

GEOCRE'S No. _____

DIST. 6 REGION _____

W.P. No. _____

CONT. No. _____

W. O. No. 92-11004STR. SITE No. 37-248HWY. No. MainLOCATION Bridge #1, GardinerExpy & Lakeshore Blvd. OverNo of PAGES - 1 - Thunder River

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT. _____

REMARKS: _____



Golder Associates Ltd.
CONSULTING ENGINEERS

DRAFT REPORT ON

GARDINER EXPRESSWAY

HUMBER RIVER BRIDGE REPLACEMENT PROJECT

BRIDGE 1 AND SOUTH KINGSWAY ON-RAMP

TORONTO, ONTARIO

Submitted to:

Delcan Corporation
133 Wynford Drive
North York, Ontario
M3C 1K1

Distribution:

3 copies - Delcan Corporation
North York, Ontario

1 copy - Golder Associates Ltd.
Mississauga, Ontario

February 1992

911-1315



Golder Associates Ltd.
CONSULTING ENGINEERS

February 12, 1992

911-1315

Delcan Corporation
133 Wynford Drive
North York, Ontario
M3C 1K1

Attention: Mr. W. V. Anderson, P. Eng.

RE: DRAFT GEOTECHNICAL REPORT
GARDINER EXPRESSWAY - HUMBER RIVER BRIDGE
REPLACEMENT PROJECT
BRIDGE 1 AND SOUTH KINGSWAY ON-RAMP
TORONTO, ONTARIO

Dear Sirs:

Please find enclosed a copy of the above updated draft report for your review and comment. The report is based on our initial understanding of the proposed layout of Bridge 1 at the South Kingsway on ramp. We have addressed comments and questions raised by MTO during the Structural Design Review Sub-committee meeting of December 17, 1991, and have addressed most of the issues discussed during our meeting of January 29, 1992.

The drawings included in this report have not been revised to reflect the most recent structure alignments, however, our final report will incorporate the final selected alignments on the drawings. A borehole has been completed at the revised location of the west abutment (Bridge 1) but not been incorporated into this draft. As a note, the bedrock surface was encountered at Elevation 68.3 m in the borehole which was located at the south limit of the west abutment.

Nine samples of fill materials obtained from boreholes located on the existing road embankments on the east side of the river were submitted for chemical analysis in accordance with the Lakefill Quality Control guidelines. The samples were chosen to provide general coverage of areas where the existing road embankments will be removed during construction. The results of the testing indicate that apart from slightly elevated levels of lead in one sample, ammonia in three samples and iron in all samples, the concentrations of parameters tested are within the accepted guidelines for open water lakefill disposal. All of the concentrations were below the guidelines for restricted disposal. Further discussion will be provided in the final report.

Delcan Corporation
Mr. W. V. Anderson, P. Eng.

February 12, 1992
911-1315

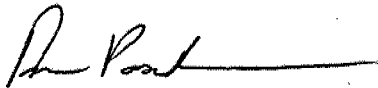
If you have any questions or comments on this draft report, please do not hesitate to contact us.

Yours truly,

GOLDER ASSOCIATES LTD.



J. Westland, P. Eng.



A. S. Poschmann, P. Eng.

JW/ASP/pds

TABLE OF CONTENTS

<u>SECTION</u>	<u>PAGE NO.</u>
Cover Letter	i
1.0 INTRODUCTION	1
2.0 SITE AND PROJECT DESCRIPTION	2
3.0 INVESTIGATION PROCEDURES	4
4.0 SUBSURFACE CONDITIONS	6
4.1 Fill	7
4.2 Sands to Silts	7
4.3 Organic Silt	8
4.4 Sand & Gravel	10
4.5 Clayey Silt to Silty Clay	10
4.6 Interlayered Clayey Silt, Silt and Silty Clay	10
4.7 Clayey Silt to Silty Clay Till	11
4.8 Residual Soil	11
4.9 Shale Bedrock	11
5.0 ENGINEERING RECOMMENDATIONS	13
5.1 Bridge 1 Approach Embankments	13
5.1.1 Embankment Settlement	14
5.1.1.1 West Abutment of Bridge 1	14
5.1.1.2 East Abutment of Bridge 1	14
5.1.1.2.1 Pre-load and Surcharge Embankments	15
5.1.1.2.2 Electro-Osmotic Treatment	17
5.1.1.2.3 Light-Weight Fill	17
5.1.1.2.4 Pile Supported Slab	18
5.2 Bridge Foundations	19
5.2.1 Driven Steel H-Piles	19
5.2.2 Cast In Situ Concrete Piles	20
5.2.3 Driven Pipe Piles	23
5.2.4 Resistance to Lateral Loads	23
5.2.5 Vibrations During Construction	25
5.2.6 Corrosion of Subgrade Steel	26
5.2.7 Frost Protection	27
5.2.8 Design Considerations for the East Abutment of Bridge 1	27
5.2.8.1 Negative Skin Friction	27
5.2.8.2 Lateral Deformation	29

TABLE OF CONTENTS CONTINUED

5.3	South Kingsway Ramp	29
5.3.1	Backfilling of Excavations prior to Ramp Construction	30
5.3.2	Earth Embankment	30
5.3.3	Geo-grid Reinforced Earth Embankment	31
5.3.4	Reinforced Earth Wall	31
5.3.5	Structural Cantilevered Retaining Wall	32
5.3.6	Influence on Existing Structure	33

TABLE 1 RESULTS OF DIAMETRAL POINT LOAD TESTS

In order following
Page 34

LIST OF ABBREVIATIONS AND SYMBOLS

RECORD OF BOREHOLE AND CONE PENETRATION TEST SHEETS

FIGURES 1 TO 13 INCLUSIVE

APPENDIX A - SELECTED BOREHOLE LOGS FROM GOLDER ASSOCIATES
REPORT 881-1312, NOVEMBER 1988.

1.0 INTRODUCTION

Golder Associates has been retained by the Municipality of Metropolitan Toronto to carry out a geotechnical investigation for the final design stage of the proposed bridge structures and associated embankment and retaining structures for the F. G. Gardiner Expressway and Lakeshore Boulevard crossing of the Humber River. The work has been carried out in consultation and coordination with Delcan Corporation, the design engineering consultant for the project. This report presents the results of the investigation and design recommendations for Bridge 1 of the project and for the on-ramp structure from the South Kingsway onto the eastbound Gardiner Expressway. The purpose of the investigation was to determine the subsoil, bedrock and groundwater conditions at the site by means of a limited number of boreholes and, based on our interpretation of these data, to provide recommendations for the geotechnical aspects of the design of the proposed structures.

Golder Associates has carried out a preliminary geotechnical investigation of the site and the results are presented in Golder Associates Report No. 881-1312, dated November 1988. Reference should be made to the above report for factual data such as borehole logs and laboratory testing results on soil samples as these have been used in the analyses and have been incorporated into the engineering design recommendations contained in this report. A selected number of the most relevant borehole logs from the previous investigation is included in an appendix to this report for ease of reference.

2.0 SITE AND PROJECT DESCRIPTION

The site is located just north of Lake Ontario on the Humber River which forms the boundary between the City of Toronto and the City of Etobicoke. The site extends north to the CN Rail tracks, east to Windermere Avenue and west to Palace Pier court, as is illustrated on Figure 1. Three bridge structures, one westbound and two eastbound, currently carry the Gardiner Expressway and Lakeshore Boulevard traffic over the Humber River.

Because of the limited available bridge width, the Westbound Lakeshore is funnelled onto the westbound Gardiner Expressway in this area. The westbound bridge is a 4-span, 4-lane wide structure. The eastbound bridges are 6-span structures that comprise 5 lanes. Construction records indicate that the bridge piers are founded on piles taken to bedrock, while the abutments are founded on piles terminated within the overburden at a depth of about 14 m. Differential settlement between the bridge piers and abutments since construction has caused an uneven road grade through this area and has contributed to the deterioration of the existing bridge structures at the site.

It is proposed to widen the Gardiner Expressway to four lanes in each direction and to provide independent crossings of the Humber River for the eastbound and westbound Lakeshore Boulevard. Their construction will increase the total number of traffic lanes crossing the Humber River from 9 lanes to 15 lanes and will, therefore, require that additional bridges be built across the Humber River. In view of the conditions of the existing structures and their location, it is proposed to demolish the existing structures, replacing them with 6 new bridges to be built over the course of 5 years. The proposed layout of the new bridges is shown, overlain on the existing conditions at the site, on Figure 2.

It is understood that the southern most bridge (Bridge No. 1) will be constructed first, along an alignment south of the existing structures. This report provides the results of the final geotechnical investigation and corresponding geotechnical engineering recommendations for design of Bridge No. 1 and the elevated section of the South Kingsway on-ramp to the eastbound Gardiner Expressway.

As was identified in Golder Associates preliminary geotechnical report, the major geotechnical design constraint at the site is the presence of an infilled valley some 250 m wide extending some 50 m west and 110 m east of the existing Humber River channel. The overburden materials within the infilled valley consist of surficial fill, sand and silts overlying an extensive deposit of compressible organic silt to the east and sand to silty sand to the west. The site is underlain by shale bedrock of the Meaford Dundas Formation, which is immediately overlain by a basal silty clay to clayey silt till.

D
R
A
F
T

3.0 INVESTIGATION PROCEDURES

The fieldwork for the investigation was carried out between April 24 and May 31, 1991 during which time a total of nine sampled boreholes, numbered 102 to 105 and 108 to 112, and three piezocone penetration tests (CPTs), numbered 101 to 103, were put down at the site. The boreholes and CPT locations from this and the previous investigation are illustrated on Figure 2. The boreholes were advanced using 200 mm diameter hollow stem augers and, at depth, NX casing and wash boring techniques. The CPT's were advanced by "pushing" the cone into the ground at a constant rate of 2 cm/s using an hydraulic ram system. The boreholes were advanced and the cones were pushed with a track-mounted Ackel AD 11 power auger drill rig supplied and operated by K & S Drilling Limited.

The boreholes were all advanced to bedrock and ranged in depth from 12 m to 36.5 m below ground surface. Overburden samples were obtained at 0.75 m to 1.5 m depth intervals with a 50 mm O.D. split spoon sampler, used as part of the Standard Penetration Test. Within the organic silt to organic clayey silt deposit, 75 mm thin walled tube samples were obtained at selected locations in place of split spoon samples. Additionally, in situ field vane tests were carried out within the organic deposit to measure the material's undrained vane shear strength. The bedrock was cored over lengths of between 3.5 m and 4.5 m using an NQ core barrel at the locations of Boreholes 102 to 105 and Borehole 108. Piezometers were installed in seven of the boreholes to permit monitoring of the groundwater level at the borehole locations.

The three cone penetration tests were put down to provide further data on the nature and extent of the organic silt deposit in the vicinity of the east abutment and embankment of Bridge 1. The cone was pushed to depths ranging from 9 m to 21 m below ground surface, terminating at the point that the drill rig was lifted off of the ground which was inferred to define the base of the organic deposit. Prior to advancing the cone, the upper fill soils were "pre-bored" to avoid premature refusal to cone advance. Over the length of cone penetration a continuous record of tip resistance, sleeve friction and dynamic porewater pressure was obtained.

The fieldwork was supervised throughout by an engineer who cleared the locations of buried services, directed the drilling and sampling operations, logged the boreholes and placed the

samples obtained in labelled, air-tight containers. The samples were identified in the field and returned to our laboratory for further examination and testing. Testing for index properties such as moisture content, organic content, grain size distribution and plasticity indices were carried out on selected samples and an oedometer test was carried out on one sample of the organic silt deposit. The as-drilled test hole locations and the ground surface elevations at the locations were measured by Golder Associates field staff. The elevations are referenced to Geodetic datum.

D
R
A
F
T

4.0 SUBSURFACE CONDITIONS

The detailed subsurface conditions encountered in each of the borings, together with the results of the laboratory tests carried out on selected samples, are given on the attached Record of Borehole sheets, Record of Cone Penetration Test sheets and on Figures 3 to 13. It should be noted that the stratigraphic boundaries shown on the Record of Borehole sheets and on the stratigraphic sections are inferred from non-continuous sampling and represent transitions between soil types rather than exact planes of geological change. Conditions will vary between and beyond the borehole locations.

As was previously noted, subsurface data reported in our preliminary geotechnical report has been incorporated into our analysis for this report and the applicable borings have been plotted on the stratigraphic sections that illustrate the subsurface conditions as encountered across the site. The records of those applicable borings are contained in Appendix A.

The portion of the site covered by this investigation follows an alignment that is south of Lakeshore Boulevard eastbound, from 75 m west of the Humber River to about 130 m east of the Humber River and between Lakeshore Boulevard east and the Gardiner Expressway from Windermere Avenue west to the jog in Lakeshore Boulevard (refer to Figure 2). The subsurface soil and groundwater conditions encountered in the boreholes put down along this portion of the site are summarized and illustrated by the stratigraphic sections presented on Figures 3 and 4. An inspection of these figures reveals two distinct subsurface regions at the site; an infilled valley and a "tableland" on either side of the infilled valley. The infilled valley extends from about 50 m west of the Humber River to about 110 m east of the Humber River; the "tableland" extends to the east and west site boundaries on either side of the infilled valley.

The ground surface across the site is relatively flat and is underlain by fill soils which are in turn underlain by sands and silty sand. The latter deposits are interpreted to be relatively recent natural lacustrine deposits. In the "table land" region, the upper sands are underlain by lacustrine clay to silt deposits which are in turn underlain by clay till deposits and/or residual soils which immediately overlie shale bedrock. Apart from the infilled valley, the bedrock surface has a relatively gentle overall downward dip from west to east. At the extreme west end of the site,

the bedrock surface is at about Elevation 70 m, while at the extreme east end of the site the top of bedrock elevation is about 61 m.

Within the infilled valley region, the fill deposit and upper sand deposit is underlain by an organic silt to organic clayey silt deposit up to 20 m thick on the east side of the Humber River. On the west side of the Humber River, the surficial deposits are underlain by 2 m to 3 m of organic silt which is in turn underlain by up to 16 m of sand to sand and gravel. This region is also underlain by shale bedrock which is immediately overlain by basal tills and/or residual soils.

The following sections provide a discussion of the in situ and laboratory tests carried on the principle soil types encountered during the investigation.

4.1 Fill

Fill soils, ranging in thickness from 1.1 m to 5.5 m were encountered at all the borehole locations. The fill encountered ranged in composition from sand and gravel to clayey silt and contained varying amounts of organic matter, concrete, brick and asphalt rubble. A slight hydrocarbon odour was noted in the samples of fill material retrieved from Borehole 112. The fill soil has a varying consistency and state of packing, with Standard Penetration Test 'N' values ranging from 3 to 40 blows / 0.3 m penetration.

4.2 Sands to Silts

Cohesionless sand to silt deposits occur across the site and range in composition from sand, trace silt to silt, some sand, trace clay. The sand to silt deposits immediately underlie fill soils at all borehole locations except at the location of Borehole 102 where, coincidentally, the fill is thickest. These upper sand to silt deposits are up to about 7 m thick. Cohesionless deposits with total thicknesses of 10 m to 12 m were also encountered at depth below organic deposits within the infilled valley on the west side of the Humber River. On the east side of the Humber River, cohesionless sand deposits were encountered at depth only at the location of Borehole 6, where it was about 6.5 m thick and was located below the organic silt deposit. Relatively thin layers of this type of soil were encountered sporadically as interlayers within the organic silt deposit and as interlayers within lacustrine clay deposits at the east end of the site.

The soil type has been identified as two separate units on the stratigraphic Sections; (i) sand, (ii) silt and fine sand to silty sand. There is no clear pattern to the distribution of these two sub-groupings which suggests that the particle size distribution varies spatially, likely having been controlled by local conditions at the time of deposition. The particle size distributions of samples of sand with some silt are shown on Figure 5A, those of sand with trace silt on Figure 5B; the particle size distribution of one sample of silt with some sand is shown on Figure 6.

The sand to silt soils are generally in a loose to compact state of packing and the average Standard Penetration Test 'N'-value measured is about 15 blows /0.3 m penetration. The measured natural moisture content of samples of these soils obtained during the current investigation ranged from 13 per cent to 34 per cent.

4.3 Organic Silt

The surficial sand to silt deposits are underlain by a deposit of organic silt to clayey silt within the infilled valley region of the site. The deposit is thickest on the east side of the Humber River where it is up to 19 m thick. The deposit tapers to a thickness of less than 2 m at the east and west ends of the infilled valley region and occurs as sporadic interlayers of thickness less than 1.5 m in the "tableland" region of the site.

The deposit is stratified, containing layers varying in composition from organic clayey silt to organic silt with occasional layers of organic silty sand. The particle size distribution of a sample of organic clayey silt is illustrated on Figure 7. Particle size distribution curves representative of the organic silt and organic silty sand layers are presented in Golder Associates preliminary geotechnical report. The organic content of the deposit measured during the current investigation ranged from 2 per cent to 10 per cent, while measurements made during the preliminary investigation showed a range of 7 per cent to 18 per cent. Unit weight measurements made on samples of the deposit range from 11.2 kN/m³ to 14.4 kN/m³. The water content of samples measured during this investigation ranged from 29 per cent to 66 per cent which is consistent with values measured during the preliminary investigation which were typically between 40 per cent and 60 per cent. Atterberg Limit tests carried out as part of both investigations have shown that the natural water content of the material is near its liquid limit. The plasticity indices

measured on samples during the current investigation were between 8 per cent and 12 per cent which is lower than the range of 20 per cent to 40 per cent measured on samples during the preliminary investigation. This observation suggests that the deposit is more variable than was indicated by testing during the preliminary geotechnical investigation.

As part of the preliminary geotechnical investigation, consolidation tests were carried out on seven samples of the organic deposit. The results of the testing indicated compression indices of 1.1 to 1.4 for the more plastic material and values of 0.6 to 0.9 for samples of lower plasticity. The preconsolidation pressure inferred for the deposit ranged from 130 kPa to 250 kPa, with the expected trend of increasing preconsolidation pressure with depth. Coefficients of secondary compression ranged from .001 to .007 for stress levels below the preconsolidation pressure and ranged from 0.16 to 0.26 for stress levels above the preconsolidation pressure. One consolidation test was carried out on a sample of the organic silt deposit during the current investigation. The results of this testing are given on Figure 13.

In situ field vane tests carried out in the organic deposit indicate undrained vane shear strengths generally greater than 96 kPa (the maximum measurable by the field test) with minimum values of about 60 kPa to 80 kPa. This is consistent with the values measured during the previous geotechnical investigation and as was discussed in the preliminary report, the dilatant nature and occasionally fibrous texture of the organic silt deposit could result in the undrained shear strength being overestimated. Indeed, the measured vane shear strength is higher than would be suggested by correlation with the pre-consolidation pressure of the deposit. Typically the vane shear strength is about 0.2 of the pre-consolidation pressure which, for this deposit, would imply an undrained shear strength ranging from 25 kPa to 50 kPa. A more rigorous laboratory testing program would be necessary to evaluate the shear strength of this deposit if embankments greater than 4 m in height were to be constructed at the site.

One sample (Borehole 104, Sample 11) of the organic silt was submitted for chemical analysis to assess the corrosion potential of piles permanently embedded in the deposit. The test results are listed below:

pH	7.4
Oxidation Reduction Potential	226 mV
Sulphide Content	<3 ppm

4.4 Sand & Gravel

Discontinuous layers of sand and gravel were encountered immediately below the organic silt in Boreholes 103 and 104. Where encountered, the sand and gravel deposit ranged from about 2 m to 5 m thick. Standard Penetration Test 'N' values were typically between 10 and 30 blows/0.3 m penetration, suggesting the deposit is generally in a compact state of packing. The particle size distributions of two samples of this material are shown on Figure 8.

4.5 Clayey Silt to Silty Clay

Within the infilled valley region of the site, a deposit of clayey silt to silty clay varying in thickness from about 3 m to 5 m was encountered below the organic silt deposit at the locations of Boreholes 3 and 104. A similar material was encountered sporadically in the east "tableland" region of the site between the upper sands and the basal tills. The thickness in this area ranged from about 1 m to 3 m. Standard Penetration Test 'N' values measured in the deposit ranged from less than 1 to 18 blows/0.3 m penetration. Measured vane shear strengths ranged from 70 kPa to 80 kPa and suggest that the low 'N' values were the result of the heavy rod weight when sampling at depth. The consistency of the deposit is, therefore, best described as stiff to very stiff. The water content of samples of the deposit ranged from 20 per cent to 25 per cent. The particle size distribution of a sample of the deposit is illustrated on Figure 9.

4.6 Interlayered Clayey Silt, Silt and Silty Clay

A deposit consisting of interlayered clayey silt, silt and silty clay was encountered in boreholes put down at the eastern half of the site, between the upper sands to silts and the basal tills. The deposit is stratified, having relatively thin layers that range in composition from silt to silty clay. The deposit is thickest near the extreme east end of the site where it is about 6 m thick at the location of Borehole 112. At other locations, layers from 1 m to 3 m thick were encountered. The particle size distribution for an homogenized sample of this deposit is illustrated in Figure 10. The Standard Penetration Test 'N' values ranged from 7 to 43 blows/0.3 m penetration, with most values between 10 and 25 blows/0.3 m penetration, indicating a generally stiff to very stiff consistency. The water content of samples of the soil ranged from 16 per cent to 21 per cent.

4.7 Clayey Silt to Silty Clay Till

A discontinuous clayey silt to silty clay till deposit immediately overlies bedrock across the site. The deposit was encountered in all boreholes except Boreholes 1, 103, 104 and 9. Where encountered the till deposit was between 1 m and 6 m thick. The particle size distribution of two samples of the deposit, both representative of the silty clay till, are illustrated on Figure 11. The Standard Penetration Test 'N' values measured in this deposit range from 6 to greater than 100 blows/0.3 m penetration. The consistency of the deposit is generally stiff to very stiff but there are zones that are of a firm consistency and others that are of a hard consistency. The water content measured on samples of this deposit ranged from 16 per cent to 21 per cent. Atterberg limit tests carried out on two samples of the deposit measured plasticity indices of 12 per cent to 14 per cent and indicated that the natural water content of the samples was close to the material's plastic limit.

4.8 Residual Soil

During the preliminary geotechnical investigation, deposits of clayey silt residual soil were encountered at the locations of Boreholes 6 and 9. These soils are believed to be the result of the complete in situ weathering of shale bedrock and as a result have a relatively low water content and a hard consistency. Where encountered, the residual soil was about 1 m thick, however, it must be stressed that the transition between a residual soil and the underlying bedrock is subtle and occurs gradually.

4.9 Shale Bedrock

The top of the bedrock surface has been inferred from auger refusal or from the retrieval of split spoon or rock core samples. From west to east across the site, in the "tableland" region, the bedrock surface has an overall downward gradient from an elevation of about 70 m at the west end of the site to an elevation of about 61 m at the east end of the site. Within the infilled valley region of the site, the bedrock surface dips down to an elevation of about 41 m at the location of Borehole 3. As boreholes were not carried out within the Humber River, the bedrock elevation beneath the river channel was not defined.

The bedrock at the site consists of a finely laminated shale containing interbeds of shaley and fossiliferous limestone. At the locations where core drilling was carried out, shale typically comprised greater than 75 per cent of the core run length. Limestone interbeds were typically 50 mm to 200 mm thick where encountered. However, at the locations of Boreholes 104 and 108, individual core runs were retrieved where shaley limestone comprised the majority of the retrieved core length.

Typically, the upper 0.5 m to 1 m of the bedrock was in a highly to completely weathered state, however, at the location of Borehole 7, this condition extended over a depth of about 2.5 m. Over the length of the core drilled sections, the bedrock was generally in a moderately weathered to fresh condition, with the amount of weathering generally decreasing with depth.

The intact strength of core sample specimens may be inferred from the results of diametral point load tests. Such tests were carried out on core samples obtained from the investigation and are plotted on the Record of Borehole sheets and tabulated in Table 1. For shale specimens, the diametral point load index ranged from 0.096 MPa to 0.87 MPa and averaged 0.34 MPa. For limestone specimens, the diametral point load index ranged from 0.96 MPa to 5.1 MPa and averaged 2.35 MPa.

The Rock Quality Designation (RQD), as measured on the core runs retrieved, was generally in the range of 20 per cent to 60 per cent. At the location of Borehole 103, however, the RQD was less than 10 per cent over the upper 3 m of the recovered core. Fractures within the rock were generally horizontal, reflecting the predominantly horizontal bedding within the rock mass. The fracture index, which is a measure of the number of fractures per 0.3 m of core length, was generally around 5 fractures per 0.3 m, although zones of highly fractured and broken core were retrieved as noted on the Borehole logs.

5.0 ENGINEERING RECOMMENDATIONS

This section of the report provides recommendations on the geotechnical aspects of the design of the works based on our interpretation of the factual information obtained during the investigation. It should be noted that the interpretation and recommendations are intended for use only by the design engineer. Where comments are made on construction they are provided only in order to highlight those aspects which could affect the design of the project. Construction techniques are uniquely related to experience and equipment available to the Contractor. Those requiring information on aspects of construction should make their own interpretation of the factual data, as it may affect equipment selection, proposed construction methods, work program and scheduling.

The recommendations made within this section are for the geotechnical aspects of the approach embankment and foundations for Bridge 1 and for the South Kingsway on-ramp onto the eastbound Gardiner Expressway (Figure 2). It is understood that Bridge 1 will be a 3-span structure, with a central span of 80 m over the Humber River and side-spans of about 64 m between the piers and the abutments. The South Kingsway ramp will rise from grade level where it emerges from under the Gardiner Expressway to the level of the Gardiner Expressway (a height of about 5 m) at a point about 100 m west of Windermere Avenue.

Results of calculations of time dependent settlements and lateral deformations of the subsoils under the proposal embankments are presented in the following sections. Actual settlements are affected by local variations in stratigraphy, soil fabric and preconsolidation pressure of the deposits. The results presented herein are our best estimate of the settlements likely to occur and are provided in order to optimize the design of the foundations. Settlement estimates should be confirmed by monitoring during construction.

5.1 Bridge 1 Approach Embankments

Based on the site and road grade profiles we have received, it appears that a 1.0 m to 2.0 m high approach embankment will be required at the west abutment of Bridge 1 and an approach embankment about 3 m high will be required at the east abutment of Bridge 1. These embankments may be constructed of an earth fill that is free of organic matter, refuse, rubble and

cobbles greater than 150 mm in size. The earth fill should be placed in loose lifts not exceeding 300 mm in thickness and compacted to 95 per cent of the material's Standard Proctor maximum dry density. The fill should be placed at a moisture content not exceeding 2 per cent of material's optimum moisture content, as established by the Standard Proctor density test. The compactive effort should extend to the edge of the embankment and to this end, the embankment should be overbuilt by about 0.3 m laterally and the sideslopes trimmed back to the required lines and grades at the completion of fill placement. For embankments constructed of fill meeting the above requirements, maximum embankment side slopes of 2 horizontal to 1 vertical may be used for design.

Prior to placing the engineered fill, the sub-grade should be prepared by stripping off the existing vegetation and topsoil (a depth of about 0.3 m) and proof-rolling the exposed sub-grade. The proof-rolling should be witnessed by a geotechnical engineer and any soft spots delineated during proof-rolling should be sub-excavated and backfilled with clean earth fill compacted to 95 per cent of the Standard Proctor maximum dry density.

5.1.1 Embankment Settlement

5.1.1.1 West Abutment of Bridge 1

Based on the subsurface conditions encountered near the west abutment of Bridge 1 and on our understanding of the embankment height, it is anticipated that the total settlement of the embankment will be about 50 mm. About half of the total settlement will occur as immediate settlement during construction of the embankment and the remainder of the total settlement will occur over time as the result of primary and secondary consolidation of the organic silt deposit.

5.1.1.2 East Abutment of Bridge 1

The borehole data suggests that about 18 m of organic silt exists beneath the proposed western edge of the east abutment. The cone penetration test data indicates that the thickness of the deposit decreases by about 1 m for every 3 m eastward from the edge of the abutment. As the preliminary geotechnical investigation emphasized, the organic deposit is very compressible. Given the compressible nature of the deposit, the fact that it thins eastward and that the

embankment height decreases eastward from the abutment, there is the potential for significant differential settlement of the east approach embankment of Bridge 1.

Settlement calculations have been carried out based on an embankment geometry as follows:

Height:	3 m
Crest width:	15 m
Side slopes:	2 horizontal to 1 vertical

The thickness of the compressible organic deposit has been taken to be 18 m. Based on these conditions, the total primary consolidation settlement calculated for the embankment is about 70 mm to 100 mm. Following completion of primary consolidation settlement another 75 mm to 100 mm of secondary consolidation settlement has been calculated to occur over a period of about 100 years; 50 mm of which is calculated to occur in the first 25 years. In light of the foregoing discussion, the total calculated settlement is equal to the differential settlement along the length of the embankment.

This amount of settlement will impact on the performance of embankment, its approach slabs and pavements and the abutment foundations. The embankment settlement may be reduced by lowering the embankment height or by moving the abutment eastward to a location where the compressible organic deposit is thinner. It is our understanding that both of these options are undesirable: the former because it will alter final road grades and result in a portion of the deck section being below grade, and the latter because of the increased bridge span that would be required.

5.1.1.2.1 Pre-load and Surcharge Embankments

Pre-loading of the organic deposit offers a method of reducing the magnitude of the settlement which will impact the embankment structure. In this case, the embankment would be built well ahead of piling for the bridge abutments and the construction of wing walls and the approach slab, allowing the organic silt to consolidate somewhat in advance of the structure's completion. The rate of consolidation of the organic silt deposit is difficult to predict because of the

importance of soil fabric, particularly the presence of organic silty sand seams, which tend to affect the rate of consolidation settlement. It is estimated that the time required to complete primary consolidation settlement could be between 0.5 years and 2.5 years. A reasonable proportion of the anticipated primary consolidation settlement could occur during pre-loading over a 0.5 year to 1 year period. In view of the uncertainty as to the time rate of primary consolidation settlement, it is recommended that any pre-loading be carried out in conjunction with a settlement and pore pressure monitoring program. Such a monitoring program would establish the rate of embankment consolidation over a relatively short time period allowing comparisons with, and refinement of, the settlement analyses.

Should the rate of consolidation settlement be found to be at the lower end of the calculated range it may be increased by installing closely spaced wick drains within the organic deposit. Such drains would be installed over the area of the embankment and their spacing would be dependent on the time available to complete the pre-loading and on the behaviour of the organic deposit as inferred from the monitoring program.

Wick drains would be installed through the embankment and porewater would be released from the wick drains into the upper sand layer. The pre-loading process, in combination with the possible use of wick drains can be used to induce virtually all primary consolidation settlement prior to abutment construction, leaving about 75 mm to 100 mm of post construction secondary consolidation settlement over the design life of the structure.

A further decrease in post construction settlement may be achieved by surcharging the embankment during the pre-loading phase. For example, if 1 m of additional fill is added to the embankment during pre-loading, we calculate that about 125 mm of primary consolidation settlement could occur, leaving about 50 mm to 75 mm of the calculated post-construction secondary compression to occur over a 100 year period. Surcharging of the embankment with more than 1 m of fill is not recommended, as the pre-consolidation pressure of the deposit could be exceeded. This would result in a slower rate of consolidation which would require wick drains at a much closer spacing in order to accelerate the consolidation process. Furthermore,

the magnitude secondary consolidation settlement would also increase as a result of exceeding the organic deposit's preconsolidation pressure.

5.1.1.2.2 Electro-Osmotic Treatment

Another means of consolidating the organic silt deposit ahead of the completion of construction is by the process of electro-osmosis. This technique for soil treatment utilizes a direct electric current to induce water flow toward electrodes which are inserted through the compressible layers^{1,2}. Preliminary laboratory testing has indicated that the method is feasible for the Humber River site and could be used to induce the estimated 100 year settlement prior to embankment construction. An important advantage of electro-osmotic treatment is that it may be applied and be effective in the area west of the proposed location of the east abutment of Bridge 1, thus reducing the eastern end span of Bridge 1. Further testing of the site soils would be necessary to assess the extent, duration and cost of electro-osmotic treatment for this site.

5.1.1.2.3 Light-Weight Fill

An alternative embankment design utilizing light-weight fill may also be considered for use at the site. It is possible to form the embankment core of materials such as rigid sheets of foamed polystyrene insulation which are weather resistant, strong and lightweight. An advantage of such an approach is that it would eliminate the requirement to pre-build the embankment in advance of piling. To reduce the quantity of such materials a vertical facing could be placed on the outside of the core, eliminating the need for 2 horizontal to 1 vertical side slopes. Such lightweight core materials must be protected from gasoline spills to prevent deterioration and therefore, a synthetic geomembrane is recommended to encapsulate to light-weight core materials. A more traditional source of lightweight fill is fly ash. This has to be carefully selected from the available sources to obtain the desirable properties and is subject to environmental testing as a non-hazardous waste.

-
1. Ho, K.S., 1990. Electrical strengthening of soft sensitive clays, Ph.D. Thesis, The University of Western Ontario, London, Ontario.
 2. Lo, K.Y., Ho, K.S. and Incullet, I.I., 1991. Field test of electro-osmosis strengthening of soft sensitive clay. Canadian Geotechnical Journal, 28:74-83.

By excavating existing soil from beneath the embankment area and replacing the removed soil with light-weight fill, the net additional load on the organic silt can be reduced to zero, thereby minimizing post-construction embankment settlement. Clearly, such a fully-compensated embankment will be most feasible if a polystyrene core is used for the embankment.

5.1.1.2.4 Pile Supported Slab

An at grade level reinforced concrete slab, founded on piles taken to bedrock, may be used to support the entire embankment thereby eliminating additional load on the compressible organic clayey silt. This alternative also has the advantage of eliminating the need to execute a separate contract in advance of the main contract for Bridge 1 construction.

Driven steel H-piles may be used to support the slab and these should be designed and constructed in accordance with the recommendations given in section 5.2.1. The embankment support slab should extend over the full length of the embankment and should extend laterally out under the embankment side-slope to the point at which the fill is less than 1.5 m thick. Alternatively, the edge of the slab may be used to support a retaining wall on either side of the approach embankment. The area under the roadway immediately east of the support slab should be back-filled with OPSS Granular "A", compacted to 100 per cent of the material's Standard Proctor maximum dry density. The zone of compacted granular "A" should extend from the base of the slab, upward at a maximum gradient of 4 horizontal to 1 vertical until the subgrade level of the approach pavement is reached.

Driving of piles for the embankment slab in the vicinity of existing services such as the Consumers gas pipeline could induce potentially damaging vibrations and settlements. It is recommended that pile driving commence at the furthest point from the gas main and that the ground vibration and settlement be monitored during driving of the first few piles. From this data it will be possible to determine if it will be necessary to expose the pipeline and/or provide support at the pipe during nearby pile driving activities.

5.2 Bridge Foundations

The variable subsurface conditions and the presence of a thick compressible organic deposit across much of the site make the use of shallow spread footings or raft foundations unsuitable for the bridge piers and abutments. It is considered that deep foundations which carry the bridge loads to the bedrock provide the only suitable foundation alternatives for the bridge structures at this site. While the load carrying capacity of the foundation alternatives is an important design consideration, the nature of the soils at this site and their effect on constructability, the presence and proximity of pre-existing structures and the potential for long-term scour of the river bed provide equally important design considerations for the selection of deep foundations.

Driven displacement piles such as steel pipe piles or pre-cast concrete piles would cause disturbance and displacement of the organic silt deposit during driving. Such displacement and disturbance could cause distress to adjacent completed structures (either newly constructed or presently existing) during the proposed staged construction program. These displacement piles are, however, axisymmetric and are capable of resisting relatively high lateral loads and internal moments. Low-displacement piles such as driven steel H-piles and non-displacement piles such as augered cast in situ concrete piles impose less disturbance on adjacent structures but, in the case of steel H-piles, have a weak axis which restricts horizontal load carrying capacity. In the case of cast in situ piles there are a number of difficulties associated with installing such piles at the site and in providing quality assurance during construction. Based on geotechnical considerations, driven steel H-piles are considered the most feasible foundation alternative for the project. Design recommendations and discussions on design related construction issues are provided for steel H-piles, cast in situ piles and pipe piles in the following sections.

5.2.1 Driven Steel H-Piles

The axial capacity of steel H-piles driven into the shale bedrock at the site is governed by the structural capacity of the steel member and the bearing capacity and deformation characteristics of shale founding stratum. The factored axial load capacity at Ultimate Limited States (ULS) can be assumed to be 1600 kN for a HP 310 x 110 steel H-pile driven to practical refusal with a suitable pile hammer. The load capacity at Serviceability Limit State (SLS) for the same pile is 1150 kN. Lateral loads imposed on the abutment may be partially resisted by battering the piles

and the lateral resistance supplied by the surrounding soil may be calculated in accordance with the recommendations provided in section 5.2.4.

The piles should be driven to an initial set equal to or greater than 10 blows per 12 mm of penetration (unless abrupt peaking occurs) using a hammer with rated energy of about 50 kilojoules but not exceeding 60 kilojoules. On reaching the required set, the hammer energy should be reduced by about 75 per cent and the pile should then be redriven by increasing the hammer energy slowly up to the maximum rated energy over about 40 blows. A final set of no less than 10 blows per 12 mm of penetration should be obtained at the maximum hammer energy. This setting process is intended to set the pile point into the bedrock and to avoid excessive damage to the pile tip and section. Provision should be made to retap all piles to confirm the set after adjacent piles have been driven. Piles that do not meet the set criteria on the first restrike would require additional striking. The above set criteria should be reviewed at the time of construction in light of the contractor's proposed equipment, so that over-driving and possible damage to the piles is avoided.

The presence of construction rubble such as bricks, asphalt and concrete pieces was noted within the fill materials encountered during the geotechnical investigation. While not encountered during the investigation, it is possible that large concrete slabs may be present within the fill materials and these could provide obstructions to pile driving. Additionally, buried river bank retaining structures may be present in the vicinity of the Humber River. For these reasons, and to provide adequate seating within the bedrock, it is recommended that the ends of the piles be fitted with appropriate driving shoes (Oslo or Titus points).

5.2.2 Cast in Situ Concrete Piles

Cast in situ concrete piles socketted into the bedrock at the site are an alternative means of supporting the bridge structure. The piles would be formed within augered holes advanced through the overburden and extended into the bedrock. The load transfer between the concrete pile and the rock may be designed based on sidewall adhesion to the bedrock socket wall or on end bearing on the base of the bedrock socket. Because of the different stress/strain relationships for the two load carrying mechanisms, and the brittle nature of the rock/concrete interface, the

ultimate load carrying capacity of the socket is not the sum of the capacity in sidewall shear and end-bearing. Design methods are available to account for the interaction of sidewall and end-bearing resistance; however, due to the difficulty in preparing the base of the socket and the different stress/strain relationships in shear and end-bearing, the design has been based on side wall adhesion only.

For a vertical sided socket, with a rough sidewall, the socket may be designed based on a sidewall adhesion of 800 kPa at Ultimate Limit States. Because of weathering near the top of the bedrock surface, the upper 1 m of the socket length should not be included as part of the surface area of the socket. The axial capacity of a 0.9 m diameter socket is shown below. As an example, a pile drilled 4 m below the bedrock surface would have a factored capacity of 6800 kN at Ultimate Limit States. The capacity of the pile at Serviceability Limit States will be controlled by the structural characteristics of the pile.

**LOAD CARRYING CAPACITIES OF A
0.9 m DIAMETER ROCK SOCKET**

<u>Depth of Socket below top of Bedrock (m)</u>	<u>Capacity at Ultimate Limit States (kN)</u>
2 m	2250
3 m	4500
4 m	6800

As indicated, the above design value assumes that the sidewall of the socket will be rough. This requirement precludes the use of coring equipment to form the socket unless the sidewalls are mechanically roughened after coring is completed. Additionally, the above design value assumes that the augered hole is not advanced using bentonite based drilling fluids. Such fluids could form a filter cake on the side of the socket, reducing the adhesion between concrete and rock.

The sockets of augered piles in rock are normally hand-cleaned and inspected prior to concrete placement. For the piers and the east abutment of Bridge 1, however, it is not considered feasible to dewater the caisson excavations because of the high piezometric pressure within the fractured bedrock. The sidewalls of the socket should be cleaned using a water jet lowered

down the open hole and rotated within the socket. Debris should then be removed from the bottom of the socket.

Concrete must be placed in cast in situ piles using tremie techniques. That is, the concrete must be discharged at the base of the pier, and flow upward to the ground surface. Typically, the temporary casing used to maintain the hole as it is drilled, is withdrawn as the concrete is displaced upward. The thick organic deposits on the east side of the Humber River may tend to squeeze inward during casing withdrawal and concrete may flow into areas of overbreak caused during augering. These phenomena cause "necking" of the piles, while such a condition may be checked after construction by dynamic monitoring, the condition cannot realistically be corrected without drilling additional piles. In view of the space constraints on this particular project, it is recommended that the design of cast in situ piles incorporate a permanent steel liner to prevent "necking" of the piles.

Two options exist for the installation and provision of the permanent steel liner for piles at the site. A single liner system could be used, in which case a heavy duty steel liner, capable of being driven, vibrated or oscillated through the overburden would be advanced to and keyed into the bedrock surface, where it would remain. Alternatively, a heavy duty liner as described above could be used to advance the hole, and a thinner inner liner would be inserted after the hole was drilled to act as the permanent liner. The outer temporary casing would be withdrawn for re-use at the other pile installations. While the double liner system would incur lower material costs, it presents practical difficulties for construction, one of which is supporting the inner liner above the bedrock socket during withdrawal of the outer temporary liner.

In view of the construction difficulties and uncertainties associated with cast in situ piles, if these piles are adopted, it is recommended that at least one test pile be installed prior to the start of construction of the pier and abutment foundations. The test pile would be load tested to confirm the design capacity of the pier and assess the contractor's installation technique. Because installation technique could have such an impact of the performance of the concrete pile foundation, it is recommended that a geotechnical engineer be on site full-time during the installation of cast in situ concrete piles.

5.2.3 Driven Pipe Piles

The axial capacity of driven pipe piles, like that of steel H-piles, is governed by both the structural capacity of the steel member and the bearing capacity and deformation characteristics of the shale founding stratum. The factored axial load capacity at ULS can be assumed to be 1900 kN for a 455 mm diameter pipe pile of 12.7 mm wall thickness, formed of grade 240W steel and driven to practical refusal with a suitable hammer. The load capacity at SLS for the same pile is 1400 kN. Lateral loads on the pipe piles may be partially resisted by battering the piles and the resistance provided by the surrounding soil may be calculated based on the recommendations given in section 5.2.4.

It is recommended that the pipe piles be driven open-ended. This will somewhat decrease the disturbance caused to adjacent structures and piles as a result of soil displacement during driving. By driving the piles open-ended it will also be possible to clear obstructions using a churn drill or chopping bit. In this regard, the presence of construction rubble such as bricks, asphalt and concrete pieces, as noted in the fill soils on the borehole logs, suggests that the tip of the pipe piles should be reinforced using an internal steel ring. An external steel reinforcing ring should not be used if lateral resistance of surrounding soil is to be relied on in the design.

The pipe piles should be driven to the same set criteria as outlined in section 5.2.1, with the exception that the hammer should have a rated energy of about 65 kilojoules and should not exceed 80 kilojoules. Upon achieving the required set, the pipe piles may be washed clean and backfilled with concrete, placed using a tremie pipe extended to the base of the pile.

5.2.4 Resistance to Lateral Loads

For resistance to lateral loads, the horizontal reaction to the piles or caissons can be calculated from the expression:

$$k_h = n_h z/b$$

where

k_h is the horizontal reaction pressure per unit deflection (MPa/m)

n_h is the constant of horizontal subgrade reaction (MPa/m)

z is the depth below ground surface (m)

b is the caisson or pile diameter (m)

The following design values may be assumed for the major soil types at this site.

Fill: n_h times $z = \text{constant} = 4 \text{ MPa}$
Sand to Silt: $n_h = 2.5 \text{ MPa/m}$
Organic Silt: $n_h = .2 \text{ MPa/m}$

Group action for lateral loading should be considered when the pile or caisson spacing in the direction of loading is less than 8 pile diameters. This can be evaluated by a reduction of the coefficient of subgrade reaction (k_h) with respect to the pile spacing as follows:

<u>Spacing</u>	<u>Reduction Factor</u>
8 B	1.0
6 B	0.7
4 B	0.4
3 B	0.25

It is understood that the pile foundations of the bridge piers will be designed to act as free standing columns over the maximum potential scour depth at the site. In the case of steel H-piles, the moment capacity of the pile over this upper zone may have to be increased. One method of increasing the rigidity of the pile over this zone would be to encase the required length of the H-section within a concrete filled pipe. In this scheme, the pipe section would be driven to the required depth, the soil within the pipe would be washed out and the H-pile would then be driven through the pipe to refusal on bedrock, as discussed in section 5.2.1. Concrete or grout would then be tremied into place within the pipe pile length. The largest commonly available pipe section, with an outside diameter of 457.2 mm and a standard wall thickness of 9.5 mm, would only accommodate an HP 310x110 H-pile without allowance for driving points or to permit concrete to flow between the two steel sections. This suggests that a larger diameter, non-standard pipe or hollow steel section will have to be used if this form of structural reinforcement is to be used.

It is understood that consideration is also being given to increasing the lateral resistance over the scour zone by construction of temporary cofferdams to permit excavation to the scour depth within the footing area and backfilling the cofferdams with concrete. This method is feasible; however, it is probable that the concrete would have to be tremied in place at the west pier location where water bearing sand deposits are present. On the east side of the river, it may be possible to form a cut-off to groundwater flow by extending the cofferdam into the organic deposit. Recommendations for cofferdam design will be provided once the design depth is selected. It should be noted that if concrete is placed within the cofferdam, the side adjacent to the river will have to be designed to withstand the net outward pressure imposed prior to concrete set.

5.2.5 Vibrations During Construction

The installation of deep foundations at this site will induce ground vibrations. The proximity of other functioning bridge structures to the construction area increases the risk that the vibrations induced during construction could have damaging impact on the existing structures. The tolerance of a structure to ground vibrations is often related to the peak particle velocity of the ground motion.

For the driven piles or driven casings for cast in situ piers, the upper bound peak particle velocity at a distance from the source can be calculated from the following formula:

$$V = \frac{1.5 E^{1/2}}{d}$$

Where

V is peak particle velocity (mm/s)

E is hammer energy (joules)

d is distance from source (m)

This empirical relation is derived from cases where diesel hammers or drop hammers have been used. Higher efficiency hydraulic pile driving hammers impact a greater proportion of their energy to the pile and as a result, the above formula may underestimate the upper bound peak particle velocity.

The use of vibratory hammers to install or remove liners for cast in situ piles have the potential to induce greater damage to structures than impact hammers. Because the vibratory hammer impacts a sustained vibration, it is possible for harmonic resonance to be induced in structures having a natural frequency close to that of the hammer. Oscillators, which impart a low frequency rotation to liners probably induce the least damaging vibration during installation.

Damage can also be caused by vibration induced settlement of loose sands and sensitive clay. The construction of the foundations of the west pier of Bridge 1 will be a minimum of about 15 m from the west abutment of the existing eastbound Lakeshore Blvd bridge which is founded on piles terminated within the sand deposits on the west side of the Humber River. It is estimated that vibrations from pile driving or from the use of vibratory hammers for casing installation, could induce up to 25 mm of abutment settlement due to compression of the sands below the piles. Because of the plastic and non-brittle nature of the organic silt deposit, vibration induced settlement of this deposit is not anticipated.

In view of the fact that subsequent phases of construction may involve piling or caisson installation in closer proximity to existing structures, it is recommended that settlement monitoring be carried out during piling or caisson installation to establish a site specific relation between induced settlement and distance from foundation installation works.

5.2.6 Corrosion of Subgrade Steel

Below a depth of about 1 m, corrosion of subgrade steel within inorganic natural soils, in freshwater environments, does not normally occur to any significant extent. Within organic soils, anaerobic, sulphate reducing bacteria can corrode steel if present and if living in a suitable environment. Chemical tests carried out on one sample of the organic silt deposit indicate that the organic silt at the site is an unsuitable environment for anaerobic corrosion. The pH is near neutral, sulphides which are the product of anaerobic bacteria are at less than 3 ppm and the oxidation - reduction potential of the soil is above 100 mV. Similar test results were obtained on a sample of the organic soil obtained during investigations for other bridges on the project.

Another possible cause of subgrade steel corrosion is stray current. It is our understanding that sources of stray current such as electric transit ways are not present at the site or planned as part of the Humber Bridges project.

5.2.7 Frost Protection

To prevent adfreezing to piles and uplift forces on the underside of the pile caps, the base of the pile caps should be maintained at least 1.2 m below the final site grade. For winter construction conditions, temporary insulation such as straw or earth fill should be placed around any exposed pile caps to provide frost protection. These same requirements apply to the horizontal distance that piles should be maintained behind erosion protection works at pier locations. If the piles cannot be maintained at least 1.2 m from the edge of permanent sheet-pile walls, then rigid polystyrene foam insulation may be placed on the inside of the sheet-piling to provide the necessary frost protection. Recommendations for the required insulation thickness will be provided if necessary.

5.2.8 Design Considerations for the East Abutment of Bridge 1

As was discussed in Section 5.1.1, the approach embankments for the east abutment of Bridge 1 will induce primary and secondary consolidation of the underlying organic silt deposit, unless the embankment itself is structurally supported or has a compensated foundation using light-weight fill. That portion of consolidation settlement which occurs after the installation of the deep foundations of the east abutment will induce additional loads from the effect of negative skin friction and will induce lateral loads as a result of lateral "squeezing" of the deposit.

5.2.8.1 Negative Skin Friction

Negative skin friction is a force induced on a pile as a result of the relative downward movement of a consolidating soil layer surrounding the pile. The magnitude of the force on an individual pile is dependent on the relative movement between the pile and the soil layer at each point along the pile. The unit friction load tends to increase with increasing relative motion up to a limiting large strain or "full-slip" condition. In the case of pile groups, the total negative skin friction may be less than the sum of the negative skin friction loads for individual piles due to the "shielding" effect that the outer piles of the group have on the inner piles.

At this point in the design process, there are not enough known parameters to evaluate the interaction between the foundation structure and the soil. Among the parameters required are the loads, the foundation stiffness and the anticipated amount of post construction foundation settlement which is dependent on the measures taken to mitigate embankment settlement. The upper bound value of the negative skin friction acting on piles at the location of the east abutment of Bridge 1 may be calculated based on an average unit negative skin friction value of 37 kPa. For HP 310 x 110 piles this represents a load of 800 kN; a load of about 1900 kN is obtained for a 0.9 m diameter concrete pier.

If a pre-load embankment or a surcharge embankment were utilized to reduce consolidation settlement, the magnitude of negative skin friction will be less than the upper bound value providing that the post-construction settlement is less than that required to induce the "full-slip" condition adjacent to the piles. If a pile supported slab or a light-weight fill embankment were constructed or if electro-osmotic treatment of the organic clayey silts was carried out, there will be no post-construction compression of the organic clayey silt and, therefore, negative skin friction loads will not be imposed on the piles.

*Only if there is no additional
surcharge on the original grade*

In evaluating the load carrying capacity of the structural elements of the foundations, two separate cases should be considered; dead load plus negative skin friction load and dead load plus live load, with the highest sum governing the design. Because live loads impose a transitory downward deformation to the pile, the soil actually moves upward relative to the pile thus, acting to help support the transitory load and negating the negative skin friction effect.

The magnitude of the negative skin friction load on driven steel H-piles may be reduced by providing the piles with a bitumen coating prior to driving. Similarly, if a smooth steel permanent liner is used in the construction of cast in place concrete piers, a bitumen coating on the outside of the liner will reduce the magnitude of the negative skin friction load. For design purposes, it may be assumed that the negative skin friction load will be reduced by one half if an appropriate bitumen coating is applied.

5.2.8.2 Lateral Deformation

Lateral deformation of piles or piers will be induced by the lateral deformation (or "squeezing") of the consolidating organic silt layer beneath the embankment of the east abutment. Calculation of the magnitude of this deformation presents a relatively complex soil-structure interaction problem requiring the stiffness of the individual foundation members, the group configuration and the vertical soil settlement after construction as inputs to the analysis.

Based on the information available, the upper bound horizontal displacement of HP 310 x 110 steel piles will be about 0.5 times the post construction settlement as given in Section 5.1.1. For 0.9 m diameter concrete piers, the upper bound lateral deformation of the pier will be about 0.3 times the post construction settlement. Further refinement of the estimate of lateral deformation of the piers or piles is possible as the final design proceeds.

Once again, if a pile supported slab, lightweight fill or electro-osmotic treatment of the organic clayey silt is used at the site, there will be no post-construction compression of the organic clayey silt and lateral pile deformation as a result of "squeezing" will be avoided.

5.3 South Kingsway Ramp

The entrance ramp from the South Kingsway onto the eastbound Gardiner Expressway will be about 250 m long. The alignment is between the eastbound Gardiner Expressway and the westbound Lakeshore Boulevard and is on land currently occupied by a service station and a hotel. The ramp will rise from ground level at its west end, up to the level of the Gardiner Expressway at Windermere Avenue; a height of about 5 m.

A number of options exist for design and construction of the ramp: a conventional earth embankment, a geo-grid reinforced earth embankment, a reinforced earth wall and a cantilevered structural retaining wall. It is understood that the selection of the ramp alternative will be based on the net cost to the project which will incorporate the construction cost of the structure and the residual value of the property remaining after ramp construction.

5.3.1 Backfilling of Excavations prior to Ramp Construction

As previously discussed, the proposed ramp alignment passes over the locations of an existing service station and a hotel. It is understood that these structures will be demolished prior to ramp construction. The demolition of the hotel will expose the basement and decommissioning of the service station will likely involve the excavation and removal of underground storage tanks. The nature of the backfill for these excavations will impact the design of the South Kingsway on-ramp.

In the case of the conventional earth embankment, the geo-grid reinforced embankment and the reinforced earthwall options, the structures will be founded essentially at ground surface, on the existing fill soils. Each of these options are settlement tolerant, however, the fill placed within excavations which will underlie the structures should be compatible with the fill presently existing at the site, to avoid severe differential settlement over short distances or across sharp boundaries. It is, therefore, recommended that excavations underlying the proposed ramp location be backfilled with a clean soil fill, placed in 0.5 m thick lifts and compacted to 95 per cent of material's Standard Proctor maximum dry density. The use of rubble fill within the hotel basement is not recommended as this will create a locally "stiff" zone, across which potentially damaging differential settlement could occur.

As will be discussed, if structural cantilevered retaining walls are used at the site of the ramp, they will have deep foundations and will not be impacted by the nature of backfill to excavations.

5.3.2 Earth Embankment

The South Kingsway on-ramp may be constructed on an earth embankment having sideslopes of 2 horizontal to 1 vertical providing the recommendations for the Bridge 1 Approach Embankments (Section 5.1) are adhered to.

Prior to placing fill for the embankment, the existing asphalt at the site should be broken up and removed from the area underlying the embankment. Additionally, any topsoil which is present under the area of the proposed embankment should be removed. The exposed subgrade should

then be proof-rolled in the presence of a geotechnical engineer and any soft spots delineated during proof-rolling should be subexcavated and backfilled with dry compacted soil.

5.3.3 Geo-grid Reinforced Earth Embankment

The appropriate use of geo-grid reinforcement within an earth embankment will permit the construction of embankments with sideslopes having an inclination of the order of 1 horizontal to 1 vertical. The strength, spacing and width of geo-grid reinforcement is dependent on the height and inclination of the embankment and the type of soil used in the construction of the embankment. Soil types which are suitable for the construction of a conventional unreinforced embankment are generally suitable for use in a geo-grid reinforced embankment.

Construction of the geo-grid reinforced embankment should be carried out on a sub-grade prepared in the manner discussed in Section 5.3.2. The magnitude of settlement of the embankment could approach 100 mm to 130 mm near Windermere Avenue where the embankment is highest. Most of this settlement will occur during construction and, providing that the excavation backfilling measures discussed in Section 5.3.1 are adhered to, differential settlement should occur gradually over the length of the embankment.

5.3.4 Reinforced Earth Wall

A still greater amount of land will be saved by the use of reinforced earth walls for the South Kingsway Ramp. These walls utilize galvanized steel strips to reinforce a cohesionless granular fill soil. Precast concrete panels are used as an exterior facing. The panels interlock but are not rigidly bonded and are therefore, relatively tolerant to settlement. It is understood that the internal design of such a wall would be carried out by others. At the east end of the ramp, where the wall will be constructed immediately adjacent to the existing structural wall of the eastbound Gardiner Expressway, the reinforced earth wall should be designed to resist the following lateral earth pressure loads which are expressed as equivalent fluid pressures:

At ultimate limit states: 8.0 kPa/m
At serviceability limit states: 6.5 kPa/m.

For the proposed maximum height of 5 m, and an assumed width of 8 m, fill soils and the underlying sand deposit at the site would provide adequate bearing capacity for the structure. The soils are, however, relatively compressible and 100 mm to 130 mm of total settlement may be anticipated at the east end of the ramp, with most of this settlement occurring during construction. The recommendations of Section 5.3.1 regarding excavation backfill should be followed to minimize differential settlement and the subgrade should be prepared in the manner discussed in Section 5.3.2. Additionally, at locations where the panel footing extends over existing basement walls, the wall should be broken out down to basement level and be replaced by backfill as specified in Section 5.3.1. The fill soils at the site are variable, and as such, have varying degrees of frost susceptibility. It is recommended that the footing for the facing panels be founded at a minimum depth of 1.2 m below final grade or, if a shallower founding depth is desired, that insulation be provided to the footings. Recommendations on the thickness and extent of insulation can be provided, if necessary.

The designers of the reinforced earth wall should review these recommendations and the estimated magnitude of total settlement and determine for themselves if the structure will perform adequately under the prevailing site conditions. Settlements may be reduced by excavating and removing the existing fill soil and replacing it with well compacted cohesionless soil. For example, calculations indicate that if the upper 1 m of fill soil is removed and replaced with compacted granular soil, the total settlement will be reduced to about 60 mm to 80 mm. For this reduction the imported soil should conform to OPSS granular "B" specifications, and should be placed in 300 mm thick lifts and compacted to 100 per cent of the material's Standard Proctor maximum dry density.

5.3.5 Structural Cantilevered Retaining Wall

A reinforced concrete cantilever retaining wall may also be used to form the South Kingsway on-ramp. The walls should be designed to resist the following earth pressure loads, which are expressed as equivalent fluid pressures:

At ultimate limit states: 8.0 kPa/m

At serviceability limit states: 6.5 kPa/m

The above earth pressures are based on a properly compacted free-draining backfill behind the wall and "active" earth pressure conditions. The free-draining backfill behind the wall should be placed in the zone from the back of the wall to a line projected from the heel of the wall upward at a 1 horizontal to 1 vertical gradient. In order for "active" earth pressure conditions to be mobilized the wall must permit a lateral deformation of the top of the wall of 0.5 per cent of the retained height of soil. Backfill to the wall should contain not more than 5 per cent by weight of material passing the 75 μ m sieve. The backfill should be placed in loose lifts not exceeding 200 mm in thickness and be compacted to 95 per cent of the material's Standard Proctor maximum dry density, using relatively light weight compaction equipment. Longitudinal drains should be located immediately below the base of the wall, providing positive drainage of the backfill.

The fill and upper sand soils at the site will not provide suitable founding conditions for a cantilevered structural retaining wall. Deep foundations, such as driven steel H piles or cast in situ concrete piers, bearing on or within the bedrock underlying the site are recommended for the support of cantilevered retaining walls at the site. The load carrying capacities and design recommendations for deep foundations as given in Section 5.2.1 and Section 5.2.2 should be followed for the South Kingsway on-ramp.

5.3.6 Influence on Existing Structure

It is understood that the existing retaining wall along the south side of the Gardiner Expressway, west of Windermere Avenue, is founded on deep foundations which extend to bedrock. The placement of fill adjacent to this structure during construction of the South Kingsway on ramp will induce additional loads on the southern most row of piles as a result of compression of the subsoils. Therefore, some distortion of the wall should be anticipated during construction. An earth or geo-grid reinforced embankment and a structural retaining wall or reinforced earth

retaining wall designed to support the previously given earth pressures, will support any of the load imposed as a result of distortion of the existing wall.

GOLDER ASSOCIATES LTD.

J. Westland, P.Eng.

A.S. Poschmann, P.Eng.

JW/ASP/dh/pds

D
R
A
F
T

TABLE 1
RESULTS OF DIAMETRAL POINT LOAD TESTS

BOREHOLE NUMBER	ELEVATION (m)	Is(50) (MPa)	ROCK TYPE	BOREHOLE NUMBER	ELEVATION (m)	Is(50) (MPa)	ROCK TYPE
102	69.25	3.049	S.L.	105	45.66	0.519	S
	69.00	0.193	S		45.53	0.388	S
	68.26	0.487	S		45.36	0.483	S
	67.90	0.293	S		45.15	0.387	S
	67.65	2.489	S.L.		44.89	2.425	S.L.
	67.60	2.393	S.L.		44.85	2.705	S.L.
	67.45	0.869	S		44.54	0.289	S
	67.19	1.922	S.L.		44.14	1.342	S.L.
	67.11	3.076	F.L.		44.06	1.246	S.L.
	66.94	1.612	S.L.		43.86	0.386	S
	66.63	0.194	S		43.45	0.386	S
	66.08	0.193	S		42.97	0.288	S
	65.47	0.579	S		42.79	0.193	S.L.
	65.31	0.577	S		42.64	2.726	S.L.
103	51.02	3.410	F.L.	108	64.04	2.726	F.L.
	49.26	3.413	S.L.		63.89	0.581	S
	47.74	2.594	S.L.		63.79	0.493	S
	47.03	0.170	S.L.		63.59	0.295	S
	46.85	0.345	S.L.		63.33	1.055	F.L.
					63.21	0.483	S
104	44.56	5.130	S.L.		62.98	1.817	S.L.
	44.30	0.099	S		62.88	1.339	F.L.
	43.51	0.098	S		62.34	3.845	S.L.
	42.88	0.964	S.L.		62.22	0.117	S
	42.65	1.534	S.L.		61.96	0.116	S
	42.57	2.118	S.L.		61.71	0.096	S
	42.09	0.675	S		61.58	1.153	S.L.
	40.95	0.201	S		61.13	2.889	S.L.
	40.59	0.233	S		60.84	0.103	S
LEGEND				S.L. = Shaley Limestone			
				F.L. = Fossiliferous Limestone			
				S = Shale			

*Is(50): Diametral Point Load Index (Corrected)

Rock Type	Average Is50(MPa)	Maximum Is50(MPa)	Minimum Is50(MPa)
Limestone (S.L. & F.L.)	2.35	5.13	0.964
Shale	0.34	0.87	0.096

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

<i>Relative Density</i>	' <i>N</i> ' 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

<i>Consistency</i>	kPa	' <i>Cu</i> ' 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 hydrometer¹
Q undrained triaxial²
R consolidated undrained triaxial²
S drained triaxial
U unconfined compression
V field vane test

NOTES:

¹Combined analyses when 5 to 95 per cent of the material passes the No. 200 sieve.

²Undrained triaxial tests in which pore pressures are measured are shown as \bar{Q} or \bar{R} .

LIST OF SYMBOLS

I. GENERAL

π	$= 3.1416$
e	$=$ base of natural logarithms 2.7183
$\log_e a$ or $\ln a$	natural logarithm of a
$\log_{10} a$ or $\log a$	logarithm of a to base 10
t	time
g	acceleration due to gravity
V	volume
W	weight
M	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_c	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
ϕ'	effective angle of shearing resistance, or friction
c_u	apparent cohesion*
ϕ_u	apparent angle of shearing resistance, or friction
μ	coefficient of friction
S_f	sensitivity

$\left. \begin{matrix} c' \\ \phi' \end{matrix} \right\} \begin{matrix} \text{in terms of effective} \\ \text{stress} \end{matrix}$
 $\tau_f = c' + \sigma' \tan \phi'$

$\left. \begin{matrix} c_u \\ \phi_u \end{matrix} \right\} \begin{matrix} \text{in terms of total stress} \end{matrix}$
 $\tau_f = c_u + \sigma \tan \phi_u$

*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.

strength $\tau_f = c_u$ is taken

PROJECT: 911-1315

RECORD OF BOREHOLE 102

SHEET 1 OF 2

LOCATION: SEE FIGURE 2

BORING DATE: MAY 6, 1991

DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm



DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE		SAMPLES		DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m		HYDRAULIC CONDUCTIVITY, k, cm/s		ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION	
		DESCRIPTION	STRATA PLOT ELEV. DEPTH (m)	NUMBER	TYPE	BLOWS/0.3m	SHEAR STRENGTH		WATER CONTENT, PERCENT			
							Cu, kPa	rel. V - + rem. V - @ U - O	Wp			W
0	POWER AUGER BORING 203mm O.D. HOLLOW STEM AUGER	GROUND SURFACE	79.13									
		TOPSOIL	0.01									
2		Stiff to very stiff, brown clayey silt, some sand and gravel, fragments of bricks, asphalt, trace organics. (FILL)		1	50 DO	14						
				2	50 DO	24						
				3	50 DO	16						
			76.23 2.90	4	50 DO	7						
4		Loose to dense, greyish brown silty sand, trace gravel, occ. pockets of clayey silt, trace wood, occ. asphalt and ceramic fragments. (FILL)		5	50 DO	13						
				6	50 DO	35						
			73.63 5.50	7	50 DO	8						
6		Firm, dark grey ORGANIC SILT, trace to some sand, occ. clayey silt seams, occ. wood and shell fragments.		8	50 DO	28						
			72.13 7.00	9	50 DO	100/15						
8		Very stiff, brown and grey mottled SILTY CLAY, trace sand and gravel, occ. oxidized fissures. (TILL)										
			69.99 9.14									
			69.38 9.75									
10		Weathered, grey SHALE. (BEDROCK)										
12												
14												
16												
18												
20												
		BOREHOLE CONTINUED. FOR BEDROCK CORING DETAILS, SEE SHEET 2.										

15 5 PERCENT AXIAL STRAIN AT FAILURE

DEPTH SCALE

1 to 100

Golder Associates

LOGGED: AP

CHECKED: ASP

DATA INPUT: SK JUNE 25/91

PROJECT: 911-1315

RECORD OF BOREHOLE 102

SHEET 2 OF 2



LOCATION: SEE FIGURE 2

DRILLING DATE: MAY 6, 1991

DATUM: GEODETIC

INCLINATION: -90 AZIMUTH:

DRILL RIG: ACKER

DRILLING CONTRACTOR: K & S DRILLING

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (m/min)	FLUSH % RETURN	FR-FRACTURE CL-CLEAVAGE SH-SHEAR VN-VEIN	F-FAULT J-JOINT P-POLISHED S-SLICKENSIDED	SM-SMOOTH R-ROUGH ST-STEPPED PL-PLANAR	FL-FLEXURED UE-UNEVEN W-WAVY C-CURVED	HYDRAULIC CONDUCTIVITY k, cm/sec	WATER LEVEL INSTRUMENTATION
0													
2													
4		CONTINUED FROM SHEET 1											
6													
8													
10				69.38									
12				9.75									
14													
16													
18													
20													

Slightly weathered to fresh, grey SHALE (75%) interbedded with fresh, light grey SHALEY LIMESTONE (18%) and FOSSILIFEROUS LIMESTONE (7%).

END OF BOREHOLE

Limestone and siltstone interbeds generally less than 50mm except at the following Elevations:

69.21m : 50mm sl
68.95m : 50mm sl
68.12m : 63mm sl
67.74m : 100mm sl
67.48m : 140mm sl
67.23m : 100mm fl
67.08m : 190mm sl + fl
66.61m : 90mm sl
66.04m : 63mm sl

NOTE:
sl - shaley limestone
fl - fossiliferous limestone

Highly weathered
(13mm)

Oxidized fissures
at Elev. 67.2m

BENTONITE
SEAL

PEA
GRAVEL

WATER LEVEL IN
OPEN BOREHOLE
AT ELEV. 73.5m
DURING DRILLING.

WATER LEVEL IN
PIEZOMETER AT
ELEV. 73.5m
ON MAY 31/91.

DEPTH SCALE:

1 to 100

Golder Associates

LOGGED: AP

DATE: MAY 6, 1991

CHECKED: ASP

DATA INPUT: SK JUNE 25/91

PROJECT: 911-1315

RECORD OF BOREHOLE 103

SHEET 1 OF 3

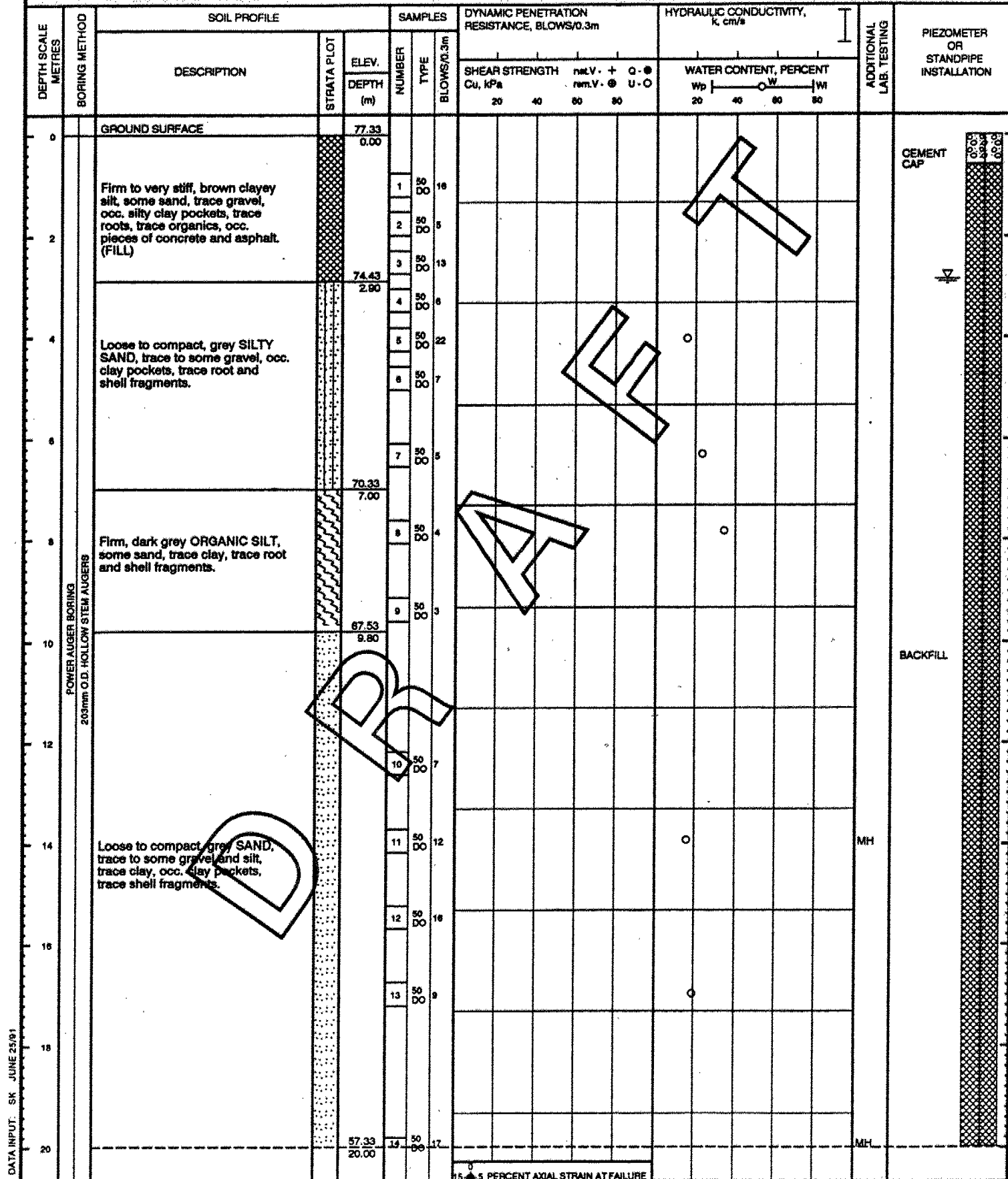
LOCATION: SEE FIGURE 2

BORING DATE: MAY 7, 1991

DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm



DEPTH SCALE

1 to 100

Golder Associates

LOGGED: AP

CHECKED: ASP

PROJECT: 911-1315

RECORD OF BOREHOLE 103

SHEET 2 OF 3



LOCATION: SEE FIGURE 2

BORING DATE: MAY 7, 1991

DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE		SAMPLES		DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m	HYDRAULIC CONDUCTIVITY, k, cm/s	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION	
		DESCRIPTION	STRATA PLOT ELEV. DEPTH (m)	NUMBER	TYPE					BLOWS/0.3m
20	POWER AUGER BORING 203mm O.D. HOLLOW STEM AUGERS	Loose to compact, grey SAND, trace to some gravel and silt, trace silt, occ. clay pockets, trace shell fragments.	57.33 20.00	14	50 DO	17			MH	BACKFILL
22		Compact, grey SAND and GRAVEL, trace silt, trace clay.	55.73 21.60	15	50 DO	13			MH	
24			52.03 25.30	16	50 DO	25				
26		Very dense, grey GRAVEL, some sand in silty clay matrix. (TILL)	51.13 26.20	17	50 DO	100/ 10			MH	
28		BOREHOLE CONTINUED, FOR BEDROCK CORING DETAILS, SEE SHEET 3.								
30										
32										
34										
36										
38										
40										

DATA INPUT: SK JUNE 25/91

15 5 PERCENT AXIAL STRAIN AT FAILURE

DEPTH SCALE

1 to 100

Golder Associates

LOGGED: AP

CHECKED: ASP

PROJECT: 911-1315

RECORD OF BOREHOLE 103

SHEET 3 OF 3



LOCATION: SEE FIGURE 2

DRILLING DATE: MAY 8, 1991

DATUM: GEODETIC

INCLINATION: -90 AZIMUTH:

DRILL RIG: ACKER

DRILLING CONTRACTOR: K & S DRILLING

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (mm/min)	FLUSH % RETURN	FR-FRACTURE CL-CLEAVAGE SH-SHEAR VN-VEIN	F-FAULT J-JOINT P-POLISHED S-SLICKENSIDED	SM-SMOOTH R-ROUGH ST-STEPPED PL-PLANAR	FL-FLEXURED UE-UNEVEN W-WAVY C-CURVED	HYDRAULIC CONDUCTIVITY k, cm/sec	DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION						
															RECOVERY		R.Q.D. %	FRACT. INDEX PER 3 m	DIP w.r.t. CORE AXIS	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION
															TOTAL CORE %	SOLID CORE %				
20																				
22																				
24																				
26				51.13 26.20																
28	NQ RC	Highly to moderately weathered, grey SHALE (85%) with interbedded, fresh, grey SHALEY LIMESTONE (15%).		48.99 26.34	18	0.040	75-80													
30	NQ RC	Slightly weathered to fresh, grey SHALE (93%) interbedded with grey, fresh SHALEY LIMESTONE (7%).		46.70 26.34	19	0.027	85-95													
32	NQ RC	END OF BOREHOLE		46.70 26.34	20	0.030	85-100													
34		Limestone interbeds generally less than 50mm except at the following Elevations: 51.12m : 190mm sl + fl 49.34m : 90mm sl 48.85m : 63mm sl 47.74m : 63mm sl NOTE: sl - shaley limestone fl - fossiliferous limestone																		
36																				
38																				
40																				

T

F

R

D

Highly weathered (100mm)
Clay Seam (25mm)
Clay Seam (50mm)
Highly Weathered (240mm)
Broken Core (250mm)
Clay Seam (38mm)
Broken Core (200mm)

Broken Core (100mm)
Broken Core (240mm)
Clay Seam (12mm)
Broken Core (114mm)
Highly Weathered (12mm)

Oxidized fissures between Elev. 48.73m and Elev. 48.60m.

BACKFILL

BENTONITE SEAL

PEA GRAVEL

WATER LEVEL IN OPEN BOREHOLE AT ELEV. 74.4m DURING DRILLING.

WATER LEVEL IN PIEZOMETER AT ELEV. 74.5m ON MAY 31/91.

DEPTH SCALE:

1 to 100

Golder Associates

LOGGED: AP

DATE: MAY 8, 1991

CHECKED: ASP

DATA INPUT: SK JUNE 25/91

PROJECT: 911-1315

RECORD OF BOREHOLE 104

SHEET 1 OF 3

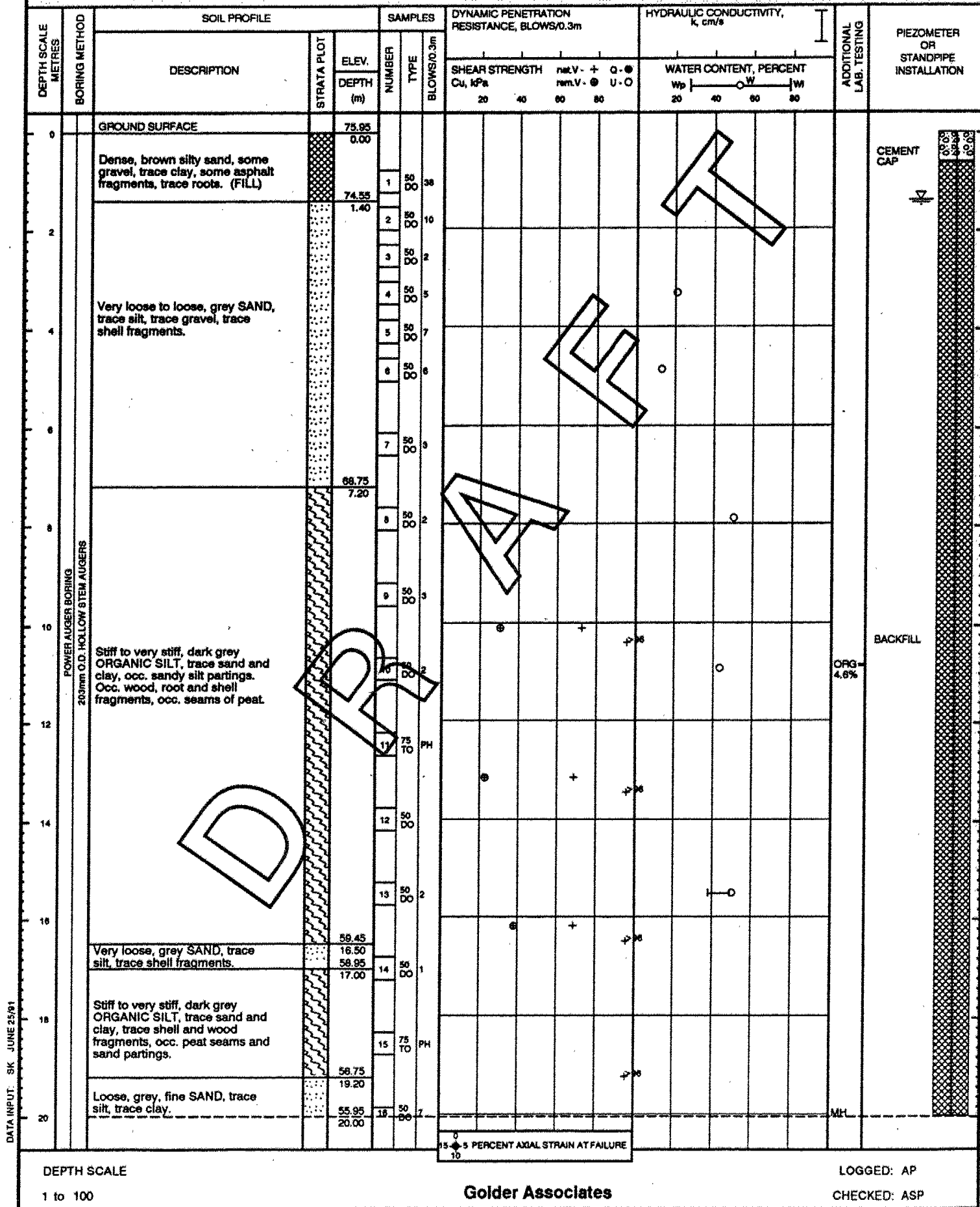
LOCATION: SEE FIGURE 2

BORING DATE: MAY 1-2, 1991

DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm



PROJECT: 911-1315

RECORD OF BOREHOLE 104

SHEET 2 OF 3

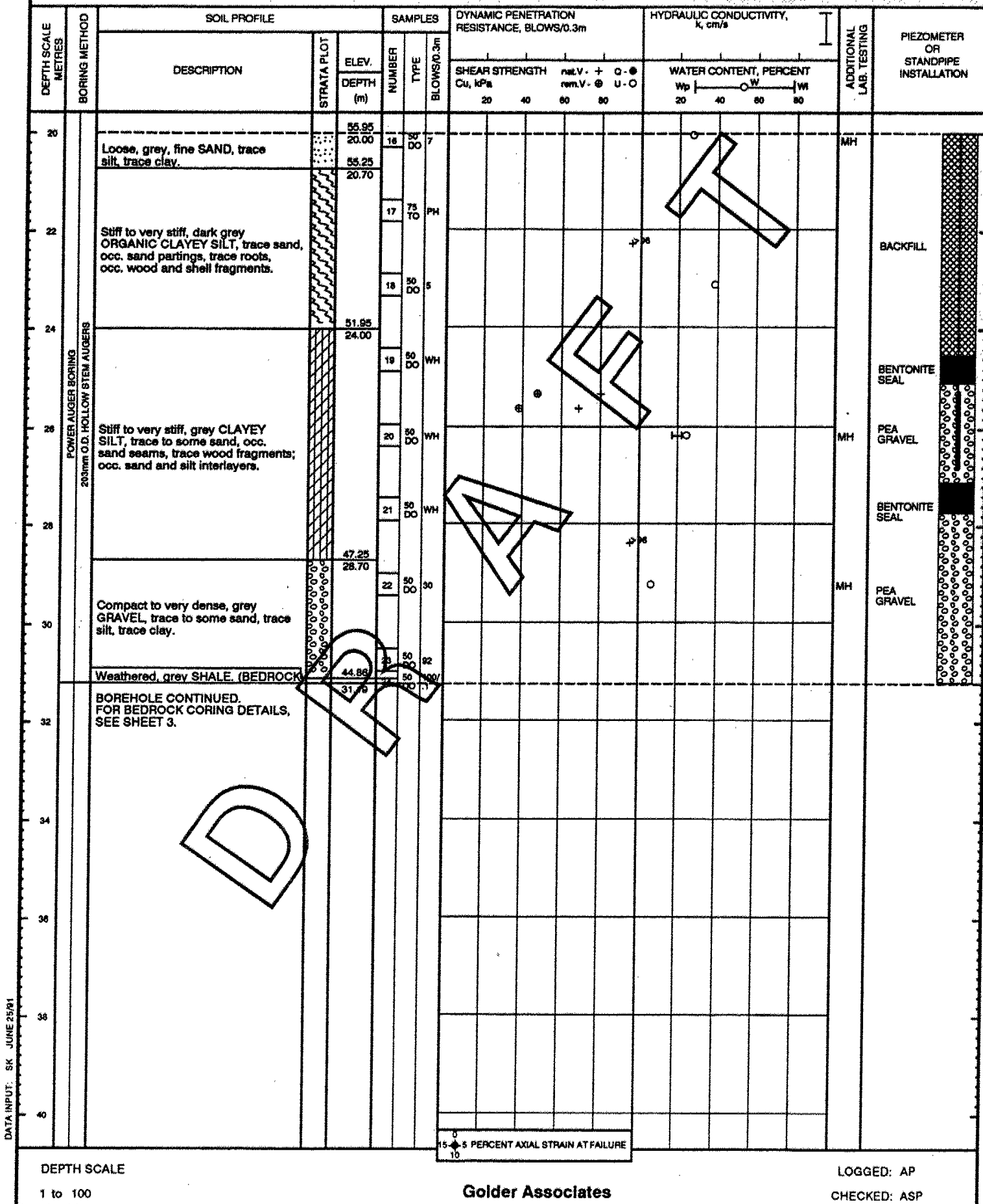
LOCATION: SEE FIGURE 2

BORING DATE: MAY 1-2, 1991

DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm



PROJECT: 911-1315

LOCATION: SEE FIGURE 2

INCLINATION: -90

AZIMUTH:

RECORD OF BOREHOLE 104

DRILLING DATE: MAY 3, 1991

DRILL RIG: ACKER

DRILLING CONTRACTOR: K & S DRILLING

SHEET 3 OF 3

DATUM: GEODETIC



DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (mm/min)	FLUSH % RETURN	FR-FRACTURE CL-CLEAVAGE SH-SHEAR VN-VEIN	F-FAULT J-JOINT P-POLISHED S-SLICKENSIDED	SM-SMOOTH R-ROUGH ST-STEPPED PL-PLANAR	FL-FLEXURED UE-UNEVEN W-WAVY C-CURVED	DISCONTINUITY DATA				HYDRAULIC CONDUCTIVITY k, cm/sec	DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION	
												RECOVERY		R.Q.D. %	FRACT. INDEX PER 3 m				TYPE AND SURFACE DESCRIPTION
												TOTAL CORE %	SOLID CORE %						
												80 60 40 20	80 60 40 20						
20																			
22																			
24																			
26																			
28																			
30																			
32	NQ RC	Moderately weathered to fresh, grey SHALE (96%) with occasional layer of fresh, grey LIMESTONE (4%).		44.78 31.19	25	0.034	95												
34	NQ RC	Fresh, grey SHALEY LIMESTONE (44%) interbedded with moderately weathered to fresh, grey SHALE (43%) and SILTSTONE (13%).		43.08 32.89	26	0.040	95												
36	NQ RC	Slightly weathered to fresh, grey SHALE (93%) with occasional layer of fresh, grey LIMESTONE (7%).		41.85 34.10	27	0.038	95-100												
38		END OF BOREHOLE		40.19 35.76															
40		Limestone and siltstone interbeds generally less than 50mm except at the following Elevations: 44.59m : 75mm sl 42.86m : 38mm si 42.67m : 127mm sl 42.47m : 190mm sl 41.34m : 50mm sl NOTE: sl - shaley limestone si - siltstone																	

DATA INPUT: SK JUNE 25/91

DEPTH SCALE:

1 to 100

Golder Associates

LOGGED: AP

DATE: MAY 3, 1991

CHECKED: ASP

WATER LEVEL IN
OPEN BOREHOLE
AT ELEV. 74.4m
DURING DRILLING.WATER LEVEL IN
PIEZOMETER AT
ELEV. 74.8m
ON MAY 31/91.

PROJECT: 911-1315

RECORD OF BOREHOLE 105

SHEET 1 OF 3

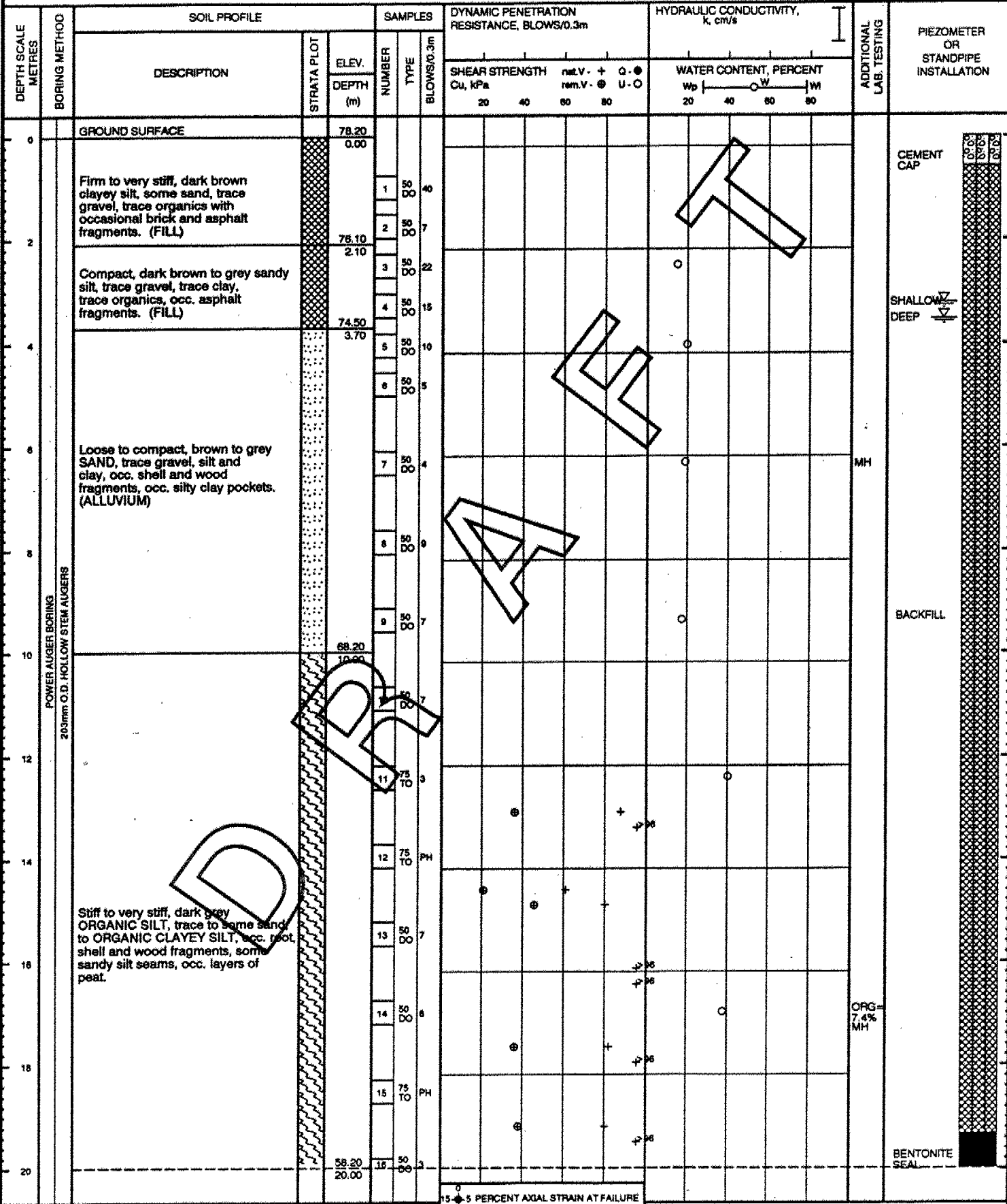
LOCATION: SEE FIGURE 2

BORING DATE: APRIL 26-29/91

DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm



DEPTH SCALE

1 to 100

LOGGED: SC

CHECKED: ASP

Golder Associates

DATA INPUT: SK JUNE 25/91

PROJECT: 911-1315

RECORD OF BOREHOLE 105

SHEET 2 OF 3

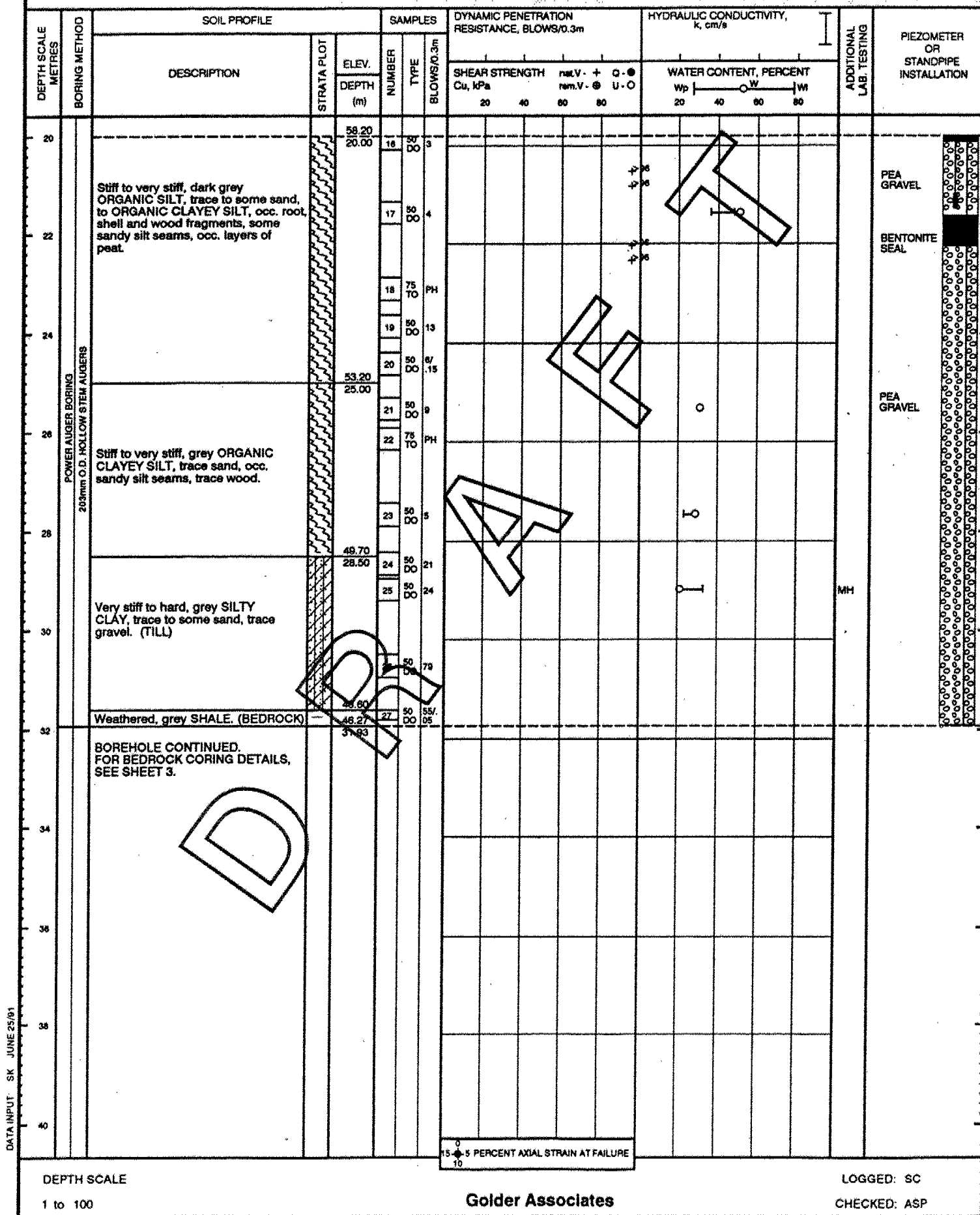
LOCATION: SEE FIGURE 2

BORING DATE: APRIL 26-29/91

DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm



PROJECT: 911-1315

RECORD OF BOREHOLE 105

SHEET 3 OF 3

LOCATION: SEE FIGURE 2

DRILLING DATE: APRIL 30, 1991

DATUM: GEODETIC

INCLINATION: -90 AZIMUTH:

DRILL RIG: ACKER

DRILLING CONTRACTOR: K & S DRILLING



DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (mm/min)	FLUSH % RETURN	FR-FRACTURE CL-CLEAVAGE SH-SHEAR VN-VEIN	F-FAULT J-JOINT P-POLISHED S-SUCKENSIDED	SM-SMOOTH R-ROUGH ST-STEPPED PL-PLANAR	FL-FLEXURED UE-UNEVEN W-WAVY C-CURVED	DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
								TOTAL CORE %	SOLID CORE %	R.Q.D. %	FRACT. INDEX PER 3m	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	HYDRAULIC CONDUCTIVITY k, cm/sec
20								8000	8000	8000	8000		
22								8000	8000	8000	8000		
24								8000	8000	8000	8000		
26								8000	8000	8000	8000		
28								8000	8000	8000	8000		
30								8000	8000	8000	8000		
32				46.27				8000	8000	8000	8000		
32				41.93				8000	8000	8000	8000		
34								8000	8000	8000	8000		
34								8000	8000	8000	8000		
36								8000	8000	8000	8000		
38								8000	8000	8000	8000		
38				41.78				8000	8000	8000	8000		
38				36.42				8000	8000	8000	8000		
40								8000	8000	8000	8000		

CONTINUED FROM SHEET 2

Slightly weathered, grey SHALE
(86%) interbedded with fresh,
grey SHALEY LIMESTONE (14%)

END OF BOREHOLE
Limestone interbeds generally
less than 50mm except at the
following Elevations:

46.05m : 50mm sl
44.91m : 90mm sl
44.11m : 90mm sl
42.06m : 75mm sl

NOTE:
sl - shaley limestone

Broken Core (250mm)
Broken Core (75mm)

Clay Seam (75mm)

Broken Core (25mm)

Broken Core (50mm)

PEA
GRAVELBENTONITE
SEALPEA
GRAVEL

WATER LEVEL IN
OPEN BOREHOLE
AT ELEV. 75.2m
DURING DRILLING.

WATER LEVEL IN
SHALLOW
PIEZOMETER AT
ELEV. 75.0m AND
IN DEEP
PIEZOMETER AT
ELEV. 74.7m
ON MAY 31/91.

DEPTH SCALE:

1 to 100

Golder Associates

LOGGED: SC

DATE: APRIL 30/91

CHECKED: ASP

DATA INPUT: SK JUNE 25/91

PROJECT: 911-1315

RECORD OF BOREHOLE 108

SHEET 1 OF 2



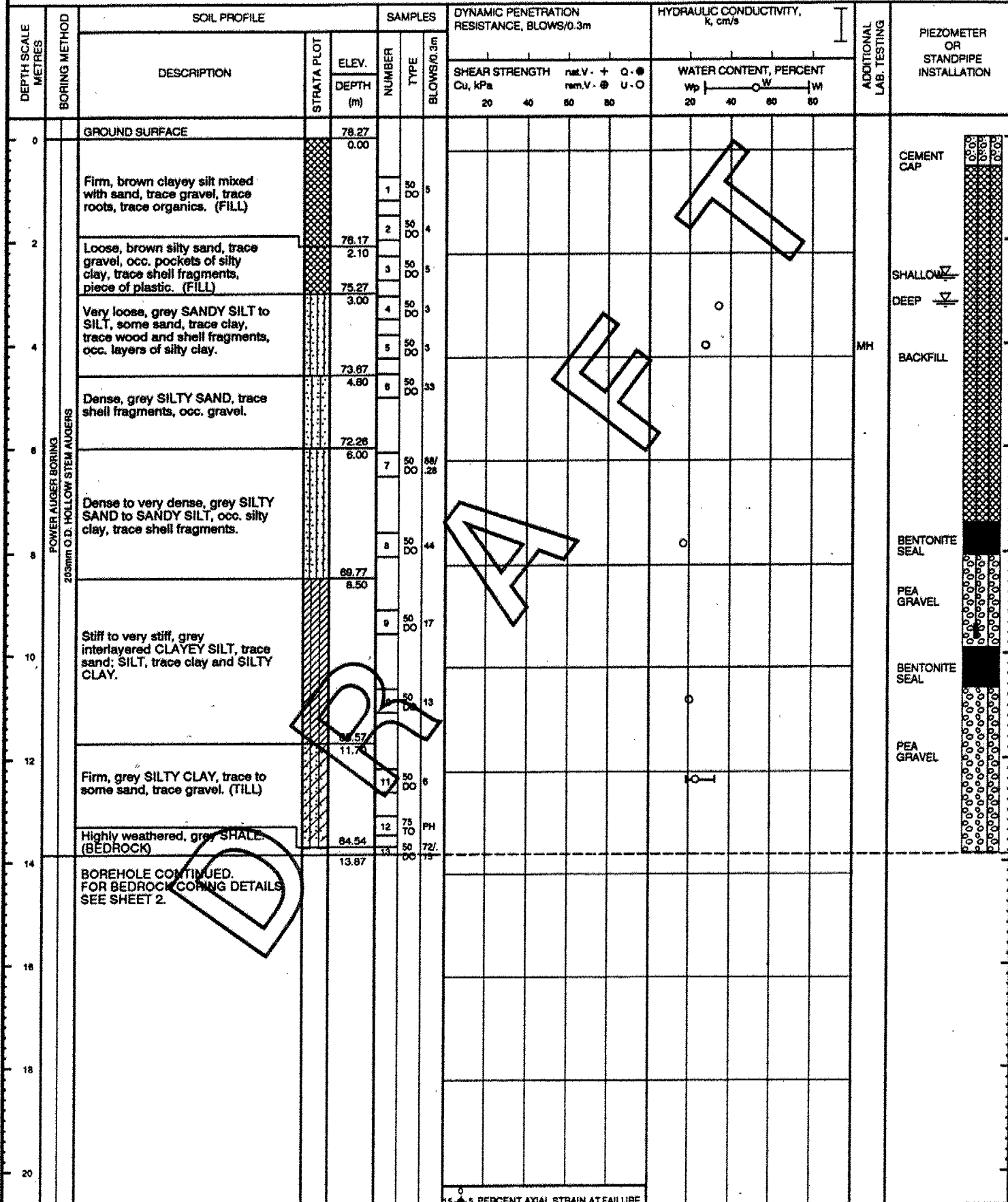
LOCATION: SEE FIGURE 2

BORING DATE: APRIL 25/91

DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm



DEPTH SCALE

1 to 100

Golder Associates

LOGGED: SC

CHECKED: ASP

DATA INPUT: SK JUNE 25/91

PROJECT: 911-1315

RECORD OF BOREHOLE 108

SHEET 2 OF 2

LOCATION: SEE FIGURE 2

DRILLING DATE: APRIL 25, 1991

DATUM: GEODETIC

INCLINATION: -90 AZIMUTH:

DRILL RIG: ACKER

DRILLING CONTRACTOR: K & S DRILLING



DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (mm/min)	FLUSH	FR-FRACTURE CL-CLEAVAGE SH-SHEAR VN-VEIN	F-FAULT J-JOINT P-POLISHED S-SUCKENSIDED	SM-SMOOTH R-ROUGH ST-STEPPED PL-PLANAR	FL-FLEXURED UE-UNEVEN W-WAVY C-CURVED	HYDRAULIC CONDUCTIVITY k, cm/sec	WATER LEVEL INSTRUMENTATION	NOTES
10														
12		CONTINUED FROM SHEET 1												
14				84.40										
				13.87										
14		Slightly weathered to fresh, grey SHALE (83%) interbedded with fresh, grey SHALEY LIMESTONE (17%).			14	0.054	80							PEA GRAVEL BENTONITE SEAL
				63.12										
18		Fresh, grey SHALEY LIMESTONE (37%) and FOSSILIFEROUS LIMESTONE (13%) interbedded with slightly weathered to fresh, grey SHALE (47%).			15	0.069	85							PEA GRAVEL
				62.09										
18				16.18										
				60.74										
18		Fresh, grey SHALE (81%) interbedded with fresh, grey SHALEY LIMESTONE (19%).			16	0.098	80							
				17.53										
18		END OF BOREHOLE												
20		Limestone interbeds generally less than 50mm except at the following Elevations:												
		64.00m : 75mm sl												
		63.38m : 63mm sl												
		63.04m : 75mm sl												
		62.97m : 100mm sl												
		62.81m : 140mm sl + fl												
		62.63m : 63mm sl												
		62.34m : 75mm sl												
		62.14m : 50mm sl												
22		61.56m : 100mm sl												
		61.22m : 127mm sl												
24		NOTE:												
		sl - shaley limestone												
		fl - fossiliferous limestone												
26														
28														
30														

DEPTH SCALE:

1 to 100

Golder Associates

LOGGED: SC

DATE: APRIL 25/91

CHECKED: ASP

DATA INPUT: SK JUNE 25/91

PROJECT: 911-1315

RECORD OF BOREHOLE 109

SHEET 1 OF 1

LOCATION: SEE FIGURE 2

BORING DATE: APRIL 24/91

DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm



DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE		SAMPLES		DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m		HYDRAULIC CONDUCTIVITY, k, cm/s		ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION		
		DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	NUMBER	TYPE	BLOWS/0.3m	SHEAR STRENGTH				WATER CONTENT, PERCENT	
								CU, kPa	nom. V - \oplus U - \odot			Wp	W
0	POWER AUGER BORING 203mm O.D. HOLLOW STEM AUGERS	GROUND SURFACE		77.69									
		ASPHALTIC CONCRETE		77.44									
				0.25									
		Loose to compact, brown silty sand, trace gravel, trace organics, occ. pockets of silty clay. (FILL)			1	50 DO	7						
2				75.59									
		Soft, dark grey ORGANIC SILT, trace wood fragments, occ. sand seams.		2.10	2	50 DO	11						
				74.79									
		Loose, grey SILTY SAND mixed with silty clay, some organics (wood).		2.90	3	50 DO	4						
				73.89									
4				3.80	4	50 DO	3						
		Compact brown to greyish brown SAND, trace gravel, trace shell fragments, occ. organics.			5	50 DO	16						
				72.19									
				5.50	6	50 DO	17						
6			Very loose to loose, grey SAND, occ. silty clay, trace shell fragments.		7	50 DO	4						
					8	50 DO	3						
				68.89									
			Soft, brown PEAT, trace clay, trace shell fragments.		8.90	9	50 DO	3			ORG-10%		
10				67.49									
			Firm, grey SILTY CLAY, trace sand, occ. clayey silt seams, occ. root fragments.		10.20	10	50 DO	5					
				65.49									
12		Stiff, brown SILTY CLAY, trace sand and gravel. (TILL)		12.20	11	50 DO	13						
			64.13										
14		Completely weathered, grey SHALE (BEDROCK)		13.56	12	50 DO	80/13/05						
			63.57										
		END OF BOREHOLE		14.12	13	50 DO	05						
16													
18													
20													

WATER LEVEL IN OPEN BOREHOLE AT ELEV. 75.1m ON COMPLETION OF DRILLING.

15 \blacklozenge 5 PERCENT AXIAL STRAIN AT FAILURE

ORG-10%

WATER LEVEL IN
OPEN BOREHOLE
AT ELEV. 75.1m
ON COMPLETION
OF DRILLING.

0 15 5 PERCENT AXIAL STRAIN AT FAILURE

DEPTH SCALE

1 to 100

Golder Associates

LOGGED: SC

CHECKED: ASP

DATA INPUT: SK JUNE 25/91

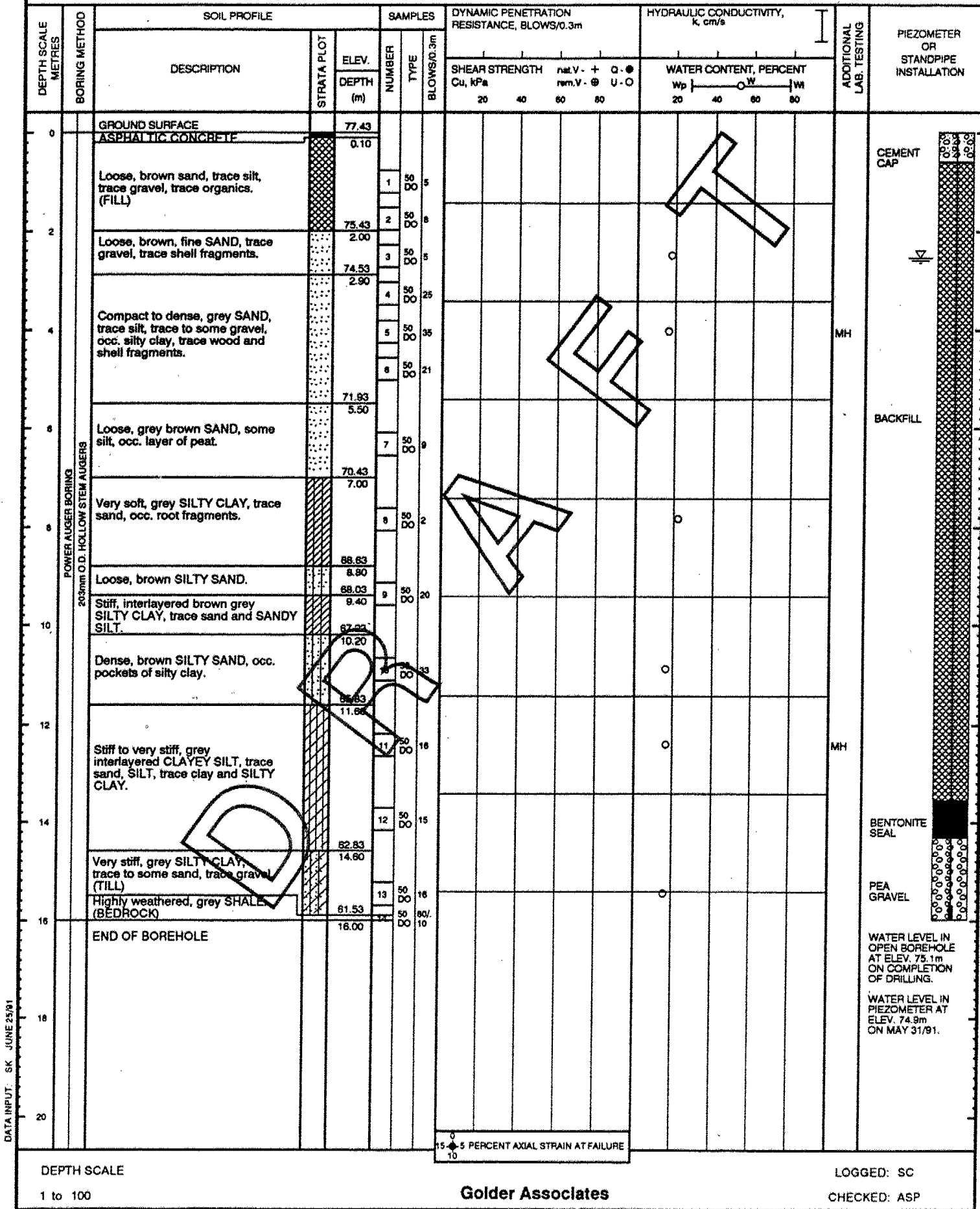
LOCATION: SEE FIGURE 2

BORING DATE: APRIL 24/91

DATUM: GEODETIC.

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm



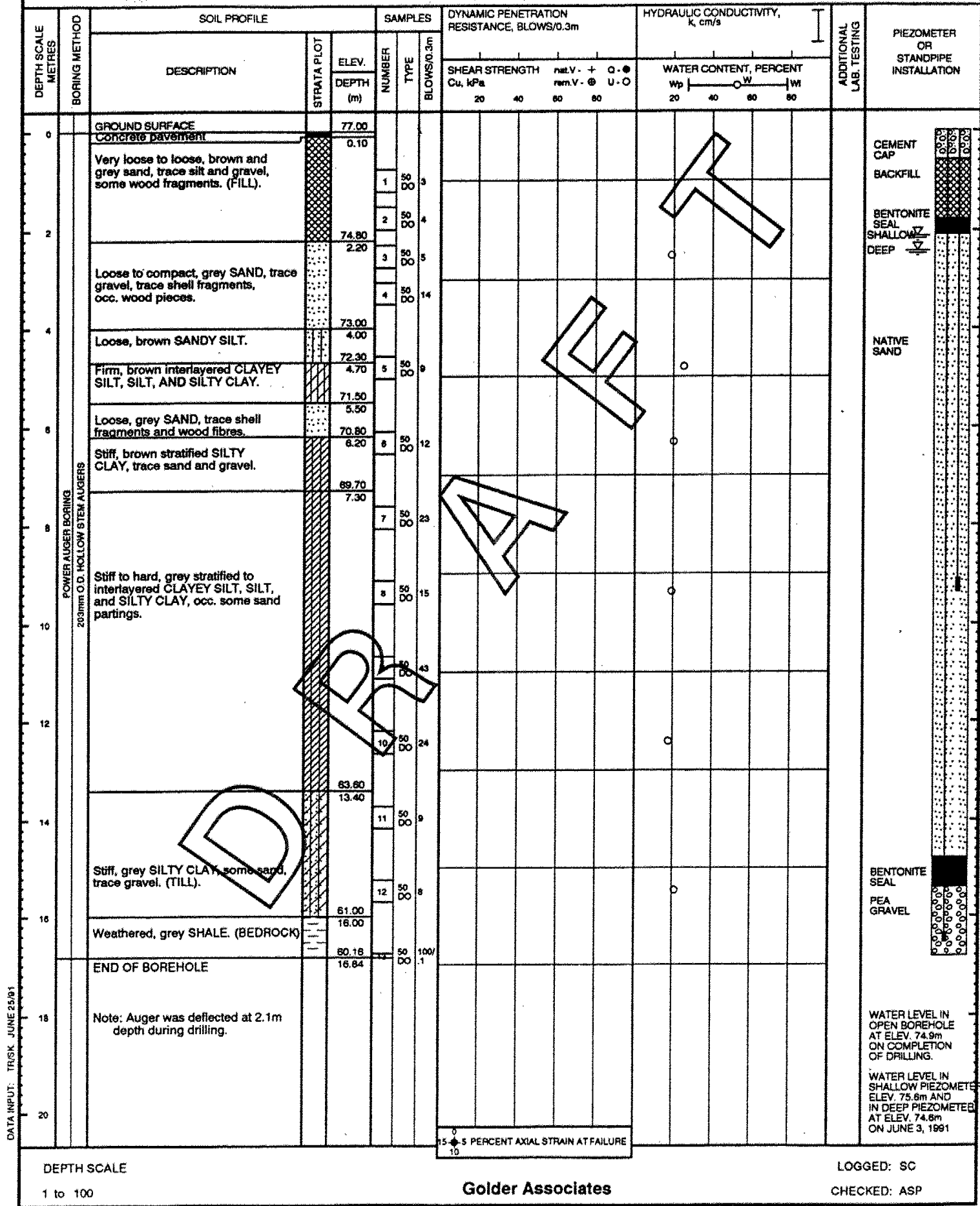
LOCATION: SEE FIGURE 2

BORING DATE: MAY 30, 1991

DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 53.5kg; DROP, 760mm



PROJECT: 911-1315

RECORD OF BOREHOLE 112

SHEET 1 OF 1

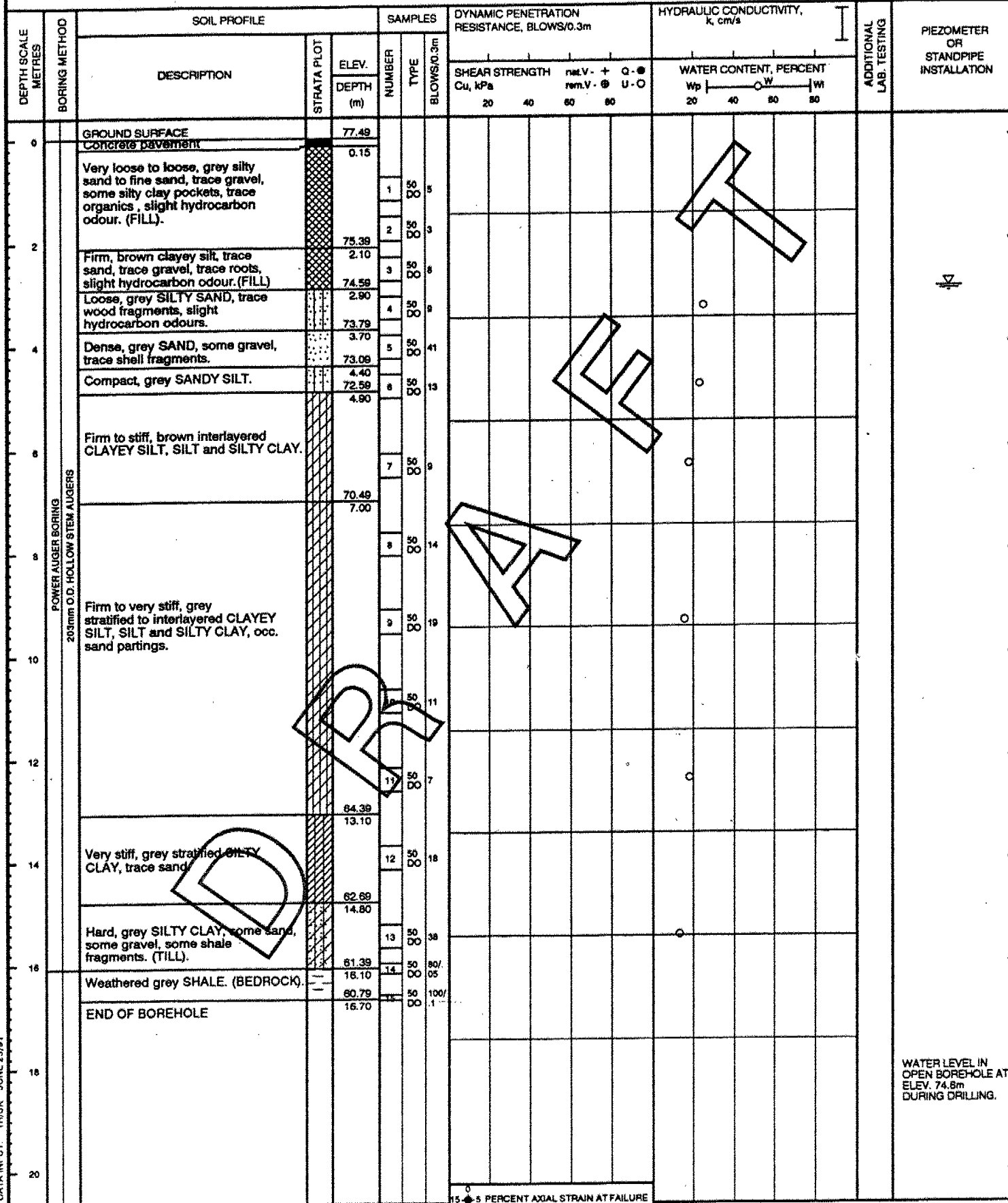
LOCATION: SEE FIGURE 2

BORING DATE: MAY 30, 1991

DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm



DEPTH SCALE

1 to 100

Golder Associates

LOGGED: SC

CHECKED: ASP

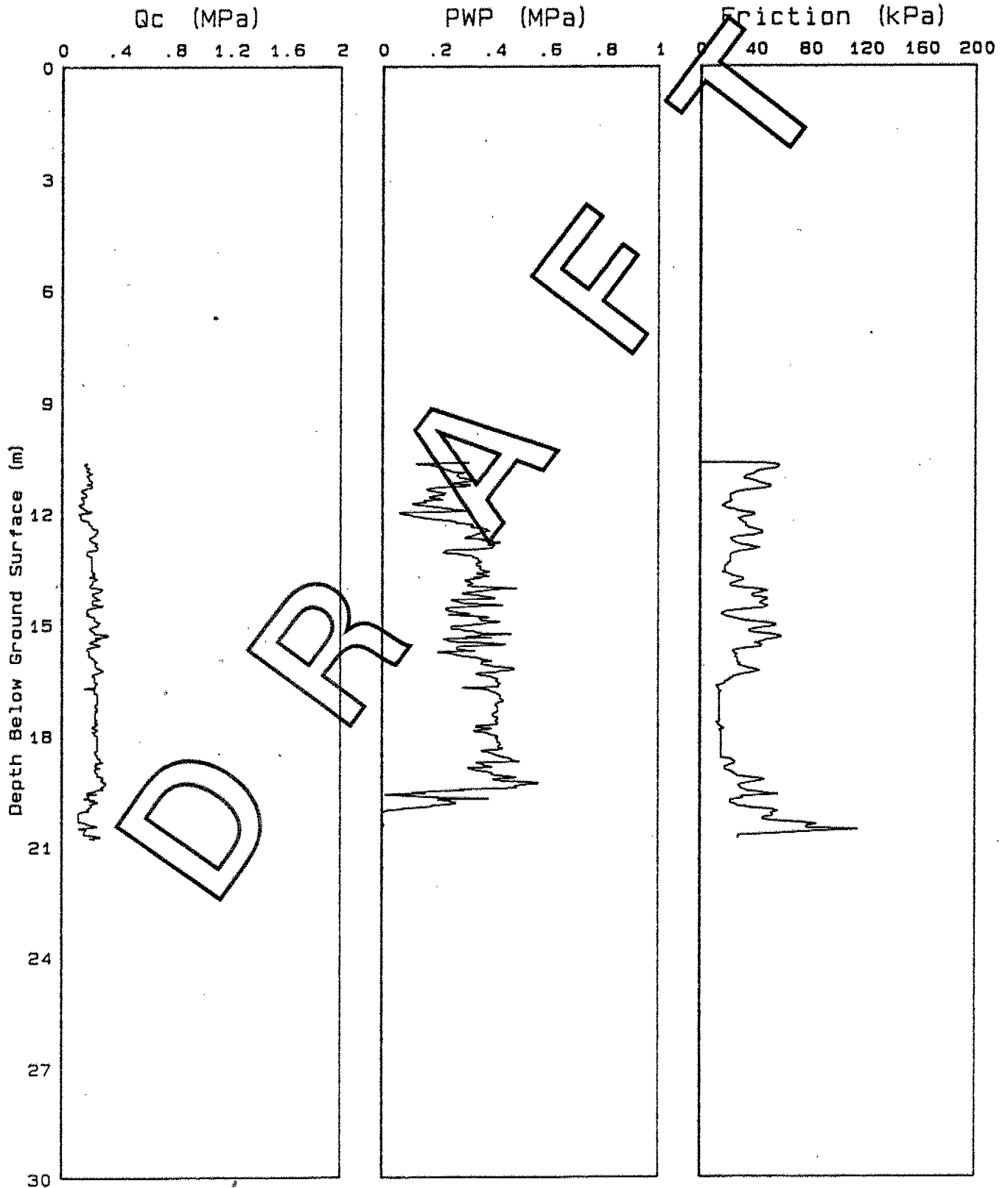
DATA INPUT: TRISK JUNE 25/91

RECORD OF CONE PENETRATION TEST 101

CPT Test : CPT106
 Test Date : 09 May 1991
 Location : SEE FIGURE 2

Page 1 of 1
 Operator : AP

Ground Surf. Elev. : 78.55 m
 Pre-bore Depth : 10.7 m
 Water Table Depth : 2.7 m



13 MAY 1991

SK

911-1315

RECORD OF CONE PENETRATION

TEST 102

CPT Test : CPT107

Page 1 of 1

Ground Surf. Elev. : 78.48m

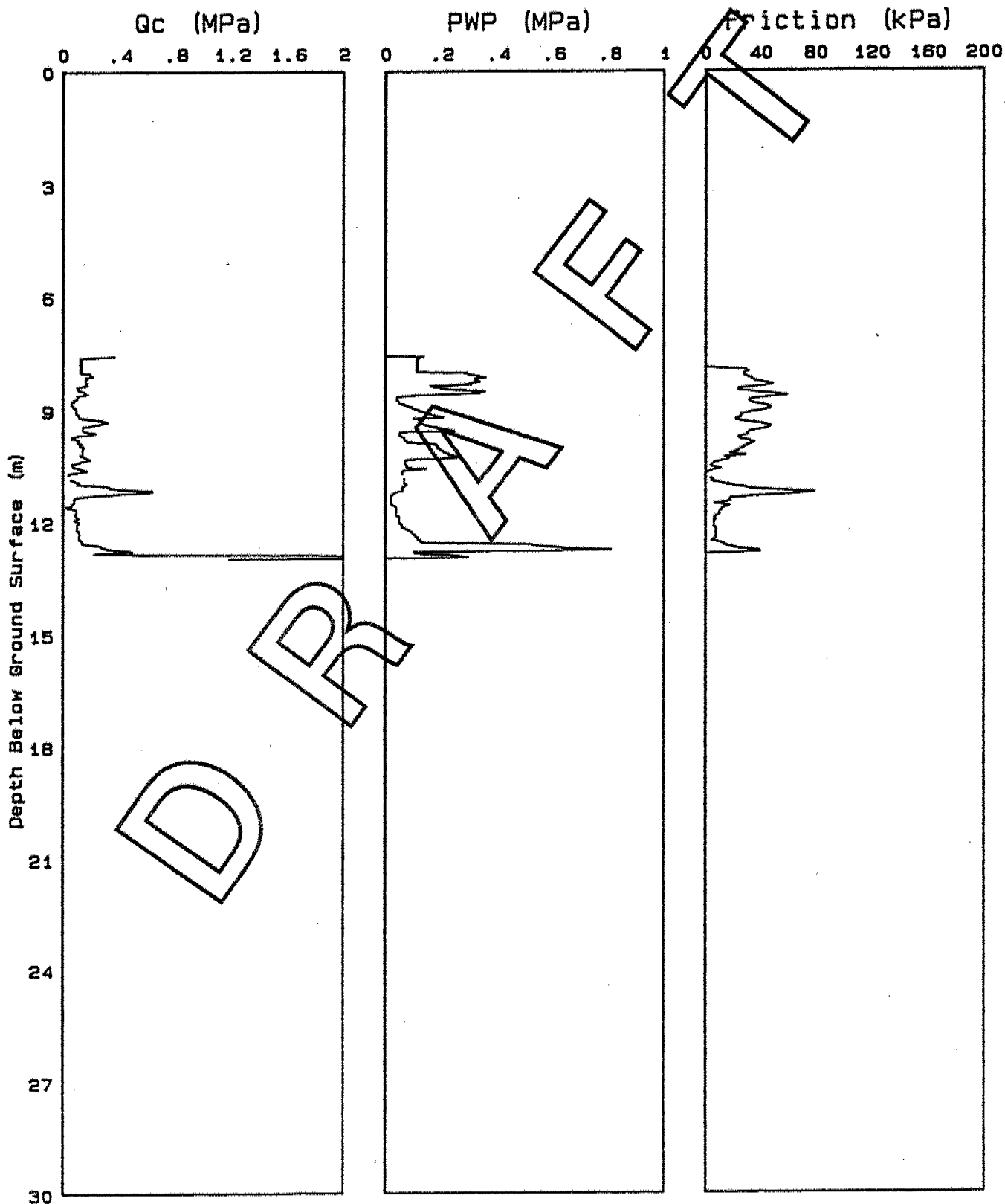
Test Date : 09 May 1991

Operator : AP

Pre-bore Depth : 7.6 m

Location : SEE FIGURE 2

Water Table Depth : 2.7 m



13 MAY 1991

SK

911-1315

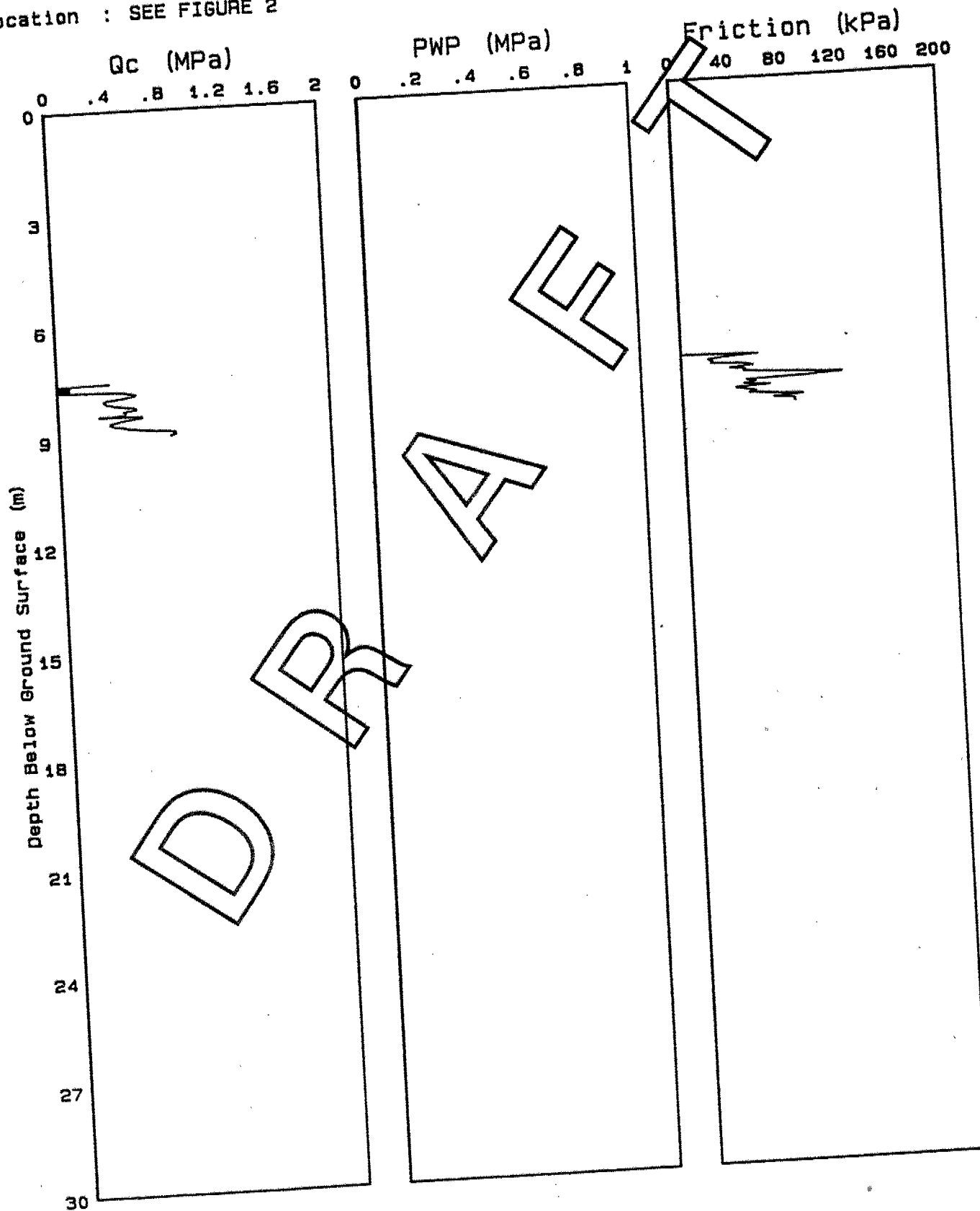
RECORD OF CONE PENETRATION

TEST

CPT Test : CPT108
Test Date : 09 May 1991
Location : SEE FIGURE 2

Page 1 of 1
Operator : AP

Ground Surf. Elev. : 78.61m
Pre-bore Depth : 7.5 m
Water Table Depth : 2.7 m



SK

911-1315

SITE LOCATION MAP

FIGURE 1



FORM PRODUCED JUNE 1986

Form CA-D-4 (imperial)

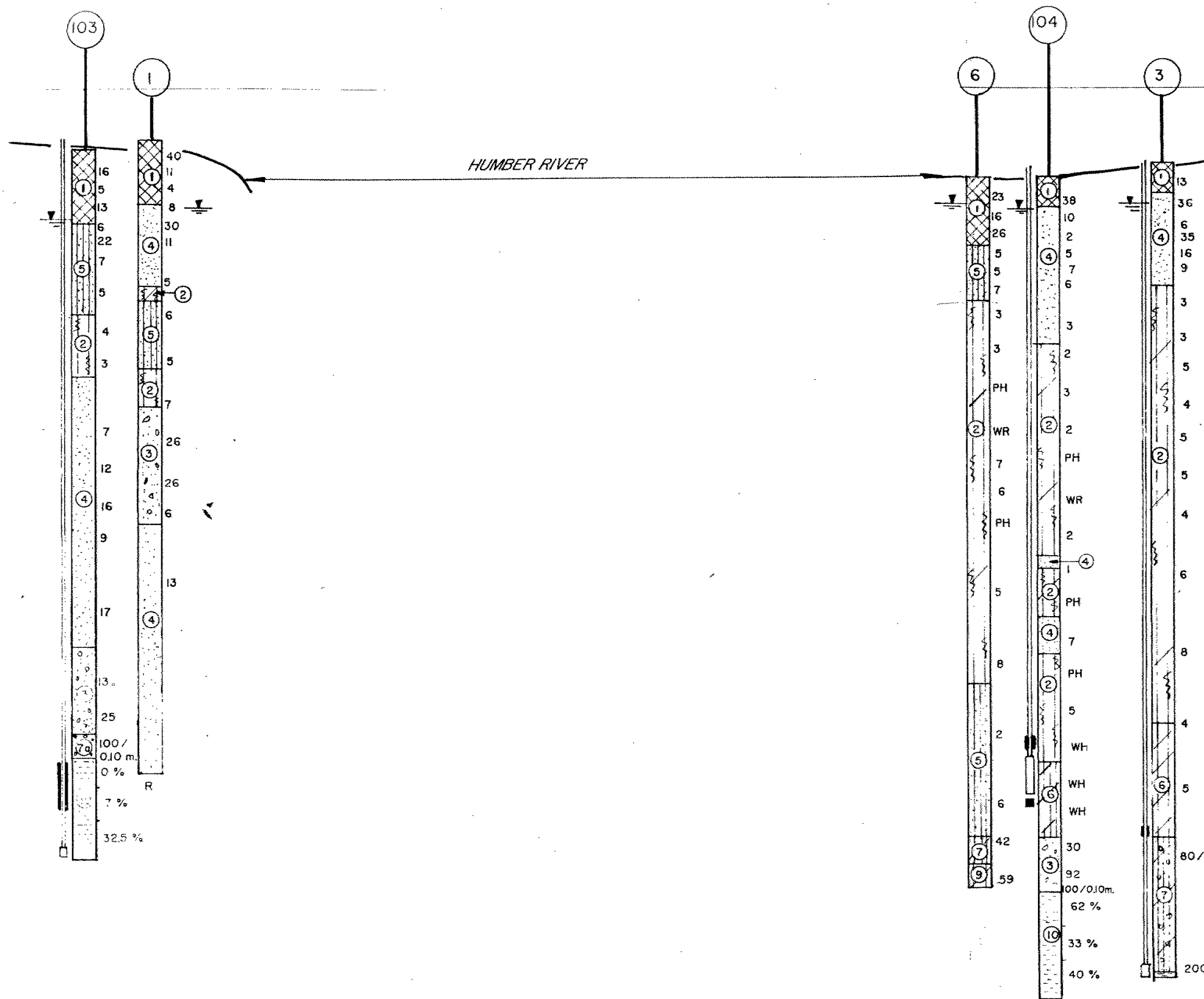
Date JULY 16, 1991

Project 911-1315

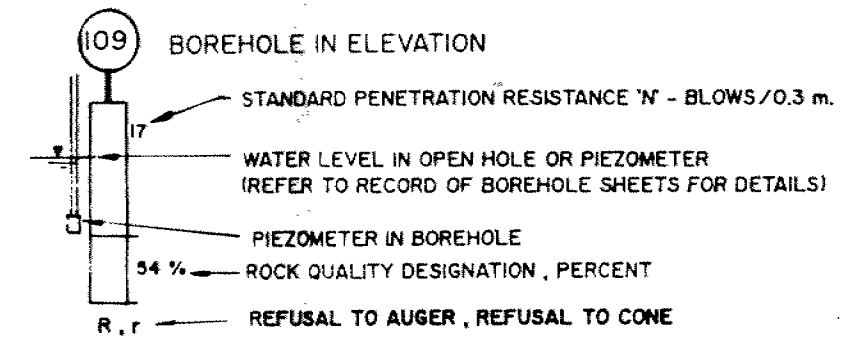
Golder Associates

Drawn D.M.

Chkd.



LEGEND



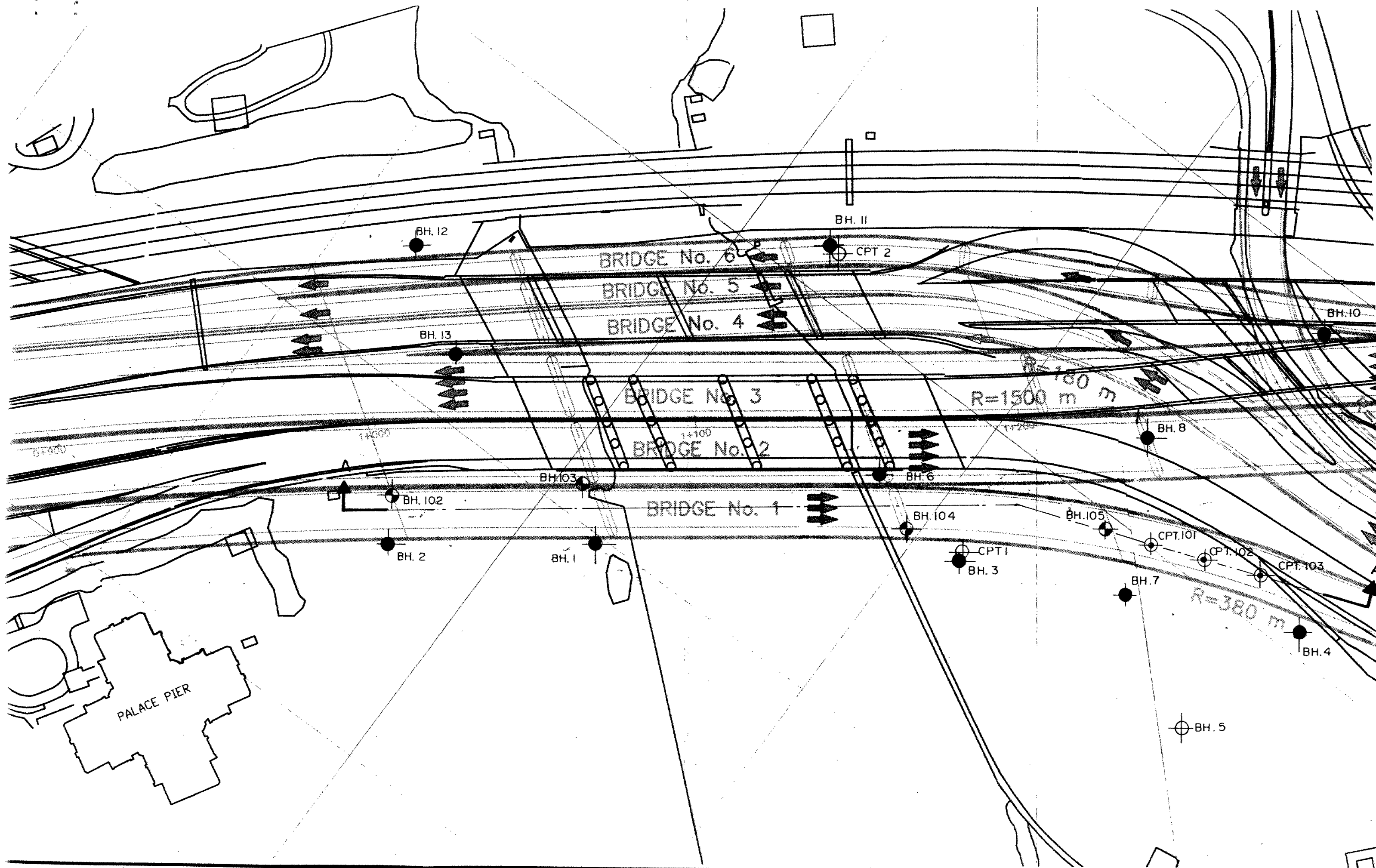
STRATIGRAPHY

- FILL
- ORGANIC SILT TO ORGANIC CLAYEY SILT
- SAND AND GRAVEL
- SAND
- SILT AND FINE SAND TO SILTY SAND
- CLAYEY SILT TO SILTY CLAY
- CLAYEY SILT TO SILTY CLAY TILL
- GRAVEL TILL
- INTERLAYERED CLAYEY SILT, SILT AND SILTY CLAY
- RESIDUAL SOIL
- SHALE BEDROCK

DRAFT

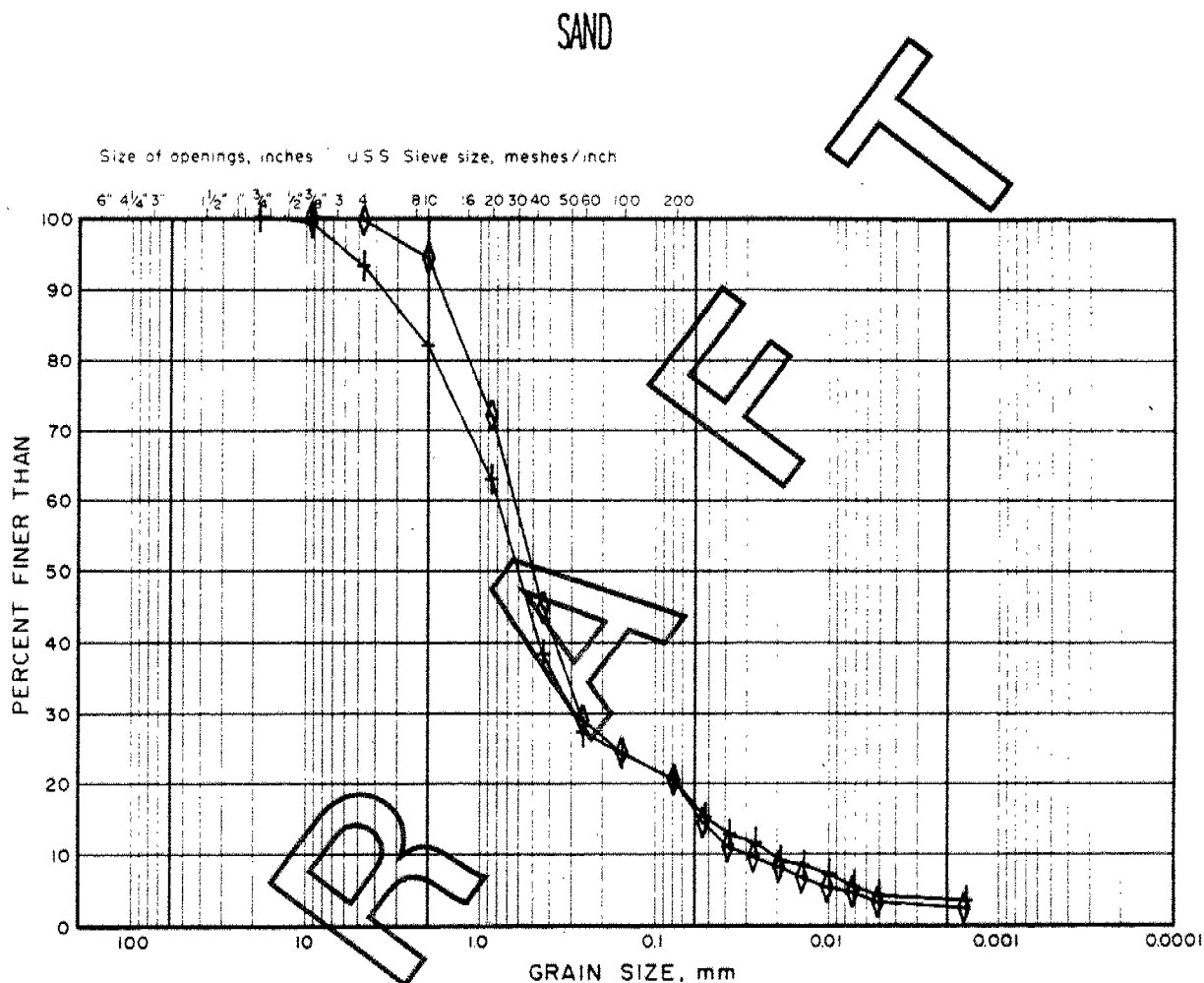
SCALES

HORIZONTAL 1 : 500
VERTICAL 1 : 200



GRAIN SIZE DISTRIBUTION

FIGURE 5A



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
+	103	11	63.1
◇	103	14	57.0

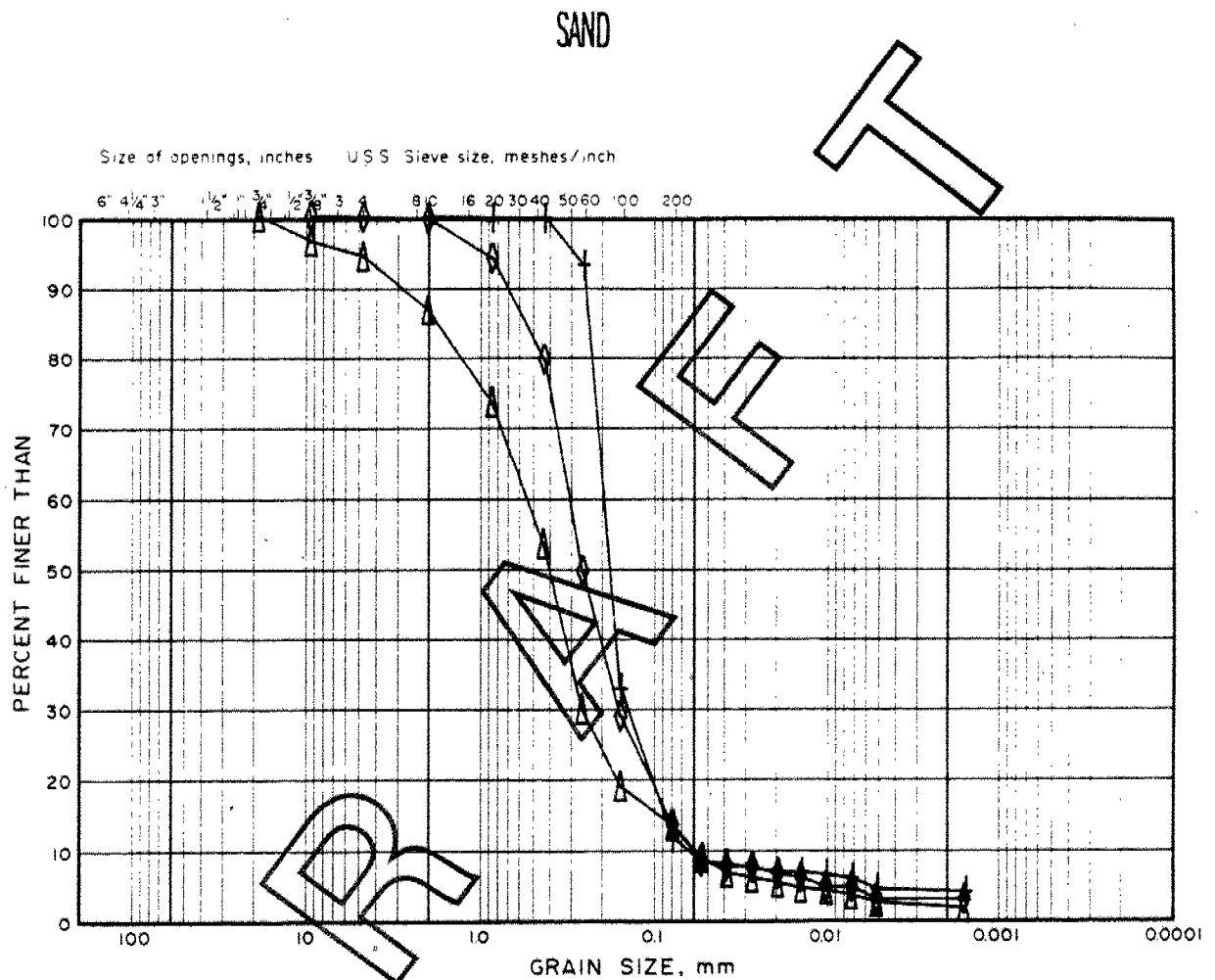
911-1315

Project

Golder Associates

GRAIN SIZE DISTRIBUTION

FIGURE 5B



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

LEGEND

SYMBOL BOREHOLE SAMPLE ELEVATION (m)

+	104	16	55.7
◇	105	7	71.6
Δ	110	5	73.2

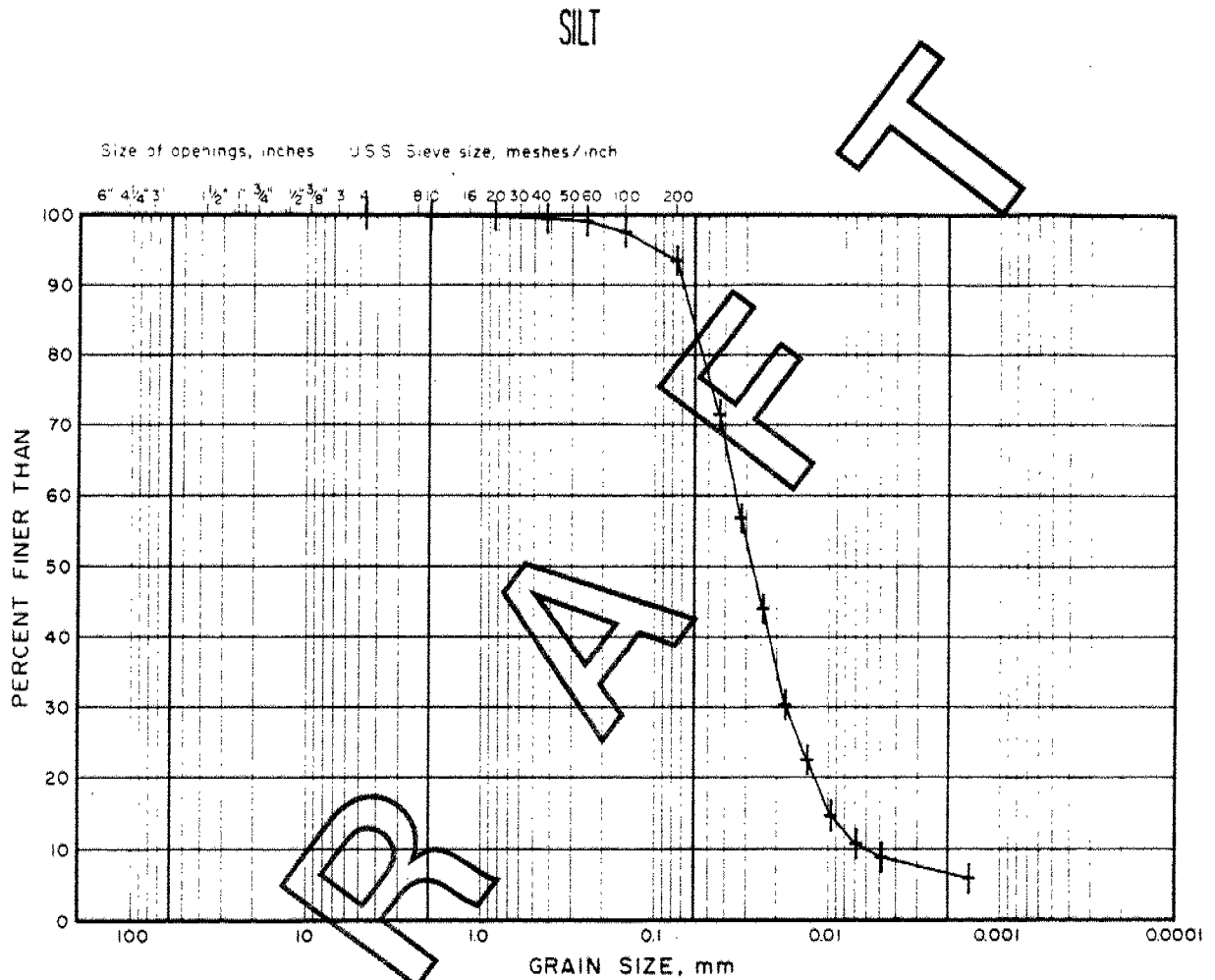
911-1315

Project

Golder Associates

GRAIN SIZE DISTRIBUTION

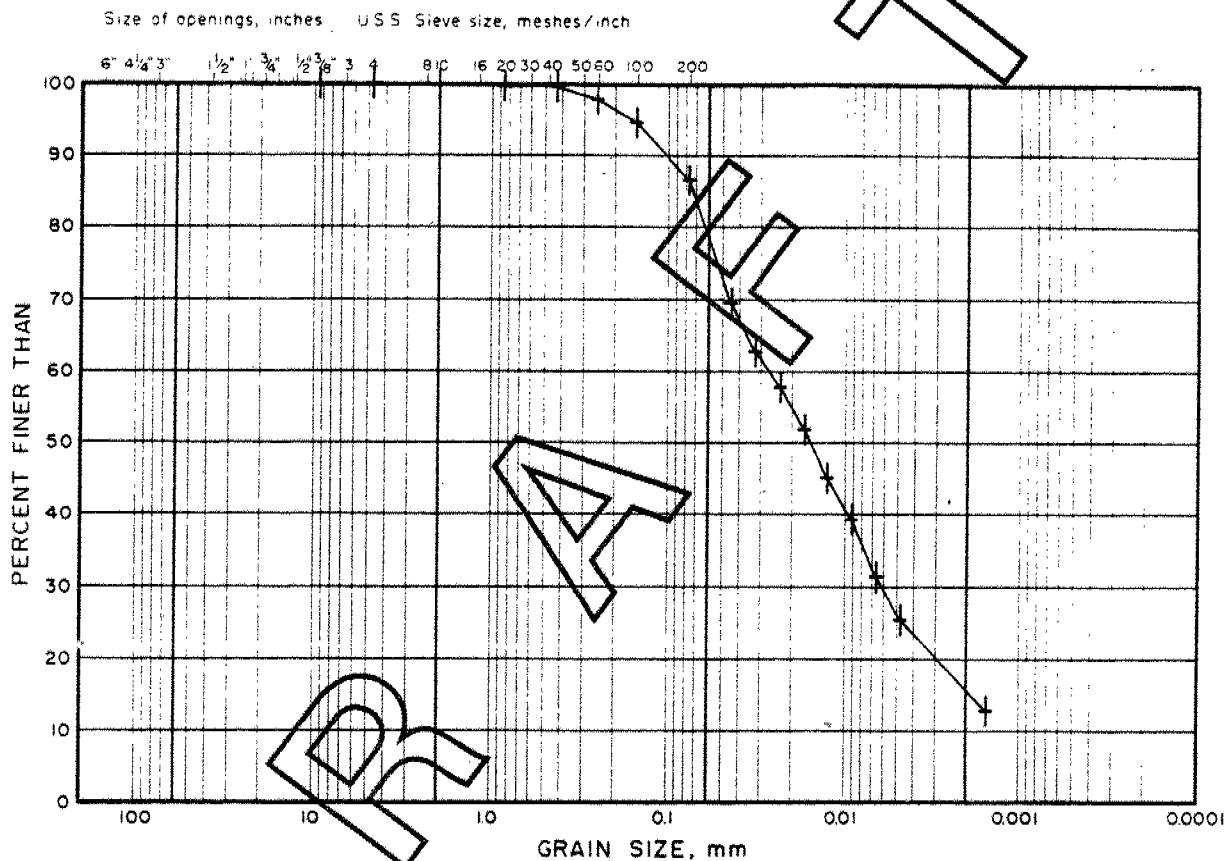
FIGURE 6



GRAIN SIZE DISTRIBUTION

FIGURE 7

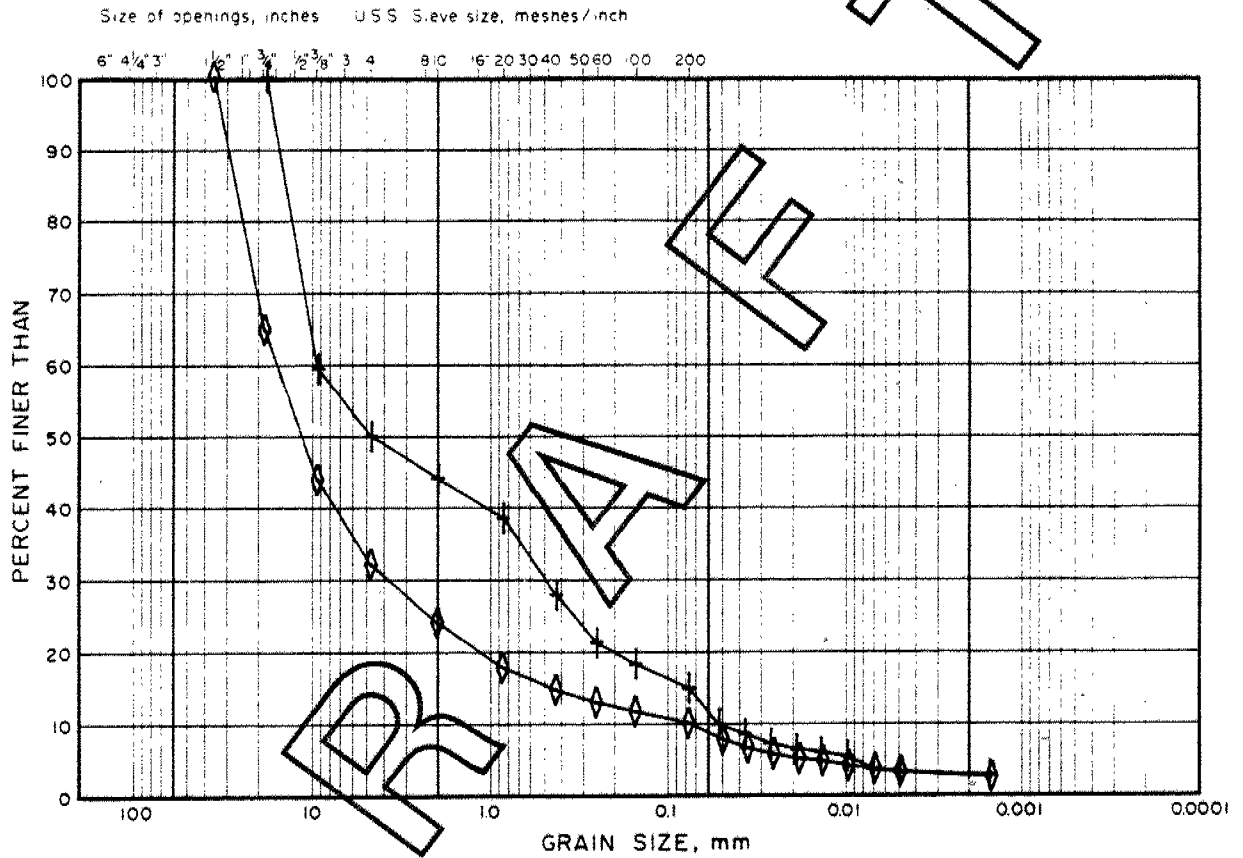
ORGANIC CLAYEY SILT



GRAIN SIZE DISTRIBUTION

FIGURE 8

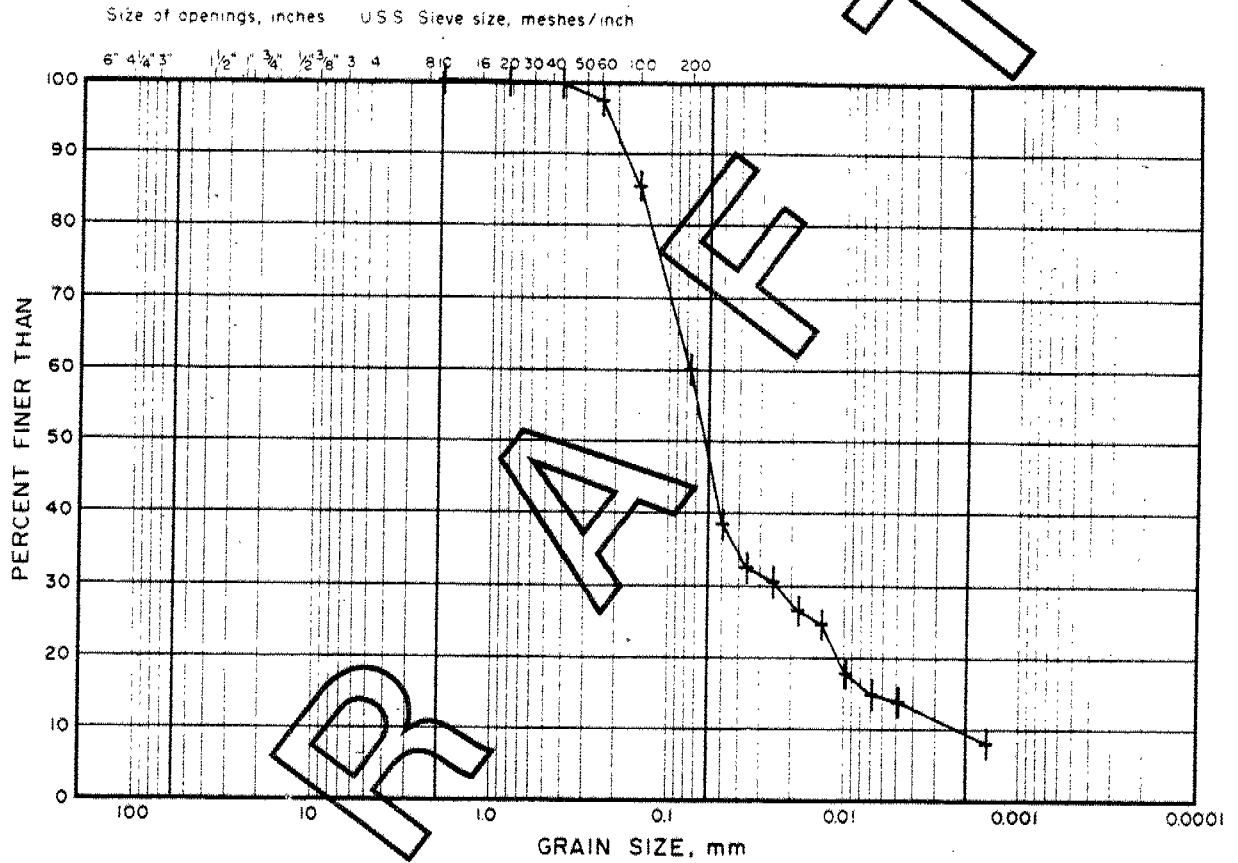
SAND AND GRAVEL TO GRAVEL



GRAIN SIZE DISTRIBUTION

FIGURE 9

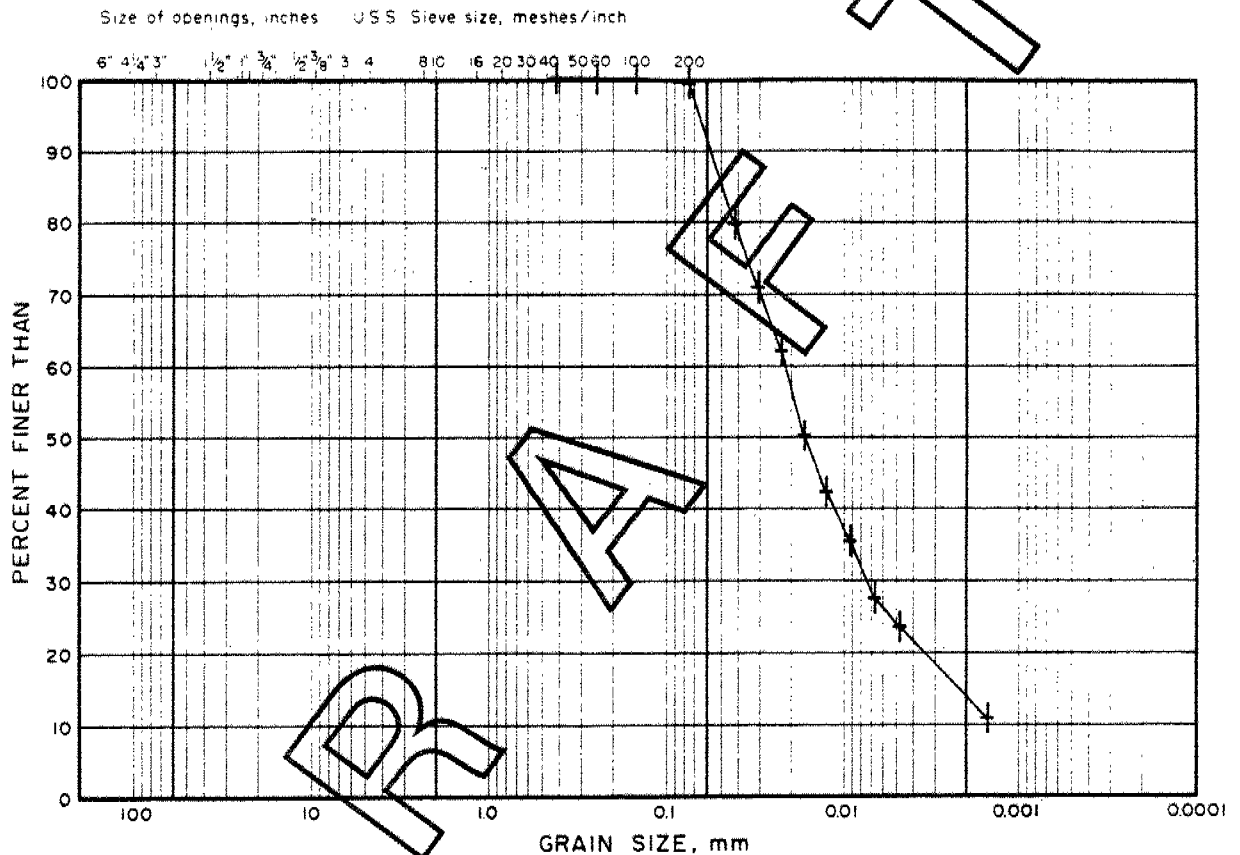
CLAYEY SILT



GRAIN SIZE DISTRIBUTION

FIGURE 10

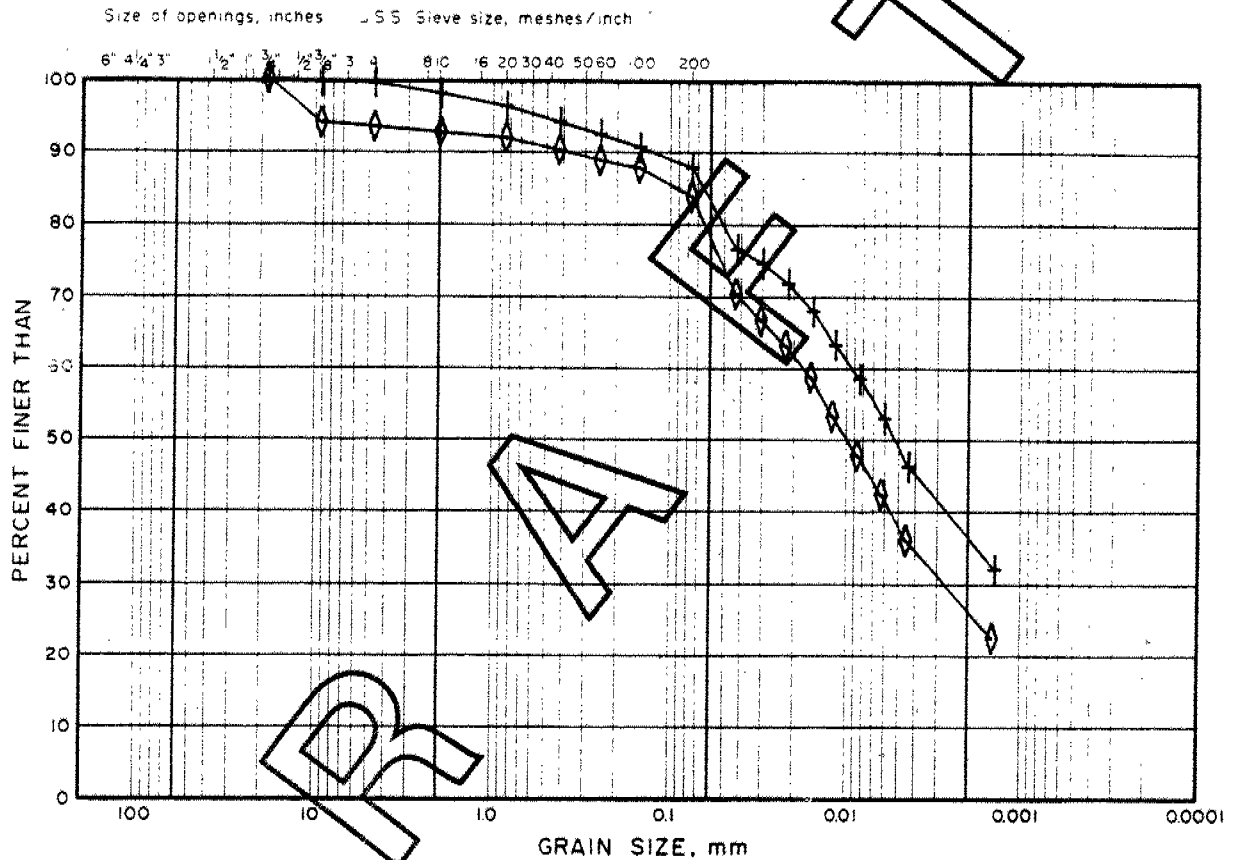
LAYERED CLAYEY SILT, SILT, SILTY CLAY



GRAIN SIZE DISTRIBUTION

FIGURE II

SILTY CLAY (TILL)



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

LEGEND

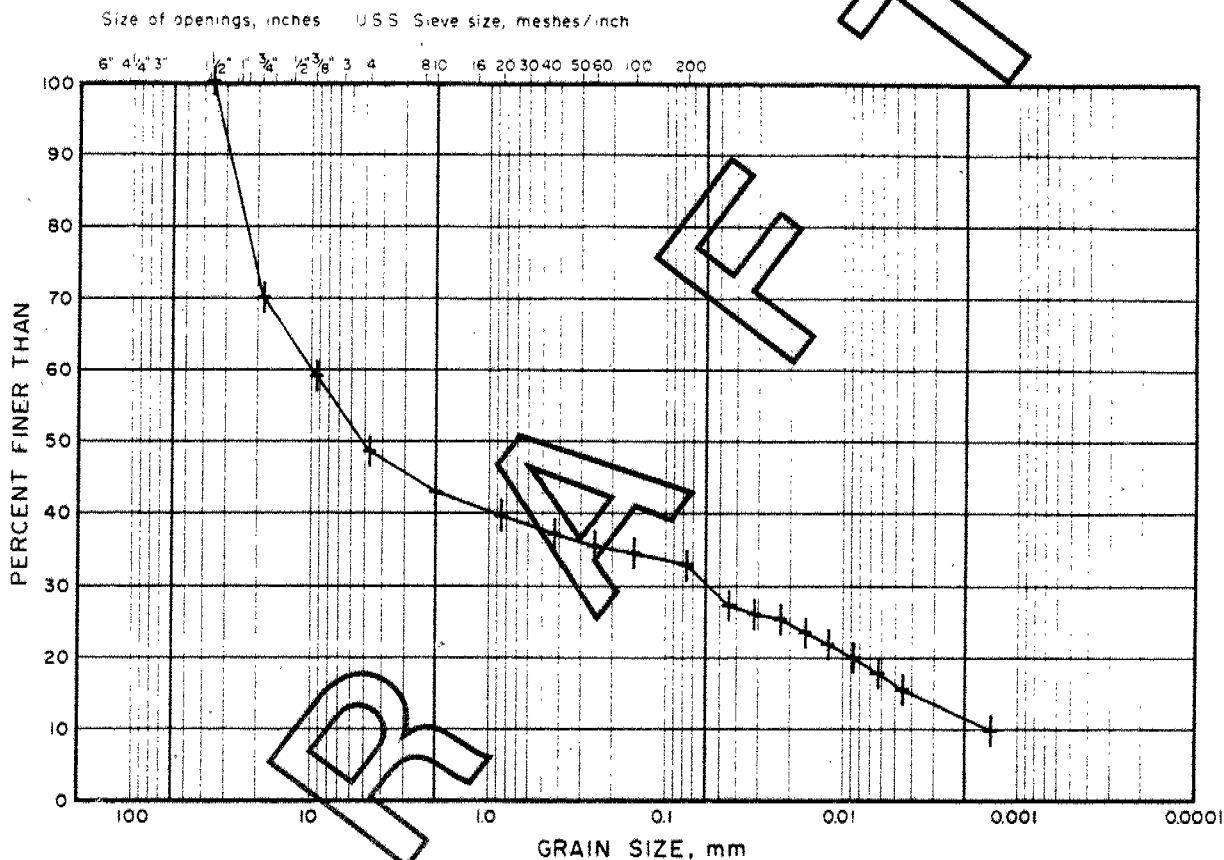
SYMBOL BOREHOLE SAMPLE ELEVATION (m)

+	102	8	71.1
◇	105	25	48.8

GRAIN SIZE DISTRIBUTION

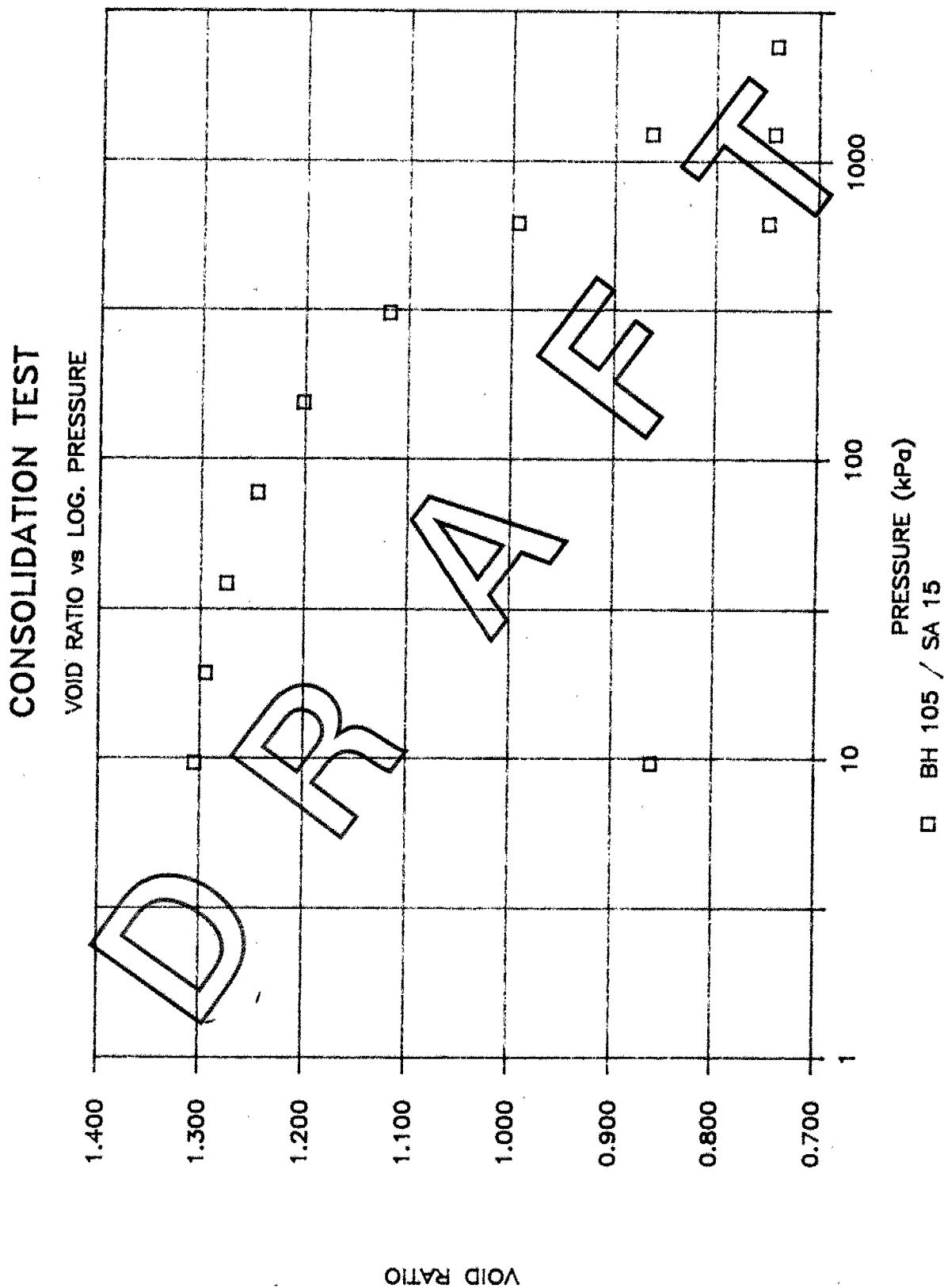
FIGURE 12

GRAVEL (TILL)



CONSOLIDATION TEST VOID RATIO VS. LOG PRESSURE

FIGURE 13



APPENDIX A

SELECTED BOREHOLE LOGS FROM

GOLDER ASSOCIATES REPORT

881-1312

NOVEMBER 1988

February 1992

911-1315

RECORD OF BOREHOLE 1

SHEET 1 OF 2

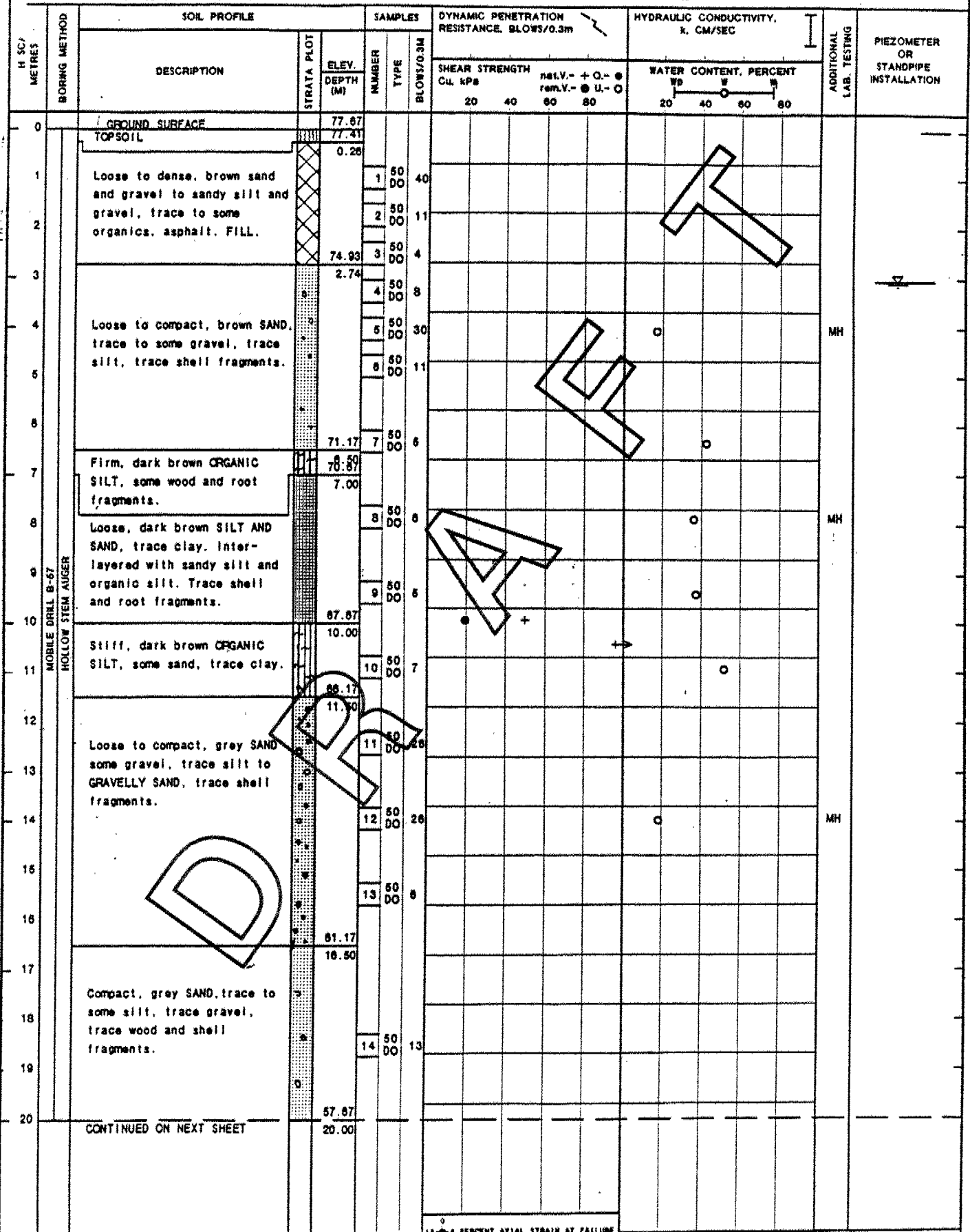
LOCATION SEE FIGURE 2

BORING DATE MAY 8th 1988

DATUM GEODETIC

SAMPLER HAMMER, 63.5kg, DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg, DROP, 760mm



DEPTH SCALE

1 : 100

0
10 PERCENT AXIAL STRAIN AT FAILURE
10

Golder Associates

LOGGED R.F.

CHECKED ASP.

RECORD OF BOREHOLE 1

SHEET 2 of 2

LOCATION: SEE FIGURE 2

BORING DATE MAY 6/9, 1988

DATUM: GEODETTIC

SAMPLER: HAMMER, 63.5kg, DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg, DROP, 760mm



1 SCA METERS	BORING METHOD	SOIL PROFILE		SAMPLES		DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m	HYDRAULIC CONDUCTIVITY, k, CM/SEC	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION	
		DESCRIPTION	STRATA PLOT ELEV. DEPTH (M)	NUMBER	TYPE	SHEAR STRENGTH Cu, kPa nat.V. - + Q - ● rem.V. - ● U - ○	WATER CONTENT, PERCENT 20 40 60 80			
20	MOBILE DRILL B-67 N/A CASING	CONTINUED FROM PREVIOUS PAGE	57.87 20.00							
21		Compact, grey SAND, trace to some silt, trace gravel, trace wood and shell fragments	[Strata Plot: Dotted pattern from 20.00 to 27.13]	15	60	WS				
22										
23										
24										
25										
26										
27										
28				END OF BOREHOLE	50.64					
29				REFUSAL TO TRI-CONE ADVANCE	27.13					
30										
31										
32										
33										
34										
35										
36										
37										
38										
39										
40										

WATER LEVEL IN
OPEN HOLE AT
ELEVATION 74.6m
ON COMPLETION
OF DRILLING.

DEPTH SCALE

1:100

Golder Associates

LOGGED R.F.

CHECKED A.S.P.

RECORD OF BOREHOLE - 2

SHEET 1 of 1

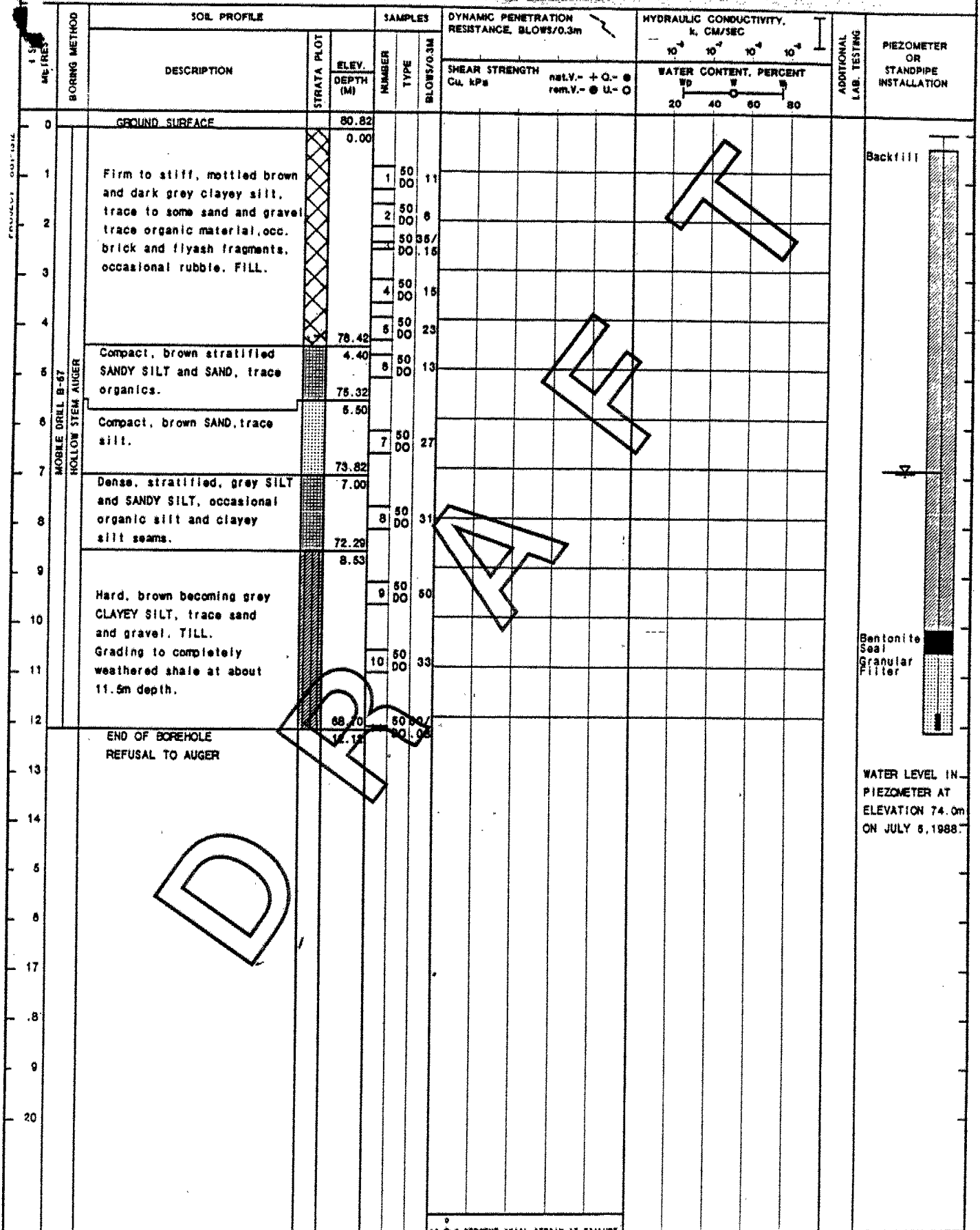
LOCATION SEE FIGURE 2

BORING DATE MAY 10/11, 1988

DATUM GEODETIC

SAMPLER HAMMER, 63.5kg, DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg, DROP, 760mm



DEPTH SCALE

1 : 100

Golder Associates

LOGGED J.W.

CHECKED A.S.P.

RECORD OF BOREHOLE 3

SHEET 1 of 2

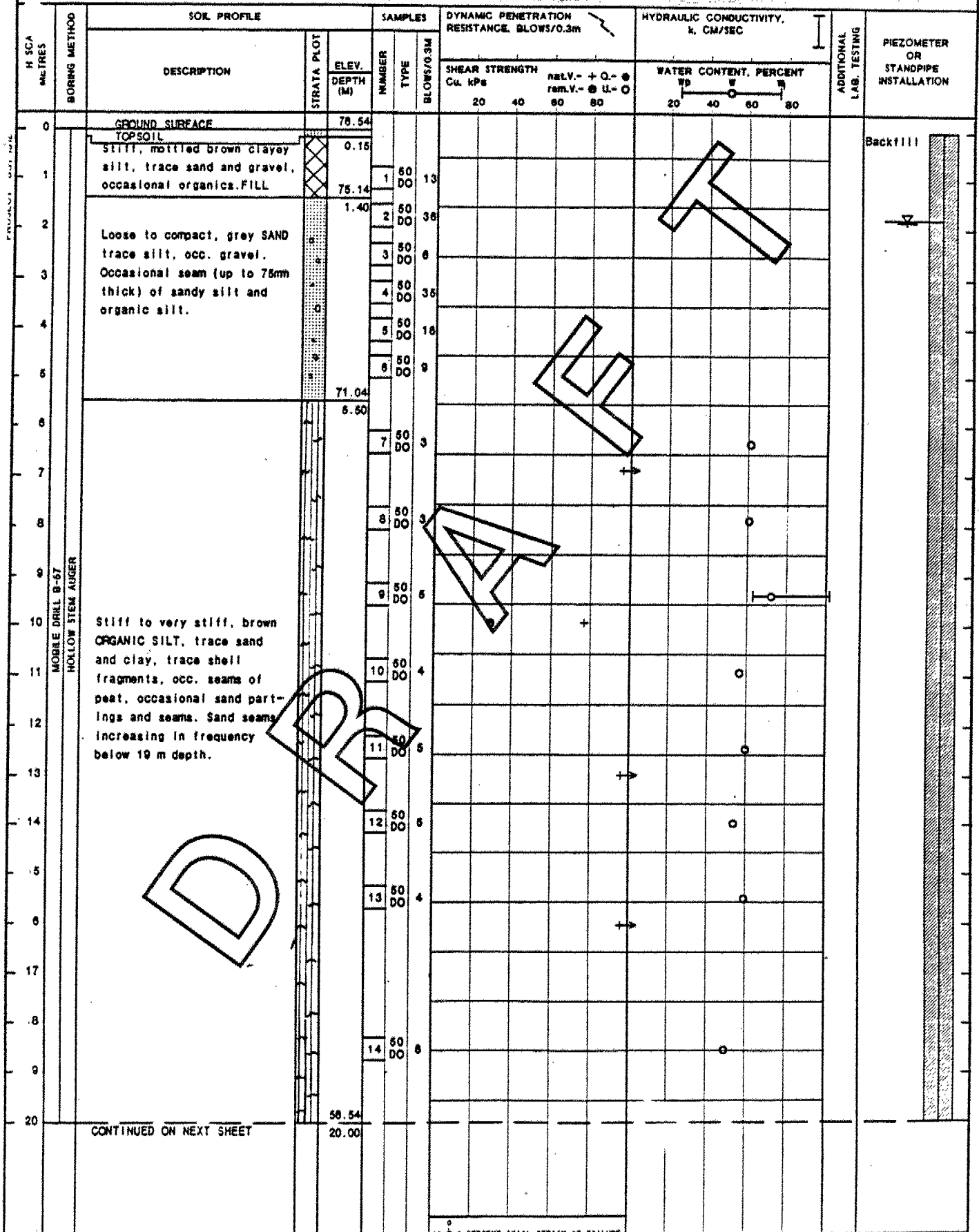
LOCATION: SEE FIGURE 2

BORING DATE MAY 11-13, 1988

DATUM: GEODETIC

SAMPLER: HAMMER, 83.5kg, DROP, 780mm

PENETRATION TEST: HAMMER, 83.5kg, DROP, 780mm



DEPTH SCALE

1 : 100

Golder Associates

LOGGED J.W.

CHECKED A.S.P.

RECORD OF BOREHOLE - 3

SHEET 2 of 2

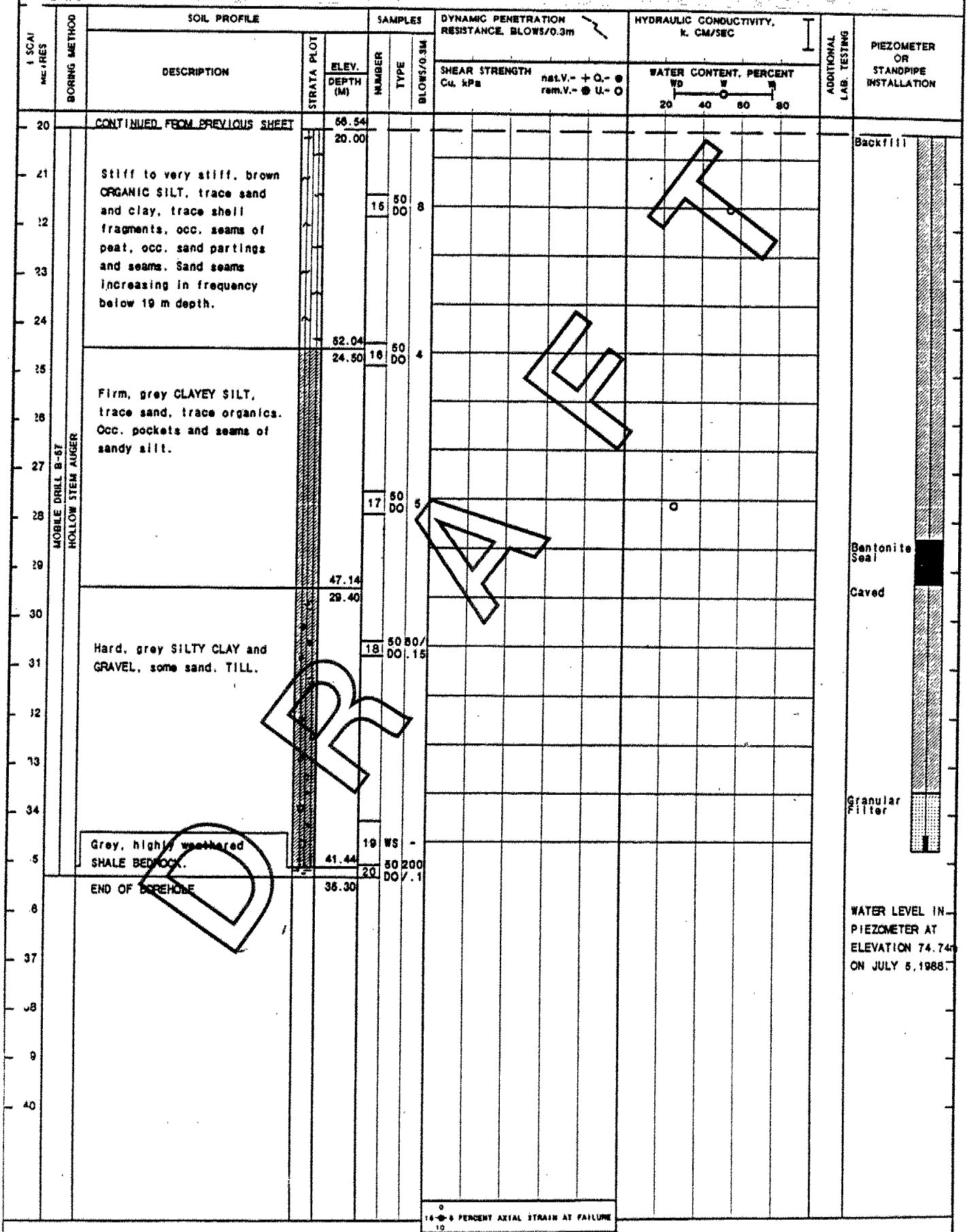
LOCATION SEE FIGURE 2

BORING DATE May 11-13, 1988

DATUM GEODETIC

SAMPLER HAMMER, 83.5kg, DROP, 760mm

PENETRATION TEST HAMMER, 83.5kg, DROP, 760mm



DEPTH SCALE

1: 100

Golder Associates

LOGGED J.W.

CHECKED A.S.P.

RECORD OF BOREHOLE 4

SHEET 1 OF 1

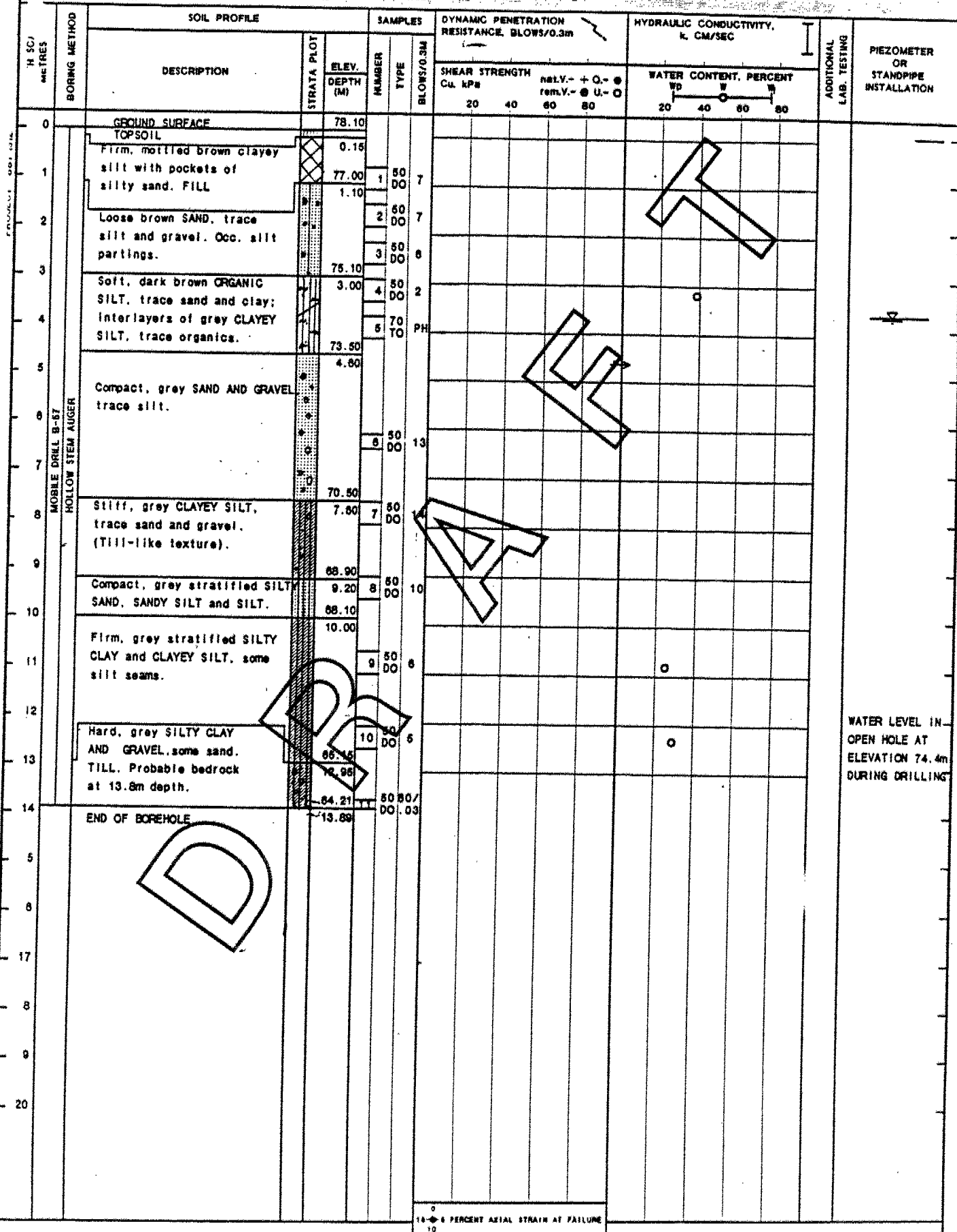
LOCATION SEE FIGURE 2

BORING DATE MAY 17, 1988

DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg, DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg, DROP, 760mm



DEPTH SCALE

1 : 100

Golder Associates

LOGGED J.W.

CHECKED A.S.P.

RECORD OF BOREHOLE 6

SHEET 1 of 2

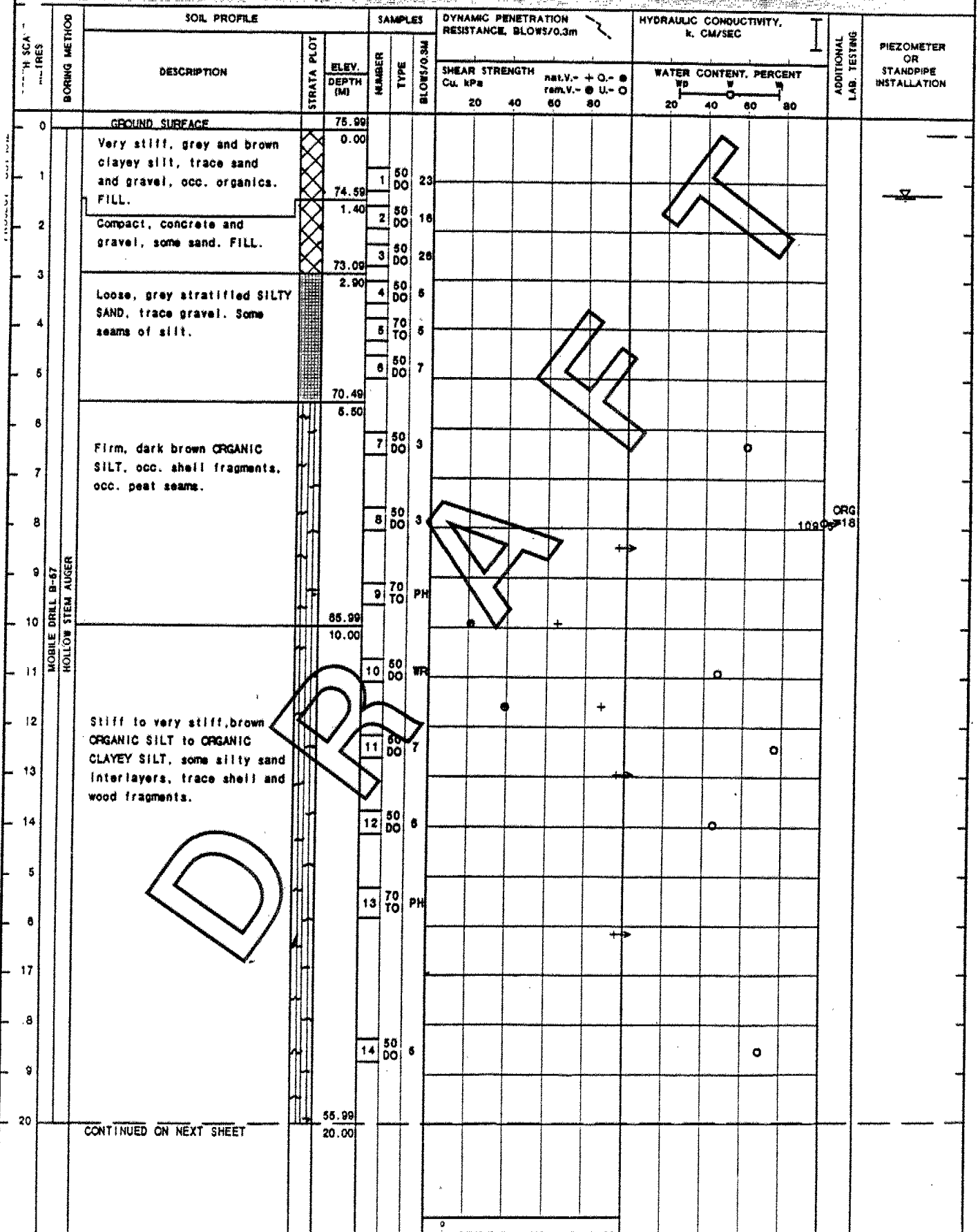
LOCATION SEE FIGURE 2

BORING DATE MAY 18-19, 1968

DATUM GEODETIC

SAMPLER HAMMER, 63.5kg, DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg, DROP, 760mm



DEPTH SCALE

1: 100

Golder Associates

LOGGED J.W.

CHECKED A.C.D.

RECORD OF BOREHOLE 6

LOCATION SEE FIGURE 2

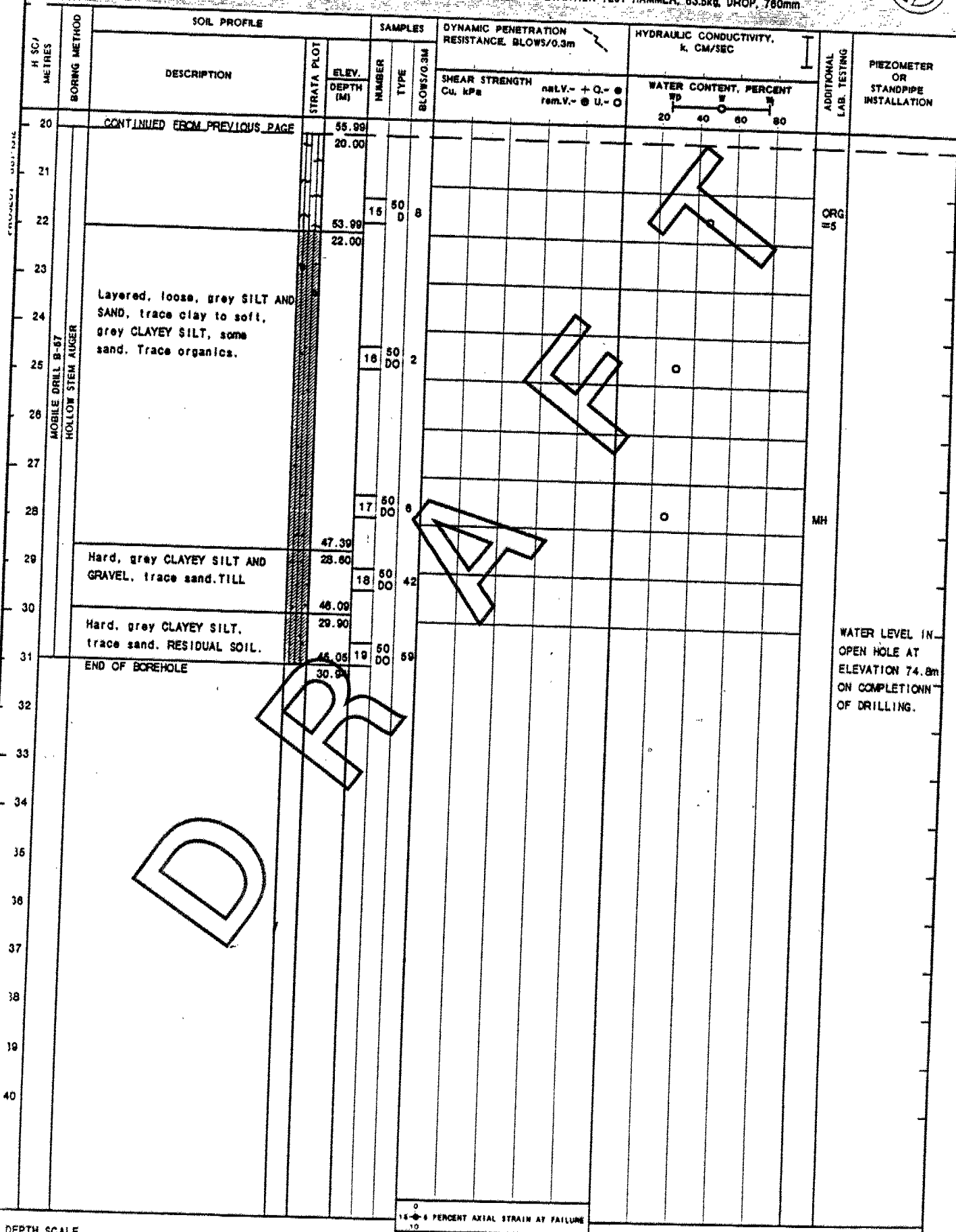
BORING DATE MAY 18-19, 1988

SHEET 2 of 2

SAMPLER HAMMER, 63.5kg, DROP, 760mm

DATUM GEODETIC

PENETRATION TEST HAMMER, 63.5kg, DROP, 760mm



DEPTH SCALE

1 : 100

Golder Associates

RECORD OF BOREHOLE 7

SHEET 1 of 2

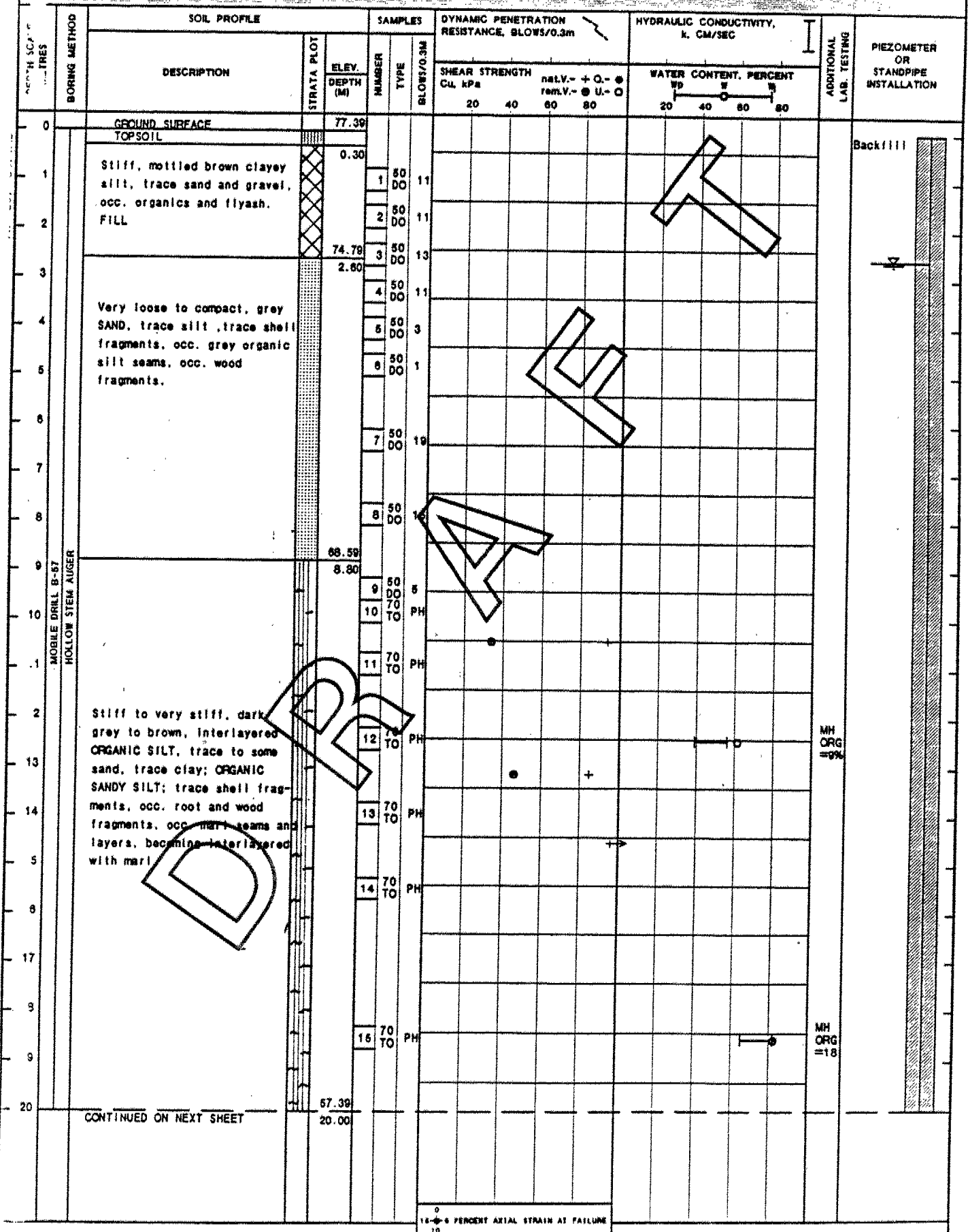
LOCATION SEE FIGURE 2

BORING DATE JUNE 16-17, 1988

DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg, DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg, DROP, 760mm



DEPTH SCALE

LOGGED J.W.

Golder Associates

RECORD OF BOREHOLE 7

SHEET 2 of 2

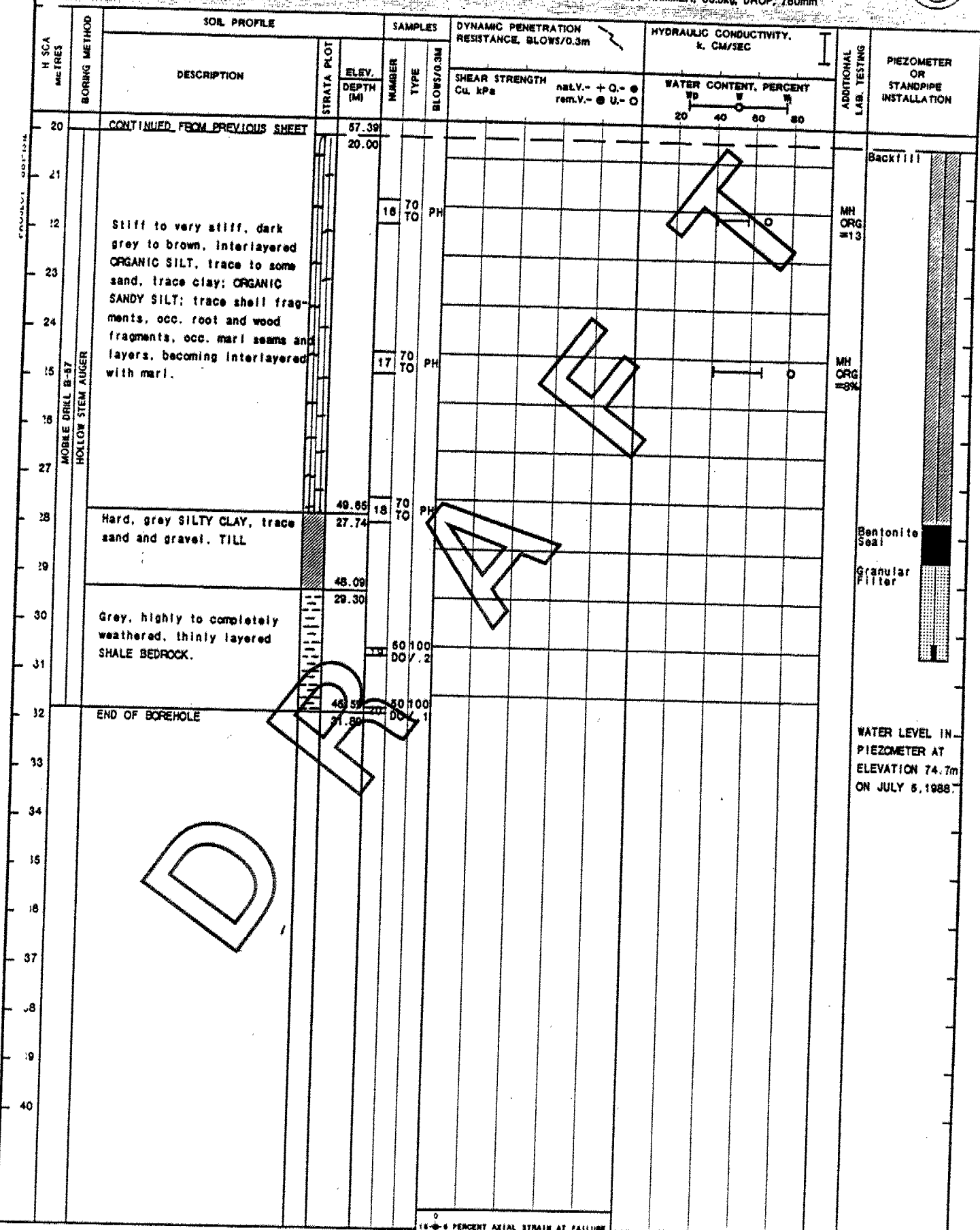
LOCATION SEE FIGURE 2

BORING DATE JUNE 16-17, 1988

DATUM GEODETIC

SAMPLER HAMMER, 63.5kg, DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg, DROP, 760mm



DEPTH SCALE

1 : 100

Golder Associates

LOGGED J.W.

CHECKED A.S.P.

RECORD OF BOREHOLE 9

SHEET 1 of 1

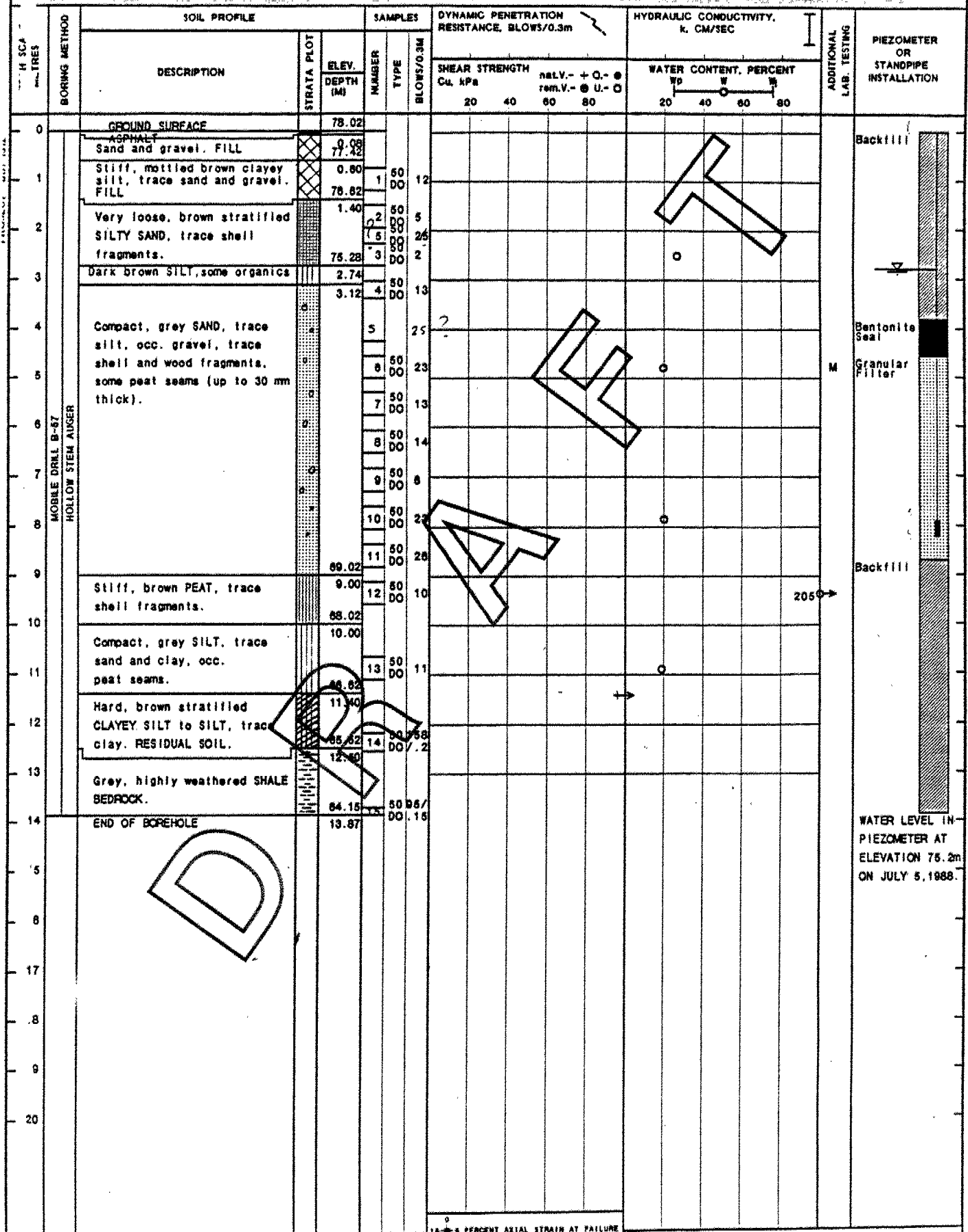
LOCATION SEE FIGURE 2

BORING DATE JUNE 21-22, 1988

DATUM GEODETIC

SAMPLES HAMMER, 63.5kg, DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg, DROP, 760mm



Doord Co.

REPLY
COPY

Structural Approvals

714 Floor Atrium

SEND
TO

DEPT.

DATE

SUBJECT

Foundation Design Dept.

June 11/92

Humber Bridge Project

As discussed in our telephone conversation of June 11th, we met with Golder and Dorman on June 5th. I have attached 2 pieces of correspondence pertinent to our at that time.

In our opinion the present proposal for pilot abutments, caissons at pier and piles approach is feasible. Outstanding issues are 1) local capacity of caissons under severe conditions and 2) cost effectiveness of pier approach study.

REPLY

We have suggested that the experience of the various engineering industry should be explored regarding underpinning caisson installation and inspection.

If there are any questions, please call

REPLY FROM

REPLY DATE

D. Doord, P.E.
Sr. Foundation Engineer

SENT BY:

6- 5-92 ; 9:46 ; GOLDER ASSOCIATES -416 235 5240

:# 2/ 9

Golder Associates Ltd.

2180 Meadowvale Boulevard
Mississauga, Ontario, Canada L5N 5R3
Telephone (416) 567 4444
Fax (416) 567 6561



June 4, 1992

911-1315

Ministry of Transportation
Foundation Design Section
Room 313, Central Building
1201 Wilson Avenue
Downsview, Ontario
M3M 1J8

ATTENTION: Mr. M. Devata, Chief Foundation Engineer, MTO

**RE: SUMMARY OF BRIDGE FOUNDATION ALTERNATIVES
HUMBER BRIDGES PROJECT
TORONTO, ONTARIO**

Dear Sirs:

Further to your correspondence dated May 29, 1992, we provide the following information regarding the foundation design alternatives for the proposed bridges involved with the Humber Bridges Project and the alternatives considered for the east approach embankment to the south bridge. We have attached copies of our letters sent to Delcan Corporation dated May 14, 1992 (addressing the use of caissons for the bridge piers adjacent to the river) and May 6, 1992 (addressing the east approach embankment alternatives for the south bridge - Bridge 1).

We trust that this summary package in conjunction with the information contained in our draft report will be sufficient for purposes of discussion at our meeting of June 5, 1992.

Foundations

Some background information is provided in our letter dated May 14, 1992. In summary, caissons are currently proposed for the support of the bridge piers which are immediately adjacent to the Humber River. The caissons should be designed with a permanent liner which would extend to the bedrock surface. The caisson excavation would then be extended by augering into the bedrock to form the bedrock socket below the liner.

The proposed design loads for various socket diameters and socketted length are provided in the table on page 3 of the letter. The design loads are based on shaft resistance within the bedrock below a depth of 1 m below the bedrock surface. A value of 800 kPa has been assumed for the rock-concrete adhesion if the capacity can be confirmed by a load test; the value is reduced to 600 kPa without a load test.

The lower value is given on the assumption that inspection of the bedrock socket will not be possible. This assumption has been based on discussions with contractors involved with the

SENT BY:

6- 5-92 ; 9:47 ; GOLDER ASSOCIATES -416 235 5240

:# 3/ 9

Ministry of Transportation
Mr. M. Devata, P. Eng.

-2-

June 4, 1992
911-1315

Palace Pier construction on the west side of the Humber River immediately south of the site. We have been informed that the first caisson installed at that site was essentially used as a test caisson to assess the feasibility of dewatering the caissons for inspection. The depth of the caissons and the amount of water inflow which occurred was such that dewatering and inspection were not possible.

Driven steel H-piles are proposed for the support of the remainder of the bridge piers and abutments at the site. As discussed in our draft report, the design capacities for steel H-piles driven to practical refusal on the shale bedrock is 1600 kN at ULS and 1150 kN at SLS.

East Approach Embankment

The geotechnical considerations associated with the alternatives which have been discussed for the east approach to Bridge 1 are summarized in the attached letter dated May 6, 1992. In summary, a Consumers Gas main extends along the edge of the proposed approach embankment and there is concern for the consequences of the settlement due to compression of the underlying organic silts. Subexcavation of the insitu soils and constructing the embankment out of light weight fill can reduce the long term settlement to about 3 cm. This approach, however, requires excavations up to 5 m deep and requires a relatively stiff pretensioned shoring system for support of the excavation adjacent to the gas main. A structural and cost comparison of this subexcavation/light weight fill method and the alternative of the pile supported slab approach has been carried out by Delcan Corporation.

Yours truly,

GOLDER ASSOCIATES LTD.



A. S. Poschmann, P. Eng.
ASP/pds

Att: as noted above

cc: Delcan Corporation
Attention: Mr. W.V. Anderson, P.Eng.

Golder Associates

SENT BY:

6- 5-92 ; 9:48 ; GOLDER ASSOCIATES -416 235 5240

4/ 9

Golder Associates Ltd.

2180 Meadowvale Boulevard
Mississauga, Ontario, Canada L5N 5S3
Telephone (416) 567-4444
Fax (416) 567-6561



May 14, 1992

911-1315

Delcan Corporation
133 Wynford Drive
North York, Ontario
M3C 1K1

ATTENTION: Mr. W. V. Anderson, P. Eng.

**RE: SUMMARY OF BRIDGE FOUNDATION ALTERNATIVES
HUMBER BRIDGES PROJECT
TORONTO, ONTARIO**

Dear Sirs:

Further to our recent Technical Subcommittee meetings of April 14, April 28, and May 12, 1992, this letter provides a summary of these discussions and our recommendations related to foundation design for the above project.

Background

Three feasible foundation options for the Bridge Structures were discussed in our draft report number 911-1315, dated February 1992. The options addressed are:

- Driven Steel H-Piles
- Driven Pipe Piles
- Cast In Situ Concrete Piles

Based on the site conditions and on construction considerations, driven steel H-Piles were recommended for the project.

Since issuing the report, preliminary scour depth calculations have been carried out and suggest that scour of the order of 20 m deep may occur in the Humber River channel under the Regional Storm event. Bank support, such as anchored sheet piling, designed to withstand such a scour depth is understood to be impractical for this project. Therefore, consideration is being given to designing the foundations of bridge piers adjacent to the Humber River as free standing columns. We understand that such columns will require a diameter of 0.9 m to 1.2 m. This precludes the use of driven pipe piles and leaves cast in situ concrete piles as the currently preferred foundation for bridge piers adjacent to the river.

SENT BY:

6- 5-92 ; 9:48 ; GOLDER ASSOCIATES -416 235 5240

;/ 5/ 9

Delcan Corporation
Mr. W.V. Anderson, P. Eng.

-2-

May 14, 1992
911-1315

Casts In Situ Concrete Piles

The recommendations for cast in situ concrete piles provided in our February 1992 report assumed:

- Design based on shaft friction (using a concrete to bedrock adhesion of 800 kPa) alone due to concerns for adequate cleaning of the caisson base at greater than 30 m depth in fractured rock.
- The sidewalls of the rock socket would be cleaned using a water jet.
- The design load carrying capacity and the contractor's installation method would be confirmed with a load test.

The possibility of carrying out a test caisson excavation at the site to examine the bedrock and to assess potential construction difficulties was discussed in our April 1992 meetings and cost estimates were provided in our May 6, 1992 letter. It has been concluded during our discussions with contractors and at the meetings that dewatering a test caisson to allow rock inspection is not feasible at the site and that the technology and equipment necessary to form load carrying rock sockets in the Toronto area Dundas Shale is available and has been proven. Hence, a test caisson excavation by itself is not considered to provide significant additional information for the design of the project.

A fully installed caisson, and a load test would provide useful data for the project design, but would cost about \$150,000. Such a test could confirm the recommended 800 kPa concrete to rock adhesion strength and could possibly justify a higher design value. We understand, however, that regardless of the axial capacity of the sockets, a configuration of 6 piles per bridge pier will be necessary to form the free standing foundation structure and resist the applied horizontal loads. Therefore, while the provision of a test caisson and the carrying out of a load test may increase the allowable socket design stress and decrease the required socket depth, it will not decrease the total number of deep foundation units required for the project.

The following table provides a summary of the design load carrying capacity of rock sockets at the site. The design values provided assume that the upper 1 m of shale does not contribute to the axial capacity and that no contribution to the resistance is provided by end-bearing.

SENT BY:

6- 5-92 ; 9:49 ; GOLDER ASSOCIATES -416 235 5240

;/# 6/ 9

Delcan Corporation
Mr. W.V. Anderson, P. Eng.

-3-

May 14, 1992
911-1315

LOAD CARRYING CAPACITIES OF ROCK SOCKETS HUMBER RIVER BRIDGES PROJECT

Depth of Socket Below Top of Bedrock (m)	SOCKET CAPACITY AT ULTIMATE LIMIT STATES (kN)			
	0.9 DIAMETER SOCKET		1.2 m DIAMETER SOCKET	
	Capacity Confirmed by Load Tests	No Load Tests	Capacity Confirmed by Load Tests	No Load Tests
2 m	2250	1700	3000	2250
3 m	4500	3400	6000	4500
4 m	6800	5100	9500	6800

As has been discussed in our report and at our meetings, the cast in situ concrete piles should be designed with a permanent steel liner to avoid "necking" of the piles during construction. The liner may either be a heavy duty steel liner which would be used to advance the caisson holes and which would be left in place, or it may be a thin steel shell installed within the heavy duty liner after excavation.


Summary

The recommended foundation alternative for the Humber River Bridge Project is driven steel H-Piles for abutments and piers located away from the River. The preliminary scour calculations suggest that deep scour of the river will occur under the regional storm event and therefore, cast in situ concrete piles socketed into bedrock are the currently preferred foundation alternative for bridge piers adjacent to the River. Contractors bidding on the project may be given the option of using cast in situ concrete piles in place of driven steel H-piles at abutment and piers away from the river if this will reduce overall construction costs.

We trust that this letter meets with your current requirements. If you have any questions or comments, please do not hesitate to contact us.

Yours truly,

GOLDER ASSOCIATES LTD.


J. Westland, P. Eng.


A. S. Poschmann, P. Eng.

JW/ASP/pds

Golder Associates

SENT BY:

6- 5-92 ; 9:51 ; GOLDER ASSOCIATES -416 235 5240

7/ 9
u

Golder Associates Ltd.

2180 Meadowvale Boulevard
Mississauga, Ontario, Canada L5N 3J3
Telephone (416) 567-4444
Fax (416) 567-6561



May 6, 1992

911-1315

Delcan Corporation
133 Wynford Drive
North York, Ontario
M3C 1K1

ATTENTION: Mr. W. V. Anderson, P. Eng.

RE: REVIEW OF LIGHT-WEIGHT FILL EMBANKMENT AND
TEST CAISSON EXCAVATION
HUMBER RIVER BRIDGES PROJECT
TORONTO, ONTARIO

Dear Sirs:

Further to the Technical Subcommittee (Structures) meeting of April 14, 1992; this letter provides our review of the geotechnical issues raised during this meeting. Specifically, we undertook to review the design of a light-weight fill embankment for the east abutment of Bridge 1 and to obtain quotations for the construction of a test caisson excavation at the site.

East Abutment to Bridge 1

The east abutment of Bridge 1 will be up to 2.7 m above the existing grade. The subsurface soils at this location include about 18 m of highly compressible organic silt. The impact of settlement of this deposit on the abutment performance has been recognized as a major design constraint since the start of the project. A number alternatives to minimize post construction settlement have been considered and are discussed in our draft report no. 911-1315, dated February, 1992. The alternatives discussed include:

- Pre-loaded Embankment
- Surcharged Embankment
- Electro-Osmotic Treatment
- Light-Weight Fill
- Pile Supported Grade Slab

The light-weight fill alternatives discussed in the draft report included the use of foamed polystyrene which is relatively expensive and requires protection against fuel spills and the use of flyash which requires registration as a non-hazardous waste. At the subcommittee meeting the use of light-weight blast furnace slag as an embankment fill was suggested and an assessment of the use of this material on embankment performance was subsequently carried out.

Settlement calculations were carried out for two design cases; one with slag placed from ground surface and the other with existing fill removed to elevation 75 m (water table elevation) and replaced by slag which would than be built up to the underside of the pavement structure. For

SENT BY:

6- 5-92 ; 9:52 ; GOLDER ASSOCIATES -416 235 5240

8/ 9

Delcan Corporation
Mr. K. D. Price, P. Eng.

-2-

May 6, 1992
911-1315

calculation purposes, the pavement structure was assumed to be 1 m thick. The calculated settlements for the two cases are summarized in the following table.

**SETTLEMENT CALCULATION RESULTS
EAST ABUTMENT BRIDGE 1
LIGHT-WEIGHT BLAST FURNACE SLAG**

Case	Description	Primary Consolidation (mm)	Secondary Compression (mm)	Total Settlement (mm)
(i)	Slag placed from existing grade.	50 - 70	40 - 50	90 - 120
(ii)	Existing fill excavated to Elev. 75 m and replaced by slag.	-25	-5	-30

Because the embankment is only about 2.7 m above existing grade, much of the embankment will consist of pavement structure and relatively little blast furnace slag can be incorporated into the design Case (i) and, as a result, the total calculated settlement in this case is close to that calculated for an embankment formed of conventional soil fill. Case (ii), involving excavation and replacement of existing fill soils, provides a significant reduction in calculated total settlement. It should be pointed out, however, that the excavation of existing fill soils under the east abutment of Bridge 1 will require a cut of 4 m to 5 m depth immediately adjacent to eastbound Lakeshore Blvd. Such an excavation would have to be shored and in view of the Consumers Gas main which runs near the south curb of Lakeshore Blvd., a relatively stiff pretensioned shoring system would have to be used. However, regardless of the shoring system used, some lateral deformation and settlement of the gas main would result from the excavation.

Test Caisson

It is understood that the depth of scour of the Humber River bed under the design storm conditions necessitates that the bridge piers act as free standing columns over a substantial depth. For this reason, large diameter (up to 1.2 m) caissons are being considered for the bridge piers. Socketed piers were discussed as a foundation alternative in Golder Associates draft report. In our report the ultimate design capacities for rock socketed caissons were provided under the following assumptions.

- Shaft friction only would support the load because cleaning the caisson base at greater than 30 m depth in fractured rock may not be feasible.
- The sidewalls of the rock socket would be cleaned using a water jet.
- The design load carrying capacity and the contractor's installation method would be confirmed with a load test.

Golder Associates

SENT BY:

6- 5-92 ; 9:53 ; GOLDER ASSOCIATES -416 235 5240

:# 9/ 9

Delcan Corporation
Mr. K. D. Price, P. Eng.

-3-

May 6, 1992
911-1315

During the sub-committee meeting it was agreed that a load test for such high capacity caissons would be quite expensive and it was suggested that a test caisson excavation at the design stage might allow inspection of the bedrock and would also provide insight into potential construction difficulties for the large diameter caissons.

Golder Associates undertook to obtain cost estimates for the mobilization of equipment to carry out a test excavation. Since at this time the exact locations of caissons are not known, the cost estimate was for excavation of the caissons with backfill by auger cuttings only. Four contractors were requested to provide a cost estimate and two formally replied. The cost estimates are as follows:

Deep Foundations Contractors Inc.	\$29,970.00
Franki Canada Limited	\$43,500.00


In discussing the project with the contractors, their experience with rock-socketed caissons at nearby sites was noted. There was general agreement that dewatering the caissons to allow inspection of the rock socket was not likely possible. It was also clear, however, that the construction of large diameter caissons with 4 m to 5 m deep rock sockets is feasible at the site. In view of this information and the cost of the proposed test caisson we do not believe that significant information affecting the design of the bridge foundations will be gained from the test excavation alone.

Further design data would be gained by carrying out the test excavation, forming a permanent reinforced concrete caisson and carrying out a load test. It is estimated, however, that such a program would cost about \$150,000. The cost of a load test during the construction contract would be substantially less.

We trust that this information is sufficient for your current requirements and look forward to continuing to work with you on this project.

Yours truly,

GOLDER ASSOCIATES LTD.


J. Westland, P. Eng.


A. S. Poschmann, P. Eng.

JW/ASP/pds

Golder Associates

Removal of the existing bridges will include the demolition of the existing river bank piers to the water line and the integration of the remaining portion with the new channel bank. The three piers within the river will be cut back to above the high water level and left in place to avoid disturbance and resuspension of the potential hazardous sediment on the river bed.

The length of the proposed bridges runs from 170 m to 300 m. Each bridge will have piers located adjacent to the east and west river banks. There will not be any new piers located within the river. Some of the bridges will have additional piers located in the overbank area east of the river.

The hydraulic analysis for the existing bridges indicates that the return periods flood flows are contained within the river banks. As there will be no additional piers/obstructions in the river and the existing river bank will be maintained the hydraulic capacity and flood levels will remain unchanged.

The Regulatory storm is above the river bank elevation and utilizes the overbank area. As the overbank area will be wider than the existing condition, no change or a reduction in the Regulatory flood level is expected.

Soffit Elevation

Ministry of Transportation Directive B-100 requires the bridge to be designed for the 100 year flood. The calculated 100 year flood level ranges from 75.2 m downstream of the existing Lakeshore Boulevard bridge to 75.9 m downstream of the CNR bridge. The calculation assumes that the 100 year flood on the Humber River coincides with a level on Lake Ontario only 0.1 m below the maximum mean annual lake level. This elevation is only 0.6 m below the maximum daily elevation and is therefore a reasonable assumption. With a 1.0 m clearance, the minimum soffit elevation should be 76.2 m at Bridge 1 and 76.9 m at Bridge 6. The lowest soffit elevations are approximately 78.0 m at the west bank pier for Bridge 1 and 77.2 m at the east bank pier for Bridge 4. Therefore the proposed bridges provide sufficient clearance above the design high water level.

The Regulatory flood level at this location ranges from 76.1 m downstream of Lakeshore Boulevard bridge to 77.6 m downstream of the CNR bridge. As noted above there are no new piers to be located within the main channel and the overbank area will be increased with the proposed bridges. Therefore the proposed bridges should not have any adverse impact on the Regulatory flood level. In addition, there is a head loss of 0.8 m through the CNR bridge which is the control section for upstream flood levels. Any portions of the proposed bridges below 77.6 m should be designed to resist movement due to buoyancy or hydrodynamic forces.

Scour

Due to the existing piers within the channel, the channel section through the existing Lakeshore Boulevard bridge has been used to estimate general and local scour. A cross-section of the channel perpendicular to the flow is attached as Figure 1.

General Scour

Under the 100 year design flood, the channel velocity will be approximately 3.7 m/s. The existing sediment in the river bed ranges from a clayey silt with some sand to a gravelly sand with trace silt. The competent velocity of this material at a flow depth of 3 m is in the order of 0.6 m/s and there would be general scour throughout the channel. At the likely scour depth the boreholes on the east bank indicate bed material to be organic silt to organic clayey silt while on the west bank the material ranges from sand and gravel to sand. Using the cohesionless sand as the worst case ($D_{60} = 0.5$ mm), the competent velocity would be 0.9 m/s at a flow depth of approximately 11 m. The average general scour would put the river bed at elevation 64.5 m as shown on the attached figure.

For the Regional Storm, the channel velocity is 6.1 m/s. A flow depth of 20 m would be required to reduce the channel velocity to the competent velocity of 1.3 m/s at this depth. The average general scour elevation would be 57.5 m.

The above general scour calculations assume that the existing piers adjacent to the river bank are incorporated into the river bank and provide a vertical channel bank to below the scour level.

Local Scour

Local scour will occur at the existing piers located within the channel. The piers adjacent to the river bank will be part of the future channel bank and will not be subject to future local scour. While protection of the existing channel piers is not of concern, the local scour may affect the scour depth at the channel bank.

The existing piers are 2.75 m in diameter however, the pile cap/footing is located just below the river bottom and would be exposed by the general scour. The pile cap is 4.6 m x 4.6 m square and the calculated local scour would be 9.1 m deep. The top of the scour hole would extend approximately 17 m beyond the outside of the footing.

The local scour has been added to the general scour on Figure 1 and would result in an increase in the scour depth at the west bank by about 2.5 m.

The proposed piers would also be subject to local scour under the Regulatory event as the flow overtops the channel banks. The proposed piers have an oval shape with a width of approximately 2.5 m. The local scour for these piers is approximately 3.8 m. This depth of scour would expose the pile cap and local scour could increase to 12 m deep. A rip rap apron extending 4 m beyond the pier could be used to minimize the impact of local scour in the overbank areas.

Summary

The return period flows are contained within the channel while the Regulatory flood would overtop the channel.

General scour has been calculated assuming vertical channel banks at the location of the existing piers adjacent to the banks of the river. The general scour ranges from 9 m to 15 m below the existing channel bed for the 100 year and Regulatory floods respectively. Local scour due to the existing piers would increase the scour depth at the west bank by 2.5 m.

The new piers and footings/piles will need to be designed to accept exposure of the piles down to elevation 55 m during the Regulatory flood. Alternatively, bank protection using sheet steel piling driven below elevation 55 m would be required to protect the channel bank from scour and erosion and to maintain the integrity of the river bank piers.

With in the overbank area, a rip rap apron extending 4 m beyond the pier is recommended to minimize local scour.

LAKE ONTARIO AT TORONTO (STATION NO. 02HC048)

1906 TO 1987

	Water Level Elevations	
	IGLD	*Metro Geodetic
Maximum Instantaneous	75.69m	75.89m
Maximum Daily	75.63m	75.83m
Maximum Mean Monthly	75.61m	75.81m
Maximum Mean Annual	75.11m	75.31m
Minimum Daily	73.52m	73.72m
Minimum Mean Monthly	73.61m	73.81m
Minimum Mean Annual	73.86m	74.06m

* Use Metro Geodetic for this project.

HUMBER RIVER FLOOD LEVELS

Event	Discharge m ³ /s	Velocity			Water Level Elevation		
		West	Channel	East	SEC 50.010 m	SEC 50.021 m	SEC 50.025 m
5 year	224	-	1.7	-	75.2	75.2	75.2
10 year	292	-	2.2	-	75.2	75.2	75.3
25 year	388	-	2.8	-	75.2	75.2	75.4
50 year	469	-	3.3	-	75.2	75.2	75.6
100 year	553	-	3.7	-	75.2	75.2	75.9
Regulatory	1580	1.1	6.1	1.2	76.1	77.0	77.6

Notes:

Data provided by MTRCA

SEC 50.010 - at Lake Ontario

SEC 50.021 - downstream face of existing Lakeshore Bridge

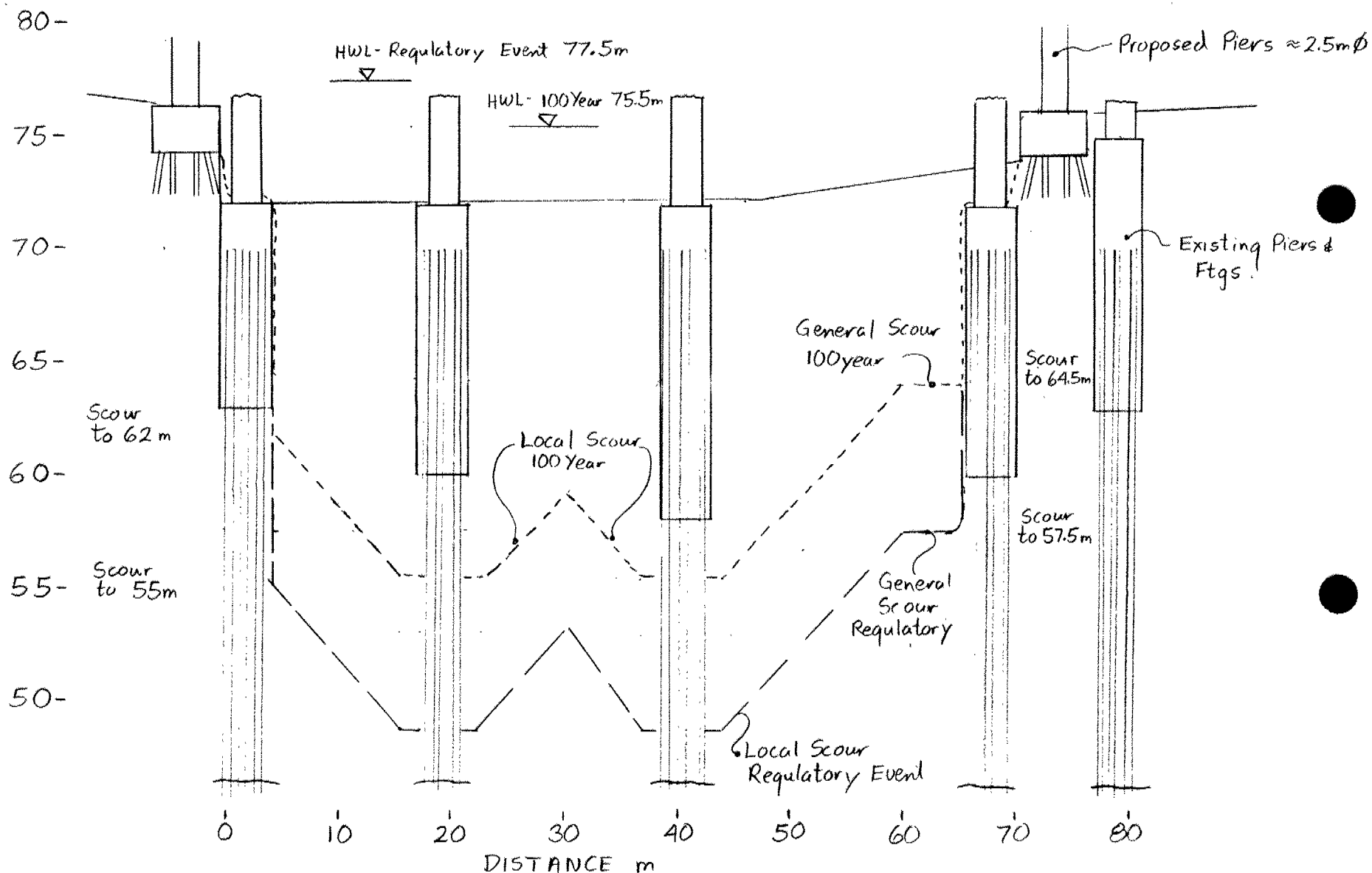
SEC 50.025 - downstream face of existing CNR bridge

Velocity - Typical velocity at Sec. 50.023 located between the existing Gardiner Expressway and Lakeshore Boulevard bridges.

West - West overbank

East - East overbank

HUMBER BRIDGES
 SECTION PERPENDICULAR TO FLOW
 LOOKING UPSTREAM
 APPROX. BRIDGES 3-4



GENERAL AND LOCAL SCOUR
 FIGURE 1

HUMBER BRIDGES GENERAL SCOUR

Typical Section thru existing piers - Lakeshore Blvd Bridge

Top width 73m

Side Slope - Vertical

Obstructions - 4@ 2.75m = 11m to elev 72 ± (open width = 62m)
 - 4@ 4.57m = 18.3m to elev 60 ± (open width = 54.7m)

Event	Flood Elev m	Channel			Total Depth m	Area m ²	Vel m/s	D ₅₀ mm	Competent Vel m/s	Bed Elev. m
		Q m ³ /s	Vel m/s	Area m ²						
100	75.5	553	3.7	150	10	572	0.97	0.5	0.82	
					11	626	0.88	0.5	0.85	64.5
									<u>Okay</u>	
Req.	77.5	1485	6.1	259	20	1134	1.3	0.5	1.2	57.5
									<u>Okay</u>	

Local Scour - Proposed Piers - 2.5 m ϕ

$$dsp = C_L C_s w = 1.5(1.0)(2.5) = 3.75m$$

- this would expose pile cap 6m wide x 8m long - rectangular

$$dsp = (2.0)(1.0)(6) = 12m$$

- however pile cap only 2m deep \therefore local scour some where between 4 and 12 m

Local Scour - Existing Piers. $9'\phi = 2.75\text{m}$.

$$dsp = 1.5(1.0)(2.75) = 4.1\text{m}$$

- however pile cap/ftg would be exposed by general scour

cap = 15' x 15' square x 40' deep; 4.57m x 4.57m x 12.2m.

$$dsp = 2.0(1.0)(4.57) = 9.1\text{m}$$

- top width of scour hole

- assume angle of repose of 30° for the bed material

- assume bottom width is 1.5m beyond the ftg.

$$\begin{aligned} \text{Top width} &= (\tan(90 - \phi) \times \text{local scour depth} + 1.5)2 + \text{ftg} \\ &= 39\text{m} \end{aligned}$$

$$\text{Bottom} = 1.5 \times 2 + \text{ftg} =$$

To: A. A. Witecki, P. Eng.
Municipal Engineer
Approvals Section
Structural Office

Date: 1992 03 17

Attention: D. Lai

From: Foundation Design Section
Room 315, Central Bldg.

Re: Metro Toronto
Bridge #1 (Gardiner Expressway and Lakeshore Blvd.) over Humber River
and
Ramp South Kingsway to EB Gardiner Expressway
WO 92-11004, Site #37-248
District 6, Toronto

As requested, we have reviewed the draft report from Golder Assoc. Ltd. dated February 1992. In our opinion this is a complex project and it would be prudent for senior staff from both of our offices to meet with Delcan and Golder to discuss alternatives. We suggest that we could streamline the process if we were to contact Golder directly so that we can ensure appropriate representation from their office and proper site plans and stratigraphy drawings for the meeting.

The following comments are generalized and are intended for internal MTO purposes only and should not be distributed to the consultant. Specifics can be addressed at the proposed meeting.

- 1) In our opinion the report is comprehensive.
- 2) However, there is not enough information in this report to base our comments on our own interpretation of subsurface conditions. Therefore these comments are based on the assumption that Golder's assessment of subsurface conditions is accurate.
- 2) The major concerns at the site are
 - a) weak foundation soils
 - b) settlements at the approaches and ramps
 - c) the impact of settlement-induced lateral forces on deep foundations in view of their proximity of proposed embankments to existing structures
 - d) ensuring lateral capacity of foundation elements in view of the scour potential in the river valley
- 3) The Golder report provides a variety of solutions (settlement acceleration, embankment material, retaining structures), some of which are more realistic than others. We suggest that Golder should recommend preferred alternatives.
- 4) We concur that deep foundations are required.

- 5) We have less concern about the practicality of caissons than Golder and would recommend that the selection of type of deep foundation should be based on comparison of costs.
- 6) If piles are selected, we recommend that MTO specifications for pile driving should be followed. We are concerned that Golder's negative skin friction reduction in pile capacity may be excessive.
- 7) If caissons are selected, capacities will require careful review. MTO would not restrict calculation of caisson capacity to socket sidewall. Consequently socket depth would be considerably less and caisson capacity would be significantly higher (with shallow sockets). MTO would not require load tests as a prerequisite for acceptance of the caisson option.
- 8) Recommendations are required for dewatering and earth pressure calculations.
- 9) Recommendations regarding slope stability should be clarified and justified by analyses.
- 10) We require clarification of the proximity of Bridge #1 to existing structures in order to assess concerns about vibration and settlement induced lateral forces on deep foundations.
- 11) Consideration should be given to pile bent construction in order to eliminate dewatering concerns.
- 12) We have concerns regarding the effectiveness of electro-osmosis as a settlement-accelerator in organic soil.

If there are any questions, or if you require our participation in a meeting with Golder, please call.



D. Dundas, P. Eng.
Sr. Foundation Engineer

for

M. Devata, P. Eng.
Chief Foundation Engineer