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REMARKS:

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REPORT ON

GEOTECHNICAL INVESTIGATION
PROPOSED HIGHWAY 17 (NEW)
ROOT RIVER TO LOWER ECHO RIVER
DISTRICT 18, SAULT STE. MARIE, ONTARIO

Submitted to:

McCormick Rankin Corporation
2655 North Sheridan Way
Mississauga, Ontario
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June 23,1997 971-1103

McCormick Rankin Corporation
2655 North Sheridan Way
Mississauga, Ontario
L5K 2P8

Attention: Mr. Gord Firth, P.Eng.

**RE: GEOTECHNICAL INVESTIGATION
PROPOSED HIGHWAY 17 (NEW)
ROOT RIVER TO LOWER ECHO RIVER
DISTRICT 18, SAULT STE. MARIE**

Dear Sirs:

Please find enclosed 6 copies of our report providing recommendations for the geotechnical aspects of the final design phase of the proposed realignment of the section of the Highway 17 within the Garden River First Nation Property. Specifically, recommendations are provided for the design of the bridge foundations and associated works at each of the six bridge sites.

We trust that the contents of this report are satisfactory for your design requirements. Should you have any questions regarding the information provided herein, or require further information, please contact the undersigned.

Yours truly,

GOLDER ASSOCIATES LTD.

ORIGINAL SIGNED BY

A.S. Poschmann, P. Eng.
Principal

AP/ASP/pds

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1.0 INTRODUCTION

Golder Associates Ltd. has been retained by McCormick Rankin Corporation to carry out a geotechnical investigation for the final design of the bridges and associated works for the planned realignment of Highway 17, hereafter referred to as Highway 17 (New), within the Garden River First Nation property, to the east of the City of Sault Ste. Marie. Golder Associates carried out the geotechnical investigation for the preliminary design of the bridges, embankments and roadworks in areas of wetlands. The results of the preliminary investigation and recommendations for design were documented in our Report No. 941-1364, dated June 1996.

The purpose of this investigation was to determine the subsurface conditions at each of the proposed bridge structure sites by means of a limited number of boreholes, in-situ tests and laboratory tests on selected samples. Based on our interpretation of the data obtained, recommendations on the geotechnical aspects of design of the proposed works are provided. Comments are also provided on anticipated construction problems where they may affect the design of the proposed bridges and approach embankments.

The terms of reference for the initial scope of work are outlined in our proposal letter P71-1007, dated January 13, 1997. During the course of the field work, the number of boreholes and extent of testing was revised slightly to accommodate the subsoil and site conditions as encountered.

2.0 SITE AND PROJECT DESCRIPTION

The works described in this report are associated with the 17 km long section of the realignment of Highway 17 west Sault Ste. Marie which crosses through the Garden River First Nation Reserve. Specifically, this section extends from the east side of Root River to the south side of Lower Echo River.

It is our understanding that the proposed works include a four lane divided highway, four creek crossings and two highway overpasses, namely:

- Root River Crossing - one span twin structure,
- Belleau Creek Crossing - one span twin structure,
- Jardin Mines Road Underpass – one span single structure,
- Garden River Crossing - three span twin structure,
- Noonday Road Underpass – one span single structure, and
- Lower Echo River Crossing - one span twin structure.

In general, cuts and embankments at the bridge sites are less than 3 m except in the area of the Garden River crossing where the proposed road grade requires an embankment height of up to 11.5 m.

The proposed horizontal alignment for Highway 17 (New) and the locations of the bridges were originally provided to us on 1:5000 plan drawings prepared by McCormick Rankin Corporation dated January 31, 1995. The vertical alignment was subsequently provided by McCormick Rankin in digital format (plan and profile drawings of the study area) on March 10, 1997. The comments and recommendations in this report refer to the alignment prepared by McCormick Rankin and received on March 10, 1997. Further vertical alignment revisions have been made since the March 10 data was received and reference is also made to the revised data where applicable.

Each bridge site is described separately together with the subsurface conditions and engineering recommendations in Sections 5.0 to 10.0 of this report.

2.1 Background Information

An investigation was carried out by Golder Associates for the preliminary design of the planned realignment of Highway 17 (New) between the City of Sault Ste. Marie and Bar River Road. In addition, a number of geotechnical investigations were carried out by MTO in the 1970's as part of the preliminary alignment studies for the Highway 17 realignment. These investigations were carried out at bridge locations in areas where potentially high embankments were required, at bridge locations and within muskeg areas. In addition, there were a number of gravel pit studies which have been completed for the existing pits in the general vicinity of the proposed highway alignment. The relevant investigations included in the above information is provided in the following table:

LOCATION	REPORT DESIGNATION	DATE	INFORMATION	COMMENTS
Highway 17 between the City of Sault Ste. Marie and Bar River Road	Golder Associates: No. 951-1364	June 1996	General site description and recommendations 1 borehole at each of bridge sites (eight in total) plus boreholes, probeholes and hand auger holes along the alignment	Preliminary design recommendations for bridge foundations.
Garden River Bridge	MTO: WP 903-72-18	Nov. 12, 1975	5 Boreholes	Proposed alignments Schemes A & B
Lower Echo River Bridge	MTO: WP 903-72-01 Part II	March 18, 1975	2 boreholes	Lines A, B, C and D

The following discussion incorporates the previous information as relevant to the specific sites of the project. The relevant Record of Borehole sheets from the investigations carried out previously by Golder Associates and MTO are included in Appendix A.

3.0 INVESTIGATION PROCEDURES

The field work for this investigation was carried out between February 4 and April 1, 1997. At this time, fourteen boreholes and twelve piezocone tests were carried out at all of the six proposed bridge sites. In addition, five piezocone tests were carried out along the alignment of the north and south approach embankments at the Lower Echo River bridge site. The investigation was carried out using bombardier mounted CME 55 drill rigs supplied and operated by Marathon Drilling Inc. of Sudbury.

In each boring, samples were obtained at regular intervals of depth using 50 mm outside diameter split spoon samplers, in accordance with Standard Penetration Test (SPT) procedures. At selected locations, relatively undisturbed soil samples were obtained using 75 mm outside diameter thin-walled Shelby tubes. Where soft cohesive deposits were encountered, in-situ field vane tests were carried out to measure the undrained shear strength of the deposit. Groundwater conditions in the open boreholes were observed throughout the drilling operations, and 12.5 mm and 25 mm diameter plastic piezometers were installed in ten of the twelve boreholes to permit monitoring of the groundwater levels. The water levels in the piezometers were monitored during our investigation with the last readings obtained between March 27 and 31, 1997. The detailed subsurface soil and groundwater conditions encountered in the boreholes are given on the attached Record of Borehole sheets.

Piezocone tests were carried out at each bridge site. The piezo-cone penetration test (CPT) is an in-situ technique for geotechnical site characterization studies. The piezocone used for the investigation was an acoustic type with three electronic channels for measuring cone tip resistance, sleeve friction and pore water pressure. The piezocone consists of a special rod equipped with electronic sensing elements which is pushed at a constant rate into the ground with readings being logged at an interval of 0.025 m. This essentially constitutes a continuous probing of the changing soil conditions from which a continuous soil profile can be interpreted. The CPT equipment was advanced to practical refusal, using the hydraulic ram system on the drill rig. The results of the CPT are summarized on the Cone Penetration Test sheets included in Appendix B.

The Cone Penetration Test sheets show the measured data recorded in the field (i.e. tip resistance corrected for end area effects (Qt), porewater pressure (PWP) and sleeve friction resistance) and interpreted parameters such as undrained shear strength (s_u), classification index (Ic) and SPT blow counts corrected for energy and depth (N_{60}). The classification index is used to differentiate between the various soil types to obtain an inferred stratigraphy. The porewater pressure plots

(PWP) shown on the CPT sheets include a hydrostatic porewater pressure line (dashed) based on the input water level corresponding to the water table.

The field work was supervised on a full time basis by members of our technical staff who located the boreholes in the field, directed the drilling, sampling and in-situ testing operations, and logged the boreholes. The soil samples were identified in the field, placed in containers and transported back to our laboratory in Mississauga for further examination. Index and classification tests were carried out on selected samples. Oedometer testing was carried out on four of the Shelby tube samples to determine the stress history, strength and compression characteristics of the silty clay deposit. The results of the oedometer tests are summarized in Table 2. One undrained triaxial test with consolidation was carried out on a sample of the silty clay.

The borehole and piezocone test locations and elevations were surveyed by McCormick Rankin and we understand that they are referenced to the Geodetic Datum. The approximate locations of the boreholes and CPTs (based on the highway alignment chainage referred to on the Record of Borehole) are shown on Figures 1 to 6. The ground surface elevations and locations of the boreholes and piezocone tests are summarized in Table 1.

4.0 GENERAL SITE GEOLOGY AND STRATIGRAPHY

4.1 Site Geology

From published geologic information, the site is located in the physiographic region known as Canadian Precambrian Shield. The shield terrain comprises large expanses of intrusive rocks (gneisses and granites) cut by east-west trending metasediments and metavolcanics called "greenstone belts". The rocks are geologically complex with considerable folding, intrusive activity, regional metamorphism and faulting. Pleistocene lacustrine/fluvial deposits and recent swamp sediments have been laid down in depressions and are associated with the Glacial Lake Algonquin. Some 11,000 years ago, the glacier front was positioned in the area of the north margin of Lake Superior and was in contact with several high level glacial lakes. Due to periodic oscillations of the ice, the levels of these lakes fluctuated, opening and closing various outflow channels. During this time, the lacustrine sediments, which are typically varved/stratified clays, were deposited. Shore features in the area of Sault Ste. Marie were developed with the surface modified by wave and current action during the various glacial lake stages and with the deposition of stratified sands and gravel of glaciofluvial ice contact origin.

The present day surface is typically a flat plain interrupted in places by bedrock protrusions and dissected by fault controlled bedrock valleys. The drainage system is complicated with a series of deep channels carved within lacustrine and glaciofluvial deposits.

4.2 General Site Stratigraphy

In general, the subsoils encountered in the boreholes put down in the project areas consist of topsoil overlying either an extensive silty clay deposit of glaciofluvial and/or lacustrine origin or sand and/or silt and sand and gravel deposits in turn underlain by silty clay to clay. No bedrock was encountered in the boreholes put down during the current investigation nor during the investigation carried out at the bridge sites in 1995.

The detailed subsurface soil and groundwater conditions encountered in the boreholes and inferred from piezocone tests put down as part of this investigation, together with the results of the laboratory tests carried out on selected soil samples, are given on the attached Record of Borehole and CPT sheets and on Figures 7 to 14, following the text of this report. The stratigraphic boundaries shown on the borehole logs are inferred from non-continuous sampling and, therefore, represent transitions between soil types rather than exact planes of geological change. Subsoil conditions will vary between and beyond the borehole locations. The locations of the boreholes and stratigraphic sections showing the inferred subsurface conditions at each site are given on Figures 1 to 6. A more detailed description of the subsurface conditions encountered at each structure site is provided in the following sections of this report which address each of the six bridge sites separately as follows:

- Root River Crossing – one span twin structure,
- Belleau Creek crossing – one span twin structure,
- Jardin Mines Road Underpass - one span single structure,
- Garden River Crossing – three span twin structure,
- Noonday Road Underpass –one span single structure,
- Lower Echo River Crossing – one span twin structure.

5.0 ROOT RIVER CROSSING

5.1 Site and Project Description

The alignment of the proposed Highway 17 (New) crosses Root River at the location where the river changes its direction of flow from the northwest to the south. The water level in the river was at about Elevation 177.8 m at the time of our current investigation. The east river bank is approximately 8 m in height (top of the bank at about Elevation 186.5 m) with the river flowing directly at the toe of the slope. The east bank is oversteepened with overhanging tree roots and sandy soils exposed at the slope surface, where not covered by snow. The west bank of the river is approximately 2.5 m to 3 m in height with the existing ground surface at approximately Elevation 180.6 m at the top of the bank.

As shown on the general arrangement plan it is understood that the highway will be carried over Root River by a single span twin structure with span length between abutment bearings of 36 m. The final road grade at the crossing will be at about Elevation 186.5 m, coincident with the existing ground surface at the crest of the east bank and about 6 m above the flood plain on the west side of the river.

5.2 Subsurface Conditions

Relevant information on subsurface conditions was obtained from one borehole (Borehole 97-1) and three piezocone tests (CPT 4 to CPT 6) put down during the current investigation and one borehole (Borehole 94-4) drilled during our 1995 investigation carried out for the preliminary design of the bridge (Report No. 941-1364). Borehole 97-1 was drilled to 30.5 m depth on the west bank of the river and Borehole 95-4 was advanced from the top of the east bank to a depth of approximately 25 m below ground surface. A dynamic cone test was carried out adjacent to Borehole 95-4 and was continued to a depth of approximately 46.9 m below ground surface. The piezocone tests, CPT-4 and CPT-5, located on the east bank were advanced to 49.4 m and 50.1 m depths and CPT-6 located on the west bank was advanced to 21.4 m depth to practical refusal to further penetration. The approximate locations of the boreholes and piezocone tests and the general stratigraphy of the site are shown on Figure 1.

In general, the soils encountered consist of a granular deposit underlain by an extensive deposit of lacustrine silty clay which extends to the base of the sampled boreholes and piezocone tests and is inferred for the full depth of the dynamic cone test in Borehole 95-4.

5.2.1 Granular Deposits

A granular layered deposit consisting of sand, silt, silty sand and sand and gravel extends to a depth of 13.5 m to about 18 m (between Elevations 172.7 m and 168.5 m) on the east side of the river and to approximately 7 m depth (Elevation 173. 4 m) on the west side of the river. Interlayers of clayey silt and silty clay with organic silt seams were noted within this deposit at random depths. In Borehole 95-4, a 0.2 m thick layer of organic silt was observed at a depth of 4.7 m. Two silty clay layers, about 0.8 m and 1.5 m in thickness, were encountered at approximately 2.6 m and 14.8 m depths, respectively, in Borehole 95-4. SPT 'N' values measured in this deposit range from less than 1 blow to 10 blows for 0.3 m of penetration. The 'N' values when corrected for depth range from 2 to 20 blows per 0.3 m penetration indicating a very loose to compact state of packing.

5.2.2 Lacustrine Clay

Underlying the granular deposit is an extensive deposit of lacustrine clay. The lacustrine deposit is typically irregularly layered with silty clay, clay, silt and clayey silt layers The lacustrine clay was encountered to a depth of about 50 m (Elevation 136 m) and was not penetrated completely in either the borings or piezocone tests. Based on available geological information, it is probable that this deposit extends to considerable depth. Approximately 1 m of sand was encountered at 21.5 m depth in Borehole 97-1. The colour of the silty clay deposit is typically brown to red brown. In situ vane shear tests and peizocone testing carried out within this deposit measured undrained shear strengths increasing with depth from about 40 kPa to 130 kPa from the top of the deposit to about 26 m depth (Elevation 160 m). These values indicate that this deposit has a soft to stiff consistency. Below Elevation 160 m, the undrained shear strength, as measured during piezocone testing, increases from 130 kPa to 180 kPa indicating a stiff to very stiff consistency. The undrained shear strength profile of the clay deposit is shown on Figure 15.

The Atterberg Limits test results indicate ranges of liquid limit from 59 per cent to 83 per cent and plasticity indices from 33 per cent to 53 per cent. The natural water content of the silty clay samples ranged from 30 per cent to 65 per cent. Oedometer testing was carried out on two silty clay samples collected from approximately 12.7 m and 27.9 m depth to determine the stress history, strength and compression characteristics of the deposit. The results of oedometer tests are provided in Appendix C and summarized in Table 2. Based on the results of the tests the deposit is normally consolidated to slightly overconsolidated.

5.2.3 Groundwater Conditions

Details of the piezometer installations and the water level recorded in the piezometers are shown on the Record of Borehole sheets. The water level in Borehole 97-1 located on the west bank of the river was at Elevation 178.3 m in the shallow piezometer and at Elevation 178.0 m in the deep piezometer on February 13, 1997. The groundwater level in Borehole 95-4 was at Elevation 179.3 m. The water levels are slightly above the adjacent river level at the time of the investigation based on field observations. It should be noted that the water levels are subject to seasonal fluctuations.

5.3 Engineering Recommendations

This section of the report provides our recommendations on the geotechnical aspects of design of the Root River bridge based on our interpretation of the factual information obtained during our recent and previous field investigations and, where pertinent, from previous investigations. It should be noted that the interpretation and recommendations are intended for use only by the design engineer and the information may not be sufficient for construction. Where comments are made on construction they are provided only in order to highlight those aspects which could affect the design of the project. Those requiring information on aspects of construction should make their own interpretation of the factual information provided as it may affect equipment selection, proposed construction method and scheduling.

5.3.1 Bridge Foundations

The general arrangement plan provided by McCormick Rankin on June 11,1997, indicates that the top of the abutment pile caps will be at Elevation 180.64 m and the base at approximately Elevation 180.0 m. The subsoils encountered in the boreholes put down at this site are not suitable for support of shallow spread footings and deep foundations are recommended for the support of the abutments. Given the relatively loose nature of the cohesionless deposit encountered in the upper 18 m (about Elevation 173 m), the carrying capacity of piles terminated above this depth would be relatively low. Therefore, it is recommended that the piles be extended into the firm to stiff silty clay deposit below about 18 m depth.

Based on the subsurface data obtained, the following factored load resistance values at Ultimate Limit States (ULS) can be assumed:

Pile Type	Assumed Base of Pile Cap Elevation (m)	Factored Geotechnical Resistance at ULS for Pile with Driven Length of:						
		20 m	25 m	30 m	35 m	40 m	45	50 m
Closed-end pipe pile 324 mm OD	180	200	300	400	500	620	730	850
HP 310x79		250	370	500	610	750	900	1050

A resistance factor of 0.6 was used to obtain the factored geotechnical resistance, which assumes that pile load testing (static test) will be carried out during construction. Where dynamic testing consisting of strain gauge measurements of acceleration and strain induced in the pile is carried out, a resistance factor of 0.5 should be used for design. A resistance factor of 0.4 would have to be used where there is no confirmatory field testing of the piles. Given the sensitivity of the pile capacity (based to a large degree on shaft friction) it would be preferable to utilize dynamic monitoring in addition to one static load test for confirmation of pile capacity to assess the potential variation in the performance of the piles across the site.

The geotechnical resistance at Serviceability Limit States (SLS) is dependent on the settlement of the pile group and, therefore, is governed by the size of the pile group. Based on the plan provided on June 11, 1997, it is understood that the support for abutments will be provided by two rows of HP310x79 piles spaced at approximately 2 m and with approximately 2 m distance between the rows; the back piles are vertical and the front piles are inclined at 1 horizontal to 3 vertical. The piles will be driven to Elevation 134 (about 46 m long) . For this pile configuration, a geotechnical resistance at SLS of 700 kN can be assumed for design. This SLS value refers to 25 mm of settlement for the pile group.

For resistance to lateral loads, the horizontal reaction to the pile can be calculated from the expression:

$$k_s = z \times n_h / d$$

where

k_s = coefficient of horizontal subgrade reaction (MPa/m)
d = pile diameter (m)
n_h = constant of horizontal subgrade reaction (MPa)
z = depth below ground surface (m).

The constant of horizontal subgrade reaction, n_h, depends on the soil type and soil density/consistency around the pile shaft. For the design of resistance to lateral loads the following values (or ranges of values) may be assumed:

Elevation (m)	Soil Type	z x n_h (MPa)
180.6 to 173.5	Sand, silt and sand and gravel, interlayered, very loose to compact	z x 1.0 to z x 4.0
173.5 to 168.5	Silty clay, firm and sand, loose	z x 2.0
168.5 to 160.0	Silty clay, firm to stiff	2.0 to 3.0 (constant with depth)
160.0 to 136.0	Silty clay, stiff to very stiff	3.0 to 5.0 (constant with depth)

Group action for lateral loading should be considered when the pile spacing in the direction of loading is less than 6 to 8 pile diameters. Group action can be evaluated by reducing the coefficient of lateral subgrade reaction in the direction of loading by a reduction factor R as follows:

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

5.3.2 Lateral Earth Pressure

The lateral pressures acting on the bridge abutments and the retaining walls 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:

- Selected free-draining granular fill meeting the specifications of OPSS Granular A or B should be used as backfill behind the walls. All granular fill should be placed in lifts of loose thickness not greater than 200 mm and compacted to 95 per cent of the material's Standard Proctor maximum dry density.

- Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill.

- The granular fill may be placed either in a zone with width equal to at least 1.2 m behind the back of the stem (Case I) or within the wedge-shaped zone defined by a 60 degree line extending up and back from the bottom of the rear face of the footing (Case II).

- 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.

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

Soil unit weight	20 kN/m^3	(assuming the embankment fills are comprised of general earth fill (select fill))

Coefficients of lateral earth pressure:

'active'	0.33
'at rest'	0.5

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

	Granular A	**Granular B**
Soil unit weight	22 kN/m^3	21 kN/m^3
Coefficients of lateral earth pressure		
'active'	0.27	0.31
'at rest'	0.43	0.47

It should be noted that the above design parameters assume level backfill and ground surface behind the wall.

5.3.3 Excavations

The drawings provided on June 11, 1997, indicate that the east river bank will be graded to an inclination of 2 horizontal to 1 vertical. This is considered to be suitable for the site conditions. Depending on the gradation of the crushed rock slope protection proposed, a filter blanket/ geotextile should be provided on the river banks under the crushed rock to restrict the migration of the fine soil particles from the native soil mass (sand) into the crushed rock. This blanket/geotextile should extend to the maximum water level.

It is understood that the base of the pile cap for the abutments will be at approximately Elevation 180.0 m. The ground surface on the west side of Root River is approximately at the proposed top of the pile cap; on the east side of the river the excavation for the pile cap construction will extend approximately 6 m below the existing ground surface or about 3 m to 4 m below the proposed slope face. The excavation will extend through the loose granular deposit and the base will be maintained above the measured groundwater level.

Seasonal fluctuations in groundwater table and river water level fluctuations (maximum level Elevation 181.9 m) should be considered in planning the construction. In general, the sands are sensitive to disturbance and where they form excavation base or side walls below the groundwater level, the soil undergo rapid loosening, due to upward water seepage. Excavations which will be open for relatively short period of time can be made in temporary unsupported cut with side slopes formed not steeper than 1.5 horizontal to 1 vertical.

Perched water flow through the granular deposits should be expected; it is anticipated that the flow can be controlled by pumping from properly filtered sumps. Care should be taken to direct surface water away from the open excavation.

All excavations should be carried out in accordance with the current Occupational Health and Safety Act.

5.3.4 Approach Embankments

On the west side of the river, an approach embankment will be required to cross the floodplain. The embankment will be up to 6 m high and will be subject to settlement due to consolidation of the loose sands underlying the site. This settlement is expected to occur during construction of the embankment. There are numerous sandy layers within the underlying soft silty clay deposit and consolidation of this deposit under the embankment loading is anticipated to be relatively fast.

Monitoring of the embankment settlement during construction must be carried out to confirm the assumptions made with respect to rate and magnitude of consolidation to permit adjustments in design if required. It is recommended that installation of the piles for the west abutment be delayed until at least one year after embankment construction.

5.3.5 Subgrade Preparation and Embankment Construction

In general, topsoil and organic deposits should be stripped from below the fill embankment areas and all subgrade soils should be proof-rolled prior to fill placement. The subgrade in this area is expected to consist of sand, silty sand or silt. There are likely to be areas where the groundwater level is at shallow depth. Pumping and rutting of the subgrade will occur with extensive traffic over the subgrade.

Construction of the embankment above the prepared subgrade may be carried out using clean earth fill or granular fill. Clean earth fill (Select Subgrade Material, SSM, or Select Borrow Material, SBM) should be free of topsoil, organics, rubble, cobble sizes greater than 150 mm or other deleterious materials and have a moisture content at the time of placement within 2 per cent of the material's optimum moisture content. All embankment fills should be placed in regular lifts with loose thickness not exceeding 300 mm, and be compacted to at least 95 per cent of the material's Standard Proctor maximum dry density. The final lift prior to placement of the granular subbase or base course should be compacted to 100 per cent of the Standard Proctor maximum dry density. Inspection and field density testing should be carried out by qualified geotechnical personnel during all fill placement operations to ensure that appropriate materials are used and that adequate levels of compaction have been achieved.

Embankment side slopes formed at 2 horizontal to 1 vertical or flatter are considered suitable for embankments constructed of properly compacted clean earth fill as indicated above.

6.0 BELLEAU CREEK CROSSING

6.1 Site and Project Description

The Belleau Creek valley at the proposed Highway 17 (New) crossing is approximately 2 m to 3.5 m in height with the creek channel meandering in the valley floor. Oversteepening of the valley slopes was noted at some locations. Since the slopes were snow covered during our investigation, the conditions of the slope face (exposed soils, vegetations, seepage, etc.) were not observed.

Based on the General Arrangement plan provided on June 11, 1997, it is understood that Belleau Creek will be realigned in the area of the proposed bridge and that the highway crossing will be a single span twin structure with abutments located on either side of the realigned creek and span length of 29 m. The final road grade at the crossing will be at about Elevation 186 m and will be about 4 m to 7 m above the existing ground surface on both sides of the creek.

6.2 Subsurface Conditions

One borehole (97-2) and three piezocone tests (CPT-1 to CPT-3) were put down at the proposed crossing site during the current investigation and one borehole (95-5) was drilled during our preliminary investigation carried our in 1995. It should be noted that the borehole locations as plotted on Figure 2 (based on the survey data provided) do not appear to coincide with our field observations.

Subsoils encountered in the boreholes and interpreted from the piezocone tests consist of an extensive granular deposit extending to approximately 20.7 m depth underlain by lacustrine clay which extends to the depth investigated of 26.2 m. A silty clay layer up to 5 m thick which was encountered within the granular deposit on the west side of the creek separates the upper very loose deposit of interlayered sand and silt from the underlying loose to compact sand.

6.2.1 Upper Granular Deposits

A deposit of brown, well graded sand with some silt and containing seams of oxidized sandy silt underlies the topsoil in Borehole 97-2 and was inferred from all piezocone tests extending to about 2 m depth. The upper sand was not encountered in Borehole 95-5, where a silt deposit with trace to some sand underlies the topsoil. This silt deposit was found/inferred underlying the sand in Borehole 97-2 and all piezocone tests. Trace organic matter and interlayers of silty clay, clayey silt and sandy silt to sand were encountered within this deposit.

Standard Penetration Test (SPT) 'N' values measured in this deposit range from less than 1 blow (weight of hammer) to 8 blows for 0.3 m of penetration indicating a very loose to loose state of packing. The measured natural water contents of selected samples of this sand range from about 6 to 58 per cent. Higher values of about 41 to 58 per cent were measured within this deposit where silty clay and organic layers were present.

The deposit extends to between 3. 0 m to 4.6 m depth, corresponding to Elevation 180.8 m and 178.2 m, respectively.

6.2.2 Upper Silty Clay

Some 3.3 m to 5 m of soft, brown silty clay underlies the granular deposits in Borehole 97-2 and in CPT-3, located on the west side of the creek. The base of the silty clay layer was at a depth of about 7.5 m (Elevation 176.7 m). One in situ vane shear test carried out at about 6.6 m depth measured an undrained shear strength of 25 kPa. These values indicate that this deposit has a soft consistency. The measured natural water contents of two samples of this deposit were 50 per cent and 58 per cent.

6.2.3 Lower Sand

Fine to medium sand with trace silt, clay and organic matter was encountered to the depths investigated of 11.5 m to 16 m; where fully penetrated in borehole 97-2, the sand was found to extend to 20.7 m depth (Elevation 163.5 m). Seams of silt and clayey silt were noted throughout this deposit throughout.

SPT 'N' values measured in this deposit range typically from 4 to 29 blows for 0.3 m of penetration indicating a very loose to compact state of packing. The low 'N' values of 1 to 4 blows for 0.3 m of penetration measured in Borehole 95-4 were obtained probably due to blowing sand in the augers. When corrected for depth, the 'N' values range from 2 to 32 blows per 0.3 m penetration. The measured natural water contents of selected samples of this deposit range from about 23 per cent to 26 per cent.

6.2.4 Lacustrine Clay

In Borehole 97-2, the sand is underlain by a deposit of lacustrine clay. The lacustrine clay was investigated to about 26.6 m depth (Elevation 157.5 m) and was not penetrated completely in the boring. The silty clay deposit is brown to red brown in colour and is irregularly layered. In situ vane shear tests carried out within the silty clay measured undrained shear strengths of about 50 kPa at about 26 m depth (Elevation 159 m) indicating that this deposit has a soft to firm consistency.

6.2.5 Groundwater Conditions

Details of the piezometer installations and the water level recorded in the piezometers are shown on the Record of Borehole sheets. The water level in Borehole 97-2 located on the west bank of the creek was at Elevation 178.2 m and at Elevation 177.2 m in Borehole 95-5 on March 29, 1997. The water levels are at or slightly below the adjacent creek level at the time of the investigation. It should be noted that the water levels are subject to seasonal fluctuations.

6.3 Engineering Recommendations

This section of the report provides our recommendations on the geotechnical aspects of design of the works at the Belleau Creek crossing based on our interpretation of the factual information obtained during our recent and previous field investigations and, where pertinent, from previous investigations. It should be noted that the interpretation and recommendations are intended for use only by the design engineer and the information may not be sufficient for construction. Where comments are made on construction they are provided only in order to highlight those aspects which could affect the design of the project. Those requiring information on aspects of construction should make their own interpretation of the factual information provided as it may affect equipment selection, proposed construction method and scheduling.

The geotechnical recommendations contained herein relate to the design of the bridge foundations and approach embankment construction and are based on information obtained from the plans and profiles provided by McCormick Rankin Corporation.

6.3.1 Bridge Foundations

The general arrangement plan provided by McCormick Rankin indicates that the base of abutment pile caps will be at Elevation 180.1 m, up to 3 m below the existing ground surface on the east side of the river and at Elevation 179.7 m at or about 0.5 m above the existing ground surface on the west side of the river. The subsoils encountered in the boreholes and inferred from the piezocone tests put down at this site are not suitable for support of shallow spread footings; therefore, deep foundations are recommended for the support of the abutments. Given the relatively loose nature of the granular deposit and soft consistency of the cohesive deposit encountered in the upper 20 m (to about Elevation 163.5 m), the carrying capacity of piles terminated above this depth would be relatively low. Therefore, it is recommended that the piles be extended into the silty clay deposit below about 20 m depth. Based on our interpretation of the information obtained, the recommended values of factored geotechnical resistance at Ultimate Limit States ULS for steel H-piles or pipe piles driven to different depths are shown in the table below:

Pile Type	Base of Pile Cap Elevation (m)	Factored Load Carrying Capacity at ULS for Driven Pile Length of:				
		16 m	20 m	34 m	39m	44 m
Closed-end pipe pile 324 mm OD	180.1(east) 179.7 (west)	430	450	500	700	920
HP 310x79		400	520	620	860	1020

A resistance factor of 0.6 was used to obtain the factored geotechnical resistance, which assumes that pile load testing (static test) will be carried out during construction. Where dynamic testing consisting of strain gauge measurements of acceleration and strain induced in the pile is carried out, a resistance factor of 0.5 should be used for design. A resistance factor of 0.4 would have to be used where there is no confirmatory field testing of the piles. Given the sensitivity of the pile capacity, based to a large degree on shaft friction, it would be preferable to utilize dynamic monitoring in addition to one static load test for confirmation of pile capacity to assess the potential variation in performance of the piles across the site.

The geotechnical resistance at Serviceability Limit States (SLS) is dependent on the settlement of the pile group and, therefore, is governed by the size of the pile group. Based on the plan provided on June 11, 1997, it is understood that the support for abutments will be provided by one row of ten HP310x79 piles spaced at 1.5 m. For the above pile group configuration with the piles driven to Elevation 135.7 m (piles approximately 44 m long), a geotechnical resistance at SLS of 600 kN can be used for design.

The above SLS value refers to 25 mm of settlement for the pile group. For resistance to lateral loads, the horizontal reaction to the pile can be calculated from the expression:

$$k_s = z \times n_h / d$$

where

k_s = coefficient of horizontal subgrade reaction (MPa/m)
d = pile diameter (m)
n_h = constant of horizontal subgrade reaction (MPa/m)
z = depth (m).

The constant of horizontal subgrade reaction depends on the soil type and soil density/consistency surrounding the pile shaft. For design of resistance to lateral loads, the values (or range of values) indicated in the table below may be assumed:

Elevation (m)	**Soil Type**	**$z \times n_h$ (MPa)**
182.0 to 179.5	Silt, very loose	z x 1.0
179.5 to 177.0	Silty Clay and silt, very soft/loose	0.5 (constant with depth)
177.0 to 163.5	Sand, loose to compact	z x 1.0 to z x 4.0
163.5 to 159.0	Silty clay, firm	2.0 (constant with depth)
159.0 to 132.0	Silty clay, stiff	3.0 (constant with depth)

Group action for lateral loading should be considered when the pile spacing in the direction of loading is less than 6 to 8 pile diameters. Group action can be evaluated by reducing the coefficient of lateral subgrade reaction in the direction of loading by a reduction factor R as follows:

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

6.3.2 Lateral Earth Pressures

The lateral pressures acting on the bridge abutments and the retaining walls 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:

- Selected free-draining granular fill meeting the specifications of OPSS Granular A or B should be used as backfill behind the walls. All granular fill should be placed in lifts of loose thickness not greater than 200 mm and compacted to 95 per cent of the material's Standard Proctor maximum dry density.

- Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill.

- The granular fill may be placed either in a zone with width equal to at least 1.2 m behind the back of the stem (Case I) or within the wedge-shaped zone defined by a 60 degree line extending up and back from the bottom of the rear face of the footing (Case II).

- 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.

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

Soil unit weight	20 kN/m^3	assuming the embankment fills are comprised of general earth fill select fill

Coefficients of lateral earth pressure:

'active'	0.33
'at rest'	0.5

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

	Granular A	Granular B
Soil Unit Weight	22 kN/m^3	21 kN/m^3
Coefficients of lateral earth pressure		
'active'	0.27	0.31
'at rest'	0.43	0.47

It should be noted that the above design parameters assume level backfill and ground surface behind the wall.

6.3.3 Excavations

The general arrangement plan (provided June 11, 1997) indicates that the base of the abutment pile caps will be at Elevations 180.1 m and 179.7 m on the west and east sides of the creek, respectively. Three of the four abutments appear to be at or above the existing ground surface. The ground surface at the abutments varies from approximately Elevation 179 m to Elevation 183 m, respectively. Excavation for pile cap construction at the south-east abutment will be up to 3 m below the existing ground surface and extend through the loose granular deposit. The base of excavation will be most likely above the groundwater level which was measured in the piezometers at approximately Elevation 178 m. Excavations which will be open for relatively short period of time can be made in temporary unsupported cut with side slopes maintained not steeper than 1.5 horizontal to 1 vertical.

Perched water flow through the granular deposits should be expected; it is anticipated that the flow can be controlled by pumping from properly filtered sumps. Care should be taken to direct surface water away from the open excavation.

Seasonal fluctuations in groundwater table with maximum water level at Elevation 181.7 m (general arrangement plan) should be considered in planning construction. In general, the sands are sensitive to disturbance and where they form the excavation base or site walls below the groundwater level, the soils undergo rapid loosening due to upward seepage.

All excavations should be carried out in accordance with the current Occupational Health and Safety Act.

6.3.4 Belleau Creek Channel Realignment

It is understood that the Belleau Creek channel will be realigned to straighten the creek in the area of the bridge and that the base will be at approximately Elevation 178 m. The soils forming the channel banks and base will typically consist of silt to silty sand and sand. A silty clay deposit was encountered at and below the channel base level in Borehole 97-2 and in CPT-3 located on the west side of the existing creek and should be anticipated at some locations along the creek channel. The variation in the subsurface conditions at and below the channel bed level, together with the measured groundwater levels at the borehole locations, indicate that potential for upwelling should be considered in design of the realigned creek.

It is considered that side slopes of 2 horizontal to 1 vertical are feasible for the channel banks. The soils at the creek level are susceptible to erosion due to creek flow and loosening during excavation. Some sloughing of the banks may occur during and following construction until rock erosion protection is placed. The rock erosion protection should be placed on a filter blanket/geotextile to restrict the migration of fine soil particles from the native soil into the erosion protection.

6.3.5 Approach Embankment

The final road grade at the crossing will be at about Elevation 186 m and with the current highway/bridge alignment (plan provided on June 11, 1997), approach embankments will have variable heights along the eastbound and westbound lanes.

For the westbound lanes, the east abutment is located close to the current Belleau Creek channel and the embankment height at the approach slab will be about 8 m reducing to about 5 m behind the approach slab. At the west abutment, the embankment height at the west approach slab will be about 5.5 m.

For the eastbound lanes, the east approach embankment will be about 5 m high in the area of the abutment and approach slab. The west approach embankment spans over the current Belleau Creek channel and heights of 7 m to 8 m will be required. The MSE wall construction spans over the existing creek channel.

The subsoil stratigraphy at the site is variable with an upper soft clay layer evident on the west side of the existing creek channel but not on the east sides. The lower silty clay deposit is at depths of 15 m to 20 m below ground surface. These conditions, together with the variable and significant height of embankments, result in anticipated variable settlement of the embankments.

With the shallow depth and small thickness of the upper soft clay layer where it is present, it is anticipated that 100 per cent of the primary consolidation will be completed within 0.5 to 1 year. Consolidation settlement of about 40 mm to 240 mm will occur for the range in embankment heights proposed.

The consolidation of the lower clay deposit will occur over a longer period of the time (up to 50 years) and the magnitude of settlement will range from about 200 mm for a 2.5 m high embankment to 1200 mm for a 8 m high embankment.

An assessment of the effect of using light-weight fill was made for an 8 m high embankment. It was found that by preloading with full weight fill and then subexcavating and replacing with 6 m of light weight fill covered by 2 m of road structure, there would still be about 500 mm of long term settlement remaining. For this reason, light weight fill is not recommended.

Given the variability of the settlement anticipated, it is recommended that preloading of the embankment area be carried out for as long as possible at this site. After a 2 year preload period, about 500 mm and 900 mm of settlement will be remaining for a 5.5 m and 8 m high embankment, respectively. In order to compensate for the loading and remaining settlement, the use of styrofoam within the embankment should be considered. The styrofoam should be placed such that the height of the earth fill/road structure above the styrofoam is not greater than 3 m in the case of the 7 m to 8 m high embankment and not greater than 2 m in the case of the 5 m high embankment. The 8 m height applies to the east abutment of the westbound lanes and the west abutment of the eastbound lanes. The styrofoam should extend to at least 5 m back from the abutment pile cap and then be tapered by benching to at least the limit of the MSE wall..

Monitoring of embankment settlement during the preload stage must be carried out to confirm the assumptions made with respect to the rate and magnitude of consolidation and to permit adjustments in design if required.

6.3.6 Subgrade Preparation and Embankment Construction

In general, topsoil and organic deposits should be stripped from below the fill embankment areas and all subgrade soils should be proof-rolled prior to fill placement. The subgrade in this area is expected to consist of sand, sandy silt or silt.

Construction of the embankment above the prepared subgrade may be carried out using clean earth fill or granular fill. Clean earth fill (Select Subgrade Material, SSM, or Select Borrow Material, SBM) should be free of topsoil, organics, rubble, cobble sizes greater than 150 mm or other deleterious materials and have a moisture content at the time of placement within 2 per cent of the material's optimum moisture content. All embankment fills should be placed in regular lifts with loose thickness not exceeding 300 mm, and be compacted to at least 95 per cent of the material's Standard Proctor maximum dry density. The final lift prior to placement of the granular subbase or base course should be compacted to 100 per cent of the Standard Proctor maximum dry density. Inspection and field density testing should be carried out by qualified geotechnical personnel during all fill placement operations to ensure that appropriate materials are used and that adequate levels of compaction have been achieved.

Embankment side slopes formed at 2 horizontal to 1 vertical or flatter are considered suitable for embankments constructed of clean earth fill as indicated above.

7.0 JARDIN MINES ROAD UNDERPASS

7.1 Site and Project Description

The proposed Jardin Mines Road underpass is located some 700 m west of the existing Jardin Mines Road. The site is relatively flat with the ground surface ranging from Elevation 188.5 m to 188.8 m across the site.

It is understood that the Jardin Mines Underpass will be a two span or a single span structure. As shown on the general arrangement plan provided on June 11, 1997, the single span structure will be 46.4 m in length. The proposed new Jardin Mines Road at the underpass will be at about Elevation 193.5 m. The proposed Highway 17 (New) grade will be at about Elevation 187.0 m, approximately 1.5 m to 1.8 m below the existing ground surface.

7.2 Subsurface Conditions

One borehole (97-3) and three piezocone tests (CPT-7 to CPT-9) were put down at the proposed crossing site during the current investigation and one borehole (95-6) was drilled during our preliminary investigation carried out in 1995. Boreholes 95-6 and 97-3 were advanced to 15.9 m and 24.8 m depth, respectively. The piezocones, originally planned to be advanced to 25 m, met refusal on a compact sand layer encountered between Elevation 170.5 m and 165.9 m (about 16.6 m to 22.8 m depths). In addition to the planned program, a dynamic cone was driven from about 19 m to 31.7 m depth; significant increase in driving resistance was met below about 30 m depth. To confirm the presence of dense/very stiff strata at depth, we returned to the site to carry out additional drilling with soil sampling from 23.2 m to 40.2 m depth (Borehole 97-13).

In general, subsoils encountered in the boreholes and interpreted from the piezocone tests consist of very loose to loose interlayered granular deposit extending to about 6 m to 13 m depth underlain by an extensive lacustrine deposit of silty clay to clay which extends to the depth investigated.

7.2.1 Upper Interlayered Granular Deposits

Some 0.15 m to 0.2 m of topsoil was encountered at ground surface in each of the boreholes. Underlying the topsoil is a granular deposit consisting of sand and silty sand interlayered with soft to very soft clayey silt, silty clay. Trace organic matter and organic partings were noted in this deposit. The interlayered granular deposit was inferred to extend to Elevation 175 m (13.5 m depth) in CPT-7 and to about Elevation 183 (about 6 m depth) in CPT-9.

Standard Penetration Test (SPT) 'N' values measured in this deposit range from less than 1 blow (weight of hammer) to 9 blows for 0.3 m of penetration indicating a very loose to loose state of packing. One 'N' value of 18 was obtained within the sand layer at about 4 m depth. The measured natural water contents of selected samples of the sand range from about 12 to 43 per cent. Higher values of about 38 per cent and 43 per cent were measured within a silty clay layer. The results of a grain size distribution tests carried out on one sample of this sand deposit is shown on Figure 8.

In situ vane shear tests carried out within the more cohesive portions of this deposit in Borehole 95-6 measured undrained shear strengths of 22 kPa and 25 kPa. These values indicate that this deposit has a soft consistency. Remoulded strengths indicate a sensitivity of 2.5.

7.2.2 Upper Lacustrine Clay

Some 3.4 m to 9.5 m of brown, irregularly layered silty clay to clay and clayey silt underlies the granular deposits in all of the boreholes and piezocone tests. The base of the silty clay layer was between about 15.1 m depth (Elevation 173.1 m) and 18 m depth (Elevation 170.7 m). Borehole 95-3 was terminated in this deposit at 15.9 m depth (Elevation 172.7 m). In situ vane shear tests carried within this layer measured undrained shear strengths in the range of 45 kPa to 85 kPa. These values indicate that the deposit has a firm to stiff consistency. Remoulded strengths of about 15 kPa to 22 kPa correspond to a sensitivity between 3 and 4.5 indicating that the material is moderately sensitive to sensitive.

The measured natural water contents of two samples of this deposit were 55 per cent and 62 per cent. One Atterberg limit test indicated liquid limit of 81 per cent and plastic limit of 31 per cent. These values indicate that the material is of high plasticity.

7.2.3 Lower Granular Deposit

Fine to medium sand and silty sand with trace clay and organic matter was encountered below 15.1 m depth (Elevation 173.1 m) to 18 m depth (Elevation 170.7 m) underlying the lacustrine clay. The sand and silty sand was inferred from all piezocone tests to extend up to 18.0 m depth, where practical refusal to further penetration was met probably on a denser sand layer. The deposit continues in Borehole 97-3, where it was found to extend to 23.2 m depth (Elevation 165.5 m). In Borehole 97-3, the upper 2.7 m of the deposit is interlayered with silt and silty clay.

SPT 'N' values measured in this deposit range from 8 to 12 blows for 0.3 m of penetration indicating a loose to compact state of packing. The measured natural water contents of selected samples of this deposit range from about 20 per cent to 39 per cent.

7.2.4 Lower Lacustrine Deposit

A lacustrine deposit consisting of silty clay to clay and clayey silt to silt was encountered underlying the sand in Borehole 97-3 and extending from the sampled depth of 22.8 m in Borehole 97-13, which was drilled some 3 m east of Borehole 97-3. The silty clay deposit is brown to red brown in colour and is irregularly layered. Seams of grey silt and clayey silt were noted in this deposit at random depths.

The lacustrine deposit was investigated to about 40.2 m depth (Elevation 148.2 m) and was not penetrated completely in the boring. In situ vane shear tests carried out within the silty clay measured undrained shear strength in a range of 70 kPa to in excess of 96 kPa (the maximum strength value measurable by the vane being 96 kPa). Remoulded strengths of about 30 kPa to 45 kPa correspond to a sensitivity of 2 to 4, indicating that the material is moderately sensitive.

The measured natural water contents of selected samples of this deposit ranged between 38 per cent and 58 per cent. One Atterberg limit test indicated a liquid limit of 80 per cent and plastic limit of 30 per cent. These values indicate that the material is of high plasticity.

7.2.5 Groundwater Conditions

Details of the piezometer installations and the water level recorded in the piezometers are shown on the Record of Borehole sheets. The water level in Borehole 97-3 located in the vicinity of the north abutment was at Elevation 186.3 m (about 2.4 m below ground surface) in both the deep and shallow piezometers. The groundwater level was at Elevation 185.6 m (about 3 m below ground surface) in the piezometer installed in Borehole 95-6. It should be noted that the water levels are subject to seasonal fluctuations.

7.3 Engineering Recommendations

This section of the report provides our recommendations on the geotechnical aspects of design of the Jardin Mines Road bridge based on our interpretation of the factual information obtained during our recent and previous field investigations and, where pertinent, from previous investigations. It should be noted that the interpretation and recommendations are intended for use only by the design engineer and the information may not be sufficient for construction. Where comments are made on construction they are provided only in order to highlight those aspects which could affect the design of the project. Those requiring information on aspects of construction should make their own interpretation of the factual information provided as it may affect equipment selection, proposed construction method and scheduling.

The geotechnical recommendations contained herein relate to the design of the bridge foundations and approach embankment construction and are based on information obtained from the plans and profiles provided by McCormick Rankin Corporation.

7.3.1 Bridge Foundations

Based on the general arrangement plan provided on June 11, 1997, it is understood that the Jardin Mines Underpass will be a single span structure about 46.4 m in length. The base of the abutment pile caps is proposed approximately at Elevation 188.5 m. The proposed Highway 17 (New) grade will be at about Elevation 187 m, approximately 1.5 m to 1.8 m below the existing ground surface. The proposed new Jardin Mines Road grade at the underpass will be at about Elevation 193.5 m.

The subsoils encountered in the boreholes and inferred from the piezocone tests put down at this site are not suitable for support of shallow spread footings; therefore, deep foundations are recommended for the support of the abutments and the pier. Given the relatively loose nature of the granular deposit and soft consistency of cohesive deposit encountered in the upper 13.5 m (to about Elevation 175 m), the carrying capacity of piles terminated above this depth would be relatively low. Consideration can be given to the use of closed-end pipe piles and steel H-piles. Based on our interpretation of the information obtained, the recommended values of factored geotechnical resistance at ULS for closed-end pipe pile 324 mm in diameter or HP310x79 piles driven to different depths at the abutments are as follows:

	Factored Geotechnical Resistance for Driven Pile Length of					
	26 m	**31 m**	**35 m**	**38 m**	**40 m**	**46.5 m**
Closed-end pipe pile 324 mm OD or HP 310x79	520	640	740	810	850	950
Pile terminated in soil (embedded depth)	Silty clay					

A resistance factor of 0.6 was used to obtain the factored geotechnical resistance, which assumes that pile load testing (static test) will be carried out during construction. Where dynamic testing consisting of strain gauge measurements of acceleration and strain induced in the pile is carried out, a resistance factor of 0.5 should be used for design. A resistance factor of 0.4 would have to be used where there is no confirmatory field testing of the piles. Given the sensitivity of the pile capacity (based to a large degree on shaft friction), it would be preferable to utilize dynamic monitoring during pile installation in addition to one static load test for confirmation of the pile capacity to assess the potential variation in pile performance across the site.

The geotechnical resistance at Serviceability Limit States (SLS) is dependent on the settlement of the pile group and, therefore, is governed by the size of the pile group. Based on the general arrangement plan provided, it is understood that the support for abutments will be provided by a group of 10 or more piles arranged in two rows placed 0.7 m apart with the piles in each row staggered such that the distance between the pile in the front and back rows is approximately 1.6 m. For the above pile group configuration, a geotechnical resistance at SLS of 600 kN can be used for design of piles driven to Elevation 142 m (46.5 m long).

A geotechnical resistance at SLS of 700 kN can be used for design for the pile group configuration consisting of two rows of piles with vertical back piles and front piles inclined at 1 horizontal to 3 vertical (rows at approximately 2 m spacing, piles at approximately 2 m spacing; no staggering of piles) driven to Elevation 142 m. The above SLS values refer to 25 mm of settlement for the pile group.

For resistance to lateral loads, the horizontal reaction to the pile can be calculated from the expression:

$$k_s = z \times n_h / d$$

where

k_s = coefficient of horizontal subgrade reaction (MPa/m)
d = pile diameter (m)
n_h= constant of horizontal subgrade reaction (MPa/m)
z = depth (m)

The constant of horizontal subgrade reaction depends on the soil type and soil density/consistency around the pile shaft. For design of resistance to lateral loads, the values (or range of values) indicated in the table below may be assumed.

Elevation (m)	Soil Type	z x n_h (MPa)
187.0 to 185 .0	Sand and silty clay, very loose to loose or very soft	z x1.0
185.0 to 182.0	Sands and silts, compact	z x 4.0
182.0 to 179.0	Sands and silts, loose to compact	z x 1.0 to z x 4.0
179.0 to 177.0	Silt and silty clay, loose/firm	2.0 (constant with depth)
177.0 to 171.0	Silty clay, stiff	3.0 (constant with depth)
171.0 to 165.5	Sands and silts, loose to compact	z x 1.0 to z x 4.0
165.5 to 148.5	Silty clay, stiff to very stiff	5.0 (constant with depth)

Group action for lateral loading should be considered when the pile spacing in the direction of loading is less than 6 to 8 pile diameters. Group action can be evaluated by reducing the coefficient of lateral subgrade reaction in the direction of loading by a reduction factor R as follows:

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

7.3.2 Lateral Earth Pressures

The lateral pressures acting on the bridge abutments and the retaining walls 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:

- Selected free-draining granular fill meeting the specifications of OPSS Granular A or B should be used as backfill behind the walls. All granular fill should be compacted in lifts of loose thickness not greater than 200 mm to 95 per cent of the material's Standard Proctor maximum dry density.

- Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill.

- The granular fill may be placed either in a zone with width equal to at least 1.2 m behind the back of the stem (Case I) or within the wedge-shaped zone defined by a 60 degree line extending up and back from the bottom of the rear face of the footing (Case II).

- 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.

- For Case I, the pressures are based on the in-situ soils and the following parameters (unfactored) may be assumed:

Soil unit weight	20 kN/m^3	(assuming that clean earth fill is used for embankment construction)
Coefficients of lateral earth pressure:		
'active'	0.33	
'at rest'	0.5	

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

	Granular A	**Granular B**
Soil unit weight	22 kN/m^3	21 kN/m^3
Coefficients of lateral earth pressure		
'active'	0.27	0.31
'at rest'	0.43	0.47

It should be noted that the above design parameters assume level backfill and ground surface behind the wall.

7.3.3 Excavation

Assuming the abutment pile cap to be at Elevation 188.5 m, and the existing ground surface in the area of the proposed bridge to be between Elevations 187 m and 188.7 m, only minor excavation for the pile cap construction will be required, extending to up to about 1.5 m below the existing ground surface through the loose granular deposit. The base of excavation will most likely be above the ground water level which was measured in the shallow piezometer at Elevation 186.4 m (approximately 2 m below the ground surface). Excavations which will be open for relatively short period of time can be made in temporary unsupported cut with side slopes maintained not steeper than 1.5 horizontal to 1 vertical. Some perched flow through the granular deposits should be expected; it is anticipated that the flow can be controlled by pumping from properly filtered sumps.

The general arrangement drawing indicates that the base of the MSE wall will be at about Elevation 185.5 m. The base of the excavations for wall construction will extend to about 1 m below the groundwater level and water flow into excavations made through the upper granular deposits will be substantial. In general, the sands are sensitive to disturbance and where they form the excavation base or side walls below the groundwater level, the soils undergo rapid loosening due to upward water seepage. Groundwater lowering or control will be required. The use of closed steel sheetpiling can be used as temporary support to the excavation and to form a cut-off to groundwater flow. The cut-off wall must extend to sufficient depth below the base of the excavation to minimize piping of the sands forming the base. The cut-off wall can be embedded into the upper lacustrine clay which was inferred at variable depths but generally at about Elevation 177 m at the proposed abutment locations. Provision for pumping through properly filtered sumps at the base of excavation must be made.

Care should be taken to direct surface water away from the open excavation. All excavations should be carried out in accordance with the current Occupational Health and Safety Act.

It should be noted that the proposed central ditch excavation, as shown on the general arrangement plan (June 11, 1997), with base at about Elevation 185 m, will be extended to below the groundwater table or measured (Elevation 186.4 m). It should be confirmed that adequate drainage to the proposed Highway 17 (New) road grade (Elevation 186.4 m) is provided.

7.3.4 Approach Embankments

The final road grade at the crossing will be at about Elevation 193.5 m and an approach embankment about 5 m high will be required on both sides of the bridge structure.

The approach embankment will undergo some settlement due to consolidation of the loose sands and silty clay deposits underlying the site. The settlement due to consolidation of the sands and silts is expected to occur during construction of the embankment. However, long term consolidation settlement of the underlying soft to firm silty clay deposit will occur. It is anticipated that for the typical embankment crest width of 12 m and side slopes at 2 horizontal to 1 vertical, the long-term settlement will be in the range of 500 mm to 650 mm. Preloading and/or surcharging together with wick drain installation will be required to reduce the post construction settlement.

About 350 mm of the settlement occurs as a result of consolidation of the upper silty clay deposit which is of variable thickness. It is recommended that a 1 m surcharge be applied to the embankment and that wick drains be used to speed up the consolidation process of this upper clay deposit. The wick drains should be installed at 2 m centres in a square grid pattern and be extended to the base of the upper clay deposit at about Elevation 170 m. It is considered that a preload period of 2 to 3 years together with the wick drains will result in compensating for the long-term settlement.

The settlement of the embankment due to consolidation of the lower clay will occur over a longer period of time. Due to the plastic nature of the clay, wick drains are not likely to be effective within this lower clay deposit. The use of a 3 m thick layer of styrofoam within the embankment immediately behind the abutments would compensate for this long term settlement where applied in conjunction with the preloading.

Monitoring of the embankments during the preload period and subsequent to pavement construction should be carried out to confirm the rate of settlement and to allow for adjustment in design, if required.

7.3.5 Subgrade Preparation and Embankment Construction

In general, topsoil and organic deposits should be stripped from below the fill embankment areas and all subgrade soils should be proof-rolled prior to fill placement. The subgrade consists of sand and silty sand. There are likely to be areas where the groundwater level is at shallow depth. Pumping and rutting of the subgrade will occur with extensive traffic over the subgrade.

Construction of the embankment above the prepared subgrade may be carried out using clean earth fill or granular fill. Clean earth fill (Select Subgrade Material, SSM or Select Borrow Material, SBM) should be free of topsoil, organics, rubble, cobble sizes greater than 150 mm or other deleterious materials and have a moisture content at the time of placement within 2 per cent of the material's optimum moisture content. All embankment fills should be placed in regular lifts with loose thickness not exceeding 300 mm, and be compacted to at least 95 per cent of the material's Standard Proctor maximum dry density. The final lift prior to placement of the granular subbase or base course should be compacted to 100 per cent of the Standard Proctor maximum dry density. Inspection and field density testing should be carried out by qualified geotechnical personnel during all fill placement operations to ensure that appropriate materials are used and that adequate levels of compaction have been achieved.

Embankment side slopes formed at 2 horizontal to 1 vertical or flatter are considered suitable for embankments constructed of clean earth fill as indicated above.

8.0 GARDEN RIVER CROSSING

8.1 Site and Project Description

The Garden River valley at the proposed Highway 17 (New) crossing is approximately 16 m in height with the top of the valley at about Elevation 195 m. The west valley slope is oversteepened with the river flowing directly at the toe of the slope. There has been a recent failure of the full height of the bank leaving overhanging tree roots at the crest and failure debris at the toe. The slope failure has exposed sandy soils along the full height of the slope. To the east of the river, there is an approximately 80 m wide floodplain which gently slopes upward to about Elevation 182 m at the toe of the east valley slope. The east valley bank then rises relatively steeply to Elevation 195 m. The water level in the river was at about Elevation 180.4 m at the time of our current investigation.

It is understood that Highway 17 (New) will be carried over Garden River by a three span twin structure about 90 m in length. As shown on the general arrangement plan provided, the final road grade will be at about Elevation 192 m on the west side of the valley crossing and at Elevation 192.5 m on the east side of the valley crossing. The road will be constructed in cut approximately 2.5 to 3 m below the existing ground surface at the crest of the west valley slope. On the east side of the river, the proposed highway will be about 12 m to 13 m above the floodplain.

8.2 Subsurface Conditions

Relevant information on subsurface conditions was obtained from five boreholes (Boreholes 97-4 to 97-8) put down during the current investigation, one borehole (Borehole 95-7) drilled during our previous investigation carried out in 1994 for the preliminary design of the bridge (Report No. 941-1364) and one borehole (Borehole 3) put down by the MTO in the fall of 1975.

It should be noted that the location of Borehole 97-6 as shown on Figure 4 (based on the survey data provided) does not confer with our field measurements and should be closer to the proposed east abutment alignment.

Boreholes 97-4 and 97-5 were put down on the west side of the river valley at the north and south abutment locations to about 23.3 m and 24.8 m depths (Elevation 171.2 m to 170.8 m), respectively. Boreholes 97-6 to 97-9 were put down on the east side of the river and advanced to between 25 m and 27.4 m depths (Elevation 154 m to 155.6 m). Borehole 95-7 was advanced to a depth of about 25.0 m below ground surface at the crest of the east river valley slope and located approximately 130 m east of the river. The MTO Borehole 75-3 was put down at the west bank of Garden River in close proximity to the proposed Highway 17 centreline and was advanced to a depth of approximately 12.7 m (Elevation 170.3 m).

The approximate locations of the boreholes and interpreted soil stratigraphy encountered in the boreholes are shown on Figure 2D.

In general, the soils encountered in the boreholes consist of an extensive granular deposit of sand, sandy silt, silt and sand and gravel extending to the depths investigated. Interlayers of silty clay were noted within the deposit in all of the boreholes.

8.2.1 Upper Sands and Silts

On the west side of the valley, very loose to loose, well graded sand was encountered extending to a depth of about 3.0 m (Elevation 192.3 m). Oxidized sand seams and layering were noted in the sand. The sand deposit is brown to grey in colour. Below 3 m depth, the sand grades to silty sand and sandy silt with trace clay and trace rootlets. The deposit becomes grey in colour. The base of the sandy silt was estimated to be at 5.8 m to 6.7 m depth (Elevations 189.4 m to 188.6 m). The sandy silt is in turn, underlain by up to 6 m thick interlayered deposit comprised of grey, silt, sandy silt and sand with trace clay and trace organic matter. The base of the interlayered deposit was at 11.6 m depth (Elevation 183.7 m).

Standard Penetration Test (SPT) 'N' values measured in these deposits range from 2 to 8 blows for 0.3 m of penetration indicating a very loose to loose state of packing. The measured natural water contents of selected samples range from about 6 to 48 per cent. Higher values of about 35 to 48 per cent were measured within the interlayered deposit where a silty clay and organic seams were present. The results of grain size distribution test carried out on one sample of this deposit is shown on Figure 6.

8.2.2 Sand and Gravel

Underlying the fine grained deposit is some 2.3 m to 2.7 m of sand and gravel with trace silt; cobbles and boulders were inferred based on auger resistance during drilling. The sand and gravel is a very dense compact state of packing; the measured SPT 'N' values range from 68 blows to in excess of 100 blows for 0.3 m of penetration. The base of the very dense sand and gravel was between 13.9 m and 14.2 m depth (Elevations 181 m and 181.5 m).

8.2.3 Lower Granular Deposit

Another granular deposit consisting of very loose to compact, grey well graded sand underlies the sand and gravel on the west site of the river and extends from the ground surface on the east side of the river. Some 1.5 m of interlayered silty clay and sand was found at about 17.7 m depth (Elevation 177 m) in Borehole 97-5 put down on the west side of the river and 2 m to 3 m thick layer of silty clay and sand extends from about 2 m depth (Elevation 179 m) on the east side of the river. Another layer of silty clay about 0.5 m to 1m thick was encountered at greater depths (between 15.5 m to 20 m) in the boreholes put down on the east side of the river.

On the west side of the river valley, the sand is typically in a very loose to compact state of packing to about Elevation 174 m (21 m below the crest of the valley); the measured SPT 'N' values range from less than 1 blow to 19 blows for 0.3 m of penetration. Below Elevation 174 m, the granular deposit becomes dense to very dense.

On the east side of the river, boreholes put down on the floodplain encountered very loose to loose granular deposits to about Elevation 163 m (18 m depth). Below Elevation 163 m, the granular deposit becomes coarser and is in a dense to very dense state of packing; the measured SPT 'N' values range from 40 to 100 blows for 0.3 m of penetration. The measured natural water contents of selected samples of this deposit range from 17 per cent to 26 per cent.

The subsoils encountered on the west bank edge of the river (MTO Borehole 75-3) in the area of the proposed west pier consist generally of sand deposits. Above a depth of about 9.8 m (Elevation 173 m), the sand is typically in a loose state of packing; a thin layer of silty clay was encountered at about 6.4 m depth. Below about 9.8 m depth the deposit consists of very dense sand and gravel extending to the depth investigated.

8.2.4 Groundwater Conditions

The water levels in the piezometers installed into boreholes put down on the west side of the river were measured between about Elevation 180 m to 181 m, coinciding with the water level in the river. Artesian water conditions, were encountered in the three boreholes (Borehole 97-5 to 97-8) on the east side of the river. In Borehole 97-7, artesian water condition was noted 1 hour after completion of piezometer installation with water level up to 1 m above ground surface. On March 29, 1997, the piezometer tubing was frozen to about 0.3 m above ground surface.

The artesian flow in Borehole 97-8 was noted during drilling after advancing the augers to 16 m depth (Elevation 164 m). The water level in both the deep and shallow piezometers installed in this borehole was at ground surface (Elevation 180.4 m) on completion of piezometer installation and at Elevation 180 m (approximately 0.4 m below the existing ground surface). In Borehole 97-6, the water level was observed to be at ground surface upon completion of drilling and on February 25, 1997. When measured on March 27, 1997, the piezometer tubing was frozen to 0.3 m above the existing ground. It should be noted that the water levels are subject to seasonal fluctuations.

8.3 Engineering Recommendations

This section of the report provides our recommendations on the geotechnical aspects of design of the Garden River bridge based on our interpretation of the factual information obtained during our recent and previous field investigations and, where pertinent, from previous investigations. It should be noted that the interpretation and recommendations are intended for use only by the design engineer and the information may not be sufficient for construction. Where comments are made on construction they are provided only in order to highlight those aspects which could affect the design of the project. Those requiring information on aspects of construction should make their own interpretation of the factual information provided as it may affect equipment selection, proposed construction method and scheduling.

The geotechnical recommendations contained herein relate to the design of the bridge foundations, approach embankment construction and permanent cut slopes and are based on information obtained from the plans and profiles provided by McCormick Rankin Corporation.

8.3.1 Bridge Foundations

It is understood that the Highway 17(New) will be carried over Garden River by a three span twin structure about 90 m in length. As shown on the general arrangement plan provided (June 11, 1997), the final road grade will be at about Elevation 192 m on the west side of the crossing and at Elevation 192.5 m on the east side of the crossing. The base of the pile cap at the west abutment will be at Elevation 183.67 m and at the east abutment at approximately Elevation 184.18 m. The bases of pile caps at the piers are indicated at approximately Elevation 178 m (top of cap at Elevation 178.5 m) and Elevation 178.5 m (top of cap at Elevation 180.0 m) on the west and the east side of the river, respectively.

The subsoils encountered in the boreholes put down at this site are not suitable for support of shallow spread footings; therefore, deep foundations are recommended for the support of the abutments and the pier. Consideration can be given to the use of closed-end pipe piles or steel H-piles driven to the dense sands encountered at about Elevation 162 m on the east side of the river and to Elevation 168 m to 170 m on the west side of the river. Based on our interpretation of the information obtained, the recommended values of factored geotechnical resistance at ULS for closed-end pipe piles 324 mm in diameter or HP310x79 piles driven to be found within the dense sand deposit (elevation varying at the abutments and piers) are as follows:

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	Factored Geotechnical Resistance at ULS			
	West Abutment	**West Pier**	**East Pier**	**East Abutment**
Pile Tip Elevation (m)	168.0	170.0 m	162.0 m	
Assumed Base of Pile Cap Elevation (m)	183.67	178.0	178.5	184.18
Closed-end pipe pile 324 mm OD	1000 kN	900 kN	1000 kN	1000 kN
HP 310x79	900 kN	700 kN	900 kN	900 kN

The geotechnical resistance at Serviceability Limit States for the pipe piles or H-piles as indicated above may be taken as:

800 kN for the west and east abutments and east pier,

700 kN for the west pier.

It should be noted, that due to the presence of a very dense sand and gravel layer at about the pile cap level at the west abutment, preaugering for pile installation at this foundation unit will probably be required.

The piles driven at the east pier to the elevation shown will penetrate slightly into the granular layer where artesian water pressures were noted. With the base of the pile cap at Elevation 178.5 m, the head of water above the base of the excavation will be about 3 m. It is understood that consideration is being given to the use of a sheet pile cofferdam with a tremie seal placed at the base and that the piles will be driven through the tremie seal. Provided that the seal is poured tight to the cofferdam and that there is not significant flow of water noted along the piles during installation, it is considered that the tremie and concrete footing will be sufficient to prevent long-term migration of fines along the piles. With the pile cap above ground surface at the east abutment, similar conditions may be encountered; however, the piles will be driven through a granular pad built above ground surface and no specific concerns with respect to migration of fines are anticipated.

A resistance factor of 0.6 was used to obtain the factored geotechnical resistance, which assumes that pile load testing (static test) will be carried out during construction. Where dynamic testing consisting of strain gauge measurements of acceleration and strain induced in the pile is carried out, a resistance factor of 0.5 should be used for design. A resistance factor of 0.4 would have to be used where there is no confirmatory field testing of the piles. Given the sensitivity of the pile capacity, based to a large degree on shaft friction, it would be preferable to utilize dynamic monitoring in addition to one static load test for confirmation of pile capacity to permit assessment of potential variation in performance of the piles across the site.

For resistance to lateral loads, the horizontal reaction to the pile can be calculated from the expression:

$$k_s = z \times n_h / d,$$

where

k_s = coefficient of horizontal subgrade reaction (MPa/m)
d = pile diameter (m)
n_h= constant of horizontal subgrade reaction MPa/m)
z = depth (m)

The constant of horizontal subgrade reaction depends on the soil type and soil density/consistency around the pile shaft. For design of resistance to lateral loads, the values (or range of values) indicated in the tables below may be assumed.

- west abutment:

Elevation (m)	**Soil Type**	**$z \times n_h$ (MPa)**
184.0 to 181.0	Sand and gravel, very dense	z x 10.0
181.0 to 174.0	Sand, very loose to compact	Z x 1.0 to z x 4.0
174.0 to 170.0	Sand, compact to dense	Z x 4.0 to z x 6.0

- east abutment:

Elevation (m)	Soil Type	z x n_h (MPa)
184.0 to 181.5	Embankment fill, compact	z x 2.0
181.5 to 179.0	Sand and gravel, compact to dense	Z x 4.0 to z x 6.0
179.0 to 165.0	Sand, very loose to loose	z x 1.0
165.0 to 164.0	Silty clay, stiff	3.0 (constant with depth)
164.0 to 154.0	Sand, dense to very dense	z x 10.0

- west pier:

Elevation (m)	Soil Type	z x n_h (MPa)
178.0 to 173.0	Sand, loose	z x 1.0
173.0 to 170.0	Sand to gravely sand, very dense	z x 10.0

- east pier:

Elevation (m)	Soil Type	z x nh (MPa)
178.5 to 164.0	Sand, loose	z x 1.0
164.0 to 160.0	Sand, compact to very dense	Z x 4.0 to z x 6.0
174.0 to 170.0	Sand, very dense	z x 10.0

Group action for lateral loading should be considered when the pile spacing in the direction of loading is less than 6 to 8 pile diameters. Group action can be evaluated by reducing the coefficient of lateral subgrade reaction in the direction of loading by a reduction factor R as follows:

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

8.3.2 Lateral Earth Pressures

The lateral pressures acting on the bridge abutments and the retaining walls 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:

- Selected free-draining granular fill meeting the specifications of OPSS Granular A or B should be used as backfill behind the walls. All granular fill should be compacted in lifts of loose thickness not greater than 200 mm to 95 per cent of the material's Standard Proctor maximum dry density.

- Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill.

- The granular fill may be placed either in a zone with width equal to at least 1.2 m behind the back of the stem (Case I) or within the wedge-shaped zone defined by a 60 degree line extending up and back from the bottom of the rear face of the footing (Case II).

- 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.

- For Case I, the pressures are based on the in-situ soils/embankment fill materials and the following parameters (unfactored) may be assumed:

Soil unit weight	20 kN/m^3	(assuming the in-situ soils/embankment fill are comprised of loose granular and/or clean earth fill)

Coefficients of lateral earth pressure:

'active'	0.33
'at rest'	05

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

-

	Granular A	Granular B
Soil unit weight	22 kN/m^3	21 kN/m^3
Coefficients of lateral earth pressure		
'active'	0.27	0.31
'at rest'	0.43	0.47

It should be noted that the above design parameters assume level backfill and ground surface behind the wall.

8.3.3 Excavations

It is understood that the base of the pile cap will be at Elevation 183.7 m and at Elevation 184.2 m at the west and east abutments, respectively, and at about Elevation 178 m and 178.5 m at the piers. The final road grade will be at about Elevation 192 m on the west side of the crossing and at approximately Elevation 192.5 m on the east side of the crossing.

At the abutments, it is assumed that general stripping to the subgrade level and slope grading would be carried out prior to pile installation. The excavations for the pile cap construction at the west abutment will, therefore, extend to about 4 m below the grade and will be through the loose granular deposit. The base of excavation will most likely be above the groundwater level. Excavations which will be open for relatively short period of time can be made in temporary unsupported cut with side slopes maintained not steeper than 1.5 horizontal to 1 vertical. Some groundwater flow from perched levels within the granular deposits may occur; it is anticipated that the flow can be controlled by pumping from properly filtered sumps.

At the piers, the excavation will extend up to 2 m below the existing ground surface. The base of the excavation will extend below the groundwater level and water flow into excavations made through the granular deposits will be substantial. In general, the sands are sensitive to disturbance and where they form the excavation base or side walls below the groundwater level, the soils undergo rapid loosening due to upward water seepage. Some form of groundwater lowering or control will be required. The use of closed steel sheetpiling can be used as temporary support to the excavation and to form a cut-off to groundwater flow. The cut-off wall must extend to sufficient depth below the base of the excavation to minimize piping of the sands forming the base. Provision for pumping through properly filtered sumps at the base of excavation must be made. Alternatively, pumping from deep wells screened within the granular deposits to lower the groundwater level to below the base of the pier pile cap elevation should be considered.

Care should be taken to direct surface water away from the open excavation. All excavations should be carried out in accordance with the current Occupational Health and Safety Act.

8.3.4 Permanent Cuts and Approach Embankment

The proposed road grade at the crossing will involve approximately 4 m cut below the existing ground surface on the west side of the Garden River valley (from Elevation 196 m to Elevation 192 m approximately) and about 3.5 m cut on the west side of the Garden River valley (from about Elevation 196 m to about Elevation 192.5 m). Based on the previous and recent subsurface information, the cuts will generally be made through loose to compact sand and silty sand deposits and will be above the groundwater level. Permanent cut slopes formed at 2 horizontal to 1 vertical are considered suitable for the site.

Given the current condition of the valley bank on the west side of the river (on-going erosion and surficial slope Instability), it is recommended that provision be made for some regrading of the slope. In addition, some rip-rap placement at the slope toe may be required depending on the final layout and configuration of the west pier.

On the east side of the river the proposed highway will be about 12 m to 13 m above the floodplain level and an approach embankment will be required. Side slopes of the approach embankments may be formed at 2 horizontal to 1 vertical; given the height of the embankment, a 2 m bench formed at mid-height is considered appropriate.

The approach embankment will undergo some settlement due to consolidation of the loose sands and silts underlying the site. The settlement due to consolidation of the sands and silts is expected to occur during construction of the embankment.

8.3.5 Subgrade Preparation and Embankment Construction

In general, topsoil and organic deposits should be stripped from below the fill embankment areas and all subgrade soils should be proof-rolled prior to fill placement. The subgrade consists of sand and silty sand. There are likely to be areas where the groundwater level is at shallow depth. Pumping and rutting of the subgrade will occur with extensive traffic over the subgrade.

Construction of the embankment above the prepared subgrade may be carried out using clean earth fill or granular fill. Clean earth fill (Select Subgrade Material, SSM or Select Borrow Material, SBM) should be free of topsoil, organics, rubble, cobble sizes greater than 150 mm or other deleterious materials and have a moisture content at the time of placement within 2 per cent of the material's optimum moisture content. All embankment fills should be placed in regular lifts with loose thickness not exceeding 300 mm, and be compacted to at least 95 per cent of the material's Standard Proctor maximum dry density. The final lift prior to placement of the granular subbase or base course should be compacted to 100 per cent of the Standard Proctor maximum dry density. Inspection and field density testing should be carried out by qualified geotechnical personnel during all fill placement operations to ensure that appropriate materials are used and that adequate levels of compaction have been achieved.

9.0 NOONDAY ROAD UNDERPASS

9.1 Site and Project Description

The proposed Noonday Road underpass is located some 300 m east of the existing Noonday Road. The ground surface across the site is relatively flat and poorly drained with ground ranging from Elevation 193.0 m to 193.5 m.

As shown on the general arrangement plan provided June 11, 1997, the Noonday Road underpass will be a single span structure about 50 m in length. The abutment pile caps are indicated at about Elevation 195.5 m. The proposed new Noonday Road at the underpass will be at about Elevation 200 m and the proposed Highway 17 (New) grade will be at about Elevation 193.5 m, at about the existing ground surface.

9.2 Subsurface Conditions

Four boreholes (97-9 to 97-11 and 97-14) were put down at the proposed underpass site during the current investigation and one borehole (95-8) was drilled during our preliminary investigation carried out in 1995. Boreholes 95-9 to 97-11 were advanced to depths of 18.7 m to 30.5 m. In addition, a dynamic cone was driven from ground surface to about 21 m depth to confirm the relative density of the sands it was thought that since the Standard Penetration Test 'N' values were being impacted by blowing sand due to water pressure. Practical refusal to driving the dynamic cone was met at 21 m depth, mainly due to build up of skin friction on the drill rod through the sands. To confirm the presence of dense strata below a depth of 20 m, subsequent drilling was carried out with soil sampling and dynamic cone testing between 22.8 m and 43.9 m depth.

In summary, the subsoils consist of an extensive cohesionless deposit extending to the depth investigated.

9.2.1 Upper Interlayered Granular Deposits

A very loose to compact interlayered granular deposit consisting of sands, silty sand, silt and sandy silt was encountered to a depth of about 6 m to 8 m below the ground surface (from Elevation 187.8 m to Elevation 185 m). The upper 1.5 m to 2 m of the deposit is interlayered with amorphous peat. Below this depth, trace organic matter (rootlets) was noted in the deposit. The deposit is brown and grey in colour.

Standard Penetration Test (SPT) 'N' values measured in these deposits range from less than 1 (weight of hammer) to 12 blows for 0.3 m of penetration indicating a very loose to compact state of packing. The measured natural water contents of selected samples range from about 24 to 34 per cent. One higher value of 69 per cent was measured within the upper portion of the deposit where a peat layer was present. A grain size distribution test carried out on one sample of this deposit is shown on Figure 11.

9.2.2 Lower Granular Deposits

The layered deposit is underlain by a coarser grained deposit ranging in composition from sand with some gravel to sand and gravel and containing trace to some silt and occasional silt interlayers. Between about 8 m and 12 m depth, the deposit is in a compact to very dense state of packing. The sand becomes finer and looser between 12 m and 17 m depth (between Elevations 181 m and 176 m). Below 17 m depth, the granular deposit is in a compact to dense state of packing to about 36 m depth (Elevation 157 m). About 3.5 m of very stiff silty clay underlies the sand, which in turn is underlain by an interlayered deposit of dense sand and hard silty clay which extends to the depth investigated of 43.9 m (Elevation 149.6 m).

The results of grain size distribution tests carried out on two samples of the deposit are shown on Figures 12 and 13.

9.2.5 Groundwater Conditions

Details of the piezometer installations and the water level recorded in the piezometers are shown on the Record of Borehole sheets. The water level in piezometers installed in Borehole 97-9 and Borehole 97-10 were at Elevation 181.5 m and 181.8 m (about 11 m below ground surface). However, there is evidence of an upper perched water table close to ground surface within the upper interlayered sands. It should be noted that the water levels are subject to seasonal fluctuations.

9.3 Engineering Recommendations

This section of the report provides our recommendations on the geotechnical aspects of design of the Noonday Road bridge based on our interpretation of the factual information obtained during our recent and previous field investigations and, where pertinent, from previous investigations. It should be noted that the interpretation and recommendations are intended for use only by the design engineer and the information may not be sufficient for construction. Where comments are made on construction they are provided only in order to highlight those aspects which could affect the design of the project. Those requiring information on aspects of construction should make their own interpretation of the factual information provided as it may affect equipment selection, proposed construction method and scheduling.

The geotechnical recommendations contained herein relate to the design of the bridge foundations, approach embankment construction and permanent cut slopes and are based on information obtained from the plans and profiles provided by McCormick Rankin Corporation.

9.3.1 Bridge Foundations

The general arrangement plan provided June 11, 1997, indicates that the Noonday Road Underpass will be a single span structure with a span length of 50 m. The proposed Highway 17 (New) grade will be at about Elevation 193.5 m, at about the existing ground surface. The proposed Noonday Road grade will be at Elevation 200 m. The base of the abutment pile caps is indicated at about Elevation 195.5 m. MSE walls will be constructed at both abutments with a proposed founding level at Elevation 192 m.

The subsoils encountered in the boreholes put down at this site are not suitable for support of shallow spread footings, therefore, deep foundations are recommended. Consideration can be given to the use of closed-end pipe piles and steel H-piles. Based on our interpretation of the information obtained, the recommended values of factored geotechnical resistance at ULS for closed-end pipe pile 324 mm in diameter or HP310x79 piles driven to different depths are as follows:

	Factored Geotechnical Resistance for Driven Pile Length of:					
	29 m	**34 m**	**37 m**	**41 m**	**44 m**	**48.5 m**
Closed-end pipe pile 324 mm OD or HP 310x79	500	630	700	730	890	990
Pile terminated in soil: (embedded length)	sand	sand	sand	2.5 m in stiff to very stiff silty clay	Interlayered dense sand and hard clay	

The geotechnical resistance at Serviceability Limit States (SLS) is dependent on the settlement of the pile group and, therefore, is governed by the size of the pile group. Assuming that the pile group has a configuration of 10 or more piles arranged in two rows placed 0.7 m apart, with the piles in each row staggered such, that the distance between the piles in the front and back rows is approximately 1.6 m, a geotechnical resistance at SLS of 700 kN can be used for design. The above SLS value refers to 25 mm of settlement for the pile group.

A resistance factor of 0.6 was used to obtain the factored geotechnical resistance, which assumes that pile load testing (static test) will be carried out during construction. Where dynamic testing consisting of strain gauge measurements of acceleration and strain induced in the pile is carried out, a resistance factor of 0.5 should be used for design. A resistance factor of 0.4 would have to be used where there is no confirmatory field testing of the piles. Given the sensitivity of the pile capacity, based to a large degree on shaft friction, it would be preferable to utilize dynamic monitoring in addition to one static load test for confirmation of pile capacity to permit assessment of variations in performance of the piles across the site.

For resistance to lateral loads, the horizontal reaction to the pile can be calculated from the expression:

$$k_s = z \times n_h / d,$$

where

k_s = coefficient of horizontal subgrade reaction (MPa/m),
d = pile diameter (m),
n_h= constant of horizontal subgrade reaction (MPa/m),
z = depth (m).

The constant of horizontal subgrade reaction depends on the soil type and soil density/consistency surrounding the pile shaft. For design of resistance to lateral loads, the values (or range of values) given in the table below may be assumed:

Elevation (m)	**Soil Type**	**$z \times n_h$ (MPa)**
195.5 to 187.0	Sand , very loose to loose	z x 1.0
187.0 to 186.0	Sands, compact	z x 4.0
186.0 to 181.0	Sand/sand and gravel, compact to dense	z x 4.0 to z x 6.0
181.0 to 176.0	Sand, very loose to loose	z x 1.0
176.0 to 174.0	Sand, compact	z x 4.0
174.0 to 170.5	Sand, very loose to compact	z x 1.0 to z x 4.0
170.5 to 161.0	Sand, compact to dense	z x 4.0 to z x 6.0

161.0 to 157.0	Sand, compact	z x 4.0
157.0 to 154.0	Silty clay, stiff to very stiff	4.0 (constant with depth)
154.0 to 149.6	Sand and silty clay, dense to very dense/hard	10.0 (constant with depth)

Group action for lateral loading should be considered when the pile spacing in the direction of loading is less than 6 to 8 pile diameters. Group action can be evaluated by reducing the coefficient of lateral subgrade reaction in the direction of loading by a reduction factor R as follows:

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

9.3.2 Lateral Earth Pressures

The lateral pressures acting on the bridge abutments and the retaining walls 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:

- Selected free-draining granular fill meeting the specifications of OPSS Granular A or B should be used as backfill behind the walls. All granular fill should be compacted in lifts of loose thickness not greater than 200 mm to 95 per cent of the material's Standard Proctor maximum dry density.

- Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill.

- The granular fill may be placed either in a zone with width equal to at least 1.2 m behind the back of the stem (Case I) or within the wedge-shaped zone defined by a 60 degree line extending up and back from the bottom of the rear face of the footing (Case II).

- 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.

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

Soil unit weight	20 kN/m^3	(assuming that clean earth fill is used for construction)
Coefficients of lateral earth pressure:		
'active'	0.33	
'at rest'	0.5	

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

	Granular A	**Granular B**
Soil unit weight	22 kN/m^3	21 kN/m^3
Coefficients of lateral earth pressure		
'active'	0.27	0.31
'at rest'	0.43	0.47

It should be noted that the above design parameters assume level backfill and ground surface behind the wall.

9.3.3 Excavations

It is assumed that the base of the abutment pile caps will be at Elevation 195.5 m, up to 2 m above the existing ground surface. The pile caps will be founded within the embankment fill. The proposed MSE walls at the bridge abutments will be founded at approximately Elevation 192 m.

Excavations for retaining wall construction will extend to about 1.4 m below the existing ground surface and will be carried out through very loose to compact sands interlayered with peat and silts. The base of excavation will most likely be below the upper perched water table and some minor water flow into excavations should be expected.

Care should be taken to direct surface water away from the open excavation. All excavations should be carried out in accordance with the current Occupational Health and Safety Act.

9.3.4 Approach Embankments

The proposed grade of Highway 17 at the underpass will be at about Elevation 200 m and approach embankments about 6.5 m high will be required on both sides of the bridge structure.

The approach embankments will undergo some settlement due to consolidation of the loose to compact sands; however, the settlement is expected to occur during construction of the embankment. Long term settlements after road construction is anticipated to be negligible.

9.3.5 Subgrade Preparation and Embankment Construction

In general, topsoil and organic deposits should be stripped from below the fill embankment areas and all subgrade soils should be proof-rolled prior to fill placement. The subgrade consists of sand and silty sand. There are likely to be areas where the groundwater level is at shallow depth. Pumping and rutting of the subgrade will occur with extensive traffic over the subgrade.

Construction of the embankment above the prepared subgrade may be carried out using clean earth fill or granular fill. Clean earth fill (Select Subgrade Material, SSM or Select Borrow Material, SBM) should be free of topsoil, organics, rubble, cobble sizes greater than 150 mm or other deleterious materials and have a moisture content at the time of placement within 2 per cent of the material's optimum moisture content. All embankment fills should be placed in regular lifts with loose thickness not exceeding 300 mm, and be compacted to at least 95 per cent of the material's Standard Proctor maximum dry density. The final lift prior to placement of the granular subbase or base course should be compacted to 100 per cent of the Standard Proctor maximum dry density. Inspection and field density testing should be carried out by qualified geotechnical personnel during all fill placement operations to ensure that appropriate materials are used and that adequate levels of compaction have been achieved.

Embankment side slopes formed at 2 horizontal to 1 vertical or flatter are considered suitable for embankments constructed of clean earth fill as indicated above.

10.0 LOWER ECHO RIVER CROSSING

10.1 Site and Project Description

The Lower Echo River valley at the proposed Highway 17 (New) crossing is approximately 6 m in height with the top of the valley at about Elevation 178.6 m on the north side and at Elevation 179.2 m at the south side of the river. The valley slopes are relatively flat.

It is understood that Highway 17 (New) will be carried over Lower Echo River by a single span twin structure about 50 m in length. As shown on the general arrangement plan provided June 13, 1997, the final road grade will be at about Elevation 181.4 m, approximately 2.5 m above the existing ground surface.

10.2 Subsurface Conditions

One borehole (Borehole 97-12) and three piezocone tests (CPT-10, CPT-11 and CPT-16) were put down at the proposed crossing site during the current investigation and one borehole (Borehole 95-9) was drilled during our previous investigation carried out in 1995. In addition, there is one borehole (numbered 4) on the south side of Lower Echo River close to the proposed Highway 17 centreline which was put down as part of an investigation by MTO in the spring of 1975.

In Borehole 97-12 put down on the south side of the river, soil sampling and testing was carried out to a depth of 45.4 m and dynamic cone penetration testing was carried out to refusal at a depth of 50.3 m where practical refusal was met. The previous Borehole 95-9 was put down on the north side of the river to a depth of approximately 40 m below ground surface. The piezocones were advanced to refusal depths between 43.6 m and 46.2 m.

In general, the soils encountered in the boreholes and interpreted from the piezocone tests consist of about 0.2 m of topsoil overlying a thin veneer of granular soil up to 0.6 m thick, which is in turn underlain by an extensive deposit of lacustrine silty clay. The silty clay was encountered to the full depth investigated. The deposit is irregularly layered with silty clay, clayey silt and silt. The colour of this deposit is typically brown to greyish brown.

Standard Penetration Test (SPT) 'N' values measured in this deposit range from less than 1 blow (weight of rods) to 4 blows for 0.3 m of penetration. In situ vane shear tests carried out within this deposit indicate that the undrained shear strength increases with depth from about 20 kPa to

90 kPa at about 40 m depth. These values indicate that the deposit has a soft to firm consistency. Higher undrained shear strengths (up to 200 kPa) were inferred from the piezocone testing below about Elevation 137 m (42 m depth) on both sides of the river indicating a very stiff to hard consistency. Remoulded strengths range from about 3 kPa to 57 kPa corresponding to a sensitivity between 2 and 7; however, typically between 2 and 3.5, indicating that some portions of the deposit are sensitive, however, typically being between 2 and 3.5. The profile of shear strength of the silty clay with depth is shown on Figure 16. The results of grain size distribution tests are presented on Figure 14.

The measured natural water contents of selected samples of this deposit range from 33 per cent to 88 per cent. Atterberg limit tests performed on selected samples of the clay deposit indicate liquid limits between 45 and 84 per cent and plastic limits between 22 and 37 per cent. In order to determine the stress history, strength and compression characteristics of the deposit, oedometer testing was carried out on four silty clay samples; two samples from Borehole 97-12 and two samples from Borehole 95-9. The results of the consolidation tests indicate that the deposit is normally consolidated to slightly overconsolidated. The compression indices for the samples tested range from 0.32 to 1.0. The results of oedometer tests are included in Appendix C and summarized in Table 2.

Details of the piezometer installations and the water levels recorded in the piezometers are shown on the Record of Borehole sheets. The water level in the piezometer installed in Borehole 97-12, with tip at about 30 m depth, was measured at ground surface (Elevation 179.2 m) on March 29, 1997. The water level in the piezometer installed in Borehole 95-9, with tip at about 38.4 m depth, was measured at 1.3 m above the ground surface on October 5, 1995; however, when measured on February 12, 1997, the water level was at 11.4 m depth (Elevation 167.1 m). It should be noted that the water levels are subject to seasonal fluctuations.

10.3 Engineering Recommendations

This section of the report provides our recommendations on the geotechnical aspects of design of the works at the Lower Echo River bridge based on our interpretation of the factual information obtained during our field investigations and, where pertinent, from previous investigations carried out by others. It should be noted that the interpretation and recommendations are intended for use only by the design engineer and the information may not be sufficient for construction. Where comments are made on construction they are provided only in order to highlight those aspects which could affect the design of the project. Those requiring information on aspects of construction should make their own interpretation of the factual information provided as it may affect equipment selection, proposed construction method and scheduling.

The geotechnical recommendations contained herein relate to the design of the bridge foundations and approach embankment construction and are based on information obtained from the plans and profiles provided by McCormick Rankin Corporation.

10.3.1 Bridge Foundations

The general arrangement plan provided on June 13, 1997, indicates that the top of the abutment pile caps will be at Elevations 176.0 m and 176.2 m, on the north and south sides of the river, respectively. It is assumed that the base of the pile caps will be at Elevations 175.0 m and 175.2 m, respectively, some 2.5 m to 3 m below the existing ground surface.

The subsoils at the site, as encountered in the boreholes and inferred from the piezocone tests, are not suitable for the support of shallow spread footings; therefore deep foundations are recommended for the support of the abutments. Given the extensive depth of soft to stiff lacustrine silty clay underlying the site, consideration could be given to the use of piles driven to bear on the very stiff to hard interlayered stratum which was encountered at about Elevation 137 m.

Approach embankments are required on both sides of the proposed bridge structure. Under the surcharge from the approach embankment placed in the abutment area, downward movement of the silty clay around the pile will occur as it consolidates, and this downward movement will result in downdrag-forces or negative skin friction acting on the pile shaft. The negative skin friction acting on the pile will be about the same as the positive skin friction. Measures will have to be implemented to minimize or eliminate the negative skin friction.

10.3.1.1 Methods to Minimize Negative Skin Friction

It is understood that approach embankments about 2.5 m in height above the existing ground surface, will be required adjacent to both sides of the Lower Echo River bridge structure.

The silty clay deposit underlying the site is highly compressible and even nominal loading will result in consolidation settlement of the strata. Furthermore, surcharge from the approach embankment placed in the abutment area, will cause consolidation of the silty clay around the piles and downward movement resulting in negative skin friction acting on the pile shafts.

Negative skin friction will occur along the portion of the pile shaft where the displacement of the pile is smaller than that of the surrounding soil and mobilized shearing resistance is in a downward direction. The length of the pile affected is a function of the magnitude of the soil settlement. These frictional forces will increase the load transferred to the pile and thereby to the stratum beneath the pile tip.

Some measures should be undertaken to eliminate consolidation settlement of the soils around the pile and thereby minimize negative skin friction. The following means were investigated to assess feasibility of compensation for the load increase due to approach embankment construction:

- Preloading
- applying a surcharge load, and
- use of lightweight material (styrofoam) in combination with subexcavation.

Given the susceptibility of the deposit to consolidation under only nominal loading, consideration should be given to preloading and surcharging. The objective of preloading in excess of the final load is to increase the effective stress in the compressible stratum to a magnitude greater or equal to the effective stress under the final loading, thereby reducing or eliminating post construction settlement. If the effective stress can be increased sufficiently, post construction settlements may be largely eliminated. The consolidation settlements are time dependent and it is understood that the construction of the road embankment can be carried out with a 4 year delay time before paving is required. Preloading with a 5 m high embankment above the existing ground surface (i.e. 2.5 m surcharge) over a 4 year period time, will induce approximately 55 mm to 320 mm of consolidation settlement (with the range based on the range in assumed coefficient of compressibility as discussed in Section 10.3.3).

For the proposed final 2.5 m high approach embankment, without surcharging, it was calculated that approximately of 400 mm of settlement will occur due to primary consolidation; this settlement would occur over a period of more than 50 years without any prior measures taken to reduce settlement.

As noted above, up to 320 mm of settlement could be dissipated in a 4 year period (350 mm in 5 years) by preloading the site with a 5 m high embankment. However, without additional methods to enhance stability of the embankment adjacent to the river, the height of the preload embankment would be governed by the 3 horizontal to 1 vertical end slope of the embankment. Therefore, without additional methods to enhance stability, the preloading can only be constructed to about 1 m to 2 m height over the critical area immediately behind the abutment.

Surcharge/Preload Embankment

An assessment of the potential configuration of the surcharge load and distance from the abutment has been made. It was found that to compensate for the final embankment loading after removal of surcharge, subexcavation to 3 m depth below the original ground surface and replacement with 4.5 m of styrofoam (3 m within the subexcavation and 1.5 m forming the subgrade) overlain by 1 m pavement structure is required within the 3 m to 5 m distance behind the centreline of the abutments. This will compensate for the remaining primary consolidation settlement where only minor settlement was induced by the nominal preload. There may still be some extent of secondary settlement.

If the preload embankment can be constructed to 5 m high in the area of the abutments, only nominal subexcavation together with 1 m of styrofoam will be sufficient. Due to the proximity of the abutment to the river bank, a steepened end slope at 1 horizontal to 1 vertical (1:1) is required to allow 5 m embankment height at the abutment location. The use of geogrid reinforcement within the preload embankment would allow steepening of the end slope and serve to increase the stability of the embankment where it is in proximity to the river bank.

Slope stability analyses were carried out using the commercially available computer program SLOPE/W (version 3.0), following the general limit equilibrium method of analysis developed by Morgenstern and Price (1965) to determine the geogrid reinforcement extent which results in an adequate factor of safety for the slope. The analysis was based on a total stress approach involving fully saturated founding soils under undrained conditions. An adequate factor of safety against failure has been calculated for the 5 m high preload embankment with 1:1 end slope with provision of geogrid reinforcement extending 15 m back from the crest of the slope, typically, at 1.5 m spacing with the lower layer placed on the prepared subgrade.

After the preload embankment has been in place for 4 years, the remaining long term settlement of the proposed 2.5 m high embankment can be compensated for by placement of styrofoam to a height of about 1 m above the original ground surface. Due to the presence of the surficial organic deposits, this could require a total styrofoam height of up to about 1.5 m, assuming the organic deposit thickness does not exceed 0.5 m. The styrofoam placement should extend to about 5 m behind the back face of the pile cap.

10.3.1.2 Geotechnical Resistance for Pile Foundations

For design, a factored geotechnical resistance at Ultimate Limit States (ULS) of 850 kN may be assumed for HP 310x79 piles founded at or below Elevation 134 m. Negative skin friction will occur around the pile due to consolidation settlement induced by the approach embankment loading in the abutment area. For the 41 m length of pile driven to Elevation 134 m, a negative skin friction load of 1330 kN is calculated unless one of the methods of embankment construction described in Section 10.3.1.1 to eliminate negative skin friction are adopted.

A resistance factor of 0.6 was used to obtain the factored geotechnical resistance, which assumes that pile load testing (static test) will be carried out during construction. Where dynamic testing consisting of strain gauge measurements of acceleration and strain induced in the pile is carried out, a resistance factor of 0.5 should be used for design. A resistance factor of 0.4 would have to be used where there is no confirmatory field testing of the piles. Given the sensitivity of the pile capacity, based to a large degree on shaft friction, it would be preferable to utilize dynamic monitoring in addition to one static load test for confirmation of pile capacity to permit an assessment of potential variations in pile performance across the site.

The geotechnical resistance at Serviceability Limit States (SLS) is dependent on the settlement of the pile group and, therefore, is governed by the size of the pile group. It is understood that the integral abutment option is not feasible and that, therefore, the foundation layout would probably consist of two or more rows of piles. For the pile configuration consisting of two rows of piles with vertical back piles and front piles inclined at 1 horizontal to 3 vertical (rows at approximately 2 m spacing; piles at 2 m spacing), a geotechnical resistance at SLS of 600 kN can be assumed for design of piles driven to Elevation 134 m. The above geotechnical resistance at SLS was obtained assuming that the settlement induced by the approach embankment is eliminated as described in section 10.3.2.1. The elimination of the negative skin friction requires that settlement of the soil around the pile be less than or equal to the settlement of the pile under SLS loading (assumed 25 mm).

For resistance to lateral loads, the horizontal reaction to the pile can be calculated from the expression:

$$k_s = z \times n_h / d$$

where

k_s = coefficient of horizontal subgrade reaction (MPa/m),
d = pile diameter (m),
n_h= constant of horizontal subgrade reaction (MPa/m),
z = depth (m).

The constant of horizontal subgrade reaction depends on soil type and soil density/consistency. For resistance to lateral loads, the horizontal reaction to the pile should be assessed using the values for n_h times z as given below:

Elevation (m)	**z x n_h (MPa)**
175.0 to 170.0	2.0
170.0 to 137.0	3.0
137.0 to 134.0	5.0
below 134.0	10.0

Group action for lateral loading should be considered when the pile spacing in the direction of loading is less than 6 to 8 pile diameters. Group action can be evaluated by reducing the coefficient of lateral subgrade reaction in the direction of loading by a reduction factor R given below:

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

10.3.2 Lateral Earth Pressures

The lateral pressures acting on the bridge abutments and the retaining walls 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:

- Selected free-draining granular fill meeting the specifications of OPSS Granular A or B should be used as backfill behind the walls. All granular fill should be compacted in lifts of loose thickness not greater than 200 mm to 95 per cent of the material's Standard Proctor maximum dry density.

- Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill.

- The granular fill may be placed either in a zone with width equal to at least 1.2 m behind the back of the stem (Case I) or within the wedge-shaped zone defined by a 60 degree line extending up and back from the bottom of the rear face of the footing (Case II).

- 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.

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

Soil unit weight	20 kN/m^3	assuming the embankment fills are comprised of general earth fill (select fill) construction)

Coefficients of lateral earth pressure:

'active'	0.33
'at rest'	0.5

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

	Granular A	**Granular B**
Soil unit weight	22 kN/m^3	21 kN/m^3

Coefficients of lateral earth pressure		
'active'	0.27	0.31
'at rest'	0.43	0.47

It should be noted that the above design parameters assume level backfill and ground surface behind the wall.

10.3.3 Road Embankment Settlement

The final road grade at the crossing will be at about Elevation 181.3 m and approach embankments about 2 to 3 m in height will be required outside of the abutment areas on both sides of Lower Echo River. The settlement of the embankment at the abutments was discussed in Section 10.3.1.1. The silty clay and organic deposits are highly compressible and settlement of the road embankment will occur due to consolidation settlement of the underlying strata. Based on the field information obtained from the boreholes and piezocone tests in the area and laboratory testing carried out on the silty clay samples, it is estimated that approximately 350 mm and 550 mm of settlement for 2 m and 3 m high embankments, respectively, will occur due to primary consolidation of the silty clay deposit induced by the embankment loading without any prior measures to reduce settlement (see Figure 17). Time required to obtain 50 per cent and 90 per cent of consolidation is estimated to be approximately 10 years and 50 years, respectively. For this estimate, we have assumed that the embankments are 12 m wide (i.e. one direction of divided highway) and have 2 horizontal to 1 vertical side slopes.

A reduction in the time required to complete primary consolidation and reduction of the magnitude of consolidation settlement for the final road embankment can be achieved by one of the following methods:

- Preloading and applying a surcharge load,
- Light weight fill (such as fly ash) or styrofoam, and/or
- use of geosynthetic "wick"/strip drains

The use of lightweight fill material (outside of the bridge abutment areas) is not considered feasible for the road embankment at this site due to the limited height of the embankment and therefore limited thickness which can be replaced with lightweight fill. Alternatively, the time for the consolidation process could be shortened by use of geosynthetic "wick"/strip drains together with preloading. The spacing between the drains will control the rate of consolidation. Selection of drain spacing depends on the percent of consolidation required prior to start of road construction and economic considerations. A preliminary assessment of the drain requirements, indicates that the spacing of wick drains would have to be relatively small and extended to significant depth and therefore it is considered that wick drain use may lead to prohibitive construction costs. In addition, the high plastic and homogeneous nature of the clay deposit is such that the success of wick drain consolidation is not assured.

Given the susceptibility of the deposit to consolidation under only nominal loading, consideration should be given to preloading and surcharging. Preloading with a 5 m high embankment above the original ground surface (i.e. 2.5 m surcharge) over a 4 year period time, will induce approximately 55 mm to 320 mm of consolidation settlement (350 mm in 5 years) with the range based on the range in assumed coefficient of compressibility as discussed below.

Estimated settlements for 2 m to 3 m high embankments after application of surcharge (5 m high embankment) for four years are illustrated on Figure 17 and are summarized below:

Embankment Height	Settlement in time after completion of embankment (excl.4 years preloading time) in mm		
(m)	10 years	20 years	30 years
2	70	100	120
2.5	105	165	200
3	140	230	280

In the above estimates of the settlement, an average value of coefficient of the compressibility was assumed based on the laboratory test results obtained to date for the silty clay deposit. This assumption also applies to Figure 17 where we have estimated that 320 mm of settlement can occur in a 4 year period. As indicated previously, there is a range for this coefficient of compressibility given by the laboratory test results and the lower bound limit results in only 60 mm of settlement occurring in a 4 year period. The deposit is definitely variable and interlayered; therefore, it is likely that the settlements will occur faster than estimated by the lower bound assumption. However, this lower bound is presented to indicate the possible 'worst case' scenario.

As noted previously, the remaining long term settlement of the final 2.5 m high embankment can be compensated for by subexcavation of the preload fill immediately behind the abutment and placement of styrofoam within the embankment. The styrofoam height should be 1.5 m (to a height of 1 m above ground surface with 0.5 m subexcavation) and it should extend to 5 m behind the back face of the pile cap.

Monitoring of the embankments during the preload/surcharge period and subsequent to the final road construction should be carried out to confirm the rate and magnitude of settlement and to allow adjustment in design if required.

10.3.4 Subgrade Preparation and Embankment Construction

In general, topsoil and surficial organic deposits should be stripped from below the fill embankment areas and all subgrade soils should be proof-rolled prior to fill placement. The subgrade in this area is expected to consist of lacustrine silty clay. This clay deposit is extremely sensitive to disturbance by construction traffic and water flow. Measures to allow construction traffic and proper placement of embankment fills will likely be required and could consist of a rockfill layer or geogrid placed at the prepared subgrade level.

Construction of the embankment above the prepared subgrade may be carried out using clean earth fill or granular fill. Clean earth fill (Select Subgrade Material, SSM, or Select Borrow Material, SBM) should be free of topsoil, organics, rubble, cobble sizes greater than 150 mm or other deleterious materials and should have a moisture content at the time of placement within 2 per cent of the material's optimum moisture content. All embankment fills should be placed in regular lifts with loose thickness not exceeding 300 mm, and be compacted to at least 95 per cent of the material's Standard Proctor maximum dry density. The final lift prior to placement of the granular subbase or base course should be compacted to 100 per cent of the Standard Proctor maximum dry density. Inspection and field density testing should be carried out by qualified geotechnical personnel during all fill placement operations to ensure that appropriate materials are used and that adequate levels of compaction have been achieved.

Embankment side slopes formed at 2 horizontal to 1 vertical or flatter are considered suitable for the site where the embankments are constructed of clean earth fill as indicated above and provided that the organic materials are removed from within the embankment footprint.

The geogrid installed to enable the surcharge end slope to be built at a 1 to 1 slope should be extended to about 15 m back from the final crest of the embankment. When the surcharge is removed and the excavation for the abutment construction and styrofoam placement is carried out, the geogrid should be rolled back and retained. Once the abutment has been constructed and backfilled to original ground level, the geogrid should be reinstalled as the embankment fill is placed adjacent to the abutment to ensure the internal stability of the final slopes in the proximity of the river. With the geogrid retained, an end slope of 2 horizontal to 1 vertical may be feasible although the final configuration of the slope in relation to the rock protection, the sheet piling and the abutment wall should be examined to assess the workable length of geogrid within the final slope.

10.3.5 Excavations

The general arrangement plan indicates that the base of the abutment pile caps will be at about Elevation 175 m and that the ground surface at the river banks is at about Elevation 178.6 m to 178 m. Excavations for pile cap construction will extend to approximately 6 m depth from the top of the final approach embankment and about 2.5 m to 3 m below the original ground surface. The excavation for the pile cap construction will extend through the preload embankment fill and geogrid reinforcement and into the native silty clay deposit. The base of excavation will be below the groundwater level, which was measured in the piezometers to be at existing ground surface, and most likely below the river level.

Due to the possibility of encountering granular waterbearing layers within the upper portion of the lacustrine silty clay deposit, some form of water control will be required. Consideration could be given to the use of braced driven closed sheet piling for temporary support to the excavation and as the groundwater control measure. The lateral earth pressure distribution shown on Figure 18 may be used for design of a braced support system using the following design parameters:

$$\gamma = 17\ \text{kN/m}^3$$

$$\gamma_w = 9.8\ \text{kN/m}^3$$

$$K = 0.3$$

Design of the braced closed sheet piling for temporary support should include a check for heave at the base of the excavation; a value of undrained shear strength of 20 kPa can be assumed for design. The sheet piling must also be checked for vertical capacity using an undrained shear strength of 20 kPa or the shear strength profile as provided in Figure 16.

Some water flow into the sheetpile cofferdam excavation should be expected; it is anticipated that the flow can be controlled by pumping from properly filtered sumps. Care should be taken to direct surface water away from the open excavation.

All excavations should be carried out in accordance with the current Occupational Health and Safety Act.

GOLDER ASSOCIATES LTD.

ORIGINAL SIGNED BY

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SUMMARY OF BOREHOLE AND PIEZOCONE TEST LOCATIONS
GROUND SURFACE ELEVATION HIGHWAY 17 (NEW)

SITE	Borehole No.	Ground Surface Elevation (m)	Chainage (m)	Offset (m)	Northing	Easting
Root River	97-1	180.378	10+320.609	13.012 LT	228.762	217.012
	CPT-4	186.064	10+360.542	16.048 LT	251.533	250.326
	CPT-5	186.507	10+356.816	7.939 RT	228.816	258.885
	CPT-6	180.158	10+316.110	10.921 RT	205.887	225.364
Belleau Creek	97-2	184.139	11+102.972	0.280 LT	468.777	958.814
	CPT-1	182.665	11+187.521	14.752 LT	491.711	1041.804
	CPT-2	183.648	11+174.333	13.769 RT	462.205	1030.993
	CPT-3	183.838	11+120.284	13.022 LT	483.483	974.559
Jardin Mines	97-3	188.688	16+136.6	24.9 LT	273.187	5778.248
	97-13	188.690	16+140	2.5 LT	-	-
	CPT-7	188.512	16+122.5	27.6 RT	316.195	5745.000
	CPT-8	188.576	16+133	CL	294.775	5765.328
	CPT-9	188.820	16+139	31.3 LT	268.207	5782.929
Garden River	97-4	195.136	17+010.137	16.89 LT	629.502	6577.757
	97-5	195.319	17+004.820	12.839 RT	652.763	6558.493
	97-6	181.404	17+098.631	24.5 LT	669.195	6658.400
	97-7	180.610	17+086.386	11.553 RT	693.079	6628.728
	97-8	180.403	17+079.782	12.704 LT	669.010	6635.991
Noonday	97-9	193.424	17+651.360	30.06 LT	994.650	7107.494
	97-10	193.430	17+648.200	1.24 LT	1015.738	7087.598
	97-14	193.400	17+645.2	1.200	-	-
	97-11	192.933	17+647.337	33.256 RT	1042.739	7066.111
Echo River	97-12	179.160	25+784.090	22.908 LT	5081.835	13990.560
	CPT-10	178.976	25+716.879	19.366 LT	5034.055	13943.157
	CPT-11	178.674	25+708.815	20.8 RT	5054.776	13907.839
	CPT-12	178.297	25+500.124	3.194 LT	4957.728	13853.338
	CPT-13	177.565	25+501.662	4.470 LT	4883.427	13788.718
	CPT-14	177.562	25+401.292	2.471 LT	4809.884	13720.384
	CPT-15	177.654	25+302.422	2.371 LT	4735.862	13654.839
	CPT-16	178.305	25+772.234	19.157 RT	5101.004	13951.285
	14013.484	14013.484	14013.484	14013.484	14013.484	14013.484

TABLE 2
SUMMARY OF CONSOLIDATION DATA
HIGHWAY 17 (NEW)

Borehole No.	Sample No.	Depth (m)	Gamma kN/m^3	w_L (%)	I_p (%)	S (%)	C_c	C_r	Effective Overburden Pressure (kPa)	Effective Preconsolidation Pressure (kPa)
97-1	14	12.65	15.64	73.2	43.5	77.4	1.06	0.17	91	145
97-1	27	27.89	17.06	64.7	36.4	96.8	0.765	0.065	200	261
97-2	6	6.55	15.93	70.5	40.3	101.9	1.01	0.056	38.6	38.8
95-9	12	20	17.93	45.0	21.7	96.5	0.32	0.03	162	180
97-2	27	33.99	16.36	64.9	36.9	98.1	0.92	0.0525	215	230
95-9	20	39	17.30	47.4	22.3	95.8	0.33	0.06	120	145

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

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

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

III. SOIL DESCRIPTION

(a) Cohesionless Soils

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

(b) Cohesive Soils

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

IV. SOIL TESTS

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

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

LIST OF SYMBOLS

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

I. GENERAL

π = 3.1416

ln x, natural logarithm of x

$\log_{10}$ x or log x, logarithm of x to base 10

g acceleration due to gravity

t time

F factor of safety

V volume

W weight

II. STRESS AND STRAIN

γ shear strain

Δ change in, e.g. in stress: $\Delta\sigma$

ε linear strain

ε_v volumetric strain

η coefficient of viscosity

ν Poisson's ratio

σ total stress

σ' effective stress ($\sigma' = \sigma - u$)

σ'_{vo} initial effective overburden stress

$\sigma_1, \sigma_2, \sigma_3$ principal stresses (major, intermediate, minor)

σ_{oct} mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$

τ shear stress

u porewater pressure

E modulus of deformation

G shear modulus of deformation

K bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

$\rho(\gamma)$ bulk density (bulk unit weight*)

$\rho_d(\gamma_d)$ dry density (dry unit weight)

$\rho_w(\gamma_w)$ density (unit weight) of water

$\rho_s(\gamma_s)$ density (unit weight) of solid particles

γ' unit weight of submerged soil ($\gamma' = \gamma - \gamma_w$)

D_R relative density (specific gravity)of solid particles ($D_R = \rho_s/\rho_w$) (formerly G_s)

e void ratio

n porosity

S degree of saturation

* Density symbol is ρ. Unit weight symbol is γ 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_α coefficient of secondary consolidation

m_v coefficient of volume change

c_v coefficient of consolidation

T_v time factor (vertical direction)

U degree of consolidation

σ'_p pre-consolidation pressure

OCR Overconsolidation ratio $= \sigma'_p/\sigma'_{vo}$

(e) Shear Strength

τ_p, τ_r peak and residual shear strength

ϕ' effective angle of internal friction

δ angle of interface friction

μ coefficient of friction $= \tan\delta$

c' effective cohesion

c_u, s_u undrained shear strength ($\phi = 0$ analysis)

p mean total stress $(\sigma_1 + \sigma_3)/2$

p' mean effective stress $(\sigma'_1 + \sigma'_3)/2$

q $(\sigma_1 - \sigma_3)/2$ or $(\sigma'_1 - \sigma'_3)/2$

q_u compressive strength $(\sigma_1 - \sigma_3)$

S_t sensitivity

Notes: 1. $\tau = c' + \sigma' \tan\phi'$

2. Shear strength = (Compressive strength)/2

M1103001.BHS

PROJECT: 971-1103

RECORD OF BOREHOLE 97-1

LOCATION: STA. 10+320.61, o/s 13.01N, 228.76N, 217.01E

BORING DATE: FEB.6,7/97

DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

BORING METHOD: CME 55 TRACK MOUNT, 93mm I.D. HOLLOW STEM AUGERS

SOIL PROFILE

DESCRIPTION	ELEV. / DEPTH (m)
GROUND SURFACE	180.38 / 0.00
Sandy silt with fine to medium sand layers, trace organics. Very loose Brown	178.25 / 2.13
Sand, fine to medium, trace silt, Very loose Brown	177.33 / 3.05
Sand, fine to coarse Very loose Grey	176.72 / 3.66
Sand and gravel grading to sand, some gravel. Very loose Grey	174.74 / 5.64
Silt, trace to some sand, trace organics Layered Loose Grey	173.37 / 7.01
Silty clay to clay and clayey silt to silt, trace sand Irregularly layered Firm to stiff Greyish brown	
CONTINUED ON NEXT PAGE	

SAMPLES

NUMBER	TYPE	BLOWS/0.3m
1	50 DO	2
2	50 DO	2
3	50 DO	2
4	50 DO	1
5	50 DO	2
6	50 DO	1
7	50 DO	3
8	50 DO	5
9	50 DO	2
10	AS	
11	75 TO	PH

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m

SHEAR STRENGTH Cu, kPa — nat V - + Q - ● ; rem V - ⊕ U - ○ (20, 40, 60, 80)

HYDRAULIC CONDUCTIVITY, k, cm/s

WATER CONTENT, PERCENT — Wp, W, Wl (20, 40, 60, 80)

ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION: BENTONITE SEAL; SAND FILTER; CAVED

DEPTH SCALE 1 to 50

LOGGED: AOB

CHECKED: AMP

Golder Associates

DATA INPUT: ps JUNE 5/97

SOILM6

PROJECT: 971-1103

RECORD OF BOREHOLE 97-1

LOCATION: STA. 10+320.61, o/s 13.01N; 228.76N, 217.01E

BORING DATE: FEB.6,7/97

DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

CONTINUED FROM PREVIOUS PAGE

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE – DESCRIPTION	ELEV. DEPTH (m)	SAMPLE NUMBER	TYPE	BLOWS/0.3m	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
10–20	CME 55 TRACK MOUNT, 93mm I.D. HOLLOW STEM AUGERS	Silty clay to clay and clayey silt to silt, trace sand Irregularly layered Firm to stiff Greyish brown to brown						
				12	50 DO	1		
				13	AS			
				14	75 TO	PH	MH C ≈16kN/m³	CAVED
				15	AS			
				16	50 DO	4		
				17	AS			
				18	AS			
				19	75 TO	PH		BENTONITE SEAL
				20	50 DO	5		
				21	75 TO	PH		

Chart columns: DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m; SHEAR STRENGTH Cu, kPa (nat V - +, rem V - ⊕, Q - ●, U - ○; 20, 40, 60, 80); HYDRAULIC CONDUCTIVITY, k, cm/s; WATER CONTENT, PERCENT (Wp —○W— Wl; 20, 40, 60, 80)

CONTINUED ON NEXT PAGE

DEPTH SCALE 1 to 50

Golder Associates

LOGGED: AOB

CHECKED: AMP

M1103001.BHS

PROJECT: 971-1103

RECORD OF BOREHOLE 97-1

LOCATION: STA. 10+320.61, o/s 13.01N; 228.76N, 217.01E

BORING DATE: FEB.6,7/97

DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

Boring method: CME 55 TRACK MOUNT, 93mm I.D. HOLLOW STEM AUGERS

CONTINUED FROM PREVIOUS PAGE

Depth (m)	Description	Elev. / Depth (m)	Sample No.	Type	Blows/0.3m	Additional lab testing	Piezometer or standpipe installation
20–21.49			21	75 TO	PH		
			22	AS			
			23	AS			
21.49–22.56	Fine sand, some silt and silt, some sand to sandy silt. Interlayered Loose Brown	158.89 / 21.49	24	50 DO	7		
22.56–30	Silty clay to clay and clayey silt to silt, trace sand Irregularly layered Firm to stiff Brown	157.82 / 22.56	25	50 DO	5		SAND FILTER /CAVED MATERIAL
			26	75 TO	PH		
			27	75 TO	PH	MH C =17.1kN/m³	

CONTINUED ON NEXT PAGE

Chart columns: DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m; SHEAR STRENGTH Cu, kPa (nat V - +, rem V - ⊕, Q - ●, U - ○) 20 40 60 80; HYDRAULIC CONDUCTIVITY, k, cm/s; WATER CONTENT, PERCENT Wp — W — Wl 20 40 60 80

DATA INPUT: ps JUNE 5/97

SOILM6

DEPTH SCALE 1 to 50

Golder Associates

LOGGED: AOB

CHECKED: AMP

M1103001.BHS

PROJECT: 971-1103

RECORD OF BOREHOLE 97-1

LOCATION: STA. 10+320.61, o/s 13.01N; 228.76N, 217.01E

BORING DATE: FEB.6,7/97

DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES: NUMBER	TYPE	BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m; SHEAR STRENGTH Cu, kPa (nat V - +, rem V - ⊕, Q - ●, U - ○) 20 40 60 80	HYDRAULIC CONDUCTIVITY, k, cm/s; WATER CONTENT, PERCENT Wp W Wl 20 40 60 80	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
30		CONTINUED FROM PREVIOUS PAGE									
				149.90				⊕ +			
		END OF BOREHOLE		30.48							
31											NOTE: Water level in shallow piezometer at Elev. 178.3m and in deep piezometer at Elev. 178.0m on Feb.13, 1997 and at Elev. 178.3m in both piezometers on March 27, 1997.
32											
33											
34											
35											
36											
37											
38											
39											
40											

DATA INPUT ps JUNE 5/97

DEPTH SCALE
1 to 50

Golder Associates

LOGGED: AOB
CHECKED: AMP

SOILM6

M1103002.BHS

PROJECT: 971-1103

RECORD OF BOREHOLE 97-2

LOCATION: STA.11+102.972, o/s 0.28N; 468.777N,958.814E

BORING DATE: FEB.4,5/97

DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

BORING METHOD: CME 55 TRACK MOUNT, 93mm I.D. HOLLOW STEM AUGERS

SOIL PROFILE – DESCRIPTION	ELEV. / DEPTH (m)	SAMPLE NUMBER	TYPE	BLOWS/0.3m	PIEZOMETER OR STANDPIPE INSTALLATION
GROUND SURFACE	184.14				
TOPSOIL	0.00				BACKFILL
Sand, fine to coarse, some silt, trace organics, occ. oxidized fine sandy silt seam. Very loose Brown	0.10	1	50 DO	3	
		2	50 DO	3	BENTONITE SEAL
Silt, trace to some sand, trace clay, trace organics. Very loose Dark grey	182.01 / 2.13	3	50 DO	1	
		4	50 DO	2	
		5	50 DO	1	BACKFILL
Silty clay, trace sand Soft Brown	179.57 / 4.57	6	50 DO	1	
		7	50 DO	1	
		8	AS		
		9	75 TO	PH	
Sandy silt, trace to some clay. Loose, Grey	176.68 / 7.46				BENTONITE SEAL
Sand, fine to medium, trace silt, occ. clayey silt and sandy silt seams. Compact Grey	176.37 / 7.77	10	50 DO	18	
		11	50 DO	29	

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m

HYDRAULIC CONDUCTIVITY, k, cm/s

SHEAR STRENGTH Cu, kPa — nat V - + ; rem V - ⊕ ; Q - ● ; U - ○ — 20 40 60 80

WATER CONTENT, PERCENT — Wp ⊢—○W—⊣ Wl — 20 40 60 80

ADDITIONAL LAB. TESTING

Depth scale (metres): 0 1 2 3 4 5 6 7 8 9 10

CONTINUED ON NEXT PAGE

DEPTH SCALE
1 to 50

Golder Associates

LOGGED: AOB
CHECKED: AMP

DATA INPUT: ps april 3/97

SOILM6

M1103002 BHS

PROJECT: 971-1103

RECORD OF BOREHOLE 97-2

LOCATION: STA.11+102.972, o/s 0.28N; 468.777N,958.814E

BORING DATE: FEB.4,5/97

DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

Boring method: CME 55 TRACK MOUNT, 93mm I.D. HOLLOW STEM AUGERS

CONTINUED FROM PREVIOUS PAGE

DEPTH SCALE METRES	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLE NUMBER	TYPE	BLOWS/0.3m	WATER CONTENT	PIEZOMETER OR STANDPIPE INSTALLATION
10	Sand, fine to medium, trace silt, occ. clayey silt and sandy silt, occ. clay seams. Compact Grey						CAVED MATERIALS
11			12	50 DO	12		
12			13	50 DO	12	○	
14			14	50 DO	22		
15			15	50 DO	12	○	
17			16	50 DO	6		
18			17	50 DO	29	○	
20			18	50 DO	10	○	

Column headings: DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20, 40, 60, 80); SHEAR STRENGTH Cu, kPa (nat V - +, rem V - ⊕, Q - ●, U - ○); HYDRAULIC CONDUCTIVITY, k, cm/s; WATER CONTENT, PERCENT Wp —○W— Wl (20, 40, 60, 80); ADDITIONAL LAB. TESTING; STRATA PLOT.

CONTINUED ON NEXT PAGE

DATA INPUT: ps april 3/97

DEPTH SCALE 1 to 50

Golder Associates

LOGGED: AOB

CHECKED: AMP

SOLM6

PROJECT: 971-1103

RECORD OF BOREHOLE 97-2

SHEET 3 OF 3

LOCATION: STA.11+102.972, o/s 0.28N; 468.777N,958.814E BORING DATE: FEB.4,5/97 DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg; DROP, 760mm PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

M1103002 BHS

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. / DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.3m	SHEAR STRENGTH Cu, kPa (nat V - +, rem V - ⊕, Q - ●, U - ○)	WATER CONTENT, PERCENT (Wp, W, Wl)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
20	CME 55 TRACK MOUNT 93mm I.D. HOLLOW STEM AUGERS	CONTINUED FROM PREVIOUS PAGE									
		Sandy silt, trace clay. Compact Grey		164.03 / 20.11	18	50 DO	10		○ (~20)		SAND FILTER
		Silty clay to clay and clayey silt to silt, trace sand. Irregularly layered Soft to firm Brown		163.41 / 20.73	19	50 DO	2		○ (~35)		
					20	50 DO	2		○ (~58)		
					21	50 DO	5		○ (~58)		CAVED
					22	AS		⊕ +			
				157.93 / 26.21				⊕ +			
		END OF BOREHOLE									

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m

HYDRAULIC CONDUCTIVITY, k, cm/s

Depth scale marks: 20, 21, 22, 23, 24, 25, 26, 27, 28, 29, 30

NOTE: Water level in piezometer at Elev. 178.3m on Feb.13, 1997 and at Elev. 178.2m on March 27, 1997.

DATA INPUT: ps april 3/97

SOILM6

DEPTH SCALE 1 to 50

Golder Associates

LOGGED: AOB

CHECKED: AMP

M1103003.BHS

PROJECT: 971-1103

RECORD OF BOREHOLE 97-3

SHEET 1 OF 3

LOCATION: STA.16+136.6, o/s 24.9m; 278.187N, 5778.248E

BORING DATE: JAN.8,9/97

DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

Column headings: DEPTH SCALE METRES | BORING METHOD | SOIL PROFILE (DESCRIPTION, STRATA PLOT, ELEV. / DEPTH (m)) | SAMPLES (NUMBER, TYPE, BLOWS/0.3m) | DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (10, 20, 30, 40) | SHEAR STRENGTH Cu, kPa (20, 40, 60, 80), nat V - +, Q - ●, rem V - ⊕, U - ○ | HYDRAULIC CONDUCTIVITY, k, cm/s | WATER CONTENT, PERCENT, Wp —○W— Wl (20, 40, 60, 80) | ADDITIONAL LAB. TESTING | PIEZOMETER OR STANDPIPE INSTALLATION

Depth scale: 0 to 10 m

Boring method: CME 55 TRACK MOUNT, 93mm I.D. HOLLOW STEM AUGERS

Description	Elev. (m)	Depth (m)
GROUND SURFACE	188.69	0.00
TOPSOIL		0.15
Sand, fine, trace to some silt, trace organics. Very loose Brown and grey	187.32	1.37
Silty sand, trace organics. Loose Grey	186.56	2.13
Clayey silt, trace to some sand, occ. seams of silty clay, sand and sandy silt. Very soft Grey	185.03	3.66
Sand, fine to coarse, some gravel. Compact Grey	184.27	4.42
Sand, fine to medium, trace silt, Very loose Grey		
Sand and gravel Loose Grey	179.40	9.29
Silt to sandy silt, trace clay, trace sand. Loose Grey		9.45
CONTINUED ON NEXT PAGE		

Number	Type	Blows/0.3m	Additional Lab. Testing
1	50 DO	3	
2	50 DO	4	
3	50 DO	2	
4	50 DO	2	
5	50 DO	18	
6	50 DO	3	
7	50 DO	2	MH
8	50 DO	2	
9	50 DO	8	

Piezometer or standpipe installation: CAVED MATERIAL; BENTONITE SEAL; SAND FILTER

DATA INPUT: ps JUNE 5/97

DEPTH SCALE 1 to 50

Golder Associates

LOGGED: AOB

CHECKED: AMP

SOILM6

PROJECT: 971-1103

RECORD OF BOREHOLE 97-3

SHEET 2 OF 3

LOCATION: STA.16+136.6, o/s 24.9m; 278.187N, 5778.248E

BORING DATE: JAN.8,9/97

DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

M1103003.BHS

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES BLOWS/0.3m	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
10	CME 55 TRACK MOUNT; 93mm I.D. HOLLOW STEM AUGERS	CONTINUED FROM PREVIOUS PAGE		178.68					
		Sand, fine to coarse. Loose Grey		10.06					CAVED MATERIAL
11				177.72 / 10.97	10	50 DO	4		
12		Silty clay to clay and clayey silt, trace sand, occ. grey silt seams. Irregularly layered Soft to stiff Brown			11	50 DO	4		
13								MH	
14					12	75 TO	PH		
15					13	50 DO	5	MH	
16									
17					14	75 TO	PH		BENTONITE SEAL
18		Fine sand, silt and silty clay. Interlayered Loose to compact, soft to stiff. Brown to grey		170.71 / 17.98	15	50 DO	12		SAND FILTER
19									
20		CONTINUED ON NEXT PAGE			16	50 DO	8		

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m: 10 20 30 40

SHEAR STRENGTH Cu, kPa: nat V - + Q - ● rem V - ⊕ U - O; 20 40 60 80

HYDRAULIC CONDUCTIVITY, k, cm/s

WATER CONTENT, PERCENT: Wp —O W— Wl; 20 40 60 80

DATA INPUT: ps JUNE 5/97

SOILM6

DEPTH SCALE 1 to 50

Golder Associates

LOGGED: AOB

CHECKED: AMP

PROJECT: 971-1103

RECORD OF BOREHOLE 97-3

LOCATION: STA.16+136.6, o/s 24.9m; 278.187N, 5778.248E

BORING DATE: JAN.8,9/97

DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

Column headings: DEPTH SCALE METRES | BORING METHOD | SOIL PROFILE (DESCRIPTION, STRATA PLOT, ELEV. DEPTH (m)) | SAMPLES (NUMBER, TYPE, BLOWS/0.3m) | DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (10, 20, 30, 40) | SHEAR STRENGTH Cu, kPa (nat V - +, rem V - ⊕, Q - ●, U - O; 20, 40, 60, 80) | HYDRAULIC CONDUCTIVITY, k, cm/s | WATER CONTENT, PERCENT (Wp, W, Wl; 20, 40, 60, 80) | ADDITIONAL LAB. TESTING | PIEZOMETER OR STANDPIPE INSTALLATION

Boring method: CME 55 TRACK MOUNT, 93mm I.D. HOLLOW STEM AUGERS

CONTINUED FROM PREVIOUS PAGE

Description	Elev. / Depth (m)	Sample Number	Type	Blows/0.3m
(continued)	167.97 / 20.72	16	50 DO	6
Sand, fine to medium, trace silt. Compact Grey	165.52 / 23.17	17	50 DO	12
		18	50 DO	11
Silty clay to clay and clayey silt to silt, trace sand. Irregularly layered Firm Brown to red brown	163.85 / 24.84	19	50 DO	9
END OF BOREHOLE				

Depth scale: 20, 21, 22, 23, 24, 25, 26, 27, 28, 29, 30

SAND FILTER

NOTE:
Water level in deep piezometer at Elev. 185.2m and in shallow piezometer at Elev. 186.4m on Feb.13, 1997 and at Elev. 186.3m in both piezometers on March 27, 1997.

DATA INPUT ps JUNE 5/97

DEPTH SCALE
1 to 50

Golder Associates

LOGGED: AOB
CHECKED: AMP

SCILM6

RECORD OF BOREHOLE 97-4

PROJECT: 971-1103

LOCATION: STA.17+010.137,o/s 16.89N;629.502N,6577.757E

BORING DATE: FEB.10/97

SAMPLER HAMMER, 63.5kg; DROP, 760mm

DATUM: GEODETIC

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

Boring method: CME 75 BOMBADIER, 122mm I.D. HOLLOW STEM AUGERS

Depth scale (metres)	Description	Elev. / Depth (m)	Sample number	Type	Blows/0.3m	Water content (%)	Additional lab. testing	Piezometer or standpipe installation
0	GROUND SURFACE	195.14						
	TOPSOIL	0.00						
	Sand, fine to coarse, trace silt, trace gravel, becoming layered below 1.4m depth. Very loose to loose. Brown to grey	0.10						
1			1	50 DO	6			
2			2	50 DO	6			▽
			3	50 DO	3			
3	Silty sand to sandy silt, trace clay, trace rootlets. Very loose. Grey	192.25 / 2.89	4	50 DO	2			
5			5	50 DO	2			
6	Silt, some sand, sandy silt and sand, trace organics, trace clay. Interlayered. Very loose. Grey	189.35 / 5.79	6	50 DO	2			
8			7	50 DO	2		MH	
9			8	50 DO	WH			
10								

CONTINUED ON NEXT PAGE

Column headings: DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m; SHEAR STRENGTH Cu, kPa (nat V - +, rem V - ⊕, Q - ●, U - ○) 20 40 60 80; HYDRAULIC CONDUCTIVITY, k, cm/s; WATER CONTENT, PERCENT Wp —○W— Wl 20 40 60 80

DEPTH SCALE 1 to 50

LOGGED: DM

CHECKED: AMP

Golder Associates

M1103004.BHS

DATA INPUT: ps JUNE 5/97

SOILM6

M1103004.BHS

PROJECT: 971-1103

RECORD OF BOREHOLE 97-4

SHEET 2 OF 3

LOCATION: STA.17+010.137,o/s 16.89N;629.502N;6577.757E

BORING DATE: FEB.10/97

DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

Boring method: CME 75 BOMBADIER, 122mm I.D. HOLLOW STEM AUGERS

DEPTH SCALE METRES	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLE NUMBER	TYPE	BLOWS/0.3m	WATER CONTENT (approx. %)	ADDITIONAL LAB. TESTING
10	CONTINUED FROM PREVIOUS PAGE						
	Silt, some sand, sandy silt and sand, trace organics, trace clay. Interlayered. Very loose. Grey	183.56 / 11.58	9	50 DO	3	~25	
	Sand and gravel, trace silt, cobbles and boulders. Very dense. Brown and grey	180.97 / 14.17	10	50 DO	68	~10	
			11	50 DO	100 /.1	~10	
	Sand, fine to coarse, trace silt. Very loose to compact. Grey		12	50 DO	3	~18	
			13	50 DO	12	~21	M
			14	50 DO	WH	~20	
20	CONTINUED ON NEXT PAGE		15	50 DO	10		

Column headings also include: DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m; HYDRAULIC CONDUCTIVITY, k, cm/s; SHEAR STRENGTH Cu, kPa (nat V - +, Q - ●, rem V - ⊕, U - O) 20 40 60 80; WATER CONTENT, PERCENT Wp W Wl 20 40 60 80; STRATA PLOT; PIEZOMETER OR STANDPIPE INSTALLATION

DATA INPUT ps JUNE 5/97

SOILM6

DEPTH SCALE 1 to 50

Golder Associates

LOGGED: DM

CHECKED: AMP

M1103004.BHS

PROJECT: 971-1103

RECORD OF BOREHOLE 97-4

SHEET 3 OF 3

LOCATION: STA.17+010.137,o/s 16.89N;629.502N,6577.757E

BORING DATE: FEB.10/97

DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m / SHEAR STRENGTH Cu, kPa (nat V - +, rem V - ⊕, Q - ●, U - ○) 20 40 60 80	HYDRAULIC CONDUCTIVITY, k, cm/s / WATER CONTENT, PERCENT Wp ⊢—○W—⊣ Wl 20 40 60 80	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
20		CONTINUED FROM PREVIOUS PAGE									
	CME 75 BOMBADIER; 122mm I.D. HOLLOW STEM AUGERS				15	50 DO	10				
21		Sand, fine to coarse, trace silt. Very loose to compact Grey									
					16	50 DO	12		○ (approx. 16)		
22											
23				171.82 / 23.32	17	50 DO	15		○ (approx. 20)		
		END OF BOREHOLE									
24		NOTE: Practical auger refusal at 12.80m on boulder. Borehole moved 2m north and re-drilled without sampling to 13.7m depth.									NOTE: Water level in open borehole at Elev. 193.0m during drilling.
25											
26											
27											
28											
29											
30											

DATA INPUT. ps JUNE 5/97

SOILM6

DEPTH SCALE
1 to 50

Golder Associates

LOGGED: DM
CHECKED: AMP

M1103005.BHS

PROJECT: 971-1103

RECORD OF BOREHOLE 97-5

SHEET 1 OF 3

LOCATION: STA.17+004.82,o/s 12.84S; 652.763N,6558.493E

BORING DATE: FEB.9,10/97

DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

Column headings: DEPTH SCALE METRES | BORING METHOD | SOIL PROFILE (DESCRIPTION, STRATA PLOT, ELEV. / DEPTH (m)) | SAMPLES (NUMBER, TYPE, BLOWS/0.3m) | DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20, 40, 60, 80) | SHEAR STRENGTH Cu, kPa (nat V - +, Q - ●, rem V - ⊕, U - ○) | HYDRAULIC CONDUCTIVITY, k, cm/s | WATER CONTENT, PERCENT (Wp ⊢—○W—⊣ Wl; 20, 40, 60, 80) | ADDITIONAL LAB. TESTING | PIEZOMETER OR STANDPIPE INSTALLATION

Boring method: CME 55 TRACK MOUNT; 93mm I.D. HOLLOW STEM AUGERS

Elev. / Depth (m)	Description
195.32 / 0.00	GROUND SURFACE
195.32 / 0.00	Sand, fine to coarse, trace silt, occ. oxidized sand seams becoming layered below 1.4m depth. Loose. Brown to grey
192.88 / 2.44	Silty sand to fine sandy silt, trace clay, trace rootlets. Very loose to loose. Grey
188.61 / 6.71	Silt, sandy silt and sand, trace clay, trace organics. Very loose to loose. Grey

Sample Number	Type	Blows/0.3m
1	50 DO	7
2	50 DO	8
3	50 DO	7
4	50 DO	3
5	50 DO	2
6	50 DO	2
7	50 DO	2
8	50 DO	3
9	50 DO	3
10	50 DO	5

Piezometer or standpipe installation: BENTONITE SEAL (about 0–1.1 m); BACKFILL; BENTONITE SEAL (about 6.8–7.4 m)

Depth scale: 0, 1, 2, 3, 4, 5, 6, 7, 8, 9, 10

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DATA INPUT: PS JUNE 5/97

SOILM6

DEPTH SCALE
1 to 50

Golder Associates

LOGGED: AOB
CHECKED: AMP

PROJECT: 971-1103

RECORD OF BOREHOLE 97-5

SHEET 2 OF 3

LOCATION: STA.17+004.82,o/s 12.84S; 652.763N,6558.493E

BORING DATE: FEB.9,10/97

DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

Boring method: CME 55 TRACK MOUNT, 93mm I.D. HOLLOW STEM AUGERS

CONTINUED FROM PREVIOUS PAGE

DEPTH SCALE METRES	SOIL PROFILE DESCRIPTION	ELEV. / DEPTH (m)	SAMPLE NUMBER	TYPE	BLOWS/0.3m	WATER CONTENT, PERCENT (W)	PIEZOMETER OR STANDPIPE INSTALLATION
10–11.58	Silt, sandy silt and sand, trace clay, trace organics. Interlayered Very loose to loose Grey	183.74 / 11.58	11	50 DO	4	~47	
11.58–13.87	Sand and gravel, trace to some silt, trace cobbles and boulders. Very dense Grey	181.45 / 13.87	12	50 DO	88	~6	CAVED MATERIAL
13.87–17.67	Sand, fine to medium, trace silt. Loose to compact Brown to grey	177.65 / 17.67	13	50 DO	22		
			14	50 DO	7	~20	(water level ▽)
			15	50 DO	6		
17.67–19.20	Silty clay and fine sand Interlayered Loose/stiff Grey and brown	176.12 / 19.20	16	50 DO	8	~35	
19.20–20	Sand, fine to coarse, trace silt, trace gravel. Very loose Grey		17	50 DO	1		

CONTINUED ON NEXT PAGE

DEPTH SCALE 1 to 50

Golder Associates

LOGGED: AOB

CHECKED: AMP

M1103005.BHS

DATA INPUT: PS JUNE 5/97

SOILM6

PROJECT: 971-1103

RECORD OF BOREHOLE 97-5

LOCATION: STA.17+004.82,o/s 12.84S; 652.763N,6558.493E

BORING DATE: FEB.9,10/97

DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. / DEPTH (m)	SAMPLES: NUMBER	TYPE	BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m; SHEAR STRENGTH Cu, kPa (nat V - +, rem V - ⊕, Q - ●, U - O) 20 40 60 80	HYDRAULIC CONDUCTIVITY, k, cm/s; WATER CONTENT, PERCENT (Wp, W, Wl) 20 40 60 80	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
20	CME 55 TRACK MOUNT; 93mm I.D. HOLLOW STEM AUGERS	CONTINUED FROM PREVIOUS PAGE									
		Sand, fine to coarse, trace silt, trace gravel. Very loose Grey			17	50 DO	1				SAND FILTER
21					18	50 DO	23		O (approx. 20)		
22–23					19	50 DO	13				
24				170.48 / 24.84	20	50 DO	37				
25		END OF BOREHOLE									
26–30											

NOTE:
Water level in augers at Elev. 182.5m on completion of drilling.

Water level in piezometer at Elev. 181.0m on Feb.13, 1997 and at Elev. 180.12m on March 28, 1997.

DEPTH SCALE
1 to 50

Golder Associates

LOGGED: AOB
CHECKED: AMP

M1103006.BHS

PROJECT: 971-1103

RECORD OF BOREHOLE 97-6

SHEET 1 OF 3

LOCATION: STA.17+098.631,o/s 24.5N; 669.195N,6658.400E

BORING DATE: FEB.23/97

DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

Column headings: DEPTH SCALE METRES | BORING METHOD | SOIL PROFILE (DESCRIPTION, STRATA PLOT, ELEV. DEPTH (m)) | SAMPLES (NUMBER, TYPE, BLOWS/0.3m) | DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (40, 80, 120, 160) / SHEAR STRENGTH Cu, kPa (20, 40, 60, 80) nat V - + Q - ● rem V - ⊕ U - O | HYDRAULIC CONDUCTIVITY, k, cm/s / WATER CONTENT, PERCENT Wp —O W— Wl (20, 40, 60, 80) | ADDITIONAL LAB. TESTING | PIEZOMETER OR STANDPIPE INSTALLATION

Boring method: CME 55 TRACK MOUNT; 93mm I.D. HOLLOW STEM AUGERS

DESCRIPTION	ELEV. / DEPTH (m)
GROUND SURFACE	181.40 / 0.00
Sand, fine to medium, some silt, trace gravel, trace rootlets. Compact Brown	180.18 / 1.22
Sand, some silt and gravel, some cobbles and boulders (river bed). Dense Grey	179.27 / 2.13
Silty clay, some sand and fine sand, some silt, trace gravel. Interlayered Soft to firm/very loose to loose Brown	176.20 / 5.20
Sand, fine to medium, trace silt and gravel. Very loose to loose Grey	
CONTINUED ON NEXT PAGE	

NUMBER	TYPE	BLOWS/0.3m
1	50 DO	20
2	50 DO	36
3	50 DO	3
4	50 DO	4
5	50 DO	5
6	50 DO	1
7	50 DO	3
8	50 DO	4
9	50 DO	4
10	50 DO	6

Piezometer installation: BACKFILL; BENTONITE SEAL; SAND FILTER

DATA INPUT: ps june 5/97

SOILM6

DEPTH SCALE 1 to 50

Golder Associates

LOGGED: AOB

CHECKED: AMP

M1103006.BHS

PROJECT: 971-1103

RECORD OF BOREHOLE 97-6

SHEET 2 OF 3

LOCATION: STA.17+098.631,o/s 24.5N; 669.195N,6658.400E

BORING DATE: FEB.23/97

DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

Boring method: CME 55 TRACK MOUNT, 93mm I.D. HOLLOW STEM AUGERS

CONTINUED FROM PREVIOUS PAGE

DEPTH SCALE METRES	DESCRIPTION	ELEV. DEPTH (m)
10 – 16.91	Sand, fine to medium, trace silt and gravel. Very loose to loose Grey	
16.91 – 17.40	Silty clay, some sand, thin silt partings. Stiff Brown and grey	164.49 / 16.91
17.40 – 20	Sand, medium to coarse, trace silt, trace to some gravel, occ. cobble. Very dense Grey	164.00 / 17.40

SAMPLE NUMBER	TYPE	BLOWS/0.3m
11	50 DO	10
12	50 DO	8
13	50 DO	2
14	50 DO	3
15	50 DO	11
16	50 DO	68
17	50 DO	82

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m: 40 80 120 160

HYDRAULIC CONDUCTIVITY, k, cm/s

SHEAR STRENGTH Cu, kPa: nat V - + Q - ● rem V - ⊕ U - ○ — 20 40 60 80

WATER CONTENT, PERCENT: Wp —○W— Wl — 20 40 60 80

ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION: SAND FILTER; BENTONITE SEAL

CONTINUED ON NEXT PAGE

DEPTH SCALE 1 to 50

Golder Associates

LOGGED: AOB

CHECKED: AMP

DATA INPUT. ps june 5/97

SOILM6

PROJECT: 971-1103

RECORD OF BOREHOLE 97-6

LOCATION: STA.17+098.631,o/s 24.5N; 669.195N,6658.400E

BORING DATE: FEB.23/97

DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

Column headings: DEPTH SCALE METRES | BORING METHOD | SOIL PROFILE: DESCRIPTION, STRATA PLOT, ELEV. DEPTH (m) | SAMPLES: NUMBER, TYPE, BLOWS/0.3m | DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (40, 80, 120, 160); SHEAR STRENGTH Cu, kPa (nat V - +, rem V - ⊕, Q - ●, U - ○) (20, 40, 60, 80) | HYDRAULIC CONDUCTIVITY, k, cm/s; WATER CONTENT, PERCENT, Wp —○W— Wl (20, 40, 60, 80) | ADDITIONAL LAB. TESTING | PIEZOMETER OR STANDPIPE INSTALLATION

CONTINUED FROM PREVIOUS PAGE

Boring method: CME 55 TRACK MOUNT, 93mm I.D. HOLLOW STEM AUGERS

Description (20.0 m to 27.43 m): Sand, medium to coarse, trace silt, trace to some gravel, occ. cobble. Very dense. Grey

Sample Number	Type	Blows/0.3m
17	50 DO	82
18	50 DO	47
19	50 DO	85
20	50 DO	>50
21	50 DO	54

Elev. 153.97, Depth 27.43: END OF BOREHOLE

Piezometer: SAND FILTER

NOTE:
Water level at ground surface upon completion of drilling.

Water level in both piezometers at Elev. 181.4m on Feb.25, 1997.

Water level in shallow piezometer at Elev. 181.1m and deep piezometer tubing frozen at Elev. 181.7m (0.3m above ground surface) on March 27, 1997.

Depth scale: 20, 21, 22, 23, 24, 25, 26, 27, 28, 29, 30

DATA INPUT: ps June 5/97

DEPTH SCALE 1 to 50

LOGGED: AOB

CHECKED: AMP

PROJECT: 971-1103

RECORD OF BOREHOLE 97-7

SHEET 1 OF 3

LOCATION: STA.17+086.39, o/s 11,55S; 693.079N,6628.73E

BORING DATE: FEB.19/97

DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

Column headings: DEPTH SCALE METRES | BORING METHOD | SOIL PROFILE (DESCRIPTION, STRATA PLOT, ELEV. DEPTH (m)) | SAMPLES (NUMBER, TYPE, BLOWS/0.3m) | DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (40, 80, 120, 160) | SHEAR STRENGTH Cu, kPa (nat V - +, Q - ●, rem V - ⊕, U - O; 20, 40, 60, 80) | HYDRAULIC CONDUCTIVITY, k, cm/s | WATER CONTENT, PERCENT (Wp, W, Wl; 20, 40, 60, 80) | ADDITIONAL LAB. TESTING | PIEZOMETER OR STANDPIPE INSTALLATION

Boring method: CME 55 TRACK MOUNT, 93mm I.D. HOLLOW STEM AUGERS

Description	Elev. / Depth (m)
GROUND SURFACE	180.61 / 0.00
Sand, fine to medium, some gravel, occ. cobbles and boulders. Loose Grey	179.11 / 1.50
Sand, medium, trace gravel, trace silt. Very loose to loose Brown	175.41 / 5.20
Sand, medium, trace to little silt and gravel, occ. silty clay seam. Compact becoming very loose below 8.5m depth. Grey	
CONTINUED ON NEXT PAGE	

Sample Number	Type	Blows/0.3m
1	50 DO	2
2	50 DO	3
3	50 DO	4
4	50 DO	5
5	50 DO	2
6	50 DO	13
7	50 DO	19
8	50 DO	10
9	50 DO	2

Piezometer installation: BACKFILL; BENTONITE SEAL; CAVED MATERIAL

DEPTH SCALE 1 to 50

Golder Associates

LOGGED: AOB

CHECKED: AMP

M1103007.BHS

PROJECT: 971-1103

RECORD OF BOREHOLE 97-7

LOCATION: STA.17+086.39, o/s 11.55S; 693.079N,6628.73E

BORING DATE: FEB.19/97

DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m / SHEAR STRENGTH Cu, kPa	HYDRAULIC CONDUCTIVITY, k, cm/s / WATER CONTENT, PERCENT	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
10	CME 55 TRACK MOUNT 93mm I.D. HOLLOW STEM AUGERS	CONTINUED FROM PREVIOUS PAGE									
11					10	50 DO	2				
12		Sand, medium, trace to little silt and gravel, occ. silty clay seam. Compact becoming very loose below 8.5m depth. Grey			11	50 DO	2		w ≈ 20		
14					12	50 DO	1				
15					13	50 DO	2		w ≈ 20		CAVED MATERIAL
17					14	50 DO	3				
18					15	50 DO	13		w ≈ 20		
20		CONTINUED ON NEXT PAGE			16	50 DO	28				

Legend: Dynamic penetration resistance scale 40, 80, 120, 160; Shear strength: nat V - + Q - ●, rem V - ⊕ U - O; scale 20, 40, 60, 80. Water content: Wp —O W— Wl; scale 20, 40, 60, 80.

DATA INPUT: PS JUNE 5/97

SOILM6

DEPTH SCALE 1 to 50

LOGGED: AOB

CHECKED: AMP

Golder Associates

M1103007.BHS

PROJECT: 971-1103

RECORD OF BOREHOLE 97-7

SHEET 3 OF 3

LOCATION: STA.17+086.39, o/s 11.55S; 693.079N,6628.73E

BORING DATE: FEB.19/97

DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE – DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (40, 80, 120, 160); SHEAR STRENGTH Cu, kPa (nat V - +, Q - ●, rem V - ⊕, U - O) (20, 40, 60, 80)	HYDRAULIC CONDUCTIVITY, k, cm/s; WATER CONTENT, PERCENT (Wp, W, Wl) (20, 40, 60, 80)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
20	CME 55 TRACK MOUNT / 93mm I.D. HOLLOW STEM AUGERS	CONTINUED FROM PREVIOUS PAGE		160.51							
		Stratified silty clay, some sand, silt parting and sand, trace silt and clay. Interlayered Very stiff/dense Brown and grey		20.10	16	50 DO	28		O		
21				159.91 / 20.70							
		Sand, medium to coarse, some gravel, trace silt, occ. cobbles. Dense to very dense Grey			17	50 DO	34				
22											CAVED MATERIAL
23					18	50 DO	76				
24					19	50 DO	79		O		
25											
26				154.24	20	50 DO	99				
		END OF BOREHOLE		26.37							
27											
28											
29											
30											

NOTE:
Artesian water condition was noted 1 hr after completion of piezometer installation with water level up to 1m above ground surface.

Piezometer tubing frozen at Elev. 180.9m (about 0.3m above ground surface) on March 29, 1997.

DATA INPUT: PS JUNE 5/97

DEPTH SCALE
1 to 50

Golder Associates

LOGGED: AOB
CHECKED: AMP

SOLM6

M1103007.BHS

PROJECT: 971-1103

RECORD OF BOREHOLE 97-7

LOCATION: STA.17+086.39, o/s 11.55S; 693.079N,6628.73E

BORING DATE: FEB.19/97

DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

BORING METHOD: CME 55 TRACK MOUNT, 93mm I.D. HOLLOW STEM AUGERS

Column headings: DEPTH SCALE METRES | BORING METHOD | SOIL PROFILE (DESCRIPTION, STRATA PLOT, ELEV. DEPTH (m)) | SAMPLES (NUMBER, TYPE, BLOWS/0.3m) | DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (40, 80, 120, 160) | SHEAR STRENGTH Cu, kPa (nat V - + Q - ●, rem V - ⊕ U - ○; 20, 40, 60, 80) | HYDRAULIC CONDUCTIVITY, k, cm/s | WATER CONTENT, PERCENT (Wp —○W— Wl; 20, 40, 60, 80) | ADDITIONAL LAB. TESTING | PIEZOMETER OR STANDPIPE INSTALLATION

Soil Profile

Elev. / Depth (m)	Description
180.61 / 0.00	GROUND SURFACE
180.61 – 179.11 (0.00 – 1.50)	Sand, fine to medium, some gravel, occ. cobbles and boulders. Loose Grey
179.11 – 175.41 (1.50 – 5.20)	Sand, medium, trace gravel, trace silt. Very loose to loose Brown
175.41 / 5.20 –	Sand, medium, trace to little silt and gravel, occ. silty clay seam. Compact becoming very loose below 8.5m depth. Grey

Samples

Number	Type	Blows/0.3m
1	50 DO	2
2	50 DO	3
3	50 DO	4
4	50 DO	5
5	50 DO	2
6	50 DO	13
7	50 DO	19
8	50 DO	10
9	50 DO	2

Piezometer or Standpipe Installation

BACKFILL

BENTONITE SEAL

CAVED MATERIAL

CONTINUED ON NEXT PAGE

DEPTH SCALE 1 to 50

LOGGED: AOB

CHECKED: AMP

Golder Associates

DATA INPUT: PS JUNE 5/97

SOILM6

M1103007 BHS

PROJECT: 971-1103

RECORD OF BOREHOLE 97-7

SHEET 2 OF 3

LOCATION: STA.17+086.39, o/s 11.55S; 693.079N,6628.73E

BORING DATE: FEB.19/97

DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES: NUMBER	TYPE	BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (40, 80, 120, 160); SHEAR STRENGTH Cu, kPa (20, 40, 60, 80) nat V - + Q - ● rem V - ⊕ U - O	HYDRAULIC CONDUCTIVITY, k, cm/s; WATER CONTENT, PERCENT Wp —O W— Wl (20, 40, 60, 80)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
10	CME 55 TRACK MOUNT; 93mm I.D. HOLLOW STEM AUGERS	CONTINUED FROM PREVIOUS PAGE									
11					10	50 DO	2				
12		Sand, medium, trace to little silt and gravel, occ. silty clay seam. Compact becoming very loose below 8.5m depth. Grey			11	50 DO	2		O		
13											
14					12	50 DO	1				
15											CAVED MATERIAL
					13	50 DO	2		O		
16											
17					14	50 DO	3				
18					15	50 DO	13		O		
19											
20		CONTINUED ON NEXT PAGE			16	50 DO	28				

DEPTH SCALE
1 to 50

Golder Associates

LOGGED: AOB
CHECKED: AMP

DATA INPUT: PS JUNE 5/97

SOILM6

M1103008.BHS

PROJECT: 971-1103

RECORD OF BOREHOLE 97-8

SHEET 1 OF 3

LOCATION: STA.17+079.78, o/s 12.704N; 669.01N,6635.99E

BORING DATE: FEB.23/97

DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

Column headings: DEPTH SCALE METRES | BORING METHOD | SOIL PROFILE (DESCRIPTION, STRATA PLOT, ELEV. DEPTH (m)) | SAMPLES (NUMBER, TYPE, BLOWS/0.3m) | DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (40, 80, 120, 160) | SHEAR STRENGTH Cu, kPa (nat V - +, Q - ●, rem V - ⊕, U - ○; 20, 40, 60, 80) | HYDRAULIC CONDUCTIVITY, k, cm/s | WATER CONTENT, PERCENT (Wp ├──○W──┤ Wl; 20, 40, 60, 80) | ADDITIONAL LAB. TESTING | PIEZOMETER OR STANDPIPE INSTALLATION

Boring method: CME 55 TRACK MOUNT, 93mm I.D. HOLLOW STEM AUGERS

ELEV. / DEPTH (m)	DESCRIPTION
180.40	GROUND SURFACE
0.00	Sand, fine, some silt, trace gravel, occ. cobble and boulders. Loose Grey
178.73 / 1.67	Silty clay, some sand and fine sand, trace gravel, some silt. Firm/loose Grey
176.70 / 3.70	Sand, fine to medium, trace silt, trace gravel. Loose Grey

NUMBER	TYPE	BLOWS/0.3m
1	50 DO	6
2	50 DO	6
3	50 DO	5
4	50 DO	4
5	50 DO	2
6	50 DO	4
7	50 DO	2
8	50 DO	6
9	50 DO	8

Piezometer installation: BACKFILL; BENTONITE SEAL; SAND FILTER

CONTINUED ON NEXT PAGE

DEPTH SCALE 1 to 50

Golder Associates

LOGGED: AOB

CHECKED: AMP

DATA INPUT: PS JUNE 5/97

SOILM6

M1103008 BHS

PROJECT: 971-1103

LOCATION: STA.17+079.78, o/s 12.704N; 669.01N,6635.99E

SAMPLER HAMMER, 63.5kg; DROP, 760mm

RECORD OF BOREHOLE 97-8

BORING DATE: FEB.23/97

DATUM: GEODETIC

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

Boring method: CME 55 TRACK MOUNT, 93mm I.D. HOLLOW STEM AUGERS

DEPTH SCALE METRES	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLE NUMBER	TYPE	BLOWS/0.3m	WATER CONTENT, PERCENT	PIEZOMETER OR STANDPIPE INSTALLATION
10	CONTINUED FROM PREVIOUS PAGE						
11	Sand, fine to medium, trace silt, trace gravel. Loose Grey		10	50 DO	8	o	SAND FILTER
12			11	50 DO	4		
14			12	50 DO	5		
15		164.86 / 15.54	13	50 DO	11		
16	Silty clay, some sand, occ. silty sand seam. Stiff Brown	164.40 / 16.00					BENTONITE SEAL
17	Sand, medium to coarse, trace silt, trace to some gravel, occ. cobble and boulders. Dense to very dense Grey		14	50 DO	40		
18			15	50 DO	53	o	
20	CONTINUED ON NEXT PAGE		16	50 DO	63		

Column headings also include: BORING METHOD; STRATA PLOT; DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (40, 80, 120, 160); SHEAR STRENGTH Cu, kPa (nat V - + Q - ●, rem V - ⊕ U - O; 20, 40, 60, 80); HYDRAULIC CONDUCTIVITY, k, cm/s; WATER CONTENT, PERCENT (Wp ├──O W──┤ Wl; 20, 40, 60, 80); ADDITIONAL LAB. TESTING.

DEPTH SCALE

1 to 50

Golder Associates

LOGGED: AOB

CHECKED: AMP

DATA INPUT: PS JUNE 5/97

SOILM6

M1103008.BHS

PROJECT: 971-1103

RECORD OF BOREHOLE 97-8

SHEET 3 OF 3

LOCATION: STA.17+079.78, o/s 12.704N; 669.01N,6635.99E

BORING DATE: FEB.23/97

DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. / DEPTH (m)	SAMPLES: NUMBER	TYPE	BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (40, 80, 120, 160) / SHEAR STRENGTH Cu, kPa (nat V - +, rem V - ⊕, Q - ●, U - ○) (20, 40, 60, 80)	HYDRAULIC CONDUCTIVITY, k, cm/s / WATER CONTENT, PERCENT (Wp, W, Wl) (20, 40, 60, 80)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
20	CME 55 TRACK MOUNT; 93mm I.D. HOLLOW STEM AUGERS	CONTINUED FROM PREVIOUS PAGE									
		Sand, medium to coarse, trace silt, trace to some gravel, occ. cobble and boulders. Dense to very dense Grey			16	50 DO	63				SAND FILTER
21											
					17	50 DO	98				
22											
23					18	50 DO	100				
24					19	50 DO	70				
				155.56 / 24.84							
25		END OF BOREHOLE									
26											
27											
28											NOTE: Artesian water conditiond noted during drilling after advancing augers to 16m depth. Water level in open borehole at ground surface upon completion of drilling. Water level in both piezometers at Elev. 180.4m on Feb.24, 1997 and at Elev. 180m on March 29, 1997.
29											
30											

DATA INPUT: PS JUNE 5/97

DEPTH SCALE
1 to 50

SOILM6

Golder Associates

LOGGED: AOB
CHECKED: AMP

M1103009.BHS

PROJECT: 971-1103

RECORD OF BOREHOLE 97-9

SHEET 1 OF 3

LOCATION: STA. 17+651.36, o/s 30.06N; 994.65N,7107.49E

BORING DATE: FEB.11/97

DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

Column headings: DEPTH SCALE METRES | BORING METHOD | SOIL PROFILE (DESCRIPTION, STRATA PLOT, ELEV. / DEPTH (m)) | SAMPLES (NUMBER, TYPE, BLOWS/0.3m) | DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (10, 20, 30, 40) | SHEAR STRENGTH Cu, kPa (nat V - + Q - ●; rem V - ⊕ U - O) (20, 40, 60, 80) | HYDRAULIC CONDUCTIVITY, k, cm/s | WATER CONTENT, PERCENT (Wp ⊢—O W—⊣ Wl) (20, 40, 60, 80) | ADDITIONAL LAB. TESTING | PIEZOMETER OR STANDPIPE INSTALLATION

Boring method: CME 55 TRACK MOUNT, 93mm I.D. HOLLOW STEM AUGERS

Soil profile

Elev. (m)	Depth (m)	Description
193.42	0.00	GROUND SURFACE
		Sand, fine, to sandy silt and amorphous peat, some sand. Interlayered. Loose/soft to firm. Grey and brown
191.29	2.13	Sand, fine, some silt, occ. rootlets. Layered. Very loose. Grey
190.52	2.90	Sandy silt to sand and silt, trace clay, trace organics. Very loose. Grey
187.78	5.64	Sand, medium to coarse, some gravel. Layered. Compact to loose. Grey
		CONTINUED ON NEXT PAGE

Samples

Number	Type	Blows/0.3m	Additional Lab. Testing
1	50 DO	7	
2	50 DO	2	
3	50 DO	WH	
4	50 DO	1	
5	50 DO	WH	
6	50 DO	1	MH
7	50 DO	30	
8	50 DO	19	
9	50 DO	9	
10	50 DO	13	

Piezometer or standpipe installation: BENTONITE SEAL; CAVED MATERIAL

DEPTH SCALE 1 to 50

Golder Associates

LOGGED: AOB

CHECKED: AMP

DATA INPUT PS JUNE 5/97

SOILM6

PROJECT: 971-1103

RECORD OF BOREHOLE 97-9

LOCATION: STA. 17+651.36, o/s 30.06N; 994.65N,7107.49E

BORING DATE: FEB.11/97

DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

Column headings: DEPTH SCALE METRES | BORING METHOD | SOIL PROFILE: DESCRIPTION, STRATA PLOT, ELEV. DEPTH (m) | SAMPLES: NUMBER, TYPE, BLOWS/0.3m | DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (10, 20, 30, 40) | SHEAR STRENGTH Cu, kPa (nat V - +, Q - ●, rem V - ⊕, U - ○; 20, 40, 60, 80) | HYDRAULIC CONDUCTIVITY, k, cm/s | WATER CONTENT, PERCENT (Wp ⊢—○W—⊣ Wl; 20, 40, 60, 80) | ADDITIONAL LAB. TESTING | PIEZOMETER OR STANDPIPE INSTALLATION

Boring method: CME 55 TRACK MOUNT; 93mm I.D. HOLLOW STEM AUGERS

CONTINUED FROM PREVIOUS PAGE

Depth scale (m)	Description	Elev. / Depth (m)	Sample number	Type	Blows/0.3m	Water content	Additional lab. testing	Piezometer
10		183.22 / 10.20						
10–11.89	Coarse sand and gravel, trace silt. Dense grey		11	50 DO	47	○	MH	CAVED MATERIAL
11.89–20	Sand, fine, trace silt to silty sand, occ. coarse sand layers. Loose to compact Grey	181.53 / 11.89	12	50 DO	4			
			13	50 DO	7	○	MH	BENTONITE SEAL
			14	50 DO	6			
			15	50 DO	10	○		FILTER SAND
			16	50 DO	20			
20			17	50 DO	10			

CONTINUED ON NEXT PAGE

DEPTH SCALE 1 to 50

Golder Associates

LOGGED: AOB

CHECKED: AMP

PROJECT: 971-1103

RECORD OF BOREHOLE 97-9

LOCATION: STA. 17+651.36, o/s 30.06N; 994.65N,7107.49E

BORING DATE: FEB.11/97

DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES: NUMBER	TYPE	BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (10 20 30 40) / SHEAR STRENGTH Cu, kPa (nat V - +, rem V - ⊕, Q - ●, U - O) (20 40 60 80)	HYDRAULIC CONDUCTIVITY, k, cm/s / WATER CONTENT, PERCENT Wp—O W—Wl (20 40 60 80)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
20		CONTINUED FROM PREVIOUS PAGE									
				173.15	17	50 DO	10		O		
		END OF BOREHOLE		20.27							
21											NOTE: Water level in open borehole at Elev. 192.5m upon completion of drilling.
22											Water level in piezometer at Elev. 181.8m on Feb.22, 1997 and at Elev. 182.0m on March 27, 1997.
23											
24											
25											
26											
27											
28											
29											
30											

DATA INPUT: PS JUNE 5/97

DEPTH SCALE

1 to 50

Golder Associates

LOGGED: AOB

CHECKED: AMP

SOILM6

PROJECT: 971-1103

RECORD OF BOREHOLE 97-10

SHEET 1 OF 3

LOCATION: STA. 17+648.20, o/s 1.24N; 1015.74N,7087.60E

BORING DATE: FEB.12/97

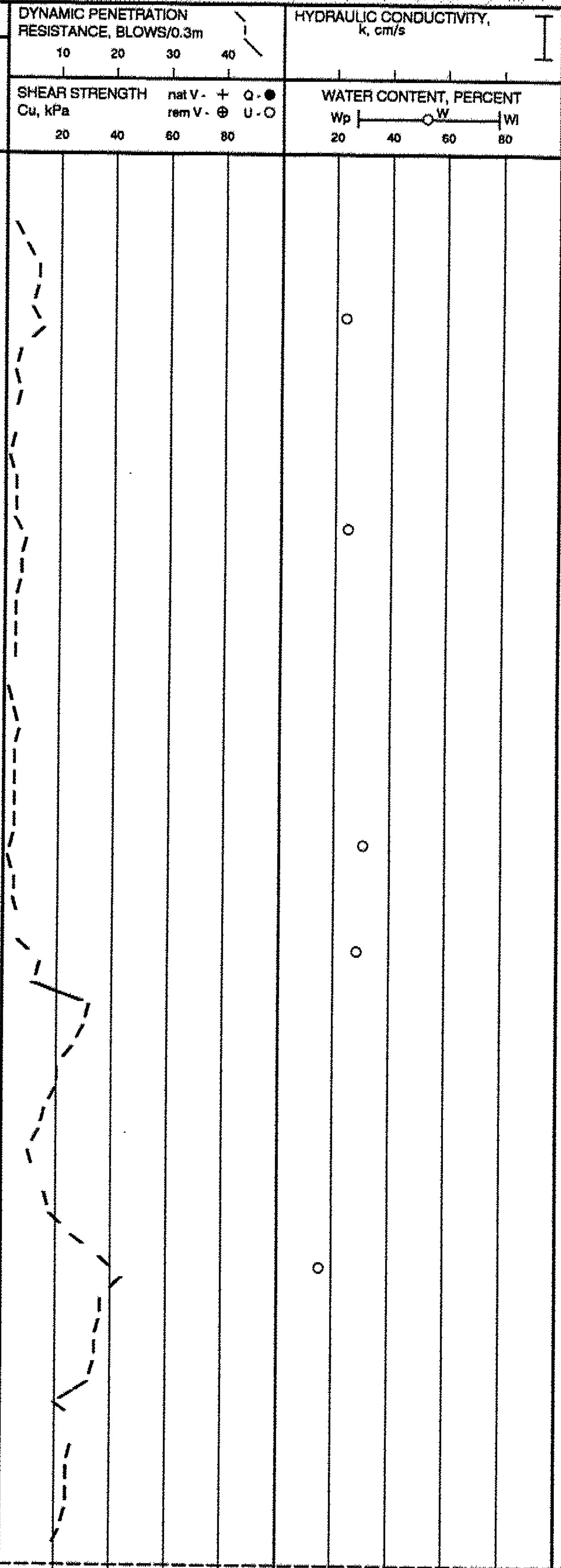
DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

BORING METHOD: CME 55 TRACK MOUNT; 93mm I.D. HOLLOW STEM AUGERS

DEPTH SCALE METRES	SOIL PROFILE – DESCRIPTION	ELEV. / DEPTH (m)	SAMPLE NUMBER	TYPE	BLOWS/0.3m
0	GROUND SURFACE	193.43			
	Sand, fine to coarse, oxidized, trace to some silt and amorphous peat, some sand. Interlayered Compact/stiff Grey and brown	0.00			
1			1	50 DO	12
	Sand, fine to medium, trace to some silt. Layered Loose to very loose Grey	191.91 / 1.52	2	50 DO	7
2			3	50 DO	3
3	Sandy silt to silt, some sand, trace organics. Very loose Grey	190.08 / 3.35			
4			4	50 DO	1
5			5	50 DO	1
			6	50 DO	1
6	Sand, medium to coarse, trace silt, trace gravel, trace oxidization. Layered Compact to dense Grey	187.03 / 6.40	7	50 DO	20
7					
8			8	50 DO	41
9			9	50 DO	26
10	CONTINUED ON NEXT PAGE				



ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION: BENTONITE SEAL; CAVED MATERIAL

DEPTH SCALE 1 to 50

Golder Associates

LOGGED: AOB

CHECKED: AMP

M1103010.BHS

PROJECT: 971-1103

RECORD OF BOREHOLE 97-10

SHEET 2 OF 3

LOCATION: STA. 17+648.20, o/s 1.24N; 1015.74N,7087.60E

BORING DATE: FEB.12/97

DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

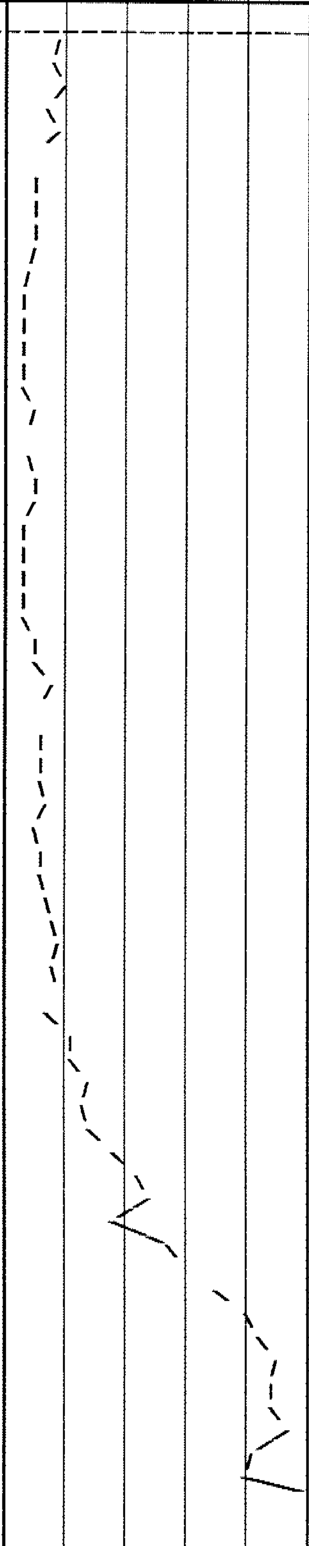
Boring method: CME 55 TRACK MOUNT; 93mm I.D. HOLLOW STEM AUGERS

CONTINUED FROM PREVIOUS PAGE

DEPTH SCALE METRES	DESCRIPTION	ELEV. / DEPTH (m)	SAMPLE NUMBER	TYPE	BLOWS/0.3m	WATER CONTENT	ADDITIONAL LAB. TESTING
10	Sand, fine, trace silt to silty sand, occ. medium and coarse sand layers. Very loose to compact Grey	183.22 / 10.21					
11			10	50 DO	26	○	
12			11	50 DO	15		
14			12	50 DO	1	○	M
15			13	50 DO	6	○	
17			14	50 DO	6		
18		174.69 / 18.74	15	50 DO	3		
19	END OF BOREHOLE						

CONTINUED ON NEXT PAGE

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (10, 20, 30, 40)
SHEAR STRENGTH Cu, kPa: nat V - + Q - ●; rem V - ⊕ U - ○ (20, 40, 60, 80)
HYDRAULIC CONDUCTIVITY, k, cm/s
WATER CONTENT, PERCENT: Wp —○W— Wl (20, 40, 60, 80)



PIEZOMETER OR STANDPIPE INSTALLATION:
BENTONITE SEAL
SAND FILTER

NOTE: Water level in open borehole at Elev. 181.2m on completion of drilling.

DATA INPUT: PS JUNE 5/97

SOILM6

DEPTH SCALE
1 to 50

Golder Associates

LOGGED: AOB
CHECKED: AMP

RECORD OF BOREHOLE 97-10

PROJECT: 971-1103

LOCATION: STA. 17+648.20, o/s 1.24N; 1015.74N,7087.60E

BORING DATE: FEB.12/97

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

DATUM: GEODETIC

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES: NUMBER	TYPE	BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (10 20 30 40); SHEAR STRENGTH Cu, kPa (20 40 60 80) nat V - + Q - ● rem V - ⊕ U - ○	HYDRAULIC CONDUCTIVITY, k, cm/s; WATER CONTENT, PERCENT Wp —○W— Wl (20 40 60 80)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
20		CONTINUED FROM PREVIOUS PAGE									Water level in piezometer at Elev. 181.5m on Feb.22, 1997 and at Elev. 181.4m on March 27, 1997.
21											
22											
23											
24											
25											
26											
27											
28											
29											
30											

DEPTH SCALE
1 to 50

Golder Associates

LOGGED: AOB
CHECKED: AMP

M1103010.BHS
DATA INPUT: PS JUNE 5/97
SOILM6

PROJECT: 971-1103

RECORD OF BOREHOLE 97-11

SHEET 1 OF 4

LOCATION: STA.17+647.34, o/s 16.89N;1042.74N,7066.111E

BORING DATE: FEB.13/97

DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

Boring method: CME 55 TRACK MOUNT; 93mm I.D. HOLLOW STEM AUGERS

Depth (m)	Soil Profile Description	Elev. / Depth (m)	Sample Number	Type	Blows/0.3m	Water Content (%)	Piezometer or Standpipe Installation
0	GROUND SURFACE	192.93 / 0.00					
	Sand, fine to coarse, oxidized, trace silt, trace gravel, trace organics. Loose Brown		1	50 DO	4	○	BACKFILL
1.52	Sand, fine, trace to some silt, trace clay, trace rootlets. Layered Very loose to loose Grey	191.41 / 1.52	2	50 DO	3		
			3	50 DO	1	○	BENTONITE SEAL
			4	50 DO	3		
3.65	Sandy silt to silt, trace to some sand, trace clay, trace organics, trace gravel. Very loose Grey	189.28 / 3.65	5	50 DO	1	○	CAVED MATERIALS
			6	50 DO	1	○	
			7	50 DO	1	○	
			8	50 DO	2		
8.08	Sand, medium to coarse, oxidized, some silt, trace gravel. Dense Brown	184.85 / 8.08	9	50 DO	41	○	
10	CONTINUED ON NEXT PAGE						

Column headings: DEPTH SCALE METRES; BORING METHOD; SOIL PROFILE (DESCRIPTION, STRATA PLOT, ELEV. DEPTH (m)); SAMPLES (NUMBER, TYPE, BLOWS/0.3m); DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (10, 20, 30, 40); SHEAR STRENGTH Cu, kPa (nat V - + Q - ●, rem V - ⊕ U - ○) (20, 40, 60, 80); HYDRAULIC CONDUCTIVITY, k, cm/s; WATER CONTENT, PERCENT (Wp —○W— Wl) (20, 40, 60, 80); ADDITIONAL LAB. TESTING; PIEZOMETER OR STANDPIPE INSTALLATION

DEPTH SCALE

1 to 50

Golder Associates

LOGGED: AOB

CHECKED: AMP

M1103011.BHS

DATA INPUT. PS JUNE 5/97

SCILM6

PROJECT: 971-1103

RECORD OF BOREHOLE 97-11

SHEET 2 OF 4

LOCATION: STA.17+647.34, o/s 16.89N;1042.74N,7066.111E

BORING DATE: FEB.13/97

DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

Boring method: CME 55 TRACK MOUNT, 93mm I.D. HOLLOW STEM AUGERS

CONTINUED FROM PREVIOUS PAGE

DEPTH SCALE METRES	SOIL PROFILE DESCRIPTION	ELEV. / DEPTH (m)	SAMPLE NUMBER	TYPE	BLOWS/0.3m	WATER CONTENT, PERCENT	PIEZOMETER OR STANDPIPE INSTALLATION
10							
10.66	Sand, trace to some silt to silty sand, trace gravel, occ. medium and coarse sand layers. Loose to compact Grey	182.27 / 10.66					
11			10	50 DO	13		
12							▽
			11	50 DO	6	o (~22)	
14			12	50 DO	4		
15			13	50 DO	18	o (~20)	CAVED MATERIALS
17			14	50 DO	4		
18			15	50 DO	10		
20			16	50 DO	7		

CONTINUED ON NEXT PAGE

DEPTH SCALE 1 to 50

LOGGED: AOB

CHECKED: AMP

PROJECT: 971-1103

RECORD OF BOREHOLE 97-11

LOCATION: STA.17+647.34, o/s 16.89N;1042.74N,7066.111E

BORING DATE: FEB.13/97

DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES: NUMBER	TYPE	BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (10 20 30 40); SHEAR STRENGTH Cu, kPa (nat V - +, Q - ●, rem V - ⊕, U - ○) (20 40 60 80)	HYDRAULIC CONDUCTIVITY, k, cm/s; WATER CONTENT, PERCENT Wp — W — Wl (20 40 60 80)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
20	CME 55 TRACK MOUNT; 93mm I.D HOLLOW STEM AUGERS	CONTINUED FROM PREVIOUS PAGE									
					16	50 DO	7				
21											
					17	50 DO	3				
22											
23					18	50 DO	19				
24		Sand, fine, trace to some silt to silty sand, trace gravel, occ. medium and coarse sand layers. Loose to compact Grey									
					19	50 DO	21				
25											CAVED MATERIALS
26					20	50 DO	19				
27											
28											
29											
30		CONTINUED ON NEXT PAGE									

DEPTH SCALE
1 to 50

Golder Associates

LOGGED: AOB
CHECKED: AMP

DATA INPUT: PS JUNE 5/97

M1103011.BHS

PROJECT: 971-1103

RECORD OF BOREHOLE 97-11

SHEET 4 OF 4

LOCATION: STA.17+647.34, o/s 16.89N;1042.74N,7066.111E

BORING DATE: FEB.13/97

DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES: NUMBER	TYPE	BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (10 20 30 40); SHEAR STRENGTH Cu, kPa nat V - + Q - ● rem V - ⊕ U - ○ (20 40 60 80)	HYDRAULIC CONDUCTIVITY, k, cm/s; WATER CONTENT, PERCENT Wp ⊢—○W—⊣ Wl (20 40 60 80)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
30	CME 55 TRACK MOUNT 93mm I.D. HOLLOW STEM AUGERS	CONTINUED FROM PREVIOUS PAGE									
		Sand, fine, trace to some silt to silty sand, trace gravel, occ. medium and coarse sand layers. Loose to compact Grey		161.99	21	50 DO	31				
31		END OF BOREHOLE		30.94							
32											NOTE: Water level in piezometer at Elev. 181.2m on March 27, 1997.
33											
34											
35											
36											
37											
38											
39											
40											

DATA INPUT: PS JUNE 5/97

DEPTH SCALE

1 to 50

Golder Associates

LOGGED: AOB

CHECKED: AMP

SOILM6

PROJECT: 971-1103

RECORD OF BOREHOLE 97-12

LOCATION: STA.25+784.09,o/s 22.91N; 5034.06N,13943.16E

BORING DATE: FEB.26/97

DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE – DESCRIPTION	ELEV. / DEPTH (m)	SAMPLE NUMBER	TYPE	BLOWS/0.3m	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
0		GROUND SURFACE	179.16 / 0.00					BENTONITE SEAL
1	CME 55 TRACK MOUNT; 93mm I.D. HOLLOW STEM AUGERS	Silty clay to clay and clayey silt, trace sand, occ. thin seams of grey silt. Irregularly layered. Soft to stiff. Greyish brown		1	50 DO	3		
				2	50 DO	1		
				3	50 DO	1		
				4	50 DO	1		BACKFILL
6				6	75 TO		=15.9kN/m³; MH; C	
7				7	AS			
8				8	50 DO	1		
9				9	75 TO			
10		CONTINUED ON NEXT PAGE						

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m: 40, 80, 120, 160

SHEAR STRENGTH Cu, kPa (nat V - +; rem V - ⊕; Q - ●; U - ○): 20, 40, 60, 80

HYDRAULIC CONDUCTIVITY, k, cm/s

WATER CONTENT, PERCENT: Wp —○W— Wl

CONTINUED ON NEXT PAGE

DEPTH SCALE 1 to 50

Golder Associates

LOGGED: AOB

CHECKED: AMP

M1103012.BHS

PROJECT: 971-1103

RECORD OF BOREHOLE 97-12

LOCATION: STA.25+784.09,o/s 22.91N; 5034.06N,13943.16E

BORING DATE: FEB.26/97

DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE – DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (40, 80, 120, 160); SHEAR STRENGTH Cu, kPa (20, 40, 60, 80) nat V - + Q - ● rem V - ⊕ U - ○	HYDRAULIC CONDUCTIVITY, k, cm/s; WATER CONTENT, PERCENT Wp —o W— Wl	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
10		CONTINUED FROM PREVIOUS PAGE									
11					10	50 DO	1				
12					11	AS					
13					12	75 TO					
					13	AS					
14	CME 55 TRACK MOUNT; 93mm I.D. HOLLOW STEM AUGERS	Silty clay to clay and clayey silt, trace sand, occ. thin seams of grey silt. Irregularly layered. Soft to stiff. Greyish brown			14	50 DO	1				
15					15	75 TO					BACKFILL
16											
17					16	50 DO	1				
18											
19					17	75 TO					
20		CONTINUED ON NEXT PAGE			18	50 DO	1				

DEPTH SCALE
1 to 50

Golder Associates

LOGGED: AOB
CHECKED: AMP

DATA INPUT: ps april 4/97

SOILM6

M1103012.BHS

PROJECT: 971-1103

RECORD OF BOREHOLE 97-12

LOCATION: STA.25+784.09,o/s 22.91N; 5034.06N,13943.16E

BORING DATE: FEB.26/97

DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. / DEPTH (m)	SAMPLES: NUMBER	TYPE	BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (40, 80, 120, 160) / SHEAR STRENGTH Cu, kPa (nat V - +, Q - ●, rem V - ⊕, U - O) (20, 40, 60, 80)	HYDRAULIC CONDUCTIVITY, k, cm/s / WATER CONTENT, PERCENT (Wp, W, Wl)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
20		CONTINUED FROM PREVIOUS PAGE									
	CME 55 TRACK MOUNT, 93mm I.D. HOLLOW STEM AUGERS				18	50 DO	1				BACKFILL
21				158.16 / 21.00							
					19	75 TO					
22											
23					20	50 DO					
24		Silty clay to clay and clayey silt, trace sand, occ. thin seams of grey silt. Irregularly layered. Firm to stiff becoming stiff below 23m depth. Reddish brown									
					21	75 TO					
25											
26					22	50 DO	1				
27											
					23	75 TO					
28											BENTONITE SEAL
29											SAND FILTER
					24	50 DO	1				
30		CONTINUED ON NEXT PAGE									

DEPTH SCALE

1 to 50

Golder Associates

LOGGED: AOB

CHECKED: AMP

DATA INPUT: ps april 4/97

SOILM6

M1103012.BHS

PROJECT: 971-1103

RECORD OF BOREHOLE 97-12

SHEET 4 OF 6

LOCATION: STA.25+784.09,o/s 22.91N; 5034.06N,13943.16E

BORING DATE: FEB.26/97

DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

Column headings: DEPTH SCALE METRES | BORING METHOD | SOIL PROFILE (DESCRIPTION, STRATA PLOT, ELEV. DEPTH (m)) | SAMPLES (NUMBER, TYPE, BLOWS/0.3m) | DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (40, 80, 120, 160) / SHEAR STRENGTH Cu, kPa (nat V - +, rem V - ⊕, Q - ●, U - ○; 20, 40, 60, 80) | HYDRAULIC CONDUCTIVITY, k, cm/s / WATER CONTENT, PERCENT (Wp, W, Wl) | ADDITIONAL LAB. TESTING | PIEZOMETER OR STANDPIPE INSTALLATION

CONTINUED FROM PREVIOUS PAGE

Boring method: CME 55 TRACK MOUNT; 93mm I.D. HOLLOW STEM AUGERS

Depth 30–40 m:

Silty clay to clay and clayey silt, trace sand, occ. thin seams of grey silt.
Irregularly layered
Stiff
Reddish brown

Sample Number	Type	Blows/0.3m
25	75 TO	
26	50 DO	1
27	75 TO	
28	50 DO	1
29	75 TO	
30	50 DO	1
31	75 TO	

Additional lab testing (at approx. 34 m): GIU, MH, C, γ = 18.4kN/m^3

NOTE:
Piezometer tubing frozen at ground surface (Elev. 179.19m) on March 31, 1997.

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DATA INPUT: ps april 4/97

SOILM6

DEPTH SCALE
1 to 50

Golder Associates

LOGGED: AOB

CHECKED: AMP

M1103012.BHS

PROJECT: 971-1103

RECORD OF BOREHOLE 97-12

SHEET 5 OF 6

LOCATION: STA.25+784.09,o/s 22.91N; 5034.06N,13943.16E

BORING DATE: FEB.26/97

DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES: NUMBER	TYPE	BLOWS/0.3m
40	CME 55 TRACK MOUNT; 93mm I.D. HOLLOW STEM AUGERS	CONTINUED FROM PREVIOUS PAGE					
40					31	75 TO	
41–42		Silty clay to clay and clayey silt, trace sand, occ. thin seams of grey silt. Irregularly layered Stiff Reddish brown			32	50 DO	1
44–45					33	50 DO	2
45.42	DYNAMIC CONE	END OF BOREHOLE		133.74 45.42			
50		CONTINUED ON NEXT PAGE					

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m: 40, 80, 120, 160

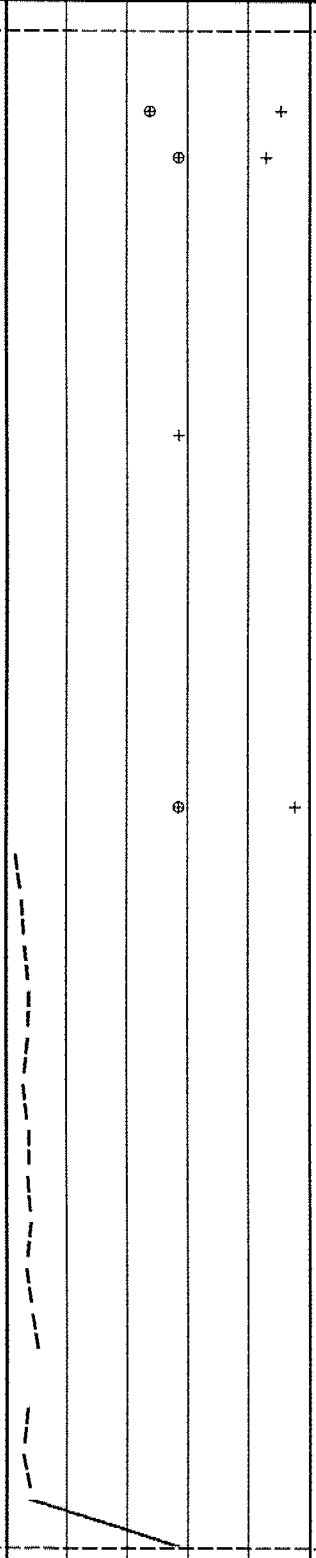
SHEAR STRENGTH Cu, kPa: nat V - + ; Q - ● ; rem V - ⊕ ; U - ○ : 20, 40, 60, 80

HYDRAULIC CONDUCTIVITY, k, cm/s

WATER CONTENT, PERCENT: Wp —○W— Wl

ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION



DEPTH SCALE

1 to 50

Golder Associates

LOGGED: AOB

CHECKED: AMP

DATA INPUT: ps april 4/97

SOILM6

PROJECT: 971-1103

RECORD OF BOREHOLE 97-12

LOCATION: STA.25+784.09,o/s 22.91N; 5034.06N,13943.16E

BORING DATE: FEB.26/97

DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (40, 80, 120, 160); SHEAR STRENGTH Cu, kPa (nat V - +, rem V - ⊕, Q - ●, U - ○) (20, 40, 60, 80)	HYDRAULIC CONDUCTIVITY, k, cm/s; WATER CONTENT, PERCENT (Wp, W, Wl)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
50	TRACK MOUNT CME 55 / DYNAMIC CONE	CONTINUED FROM PREVIOUS PAGE									
		END OF DYNAMIC PENETRATION TEST		128.87 50.29							
51											
52											
53											
54											
55											
56											
57											
58											
59											
60											

DATA INPUT: ps april 4/97

DEPTH SCALE

1 to 50

Golder Associates

LOGGED: AOB

CHECKED: AMP

PROJECT: 971-1103

RECORD OF BOREHOLE 97-13

SHEET 1 OF 5

LOCATION: STA. 16+140, o/s 25m; 278.187N, 5781.3E

BORING DATE: MARCH 27-29/97

DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. / DEPTH (m)	SAMPLES: NUMBER	TYPE	BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m; SHEAR STRENGTH Cu, kPa (nat V - +, rem V - ⊕, Q - ●, U - ○) 20 40 60 80	HYDRAULIC CONDUCTIVITY, k, cm/s; WATER CONTENT, PERCENT Wp ├─○W─┤ Wl 20 40 60 80	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
0	CME 55 TRACK MOUNT; 108mm I D HOLLOW STEM AUGERS	GROUND SURFACE		188.69 / 0.00							
1		Borehole 97-13 drilled 3m east of Borehole 97-3. For soil description refer to Record of Borehole 97-3.									
2											
3											
4											▽
5											
6											
7											
8											
9											
10		CONTINUED ON NEXT PAGE									

DEPTH SCALE
1 to 50

Golder Associates

LOGGED: AOB
CHECKED: AMP

M1103013.BHS
DATA INPUT: ps JUNE 5/97
SOILM6

M1103013.BHS

PROJECT: 971-1103

RECORD OF BOREHOLE 97-13

LOCATION: STA. 16+140, o/s 25m; 278.187N, 5781.3E

BORING DATE: MARCH 27-29/97

DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES: NUMBER	TYPE	BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m; SHEAR STRENGTH Cu, kPa (nat V - + Q - ●; rem V - ⊕ U - O) 20 40 60 80	HYDRAULIC CONDUCTIVITY, k, cm/s; WATER CONTENT, PERCENT Wp —O W— Wl 20 40 60 80	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
10		CONTINUED FROM PREVIOUS PAGE									
11	CME 55 TRACK MOUNT 108mm I.D. HOLLOW STEM AUGERS	Borehole 97-13 drilled 3m east of Borehole 97-3. For soil description refer to Record of Borehole 97-3.									
12											
13											
14											
15											
16											
17											
18											
19											
20		CONTINUED ON NEXT PAGE									

DATA INPUT: ps JUNE 5/97

DEPTH SCALE
1 to 50

Golder Associates

LOGGED: AOB
CHECKED: AMP

SOILMS

M1103013.BHS

PROJECT: 971-1103

RECORD OF BOREHOLE 97-13

SHEET 1 OF 5

LOCATION: STA. 16+140, o/s 25m; 278.187N, 5781.3E

BORING DATE: MARCH 27-29/97

DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES: NUMBER	TYPE	BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m; SHEAR STRENGTH Cu, kPa (nat V - +, Q - ●, rem V - ⊕, U - ○) 20 40 60 80	HYDRAULIC CONDUCTIVITY, k, cm/s; WATER CONTENT, PERCENT Wp —○W— Wl 20 40 60 80	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
0	CME 55 TRACK MOUNT 108mm I.D. HOLLOW STEM AUGERS	GROUND SURFACE		188.69 0.00							
1		Borehole 97-13 drilled 3m east of Borehole 97-3. For soil description refer to Record of Borehole 97-3.									
2											
3											
4											▽
5											
6											
7											
8											
9											
10		CONTINUED ON NEXT PAGE									

DATA INPUT: ps JUNE 5/97

DEPTH SCALE
1 to 50

Golder Associates

LOGGED: AOB
CHECKED: AMP

SOILM6

M1103013.BHS

PROJECT: 971-1103

RECORD OF BOREHOLE 97-13

LOCATION: STA. 16+140, o/s 25m; 278.187N, 5781.3E

BORING DATE: MARCH 27-29/97

DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES: NUMBER	TYPE	BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m; SHEAR STRENGTH Cu, kPa (nat V - +, rem V - ⊕, Q - ●, U - ○) 20 40 60 80	HYDRAULIC CONDUCTIVITY, k, cm/s; WATER CONTENT, PERCENT Wp —o W— Wl 20 40 60 80	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
10	CME 55 TRACK MOUNT; 108mm I.D. HOLLOW STEM AUGERS	CONTINUED FROM PREVIOUS PAGE									
11		Borehole 97-13 drilled 3m east of Borehole 97-3. For soil description refer to Record of Borehole 97-3.									
12											
13											
14											
15											
16											
17											
18											
19											
20		CONTINUED ON NEXT PAGE									

DATA INPUT: ps JUNE 5/97

SOILM6

DEPTH SCALE
1 to 50

Golder Associates

LOGGED: AOB
CHECKED: AMP

M1103013 BHS

PROJECT: 971-1103

RECORD OF BOREHOLE 97-13

SHEET 3 OF 5

LOCATION: STA. 16+140, o/s 25m; 278.187N, 5781.3E

BORING DATE: MARCH 27-29/97

DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE – DESCRIPTION	STRATA PLOT	ELEV. / DEPTH (m)	SAMPLES – NUMBER	SAMPLES – TYPE	SAMPLES – BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m / SHEAR STRENGTH Cu, kPa (nat V - +, rem V - ⊕, Q - ●, U - ○) 20 40 60 80	HYDRAULIC CONDUCTIVITY, k, cm/s / WATER CONTENT, PERCENT (Wp, W, Wl) 20 40 60 80	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
20		CONTINUED FROM PREVIOUS PAGE									
21–22	CME 55 TRACK MOUNT 108mm I.D. HOLLOW STEM AUGERS	Borehole 97-13 drilled 3m east of Borehole 97-3. For soil description refer to Record of Borehole 97-3.									
23–30	CME 55 TRACK MOUNT 108mm I.D. HOLLOW STEM AUGERS; WASH BORING / NQ CASING	Silty clay to clay and clayey silt to silt, trace sand. Irregularly layered Firm to very stiff Brown to red brown	hatched	165.85 / 22.84	1	50 DO	7				
					2	50 DO	8				
					4	50 TO	PH				
					6	50 DO	7				
					8	50 TO	PH				
30		CONTINUED ON NEXT PAGE									

DEPTH SCALE 1 to 50

Golder Associates

LOGGED: AOB

CHECKED: AMP

DATA INPUT: ps JUNE 5/97

SOILM6

PROJECT: 971-1103

RECORD OF BOREHOLE 97-13

LOCATION: STA. 16+140, o/s 25m; 278.187N, 5781.3E

BORING DATE: MARCH 27-29/97

DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m / SHEAR STRENGTH Cu, kPa (nat V - +, rem V - ⊕, Q - ●, U - ○) 20 40 60 80	HYDRAULIC CONDUCTIVITY, k, cm/s / WATER CONTENT, PERCENT Wp —○W— Wl 20 40 60 80	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
30	CME 55 TRACK MOUNT / WASH BORING / NQ CASING	CONTINUED FROM PREVIOUS PAGE									
31					10	50 DO	10				
34		Silty clay to clay and clayey silt to silt, trace sand. Irregularly layered Firm to very stiff Brown to red brown			12	50 DO	11				
37					14	50 DO	9				
40		CONTINUED ON NEXT PAGE			16	50 DO	12				

DEPTH SCALE

1 to 50

Golder Associates

LOGGED: AOB

CHECKED: AMP

DATA INPUT: ps JUNE 5/97

SOILM6

PROJECT: 971-1103

RECORD OF BOREHOLE 97-13

LOCATION: STA. 16+140, o/s 25m; 278.187N, 5781.3E

BORING DATE: MARCH 27-29/97

DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m; SHEAR STRENGTH Cu, kPa (nat V - +, rem V - ⊕, Q - ●, U - ○) 20 40 60 80	HYDRAULIC CONDUCTIVITY, k, cm/s; WATER CONTENT, PERCENT Wp W Wl 20 40 60 80	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
40		CONTINUED FROM PREVIOUS PAGE									
				148.46	16	50 DO	12				
		END OF BOREHOLE		40.23							
41											NOTE: Water level in open borehole at Elev. 185.0m during drilling.
42											
43											
44											
45											
46											
47											
48											
49											
50											

DATA INPUT: ps JUNE 5/97

DEPTH SCALE

1 to 50

Golder Associates

LOGGED: AOB

CHECKED: AMP

M1103014.BHS

PROJECT: 971-1103

RECORD OF BOREHOLE 97-14

SHEET 1 OF 5

LOCATION: STA. 17+645.2, o/s 1.2mN;

BORING DATE: MAR.29-APR.1/97

DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. / DEPTH (m)	SAMPLES: NUMBER	TYPE	BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (40, 80, 120, 160); SHEAR STRENGTH Cu, kPa (20, 40, 60, 80) nat V - + Q - ● rem V - ⊕ U - ○	HYDRAULIC CONDUCTIVITY, k, cm/s; WATER CONTENT, PERCENT Wp —o W — Wl (20, 40, 60, 80)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
0	CME 55 TRACK MOUNT WASH BORING, HQ CASING	GROUND SURFACE		193.40 0.00							
1		Borehole 97-14 drilled 3m west of Borehole 97-10. For soil description refer to Record of Borehole 97-10.									
2											
3											
4											
5											
6											
7											
8											
9											
10		CONTINUED ON NEXT PAGE									

DEPTH SCALE
1 to 50

Golder Associates

LOGGED: AOB
CHECKED: AMP

DATA INPUT. PS JUNE 5/97

SOILM6

M1103014.BHS

PROJECT: 971-1103

RECORD OF BOREHOLE 97-14

SHEET 2 OF 5

LOCATION: STA. 17+645.2, o/s 1.2mN;

BORING DATE: MAR.29-APR.1/97

DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. / DEPTH (m)	SAMPLES: NUMBER	TYPE	BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (40, 80, 120, 160); SHEAR STRENGTH Cu, kPa (nat V - +, Q - ●, rem V - ⊕, U - ○) (20, 40, 60, 80)	HYDRAULIC CONDUCTIVITY, k, cm/s; WATER CONTENT, PERCENT (Wp, W, Wl) (20, 40, 60, 80)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
10	CME 55 TRACK MOUNT WASH BORING, HQ CASING	CONTINUED FROM PREVIOUS PAGE									
11		Borehole 97-14 drilled 3m west of Borehole 97-10. For soil description refer to Record of Borehole 97-10.									
12											
13											
14											
15											
16											
17											
18											
19											
20		CONTINUED ON NEXT PAGE									

DATA INPUT: PS JUNE 5/97

DEPTH SCALE
1 to 50

LOGGED: AOB
CHECKED: AMP

Golder Associates

SOILM6

M1103014.BHS

PROJECT: 971-1103

RECORD OF BOREHOLE 97-14

SHEET 3 OF 5

LOCATION: STA. 17+645.2, o/s 1.2mN;

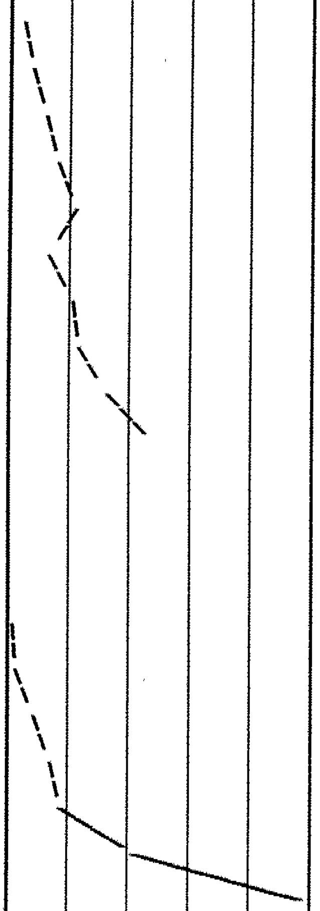
BORING DATE: MAR.29-APR.1/97

DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE – DESCRIPTION	STRATA PLOT	ELEV. / DEPTH (m)	SAMPLES – NUMBER	SAMPLES – TYPE	SAMPLES – BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (40, 80, 120, 160) / SHEAR STRENGTH Cu, kPa (nat V - +, rem V - ⊕, Q - ●, U - ○) (20, 40, 60, 80)	HYDRAULIC CONDUCTIVITY, k, cm/s / WATER CONTENT, PERCENT (Wp, W, Wl) (20, 40, 60, 80)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
20		CONTINUED FROM PREVIOUS PAGE									
21		Borehole 97-14 drilled 3m west of Borehole 97-10. For soil description refer to Record of Borehole 97-10.									
22											
				170.54 / 22.86							
23					1	50 DO	31		○		
24											
25	CME 55 TRACK MOUNT WASH BORING, HQ CASING	Sand, fine, trace to some silt to silty sand, occ. medium and coarse sand seams. Compact to dense Grey									
26											
27											
28					2	50 DO	27				
29											
30					3	50 DO	19		○		
		CONTINUED ON NEXT PAGE									



DATA INPUT: PS JUNE 5/97

DEPTH SCALE
1 to 50

Golder Associates

LOGGED: AOB
CHECKED: AMP

SOILM6

PROJECT: 971-1103

RECORD OF BOREHOLE 97-14

SHEET 4 OF 5

LOCATION: STA. 17+645.2, o/s 1.2mN;

BORING DATE: MAR.29-APR.1/97

DATUM: GEODETIC

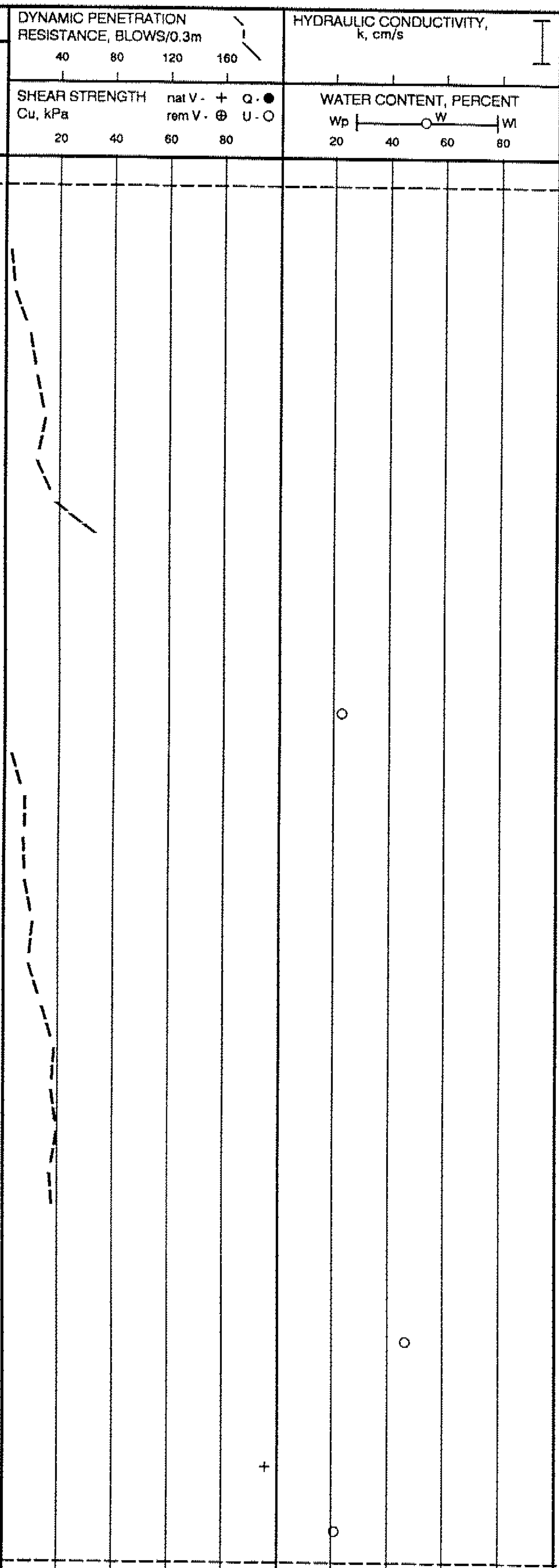
SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

BORING METHOD: CME 55 TRACK MOUNT, WASH BORING, HQ CASING

CONTINUED FROM PREVIOUS PAGE

DEPTH SCALE METRES	SOIL PROFILE – DESCRIPTION	ELEV. / DEPTH (m)	SAMPLE NUMBER	TYPE	BLOWS/0.3m
30–36	Sand, fine, trace to some silt to silty sand, occ. medium and coarse sand seams. Compact to dense. Grey		3	50 DO	19
			4	50 DO	28
			5	50 DO	18
36–39.5	Silty clay, trace sand, occ. fine sand seam. Stiff to very stiff. Brown	157.40 / 36.00	7	50 DO	9
39.5–40	Sand, fine, trace to some silt, trace clay. Dense. Grey	153.90 / 39.50	9	50 DO	30



CONTINUED ON NEXT PAGE

DEPTH SCALE 1 to 50

Golder Associates

LOGGED: AOB

CHECKED: AMP

M1103014.BHS

DATA INPUT PS JUNE 5/97

SOILM6

PROJECT: 971-1103

RECORD OF BOREHOLE 97-14

LOCATION: STA. 17+645.2, o/s 1.2mN;

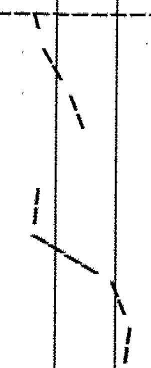
BORING DATE: MAR.29-APR.1/97

DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. / DEPTH (m)	SAMPLES: NUMBER	TYPE	BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (40, 80, 120, 160); SHEAR STRENGTH Cu, kPa (nat V - +, rem V - ⊕, Q - ●, U - O) (20, 40, 60, 80)	HYDRAULIC CONDUCTIVITY, k, cm/s; WATER CONTENT, PERCENT (Wp, W, Wl) (20, 40, 60, 80)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
40	CME 55 TRACK MOUNT; WASH BORING, HQ CASING	CONTINUED FROM PREVIOUS PAGE									
				153.17 / 40.23							
41		Sand, fine, grey, trace to some silt, trace clay and brown silty clay, some sand. Interlayered Dense/hard			10	50 DO	54				
42											
43					11	50 DO	42				
44		END OF BOREHOLE		149.51 / 43.89							
45											
46											
47											
48											
49											
50											



DATA INPUT: PS JUNE 5/97

SOILM6

DEPTH SCALE

1 to 50

Golder Associates

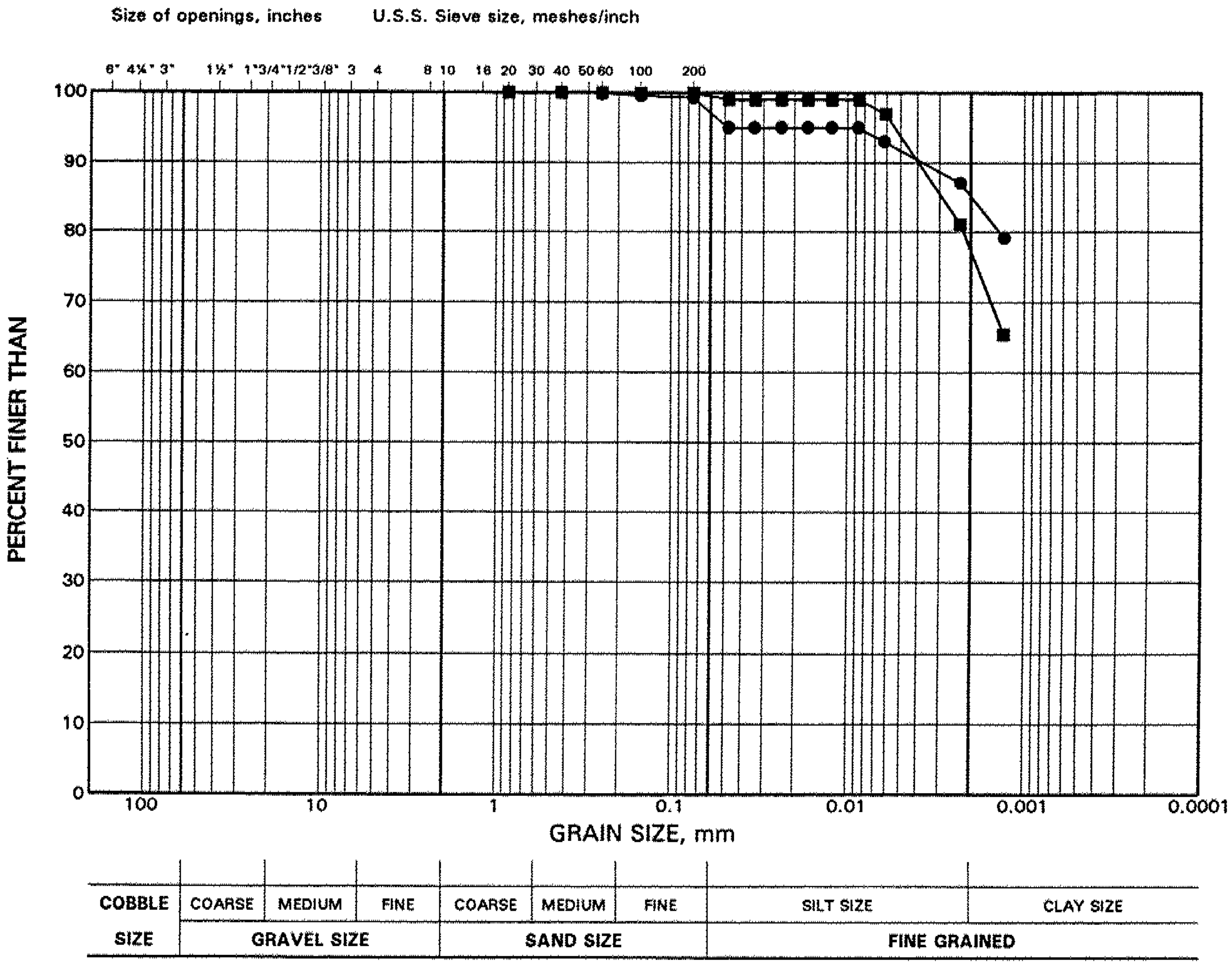
LOGGED: AOB

CHECKED: AMP

OVERSIZE DRAWING(S)

GRAIN SIZE DISTRIBUTION
Clay to Silty Clay

FIGURE 7

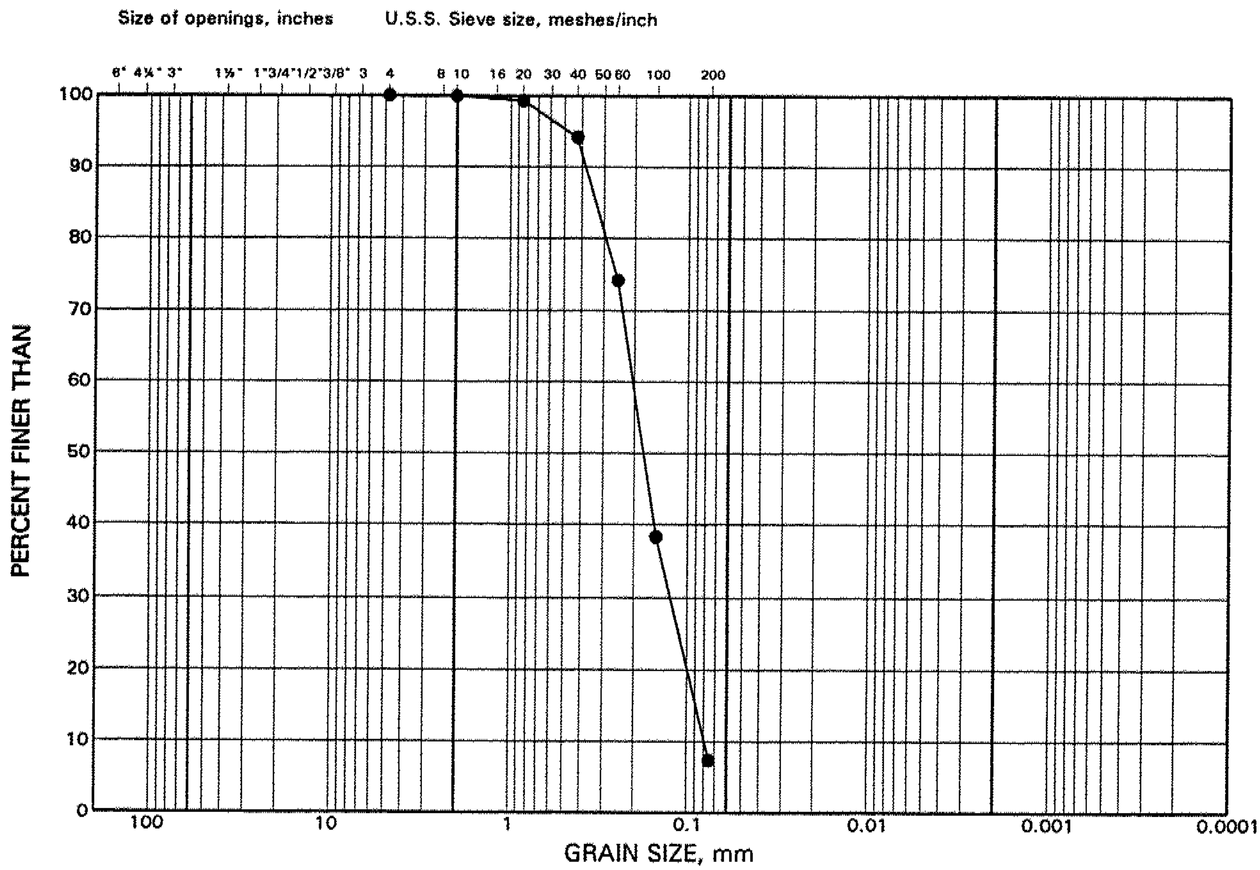


LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	97-1	14	167.5
■	97-1	27	152.3

GRAIN SIZE DISTRIBUTION
Sand

FIGURE 8



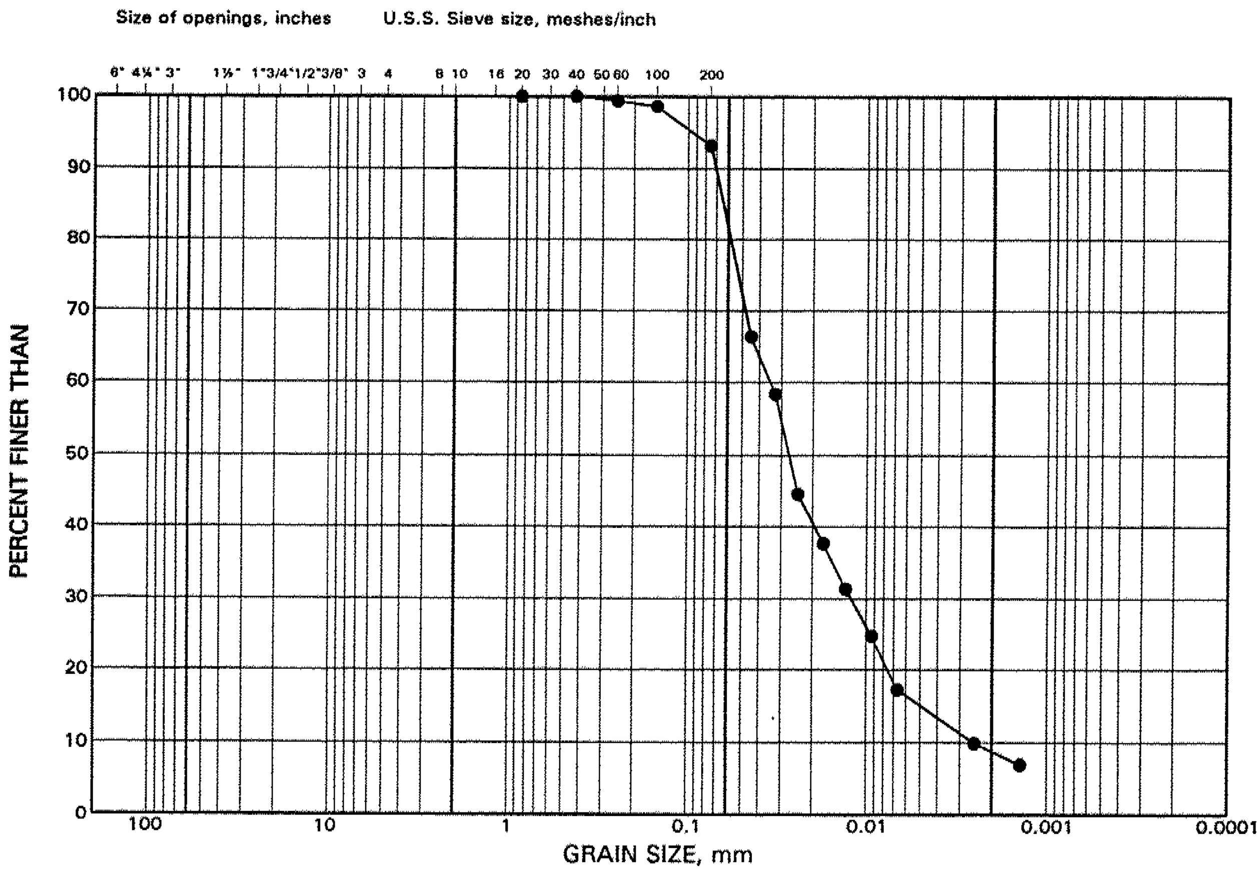
COBBLE SIZE	COARSE	MEDIUM	FINE	COARSE	MEDIUM	FINE	SILT SIZE	CLAY SIZE
	GRAVEL SIZE			SAND SIZE			FINE GRAINED	

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	97-3	7	182.0

GRAIN SIZE DISTRIBUTION
Silt

FIGURE 9



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

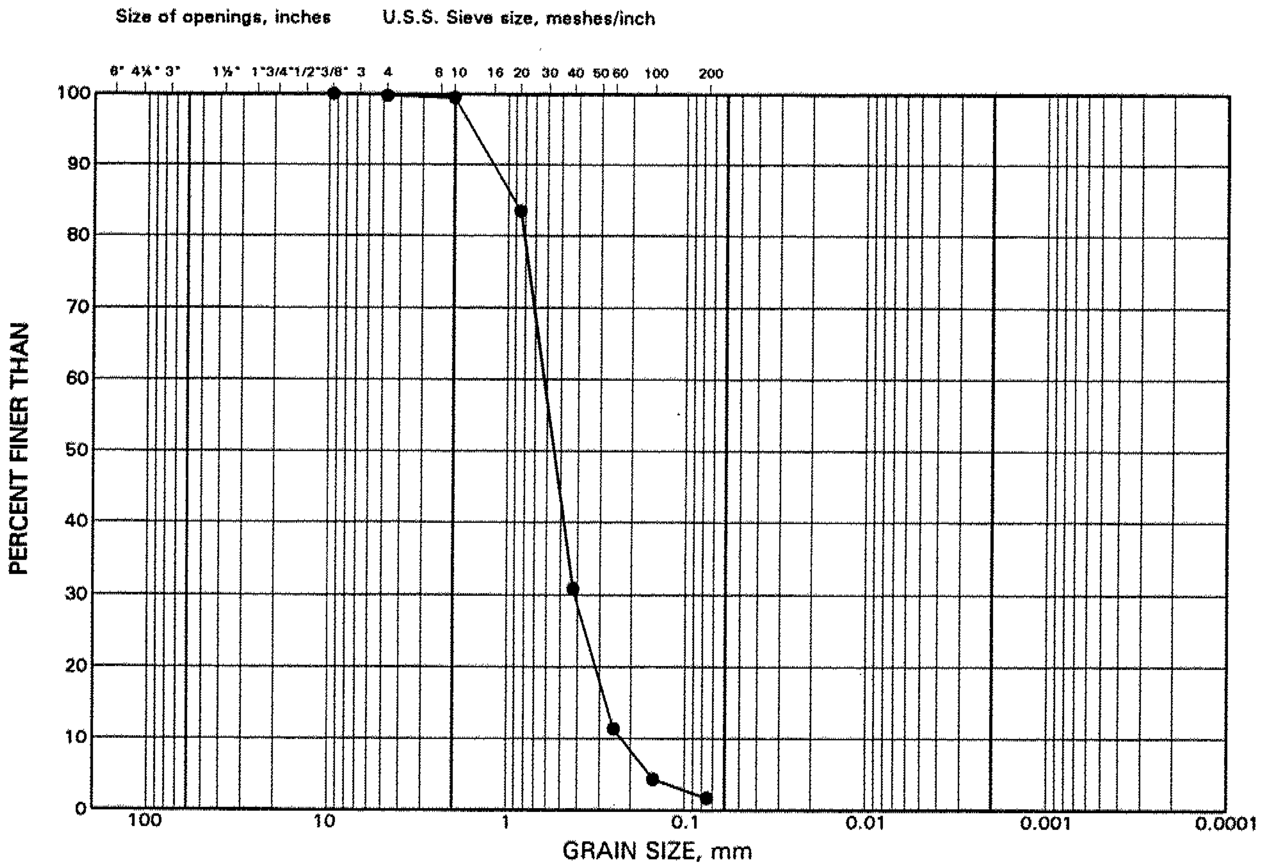
LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	97-4	7	187.0

Golder Associates

GRAIN SIZE DISTRIBUTION
Sand

FIGURE 10



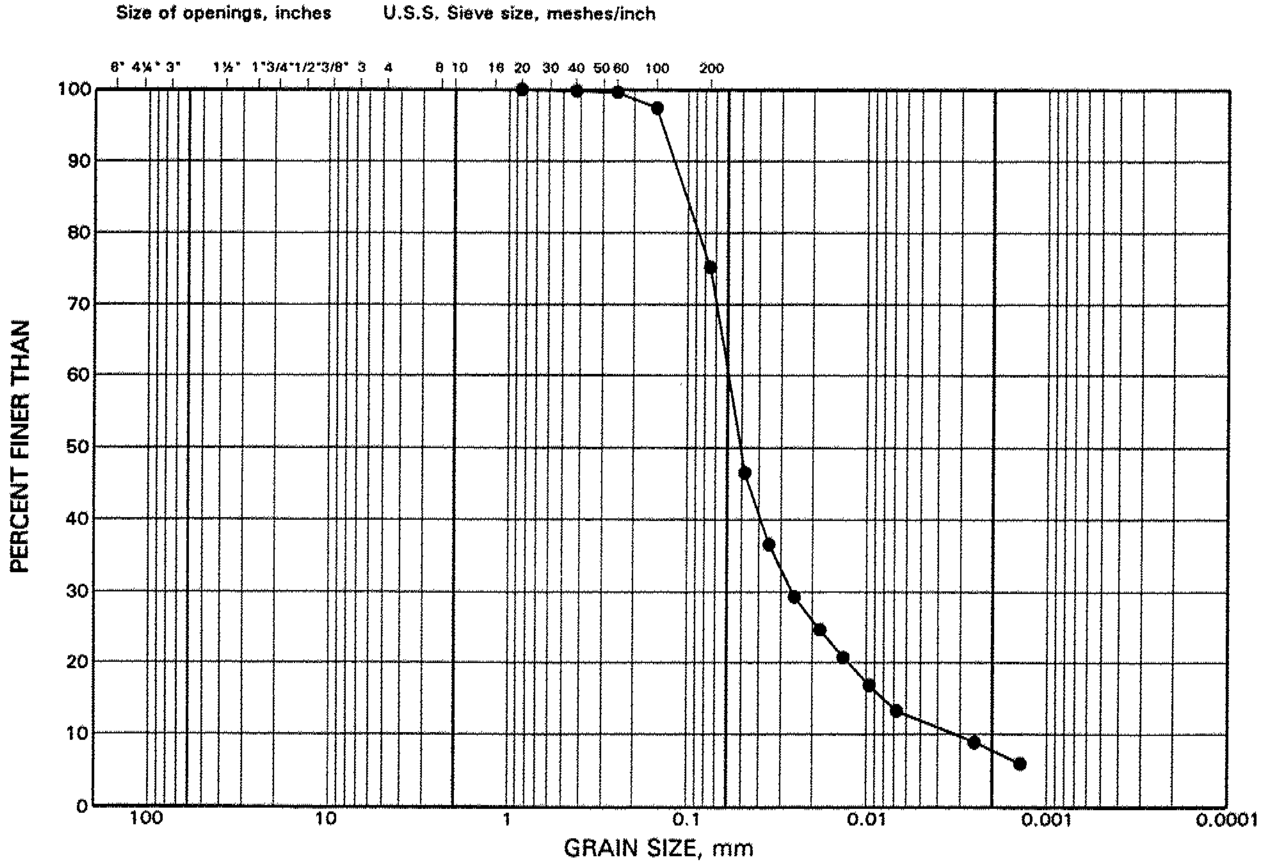
COBBLE SIZE	COARSE	MEDIUM	FINE	COARSE	MEDIUM	FINE	SILT SIZE	CLAY SIZE
	GRAVEL SIZE			SAND SIZE			FINE GRAINED	

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	97-4	13	177.9

GRAIN SIZE DISTRIBUTION
Sand and Silt

FIGURE 11



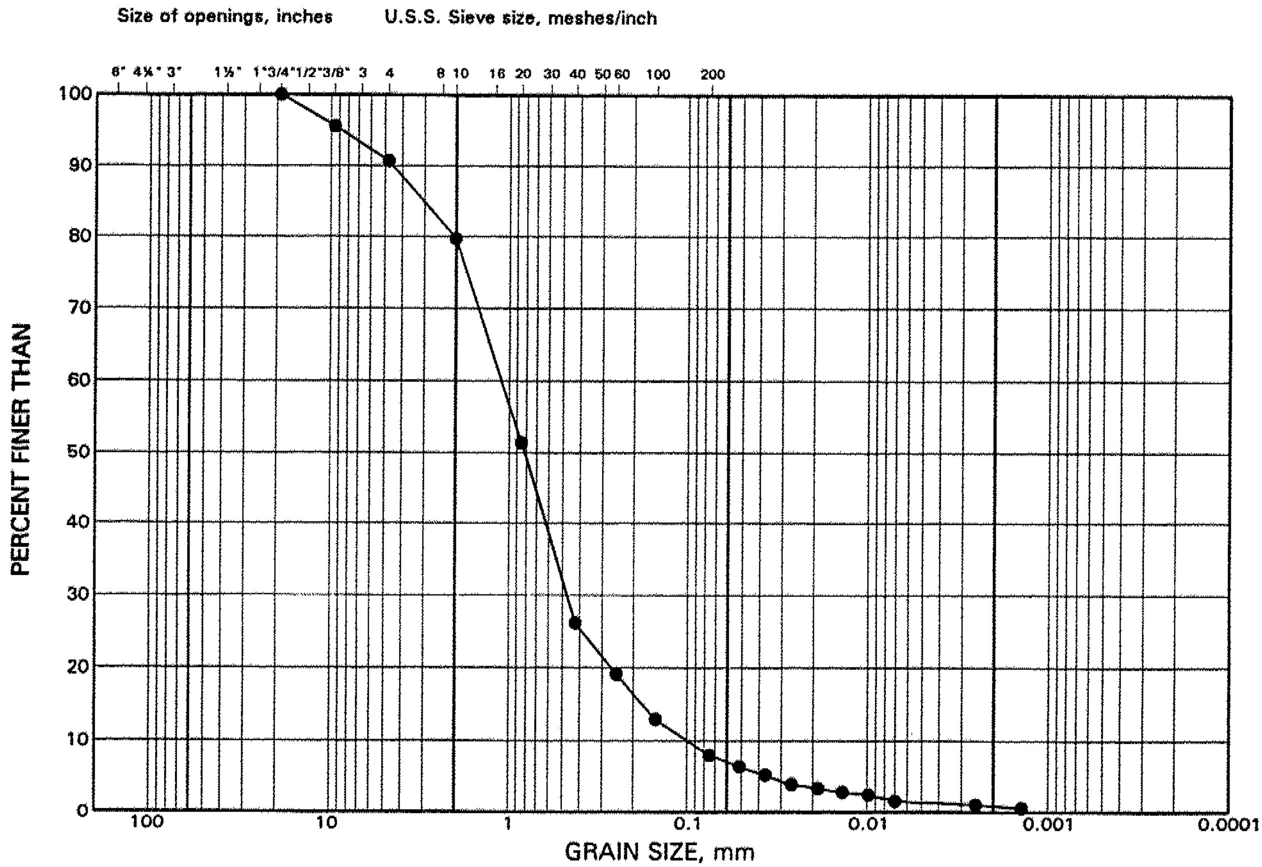
COBBLE	COARSE	MEDIUM	FINE	COARSE	MEDIUM	FINE	SILT SIZE	CLAY SIZE
SIZE	GRAVEL SIZE			SAND SIZE			FINE GRAINED	

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	97-9	6	188.4

GRAIN SIZE DISTRIBUTION
Sand and Gravel

FIGURE 12



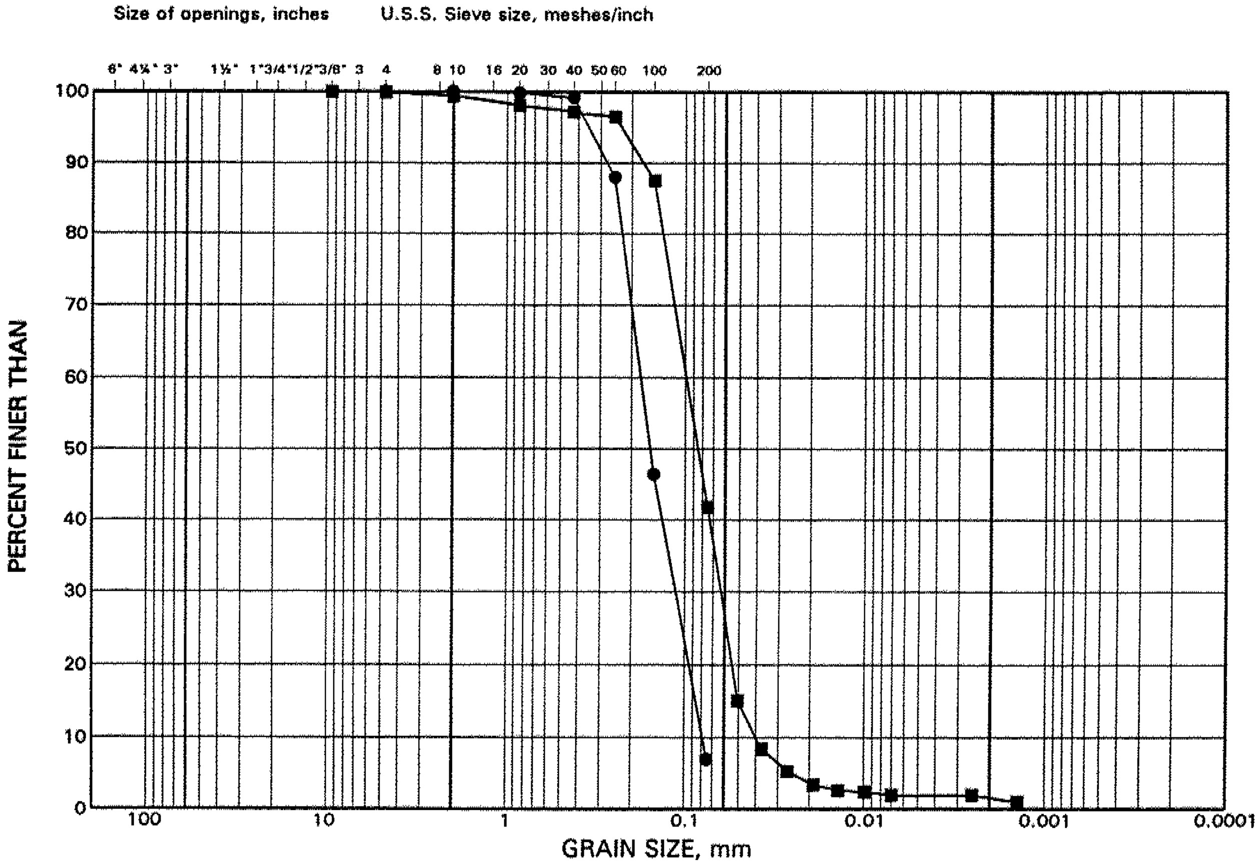
COBBLE	COARSE	MEDIUM	FINE	COARSE	MEDIUM	FINE	SILT SIZE	CLAY SIZE
SIZE	GRAVEL SIZE			SAND SIZE			FINE GRAINED	

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	97-9	11	182.3

GRAIN SIZE DISTRIBUTION
Sand to Silty Sand

FIGURE 13



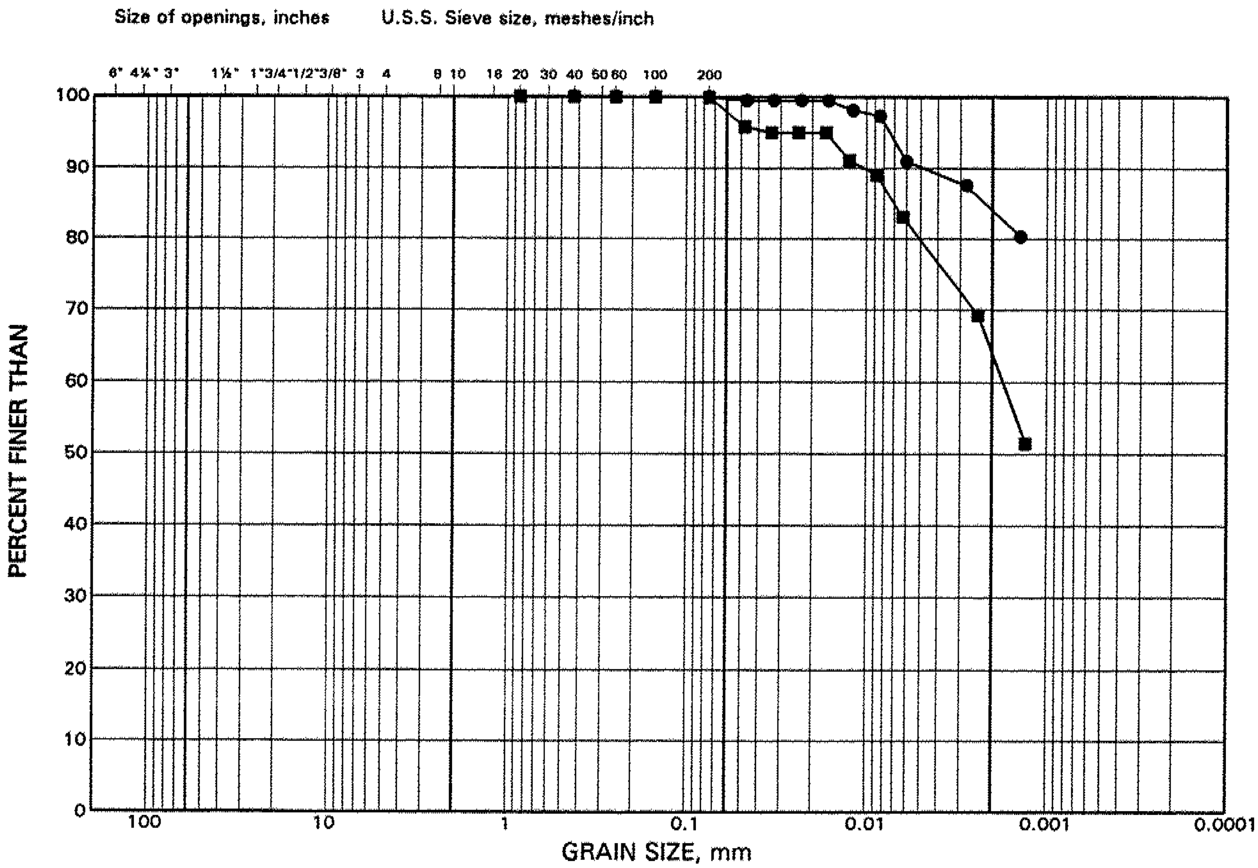
COBBLE	COARSE	MEDIUM	FINE	COARSE	MEDIUM	FINE	SILT SIZE	CLAY SIZE
SIZE	GRAVEL SIZE			SAND SIZE			FINE GRAINED	

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	97-9	13	179.2
■	97-10	12	179.2

GRAIN SIZE DISTRIBUTION
Clay to Silty Clay

FIGURE 14



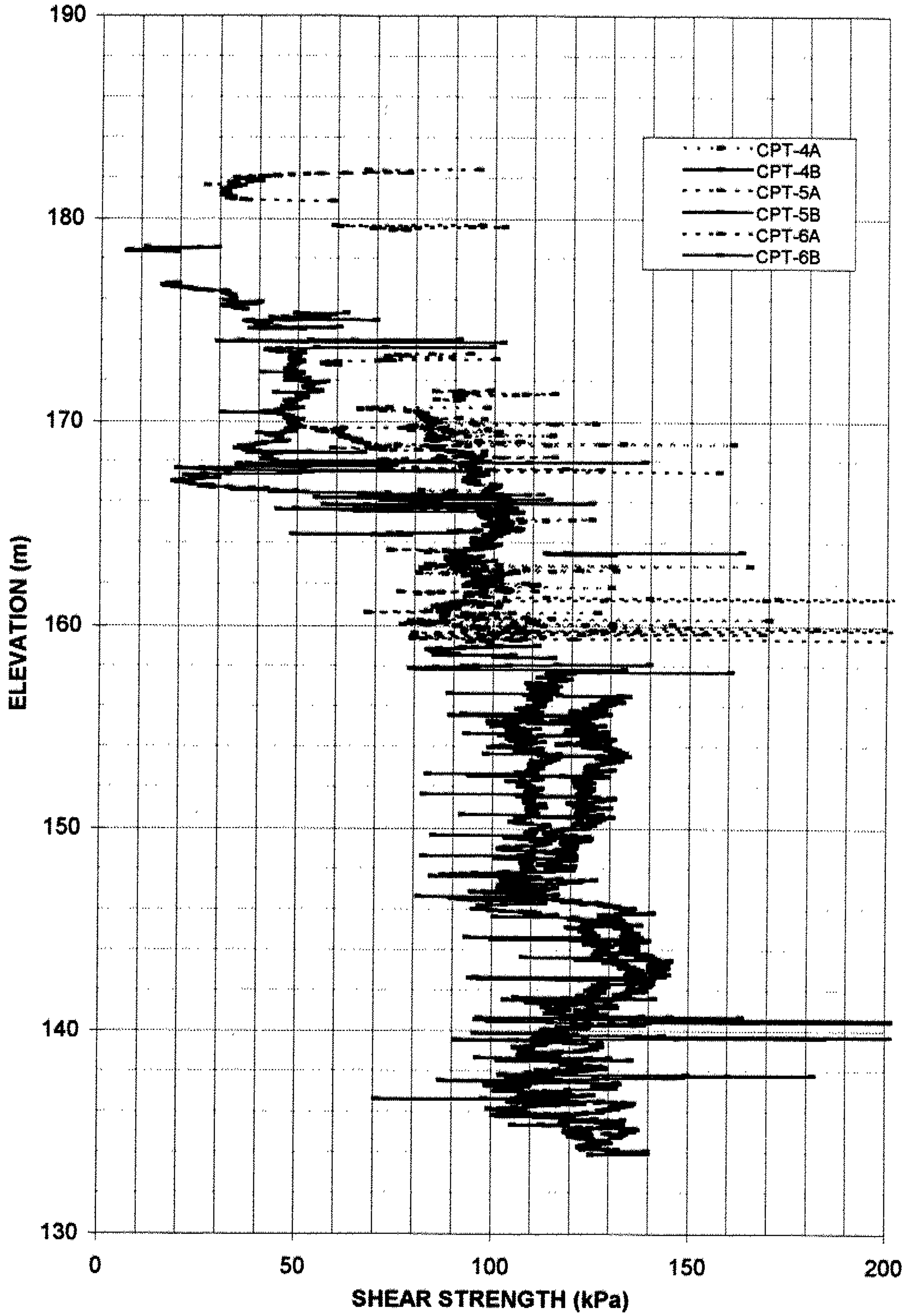
COBBLE	COARSE	MEDIUM	FINE	COARSE	MEDIUM	FINE	SILT SIZE	CLAY SIZE
SIZE	GRAVEL SIZE			SAND SIZE			FINE GRAINED	

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	97-12	6	172.4
■	97-12	27	145.0

IN-SITU VANE SHEAR STRENGTH
ROOT RIVER CROSSING AREA

FIGURE 15

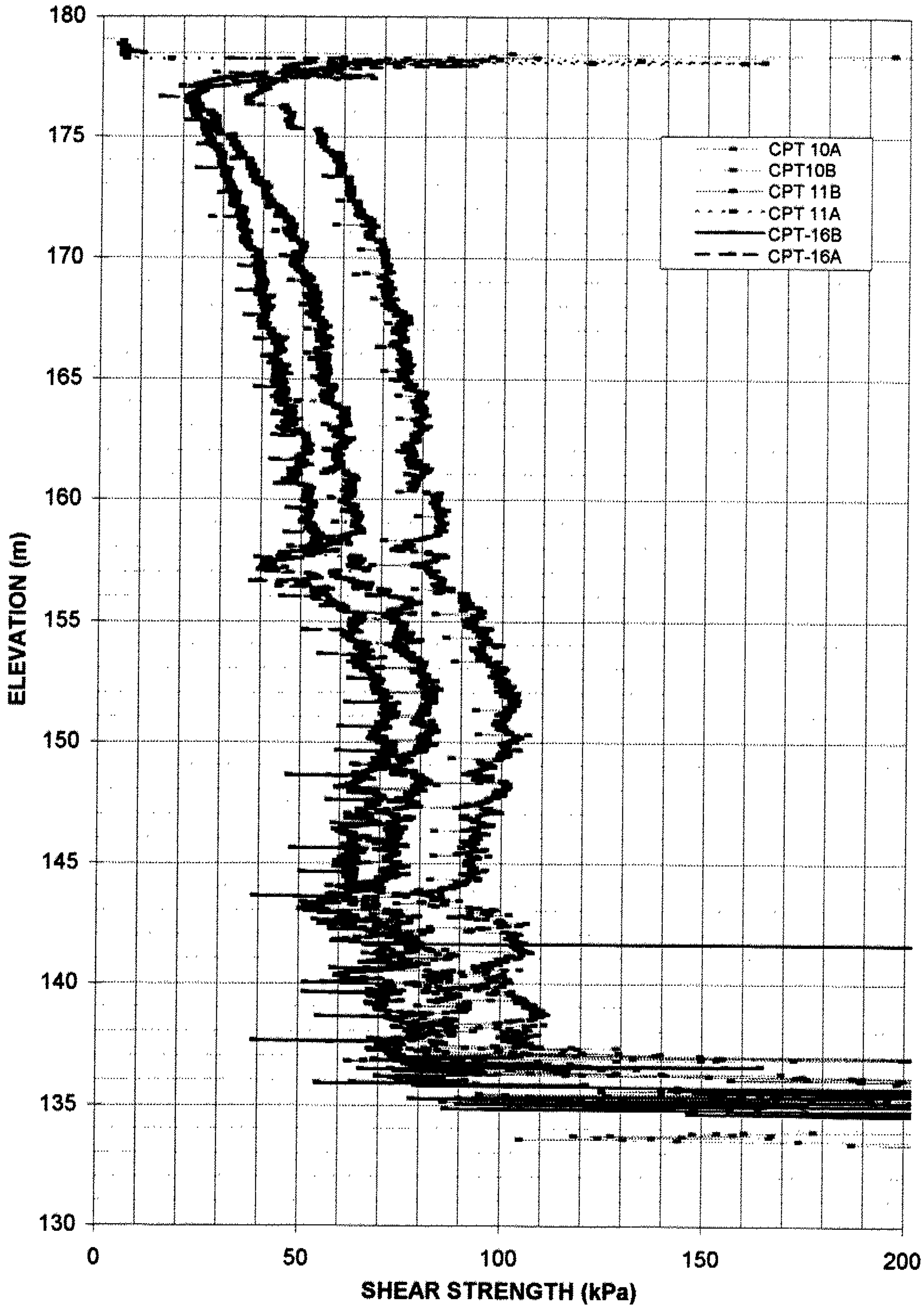


Date June 13, 1997
Project 97-1103

Golder Associates

Drawn AP
Chkd. ASP

IN-SITU VANE SHEAR STRENGTH LOWER ECHO RIVER CROSSING AREA	FIGURE 16



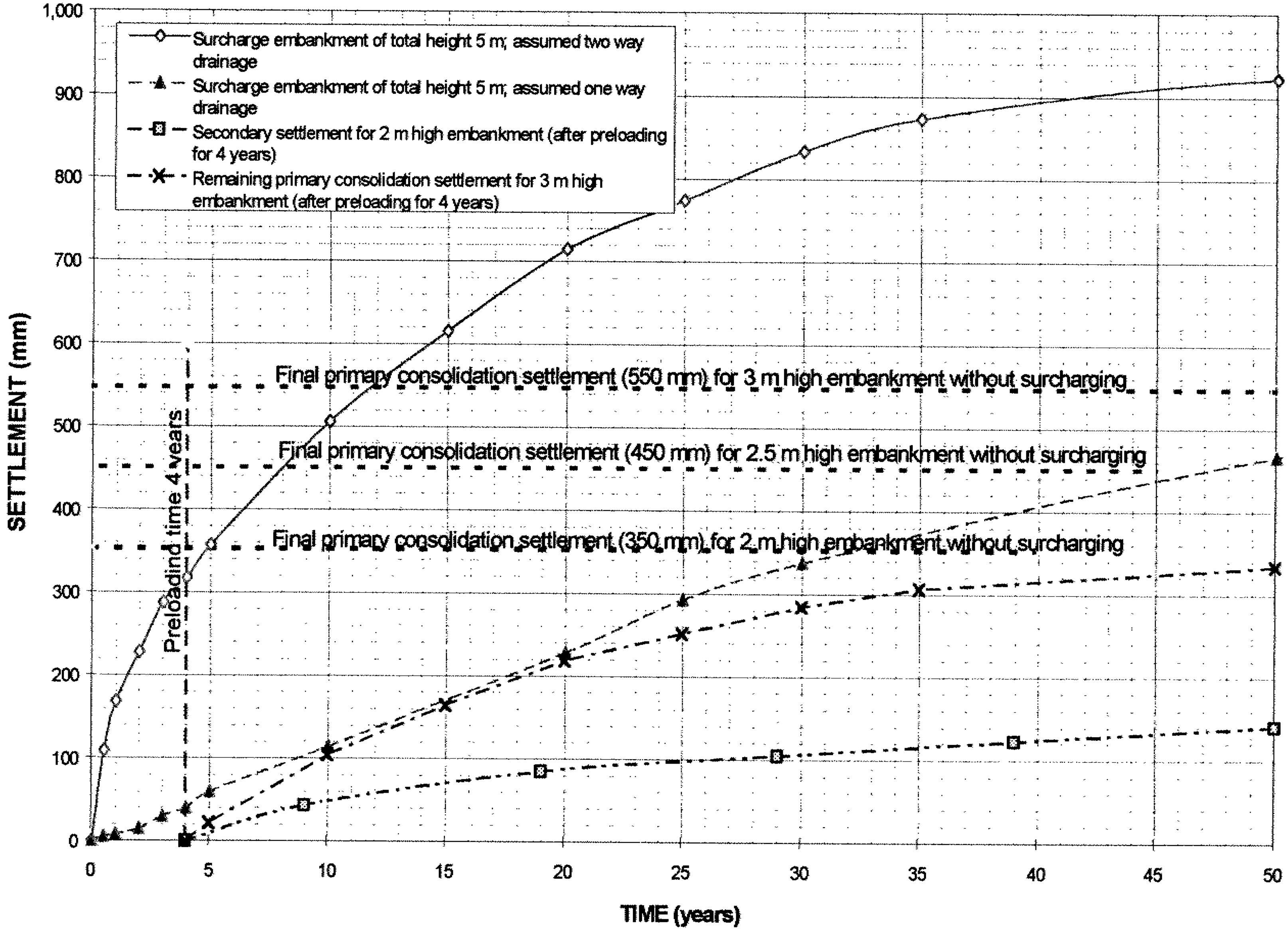
Date June 13, 1997
Project 97-1103

Golder Associates

Drawn AP
Chkd. ASP

Date June 13, 1997
Project 97-1103

Golder Associates



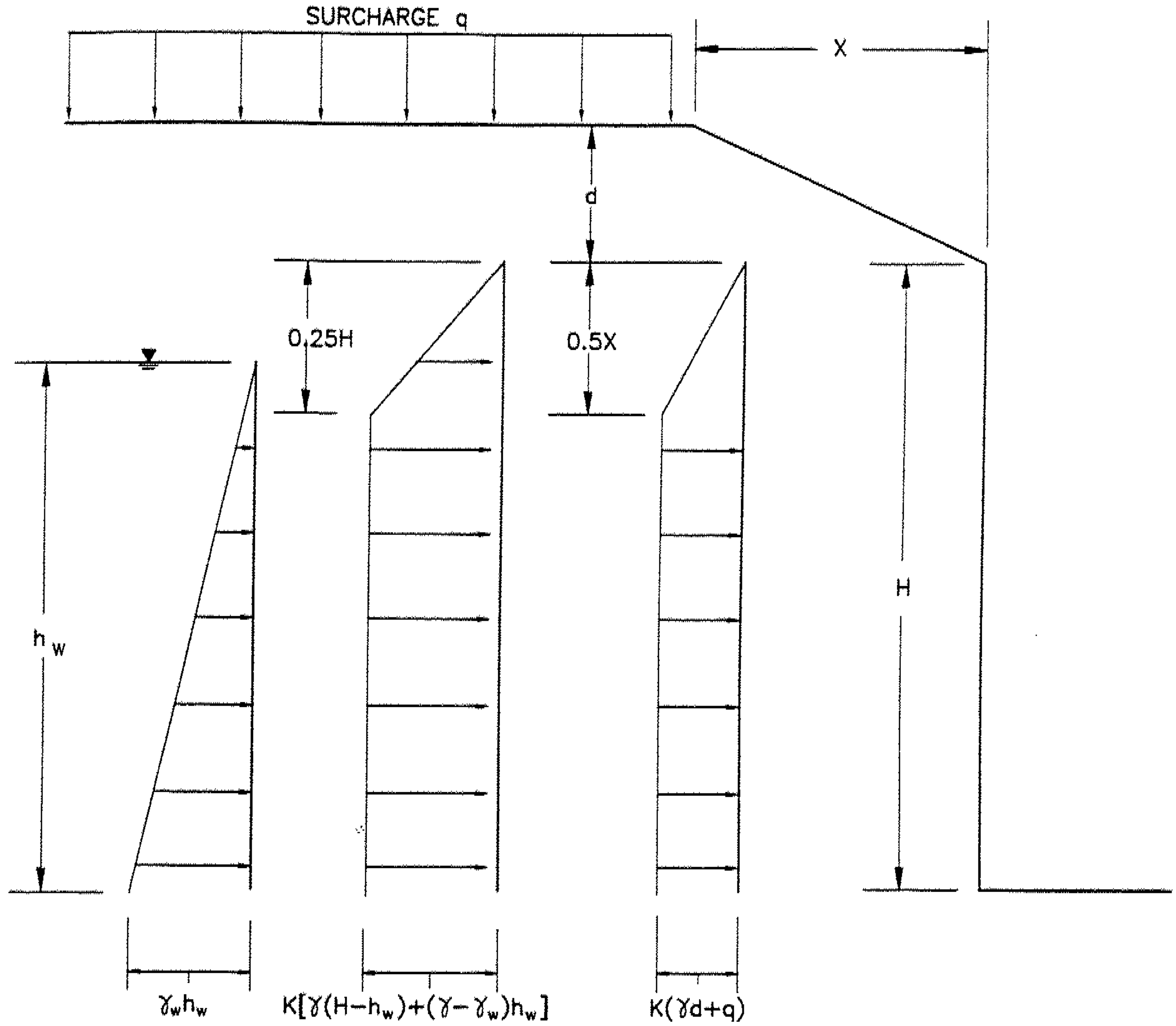
Drawn AP
Chkd. ASP

ESTIMATE OF SETTLEMENTS FOR 2 m AND 3 m HIGH ROAD EMBANKMENTS AND 5 m HIGH SURCHARGE EMBANKMENT LOWER ECHO RIVER AREA

FIGURE 17

DESIGN LATERAL EARTH PRESSURE DISTRIBUTION BRACED EXCAVATIONS IN COHESIVE SOILS

FIGURE 18



γ = UNIT WEIGHT OF SOIL

γ_w = UNIT WEIGHT OF WATER

K = EARTH PRESSURE COEFFICIENT

REFER TO TEXT OF REPORT FOR DESIGN VALUES

NOTES

1. FOR INTACT CLAY, USE h_w =0.
2. FOR HORIZONTAL GROUND BEHIND WALL, x=0

Date JUNE 1995

Project 971-1103

Golder Associates

Drawn KD

Chkd [signature]

P:\951-1315\K131506.DWG

APPENDIX A

RECORD OF BOREHOLE LOGS - MINISTRY OF TRANSPORTATION
BOREHOLE 3- REPORT W.P. 903-72-18

RECORD OF BOREHOLE SHEETS
AND LABORATORY TEST RESULTS
GOLDER ASSOCIATES LTD
BOREHOLES 95-4 TO 95-9 - REPORT NO. 961-1364

ENGINEERING SERVICES BRANCH - GEOTECHNICAL OFFICE - SOIL MECHANICS SECTION

RECORD OF BOREHOLE Nº 3

WP 903-72-18 | LOCATION Co-ords. 916,805 N; 963,978 E. Scheme 'B' | ORIGINATED BY HS

DIST 18 HWY 17 | BORING DATE October 6, 1975 | COMPILED BY HS

DATUM Geodetic | BOREHOLE TYPE Washboring, NX Casing | CHECKED BY [signature]

ELEV / DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES	GROUND WATER / ELEV	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH (○ UNCONFINED, ● QUICK TRIAXIAL, + FIELD VANE, × LAB VANE; 400 800 1200 1600 2000)	LIQUID LIMIT w_L / PLASTIC LIMIT w_P / WATER CONTENT w (WATER CONTENT %)	UNIT WEIGHT γ	REMARKS % GR SA SI CL
600.0	Ground Level									
0.0	Topsoil									
	Sandy silt									
595.0	Very Loose		1	SS	3					
5.0	Sand - Fine to Medium		2	SS	82	*				
	Trace to some gravel.					590				
			3	SS	7	▽				
	Loose		4	SS	4					
			5	SS	6	580				
	silty clay to clay Stiff						+ s3.2			
			6	SS	9					
567.5			7	SS	8	570				
32.5	Sand with gravel									
			8	SS	50					
558.5	Dense to Very Dense		9	SS	100/10"	560				
41.5	End of Borehole									
						550				

Note: Three attempts were made to set up the borehole, in order to avoid cobbles and/or boulders which are present in th upper 5 ft. of the granular deposit.

* Coarse gravel obstructed the advancement of the spoon in sample No.2, thus its blows/foot count is considered non-representative.

20
15 ⌖ 5 % STRAIN AT FAILURE

RECORD OF BOREHOLE 95-4

PROJECT: 941-1364
LOCATION: SEE FIGURE 2B
SAMPLER HAMMER, 63.5kg; DROP, 760mm
BORING DATE: SEPT.27-28/95
SHEET 1 OF 5
DATUM: GEODETIC
PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

J1364004.BHS

BORING METHOD: TRACK MOUNTED CME 55 POWER AUGER, 108mm ID HOLLOW STEM AUGERS

DEPTH SCALE METRES	SOIL PROFILE DESCRIPTION	ELEV. / DEPTH (m)	SAMPLE NUMBER	TYPE	BLOWS/0.3m	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
0	GROUND SURFACE	186.30					
	TOPSOIL	0.00					BENTONITE SEAL
	Sand, some silt, with layers of silty sand, sandy silt, sand and silt, silt and clayey silt Occasional organics Very loose to loose Brown to grey	186.00 0.30	1	50 DO	2		
1			2	50 DO	4		
2			3	50 DO	2		
	Silty clay, some sand and silt Very soft Brown and grey	183.70 2.60	4	50 DO	1		
3	Sand, some silt with layers of silty sand, sandy silt, sand and silt, silt and clayey silt with organic silt layers from 4.7m to 4.9m depth. Very loose to loose Brown to grey	182.90 3.40	5	50 DO	4		
4			6	50 DO	7		CUTTINGS
5			7	50 DO	2		
	Interlayered, sandy silt and clayey silt, occasional organics and silty clay seams Loose to very loose/soft Grey	5.00	8	50 DO	5	MH	
6			9	50 DO	4		
7	Sand, fine grading to coarse, trace silt and fine gravel Compact Grey to brown	179.10 7.20					▽
8			10	50 DO	17		
	Interlayered, sandy silt and clayey silt, trace organics. Very loose/soft Grey	177.60 8.70					
9			11	50 DO	1		
10	CONTINUED ON NEXT PAGE						

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m: 20 40 60 80
SHEAR STRENGTH Cu, kPa: nat V - + Q - ● rem V - ⊕ U - ○ : 20 40 60 80
HYDRAULIC CONDUCTIVITY, k, cm/s
WATER CONTENT, PERCENT: Wp ⊢—○W—⊣ Wl : 20 40 60 80

DATA INPUT PS JAN 22/96

DEPTH SCALE
1 to 50

Golder Associates

LOGGED: GEB
CHECKED: ASP

PROJECT: 941-1364

RECORD OF BOREHOLE 95-4

LOCATION: SEE FIGURE 2B

BORING DATE: SEPT.27-28/95

DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

BORING METHOD: TRACK MOUNTED CME 55 POWER AUGER, 108mm ID HOLLOW STEM AUGERS

CONTINUED FROM PREVIOUS PAGE

DEPTH SCALE METRES	SOIL PROFILE DESCRIPTION	ELEV. / DEPTH (m)	SAMPLE NUMBER	TYPE	BLOWS/0.3m	WATER CONTENT (W)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
10–14.8	Interlayered sandy silt and clayey silt, occ. organics Very loose/soft Grey							
11			12	50 DO	WH	o	MH	
12.5			13	50 DO	4			
13.2								CAVED
14			14	50 DO	3	o		
14.8–16.3	Silty clay Very soft Brownish grey to grey	171.50 / 14.80						
15.5			15	50 DO	WH	o		
16.3–17.8	Silty fine sand Very loose Grey	170.00 / 16.30						
17			16	50 DO	1	o		
17.8–20	Irregularly layered silty clay to clay and clayey silt to silt, trace sand Firm to stiff Greyish brown	168.50 / 17.80						
18.6			17	50 DO	1	o		
19.2								BENTONITE SEAL
20			18	50 DO	WH		MH	SAND

CONTINUED ON NEXT PAGE

DATA INPUT: PS JAN 22/96

DEPTH SCALE 1 to 50

LOGGED: GEB

CHECKED: ASP

SOILMS

PROJECT: 941-1364

RECORD OF BOREHOLE 95-4

LOCATION: SEE FIGURE 2B

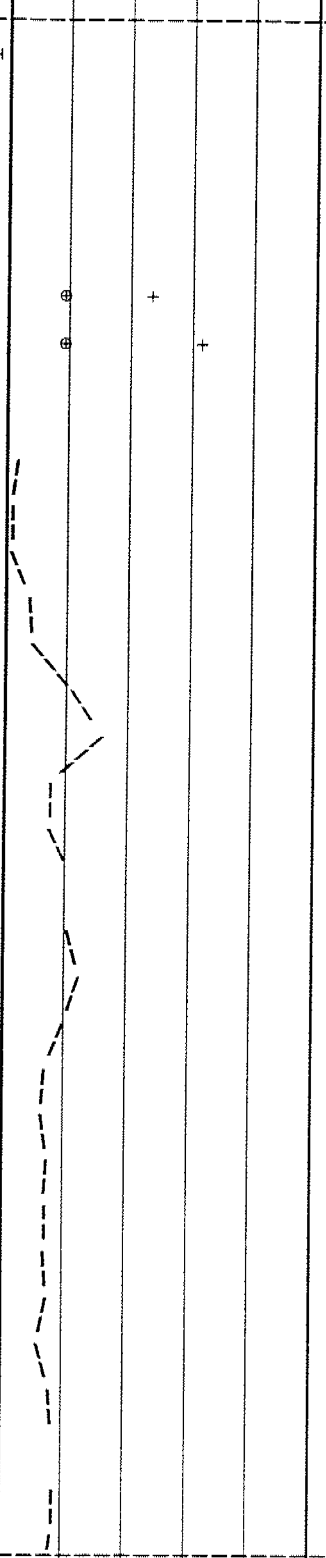
BORING DATE: SEPT.27-28/95

DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE – DESCRIPTION	STRATA PLOT	ELEV. / DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20, 40, 60, 80); SHEAR STRENGTH Cu, kPa (nat V - +, rem V - ⊕, Q - ●, U - O) (20, 40, 60, 80)	HYDRAULIC CONDUCTIVITY, k, cm/s; WATER CONTENT, PERCENT (Wp, W, Wl) (20, 40, 60, 80)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
20	TRACK MOUNTED CME 55 POWER AUGER; 108mm ID HOLLOW STEM AUGERS	CONTINUED FROM PREVIOUS PAGE									
		Irregularly layered silty clay to clay and clayey silt to silt, trace sand Firm to stiff Greyish brown			18	50 DO	WH			MH	
21											
22				164.05 22.25							
		END OF BOREHOLE Start of Dynamic Cone Test									Water level in augers at about 18.0m depth below ground surface upon completion of drilling. Water level in piezometer at Elev. 179.3m on October 5,1995.
23											
24											
25											
26											
27											
28											
29											
30		CONTINUED ON NEXT PAGE									



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DEPTH SCALE 1 to 50

Golder Associates

LOGGED: GEB

CHECKED: ASP

J1364004.BHS

PROJECT: 941-1364

LOCATION: SEE FIGURE 2B

SAMPLER HAMMER, 63.5kg; DROP, 760mm

RECORD OF BOREHOLE 95-4

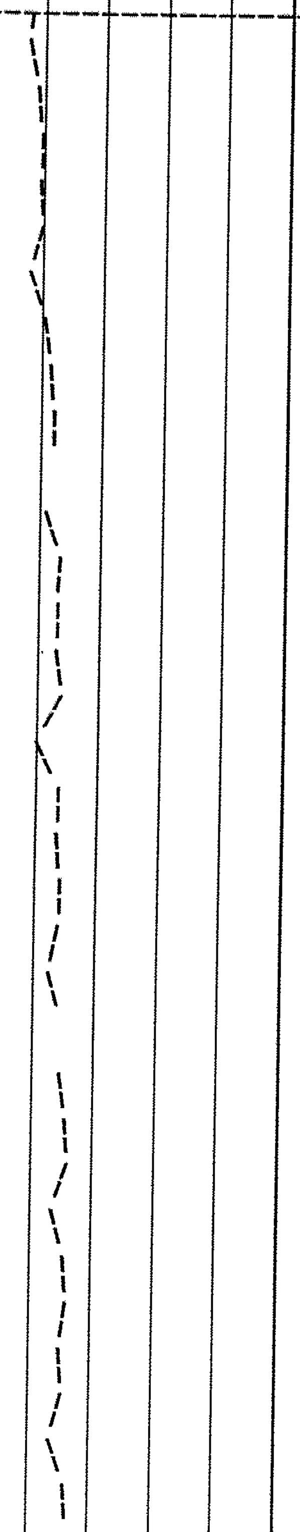
BORING DATE: SEPT.27-28/95

SHEET 4 OF 5

DATUM: GEODETIC

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES: NUMBER	TYPE	BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20 40 60 80); SHEAR STRENGTH Cu, kPa (20 40 60 80) nat V - + Q - ● rem V - ⊕ U - O	HYDRAULIC CONDUCTIVITY, k, cm/s; WATER CONTENT, PERCENT Wp —O W— Wl (20 40 60 80)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
30		CONTINUED FROM PREVIOUS PAGE									
31											
32											
33											
34											
35											
36											
37											
38											
39											
40		CONTINUED ON NEXT PAGE									



DEPTH SCALE

1 to 50

Golder Associates

LOGGED: GEB

CHECKED: ASP

J1364004 BHS

PROJECT: 941-1364

RECORD OF BOREHOLE 95-4

SHEET 5 OF 5

LOCATION: SEE FIGURE 2B

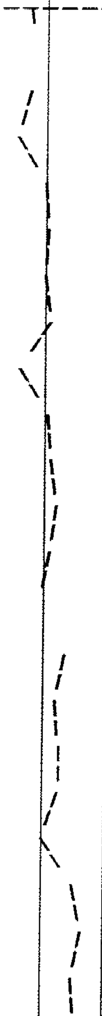
BORING DATE: SEPT.27-28/95

DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES: NUMBER	TYPE	BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20 40 60 80); SHEAR STRENGTH Cu, kPa (nat V - + Q - ● rem V - ⊕ U - O) (20 40 60 80)	HYDRAULIC CONDUCTIVITY, k, cm/s; WATER CONTENT, PERCENT Wp ├──O W──┤ Wl (20 40 60 80)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
40		CONTINUED FROM PREVIOUS PAGE									
41											
42											
43											
44											
45											
46											
		End of Dynamic Cone Test		139.60 46.70							
47											
48											
49											
50											



DATA INPUT: PS JAN.22/96

DEPTH SCALE

1 to 50

LOGGED: GEB

CHECKED: ASP

Golder Associates

SOILM6

PROJECT: 941-1364

LOCATION: SEE FIGURE 2C

SAMPLER HAMMER, 63.5kg; DROP, 760mm

RECORD OF BOREHOLE 95-5

BORING DATE: SEPT.29/95

DATUM: GEODETIC

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

BORING METHOD: TRACK MOUNTED CME 55 POWER AUGER, 108mm ID HOLLOW STEM AUGERS

DEPTH SCALE METRES	SOIL PROFILE DESCRIPTION	ELEV. / DEPTH (m)	SAMPLE NUMBER	TYPE	BLOWS/0.3m	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
0	GROUND SURFACE	182.20					
	TOPSOIL	0.00					BENTONITE SEAL
	Silt, some sand and clay with interlayers of sandy silt to sand and clayey silt, trace organics. Very loose to loose. Greyish brown to brownish grey.	0.10	1	50 DO	4		
1			2	50 DO	8		
2			3	50 DO	6	MH	CUTTINGS
3			4	50 DO	2		
			5	50 DO	WH		
4	Sand, fine, trace silt and clay, occ. organics and grey silt to clayey silt seams. Very loose to compact. Greyish and reddish brown to brownish grey	178.20 / 4.00	6	50 DO	6	MH	BENTONITE SEAL (water level)
5			7	50 DO	5		
6			8	50 DO	6		
			9	50 DO	12		
7							CAVED
8			10	50 DO	4		
9			11	50 DO	14		
10	CONTINUED ON NEXT PAGE						

Chart columns: DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m; SHEAR STRENGTH Cu, kPa (nat V - +, rem V - ⊕, Q - ●, U - ○); HYDRAULIC CONDUCTIVITY, k, cm/s; WATER CONTENT, PERCENT (Wp ⊢—○W—⊣ Wl) 20 40 60 80

CONTINUED ON NEXT PAGE

DEPTH SCALE 1 to 50

Golder Associates

LOGGED: GEB

CHECKED: ASP

DATA INPUT: PS JAN.22/96

PROJECT: 941-1364

RECORD OF BOREHOLE 95-5

SHEET 2 OF 2

LOCATION: SEE FIGURE 2C

BORING DATE: SEPT.29/95

DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m; SHEAR STRENGTH Cu, kPa (nat V - +, rem V - ⊕, Q - ●, U - O)	HYDRAULIC CONDUCTIVITY, k, cm/s; WATER CONTENT, PERCENT (Wp, W, Wl; 20 40 60 80)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
10	TRACK MOUNTED CME 55 POWER AUGER, 108mm ID HOLLOW STEM AUGERS	CONTINUED FROM PREVIOUS PAGE									
11		Sand, fine, trace silt and clay, occ. organics and grey silt to clayey silt seams. Very loose to compact Greyish and reddish brown to brownish grey.			12	50 DO	5		O		
12											CAVED
					13	50 DO	1		O		
13											
14											
15					14	50 DO	10		O		
				166.35 / 15.85							
16		END OF BOREHOLE									Water level in piezometer at Elev. 177.9m depth on October 5,1995.
17											
18											
19											
20											

DATA INPUT PS JAN 22/96

DEPTH SCALE

1 to 50

Golder Associates

LOGGED: GEB

CHECKED: ASP

J1364006.BHS

PROJECT: 941-1364

LOCATION: SEE FIGURE 2D

SAMPLER HAMMER, 63.5kg; DROP, 760mm

RECORD OF BOREHOLE 95-6

BORING DATE: OCT 2/95

SHEET 1 OF 2

DATUM: GEODETIC

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

BORING METHOD: TRACK MOUNTED CME 55 POWER AUGER, 95mm ID HOLLOW STEM AUGERS

DEPTH SCALE METRES	SOIL PROFILE – DESCRIPTION	ELEV. / DEPTH (m)
0	GROUND SURFACE	188.50
	TOPSOIL	0.00
	Silty sand, some rootlets. Very loose Brown	188.35 / 0.15
	Sand, trace to some silt, trace fine gravel Very loose to loose Brown	188.10 / 0.40
2	Silty sand, trace organics Very loose to loose Brown	186.50 / 2.00
	Interlayered, sandy silt and clayey silt, occ. seams of silty clay with trace sand, organic partings from 2.20m to 3.7m depth. Very soft to soft/very loose to loose Grey	186.30 / 2.20
7	Interlayered silty sand, clayey silt and sandy silt. Occasional organic seams Very loose to loose Grey	181.50 / 7.00
8	Sand, trace silt Loose Brownish grey	180.40 / 8.10
9	Irregularly layered silty clay to clay and clayey silt, trace sand, occ. grey silt seam. Firm to stiff Greyish to reddish brown	179.30 / 9.20
10	CONTINUED ON NEXT PAGE	

SAMPLE NUMBER	TYPE	BLOWS/0.3m	NOTES
1	50 DO	3	
2	50 DO	9	
3	50 DO	5	
4	50 DO	WH	
5	50 DO	1	
6	50 DO	WH	non-plastic; ADDITIONAL LAB. TESTING: MH
7	50 DO	PM	
8	50 DO	5	
9	50 DO	2	

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m

SHEAR STRENGTH Cu, kPa: nat V - + Q - ● ; rem V - ⊕ U - O (20, 40, 60, 80)

HYDRAULIC CONDUCTIVITY, k, cm/s

WATER CONTENT, PERCENT: Wp ├──O W──┤ Wl (20, 40, 60, 80)

PIEZOMETER OR STANDPIPE INSTALLATION: CUTTINGS

DEPTH SCALE 1 to 50

Golder Associates

LOGGED: GEB

CHECKED: ASP

J1364006.BHS

PROJECT: 941-1364

LOCATION: SEE FIGURE 2D

SAMPLER HAMMER, 63.5kg; DROP, 760mm

RECORD OF BOREHOLE 95-6

BORING DATE: OCT.2/95

DATUM: GEODETIC

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE – DESCRIPTION	STRATA PLOT	ELEV. / DEPTH (m)	SAMPLES – NUMBER	SAMPLES – TYPE	SAMPLES – BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m; SHEAR STRENGTH Cu, kPa (nat V - +, rem V - ⊕, Q - ●, U - O) 20 40 60 80	HYDRAULIC CONDUCTIVITY, k, cm/s; WATER CONTENT, PERCENT Wp —O W— Wl 20 40 60 80	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
10	TRACK MOUNTED CME 55 POWER AUGER; 95mm ID HOLLOW STEM AUGERS	CONTINUED FROM PREVIOUS PAGE									
11–15		Irregularly layered silty clay to clay and clayey silt, trace sand, occ. grey silt seams Firm to stiff Greyish to reddish brown			10	50 DO	WH				BENTONITE SEAL
											SAND
					11	50 DO	WH				CAVED
16		END OF BOREHOLE		172.65 15.85							Water level in augers at about 14.0m depth below ground surface upon completion of drilling. Water level in piezometer at Elev. 185.6m on October 5,1995.
17											
18											
19											
20											

DATA INPUT. PS JAN 22/96

DEPTH SCALE

1 to 50

Golder Associates

LOGGED: GEB

CHECKED: ASP

SOILM6

J1364007.BHS

PROJECT: 941-1364

LOCATION: SEE FIGURE 2E

SAMPLER HAMMER, 63.5kg; DROP, 760mm

RECORD OF BOREHOLE 95-7

BORING DATE: OCT.2-3/95

SHEET 1 OF 3

DATUM: GEODETIC

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

BORING METHOD: TRACK MOUNTED CME 55 POWER AUGER, 95mm ID HOLLOW STEM AUGERS

SOIL PROFILE DESCRIPTION	ELEV. / DEPTH (m)	SAMPLE NUMBER	TYPE	BLOWS/0.3m	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
GROUND SURFACE	194.30					
TOPSOIL	0.00					BENTONITE SEAL
Sand, some gravel, trace silt Fine to coarse, occ. organic Loose Brown	0.15	1	50 DO	6		
Coarse sand to sand and gravel Compact Brown	193.60 / 0.70	2	50 DO	15		
Medium to coarse sand Loose Brown	192.90 / 1.40					SAND
Fine sand, some silt Loose Brown	192.50 / 1.80	3	50 DO	9		
		4	50 DO	8		
Interlayered sand, silty sand and sandy silt, occ. organics Loose to very loose Brown	191.30 / 3.00	5	50 DO	4		CUTTINGS
		6	50 DO	2		
Interlayered silty sand and sandy silt, trace clay, occ. fine sand layer, frequent organics Very loose Grey	190.00 / 4.30	7	50 DO	2	MH	
		8	50 DO	3		
		9	50 DO	1		
-Free water in samples 6 to 9		10	50 DO	2		
Sand, medium to coarse, trace silt and gravel Compact Brown	185.60 / 8.70	11	50 DO	23		
CONTINUED ON NEXT PAGE						CAVED

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m

SHEAR STRENGTH Cu, kPa — nat V - + Q - ● rem V - ⊕ U - O

HYDRAULIC CONDUCTIVITY, k, cm/s

WATER CONTENT, PERCENT Wp —O W— Wl: 20 40 60 80

DEPTH SCALE 1 to 50

Golder Associates

LOGGED: GEB

CHECKED: ASP

DATA INPUT: JAN 22/96

J1364007.BHS

PROJECT: 941-1364

LOCATION: SEE FIGURE 2E

SAMPLER HAMMER, 63.5kg; DROP, 760mm

RECORD OF BOREHOLE 95-7

BORING DATE: OCT.2-3/95

SHEET 2 OF 3

DATUM: GEODETIC

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

BORING METHOD: TRACK MOUNTED CME 55 POWER AUGER, 95mm ID HOLLOW STEM AUGERS

CONTINUED FROM PREVIOUS PAGE

DESCRIPTION	ELEV. / DEPTH (m)
	184.10 / 10.20
Sandy silt, Very loose, Grey to brown	183.40 / 10.90
Sand, medium to coarse, trace silt and gravel. Compact, Brown	181.00 / 13.30
Sandy silt, occ. organics, Very loose, Grey	179.70 / 14.60
Gravel, some sand and silt, Very dense, Brown. -Free water in sample 15	178.00 / 16.30
Sand, fine, trace silt and clay, Very loose to loose, Brown to reddish grey	

SAMPLE NUMBER	TYPE	BLOWS/0.3m
12	50 DO	14
13	50 DO	21
14	50 DO	2
15	50 DO	69
16	50 DO	4
17	50 DO	3

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m

SHEAR STRENGTH Cu, kPa: nat V - + Q - ●; rem V - ⊕ U - O

HYDRAULIC CONDUCTIVITY, k, cm/s

WATER CONTENT, PERCENT: Wp —O W— Wl (20, 40, 60, 80)

ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION: CAVED & BENTONITE; BENTONITE SEAL; CAVED

Depth scale (metres): 10, 11, 12, 13, 14, 15, 16, 17, 18, 19, 20

CONTINUED ON NEXT PAGE

DEPTH SCALE 1 to 50

Golder Associates

LOGGED: GEB

CHECKED: ASP

DATA INPUT: PS JAN 22/96

SOIL M6

J1364007.BHS

PROJECT: 941-1364

LOCATION: SEE FIGURE 2E

SAMPLER HAMMER, 63.5kg; DROP, 760mm

RECORD OF BOREHOLE 95-7

BORING DATE: OCT 2-3/95

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

SHEET 3 OF 3

DATUM: GEODETIC

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. / DEPTH (m)	SAMPLES: NUMBER	TYPE	BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m; SHEAR STRENGTH Cu, kPa (nat V - + Q - ●; rem V - ⊕ U - ○)	HYDRAULIC CONDUCTIVITY, k, cm/s; WATER CONTENT, PERCENT (Wp ├─○W─┤ Wl; 20 40 60 80)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
20	TRACK MOUNTED CME 55 POWER AUGER; 95mm ID HOLLOW STEM AUGERS	CONTINUED FROM PREVIOUS PAGE									
20–25		Sand, fine, trace silt and clay Very loose to loose Brown to reddish grey			18	50 DO	3		○	MH	CAVED
					19	50 DO	8		○		
25				169.31 24.99							
25–27.43		Sand (inferred from easy auger advance)									
		END OF BOREHOLE Note: Sand "blowback" of 0.6m and 2.4m inside augers when sampling at 19m and 25m depths respectively.		166.87 27.43							Water level in augers at about 13.7m depth below ground surface upon completion of drilling. Water level in piezometer at Elev. 177.2m on October 6,1995.

Depth scale: 20, 21, 22, 23, 24, 25, 26, 27, 28, 29, 30

JAN 22

TA INP

DEPTH SCALE

1 to 50

Golder Associates

LOGGED: GEB

CHECKED: ASP

J1364008 BHS

PROJECT: 941-1364

LOCATION: SEE FIGURE 2F

SAMPLER HAMMER, 63.5kg; DROP, 760mm

RECORD OF BOREHOLE 95-8

BORING DATE: OCT.4/95

DATUM: GEODETIC

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE – DESCRIPTION	ELEV. / DEPTH (m)	SAMPLES – NUMBER	TYPE	BLOWS/0.3m	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
0	TRACK MOUNTED CME 55 POWER AUGER; 95mm ID HOLLOW STEM AUGERS	GROUND SURFACE	193.70					
		Organic sandy silt Very loose Dark brown	0.00					
		Interlayered fine sand, some silt to sandy silt, trace clay, occ. clayey silt seams, frequent organic partings. Very loose Brown, becoming grey below 1m depth.	193.40 0.30	1	50 DO	4		
1				2	50 DO	2		CUTTINGS
2				3	50 DO	1		
				4	50 DO	2		
3				5	50 DO	1		
4				6	50 DO	WH		BENTONITE SEAL
5				7	50 DO	WH		
				8	50 DO	WH		
6				9	50 DO	WH	MH	
7		Free water noted in samples 1 to 7						
8		Sand, coarse to fine, some silt, trace fine gravel Occ. grey silt parting layer Compact Brown	185.70 8.00	10	50 DO	14		
9								CUTTINGS
				11	50 DO	10		
10								

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m

SHEAR STRENGTH Cu, kPa — nat V - + Q - ● rem V - ⊕ U - ○

HYDRAULIC CONDUCTIVITY, k, cm/s

WATER CONTENT, PERCENT — Wp ⊢ W ○ ⊣ Wl — 20 40 60 80

CONTINUED ON NEXT PAGE

DEPTH SCALE 1 to 50

Golder Associates

LOGGED: GEB

CHECKED: ASP

DATA INPUT. PS JAN 22/96

SOIL M6

J1364008.BHS

PROJECT: 941-1364

LOCATION: SEE FIGURE 2F

SAMPLER HAMMER, 63.5kg; DROP, 760mm

RECORD OF BOREHOLE 95-8

BORING DATE: OCT.4/95

SHEET 2 OF 2

DATUM: GEODETIC

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE – DESCRIPTION	STRATA PLOT	ELEV. / DEPTH (m)	SAMPLES – NUMBER	TYPE	BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m; SHEAR STRENGTH Cu, kPa (nat V - +, rem V - ⊕, Q - ●, U - ○)	HYDRAULIC CONDUCTIVITY, k, cm/s; WATER CONTENT, PERCENT Wp —○W— Wl (20, 40, 60, 80)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
10	TRACK MOUNTED CME 55 POWER AUGER; 95mm ID HOLLOW STEM AUGERS	CONTINUED FROM PREVIOUS PAGE									
				183.50 / 10.20							
11		Medium to coarse sand and gravel, trace silt and clay, fine to medium sand layers from 10.9m to 11.7m depth. Compact to very dense Brown			12	50 DO	62		○		CAVED
12				181.10 / 12.60	13	50 DO	15		○	MH	
13		Sand, fine to medium, trace gravel and silt Loose Brown									SAND FILTER & CAVED
14				179.37 / 14.33	14	50 DO	4		○		
		END OF BOREHOLE									Water level in augers at about 12.8m depth below ground surface upon completion of drilling. Water level in piezometer at Elev. 181.6m on October 6,1995.
15											
16											
17											
18											
19											
20											

DATA INPUT: [illegible] JAN 22/96

DEPTH SCALE
1 to 50

Golder Associates

LOGGED: GEB
CHECKED: ASP

RECORD OF BOREHOLE 95-9

PROJECT: 941-1364
LOCATION: SEE FIGURE 2G
SAMPLER HAMMER, 63.5kg; DROP, 760mm

BORING DATE: SEPT.19-21/95

DATUM: GEODETIC
PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

BORING METHOD: TRACK MOUNTED CME 55 POWER AUGER, 108mm ID HOLLOW STEM AUGERS

Column headings: DEPTH SCALE METRES | BORING METHOD | SOIL PROFILE (DESCRIPTION, STRATA PLOT, ELEV. DEPTH (m)) | SAMPLES (NUMBER, TYPE, BLOWS/0.3m) | DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m | SHEAR STRENGTH Cu, kPa (nat V - +, Q - ●, rem V - ⊕, U - ○) 20 40 60 80 | HYDRAULIC CONDUCTIVITY, k, cm/s | WATER CONTENT, PERCENT (Wp, W, Wl) 20 40 60 80 | ADDITIONAL LAB TESTING | PIEZOMETER OR STANDPIPE INSTALLATION

Description	Elev. / Depth (m)
GROUND SURFACE	178.50
TOPSOIL	0.00 / 178.30
Silty sand, some rootlets Loose Brown	0.20 / 177.70
Irregularly layered silty clay with grey silt laminations Soft Brown to greyish brown	0.80 / 177.10
Irregularly layered silty clay to clay and clayey silt to silt, trace sand Soft to firm Greyish brown	1.40

Sample Number	Type	Blows/0.3m
1	50 DO	4
2	50 DO	4
3	50 DO	1
4	50 DO	WH
5	50 DO	PM
6	50 DO	PM

Piezometer or standpipe installation: CUTTINGS

CONTINUED ON NEXT PAGE

DEPTH SCALE
1 to 50

LOGGED: GEB
CHECKED: ASP

Golder Associates

J1364009.BHS
DATA INPUT PS JAN 22/96
SOILMS

PROJECT: 941-1364

RECORD OF BOREHOLE 95-9

LOCATION: SEE FIGURE 2G

BORING DATE: SEPT.19-21/95

DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE – DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m / SHEAR STRENGTH Cu, kPa (nat V - +, rem V - ⊕, Q - ●, U - ○) 20 40 60 80	HYDRAULIC CONDUCTIVITY, k, cm/s / WATER CONTENT, PERCENT Wp —○W— Wl 20 40 60 80	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
8	TRACK MOUNTED CME 55 POWER AUGER; 108mm ID HOLLOW STEM AUGERS	CONTINUED FROM PREVIOUS PAGE									
		Irregularly layered silty clay to clay and clayey silt to silt, trace sand Soft to firm Greyish brown									CUTTINGS
9					7	50 DO	WR				
11					8	75 TO	PH				
12					9	50 DO	PM				
15					10	50 DO	WR				
18		CONTINUED ON NEXT PAGE									

DEPTH SCALE
1 to 50

Golder Associates

LOGGED: GEB
CHECKED: ASP

PROJECT: 941-1364

RECORD OF BOREHOLE 95-9

LOCATION: SEE FIGURE 2G

BORING DATE: SEPT.19-21/95

DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE / SHEAR STRENGTH Cu, kPa (nat V - +, rem V - ⊕, Q - ●, U - ○)	HYDRAULIC CONDUCTIVITY, k, cm/s / WATER CONTENT, PERCENT (Wp, W, Wl)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
18	TRACK MOUNTED CME 55 POWER AUGER; 108mm ID HOLLOW STEM AUGERS	CONTINUED FROM PREVIOUS PAGE									
		Irregularly layered silty clay to clay and clayey silt to silt, trace sand Soft to firm Greyish brown			11	50 DO	WR				
20					12	75 TO	PH			MH C	
22					13	50 DO	WR				
23		Irregularly layered silty clay to clay and clayey silt, trace sand. Stiff Grey to greyish brown, becoming reddish brown with grey silt interlayers below 30.1m depth.		155.60 22.90							
24					14	50 DO	WR				
25											CUTTINGS
27					15	50 DO	WR				
28		CONTINUED ON NEXT PAGE									

DEPTH SCALE
1 to 50

Golder Associates

LOGGED: GEB
CHECKED: ASP

J1364009 BHS
DATA INPUT: PS JAN 22/96
SO.1.M6

J1364009.BHS

PROJECT: 941-1364

RECORD OF BOREHOLE 95-9

LOCATION: SEE FIGURE 2G

BORING DATE: SEPT.19-21/95

DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES: NUMBER	TYPE	BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m; SHEAR STRENGTH Cu, kPa (nat V - +, rem V - ⊕, Q - ●, U - ○) 20 40 60 80	HYDRAULIC CONDUCTIVITY, k, cm/s; WATER CONTENT, PERCENT Wp — W — Wl 20 40 60 80	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
28	TRACK MOUNTED CME 55 POWER AUGER; 108mm ID HOLLOW STEM AUGERS	CONTINUED FROM PREVIOUS PAGE			15	50 DO	WR				
29					16	75 TO	PH				
30											
31					17	50 DO	WR				
32		Irregularly layered silty clay to clay and clayey silt, trace sand. Stiff. Grey to greyish brown, becoming reddish brown with grey silt interlayers below 30.1m depth.									
33											CUTTINGS
34					18	50 DO	WR				
35											
36											
37					19	50 DO	WH				BENTONITE SEAL
38		CONTINUED ON NEXT PAGE									

DATA INPUT: PS JAN.22/96

SOILM6

DEPTH SCALE

1 to 50

Golder Associates

LOGGED: GEB

CHECKED: ASP

PROJECT: 941-1364

LOCATION: SEE FIGURE 2G

SAMPLER HAMMER, 63.5kg; DROP, 760mm

RECORD OF BOREHOLE 95-9

BORING DATE: SEPT.19-21/95

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

DATUM: GEODETIC

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m / SHEAR STRENGTH Cu, kPa (nat V - +, rem V - ⊕, Q - ●, U - O) 20 40 60 80	HYDRAULIC CONDUCTIVITY, k, cm/s / WATER CONTENT, PERCENT Wp —O W— Wl 20 40 60 80	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
38	TRACK MOUNTED CME 55 POWER AUGER; 108mm ID HOLLOW STEM AUGERS	CONTINUED FROM PREVIOUS PAGE									
		Varved silty clay to clay and clayey silt, trace sand. Stiff. Grey to greyish brown, becoming reddish brown with grey silt varves below 30.1m depth.									SAND
39					20	75 TO	PH			MH C	
					21	50 DO	WH				
				138.57							
40		END OF BOREHOLE		39.93							Water level at 1.32m above ground surface on October 5,1995.
41											
42											
43											
44											
45											
46											
47											
48											

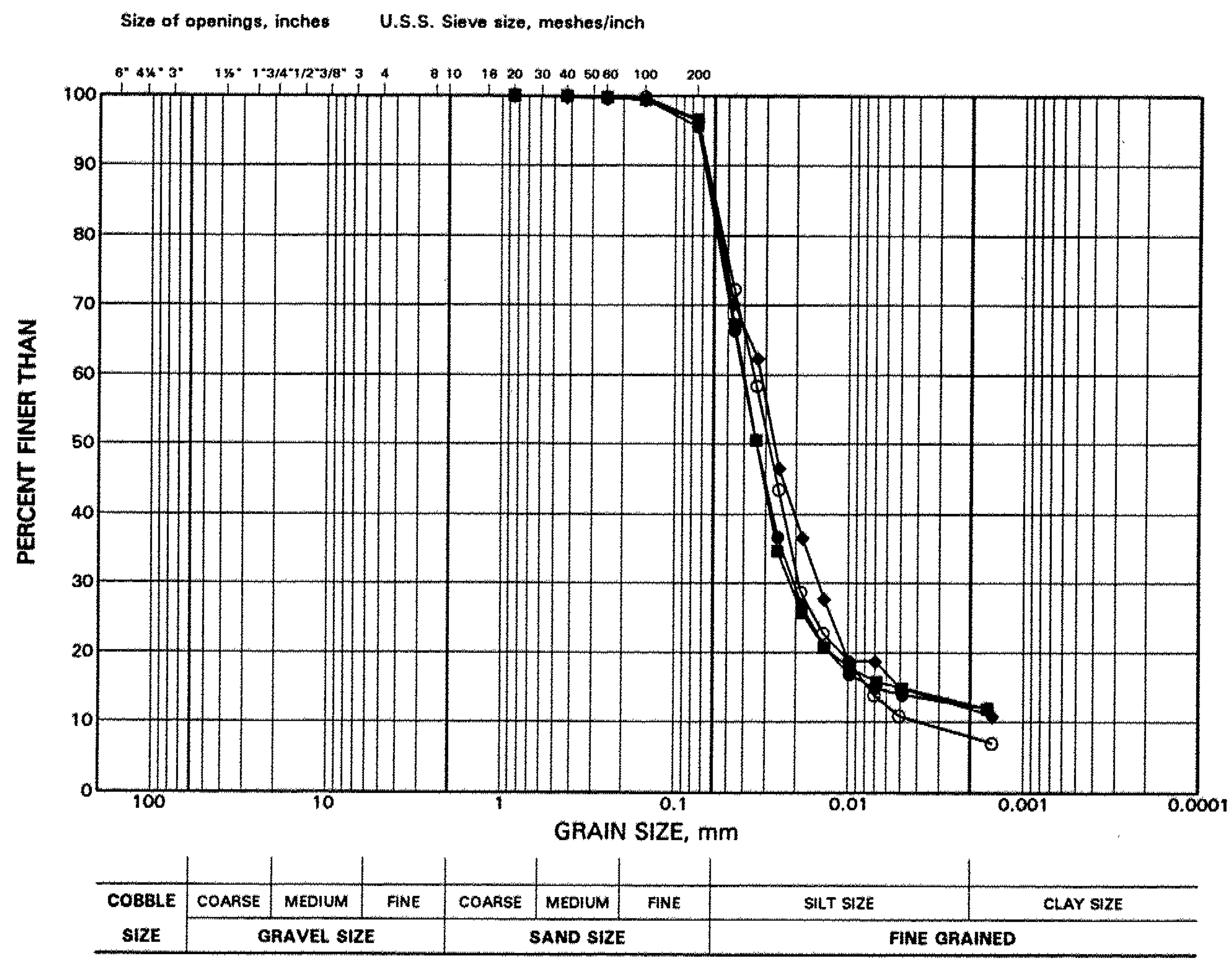
DEPTH SCALE
1 to 50

Golder Associates

LOGGED: GEB
CHECKED: ASP

GRAIN SIZE DISTRIBUTION
Interlayered Sandy Silt and Clayey Silt

FIGURE 6

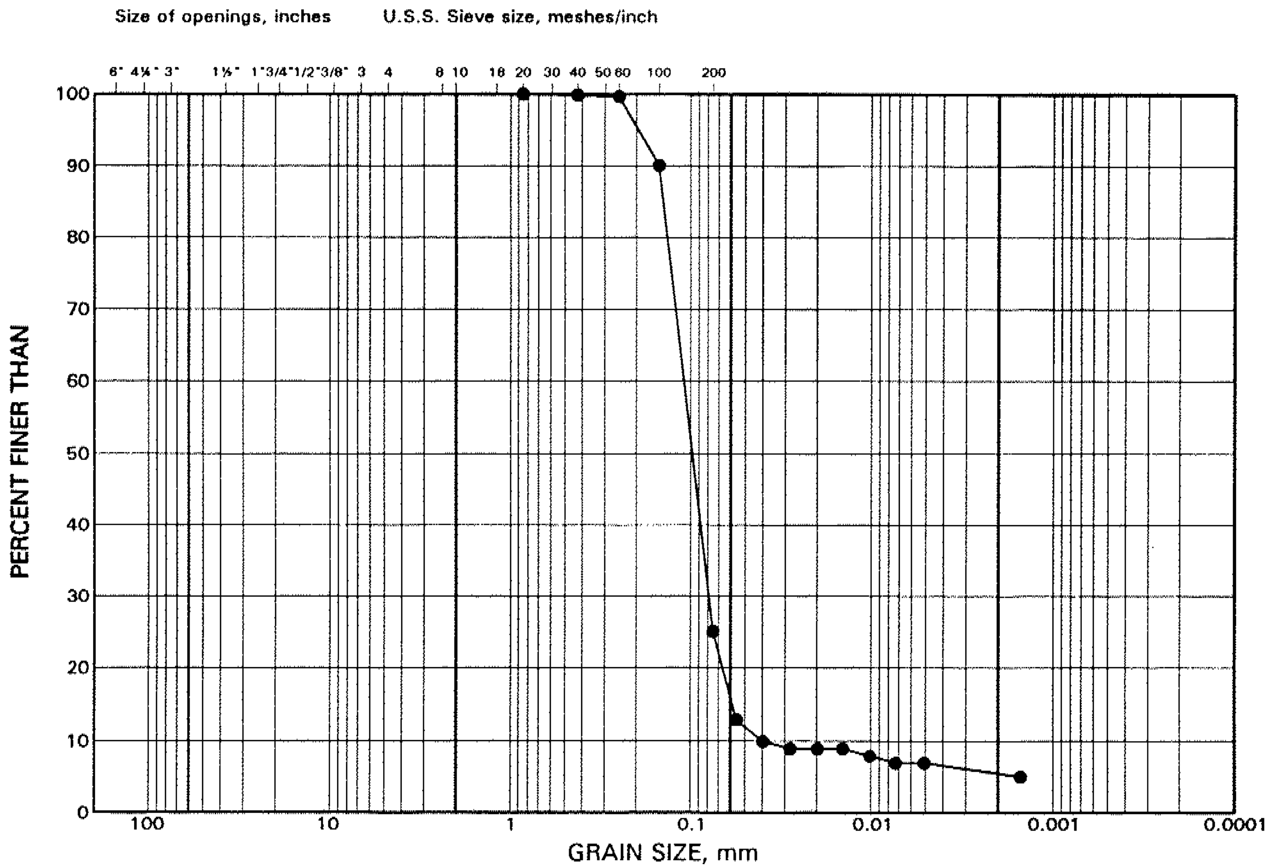


LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	95-4	8B	180.4
■	95-4	12	175.0
◆	95-5	3A	180.3
○	95-PH4	3	190.2

GRAIN SIZE DISTRIBUTION
Sand

FIGURE 7



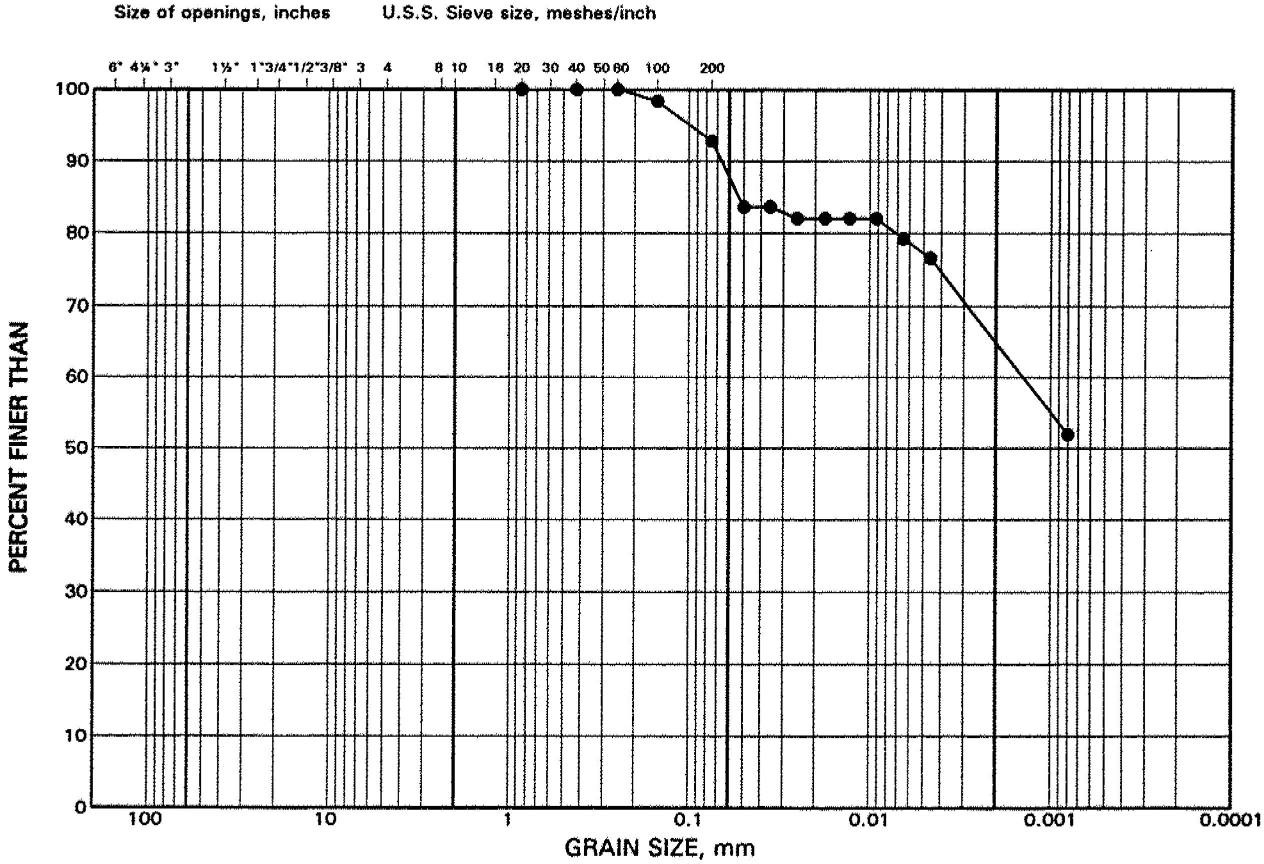
COBBLE	COARSE	MEDIUM	FINE	COARSE	MEDIUM	FINE	SILT SIZE	CLAY SIZE
SIZE	GRAVEL SIZE			SAND SIZE			FINE GRAINED	

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	95-5	6B	177.8

GRAIN SIZE DISTRIBUTION
Clay

FIGURE 8



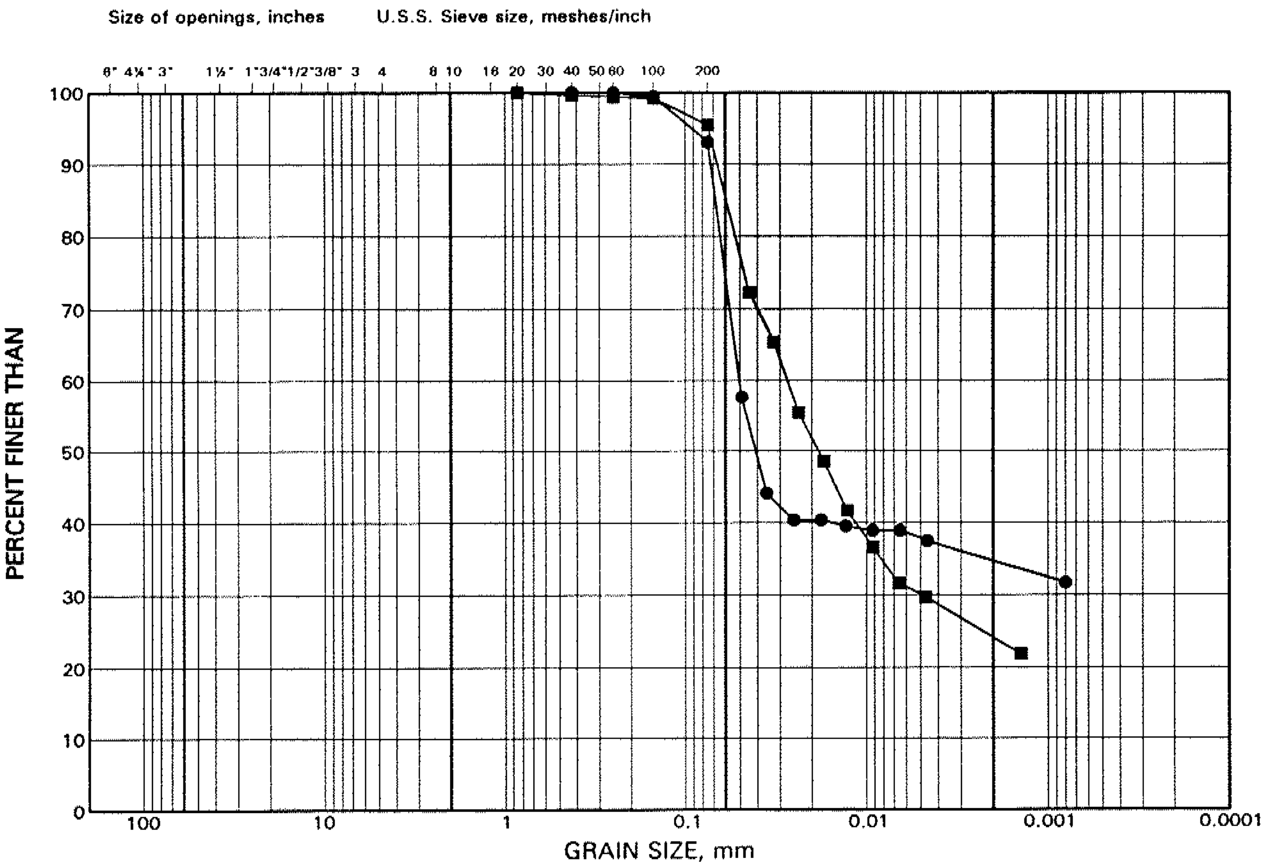
COBBLE	COARSE	MEDIUM	FINE	COARSE	MEDIUM	FINE	SILT SIZE	CLAY SIZE
SIZE	GRAVEL SIZE			SAND SIZE			FINE GRAINED	

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	95-4	18	165.9

GRAIN SIZE DISTRIBUTION
Interlayered Silt and Silty Clay

FIGURE 5



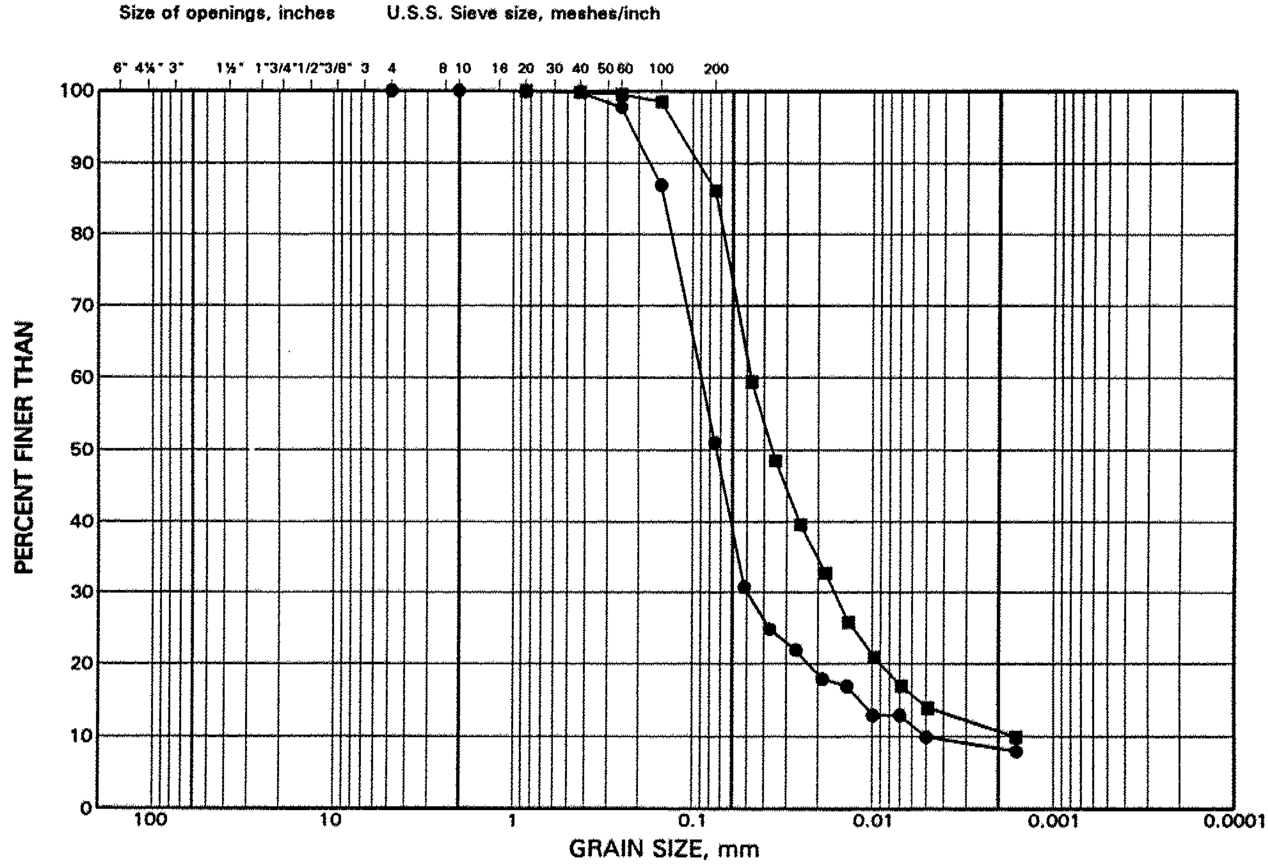
COBBLE	COARSE	MEDIUM	FINE	COARSE	MEDIUM	FINE	SILT SIZE	CLAY SIZE
SIZE	GRAVEL SIZE			SAND SIZE			FINE GRAINED	

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	95-PH3	3B	191.7
■	95-6	6	184.1

GRAIN SIZE DISTRIBUTION
Interlayered Silty Sand and Sandy Silt

FIGURE 9



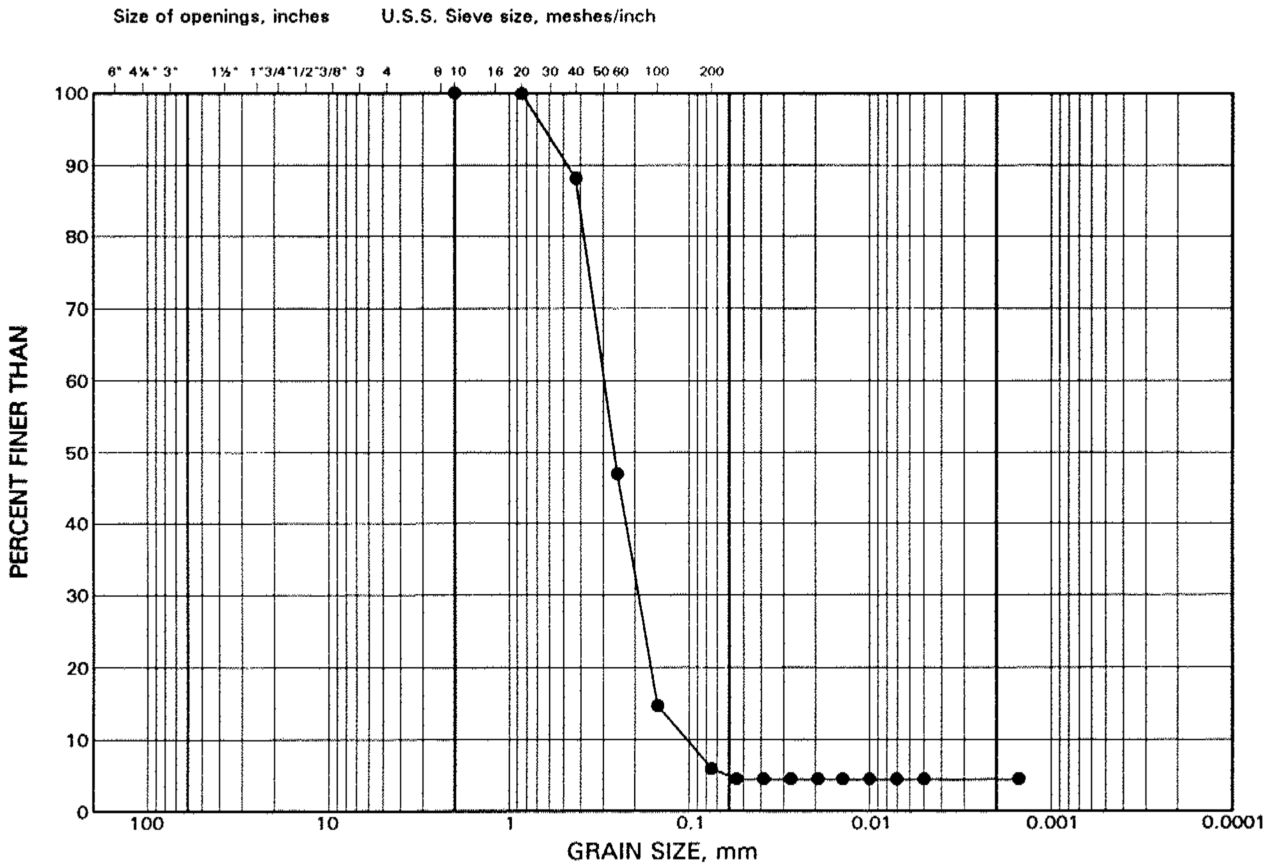
COBBLE	COARSE	MEDIUM	FINE	COARSE	MEDIUM	FINE	SILT SIZE	CLAY SIZE
SIZE	GRAVEL SIZE			SAND SIZE			FINE GRAINED	

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	95-7	7	189.1
■	95-8	9	187.0

GRAIN SIZE DISTRIBUTION
Sand

FIGURE 10



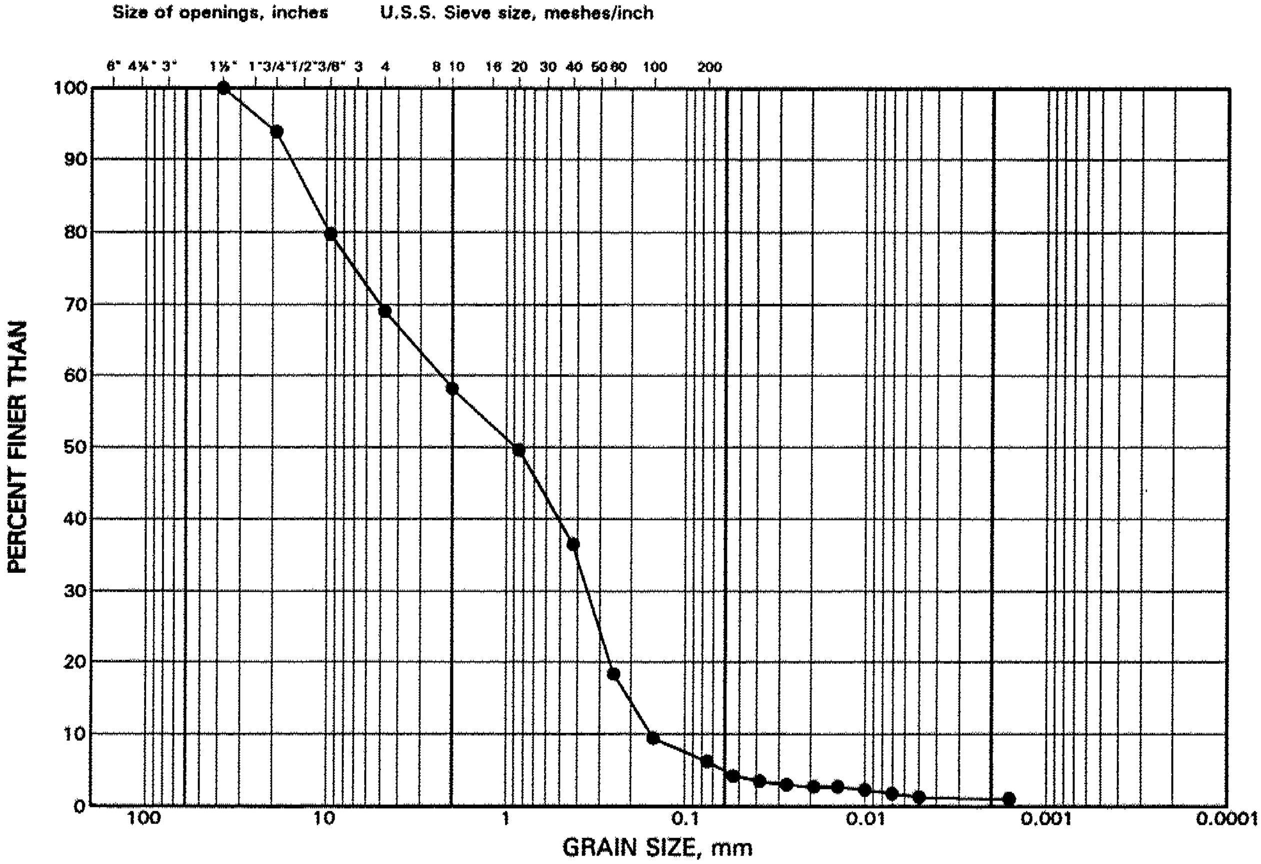
COBBLE	COARSE	MEDIUM	FINE	COARSE	MEDIUM	FINE	SILT SIZE	CLAY SIZE
SIZE	GRAVEL SIZE			SAND SIZE			FINE GRAINED	

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	95-7	18	172.4

GRAIN SIZE DISTRIBUTION
Sand & Gravel

FIGURE 11



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

LEGEND

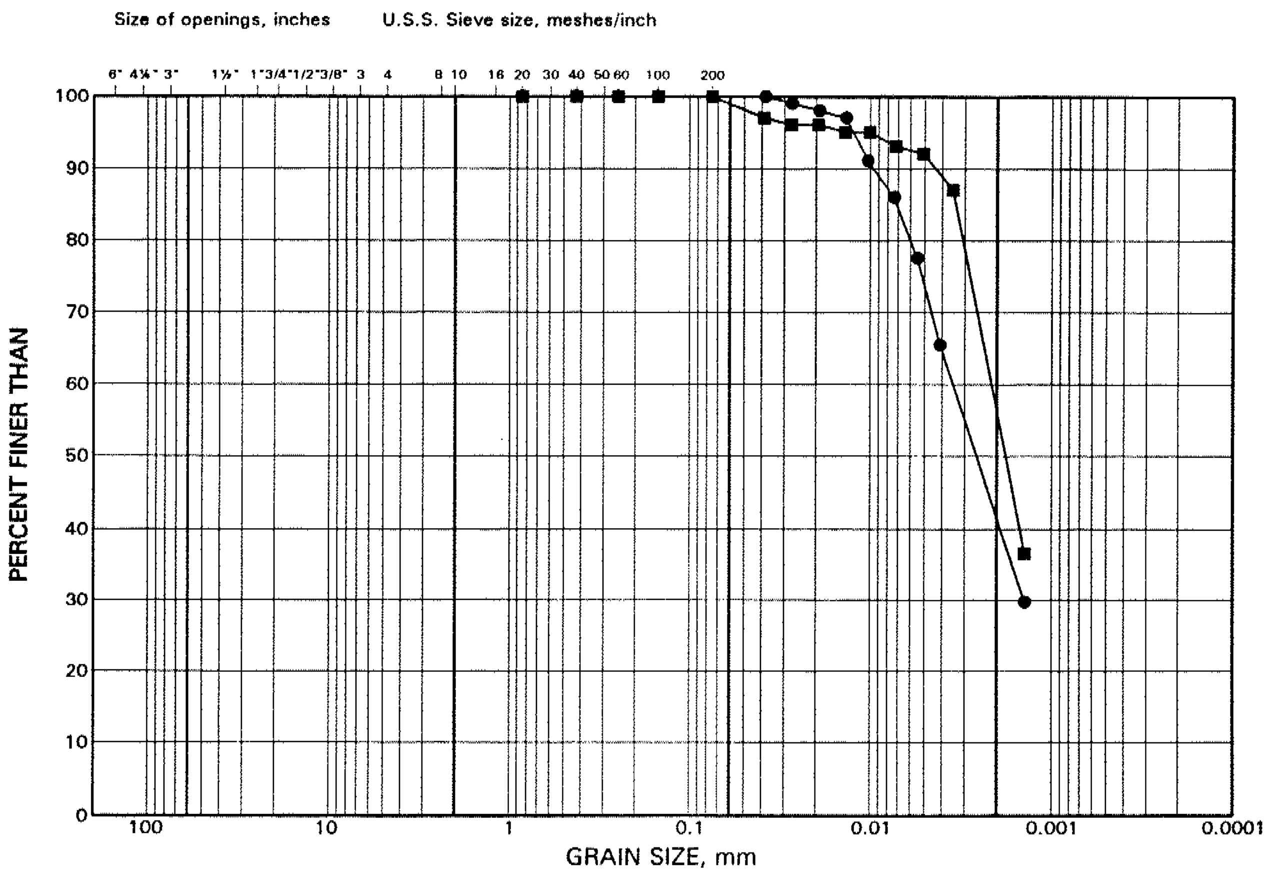
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	95-8	13	180.9

Project 941-1364

Golder Associates

GRAIN SIZE DISTRIBUTION
Silty Clay

FIGURE 12



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	95-9	12	158.4
■	95-9	20	139.5

CONSOLIDATION SUMMARY

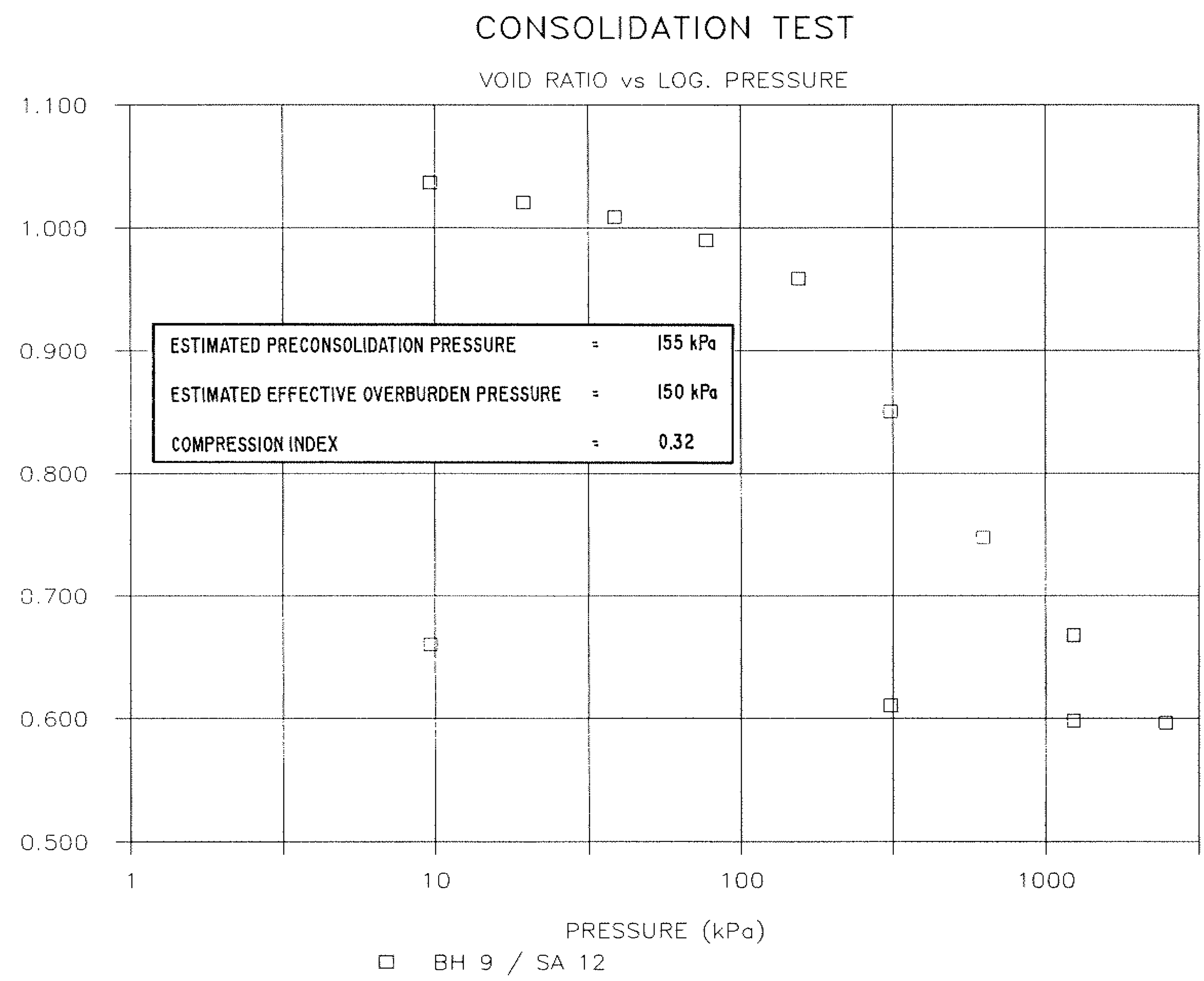
FIGURE 17a

PROJECT 941-1364 SPECIFIC GRAVITY 2.70 assumed DATE STARTED 95-10-18
SAMPLE BH 9 / SA 12 DRY WEIGHT,gm 81.1 DATA COMPLETED 95-10-30
AREA(mm2)3157.06 SOLIDS HT.2HS 9.514

Load kPa	Corr. Height mm	Void Ratio	Average Height mm	t90 sec	t50 sec	cv. t90 cm2/s	cv. t50	k cm/S	mv m2/kN
0.00	19.100	1.008	19.100						
9.68	18.920	0.989	19.010	14		5.47E-02		5.23E-06	9.75E-04
19.36	18.773	0.973	18.847	14		5.38E-02		4.17E-06	7.92E-04
38.73	18.660	0.961	18.717	37		2.01E-02		6.01E-07	3.05E-04
77.46	18.485	0.943	18.573	127		5.76E-03		1.33E-07	2.37E-04
154.92	18.197	0.913	18.341	71		1.00E-02		1.92E-07	1.95E-04
309.83	17.189	0.807	17.693	893		7.43E-04		2.48E-08	3.41E-04
619.66	16.232	0.706	16.710	583		1.02E-03		1.61E-08	1.62E-04
1239.32	15.491	0.628	15.861	197		2.71E-03		1.66E-08	6.26E-05
2478.65	14.829	0.559	15.160	20		2.44E-02		6.68E-08	2.80E-05
1239.32	14.846	0.560	14.838						7.35E-07
309.83	14.964	0.573	14.905						6.61E-06
9.68	15.421	0.621	15.192						7.98E-05

Notes:
k calculated using Cv based on t90 values.

Water Content % ,initial	36.0	Liquid Limit %	45.0
Water Content % , final	25.8	Plastic Limit %	23.3
		Plastic Index %	21.7
Original Volume,cc	60.299	Liquidity Index	0.585
Volume of Solids,cc	30.04		
Volume of Voids,cc	30.26	Unit Weight,kN/m3	17.93
Degree of Saturation %	96.47	Dry Unit Weight,kN/m3	13.18



Golder Associates

CONSOLIDATION TEST
VOID RATIO VS. LOG PRESSURE

FIGURE 17b

CONSOLIDATION SUMMARY

FIGURE 18a

PROJECT 941-1364
SAMPLE BH 9 / SA 20
AREA(mm2)3157.06

SPECIFIC GRAVITY 2.70 assumed
DRY WEIGHT,gm 75.05
SOLIDS HT.2HS 8.804

DATE STARTED 95-10-26
DATA COMPLETED 95-11-06

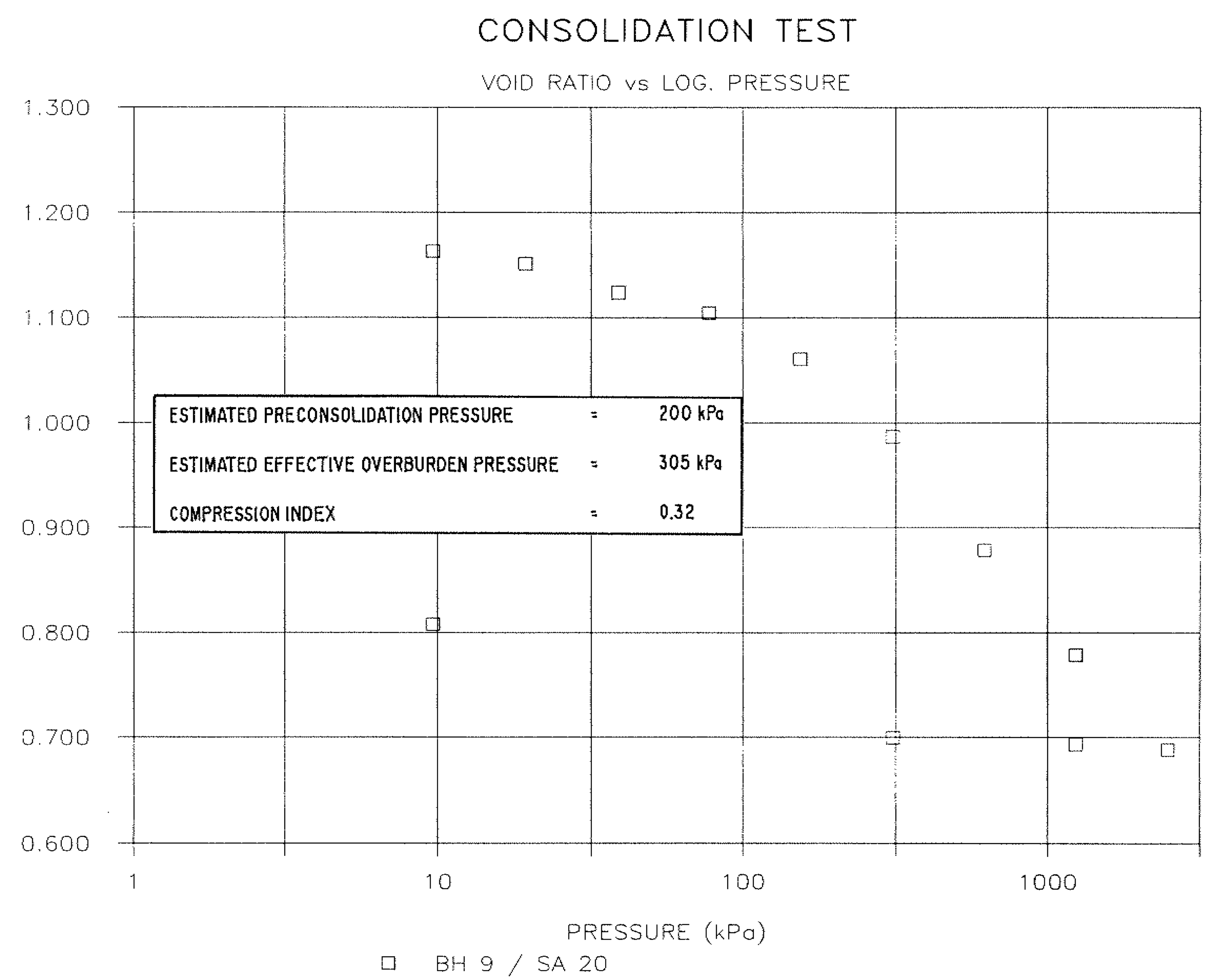
Load kPa	Corr. Height mm	Void Ratio	Average Height mm	t90 sec	t50 sec	cv. t90 cm2/s	cv. t50	k cm/S	mv m2/kN
0.00	19.100	1.169	19.100						
9.68	19.045	1.163	19.073	13		5.93E-02		1.73E-06	2.97E-04
19.36	18.943	1.152	18.994	20		3.82E-02		2.07E-06	5.52E-04
38.73	18.701	1.124	18.822	30		2.50E-02		1.60E-06	6.54E-04
77.46	18.532	1.105	18.617	90		8.16E-03		1.83E-07	2.29E-04
154.92	18.147	1.061	18.339	90		7.92E-03		2.02E-07	2.60E-04
309.83	17.495	0.987	17.821	60		1.12E-02		2.42E-07	2.20E-04
619.66	16.537	0.878	17.016	120		5.12E-03		8.11E-08	1.62E-04
1239.32	15.658	0.778	16.098	50		1.10E-02		7.99E-08	7.43E-05
2478.65	14.870	0.689	15.264	50		9.88E-03		3.22E-08	3.33E-05
1239.32	14.915	0.694	14.893						1.92E-06
309.83	14.968	0.700	14.942						2.96E-06
9.68	15.914	0.807	15.441						1.65E-04

Notes:
k calculated using Cv based on t90 values.

Water Content % ,initial	41.5
Water Content % , final	35.5
Original Volume,cc	60.299
Volume of Solids,cc	27.80
Volume of Voids,cc	32.50
Degree of Saturation %	95.82

Liquid Limit %	47.4
Plastic Limit %	25.1
Plastic Index %	22.3
Liquidity Index	0.735
Unit Weight,kN/m3	17.30
Dry Unit Weight,kN/m3	12.23

Golder Associates



Golder Associates

CONSOLIDATION TEST
VOID RATIO VS. LOG PRESSURE

FIGURE 18b

APPENDIX B

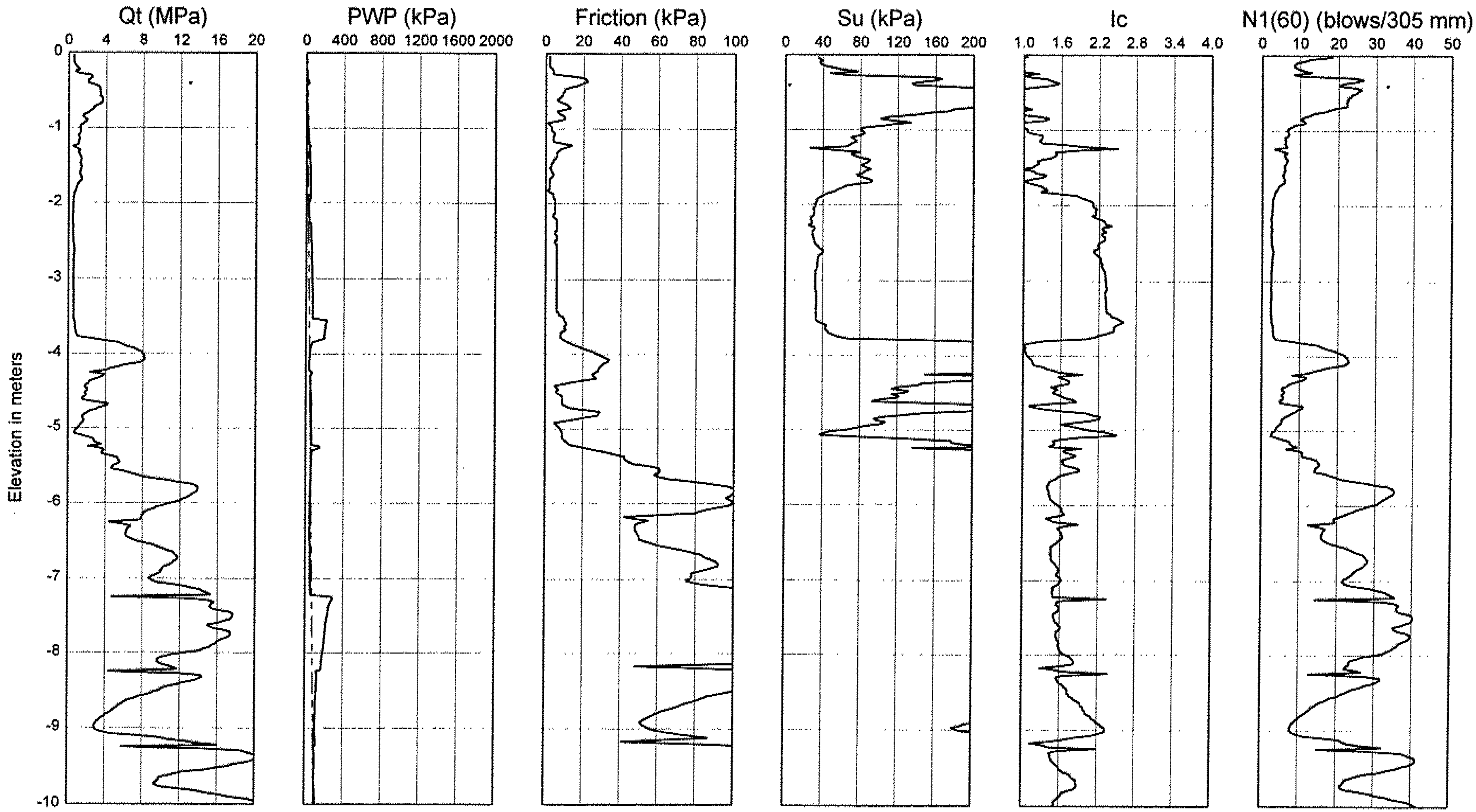
Cone Penetration Test Sheets
CPT-1 to CPT-17

Cone Penetration Test - 1

Test Date : 4-2-97
Location : SEE FIGURE

Operator : Golder Associates

Ground Surf. Elev. : 0.00
Water Table Depth : 0.00



Qt normalized for
unequal end area effects

Su = (Qt - SigmaV) / Nk
Nk = 15
Gamma = 17 kN/m³

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

After Jefferies and Davies (1993)

PROJECT NO. 971-1103 DATE: DRAWN BY: AJW

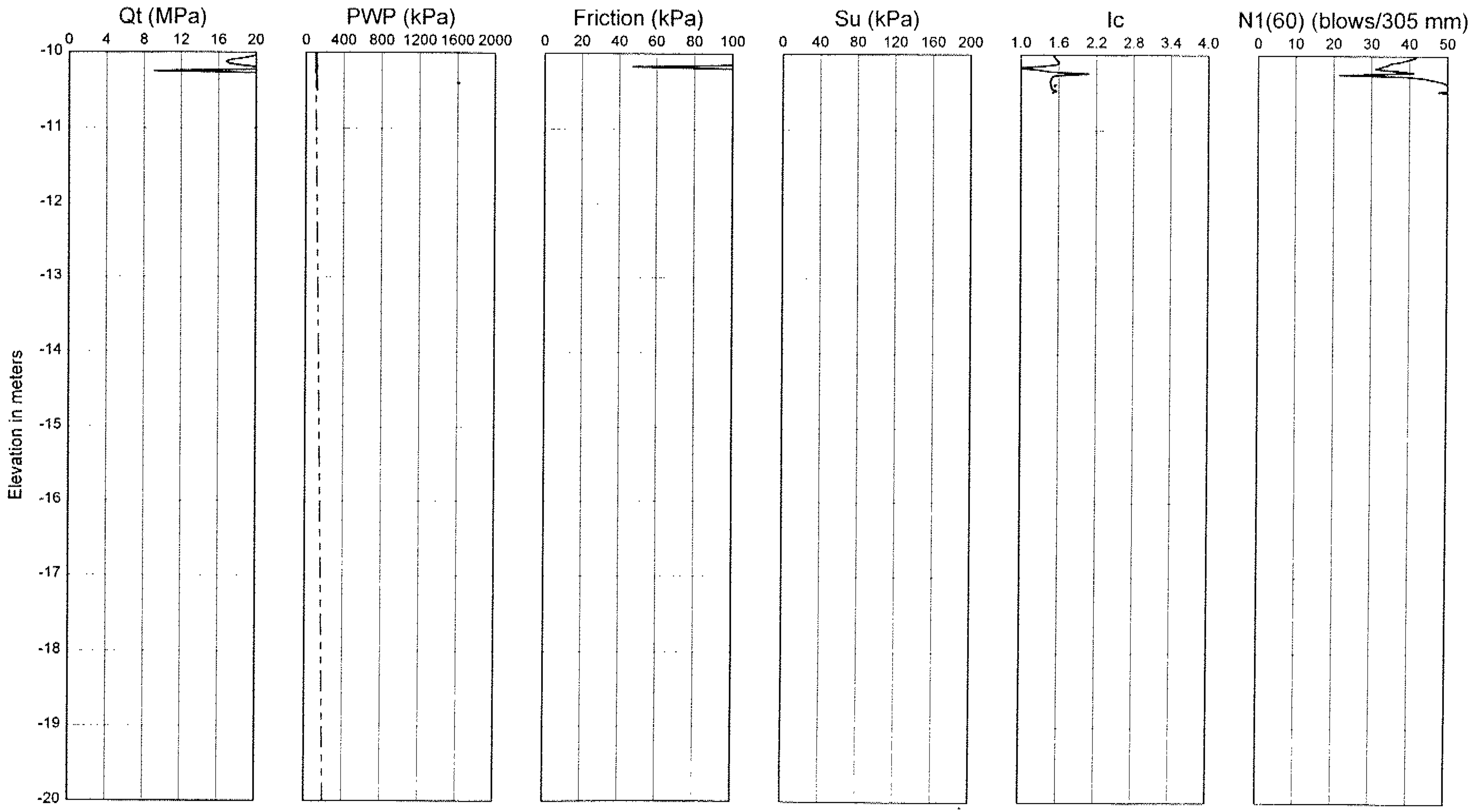
HWY17

Cone Penetration Test - 1

Test Date : 4-2-97
Location : SEE FIGURE

Operator : Golder Associates

Ground Surf. Elev. : 0.00
Water Table Depth : 0.00



Qt normalized for unequal end area effects

Su = (Qt - SigmaV) / Nk
Nk = 15
Gamma = 17 kN/m³

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

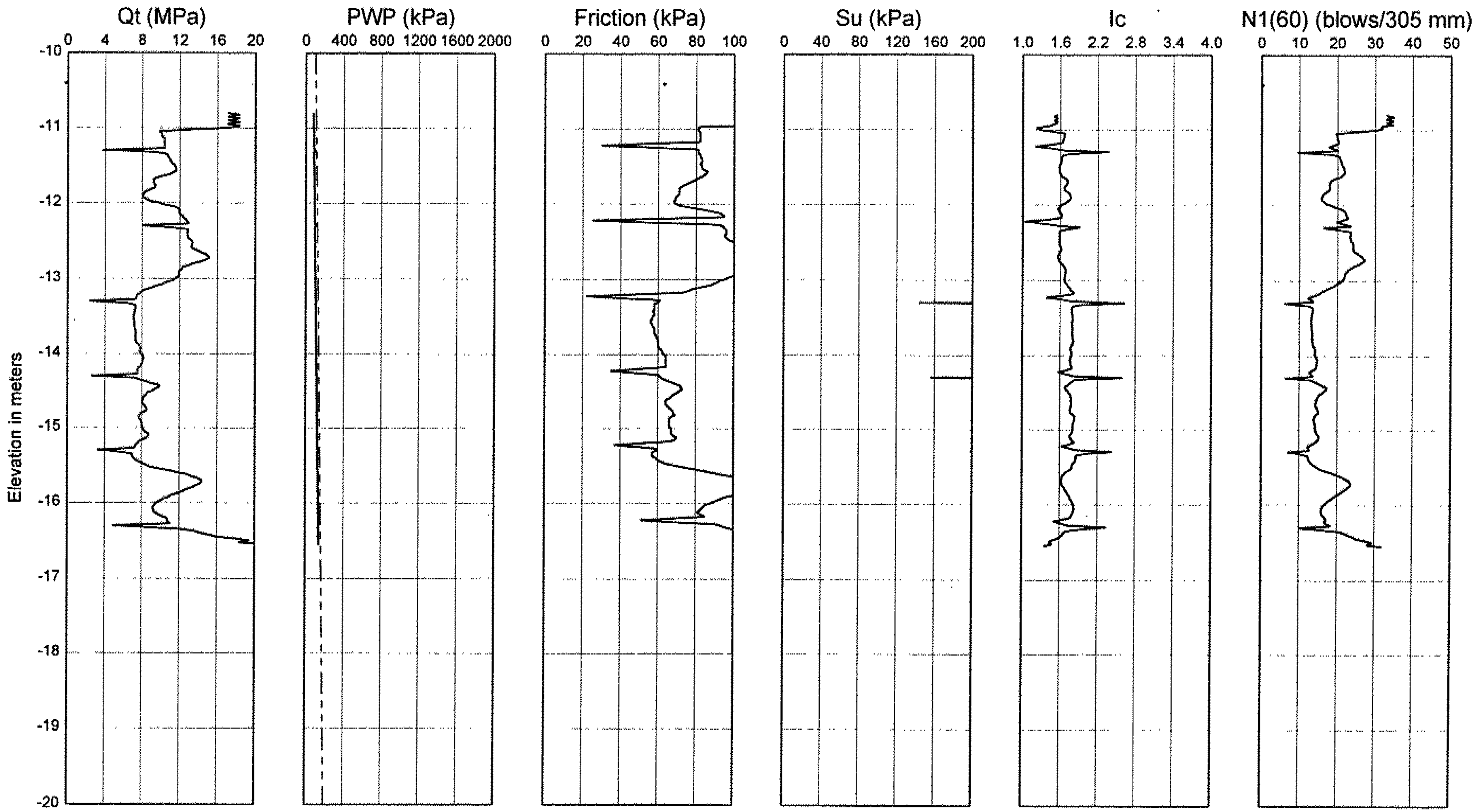
After Jefferies and Davies (1993)

Cone Penetration Test - 1

Test Date : 4-2-97
Location : SEE FIGURE

Operator : Golder Associates

Ground Surf. Elev. : 0.00
Water Table Depth : 0.00



Qt normalized for
unequal end area effects

Su = (Qt - SigmaV) / Nk
Nk = 15
Gamma = 17 kN/m³

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

After Jefferies and Davies (1993)

PROJECT NO. 971-1103 DATE: DRAWN BY: AJW

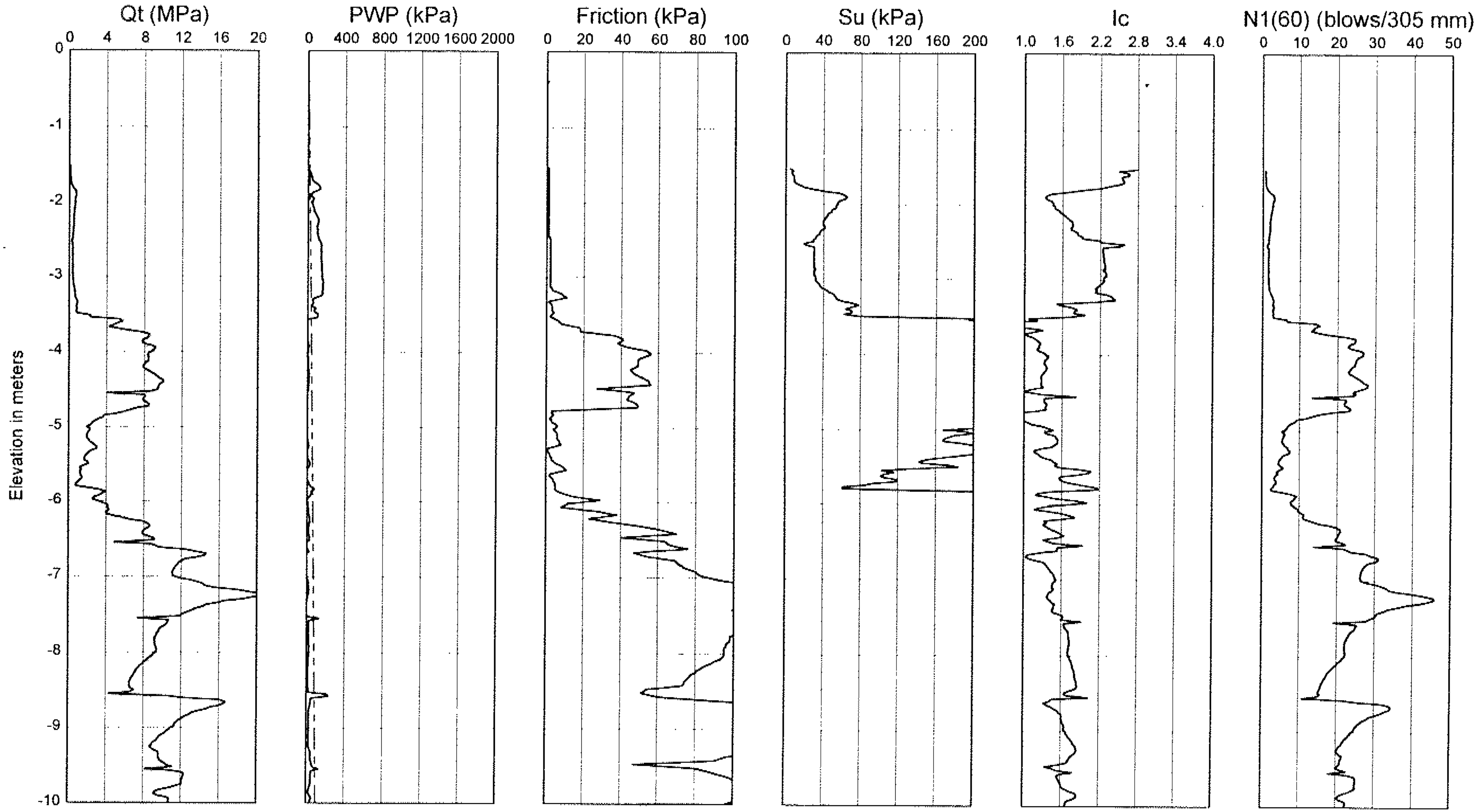
HWY17

Cone Penetration Test - 2

Test Date : 4-2-97
Location : SEE FIGURE

Operator : Golder Associates

Ground Surf. Elev. : 0.00
Water Table Depth : 0.00



Qt normalized for
unequal end area effects

Su = (Qt - SigmaV) / Nk
Nk = 11
Gamma = 17 kN/m³

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

After Jefferies and Davies (1993)

PROJECT NO. 971-1103 DATE: DRAWN BY: AJW

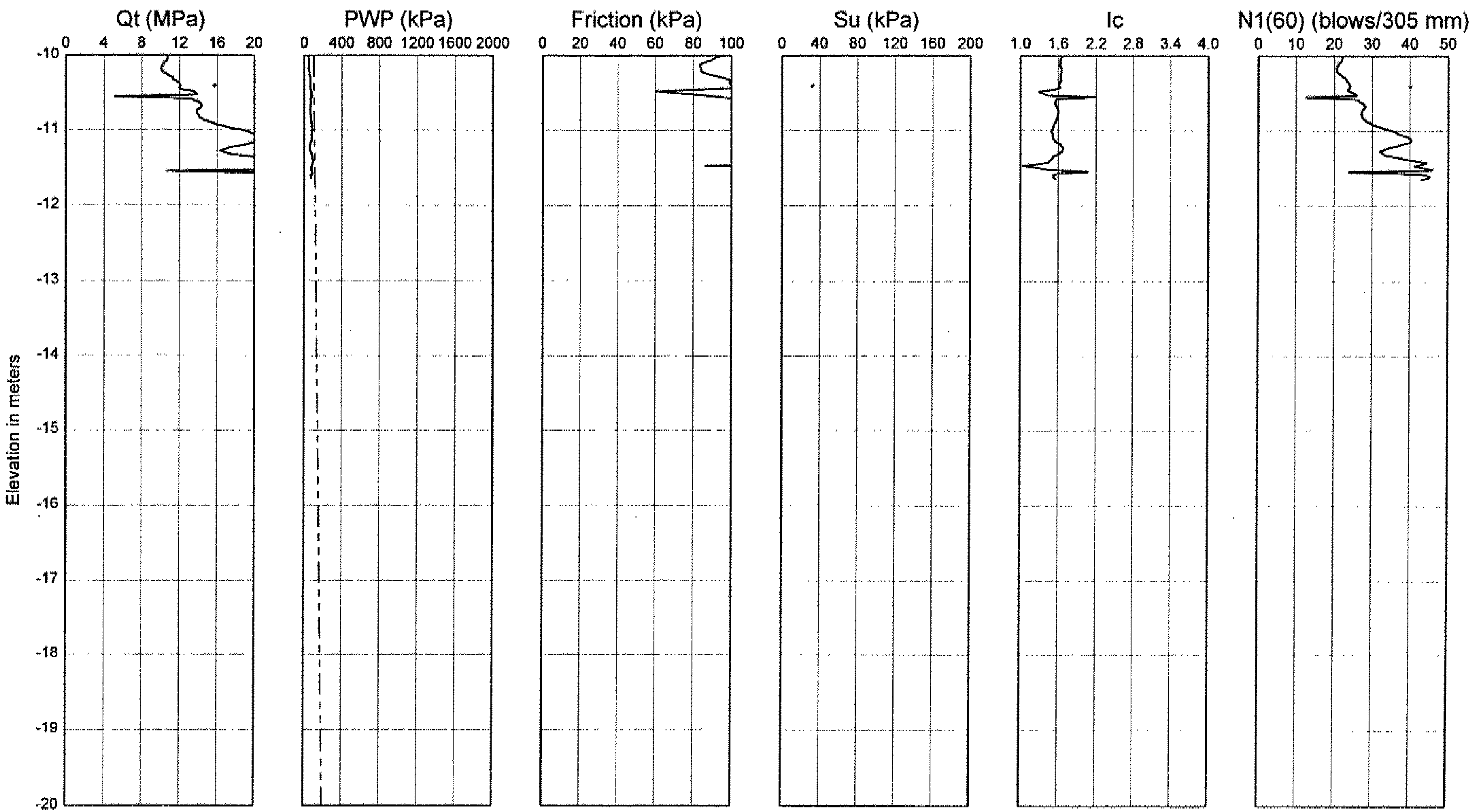
HWY17

Cone Penetration Test - 2

Test Date : 4-2-97
Location : SEE FIGURE

Operator : Golder Associates

Ground Surf. Elev. : 0.00
Water Table Depth : 0.00



Qt normalized for
unequal end area effects

Su = (Qt - SigmaV) / Nk
Nk = 11
Gamma = 17 kN/m³

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

After Jefferies and Davies (1993)

PROJECT NO. 971-1103 DATE: DRAWN BY: AJW

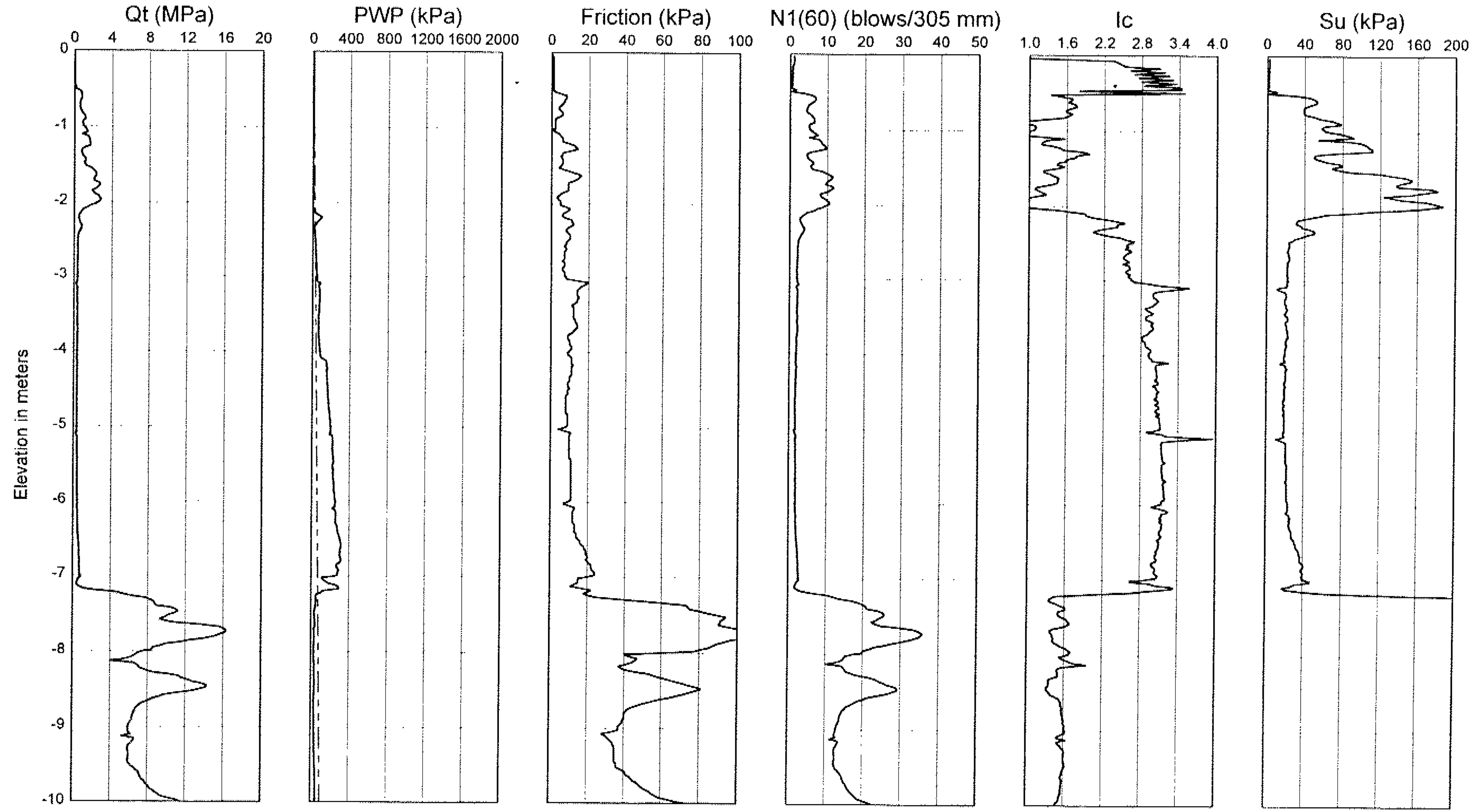
HWY17

Cone Penetration Test - 3

Test Date : 4-2-97
Location : SEE FIGURE

Operator : Golder Associates

Ground Surf. Elev. : 0.00
Water Table Depth : 0.00



Qt normalized for
unequal end area effects

After Jefferies and Davies (1993)

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

Su = (Qt - SigmaV) / Nk
Nk = 15
Gamma = 17 kN/m³

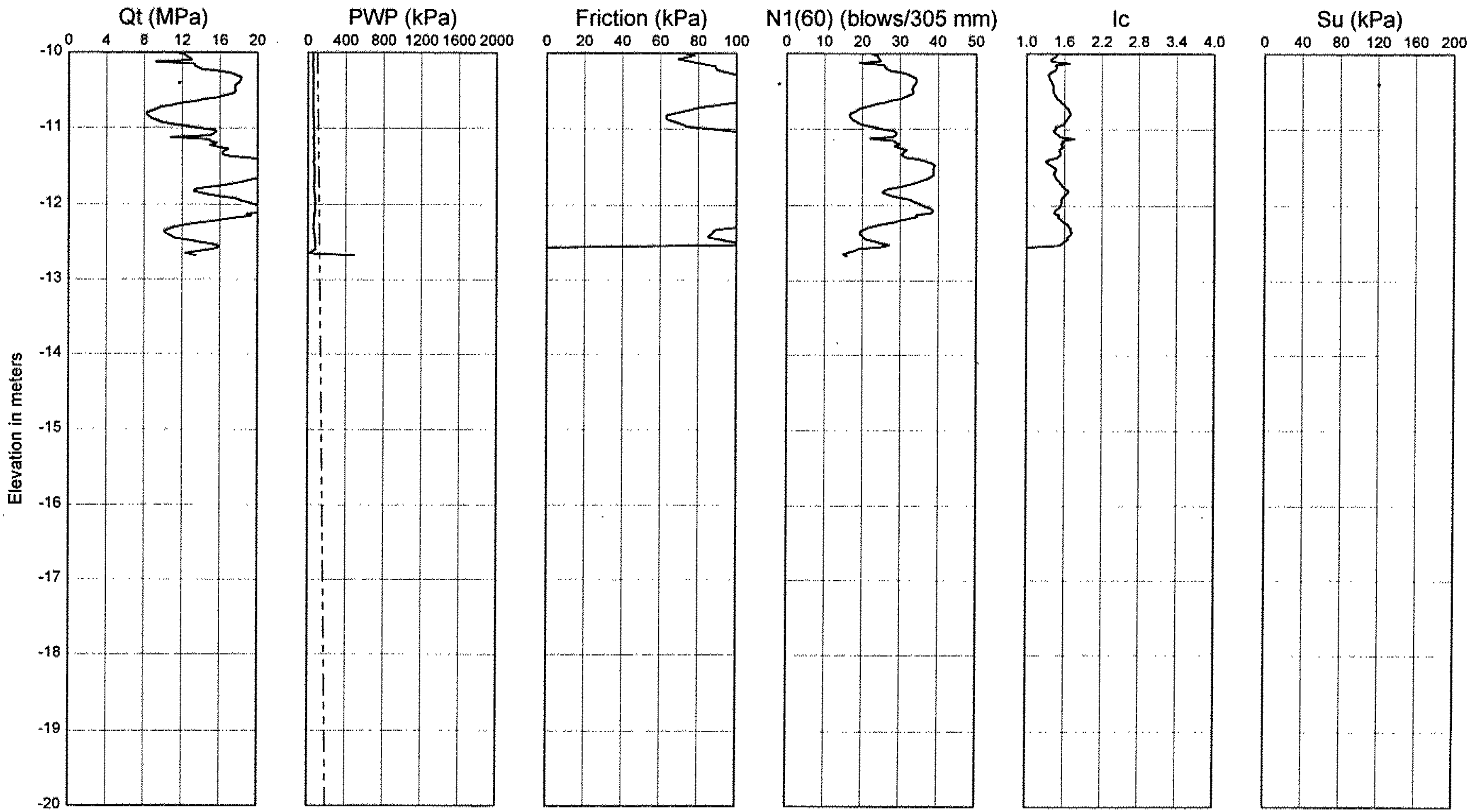
PROJECT NO. 971-1103 DATE. DRAWN BY: AJW

Cone Penetration Test - 3

Test Date : 4-2-97
Location : SEE FIGURE

Operator : Golder Associates

Ground Surf. Elev. : 0.00
Water Table Depth : 0.00



Qt normalized for
unequal end area effects

After Jefferies and Davies (1993)

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

Su = (Qt - SigmaV) / Nk
Nk = 15
Gamma = 17 kN/m³

PROJECT NO. 971-1103 DATE: DRAWN BY: AJW

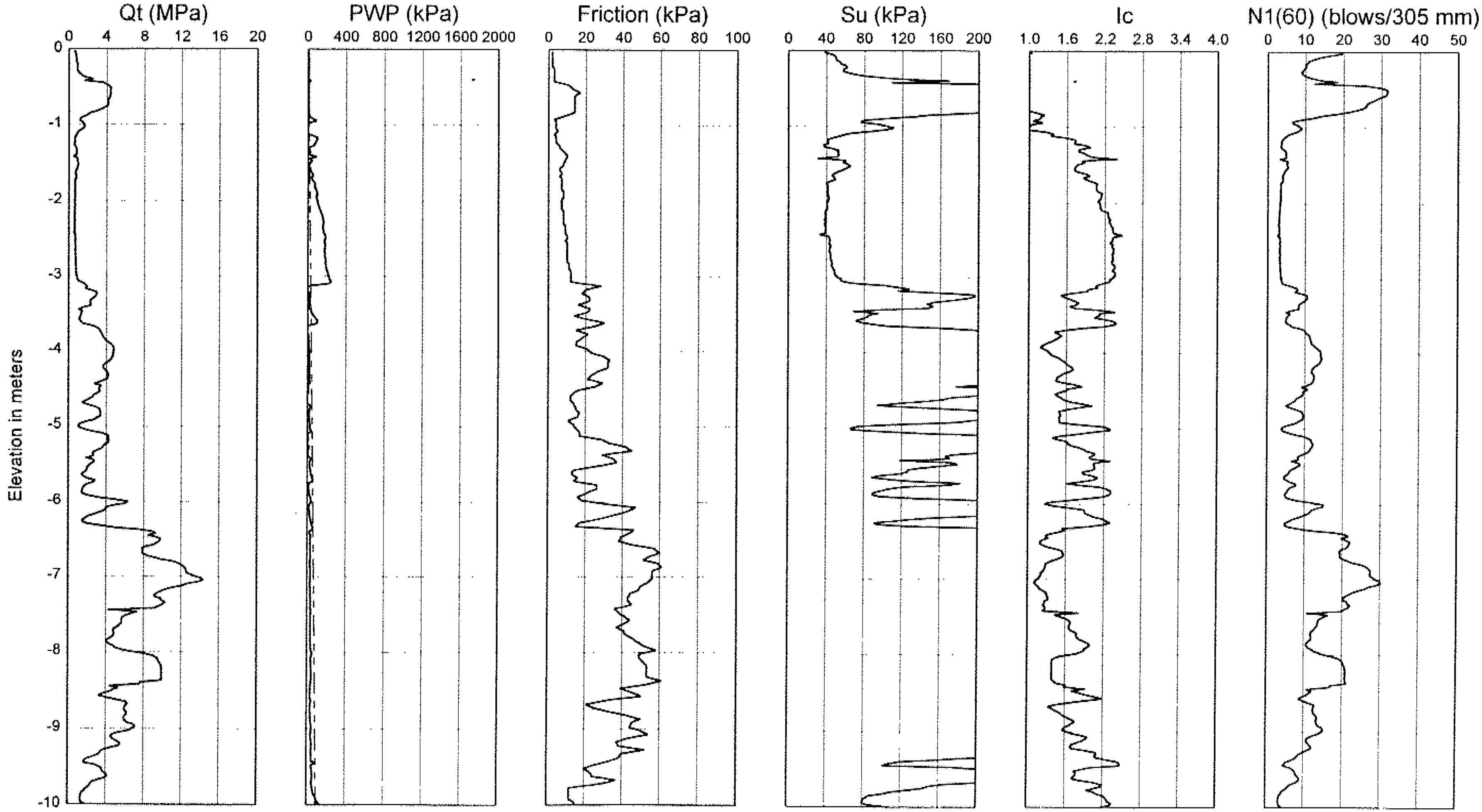
HWY17

Cone Penetration Test - 4

Test Date : 5-2-97
Location : SEE FIGURE

Operator : Golder Associates

Ground Surf. Elev. : 0.00
Water Table Depth : 0.00



Qt normalized for
unequal end area effects

Su = (Qt - SigmaV) / Nk
Nk = 15
Gamma = 17 kN/m³

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

After Jefferies and Davies (1993)

PROJECT NO. 971-1103 DATE: DRAWN BY: AJW

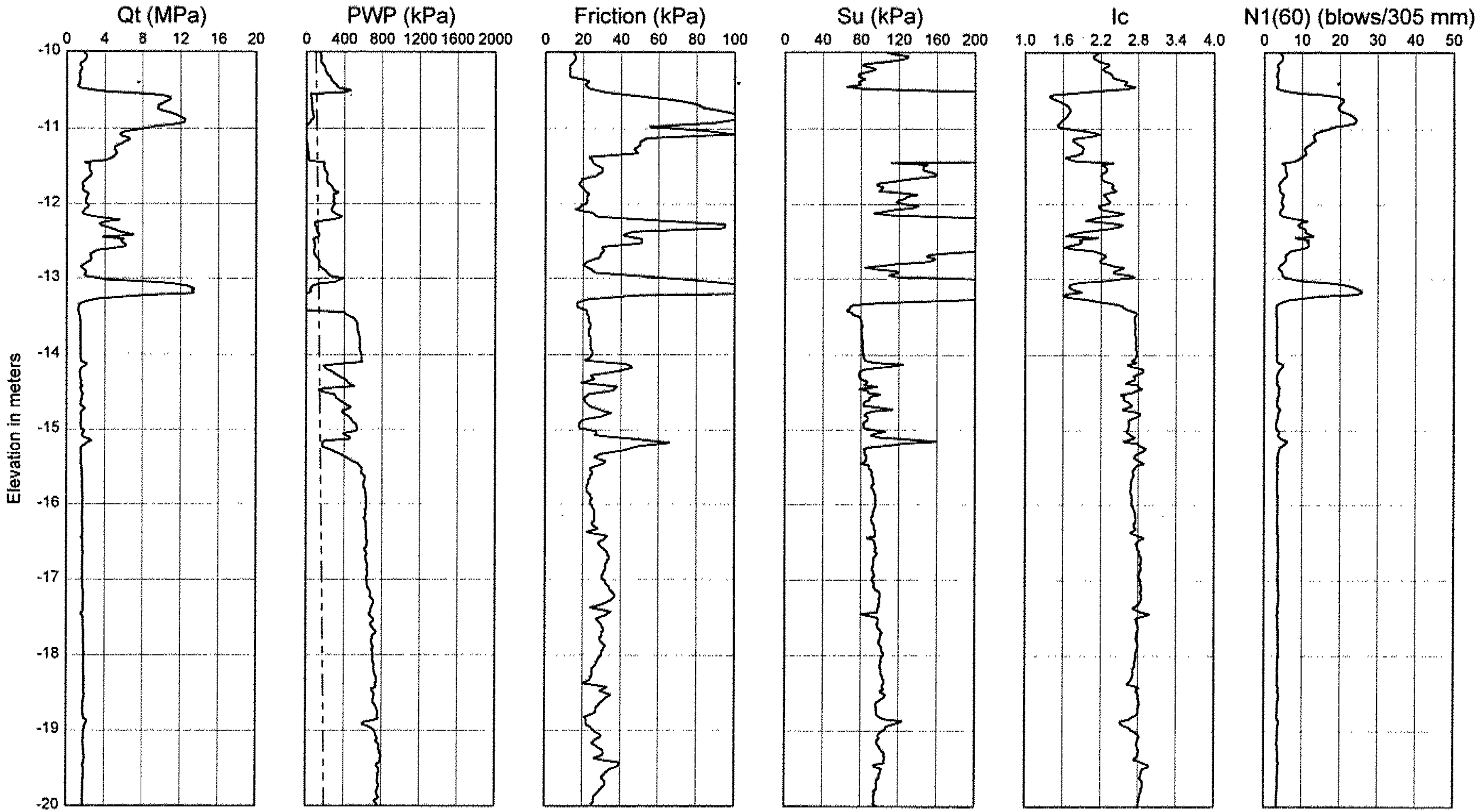
HWY17

Cone Penetration Test - 4

Test Date : 5-2-97
Location : SEE FIGURE

Operator : Golder Associates

Ground Surf. Elev. : 0.00
Water Table Depth : 0.00



Qt normalized for
unequal end area effects

Su = (Qt - SigmaV) / Nk
Nk = 15
Gamma = 17 kN/m³

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

After Jefferies and Davies (1993)

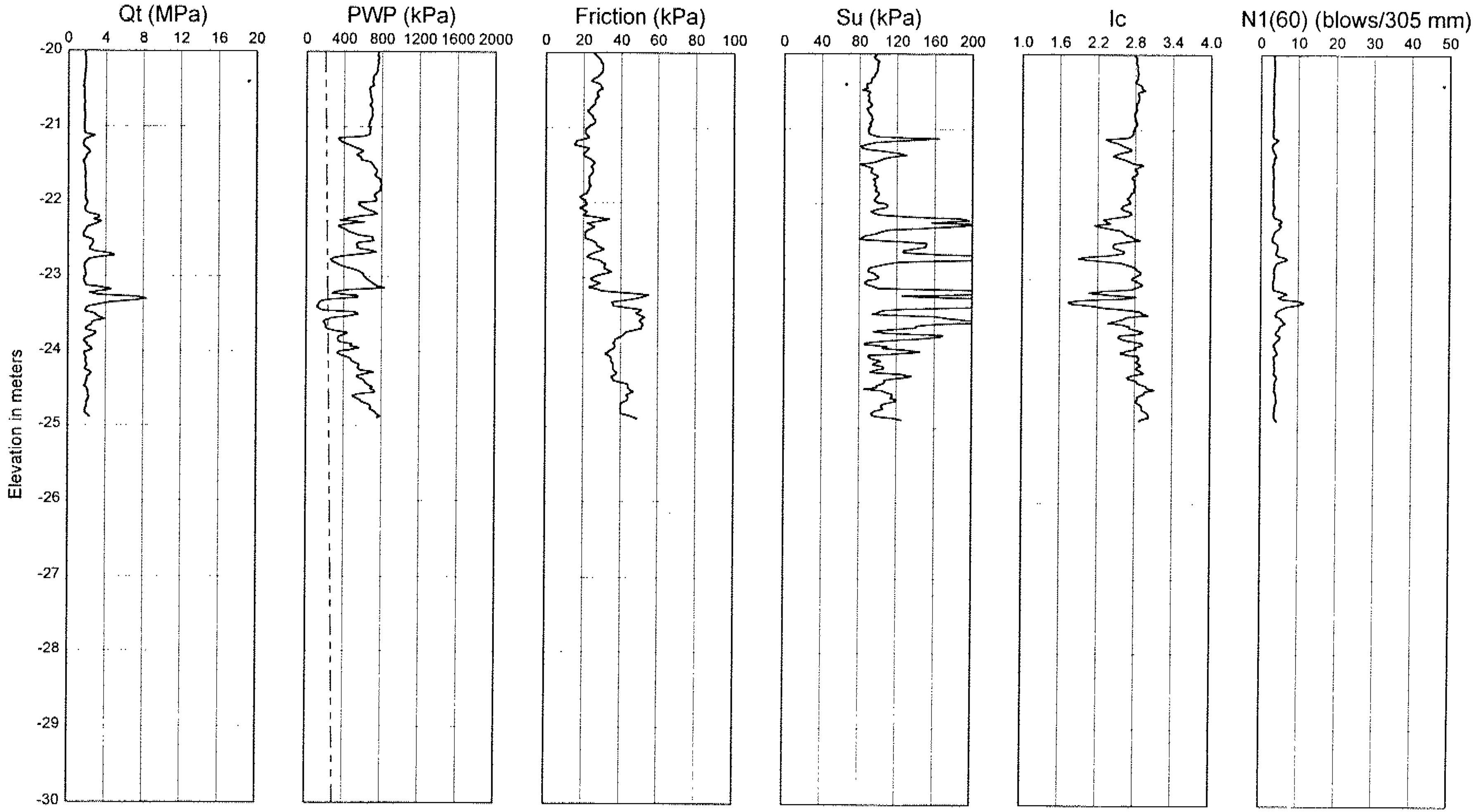
PROJECT NO. 971-1103 DATE: DRAWN BY: AJW

Cone Penetration Test - 4

Test Date : 5-2-97
Location : SEE FIGURE

Operator : Golder Associates

Ground Surf. Elev. : 0.00
Water Table Depth : 0.00



Qt normalized for
unequal end area effects

Su = (Qt - SigmaV) / Nk
Nk = 15
Gamma = 17 kN/m³

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

After Jefferies and Davies (1993)

PROJECT NO. 971-1103 DATE: DRAWN BY: AJW

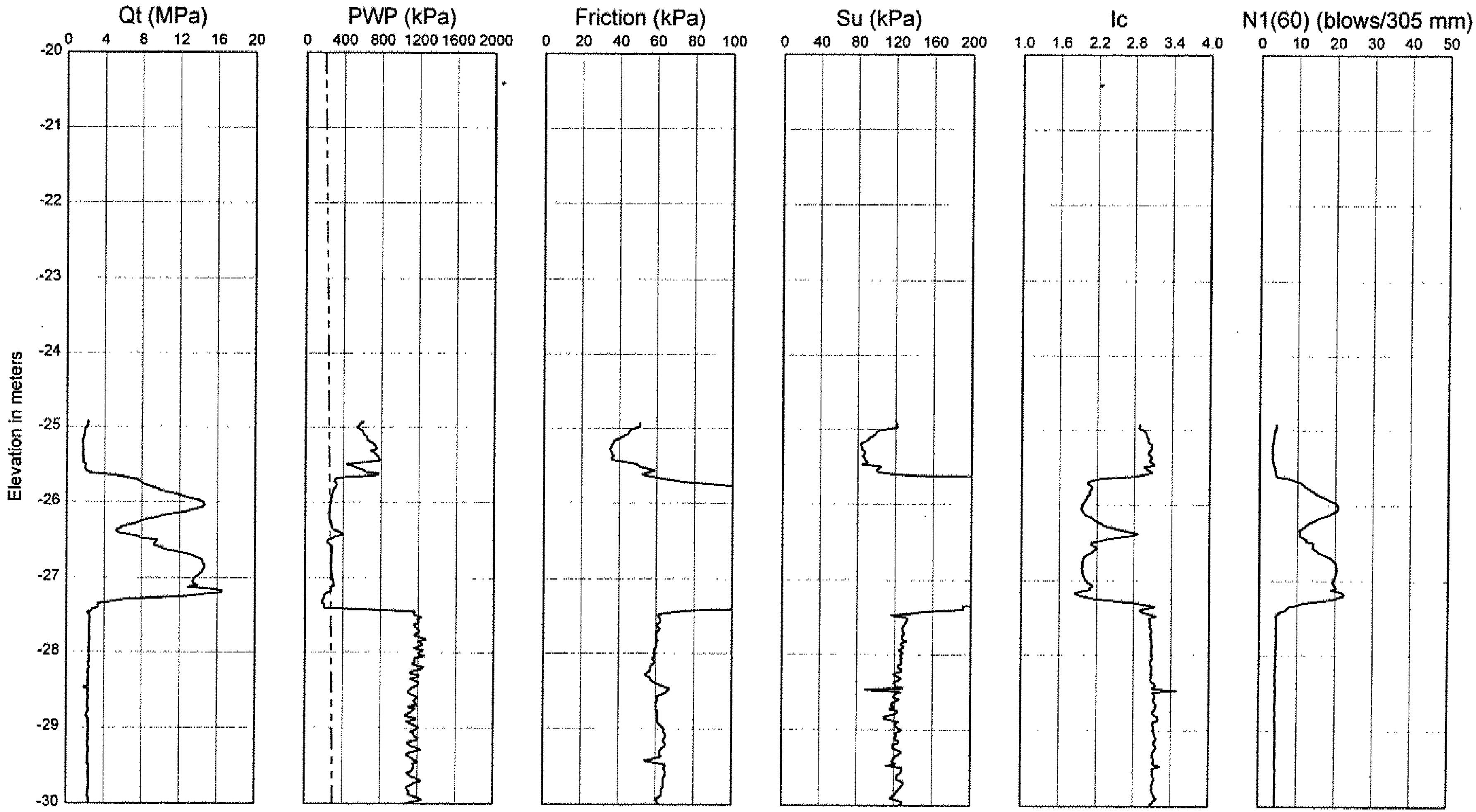
HWY17

Cone Penetration Test - 4

Test Date : 5-2-97
Location : SEE FIGURE

Operator : Golder Associates

Ground Surf. Elev. : 0.00
Water Table Depth : 0.00



Qt normalized for
unequal end area effects

Su = (Qt - SigmaV) / Nk
Nk = 15
Gamma = 17 kN/m³

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

After Jefferies and Davies (1993)

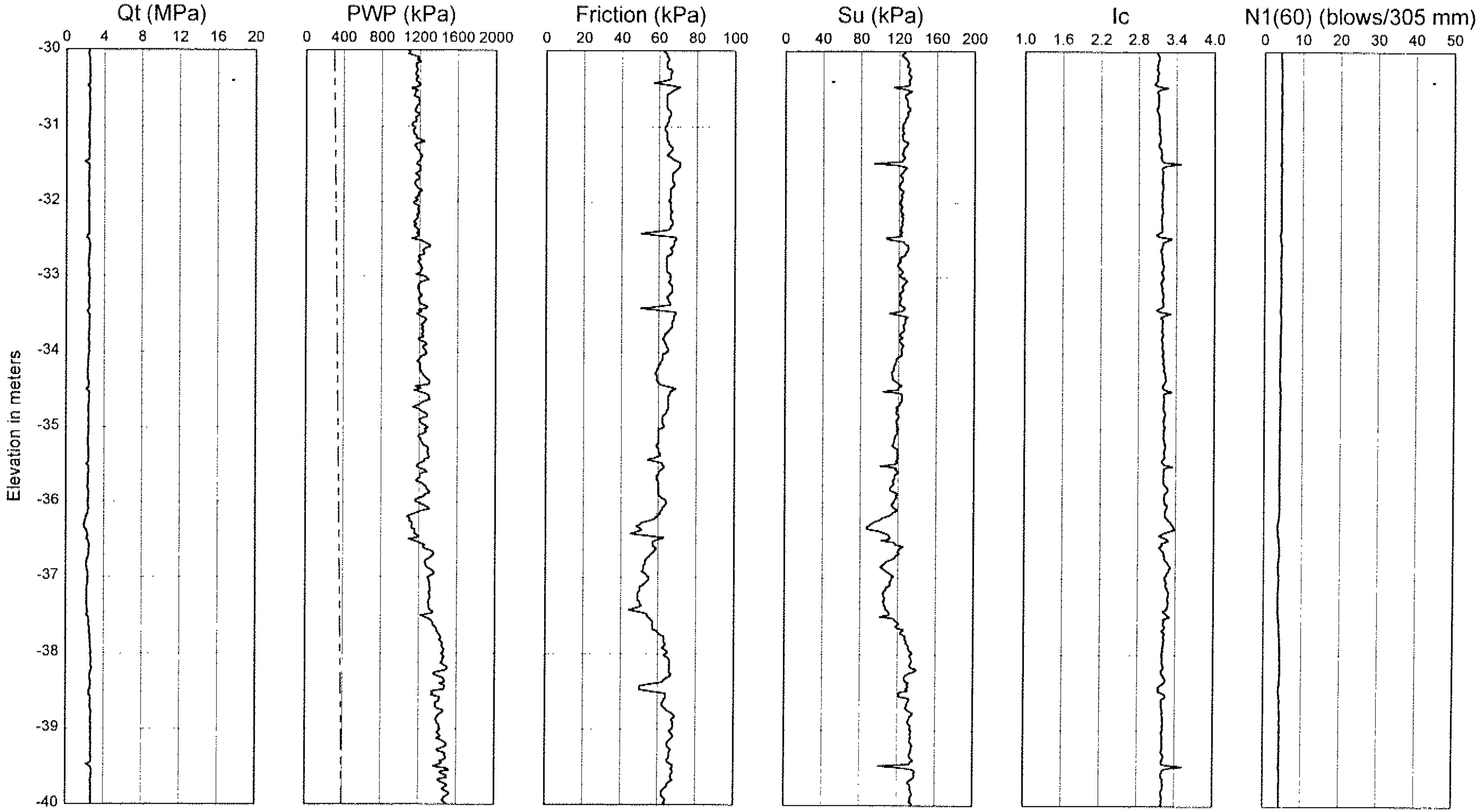
PROJECT NO. 971-1103 DATE: DRAWN BY: AJW

Cone Penetration Test - 4

Test Date : 5-2-97
Location : SEE FIGURE

Operator : Golder Associates

Ground Surf. Elev. : 0.00
Water Table Depth : 0.00



Qt normalized for
unequal end area effects

Su = (Qt - SigmaV) / Nk
Nk = 15
Gamma = 17 kN/m³

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

After Jefferies and Davies (1993)

PROJECT NO. 971-1103 DATE: DRAWN BY: AJW

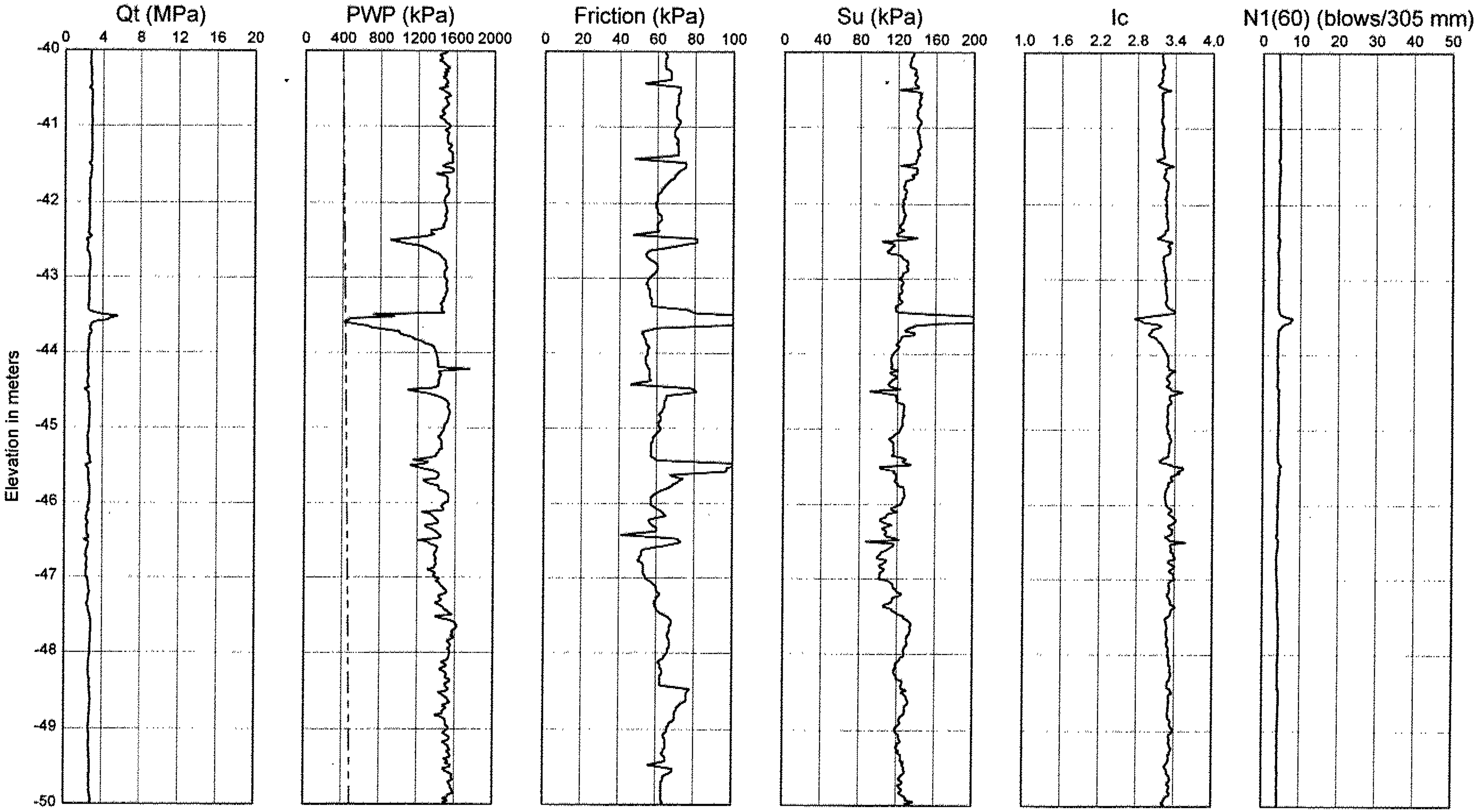
HWY17

Cone Penetration Test - 4

Test Date : 5-2-97
Location : SEE FIGURE

Operator : Golder Associates

Ground Surf. Elev. : 0.00
Water Table Depth : 0.00



Qt normalized for unequal end area effects

Su = (Qt - SigmaV) / Nk
Nk = 15
Gamma = 17 kN/m³

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

After Jefferies and Davies (1993)

PROJECT NO. 971-1103 DATE: DRAWN BY: AJW

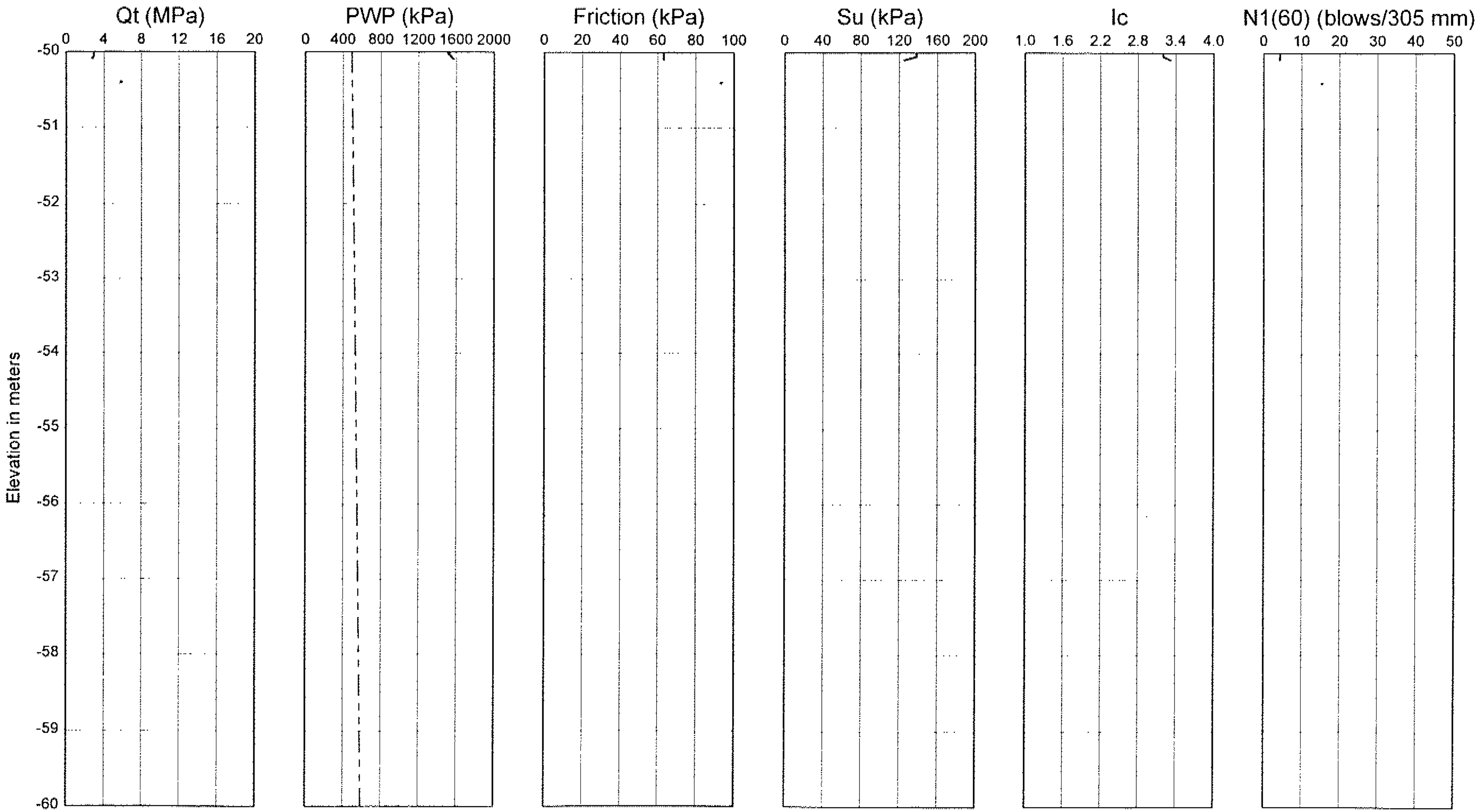
HWY17

Cone Penetration Test - 4

Test Date : 5-2-97
Location : SEE FIGURE

Operator : Golder Associates

Ground Surf. Elev. : 0.00
Water Table Depth : 0.00



Qt normalized for
unequal end area effects

Su = (Qt - SigmaV) / Nk
Nk = 15
Gamma 17 kN/m³

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

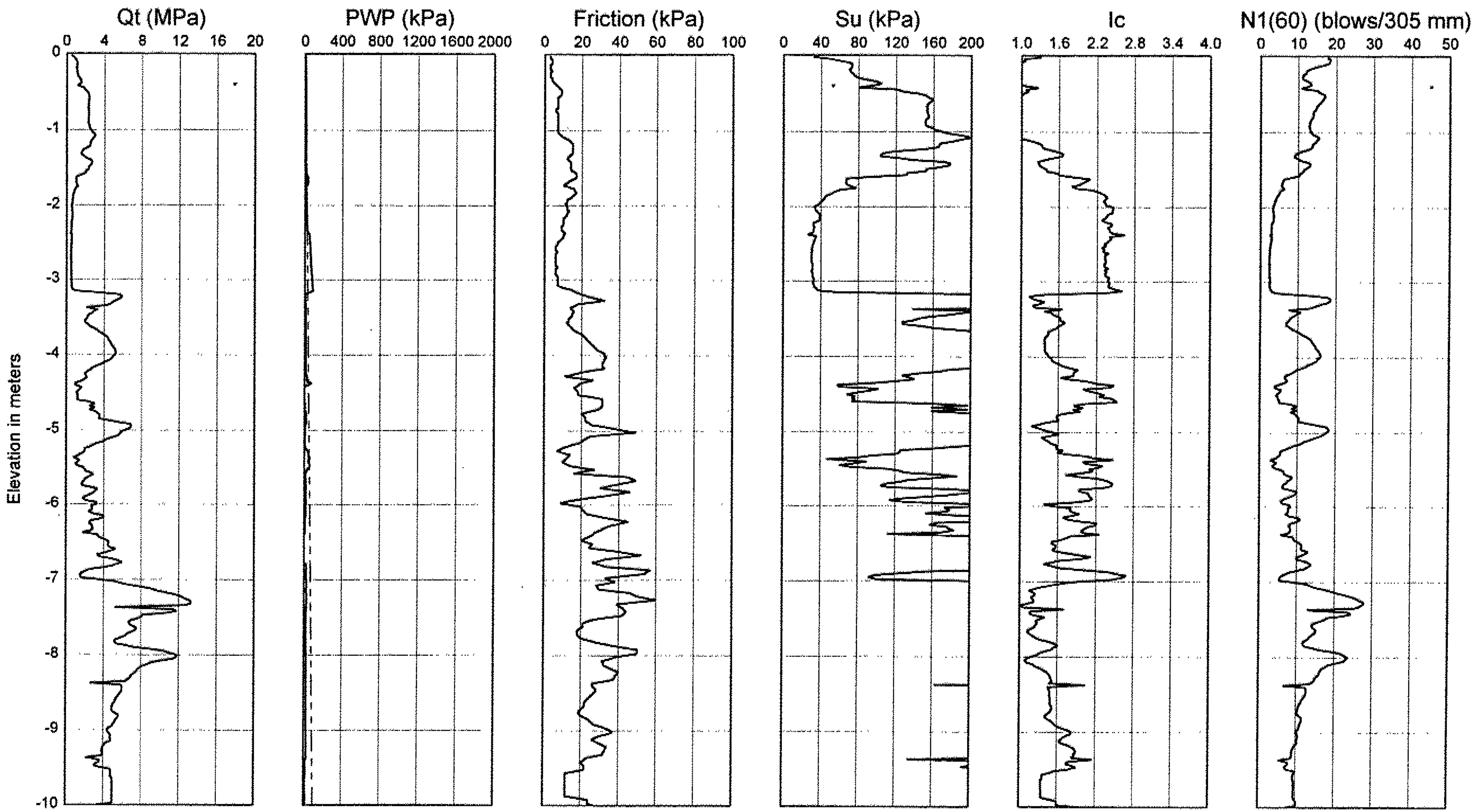
After Jefferies and Davies (1993)

Cone Penetration Test - 5

Test Date : 0-/0 /0-
Location : SEE FIGURE

Operator : Golder Associates

Ground Surf. Elev. : 0.00
Water Table Depth : 0.00



Qt normalized for unequal end area effects

Su = (Qt - SigmaV) / Nk
Nk = 15
Gamma = 17 kN/m³

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

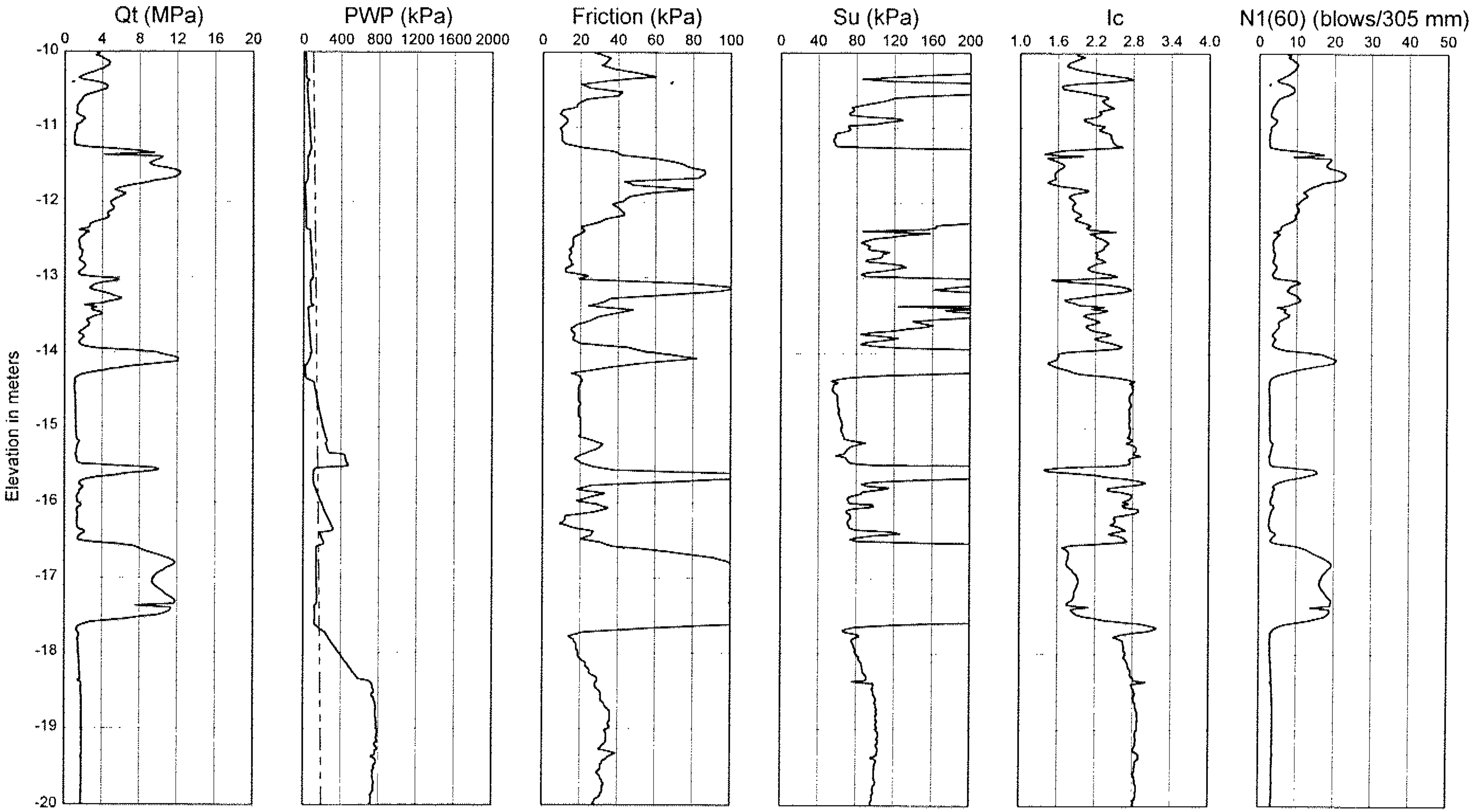
After Jefferies and Davies (1993)

Cone Penetration Test - 5

Test Date : 0-/0 /0-
Location : SEE FIGURE

Operator : Golder Associates

Ground Surf. Elev. : 0.00
Water Table Depth : 0.00



Qt normalized for
unequal end area effects

Su = (Qt - SigmaV) / Nk
Nk = 15
Gamma = 17 kN/m³

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

After Jefferies and Davies (1993)

PROJECT NO. 971-1103 DATE: DRAWN BY: AJW

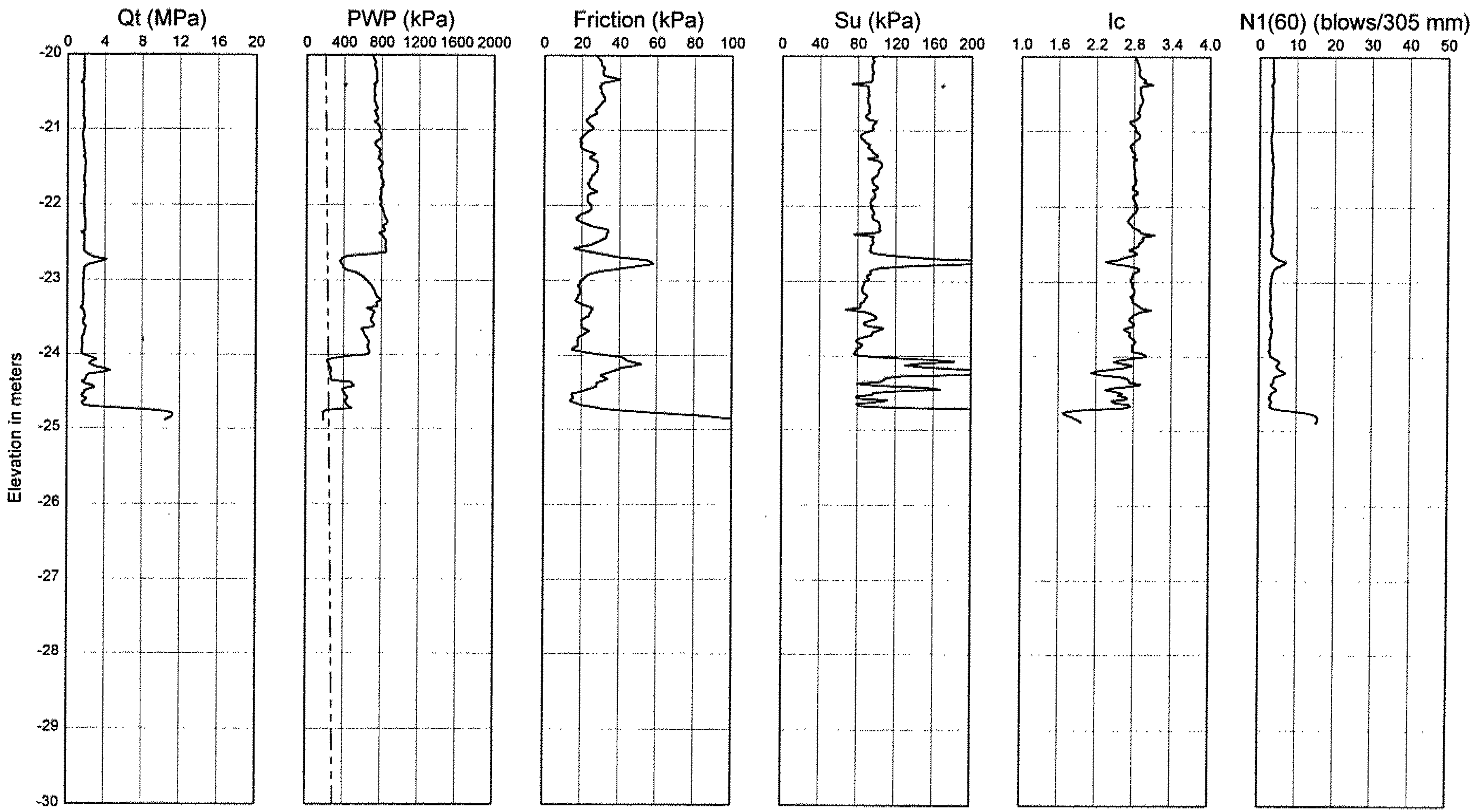
HWY 17

Cone Penetration Test - 5

Test Date : 0-/0 /0-
Location : SEE FIGURE

Operator : Golder Associates

Ground Surf. Elev. : 0.00
Water Table Depth : 0.00



Qt normalized for
unequal end area effects

Su = (Qt - SigmaV) / Nk
Nk = 15
Gamma = 17 kN/m³

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

After Jefferies and Davies (1993)

PROJECT NO. 971-1103 DATE: DRAWN BY: AJW

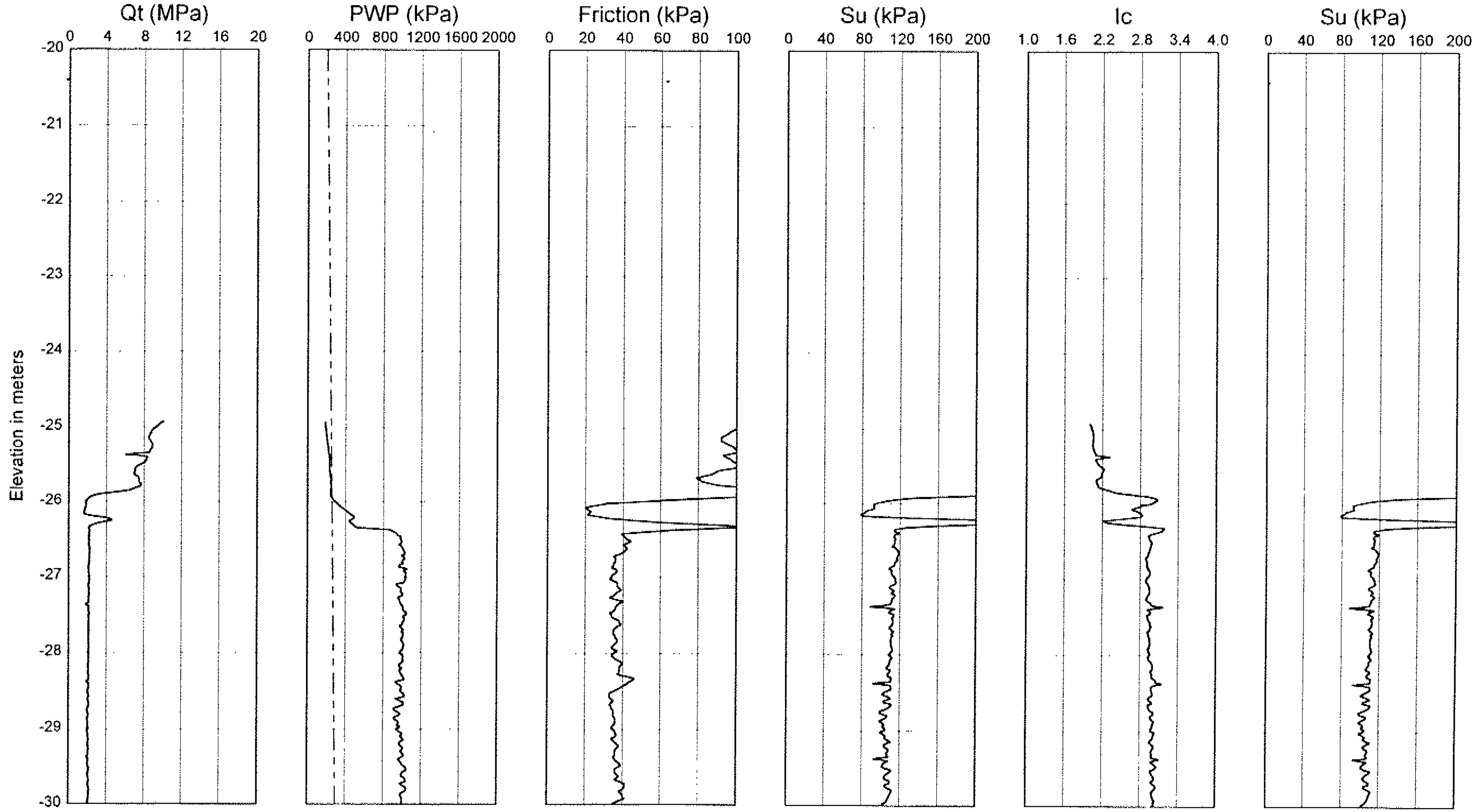
HWY 17

Cone Penetration Test - 5

Test Date : 0-/0 /0-
Location : SEE FIGURE

Operator : Golder Associates

Ground Surf. Elev. : 0.00
Water Table Depth : 0.00



Qt normalized for
unequal end area effects

Su = (Qt - SigmaV) / Nk
Nk = 15
Gamma = 17 kN/m³

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

Su = (Qt - SigmaV) / Nk
Nk 15
Gamma = 17 kN/m³

PROJECT NO. 971-1103 DATE: DRAWN BY. AJW

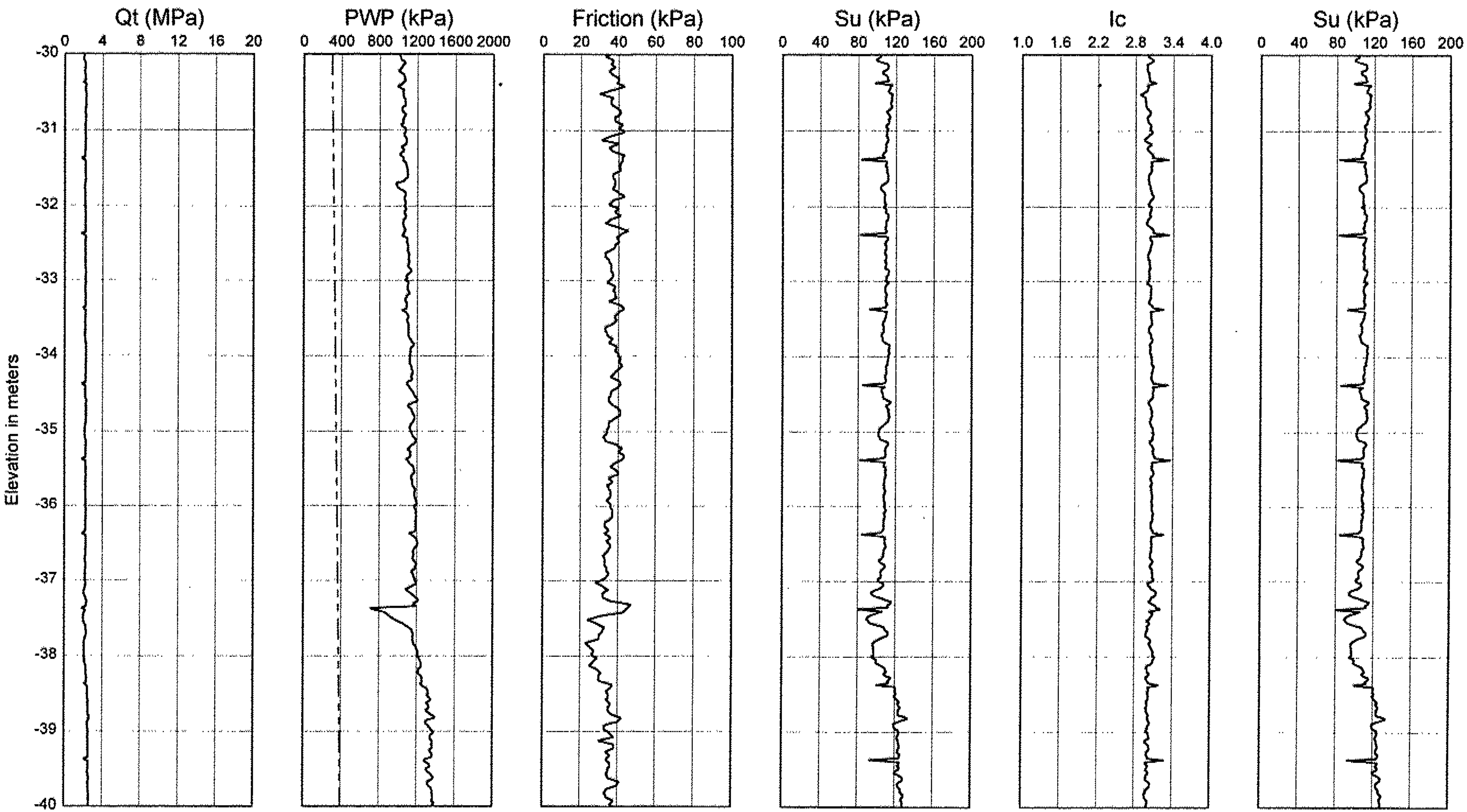
HWY 17

Cone Penetration Test - 5

Test Date : 0-/0 /0-
Location : SEE FIGURE

Operator : Golder Associates

Ground Surf. Elev. : 0.00
Water Table Depth : 0.00



Qt normalized for
unequal end area effects

Su = (Qt - SigmaV) / Nk
Nk = 15
Gamma = 17 kN/m³

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

Su = (Qt - SigmaV) / Nk
Nk = 15
Gamma = 17 kN/m³

PROJECT NO. 971-1103 DATE: DRAWN BY: AJW

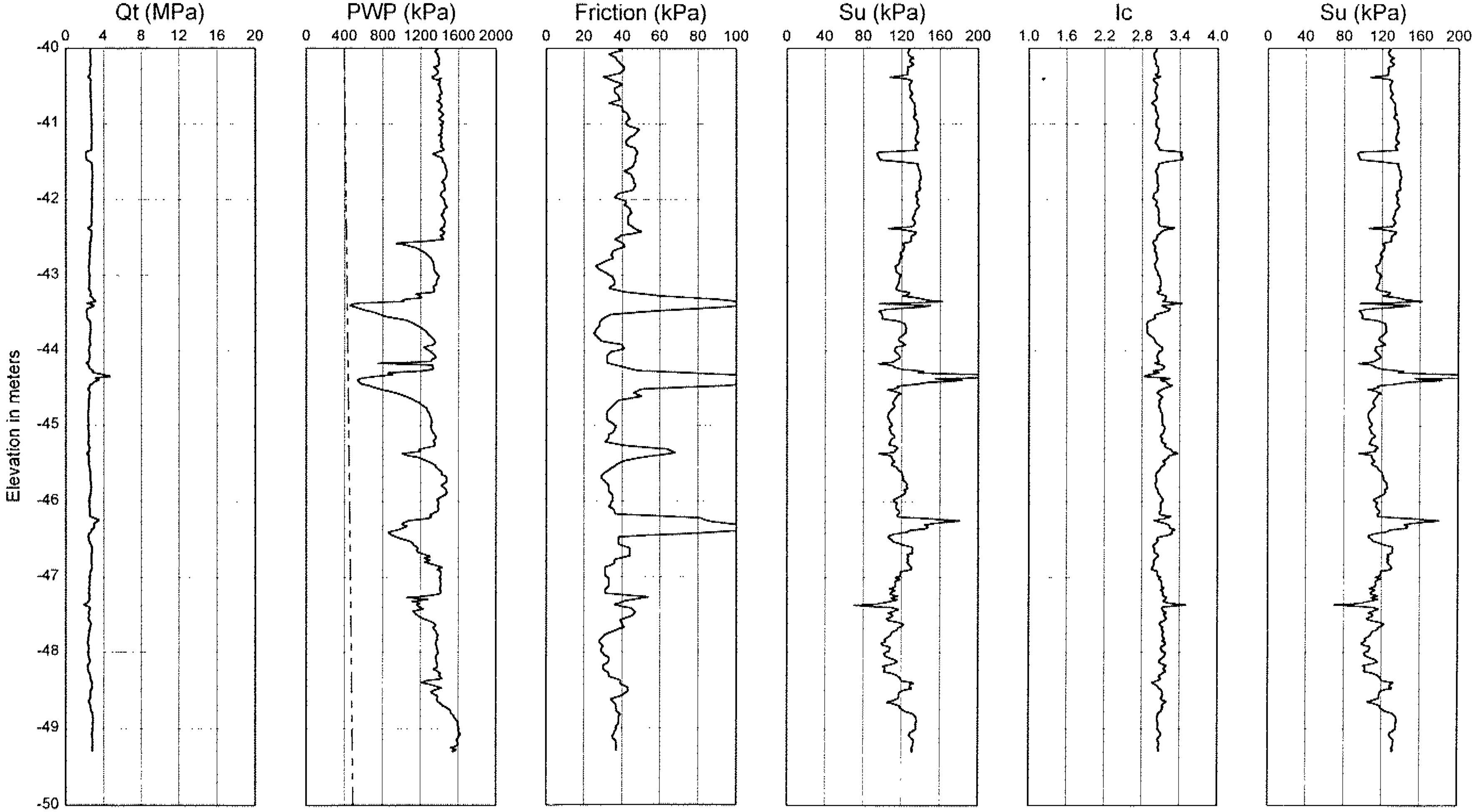
HWY 17

Cone Penetration Test - 5

Test Date : 0-/0 /0-
Location : SEE FIGURE

Operator : Golder Associates

Ground Surf. Elev. : 0.00
Water Table Depth : 0.00



Qt normalized for unequal end area effects

Su = (Qt - SigmaV) / Nk
Nk 15
Gamma = 17 kN/m³

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

Su = (Qt - SigmaV) / Nk
Nk 15
Gamma = 17 kN/m³

PROJECT NO. 971-1103 DATE: DRAWN BY. AJW

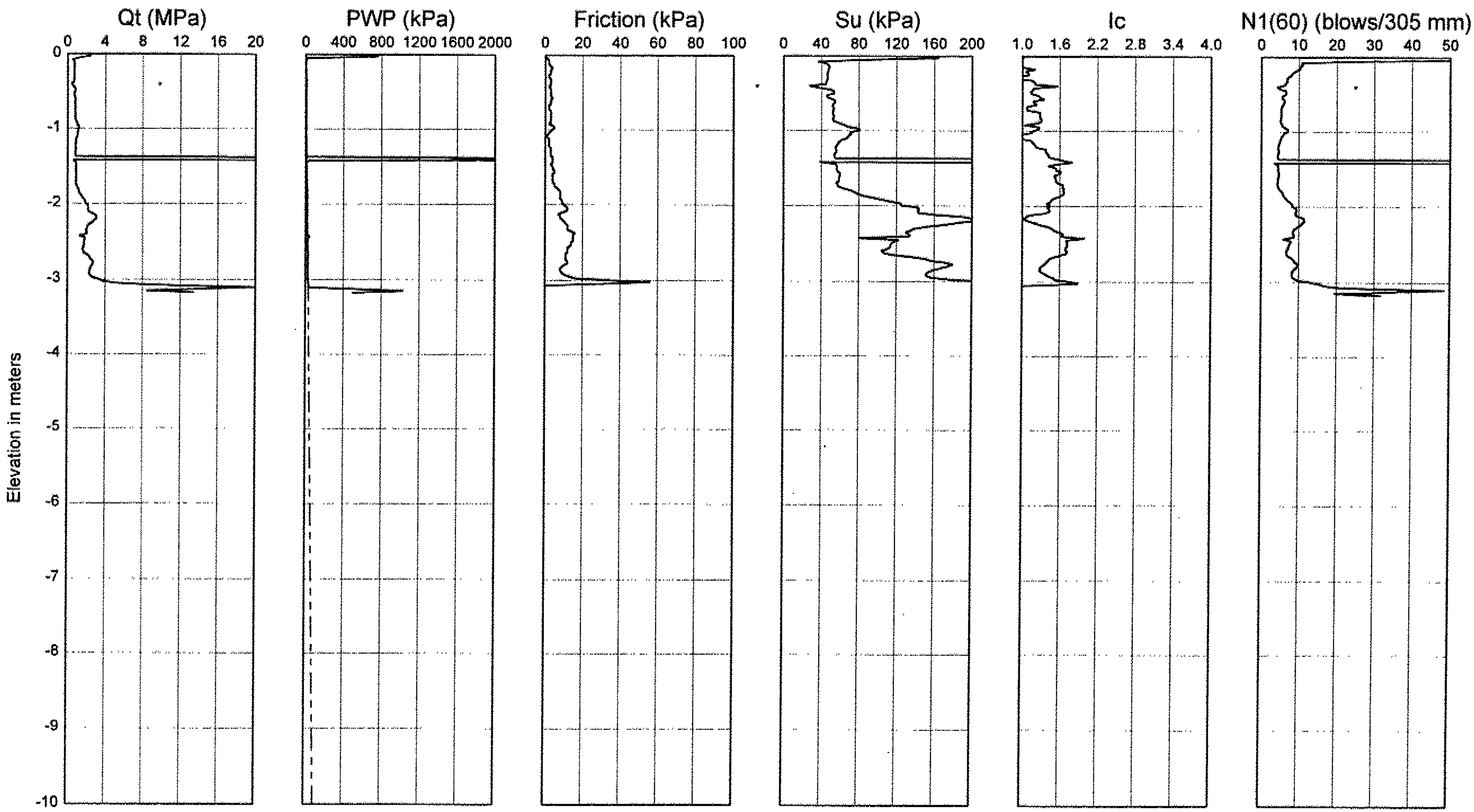
HWY 17

Cone Penetration Test - 6

Test Date : 6-2-97
Location : SEE FIGURE

Operator : Golder Associates

Ground Surf. Elev. : 0.00
Water Table Depth : 0.00



Qt normalized for
unequal end area effects

Su = (Qt - SigmaV) / Nk
Nk = 15
Gamma = 17 kN/m³

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

After Jefferies and Davies (1993)

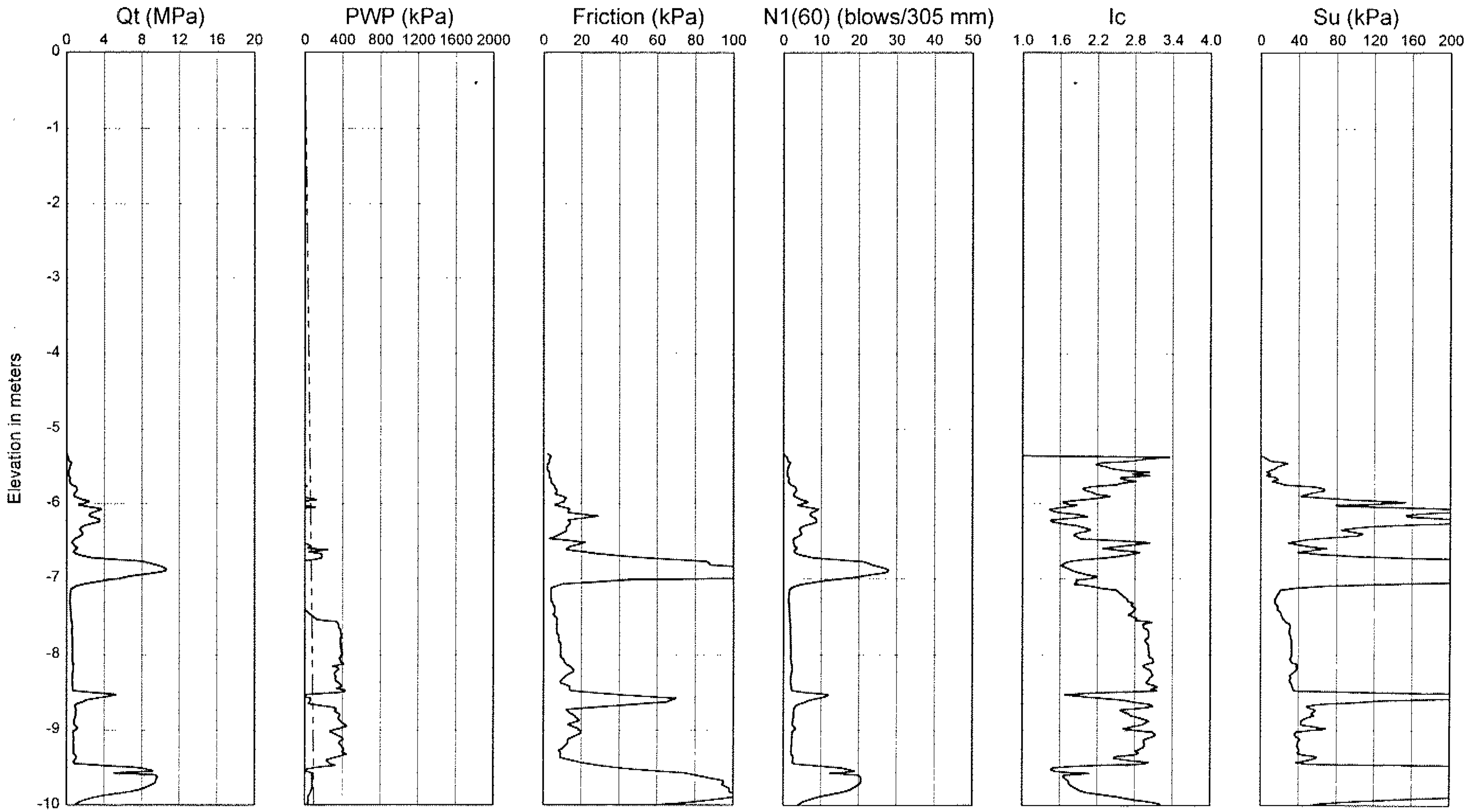
PROJECT NO. 971-1103 DATE: DRAWN BY: AJW HWY 17

Cone Penetration Test - 6

Test Date : 6-2-97
Location : SEE FIGURE

Operator : Golder Associates

Ground Surf. Elev. : 0.00
Water Table Depth : 0.00



Qt normalized for
unequal end area effects

After Jefferies and Davies (1993)

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

Su = (Qt - SigmaV) / Nk
Nk = 15
Gamma = 17 kN/m³

PROJECT NO. 971-1103 DATE. DRAWN BY: AJW

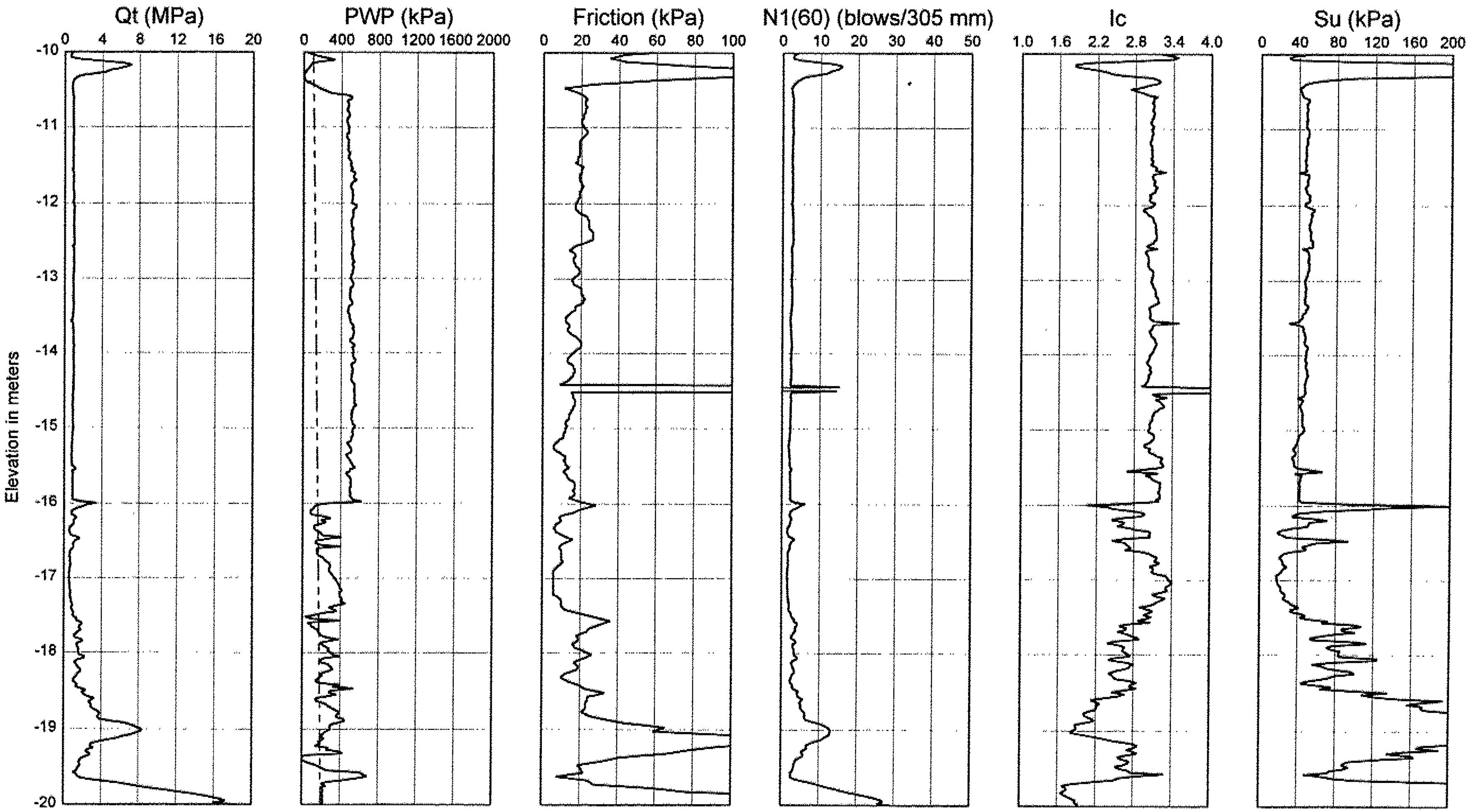
HWY 17

Cone Penetration Test - 6

Test Date : 6-2-97
Location : SEE FIGURE

Operator : Golder Associates

Ground Surf. Elev. : 0.00
Water Table Depth : 0.00



Qt normalized for unequal end area effects

After Jefferies and Davies (1993)

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

Su = (Qt - SigmaV) / Nk
Nk = 15
Gamma = 17 kN/m³

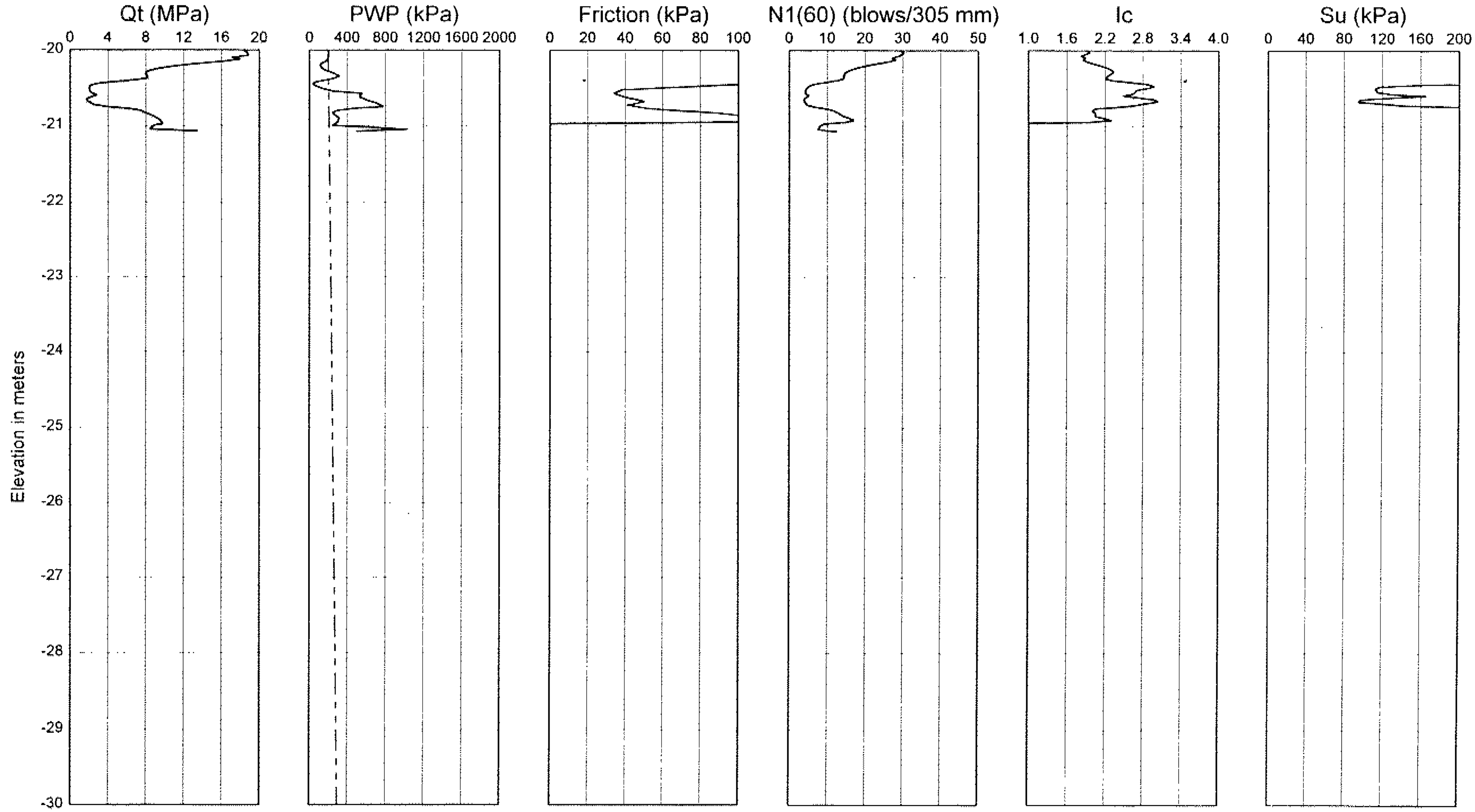
PROJECT NO. 971-1103 DATE: DRAWN BY: AJW

Cone Penetration Test - 6

Test Date : 6-2-97
Location : SEE FIGURE

Operator : Golder Associates

Ground Surf. Elev. : 0.00
Water Table Depth : 0.00



Qt normalized for unequal end area effects

After Jefferies and Davies (1993)

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

Su = (Qt - SigmaV) / Nk
Nk = 15
Gamma = 17 kN/m³

PROJECT NO. 971-1103 DATE: DRAWN BY. AJW

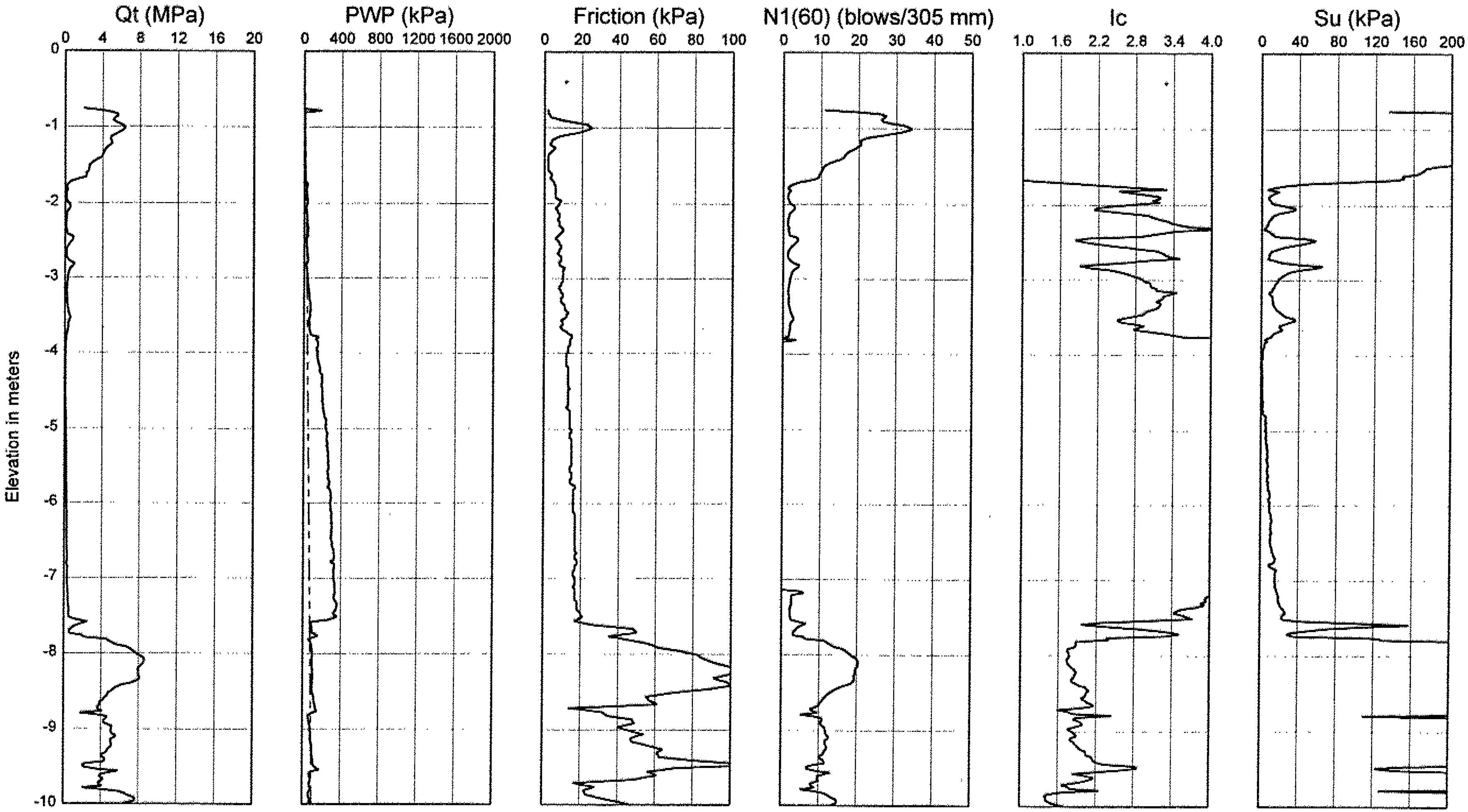
HWY 17

Cone Penetration Test - 7

Test Date : 10-2-97
Location : SEE FIGURE

Operator : Golder Associates

Ground Surf. Elev. : 0.00
Water Table Depth : 0.00



Qt normalized for unequal end area effects

After Jefferies and Davies (1993)

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

Su = (Qt - SigmaV) / Nk
Nk = 15
Gamma = 17 kN/m³

PROJECT NO. 971-1103 DATE: DRAWN BY: AJW

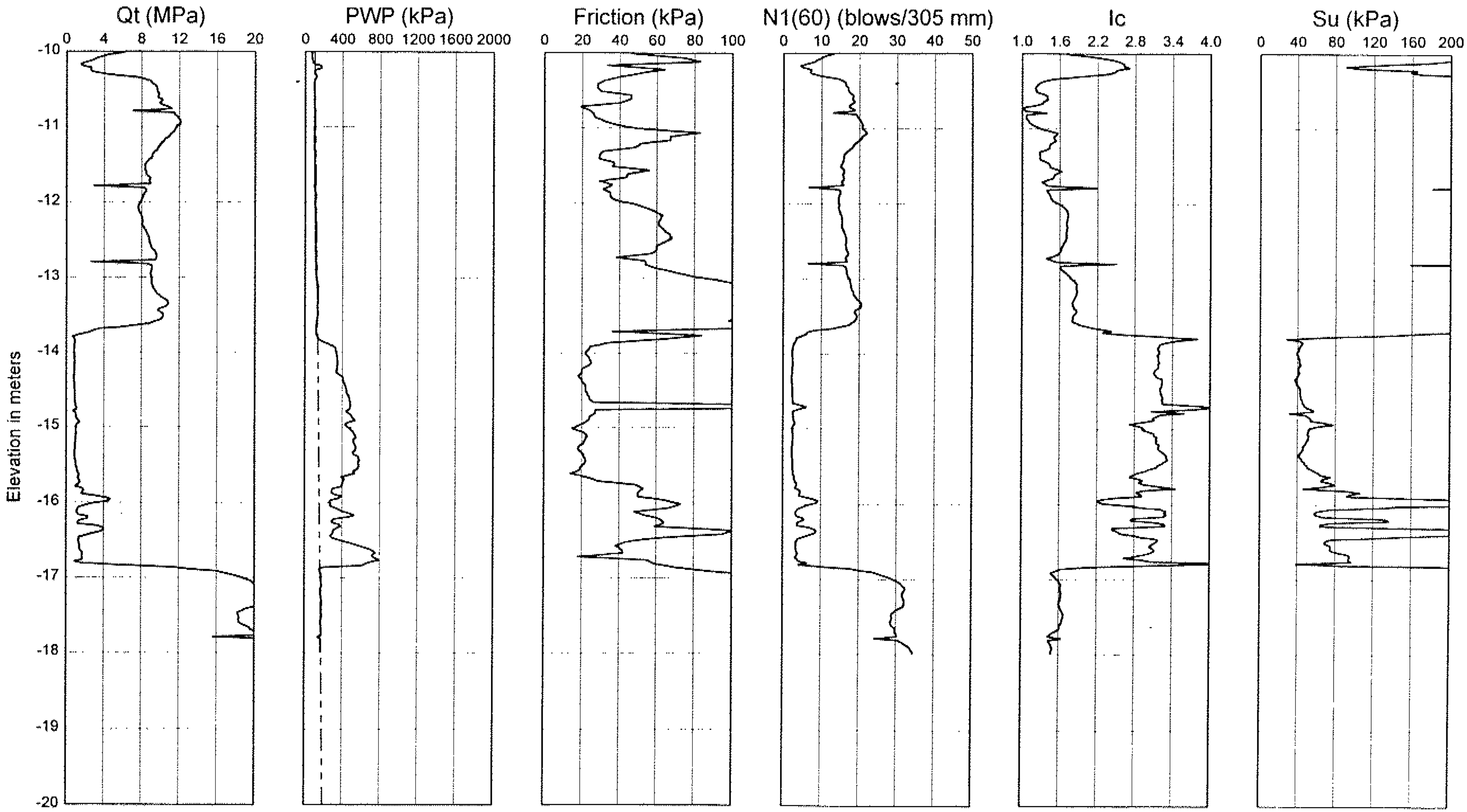
HWY 17

Cone Penetration Test - 7

Test Date : 10-2-97
Location : SEE FIGURE

Operator : Golder Associates

Ground Surf. Elev. : 0.00
Water Table Depth : 0.00



Qt normalized for
unequal end area effects

After Jefferies and Davies (1993)

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

Su = (Qt - SigmaV) / Nk
Nk = 15
Gamma = 17 kN/m³

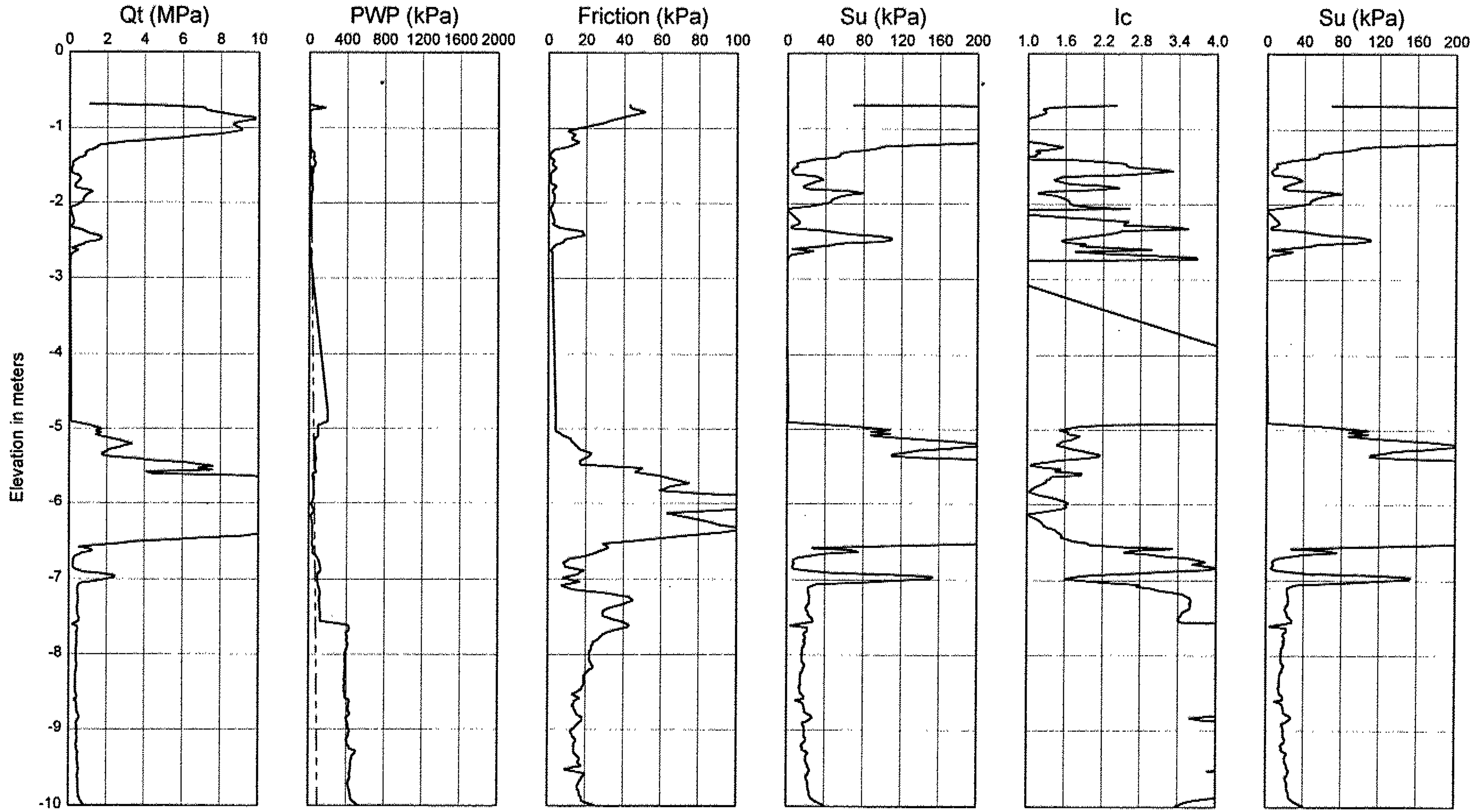
PROJECT NO. 971-1103 DATE: DRAWN BY: AJW

Cone Penetration Test - 8

Test Date : 0-/0 /0-
Location : SEE FIGURE

Operator : Golder Associates

Ground Surf. Elev. : 0.00
Water Table Depth : 0.00



Qt normalized for
unequal end area effects

Su = (Qt - SigmaV) / Nk
Nk = 15
Gamma = 17 kN/m³

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

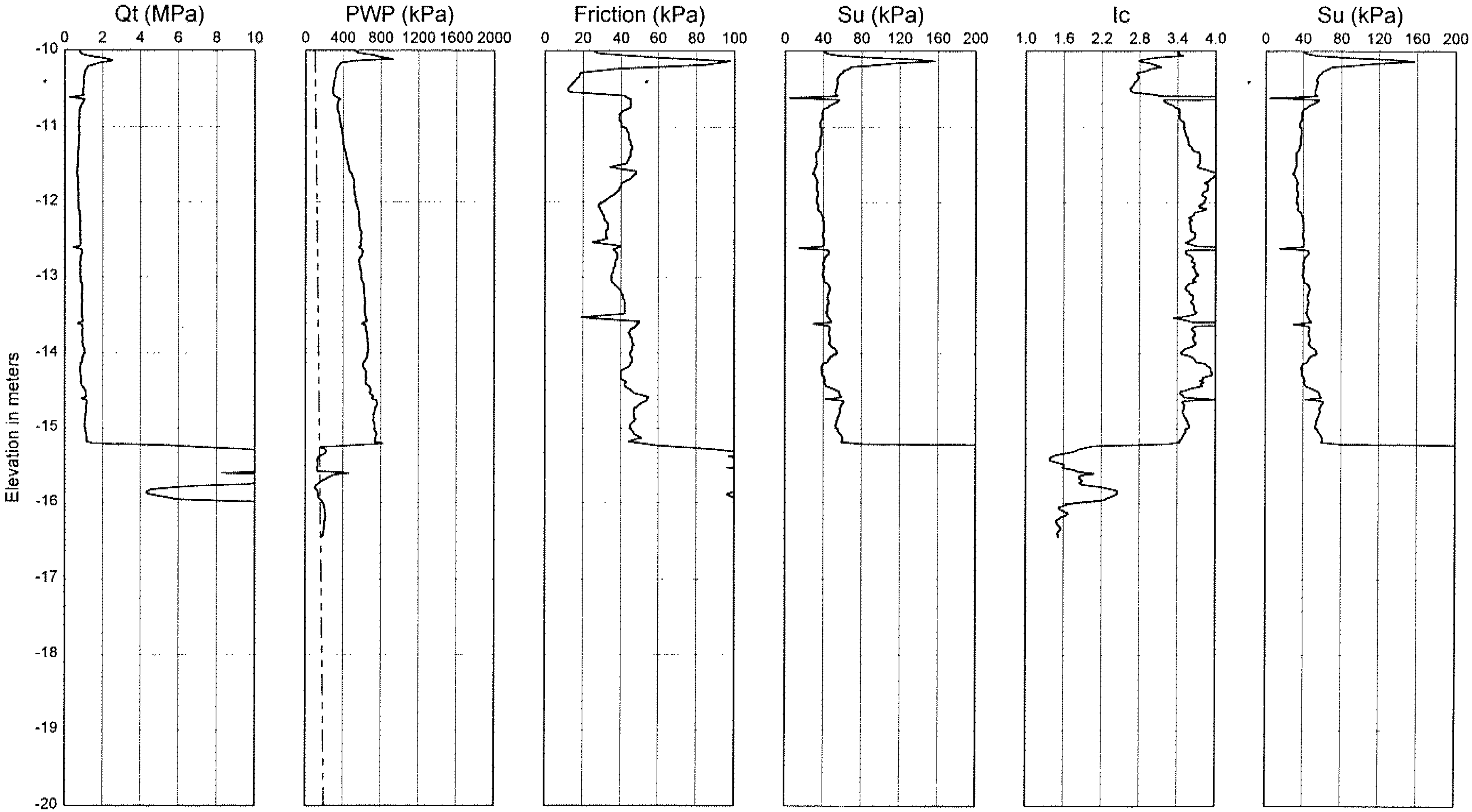
Su = (Qt - SigmaV) / Nk
Nk = 15
Gamma = 17 kN/m³

Cone Penetration Test - 8

Test Date : 0-/0 /0-
Location : SEE FIGURE

Operator : Golder Associates

Ground Surf. Elev. : 0.00
Water Table Depth : 0.00



Qt normalized for
unequal end area effects

Su = (Qt - SigmaV) / Nk
Nk = 15
Gamma = 17 kN/m³

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

Su = (Qt - SigmaV) / Nk
Nk = 15
Gamma = 17 kN/m³

PROJECT NO 971-1103 DATE: DRAWN BY:

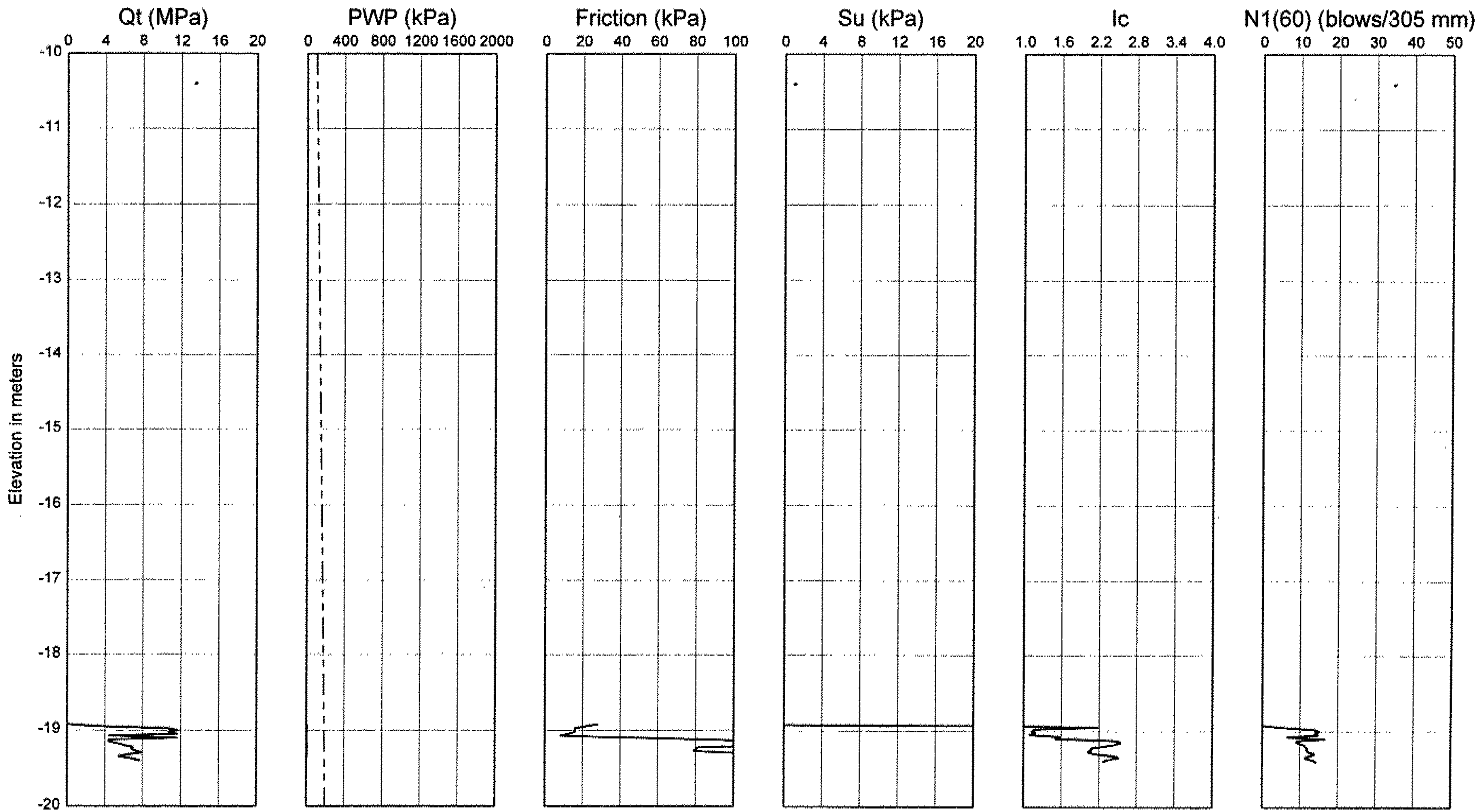
HWY 17

Cone Penetration Test - 8A

Test Date : 0-/0 /0-
Location : SEE FIGURE

Operator : Golder Associates

Ground Surf. Elev. : 0.00
Water Table Depth : 0.00



Qt normalized for
unequal end area effects

Su = (Qt - SigmaV) / Nk
Nk = 15
Gamma = 17 kN/m³

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

After Jefferies and Davies (1993)

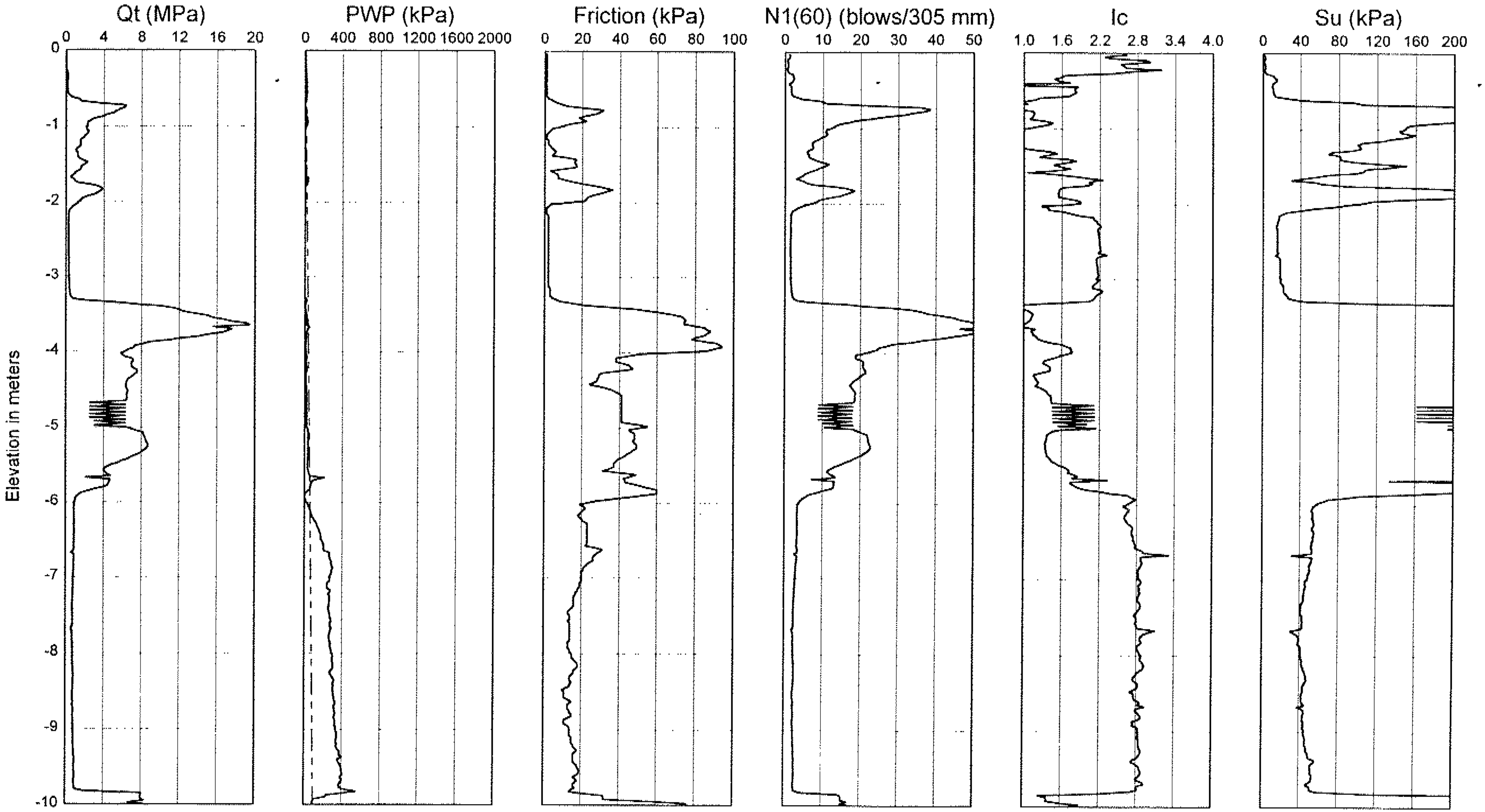
PROJECT NO. 971-1103 DATE: DRAWN BY:

Cone Penetration Test - 9

Test Date : 0-/0 /0-
Location : SEE FIGURE

Operator : Golder Associates

Ground Surf. Elev. : 0.00
Water Table Depth : 0.00



Qt normalized for
unequal end area effects

After Jefferies and Davies (1993)

After Jefferies and Davies (1993)
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

Su = (Qt - SigmaV) / Nk
Nk = 15
Gamma = 17 kN/m³

PROJECT NO. 971-1103 DATE: DRAWN BY:

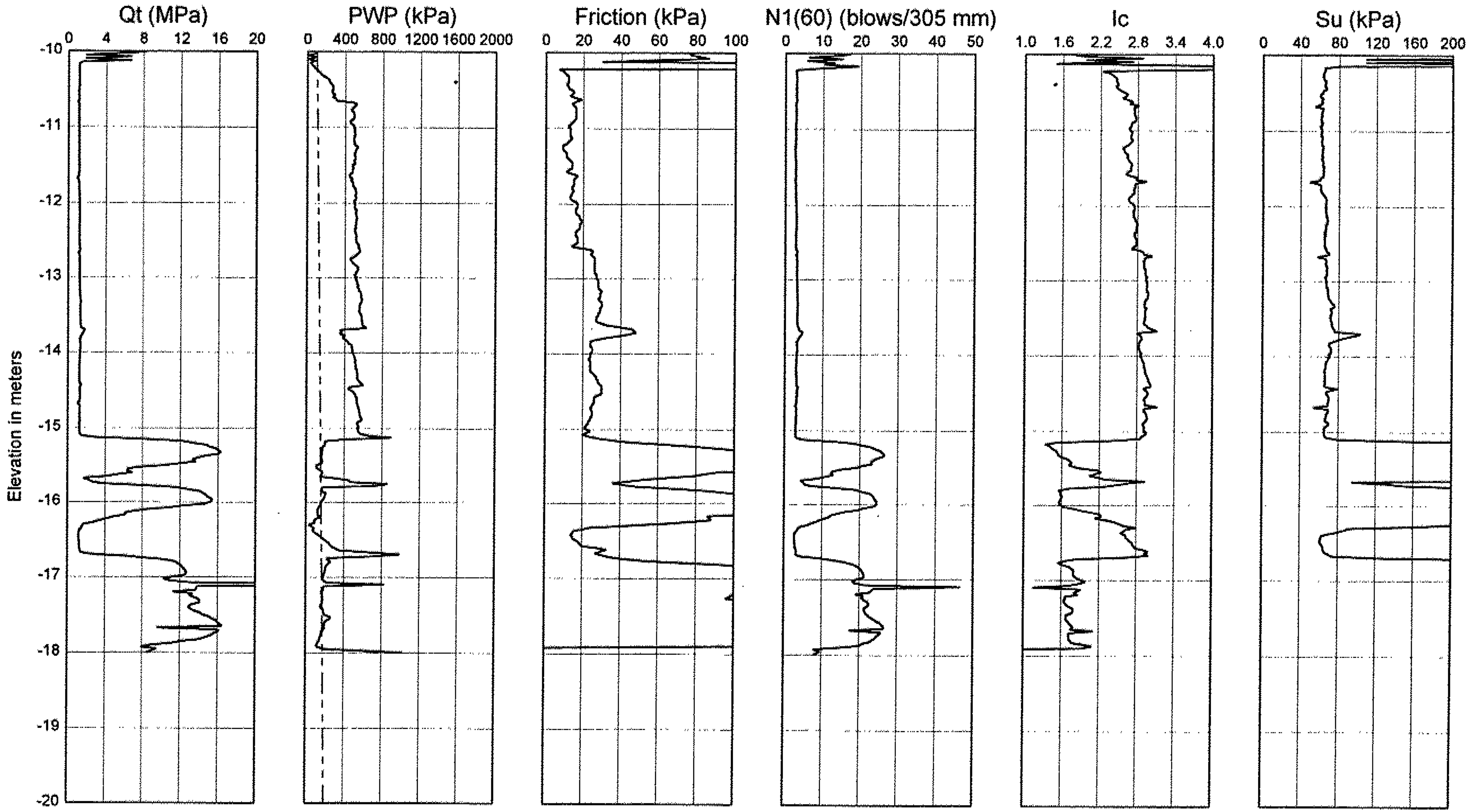
HWY 17

Cone Penetration Test - 9

Test Date : 0-/0 /0-
Location : SEE FIGURE

Operator : Golder Associates

Ground Surf. Elev. : 0.00
Water Table Depth : 0.00



Qt normalized for unequal end area effects

After Jefferies and Davies (1993)

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

Su = (Qt - SigmaV) / Nk
Nk = 15
Gamma = 17 kN/m³

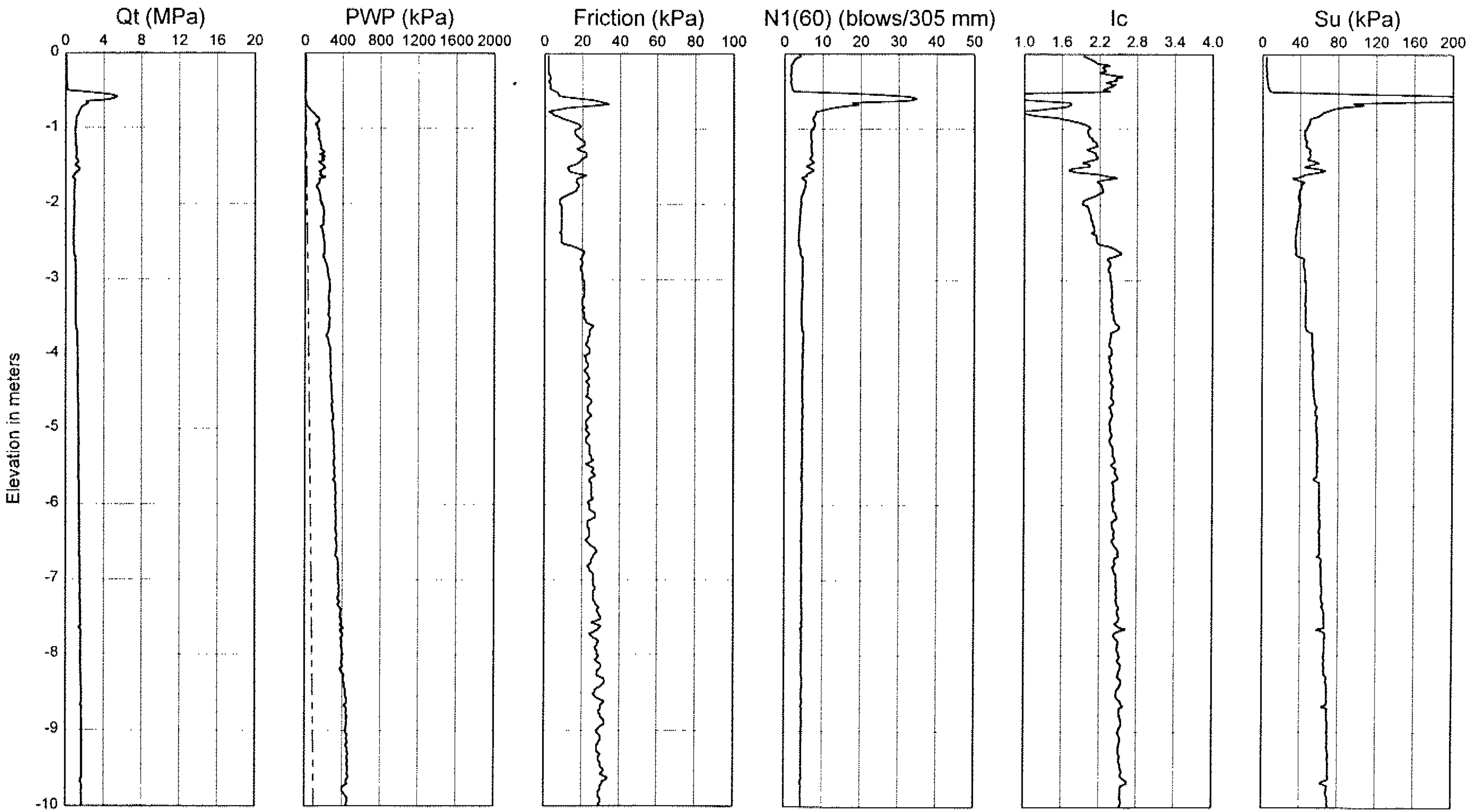
PROJECT NO. 971-1103 DATE: DRAWN BY:

Cone Penetration Test - 10

Test Date : 0-/0 /0-
Location : SEE FIGURE

Operator : Golder Associates

Ground Surf. Elev. : 0.00
Water Table Depth : 0.00



Qt normalized for
unequal end area effects

After Jefferies and Davies (1993)

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

Su = (Qt - SigmaV) / Nk
Nk = 22
Gamma = 17 kN/m³

PROJECT NO. 971-1103 DATE: DRAWN BY:

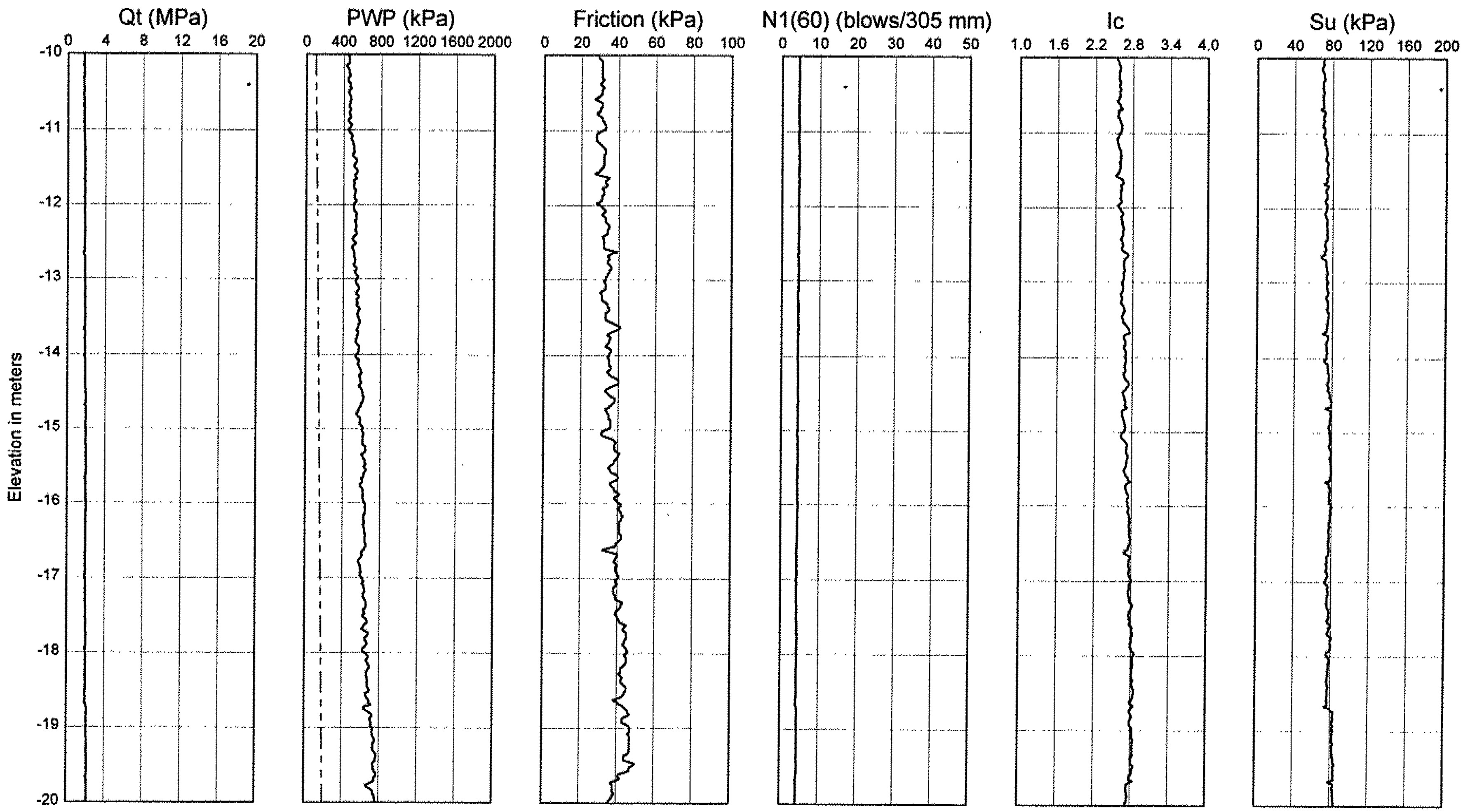
HWY 17

Cone Penetration Test - 10

Test Date : 0-/0 /0-
Location : SEE FIGURE

Operator : Golder Associates

Ground Surf. Elev. : 0.00
Water Table Depth : 0.00



Qt normalized for unequal end area effects

After Jefferies and Davies (1993)

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

Su = (Qt - SigmaV) / Nk
Nk = 22
Gamma = 17 kN/m³

PROJECT NO. 971-1103 DATE: DRAWN BY:

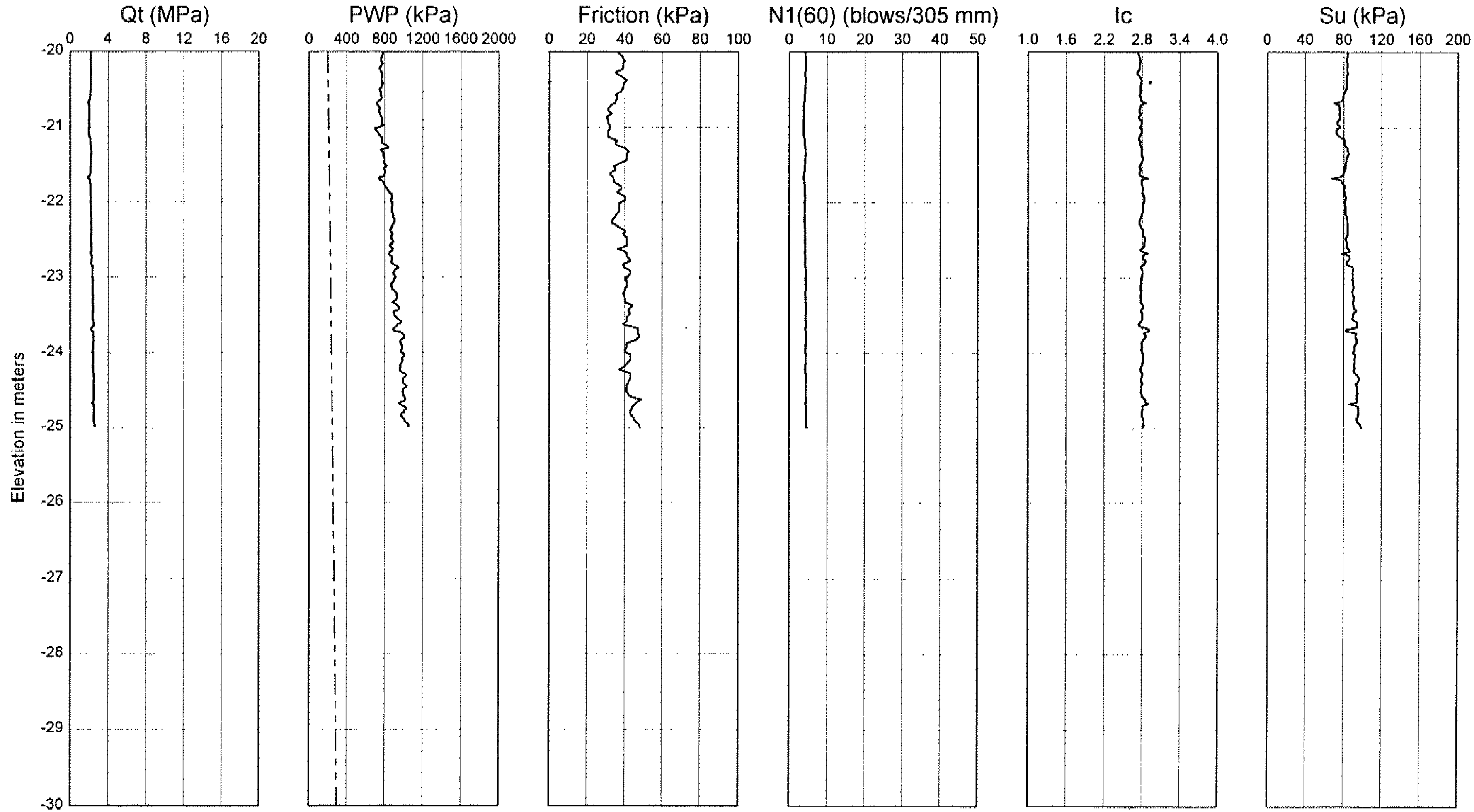
HWY 17

Cone Penetration Test - 10

Test Date : 0-/0 /0-
Location : SEE FIGURE

Operator : Golder Associates

Ground Surf. Elev. : 0.00
Water Table Depth : 0.00



Qt normalized for
unequal end area effects

After Jefferies and Davies (1993)

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

Su = (Qt - SigmaV) / Nk
Nk = 22
Gamma = 17 kN/m³

PROJECT NO. 971-1103 DATE: DRAWN BY:

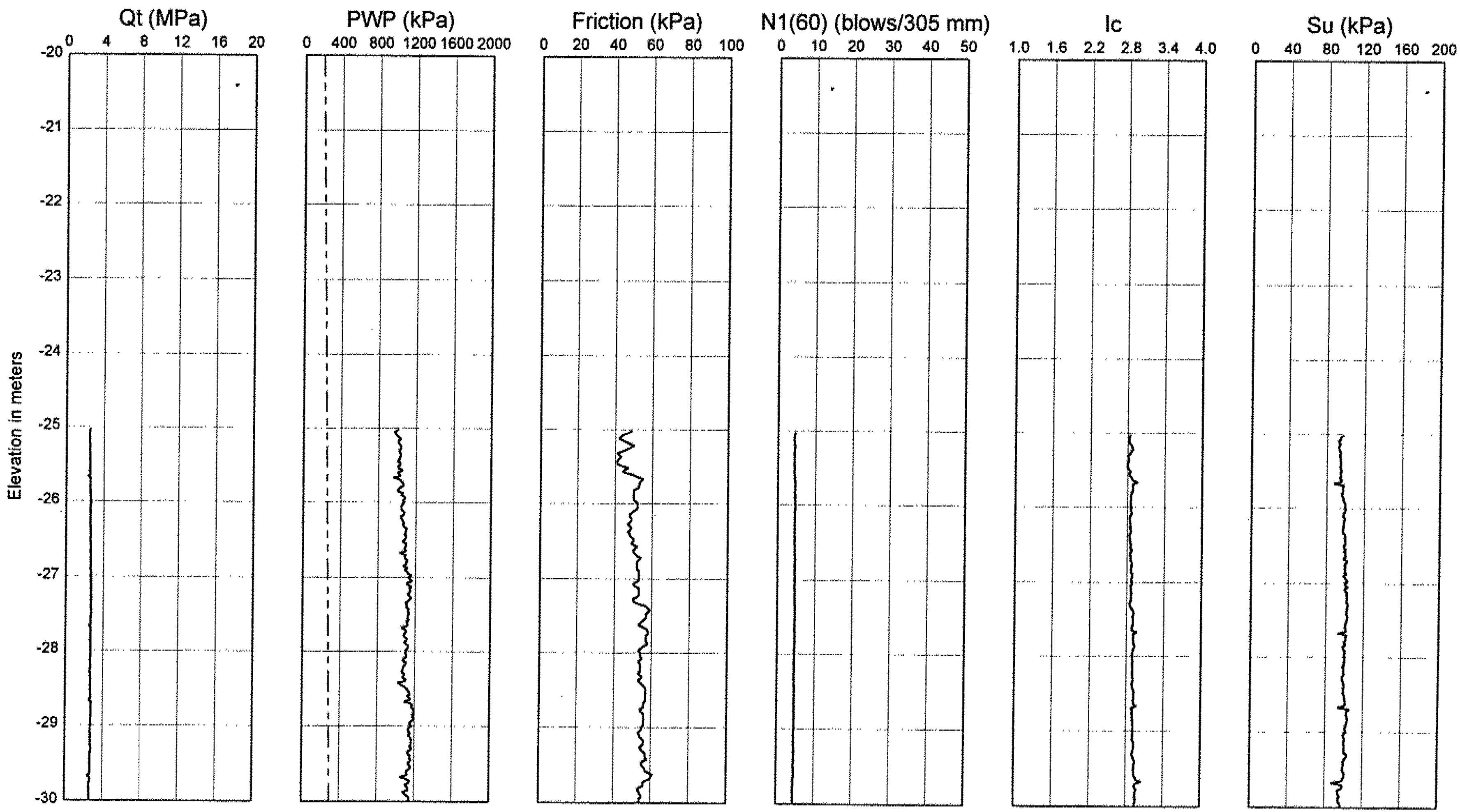
HWY 17

Cone Penetration Test - 10

Test Date : 0-/0 /0-
Location : SEE FIGURE

Operator : Golder Associates

Ground Surf. Elev. : 0.00
Water Table Depth : 0.00



Qt normalized for unequal end area effects

After Jefferies and Davies (1993)

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

Su = (Qt - SigmaV) / Nk
Nk = 22
Gamma = 17 kN/m³

PROJECT NO. 971-1103 DATE: DRAWN BY:

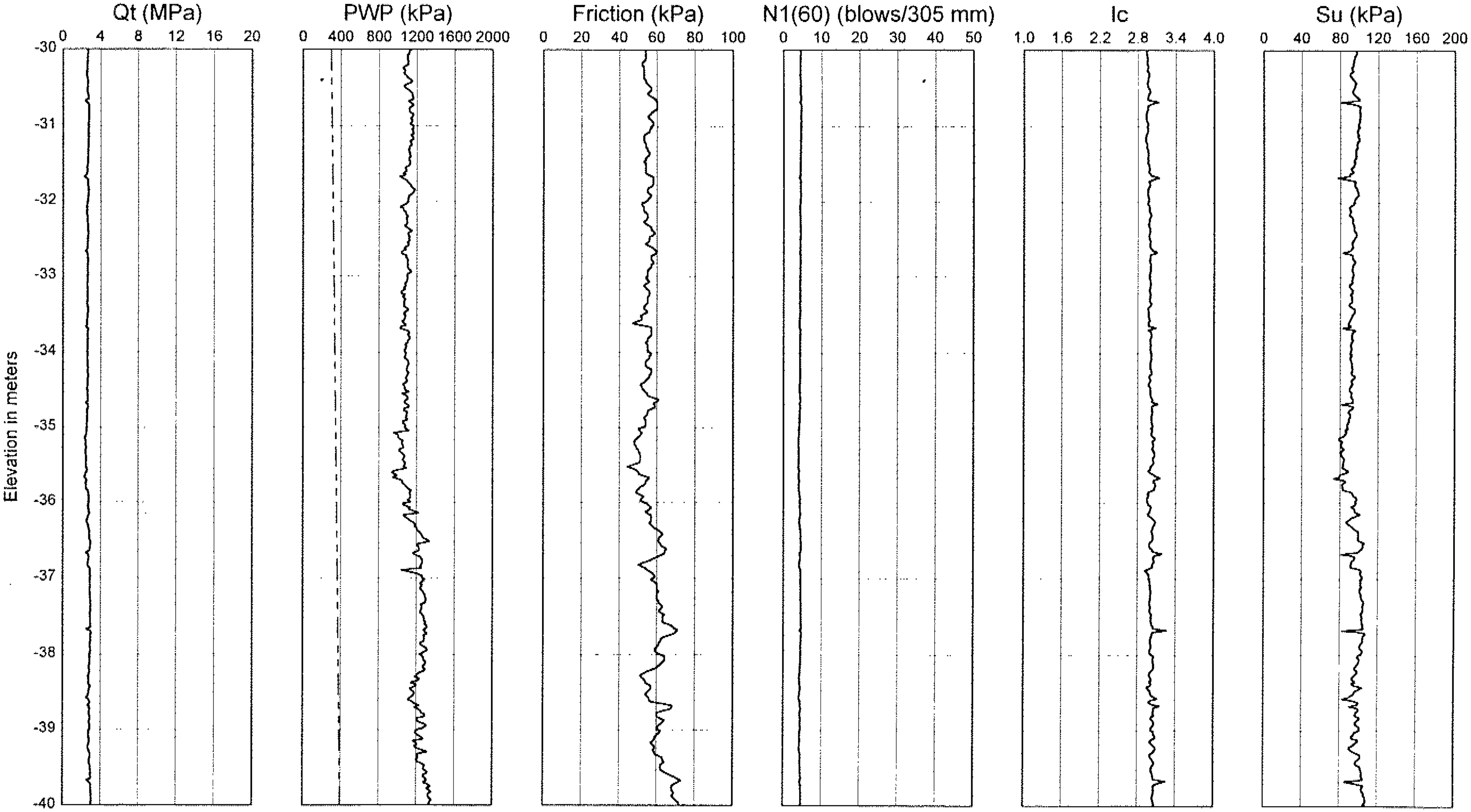
HWY 17

Cone Penetration Test - 10

Test Date : 0-/0 /0-
Location : SEE FIGURE

Operator : Golder Associates

Ground Surf. Elev. : 0.00
Water Table Depth : 0.00



Qt normalized for unequal end area effects

After Jefferies and Davies (1993)

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

Su = (Qt - SigmaV) / Nk
Nk = 22
Gamma = 17 kN/m³

PROJECT NO. 971-1103 DATE. DRAWN BY·

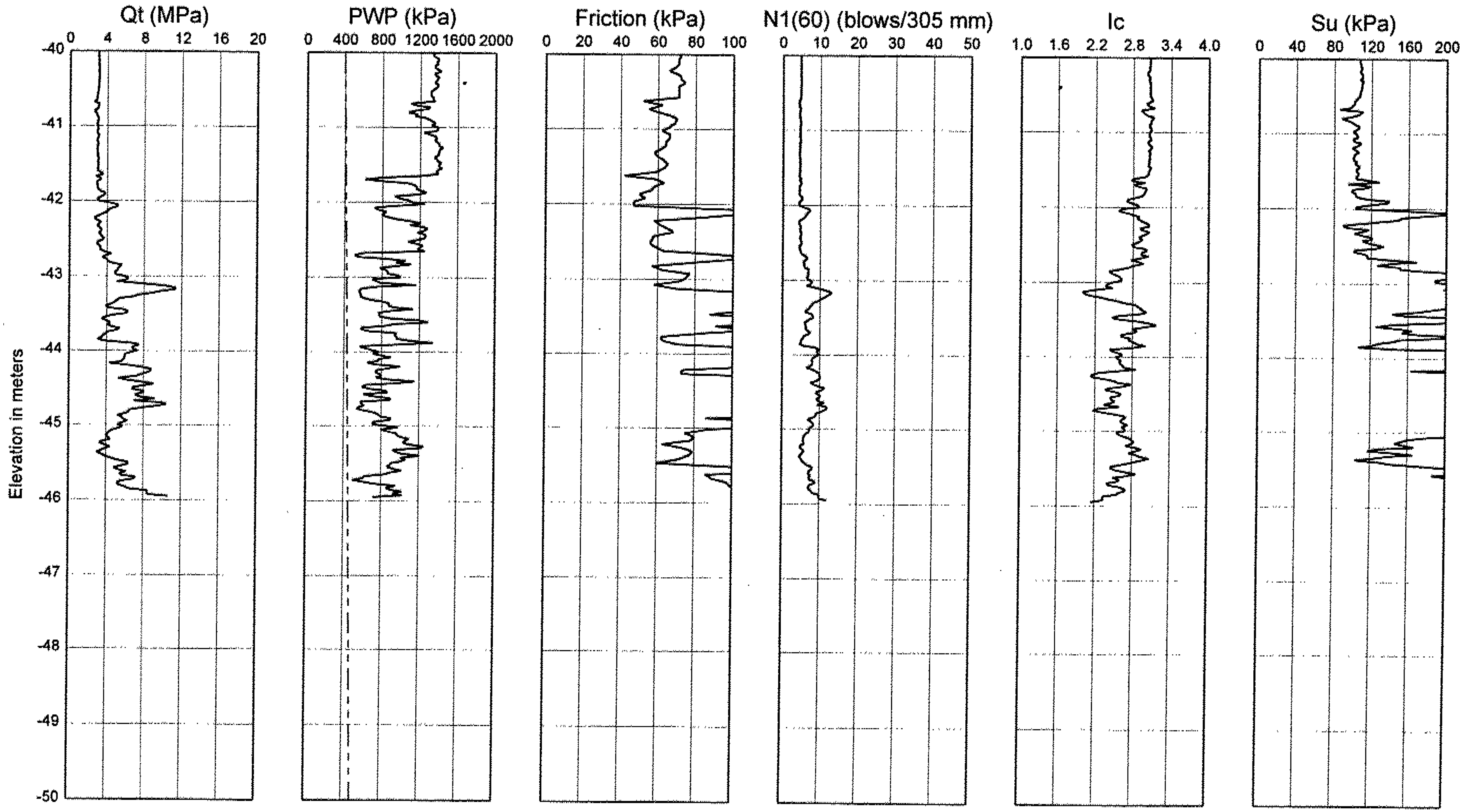
HWY 17

Cone Penetration Test - 10

Test Date : 0-/0 /0-
Location : SEE FIGURE

Operator : Golder Associates

Ground Surf. Elev. : 0.00
Water Table Depth : 0.00



Qt normalized for
unequal end area effects

After Jefferies and Davies (1993)

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

Su = (Qt - SigmaV) / Nk
Nk = 22
Gamma = 17 kN/m³

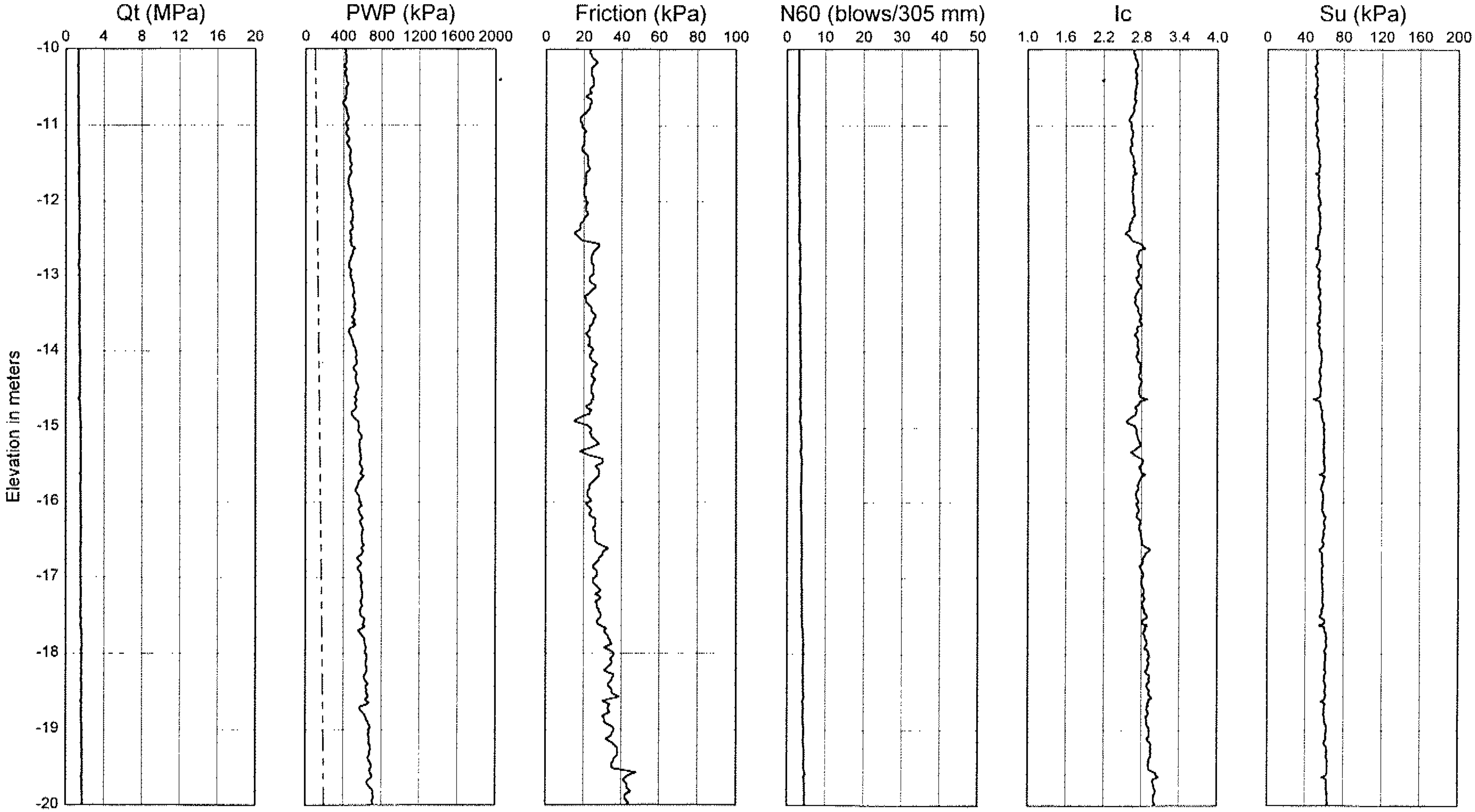
PROJECT NO. 971-1103 DATE: DRAWN BY:

Cone Penetration Test - 11

Test Date : 0-/0 /0-
Location : SEE FIGURE

Operator : Golder Associates

Ground Surf. Elev. : 0.00
Water Table Depth : 0.00



Qt normalized for
unequal end area effects

After Jefferies and Davies (1993)

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

Su (Qt - SigmaV) / Nk
Nk - 22
Gamma = 17 kN/m³

PROJECT NO. 971-1103 DATE: DRAWN BY:

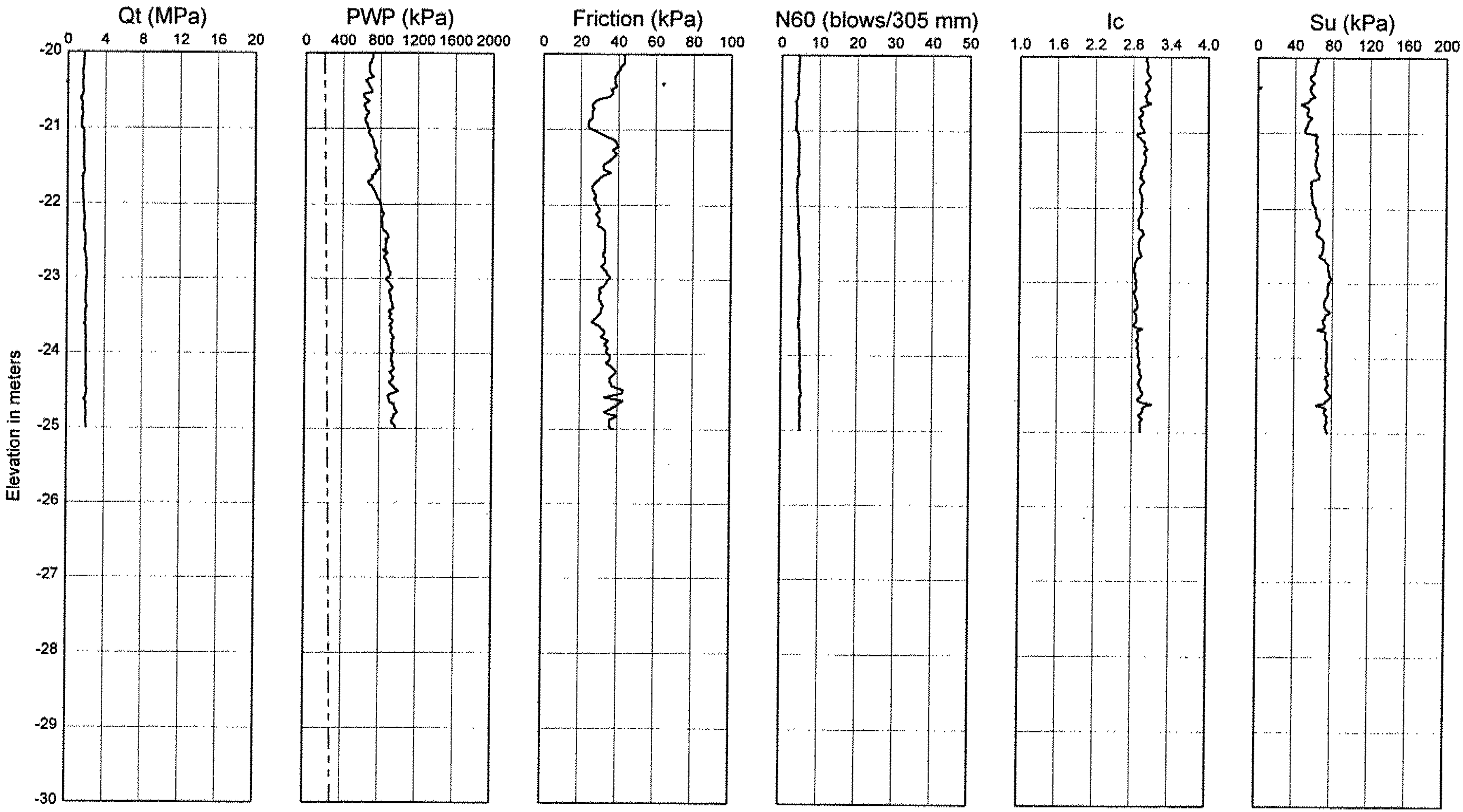
HWY 17

Cone Penetration Test - 11

Test Date : 0-/0 /0-
Location : SEE FIGURE

Operator : Golder Associates

Ground Surf. Elev. : 0.00
Water Table Depth : 0.00



Qt normalized for
unequal end area effects

After Jefferies and Davies (1993)

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

Su = (Qt - SigmaV) / Nk
Nk = 22
Gamma = 17 kN/m³

PROJECT NO. 971-1103 DATE: DRAWN BY:

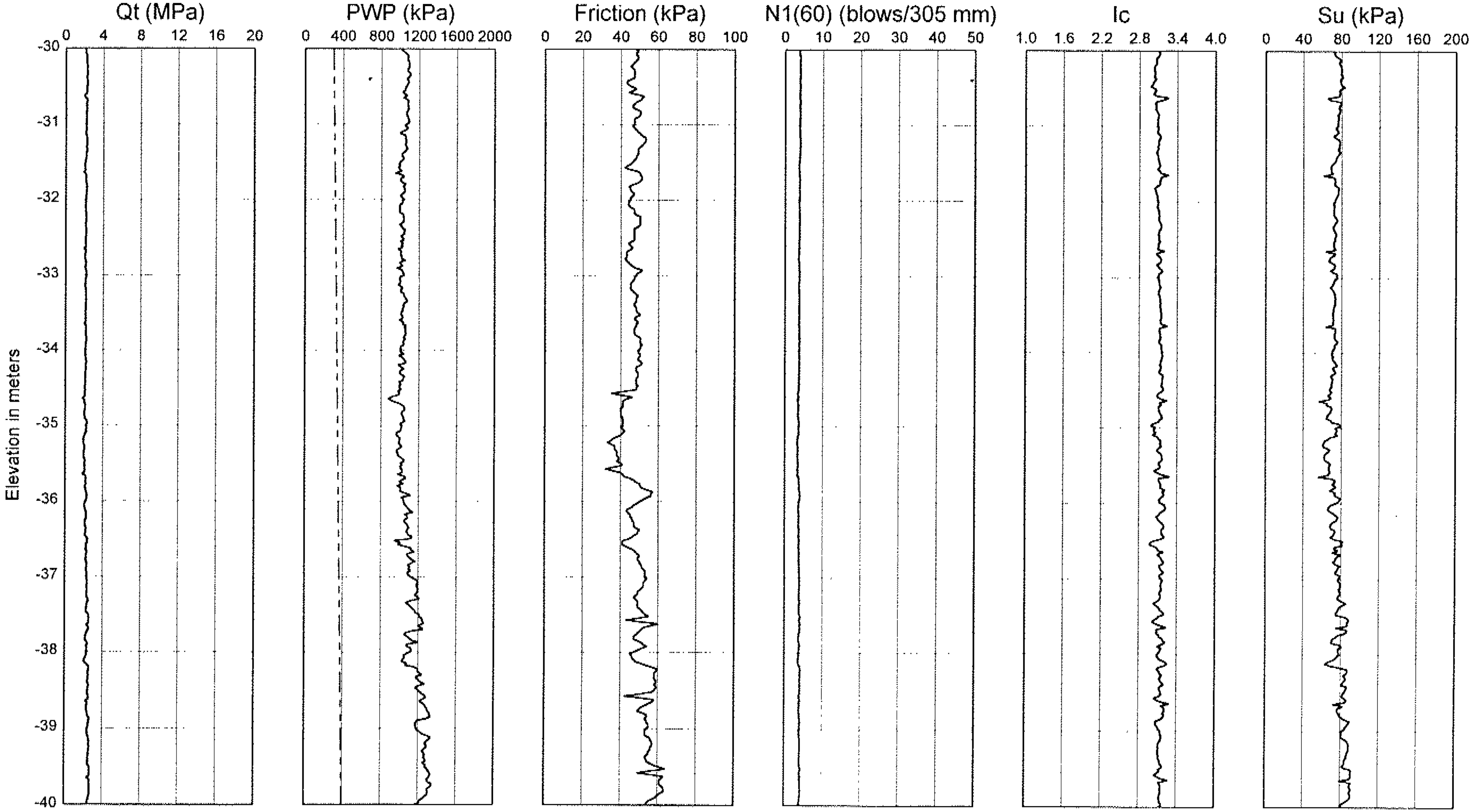
HWY 17

Cone Penetration Test - 11

Test Date : 0-/0 /0-
Location : SEE FIGURE

Operator : Golder Associates

Ground Surf. Elev. : 0.00
Water Table Depth : 0.00



Qt normalized for
unequal end area effects

After Jefferies and Davies (1993)

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

Su = (Qt - SigmaV) / Nk
Nk = 22
Gamma = 17 kN/m³

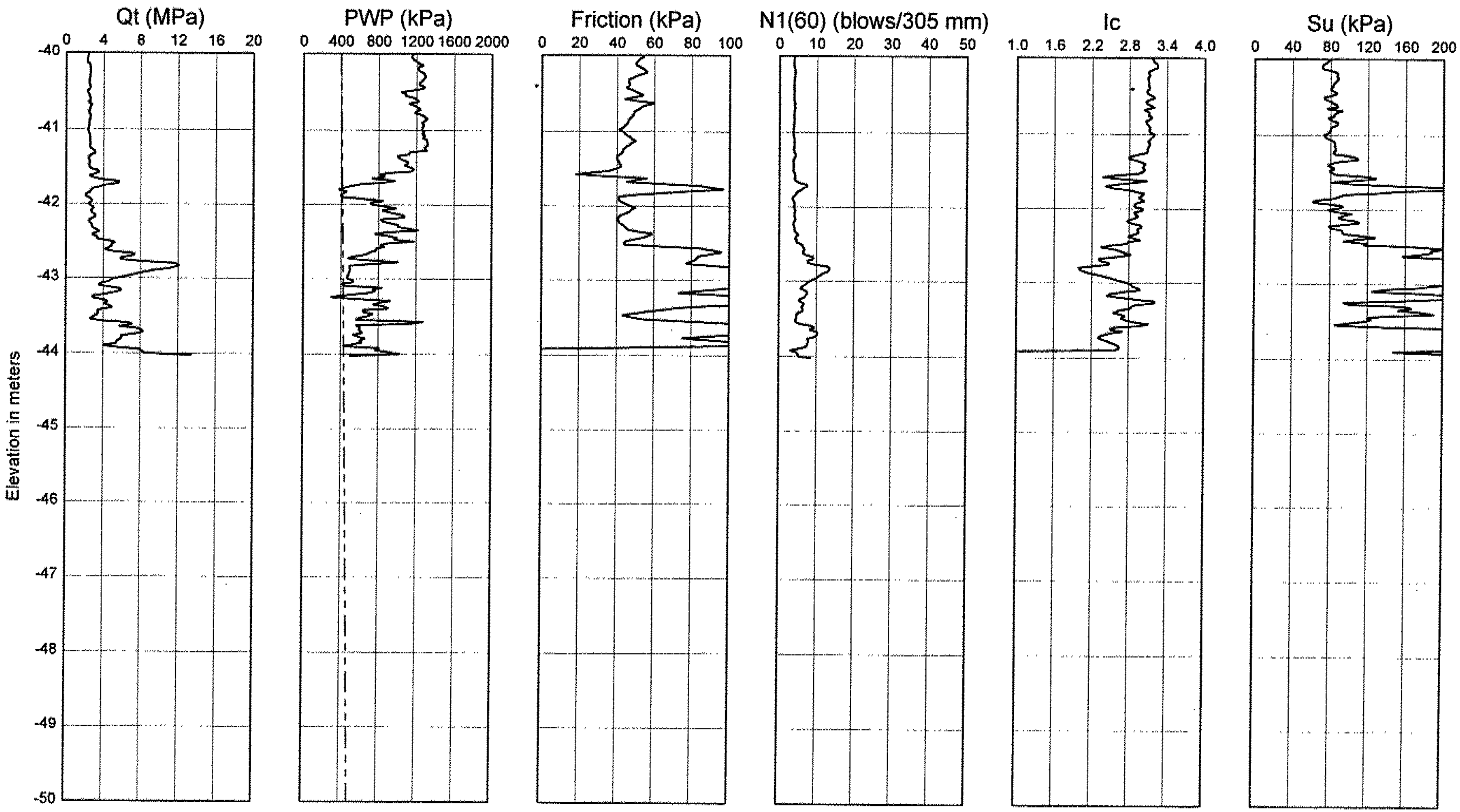
PROJECT NO 971-1103 DATE: DRAWN BY:

Cone Penetration Test - 11

Test Date : 0-/0 /0-
Location : SEE FIGURE

Operator : Golder Associates

Ground Surf. Elev. : 0.00
Water Table Depth : 0.00



Qt normalized for
unequal end area effects

After Jefferies and Davies (1993)

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

Su = (Qt - SigmaV) / Nk
Nk = 22
Gamma = 17 kN/m³

PROJECT NO. 971-1103 DATE: DRAWN BY:

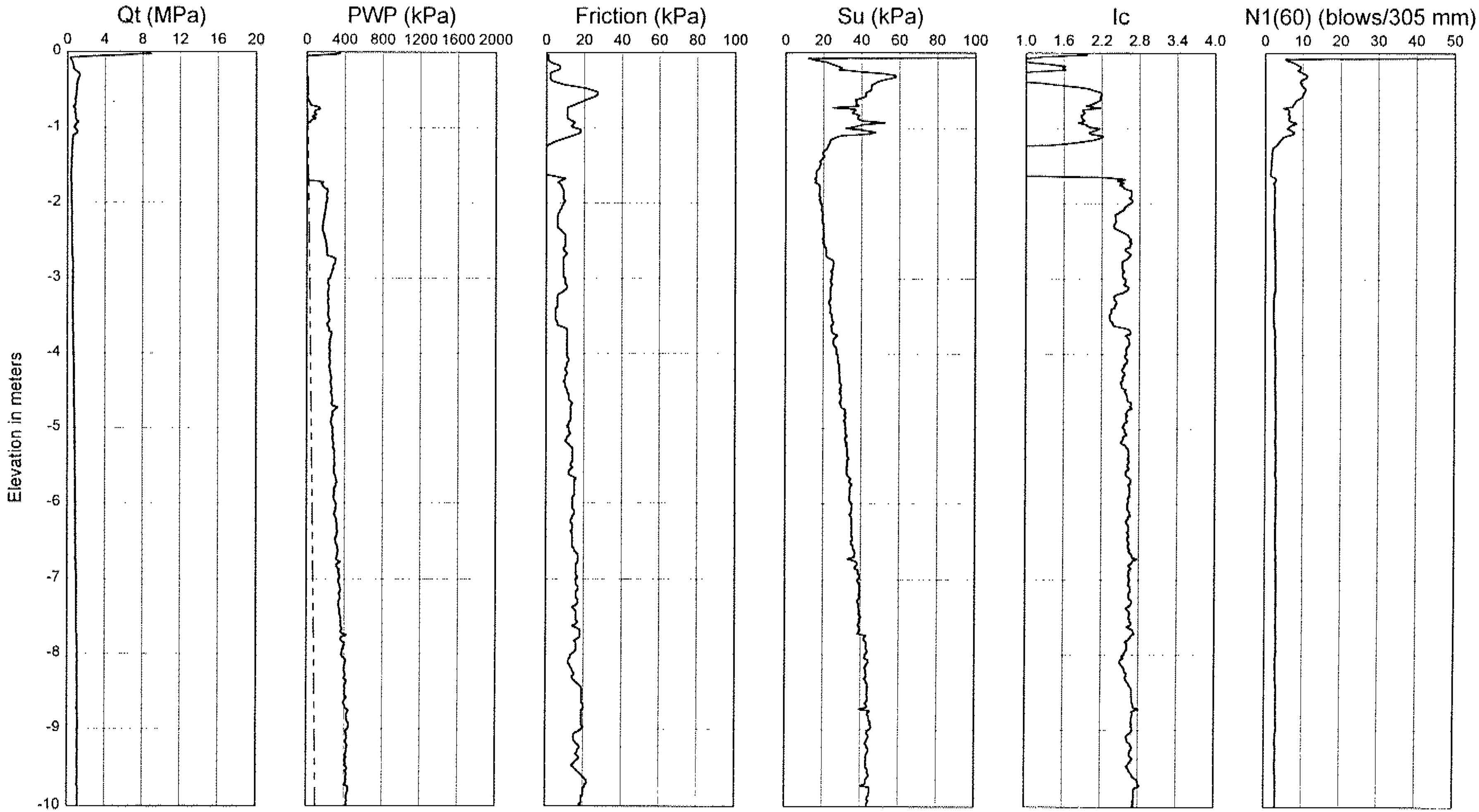
HWY 17

Cone Penetration Test - 12

Test Date : 0-/0 /0-
Location : SEE FIGURE

Operator : Golder Associates

Ground Surf. Elev. : 0.00
Water Table Depth : 0.00



Qt normalized for
unequal end area effects

Su = (Qt - SigmaV) / Nk
Nk = 22
Gamma = 17 kN/m³

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

After Jefferies and Davies (1993)

PROJECT NO. 971-1103 DATE. DRAWN BY.

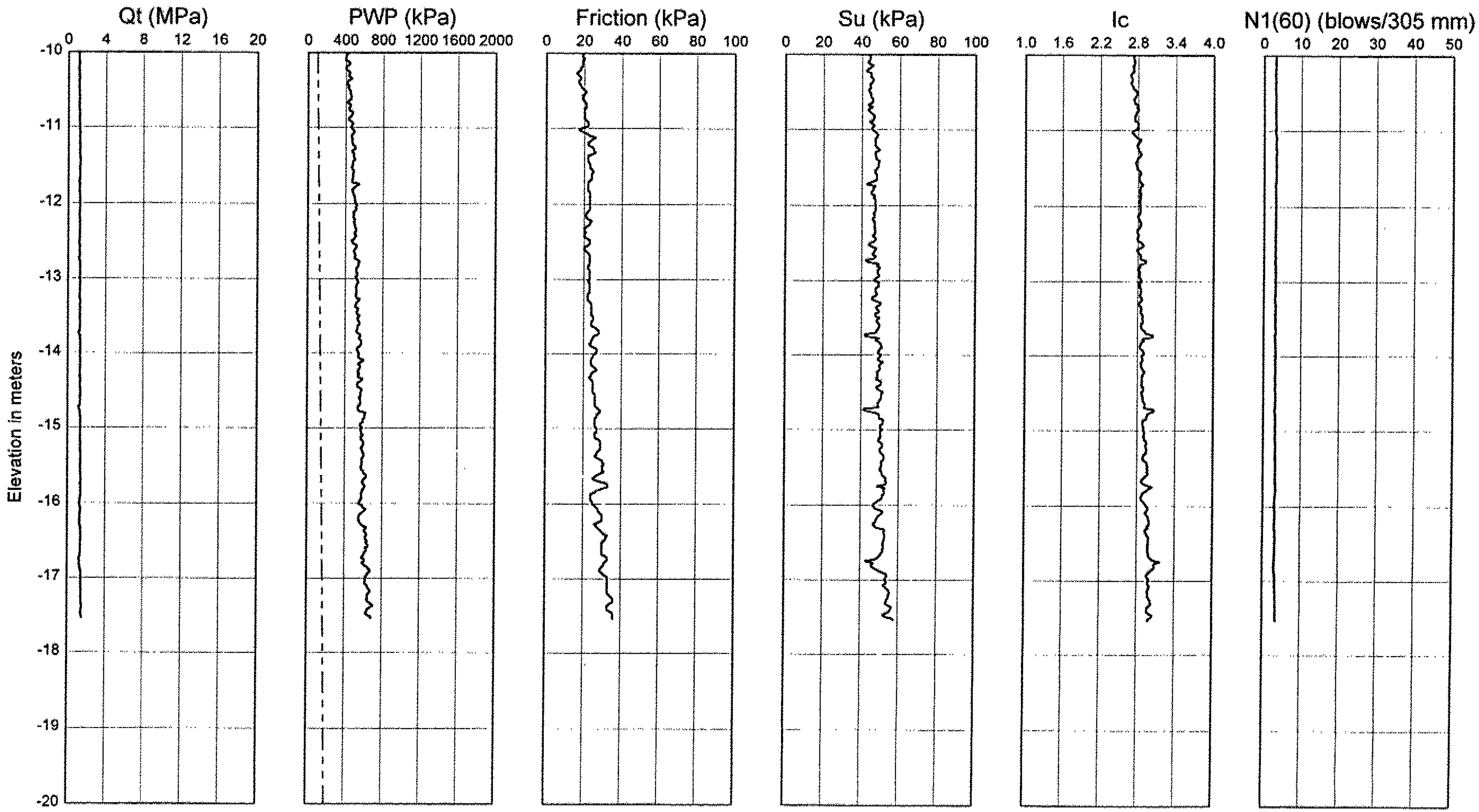
HWY 17

Cone Penetration Test - 12

Test Date : 0-/0 /0-
Location : SEE FIGURE

Operator : Golder Associates

Ground Surf. Elev. : 0.00
Water Table Depth : 0.00



Qt normalized for
unequal end area effects

Su = (Qt - SigmaV) / Nk
Nk = 22
Gamma = 17 kN/m³

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

After Jefferies and Davies (1993)

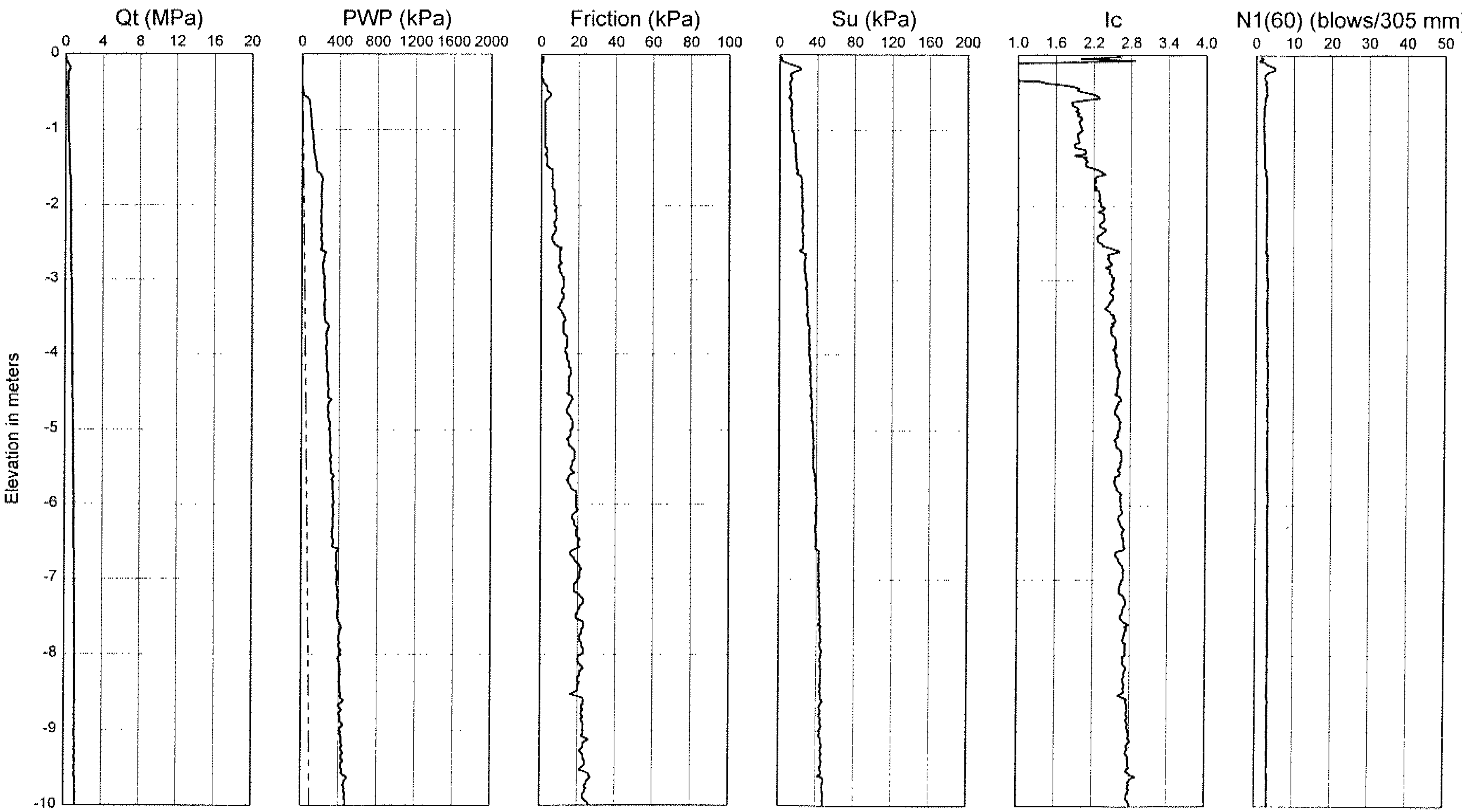
PROJECT NO. 971-1103 DATE: DRAWN BY:

Cone Penetration Test - 13

Test Date : 0-/0 /0-
Location : SEE FIGURE

Operator : Golder Associates

Ground Surf. Elev. : 0.00
Water Table Depth : 0.00



Qt normalized for
unequal end area effects

Su = (Qt - SigmaV) / Nk
Nk = 22
Gamma = 17 kN/m³

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

After Jefferies and Davies (1993)

PROJECT NO 971-1103 DATE: DRAWN BY:

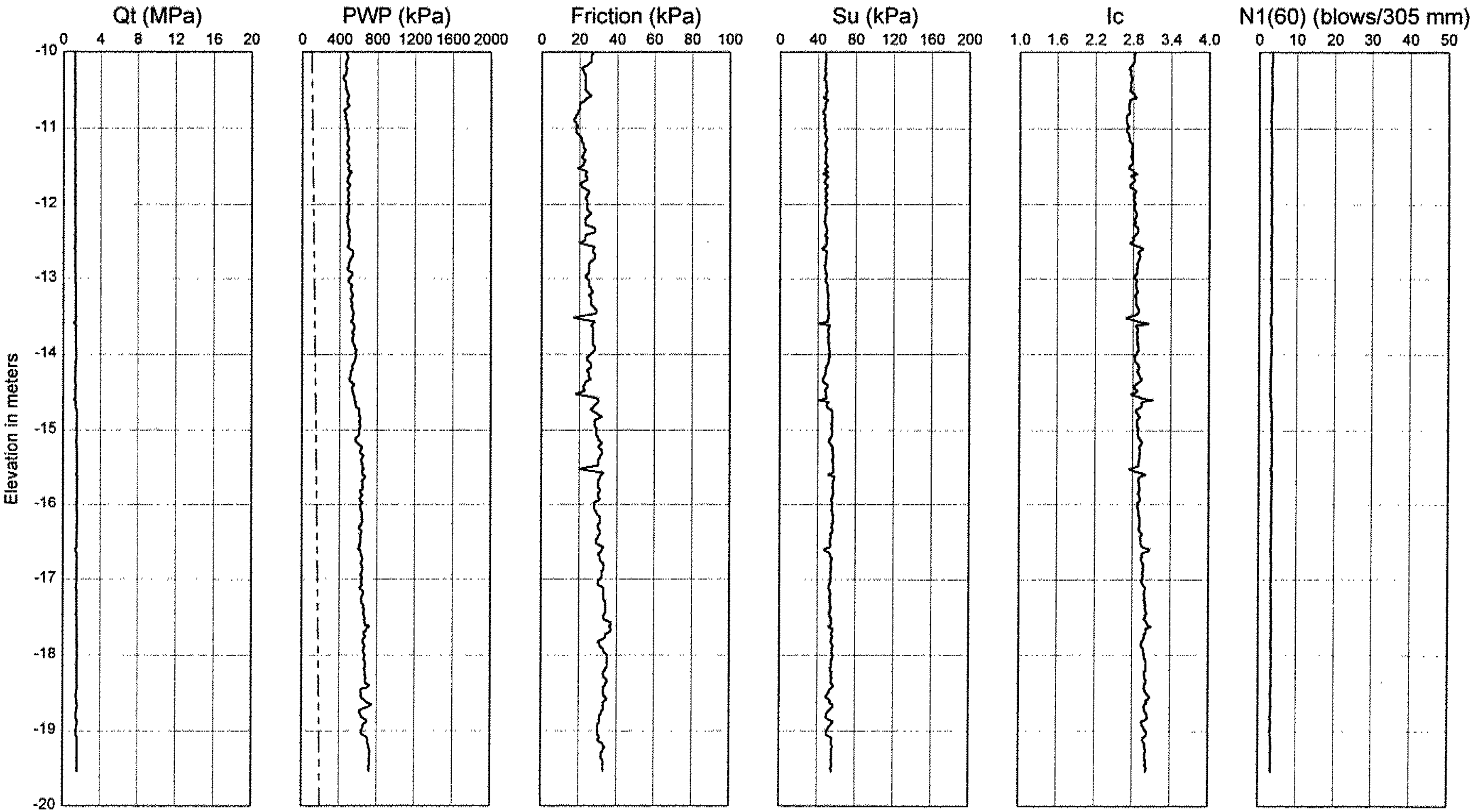
HWY 17

Cone Penetration Test - 13

Test Date : 0-/0 /0-
Location : SEE FIGURE

Operator : Golder Associates

Ground Surf. Elev. : 0.00
Water Table Depth : 0.00



Qt normalized for
unequal end area effects

Su = (Qt - SigmaV) / Nk
Nk = 22
Gamma = 17 kN/m³

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

After Jefferies and Davies (1993)

PROJECT NO. 971-1103 DATE: DRAWN BY:

HWY 17

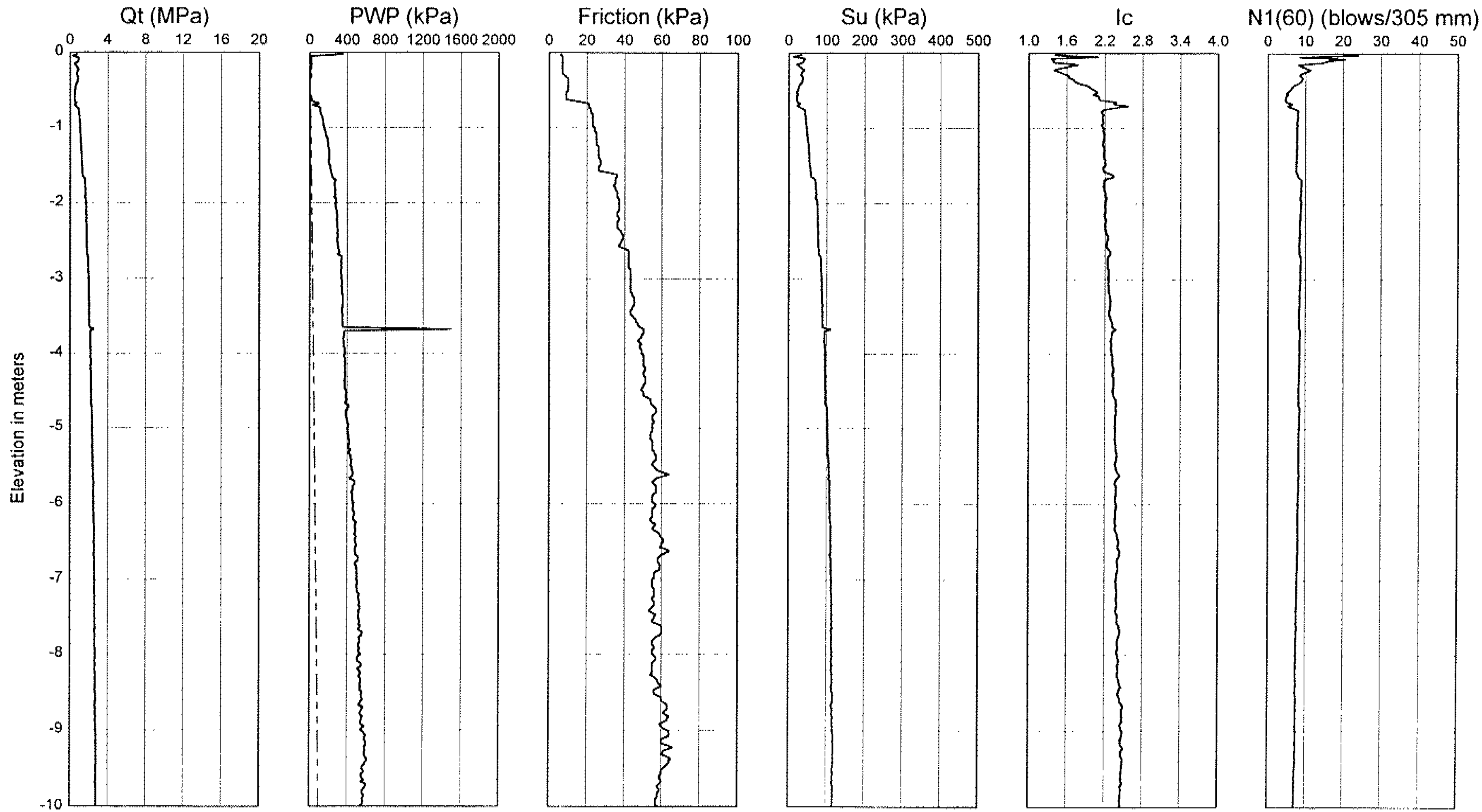
Golder Associates

Cone Penetration Test - 14

Test Date : 0-/0 /0-
Location : SEE FIGURE

Operator : Golder Associates

Ground Surf. Elev. : 0.00
Water Table Depth : 0.00



Qt normalized for
unequal end area effects

Su = (Qt - SigmaV) / Nk
Nk = 22
Gamma = 17 kN/m³

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

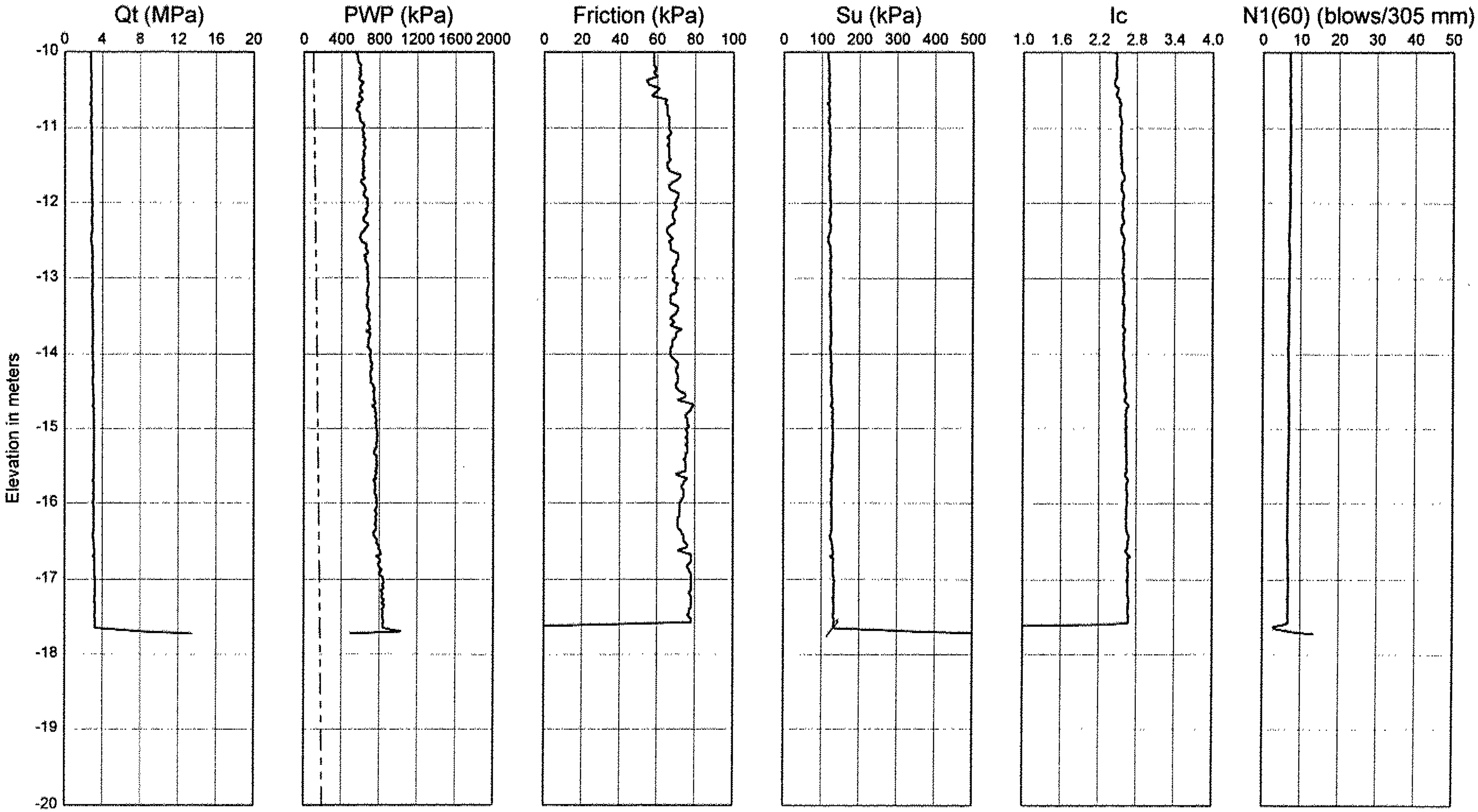
After Jefferies and Davies (1993)

Cone Penetration Test - 14

Test Date : 0-/0 /0-
Location : SEE FIGURE

Operator : Golder Associates

Ground Surf. Elev. : 0.00
Water Table Depth : 0.00



Qt normalized for
unequal end area effects

Su = (Qt - SigmaV) / Nk
Nk = 22
Gamma = 17 kN/m³

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

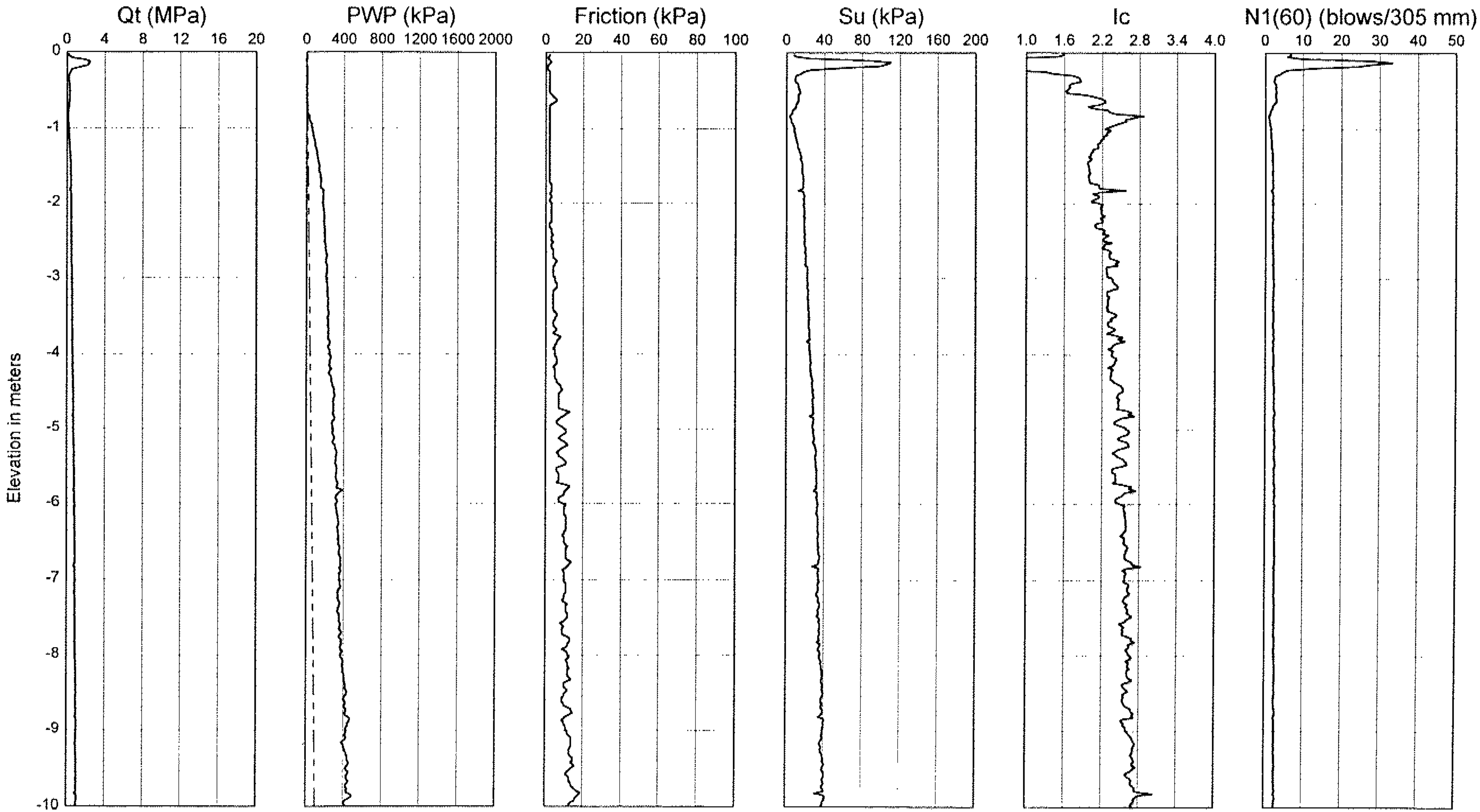
After Jefferies and Davies (1993)

Cone Penetration Test - 15

Test Date : 0-/0 /0-
Location : SEE FIGURE

Operator : Golder Associates

Ground Surf. Elev. : 0.00
Water Table Depth : 0.00



Qt normalized for
unequal end area effects

Su (Qt - SigmaV) / Nk
Nk 22
Gamma = 17 kN/m³

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

After Jefferies and Davies (1993)

PROJECT NO 971-1103 DATE. DRAWN BY:

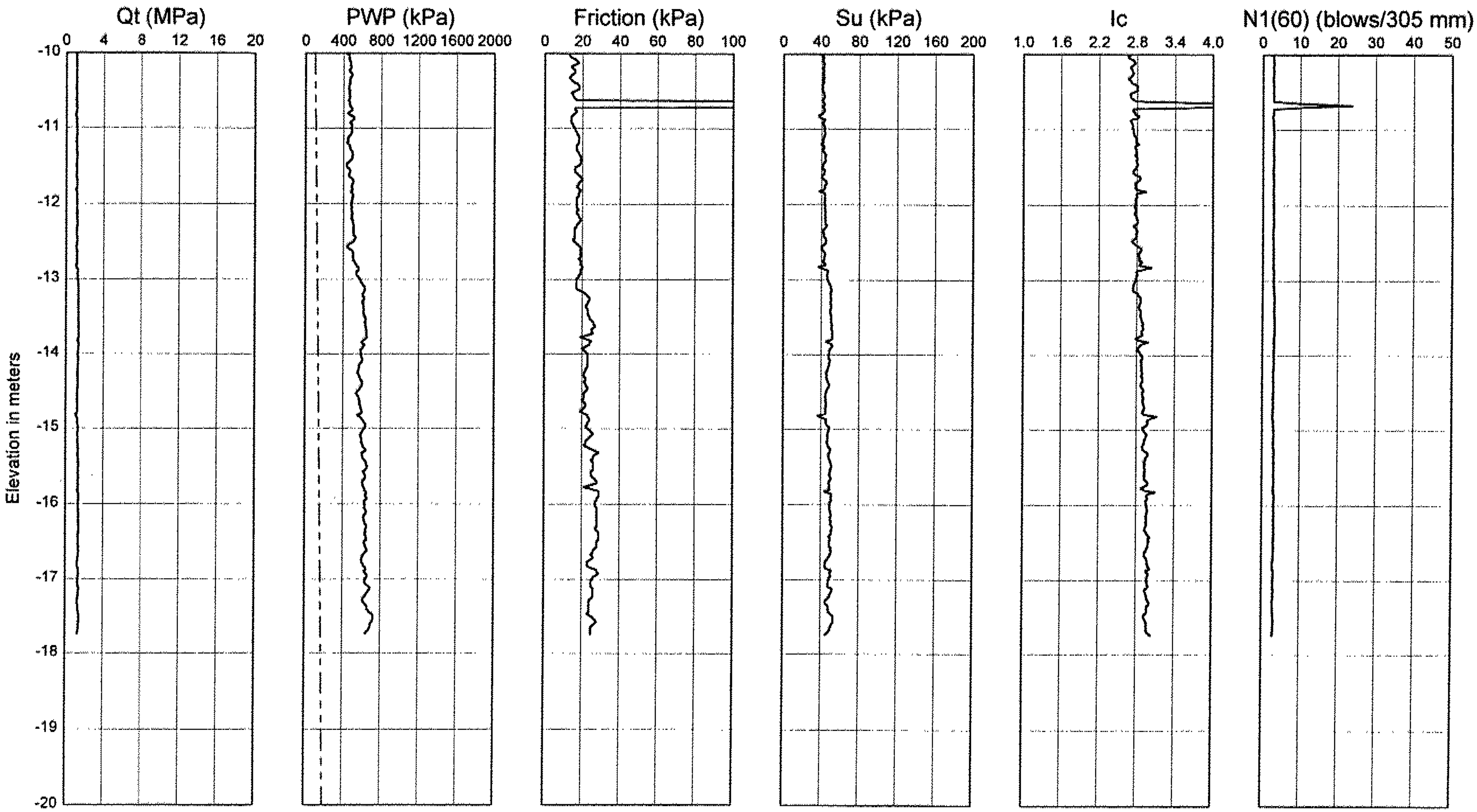
HWY 17

Cone Penetration Test - 15

Test Date : 0-/0 /0-
Location : SEE FIGURE

Operator : Golder Associates

Ground Surf. Elev. : 0.00
Water Table Depth : 0.00



Qt normalized for
unequal end area effects

Su = (Qt - SigmaV) / Nk
Nk = 22
Gamma = 17 kN/m³

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

After Jefferies and Davies (1993)

PROJECT NO. 971-1103 DATE: DRAWN BY:

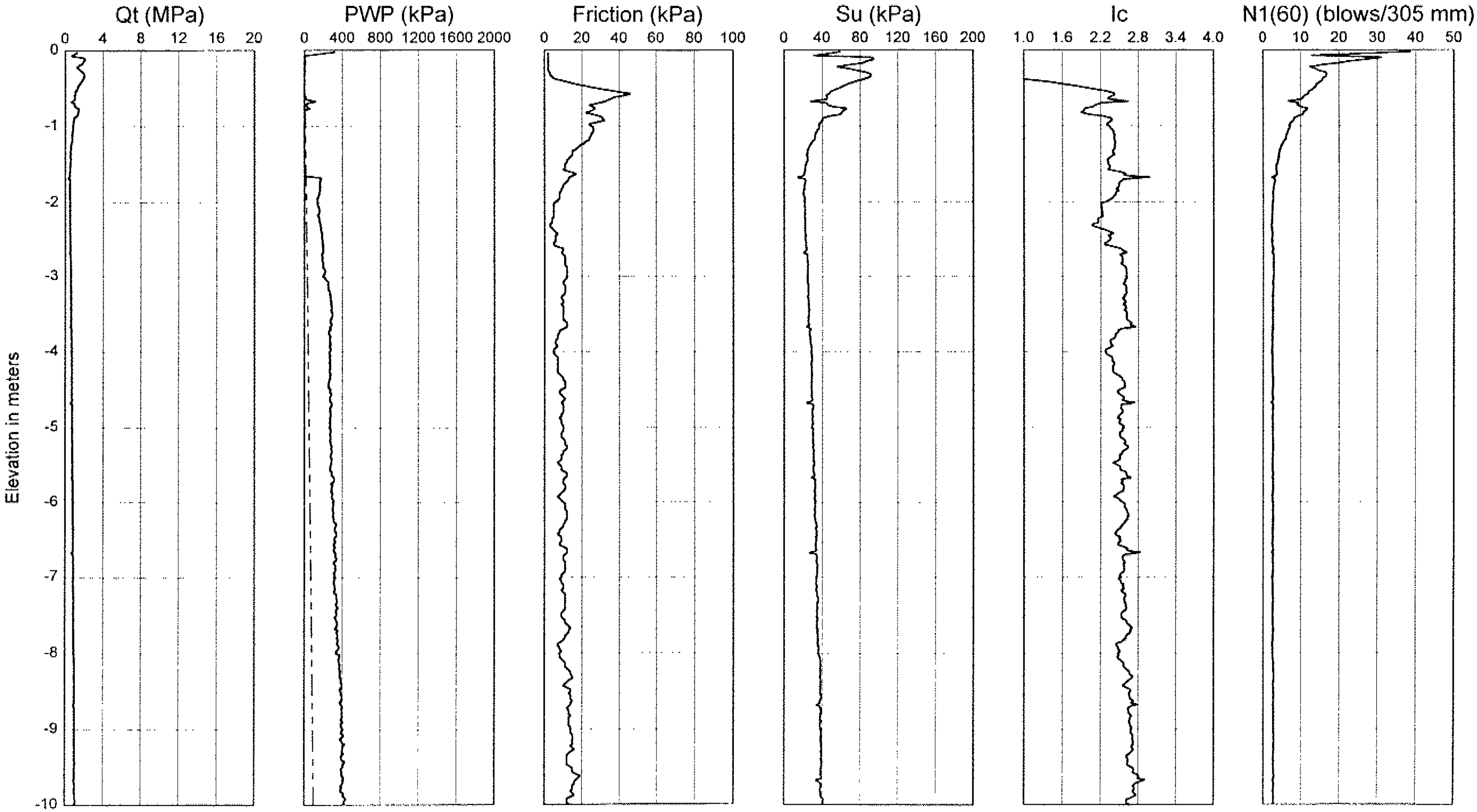
HWY 17

Cone Penetration Test - 16

Test Date : 0-/0 /0-
Location : SEE FIGURE

Operator : Golder Associates

Ground Surf. Elev. : 0.00
Water Table Depth : 0.00



Qt normalized for
unequal end area effects

Su = (Qt - SigmaV) / Nk
Nk = 22
Gamma = 17 kN/m³

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

After Jefferies and Davies (1993)

PROJECT NO 971-1103 DATE: DRAWN BY: HWY 17

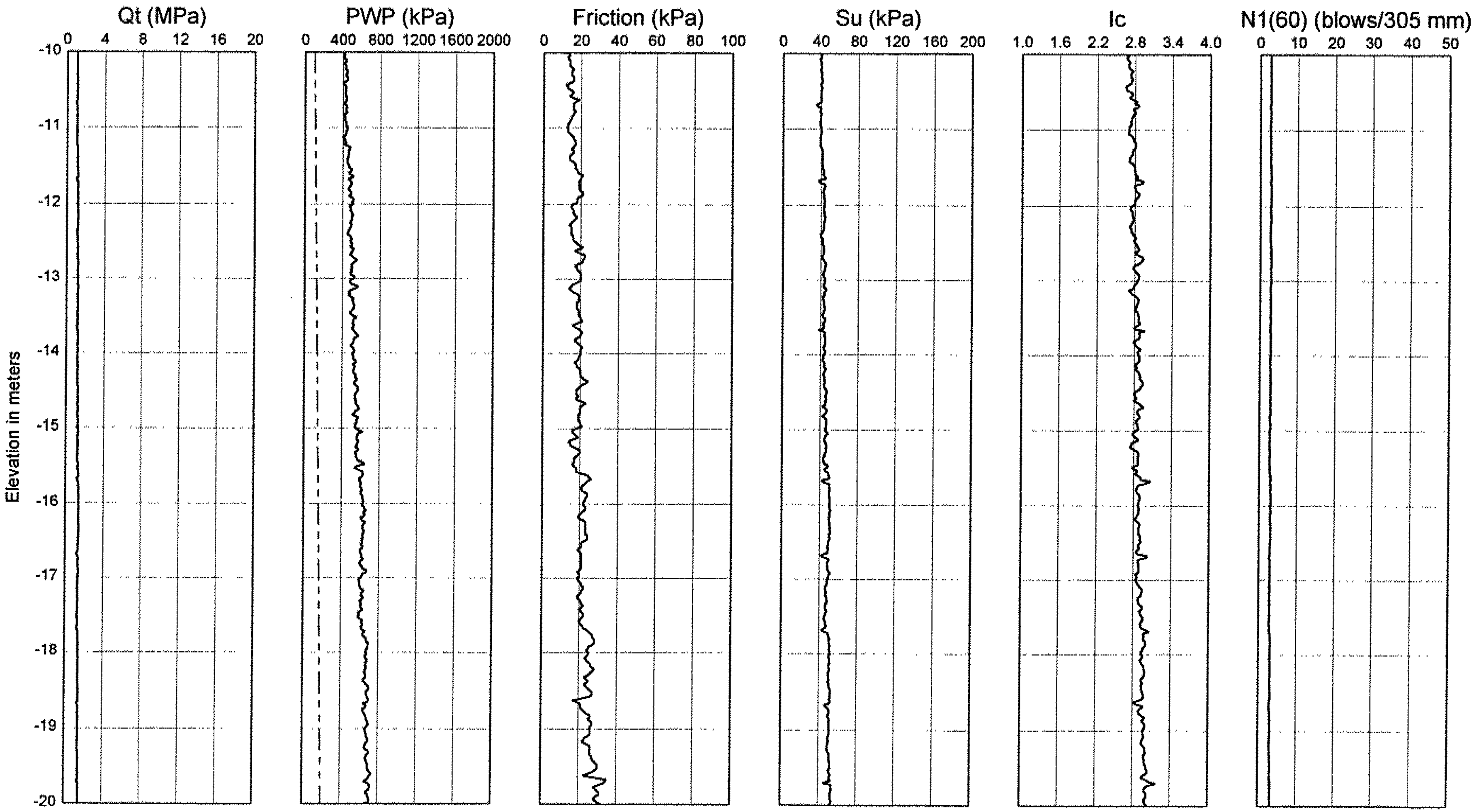
Golder Associates

Cone Penetration Test - 16

Test Date : 0-/0 /0-
Location : SEE FIGURE

Operator : Golder Associates

Ground Surf. Elev. : 0.00
Water Table Depth : 0.00



Qt normalized for unequal end area effects

Su = (Qt - SigmaV) / Nk
Nk = 22
Gamma = 17 kN/m³

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

After Jefferies and Davies (1993)

Golder Associates

PROJECT NO. 971-1103 DATE: DRAWN BY:

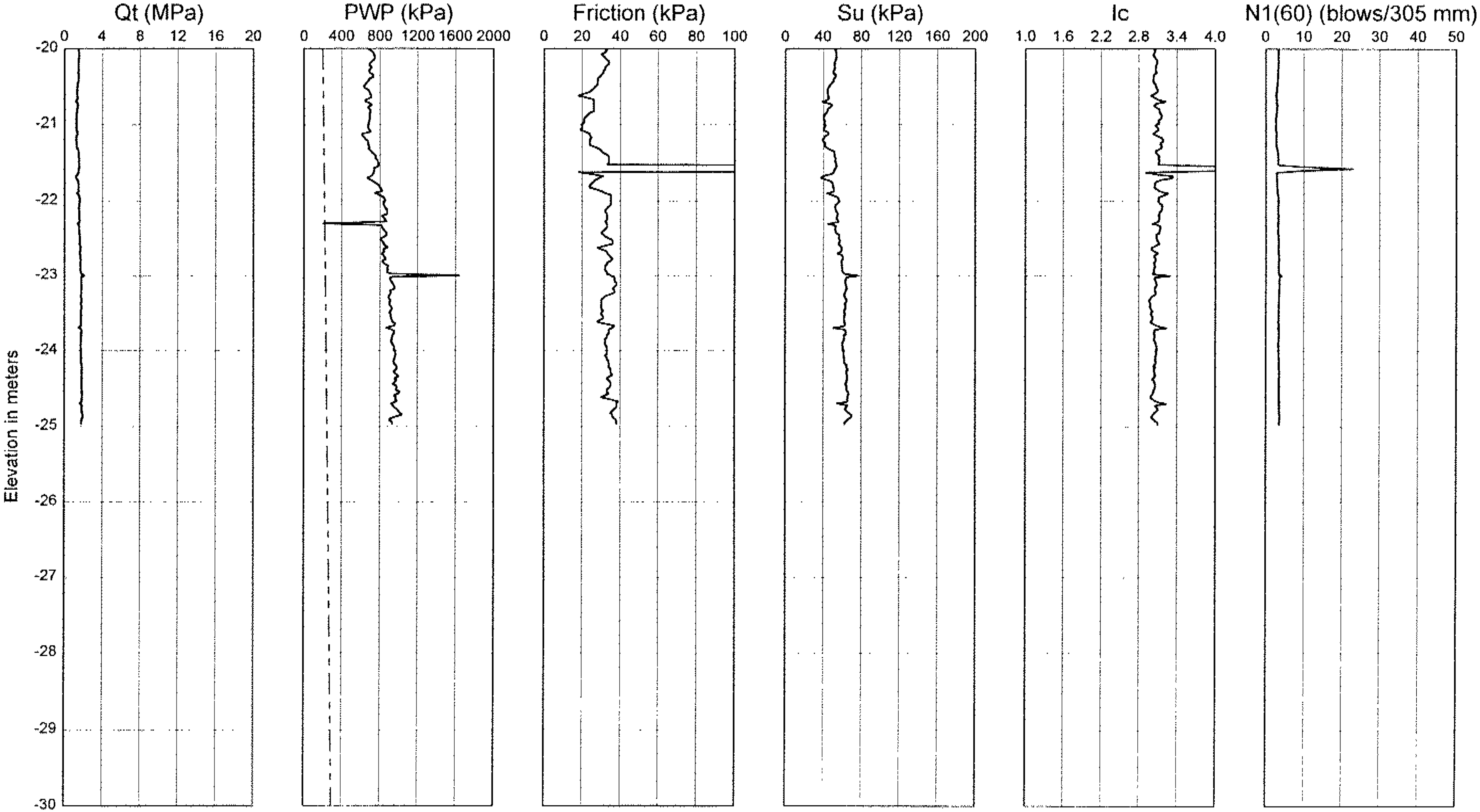
HWY 17

Cone Penetration Test - 16

Test Date : 0-/0 /0-
Location : SEE FIGURE

Operator : Golder Associates

Ground Surf. Elev. : 0.00
Water Table Depth : 0.00



Qt normalized for
unequal end area effects

Su = (Qt - SigmaV) / Nk
Nk = 22
Gamma = 17 kN/m³

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

After Jefferies and Davies (1993)

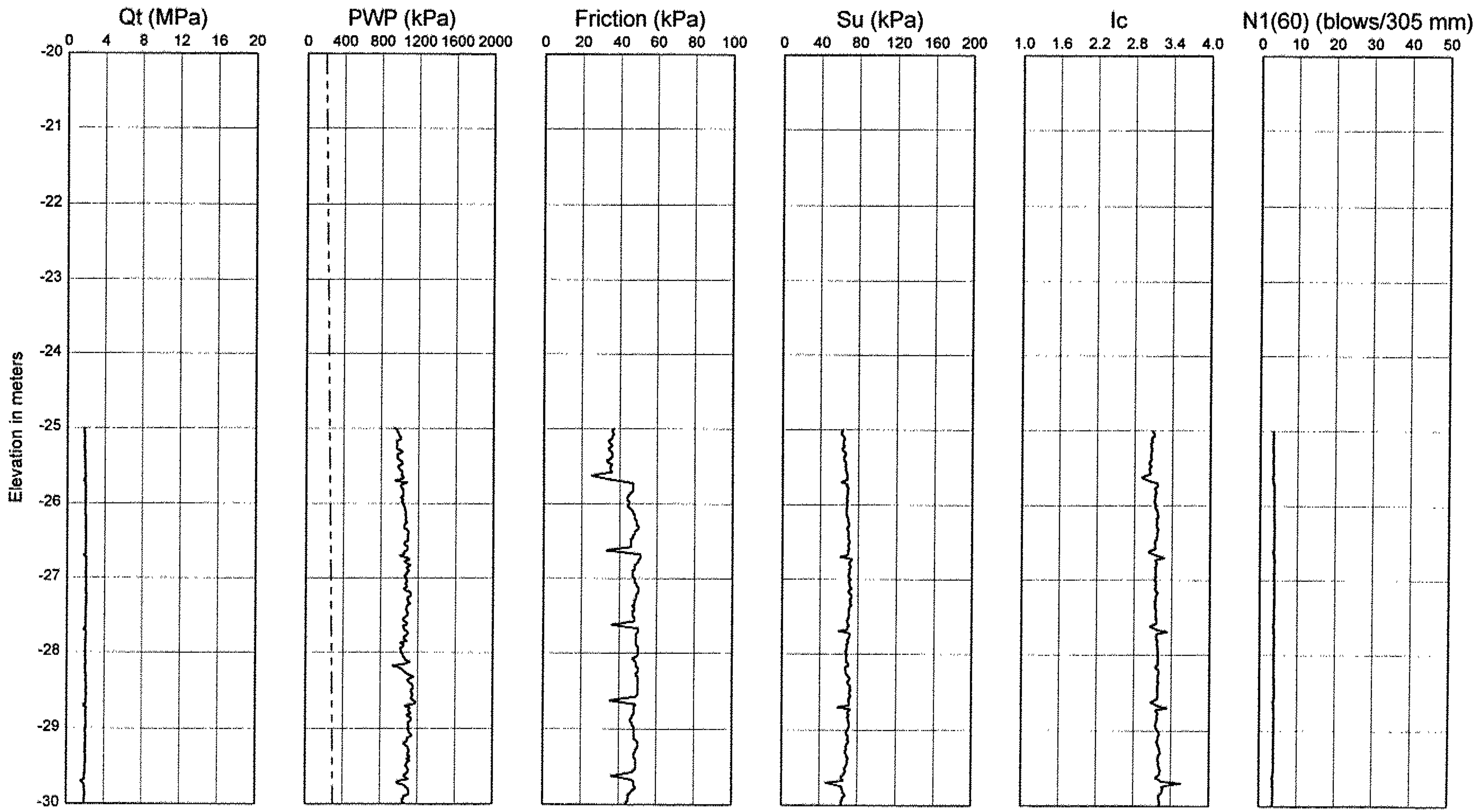
PROJECT NO. 971-1103 DATE: DRAWN BY:

Cone Penetration Test - 16

Test Date : 0-/0 /0-
Location : SEE FIGURE

Operator : Golder Associates

Ground Surf. Elev. : 0.00
Water Table Depth : 0.00



Qt normalized for
unequal end area effects

Su = (Qt - SigmaV) / Nk
Nk = 22
Gamma = 17 kN/m³

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

After Jefferies and Davies (1993)

PROJECT NO. 971-1103 DATE: DRAWN BY:

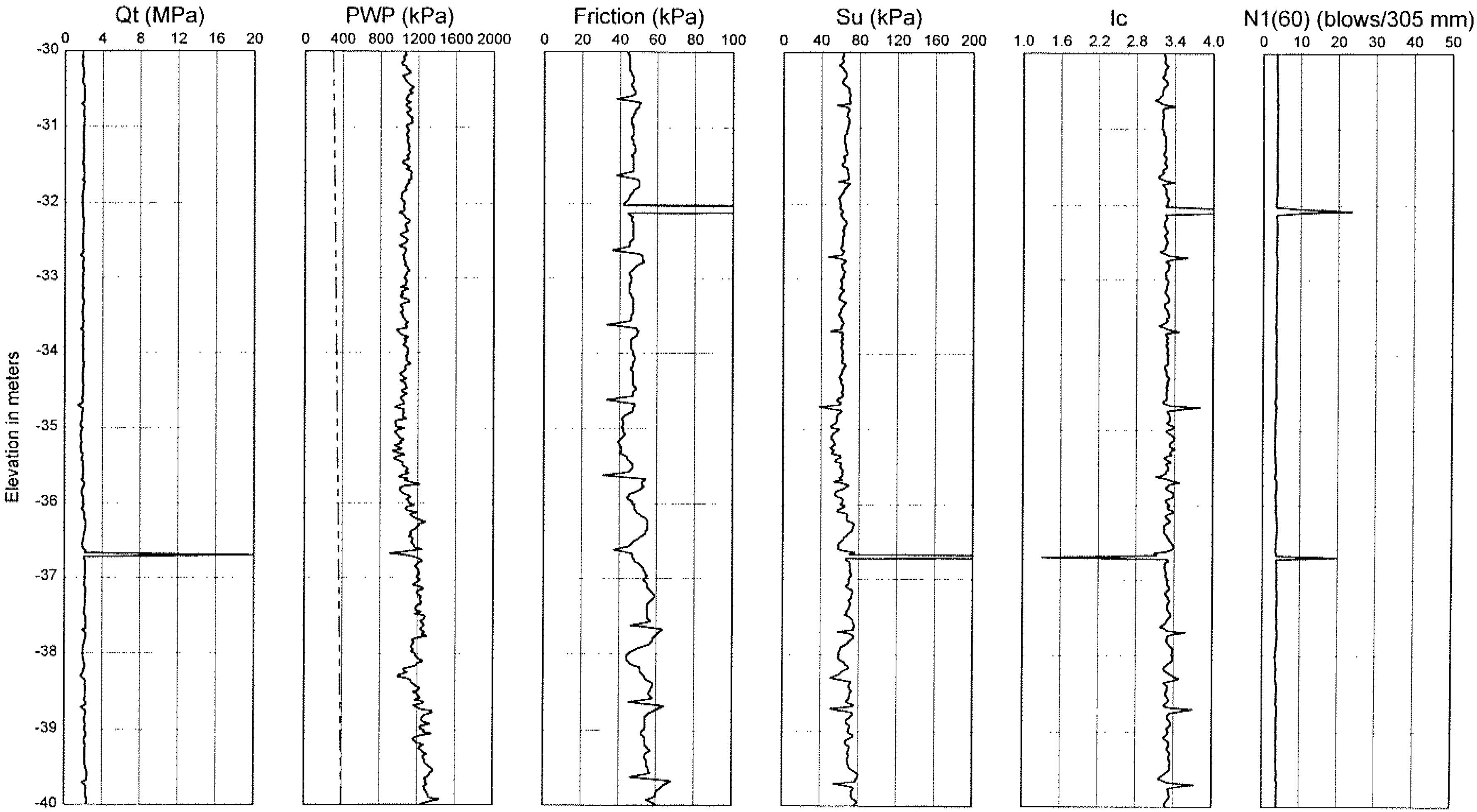
HWY 17

Cone Penetration Test - 16

Test Date : 0-/0 /0-
Location : SEE FIGURE

Operator : Golder Associates

Ground Surf. Elev. : 0.00
Water Table Depth : 0.00



Qt normalized for
unequal end area effects

Su = (Qt - SigmaV) / Nk
Nk = 22
Gamma = 17 kN/m³

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

After Jefferies and Davies (1993)

PROJECT NO. 971-1103 DATE: DRAWN BY:

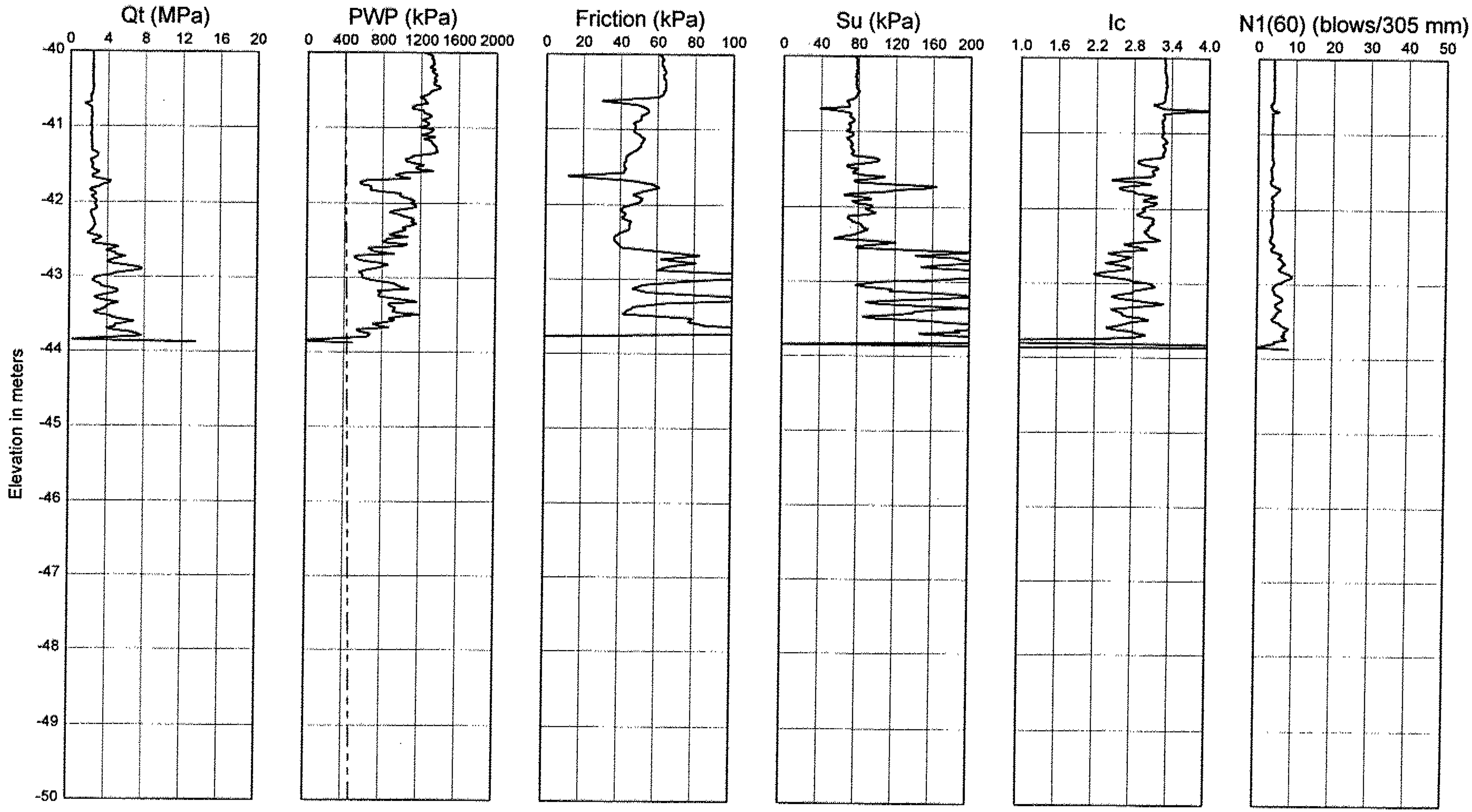
HWY 17

Cone Penetration Test - 16

Test Date : 0-/0 /0-
Location : SEE FIGURE

Operator : Golder Associates

Ground Surf. Elev. : 0.00
Water Table Depth : 0.00



Qt normalized for
unequal end area effects

Su = (Qt - SigmaV) / Nk
Nk = 22
Gamma = 17 kN/m³

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

After Jefferies and Davies (1993)

PROJECT NO. 971-1103 DATE: DRAWN BY:

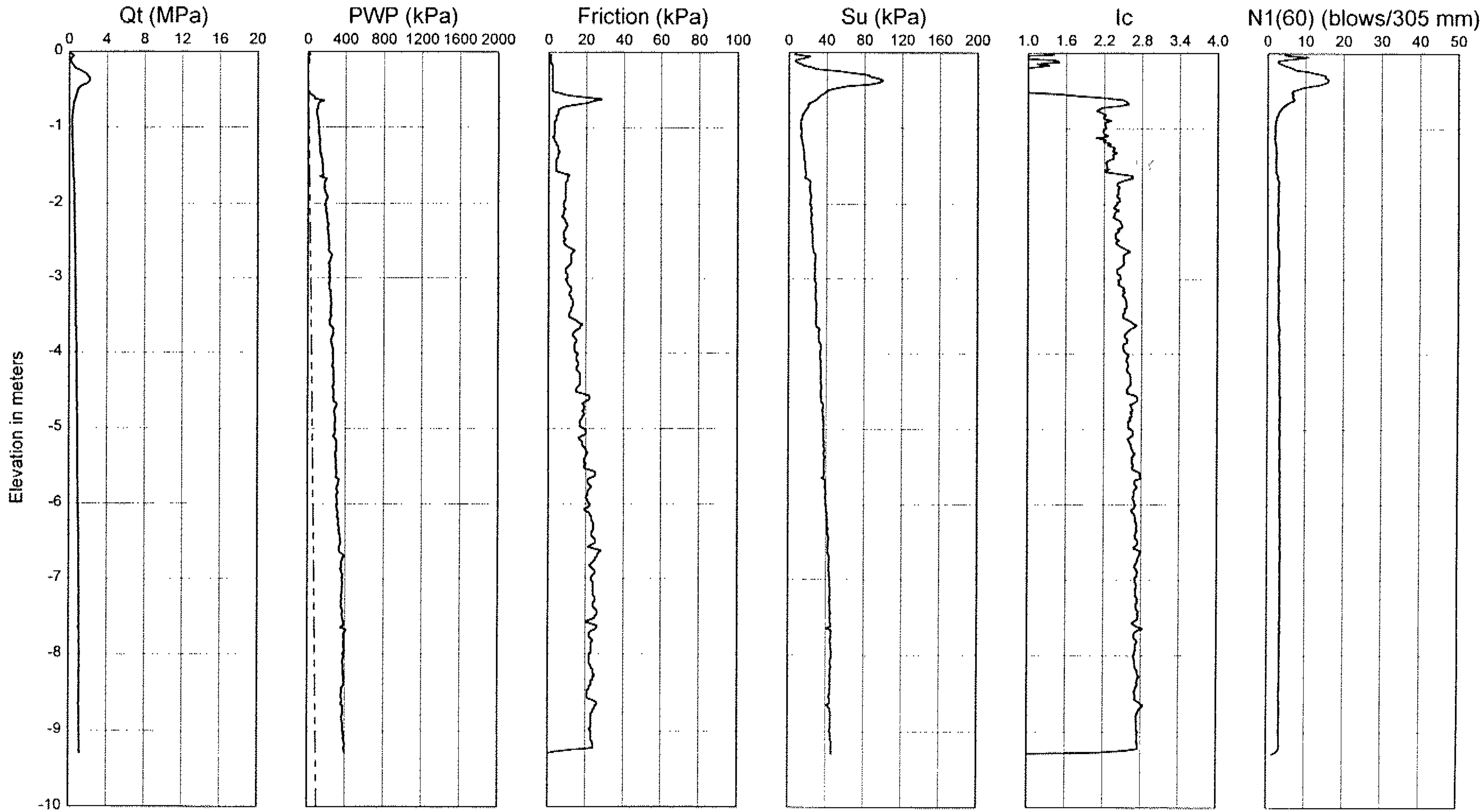
HWY 17

Cone Penetration Test - 17

Test Date : 0-/0 /0-
Location : SEE FIGURE

Operator : Golder Associates

Ground Surf. Elev. : 0.00
Water Table Depth : 0.00



Qt normalized for
unequal end area effects

Su = (Qt - SigmaV) / Nk
Nk = 22
Gamma = 17 kN/m³

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

After Jefferies and Davies (1993)

APPENDIX C

OEDOMETER TESTS RESULTS

CONSOLIDATION SUMMARY

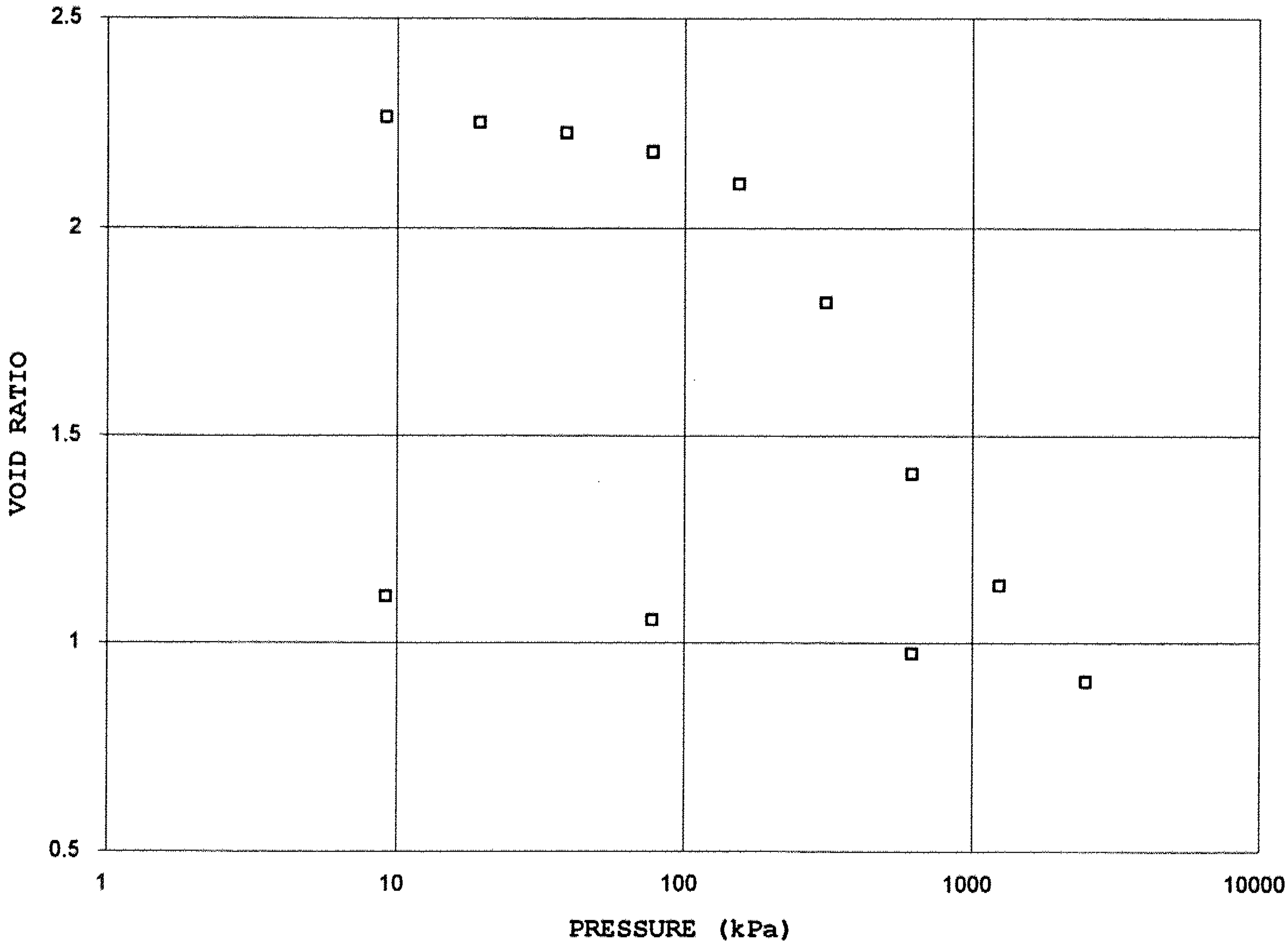
FIGURE C-1

PROJECT	971-1103	SPECIFIC GRAVITY	2.70 assumed	DATE STARTED	97-03-05
SAMPLE	BH 97-1/SA 14	DRY WEIGHT,gm	49.66	DATE COMPLETED	97-03-05
AREA(mm2)	3152.0	SOLIDS HT.2HS	5.835	SAMPLE DEPTH ,m	12.65

Load kPa	Corr. Height mm	Void Ratio	Average Height mm	t90 sec	cv. t90 cm2/s	k cm/S	mv m2/kN	Total Work kJ/m3
0.00	19.090	2.272	19.090					
9.15	19.042	2.263	19.066	181	4.26E-03	1.15E-07	2.75E-04	0.012
19.40	18.965	2.250	19.004	379	2.02E-03	7.79E-08	2.08E-04	0.069
38.79	18.816	2.225	18.891	427	1.77E-03	6.99E-08	2.01E-04	0.298
77.58	18.556	2.180	18.686	496	1.49E-03	5.14E-08	1.76E-04	1.102
155.16	18.121	2.105	18.339	649	1.10E-03	3.16E-08	1.47E-04	3.830
310.32	16.447	1.819	17.284	3937	1.61E-04	8.91E-09	2.83E-04	25.330
620.64	14.052	1.408	15.250	3342	1.48E-04	5.84E-09	2.02E-04	93.113
1241.28	12.473	1.138	13.263	2428	1.54E-04	2.01E-09	6.66E-05	197.723
2482.56	11.130	0.907	11.802	1240	2.38E-04	1.32E-09	2.83E-05	398.201
620.64	11.511	0.973	11.321					
77.58	11.988	1.054	11.750					
9.15	12.321	1.111	12.155					

Notes:
Test carried out with "quick" consolidation procedure.
Value t90 calculated from plot of stage ratio, t/d, versus elapsed time, minutes.

Water Content % ,initial	65.1	Liquid Limit %	73.2
Water Content % , final	45.5	Plastic Limit %	29.7
		Plastic Index %	43.5
Original Volume,cc	60.17	Liquidity Index	0.813
Volume of Solids,cc	18.39		
Volume of Voids,cc	41.78	Unit Weight,kN/m3	15.64
Degree of Saturation %	77.4	Dry Unit Weight,kN/m3	9.47



VOID RATIO VS. LOG PRESSURE
BOREHOLE 97-1 SAMPLE 14

FIGURE C-2

Golder Associates

CONSOLIDATION SUMMARY

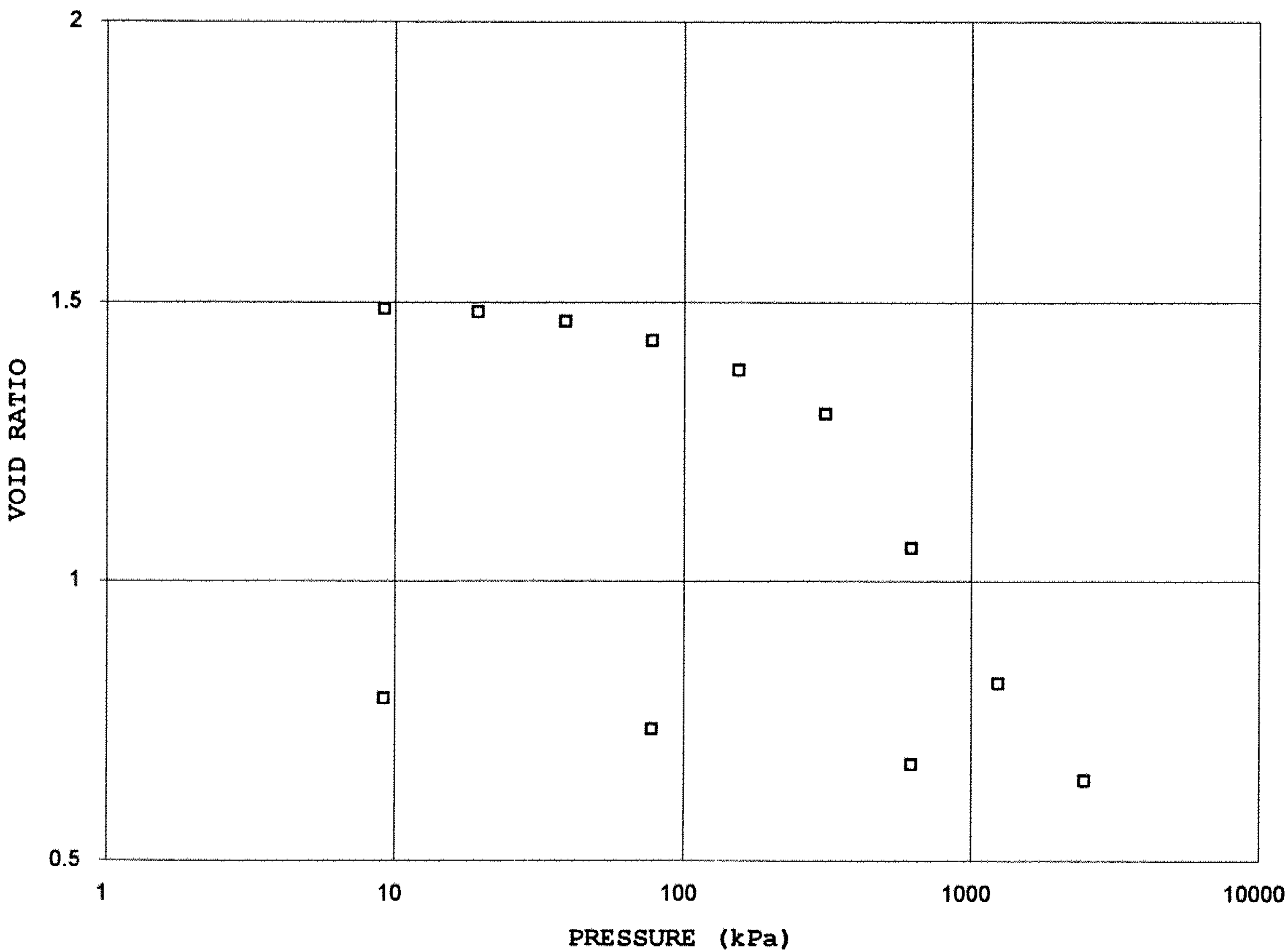
FIGURE C-3

PROJECT	971-1103	SPECIFIC GRAVITY	2.70 assumed	DATE STARTED	97-03-05
SAMPLE	BH 97-1/SA 27	DRY WEIGHT,gm	65.3	DATE COMPLETED	97-03-05
AREA(mm2)	3152.0	SOLIDS HT.2HS	7.673	SAMPLE DEPTH ,m	27.89

Load kPa	Corr. Height mm	Void Ratio	Average Height mm	t90 sec	cv. t90 cm2/s	k cm/S	mv m2/kN	Total Work kJ/m3
0.00	19.100	1.489	19.100					
9.15	19.091	1.488	19.096					0.002
19.40	19.046	1.482	19.069	904	8.53E-04	1.92E-08	1.21E-04	0.036
38.79	18.918	1.466	18.982	1560	4.90E-04	1.66E-08	1.73E-04	0.231
77.58	18.656	1.431	18.787	1220	6.13E-04	2.13E-08	1.77E-04	1.037
155.16	18.254	1.379	18.455	1122	6.44E-04	1.71E-08	1.36E-04	3.545
310.32	17.649	1.300	17.952	807	8.47E-04	1.69E-08	1.02E-04	11.259
620.64	15.799	1.059	16.724	3000	1.98E-04	6.05E-09	1.56E-04	60.051
1241.28	13.943	0.817	14.871	1148	4.08E-04	6.27E-09	7.83E-05	169.416
2482.56	12.597	0.642	13.270	1440	2.59E-04	1.44E-09	2.84E-05	349.158
620.64	12.823	0.671	12.710					
77.58	13.303	0.734	13.063					
9.15	13.729	0.789	13.516					

Notes:
Test carried out with "quick" consolidation procedure.
Value t90 calculated from plot of stage ratio, t/d, versus elapsed time, minutes.

Water Content % ,initial	53.4	Liquid Limit %	64.7
Water Content % , final	41.3	Plastic Limit %	28.3
		Plastic Index %	36.4
Original Volume,cc	60.17	Liquidity Index	0.690
Volume of Solids,cc	24.19		
Volume of Voids,cc	35.99	Unit Weight,kN/m3	17.06
Degree of Saturation %	96.8	Dry Unit Weight,kN/m3	11.12



VOID RATIO VS. LOG PRESSURE
BOREHOLE 97-1 SAMPLE 1

FIGURE C-4

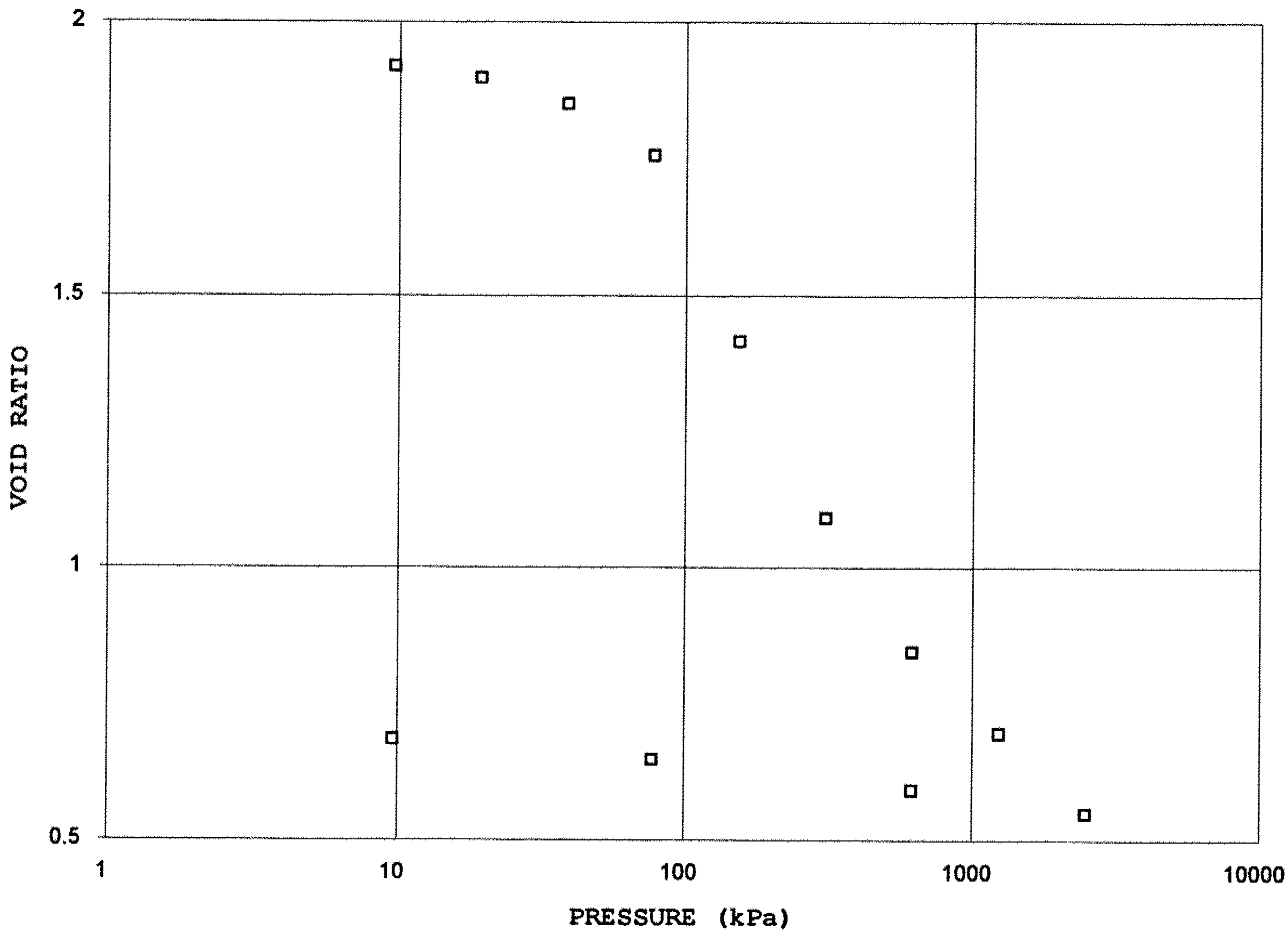
CONSOLIDATION SUMMARY	FIGURE C-5

PROJECT	971-1103	SPECIFIC GRAVITY	2.70 assumed	DATE STARTED	97-03-07
SAMPLE	BH 97-12/SA 6	DRY WEIGHT,gm	55.48	DATE COMPLETED	97-03-07
AREA(mm2)	3152.0	SOLIDS HT.2HS	6.519	SAMPLE DEPTH ,m	6.55

Load kPa	Corr. Height mm	Void Ratio	Average Height mm	t90 sec	cv. t90 cm2/s	k cm/S	mv m2/kN	Total Work kJ/m3
0.00	19.090	1.928	19.090					
9.70	19.027	1.919	19.059	1389	5.54E-04	1.85E-08	3.40E-04	0.016
19.40	18.881	1.896	18.954	1130	6.74E-04	5.21E-08	3.94E-04	0.128
38.79	18.571	1.849	18.726	1366	5.44E-04	4.47E-08	4.19E-04	0.605
77.58	17.955	1.754	18.263	1180	5.99E-04	4.89E-08	4.16E-04	2.535
155.16	15.741	1.415	16.848	1930	3.12E-04	4.57E-08	7.47E-04	16.885
310.32	13.629	1.091	14.685	2334	1.96E-04	1.37E-08	3.57E-04	48.112
620.64	12.023	0.844	12.826	1006	3.47E-04	9.21E-09	1.36E-04	102.963
1241.28	11.055	0.696	11.539	916	3.08E-04	2.47E-09	4.09E-05	177.916
2482.56	10.096	0.549	10.576	574	4.13E-04	1.64E-09	2.02E-05	339.434
620.64	10.379	0.592	10.238					
77.58	10.737	0.647	10.558					
9.70	10.981	0.684	10.859					

Notes:
Test carried out with "quick" consolidation procedure.
Value t90 calculated from plot of stage ratio, t/d, versus elapsed time, minutes.

Water Content % ,initial	72.8	Liquid Limit %	70.5
Water Content % , final	33.5	Plastic Limit %	30.2
		Plastic Index %	40.3
Original Volume,cc	60.17	Liquidity Index	1.057
Volume of Solids,cc	20.55		
Volume of Voids,cc	39.62	Unit Weight,kN/m3	15.93
Degree of Saturation %	101.9	Dry Unit Weight,kN/m3	9.22



VOID RATIO VS. LOG PRESSURE
BOREHOLE 97-12 SAMPLE 6

FIGURE C-6

CONSOLIDATION SUMMARY

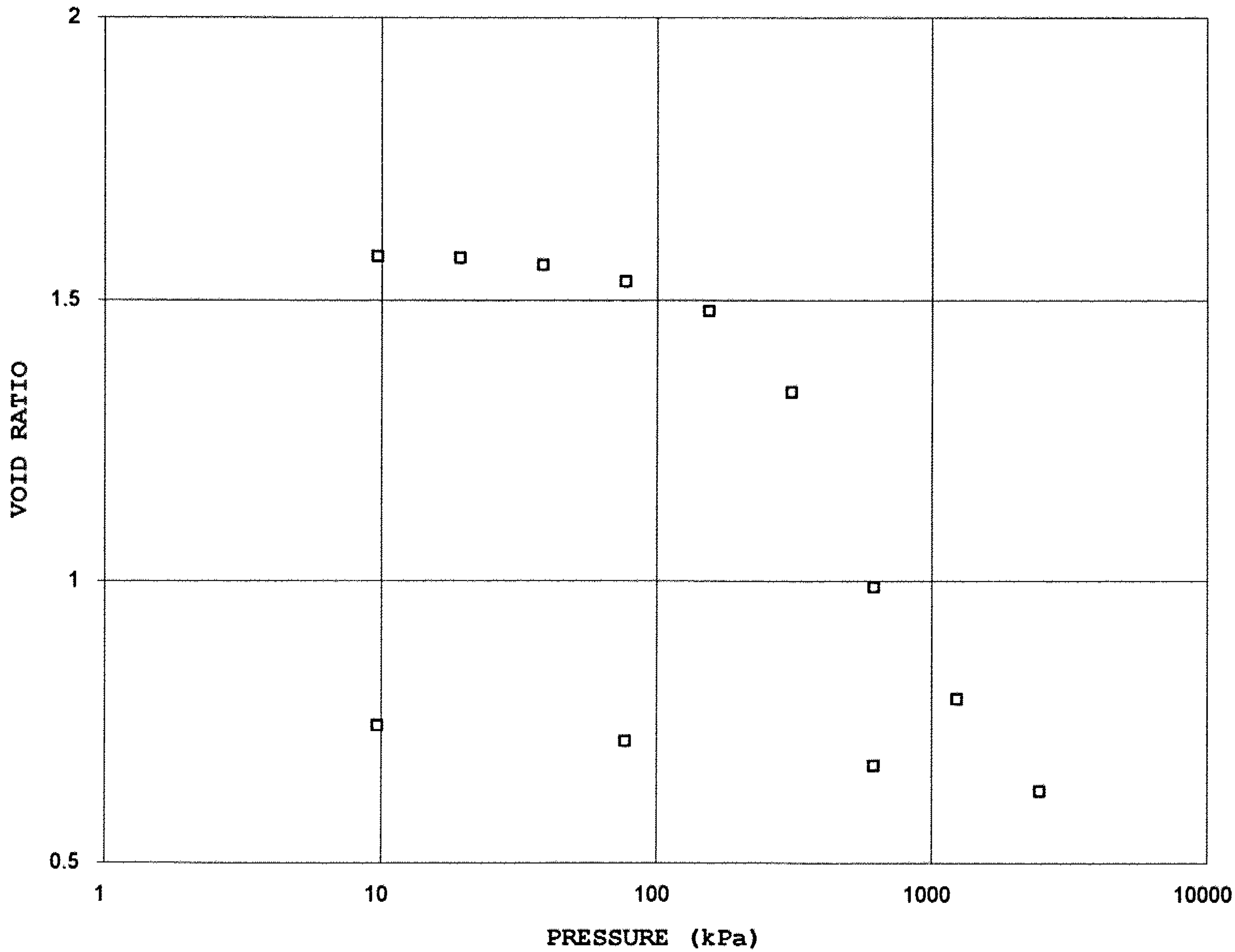
FIGURE C-7

PROJECT	971-1103	SPECIFIC GRAVITY	2.70 assumed	DATE STARTED	97-03-07
SAMPLE	BH 97-12/SA 27	DRY WEIGHT,gm	63.04	DATE COMPLETED	97-03-07
AREA(mm2)	3158.5	SOLIDS HT.2HS	7.392	SAMPLE DEPTH ,m	33.99

Load kPa	Corr. Height mm	Void Ratio	Average Height mm	t90 sec	cv. t90 cm2/s	k cm/S	mv m2/kN	Total Work kJ/m3
0.00	19.068	1.579	19.068					
9.68	19.057	1.578	19.063	26	2.96E-02	1.73E-07	5.96E-05	0.003
19.36	19.039	1.576	19.048	36	2.14E-02	2.04E-07	4.88E-05	0.017
38.71	18.943	1.563	18.991	289	2.65E-03	6.75E-08	1.30E-04	0.163
77.42	18.726	1.533	18.835	428	1.76E-03	5.06E-08	1.47E-04	0.828
154.85	18.336	1.480	18.531	426	1.71E-03	4.42E-08	1.32E-04	3.247
309.69	17.275	1.337	17.806	1605	4.19E-04	1.47E-08	1.80E-04	16.687
619.39	14.707	0.990	15.991	1431	3.79E-04	1.61E-08	2.17E-04	85.743
1238.78	13.242	0.791	13.975	812	5.10E-04	6.20E-09	6.20E-05	178.291
2477.55	12.023	0.626	12.633	476	7.11E-04	3.59E-09	2.58E-05	349.346
619.39	12.363	0.672	12.193					
77.42	12.682	0.716	12.523					
9.68	12.890	0.744	12.786					

Notes:
Test carried out with "quick" consolidation procedure.
Value t90 calculated from plot of stage ratio, t/d, versus elapsed time, minutes.

Water Content % ,initial	57.4	Liquid Limit %	64.9
Water Content % , final	33.5	Plastic Limit %	28.0
		Plastic Index %	36.9
Original Volume,cc	60.23	Liquidity Index	0.797
Volume of Solids,cc	23.35		
Volume of Voids,cc	36.88	Unit Weight,kN/m3	16.36
Degree of Saturation %	98.1	Dry Unit Weight,kN/m3	10.39



VOID RATIO VS. LOG PRESSURE
BOREHOLE 97-12 SAMPLE 27

FIGURE C-8

Golder Associates

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Golder Associates

REPORT ON

GEOTECHNICAL INVESTIGATION
PROPOSED HIGHWAY 17 (NEW)
BAR RIVER ROAD TO BLACK ROAD
SAULT STE. MARIE, ONTARIO
YOUR FILE W.O. 2872-94

Submitted to:

McCormick Rankin & Associates Ltd.
2655 North Sheridan Way
Mississauga, Ontario
L5K 2P8

4 copies - McCormick Rankin & Associates Ltd.
Mississauga, Ontario

2 copies - Golder Associates Ltd.
Mississauga, Ontario

June 1996

941-1364

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Lists of Abbreviations and Symbols
List of Abbreviations used in Wetland Summary Tables - Figures 3B to 3E

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Figure 1A	Site Location Plan
Figure 1B	Horizontal Alignment
Figure 2A	Borehole Location Plan, Black Creek Crossing
Figure 2B	Borehole Location Plan, Root River Bridge
Figure 2C	Borehole Location Plan, Belleau / Beaver Creek Bridge
Figure 2D	Borehole Location Plan, Jardin Mines Road Bridge
Figure 2E	Borehole Location Plan, Garden River Bridge
Figure 2F	Borehole Location Plan, Noonday River Bridge
Figure 2G	Borehole Location Plan, Echo River Bridge
Figure 2H	Borehole Location Plan, CPR Crossing
Figure 3A	Borehole Location Plan, Black Road
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Figure 3E	Borehole Location Plan, Highway 638
Figures 4 to 16	Grain Size Distribution
Figures 17 and 18	Consolidation Test Results

APPENDIX I	Record of Borehole Logs	- Ministry of Transportation
	Borehole 3	- Report W.P. 903-72-18
	Boreholes 3 and 4	- Report W.P. 903-72-01 Part 2
	Boreholes 9, 12 through 18	- Report W.P. 903-72-01 Part 1

1.0 INTRODUCTION

Golder Associates Ltd. has been retained by McCormick Rankin & Associates Ltd. to carry out a geotechnical investigation for the preliminary design of the bridges and associated roadworks for the planned realignment of Highway 17, hereafter referred to as Highway 17 (New), between Black Road in the City of Sault Ste. Marie and Bar River Road to the south-east (see Figure 1A). The purpose of this investigation is to determine the subsurface conditions at selected widely spaced locations along the proposed highway alignment and, based on our interpretation of the data obtained, to provide recommendations on the geotechnical aspects of preliminary design of the proposed works. Specific emphasis was placed on identifying the subsurface conditions at each of the proposed bridge structure locations, in areas of significant proposed embankment height and where the alignment crosses areas of wetland.

The terms of reference for the initial scope of work are outlined in our proposal letter P41-1207, dated May 30, 1994. The scope was subsequently expanded to include additional bridge locations, areas of wetlands and embankments. During the course of the field work, the number of boreholes and extent of testing was revised slightly to accommodate the subsoil and site conditions as encountered.

2.0 SITE AND PROJECT DESCRIPTION

The roadworks considered in this report are associated with the realignment of Highway 17 between Sault Ste. Marie and Bar River Road, a distance of approximately 27 km. The proposed Highway 17 (New) is to run between approximately 0.5 km to 3 km north and east of the existing Highway 17, crossing through the Garden River First Nation Reserve and eventually the Batchewana First Nation Reserve. The current Highway 17 extending east from Sault Ste. Marie is a two lane undivided highway.

It is our understanding that the proposed works include:

- five river and creek crossings (Root, Garden and Echo Rivers and Belleau/Beaver and Black Creek),
- two highway overpasses (Jardin Mines and Noonday Roads),
- one railway crossing (Canadian Pacific Railway at Maple Leaf Road),
- wetland/swamp crossings
- embankments

In general, cuts and embankments are less than about 2 m. In the area of the Garden River crossing, cuts of up to 8 m are proposed. To the south of Echo River, embankments of up to 5 m in height are proposed. At the CPR tracks and in the vicinity, the proposed road grade requires embankments up to 9 m in height.

The proposed horizontal alignment for Highway 17 (New) was originally provided to us on 1:5000 plan drawings prepared by McCormick Rankin Ltd., dated January 31, 1995. Subsequent to alignment revisions and finalizing of the vertical alignment, plan and profile drawings for specific sections of the study were provided in September 1995. The comments and recommendations in this report refer to the alignment as shown on the profiles and 1:2000 plans prepared by McCormick Rankin and received on November 24, 1995.

2.1 Background Information

A number of geotechnical investigations were carried out in the 1970's as part of the preliminary alignment studies for Highway 17 (New). These investigations were carried out at potential high embankment and bridge locations and within muskeg areas. In addition, there are a number of gravel pit studies which have been completed for the existing pits in the general vicinity of the proposed highway alignment. The above information is provided in the following table:

BACKGROUND INFORMATION SUMMARY

LOCATION	M.T.O. REPORT DESIGNATION	DATE	INFORMATION	COMMENTS
Sault Ste. Marie East to Jordan Mines Road (including proposed Root River Bridge)	WP 903-72-01 and WP 903-72-16	June 22, 1978	General site description and recommendations	No boreholes or test pits
Existing Hwy. 17 Root River Bridge	WP 279-85-01	May 5, 1986	7 Boreholes	
G.R.F.N. Pit # 53 (NW of Garden River Crossing)	Gravel Deposits Survey (i) Ballast Pit Survey Plan (WP 903-72-01)	Feb. 20, 1980 Dec. 6, 1991	Miscellaneous Shallow test pits and Shallow test holes	Formerly designated CPR Gravel Pit
Garden River Bridge	WP 903-72-18	Nov. 12, 1975	5 Boreholes	Proposed alignments Schemes A & B
G.R.F.N. Pit # 97 (NE of Garden River Crossing)	Ballast Pit Survey Plan (WP 903-72-01)	Dec. 19, 1991	Shallow test holes	
Talus Deposit (NE of Trap Rock Road Crossing)	Gravel Deposit Survey (i)	Feb. 20, 1980	Observations	
G.R.F.N. Pit #65 (NW of Lower Echo River Crossing)	Gravel Deposit Survey (i) Ballast Pit Survey Plan (WP 903-72-01)	Feb. 20, 1980 Dec. 17, 1991	Miscellaneous testing Shallow test holes	Formerly designated The Maple Ridge Pit
2 Muskeg Areas in vicinity of Echo River	WP 903-72-00	Sept. 8, 1977	Probes	Probes located on lines B & D Exact locations not known
Lower Echo River Bridge	WP 903-72-01 Part II	March 18, 1975	2 boreholes	Lines A, B, C and D
Existing Hwy. 17 Echo Bay Bridge	WP 185-79-01	Dec. 1989	6 boreholes	
Embankment at Sta. 150 South of Echo River and North of Hwy. 638	WP 903-72-01 Part IV	March 25, 1975	2 boreholes	
Hwy. 638 in vicinity of crossing	WP 283-85-01	Sept. 27, 1987	7 shallow boreholes	
CPR Tracks	WP 903-72-01 Part I Addendum	Mar 5, 1975 May 7, 1975	13 boreholes	Determination of crossing alignment for lines A, B, C, D, 4 and 5

Notes: (i) Report entitled "A Survey of the Known Gravel and Talus "Trap Rock" Deposits of the Garden River Indian Reserve No. 14, District of Algoma," by C.J. Kuryliw, P.Eng., to Dept. of Indian and Northern Affairs, dated February 15, 1980.

3.0 INVESTIGATION PROCEDURES

The field work for this investigation was carried out between September 20 and October 6, 1995. At this time, eleven boreholes and five probeholes were put down at each of the six proposed bridge sites and along the road alignment using a bombardier mounted CME 55 drill rig supplied and operated by Marathon Drilling Inc. of Sudbury. In addition, 19 hand augered probe holes were put down to depths up to 1.5 m in areas of swamp and wetlands.

In each boring, samples were obtained at regular intervals of depth using 50 mm outside diameter split spoon samplers, in accordance with Standard Penetration Test (SPT) procedures. At selected locations, relatively undisturbed samples of silty clay were obtained using 75 mm outside diameter thin-walled Shelby tubes. Where soft cohesive deposits were encountered, in-situ field vane testing was carried out to measure undrained shear strength of the deposit. Groundwater conditions in the open boreholes were observed throughout the drilling operations, and 12.5 mm diameter plastic piezometers were installed at the base of each borehole to permit monitoring of the groundwater levels. The water levels in the piezometers were measured on October 5 and 6, 1995.

The field work was supervised on a full time basis by a member of our engineering staff who located the boreholes in the field, directed the drilling, sampling and in-situ testing operations, and logged the boreholes. The soil samples were identified in the field, placed in airtight containers and transported back to our laboratory in Mississauga for further examination. Index and classification tests were carried out on selected samples. Oedometer testing was carried out on two of the Shelby tube samples to determine the stress history, strength and compression characteristics of the silty clay deposit.

The borehole locations were referenced in the field by members of our staff with respect to prominent topographical features and the chainage stakes which had been established in the field by McCormick Rankin. The highway alignment chainage referred to throughout this report is shown on Figure 1B and incorporates a chainage of 10+000 at all juristictional boundaries. The ground surface elevations at the borehole locations were provided by McCormick Rankin and are inferred from the alignment profile which is referenced to the Geodetic Datum. The borehole locations and elevations are approximate.

4.0 SUBSURFACE CONDITIONS

The detailed subsurface soil and groundwater conditions encountered in the boreholes and probeholes put down as part of this investigation, together with the results of the laboratory tests carried out on selected soil samples, are given on the attached Record of Borehole sheets and Figures 4 to 18 following the text of this report. The indicated stratigraphic boundaries on the boreholes logs are inferred from non-continuous sampling and, therefore, represent transitions between soil types rather than exact planes of geological change. Subsoil conditions will vary between and beyond the hole locations.

The Record of Borehole sheets from the investigations carried out previously by MTO are included in Appendix A. The following discussion incorporates the previous information as relevant to the specific sections of the alignment.

The subsurface conditions encountered in the boreholes are variable. In general, however, the subsoils along the alignment consist of organic silt or topsoil overlying either an extensive silty clay deposit directly or sand and/or silt deposits in turn underlain by silty clay. A sandy till deposit was occasionally encountered below the silty clay. Red sandstone bedrock was encountered below the till in two of the boreholes put down north of the CPR. Occasional areas were noted where a number of boulders were present at ground surface (specifically, where the alignment crosses Echo River and Government Roads). A more detailed account of the subsurface conditions encountered along the alignment follows.

It should be noted that the proposed alignment crosses at least six areas of wetland. Probeholes were put down within four of these wetlands. This information is summarized separately in tabular form together with the augerhole location plan for each wetland (see Figures 3B to 3E).

4.1 Black Road (Sta. 10+000) to Jardin Mines Road (Approx. Sta. 16+700)

6.7 km

Within this section of the alignment, four boreholes and two probeholes holes were put down at the approximate locations shown on Figures 2A through 2D and Figure 3A. Fourteen hand auger holes were put down in areas of wetland approximately at the locations shown on Figures 3B and 3C.

4.1.1 Organics/Topsoil

Dark brown organic silt to organic sandy silt topsoil was encountered at ground surface in each of the boreholes and probeholes put down along the section of the alignment. The thickness of the topsoil layer generally varies between 0.1 m and 0.3 m along much of the alignment between Sta. 10+000 and Sta. 16+700. Up to about 0.85 m of topsoil was encountered at hand auger holes 95-HA11 to 95-HA14 located in the vicinity of Sta 15+500.

4.1.2 Upper Sand

A granular deposit of grey to brown sand with trace to some silt and containing layers of silty sand (up to 1.5 m thick) underlies the topsoil in Borehole 95-1 to a depth of 14.8 m below ground surface. This upper sand deposit was also encountered in Boreholes 95-4 and 95-6 to a depth of 2.6 m and 2.2 m below ground surface respectively. Occasional organic silt partings and interlayers of sandy silt, silt, clayey silt and silty clay were encountered within this deposit. In Borehole 95-4, a 0.2 m thick layer of organic silt was observed at a depth of 4.7 m. The results of grain size distribution tests carried out on selected samples of the sand deposit are shown on Figure 4.

Standard Penetration Test (SPT) 'N' values measured in this deposit range from less than 1 blow (weight of hammer) to 10 blows for 0.3 m of penetration indicating a very loose to loose state of packing. The measured natural water contents of selected samples of this deposit range from about 5 to 33 per cent. Higher values of about 49 and 78 per cent were measured within this deposit in a silty clay and organic layer, respectively.

4.1.3 Interlayered Silts

Underlying the upper sand deposit in Boreholes 95-4 and 95-6 and the topsoil in Borehole 95-5 and Probeholes 95-PH3 and 95-PH4, is a deposit of interlayered silts. This deposit generally consists of interlayered sandy silt, silt and clayey silt to silty clay but the deposit also contains layers of sand and silty sand. Occasional organic seams were noted throughout the deposit. Sand layers of 1.1 m to 1.5 m thick, were encountered in Boreholes 95-4 and 95-6 at depths of 7.2 m and 8.1 m, respectively. The results of grain size distribution tests carried out on selected samples of this deposit are shown on Figures 5 and 6.

At Probeholes 95-PH3 and 95-PH4 and Borehole 95-5, the interlayered silts underlie the topsoil and extend to depths of 2.2 m, 4.9 m and 4.0 m respectively, below ground surface. At Borehole 95-4, the silts were encountered between 5.0 m and 14.8 m depth below ground surface. The interlayered silts extended to 9.2 m below ground surface at Borehole 95-6.

Where this deposit underlies the topsoil directly, it is brown to grey in colour, likely due to weathering. However, where this deposit underlies the upper sand it is grey in colour. SPT 'N' values measured in this deposit range from less than 1 blow (weight of hammer) to 8 blows for 0.3 m of penetration indicating a very loose to loose state of packing. An SPT 'N' value of 17 was measured in Borehole 95-4 within the 1.5 m layer of sand.

In situ vane shear tests carried out within the more cohesive portions of this deposit in Borehole 95-6 measured undrained shear strengths of 22 kPa and 25 kPa. These values indicate that this deposit has a soft consistency. Remoulded strengths indicate a sensitivity of 2.5.

The measured natural water contents of selected samples of this deposit range from about 23 per cent to 45 per cent, the higher water contents corresponding to the clayier samples. Atterberg limits tests performed on two samples of the clayier portions of this deposit indicated liquid limits of about 38 per cent and plastic limits of about 20 per cent. These values indicate that this material is of low plasticity.

4.1.4 Lower Sand

In Boreholes 95-4, 95-5 and 95-6, the interlayered silt deposit is underlain by another granular deposit consisting of brown to grey silty sand to sand with trace silt, gravel and clay. Occasional seams of silt to clayey silt were noticed. In Boreholes 95-4 and 95-6, this deposit is thin (1.5 m and 1.1 m thick, respectively) and directly overlies the silty clay deposit described in the following section. In Borehole 95-5, the sand deposit was encountered at a depth of about 4.0 m below ground surface but was not penetrated in the borehole which was terminated at Elevation 166.4 m (15.9 m depth). The result of a grain size distribution test carried out on one sample of this deposit is shown on Figure 7.

SPT 'N' values measured in this deposit range from 1 to 14 blows for 0.3 m of penetration indicating a very loose to compact state of packing. The measured natural water contents of selected samples of this deposit range from 25 to 27 per cent.

4.1.5 Lacustrine Clay

With the exception of Borehole 95-5, a deposit of lacustrine clay with trace sand was encountered immediately below the cohesionless sand or interlayered silt deposits in each of the borings put down in this area. The lacustrine clay was encountered at depths ranging from about 2.2 m to 17.8 m below ground surface about Elevation 191.6 m and Elevation 168.5 m, but was not penetrated completely in either of the borings. Based on available geological information, it is probable that this deposit extends to considerable depth. The dynamic cone test performed in Borehole 95-4 indicates that this lacustrine deposit extends to at least Elevation 139.8 m (thickness greater than 30 m).

The result of a grain size distribution test carried out on one sample of this deposit is shown on Figure 8. The silty clay deposit, for the most part, is irregularly layered with silt and clayey silt with some portions being more pronouncely layered than others. The colour of this deposit is typically greyish to reddish brown.

SPT 'N' values measured in this deposit range from less than 1 blow (weight of hammer) to 4 blows for 0.3 m of penetration. The dynamic cone test performed in Borehole 95-4 showed that the blows required for 0.3 m of penetration increase with depth and about 50 blows per 0.3 m were measured at 46.5 m depth below ground surface. In situ vane shear tests carried out within this deposit measured undrained shear strengths increasing with depth from about 21 kPa to 73 kPa. These values indicate that this deposit has a soft to stiff consistency. Remoulded strengths at about 5 kPa to 20 kPa correspond to a sensitivity between 3 and 5 indicating that the material is sensitive.

The measured natural water contents of selected samples of this deposit range from 32 to 80 percent. Atterberg limit tests performed on selected samples of this deposit indicated liquid limits between 59 and 82 per cent and plastic limits between 26 and 34 per cent. These values indicate that this material is of medium to high plasticity.

4.1.6 Groundwater Conditions

Water was encountered during drilling at depths of between 1.5 m and 8.7 m below ground surface. Piezometers were installed in each of the boreholes put down in this area. The water levels measured in the piezometers sealed into the silty clay deposit in Boreholes 95-1, 95-4 and 95-6 were measured to be between about Elevation 185.6 m and 179.3 m (depths between 2.9 m and 7.0 m below ground surface). The water level measured in the piezometer sealed into the sand deposit in Borehole 95-5 was measured to be at about Elevation 177.9 m (4.3 m depth below ground surface).

4.1.7 Wetlands

Within this area, the proposed alignment crosses two relatively large wetlands; one at the Link Road intersection and the second at Peltier Creek. The results of probeholes put down in these wetlands are shown in the tables following Figures 3B and 3C.

Link Road Intersection (Root River) Wetland

Six hand auger holes numbered 95-HA1 to 95-HA6 were put down between Sta. 11+000 and Root River at the locations shown on Figure 3B. The wetland area extends west from about Sta. 11+500 along the highway alignment and south along Link Road. The subsoils encountered during probing generally consist of up to 0.3 m of soft peat underlain by soft clayey silt. At the location of hole 95-HA6 (closer to Root River) about 1.2 m of loose silty sand to sandy silt was encountered. To the east of the Link Road intersection, firm silty clay was encountered at 0.4 m depth in holes 95-HA4 and -HA5 and at 1.4 m depth in hole 95-HA6. Firm strata were not encountered in holes 95-HA1 to -HA3 located on Link Road and to the east of the intersection. These holes were terminated at 0.6 m depth in soft clayey silt. West of about Sta. 11+500 and on Link Road, the area was wet with standing water evident at the hole locations.

Peltier Creek Wetland (approximate Sta. 14+375 to Sta. 15+700) ≈ 1325m

Eight hand auger holes numbered 95-HA7 to -HA14 were put down within the wetland area at the locations shown on Figure 3C. Up to 1.2 m of soft peat and organic silt was encountered underlain by soft silty clay/clayey silt and loose silty sand. Firm strata were not found in the auger holes which were terminated at depths ranging from 0.6 m and 1.5 m. The area was generally wet with up to 0.15 m of standing water at the hole locations.

4.2 Jardin Mines Road (Approx. Sta. 16+700) to Echo River Road (Approx. Sta. 24+200)

Within this section of the alignment, two boreholes were put down during the current investigation at the approximate locations shown on Figures 2E and 2F. In 1975 MTO had put down two boreholes east and west of Garden River (see Figure 2E). Additionally, three gravel pit surveys were carried out in this area in December 1991. This information is presented in Section 5.6.

4.2.1 Organic/Topsoil

Dark brown organic silt to organic sandy silt topsoil was encountered at ground surface in each of the boreholes along this section of the alignment. The thickness of this topsoil layer varies between 0.15 m and 0.3 m.

4.2.2 Upper Sand

The topsoil in Borehole 95-7 is underlain by a deposit consisting of brown sand with some gravel, trace to some silt and layers of silty sand to sandy silt which extends to a depth of about 4.3 m below ground surface. Standard Penetration Test (SPT) 'N' values measured in this deposit range from 2 to 15 blows for 0.3 m of penetration indicating a very loose to compact state of packing. The measured natural water contents of selected samples of this deposit range from 8 per cent to 25 per cent.

4.2.3 Interlayered Silty Sand and Sandy Silt

Underlying the upper sand deposit in Boreholes 95-7 and the surficial organic silt in Borehole 95-8, is a deposit of interlayered brown to grey sandy silt and silty sand with trace clay, frequent organics and occasional sand layer and clayey silt seam. At Borehole 95-7, the interlayered silts were encountered between depths of 4.3 m and 8.7 m. At Borehole 95-8, interlayered silts extended to a depth of 8.0 m. The results of grain size analysis carried out on selected samples of this deposit are shown on Figure 9.

SPT 'N' values measured in this deposit range from less than 1 blow (weight of hammer) to 4 blows per 0.3 m of penetration, indicating a very loose state of packing. The measured natural water contents of selected samples of this deposit range from 20 per cent to 49 percent. Atterberg limit tests performed on two samples of this deposit indicated a liquid limit of about 27 per cent, the test results indicated the samples were non-plastic.

4.2.4 Lower Sand

In each of the boreholes for the current investigation, the interlayered deposit is underlain by another deposit of sand with trace to some silt and trace gravel and clay. In Borehole 95-7, interlayers of sandy silt with occasional organics are present in about the top 6.0 m of the deposit (to a depth of 14.6 m). The result of a grain size analysis carried out on a selected sample of this deposit is shown on Figure 10. This deposit is generally brown in colour; the interlayers of silt are grey in colour and in Borehole 95-7 the sand becomes grey at depth. SPT 'N' values measured in this deposit range from 2 to 23 blows for 0.3 m of penetration indicating a very loose to compact state of packing.

Within the sand deposit a 1.7 m to 2.4 m thick layer of gravel with some sand to sand and gravel with sand layers, trace silt and clay was encountered in each of the boreholes. The result of a grain size distribution test carried out on a selected sample within this layer is shown on Figure 11. SPT 'N' values measured in this gravel layer were 62 and 69 blows per 0.3 m of penetration, indicating a very dense state of packing. The 'N' values drop to 15 blows for 0.3 m of penetration in Borehole 95-8 at the bottom of this layer, possibly due to disturbance as a result of the water pressures, at this depth.

The measured natural water contents of selected samples of this deposit range from 4 per cent to 26 per cent.

4.2.5 Groundwater Conditions

Water was encountered during the drilling of Borehole 95-7 between depths of approximately 3.8 m and 7.2 m and then again below about 15.2 m. Standing water was noted at ground surface in the vicinity of Borehole 95-8 and water was encountered during drilling to a depth of about 5.3 m and then again below about 12.1 m depth. Piezometers were installed in the lower sands in each of the two boreholes put down in this area during the current investigation. The water levels in these piezometers was measured to be at about Elevations 181.6 m and 177.2 m (corresponding to depths of 12.1 m and 17.1 m respectively below the ground surface).

4.2.6 Wetlands

Within this area, the proposed alignment crosses an area of wetland around the proposed Noonday Road overpass. Probeholes were not put down in this area due to the inaccessibility of the area.

4.3 Echo River Road (Approx. Sta. 24+200) to Maple Leaf Road (Approx. Sta. 16+700)

Within this section of the alignment, five boreholes, three probeholes and five hand auger holes were put down at the approximate locations shown on Figures 2G and 2H and Figures 3D and 3E.

4.3.1 Organics/Topsoil

About 0.2 m to 0.4 m of dark brown organic sandy silt to silty clay was encountered at ground surface in Boreholes 95-9, 95-10 and 95-11. About 0.15 m to 0.3 m of black to dark brown fibrous organic silt with trace to some sand was encountered at ground surface in Probeholes 95-PH1, 95-PH2 and 95-PH5. An organic content of 77 per cent was measured on one sample of organic silt.

4.3.2 Sandy Silt to Silty Sand

The topsoil and organic silt in Borehole 95-9 and Probeholes 95-PH2 and 95-PH5 is underlain by about 0.2 m to 0.6 m of loose to very loose deposit of brown sandy silt with some organics to sandy organic silt. Underlying the topsoil at Probehole 95-PH1 is a 1.3 m thick lacustrine deposit of silty clay with trace sand and frequent organic matter. This silty clay layer is in turn underlain by about 0.8 m of sandy silt with some organics.

SPT 'N' values measured in this deposit were 4 and 7 blows per 0.3 m of penetration, indicating that the deposit is in a very loose to loose state of packing. The measured water content on samples from this deposit ranged between 20 per cent and 27 per cent.

4.3.3 Lacustrine Clay

A deposit of lacustrine clay with trace sand was encountered immediately below the thin sandy silt deposit except for Probehole 95-PH1 and Boreholes 95-10 and 95-11 where it underlies the organic deposit directly. The clay deposit was not penetrated in Borehole 95-9 and Probehole 95-PH1 and 95-PH2 which extended to Elevation of 138.6 m (depth of 39.9 m below ground surface). The clay deposit is only about 2.3 m thick at Borehole 95-13 near the CPR crossing and it was not encountered in Borehole 95-12.

The results of grain size distribution tests carried out on selected samples of this deposit are shown on Figures 12 and 13. For the most part, the deposit is irregularly layered with silty clay, clayey silt and silt with the layering more pronounced in some portions. The colour of this deposit is typically greyish to reddish brown. In Probehole 95-PH1, the upper 3.2 m of this deposit contains occasional fibrous peat layers and is brownish grey to grey in colour, indicative of a high organic content.

SPT 'N' values measured in this deposit range from less than 1 blow (weight of rods) to 7 blows for 0.3 m of penetration. In situ vane shear tests carried out within this deposit measured undrained shear strengths increasing with depth from about 15 kPa to 90 kPa. These values indicate that this deposit has a soft to firm consistency. Remoulded strengths, range from about 4 kPa to 47 kPa corresponding to a sensitivity between 2 and 5 and indicating that some portions of the deposit are sensitive.

The measured natural water contents of selected samples of this deposit range from 32 per cent to 88 per cent. A lower water content of 26 per cent was measured in the sandy silty interlayered in the top portion of the deposit in Probehole 95-PH1. Atterberg limit tests performed on selected samples of the lacustrine clay deposit indicated liquid limits between 36 and 84 per cent and plastic limits between 18 and 36 per cent. These values indicate that this material is of medium to high plasticity.

Consolidation testing was carried out on two samples of the lacustrine clay from Borehole 95-9. The results (shown on Figures 17 and 18) indicate effective preconsolidation pressures of 155 kPa for Sample 12 at 20 m depth and 200 kPa for Sample 20 at 39 m depth. These results indicate that the deposit is normally consolidated. The estimated effective overburden pressure at 39 m depth is lower than the preconsolidation pressure; however, this could be a function of the assumed unit weight and the piezometric pressure. Alternatively the results at this depth could be a consequence of a soft zone within the irregularly layered silty clay deposit.. The compression index for both samples was about 0.32.

4.3.4 Silty Sand Till

The lacustrine clay at Probehole 95-PH5 and Borehole 95-10 and the upper sand at Borehole 95-12 is underlain by a till deposit consisting of grey silty sand with some gravel and trace to some clay grading to silty sand and gravel with trace clay. Recorded SPT 'N' values in the till range from 2 to 27 blows per 0.3m of penetration. Measured natural water contents of selected samples of the till varied between 8 and 13 per cent. The results of grain size distribution tests carried out on selected samples of this deposit are shown on Figure 14.

4.3.5 Interlayered Sandy Silt to Silty Sand

Underlying the lacustrine clay at Borehole 95-11 was a 3.0 m thick layer of interlayered sandy silt to silty sand with occasional silty clay layers and some gravel. The majority of the measured 'N' values in the deposit were less than 1 blow per 0.3 m of penetration (weight of the hammer). Near the base of the deposit, one 'N' value of 17 blows per 0.3 m penetration was measured, possibly due to the increased gravel content. The natural water content of samples from this deposit ranged between 22 and 36 per cent.

4.3.6 Silty Sand to Sand and Gravel

At Boreholes 95-12 and 95-13, near the CPR crossing, the subsurface conditions are quite different from those drilled to the north and predominantly consist of sand with trace silt to silty sand. At Borehole 95-12, a silty sand till deposit was encountered between 2.2 m and 3.7 m depth. Above and below the till is a silty sand deposit which grades to coarse sand some silt. At 5.3 m and 7.2 m depth there is a 0.7 m thick sand and gavel layer with trace silt. Sand with some silt and trace to some gravel is present above and below the silty clay layer in Borehole 95-13.

The SPT 'N' values vary from less than 1 blow (weight of rods) to 8 blows per 0.3 m of penetration, indicating a very loose to loose state of packing. In the sand and gravel layers, one measured 'N' value was 15 blows per 0.3 m of penetration which indicates that the sand and gravel is in a compact state of packing. The natural water content of selected samples varied between 6 and 27 per cent. The results of grain size distribution tests carried out on selected samples of this deposit are shown on Figures 15 and 16.

4.3.7 Bedrock

Bedrock was encountered in Boreholes 95-10 and 95-11 at about Elevation 178.4 m (depths of between 9.4 and 10.2 m below ground surface). The bedrock consists of dark reddish brown weathered sandstone.

4.3.8 Groundwater Conditions

Piezometers were installed in the lacustrine clay deposit in Boreholes 95-9 and Probeholes 95-PH2. The measured water level in Borehole 95-9 and Probehole 95-PH2 was at Elevation 179.8 m and 176.7 m, respectively. The water level in Borehole 95-9 was at least 1.3 m above ground surface (at the top of the piezometer tube) indicating artesian pressure at depth. A shallow piezometer was installed in the upper clay at Borehole 95-10 and the water level was measured to be at ground surface.

A piezometer was installed in the silty sand till underlying the lacustrine clay at Probehole 95-PH5. At the time of water level measurement, the water was flowing out the top of the standpipe which was at Elevation 181.2 m. Standing water was noted at ground surface at this probehole location.

Two piezometers were installed in the bedrock, one each in Boreholes 95-10 and 95-11. The water level in Borehole 95-11 was measured at Elevation 189.6 m; in Borehole 95-10, the water was flowing out the top of the standpipe which was at Elevation 189.3 m (1.5 m above ground surface).

A piezometer was installed in the silty sand in Boreholes 95-12 and 95-13; the water level was near the ground surface at an Elevation of 188.0 m and 187.35 m respectively.

4.3.9 Wetlands

Within this area, the proposed alignment crosses two large areas of wetland; one immediately north of Echo River and the second south of Echo Bay. The results of probeholes put down in these wetlands are shown in the tables following Figures 3D and 3E. It was noted that a third wetland area exists to the south of Borehole 95-13; however, probeholes were not put down in this area.

Echo River Wetland (approximate Sta. 24+200 to Sta. 25+700)

Three hand auger holes numbered 95-HA15 to HA17 were put down within the wetland area at the locations shown on Figure 3D. Up to 0.3 m of soft organic silt/peat was encountered underlain by a thin layer of silty sand to sandy silt. Firm brown silty clay was encountered at about 0.3 m to 0.4 m depth. The area was generally wet with standing water up to 0.3 m depth at the hole locations.

Highway 638 Wetland (approximate Sta. 11+460 to Sta. 13+000)

Two hand auger holes (95-HA18 and -HA19) and three probeholes (95-PH1, -PH2 and -PH5) were put down within the wetland area at the locations shown on Figure 3E. About 0.2 m to 0.5 m of soft organic silt/peat was encountered underlain by soft silty clay which contains silty sand interlayers at some locations. The silty clay deposit was found to extend to greater than 9 m depth at the probehole locations. The area was generally wet with standing water up to 0.15 m deep at the hole locations.

5.0 ENGINEERING RECOMMENDATIONS

This section of the report discusses the geotechnical aspects of preliminary design of the proposed realignment of Highway 17. The design recommendations provided are based on our interpretation of the factual information obtained during the recent field investigation and, where pertinent, previous investigations performed by the Ministry of Transportation of Ontario (MTO) in the mid 70's. This section of the report is intended for the guidance of the design engineer only during the preliminary design phase of the project. Further subsurface investigation will be required for final design of the works. Comments on construction are provided only where they may affect the preliminary design; the information is not sufficient for construction purposes.

The roadworks considered in this report are associated with the realignment of Highway 17 (New) from Black Road in the City of Sault Ste. Marie, east and south to Bar River Road, a distance of about 27 km. The geotechnical recommendations contained herein relate to the preliminary design of the bridge foundations, embankment construction, permanent cut slopes, subgrade preparation and pavement design, and are based on information obtained from the plans and profiles provided by McCormick Rankin & Associates Ltd.

5.1 Bridge Structures

A total of eight bridge locations consisting of 12 structures (e.g. four structures are twinned) will ultimately be required along the proposed road alignment: at each of the crossings of the Root, Garden and Echo Rivers, Black and Belleau/Beaver Creeks, at both the Jardin Mines Road and Noonday Road overpasses and at the CPR crossing. The following is a discussion of foundation alternatives for each bridge site individually with respect to the subsoils encountered in our investigation or, where appropriate, earlier investigations by MTO. The recommendations are provided for preliminary design assessment; further subsurface investigation will be required at each of the bridge sites for final design once the foundation layout has been established.

5.1.1 Link Road over Black Creek

The proposed Link Road crosses Black Creek about midpoint between Highway 17 (New) and the existing highway (Trunk Road). The north bank of the creek is about 2.4 m high above the creek level at Elevation 182 m. The south bank of the creek rises steeply up to about Elevation 186 m then gently to about Elevation 189 m to the generally flat tableland extending to the intersection with the CPR tracks and Trunk Road.

Borehole 95-1 was put down to about 15.9 m depth on the south side of the Black Creek valley and close to the crest of the creek bank. Subsoils encountered in the borehole consist of 0.1 m of topsoil overlying an extensive deposit of generally very loose to loose reddish brown to grey sand with some silt, extending to about 14.8 m depth. Between about 10.2 m and 11.7 m the sand is in a compact state of packing. Interlayers of sandy silt, silty sand and silty clay were noted in the lower portion of the deposit below 11.7 m depth. The sands are underlain by a cohesive deposit consisting of very soft irregularly layered silty clay to clay and clayey silt with trace of sand. The groundwater level was at Elevation 181.2 m, some 5 m below the existing ground surface and coincident with the creek level.

It is our understanding that a culvert or a bridge will be constructed at the Black Creek crossing. The structure could be about 25 m to 30 m in length and embankments up to 2.5 m in height will be required at the approaches.

The subsoils are not suitable for support of shallow spread footings and deep foundations are recommended for support of the bridge/culvert. Consideration could be given to the use of close-ended steel pipe piles or timber piles. For preliminary design purposes, assuming a pile cap at about Elevation 182 m, a load carrying capacity at Ultimate Limit States (ULS) of 225 kN can be assumed for 9 m long 324 mm OD pipe pile driven into the sand (tip at about Elevation 173 m). For size 36 timber piles with minimum length of 12 m, a factored load capacity at ULS of 300 kN may be assumed. The load capacity at Serviceability Limit States (SLS) will be governed by the size of the pile group since it will be dependent on the settlement of the pile group which will result from consolidation of the underlying silty clay deposit. A detailed settlement analysis for the pile group configuration should be carried out as part of the final design and after further subsurface information at the site is obtained.

For the case of a culvert, the lateral pressure acting on the side walls of the box culvert will depend on the type and method of placement of the backfill materials and on the nature of the soils behind the backfill. A triangular pressure distribution should be assumed acting on the walls with coefficient of lateral earth pressure, K_o, equal to 0.5 and soil unit weight equal to 21 kN/m^3. A compaction surcharge equal to 16 kPa should be included in the lateral earth pressures for the structural design of the walls in accordance with OHBDC Figure 6.7.4.3.

Selected free-draining granular fill meeting the specifications of OPSS Granular A or B should be used as backfill around the culvert. All granular fill should be compacted in lifts of loose thickness not greater than 200 mm to 95 per cent of the material's Standard Proctor maximum dry density.

Construction of the bridge/culvert structure will likely require excavations to below the groundwater level. Groundwater flow into the excavations through the sands below the watertable (Elevation 181 m to 182 m) will be substantial and groundwater control (dewatering or cut-off) will be required. Where space permits, temporary unsupported excavations can be made with side slopes not steeper than 1.5 horizontal to 1 vertical (1.5H:1V) in the granular deposit above the groundwater level. The same slope inclination may be used below the water table provided that adequate dewatering has been implemented. Alternatively, a temporary support system consisting of soldier pile and lagging walls in conjunction with dewatering or steel sheetpiling to form a cut off may be utilized.

Embankments of heights up to 2.5 m will be subject to some settlement due to consolidation of the very loose to loose sands. This settlement is anticipated to occur during construction of the embankment and long term settlement will be negligible.

5.1.2 Root River

Borehole 95-4 was advanced from the top of the east bank of the Root River to a depth of approximately 25 m below ground surface. A dynamic cone test was continued beyond the bottom of the borehole to a depth of approximately 46.9 m below ground surface. It should be noted that the east bank rises steeply up from a narrow floodplain to a height of approximately 8 m above the river water level. The west bank immediately adjacent to the river is only about 2.5 m in height but due to access problems a borehole planned at the west bank had to be cancelled.

Subsoils encountered in the borehole consist of about 0.3 m of topsoil overlying a layered cohesionless deposit extending to a depth of about 17.8 m below ground surface. This deposit consists of very loose to compact, brown to grey interlayered sands, silty sands and sandy silts with occasional thin layers of clayey silt. Two silty clay layers, about 0.8 m and 1.5 m in thickness, were encountered at approximately 2.6 m and 14.8 m depths respectively. The cohesionless deposit is underlain by a deposit of firm to stiff, greyish brown silty clay which extends to the base of the sampled borehole and is inferred for the full depth of the dynamic cone test. The groundwater level was at Elevation 179.3 m which is slightly above the adjacent river level.

It is our understanding that the proposed twin bridge structures to carry Highway 17 (New) over Root River will be single span structures about 35 m in length. It is further understood that the final road grade at the river crossing is to be at approximately Elevation 186.2 m, about coincident with the existing ground surface at the crest of the east valley slope and about 6 m above the flood plain level at the west edge of the river.

The subsoils encountered in the borehole put down at this site are not suitable for support of shallow spread footings and deep foundations are recommended for the support of the abutments. Given the relatively loose nature of the cohesionless deposit encountered in the upper 18 m or so below ground surface, the carrying capacity of piles terminated above this depth would be relatively low. We recommend that the piles be extended into the firm to stiff silty clay deposit below about 18 m depth and that consideration be given to the use of steel H-piles or steel pipe piles or timber piles. For preliminary design purposes, a factored load carrying capacity at ULS of 400 kN may be assumed for a 22 m long HP 310x79 pile driven approximately 10 m into the silty clay deposit (to about Elevation 158 m). For timber piles, size 36 or larger and minimum length of 15 m, a factored load capacity at ULS of 300 kN may be assumed. These load capacities assume that the pile cap would be located about 6 m below the existing ground surface on the east side of the river (about Elevation 180 m).

The load capacity at SLS will be governed by the size of the pile group since it will be dependent on the settlement of the pile group and/or the settlement induced by the approach embankments. A detailed settlement analysis based on the pile group configuration should be carried out as part of the final design since the SLS capacity will likely govern the design.

Water flow into excavations made though the upper sandy deposits below the groundwater level will be substantial. Groundwater lowering or provision of a sheetpile cut-off may be required depending on the proposed pile cap elevation.

The proposed 6 m high embankment crossing the floodplain will be subject to settlement due to consolidation of the loose sands underlying the site. This settlement is expected to occur during construction of the embankment. It is anticipated that for typical embankment widths, surcharge loading onto the underlying soft clay deposit will be negligible and, therefore, the long term settlement will be nominal.

5.1.3 Belleau/Beaver Creek

Borehole 95-5 was put down on the east bank of Beaver Creek to a depth of about 14.6 m. As a result of access restrictions, the borehole is located approximately 23.5 m east of the edge of the creek.

Subsoils encountered in the borehole consist of about 0.1 m of topsoil overlying a very loose to loose deposit of brown to grey interlayered sands, silty sands and sandy silts extending to a depth of about 5.2 m. Occasional thin layers of clayey silt were also noticed. Underlying this fine grained deposit is a granular deposit of very loose to compact, brown to grey sand with trace to some silt which extends to the depth investigated. The groundwater level was at about Elevation 178 m (about 4.3 m below ground surface at the borehole location).

It is our understanding that the proposed twin bridge structures to carry Highway 17 (New) over Beaver Creek will be single span structures about 39 m in length. We further understand that the final road grade at the creek crossing is to be at approximately Elevation 186.7 m, making it about

2.4 m and 3.8 m above the existing ground surface at the top of the west and east banks, respectively.

Deep foundations are recommended for the support of the abutments. Loose cohesionless deposits were encountered for the full depth of the borehole (about 16 m deep) and consideration should, therefore, be given to the use of close ended steel pipe piles or timber piles. For preliminary design purposes, assuming a pile cap about 2 m below the existing ground surface, a load carrying capacity at ULS of 260 kN may be assumed for a 16 m long 324 mm OD pipe pile driven into the sands to about Elevation 164 m. For size 36 timber piles with minimum length of 12 m, a factored load capacity at ULS of 300 kN may be assumed.

The load capacity at SLS will be governed by the size of the pile group since it will be dependent on the settlement of the pile group. A detailed settlement analysis based on the pile group configuration should be carried out as part of the final design since serviceability limit states could govern the design.

Embankments of heights up to 4 m will be subject to some settlement due to consolidation of the loose sands. This settlement is expected to occur during construction of the embankment and long term settlement will be negligible.

5.1.4 Jardin Mines Road

Borehole 95-6 was advanced to a depth of approximately 15.9 m below ground surface in close proximity to where the re-routed Jardin Mines Road is proposed to cross over Highway 17 (New).

Subsoils encountered in the borehole consist of about 0.2 m of topsoil overlying a deposit of very loose to loose, brown silty sand to sand extending to a depth of about 2.2 m. Underlying this granular deposit is a fine grained deposit of very loose/soft, grey interlayered sandy silt and clayey silt to silty clay which extends to a depth of about 8.1 m below ground surface. This deposit is underlain by a layer of greyish brown sand approximately 1.1 m thick. This sand layer is in turn underlain by a deposit of stiff, brownish grey silty clay which continues from a depth of about 9.2

m to the end of the borehole. The groundwater level was at Elevation 185.6 m (about 3 m below ground surface) at the borehole location.

It is our understanding that the bridge proposed to carry the re-routed Jardin Mines Road over Highway 17 (New) will be about 55 m in length and could be a single or double span structure. The proposed Highway 17 (New) grade is at about Elevation 186.5 m, about 2.5 m to 3 m below the existing grade and the proposed Jardin Mines Road grade will likely be at about 3 m above existing ground surface.

Very loose fine grained deposits were encountered in the upper 9.2 m below ground surface. The piles should be extended into the underlying firm to stiff silty clay deposit below about Elevation 179 m and consideration should be given to the use of steel H-piles, close ended steel pipe piles or timber piles. For preliminary design purposes, assuming a pile cap 3 m below the existing ground surface, a factored load capacity at ULS of 250 kN may be assumed for steel H-piles or pipe piles driven to Elevation 171 m. For size 36 timber piles with a minimum length of 15 m, a factored load capacity at ULS OF 300 kN may be assumed.

The load capacity at SLS will be governed by the size of the pile group since it will be dependent on the settlement of the pile group. A detailed settlement analysis based on the pile group configuration should be carried out as part of the final design.

The proposed 3 m high embankments will undergo some settlement due to consolidation of the sands. In addition, there may be sufficient loading to effect consolidation settlement of the underlying clays depending on the configuration of the road embankment. Settlement of the sands will occur during embankment construction; consolidation of the underlying clay could induce long term embankment settlement. The magnitude of settlement should be checked once the embankment configuration is confirmed.

It is noted that the proposed Highway 17 (New) road grade is close to (about 0.9 m above) the measured groundwater level. Depending on the proposed pile cap level, groundwater control may be required during excavation and pile cap construction.

5.1.5 Garden River

Borehole 95-7 was advanced to a depth of about 25.0 m below ground surface at the crest of the east river valley slope. It should be noted that the east valley slope is about 13 m high, rising steeply from a wide floodplain and consequently, Borehole 95-7 is located approximately 130 m east of the river. A borehole was put down at the west bank of Garden River as part of an investigation carried out by MTO in the fall of 1975. This borehole (numbered 3) was advanced to a depth of approximately 12.5 m in close proximity to the proposed Highway 17 (New) centreline.

The subsoils encountered in the MTO borehole on the west bank edge of the river consist generally of sand deposits. Above a depth of about 9.8 m, the sand is typically in a loose state of packing; a thin layer of silty clay was encountered at about 6.4 m depth. Below about 9.8 m depth the deposit consists of very dense sand and gravel. Water was encountered at about 3.6 m depth, approximately coincident to the water level in the river.

At the location of Borehole 95-7, the subsoils consist of about 0.2 m of topsoil underlain by an extensive granular deposit. Loose to compact, brown sand was encountered extending to a depth of about 3.0 m underlain by about 4.7 m of very loose, brown to grey, interlayered sand, silty sand and sandy silt with occasional thin layers of clayey silt. Underlying this fine grained deposit is another granular deposit consisting of compact, brown to brownish grey sand and containing two layers of very loose, grey sandy silt layers approximately 0.7 and 1.3 m thick between about 10.2 m and 14.6 m depth. Directly below the second sandy silt layer is a layer of very dense gravel with some sand and silt. The sands below about 14.6 m depth and continuing to the end to the borehole are in a loose to very loose state of packing.

It is our understanding that the proposed twin bridge structures to carry Highway 17 (New) over Garden River will be three span structures, with a total length of about 75 m and proposed bridge deck at approximately Elevation 189.7 m. The road grade is about 6.3 m below the existing ground surface at the west river bank and about 9.2 m above the existing ground surface at the east end of the bridge.

There is significant variation of subsoils at this site based on the results of our borehole and the MTO borehole put down on/close to the highway alignment. The latter is closer to the proposed bridge and is likely more representative of the subsoils in the immediate area of the west abutment of the proposed bridge. A very dense granular deposit was encountered below about 9.9 m depth underlying loose sands. Consideration could be given to the use of 310x79 steel H-piles or close ended pipe piles driven to about Elevation 171 m at the west abutment. In the borehole to the east of the bridge, compact sands and gravels were encountered to about 5 m below the proposed pile cap level (Elevation 183 m) underlain by very loose sands. Assuming similar conditions at the east abutment, consideration should be given to close ended pipe piles driven through the compact sands into the underlying deposits. For preliminary design purposes, a load carrying capacity at ULS of 460 kN may be assumed for 11m long piles on the west side; for 16 m long piles on the east side, a value of 260 kN may be assumed. For the pier foundations, the pile capacities of the adjacent abutments may be assumed. The pile capacity at the west pier, however, will depend on the pile cap elevation and the length of pile achievable above the dense sand deposit.

The load carrying capacity at SLS will depend on the extent of the dense sands for the west abutment and pier and on the pile group configuration for the east abutment and pier. If practical refusal (or at least significant driving resistance) is met for piles driven into dense sands at the west abutment and pier, settlement of the piles could be negligible compared to that at the east abutment and pier. The SLS capacity at the east abutment and pier will be dependent on the settlement of the pile group. A detailed settlement analysis will be required at final design to assess the potential for differential settlement between the east and west piers.

Settlement of the proposed 9 m high embankments in the floodplain is expected to be minimal as a result of settlement of the in situ soils. The majority of long term settlement will be due to consolidation of the embankment materials themselves; settlement of the in situ sands will occur during construction. minimal

5.1.6 Noonday Road

Borehole 95-8 was advanced to a depth of approximately 14.3 m below ground surface in close proximity to where the re-routed Noonday Road is proposed to cross over the Highway 17 (New).

Subsoils encountered in the borehole consist of about 0.3 m of topsoil overlying a deposit consisting of very loose, brown to grey interlayered sands, silty sands and sandy silts which extend to a depth of about 8.0 m below ground surface. The layered deposit is underlain by a coarser grained deposit ranging in composition from sand with some gravel to sand and gravel containing trace to some silt and occasional silt interlayers. Between about 8 m and 13.2 m depth, the deposit is in a compact to very dense state of packing; below a depth of about 13.2 m the sand becomes much looser. The water level in the piezometer was measured to be at about Elevation 181.6 m; however, there is evidence of an upper water table close to ground surface within the upper interlayered sands.

It is our understanding that the bridge proposed to carry the re-routed Noonday Road over the Highway 17 (New) will be about 55 m in length and could be a single or double span structure. Furthermore, we understand that the pavement surface of the new highway under the Noonday Road overpass is to be at approximately Elevation 191.0 m, about 2.7 m below the existing ground surface.

The soils at the site consist of relatively loose fine grained deposits underlain at about 8 m depth by compact sands. The abutments could be supported on deep foundations such as close ended steel pipe piles driven into the compact sands below about 8 m depth. Assuming a pile cap at about 3 m below the existing ground surface, a load carrying capacity at ULS of 240 kN may be assumed for 324 mm OD pipe piles driven to Elevation 179 m. The corresponding value at SLS will depend on the configuration of the pile group and a settlement analysis should be carried out for final design once the pile group configuration is established.

Excavations for pile cap construction will likely be extended to below the upper groundwater level within the sands. Water inflow to the excavation could be significant where coarser sands are intercepted. Groundwater lowering or provision of a cut-off will be required depending on the pile cap elevation. It is noted that the proposed Highway 17 (New) grade will be below the upper groundwater level. The extent of permanent drainage requirements for the road will have to be established during final design.

Approach embankments may be up to 3 m above ground surface. Settlement of the sands underlying the embankment is expected to occur during construction and long term settlement of the embankment will be negligible.

5.1.7 Echo River

Borehole 95-9 was put down on the north side of Echo River to a depth of approximately 39.9 m below ground surface. A borehole (numbered 4) exists on the south side of Echo River close to the proposed Highway 17 (New) centreline and was put down as part of an investigation by MTO in the spring of 1975.

The subsoils encountered in both boreholes consist of about 0.2 m of topsoil overlying a thin veneer of silty sand approximately 0.6 m thick which is in turn underlain by an extensive deposit of very soft to stiff, brown to grey silty clay. The full depth of the silty clay was not penetrated within the depth investigated (about 40 m).

It is our understanding that the proposed twin bridge structures to carry Highway 17 (New) over Echo River will be single span structures about 44 m in length. Furthermore, we understand that the proposed road grade over the river is to be at approximately Elevation 181.1 m, making it about 2.5 m and 2.0 m above the existing ground surface at the top of the west and east river banks, respectively.

The subsoils at the site are not considered suitable for the support of shallow footings. Given the extensive depth of soft to firm silty clay encountered in the boreholes put down at this site, friction piles are the only feasible form of deep foundations. As such, consideration should be given to the use of timber piles or alternatively steel H-piles, although H-piles would have to be of significant length to make them economically feasible.

Assuming a pile cap 4 m below the existing ground surface, a load carrying capacity at ULS of 190 kN may be assumed for 15 m long type 36 timber piles (average diameter of 300 mm). For 30 m long HP 310x79 piles driven into the clay to a depth of approximately 34 m, a load carrying capacity at ULS of 300 kN may be assumed. The corresponding load capacity at SLS will be dependent on the configuration of the pile group and the settlement induced by the pile loading itself together with the embankment loading. A detailed analysis of the pile group settlement must be carried out at the time of final design once the configuration is determined.

Settlement of the approach embankments which are up to 3 m in height will occur due to consolidation settlement of the underlying silty clay. It is estimated that at least 700 mm of settlement will occur due to primary and secondary consolidation of the silty clay deposit within 10 years of completion of the embankment. This magnitude of settlement is representative of 90% consolidation; 50% of the settlement is likely to occur within 3 years following construction. For this estimate, we have assumed that the embankment is 3 m high, 8 m wide (i.e. one direction of divided highway) and has 2 horizontal to 1 vertical side slopes.

The magnitude of settlement can be reduced by embankment preloading, embankment surcharging or use of a lightweight fill material. Given the susceptibility of this deposit to consolidation under only nominal loading, it is recommended that consideration be given to preloading or surcharging. Alternatively, the time for the consolidation process could be shortened by applying electro-osmosis to the deposit. Testing of block samples would be required to assess the applicability of this alternative.

In general, an adequate factor of safety against failure has been calculated for embankments up to 3 m in height where soft irregularly layered lacustrine silty clay deposits are present at shallow depth below ground surface. For the grade raise as proposed immediately adjacent to the river banks, however, a factor of safety of less than 1.3 was obtained for a failure surface extending through the embankments and into the river. It is recommended that the abutments be maintained as far as possible from the edge of the river. The use of geogrid reinforcement within the embankment fill could serve to increase the stability of the embankment.

5.1.8 Canadian Pacific Railway

Borehole 95-12 was put down during current investigation on the north side of the CPR tracks to a depth of 7.9 m below the existing ground surface. Also relevant to this site are two boreholes (numbered 16 and 17) put down on each side of the CPR tracks as part of an investigation carried out by MTO in March 1975.

In general, the subsoils encountered in the boreholes consist of granular deposits which range in composition from silty sand to sand and gravel. Layering is evident within the deposit at some locations. Generally the granular materials are in a loose to compact state of packing; however, very loose zones were evident at variable depths. Below depths of 7.5 m and 11 m in Boreholes 16 and 17, respectively, the deposit is described as consisting essentially of silty sand with cobbles. This stratum extended to depths of 8.8 m and 12.2 m in Boreholes 16 and 17, respectively, where refusal to further auger penetration was met. Refusal to auger penetration was met at about 8 m depth (Elevation 180.9 m) in Borehole 95-12. The groundwater level was at Elevation 188 m, some 0.8 m below ground surface, at the location of Borehole 95-12.

It is our understanding that the proposed twin bridge structures to carry Highway 17 (New) over the CPR tracks will be one span structures, with a total length of about 20 m. The proposed road grade at the bridge is at approximately Elevation 198.7 m resulting in approach embankments about 9 m in height.

The results of the boreholes indicate that endbearing piles should be feasible for support of the bridge. Given the proposed height of embankment, consideration could also be given to construction of the abutments on a bank seat within the embankment.

The upper soils are variable; however, refusal to auger penetration was encountered in all of the boreholes at depths corresponding to Elevations 179.5 m to 180.9 m. It is anticipated that piles would be driven through the approach embankments and consideration should therefore be given to the use of steel H-piles driven to practical refusal which can be anticipated at about Elevations 179 m to 180 m. Assuming a pile cap at about 6 m below the proposed road grade, a load carrying capacity at ULS of 1600 kN may be assumed for 310 x 110 H-piles driven to practical refusal. Assuming that auger refusal implies the presence of bedrock, settlement of the piles will be negligible and SLS is not applicable. Further drilling (including bedrock coring) at this site will be required to confirm the founding conditions.

Approach embankments will be up to 9 m above ground surface. Settlement of the sands underlying the embankment is expected to occur during construction and long term settlement of the embankment will be due to consolidation of the embankment materials themselves.

5.1.9 Lateral Earth Pressures

The lateral pressures acting on the bridge abutments and the retaining walls 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:

- Selected free-draining granular fill meeting the specifications of OPSS Granular A or B should be used as backfill behind the walls. All granular fill should be compacted in lifts of loose thickness not greater than 200 mm to 95 per cent of the material's Standard Proctor maximum dry density.

- Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill.

- The granular fill may be placed either in a zone with width equal to at least 1.2 m behind the back of the stem (Case I) or within the wedge-shaped zone defined by a 60 degree line extending up and back from the bottom of the rear face of the footing (Case II).

- 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.

- For Case I, the pressures are based on the in-situ soils and the following parameters (unfactored) may be assumed:

Soil unit weight	22 kN/m^3	(assuming the in-situ soils are comprised of silty clay till)
Coefficients of lateral earth pressure:		
'active'	0.31	
'at rest'	0.47	

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

	Granular A	**Granular B**
Soil Unit Weight	22 kN/m^3	21 kN/m^3
Coefficient of Lateral Earth pressure		
'active'	0.27	0.31
'at rest'	0.43	0.47

It should be noted that the above design parameters assume level backfill and ground surface behind the wall. A portion of the wall length will have sloping ground behind the wall. Further design parameters can be provide once details of the walls are known.

5.2 Embankments and Permanent Cuts

In general, the proposed road grade involves cuts and embankments less than about 2 m. At the wetland crossings, embankments up to 3 m in height will be required. The following sections provide recommendations with respect to embankment stability and permanent cuts in areas where embankments greater than 2 m are proposed in areas of soft clay and, where cuts greater than 3 m are proposed.

5.2.1 Garden River - Permanent Cut Slopes

The proposed road grade involves a cut of up to about 3.5 m on the east side of the Garden River valley (Elev. 191.5 m to 192 m) and up to about 8 m on the west side (Elev. 185 m to 189 m).

Based on the previous and recent subsurface information, the cuts will generally be made through loose to compact sand/sand and gravel deposits and will be above the groundwater level. The groundwater table appears to be governed by the adjacent river level which is at about Elevation 180 m. In the area of the existing and proposed Jardin Mines Road excavations for road construction may intercept the groundwater table; however, it is anticipated that the proposed final road grade will be at or above the groundwater level. Permanent cut slopes formed at 2.0 horizontal to 1 vertical are considered suitable for preliminary design. Further investigation to confirm groundwater levels in relation to the cut slopes should be carried out for final design.

5.2.2 Lower Echo River Bridge and South to Hwy. 638 - Embankment Stability

The proposed road grade elevation in this area will involve embankment construction as follows:

Echo River and vicinity:	2 m embankment - road grade ranging from Elev. 180 m to 182 m
Sta. 11+400 to 11+700:	3 m to 5 m embankment - road grade ranging from Elev. 188 m to 182.5 m
Sta. 12+000 to 12+500:	2 m embankment - road grade at about Elev. 179 m.

The subsoils in this area consist generally of soft to stiff silty clay. Embankments 2 m to 4 m in height with side slopes no steeper than 2 horizontal to 1 vertical are considered feasible. Embankments greater than 4 m in height will require berms or flatter side slopes. If the section with greater than 4 m embankments is relatively short and the maximum height is 5 m, consideration could be given to providing flatter side slopes (i.e. 3 horizontal to 1 vertical).

5.2.3 CPR Tracks and Vicinity (Sta. 15+500 to 16+300)

The proposed road profile indicates embankments up to 6.6 m in height will be required to the north of the CPR tracks (ground surface about 192 and embankment crest at 198.6 m) in the area where soft silty clay subsoils are present. At and near the CPR tracks, the embankment will be about 9 m in height and the subsoils consist of loose to compact sand deposits. At about 100 m north and 80 m south of the CPR tracks, soft silty clay deposits are found overlying the sands. The following sections have embankments greater than 4 m in height and subsoils comprised of silty clay:

80 m south of Maple Leaf Road to about 2000 m south of Maple Leaf Road

It is suggested that for the length of the section, consideration could be given to flattening the side slopes to 3 horizontal to 1 vertical, provided the maximum height is 5 m. Alternatively, a 6 m wide 3 m high berm at the toe of the slope should be allowed for.

100 m north of CPR tracks to about 300 m north of CPR tracks

Benching and/or a berm is required in this area for the embankment which may be up to 6.6 m in height. Based on the profile data and plan information, a berm with width of 17 m and height of 3 m (at the toe of the embankment) may be required.

500 m north of CPR tracks

There is a short stretch where the embankment may be up to 4.4 m in height. It is suggested that consideration be given to flattening the side slope to 3 horizontal to 1 vertical.

5.3 Subgrade Preparation and Embankment Construction

In general, topsoil and organic deposits should be stripped from below the fill embankment areas and all subgrade soils should be proof-rolled prior to fill placement. In the wetland areas and in those areas where soft silty clay deposits are at the subgrade level, difficult working conditions will be encountered due to the groundwater level and the sensitivity of the soils forming the subgrade. In addition, there are likely to be areas where the subgrade consists of sandy silt or silt with the groundwater level at shallow depth. Pumping and rutting of the subgrade will occur with excessive construction traffic over the subgrade. Improvement of the subgrade either by use of geogrid or by placement of large size stone will be required to achieve adequate compaction of the lower lifts of the embankment fill.

Construction of the embankment above the prepared subgrade may be carried out using clean earth fill or granular fill. Clean earth fill (Select Subgrade Material, SSM or Select Borrow Material, SBM) is fill soil free of topsoil, organics, rubble, cobble sizes greater than 150 mm or other deleterious materials and which has a moisture content at the time of placement within 2 per cent of the material's optimum moisture content. All embankment fills should be placed in regular lifts with loose thickness not exceeding 300 mm, and be compacted to at least 95 per cent of the material's Standard Proctor maximum dry density. The final lift prior to placement of the granular subbase or base course should be compacted to 100 per cent of the Standard Proctor maximum dry density. Inspection and field density testing should be carried out by qualified geotechnical personnel during all fill placement operations to ensure that appropriate materials are used and that adequate levels of compaction have been achieved.

5.4 Pavement Design

The subsurface information obtained to date along the route indicates that the subgrade soils along the Highway 17 (New) alignment will be variable. In general, the following are the subsoil conditions and anticipated type of soil at the subgrade level along the highway alignment:

- **Black Road (Sta. 10+000) to Echo River Road (Sta. 24+200):**
  - organic silt/topsoil overlying an extensively granular deposit consisting of very loose to loose silty sand, sand and silt underlain at depth by cohesive deposits (BHs: 95-1, 95-4 to 95-8)
  - granular subgrade

- **Echo River Road (Sta. 24+200) to 100 m north of CPR crossing (Sta. 16+000):**
  - organic silt/topsoil overlying an extensive soft cohesive deposit (BHs: 95-9 to 95-11, PH1, PH-2 and PH-5)
  - cohesive subgrade

- **Sta. 16+000 to 100 m south of Maple Leaf Road (Sta. 16+400):**
  - topsoil overlying an extensive very loose to loose granular deposit of silty sand and sand (BH 95-12)
  - granular subgrade

- **Sta. 16+400 south:**
  - topsoil overlying an extensive soft cohesive deposit.
  - cohesive subgrade

In addition, the proposed alignment crosses at least six areas of wetland as noted in Section 4. Generally, highway construction over wetland/swamp areas can be carried out with subexcavation of organics and soft deposits where not more than about 4 m of subexcavation is required. Where there is more than 4 m of organics and soft deposits, subexcavation is excessive and consideration is given to controlled displacement and/or surcharging of the soft compressive deposits for embankment construction.

Within three of the four wetlands investigated, the probeholes put down indicate that there is less than 4 m of organics and soft materials within the proposed highway right-of-way. Firm strata were encountered at the following depths:

Link Road Intersection Wetland	- typically less than 1 m depth
Echo River Wetland	- typically less than 0.5 m depth
Highway 638 Wetland	- typically 0.5 m to 1.5 m depth.

Based on the profile drawings provided, embankments for road construction within the wetland areas will be less than 3 m high. It is considered that this height of embankment can be constructed in the above wetland areas with removal of the organic and upper soft clay deposits which were found to extend to depths of up to 1.5 m.

In the area of the Peltier Creek Wetland, firm strata were not inferred in the probeholes put down during this investigation. Additional subsurface investigation should be carried out in order to determine the depth of the organic and soft deposits.

Recommendations for pavement design thickness are in general accordance with the MTO Pavement Design and Rehabilitation Manual, SDO-90-01. The draft design (2004) traffic volumes were provided on a summary page dated March, 1996, and indicate AADT values ranging from 4,250 to 10,600 for Highway 17 (New). The following pavement structures have been based on a highway/freeway class of road for the design AADT values and thicknesses are provided for both the granular and cohesive subgrade conditions anticipated within the sections of highway alignment outlined above.

SUGGESTED FLEXIBLE PAVEMENT STRUCTURES

HIGHWAY 17 (NEW) - SAULT STE. MARIE TO BAR RIVER ROAD

AADT	PAVEMENT STRUCTURE ELEMENTS	THICKNESS OF PAVEMENT STRUCTURE ELEMENT	
		GRANULAR SUBGRADE	COHESIVE SUBGRADE
Greater than 4000 AADT	HM B SB GBE	130 150 450 710	130 150 750 910

Notes: HM - Hot Mix Asphalt
B - Granular 'A' Base Course
SB - Granular 'B' Subbase Course
GBE - Granular Base Equivalency Thickness

There will be portions of the highway where the proposed road grade requires embankment construction. In these areas the pavement structure thickness will depend on the type of embankment fill.

Where a change in thickness of the pavement structure occurs along the alignment, the structure thickness should be tapered at 10:1 or 20:1 or transition treatment as per OPSD's to prevent probable distress resulting from abrupt changes in pavement profile.

All pavement granular base and subbase materials should be compacted to 100 per cent of Standard Proctor maximum dry density. The binder course should be placed and compacted in two lifts and all asphaltic materials should be compacted to at least 97 per cent of their Marshall densities.

Where the proposed Highway 17 (New) is constructed in cut in the area of Noonday Road, the grade will be below the upper groundwater level as evident in the borehole at this site. Between the Garden River valley and the proposed Jardin Miness Road, the proposed road grade dips to about Elevation 184.5 m which is below the groundwater level measured at the proposed Jardin Miness Road (Elevation 185.6 m). The actual groundwater level in these two areas should be established to determine the extent of permanent drainage requirements for the road during final design.

5.5 Reuse of Site Soils

Materials excavated from those locations where Highway 17 (New) is to be constructed in cut can potentially be reused on or off site, subject to both geotechnical and environmental requirements. Soil samples obtained during this investigation were not submitted for chemical analyses to determine the environmental suitability of these materials for reuse or disposal.

In general the material to be excavated in conjunction with construction of the highway will consist of sands and silts underlying the topsoil/ organic materials. In some areas, excavation to the road subgrade level will encounter silty clay deposits. The silty clay deposits are not suitable for re-use in embankment construction due to the high water content and sensitivity of the soils. The sands and silts will generally be suitable for re-use; however, stockpiling and drying may be required in some areas to permit compaction.

Representative samples of soils to be reused for engineered backfill should be submitted to a qualified geotechnical laboratory for suitability testing prior to placement and compaction. In order to achieve the desired density, it is considered that the candidate material should have a natural water content within 2 per cent of the optimum water content for compaction.

5.6 Gravel Pit Information

There are three gravel pits along the proposed alignment of Highway 17 (New) within the Garden River First Nation (GRFN) boundaries (see Figure 1A). Information on these three pits are contained in the following sources:

- "Report on a Survey of the Known Gravel and Talus "Trap Rock" Deposits of the Garden River Indian Reserve No. 14, District of Algoma, Ontario", prepared by Chester J. Kiryliw, dated February 15, 1980. - provides a summary of GRFN PITS 53 and 65.

- Log of Test Holes and Data Sheet entitled "Report 5 - All Information", dated October 3, 1994, and corresponding site plans for GRFN Pit 53, 65 and 97, dated December 6, 17, and 19, 1991, respectively, prepared by MTO.

The three gravel pits are summarized below:

- **Gravel Pit 53 - CPR Ballast Pit**

The alignment of the proposed Highway 17 (New) crosses through the GRFN Gravel Pit 53 as it approaches the west bank of Garden River. A survey of the granular material contained in Pit 53 (previously known as the CPR Ballast Pit) was carried out in December 1991 and summarized in "Report 5" mentioned above. This survey consisted of 36 test holes to depths of between 2.4 m and 4.8 m. It indicated that, at the time of the investigation, approximately 115,000 tonnes of granular material existed in this pit; 65,000 tonnes crushable and an additional 50,000 tonnes non-crushable. The table below gives an indication of the gradation of the available granular material:

% Retained on Sieve					
4.75 mm		25 mm		100 mm	
0	70	0	50	0	15

The report indicates the following potential concerns regarding the use of available materials as Granular 'A' and 'B' Type I:

Granular 'A'	selection and sand control are required; per cent crushed may be a problem in sections
Granular 'B' Type I	oversize will need removing or crushing; compaction will be difficult in sections due to lack of fines while other areas will be too fine

In order to reach the useable granular material, between about 0.3 m and 2.1 m of unusable surficial material may have to be stripped. Furthermore, fine sand, silt and clay interlayers were encountered within the granular deposits and will need to be separated during excavation. It should be noted that, at the time of this survey, the pit face was sloughed and partially overgrown.

The results of Borehole 95-6 located to the west of the CPR pit on the highway alignment indicate that relatively clean sands are present to about Elevation 186.5 m. This is consistent with the elevation of the lower levels of the pit as shown on the contour plans provided for 1991 conditions. The proposed road grade in the area of the proposed Jardin Mines Road bridge is at about Elevation 186 m; to the east the proposed grade dips to about Elevation 184.5 m. Given the available information, it is probable that the proposed cut will generally extend to below the "clean" sand deposit and into the underlying silty sand to interlayered sandy and clayey silt deposit. The materials excavated in the lower 0.5 m to 2 m of the cut will therefore not be suitable as Granular 'A' or 'B' Type I.

Immediately west of the river, MTO Borehole 4 indicates that a clean sand to sandy gravel deposit exists to about Elevation 186.8 m (at which depth the borehole was terminated). A silty clay interlayer was encountered between the sand deposit and the sandy gravel deposit near the bottom of the borehole. The proposed road grade in the area of this borehole is at about 186.5 m. Given the available information, it is probable that the proposed cut on the west side of Garden River will generally be through the relatively "clean" sands which may be suitable as Granular 'B' Type I; portions will be suitable as Granular 'A'.

The preliminary profile for Highway 17 (New) in this area is up to 7 m below grade. Discounting for unusable material at the top and bottom of the granular deposit, there may be up to 220,000 m^3 of material within the proposed highway excavation limits that may be suitable for use as Granular 'A' or Granular 'B' Type I.

- **Gravel Pit 97 - Garden River Pit**

The alignment of the proposed Highway 17 (New) crosses south of the GRFN Gravel Pit 97 as it approaches the east bank of Garden River. A survey of the granular material contained in Pit 97 (previously known as the Garden River Pit) was carried out in December 1991 and summarized in "Report 5" mentioned above. This survey consisted of 66 test holes to depths of between 0.9 m and 4.8 m. It indicated that, at the time of the investigation, approximately 295,000 tonnes of granular material existed in this pit; 195,000 tonnes crushable and an additional 100,000 tonnes non-crushable. The table below gives an indication of the gradation of the available granular material:

% Retained on Sieve					
4.77 mm		25 mm		100 mm	
0	75	0	60	0	30

The report indicates the following potential concerns regarding the use of available materials as Granular 'A' and 'B' Type I:

Granular 'A'	selection and sand control are required; per cent crushed may be a problem in sections
Granular 'B' Type I	oversize will need removing or crushing; compaction will be difficult in sections due to lack of fines; some material is too coarse for the purpose; selection is required

In order to reach the useable granular material, between about 0.2 m and 2.1 m of unusable surficial material may have to be stripped. Furthermore, fine sand, silt and clay interlayers were encountered within the granular deposits and will need to be separated during excavation. Also, some boulders up to 0.9 m were encountered.

The results of Borehole 95-7 located south of the Garden River Pit on the highway alignment indicate that relatively clean coarse sands and gravel are present to about Elevation 192.5 m. This is consistent with the base of the potentially useable granular deposits as indicated for the existing gravel pit. In the Borehole 95-7, these coarse sands are underlain by fine sand which is present to about Elevation 190 m and which overlies interlayered silty sand and sandy silt. These lower deposits are not suitable as Granular 'A' or 'B'. The proposed road grade on the east side of Garden River in this vicinity is at about Elevation 192 m; only the material excavated from above Elevation 192.5 m may be suitable as Granular 'A' or 'B' Type I.

The preliminary profile for Highway 17 (New) in this area is about 4 m below grade. About 3.5 m of this is above Elevation 192.5 m. Discounting for unusable material at the top of the granular deposit, there may be up to 70,000 m^3 of material within the proposed highway excavation limits that may be suitable for use as Granular 'A' or Granular 'B' Type I.

- **Gravel Pit 65 - Maple Ridge Pit**

The alignment of the proposed Highway 17 (New) crosses through the GRFN Gravel Pit 65 as it approaches Echo River Road from the west. A survey of the granular material contained in Pit 53 (previously known as the Maple Ridge Pit) was carried out in December 1991 and summarized in "Report 5" mentioned above. This survey consisted of 51 test holes to depths of between 0.4 m and 4.8 m. It indicated that, at the time of the investigation, approximately 125,000 tonnes of granular material existed in this pit; 95,000 tonnes crushable and an additional 30,000 tonnes non-crushable. The table below gives an indication of the gradation of the available granular material:

% Retained on Sieve					
4.77 mm		25 mm		100 mm	
0	80	0	65	0	25

The report indicates the following potential concerns regarding the use of available materials as Granular 'A' and 'B' Type I:

Granular 'A'	selection and sand control are required; per cent crushed may be a problem in sections
Granular 'B' Type I	oversize will need removing or crushing; compaction will be difficult in sections due to lack of fines; some material is too coarse for the purpose; selection is required

In order to reach the useable granular material, between about 0.1 m and 2.1 m of unusable surficial material may have to be stripped. Furthermore, silt and clay interlayers were encountered within the granular deposits and will need to be separated during excavation. It should be noted that, at the time of this survey, the pit face was sloughed

The proposed road grade where Highway 17 (New) crosses through the pit area is at about Elevation 195 m. The existing gravel pit information indicates that the lowest base area of the east pit in 1991 was at about Elevation 200 m and that there was at least an additional 4 m of fine to medium granular below the pit floor. The base of the west pit was at about Elevation 192 m in 1991 and the test holes indicate that there is at least 4.5 m of "dirty" sands below the base. The adjacent low lying lands are known to be underlain by organics and silty clay deposits below about Elevation 185 m. Given this information, the cut for the proposed road grade appears to be terminated within the "clean" sands and there may be a further 1 m to 2 m of granular thickness remaining in place. The fine to medium granular will not generally be suitable for use as Granular 'A' but maybe suitable for use as Granular 'B' Type I.

The preliminary profile for Highway 17 (New) in this area is up to 25 m below grade. Discounting for unusable material at the top of the granular deposit, there may be up to 380,000 m^3 of material within the proposed highway excavation limits that may be suitable for use as Granular 'A' or Granular 'B' Type I.

GOLDER ASSOCIATES LTD.

G.E. Breeze, B.Sc. Eng.

A.S. Poschmann, P. Eng.
Principal

GEB/ASP/pds

Table 1
Hwy 17 New - Sault Ste. Marie to Bar River Road
Borehole/Probehole Locations

Borehole	Nearest Structure/ Landmark	Approximate Station	Offset m	Direction	Elevation, m	
					Borehole	Centre line
1	Black Creek	0+434	3.5	S of creek bank	186.2	187.6
4	Root River	10+351	*		186.3	186
5	Beaver Creek	11+174.5	0.6	N of centreline	182.2	186.9
6	Old Jardin Mines Road	16+096	*		188.5	186.5
7	Garden River	17+202	*		194.3	191.4
8	Noonday Road	17+533	*		193.7	191.3
9	Echo River	25+713	3	W of centreline	178.5	181.1
10	Watson Road	15+591	0.8	W of centreline	187.8	192.6
11	Government Road	15+910	60	W of centreline	188.5	-
12	CPR Tracks	16+091	*		188.8	198.7
13	Maple Leaf Road	16+400	15	W of centreline - at "treeline"	187.2	193.4
PH1	Hwy 638	12+710	5.5	W of centreline	176.9	179
PH2	Hwy 638	12+310	6.5	W of centreline	176.8	179
PH3	Black Road	10+048	20	S of centreline	193.8	195.1
PH4	Batchewana Prop. Limit	11+100	13	S of centreline	192.3	193.8
PH5	Echo Lake Road	11+510.5	2	E of centreline	180.2	185.2

Notes:

1. Data interpreted from profiles and plans provided by McCormick Rankin and include station conversion factors as provided to reflect revised chainage along alignment.
2. ' * ' Indicates hole put down approximately on alignment centreline.

LIST OF ABBREVIATIONS

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

I. SAMPLE TYPES

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

II. PENETRATION RESISTANCES

Dynamic Penetration Resistance:
The number of blows by a 63.5 kg (140 lb) hammer dropped 760 mm (30 in.) to drive uncased a 50 mm (2 in.) diameter, 60° cone attached to "A" size drill rods for a distance of 0.3 m (12 in.).

Standard Penetration Resistance, *N*:
The number of blows by a 63.5 kg (140 lb) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) drive open sampler for a distance of 0.3 m (12 in.).

WH sampler advanced by static weight—weight, hammer

PH sampler advanced by pressure—pressure, hydraulic

PM sampler advanced by pressure—pressure, manual

III. SOIL DESCRIPTION

(a) *Cohesionless Soils*

Relative Density	'N' Blows/0.30m 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	'Cu' kPa	'Cu' psf.
Very soft	0 to 12	0 to 250
Soft	12 to 25	250 to 500
Firm	25 to 50	500 to 1000
Stiff	50 to 100	1000 to 2000
Very stiff	100 to 200	2000 to 4000
Hard	over 200	over 4000

IV. SOIL TESTS

C consolidation test
H hydrometer analysis
M sieve analysis
MH combined analysis, sieve and hydrometer[1]
Q undrained triaxial[2]
R consolidated undrained triaxial[2]
S drained triaxial
U unconfined compression
V field vane test

NOTES:
[1]Combined analyses when 5 to 95 per cent of the material passes the No. 200 sieve.
[2]Undrained triaxial tests in which pore pressures are measured are shown as $\bar{Q}$ or $\bar{R}$.

LIST OF SYMBOLS

I. GENERAL

$\pi = 3.1416$

e = base of natural logarithms 2.7183

$\log_e a$ or $\ln a$, natural logarithm of a

$\log_{10} a$ or $\log a$, logarithm of a to base 10

t	time
g	acceleration due to gravity
V	volume
W	weight
M	moment
F	factor of safety

II. STRESS AND STRAIN

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

III. SOIL PROPERTIES

(a) *Unit weight*

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

(b) *Consistency*

w_L	liquid limit
w_P	plastic limit
I_P	plasticity index
w_S	shrinkage limit
I_L	liquidity index $= (w - w_P)/I_P$
I_C	consistency index $= (w_L - w)/I_P$
e_{max}	void ratio in loosest state
e_{min}	void ratio in densest state
D_r	relative density $= (e_{max} - e)/(e_{max} - e_{min})$

(c) *Permeability*

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

(d) *Consolidation (one-dimensional)*

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

(e) *Shear strength*

τ_f	shear strength	
c'	effective cohesion intercept	in terms of effective stress $\tau_f = c' + \sigma' \tan \phi'$
ϕ'	effective angle of shearing resistance, or friction	
c_u	apparent cohesion*	in terms of total stress $\tau_f = c_u + \sigma \tan \phi_u$
ϕ_u	apparent angle of shearing resistance, or friction	
μ	coefficient of friction	
S_t	sensitivity	

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

LIST OF ABBREVIATIONS USED IN
WETLAND SUMMARY TABLES - FIGURES 3B TO 3E

peat	pt.
organic	org.
sand	sd.
clay	clay
silt	silt
sandy	sdy.
silty	sty.
clayey	cly.
soft	so.
firm	fi.
stiff	st.
hard	ha.
dark	dk.
brown	brn.
grey	grey
green	grn.
loose	lse.
compact	com.
dense	dse.
interlayered	interl'd.
some	sm.
trace	tr.

J1364001.BHS

PROJECT: 941-1364

RECORD OF BOREHOLE 95-1

SHEET 1 OF 2

LOCATION: SEE FIGURE 2A

BORING DATE: SEPT.27-28/95

DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

DEPTH SCALE METRES: 0 to 10

BORING METHOD: TRACK MOUNTED CME 55 POWER AUGER, 108mm ID HOLLOW STEM AUGERS

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m

HYDRAULIC CONDUCTIVITY, k, cm/s

SHEAR STRENGTH Cu, kPa — nat V - + Q - ● ; rem V - ⊕ U - O

WATER CONTENT, PERCENT — Wp, W, Wl — 20 40 60 80

Soil Profile

Description	Elev. / Depth (m)
GROUND SURFACE	186.20
TOPSOIL	0.00
Silty sand, some rootlets Very loose Oxidized appearance	0.10
Fine sand, fine, some silt, trace clay Loose Greyish to reddish brown	185.50 0.70
Fine sand, some silt, trace clay, occasional organic silt partings Very loose Brownish grey	181.70 4.50
Silty sand with sandy silt interlayers, trace organics Very loose Grey	177.50 8.70

Samples

Number	Type	Blows/0.3m	Additional Lab. Testing
1	50 DO	1	
2	50 DO	9	
3	50 DO	8	
4	50 DO	9	
5	50 DO	10	
6	50 DO	7	
7	50 DO	2	MH
8	50 DO	1	
9	50 DO	WH	
10	50 DO	1	
11	50 DO	1	

Piezometer or Standpipe Installation

CUTTINGS

BENTONITE SEAL

CUTTINGS

CAVED

BENTONITE SEAL

CAVED

CONTINUED ON NEXT PAGE

J1364001.BHS

PROJECT: 941-1364

RECORD OF BOREHOLE 95-1

LOCATION: SEE FIGURE 2A

BORING DATE: SEPT.27-28/95

DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

BORING METHOD: TRACK MOUNTED CME 55 POWER AUGER, 108mm ID HOLLOW STEM AUGERS

DEPTH SCALE METRES	SOIL PROFILE: DESCRIPTION	ELEV. / DEPTH (m)	SAMPLES: NUMBER	TYPE	BLOWS/0.3m	PIEZOMETER OR STANDPIPE INSTALLATION
10	CONTINUED FROM PREVIOUS PAGE	176.00 / 10.20				
10–11	Fine sand, some silt, trace clay, trace organics Compact Grey					
11			12	50 DO	20	
		174.50 / 11.70				
12	Sand, some silt with layers of sandy silt, silty sand and silty clay Very loose Grey to reddish brown					SAND
			13	50 DO	WH	
13						SAND & CAVED
14			14	50 DO	1	
		171.40 / 14.80				CAVED
15	Irregularly layered silty clay to clay and clayey silt, trace sand Very soft Greyish to reddish brown		15	50 DO	WH	
		170.35 / 15.85				
16	END OF BOREHOLE					Water level in augers at about 12.2m depth below ground surface upon completion of drilling. Water level in piezometer at elevation 181.2m depth on October 5,1995.
17						
18						
19						
20						

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m

SHEAR STRENGTH Cu, kPa — nat V - + Q - ● rem V - ⊕ U - O

HYDRAULIC CONDUCTIVITY, k, cm/s

WATER CONTENT, PERCENT — Wp ├──O W──┤ Wl — 20 40 60 80

ADDITIONAL LAB. TESTING

DATA INPUT: PS JAN 22/96

DEPTH SCALE 1 to 50

Golder Associates

LOGGED: GEB

CHECKED: ASP

SOILMS

RECORD OF BOREHOLE 95-4

J1364004.BHS

PROJECT: 941-1364

SHEET 1 OF 5

LOCATION: SEE FIGURE 2B

BORING DATE: SEPT.27-28/95

DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

Boring method: TRACK MOUNTED CME 55 POWER AUGER, 108mm ID HOLLOW STEM AUGERS

DEPTH SCALE METRES	SOIL PROFILE – DESCRIPTION	ELEV. / DEPTH (m)
0	GROUND SURFACE	186.30 / 0.00
	TOPSOIL	186.00 / 0.30
	Sand, some silt, with layers of silty sand, sandy silt, sand and silt, silt and clayey silt. Occasional organics. Very loose to loose. Brown to grey	183.70 / 2.60
	Silty clay, some sand and silt. Very soft. Brown and grey	182.90 / 3.40
	Sand, some silt with layers of silty sand, sandy silt, sand and silt, silt and clayey silt with organic silt layers from 4.7m to 4.9m depth. Very loose to loose. Brown to grey	5.00
	Interlayered, sandy silt and clayey silt, occasional organics and silty clay seams. Loose to very loose/soft. Grey	179.10 / 7.20
	Sand, fine grading to coarse, trace silt and fine gravel. Compact. Grey to brown	177.60 / 8.70
	Interlayered, sandy silt and clayey silt, trace organics. Very loose/soft. Grey	
10	CONTINUED ON NEXT PAGE	

SAMPLE NUMBER	TYPE	BLOWS/0.3m	ADDITIONAL LAB. TESTING
1	50 DO	2	
2	50 DO	4	
3	50 DO	2	
4	50 DO	1	
5	50 DO	4	
6	50 DO	7	
7	50 DO	2	
8	50 DO	5	MH
9	50 DO	4	
10	50 DO	17	
11	50 DO	1	

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m: 20 40 60 80

SHEAR STRENGTH Cu, kPa: nat V - + Q - ● rem V - ⊕ U - ○ : 20 40 60 80

HYDRAULIC CONDUCTIVITY, k, cm/s

WATER CONTENT, PERCENT: Wp —○W— Wl : 20 40 60 80

PIEZOMETER OR STANDPIPE INSTALLATION: BENTONITE SEAL; CUTTINGS

DATA INPUT [illegible] JAN.22/[illegible]

DEPTH SCALE 1 to 50

LOGGED: GEB

CHECKED: ASP

Golder Associates

J136404.BHS

PROJECT: 941-1364

RECORD OF BOREHOLE 95-4

LOCATION: SEE FIGURE 2B

BORING DATE: SEPT.27-28/95

DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

Column headings: DEPTH SCALE METRES | BORING METHOD | SOIL PROFILE (DESCRIPTION, STRATA PLOT, ELEV. DEPTH (m)) | SAMPLES (NUMBER, TYPE, BLOWS/0.3m) | DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20, 40, 60, 80); SHEAR STRENGTH Cu, kPa (nat V - +, Q - ●, rem V - ⊕, U - O; 20, 40, 60, 80) | HYDRAULIC CONDUCTIVITY, k, cm/s; WATER CONTENT, PERCENT (Wp —O W— Wl; 20, 40, 60, 80) | ADDITIONAL LAB. TESTING | PIEZOMETER OR STANDPIPE INSTALLATION

Boring method: TRACK MOUNTED CME 55 POWER AUGER, 108mm ID HOLLOW STEM AUGERS

CONTINUED FROM PREVIOUS PAGE

Depth (m)	Description	Elev. / Depth (m)	Sample No.	Type	Blows/0.3m	Additional Lab. Testing	Piezometer or Standpipe Installation
10–14.80	Interlayered sandy silt and clayey silt, occ. organics Very loose/soft Grey	171.50 14.80	12	50 DO	WH	MH	CAVED
			13	50 DO	4		
			14	50 DO	3		
14.80–16.30	Silty clay Very soft Brownish grey to grey	170.00 16.30	15	50 DO	WH		
16.30–17.80	Silty fine sand Very loose Grey	168.50 17.80	16	50 DO	1		
17.80–20	Irregularly layered silty clay to clay and clayey silt to silt, trace sand Firm to stiff Greyish brown		17	50 DO	1		BENTONITE SEAL
			18	50 DO	WH	MH	SAND

CONTINUED ON NEXT PAGE

DATA INPUT: PS JAN 22/96

DEPTH SCALE
1 to 50

Golder Associates

LOGGED: GEB

CHECKED: ASP

SOILM6

PROJECT: 941-1364

RECORD OF BOREHOLE 95-4

LOCATION: SEE FIGURE 2B

BORING DATE: SEPT.27-28/95

DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. / DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20 40 60 80); SHEAR STRENGTH Cu, kPa (nat V - +, Q - ●, rem V - ⊕, U - O) (20 40 60 80)	HYDRAULIC CONDUCTIVITY, k, cm/s; WATER CONTENT, PERCENT Wp —O W— Wl (20 40 60 80)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
20	TRACK MOUNTED CME 55 POWER AUGER; 108mm ID HOLLOW STEM AUGERS	CONTINUED FROM PREVIOUS PAGE									
		Irregularly layered silty clay to clay and clayey silt to silt, trace sand Firm to stiff Greyish brown			18	50 DO	WH			MH	
22				164.05 22.25							
		END OF BOREHOLE Start of Dynamic Cone Test									Water level in augers at about 18.0m depth below ground surface upon completion of drilling. Water level in piezometer at Elev. 179.3m on October 5,1995.
30		CONTINUED ON NEXT PAGE									

DEPTH SCALE

1 to 50

Golder Associates

LOGGED: GEB

CHECKED: ASP

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J1364004.BHS

PROJECT: 941-1364

RECORD OF BOREHOLE 95-4

SHEET 4 OF 5

LOCATION: SEE FIGURE 2B

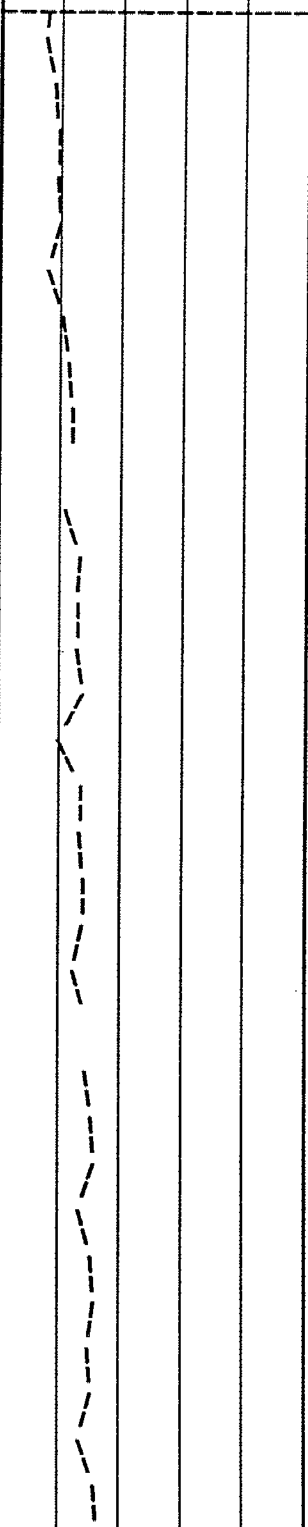
BORING DATE: SEPT.27-28/95

DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. / DEPTH (m)	SAMPLES: NUMBER	TYPE	BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20 40 60 80); SHEAR STRENGTH Cu, kPa (nat V - + Q - ●; rem V - ⊕ U - O) (20 40 60 80)	HYDRAULIC CONDUCTIVITY, k, cm/s; WATER CONTENT, PERCENT Wp —O W— Wl (20 40 60 80)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
30		CONTINUED FROM PREVIOUS PAGE									
31											
32											
33											
34											
35											
36											
37											
38											
39											
40		CONTINUED ON NEXT PAGE									



DATA INPUT: PS JAN.22/96

DEPTH SCALE
1 to 50

Golder Associates

LOGGED: GEB
CHECKED: ASP

SOILM6

PROJECT: 941-1364

LOCATION: SEE FIGURE 2B

SAMPLER HAMMER, 63.5kg; DROP, 760mm

RECORD OF BOREHOLE 95-4

BORING DATE: SEPT.27-28/95

DATUM: GEODETIC

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. / DEPTH (m)	SAMPLES: NUMBER	TYPE	BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20 40 60 80); SHEAR STRENGTH Cu, kPa (nat V - +, Q - ●, rem V - ⊕, U - O) (20 40 60 80)	HYDRAULIC CONDUCTIVITY, k, cm/s; WATER CONTENT, PERCENT Wp ⊢—O W—⊣ Wl (20 40 60 80)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		CONTINUED FROM PREVIOUS PAGE									
40											
41											
42											
43											
44											
45											
46				139.60							
		End of Dynamic Cone Test		46.70							
47											
48											
49											
50											

DEPTH SCALE

1 to 50

Golder Associates

LOGGED: GEB

CHECKED: ASP

J1364005.BHS

PROJECT: 941-1364

RECORD OF BOREHOLE 95-5

LOCATION: SEE FIGURE 2C

BORING DATE: SEPT.29/95

DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

DEPTH SCALE METRES | BORING METHOD | SOIL PROFILE: DESCRIPTION, STRATA PLOT, ELEV. DEPTH (m) | SAMPLES: NUMBER, TYPE, BLOWS/0.3m | DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m | SHEAR STRENGTH Cu, kPa (nat V - +, Q - ●, rem V - ⊕, U - O) | HYDRAULIC CONDUCTIVITY, k, cm/s | WATER CONTENT, PERCENT (Wp, W, Wl; 20, 40, 60, 80) | ADDITIONAL LAB. TESTING | PIEZOMETER OR STANDPIPE INSTALLATION

Boring method: TRACK MOUNTED CME 55 POWER AUGER; 108mm ID HOLLOW STEM AUGERS

Elev. / Depth (m)	Description
182.20	GROUND SURFACE
0.00	TOPSOIL
0.10	Silt, some sand and clay with interlayers of sandy silt to sand and clayey silt, trace organics. Very loose to loose. Greyish brown to brownish grey.
178.20 / 4.00	Sand, fine, trace silt and clay, occ. organics and grey silt to clayey silt seams Very loose to compact Greyish and reddish brown to brownish grey

Sample Number	Type	Blows/0.3m	Additional Lab. Testing
1	50 DO	4	
2	50 DO	8	
3	50 DO	6	MH
4	50 DO	2	
5	50 DO	WH	
6	50 DO	6	MH
7	50 DO	5	
8	50 DO	6	
9	50 DO	12	
10	50 DO	4	
11	50 DO	14	

Piezometer or standpipe installation: BENTONITE SEAL; CUTTINGS; BENTONITE SEAL; CAVED

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JAN 22

A INP

DEPTH SCALE 1 to 50

Golder Associates

LOGGED: GEB

CHECKED: ASP

PROJECT: 941-1364

RECORD OF BOREHOLE 95-5

SHEET 2 OF 2

LOCATION: SEE FIGURE 2C

BORING DATE: SEPT.29/95

DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES: NUMBER	TYPE	BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m / SHEAR STRENGTH Cu, kPa (nat V - +, Q - ●, rem V - ⊕, U - O)	HYDRAULIC CONDUCTIVITY, k, cm/s / WATER CONTENT, PERCENT (Wp, W, Wl; 20 40 60 80)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
10		CONTINUED FROM PREVIOUS PAGE									
11	TRACK MOUNTED CME 55 POWER AUGER; 108mm ID HOLLOW STEM AUGERS	Sand, fine, trace silt and clay, occ. organics and grey silt to clayey silt seams. Very loose to compact Greyish and reddish brown to brownish grey.			12	50 DO	5		O		
12											CAVED
					13	50 DO	1		O		
13											
14											
15					14	50 DO	10		O		
				166.35 15.85							
16		END OF BOREHOLE									Water level in piezometer at Elev. 177.9m depth on October 5,1995.
17											
18											
19											
20											

DATA INPUT: PS JAN 22/96

DEPTH SCALE 1 to 50

Golder Associates

LOGGED: GEB

CHECKED: ASP

PROJECT: 941-1364

RECORD OF BOREHOLE 95-6

SHEET 1 OF 2

LOCATION: SEE FIGURE 2D

BORING DATE: OCT.2/95

DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

BORING METHOD: TRACK MOUNTED CME 55 POWER AUGER; 95mm ID HOLLOW STEM AUGERS

Column headings: DEPTH SCALE METRES | BORING METHOD | SOIL PROFILE (DESCRIPTION, STRATA PLOT, ELEV. DEPTH (m)) | SAMPLES (NUMBER, TYPE, BLOWS/0.3m) | DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m | SHEAR STRENGTH Cu, kPa (nat V - +, rem V - ⊕, Q - ●, U - ○) 20 40 60 80 | HYDRAULIC CONDUCTIVITY, k, cm/s | WATER CONTENT, PERCENT Wp —○W— Wl 20 40 60 80 | ADDITIONAL LAB. TESTING | PIEZOMETER OR STANDPIPE INSTALLATION

DESCRIPTION	ELEV. / DEPTH (m)
GROUND SURFACE	186.50
TOPSOIL	0.00
Silty sand, some rootlets. Very loose Brown	0.15 / 186.10 / 0.40
Sand, trace to some silt, trace fine gravel Very loose to loose Brown	186.50 / 2.00
Silty sand, trace organics Very loose to loose Brown	186.30 / 2.20
Interlayered, sandy silt and clayey silt, occ. seams of silty clay with trace sand, organic partings from 2.20m to 3.7m depth. Very soft to soft/very loose to loose Grey	181.50 / 7.00
Interlayered silty sand, clayey silt and sandy silt. Occasional organic seams Very loose to loose Grey	180.40 / 8.10
Sand, trace silt Loose Brownish grey	179.30 / 9.20
Irregularly layered silty clay to clay and clayey silt, trace sand, occ. grey silt seam. Firm to stiff Greyish to reddish brown	

SAMPLE NUMBER	TYPE	BLOWS/0.3m	NOTES
1	50 DO	3	
2	50 DO	9	
3	50 DO	5	
4	50 DO	WH	
5	50 DO	1	
6	50 DO	WH	non-plastic; MH
7	50 DO	PM	
8	50 DO	5	
9	50 DO	2	

PIEZOMETER OR STANDPIPE INSTALLATION: CUTTINGS

CONTINUED ON NEXT PAGE

DEPTH SCALE 1 to 50

LOGGED: GEB

Golder Associates

CHECKED: ASP

PROJECT: 941-1364

LOCATION: SEE FIGURE 2D

SAMPLER HAMMER, 63.5kg; DROP, 760mm

RECORD OF BOREHOLE 95-6

BORING DATE: OCT.2/95

DATUM: GEODETIC

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.3m	SHEAR STRENGTH Cu, kPa (nat V - +, rem V - ⊕)	WATER CONTENT, PERCENT (Wp, W, Wl)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
10		CONTINUED FROM PREVIOUS PAGE									
11–15.85	TRACK MOUNTED CME 55 POWER AUGER; 95mm ID HOLLOW STEM AUGERS	Irregularly layered silty clay to clay and clayey silt, trace sand, occ. grey silt seams. Firm to stiff. Greyish to reddish brown			10	50 DO	WH				BENTONITE SEAL
					11	50 DO	WH				SAND; CAVED
15.85		END OF BOREHOLE		172.65 15.85							Water level in augers at about 14.0m depth below ground surface upon completion of drilling. Water level in piezometer at Elev. 185.6m on October 5,1995.

Dynamic penetration resistance, blows/0.3m: 20, 40, 60, 80

Hydraulic conductivity, k, cm/s

DEPTH SCALE 1 to 50

Golder Associates

LOGGED: GEB

CHECKED: ASP

RECORD OF BOREHOLE 95-7

PROJECT: 941-1364
LOCATION: SEE FIGURE 2E
SAMPLER HAMMER, 63.5kg; DROP, 760mm
BORING DATE: OCT.2-3/95
SHEET 1 OF 3
DATUM: GEODETIC
PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

BORING METHOD: TRACK MOUNTED CME 55 POWER AUGER, 95mm ID HOLLOW STEM AUGERS

Column headings: DEPTH SCALE METRES | BORING METHOD | SOIL PROFILE (DESCRIPTION, STRATA PLOT, ELEV. DEPTH (m)) | SAMPLES (NUMBER, TYPE, BLOWS/0.3m) | DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m | SHEAR STRENGTH Cu, kPa (nat V - +, Q - ●, rem V - ⊕, U - O) | HYDRAULIC CONDUCTIVITY, k, cm/s | WATER CONTENT, PERCENT (Wp, W, Wl; 20, 40, 60, 80) | ADDITIONAL LAB. TESTING | PIEZOMETER OR STANDPIPE INSTALLATION

Description	Elev. / Depth (m)
GROUND SURFACE	194.30
TOPSOIL	0.00
Sand, some gravel, trace silt Fine to coarse, occ. organic Loose Brown	0.15
Coarse sand to sand and gravel Compact Brown	193.60 0.70
Medium to coarse sand Loose Brown	192.90 1.40
Fine sand, some silt Loose Brown	192.50 1.80
Interlayered sand, silty sand and sandy silt, occ. organics Loose to very loose Brown	191.30 3.00
Interlayered silty sand and sandy silt, trace clay, occ. fine sand layer, frequent organics Very loose Grey -Free water in samples 6 to 9	190.00 4.30
Sand, medium to coarse, trace silt and gravel Compact Brown	185.60 8.70
CONTINUED ON NEXT PAGE	

Sample Number	Type	Blows/0.3m	Additional Lab. Testing
1	50 DO	6	
2	50 DO	15	
3	50 DO	9	
4	50 DO	8	
5	50 DO	4	
6	50 DO	2	
7	50 DO	2	MH
8	50 DO	3	
9	50 DO	1	
10	50 DO	2	
11	50 DO	23	

Piezometer or standpipe installation: BENTONITE SEAL; SAND; CUTTINGS; CAVED

DEPTH SCALE 1 to 50

LOGGED: GEB
CHECKED: ASP

Golder Associates

J1364007.BHS

PROJECT: 941-1364

RECORD OF BOREHOLE 95-7

SHEET 2 OF 3

LOCATION: SEE FIGURE 2E

BORING DATE: OCT.2-3/95

DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

Boring method: TRACK MOUNTED CME 55 POWER AUGER, 95mm ID HOLLOW STEM AUGERS

CONTINUED FROM PREVIOUS PAGE

Description	Elev. / Depth (m)	Sample Number	Type	Blows/0.3m
	184.10 / 10.20			
Sandy silt Very loose Grey to brown	183.40 / 10.90	12	50 DO	14
Sand, medium to coarse, trace silt and gravel. Compact Brown	181.00 / 13.30	13	50 DO	21
Sandy silt, occ. organics Very loose Grey	179.70 / 14.60	14	50 DO	2
Gravel, some sand and silt Very dense Brown -Free water in sample 15	178.00 / 16.30	15	50 DO	69
Sand, fine, trace silt and clay Very loose to loose Brown to reddish grey		16	50 DO	4
		17	50 DO	3

Piezometer or standpipe installation: CAVED & BENTONITE; BENTONITE SEAL; CAVED

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DEPTH SCALE 1 to 50

Golder Associates

LOGGED: GEB

CHECKED: ASP

DATA INPUT: PS JAN.22/96

SOILMS

PROJECT: 941-1364

RECORD OF BOREHOLE 95-7

LOCATION: SEE FIGURE 2E

BORING DATE: OCT.2-3/95

DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

Boring method: TRACK MOUNTED CME 55 POWER AUGER; 95mm ID HOLLOW STEM AUGERS

DEPTH SCALE METRES	SOIL PROFILE – DESCRIPTION	ELEV. / DEPTH (m)	SAMPLE NUMBER	TYPE	BLOWS/0.3m	WATER CONTENT, PERCENT (Wp 20 40 60 80 Wl)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
20	CONTINUED FROM PREVIOUS PAGE							
21–25	Sand, fine, trace silt and clay Very loose to loose Brown to reddish grey		18	50 DO	3	O	MH	CAVED
		169.31 24.99	19	50 DO	8	O		
25–27	Sand (inferred from easy auger advance)	166.87 27.43						
	END OF BOREHOLE							
28	Note: Sand "blowback" of 0.6m and 2.4m inside augers when sampling at 19m and 25m depths respectively.							Water level in augers at about 13.7m depth below ground surface upon completion of drilling. Water level in piezometer at Elev. 177.2m on October 6,1995.
29								
30								

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m

HYDRAULIC CONDUCTIVITY, k, cm/s

SHEAR STRENGTH Cu, kPa — nat V - + Q - ● rem V - ⊕ U - O

DEPTH SCALE 1 to 50

Golder Associates

LOGGED: GEB

CHECKED: ASP

J1364008.BHS

PROJECT: 941-1364

RECORD OF BOREHOLE 95-8

LOCATION: SEE FIGURE 2F

BORING DATE: OCT.4/95

DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

Boring method: TRACK MOUNTED CME 55 POWER AUGER, 95mm ID HOLLOW STEM AUGERS

Depth (m)	Description	Elev. / Depth (m)	Sample No.	Type	Blows/0.3m	Additional Lab. Testing	Piezometer or Standpipe Installation
0	GROUND SURFACE	193.70					
	Organic sandy silt, Very loose, Dark brown	0.00					
		193.40 / 0.30	1	50 DO	4		CUTTINGS
	Interlayered fine sand, some silt to sandy silt, trace clay, occ. clayey silt seams, frequent organic partings. Very loose. Brown, becoming grey below 1m depth.		2	50 DO	2		
			3	50 DO	1		
			4	50 DO	2		BENTONITE SEAL
			5	50 DO	1		
			6	50 DO	WH		
			7	50 DO	WH		
			8	50 DO	WH		
			9	50 DO	WH	MH	
	Free water noted in samples 1 to 7						
8		185.70 / 8.00	10	50 DO	14		
	Sand, coarse to fine, some silt, trace fine gravel. Occ. grey silt parting layer. Compact. Brown						CUTTINGS
			11	50 DO	10		
10							

CONTINUED ON NEXT PAGE

DEPTH SCALE 1 to 50

Golder Associates

LOGGED: GEB

CHECKED: ASP

PROJECT: 941-1364

RECORD OF BOREHOLE 95-8

SHEET 2 OF 2

LOCATION: SEE FIGURE 2F

BORING DATE: OCT.4/95

DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.3m	WATER CONTENT, PERCENT (Wp — W — Wl; 20 40 60 80)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
10	TRACK MOUNTED CME 55 POWER AUGER, 95mm ID HOLLOW STEM AUGERS	CONTINUED FROM PREVIOUS PAGE								
				183.50						
10–12		Medium to coarse sand and gravel, trace silt and clay, fine to medium sand layers from 10.9m to 11.7m depth. Compact to very dense Brown		10.20	12	50 DO	62	o		CAVED
12				181.10	13	50 DO	15	o	MH	
13–14		Sand, fine to medium, trace gravel and silt Loose Brown		12.60	14	50 DO	4	o		SAND FILTER & CAVED
				179.37						
		END OF BOREHOLE		14.33						Water level in augers at about 12.8m depth below ground surface upon completion of drilling. Water level in piezometer at Elev. 181.6m on October 6,1995.
15										
16										
17										
18										
19										
20										

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m

HYDRAULIC CONDUCTIVITY, k, cm/s

SHEAR STRENGTH Cu, kPa — nat V - + Q - ● rem V - ⊕ U - O

DEPTH SCALE 1 to 50

LOGGED: GEB

CHECKED: ASP

Golder Associates

J1364008.BHS

DATA INPUT: PS JAN 22/96

SOILM6

PROJECT: 941-1364

RECORD OF BOREHOLE 95-9

LOCATION: SEE FIGURE 2G

BORING DATE: SEPT.19-21/95

DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

J1364009.BHS

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE – DESCRIPTION	STRATA PLOT	ELEV. / DEPTH (m)	SAMPLES – NUMBER	TYPE	BLOWS/0.3m
0	TRACK MOUNTED CME 55 POWER AUGER, 108mm ID HOLLOW STEM AUGERS	GROUND SURFACE		178.50			
		TOPSOIL		0.00			
		Silty sand, some rootlets Loose Brown		178.30 0.20	1	50 DO	4
1		Irregularly layered silty clay with grey silt laminations Soft Brown to greyish brown		177.70 0.80	2	50 DO	4
2		Irregularly layered silty clay to clay and clayey silt to silt, trace sand Soft to firm Greyish brown		177.10 1.40	3	50 DO	1
3					4	50 DO	WH
5					5	50 DO	PM
6					6	50 DO	PM
8		CONTINUED ON NEXT PAGE					

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m: 20, 40, 60, 80

SHEAR STRENGTH Cu, kPa: nat V - + ; rem V - ⊕ ; Q - ● ; U - ○

HYDRAULIC CONDUCTIVITY, k, cm/s

WATER CONTENT, PERCENT: Wp —○W— Wl; 20, 40, 60, 80

ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION: CUTTINGS

DEPTH SCALE 1 to 50

DATA INPUT: PS JAN.22/96

SOILM6

LOGGED: GEB

CHECKED: ASP

Golder Associates

J1364009.BHS

PROJECT: 941-1364

RECORD OF BOREHOLE 95-9

SHEET 2 OF 5

LOCATION: SEE FIGURE 2G

BORING DATE: SEPT.19-21/95

DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES: NUMBER	TYPE	BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m; SHEAR STRENGTH Cu, kPa (nat V - +, rem V - ⊕, Q - ●, U - O) 20 40 60 80	HYDRAULIC CONDUCTIVITY, k, cm/s; WATER CONTENT, PERCENT Wp — W — Wl 20 40 60 80	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
8	TRACK MOUNTED CME 55 POWER AUGER; 108mm ID HOLLOW STEM AUGERS	CONTINUED FROM PREVIOUS PAGE									
		Irregularly layered silty clay to clay and clayey silt to silt, trace sand Soft to firm Greyish brown									CUTTINGS
9					7	50 DO	WR				
11					8	75 TO	PH				
12					9	50 DO	PM				
15					10	50 DO	WR				
18		CONTINUED ON NEXT PAGE									

DEPTH SCALE

1 to 50

Golder Associates

LOGGED: GEB

CHECKED: ASP

DATA INPUT: PS JAN 22/96

SOILM8

PROJECT: 941-1364

RECORD OF BOREHOLE 95-9

LOCATION: SEE FIGURE 2G

BORING DATE: SEPT.19-21/95

DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

Boring method: TRACK MOUNTED CME 55 POWER AUGER, 108mm ID HOLLOW STEM AUGERS

CONTINUED FROM PREVIOUS PAGE

DEPTH SCALE METRES	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLE NUMBER	TYPE	BLOWS/0.3m	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
18–22.9	Irregularly layered silty clay to clay and clayey silt to silt, trace sand Soft to firm Greyish brown		11	50 DO	WR		
			12	75 TO	PH	MH C	
			13	50 DO	WR		
22.9–28	Irregularly layered silty clay to clay and clayey silt, trace sand. Stiff Grey to greyish brown, becoming reddish brown with grey silt interlayers below 30.1m depth.	155.60 22.90	14	50 DO	WR		CUTTINGS
			15	50 DO	WR		

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DEPTH SCALE 1 to 50

Golder Associates

LOGGED: GEB

CHECKED: ASP

J1364009 BHS

PROJECT: 941-1364

RECORD OF BOREHOLE 95-9

SHEET 4 OF 5

LOCATION: SEE FIGURE 2G

BORING DATE: SEPT.19-21/95

DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE – DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES – NUMBER	TYPE	BLOWS/0.3m	SHEAR STRENGTH Cu, kPa (nat V +, rem V ⊕)	WATER CONTENT, PERCENT (Wp – W – Wl)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		CONTINUED FROM PREVIOUS PAGE									
28					15	50 DO	WR				
29					16	75 TO	PH				
30								⊕, +			
31					17	50 DO	WR		Wp–W–Wl		
32	TRACK MOUNTED CME 55 POWER AUGER; 108mm ID HOLLOW STEM AUGERS	Irregularly layered silty clay to clay and clayey silt, trace sand. Stiff Grey to greyish brown, becoming reddish brown with grey silt interlayers below 30.1m depth.						⊕, +; ⊕, +			
33											CUTTINGS
34					18	50 DO	WR		O		
35											
36								⊕, +			
37					19	50 DO	WH		O		BENTONITE SEAL
38								⊕, +; ⊕, +			
		CONTINUED ON NEXT PAGE									

DATA INPUT: PS JAN 22/96

DEPTH SCALE

1 to 50

Golder Associates

LOGGED: GEB

CHECKED: ASP

SOILMS

RECORD OF BOREHOLE 95-9

PROJECT: 941-1364
LOCATION: SEE FIGURE 2G
SAMPLER HAMMER, 63.5kg; DROP, 760mm
BORING DATE: SEPT.19-21/95
SHEET 5 OF 5
DATUM: GEODETIC
PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

J1364009.BHS

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. / DEPTH (m)	SAMPLES: NUMBER	TYPE	BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m; SHEAR STRENGTH Cu, kPa (nat V - +, Q - ●, rem V - ⊕, U - ○) 20 40 60 80	HYDRAULIC CONDUCTIVITY, k, cm/s; WATER CONTENT, PERCENT Wp ─○W─ Wl 20 40 60 80	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		CONTINUED FROM PREVIOUS PAGE									
38	TRACK MOUNTED CME 55 POWER AUGER; 108mm ID HOLLOW STEM AUGERS	Varved silty clay to clay and clayey silt, trace sand. Stiff. Grey to greyish brown, becoming reddish brown with grey silt varves below 30.1m depth.									SAND
39					20	75 TO	PH			MH C	
					21	50 DO	WH				
				136.57							
40		END OF BOREHOLE		39.93							Water level at 1.32m above ground surface on October 5,1995.
41											
42											
43											
44											
45											
46											
47											
48											

DATA INPUT: PS JAN 22/96

DEPTH SCALE
1 to 50

Golder Associates

LOGGED: GEB
CHECKED: ASP

SOILM6

RECORD OF BOREHOLE 95-10

PROJECT: 941-1364 | SHEET 1 OF 2
LOCATION: SEE FIGURE 2H | BORING DATE: SEPT.21-22/95 | DATUM: GEODETIC
SAMPLER HAMMER, 63.5kg; DROP, 760mm | PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

J3640010.BHS

BORING METHOD: TRACK MOUNTED CME 55 POWER AUGER, 108mm ID HOLLOW STEM AUGERS

DEPTH SCALE METRES	SOIL PROFILE DESCRIPTION	ELEV. / DEPTH (m)	SAMPLE NUMBER	TYPE	BLOWS/0.3m	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
-2							PIEZO. 95-10B
-1	GROUND SURFACE						
0	Organic silt, trace sand, some grey silty clay. Soft. Dark brown and black	187.80 / 0.00	1	50 DO	3		PIEZO. 95-10A; CUTTINGS; BENTONITE SEAL
0.5	Irregularly layered silty clay and clayey silt, occ. rootlets, some grey silt laminations and inclusions, some sand, occ. gravel. Soft. Greyish/reddish brown to grey.	187.30 / 0.50	2	50 DO	3		CUTTINGS; BENTONITE SEAL
			3	50 DO	5		SAND; PIEZO. 95-10A; BENTONITE SEAL
			4	50 DO	WH		
4	Silty sand and gravel, trace clay. (TILL) Compact, becoming very loose from 7.2m to 8.7m depth. Grey	183.80 / 4.00	5	50 DO	21		
			6	50 DO	16	MH	CAVED
			7	50 DO	14		
			8	50 DO	13		
			9	50 DO	2		
8	CONTINUED ON NEXT PAGE						

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20, 40, 60, 80)
SHEAR STRENGTH Cu, kPa: nat V - +, rem V - ⊕, Q - ●, U - O
HYDRAULIC CONDUCTIVITY, k, cm/s
WATER CONTENT, PERCENT: Wp —O W— Wl (20, 40, 60, 80)

DATA INPUT: PS JAN 22/96

DEPTH SCALE 1 to 50 | Golder Associates | LOGGED: GEB | CHECKED: ASP

SOILM6

PROJECT: 941-1364

RECORD OF BOREHOLE 95-10

SHEET 2 OF 2

LOCATION: SEE FIGURE 2H

BORING DATE: SEPT.21-22/95

DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

BORING METHOD: TRACK MOUNTED CME 55 POWER AUGER, 108mm ID HOLLOW STEM AUGERS

DEPTH SCALE METRES	SOIL PROFILE: DESCRIPTION	ELEV. / DEPTH (m)	SAMPLES: NUMBER	TYPE	BLOWS/0.3m	WATER CONTENT, PERCENT	PIEZOMETER OR STANDPIPE INSTALLATION
	CONTINUED FROM PREVIOUS PAGE						
8	Silty sand and gravel, trace clay. (TILL) Compact, becoming very loose from 7.2m to 8.7m depth. Grey	179.10 / 8.70	9	50 DO	2		
9	Sandy silt, trace clay Compact Grey	178.43 / 9.37					
	Sandstone, weathered (bedrock) Dark reddish brown		10	50 DO	28/0.2	o (~17)	
10							SAND
		176.80 / 11.00		50 DO	46/.03		PIEZO. 95-10B
11	END OF BOREHOLE Refusal to Auger Penetration						Water level in augers at about 3.8m depth below ground surface upon completion of drilling. Water level in piezometer 95-10A at Elev. 187.8m (ground surface) on October 6,1995. Water level at top of 95-10B piezometer pipe (1.52m above ground surface and flowing) on October 6,1995.

Column headings also printed: DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m; SHEAR STRENGTH Cu, kPa (nat V - + Q - ●; rem V - ⊕ U - O) 20 40 60 80; HYDRAULIC CONDUCTIVITY, k, cm/s; WATER CONTENT, PERCENT Wp —O W— Wl 20 40 60 80; ADDITIONAL LAB. TESTING; STRATA PLOT

Depth scale: 8 9 10 11 12 13 14 15 16 17 18

J3640010.BHS

DATA INPUT: PS JAN 22/96

DEPTH SCALE 1 to 50

Golder Associates

LOGGED: GEB

CHECKED: ASP

SOILM6

PROJECT: 941-1364

RECORD OF BOREHOLE 95-11

LOCATION: SEE FIGURE 2H

BORING DATE: SEPT.22/95

DATUM: GEODETIC

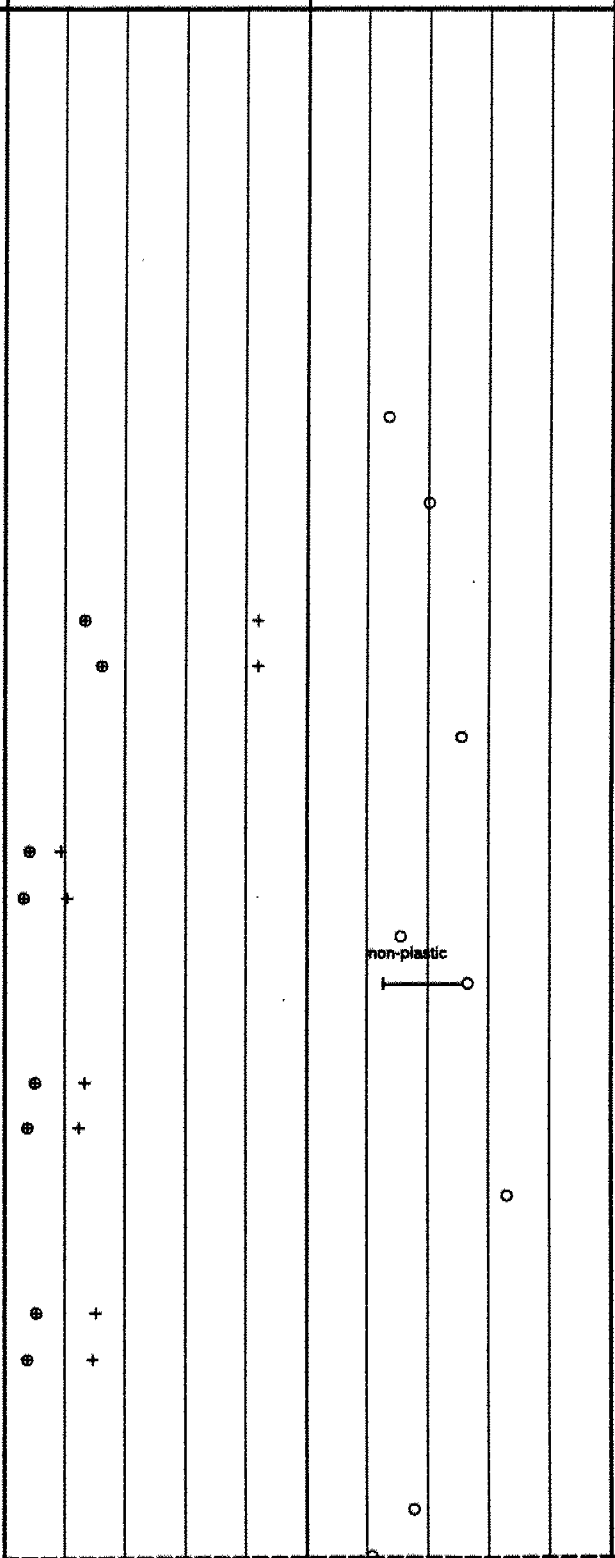
SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

BORING METHOD: TRACK MOUNTED CME 55 POWER AUGER, 108mm ID HOLLOW STEM AUGERS

DESCRIPTION	ELEV. / DEPTH (m)	SAMPLE NUMBER	TYPE	BLOWS/0.3m	PIEZOMETER OR STANDPIPE INSTALLATION
GROUND SURFACE	188.50				
Oragnic silty clay, some sand, Soft Dark greyish brown	0.00	1	50 DO	5	BENTONITE SEAL
Irregularly layered silty clay and clayey silt. Firm to stiff, becoming soft below 2.20m depth. Greyish to reddish brown with grey silt seams and layers.	188.10 / 0.40	2	50 DO	7	
		3	50 DO	WH	CUTTINGS
		4	50 DO	WH	
		5	50 DO	PM	
					BENTONITE SEAL
Interlayered sandy silt to silty sand with occ. grey silty clay layer. Some gravel below 8.7m depth. Very loose to 8.7m becoming compact. Grey	181.30 / 7.20	6	50 DO	WH	CUTTINGS

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m — SHEAR STRENGTH Cu, kPa (nat V - +, rem V - ⊕, Q - ●, U - ○) — HYDRAULIC CONDUCTIVITY, k, cm/s — WATER CONTENT, PERCENT (Wp, W, Wl)



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DEPTH SCALE 1 to 50

LOGGED: GEB

CHECKED: ASP

Golder Associates

J3640011.BHS

DATA INPUT: PS JAN.22/96

SOILM6

PROJECT: 941-1364

RECORD OF BOREHOLE 95-11

SHEET 2 OF 2

LOCATION: SEE FIGURE 2H

BORING DATE: SEPT.22/95

DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

J3640011.BHS

DEPTH SCALE METRES	BORING METHOD	DESCRIPTION	ELEV. DEPTH (m)	NUMBER	TYPE	BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m / SHEAR STRENGTH Cu, kPa	HYDRAULIC CONDUCTIVITY, k, cm/s / WATER CONTENT, PERCENT Wp — W — Wl	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		CONTINUED FROM PREVIOUS PAGE								
8	TRACK MOUNTED CME 55 POWER AUGER, 108mm ID HOLLOW STEM AUGERS	Interlayered sandy silt to silty sand with occ. grey silty clay layer. Some gravel below 8.7m depth. Very loose to 8.7m depth becoming compact. Grey		6	50 DO	WH				CAVED
9				7	50 DO	17				
10			178.30 / 10.20							SAND
		Sandstone, weathered (bedrock) Dark reddish brown	177.88 / 10.62		50 DO	60/ .03				
11		END OF BOREHOLE Refusal to Sampler Penetration								Water level at about 2.0m depth below ground surface upon completion of drilling. Water level in piezometer at Elev. 189.6m (1.1m above ground surface) on October 6,1995.
12										
13										
14										
15										
16										
17										
18										

DATA INPUT: PS JAN.22/96

DEPTH SCALE 1 to 50

Golder Associates

LOGGED: GEB

CHECKED: ASP

SOILM6

J3640012.BHS

PROJECT: 941-1364

RECORD OF BOREHOLE 95-12

SHEET 1 OF 1

LOCATION: SEE FIGURE 2H

BORING DATE: SEPT.25/95

DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

BORING METHOD: TRACK MOUNTED CME 55 POWER AUGER, 108mm ID HOLLOW STEM AUGERS

Column headings: DEPTH SCALE METRES | BORING METHOD | SOIL PROFILE (DESCRIPTION, STRATA PLOT, ELEV. DEPTH (m)) | SAMPLES (NUMBER, TYPE, BLOWS/0.3m) | DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m | SHEAR STRENGTH Cu, kPa (nat V - +, rem V - ⊕, Q - ●, U - O) | HYDRAULIC CONDUCTIVITY, k, cm/s | WATER CONTENT, PERCENT (Wp, W, Wl; 20, 40, 60, 80) | ADDITIONAL LAB. TESTING | PIEZOMETER OR STANDPIPE INSTALLATION

Description	Elev. / Depth (m)	Sample Number	Type	Blows/0.3m	Additional Lab. Testing	Piezometer or Standpipe Installation
GROUND SURFACE	188.80					
TOPSOIL	0.00					
Sand, trace to some gravel; trace silt, occ. organic Loose to compact Brown	0.15	1	50 DO	5		FILL
		2	50 DO	8		BENTONITE SEAL ▽
		3	50 DO	17		CUTTINGS
Silty sand, some gravel, trace clay (TILL) Compact Reddish to greyish brown	186.80 / 2.00	4	50 DO	27	MH	
		5	50 DO	17		
Silty sand, some gravel, trace clay, slightly layered. Loose Reddish to greyish brown.	185.10 / 3.70	6	50 DO	8		CAVED
		7	50 DO	9		
Sand and gravel, trace to some silt, trace clay. Compact Greyish to reddish brown	183.50 / 5.30	8	50 DO	15	MH	
Coarse sand, some silt Loose Greyish to reddish brown	182.80 / 6.00	9	50 DO	8		
Sand and garvel, trace to some silt, trace clay. Compact Greyish to reddish brown	181.60 / 7.20	10	50 DO	105		
END OF BOREHOLE Refusal to Auger and Sampler Penetration (possible boulder) Note: Gravel stuck in sampler and possible boulder likely cause of higher blows recorded in sample 10	180.88 / 7.92					Water level in auger at about 0.5m depth below ground surface upon completion of drilling. Water level in piezometer at Elev. 188.0 on October 6,1995.

DEPTH SCALE 1 to 50

LOGGED: GEB

CHECKED: ASP

Golder Associates

DATA INPUT: PS JAN 22/96

SOILM6

J3640013.BHS

RECORD OF BOREHOLE 95-13

PROJECT: 941-1364 | SHEET 1 OF 2

LOCATION: SEE FIGURE 2H | BORING DATE: SEPT.30/95 | DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg; DROP, 760mm | PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

BORING METHOD: TRACK MOUNTED CME 55 POWER AUGER, 108mm ID HOLLOW STEM AUGERS

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20, 40, 60, 80)

SHEAR STRENGTH Cu, kPa — nat V - + Q - ● ; rem V - ⊕ U - O (20, 40, 60, 80)

HYDRAULIC CONDUCTIVITY, k, cm/s

WATER CONTENT, PERCENT — Wp ├──O W──┤ Wl (20, 40, 60, 80)

SOIL PROFILE – DESCRIPTION	ELEV. / DEPTH (m)
GROUND SURFACE	187.20 / 0.00
Sandy organic silt, Very loose, Dark brown	186.90 / 0.30
Sand, some silt, Very loose, Greyish brown	185.80 / 1.40
Irregularly layered silty clay and clayey silt, occ. gravel, trace organics. Firm. Reddish to greyish brown with grey interlayers, becoming grey with reddish to greyish brown interlayers below 2.7m depth.	183.50 / 3.70
Fine sand, trace to some silt, Very loose, Grey	179.40 / 7.80
Sandy silt, some coarse sand, Loose, Grey	179.20 / 8.00
Silty sand, some gravel, trace clay (TILL), Loose, Grey	178.50 / 8.70
CONTINUED ON NEXT PAGE	

SAMPLE NUMBER	TYPE	BLOWS/0.3m	ADDITIONAL LAB. TESTING
1	50 DO	4	
2	50 DO	WH	
3	50 DO	1	
4	50 DO	WH	
5	50 DO	WR	
6	50 DO	WH	
7	50 DO	WR	MH
8	50 DO	1	
9	50 DO	7	MH

PIEZOMETER OR STANDPIPE INSTALLATION: FILL; CUTTINGS; BENTONITE SEAL; FILTER SAND & CAVED

DEPTH SCALE METRES: -1, 0, 1, 2, 3, 4, 5, 6, 7, 8, 9

DATA INPUT: PS JAN.22/96

DEPTH SCALE 1 to 50

Golder Associates

LOGGED: GEB

CHECKED: ASP

SOILM6

J3640013.BHS

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m / SHEAR STRENGTH Cu, kPa (nat V - +, Q - ●, rem V - ⊕, U - O) 20 40 60 80	HYDRAULIC CONDUCTIVITY, k, cm/s / WATER CONTENT, PERCENT Wp —O W— Wl 20 40 60 80	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		CONTINUED FROM PREVIOUS PAGE									
9	TRACK MOUNTED CME 55 POWER AUGER, 108mm ID HOLLOW STEM AUGERS	Sand, trace gravel to sand and fine gravel Graded/layered appearance Loose Grey		177.40 9.80	10	50 DO	9		O (approx. 20)		
10–12		Sand, trace gravel to sand and gravel inferred by augering		175.00 12.20							FILTER SAND & CAVED
		END OF BOREHOLE Note: Sand "blowback" of 4m inside augers when sampling at 10.7m depth (therefore no sample taken)									Water level at top of piezometer pipe (0.15m above ground surface) on October 6,1995.

Depth scale: 9, 10, 11, 12, 13, 14, 15, 16, 17, 18, 19

DATA INPUT: PS JAN 22/96

J36400P1.BHS

PROJECT: 941-1364

RECORD OF PROBE HOLE 95-PH1

SHEET 1 OF 2

LOCATION: SEE FIGURE 3E

BORING DATE: SEPT.26/95

DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

BORING METHOD: TRACK MOUNTED CME 55 POWER AUGER, 108mm ID HOLLOW STEM AUGERS

Soil Profile Description	Elev. (m)	Depth (m)	Sample Number	Type	Blows/0.3m
GROUND SURFACE	176.90	0.00			
Organic silt, some sand, fibrous. Very soft. Dark brown		0.15	1	50 DO	1
Silty clay, trace sand, frequent sand and organic matter pockets. Very soft. Brownish grey to grey	175.50	1.40	2	50 DO	1
Sandy silt, trace organic. Loose. Grey	174.70	2.20	3	50 DO	7
Irregularly layered silty clay and clayey silt, trace sand, occ. fibrous peat layers. Soft. Brownish grey to grey	173.50	3.40	4	50 DO	WH
Irregularly layered silty clay to clay and clayey silt to silt, interlayers of silty sand and organic inclusions. Soft. Greyish to reddish brown			5	50 DO	WH
			6	50 DO	WH
			7	50 DO	PM

Additional lab testing: >365 (at ground surface)

Chart columns: DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m; SHEAR STRENGTH Cu, kPa (nat V - +, rem V - ⊕, Q - ●, U - ○) 20 40 60 80; HYDRAULIC CONDUCTIVITY, k, cm/s; WATER CONTENT, PERCENT (Wp, W, Wl) 20 40 60 80; ADDITIONAL LAB. TESTING; PIEZOMETER OR STANDPIPE INSTALLATION

Depth scale (metres): 0 1 2 3 4 5 6 7 8 9 10

CONTINUED ON NEXT PAGE

DEPTH SCALE 1 to 50

Golder Associates

LOGGED: GEB

CHECKED: ASP

DATA INPUT: PS JAN.22/96

3MPROBE

J35400P1.BHS

PROJECT: 941-1364

RECORD OF PROBE HOLE 95-PH1

SHEET 2 OF 2

LOCATION: SEE FIGURE 3E

BORING DATE: SEPT.26/95

DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m / SHEAR STRENGTH Cu, kPa (nat V - +, Q - ●, rem V - ⊕, U - O) 20 40 60 80	HYDRAULIC CONDUCTIVITY, k, cm/s / WATER CONTENT, PERCENT Wp W Wl 20 40 60 80	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		CONTINUED FROM PREVIOUS PAGE									
10 – 12.80	TRACK MOUNTED CME 55 POWER AUGER, 108mm ID HOLLOW STEM AUGERS	Irregularly layered silty clay to clay and clayey silt to silt, interlayers of silty sand and organic inclusions. Soft Greyish to reddish brown		164.10 / 12.80	8	50 DO	PM	⊕ +	O		
		END OF BOREHOLE									Water level in open hole at about 2.4m depth (but still rising) upon completion of drilling.

Depth scale: 10, 11, 12, 13, 14, 15, 16, 17, 18, 19, 20

DATA INPUT: PS JAN 22/96

DEPTH SCALE
1 to 50

Golder Associates

LOGGED: GEB
CHECKED: ASP

SMPROBE

PROJECT: 941-1364

RECORD OF PROBE HOLE 95-PH2

LOCATION: SEE FIGURE 3E

BORING DATE: SEPT.26/95

DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

J36400P2.BHS

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. / DEPTH (m)	SAMPLES: NUMBER	TYPE	BLOWS/0.3m	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		GROUND SURFACE		176.80					
0	TRACK MOUNTED CME 55 POWER AUGER; 108mm ID HOLLOW STEM AUGERS	Organic silt, fibrous Very soft Black		0.00 176.55 0.25					
		Organic sandy silt, some gravel, occ. cobble Very soft Brown		176.35 0.45	1	50 DO	PM	ORG =77%	BENTONITE SEAL
2					2	50 DO	PM	MH	CUTTINGS
3					3	50 DO	PM		
4		Irregularly layered silty clay to clay and clayey silt to silt, trace sand, trace organic matter from 0.45m to 1.10m depth. Soft to firm Brownish grey to greyish brown							BENTONITE SEAL SAND BENTONITE SEAL
5					4	50 DO	PM		
6					5	50 DO	PM		
7									CUTTINGS
8									
9									
10		CONTINUED ON NEXT PAGE							

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m

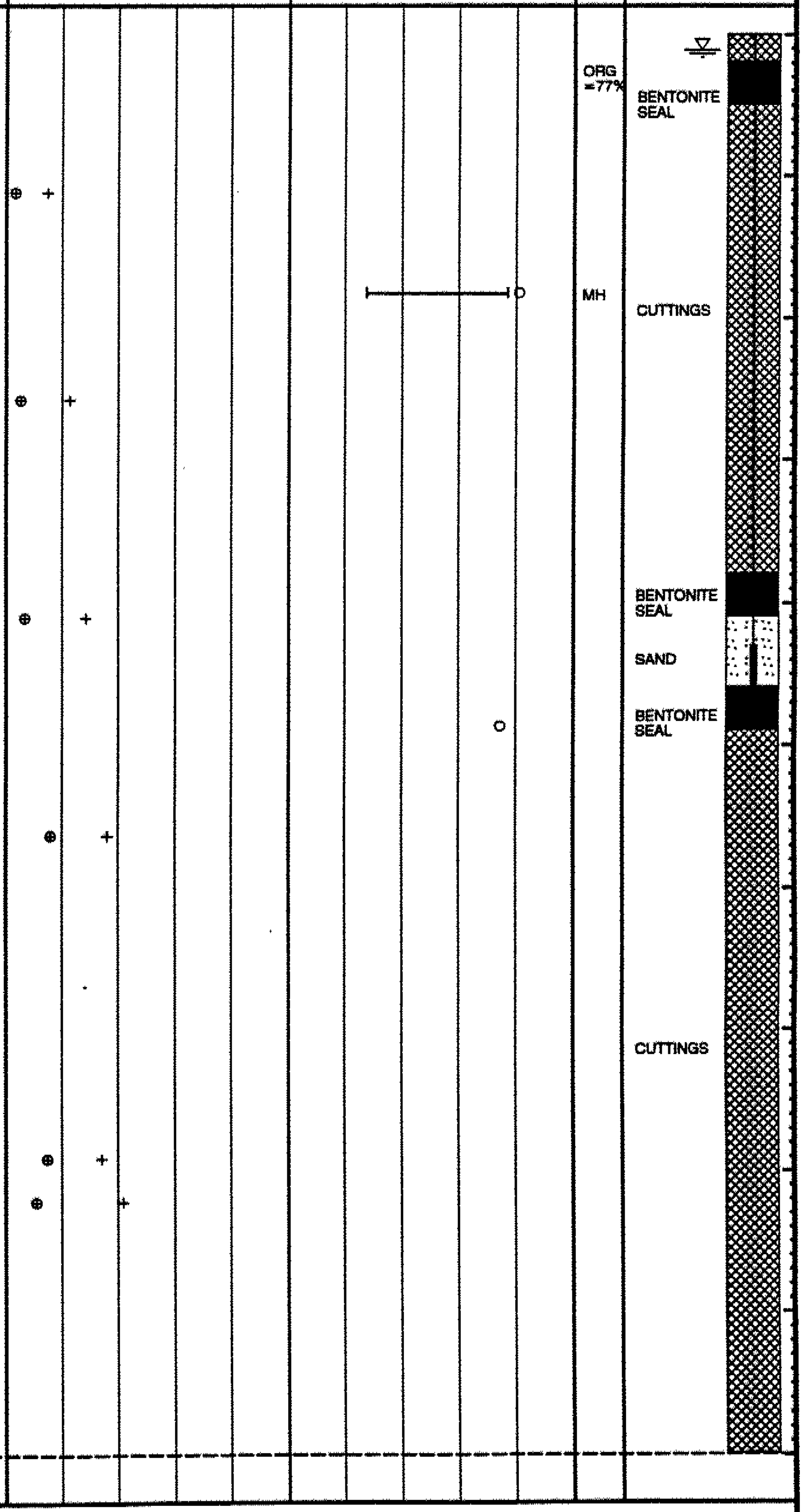
SHEAR STRENGTH Cu, kPa — nat V - + Q - ● rem V - ⊕ U - O

20 40 60 80

HYDRAULIC CONDUCTIVITY, k, cm/s

WATER CONTENT, PERCENT Wp ├──O W──┤ Wl

20 40 60 80



DATA INPUT: PS JAN.22/96

SMPROBE

DEPTH SCALE

1 to 50

Golder Associates

LOGGED: GEB

CHECKED: ASP

PROJECT: 941-1364

RECORD OF PROBE HOLE 95-PH2

SHEET 2 OF 2

LOCATION: SEE FIGURE 3E

BORING DATE: SEPT.26/95

DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE – DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES – NUMBER	TYPE	BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m; SHEAR STRENGTH Cu, kPa (nat V - +, rem V - ⊕, Q - ●, U - O) 20 40 60 80	HYDRAULIC CONDUCTIVITY, k, cm/s; WATER CONTENT, PERCENT Wp —O W— Wl 20 40 60 80	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		CONTINUED FROM PREVIOUS PAGE									
10–11	TRACK MOUNTED CME 55 POWER AUGER; 108mm ID HOLLOW STEM AUGERS	Irreugularly layered silty clay to clayey silt, trace sand, occ. grey silt inclusion below 9.0m depth. Soft to firm Brownish grey		164.91	6	50 DO	1		O (approx. 57)		CUTTINGS
12		END OF BOREHOLE		11.89							Water level in open hole at about 10.7m depth below ground surface upon completion of drilling Water level in piezometer at Elev. 176.7m on October 6,1995.
13											
14											
15											
16											
17											
18											
19											
20											

DEPTH SCALE

1 to 50

Golder Associates

LOGGED: GEB

CHECKED: ASP

J36400P3.BHS

PROJECT: 941-1364

RECORD OF PROBE HOLE 95-PH3

SHEET 1 OF 1

LOCATION: SEE FIGURE 3A

BORING DATE: SEPT.27/95

DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.3m	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
0	TRACK MOUNTED CME 55 POWER AUGER, 108mm ID HOLLOW STEM AUGERS	GROUND SURFACE		193.80					
		TOPSOIL		0.00					
		Silt, some sand, trace clay, some rootlets. Loose. Grey and brown with frequent oxidized silt interlayers		0.15	1	50 DO	4		
1					2	50 DO	6		
		Clayey silt with sand. Soft. Grey to brown		192.00 / 1.80	3	50 DO	2	MH	
2		Irregularly layered silty clay to clay and clayey silt, trace sand. Soft. Greyish to reddish brown with occ. grey silt inclusion.		191.60 / 2.20	4	50 DO	4		
3					5	50 DO	WH		
5					6	50 DO	PM		
6				187.09 / 6.71	7	50 DO	PM		
7		END OF BOREHOLE							Water level in open hole at about 2.4m depth below ground surface upon completion of drilling.
8									
9									
10									

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m

SHEAR STRENGTH Cu, kPa — nat V - + Q - ● rem V - ⊕ U - O (20, 40, 60, 80)

HYDRAULIC CONDUCTIVITY, k, cm/s

WATER CONTENT, PERCENT Wp — W — Wl (20, 40, 60, 80)

DATA INPUT: PS JAN 22/96

DEPTH SCALE 1 to 50

Golder Associates

LOGGED: GEB

CHECKED: ASP

SMPROBE

PROJECT: 941-1364

RECORD OF PROBE HOLE 95-PH4

SHEET 1 OF 1

LOCATION: SEE FIGURE 3A

BORING DATE: SEPT.27/95

DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

BORING METHOD: TRACK MOUNTED CME 55 POWER AUGER, 108mm ID HOLLOW STEM AUGERS

DEPTH SCALE METRES	SOIL PROFILE DESCRIPTION	ELEV. / DEPTH (m)	SAMPLE NUMBER	TYPE	BLOWS/0.3m	SHEAR STRENGTH Cu, kPa (nat V - +, rem V - ⊕, Q - ●, U - ○)	WATER CONTENT, PERCENT (Wp, W, Wl)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
0	GROUND SURFACE	192.30							
	TOPSOIL	0.00							
	Silt, some sand, trace clay, trace organics Very loose to loose Frequent oxidized fissures Brownish grey to grey	192.00 0.30	1	50 DO	2		○		
1			2	50 DO	8		○		
2			3	50 DO	6		○	MH	
	Interlayered, grey sandy silt to sand and greyish to reddish brown silty clay. Soft/very loose	190.10 2.20	4	50 DO	3		○		
3			5	50 DO	2		○		
4			6	50 DO	3				
5	Irregularly layered silty clay to clay and clayey silt to silt, trace sand. Firm Greyish to reddish brown	187.40 4.90	7	50 DO	WH		○		
						⊕ +			
6			8	50 DO	PM		○		
	EDN OF BOREHOLE	185.59 6.71							Water level in augers at about 5.2m depth below ground surface upon completion of drilling.
7									
8									
9									
10									

DEPTH SCALE 1 to 50

Golder Associates

LOGGED: GEB

CHECKED: ASP

J38400P4.BHS

DATA INPUT: PS JAN.22/96

SMPROBE

J36400P5.BHS

PROJECT: 941-1364

RECORD OF PROBE HOLE 95-PH5

SHEET 1 OF 2

LOCATION: SEE FIGURE 3E

BORING DATE: SEPT.30/95

DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

Boring method: TRACK MOUNTED CME 55 POWER AUGER, 108mm ID HOLLOW STEM AUGERS

DEPTH SCALE METRES	DESCRIPTION	ELEV. / DEPTH (m)	SAMPLE NUMBER	TYPE	BLOWS/0.3m	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
0	GROUND SURFACE	180.20					
	Organic sandy silt Soft Black	0.00	1	50 DO	4	ORG =26%	BENTONITE SEAL
	Sand to sandy silt, some organics Very loose Brown	179.90 0.30					
	Irregularly layered silty clay to clay and clayey silt to silt, trace sand, grey silt interlayers below 4.1m depth. Firm Greyish to reddish brown	179.70 0.50	2	50 DO	1		
			3	50 DO	PM		
			4	50 DO	WH		CUTTINGS
			5	50 DO	PM		
			6	75 TO	PH		
			7	50 DO	PM		CAVED
8	CONTINUED ON NEXT PAGE						

CONTINUED ON NEXT PAGE

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m: 20 40 60 80

SHEAR STRENGTH Cu, kPa — nat V - + Q - ● rem V - ⊕ U - O

HYDRAULIC CONDUCTIVITY, k, cm/s

WATER CONTENT, PERCENT Wp —O W— Wl: 20 40 60 80

DEPTH SCALE 1 to 50

Golder Associates

LOGGED: GEB

CHECKED: ASP

DATA INPUT: PS JAN.22/96

SMPROBE

PROJECT: 941-1364

LOCATION: SEE FIGURE 3E

SAMPLER HAMMER, 63.5kg; DROP, 760mm

RECORD OF PROBE HOLE 95-PH5

BORING DATE: SEPT.30/95

DATUM: GEODETIC

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE / SHEAR STRENGTH Cu, kPa	HYDRAULIC CONDUCTIVITY / WATER CONTENT, PERCENT	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
	TRACK MOUNTED CME 55 POWER AUGER 108mm ID HOLLOW STEM AUGERS	CONTINUED FROM PREVIOUS PAGE									
8		Irregularly layered silty clay to clay and clayey silt to silt, trace sand, Grey silt interlayers below 4.1m depth. Firm Greyish to reddish brown		171.10 / 9.10	7	50 DO	PM	⊕ +			
9											BENTONITE SEAL
		Silty sand, some gravel, trace to some clay (TILL) Compact Grey			8	50 DO	17		o		CAVED
10											SAND
											SAND
11				168.92 / 11.28	9	50 DO	20		o	MH	
		END OF BOREHOLE									Water level in augers at about 5.9m depth below ground surface upon completion of drilling. Water level at top of piezometer pipe (1.0m above ground surface and flowing) on October 6,1995.
12											
13											
14											
15											
16											
17											
18											

Shear strength legend: nat V - +, rem V - ⊕, Q - ●, U - O. Scales: 20, 40, 60, 80. Water content: Wp —O W— Wl; 20, 40, 60, 80.

DEPTH SCALE
1 to 50

Golder Associates

LOGGED: GEB
CHECKED: ASP

J3640OPS.BHS

DATA INPUT: PS JAN 22/96

SMPROBE

SITE LOCATION PLAN

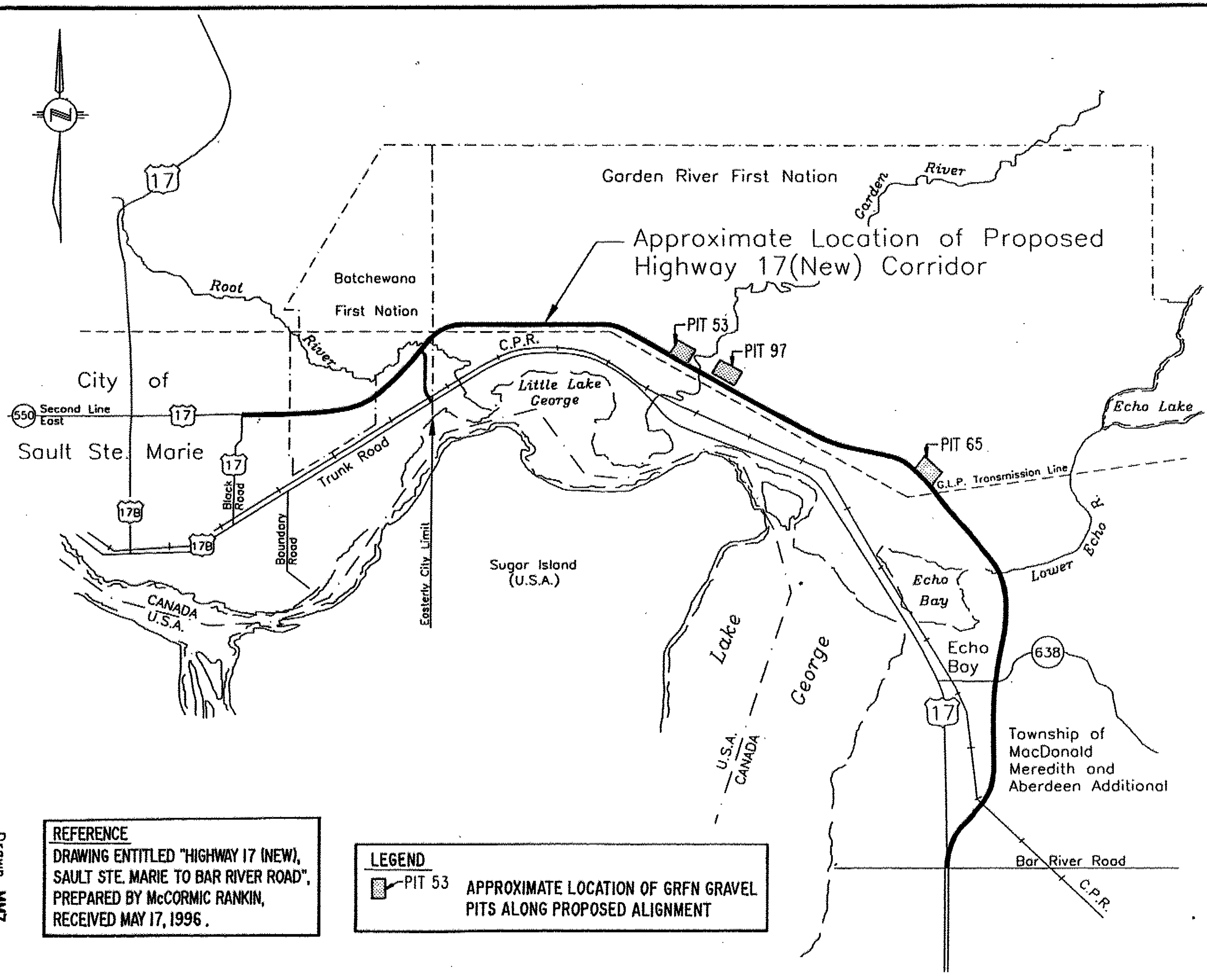


FIGURE 1A

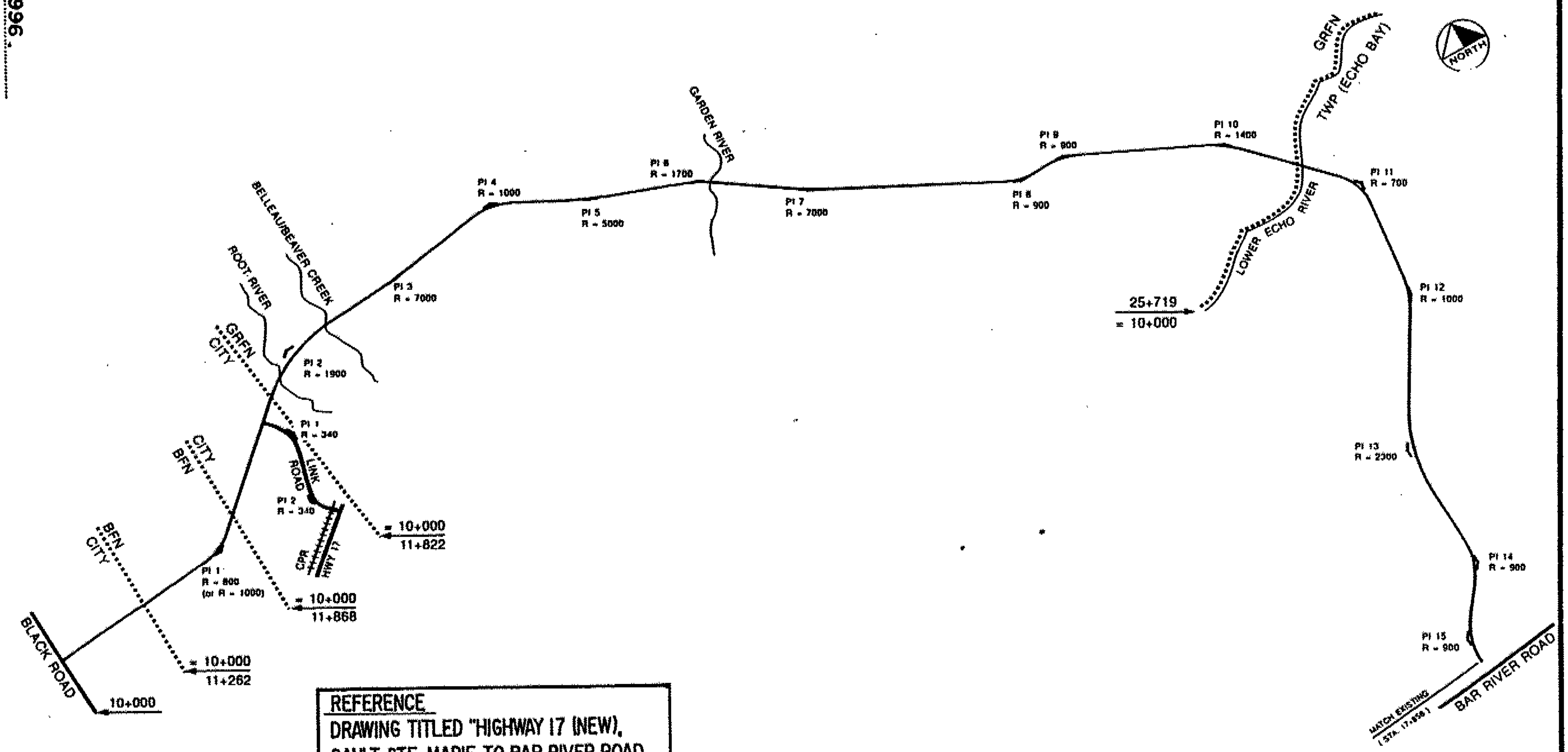
Golder Associates

Date JUNE, 1996.
Project 941-1364

Drawn MMZ
Chkd. [signature]

Date JUNE 1996
Project 941-1364

Golder Associates



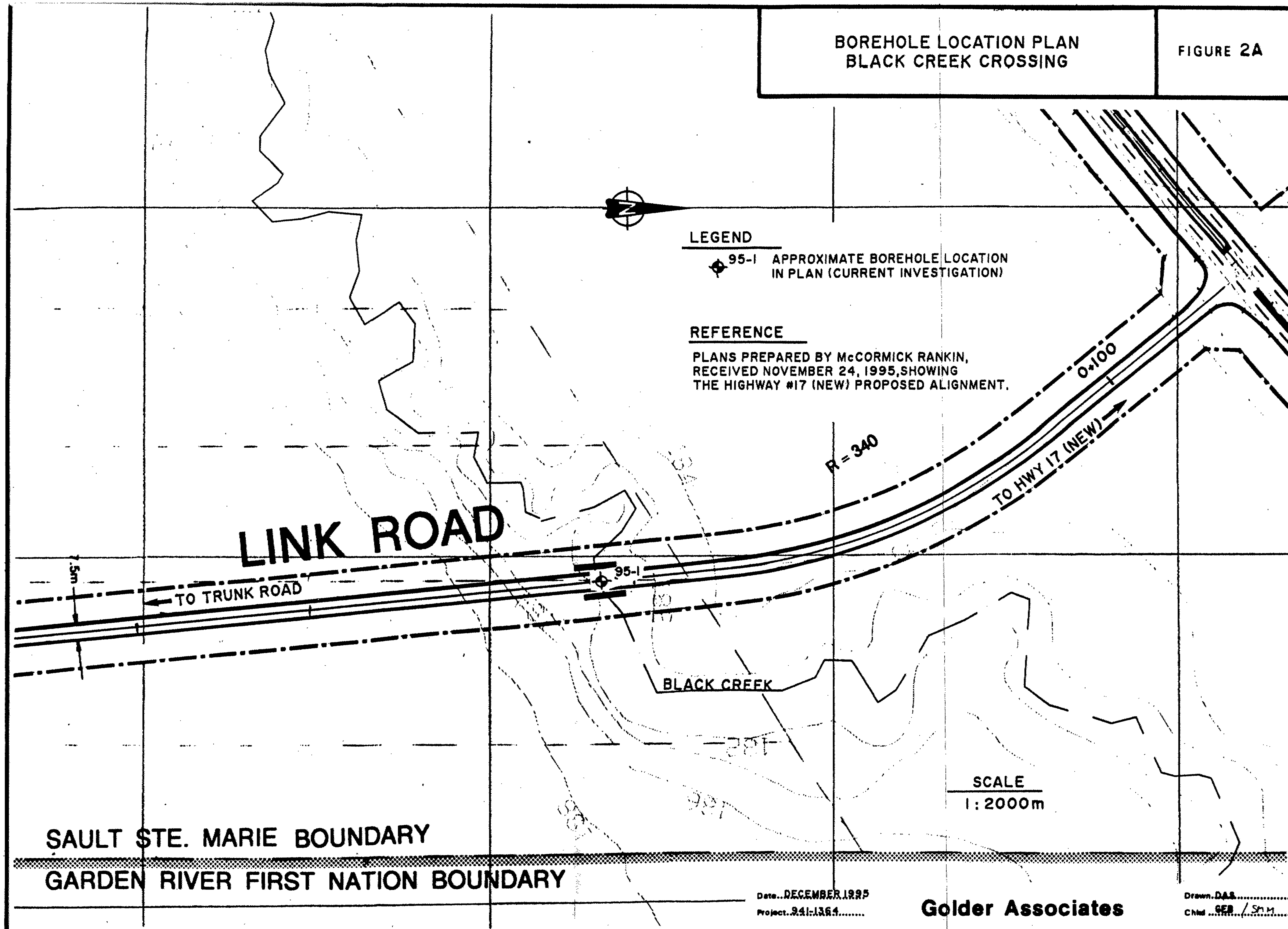
REFERENCE
DRAWING TITLED "HIGHWAY 17 (NEW), SAULT STE. MARIE TO BAR RIVER ROAD, HORIZONTAL ALIGNMENT", PREPARED BY McCORMICK RANKIN, RECEIVED MAY 17, 1996.

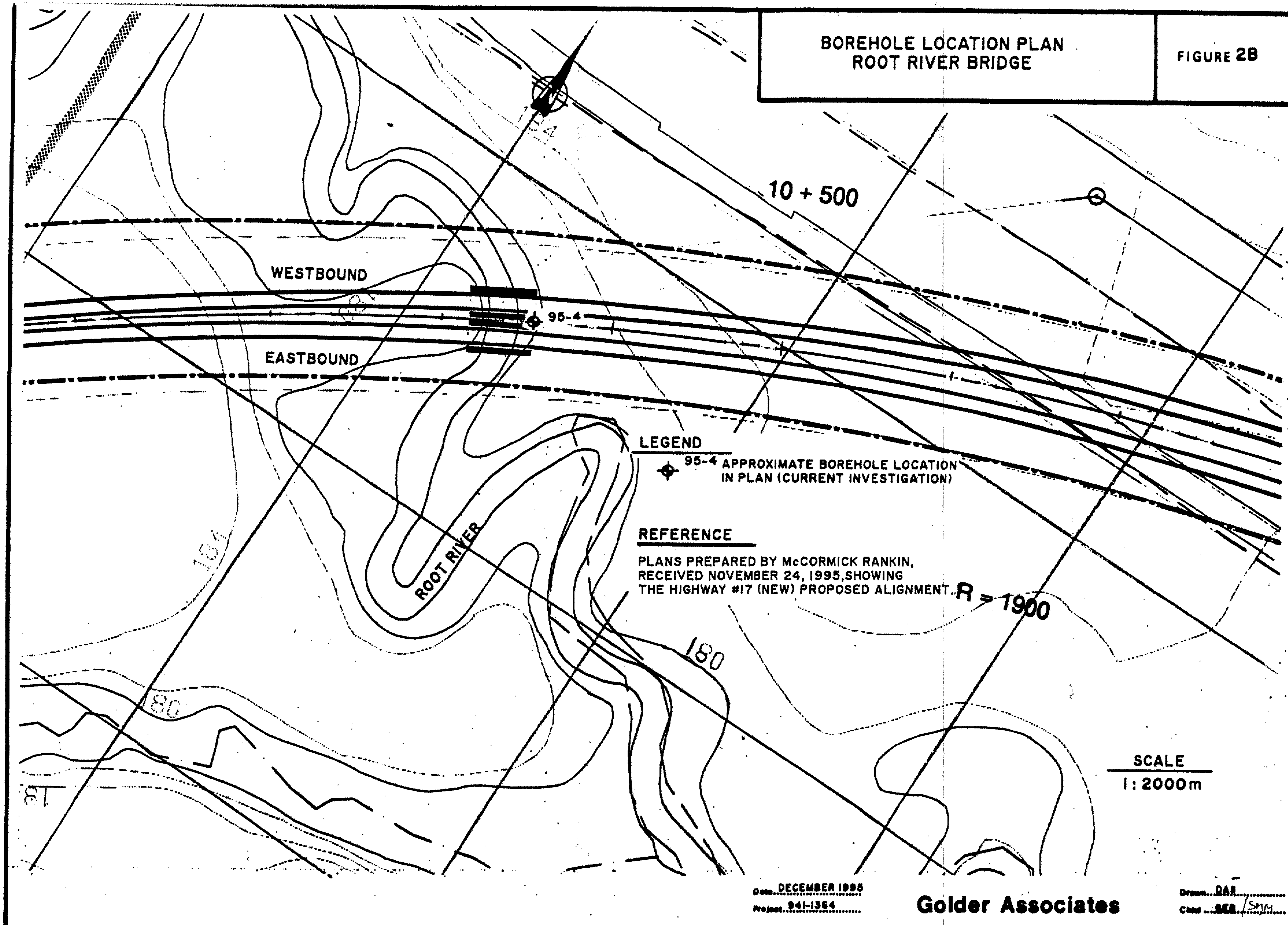
NTS

Drawn MMZ
Chkd. [signature]

HORIZONTAL ALIGNMENT

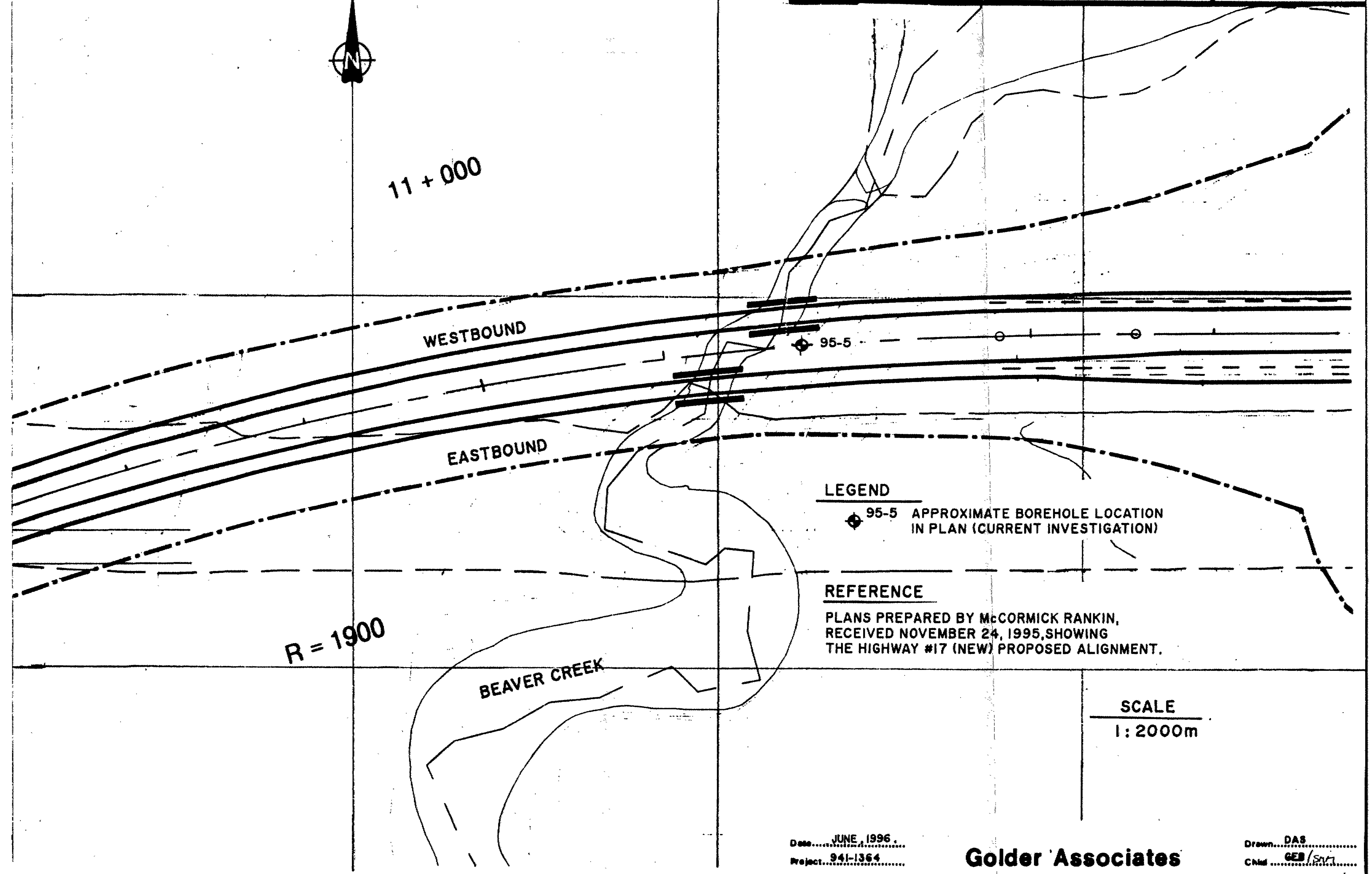
FIGURE 1B


BOREHOLE LOCATION PLAN
BLACK CREEK CROSSING
FIGURE 2A
LEGEND
95-1 APPROXIMATE BOREHOLE LOCATION IN PLAN (CURRENT INVESTIGATION)
REFERENCE
PLANS PREPARED BY McCORMICK RANKIN, RECEIVED NOVEMBER 24, 1995, SHOWING THE HIGHWAY #17 (NEW) PROPOSED ALIGNMENT.
0+100
R = 340
TO HWY 17 (NEW)
LINK ROAD
95-1
TO TRUNK ROAD
7.5m
BLACK CREEK
185
186
SCALE
1 : 2000m
SAULT STE. MARIE BOUNDARY
GARDEN RIVER FIRST NATION BOUNDARY
Date DECEMBER 1995
Project 941-1364
Golder Associates
Drawn DAS
Chkd GEB / SMM


BOREHOLE LOCATION PLAN
ROOT RIVER BRIDGE
FIGURE 2B
10 + 500
WESTBOUND
95-4
EASTBOUND
ROOT RIVER
LEGEND
95-4 APPROXIMATE BOREHOLE LOCATION IN PLAN (CURRENT INVESTIGATION)
REFERENCE
PLANS PREPARED BY McCORMICK RANKIN, RECEIVED NOVEMBER 24, 1995, SHOWING THE HIGHWAY #17 (NEW) PROPOSED ALIGNMENT.
R = 1900
180
180
SCALE
1:2000m
Date DECEMBER 1995
Project 941-1364
Golder Associates
Drawn DAS
Chkd GEB/SMM

BOREHOLE LOCATION PLAN
BELLEAU/BEAVER CREEK BRIDGE

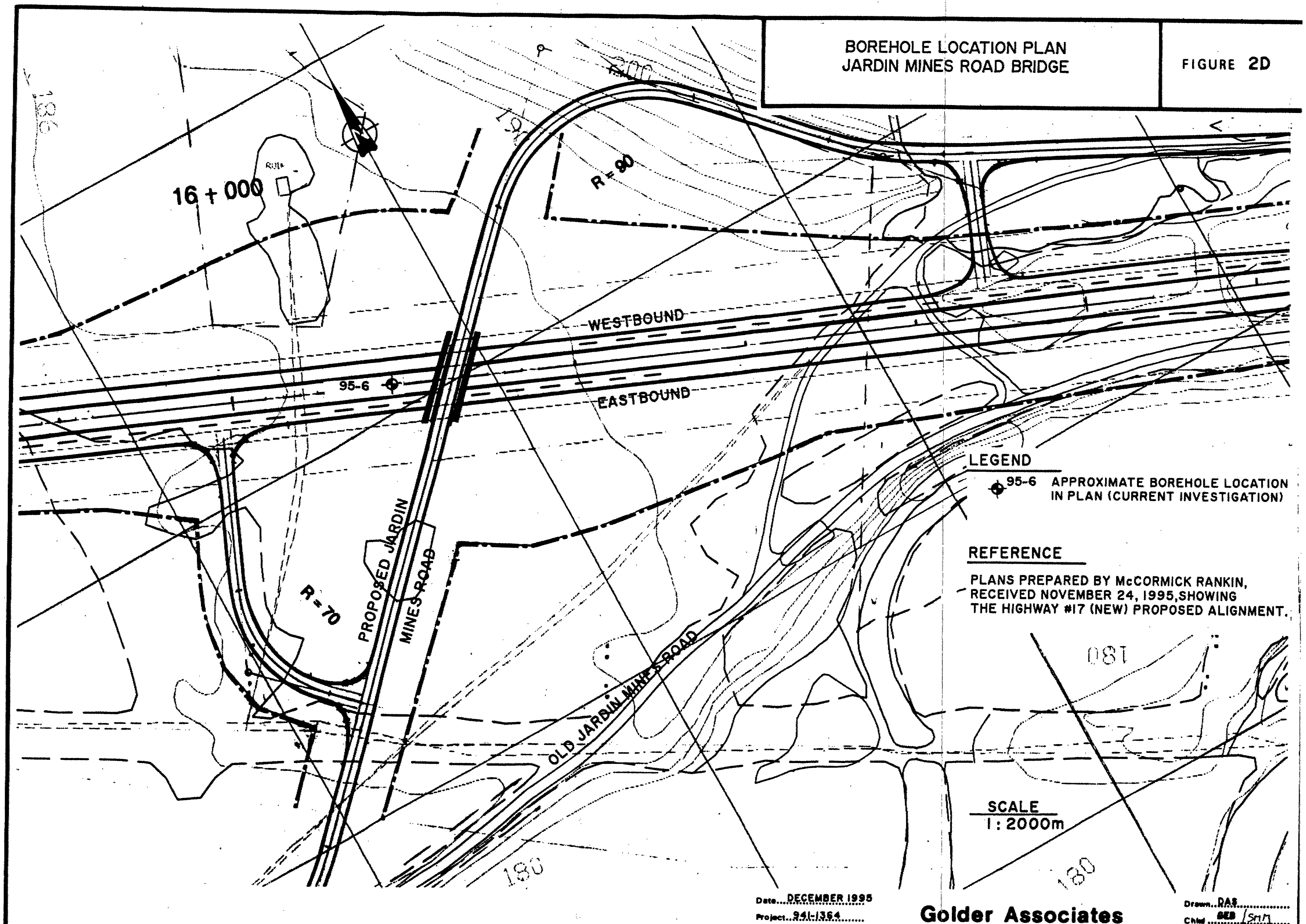
FIGURE 2C



Date JUNE, 1996.
Project 941-1364

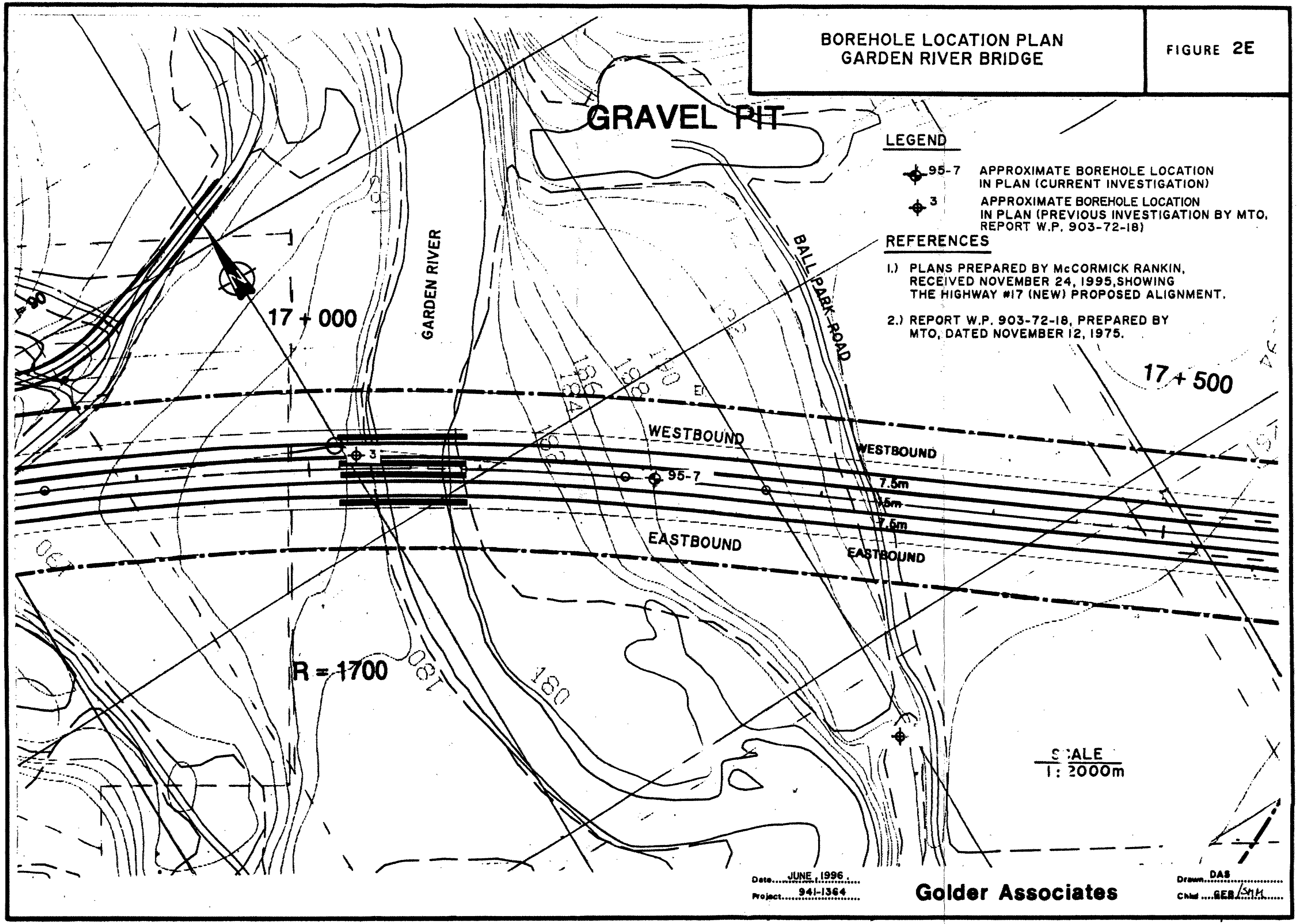
Golder Associates

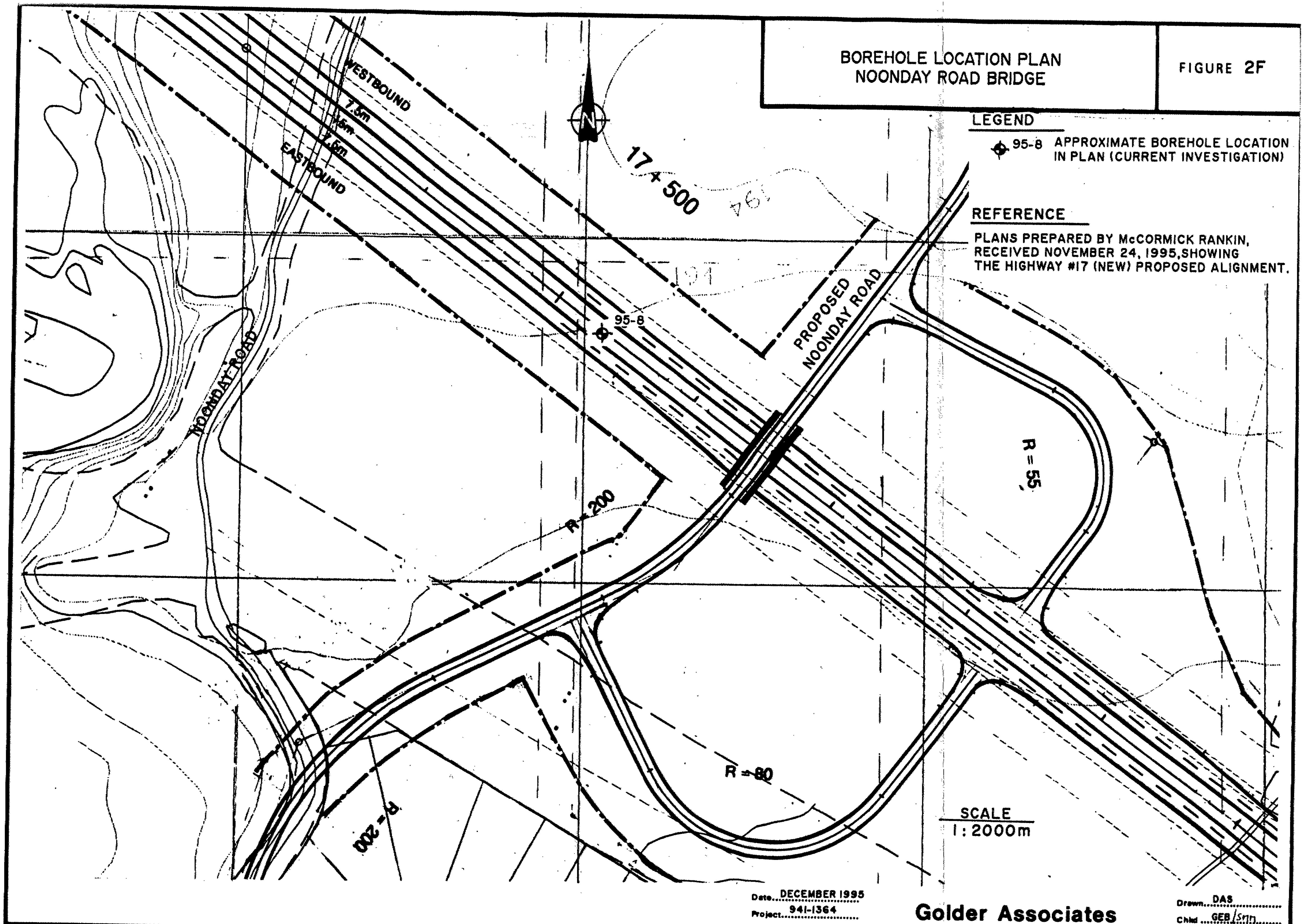
Drawn DAS
Chkd GEB/SnM



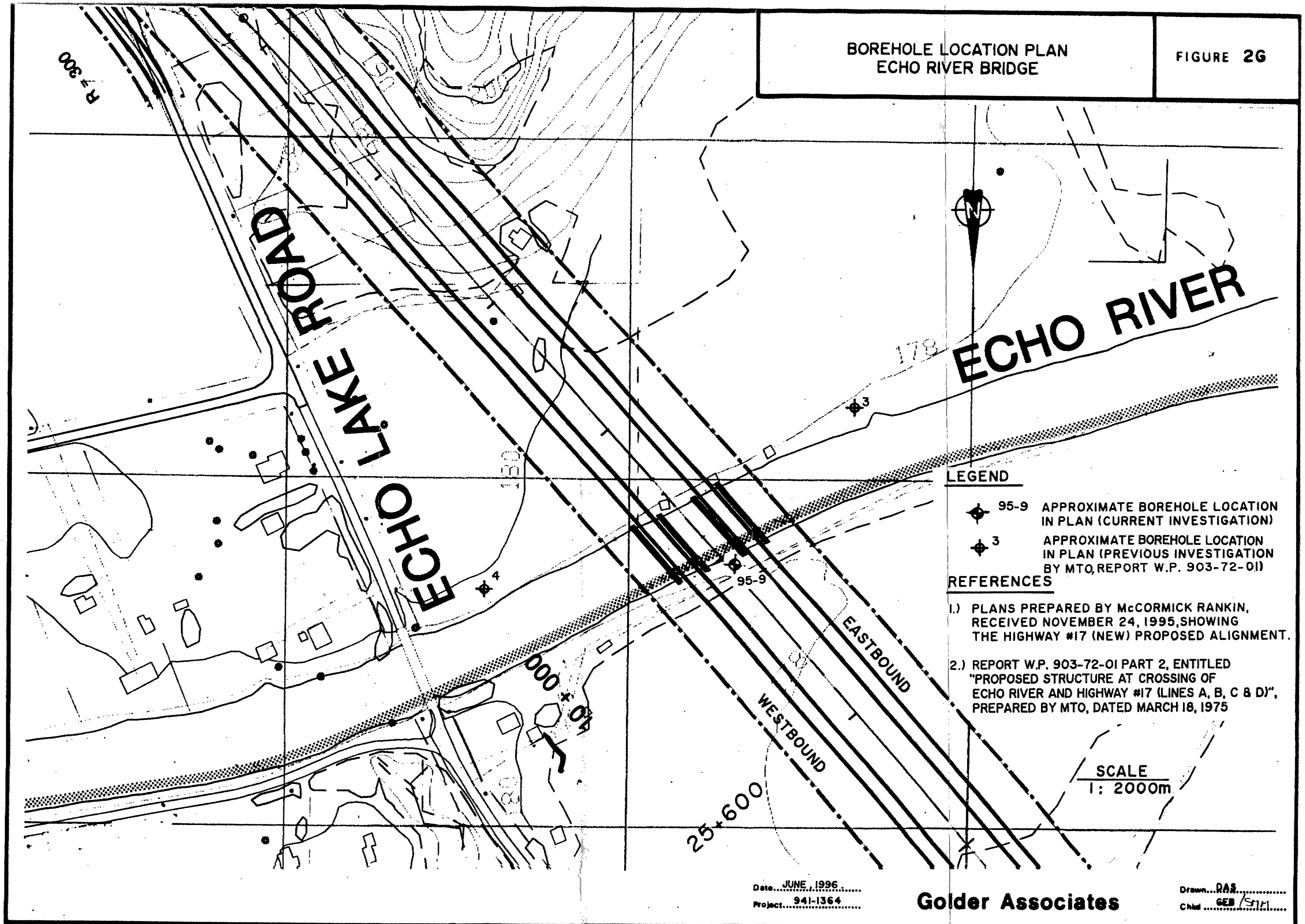
BOREHOLE LOCATION PLAN
JARDIN MINES ROAD BRIDGE

FIGURE 2D

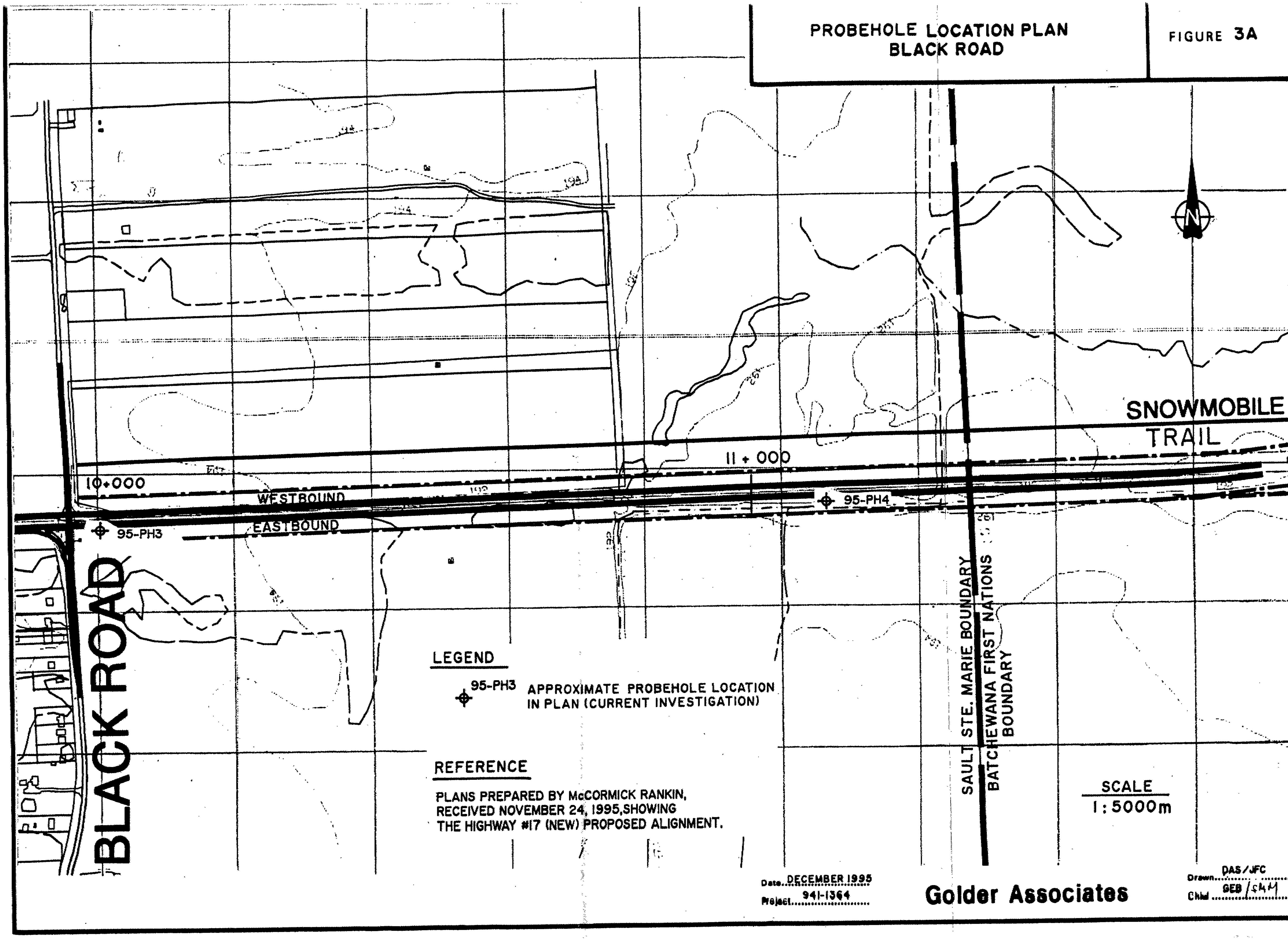

BOREHOLE LOCATION PLAN
GARDEN RIVER BRIDGE
FIGURE 2E
GRAVEL PIT
LEGEND
95-7 APPROXIMATE BOREHOLE LOCATION IN PLAN (CURRENT INVESTIGATION)
3 APPROXIMATE BOREHOLE LOCATION IN PLAN (PREVIOUS INVESTIGATION BY MTO, REPORT W.P. 903-72-18)
REFERENCES
1.) PLANS PREPARED BY McCORMICK RANKIN, RECEIVED NOVEMBER 24, 1995,SHOWING THE HIGHWAY #17 (NEW) PROPOSED ALIGNMENT.
2.) REPORT W.P. 903-72-18, PREPARED BY MTO, DATED NOVEMBER 12, 1975.
GARDEN RIVER
BALL PARK ROAD
17 + 000
17 + 500
WESTBOUND
EASTBOUND
WESTBOUND
EASTBOUND
7.5m
15m
7.5m
95-7
3
R = 1700
180
190
SCALE
1 : 2000m
Date JUNE 1996
Project 941-1364
Golder Associates
Drawn DAS
Chkd GEB



BOREHOLE LOCATION PLAN
NOONDAY ROAD BRIDGE
FIGURE 2F
LEGEND
95-8 APPROXIMATE BOREHOLE LOCATION IN PLAN (CURRENT INVESTIGATION)
REFERENCE
PLANS PREPARED BY McCORMICK RANKIN, RECEIVED NOVEMBER 24, 1995, SHOWING THE HIGHWAY #17 (NEW) PROPOSED ALIGNMENT.
WESTBOUND
7.5m
15m
7.5m
EASTBOUND
17+500
95-8
PROPOSED NOONDAY ROAD
NOONDAY ROAD
R = 200
R = 55
R = 80
R = 200
SCALE
1:2000m
Date DECEMBER 1995
Project 941-1364
Golder Associates
Drawn DAS
Chkd GEB/smn


BOREHOLE LOCATION PLAN
ECHO RIVER BRIDGE
FIGURE 2G
R=300
ECHO LAKE ROAD
ECHO RIVER
178
180
95-9
3
4
EASTBOUND
WESTBOUND
40+000
25+600
LEGEND
95-9 APPROXIMATE BOREHOLE LOCATION IN PLAN (CURRENT INVESTIGATION)
3 APPROXIMATE BOREHOLE LOCATION IN PLAN (PREVIOUS INVESTIGATION BY MTO, REPORT W.P. 903-72-01)
REFERENCES
1.) PLANS PREPARED BY McCORMICK RANKIN, RECEIVED NOVEMBER 24, 1995, SHOWING THE HIGHWAY #17 (NEW) PROPOSED ALIGNMENT.
2.) REPORT W.P. 903-72-01 PART 2, ENTITLED "PROPOSED STRUCTURE AT CROSSING OF ECHO RIVER AND HIGHWAY #17 (LINES A, B, C & D)", PREPARED BY MTO, DATED MARCH 18, 1975
SCALE
1: 2000m
Date JUNE 1996
Project 941-1364
Golder Associates
Drawn DAS
Chkd GEB

OVERSIZE DRAWING(S)

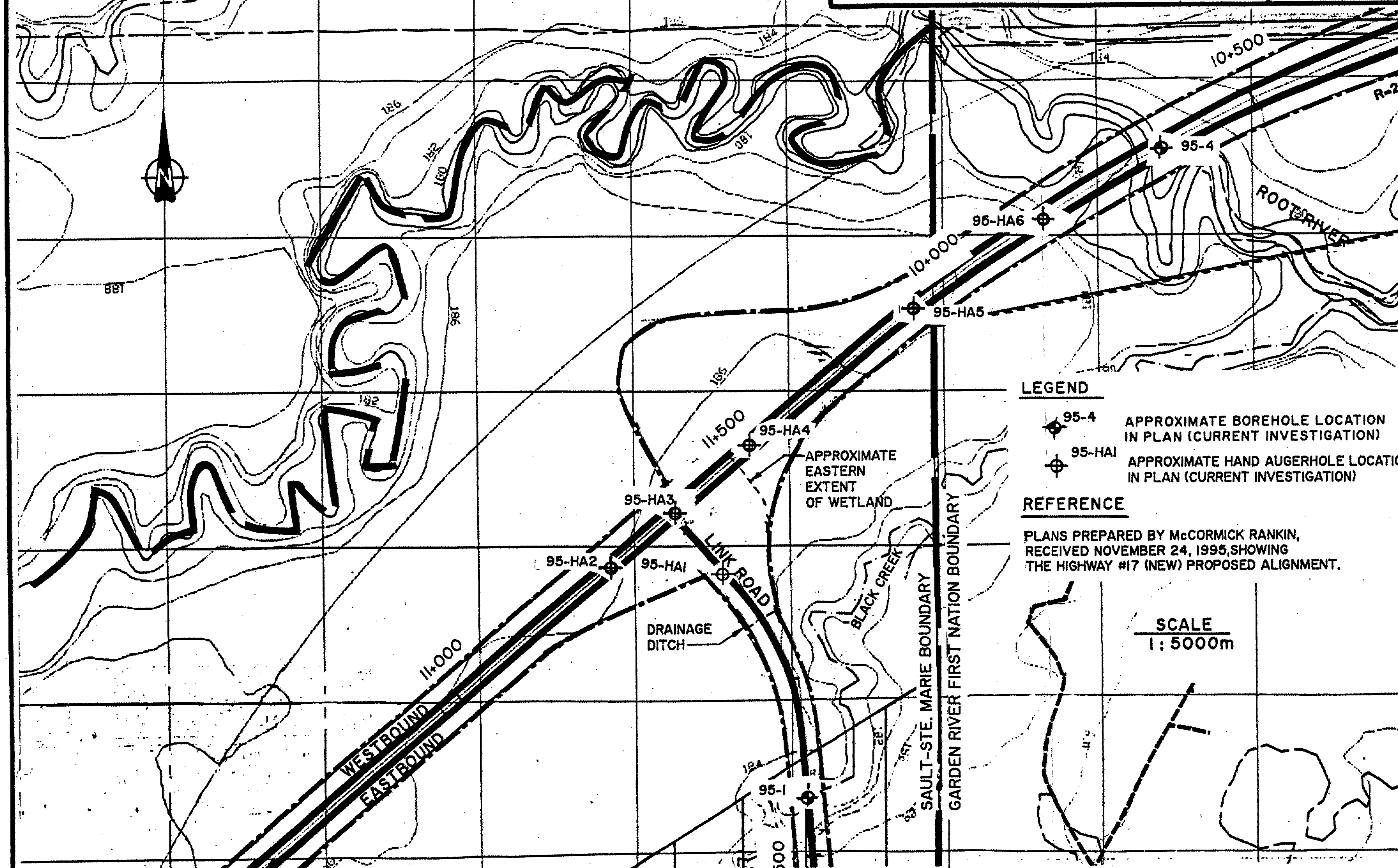


PROBEHOLE LOCATION PLAN
BLACK ROAD

FIGURE 3A

PROBEHOLE LOCATION PLAN
ROOT RIVER

FIGURE: 3B



LEGEND

95-4 APPROXIMATE BOREHOLE LOCATION IN PLAN (CURRENT INVESTIGATION)

95-HAI APPROXIMATE HAND AUGERHOLE LOCATION IN PLAN (CURRENT INVESTIGATION)

REFERENCE

PLANS PREPARED BY McCORMICK RANKIN, RECEIVED NOVEMBER 24, 1995, SHOWING THE HIGHWAY #17 (NEW) PROPOSED ALIGNMENT.

SCALE
1 : 5000m

Date DECEMBER 1995
Project 941-1364

Golder Associates

Drawn DAS
Chkd GEB / SMM

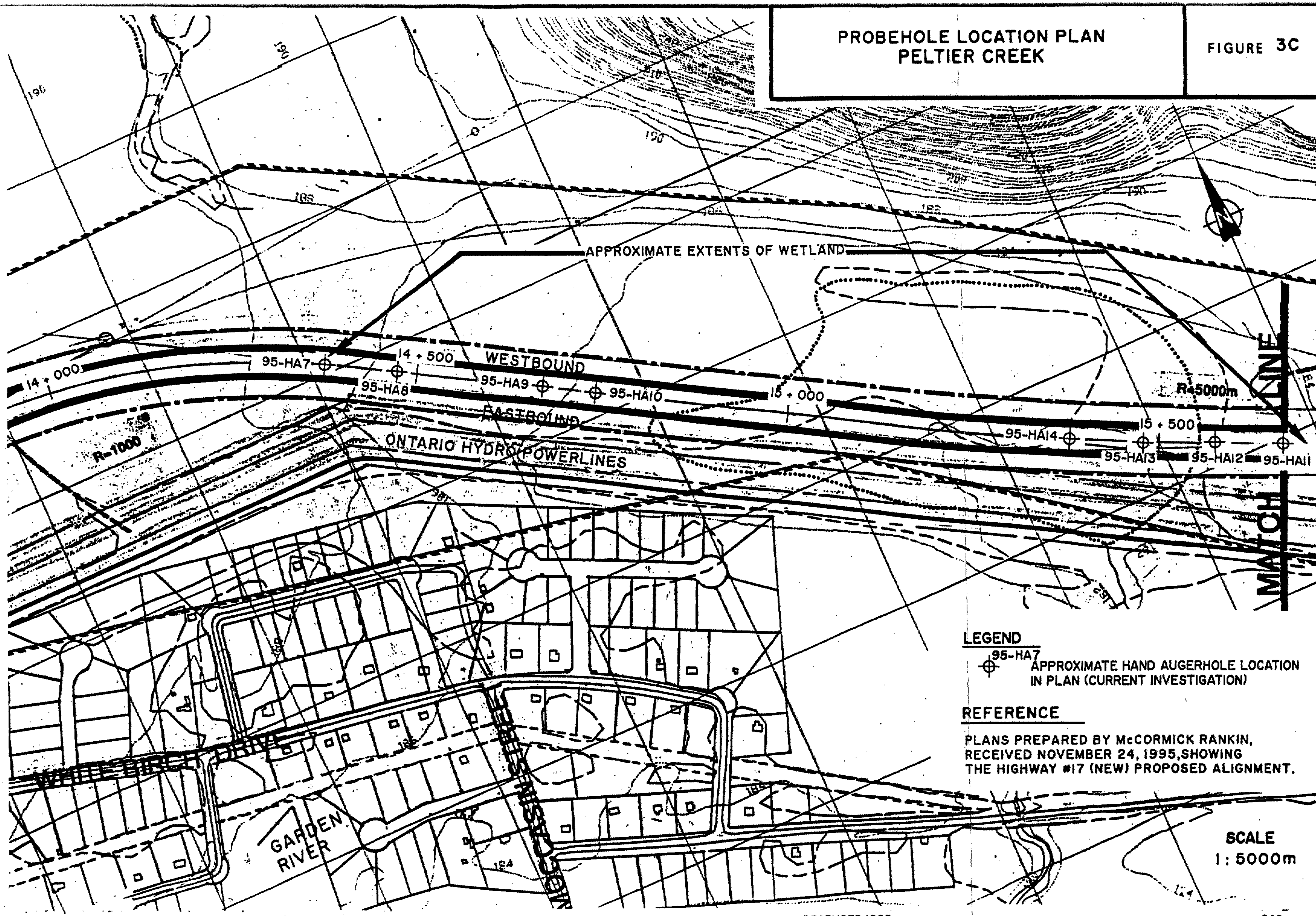
Summary of Augerhole Results
Link Road Wetland (Figure 3B)
Hwy 17(new) - Sault Ste. Marie to Bar River Road

Hand Augerhole Number	Approximate Location	Depth m			Material
95-HA1	0+100	0.00	to	0.30	So. dk. brn. pt.
		0.30	to	0.61	So. grey cly. silt.
	0.61 m; End of hole				water ponded at ground surface
95-HA2	11+175	0.00	to	0.10	So. dk. brn. pt.
		0.10	to	0.61	So. grey cly. silt.
	0.61 m; End of hole				water near ground surface
95-HA3	11+375	0.00	to	0.15	So. dk. brn. pt.
		0.15	to	0.61	So. grey cly. silt.
	0.61 m; End of hole				wet
95-HA4	11+505	0.00	to	0.08	So. dk. brn. pt.
		0.08	to	0.46	So. grey cly. silt.
		0.46	to	0.61	Fi. brn. sty. clay
	0.61 m; End of hole				
95-HA5	11+805	0.00	to	0.05	So. dk. brn. org. silt
		0.05	to	0.18	So. brn. cly. silt
		0.18	to	0.41	So. grey cly. silt
		0.41	to	0.61	Fi. brn. sty. clay
	0.61 m; End of hole				water below 0.25 m depth
95-HA6	10+190	0.00	to	0.15	So. brn. cly. silt, sm. sand
		0.15	to	0.30	Lse. brn. sdy. silt
		0.30	to	0.61	Lse. brn. sty. sand
		0.61	to	1.40	Lse. brn. sand, sm. silt
		1.40	to	1.52	St. grey cly. silt
	1.52 m; End of hole				water below 1.4 m depth

Notes:
1) Hand augerhole locations shown on Figure 3B.
2) Area becomes dry east of approximately Sta. 11+485 and south of approximately Sta. 0+120.
3) Wetland extent west of Sta. 11+175 was not determined.

PROBEHOLE LOCATION PLAN
PELTIER CREEK

FIGURE 3C



LEGEND

95-HA7 APPROXIMATE HAND AUGERHOLE LOCATION IN PLAN (CURRENT INVESTIGATION)

REFERENCE

PLANS PREPARED BY McCORMICK RANKIN, RECEIVED NOVEMBER 24, 1995, SHOWING THE HIGHWAY #17 (NEW) PROPOSED ALIGNMENT.

SCALE
1 : 5000m

Date DECEMBER 1995
Project 941-1364

Golder Associates

Drawn DAS
Chkd GEB / SMM

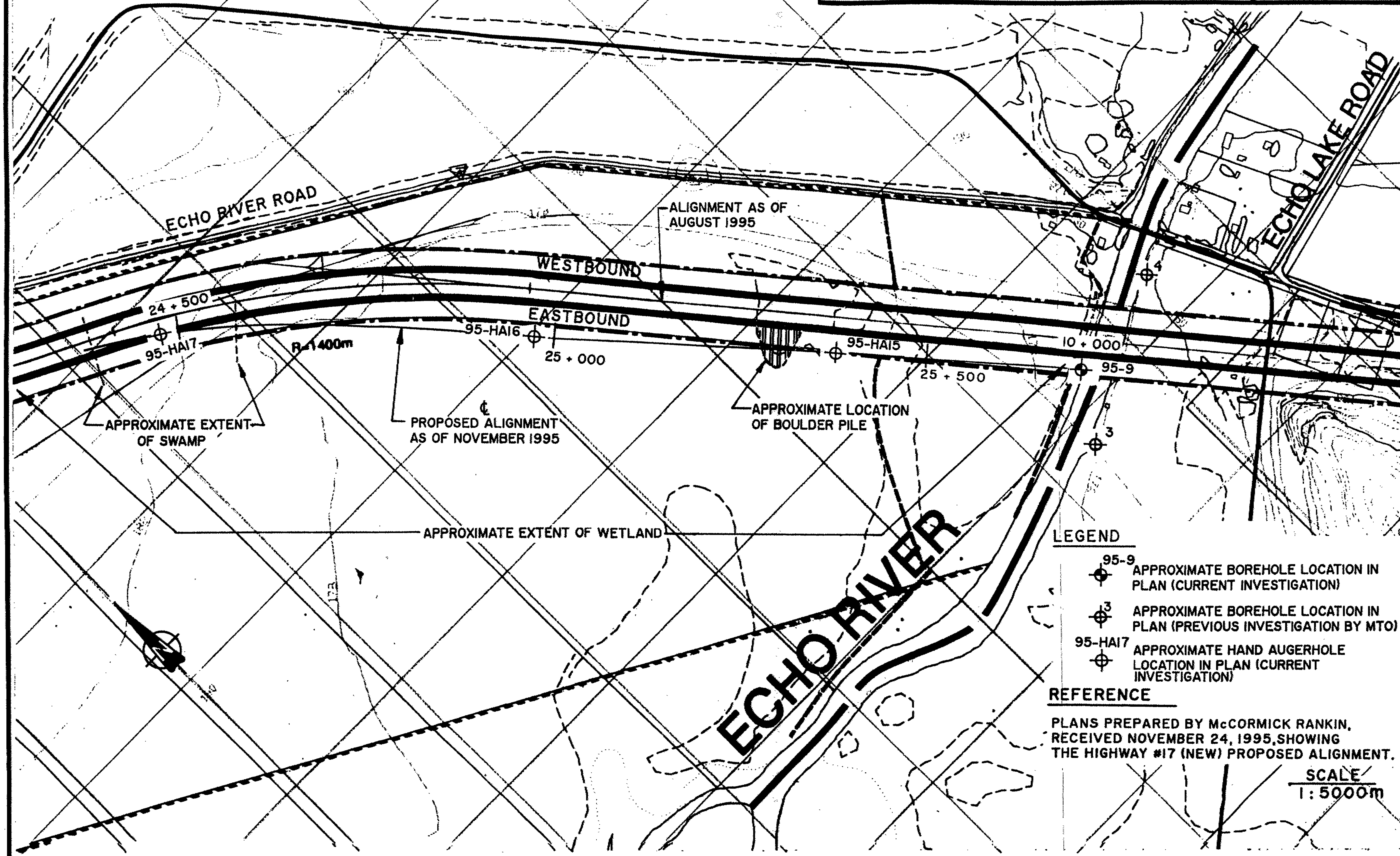
Summary of Augerhole Results
Peltier Creek Wetland (Figure 3C)
Hwy 17(new) - Sault Ste. Marie to Bar River Road

Hand Augerhole Number	Approximate Location	Depth m			Material
95-HA7	14+375	0.00	to	0.10	So. dk. brn. pt.
		0.10	to	0.61	So. brn. cly. silt, sm. sand
	0.61 m; End of hole				wet then very wet below 0.2 m depth
95-HA 8	14+475	0.00	to	0.23	So. dk. brn. pt.
		0.23	to	0.61	So. grey cly. silt
	0.61 m; End of hole				
95-HA9	14+675	0.00	to	0.13	So. dk. brn. pt.
		0.13	to	0.36	So. grey cly. silt
		0.36	to	0.61	Lse. grey sty. sand
	0.61 m; End of hole				water below 0.1 m depth
95-HA10	14+750	0.00	to	0.41	So. dk. brn. pt.
		0.41	to	1.52	So. grey sty. clay
	1.52 m; End of hole				water up to 0.15 m above ground
95-HA14	15+375	0.00	to	0.86	So. dk. brn. pt.
		0.86	to	1.22	So. brn. org. silt
		1.22	to	1.52	So. grn/grey sty. clay, sm. org.
	1.52 m; End of hole				
95-HA13	15+475	0.00	to	0.89	So. dk. brn. pt.
		0.89	to	1.22	So. brn. org. silt
		1.22	to	1.52	So. grey sty. clay, sm. org.
	1.52 m; End of hole				water up to 0.1 m above ground
95-HA12	15+575	0.00	to	0.79	So. dk. brn. pt.
		0.79	to	1.07	So. brn./grey sty. clay, sm. org.
		1.07	to	1.52	So. grey sty. clay
	1.52 m; End of hole				water up to 0.1 m above ground
95-HA11	15+675	0.00	to	0.79	So. dk. brn. pt.
		0.79	to	1.17	So. brn. org. silt
		1.17	to	1.52	So. grey cly. silt, sm. sand & org.
	1.52 m; End of hole				

Notes:
1) Hand augerhole locations shown on Figure 3C.
2) Area becomes drier west of approximately Sta. 14+675 and east of approximately Sta. 15+615.
3) Area becomes wet again between approximately Sta. 14+375 and Sta. 14+475.
4) Area of muskeg exists from eastern extent of wetland to approximately Sta. 15+755.

PROBEHOLE LOCATION PLAN
ECHO RIVER

FIGURE 3D



LEGEND

95-9 APPROXIMATE BOREHOLE LOCATION IN PLAN (CURRENT INVESTIGATION)

3 APPROXIMATE BOREHOLE LOCATION IN PLAN (PREVIOUS INVESTIGATION BY MTO)

95-HAI7 APPROXIMATE HAND AUGERHOLE LOCATION IN PLAN (CURRENT INVESTIGATION)

REFERENCE

PLANS PREPARED BY McCORMICK RANKIN, RECEIVED NOVEMBER 24, 1995, SHOWING THE HIGHWAY #17 (NEW) PROPOSED ALIGNMENT.

SCALE 1 : 5000m

Date DECEMBER 1995
Project 941-1364

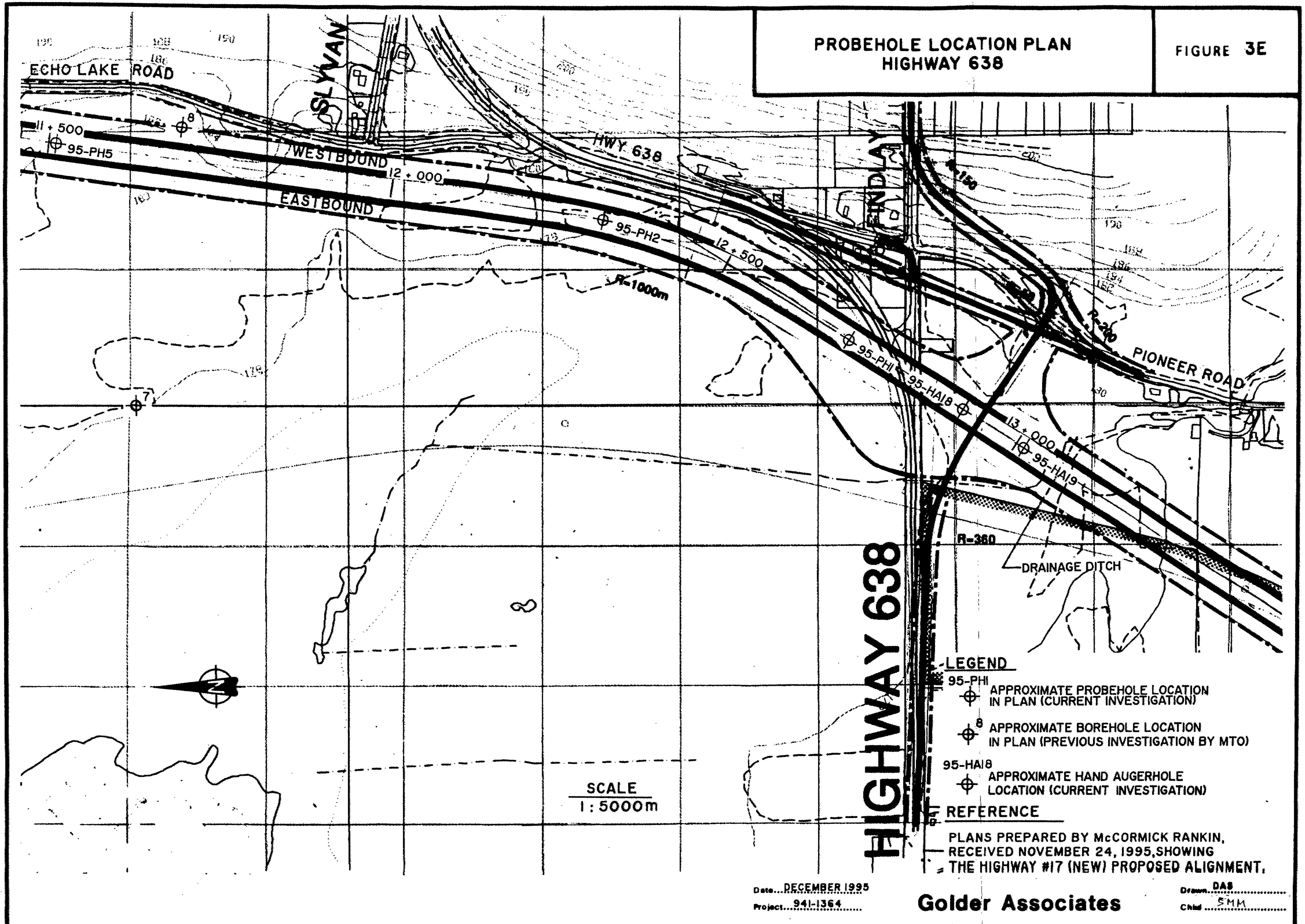
Golder Associates

Drawn DAS
Chkd GEB / SMM

Summary of Augerhole Results
Echo River Wetland (Figure 3D)
Hwy 17(new) - Sault Ste. Marie to Bar River Road

Hand Augerhole Number	Approximate Location	Depth m			Material
95-HA15	25+375	0.00	to	0.15	So. blk. org. silt
		0.15	to	0.23	Lse. grey sty. sand
		0.23	to	0.61	Fi. brn. sty. clay
	0.61 m; End of hole				water ponded on surface
95-HA16	25+975	0.00	to	0.18	So. brn. org. silt
		0.18	to	0.36	Lse. brn. sdy. silt, sm. org.
		0.36	to	0.61	Fi. brn. sty. clay
	0.61 m; End of hole				very wet
95-HA17	24+475	0.00	to	0.30	So. dk. brn. pt.
		0.30	to	0.38	Com. grey sty. sand
		0.38	to	0.61	Fi. brn. sty. clay
	0.61 m; End of hole				water up to 0.3 m above ground

Notes:
1) Hand augerhole locations shown on Figure 3D.
2) Area becomes wet south of approximately Sta. 24+175 and continues to approximately Sta. 25+705.
3) Noticeably wetter between approximately Sta. 24+375 and Sta. 24+575.
4) Surface water observed flowing westerly near approximately Sta. 24+985.
5) Area of muskeg exists between approximately Sta. 25+175 and the southern extent of the wetland.
6) Mound of boulders exists between approximately Sta. 25+275 and Sta. 25+325.


PROBEHOLE LOCATION PLAN
HIGHWAY 638
FIGURE 3E
ECHO LAKE ROAD
SLYVAN
FINDLAY
HWY 638
WESTBOUND
EASTBOUND
11 + 500
12 + 000
12 + 500
13 + 000
95-PH5
95-PH2
95-PH1
95-HA18
95-HA19
R=1000m
R=150
R=200
R=360
PIONEER ROAD
DRAINAGE DITCH
HIGHWAY 638
SCALE
1:5000m
LEGEND
95-PH1
APPROXIMATE PROBEHOLE LOCATION IN PLAN (CURRENT INVESTIGATION)
APPROXIMATE BOREHOLE LOCATION IN PLAN (PREVIOUS INVESTIGATION BY MTO)
95-HA18
APPROXIMATE HAND AUGERHOLE LOCATION (CURRENT INVESTIGATION)
REFERENCE
PLANS PREPARED BY McCORMICK RANKIN, RECEIVED NOVEMBER 24, 1995, SHOWING THE HIGHWAY #17 (NEW) PROPOSED ALIGNMENT.
Date DECEMBER 1995
Project 941-1364
Golder Associates
Drawn DAS
Chkd SMM

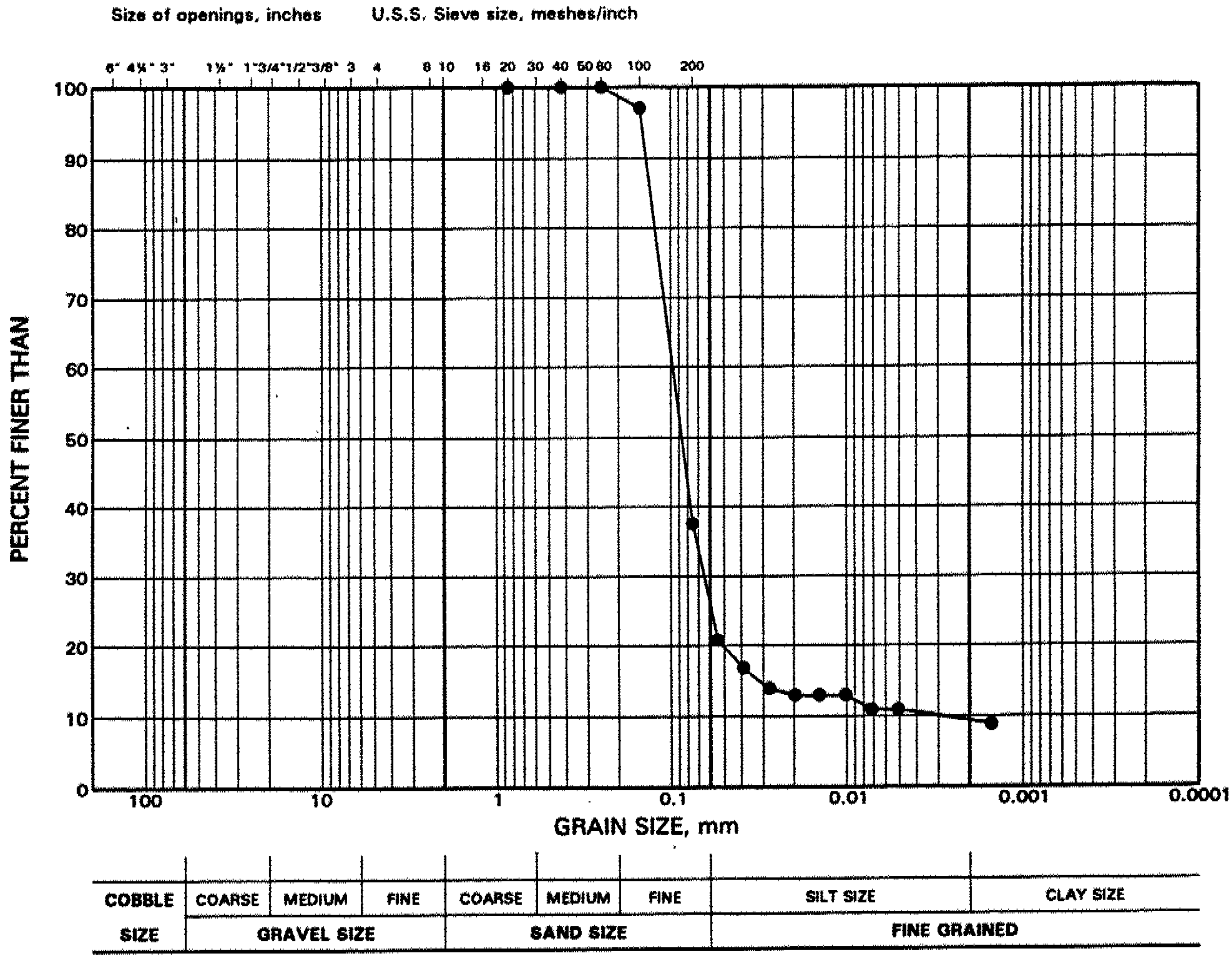
Summary of Augerhole Results
Highway 638 Wetland (Figure 3E)
Hwy 17(new) - Sault Ste. Marie to Bar River Road

Hand Augerhole Number	Approximate Location	Depth m			Material
95-PH5	11+510.5	0.00	to	0.30	So. blk. sdy. org. silt
		0.30	to	0.50	V. lse. brn. sand to sdy. silt, sm. org.
		0.50	to	9.10	Fi. grey to brn. sty. clay, tr. sand
		9.10	to	11.28	Com. grey sty. sand, sm. gvl, tr. to sm. clay (till).
					water ponding on surface
					till layer under artesian presure
95-PH2	12+310	0.00	to	0.25	V. so. blk. pt.
		0.25	to	0.45	V. so. brn. sdy. org. silt, sm. grvl
		0.45	to	11.89	So. to fi. grey to brn. sty. clay, tr. sand
	11.89 m; End of hole				water at 0.1 m depth
95-PH1	12+710	0.00	to	0.15	V. so. dk. brn. pt, sm sand
		0.15	to	1.40	So. grey sty. clay, tr. sand & org.
		1.40	to	2.20	Lse. grey sdy. silt, tr. org.
		2.20	to	12.80	So. to fi. grey to brn. sty. clay, tr. sand
	12.80 m; End of hole				water at ground surface
95-HA18	12+910	0.00	to	0.33	So. dk. brn. pt.
		0.33	to	0.43	So. brn. org. silt
		0.43	to	1.52	So. grey sty. clay
	1.52 m; End of hole				water up to 0.1 m above ground
95-HA19	13+010	0.00	to	0.18	So. brn. org. silt
		0.18	to	1.02	Lse./com. brn. sty. sand, sm. org.
		1.02	to	1.52	Fi. grey sty. clay
	1.52 m; End of hole				water at 0.15 m depth

Notes:
1) Hand augerhole locations shown on Figure 3E.
2) Area becomes wet south of approximately Sta. 11+460 and continues to approximately Sta. 12+960.

GRAIN SIZE DISTRIBUTION
Sand some Silt

FIGURE 4



LEGEND

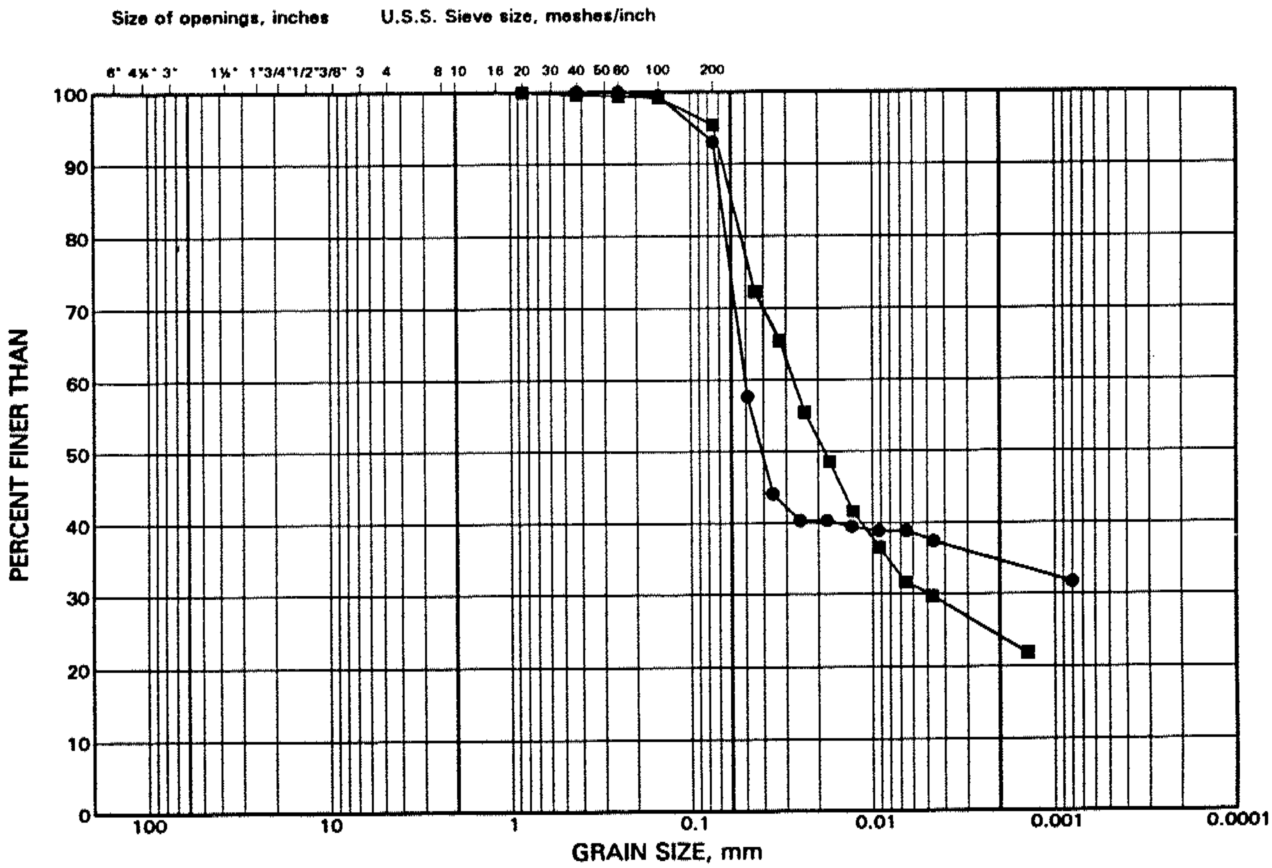
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	95-1	7	181.1

Project 941-1364

Golder Associates

GRAIN SIZE DISTRIBUTION
Interlayered Silt and Silty Clay

FIGURE 5

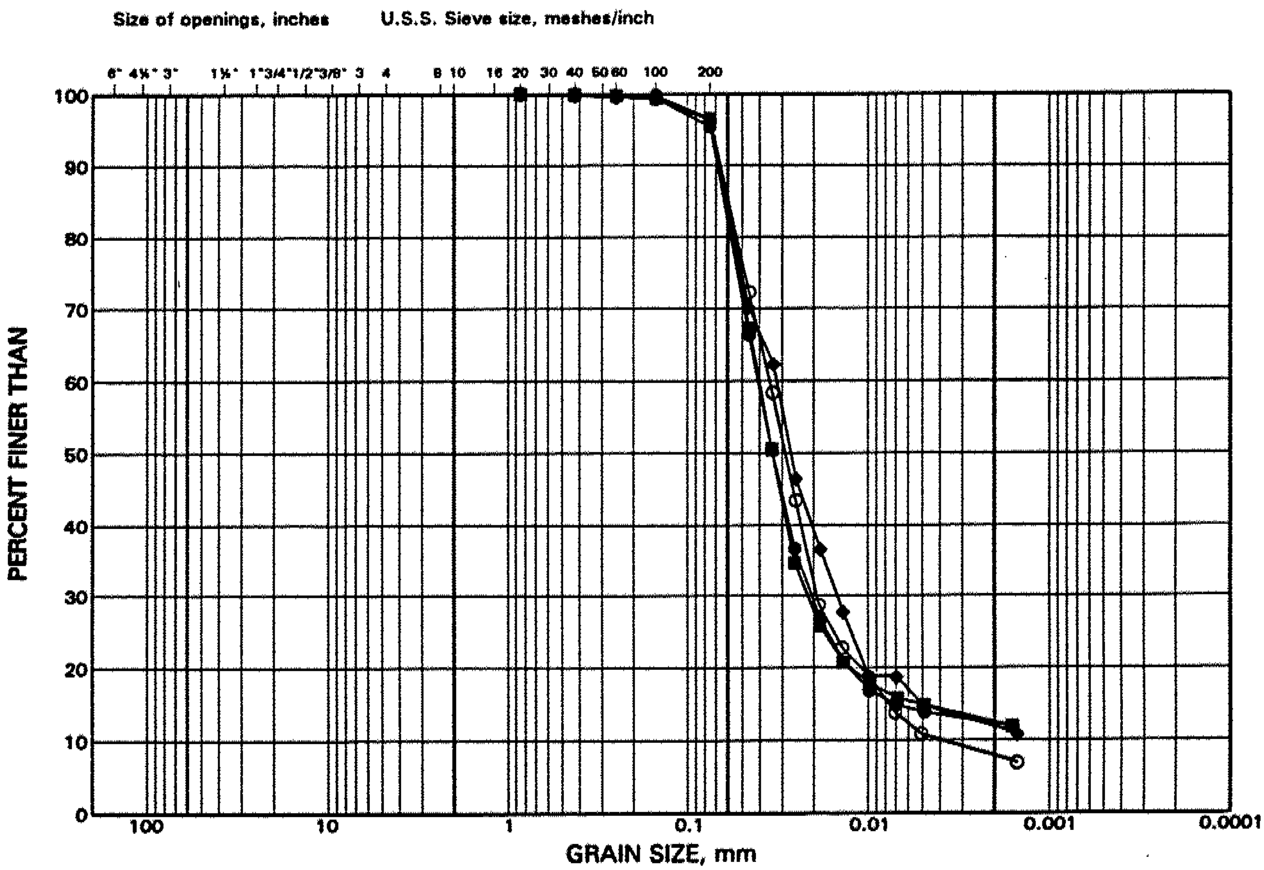


COBBLE	COARSE	MEDIUM	FINE	COARSE	MEDIUM	FINE	SILT SIZE	CLAY SIZE
SIZE	GRAVEL SIZE			SAND SIZE			FINE GRAINED	

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	95-PH3	3B	191.7
■	95-6	6	184.1

GRAIN SIZE DISTRIBUTION Interlayered Sandy Silt and Clayey Silt	FIGURE 6



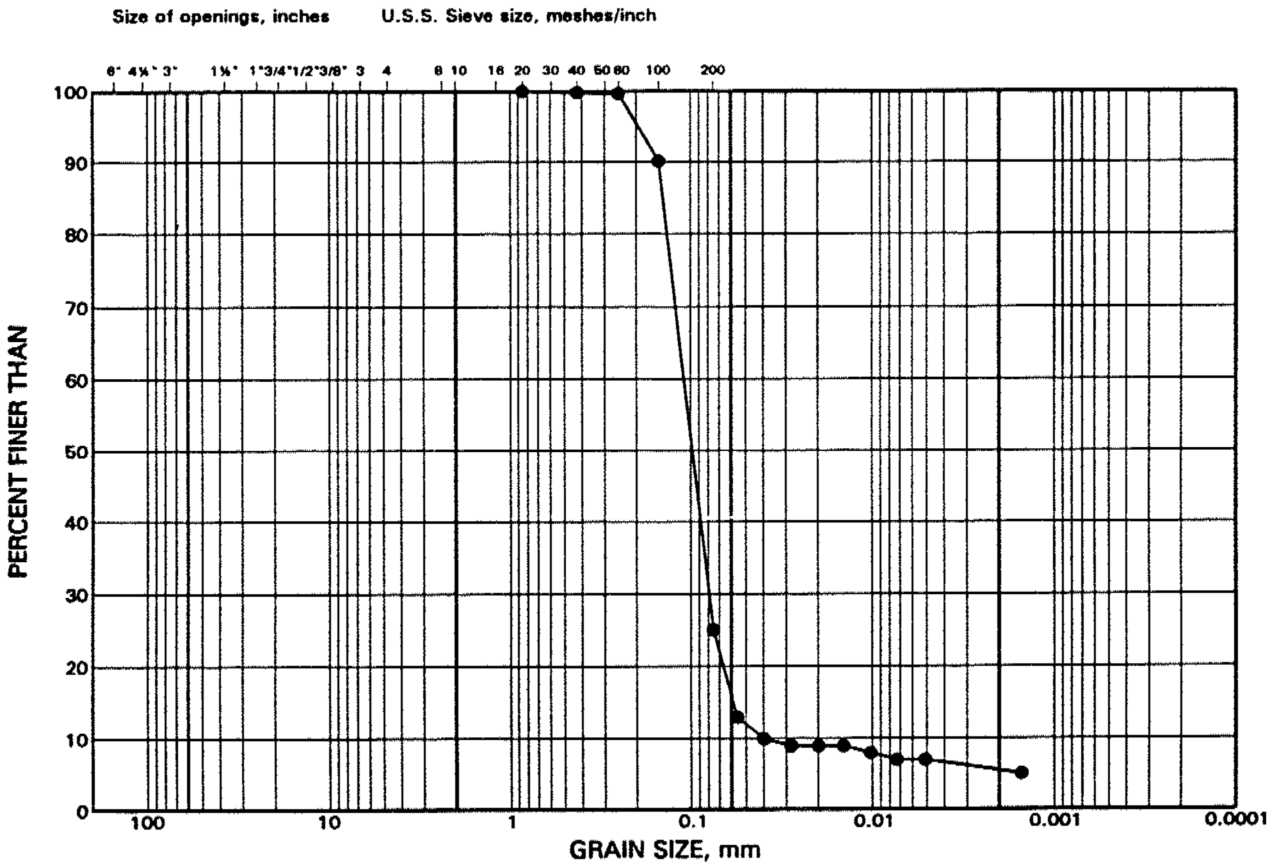
COBBLE	COARSE	MEDIUM	FINE	COARSE	MEDIUM	FINE	SILT SIZE	CLAY SIZE
SIZE	GRAVEL SIZE			SAND SIZE			FINE GRAINED	

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	95-4	8B	180.4
■	95-4	12	175.0
◆	95-5	3A	180.3
○	95-PH4	3	190.2

GRAIN SIZE DISTRIBUTION
Sand

FIGURE 7



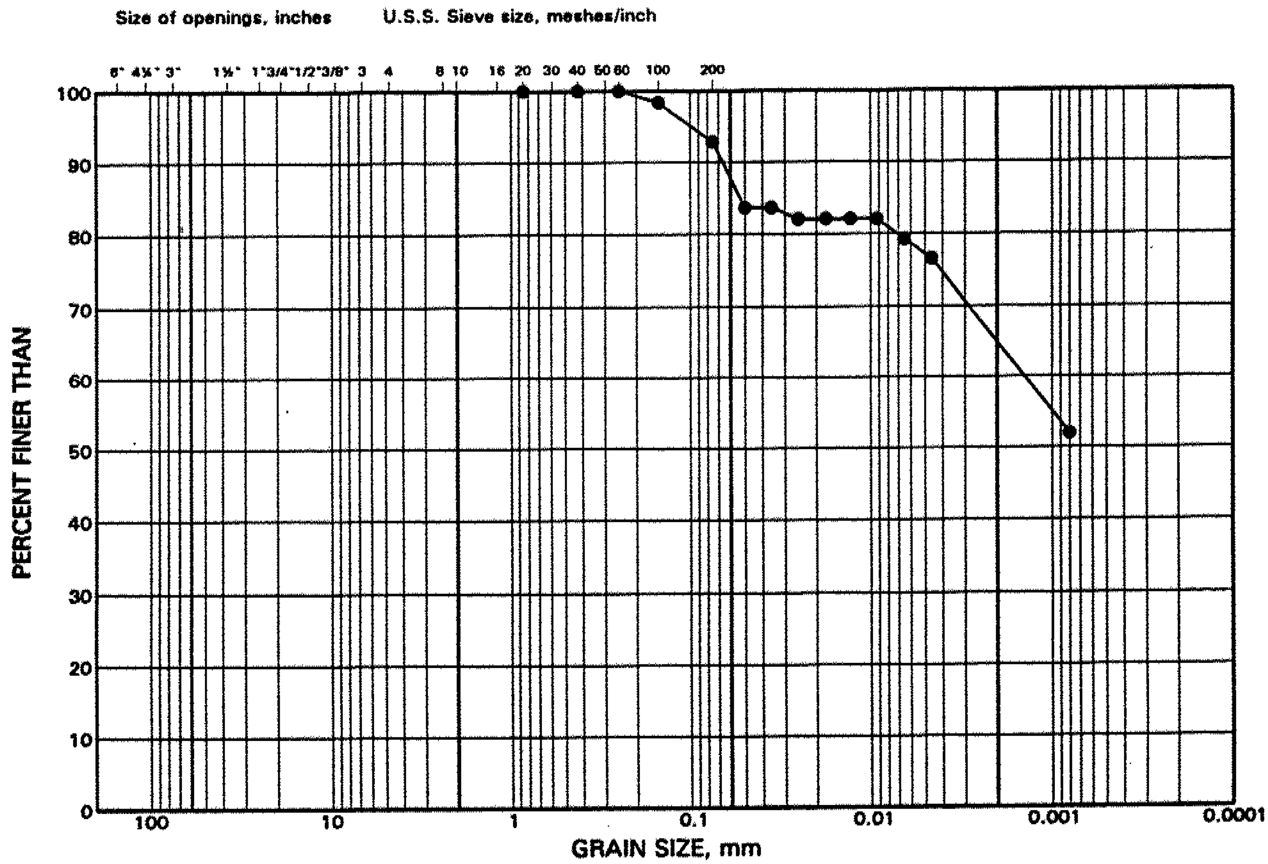
COBBLE SIZE	COARSE	MEDIUM	FINE	COARSE	MEDIUM	FINE	SILT SIZE	CLAY SIZE
	GRAVEL SIZE			SAND SIZE			FINE GRAINED	

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	95-5	6B	177.8

GRAIN SIZE DISTRIBUTION
Clay

FIGURE 8



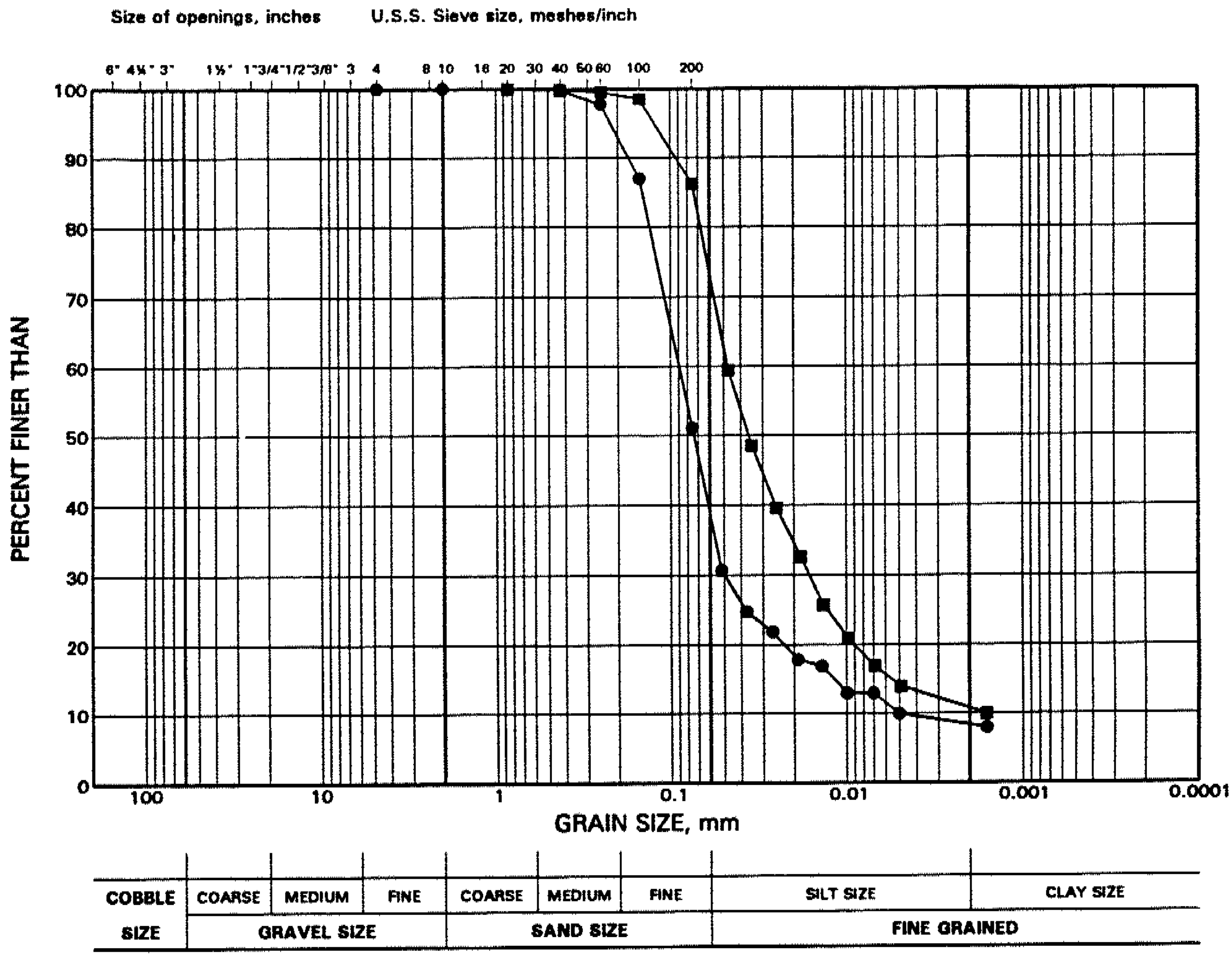
COBBLE SIZE	COARSE	MEDIUM	FINE	COARSE	MEDIUM	FINE	SILT SIZE	CLAY SIZE
	GRAVEL SIZE			SAND SIZE			FINE GRAINED	

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	95-4	18	165.9

GRAIN SIZE DISTRIBUTION
Interlayered Silty Sand and Sandy Silt

FIGURE 9

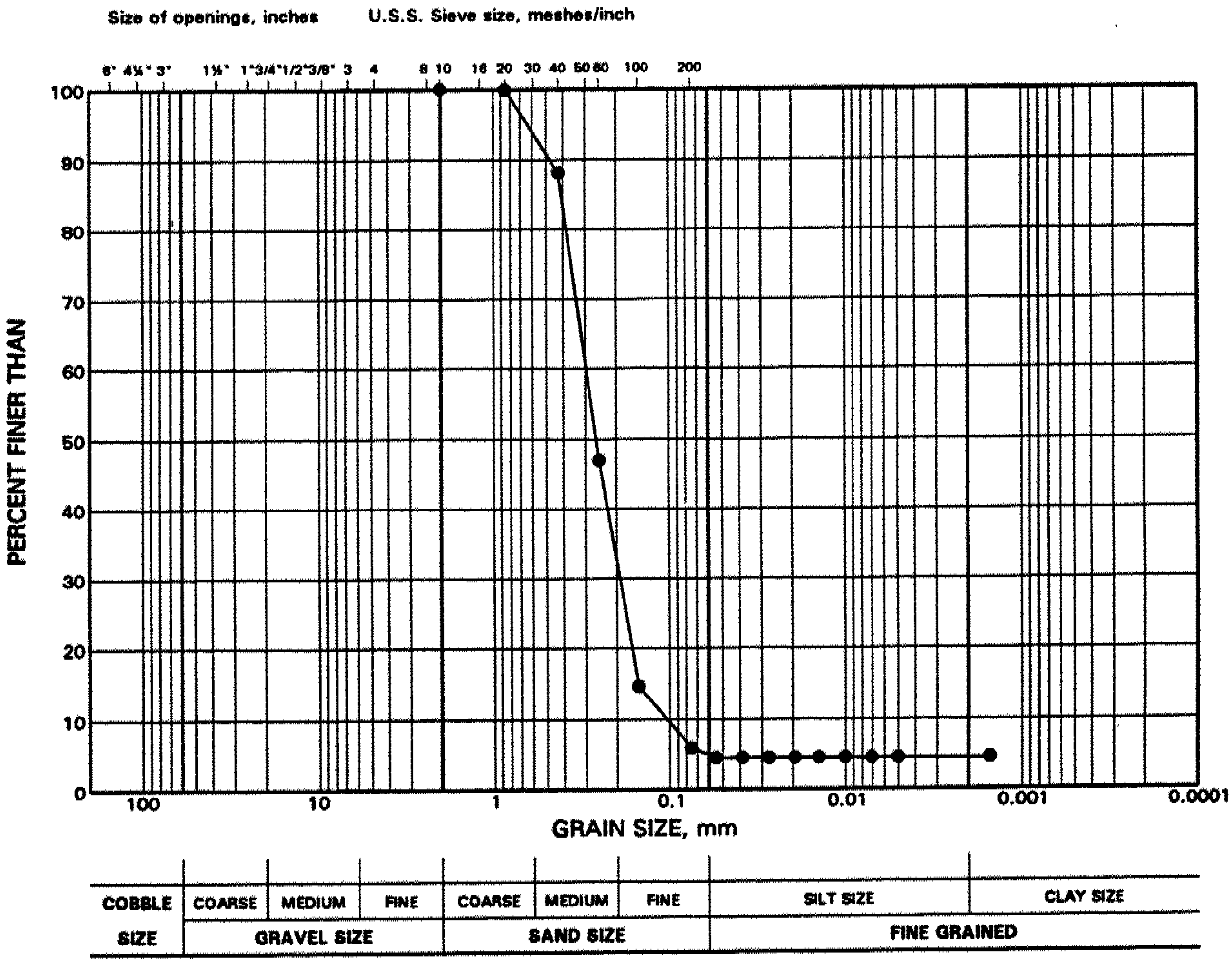


LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	95-7	7	189.1
■	95-8	9	187.0

GRAIN SIZE DISTRIBUTION
Sand

FIGURE 10

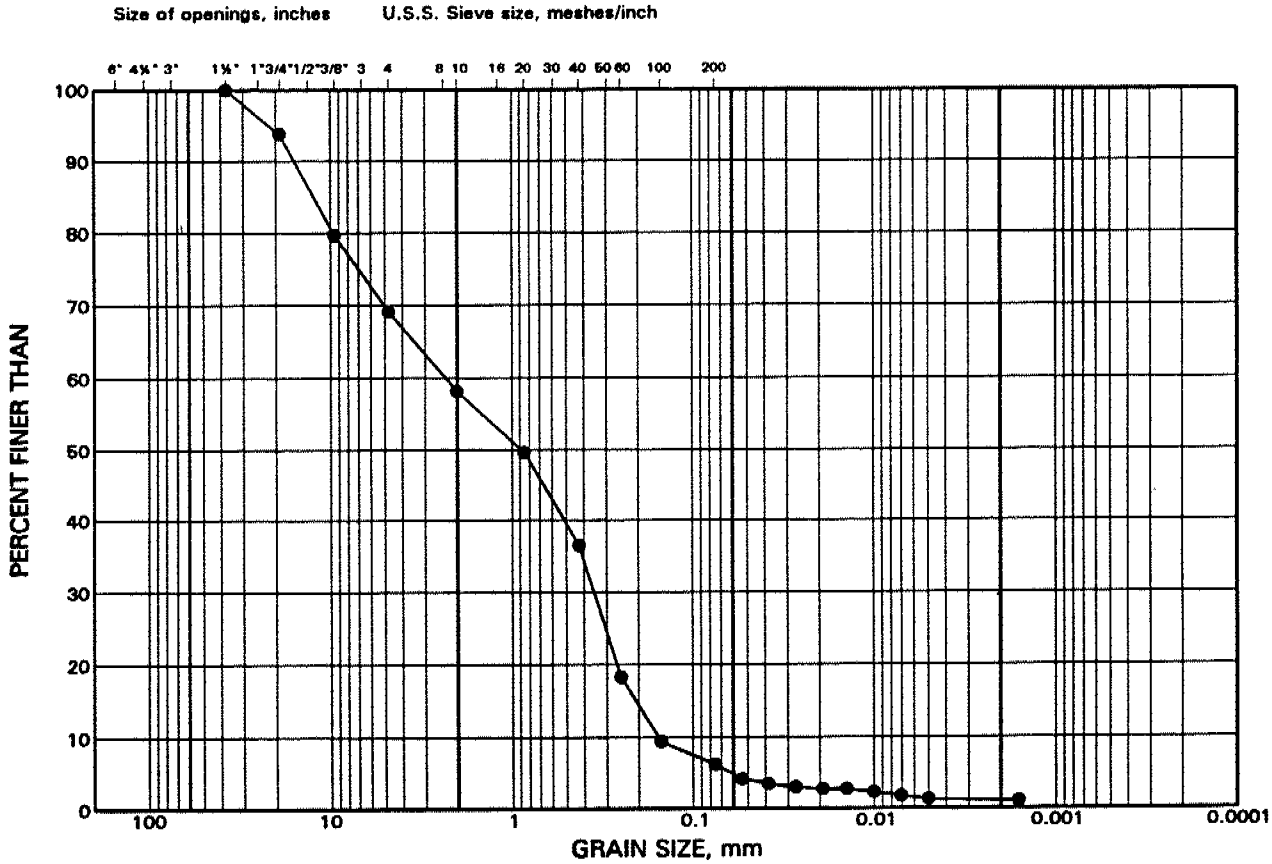


LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	95-7	18	172.4

GRAIN SIZE DISTRIBUTION
Sand & Gravel

FIGURE 11



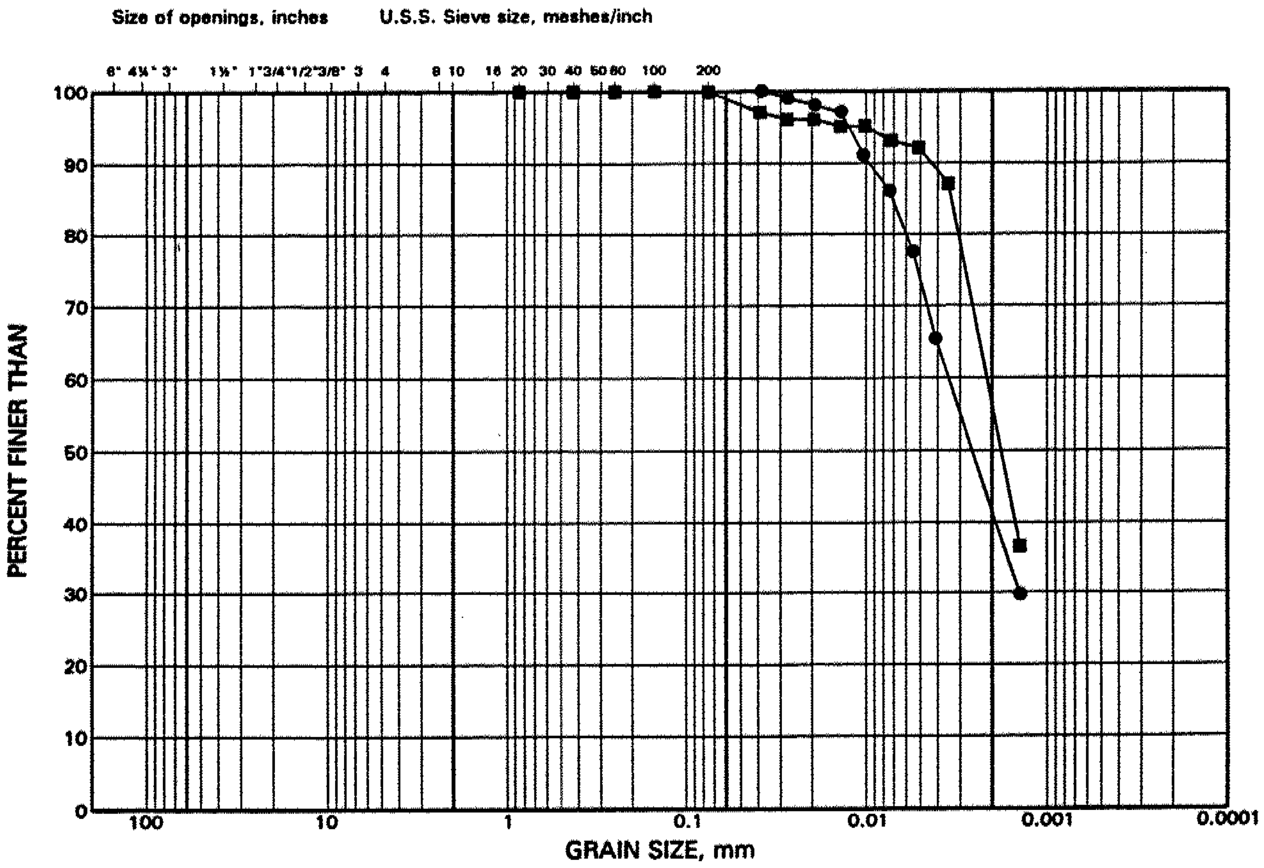
COBBLE	COARSE	MEDIUM	FINE	COARSE	MEDIUM	FINE	SILT SIZE	CLAY SIZE
SIZE	GRAVEL SIZE			SAND SIZE			FINE GRAINED	

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	95-8	13	180.9

GRAIN SIZE DISTRIBUTION
Silty Clay

FIGURE 12



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	95-9	12	158.4
■	95-9	20	139.5

Project 941-1364

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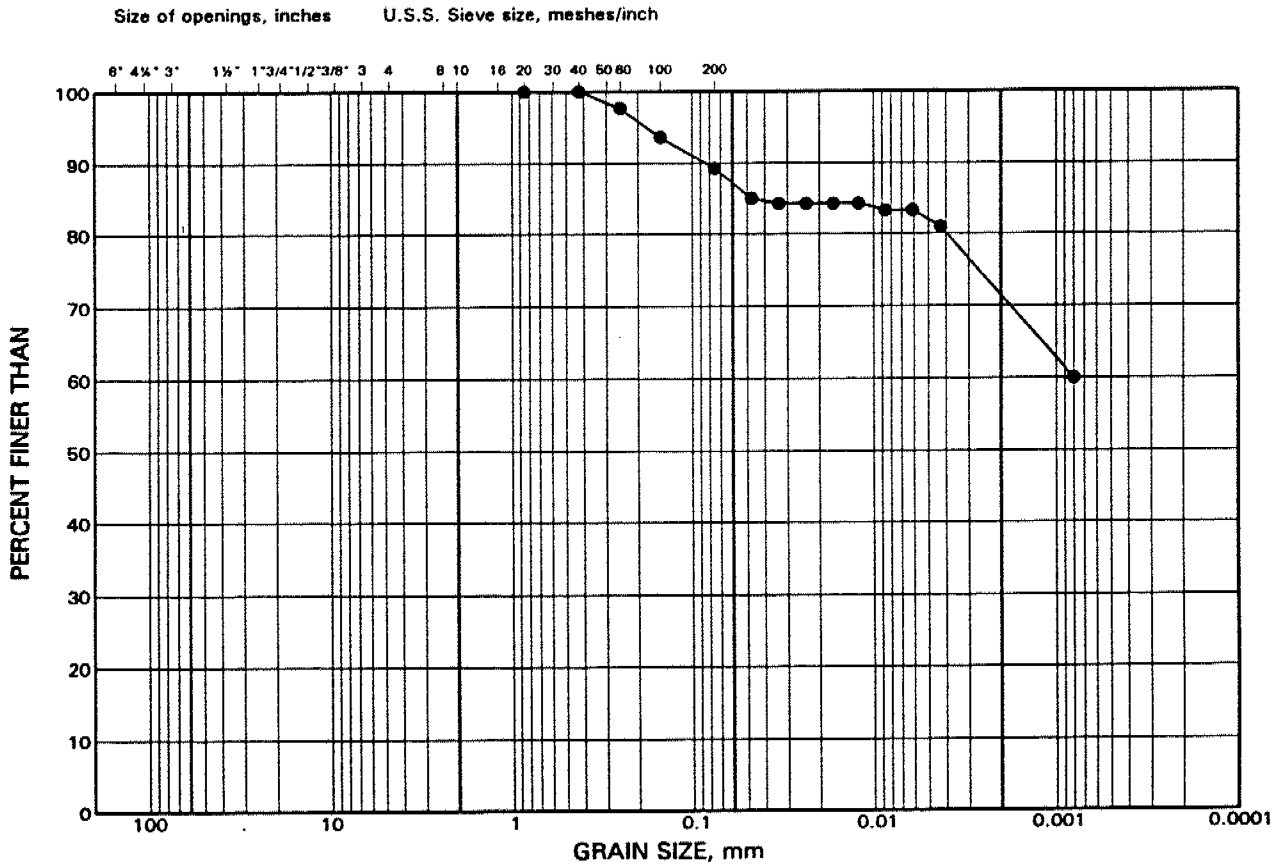
GRAIN SIZE DISTRIBUTION
Clay

FIGURE 13



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

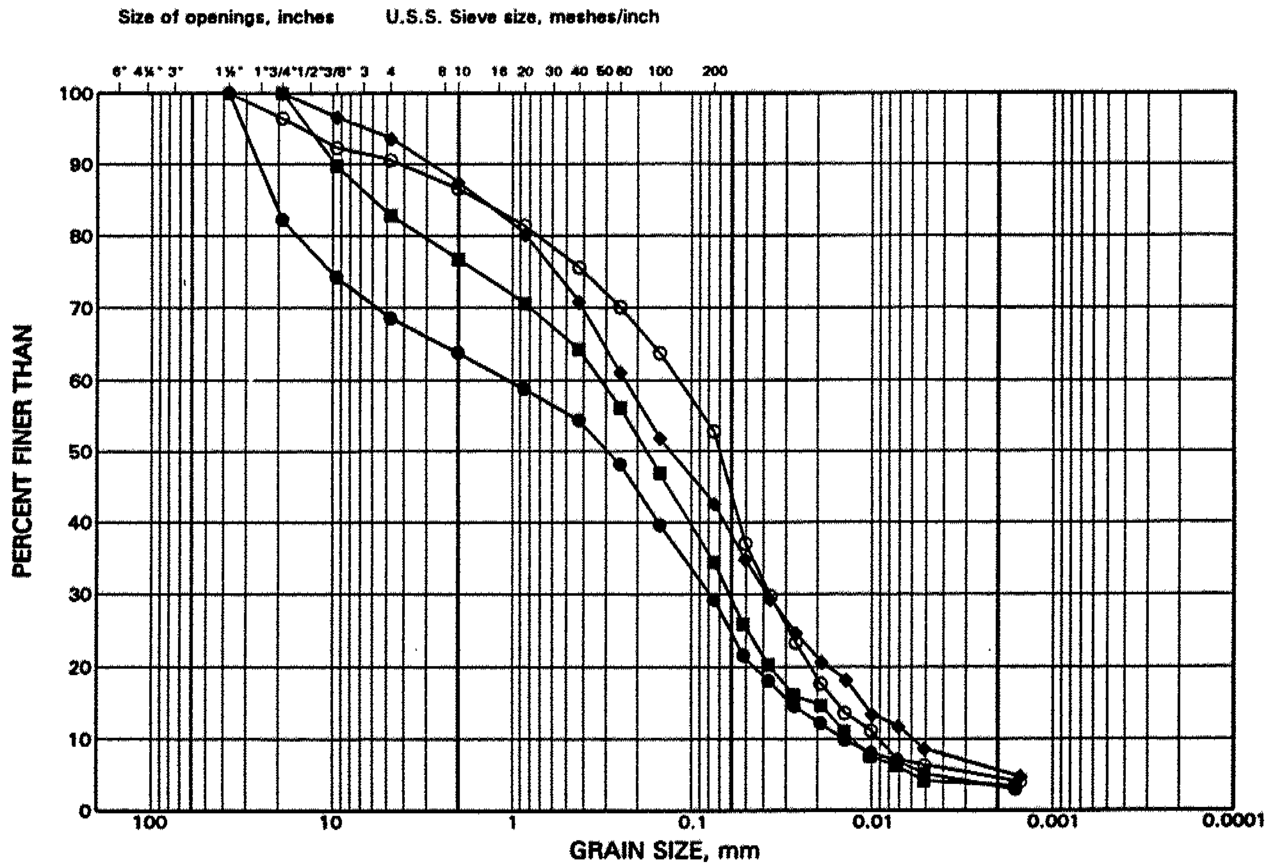
LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	95-PH2	2	174.7

Golder Associates

GRAIN SIZE DISTRIBUTION
Silty Sand (Till)

FIGURE 14



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	95-10	6	182.6
■	95-12	4	185.9
◆	95-13	9C	179.0
○	95-PH5	9	168.9

Project 941-1364

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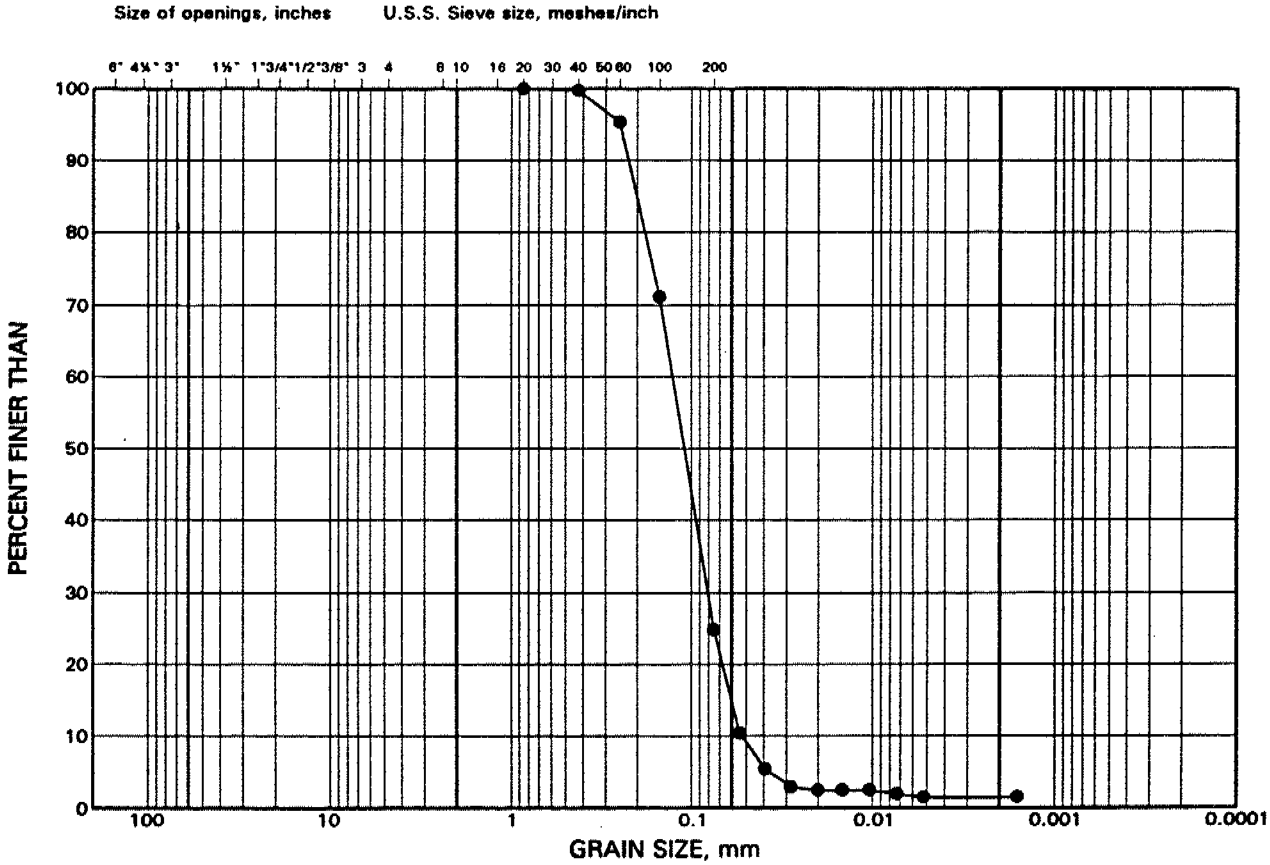
GRAIN SIZE DISTRIBUTION
Sand

FIGURE 15



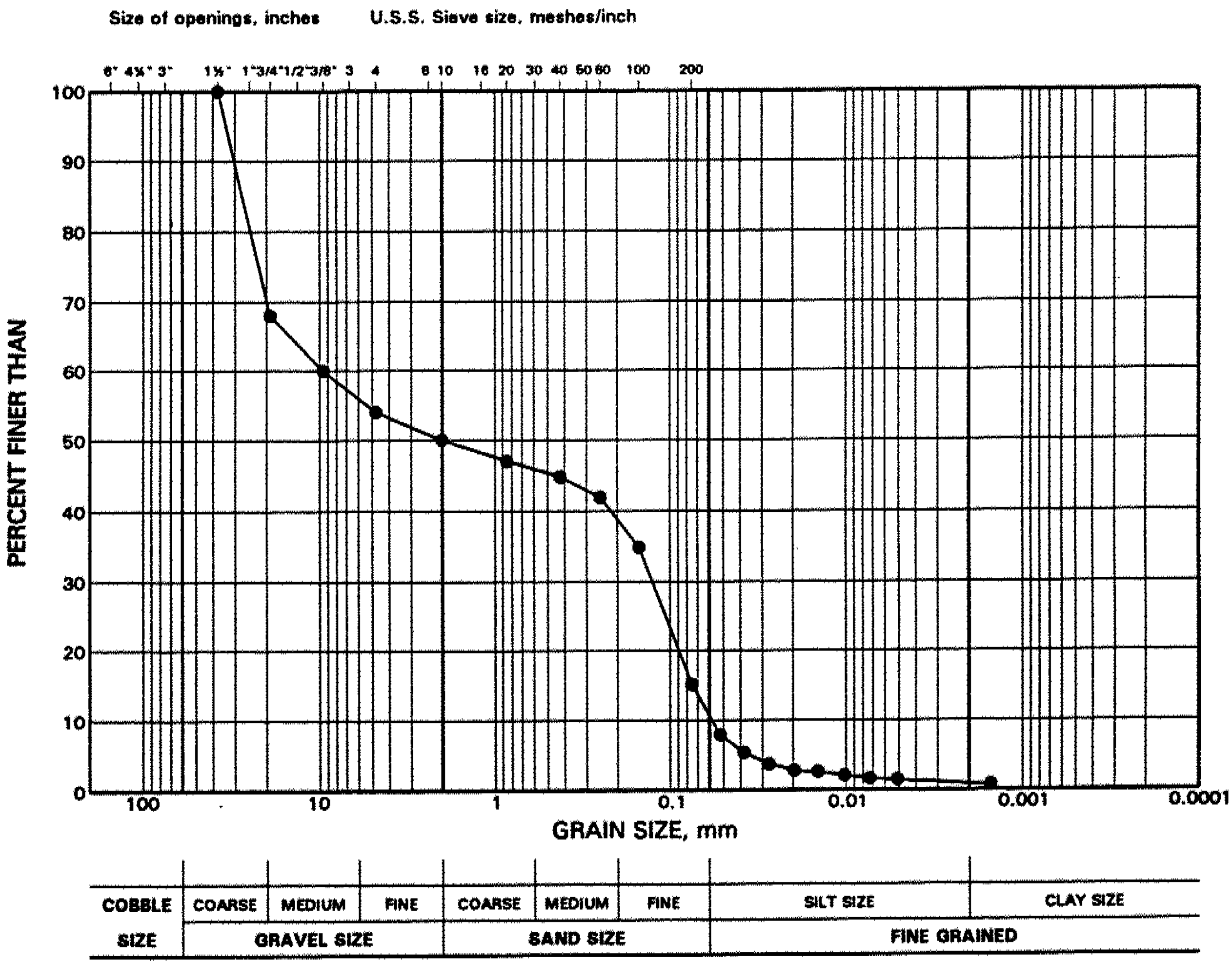
COBBLE	COARSE	MEDIUM	FINE	COARSE	MEDIUM	FINE	SILT SIZE	CLAY SIZE
SIZE	GRAVEL SIZE			SAND SIZE			FINE GRAINED	

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	95-13	7	181.3

GRAIN SIZE DISTRIBUTION
Sand & Gravel

FIGURE 16



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	95-12	8	182.9

CONSOLIDATION SUMMARY

FIGURE 17a

PROJECT 941-1364
SAMPLE BH 9 / SA 12
AREA(mm2) 3157.06

SPECIFIC GRAVITY 2.70 assumed
DRY WEIGHT,gm 81.1
SOLIDS HT.2HS 9.514

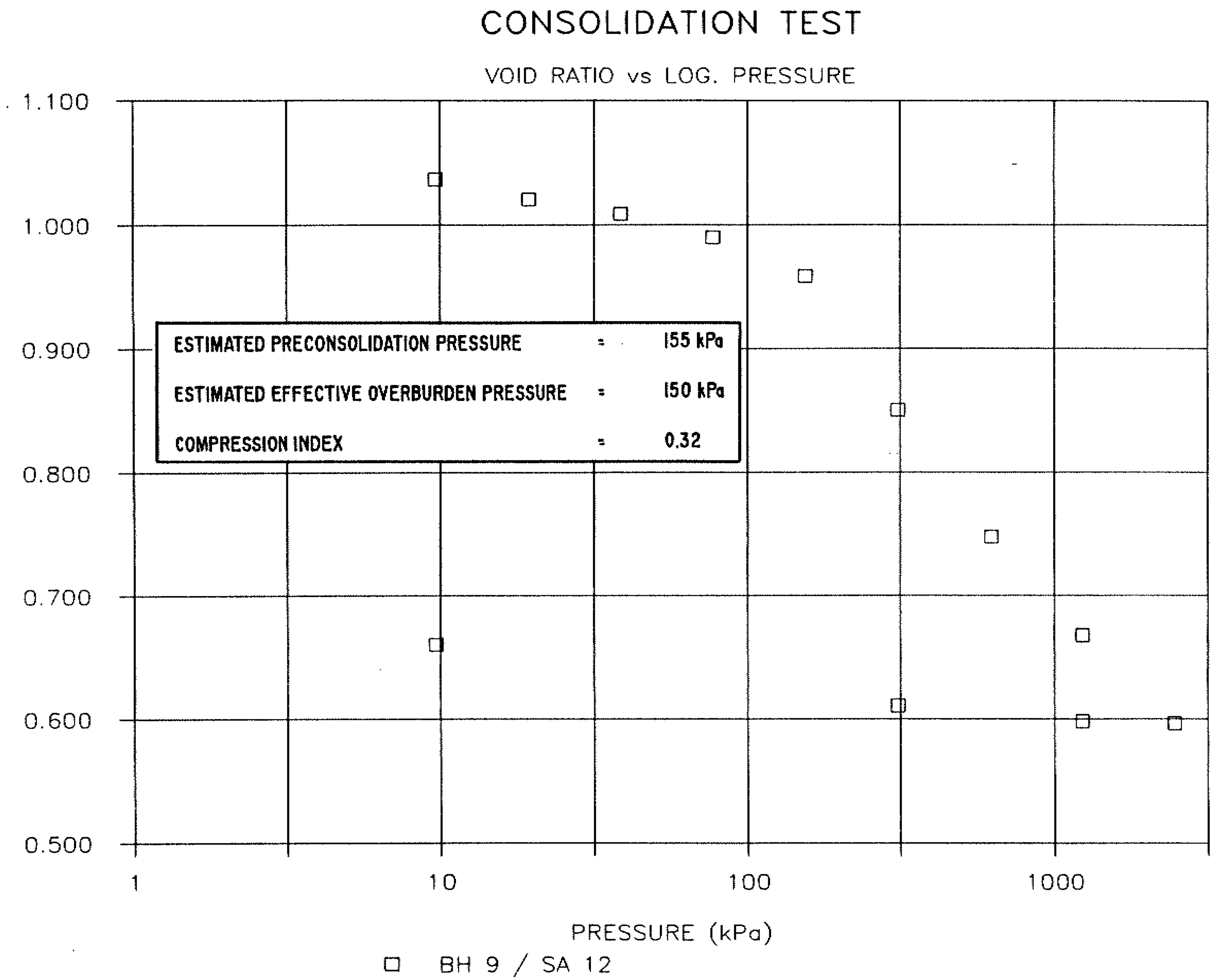
DATE STARTED 95-10-18
DATA COMPLETED 95-10-30

Load kPa	Corr. Height mm	Void Ratio	Average Height mm	t90 sec	t50 sec	cv. t90 cm2/s	cv. t50 cm2/s	k cm/S	mv m2/kN
0.00	19.100	1.008	19.100						
9.68	18.920	0.989	19.010	14		5.47E-02		5.23E-06	9.75E-04
19.36	18.773	0.973	18.847	14		5.38E-02		4.17E-06	7.92E-04
38.73	18.660	0.961	18.717	37		2.01E-02		6.01E-07	3.05E-04
77.46	18.485	0.943	18.573	127		5.76E-03		1.33E-07	2.37E-04
154.92	18.197	0.913	18.341	71		1.00E-02		1.92E-07	1.95E-04
309.83	17.189	0.807	17.693	893		7.43E-04		2.48E-08	3.41E-04
619.66	16.232	0.706	16.710	583		1.02E-03		1.61E-08	1.62E-04
1239.32	15.491	0.628	15.861	197		2.71E-03		1.66E-08	6.26E-05
2478.65	14.829	0.559	15.160	20		2.44E-02		6.68E-08	2.80E-05
1239.32	14.846	0.560	14.838						7.35E-07
309.83	14.964	0.573	14.905						6.61E-06
9.68	15.421	0.621	15.192						7.98E-05

Notes:
k calculated using Cv based on t90 values.

Water Content % ,initial	36.0	Liquid Limit %	45.0
Water Content % , final	25.8	Plastic Limit %	23.3
		Plastic Index %	21.7
Original Volume,cc	60.299	Liquidity Index	0.585
Volume of Solids,cc	30.04		
Volume of Voids,cc	30.26	Unit Weight,kN/m3	17.93
Degree of Saturation %	96.47	Dry Unit Weight,kN/m3	13.18

Golder Associates



Golder Associates

CONSOLIDATION TEST
VOID RATIO VS. LOG PRESSURE

FIGURE 17b

# CONSOLIDATION SUMMARY	FIGURE 18a

PROJECT 941-1364 SPECIFIC GRAVITY 2.70 assumed DATE STARTED 95-10-26
SAMPLE BH 9 / SA 20 DRY WEIGHT,gm 75.05 DATA COMPLETED 95-11-06
AREA(mm2)3157.06 SOLIDS HT.2HS 8.804

Load kPa	Corr. Height mm	Void Ratio	Average Height mm	t90 sec	t50 sec	cv. t90 cm2/s	cv. t50 cm2/s	k cm/S	mv m2/kN
0.00	19.100	1.169	19.100						
9.68	19.045	1.163	19.073	13		5.93E-02		1.73E-06	2.97E-04
19.36	18.943	1.152	18.994	20		3.82E-02		2.07E-06	5.52E-04
38.73	18.701	1.124	18.822	30		2.50E-02		1.60E-06	6.54E-04
77.46	18.532	1.105	18.617	90		8.16E-03		1.83E-07	2.29E-04
154.92	18.147	1.061	18.339	90		7.92E-03		2.02E-07	2.60E-04
309.83	17.495	0.987	17.821	60		1.12E-02		2.42E-07	2.20E-04
619.66	16.537	0.878	17.016	120		5.12E-03		8.11E-08	1.62E-04
1239.32	15.658	0.778	16.098	50		1.10E-02		7.99E-08	7.43E-05
2478.65	14.870	0.689	15.264	50		9.88E-03		3.22E-08	3.33E-05
1239.32	14.915	0.694	14.893						1.92E-06
309.83	14.968	0.700	14.942						2.96E-06
9.68	15.914	0.807	15.441						1.65E-04

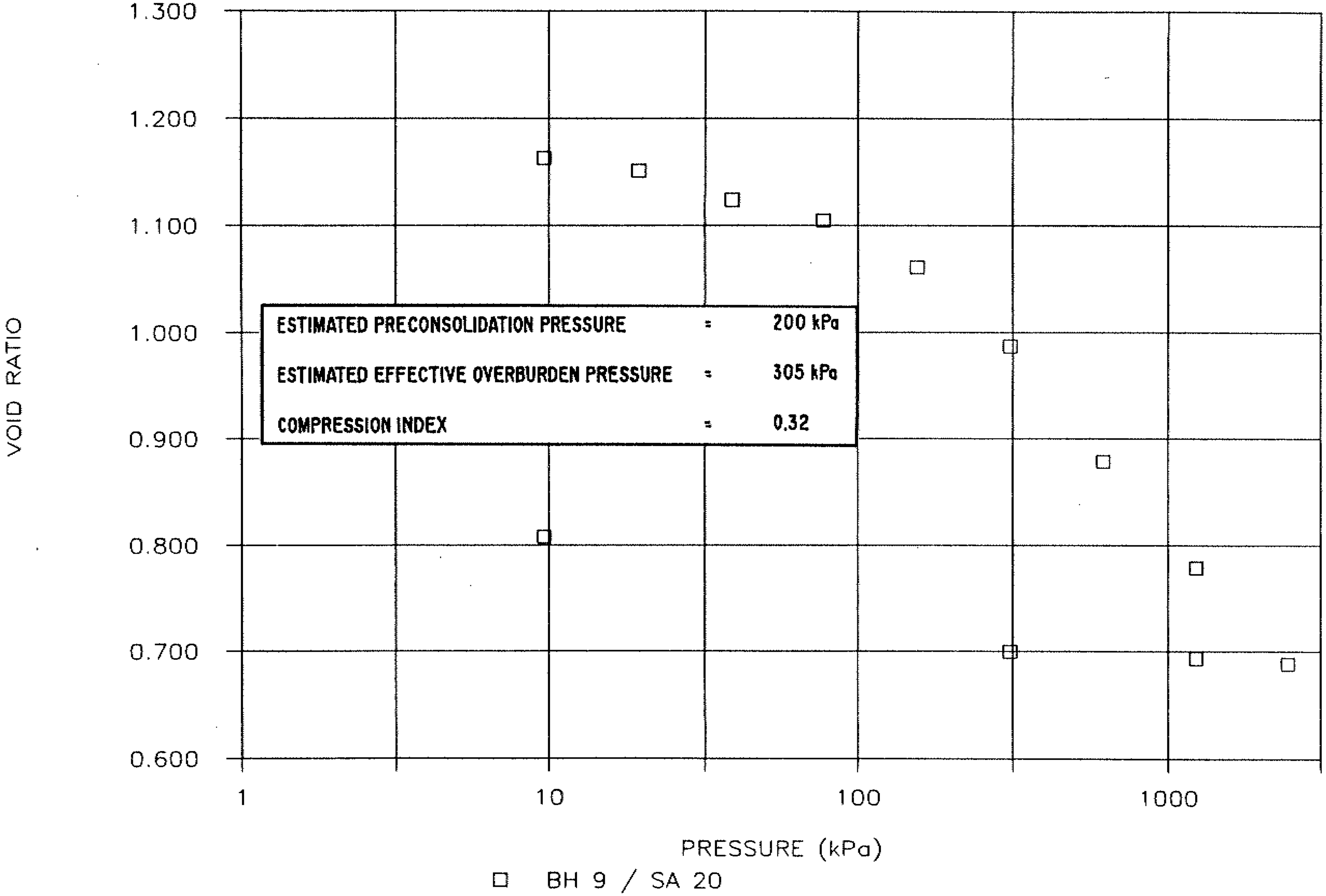
Notes:
k calculated using Cv based on t90 values.

Water Content % ,initial	41.5	Liquid Limit %	47.4
Water Content % , final	35.5	Plastic Limit %	25.1
		Plastic Index %	22.3
Original Volume,cc	60.299	Liquidity Index	0.735
Volume of Solids,cc	27.80		
Volume of Voids,cc	32.50	Unit Weight,kN/m3	17.30
Degree of Saturation %	95.82	Dry Unit Weight,kN/m3	12.23

Golder Associates

CONSOLIDATION TEST

VOID RATIO vs LOG. PRESSURE



CONSOLIDATION TEST
VOID RATIO VS. LOG PRESSURE

FIGURE 18b

APPENDIX I

Record of Borehole Logs	-	**Ministry of Transportation**
Borehole 3	-	**Report W.P. 903-72-18**
Boreholes 3 and 4	-	**Report W.P. 903-72-01 Part 2**
Boreholes 1 and 2	-	**Report W.P. 903-72-01-Part 1**
Boreholes 9, 12 through 18	-	**Report W.P. 903-72-01 Part 1 - Addendum**

ENGINEERING SERVICES BRANCH-GEOTECHNICAL OFFICE-SOIL MECHANICS SECTION

RECORD OF BOREHOLE Nº 3

WP 903-72-18 | LOCATION Co-ords. 916,805 N; 963,978 E. Scheme 'B' | ORIGINATED BY HS

DIST 18 HWY 17 | BORING DATE October 6, 1975 | COMPILED BY HS

DATUM Geodetic | BOREHOLE TYPE Washboring, NX Casing | CHECKED BY [signature]

SOIL PROFILE		SAMPLES			GROUND WATER	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 400 800 1200 1600 2000	LIQUID LIMIT —w_L PLASTIC LIMIT —w_P WATER CONTENT —w / WATER CONTENT %	UNIT WEIGHT γ	REMARKS % GR SA SI CL
ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	'N' VALUES	ELEV				
600.0	Ground Level								
0.0	Topsoil; Sandy silt								
595.0	Very Loose	1	SS	3					
5.0	Sand - Fine to Medium	2	SS	82*					
	Trace to some gravel.	3	SS	7	590				
	Loose	4	SS	4					
		5	SS	6	580				
	silty clay to clay - Stiff					+ s3.2			
		6	SS	9					
567.5		7	SS	8	570				
32.5	Sand with gravel	8	SS	50					
558.5	Dense to Very Dense	9	SS	100/10"	560				
41.5	End of Borehole				550				

Note: Three attempts were made to set up the borehole, in order to avoid cobbles and/or boulders which are present in th upper 5 ft. of the granular deposit.

* Coarse gravel obstructed the advancement of the spoon in sample No.2, thus its blows/foot count is considered non-representative.

20
15 ◇ 5 % STRAIN AT FAILURE
10

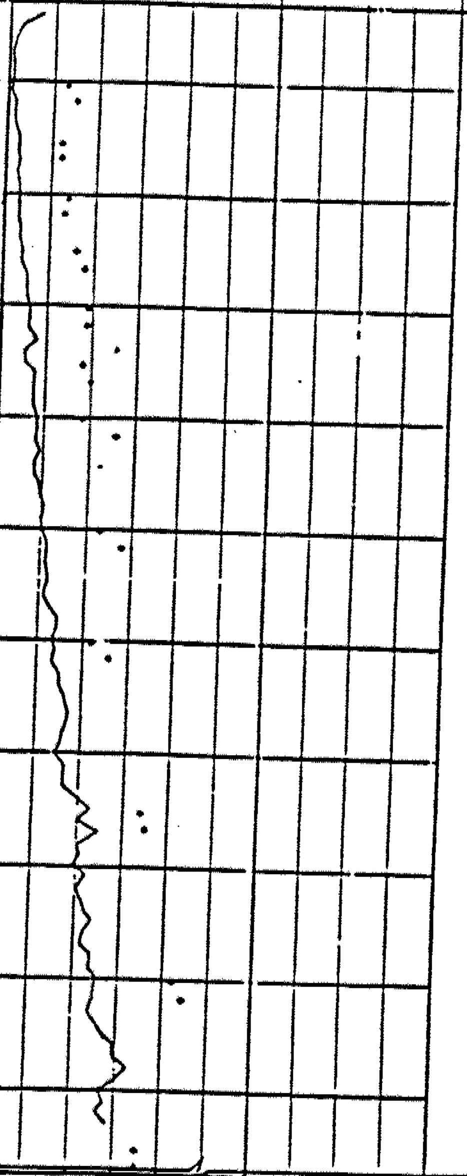
MINISTRY OF TRANSPORTATION AND COMMUNICATIONS - ONTARIO

ENGINEERING SERVICES BRANCH GEOTECHNICAL OFFICE SOIL MECHANICS SECTION

RECORD OF BOREHOLE No 3

W.P. 903-72-01
LOCATION Hwy.17 Line'D' & Echo River (Sta. 1015)
DIST 18 HWY 17
BORING DATE March 1-3, 1975
DATUM Geodetic
BOREHOLE TYPE Hollow Stem Auger & Cone Test
ORIGINATED BY HS
COMPILED BY HH
CHECKED BY

SOIL PROFILE			SAMPLES				DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH (○ UNCONFINED, ● QUICK TRIAXIAL, + FIELD VANE, ▲ LAB VANE; 400 800 1200 1600 2000)	LIQUID LIMIT, PLASTIC LIMIT, WATER CONTENT %	UNIT WEIGHT γ	REMARKS (% GR SA SI CL)
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER / ELEV				
587.9	Ground Level									
	Clay to silty clay		1	SS	9					
			2	SS	2	580				
			3	SS	1/18"					
	Soft		4	SS	1/18"	570				
	Firm		5	TW	PH					
			6	SS	1/18"	560				
						550				
			7	TW	PH	540				
			8	SS	1/18"	530				
						520				
	Stiff		9	SS	1/18"	510				
			10	SS	1/18"	500				
						490				
			11	SS	1/18"					Augered to 132.
455.0	End of borehole									



20
15 ◇ 5 % STRAIN AT FAILURE
10

OFFICE REPORT ON SOIL EXPLORATION

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS – ONTARIO

ENGINEERING SERVICES BRANCH · GEOTECHNICAL OFFICE · SOIL MECHANICS SECTION

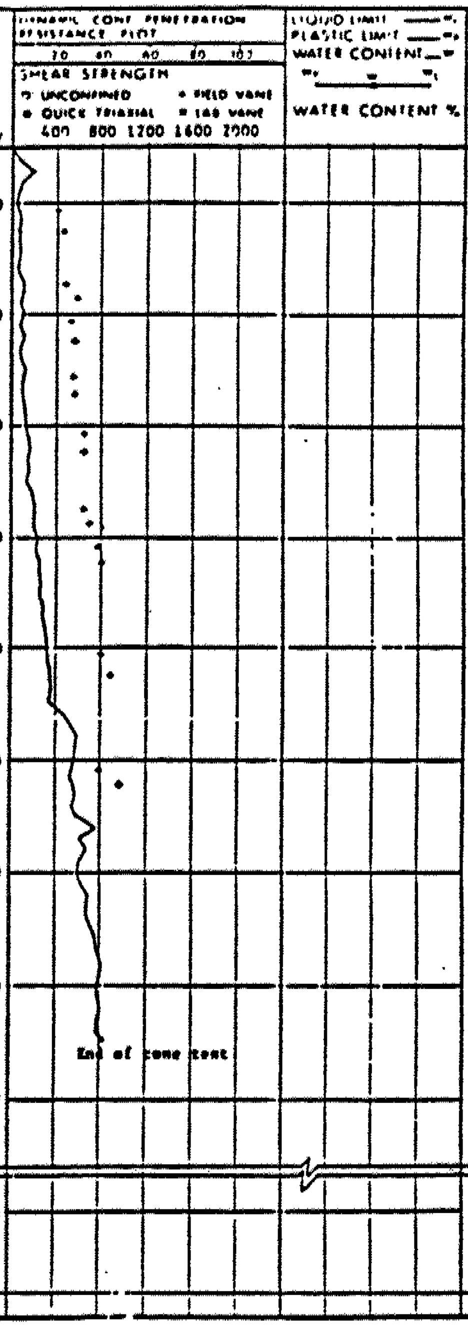
RECORD OF BOREHOLE Nº 4

W.P. 303-72-01 LOCATION Hwy.17 Lines "A:B:& C" & Echo River (Sta. 101±) ORIGINATED BY HS

DIST 18 HWY 17 BORING DATE March 4-5, 1975 COMPILED BY

DATUM Geodetic BOREHOLE TYPE Hollow Stem Auger and Cone Test CHECKED BY

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES "N" VALUES	GROUND WATER / ELEV	REMARKS (% GR SA SI CL)
595.							
	Topsoil						
1.0	Sandy silt		1	AS			
3.0	Clay to silty clay					590	
			2	TW	PH		
						580	
	Soft						
	Firm					570	
			3	TW	PH		
						560	
						550	
						540	
						530	
	Stiff					520	
						510	
						470	
463.0							Augered to 132 ft.
132.0	End of Borehole						



20
15 ⊕ 5 % STRAIN AT FAILURE
10

OFFICE REPORT ON SOIL EXPLORATION

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS–ONTARIO

ENGINEERING SERVICES BRANCH–GEOTECHNICAL OFFICE–SOIL MECHANICS SECTION

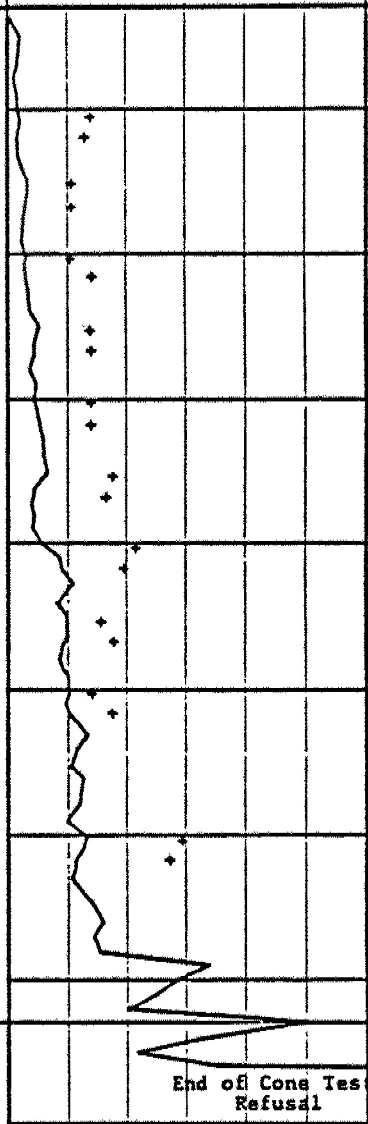
RECORD OF BOREHOLE Nº 1

W.P. 903-72-01	LOCATION Hwy. 17 & C.P.R. (As shown on Plan)	ORIGINATED BY HS
DIST. 18 HWY. 17	BORING DATE Feb. 24 -26, 1975	COMPILED BY MM
DATUM Geodetic	BOREHOLE TYPE Hollow Stem Auger & Cone Test	CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER ELEV.
ELEV. / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES	
617.±	Ground Level					
0.0	Topsoil					
614.0 / 3.0	Sandy Silt Desiccated or Frozen					
	Clay to Silty Clay, Soft		1	SS	2	▼ 610
			2	TW	PH	
			3	SS	1	600
			4	SS	1	
			5	SS	1	590
	Firm		6	SS	1	
			7	TW	PH	580
			8	SS	1	
			9	SS	1	570
			10	TW	PH	560
556.0 / 61.0	Silty fine sand to sandy silt. Compact to Dense		11	SS	22	
			12	SS	29	550
547.2 / 69.8	End of Borehole Refusal		13	SS	70	
						540

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE — 400 800 1200 1600 2000



LIQUID LIMIT — w_L PLASTIC LIMIT — w_P WATER CONTENT — w

WATER CONTENT %

UNIT WEIGHT γ

REMARKS % GR SA SI CL

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS–ONTARIO

ENGINEERING SERVICES BRANCH-GEOTECHNICAL OFFICE-SOIL MECHANICS SECTION

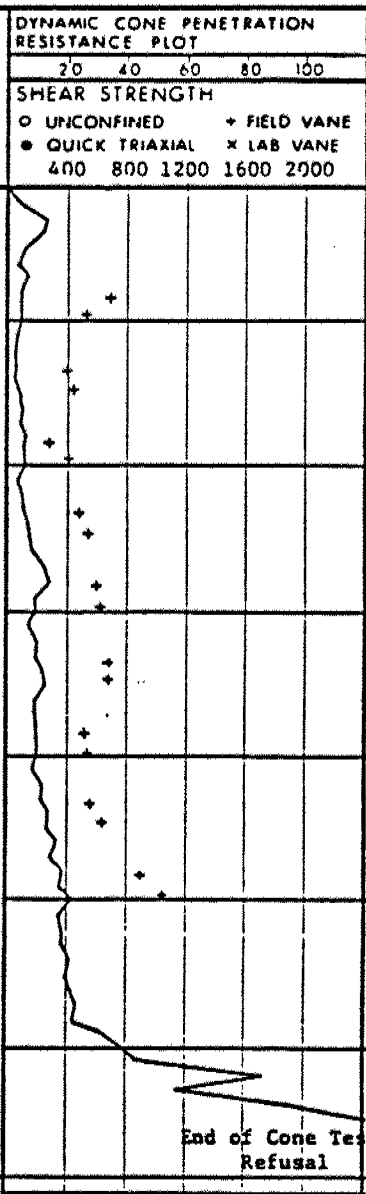
RECORD OF BOREHOLE Nº 2

W.P. 903-72-01 LOCATION Hwy. 17 & C.P.R. (As shown on Plan) ORIGINATED BY HS

DIST. 18 HWY. 17 BORING DATE February 26-28, 1975 COMPILED BY MM

DATUM Geodetic BOREHOLE TYPE Hollow Stem Auger and Cone Test CHECKED BY [signature]

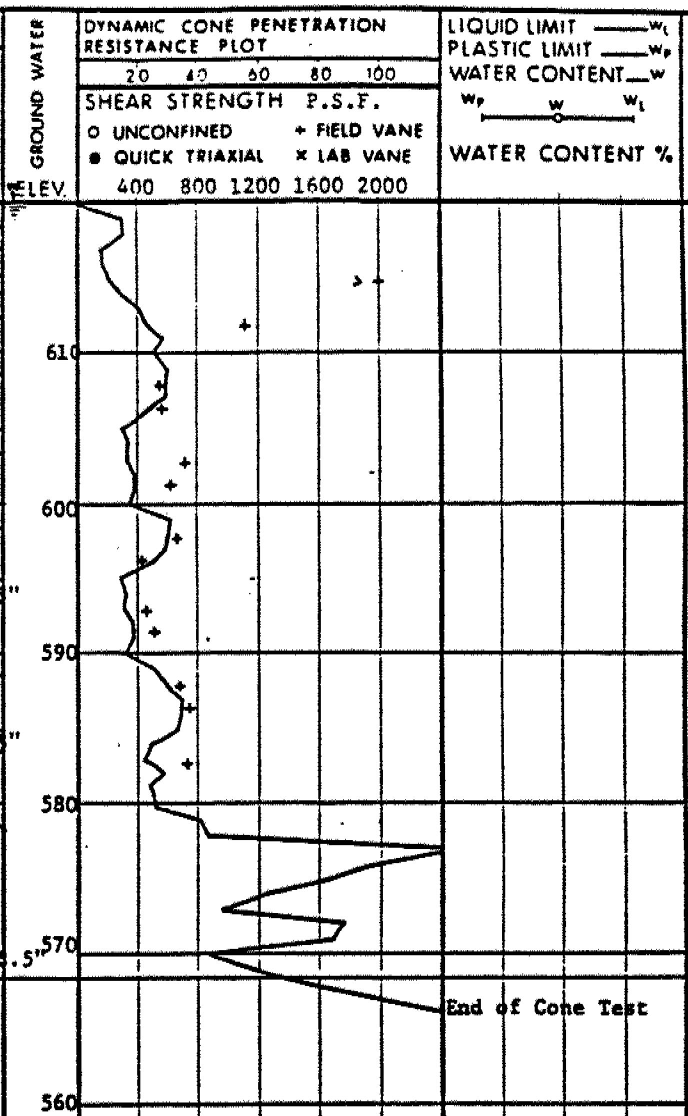
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES	GROUND WATER ELEV.	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH	LIQUID LIMIT, PLASTIC LIMIT, WATER CONTENT %	UNIT WEIGHT γ	REMARKS % GR SA SI C
619.±	Ground Level									
0.0	Topsoil									
616.0	Sandy Silt									
3.0	Clay to Silty Clay — Desiccated or Frozen		1	SS	4					
	Firm					610				
			2	SS	1					
	Soft		3	SS	1					
						600				
			4	TW	PH					
			5	SS	1					
						590				
			6	SS	1					
	Firm		7	SS	1					
						580				
			8	TW	PH					
						570				
			9	SS	2					
			10	SS	3					
560.0						560				
59.0	Silty fine sand to sandy silt. Compact to Dense		11	SS	34					
551.3			12	SS	30					
67.7	End of Borehole Refusal					550				



DIST. 18 HWY. 17 BORING DATE MARCH 22, 1975 COMPILED BY C.McK.

DATUM GEODETIC BOREHOLE TYPE AUGER AND SAMPLE WITH CME 55 MACHINE CHECKED BY

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	'N' VALUES
620.0	GROUND LEVEL			
	Topsoil	1	SS	8
	Clay, some sand			
615.0	V. Stiff	2	SS	13
5.0	Silty Clay to clay	3	SS	5
	V. Stiff			
	Soft to Firm	4	TW	PH
		5	SS	2
		6	TW	PH
		7	SS	1/18"
		8	TW	PH
		9	SS	1/18"
581.5 / 38.5	Sand & Gravel	10	SS	27
	Compact to V.Dense			
568.5 / 51.5	End of Borehole	11	SS	100/4.5"
	Refusal			



LIQUID LIMIT — w_L
PLASTIC LIMIT — w_P
WATER CONTENT — w
WATER CONTENT %

UNIT WEIGHT γ

REMARKS % GR. SA SI

20 / 15 ◇ 5 / 10 % STRAIN AT FAILURE

OFFICE REPORT ON SOIL EXPLORATION

DIST. 18 HWY. 17 BORING DATE APRIL 26, 1975 COMPILED BY H.S.

DATUM GEODETIC BOREHOLE TYPE HOLLOW STEM - 2 3/4" CHECKED BY [illegible]

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLE NUMBER	TYPE	'N' VALUES	GROUND WATER ELEV.
617.0	GROUND LEVEL				
0.0	Topsoil				
	Silty Sand to Sand	1	SS	10	
		2	SS	4	
	Very loose to loose	3	SS	2	610
		4	SS	4	
		5	SS	2	
		6	SS	6	600
596.0					
21.0	Sand & Gravel compact / v.dense	7	SS	11	
		8	SS	71	590
	with cobbles	9	SS	100/ 6"	
	and few boulders	10	SS	76/ 3"	580
		11	SS	200/ 3"	
570.0					
47.0	End of Borehole Refusal to Augering probable boulder				570
					560

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH P.S.F.: O UNCONFINED; + FIELD VANE; ● QUICK TRIAXIAL; × LAB VANE

LIQUID LIMIT —— w_L; PLASTIC LIMIT —— w_P; WATER CONTENT —— w

WATER CONTENT %

UNIT WEIGHT γ

REMARKS % GR SA SI

20 15 ◇ 5 10 % STRAIN AT FAILURE

DIST. 18 HWY. 17

DATUM GEODETIC

BORING DATE APRIL 28, 1975

BOREHOLE TYPE HOLLOW STEM AUGERS - 2 3/4"

ORIGINATED BY HS

COMPILED BY H.S.

CHECKED BY

SOIL PROFILE: ELEV DEPTH	SOIL PROFILE: DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: 'N' VALUES	GROUND WATER	ELEV.	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH P.S.F. ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 400 800 1200 1600 2000	LIQUID LIMIT w_L / PLASTIC LIMIT w_P / WATER CONTENT w / WATER CONTENT %	UNIT WEIGHT γ	REMARKS GR SA S %
615.0	GROUND LEVEL										
0.0	Topsoil		1	SS	10	▼					
	Silty Sand to Sand		2	SS	2/18"		610				
	Very Loose to loose		3	SS	4						
			4	SS	4		600				
590.5			5	SS	4						
24.5	Sand & Gravel Compact with V. Dense cobbles		6	SS	12		590				
			7	SS	96						
579.5			8	SS	100/6"		580				
35.5	End of Borehole Refusal to Augering Probable Boulder										
							570				

20 / 15 ◇ 5 / 10 % STRAIN AT FAILURE

13 ?

W.P. ... LOCATION HWY. 17 & CPR (AS SHOWN ON PLAN) ORIGINATED BY H.S.

DIST. 18 HWY. 17 BORING DATE APRIL 28, 1975 COMPILED BY H.S.

DATUM GEODETIC BOREHOLE TYPE HOLLOW STEM AUGERS - 2 3/4" CHECKED BY

ELEV. / DEPTH	SOIL PROFILE – DESCRIPTION	STRAT PLOT	SAMPLES – NUMBER	TYPE	'N' VALUES	GROUND WATER ELEV.	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH P.S.F. (○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE) 400 800 1200 1600 2000	LIQUID LIMIT w_L / PLASTIC LIMIT w_P / WATER CONTENT w / WATER CONTENT %	UNIT WEIGHT γ	REMARKS % GR SA SI
616.0	GROUND LEVEL									
0.0	Topsoil									
613.0 / 3.0	Clay, Trace of Sand Firm		1	SS	7	▽				
			2	SS	2	610	+S=8.0 +S=5.9			
617.0 / 9.0	Silty Sand to Sand Very Loose		3	SS	2					
			4	SS	4	600				
592.0 / 24.0	Sand and Gravel with cobbles V. Loose		5	SS	2	590				
	V. Dense		6	SS	69					
583.5 / 32.5	End of Borehole Refusal to Augering Probable Boulder					580				

20 / 15 ◇ 5 / 10 % STRAIN AT FAILURE

W.P. [illegible] LOCATION [illegible] ORIGINATED BY H.S.

DIST. 18 HWY. 17 BORING DATE APRIL 28, 1975 COMPILED BY H.S.

DATUM GEODETIC BOREHOLE TYPE HOLLOW STEM AUGERS - 2 3/4" CHECKED BY [illegible]

ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES	GROUND WATER ELEV.	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH P.S.F. o UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	LIQUID LIMIT w_L PLASTIC LIMIT w_P WATER CONTENT w / WATER CONTENT %	UNIT WEIGHT γ	REMARKS % GR SA SI
617.0	GROUND LEVEL									
0.0	Topsoil					▽				
	Silty Sand to Sand		1	SS	12					
			2	SS	4	610				
	V. Loose to Compact		3	SS	3					
						600				
			4	SS	3					
591.0 / 26.0	Sand and Gravel with cobbles V. Dense		5	SS	100	590				
588.5 / 28.5	End of Borehole Refusal to Augering Probable Boulder									
						580				

20 / 15 ◇ 5 / 10 % STRAIN AT FAILURE

15 ?

OFFICE REPORT ON SOIL EXPLORATION

W.P. [illegible] LOCATION [illegible] ORIGINATED BY [illegible]

DIST. 18 HWY. 17 BORING DATE APRIL 28-29, 1975 COMPILED BY H.S.

DATUM GEODETIC BOREHOLE TYPE HOLLOW STEM AUGERS - 2-3/4" CHECKED BY [illegible]

ELEV. / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	'N' VALUES	GROUND WATER / ELEV
618.0 (188.36)	GROUND LEVEL				
0.0	Topsoil				▽
	Silty Sand to Sand	1	SS	4	
	Very Loose				610
		2	SS	3	
601.5					
16.5 (5.03)	Sand & Gravel				600
		3	SS	19	
	With cobbles Compact / V. Dense	4	SS	100/4"	
589.0					590
29.0 (8.84)	End of Borehole Refusal to Augering Probable Boulder				
					580

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH P.S.F.: O UNCONFINED; ● QUICK TRIAXIAL; + FIELD VANE; × LAB VANE

LIQUID LIMIT — w_L; PLASTIC LIMIT — w_P; WATER CONTENT — w; WATER CONTENT %

UNIT WEIGHT γ

REMA[RKS] % GR SA

20 / 15 ◆ 5 / 10 % STRAIN AT FAILURE

167

W.P. ____ LOCATION ____ ORIGINATED BY ____

DIST. 18 HWY. 17 BORING DATE APRIL 29, 1975 COMPILED BY HS

DATUM GEODETIC BOREHOLE TYPE HOLLOW STEM AUGERS – 2 3/4" CHECKED BY ____

SOIL PROFILE			SAMPLES			GROUND WATER	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH P.S.F. (○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE)	LIQUID LIMIT w_L / PLASTIC LIMIT w_P / WATER CONTENT w / WATER CONTENT %	UNIT WEIGHT γ	REMARKS % GR SA S
ELEV. / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES	ELEV.				
623.0 (189.89)	GROUND LEVEL									
0.0	Sand; Some gravel									
			1	SS	31	620				
			2	SS	17					
613.5	Compact to Dense		3	SS	34					
9.5 / 2.9	Silty Sand to Sand,		4	SS	15					
						610				
	Some Gravel		5	SS	10					
	Loose to Compact		6	SS	12					
						600				
			7	SS	6					
			8	SS	10					
						590				
20.7?	Cobbles									
593.0										
40.0 / 12.19	End of Borehole Refusal to Auger probable boulder					580				

20 / 15 ◇ 5 / 10 % STRAIN AT FAILURE

17?

OFFICE REPORT ON SOIL EXPLORATION

LOCATION LONG BAY (AS SHOWN ON PLAN)

DIST. 18 HWY. 17

BORING DATE APRIL 29, 1975

DATUM GEODETIC

BOREHOLE TYPE HOLLOW STEM AUGERS - 2 3/4"

ORIGINATED BY H.S.

COMPILED BY H.S.

CHECKED BY [illegible]

SOIL PROFILE: ELEV / DEPTH	SOIL PROFILE: DESCRIPTION	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: 'N' VALUES	GROUND WATER ELEV.	SHEAR STRENGTH P.S.F. (+ FIELD VANE)	REMARKS
615.0	GROUND LEVEL						
0.0 / 612.0	Topsoil						
3.0	Silty Clay to Clay — Soft / Firm	1	SS	2	610	+S=3.5	
		2	SS	1		+S=2.4	
						+S=2.6	
		3	TW	PM		+S=7.5	
						+S=6.2	
		4	SS	2	600		
596.0						+S=5.7	
19.0	Silty Sand to Sand	5	SS	4			W.L. not establis[hed]
	Very Loose	6	SS	4	590		
581.8		7	SS	2			
		8	SS	[illegible]"			
33.2	End of Borehole Refusal to Augering Probable Boulder				580		

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH P.S.F.: O UNCONFINED; + FIELD VANE; ● QUICK TRIAXIAL; × LAB VANE — 400 800 1200 1600 2000

LIQUID LIMIT — w_L; PLASTIC LIMIT — w_P; WATER CONTENT — w; WATER CONTENT %

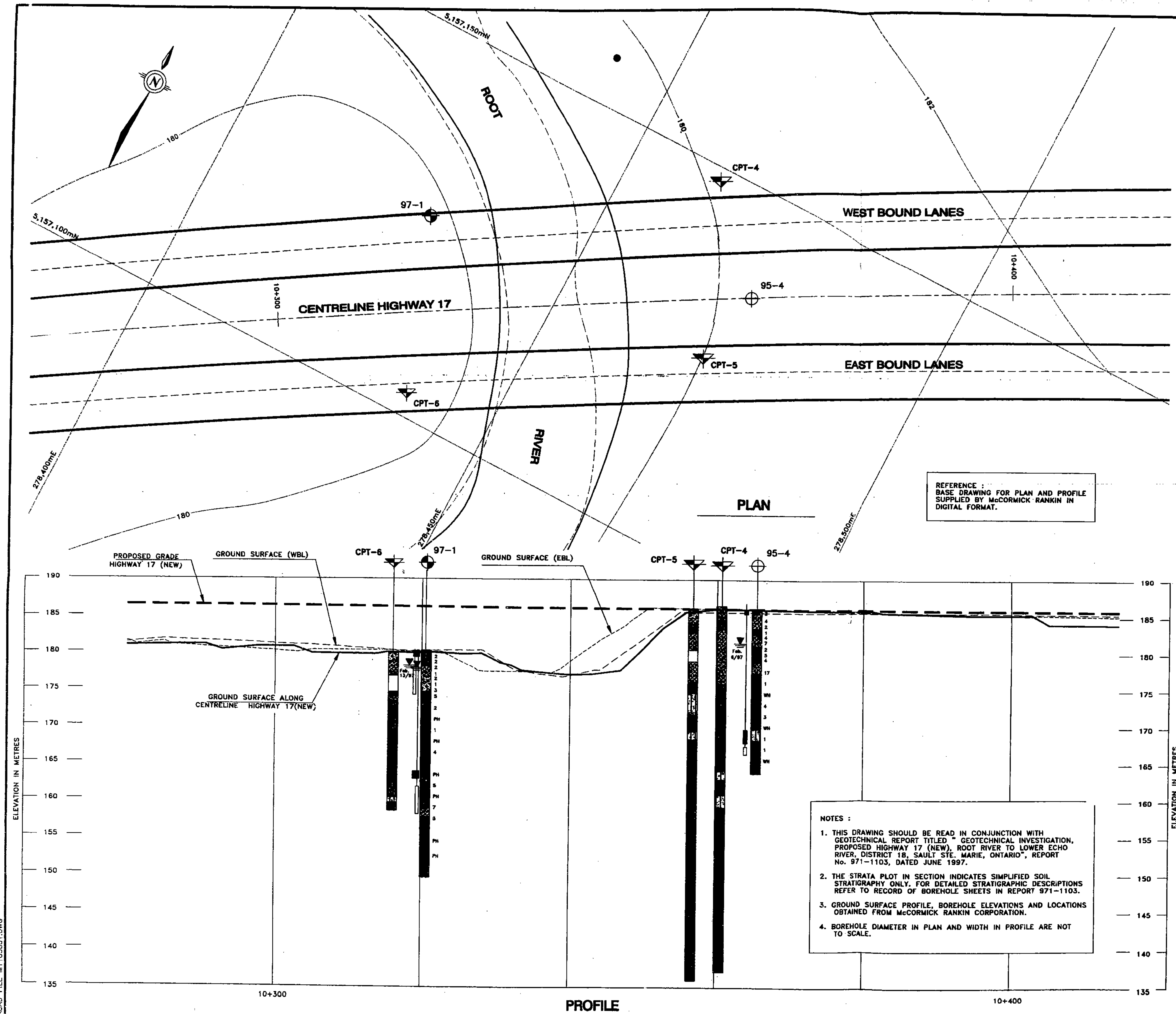
UNIT WEIGHT γ

% GR SA S[I]

15 / 20 / 5 / 10 % STRAIN AT FAILURE

18

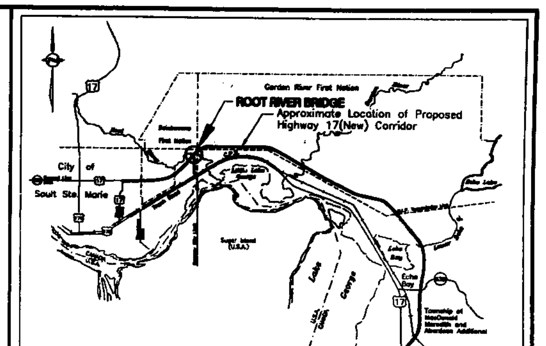
OFFICE REPORT ON SOIL EXPLORATION



REFERENCE :
BASE DRAWING FOR PLAN AND PROFILE SUPPLIED BY McCORMICK RANKIN IN DIGITAL FORMAT.

NOTES :

1. THIS DRAWING SHOULD BE READ IN CONJUNCTION WITH GEOTECHNICAL REPORT TITLED " GEOTECHNICAL INVESTIGATION, PROPOSED HIGHWAY 17 (NEW), ROOT RIVER TO LOWER ECHO RIVER, DISTRICT 18, SAULT STE. MARIE, ONTARIO", REPORT No. 971-1103, DATED JUNE 1997.
2. THE STRATA PLOT IN SECTION INDICATES SIMPLIFIED SOIL STRATIGRAPHY ONLY. FOR DETAILED STRATIGRAPHIC DESCRIPTIONS REFER TO RECORD OF BOREHOLE SHEETS IN REPORT 971-1103.
3. GROUND SURFACE PROFILE, BOREHOLE ELEVATIONS AND LOCATIONS OBTAINED FROM McCORMICK RANKIN CORPORATION.
4. BOREHOLE DIAMETER IN PLAN AND WIDTH IN PROFILE ARE NOT TO SCALE.



HIGHWAY 17 (New)
SAULT STE. MARIE to BAR RIVER ROAD

LEGEND

97-1 BOREHOLE LOCATION – CURRENT INVESTIGATION
95-4 BOREHOLE LOCATION – PREVIOUS INVESTIGATION BY GOLDER ASSOC. LTD. (REPORT 941-1364)
CPT-5 CONE PENETRATION TEST – CURRENT INVESTIGATION

BENTONITE SEAL
WATER LEVEL
PIEZOMETER
'N' VALUE, blows/0.3m
STRATA PLOT (SEE DESCRIPTIONS BELOW)

STATIGRAPHY DESCRIPTIONS

- TOPSOIL
- SANDY SILT with fine to medium sand layers, trace organics Very loose, brown
- SAND, fine to coarse, trace to some silt, trace gravel, trace organics, Very loose to compact, brown and grey
- SAND and GRAVEL, grading to sand, with some gravel Very loose, grey
- SILT, SANDY SILT and CLAYEY SILT, interlayered, trace organics Loose to very loose/soft, grey
- SILTY CLAY to CLAY and CLAYEY SILT to SILT, trace sand, irregularly layered, Firm to stiff, greyish brown to brown
- SILTY SAND, Very loose

0 5.0 10.0 15.0 20.0 25.0m
SCALE 1:250

NO.	DATE	REVISION	BY	CHK	ENG	APP

Golder Associates
Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA

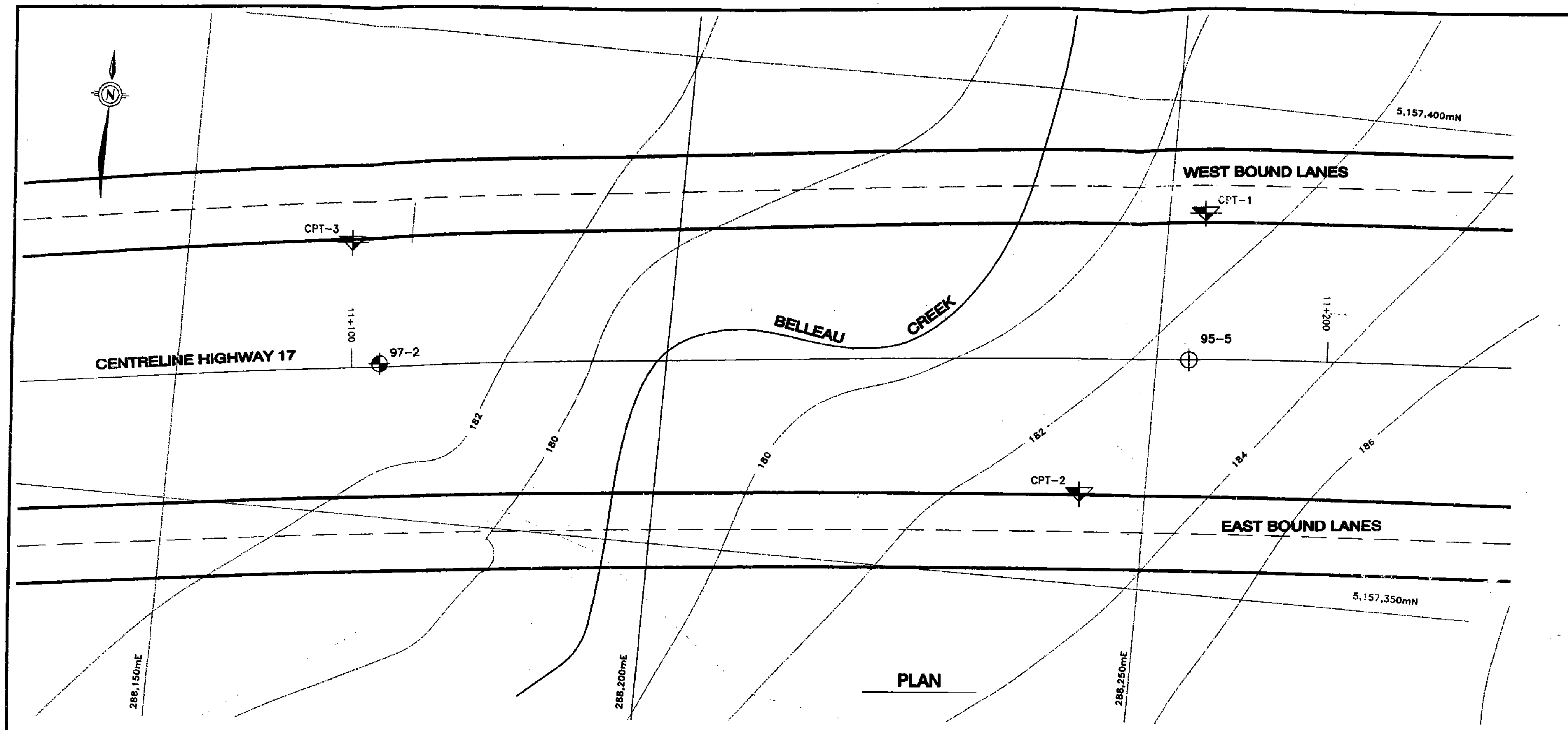
McCORMICK RANKIN CORPORATION
MISSISSAUGA, ONTARIO

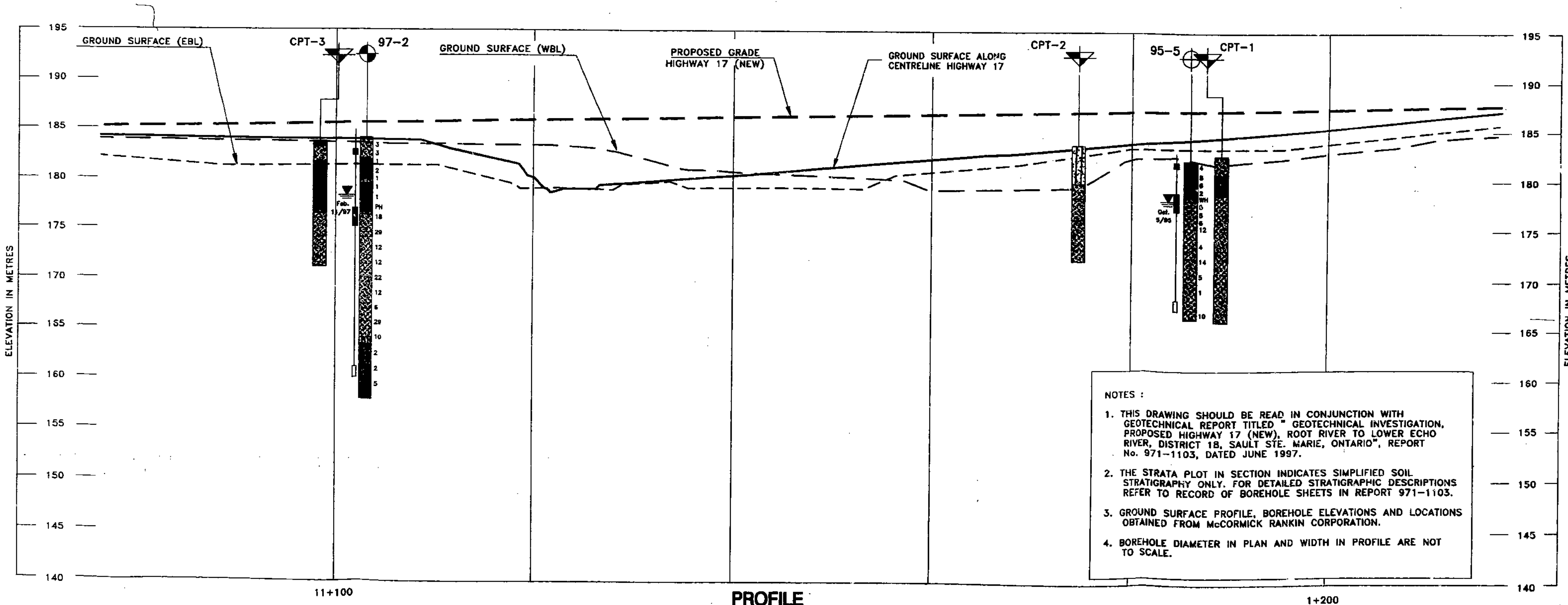
HIGHWAY 17

PLAN AND PROFILE
ROOT RIVER BRIDGE

DRAWN: MHW CHKD: AMP
DATE: MARCH 16, 1997
PROJECT NO.: 971-1103
DRAWING NUMBER 1
REV. A

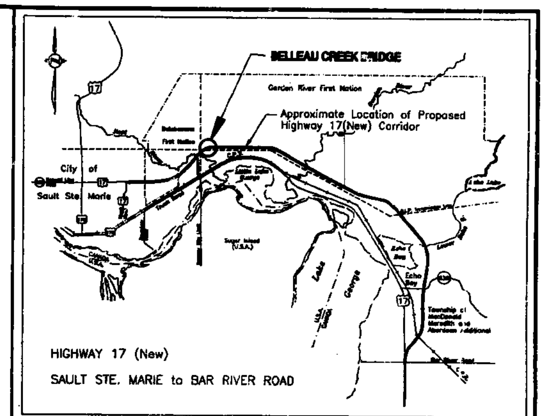
ACAD FILE M1103001.DWG
1:250





NOTES :

1. THIS DRAWING SHOULD BE READ IN CONJUNCTION WITH GEOTECHNICAL REPORT TITLED " GEOTECHNICAL INVESTIGATION, PROPOSED HIGHWAY 17 (NEW), ROOT RIVER TO LOWER ECHO RIVER, DISTRICT 18, SAULT STE. MARIE, ONTARIO", REPORT No. 971-1103, DATED JUNE 1997.
2. THE STRATA PLOT IN SECTION INDICATES SIMPLIFIED SOIL STRATIGRAPHY ONLY. FOR DETAILED STRATIGRAPHIC DESCRIPTIONS REFER TO RECORD OF BOREHOLE SHEETS IN REPORT 971-1103.
3. GROUND SURFACE PROFILE, BOREHOLE ELEVATIONS AND LOCATIONS OBTAINED FROM McCORMICK RANKIN CORPORATION.
4. BOREHOLE DIAMETER IN PLAN AND WIDTH IN PROFILE ARE NOT TO SCALE.



LEGEND

97-1 BOREHOLE LOCATION – CURRENT INVESTIGATION

95-4 BOREHOLE LOCATION – PREVIOUS INVESTIGATION BY GOLDER ASSOC. LTD. (REPORT 941-1364)

CPT-5 CONE PENETRATION TEST – CURRENT INVESTIGATION

BENTONITE SEAL

WATER LEVEL

PIEZOMETER

'N' VALUE, blows/0.3m

STRATA PLOT (SEE DESCRIPTIONS BELOW)

STATIGRAPHY DESCRIPTIONS

- TOPSOIL
- SILTY SAND, Very loose
- SAND, fine to coarse, trace to some silt, trace clay and organics, Very loose to compact, brown to grey
- SILT, trace to some sand, trace organics Very loose to loose, brown to grey
- SILTY CLAY to CLAY and CLAYEY SILT to SILT, trace sand, Irregularly layered, Soft to firm, brown

REFERENCE :
BASE DRAWING FOR PLAN AND PROFILE SUPPLIED BY McCORMICK RANKIN IN DIGITAL FORMAT.

0 5.0 10.0 15.0 20.0 25.0m
SCALE 1:250

NO.	DATE	REVISION	BY	CHK	ENG	APP

Golder Associates

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA

McCORMICK RANKIN CORPORATION
MISSISSAUGA, ONTARIO

HIGHWAY 17

PLAN AND PROFILE
BELLEAU CREEK BRIDGE

DRAWN: MHW	CHKD: AMP	2	A
DATE: MARCH 18, 1997			
PROJECT NO.: 971-1103		DRAWING NUMBER	REV.

ACAD FILE M110302A.DWG

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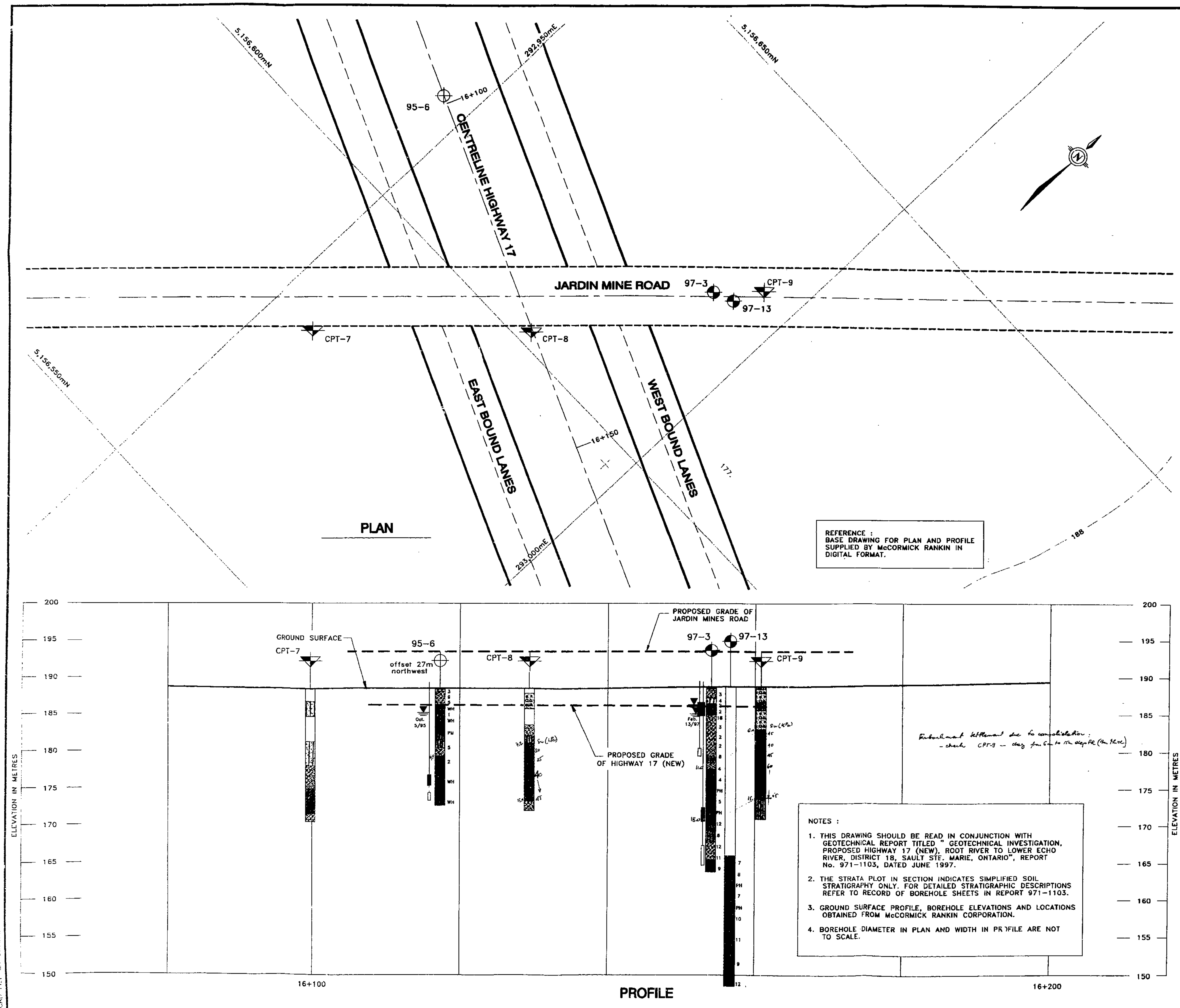
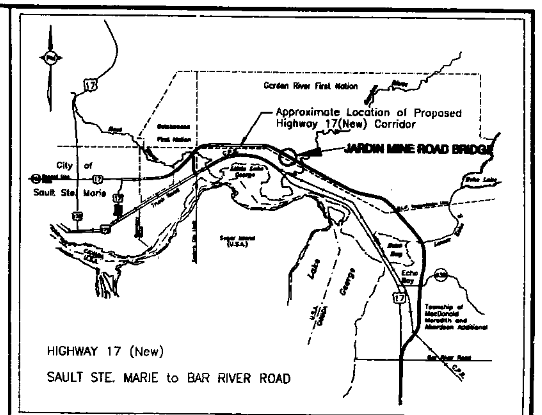




LEGEND

97-1 BOREHOLE LOCATION – CURRENT INVESTIGATION

95-4 BOREHOLE LOCATION – PREVIOUS INVESTIGATION BY GOLDER ASSOC. LTD. (REPORT 941-1364)

CPT-5 CONE PENETRATION TEST – CURRENT INVESTIGATION

BENTONITE SEAL
WATER LEVEL
PIEZOMETER
'N' VALUE, blows/0.3m
STRATA PLOT (SEE DESCRIPTIONS BELOW)

STRATIGRAPHY DESCRIPTIONS

- TOPSOIL
- SAND, SILTY SAND and SILTY CLAY, interlayered, Compact/Very stiff
- SAND, fine to medium, trace to some silt, occasional organics Very loose to compact, brown and grey
- SAND and GRAVEL
- SAND, SILT and SILTY CLAY, interlayered, trace organics, Loose to compact/soft to stiff, brown to grey
- CLAYEY SILT to SILT, trace sand, irregularly layered Firm to very stiff, greyish brown
- SILTY SAND, trace organics, Loose, brown and grey

0 5.0 10.0 15.0 20.0 25.0m
SCALE 1:250

NO.	DATE	REVISION	BY	CHK	ENG	APP

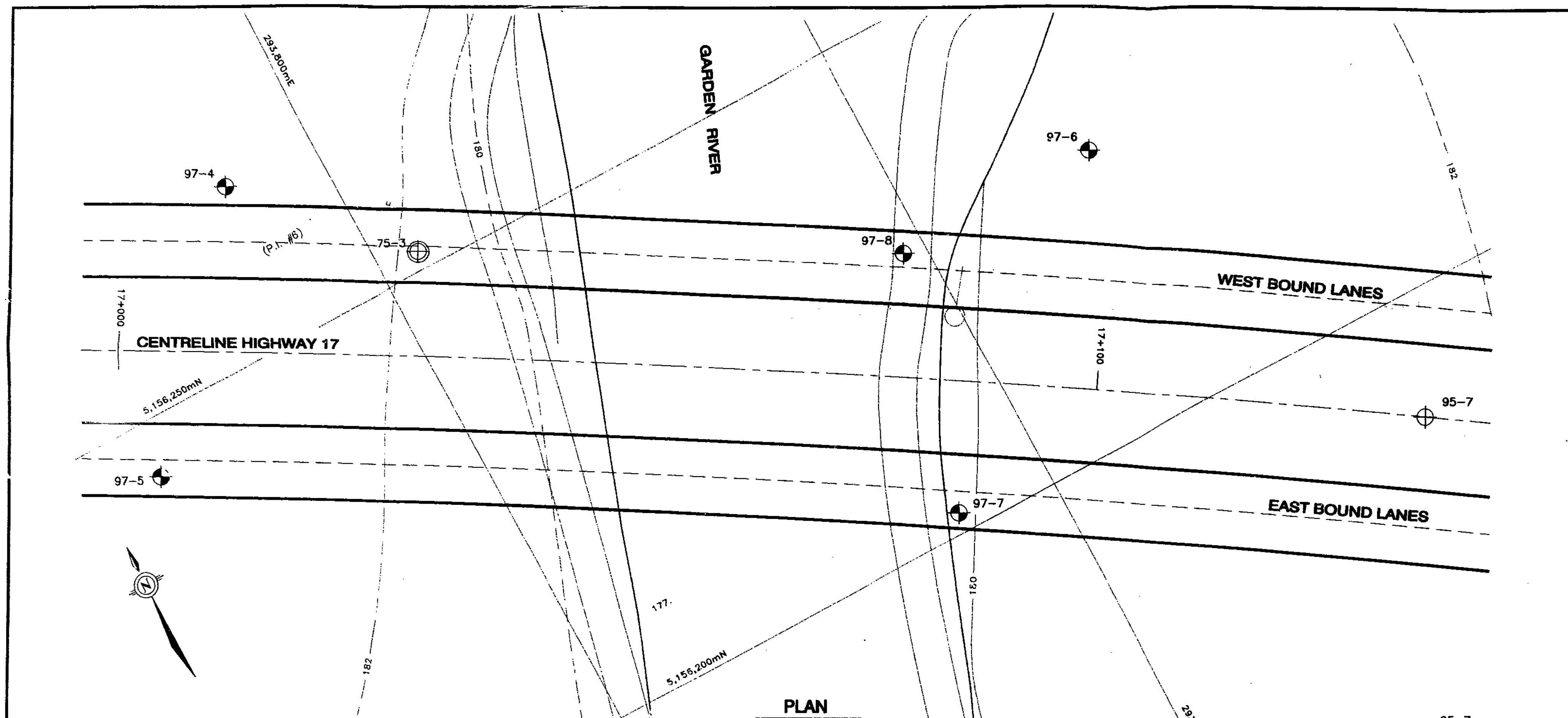
Golder Associates
Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA

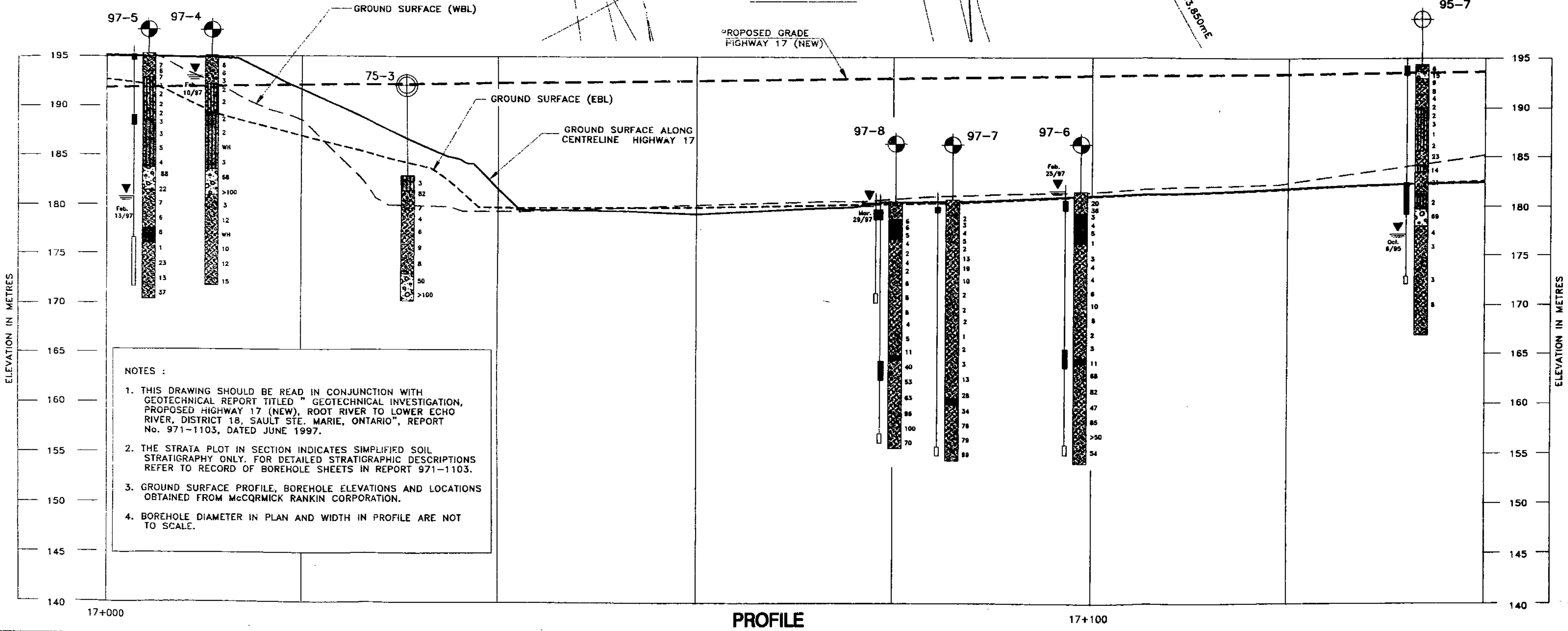
McCORMICK RANKIN CORPORATION
MISSISSAUGA, ONTARIO

HIGHWAY 17

PLAN AND PROFILE
JARDIN MINES ROAD BRIDGE

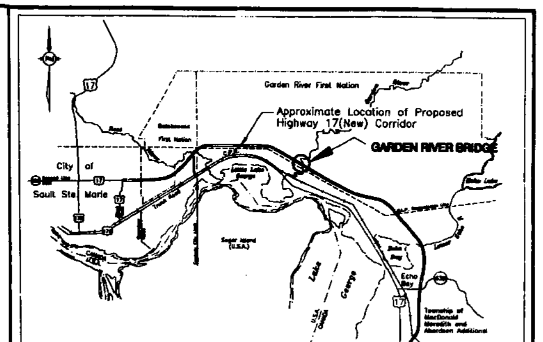
DRAWN: MHW	CHKD: AMP	3	A
DATE: MARCH 16, 1997		DRAWING NUMBER	REV.
PROJECT NO.: 971-1103			





NOTES :

1. THIS DRAWING SHOULD BE READ IN CONJUNCTION WITH GEOTECHNICAL REPORT TITLED " GEOTECHNICAL INVESTIGATION, PROPOSED HIGHWAY 17 (NEW), ROOT RIVER TO LOWER ECHO RIVER, DISTRICT 18, SAULT STE. MARIE, ONTARIO", REPORT No. 971-1103, DATED JUNE 1997.
2. THE STRATA PLOT IN SECTION INDICATES SIMPLIFIED SOIL STRATIGRAPHY ONLY. FOR DETAILED STRATIGRAPHIC DESCRIPTIONS REFER TO RECORD OF BOREHOLE SHEETS IN REPORT 971-1103.
3. GROUND SURFACE PROFILE, BOREHOLE ELEVATIONS AND LOCATIONS OBTAINED FROM McCQRMICK RANKIN CORPORATION.
4. BOREHOLE DIAMETER IN PLAN AND WIDTH IN PROFILE ARE NOT TO SCALE.



HIGHWAY 17 (New)
SAULT STE. MARIE to BAR RIVER ROAD

LEGEND

97-1 BOREHOLE LOCATION – CURRENT INVESTIGATION

95-4 BOREHOLE LOCATION – PREVIOUS INVESTIGATION BY GOLDER ASSOC. LTD. (REPORT 941-1364)

75-3 BOREHOLE LOCATION – PREVIOUS INVESTIGATION BY MTO (1975)

CPT-5 CONE PENETRATION TEST – CURRENT INVESTIGATION

BENTONITE SEAL
WATER LEVEL
PIEZOMETER
'N' VALUE, blows/0.3m
STRATA PLOT (SEE DESCRIPTIONS BELOW)

STATIGRAPHY DESCRIPTIONS

- TOPSOIL
- SANDY SILT and SILTY SAND, trace clay, some organics interlayered, Very loose, grey
- SAND, fine to coarse, trace gravel, silt and silty clay interlayered, Very loose to very dense, brown to grey
- SAND and GRAVEL, trace silt, cobbles and boulders Compact to very dense, brown and grey
- SILT, SANDY SILT and SAND, trace clay, trace organics, interlayered, Very loose to loose, grey
- SILTY CLAY, some sand, silt and sand partings/seams and fine sand, interlayered, Soft to firm/very loose to loose

REFERENCE :
BASE DRAWING FOR PLAN AND PROFILE SUPPLIED BY McCORMICK RANKIN IN DIGITAL FORMAT.

0 5.0 10.0 15.0 20.0 25.0m
SCALE 1:250

NO.	DATE	REVISION	BY	CHK	ENG	APP

Golder Associates

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA

McCORMICK RANKIN CORPORATION
MISSISSAUGA, ONTARIO

HIGHWAY 17

PLAN AND PROFILE
GARDEN RIVER BRIDGE

DRAWN: MHW	CHKD: AMP	4	A
DATE: MARCH 16, 1997		DRAWING NUMBER	REV.
PROJECT NO.: 971-1103			

ACAD FILE M1103O4A.DWG

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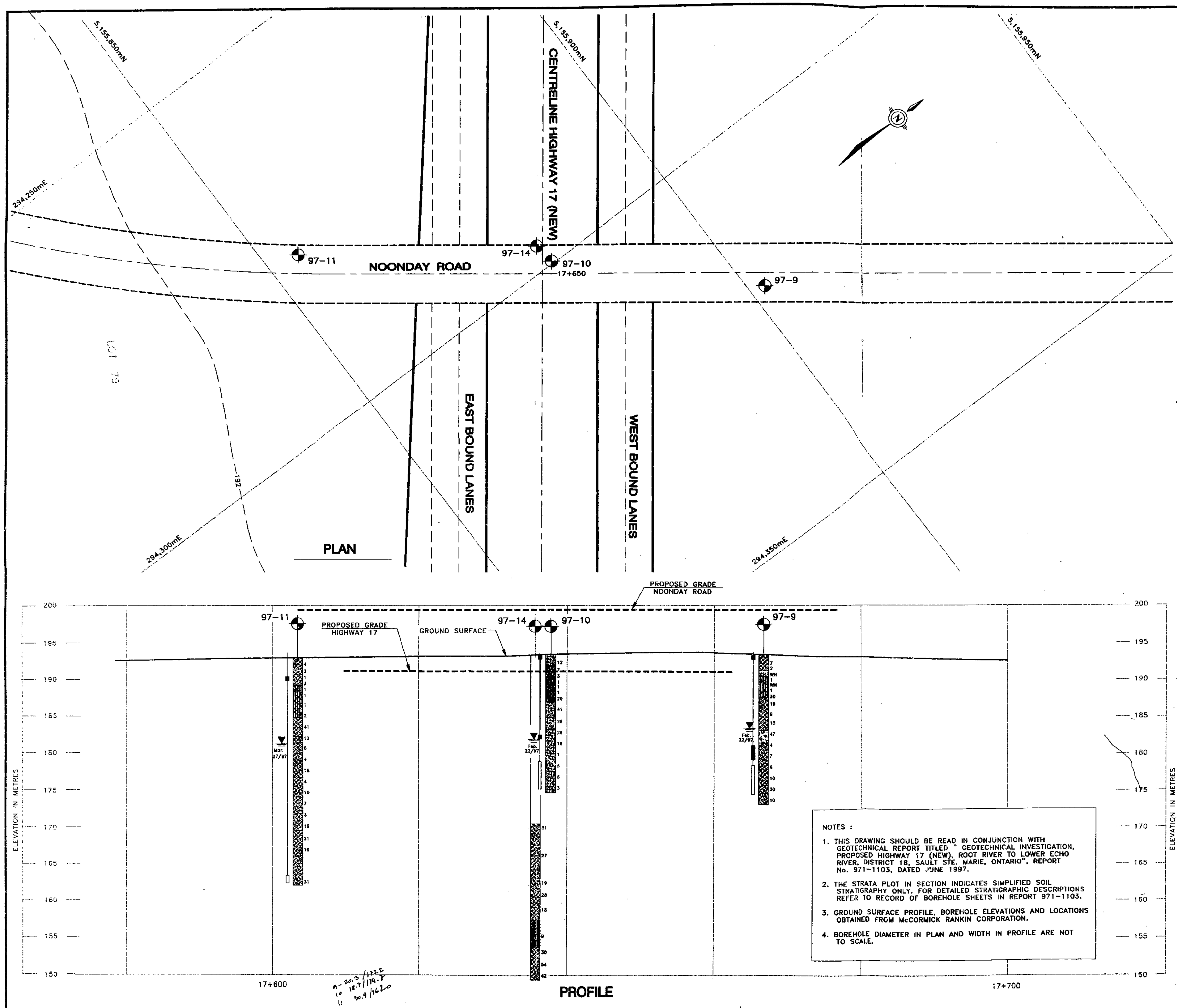

NOTES :

1. THIS DRAWING SHOULD BE READ IN CONJUNCTION WITH GEOTECHNICAL REPORT TITLED " GEOTECHNICAL INVESTIGATION, PROPOSED HIGHWAY 17 (NEW), ROOT RIVER TO LOWER ECHO RIVER, DISTRICT 18, SAULT STE. MARIE, ONTARIO", REPORT No. 971-1103, DATED JUNE 1997.
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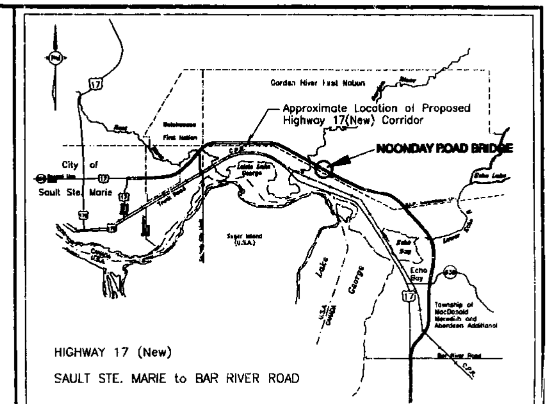
3. GROUND SURFACE PROFILE, BOREHOLE ELEVATIONS AND LOCATIONS OBTAINED FROM McCORMICK RANKIN CORPORATION.
4. BOREHOLE DIAMETER IN PLAN AND WIDTH IN PROFILE ARE NOT TO SCALE.



LEGEND

97-1 BOREHOLE LOCATION - CURRENT INVESTIGATION

BENTONITE SEAL
WATER LEVEL
PIEZOMETER
'N' VALUE, blows/0.3m
STRATA PLOT (SEE DESCRIPTIONS BELOW)

STATIGRAPHY DESCRIPTIONS

- TOPSOIL
- SANDY SILT To SAND and SILT, trace clay, trace gravel, trace organics Very loose, grey
- SAND, fine to coarse, trace to some silt, trace to some gravel, trace organics, Very loose to dense, brown to grey
- SAND and GRAVEL, trace silt, Dense, grey
- SILTY CLAY, trace sand, occas. fine sand seams Stiff to very stiff, brown

REFERENCE :
BASE DRAWING FOR PLAN AND PROFILE SUPPLIED BY McCORMICK RANKIN IN DIGITAL FORMAT.

0 5.0 10.0 15.0 20.0 25.0m
SCALE 1:250

NO.	DATE	REVISION	BY	CHK	ENG	APP

Golder Associates

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA

McCORMICK RANKIN CORPORATION
MISSISSAUGA, ONTARIO

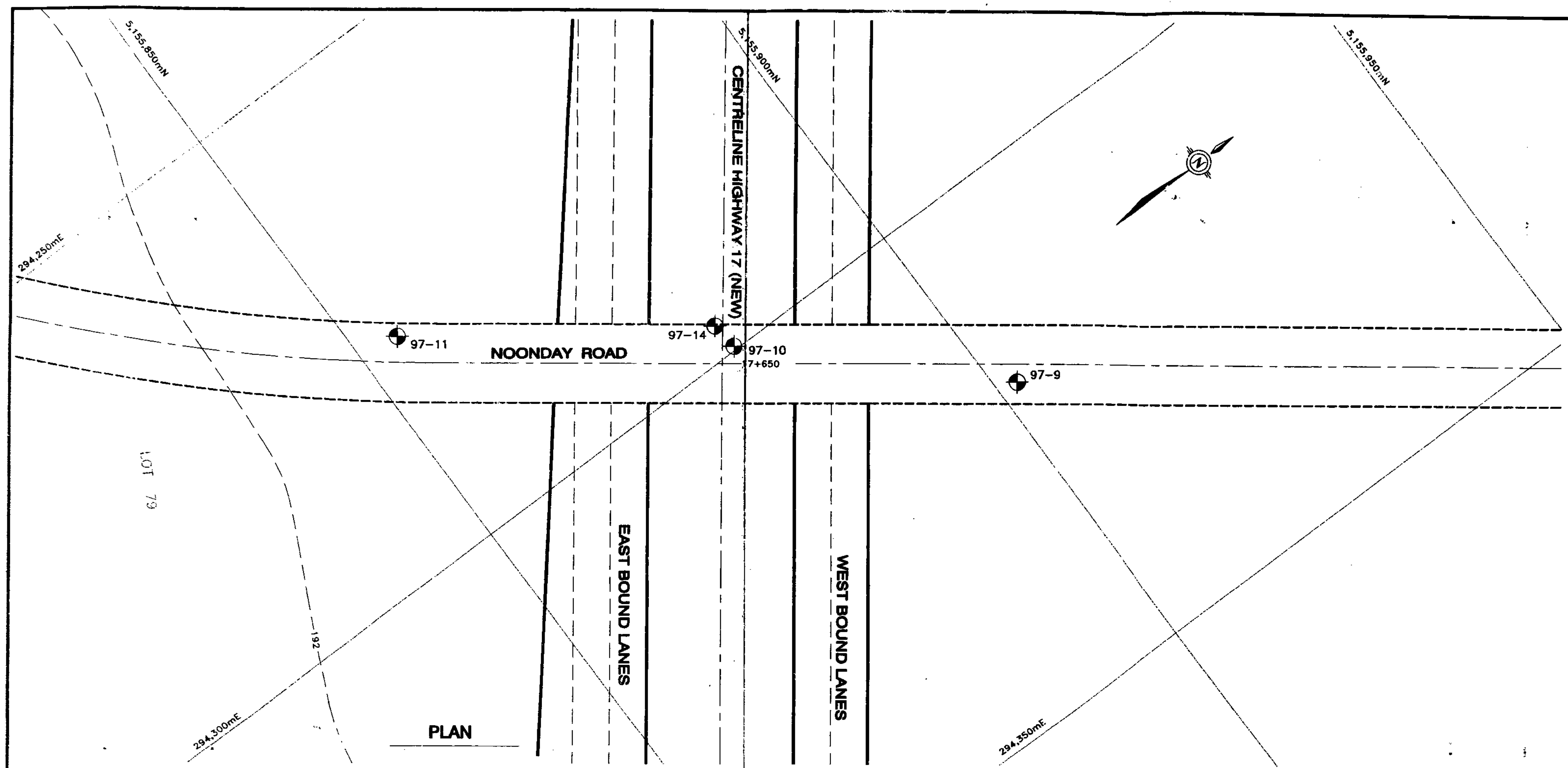
HIGHWAY 17

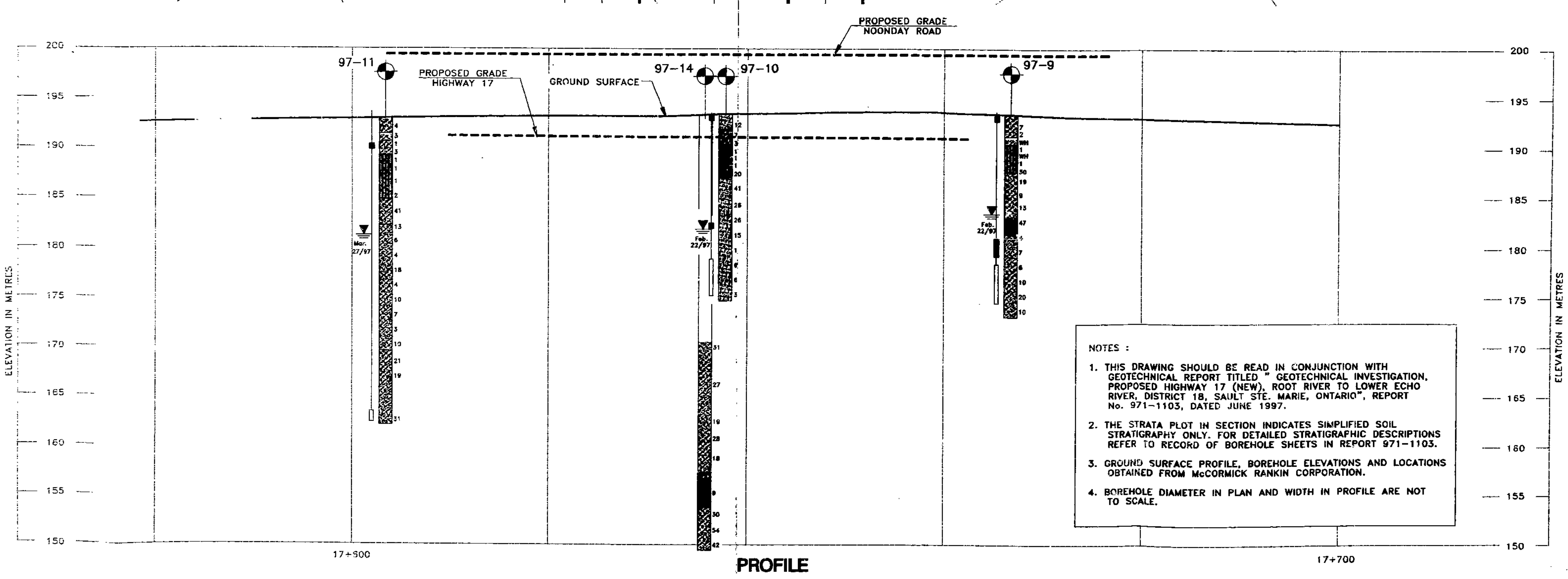
PLAN AND PROFILE
NOONDAY ROAD BRIDGE

DRAWN: MHW	CHKD: AMP	5	A
DATE: MARCH 18, 1997		DRAWING NUMBER	REV.
PROJECT NO.: 971-1103			

ACAD FILE M1103056.DWG

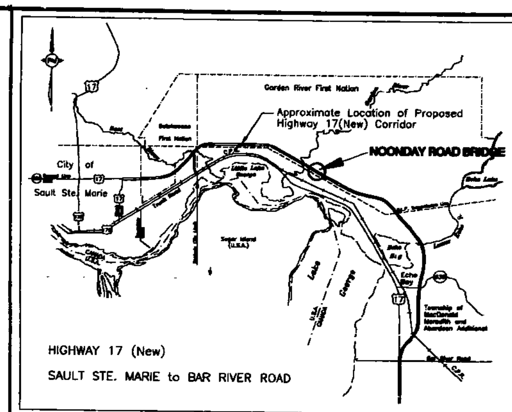
1:250





NOTES :

1. THIS DRAWING SHOULD BE READ IN CONJUNCTION WITH GEOTECHNICAL REPORT TITLED " GEOTECHNICAL INVESTIGATION, PROPOSED HIGHWAY 17 (NEW), ROOT RIVER TO LOWER ECHO RIVER, DISTRICT 18, SAULT STE. MARIE, ONTARIO", REPORT No. 971-1103, DATED JUNE 1997.
2. THE STRATA PLOT IN SECTION INDICATES SIMPLIFIED SOIL STRATIGRAPHY ONLY. FOR DETAILED STRATIGRAPHIC DESCRIPTIONS REFER TO RECORD OF BOREHOLE SHEETS IN REPORT 971-1103.
3. GROUND SURFACE PROFILE, BOREHOLE ELEVATIONS AND LOCATIONS OBTAINED FROM McCORMICK RANKIN CORPORATION.
4. BOREHOLE DIAMETER IN PLAN AND WIDTH IN PROFILE ARE NOT TO SCALE.



HIGHWAY 17 (New)
SAULT STE. MARIE to BAR RIVER ROAD

LEGEND

97-1 BOREHOLE LOCATION – CURRENT INVESTIGATION

BENTONITE SEAL
WATER LEVEL
PIEZOMETER
'N' VALUE, blows/0.3m
STRATA PLOT (SEE DESCRIPTIONS BELOW)

STATIGRAPHY DESCRIPTIONS

- TOPSOIL
- SANDY SILT To SAND and SILT, trace clay, trace gravel, trace organics Very loose, grey
- SAND, fine to coarse, trace to some silt, trace to some gravel, trace organics, Very loose to dense, brown to grey
- SAND and GRAVEL, trace silt, Dense, grey
- SILTY CLAY, trace sand, occas. fine sand seams Stiff to very stiff, brown

REFERENCE :
BASE DRAWING FOR PLAN AND PROFILE SUPPLIED BY McCORMICK RANKIN IN DIGITAL FORMAT.

0 5.0 10.0 15.0 20.0 25.0m
SCALE 1:250

NO.	DATE	REVISION	BY	CHK	ENG	APP

Golder Associates

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA

McCORMICK RANKIN CORPORATION
MISSISSAUGA, ONTARIO

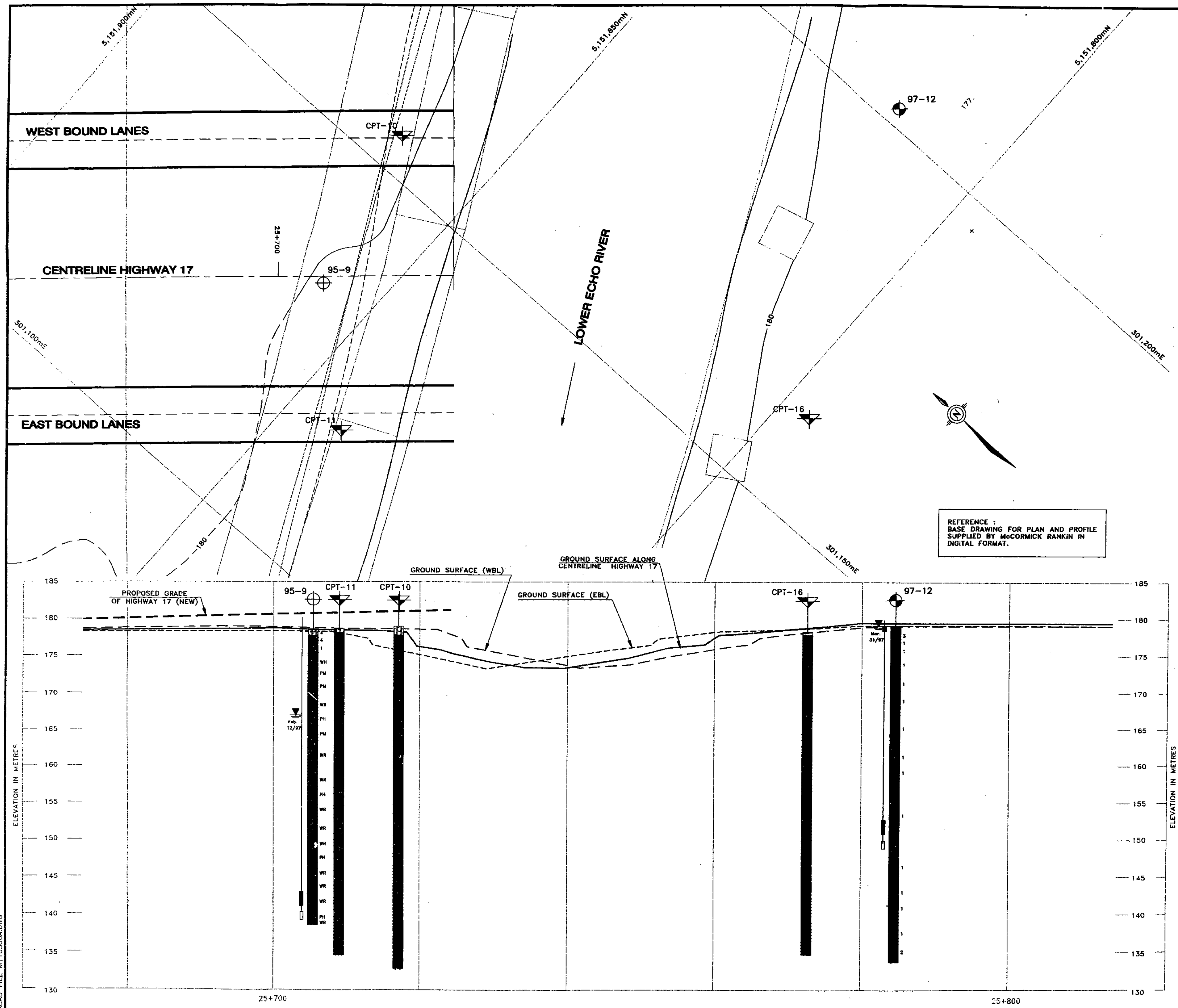
HIGHWAY 17

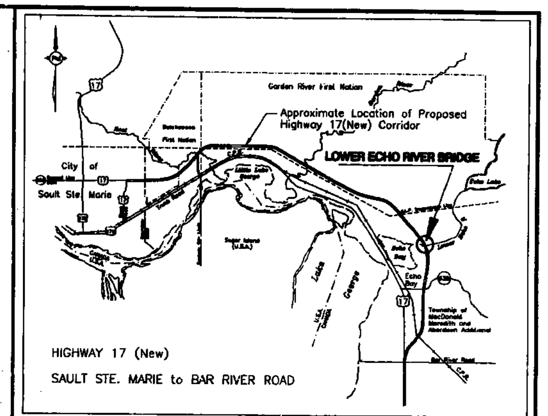
PLAN AND PROFILE
NOONDAY ROAD BRIDGE

DRAWN: MHW	CHKD: AMP	5	A
DATE: MARCH 16, 1997			
PROJECT NO.: 971-1103		DRAWING NUMBER	REV.

ACAD FILE M11030pb.DWG

1:250





LEGEND

97-12 BOREHOLE LOCATION – CURRENT INVESTIGATION

95-9 BOREHOLE LOCATION (approx.)– PREVIOUS INVESTIGATION BY GOLDER ASSOC. LTD. (REPORT 941-1364)

CPT-5 CONE PENETRATION TEST – CURRENT INVESTIGATION

BENTONITE SEAL
WATER LEVEL
PIEZOMETER
'N' VALUE, blows/0.3m
STRATA PLOT (SEE DESCRIPTIONS BELOW)

STATIGRAPHY DESCRIPTIONS

TOPSOIL

SILTY fine SAND, Very loose

SILTY CLAY to CLAY and CLAYEY SILT, with seams of silt irregularly layered, Firm to stiff, reddish brown

NOTES :

1. THIS DRAWING SHOULD BE READ IN CONJUNCTION WITH GEOTECHNICAL REPORT TITLED " GEOTECHNICAL INVESTIGATION, PROPOSED HIGHWAY 17 (NEW), ROOT RIVER TO LOWER ECHO RIVER, DISTRICT 18, SAULT STE. MARIE, ONTARIO",REPORT 971-1103, DATED JUNE 1997.
2. THE STRATA PLOT IN SECTION INDICATES SIMPLIFIED SOIL STRATIGRAPHY ONLY. FOR DETAILED STRATIGRAPHIC DESCRIPTIONS, REFER TO RECORD OF BOREHOLE SHEETS IN REPORT 971-1103.
3. GROUND SURFACE PROFILE, BOREHOLE ELEVATIONS AND LOCATIONS OBTAINED FROM McCORMICK RANKIN CORPORATION.
4. BOREHOLE DIAMETER IN PLAN AND WIDTH IN PROFILE ARE NOT TO SCALE.

0 5.0 10.0 15.0 20.0 25.0m
SCALE 1:250

NO.	DATE	REVISION	BY	CHK	ENG	APP

Golder Associates

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA

McCORMICK RANKIN CORPORATION

MISSISSAUGA, ONTARIO

HIGHWAY 17

PLAN AND PROFILE
LOWER ECHO RIVER BRIDGE

DRAWN: MHW	CHKD: AMP	
DATE: MARCH 16, 1997		6 DRAWING NUMBER
PROJECT NO.: 971-1103		REV. A

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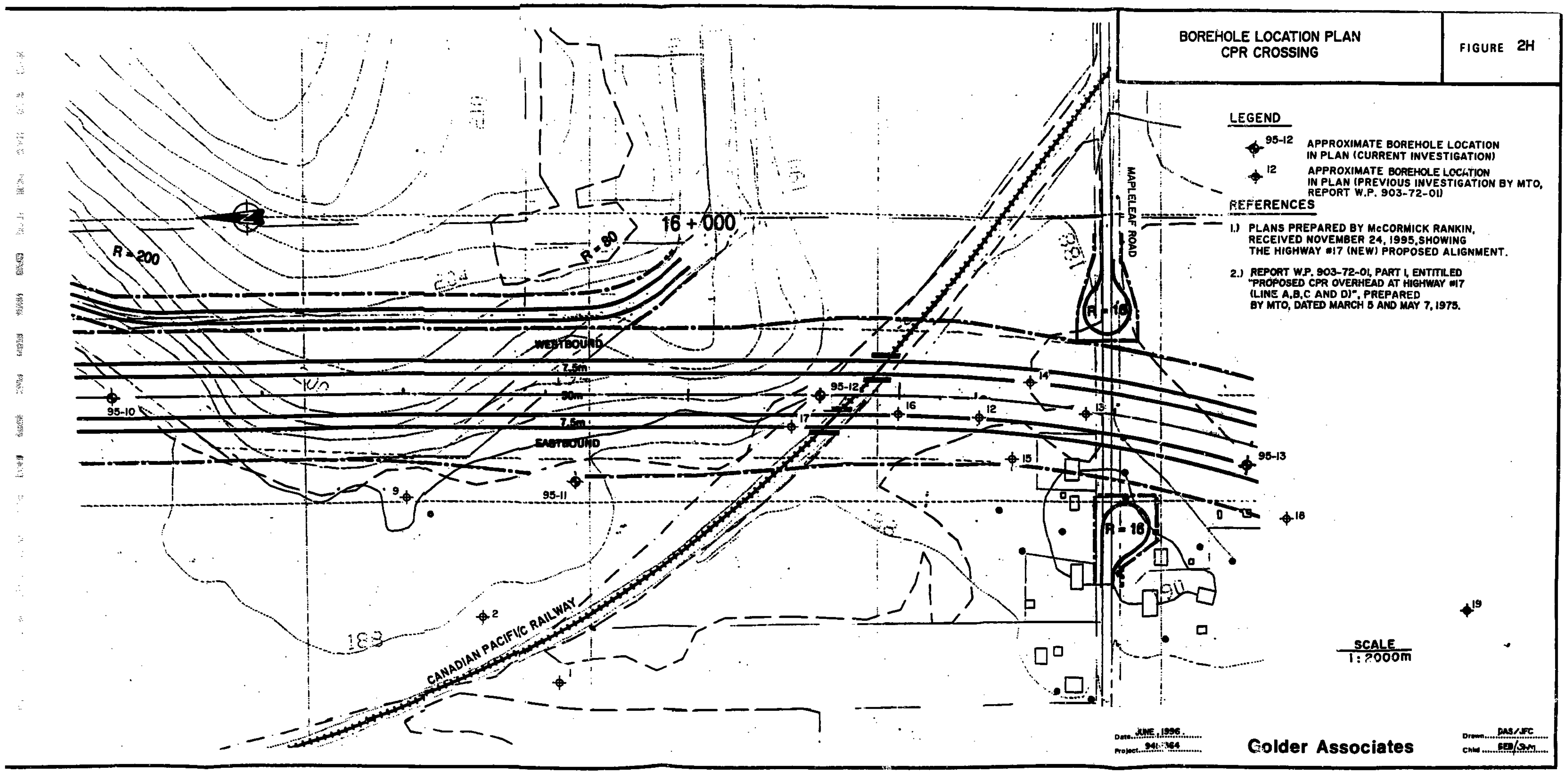
BOREHOLE LOCATION PLAN
CPR CROSSING
FIGURE 2H
LEGEND
95-12 APPROXIMATE BOREHOLE LOCATION IN PLAN (CURRENT INVESTIGATION)
12 APPROXIMATE BOREHOLE LOCATION IN PLAN (PREVIOUS INVESTIGATION BY MTO, REPORT W.P. 903-72-01)
REFERENCES
1.) PLANS PREPARED BY McCORMICK RANKIN, RECEIVED NOVEMBER 24, 1995, SHOWING THE HIGHWAY #17 (NEW) PROPOSED ALIGNMENT.
2.) REPORT W.P. 903-72-01, PART I, ENTITLED "PROPOSED CPR OVERHEAD AT HIGHWAY #17 (LINE A,B,C AND D)", PREPARED BY MTO, DATED MARCH 5 AND MAY 7, 1975.
MAPLELEAF ROAD
CANADIAN PACIFIC RAILWAY
16 + 000
R = 200
R = 80
R = 16
WESTBOUND
EASTBOUND
7.5m
30m
95-10
95-11
95-12
95-13
SCALE
1:2000m
Date JUNE, 1996
Project
Golder Associates
Drawn DAS/JFC