

DOCUMENT MICROFILMING IDENTIFICATION

GEOCRES No. 316-199

DIST. 9 REGION

W.P. No. 128-87-07/08

CONT. No. 91-46

W. O. No.

STR. SITE No. 3-550

HWY. No. 416

LOCATION Hwy 416 & Jack River

No. of PAGES -

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OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.

REMARKS:

DIMENSIONS ARE IN MILLIMETRES
UNLESS OTHERWISE SHOWN.
ELEVATIONS, COORDINATES, CURVE
AND ALIGNMENT DATA ARE IN METRES.
STATIONS ARE IN KILOMETRES + METRES.

DIST. 9

CONT No

WP No 128-87-07/08

JOCK RIVER BRIDGES
STRUCTURE 22A - HWY 416 NBL
STRUCTURE 22B - HWY 416 SBL
GENERAL ARRANGEMENT

SHEET



Proctor & Redfern Limited
Consulting Engineer and Architect
Toronto Kingston

E.O. 89M01

GENERAL NOTES

1. CLASS OF CONCRETE
- ALL CONCRETE 30 Mpa
2. CLEAR COVER TO REINFORCING
- | | |
|-----------------------|----------|
| FOOTINGS | 100 ± 25 |
| ABUTMENTS & WINGWALLS | |
| FRONT FACE | 80 ± 20 |
| BACK FACE | 70 ± 20 |
| PIERS | 80 ± 20 |
| DECK | |
| TOP | 70 ± 20 |
| BOTTOM | 40 ± 10 |
| CAISSON | 80 ± 20 |
| REMAINDER | 70 ± 20 |
- UNLESS OTHERWISE SPECIFIED
3. REINFORCING STEEL SHALL BE GRADE 400 UNLESS OTHERWISE SPECIFIED. BAR MARKS WITH SUFFIX C DENOTE COATED BARS.
4. CONSTRUCTION NOTE
- IF THE ACTUAL BEARING HEIGHTS ARE DIFFERENT FROM THE ASSIGNED BEARING HEIGHTS GIVEN WITH THE BEARING DESIGN DATA, THE CONTRACTOR SHALL ADJUST THE BEARING SEAT ELEVATIONS AND THE REINFORCING STEEL TO SUIT THE ACTUAL HEIGHTS.

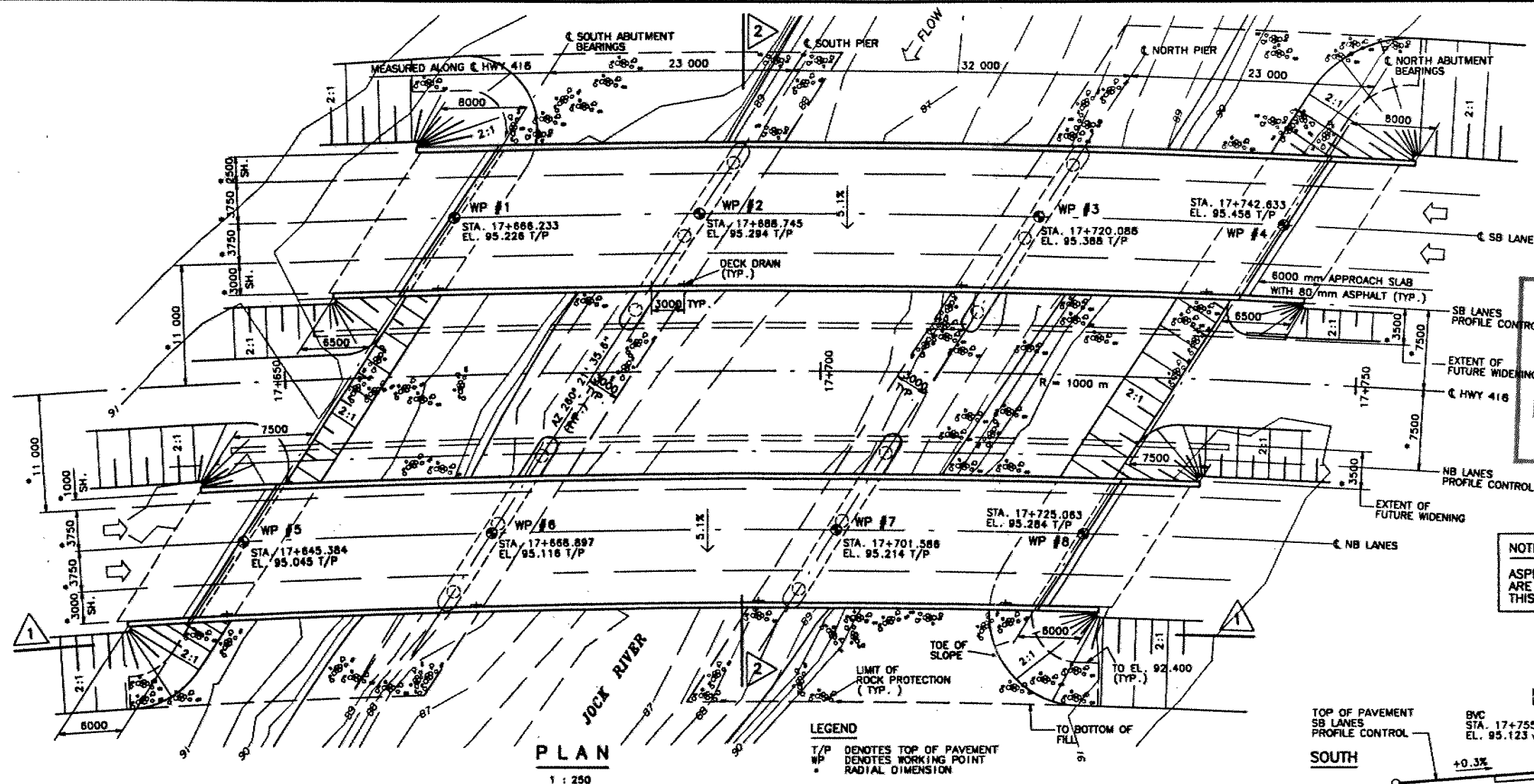
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6. NBL PIERS
7. NBL ABUTMENTS
8. NBL WINGWALLS
9. NBL STRUCTURAL STEEL I
10. NBL STRUCTURAL STEEL II
11. NBL DECK REINFORCING
12. NBL BARRIER WALLS
13. NBL 6000 mm APPROACH SLABS
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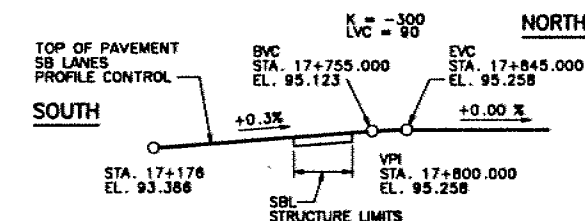
DD-3503 MINIMUM GRANULAR BACKFILL REQUIREMENTS

DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

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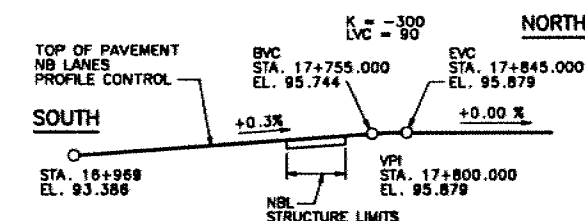
PRINTED
JAN 16 1991
PROCTOR & REDFERN
LIMITED

NOTE :
ASPHALT & WATERPROOFING
ARE NOT INCLUDED IN
THIS CONTRACT



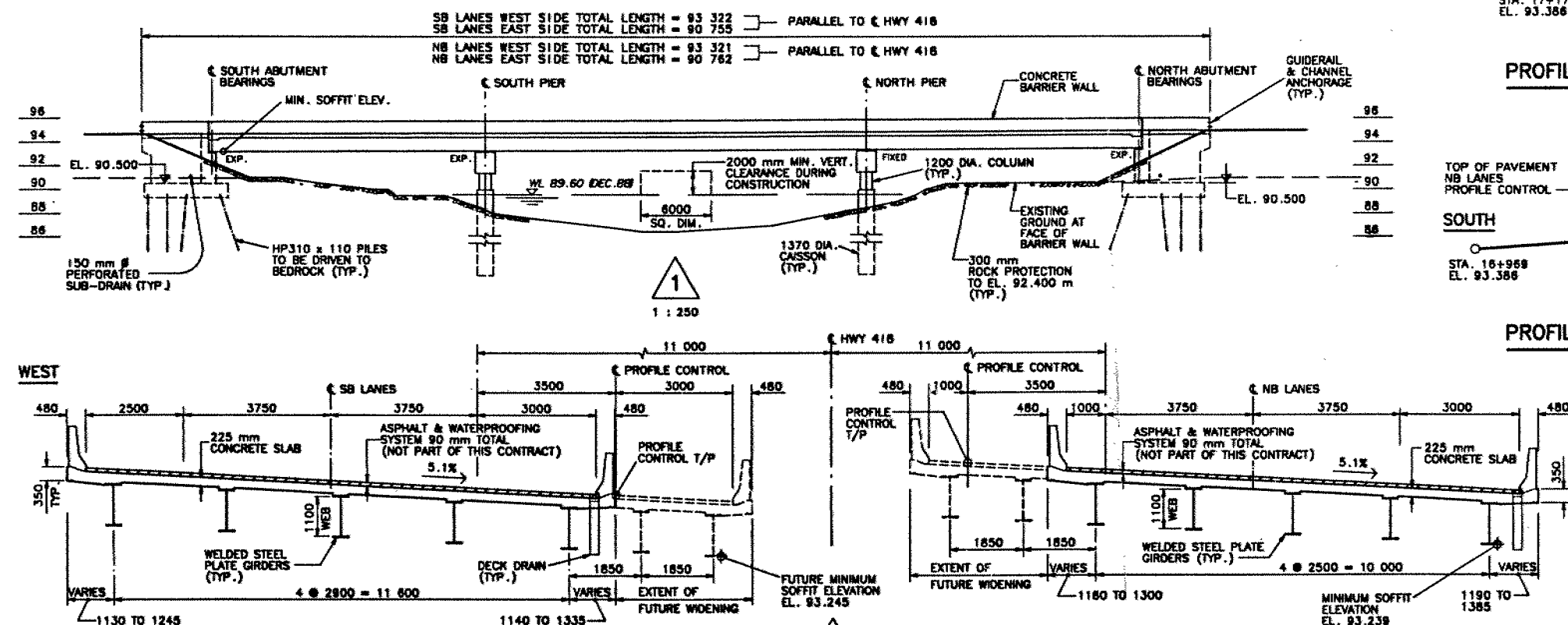
PROFILE HWY 416 SB LANES

N. T. S.



PROFILE HWY 416 NB LANES

N. T. S.



EAST



B.M. 90.631

N & W IN ROOT OF 1.0 MAP
30.6 RT. 17 + 896.2

1 1 75

HP 310 x 110 PILE DATA (SOUTHBOUND LANES)

LOCATION	SOUTH ABUTMENT	NORTH ABUTMENT
TYPE	1 : 3 BATTER	1 : 10 BATTER
NUMBER	11	9
ESTIMATED LENGTH (m)	12.5	12.0
ESTIMATED PILE TIP ELEVATION	77.5	77.5
MAX. COMB. FACTORED LOADS ULS	1225 kN	1225 kN
MAX. COMB. FACTORED LOADS SLS II	850 kN	850 kN

HP 310 x 110 PILE DATA (NORTHBOUND LANES)

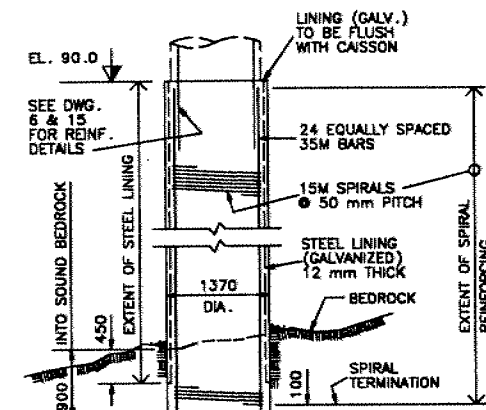
LOCATION	SOUTH ABUTMENT	NORTH ABUTMENT
TYPE	1 : 3 BATTER	1 : 10 BATTER
NUMBER	10	9
ESTIMATED LENGTH (m)	12.5	12.0
ESTIMATED PILE TIP ELEVATION	77.5	77.5
MAX. COMB. FACTORED LOADS ULS	1225 kN	1225 kN
MAX. COMB. FACTORED LOADS SLS II	850 kN	850 kN

CAISSON PILE DATA (SOUTHBOUND LANES)

LOCATION	SOUTH PIER	NORTH PIER
NUMBER	3	3
ESTIMATED LENGTH (m)	13.5	13.5
ESTIMATED CAISSON TIP ELEVATION	76.8	76.8
MAX. COMB. FACTORED LOADS ULS	7300 kN	7300 kN

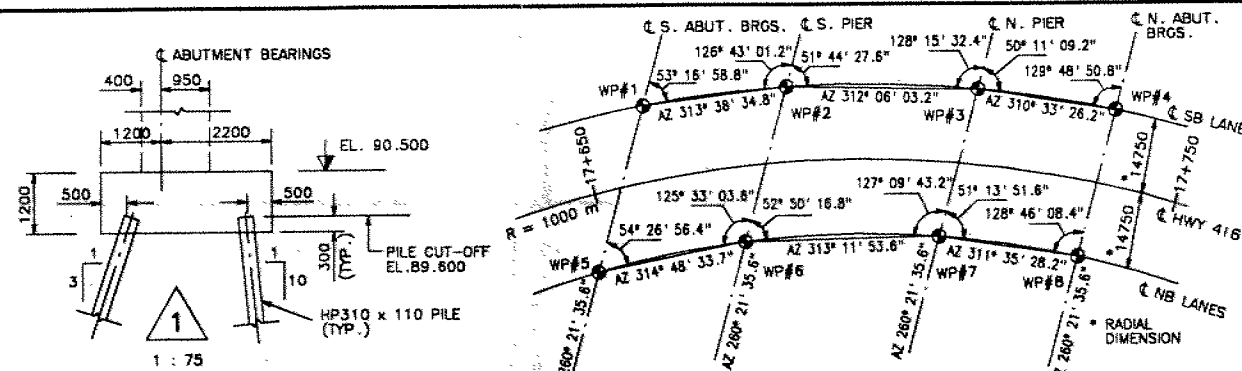
CAISSON PILE DATA (NORTHBOUND LANES)

LOCATION	SOUTH PIER	NORTH PIER
NUMBER	3	3
ESTIMATED LENGTH (m)	13.5	13.5
ESTIMATED CAISSON TIP ELEVATION	76.8	76.8
MAX. COMB. FACTORED LOADS ULS	7300 kN	7300 kN



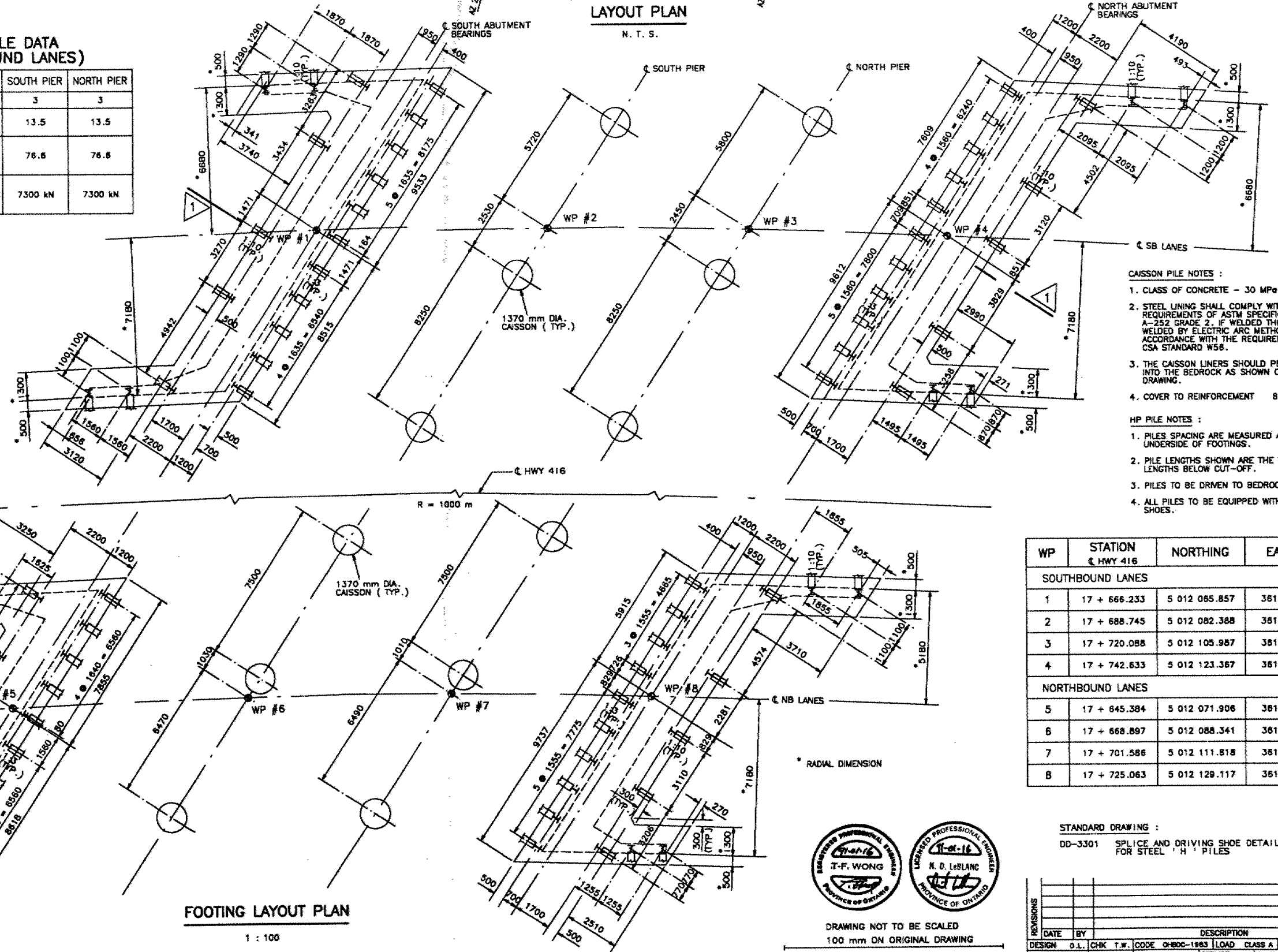
CAISSON DETAIL

1 : 50



LAYOUT PLAN

N. T. S.



FOOTING LAYOUT PLAN

1 : 100

METRIC

DIMENSIONS ARE IN MILLIMETRES
UNLESS OTHERWISE SHOWN.
ELEVATIONS, COORDINATES, CURVE
AND ALIGNMENT DATA ARE IN METRES.
STATIONS ARE IN KILOMETRES + METRES.



FOR CONSTRUCTION

CONT No
WP No 128-87-07/08

JOCK RIVER BRIDGES
STRUCTURE 22A - HWY 416 NBL
STRUCTURE 22B - HWY 416 SBL
FOOTING & PILING LAYOUT

Proctor & Redfern Limited
Consulting Engineer and Architect
Toronto Kingston

E.O. 89M01

CAISSON PILE NOTES :

1. CLASS OF CONCRETE - 30 MPa.
2. STEEL LINING SHALL COMPLY WITH THE REQUIREMENTS OF ASTM SPECIFICATION A-252 GRADE 2. IF WELDED THEY SHALL BE WELDED BY ELECTRIC ARC METHOD IN ACCORDANCE WITH THE REQUIREMENTS OF CSA STANDARD W58.
3. THE CAISSON LINERS SHOULD PENETRATE INTO THE BEDROCK AS SHOWN ON THE DRAWING.
4. COVER TO REINFORCEMENT 80 +/- 20 mm

HP PILE NOTES :

1. PILES SPACING ARE MEASURED AT THE UNDERSIDE OF FOOTINGS.
2. PILE LENGTHS SHOWN ARE THE THEORETICAL LENGTHS BELOW CUT-OFF.
3. PILES TO BE DRIVEN TO BEDROCK.
4. ALL PILES TO BE EQUIPPED WITH DRIVING SHOES.

WP	STATION @ HWY 416	NORTHING	EASTING
SOUTHBOUND LANES			
1	17 + 666.233	5 012 085.857	361 267.230
2	17 + 688.745	5 012 082.388	361 251.464
3	17 + 720.088	5 012 105.987	361 230.140
4	17 + 742.633	5 012 123.367	361 215.266
NORTHBOUND LANES			
5	17 + 645.384	5 012 071.906	361 302.839
6	17 + 668.897	5 012 088.341	361 286.513
7	17 + 701.586	5 012 111.818	361 264.468
8	17 + 725.063	5 012 129.117	361 249.114

STANDARD DRAWING :
DD-3301 SPLICE AND DRIVING SHOE DETAILS
FOR STEEL 'H' PILES



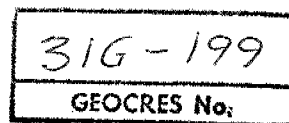
DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

DATE	BY	DESCRIPTION
DESIGN	D.L. CHK T.W.	CODE 0800-1983 LOAD CLASS A DATE DEC. 1990
DRAWING	C.K. CHK D.L. SITE 05-550	STRUCT SCHEME DWG 4



Golder Associates Ltd.

CONSULTING ENGINEERS



REPORT TO

MINISTRY OF TRANSPORTATION ONTARIO

FOUNDATION INVESTIGATION

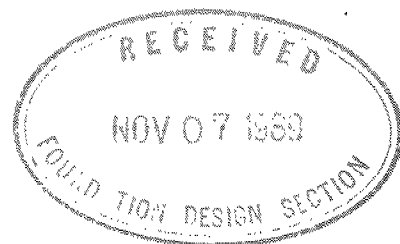
PROPOSED JOCK RIVER BRIDGES

HIGHWAY 416

CONT. 91-46

W.P. 128-87-07/08 DISTRICT 9 (OTTAWA)

NEPEAN, ONTARIO



Distribution:

15 copies - Ministry of Transportation Ontario
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November 1989

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ABSTRACT

This report presents the results of a subsurface investigation carried out at the site of the proposed twin bridge crossing of the Jock River by Highway 416 in Nepean, Ontario. Based on present plans, the crossing is to consist of two, 3 span bridges. The approach embankment height at the abutments is to be between 4.2 and 4.8 metres. The purpose of this investigation is to provide engineering recommendations on the geotechnical design aspects of the project, including foundation design, expected settlement of the approach embankments and an assessment of the embankment slope stability.

The subsurface conditions in the area of the proposed structures were determined by means of six (6) boreholes and seventeen (17) static cone penetration tests. The site was found to be underlain by surficial deposits of topsoil, alluvium, and sand/silty sand followed by a thick deposit of sensitive silty clay. The upper 5 to 7 metres of the grey silty clay has a low plasticity and an undrained shear strength of between 15 and 45 kilopascals. Below the low plasticity silty clay, the silty clay has a medium to high plasticity; vane shear strength testing gave undrained shear strengths of between 17 and 40 kilopascals. Oedometer consolidation testing indicates that the apparent preconsolidation pressure ranges from 85 to 140 kilopascals which is about 50 to 80 kilopascals in excess of the existing overburden pressure for the samples. The CPT data indicates that the silty clay deposits have been preconsolidated by about 60 kilopascals in excess of the existing overburden pressure although, in some zones, the deposits appear to have a lower preconsolidation pressure.

The silty clay deposits are underlain by deposits of sand, sand and gravel and glacial till containing cobbles and boulders.

Limestone bedrock was encountered at depths of between 12.9 and 14.4 metres below ground surface (elevation 76.2 to 78.8 metres).

The groundwater level was found to range from 0.4 to 1.4 metres below ground surface (elevation 89.5 to 91.4 metres).

Recommendations are provided for the support of the proposed bridge abutments and piers on end bearing steel H-piles. Due to consolidation settlement of the silty clay beneath the approach embankment, negative skin friction loads should be considered. The abutment piles should be battered both towards the river and towards the embankment. The pile caps should be provided with at least 1.8 metres of earth cover for frost protection.

Recommendations are provided for backfilling the abutment and for determining the horizontal load on the abutments.

To prevent excessive settlement of the embankments, it is recommended that the maximum embankment loading be 60 kilopascals.

This loading could result in a long term consolidation settlement of between 100 and 250 millimetres with a most probable settlement of 150 millimetres.

The factor of safety against rotational failure of the proposed embankments and abutments will be adequate provided that the embankment load is less than 60 kilopascals.

Due to stability and settlement constraints throughout this section of the highway, consideration should be given to extending the proposed CNR overpass structure to the Jock River or to lowering the highway grade and providing an underpass at the CNR railway crossing.

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1.0 INTRODUCTION

Golder Associates Ltd. has been retained by the Ministry of Transportation Ontario (MTO) to carry out a subsurface investigation at the site of the proposed twin bridge crossing of the Jock River by Highway 416 (see Key Plan, Figure 1). The purpose of the investigation was to determine the subsurface conditions at the site. Based on the factual information obtained, we were to provide engineering recommendations for the founding of the bridge structures and an assessment of the slope stability and the expected settlement of the approach embankments.

As proposed, the Highway 416 will cross the Jock River to the east of Moody Drive by means of two, 3 span bridges. The approach embankments will be a maximum of 4.2 metres above the existing ground surface at the south abutments and a maximum of 4.8 metres above the existing ground surface at the north abutments. The proposed roadway grade is to be about 8.5 metres above the river bottom at its lowest point.

The abutments for the bridge are to be set back from the crest of the river bank by between about 11 and 17 metres. As proposed, the four centre piers for the bridges are to be constructed in the river channel.

North of the proposed bridges, the embankment height is to increase as the roadway grade rises towards the CNR railway overpass near Strandherd Drive.

2.0 SITE DESCRIPTION AND GEOLOGY

The site of the proposed bridge is characterized by a relatively flat topography with a slight rise in grade south of the Jock River. Based on available contour plans, the banks to the river are sloped at about 2 to 4 horizontal to 1 vertical.

Geological maps suggest that the area of the structure is underlain by deposits of sensitive silty clay followed by limestone bedrock.

A previous subsurface investigation was carried out by Golder Associates along the Jock River at Cedarview Road. The subsurface conditions in this area were shown to consist of silty sand and organic silt alluvium over a deep deposit of sensitive silty clay; bedrock was encountered at about 20 metres below ground surface. The silty clay was shown to have a shear strength ranging from 19 to 38 kilopascals. A consolidation test carried out on a sample of the firm grey silty clay showed an apparent preconsolidation pressure of about 120 kilopascals, which is about 30 kilopascals in excess of the overburden pressure for the sample.

3.0 PROCEDURE

The field work for this investigation was carried out in two stages between June 14 and June 30, 1989.

During the first stage of the field work, four (4) boreholes were put down at the abutment locations using a track mounted hollow stem auger drill rig supplied and operated by Marathon Drilling Co. Ltd. of Gloucester, Ontario. All boreholes were advanced to the bedrock through about 13 to 14 metres of overburden. The limestone bedrock was core drilled in BXL size for a minimum of 3 metres in each of the boreholes. In addition, two boreholes were put down in the approach embankment area and taken to auger refusal. In situ vane testing was carried out to determine the shear strength characteristics of the silty clay. Standard penetration tests were carried out in the boreholes and samples of the soils encountered were recovered using drive open sampling equipment. In addition, relatively undisturbed 70 millimetre inside diameter shelly tube samples of the clay were taken in order that oedometer testing could be carried out to determine the consolidation characteristics of the silty clay subsoil.

Standpipes were sealed into all of the boreholes to determine the groundwater conditions at the site.

Samples of the soils encountered were taken to our laboratory for examination and classification testing. Samples of the soil encountered were tested for moisture content, liquid and plastic limits and grain size distribution. Consolidation testing was carried out on four samples of the sensitive silty clay.

Detailed logs of the soil and groundwater conditions encountered in the boreholes are given on the Record of Borehole sheets following the text of the report. The results of the laboratory testing are given on the Record of Borehole sheets and on Figures 2 to 8.

During the second stage of the field investigation, seventeen (17) static cone penetration tests (CPT) were carried out using equipment supplied and operated by Golder Associates. The CPT equipment was attached to the CME drill rig. The cone tests were advanced with continuous electronic recording of tip resistance, sleeve friction and pore water pressure until practical refusal was encountered. The CPT testing was carried out adjacent to most of the boreholes to allow comparison and calibration of the test results.

The results of the CPT testing are given on the Record of CPT sheets following the Record of Borehole sheets. The inferred subsurface stratigraphy from the testing is also shown on the CPT logs.

The drilling and CPT work was supervised throughout by a member of our engineering staff who directed the drilling, sampling and in situ testing, logged the boreholes and collected the CPT records.

The approximate locations of the boreholes and CPT holes are given on the Site Plan, Drawing 1288707/08-A. The subsurface information

obtained in the boreholes together with the proposed bridge grades are given on Drawing 1288707/08-B.

The borehole locations and elevations were determined by MTO survey personnel. The locations of the CPT tests were determined by Golder Associates personnel with reference to the staked boreholes. The approximate ground surface elevations at the CPT test locations were determined from the borehole survey information provided by MTO. The elevations are referenced to Geodetic datum.

4.0 SUBSURFACE CONDITIONS

4.1 General

As previously indicated, the detailed soil and groundwater conditions determined from the boreholes are given on the Record of Borehole sheets following the text of this report. The following sections present a detailed description of the soil and groundwater conditions encountered in the boreholes as well as the inferred conditions encountered in the CPT holes.

4.2 Topsoil, Alluvium

All of the boreholes put down during the investigation encountered surficial deposits of topsoil and/or alluvium.

Surficial deposits of topsoil having a thickness of about 0.2 to 0.3 metres were encountered along the south side of the river (boreholes 22B5, 22A1, 22B1) and at borehole 22A8.

Boreholes 22B4 and 22A4 put down adjacent to the north bank of the river encountered about 0.2 metres of topsoil, followed by alluvium deposits having a thickness ranging from 1.7 to 3.2 metres. The upper 1.1 to 1.4 metres of the alluvium deposit was found to be composed mostly of silty clay with silty sand and trace amounts of organic matter; beneath this zone, silty sand containing organic

silt, wood, shells and organic matter was encountered. Standard penetration tests carried out within the silty sand alluvium gave N values of 1 to 2 blows per 0.3 metres, which reflect a very loose relative density. The moisture content of the silty sand alluvium at borehole 22A4 was found to be between 71 and 217 percent. The moisture content of 217 percent for the sample from borehole 22A4 could reflect a high organic content for the portion of the sample tested and may not be representative of the entire deposit. The organic content of one sample of the silty sand alluvium was found to be about 9 percent.

4.3 Surficial Sand, Silty Sand

Deposits of sand and silty sand having a combined thickness of about 2.4 metres were encountered beneath the topsoil in borehole 22B5 put down south of the Jock River. Standard penetration testing carried out at borehole 22B5 gave N values of 1 to 5 blows per 0.3 metres which reflects a very loose to loose relative density for this material.

4.4 Sensitive Silty Clay

The surficial topsoil, alluvium and sandy deposits are underlain by an extensive deposit of sensitive silty clay. The upper 0.8 to 1.2 metres of the deposit at boreholes 22A1, 22B1 and 22A8 was found to be weathered to a grey brown crust. Beneath the weathered zone at the above boreholes and at the other boreholes put down during the investigation, the silty clay is grey in colour and has a soft to firm consistency.

The upper part of the grey silty clay, which extends to depths of about 5 to 7 metres, contains sand seams and generally has low plasticity. Atterberg limit tests on this silty clay gave liquid limit values of 20 to 27 and plasticity indices of 5 to 10, as shown on the Plasticity Chart, Figure 2. In situ vane testing carried out in the upper grey silty clay gave undrained shear

strengths of 15 to 45 kilopascals, which indicate a soft to firm consistency. The moisture content of the upper silty clay ranges from about 25 to 62 percent.

Consolidation tests were carried out on two samples of the upper grey silty clay; the results of this testing are given on Figures 4 and 5. The oedometer testing showed apparent preconsolidation pressures of about 85 and 100 kilopascals, which are about 50 to 60 kilopascals in excess of the existing overburden pressure for the samples. The compression index of the samples was determined to be about 0.9.

Below the upper low plasticity silty clay, the silty clay is not noticeably layered. Atterberg limit test results for this clay are summarized on the Plasticity Chart, Figure 3; the liquid limit values range from 42 to 70 and the plasticity indices range from 20 to 42, indicating a clay of medium to high plasticity. The in situ vane testing gave undrained shear strength values from 17 to about 40 kilopascals, indicating a soft to firm consistency. The moisture content of the lower silty clay ranges from about 50 to 85 percent, which approaches or exceeds the measured liquid limit values.

The results of two consolidation tests carried out on samples of this silty clay are given on Figures 6 and 7. The testing showed apparent preconsolidation pressures of 120 and 140 kilopascals, which are about 60 to 80 kilopascals in excess of the existing overburden pressure. The compression indices were determined to be 0.9 and 1.4.

The CPT test results were also used to determine the probable preconsolidation for the silty clay deposit. Correlation data between the cone bearing pressure and overconsolidation ratio (OCR) given on Figure 9 was used to prepare continuous plots of preconsolidation pressure versus depth. The validity of this relationship for this site was checked using the oedometer

consolidation data obtained from this investigation; data from the oedometer tests carried out on samples recovered during this investigation and during a previous investigation by MTO (as shown on Figure 9) were found to agree with the previous findings. Plots of the preconsolidation pressure versus depth for each of the CPT logs are given on the Summary of Preconsolidation Pressure Profiles, Figures 10 and 11. The oedometer preconsolidation pressure values generally support the cone test findings.

In general, the cone data indicates that the silty clay deposits have been preconsolidated by about 60 kilopascals in excess of the existing overburden pressure, although in some zones, the deposits appear to have a lower preconsolidation pressure.

The CPT results were also used to generate continuous profiles of undrained shear strength; the results of this work are given on Figures 12 and 13 together with the vane shear strength information obtained in adjacent boreholes. The cone shear strength was determined using a ratio of undrained shear strength to preconsolidation pressure of 0.20, which has been shown to give results which correspond to the average mobilized strength at failure in the foundation of embankments built on inorganic soft clays. On the north side of the river, the cone shear strengths show fairly good agreement with the vane shear strengths and generally range from about 17 to 30 kilopascals in the upper silty clay and about 30 to 40 kilopascals in the lower silty clay. On the south side the cone undrained shear strengths are generally above the vane strengths; the cone shear strengths were found to range from about 20 kilopascals in the upper part of the stratum to about 40 kilopascals at depth.

4.5 Sand, Sand and Gravel

Thin deposits of sand or sand and gravel were encountered beneath the silty clay at most of the boreholes. These deposits contained some cobbles and occasional boulders.

4.6 Glacial Till

The silty clay, sand, and sand and gravel strata are underlain by a mantle of glacial till. The glacial till was encountered at depths ranging from about 11.0 to 12.7 metres below ground surface (elevation 78.1 to 81.5 metres) and has a thickness of about 1.0 to 3.1 metres.

The glacial till consists of a heterogeneous mixture of all grain sizes but may be generally described as silty sand or sandy silt containing gravel, clay, cobbles and boulders. The results of a grain size distribution tests on samples of the glacial till are given on Figure 8 and are summarized on the Record of Borehole sheets. It should be noted that the gradation tests were carried out on 38 millimetre I.D. split barrel samples and so do not reflect the presence of cobbles or boulders.

In two of the boreholes, it was necessary to use diamond drilling techniques to penetrate to the surface of the bedrock due to the bouldery nature of the glacial till. The relative density of the glacial till is considered to be dense to very dense based on the standard penetration test results obtained in the stratum.

4.7 Bedrock

The four cored boreholes at the abutment locations encountered limestone bedrock at a depth of about 12.9 to 14.4 metres below existing ground surface (elevation 76.2 to 78.8 metres). Auger refusal was encountered at depths of 13.6 to 14.3 metres in the other two boreholes put down at the site. The limestone contained some shale interbeds, typical of the limestone of the Ottawa formation.

A measure of the quality of the bedrock retrieved from the borehole is shown on the respective Record of Borehole sheets as the percent total core recovery (T.C.R.) and Rock Quality Designation (R.Q.D.);

for definitions of these parameters, reference should be made to the Explanation of Terms sheet following the text of this report. The solid core recovery (S.C.R.) refers to the percentage of solid drill core, regardless of length, recovered at full diameter, measured relative to the length of total core run. The amount of core lost during coring was very low, resulting in core recovery values of between 99 and 100 percent. The R.Q.D. values ranged from 81 to 97 percent and the S.C.R. values ranged from 87 to 98 percent, which reflect the good quality of the limestone bedrock.

4.8 Groundwater

Groundwater levels were obtained from standpipes sealed in the completed boreholes. On June 30, 1989 the groundwater level was found to range from 0.4 to 1.4 metres below ground surface (elevation 89.5 to 91.4 metres). Minor downward drainage gradients are apparent from multiple standpipe installations. The shallow depth groundwater levels are expected to be higher during wet periods of the year such as the early spring.

5.0 PROPOSED BRIDGE STRUCTURES

5.1 Foundations

The foundation stratum at and below river bottom elevation has a shear strength of only about 17 kilopascals, which is too low to support the bridge foundations. It is recommended that the foundation loads be transferred to the underlying limestone bedrock by the use of end bearing piles. The silty clay deposit is sensitive to disturbance and significant loss of strength in the clay stratum due to pile driving may reduce the stability of the approach embankments. In order to minimize disturbance of the sensitive clay deposit, it is recommended that steel H piles be used.

Due to the bouldery nature of the sand and gravel and glacial till deposits, the H piles should be equipped with a cast steel driving shoe. A heavy pile section is preferred to penetrate the bouldery glacial till.

As a design example, the allowable load for a HP 310x110 steel H-pile at Serviceability Limit States (SLS) may be taken as 1150 kilonewtons; at Ultimate Limit States (ULS), the factored capacity can be taken as 1600 kilonewtons. In this case, the H pile should be set to a termination of 8 blows for the last 12 millimetres of penetration using a hammer transferring about 60 kilojoules of energy per blow to the pile.

Based on piling experience in this area, it is possible that several rounds of restriking could be required to achieve permanence of the final set. Therefore, provision should be made for restriking all of the piles at least once to confirm the set. Piles that do not meet the design set criteria on the first restrike would require additional restriking. A minimum of two days should be allowed before restriking a pile.

Allowance should be made for pile load testing at the time of construction.

Skin friction loads can be induced on piles due to consolidation settlement of the silty clay beneath the approach embankments. The negative skin friction loads on the piles can be determined using an effective stress approach as described in the Commentary to the Ontario Highway Bridge Design Code (OHBDC) section C6-8.3.3.2. The skin friction force per unit area of pile can be determined by

$$f_s = 0.25 p_z'$$

where p_z' = unit effective vertical stress at depth z .

For design purposes, the effective unit weight of the silty clay can be taken as 7 kilonewtons per cubic metre.

The negative skin friction loads should be assumed to act only with the permanent (dead) loads.

As a design example, the negative skin friction load on a HP310x110 steel H pile driven through 10 metres of silty clay could be taken as 300 kilonewtons at Serviceability Limit States and 375 kilonewtons at Ultimate Limit States, if the embankment loading is 60 kilopascals.

Since some settlement will result from the consolidation of the clay stratum under the approach embankment loading, it is recommended that batter piles be installed in a direction away from the structure, as well as in the normal direction towards the river, to resist movement of the abutments towards the centre of settlement below the approach embankments.

The pile caps should be provided with at least 1.8 metres of earth cover for frost protection purposes. The abutments should be

backfilled with compacted non frost susceptible, free draining backfill such as that meeting MTO Granular B Type I or II for at least 1.5 metres beyond the inside face of the abutment.

If lateral movement at the top of the abutment of about 0.05 percent of the retained height can be tolerated, "active" earth pressure coefficients (K_a) should be used in determining the horizontal load on the abutments. If no movement can be permitted, then "at rest" pressure coefficients (K_o) should be used.

Assuming that a sandy earth fill material having a unit weight of about 18 kilonewtons per cubic metre is used behind the abutments for embankment settlement considerations, then the following earth pressure coefficients may be used in determining the lateral load acting on the abutments:

Earth Pressure
Coefficient

At Ultimate Limit States

"at rest" condition	0.56
"active" condition	0.40

At Serviceability Limit States

"at rest" condition	0.47
"active" condition	0.31

Earth pressure parameters for other materials could be provided if necessary.

To prevent compaction induced stress on the abutment walls, the granular fill within 1 metre of the walls should be compacted only with a plate tamper or light vibratory steel drum roller.

Highway live loads should be considered on the abutments unless approach slabs are used.

5.2 Settlement of Approach Embankments

The proposed roadway grade will be a maximum of 4.2 metres above the existing ground surface at the south abutments and a maximum of 4.8 metres above existing ground surface at the north abutments.

Based on the results of the consolidation cone test data, the silty clay has generally been preconsolidated by about 60 kilopascals in excess of the existing overburden pressure, although, in some zones of the deposits, the preconsolidation pressure may be lower.

For embankment fill heights that impose a loading beyond the preconsolidation pressure, settlement of embankments tends to be excessive and unpredictable. This excessive loading may also produce rotational effects on bridge abutments and result in the need for long term maintenance of the approaches. For instance, previous experience in this area suggests that the consolidation settlement could be about 600 millimetres if the embankments are constructed to a height of about 4.8 metres with material having a unit weight of about 21 kilonewtons per cubic metre. It is therefore recommended that embankment loadings not exceed the apparent preconsolidation pressure for the sensitive silty clay. Thus for design purposes, it is recommended that the maximum embankment loading be 60 kilopascals. This loading over some 10 metres of silty clay could result in a long term consolidation settlement at the centre of the embankment of between 100 and 250 millimetres with a most probable settlement of about 150 millimetres. Based on previous embankment settlement monitoring in this area, the time required for 90 percent of the consolidation settlement to occur could be about 10 years; it is expected to take about 2 to 4 years for about 50 percent of the settlement to occur. Previous experience also suggests that the primary consolidation should be mostly complete about 2 years after construction and that subsequent settlement will be due mainly to secondary compression.

On the north side of the Jock River, some additional settlement should be expected near the abutment due to compression of the loose organic alluvium; this settlement will necessitate maintenance in adjusting the approach grade to the pile supported abutment. Consideration should be given to constructing embankments well in advance of bridge construction to allow some of the settlement to take place prior to final road grading. Approach slabs to the bridges should be used to maintain as smooth an approach as possible.

Assuming that a relatively light weight earth fill material (such as clean uniform sand or clear crushed stone) with a unit weight of about 17 to 18.5 kilonewtons per cubic metre is used, the maximum height of earth fill embankment would be about 3.2 to 3.5 metres. The allowable embankment fill height could be increased by the use of light weight fill (such as slag), by using a combination of earth fill and no weight fill (such as Durofoam, Styrofoam, etc.), or by providing void structures beneath the embankments (such as culverts). Note that the use of no weight fill in the area behind the abutments would reduce the horizontal loading on the abutments and increase the overall rotational stability of the abutments.

As discussed previously, the roadway grade is to rise north of the proposed Jock River bridge for the CNR railway overpass. Due to settlement and embankment side slope stability considerations north of the Jock River crossing, it is understood that a bridge structure is being considered in lieu of embankments. Therefore, as an alternative to using light weight and/or no weight fill to achieve the required grade at the Jock River crossing, consideration may also be given to extending the proposed CNR overpass structure to the Jock River.

In view of the embankment slope stability and settlement considerations for both the Jock River and CNR crossings, it is suggested that study be given to lowering the grade throughout this

section of the highway and to providing an underpass at the CNR railway crossing. In this case, the roadway grade at the Jock River bridges could be lowered so that the embankments could be constructed entirely of locally available earth fill materials. Between the Jock River bridges and this cut for the CNR crossing, the roadway could be constructed generally about 1 metre above existing grade which would optimize the pavement design in this section. If an underpass is used at the CNR crossing, a tied back diaphragm wall structure may be considered as one of the design options for the cut section.

5.3 Stability of Approach Embankments

In evaluating the rotational stability of the proposed approach embankments and abutments, an undrained, total stress approach was used. Based on the vane shear test results and the cone data, a design undrained shear strength of 17 kilopascals was generally used for the upper silty clay. For the most part, the lower silty clay was assumed to have an undrained shear strength of 27 kilopascals. Since the vane shear strength data at the southwest abutment (borehole 22B1) showed a relatively constant vane shear strength with depth, a design undrained shear strength of 19 kilopascals was assumed throughout the deposit at this location.

If settlement criteria are used in establishing the maximum height of the embankments (i.e. maximum stress increase of 60 kilopascals) near the bridge structures, then the factor of safety against rotational failure of the approach embankments would be adequate if 2 horizontal to 1 vertical side slopes are used. The short term rotational stability of the abutments should also be adequate provided that the abutments are set back from the crest of the slope to the river by between about 11 to 17 metres, as presently proposed.

As discussed, we do not recommend that the embankment loading be taken beyond 60 kilopascals from an embankment settlement point of

view. If higher loadings are used, however, side slopes of 2 horizontal to 1 vertical are appropriate for embankments up to about 4 metres in height and constructed of sandy earth fill material having a total unit weight of about 18 kilonewtons per cubic metre. For embankments constructed of a similar material and between 4.0 and 4.8 metres in height, a mid height berm having a width of 7 metres would be required. Careful monitoring would be required during construction where the embankment load exceeds 60 kilopascals; stage construction may be necessary. The monitoring instrumentation would include pneumatic type piezometers to measure the porewater pressure in the silty clay, slope indicators to monitor horizontal displacement in the silty clay deposit with depth, and settlement plates. Notwithstanding, embankment loading beyond 60 kilopascals within 20 metres of the abutments is not recommended due to both stability and settlement constraints.

To prevent erosion of the river banks near the abutments and piers, the banks could be protected with rip rap underlain by a filter fabric. Additional details on this treatment can be provided, if required.

5.4 Construction Considerations

No unusual side slope stability problems are anticipated in carrying out excavations for the pile caps in the overburden materials at the site above the groundwater level; normal construction side slopes of 1 horizontal to 1 vertical could be used. However, excavation below the groundwater level through the surficial silty sand or sand deposits (such as at boreholes 22B5 and 22A4) will present some constraints. Groundwater inflow from these deposits could be substantial and could result in sloughing of the side slopes to the excavation. As such, flatter than normal side slopes (4 horizontal to 1 vertical) will be required for short term excavations in these deposits together with pumping from the excavation. Where the excavation is carried out within the silty clay below the groundwater level, normal 1 horizontal to 1 vertical

side slopes are appropriate and the groundwater inflow should be handled by pumping from sumps within the excavation.

The silty clay soils at this site are highly sensitive to disturbance due to ponded water and construction traffic. Therefore, it may be necessary to provide a 0.5 metre thick granular working mat consisting of compacted coarse granular material for construction vehicle access.

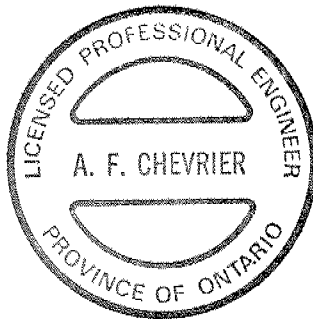
The soils at this site are highly susceptible to frost heaving. Therefore, the exposed subgrade around the piles should be protected from freezing using straw, insulation blankets or granular backfill to prevent pile jacking due to freezing of the soil around the piles.

Yours truly,

GOLDER ASSOCIATES LTD.



A.F. Chevrier, P. Eng.



F.J. Heffernan, P. Eng.

AFC/FJH/yc
Disk 6

EXPLANATION OF TERMS USED IN REPORT

N VALUE: THE STANDARD PENETRATION TEST (SPT) N VALUE IS THE NUMBER OF BLOWS REQUIRED TO CAUSE A STANDARD 51mm O.D. SPLIT BARREL SAMPLER TO PENETRATE 0.3m INTO UNDISTURBED GROUND IN A BOREHOLE WHEN DRIVEN BY A HAMMER WITH A MASS OF 63.5kg, FALLING FREELY A DISTANCE OF 0.76m. FOR PENETRATIONS OF LESS THAN 0.3m N VALUES ARE INDICATED AS THE NUMBER OF BLOWS FOR THE PENETRATION ACHIEVED. AVERAGE N VALUE IS DENOTED THUS \bar{N} .

DYNAMIC CONE PENETRATION TEST: CONTINUOUS PENETRATION OF A CONICAL STEEL POINT (51mm O.D. 60° CONE ANGLE) DRIVEN BY 475 J IMPACT ENERGY ON 'A' SIZE DRILL RODS. THE RESISTANCE TO CONE PENETRATION IS MEASURED AS THE NUMBER OF BLOWS FOR EACH 0.3m ADVANCE OF THE CONICAL POINT INTO THE UNDISTURBED GROUND.

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

CONSISTENCY: COHESIVE SOILS ARE DESCRIBED ON THE BASIS OF THEIR UNDRAINED SHEAR STRENGTH (c_u) AS FOLLOWS:

c_u (kPa)	0 - 12	12 - 25	25 - 50	50 - 100	100 - 200	> 200
	VERY SOFT	SOFT	FIRM	STIFF	VERY STIFF	HARD

DENSENESS: COHESIONLESS SOILS ARE DESCRIBED ON THE BASIS OF DENSENESS AS INDICATED BY SPT N VALUES AS FOLLOWS:

N (BLOWS/0.3m)	0 - 5	5 - 10	10 - 30	30 - 50	> 50
	VERY LOOSE	LOOSE	COMPACT	DENSE	VERY DENSE

ROCKS ARE DESCRIBED BY THEIR COMPOSITION AND STRUCTURAL FEATURES AND / OR STRENGTH.

RECOVERY: SUM OF ALL RECOVERED ROCK CORE PIECES FROM A CORING RUN EXPRESSED AS A PERCENT OF THE TOTAL LENGTH OF THE CORING RUN

MODIFIED RECOVERY: SUM OF THOSE INTACT CORE PIECES, 100mm+ IN LENGTH EXPRESSED AS A PERCENT OF THE LENGTH OF THE CORING RUN. THE ROCK QUALITY DESIGNATION (RQD), FOR MODIFIED RECOVERY, IS:

RQD (%)	0 - 25	25 - 50	50 - 75	75 - 90	90 - 100
	VERY POOR	POOR	FAIR	GOOD	EXCELLENT

JOINTING AND BEDDING:

SPACING	50mm	50 - 300mm	0.3m - 1m	1m - 3m	> 3m
JOINTING	VERY CLOSE	CLOSE	MOD. CLOSE	WIDE	VERY WIDE
BEDDING	VERY THIN	THIN	MEDIUM	THICK	VERY THICK

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

SS SPLIT SPOON	TP THINWALL PISTON
WS WASH SAMPLE	OS OSTERBERG SAMPLE
ST SLOTTED TUBE SAMPLE	RC ROCK CORE
BS BLOCK SAMPLE	PH TW ADVANCED HYDRAULICALLY
CS CHUNK SAMPLE	PM TW ADVANCED MANUALLY
TW THINWALL OPEN	FS FOIL SAMPLE

STRESS AND STRAIN

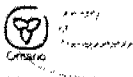
u_w	kPa	PORE WATER PRESSURE
r_u	1	PORE PRESSURE RATIO
σ	kPa	TOTAL NORMAL STRESS
σ'	kPa	EFFECTIVE NORMAL STRESS
τ	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
ϵ	%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
μ	1	COEFFICIENT OF FRICTION

MECHANICAL PROPERTIES OF SOIL

m_v	kPa ⁻¹	COEFFICIENT OF VOLUME CHANGE
C_c	1	COMPRESSION INDEX
C_s	1	SWELLING INDEX
C_α	1	RATE OF SECONDARY CONSOLIDATION
c_v	m ² /s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
T_v	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
σ'_{vo}	kPa	EFFECTIVE OVERBURDEN PRESSURE
σ'_p	kPa	PRECONSOLIDATION PRESSURE
τ_f	kPa	SHEAR STRENGTH
c'	kPa	EFFECTIVE COHESION INTERCEPT
ϕ'	-°	EFFECTIVE ANGLE OF INTERNAL FRICTION
c_u	kPa	APPARENT COHESION INTERCEPT
ϕ_u	-°	APPARENT ANGLE OF INTERNAL FRICTION
τ_R	kPa	RESIDUAL SHEAR STRENGTH
τ_r	kPa	REMOULDED SHEAR STRENGTH
S_f	1	SENSITIVITY = $\frac{c_u}{\tau_r}$

PHYSICAL PROPERTIES OF SOIL

ρ_s	kg/m ³	DENSITY OF SOLID PARTICLES	e	1, %	VOID RATIO	e_{min}	1, %	VOID RATIO IN DENSEST STATE
γ_s	kn/m ³	UNIT WEIGHT OF SOLID PARTICLES	n	1, %	POROSITY	I_D	1	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
ρ_w	kg/m ³	DENSITY OF WATER	w	1, %	WATER CONTENT	D	mm	GRAIN DIAMETER
γ_w	kn/m ³	UNIT WEIGHT OF WATER	S_r	%	DEGREE OF SATURATION	D_n	mm	n PERCENT - DIAMETER
ρ	kg/m ³	DENSITY OF SOIL	w_L	%	LIQUID LIMIT	C_u	1	UNIFORMITY COEFFICIENT
γ	kn/m ³	UNIT WEIGHT OF SOIL	w_p	%	PLASTIC LIMIT	h	m	HYDRAULIC HEAD OR POTENTIAL
ρ_d	kg/m ³	DENSITY OF DRY SOIL	w_s	%	SHRINKAGE LIMIT	q	m ³ /s	RATE OF DISCHARGE
γ_d	kn/m ³	UNIT WEIGHT OF DRY SOIL	I_p	%	PLASTICITY INDEX = $w_L - w_p$	v	m/s	DISCHARGE VELOCITY
ρ_{sat}	kg/m ³	DENSITY OF SATURATED SOIL	I_L	1	LIQUIDITY INDEX = $\frac{w - w_p}{I_p}$	i	1	HYDRAULIC GRADIENT
γ_{sat}	kn/m ³	UNIT WEIGHT OF SATURATED SOIL	I_C	1	CONSISTENCY INDEX = $\frac{w_L - w}{I_p}$	k	m/s	HYDRAULIC CONDUCTIVITY
ρ'	kg/m ³	DENSITY OF SUBMERGED SOIL	e_{max}	1, %	VOID RATIO IN LOOSEST STATE	j	kn/m ³	SEEPAGE FORCE
γ'	kn/m ³	UNIT WEIGHT OF SUBMERGED SOIL						



RECORD OF BOREHOLE No 22A1

METRIC

W P 128-87-07/08 LOCATION Co-ords N5 012 070.6; E 361 313.1 ORIGINATED BY DJS
DIST 9 HWY 416 BOREHOLE TYPE Hollow Stem Auger, BXL Rock Coring COMPILED BY FJH
DATUM Geodetic DATE June 19-20, 1989 CHECKED BY AFC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT LIMIT			UNIT WEIGHT Y KN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION [%] GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20 40 60 80 100							W _p	W	W _L
								SHEAR STRENGTH kPa									
								○ UNCONFINED	+ FIELD VANE	WATER CONTENT (%)							
								● QUICK TRIAXIAL	x LAB VANE	20 40 60							
91.7	Ground Surface																
0.0	Topsoil																
0.2	Silty Clay (weathered crust)																
91.0	Grey brown						91										
0.7	Fine sand, some silt Very loose		1	SS	2												
1.0	Silty Clay with some sand seams (weathered crust)																
89.9	Stiff Grey brown		2	SS	WH*		90										
1.8																	
	Silty Clay trace to some sand seams						89										
	Soft to firm Grey		3	SS	PM												
							88										
87.0							87										
4.7			4	SS	1												
	Silty Clay						86										
	Soft to firm Grey																
			5	TP	PH												
							85										
			6	SS	PM		84										
							83										
			7	SS	PM		82										
81.4																	
10.3	Silty sand & gravel with some cobbles						81										
80.6	Loose Grey		8	SS	4												
11.1	Silty Sand, some gravel, clay and boulders (Glacial till) Dense to very dense						80										
			9	SS	42												
78.8			10	RC BXL	--		79										
12.9	Continued						78										
						</											

*Sank under weight
of hammer

+3, x5 Numbers refer to 20
Sensitivity 15-5 (%) STRAIN AT FAILURE
10

OFFICE REPORT ON SOIL EXPLORATION



Ministry
of
Transportation

RECORD OF BOREHOLE No 22AI

METRIC

N P 128-87-07/08 LOCATION Co-ords N 5 012 070.6; E 361 313.1 ORIGINATED BY DJS
DIST 9 HWY 416 BOREHOLE TYPE Hollow Stem Auger COMPILED BY FJH
DATUM Geodetic DATE June 19-20, 1989 CHECKED BY APC

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV. DEPTH.	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE			'N' VALUES	SHEAR STRENGTH kPa								
							20	40	60	80	100		20	40	60	
78.6	Continued															
12.9	Limestone bedrock occasional shale interbed		11	RC BXL	TCR= 99% RQD= 84% SCR= 97%											
	Sound Grey															
	25 mm seam from 13.69 to 13.72															
75.7																
16.0	End of hole															
	*TCR: Total Core Recovery RQD: Rock Quality Designation SCR: Solid Core Recovery															

OFFICE REPORT ON SOIL EXPLORATION

+3, x⁵; Numbers refer to
Sensitivity

20
15 \div 5 (%) STRAIN AT FAILURE
10

RECORD OF BOREHOLE No 22A4

METRIC

W P 128-87-07/08 LOCATION Co-ords N 5 012 128.0; E 361 258.3 ORIGINATED BY DJS
 DIST 9 HWY 416 BOREHOLE TYPE Hollow Stem Auger, BXL Rock Coring COMPILED BY FJH
 DATUM Geodetic DATE June 14, 1989 CHECKED BY AEC

OFFICE REPORT ON SOIL EXPLORATION

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100					
90.8	Ground Surface																GR SA SI CL
0.0	Topsoil																
0.2	Silty Clay some silty sand, trace organic matter (alluvium)		1	SS	4												
89.2	Very stiff Grey brown																
1.6	Silty Sand trace organic silt, wood and shells (alluvium)		2	SS	2												
			3	SS	1												
87.4	Very loose Grey brown		4	SS	1												
3.4	Silty clay occasional sand seam Soft Grey		5	SS	PM												
45.3																	
5.5	Silty Clay		6	TP	PH												
	Soft to firm Grey		7	SS	PM												
			8	SS	PM												
79.7			9	SS	PM												
11.1	Silty clay trace gravel occasional sand seam		10	SS	2												
78.9	Very stiff Grey																
11.9	Fine to coarse sand & gravel some silt, occasional cobble																
78.1	Very dense Grey		11	SS	78												
12.7	Sandy silt some gravel and clay, occasional cobble and boulder (glacial till)																
77.1	Very dense Grey																
13.7	Continued																

+3, x5: Numbers refer to 20
Sensitivity 15 - 5 (%) STRAIN AT FAILURE
10

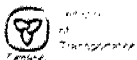
RECORD OF BOREHOLE No 22A4

METRIC

W P 12R-87-07/08 LOCATION Co-ords N 5 012 128.0; E 361 258.3 ORIGINATED BY DJS
 DIST 9 HWY 416 BOREHOLE TYPE Hollow Stem Auger COMPILED BY FJH
 DATUM Geodetic DATE June 14, 1989 CHECKED BY APC

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT Y	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE			20	40	60	80	100					
77.1	Continued															
12.7	Limestone bedrock with occasional black shale interbed		12	RC BLX	TCR= 100% RQD= 90% SCR= 98%	77										
	Sound Grey					76										
						75										
74.1																
12.7	End of Hole					74										
	*TCR: Total Core Recovery RQD: Rock Quality Designation SCR: Solid Core Recovery															

OFFICE REPORT ON SOIL EXPLORATION



RECORD OF BOREHOLE No 22A8

METRIC

W.P. 128-97-07/03 LOCATION Co-ords N 5 012 164.7; E 361 219.7 ORIGINATED BY DJS
DIST 9 HWY 416 BOREHOLE TYPE Hollow Stem Auger COMPILED BY FJH
DATUM Geodetic DATE June 22, 1989 CHECKED BY APC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				NATURAL MOISTURE CONTENT			UNIT WEIGHT Y	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES			20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	W _p	W	W _L		
90.9	Ground Surface															GR SA SI CL
0.0	Topsoil															
0.3	Silty clay some sand layers (weathered crust)		1	SS	5											
89.0	Very stiff Grey															
1.5			2	SS	WH											
	Silty clay trace to some sand seams		3	TP	PH											
	Soft to firm Grey															
85.2			4	SS	PM											
5.3			5	SS	PM											
	Silty clay		6	SS	PM											
	Soft to firm Grey		7	SS	WR											
79.7			8	SS	WR											
10.8	Silty clay some sand and gravel layers															
78.9	Soft to firm Grey															
11.6			9	SS	56											
	Silty sand some gravel, cobbles and boulders (glacial till)															
	Very dense Grey															
76.2																
14.3	End of hole Auger refusal * Bank under weight of															

+3, x5: Numbers refer to 20
Sensitivity 15-25 (%) STRAIN AT FAILURE
10

OFFICE REPORT ON SOIL EXPLORATION



Canada

RECORD OF BOREHOLE No 22BI

METRIC

W.P. 128-87-07/04 LOCATION Co-ords N 5 012 066.5; E 361 258.0 ORIGINATED BY DJS
DIST 9 H.V. 416 BOREHOLE TYPE Hollow Stem Auger COMPILED BY KJH
DATUM Geodetic DATE June 16, 1989 CHECKED BY APC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT LIMIT			UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT. PLT	NUMBER	TYPE	'N' VALUES			20 40 60 80 100	20 40 60 80 100	W _p	W	W _L		
91.2	Ground Surface													GR SA SI CL
0.0	Topsoil													
0.2	Silty clay trace silty sand													
90.2	Stiff Grey brown		1	SS	1									
1.0	Fine to medium sand Very loose Brown													
1.2	Silty clay some gravel and cobbles Grey													
1.5														
	Silty clay some sand seams, trace shells		2	SS	1									
	Soft Grey													
			3	SS	WR *									
86.0														
5.2														
	Silty Clay		4	SS	PM									
	Soft Grey													
			5	SS	PM									
81.6			6	SS	PM									
9.6	Silty clay trace sand and gravel													
81.0	Firm Grey													
10.2	Fine to coarse sand and gravel occasional cobble, trace to some Loose silt Grey		7	SS	11									
80.2														
11.0	Silty sand some gravel and clay occasional cobble and boulder (glacial till) Dense to very dense Grey		8	SS	112 0.2 m									
78.3														
12.9	Continued													
	*Sank under weight of rods													30 52 14 4

+3, x5: Numbers refer to 20
Sensitivity 15 \rightarrow 5 (%) STRAIN AT FAILURE
10

OFFICE REPORT ON SOIL EXPLORATION



Ontario

RECORD OF BOREHOLE No 22BI

METRIC

W.F. 108-87-07/08 LOCATION Co-ords N 012 066.5;E 361 258.0 ORIGINATED BY DJR
 DIST 2 HWY 416 BOREHOLE TYPE Hollow Stem Auger, BXL Rock Coring COMPILED BY FGH
 DATUM Geodetic DATE June 16, 1989 CHECKED BY APC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT Y	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	N VALUES			20	40	60	80	100					
78.3	Continued																
12.9	Limestone bedrock with occasional black shale interbed			RC	TCR=		78										
	Sound Grey			BXL	RQD=		77										
					SCR=		76										
75.2					96%												
16.0	End of hole						75										
	*TCR: Total Core Recovery RQD: Rock Quality Designation SCR: Solid Core Recovery																

*3, *5: Numbers refer to
Sensitivity

20
15 5 (%) STRAIN AT FAILURE
10

OFFICE REPORT ON SOIL EXPLORATION



RECORD OF BOREHOLE No 22B4

METRIC

W.P. 12B-87-07/08 LOCATION Cochrane N 5 012 131.5: E 361 206.9 ORIGINATED BY DJH
 DIST 9 HWY 416 BOREHOLE TYPE Hollow Stem Auger, BXL Rock Coring COMPILED BY FJH
 DATUM Geodetic DATE June 12, 13 & 14, 1989 CHECKED BY APC

OFFICE REPORT ON SOIL EXPLORATION

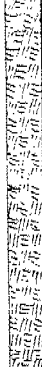
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ KN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES			20 40 60 80 100	20 40 60 80 100					
90.6	Ground Surface													
0.0	Topsoil					Seal								
0.2	Silty clay and silty sand (alluvium)		1	SS	4		90							
89.3	Very stiff Grey brown													
1.3	Sandy silt topsoil													
1.6	Silty sand trace shells		2	SS	2		89							
88.7	and organic matter (alluvium)													
1.9	Very loose Dark grey brown		3	SS	WH *		88							
						Seal								
			4	TP	PH		97						17.3	
	Silty clay with occasional sand seam		5	SS	PM		86							
	Soft to firm Grey		6	SS	PM		85							
83.6							84							
7.0	Silty Clay		7	SS	PM		83							
	Firm to stiff Grey						82							
81.3			8	SS	PM		81							
9.3	Silty clay with gravel and cobbles occasional sand seam		9	SS	>100		80							
80.5	Very stiff Grey													
10.1	Fine to coarse sand and gravel, trace to some silt		10	SS	38		79							
79.3	Dense to very dense Grey						78							
11.3	Silty sand some gravel and clay, numerous cobbles and boulders (glacial till)		11	SS	>100		77							
			12	RC	TCR=									
			13	BXL	66%									
			14	RC	TCR=									
			15	BXL	59%									
			16	RC	TCR=									
			17	BXL	50%									
			18	SS	107									
			19	RC	TCR=									
			20	BXL	78%									
76.2	Very dense Grey						76							
14.4	Continued													

+3, x⁵: Numbers refer to Sensitivity
 20
 15 \div 5 (%) STRAIN AT FAILURE
 10

RECORD OF BOREHOLE No 22B4

METRIC

W P 125-57-07/08 LOCATION Co-ords N 5 012 123.5; E 361 206.9 ORIGINATED BY DJS
 DIST 9 HWY 416 BOREHOLE TYPE Hollow Stem Auger, BXL Rock Coring COMPILED BY FJH
 DATUM Geodetic DATE June 12, 13 & 14, 1989 CHECKED BY APC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100					
76.2	Continued																
14.4	Limestone bedrock with occasional black shale interbed		17	RC BXL	TCR=100% ROD=81% SCR=87%	**	76										
			18	RC BXL	TCR=100% ROD=97% SCR=97%		75										
			19	RC BXL	TCR=100% ROD=90% SCR=98%		74										
72.4	Sound	Grey					73										
18.2	End of hole						72										
	* Sank under Weight of hammer																
	** TCR: Total Core Recovery																
	ROD: Rock Quality Designation																
	SCR: Solid Core Recovery																

OFFICE REPORT ON SOIL EXPLORATION

RECORD OF CPT 1

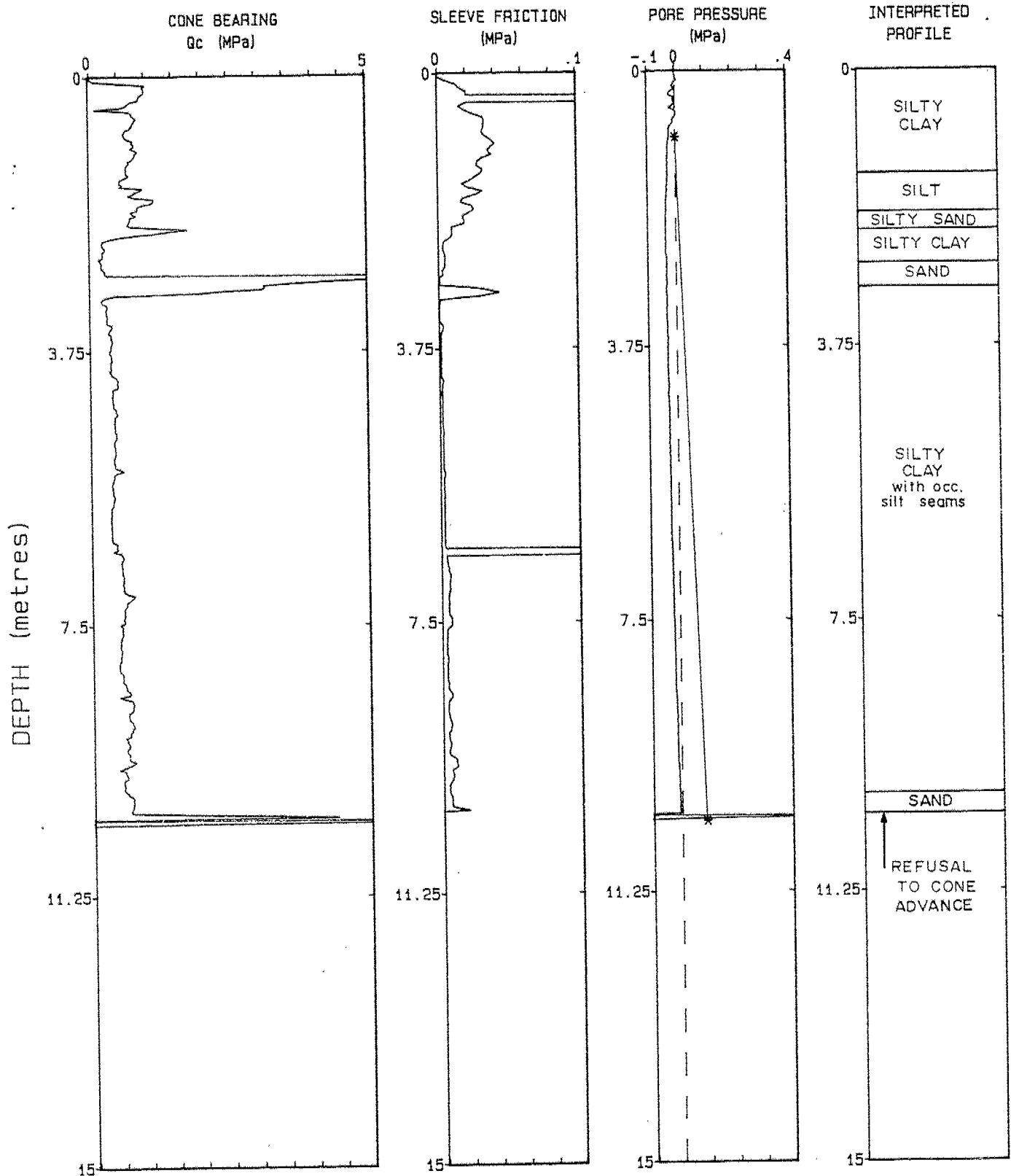
Location: See Dwg. No. 1288707/08A DATE 89-06-27

Page No: 1 / 1

Prebore Depth: 0.0m

Cone: 3015

Project No: 891-2251



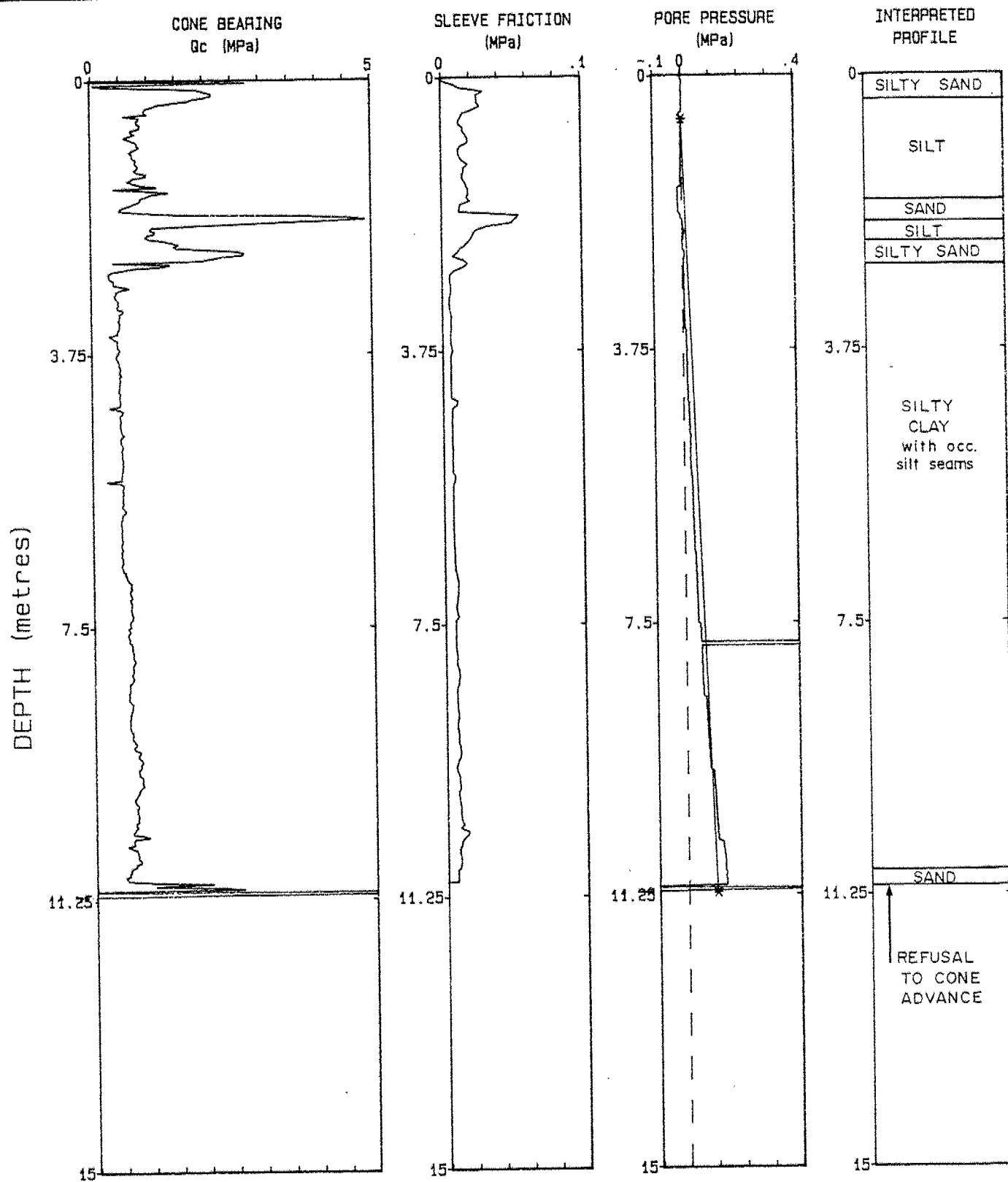
Depth Increment : .025 m

Max Depth : 10.275 m

RECORD OF CPT2

Location: See Dwg. No. 1288707/08A DATE 89-06-28
Prebore Depth: 0.0m Cone: 3015

Page No: 1 / 1
Project No: 891-2251



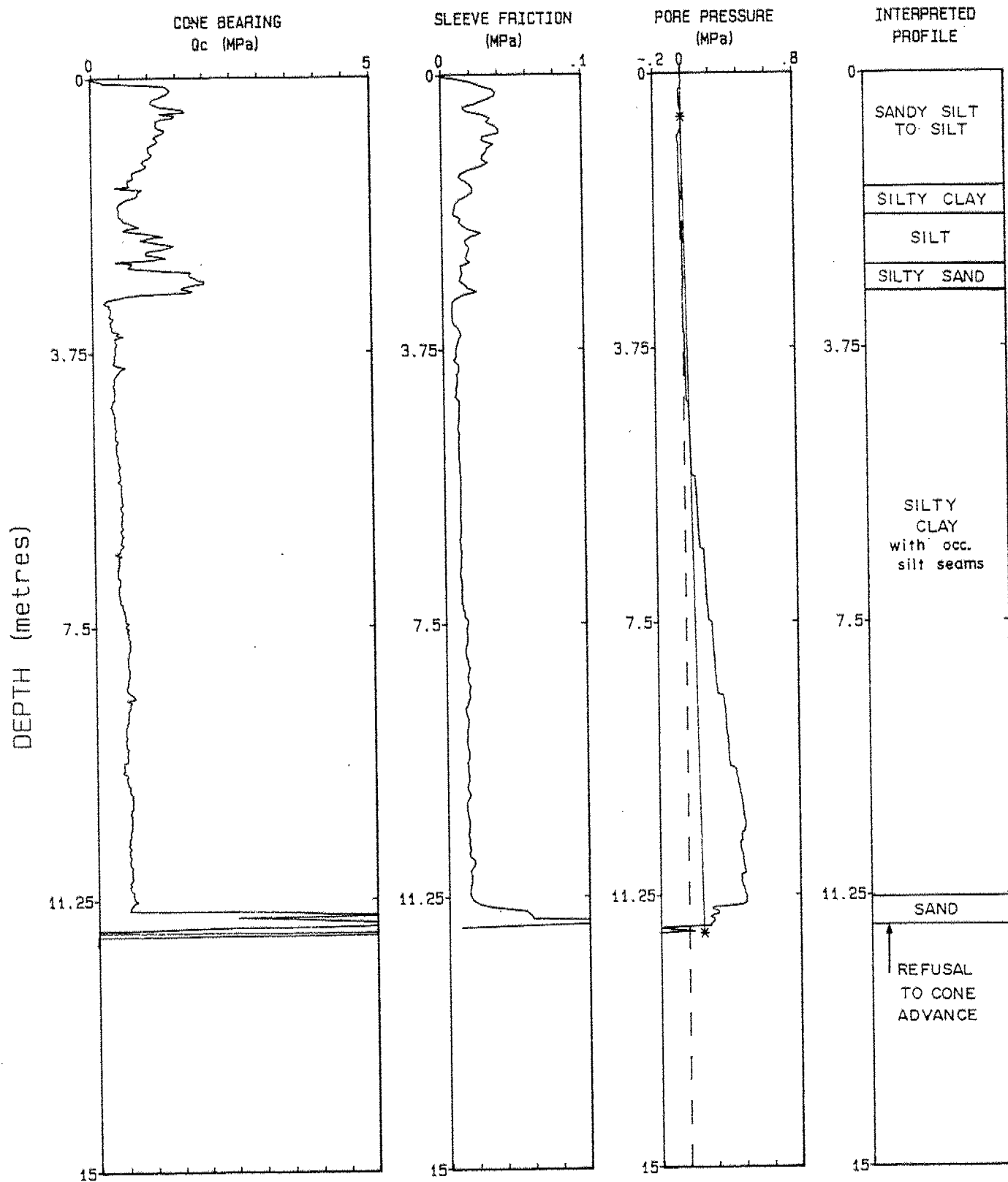
Depth Increment : .025 m

Max Depth : 11.225 m

RECORD OF CPT3

Location: See Dwg. No. 1288707/08A DATE 89-06-28
Prebore Depth: 0.0m Cone: 3015

Page No: 1 / 1
Project No: 891-2251



Depth Increment : .025 m

Max Depth : 11.775 m

RECORD OF CPT4

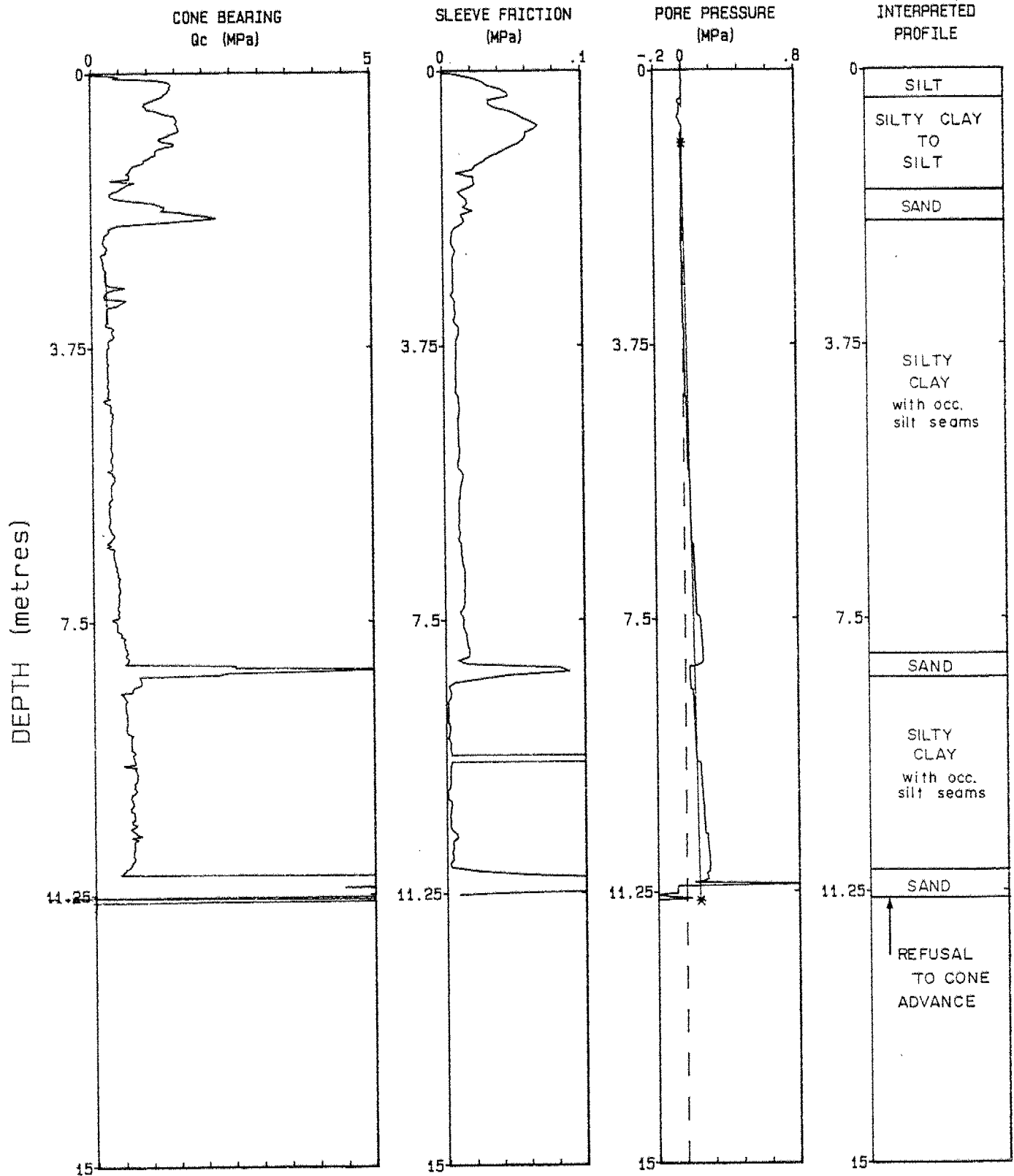
Location: See Dwg. No. 1288707/08A DATE 89-06-28

Page No: 1 / 1

Prebore Depth: 0.0m

Cone: 3015

Project No: 891-2251



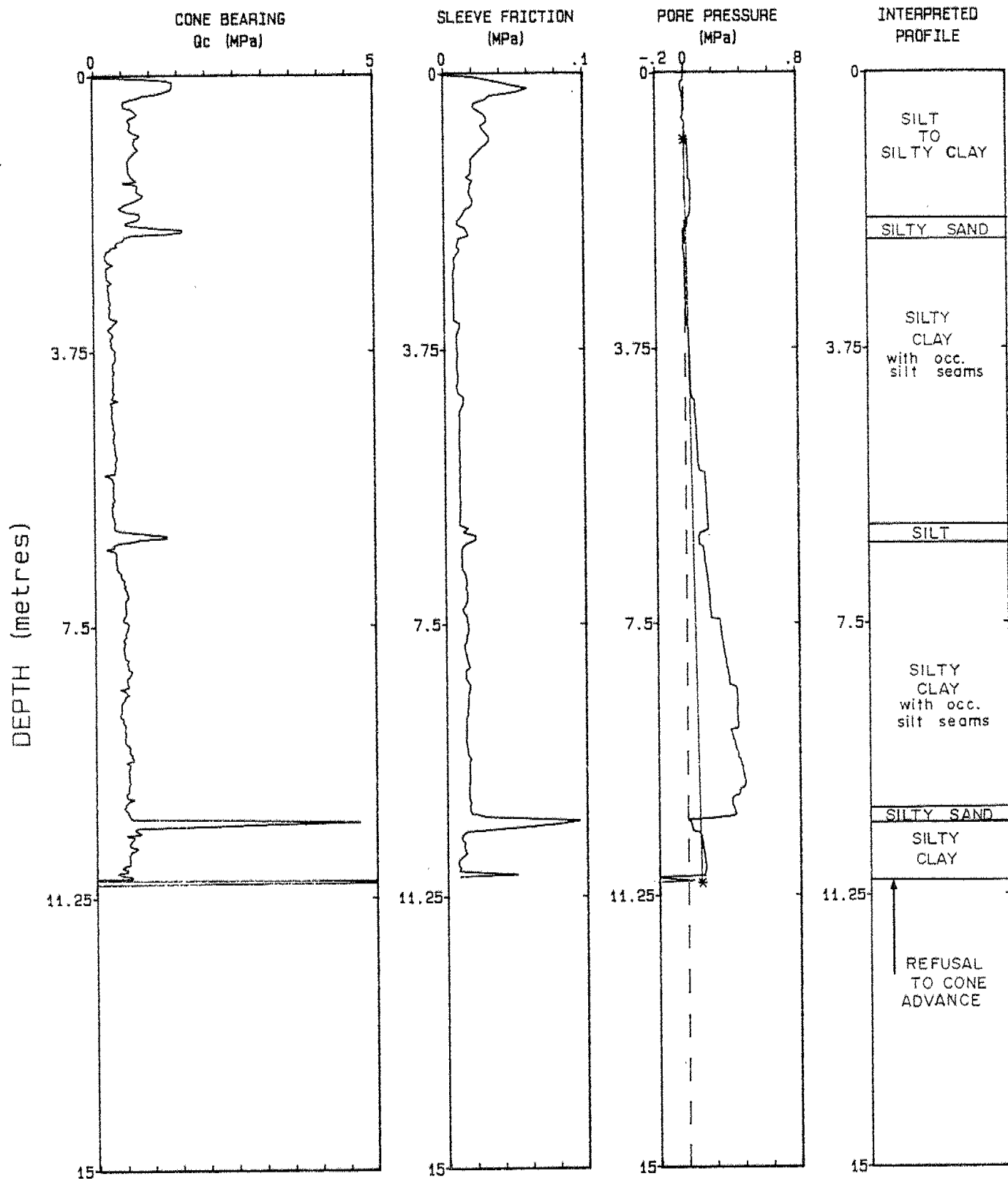
Depth Increment : .025 m

Max Depth : 11.375 m

RECORD OF CPT5

Location: See Dwg. No. 1288707/08A DATE 89-06-28
Prebore Depth: 0.0m Cone: 3015

Page No: 1 / 1
Project No: 891-2251



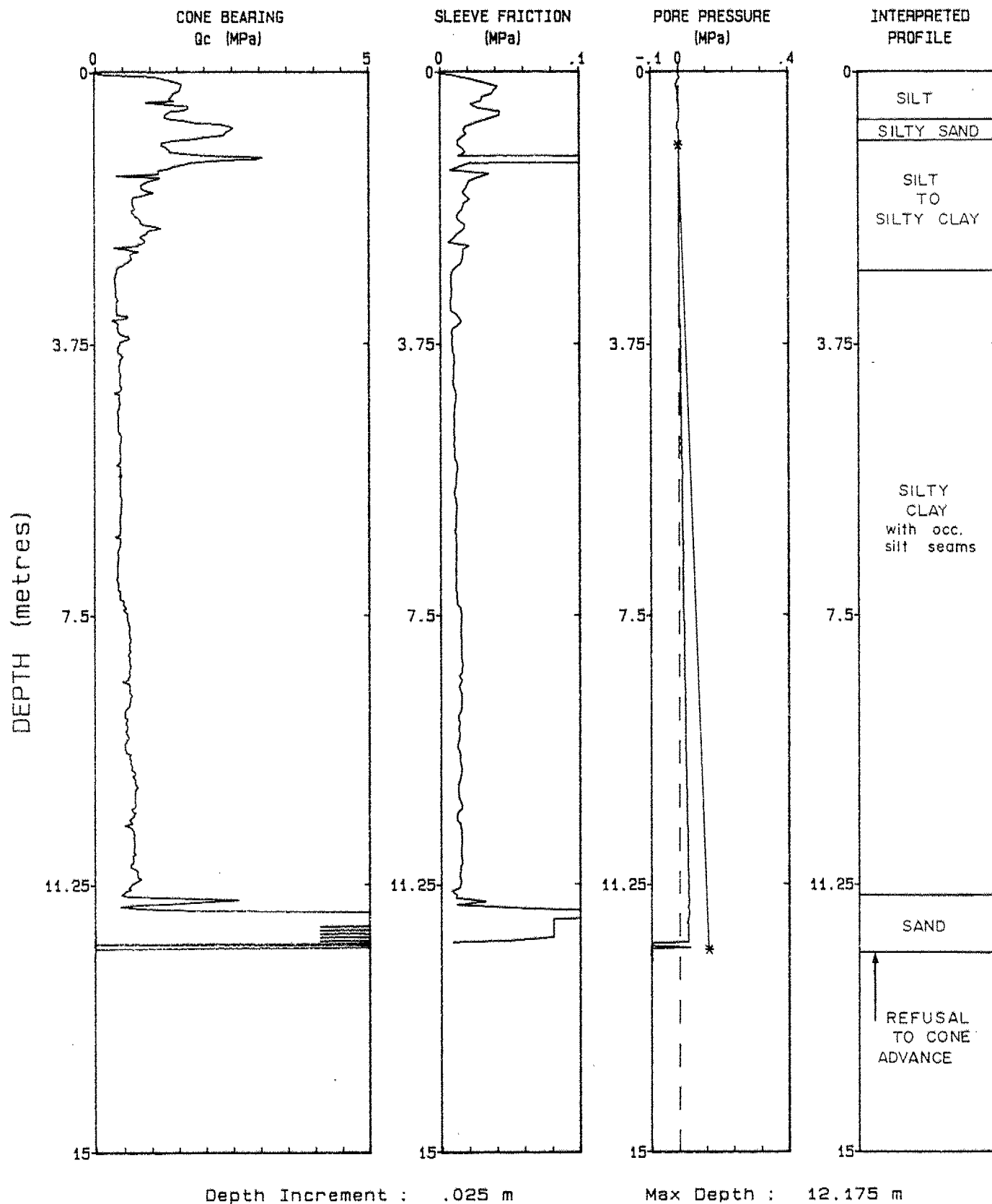
Depth Increment : .025 m

Max Depth : 11.075 m

RECORD OF CPT6

Location: See Dwg. No. 1288707/08A DATE 89-06-28
Prebore Depth: 0.0m Cone: 3015

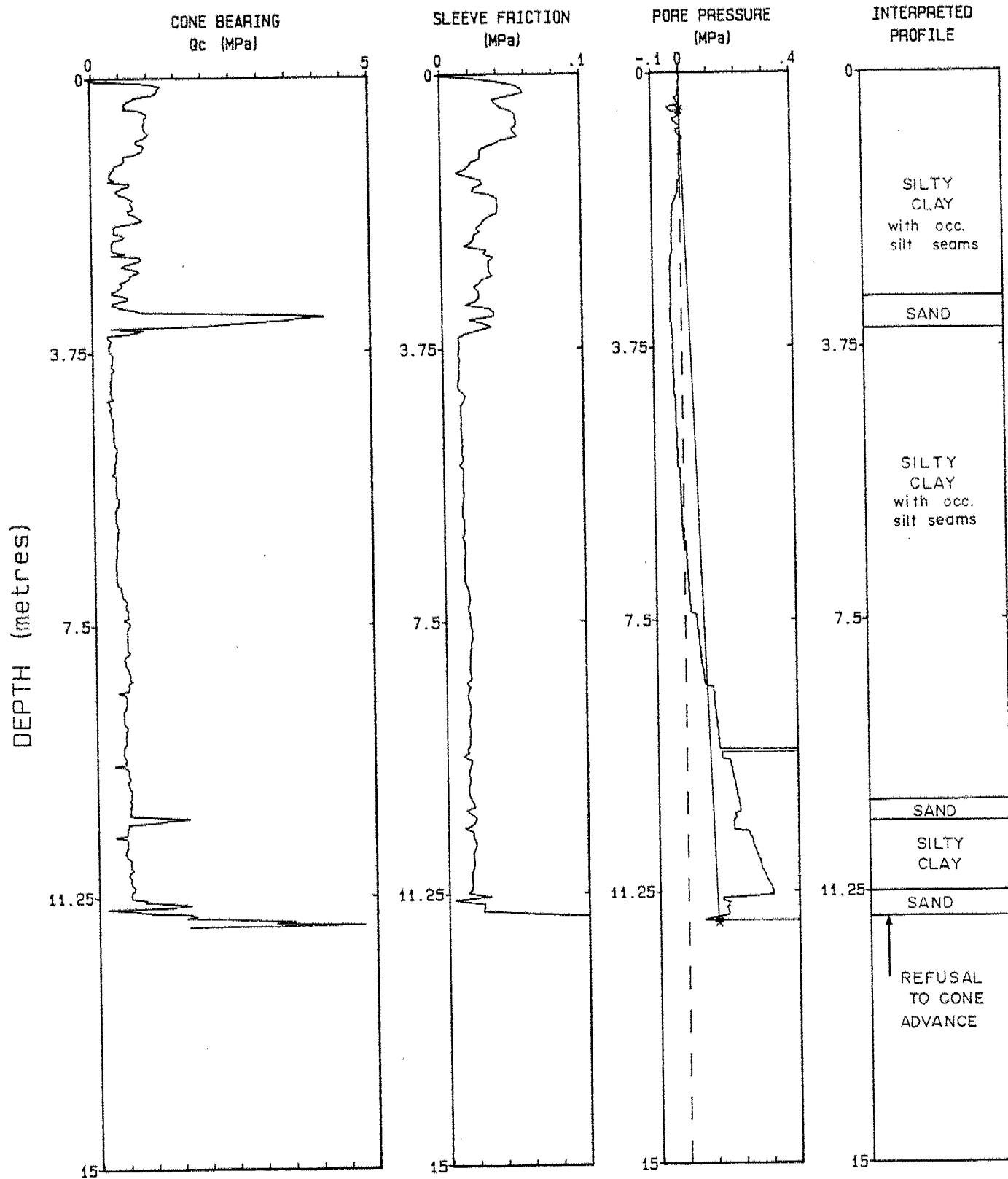
Page No: 1 / 1
Project No: 891-2251



RECORD OF CPT7

Location: See Dwg. No. 1288707/08A DATE 89-06-28
Prebore Depth: 0.0m Cone: 3015

Page No: 1 / 1
Project No: 891-2251



Depth Increment : .025 m

Max Depth : 11.675 m

RECORD OF CPT8

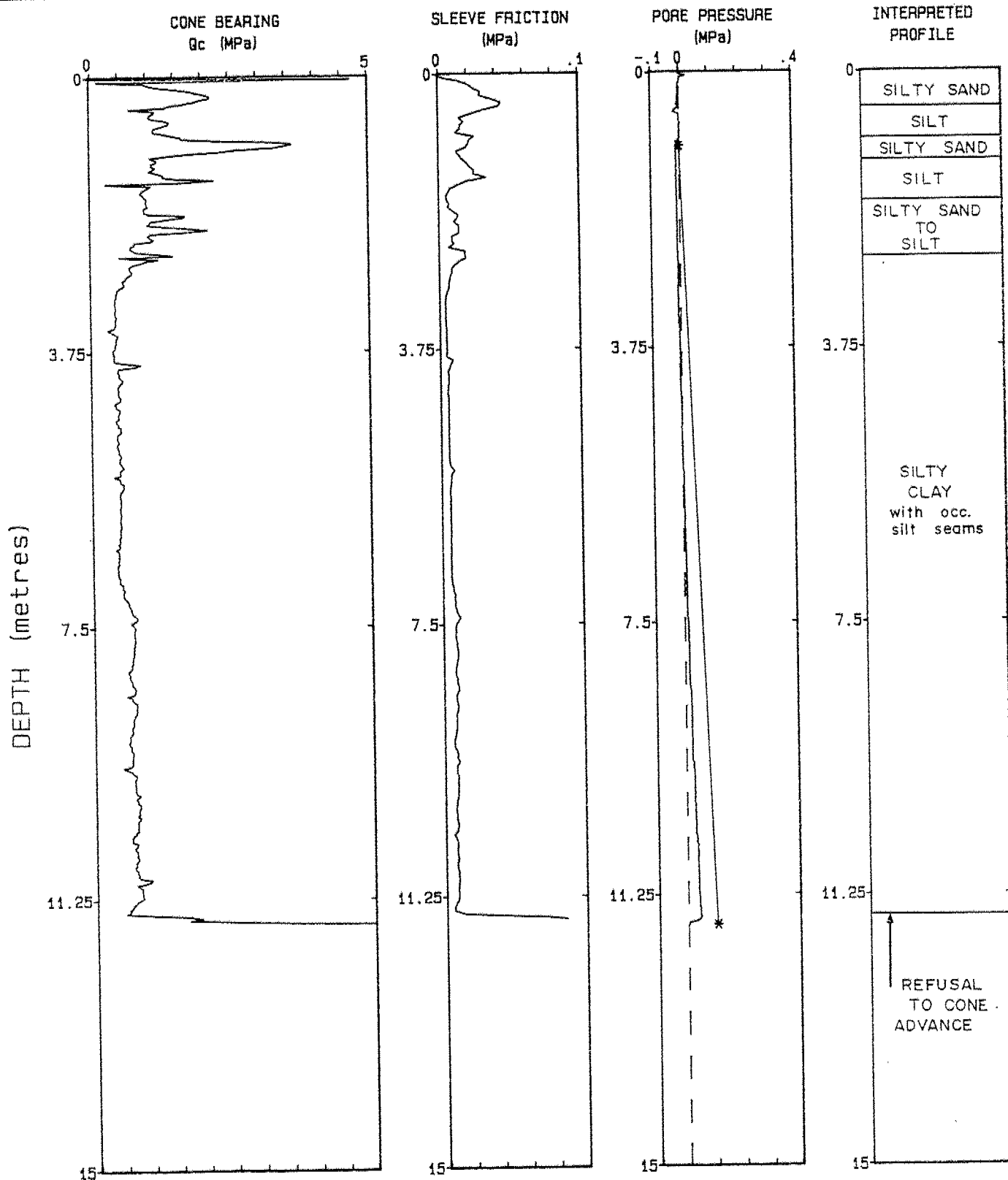
Location: See Dwg. No. 1288707/08A DATE 89-06-28

Page No: 1 / 1

Prebore Depth: 0.0m

Cone: 3015

Project No: 891-2251



Depth Increment : .025 m

Max Depth : 11.675 m

RECORD OF CPT9

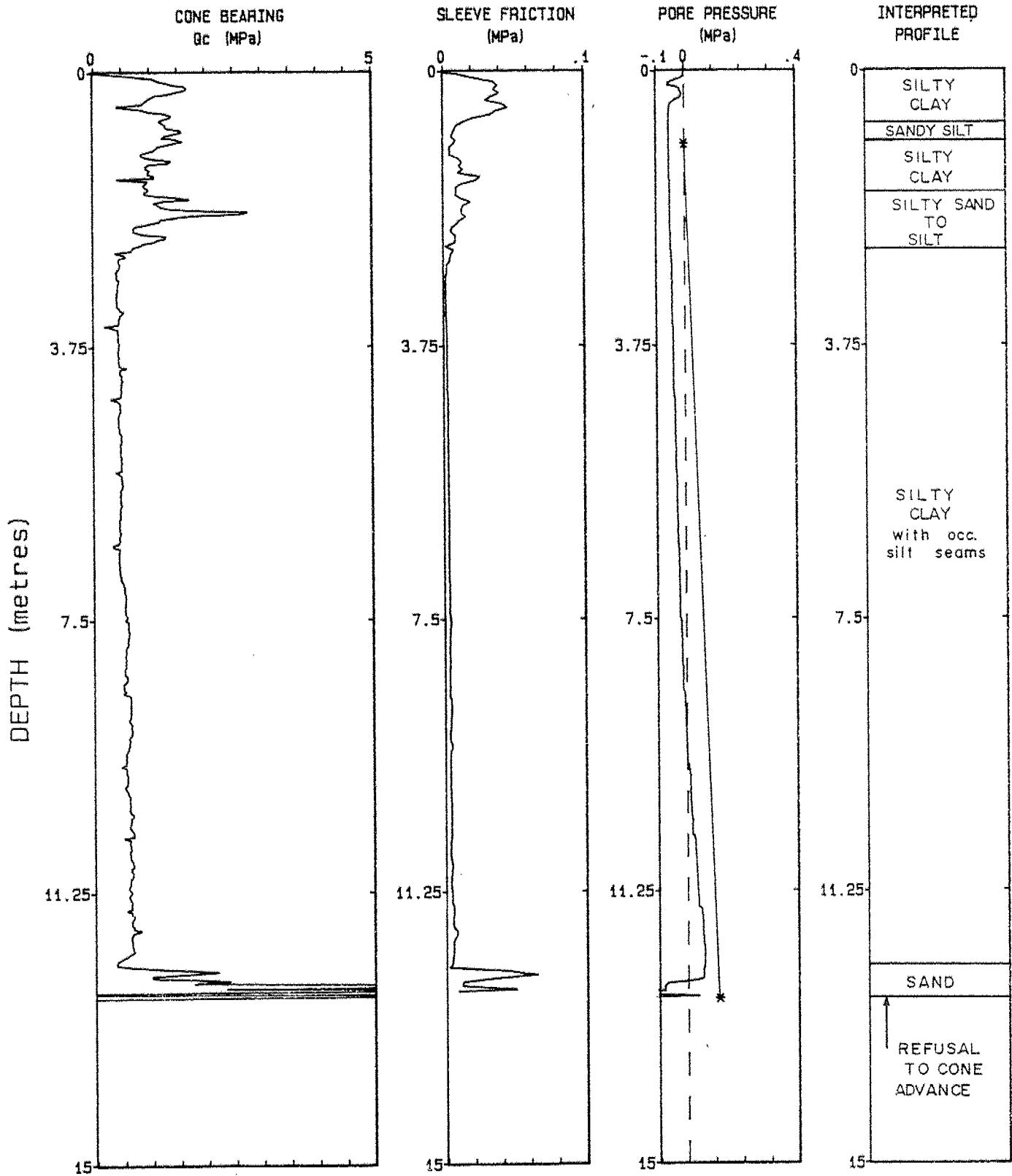
Location: See Dwg. No. 1288707/08A DATE 89-06-29

Page No: 1 / 1

Prebore Depth: 0.0m

Cone: 3015

Project No: 891-2251



Depth Increment : .025 m

Max Depth : 12.725 m

RECORD OF CPT 10

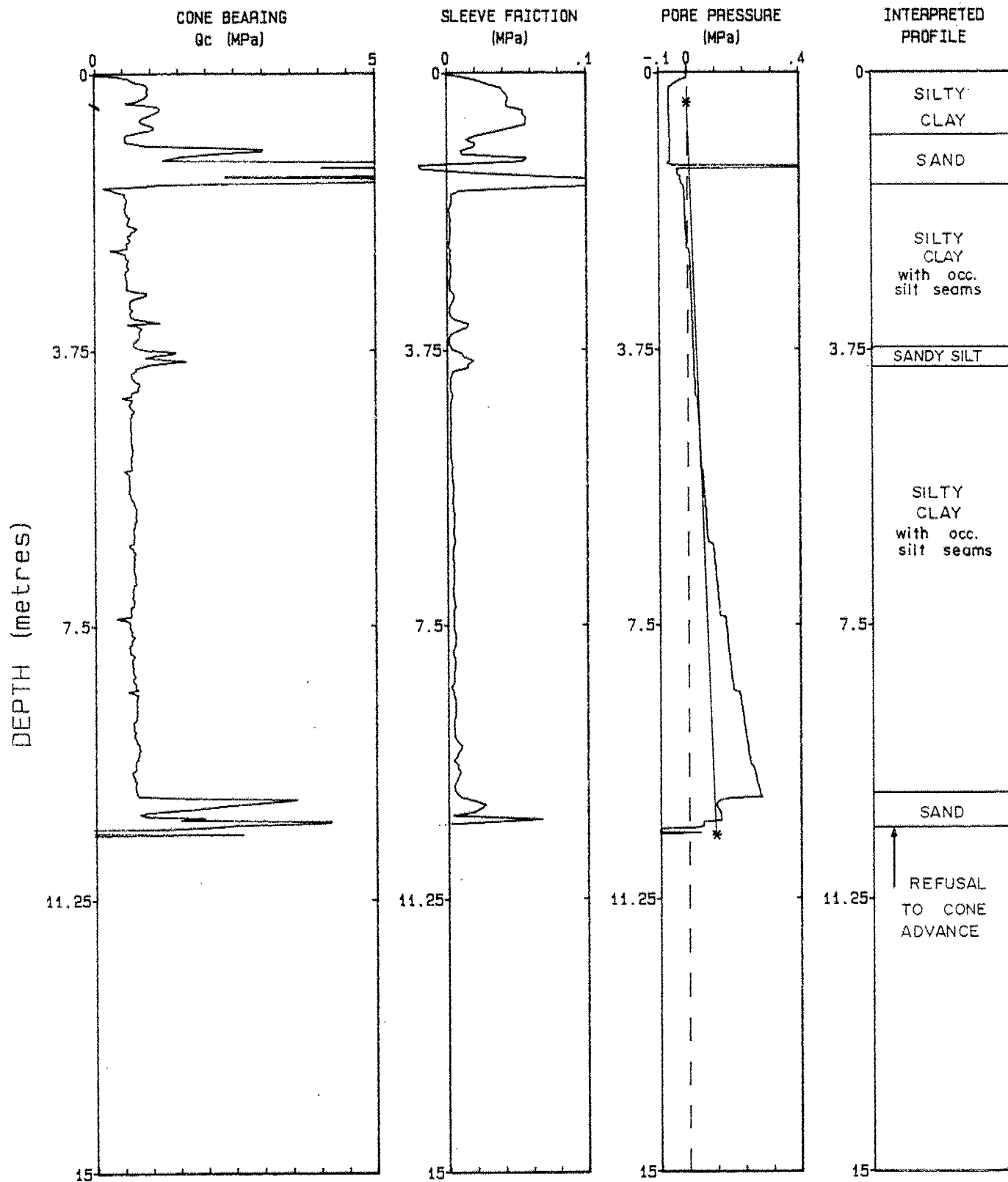
Location: See Dwg. No. 1288707/08A DATE 89-06-29

Page No: 1 / 1

Prebore Depth: 0.0m

Cone: 3015

Project No: 891-2251



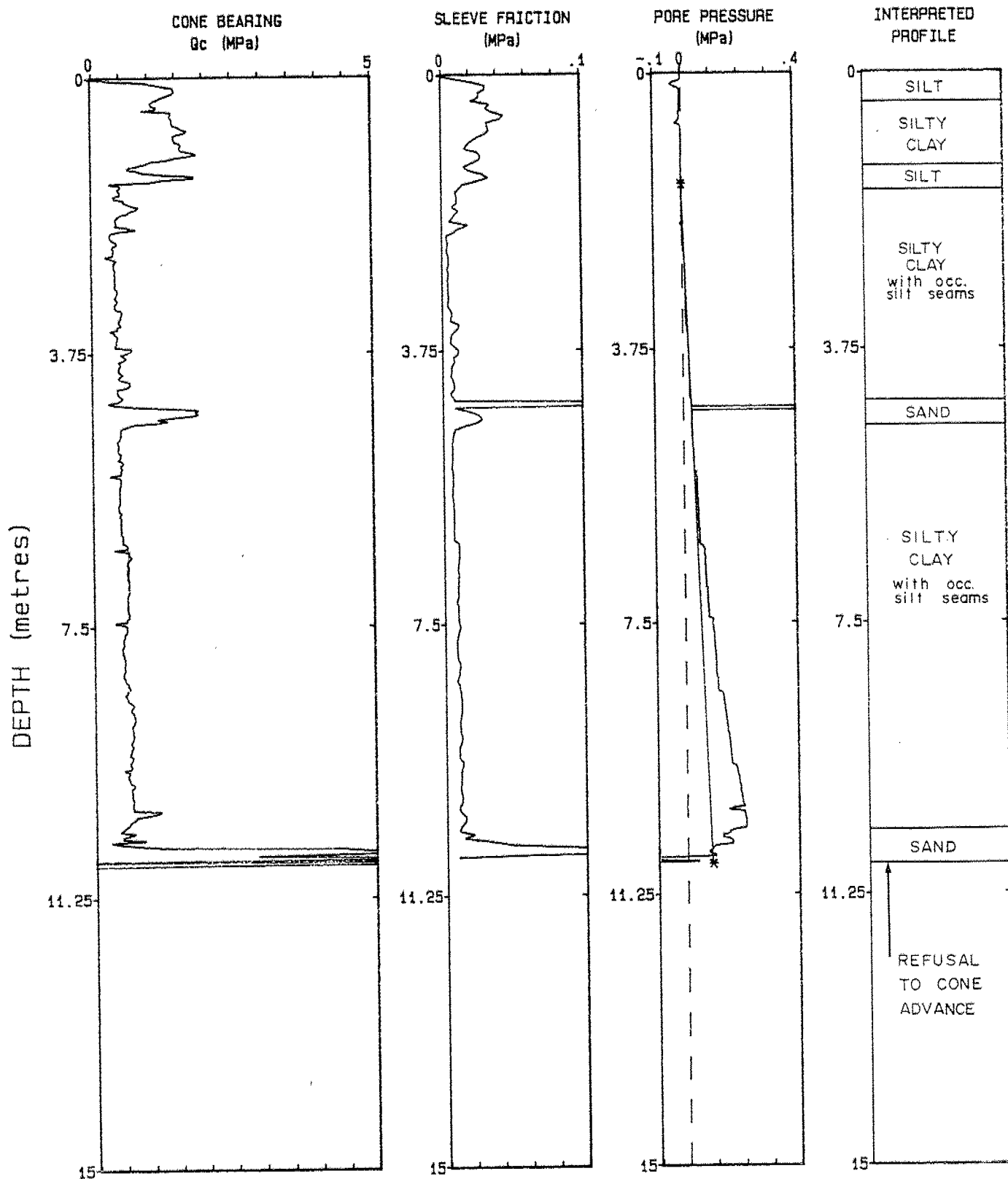
Depth Increment : .025 m

Max Depth : 10.375 m

RECORD OF CPT 11

Location: See Dwg. No. 1288707/08A DATE 89-06-29
Prebore Depth: 0.0m Cone: 3015

Page No: 1 / 1
Project No: 891-2251



Depth Increment : .025 m

Max Depth : 10.825 m

RECORD OF CPT 12

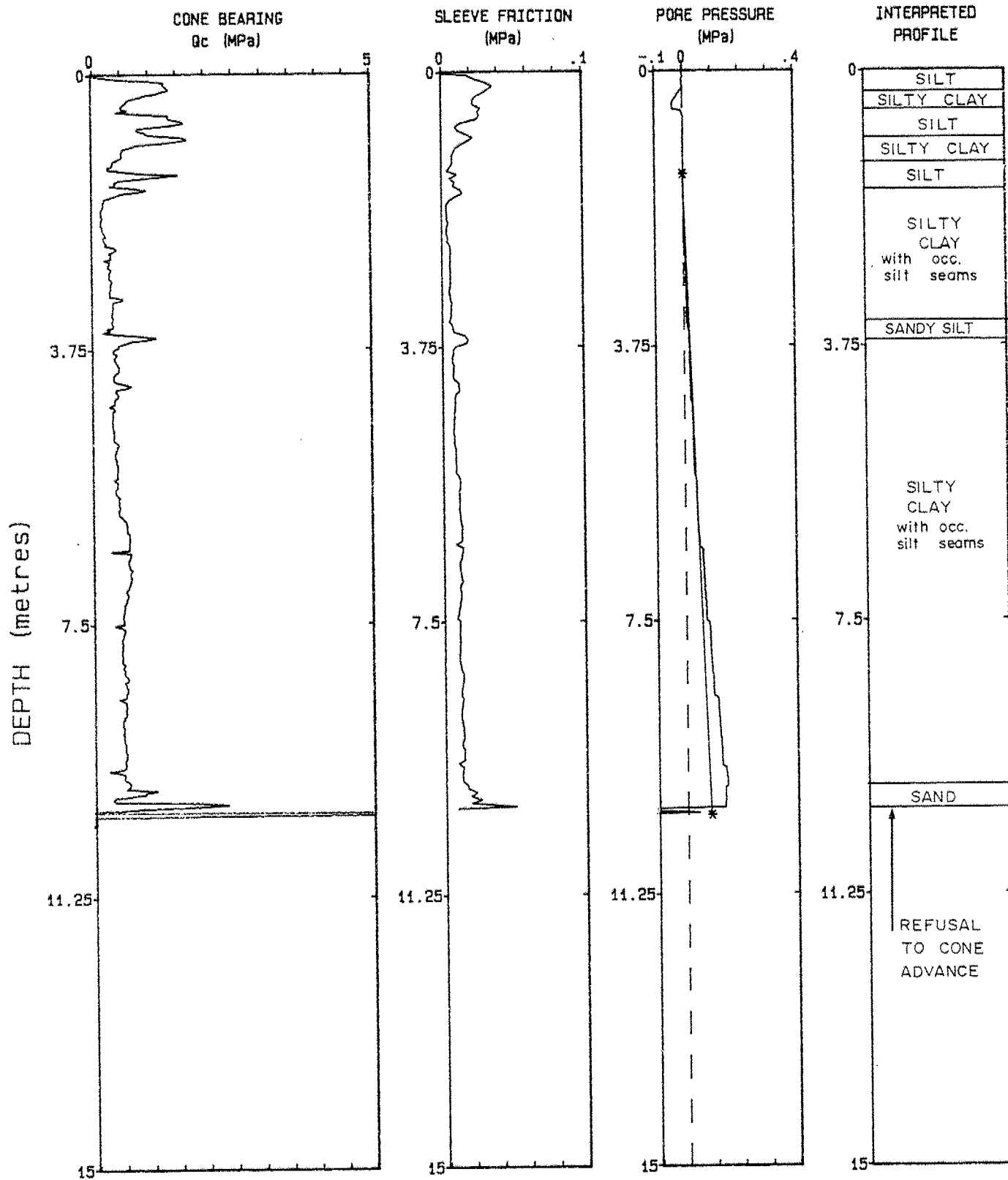
Location: See Dwg. No. 1288707/08A DATE 89-06-29

Page No: 1 / 1

Prebore Depth: 0.0m

Cone: 3015

Project No: 891-2251



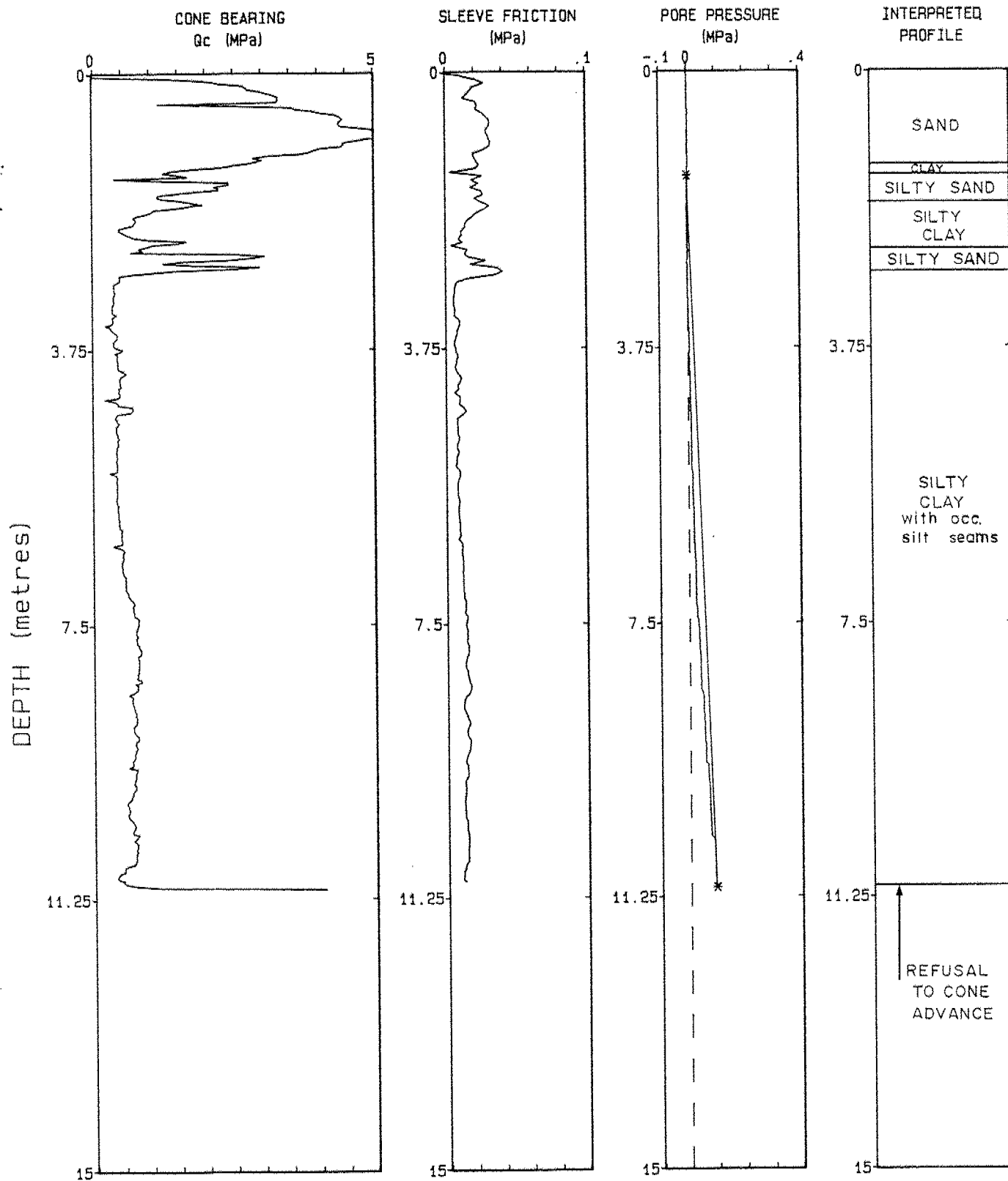
Depth Increment : .025 m

Max Depth : 10.175 m

RECORD OF CPT 13

Location: See Dwg. No. 1288707/08A DATE 89-06-29
Prebore Depth: 0.0m Cone: 3015

Page No: 1 / 1
Project No: 891-2251



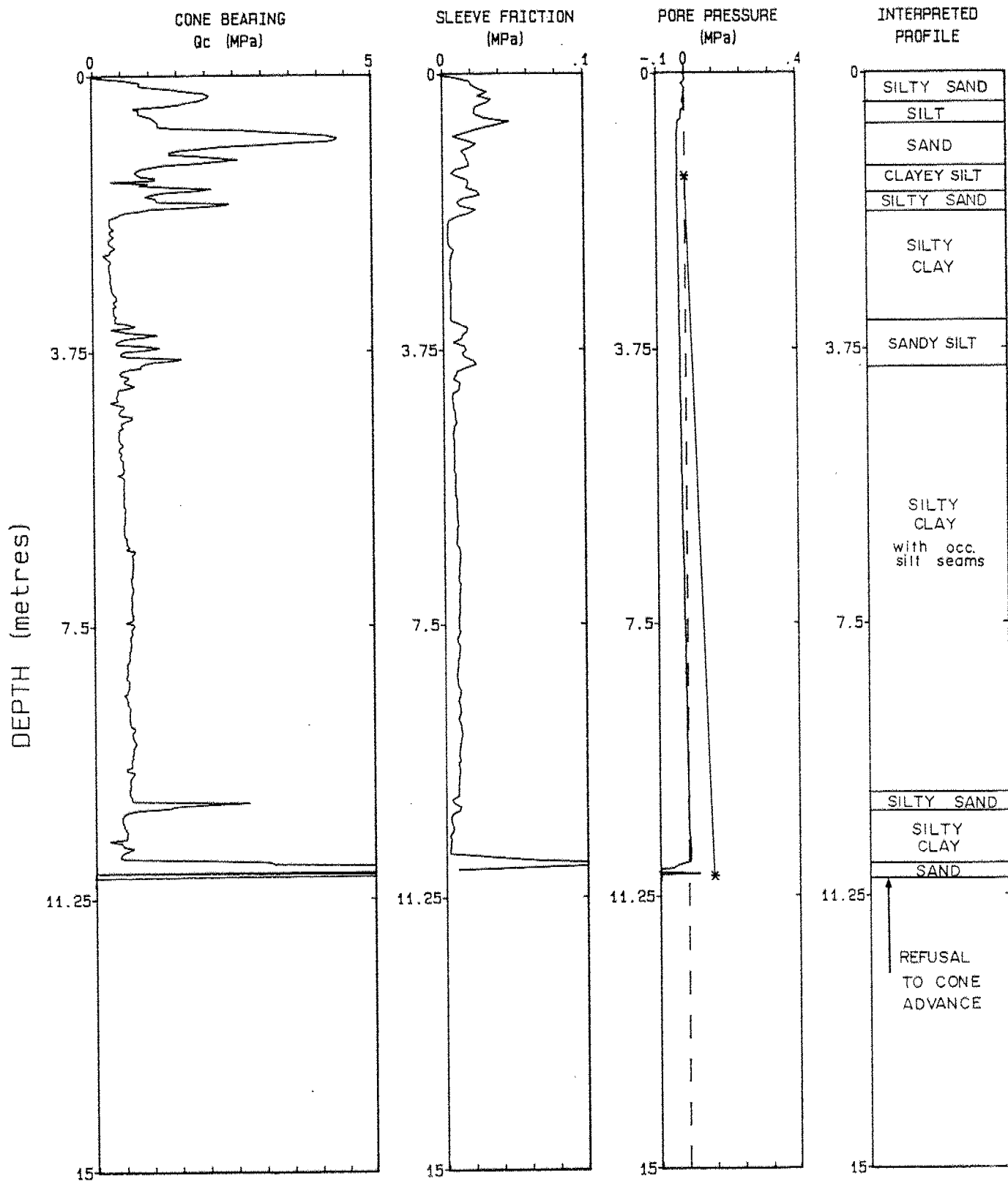
Depth Increment : .025 m

Max Depth : 11.125 m

RECORD OF CPT 14

Location: See Dwg. No. 1288707/08A DATE 89-06-29
 Prebore Depth: 0.0m Cone: 3015

Page No: 1 / 1
 Project No: 891-2251



Depth Increment : .025 m

Max Depth : 10.975 m

RECORD OF CPT 15

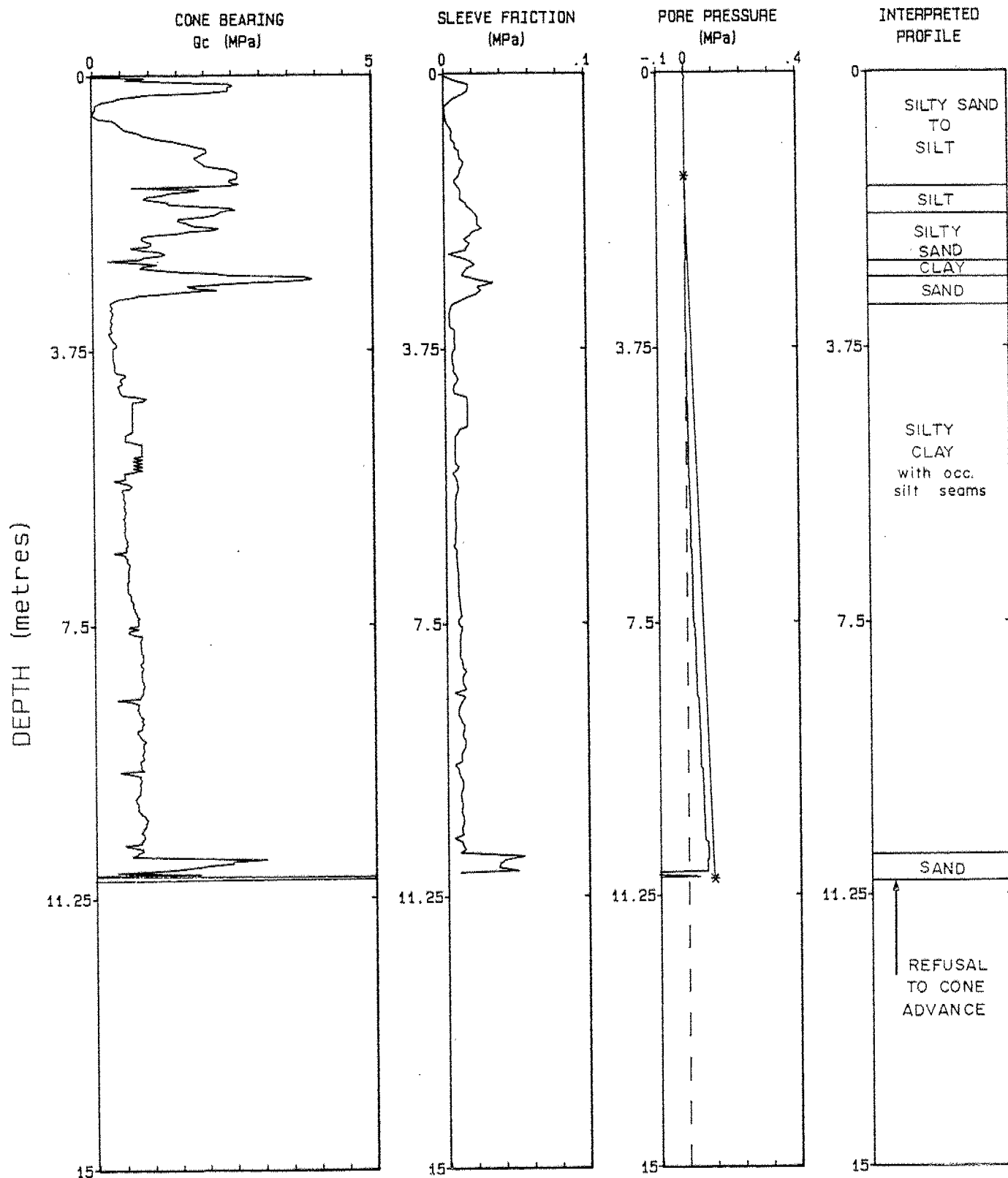
Location: See Dwg. No. 1288707/08A DATE 89-06-30

Page No: 1 / 1

Prebore Depth: 0.0m

Cone: 3015

Project No: 891-2251



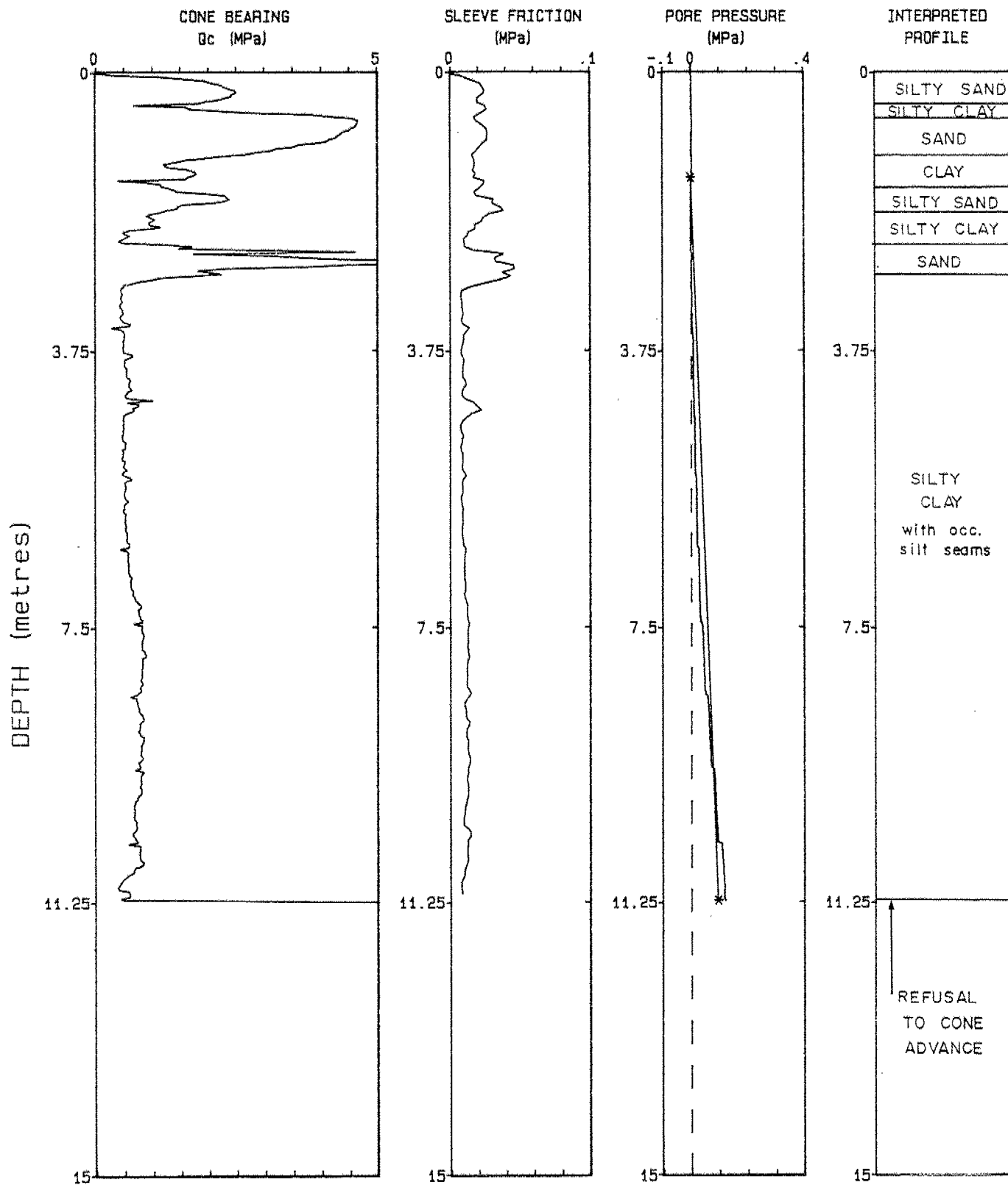
Depth Increment : .025 m

Max Depth : 11.025 m

RECORD OF CPT 16

Location: See Dwg. No. 1288707/08A DATE 89-06-30
 Prebore Depth: 0.0m Cone: 3015

Page No: 1 / 1
 Project No: 891-2251



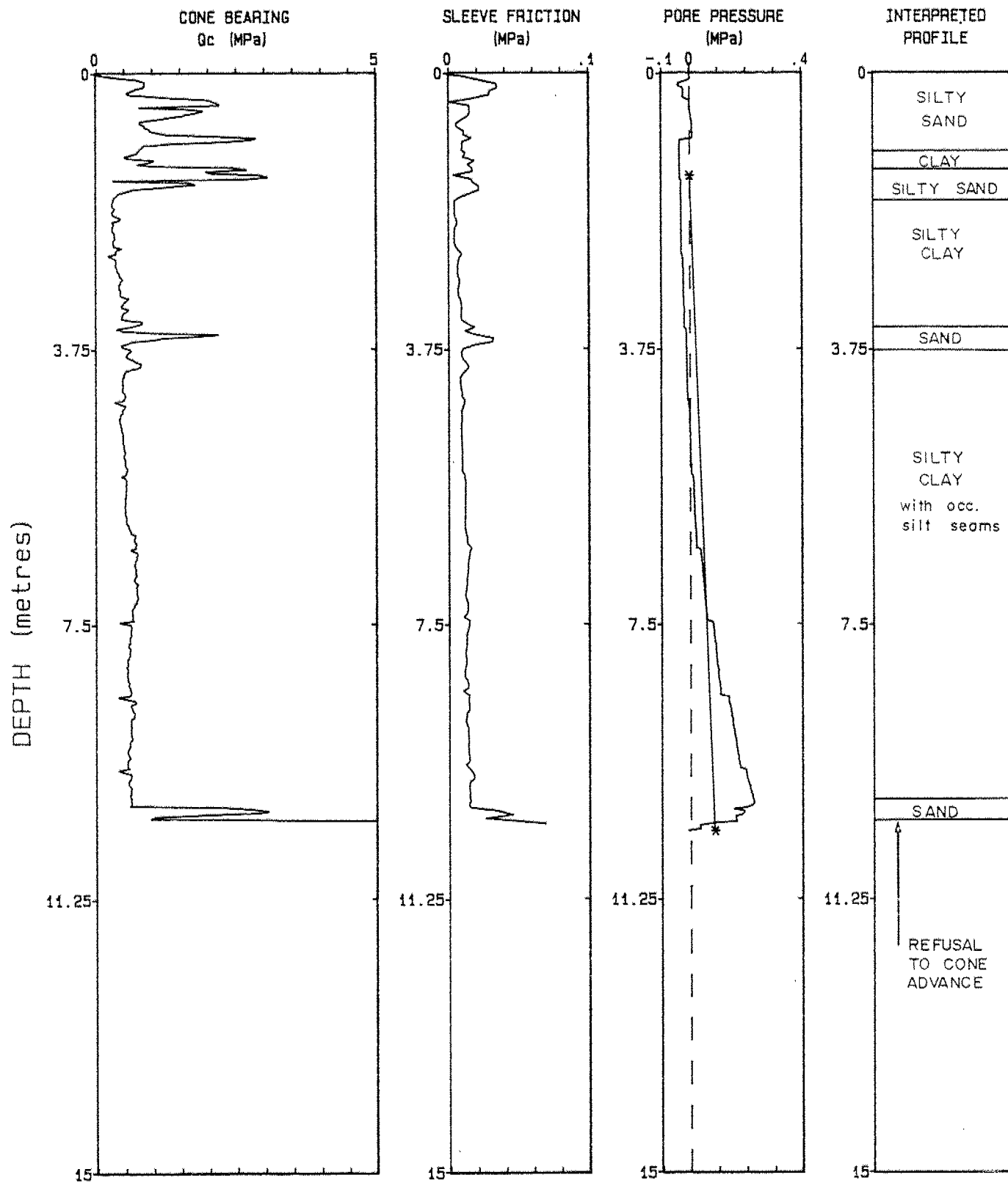
Depth Increment : .025 m

Max Depth : 11.225 m

RECORD OF CPT 17

Location: See Dwg. No. 1288707/08A DATE 89-06-30
Prebore Depth: 0.0m

Page No: 1 / 1
Cone: 3015
Project No: 891-2251

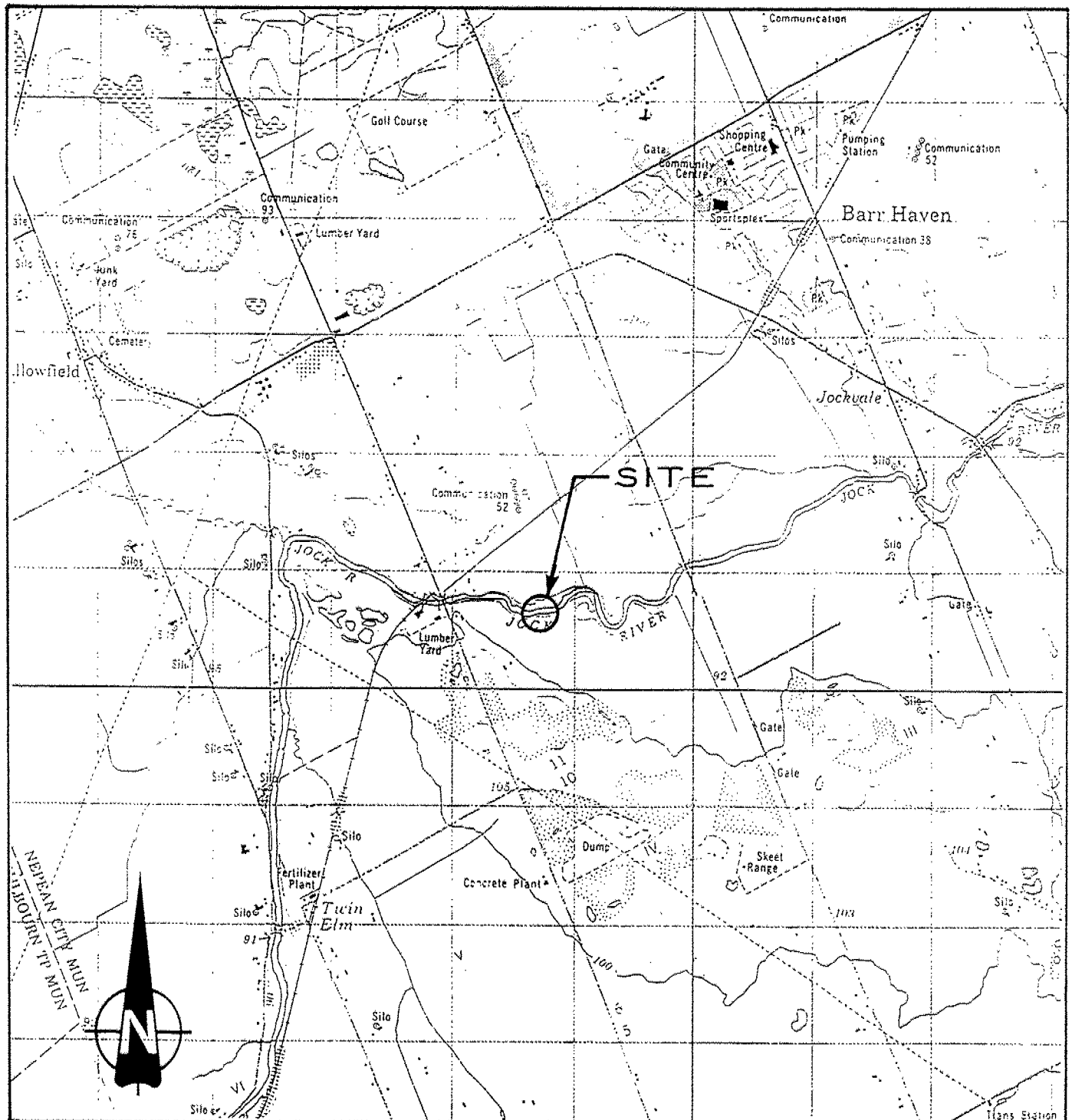


Depth Increment : .025 m

Max Depth : 10.325 m

KEY PLAN

FIGURE 1



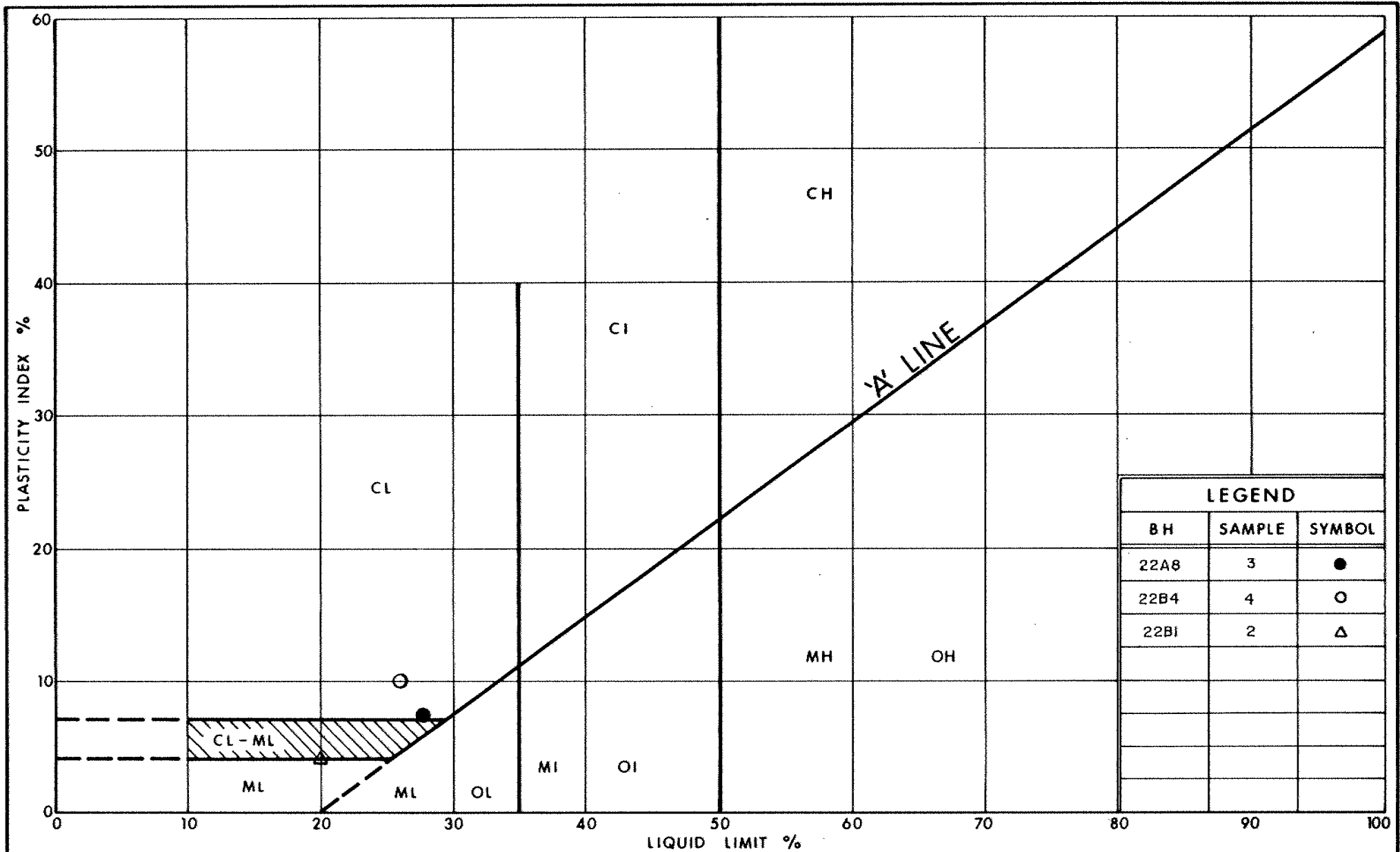
SCALE
1 : 50,000

SPECIAL NOTE
THIS DRAWING IS TO BE READ IN CONJUNCTION
WITH ACCOMPANYING REPORT

Date JUNE 30, 1989
Project 891-2251

Golder Associates

Drawn JC
Chkd. RC

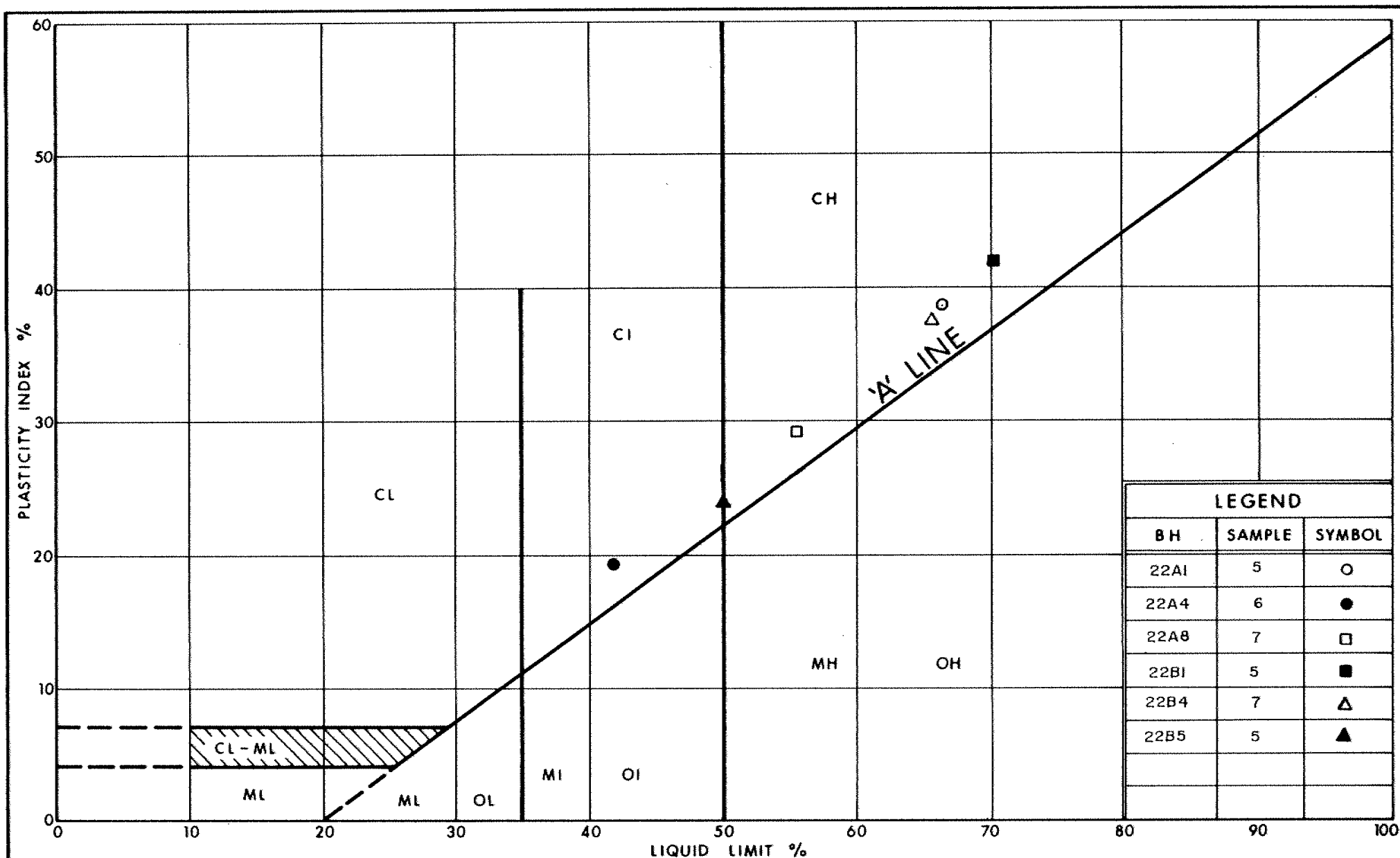


Ministry of
Transportation and
Communications

PLASTICITY CHART UPPER SILTY CLAY

FIG No 2

W P 128-87-07/08



Ontario

Ministry of
Transportation and
Communications

PLASTICITY CHART LOWER SILTY CLAY

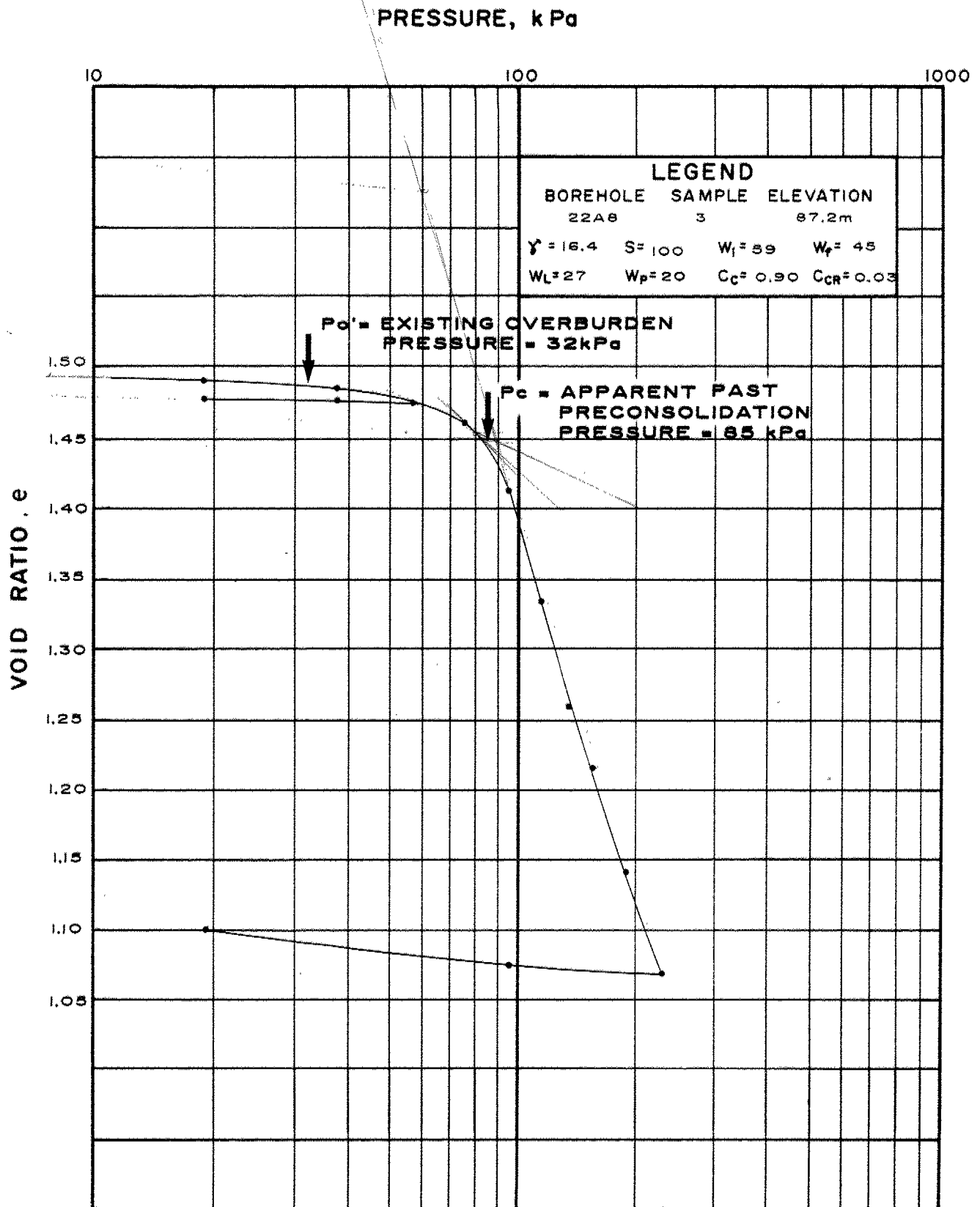
FIG No 3

W P 128-87-07/08

VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

FIGURE 4

WP 128-87-07/08

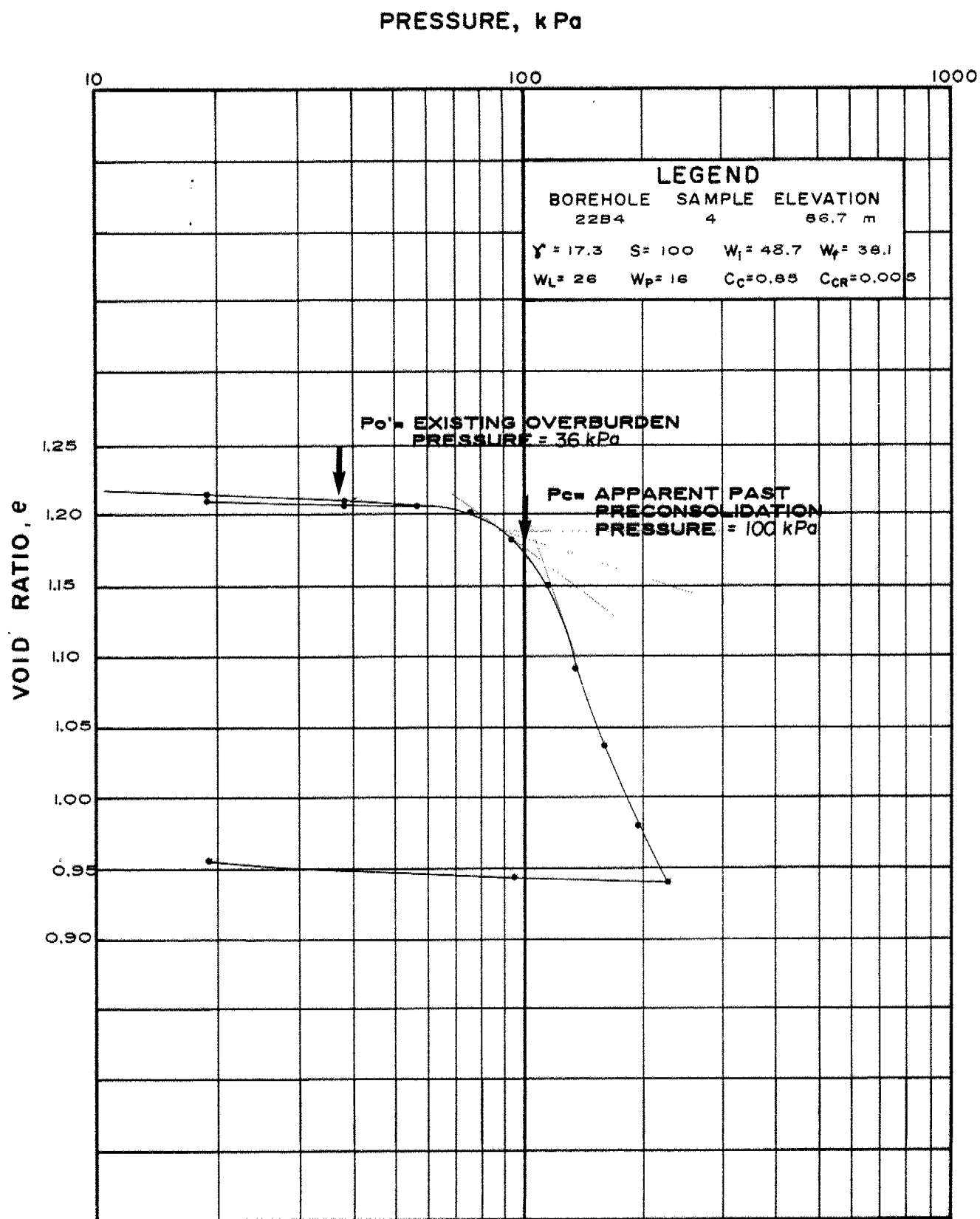


Golder Associates

VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

FIGURE 5

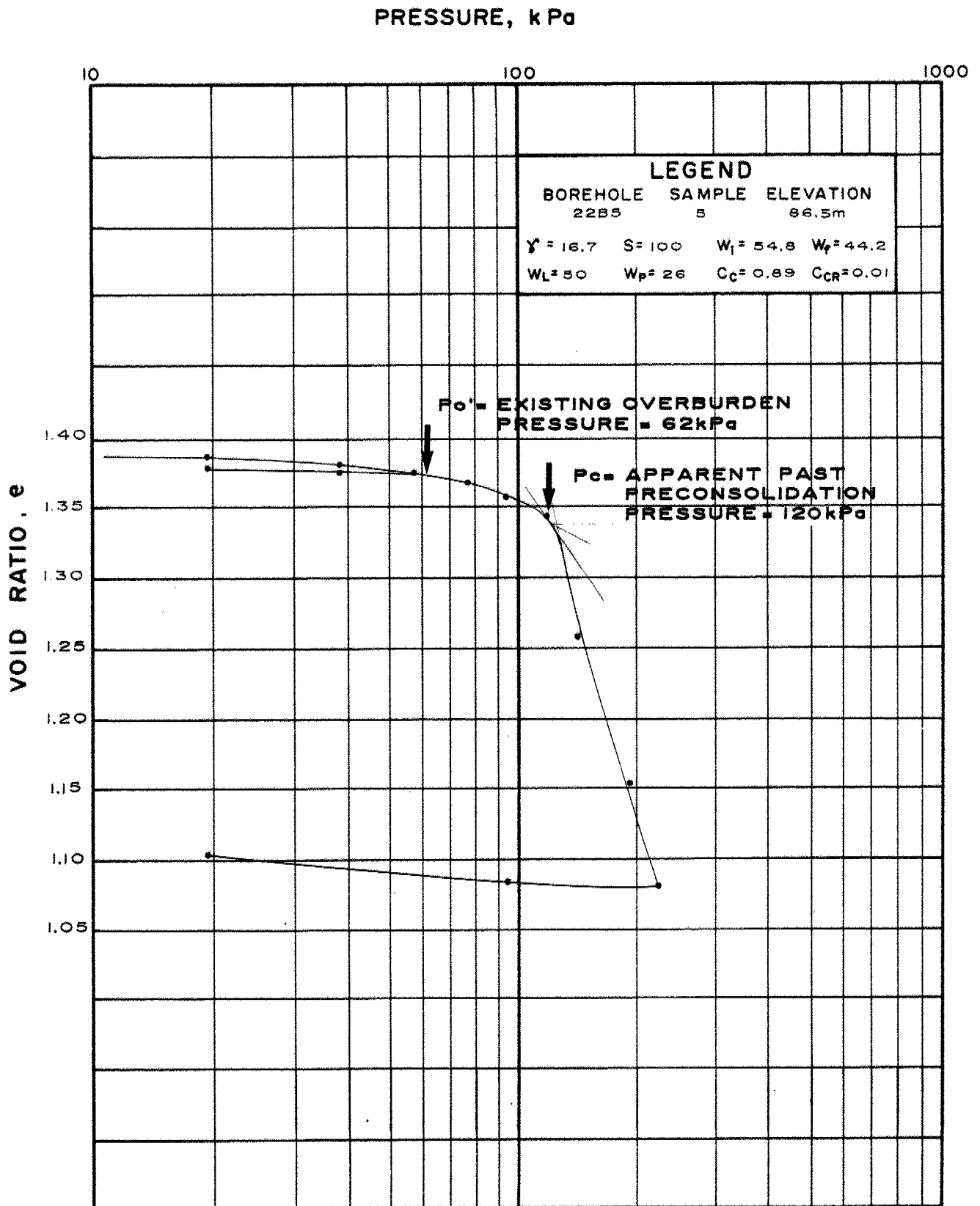
WP 128-87-07/08



VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

FIGURE 6

WP 128-87-07/08



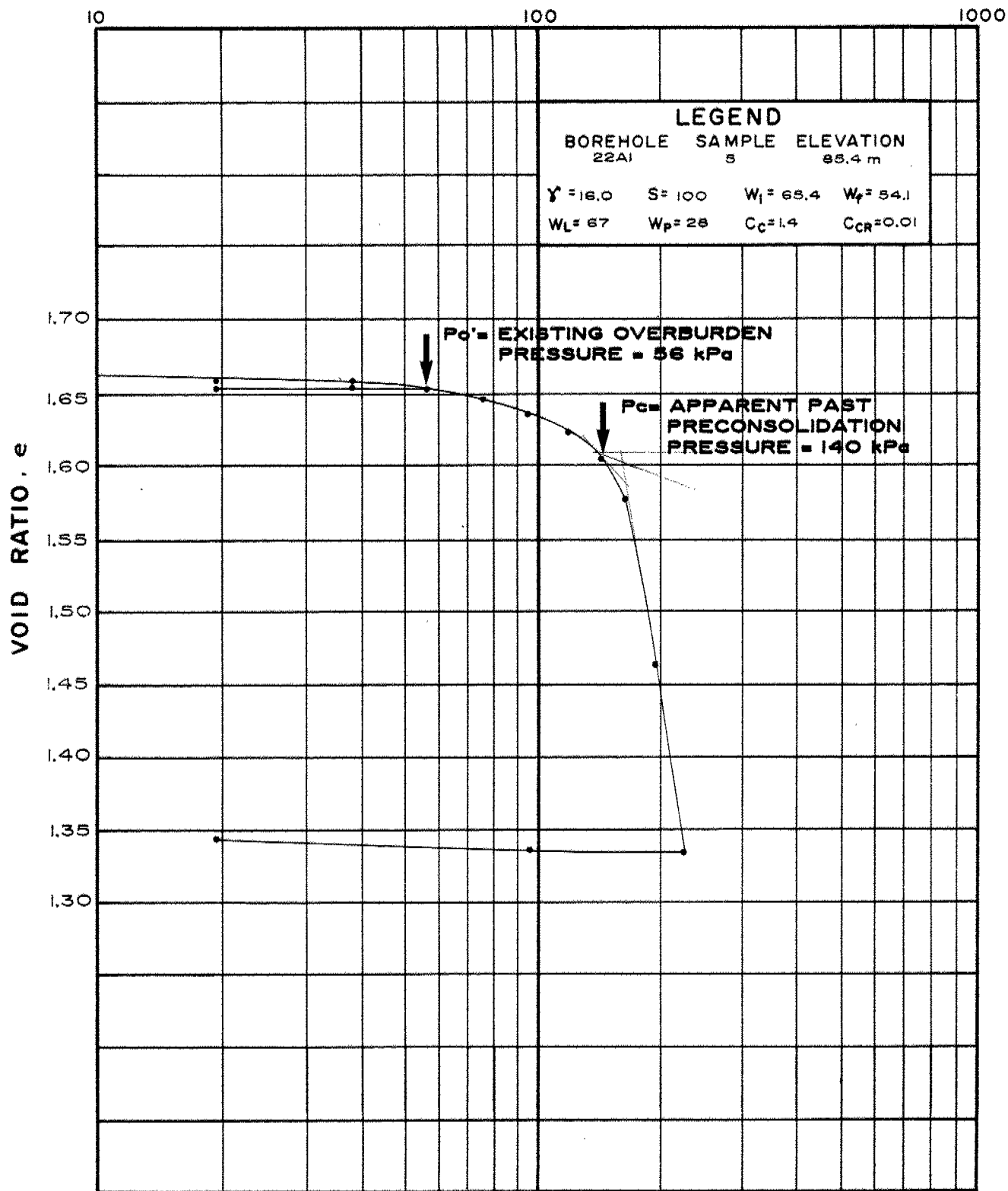
Golder Associates

VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

FIGURE 7

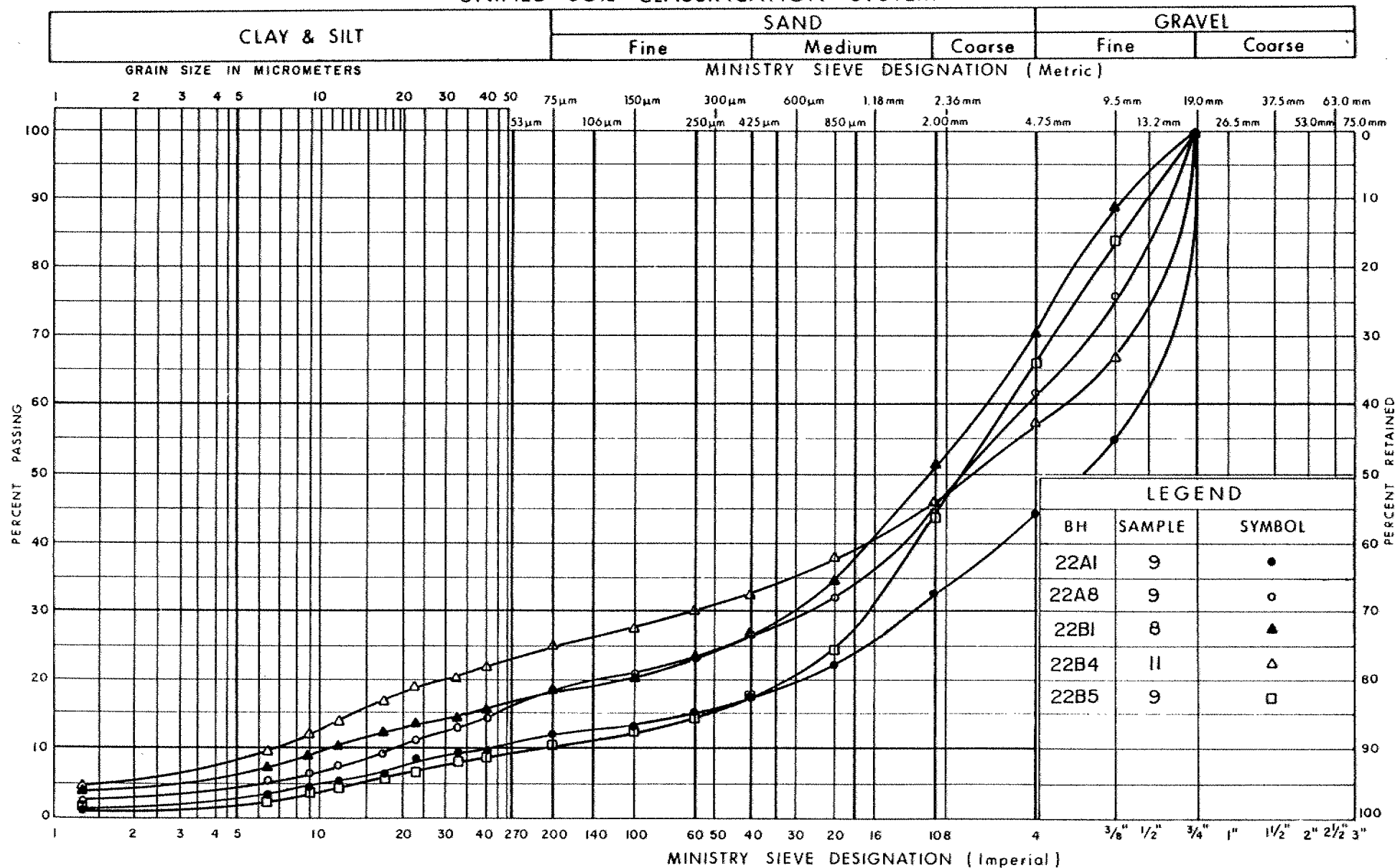
WP 128-87-07/08

PRESSURE, kPa



Golder Associates

UNIFIED SOIL CLASSIFICATION SYSTEM



Ministry of
Transportation

GRAIN SIZE DISTRIBUTION
SILTY SAND, with gravel, trace to some clay

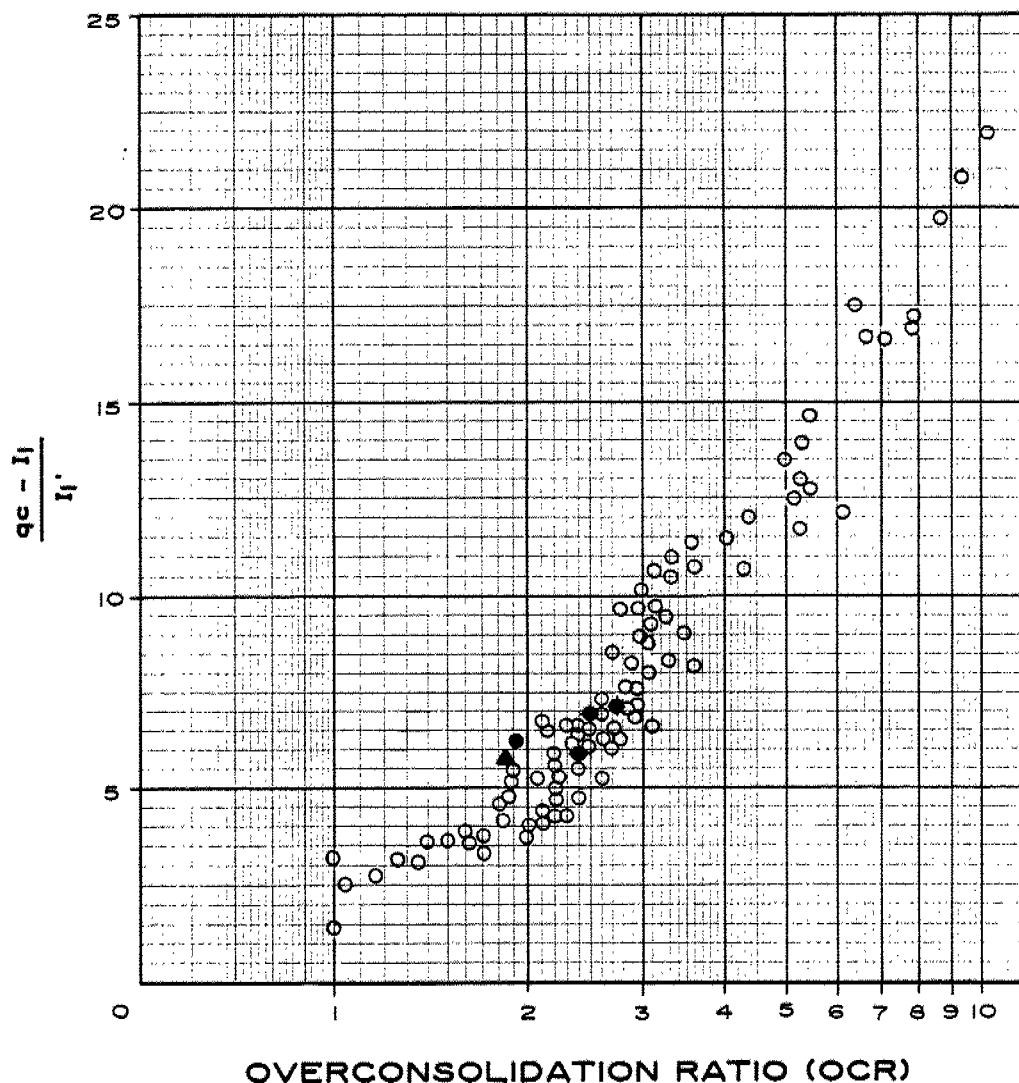
FIG No 8

W P 128-87-07/08

$\frac{qc - I_1}{I_1'}$ vs. OCR CORRELATION

FIGURE 9

WP 128-87-07/08



LEGEND

- PRESENT INVESTIGATION
- ▲ MTO BOREHOLE 89-1
- DATA FROM PREVIOUS INVESTIGATIONS BY GOLDER ASSOCIATES

Date AUG. 24, 1989
Project 691-2251

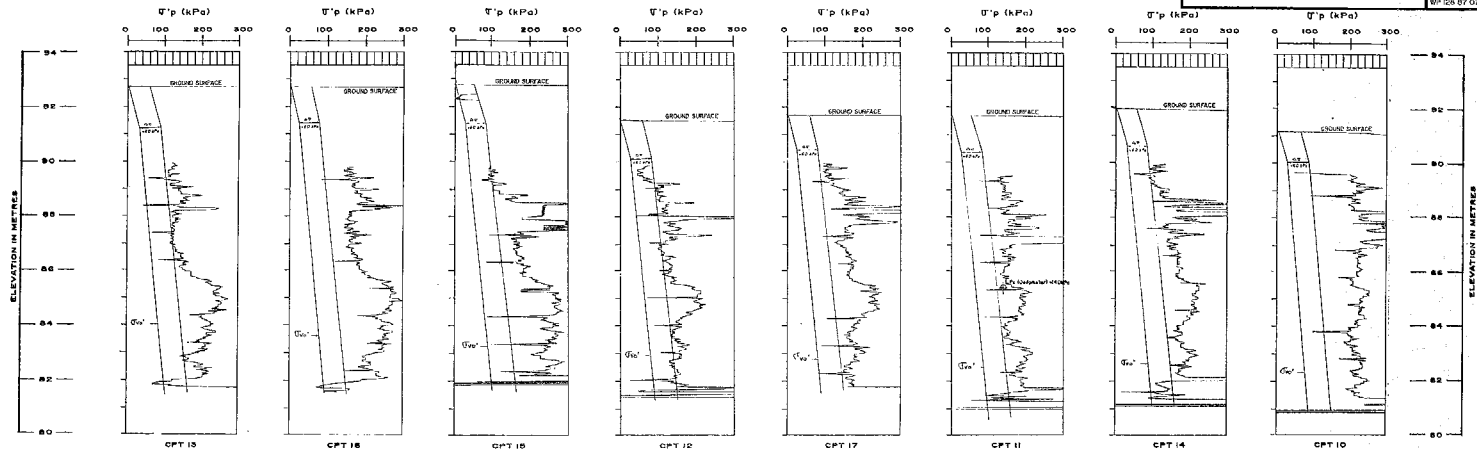
Golder Associates

Drawn JC
Chkd. AS

OVERSIZE DRAWING

SUMMARY OF PRECONSOLIDATION
PRESSURE PROFILES
SOUTH SIDE OF JOCK RIVER

FIGURE 10
WP 126 67 07/06



VERTICAL SCALE 1:80

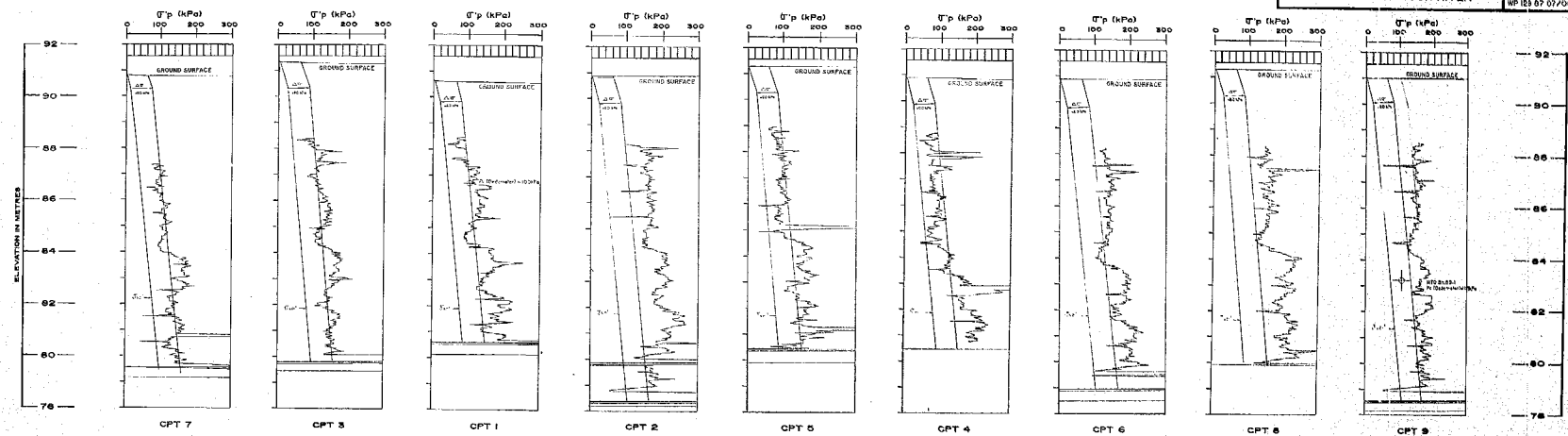
Date: AUG. 28, 1988
Project: 88-1-10001

Golder Associates

Drawn: JC
Checked: CND

SUMMARY OF PRECONSOLIDATION
PRESSURE PROFILES
NORTH SIDE OF JOCK RIVER

FIGURE II
WP 123.07.07/08



VERTICAL SCALE 1:60

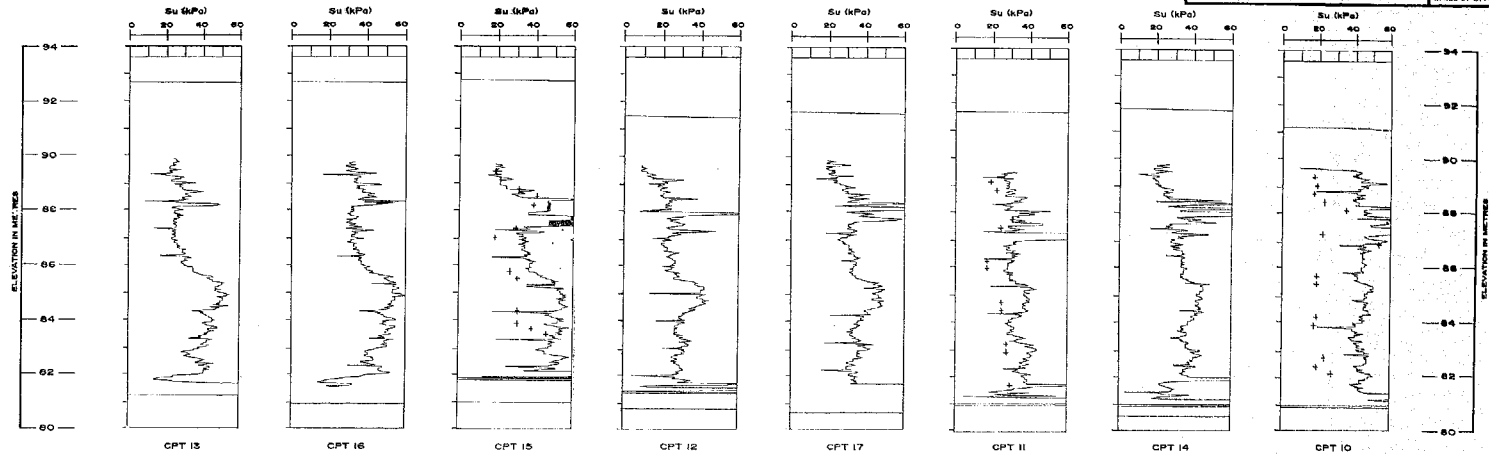
Drawn: NMS, SS, 10/88
Project: 820-0001

Gokier Associates

City: ...

SUMMARY OF UNDRAINED
SHEAR STRENGTH PROFILES
SOUTH SIDE OF JOCK RIVER

FIGURE 12
WP 128 87 07/08



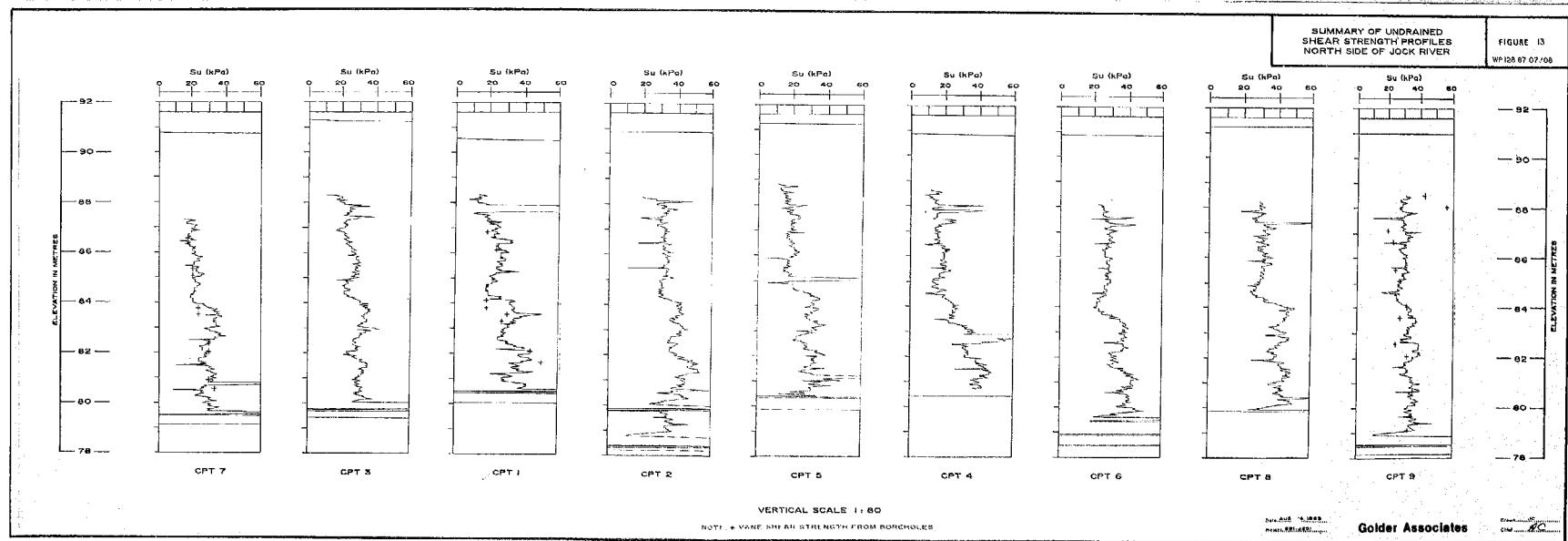
VERTICAL SCALE 1:60

NOTE: + VANE SHEAR STRENGTH FROM BOREHOLES

Date: 01/04/2009
Project: 891-8251

Golder Associates

Drawn: JC
CWD



METRIC

DIENSIONS ARE IN METRES
AND/OR MILLIMETRES UNLESS
OTHERWISE SHOWN. STATIONS
IN KILOMETRES & METRES

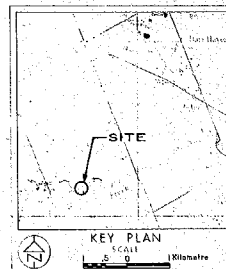
CONT No
WP No 128-87-07/08



JOCK RIVER

SHEET

Golder Associates Ltd.



LEGEND

- Bore Hole
- ⋯ Static Cone Penetration Test (Cone)
- ⊕ Bore Hole & Cone
- ⊕ Blows/0.3m (5th Pen Test, 475 lbf/blow)
- ⊕ W.L. at 100% of Liquid Limit

No.	ELEVATION	NORTH	EAST
22A1	90.2m	5 012 070.8	361 313.1
22A4	90.6m	5 012 128.0	361 258.3
22A8	90.8m	5 012 164.7	361 219.7
22B1	91.2m	5 012 066.5	361 258.0
22B4	90.6m	5 012 123.5	361 206.9
22B5	92.6m	5 012 031.7	361 302.2
CPT1		5 012 180.0	361 207.8
CPT2		5 012 131.0	361 225.3
CPT3		5 012 142.7	361 236.1
CPT4		5 012 147.0	361 214.0
CPT5		5 012 140.8	361 201.5
CPT6		5 012 162.0	361 188.0
CPT7		5 012 156.2	361 238.1
CPT8		5 012 164.5	361 183.0
CPT9		5 012 054.5	361 165.5
CPT10		5 012 064.0	361 228.4
CPT11		5 012 056.0	361 310.0
CPT12		5 012 056.0	361 310.0
CPT13		5 012 056.0	361 341.0
CPT14		5 012 055.5	361 251.0
CPT15		5 012 033.0	361 291.5
CPT16		5 012 034.7	361 221.0
CPT17		5 012 084.5	361 300.0

NOTE

The boundaries between and strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence.

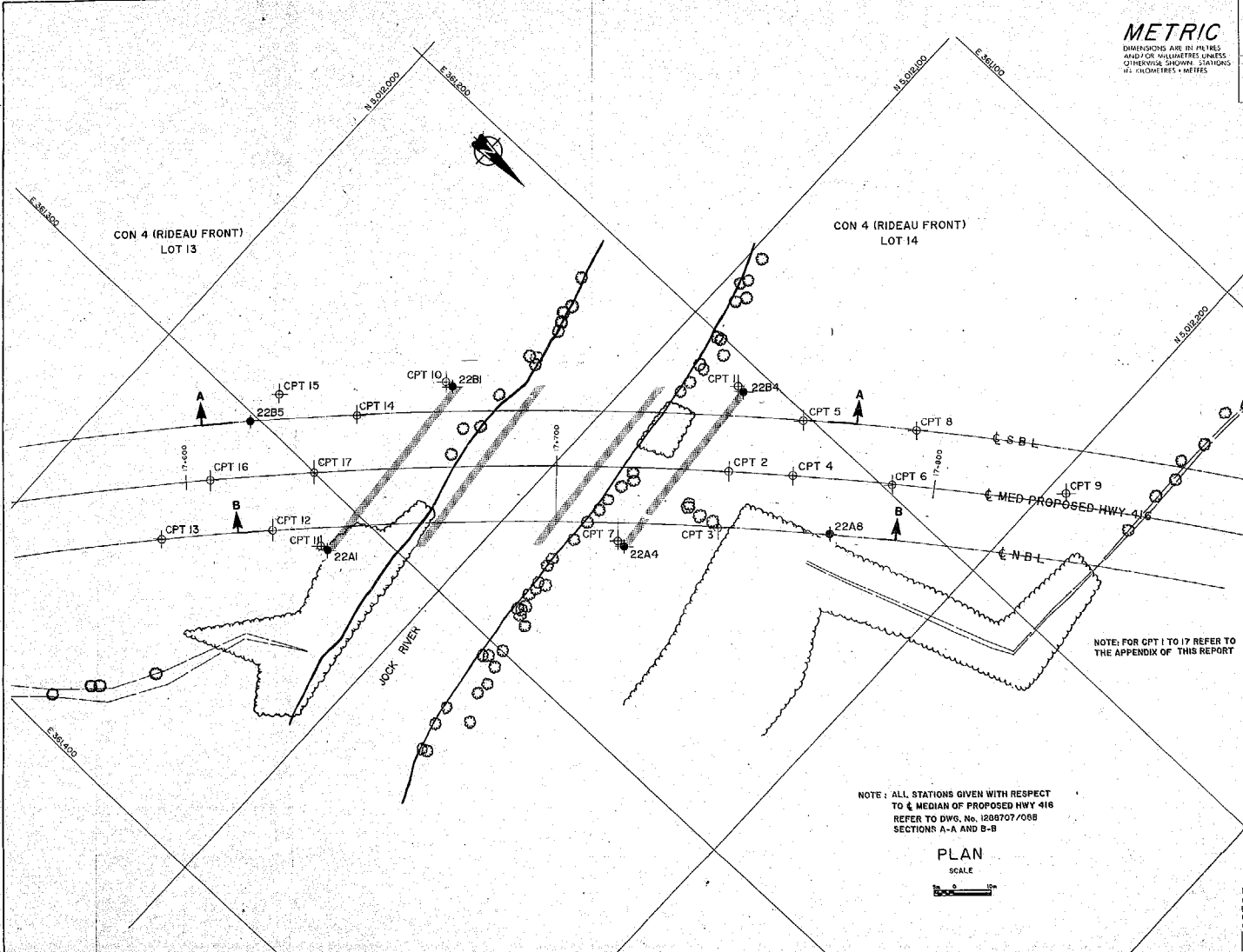
NOTE: The complete foundation investigation and design report for this project and other related documents may be examined at the Engineering Materials Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with the conditions of Section 107.2 of Form 100.

No.	DATE	BY	DESCRIPTION
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NOTE: ALL STATIONS GIVEN WITH RESPECT TO & MEDIAN OF PROPOSED HWY 416 REFER TO DWG. No. 1288707/088 SECTIONS A-A AND B-B

PLAN

SCALE



SCALE



