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REPORT ON

**FOUNDATION INVESTIGATION AND DESIGN
RUNNING CREEK BRIDGE REPLACEMENT
HIGHWAY 40, SITE 13-6, GWP 60-99-00
TOWNSHIP OF CHATHAM (GORE)
MUNICIPALITY OF CHATHAM-KENT
DISTRICT 32, CHATHAM**

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December 2000

001-3045
Geocres No. 40J9-20

TABLE OF CONTENTS

Table of Contents	i
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<u>SECTION</u>	<u>PAGE</u>
----------------	-------------

PART A – FIELD INVESTIGATION

1.0	INTRODUCTION.....	1
2.0	SITE DESCRIPTION	2
2.1	Site Location	2
2.2	Physiography	2
3.0	INVESTIGATION PROCEDURES	3
4.0	SUBSURFACE CONDITIONS	5
4.1	General Stratigraphy	5
4.2	Fill and Topsoil	5
4.3	Silt and Sandy Silt	6
4.4	Sands	6
4.5	Silty Clay	6
4.6	Sandy Silt Till	8
4.7	Shale Bedrock	8
4.8	Groundwater Conditions	8

PART B – FOUNDATION DESIGN

5.0	ENGINEERING RECOMMENDATIONS	10
5.1	Replacement Bridge Foundations	10
5.1.1	Pile Axial Resistance	10
5.1.2	Horizontal Resistance	12
5.2	Detour Bridge Foundations	14
5.3	Lateral Earth Pressure.....	15
5.4	Approach Embankments	16
5.4.1	Settlement.....	16
5.4.2	Stability	18
5.4.3	Traffic Protection	19
5.5	Subgrade Preparation and Embankment Construction.....	20
5.6	Temporary Excavations and Groundwater Control.....	20

TABLE OF CONTENTS Continued

In Order
Following
Page 21

LIST OF ABBREVIATIONS

LIST OF SYMBOLS

LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

RECORD OF BOREHOLE SHEETS

- FIGURE 1 – Site Location Map
- FIGURE 2 – Borehole Locations
- FIGURES 3 and 4 – Soil Strata Profile and Cross Sections
- FIGURE 5 – Field Vane Shear Strength vs. Elevation

- APPENDIX A – Piezo-cone Penetration Test Results
- APPENDIX B – Laboratory Test Data (Figures B-1 to B-9)
- APPENDIX C – Site Photographs

PART A – FIELD INVESTIGATION

FOUNDATION INVESTIGATION AND DESIGN

RUNNING CREEK BRIDGE REPLACEMENT

HIGHWAY 40, SITE 13-6, GWP 60-99-00

TOWNSHIP OF CHATHAM (GORE)

MUNICIPALITY OF CHATHAM-KENT

DISTRICT 32, CHATHAM

1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by Dillon Consulting Limited (Dillon) to carry out a foundation investigation for the replacement of the Running Creek Bridge (Site 13-6, GWP 60-99-00) as part of the improvements of Highway 40 between Chatham and Sarnia, Ontario. The proposed work consists of replacing the existing three span bridge structure with a wider and longer three span structure.

The purpose of the investigation was to determine the subsurface conditions at the locations of the proposed replacement bridge, approach fills and detour bridge by drilling boreholes, carrying out in-situ tests and performing laboratory tests on selected samples. Based on our interpretation of the data obtained, recommendations on the geotechnical aspects of the foundation design and construction for the abutments and for the new bridge piers are provided.

Dillon provided the plan and profile of the proposed Highway 40 alignment at the Running Creek Bridge to Golder. The centreline and stations of the proposed alignment were surveyed by others prior to commencing the foundation field investigation program.

The terms of reference and scope of work for the investigation are outlined in Golder's proposal P01-3013, dated February 2000, that forms part of the Consultant's Agreement (Number 3005-A-000069) for this project. The two initially proposed approach fill boreholes, as outlined in our proposal, were replaced by piezo-cone penetration test holes. The work was carried out in accordance with the Quality Control Plan for this project.

2.0 SITE DESCRIPTION

2.1 Site Location

The project area covered by this report extends along Highway 40 from approximately Stations 10+750 to 10+900 in the Township of Chatham (Gore) where Highway 40 crosses Running Creek. The site is situated approximately 1 kilometre west of the west limit of Wallaceburg, Ontario (see Figure 1). The highway runs approximately east-west and the creek flows southwest and northeast towards the North Sydenham River or the Chenal Ecarte depending on the prevailing water levels.

The existing bridge deck is about 2 metres above current creek level. Based on available information provided to Golder, the existing three span concrete bridge structure is founded on friction piles. The ground surface at the existing abutment locations is at about elevation 176.7 metres and the Running Creek bed is at about elevation 172.2 metres. The water level in the creek is currently at about elevation 175 metres.

The proposed replacement bridge is located at the same location as the existing bridge. The proposed highway surface on the replacement bridge is at about elevation 178.2 metres at midspan. The detour bridge will be some 35 metres north of the existing bridge site and will consist of a two lane, single span, Bailey Bridge with a deck elevation of about 176.4 metres.

2.2 Physiography

The site lies within the physiographic region of Southwestern Ontario known as the St. Clair Clay Plain¹, specifically an area known as the Chatham Flats. The soil conditions in the areas adjacent to Lake St. Clair and the St. Clair River typically consist of clay loam with occasional silty or sandy knolls overlying deep deposits of fairly uniform clay laid down across Kent County by the glacial and post-glacial lakes which existed during and after the final retreat of the last glaciation. The region is generally underlain by black shale bedrock of the Kettle Point formation of Upper Devonian Age. The rock surface is typically found at depths of about 20 to 30 metres below the ground surface.

¹ The Physiography of Southern Ontario; Ontario Geological Survey, Special Volume 2. By Chapman and Putnam, 1984.

3.0 INVESTIGATION PROCEDURES

The field work for this investigation was carried out between August 10 and 31, 2000 during which time, eight boreholes and two piezo-cones were advanced at the site of the proposed replacement bridge and detour. The boreholes were drilled and sampled to depths ranging from about 8.1 metres to 37.6 metres. The bedrock surface and conditions were proven in four boreholes by rock coring. The borehole locations are shown in plan on Figure 2. Additional boreholes and coreholes drilled for pavement reconstruction in conjunction with the bridge replacement are being reported in a separate Pavement Design Report.

The investigation was carried out using CME 55, 75 and 750 drill rigs mounted on all terrain vehicles and a truck supplied and operated by Lantech Drilling Services Inc. The boreholes were advanced using 208 millimetre outside diameter continuous flight hollow stem augers or rotary wash boring with NW sized casing through overburden soils, and using an NQ size core barrel for bedrock coring. Soil samples were obtained at regular intervals of depth using a 50 millimetre outside diameter split-spoon sampler in accordance with standard penetration test procedures, or a 80 millimetre outside diameter thin walled open Shelby tube sampler. In-situ vane shear tests were also carried out at intervals in the clayey deposit. The cored length of bedrock in the four boreholes at the replacement bridge was 1.6 to 3.2 metres. The bedrock surface elevation was consistent across the bridge site. Two shallow standpipes and a deep piezometer were installed in boreholes 2, 3 and 6 to measure the groundwater conditions at these locations. All of the boreholes were backfilled using Ministry of Transportation, Ontario (MTO) recommended procedures. Water levels in the installations were obtained on September 1 and 13, 2000 to determine stabilized levels at that time.

The field work was supervised throughout by a member of our engineering staff, who located the boreholes, cleared the borehole locations for underground services, supervised the drilling, sampling and in-situ testing operations, logged the boreholes, and examined and cared for the soil and rock samples. The samples were identified in the field, placed in containers, labeled and transported to our London laboratory. Laboratory testing on selected samples included natural water content, Atterberg limits, grain size analyses, specific gravity and oedometer (consolidation) tests. The results of the field and laboratory testing are given on the Record of Borehole sheets and in Appendices A and B, respectively. Site photographs of the drilling operations are provided in Appendix C.

The following table summarizes the proposed locations (at the time of drilling) of foundation units and boreholes, referenced to the proposed highway centreline stations.

<u>FOUNDATION UNIT/ BOREHOLE NUMBER</u>	<u>APPROXIMATE STATION (m)</u>	<u>BOREHOLES</u>
Replacement Bridge		
West Abutment	10+856	1
West Pier	10+839	8
East Pier	10+817	5
East Abutment	10+800	2
Detour Bridge		
West Abutment	10+840	6, 7
East Abutment	10+788	3, 4

The Piezo-cone penetration tests were advanced at about Stations 10+780 and 10+886 for the east and west approaches, respectively. The results of the cone penetration testing are provided in Appendix A.

The as drilled borehole locations were surveyed by Golder staff using the co-ordinate system on the drawings provided by Dillon. The elevations at the borehole locations were referenced to a brass tablet in the top portion of the southeast wingwall of the existing bridge. The benchmark is numbered 81U103 and is understood to have an elevation of 176.901 metres, referred to geodetic datum.

4.0 SUBSURFACE CONDITIONS

The detailed subsurface conditions encountered in the boreholes advanced during this investigation are presented on the Record of Borehole sheets, the piezo-cone penetration test results are presented in Appendix A and the laboratory test results are contained in Appendix B. It should be noted that the stratigraphic boundaries indicated on the borehole records are inferred from non-continuous sampling, observations of drilling progress, results of standard penetration tests and in-situ vane shear tests. These boundaries typically represent transitions from one soil type to another and should not be regarded as exact planes of geological change. Further, subsurface conditions will vary between and beyond the borehole locations.

The ground surface elevation at the borehole and cone penetration test locations varied from about elevation 176.5 metres to 177.1 metres. The boreholes were sampled at regular intervals of depth.

4.1 General Stratigraphy

The general stratigraphy at the site consists primarily of surficial topsoil and fill layers between ground surface at about elevation 177 metres and elevation 175 metres where silty deposits were encountered. These surficial layers were underlain by a thick deposit of cohesive soils between about elevation 169 metres and 144 metres. Thin granular and/or sandy silt till layers were encountered between the cohesive deposit and the top of the shale bedrock at about elevation 142 metres.

At the locations of boreholes 1, 6, 7 and 8, a 3.8 to 5.9 metres thick granular deposit overlies the cohesive soils. These granular soils may have been deposited by a former river/creek channel west of the existing creek channel.

4.2 Fill and Topsoil

Topsoil and fill layers were encountered in all of the boreholes, except borehole 8 which was drilled in the creek channel. The topsoil layers were about 0.1 to 0.6 metres thick and were generally encountered at ground surface or beneath layers of fill between elevations 175.0 and 177.1 metres. Two standard penetration tests in the topsoil indicated N values of 6 and 12 blows per 0.3 metres penetration and the samples had water contents of 23 and 48 per cent.

Clayey silt and sandy silt fill layers some 1.0 to 1.9 metres thick were encountered in boreholes 1 to 7 between elevations 174.9 and 177.0 metres. The fill layers had N values between 3 and 19 blows per 0.3 metres of penetration and water contents between 12 and 56 per cent. The fill was underlain by about 0.3 metres of topsoil in boreholes 2 and 6 and 0.8 metres of peat in borehole 5.

4.3 Silt and Sandy Silt

Beneath the fill and topsoil layers in boreholes 1 and 4 to 7, and at the creek bed in borehole 8, layers of sandy silt and silt were encountered. These layers were 0.8 to 5.9 metres thick and were encountered between elevations 169.1 and 175.4 metres. The silt and sandy silt generally had standard penetration test N values of 1 to 9 blows per 0.3 metres of penetration, but were as low as the weight of the sampling rods in the creek bed in borehole 8. The water contents of the silt and sandy silt samples ranged between 15 and 55 per cent with an average of about 31 per cent. Figure B-1 shows the grain size distribution curve for a sample of sandy silt from borehole 1.

4.4 Sands

Sand layers with varying amounts of silt and gravel were encountered beneath the sandy silt at elevations between 169.1 and 174.9 metres in boreholes 1 and 7, between silty clay layers in borehole 4 and beneath the silty clay deposit between elevation 144.0 and 144.7 metres in boreholes 2 and 8. The sands were generally about 0.6 metres thick, except in borehole 1 where the sand was about 4 metres thick and underlain by 0.4 metres of sand and gravel. The granular deposits had standard penetration test N values between 1 and 10 blows per 0.3 metres penetration and water contents between 16 and 31 per cent, with an average of about 25 per cent. Grain size distribution curves for three sand samples are shown on Figures B-2 and B-3.

4.5 Silty Clay

Below the surficial soil deposits, an extensive deposit of grey silty clay, with occasional clayey silt and silty sand layers was encountered in all of the boreholes. The top of this deposit varies between elevation 168.7 and 175.2 metres. A stiffer crust was generally present near the top of the silty clay. The bottom of the deposit was between elevation 143.8 and 144.8 metres for a total thickness between 24.1 and 30.0 metres. The lower portion of this deposit was noted to have small natural gas pockets during drilling of the boreholes. Standard penetration test N values ranged from the weight of rods to 7 blows per 0.3 metres of penetration, with an average of less

than one blow per 0.3 metres of penetration based on 105 tests. In-situ vane shear testing in the silty clay deposit indicated undrained shear strengths between 19 and 91 kilopascals, indicating a soft to stiff consistency, with an average strength of 44 kilopascals based on 69 tests. Figure 5 summarizes the measured field vane shear tests that were carried out as part of the current investigation. The figure clearly shows the crust in the silty clay above elevation 170 metres and a trend of increasing vane (undrained) shear strength (s_u) with depth below about elevation 165 metres, ranging from a minimum value of about 20 kilopascals to about 60 kilopascals at the bottom of the deposit.

Figures B-4 to B-6 in Appendix B show gradation curves for fourteen samples of this deposit. Measured water contents of the samples from this deposit were between 26 and 57 per cent, with an average of about 40 per cent. The average plastic and liquid limits, based on 14 samples tested, are 23 and 42 per cent, respectively, with an average plasticity index of 19 per cent. The Atterberg limits for this deposit are summarized on a Plasticity Chart in Figure B-8 in Appendix B.

Figures B-9 and B-10 in Appendix B show the void ratio (e) vs. log effective vertical pressure (σ') for two samples of the silty clay deposit that were obtained from boreholes 1 and 2 using thin walled Shelby tube samplers. The soil samples were assessed to be undisturbed. The e -log σ' curves are considered to reflect the silty nature of the soil, although it may also reflect that some degree of disturbance may have been imparted to the sample. The key consolidation parameters interpreted from the test results are:

<u>PARAMETER</u>	<u>BOREHOLE 1 SAMPLE 14</u>	<u>BOREHOLE 2 SAMPLE 8</u>
Initial void ratio	1.29	1.16
Estimated preconsolidation pressure, σ'_p (kPa)	120	130
Existing overburden pressure, σ'_{po} (kPa)	120	110
Overconsolidation Ratio, OCR	1.0	1.2
Recompression Index, C_r (based on rebound portion of curve)	0.12	0.10
Compression Index, C_c	0.47	0.37

4.6 Sandy Silt Till

A 1.7 to 2.4 metres thick layer of grey sandy silt till was encountered beneath the silty clay in the boreholes that penetrated the silty clay deposit (boreholes 1, 2, 4, 5, 7 and 8). The till layer was between elevations 141.7 and 144.5 metres. Standard penetration test N values in the till ranged from 34 to over 100 blows per 0.3 metres of penetration indicating dense to very dense conditions. Measured water contents of samples from this deposit ranged from 10 to 17 per cent. Grain size curves for two samples recovered from the standard penetration testing are shown on Figure B-7 in Appendix B.

4.7 Shale Bedrock

The bedrock surface was encountered between elevations 141.7 and 142.3 metres in boreholes 1, 2, 3, 5 and 8. The bedrock surface typically had standard penetration test N values of greater than 100 blows per 75 to 100 millimetres of penetration. The dark grey shale bedrock is slightly weathered and thinly bedded near the surface with rock quality designation values as low as 0 per cent, becoming more massive and black with depth with rock quality designation values as high as 98 per cent. A statistical summary of the rock core data is as follows:

INDEX	NO. OF TESTS	PERCENTAGE OF CORE RUN		
		Minimum	Maximum	Average
Total Core Recovery (TCR)	5	70	100	86
Solid Core Recovery (SCR)	5	14	99	74
Rock Quality Designation (RQD)	5	0	98	69

4.8 Groundwater Conditions

Two standpipes and one piezometer were installed in boreholes 2, 3 and 6. The water levels encountered in the boreholes during drilling between August 10 and 31, 2000 and measured in the installations on September 1 and 13, 2000 are summarized in the following table. It should be noted that the groundwater level is subject to seasonal fluctuations.

BOREHOLE NUMBER	GROUND SURFACE ELEVATION (m)	ENCOUNTERED GROUNDWATER LEVEL		MEASURED GROUNDWATER LEVEL			
		Depth (m)	Elevation (m)	SEPTEMBER 1, 2000		SEPTEMBER 13, 2000	
				Depth (m)	Elevation (m)	Depth (m)	Elevation (m)
1	176.70	2.90	173.80				
2	176.67	3.05	173.62	2.50	174.17	2.01	174.66

3	176.73	5.18	171.55	1.68	175.05	1.80	174.93
4	176.60	Dry	Dry				
5	176.64	1.82	174.82				
6	177.08	2.13	174.95	2.44	174.64	1.91	175.17
7	177.14	2.13	175.01				
8	176.72	1.71	175.01				

The measured water level in the creek was at elevation 174.96 metres on September 1, 2000.

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PART B – FOUNDATION DESIGN

FOUNDATION INVESTIGATION AND DESIGN

RUNNING CREEK BRIDGE REPLACEMENT

HIGHWAY 40, SITE 13-6, GWP 60-99-00

TOWNSHIP OF CHATHAM (GORE)

MUNICIPALITY OF CHATHAM-KENT

DISTRICT 32, CHATHAM

5.0 ENGINEERING RECOMMENDATIONS

This section of the report provides our interpretation of the factual geotechnical data obtained during the investigation. The recommendations provided are intended for the guidance of the design engineer. Where comments are made on construction, they are provided to highlight aspects of construction that could affect the design of the project. Contractors bidding on or undertaking the works must make their own interpretation of the subsurface information provided as it affects their proposed construction methods, costs, equipment selection, scheduling and the like.

The proposed work consists of a three span bridge structure and a detour bridge (see Figure 2). The proposed replacement bridge is located at the same location as the existing bridge. The proposed highway surface on the replacement bridge is at about elevation 178.2 metres at the abutments. The detour bridge will be some 35 metres north of the existing bridge site and will consist of a two lane, single span Bailey Bridge with a deck elevation of about 176.4 metres.

5.1 Replacement Bridge Foundations

The subsoils encountered at the replacement bridge location are not considered suitable to support shallow spread footings. Deep foundations, such as steel H-piles driven to practical refusal on bedrock, are recommended for support of the abutments and piers. H-piles are recommended because they will easily penetrate the clay deposit and minimize the amount of disturbance imparted to the clay given their shape and small cross-sectional area.

For frost action protection, the base of pile caps and other footings should be provided with a minimum soil cover of 1.2 metres.

5.1.1 Pile Axial Resistance

For HP 310 x 110 piles driven to refusal in the shale bedrock at about elevation 142 metres at the replacement bridge location, based on proposed pile cut off elevations of 171.5 and 175.0 metres at the piers and abutments, respectively, pile lengths will be about 29.5 and 33.0 metres. A factored axial resistance at Ultimate Limit States (ULS) of 2,000 kilonewtons per pile may be assumed for design. This value represents a structural limitation of the pile rather than a geotechnical limitation. The pile tips should be suitably reinforced with rock points to ensure penetration and adequate seating as per current MTO practice (Standard Ontario Provincial

Standard Drawings (OPSD) 3301.00 and Ontario Provincial Standard Specifications (OPSS) 903.07.02.05).

A Serviceability Limit States (SLS) value is not provided because the shale bedrock is considered to be an unyielding material. Under such conditions, SLS values (for 25 millimetres of settlement) do not govern design because the SLS value is much higher than the ULS values.

Downdrag Load (Negative Skin Friction)

As will be discussed in a later section, the height of the approach embankments for the replacement bridge is about 1.5 metres and some consolidation settlement of the underlying thick clay deposits will take place as a result of additional loading associated with increased vertical grades due to approach embankments adjacent to the abutments. The consolidation settlement is time-dependent and will not completely occur during the construction period. That is, post-construction settlement of the clay deposit will take place. Because the piles are end-bearing on bedrock, a small amount of settlement of the clay relative to the pile will result in the development of negative skin friction acting on the piles. Therefore, negative skin friction or downdrag loads will need to be taken into account during design of the piles supporting the replacement bridge abutments.

The magnitude of the downdrag load acting on a pile is a function of the adhesion (skin friction) that develops between the pile and the clay, and the surface area of the pile within the clay deposit.

The total downdrag load is a function of the surface area of the pile within the cohesive soil. The negative skin friction acting on a single pile has been calculated to be 200 kilonewtons using OHBDC methods. The load calculated in this manner is a nominal (unfactored) load. The structural engineer needs to multiply this load by a load factor of 1.25, as defined in OHBDC, and include it as part of the dead load effects acting on the piles at the abutments.

Set Criteria

Set criteria are highly dependent on pile driving hammer type and selected pile. The set criteria can be determined through a variety of methods, including empirical correlations and wave equation analyses, at the time of construction once the hammer and pile types are known. The choice of set criteria is dependent on the experience of the engineer, and traditional use where a substantial database has been developed over the years. The criteria also needs to be set to avoid overdriving and possible damage to the piles. Pile driving should be in conformance with OPSS 903 and provision should be made to re-drive selected piles as per MTO's specifications.

Pile Driving Note

The pile driving note to be added to the drawings is Note 4 in Clause 2.5.11 of the Structural Manual – “Piles to be driven to bedrock”.

5.1.2 Horizontal Resistance

The design of piles subjected to lateral loads should take into account such factors as 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 moments, the soil resistance that can be mobilized, the maximum tolerable lateral deflection at the head of the pile and pile group effects. For a longer, more flexible pile, its maximum yield moment may be reached prior to mobilization of the lateral geotechnical resistance. For design purposes, both the structural and geotechnical resistances should be determined to establish the governing case.

According to the Canadian Foundation Engineering Manual, the component of horizontal soil reaction to a vertical pile in cohesive soils can be estimated using the following formula:

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

where: k_h = coefficient of horizontal subgrade reaction (kPa/m)
 s_u = undrained shear strength of the soil (kPa)
 b = pile width or diameter (m)

The in-situ vane shear test results indicate that the undrained shear strength of the clay deposit increases with depth (see Figure 5). For the purpose of design, it may be assumed that s_u is 25 kilopascals in the crust and increases linearly with depth below elevation 165 metres and may be calculated as follows:

$$s_u = 23.5 + 1.8d \text{ (units of kPa)}$$

where: d = depth below elevation 165 (m)

The following table provides the generalized undrained shear strength profile for the silty clay materials at the site:

<u>ELEVATION</u> (m)	<u>s_u</u> (kPa)
175 to 166	25.0
165	23.5
164	25.3
163	27.1
162	28.9
161	30.7
160	32.5
159	34.3
158	36.1
157	37.9
156	39.7
155	41.5
154	43.3
153	45.1
152	46.9
151	48.7
150	50.5
149	52.3
148	54.1
147	55.9
146	57.7
145	59.5

Group action for lateral loading should be considered when the pile spacing in the direction of 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:

SUBGRADE REDUCTION FACTORS

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

It should be pointed out that geotechnical resistance to develop lateral load is very strain dependent. For the soil profile at this site, it is recommended that lateral loads be accommodated using battered piles.

5.2 Detour Bridge Foundations

The subsoils encountered at the location of the proposed detour bridge are not considered suitable to support shallow spread footings. Deep foundations, such as steel H-piles driven to practical refusal on bedrock are recommended for the support of the proposed Bailey Bridge.

All of the piling recommendations provided in Section 5.1 for the replacement bridge foundations are considered relevant for the detour bridge foundations. However, it is anticipated that the pile tips will reach the shale bedrock at about elevation 141.5 metres, for total pile lengths in the order of 33.5 metres based on a cut off elevation of 175.0 metres.

5.3 Lateral Earth Pressure

The lateral pressures acting on the bridge abutments will depend on the type and method of placement of the backfill materials, on the nature of the soils behind the backfill and on the subsequent lateral movement of the structure. The following recommendations are made concerning the design of the abutments and the retaining walls in accordance with OHBDC 6-7.

- Select free-draining granular fill meeting the specifications of OPSS Granular A or Granular B, Type II but with less than 5 per cent passing the 200 sieve should be used as backfill behind the walls. All granular fill should be compacted in lifts of loose thickness not greater than 200 millimetres to 95 per cent of the material's Standard Proctor maximum dry density.
- Alternatively, lightweight fill, such as slag material, may be used to limit consolidation settlement. Slag fill material is lightweight, possesses good strength (frictional) characteristics, is non-frost susceptible, is quite pervious and possesses good drainage characteristics.
- Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill.
- The granular fill may be placed either in a zone with width equal to at least 1.2 metres behind the back of the stem (Case I – OHBDC, Figure 6-7.4.1) or within the wedge-shaped zone defined by a 1.5 horizontal to 1 vertical line extending up and back from the bottom of the rear face of the footing as shown by OPSD-3501.000 (Case II).
- If lightweight fill is used, particle breakage or crushing of particles will occur during compaction if over compacted. Therefore, careful construction control is required to achieve adequate compaction without crushing. The lightweight backfill should be placed in loose lifts of 300 millimetres and compacted by eight passes of a manually guided tamper such as Bomag BPR 30/38 or equivalent in accordance with OPSS 206.07.
- If the wall support allows lateral yielding of the stem (unrestrained structure), active earth pressures may be used in the geotechnical design of the structure. If the abutment support does not allow lateral yielding (restrained structure), at-rest pressures should be assumed for geotechnical design.

- A compaction surcharge equal to 16 kPa should be included in the lateral earth pressures for the structural design of the abutment wall in accordance with OHBDC Figure 6-7.4.3. Compaction equipment should be restricted as per OPSS 501.06.
- For Case I, the pressures will be based on the in-situ soil and proposed embankment fill. The following parameters (unfactored) may be assumed:

	<u>EARTH FILL</u>	<u>LIGHTWEIGHT FILL</u>	<u>ULTRA LIGHTWEIGHT FILL</u>
Soil Unit Weight (kN/m ³) [assuming the compacted clean earth fill is Select Subgrade Material (OPSS 1010)]	21	14	11.5
Coefficients of Lateral Earth Pressure:			
'active'	0.33	0.27	0.27
'at rest'	0.50	0.43	0.43

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

	<u>GRANULAR A</u>	<u>GRANULAR B (TYPE II)</u>	<u>LIGHTWEIGHT FILL</u>	<u>ULTRA LIGHTWEIGHT FILL</u>
Soil Unit Weight (kN/m ³)	22	21	14	11.5
Coefficients of Lateral Earth Pressure				
'active'	0.27	0.31	0.27	0.27
'at rest'	0.43	0.47	0.43	0.43

It should be noted that the above design parameters assume level backfill at ground surface behind the wall. Other aspects of the abutment granular backfill requirements with respect to sub-drains and frost tapers should be in accordance with OPSD-3501.00.

5.4 Approach Embankments

5.4.1 Settlement

Based on a finished road surface elevation at elevation 178.2 metres at the abutments of the new structure, approximately 1.6 metres of fill is required for the proposed bridge approach embankments. It is understood that no fill will be required for the detour bridge approaches. Settlement analyses were carried out for the Running Creek Bridge approach embankments based on the available borehole, oedometer and in-situ vane shear strength data. The following parameters were used in the analysis:

<u>SOIL UNIT</u>	<u>COMPRESSION INDEX, C_c</u>	<u>RECOMPRESSION INDEX, C_r</u>	<u>INITIAL VOID RATIO, e₀</u>
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Silty Clay	0.4	0.1	1.2
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The embankment fill loads were modelled as a rectangular wedge having the dimensions of the proposed fill. Further, the portion of the settlement due to the granular deposit at the west approach was calculated to be about 30 millimetres. Based on the results of the analysis, the following conclusions are made:

- For the west approach at the abutment, the maximum total settlement is estimated to be in the range of 80 millimetres to 100 millimetres, about a third of which will occur during construction with the remainder over ten years.
- For the east approach, the maximum total settlement is estimated to be about 100 millimetres to 150 millimetres, most of which should occur as long-term consolidation settlement over five to ten years or so.

Alternatively, the magnitude of settlement may be reduced by using lightweight fill to construct the approach embankments. The results of the analyses indicated that lightweight fill would reduce the settlements for the east approach to about 60 millimetres and to as little as 40 millimetres if 1 metre thickness of the existing fill materials is subexcavated and replaced with lightweight fill. Settlements would reduce to about 35 millimetres with the use of ultra lightweight fill (such as Litex 143) and 1 metre of subexcavation and replacement. We understand from the supplier that Litex 143 would be available for this project.

The following is a summary of the settlement analyses for the approach embankments:

	EMBANKMENT SETTLEMENT NEAR ABUTMENT (mm)	
	<u>East</u>	<u>West</u>
Granular Fill	100 – 150	80 – 100
Lighweight Fill	60	50
Lightweight Fill with 1 metre subexcavation	40	40
Ultra lightweight fill with 1 metre subexcavation	35	35

If the lightweight fill with 1 metre subexcavation option is selected, the subexcavation should extend the full width of the embankment and at least 30 metres from the abutments plus a 1 in 10 taper zone.

If the detour bridge is not constructed and the construction of the replacement bridge is staged to allow traffic access, some of the anticipated settlement will occur differentially between the embankment construction stages.

5.4.2 Stability

Stability analyses were carried out for the banks of the creek to determine the effects of loads induced by the embankment fill and traffic at the replacement bridge site and by shallow foundations and traffic loading at the proposed detour bridge location. The analyses considered the effects of the loads applied at the top of the bank together with the interaction between the crust and the underlying softer silty clay soils as well as variations in the subsurface conditions at the various borehole locations. Based on the results of the analyses, the following conclusions are made regarding the stability of the creek banks:

- Placing the proposed Bailey bridge on conventional spread footings or cribs results in a factor of safety less than unity.
- The factor of safety is increased to 1.8 if the Bailey bridge is constructed on driven piles.

- The factor of safety is in the order of 1.5 for the replacement bridge embankments for the proposed 3 horizontal to 1 vertical slopes.

The overall stability is increased if lighter backfill material is used.

5.4.3 Traffic Protection

It is anticipated that the roadway protection consisting of vertical shoring and detouring of traffic onto the roadway shoulders will be required as part of the traffic staging if the detour bridge is not constructed. The geotechnical parameters that may be used for the design of the vertical shoring are summarized below:

Existing Embankment Fill/Native Soil

Total unit weight, γ	20 kN/m ³
At rest lateral earth pressure coefficient, k_o	0.55
Active lateral earth pressure coefficient, k_A	0.38
Passive lateral earth pressure coefficient, k_P	2.66

Surcharge Loading

Live load surcharge due to traffic, q_{ST}	18 kPa
Live load surcharge due to compaction, q_{SC}	16 kPa

Additional Embankment Fill

Total unit weight, γ :	Granular Fill	21 kN/m ³
	Lightweight Fill	14 kN/m ³
	Ultra Lightweight Fill	11.5 kN/m ³
Lateral earth pressure coefficient, K		0.5

For design of the shoring, the lateral earth pressure distribution should be taken as a truncated triangular shape defined by the live load surcharge for traffic, vertical effective stress, and the coefficient of earth pressure at rest with an additional compaction surcharge defined by an inverted triangular loading with a maximum equal to q_{SC} consistent with Section 6-7 of the Ontario Highway Bridge Design Code. No component for water pressure is required.

5.5 Subgrade Preparation and Embankment Construction

Topsoil and organic deposits should be stripped from within the plan limits of the proposed embankments.

It is considered that the approach fills may be constructed to proposed underside of subbase elevation using approved, non-frost susceptible earth borrow or lightweight fill, depending on the acceptable magnitude of settlement. Embankment materials should be placed in maximum 300 millimetre thick loose lifts and compacted. Side slopes should not exceed an inclination of 2.0 horizontal to 1 vertical. Upon completion of embankment and pavement construction, the side slopes should be trimmed to the final inclination and be properly topsoiled and seeded, as appropriate. Surface water should be directed to localized discharge points leading to rock lined drainage channels located on the slope.

Inspection and field density testing should be carried out by qualified geotechnical personnel during all fill placement operations to ensure that appropriate materials are used and that adequate levels of compaction have been achieved.

5.6 Temporary Excavations and Groundwater Control

It is anticipated that the excavations for the replacement bridge abutment will extend to elevation 174.5 metres. These excavations will be in fill materials, silty clay and/or silts. The detour bridge excavations may encounter the silty clay deposits. The fill and silty soils are classified as Type 3 and the silty clay deposit as Type 2 according to the Occupational Health and Safety Act of Ontario. All excavations should be carried out according to the latest edition of the Occupational Health and Safety Act and Regulations for Construction Projects. Only minimal groundwater seepage is anticipated.

Temporary cuts made no steeper than 1 horizontal to 1 vertical should be stable during normal construction duration, although some localized surficial sloughing may be experienced in areas of higher groundwater seepage. Conventional excavation equipment would be suitable for excavating the soils. All surface water must be directed away from the excavation and not permitted to enter the excavation.

The excavations for the piers will extend to elevation 170 metres. Following the installation of steel sheet piling, excavation in the wet would be carried out, the foundation piles driven, a one metre thick tremie plug placed and the sheeted cofferdam dewatered for pier construction.

GOLDER ASSOCIATES LTD.

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LIST OF ABBREVIATIONS

The abbreviations commonly employed on each "Record of Borehole", on the figures and in the text of the report, are as follows:

I. SAMPLE TYPES

<i>AS</i>	auger sample
<i>CS</i>	chunk sample
<i>DO</i>	drive open
<i>DS</i>	Denison type sample
<i>FS</i>	foil sample
<i>RC</i>	rock core
<i>ST</i>	slotted tube
<i>TO</i>	thin-walled, open
<i>TP</i>	thin-walled, piston
<i>WS</i>	wash sample
<i>SS</i>	split spoon

II. PENETRATION RESISTANCES

Dynamic Penetration Resistance:

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 0.3 m (12 in.).

Standard Penetration Resistance, 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 0.3 m (12 in.).

<i>WH</i>	sampler advanced by static weight-weight, hammer
<i>PH</i>	sampler advanced by hydraulic force
<i>PM</i>	sampler advanced by manual force

III. SOIL DESCRIPTION

(a) Cohesionless Soils

	"N" Blows/0.3 m or Blow/ft.
Relative Density	
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	over 50

(b) Cohesive Soils

	"Cu" = "Su"	
Consistency	kPa	psf
Very soft	0 to 12	0 to 250
Soft	12 to 25	250 to 500
Firm	25 to 50	500 to 1000
Stiff	50 to 100	1000 to 2000
Very stiff	100 to 200	2000 to 4000
Hard	over 200	over 4000

IV. SOIL TESTS

<i>C</i>	consolidation test
<i>H</i>	hydrometer analysis
<i>M</i>	sieve analysis
<i>MH</i>	combined analysis, sieve and hydrometer ¹
<i>Q</i>	undrained triaxial ²
<i>R</i>	consolidated undrained triaxial ²
<i>S</i>	drained triaxial
<i>U</i>	unconfined compression
<i>V</i>	field vane test
<i>Chem</i>	chemical analysis

NOTES:

1. Combined analyses when 5 to 95 per cent of the material passes the No. 200 sieve.
2. Undrained triaxial tests in which pore pressures are measured are shown as Q or R.

LIST OF SYMBOLS

I. GENERAL

π	= 3.1416
e	= base of natural logarithms 2.7183
\log_e	a or \ln a, natural logarithm of a
\log_{10}	a or \log a, logarithm of a to base 10
t	time
g	acceleration due to gravity
V	volume
W	weight
m	mass
M	moment
F	factor of safety

II. STRESS AND STRAIN

u	pore pressure
σ	normal stress
σ'	normal effective stress (σ is also used)
τ	shear stress
ε	linear strain
ε_{sy}	shear strain
ν	Poisson's ration (μ is also used)
E	modulus of linear deformation (Young's modulus)
G	modulus of shear deformation
K	modulus of compressibility
η	coefficient of viscosity

III. SOIL PROPERTIES

(a) Unit weight

γ	unit weight of soil (bulk density)
γ_s	unit weight of solid particles
γ_w	unit weight of water
γ_d	unit dry weight of soil (dry density)
γ'	unit weight of submerged soil
G_s	specific gravity of solid particles $G_s = \gamma_s/\gamma_w$
e	void ratio
n	porosity
w	water content
S_r	degree of saturation

(b) Consistency

w_L	liquid limit
w_P	plastic limit
I_P	plasticity index
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
D_r	relative density = $(e_{max} - e)/(e_{max} - e_{min})$

(c) Permeability

h	hydraulic head or potential
q	rate of discharge
v	velocity of flow
i	hydraulic gradient
κ	coefficient of permeability
j	seepage force per unit volume

(d) Consolidation (one-dimensional)

m_v	coefficient of volume change = $-\Delta e/(1+e)\Delta\sigma'$
C_c	compression index = $-\Delta e/\Delta\log_{10}\sigma'$
c_v	coefficient of consolidation
T_F	time factor = $c_v t/d^2$ (d , drainage path)
U	degree of consolidation

(e) Shear strength

τ_f	shear strength	$\left. \begin{array}{l} \text{in terms} \\ \text{of effective} \\ \text{stress} \end{array} \right\} \tau_f = c' + \sigma' \tan \phi$
c'	effective cohesion intercept	
ϕ'	effective angle of shearing resistance, or friction	
S_u	apparent cohesion*	
ϕ_u	apparent angle of shearing resistance, or friction	$\left. \begin{array}{l} \text{in terms of} \\ \text{total stress} \end{array} \right\} \tau_f = cu + \sigma \tan \phi_u$
μ	coefficient of friction	
S_t	sensitivity	

*For the case of a saturated cohesive soil, $\phi_u = 0$ and the undrained shear strength $\tau_f = S_u$ is taken as half the undrained compressive strength.

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

<u>Description</u>	<u>Bedding Plane Spacing-</u>
Very thickly bedded	>2 m
Thickly bedded	0.6 m to 2m
Medium bedded	0.2 m to 0.6m
Thinly bedded	60 m 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

<u>Description</u>	<u>Spacing</u>
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 (TCR)

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

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
M F - Mechanical Fracture	C - Curved
- Parallel To	
⊥ - Perpendicular To	

PROJECT 001-3045		RECORD OF BOREHOLE No 1		1 OF 3	METRIC
W.P. 60-99-00		LOCATION 4717085.655 N, 311262.796 E		ORIGINATED BY D.J.M.	
DIST 32 HWY 40		BOREHOLE TYPE CONTINUOUS FLIGHT HOLLOW STEM AUGERS / NQ ROCK CORING		COMPILED BY B. G.	
DATUM GEODETIC		DATE 10.08.00 - 16.08.00		CHECKED BY <i>mm</i>	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100							20 40 60 80 100	
								SHEAR STRENGTH kPa							WATER CONTENT (%)	
176.70	GROUND SURFACE															
0.00	Topsoil, clayey Black															
176.09																
0.61	Fill, clayey silt, trace sand, trace organics Firm, Brown		1	SS	6											
175.33																
1.37	Fill, silt, trace clay, trace sand, trace organics Loose, Brown		2	SS	4											
174.57																
2.13	Sandy Silt, trace organics, Very Loose, Grey		3	SS	3									0 26 66 8		
			4	SS	2											
173.04																
3.66	Sand, fine to medium, trace silt, Very Loose to Loose, Grey		5	SS	3									0 88 9 3		
			6	SS	5											
			7	SS	7											
			8	SS	6											
			9	SS	7									0 86 10 4		
169.08																
7.62	Sand and Gravel, Loose, Grey		10	SS	4											
168.68																
8.02	Silty Clay, Soft to Stiff, Grey		11	SS	2											
			12	SS	WH									0 1 63 36		
			13	SS	WR											
			14	SH	PH									0 0 35 65		
			15	SS	WR											

ON MOT 001-3045.GPJ ON MOT.GDT 19-09-00 DATA INPUT:

Continued Next Page

+ 3. X 3. Numbers refer to 3% STRAIN AT FAILURE
Sensitivity

PROJECT <u>001-3045</u>		RECORD OF BOREHOLE No 1		2 OF 3	METRIC
W.P. <u>60-99-00</u>	LOCATION <u>4717085 655 N, 311262.796 E</u>	ORIGINATED BY <u>D.J.M.</u>			
DIST <u>32</u> HWY <u>40</u>	BOREHOLE TYPE <u>CONTINUOUS FLIGHT HOLLOW STEM AUGERS / NQ ROCK CORING</u>	COMPILED BY <u>B. G.</u>			
DATUM <u>GEODETIC</u>	DATE <u>10.08.00 - 16.08.00</u>	CHECKED BY <u><i>AMH</i></u>			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)			
								20 40 60 80 100										
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE										
	Silty Clay, Soft to Stiff, Grey		16	SS	WR		161											
								2.9 + 2.3										
			17	SS	WR		160								0 0 29 71			
								2.9 + 2.2										
			18	SH	PH		158											
								1.7 + 2.7										
			19	SS	WR		157											
								2.1 + 2.2										
			20	SS	WR		155								Sa. 20 lost			
								2.1 + 1.9										
			21	SS	WR		154								0 3 36 61			
								2.4 + 2.0										
		22	SS	WR		152												
							2.2 + 1.9											
		23	SS	WR		151												
							2.5 + 2.6											
		24	SS	WR		149												
							3.2 + 3.2											
148.05 28.65	Silty Clay, trace sand, trace gravel, Stiff, Grey		25	SS	WR		148							0 2 35 63				
							147											
								3.3 +										

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

ON_MOT_001-3045.GPJ ON_MOT.GDT 19-09-00 DATA INPUT:

PROJECT 001-3045				RECORD OF BOREHOLE No 1				3 OF 3		METRIC				
W.P. 60-99-00				LOCATION 4717085.655 N, 311262.796 E				ORIGINATED BY D.J.M.						
DIST 32 HWY 40				BOREHOLE TYPE CONTINUOUS FLIGHT HOLLOW STEM AUGERS / NQ ROCK CORING				COMPILED BY B. G.						
DATUM GEODETIC				DATE 10.08.00 - 16.08.00				CHECKED BY <i>[Signature]</i>						
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100					
144.54	Silty Clay, trace sand, trace gravel. Stiff, Grey	[Pattern]	26	SS	WR	[Pattern]	146	2.8						
32.16	Sandy Silt, trace clay, trace gravel, (Till), Dense to Very Dense, Grey	[Pattern]	27	SS	34	[Pattern]	145	3						2 51 36 11
			28	SS 102/229m		[Pattern]	144	2.1						
142.10						[Pattern]	143							
34.60	Shale Bedrock, clay infilled fractures, slightly weathered, thinly bedded, Dark grey to black	[Pattern]	29	NO RQD/100m		[Pattern]	142							
			30	NQ RC		[Pattern]	141							
140.46	END OF BOREHOLE													
36.24	Water level encountered in borehole at elev. 173.80m Aug. 10, 2000													

ON_MOT 001-3045.GPJ ON_MOT.GDT 19-09-00 DATA INPUT:

PROJECT <u>001-3045</u>		RECORD OF BOREHOLE No 2		1 OF 3	METRIC
W.P. <u>60-99-00</u>	LOCATION <u>4717098.124 N, 311326.176 E</u>	ORIGINATED BY <u>D.J.M.</u>			
DIST <u>32</u> HWY <u>40</u>	BOREHOLE TYPE <u>CONTINUOUS FLIGHT HOLLOW STEM AUGERS / NQ ROCK CORING</u>	COMPILED BY <u>B. G.</u>			
DATUM <u>GEODETIC</u>	DATE <u>17.08.00 - 21.08.00</u>	CHECKED BY <u><i>[Signature]</i></u>			

[illegible]

ON_MOT 001-3045.GPJ ON_MOT.GDT 19-09-00 DATA INPUT:

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

PROJECT 001-3045			RECORD OF BOREHOLE No 2			2 OF 3			METRIC					
W.P. 60-99-00			LOCATION 4717098.124 N. 311326.176 E			ORIGINATED BY D.J.M.								
DIST 32 HWY 40			BOREHOLE TYPE CONTINUOUS FLIGHT HOLLOW STEM AUGERS / NQ ROCK CORING			COMPILED BY B. G								
DATUM GEODETIC			DATE 17.08.00 - 21.08.00			CHECKED BY <i>AMP</i>								
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100					
159.91	Silty Clay, trace sand, Soft to Firm, Grey		12	SS	WR									
16.76	Silty Clay, trace sand, Firm, Grey													
			13	SS	WR									
			14	SS	1									
			15	SS	3									
		16	SS	3										
		17	SS	1										
152.90	Silty Clay, Firm, Grey													
23.77			18	SS	1									
			19	SS	WR									
			20	SS	WR									
148.32	Silty Clay, trace sand, trace gravel, Firm, Grey and Brown													
28.35			21	SS	2									

ON_MOT 001-3045.GPJ ON_MOT.GDT 19-09-00 DATA INPUT:

Continued Next Page

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

PROJECT 001-3045			RECORD OF BOREHOLE No 2			3 OF 3			METRIC						
W.P. 60-99-00			LOCATION 4717098 124 N. 311326 176 E			ORIGINATED BY D.J.M.									
DIST 32 HWY 40			BOREHOLE TYPE CONTINUOUS FLIGHT HOLLOW STEM AUGERS / NQ ROCK CORING			COMPILED BY B. G.									
DATUM GEODETTIC			DATE 17.08.00 - 21.08.00			CHECKED BY <i>MAH</i>									
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	20 40 60					
	Silty Clay, trace sand, trace gravel, Firm, Grey and Brown		22	SS	WR		146							Sa. 22 lost	
144.67 32.00	Silty fine Sand, Compact, Grey		23	SS	10		145								
144.06 32.61	Sandy Silt, trace clay, trace gravel, cobbles (Till) Very Dense, Grey		24	SS	76		144								
142.38 34.29	Shale Bedrock, slightly weathered, and thinly bedded to 36.0m, becoming more massive and darker with depth, Dark grey to black		25	SS	100/0mm		143							4 40 44 12	
			26	NQ RC			142								
			27	NQ RC			141								
							140								
139.18 37.49	END OF BOREHOLE Water level encountered in borehole at elev. 173.62m Aug. 17, 2000 Water level measured in Piezometer at elev. 174.17m Sept. 1, 2000 Water level measured in Piezometer at elev. 174.66m Sept. 13, 2000														

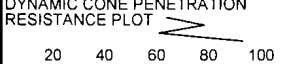
ON_MOT 001-3045.GPJ ON_MOT.GDT 19-09-00 DATA INPUT:

PROJECT 001-3045		RECORD OF BOREHOLE No 3		1 OF 1	METRIC
W.P. 60-99-00		LOCATION 4717112.303 N. 311328.402 E		ORIGINATED BY D.J.M.	
DIST 32 HWY 40		BOREHOLE TYPE CONTINUOUS FLIGHT HOLLOW STEM AUGERS		COMPILED BY B. G.	
DATUM GEODETIC		DATE 22.08.00 - 22.08.00		CHECKED BY <i>[Signature]</i>	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)
								○ UNCONFINED ● QUICK TRIAXIAL	+ FIELD VANE x LAB VANE						
176.73 0.00 0.15	GROUND SURFACE Topsoil, silty, Black Fill, sandy silt, trace topsoil, trace gravel, Compact, Brown		1	SS	14										
175.21 1.52	Silty Clay, trace sand, Very Stiff to Firm. Grey		2	SS	7									Sa. 2 lost	
			3	SS	4										
			4	SS	5										
			5	SS	6									0 0 58 42	
			6	SS	1										
			7	SS	1									0 0 62 38	
168.65 8.08	END OF BOREHOLE Water level encountered in borehole at elev. 171.55m Aug. 22, 2000 Water level measured in Piezometer at elev. 175.05m Sept. 1, 2000 Water level measured in Piezometer at elev. 174.93m Sept. 13, 2000														

ON MOT 001-3045.GPJ ON MOT GDT 19-09-00 DATA INPUT:

PROJECT 001-3045		RECORD OF BOREHOLE No 4		1 OF 3	METRIC
W.P. 60-99-00		LOCATION 4717119.480 N, 311333.846 E		ORIGINATED BY D.J.M.	
DIST 32 HWY 40		BOREHOLE TYPE NW CASING / NO ROCK CORING		COMPILED BY B.G.	
DATUM GEODETIC		DATE 22.08.00 - 31.08.00		CHECKED BY <i>DMH</i>	

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								
176.60	GROUND SURFACE												
176.00	Topsoil, silty, Black												
0.21	Fill, silt, with sand, trace topsoil. Loose Brown and black												
175.38			1	SS	6		176						
1.22	Silt, with sand, trace topsoil Loose, Brown		2	SS	7		175						Sa. 2 lost
174.47													
2.13	Silty Clay, trace sand Stiff to Firm, Grey		3	SS	4		174						
			4	SS	4		173						0 0 56 44
			5	SS	4		172						Sa. 5 lost
			6	SS	4		171						
			7	SS	2		170						
							169						
168.98													
7.62	Silty Sand, trace gravel, trace clay, Very Loose, Grey		8	SS	1		169						9 72 10 9
168.37							168						
8.23	Silty Clay, Soft to Firm, Grey		9	SS	2		167						
			10	SS	WR		166						
			11	SS	WR		165						
			12	SS	WH		164						
							163						
							162						

ON MOT 001-3045.GPJ ON MOT.GDT 19-09-00 DATA INPUT:

Continued Next Page

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

ON MOT 001-3045.GPJ ON MOT.GDT 19-09-00 DATA INPUT:

PROJECT <u>001-3045</u>		RECORD OF BOREHOLE No 4		3 OF 3		METRIC	
W.P. <u>60-99-00</u>		LOCATION <u>4717119 480 N, 311333 846 E</u>		ORIGINATED BY <u>D.J.M.</u>			
DIST <u>32</u> HWY <u>40</u>		BOREHOLE TYPE <u>NW CASING / NQ ROCK CORING</u>		COMPILED BY <u>B. G</u>			
DATUM <u>GEODETIC</u>		DATE <u>22.08.00 - 31.08.00</u>		CHECKED BY <u><i>[Signature]</i></u>			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
								○ UNCONFINED ● QUICK TRIAXIAL	+ FIELD VANE × LAB VANE					
								20 40 60 80 100						
</														

ON_MOT_001-3045.GPJ ON_MOT.GDT 19-09-00 DATA INPUT:

PROJECT 001-3045		RECORD OF BOREHOLE No 5		1 OF 3	METRIC
W.P. 60-99-00		LOCATION 4717085.349 N. 311304.144 E		ORIGINATED BY D.J.M.	
DIST 32 HWY 40		BOREHOLE TYPE NW CASING / NO ROCK CORING		COMPILED BY B. G.	
DATUM GEODETIC		DATE 28.08.00 - 29.08.00		CHECKED BY <i>ama</i>	

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							WATER CONTENT (%)			
176.64	GROUND SURFACE																	
176.48	Topsoil, Fill, silty clay, with gravel, Black																	
0.24	Fill, silt, with clay, trace sand, gravel, topsoil, Very Loose, Black		1	SS	3													
			2	SS	3													
174.51																		
2.13	Peat, Fibrous, Very Soft, Black		3	SS	1													
173.74																		
2.90	Silt, with sand, trace organics, Very Loose, Grey		4	SS	1													
172.98																		
3.66	Silty Clay, Soft to Stiff, Grey		5	SS	4													
			6	SS	4													
			7	SS	2													
			8	SS	1													
			9	SS	2													
			10	SS	WH													
165.67																		
10.97	Silty Clay, trace gravel, occasional silt and sand pockets Firm, Grey		11	SS	WR													
			12	SS	WR													
			13	SS	WR													

ON_MOT 001-3045.GPJ ON_MOT.GDT 19-09-00 DATA INPUT:

Continued Next Page

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

ON MOT 001-3045.GPJ ON MOT GDT 19-09-00 DATA INPUT:

ON MOT 001-3045.GPJ ON MOT.GDT 19-09-00 DATA INPUT:

PROJECT <u>001-3045</u>		RECORD OF BOREHOLE No 6		1 OF 1	METRIC
W.P. <u>60-99-00</u>	LOCATION <u>4717131.555 N. 311282.282 E</u>	ORIGINATED BY <u>D.J.M</u>			
DIST <u>32</u> HWY <u>40</u>	BOREHOLE TYPE <u>CONTINUOUS FLIGHT HOLLOW STEM AUGERS</u>	COMPILED BY <u>B.G.</u>			
DATUM <u>GEODETIC</u>	DATE <u>29.08.00 - 29.08.00</u>	CHECKED BY <u>DMH</u>			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)
								○ UNCONFINED ● QUICK TRIAXIAL	+ FIELD VANE x LAB VANE						
177.08	GROUND SURFACE														
0.09	Topsoil, silty, Black Fill, sandy silt, trace topsoil, trace gravel, Loose, Brown		1	SS	7										
175.56															
1.52	Topsoil, silty, Black, Compact		2	SS	12										
175.25	Sandy Silt, trace organics, Very Loose, Brown														
1.83			3	SS	3										
			4	SS	1										
			5	SS	1										
			6	SS	1										
171.44															
5.64	Clayey Silt, trace sand, trace organics, Stiff, Grey		7	SS	2										
169.76															
7.32	Silty Clay, trace sand, Stiff, Grey		8	SS	3										
169.00															
8.08	END OF BOREHOLE														
	Water level encountered in borehole at elev. 174.95m Aug. 29, 2000														
	Water level measured in Piezometer at elev. 174.64m Sept. 1, 2000														
	Water level measured in Piezometer at elev. 175.18m Sept. 13, 2000														

ON_MOT_001-3045.GPJ ON_MOT.GDT 19-09-00 DATA INPUT:

PROJECT 001-3045			RECORD OF BOREHOLE No 7			1 OF 3			METRIC					
W.P. 60-99-00			LOCATION 4717123 224 N, 311277.919 E			ORIGINATED BY D.J.M.								
DIST 32 HWY 40			BOREHOLE TYPE NW CASING			COMPILED BY B.G.								
DATUM GEODETIC			DATE 30.08.00 - 30.08.00			CHECKED BY <i>[Signature]</i>								
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
177.14	GROUND SURFACE													
0.00	Topsoil, silty, Brown													
0.15	Fill, sandy silt, trace topsoil, Loose Brown		1	SS	5									
175.92														
1.22	Sandy Silt, trace organics, Loose, Grey		2	SS	9									
174.85														
2.29	Silty fine Sand, Very Loose, Grey		3	SS	3									
174.24														
2.90	Silt, with sand, trace clay, trace organics, fine sand layers, Very Loose, Grey		4	SS	2									
			5	SS	1									
			6	SS	1									
			7	SS	2									
170.43														
6.71	Clayey silt, trace sand, trace organics, Firm, Grey		8	SS	1									
169.52														
7.62	Silty Clay, trace sand, silt layers, Firm, Grey													
			9	SS	WR									
			10	SS	WR									
165.56														
11.58	Silty Clay, trace gravel, silt lenses, Firm, Grey		11	SS	WR									
			12	SS	WR									
			13	SS	WR									

ON MOT 001-3045.GPJ ON MOT.GDT 19-09-00 DATA INPUT:

Continued Next Page

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

PROJECT 001-3045			RECORD OF BOREHOLE No 7			2 OF 3		METRIC												
W.P. 60-99-00			LOCATION 4717123 224 N. 311277.919 E			ORIGINATED BY D.J.M.														
DIST 32 HWY 40			BOREHOLE TYPE NW CASING			COMPILED BY B. G.														
DATUM GEODETIC			DATE 30.08.00 - 30.08.00			CHECKED BY														
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 (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES															
	Silty Clay, trace gravel, silt lenses. Firm. Grey																			
			14	SS	WR															
			15	SS	1															
			16	SS	1															
			17	SS	WR															
			18	SS	1															
			19	SS	WR															
			20	SS	WR															
			21	SS	WR															
			22	SS	WR															

ON_MOT_001-3045.GPJ ON MOT.GDT 19-09-00 DATA INPUT:

Continued Next Page

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

PROJECT 001-3045			RECORD OF BOREHOLE No 7			3 OF 3			METRIC					
W.P. 60-99-00			LOCATION 4717123.224 N. 311277.919 E			ORIGINATED BY D.J.M.								
DIST 32 HWY 40			BOREHOLE TYPE NW CASING			COMPILED BY B. G.								
DATUM GEODETIC			DATE 30.08.00 - 30.08.00			CHECKED BY <i>[Signature]</i>								
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE						
144.83	Silty Clay, trace gravel, silt lenses Firm, Grey		23	SS	WR		147							
			24	SS	WR		146							
32.31	Sandy Silt, trace clay, gravel, (Till), Dense to Very Dense, Grey		25	SS	34		145							
142.45							144							
							143							
34.69	END OF BOREHOLE Water level encountered in borehole at 175.01m during drilling Aug. 30, 2000		26	SS 10/254m										

ON_MOT 001-3045.GPJ ON_MOT.GDT 19-09-00 DATA INPUT:

PROJECT 001-3045			RECORD OF BOREHOLE No 8			1 OF 3			METRIC						
W.P. 60-99-00			LOCATION 4717095.098 N. 311284.224 E			ORIGINATED BY D.J.M.									
DIST 32 HWY 40			BOREHOLE TYPE NW CASING / NQ ROCK CORING			COMPILED BY B.G.									
DATUM GEODETIC			DATE 31.08.00 - 31.08.00			CHECKED BY <i>[Signature]</i>									
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa	WATER CONTENT (%)						
176.72	PAVEMENT SURFACE														
0.00	Asphalt														
0.24	Concrete														
	Air Space														
175.01							176								
1.71	Water						175								
173.82							174								
2.90	Sandy Silt, with organics, Very Loose, Grey		1	SS	WR		173								
173.21							172								
3.51	Sandy Silt, Very Loose, Grey						171								
			2	SS	4		170								
170.47							169								
6.25	Sandy Silt, with silty clay, sand and peat layers, Very Loose, Grey		3	SS	2		168								
							167								
169.10							166								
7.62	Silty Clay, trace gravel, Firm, Grey		4	SS	WR		165								
							164								
			5	SS	WR		163								
							162								
			6	SS	WR										
			7	SS	WR										
			8	SS	WR										






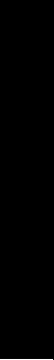
ON_MOT 001-3045.GPJ ON_MOT.GDT 19-09-00 DATA INPUT:

Continued Next Page

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

ON MOT 001-3045.GPJ ON MOT.GDT 19-09-00 DATA INPUT:

PROJECT <u>001-3045</u>		RECORD OF BOREHOLE No 8		3 OF 3	METRIC
W.P. <u>60-99-00</u>		LOCATION <u>4717095.098 N, 311284 224 E</u>		ORIGINATED BY <u>D.J.M.</u>	
DIST <u>32</u> HWY <u>40</u>		BOREHOLE TYPE <u>NW CASING / NQ ROCK CORING</u>		COMPILED BY <u>B. G.</u>	
DATUM <u>GEODETIC</u>		DATE <u>31.08.00 - 31.08.00</u>		CHECKED BY <u>[Signature]</u>	

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			20 40 60 80 100							W _p	W	W _L			
								SHEAR STRENGTH kPa										WATER CONTENT (%)		
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE												
144.56	Silty Clay, trace gravel, Firm, Grey		19	SS	1		146							Sa. 19 lost						
								145												
32.16	Sand fine to medium, trace silt, Loose, Grey						144													
143.95																				
32.77	Sandy Silt, trace clay, gravel, (Till), Very Dense, Grey		20	SS	62		143													
142.28			21	SS	111/254m		142													
34.44	Shale Bedrock. Slightly weathered and thinly bedded to 34.7m, becoming more massive and darker with depth, Dark Grey to Black							141												
			22	NQ RC			140													
139.14																				
37.58	END OF BOREHOLE Creek water level encountered in borehole at elev. 175.01m during drilling Aug. 31, 2000																			

ON_MOT_001-3045.GPJ ON_MOT.GDT 19-09-00 DATA INPUT:

SITE LOCATION MAP

FIGURE 1

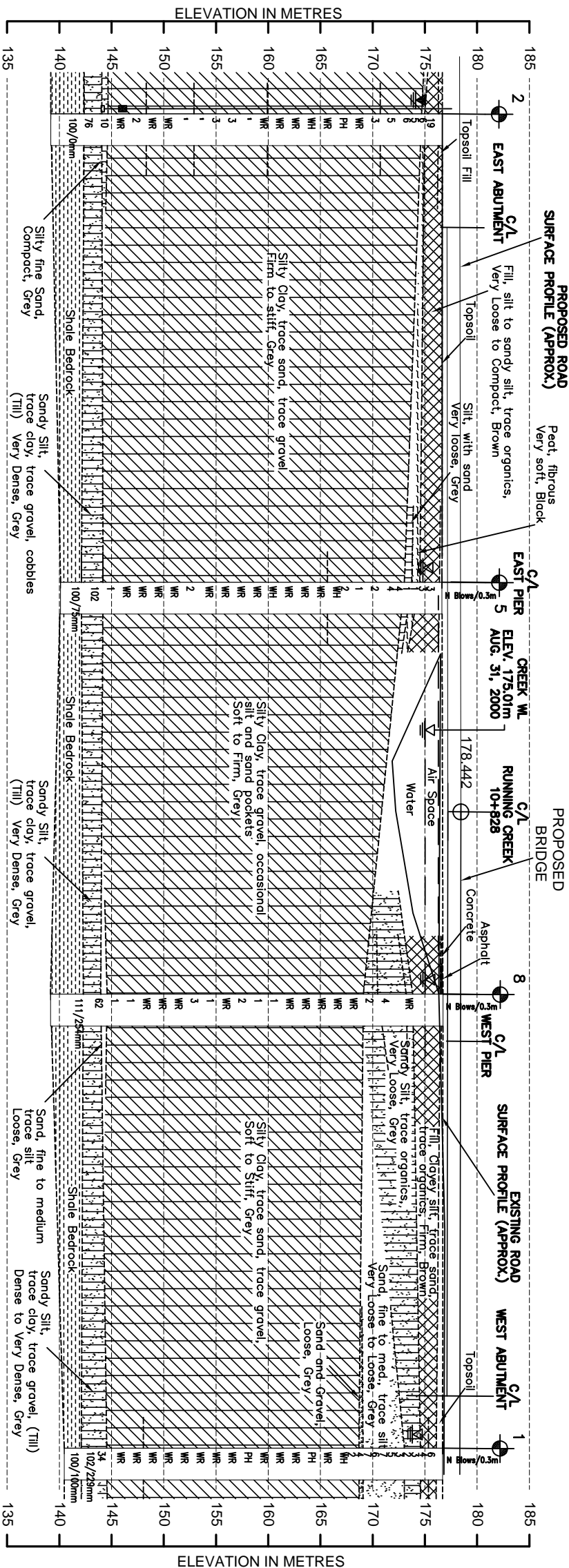
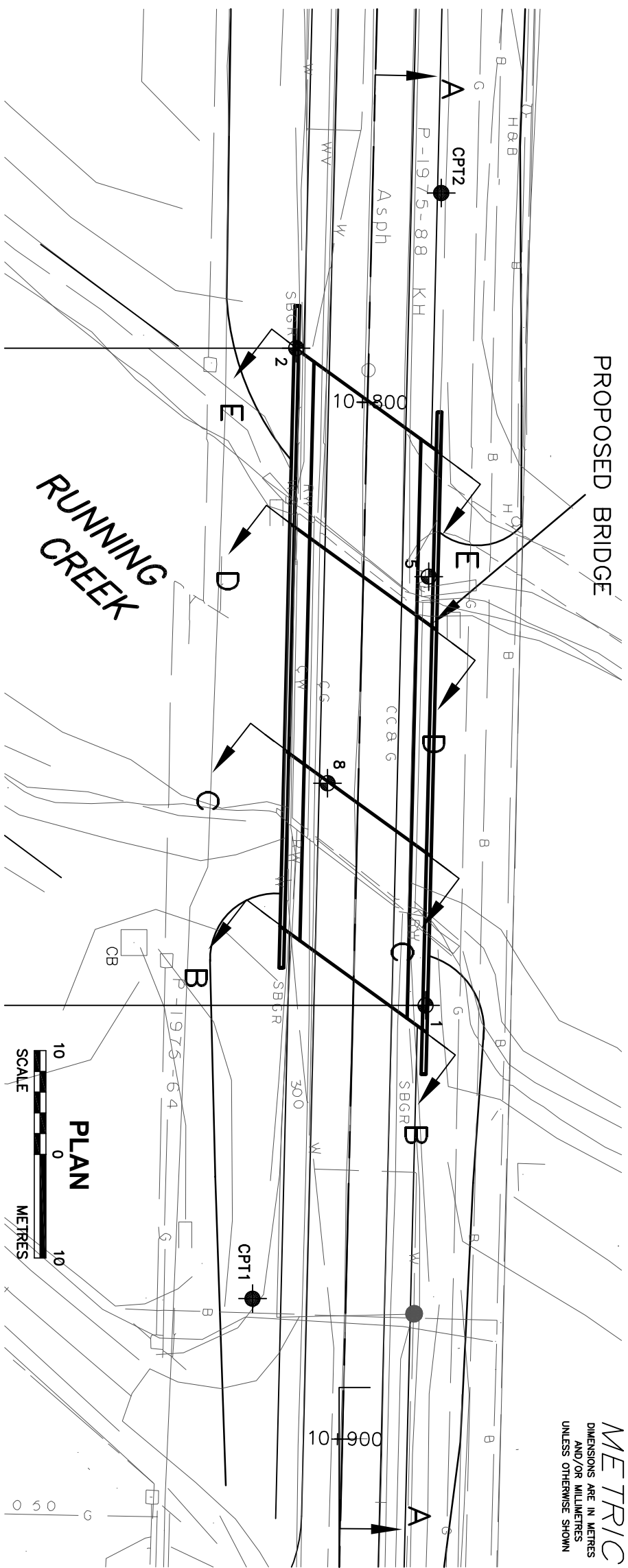




Date SEPT. 14, 2000
Project 001-3045

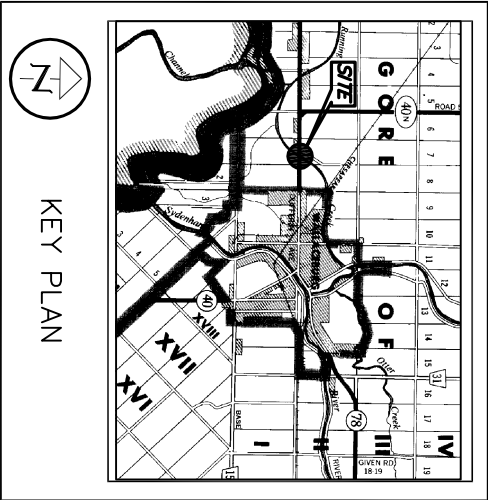


REFERENCE
RAND McNALLY AND Co.
PUBLISHED BY: ALLMAPS CANADA LTD.
SCALE: 1:250,000

Drawn B.G.
Chkd



<p>CONT. No. WP No. 60-99-00</p>	
<p>HIGHWAY 40 RUNNING CREEK BRIDGE BOREHOLE LOCATIONS & SOIL STRATA</p>	<p>SHEET</p>
<div style="display: flex; justify-content: space-between; align-items: center;"> <div style="text-align: right;">  <p>Golder Associates Ltd.</p> </div> <div style="text-align: left;"> <p>Golder Associates Ltd. LONDON, ONTARIO, CANADA</p> </div> </div>	



LEGEND			
<p>Borehole by Golder Associates</p> <p>Dynamic Cone Penetration Test by Golder Associates</p> <p>Seal</p> <p>Piezometer</p>			
N WL WL during drilling	Blows/0.3m (Std. Pen. Test: 4.75 j/blow) WL in piezometer Aug. / Sept. 2000		
No.	ELEVATION (metres)	CO-ORDINATES NORTHING	EASTING
1	176.70	4717085.655	311262.796
2	176.67	4717098.124	311326.176
5	176.64	4717085.549	311304.144
8	176.72	4717095.098	311284.224
CPT1	176.24	4717102.331	311234.516
CPT2	176.50	4717084.143	311341.133

NOTES

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

NO.	DATE	BY
		REVISION


HWY. No.	40	PROJECT NO.:	001-3045
SUB/M.D.	-	CHKD. AMH	
DRAWN: WDF	CHKD. AMH	DATE: APR. 2002	
		APPD.	FIGURE 2

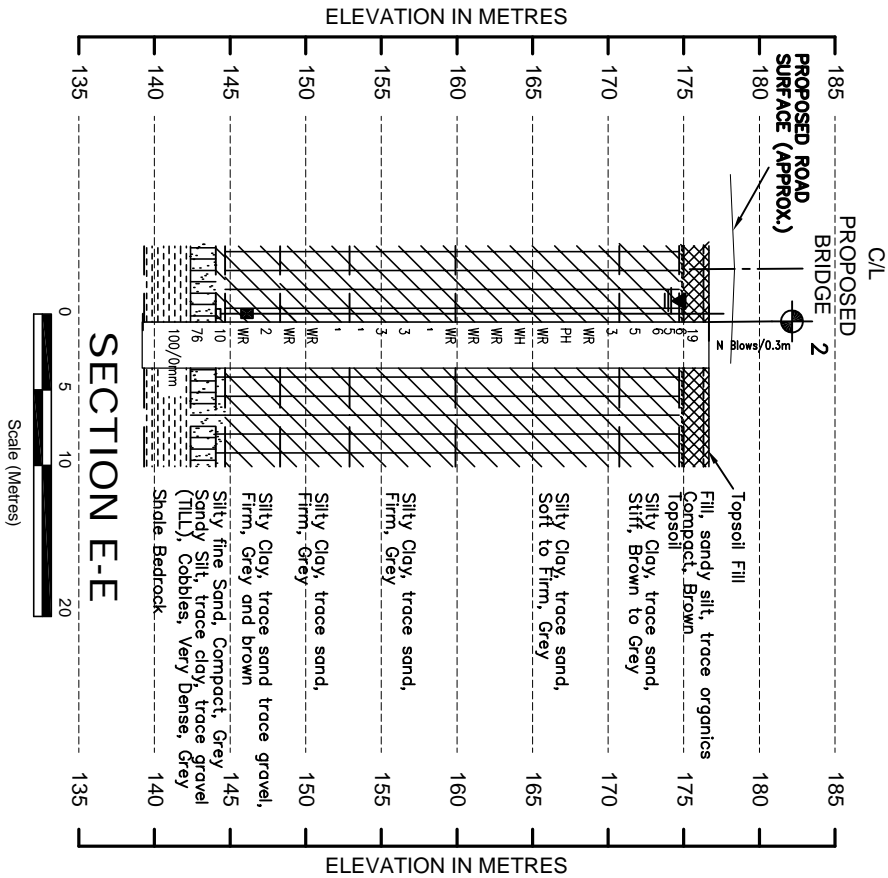
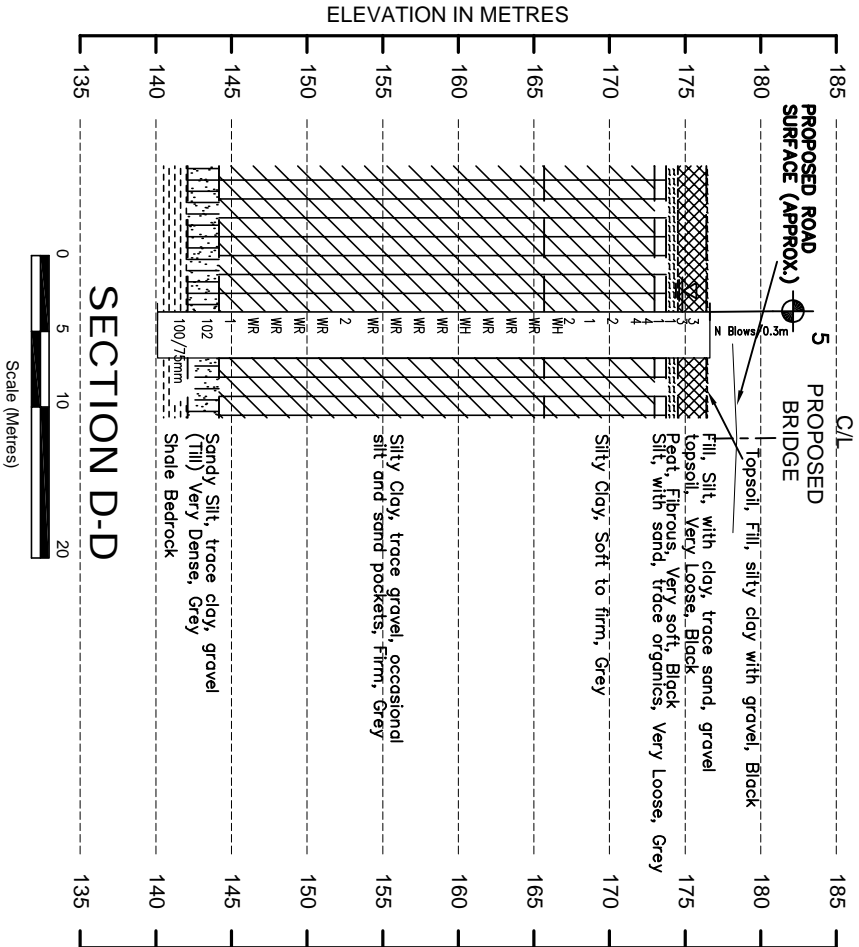
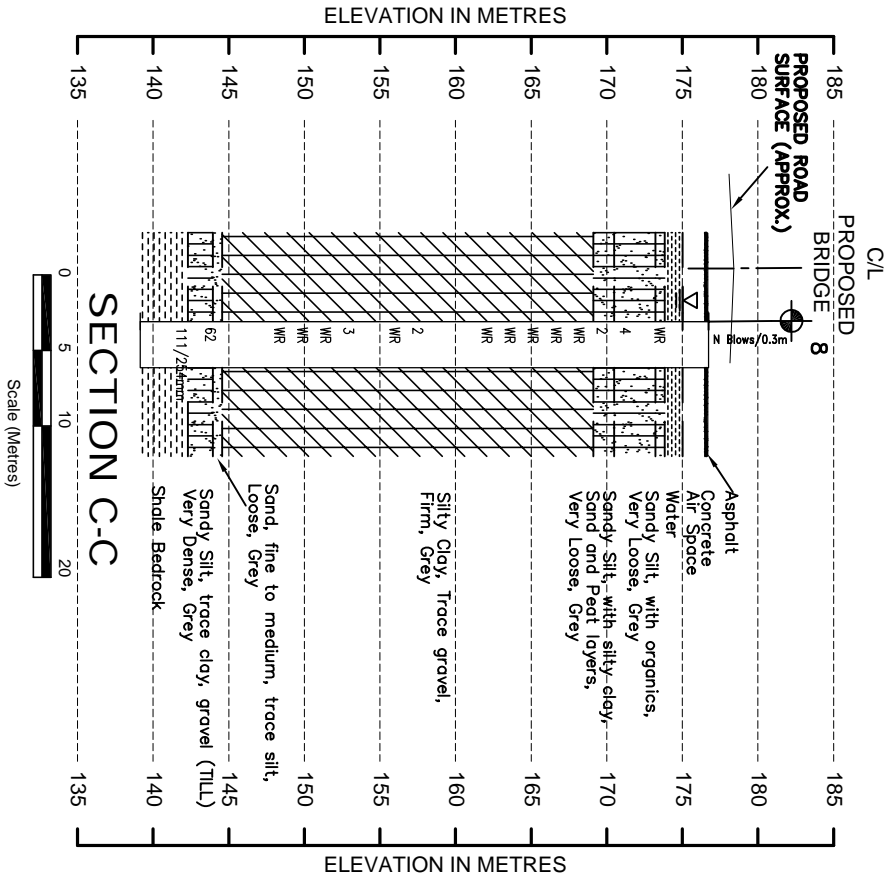
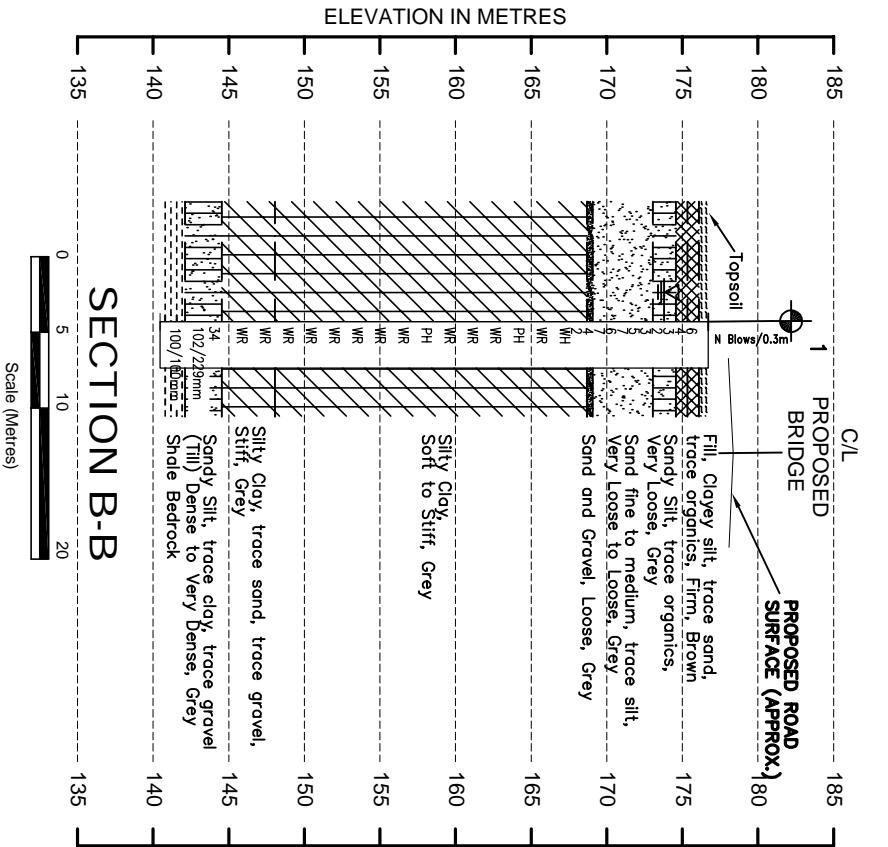
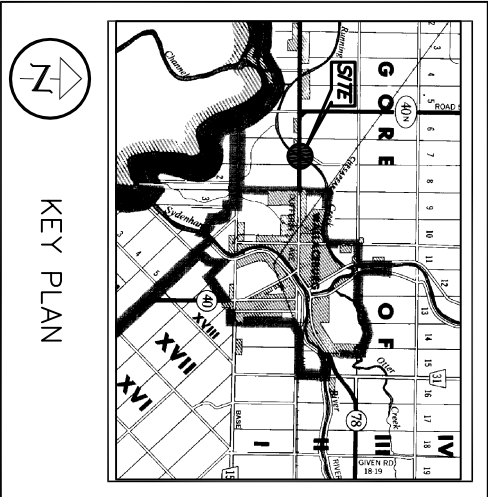
METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN







CONT. No.
WP No. 60-99-00

SHEET

HIGHWAY 40
RUNNING CREEK BRIDGE
SOIL STRATA CROSS SECTIONS

**Golder Associates Ltd.**
LONDON, ONTARIO, CANADA



LEGEND			
	Borehole by Golder Associates		
	Seal		
	Piezometer		
	N Blows/0.3m (Std. Pen. Test, 475 j/blow)		
	WL in piezometer Aug. / Sept. 2000		
	WL during drilling		
No.	ELEVATION (metres)	CO-ORDINATES NORTHING EASTING	
1	176.70	4717085.655	311262.796
2	176.67	4717098.124	311326.176
5	176.64	4717085.349	311304.144
8	176.72	4717095.098	311284.224

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

NO.	DATE	BY	REVISION

Geocres No. 40J9-20			
HMT. No.	40	PROJECT NO.:	001-3045
SUBM'D.	-	CHKD.	AMH
DATE:	APR. 2002	DATE:	APR. 2002
DRAWN:	WDF	CHKD.	AMH
APPD.			
		FIGURE	3

METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

DIST 32 CHATHAM
CONT. No.
WP No. 60-99-00

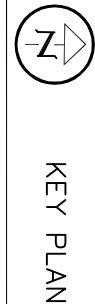
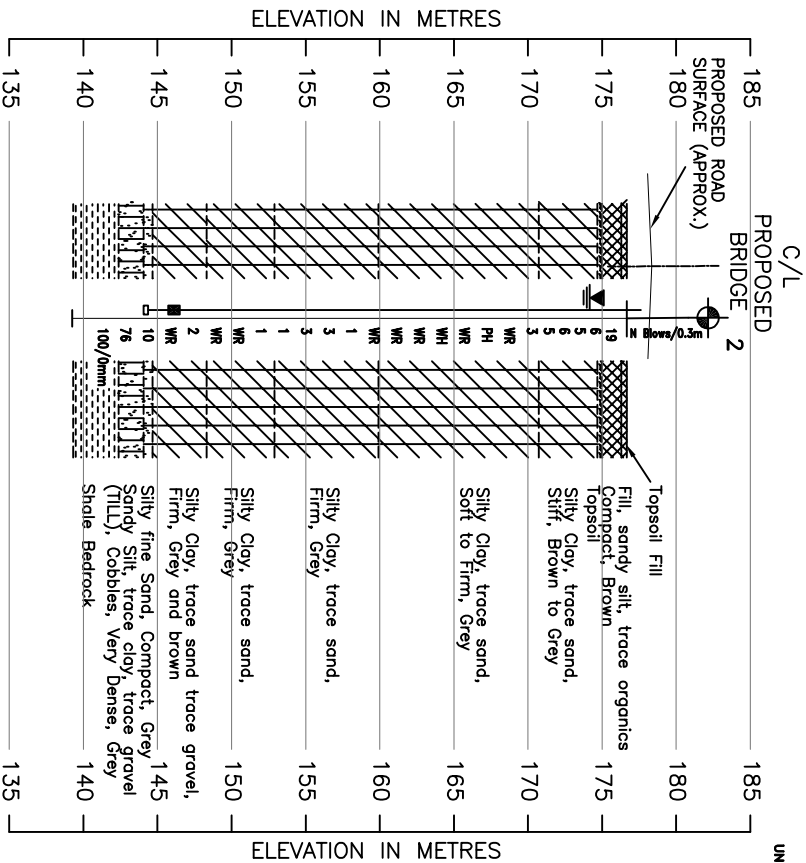
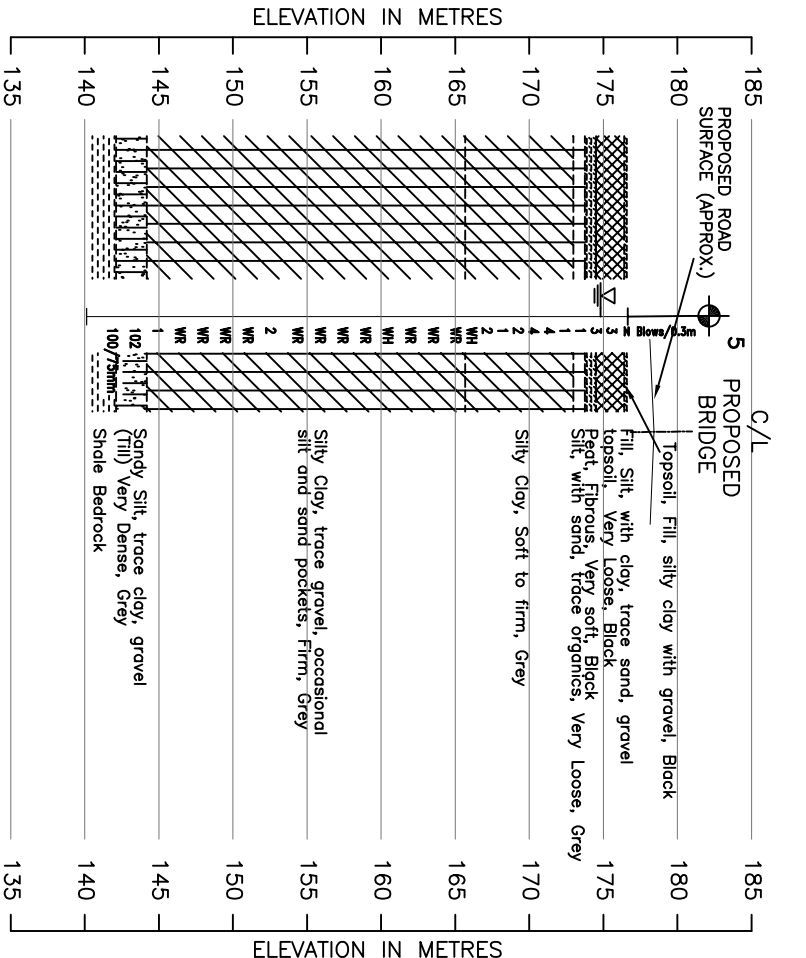


HIGHWAY 40
RUNNING CREEK BRIDGE
SOIL STRATA CROSS SECTIONS

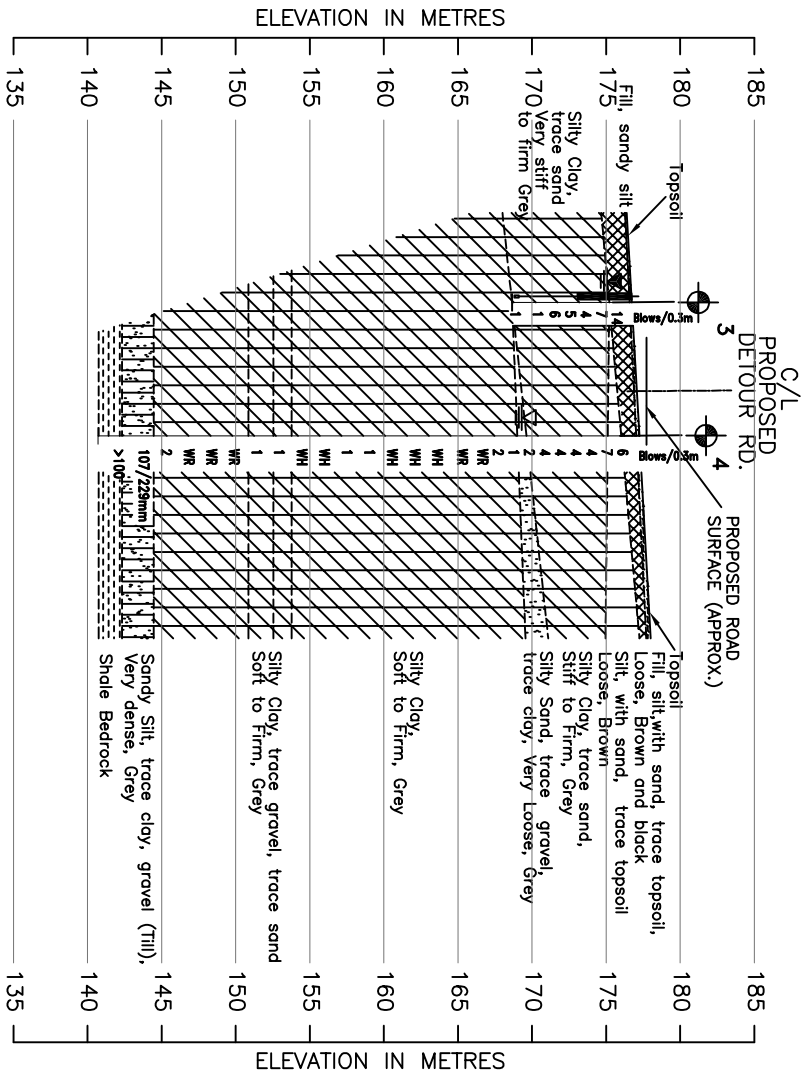
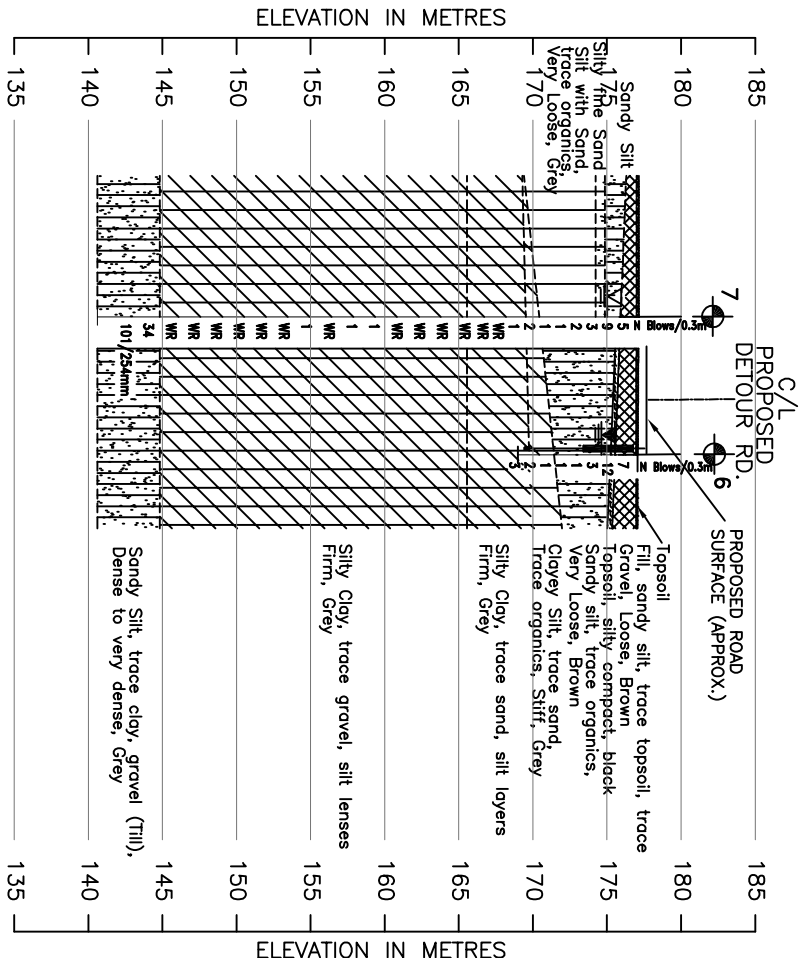
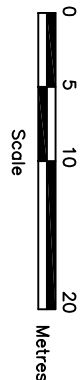
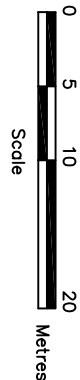
SHEET



Golder Associates Ltd.
LONDON, ONTARIO, CANADA



KEY PLAN



LEGEND

- Borehole by Golder Associates
- Seal
- Piezometer
- Blows/0.3m (Std. Pen. Test, 475 J/blow)
- WL in piezometer on Aug./Sept. 2000
- WL during drilling

No.	ELEVATION	LOCATION	NORTHING	EASTING
2	176.67		4717098.124	311326.176
3	176.73		4717112.303	311328.402
4	176.60		4717119.480	311333.846
5	176.64		4717085.349	311304.144
6	177.08		4717131.555	311282.282
7	177.14		4717123.224	311277.919

NOTES

The boundaries between soil strata have been established by the Golder Associates Ltd. geotechnical investigation. Boundaries are assumed from geological evidence.

NO.	DATE	BY	REVISION

Geocres No. 40J9-20	PROJECT NO.: 001-3045	FIGURE 4
HWY. No. 40	CHKD: A.M.H. DATE: SEPT. 14, 00	
SUBWD.	CHKD: A.M.H. APPD.	
DRAWN: B.G.		

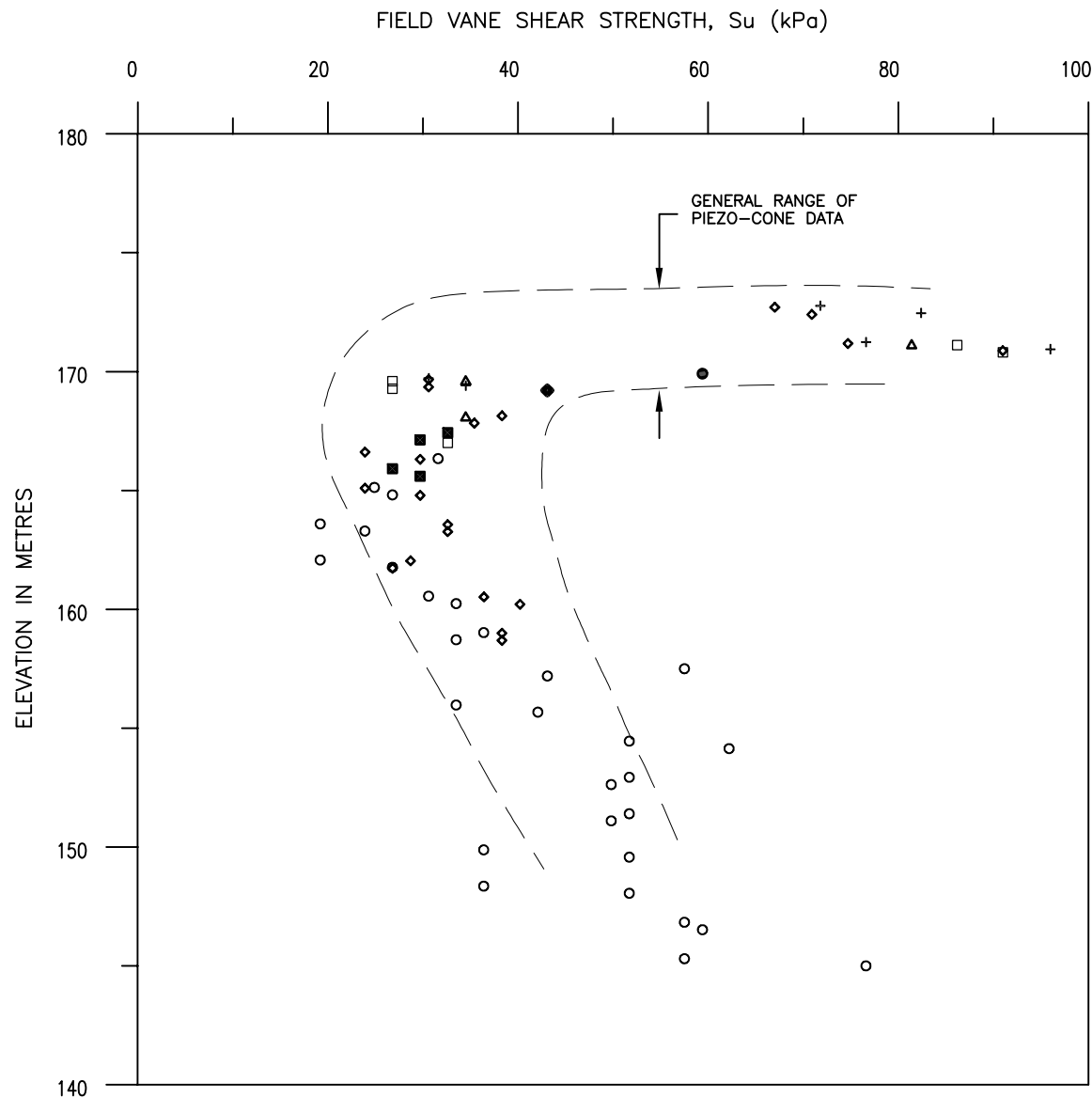
1 = 1 metric

D size dwg 22" x 32" 11" x 17" plot half scale

00304503.DWG

FIELD VANE SHEAR STRENGTH,
Su (kPa) VS. ELEVATION

FIGURE 5



LEGEND	
○ Borehole 1	△ Borehole 5
◇ Borehole 2	● Borehole 6
+ Borehole 3	◆ Borehole 7
□ Borehole 4	■ Borehole 8

APPENDIX A

PIEZO-CONE PENETRATION TEST RESULTS

GENERAL COMMENTS ON PIEZO-CONE PENETRATION TESTING

The piezo-cone penetration test (CPT) is a state-of-the-art, in-situ technique for geotechnical site characterization studies. The CPT consists of a special rod 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 of 2 cm/s into the ground using a drill rig. A reliable continuous stratigraphic profile, together with important engineering properties such as strength and density, can be interpreted from the CPT results.

Further, because the CPT data are collected on a continuous basis, detection of thin soft layers and pervious layers is possible. This is important as these zones control the behaviour and performance of the soil mass. These layers may go undetected by conventional drilling and sampling operations.

The CPT's were carried out on August 16, 2000 and pushed to depths of from 1.5 to 24.0 metres. The locations of the CPT's are shown on Figure 2.

The Cone Penetration Test sheets show the measured data recorded in the field (tip resistance corrected for end area effects (Q_t), porewater pressure (PWP) and sleeve friction resistance) and the two interpreted parameters of classification index (I_c) and undrained shear strengths (S_u). The classification index provides an inferred stratigraphy. The S_u values are obtained from correlation with the in-situ shear vane strengths and oedometer test results using a cone factor (N_k) of 17. It should be noted that the S_u values shown on the CPT records are only applicable when the I_c is greater than 2.6.

The porewater pressure plots shown on the CPT sheets include a hydrostatic porewater pressure line (dotted) based on the input water level.

The results from CPT 1 do not agree well with the shear vane strengths from Borehole 1. The borehole values are significantly lower between depths of about 10 m and 20 m below ground surface. The values appear to start matching up below a depth of about 20 m. This may be due to the presence of a former river/creek channel at the location of Borehole 1. The results from CPT 2 agree well with the data from Borehole 2.

The preconsolidation stress (σ'_p) in the silty clay has been established using the oedometer testing. The relationship between σ'_p at depth and S_u from CPT and in-situ vane shear strength is as follows:

$$S_u = 0.22 \sigma'_p$$

Where σ'_p = preconsolidation pressure (kPa)

S_u = in situ shear strength (kPa)

Reference for Robertson and Wride shown on cone plots (Ic determination)

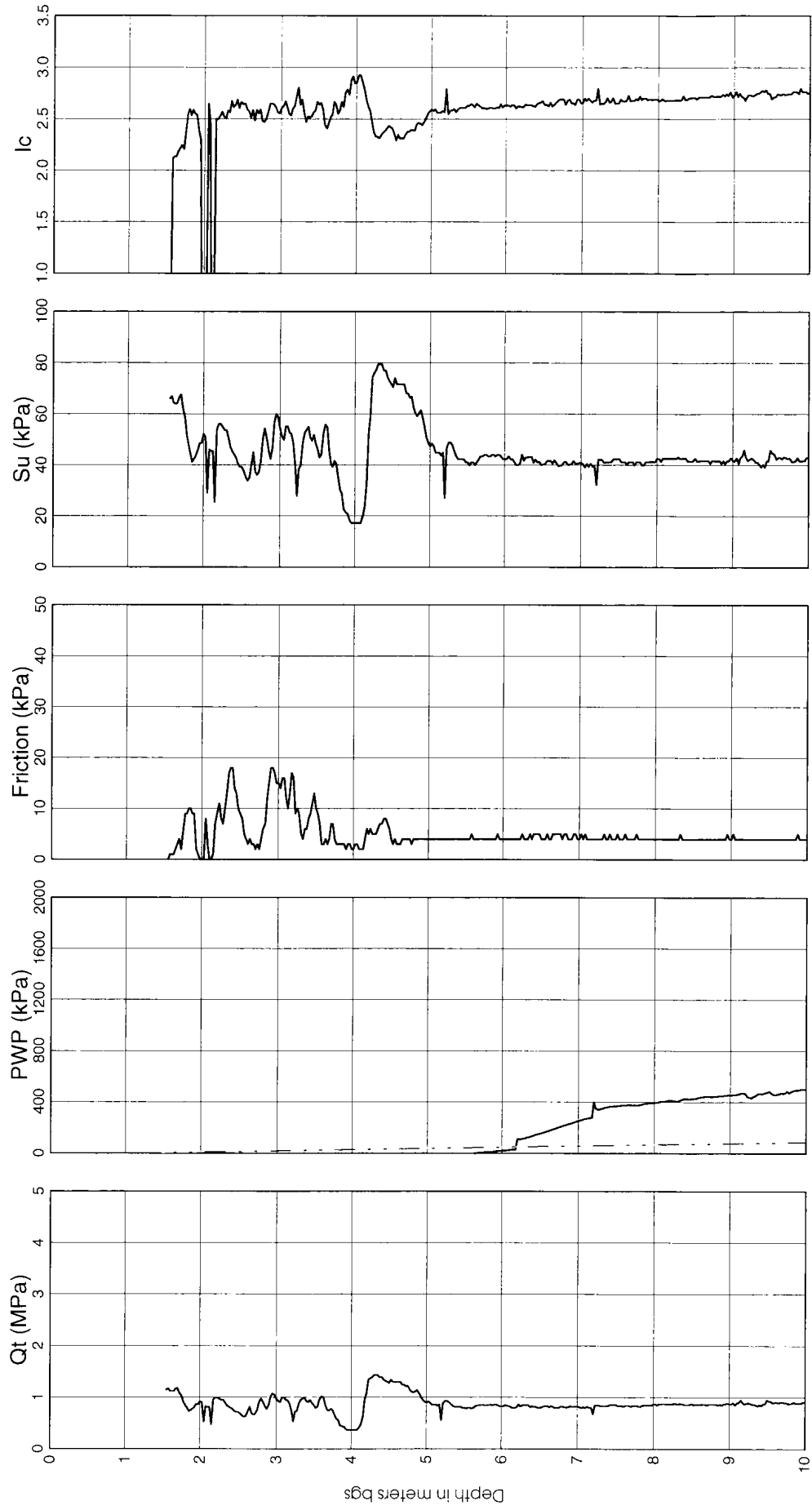
Robertson, P.K. and Wride, C.E. (1998) "Evaluating cyclic liquefaction potential using the Cone Penetration Test" CGJ, 35, pp 442-459

Cone Penetration Test - 1

Test Date : August 16, 2000
Location : see figure

Operator : Golder Associates

Ground Surf. Elev. : 176.24
Water Table Depth : 1.20



Qt normalized for unequal end area effects

$S_u = (Q_t - \text{Sigma } V) / N_k$

$N_k = 17$

$\text{Gamma} = 18 \text{ kN/m}^3$

After Robertson and Wride (1998)

$I_c < 1.31$ - Gravely sands

$1.31 < I_c < 2.05$ - Clean to silty sand

$2.05 < I_c < 2.60$ - Silty sand to sandy silt

$2.60 < I_c < 2.95$ - Clayey silt to silty clay

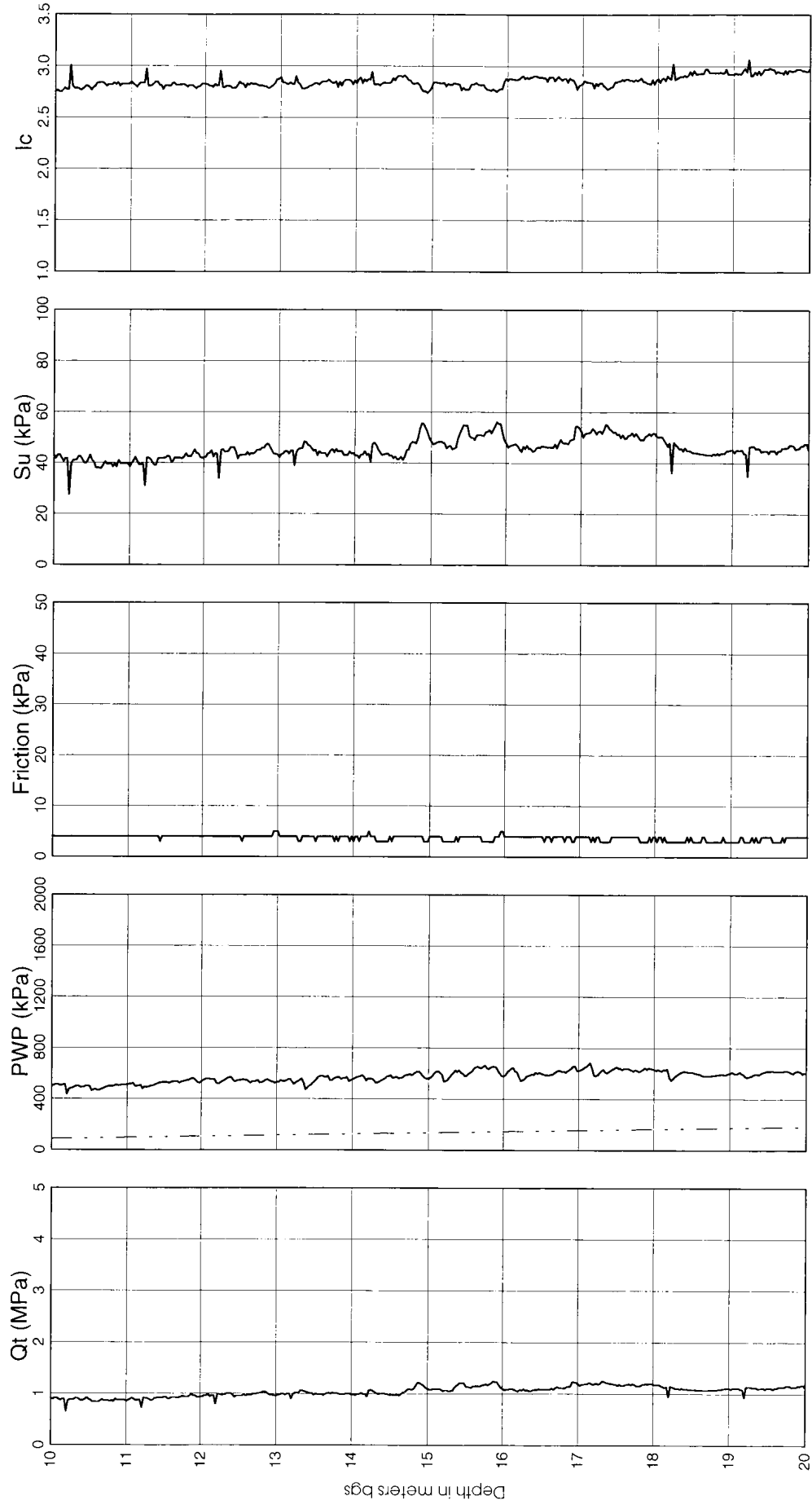
$2.95 < I_c < 3.60$ - Clays

Cone Penetration Test - 1

Test Date : August 16, 2000
Location : see figure

Operator : Golder Associates

Ground Surf. Elev. : 176.24
Water Table Depth : 1.20

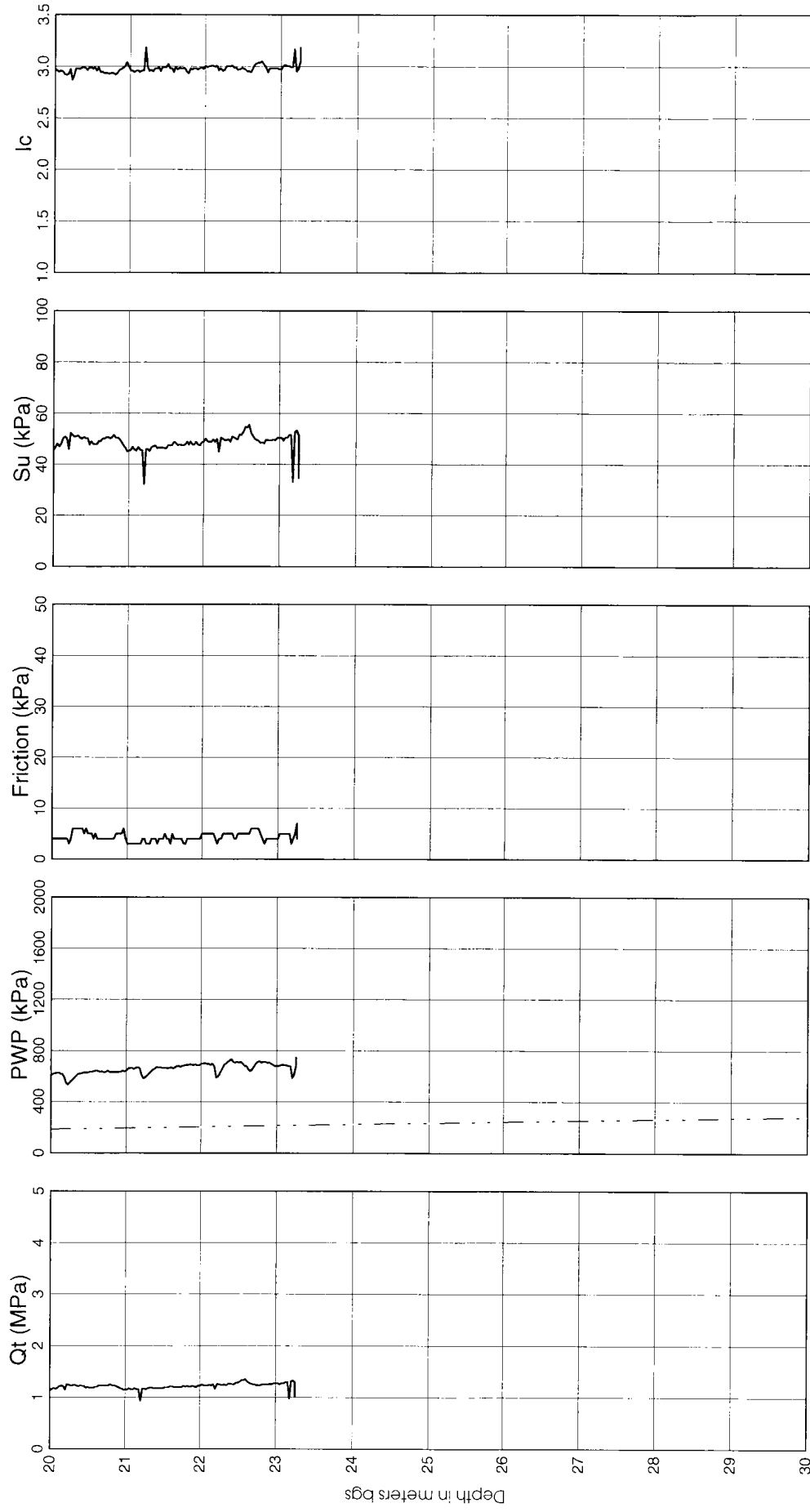


Cone Penetration Test - 1

Test Date : August 16, 2000
Location : see figure

Operator : Golder Associates

Ground Surf. Elev. : 176.24
Water Table Depth : 1.20



Qt normalized for
unequal end area effects

$S_u = (Q_t - \text{Sigma } V) / N_k$
 $N_k = 17$
 $\text{Gamma} = 18 \text{ kN/m}^3$

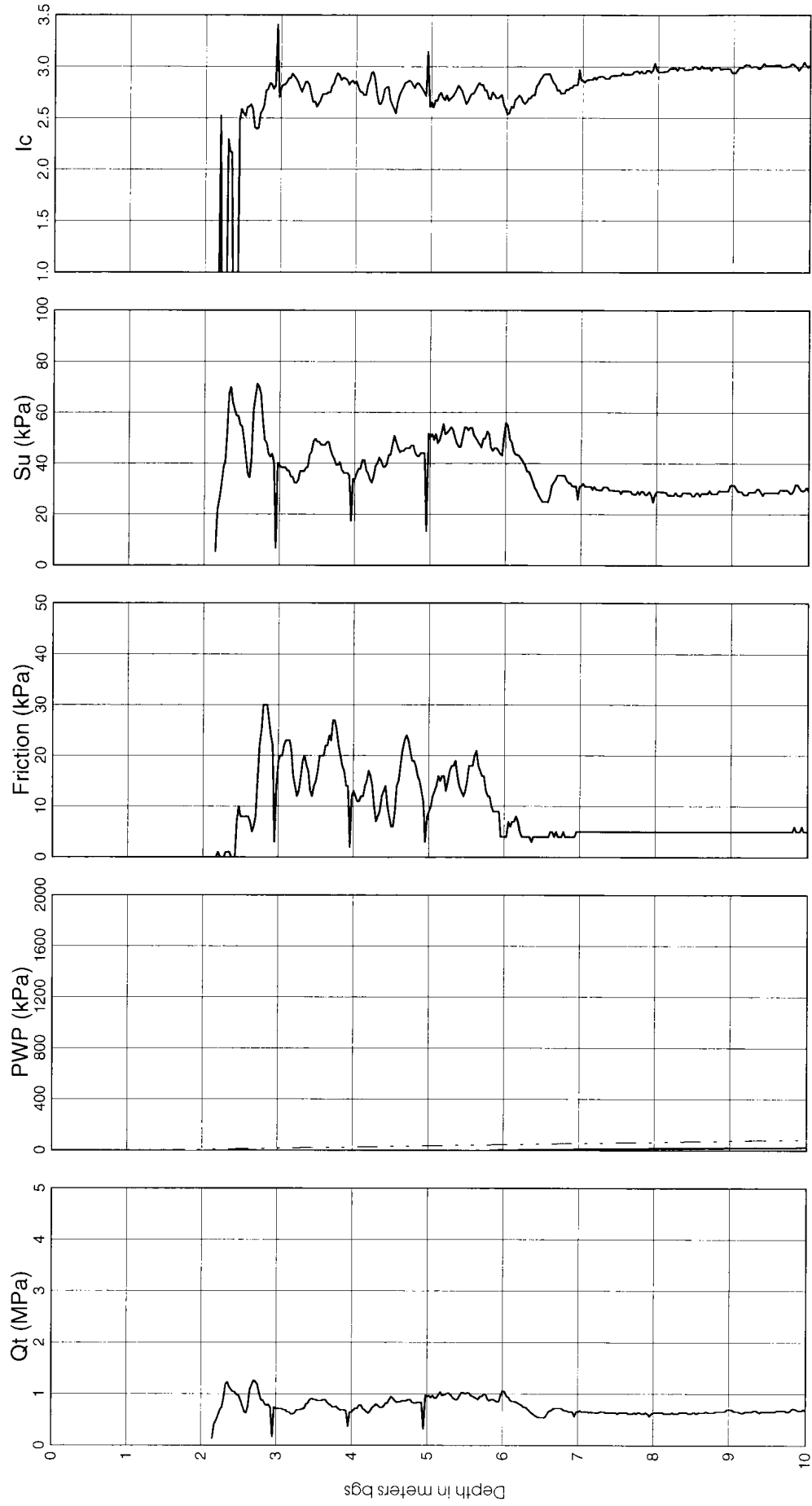
After Robertson and Wride (1998)
 $I_c < 1.31$ - Gravely sands
 $1.31 < I_c < 2.05$ - Clean to silty sand
 $2.05 < I_c < 2.60$ - Silty sand to sandy silt
 $2.60 < I_c < 2.95$ - Clayey silt to silty clay
 $2.95 < I_c < 3.60$ - Clays

Cone Penetration Test - 2

Test Date : August 16, 2000
Location : see figure

Operator : Golder Associates

Ground Surf. Elev. : 176.96
Water Table Depth : 1.50



Qt normalized for
unequal end area effects

$Su = (Q_t - \text{Sigma } V) / N_k$
 $N_k = 17$
 $\text{Gamma} = 18 \text{ kN/m}^3$

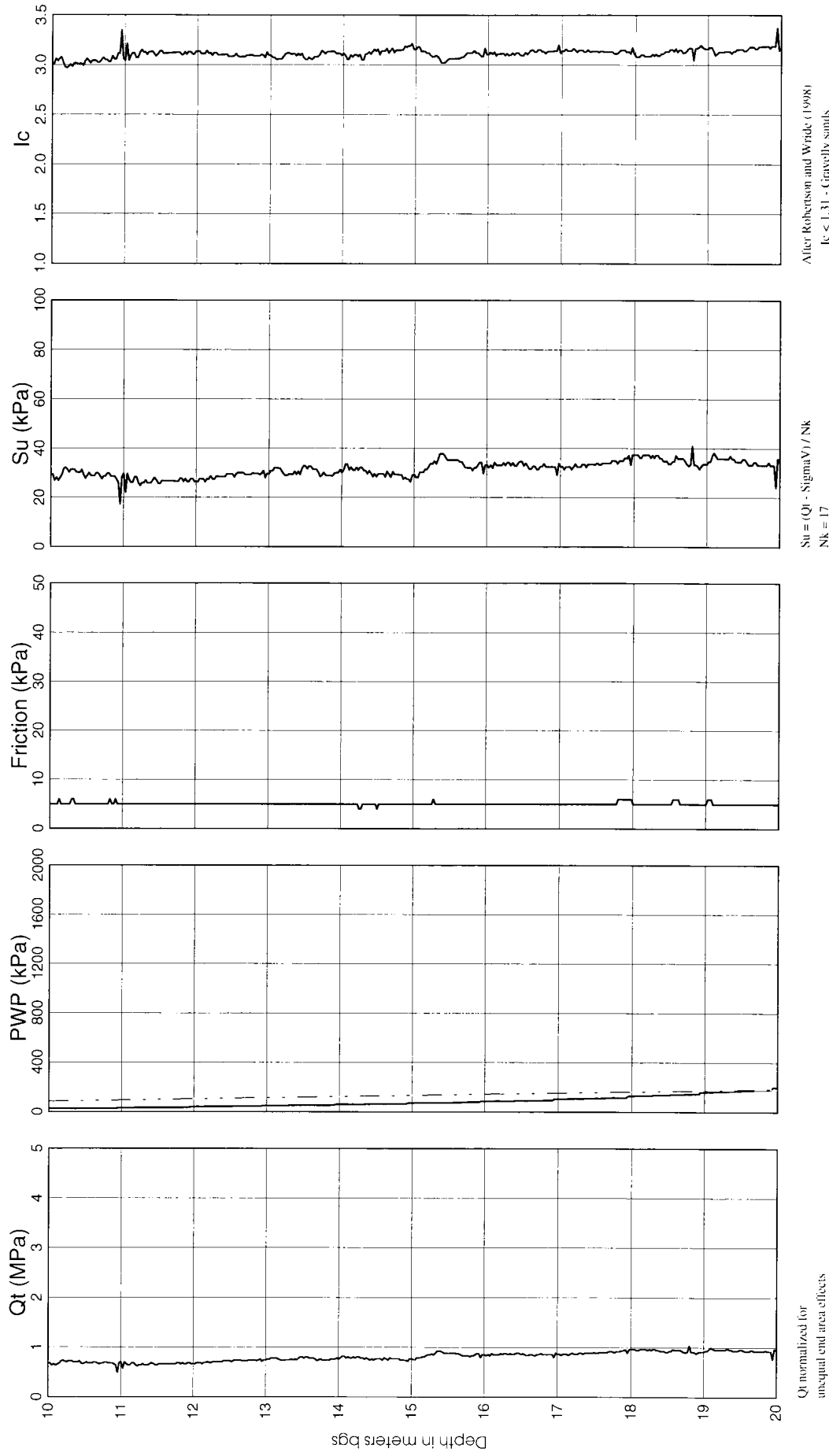
After Robertson and Wride (1998)
 $I_c < 1.31$ - Gravely sands
 $1.31 < I_c < 2.05$ - Clean to silty sand
 $2.05 < I_c < 2.60$ - Silty sand to sandy silt
 $2.60 < I_c < 2.95$ - Clayey silt to silty clay
 $2.95 < I_c < 3.60$ - Clays

Cone Penetration Test - 2

Test Date : August 16, 2000
Location : see figure

Operator : Golder Associates

Ground Surf. Elev. : 176.96
Water Table Depth : 1.50

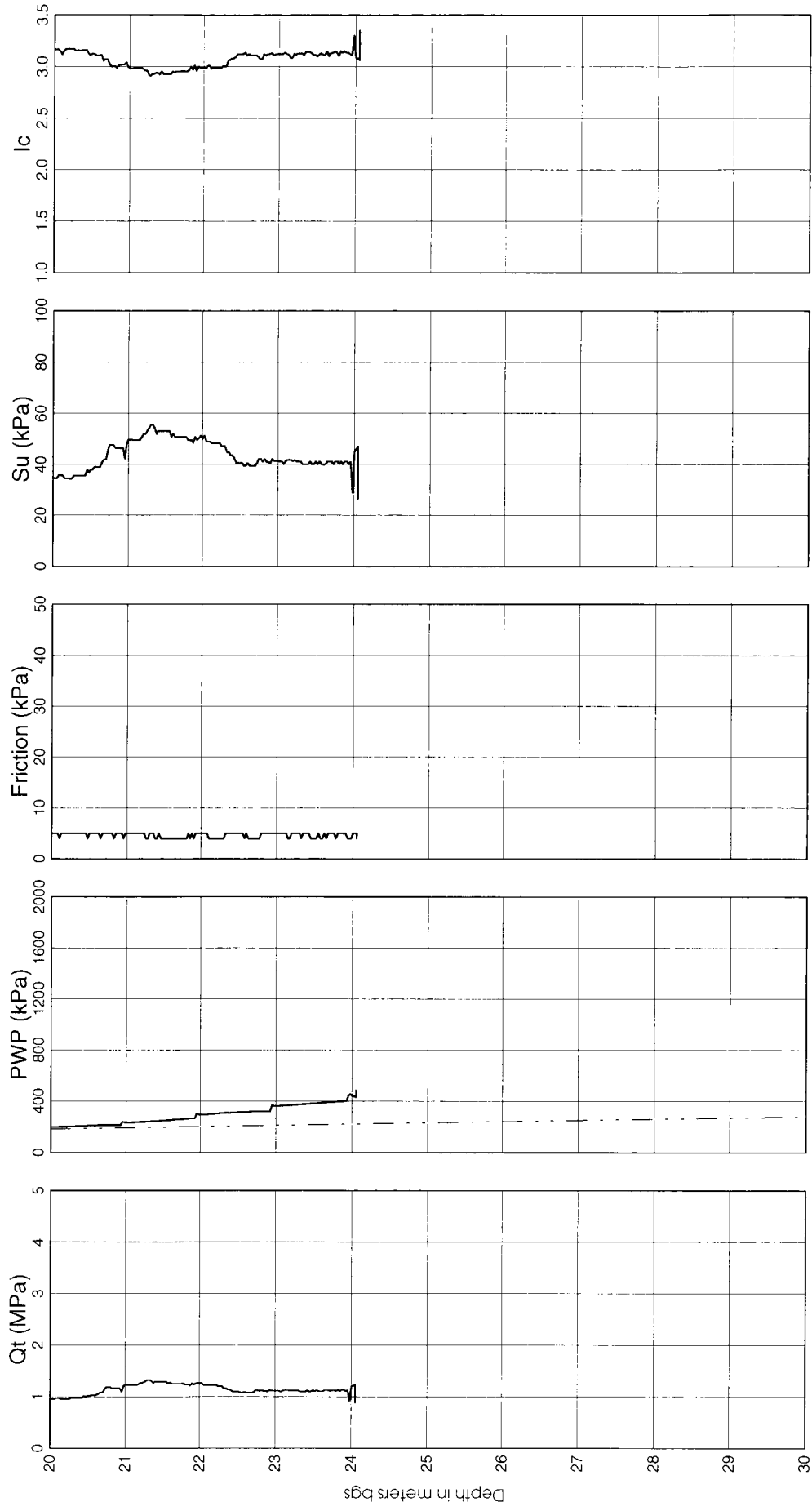


Cone Penetration Test - 2

Test Date : August 16, 2000
Location : see figure

Operator : Golder Associates

Ground Surf. Elev. : 176.96
Water Table Depth : 1.50



Qt normalized for
unequal end area effects

$S_u = (Q_t - \sigma_{vm}) / N_k$
 $N_k = 17$
 $\gamma_{\text{mean}} = 18 \text{ kN/m}^3$

After Robertson and Wride (1998)
 $I_c < 1.31$ - Gravely sands
 $1.31 < I_c < 2.05$ - Clean to silty sand
 $2.05 < I_c < 2.60$ - Silty sand to sandy silt
 $2.60 < I_c < 2.95$ - Clayey silt to silty clay
 $2.95 < I_c < 3.60$ - Clays

APPENDIX B

LABORATORY TEST DATA

TABLE B-I

SUMMARY OF LABORATORY TESTING RESULTS

Running Creek Bridge
Wallaceburg, Ontario
GWP 60-99-00

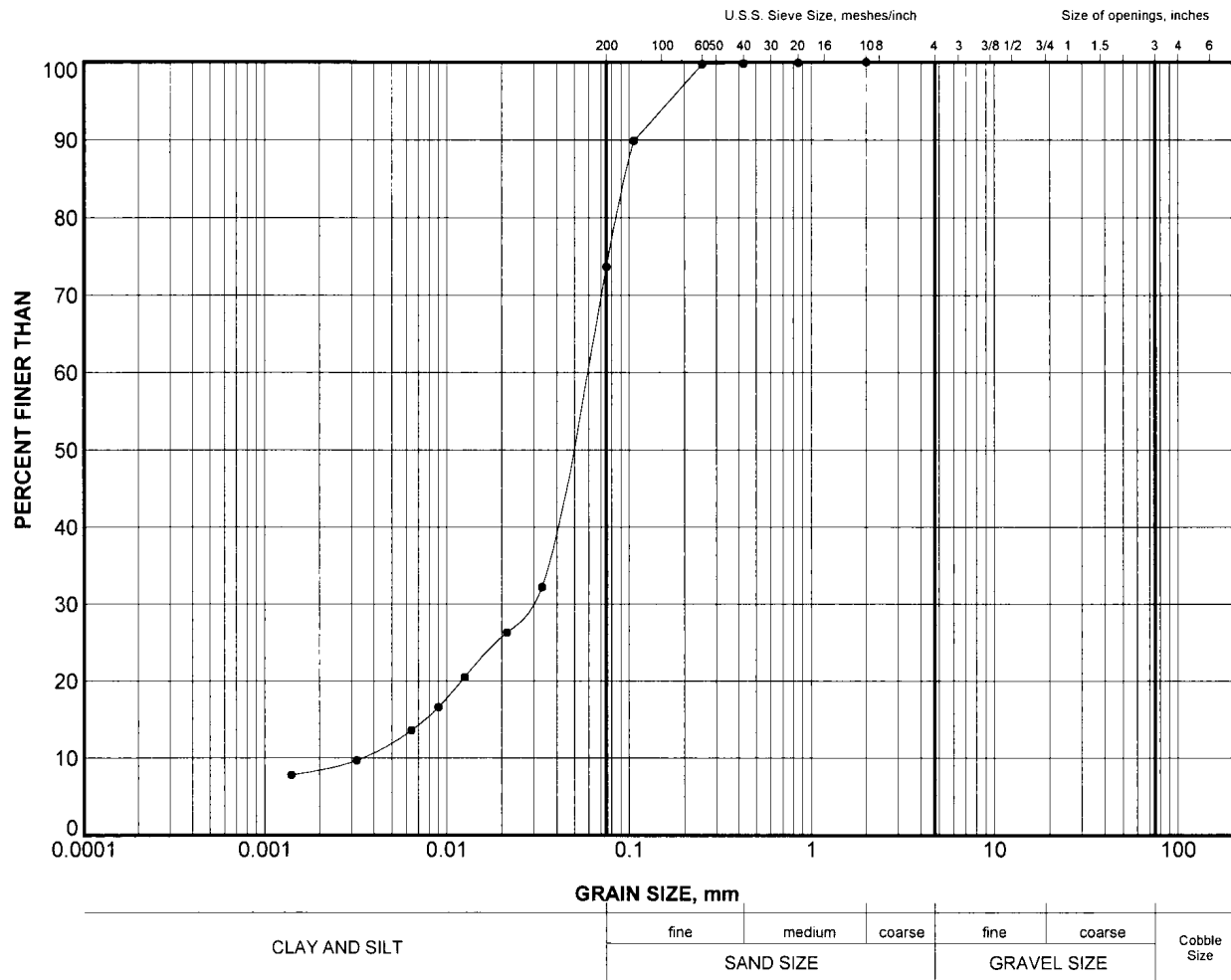
SOIL TYPE	TEST	NO. OF TESTS	AVERAGE (%)	MINIMUM (%)	MAXIMUM (%)
Topsoil	natural moisture content	2	36	23	48
Fill	natural moisture content	9	31	12	56
Peat	natural moisture content	1	65	-	-
Sand	natural moisture content	9	25	16	31
Sandy Silt	natural moisture content	15	31	15	55
Silty Clay	natural moisture content	105	40	26	57
Sandy Silt Till	natural moisture content	8	12	10	17
Weathered Shale Bedrock	natural moisture content	5	9	8	13
Silty Clay	liquid limit	14	42	36	55
Silty Clay	plastic limit	14	23	15	28
Silty Clay	specific gravity	2	2.7	2.70	2.71

NOTE: Table to be read in conjunction with accompanying report.

GRAIN SIZE DISTRIBUTION

FIGURE B-1

SANDY SILT



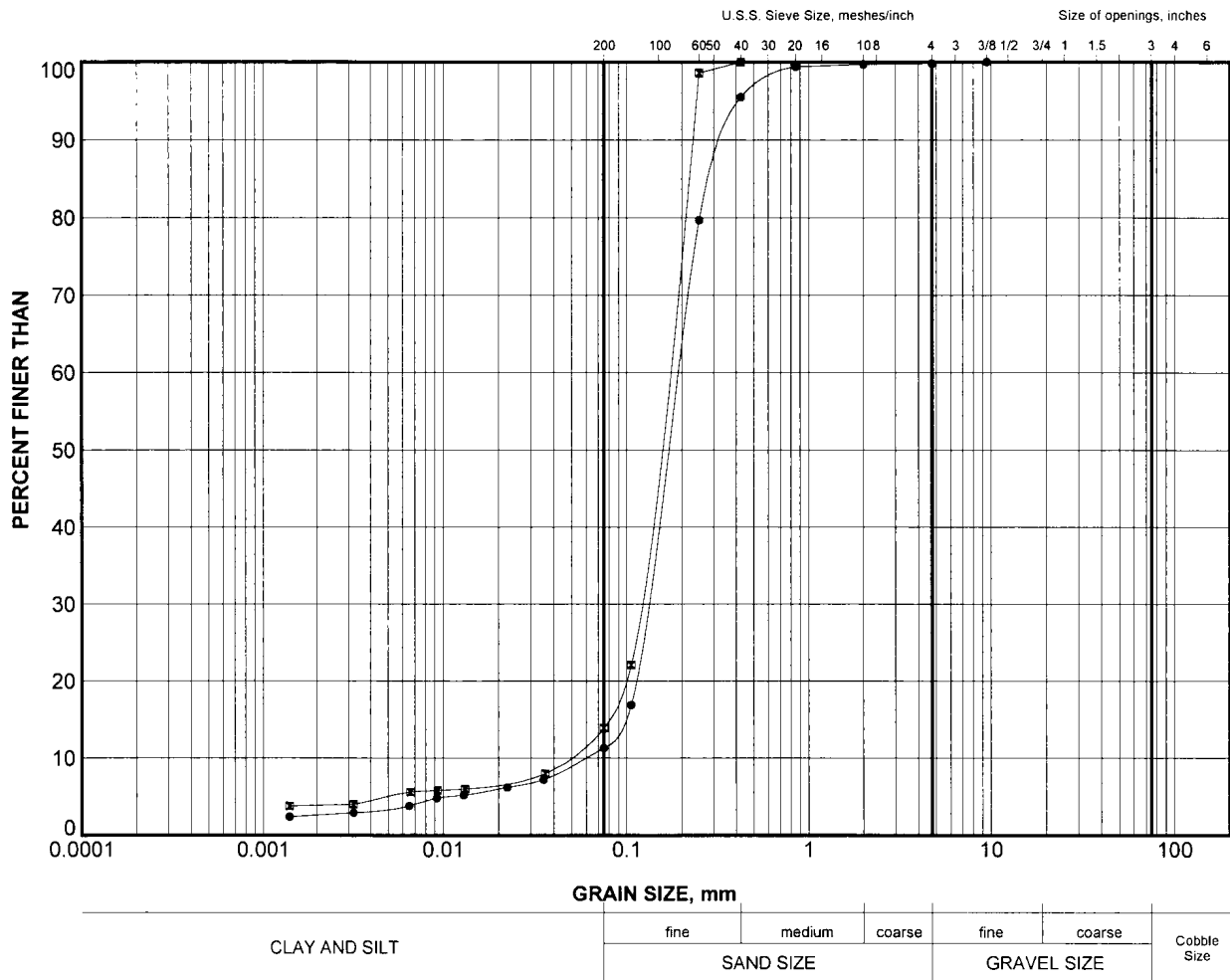
LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	1	3	174.2

GRAIN SIZE DISTRIBUTION

FIGURE B-2

Fine to Medium SAND



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	1	5	172.7
⊠	1	9	169.6

LDN_MTO_001-3045.GPJ GLDR LDN.GDT 17-09-00 DATA INPUT:

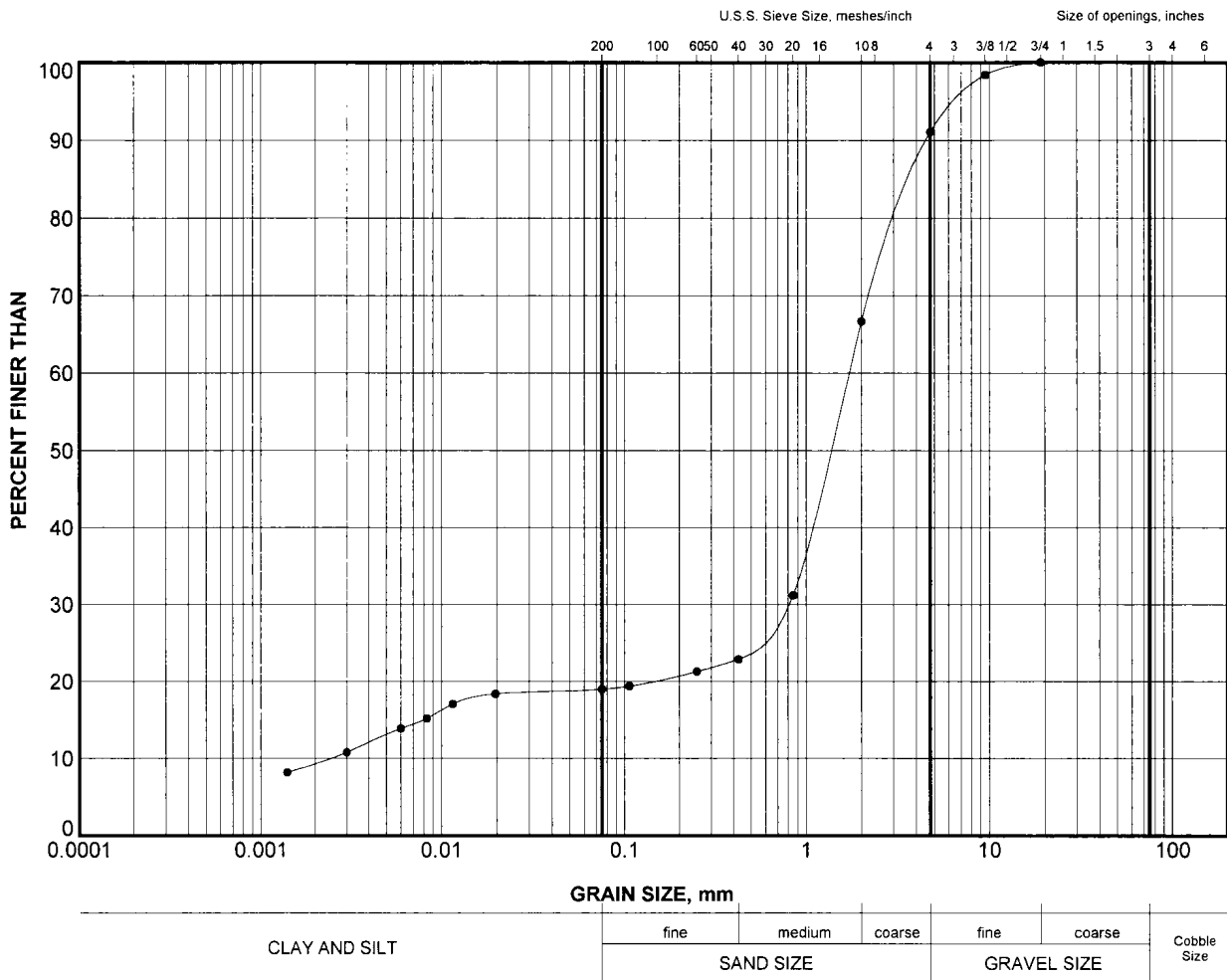
Project: 001-3045



GRAIN SIZE DISTRIBUTION

FIGURE B-3

SILTY SAND



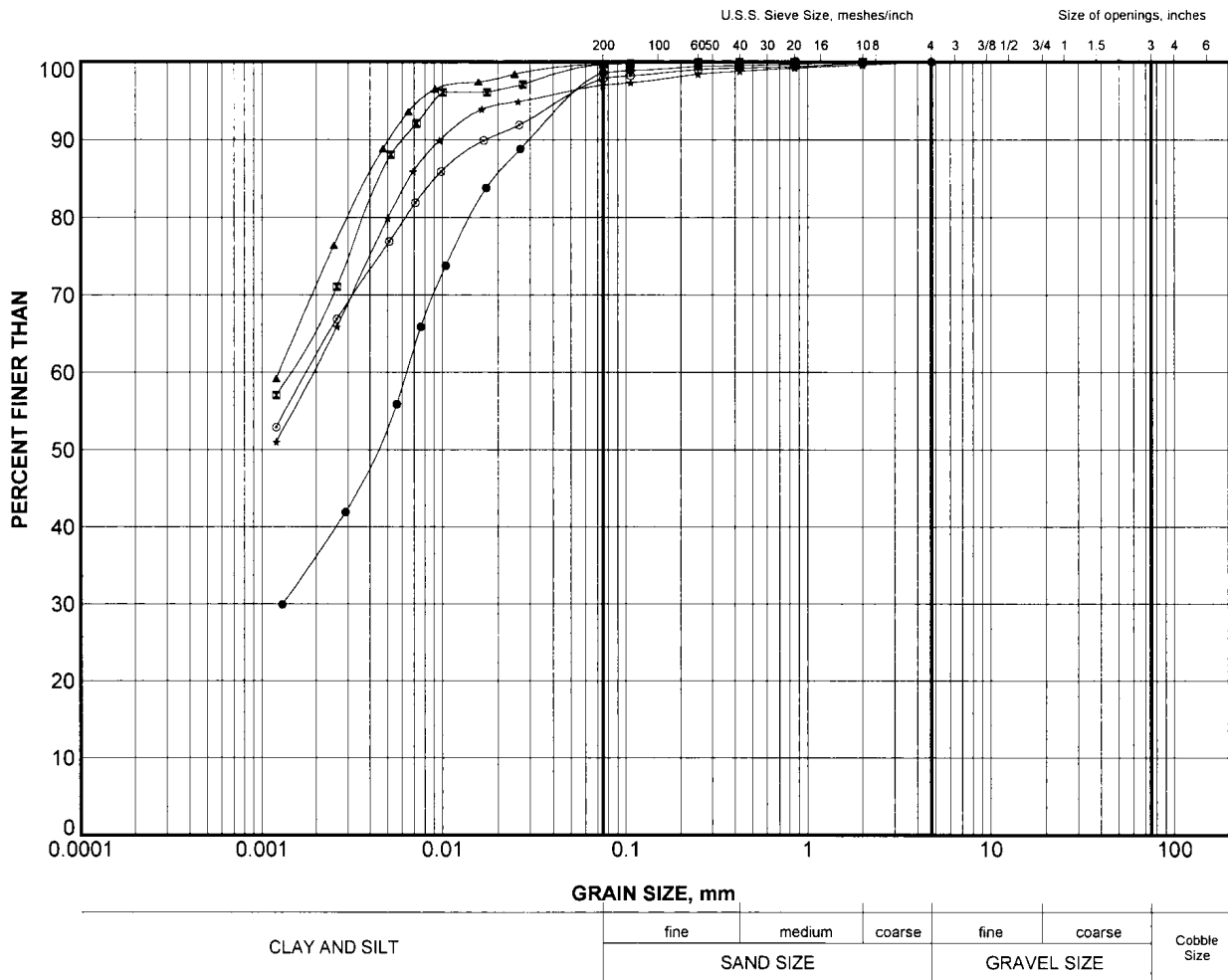
LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	4	8	168.7

GRAIN SIZE DISTRIBUTION

FIGURE B-4

SILTY CLAY



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	1	12	167.3
⊠	1	14	164.1
▲	1	17	159.7
★	1	21	153.6
⊙	1	25	147.5

LDN.MTO.001-3045.GPJ GLDR LDN.GDT 17-09-00 DATA INPUT:

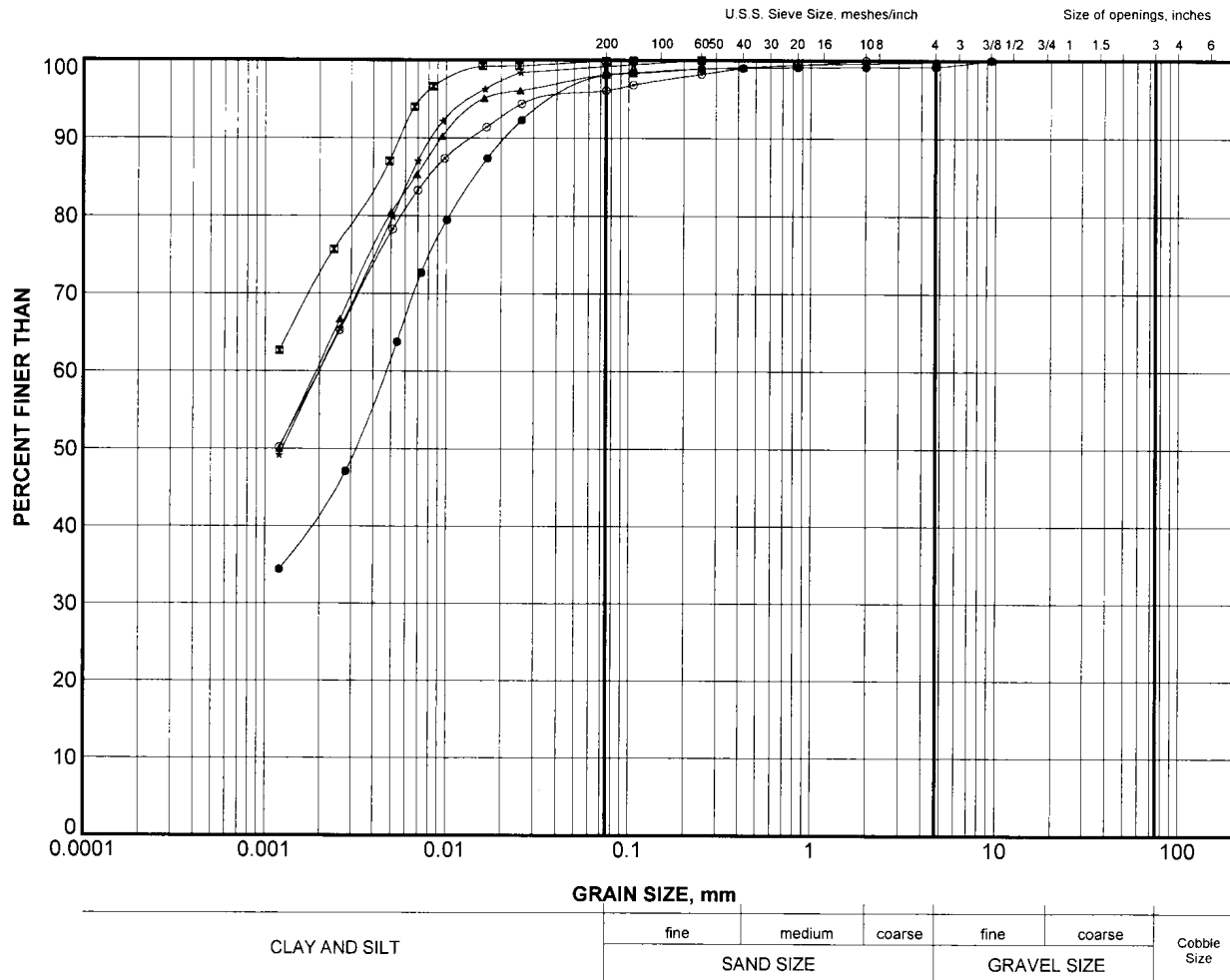
Project: 001-3045



GRAIN SIZE DISTRIBUTION

FIGURE B-5

SILTY CLAY

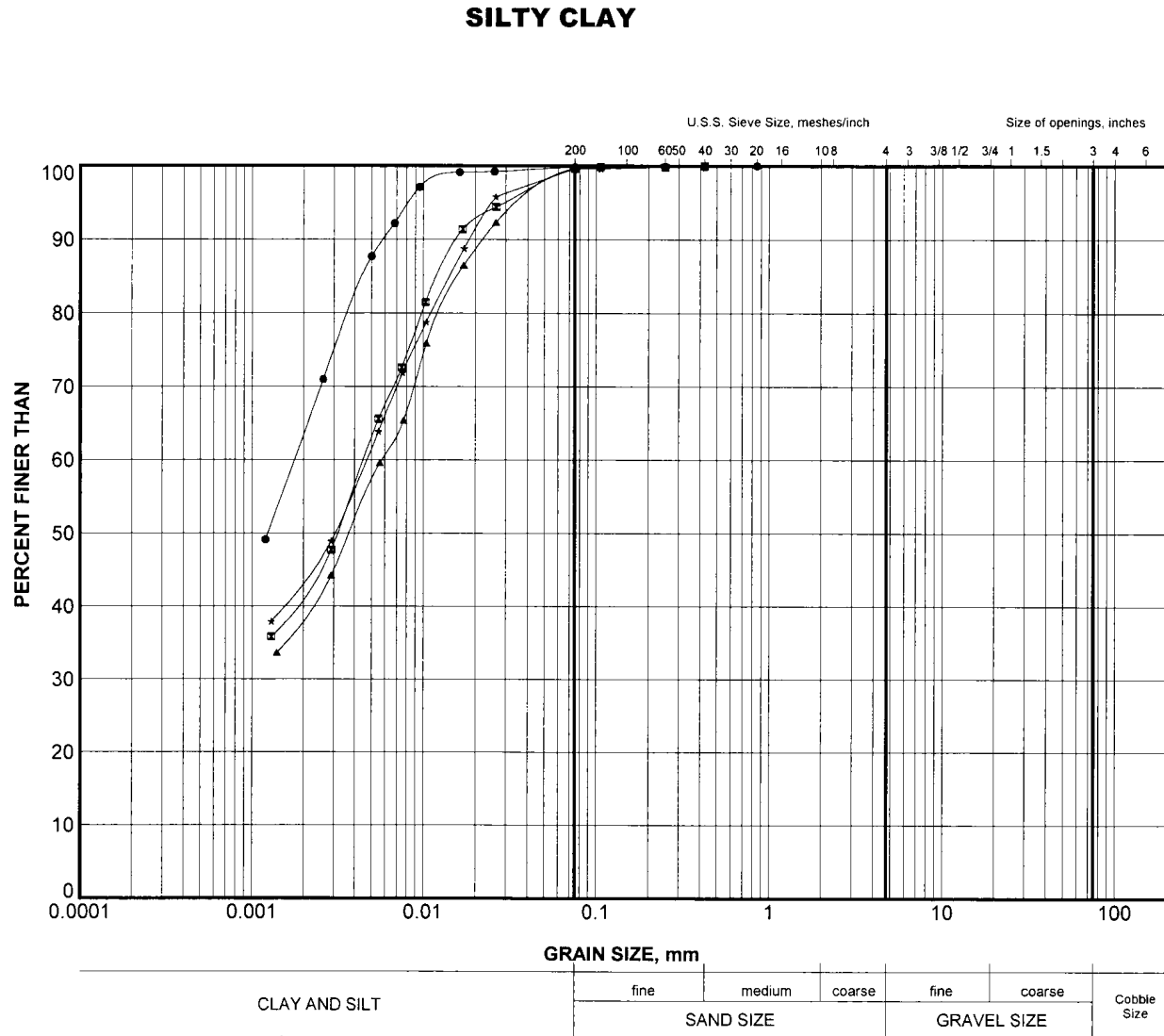


LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	2	5	171.9
⊠	2	8	167.1
▲	2	11	162.7
★	2	14	158.2
⊙	2	18	152.1

GRAIN SIZE DISTRIBUTION

FIGURE B-6

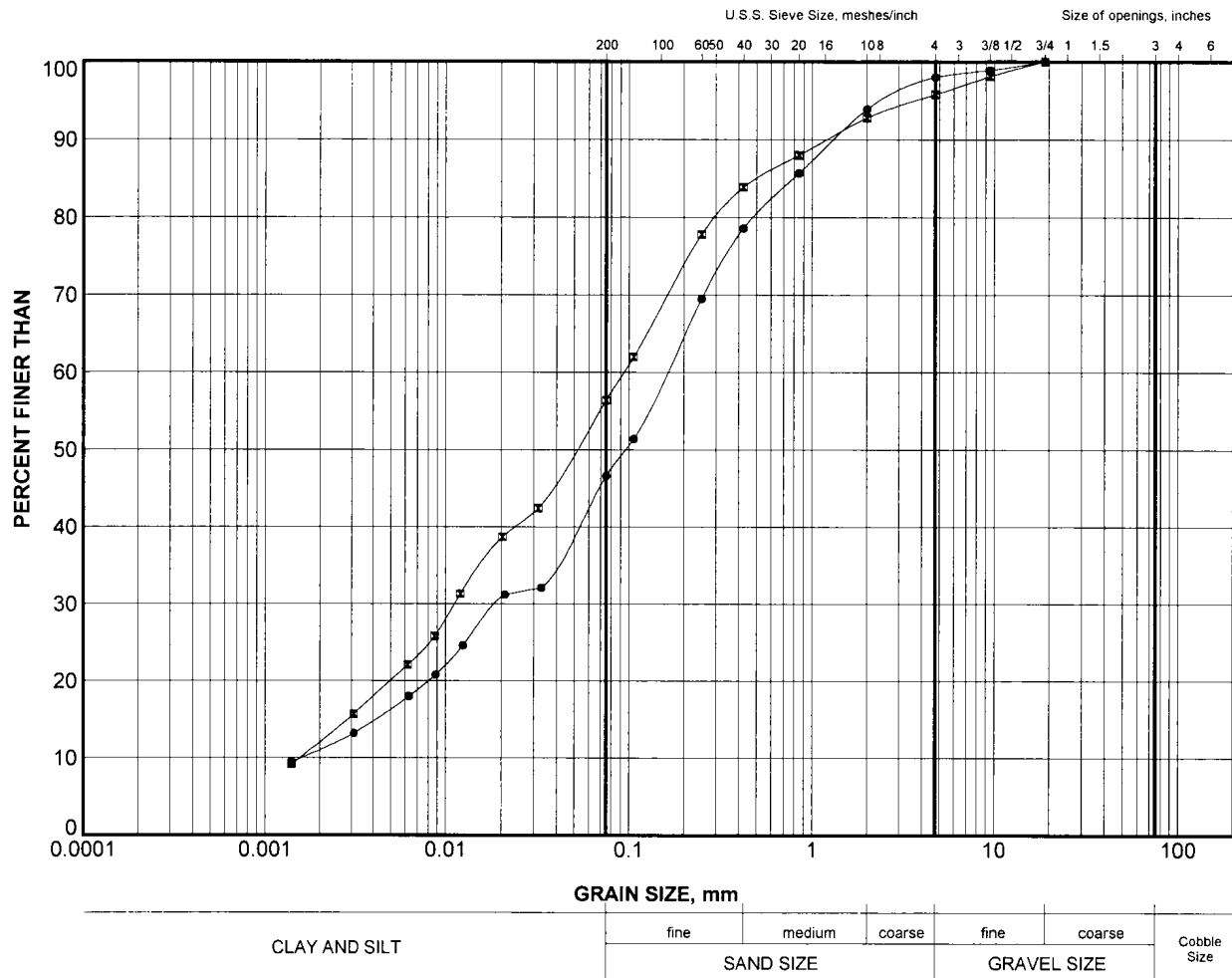


LEGEND			
SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	2	20	149.0
⊠	3	5	171.9
▲	3	7	168.8
★	4	4	173.3

GRAIN SIZE DISTRIBUTION

FIGURE B-7

SANDY SILT (TILL)



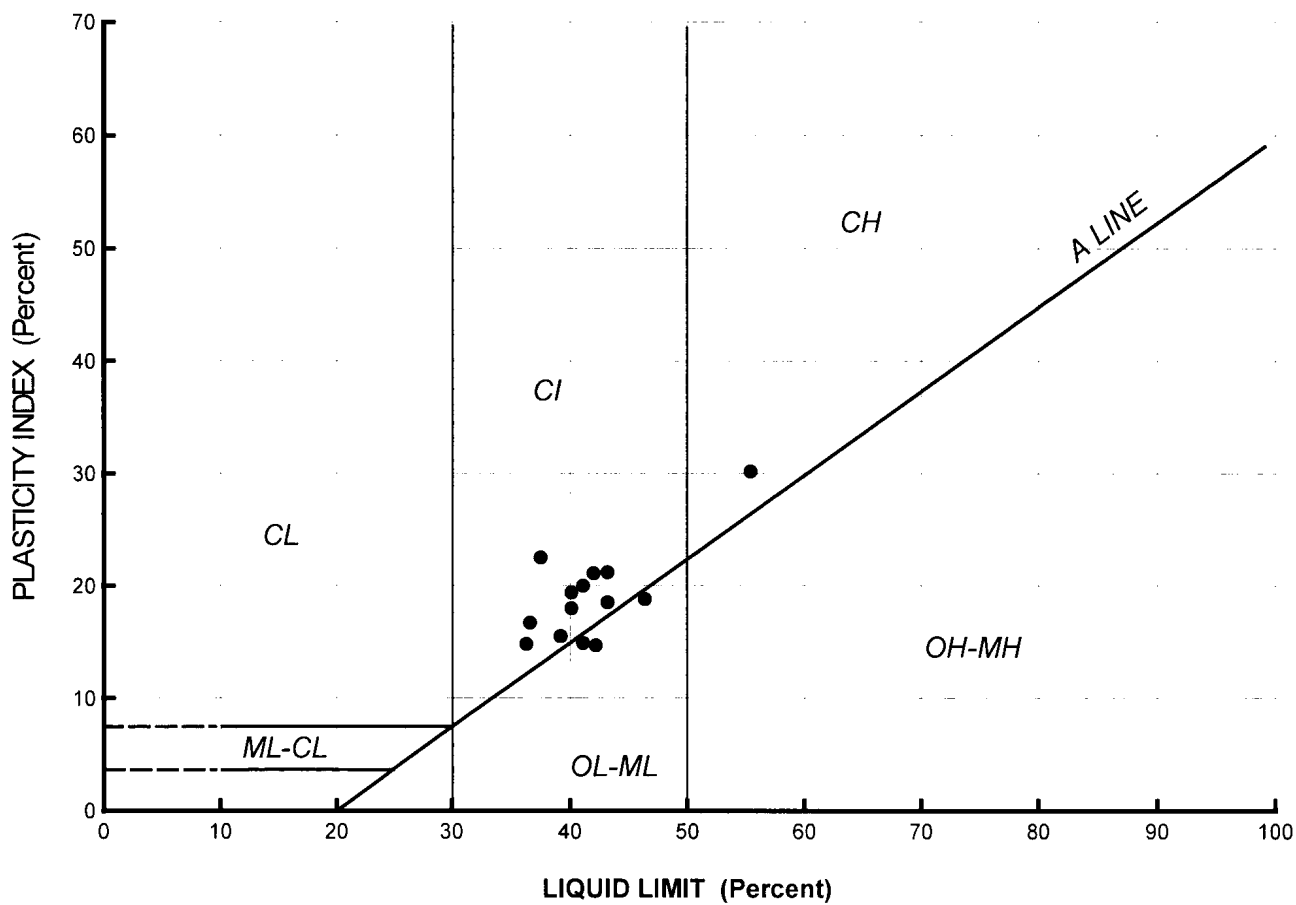
LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	1	27	144.4
⊠	2	24	142.9

LDN MTO 001-3045.GPJ GLDR LDN.GDT 17:09:00 DATA INPUT:

PLASTICITY CHART

FIGURE B-8

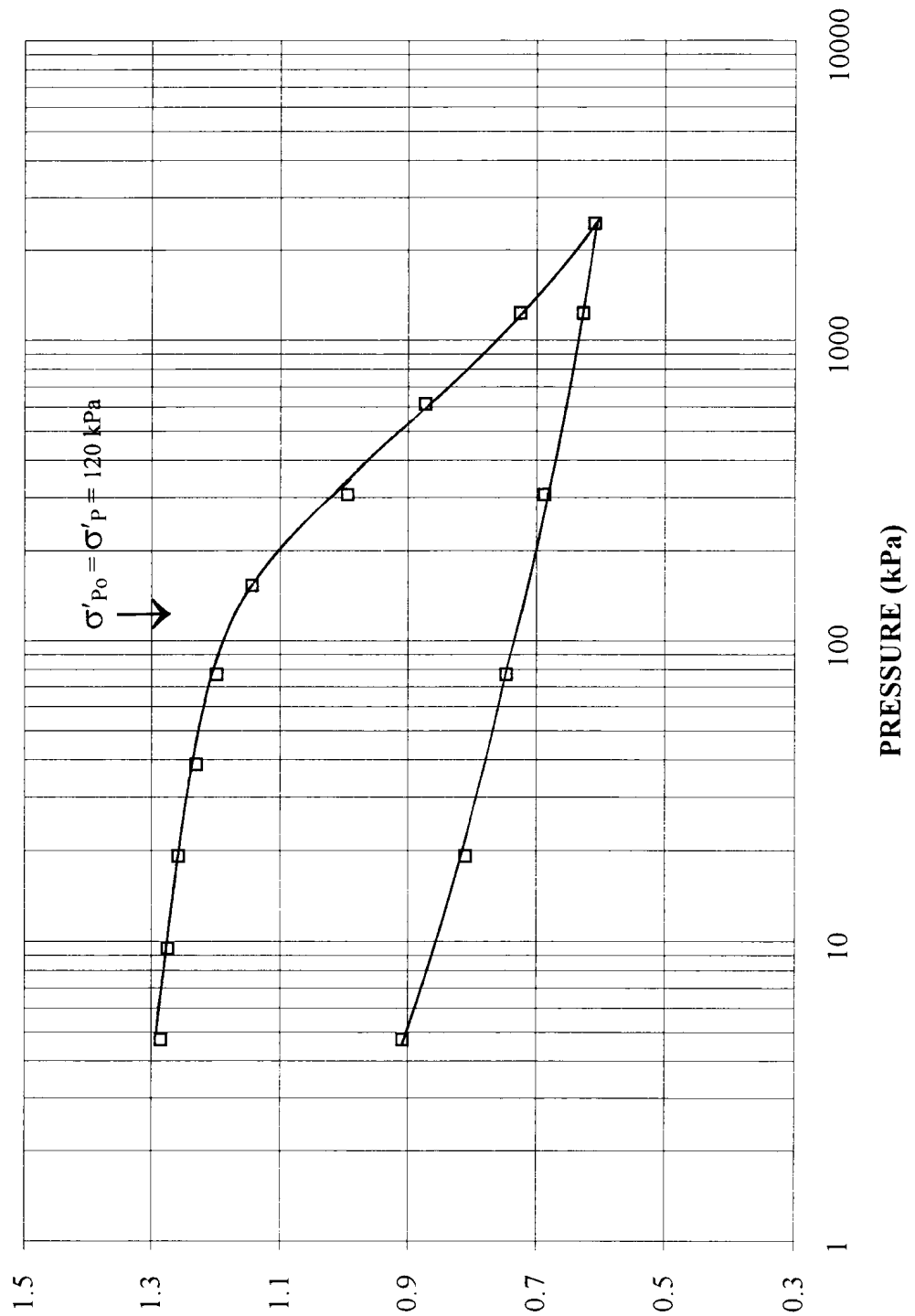


SOIL TYPE	PLASTICITY
C = Clay	L = Low
M = Silt	I = Intermediate
O = Organic	H = High

CONSOLIDATION TEST
VOID RATIO VS. LOG PRESSURE

FIGURE B-9

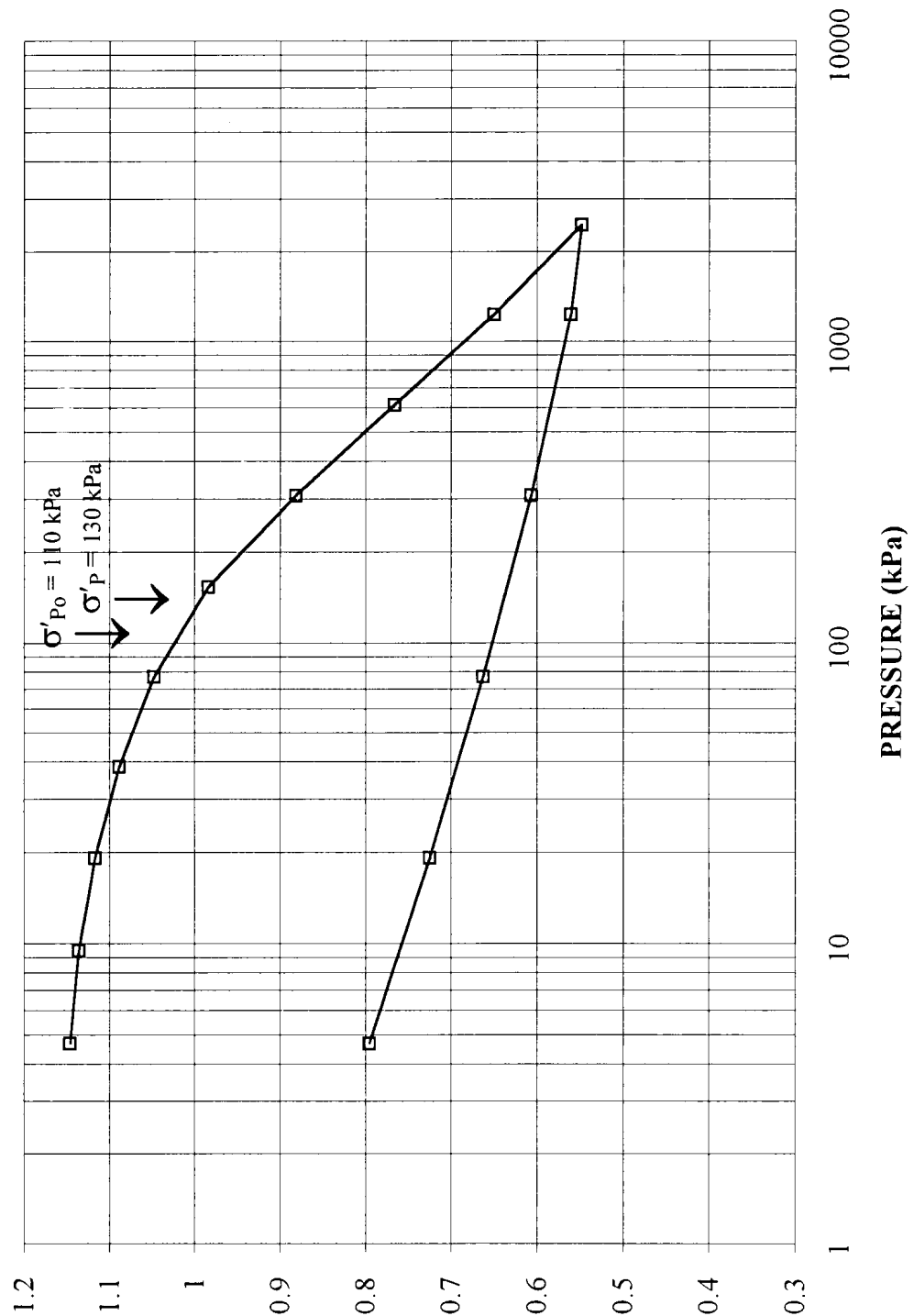
CONSOLIDATION TEST
VOID RATIO vs PRESSURE
BH 1 SA 14



CONSOLIDATION TEST
VOID RATIO VS. LOG PRESSURE

FIGURE B-10

CONSOLIDATION TEST
VOID RATIO vs PRESSURE
BH 2 SA 8



Project No. 001-3045

VOID RATIO

APPENDIX C

SITE PHOTOGRAPHS

SITE PHOTOGRAPHS



Photo 1: Drilling boreholes at northeast end of site and through existing bridge.



Photo 2: Drilling borehole 8 through existing bridge deck.