations. The clayey silt tills are underlain by a thick sandy silt till deposit containing cobbles and boulders in turn underlain by shale bedrock of the Queenston Formation. The total overburden thickness is up to 25.9 m as encountered in the two deepest boreholes at the site where bedrock was proven by coring. A more detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

4.2.1 Topsoil and Fill

Topsoil was encountered at the existing ground surface in Boreholes BR9-1 to BR9-9B, RSER-6 and RSER-7. The surface of the topsoil ranged between Elevation 79.8 m to 81.2 m and ranged from 0.1 m to 0.15 m thick.

Fill materials were encountered, either immediately below the topsoil or at ground surface in all boreholes except BR9-8 to BR9-9B and RSER-6. In Boreholes BR9-1 to BR9-7 and RSER-7, the fill consists of sandy silt and extends to between 0.6 m and 1.5 m depth below ground surface. The fill in Borehole BR9-2 contains traces of asphalt pieces. In Borehole 9, the fill ranges in composition between silty sand and clayey silt and was approximately 0.6 m thick. The fill in Borehole BR9-3 extends to 1.5 m depth and consists of clayey silt. Trace organics and/or rootlets were noted in the fill in Boreholes BR9-1, BR9-3 to BR9-7 and 9.

In Borehole RSER-6, a brown to reddish-brown clayey silt deposit containing some sand, trace gravel, organics and oxidised stains was encountered below the topsoil. The surface of the clayey silt was encountered at Elevation 81.1 m and the layer was 2.0 m thick.

Standard Penetration Testing (SPT) measured 'N' values within the non-cohesive fill ranged between 4 and 33 blows per 0.3 m of penetration, indicating a loose to dense state of packing; typically loose to compact. SPT measured 'N' values with the cohesive fill ranged between 11 and 32 blows per 0.3 m of penetration, indicating a stiff to hard consistency; typically stiff.

The natural water contents measured on six samples of the fill range from 10 to 18 percent.

4.2.2 Clayey Silt

A deposit of grey and brown clayey silt containing some sand and trace to some gravel was encountered below the topsoil and/or fill at Boreholes BR9-1, BR9-2, 9, RSER-6 and RSER-7. The clayey silt ranged from 0.6 m to 1.8 m thick, and the surface was encountered between Elevations 79.0 m and 79.9 m.

At the borehole locations, Standard Penetration Testing (SPT) measured 'N' values ranged between 13 and 27 blows per 0.3 m of penetration, indicating a stiff to very stiff consistency.

The natural water contents measured on seven samples of the clayey silt were between 18 and 23 percent.

4.2.3 Clayey Silt Till

A deposit of clayey silt till was encountered underlying the fill and/or clayey silt deposit in all Boreholes. The clayey silt till contains trace to some sand and gravel. Occasional silty clay seams were noted in Borehole RSER-6. The clayey silt till was typically grey in colour 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 Elevation 77.5 m and 79.1 m in the boreholes, and the thickness varied from 11.6 m to 15.3 m. This deposit was not fully penetrated in Boreholes BR9-4 to BR9-9B, RSER-6 and RSER-7.

In the 2004 investigation (BR9-4 and BR9-9B), the upper 2.1 m to 3.2 m of clayey silt till deposit encountered between about Elevation 77.3 m to Elevation 78.1 m is composed of a weathered brown to grey-brown crust containing trace to some sand and gravel. Standard Penetration Testing (SPT) in the weathered crust measured 'N' values ranged between 5 and 8 blows per 0.3 m of penetration. In situ field vane testing was carried out using a standard MTO 'N' vane measured shear strength of 95 kPa or greater indicating a stiff to very stiff consistency. This is similar to the clayey silt deposit found in the 2003 investigation.

The upper 2.4 m to 4.6 m of the clayey silt till deposit from the 2003 investigation and approximately 1.7 m to 3.2 m (between about Elevation 79.4 m to Elevation 76.9 m) below the weathered clayey silt till crust from the 2004 investigation is a 'softened' portion of the clayey silt till deposit. The SPT measured 'N' values within the upper portion of the clayey silt ranged between 3 and 11 blows per 0.3 m of penetration.

Grain size distribution on four samples from this deposit are shown on Figure A1 in Appendix A.

The natural water content measured on forty-eight samples of this deposit ranged between 9 and 31 percent, with an average of 24 percent. Atterberg limits testing was carried out on eighteen samples of the clayey silt till deposit. The results of the Atterberg limits tests are plotted on the plasticity chart on Figure A2 Appendix A. It should be noted that the test result from Borehole BR9-4 Sample 5 and RSER-6 Sample 5 indicates that the material is considered to be a silty clay till of intermediate plasticity; however, in general, the deposit is classified as a clayey silt of low plasticity. Water contents and Atterberg limits are summarised on Figure 1.

The results of in situ vane tests carried out using a standard MTO 'N' vane are shown on the Record of Borehole sheets. The results of the vane tests and undrained shear strength profiles from the CPTs is summarised on Figure 2. Based on the in situ vane tests, the undrained shear strength of the 'softened' portion of the clayey silt till deposit measured shear strength between 42 kPa and 77 kPa, indicating that this portion of the deposit has a firm to stiff consistency. These results are generally consistent with the results from the CPTs that estimate the undrained

shear strength to vary from 30 kPa to 60 kPa in the ‘softened’ portion of the till deposit. The undrained shear strength profiles from the CPTs are shown on the Cone Penetration Test sheets

Laboratory oedometer (consolidation) tests were performed on five samples of the ‘softened’ portion of the clayey silt till obtained from Boreholes BR9-1, BR9-3, BR9-4, BR9-7 and RSER-6 are provided on Figures A3 to A16 in Appendix A. The results of these oedometer tests and the preconsolidation pressure profiles from the CPTs are summarised in Figure 3. The following table summarises the oedometer test results.

Borehole and Sample No.	Elevation (m)	$S_{vo} \zeta$ (kPa)	$S_p \zeta$ (kPa)	OCR	e_o	C_r	C_c	c_v^* (cm²/s)	c_h^* (cm²/s)
BR9-1 SA 5B	76.0 – 75.4	93	200	2.2	0.52	0.044	0.184	2.4×10^{-3}	-
BR9-3 SA 5B	77.3 – 76.6	70	220	3.2	0.78	0.058	0.275	2.6×10^{-3}	-
BR9-4 SA 7	76.3 – 75.7	55	120	2.2	0.73	0.041	0.200	8.3×10^{-3}	5.9×10^{-3}
BR9-7 SA 8	75.4 – 74.8	55	165	3.0	0.72	0.051	0.248	2.2×10^{-3}	2.4×10^{-3}
RSER-6 SA 6	76.6 – 76.0	80	155	1.9	0.64	0.028	0.207	1.7×10^{-3}	-

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

where: $S_{vo} \zeta$ is the effective overburden pressure in kPa
 $S_p \zeta$ 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 vertical coefficient of consolidation in cm²/s
 c_h is the horizontal coefficient of consolidation in cm²/s

Laboratory consolidated undrained (CIU) triaxial compression test were carried out on carefully trimmed specimen of the clayey silt till obtained from Borehole BR9-1 Sample 5B and BR9-7 Sample 6. The test results indicate an effective angle of shearing resistance of 30° (degrees) and effective shear resistance of 0 kPa. Details of the test results are shown on Figure A17 and A18 in Appendix A.

In situ porewater pressure dissipation tests were also carried out in the ‘softened’ clayey silt till using the CPT unit. The results of the dissipation tests are provided on Figure 4. The measured horizontal coefficient of consolidation, c_h , values ranges from 2.1×10^{-2} cm²/s to 1.6×10^{-3} cm²/s with an average value of 2.4×10^{-3} cm²/s. These results are generally consistent with the results from the oedometer tests carried out on Borehole BR9-4 Sample 4 and BR-7 Sample 8 where horizontal coefficient of consolidation were assessed.

The lower portion of clayey silt till deposit had measured SPT ‘N’ values ranging from 13 to 137 blows per 0.3 m of penetration, suggesting a stiff to hard consistency. Typically the ‘N’ values increased with depth. The top of this deposit varies from Elevation 72.7 m to Elevation

75.7 m. In Boreholes BR9-1, BR9-2, BR9-3 and 9 the stratum was penetrated and ranged between 7.0 m and 11.8 m in thickness. Shale and limestone pieces were encountered throughout the deposit, becoming more frequently with depth.

At the base of the till deposit, a thin layer of reddish-brown clayey silt containing some sand and gravel was encountered in Boreholes BR9-1, BR9-2 and 9. Occasional weathered shale and limestone fragments were observed in the deposit. The top of this layer was encountered between Elevation 65.4 m and 67.5 m. This deposit was between 2.9 m and 3.7 m in thickness in Boreholes BR9-1 and BR9-2.

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.

The natural water content measured on selected samples of this hard clayey silt till layer ranged from 6 to 7 percent.

4.2.4 Sandy Silt Till

A deposit of red sandy silt to sandy silt and gravel till, was encountered below the reddish-brown clayey silt till in Boreholes BR9-1 and BR9-2 and beneath the grey to reddish-grey clayey silt till in BR9-3. The deposit contains varying amounts of clay and gravel, cobbles and boulders. This deposit is considered to be the 'Wentworth' till sheet. The surface of the deposit was encountered between Elevation 62.4 m and 64.6 m. The deposit was penetrated in BR9-2 and BR9-3 and the thickness of the deposit was about 3.3 m and 9.1 m, respectively. Borehole BR9-1 was terminated in this deposit proving a thickness of 4.1 m.

Borehole BR9-3 was advanced through this till deposit by augering and by rock coring where refusal to augering and spoon advance was encountered. 'NQ' coring was carried out between 17.8 m and 21.3 m depths and between 22.6 m and 25.9 m depths within the deposit. Limestone and siltstone boulders were recovered from the rock core between 17.8 m and 21.3 m depth and granitic and dioritic boulders were recovered from the rock core between 22.6 m and 25.9 m depth at the base of the deposit. Grain size distribution on one sample of this deposit (for the portion less than 20 mm) is shown on Figure A19 in Appendix A.

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

The natural water content measured on five samples of this deposit were between 7 and 14 percent.

Atterberg limits testing was carried out on one sample of the sandy silt till deposit. The liquid limit was 20 percent and the plastic limit was 14.5 percent giving a plasticity index of 5.5 percent. The result of this testing indicate that the sandy silt till is a silt of low plasticity. The result of the Atterberg limits test are also plotted on the plasticity chart on Figure A20 Appendix A.

4.2.5 Clayey Silt (Residual Soil)

A thin layer of red clayey silt was encountered beneath the sandy silt till layer in Borehole BR9-2. The deposit contains some sand and gravel with shale pieces and is described as a residual soil derived from the weathering of the underlying shale bedrock. The deposit was encountered at Elevation 59.1 m and is 2.5 m thick.

A SPT 'N' values within the till was greater than 100 blows per 0.3 m of penetration indicating a hard consistency.

4.2.6 Bedrock

Bedrock was encountered in Borehole BR9-2 and was possibly encountered in Borehole BR9-3. The poor recovery for the core samples obtained in Borehole BR9-3 make it difficult to determine whether the deposit is an extension of the sandy silt (Wentworth) till containing predominantly shale boulders or whether it is in fact highly weathered shale bedrock. The elevation and depth of the bedrock surface or inferred bedrock is given in the table below:

<i>Borehole</i>	<i>Ground Surface Elevation (m)</i>	<i>Bedrock Depth (m)</i>	<i>Bedrock Surface Elevation (m)</i>
BR9-2	80.7	24.1	56.6
BR9-3	80.3	25.9	54.4

The bedrock samples obtained consist of reddish-grey, moderately to highly weathered, thinly layered, very fine grained, very weak to medium strong calcareous shale of the Queenston formation. Seams and layers of limestone and siltstone were present within the shale; in Borehole BR9-2, the limestone seams/layers comprised about 13 percent of the core recovered. The Total Core Recovery was between 13 and 100 percent. The Rock Quality Designation (RQD) measured on the core samples in Borehole BR9-2 ranged from about 45 to 88 percent but was typically 75 to 88 percent, indicating a rock mass of poor to good quality, typically good. The Total Core Recovery in Borehole BR9-3 ranged from about 15 to 50 percent (as compared to 95 to 100 percent in Borehole BR9-2). The RQD was typically 0percent indicating that, if it is actually rock, it is of very poor quality.

Point load strength tests were performed on selected samples of the rock core from Boreholes BR9-2 and BR9-3. Diametral point load strength index values are shown on the Record of Drillhole sheets. Approximate point load index values range from 0 MPa to 1.57 MPa in the shale and from 1.77 MPa to 4.71 MPa in the limestone. In the shale, this corresponds to an estimated unconfined compressive strength (UCS) ranging from 0MPa to 36 MPa. Using the Intact Rock Strength Classification table, these values indicate that the shale is very weak to medium strong. In the limestone, this corresponds to an estimated unconfined compressive strength (UCS) ranging from 41 MPa to 108 MPa indicating a medium strong to very strong rock type.

4.2.7 Groundwater Conditions

The water levels were noted during and after the drilling and coring operations in the boreholes. Piezometers were installed in Boreholes RSER-6, RSER-7, 9 and BR9-2. The piezometers in Boreholes RSER-6, RSER-7 and 9 were sealed within the clayey silt till deposit while the piezometer in the BR9-2 was sealed into the bedrock. Details of the piezometer installations are shown in the Record of Boreholes following the text of this report. Water levels in the piezometers are summarised in the table below:

<i>Borehole</i>	<i>Ground Surface Elevation (m)</i>	<i>Ground Water Level Depth (m)</i>	<i>Ground Water Level Elevation (m)</i>	<i>Date</i>
RESR-6	81.2	0.5	80.7	June 1, 2004
BR9-2	80.7	5.7	75.0	October 22, 2003
9	79.6	5.5	74.1	October 22, 2003
RSER-7	80.7	3.3	77.4	October 22, 2003

The groundwater level at the borehole locations typically appears to be coincident with the base of the softened upper portion of the clayey silt till. The groundwater table is likely controlled by the water level in Lake Ontario and is expected to slope slightly downwards toward the lake.

It should be noted that groundwater levels are expected to fluctuate seasonally and are expected to rise during wet periods of the year.



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**PART B
DETAIL DESIGN
BRIDGE 9; RHCE S-E RAMP
FOUNDATION INVESTIGATION AND DESIGN
RED HILL CREEK EXPRESSWAY INTERCHANGE
G.W.P. 441-97-00
MINISTRY OF TRANSPORTATION, ONTARIO
HAMILTON, ONTARIO**

5.0 ENGINEERING RECOMMENDATIONS

This section of the report provides foundation design recommendations for the proposed Bridge 9 as part of the QEW/Red Hill Creek Expressway Interchange. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during three phases of the subsurface investigation at this site. The interpretation and recommendations provided are intended only to provide the designers with sufficient information to assess the feasible foundation alternatives and to design the proposed structure foundations. As such, where comments are made on construction they are provided only in order to highlight those aspects which could affect the design of the project. Those requiring information on aspects of construction should make their own interpretation of the factual information provided as it may affect equipment selection, proposed construction methods, scheduling and the like.

It is understood that the proposed bridge will carry the Red Hill Creek Expressway south-to-east (S-E) ramp over the proposed Centennial Parkway west-to-north/south (W-N/S) ramp. The existing Centennial Parkway ramp will be re-aligned as part of this project. The proposed bridge is a 3-span bridge with variable span lengths up to 35 m long. Conventional wing walls will be used at the northeast and southwest corners of the bridge. At the northwest and southeast corners, retaining walls are required to extend beyond the limits of the wing walls. The proposed embankments will be up to about 8 m and 9 m in height at the east and west approaches, respectively.

5.1 General

Different alternatives for the abutment and pier foundations were considered and a summary is presented in Table 1, following the text of this report. Shallow foundations are not recommended for support of the bridge due to the anticipated settlements as a result of the presence of the softened till deposit. Caissons extended into the bedrock are not recommended given the extremely variable and bouldery till deposits and variable bedrock conditions. Steel H-piles driven to refusal just into the very dense sandy silt till for support of the piers and abutments is considered to be the most feasible option from a foundation perspective. The wing walls at the northeast and southwest corners of the bridge 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; alternatives for mitigation of this settlement are 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 pre-loaded. In this case, differential settlement of up to 25 mm could occur 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 assuming that the underling soil has been pre-loaded. 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, $\tan \delta'$, between the cast-in-place concrete footing and the compacted Granular 'A' 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 may be used for support of the abutments and concrete wing walls. It is not considered practical to drive the piles to the surface of the bedrock due to the density of and the presence of numerous cobbles and boulders inferred within the sandy silt (Wentworth) till deposit overlying the bedrock.

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

<i>Foundation Location</i>	<i>Relevant Boreholes</i>	<i>Design Pile Tip Elevation (m)</i>
West Abutment	BR9-1	65
West Pier	BR9-2	64
East Pier	9	64
East Abutment	BR9-3	63

5.3.1 Axial Geotechnical Resistance

For HP 310x110 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 67 m.

Pile installation should be in accordance with SP903S01. The pile termination or set criteria will be dependent on the pile driving hammer type, helmet, selected pile and length of pile. The criteria must therefore be established at the time of construction after the piling equipment is known. The 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 Formula :

- “Piles to be driven in accordance with Standard SS103-11 using an ultimate capacity of 3,500 kN per pile, but must be driven below El. 67 m for the west abutment and pier 1, and below El. 65 m for the east abutment and pier 2.”

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 and wing walls as a consequence of settlement of the ground with respect to the pile. The abutment pile structural design should be based on the full

downdrag load acting on the piles. The estimated unfactored downdrag load acting on the HP 310x110 piles may be taken as 200 kN per pile at the abutment location.

The load calculated in this manner is a 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 pre-loading of the abutment areas and subsequent use of lightweight fill in at least a portion of the embankments as discussed in Section 5.6.

5.3.3 Resistance to Lateral Loads

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

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

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

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

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

<i>Soil Unit</i>	<i>Elevation</i>	<i>s_u</i> (kPa)
Soft to firm clayey silt till	Between Elev. 79 m and 75 m	30
Stiff to hard clayey silt till	Below Elev. 75 m	100

Group action for lateral loading should also be considered when the pile spacing in the direction of the loading is less than six to eight pile diameters. Group action can be evaluated by reducing

the coefficient of horizontal subgrade reaction in the direction of loading by a reduction factor, R , as follows:

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

5.3.4 Frost Protection

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

5.4 Caissons

Consideration could be given to the use of caissons socketted into the hard clayey silt till or the very dense sandy silt till for support of the bridge. Due to the difficulties anticipated with extending the caissons to/into the bedrock as a result of the number of boulders within the sandy silt till and the highly variable condition of the shale bedrock, the use of caissons socketted into the bedrock is not recommended. If consideration is being given to the use of caissons, the following design base elevations may be used at the various foundation elements:

<i>Foundation Location</i>	<i>Design Caisson Founding Elevation (m)</i>
West Abutment	64
West Pier	62 to 64
East Pier	62 to 64
East Abutment	64

5.4.1 Axial Geotechnical Resistance

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

<i>Caisson Diameter (m)</i>	<i>Factored Axial Resistance</i>	
	ULS	SLS
0.9	3,200 kN	2,000 kN
1.5	7,500 kN	2,600 kN

5.4.2 Downdrag Load (Negative Skin Friction)

The estimated unfactored downdrag load acting on the caissons at the abutments may be taken as 450 kN and 750 kN for 0.9 m and 1.5 m diameter caissons, respectively. Other requirements for structural downdrag loading for the caissons should be in accordance with Section 5.3.2.

5.4.3 Resistance to Lateral Loads

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

5.4.4 Frost Protection

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

5.5 Lateral Earth Pressures for Design

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

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

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

granular backfill requirements with respect to sub-drains and frost taper should be in accordance with OPSD 3501.00 and 3504.00.

- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the wall stem, in accordance with *CHBDC* Section 6.9.3 and Figure 6.9.3. Compaction equipment should be used in accordance with OPSS 501.06. Other surcharge loadings should be accounted for in the design, as required.
- The granular fill may be placed either in a zone with width equal to at least 1.2 m behind the back of the wall stem (Case I in Figure C6.9.1(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	Lightweight Slag Fill
Soil unit weight:	21 kN/m ³	14.5 kN/m ³
Coefficients of static lateral earth pressure:		
Active, K_a	0.33	0.36
At rest, K_o	0.50	0.53

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

	GRANULAR 'A'	GRANULAR 'B' TYPE II
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 ramp varies from about Elevation 87.5 m to 90.0 m at the east and west approaches, respectively. The existing ground surface at the bridge varies from about Elevation 80 m to 81 m resulting in embankments between 7.5 m and 9.5 m in height. On the north side of the approach embankments, a retaining wall is proposed beyond the wing walls. The design of the wall will be addressed in a separate report, however, implications to global stability and settlement of the approach embankments are discussed below.

5.6.1 Subgrade Preparation and Embankment Construction

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 percent 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 percent 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

The proposed grade of the S-E Ramp varies from 87.5 m to 90.1 m at the east and west approaches, respectively. The existing ground surface at the bridge site varies from about Elevation 80 to 81 m. The embankment is anticipated to be up to 7.8 m and 9.1 m high above the existing ground surface at the northeast and southwest abutments, respectively. Based on space restrictions at this site, long retaining walls are required at the northwest and southeast corners of the site. Due to grade profile of the proposed ramp and detours, these walls will be 6.6 m and 7.7 m high at the southeast and northwest abutments, respectively.

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 and as shown on Figures 1 to 3:

<i>Soil Deposit</i>	<i>Bulk Unit Weight</i>	<i>Effective Friction Angle</i>	<i>Undrained Shear Strength</i>
Earth Fill	21 kN/m ³	32°	–
Clayey Silt Till Crust	21 kN/m ³	–	75 kPa
Firm to Stiff Clayey Silt Till	20 kN/m ³	30°	30 kPa
Very Stiff to Hard Clayey Silt Till	21 kN/m ³	30°	100 kPa
Very Dense Sandy Silt Till	21 kN/m ³	35°	–

The stability analyses indicates that the maximum embankment height that can be constructed using earth fill and 2 horizontal to 1 vertical (2H:1V) side slopes is 8 m. Figures 5 and 6 show the results of the stability analysis for the east and west approach embankments, respectively. For embankments over 8 m in height, stabilizing toe berms are required; at the west abutment, a 5 m wide, 3 m high berm is required as shown on Figure 7. Discussion of the stability analysis for each abutment area is given below.

5.6.2.1 East Abutment Area

At the east abutment area, the south retaining wall is 22 m long extending back from the front face of the east abutment. The S-E Ramp grade at this location is at about Elevation 87.8 m and the W-N/S Ramp (on the south side) is at about Elevation 81.2 m giving a proposed retaining wall height of about 6.6 m on the south side. The original ground surface on the north side is at about Elevation 80.0 m and therefore the embankment height on the north side is about 7.8 m. It should be noted that the existing QEW is located less than about 20 m to the north of the abutment.

Based on the results of the stability analyses and as shown on Figure 7, a 6.6 m high RSS wall has a factor of safety of greater than 1.3 against deep seated failure, when the wall is constructed using Granular 'B' backfill.

5.6.2.2 West Abutment Area

The north retaining wall at the west abutment is about 30 m long extending back from the front face of the abutment. The S-E Ramp grade at this location is at about Elevation 90.1 m and the W-N/S Ramp (on the north side) is at about Elevation 82.4 m giving a proposed retaining wall height of about 7.7 m. The original ground surface on the south side is at about Elevation 81.0 m giving an embankment height of about 9.1 m. For the 9.1 m high embankment on the south side, stabilizing toe berms (5 m wide and 3 m high) are required for stability as shown on Figure 6.

The ultimate ramp as well as the ramp detour on the north side in effect act as a “toe berm” on this side of the embankment to maintain the overall height of the wall at 7.7 m. With this “toe-berm” in place, the results of the stability analyses (as shown on Figure 8) indicate that a 7.7 m high RSS wall has a factor of safety of 1.3 against deep seated failure when the wall is constructed using Granular ‘B’ backfill. The “toe berm” should extend to no less than 29 m from the crest of the toe berm to the centreline of the embankment. Where the location of the north side of the proposed detour ramps is such that this 29 m distance is not provided, the toe berm should be extended beyond the edge of the proposed detour ramp.

In order to eliminate the requirement for toe berms at the west abutment, consideration could be given to the use of lightweight slag fill as retaining wall backfill for embankment construction. Another alternative which could be considered to eliminate the requirement for toe berms is sub-excavation of the soft to firm clayey silt till.

5.6.3 Approach Embankment and Settlement

Settlement of the approach embankment subgrade can be expected mainly due to consolidation of the surficial clayey silt and the softened clayey silt till deposits encountered at the approach embankments. Settlement analyses were carried out using the commercially available computer program Unisettle 3.0.

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

Settlement of the embankments as a consequence of consolidation settlement of the clayey silt till deposit underlying the earth fill embankments is calculated to be between about 175 mm and 200 mm. If the embankment is constructed fully of lightweight slag fill, up to about 125 mm of settlement of the embankments is calculated. If lightweight fill is used as RSS wall backfill (earth fill elsewhere within the embankment), the settlement would be expected to be mainly

influenced by the earth fill and would be calculated to be about 175 mm. If the option of sub-excavation of the soft to firm clayey silt till from below the approach embankments is adopted, the settlement will be controlled by the underlying stiff to hard clayey silt till and is calculated to be less than 50 mm. About 90 percent of settlement is expected to occur within about 9 months after loading.

Consideration could be given to pre-loading the embankment prior to wall construction, including the use of wick drains to reduce the length of time required for this consolidation.

It should be noted that there will be ongoing creep settlement within the subsoils at both abutments. It is estimated that approximately 20 mm of secondary (creep) settlement will occur within 50 years after completion of the pre-load period, with about 10 mm of this creep settlement occurring within the first 5 years

5.6.4 Mitigation of Stability Issues / Time Dependant Settlement

Time dependent settlements of the new embankments are expected as a result of consolidation of the underlying soft to firm clayey silt till. In addition, the presence of this material creates stability problems for the new embankment/retaining walls. In these areas, consideration could be given to pre-loading of the embankment or sub-excavation of the soft to firm clayey silt till in order to achieve the target factor of safety of 1.3 for stability and to limit the post-construction settlements and subsequent maintenance on the new roadway pavement structure.

5.6.4.1 Sub-excavation

Consideration could be given to sub-excavation of the soft to firm clayey silt till to minimize settlement and increase stability. The soft to firm deposit would have to be removed to an Elevation of 74.0 m and 74.5 m at the east and west abutment locations, respectively. This would require excavations up to between 5.6 m 6.7 m deep. Requirements with respect to side slopes and groundwater control should be in accordance with Section 5.9.

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

5.6.4.2 Pre-loading

Consideration could be given to pre-loading the embankment to limit post-construction settlements. If the embankment is pre-loaded for a minimum of 12 months, it is estimated

that 90 percent of the consolidation settlement will occur. The maximum embankment height with sides slopes of 2H:1V that can be constructed at this site using earth fill and still maintain a factor of safety of 1.3 is 8 m. The maximum embankment height with sides slopes of 1H:1V that can be constructed is 7 m.

At the east abutment area, it is understood that there is sufficient room to construct the 7.8 m high embankment with 2H:1V side slopes during the pre-load stage. At the west abutment area, there is insufficient room on the north side to construct the 2H:1V pre-load embankment side slopes as a result of the detours/construction staging. For embankment side slopes formed at 1.5H:1V, and provided that the specified “toe berm” (as discussed in Section 5.6.2.2) is constructed and is in place prior to construction of the pre-load embankment, an adequate Factor of Safety is obtained for this configuration as shown on the attached Figure 9.

5.6.4.3 Surcharging

Consideration could be given to surcharging the pre-load embankment in order to reduce the magnitude of post-construction settlement. However, given the stability considerations noted above as well as the space constraints, it may not be possible to place a surcharge at these locations.

5.6.4.4 Wick Drains

Consideration could also be given to the use of wick drains in combination with pre-loading to reduce the length of time required for consolidation. If wick drains are utilized under the pre-load embankment, it is anticipated that at least 90 percent of the estimated primary consolidation settlement will occur within about 3 months.

During the design phase of this project, it has become evident that due to schedule and cost considerations, the most feasible option to minimize settlement and improve stability is the use of wick drains in combination with pre-loading the embankment. In this regard, detailed wick drain design is given in Section 5.7 below.

5.7 Pre-load Embankment and Wick Drain Design

The following sections address the design of the proposed wick drain foundation system to be installed at the Bridge 9 approach embankments. A wick drain system is required at this site in order to allow the embankment construction to be completed within an accelerated schedule and to mitigate long-term, post-construction foundation settlements that could affect performance of the proposed RSS retaining wall systems to be constructed in this area.

The design has been carried out in accordance with the terms of reference for this work entitled, “*Wick Drain Design and Installation Monitoring, Terms of Reference, RHCE/QEW, WP 441-97-00*” dated May 12, 2004. The results of the additional field investigation that was carried out as part of this assignment, consisting of four (4) Cone Penetration Tests (CPTs) with pore pressure dissipation testing and seven (7) boreholes, and the additional laboratory testing, consisting of four (4) consolidation tests and one set of three (3) triaxial tests, have been included in Part A of this report.

It is our understanding that the east approach embankment is to be built in the fall of 2004 in a Regional Contract in order to accelerate the construction schedule. The west approach embankment is to be built in 2005 under the main Contract (Contract 1). The design of the wick drain foundation system addresses the stability and settlement of both approach embankments during and following construction.

5.7.1 Summary of Design Parameters

The following sections summarise the soil parameters selected for use in the design. Selection of the parameters was based on an assessment of the data collected during the field investigations and laboratory testing including an examination and comparison of these results using established empirical correlations and typical values found in literature.

5.7.1.1 Strength Parameters

The table below summarises the simplified stratigraphy, unit weight and associated strength parameters for the embankment and foundation soils used for assessing the stability of the critical sections of the east and west approaches. It should be noted that effective stress parameters are required for the assessment of the stability of the approach embankments when considering the appropriate sequence and rate of embankment construction on a wick drain foundation system.

Soil	Thickness (East Side) (m)	Thickness (West Side) (m)	g_{bulk} (kN/m ³)	Effective Friction Angle, f' (°)	Effective Cohesion, c' (kPa)
Earth embankment fill	7.8	9.1	21	32	0
Very stiff clayey silt till crust	1.5	2.5	21	30	5
Firm to stiff clayey silt till	4.5	4.5	20	27	0
Very stiff to hard clayey silt till	7.0	7.0	21	30	5
Very dense sandy silt till	3.0 – 10.0	3.0 – 10.0	21	35	0

The effective friction angle (ϕ') and cohesion (c') were assessed based on the results of the consolidated isotropic undrained (CIU) triaxial tests with pore pressure measurement carried out on specimens obtained from the soft to firm clayey silt till stratum. The results were compared with estimates of ϕ' from the empirical correlations with plasticity index (PI) proposed by Mitchell (1993) and Ladd et al. (1977) to estimate the values for the upper very stiff crust and lower very stiff to hard till soil layers. The values of c' and ϕ' for the very dense sandy silt till were estimated based on empirical correlations and the results of the SPT tests using the method proposed by Schmertmann (1975).

5.7.1.2 Deformation Parameters

The table below summarises the simplified stratigraphy, unit weight and associated deformation parameters employed for the assessment of the settlement of the foundation soil strata at the critical sections of the east and west approaches.

Soil	g_{bulk} (kN/m ³)	S_p' (kPa)	e_o	C_r	C_c	$C_{a(e)}$ (% / log- cycle time)
Very stiff, clayey silt till crust	21	400	0.40	0.025	0.15	-
Firm to stiff, clayey silt till	20	120	0.75	0.050	0.225	0.0027
Very stiff to hard, clayey silt till	21	600	0.50	0.025	0.15	-

The modulus of the very dense sandy silt till (underlying the lower very stiff to hard, clayey silt till) was estimated to be $E' = 75 \text{ MPa}$ based on the results of the SPT testing within this strata and employing the correlations proposed by Bowles (1984) and Kulhway and Mayne (1990).

The data tabulated above was assessed based on the results of the laboratory consolidation tests and in situ CPT tests and from a comparison with estimated values from empirical correlations based on laboratory index testing.

The preconsolidation pressure (σ_p') was evaluated from the results of the consolidation tests using the methods proposed by Casagrande (1936), Becker et al. (1987) and Onitsuka et al. (1995). It was also assessed from the results of the CPT tests using the following method proposed by Mayne and Holtz (1988):

$$\sigma_p' = 0.4(q_c - \gamma \cdot z)$$

where: q_c = tip stress measured by the CPT (kPa)
 $\gamma \cdot z$ = total vertical stress (kPa)

The following correlation relating in situ vane shear strength to preconsolidation pressure proposed by Mesri (1975) was also employed:

$$s_u = 0.22 \sigma_p'$$

where: s_u = average mobilised undrained shear strength (kPa)

A summary of these interpretations, including the selected design line for analysis is plotted versus elevation on Figure 3.

The initial void ratio (e_o) within the strata was evaluated based on measurements from the trimmed specimens used for the consolidation tests and based on the water contents measured on the SPT samples obtained during the field investigation using the following correlation:

$$e_o = w_n \cdot G_s \quad (\text{assuming 100\% saturation})$$

where: w_n = natural water content
 G_s = specific gravity (=2.77 based on 4 laboratory tests)

A summary of the estimated void ratio, including the selected design line for analysis is plotted versus elevation on Figure 10.

The recompression index (C_r) and compression index (C_c) for the strata was evaluated based on the results of the laboratory consolidation tests. These results were then combined with estimates

of C_r and C_c based on the Atterberg Limits testing and water contents measured on the SPT samples obtained during the field investigation, using the following correlations:

where: $C_c = 0.009 (w_L - 10)$ (after Terzaghi and Peck, 1967)
 w_L = liquid limit (in %)

where: $C_c = PI/74$ (after Kulhawy and Mayne, 1990)
 PI = plasticity index (in %)

where: $C_c = 0.75(e_o - 0.50)$ (after Azzouz et al., 1976)
 e_o = void ratio

and;

$C_r = PI/385$ (after Kulhawy and Mayne, 1990)
 $C_r = C_c/5$ (after Britto and Gunn, 1987)

A summary of this data, including the selected design lines for C_r and C_c used in the analysis is plotted versus elevation on Figure 11 and 12, respectively.

The coefficient of secondary consolidation ($C_{\alpha(e)}$) (i.e. creep) of the firm to stiff clay silt till stratum was assessed from the results of the 24 hour load increment, consolidation tests (considering the appropriate stress level) and from estimates based on the results of the index testing (i.e. water contents) and the empirical correlation proposed by Mesri (1973).

5.7.1.3 Rate of Consolidation

The table below summarises the average coefficients of consolidation (c_h and c_v) used in the analysis of the rate of consolidation settlement of the foundation soil strata at the critical sections of the east and west approaches.

Direction / Orientation	Average Coefficient of Consolidation (cm^2/s)
Horizontal (c_h)	4.6×10^{-3}
Vertical (c_v)	4.3×10^{-3}

The coefficient of consolidation in the vertical direction (c_v) was assessed from the results of the consolidation tests performed on horizontally trimmed specimens (HTO) of the clayey silt till. Values of c_v were also assessed from the consolidation phase of the CIU triaxial tests and from estimates based on the results of the Atterberg limits testing (i.e. liquid limit) and the empirical correlation proposed by U.S. Navy (1971).

The coefficient of consolidation in the horizontal direction (c_h) was assessed from the results of the consolidation tests performed on vertically trimmed specimens (VTO) of the clayey silt till. In addition, values of c_h were assessed from the results of the pore pressure dissipation testing carried out as part of the CPT testing at the site. A total of six pore pressure dissipation tests were performed and the results are shown on Figure 4. Based on this data, c_h was estimated using the following method proposed by Robertson et al. (1992):

$$c_h = (m/M)^2 \cdot v I_r \cdot r^2$$

where: m = gradient of the initial linear portion of the dissipation curve
 $M = 1.15$ (for CPT pore pressure sensor at position u_2)
 I_r = rigidity index, G/s_u (~ 120 for this site)
 r = radius of CPT probe (=17.8 mm)

The average coefficient of consolidation (c_v and c_h) reported above has been calculated based on a weighted average of the field and laboratory data.

5.7.1.4 Pore Pressure Parameters

The table below summarises the simplified stratigraphy and the average pore pressure coefficients (A and B) used in the analysis of the excess pore pressure response in the foundation soil strata due to embankment construction at the critical sections of the east and west approaches.

Soil	Pore Pressure Coefficient		
	$A_{(elastic)}$	$A_{(yield)}$	B
Very stiff, clayey silt till crust	0.3	-	0.95
Firm to stiff, clayey silt till	0.3	0.53	0.95
Very stiff to hard, clayey silt till	0.3	-	0.95

The pore pressure coefficients were assessed from the CIU triaxial tests with pore pressure measurement performed on specimens of the soft to firm clayey silt till. The values of $A_{(elastic)}$

were estimated from the initial straight line portions of the deviator stress and excess pore pressure versus strain curves at low strain levels. The values of $A_{(yield)}$ were estimated at higher strain levels on the non-linear portions of the deviator stress versus strain curves, but at strain levels less than failure.

5.7.2 Performance Requirements

The following criteria were established for the long-term performance of the retaining walls and approach embankments at this site:

Location	Total Post-Construction Settlement (mm)
Abutment to 30 metres beyond abutment	25
>30 metres	50

The performance criteria were considered as part of the design of the wick drain design.

5.7.3 Method of Analysis

Analyses were carried out to assess the effect of wick drain spacing on the response of the foundation soils to the proposed pre-load embankment fills. The analyses, as discussed below, include assessing the optimal wick drain spacing for the site considering the effect on rate of construction, development of excess pore pressure, embankment stability, rate of consolidation settlement and the time available within the scheduled pre-load period (i.e. 90 days) for consolidation to take place.

The analyses employed the closed form solutions for assessing the degree of consolidation by radial (or horizontal) drainage (U_h) proposed by Barron and Kjellman (1948) including the extended solutions of Hansbo (1979) developed specifically to assess the use of prefabricated geosynthetic drains (i.e. wick drains) for the consolidation of soft, cohesive strata. The extended solutions by Hansbo (1979) permit including the effects of the wick drain well resistance/discharge capacity and the effects of smear of the soil along the wick drain caused by installation on the rate of pore pressure dissipation/consolidation.

The average degree of consolidation (U) within the firm to stiff, clayey silt till was calculated considering the contributions of both the horizontal consolidation (U_h) and the vertical

consolidation (U_v). This approach is important at this site considering the relatively thin nature of the firm compressible strata. The following equation for combined vertical and radial drainage was employed:

$$U = 1 - (1 - U_v)(1 - U_h)$$

where: U_v = average degree of consolidation from vertical drainage only from Terzaghi's 1-D consolidation theory
 U_h = average degree of consolidation from radial drainage only (to the wick drains) from Hansbo (1979)

The total elastic settlement (s_E) of the upper very stiff clayey silt till crust, the lower very stiff to hard clayey silt till, and the looser very dense sandy silt till and the total primary consolidation settlement (s_C) of the firm to stiff clayey silt till, were calculated using the commercially available program Unisettle (Version 3.0) produced by Unisoft Limited using the simplified stratigraphy and deformation parameters summarised previously. The settlement analysis for the east and west pre-load embankments was assessed at the critical (i.e. highest) sections of the approaches.

The primary consolidation settlement with time, $s(t)$, was calculated based on the average degree of consolidation (U) completed at the time of interest (as described above) using the following formula:

$$s(t) = s_C \cdot U$$

where: s_C = total primary consolidation settlement
 U = average degree of consolidation at time, t

The secondary consolidation settlement (s_S) of the firm to stiff clayey silt till was calculated using the value of $C_{\alpha(\epsilon)}$ summarised previously in conjunction with the following formula:

$$s_S = C_{\alpha(\epsilon)} \cdot H_o \cdot (\Delta \log t)$$

where: $C_{\alpha(\epsilon)}$ = coefficient of secondary consolidation
 H_o = initial thickness of the firm to stiff compressible layer
 $\Delta \log t$ = the logarithm of the time period of interest

The excess pore pressure (Δu) response within the clayey silt till as a result of the embankment construction was estimated as follows:

$$\Delta u = \bar{B} \cdot \Delta s_1 = \bar{B} \cdot g \cdot \Delta H$$

where: \bar{B} = overall (or combined) pore pressure coefficient (*see below*)
 γ = bulk unit weight of embankment fill (=21 kN/m³)
 ΔH = change height of embankment fill

The overall pore pressure coefficient (\bar{B}) was calculated from the following equation proposed by Skempton (1954):

$$\bar{B} = B \left[1 - (1 - A) \left(1 - \frac{\Delta s_3}{\Delta s_1} \right) \right]$$

where: A & B = Skempton pore pressure parameters estimated from the results of the CIU triaxial tests (as summarised previously)
 $\Delta \sigma_3 / \Delta \sigma_1$ = principal stress ratio at point of interest within foundation soil below embankment (calculated from the solutions by Perloff et al. (1967))

The excess pore pressure with time, $\Delta u(t)$, was calculated based on the average degree of consolidation (U) completed at the time of interest (as described above) using the following formula:

$$\Delta u(t) = \Delta u \cdot (1 - U)$$

where: Δu = excess pore pressure
U = average degree of consolidation at time, t

It should be noted that different rates of construction were considered in the analyses. The rate of construction assigned influences the excess pore pressure calculated at any time, t, due to the fact that the slower the rate of construction, the larger the degree of consolidation that will be completed within any one construction stage.

Effective stress stability analysis of the critical sections of the staged pre-load embankment construction was carried out using the commercially available program Slope/W (Version 5.19), produced by Geo-Slope International Ltd., employing the Morgenstern-Price method of analysis. The stability analyses employed the simplified stratigraphy and strength parameters summarised previously. The total pore pressure, u_T , within the foundation strata at the appropriate time, t, is required as part of the effective stress stability analysis and was calculated as follows:

$$u_T(t) = u_o + \Delta u(t)$$

where: u_o = initial pore pressure within the foundation stratum (based on ground water table at Elevation 79.0 m)
 $\Delta u(t)$ = excess pore pressure at time, t

For all analyses, the factors of safety of numerous potential failure surfaces were computed in order to establish the minimum factor of safety. The stability analyses were performed to check that the target minimum factor of safety (FoS = 1.3) was achieved for each stage of the pre-load embankment construction.

5.7.4 Results of Analysis

The following sections present the results of the analysis assessing the affect of wick drains (at different spacings) on the rate of settlement, development of excess pore pressure and stability of the east and west pre-load embankments.

5.7.4.1 Settlement

The estimated magnitude of the elastic, primary and secondary consolidation settlements for the critical (i.e. highest) sections of the pre-load embankments is summarised in the following table.

Settlement Component	East Pre-load Embankment (mm)	West Pre-load Embankment (mm)
Elastic (upper crust)	30	50
Elastic (lower tills)	55	55
Primary Consolidation (firm to stiff clayey silt till)	150	135
Secondary (Creep) per log cycle of time	12	12

The elastic settlement of the upper stiff clayey silt till crust is anticipated to be completed in less than about 1 week following completion of construction. The elastic settlement of the lower tills (i.e. below the extent of the wick drains) is anticipated to be completed in less than about 2 months following completion of construction.

The rate of primary consolidation within the firm to stiff clayey silt till is dependent on the wick drain spacing. Estimations of settlement versus time for a range of wick drain spacings (1.0 m,

1.5 m, 1.75 m and 2.0 m) and for no wick drains are presented on Figure 13 and 14 for the east and west embankments, respectively.

A plot summarising the percent of consolidation, U , completed at the end of the currently proposed 3 month (or 90 day) pre-load period is presented on Figure 15 for four different wick drain spacings. It can be seen that in order to achieve the target of 90% consolidation within the pre-load period, a wick drain spacing of 1.0 m is required at this site.

A plot of settlement (primary and secondary) versus time for both the east and west embankment for 1 m wick drain spacing is presented on Figure 16.

5.7.4.2 Excess Pore Pressures

The estimated excess pore pressure response at the middle of the firm to stiff clayey silt till stratum (at about Elevation 76.2 m) below the critical sections of the pre-load embankments during and following construction is shown on Figures 17 and 18 for the east and west embankments, respectively. The excess pore pressures have been calculated assuming 1 m wick drain spacing and a rate of construction equivalent to about 2 m of fill placed per 24 hour period. The following table presents the same data at regular time intervals:

Time (days)	Fill Height, H (m)		Approximate Excess Pore Pressure, Du (kPa)	
	<i>East Side</i>	<i>West Side</i>	<i>East Side</i>	<i>West Side</i>
0	0	0	0	0
1	2	2	26	26
2	4	4	43	43
3	6	6	53	53
4	7.8	8	87	89
5	7.8	9.1	83	98
6	7.8	9.1	79	94
7	7.8	9.1	76	90
14	7.8	9.1	61	72
21	7.8	9.1	50	58
28	7.8	9.1	41	48
60	7.8	9.1	17	39
90	7.8	9.1	8	20
120	7.8	9.1	4	9
150	7.8	9.1	2	4
180	7.8	9.1	1	2
210	7.8	9.1	0	1
240	7.8	9.1	0	0
270	7.8	9.1	0	0
300	7.8	9.1	0	0

5.7.4.3 Stability

The impact of the excess pore pressure development (due to a 2m per 24 hour fill rate of construction) on the stability of the pre-load embankment at each stage of construction and the stability over a 24 month period is shown on Figure 19 and 20, respectively. The stability analyses were carried out assuming a 2H:1V side slope profile for the east embankment and a 1.5H:1V side slope profile for the west embankment. It can be seen that for a 2 m per 24 hour fill rate and for a wick drain spacing of 1.0 m, the Factor of Safety (FoS) at each stage of construction is greater than 1.3 for both the east and west pre-load embankments. It can also be seen that, following the completion of construction, the FoS increases with time as excess pore pressures dissipate and the nearly fully drained condition is reached.

Figure 21 and 22 show the results of the effective stress stability analysis for the east and west pre-load embankments, respectively, at the most critical stage of construction (i.e. upon application of the final fill lift). It can be seen that a FoS greater than 1.3 is maintained in both cases.

In order to ensure that the FoS is greater than 1.3 during embankment construction, the maximum allowable excess pore pressure (as measured in the field) should not exceed 95 kPa and 120 kPa for the east and west pre-load embankments, respectively. The maximum allowable excess pore pressure is higher at the west pre-load embankment due to the lower groundwater table and thicker crust in this area of the site.

5.7.5 Design Recommendations for Embankments on Wick Drain Foundation

Based on the results of the time rate consolidation analysis, considering the required 3 month pre-load period, and the embankment stability analysis considering rate of construction (as described in the previous section), the following recommendations are provided for the design layout of the wick drain foundation system.

Wick Drain Layout Requirements

Drain Details	East Pre-load Embankment	West Pre-load Embankment
Spacing	1.0 m	1.0 m
Grid Pattern	triangular	triangular
Lateral Extent	between embankment toes (approx. 20 m LT of centreline to 20 m RT)	between embankment toes (approx. 25 m LT of centreline to 25 m RT)
Longitudinal Extent	between embankment toes (approx. station 10+495 to 10+545)	between embankment toes (approx. station 10+370 to 10+435)
Vertical Extent	to about Elevation 73 m (approx. 8 m below top of drainage layer)	to about Elevation 73 m (approx. 8.5 m below top of drainage layer)

Pre-Drilling Requirements

Based on discussions with wick drain installation contractors, it is our understanding that pre-drilling through very stiff to hard crustal soils can be required to facilitate installation of wick drains at some sites. In general, in strata where SPT N-values are greater than about 15 blows per 0.3 m of penetration, mandrel penetration may be difficult. Figure 23 presents a summary of the SPT data for the both the east and west pre-load embankment areas. This data has been used to provide guidance on the depth of pre-drilling required at each area of the site.

Pre-Drill Details	East Pre-load Embankment	West Pre-load Embankment
Terminus Elevation	78.5 m	78.5 m
Depth Below Original Ground Surface	2.0 m – 2.5 m	2.5 m – 3.0 m

Construction/Installation Requirements

The following recommendations are provided as guidelines for good construction practice and to ensure that drainage from the wick drains is facilitated away from the pre-load area so that the wick drain foundation functions efficiently during the pre-load period:

- all topsoil/organics to be stripped within area of pre-load prior to placement of granular blanket and embankment fill.
- after stripping, localised grading is required to shape subgrade to 3% minimum crossfall before placing granular drainage blanket.
- shallow ditching to be constructed around the perimeter of the embankment to facilitate drainage away from the pre-load area.
- 0.3 m thick granular drainage blanket composed of Granular 'B' Type II fill to be placed on ground surface following stripping and grading.

5.7.6 Specifications

The specifications describing the requirements for the supply and installation of the wick drains as well as the specifications and contract drawings describing the type, location, supply and installation of the instrumentation required for the monitoring were supplied under separate cover.

5.7.7 Monitoring

The monitoring program for the east and west abutments as shown on the Contract Drawings should be carried out with the following frequency:

Stage	Frequency (over given time)
Baseline Reading	3 readings on 3 consecutive days, no sooner than 7 days following installation
Just prior to start of embankment construction	Once
During embankment construction	Once every 0.5 m fill lift within 20 m of the monitoring section
After end of embankment construction	Daily for 2 weeks Bi-weekly for 2 months Monthly for duration of pre-loading (assume 12 months in total)

5.8 Retaining Wall Foundations

As discussed in Section 5.6 two long retaining walls are required on the northwest and southeast corners of the bridge. At the east abutment, the south retaining wall is 22 m long extending back from the front face of the east abutment and is up to 6.6 m in height. The north retaining wall at the west abutment is about 30 m long extending back from the front face of the west abutment and is up to 7.7 m in height.

Consideration could be given to the use of a mechanically-reinforced soil retaining wall system (retained soil system or RSS wall) or a pile-supported concrete wall. The advantages, disadvantages, relative costs and risks/ consequences for the different wall options are summarized in Table 2.

5.8.1 Settlement and Stability

The settlement of the wall is governed by the embankment loading on the underlying firm to stiff clayey silt till. As discussed in Section 5.6.3, settlement underneath the wall is expected to be up to 200 mm. Settlement options are discussed in Section 5.6.4 and it is understood that the use of wick drains and pre-loading is the preferred option to minimize settlement beneath the embankment, assuming that 90 percent consolidation takes place with the use of wick drains, the settlement of the walls would be less than 25 mm.

The internal stability of the mechanically-reinforced soil walls should be checked by the RSS supplier / designer. Detailed discussion of global stability of the wall configurations are given in Section 5.6.2.

5.8.2 Geotechnical Resistance

An RSS wall typically consists of granular fill placed and compacted in layers, and reinforced with metal or fabric strips or grids. A facing material, typically pre-cast concrete panels mechanically fastened to the reinforcing strips or grids, is used to form the face of the reinforced soil structure and to prevent the loss of fill material and is supported on a strip footing. A typical RSS wall has a front facing supported on a strip footing placed at shallow depth below the ground surface in front of the wall. The footing must be founded below any topsoil, loose fill or unsuitable native soils.

Assuming that the RSS wall acts as a unit and utilizes the full width of the reinforced soil mass, which is taken as two-thirds of the height of the wall, the geotechnical resistances given in the

following table may be used for assessment of the reinforced mass founded on the properly prepared embankment fill materials or on the stiff to very stiff clayey silt deposits. These resistances assume that the embankment has been pre-loaded prior to construction of the wall. Allowance should be made for sub-excavation and replacement with compacted granular fill if unsuitable soils are encountered at the founding level of either the reinforced mass or the facing footing; an NSSP may be required for this purpose.

<i>Wall Height</i>	<i>Assumed Width</i>	<i>Factored Geotechnical Resistance at ULS</i>	<i>Geotechnical Resistance at SLS</i>
6.6 m (east abutment)	4.4 m	350 kPa	250 kPa
7.7 m (west abutment)	5.1 m	400 kPa	275 kPa

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, $\tan \delta$, between the compacted granular fill of the RSS wall and the existing fill materials or the native clayey silt soils may be taken as 0.57. This represents an unfactored value; in accordance with the CHBDC, a factor of 0.8 is to be applied in calculating the horizontal resistance

5.9 Excavations and Temporary Cut Slopes

Excavations for construction of the pile cap at the abutments and piers will typically extend through compact silty sand to sandy silt fill and stiff to very stiff clayey silt if the pile cap is located below the existing ground surface. Alternatively, the abutment pile cap may be constructed up higher within the newly constructed embankment fill. In the case of the pier pile caps, the excavation may extend into the soft to firm clayey silt till. This deposit is very susceptible to disturbance from ponded water and construction traffic. If the base of the excavation is formed within the soft to firm clayey silt till, a mud coat will probably be required for suitable working conditions.

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 stiff to very stiff clayey silt at the site are classified as Type 3 soil, according to the OHSA handbook. If excavations extend into the soft to firm clayey silt till, these soils may be classified as Type 4 soil. Temporary excavations (i.e. those which are only open for a relatively short period) through these overburden soils should be made with side slopes no steeper than 1.5

horizontal to 1 vertical (1.5H:1V) through the fill and the clayey silt; surficial sloughing may occur and face treatment may be required.

The ground water level at the site is typically located at the base of the soft to firm clayey silt till below about Elevation 75.0 m although the groundwater level was measured as high as 78.7 m at the west approach. It is anticipated the excavations at this site will typically be above the groundwater table and it is likely that sumping from properly filtered sumps will adequately control the groundwater within open-cut excavations.

Excavation support is not anticipated at this site, however, it may be required at some locations due to the proximity of the existing ramps. If temporary roadway protection is 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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Designated MTO Contact



CN/SEP/JPD/ASP/FJH/sm

N:\Active\2002\1100\021-1162 QEW-Red Hill Creek\Phase 5000 Reports\Red Hill Creek Interchange\Bridge 9\Final\021-1162 RPT 04Aug Final Report Bridge 9.doc

TABLE 1
EVALUATION OF FOUNDATION ALTERNATIVES
BRIDGE 9, RED HILL CREEK EXPRESSWAY INTERCHANGE

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

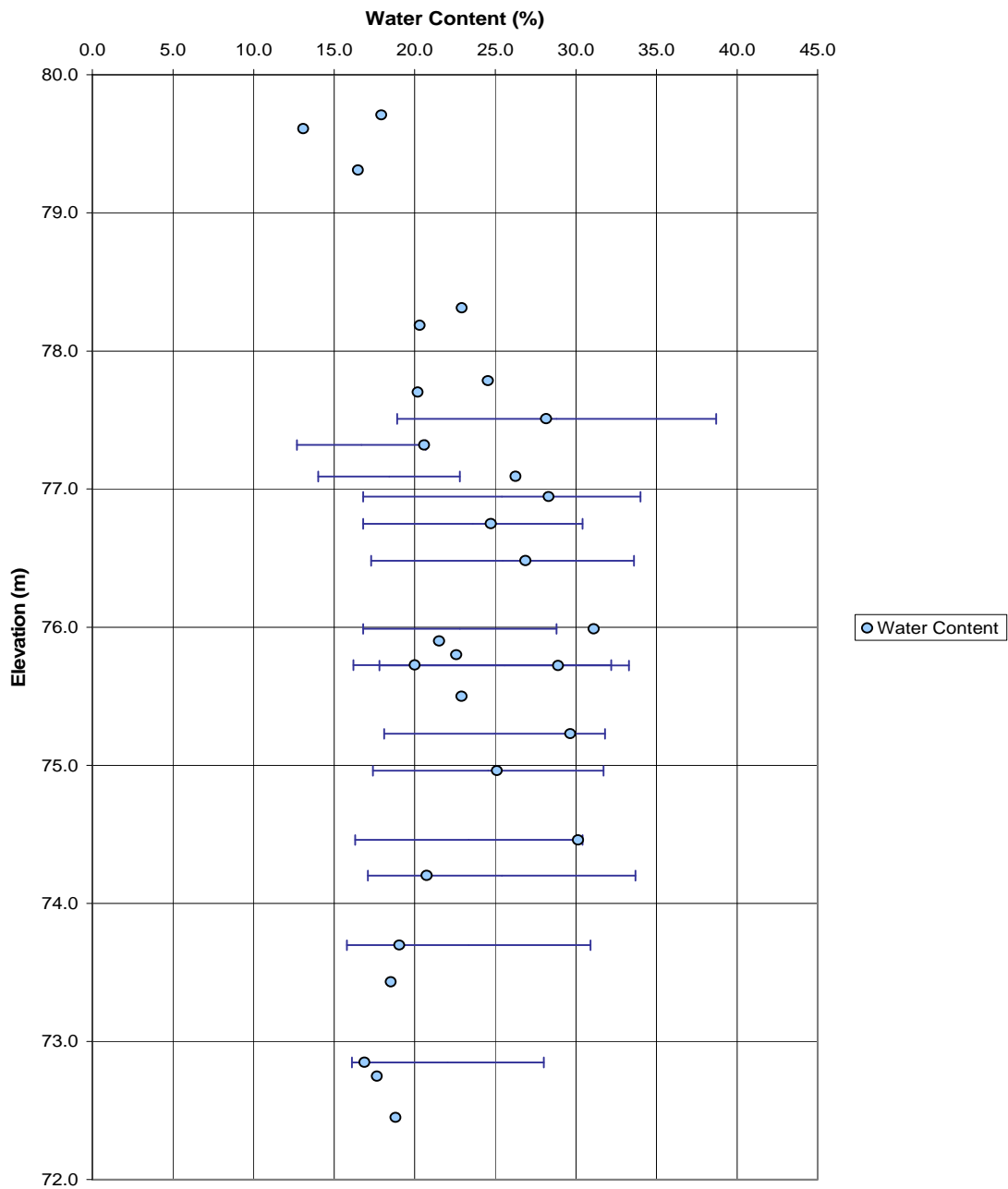
NF: Indicates that the founding option is considered not feasible.

TABLE 2
EVALUATION OF RETAINING WALL ALTERNATIVES
RED HILL CREEK EXPRESSWAY INTERCHANGE
G.W.P. 441-97-00

<i>Wall Option</i>	<i>Advantages</i>	<i>Disadvantages</i>	<i>Relative Costs</i>	<i>Risks/Consequences</i>
Retained Soil System (RSS Wall)	Relative ease of construction; foundation placed on prepared native soils; excavation not required.	Post-construction settlements due to soft clay soil even with use of lightweight fill backfill. Preloading and removal of preload required to reduce post-construction settlements	The wall itself may be less costly than pile supported wall but pre-load and removal more costly. Increased costs if lightweight fill is used as backfill.	Impacts of ground settlements minimized if preload carried out.
Concrete Cantilever Gravity Wall (Pile Supported)	Post-construction settlement not a factor for conventional cantilever gravity wall, only for downdrag on piles.	Construction of foundation will be more involved than RSS wall; heavy equipment required for pile driving; minor excavation required for pile cap. Soil anchors may be required; concern that consolidation settlement due to embankment fill loading may impact the tieback anchors.	Cost of foundation may be considerable compared to RSS wall.	Without pre-loading the approach embankment, there will be settlement of the road differential to the wall and abutment.

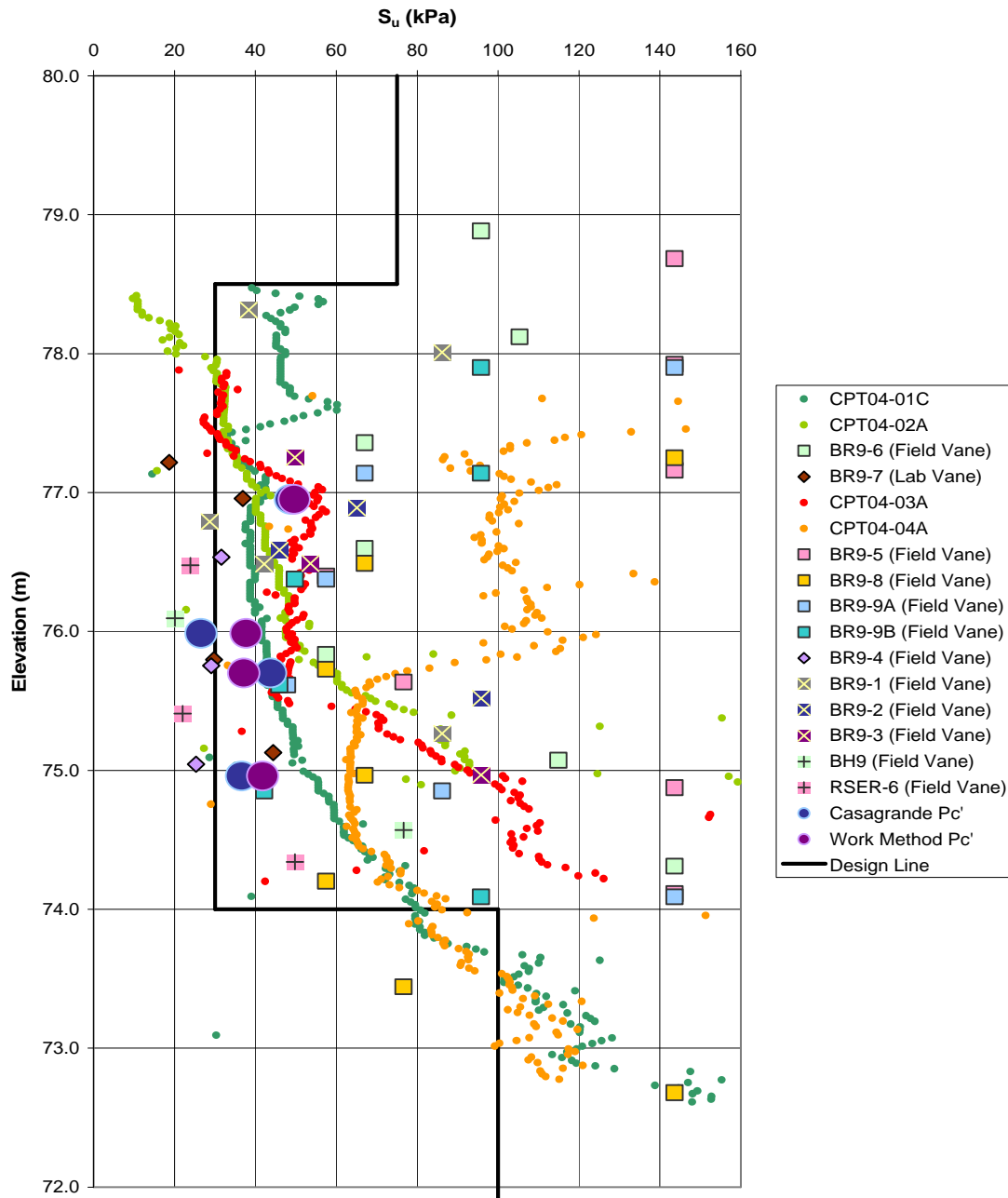
WATER CONTENT AND ATTERBERG LIMIT
QEW-RHCE BRIDGE 9

FIGURE 1



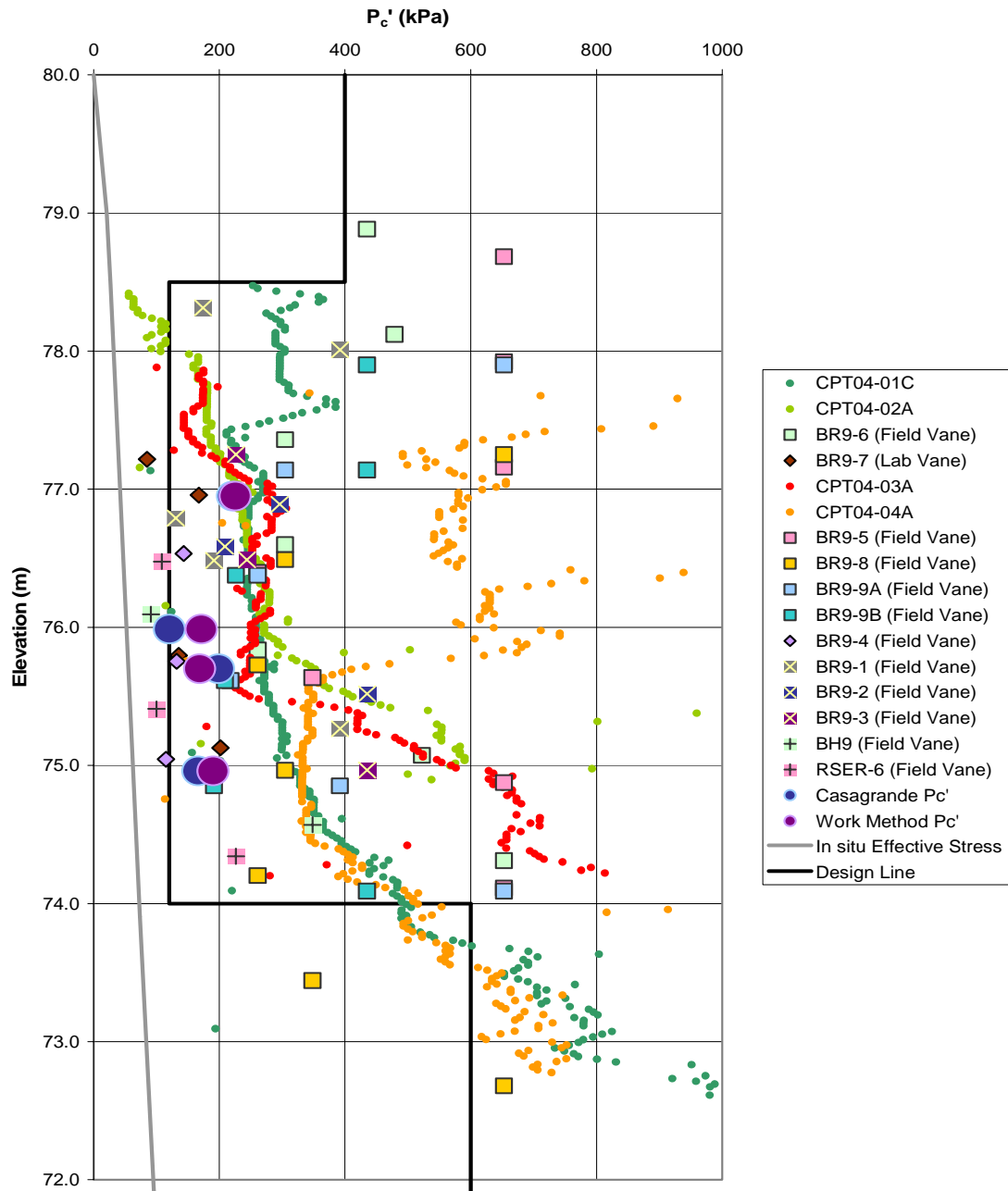
SHEAR STRENGTH AND DESIGN LINE QEW-RHCE BRIDGE 9

FIGURE 2



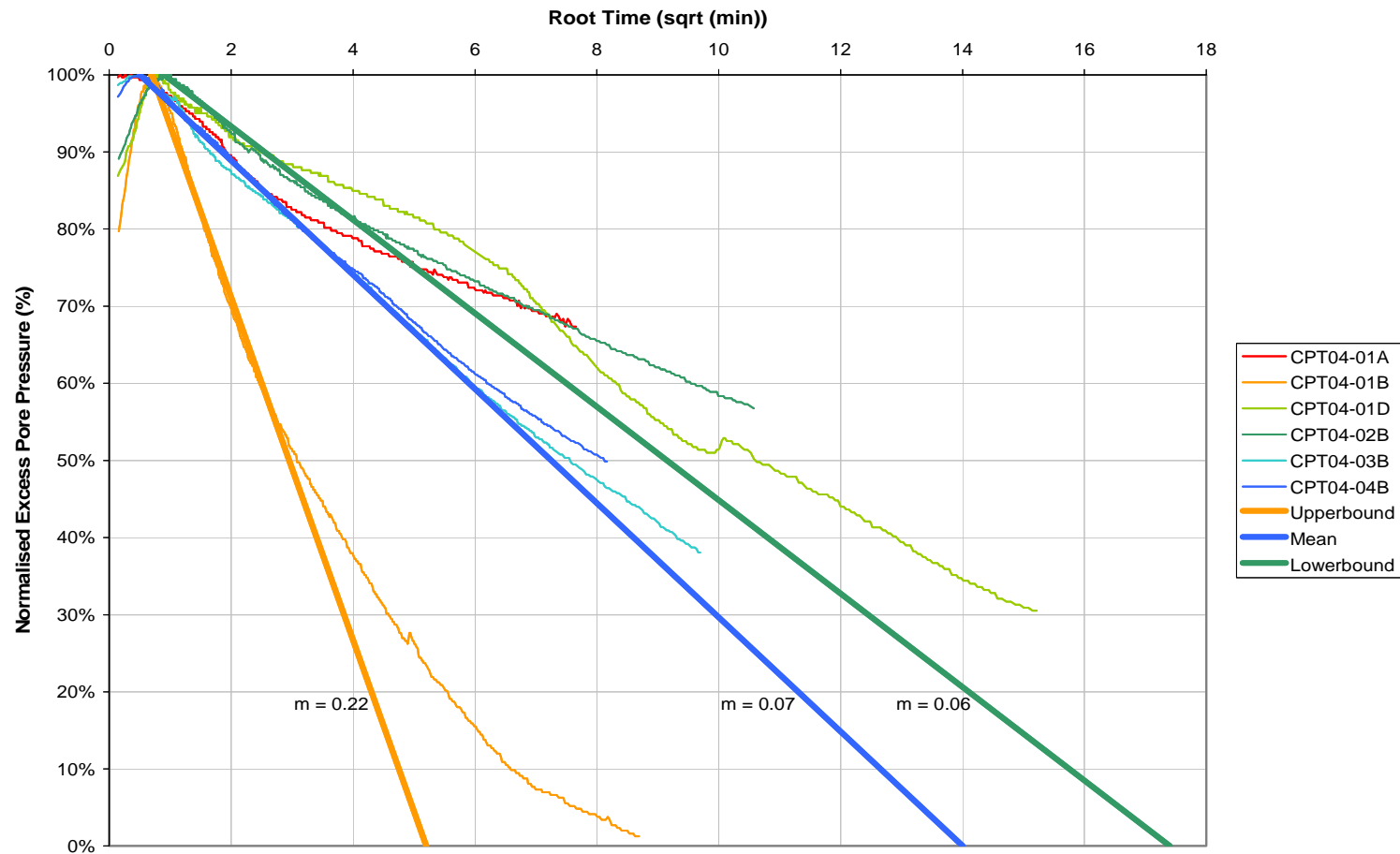
PRECONSOLIDATION PRESSURE AND DESIGN LINE QEW-RHCE BRIDGE 9

FIGURE 3



CPT PORE PRESSURE DISSIPATION TEST QEW-RHCE BRIDGE 9

FIGURE 4



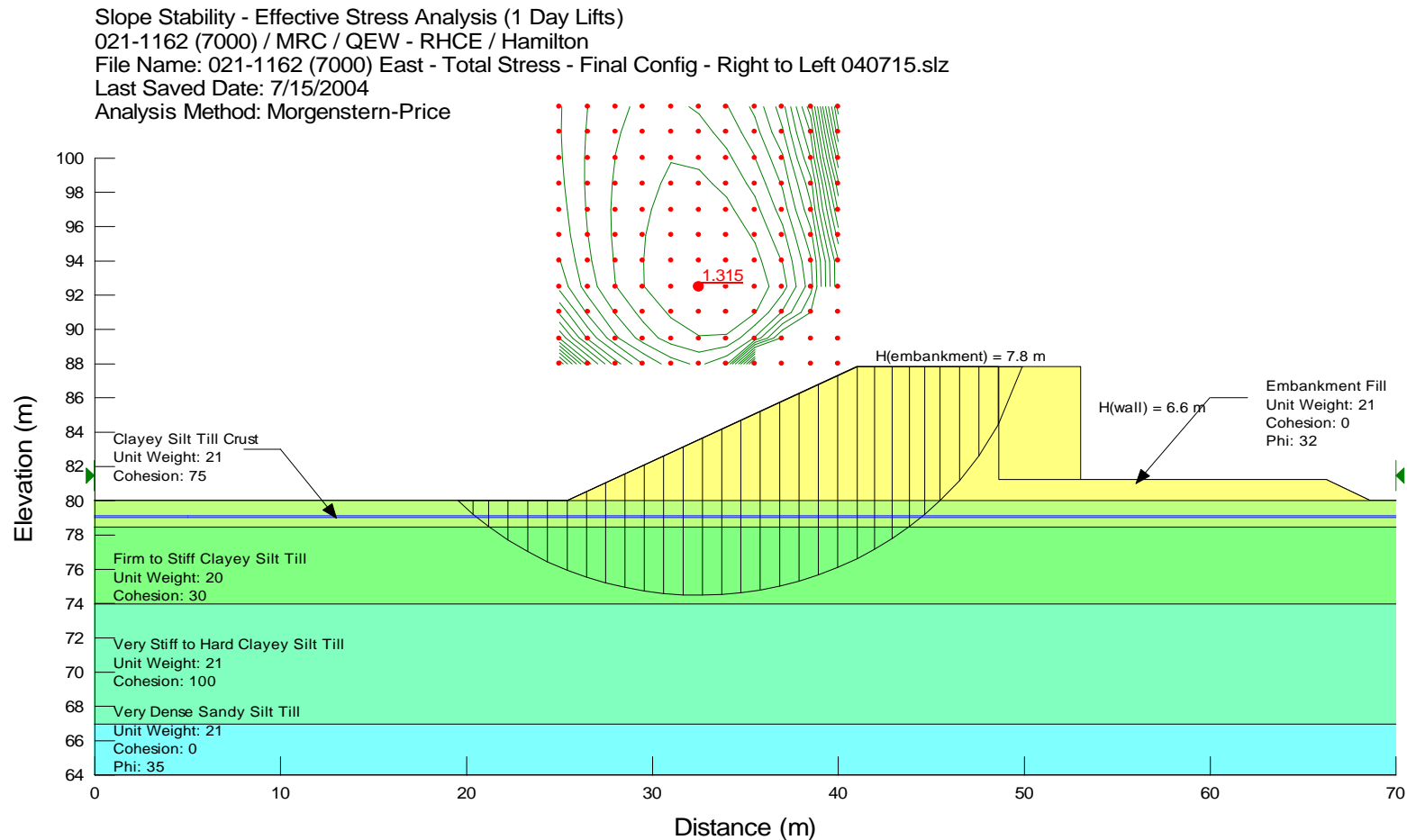
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Project: 021-1162

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APPROACH EMBANKMENT STABILITY ANALYSIS - NO WICK DRAINS **QEW-RHCE BRIDGE 9 - EAST EMBANKMENT (RIGHT TO LEFT)**

FIGURE 5



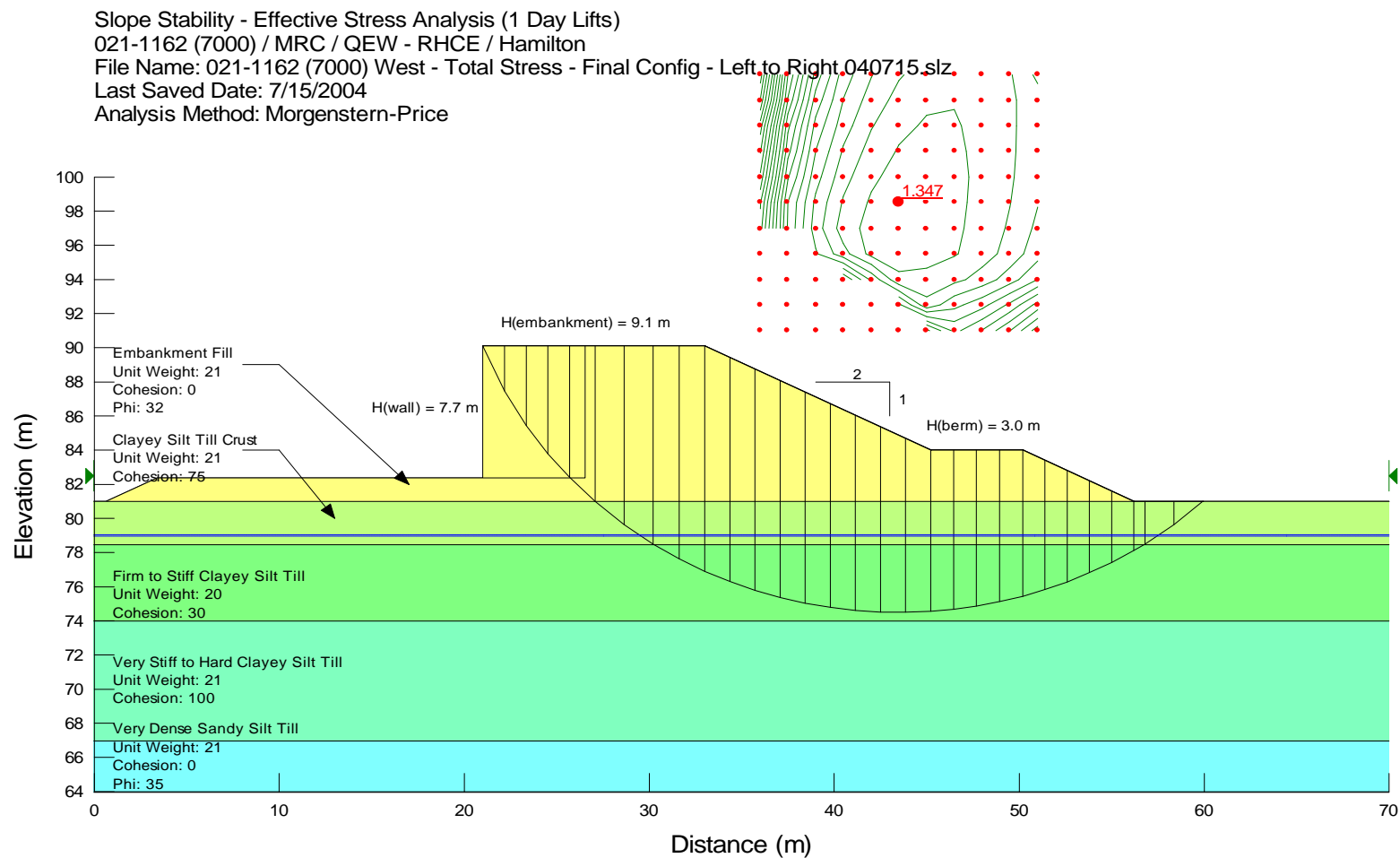
Date: July, 2004
 Project: 021-1162

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APPROACH EMBANKMENT STABILITY ANALYSIS - NO WICK DRAINS **QEW-RHCE BRIDGE 9 - WEST EMBANKMENT (LEFT TO RIGHT)**

FIGURE 6



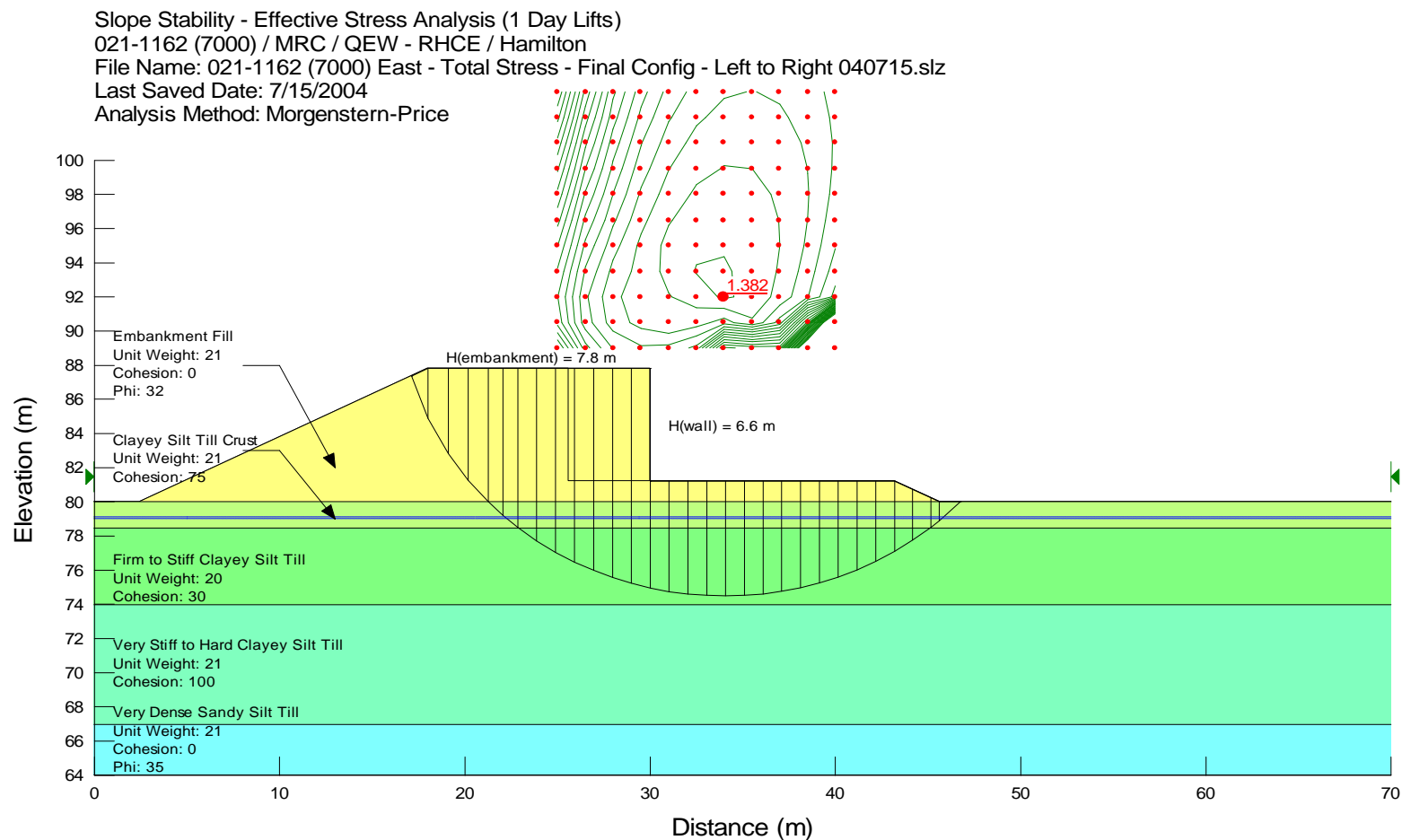
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APPROACH EMBANKMENT STABILITY ANALYSIS - NO WICK DRAINS **QEW-RHCE BRIDGE 9 - EAST EMBANKMENT (LEFT TO RIGHT)**

FIGURE 7



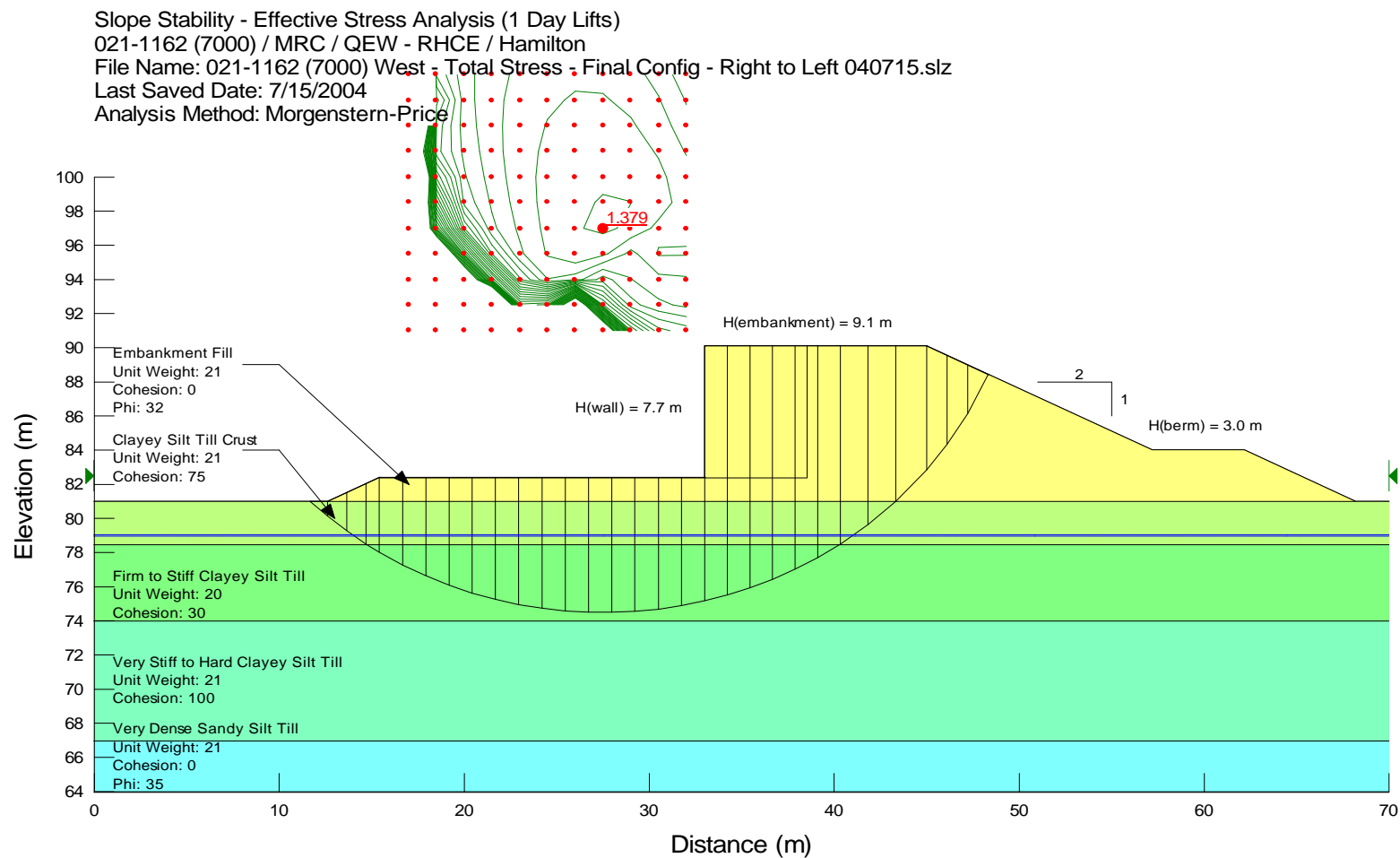
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APPROACH EMBANKMENT STABILITY ANALYSIS - NO WICK DRAINS **QEW-RHCE BRIDGE 9 - WEST EMBANKMENT (RIGHT TO LEFT)**

FIGURE 8



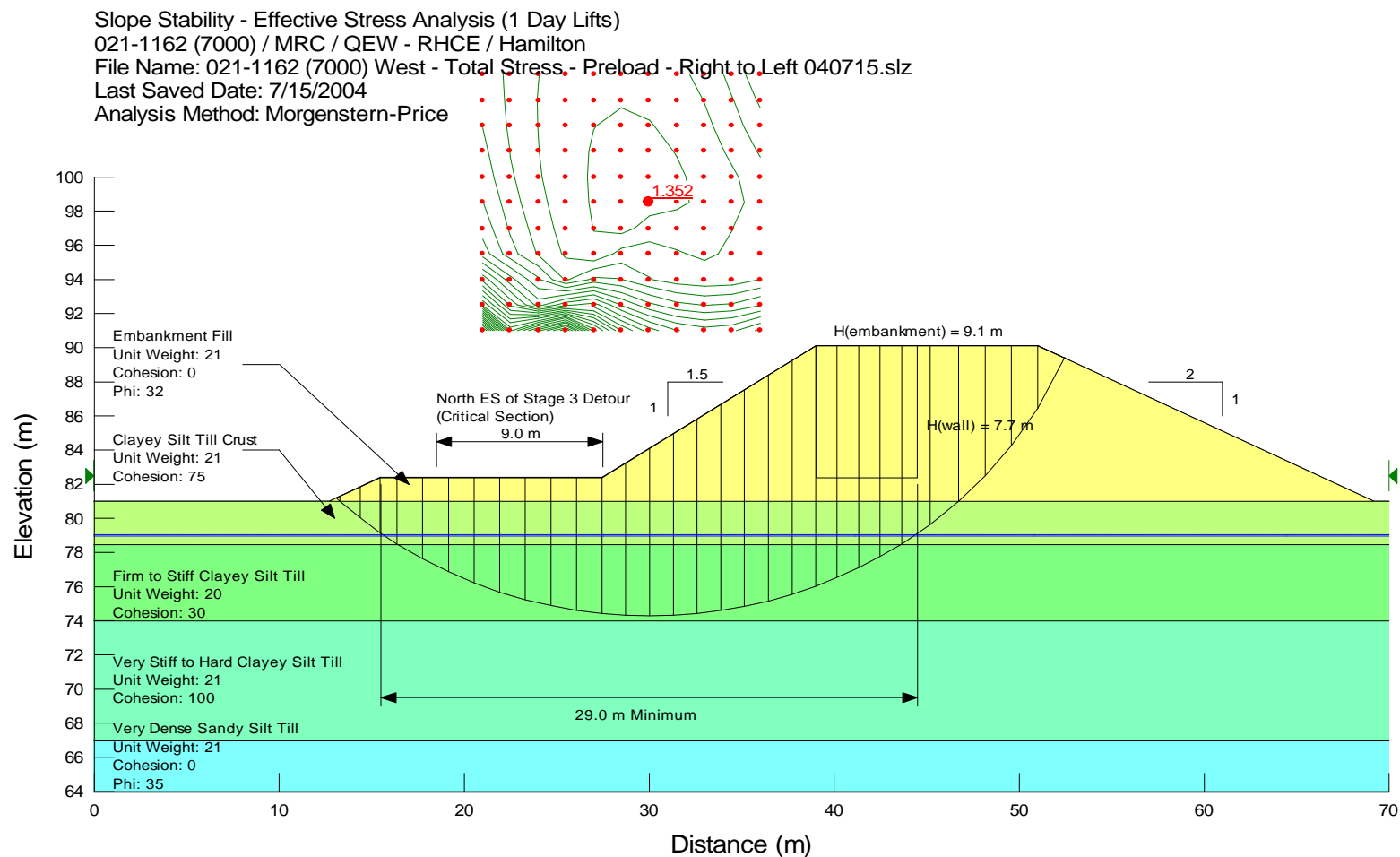
Date: July, 2004
 Project: 021-1162

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 Checked: JPD

APPROACH EMBANKMENT STABILITY ANALYSIS - PRELOAD EMBANKMENT QEW-RHCE BRIDGE 9 - WEST EMBANKMENT (RIGHT TO LEFT)

FIGURE 9



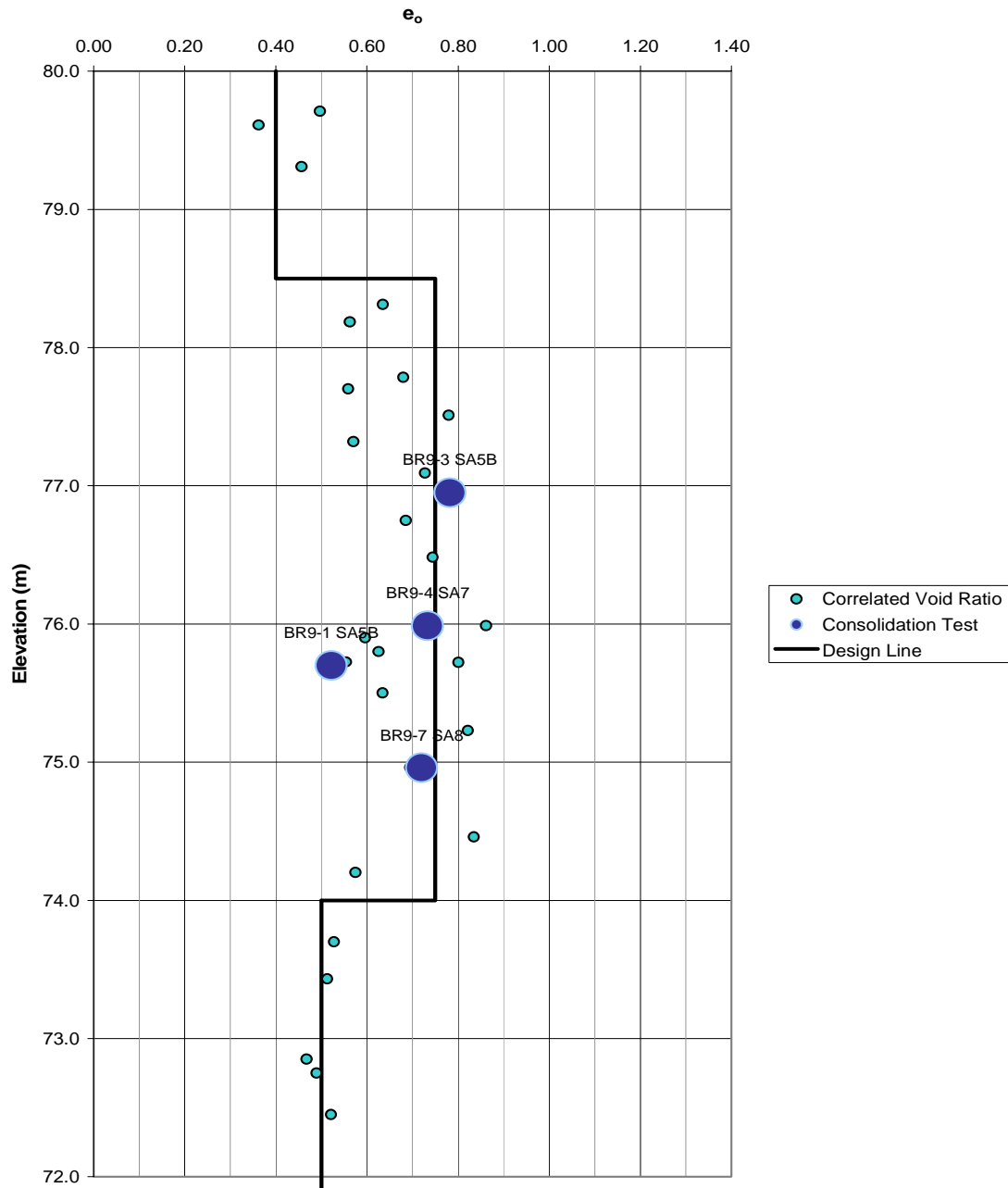
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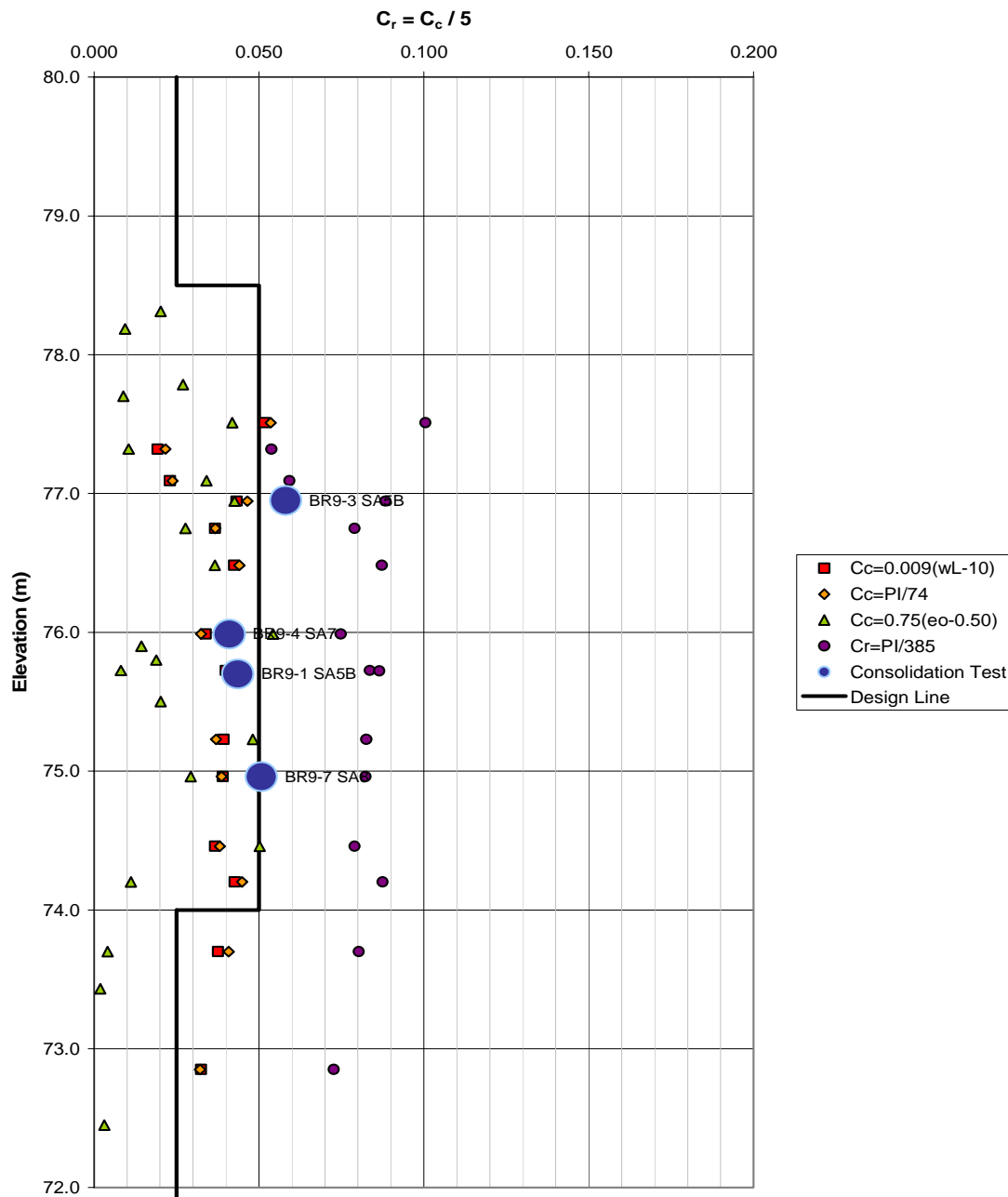
VOID RATIO AND DESIGN LINE QEW-RHCE BRIDGE 9

FIGURE 10



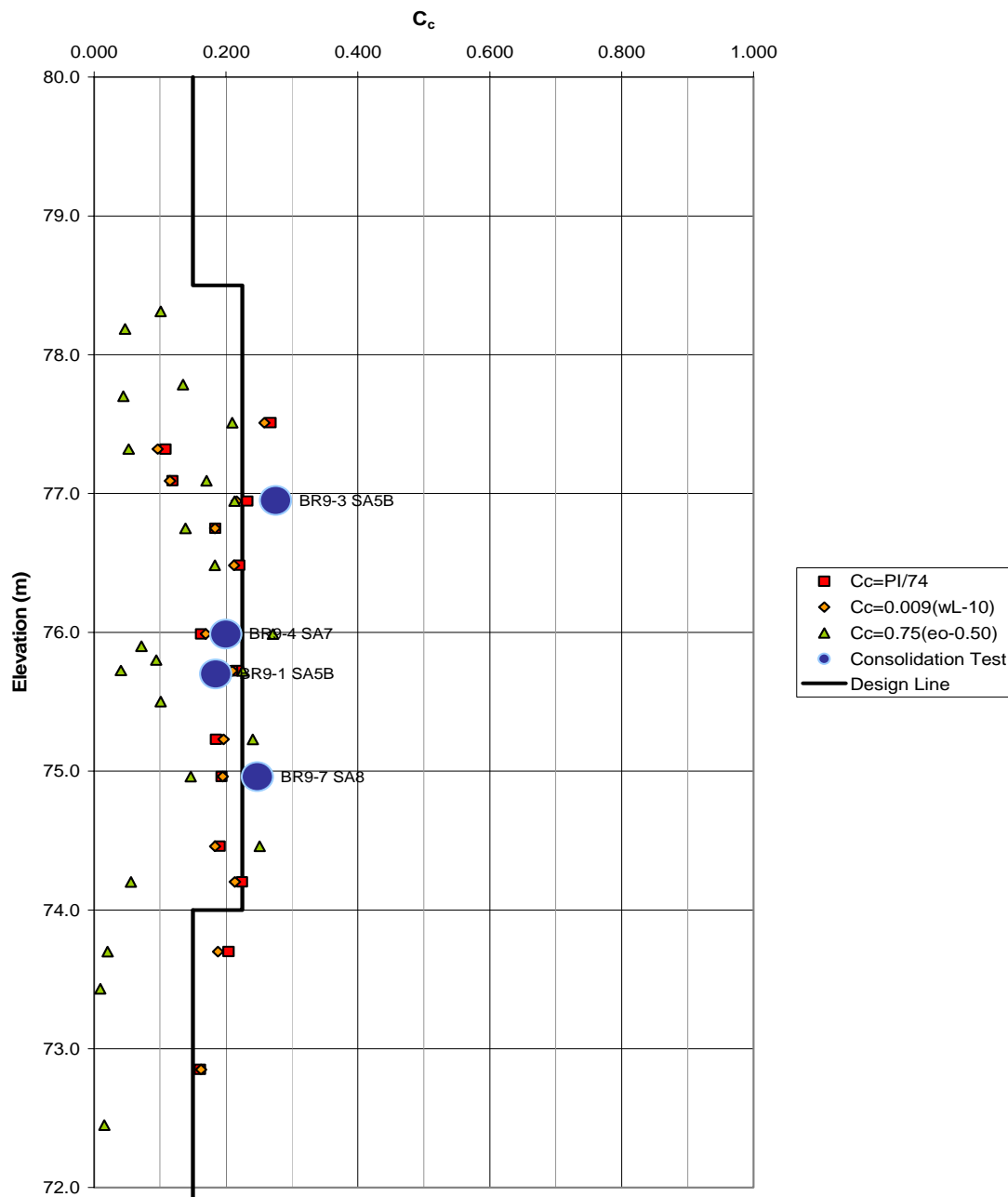
RECOMPRESSION INDEX AND DESIGN LINE QEW-RHCE BRIDGE 9

FIGURE 11



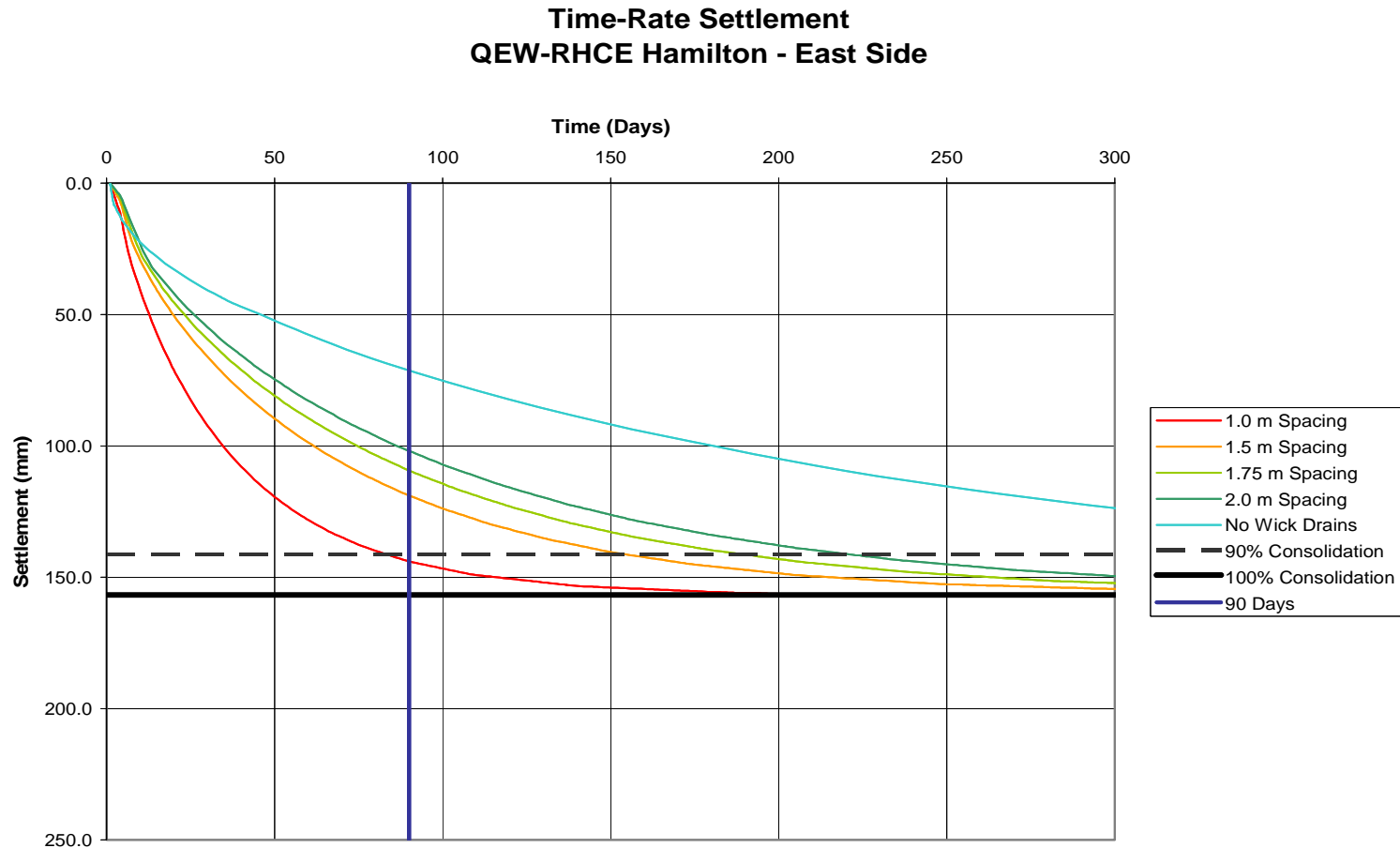
COMPRESSION INDEX AND DESIGN LINE QEW-RHCE BRIDGE 9

FIGURE 12



TIME-RATE SETTLEMENT FOR DIFFERENT WICK DRAIN SPACING
QEW-RHCE BRIDGE 9 - EAST EMBANKMENT
Primary Settlement Only

FIGURE 13



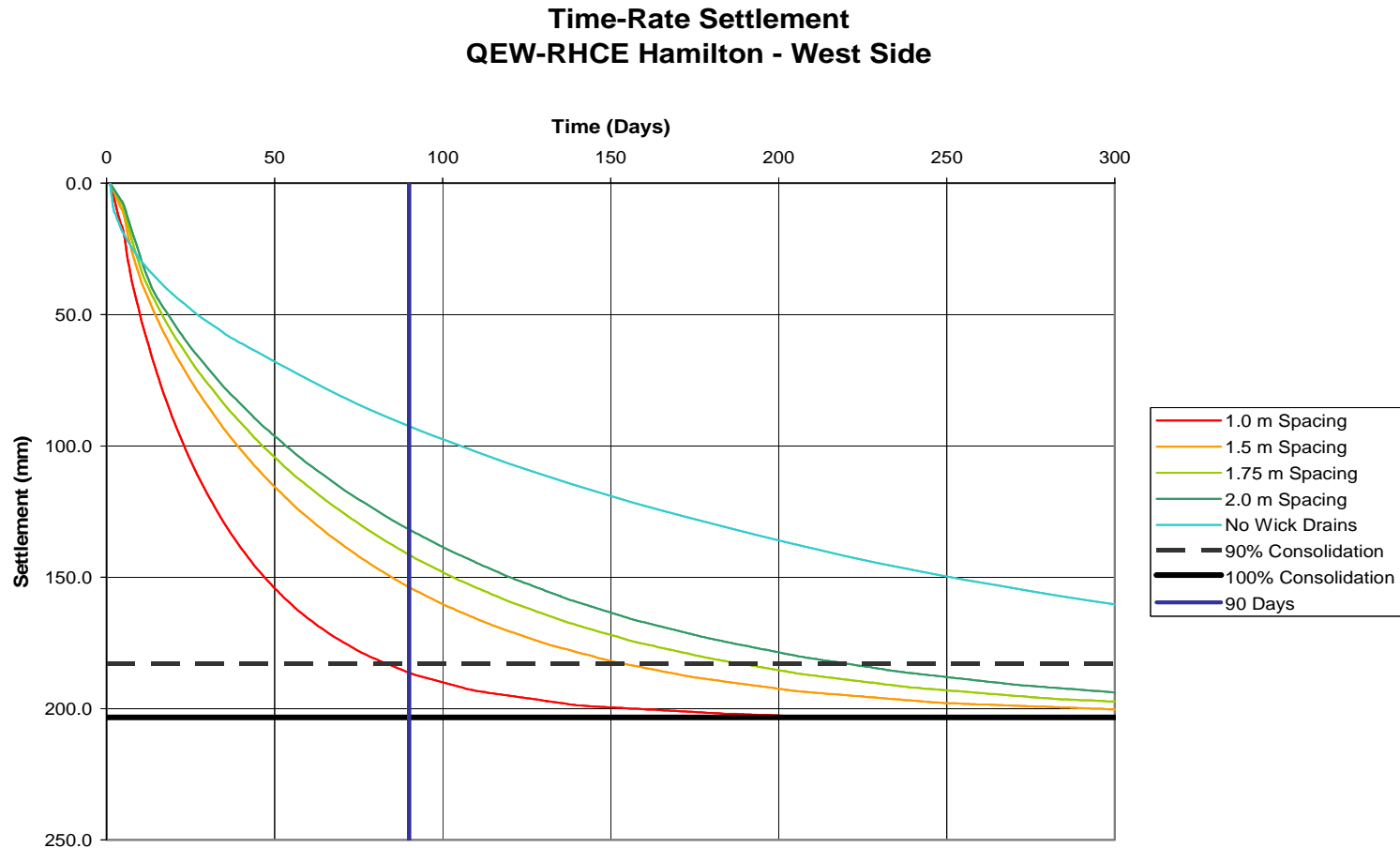
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TIME-RATE SETTLEMENT FOR DIFFERENT WICK DRAIN SPACING
QEW-RHCE BRIDGE 9 - WEST EMBANKMENT
Primary Settlement Only

FIGURE 14



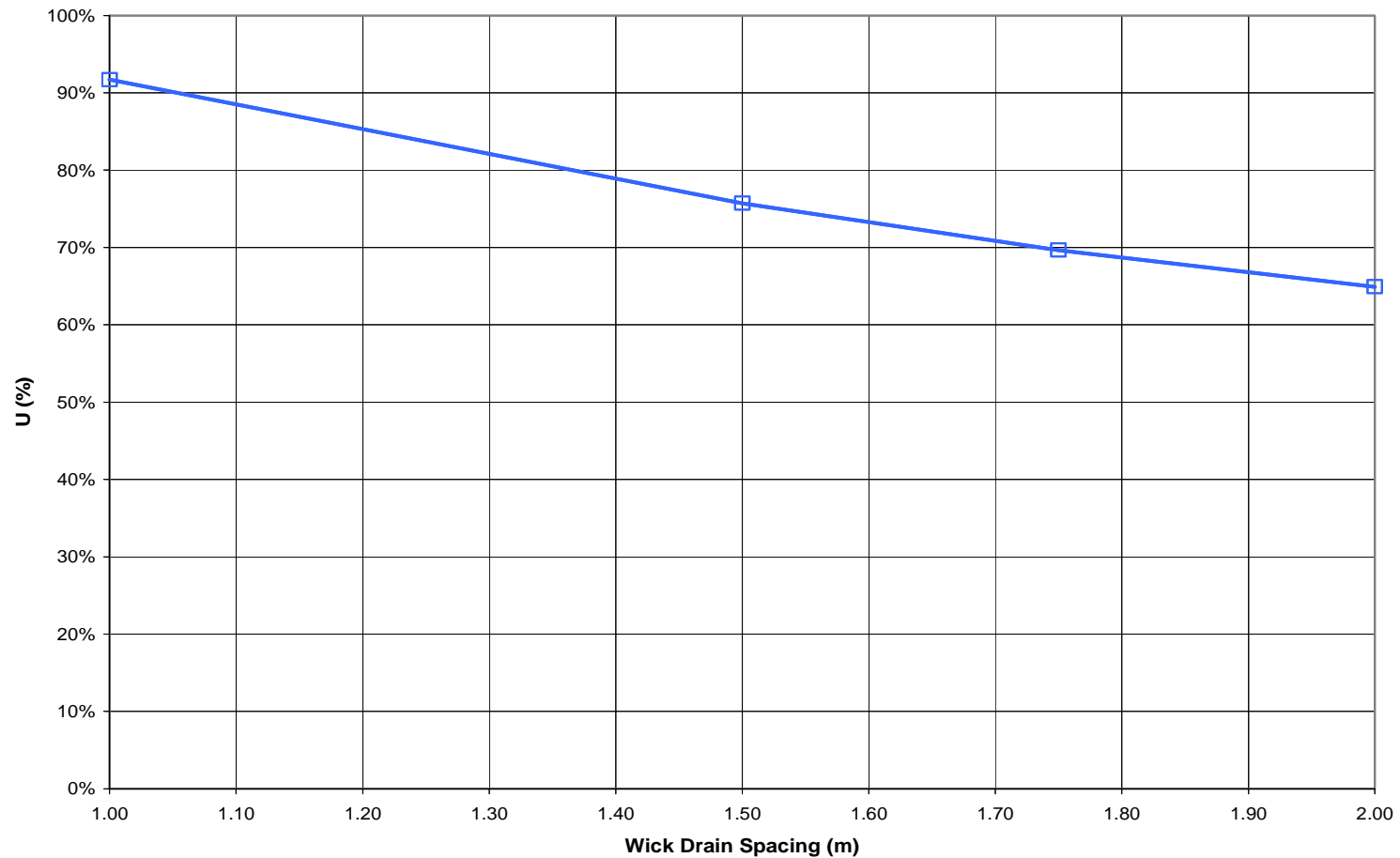
Date: July, 2004
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**PERCENT CONSOLIDATION IN 90 DAYS FOR DIFFERENT WICK DRAIN SPACING
QEW-RHCE BRIDGE 9 - EAST AND WEST EMBANKMENT**

FIGURE 15



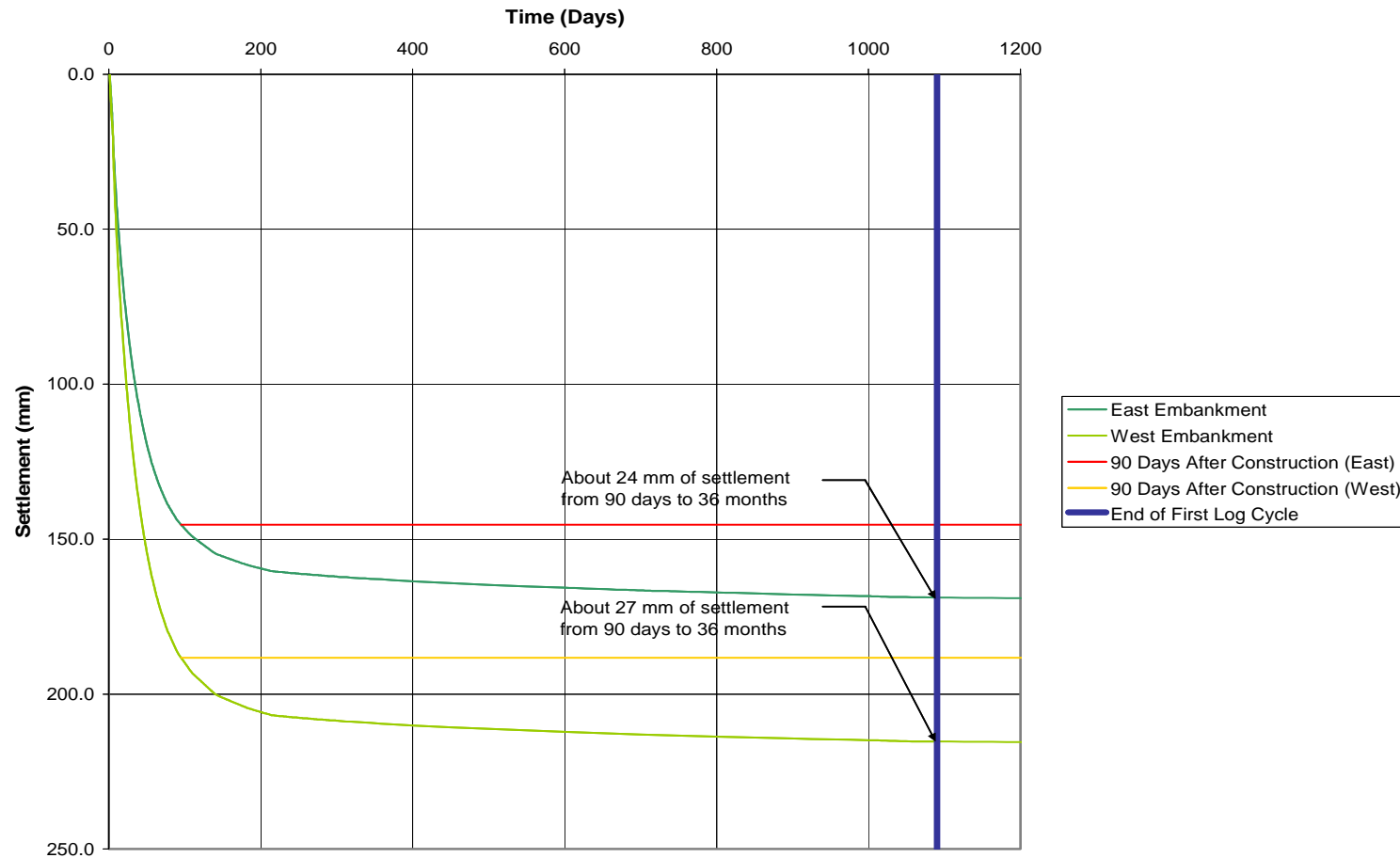
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TIME-RATE SETTLEMENT FOR 1.0 m WICK DRAIN SPACING
QEW-RHCE BRIDGE 9 - EAST AND WEST SIDE
Location: Centreline of Embankment - 5 m Behind Abutment
Primary and Secondary Settlement

FIGURE 16



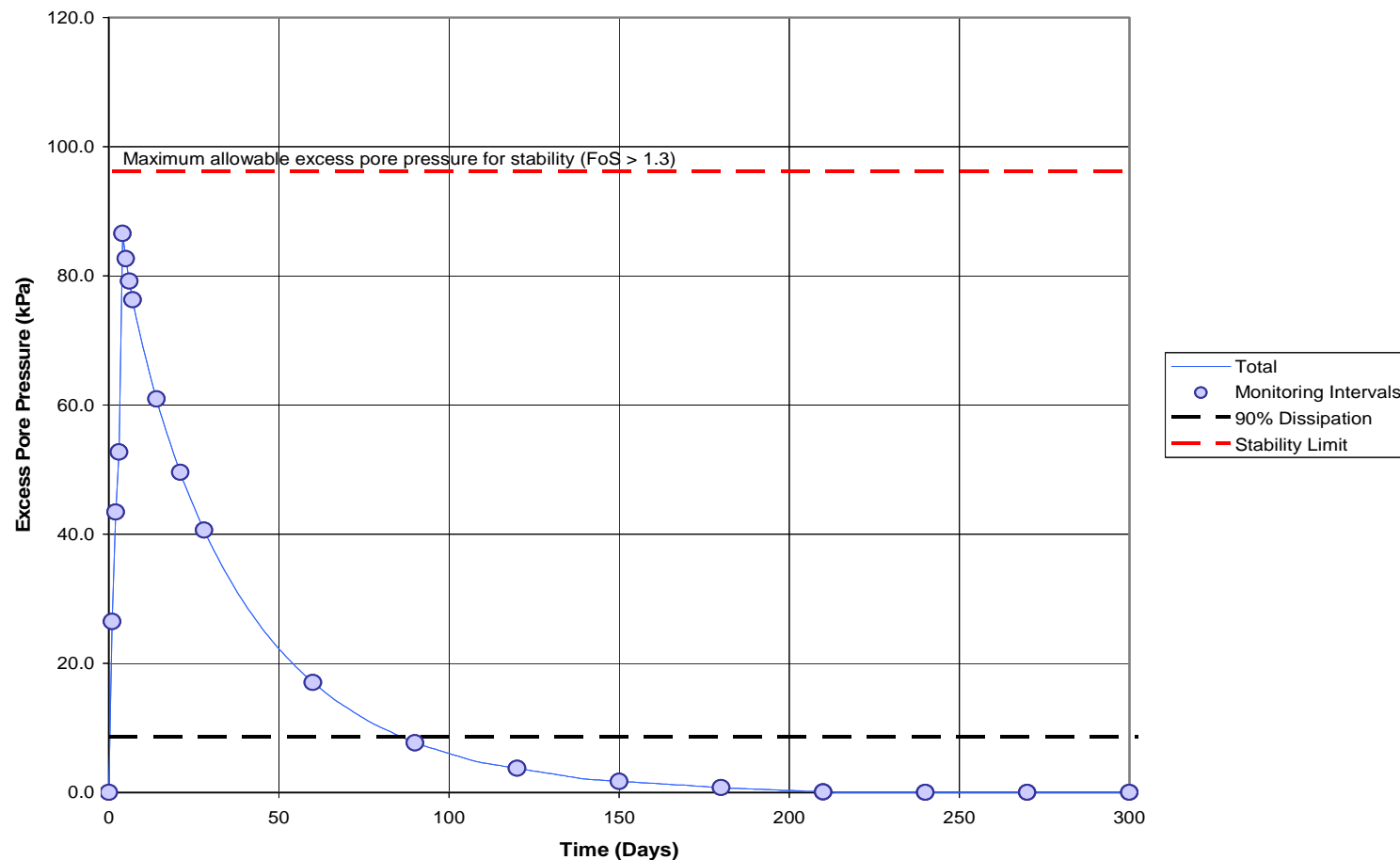
Date: July, 2004
Project: 021-1162

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EXCESS PORE PRESSURE RESPONSE
QEW-RHCE BRIDGE 9 - EAST EMBANKMENT
 Location: Centreline of Embankment - Elevation 76.2 m
 Construction Rate: 2 m / 24 hour Period

FIGURE 17



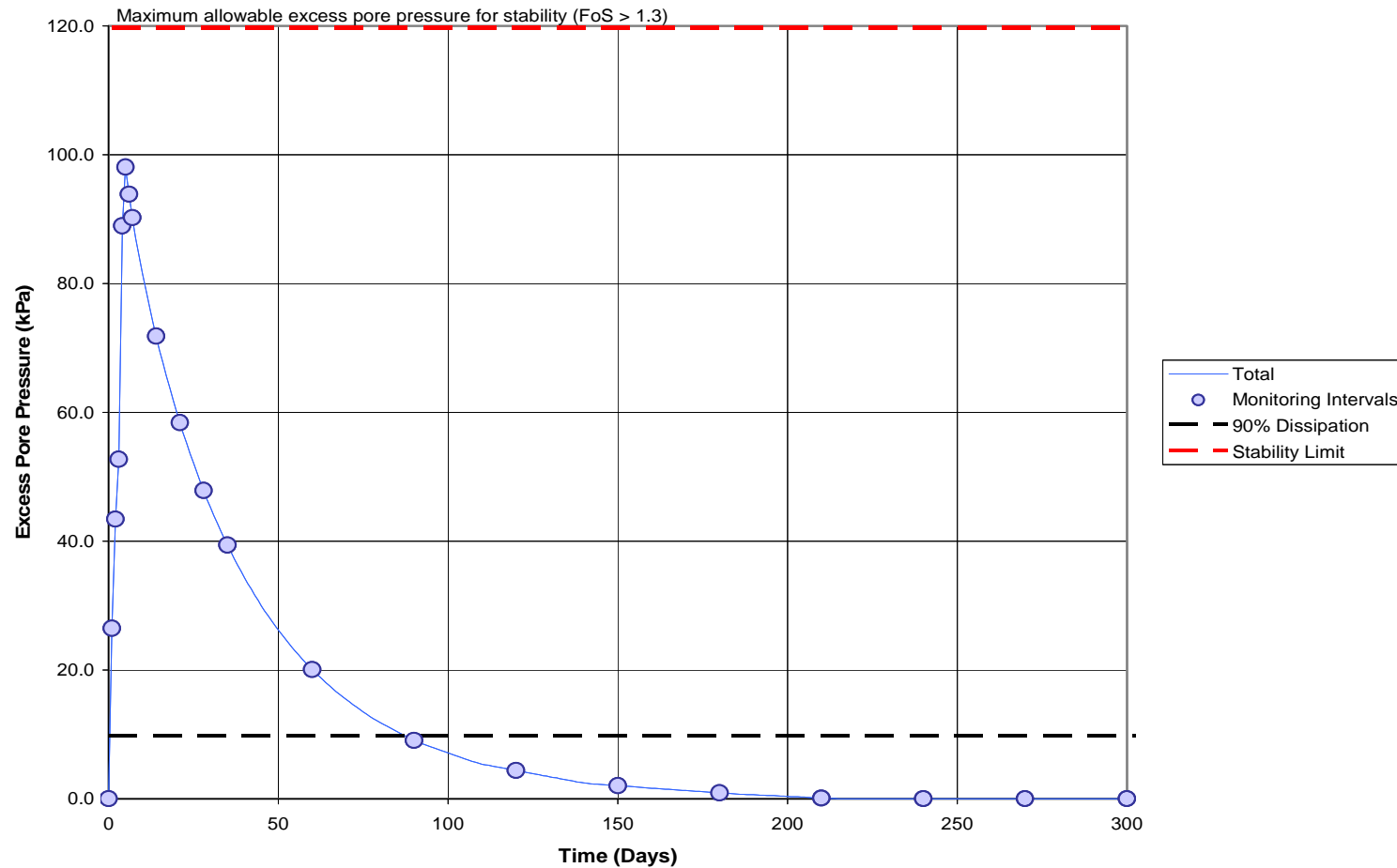
Date: July, 2004
 Project: 021-1162

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EXCESS PORE PRESSURE RESPONSE
QEW-RHCE BRIDGE 9 - WEST EMBANKMENT
Location: Centreline of Embankment - Elevation 76.2 m
Construction Rate: 2 m / 24 hour Period

FIGURE 18



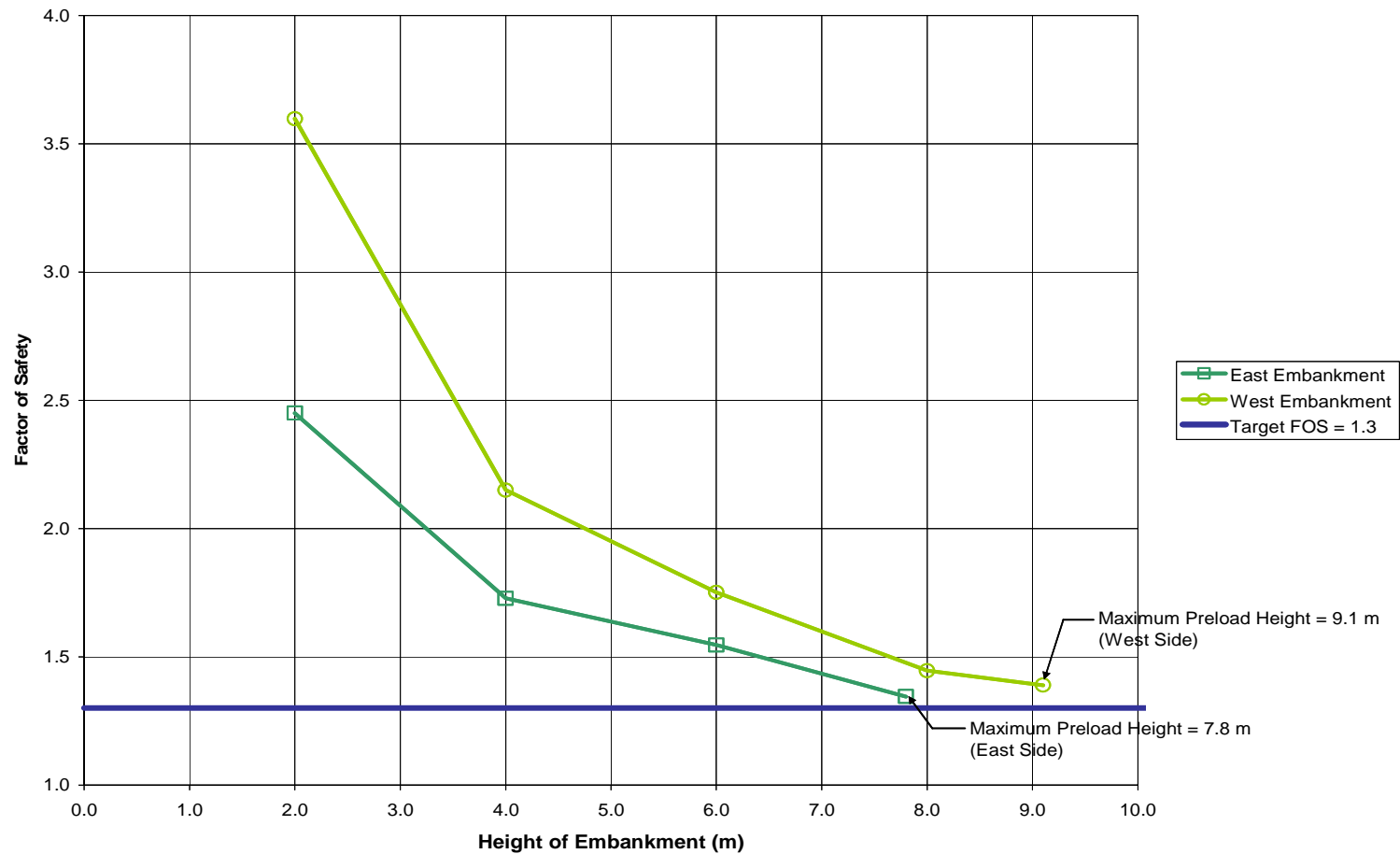
Date: July, 2004
Project: 021-1162

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**APPROACH STABILITY ANALYSIS - FACTOR OF SAFETY DURING CONSTRUCTION PERIOD
QEW-RHCE BRIDGE 9 - EAST AND WEST EMBANKMENT**

FIGURE 19



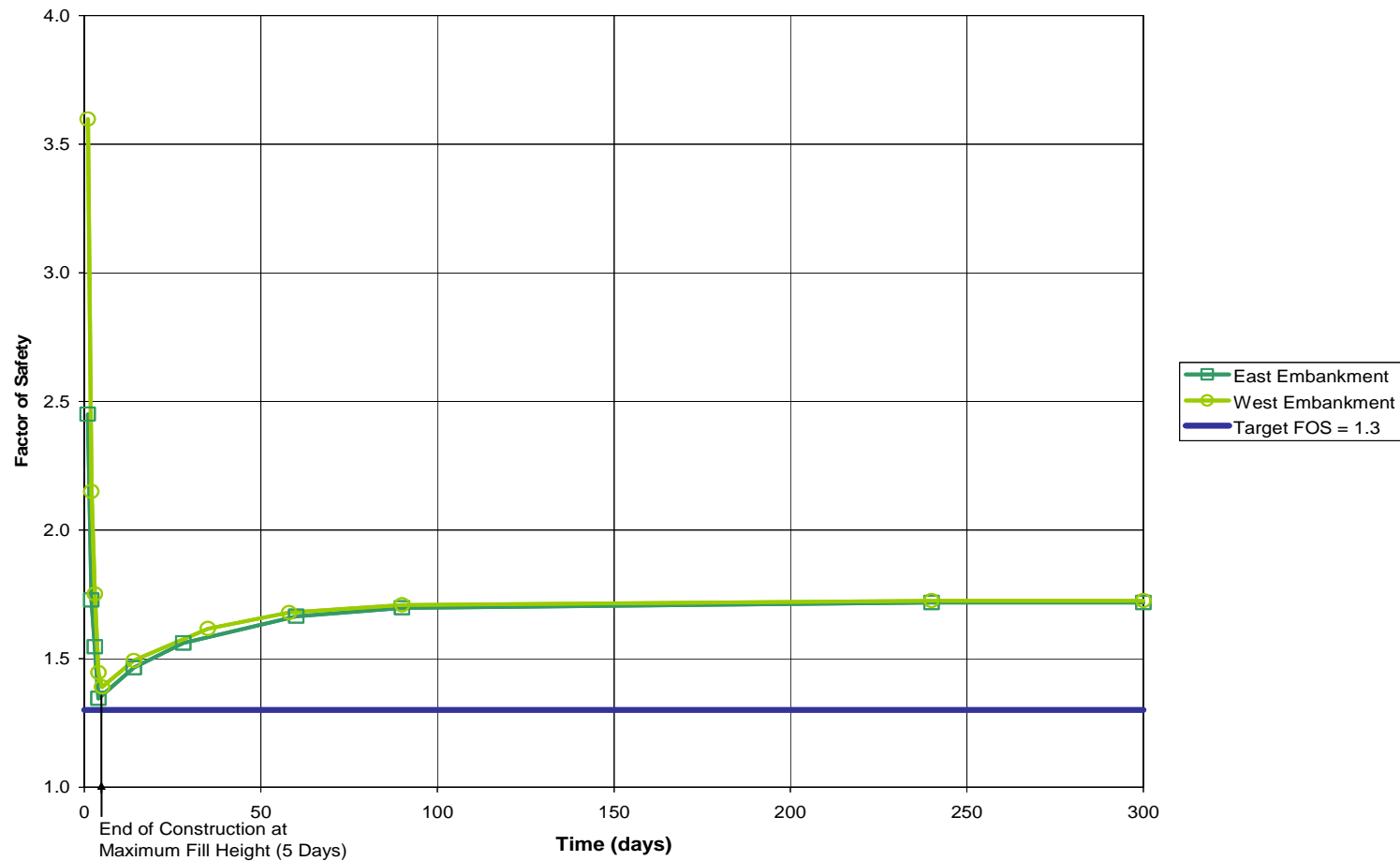
Date: July, 2004
Project: 021-1162

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APPROACH EMBANKMENT STABILITY ANALYSIS - FACTOR OF SAFETY WITH TIME
QEW-RHCE BRIDGE 9 - EAST AND WEST EMBANKMENT

FIGURE 20



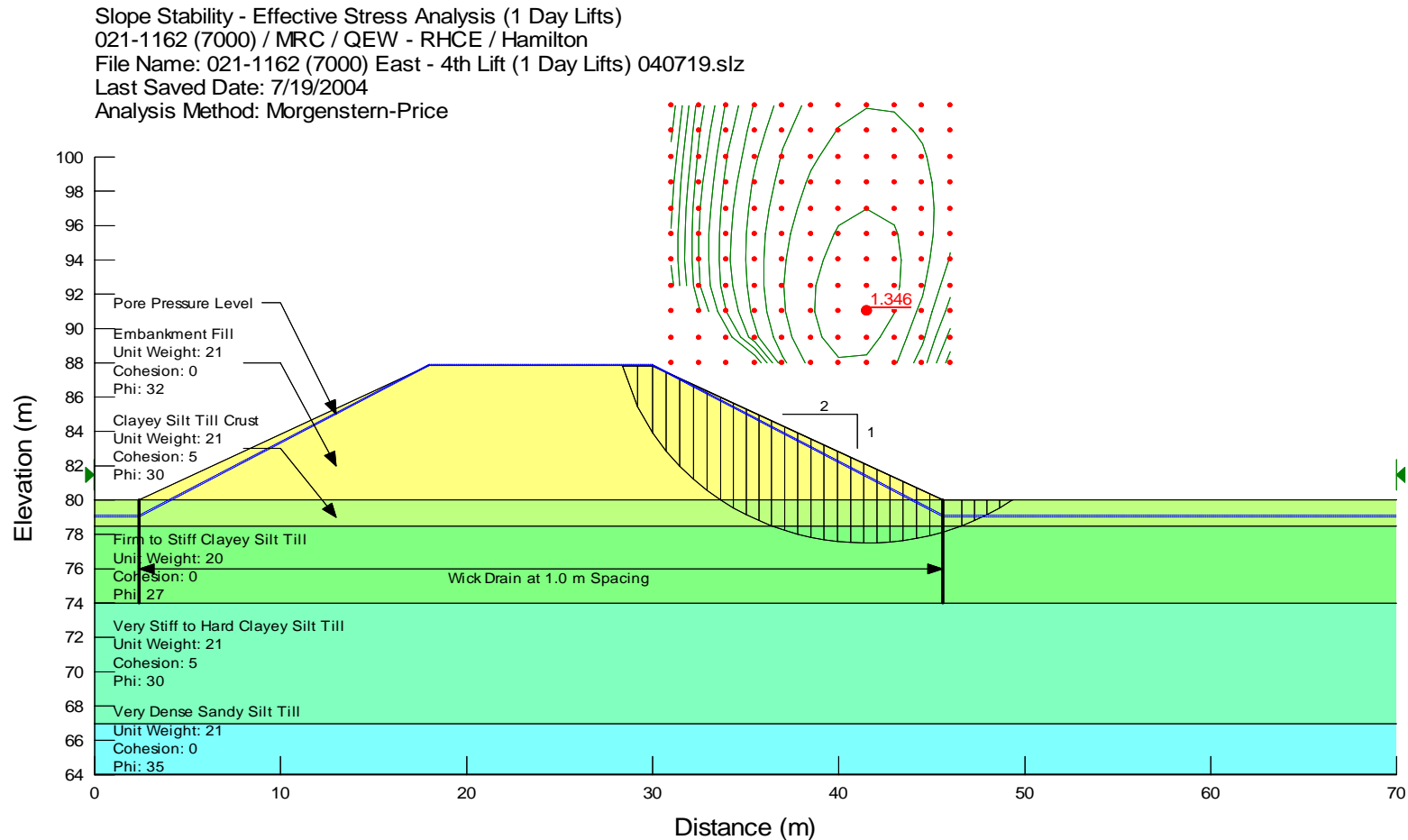
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APPROACH EMBANKMENT STABILITY ANALYSIS
QEW-RHCE BRIDGE 9 - EAST PRELOAD EMBANKMENT
AT COMPLETION OF PRELOAD CONSTRUCTION
Construction Rate: 2 m / 24 hour Period

FIGURE 21



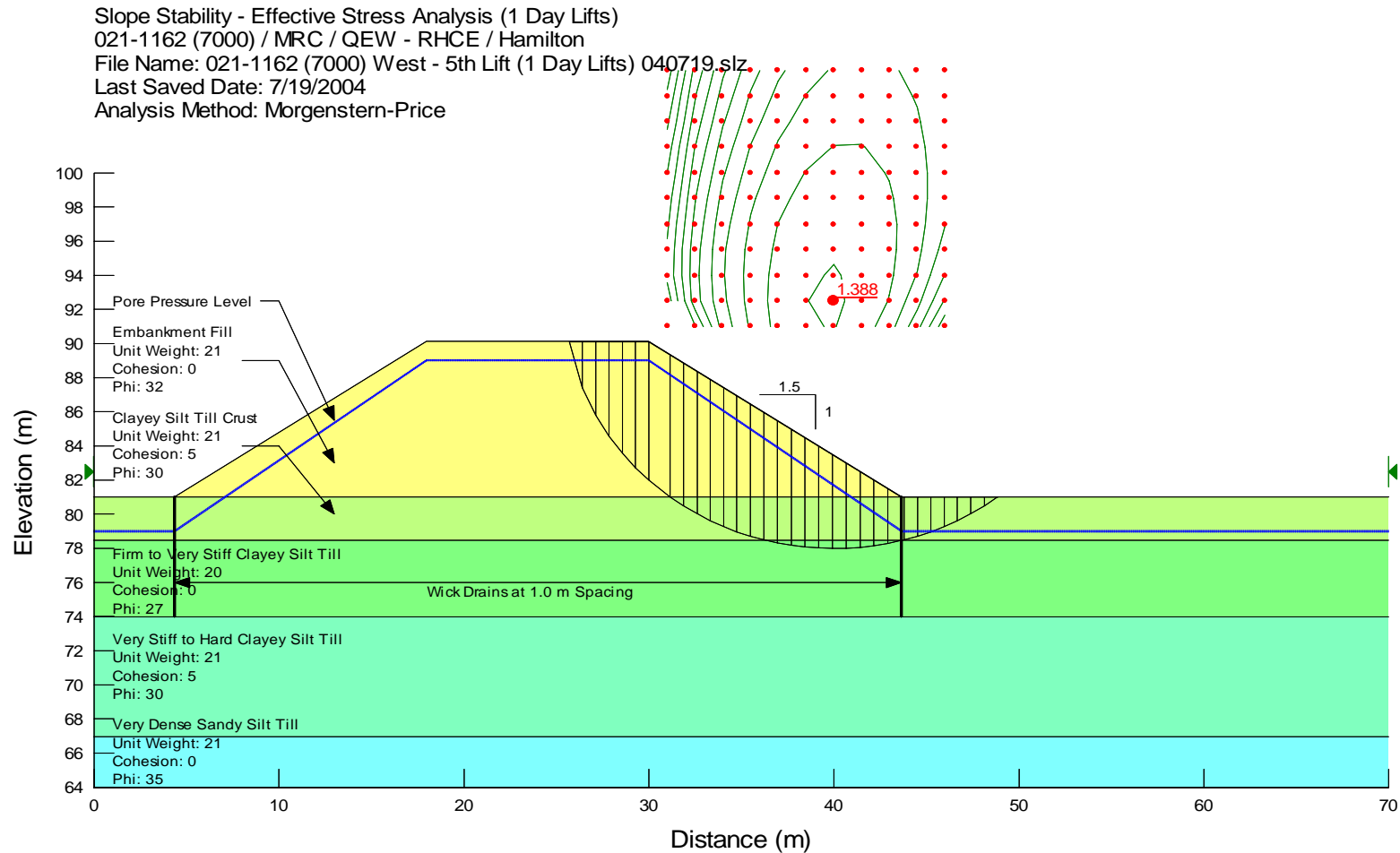
Date: July, 2004
 Project: 021-1162

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APPROACH EMBANKMENT STABILITY ANALYSIS
QEW-RHCE BRIDGE 9 - WEST PRELOAD EMBANKMENT
AT COMPLETION OF PRELOAD CONSTRUCTION
Construction Rate: 2 m / 24 hour Period

FIGURE 22



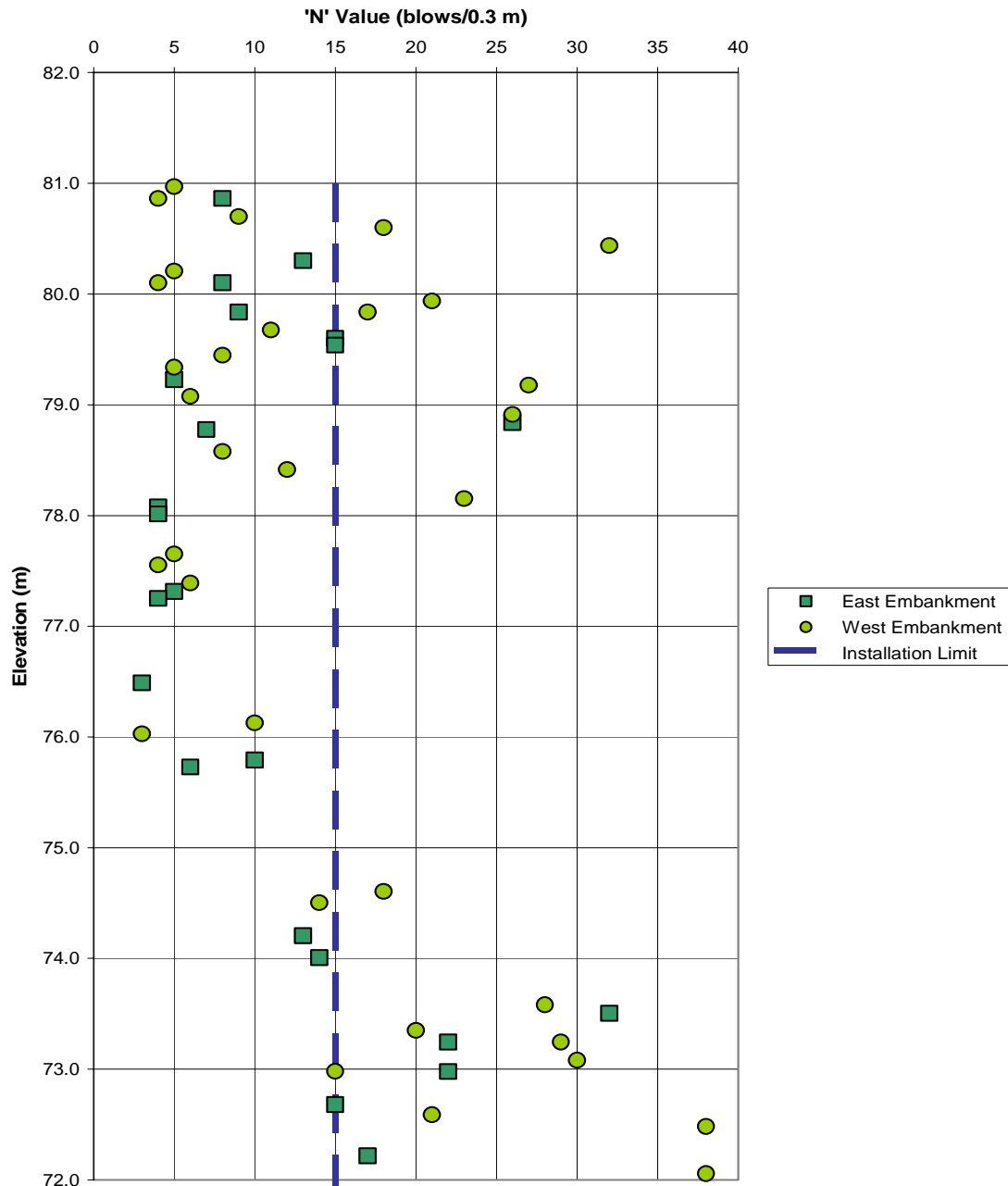
Date: July, 2004
 Project: 021-1162

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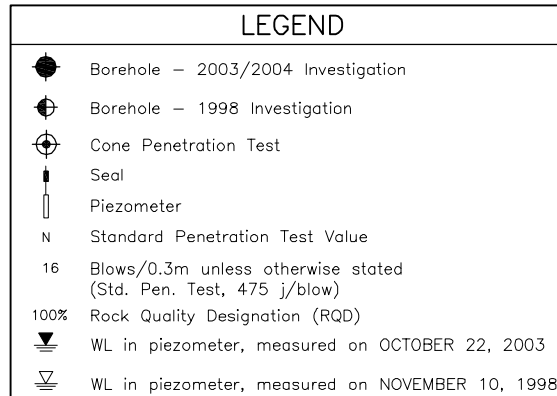
**'N' VALUES AND WICK DRAIN INSTALLATION LIMIT
QEW-RHCE BRIDGE 9**

FIGURE 23





 **Golder Associates**
MISSISSAUGA, ONTARIO, CANADA

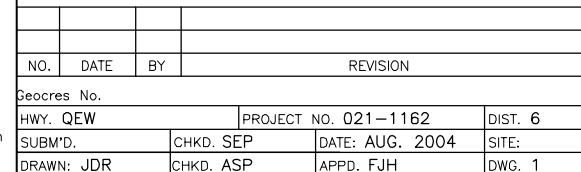


No.	ELEVATION	CO—ORDINATES	
		NORTHING	EASTING
9	79.6	4,789,705.7	283,693.0
BR9—1	80.6	4,789,752.7	283,642.4
BR9—2	80.7	4,789,726.1	283,658.2
BR9—3	80.3	4,789,681.8	283,711.0
BR9—4	80.7	4,789,756.5	283,612.0
BR9—5	80.7	4,789,749.9	283,616.5
BR9—6	80.3	4,789,692.0	283,698.0
BR9—7	80.0	4,789,696.5	283,702.9
BR9—8	80.8	4,789,763.9	283,606.9
BR9—9A	81.4	4,789,745.9	283,609.6
BR9—9B	81.4	4,789,746.7	283,609.1
CPT04—01	80.0	4,789,695.0	283,700.0
CPT04—02	80.0	4,789,688.3	283,713.0
CPT04—03	80.6	4,789,753.2	283,614.2
CPT04—04	80.6	4,789,765.1	283,606.0
RSER—5	81.2	4,789,776.4	283,576.1
RSER—6	81.2	4,789,746.7	283,610.9
RSER—7	80.7	4,789,659.9	283,737.3

General arrangement plans provided by McCormick Rankin Corporation,
ElectGA.dwg, dated July, 2004.

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.



LIST OF ABBREVIATIONS

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

I. SAMPLE TYPE

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

III. SOIL DESCRIPTION

(a) Cohesionless Soils

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

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

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

Consistency

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

(b) Cohesive Soils

c_u, s_u

Dynamic Cone Penetration Resistance; N_d :

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

PH: Sampler advanced by hydraulic pressure

PM: Sampler advanced by manual pressure

WH: Sampler advanced by static weight of hammer

WR: Sampler advanced by weight of sampler and rod

Piezo-Cone Penetration Test (CPT)

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

IV. SOIL TESTS

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

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

LIST OF SYMBOLS

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

I. General

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

II. STRESS AND STRAIN

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

III. SOIL PROPERTIES

(a) Index Properties

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

(a) Index Properties (continued)

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

(b) Hydraulic Properties

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

(c) Consolidation (one-dimensional)

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

(d) Shear Strength

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

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

LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

WEATHERING STATE

Fresh: no visible sign of weathering.

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

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

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

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

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

BEDDING THICKNESS

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

JOINT OR FOLIATION SPACING

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

GRAIN SIZE

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

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

CORE CONDITION

Total Core Recovery

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

Solid Core Recovery (SCR)

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

Rock Quality Designation (RQD)

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

DISCONTINUITY DATA

Fracture Index

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

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

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

Description and Notes

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

Abbreviations

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

PROJECT021-1162

W.P.441-97-00

DIST

DATUMGeodetic

LOCATIONN 4789752.7 ; E 283642.4

BOREHOLE TYPEPower Auger 108mm I.D. Hollow Stem Auger

DATEAugust 13, 2003

ORIGINATED BYPKS

COMPILED BYKG

CHECKED BYSEP

SOIL PROFILE

ELEV
DEPTH

DESCRIPTION

STRAT PLOT

SAMPLES

NUMBER

TYPE

"N" VALUES

GROUND WATER
CONDITIONS

ELEVATION SCALE

DYNAMIC CONE PENETRATION
RESISTANCE PLOT

20406080100

20406080100

○ UNCONFINED

● QUICK TRIAXIAL

+ FIELD VANE

× REMOULDED

20406080100

PLASTIC LIMIT

NATURAL MOISTURE CONTENT

LIQUID LIMIT

W_p

W

W_L

WATER CONTENT (%)

102030

UNIT WEIGHT

γ

kN/m³

REMARKS & GRAIN SIZE DISTRIBUTION (%)

GRSA SICSIL

80.6

GROUND SURFACE

80.1

Sandy Topsoil

79.8

Sandy SILT, some clay, trace to some gravel, trace organics

0.8

Compact Brown Moist (Fill)

79.1

Clayey SILT, some sand, trace gravel

1.5

Very Stiff Brown and grey Moist

Clayey SILT, some sand, trace gravel

Firm to stiff Grey Moist (Till)

74.5

Clayey SILT, some sand, trace gravel

6.1

Stiff to hard Grey to reddish-grey Moist (Till)

67.5

Clayey SILT, some sand and gravel, occasional shale and limestone fragments and boulders

13.1

Hard Reddish-brown Dry to moist (Till)

1

SS

18

2

SS

17

3

SS

6

4

SS

4

5

SS

3

6

SS

14

7

SS

15

8

SS

21

9

SS

29

10

SS

42

11

SS

00/0.18

1.7

2.8

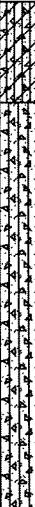
2.3

1.9

1.7

21.1

7314818








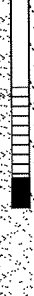


PROJECT 021-1162				RECORD OF BOREHOLE No BR9-1				2 OF 2				METRIC																		
W.P. 441-97-00				LOCATION N 4789752.7 ; E 283642.4				ORIGINATED BY PKS																						
DIST _____ HWY QEW				BOREHOLE TYPE Power Auger 108mm I.D. Hollow Stem Auger				COMPILED BY KG																						
DATUM Geodetic				DATE August 13, 2003				CHECKED BY SEP																						
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS			ELEVATION SCALE			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			WATER CONTENT (%)			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)						
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES																									
--- CONTINUED FROM PREVIOUS PAGE ---																														
64.6	Sandy SILT, some clay and gravel, shale and limestone pieces, occasional cobbles/boulders Very dense Red Moist to wet (Till)		12	SS	00/0.20																									
16.0																														
60.5			13	SS	00/0.18																									
20.1			14	SS	00/0.18																									
			15	SS	00/0.26																									
	End of Borehole																													
	Notes: 1. Open borehole dry upon completion of drilling 2. Borehole drilled 1 m west and shelly tube collected from 4.6 m to 5.2 m depth (Sample 5B)																													

PROJECT 021-1162			RECORD OF BOREHOLE No BR9-2			1 OF 2			METRIC				
W.P. 441-97-00			LOCATION N 4789726.1 ; E 283658.2			ORIGINATED BY PKS							
DIST _____ HWY QEW			BOREHOLE TYPE Power Auger 108mm I.D. Hollow Stem Auger			COMPILED BY KG							
DATUM Geodetic			DATE August 11, 2003			CHECKED BY SEP							
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT							
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x REMOULDED	PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
80.7	GROUND SURFACE												
80.9	Sandy Topsoil		1	SS	9								
79.9	Sandy SILT, some clay, trace to some gravel, trace organics, asphalt pieces												
0.8	Loose to compact Brown Moist (Fill)		2	SS	21								
	Clayey SILT, some sand, trace to some gravel		3	SS	27								
	Very stiff Brown to grey Moist		4	SS	12								
78.1	Clayey SILT, some sand, trace gravel												
2.6	Firm to stiff Grey Moist (Till)		5	SS	5								
			6	SS	10								
75.7	Clayey SILT, some sand, trace gravel												
5.0	Very stiff to hard Grey to reddish-grey Moist (Till)		7	SS	18								
			8	SS	30								
			9	SS	34								
			10	SS	34								
			11	SS	34								
			12	SS	137								
66.1	Shale and limestone pieces below 9.1 m depth												
14.6													

MISS_MTO_0211162EAMTO.GPJ ON MOT.GDT 16/7/04

Continued Next Page

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

PROJECT 021-1162			RECORD OF BOREHOLE No BR9-2			2 OF 2			METRIC										
W.P. 441-97-00			LOCATION N 4789726.1 ; E 283658.2			ORIGINATED BY PKS													
DIST _____ HWY QEW			BOREHOLE TYPE Power Auger 108mm I.D. Hollow Stem Auger			COMPILED BY KG													
DATUM Geodetic			DATE August 11, 2003			CHECKED BY SEP													
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x REMOULDED			WATER CONTENT (%) w _p — w — w _L			γ	GR SA SI CL				
--- CONTINUED FROM PREVIOUS PAGE ---																			
62.4	Clayey SILT, some sand and gravel, occasional shale and limestone fragments/cobbles Hard Reddish-brown Dry to moist (Till)		13	SS	00/0.1		65												
			14	SS	00/0.1		64												
			15	SS	00/0.1		63												
18.3	Sandy SILT, some clay and gravel, with shale and limestone pieces, occasional cobbles/boulders Very dense Red Moist to wet (Till)		16	SS	143		62												
			17	SS	00/0.1		61												
			18	SS	00/0.1		60												
59.1	Clayey SILT, some sand and gravel, with shale pieces/cobbles Hard Red Moist (Residual Soil)						59												
21.6							58												
							57												
56.6	Highly to moderately weathered, weak to strong calcareous SHALE BEDROCK (Queenston Formation) with fresh, strong to very strong limestone layers Bedrock cored from 24.1 m to 27.9 m depth For coring details see Record of Drillhole BR9-2						56												
24.1							55												
							54												
52.8	End of Borehole						53												
27.9																			

MISS_MTO_0211162EAMTO GPJ ON MOT.GDT 16/7/04

PROJECT: 021-1162

RECORD OF DRILLHOLE: BR9-2

SHEET 1 OF 1

LOCATION: N 4789726.1; E 283658.2

DRILLING DATE: August 11, 2003

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: Bomb CME 75

DRILLING CONTRACTOR: Geo-Environmental Drilling Ltd.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV.		RUN No.	PENETRATION RATE (mm/min)	FLUSH % RETURN	COLOUR	FR/FX-FRACTURE-F-FAULT CL-CLEAVAGE J-JOINT R-ROUGH SH-SHEAR P-POLISHED ST-STEPPED VN-VEIN S-SLICKENSIDED PL-PLANAR C-CURVED										SM-SMOOTH UE-UNEVEN W-WAVY C-CURVED			FL-FLEXURED MB-MECH. BREAK B-BEDDING			DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
				DEPTH (m)	INDEX					RECOVERY		R.Q.D. %	FRACT. INDEX PER 0.3	DISCONTINUITY DATA		HYDRAULIC CONDUCTIVITY K _f cm/sec											
										TOTAL CORE %	SOLID CORE %			TYPE AND SURFACE DESCRIPTION	10 ⁻⁶ 10 ⁻⁵ 10 ⁻⁴												
															80 90 100	80 90 100	5 10 15	0 10 20	0 10 20								
		Refer to previous page		56.60																							
		Moderately to highly weathered, thinly layered, reddish-grey to light grey, very fine grained, weak to medium strong, calcareous SHALE BEDROCK (Queenston Formation)		24.10	1		100																				
25		Occasional seams/layers of light grey medium strong to very strong, limestone/siltstone Run No. 1: 27% limestone Run No. 2: 23% limestone Run No. 3: 13% limestone			2		100																				
26	NQ CORING	All fractures are rough bedding																									
27					3		100																				
28		End of Drillhole		52.80 27.90																							
29																											
30																											
31																											
32																											
33																											
34																											

DEPTH SCALE

1 : 50



LOGGED: PS

CHECKED: SEP

MISS ROCK 0211162EARCH GPJ GLDR CAN GDT 3/8/04 TM

PROJECT 021-1162			RECORD OF BOREHOLE No BR9-3		1 OF 3		METRIC					
W.P. 441-97-00		LOCATION N 4789681.8 ; E 283711.0		ORIGINATED BY		PKS						
DIST _____ HWY QEW		BOREHOLE TYPE Power Auger 108mm I.D. Hollow Stem Auger		COMPILED BY		KG						
DATUM Geodetic		DATE August 7, 2003		CHECKED BY		SEP						
SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE		"N" VALUES	SHEAR STRENGTH kPa					
80.3	GROUND SURFACE											
80.0	Topsoil		1	SS	13							
79.9	Clayey SILT, some sand, trace to some gravel, trace organics		2	SS	15							
78.8	Stiff Brown Moist (Fill)											
78.8	Clayey SILT, some sand, trace gravel		3	SS	7							
78.8	Firm to stiff Grey Moist (Till)		4	SS	4							
78.8			5	SS	4							
78.8			6	SS	3							
78.8			7	SS	6							
75.3	Clayey SILT, some sand, trace gravel											
75.3	Stiff to hard Grey to reddish-grey Moist (Till)		8	SS	13							
75.3	Shale and limestone pieces below 6.1 m depth											
75.3			9	SS	15							
75.3			10	SS	21							
75.3			11	SS	17							
75.3			12	SS	41							
75.3			13	SS	67							

Continued Next Page

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

RECORD OF BOREHOLE No BR9-3

2 OF 3

METRIC

PROJECT 021-1162

W.P. 441-97-00

LOCATION N 4789681.8 ; E 283711.0

ORIGINATED BY PKS

DIST HWY QEW

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

COMPILED BY KG

DATUM Geodetic

DATE August 7, 2003

CHECKED BY SEP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		WATER CONTENT (%)								
								○ UNCONFINED + FIELD VANE	● QUICK TRIAXIAL × REMOULDED	w _p	w	w _L						
--- CONTINUED FROM PREVIOUS PAGE ---							20	40	60	80	100	10	20	30	GR	SA	SI	CL
	Clayey SILT, some sand, trace gravel Stiff to hard Grey to reddish-grey Moist (Till)		14	SS	68		65											
63.5							64											
16.8	Sandy SILT and GRAVEL with boulders, trace clay Red Wet (Till)		15	SS	107		63											
	NQ Coring carried out between 17.8 m and 21.3 m depth		16	RC			62											
	Limestone and siltstone cobbles and boulders throughout deposit based on core return		17	RC			61											
			18	RC			60											
			19	SS	000.06		59											
							58											
	NQ Coring carried out between 22.6 m and 25.9 m depth		1	RC			57											
	Granitic and dioritic boulders within deposit below 22 m based on core return		2	RC			56											
			3	RC			55											
54.4							54											
25.9	Highly weathered, red SHALE BEDROCK (Queenston Formation) with occasional limestone layers NQ coring from 22.6 m to 31.1 m For coring details see Record of Drillhole BR9-3						53											
							52											
							51											

Continued Next Page

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

MISS_MTO_0211162EAMTO.GPJ ON MOT.GDT 16/7/04

PROJECT <u>021-1162</u>		RECORD OF BOREHOLE No BR9-3		3 OF 3	METRIC
W.P. <u>441-97-00</u>	LOCATION <u>N 4789681.8 ; E 283711.0</u>	ORIGINATED BY <u>PKS</u>			
DIST <u>HWY QEW</u>	BOREHOLE TYPE <u>Power Auger 108mm I.D. Hollow Stem Auger</u>	COMPILED BY <u>KG</u>			
DATUM <u>Geodetic</u>	DATE <u>August 7, 2003</u>	CHECKED BY <u>SEP</u>			

SOIL PROFILE		SAMPLES					DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa		W _p	W			W _L
	--- CONTINUED FROM PREVIOUS PAGE ---							20 40 60 80 100	○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED	WATER CONTENT (%)				
							50			10 20 30				
49.2 31.1	End of Borehole Notes: 1. Auger and spoon refusal at 22.6 m depth (Elev. 57.6 m) 2. Borehole drilled 1 m south and shelby tube sample collected from 3.1 m to 3.7 m depth (Sample 5B) 3. Base of borehole wet upon completion of drilling and before coring operations 4. Water level in hole at 25 m depth (Elev. 55.3 m) on August 8, 2003 (24 hours after drilling operations)													

PROJECT: 021-1162

RECORD OF DRILLHOLE: BR9-3

SHEET 1 OF 1

LOCATION: N 4789681.8; E 283711.0

DRILLING DATE: August 7, 2003

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: Bomb CME 75

DRILLING CONTRACTOR: Geo-Environmental Drilling Ltd.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (m/min)	FLUSH % RETURN	FR/FX-FRACTURE F-FAULT CL-CLEAVAGE J-JOINT R-SMOOTH SH-SHEAR P-POLISHED R-ROUGH VN-VEIN S-SLICKENSIDED PL-PLANAR C-CURVED										DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	HYDRAULIC CONDUCTIVITY K _f cm/sec			DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
								RECOVERY		R.Q.D. %	FRACT. INDEX PER 0.3	DIP w.r.t. CORE AXIS	10 ⁻⁶	10 ⁻⁵	10 ⁻⁴	10 ⁻³							
								TOTAL CORE %	SOLID CORE %														
								80 80 80 80	80 80 80 80														
		Refer to previous page		56.80 23.50																			
24		Sandy SILT and GRAVEL, trace clay (TILL) with boulders of diorite, granite, limestone and shale throughout Note: Due to poor core recovery, the deposit between 25.9 m and 31.1 m depth may be an extension of the overlying till with the majority comprised of shale boulders. The following describes the core samples as retrieved: Highly weathered, very fine layered, reddish-grey, very fine grained, very weak to weak, calcareous SHALE BEDROCK (Queenston Formation) Light grey, slightly weathered, medium strong to strong, limestone/siltstone layers at 26.4 m depth (50 mm thick) and 29.3 m depth (300 mm thick)																					
25																							
26																							
27																							
28																							
29																							
30																							
31		End of Drillhole		49.20 31.10																			
32																							
33																							

DEPTH SCALE

1 : 50



LOGGED: PS

CHECKED: SEP

MISS ROCK 0211162EARCKGPJ GLDR CAN.GDT 3/8/04 TM

PROJECT <u>021-1162</u>				RECORD OF BOREHOLE No BR9-4				1 OF 1		METRIC			
W.P. <u>441-97-00</u>		LOCATION <u>N 4789756.5 ; E 283612.0</u>				ORIGINATED BY <u>CN</u>							
DIST <u> </u> HWY <u>QEW</u>		BOREHOLE TYPE <u>Power Auger 108mm I.D. Hollow Stem Auger</u>				COMPILED BY <u>JPD</u>							
DATUM <u>Geodetic</u>		DATE <u>May 28, 2004</u>				CHECKED BY <u>FJH</u>							
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT		UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		WATER CONTENT (%)			
80.9 0.0	GROUND SURFACE												
80.0 0.1	TOPSOIL												
	Silty SAND, some gravel, trace clay and rootlets (FILL) Loose Brown Moist to wet		1	SS	4								
			2	SS	4								
79.4 1.5	Clayey SILT, trace to some sand, trace gravel (TILL) Firm becoming stiff Brown becoming grey at Elev. 78.6 m Moist		3	SS	5								
			4	SS	8								
			5	TO	PH								
			6	TO	PH								
			7	TO	PH								
			8	TO	PH								
			9	TO	PH								
			10	TO	PH								
73.3 7.6	Clayey SILT, some sand, trace gravel and shale fragments (TILL) Very stiff to hard Grey Moist		11	SS	29								
			12	SS	38								
71.9 9.0	End of Borehole												

MISS_MTO_0211162EAMTO.GPJ ON_MOT.GDT 12/7/04

MISS MTO 0211162EAMTO.GPJ ON MOT.GDT 13/7/04

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

PROJECT <u>021-1162</u>		RECORD OF BOREHOLE No BR9-6		1 OF 1	METRIC
W.P. <u>441-97-00</u>		LOCATION <u>N 4789692.0 ; E 283698.0</u>		ORIGINATED BY <u>CN</u>	
DIST <u> </u> HWY <u>QEW</u>		BOREHOLE TYPE <u>Power Auger 108mm I.D. Hollow Stem Auger</u>		COMPILED BY <u>JPD</u>	
DATUM <u>Geodetic</u>		DATE <u>May 27, 2004</u>		CHECKED BY <u>FJH</u>	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	W _p	W	W _L		
80.3	GROUND SURFACE													
80.1	TOPSOIL													
79.3	Silty SAND, trace clay and rootlets (FILL) Loose Brown Moist		1	SS	8		80							
1.1	Clayey SILT, some sand, trace gravel (TILL) Stiff to very stiff Grey Moist		2	SS	8		78							
							77							
							76							
							75							
							74							
			3	SS	14		73							
72.7	Clayey SILT, some sand, trace gravel and shale fragments(TILL) Very stiff Grey Moist		4	SS	22									
72.1														
8.2	End of Borehole													

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

PROJECT 021-1162			RECORD OF BOREHOLE No BR9-7			1 OF 1		METRIC							
W.P. 441-97-00			LOCATION N 4789695.0 ; E 283700.0			ORIGINATED BY CN									
DIST _____ HWY QEW			BOREHOLE TYPE Power Auger 108mm I.D. Hollow Stem Auger			COMPILED BY JPD									
DATUM Geodetic			DATE May 27, 2004			CHECKED BY FJH									
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							
80.0	GROUND SURFACE														
8.0	TOPSOIL														
0.1	Silty SAND, trace clay and rootlets (FILL)		1	SS	9										
79.4	Loose Brown Moist		2	SS	5										
0.6	Clayey SILT, some sand, trace gravel (TILL)		3	TO	PH										
	Firm to very stiff Grey Moist		4	TO	PH										
			5	TO	PH										
			6	TO	PH										
			7	TO	PH										
			8	TO	PH										
			9	TO	PH										
			10	TO	PH										
73.2	Clayey SILT, some sand, trace gravel and shale fragments (TILL)		11	SS	22										
6.9	Very stiff Grey Moist		12	SS	17										
71.8	End of Borehole														
8.2															

MISS MTO 0211162EAMTO.GPJ ON MOT.GDT 13/7/04

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

PROJECT <u>021-1162</u>				RECORD OF BOREHOLE No BR9-9A				1 OF 1		METRIC								
W.P. <u>441-97-00</u>		LOCATION <u>N 4789745.9 ; E 283609.6</u>				ORIGINATED BY <u>CN</u>												
DIST <u> </u> HWY <u>QEW</u>		BOREHOLE TYPE <u>Power Auger 108mm I.D. Hollow Stem Auger</u>				COMPILED BY <u>JPD</u>												
DATUM <u>Geodetic</u>		DATE <u>June 2, 2004</u>				CHECKED BY <u>FJH</u>												
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT W _p W W _L			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE		"N" VALUES	SHEAR STRENGTH kPa					WATER CONTENT (%)						
81.4 0.0	GROUND SURFACE						<div style="display: flex; justify-content: space-between;"> 20 40 60 80 100 20 40 60 80 100 </div> <div style="display: flex; justify-content: space-between;"> 20 40 60 80 100 10 20 30 </div>											
	Auger to 3.1 m depth (Elev. 78.3 m) Perform in situ vane testing starting at 3.5 m depth For stratigraphy see Record of Borehole RSER-6					81												
						80												
						79												
						78												
						77												
						76												
						75												
74.1 7.3	End of Borehole																	

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

PROJECT <u>021-1162</u>				RECORD OF BOREHOLE No BR9-9B				1 OF 1		METRIC							
W.P. <u>441-97-00</u>		LOCATION <u>N 4789746.7 :E 283609.1</u>				ORIGINATED BY <u>CN</u>											
DIST <u> </u> HWY <u>QEW</u>		BOREHOLE TYPE <u>Power Auger 108mm I.D. Hollow Stem Auger</u>				COMPILED BY <u>JPD</u>											
DATUM <u>Geodetic</u>		DATE <u>June 2, 2004</u>				CHECKED BY <u>FJH</u>											
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES		ELEVATION SCALE	SHEAR STRENGTH kPa									WATER CONTENT (%)
81.4 0.0	GROUND SURFACE						20 40 60 80 100	○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					20 40 60 80 100 10 20 30				
	Auger to 3.1 m depth (Elev. 78.3 m) Perform in situ vane testing starting at 3.5 m depth For stratigraphy see Record of Borehole RSER-6						81										
							80										
							79										
							78										
							77										
							76										
							75										
74.1 7.3	End of Borehole																

MISS_MTO 0211162EAMTO.GPJ ON_MOT.GDT 16/7/04

RECORD OF BOREHOLE No 9

1 OF 2

METRIC

PROJECT 021-1162

W.P. 441-97-00

LOCATION N 4789705.7 ; E 283693.0

ORIGINATED BY GD

DIST HWY QEW

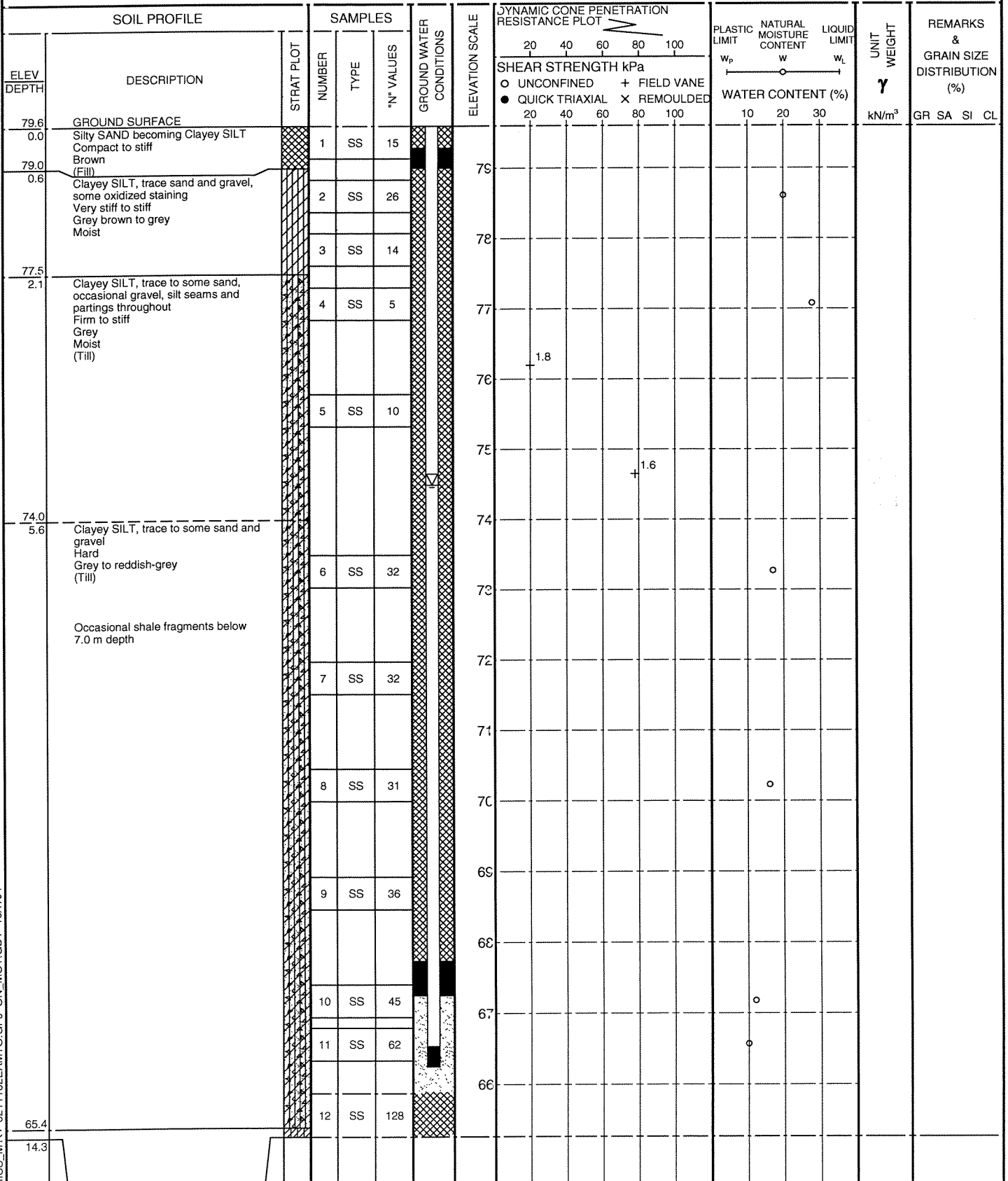
BOREHOLE TYPE Power Auger 108mm I.D. Hollow Stem Auger

COMPILED BY KG

DATUM Geodetic

DATE February 25, 1998

CHECKED BY SEP



Continued Next Page

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

PROJECT <u>021-1162</u>				RECORD OF BOREHOLE No 9				2 OF 2		METRIC				
W.P. <u>441-97-00</u>		LOCATION <u>N 4789705.7 ; E 283693.0</u>		ORIGINATED BY <u>GD</u>										
DIST <u> </u> HWY <u>QEW</u>		BOREHOLE TYPE <u>Power Auger 108mm I.D. Hollow Stem Auger</u>				COMPILED BY <u>KG</u>								
DATUM <u>Geodetic</u>		DATE <u>February 25, 1998</u>				CHECKED BY <u>SEP</u>								
SOIL PROFILE		SAMPLES				DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT		UNIT WEIGHT		REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL X REMOULDED		WATER CONTENT (%) W _p W W _L		γ	GR SA SI CL	
	--- CONTINUED FROM PREVIOUS PAGE --- Clayey SILT, some sand and gravel, with highly weathered shale fragments Hard Reddish-brown (Till) End of Borehole Notes: 1. Borehole dry upon completion of drilling 2. Water level in piezometer at Elev. 67.8 m on March 25, 1998 3. Water level in piezometer at Elev. 74.1 m on August 22 and October 22, 2003.													

PROJECT 021-1162				RECORD OF BOREHOLE No RSER-6		1 OF 1		METRIC					
W.P. 441-97-00		LOCATION N 4789746.7 E 283610.9		ORIGINATED BY PKS									
DIST HWY QEW		BOREHOLE TYPE Power Auger 108mm I.D. Hollow Stem Auger		COMPILED BY KG									
DATUM Geodetic		DATE September 23, 1998		CHECKED BY SEP									
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa					
81.2	GROUND SURFACE												
8.9	Topsoil												
	Clayey SILT, some sand, trace gravel, some organics and oxidized stains												
	Hard to stiff												
	Brown to reddish-brown												
	Dry to moist												
			1	SS	32								
			2	SS	11								
			3	SS	26								
79.1	Clayey SILT, trace to some sand, trace gravel, occasional silt seams												
2.1	Very stiff												
	Brown and grey												
	Dry to moist												
			4	SS	23								
77.6	Clayey SILT, trace to some sand, occasional gravel, occasional silty clay zones												
3.7	Soft to firm												
	Grey												
	Moist to wet (Till)												
			5	SS	6								
			6	TO	PH								
			7	TO	PH								
			8	SS	28								
			9	SS	38								
			10	SS	39								
74.5	Clayey SILT, some sand, trace gravel, occasional weathered shale fragments												
6.7	Very stiff to hard												
	Grey to reddish-grey												
	Moist (Till)												
			8	SS	28								
			9	SS	38								
			10	SS	39								
70.1	End of Borehole												
11.1	Notes:												
	1. Water level in open hole at 4.6 m depth upon completion of drilling												
	2. Water level in piezometer at Elev. 78.1 m on October 19, 1998												
	3. Water level in piezometer at Elev. 78.7 m on November 10, 1998												

MISS MTO 0211162EAMTO.GPJ ON MOT GDT 13/7/04

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

RECORD OF BOREHOLE No RSER-7

1 OF 1

METRIC

PROJECT 021-1162

W.P. 441-97-00

LOCATION N 4789659.9 ; E 283737.3

ORIGINATED BY PKS

DIST HWY QEW

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

COMPILED BY KG

DATUM Geodetic

DATE September 20, 1998

CHECKED BY SEP

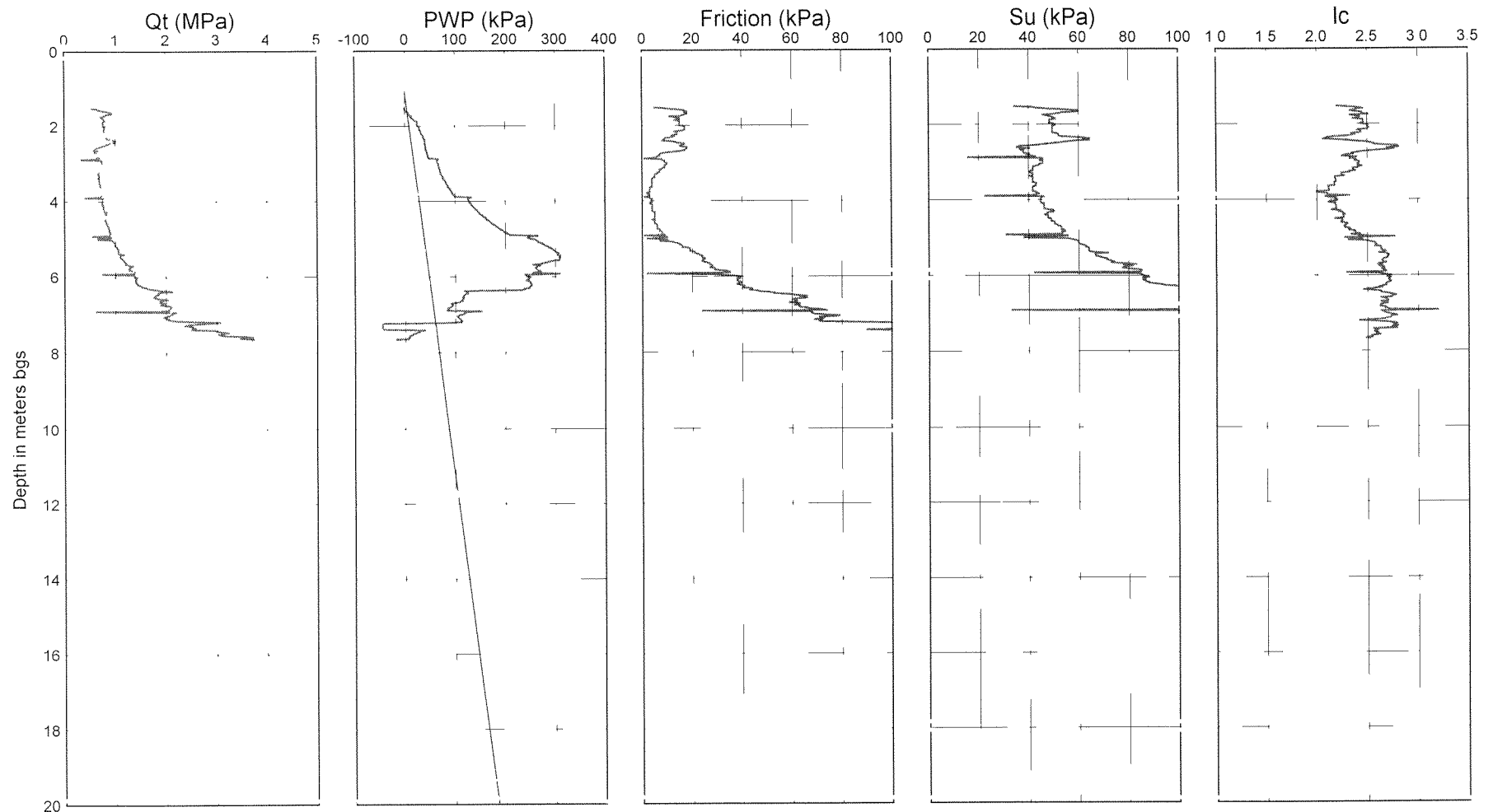
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		WATER CONTENT (%)				
80.7	GROUND SURFACE							20 40 60 80 100		10 20 30				
0.0	Topsoil		1	SS	33									
0.2	Sandy SILT, some clay, trace to some gravel, some sand seams with oxidized stains Brown Dense becoming compact Dry to moist (Fill)		2	SS	14									
79.3														
1.4	Clayey SILT, some sand, trace gravel, trace organics and oxidized stains Mottled grey and brown Stiff Moist		3	SS	13									
78.4														
2.3	Clayey SILT, some sand, occasional gravel (TILL) Firm Grey Moist		4	SS	7									
			5	SS	8									
77.1														
3.7	Clayey SILT, some sand, trace gravel, occasional shale (TILL) Grey to reddish-grey Firm to very stiff Moist		6	SS	7									
			7	SS	7									
			8	SS	11									
			9	SS	16									
			10	SS	29									
73.4	End of Borehole													
7.3	Notes: 1. Open hole dry upon completion of drilling 2. Piezometer dry on October 19, 1998 and on November 10, 1998 3. Water level in piezometer at Elev. 77.4 m on October 22, 2003													

Cone Penetration Test - CPT04-01C

Test Date : May 31, 2004
Location :

Operator : Golder Associates

Ground Surf. Elev. : 80.00
Water Table Depth : 1.10



Qt normalized for
tip area effects

Su = $(Q_t - \sigma_{vm}) / N_k$
 $N_k = 15$
 Gamma = 18 kN/m

After Jefferies and Davies (1991)

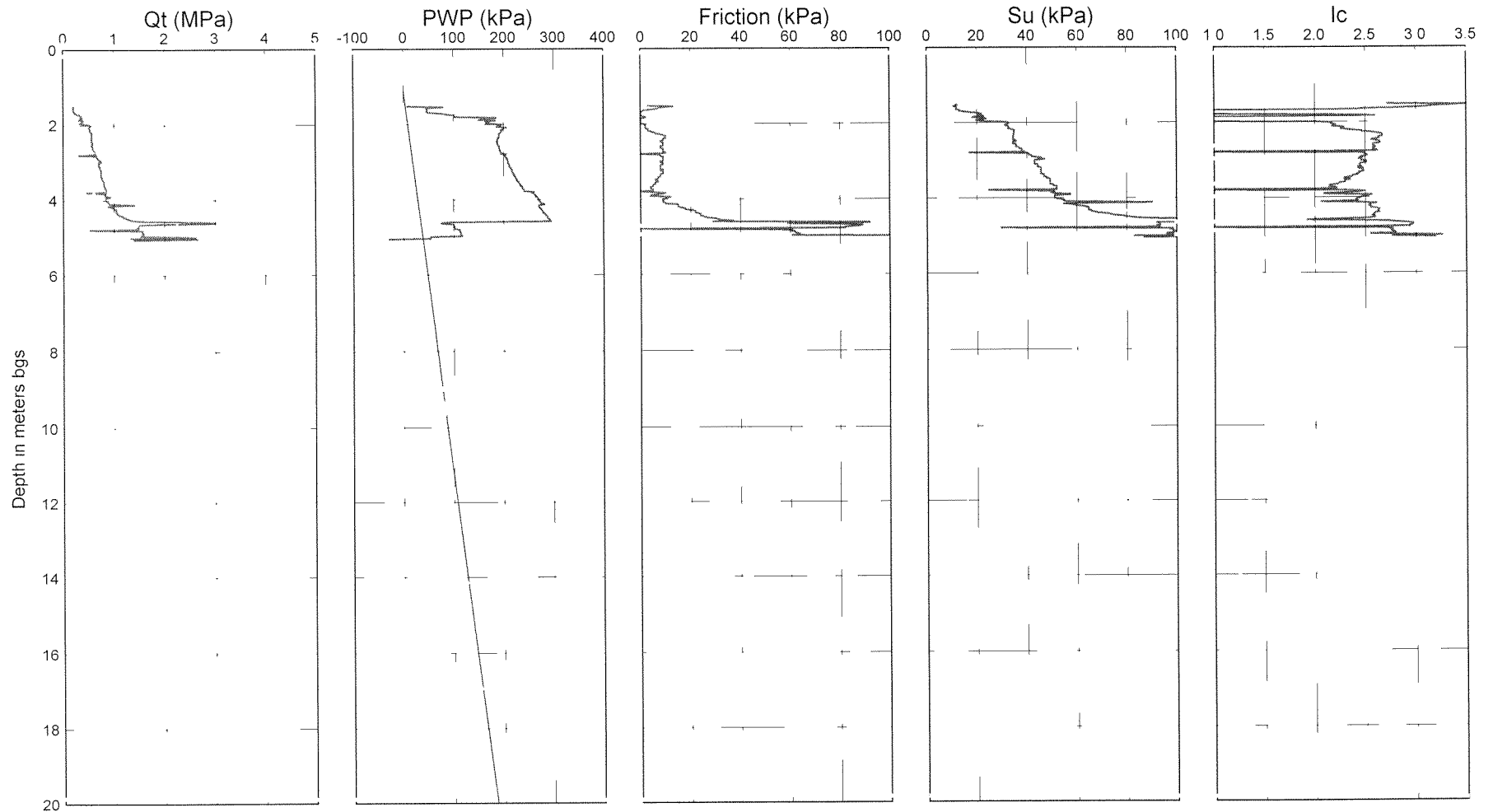
Ic 1.25 - Gravelly sands
 1.25 - Ic 1.90 - Clean to silty sand
 1.90 - Ic 2.54 - Silty sand to sandy silt
 2.54 - Ic 2.82 - Clayey silt to silty clay
 2.82 - Ic 3.22 - Clays

Cone Penetration Test - CPT04-02A

Test Date : May 31, 2004
Location :

Operator : Golder Associates

Ground Surf. Elev. : 80.00
Water Table Depth : 1.10



Qt normalized for
unequal end area effects

$S_u = (Q_t - S'_{cmax}) / N_k$
 $N_k = 15$
 $\gamma = 18 \text{ kN/m}^3$

After Jefferies and Davies (1991)

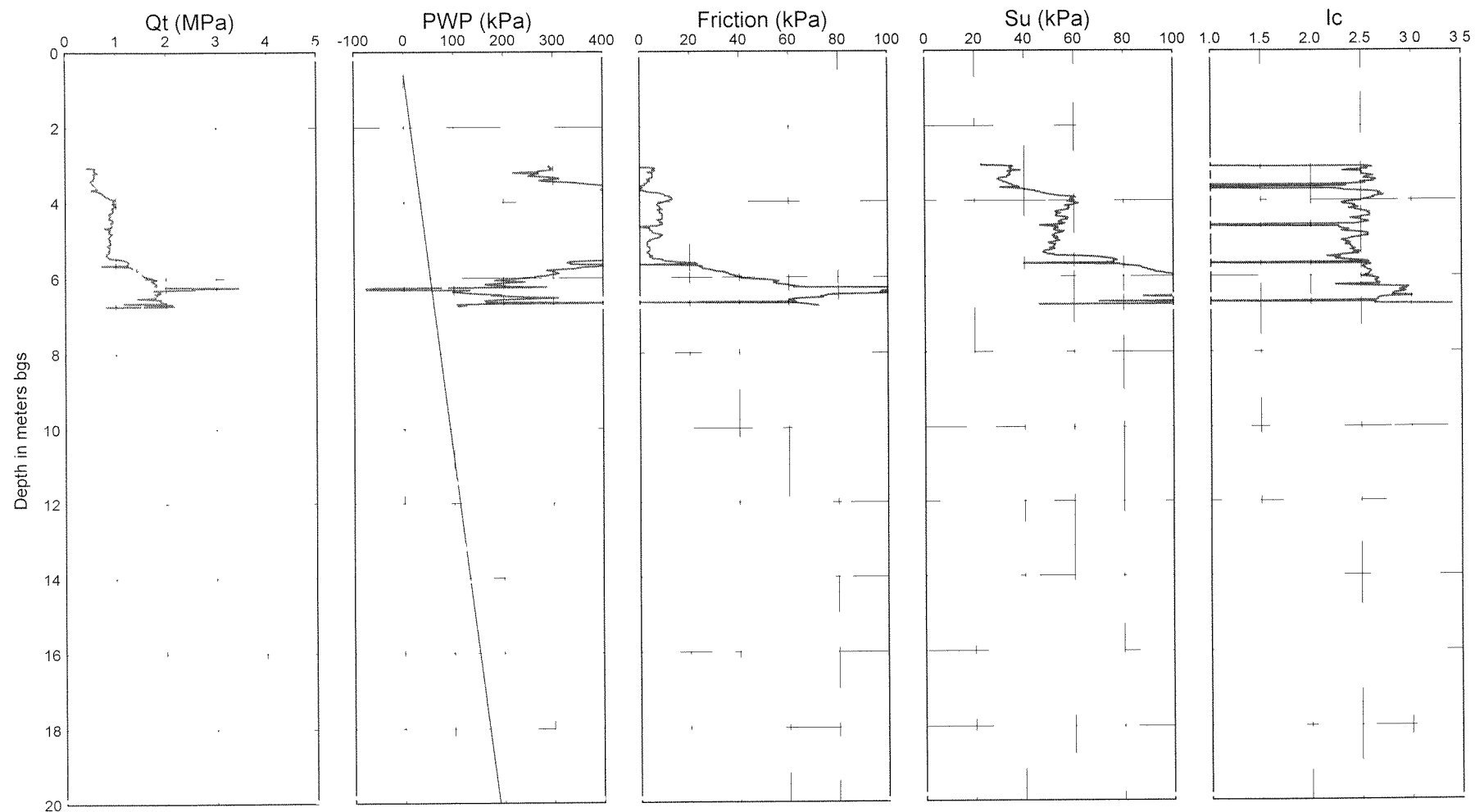
Ic 1.25 - Gravelly sands
1.25 - Ic 1.90 - Clean to silty sand
1.90 - Ic 2.54 - Silty sand to sandy silt
2.54 - Ic 2.82 - Clayey silt to silty clay
2.82 - Ic 3.27 - Clays

Cone Penetration Test - CPT04-03A

Test Date : June 1, 2004
Location :

Operator : Golder Associates

Ground Surf. Elev. : 80.90
Water Table Depth : 0.60



Qt normalized for
unequal cone effects

$S_u = (Q_t - S_{pmax}) / N_k$
 $N_k = 15$
 $\gamma = 18 \text{ kN/m}^3$

After Jefferies and Davies (1991)

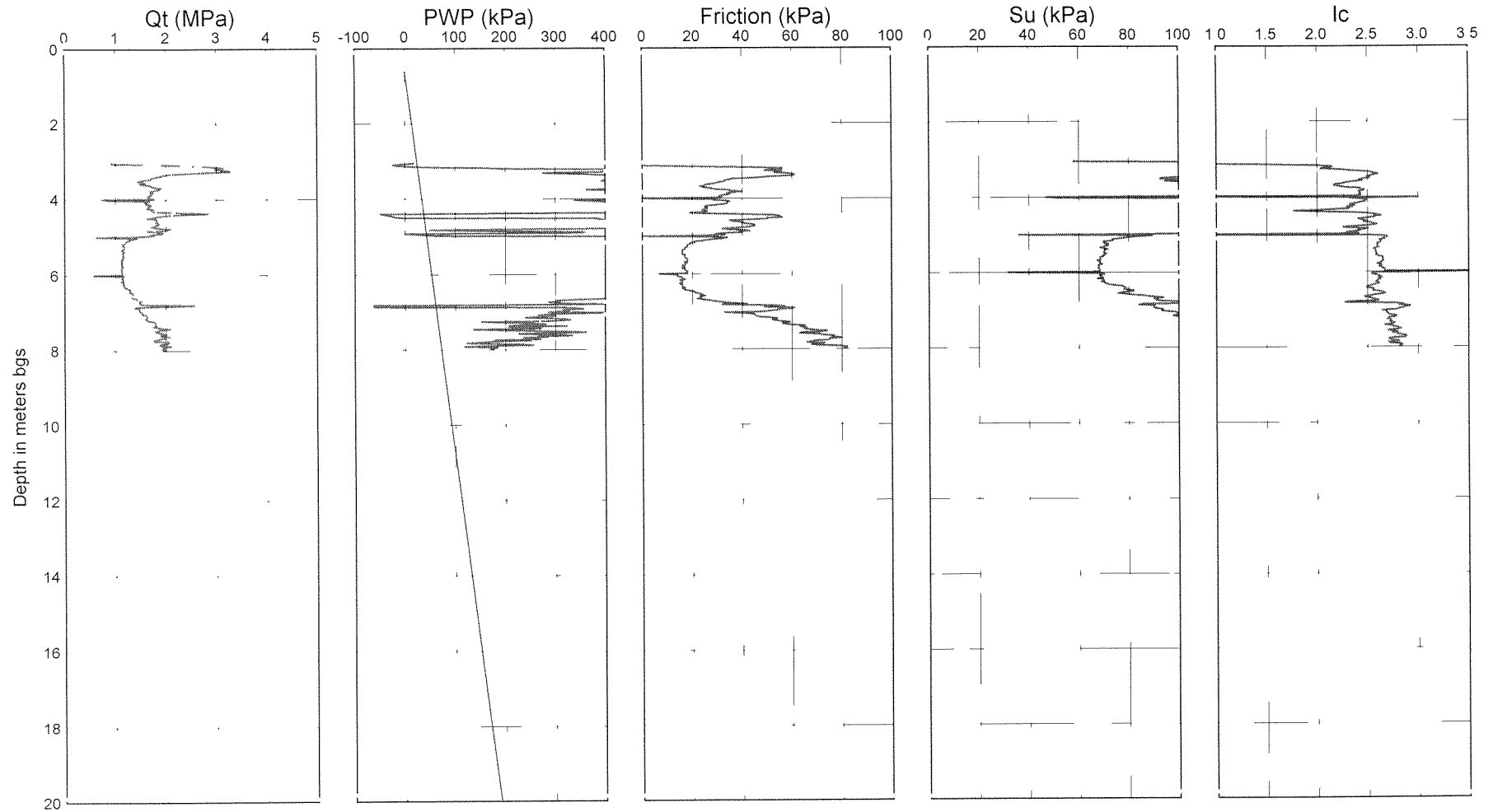
Ic 1.25 Gravelly sands
1.25 Ic 90 Clean to silty sand
1.90 Ic 2.54 Silty sand to sandy silt
2.54 Ic 2.82 Clayey silt to silty clay
2.82 Ic 3.22 Clays

Cone Penetration Test - CPT04-04A

Test Date : June 1, 2004
Location :

Operator : Golder Associates

Ground Surf. Elev. : 80.80
Water Table Depth : 0.60



Qt normalized for
unequal end area effects

$S_u = (Q_t - \sigma_{vm}) / N_k$
 $N_k = 15$
 $\gamma_{soil} = 18 \text{ kN/m}^3$

After Jefferies and Davies (1991)

Ic 1.25 - Gravelly sands
1.25 - Ic 1.90 - Clean to silty sand
1.90 - Ic 2.54 - Silty sand to sandy silt
2.54 - Ic 2.82 - Clayey silt to silty clay
2.82 - Ic 3.22 - Clays

APPENDIX A
LABORATORY TEST DATA

TABLE A1

SUMMARY OF WATER CONTENT DETERMINATIONS

PROJECT NUMBER		021-1162			
PROJECT NAME		MRC / QEW-Red Hill Creek / Hamilton			
DATE TESTED		May, 2004			
Borehole No.	Sample No.	Depth (ft)	Depth (m)	Water Content (%)	Atterberg Limits LL, PL, PI
BR9-1	2	2.5-4.0	0.76-1.22	13.1%	LL=20.7, PL=12.7, PI=8.0
BR9-1	4	10.0-11.5	3.05-3.51	20.6%	
BR9-1	5	15.0-16.5	4.57-5.03	22.6%	
BR9-1	7	25.0-26.5	7.62-8.08	17.7%	
BR9-1	9	35.0-36.5	10.67-11.13	15.6%	
BR9-1	11	45.0-45.6	13.72-13.90	6.8%	LL=32.2, PL=16.2, PI=16.0
BR9-1	13	55.0-55.4	16.76-16.89	6.8%	
BR9-1	15	65.0-65.8	19.81-20.06	13.9%	
BR9-1	5B	15.0-17.0	4.57-5.18	20.0%	
BR9-2	2	2.5-4.0	0.76-1.22	17.9%	LL=28.0, PL=16.1, PI=11.9
BR9-2	4	7.5-9.0	2.29-2.74	20.3%	
BR9-2	6	15.0-16.5	4.57-5.03	21.5%	
BR9-2	8	25.0-26.5	7.62-8.08	16.9%	
BR9-2	10	35.0-36.5	10.67-11.13	15.3%	
BR9-2	12	45.0-46.5	13.72-14.17	9.1%	LL=34.0, PL=16.8, PI=17.2
BR9-2	14	55.0-55.4	16.76-16.89	5.8%	
BR9-2	16	65.0-65.4	19.81-19.93	6.5%	
BR9-2	17	70.0-70.6	21.34-21.52	11.6%	
BR9-3	2	2.5-4.0	0.76-1.22	16.5%	
BR9-3	4	7.5-9.0	2.29-2.74	24.5%	LL=20.0, PL=14.5, PI=5.5
BR9-3	5B	10.0-12.0	3.05-3.66	28.3%	
BR9-3	7	15.0-16.5	4.57-5.03	22.9%	
BR9-3	9	25.0-26.5	7.62-8.08	18.8%	
BR9-3	11	35.0-36.5	10.67-11.13	14.9%	
BR9-3	13	45.0-46.5	13.72-14.17	11.3%	LL=38.7, PL=18.9, PI=19.8
BR9-3	15	55.0-56.5	16.76-17.22	11.0%	
BR9-4	5	10.0-12.0	3.05-3.66	28.1%	
BR9-4	6	12.5-14.5	3.81-4.42	24.7%	
BR9-4	7	15.0-17.0	4.57-5.18	31.1%	
BR9-4	8	17.5-19.5	5.33-5.94	29.6%	LL=30.4, PL=16.3, PI=14.1
BR9-4	9	20.0-22.0	6.10-6.71	30.1%	

TABLE A1

SUMMARY OF WATER CONTENT DETERMINATIONS

PROJECT NUMBER		021-1162			
PROJECT NAME		MRC / QEW-Red Hill Creek / Hamilton			
DATE TESTED		May, 2004			
Borehole No.	Sample No.	Depth (ft)	Depth (m)	Water Content (%)	Atterberg Limits LL, PL, PI
BR9-4	10	22.5-24.5	6.86-7.47	19.0%	LL=30.9, PL=15.8, PI=15.1
BR9-7	3	4.0-6.0	1.22-1.83	22.9%	
BR9-7	4	6.0-8.0	1.83-2.44	20.2%	LL=32.2, PL=17.2, PI=15.0
BR9-7	5	8.0-10.0	2.44-3.05	26.3%	LL=22.8, PL=14.0, PI=8.8
BR9-7	6	10.0-12.0	3.05-3.66	26.9%	LL=33.6, PL=17.3, PI=16.3
BR9-7	7	12.5-14.5	3.81-4.42	28.9%	LL=33.3, PL=17.8, PI=15.5
BR9-7	8	15.0-17.0	4.57-5.18	25.1%	LL=31.7, PL=17.4, PI=14.3
BR9-7	9	17.5-19.5	5.33-5.94	20.7%	LL=33.7, PL=17.1, PI=16.6
BR9-7	10	20.0-22.0	6.10-6.71	18.5%	
RSER-6	1	2.5-4.0	0.76-1.22	11.7%	
RSER-6	2	5.0-7.5	1.52-2.29	18.3%	
RSER-6	3	7.5-9.0	2.29-2.74	18.6%	
RSER-6	4	10.0-11.5	3.05-3.51	22.5%	
RSER-6	5	12.5-14.0	3.81-4.27	29.4%	LL=41.0, PL=17.8, PI=23.2
RSER-6	6	15.5-17.5	4.72-5.33	30.1%	LL=24.9, PL=13.0, PI=11.9
RSER-6	7	19.0-21.0	5.79-6.40	28.6%	
RSER-6	8	25.0-26.5	7.62-8.08	14.8%	
RSER-6	9	30.0-31.5	9.14-9.60	14.0%	
RSER-6	10	35.0-36.5	10.67-11.13	14.8%	
RSER-7	1	0.0-2.0	0.00-0.61	10.3%	
RSER-7	2	2.5-4.0	0.76-1.22	17.3%	
RSER-7	3	5.0-6.5	1.52-1.98	24.2%	
RSER-7	4	7.5-9.0	2.29-2.74	26.7%	
RSER-7	5	10.5-11.5	3.20-3.51	21.5%	
RSER-7	6	12.5-14.0	3.81-4.27	22.2%	
RSER-7	7	15.0-16.5	4.57-5.03	25.8%	
RSER-7	8	17.5-19.0	5.33-5.79	19.4%	
RSER-7	9	20.0-21.5	6.10-6.55	18.0%	
RSER-7	10	22.5-24.0	6.86-7.32	15.8%	

TABLE A2
SPECIFIC GRAVITY TEST RESULTS
ASTM D 854-00 TEST METHOD A

PROJECT NUMBER	021-1162		
PROJECT NAME	MRC / QEW-Red Hill Creek / Hamilton		
DATE TESTED	June, 2004		
Borehole No.	Sample No.	Specific Gravity	
BR9-1	5B	2.74	
BR9-3	5B	2.75	
BR9-4	7	2.78	
BR9-7	8	2.79	

Note: Test carried out on soil particles <4.75mm using distilled water.

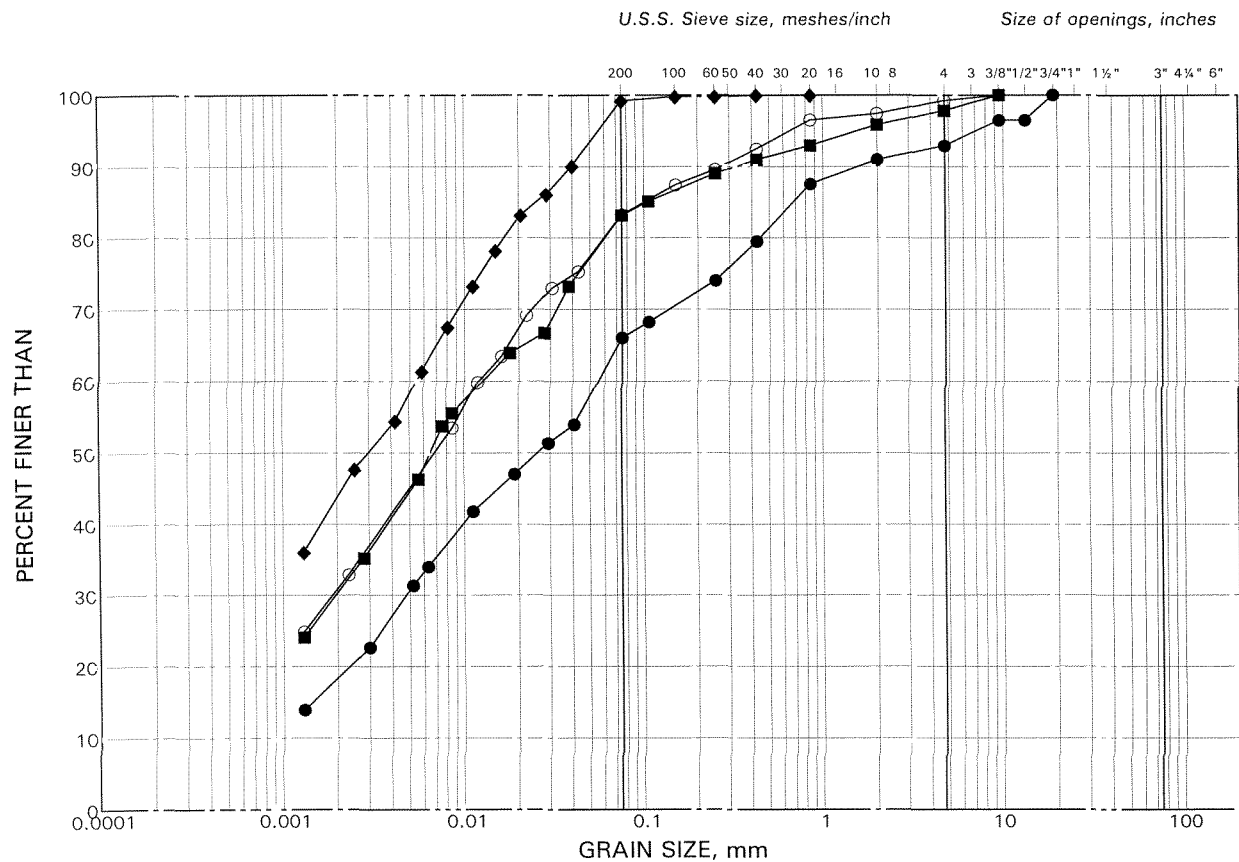
TABLE A3
SUMMARY OF LABORATORY VANE TESTING

Project Number		021-1162		Date of Testing						5/28/2004
Borehole No.	Sample No.	Sample Depth (m)	Location in shelby tube	Vane Angular Deflection	Peak	Residual	Vane Blade	Peak	Residual	Water
				Peak / Residual Degrees	Torque (Nm)	Torque (Nm)	Constant (m ³)	Shear Strength (kPa)	Shear Strength (kPa)	Content (%)
BR9-4	6	3.81-4.42	9cm from bottom	71/20	0.13	0.04	4.24E-06	31.53	8.88	28.9
BR9-4	7	4.57-5.18	7cm from bottom	88/27	0.17	0.05	4.24E-06	39.08	11.99	28.3
BR9-4	8	5.33-5.94	12cm from bottom	57/32	0.11	0.06	4.24E-06	25.32	14.21	33.7
BR9-7	5	2.44-3.05	18cm from top	42/11	0.08	0.02	4.24E-06	18.65	4.89	29.8
BR9-7	5	2.44-3.05	44cm from top	83/23	0.16	0.04	4.24E-06	36.86	10.21	22.7
BR9-7	7	3.81-4.42	23cm from top	67/25	0.13	0.05	4.24E-06	29.76	11.10	28.9
BR9-7	8	4.57-5.18	14cm from top	100/31	0.19	0.06	4.24E-06	44.41	13.77	25.9

GRAIN SIZE DISTRIBUTION

Clayey Silt (Till)

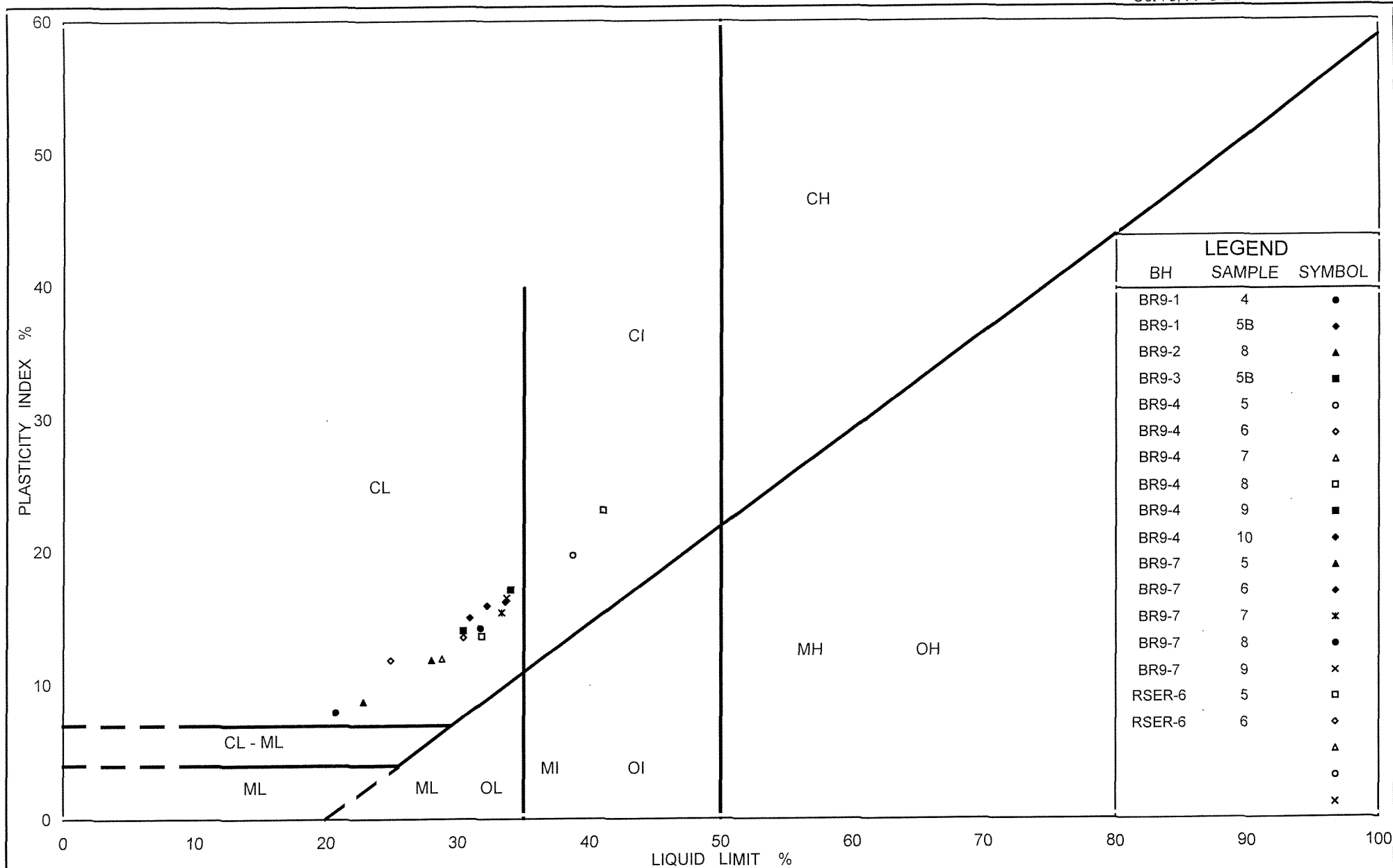
FIGURE A1



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
●	BR9-1	4	77.1
■	BR9-2	8	72.6
◆	RSER-6	5	77.0
○	RSER-6	6	76.0



Ministry of Transportation

Ontario

PLASTICITY CHART Clayey Silt (Till)

FIG No. A2

Project No. 021-1162

OEDOMETER CONSOLIDATION SUMMARY

SAMPLE IDENTIFICATION

Project Number	021-1162	Sample Number	5B
Borehole Number	BR 9-1	Sample Depth, m	4.6-5.2

TEST CONDITIONS

Test Type	Standard	Load Duration, hr	24
Oedometer Number	7		
Date Started	10/06/2003		
Date Completed	10/16/2003		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	1.90	Unit Weight, kN/m ³	21.05
Sample Diameter, cm	6.34	Dry Unit Weight, kN/m ³	17.66
Area, cm ²	31.57	Specific Gravity, measured	2.74
Volume, cm ³	59.98	Solids Height, cm	1.249
Water Content, %	19.21	Volume of Solids, cm ³	39.42
Wet Mass, g	128.76	Volume of Voids, cm ³	20.56
Dry Mass, g	108.01	Degree of Saturation, %	100.9

TEST COMPUTATIONS

Pressure kPa	Corr. Height cm	Void Ratio	Average Height cm	t ₉₀ sec	cv. cm ² /s	mv m ² /kN	k cm/s
0.00	1.900	0.522	1.900				
4.84	1.898	0.520	1.899	8	9.56E-02	2.39E-04	2.24E-06
9.48	1.895	0.517	1.896	46	1.66E-02	3.74E-04	6.07E-07
19.56	1.888	0.512	1.891	171	4.43E-03	3.55E-04	1.54E-07
39.01	1.876	0.502	1.882	321	2.34E-03	3.14E-04	7.19E-08
77.76	1.859	0.489	1.868	433	1.71E-03	2.32E-04	3.89E-08
155.27	1.835	0.469	1.847	295	2.45E-03	1.66E-04	4.00E-08
310.14	1.798	0.440	1.816	312	2.24E-03	1.24E-04	2.73E-08
620.40	1.749	0.401	1.773	496	1.34E-03	8.31E-05	1.09E-08
1240.55	1.692	0.355	1.720	321	1.95E-03	4.84E-05	9.27E-09
2480.44	1.625	0.302	1.659	141	4.14E-03	2.83E-05	1.15E-08
1240.55	1.638	0.312	1.632				
310.14	1.669	0.336	1.653				
77.76	1.705	0.366	1.687				
19.56	1.740	0.393	1.723				
4.84	1.774	0.420	1.757				

Notes:

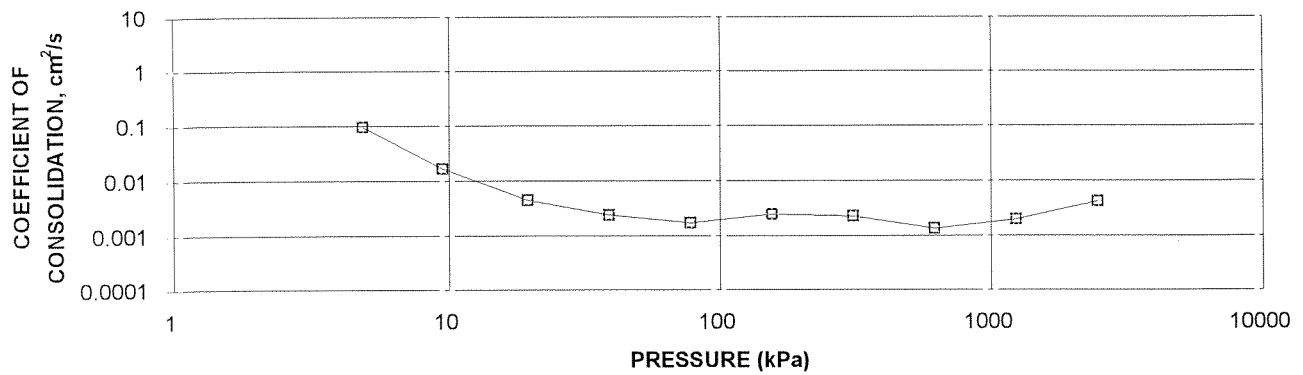
k calculated using cv based on σ_0 values.

SAMPLE DIMENSIONS AND PROPERTIES - FINAL

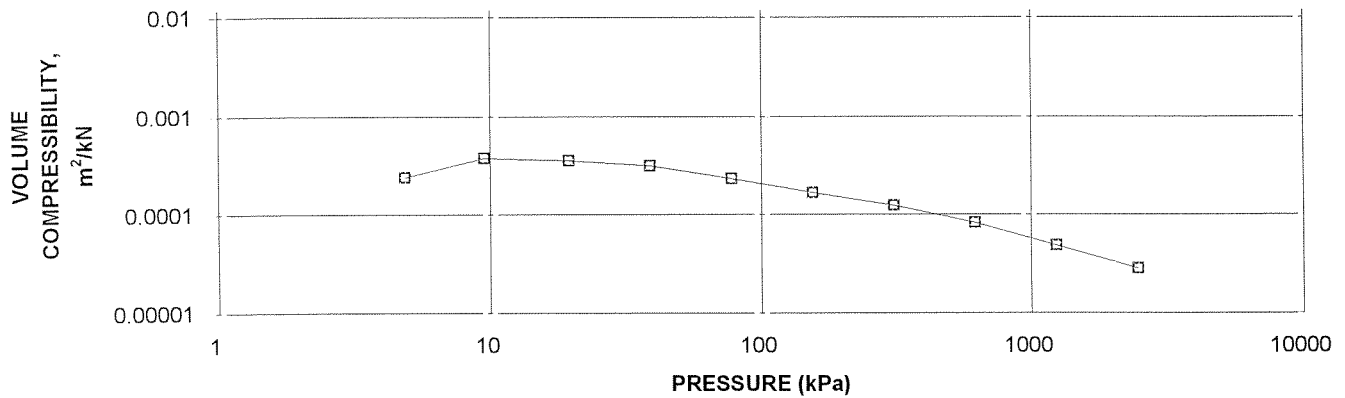
Sample Height, cm	1.77	Unit Weight, kN/m ³	22.53
Sample Diameter, cm	6.34	Dry Unit Weight, kN/m ³	18.92
Area, cm ²	31.57	Specific Gravity, measured	2.74
Volume, cm ³	55.99	Solids Height, cm	1.249
Water Content, %	19.13	Volume of Solids, cm ³	39.42
Wet Mass, g	128.67	Volume of Voids, cm ³	16.58
Dry Mass, g	108.01		

OEDOMETER CONSOLIDATION SUMMARY

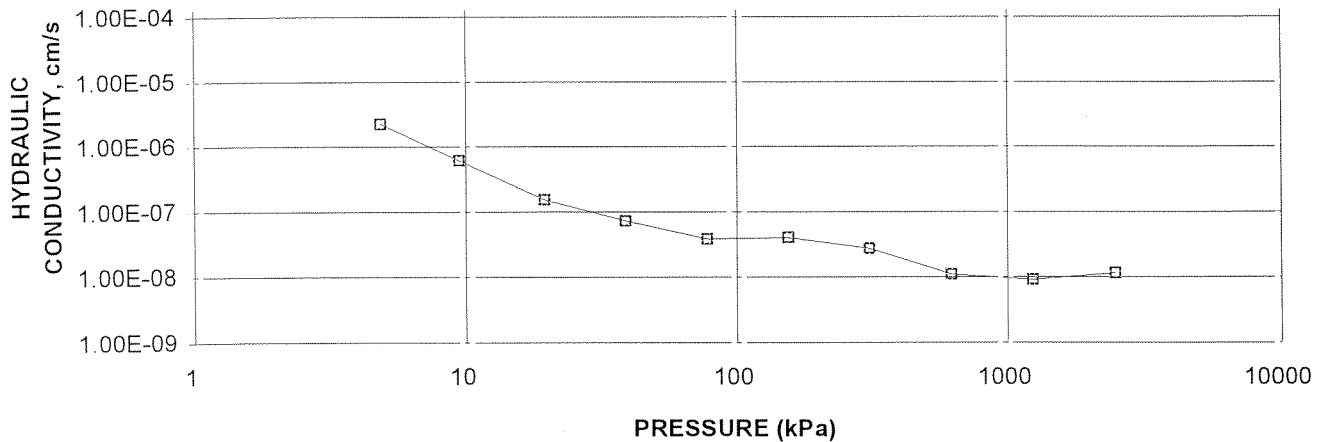
CONSOLIDATION TEST
CV cm^2/s VS PRESSURE (kPa)
BH BR9-1 SA 5B



CONSOLIDATION TEST
MV m^2/kN vs PRESSURE (kPa)
BH BR9-1 SA 5B



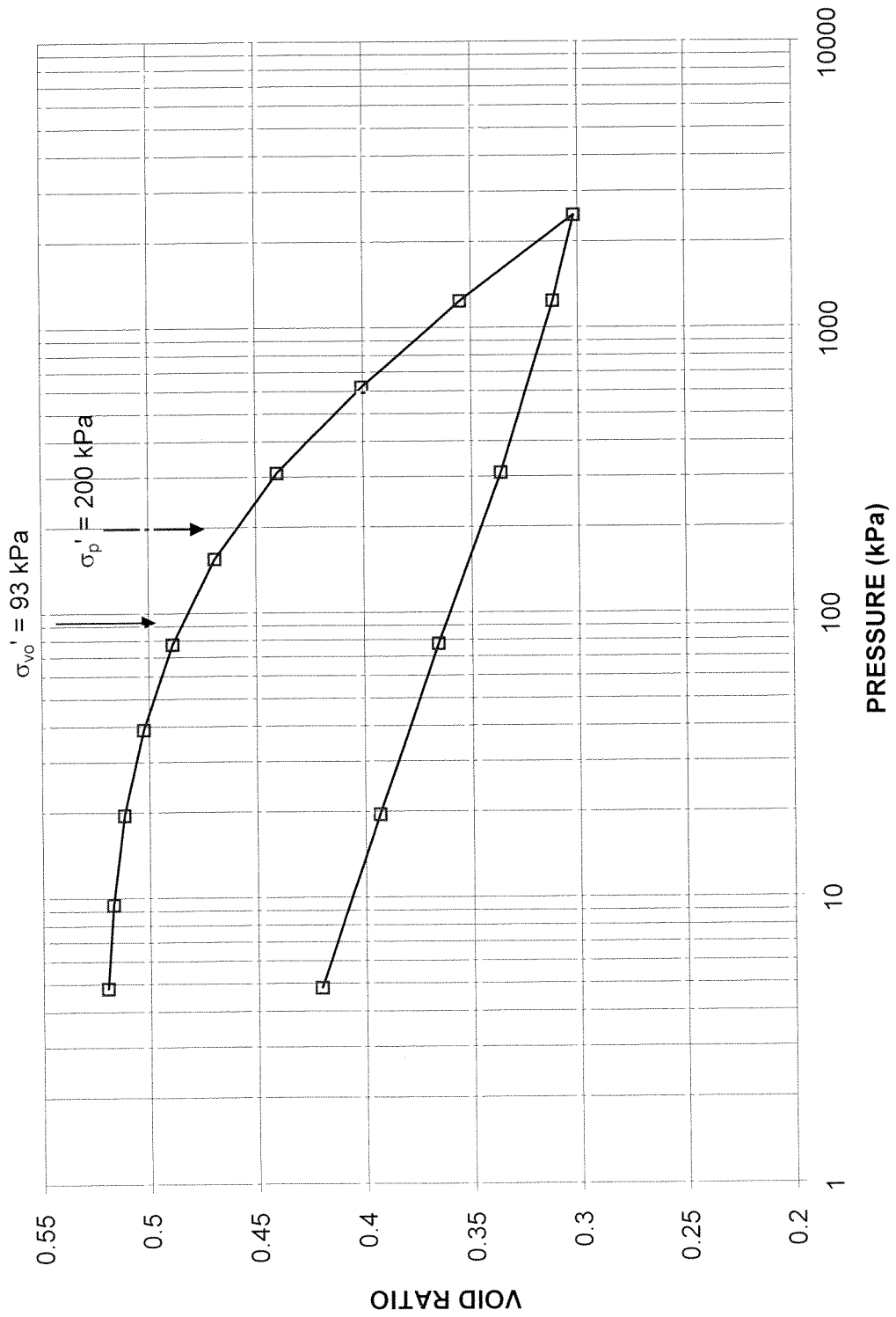
CONSOLIDATION TEST
HYDRAULIC CONDUCTIVITY vs PRESSURE
BH BR9-1 SA 5B



CONSOLIDATION TEST
VOID RATIO VS. LOG PRESSURE

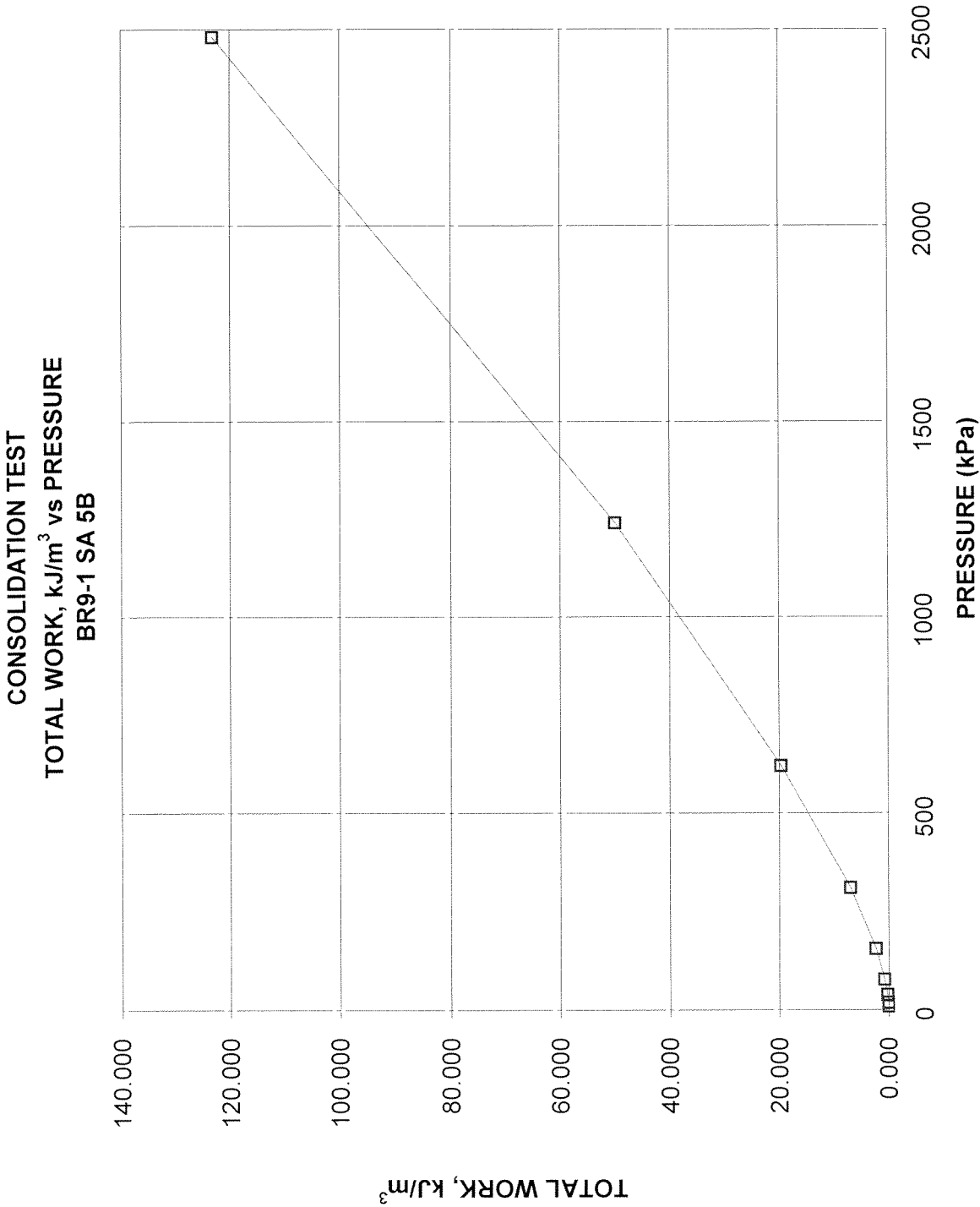
FIGURE A3

CONSOLIDATION TEST
VOID RATIO vs. PRESSURE
BR9-1 SA 5B



CONSOLIDATION TEST
TOTAL WORK VS. PRESSURE

FIGURE A4



OEDOMETER CONSOLIDATION SUMMARY

SAMPLE IDENTIFICATION

Project Number	021-1162	Sample Number	5B
Borehole Number	BR 9-3	Sample Depth, m	3.0-3.7

TEST CONDITIONS

Test Type	Standard	Load Duration, hr	24
Oedometer Number	6		
Date Started	9/25/2003		
Date Completed	10/05/2003		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	1.91	Unit Weight, kN/m ³	19.41
Sample Diameter, cm	6.34	Dry Unit Weight, kN/m ³	15.13
Area, cm ²	31.57	Specific Gravity, measured	2.75
Volume, cm ³	60.30	Solids Height, cm	1.072
Water Content, %	28.24	Volume of Solids, cm ³	33.84
Wet Mass, g	119.34	Volume of Voids, cm ³	26.46
Dry Mass, g	93.06	Degree of Saturation, %	99.3

TEST COMPUTATIONS

Pressure kPa	Corr. Height cm	Void Ratio	Average Height cm	t ₉₀ sec	cv, cm ² /s	mv m ² /kN	k cm/s
0.00	1.910	0.782	1.910				
4.77	1.908	0.780	1.909	8	9.66E-02	2.09E-04	1.97E-06
9.58	1.906	0.778	1.907	60	1.29E-02	2.07E-04	2.61E-07
19.31	1.902	0.774	1.904	190	4.04E-03	2.37E-04	9.38E-08
38.80	1.892	0.765	1.897	240	3.18E-03	2.69E-04	8.37E-08
77.63	1.877	0.751	1.885	158	4.77E-03	1.97E-04	9.19E-08
155.17	1.854	0.729	1.865	119	6.20E-03	1.60E-04	9.72E-08
310.14	1.800	0.679	1.827	177	4.00E-03	1.80E-04	7.07E-08
620.49	1.716	0.601	1.758	375	1.75E-03	1.42E-04	2.43E-08
1240.39	1.633	0.524	1.675	295	2.02E-03	6.99E-05	1.38E-08
2481.16	1.554	0.449	1.593	197	2.73E-03	3.35E-05	8.98E-09
1240.39	1.564	0.459	1.559				
310.14	1.595	0.488	1.579				
77.63	1.638	0.528	1.617				
19.31	1.685	0.572	1.662				
4.77	1.722	0.607	1.704				

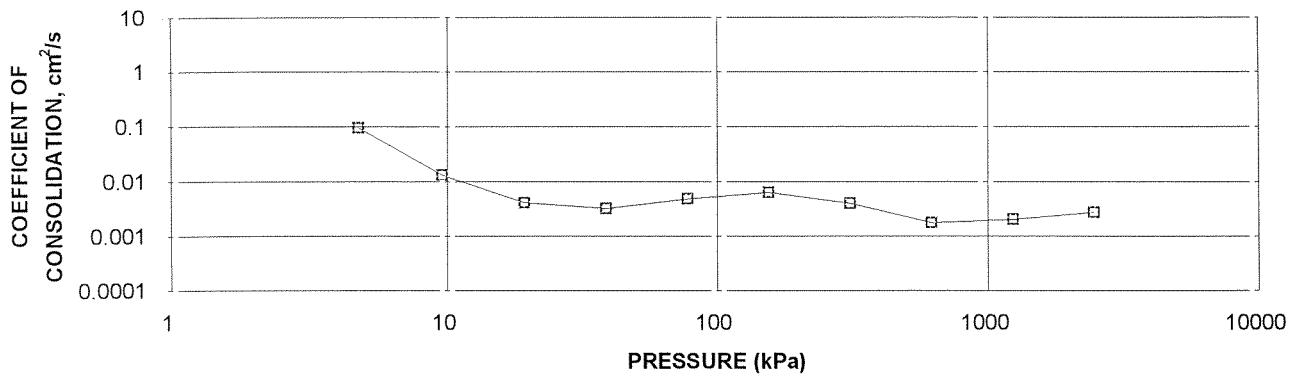
Notes:
k calculated using cv based on $\bar{\sigma}_0$ values.

SAMPLE DIMENSIONS AND PROPERTIES - FINAL

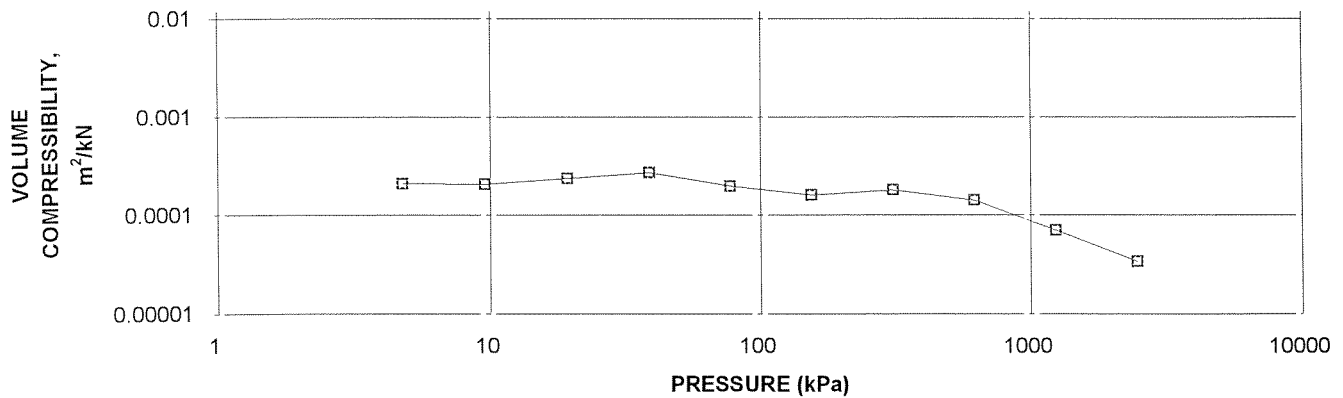
Sample Height, cm	1.72	Unit Weight, kN/m ³	20.62
Sample Diameter, cm	6.34	Dry Unit Weight, kN/m ³	16.79
Area, cm ²	31.57	Specific Gravity, measured	2.75
Volume, cm ³	54.37	Solids Height, cm	1.072
Water Content, %	22.85	Volume of Solids, cm ³	33.84
Wet Mass, g	114.32	Volume of Voids, cm ³	20.53
Dry Mass, g	93.06		

OEDOMETER CONSOLIDATION SUMMARY

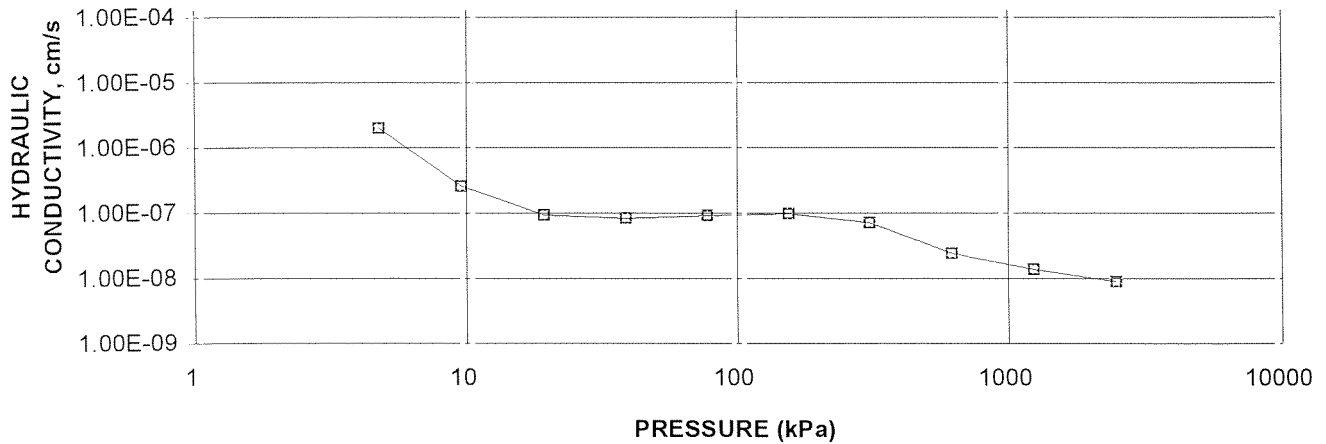
CONSOLIDATION TEST
CV cm^2/s VS PRESSURE (kPa)
BR9-3 SA 5B



CONSOLIDATION TEST
MV m^2/kN vs PRESSURE (kPa)
BR9-3 SA 5B



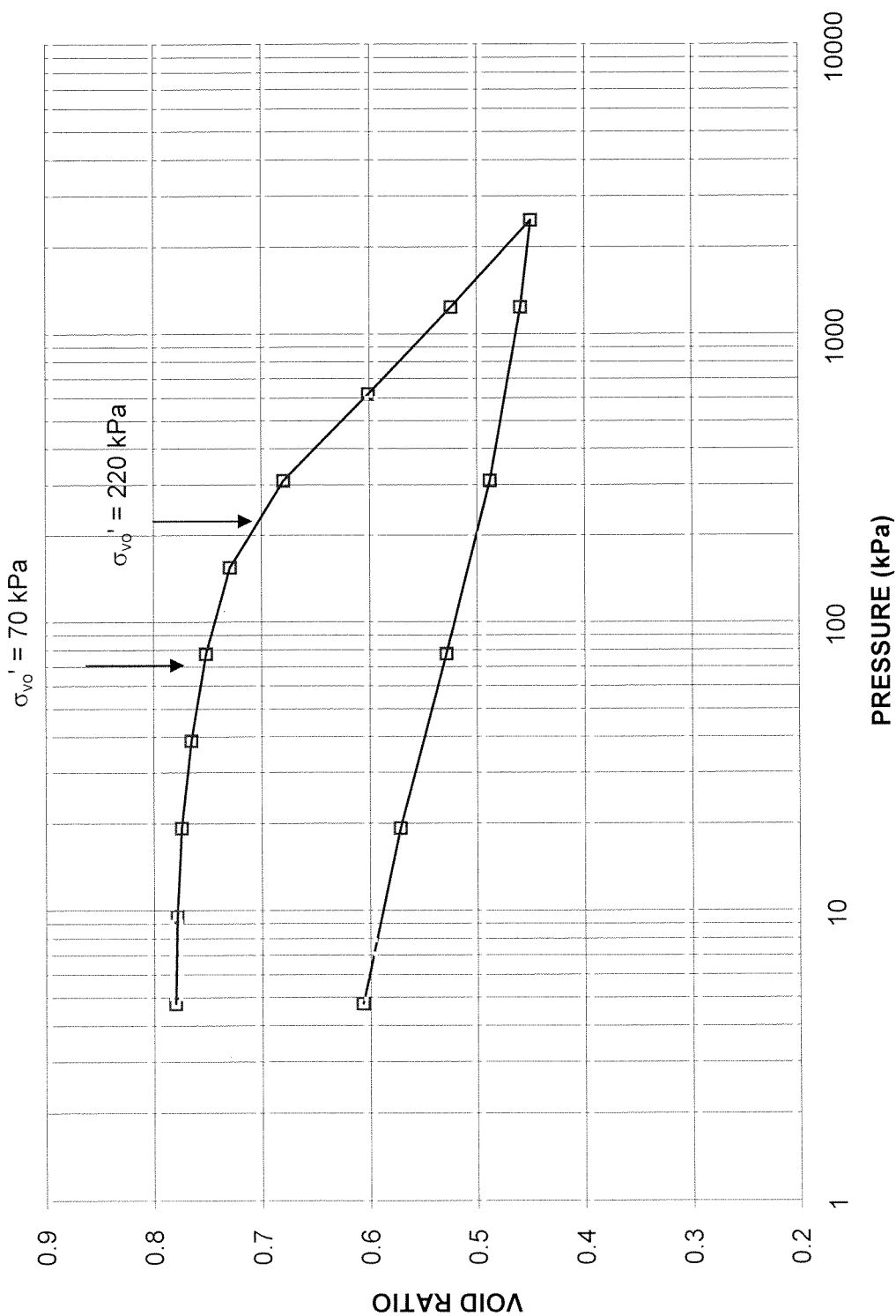
CONSOLIDATION TEST
HYDRAULIC CONDUCTIVITY vs PRESSURE
BR9-3 SA 5B



CONSOLIDATION TEST VOID RATIO VS. LOG PRESSURE

FIGURE A5

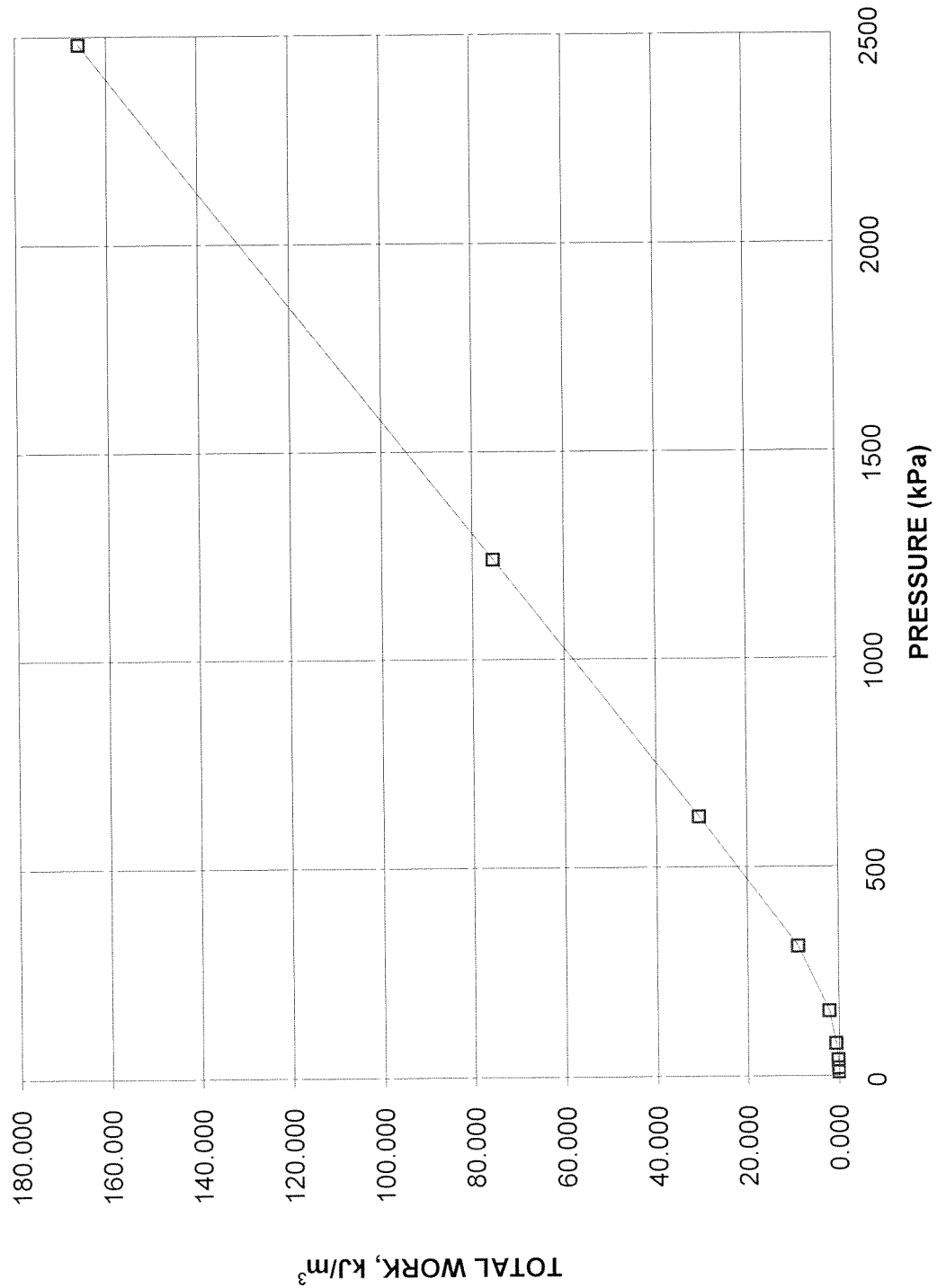
CONSOLIDATION TEST VOID RATIO vs. PRESSURE BR9-3 SA 5B



**CONSOLIDATION TEST
TOTAL WORK VS. PRESSURE**

FIGURE A6

**CONSOLIDATION TEST
TOTAL WORK, kJ/m^3 vs PRESSURE
BR9-3 SA 5B**



OEDOMETER CONSOLIDATION SUMMARY (HTO)

SAMPLE IDENTIFICATION

Project Number	021-1162	Sample Number	7
Borehole Number	BR 9-4	Sample Depth, m	4.6-5.2

TEST CONDITIONS

Test Type	Standard	Load Duration, hr	24
Oedometer Number	5		
Date Started	5/28/2004		
Date Completed	6/08/2004		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	1.91	Unit Weight, kN/m ³	19.81
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m ³	15.73
Area, cm ²	31.65	Specific Gravity, measured	2.78
Volume, cm ³	60.45	Solids Height, cm	1.102
Water Content, %	25.91	Volume of Solids, cm ³	34.88
Wet Mass, g	122.11	Volume of Voids, cm ³	25.57
Dry Mass, g	96.98	Degree of Saturation, %	98.3

TEST COMPUTATIONS

Pressure kPa	Corr. Height cm	Void Ratio	Average Height cm	t ₉₀ sec	cv. cm ² /s	mv m ² /kN	k cm/s
0.00	1.910	0.733	1.910				
4.70	1.896	0.720	1.903	89	8.63E-03	1.56E-03	1.32E-06
9.54	1.890	0.715	1.893	124	6.13E-03	6.49E-04	3.90E-07
19.29	1.877	0.703	1.884	113	6.66E-03	6.98E-04	4.55E-07
38.71	1.861	0.688	1.869	85	8.71E-03	4.31E-04	3.68E-07
77.44	1.837	0.667	1.849	60	1.21E-02	3.24E-04	3.84E-07
154.67	1.804	0.637	1.821	76	9.24E-03	2.24E-04	2.03E-07
310.09	1.756	0.593	1.780	124	5.42E-03	1.62E-04	8.58E-08
619.45	1.704	0.546	1.730	94	6.75E-03	8.80E-05	5.82E-08
1237.37	1.650	0.497	1.677	49	1.22E-02	4.58E-05	5.46E-08
2474.43	1.590	0.443	1.620	64	8.69E-03	2.54E-05	2.16E-08
1237.37	1.599	0.451	1.595				
310.09	1.623	0.472	1.611				
77.44	1.655	0.501	1.639				
19.29	1.680	0.524	1.668				
4.70	1.713	0.554	1.697				

Notes:

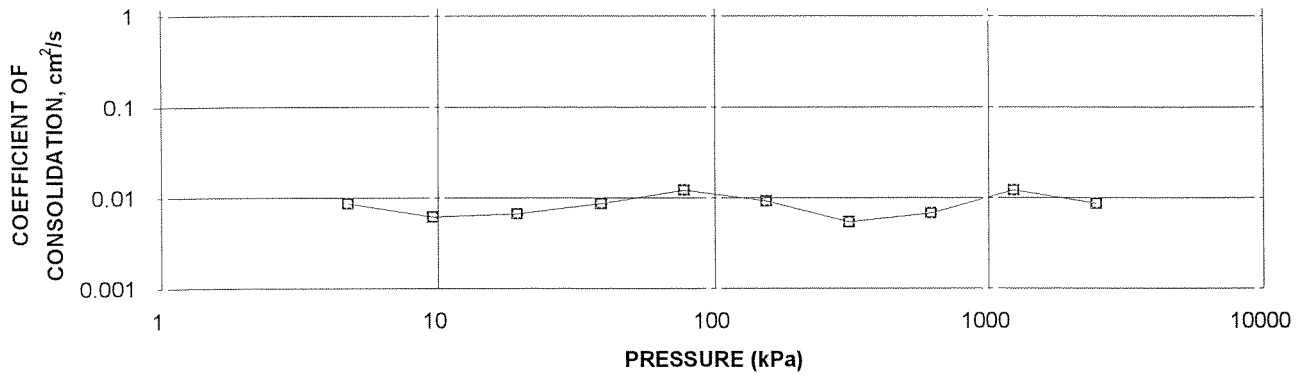
k calculated using cv based on t_{90} values.
specimen horizontally trimmed

SAMPLE DIMENSIONS AND PROPERTIES - FINAL

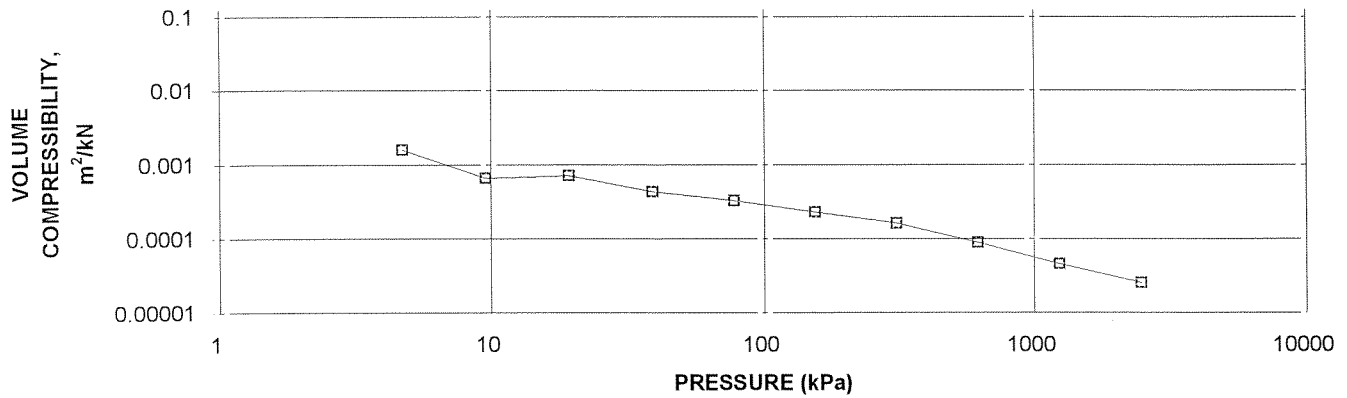
Sample Height, cm	1.71	Unit Weight, kN/m ³	21.12
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m ³	17.54
Area, cm ²	31.65	Specific Gravity, measured	2.78
Volume, cm ³	54.22	Solids Height, cm	1.102
Water Content, %	20.40	Volume of Solids, cm ³	34.88
Wet Mass, g	116.76	Volume of Voids, cm ³	19.33
Dry Mass, g	96.98		

OEDOMETER CONSOLIDATION SUMMARY

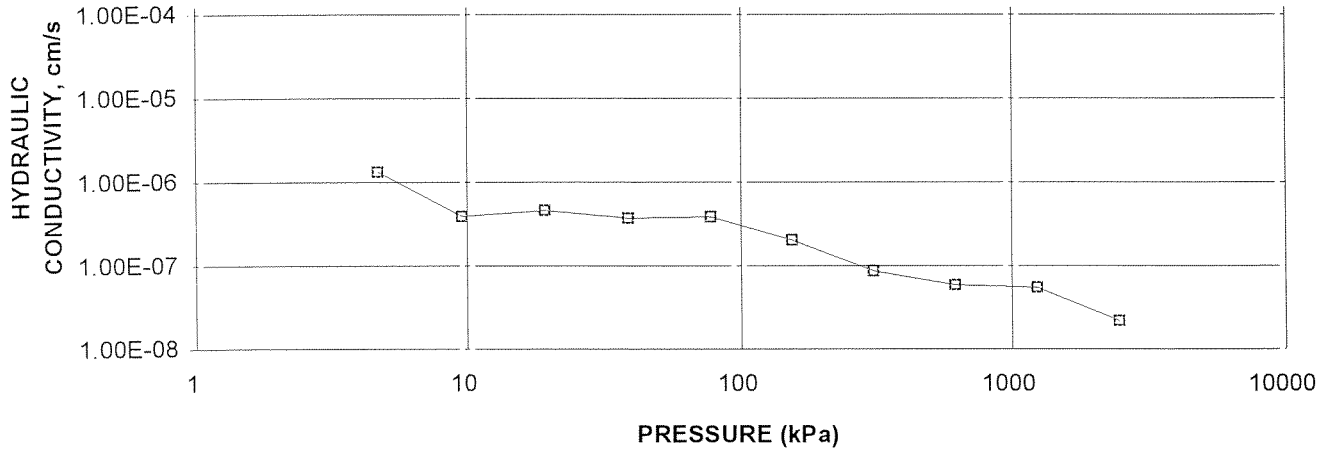
CONSOLIDATION TEST
CV cm^2/s VS PRESSURE (kPa)
BR9-4 SA 7 (HTO)



CONSOLIDATION TEST
MV m^2/kN vs PRESSURE (kPa)
BR9-4 SA 7 (HTO)



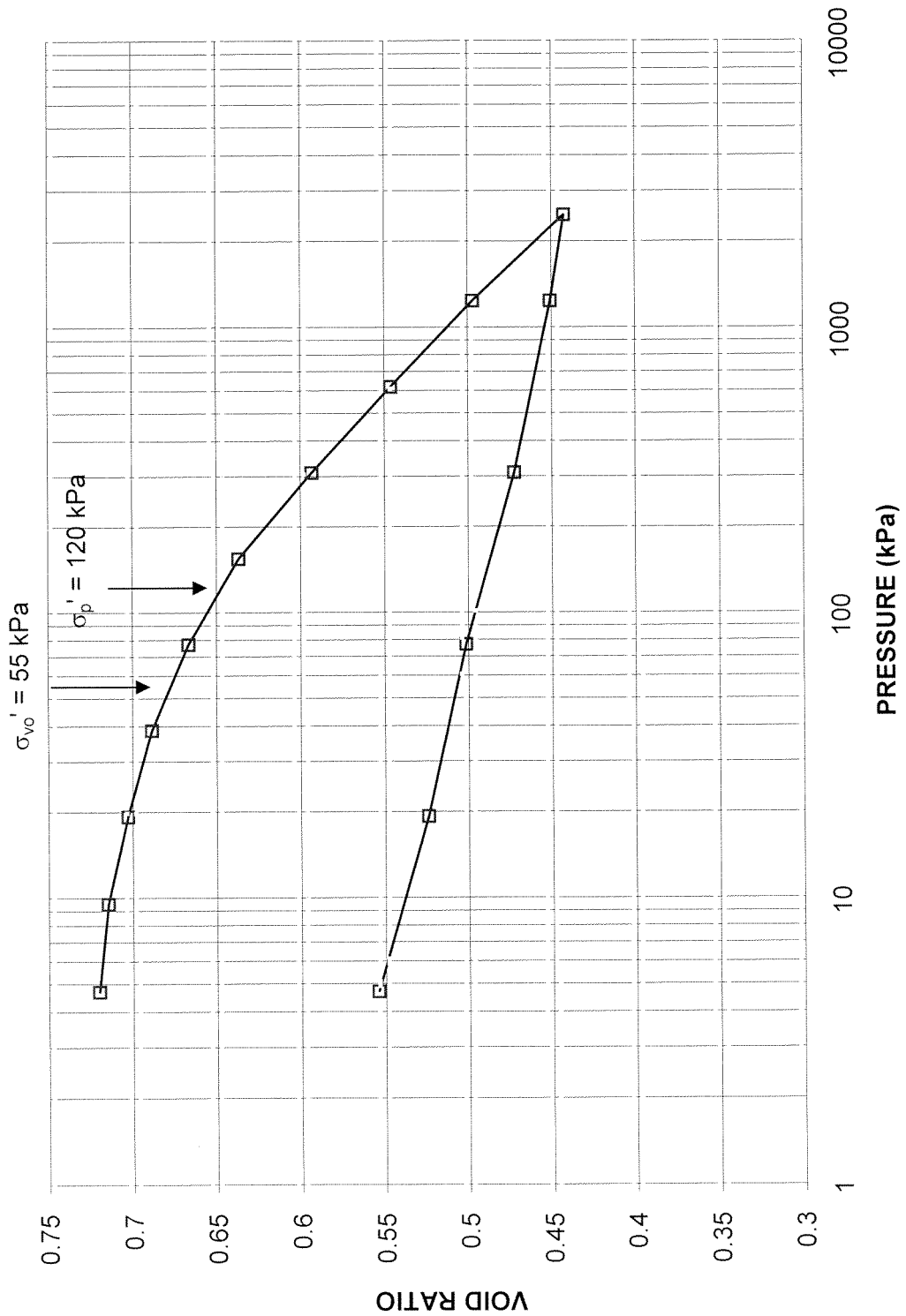
CONSOLIDATION TEST
HYDRAULIC CONDUCTIVITY vs PRESSURE
BR9-4 SA 7 (HTO)



CONSOLIDATION TEST
VOID RATIO VS. LOG PRESSURE

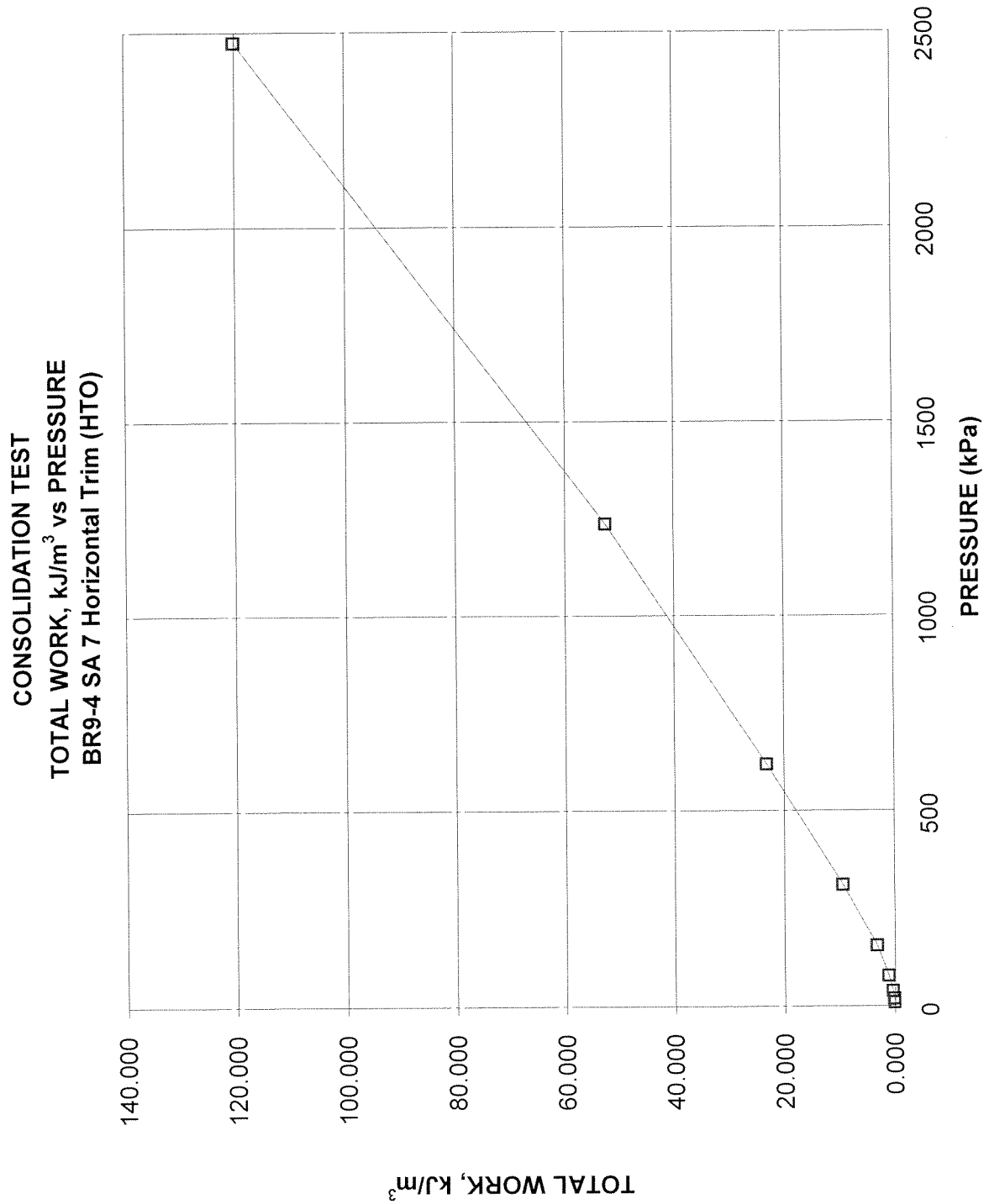
FIGURE A7

CONSOLIDATION TEST
VOID RATIO vs PRESSURE
BR9-4 SA 7 Horizontal Trim (HTO)



**CONSOLIDATION TEST
TOTAL WORK VS. PRESSURE**

FIGURE A8



OEDOMETER CONSOLIDATION SUMMARY (VTO)

SAMPLE IDENTIFICATION

Project Number	021-1162	Sample Number	7
Borehole Number	BR 9-4	Sample Depth, m	4.6-5.2

TEST CONDITIONS

Test Type	Standard	Load Duration, hr	24
Oedometer Number	6		
Date Started	5/28/2004		
Date Completed	6/08/2004		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	1.90	Unit Weight, kN/m ³	19.71
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m ³	15.51
Area, cm ²	31.67	Specific Gravity, measured	2.78
Volume, cm ³	60.17	Solids Height, cm	1.081
Water Content, %	27.08	Volume of Solids, cm ³	34.23
Wet Mass, g	120.94	Volume of Voids, cm ³	25.94
Dry Mass, g	95.17	Degree of Saturation, %	99.4

TEST COMPUTATIONS

Pressure kPa	Corr. Height cm	Void Ratio	Average Height cm	t ₉₀ sec	ch cm ² /s	mh m ² /kN	k cm/s
0.00	1.900	0.758	1.900				
4.75	1.881	0.740	1.891	60	1.26E-02	2.11E-03	2.61E-06
9.54	1.874	0.734	1.878	124	6.03E-03	7.69E-04	4.54E-07
19.25	1.861	0.722	1.868	271	2.73E-03	7.05E-04	1.88E-07
38.68	1.843	0.705	1.852	124	5.86E-03	4.88E-04	2.80E-07
75.68	1.818	0.682	1.831	103	6.90E-03	3.56E-04	2.40E-07
154.68	1.786	0.652	1.802	135	5.10E-03	2.13E-04	1.07E-07
308.19	1.744	0.613	1.765	124	5.33E-03	1.44E-04	7.52E-08
616.88	1.697	0.570	1.721	113	5.55E-03	8.01E-05	4.36E-08
1235.85	1.644	0.521	1.671	89	6.65E-03	4.51E-05	2.94E-08
2473.24	1.583	0.464	1.614	94	5.87E-03	2.59E-05	1.49E-08
1235.85	1.595	0.476	1.589				
308.19	1.606	0.486	1.601				
75.68	1.627	0.505	1.617				
19.25	1.653	0.529	1.640				
4.75	1.682	0.556	1.668				

Notes:

k calculated using ch based on σ_0 values.

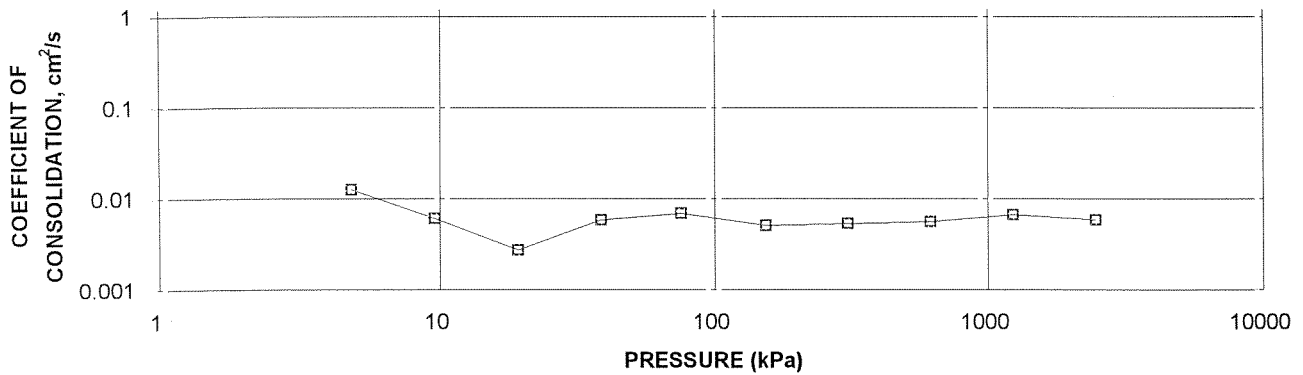
specimen vertically trimmed

SAMPLE DIMENSIONS AND PROPERTIES - FINAL

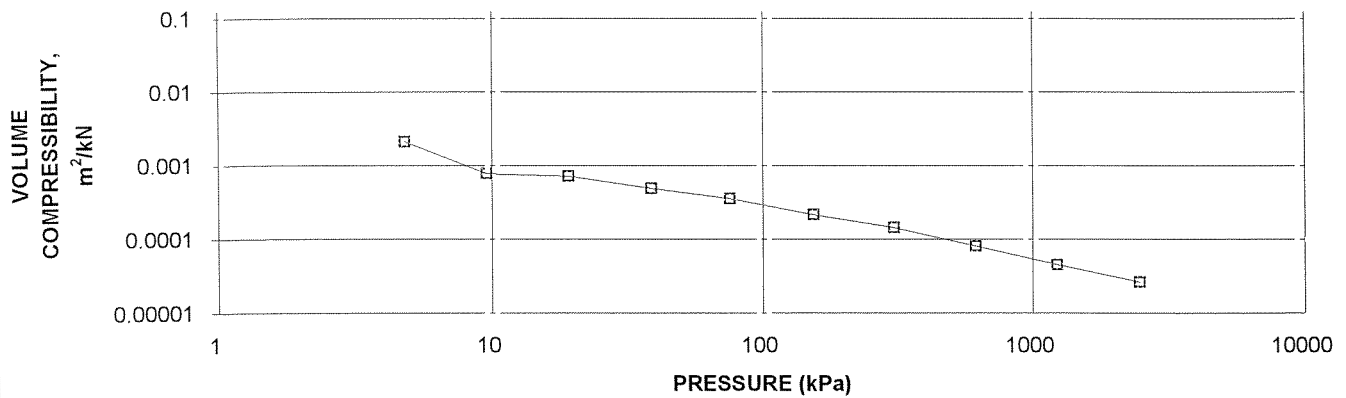
Sample Height, cm	1.68	Unit Weight, kN/m ³	21.16
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m ³	17.52
Area, cm ²	31.67	Specific Gravity, measured	2.78
Volume, cm ³	53.27	Solids Height, cm	1.081
Water Content, %	20.77	Volume of Solids, cm ³	34.23
Wet Mass, g	114.94	Volume of Voids, cm ³	19.03
Dry Mass, g	95.17		

OEDOMETER CONSOLIDATION SUMMARY

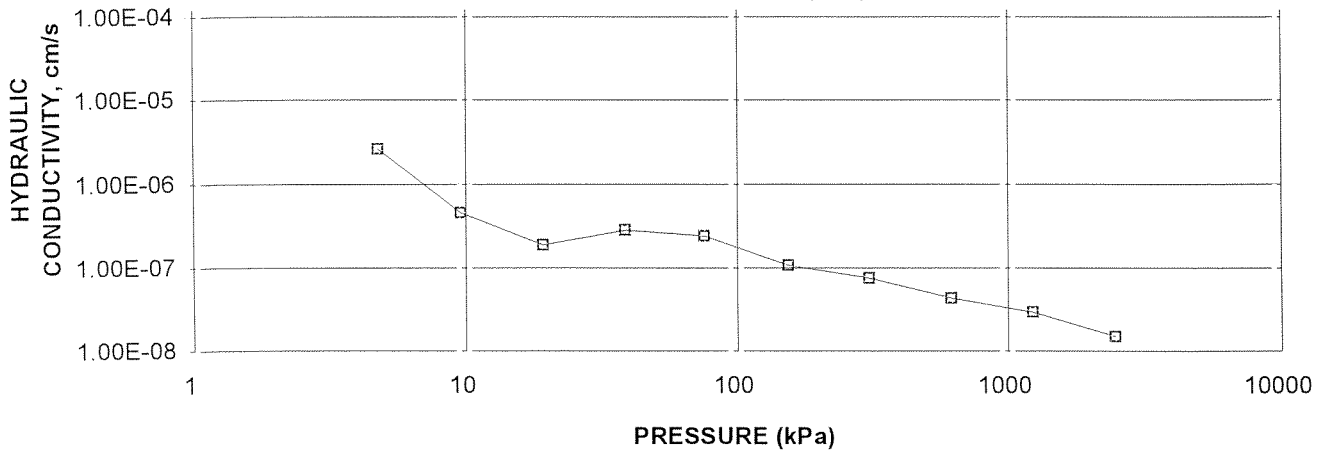
CONSOLIDATION TEST
CH cm^2/s VS PRESSURE (kPa)
BR9-4 SA 7 (VTO)



CONSOLIDATION TEST
MH m^2/kN vs PRESSURE (kPa)
BR9-4 SA 7 (VTO)



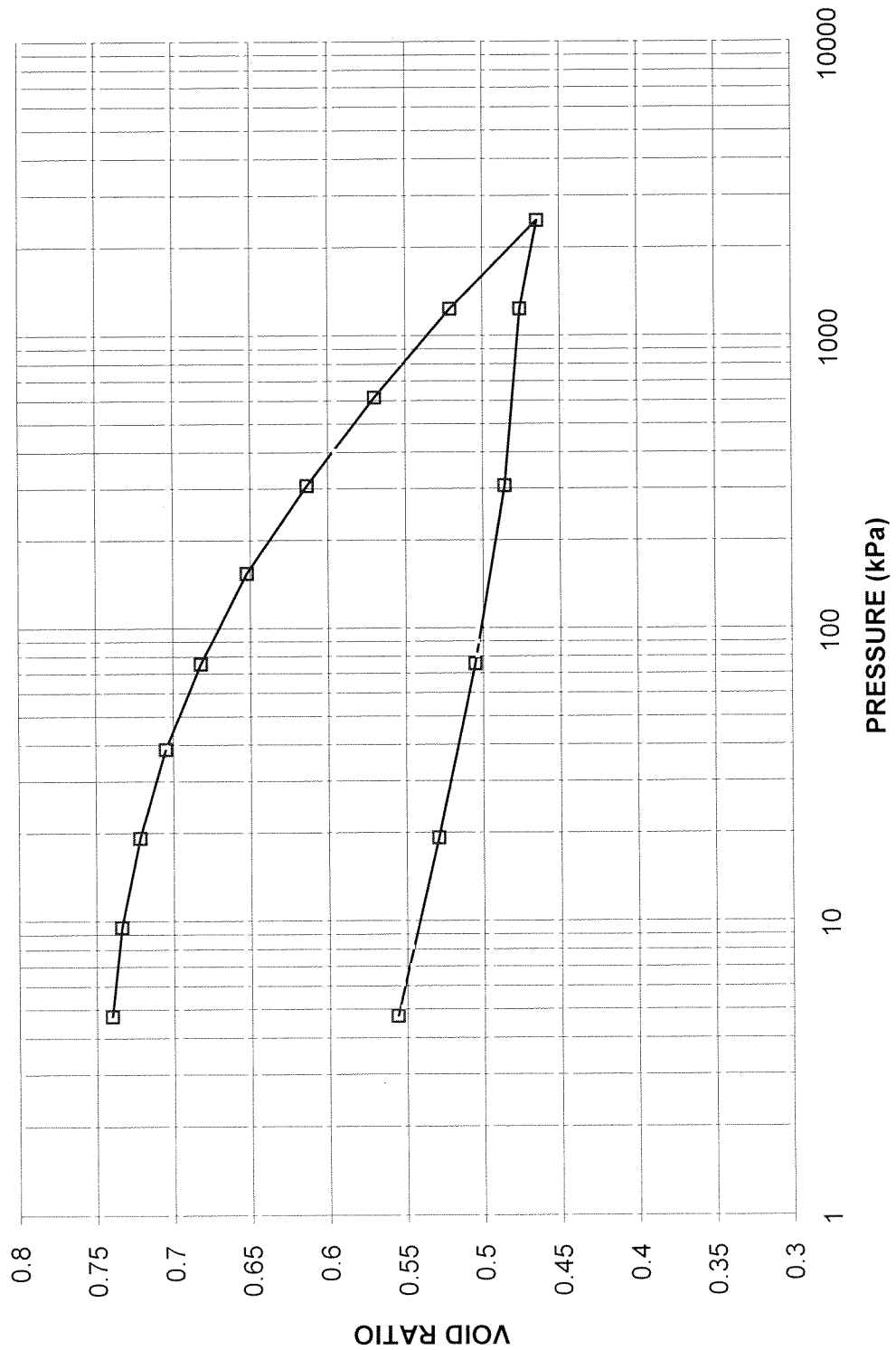
CONSOLIDATION TEST
HYDRAULIC CONDUCTIVITY vs PRESSURE
BR9-4 SA 7 (VTO)



**CONSOLIDATION TEST
VOID RATIO VS. LOG PRESSURE**

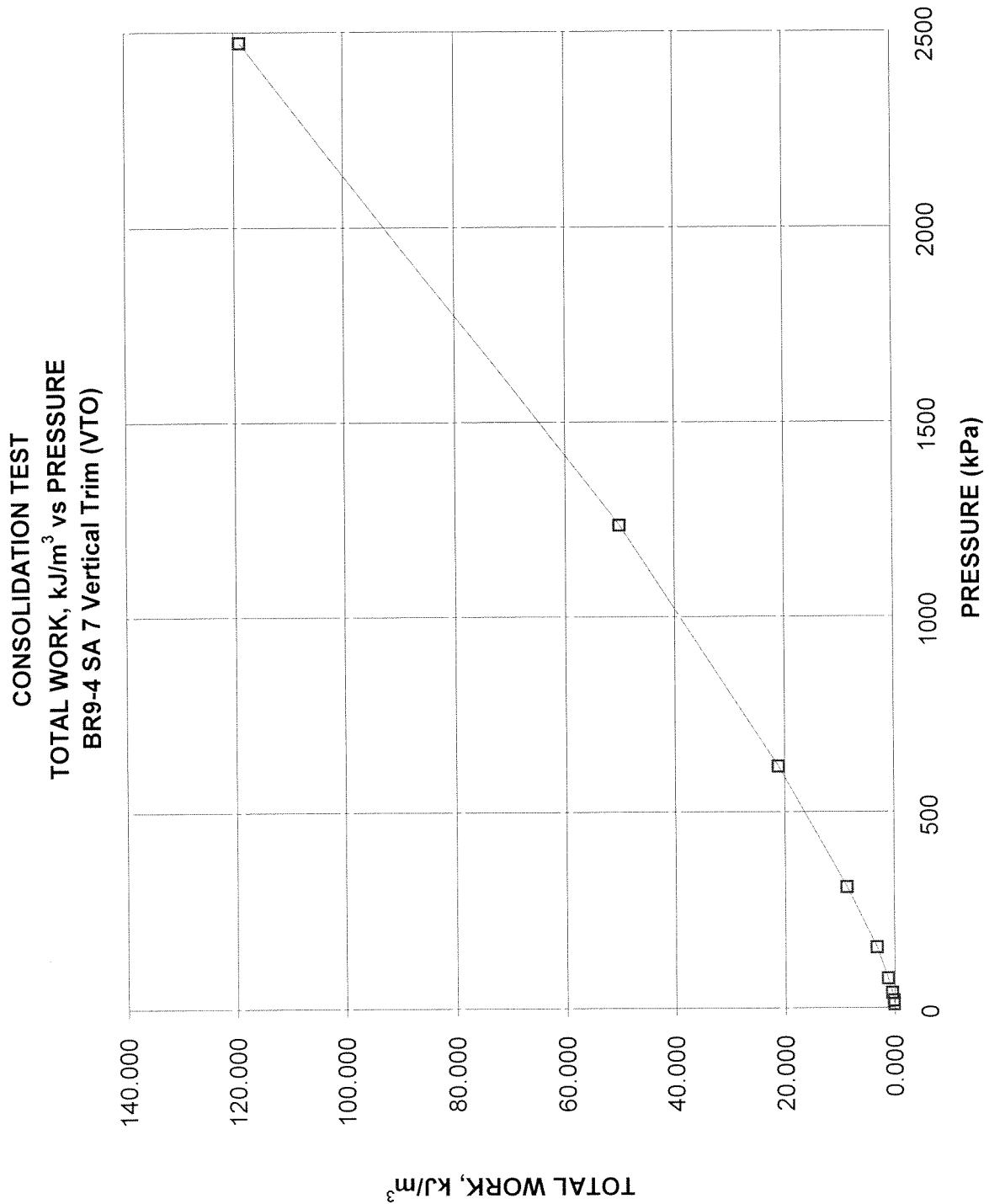
FIGURE A9

**CONSOLIDATION TEST
VOID RATIO vs PRESSURE
BR9-4 SA 7 Vertical Trim (VTO)**



CONSOLIDATION TEST
TOTAL WORK VS. PRESSURE

FIGURE A10



OEDOMETER CONSOLIDATION SUMMARY (HTO)

SAMPLE IDENTIFICATION

Project Number	021-1162	Sample Number	8
Borehole Number	BR 9-7	Sample Depth, m	4.6-5.2

TEST CONDITIONS

Test Type	Standard	Load Duration, hr	24
Oedometer Number	7		
Date Started	5/28/2004		
Date Completed	6/08/2004		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	1.92	Unit Weight, kN/m ³	20.16
Sample Diameter, cm	6.34	Dry Unit Weight, kN/m ³	15.91
Area, cm ²	31.61	Specific Gravity, measured	2.79
Volume, cm ³	60.53	Solids Height, cm	1.114
Water Content, %	26.71	Volume of Solids, cm ³	35.20
Wet Mass, g	124.43	Volume of Voids, cm ³	25.33
Dry Mass, g	98.2	Degree of Saturation, %	103.5

TEST COMPUTATIONS

Pressure kPa	Corr. Height cm	Void Ratio	Average Height cm	t ₉₀ sec	cv. cm ² /s	mv m ² /kN	k cm/s
0.00	1.915	0.720	1.915				
4.86	1.908	0.714	1.912	108	7.17E-03	7.52E-04	5.29E-07
9.52	1.904	0.710	1.906	113	6.82E-03	4.48E-04	2.99E-07
19.44	1.895	0.702	1.900	303	2.52E-03	4.74E-04	1.17E-07
38.71	1.881	0.689	1.888	232	3.26E-03	3.79E-04	1.21E-07
77.58	1.860	0.670	1.871	94	7.89E-03	2.82E-04	2.18E-07
154.95	1.824	0.638	1.842	158	4.55E-03	2.43E-04	1.08E-07
310.21	1.760	0.581	1.792	338	2.01E-03	2.15E-04	4.25E-08
619.80	1.693	0.520	1.727	197	3.21E-03	1.13E-04	3.55E-08
1239.36	1.621	0.456	1.657	356	1.64E-03	6.07E-05	9.72E-09
2479.16	1.549	0.391	1.585	271	1.97E-03	3.03E-05	5.84E-09
1239.36	1.558	0.399	1.554				
310.21	1.588	0.426	1.573				
77.58	1.626	0.460	1.607				
19.44	1.674	0.503	1.650				
4.86	1.702	0.529	1.688				

Notes:

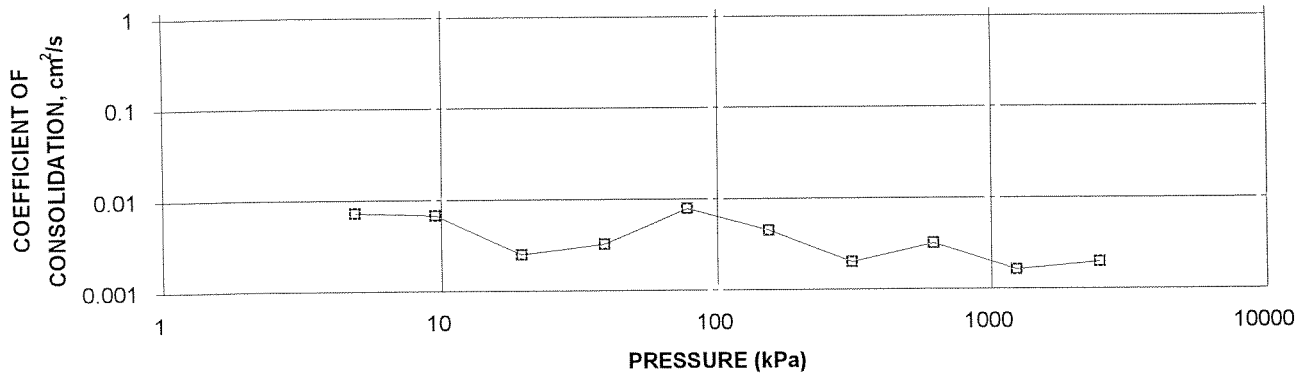
k calculated using cv based on t₉₀ values.
specimen horizontally trimmed

SAMPLE DIMENSIONS AND PROPERTIES - FINAL

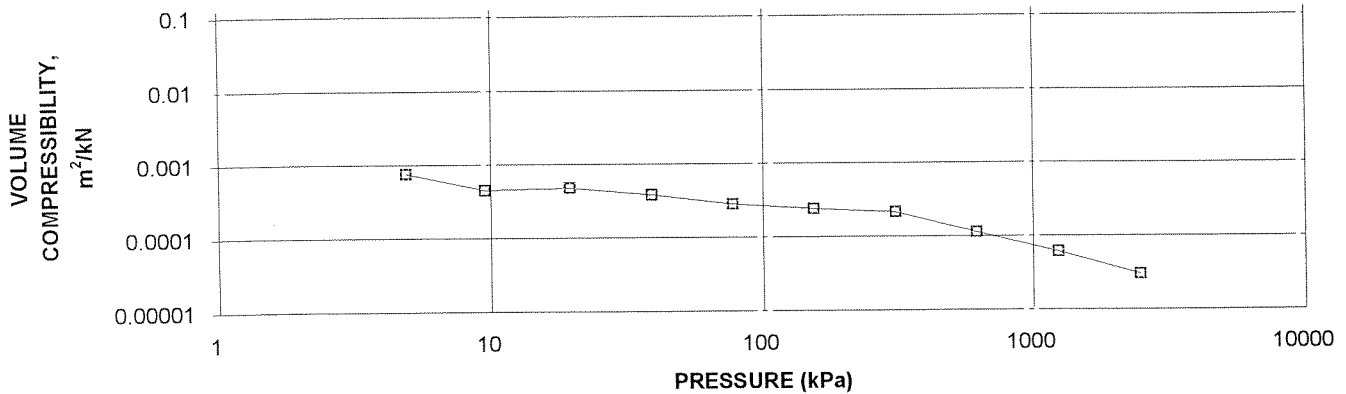
Sample Height, cm	1.70	Unit Weight, kN/m ³	21.70
Sample Diameter, cm	6.34	Dry Unit Weight, kN/m ³	17.90
Area, cm ²	31.61	Specific Gravity, measured	2.79
Volume, cm ³	53.80	Solids Height, cm	1.114
Water Content, %	21.24	Volume of Solids, cm ³	35.20
Wet Mass, g	119.06	Volume of Voids, cm ³	18.60
Dry Mass, g	98.2		

OEDOMETER CONSOLIDATION SUMMARY

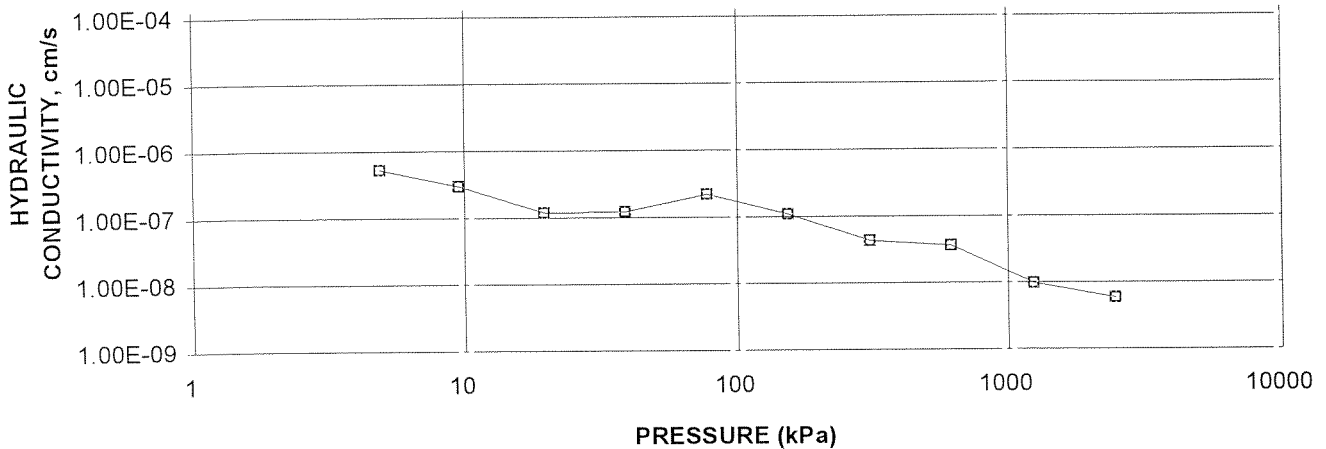
CONSOLIDATION TEST
CV cm²/s VS PRESSURE (kPa)
BR 9-7 SA 8 (HTO)



CONSOLIDATION TEST
MV m²/kN vs PRESSURE (kPa)
BR 9-7 SA 8 (HTO)



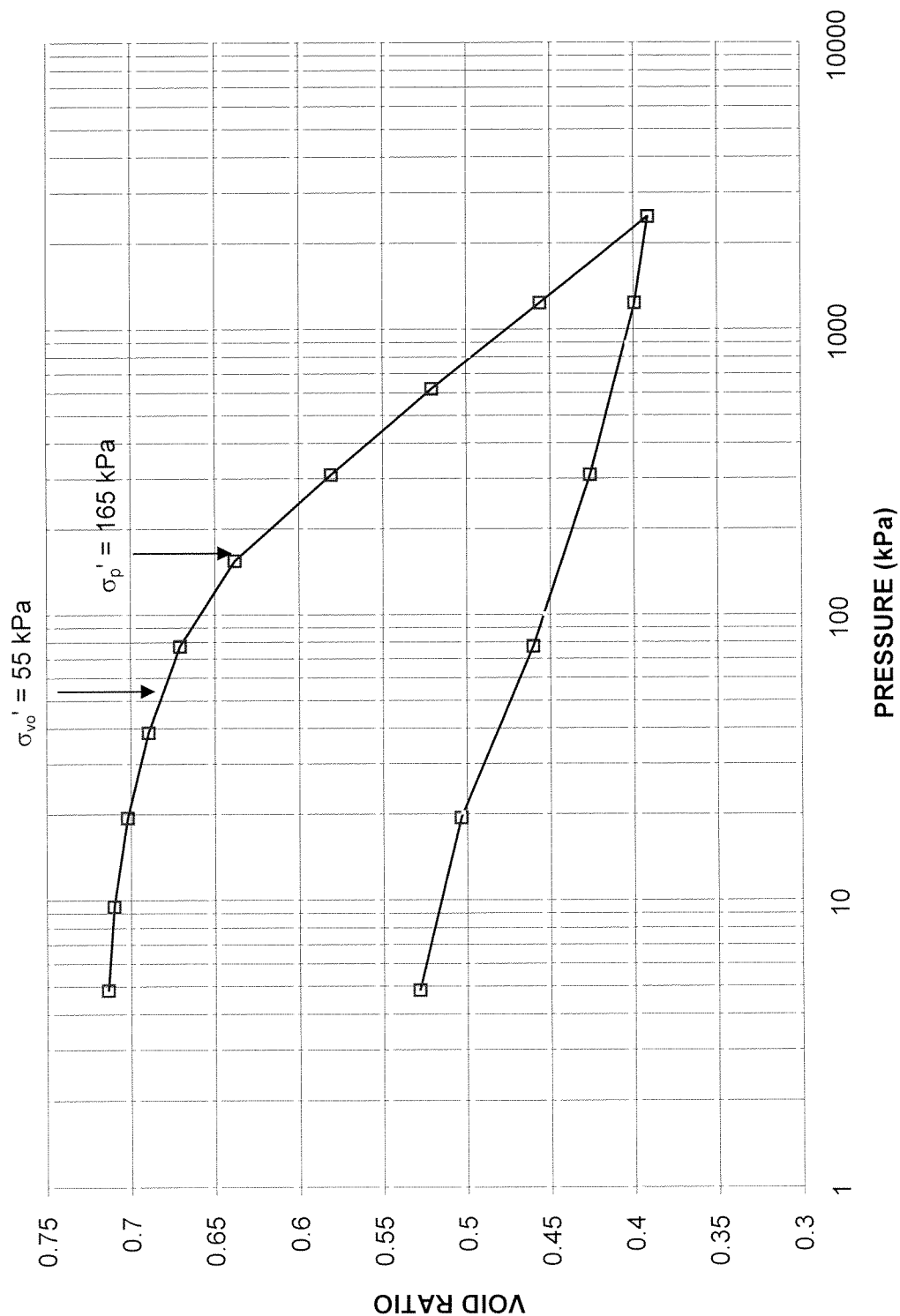
CONSOLIDATION TEST
HYDRAULIC CONDUCTIVITY vs PRESSURE
BR 9-7 SA 8 (HTO)



CONSOLIDATION TEST
VOID RATIO VS. LOG PRESSURE

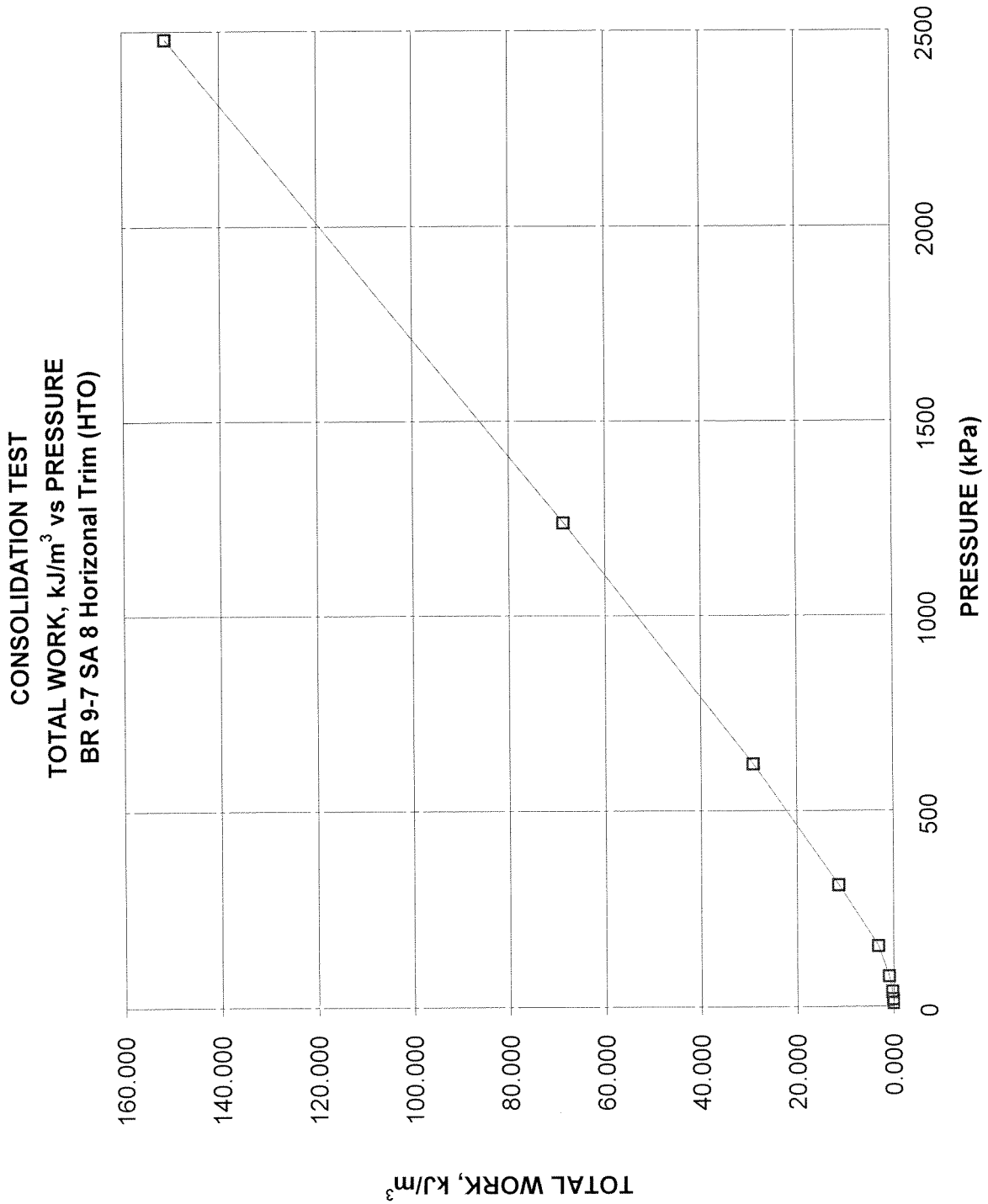
FIGURE A11

CONSOLIDATION TEST
VOID RATIO vs. PRESSURE
BR 9-7 SA 8 Horizontal Trim (HTO)



**CONSOLIDATION TEST
TOTAL WORK VS. PRESSURE**

FIGURE A12



OEDOMETER CONSOLIDATION SUMMARY (VTO)

SAMPLE IDENTIFICATION

Project Number	021-1162	Sample Number	8
Borehole Number	BR 9-7	Sample Depth, m	4.6-5.2

TEST CONDITIONS

Test Type	Standard	Load Duration, hr	24
Oedometer Number	7		
Date Started	5/28/2004		
Date Completed	6/08/2004		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	1.90	Unit Weight, kN/m ³	20.14
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m ³	15.92
Area, cm ²	31.65	Specific Gravity, measured	2.79
Volume, cm ³	60.13	Solids Height, cm	1.105
Water Content, %	26.50	Volume of Solids, cm ³	34.99
Wet Mass, g	123.48	Volume of Voids, cm ³	25.15
Dry Mass, g	97.61	Degree of Saturation, %	102.9

TEST COMPUTATIONS

Pressure kPa	Corr. Height cm	Void Ratio	Average Height cm	t ₉₀ sec	c _h cm ² /s	m _h m ² /kN	k cm/s
0.00	1.900	0.719	1.900				
4.83	1.894	0.713	1.897	103	7.41E-03	6.54E-04	4.75E-07
9.46	1.888	0.708	1.891	135	5.62E-03	6.82E-04	3.75E-07
19.51	1.874	0.695	1.881	454	1.65E-03	7.33E-04	1.19E-07
38.91	1.854	0.677	1.864	540	1.36E-03	5.43E-04	7.25E-08
77.57	1.824	0.650	1.839	540	1.33E-03	4.08E-04	5.31E-08
154.88	1.781	0.611	1.803	375	1.84E-03	2.93E-04	5.27E-08
309.92	1.729	0.564	1.755	413	1.58E-03	1.77E-04	2.74E-08
619.67	1.667	0.508	1.698	287	2.13E-03	1.05E-04	2.20E-08
1238.29	1.601	0.448	1.634	211	2.68E-03	5.62E-05	1.48E-08
2475.97	1.531	0.385	1.566	158	3.29E-03	2.98E-05	9.60E-09
1238.29	1.541	0.394	1.536				
309.92	1.568	0.418	1.555				
77.57	1.606	0.453	1.587				
19.51	1.639	0.483	1.623				
4.83	1.687	0.526	1.663				

Notes:

k calculated using c_h based on t₉₀ values.

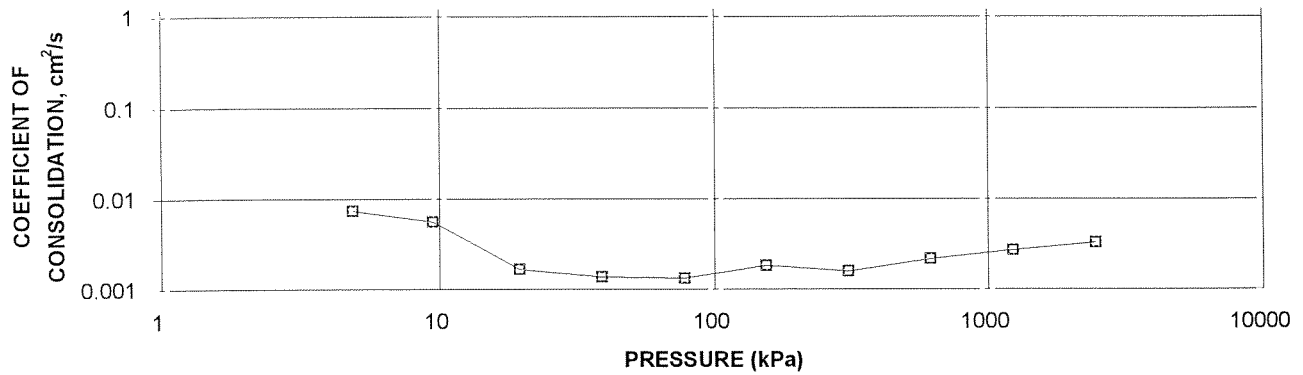
specimen vertically trimmed

SAMPLE DIMENSIONS AND PROPERTIES - FINAL

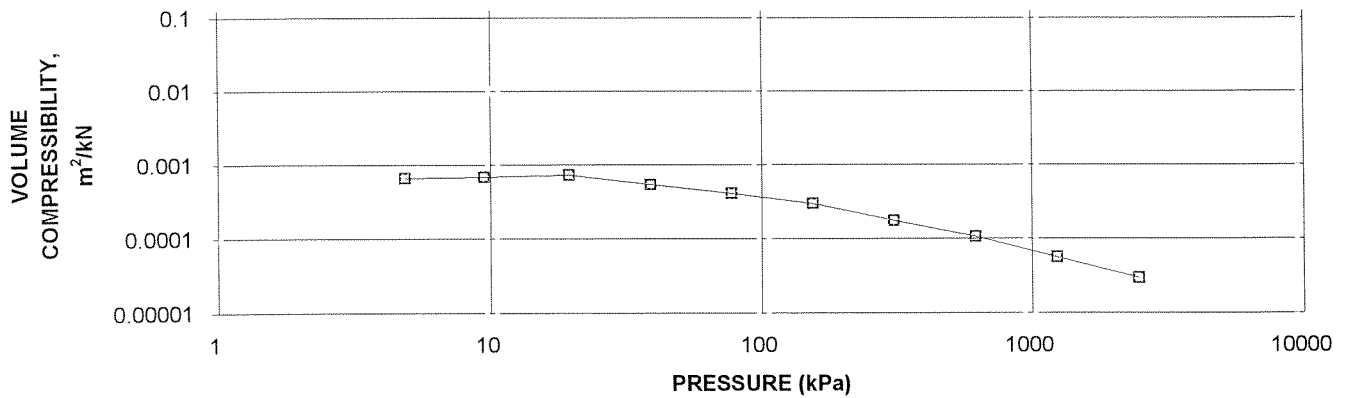
Sample Height, cm	1.69	Unit Weight, kN/m ³	21.67
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m ³	17.93
Area, cm ²	31.65	Specific Gravity, measured	2.79
Volume, cm ³	53.39	Solids Height, cm	1.105
Water Content, %	20.89	Volume of Solids, cm ³	34.99
Wet Mass, g	118.00	Volume of Voids, cm ³	18.41
Dry Mass, g	97.61		

OEDOMETER CONSOLIDATION SUMMARY

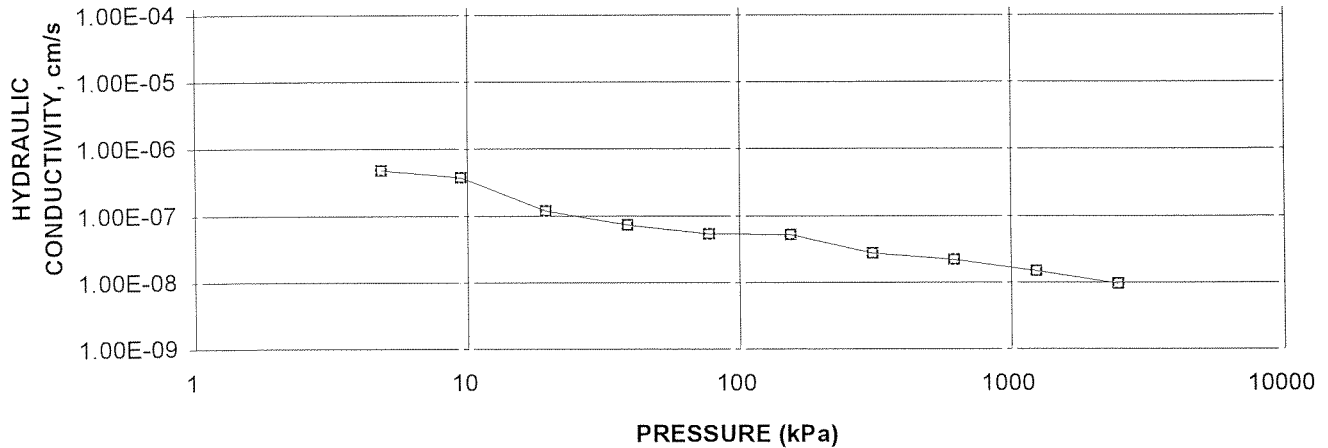
CONSOLIDATION TEST
CH cm^2/s VS PRESSURE (kPa)
BR9-7 SA 8 (VTO)



CONSOLIDATION TEST
MH m^2/kN vs PRESSURE (kPa)
BR9-7 SA 8 (VTO)



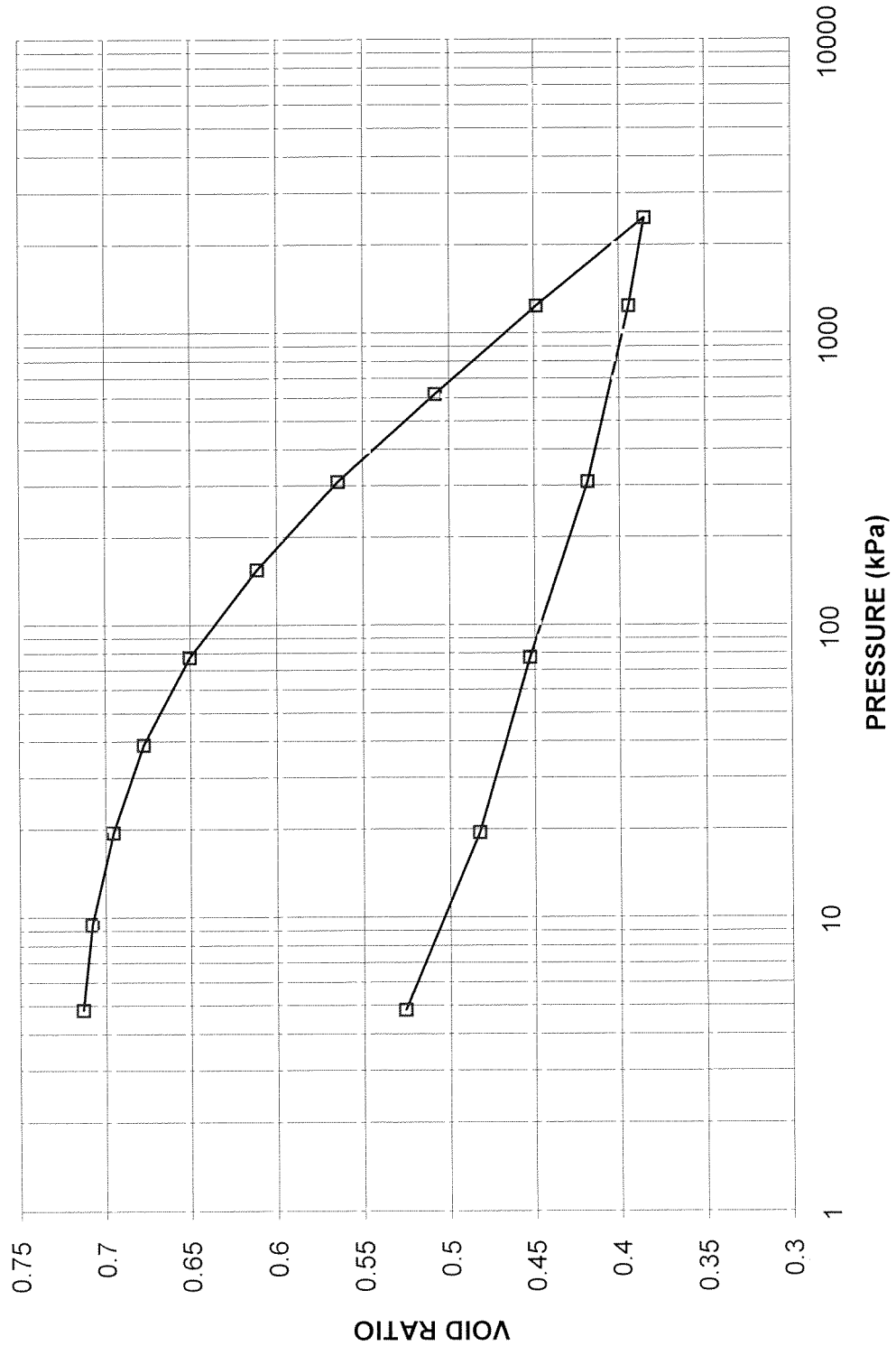
CONSOLIDATION TEST
HYDRAULIC CONDUCTIVITY vs PRESSURE
BR9-7 SA 8 (VTO)



CONSOLIDATION TEST
VOID RATIO VS. LOG PRESSURE

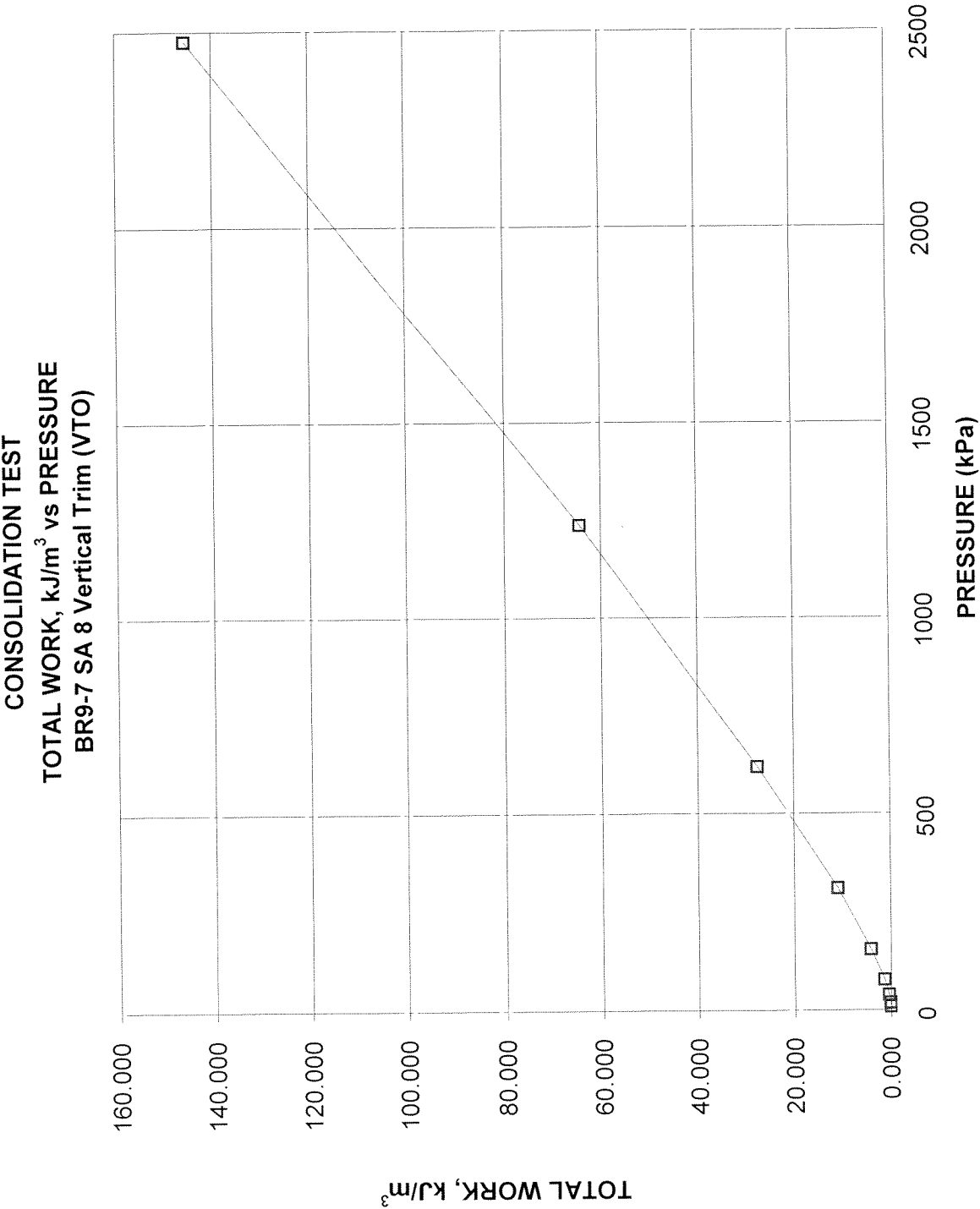
FIGURE A13

CONSOLIDATION TEST
VOID RATIO vs. PRESSURE
BR9-7 SA 8 Vertical Trim (VTO)



CONSOLIDATION TEST
TOTAL WORK VS. PRESSURE

FIGURE A14



OEDOMETER CONSOLIDATION SUMMARY

PROJECT	021-1162		SPECIFIC GRAVITY		2.70 assumed		DATE STARTED	98-10-16
BOREHOLE	RSER-6		AREA(mm ²)		3161.94		DATE COMPLETED	98-10-31
SAMPLE	6		SOLIDS HT.2HS		11.620			
DEPTH, m			DRY WEIGHT, g		99.02			
	Corr.	Void	Average			cv.	k	mv
Load	Height	Ratio	Height	t ₉₀	t ₅₀	t ₉₀		
kPa	mm		mm	sec	sec	cm ² /s	cm/s	m ² /kN
0.00	19.050	0.639	19.050					
9.67	18.762	0.615	18.906	18		4.21E-02	6.45E-06	1.56E-03
19.34	18.622	0.603	18.692	9		8.23E-02	6.13E-06	7.60E-04
38.69	18.395	0.583	18.509	10		7.26E-02	4.38E-06	6.16E-04
19.34	18.414	0.585	18.405					5.16E-05
9.67	18.460	0.589	18.437					2.50E-04
19.34	18.436	0.587	18.448	16		4.51E-02	5.76E-07	1.30E-04
38.69	18.375	0.581	18.406	23		3.12E-02	5.07E-07	1.66E-04
61.90	18.227	0.569	18.301	5		1.42E-01	4.66E-06	3.35E-04
77.37	18.137	0.561	18.182	30		2.34E-02	6.99E-07	3.05E-04
99.04	18.027	0.551	18.082	34		2.04E-02	5.32E-07	2.67E-04
125.34	17.899	0.540	17.963	41		1.67E-02	4.18E-07	2.55E-04
154.74	17.808	0.533	17.854	25		2.70E-02	4.30E-07	1.62E-04
179.50	17.709	0.524	17.759	51		1.31E-02	2.70E-07	2.10E-04
309.49	17.353	0.493	17.531	13		5.01E-02	7.06E-07	1.44E-04
618.97	16.898	0.454	17.126	128		4.86E-03	3.67E-08	7.72E-05
1237.94	16.399	0.411	16.649	70		8.39E-03	3.48E-08	4.23E-05
2475.89	15.815	0.361	16.107	82		6.71E-03	1.63E-08	2.48E-05
1237.94	15.930	0.371	15.873					
309.49	16.092	0.385	16.011					
77.37	16.281	0.401	16.187					
38.69	16.399	0.411	16.340					
9.67	16.553	0.425	16.476					

Notes:

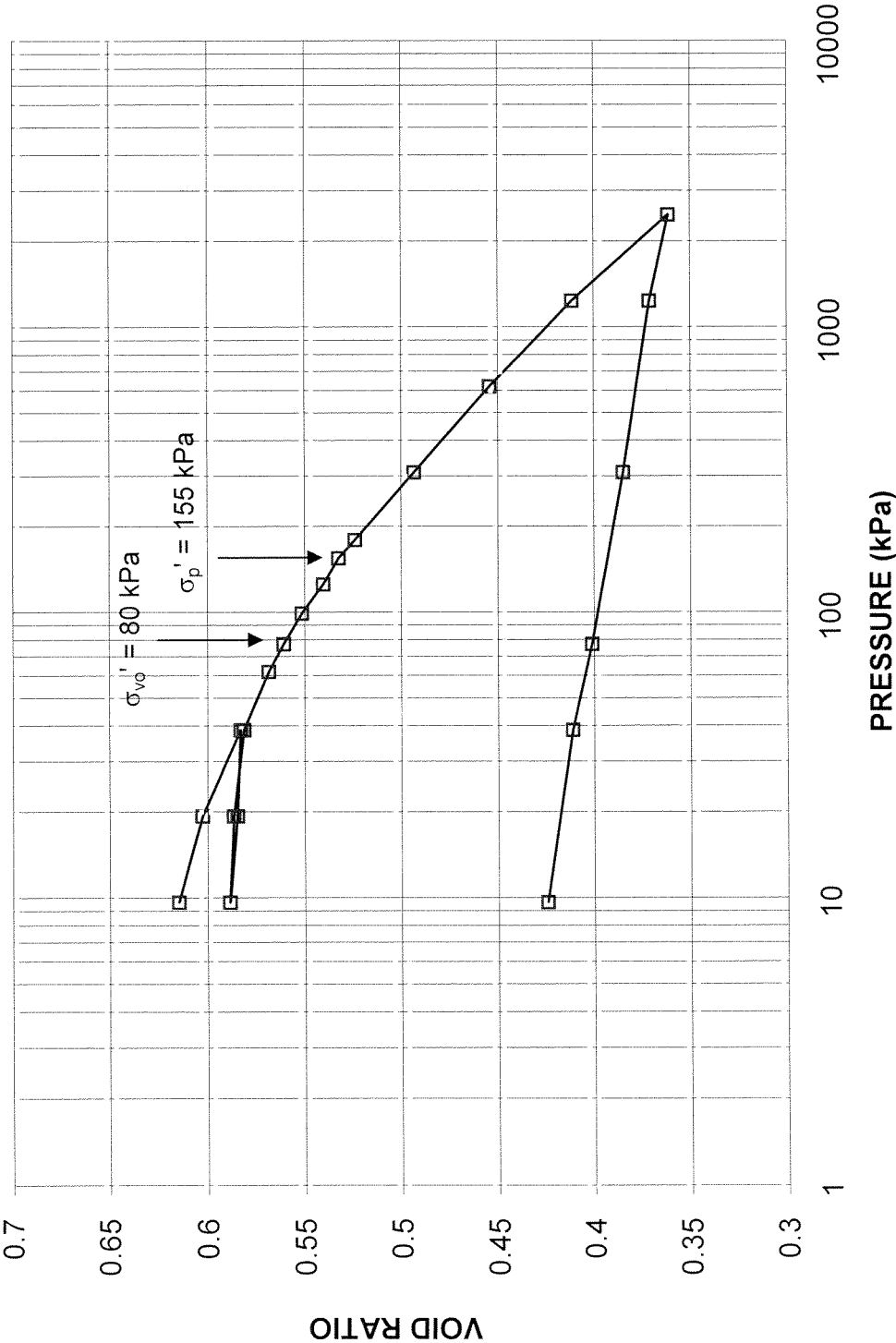
k calculated using Cv based on t90 values.

Water Content %, initial	24.0		
Water Content %, final	17.6		
Original Volume, cc	60.23		
Volume of Solids, cc	36.74		
Volume of Voids, cc	23.49	Unit Weight, kN/m ³	20.00
Degree of Saturation, %	101.2	Dry Unit Weight, kN/m ³	16.13

CONSOLIDATION TEST
VOID RATIO VS. LOG PRESSURE

FIGURE A15

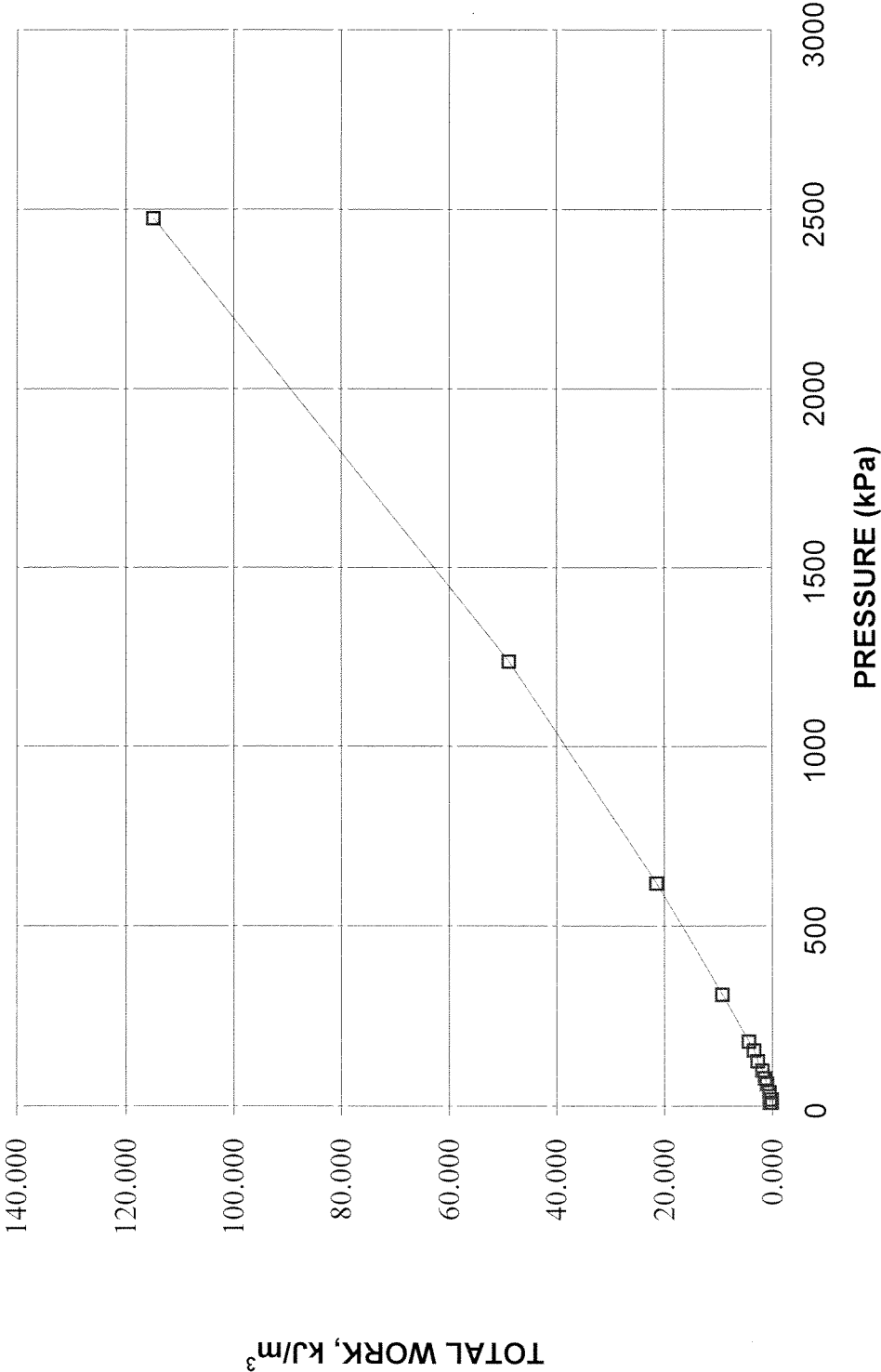
CONSOLIDATION TEST
VOID RATIO vs PRESSURE
RSER-6 SA 6



**CONSOLIDATION TEST
TOTAL WORK vs PRESSURE**

FIGURE A16

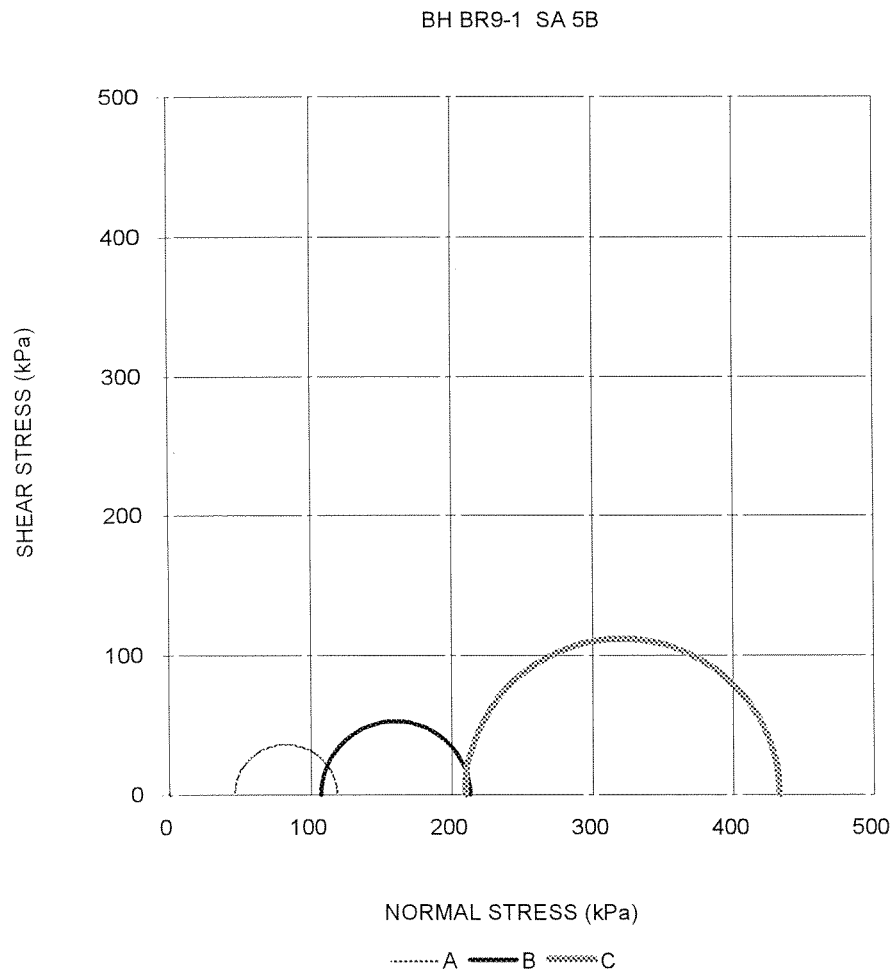
**CONSOLIDATION TEST
TOTAL WORK, kJ/m^3 vs PRESSURE
RSER-6 SA6**



CONSOLIDATED UNDRAINED TRIAXIAL WITH PORE PRESSURE MEASUREMENTS SHEET 1 OF 4			FIGURE
TEST STAGE	A	B	C
BOREHOLE NUMBER	BR9-1	BR9-1	BR9-1
SAMPLE NUMBER	5B	5B	5B
SPECIMEN DIAMETER, cm	4.93	4.98	5.00
SPECIMEN HEIGHT, cm	10.16	10.20	10.18
WATER CONTENT BEFORE CONSOLIDATION, %	24.8	25.1	22.5
CELL PRESSURE, σ_3 , kPa	210.0	425.0	505.0
BACK PRESSURE, kPa	135.0	275.0	205.0
PORE PRESSURE PARAMETER "B"	0.96	0.97	0.86
CONSOLIDATION PRESSURE, σ_c , kPa	75.0	150.0	300.0
VOLUMETRIC STRAIN DURING CONSOLIDATION, %	2.3	3.5	6.0
WATER CONTENT AFTER CONSOLIDATION, %	23.5	23.0	19.0
AVERAGE RATE OF STRAIN, %/hr	0.5	0.5	0.5
TIME TO FAILURE, DAYS	2	2	2
WATER CONTENT AFTER TEST, %	25.0	24.3	19.6
MAX. DEVIATOR STRESS, $(\sigma_1 - \sigma_3)$, kPa	72.7	105.8	223.9
AXIAL STRAIN AT $(\sigma_1 - \sigma_3)$ MAXIMUM, %	1.8	12.0	10.0
MAX EFFECTIVE PRINCIPAL STRESS			
RATIO, (σ_1 / σ_3) MAXIMUM	3.1	3.1	2.8
DEVIATOR STRESS AT (σ_1 / σ_3) MAXIMUM, kPa	72.0	105.4	222.8
AXIAL STRAIN AT (σ_1 / σ_3) MAXIMUM, %	10.4	11.2	9.1
PORE PRESSURE PARAMETER, Af, AT $(\sigma_1 - \sigma_3)$ MAXIMUM	0.47	0.93	0.79
PORE PRESSURE PARAMETER, Af, AT (σ_1 / σ_3) MAXIMUM	0.56	0.93	0.80
NATURAL WATER CONTENT, %	24.0	23.9	21.5
DRY DENSITY, Mg/m ³	1.64	1.64	1.70
FILTER DRAINS USED, y/n	y	y	y
TEST NOTES:			
CHANGED RATE OF STRAIN, %/hr	-	-	-
AXIAL STRAIN WHERE RATE OF STRAIN WAS CHANGED, %	-	-	-
FAILURE PLANE NUMBER	-	-	1.0
ANGLE OF FAILURE, DEGREES	-	-	62.0
DATE: September, 2003			
Project No. 021-1162	Golder Associates		

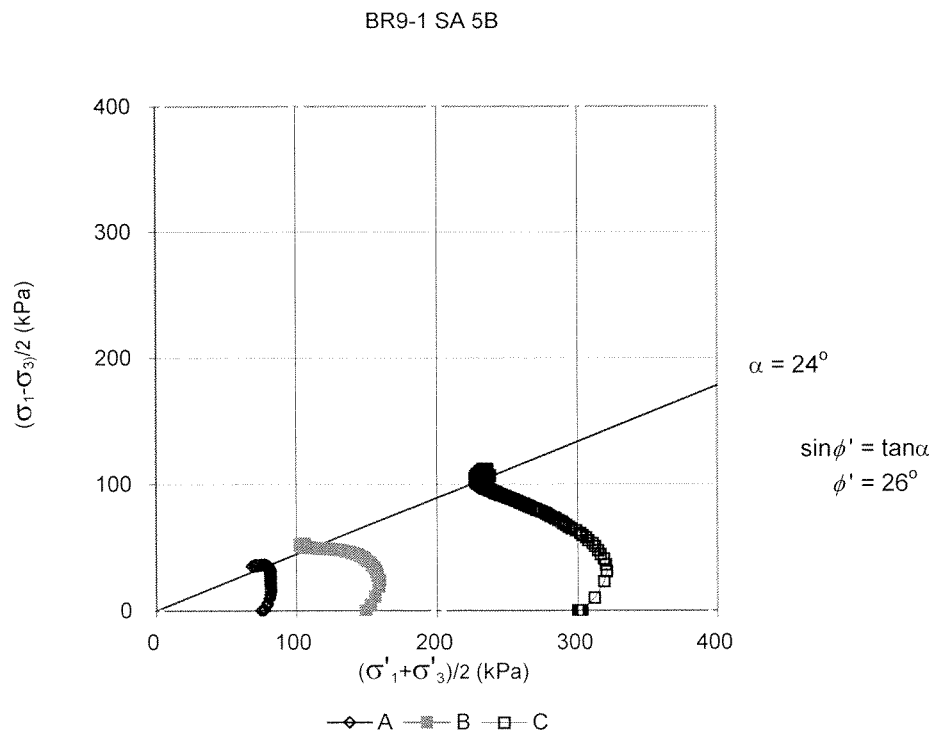
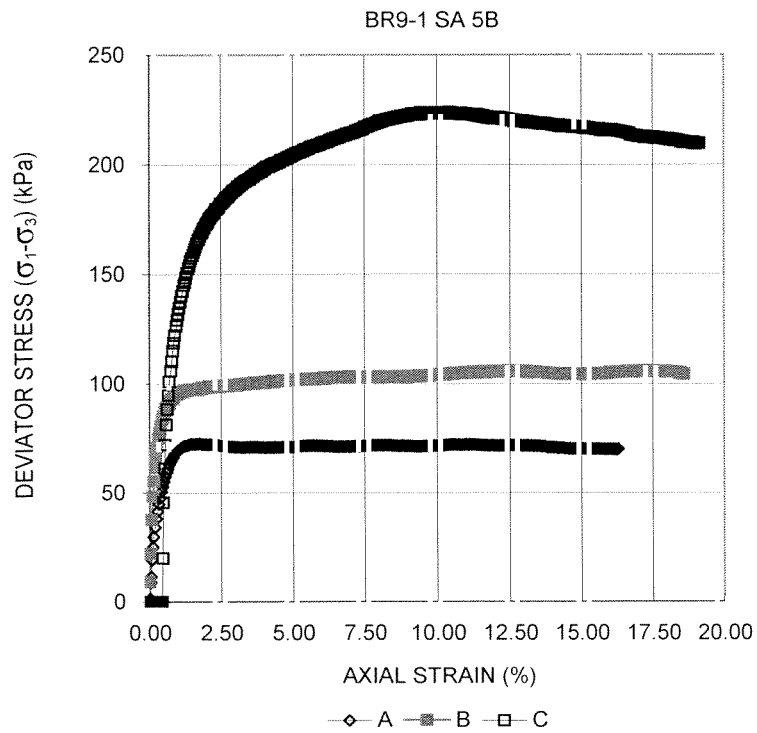
CONSOLIDATED UNDRAINED TRIAXIAL
WITH PORE PRESSURE MEASUREMENTS
SHEET 2 OF 4

FIGURE A17



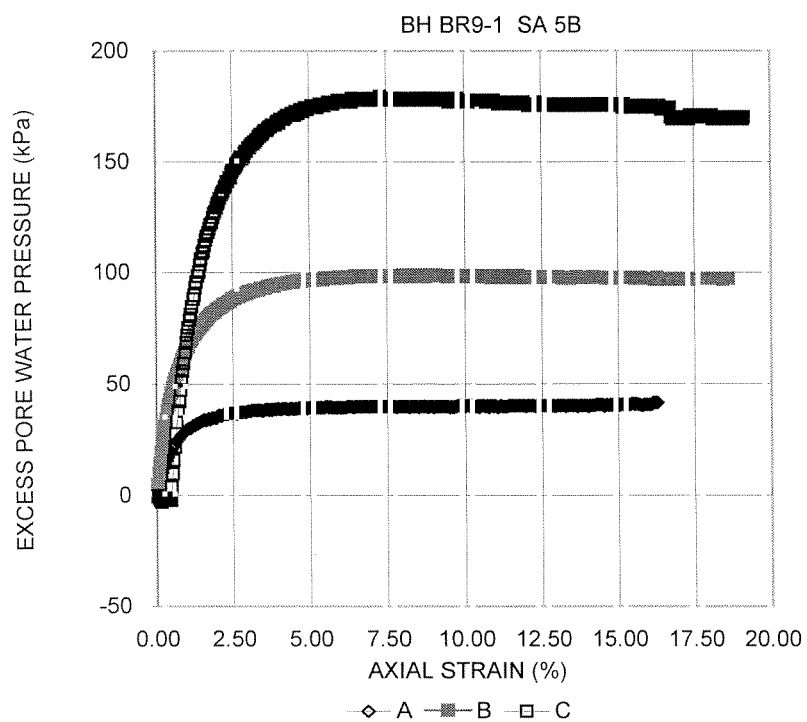
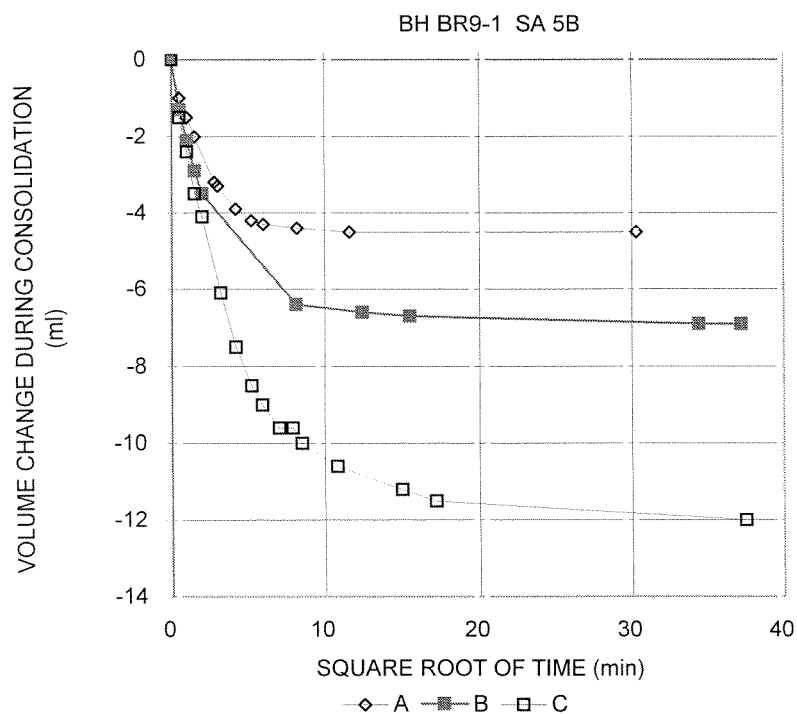
CONSOLIDATED UNDRAINED TRIAXIAL
WITH PORE PRESSURE MEASUREMENTS
SHEET 3 OF 4

FIGURE A17



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

FIGURE A17



**CONSOLIDATED UNDRAINED TRIAXIAL
WITH PORE PRESSURE MEASUREMENTS
SHEET 1 OF 4**

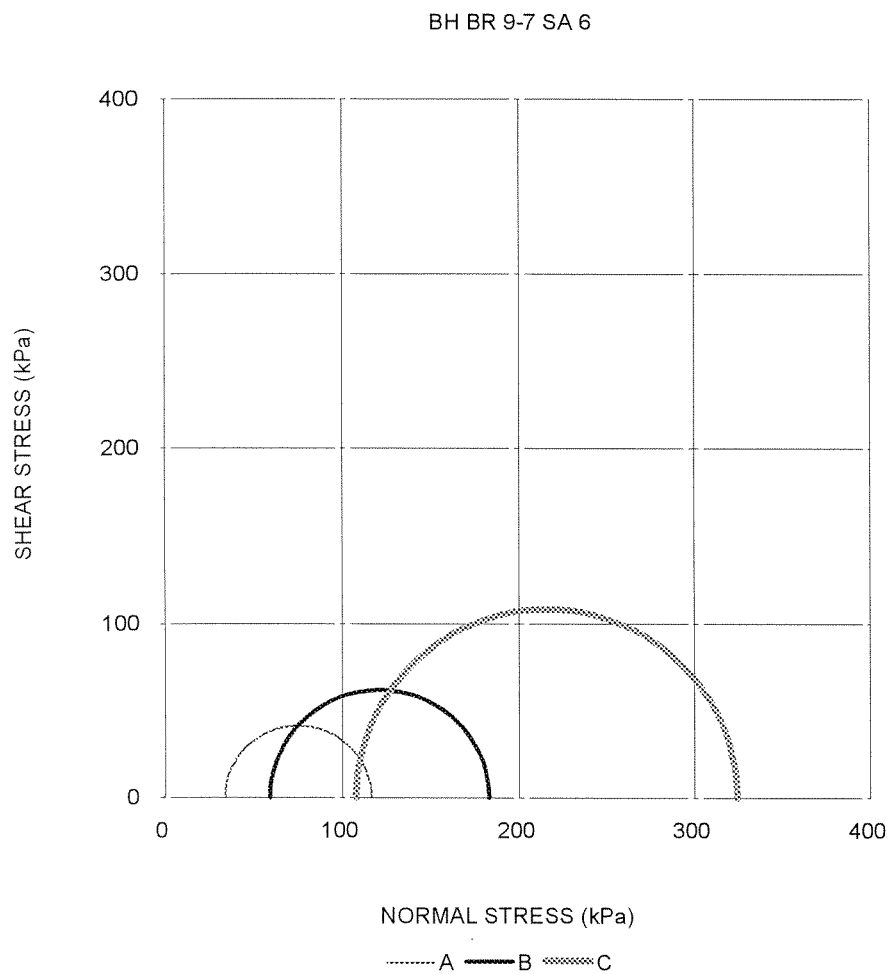
FIGURE A18

TEST STAGE	A	B	C
BOREHOLE NUMBER	BR9-7	BR9-7	BR9-7
SAMPLE NUMBER	6	6	6
SPECIMEN DIAMETER, cm	5.01	5.03	4.98
SPECIMEN HEIGHT, cm	10.12	10.11	10.08
WATER CONTENT BEFORE CONSOLIDATION, %	29.2	29.8	29.6
CELL PRESSURE, σ_3 , kPa	255.0	330.0	455.0
BACK PRESSURE, kPa	205.0	205.0	205.0
PORE PRESSURE PARAMETER "B"	0.96	0.96	0.97
CONSOLIDATION PRESSURE, σ_c , kPa	50.0	125.0	250.0
VOLUMETRIC STRAIN DURING CONSOLIDATION, %	1.8	4.2	8.3
WATER CONTENT AFTER CONSOLIDATION, %	28.1	27.1	24.4
AVERAGE RATE OF STRAIN, %/hr	0.5	0.5	0.5
TIME TO FAILURE, DAYS	1	1	1
WATER CONTENT AFTER TEST, %	26.6	26.9	24.4
MAX. DEVIATOR STRESS, $(\sigma_1 - \sigma_3)$, kPa	77.5	124.0	216.6
AXIAL STRAIN AT $(\sigma_1 - \sigma_3)$ MAXIMUM, %	2.7	9.2	13.5
MAX EFFECTIVE PRINCIPAL STRESS			
RATIO, (σ_1 / σ_3) MAXIMUM	3.8	3.1	3.0
DEVIATOR STRESS AT (σ_1 / σ_3) MAXIMUM, kPa	76.2	123.5	214.9
AXIAL STRAIN AT (σ_1 / σ_3) MAXIMUM, %	2.2	7.7	11.3
PORE PRESSURE PARAMETER, A_f , AT $(\sigma_1 - \sigma_3)$ MAXIMUM	0.28	0.53	0.66
PORE PRESSURE PARAMETER, A_f , AT (σ_1 / σ_3) MAXIMUM	0.29	0.54	0.67
NATURAL WATER CONTENT, %	27.9	28.6	27.7
DRY DENSITY, Mg/m ³	1.57	1.55	1.57
FILTER DRAINS USED, y/n	y	y	y
TEST NOTES:			
CHANGED RATE OF STRAIN, %/hr	-	-	-
AXIAL STRAIN WHERE RATE OF STRAIN WAS CHANGED, %	-	-	-
FAILURE PLANE NUMBER	bulged	bulged	bulged
ANGLE OF FAILURE, DEGREES	-	-	-

DATE: 06/11/2004

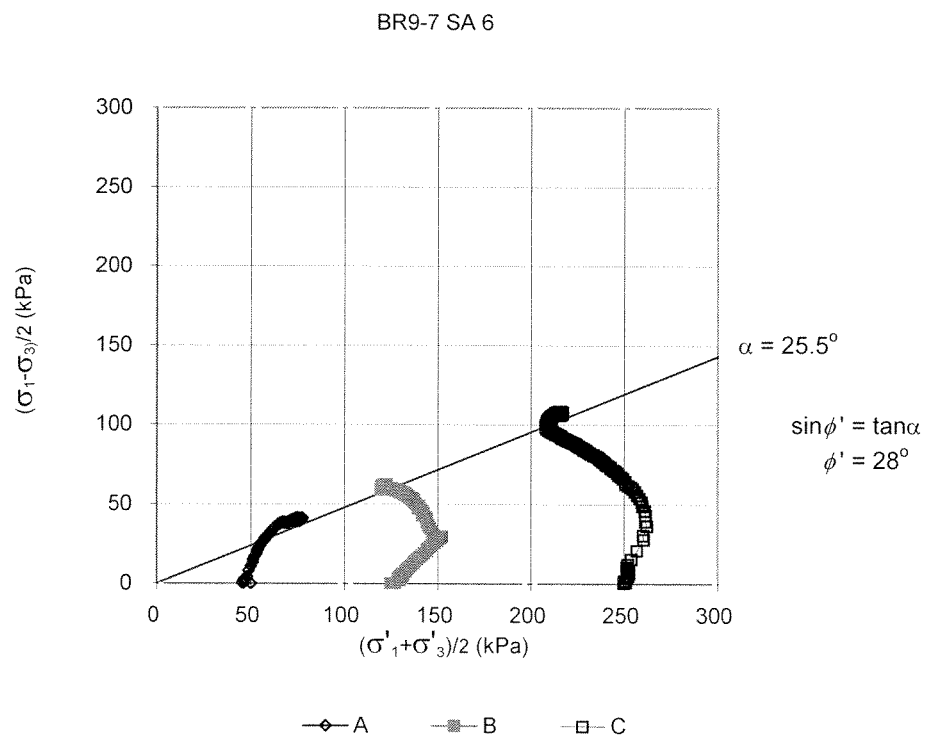
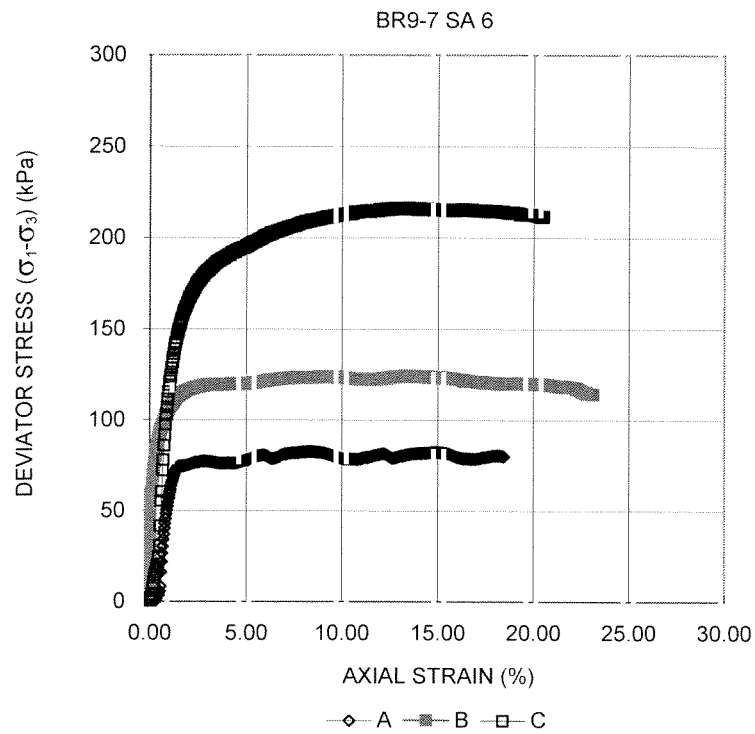
CONSOLIDATED UNDRAINED TRIAXIAL
WITH PORE PRESSURE MEASUREMENTS
SHEET 2 OF 4

FIGURE A18



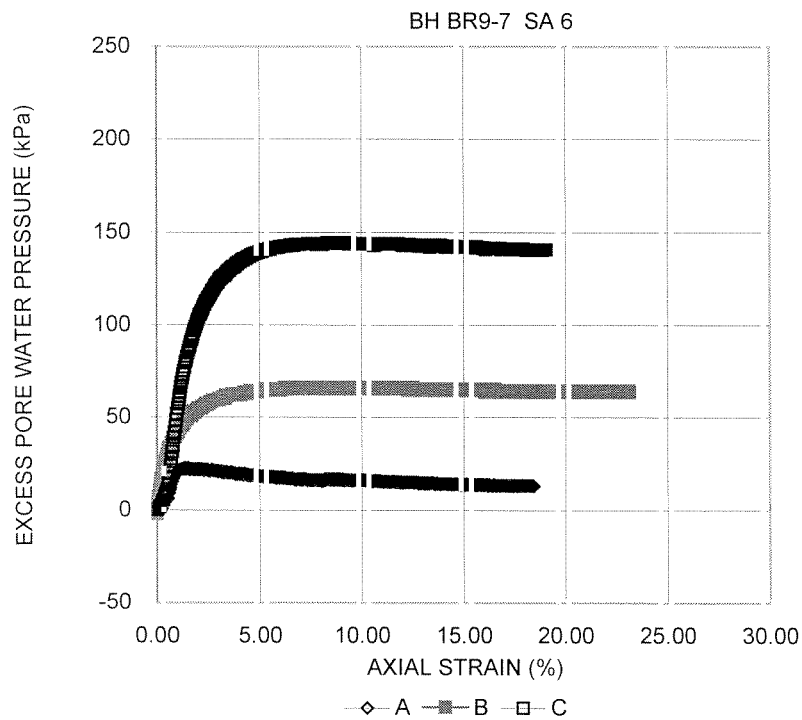
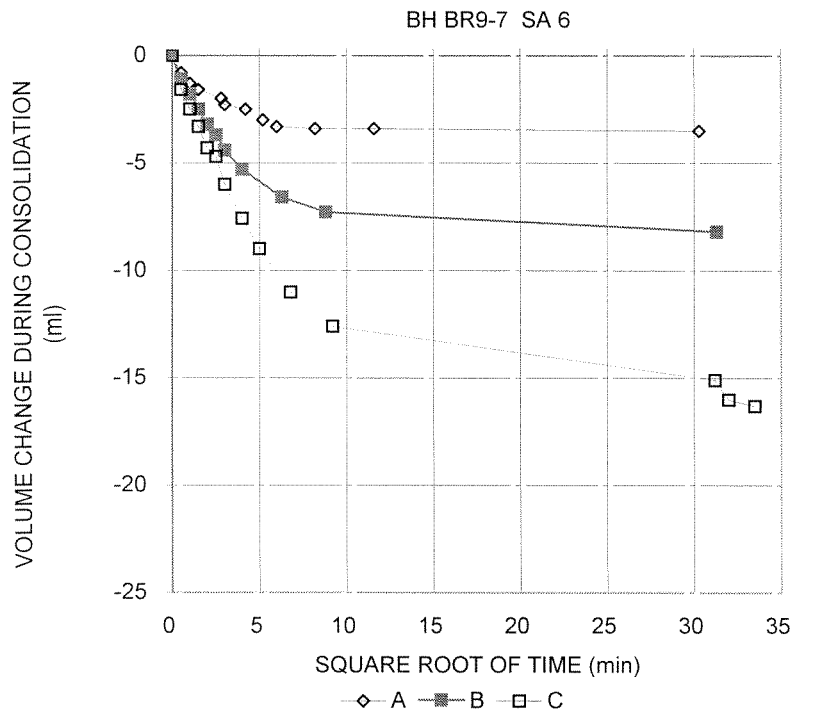
CONSOLIDATED UNDRAINED TRIAXIAL
WITH PORE PRESSURE MEASUREMENTS
SHEET 3 OF 4

FIGURE A18



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

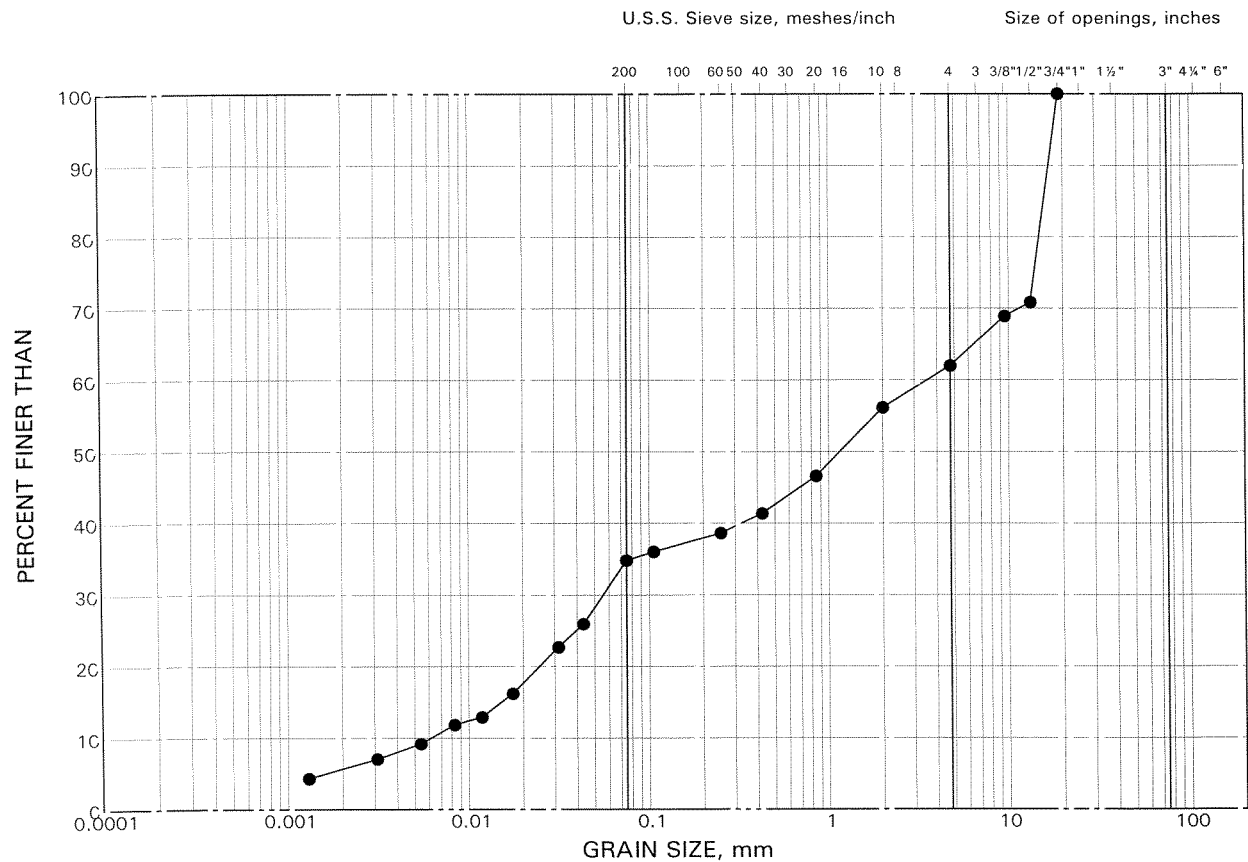
FIGURE A18



GRAIN SIZE DISTRIBUTION

Sandy Silt and Gravel (Till)

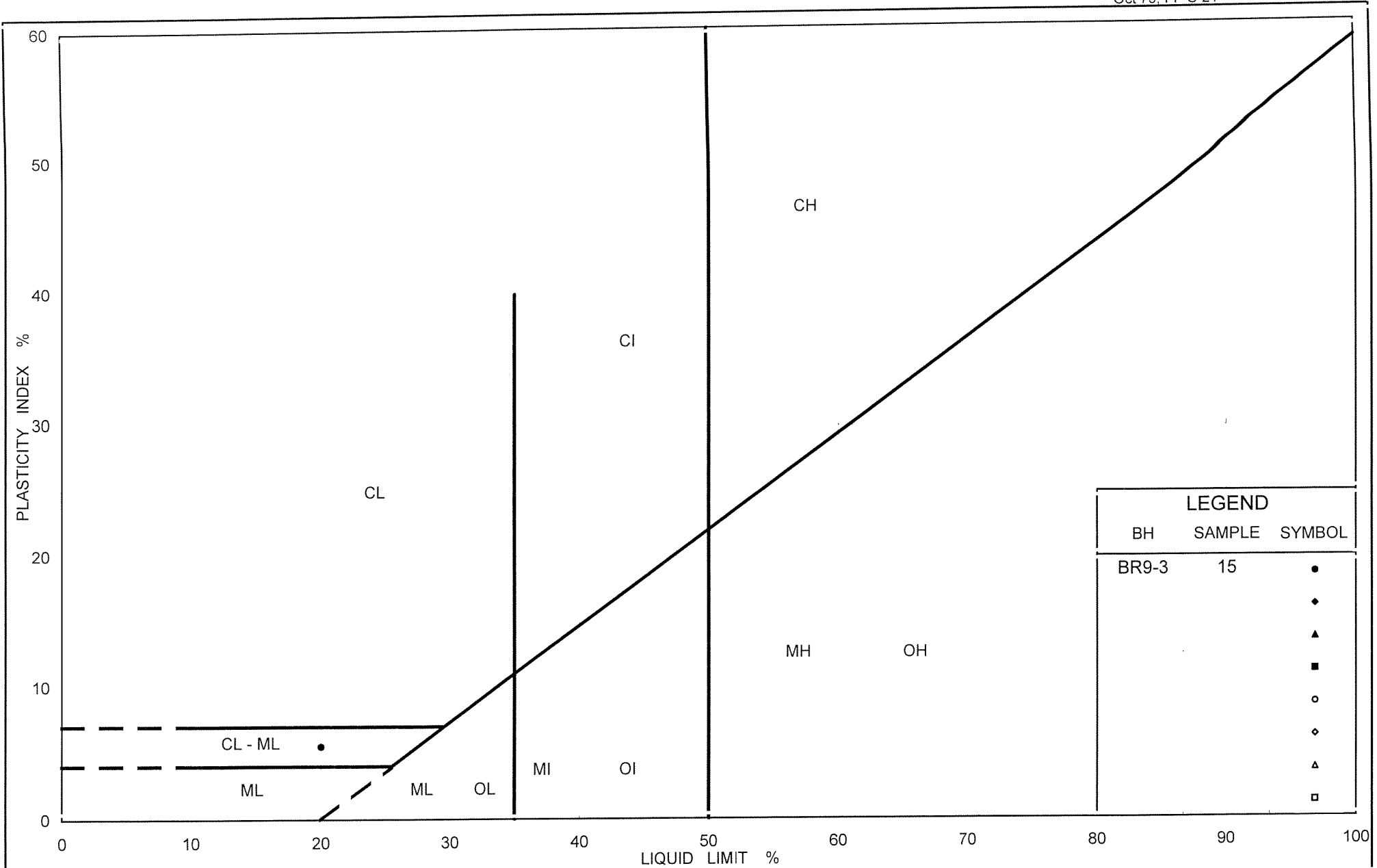
FIGURE A19



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
•	BR9-3	15	63.1



Ministry of Transportation

Ontario

PLASTICITY CHART Sandy Silt and Gravel (Till)

FIG No. A20

Project No. 021-1162