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GEOCRES No:  
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**REPORT ON**

**FOUNDATION INVESTIGATION AND DESIGN  
HIGHWAY 417 UNDERPASS BRIDGE  
AT REGION OF OTTAWA-CARLETON ROAD 22  
DISTRICT 9, WEST CARLETON  
W.P. 128-92-00 AND 451-90-06**

**Submitted to:**

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January 2001

001-2026 (Task 5002)

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**PART A**

**FOUNDATION INVESTIGATION REPORT  
FOUNDATION INVESTIGATION AND DESIGN  
HIGHWAY 417 UNDERPASS BRIDGE  
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## 1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by McCormick Rankin Corporation (McCormick Rankin) to carry out a foundation investigation for the proposed underpass bridge at the proposed crossing of Region of Ottawa-Carleton Road 22 (ROC Road 22) and the proposed Highway 417, located south east of Arnprior, Ontario. The foundation investigation was carried out for a two-span bridge with associated approach embankment. It is understood that this project involves the construction of a two-span bridge with associated approach embankments to carry the proposed realigned ROC Road 22 over the proposed Highway 417. The bridge underpass is being constructed as part of the Highway 417 extension project.

The purpose of the investigation is to determine the general subsurface and groundwater conditions at the bridge and approach embankment locations by means of drilling 9 boreholes and pushing 5 piezocones, completing in-situ testing and performing laboratory testing on selected soil samples and, based on our interpretation of the factual information obtained along with existing subsurface information available for the site, to provide engineering recommendations on the foundation design aspects of the project, including construction considerations which could influence design decisions.

The following sources which contain existing subsurface information obtained from investigations carried out in the general vicinity of the site were reviewed during the course of this investigation:

- Preliminary Foundation Investigation Report for Proposed Highway 17 Realignment and Regional Road 22, (W.P. 451-90-06, May 1992) completed by the Ministry of Transportation, Ontario (MTO).
- Pavement Design Report, Highway 417 (New Facility), from Regional Road 20 Westerly to Regional Road 22, Township of West Carleton, Regional Municipality of Ottawa-Carleton (W.P. 127-92-00) completed by Golder, Project 931-8028.
- Pavement Design Report, Highway 417 (New Facility), from Highway 15 (Realigned Easterly to Regional Road 22, Township of West Carleton, Regional Municipality of Ottawa-Carleton (W.P. 128-92-00) completed by Golder, Project No. 941-8035.
- Highway 417 Preliminary Design Study, Grainger Park Road Northerly to Highway 15 (W.P. 170-88-00) completed by Totten Sims Hubicki Associates.

The plan and profile of the proposed Highway 417 underpass bridge alignment at ROC Road 22 were provided to Golder by McCormick Rankin. The centreline at the piers and abutments of the proposed bridge were surveyed by McCormick Rankin prior to commencing the foundation investigation program.

The Terms of Reference for the scope of work are outlined in Golder's proposal P01-2035, dated February 2000 which forms part of the Consultant's Agreement (Number 4005-A-000092) for this project. The work was carried out in accordance with Golder's Quality Control Plan for this project.

## **2.0 SITE DESCRIPTION**

The project area covered by this report extends along the proposed ROC Road 22, from approximately Station 9+800 to Station 10+300 in the Township of West Carleton. The site is situated south east of Arnprior, Ontario between Highway 17 and Upper Dwyer Hill Road (Refer to Drawing A).

Regional Road 22 is an existing two lane roadway with a rural cross-section and gravel shoulders. Ditches are present on both sides of the roadway. The area is relatively flat and consists of farm land.

The site is located within the geographical region known as the Ottawa Valley clay plains. Based on previous investigations and experience, the subsoil conditions are expected to consists of sensitive silty clay overlying thin basal till. Bedrock is expected at depths of about 45 m to 50 m.

Based on available geological mapping (Geological Survey of Canada, Map 1363A) the bedrock at this site consists of dolomite or dolomitic limestone of the March and Oxford formations. Limestone of the Ottawa formation exists about 1.0 km south of the site.



### 3.0 INVESTIGATION PROCEDURES

The field work for this investigation was carried out from March 16, 2000 to May 28, 2000. During this period boreholes were laid out in the field, existing buried services were cleared. Two boreholes (Boreholes 22-1 and 22-5) were drilled at locations close to the originally proposed north and south abutments followed by three boreholes (Boreholes 22-2 to 22-4) for the originally proposed piers and four boreholes (Boreholes 22-6, 22-8, 22-10 and 22-12) and five piezocone penetration holes (Test holes 22-5A, 22-7, 22-9, 22-11 and 22-13) for the north and south approach embankments. The approximate borehole locations are shown in Drawing A. Since the boring program was carried out it was decided to employ a two-span bridge rather than the four-span bridge. Boreholes 22-2 and 22-4 are at the location of the abutments and Borehole 22-3 is at the center pier. Boreholes 22-1 and 22-5 are in the embankment area.

The investigation was carried out using two track-mounted bombardier mounted CME 55 drill rigs supplied and operated by Marathon Drilling Co. Ltd. of Gloucester, Ontario. The boreholes were advanced using 208 mm outside diameter (O.D.) continuous flight hollow stem augers. Soil samples were obtained at regular intervals of depth using a 50 mm O.D. split-spoon sampler in accordance with Standard Penetration Test (SPT) procedures. Also, selected soil samples were recovered using a 75 mm O.D. thin walled stationary piston sampler. Between samples, where possible, in-situ vane shear strength tests were carried out. An MTO 'N' size vane was used with the reaction provided by Hanson Scales purchased for this project. The scales were calibrated weekly throughout the field work using known weights of 10, 20 and 30 pounds. After determining the undrained shear strength the vane was rotated 10 to 12 times and after a pause of 60 seconds the remoulded strength was measured. Boreholes 22-1 to 22-5, 22-8 and 22-12 were advanced to practical refusal to augering at depths varying between 44.7 m and 54.3 m. Borehole 22-10 was advanced to a depth of 31.7 m. Piezocone penetration holes 22-5A, 22-7, 22-9, 22-11 and 22-13 were advanced to a depth of 44.7 m. Bedrock cores were obtained in NQ size from Boreholes 22-1 to 22-5. The groundwater conditions in the open boreholes were observed during drilling operations and piezometers were installed and sealed in Boreholes 22-1, 22-5, 22-6, 22-8, 22-10 and 22-12 to permit monitoring of the groundwater levels. Final groundwater levels were measured on May 28, 2000.

The field work was supervised throughout by members of our engineering staff, who located the boreholes, supervised the drilling, sampling and in-situ testing operations, logged the boreholes, and

examined and cared for the soil and rock samples. The soil and rock samples were identified in the field, placed in containers, labelled and transported to our Ottawa laboratory for further examination and laboratory testing. Laboratory testing on selected samples included natural water content determinations, grain size analysis and Atterberg limits. In addition four selected samples were submitted for consolidation testing; another sample was submitted to a consolidated undrained triaxial test with pore water pressure measurements. The results of laboratory testing are given on the Record of Borehole sheets and on Figures 1 to 9.

Northings, eastings and ground surface elevations at the as-drilled borehole locations were provided by McCormick Rankin. It is understood that the elevations are referred to Geodetic datum. The northings, eastings and ground elevations are shown on the Record of Borehole sheets as well as on Drawing A. The test holes locations are also shown on Drawing A.

The project was carried out in accordance with the Quality Assurance program prepared for the project.

#### **4.0 SUBSURFACE CONDITIONS**

The detailed subsurface conditions encountered in the boreholes advanced during this investigation, together with the results of the laboratory tests carried out on selected soil samples are presented on the Record of Borehole sheets and on Figures 1 to 9. The Record of Borehole sheets from a previous MTO investigation are shown in Appendix A. 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 (SPTs) and field and laboratory testing results. 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.

In summary, the subsurface conditions consist of thin surficial fill and / or topsoil overlying the main stratum of sensitive silty clay with sand seams which ranged in thickness from 44 m to 52 m across the site. A discontinuous silty sand layer with gravel, cobbles and boulders (Glacial Till) is present above the bedrock. Bedrock consisting of dolomitic limestone is present below the silty clay / silty sand with gravel, cobbles and boulders at depths interpreted to vary between 44.9 m and 54.3 m.

##### **4.1 Fill**

Fill consisting of silty sand to sand and gravel is present at ground surface at Boreholes 22-2, 22-4, 22-5, 22-6, 22-8, 22-10 and 22-12. Where present, the thickness of the fill varies between 430 mm and 580 mm.

##### **4.2 Topsoil**

Topsoil is present at ground surface or below the fill at all borehole locations except Boreholes 22-2, 22-5 and 22-10; topsoil is not reported at MTO Boreholes 1 and 2. Where present, the thickness of the topsoil varies between 150 mm and 640 mm.

#### **4.3 Silty Sand**

Fine silty sand was encountered at Borehole 22-10 below the gravel and sand fill. The thickness of the silty sand is 230 mm.

#### **4.4 Silty Clay**

Silty clay was encountered at ground surface (MTO boreholes), below the topsoil, the fill and / or below the surficial silty sand at all borehole locations. Where fully penetrated by the boreholes, the silty clay was found to have a thickness varying between 44.0 m at Borehole 22-1 and 52.0 m at Borehole 22-5.

The upper portion of the deposit consists of brown and weathered silty clay crust with a trace of fine silty sand and / or occasional sand seams. The thickness of the silty clay crust was found to vary from 3.3 m at MTO Borehole 2 to 5.6 m at Borehole 22-2. Based on the results of the in-situ vane tests the undrained shear strength values generally decrease with depth from 95 kPa to about 32 kPa; these values indicate that the consistency of the weathered silty clay is stiff to firm. Standard penetration tests carried out within the weathered crust gave 'N' values ranging from 1 blow to 15 blows per 0.3 m with the lower values closest to the base of the crust. A summary plot of engineering properties for the clay is given on Figure 1.

Natural water content tests carried out on seven samples from the brown clay yielded results ranging from about 38 percent to 71 percent with the values generally increasing with depth. The Atterberg limits were determined for one sample of the weathered crust. The result gave a liquid limit of 84 and a plasticity index of 56 percent. The result is plotted on Figure 2 and indicated a clay of high plasticity.

Grey silty clay with occasional sand seams exists below the weathered crust. Also observed was black streaking in the clay which is common to Champlain sea clays. Where fully penetrated the thickness of the grey silty clay varies between 40 m at Borehole 22-1 and 48 m at Borehole 22-5. Undrained shear strength values based on in-situ vane tests were found to vary between 18 kPa and 30 kPa for the first 5 m below the crust which is indicative of a soft to firm consistency. In-situ testing using the piezocone test equipment at five locations across the site confirmed this shear strength profile. Below 10 m depth, the clay strength increases with depth from about

20 kPa to about 90 kPa. These results are indicative of a soft to stiff consistency. Remoulded vane shear strength values indicate sensitivities up to 30 and generally about 10. These results are indicative of extra sensitive to quick clay.

Natural water content tests carried out on 51 samples from this deposit yielded values ranging from about 90 percent near the surface of the stratum to as low as 20 percent at depth. Atterberg limit were determined for seven samples. The plasticity of the clay decreases markedly with depth from 57 percent near the surface to 7 percent at 50 m depth (see Figures 1 and 2).

Consolidated undrained triaxial tests with pore pressure measurements were carried out on a sample from Borehole 22-1 from 8 m depth and the results are shown on Figures 3 to 5.

Consolidation tests were carried out on four samples of the sensitive silty clay and the results are shown on Figures 6 to 9. The four consolidation tests yielded the following results.

<i>Borehole No.</i>	<i>Depth (m)</i>	<i>Elevation (m)</i>	<i>P'<sub>o</sub> kPa</i>	<i>p'<sub>c</sub> kPa</i>	<i>P'<sub>c</sub> - P'<sub>o</sub></i>	<i>e<sub>o</sub></i>
22-5	6.1 - 6.57	94	40	95	55	1.35
22-5	9.1 - 9.65	91	60	110	50	1.45
22-10	15.2 - 15.5	84	95	115	20	1.20
22-6	30.5 - 31.0	70	210	270	60	0.94

#### 4.5 Silty Sand with Gravel, Cobbles and Boulders (Glacial Till)

A basal granular layer consisting of silty sand with gravel, cobbles and boulders was encountered above the bedrock in the boreholes at the bridge structure (Boreholes 22-1 to 22-5). The thickness of this layer varies from about 0.2 m in Borehole 22-1 to 3.2 m Borehole 22-2. One standard penetration test carried out in the deposit gave an 'N' value of 30 blows per 0.3 m of penetration which is indicative of a dense of packing. While the sample obtained was mainly silty sand with gravel, rock coring of the deposit provided evidence of cobbles and boulders. The deposit is considered to be a glacial till.

#### 4.6 Bedrock / Refusal

Bedrock or practical refusal to augering was encountered at depths ranging from 44.7 m at Borehole 22-1 to 54.3 m at Borehole 22-5. The elevation of the bedrock decreases from north to south across the bridge site from Elevation 55.8 m to 45.6 m. The bedrock consists of faintly weathered, thick bedded mainly light grey to dark grey fine grained dolomitic limestone. The bedrock is generally of fair to good quality with RQD values ranging from 46 percent to 100 percent.

#### 4.7 Groundwater Conditions

Piezometers were sealed in selected boreholes to permit groundwater level monitoring. Groundwater level readings were taken at several occasions up to 2 months after drilling (May 28, 2000); the results are presented in the following table.

Borehole No.	Ground Surface El. (m)	Piezometer Tip	Water Level Readings						
		El. (m)	Depth (m)	Upon Borehole Completion		April 25		May 28	
				El. (m)	Depth (m)	El. (m)	Depth (m)	El. (m)	Depth (m)
22-1	100.46	92.86	7.6	98.33	2.13	99.89	0.57	99.87	0.59
		70.06	30.4			95.12	5.34	95.39	5.07
22-5	99.99	91.99	8.0	97.02	3.05	98.08	1.91	98.45	1.54
		68.89	31.1			96.09	3.90	96.76	3.23
22-6	100.07	96.47	3.6			99.05	1.02	99.0	1.07
		81.17	18.9			98.45	1.62	97.96	2.11
22-8	99.94	93.34	6.6			96.47	3.47	96.91	3.03
22-10	100.56	96.91	3.65			99.73	0.83	99.57	0.99
		81.66	18.9			98.77	1.79	98.76	1.80
22-12	101.12	94.42	6.7			99.27	1.85	99.48	1.64

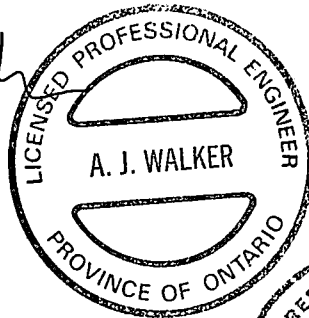
Readings taken by Golder Associates Ltd. on May 28, 2000 gave groundwater levels in the shallow piezometers from 0.6 m to 3.0 m depth. Readings taken in the deep piezometers gave groundwater levels of 1.8 m to 5 m depth suggesting there is some slight downward drainage to the bedrock.

It should be noted that the groundwater levels are subject to seasonal fluctuations.

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January 2001

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**PART B**

**FOUNDATION DESIGN REPORT  
FOUNDATION INVESTIGATION AND DESIGN  
HIGHWAY 417 UNDERPASS BRIDGE  
AT REGION OF OTTAWA-CARLETON ROAD 22  
DISTRICT 9, WEST CARLETON  
W.P. 128-92-00 AND 451-90-06**



## 5.0 DISCUSSIONS AND RECOMMENDATIONS

This section of the report provides our interpretation of the factual geotechnical data obtained during the present and previous investigations. The recommendations provided are intended for the guidance of the design engineers and are intended for this project only. The data may not be sufficient for construction and where comments are made on construction, they are provided only in order to highlight aspects of construction which 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, safety and the like.

This report addresses aspects of the foundation design which include a two-span bridge and the associated approach embankments. The locations of the two abutments and the pier are shown on Drawing A. All project information utilized for the preparation of this report, including the digital base plan, was provided by McCormick Rankin.

Based on the information provided by McCormick Rankin the proposed deck of the two-span bridge will be about 73 m long and 11 m wide. The north approach embankment is to reach a maximum elevation of 107.80 m along its centreline at about Station 9+982.300. The existing ground elevation at this location is about 100.7 m. The south approach embankment is to reach a maximum elevation of 107.77 m along its centreline at about Station 10+057.700 where the original ground is approximately at about Elevation 100.45 m. The maximum height of the north and south approach embankments is 7.1 m and 7.3 m, respectively. The approach embankments are to be 12 m wide at the top.

The subsurface conditions encountered in the boreholes completed during this investigation typically consist of topsoil and / or fill overlying sensitive silty clay. A granular basal layer consisting of silty sand with gravel, cobbles and boulders is present above the bedrock. Bedrock consisting of dolomitic limestone is present below the granular basal layer at depths interpreted to vary between 44.7 m and 54.3 m. The groundwater levels measured in the piezometers, at the time of the investigation, were between depths of 0.6 m to 3.5 m.

## **5.1 Approach Embankments**

### **5.1.1 Embankment Options**

Based on the information provided by McCormick Rankin, the proposed heights for the embankments at ROC Road 22 are 7.3 m and 7.1 m for the south and north approach embankments, respectively. A number of embankment options were considered, including construction with:

- Earthfill;
- Rockfill;
- Light-weight fill (blast furnace slag); and
- Polystyrene blocks.

These fill options were previously discussed in our facsimile transmission dated May 19, 2000, and are considered not to be feasible for the abutment areas. For example; with an earthfill embankment, long berms (>60 m) would be required and settlements of up to 2,000 mm were calculated for the abutment area and with light-weight fill, 20 m wide berms were required and settlements of up to 1,000 mm were calculated for the abutment area.

An iterative process was carried out between McCormick Rankin and Golder Associates to determine an acceptable embankment fill configuration. This process consisted of altering the abutment heights, lengthening the bridge spans, changing bridge deck types and combining various light-weight fills in order to develop a cost-effective embankment / bridge system that met settlement and stability performance criteria, as well as minimizing downdrag loads. The performance criteria could only be met if a polystyrene block type fill in combination with wick drains was used. The final configuration selected is shown on Figure 10. This configuration combines polystyrene, ultra light-weight fill, light-weight fill, rockfill and earthfill into a cost / performance optimized configuration.

The following sections discuss the performance of the optimized configuration in terms of stability and settlement. The use of surcharging, wick drains, temporary works and staging is also discussed.

### 5.1.2 Stability Analysis

A stability analysis was carried out for the proposed south embankment. The undrained shear strength of the silty clay under the north embankment is in general similar to that of the south embankment. The north embankment is slightly lower than the south and therefore, the following stability results can also be conservatively assumed to apply to both embankments.

The analysis was carried out using the commercially available program SLOPE/W using the Morgenstern and Price limit equilibrium method of analysis.

The undrained stability of the south abutment was assessed using the configuration shown on Figure 10. The following undrained shear strength ( $S_u$ ) model was used.

<i>ELEVATION (m)</i>	<i><math>S_u</math> (kPa)</i>
100	50
97	50
95	20
90	20
50	100

The model was based on the in-situ vane shear and piezo-cone penetration test (CPT) data and is also shown on Figure 1. The following table summarizes the fill properties used in the analysis.

<i>Fill Type</i>	<i>Fill Properties</i>	
	<i>Unit Weight (kN/m<sup>3</sup>)</i>	<i>Effective Strength Friction Angle (degrees)</i>
Earthfill	22	30
Rockfill	18	40
Light-weight (i)	14	35
Ultra light-weight (ii)	11.5	35
Polystyrene (iii)	1	$S_u = 27.5 \text{ kPa}$ (iv)

- (i) Pelatized water cooled blast furnace slag by Lafarge Canada (Litex 149) or equivalent.
- (ii) Pelatized water cooled blast furnace slag by Lafarge Canada (Litex 143) or equivalent.
- (iii) Styrofoam HI 40 produced by DOW or equivalent with a shear strength = 275 kPa.
- (iv) Undrained shear strength provided which includes a factor of safety, 10 to account for long-term creep and fatigue effects.

A factor of safety equal or greater than 1.3 is typically considered acceptable.

The analyses were carried out for a number of sections along the embankment for the final and surcharge conditions. The surcharge levels were determined based on the settlement assessment described below and the results of the stability analyses. Note that the embankment surcharge is 1.5 m of earthfill at the abutment reducing to 0.75 m of earthfill when the polystyrene zone ends, or about 50 m from the abutment. If desired the polystyrene / ultra light-weight / surcharge section of the embankment may be replaced with 3.5 m of earthfill with 2H:1V side slopes during the preloading period. The preload configuration with 3.5 m of earthfill should extend to the full width of the proposed embankment in its final configuration. The results of the stability analyses are summarized follows:

Section Location	Embankment Height (m)	Factor of Safety	
		Final Configuration	Surcharge Configuration
Forward Slope at Abutment	7.3	2.3	2.0
Station 10+082.7 – section with 3.4 m of polystyrene	7.2	2.45	1.8
Station 10+107.7 – ultra light-weight fill	7.0	1.7	1.5
Station 10+151.86 – light-weight fill	6.4	1.65	1.45
Station 10+249.36 – rockfill	4	2.4	2.0

The above results confirm that the design configuration has acceptable factors of safety with 2H:1V side slopes for both design and surcharge configurations.

### 5.1.3 Settlement Analysis

A deposit of silty clay, about 44 m to 54 m in thickness, exists below the proposed embankments at this site. Settlement analyses were carried out for the south approach embankment, using the embankment configuration given on Figure 10. The analyses were based on the available borehole, CPT, oedometer and in-situ vane shear strength data. The pre-consolidation stress ( $\sigma'_p$ ) profile in the silty clay was established using the oedometer, CPT and in-situ vane shear strength and the following relationship:

$$\begin{aligned} S_u &= 0.22 \sigma'_p \\ \text{Where } \sigma'_p &= \text{pre-consolidation pressure (kPa)} \\ S_u &= \text{in-situ shear strength (kPa)} \end{aligned}$$

The following pre-consolidation stress profile and consolidation parameters were used.

<i>Elevation (m)</i>	<i>Initial Void Ratio (Co)</i>	<i>Compression Index (Cc)</i>	<i>Recompression Index (Cr)</i>	<i>Pre-consolidation Stress (kPa)</i>	<i>Unit Weight (kN/m<sup>3</sup>)</i>
100 - 97	1.2	0.6	0.04	350	17.5
97 - 95	2.0	1.2	0.09	350 - 100	15.5
95 - 88	1.4	1.4	0.06	100	16
88 - 75	1.3	1.1	0.05	100 - 240	17
75 - 50	0.9	0.5	0.03	240 - 500	18

Settlement analyses were carried out using the program UNISETTLE.

The maximum settlement at the abutment on the centerline, in its final configuration, was calculated to be about 150 mm to 200 mm. This level of settlement is unacceptable from an on-going maintenance perspective and the dragload that would be induced on the piles. The time to complete 90% of the consolidation settlement (using an average  $c_v = 10^{-3} \text{ cm}^2/\text{s}$ ) is about 90 years. The time to complete 50% of the settlement is about 20 years.

The addition of a surcharge for a period of one year helps to compensate for a portion of this settlement. A surcharge of 1.5 m of earthfill would induce about 70 mm of settlement in one year. Therefore, 80 mm to 120 mm of settlement would still occur after removal of the preload. Based on an iterative process involving stability and settlement analyses, as well as the requirement to minimize dragload, it was concluded that wick drains in combination with a preload are required for the bridge abutment approach embankment areas.

The preload configuration is described above in Section 5.1.2. A detailed wick drain design is required for this site. However, preliminary calculations indicate that about 300 mm of settlement at the abutment areas can be induced in one year with the 1.5 m earthfill surcharge and wick drains. The wick drains would have to be installed to a depth of at least 25 m below ground surface in a 3 m grid pattern. The induced settlement would compensate for the anticipated primary consolidation at the abutment and therefore only a minimal amount of settlement would occur after removal of the surcharge, say less than 25 mm in 20 years. The areal extent over which the drains should be installed has not been determined but for preliminary purposes a zone equal to the width of the road plus two times the embankment height by 200 m in length may be assumed at each approach. At the wick drain design stage there will be an opportunity to incrementally increase the drain spacing and reduce the installation depth away from the abutment and therefore increase coverage under the embankment and smooth the settlement

transition between the approach embankment area without wick drains and the abutment area with wick drains.

The areas of the embankment that are outside of the influence of the wick drains will still experience time-dependent primary consolidation on the order of 100 mm in 20 years. Some maintenance of the approach embankments will be required.

It should be noted that settlement predictions are difficult to make for the site given the low pre-consolidation stresses and high compressibilities. A minor change in the pre-consolidation stress design level can produce large changes in estimated settlements. However, with the wick drain and surcharge combination the risk of settlements at the bridge abutments exceeding 25 mm following removal of the preload is considered to be low.

Settlement monitoring of the embankment should be carried out during the preload period.

#### **5.1.4 Falsework**

We understand that a post tensioned concrete deck has been selected for this bridge and that this type of deck can only tolerate about 12 mm of settlement during the time when the concrete is setting up and curing. This time period extends from about 8 hours to 21 days after the concrete pour. The deck is considered self-supporting after post-tensioning which would take place at about 21 days. We also understand that the overall load imposed by the formwork supports is equivalent to about a pressure load of 30 kPa.

A number of falsework support options were considered including, short friction piles, piles to bedrock and a preloaded granular pad. Based on an assessment by McCormick Rankin, MTO and ourselves the primary option that we have considered is a preloaded granular pad. An instrumented test section of this pad would be used to determine if the 21 day settlement performance of this pad was acceptable.

Assuming that the temporary works support system spreads the load relatively evenly, the general load increase would be about 30 kPa. A 0.5 m thick granular pad would be used to found the falsework supports on. The preload configuration may be either 2 m of earthfill (which is then removed and replaced with a 0.5 m thick granular pad) or 0.5 m of granular and 1.5 m of earthfill.

Note that the preload would be maintained for about a year. An analysis was carried out using this area load over a 12 m by 30 m area. Unisettle was used to calculate the stress change within the 50 m thick clay deposit.

The range in elastic settlement under the loading is about 25 mm to 50 mm (using an undrained elastic modulus  $E_u = 300 \times S_u$  to  $500 \times S_u$ ). The calculated total primary consolidation settlement is about 90 mm. The calculated consolidation that would occur in one year is about 50 mm ( $c_v = 0.005 \text{ cm}^2/\text{s}$ ).

After one year the preload would be removed and the falsework support system constructed and then the deck poured. The consolidation that was induced during the preload period would compensate for any primary consolidation settlement that would occur during the 21 day loading period. However, some level of elastic settlement would still occur as part of the unloading-reloading process.

The time in which the elastic settlement would occur is not known. It is generally described as occurring within the construction period, which would generally be assumed to be days to weeks. It is unclear whether all the elastic settlement would occur within 8 hours of loading. Given the settlement tolerance of 12 mm from 8 hours to 21 days we recommend that a section of the falsework preload be instrumented and that a trial unloading, reloading cycle is carried out at about 10 months after initial placement. An assessment of the effectiveness of the preload would then be carried out.

Instrumentation would consist mainly of settlement plates placed on the original ground surface and then monitored during fill placement and removal operations.

If this option was determined to be unacceptable then short friction piles could be utilized. The following may be used for preliminary assessments. In order to evaluate the theoretical capacity of the timber friction pile the analyses were carried from both total stress and effective stress approaches known as  $\alpha$  and  $\beta$  methods. The axial capacity based on these methods gave an ultimate average shaft resistance ranging from 190 kN to 265 kN for a 13 m long tapered (Size #32 untreated butt 300 mm, tip 200 mm) untreated timber pile. The axial capacity incorporating the resistance factors as per OHBDC, 1992 are as follows:

Factored axial capacity at ULS	90 kN
Geotechnical resistance at SLS (limiting deformations to 10 mm or less)	70 kN

However the actual resistance depends significantly on the following:

- driving method
- dissipation of pore-water pressure
- time effect
- unusual weak layers in the clay deposit

If static load(s) are carried out prior to construction, higher resistance factors can be applied to the results. Therefore, we recommend the axial capacity for the untreated timber piles should be obtained by in-situ static load tests at the time of construction. If load tests are not carried out the above capacities may be used for design purposes.

For long end bearing piles to bedrock, the elastic compression of the pile under load would be much greater than the allowable settlement for the falsework. As it is not known how much of this settlement would take place in the period of 8 hours to 21 days, this option has been discarded.

### 5.1.5 Staging

The following is our recommended construction sequence.

- 1) Clear site – install wick drains and place 300 mm thick granular pad in approach embankment areas.
- 2) Construct approach embankments with preload, preload in falsework area and install instrumentation. Note that the embankment preload is 1.5 m of earthfill at the abutment reducing to 0.75 m of earthfill where the polystyrene zone ends, or about 50 m from the abutment. If desired the polystyrene / ultra light-weight / preload section of the embankment may be replaced with 3.5 m of earthfill with 2H:1V side slopes in front and / or behind the abutment. The preload configuration with 3.5 m of earthfill should extend to the full width of the proposed embankment in its final configuration. The preload in the falsework area is equivalent to 2 m of earthfill to about Elevation 102.75 m. The granular pad required for the falsework of about 0.5 m may be incorporated into the total of 2 m, i.e. a 0.5 m granular pad with 1.5 m earthfill placed on top. It is not essential that the granular pad be placed at this stage.



- 3) 10 months after fill placement carry out test of falsework preload section, by removing and replacing 1.5 m of earthfill. Assess results and determine falsework support option.
- 4) One year after embankment construction, remove embankment preload. At abutments remove forward slope while maintaining 2H:1V slopes and excavate area at pier.
- 5) Drive piles and construct pile caps.
- 6) Remove falsework preload 1.5 m of earthfill; if 2 m of earthfill – remove and construct 0.5 m granular pad.
- 7) Assemble formwork and construct post tensioned concrete deck.
- 8) Complete forward slope with recommended polystyrene / rockfill / ultra light-weight fill arrangement.

Note: the key restriction is that the height of earthfill placed anywhere on the site should not exceed 3.5 m.

#### **5.1.6 Instrumentation**

The performance of the embankments should be monitored to confirm that performance is within anticipated ranges. We recommend that the primary method of monitoring pore pressure dissipation in the clay deposit should be settlement plates. Settlement plates are relatively inexpensive to install and monitor, they can be placed to provide good areal coverage, are easily replaced if damaged and the overall behavior of the clay deposit can be inferred from the time settlement data. The majority of plates should be placed under the preload on the embankments. However, plates at each abutment should be placed on original ground prior to embankment construction. As a secondary method of assessment piezometers (vibrating wire or pneumatic) should be installed in the abutment areas. Consideration should also be given to installing settlement profiler tubes and vibrating wire settlement gauges.

The performance of the falsework preload area is critical to ensuring that the post-tensioned deck does not suffer excessive cracking during curing. Therefore, a number of settlement plates should be installed prior to the placement of the preload and monitored with time. After ten months a section of the preload should be removed and the unload response measured. The preload should then be replaced and the reloading response measured over the next 21 days. Note that the replacement of the preload would have to be completed in a timely manner in order to simulate the rate of falsework loading. This data would be used to confirm the ground response to the

surcharge, with the surcharge modeling the falsework loading. If this response was unacceptable then the friction pile support option would have to be adopted.

A more detailed monitoring plan showing instrumentation locations as well as a reading schedule and preload assessment scheme will be required prior to construction.

## **5.2 Structure Foundations**

Due to the thickness of the relatively weak and compressible sensitive grey silty clay deposit underlying the weathered silty clay crust and considering the proposed embankment heights, shallow foundation support for the proposed structure is not recommended.

### **5.2.1 End Bearing Piles**

It is recommended that the structure foundation loads be transferred through the sensitive silty clay to more competent bearing at depth. This can be achieved with piles driven to end bearing on bedrock. A suitable pile type would be steel H-piles driven to practical refusal on bedrock. H-piles are recommended because they will easily penetrate the silty clay deposit and minimize the amount of disturbance imparted to the silty clay given their shape and small cross-sectional area. Based on the available borehole information the following pile tip elevations may be assumed for design:

<i>Structure</i>	<i>Assumed Pile Tip Elevation (m)</i>
South Abutment	48.3
Central Pier	49.0
North Abutment	50.4

#### **5.2.1.1 Axial Capacity**

For HP 310 x 110 piles driven to practical refusal on the dolomitic limestone bedrock, a factored axial resistance at Ultimate Limit States (ULS) of 2,000 kN may be used for design. This value represents a structural limitation for the pile rather than a geotechnical limitation. The axial resistance at serviceability limit states (SLS) is controlled by the elastic response of the pile. The SLS load should therefore be determined by the structural designer based on the elastic response

of a chosen 50 m long pile section to vertical loading and deformation. For example, if 25 mm deformation is considered applicable then, based on the assumption that the pile is end bearing, SLS for HP 310 x 110 pile would be 1,400 kN.

Pile termination or 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.

Provision should be made to re-strike on selected piles to confirm the set after adjacent piles have been driven as per Ministry's current Special Provisions.

Boulders / cobbles were inferred from several boreholes during the investigation within the basal granular layer above the bedrock. Based on this, it is considered that boulders at the site may exist immediately above the bedrock surface. The pile tips should be reinforced to minimize damage to the pile during driving (such as welding a 12 mm thick steel plate to the bottom 300 mm of each flange as per OPSD 3301.00).

The base of pile caps should be provided with a minimum soil cover of 1.8 m for frost protection purposes.

#### **5.2.1.2 Downdrag**

The maximum heights of the south and north approach embankments are 7.1 m and 7.3 m, respectively. Consolidation settlement of the underlying sensitive silty clay deposit will take place as a result of the construction of the approach embankments. The consolidation settlement will occur during the construction period and will continue for some significant time afterward. Since the piles are end bearing the consolidation settlements will result in the development of negative skin friction (downdrag loads or dragloads) acting on the piles. Down to a certain point along the pile, called the neutral point, the settlement of the soil is larger than the downward movement of the pile. The shear stresses mobilized along the pile down to the neutral point act

downward and act as a downdrag load. Below this point the downward movement of the pile is larger than the movement of the soil settlement and the mobilized shear stresses act upwards on the pile and act as positive skin friction. The neutral point is defined as the point along the pile at which the relative pile-soil movement is zero i.e. settlement of the soil is equal to the downward movement of the pile. In this case, the pile tip resting on unyielding bedrock, the predominant downward movement of the pile will be due to pile compression.

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 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 pile as described in OHBDC.

The approach embankment design was optimized to provide acceptable long-term settlements at the abutments and to minimize dragloads, see Section 5.1. This was achieved by using a combination of light-weight material, surcharging and wick drains. The future ground settlement at the abutments was estimated at under 25 mm which is similar to the elastic compression of a HP 310 x 110 under a load of 1,400 kN. We consider that for a HP 310 x 100 pile, under the proposed embankment design the elastic compression of the pile will exceed the compression of the clay deposit and the dragload may be neglected. Note, the distribution of loading on the pile will change over time as consolidation of the clay occurs. Initially the pile will behave as a friction pile with an effective length less than 50 m, i.e. the pile will initially not transfer any load to the tip, and the pile will not compress 25 mm at 1,400 kN, as given in the above example. However, as some consolidation of the clay occurs the load along the pile will redistribute and some proportion of the load will be transferred to the tip. The pile will then deform elastically to compensate for the clay compression and the dragload may effectively be neglected.

Note the preload for the falsework compensates for the settlement induced by the minor grade change at the pier location. Therefore, the piles at the pier location may also be designed without the addition of a dragload.

### 5.2.1.3 Horizontal Resistance

The horizontal load capacity of the abutment piles is dependant on the quality and compaction of the abutment fill (for perched abutment) and the strength of the underlying soils while that of the central pier piles depends on the quality and strength of the surficial and underlying soils. For conventional piled foundations, lateral resistance may be considered in accordance with Section 6-9.8.1 of the OHBDC (1991). Battered piles are likely required to increase lateral resistance.

Where significant settlement of the embankment fill behind the abutment is expected, piles should also be battered towards the embankment to avoid rotational movement of the abutment towards this centre of settlement.

Laterally applied loads can be resisted geotechnically by the driven piles through passive pressure developed in the soil in which the pile is embedded. Based on the pile tip elevations provided above, the pile lengths will vary between 44.7 m and 54.3 m.

The design of pile 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.

The horizontal soil reaction to a vertical pile 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 general undrained shear strength profile at the proposed structures shown on Figure 1 may be used for design.

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

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

It should be pointed out that geotechnical resistance to develop lateral load capacity is very strain dependent. For the soil profile at this site, it is recommended that lateral loads be accommodated using battered piles. The above equation for horizontal reaction may also be used as an approximation for battered piles.

The above horizontal reaction may also be used for seismic assessment purposes (which is an ULS condition).

The analysis of a pile under lateral loading is a problem in soil-structure interaction. The deflection of the pile is dependent on the soil response and the soil response is a function of pile deflection. An iterative solution should be employed because the soil response is a non-linear function of pile deflection and of position along the length of the pile. A non-linear analysis typically utilizes curves relating the soil response and the pile deflection ( $p - y$  curves). The calculation process involves the geotechnical and structural engineers working interactively to solve the pile structure response. The equation for horizontal subgrade reaction, given above, is based on approximate linear theory. In particular,  $k_h$  is related to the secant modulus ( $E_{50}$ ) at half the ultimate stress in an undrained test and typically a strain level of about 1% to 2%. The approach is inherently conservative for lower levels of strain and unconservative for higher levels.

This approximate approach may be used by the structural engineer to estimate the response of the bridge to lateral loading. However, if strain levels higher than about 2% are predicated for the pile(s) then the geotechnical engineer should be informed and subgrade reaction values adjusted or a non-linear analysis methodology adopted. For general guidance refer to OHBDC 3<sup>rd</sup> edition, 6-9.8.

### 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.

- The slag material is light-weight, possess high strength (frictional) characteristics, is non-frost susceptible, is quite pervious and possess good drainage characteristics. Therefore, slag material should be used as backfill material behind the abutments and the retaining walls.
- 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 slag backfill should be placed in loose lifts of 300 mm and compacted by eight passes of a manually guided tamper such as Bomag BPR 30/38 or equivalent in accordance with OPSS 206.07.
- Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill.
- 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.
- Polystyrene light-weight fill will have a minimal effect on the lateral earth pressure and may be neglected.
- The pressures may be based on the fill configuration as shown on Figure 10 and the following parameters (unfactored) may be assumed:

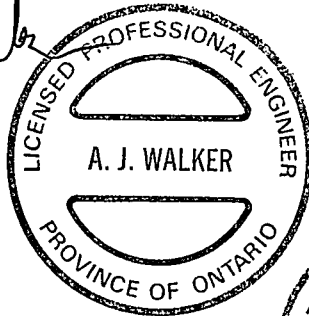
	<i>Ultra Light-Weight Fill</i>	<i>Granular A</i>	<i>Granular B Type II</i>
Soil Unit Weight	11.5 kN/m <sup>3</sup>	22 kN/m <sup>3</sup>	21 kN/m <sup>3</sup>
Coefficient of Lateral Earth Pressure			
'active'	0.27	0.27	0.31
'at rest'	0.43	0.43	0.47

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

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## LIST OF ABBREVIATIONS

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

### I SAMPLE TYPE

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

### II PENETRATION RESISTANCE

#### Standard Penetration Resistance (SPT), N:

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

#### Dynamic Penetration Resistance; $N_6$ :

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):

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

### III SOIL DESCRIPTION

#### (a) Cohesionless Soils

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

#### (b) Cohesive Soils

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

### IV. SOIL TESTS

w	water content
$w_p$	plastic limit
$w_L$	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test <sup>1</sup>
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement <sup>1</sup>
$D_R$	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 test (LV-laboratory vane test)
$\gamma$	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

$\pi$	= 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

$\gamma$	shear strain
$\Delta$	change in, e.g. in stress: $\Delta \sigma$
$\epsilon$	linear strain
$\epsilon_v$	volumetric strain
$\eta$	coefficient of viscosity
$\nu$	Poisson's ratio
$\sigma$	total stress
$\sigma'$	effective stress ( $\sigma' = \sigma - u$ )
$\sigma'_{vo}$	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stresses (major, intermediate, minor)
$\sigma_{oct}$	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
$\tau$	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
$\gamma'$	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
*	Density symbol is $\rho$ . Unit weight symbol is $\gamma$ where $\gamma = \rho g$ (i.e. mass density x acceleration due to gravity)

#### (a) Index Properties (con't.)

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)

#### (c) 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

#### (d) Consolidation (one-dimensional)

$C_c$	compression index (normally consolidated range)
$C_r$	recompression index (overconsolidated range)
$C_s$	swelling index
$C_\alpha$	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
$\sigma'_p$	pre-consolidation pressure
OCR	Overconsolidation ratio = $\sigma'_p / \sigma'_{vo}$

#### (e) Shear Strength

$\tau_p, \tau_r$	peak and residual shear strength
$\phi'$	effective angle of internal friction
$\delta$	angle of interface friction
$\mu$	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 = (Compressive strength)/2

# 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 0.6 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

<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

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

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

O:\Templates\Rock Description Terminology

## 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 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	Ca-	Calcite
FO-	Foliation/Schistosity	P-	Polished
CL -	Cleavage	S-	Slickensided
SH -	Shear Plane/Zone	SM-	Smooth
VN-	Vein	R-	Ridged/Rough
F -	Fault	ST-	Stepped
CO-	Contact	PL-	Planar
J -	Joint	FL-	Flexured
FR-	Fracture	UE-	Uneven
MF -	Mechanical	W-	Wavy
A-	Angular	C-	Curved
BP-	Bedding Plane	H-	Hackly
BL-	Blast Induced	SL-	Sludge Coated
	Parallel To	TCA-	To Core Axis
⊥	Perpendicular To	STR-	Stress Induced

PROJECT 001-2026 (5002)

# RECORD OF BOREHOLE No 22-1

1 OF 3

METRIC

W.P. 128-92-00 and 451-90-06

LOCATION N 5029116.715 E 321369.743

ORIGINATED BY M.B. / D.W.N.

DIST 9 HWY 417

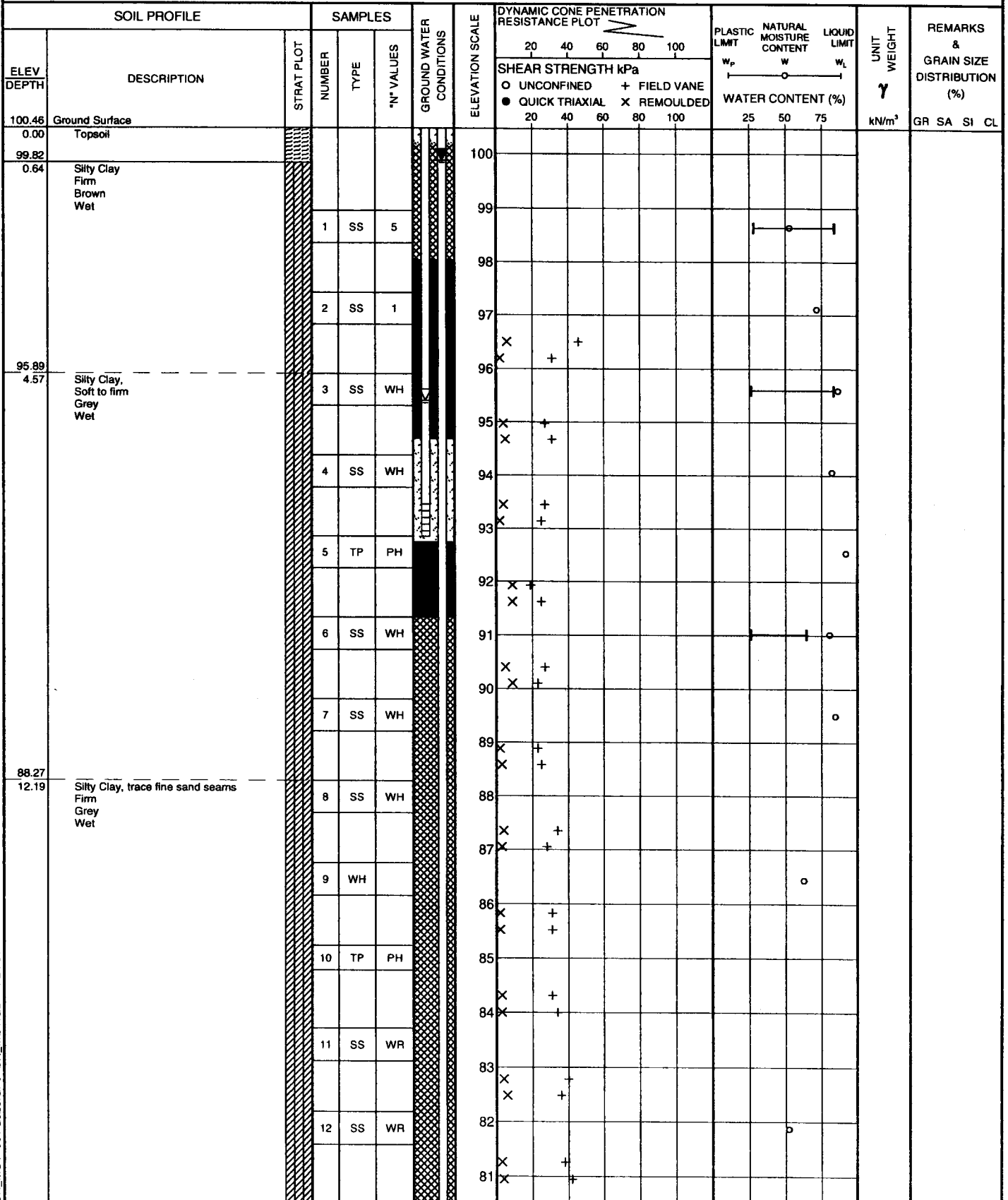
BOREHOLE TYPE CME 55 Bombardier

COMPILED BY G.C.

DATUM Geodetic

DATE Mar 16-21, 2000

CHECKED BY G.S.W.



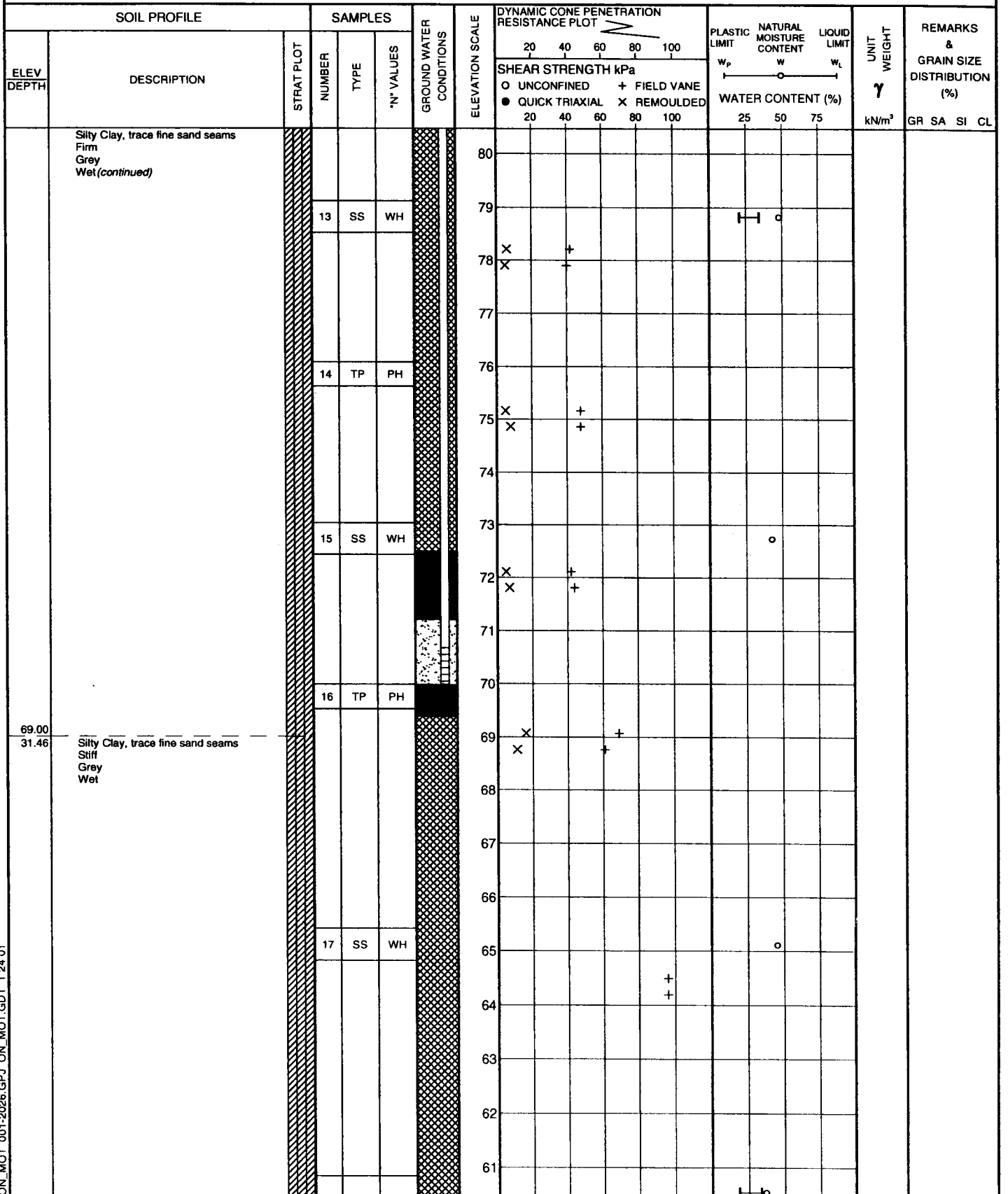
Continued Next Page

+ 3, X 3. Numbers refer to  
Sensitivity

O 3% STRAIN AT FAILURE

ON\_MOT\_001-2026.GPJ ON\_MOT\_GDT\_1 24.01

PROJECT <u>001-2026 (5002)</u>		<b>RECORD OF BOREHOLE No 22-1</b>		2 OF 3		<b>METRIC</b>	
W.P. <u>128-92-00 and 451-90-06</u>		LOCATION <u>N 5029116.715 E 321369.743</u>		ORIGINATED BY <u>M.B. / D.W.M.</u>			
DIST <u>9</u> HWY <u>417</u>		BOREHOLE TYPE <u>CME 55 Bombardier</u>		COMPILED BY <u>G.C.</u>			
DATUM <u>Geodetic</u>		DATE <u>Mar 16-21, 2000</u>		CHECKED BY <u>G.S.W.</u>			



Continued Next Page

+<sup>3</sup>, X<sup>3</sup>: Numbers refer to Sensitivity      ○ 3% STRAIN AT FAILURE

ON\_MOT\_001-2026.GPJ ON\_MOT.GDT 1 24 01

PROJECT 001-2026 (5002)			RECORD OF BOREHOLE No 22-1			3 OF 3			METRIC										
W.P. 128-92-00 and 451-90-06			LOCATION N 5029116.715 E 321369.743			ORIGINATED BY M.B. / D.W.N.													
DIST 9 HWY 417			BOREHOLE TYPE CME 55 Bombardier			COMPILED BY G.C.													
DATUM Geodetic			DATE Mar 16-21, 2000			CHECKED BY G.S.W.													
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 <sub>p</sub> w w <sub>L</sub>			γ			GR SA SI CL		
	Silty Clay, trace fine sand seams Stiff Grey Wet (continued)		18	SS	1		60												
							59												
							58												
							57												
55.81			19	SS	14		56												
55.59	Silty Sand with gravel, cobbles and boulders Compact Grey Wet (GLACIAL TILL)						55												
44.87	Faintly weathered, thickly bedded, grey to pink, fine grained, DOLOMITIC LIMESTONE						54												
							53												
52.43	Bedrock cored between 44.67m and 48.03m depth																		
48.03	For Bedrock coring details refer to Record of Drillhole 22-1 END OF BOREHOLE																		
NOTE: Water level in lower piezometer at 5.07m depth and in upper piezometer at 0.59m depth on May 28, 2000																			

ON\_MOT 001-2026.GPJ ON\_MOT.GDT 1 24 01

PROJECT: 001-2026 (5002)

## RECORD OF DRILLHOLE: 22-1

SHEET 2 OF 2

LOCATION: N 5029116.715 E 321369.743

DRILLING DATE: March 21, 2000

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 55 Bombardier

DRILLING CONTRACTOR: Marathon Drilling

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (mm/min)	FLUSH % RETURN	COLOUR % RETURN	FR-FRACTURE CL-CLEAVAGE SH-SHEAR VN-VEIN	F-FAULT J-JOINT P-POLISHED S-SLICKENSIDED	SM-SMOOTH R-ROUGH ST-STEPPED PL-PLANAR	FL-FLEXURED UE-UNEVEN W-WAVY C-CURVED	BC-BROKEN CORE MB-MECH. BREAK B-BEDDING	DISCONTINUITY DATA				HYDRAULIC CONDUCTIVITY k, 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				TYPE AND SURFACE DESCRIPTION			
																						TOTAL CORE %	SOLID CORE %	TYPE AND SURFACE DESCRIPTION
				55.59																				
				44.87	1		100								FR, R FR, R FR, R FR, R FR, R FR, R FR, R									
46	NQ Core	Faintly weathered, thickly bedded, grey to pink, fine grained, DOLOMITIC LIMESTONE			2		100								FR, R FR, R FR, R FR, R FR, R FR, R FR, R									
							3		100								FR, R FR, R FR, R FR, R FR, R FR, R FR, R							
48							52.43										FR, R FR, R FR, R FR, R FR, R FR, R FR, R							
		END OF BOREHOLE		48.03																				
50																								
52																								
54																								
56																								
58																								
60																								
62																								
64																								

ROCKINT.GPJ GLDR\_CAN.GDT 11/26/00

DRILLHOLE ROCKINT.GPJ GLDR\_CAN.GDT 11 26 00

DEPTH SCALE

1 : 100



LOGGED: D.W.M./G.C.

CHECKED: G.S.W.

ON\_MOT 001-2026.GPJ ON\_MOT.GDT 1 24 01

Continued Next Page

+<sup>3</sup>, X<sup>3</sup>: Numbers refer to Sensitivity      O<sup>3%</sup> STRAIN AT FAILURE



ON\_MOT 001-2026.GPJ ON\_MOT.GDT 1 24 01

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity      ○<sup>3%</sup> STRAIN AT FAILURE

PROJECT 001-2026 (5002)				RECORD OF BOREHOLE No 22-2				3 OF 3		METRIC						
W.P. 128-92-00 and 451-90-06				LOCATION N 5029101.989 E 321334.891				ORIGINATED BY E.S.								
DIST 9 HWY 417				BOREHOLE TYPE CME 55 Bombardier				COMPILED BY G.C.								
DATUM Geodetic				DATE Mar. 21-23, 2000				CHECKED BY G.S.W.								
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)	
40.00	Silty Clay, trace fine sand Stiff Grey with black streaking Wet		10	SS	10											
54.30	Silty Clay, silty fine sand layers, rock fragments Stiff Grey Wet Silty Sand with gravel, cobbles and boulders (GLACIAL TILL)		11	SS	7	60										
46.12																
53.70																
46.72																
50.46	Faintly weathered, thickly bedded, dark grey to light grey, fine grained DOLOMITIC LIMESTONE					55										
49.96																
47.18	Bedrock cored between 49.96m and 53.24m depth				50											
53.24																
	For bedrock coring details refer to Record of Drillhole 22-2 END OF BOREHOLE				48											

ON\_MOT 001-2026 GPJ ON\_MOT.GDT 1 24 01

PROJECT: 001-2026 (5002)

**RECORD OF DRILLHOLE: 22-2**

SHEET 2 OF 2

LOCATION: N 5029101.989 E 321334.891

DRILLING DATE: March 23, 2000

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 55 Bombardier

DRILLING CONTRACTOR: Marathon Drilling

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN NO.	PENETRATION RATE (mm/min)	FLUSH	RECOVERY TOTAL CORE % SOLID CORE %	R.Q.D. %	FRACT. INDEX PER 0.3	DIP w.r.t. CORE AXIS	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	HYDRAULIC CONDUCTIVITY K, cm/sec	DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
50	NQ Core	Faintly weathered, thickly bedded, dark grey to light grey, fine grained DOLOMITIC LIMESTONE		50.46											
				49.96	1	100						VR J, Ca, R FR, Ca, R FR, R J, FR, R MF, R			
					2	100						FR, R, Ca FR, R, Ca FR, R, Ca FR, R, Ca J, FR, R MF, R F, R, Ca			
52					3	100									
		END OF BOREHOLE		47.18 53.24											
54															
56															
58															
60															
62															
64															
66															
68															

DRILLHOLE ROCKINT.GPJ GLDR\_CAN.GDT 11 28 00

DEPTH SCALE

1 : 100



LOGGED: E.S./G.C.

CHECKED: G.S.W.

ON MOT 001-2026.GPJ ON MOT.GDT 1 24 01

+<sup>3</sup>, X<sup>3</sup>: Numbers refer to Sensitivity      ○<sup>3</sup>% STRAIN AT FAILURE

ON\_MOT 001-2026.GPJ ON\_MOT.GDT 1 24 01

Continued Next Page

+<sup>3</sup>, X<sup>3</sup>: Numbers refer to Sensitivity      O<sup>3%</sup> STRAIN AT FAILURE

ON\_MOT 001-2026.GPJ ON\_MOT.GDT 1 24 01

+<sup>3</sup>, X<sup>3</sup>: Numbers refer to Sensitivity      O<sup>3%</sup> STRAIN AT FAILURE

PROJECT: 001-2026 (5002)

**RECORD OF DRILLHOLE: 22-3**

SHEET 2 OF 2

LOCATION: N 5029071.942 E 321321.193

DRILLING DATE: March 20, 2000

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 55 Bombardier

DRILLING CONTRACTOR: Marathon Drilling

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN NO.	PENETRATION RATE (m/min)	COLOUR FLUSH % RETURN	FR-FRACTURE F-FAULT SM-SMOOTH FL-FLEXURED BC-BROKEN CORE CL-CLEAVAGE J-JOINT R-ROUGH UE-UNEVEN MB-MECH. BREAK SH-SHEAR P-POLISHED ST-STEPPED W-WAVY B-BEDDING VN-VEIN S-SLICKENSIDED PL-PLANAR C-CURVED										DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																												
								RECOVERY		R.Q.D. %	FRACT. INDEX PER 0.3	DISCONTINUITY DATA		HYDRAULIC CONDUCTIVITY k, cm/sec																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																	
								TOTAL CORE %	SOLID CORE %			DIP w.r.t CORE AXIS	TYPE AND SURFACE DESCRIPTION	10 <sup>-5</sup>	10 <sup>-4</sup>	10 <sup>-3</sup>																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																															
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DRILLHOLE ROCKGINT.GPJ GLDR.CAN.GDT 11 26 00

DEPTH SCALE

1 : 100



LOGGED: E.S./G.C.

CHECKED: G.S.W.

PROJECT 001-2026 (5002)			RECORD OF BOREHOLE No 22-4		1 OF 3		METRIC						
W.P. 128-92-00 and 451-90-06		LOCATION N 5029057.169 E 321287.179		ORIGINATED BY E.S.									
DIST 9 HWY 417		BOREHOLE TYPE CME 55 Bombardier		COMPILED BY G.C.									
DATUM Geodetic		DATE Mar. 24 & 27, 2000		CHECKED BY G.S.W.									
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT	PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES								
100.10	Ground Surface												
0.00	Sand and Gravel (FILL)												
99.65	Topsoil												
0.71	Silty Clay, mottled Stiff Brown Moist												
			1	SS	4								
94.00	Silty Clay, trace fine sand Firm Grey Wet												
6.10			2	SS	PM								
90.95	Silty Clay, occ. fine sand Firm Grey with black streaking Wet												
9.15			3	SS	PM								
	occ. shell fragments at 12.2m depth		4	SS	PM								
			5	SS	PM								
			6	SS	PM								
80.10													

ON\_MOT\_001-2026.GPJ ON\_MOT.GDT 1 24 01

Continued Next Page

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



ON\_MOT 001-2026.GPJ ON\_MOT.GDT 1 24 01

Continued Next Page

**+<sup>3</sup>, ×<sup>3</sup>:** Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

PROJECT 001-2026 (5002)			RECORD OF BOREHOLE No 22-4			3 OF 3			METRIC					
W.P. 128-92-00 and 451-90-06			LOCATION N 5029057.169 E 321287.179			ORIGINATED BY E.S.								
DIST 9 HWY 417			BOREHOLE TYPE CME 55 Bombardier			COMPILED BY G.C.								
DATUM Geodetic			DATE Mar. 24 & 27, 2000			CHECKED BY G.S.W.								
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
	Silty Clay, occ. layer of clayey silt Stiff Grey with black streaking Wet (continued)		10	SS	9		60							
							59	X						
							58	X						
							57							
							56							
							55							
54.38							54							
45.72	Silty Clay to Clayey Silt, silty fine sand seams Very Stiff Grey Wet		11	SS	12		53							
							52							
51.33							51							
48.77	Silty Sand, some gravel, cobbles and boulders (GLACIAL TILL)						50							
							49							
48.34							48							
51.76	Faintly weathered, thickly bedded, grey, fine grained, DOLOMITIC LIMESTONE						47							
							46							
	Bedrock cored between 51.76m and 54.89m depth													
45.21	For bedrock coring details refer to Record of Drillhole 22-4													
54.89	END OF BOREHOLE													

ON\_MOT\_001-2026.GPJ ON\_MOT.GDT 1 24 01

PROJECT: 001-2026 (5002)

**RECORD OF DRILLHOLE: 22-4**

SHEET 2 OF 2

LOCATION: N 5029057.169 E 321287.179

DRILLING DATE: March 27, 2000

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 55 Bombardier

DRILLING CONTRACTOR: Marathon Drilling

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV.	RUN NO.	PENETRATION RATE (m/min)	FLUSH % RETURN	FR-FRACTURE CL-CLEAVAGE SH-SHEAR VN-VEIN	F-FAULT J-JOINT P-POLISHED S-SLICKENSIDED	SM-SMOOTH R-ROUGH ST-STEPPED PL-PLANAR	FL-FLEXURED UE-UNEVEN W-WAVY C-CURVED	BC-BROKEN CORE MB-MECH. BREAK B-BEDDING	DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION				
				DEPTH (m)														
				RECOVERY											R.Q.D. %	FRACT. INDEX PER 0.3	DISCONTINUITY DATA	HYDRAULIC CONDUCTIVITY k, cm/sec
				TOTAL CORE %											SOLID CORE %	TYPE AND SURFACE DESCRIPTION	10 <sup>-1</sup> 10 <sup>-2</sup> 10 <sup>-3</sup>	
52	NO Core	Faintly weathered, thickly bedded, grey, fine grained, DOLOMITIC LIMESTONE		48.34														
51.76				1	100													
54				2	100													
		END OF BOREHOLE		45.21														
				54.89														
56																		
58																		
60																		
62																		
64																		
66																		
68																		
70																		

DEPTH SCALE

1 : 100



LOGGED: E.S./G.C.

CHECKED: G.S.W.

DRILLHOLE ROCKINT.GPJ GLDR\_CAN.GDT 11 26 00

PROJECT 001-2026 (5002)

# RECORD OF BOREHOLE No 22-5

1 OF 3

METRIC

W.P. 128-92-00 and 451-90-06

LOCATION N 5029023.839 E 321270.079

ORIGINATED BY D.W.M.

DIST 9 HWY 417

BOREHOLE TYPE CME 55 Bombardier

COMPILED BY G.C.

DATUM Geodetic

DATE Mar. 21, 22&24, 2000

CHECKED BY G.S.W.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
99.99	Ground Surface							20 40 60 80 100	20 40 60 80 100	25 50 75				
0.00	Sand and Gravel (FILL)							○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED						
99.56														
0.43	Silty Clay, mottled, trace line sand Stiff to firm Brown Moist to wet		1	SS	9									
			2	SS	2									
95.42														
4.57	Silty Clay Firm Grey brown Moist to wet		3	SS	PM									
93.90														
6.09	Silty Clay Soft to firm Grey with black streaking Wet		4	SS	WH									
			5	SS	WH									
			6	SS	WH									
			7	SS	WH									
			8	SS	WH									
			9	SS	WH									
			10	TO	WH									
			11	SS	WH									
81.70														
18.29	Silty Clay, occ. thin silty sand seams Firm to stiff Grey with black streaking Wet		12	SS	WH									

Continued Next Page

+<sup>3</sup> . X<sup>3</sup> : Numbers refer to Sensitivity O<sup>3</sup>% STRAIN AT FAILURE

ON\_MOT\_001-2026.GPJ ON\_MOT.GDT 1 24 01

ON\_MOT 001-2026.GPJ ON\_MOT.GDT 1 24 01

Continued Next Page

**+<sup>3</sup>, ×<sup>3</sup>:** Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

PROJECT 001-2026 (5002)		RECORD OF BOREHOLE No 22-5		3 OF 3		METRIC												
W.P. 128-92-00 and 451-90-06		LOCATION N 5029023.839 E 321270.079		ORIGINATED BY D.W.M.														
DIST 9 HWY 417		BOREHOLE TYPE CME 55 Bombardier		COMPILED BY G.C.														
DATUM Geodetic		DATE Mar. 21, 22&24, 2000		CHECKED BY G.S.W.														
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					WATER CONTENT (%)					
								20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL X REMOULDED					W <sub>p</sub> W W <sub>L</sub> 25 50 75			GR SA SI CL		
	Silty Clay, occ. thin silt sand / silt seams Stiff Grey Wet (continued)		19	SS	WH		59											
			20	SS	WH		58											
							57											
							56											
							55											
							54											
							53											
							52											
51.23							51											
48.76	Silty Clay to Clayey Silt, silty sand seams Stiff Grey Wet		21	SS	PM		50											
							49											
							48											
47.57							47											
52.42	Silty Sand, some gravel and cobbles Compact to dense Grey Wet (GLACIAL TILL)		22	SS	30		46											
45.64							45											
54.35	Faintly weathered, thickly bedded, grey, fine grained, DOLOMITIC LIMESTONE						44											
							43											
	Bedrock cored between 54.35m and 57.86m depth																	
42.13	For bedrock coring details refer to Record of Drillhole 22-5																	
57.86	END OF BOREHOLE																	
	Note: Water level in lower piezometer at 3.23m depth and in upper piezometer at 1.54m depth on May 28, 2000																	

ON\_MOT\_001-2026.GPJ ON\_MOT.GDT 1 24 01

PROJECT: 001-2026 (5002)

## RECORD OF DRILLHOLE: 22-5

SHEET 2 OF 2

LOCATION: N 5029023.839 E 321270.079

DRILLING DATE: March 24, 2000

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 55 Bombardier

DRILLING CONTRACTOR: Marathon Drilling

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN NO.	PENETRATION RATE (mm/min)	FLUSH	RECOVERY TOTAL CORE % SOLID CORE %	R.Q.D. %	FRACT. INDEX PER 0.3	DISCONTINUITY DATA DIP w.r.t. CORE AXIS TYPE AND SURFACE DESCRIPTION	HYDRAULIC CONDUCTIVITY k, cm/sec	DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
				45.64										
				54.35	1		100				JR, R MF, R			
					2		100				MF, R MF, R F, R, Ca F, R, Ca JR, R FR, R MF, R JR, R FR, R FR, R, Ca V, FR, R FR, R, Ca FR, R, Ca J, FR, R FR, R M, FR, R FR, R J, R, Ca J, R, Ca			
56	NO Core	Faintly weathered, thickly bedded, grey, fine grained DOLOMITIC LIMESTONE												
				42.13	3		100							
58		END OF BOREHOLE		57.86										
60														
62														
64														
66														
68														
70														
72														
74														

DRILLHOLE ROCKINT.GPJ GLDR. CAN.GDT 11 28 00

DEPTH SCALE

1 : 100



LOGGED: D.W.M./G.C.

CHECKED: G.S.W.

ON\_MOT 001-2026.GPJ ON\_MOT.GDT 1 24 01

Continued Next Page

**+<sup>3</sup>, X<sup>3</sup>:** Numbers refer to Sensitivity      **O<sup>3</sup>:** STRAIN AT FAILURE



PROJECT 001-2026 (5002)			RECORD OF BOREHOLE No 22-6			2 OF 2		METRIC											
W.P. 128-92-00 and 451-90-06		LOCATION N 5029020.896 E 321248.858		ORIGINATED BY D.W.M.															
DIST 9 HWY 417		BOREHOLE TYPE CME 55 Bombardier		COMPILED BY G.C.															
DATUM Geodetic		DATE Mar 24 & 25, 2000		CHECKED BY G.S.W.															
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			WATER CONTENT (%)			Y			GR SA SI CL		
								20 40 60 80 100	20 40 60 80 100	25 50 75	W <sub>p</sub> W W <sub>L</sub>	20 40 60 80 100	25 50 75	20 40 60 80 100	25 50 75	20 40 60 80 100	25 50 75	20 40 60 80 100	25 50 75
								O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL X REMOULDED											
75.89	Silty Clay, mottled, occ. silty sand seams Firm Grey with black streaking Wet (continued)		13	SS	PM														
24.38	Silty Clay, mottled, occ. silty sand / silt seams Stiff Grey Wet		14	TP	PM														
			15	SS	PM														
			16	SS	WH														
			17	TP	PM														
68.38	END OF BOREHOLE																		
31.69	Note: Water level in lower piezometer at 2.11m depth and in upper piezometer at 1.07m depth on May 28, 2000																		

ON\_MOT 001-2026.GPJ ON\_MOT.GDT 1 24 01

+<sup>3</sup>, X<sup>3</sup>: Numbers refer to Sensitivity      O<sup>3%</sup> STRAIN AT FAILURE

ON\_MOT 001-2026.GPJ ON\_MOT.GDT 1 24 01

Continued Next Page

+<sup>3</sup>, X<sup>3</sup>: Numbers refer to Sensitivity      O<sup>3%</sup> STRAIN AT FAILURE

ON\_MOT 001-2026.GPJ ON\_MOT.GDT 1 24 01

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity      ○<sup>3%</sup> STRAIN AT FAILURE

ON\_MOT 001-2026.GPJ ON\_MOT.GDT 1 24 01

Continued Next Page

+<sup>3</sup>, X<sup>3</sup>: Numbers refer to Sensitivity      ○<sup>3</sup>% STRAIN AT FAILURE

+<sup>3</sup>, X<sup>3</sup>: Numbers refer to Sensitivity      ○<sup>3%</sup> STRAIN AT FAILURE

ON\_MOT 001-2026.GPJ ON\_MOT.GDT 1 24 01

Continued Next Page

+<sup>3</sup>, X<sup>3</sup>: Numbers refer to Sensitivity      ○<sup>3%</sup> STRAIN AT FAILURE

PROJECT <u>001-2026 (5002)</u>		<b>RECORD OF BOREHOLE No 22-12</b>		2 OF 3	<b>METRIC</b>
W.P. <u>128-92-00 and 451-90-06</u>		LOCATION <u>N 5029195.634 E 321434.905</u>		ORIGINATED BY <u>E.S.</u>	
DIST <u>9</u> HWY <u>417</u>		BOREHOLE TYPE <u>CME 55 Bombardier</u>		COMPILED BY <u>G.C.</u>	
DATUM <u>Geodetic</u>		DATE <u>Mar. 30 &amp; 31, 2000</u>		CHECKED BY <u>G.S.W.</u>	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT  $\gamma$  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		WATER CONTENT (%)				
								20 40 60 80 100	20 40 60 80 100	W <sub>p</sub>	W	W <sub>L</sub>		
	Auger drilling from 16.46m to 47.86m depth(continued)						○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL X REMOULDED							
										</				

ON\_MOT\_001-2026.GPJ ON\_MOT.GDT 1 24 01

Continued Next Page

+<sup>3</sup>, X<sup>3</sup>: Numbers refer to Sensitivity      ○ 3% STRAIN AT FAILURE



PROJECT <u>001-2026 (5002)</u>				RECORD OF BOREHOLE <b>No 22-12</b>				3 OF 3		<b>METRIC</b>					
W.P. <u>128-92-00 and 451-90-06</u>				LOCATION <u>N 5029195.634 E 321434.905</u>				ORIGINATED BY <u>E.S.</u>							
DIST <u>9</u> HWY <u>417</u>				BOREHOLE TYPE <u>CME 55 Bombardier</u>				COMPILED BY <u>G.C.</u>							
DATUM <u>Geodetic</u>				DATE <u>Mar. 30 &amp; 31, 2000</u>				CHECKED BY <u>G.S.W.</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 $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							
	Auger drilling from 16.46m to 47.86m depth (continued)							20 40 60 80 100							
53.25															
47.86	STANDARD PENETRATION CONE 16.46m to 49.93m depth														
51.18															
49.93	END OF BOREHOLE REFUSAL TO PENETRATION CONE														
	Note: Water level in piezometer at 1.64m depth on May 28, 2000														

ON\_MOT\_001-2026.GPJ ON\_MOT.GDT 1 24 01

+<sup>3</sup>, X<sup>3</sup>: Numbers refer to  
Sensitivity

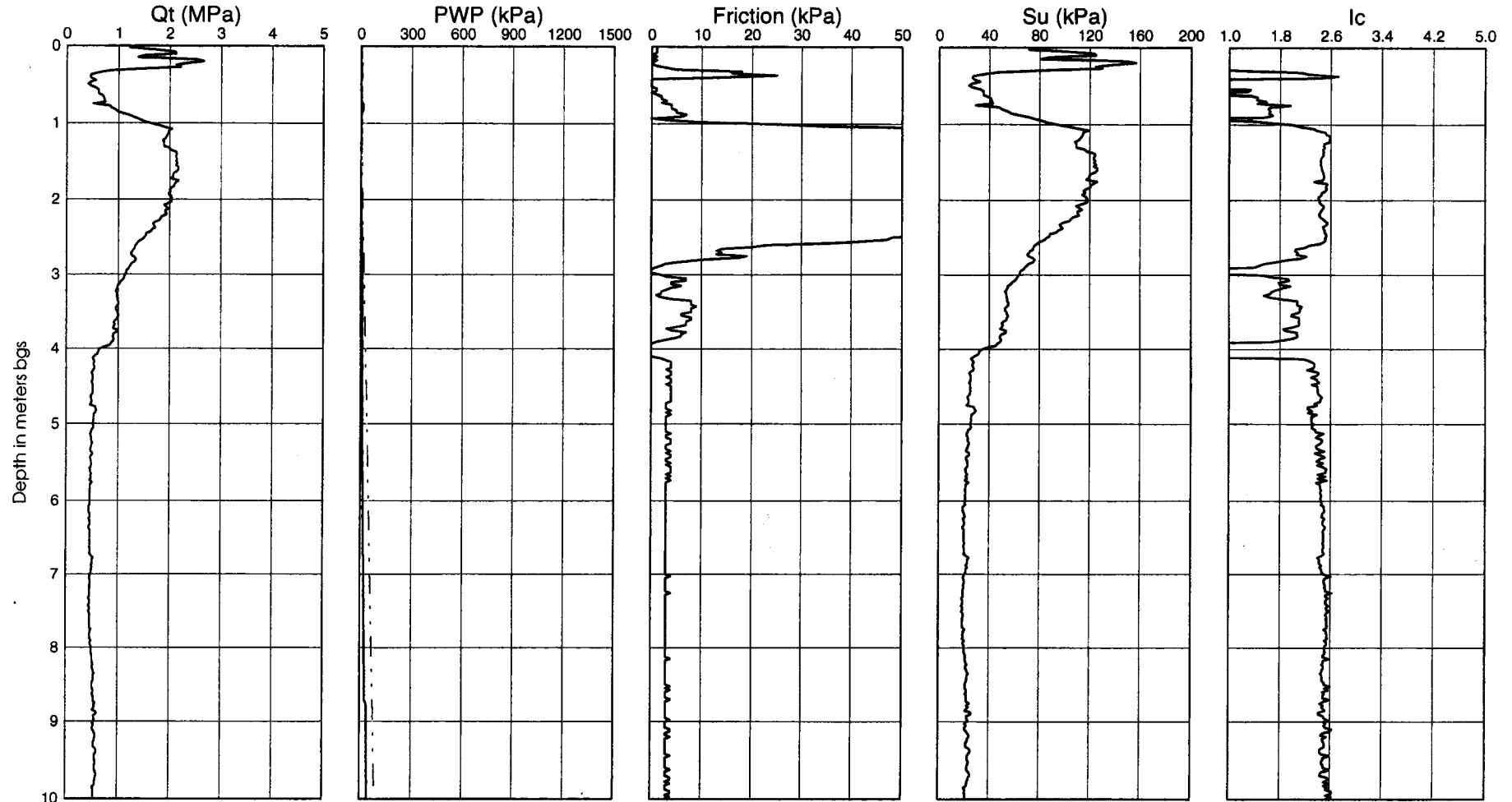
O 3% STRAIN AT FAILURE

# Cone Penetration Test - 22-5a

Test Date : Apr 10, 1900  
Location : see Figure

Operator : Golder Associates

Ground Surf. Elev. : 99.92  
Water Table Depth : 1.00



Qt normalized for  
unequal end area effects

$S_u = (Q_t - \sigma_v) / N_k$   
 $N_k = 17$   
 $\gamma = 17 \text{ kN/m}^3$

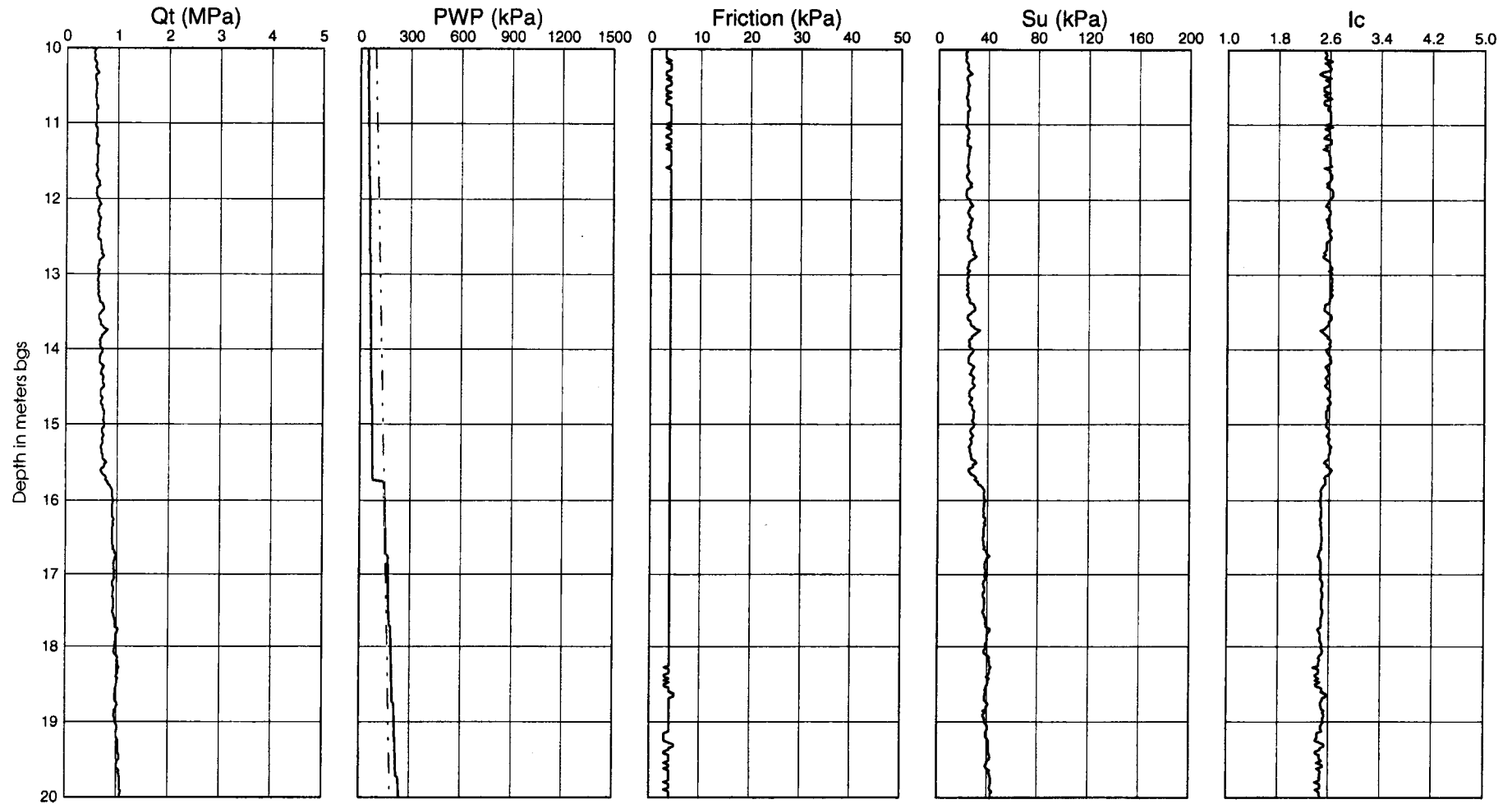
After Jefferies and Davies (1991)  
 $I_c < 1.25$  - Gravelly sands  
 $1.25 < I_c < 1.90$  - Clean to silty sand  
 $1.90 < I_c < 2.54$  - Silty sand to sandy silt  
 $2.54 < I_c < 2.82$  - Clayey silt to silty clay  
 $2.82 < I_c < 3.22$  - Clays

# Cone Penetration Test - 22-5a

Test Date : Apr 10, 1900  
Location : see Figure

Operator : Golder Associates

Ground Surf. Elev. : 99.92  
Water Table Depth : 1.00



Qt normalized for  
unequal end area effects

$S_u = (Q_t - \sigma_v) / N_k$   
 $N_k = 17$   
 $\gamma = 17 \text{ kN/m}^3$

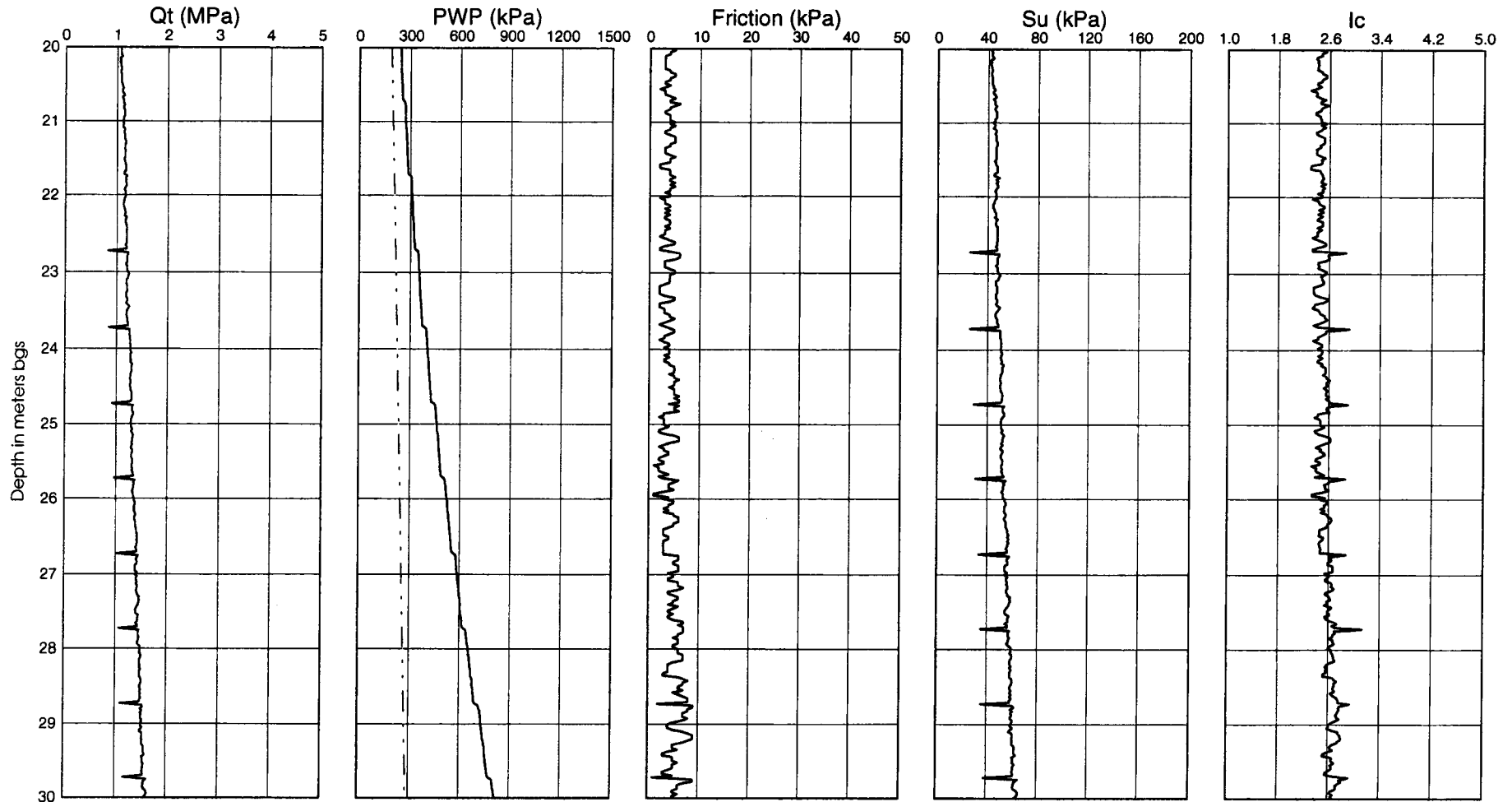
After Jefferies and Davies (1991)  
 $I_c < 1.25$  - Gravelly sands  
 $1.25 < I_c < 1.90$  - Clean to silty sand  
 $1.90 < I_c < 2.54$  - Silty sand to sandy silt  
 $2.54 < I_c < 2.82$  - Clayey silt to silty clay  
 $2.82 < I_c < 3.22$  - Clays

# Cone Penetration Test - 22-5a

Test Date : Apr 10, 1900  
Location : see Figure

Operator : Golder Associates

Ground Surf. Elev. : 99.92  
Water Table Depth : 1.00



Qt normalized for  
unequal end area effects

$S_u = (Q_t - \sigma_{v'}) / N_k$   
 $N_k = 17$   
 $\gamma = 17 \text{ kN/m}^3$

After Jeffries and Davies (1991)

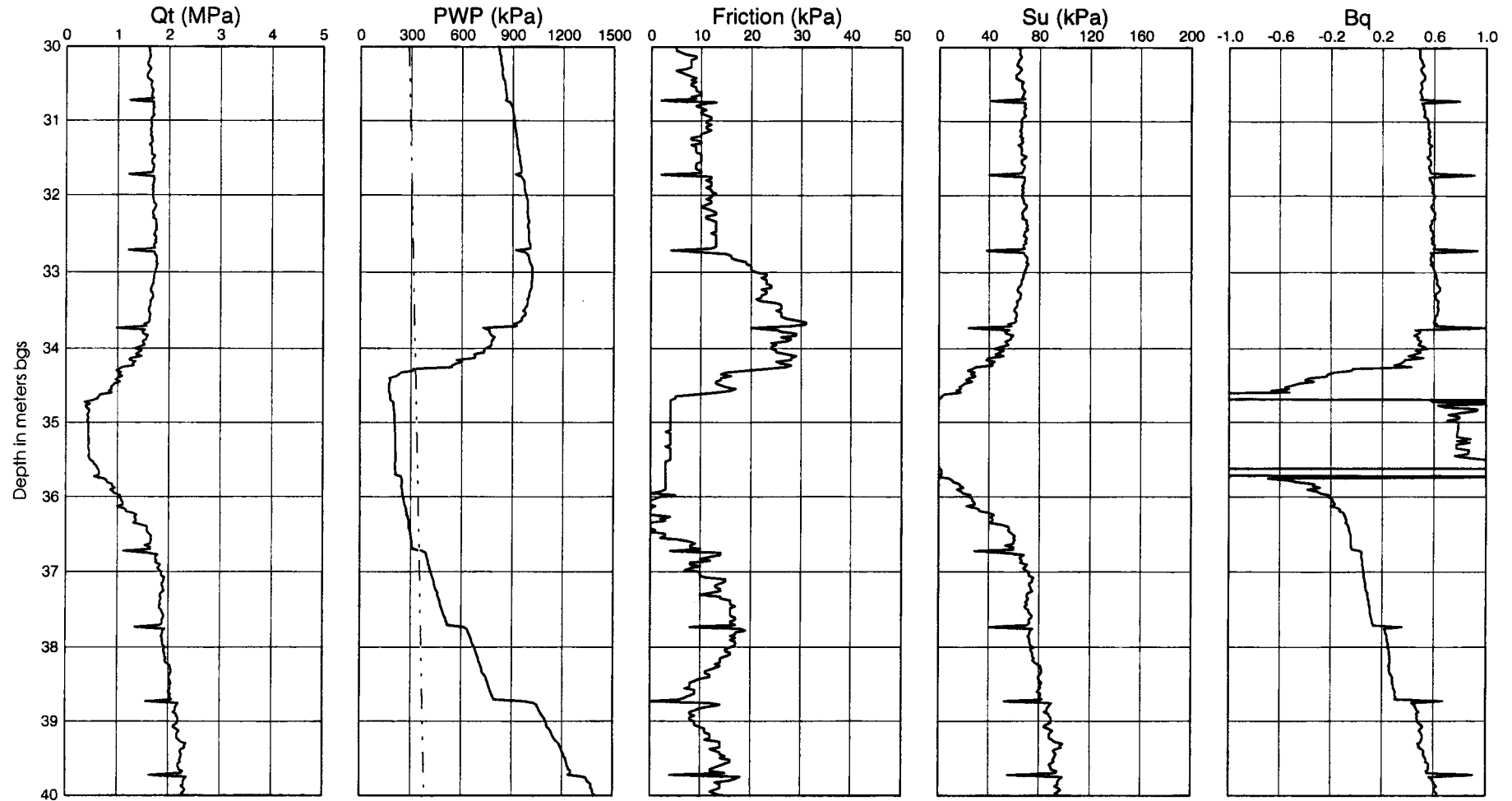
$I_c < 1.25$  - Gravelly sands  
 $1.25 < I_c < 1.90$  - Clean to silty sand  
 $1.90 < I_c < 2.54$  - Silty sand to sandy silt  
 $2.54 < I_c < 2.82$  - Clayey silt to silty clay  
 $2.82 < I_c < 3.22$  - Clays

# Cone Penetration Test - 22-5a

Test Date : Apr 10, 1900  
Location : see Figure

Operator : Golder Associates

Ground Surf. Elev. : 99.92  
Water Table Depth : 1.00



Qt normalized for  
unequal end area effects

$S_u = (Q_t - \sigma_v) / N_k$   
 $N_k = 17$   
 $\gamma = 17 \text{ kN/m}^3$

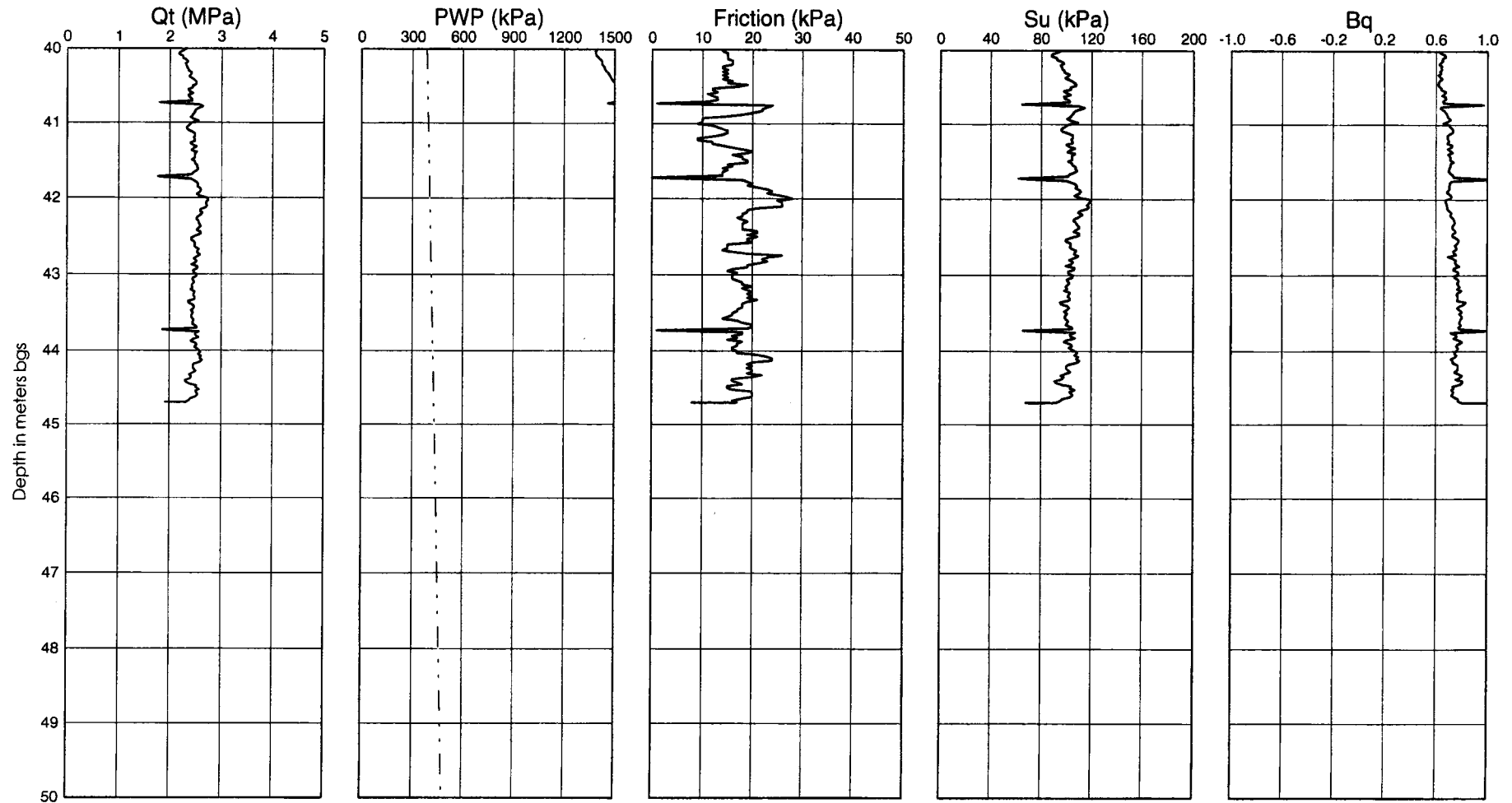
$B_q = dU / (Q_t - \sigma_v)$   
 $\gamma = 17 \text{ kN/m}^3$

# Cone Penetration Test - 22-5a

Test Date : Apr 10, 1900  
Location : see Figure

Operator : Golder Associates

Ground Surf. Elev. : 99.92  
Water Table Depth : 1.00



Qt normalized for  
unequal end area effects

$Su = (Qt - \sigma_v) / Nk$   
 $Nk = 17$   
 $\gamma = 17 \text{ kN/m}^3$

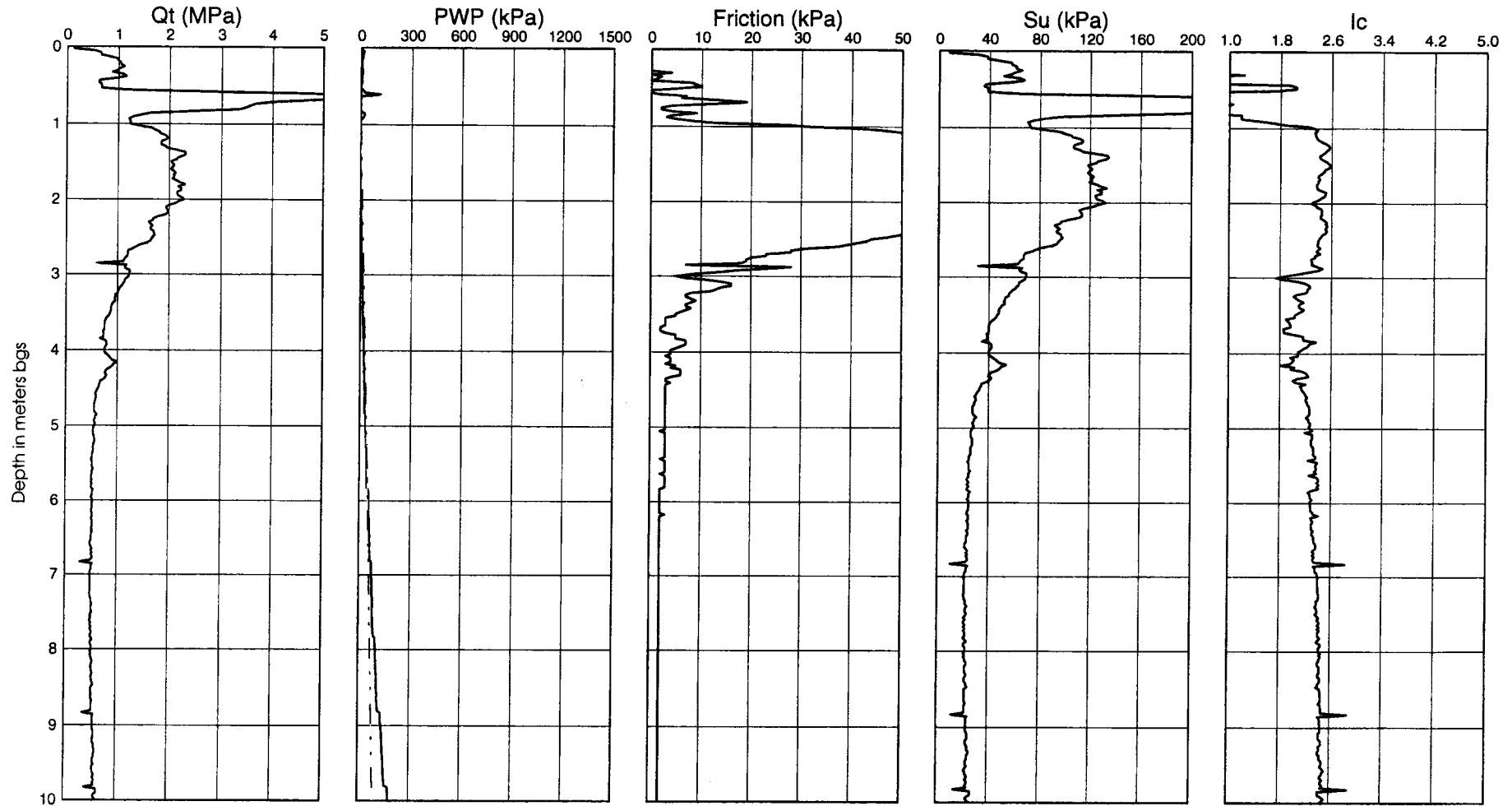
$Bq = dU / (Qt - \sigma_v)$   
 $\gamma = 17 \text{ kN/m}^3$

# Cone Penetration Test - 22-7

Test Date : Apr 07, 1900  
Location : see Figure

Operator : Golder Associates

Ground Surf. Elev. : 99.97  
Water Table Depth : 1.00



Qt normalized for  
unequal end area effects

$S_u = (Q_t - \sigma_v) / N_k$   
 $N_k = 17$   
 $\gamma = 17 \text{ kN/m}^3$

After Jeffries and Davies (1991)

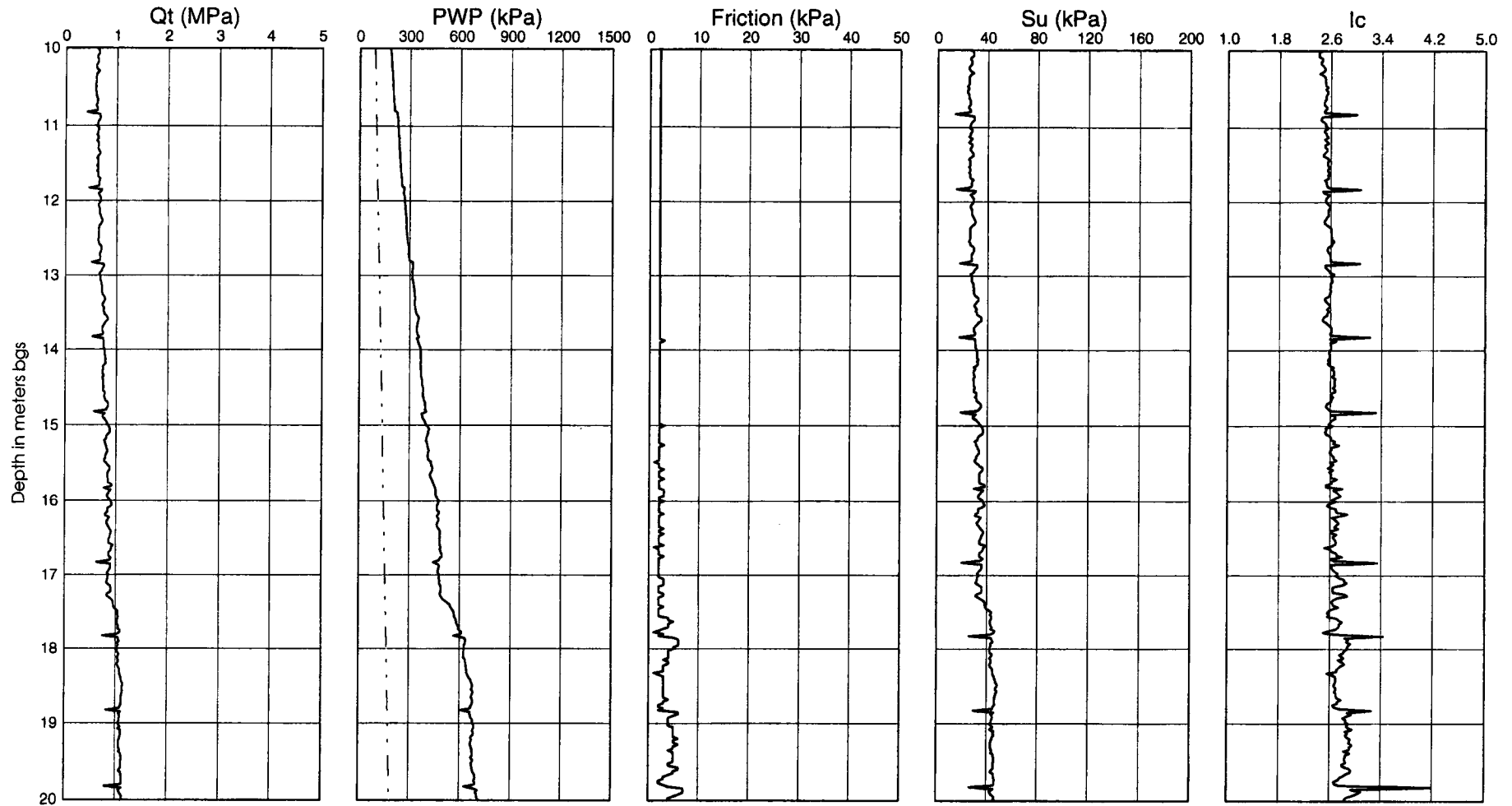
$I_c < 1.25$  - Gravelly sands  
 $1.25 < I_c < 1.90$  - Clean to silty sand  
 $1.90 < I_c < 2.54$  - Silty sand to sandy silt  
 $2.54 < I_c < 2.82$  - Clayey silt to silty clay  
 $2.82 < I_c < 3.22$  - Clays

# Cone Penetration Test - 22-7

Test Date : Apr 07, 1900  
Location : see Figure

Operator : Golder Associates

Ground Surf. Elev. : 99.97  
Water Table Depth : 1.00



Qt normalized for  
unequal end area effects

$S_u = (Q_t - \sigma_v) / N_k$   
 $N_k = 17$   
 $\gamma = 17 \text{ kN/m}^3$

After Jefferies and Davies (1991)  
 $I_c < 1.25$  - Gravelly sands  
 $1.25 < I_c < 1.90$  - Clean to silty sand  
 $1.90 < I_c < 2.54$  - Silty sand to sandy silt  
 $2.54 < I_c < 2.82$  - Clayey silt to silty clay  
 $2.82 < I_c < 3.22$  - Clays

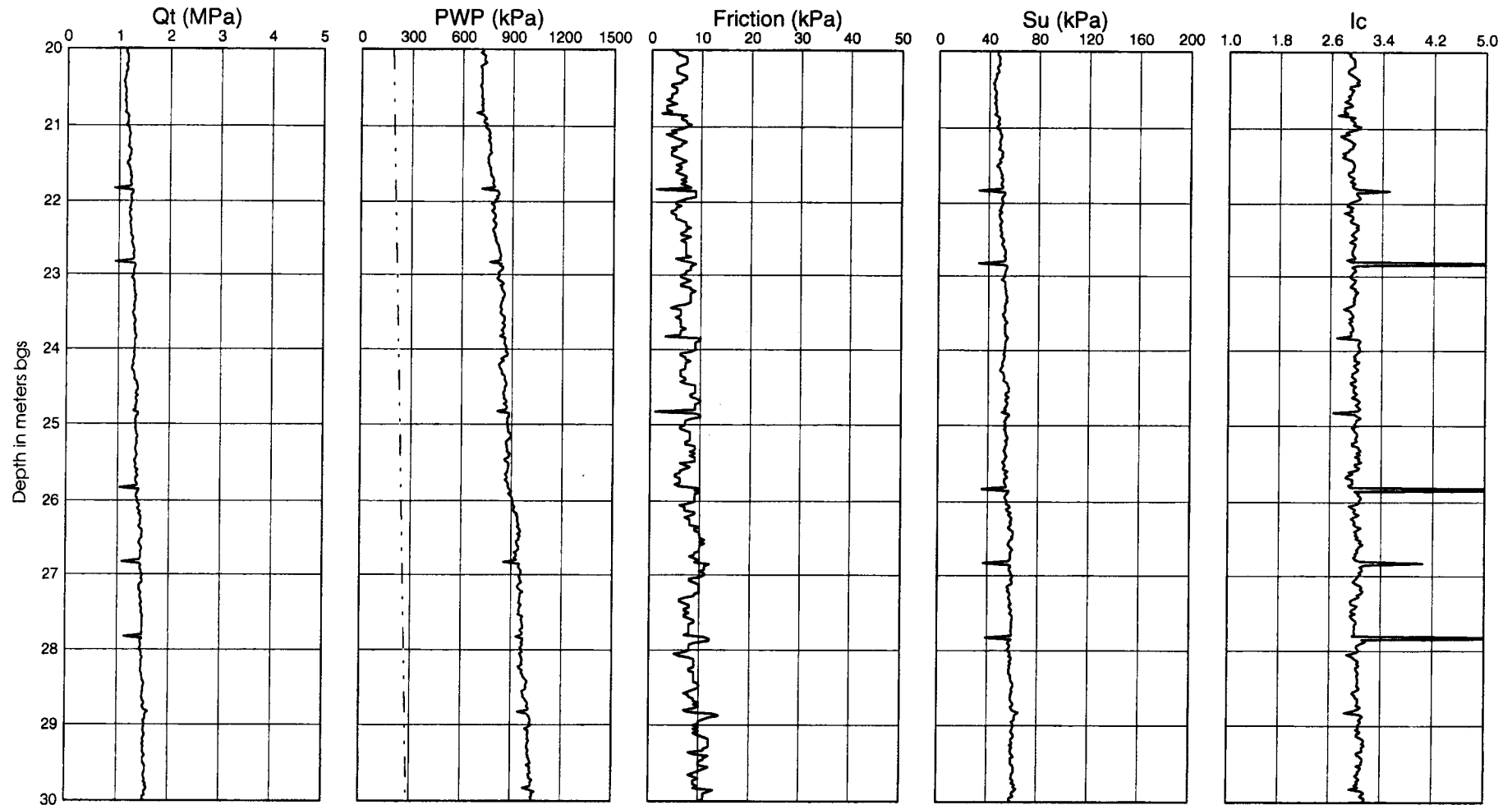


# Cone Penetration Test - 22-7

Test Date : Apr 07, 1900  
Location : see Figure

Operator : Golder Associates

Ground Surf. Elev. : 99.97  
Water Table Depth : 1.00



Qt normalized for  
unequal end area effects

$S_u = (Q_t - \sigma_{vV}) / N_k$   
 $N_k = 17$   
 $\gamma = 17 \text{ kN/m}^3$

After Jefferies and Davies (1991)

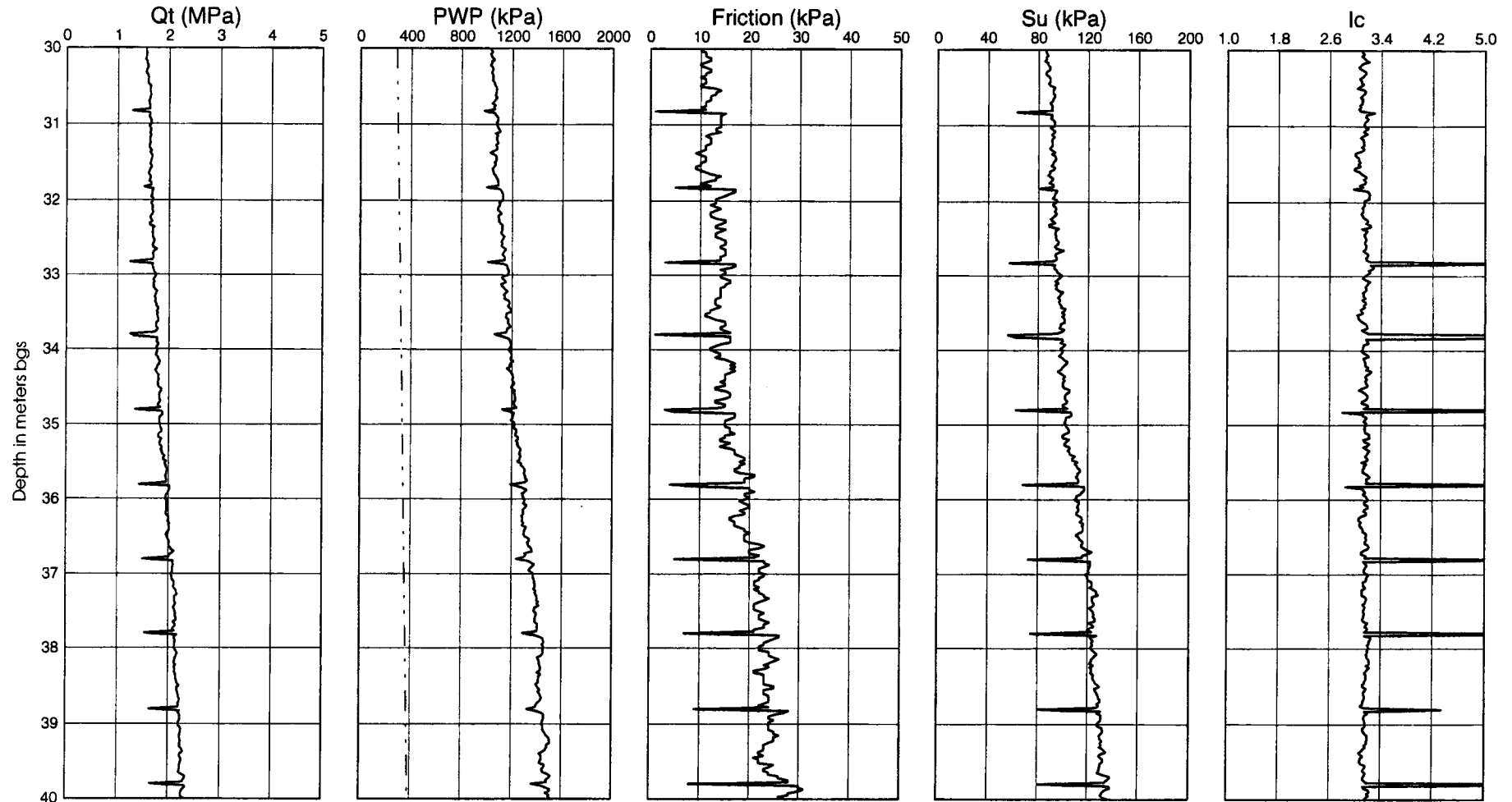
$I_c < 1.25$  - Gravely sands  
 $1.25 < I_c < 1.90$  - Clean to silty sand  
 $1.90 < I_c < 2.54$  - Silty sand to sandy silt  
 $2.54 < I_c < 2.82$  - Clayey silt to silty clay  
 $2.82 < I_c < 3.22$  - Clays

# Cone Penetration Test - 22-7

Test Date : Apr 07, 1900  
Location : see Figure

Operator : Golder Associates

Ground Surf. Elev. : 99.97  
Water Table Depth : 1.00



Qt normalized for  
unequal end area effects

$S_u = (Q_t - \sigma_v) / N_k$   
 $N_k = 12$   
 $\gamma = 17 \text{ kN/m}^3$

After Jefferies and Davies (1991)

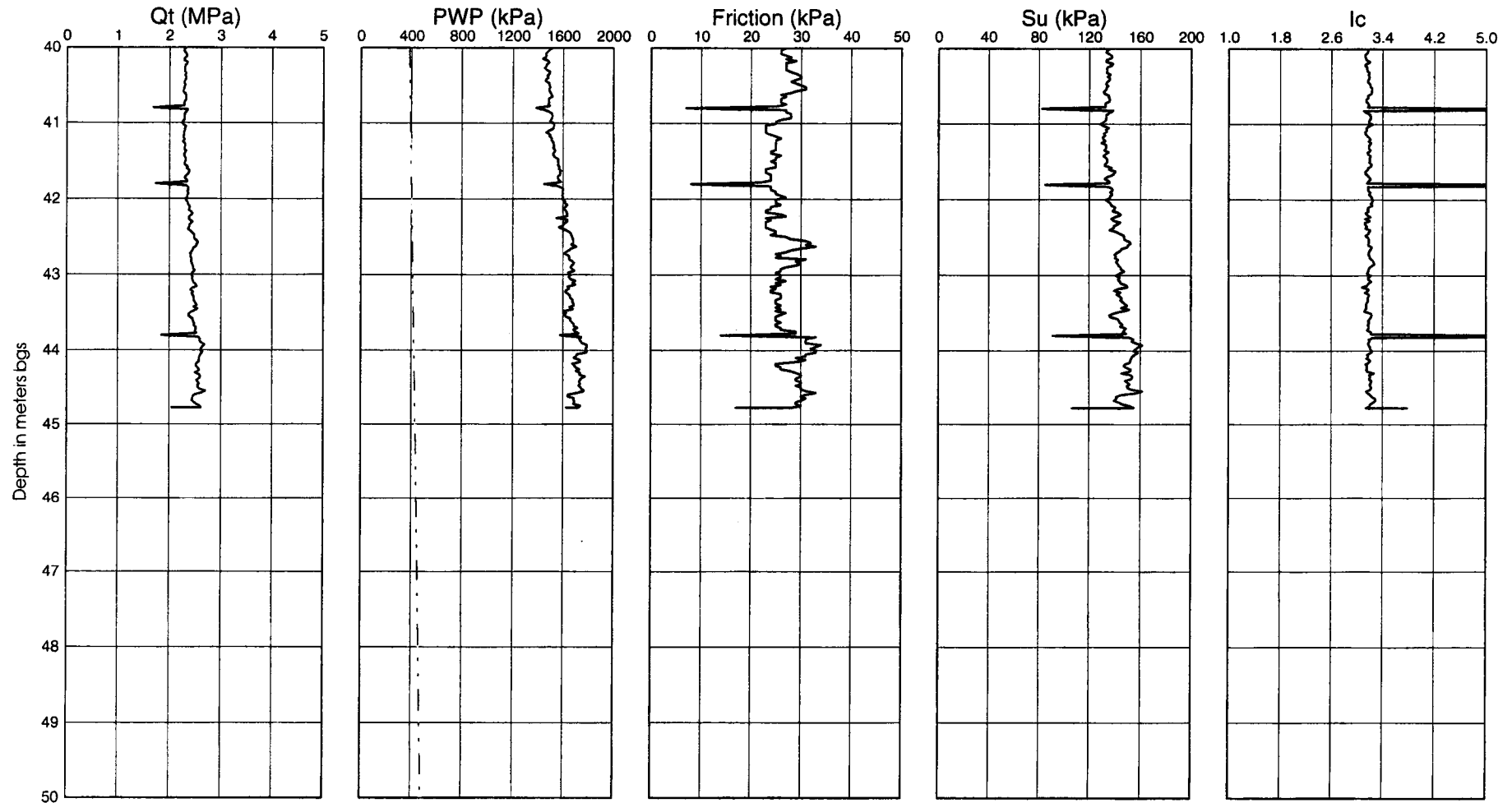
$I_c < 1.25$  - Gravelly sands  
 $1.25 < I_c < 1.90$  - Clean to silty sand  
 $1.90 < I_c < 2.54$  - Silty sand to sandy silt  
 $2.54 < I_c < 2.82$  - Clayey silt to silty clay  
 $2.82 < I_c < 3.22$  - Clays

# Cone Penetration Test - 22-7

Test Date : Apr 07, 1900  
Location : see Figure

Operator : Golder Associates

Ground Surf. Elev. : 99.97  
Water Table Depth : 1.00



Qt normalized for  
unequal end area effects

$S_u = (Q_t - \sigma_v) / N_k$   
 $N_k = 12$   
 $\gamma = 17 \text{ kN/m}^3$

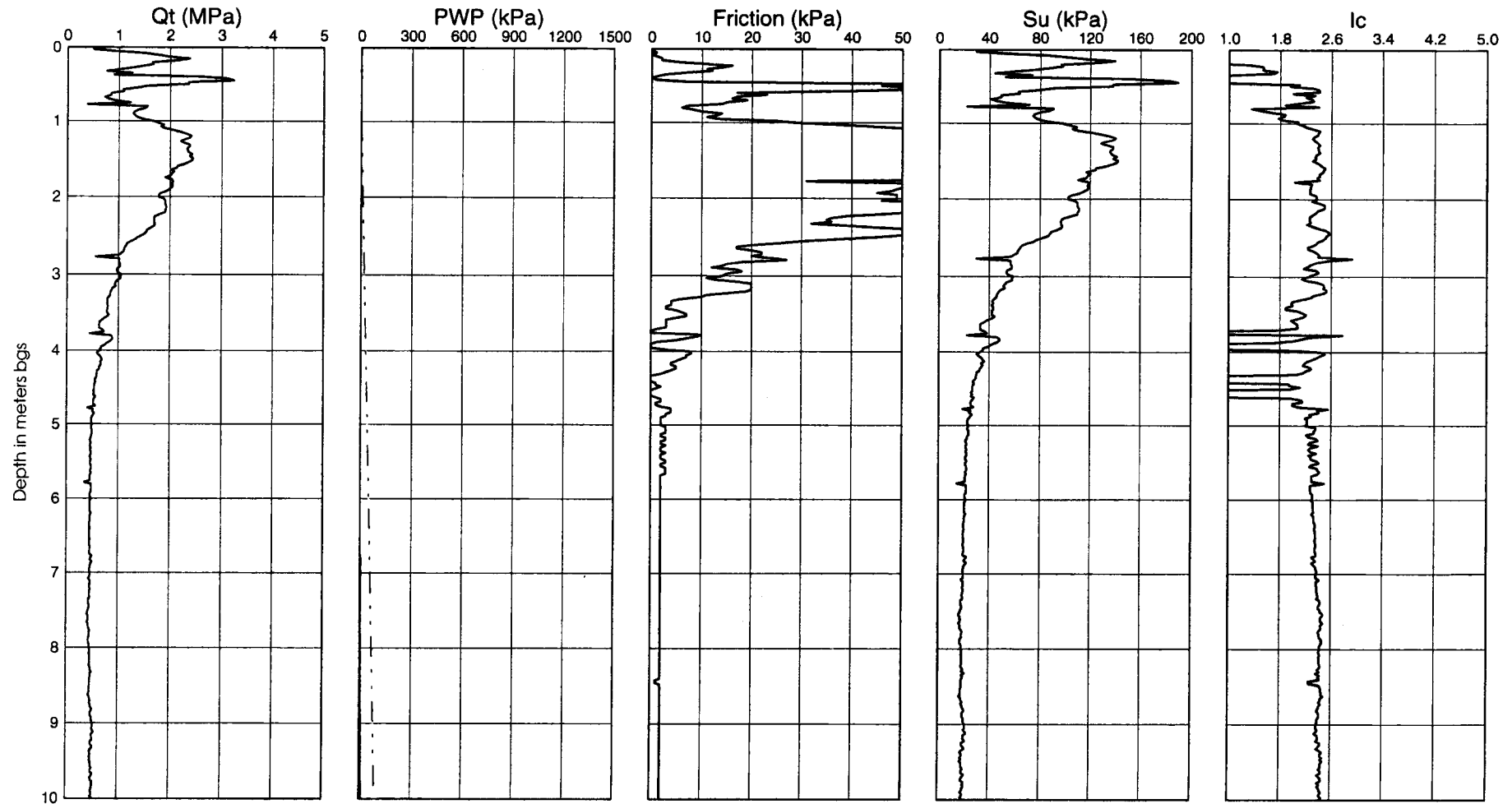
After Jefferies and Davies (1991)  
 $I_c < 1.25$  - Gravelly sands  
 $1.25 < I_c < 1.90$  - Clean to silty sand  
 $1.90 < I_c < 2.54$  - Silty sand to sandy silt  
 $2.54 < I_c < 2.82$  - Clayey silt to silty clay  
 $2.82 < I_c < 3.22$  - Clays

# Cone Penetration Test - 22-9

Test Date : Apr 07, 1900  
Location : see Figure

Operator : Golder Associates

Ground Surf. Elev. : 99.64  
Water Table Depth : 1.00



Qt normalized for  
unequal end area effects

$S_u = (Q_t - \sigma_v) / N_k$   
 $N_k = 17$   
 $\gamma = 17 \text{ kN/m}^3$

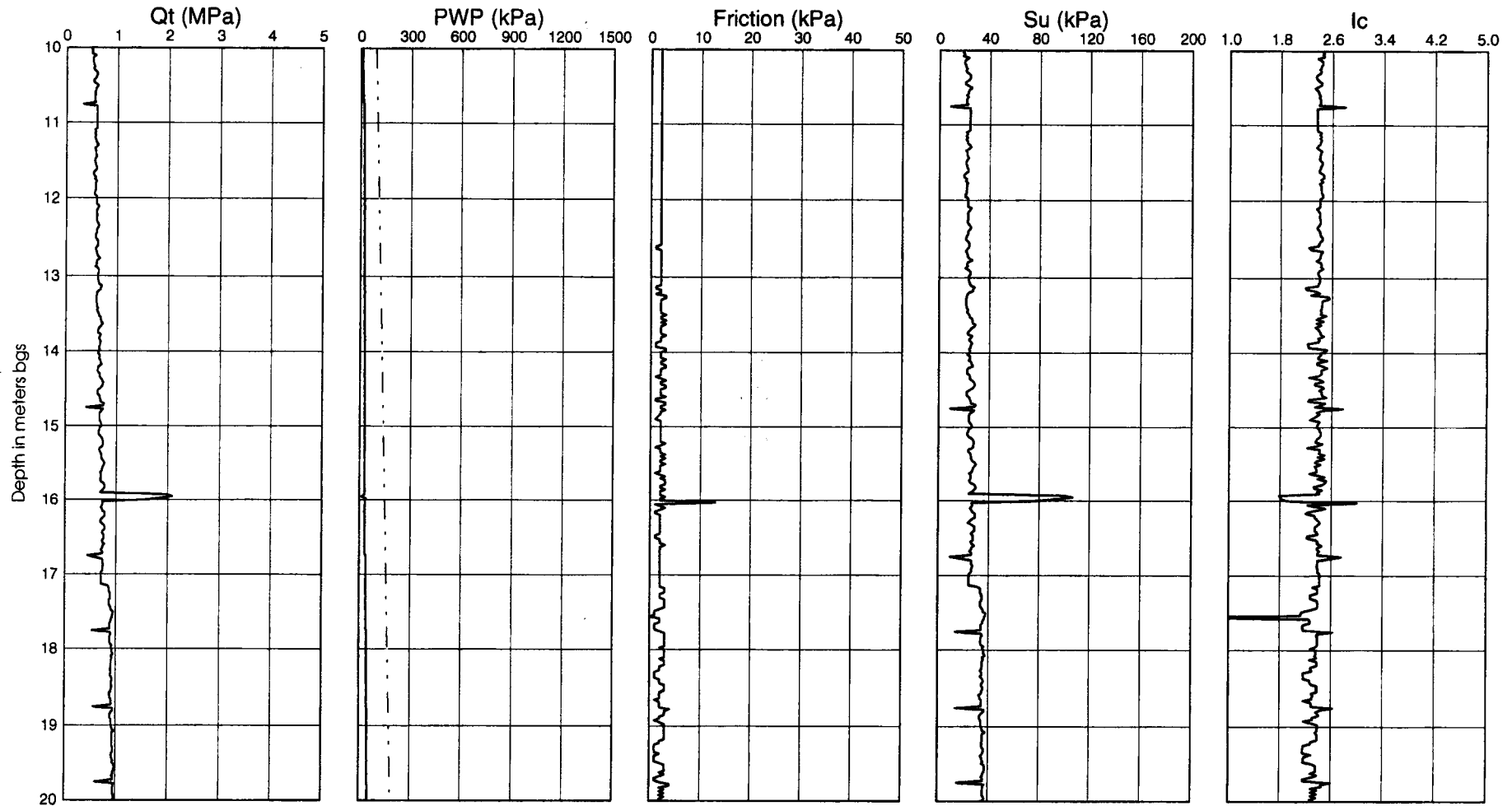
After Jefferies and Davies (1991)  
 $I_c < 1.25$  - Gravelly sands  
 $1.25 < I_c < 1.90$  - Clean to silty sand  
 $1.90 < I_c < 2.54$  - Silty sand to sandy silt  
 $2.54 < I_c < 2.82$  - Clayey silt to silty clay  
 $2.82 < I_c < 3.22$  - Clays

# Cone Penetration Test - 22-9

Test Date : Apr 07, 1900  
Location : see Figure

Operator : Golder Associates

Ground Surf. Elev. : 99.64  
Water Table Depth : 1.00



Qt normalized for  
unequal end area effects

$S_u = (Q_t - \sigma_v) / N_k$   
 $N_k = 17$   
 $\gamma = 17 \text{ kN/m}^3$

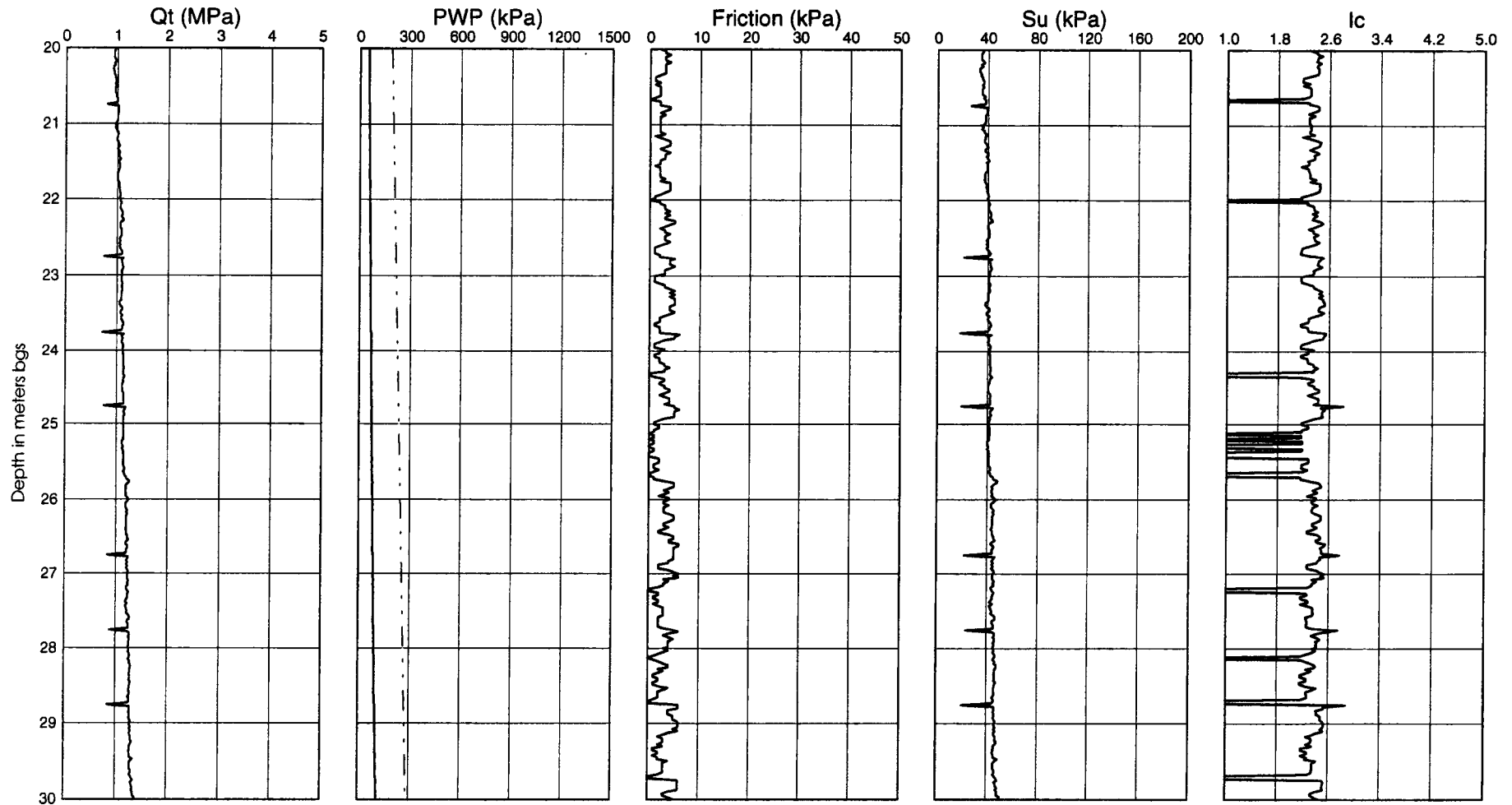
After Jefferies and Davies (1991)  
 $I_c < 1.25$  - Gravelly sands  
 $1.25 < I_c < 1.90$  - Clean to silty sand  
 $1.90 < I_c < 2.54$  - Silty sand to sandy silt  
 $2.54 < I_c < 2.82$  - Clayey silt to silty clay  
 $2.82 < I_c < 3.22$  - Clays

# Cone Penetration Test - 22-9

Test Date : Apr 07, 1900  
Location : see Figure

Operator : Golder Associates

Ground Surf. Elev. : 99.64  
Water Table Depth : 1.00



Qt normalized for  
unequal end area effects

$S_u = (Q_t - \sigma_{av}) / N_k$   
 $N_k = 17$   
 $\gamma = 17 \text{ kN/m}^3$

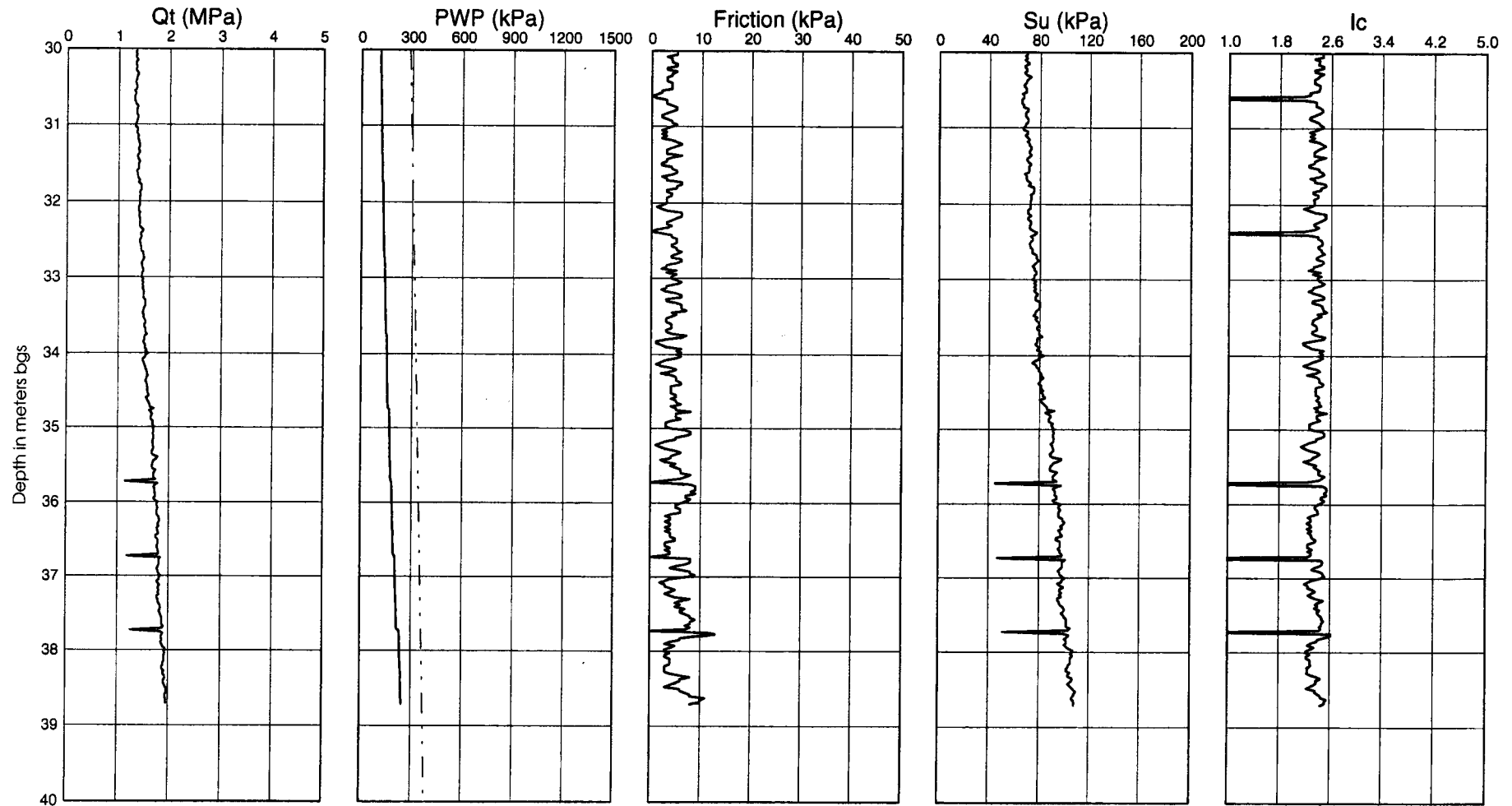
After Jefferies and Davies (1991)  
 $I_c < 1.25$  - Gravelly sands  
 $1.25 < I_c < 1.90$  - Clean to silty sand  
 $1.90 < I_c < 2.54$  - Silty sand to sandy silt  
 $2.54 < I_c < 2.82$  - Clayey silt to silty clay  
 $2.82 < I_c < 3.22$  - Clays

# Cone Penetration Test - 22-9

Test Date : Apr 07, 1900  
Location : see Figure

Operator : Golder Associates

Ground Surf. Elev. : 99.64  
Water Table Depth : 1.00



Qt normalized for  
unequal end area effects

$S_u = (Q_t - \sigma_{mv}) / N_k$   
 $N_k = 12$   
 $\gamma = 17 \text{ kN/m}^3$

After Jefferies and Davies (1991)

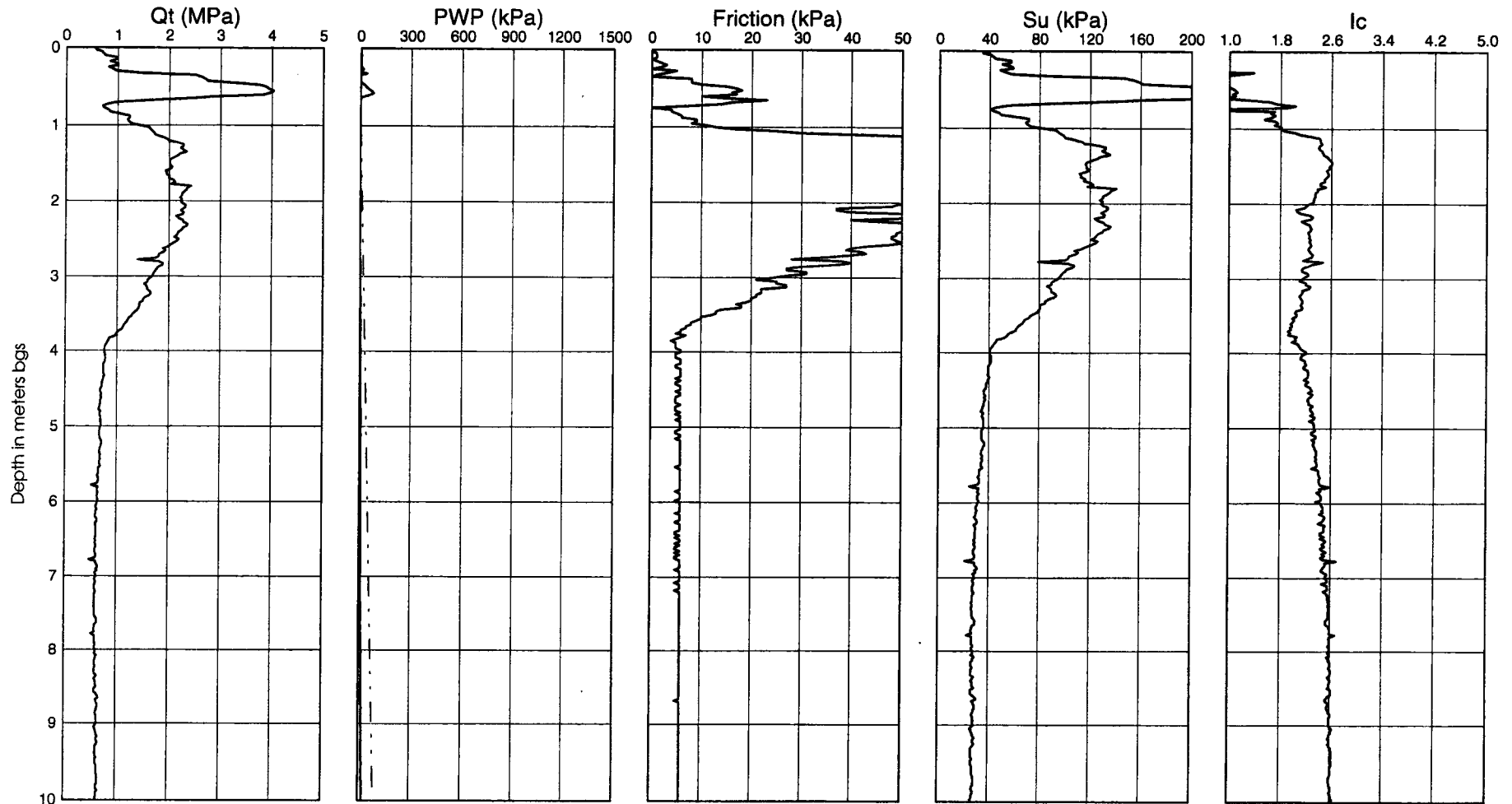
$I_c < 1.25$  - Gravelly sands  
 $1.25 < I_c < 1.90$  - Clean to silty sand  
 $1.90 < I_c < 2.54$  - Silty sand to sandy silt  
 $2.54 < I_c < 2.82$  - Clayey silt to silty clay  
 $2.82 < I_c < 3.22$  - Clays

# Cone Penetration Test - 22-11

Test Date : Apr 10, 1900  
Location : see Figure

Operator : Golder Associates

Ground Surf. Elev. : 100.70  
Water Table Depth : 1.00



Qt normalized for  
unequal end area effects

$S_u = (Q_t - \sigma_v) / N_k$   
 $N_k = 17$   
 $\gamma = 17 \text{ kN/m}^3$

After Jefferies and Davies (1991)  
 $I_c < 1.25$  - Gravelly sands  
 $1.25 < I_c < 1.90$  - Clean to silty sand  
 $1.90 < I_c < 2.54$  - Silty sand to sandy silt  
 $2.54 < I_c < 2.82$  - Clayey silt to silty clay  
 $2.82 < I_c < 3.22$  - Clays

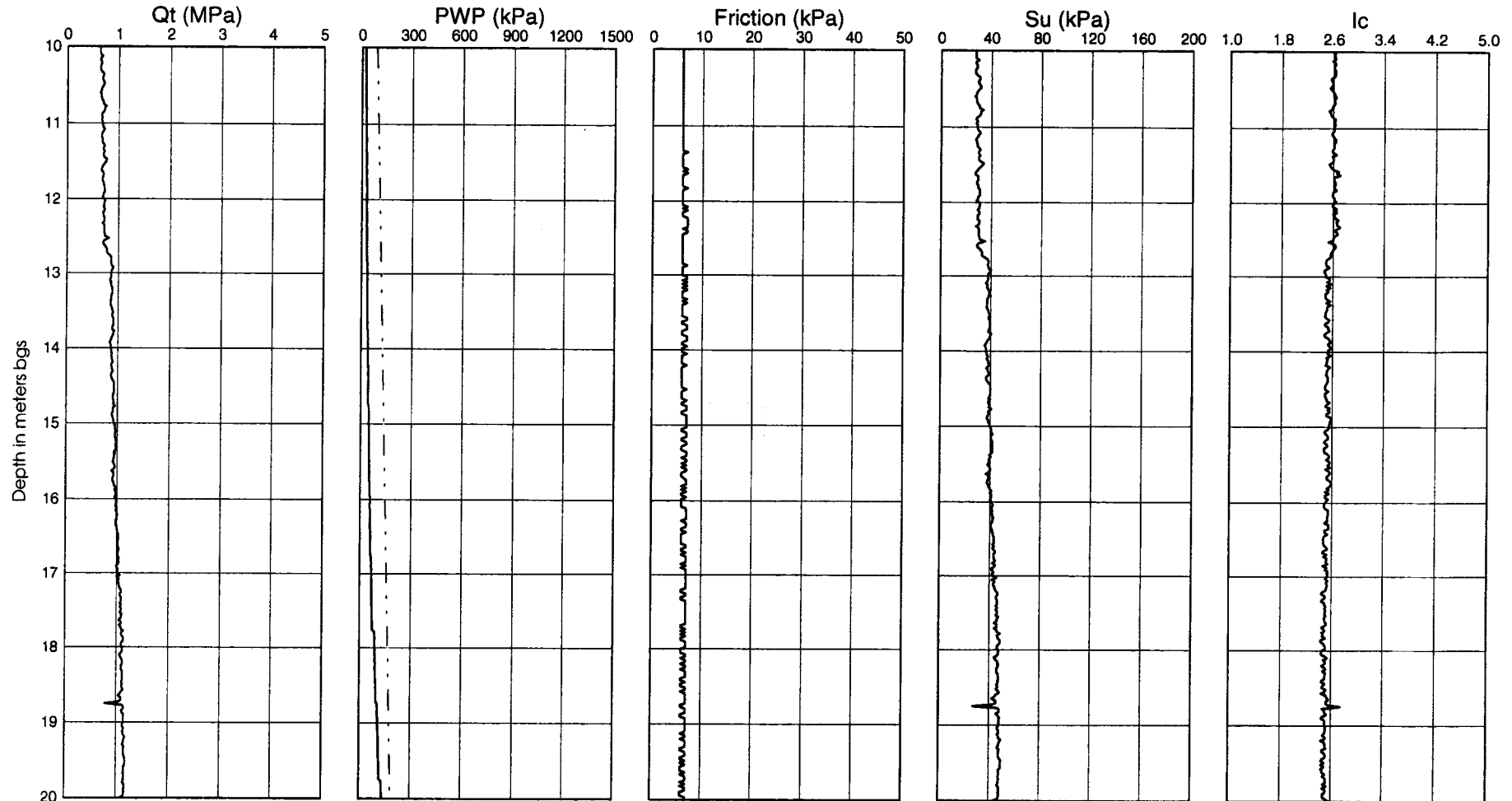


# Cone Penetration Test - 22-11

Test Date : Apr 10, 1900  
Location : see Figure

Operator : Golder Associates

Ground Surf. Elev. : 100.70  
Water Table Depth : 1.00



Qt normalized for  
unequal end area effects

$S_u = (Q_t - \sigma_v) / N_k$   
 $N_k = 17$   
 $\gamma = 17 \text{ kN/m}^3$

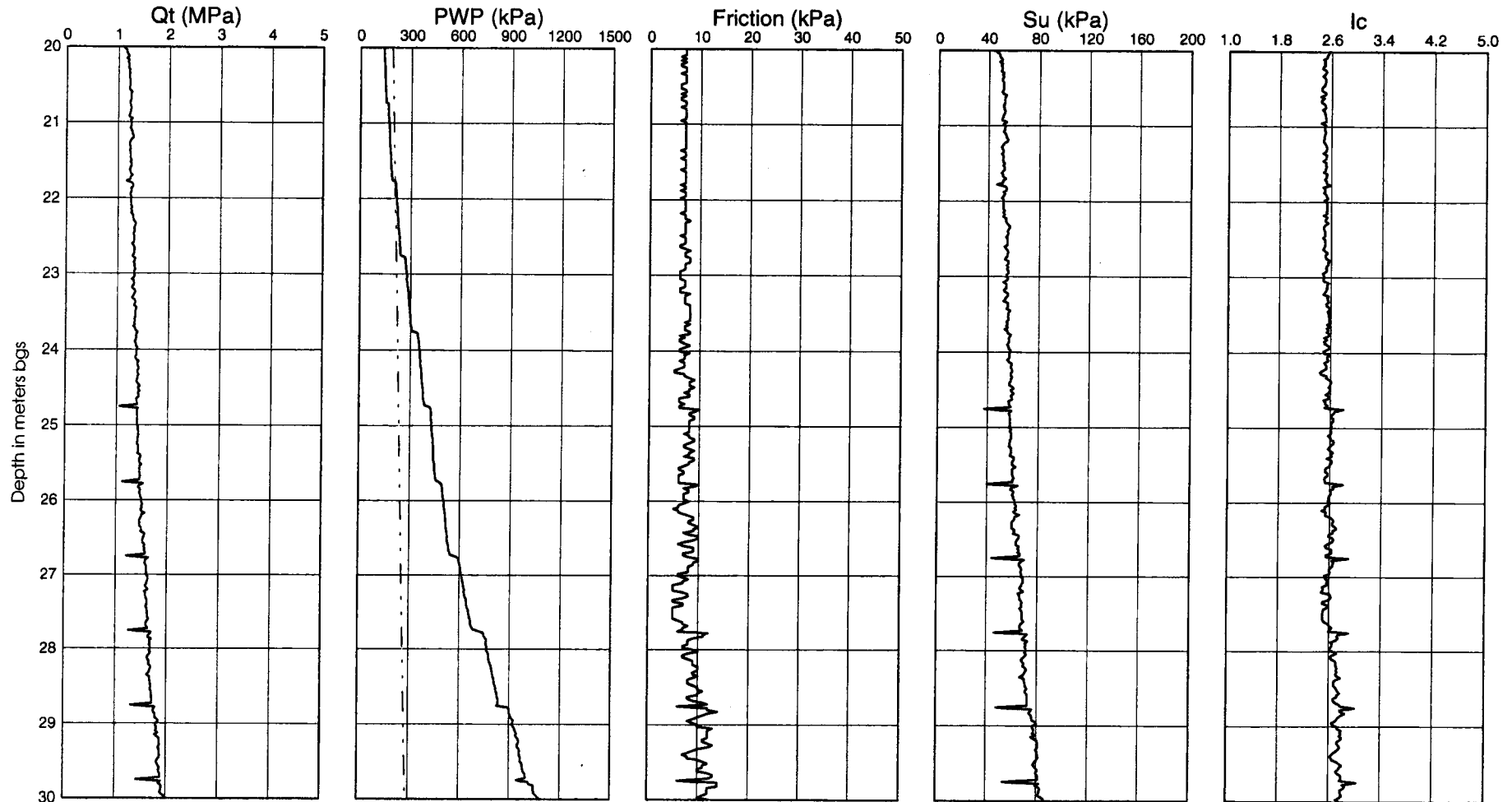
After Jefferies and Davies (1991)  
 $I_c < 1.25$  - Gravelly sands  
 $1.25 < I_c < 1.90$  - Clean to silty sand  
 $1.90 < I_c < 2.54$  - Silty sand to sandy silt  
 $2.54 < I_c < 2.82$  - Clayey silt to silty clay  
 $2.82 < I_c < 3.22$  - Clays

# Cone Penetration Test - 22-11

Test Date : Apr 10, 1900  
Location : see Figure

Operator : Golder Associates

Ground Surf. Elev. : 100.70  
Water Table Depth : 1.00



Qt normalized for  
unequal end area effects

$S_u = (Q_t - \sigma_v) / N_k$   
 $N_k = 17$   
 $\gamma = 17 \text{ kN/m}^3$

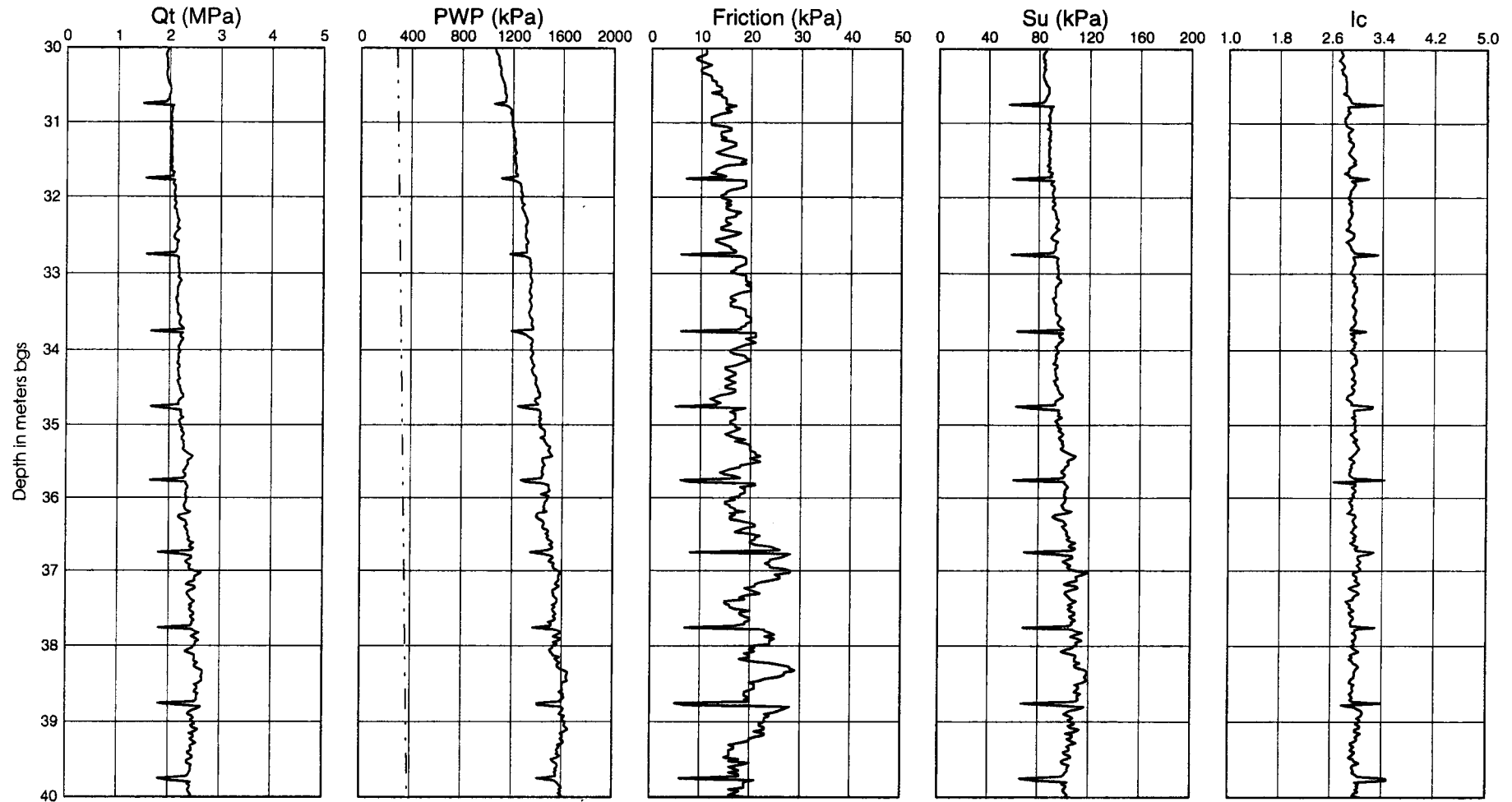
After Jefferies and Davies (1991)  
 $I_c < 1.25$  - Gravely sands  
 $1.25 < I_c < 1.90$  - Clean to silty sand  
 $1.90 < I_c < 2.54$  - Silty sand to sandy silt  
 $2.54 < I_c < 2.82$  - Clayey silt to silty clay  
 $2.82 < I_c < 3.22$  - Clays

# Cone Penetration Test - 22-11

Test Date : Apr 10, 1900  
Location : see Figure

Operator : Golder Associates

Ground Surf. Elev. : 100.70  
Water Table Depth : 1.00



Qt normalized for  
unequal end area effects

$S_u = (Q_t - \sigma_{av}) / N_k$   
 $N_k = 17$   
 $\gamma = 17 \text{ kN/m}^3$

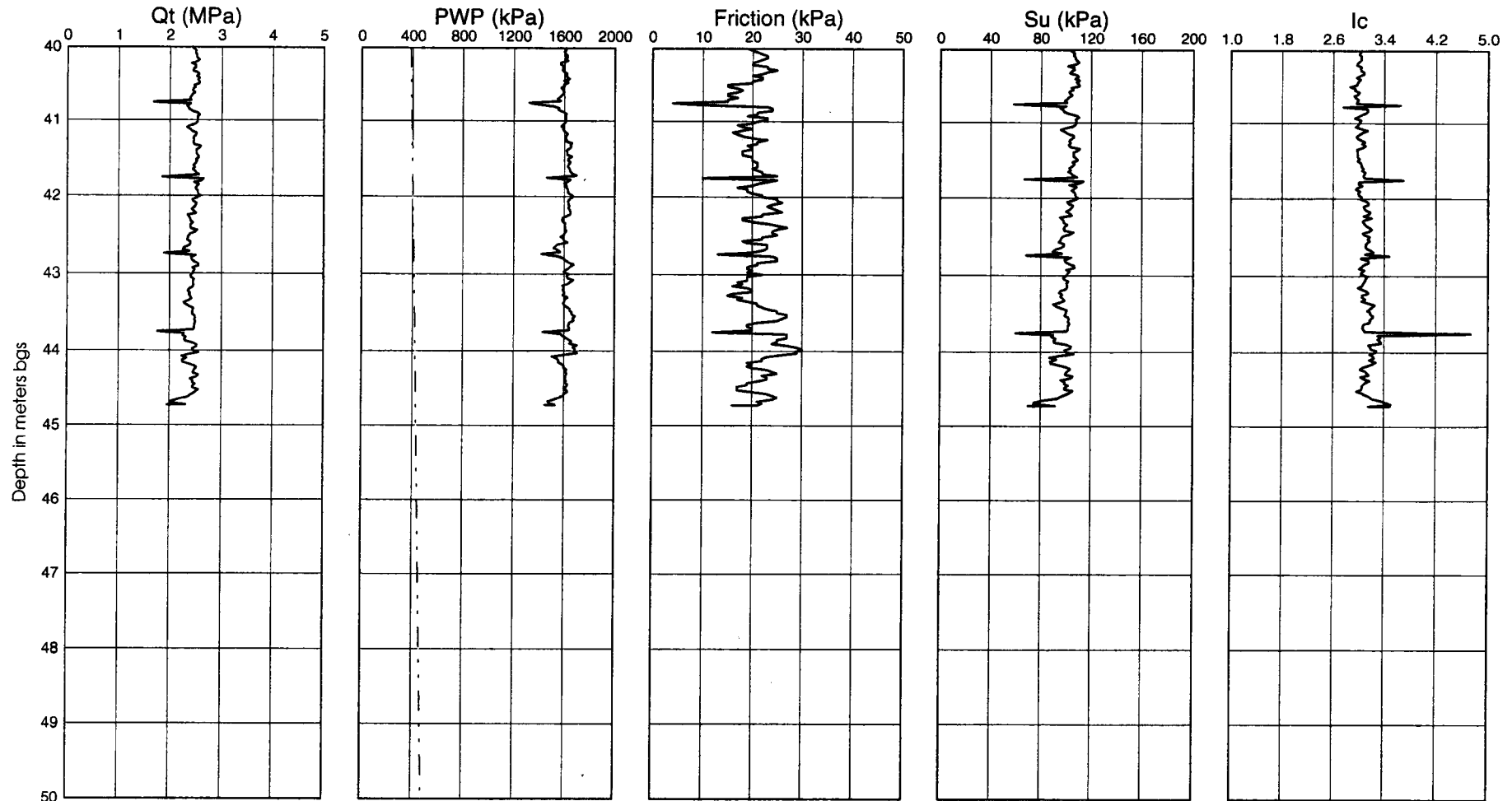
After Jeffries and Davies (1991)  
Ic < 1.25 - Gravelly sands  
1.25 < Ic < 1.90 - Clean to silty sand  
1.90 < Ic < 2.54 - Silty sand to sandy silt  
2.54 < Ic < 2.82 - Clayey silt to silty clay  
2.82 < Ic < 3.22 - Clays

# Cone Penetration Test - 22-11

Test Date : Apr 10, 1900  
Location : see Figure

Operator : Golder Associates

Ground Surf. Elev. : 100.70  
Water Table Depth : 1.00



Qt normalized for  
unequal end area effects

$S_u = (Q_t - \sigma_{vV}) / N_k$   
 $N_k = 17$   
 $\gamma = 17 \text{ kN/m}^3$

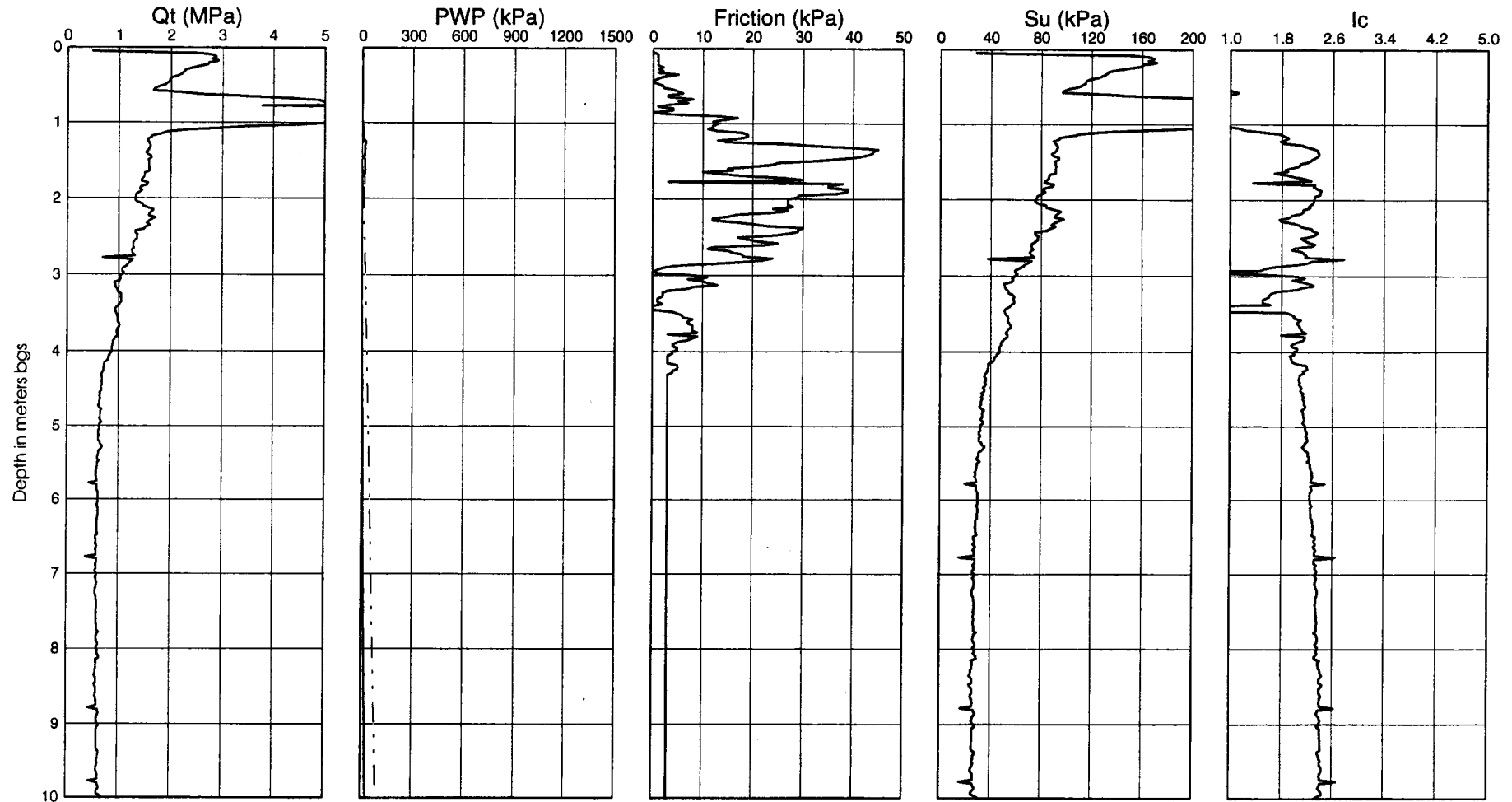
After Jefferies and Davies (1991)  
 $I_c < 1.25$  - Gravelly sands  
 $1.25 < I_c < 1.90$  - Clean to silty sand  
 $1.90 < I_c < 2.54$  - Silty sand to sandy silt  
 $2.54 < I_c < 2.82$  - Clayey silt to silty clay  
 $2.82 < I_c < 3.22$  - Clays

# Cone Penetration Test - 22-13

Test Date : Apr 10, 1900  
Location : see Figure

Operator : Golder Associates

Ground Surf. Elev. : 101.32  
Water Table Depth : 1.00



Qt normalized for  
unequal end area effects

$Su = (Qt - \sigma_v) / Nk$   
 $Nk = 17$   
 $\gamma = 17 \text{ kN/m}^3$

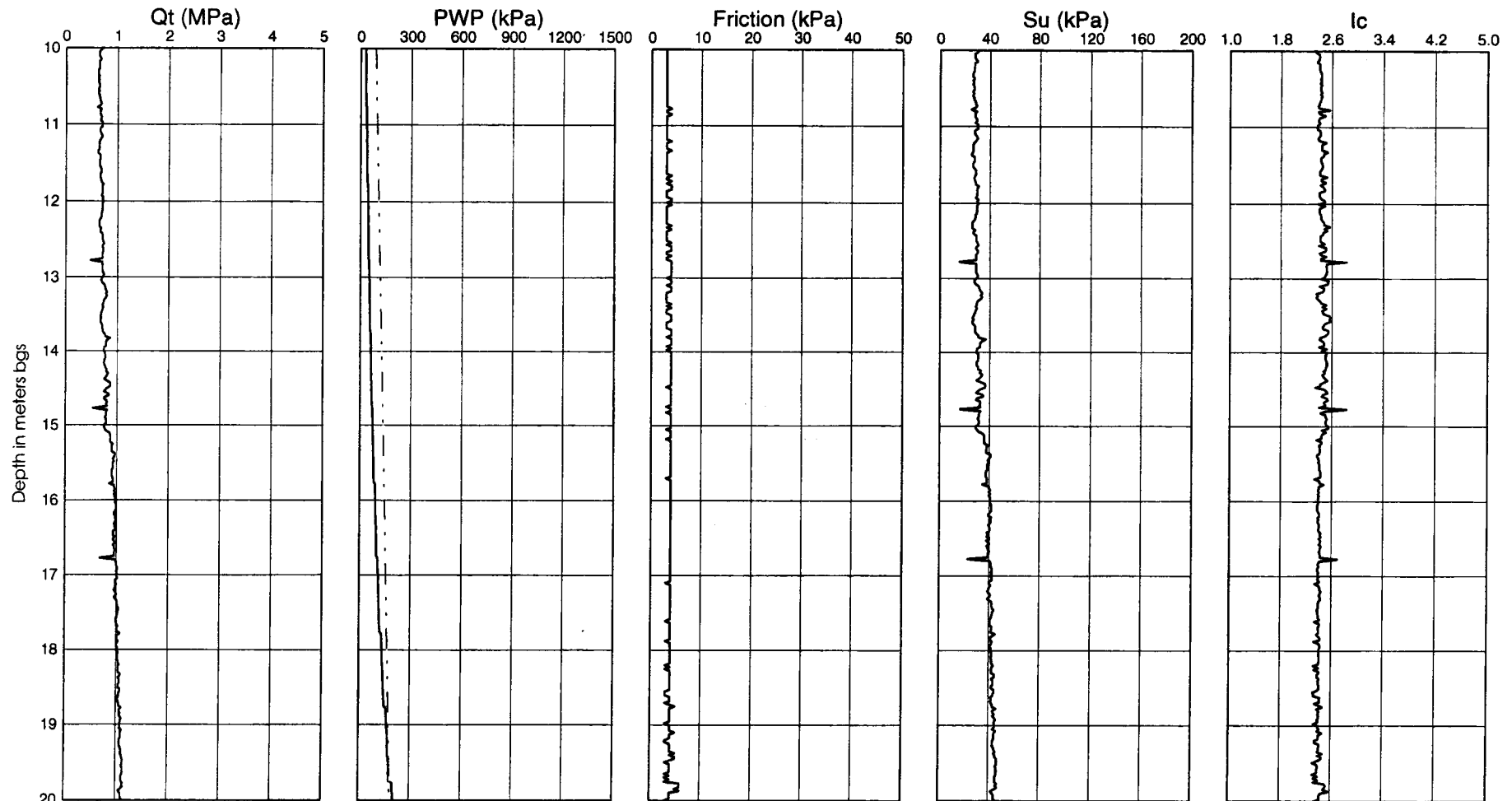
After Jefferies and Davies (1991)  
Ic < 1.25 - Gravelly sands  
1.25 < Ic < 1.90 - Clean to silty sand  
1.90 < Ic < 2.54 - Silty sand to sandy silt  
2.54 < Ic < 2.82 - Clayey silt to silty clay  
2.82 < Ic < 3.22 - Clays

# Cone Penetration Test - 22-13

Test Date : Apr 10, 1900  
Location : see Figure

Operator : Golder Associates

Ground Surf. Elev. : 101.32  
Water Table Depth : 1.00



Qt normalized for  
unequal end area effects

$S_u = (Q_t - \sigma_v) / N_k$   
 $N_k = 17$   
 $\gamma = 17 \text{ kN/m}^3$

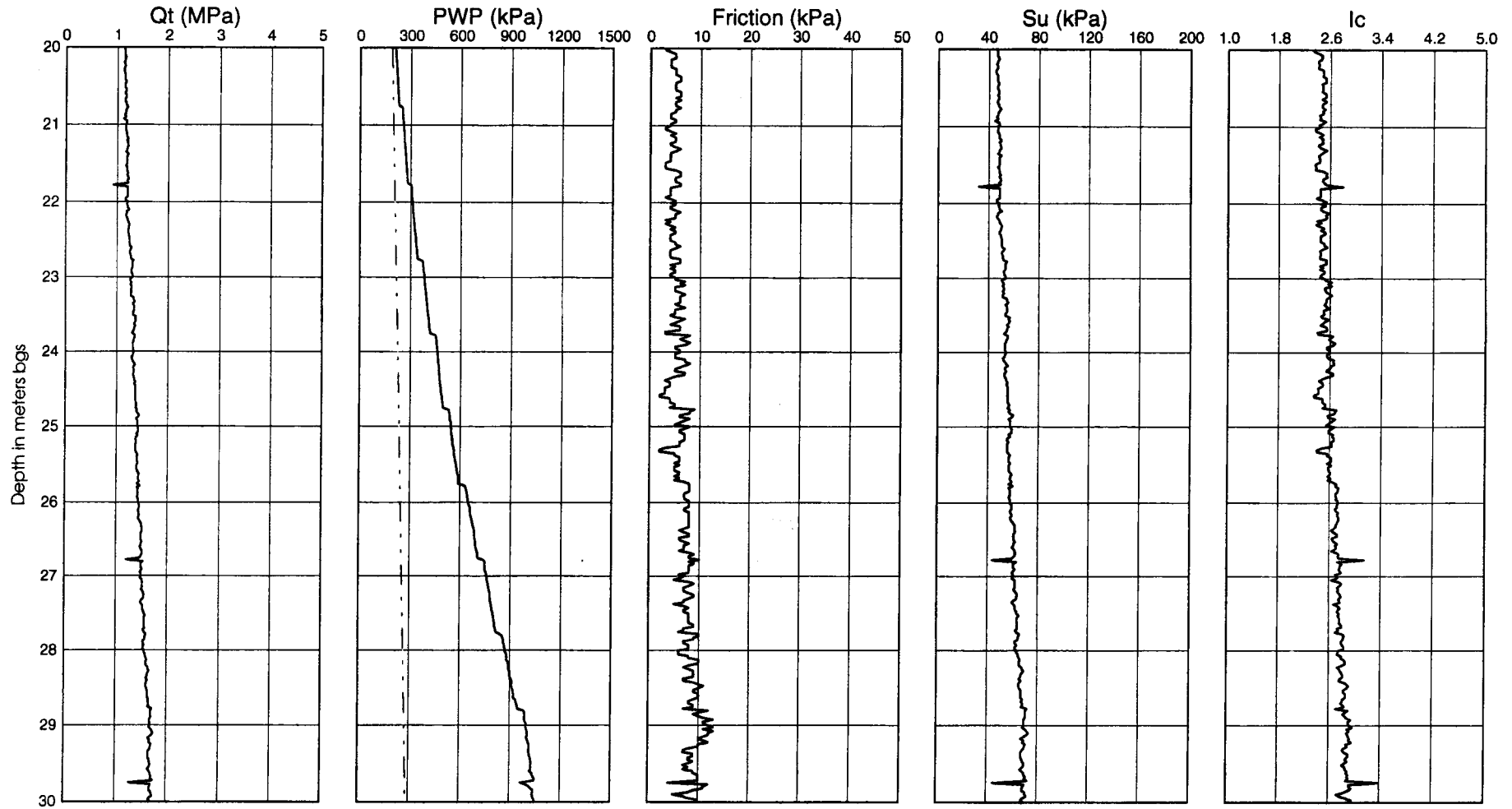
After Jefferies and Davies (1991)  
 $I_c < 1.25$  - Gravely sands  
 $1.25 < I_c < 1.90$  - Clean to silty sand  
 $1.90 < I_c < 2.54$  - Silty sand to sandy silt  
 $2.54 < I_c < 2.82$  - Clayey silt to silty clay  
 $2.82 < I_c < 3.22$  - Clays

# Cone Penetration Test - 22-13

Test Date : Apr 10, 1900  
Location : see Figure

Operator : Golder Associates

Ground Surf. Elev. : 101.32  
Water Table Depth : 1.00



Qt normalized for  
unequal end area effects

$$Su = (Qt - \sigma_v) / Nk$$

Nk = 17  
Gamma = 17 kN/m<sup>3</sup>

After Jefferies and Davies (1991)

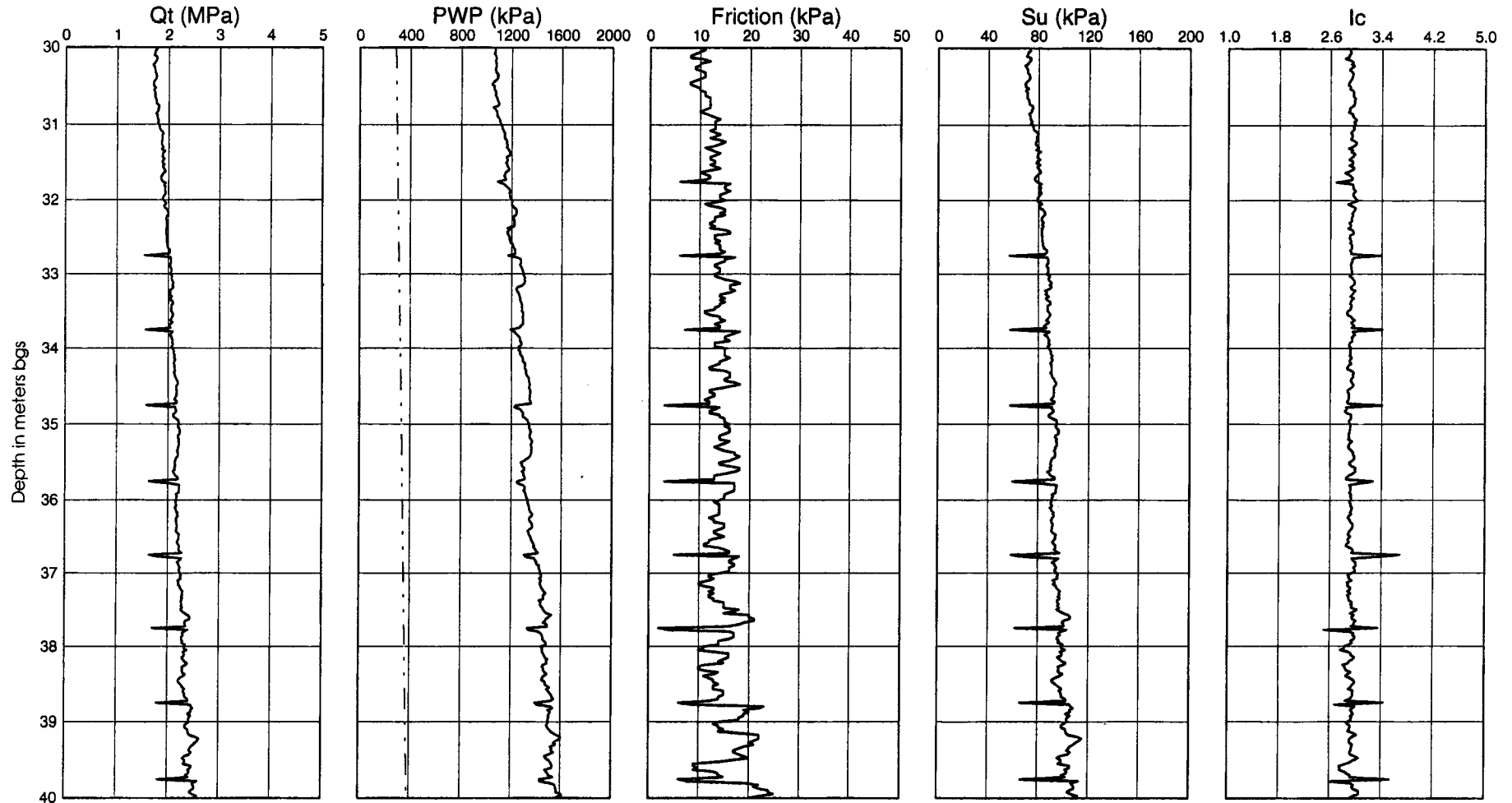
- Ic < 1.25 - Gravelly sands
- 1.25 < Ic < 1.90 - Clean to silty sand
- 1.90 < Ic < 2.54 - Silty sand to sandy silt
- 2.54 < Ic < 2.82 - Clayey silt to silty clay
- 2.82 < Ic < 3.22 - Clays

# Cone Penetration Test - 22-13

Test Date : Apr 10, 1900  
Location : see Figure

Operator : Golder Associates

Ground Surf. Elev. : 101.32  
Water Table Depth : 1.00



Qt normalized for  
unequal end area effects

$S_u = (Q_t - \sigma_{vV}) / N_k$   
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 $2.54 < I_c < 2.82$  - Clayey silt to silty clay  
 $2.82 < I_c < 3.22$  - Clays

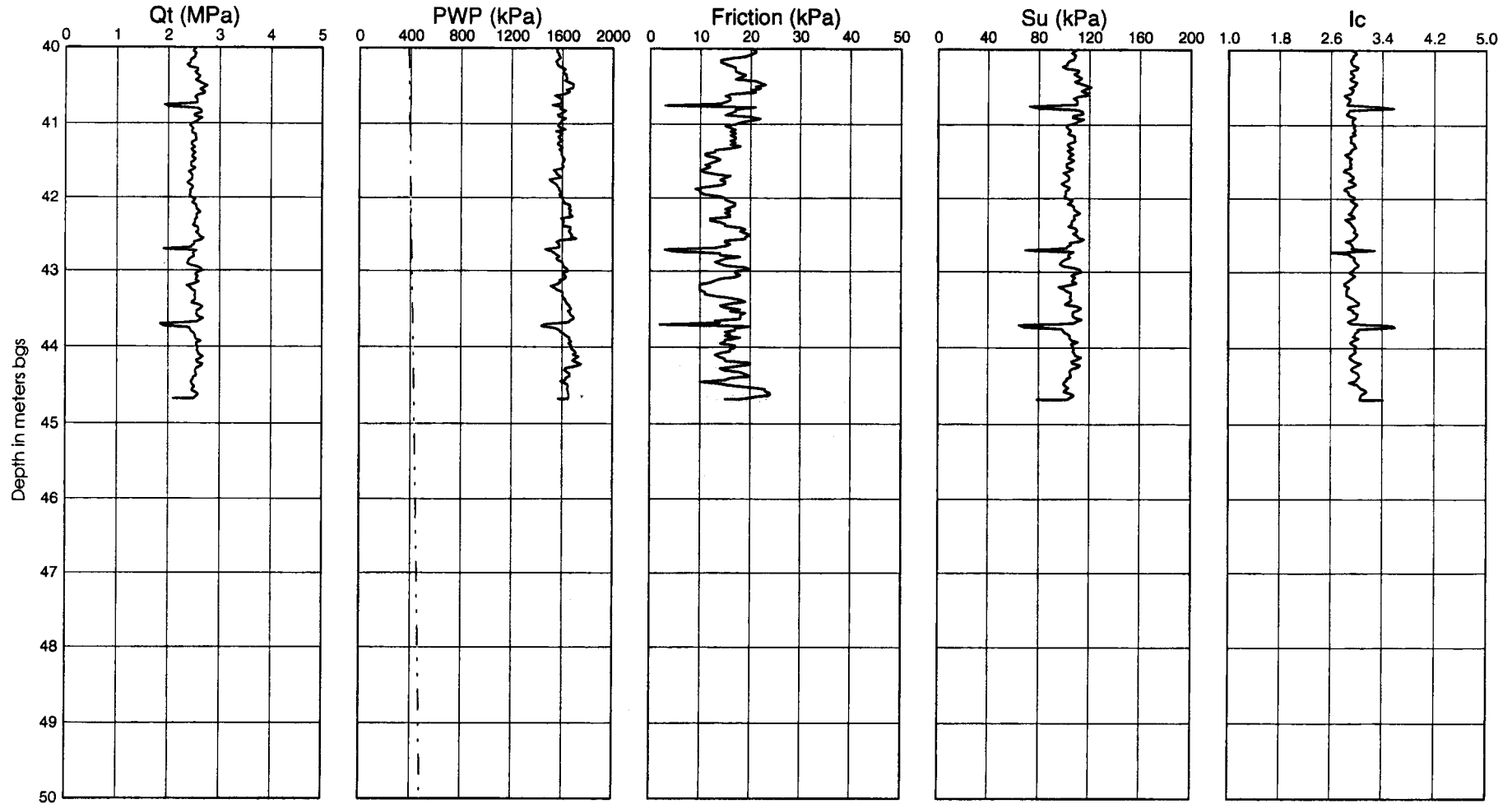


# Cone Penetration Test - 22-13

Test Date : Apr 10, 1900  
Location : see Figure

Operator : Golder Associates

Ground Surf. Elev. : 101.32  
Water Table Depth : 1.00

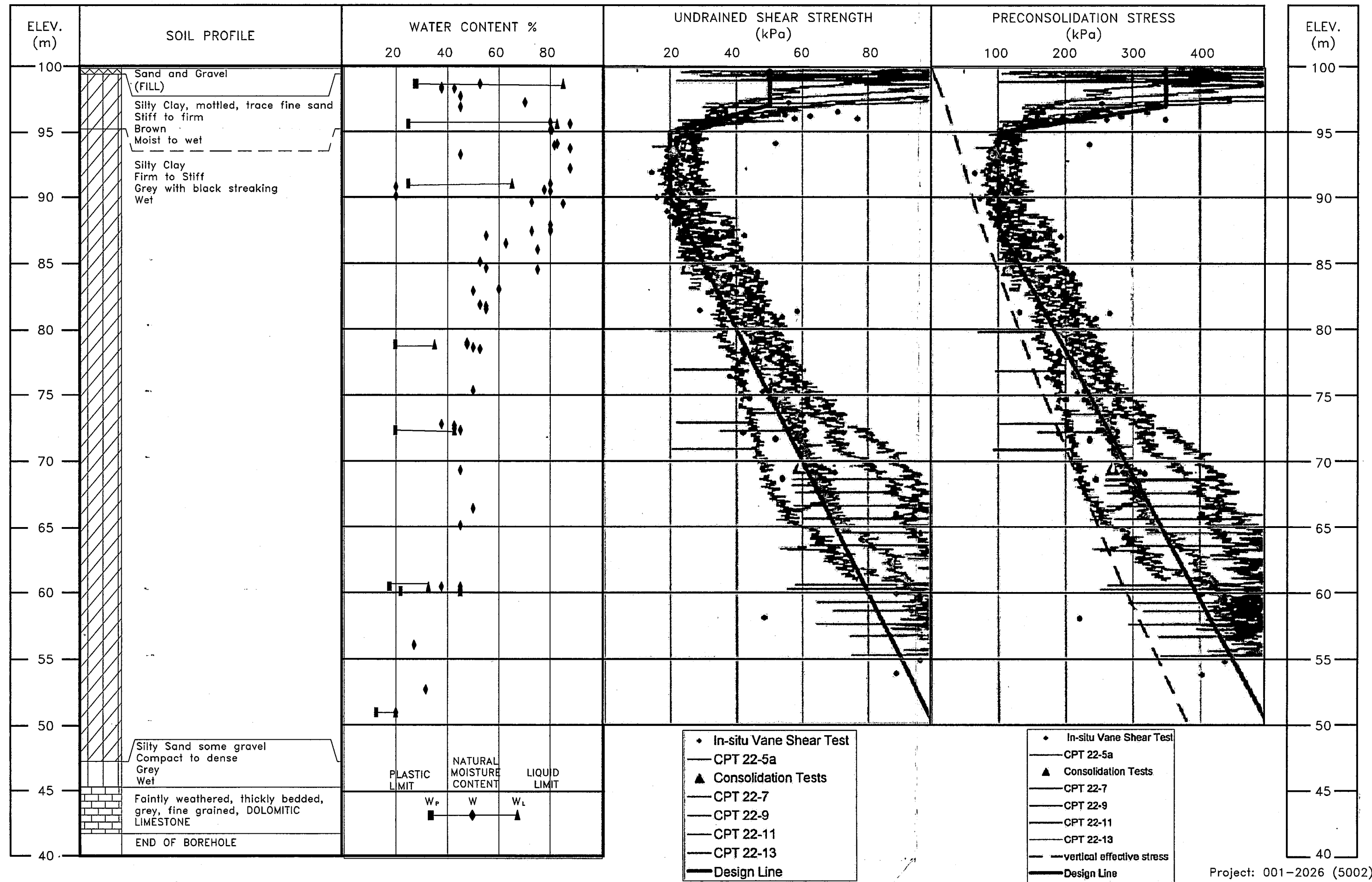


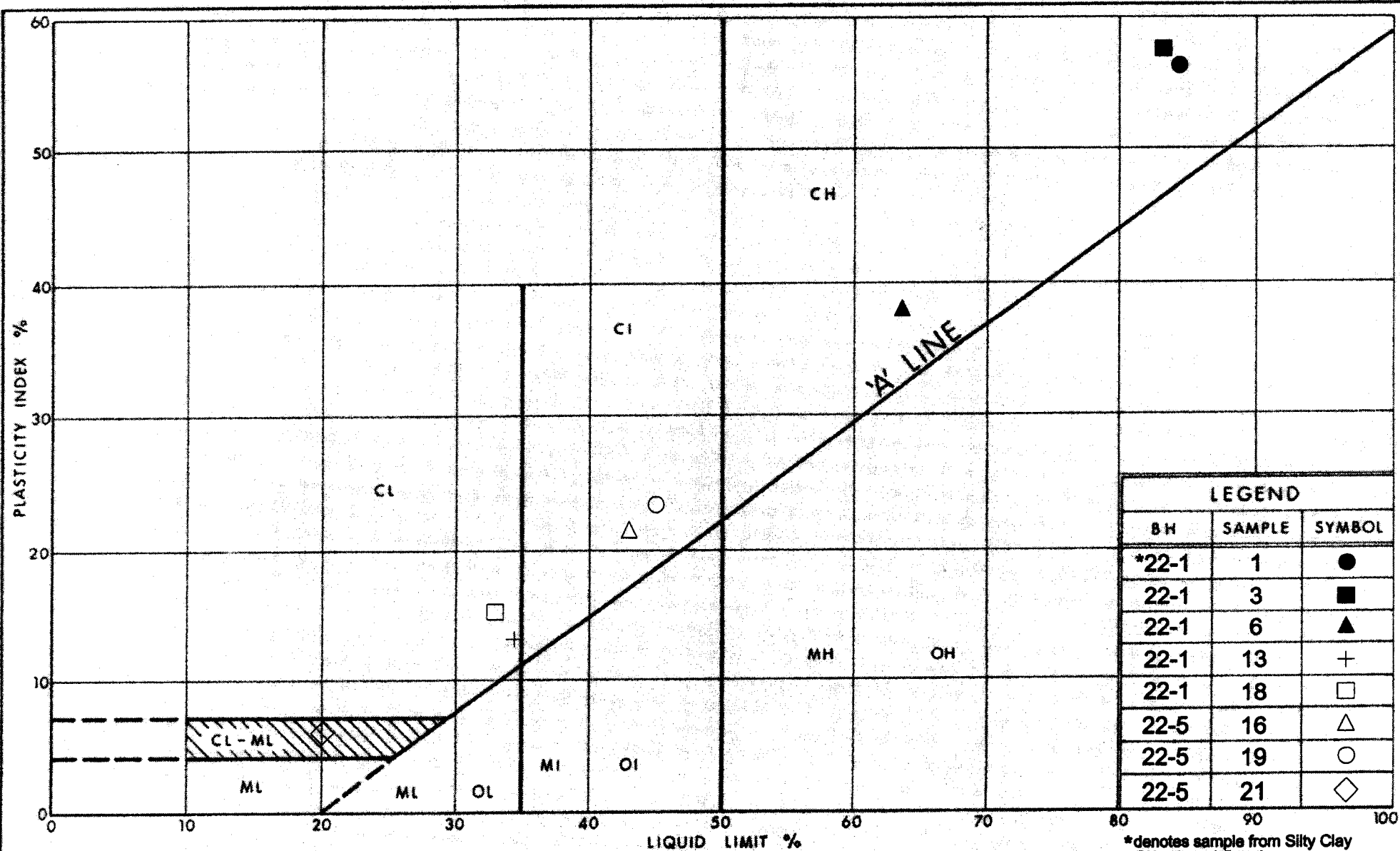
Qt normalized for  
unequal end area effects

$S_u = (Q_t - \sigma_v) / N_k$   
 $N_k = 17$   
 $\gamma = 17 \text{ kN/m}^3$

After Jefferies and Davies (1991)  
 $I_c < 1.25$  - Gravelly sands  
 $1.25 < I_c < 1.90$  - Clean to silty sand  
 $1.90 < I_c < 2.54$  - Silty sand to sandy silt  
 $2.54 < I_c < 2.82$  - Clayey silt to silty clay  
 $2.82 < I_c < 3.22$  - Clays

FIGURE 1. SOIL PROFILE - SUMMARY PLOT OF ENGINEERING PROPERTIES





Ministry of  
Transportation

Ontario

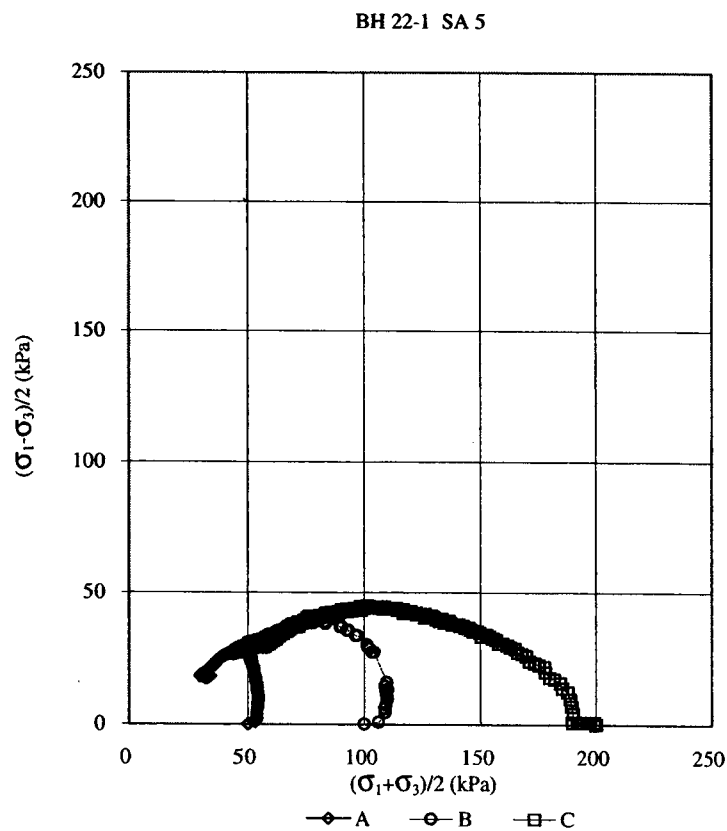
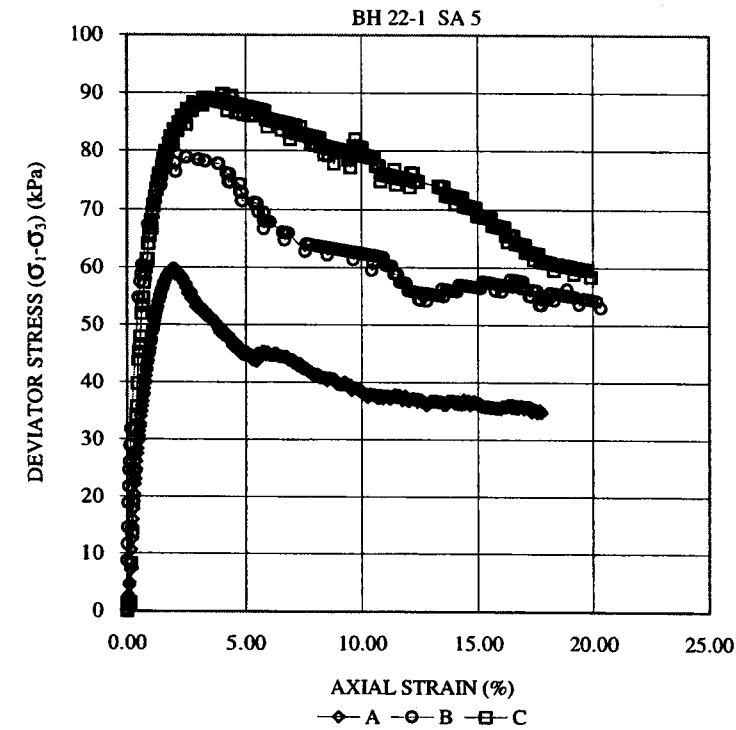
# PLASTICITY CHART SENSITIVE GREY SILTY CLAY

FIG No 2

W P 128-92-00 & 452-90-00

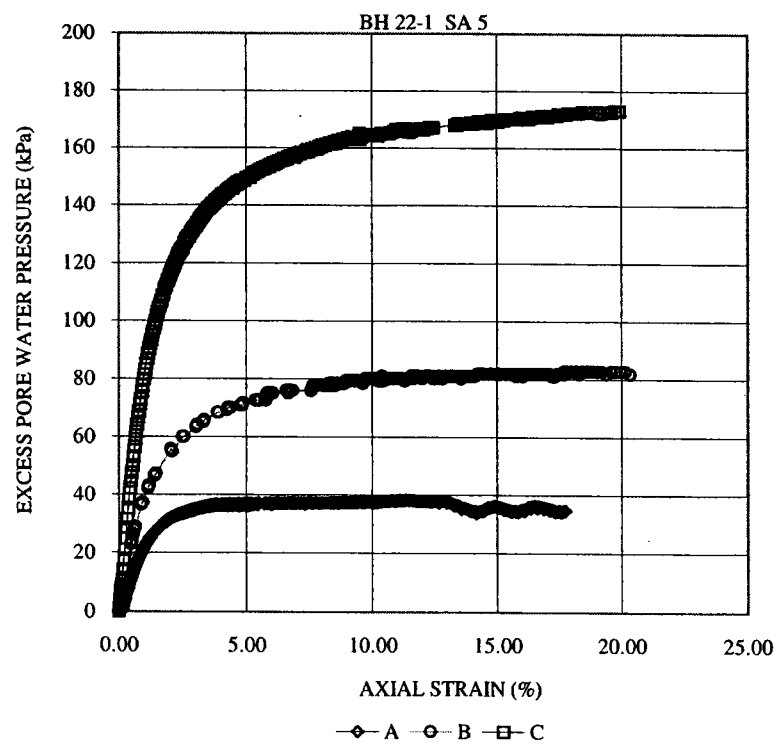
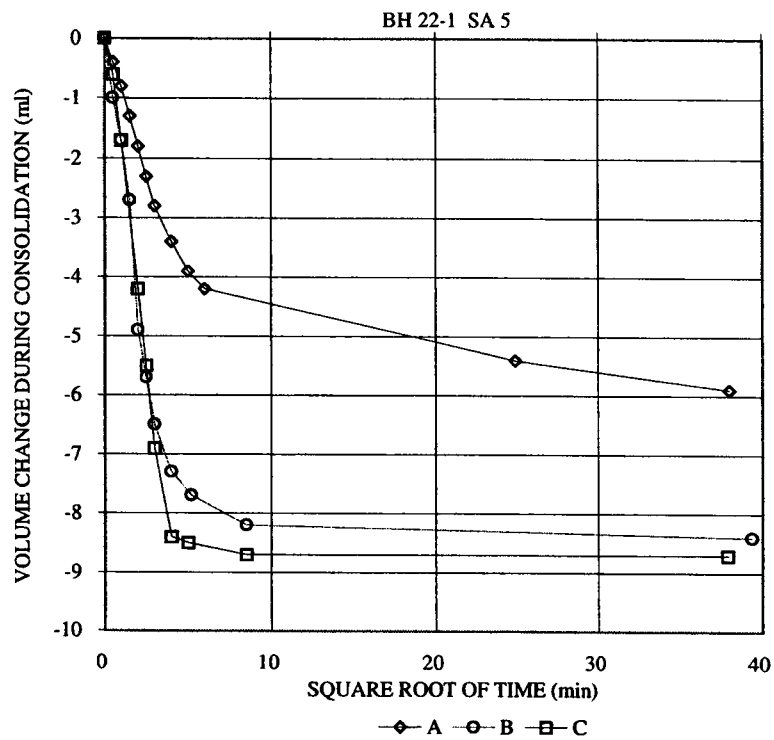
Project No. 001-2026 (5002)

CONSOLIDATED UNDRAINED TRIAXIAL WITH PORE PRESSURE MEASUREMENTS SHEET 1 OF 3		FIGURE 3	
TEST STAGE	A	B	C
BOREHOLE NUMBER	22-1	22-1	22-1
SAMPLE NUMBER	5	5	5
DEPTH, m	7.6-8.1	7.6-8.1	7.6-8.1
SPECIMEN DIAMETER, cm	5.04	5.03	5.05
SPECIMEN HEIGHT, cm	10.15	10.16	10.16
WATER CONTENT BEFORE CONSOLIDATION, %	93.1	91.6	89.9
CELL PRESSURE, $\sigma_3$ , kPa	185.0	235.0	335.0
BACK PRESSURE, kPa	135.0	135.0	135.0
PORE PRESSURE PARAMETER "B"	0.99	0.99	0.99
CONSOLIDATION PRESSURE, $\sigma_c$ , kPa	50.0	100.0	200.0
VOLUMETRIC STRAIN DURING CONSOLIDATION, %	2.9	9.7	4.3
WATER CONTENT AFTER CONSOLIDATION, %	89.3	86.2	84.6
AVERAGE RATE OF STRAIN, %/hr	0.3	0.3	0.3
TIME TO FAILURE, DAYS	2	2	2
WATER CONTENT AFTER TEST, %	89.6	84.7	79.5
MAX. DEVIATOR STRESS, $(\sigma_1 - \sigma_3)$ , kPa	59.8	78.9	89.2
AXIAL STRAIN AT $(\sigma_1 - \sigma_3)$ MAXIMUM, %	1.9	2.5	3.3
MAX EFFECTIVE PRINCIPAL STRESS RATIO, $(\sigma_1 / \sigma_3)$ MAXIMUM	4.5	3.5	2.6
DEVIATOR STRESS AT $(\sigma_1 / \sigma_3)$ MAXIMUM, kPa	57.3	76.1	89.7
AXIAL STRAIN AT $(\sigma_1 / \sigma_3)$ MAXIMUM, %	2.5	4.3	4.1
PORE PRESSURE PARAMETER, Af, AT $(\sigma_1 - \sigma_3)$ MAXIMUM	0.52	0.76	1.52
PORE PRESSURE PARAMETER, Af, AT $(\sigma_1 / \sigma_3)$ MAXIMUM	0.59	0.92	1.60
NATURAL WATER CONTENT, w, %	92.1	89.8	86.9
DRY DENSITY, $\text{mg}/\text{m}^3$	0.77	0.78	0.80
FILTER DRAINS USED, y/n	y	y	y
TEST NOTES:			
CHANGED RATE OF STRAIN, %/hr	0.5	0.5	0.5
AXIAL STRAIN WHERE RATE OF STRAIN WAS CHANGED, %	5.5	4.8	4.5
FAILURE PLANE NUMBER	Bulged	Bulged	1
ANGLE OF FAILURE, DEGREES	-	-	60
DATE:	April, 2000		
PROJECT NUMBER	001-2026		
Golder Associates			

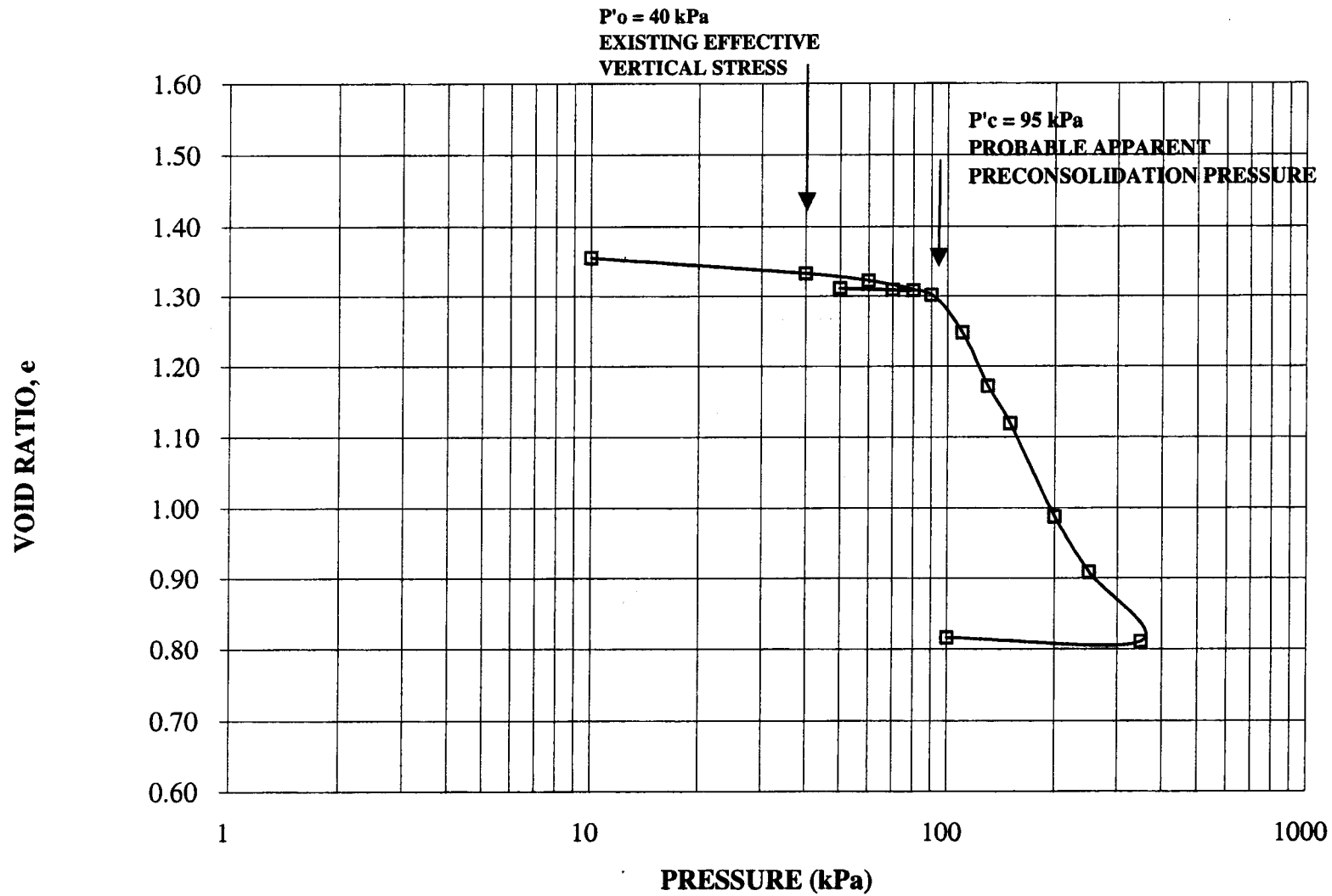


CONSOLIDATED UNDRAINED TRIAXIAL  
WITH PORE PRESSURE MEASUREMENTS  
SHEET 3 OF 3

FIGURE 5



**CONSOLIDATION TEST  
VOID RATIO vs LOG. PRESSURE  
BH 22-10 SA 4 ELEV. 94-94.5m**



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

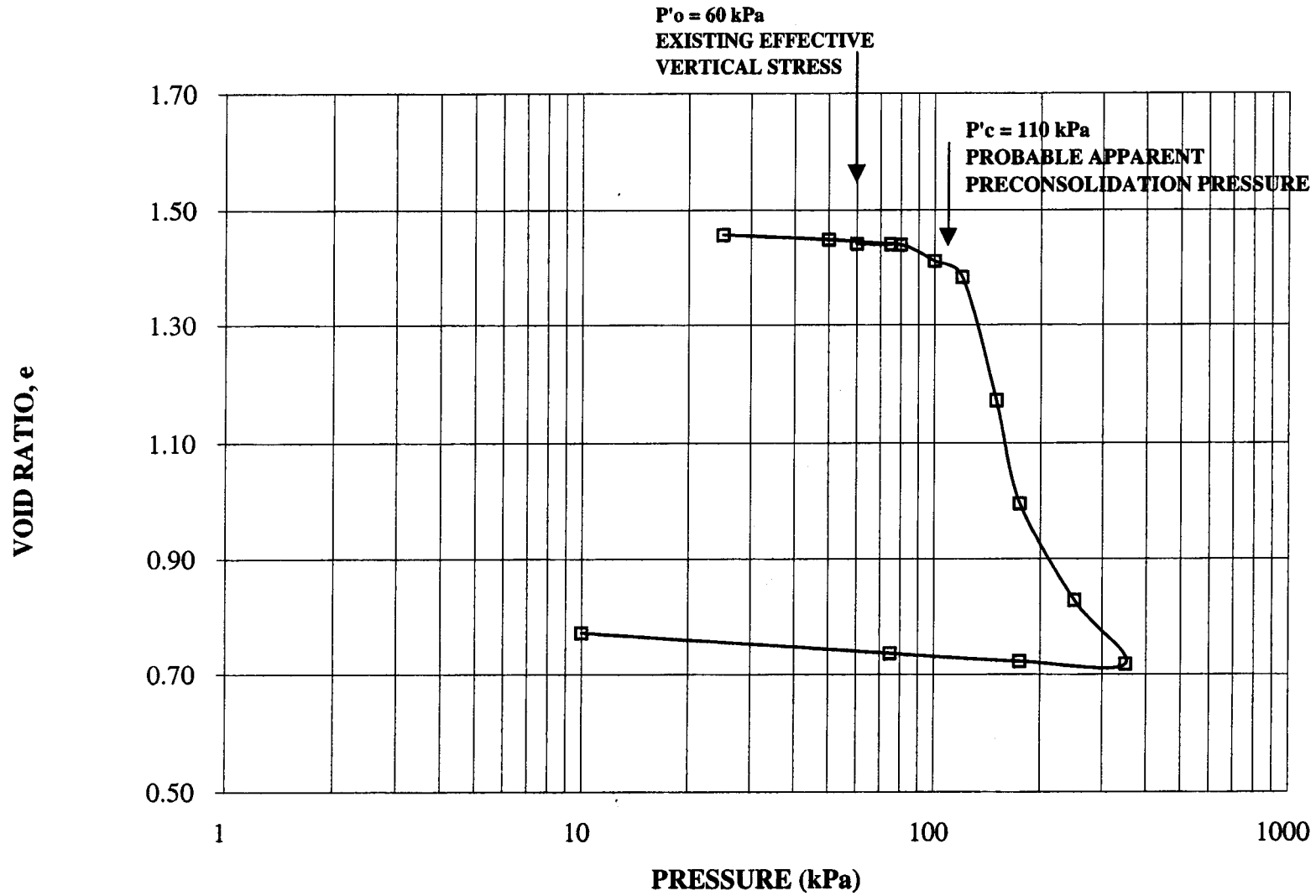
**FIGURE 6**

## CONSOLIDATION TEST VOID RATIO VS. LOG PRESSURE



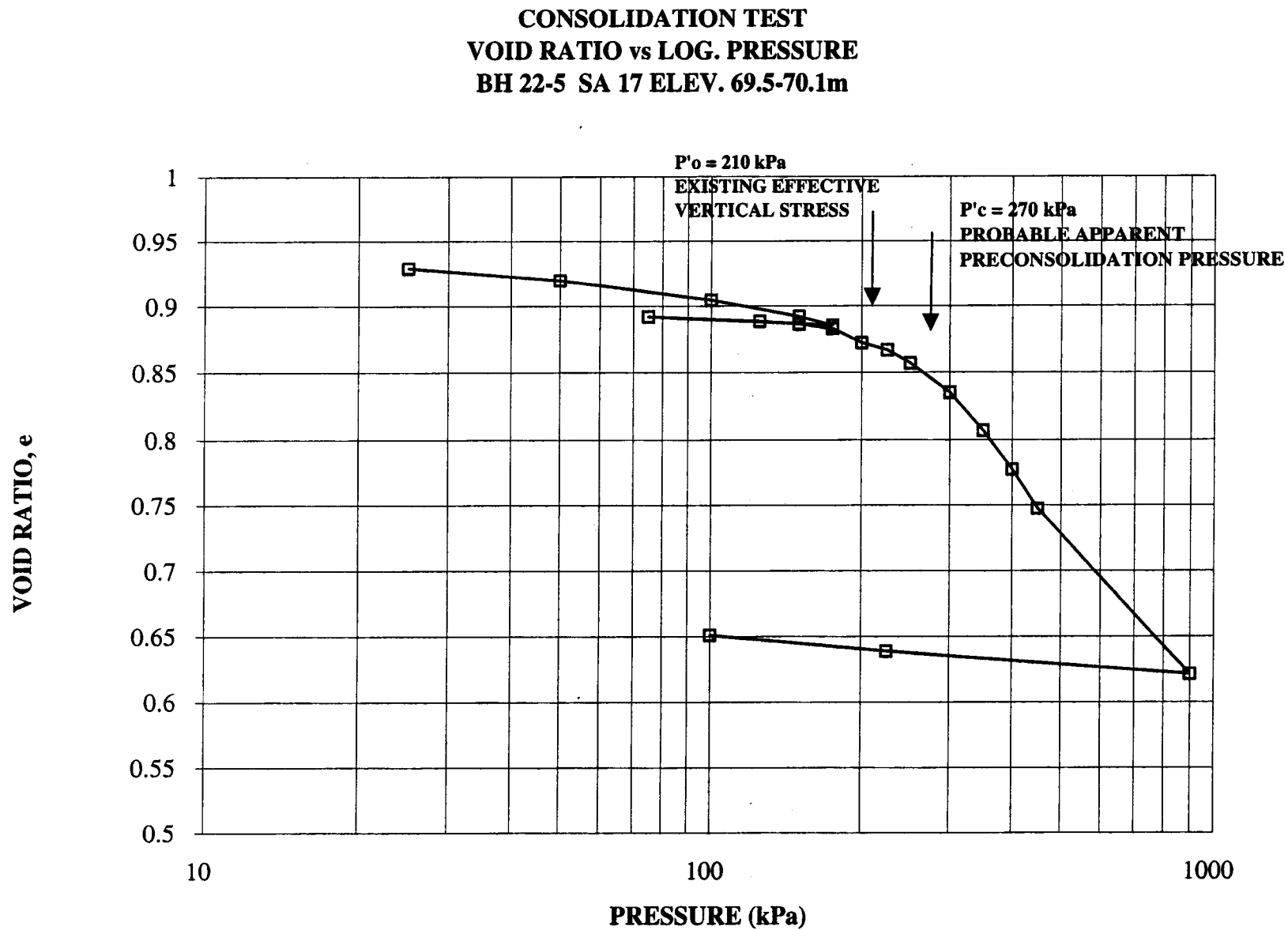


**CONSOLIDATION TEST  
VOID RATIO vs LOG. PRESSURE  
BH 22-6 SA 6 ELEV. 90.6-91.1m**



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

**FIGURE 7**



CONSOLIDATION TEST  
VOID RATIO VS. LOG PRESSURE

FIGURE 9

January 2001

001-2026 (Task 5002)

**APPENDIX A**  
**RECORD OF BOREHOLE SHEETS**  
**FROM THE 1992 INVESTIGATION BY MTO**

# RECORD OF BOREHOLE No 1

1 OF 2 METRIC

W.P. 451-90-08

LOCATION Coordinates N 5 029 102.7, E 321 356.7

DIST 9 HWY 17

BOREHOLE TYPE Mallow Stem Auger

ORIGINATED BY M.M.

DATUM Geodetic

DATE 91 10 18 - 91 10 21

COMPILED BY M.M.

CHECKED BY B.I.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT		UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20 40 60 80 100	20 40 60 80 100	WATER CONTENT (%)	WATER CONTENT (%)		
100.1	Ground Surface												
0.0													
			1	SS	12								
			2	SS	8								
	Firm to Stiff		3	SS	3								
			4	TM	PM								
	Brown Grey		5	SS	1								
			6	TM	PM								
			7	SS	0								
			8	TM	PM								
	Clay to Silty Clay		9	SS	0								
			10	TM	PM								
	Soft to Firm		11	SS	0								
			12	TM	PM								
			13	SS	0								
			14	TM	PM								
	Cloey Silt		15	SS	0								
	Firm to Stiff		16	TM	PM								
69.6													
30.5													

Continued

+3, x3, Numbers refer to Sensitivity

20 15 10 (x) STRAIN AT FAILURE

Continued



# RECORD OF BOREHOLE No 2

1 OF 2 METRIC

W.P. 451-80-08

LOCATION Coords: N 5 029 048.4, E 321 299.6

ORIGINATED BY M.M.

DIST 9

HWY 17

BOREHOLE TYPE Hollow Stem Auger

COMPILED BY M.M.

DATUM Coadette

DATE 91 10 18 - 91 10 21

CHECKED BY B.J.

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			20 40 60 80 100	20 40 60 80 100					
98.5	Ground Surface												
0.0													
	Stiff		1	SS	15							16.9	0 1 32 67
			2	SS	7								
			3	SS	5								
	Brown Grey		4	TW	PM								
			5	SS	1								
			6	TW	PM								
			7	SS	0								
	Clay to Silty Clay		8	TW	PM							14.5	
			9	SS	0							14.2	0 0 32 68
	Soft to Firm		10	TW	PM							14.8	
			11	SS	0								
			12	TW	PM							16.3	
	Clayey Silt		13	SS	0								0 1 37 62
	Firm to Stiff		14	TW	PM								
			15	SS	0								
			16	TW	PM								
69.0													
30.5													

Continued

Numbers refer to Sensitivity

20 15-25 (E) STRAIN AT FAILURE 10

Continued

## RECORD OF BOREHOLE No 2

2 OF 2

METRIC

W.P. 451-80-06

LOCATION

Coords: N S 020 0+8.4, E J21 299.6

ORIGINATED BY M.W.

DIST 9

HWY 17

BOREHOLE TYPE

Mellow Stem Auger

COMPILED BY M.W.

DATUM Ceasatic

DATE

91 10 18 - 91 10 21

CHECKED BY B.I.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT 7 kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40					
30.5	Continued		17	SS	0									
	Clay to Silty Clay		18	TM	PM									
62.5			19	SS	0									
37.0		End of Borehole												
50.4														
49.1	End of Cone Test - Probable Bedrock													

LEGEND

- EARTH PRELOAD FOR FALSEWORK  
AND FRONT OF ABUTMENTS
- GRANULAR 'B' TYPE II SURCHARGE
- EMBANKMENT CONSTRUCTION TO SUBGRADE

REFERENCE:

PROFILE BY McCORMICK RANKIN CORPORATION  
REALIGNED REGIONAL ROAD 22 - EMBANKMENT STAGE 1 PROFILE  
DRAWING No. 4209\_RR22EMBSTG1.DWG DATED 00/11/23

SCALE

HORIZONTAL 1 : 2000  
VERTICAL 1 : 200

SPECIAL NOTE

THIS DRAWING IS TO BE READ IN CONJUNCTION  
WITH ACCOMPANYING REPORT

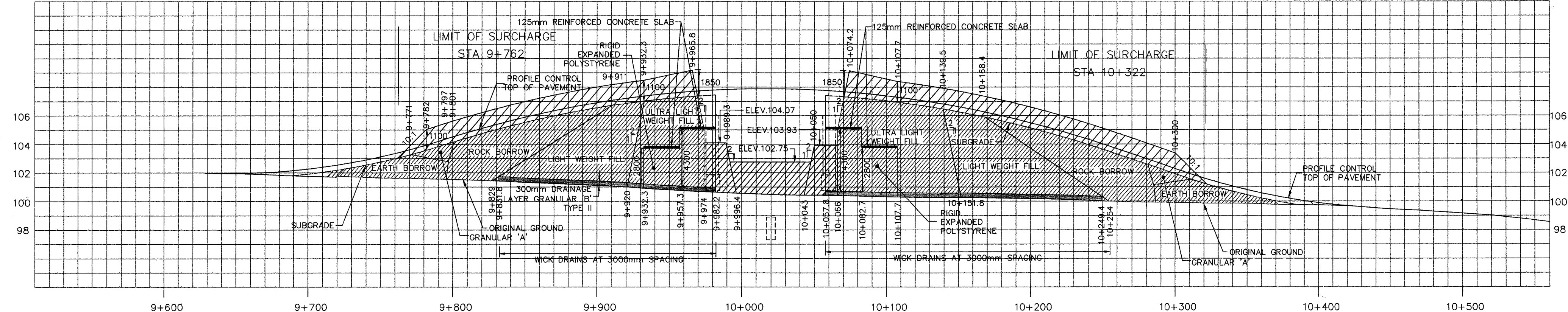
Date: Jan. 2001

Project: 001-2026-5002



Drawn: S.L.

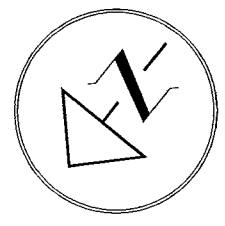
Chkd: A.J.H.



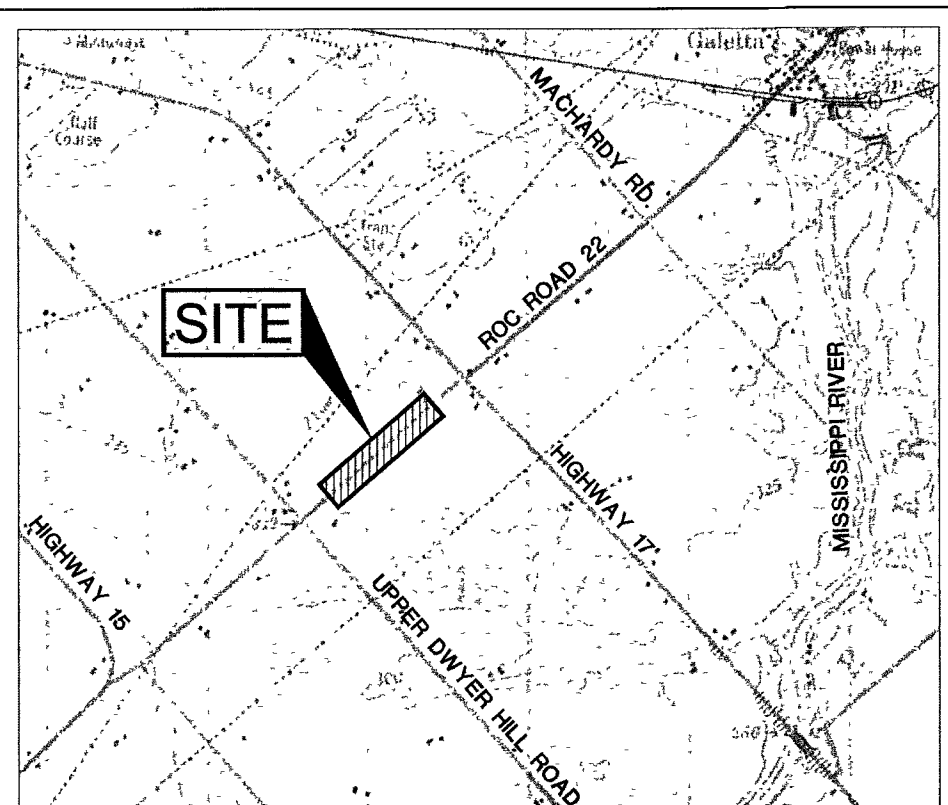
REALIGNED REGIONAL ROAD 22 – EMBANKMENT STAGE 1  
PROFILE

D:\file\00\001-2026\Task5002\acad\JAN2001\figure\_10.dwg plot at 1:2 (metric)





Golder Associates Ltd.  
OTTAWA, ONTARIO, CANADA



KEY PLAN

LEGEND

- Borehole - Current Golder Associates Ltd. Investigation
- Piezo-Cone Penetration Test - Current Golder Associates Ltd. Investigation
- Borehole - Previous MTO Investigation Geocres. No. 31F-115
- Seal
- Piezometer
- N Blows/0.3m (Std. Pen. Test, 475 j/blow)
- 100% Rock Quality Designation (RQD)
- WL in piezometer on May 28, 2000
- WL in piezometer in Oct. 1991

No.	ELEVATION	LOCATION	
		NORTHING	EASTING
22-1	100.46	5029116.715	321369.743
22-2	100.42	5029101.989	321334.891
22-3	100.22	5029071.942	321321.193
22-4	100.10	5029057.169	321287.179
22-5	99.99	5029023.839	321270.079
22-5A	99.92	5029025.802	321272.459
22-6	100.07	5029020.896	321248.858
22-7	99.97	5028990.897	321235.86
22-8	99.94	5028966.235	321190.965
22-9	99.64	5028922.922	321163.395
22-10	100.58	5029141.593	321376.318
22-11	100.70	5029152.322	321407.334
22-12	101.11	5029195.634	321434.905
22-13	101.32	5029230.803	321471.186
1 (MTO)	100.10	5029102.7	321356.7
2 (MTO)	99.50	5029048.4	321299.5

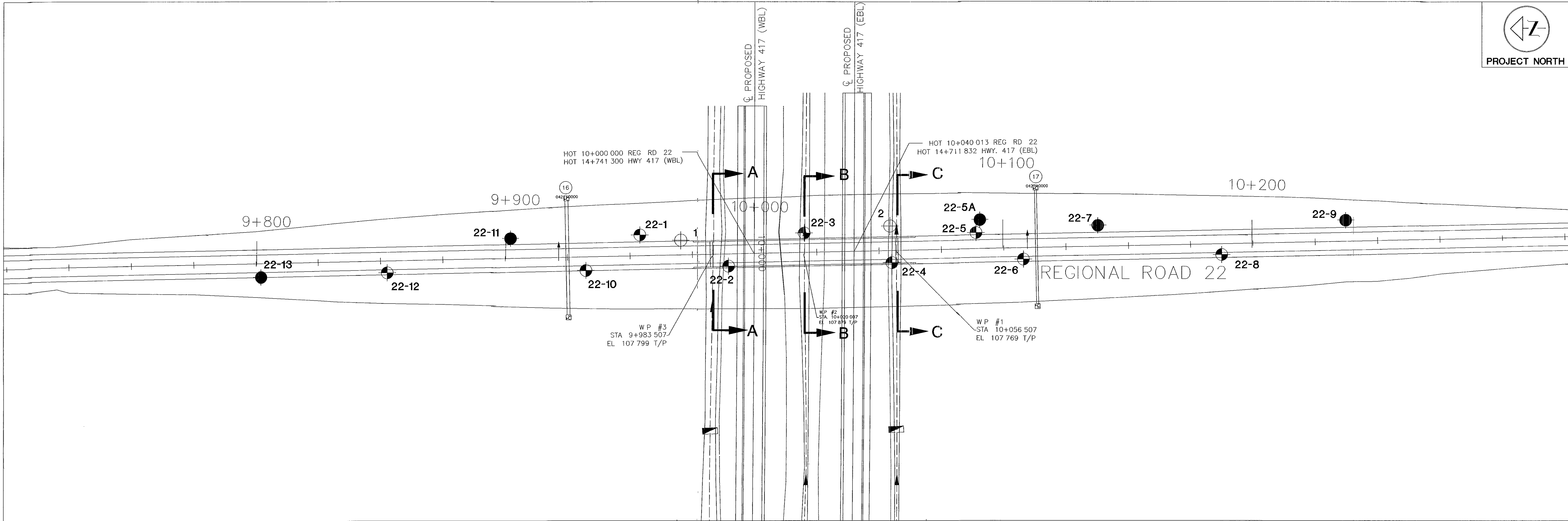
NOTES

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

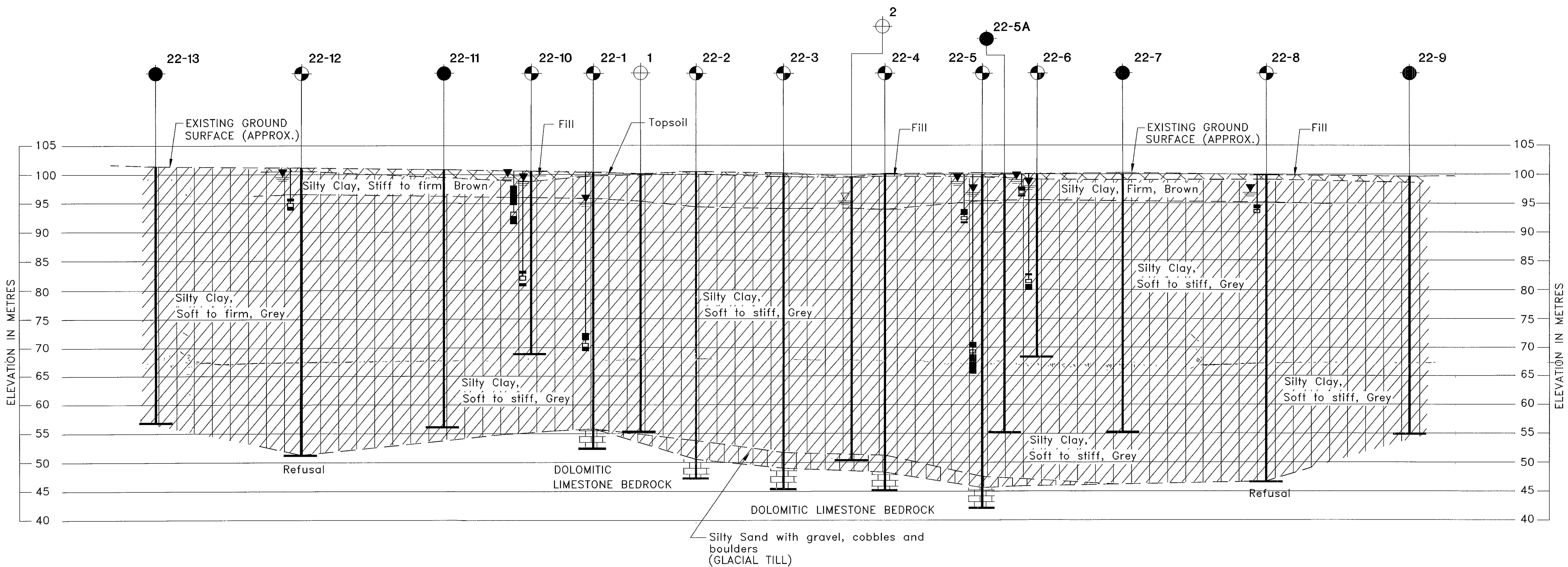
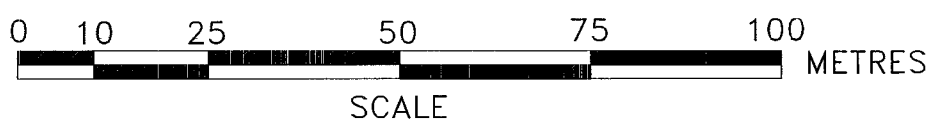
REFERENCE

This drawing was created from digital file provided by McCormick Rankin Corp.

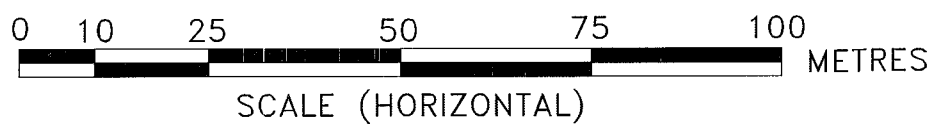
1	00/11/20	FH	REVISED BRIDGE DESIGN
NO.	DATE	BY	REVISION
Geocres No.			
HWY. No.	417	PROJECT NO.:	001-2026 (5002)
SUBM'D.	CHKD: FH	DATE:	JAN. 2001
DRAWN: SGL	CHKD: GSW	APPD.	DWG. 2



PLAN



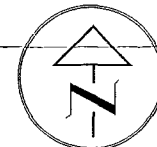
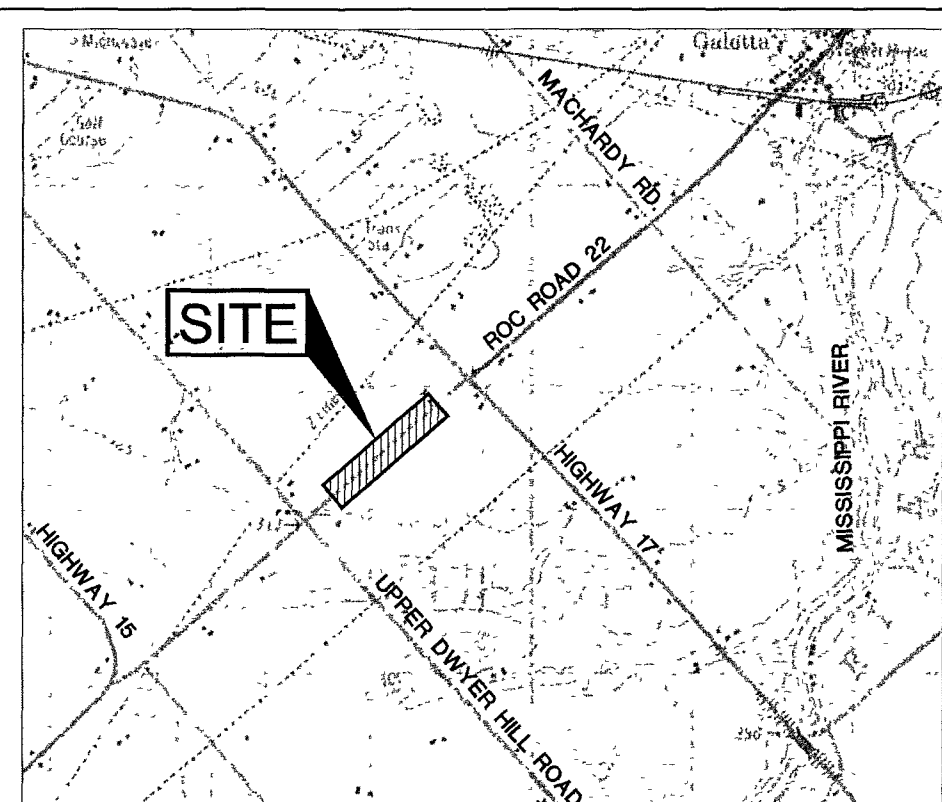
PROFILE ALONG ROC ROAD 22



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



Golder Associates Ltd.  
OTTAWA, ONTARIO, CANADA



KEY PLAN

LEGEND

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- Borehole - Previous MTO Investigation Geocres No. 31F-115
- Seal
- Piezometer
- N Blows/0.3m (Std. Pen. Test, 475 j/blow)
- Cone Blows/0.3m (60° Cone, 475 j/blow)
- 100% Rock Quality Designation (RQD)
- WL in piezometer on May 28, 2000
- WL in piezometer in Oct. 1991

No.	ELEVATION	LOCATION	
		NORTHING	EASTING
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22-4	100.10	5029057.169	321287.179
22-5	99.99	5029023.839	321270.079
22-5A	99.92	5029025.802	321272.459
22-6	100.07	5029020.896	321248.858
22-7	99.97	5028990.897	321235.86
22-8	99.94	5028966.235	321190.965
22-9	99.64	5028922.922	321163.395
22-10	100.58	5029141.593	321376.318
22-11	100.70	5029152.322	321407.334
22-12	101.11	5029195.634	321434.905
22-13	101.32	5029230.803	321471.186
1 (MTO)	100.10	5029102.7	321356.7
2 (MTO)	99.50	5029048.4	321299.5

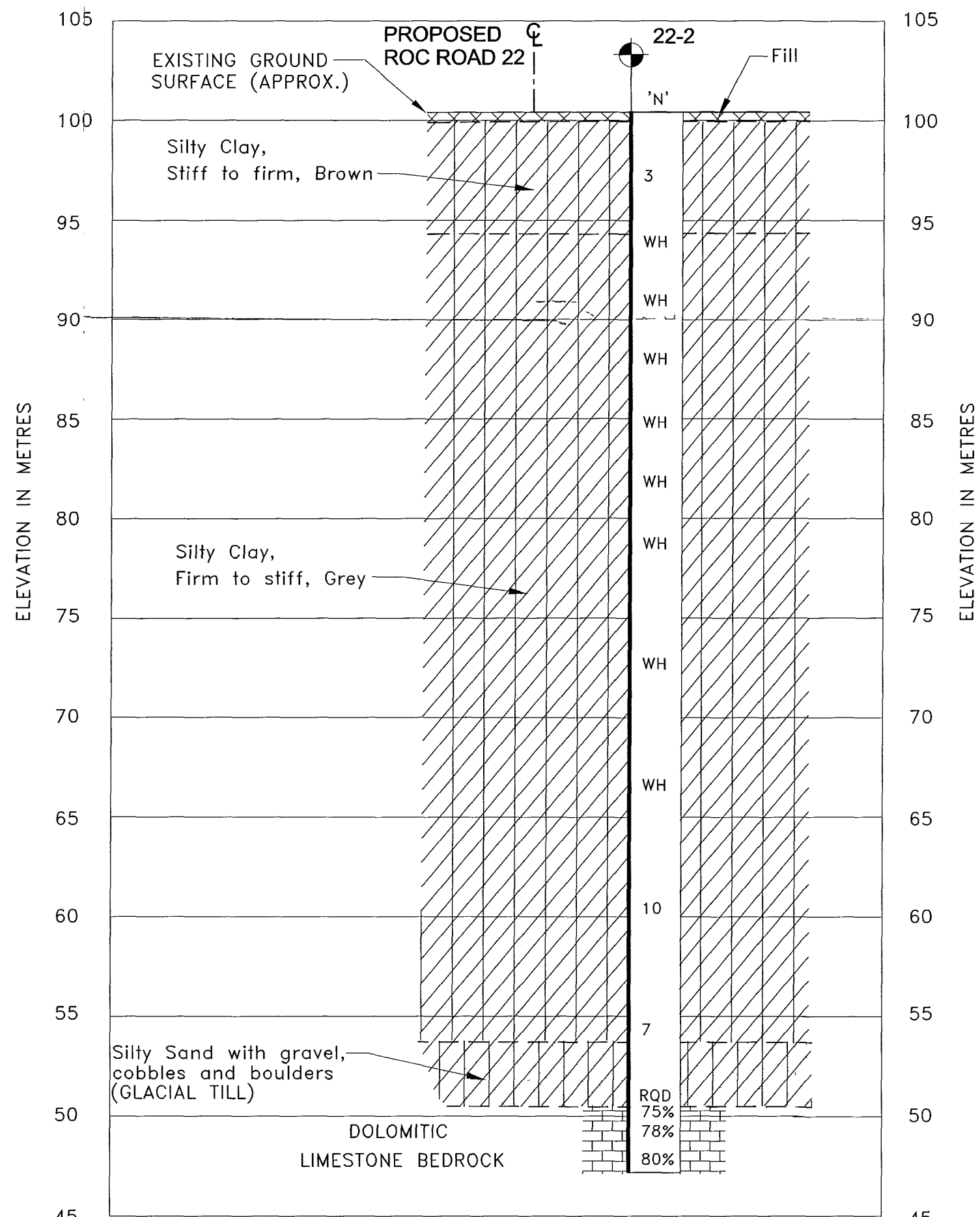
NOTES  
The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.

REFERENCE  
This drawing was created from digital file provided by McCormick Rankin Corp.

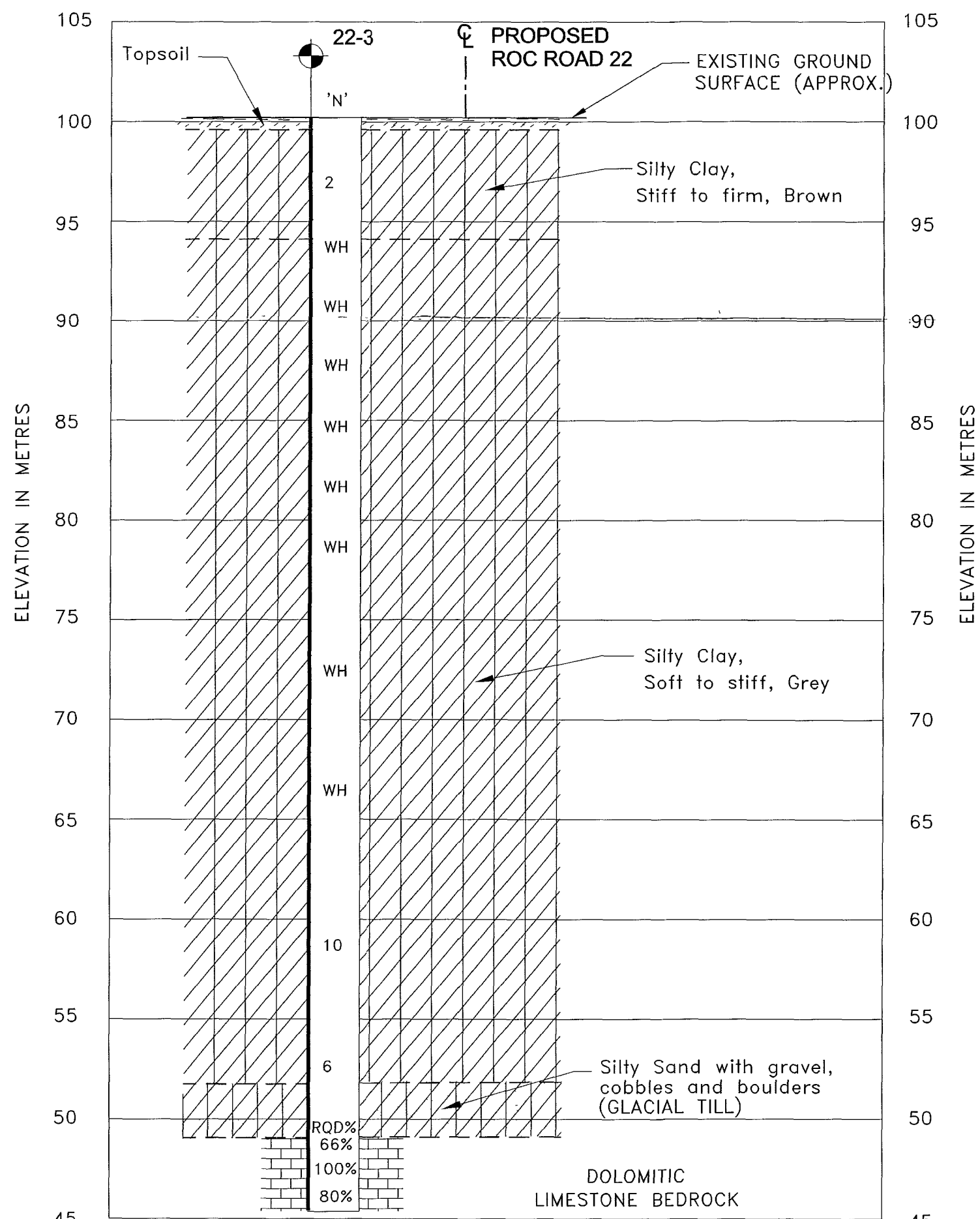
			FINAL
NO.	DATE	BY	REVISION

Geocres No.

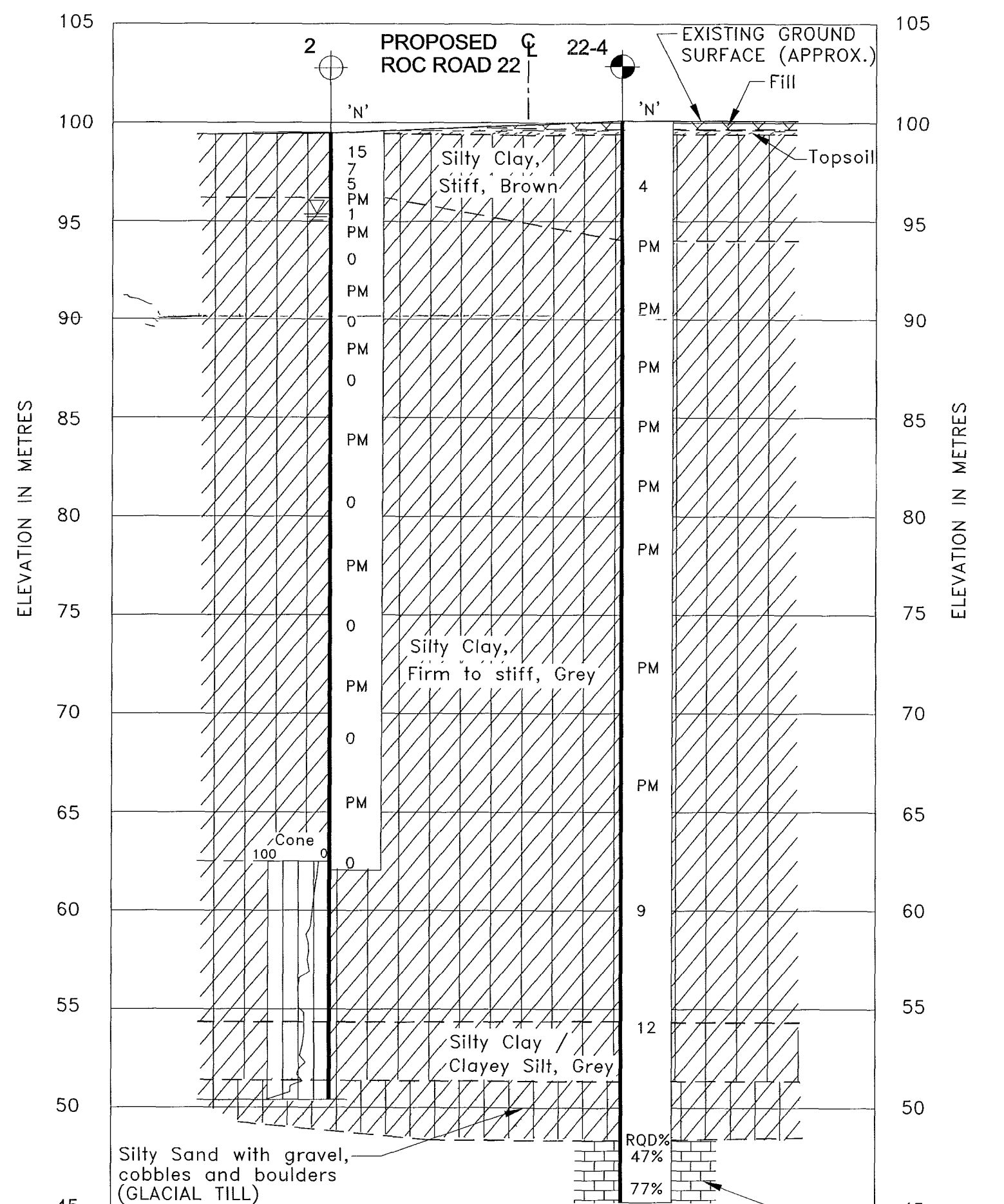
HWY. No.	417	PROJECT NO.:	001-2026 (5002)	DIST.	42
SUBM'D.		CHKD:	FH	DATE:	JAN. 2001
DRAWN:	SGL	CHKD:	GSW	APPD.	DWG. 3



SECTION A-A  
PROPOSED NORTH ABUTMENT



SECTION B-B  
PROPOSED CENTRE PIER



SECTION C-C  
PROPOSED SOUTH ABUTMENT



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