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HWY. No. 17

LOCATION Tenth Line Extension

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OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.

REMARKS:

DELCAN CORPORATION

PRELIMINARY GEOTECHNICAL INVESTIGATION

HIGHWAY 17 OVERPASS AT REGIONAL ROAD 47
10TH LINE RD.

CUMBERLAND, ONTARIO

W.P. 11-81-02

NOVEMBER 30, 1989

PROJECT No: O-19375

JACQUES, WHITFORD LIMITED

2285 ST. LAURENT BLVD
BUILDING C, UNIT 20
OTTAWA, ONTARIO K1G 4Z6

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Report

to

The Delcan Corporation

on

Preliminary Geotechnical Investigation
Highway 17 Overpass at Regional Road 47

Cumberland, Ontario

Jacques, Whitford Limited

November 30, 1989

Project No. 0-19375

150 000 m^3 (report)

$-16 \times 2 \times 200 \times 10 \approx 64000$ m^3 (Pens)

= 86000

@ \$38./m^3

= \$ ~~2,432,000~~ ±

3,268,000

≈ \$3.3 million

for fill.

\$38 / Tonne @ 1.0 Tn/m^3

= \$38 / m^3

+

Short St. + Lt wt fill

$100 m \times 30 \times 1200 = 3.6$

\$ 3.3

6.9 ←

Long St.

$200 \times 30 \times 1200 = \$ 7.2$

+ Fill + property

✓ Prel. cost analysis favours Lt wt fill option

TABLE OF CONTENTS

1.0 INTRODUCTION

This report presents the results of a preliminary geotechnical investigation carried out for the proposed Highway 17 overpass at Regional Road 47 (Tenth Line Road) in the Township of Cumberland, Ontario. The work was carried out in accordance with our letter of proposal dated October 14, 1989 and our meetings held with the Delcan Corporation on November 2 and 28, 1989. Authorization to conduct the investigation was provided by Mr. Barry T. Darch, P.Eng., of the Delcan Corporation in a letter dated October 23, 1989.

This report has been prepared specifically and solely for the proposed Regional Road 47 project feasibility study described herein. It contains all of our findings with respect to the design of the approachments, and includes preliminary geotechnical recommendations for the design and construction of the project. The report is directed to the Delcan Corporation for cost estimating purposes to assist with the selection of the bridge structure alternative.

2.0 PROPOSED DEVELOPMENT

The location of the proposed development is shown on Drawing No. O-19375-1 provided in Appendix 2 of this report. It is understood that the proposed development will consist of extending Regional Road 47 from the proposed intersection of Regional Road 34 northerly to Jeanne D'Arc Boulevard on the north side of Highway 17. Profiles along this section of the alignment indicate that it is proposed to construct the alignment at grades significantly higher than the existing ground surface elevation. It is understood that at the present time, two (2) alternatives for the overpass at the intersection of Highway 17 and the proposed Regional Road 47 extension are being considered.

2.1 Short Structure Alternative:

The Short Structure alternative consists of a short bridge structure of less than 100 metres in length. The maximum embankment fill heights would be in the order of 12 to 13 metres on the south side of the overpass and approximately 9 metres on the north. This alternative would require the placement of as much as 150,000 m^3 of embankment fill on the south side of the alignment.

2.2 Long Structure Alternative:

The Long Structure alternative consists of a longer bridge structure of more than 200 metres in length. The north abutment for this alternative would be located at the same location as the Short Structure. However the south abutment would be located further south to minimize the requirements for the high embankment fills. The long structure alternative would consist of a evenly spaced multi-pier long span structure.

3.0 SCOPE OF WORK

The scope of work for this portion of the investigation is as follows:

- To carry out a field drilling and laboratory investigation to identify the subsurface soil and groundwater conditions at the proposed embankment fill locations.

- To provide a comprehensive engineering report summarizing the results of the field and laboratory investigation.

- To provide preliminary geotechnical related recommendations for the design and construction of the two proposed overpass alternatives in light of the observed subsurface conditions.

4.0 PROCEDURE

4.1 Field Investigation

Prior to the onset of the field investigation, existing geotechnical, geological and hydrogeological information from near the proposed site was collected and reviewed. In addition to standard geological mapping information, previous geotechnical investigations for the Cumberland Trunk Sewer were obtained and reviewed. The preliminary foundation investigation and design report prepared for the Tenth Line Road extension structure by MTO in 1986 was also reviewed.

The field work for this portion of the investigation was carried out between October 23 and November 26, 1989. A total of 12 test holes, consisting of six (6) boreholes (BH 89-1 to 89-6), and six (6) Dynamic Cone Penetration tests (CPT 89-1 to 89-6) were put down at the locations shown on Drawing No. O-19375-1 in Appendix 2. Cone Penetration Tests were also carried out in Boreholes 89-1, 89-3 and 89-4 to establish practical refusal. Boreholes 89-2, 89-5 and 89-6 involved detailed sampling of soil to auger refusal.

The boreholes and cones were located at selected locations determined by the Delcan Corporation and Jacques, Whitford Limited. In selecting the borehole and cone locations, consideration was given to identifying the subsurface soil conditions in areas of maximum embankment fill heights and identifying the bedrock profile.

The boreholes and cone tests were put down to depths ranging from 6.0 to 45.5 metres below the existing grade using both truck mount and track mounted CME power auger drills suitably equipped for soil and bedrock sampling and testing.

The boreholes were advanced in soil using 200 mm outside diameter hollow-stem augers. The soils were sampled at variable intervals by conducting Standard Penetration Tests. Close sample spacing intervals were carried out at critical locations. Piston samplers were utilized to retrieve selected samples to be submitted for consolidation and other specialized laboratory testing. An estimate of the shearing strength of cohesive soil deposits was obtained by conducting insitu vane shear tests at selected depths in all boreholes.

All soil samples recovered were stored in moisture-proof containers and were sent to our Ottawa laboratory for detailed classification and testing.

Standpipe piezometers were installed in all boreholes. Several groundwater level readings were taken in the standpipes during the field investigation. Groundwater readings were last measured on November 17, 1989.

4.2 Survey

Borehole elevations and locations were surveyed by Delcan personnel using Geodetic Bench Marks and a coordinate grid system. The borehole and cone penetration test locations and elevations are included in Table 1 provided in Appendix 1.

4.3 Laboratory Testing

All soil samples returned to the laboratory were subjected to detailed visual identification. Atterberg Limit, consolidation and moisture content testing was performed on selected samples of the material collected from the boreholes.

All soil samples will be stored in the laboratory for a period of six months after issuance of this report. They will then be discarded unless we are otherwise directed.

5.0 RESULTS OF THE INVESTIGATION

5.1 Surface Condition

At the time of the field investigation, the existing ground surface was relatively flat along the proposed Regional Road 47 alignment north of approximate Station 20+000 (centerline of existing Highway 17). To the west of the proposed road centreline, there existed a small gravel road that was used for access during the installation of the previously constructed Cumberland Trunk Sewer. The elevation of this access road was typically below the elevation of the proposed roadway centerline. The ground surface along this flat section was noted to have been stripped of organic and other deleterious materials during recent construction activities in this area.

South of Highway 17 along the proposed alignment, a significant depressed area or gulley was noted extending to depths in the order of six (6) metres below the existing Highway 17 elevation. The gulley runs approximately perpendicular to the proposed alignment. South of the gulley, the ground surface is relatively flat along the proposed alignment for a distance of about 100 metres. South of this point the ground surface rises significantly at a grade of as much as 25%.

A bedrock outcrop was observed from approximate Station 19+805 to Station 19+820.

5.2 Subsurface Profile

The subsurface conditions observed in the boreholes are summarized on the Borehole Records provided in Appendix 1. An explanation of the symbols and terms used on the Borehole Records is also provided. The results of the laboratory tests are summarized on the Borehole Records and on Figures 1 to 3 in Appendix 1.

A stratigraphic profile showing the observed subsurface conditions along the proposed alignment has been prepared and is shown on Drawing No. O-19375-2 in Appendix 2. The stratigraphic profile has been provided for general discussion purposes only. Information plotted on the profiles between boreholes should not be used for detailed design or construction purposes.

The subsurface conditions encountered at the borehole locations generally consist of granular and/or clay fills overlying silty clay, which in turn overlies glacial till and/or bedrock. Generally, no topsoil or rootmat was observed in the sampled boreholes. Specific details of the subsurface materials should be obtained from the Borehole Records. A brief generalized description of the basic materials is provided below.

5.2.1 Clay Fill

Clay fill was encountered along the relatively flat section west of the centreline of the proposed Regional Road 47 at Borehole 89-3. The clay fill was approximately 0.9 m in thickness. Most of the clay fill along Regional Road 47 is likely recently placed during the construction of the Cumberland Trunk sewer. The properties of the clay fill were observed to be variable but can be generalized as firm in consistency.

5.2.2 Granular/Roadway Fill

Silty sand and gravel fill, consisting of temporary road base material was encountered in Borehole 89-1. The thickness of the granular fill was approximately 0.9 metres. The silty sand and gravel fill was found to be compact in consistency with some clay.

5.2.3 Silty Clay

Silty clay was encountered beneath the fill described above or at surface at the other all sampled borehole locations. The clay deposit was observed to extend to depths of up to 45 metres below the existing ground surface.

Atterberg limit tests carried out on samples of the clay at various depths indicate that the liquid limit ranges between 42% and 78% and the plastic limit ranges from 21% to 29%. The Plasticity Index of the clay varies between 21% and 50% indicating that the material is described as medium to highly plastic (see Figure 1 in Appendix 1).

A summary of the Atterberg Limits and field vane shear strengths in the clay with depth are plotted on Figure 2 in Appendix 1. The vane shear strength results typically increase with depth from about 50 kPa to greater than 150 kPa.

The results of consolidation tests carried out on undisturbed samples of the clay obtained at various depths are summarized on Figures 3.1 to 3.6 in Appendix 1. The consolidation test results indicate that the clay has been preconsolidated to pressures of at least 280 kPa.

The silty clay has been described in the Borehole Records as i) weathered crust, ii) grey silty clay or iii) organic, dark grey, silty clay.

Weathered Crust

The upper portion of the silty clay encountered in the sampled boreholes drilled during this investigation consists of brown to grey-brown, weathered, desiccated crust material. The depth of weathering varies from about 2.2 to 3.7 m.

The consistency of the crust material varies from stiff to hard, with the relatively weaker materials located near the bottom of the deposit. The natural moisture content of the crust material ranges between 30% and 50%. The higher moisture content material is generally located at the lower sections of the deposit.

Grey Silty Clay

Grey, silty clay was observed beneath the weathered crust material at all borehole locations. The grey silty clay extends to depths of about 9.0 to 18.3 metres beneath the existing ground surface.

The grey, silty clay has a significantly lower consistency and higher natural moisture content than the overlying weathered clay crust material. Undrained shear strength values measured using vane shear equipment range from 50 to more than 100 kPa, indicating a stiff to very stiff consistency. Remoulded vane shear strengths in the silty clay range from 5 to 36 kPa, indicating that this material is sensitive, with sensitivity values ranging from 3 to 6. Natural moisture contents of the lower clay range from 40% to more than 70%.

5.2.4 Organic, Dark Grey Silty Clay

Dark grey, silty clay was observed beneath the sensitive grey silty clay layer at all borehole locations with the exception of Borehole 89-5. The dark grey silty clay extends to depths of about 21.5 to 45.5 metres beneath the existing ground surface.

The dark grey, silty clay was organically stained with a significantly higher consistency than the overlying grey clay material. Undrained shear strength values measured using vane shear equipment range from 100 to more than 150 kPa, indicating a very stiff consistency. Remoulded vane shear strengths in this material range from 15 to 52 kPa, with sensitivity values ranging from 3 to 8. Natural moisture contents of the lower clay range from 40% to 60%.

5.2.5 Till

A thin layer of glacial till was encountered beneath the silty clay at Borehole 89-6 and inferred at Cone Penetration Test 89-1 and 89-2. The till deposit was observed to be less than 400 mm where observed. Based on geological evidence, the thickness of the glacial till is likely to be discontinuous within this area. The till may be water-bearing under pressure at some locations.

5.2.6 Bedrock

Bedrock was inferred by auger or cone refusal at all borehole and cone penetration test locations. Depth to inferred bedrock at these locations range from 6.0 to 45.5 metres below the existing grade. As shown on Drawing No. O-19375-2, the elevation of the top of the bedrock along the section between Stations 19+820 and 19+870 of the proposed Regional Road 47 realignment drops significantly to the north. A further significant nine (9) metres drop to the east in the bedrock profile was noted between Cone Penetration Test 89-6 and Borehole 89-2.

5.3 Groundwater

Water levels in the standpipes were monitored several times during and upon completion of the field investigation. The groundwater levels observed on November 17, 1989 are plotted on the Borehole Records and are also shown on the Stratigraphic Profile (Drawing No O-19375-2). As shown on the stratigraphic profile, the depth to groundwater from the existing ground surface ranges from as shallow as surface to 1.5 metres below the existing grade.

Seasonal fluctuations in the groundwater levels will occur and should be anticipated.

6.0 DISCUSSION AND RECOMMENDATIONS

6.1 Geotechnical Evaluation

Both the Short Structure and Long Structure alternatives described in Section 2.0 will require the construction of embankment fills along the proposed alignment. A significant volume of fill will be made available during excavation of the proposed cut section along Regional Road 47, immediately south of Regional Road 34. An assessment of the anticipated material from this cut has been carried out and is described in Section 6.2 below.

Construction of the embankment fills will be carried out on the deep deposit of relatively weak, sensitive and compressible clay described in Section 5.2. The characteristics of the clay will limit the height of embankment fill that can be placed without special considerations given to slope stabilization and settlement considerations. A preliminary assessment of both of these items is addressed in Sections 6.3 and 6.4.

Preliminary recommendations for the bridge structure foundations are provided in Section 6.5.

6.2 Assessment of Excavated Material

6.2.1 Estimated Quantities

The exact quantity of material to be excavated from the proposed cut section along Regional Road 47, south of Regional Road 34, will depend on the long term slope stabilization measures that are used to support this cut section south of Regional Road 34. Based on other boreholes drilled at the cut section area, a preliminary estimate of the quantity of the various materials is as follows:

Material	Estimated Quantity (m^3)
1. Silty clay	140,000
2. Glacial till	5,000
3. Blasted bedrock	30,000

It is estimated that approximately 90,000 m^3 of the silty clay excavation, will consist of weathered clay crust material.

It should be noted that these quantities are very approximate and should not be used for final design or construction purposes.

6.2.2 Use of Excavated Material as Embankment Fill

The blast rock and glacial till materials from the Regional Road 47 cut section are acceptable materials for construction of the main body of the embankment fills. The blast rock should be graded and contain particle sizes no greater than 450 mm. The upper metre of the blast rock should be blinded to prevent loss of fill material into the rock fill. Blasted rock particles greater than 75 mm should not be used for embankment fill in areas where pile foundations will be driven.

It is recommended that the excavated grey silty clay materials not be used for construction of the main body of the embankment fills. The grey clay can be used for berm and landscaping purposes. It is possible that selected portions of the weathered clay crust material could be used in the upper sections of the main body of the embankment. The remaining portion of the crust material could be used for berms and landscaping.

6.3 Embankment Stability

As discussed in Section 5.2, the foundation soil beneath the proposed embankment fill consists of up to 45 metres of stiff to very stiff, sensitive clay. The undrained shear strength profile of the clay has been plotted on Figure 2 in Appendix 1. This figure indicates that the undrained shear strength of the unweathered, grey clay beneath the crust increases with depth from about 50 kPa to greater than 150 kPa.

Preliminary stability analyses of different embankment heights constructed on the foundation clay have been carried out. The parameters used in the stability analysis are summarized on Table 2 provided in Appendix 1. The field vane shear strength results have been adjusted depending upon the Plasticity Index of the material as recommended in the literature.

The analyses have been carried out with the assumption that the main body of the embankment is constructed in one stage and consists mostly of granular material (such as blast rock, glacial till, Granular B). Where required, the berms for the embankment have been assumed to consist of cohesive material (such as clay crust or grey clay). All fills are assumed to be constructed at 2 horizontal to 1 vertical side slopes.

Stability analyses have been carried out using an in-house computer program that utilizes the Bishop's Simplified Method of Analysis. For these analyses, it has been assumed that a Factor of Safety, FS of at least 1.4 is required to maintain the embankment stability. A variety of embankment fill configurations were analyzed.

The key results of the stability analyses are summarized on Figure 4 provided in Appendix 1. The results suggest that embankment fills can be constructed to heights of up to nine (9) metres without the need for counterbalancing stabilization berms. Higher embankments will require berms of different lengths, depending upon the embankment height, as shown on Figure 4. At sloping ground locations, the embankment height used on Figure 4 should be the larger of the heights measured at the crest and the toe of the embankment. Figure 4 may be used to determine the embankment configuration in both the longitudinal and transverse directions.

Preliminary stability analyses have also been carried out with the assumption that a light-weight fill, such as a slag material is used to construct the embankment. It has been assumed that the unit weight of such a material is about 14 kN/m^3. These analyses suggest that embankment fills as high as 12 metres could be constructed without the requirement for stabilization berms. However, slag aggregate sources of the volumes required for this project are not readily available in the Ottawa-Carleton area and would therefore be expensive to import.

The stability analyses presented above are preliminary in nature and should be reviewed upon the selection of the final embankment locations and configurations.

Recommendations for construction of the embankment fills can be submitted during the final design. The following preliminary recommendations are provided for cost analysis purposes:

1) Existing fill, topsoil and other deleterious organic material should be stripped from beneath the main embankment prior to fill placement. Stripping need not be carried out beneath stabilization berms.

2) All embankment fill must be placed in lifts and compacted.

3) Slope indicator and piezometer instrumentation is recommended to monitor the foundation performance beneath embankment heights greater than about 10 metres. Depending upon the instrumentation readings, the placement of the upper few metres of the embankment may be restricted or delayed until favourable foundation performance is observed.

6.4 Embankment Settlement

Settlement beneath the embankment fills will take place immediately after construction and will continue for some time after construction. An estimate of this settlement is required to assess the impact of the settlement on the adjacent bridge structures. The required embankment fill quantities and the as-built shape of the embankment will also depend somewhat on the anticipated settlement.

The results of consolidation testing carried out on selected samples of the foundation clay indicate that the material has been preconsolidated to pressures of at least 280 kPa. This predonsolidation pressure is likely to be greater than the pressure imposed on the foundation clay by the embankment fills. Nevertheless, recompression of the clay due to high fill construction is expected to generate significant settlement.

Using the information from the laboratory testing carried out on the clay material, it is estimated that a fill height of 12 metres will experience an approximate total settlement of about 250 to 350 mm. It is assumed that the embankment fill will be placed up to two (2) years prior to completion of the road structure. Construction of the bridge structure will commence one year after placement of the embankment fill. Therefore, the amount of post-construction settlement experienced beneath the embankment fill after this two year initial consolidation period is expected to be reduced to about 75 to 100 mm.

Pre-loading the high embankment fills with an additional surcharge of at least two (2) metres and maintaining this surcharge during the two year period could reduce the post-construction settlement to less than 50 mm.

The settlement estimates presented above have been calculated assuming the embankment fill is constructed with normal fill materials. Settlements beneath light-weight fills such as the slag material discussed in Section 6.3 will be less than described above.

The embankment fill should be over-built to accommodate the anticipated settlement.

It is recommended that survey monitor pins, about one metre in length, be installed within the embankment fill to monitor the actual settlement in the field.

6.5 Bridge Structure Foundations

Steel H-piles driven to bedrock would provide a suitable foundation for the proposed bridge structures at this site. The boreholes drilled along the proposed alignment indicated that refusal to the Dynamic Cone Penetration test was encountered at various depths of up to more than 30 metres below existing grade. However, it is understood that the bridge abutment locations are not yet finalized and that further drilling and rock coring will be required to prove bedrock depths once final abutment locations have been established.

Because of the height of embankment fill to be retained by wingwalls at the abutment locations, battered piles will likely be required to resist the lateral thrust imposed by these.

The proposed pile driving equipment and installation procedure should be reviewed by the geotechnical consultant prior to mobilization to the field.

In locations of sloping bedrock, rock points should be installed on the piles prior to driving to prevent the piles from sliding on the rock surface. Special consideration will need to be given to the design of battered piles in sloping rock locations.

Steel H-piles driven to refusal may be designed using a Serviceability Limit State Type II allowable stress of 90 N/mm^2 (13 ksi) against the steel pipe pile cross-section. For example, the capacity at the SLS Type II for a 310 HP 110 H pile should exceed 1,250 kN. Allowable capacities using Ultimate Limit State Design and for other pile sizes can be submitted if required.

The total design loads acting on the piles driven at abutment locations should include the effects of full downdrag (negative skin friction) loads. Field observations reported in the literature indicate that at a given depth, the negative skin friction imposed on a single pile embedded in a settling clay is equal to approximately 20 to 30 percent of the vertical effective stress at that depth. It is recommended that the negative skin friction for a single pile be calculated as follows:

$$q_n = 0.25\ p_{effective}$$

where q_n is the negative skin friction at a given depth, and $p_{effective}$ is the effective vertical stress at the same depth.

The downdrag load on an individual pile could then be calculated as follows:

$$Q_n = q_{navg}\ A_s\ D_n$$

where Q_n is the total downdrag load, q_{navg} is the average q_n over the length of pile, A_s is the shaft area of the pile, and D_n is the length of pile embedded in the settling soil.

If necessary, negative skin frictions on piles can be reduced by the application of bituminous or other viscous fluid prior to installation. Examples in literature have indicated reductions in skin friction of up to 50% by using bituminous coatings on model test piles; it is suggested that a 30% reduction can be used as a guide.

It is noted that full transient loads do not act in combination with negative skin friction. The effect of negative skin friction on battered piles is generally more severe and will need to be addressed.

Where isolated pile supported piers are located more than 35 m from the large embankment fills, and where little to no increase in grade is anticipated, the pile design need not include downdrag forces.

The base of the pile caps should be protected against frost heave.

6.6 Other Considerations

When assessing the cost of the two structure alternatives discussed in Section 2.0, the following additional considerations should be taken into account:

1) If the material from the proposed Regional Road 47 cut section south of Regional Road 34 is used to construct the embankment fills along this alignment, it may be beneficial from a cost point of view to have the excavation and fill placement work carried out under one contract.

2) If the short structure alternative is selected, it is recommended that the embankment fills be constructed immediately in order to provide at least two years time for settlements to occur prior to construction of the pavement structure.

3) Consideration will need to be given to drainage of the low gulley section near Station 19+940 if the embankment fill for the short structure alternative is placed within this gulley. If a culvert is required to be placed in this area, it must be designed to accommodate the high embankment fill placed directly over it.

4) An assessment of the effects of the embankment fills on existing utilities such as the adjacent Cumberland Trunk Sewer line must be assessed prior to final design.

7.0 CLOSURE

The recommendations made in this report are preliminary and are in accordance with our present understanding of the project. As mentioned in this report, additional geotechnical work will be required prior to the final design.

A soils and groundwater investigation is a limited sampling of a site. Should any conditions at the site be encountered which differ from those at the test locations, we request that we be notified immediately in order to permit reassessment of our recommendations.

We trust the above information meets your present requirements. Should you have any questions or require additional information, please do not hesitate to contact us.

We thank you for the opportunity to be of service to you.

Yours very truly,

JACQUES, WHITFORD LIMITED

LICENSED PROFESSIONAL ENGINEER
C. P. DA SILVA
PROVINCE OF ONTARIO

Carlos P. Da Silva, P.Eng.

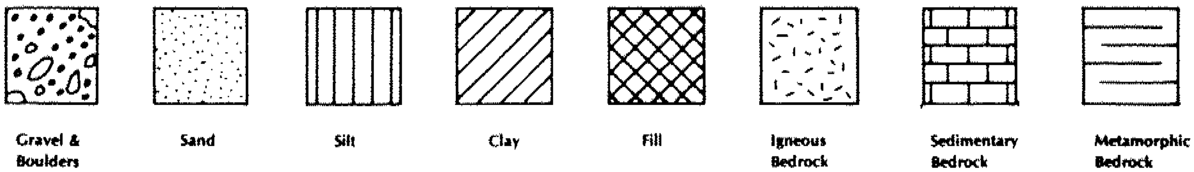
LICENSED PROFESSIONAL ENGINEER
G. J. KACK
PROVINCE OF ONTARIO

Gordon J. Kack, M.E.Sc.,P.Eng

APPENDIX 1

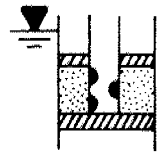
SYMBOLS AND TERMS CONTINUED

STRATA PLOT



WATER LEVEL MEASUREMENT

Borehole or Standpipe



Piezometer

SAMPLES

SS.... Split spoon sample (obtained by performing the standard penetration test)
ST.... Shelby tube or thin wall tube
PS.... Piston sample
BS.... Bulk sample
WS.... Wash sample
RC.... Rock core
AXT, BXL, etc....
Rock core samples obtained with the use of standard diamond drilling bits.

OTHER TESTS

G..... Specific gravity
H..... Hydrometer analysis
S..... Sieve analysis
γ.... Unit weight
C..... Consolidation
CD.... Consolidated drained triaxial
CU.... Consolidated undrained triaxial with pore pressure measurements
UU.... Unconsolidated undrained triaxial
DS.... Direct shear
P..... Field permeability

ROCK DESCRIPTION

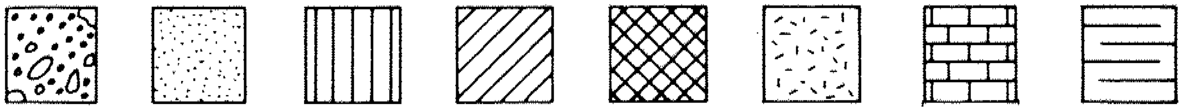
The description of bedrock is based on the rock quality designation (RQD).

The classification is based on a modified core recovery percentage in which all pieces of sound core over 100mm long are counted as recovery. The smaller pieces are considered to be due to close shearing, jointing, faulting, or weathering in the rock mass and are not counted. In most cases RQD is run on NXL core; however, it can be used on different core sizes if the bulk of the fractures caused by drilling stresses are easily distinguishable from normal insitu fractures.

RQD	ROCK QUALITY
90-100	Excellent, intact, very sound
75-90	Good, massive, moderately jointed or sound
50-75	Fair, blocky and seamy, fractured
25-50	Poor, shattered and very seamy or blocky, severely fractured
0-25	Very poor, crushed, very severely fractured.

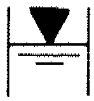
SYMBOLS AND TERMS CONTINUED

STRATA PLOT

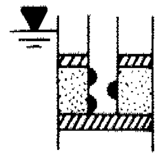


Gravel & Boulders | Sand | Silt | Clay | Fill | Igneous Bedrock | Sedimentary Bedrock | Metamorphic Bedrock

WATER LEVEL MEASUREMENT



Borehole or Standpipe



Piezometer

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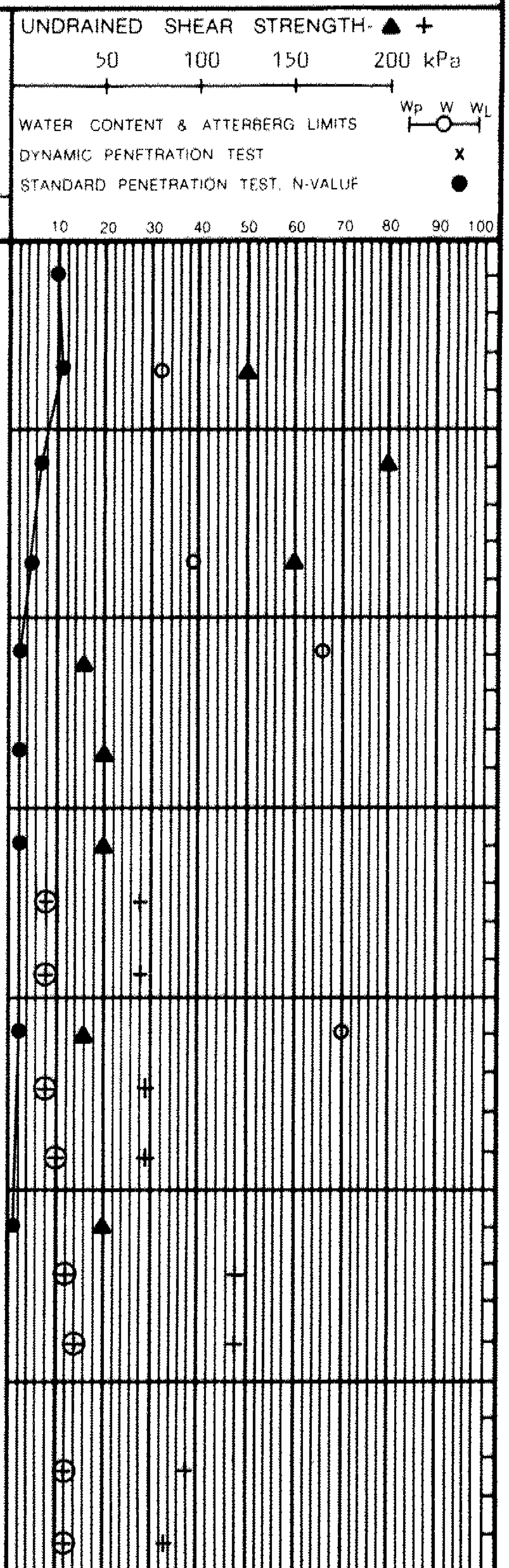
BOREHOLE RECORD

BOREHOLE No. 89-1

CLIENT: Delcan Corporation
LOCATION: Regional Road 47 at Highway 17, Cumberland, Ontario
DATES: BORING October 26, 1989 WATER LEVEL November 17, 1989

PROJECT No. 0-19375
CASING SIZE Auger
DATUM Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	SAMPLE TYPE	NUMBER	RECOVERY (mm)	N-VALUE OR RQD %	OTHER TESTS
0	56.7	Ground Surface					
		Compact, brown, silty sand and gravel, mixed with clay: FILL	SS	1	450	10	
	55.9	Hard to very stiff, brown, SILTY CLAY, crust, desicated	SS	2	350	11	
			SS	3	600	7	
			SS	4	600	5	
	53.9	Stiff to very stiff, grey, SILTY CLAY, wet	SS	5	600	2	
			SS	6	600	2	
			SS	7	600	2	
			SS	8	600	2	
			SS	9	600	1	
			PS	10	450	-	
11	45.7	Continued					



▲ Pocket Penetrometer
+ Field Vane
⊕ Remolded Field Vane

204 86/11

BOREHOLE RECORD

BOREHOLE No. 89-1

CLIENT Delcan Corporation
PROJECT No. 0-19375
LOCATION Regional Road 47 at Highway 17, Cumberland, Ontario
CASING SIZE Auger
DATES: BORING October 26, 1989
WATER LEVEL November 17, 1989
DATUM Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	RECOVERY (mm)	N-VALUE OR RQD %	OTHER TESTS
11	45.7	Continued					
11–15		Stiff to very stiff, grey SILTY CLAY, wet					
13			SS	11	600	1	
15	41.5	Very stiff, dark grey, SILTY CLAY, organic, stains, wet					
16			PS	12	450	-	
19			SS	13	600	2	
22			PS	14	450	-	
22	34.7	Continued					

UNDRAINED SHEAR STRENGTH- ▲ + 50 100 150 200 kPa

WATER CONTENT & ATTERBERG LIMITS W_P W W_L

DYNAMIC PENETRATION TEST x

STANDARD PENETRATION TEST. N-VALUE ●

10 20 30 40 50 60 70 80 90 100

▲ Pocket Penetrometer
+ Field Vane
⊕ Remolded Field Vane

204 88/11

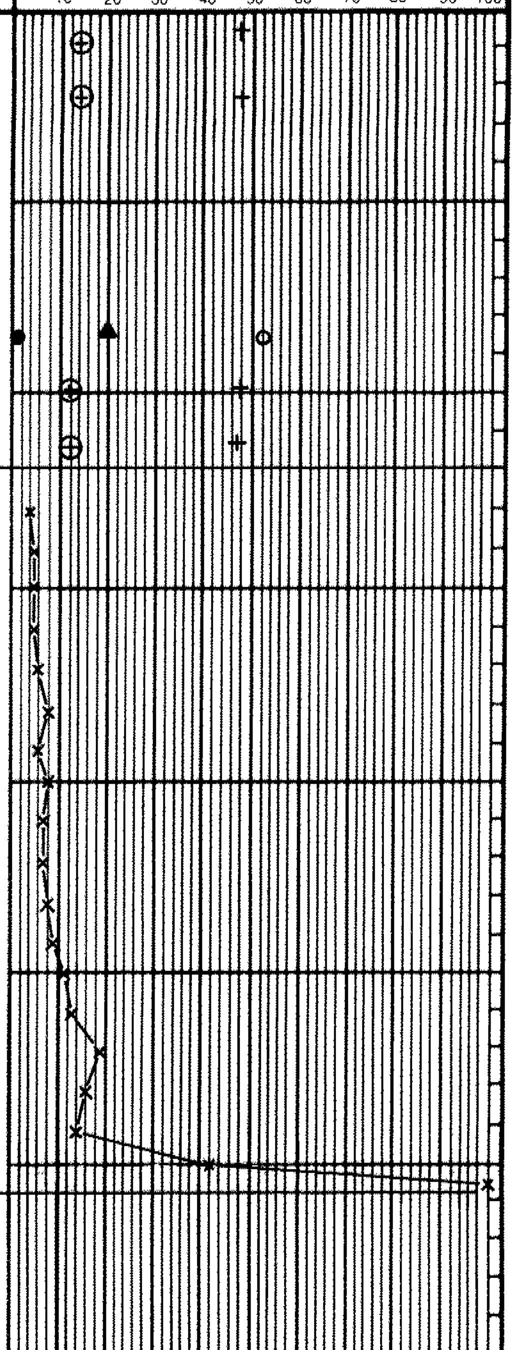
BOREHOLE RECORD

BOREHOLE No. 89-1

CLIENT: Delcan Corporation
LOCATION: Regional Road 47 at Highway 17, Cumberland, Ontario
DATES: BORING: October 26, 1989
WATER LEVEL: November 17, 1989
PROJECT No. 0-19375
CASING SIZE: Auger
DATUM: Geodetic

DEPTH (FT.)	DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES: TYPE	SAMPLES: NUMBER	SAMPLES: RECOVERY (mm)	SAMPLES: N-VALUE OR RQD %	OTHER TESTS
	22	34.7	Continued							
	22–25.6		Stiff, dark grey, SILTY CLAY, organic, stains, wet			SS	15	600	1	
		31.1	End of Borehole. Start of Dynamic Cone Penetration Test							
		25.3	End of Dynamic Cone Penetration Test (Advance refusal on dense till or possible bedrock.)							

UNDRAINED SHEAR STRENGTH- ▲ + 50 100 150 200 kPa
WATER CONTENT & ATTERBERG LIMITS W_P W W_L
DYNAMIC PENETRATION TEST x
STANDARD PENETRATION TEST. N-VALUE ●
10 20 30 40 50 60 70 80 90 100



▲ Pocket Penetrometer
+ Field Vane
⊕ Remolded Field Vane

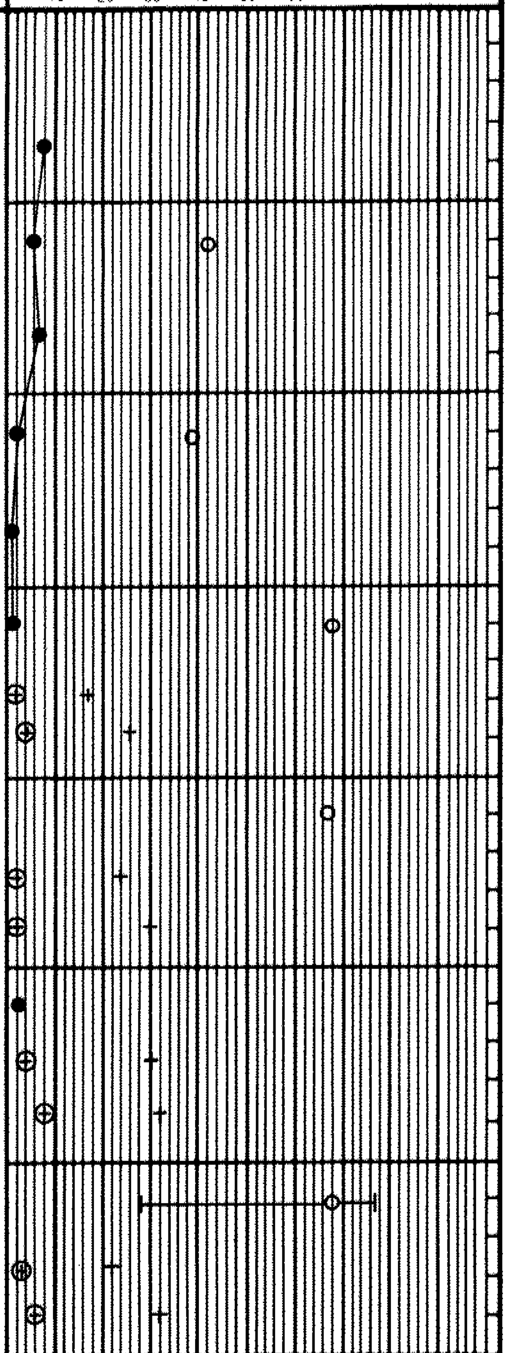
BOREHOLE RECORD

BOREHOLE No. 89-2

CLIENT Delcan Corporation
LOCATION Regional Road 47 at Highway 17, Cumberland, Ontario
DATES: BORING October 24, 1989
WATER LEVEL November 17, 1989
PROJECT No. 0-19375
CASING SIZE Auger
DATUM Geodetic

DEPTH (FT.)	DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	NUMBER	RECOVERY (mm)	N-VALUE OR RQD %	OTHER TESTS
	0	57.6	Ground Surface							
	0–3.7		Hard to stiff, brown SILTY CLAY, crust, desiccated							
	1				▼	SS	1	560	8	
	2					SS	2	600	6	
						SS	3	600	7	
	3					SS	4	600	2	
		53.9	Firm to stiff, grey SILTY CLAY, wet							
	4					SS	5	600	1	
	5					SS	6	600	1	
	6					SS	7	600	-	
	8					SS	8	600	2	
	9					SP	9	600	-	
	11	46.6	Continued							

UNDRAINED SHEAR STRENGTH +
50 100 150 200 kPa
WATER CONTENT & ATTERBERG LIMITS W_P W W_L
DYNAMIC PENETRATION TEST x
STANDARD PENETRATION TEST, N-VALUE ●
10 20 30 40 50 60 70 80 90 100



\+ Field Vane
⊕ Remolded Field Vane

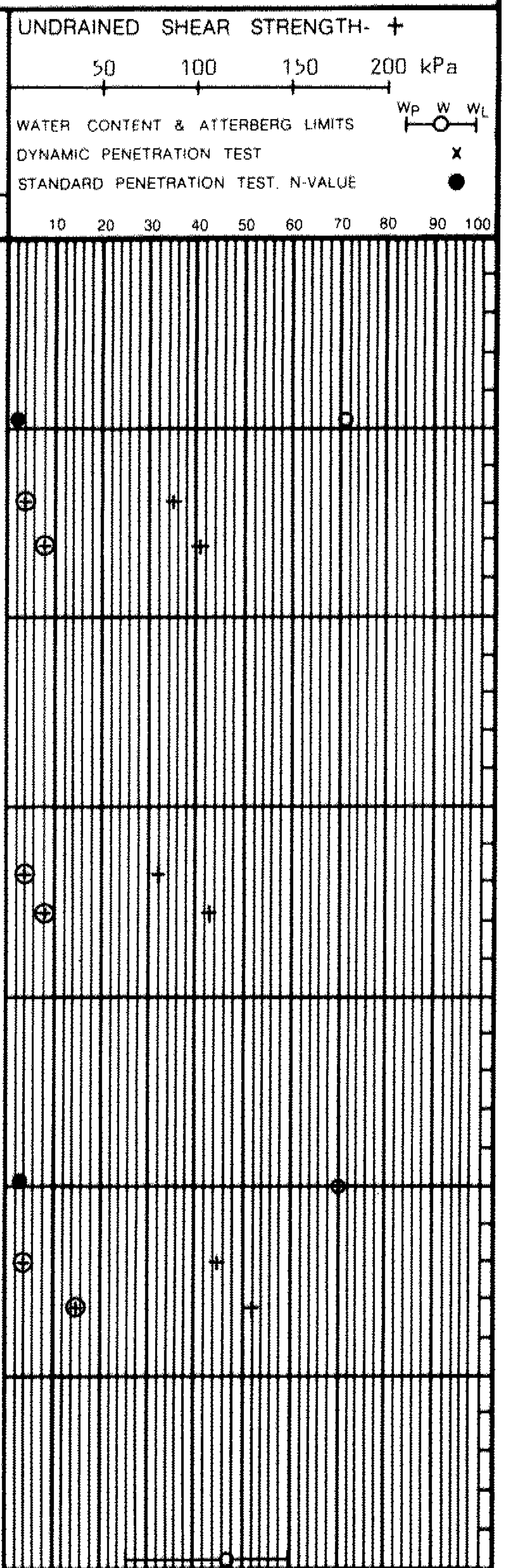
BOREHOLE RECORD

BOREHOLE No. 89-2

CLIENT Delcan Corporation
LOCATION Regional Road 47 at Highway 17, Cumberland, Ontario
DATES: BORING October 24, 1989 WATER LEVEL November 17, 1989

PROJECT No. 0-19375
CASING SIZE Auger
DATUM Geodetic

DEPTH (FT.)	DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	NUMBER	RECOVERY	N-VALUE OR RQD %	OTHER TESTS
		46.6	Continued							
	11		Firm to stiff, grey, SILTY CLAY, wet							
	12	45.4	Stiff to very stiff, dark grey, SILTY CLAY, organic stains, wet			SS	10	600	2	
	13–15					TP	11	600	PH	
	16–18	39.3	Very stiff, dark grey, SILTY CLAY, organic stains, wet			SS	12	600	3	
	19–21					TP	13	600	PH	
	22	35.6	Continued							



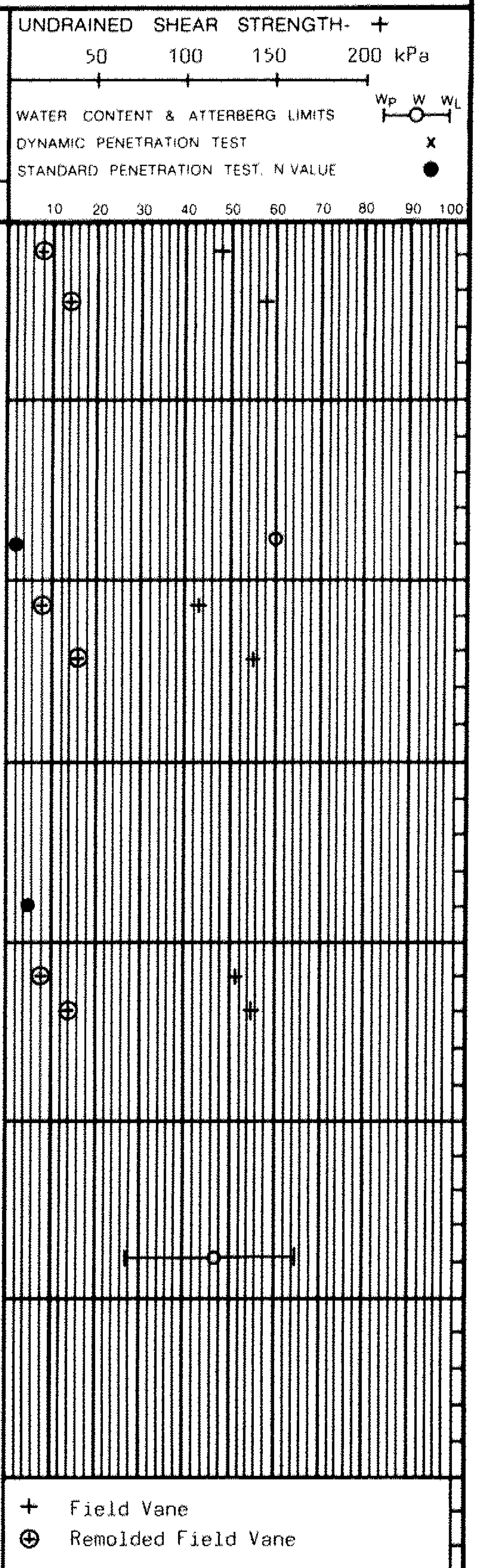
\+ Field Vane
⊕ Remolded Field Vane

BOREHOLE RECORD

BOREHOLE No. 89-2

CLIENT: Delcan Corporation
LOCATION: Regional Road 47 at Highway 17, Cumberland, Ontario
DATES: BORING: October 24, 1989
WATER LEVEL: November 17, 1989
PROJECT No. 0-19375
CASING SIZE: Auger
DATUM: Geodetic

DEPTH (FT.)	DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	NUMBER	RECOVERY	N-VALUE OR RQD %	OTHER TESTS
	22	35.6	Continued							
	22–33		Very stiff, dark grey, SILTY CLAY, organic stains, wet							
	~24.8					SS	14	600	2	
	~27.8					SS	15	600	5	
	~30.8					ST	16	600	PH	
	33	24.6	Continued							



BOREHOLE RECORD

BOREHOLE No. 89-2

CLIENT Delcan Corporation

PROJECT No. 0-19375

LOCATION Regional Road 47 at Highway 17, Cumberland, Ontario

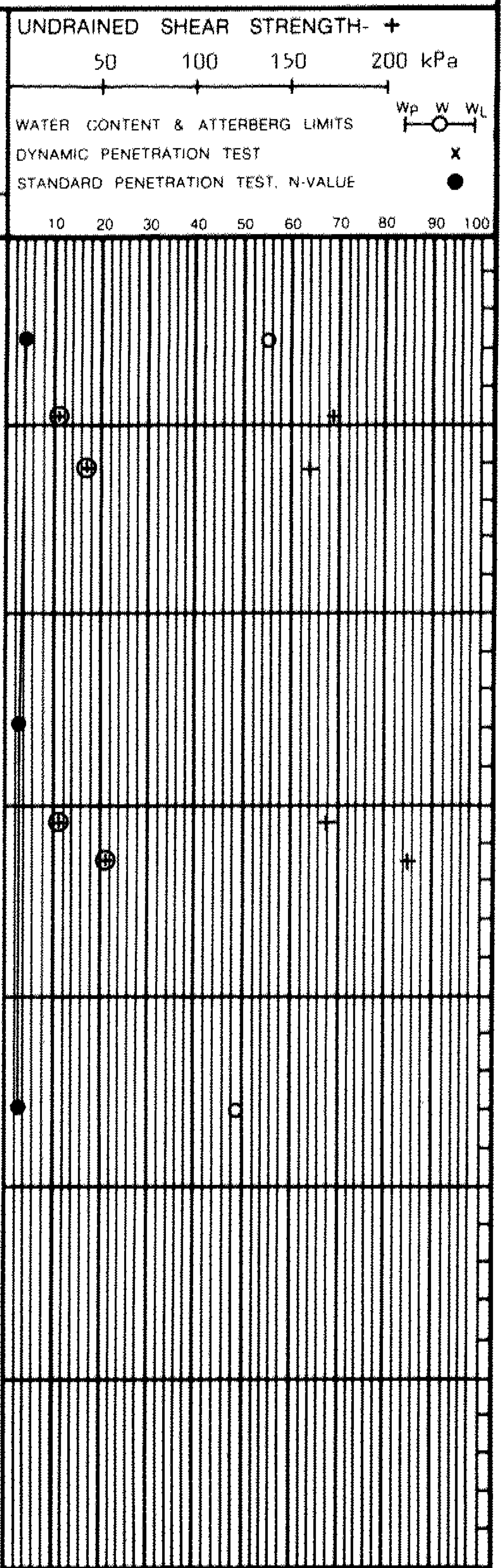
CASING SIZE Auger

DATES: BORING October 24, 1989

WATER LEVEL November 17, 1989

DATUM Geodetic

DEPTH (FT.)	DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	NUMBER	RECOVERY	N-VALUE OR RQD %	OTHER TESTS
	33	24.6	Continued							
	34		Very stiff, dark grey, SILTY CLAY, organic stains, wet			SS	17	600	4	
	35									
	36									
	37					SS	18	600	3	
	38									
	39									
	40					SS	19	600	3	
	41									
	42									
	43									
	44	13.6	Continued							



+ Field Vane

⊕ Remolded Field Vane

BOREHOLE RECORD

BOREHOLE No. 89-2

CLIENT Delcan Corporation

PROJECT No. 0-19375

LOCATION Regional Road 47 at Highway 17, Cumberland, Ontario

CASING SIZE Auger

DATES: BORING October 24, 1989 WATER LEVEL November 17, 1989

DATUM Geodetic

DEPTH (FT.)	DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	SAMPLES NUMBER	SAMPLES RECOVERY	SAMPLES N-VALUE OR RQD %	OTHER TESTS	UNDRAINED SHEAR STRENGTH- / WATER CONTENT & ATTERBERG LIMITS W_P W W_L / DYNAMIC PENETRATION TEST x / STANDARD PENETRATION TEST, N-VALUE ● / 10 20 30 40 50 60 70 80 90 100
	44	13.42	Continued								
	45	12.1	Very stiff, dark grey, SILTY CLAY, organic stains, wet								
	46		END OF BOREHOLE Auger refusal on dense till or possible bedrock								

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BOREHOLE RECORD

BOREHOLE No. 89-3

CLIENT Delcan Corporation

PROJECT No. 0-19375

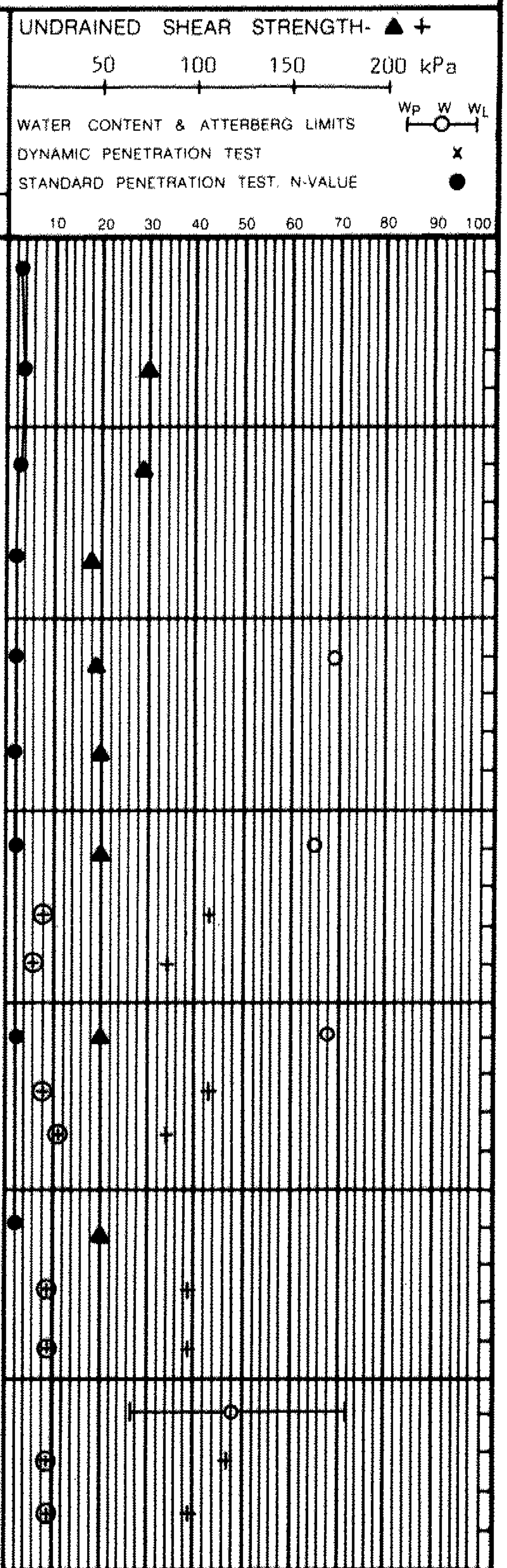
LOCATION Regional Road 47 at Highway 17, Cumberland, Ontario

CASING SIZE Auger

DATES: BORING October 26, 1989 WATER LEVEL October 31, 1989

DATUM Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	SAMPLE TYPE	NUMBER	RECOVERY mm	N-VALUE OR RQD %
0	56.7	Ground Surface				
		Firm, brown, silty clay, moist: FILL	SS	1	100	3
	55.8	Very stiff, brown, SILTY CLAY, CRUST, desiccated	SS	2	350	4
			SS	3	550	3
	54.6	Very stiff to stiff, grey, SILTY CLAY, wet	SS	4	600	2
			SS	5	600	2
			SS	6	600	2
			SS	7	600	2
			SS	8	600	2
			SS	9	600	2
			PS	10	450	PH
	46.1	Dark grey				
	45.7	Continued				



▲ Pocket Penetrometer
+ Field Vane
⊕ Remolded Field Vane

204 88/11

BOREHOLE RECORD

BOREHOLE No. 89-3

CLIENT Delcan Corporation

LOCATION Regional Road 47 at Highway 17, Cumberland, Ontario

DATES: BORING october 26, 1989 WATER LEVEL October 31, 1989

PROJECT No. 0-19375

CASING SIZE Auger

DATUM Geodetic

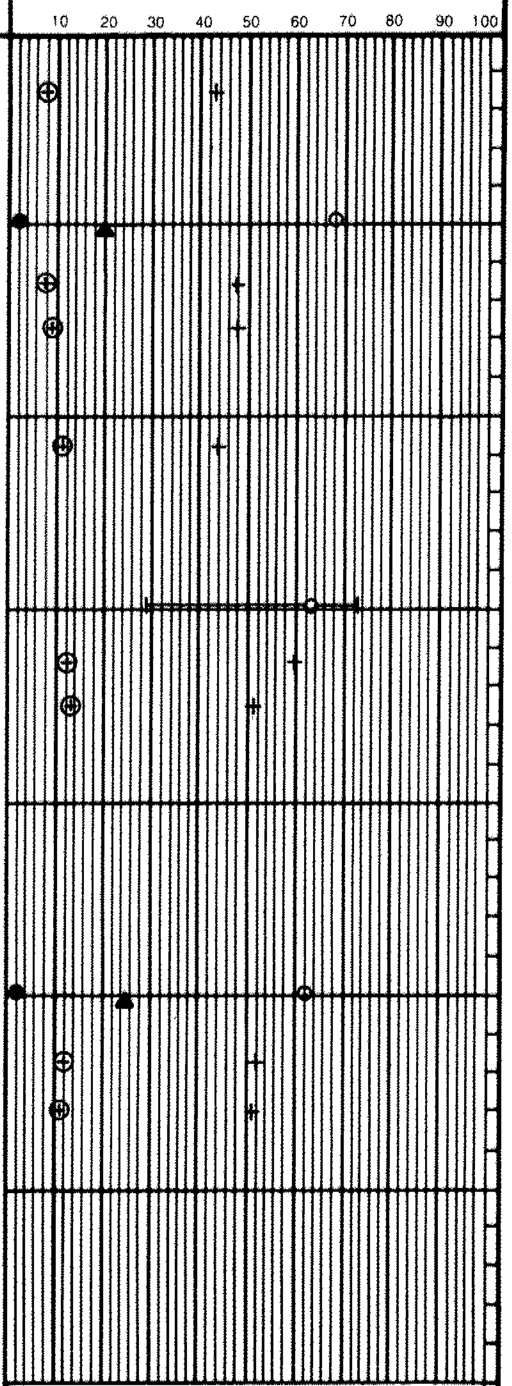
DEPTH (FT.)	DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	NUMBER	RECOVERY mm	N-VALUE OR RQD %	OTHER TESTS
	11	45.7	Continued							
			Very stiff, dark grey, SILTY CLAY, wet							
	12					SS	11	600	2	
	13–15					PS	12	450	-	
	16–18					SS	13	600	2	
	19–20	36.2								
	21		Very stiff, dark grey, SILTY CLAY, organic stains, wet			PS	14	450	-	
	22	34.7	Continued							

UNDRAINED SHEAR STRENGTH- ▲ + 50 100 150 200 kPa

WATER CONTENT & ATTERBERG LIMITS W_P W W_L

DYNAMIC PENETRATION TEST x

STANDARD PENETRATION TEST, N-VALUE ●



▲ Pocket penetrometer
+ Field Vane
⊕ Remolded Field Vane

BOREHOLE RECORD

BOREHOLE No. 89-3

CLIENT: Delcan Corporation

LOCATION: Regional Road 47 at Highway 17, Cumberland, Ontario

DATES: BORING: October 26, 1989

WATER LEVEL: October 31, 1989

PROJECT No.: 0-19375

CASING SIZE: Auger

DATUM: Geodetic

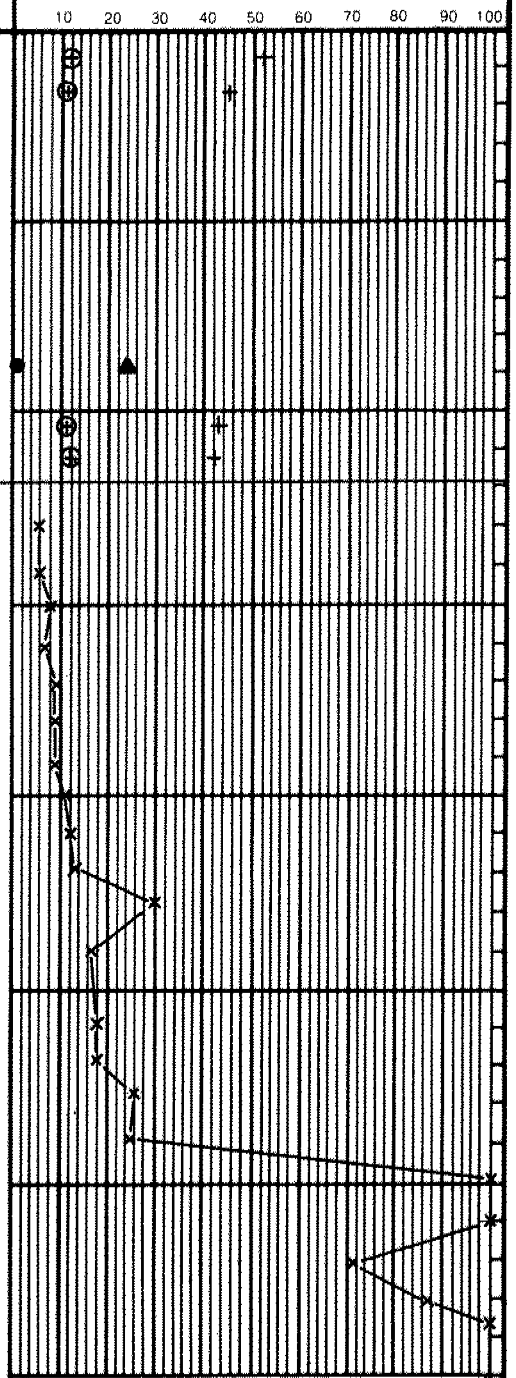
DEPTH (FT.)	DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	NUMBER	RECOVERY	N-VALUE OR RQD %	OTHER TESTS
	22	34.7	Continued							
	22–25.4	31.1	Very stiff, Dark grey, SILTY CLAY, organic stains, wet			SS	15	600	1	
	25.4–32.7	23.9	End of Borehole Start of Dynamic Cone Penetration Test							
	32.7		End of Dynamic Cone Penetration Test (refusal on dense till or possible bedrock.)							

UNDRAINED SHEAR STRENGTH- 50 100 150 200

WATER CONTENT & ATTERBERG LIMITS W_P W W_L

DYNAMIC PENETRATION TEST x

STANDARD PENETRATION TEST. N-VALUE ●



▲ Pocket Penetrometer

+ Field VAne

⊕ Remolded Field Vane

204 88/11

BOREHOLE RECORD

BOREHOLE No. 89-4

CLIENT: Delcan Corporation

PROJECT No. 0-19375

LOCATION: Regional Road 47 at Highway 17, Cumberland, Ontario

CASING SIZE: Auger

DATES: BORING October 26, 1989 WATER LEVEL November 17, 1989

DATUM: Geodetic

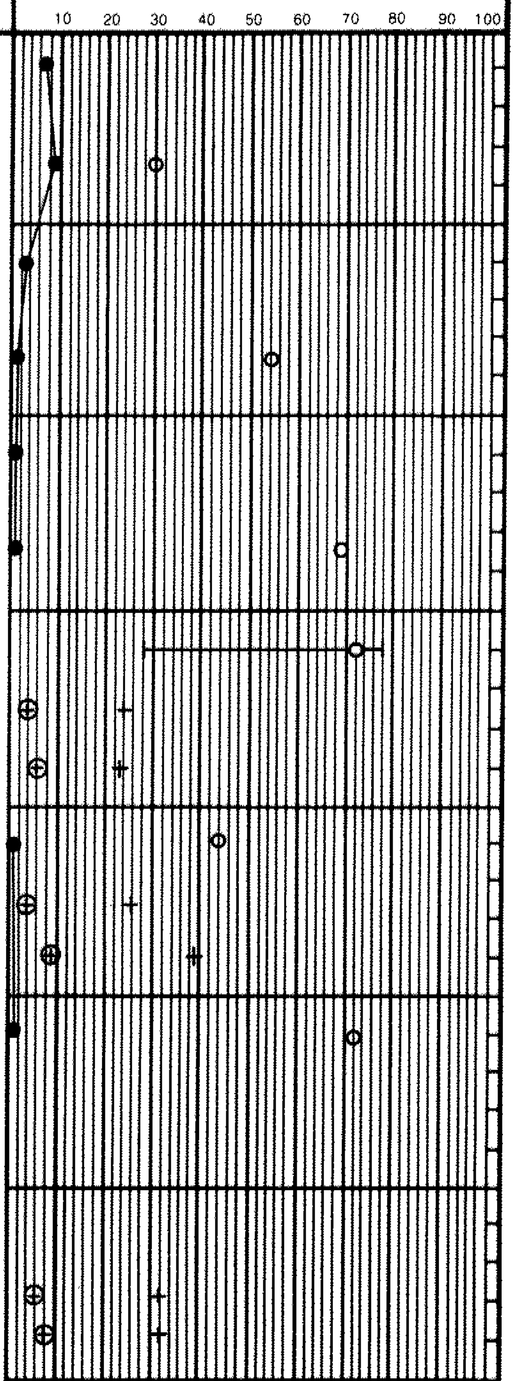
DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	TYPE	NUMBER	RECOVERY	N-VALUE OR RQD %	OTHER TESTS
0	59.4	Ground Surface					
		50mm of SILTY CLAY FILL over Very stiff, brown, SILTY CLAY, CRUST, desiccated	SS	1	410	7	
1			SS	2	530	9	
			SS	3	600	3	
2	57.3	Stiff, grey, SILTY CLAY, wet	SS	4	600	1	
3			SS	5	600	1	
4			SS	6	600	1	
5			PS	7	600	-	
6			SS	8	600	1	
7							
8			SS	9	600	1	
9			PS	10	600	-	
10							
11	48.4	Continued					

UNDRAINED SHEAR STRENGTH- 50 100 150 200 kPa

WATER CONTENT & ATTERBERG LIMITS W_P W W_L

DYNAMIC PENETRATION TEST x

STANDARD PENETRATION TEST, N VALUE ●



+ Field Vane

⊕ Remolded Field Vane

204 88/11

BOREHOLE RECORD

BOREHOLE No. 89-4

CLIENT Delcan Corporation

LOCATION Regional Road 47 at Highway 17, Cumberland, Ontario

DATES: BORING October 26, 1989 WATER LEVEL November 17, 1989

PROJECT No. 0-19375

CASING SIZE Auger

DATUM Geodetic

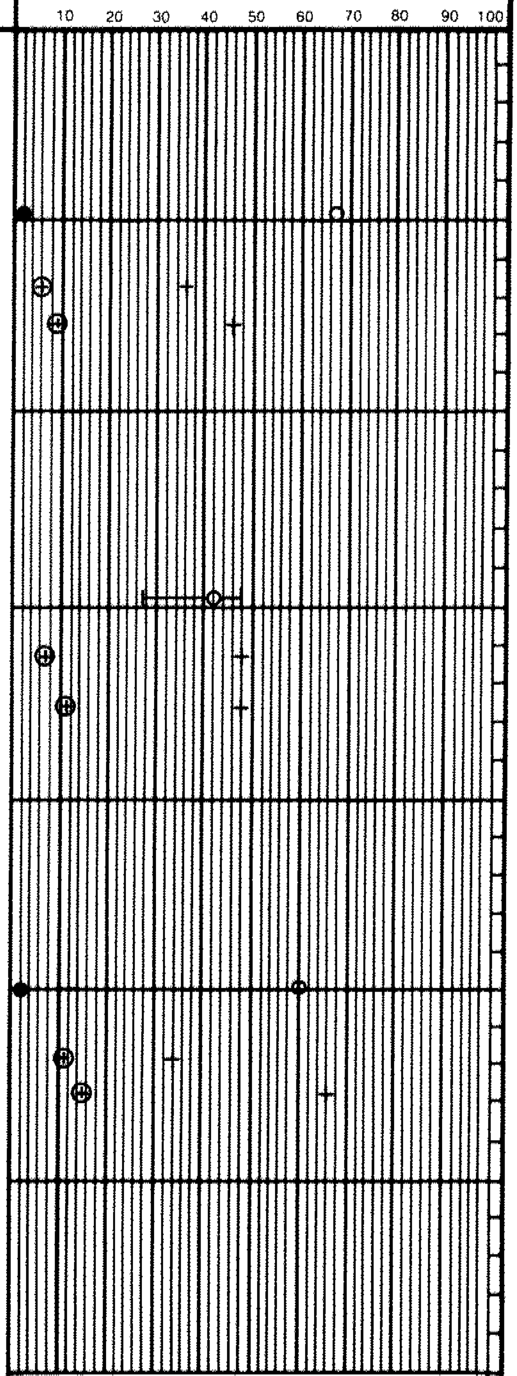
DEPTH (FT.)	DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	NUMBER	RECOVERY	N-VALUE OR RQD %	OTHER TESTS
		48.4	Continued							
	11–18		Stiff to very stiff, SILTY CLAY, wet			SS	11	600	2	
						PS	12	600	-	
	18–22	41.1	Stiff to very stiff, dark grey, SILTY CLAY, wet			SS	13	600	2	
						ST	14	600	PH	
	22	34.4	Continued							

UNDRAINED SHEAR STRENGTH- + 50 100 150 200 kPa

WATER CONTENT & ATTERBERG LIMITS W_P W W_L

DYNAMIC PENETRATION TEST x

STANDARD PENETRATION TEST, N-VALUE ●



+ Field Vane

⊕ Remolded Field Vane

BOREHOLE RECORD

BOREHOLE No. 89-4

CLIENT Delcan Corporation

LOCATION Regional Road 47 at Highway 17, Cumberland, Ontario

DATES: BORING Octiober 26, 1989

WATER LEVEL November 17, 1989

PROJECT No. 0-19375

CASING SIZE Auger

DATUM Geodetic

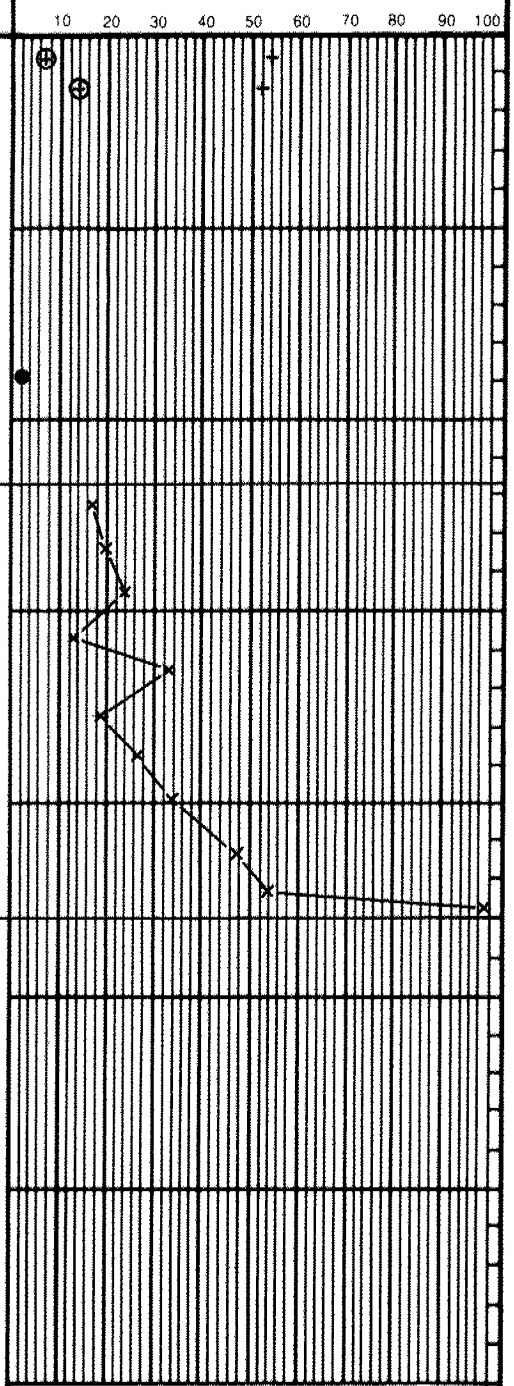
DEPTH (FT.)	DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	SAMPLES NUMBER	SAMPLES RECOVERY	SAMPLES N-VALUE OR RQD %	OTHER TESTS
	22	34.4	Continued							
	22–25.6	33.8	Very stiff, dark grey, SILTY CLAY, wet			SS	15	600	2	
	25.6–29	30.4	End of Borehole Start of Dynamic Cone Penetration Test							
	29		End of Dynamic Cone Penetration Test (Advance refusal on dense till or possible bedrock.)							

UNDRAINED SHEAR STRENGTH- 50 100 150 200 kPa

WATER CONTENT & ATTERBERG LIMITS W_P W W_L

DYNAMIC PENETRATION TEST x

STANDARD PENETRATION TEST N-VALUE ●



+ Field Vane

⊕ Remolded Field Vane

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BOREHOLE RECORD

BOREHOLE No. 89-5

CLIENT: Delcan Corporation

LOCATION: Regional Road 47 at Highway 17, Cumberland, Ontario

DATES: BORING 1989-10-23

WATER LEVEL 1989-10-31

PROJECT No. 0-19375

CASING SIZE Auger

DATUM Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	SAMPLES TYPE	NUMBER	RECOVERY (mm)	N-VALUE OR RQD %	OTHER TESTS
0	60.5	Ground Surface					
		50mm ROOTMAT					
1		Very stiff to stiff, brownish grey, SILTY CLAY CRUST	SS	1	600	10	
2			SS	2	600	5	
3	57.4	Stiff to firm, grey, SILTY CLAY	ST	3	-	-	
6			ST	4	-	-	
9	51.5	End of Borehole (Auger refusal an inferred bedrock)					

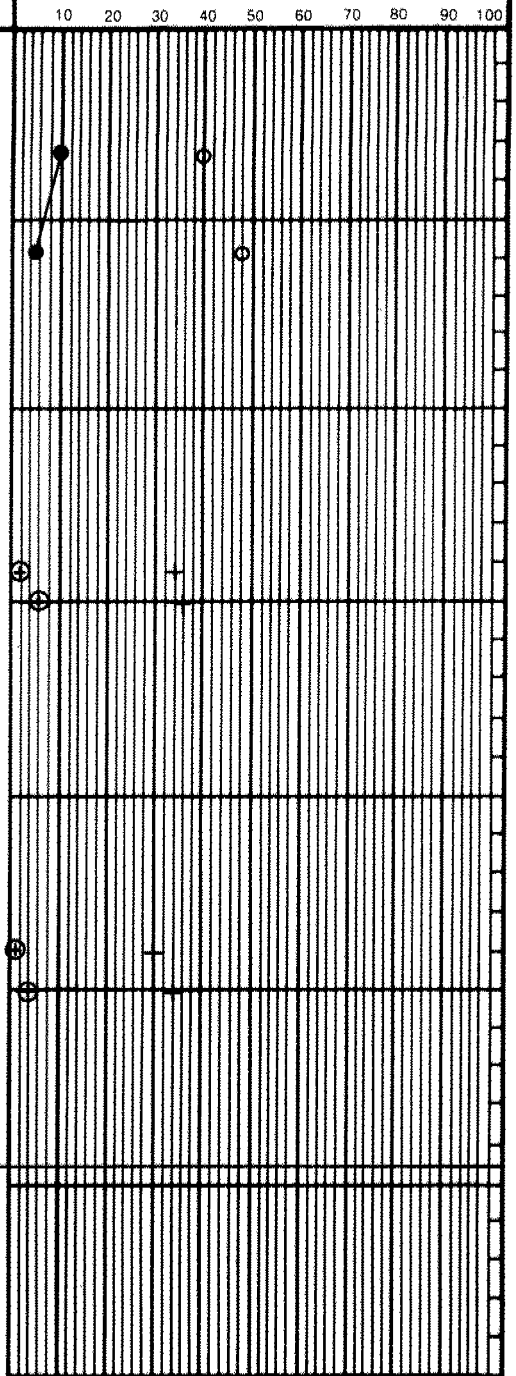
UNDRAINED SHEAR STRENGTH- +

50 100 150 200 kPa

WATER CONTENT & ATTERBERG LIMITS W_P W W_L

DYNAMIC PENETRATION TEST x

STANDARD PENETRATION TEST, N-VALUE ●



+ Field Vane

⊕ Remolded Field Vane

204 88/11

BOREHOLE RECORD

BOREHOLE No. 89-6

CLIENT Delcan Corporation

PROJECT No. 0-19375

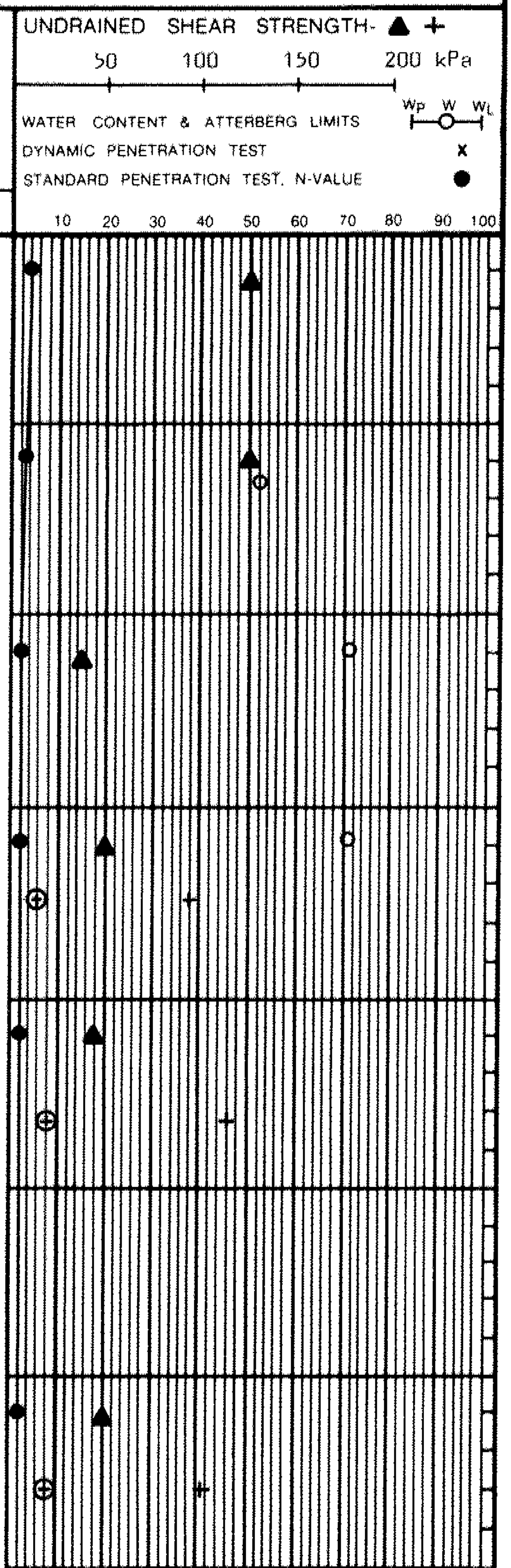
LOCATION Regional Road 47 at Highway 17, Cumberland, Ontario

CASING SIZE Auger

DATES: BORING October 28, 1989 WATER LEVEL November 17, 1989

DATUM Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	TYPE	NUMBER	RECOVERY	N-VALUE OR RQD %	OTHER TESTS
0	53.9	Ground Surface					
		25 mm of TOPSOIL over very stiff, brown, SILTY CLAY, CRUST, desiccated	SS	1	450	4	
			SS	2	600	3	
	51.3	Stiff to very stiff, grey, SILTY CLAY, organic stains, wet	SS	3	600	2	
			SS	4	600	2	
			SS	5	600	2	
	44.8	Stiff to very stiff, grey, SILTY CLAY, wet	SS	6	600	2	
	42.9	Continued					



▲ Pocket Penetrometer

\+ Field Vane

⊕ Remolded Field Vane

BOREHOLE RECORD

BOREHOLE No. 89-6

CLIENT Delcan Corporation

LOCATION Regional Road 47 at Highway 17, Cumberland, Ontario

DATES: BORING October 28, 1989 WATER LEVEL November 17, 1989

PROJECT No. 0-19375

CASING SIZE Auger

DATUM Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	TYPE	NUMBER	RECOVERY	N-VALUE OR RQD %
11	42.9	Continued				
11–21.5		Stiff to very stiff, dark grey, SILTY CLAY, wet	ST	7	450	PH
			SS	8	600	2
			SS	9	600	PM
21.5	32.4	Very dense, grey, SILTY CLAY TILL, wet	SS	10	600	46
22	31.7	End of Borehole (Auger refusal on dense till or possible bedrock.)				

UNDRAINED SHEAR STRENGTH- ▲ + : 50 100 150 200 kPa

WATER CONTENT & ATTERBERG LIMITS W_P W W_L

DYNAMIC PENETRATION TEST x

STANDARD PENETRATION TEST N-VALUE ●

10 20 30 40 50 60 70 80 90 100

▲ Pocket Penetrometer

\+ Field Vane

⊕ Remolded Field Vane

CONE PENETRATION TEST

C.P.T. No. 89-1

CLIENT Delcan Corporation

LOCATION Regional Road 47 at Highway 17, Cumberland, Ontario

DATE October 27, 1989

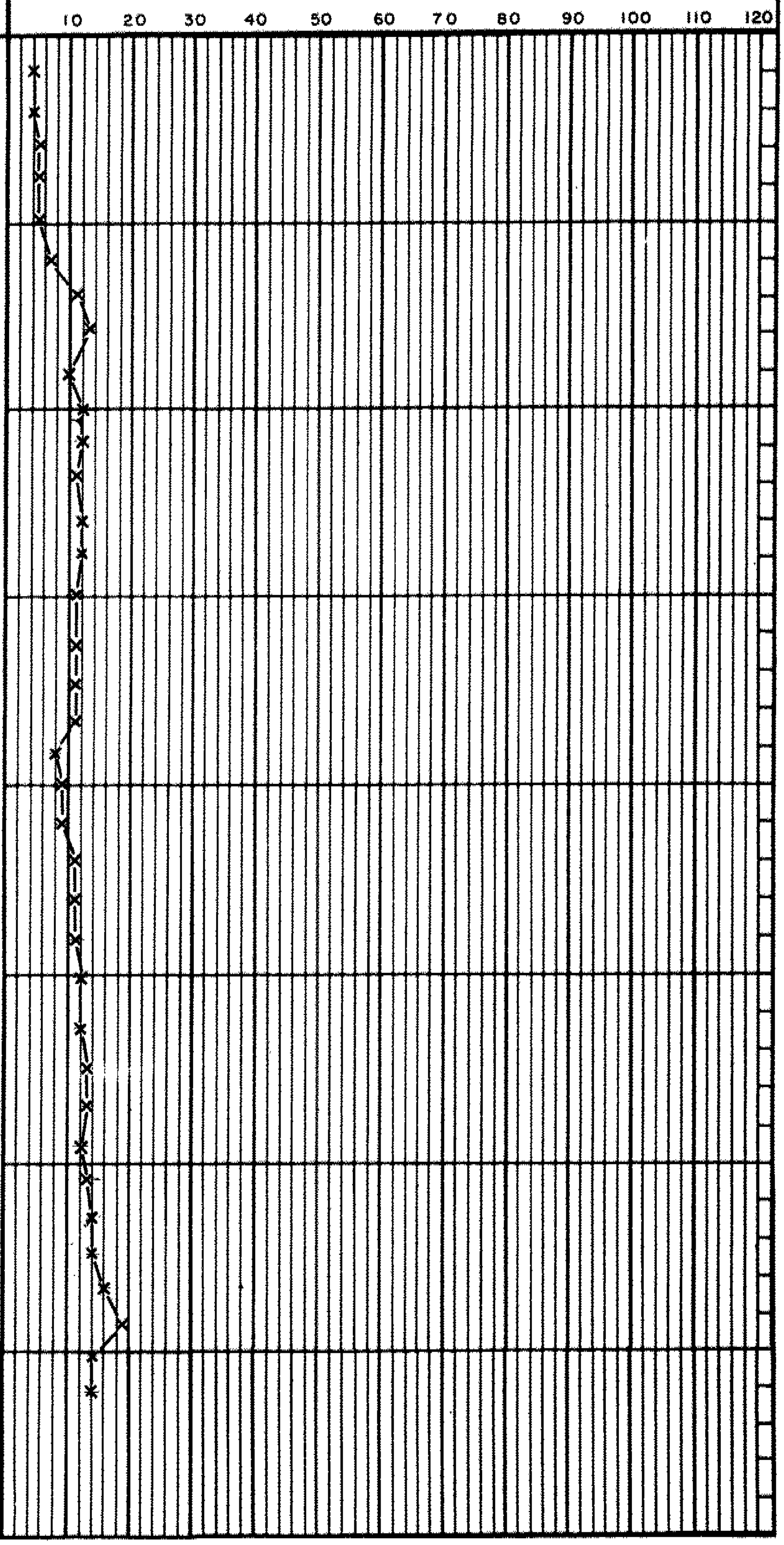
PROJECT No. 0-19375

DATUM Geodetic

DEPTH (FT.)	DEPTH (M)	ELEVATION (m)	INFERRED SOIL TYPE
		59.3	Ground Surface
	1		SILTY CLAY CRUST
	2		
	3		
		56.8	SILTY CLAY
	4		
	5		
	6		
	7		
	8		
	9		
	10		
	11		
		48.3	Continued

DYNAMIC PENETRATION RESISTANCE

— BLOWS / FT. (300 mm)



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CONE PENETRATION TEST

C.P.T. No. 89-1

CLIENT Delcan Corporation

PROJECT No. 0-19375

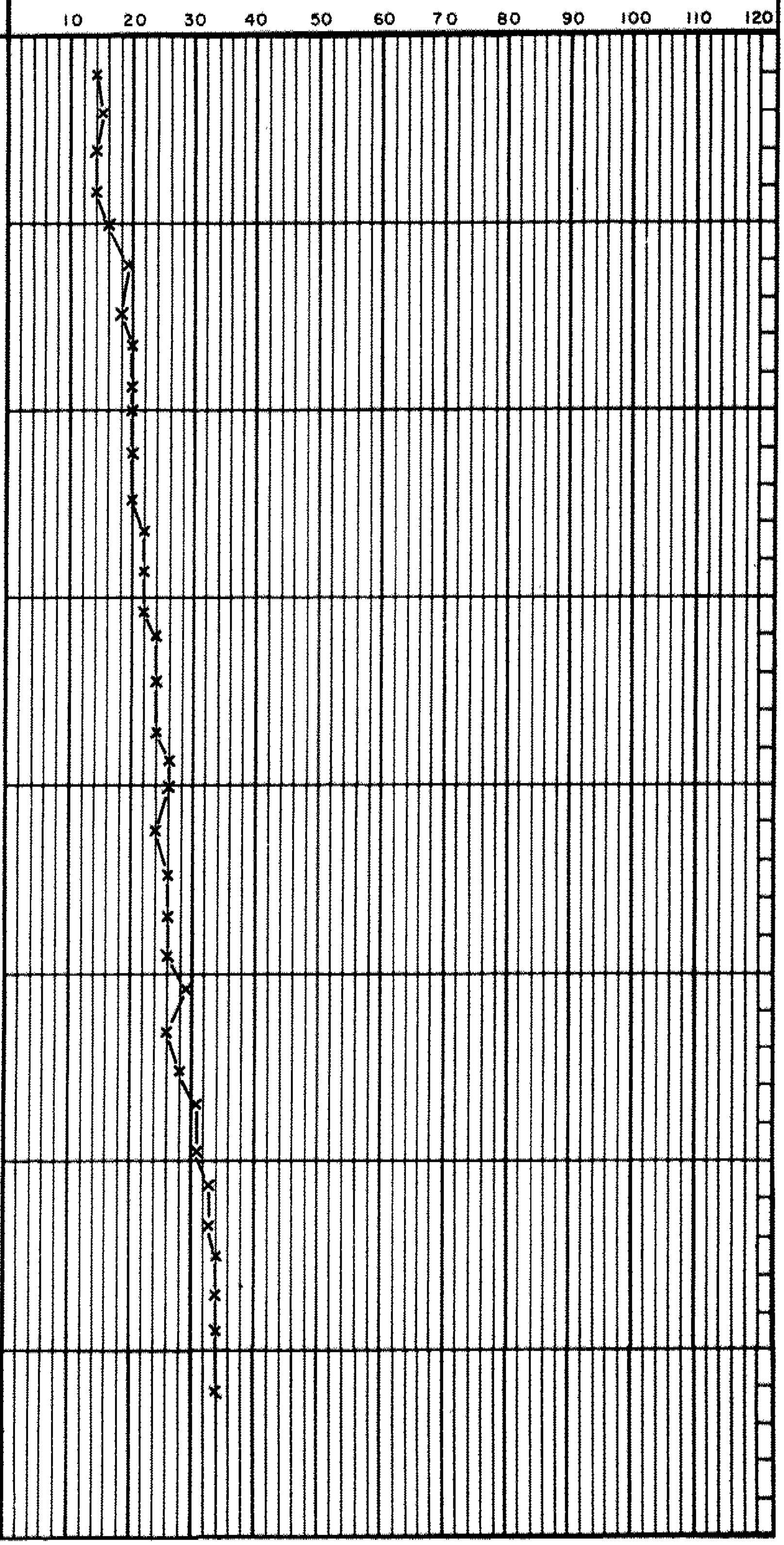
LOCATION Regional Road 47 at Highway 17, Cumberland, Ontario

DATE October 27, 1989

DATUM Geodetic

DEPTH (FT.)	DEPTH (M)	ELEVATION (m)	INFERRED SOIL TYPE
		48.3	Continued
	11		SILTY CLAY
	12		
	13		
	14		
	15		
	16		
	17		
	18		
	19		
	20		
	21		
	22	37.3	Continued

DYNAMIC PENETRATION RESISTANCE — BLOWS / FT. (300 mm)



CONE PENETRATION TEST

C.P.T. No. 89-1

CLIENT Delcan Corporation

PROJECT No. 0-19375

LOCATION Regional Road 47 at Highway 17, Cumberland, Ontario

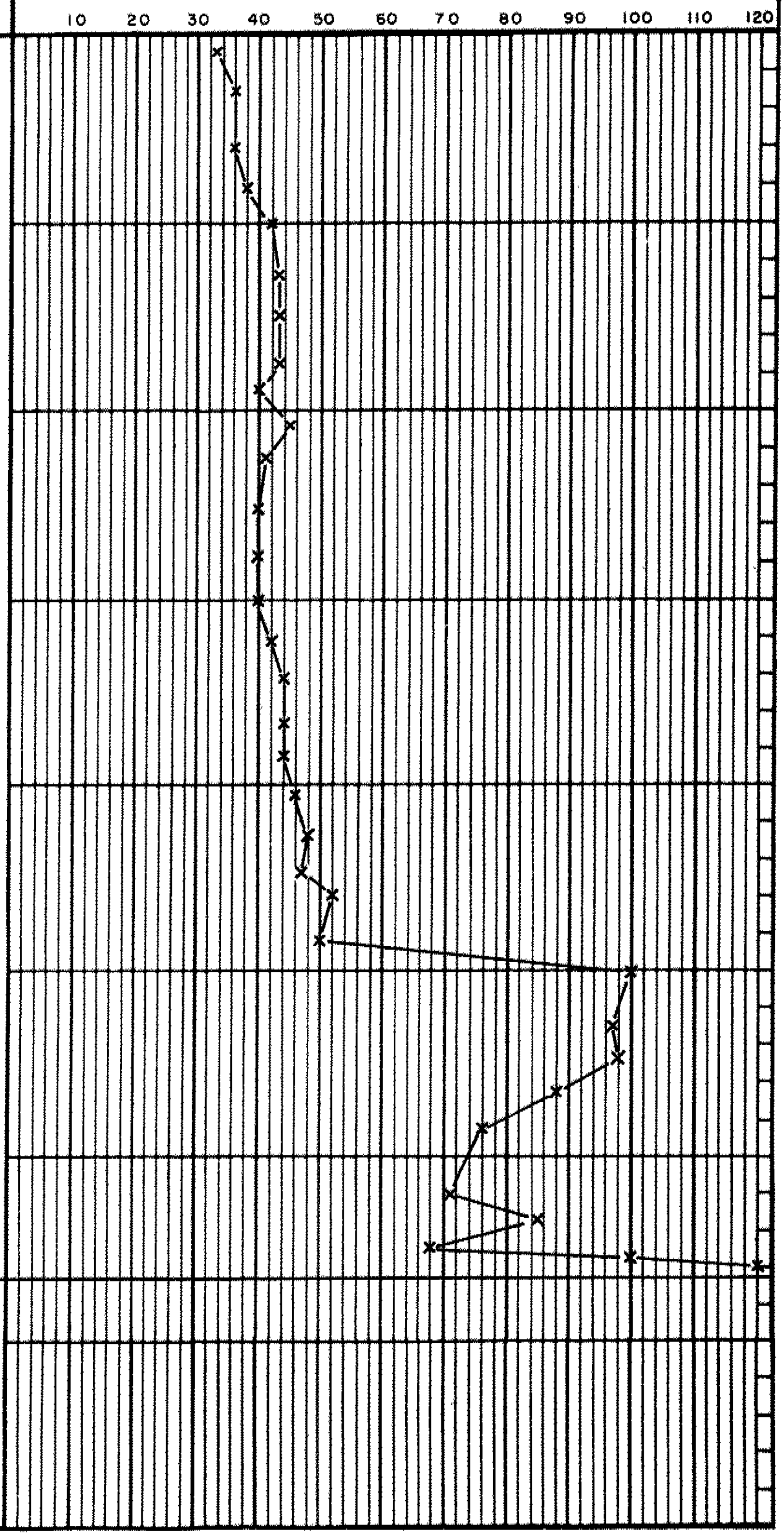
DATE October 27, 1989

DATUM Geodetic

DEPTH (FT.)	DEPTH (M)	ELEVATION (m)	INFERRED SOIL TYPE
		37.3	Continued
	22		
	23		
	24		
	25		
	26		
	27		
	28		
	29	30.0	
	30		Till or weathered Bedrock
	31		
	32	27.1	
	33		End of Dynami Cone Penetration Test (Advance refusal on dense till or possible bedrock)

DYNAMIC PENETRATION RESISTANCE

— BLOWS / FT. (300 mm)



206 3/84

CONE PENETRATION TEST

C.P.T. No. 89-2

PROJECT No. 0-19375

CLIENT Delcan Corporation

LOCATION Regional Road 47 at Highway 17, Cumberland, Ontario

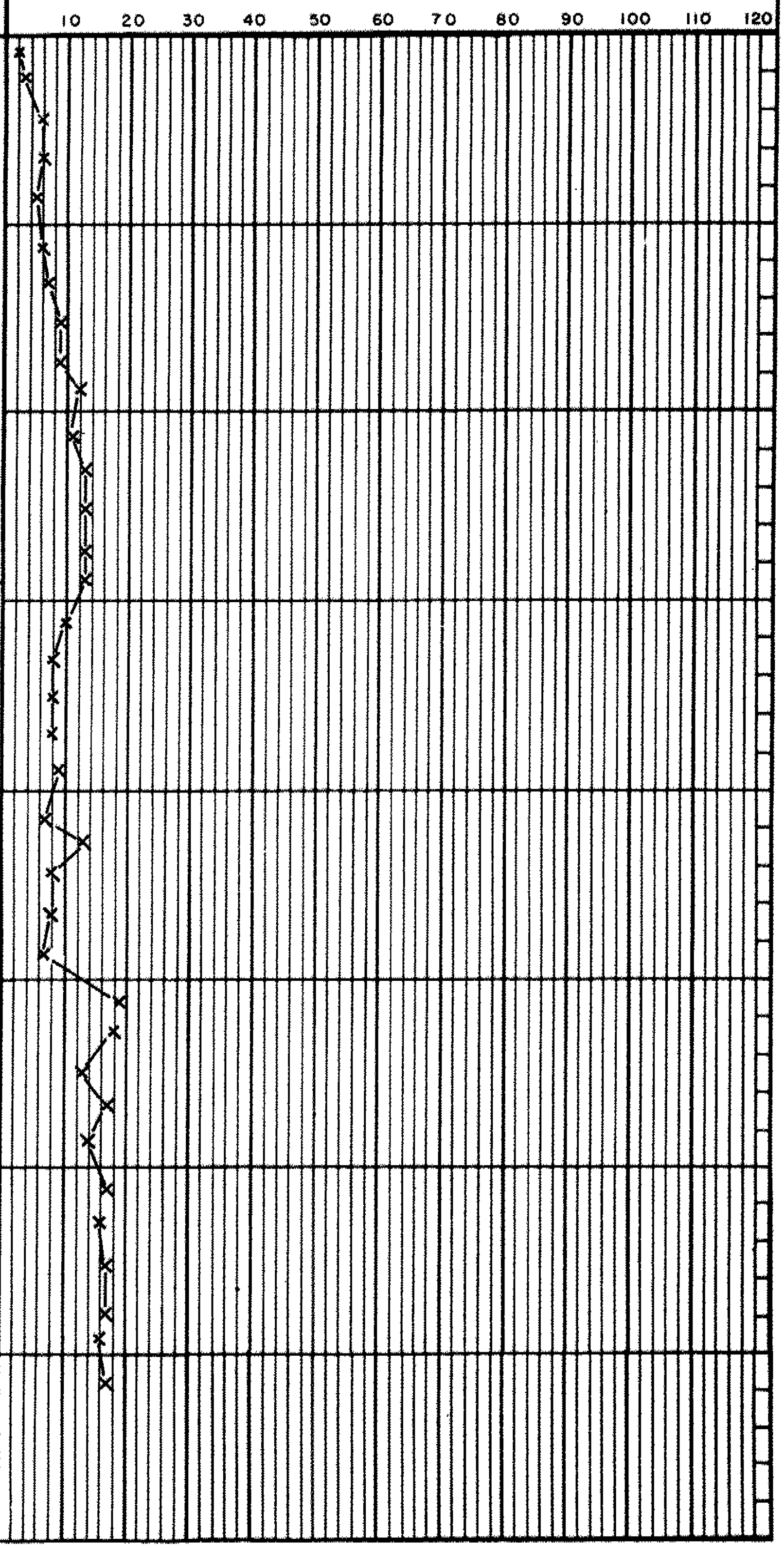
DATE November 10, 1989

DATUM Geodetic

DEPTH (FT.)	DEPTH (M)	ELEVATION (m)	INFERRED SOIL TYPE
		59.9	Ground Surface
	0		SILTY CLAY CRUST
	1		
	2		
	3		
	4	55.5	SILTY CLAY
	5		
	6		
	7		
	8		
	9		
	10		
	11	48.9	Continued

DYNAMIC PENETRATION RESISTANCE

— BLOWS / FT. (300 mm)



CONE PENETRATION TEST

C.P.T. No. 89-2

PROJECT No. 0-19375

CLIENT Delcan Corporation

LOCATION Regional Road 47 at Highway 17, Cumberland, Ontario

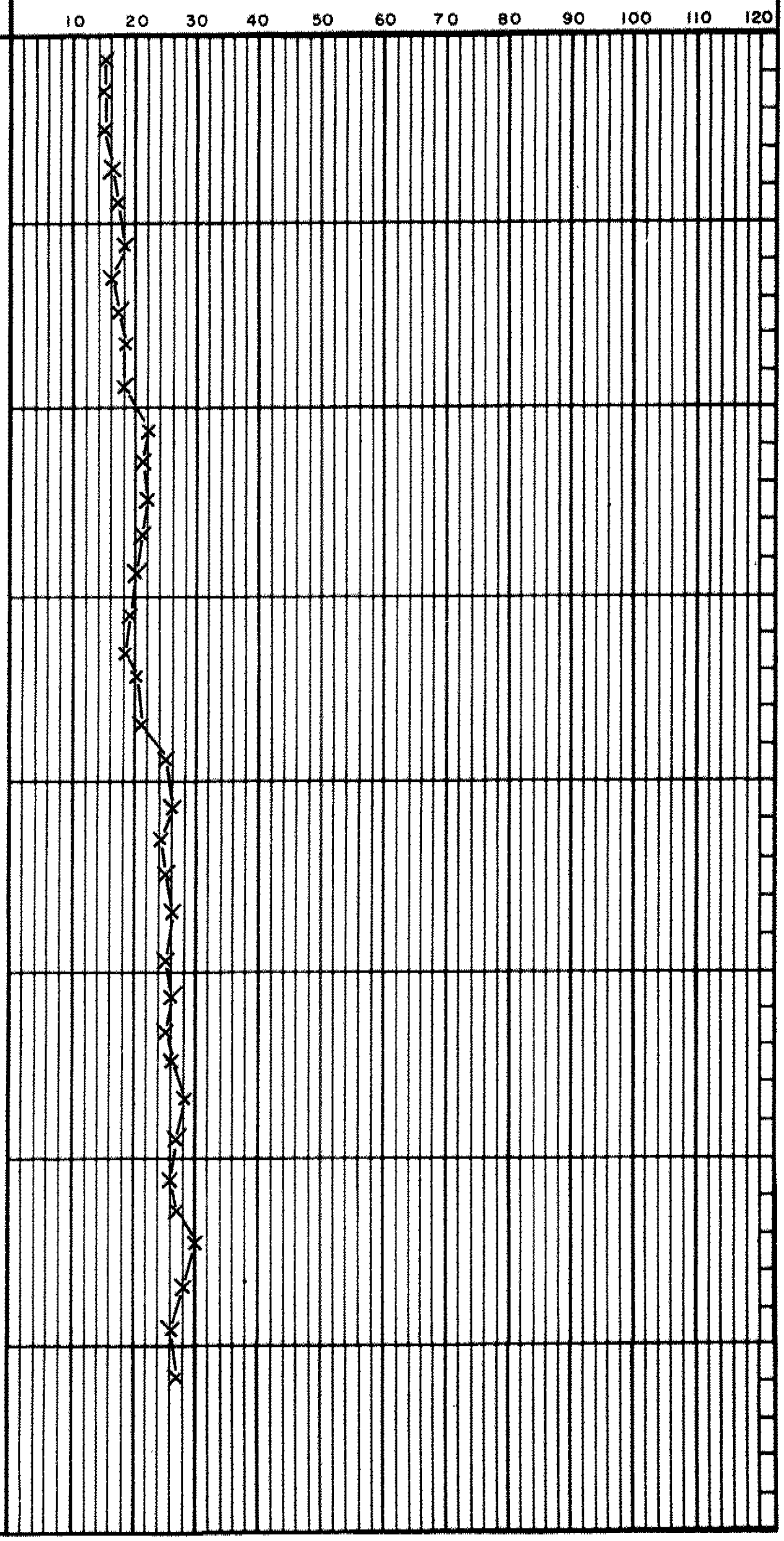
DATE November 10, 1989

DATUM Geodetic

DEPTH (FT.)	DEPTH (M)	ELEVATION (m)	INFERRED SOIL TYPE
		48.9	Continued
	11		SILTY CLAY
	12		
	13		
	14		
	15		
	16		
	17		
	18		
	19		
	20		
	21		
	22	37.9	

DYNAMIC PENETRATION RESISTANCE

— BLOWS / FT. (300 mm)



CONE PENETRATION TEST

C.P.T. No. 89-2

CLIENT Delcan Corporation

PROJECT No. 0-19375

LOCATION Regional Road 47 at Highway 17, Cumberland, Ontario

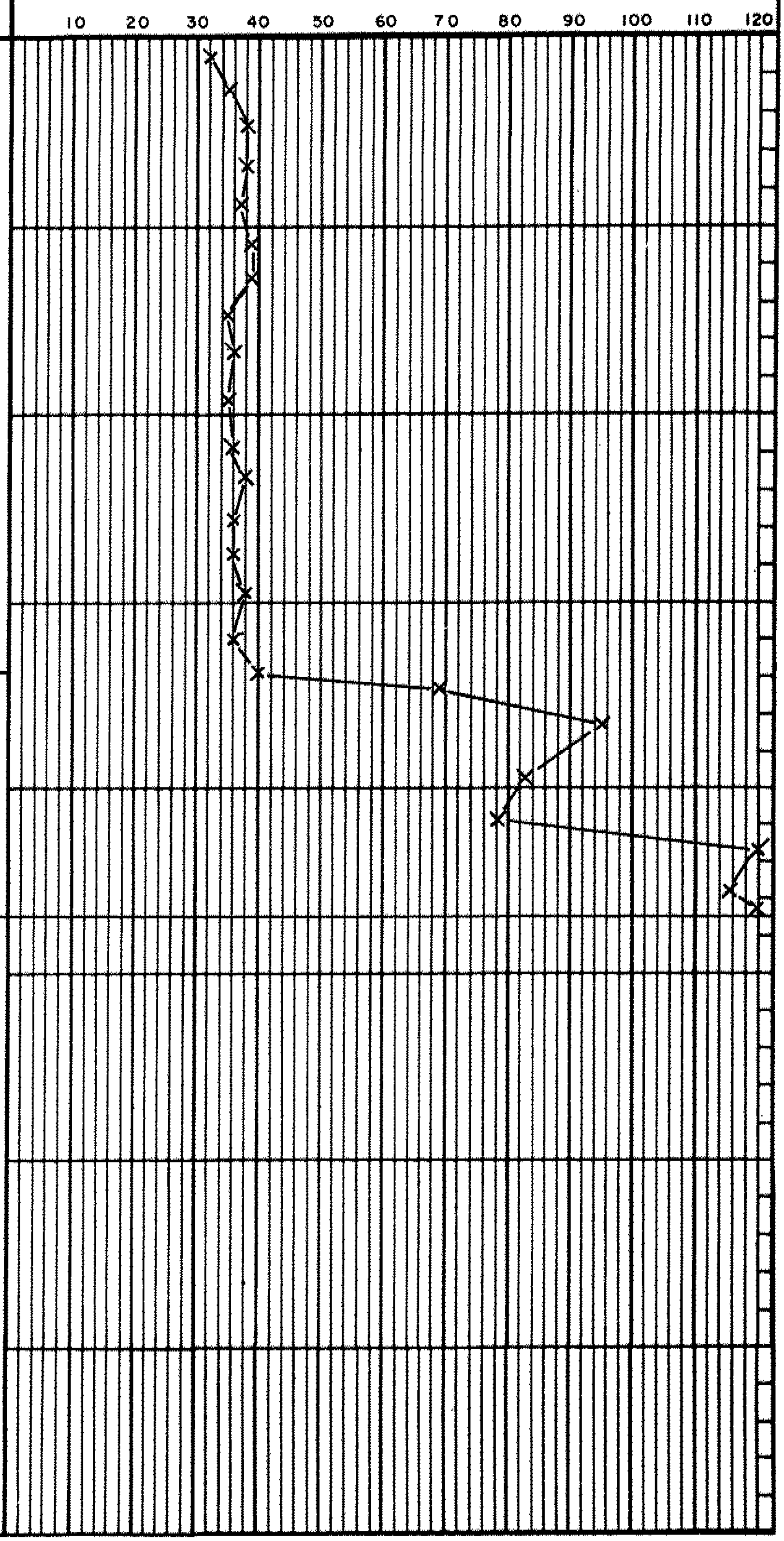
DATE November 10, 1989

DATUM Geodetic

DEPTH (FT.)	DEPTH (M)	ELEVATION (m)	INFERRED SOIL TYPE
		37.9	Continued
	22		SILTY CLAY
	23		
	24		
	25		
	26		
	27	32.7	TILL or weathered BEDROCK
	28		
	29	30.7	End of Dynamic Cone Penetration Test (Advance refusal on dense till or possible bedrock)

DYNAMIC PENETRATION RESISTANCE

— BLOWS / FT. (300 mm)



CONE PENETRATION TEST

C.P.T. No. 89-3

PROJECT No. 0-19375

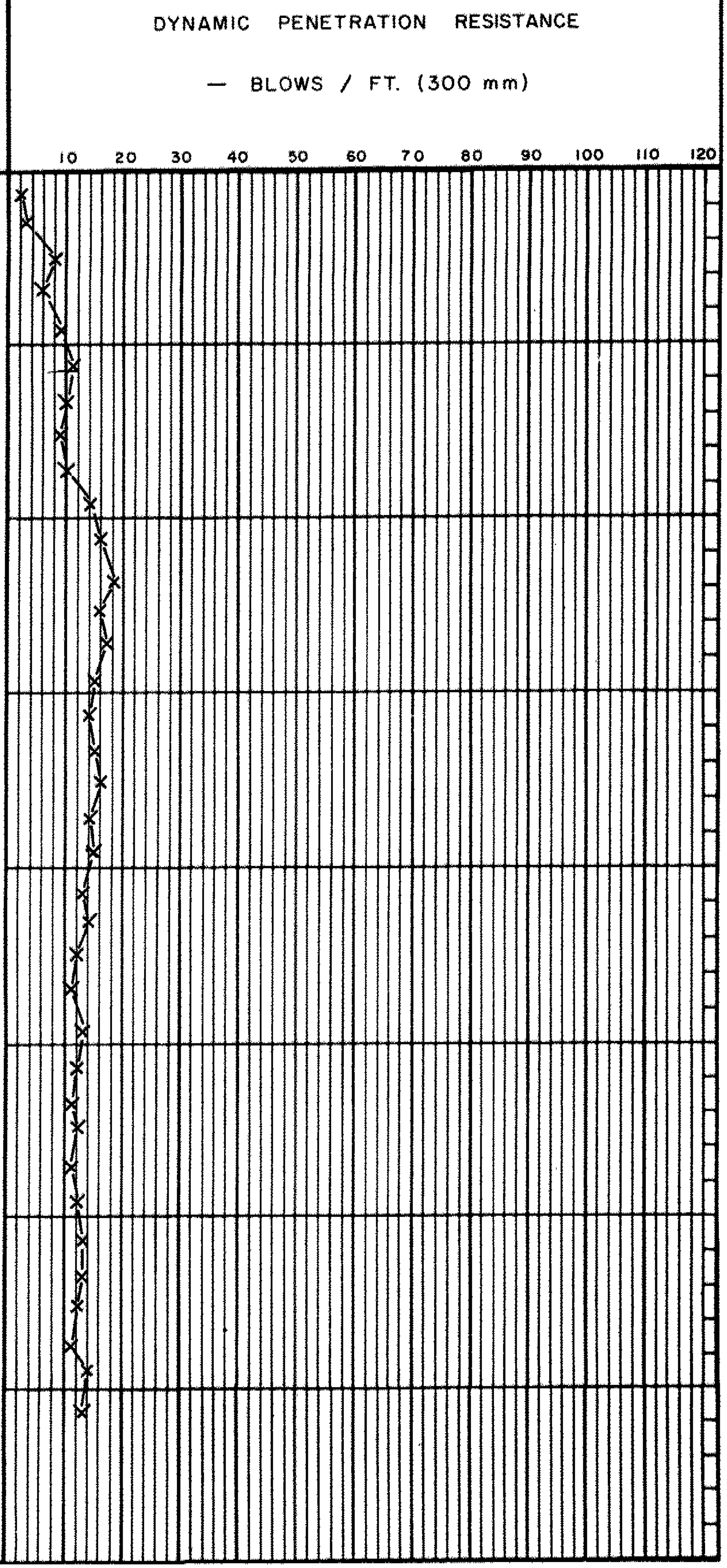
CLIENT Delcan Corporation

LOCATION Regional Road 47 at Highway 17, Cumberland, Ontario

DATE November 10, 1989

DATUM Geodetic

DEPTH (FT.)	DEPTH (M)	ELEVATION (m)	INFERRED SOIL TYPE
		60.1	Ground Surface
	0		SILTY CLAY CRUST
	1		
	2		
	3		
		56.6	SILTY CLAY
	4		
	5		
	6		
	7		
	8		
	9		
	10		
		49.1	
	11		Continued



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CONE PENETRATION TEST

C.P.T. No. 89-3

CLIENT: Delcan Corporation

LOCATION: Regional Road 47 at Highway 17, Cumberland, Ontario

DATE: November 10, 1989

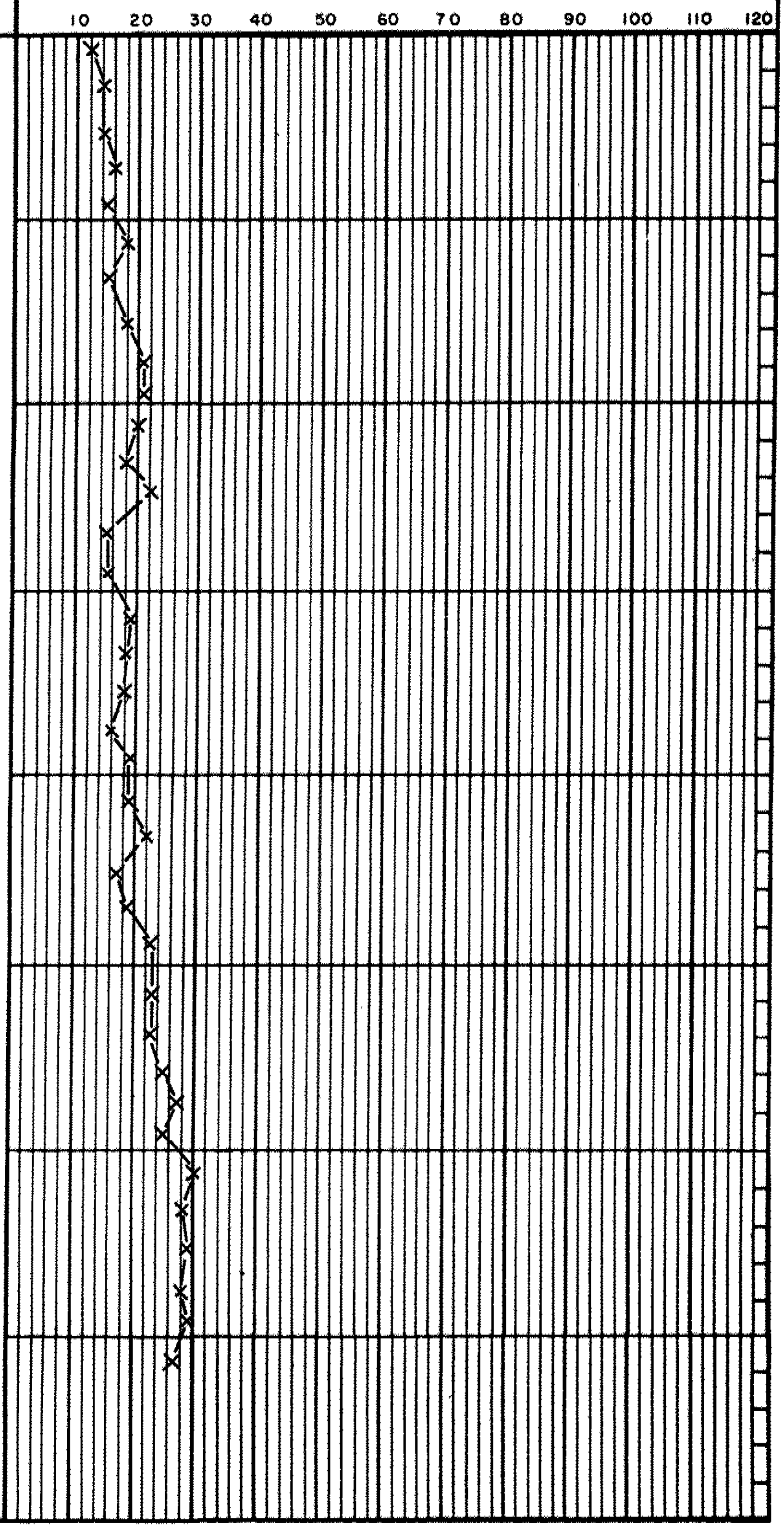
PROJECT No. 0-19375

DATUM: Geodetic

DEPTH (FT.)	DEPTH (M)	ELEVATION (m)	INFERRED SOIL TYPE
		49.1	Continued
	11		SILTY CLAY
	12		
	13		
	14		
	15		
	16		
	17		
	18		
	19		
	20		
	21		
		38.1	
	22		Continued

DYNAMIC PENETRATION RESISTANCE

— BLOWS / FT. (300 mm)



CONE PENETRATION TEST

C.P.T. No. 89-3

CLIENT Delcan Corporation

LOCATION Regional Road 47 at Highway 17, Cumberland, Ontario

DATE November 10, 1989

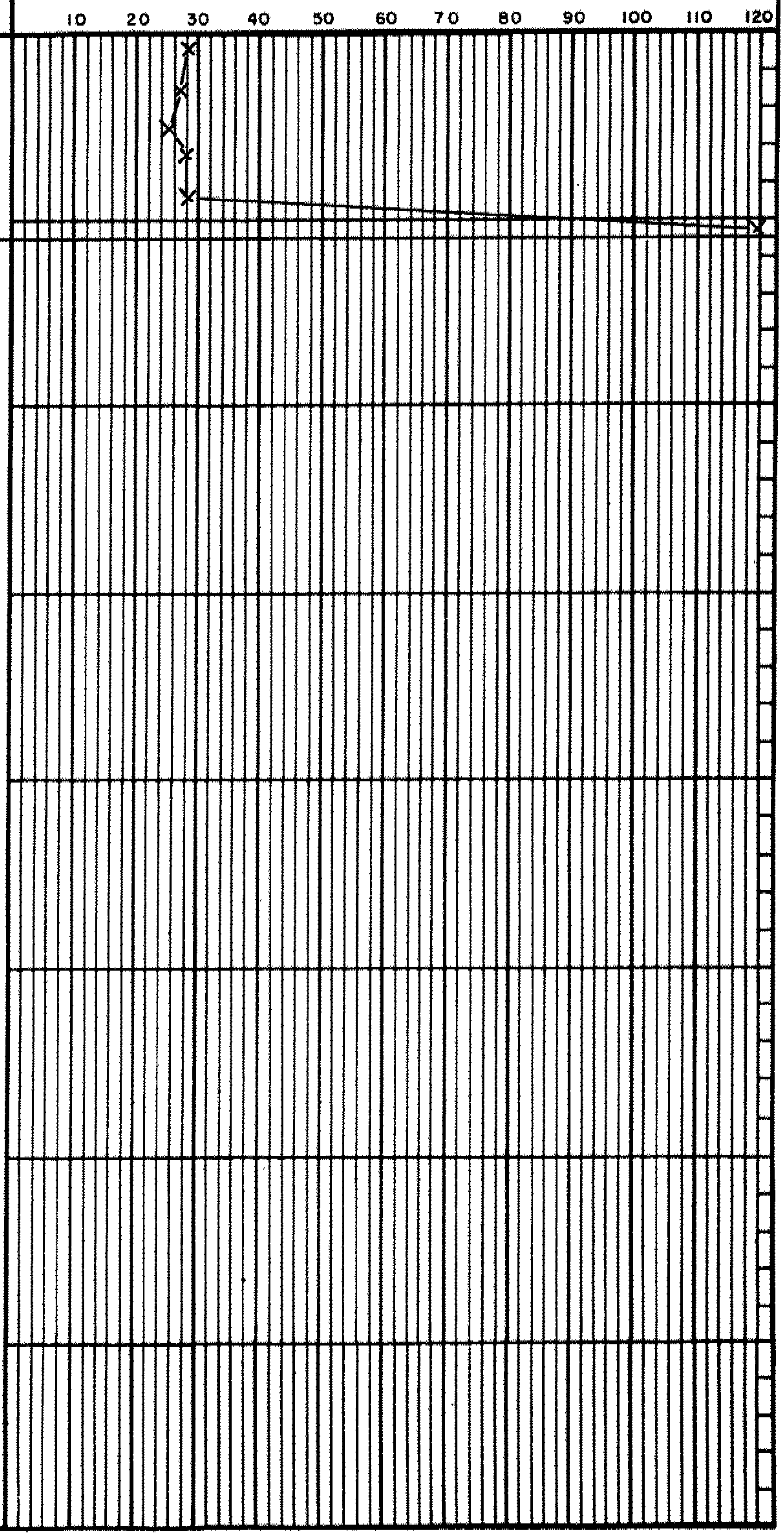
PROJECT No. 0-19375

DATUM Geodetic

DEPTH (FT.)	DEPTH (M)	ELEVATION (m)	INFERRED SOIL TYPE
		38.1	Continued
	22		SILTY CLAY
	23	36.5	
	24		End of Dynamic Cone Penetration Test (Advance refusal on dense till or possible bedrock)

DYNAMIC PENETRATION RESISTANCE

— BLOWS / FT. (300 mm)



CONE PENETRATION TEST

C.P.T. No. 89-4

PROJECT No. 0-19375

CLIENT Delcan Corportaion

LOCATION Regional Road 47 at Highway 17, Cumberland, Ontario

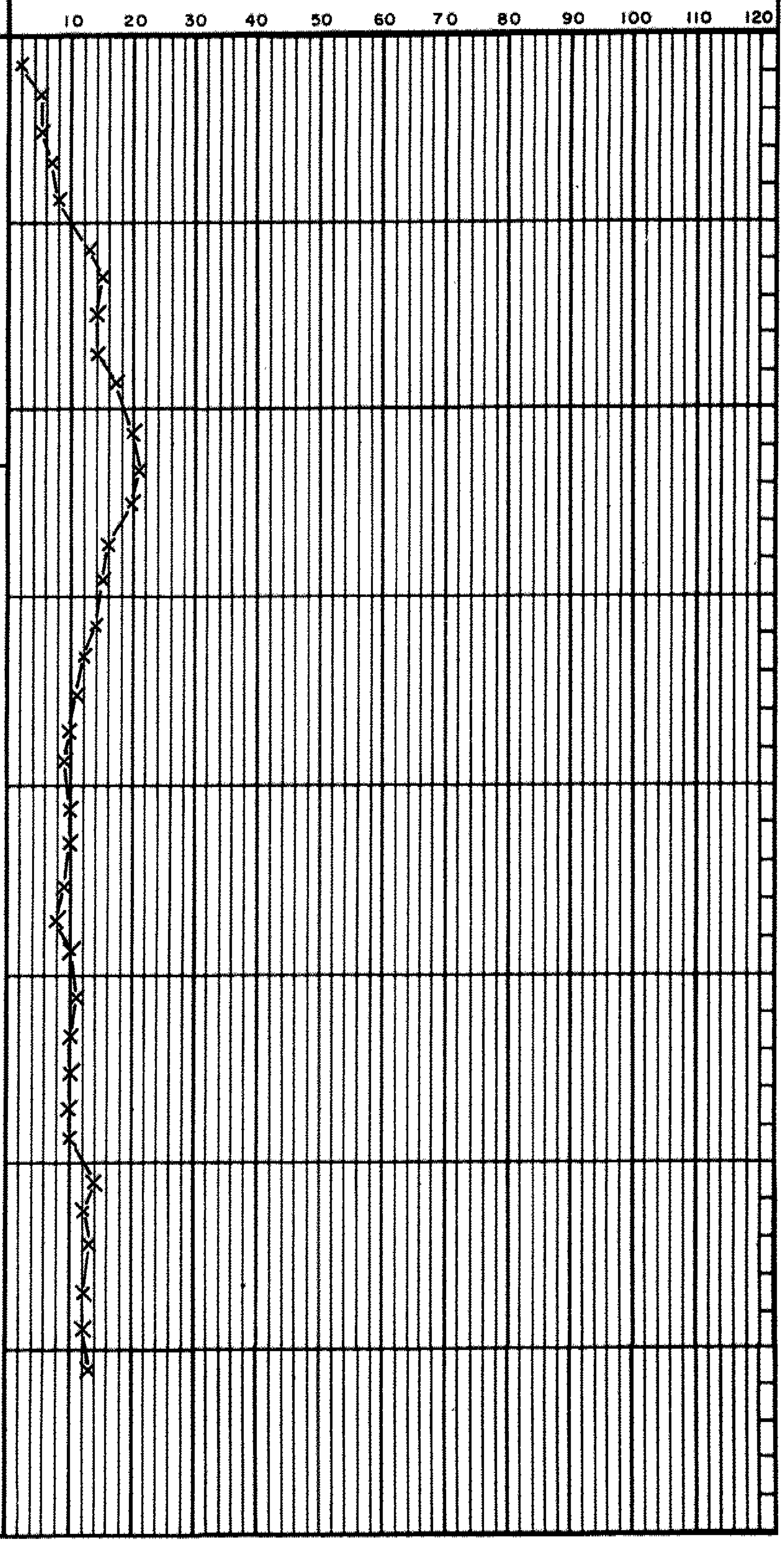
DATE November 16, 1989

DATUM Geodetic

DEPTH (FT.)	DEPTH (M)	ELEVATION (m)	INFERRED SOIL TYPE
		60.4	Ground Surface
	0		SILTY CLAY CRUST
	3.5	56.8	SILTY CLAY
	11	49.4	Continued

DYNAMIC PENETRATION RESISTANCE

— BLOWS / FT. (300 mm)



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CONE PENETRATION TEST

C.P.T. No. 89-4

PROJECT No. 0-19375

CLIENT Delcan Corporation

LOCATION Regional Road 47 at Highway 17, Cumberland, Ontario

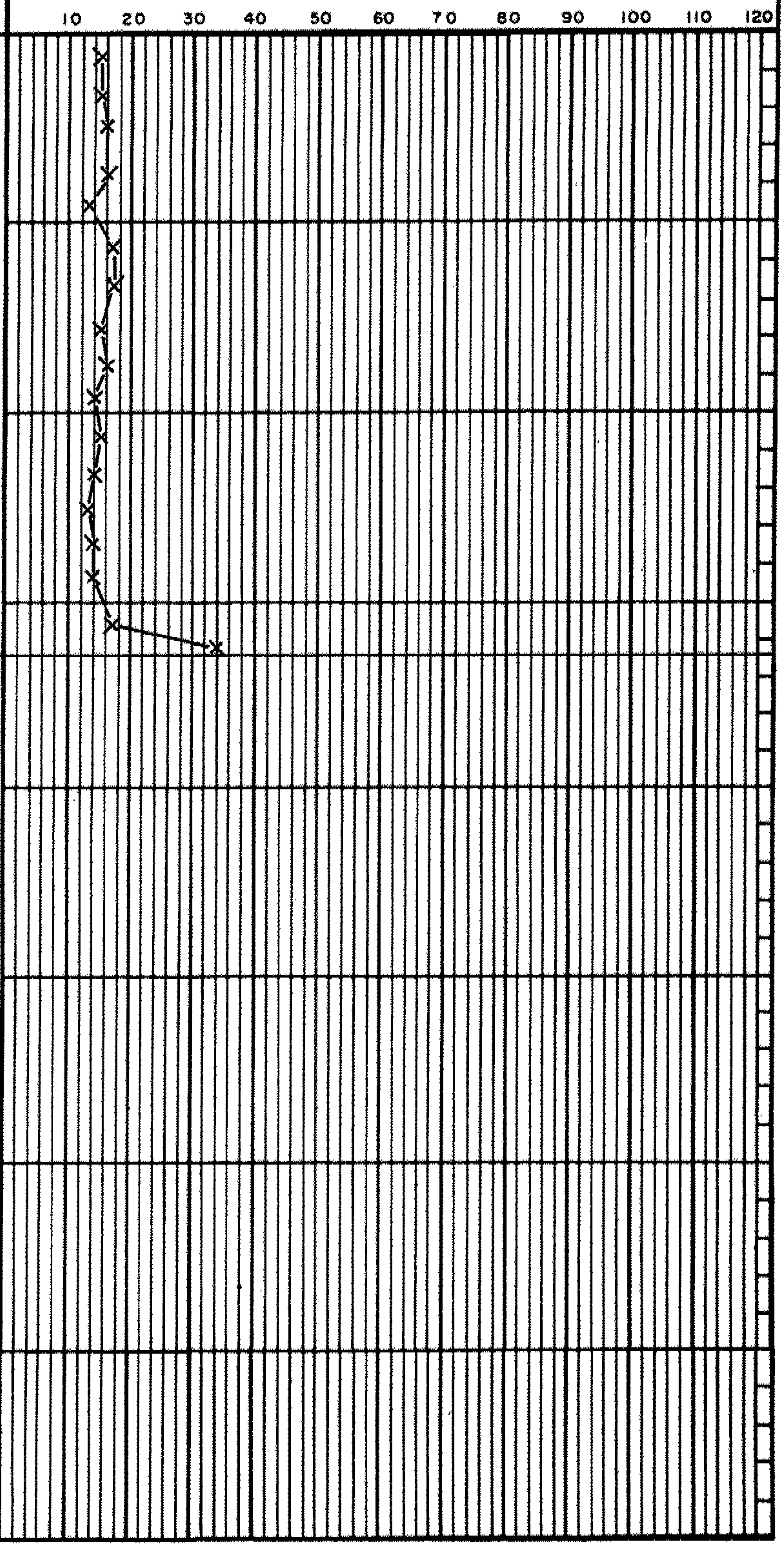
DATE November 16, 1989

DATUM Geodetic

DEPTH (FT.)	DEPTH (M)	ELEVATION (m)	INFERRED SOIL TYPE
		49.4	Continued
	11		SITLY CLAY
	12		
	13		
	14		
	15		
	16	44.5	End of Dynamic Cone Penetration Test. Note: Advance refusal was not obtained. Cone head slipped on inferred bedrock and continued to 45.5m where test was halted.

DYNAMIC PENETRATION RESISTANCE

— BLOWS / FT. (300 mm)



CONE PENETRATION TEST

C.P.T. No. 89-5

PROJECT No. 0-19375

DATUM Geodetic

CLIENT Delcan Corporation

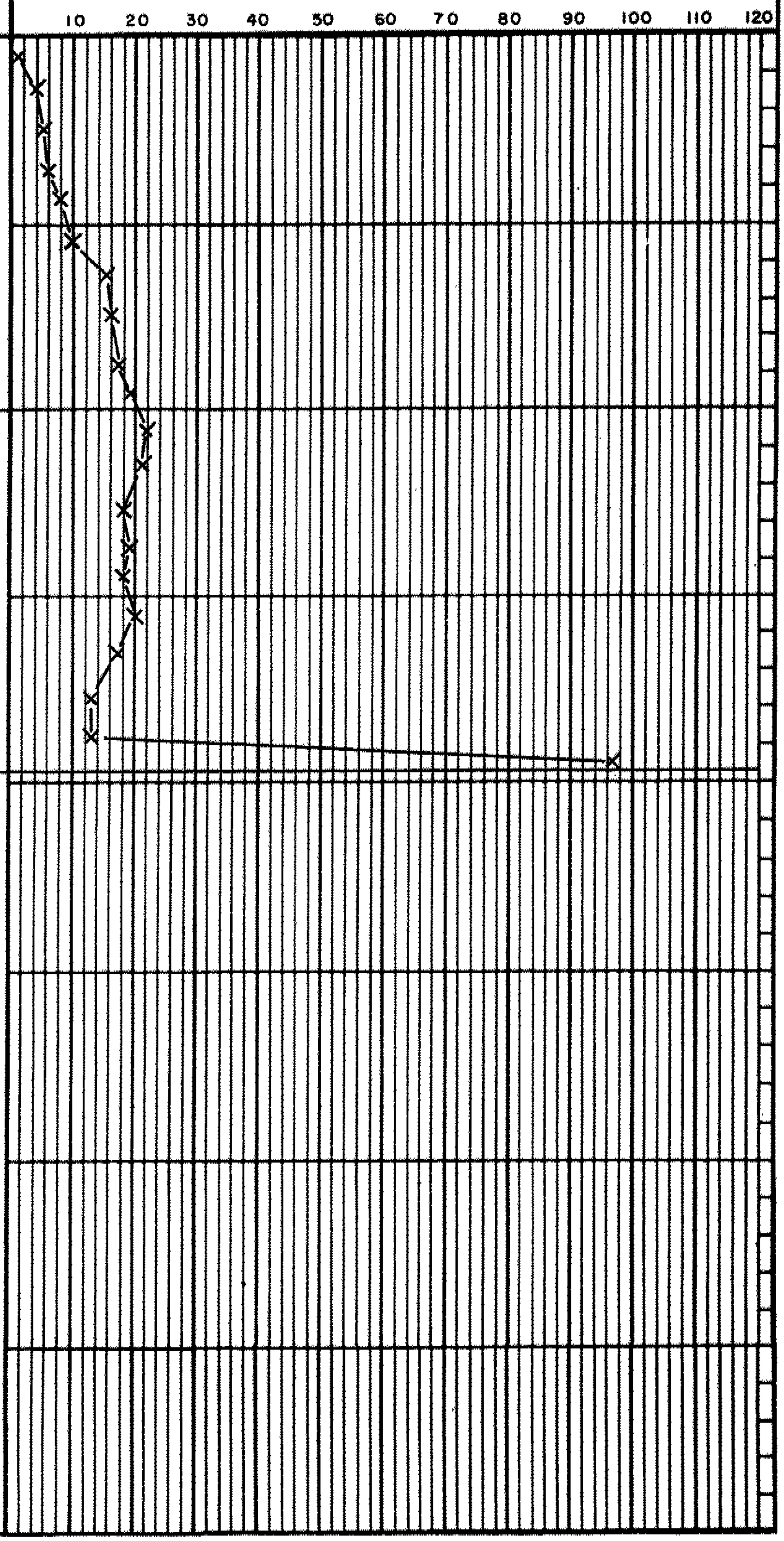
LOCATION Regional Road 47 at Highway 17, Cumberland, Ontario

DATE November 16, 1989

DEPTH (FT.)	DEPTH (M)	ELEVATION (m)	INFERRED SOIL TYPE
	0	60.7	SILTY CLAY CRUST
	1		
	2		
	3		SILTY CLAY
	4		
	5		
	6	54.7	End of Dynami Cone Penetration Test (Advance refusal on dense till or possible bedrock)

DYNAMIC PENETRATION RESISTANCE

— BLOWS / FT. (300 mm)



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CONE PENETRATION TEST

C.P.T. No. 89-6

PROJECT No. 0-19375

CLIENT Delcan Corporation

LOCATION Regional Road 47 at Highway 17, Cumberland, Ontario

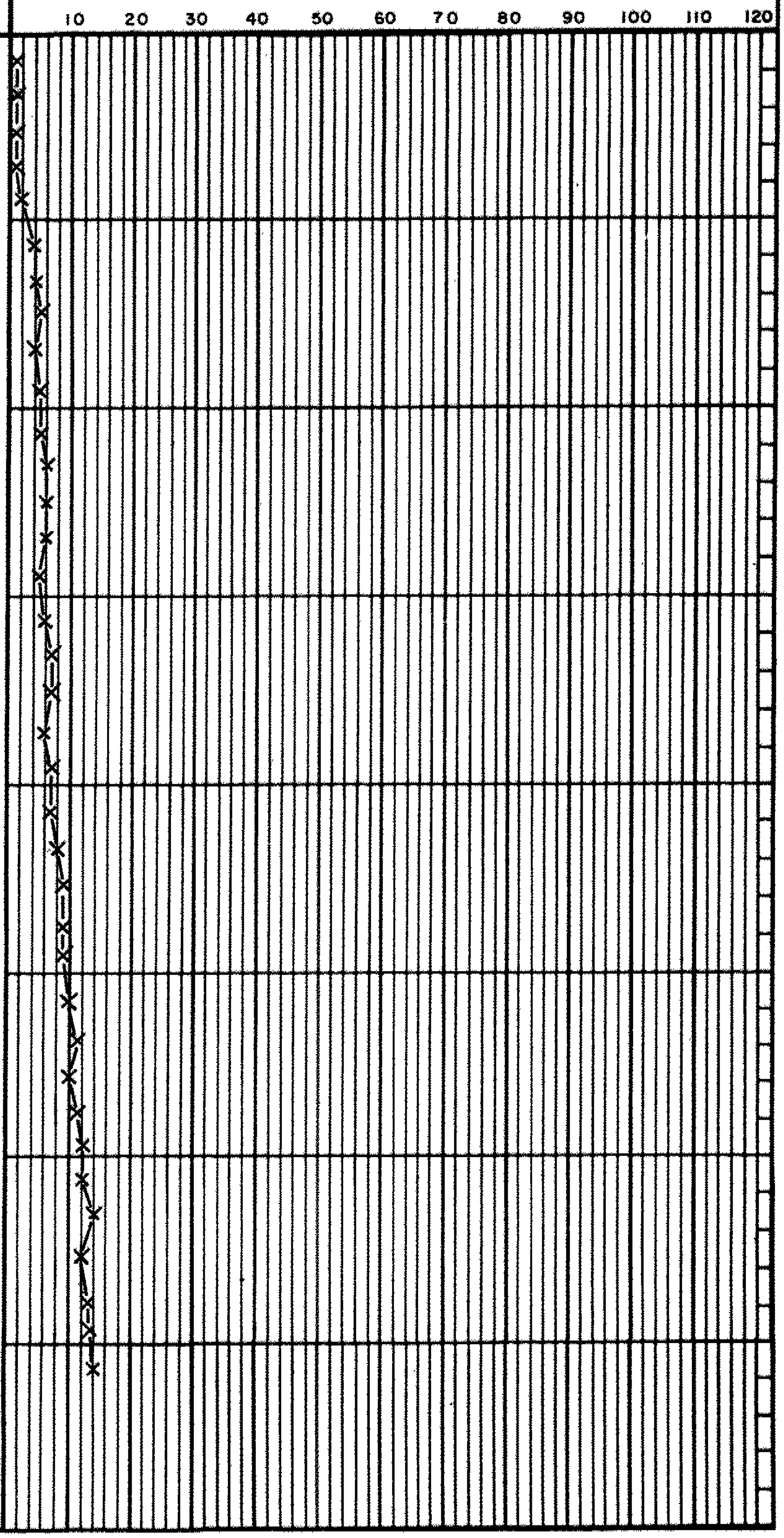
DATE November 26, 1989

DATUM Geodetic

DEPTH (FT.)	DEPTH (M)	ELEVATION (m)	INFERRED SOIL TYPE
		54.3	Ground Surface
	0		SILTY CLAY
	1		
	2		
	3		
	4		
	5		
	6		
	7		
	8		
	9		
	10		
	11	43.3	Continued

DYNAMIC PENETRATION RESISTANCE

— BLOWS / FT. (300 mm)



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CONE PENETRATION TEST

C.P.T. No. 89-6

CLIENT: Delcan Corporation

LOCATION: Regional Road 47 at Highway 17, Cumberland, Ontario

DATE: November 26, 1989

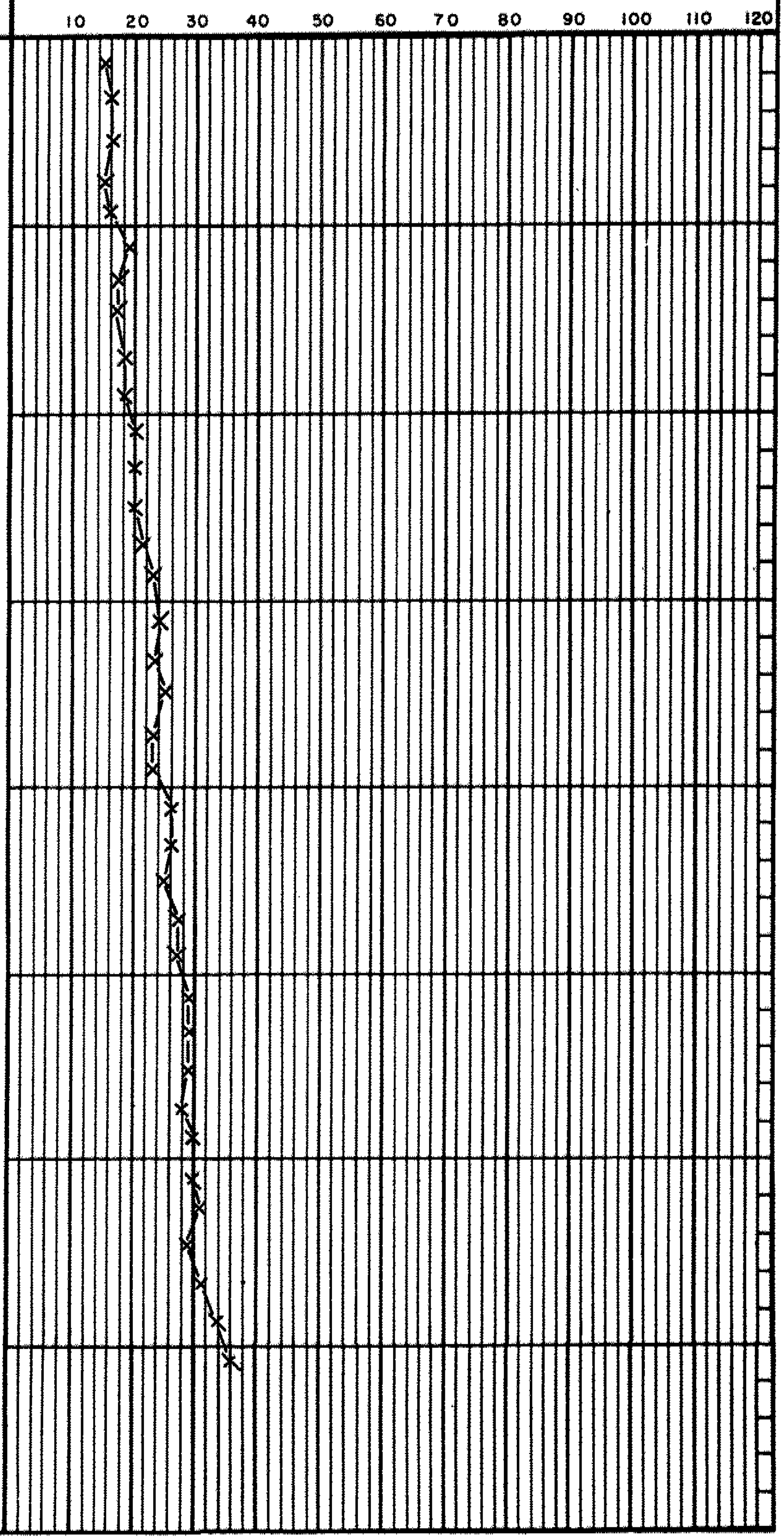
PROJECT No. 0-19375

DATUM: Geodetic

DEPTH (FT.)	DEPTH (M)	ELEVATION (m)	INFERRED SOIL TYPE
		43.3	Continued
	11		SILTY CLAY
	12		
	13		
	14		
	15		
	16		
	17		
	18		
	19		
	20		
	21		
	22	32.3	Continued

DYNAMIC PENETRATION RESISTANCE

— BLOWS / FT. (300 mm)



CONE PENETRATION TEST

C.P.T. No. 89-6

PROJECT No. 0-19375

CLIENT Delcan Corporation

LOCATION Regional Road 47 at Highway 17, Cumberland, Ontario

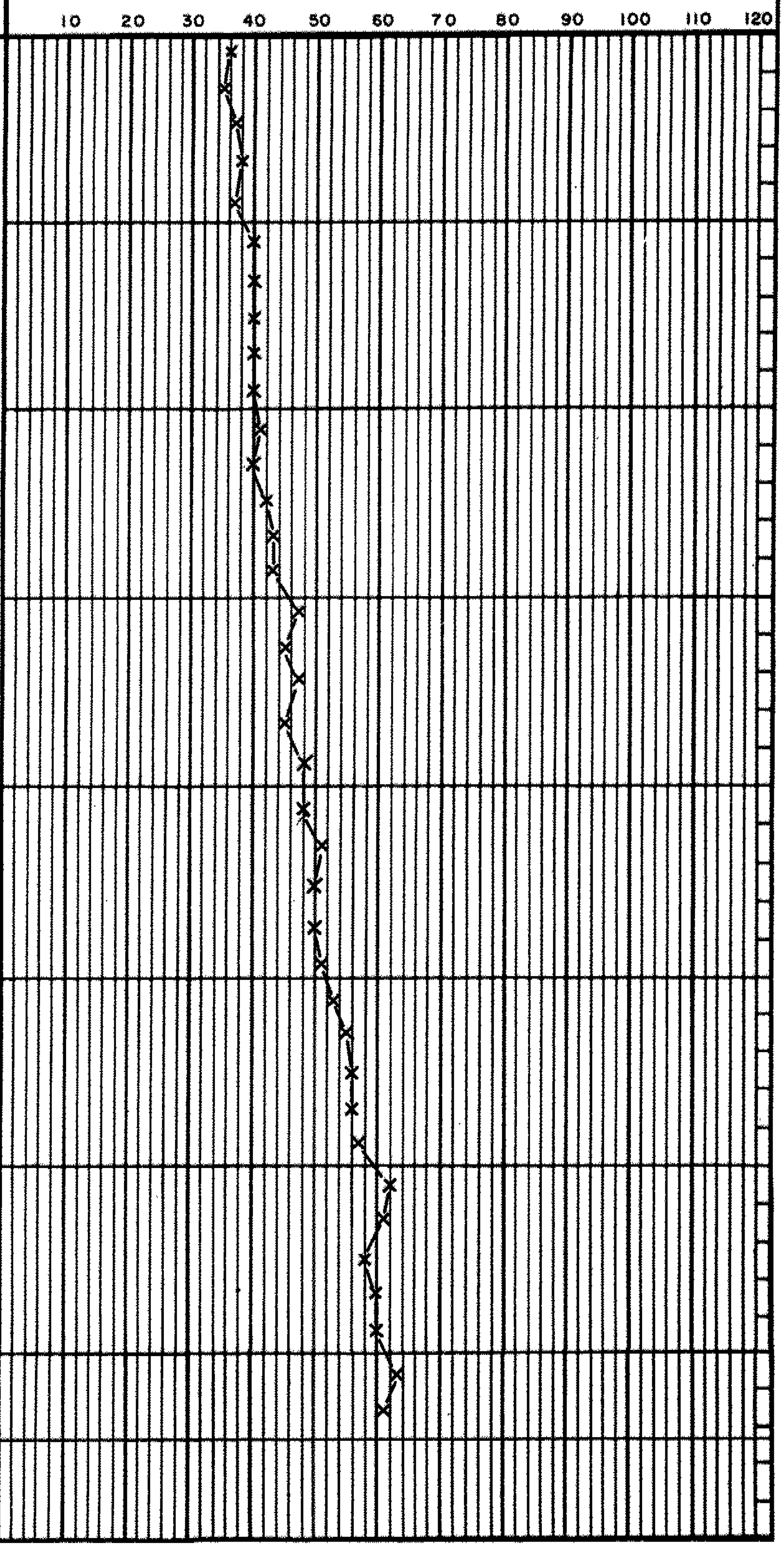
DATE November 26, 1989

DATUM Geodetic

DEPTH (FT.)	DEPTH (M)	ELEVATION (m)	INFERRED SOIL TYPE
	22	32.3	Continued
	22–33		SILTY CLAY
			(Advanced refusal on dense till or possible bedrock)
		20.9	End of Dynamic Cone Penetration Test

DYNAMIC PENETRATION RESISTANCE

— BLOWS / FT. (300 mm)



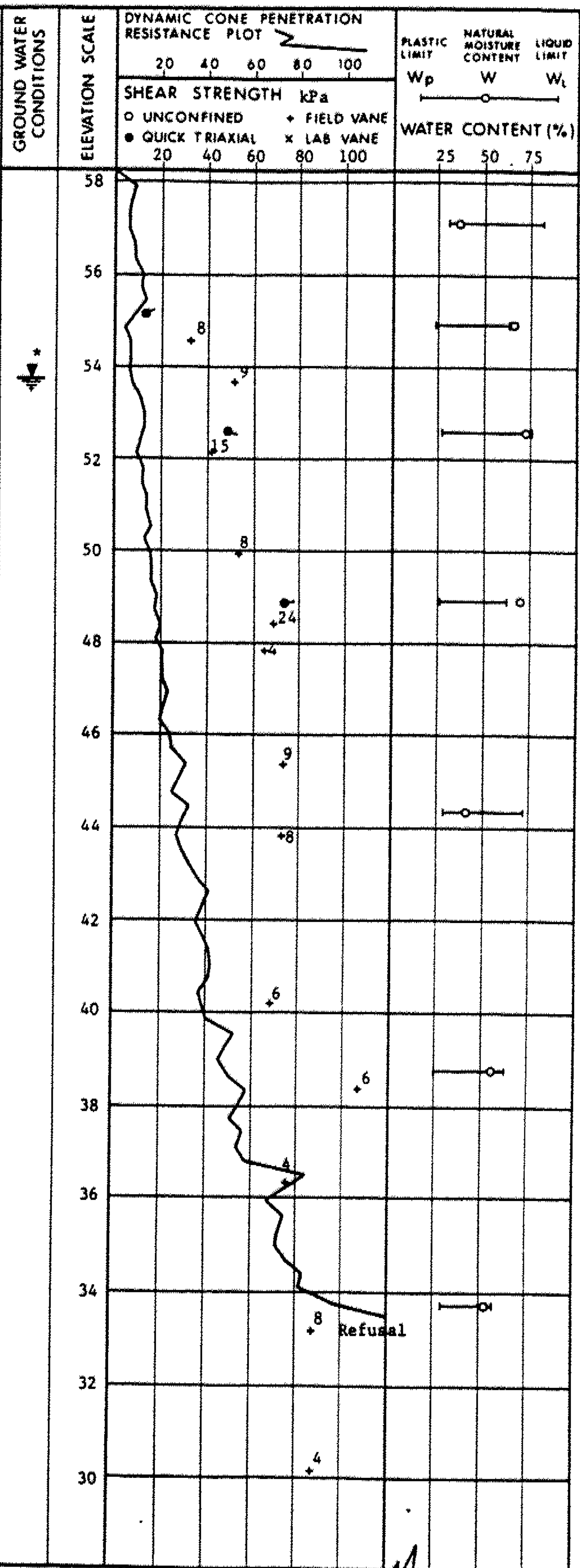
RECORD OF BOREHOLE No 1 — METRIC

W P 11-81-02 — LOCATION STA. 10 + 040 ℄ 10TH Line Road Extension — ORIGINATED BY D.Y.

DIST 9 HWY 17 — BOREHOLE TYPE Hollow Stem, Cone Test — COMPILED BY D.Y.

DATUM Geodetic — DATE 86 01 15 to 86 01 17 — CHECKED BY

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	'N' VALUES	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
58.2	Ground Surface					
0.0	Clay (Desiccated)					
	Very Stiff	1	SS	10		
		2	SS	8		
	(Brown)					
		3	TW	PH	16.3	
	Clay of High Plasticity					
		4	SS	2		
	Firm	5	TW	PM	16.8	
		6	SS	5		
		7	TW	PM	15.9	
		8	SS	7		
	Stiff					
		9	TW	PH	16.3	
		10	SS	6		
		11	TW	PH	16.3	
		12	SS	6		
		13	TW	PH	17.1	
		14	SS	12		
	(Grey)					
28.0 / 30.2						



Continued

$^{+3}$, x^5 : Numbers refer to Sensitivity

20 / 15 ◇ 5 / 10 (%) STRAIN AT FAILURE

Continued

Ontario

RECORD OF BOREHOLE No 1 Continued METRIC

W P 11-81-02 LOCATION STA. 10 + 040 ℄ 10TH Line Road Extension ORIGINATED BY D.Y.

DIST 9 HWY 17 BOREHOLE TYPE Hollow Stem, Cone Test COMPILED BY D.Y.

DATUM Geodetic DATE 86 01 15 to 86 01 17 CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	PLASTIC LIMIT Wp / NATURAL MOISTURE CONTENT W / LIQUID LIMIT WL — WATER CONTENT (%)	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES						
28.0 30.2	Continued										
	Clay of High Plasticity		15	TW	PH						
25.7	Stiff		16	SS	**		26				
32.5	End of Borehole										

Refusal to Augering
probable Bedrock

* NOTE: Watertable elevations measured in open borehole

DATE	ELEVATION
86 01 18	50.2
01 20	51.2
01 21	52.5
01 22	53.7

** NOTE: Spoon Bouncing

+3, ×5 : Numbers refer to Sensitivity

15 ⌖ 5 (20 above, 10 below) (%) STRAIN AT FAILURE

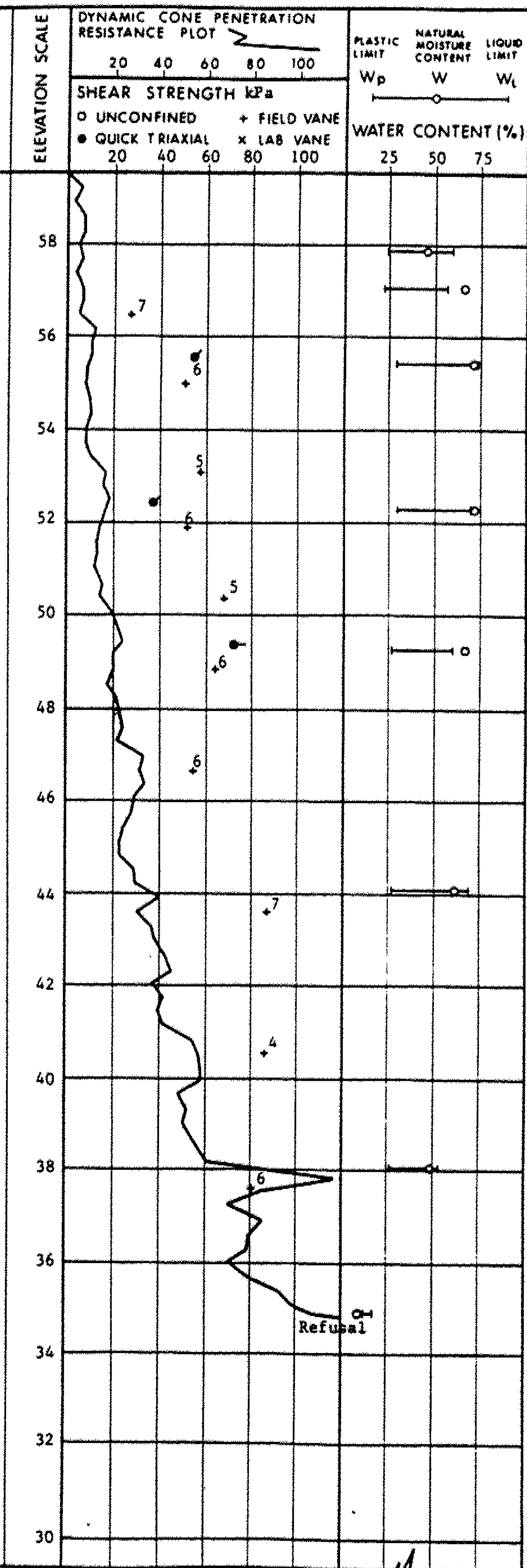
RECORD OF BOREHOLE No 2 — METRIC

W P 11-81-02 — LOCATION STA. 9 + 940 ℄ 10TH Line Road Extension — ORIGINATED BY D.Y.

DIST 9 HWY 17 — BOREHOLE TYPE Hollow Stem Auger, BX Rock Core, Cone Test — COMPILED BY D.Y.

DATUM Geodetic — DATE 86 01 17 to 86 01 22 — CHECKED BY

ELEV / DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	'N' VALUES	UNIT WEIGHT γ kN/m³	GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
59.5	Ground Surface					
0.0	Clay (Desiccated) Very Stiff (Brown)	1	SS	19		
		2	SS	9		
	Clay of High Plasticity	3	SS	2		
	Firm	4	TW	PM	15.6	
		5	SS	1		
		6	TW	PH	15.7	
	Stiff	7	SS	2		
		8	TW	PM	15.9	
		9	SS	3		
		10	TW	PH	16.2	
		11	SS	6		
		12	TW	PH	17.3	
35.3 / 24.2	(Grey)					
	Silty Sand, Some Gravel Very Dense	13	SS	65		24 33 42 1
		14	SS	**		
30.8 / 28.7	Boulders	15	BX RC	78% REC 36% RQD		
	Sound Bedrock Limestone (Grey)	15A	BX RC			
29.3 / 30.2		16	BX RC	94% REC 88% RQD		



Continued

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

Continued

RECORD OF BOREHOLE No 2 Continued METRIC

W P 11-81-02 LOCATION STA. 9 + 940 ℄ 10TH Line Road Extension ORIGINATED BY D.Y.

DIST 9 HWY 17 BOREHOLE TYPE Hollow Stem Auger, BX Rock Core, Cone Test COMPILED BY D.Y.

DATUM Geodetic DATE 86 01 17 to 86 01 22 CHECKED BY [signature]

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L / WATER CONTENT (%)	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES						
29.3	Continued										
28.8	Limestone Bedrock Sound										
30.7	End of Borehole										
	* NOTE: Watertable measured inside augers on same day as augering										
	** NOTE: Spoon Bouncing										

$+^3, \times^5$: Numbers refer to Sensitivity

15 ◇ 5 (%) STRAIN AT FAILURE (20 above, 10 below)

TABLE 1 - BOREHOLE LOCATIONS AND ELEVATIONS

Borehole	Coordinate North (Y)	Coordinate East (X)	Ground Surface Elev.(m)	Depth (m) Auger Refusal	Depth (m) Cone Refusal	Refusal Elevation (m)
BH-89-1	5038625.441	382763.750	56.723		31.4	25.32
BH-89-2	5038667.938	382814.902	57.579	45.5		12.08
BH-89-3	5038587.222	382704.250	56.724		32.8	23.92
BH-89-4	5038707.507	382727.671	59.357		29.0	30.36
BH-89-5	5038535.828	382807.504	60.542	9.0		51.54
BH-89-6	5038755.465	383011.964	53.865	22.2		31.67
CPT-89-1	5038573.868	382775.413	59.329		32.2	27.12
CPT-89-2	5038563.494	382794.762	59.894		29.2	30.69
CPT-89-3	5038549.912	382800.875	60.120		23.6	36.52
CPT-89-4	5038544.014	382803.120	60.350		15.8	44.50
CPT-89-5	5038528.928	382812.040	60.712		6.0	54.71
CPT-89-6	5038651.283	382794.618	54.334		33.4	20.93

NOTE: BH-89-1 indicates borehole
CPT-89-1 indicates cone penetration test

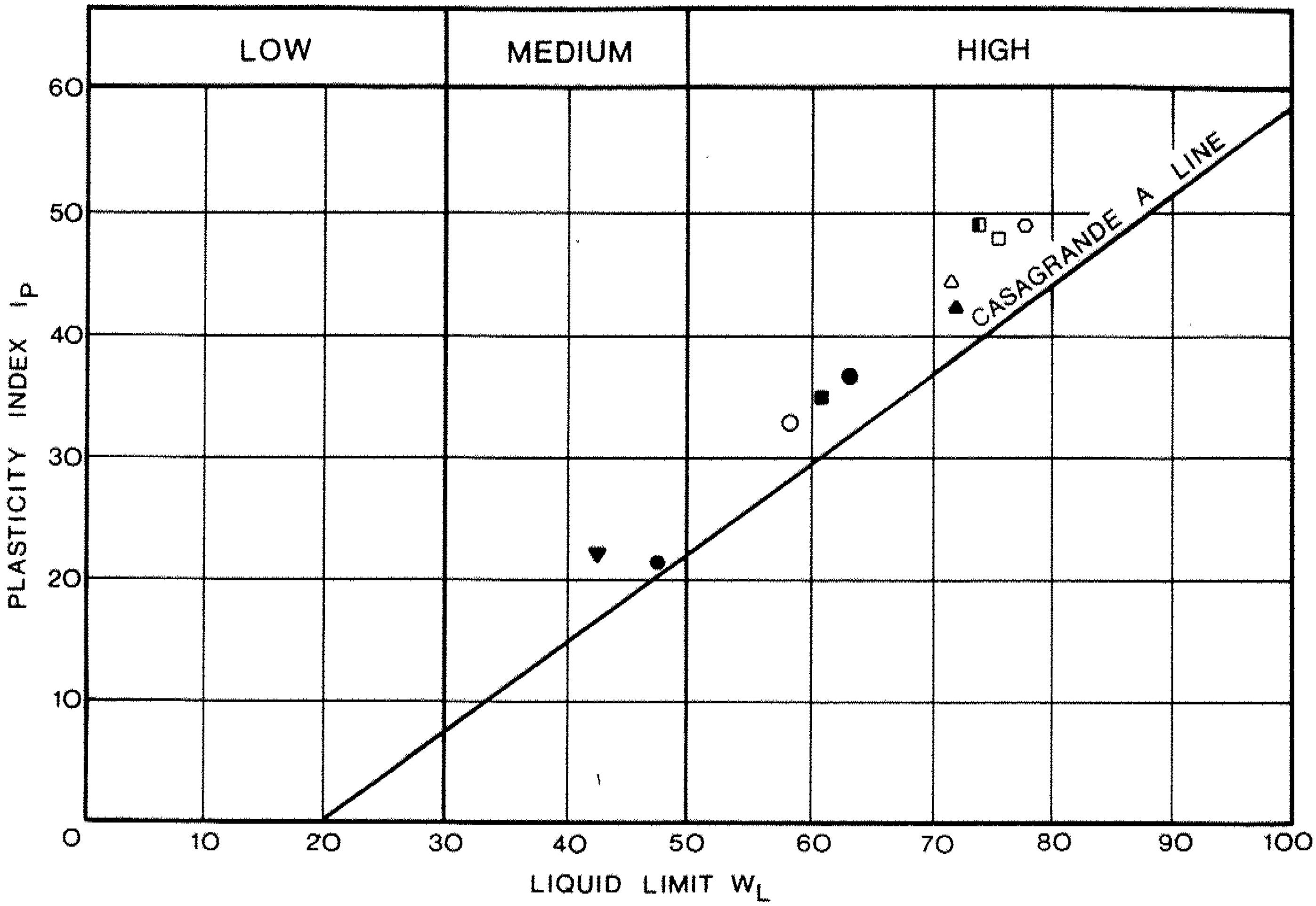
TABLE 2 PARAMETERS USED FOR STABILITY ANALYSES

Material	Elev. Interval (m)	Unit Weight (kn/m³)	Cohesion (kPa)	Friction Angle (°)	r_u Value
Main Embankment Fill	-	20	0	30	0.2
Berm Fill	-	17	5	26	0.3
Clay Crust	OGS*-54.0	17	75	0	**
Grey Clay	54.0-50.0	15	50	0	**
Grey Clay	50.0-45.0	15	56	0	**
Grey Clay	45.0-40.0	15	70	0	**
Grey Clay	40.0-35.0	15	85	0	**
Grey Clay	35.0-30.0	15	100	0	**

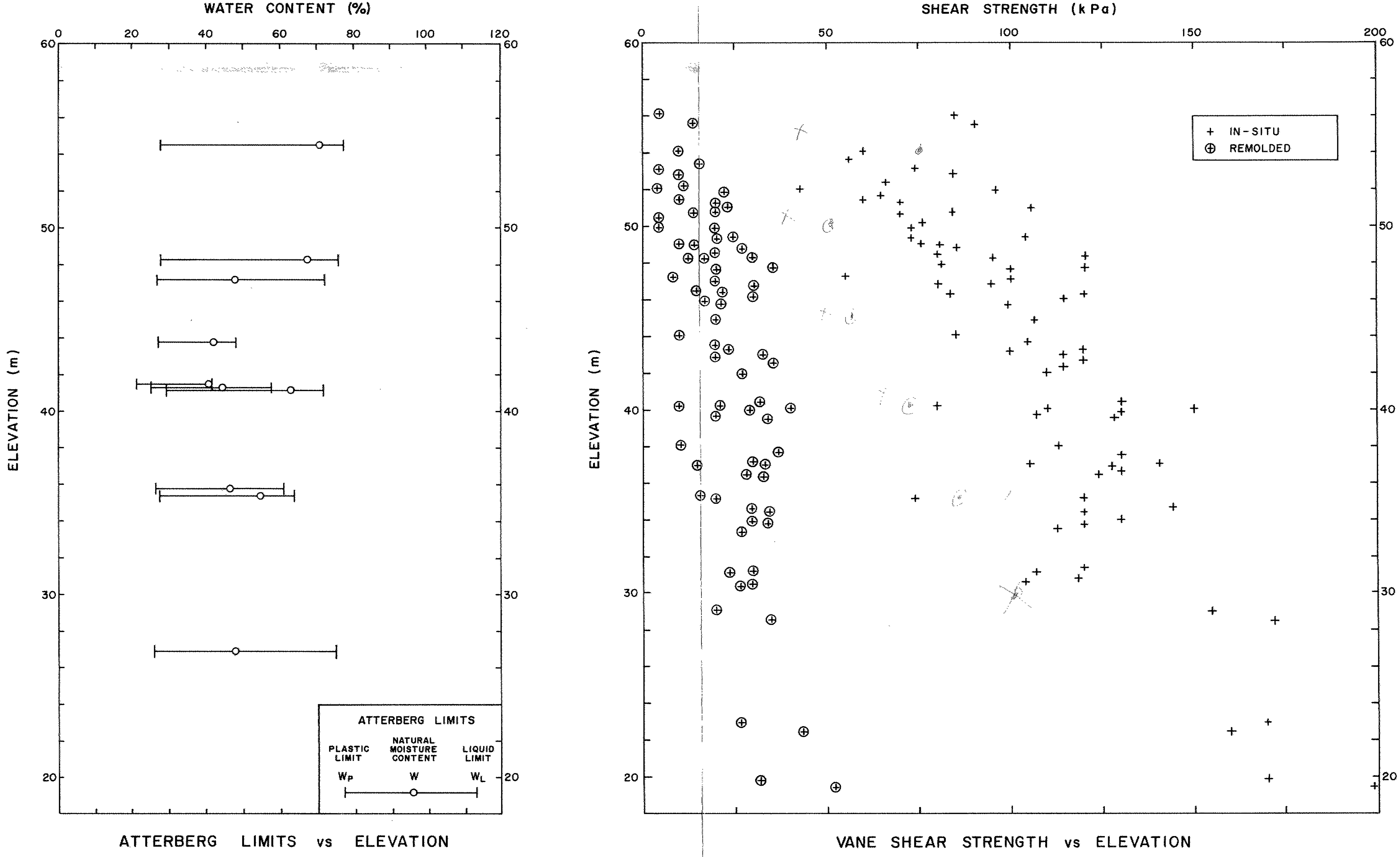
Notes: * OGS = Original Ground Surface
** Piezometric surface at OGS

PLASTICITY CHART

APPENDIX
FIGURE 1
PROJECT U-19375



SYMBOL	BOREHOLE	DEPTH (m)	DESCRIPTION
○	89-1	15.3	Inorganic SILTY CLAY of high plasticity
●	89-1	21.2	Inorganic SILTY CLAY of high plasticity
□	89-2	9.2	Inorganic SILTY CLAY of high plasticity
■	89-2	21.5	Inorganic SILTY CLAY of high plasticity
◧	89-2	30.5	Inorganic SILTY CLAY of high plasticity
△	89-3	9.4	Inorganic SILTY CLAY oq high plasticity
▲	89-3	15.1	Inorganic silty clay of high plasticity
⬡	89-4	4.8	Inorganic SILTY CLAY of high plasticity
⬢	89-4	15.5	Inorganic SILTY CLAY of medium plasticity
▼	89-6	12.4	Inorganic SILTY CLAY of medium plasticity



ATTERBERG LIMITS AND SHEAR STRENGTH vs ELEVATION
BOREHOLES 89-1 TO 89-6

PROJECT No. 0-19375

FIGURE 2

VOID RATIO - PRESSURE CURVE

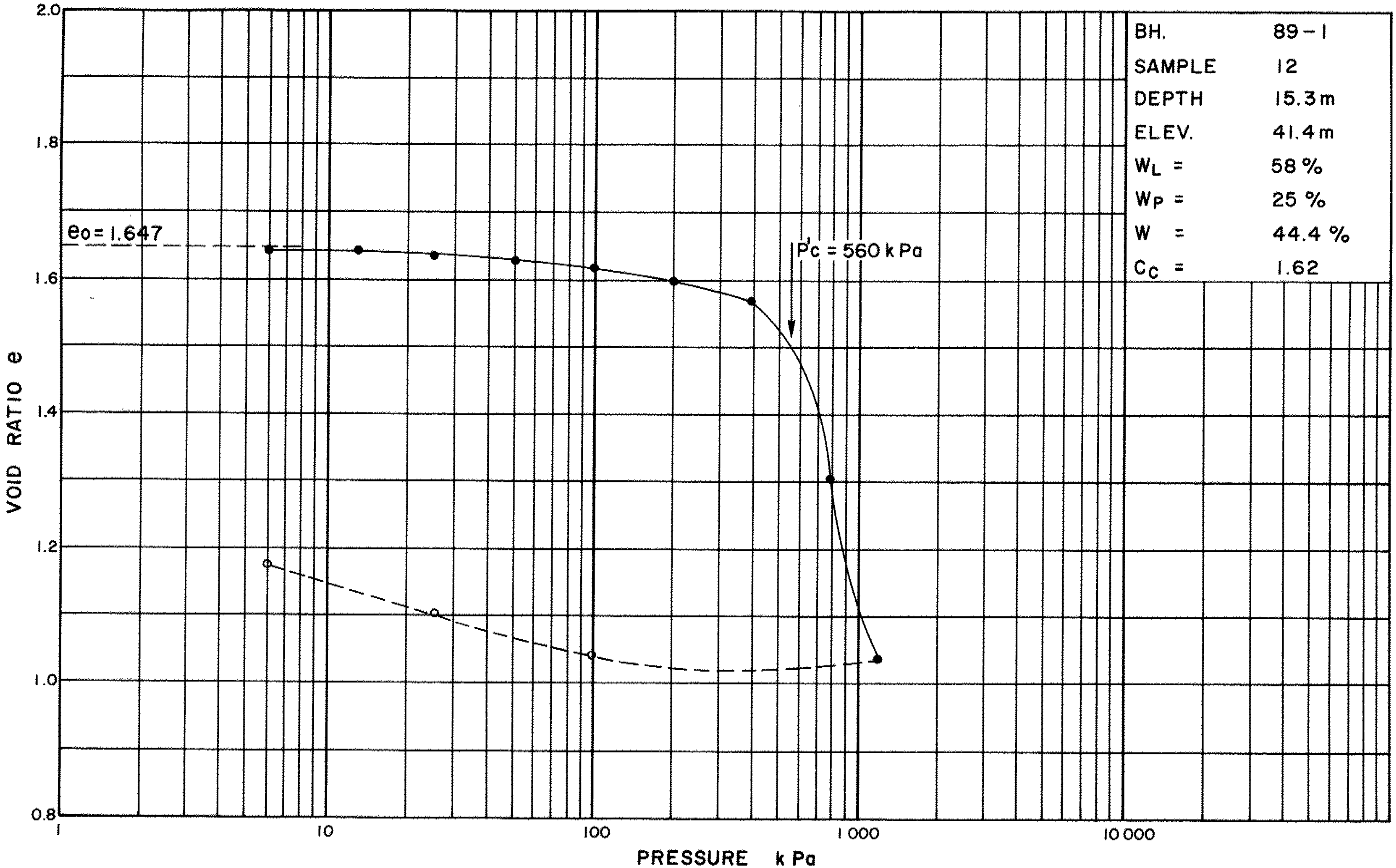


FIGURE 3.1

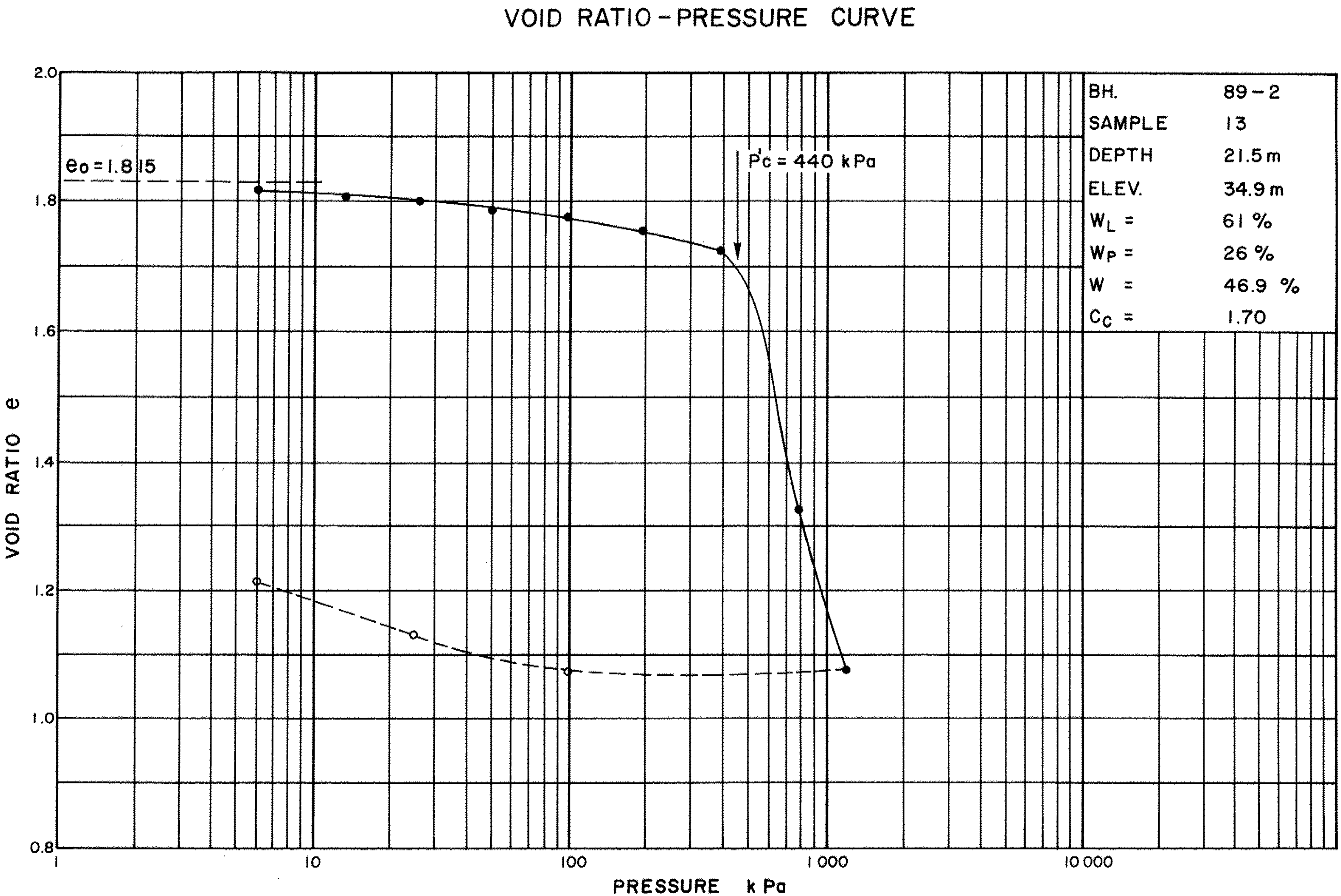

VOID RATIO - PRESSURE CURVE
VOID RATIO e
PRESSURE k Pa
2.0
1.8
1.6
1.4
1.2
1.0
0.8
1
10
100
1 000
10 000
eo = 1.815
P'c = 440 kPa
BH. 89 - 2
SAMPLE 13
DEPTH 21.5 m
ELEV. 34.9 m
WL = 61 %
WP = 26 %
W = 46.9 %
CC = 1.70

FIGURE 3.2

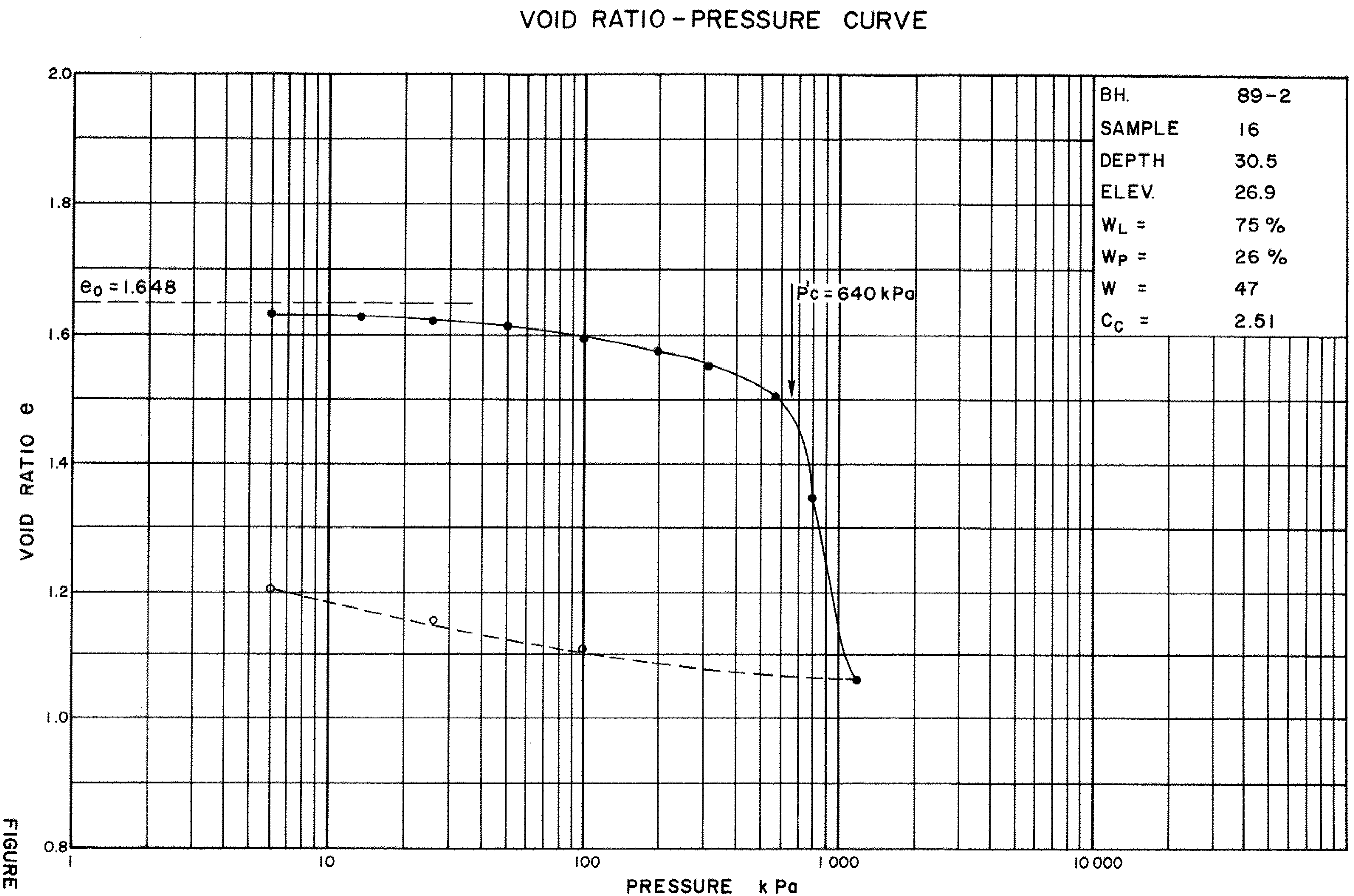

VOID RATIO - PRESSURE CURVE
BH. 89-2
SAMPLE 16
DEPTH 30.5
ELEV. 26.9
W_L = 75 %
W_P = 26 %
W = 47
C_C = 2.51
e_0 = 1.648
P'c = 640 kPa
VOID RATIO e
PRESSURE kPa
2.0
1.8
1.6
1.4
1.2
1.0
0.8
1
10
100
1 000
10 000

FIGURE 3.3

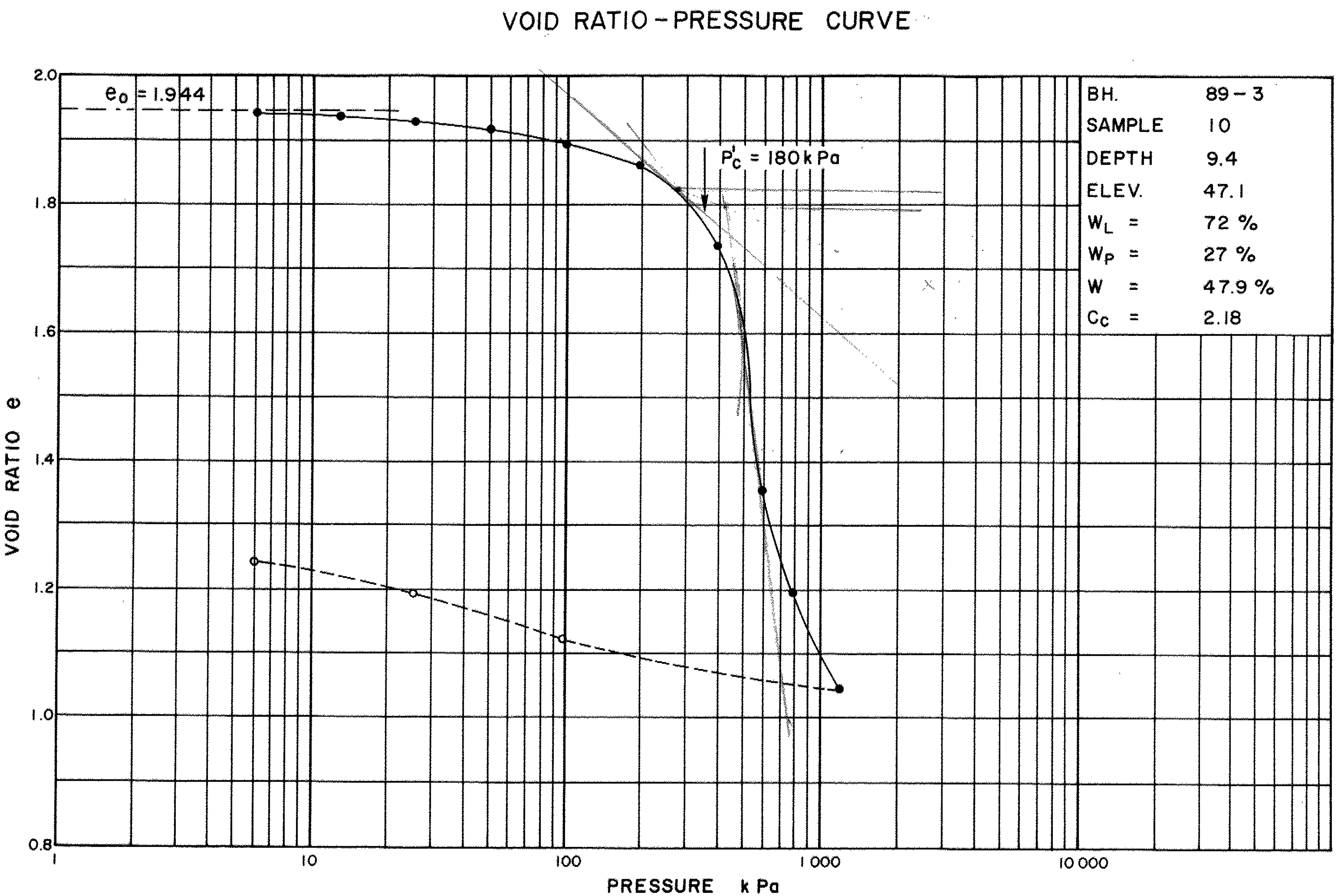

VOID RATIO - PRESSURE CURVE
e_o = 1.944
P'_c = 180 k Pa
BH. 89 - 3
SAMPLE 10
DEPTH 9.4
ELEV. 47.1
W_L = 72 %
W_P = 27 %
W = 47.9 %
C_c = 2.18
VOID RATIO e
PRESSURE k Pa
2.0
1.8
1.6
1.4
1.2
1.0
0.8
1
10
100
1 000
10 000

FIGURE 3.4

VOID RATIO - PRESSURE CURVE

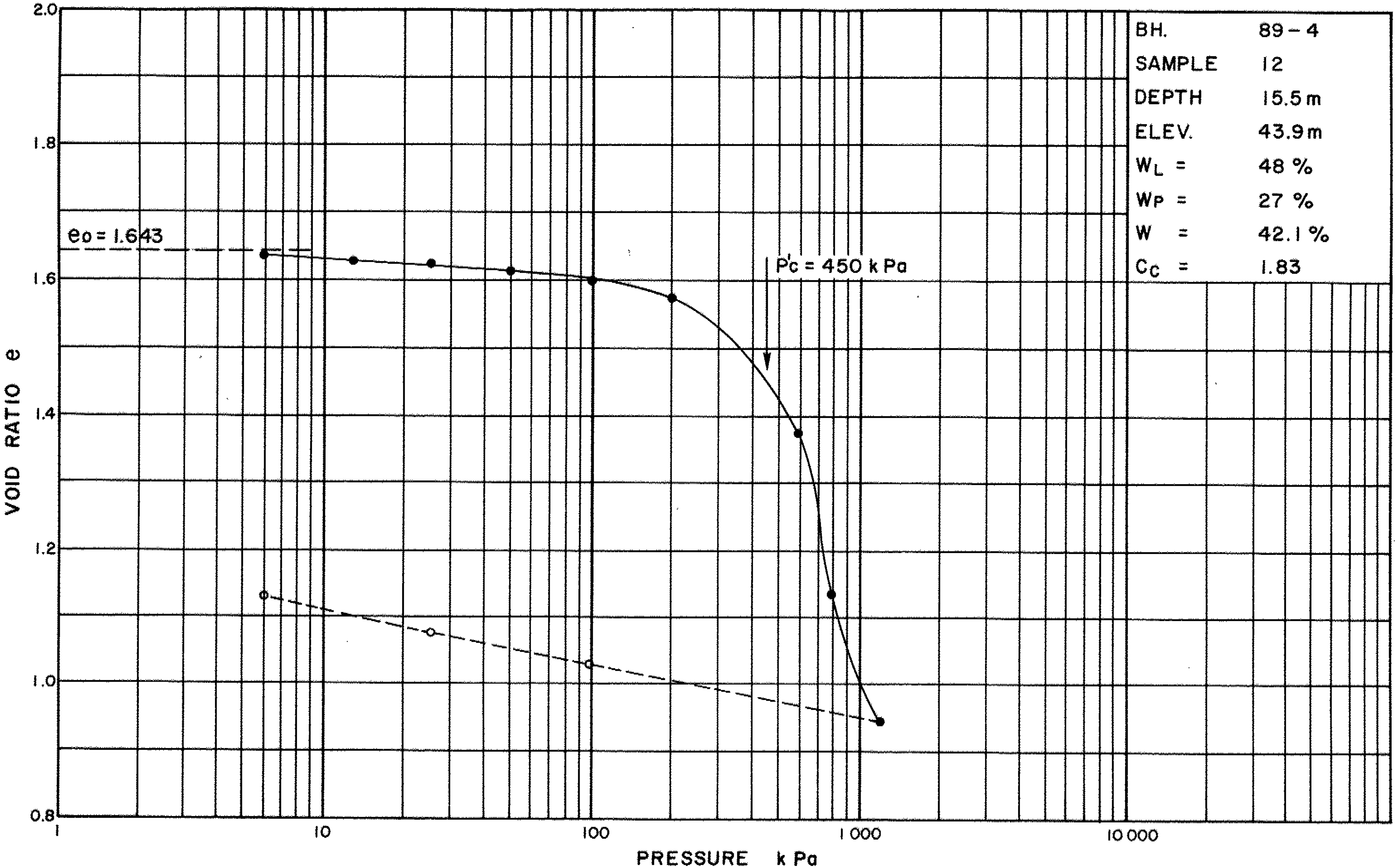


FIGURE 3.5

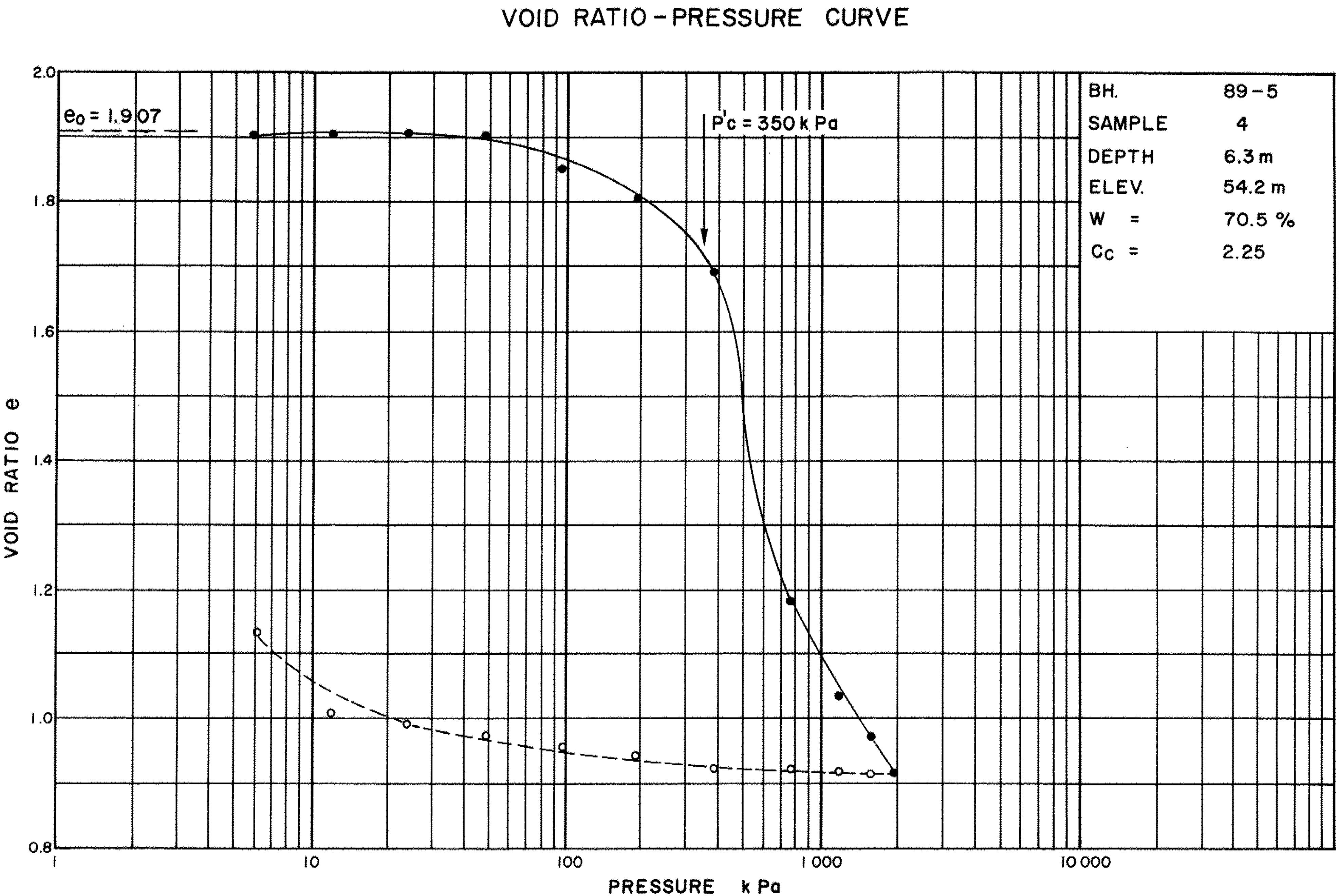
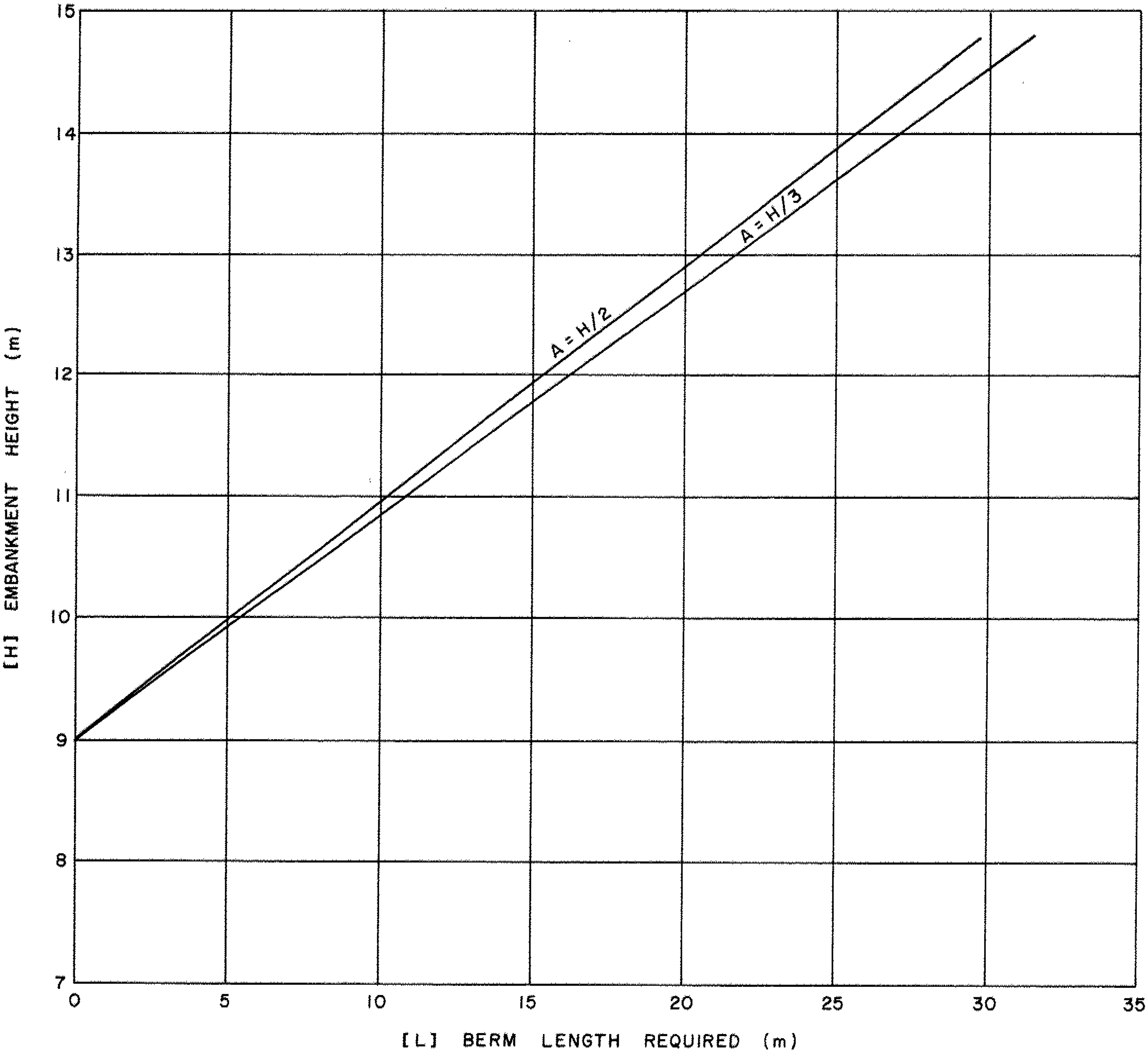
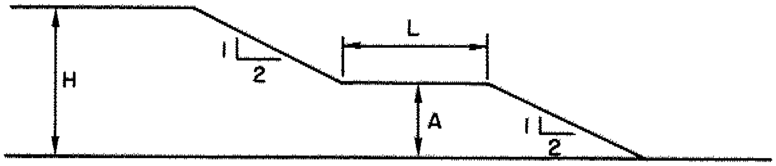

VOID RATIO - PRESSURE CURVE
$e_0 = 1.907$
P'c = 350 k Pa
BH. 89 - 5
SAMPLE 4
DEPTH 6.3 m
ELEV. 54.2 m
W = 70.5 %
Cc = 2.25
VOID RATIO e
PRESSURE k Pa
2.0
1.8
1.6
1.4
1.2
1.0
0.8
1
10
100
1 000
10 000

FIGURE 3.6



NOTE: THIS FIGURE SHOULD BE READ IN CONJUNCTION WITH THE APPENDED REPORT.



SECTION PARAMETERS

SUMMARY OF STABILITY ANALYSES

240 89/05

FIGURE 4

APPENDIX 2

OVERSIZE DRAWING

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Ministry of Transportation

Ontario

FOUNDATION DESIGN SECTION

foundation investigation and design report

ENGINEERING MATERIALS OFFICE
FOUNDATION DESIGN SECTION

CONT 93-62

WP 11-81-02 DIST #9

HWY 17 STR SITE

TENTH LINE ROAD EXTENSION STRUCTURE

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PRELIMINARY

FOUNDATION INVESTIGATION REPORT

FOR

W.P. 11-81-02

Hwy. 17 and Tenth Line Road Extension

District 9, Ottawa

INTRODUCTION

This report summarizes the information obtained from a preliminary foundation investigation carried out at the above-mentioned site, and provides preliminary recommendations pertaining to the structure foundations, and the stability and settlement at the approaches.

The field work was carried out between 86 01 15 and 86 01 22 and consisted of 2 sampled boreholes (BH 1, BH 2) advanced by a continuous flight auger machine equipped with hollow stem augers, BX casing, and BX core barrels. A dynamic cone penetration test accompanied each of the boreholes. The boreholes were advanced to depths of 32.4 m and 30.7 m.

SITE DESCRIPTION

The site, as shown in DWG 1, is located at the proposed crossing of Tenth Line Road extension at Hwy. 17, in the Township of Cumberland, Regional Municipality of Ottawa - Carleton, 1.4 km east Champlain Road.

Land use in the area is a combination of residential and open fields. To the north of the site, the land is flat and lightly wooded. To the south, approximately 15 m from Hwy. 17 is a 3 m deep gulley, and 250 m south is a 13 m high ridge with occasional rock outcrops.

Physiographically, the site is situated in the region called the "Ottawa Valley Clay Plains". This area consists of clay plains interrupted by ridges of rock or sand. (Reference Chapman & Putnam).

SUBSURFACE CONDITIONS

General

The record of Borehole Sheets in the Appendix illustrate the conditions at the borehole locations.

A surficial layer of 2-3 m thick consisting of very stiff desiccated clay of high plasticity is encountered. Beneath this layer is clay of high plasticity ranging in consistency from firm to stiff. The thickness of this stratum ranges from 22 m to 29 m, extending down to elevations between 24 m and 32 m. In BH #2 a 3.4 m layer of silty sand, some gravel, trace clay was found to underlie the clay, which in turn, is underlain by 1.5 m of boulders, and limestone bedrock.

Clay of High Plasticity

A very stiff layer of desiccated clay of high plasticity 2 m to 3 m thick underlies the relatively thin layer of topsoil. Immediately underlying the surficial material is the predominant stratum across the site; a grey clay of high plasticity. The thickness of this stratum ranges from 22 m to 29 m, extending down to elevations between 24 m and 32 m.

Physical properties of the material as determined from field and laboratory tests are summarized as follows:

		Range	Average
Unit Weight	(γ)	15.6-17.3 kN/m^3	16.3 kN/m^3
Natural Moisture Content	(W)	41.5 - 81.5%	61.7%
Liquid Limit	(W_L)	52.0 - 75.0%	64.3%
Plastic Limit	(W_P)	21.5 - 29.0%	26.1%
Shear Strength (Field)	(Cu)	26.9 - 107.0 kPa	
Sensitivity (Field)		generally 4 to less than 10	
Unconfined Strength (Lab)	(Cu)	13.7 - 73.8 kPa	

The Atterberg Limit tests are plotted on the Plasticity Chart shown in Figure 1. These results indicate that the clay is inorganic and of high plasticity. Figure 2 shows that in general the natural moisture content is above the liquid limit in the upper zones of the deposits, decreasing to values slightly below the liquid limit with depth. The consistency as determined from unconfined, undrained shear tests and field vane tests, increase with depth from firm to stiff. The relationship between shear strength and depth for both the insitu and remoulded conditions is illustrated in Figure 3.

Primary consolidation testing was carried out on 3 of samples of this material. Results of this testing is summarized as follows:

BH	Sample #	Depth (m)	Pc (kPa)	Cc	e_o
1	3	10	108	0.99	1.804
1	7	30	455	1.66	1.858
2	6	23	415	1.78	1.900

Two typical consolidation curves for material of this stratum are shown in Figures 4 and 5.

<u>Silty Sand, Some Gravel, Trace Clay</u> (Till)

This very dense, non-cohesive material was encountered beneath the clay in BH #2 only. The thickness of this layer is 3 m extending from elevation 35.3 m to 32.3 m. A grain size distribution curve for this material is shown in Figure 6.

<u>Bedrock</u>

Bedrock was established in BH #2 by obtaining BX rock core samples. The first 1.5 m of coring, from elevations 30.8 m to 32.3 m, is assumed to be a boulder zone because of the poor recovery. Bedrock is confirmed at elevations 32.3 m and consists of grey, unweathered limestone. A more detailed description of the bedrock is included in Table A, Description of Rock Core, in the Appendix.

Groundwater Conditions

The groundwater table established from readings in open boreholes 5 days after borehole completion (BH 1), and inside hollow-stem augers on the same day as augering (BH 2), indicate the water table varies from elevations 53.7 m to 54.2 m which is 4.5 m to 5.3 m below the existing ground surface.

DISCUSSION AND RECOMMENDATIONS

General

It is proposed to construct a structure to carry the Tenth Line Road extension over Hwy. 17. This structure is to accommodate the proposed widening of Hwy. 17 from an existing 2-lane highway to a 4-lane divided highway as well as two on-off ramps. The height of the approaches could be as high as 10 m above the existing ground surface.

Results of the preliminary foundation investigation indicate that underlying between 2 m and 3 m of stiff desiccated clay of high plasticity is the predominant stratum across the site; a sensitive clay of high plasticity ranging in consistency from firm to stiff with depth. The thickness of this stratum ranges from 22 m to 29 m. The clay is underlain by up to 3 m of till composed of silty sand, some gravel, trace clay, followed by 1.5 m of boulders, and in turn followed by limestone bedrock.

The presence of an extensive deposit of soft and highly compressible, sensitive clay poses a problem for the stability of the embankments, as well as the settlement induced by the approach fills. The following are our preliminary recommendations for the design and construction of the structure and the associated approach fills.

APPROACHES

Stability

The critical condition for stability of an embankment on normally or slightly over consolidated clay, as is the case with this clay deposit, generally occurs during or immediately after construction. This being the case, a total stress analysis ($\phi = 0$) provides a suitable means of assessing the stability of the embankment. The total stress analysis takes into consideration the undrained shear strength properties of the foundation and embankment soils.

In this preliminary analysis of the proposed embankment, the following assumptions were made:

1) Fill Material : non-cohesive

Bulk Density = 20.6 kN/m^3
Angle of Shearing Resistance = 28^o

2) Foundation Subsoils : sensitive clay

Bulk Density = 15.7 kN/m^3
Submerged Density = 5.9 kN/m^3
Undrained shear strength, Cu, varies with depth as follows:

Depth Below Ground Surface	Cu (kPa)
0 - 2 m	40
2 - 7.5 m	35
7.5 - 12.5 m	47
12.5 - 17.5 m	60
17.5 - 22.5 m	75
22.5 - 27.5 m	90
27.5 -	100

3) A minimum factor of safety of 1.3 is considered acceptable

4) Berms are constructed as an integral part of the embankment and not constructed separately

5) Berms occur at mid-height of the fill

6) All slopes are to be constructed at 2H:1V

7) All surficial organic material is removed prior to placing fill.

A typical section analyzed is shown on Figure A (Pg. 8). A 10 m high fill, with 2H:1V side and front slopes would experience a deep seated failure as shown by the geometry of the failure circle and the associated factor of safety of 0.92. Figure B (Pg. 8) shows that for this height of fill, a 16 m counterbalancing mid-height berm would be required in order to stabilize the embankment.

The results of the stability analysis have been summarized in terms of fill height (H) vs. required length of mid-height berm (L). This relationship is shown on Figure C (Pg. 9). As indicated by this curve no berm is required for fills less than 6.5 m in height. For fill heights between 6.5 and 10 m, the associated mid-height berm length can be obtained from the curve. This Section should be consulted if fills greater than 10 m are required.

It is to be noted that when berms are required they should be provided in both the longitudinal and transverse directions.

The stability of the approach embankment was also analyzed assuming that the fill would be constructed of "light-weight" material. For the purpose of the analysis, it was assumed that "Open Graded Pit Run" air-cooled slag is used. The maximum compacted unit weight of this material is limited to 14 kN/m^3. The results of the total stress stability analysis indicate that no berms are required for "light-weight" fills of 9-10 m in height or lower. Fills of a greater height would require stabilizing berms. If this option is considered, this Section should be contacted for further details.

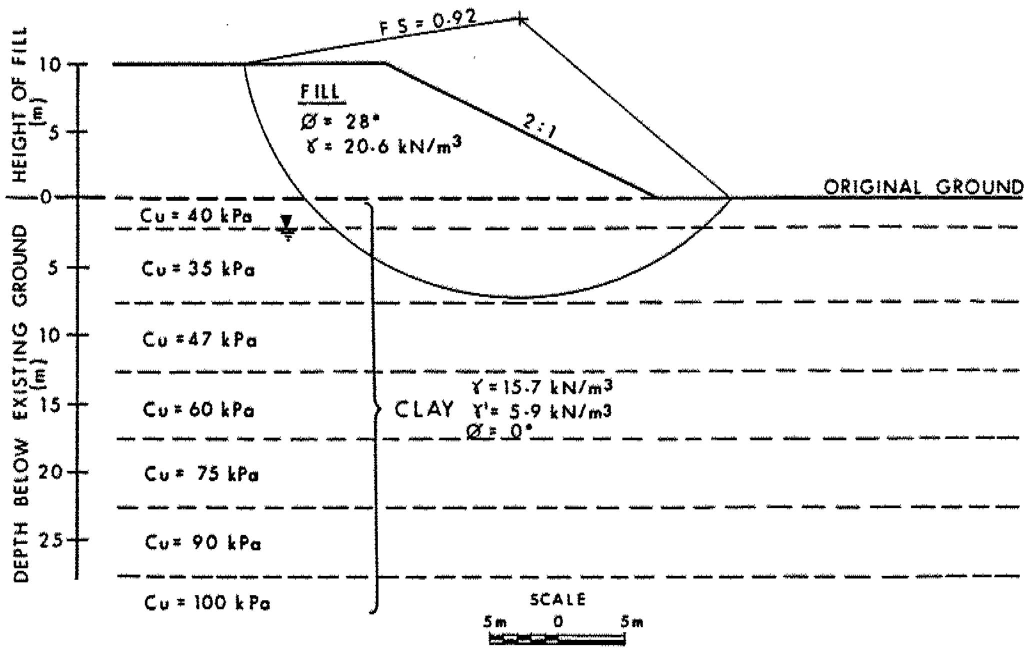


Fig A

WP 11-81-02

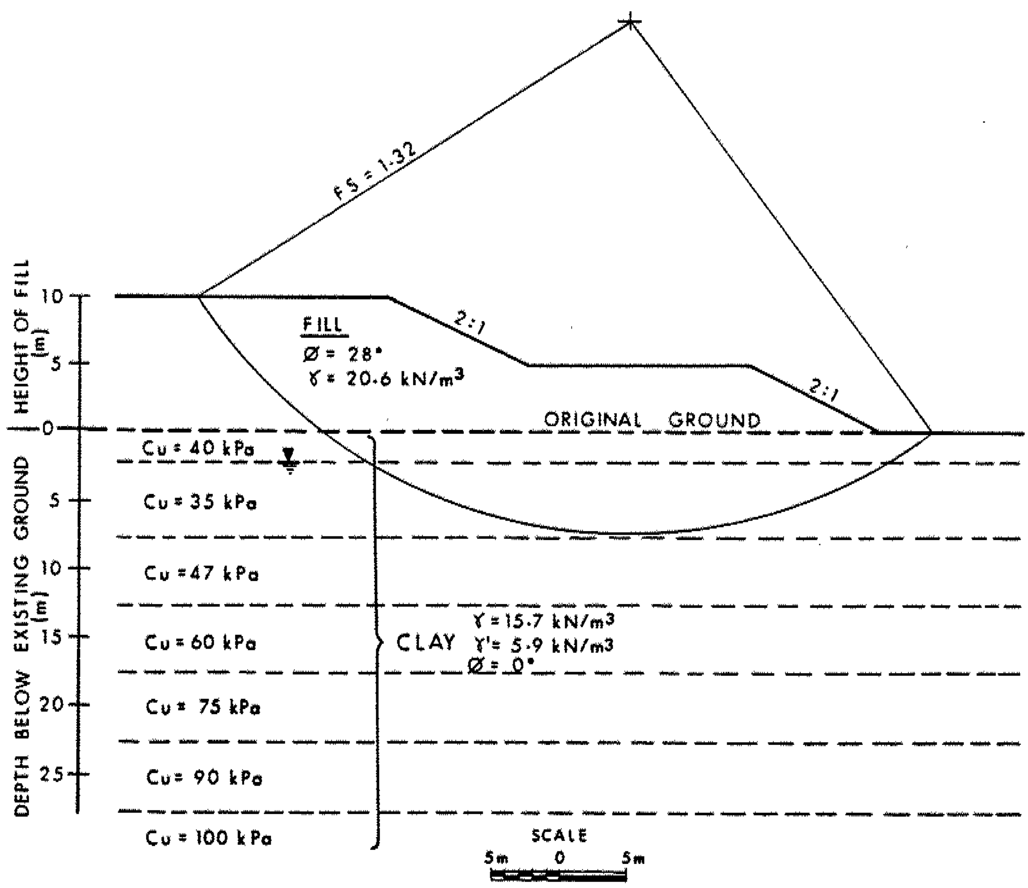


Fig B

WP 11-81-02

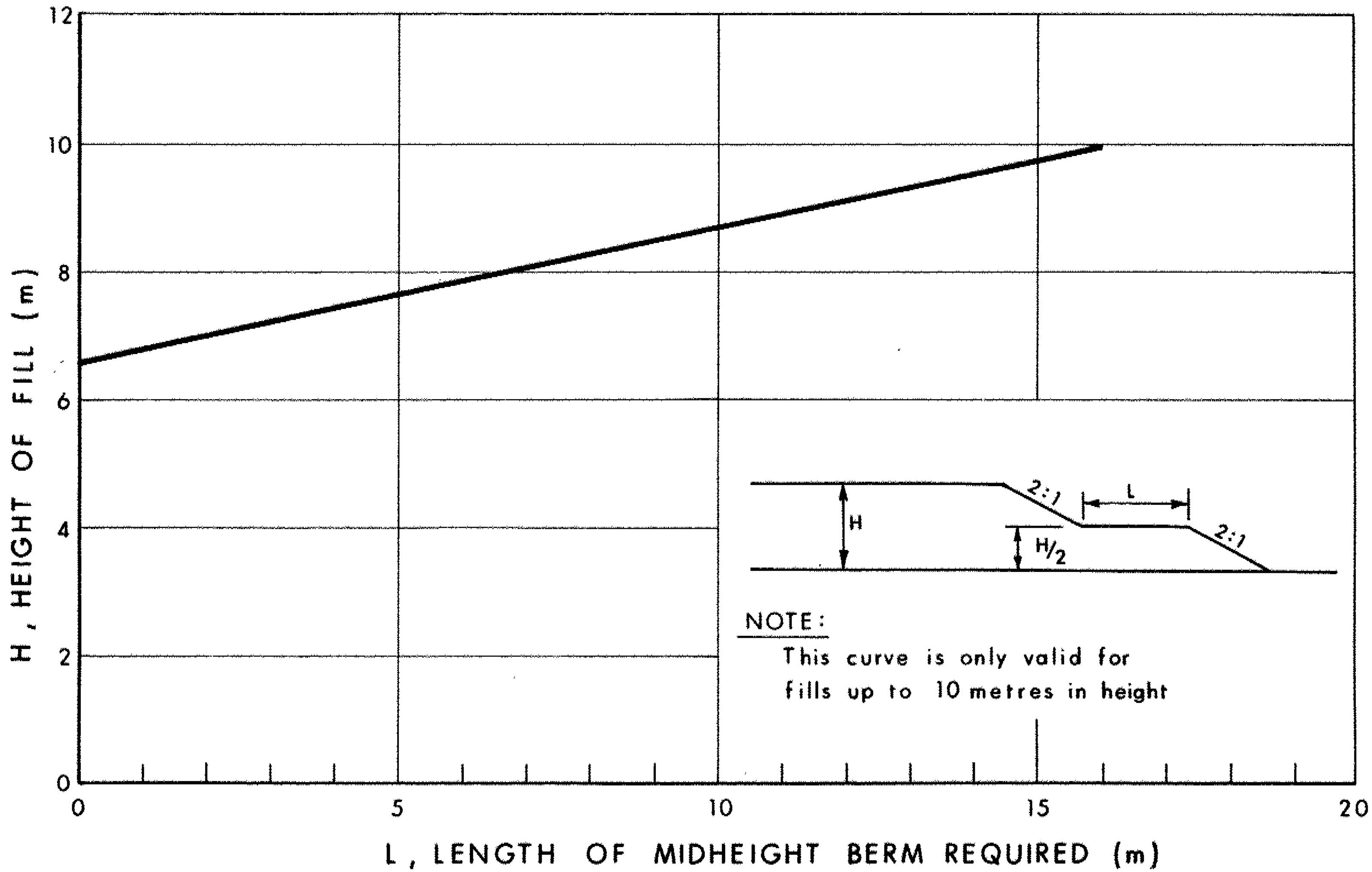


Fig C

WP 11-82-02

SETTLEMENT

The underlying compressible clay stratum will experience appreciable settlements due to consolidation under the additional stresses imposed by the proposed embankment. Settlement calculations were carried out and the results are summarized in Figure D, on the following page. In the settlement analysis, it was assumed that the weight of the fill material used would not exceed 19.6 kN/m^3, and that berms would be incorporated as per Figure C for fill heights greater than 6.5 m.

As indicated on Figure D a fill height of 10 m will experience an approximate total settlement of 60 cm at the centre of the embankment. It is anticipated that it will require 5-6 years for this total settlement to occur. Within the first year, however, it is expected the initial 40-50% of the settlement will be experienced.

In order to accelerate post-construction settlements, it is recommended that the embankments be pre-loaded with an additional 2 m surcharge for a minimum period of 12 months. It should be noted that if the surcharge is incorporated, mid-height berms of up to 23-24 m will be required during the pre-load period. This option obviously has considerable implications on property requirements.

Another option which could be considered involves the use of light-weight slag fill, previously mentioned under the "Stability" heading. It is estimated that a 10 m high embankment constructed of slag material having a maximum unit weight of 14 kN/m^3, would experience about 35 cm total settlement at the centre of the embankment. If this option is selected this Section should be contacted regarding pre-loading requirements. This alternative would reduce property requirements since fills of 9 to 10 m in height constructed of this material would not require extensive stabilizing berms.

In order to reduce or accelerate the anticipated settlements other options involving expanded polystyrene styrofoam fill or drain wicks could be considered. Additional details could be provided by this Section if these alternatives are given consideration.

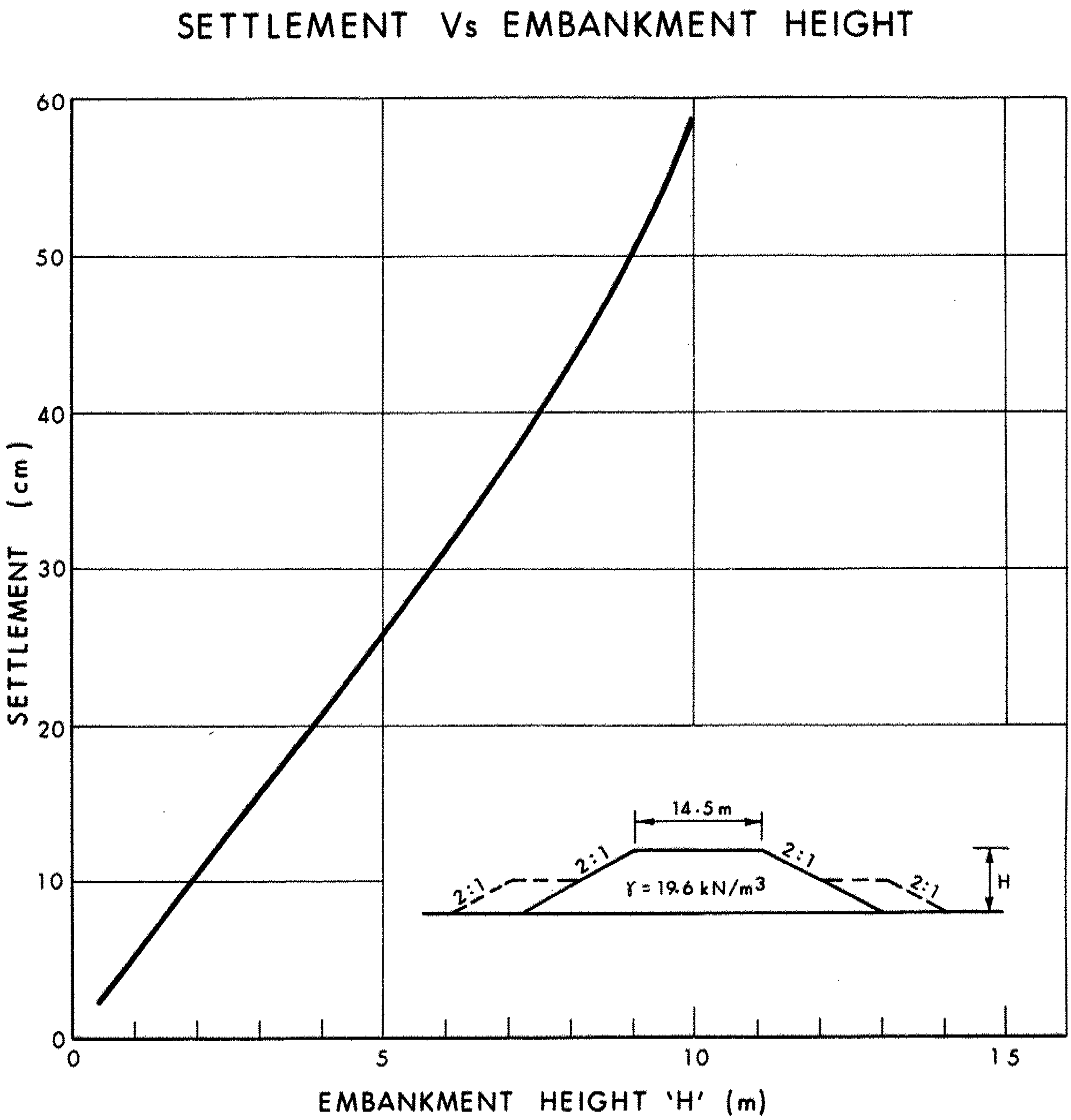


Fig. D

WP 11-81-02

STRUCTURE FOUNDATIONS

The proposed structure may be supported on steel H-piles, equipped with reinforced tips (to facilitate pile driving) and driven to bedrock. For estimating purposes the following bedrock elevations may be used:

32.4 m at south abutment	(refusal to augering)
28.7 m at north abutment	(cored)

The following design values are recommmended for the piles for the piers, assuming that no fill material is required at these two locations. If at the pier locations fill is required, these loadings may have to be reduced because of the effects of negative skin friction resulting from the consolidation of the underlying clay deposit.

Pile Type	Factored Capacity at ULS	Capacity at SLS Type II
310 HP 110	1600 kN per pile	1150 kN per pile
310 HP 79	1150 kN per pile	850 kN per pile

Negative skin friction will be imposed on the piles supporting the abutments due to settlement of the approach embankment. Futhermore, these forces, combined with lateral movement of the subsoil due to the strain imposed by the embankment loading, will tend to displace the pile laterally. In order to minimize rotation of the abutments, the wingwalls should also be supported on steel H-piles driven to bedrock. In addition, piles should be battered in both directions.

The following design values are recommended for the abutments and wing walls:

Pile Type	Factored Capacity at ULS	Capacity at SLS Type II
310 HP 110	1280 kN per pile	920 kN per pile
310 HP 79	920 kN per pile	660 kN per pile

If desired, the abutment footings (supported on steel H-piles) may be perched within the embankment fill. To facilitate pile driving, particle size in the fill immediately beneath the pile locations should not exceed 75 mm.

EARTH PRESSURE CALCULATIONS

Backfill to structures should consist of granular material in accordance with MTC Standard Special Provision #121 (83 10). Computation of earth pressures should be carried out in accordance with Section 6.6.1.2 of O.H.B.D.C.

For design purposes, the physical properties of the backfill are as follows:

Material	ϕ	γ
Granular 'A'	35°	22.0 kN/m^3
Granular 'B'	30°	21.2 kN/m^3

Geotechnically, the foundation is considered to be non-yielding as the piles will be driven to bedrock and the at-rest (Ko) applies for lateral earth pressures. Structurally, the active (Ka) condition may control the earth pressure design due to the length of the piles.

GENERAL RECOMMENDATIONS

- All topsoil and surficial organic material within the plan limits of the embankment should be removed.
- All fill material placed in the area where the piles will penetrate should be restricted to a maximum particle size of 75 mm.
- No dewatering problems are anticipated for footing excavations.
- A frost cover of 1.8 m should be provided to all underside of footings.

MISCELLANEOUS

The recommendations outlined in this report are based on a limited amount of field work. It will be necessary to carry out a detailed subsurface investigation when the final conceptual design is available. Recommendations given in this report are subject to revision at a later date.

The fieldwork for this project was carried out under the supervision of Mr. L. Politano, Project Foundations Engineer, and Ms. D. Yeo, Project (Trainee) Foundations Engineer. The description of the bedrock core was provided by Mr. E. Magni, Geologist.

This report was prepared by Ms. D. Yeo, Project (Trainee) Foundations Engineer and L. Politano, Project Foundations Engineer and was reviewed by Mr. M. Devata, Chief Foundations Engineer (East).

The drilling equipment used was owned and operated by F. E. Johnston Drilling Inc., of Ottawa.

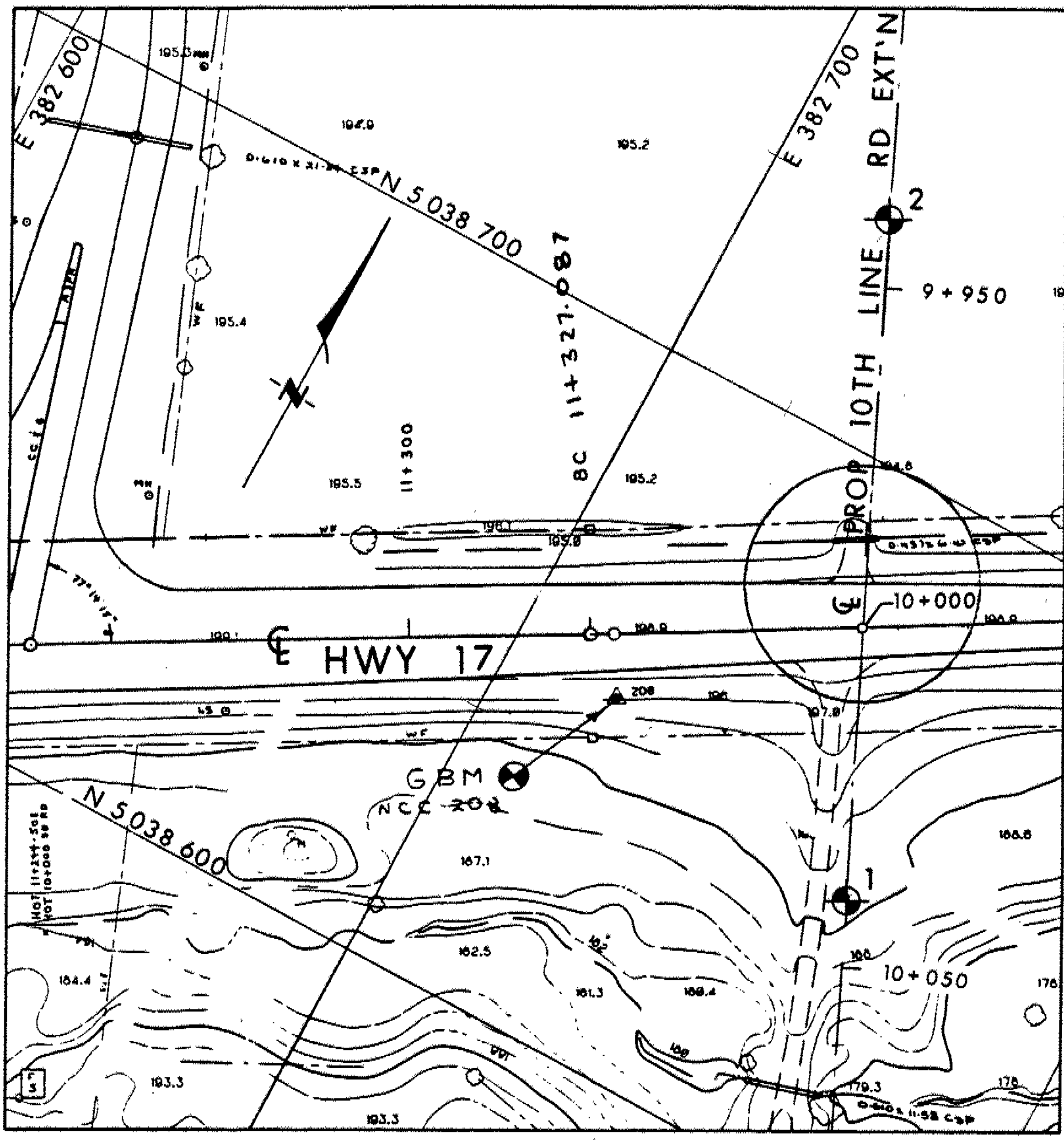
D. Yeo
Project (Trainee) Foundations Engineer

REGISTERED PROFESSIONAL ENGINEER
CIVIL
M. DEVATA
PROVINCE OF ONTARIO

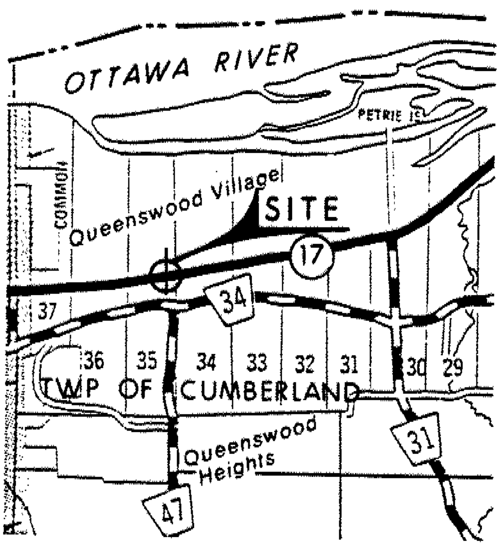
M. Devata, P. Eng.,
Chief Foundations Engineer
(East)

March 1986

APPENDIX



PLAN SCALE 1:1000



KEY PLAN

NOTE:

DISREGARD CONTOURS LINES AND SPOT ELEVATIONS.

BH 1 STA 10+040 ℄

BH 2 STA 9+940 ℄

Geocres No 31G5-143

Dist 9

Hwy 17

WP 11-81-02

Dwg No 1

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/0.3 m)	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 (R Q D), FOR MODIFIED RECOVERY, IS:

R Q D (%)	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

S S	SPLIT SPOON	T P	THINWALL PISTON
W S	WASH SAMPLE	O S	OSTERBERG SAMPLE
S T	SLOTTED TUBE SAMPLE	R C	ROCK CORE
B S	BLOCK SAMPLE	P H	T W ADVANCED HYDRAULICALLY
C S	CHUNK SAMPLE	P M	T W ADVANCED MANUALLY
T W	THINWALL OPEN	F S	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_t	1	SENSITIVITY $= \frac{c_u}{\tau_r}$

PHYSICAL PROPERTIES OF SOIL

ρ_s	kg/m^3	DENSITY OF SOLID PARTICLES
γ_s	kN/m^3	UNIT WEIGHT OF SOLID PARTICLES
ρ_w	kg/m^3	DENSITY OF WATER
γ_w	kN/m^3	UNIT WEIGHT OF WATER
ρ	kg/m^3	DENSITY OF SOIL
γ	kN/m^3	UNIT WEIGHT OF SOIL
ρ_d	kg/m^3	DENSITY OF DRY SOIL
γ_d	kN/m^3	UNIT WEIGHT OF DRY SOIL
ρ_{sat}	kg/m^3	DENSITY OF SATURATED SOIL
γ_{sat}	kN/m^3	UNIT WEIGHT OF SATURATED SOIL
ρ'	kg/m^3	DENSITY OF SUBMERGED SOIL
γ'	kN/m^3	UNIT WEIGHT OF SUBMERGED SOIL

e	1,%	VOID RATIO
n	1,%	POROSITY
w	1,%	WATER CONTENT
S_r	%	DEGREE OF SATURATION
w_L	%	LIQUID LIMIT
w_P	%	PLASTIC LIMIT
w_S	%	SHRINKAGE LIMIT
I_P	%	PLASTICITY INDEX $= w_L - w_P$
I_L	1	LIQUIDITY INDEX $= \frac{w - w_P}{I_P}$
I_C	1	CONSISTENCY INDEX $= \frac{w_L - w}{I_P}$
e_{max}	1,%	VOID RATIO IN LOOSEST STATE

e_{min}	1,%	VOID RATIO IN DENSEST STATE
I_D	1	DENSITY INDEX $= \frac{e_{max} - e}{e_{max} - e_{min}}$
D	mm	GRAIN DIAMETER
D_n	mm	n PERCENT - DIAMETER
C_U	1	UNIFORMITY COEFFICIENT
h	m	HYDRAULIC HEAD OR POTENTIAL
q	m^3/s	RATE OF DISCHARGE
v	m/s	DISCHARGE VELOCITY
i	1	HYDRAULIC GRADIENT
k	m/s	HYDRAULIC CONDUCTIVITY
j	kN/m^3	SEEPAGE FORCE

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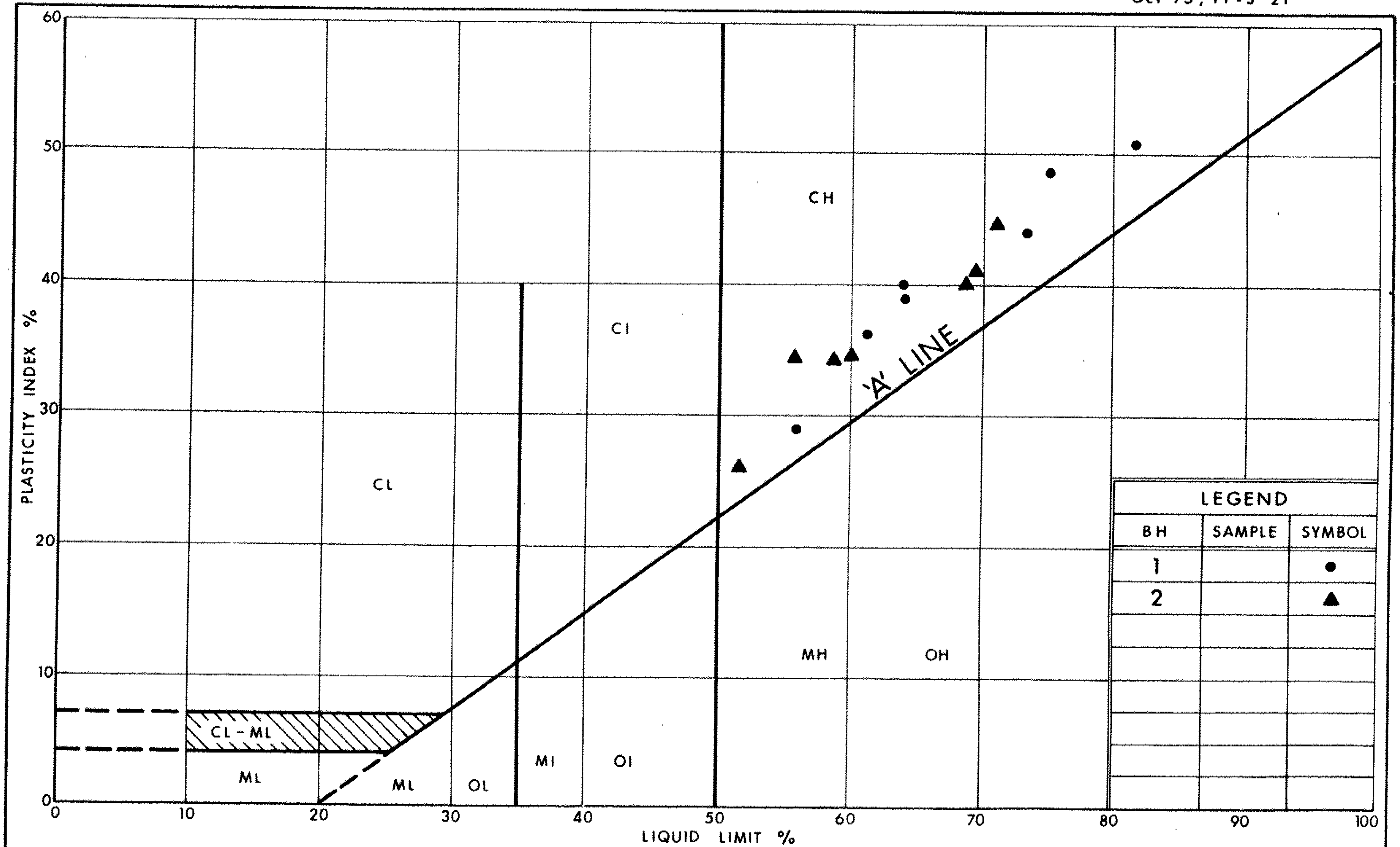

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PLASTICITY CHART
CLAY

FIG No 1
WP 11-81-02

ATTERBERG LIMITS Vs DEPTH

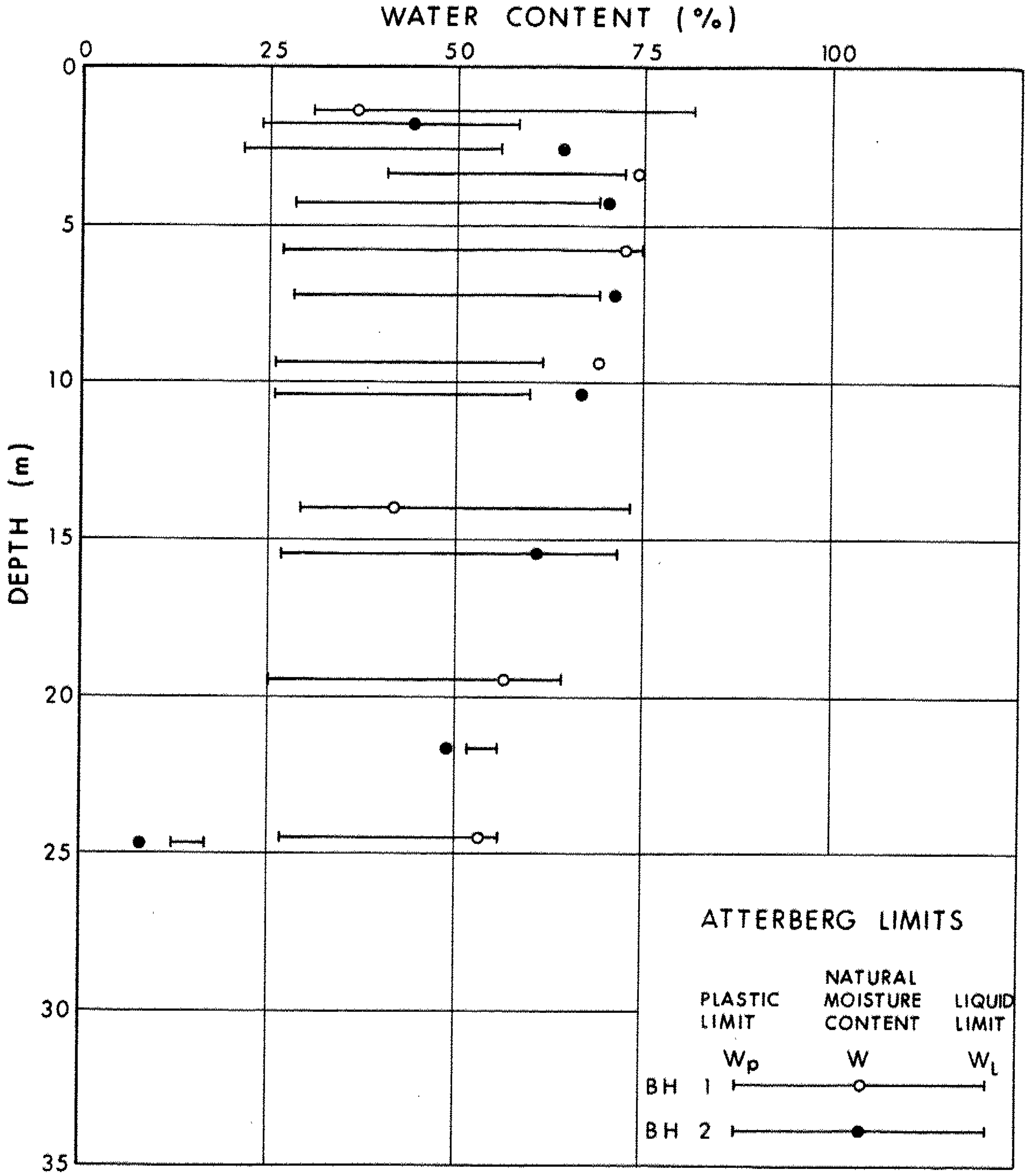

Fig No 2

WP 11-81-02

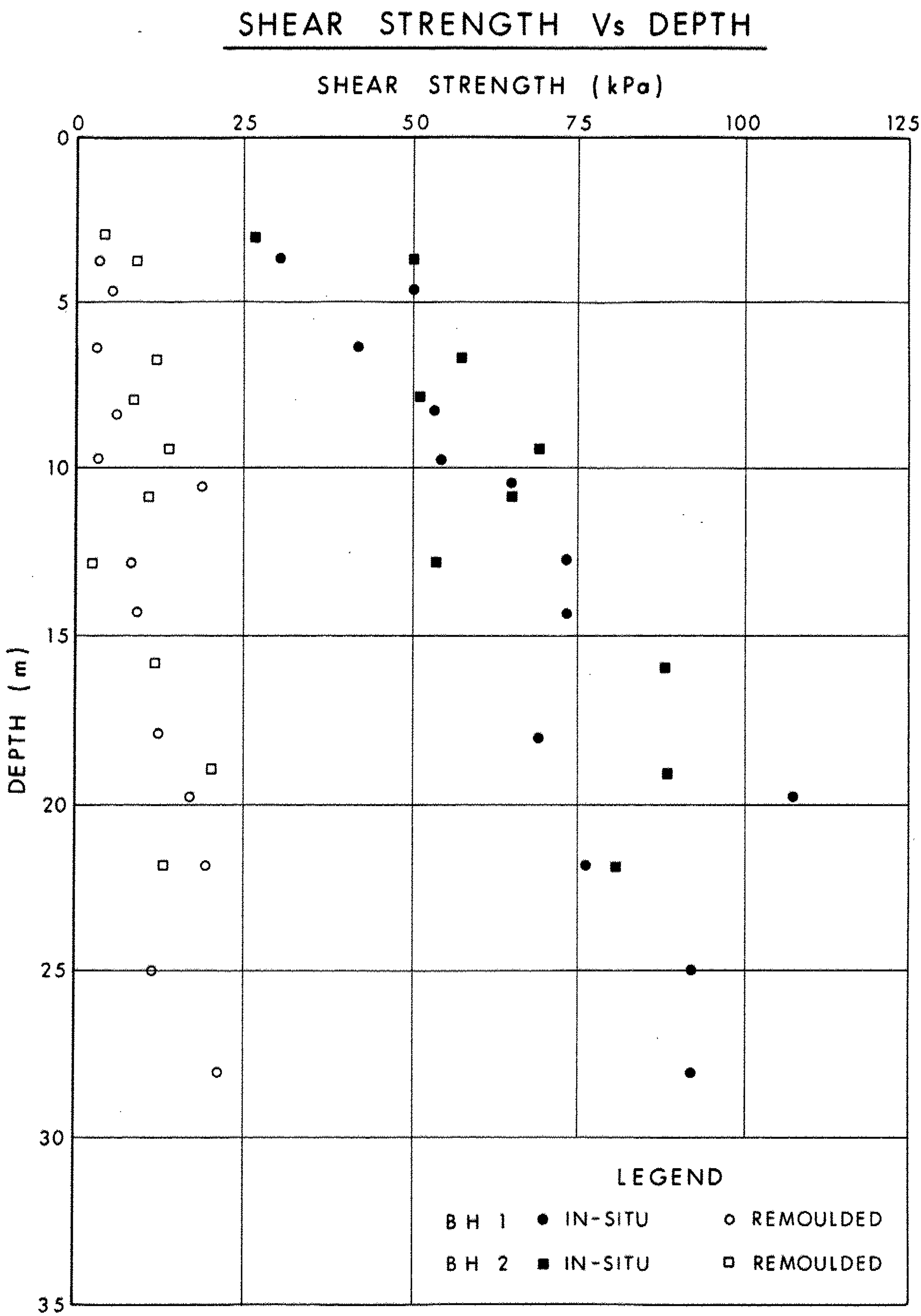

Fig No 3

WP 11-81-02

VOID RATIO - PRESSURE CURVE

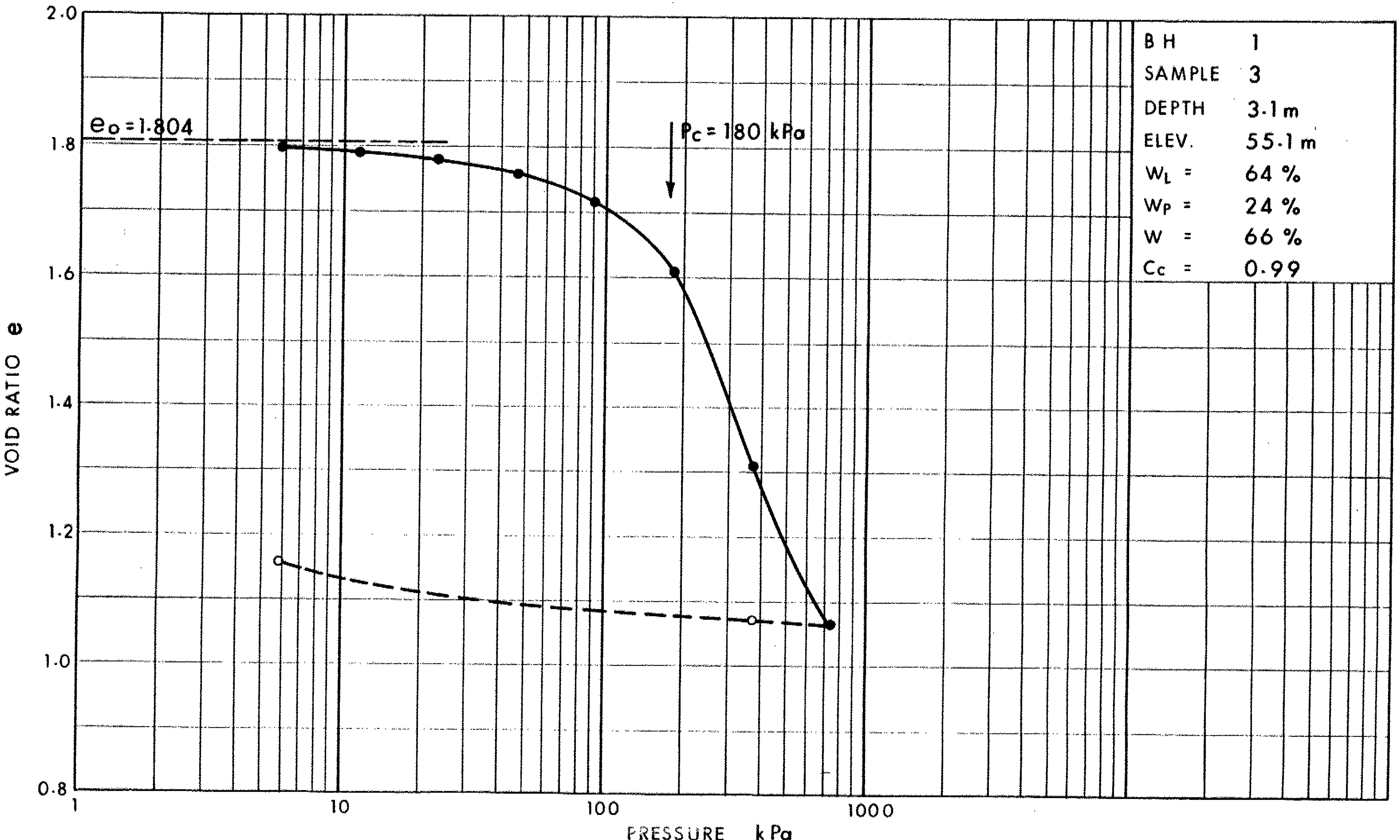

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FIG No 4

VOID RATIO - PRESSURE CURVE

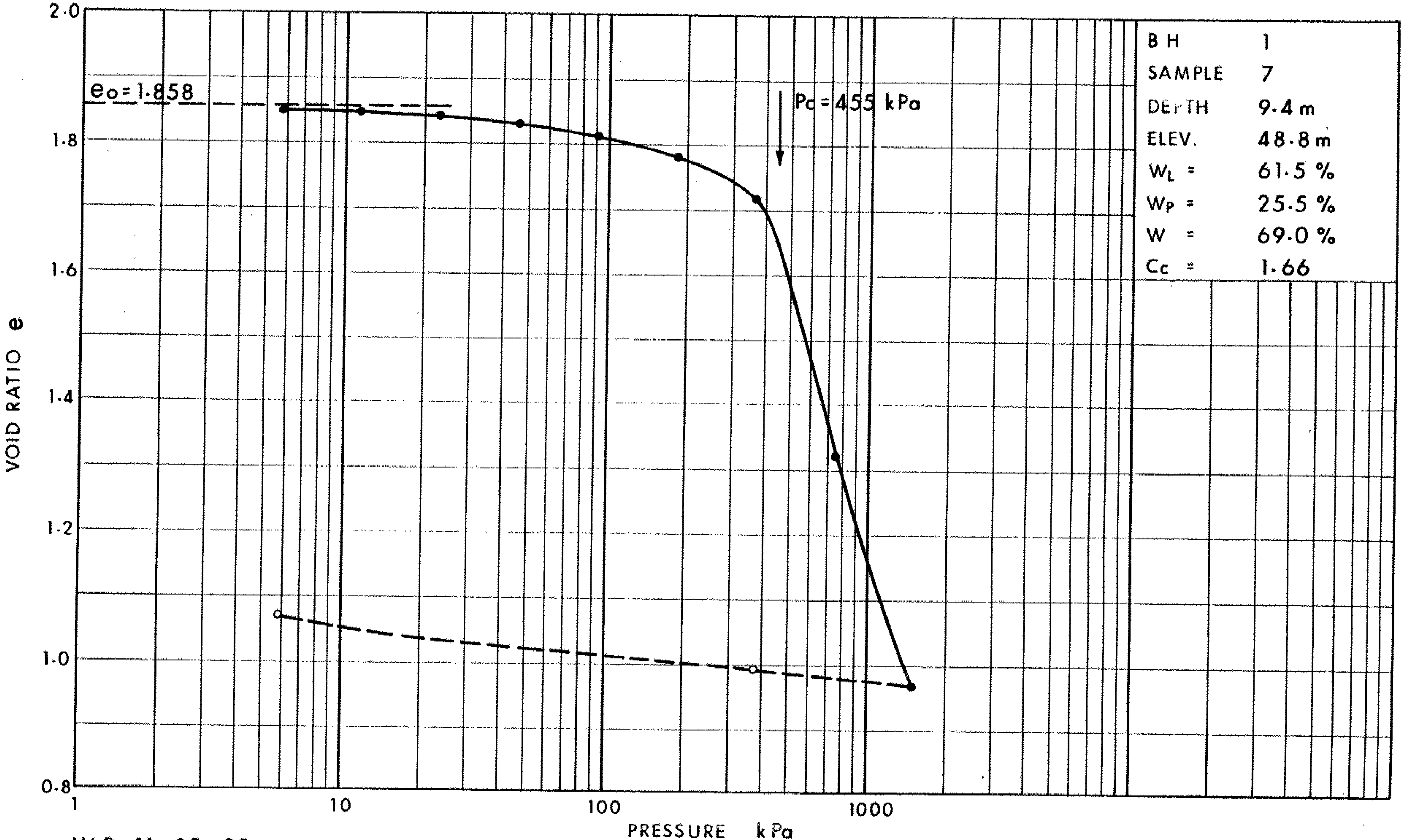

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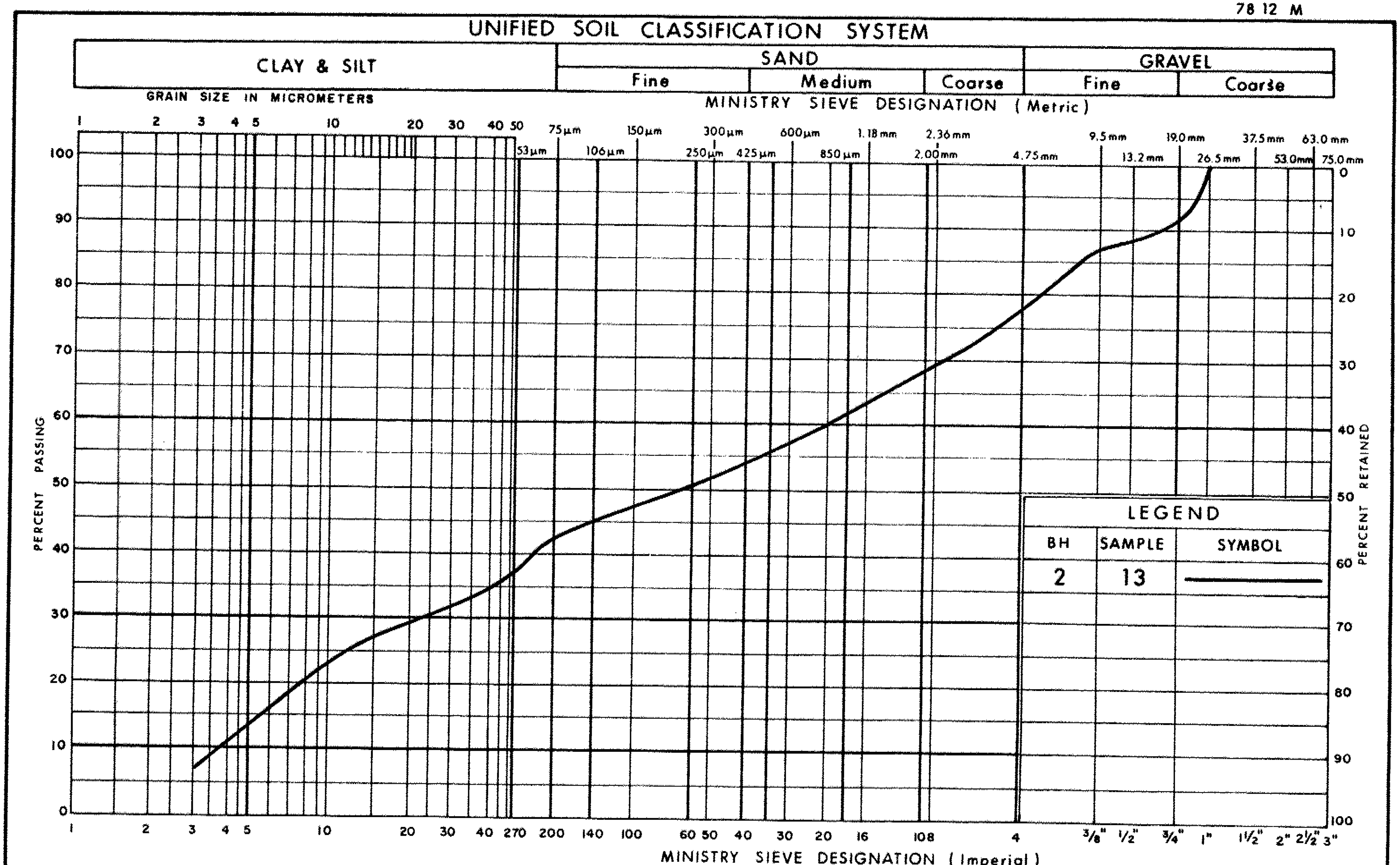
FIG No 5



Ministry of Transportation and Communications Ontario	GRAIN SIZE DISTRIBUTION SILTY SAND, SOME GRAVEL	FIG No 6 WP 11-81-02

DESCRIPTION OF ROCK CORE - W.P. 11-81-02

BOREHOLE NUMBER	DEPTH (m)	% CR*	% RQD*	CORE DESCRIPTION DEPTH (m)	CORE DESCRIPTION DESCRIPTION
2	27.22-28.65	78	36	27.22-28.65	Boulders assumed, high core loss zone
	28.65-29.38	66	17	28.65-30.69	LIMESTONE (100%), grey, unweathered, medium spaced joints (core loss from 28.65-29.38 due to drilling)
	29.38-30.69	94	88		

* CR = CORE RECOVERY ; RQD = ROCK QUALITY DESIGNATION

TABLE 'A'

Ministry of Transportation and Communications
Ontario

RECORD OF BOREHOLE No 1

METRIC

W P 11-81-02 LOCATION STA. 10 + 040 ℄ 10TH Line Road Extension ORIGINATED BY D.Y.

DIST 9 HWY 17 BOREHOLE TYPE Hollow Stem, Cone Test COMPILED BY D.Y.

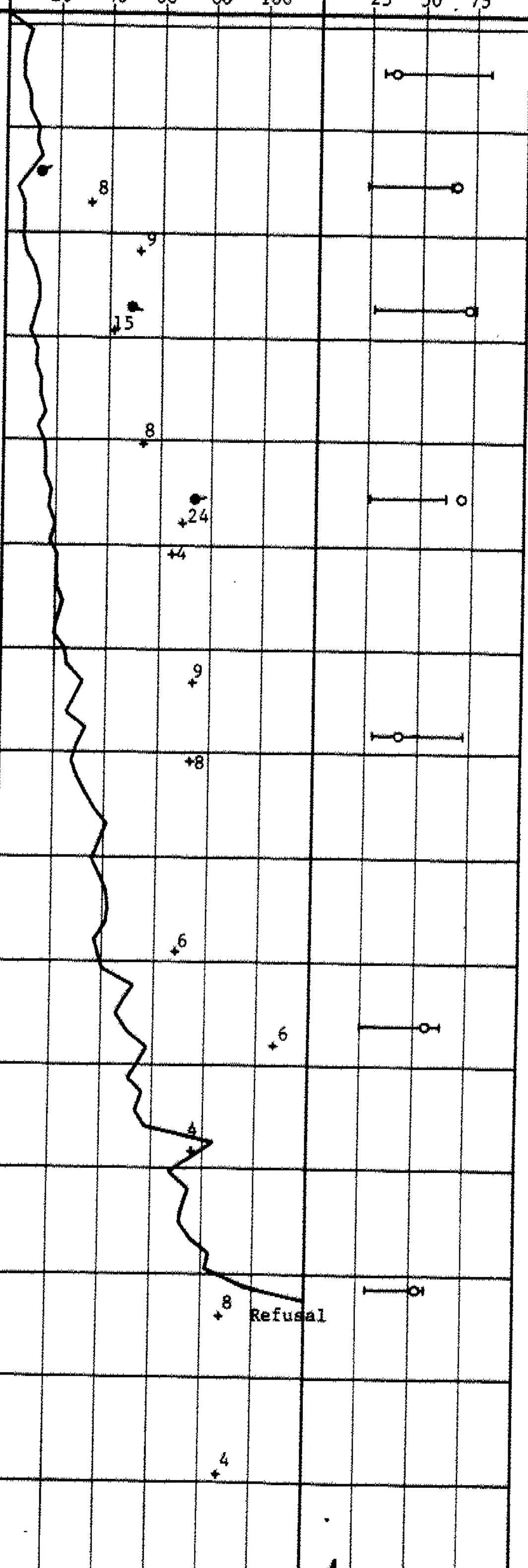
DATUM Geodetic DATE 86 01 15 to 86 01 17 CHECKED BY [signature]

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	UNIT WEIGHT γ kN/m³
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES				
58.2	Ground Surface								
0.0	Clay (Desiccated)						58		
	Very Stiff		1	SS	10				
			2	SS	8		56		
	(Brown)								
	Clay of High Plasticity		3	TW	PH				16.3
			4	SS	2	▼*	54		
	Firm		5	TW	PM		52		16.8
			6	SS	5		50		
			7	TW	PM		48		15.9
			8	SS	7		46		
	Stiff		9	TW	PH		44		16.3
			10	SS	6		42		
							40		
			11	TW	PH		38		16.3
			12	SS	6		36		
			13	TW	PH		34		17.1
							32		
			14	SS	12		30		
28.0 / 30.2	(Grey)								

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED; + FIELD VANE; ● QUICK TRIAXIAL; × LAB VANE: 20 40 60 80 100

PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%): 25 50 75



Continued

+3, ×5 : Numbers refer to Sensitivity

15 ⌖ 5 (%) STRAIN AT FAILURE (20 above, 10 below)

Continued

OFFICE REPORT ON SOIL EXPLORATION

RECORD OF BOREHOLE No 1 Continued METRIC

W P 11-81-02 LOCATION STA. 10 + 040 ℄ 10TH Line Road Extension ORIGINATED BY D.Y.

DIST 9 HWY 17 BOREHOLE TYPE Hollow Stem, Cone Test COMPILED BY D.Y.

DATUM Geodetic DATE 86 01 15 to 86 01 17 CHECKED BY [signature]

SOIL PROFILE ELEV DEPTH	DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L WATER CONTENT (%)	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
28.0 / 30.2	Continued										
	Clay of High Plasticity		15	TW	PH						
25.7	Stiff		16	SS	**		26				
32.5	End of Borehole										

Refusal to Augering
probable Bedrock

* NOTE: Watertable elevations measured in open borehole

DATE	ELEVATION
86 01 18	50.2
01 20	51.2
01 21	52.5
01 22	53.7

** NOTE: Spoon Bouncing

OFFICE REPORT ON SOIL EXPLORATION

$+^3, \times^5$: Numbers refer to Sensitivity

20 / 15 ⌖ 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation and Communications
Ontario

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RECORD OF BOREHOLE No 2 — METRIC

W P 11-81-02 — LOCATION STA. 9 + 940 ℄ 10TH Line Road Extension — ORIGINATED BY D.Y.

DIST 9 HWY 17 — BOREHOLE TYPE Hollow Stem Auger, BX Rock Core, Cone Test — COMPILED BY D.Y.

DATUM Geodetic — DATE 86 01 17 to 86 01 22 — CHECKED BY [signature]

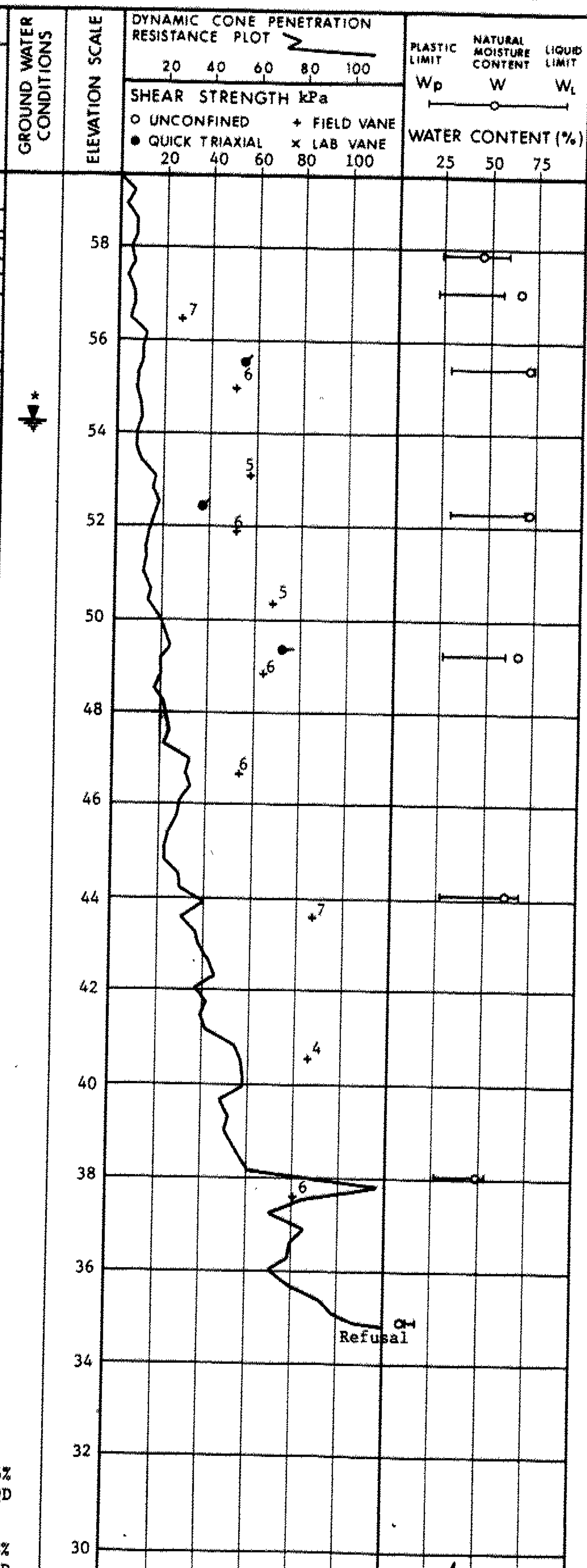
OFFICE REPORT ON SOIL EXPLORATION

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
59.5	Ground Surface
0.0	Clay (Desiccated) Very Stiff (Brown)
	Clay of High Plasticity — Firm
	Stiff
35.3 / 24.2	(Grey)
	Silty Sand, Some Gravel Very Dense
30.8 / 28.7	Boulders
29.3 / 30.2	Sound Bedrock Limestone (Grey)

SAMPLES

NUMBER	TYPE	'N' VALUES
1	SS	19
2	SS	9
3	SS	2
4	TW	PM
5	SS	1
6	TW	PH
7	SS	2
8	TW	PM
9	SS	3
10	TW	PH
11	SS	6
12	TW	PH
13	SS	65
14	SS	**
15	BX RC	78% REC 36% RQD
15A	BX RC	
16	BX RC	94% REC 88% RQD



UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
15.6	
15.7	
15.9	
16.2	
17.3	
	24 33 42 1

Continued

+3, ×5 : Numbers refer to Sensitivity

20 / 15 -◊- 5 / 10 (%) STRAIN AT FAILURE

Continued

Ministry of Transportation and Communications
Ontario

RECORD OF BOREHOLE No 2 Continued METRIC

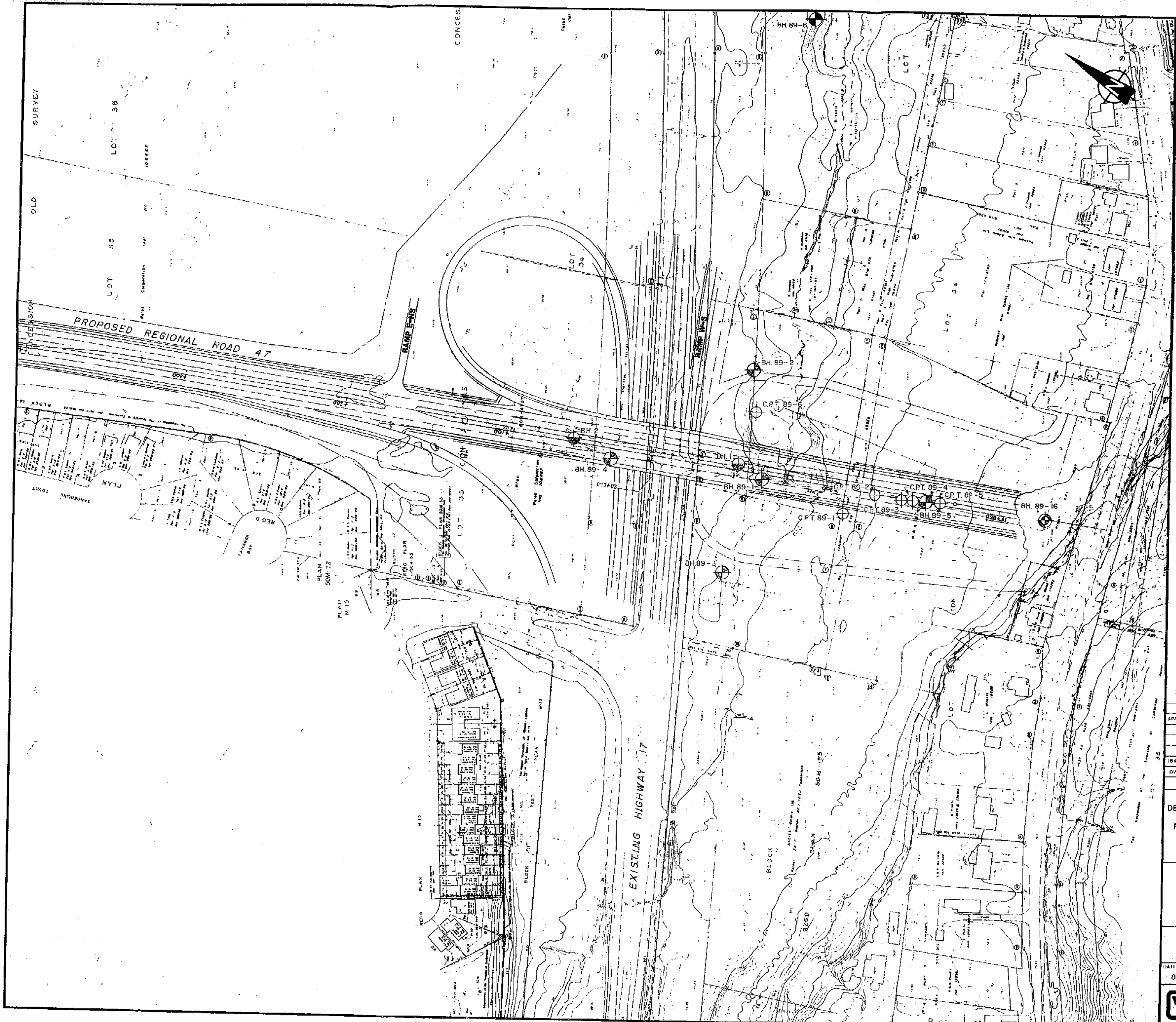
W P 11-81-02 LOCATION STA. 9 + 940 ℄ 10TH Line Road Extension ORIGINATED BY D.Y.
DIST 9 HWY 17 BOREHOLE TYPE Hollow Stem Auger, BX Rock Core, Cone Test COMPILED BY D.Y.
DATUM Geodetic DATE 86 01 17 to 86 01 22 CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	PLASTIC LIMIT W_p NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L; WATER CONTENT (%)	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES						
29.3	Continued										
28.8	Limestone Bedrock Sound										
30.7	End of Borehole										
	* NOTE: Watertable measured inside augers on same day as augering										
	** NOTE: Spoon Bouncing										

OFFICE REPORT ON SOIL EXPLORATION

$+^3$, $\times^5$: Numbers refer to Sensitivity

15 ⌖ 5 (%) STRAIN AT FAILURE (20 above, 10 below)



THIS DRAWING FORMS PART OF JACQUES, WHITFORD LTD. REPORT No. 0-19375 AND SHOULD BE READ IN CONJUNCTION WITH THE REPORT.

LEGEND

- BOREHOLE LOCATION (J.W.L. 0-19375)
- CONE PENETRATION TEST (J.W.L. 0-19375)
- BOREHOLE LOCATION (J.W.L. 0-19373)
- BOREHOLE LOCATION (M.T.O. 1986)

NOTE

LOCATIONS & ELEVATIONS SURVEYED BY DELCAN

APP'D	NO	DETAILS	DATE
REVISIONS			

DWG NO	DESCRIPTION	DATE
1841-01	DELCAN CORPORATION	89/10/26

REFERENCES

DELCAN CORPORATION
DESIGN PROJECT REGIONAL RD. 47
REGIONAL RD. 34 NORTHERLY TO
JEANNE D'ARC BOULEVARD
CUMBERLAND, ONTARIO

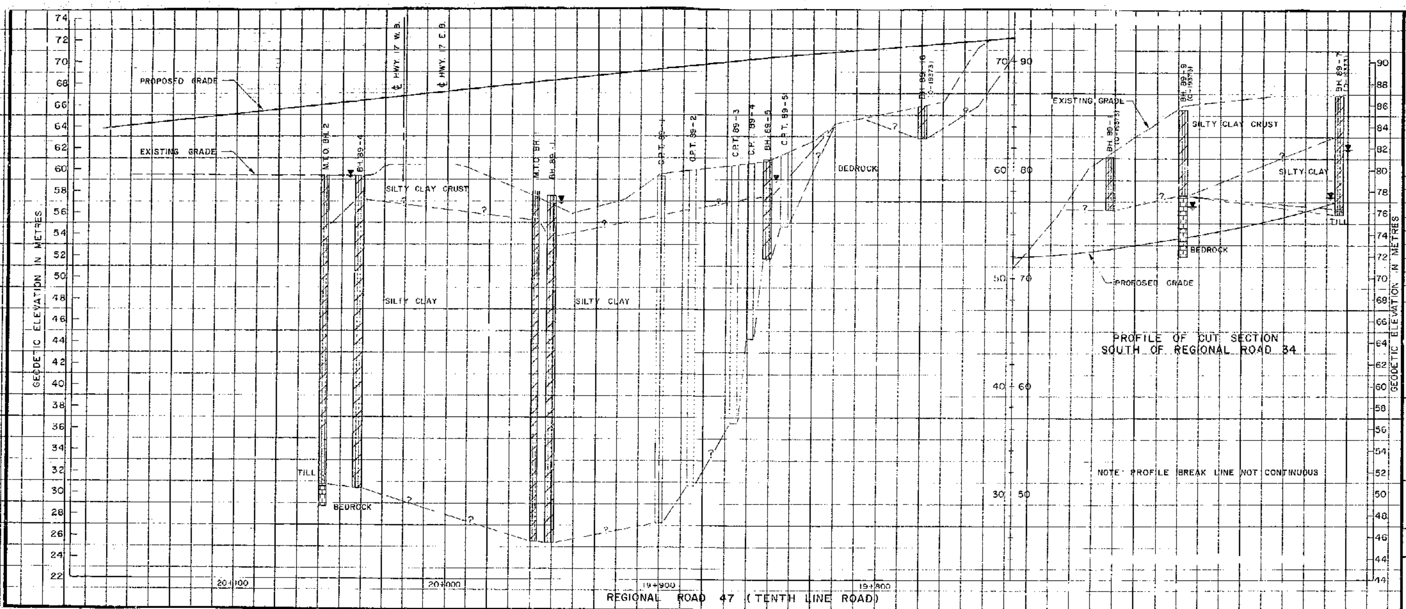
BOREHOLE LOCATIONS

Jacques, Whitford Limited
CONSULTING ENGINEERS

DATE	SCALE	DRAWN BY
89/11/30	1:1 000	NCP

APPROVED BY:

DRAWING NO: 0-19375-1



THIS DRAWING FORMS PART OF JACQUES, WHITFORD LTD. REPORT No O-19375 AND SHOULD BE READ IN CONJUNCTION WITH THE REPORT

SPECIAL NOTE:

DATA CONCERNING THE VARIOUS STRATA HAVE BEEN OBTAINED AT BOREHOLE LOCATIONS ONLY. THE SOIL STRATIGRAPHY BETWEEN BOREHOLES HAS BEEN INFERRED FROM GEOLOGICAL EVIDENCE AND SO MAY VARY FROM THAT SHOWN

NOTES

1. ▼ GROUNDWATER LEVEL NOV. 17 1989.
2. PROPOSED & EXISTING GRADE ELEVATIONS OBTAINED FROM DELCAN DRAWING No. 1532-115 DATED 86-09.

APP'D	NO.	DETAILS	DATE
		REVISIONS	

DWG. NO.	DESCRIPTION	DATE
	REFERENCES	

DELCAN CORPORATION

DESIGN PROJECT REGIONAL RD. 47
REGIONAL RD. 34 NORTHERLY TO
JEANNE D'ARC BOULEVARD

CUMBERLAND, ONTARIO

STRATIGRAPHIC PROFILE

Jacques, Whitford Limited

CONSULTING ENGINEERS

DATE	SCALE	DRAWN BY
1989/11/30	1 1000 HOR 1 200 VERT	NCP

APPROVED BY

DRAWING NO. O-19375-2