CITY OF CORNWALL

64-F-283 M

REPORT
ON
FOUNDATION INVESTIGATION
FOR PROPOSED STRUCTURE ON
McCONNELL AVENUE AT THE
C.N.R. CROSSING
CORNWALL, ONTARIO

DAMAS AND SMITH LIMITED
CONSULTING ENGINEERS

Submitted By

ASSOCIATED GEOTECHNICAL SERVICES LIMITED
211 Davenport Road, Toronto 5, Ontario.

April, 1964.

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APPENDIX

SECTION 1

INTRODUCTION

The purpose of this report is to present the results of a foundation investigation made in connection with the proposed structure on McConnell Avenue - C.N.R. crossing in Cornwall, Ontario.

The study was authorized by Damas and Smith Limited, on behalf of Mr. R. C. Adams, P.Eng., City Engineer, Cornwall, Ontario.

SECTION 2

DISCUSSION OF PROCEDURES

A primary drilling program consisting of four boreholes and six dynamic cone probes was carried out in the vicinity of the proposed structure during the month of December 1963. In order to provide additional soils and water level information, the drilling program was extended by four boreholes, four dynamic cone probes and four piezometer installations. The secondary program was carried out during February 1964.

The borehole locations for this investigation were referenced to the centreline of the existing McConnell Avenue at the centreline of the C.N.R. right-of-way. The borehole, dynamic cone probe and piezometer locations are shown on the plan in the Appendix.

The elevation of each borehole and dynamic cone probe was determined by spirit level using the top of the northern-most rail at the centreline of the road as a benchmark with an assumed elevation of 196.9.

The boring and sampling operations for this project were carried out using two Boyles screw feed, trailer-mounted drilling rigs. The field boring was carried out under the full-time supervision of a qualified Soils Technician.

The soil borings were performed by standard wash boring sequences. In this procedure, drill casing is driven into the soil by a 350 lb. hammer to a depth of 5 feet or to a change in stratum. Comparative soil density changes were noted by observing the number of blows required to drive the casing. All the soil contained inside the casing was thoroughly washed out to the bottom of the casing and the resultant wash water observed to further determine stratum changes. Sampling tools were then lowered to the bottom of the hole and samples taken. Additional lengths of casing were used as required and the procedure repeated.

Attempts were made to obtain samples of cohesionless soil by means of a 2" O.D. standard split-spoon sampler. The standard penetration test using a 140 lb. hammer falling 30 inches was recorded for each foot of sampler penetration. All samples were visually examined on the site, then placed in jars and forwarded to the Engineering Office.

All undisturbed samples of cohesive soil were obtained in 2" I.D. Shelby tubes utilizing a piston sampling apparatus. Upon removal from the borehole, the ends of the samples were classified, sealed with wax and forwarded to the Engineering Office. All undisturbed samples were extruded and visually examined in the soils laboratory.

Insitu measurements of clay shear strength were made in the boreholes using a 2" by 4" vane and a Torqometer calibrated in increments of 20 inch-lbs. The accuracy of these shear strength determinations, by estimating the readings of torque to the nearest 5 inch-lbs, was ± 25 lbs. per sq. ft. A thrust bearing was used to take the weight of the drill rods in most measurements of shear strength.

Samples of bedrock were obtained by diamond drilling using an Ax size core barrel.

All soils tests were carried out in the soils laboratory of Associated Geotechnical Services Limited. They primarily included moisture content, strength and consolidation determinations. The strength tests consisted of unconfined compression and unconsolidated, undrained triaxial tests. The consolidationtesting was carried out on a Geonor apparatus utilizing a sample area of 10 sq. cms.

SECTION 3

DISCUSSION OF SITE

3.1 Geographic Location

The proposed bridge site is located on McConnell Avenue in the north central portion of the City of Cornwall, Ontario. McConnell Avenue is one of the major traffic arteries from Highway 401 into the City of Cornwall. The site is located about one mile south of Highway No. 401.

3.2 Geology of the Site Area

The Pleistocene and Recent geology of the general geographic area have been described by E. B. Owen in Paper 51-12 of the Geological Survey of Canada. The general sequence of glacial and postglacial events described by Owens is as follows. During the advance and retreat of the last continental ice sheet, deposits of till textured soil were heaped up on the bedrock to form irregular hills and ridges. During the retreat of the ice sheet from the area, the rugged topography of the hills and ridges was somewhat subdued by the deposition of granular outwash on top of the till by the glacial meltwaters. Further subduing of the relief was accomplished by deposition of flat-lying marine sediments (mostly clay capped with a thin layer of sand) in the immediate postglacial land submergence under the Champlain Sea.

Limestone bedrock of the Ordovician Period was found at the site at a depth of approximately 33 feet below ground surface.

3.3 Soil Conditions

The soils at the site are shown in cross-section in the Appendix.

The soils at the site were found in the following order below ground surface:

1. Up to 4 feet of organic sand and garbage fill (north of the railway only)

2. 0.5 to 1.0 feet of topsoil

3. Up to 5 feet of brown fine sand (predominantly south of the railway)

4. 5 to 10 feet of stiff uniform grey clay.

5. Up to 12 feet of medium to stiff grey varved clay and silt with an occasional horizontal thin sand layer.

6. Medium dense grey silt with sand, some gravel having a till-like texture.

7. Very dense grey silt with sand, some gravel, occasional cobble, till texture.

8. Black limestone bedrock.

A four foot deep organic sand and garbage fill containing some boulders was found overlying the topsoil to the north of the railway right-of-way. This fill extended from the north toe of the railway embankment towards the north for a distance of about 350 feet. Underlying this fill on the north and at ground surface south of the tracks a 1 foot thick layer of topsoil was encountered. These strata, being of shallow depth, were not investigated other than by visual field classification as shown on the Borehole Logs.

The soils profile in the Appendix illustrates the location of a layer of brown fine sand that was encountered overlying the deep clay strata. The sand reached a maximum thickness of 6 feet in the vicinity of borehole 9, however, in most location its thickness was less than 5 feet. The layer was not encountered in boreholes 7 and 8 which are located near the north end of the proposed structure. The penetration resistance of the sand was found to vary from 4 to 9 blows per foot. No laboratory tests were carried out on this material.

Beneath the topsoil and sand, a uniform stratum of stiff grey clay was encountered. The clay was found to be weathered to a brown colour near ground surface, however, the major portion of the layer was of a grey colour. In boreholes 1 and 2, the clay contained numerous black nodules which is characteristic of Leda marine clay. The noduling is due to the presence of anaerobic bacteria. Other than the noduling, the clay was found to be massive both at natural water content and upon drying. The strength of this clay layer was determined in the field by insitu vane tests and in the laboratory by unconfined compression testing. There appeared to be no relationship between strength and depth other than the stiffening of the upper part of the layer by weathering. The range of shear strength determinations is shown on figure 1 overleaf. As can be seen on this chart, the vane shear strength of the layer varied from 700 to 1300 p. s. f. The vane sensitivity was found for the most part

CLIENT CITY OF CORNWALL

JOB. NO. 6335 LOCATION M^c CONNELL AT C.N.R.

BOREHOLE NUMBER ______ DEPTH ______

SAMPLE NUMBER ______ DATE ______

ASSOCIATED GEOTECHNICAL SERVICES Limited

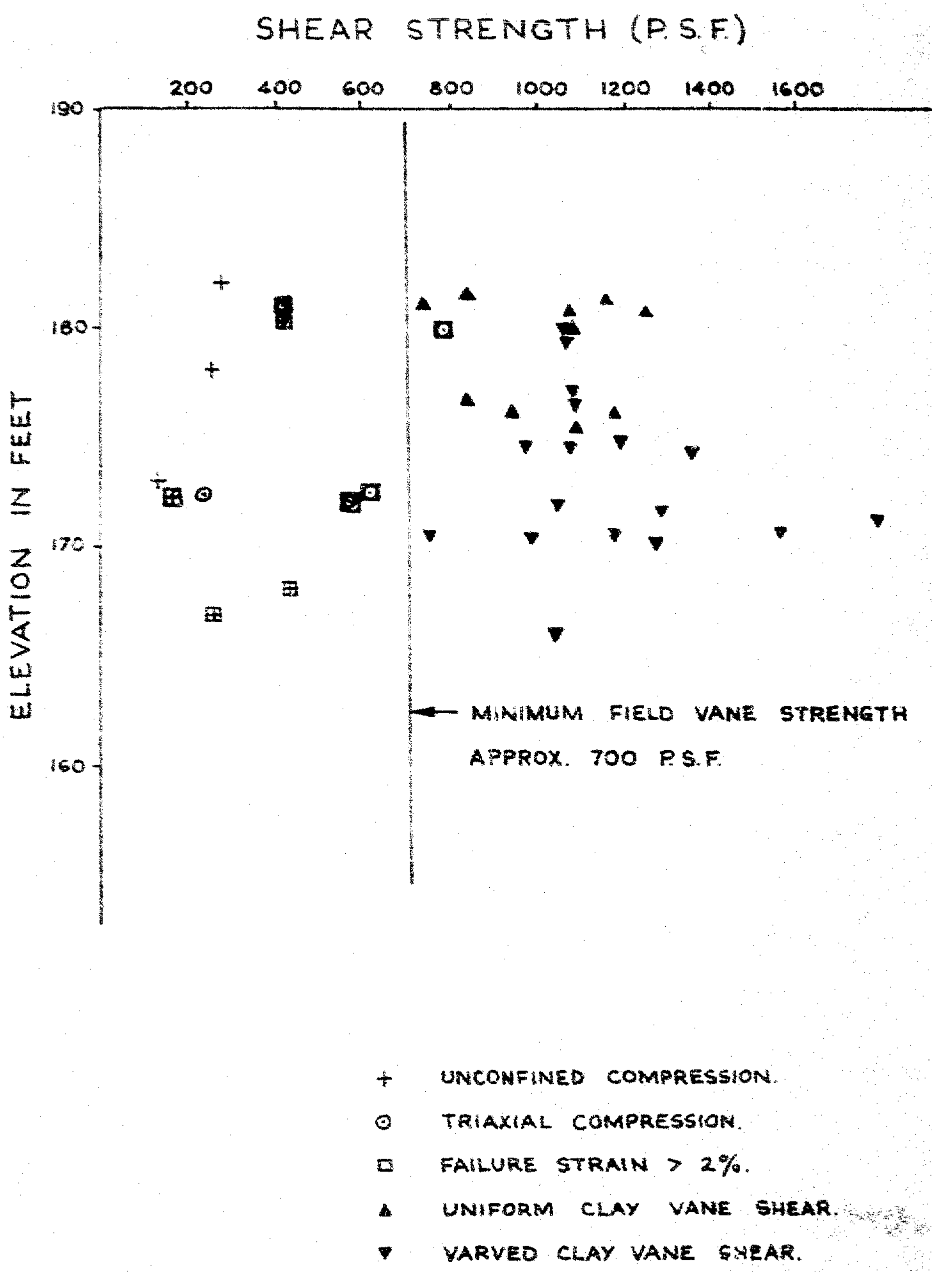


FIGURE 1

to exceed 10. Several unconfined compression tests were carried out on representative samples of this soil as shown on the borehole logs. The results of these tests have been plotted in detail on the stress-strain charts in the Appendix. In general, the laboratory tests were found to give strengths considerably lower than the field vane results even when the laboratory specimens failed at low strain. Moisture contents and unit weight determinations were carried out on the thin walled tube samples as indicated on the Borehole Logs. The majority of moisture contents in the uniform clay varied from 76.0 to 86.0 percent with an average of 78.0 percent. The unit weight of the uniform clay was found to be in the 92.0 to 102.0 p.c.f. range with an average of 98.3 p.c.f. In order to determine the settlement characteristics of the uniform grey clay layer under the load of an approach embankment, consolidation testing was carried out on two representative samples as shown on the following table.

Table Summarizing Results of
Consolidation Testing in Uniform Grey Clay Layer.

Borehole No.	Sample No.	Elevation	Pre-consolidation Pressure T.S.F.	Existing Over-burden Pressure T.S.F.	Net * Pre-consolidation Pressure T.S.F.
9	2	181.0	0.75	0.22	0.53
10	2	178.0	0.70	0.20	0.50

* in excess of estimated present overburden pressure.

Underlying the uniform grey clay, a stratum of stiff varved clay and silt was encountered in all boreholes except No. 7. This varved stratum was found to contain occassional thin layers or partings of fine sand. The continuity of these sand partings was not determined. The varves consisted of alternate dark and light layers of clay and silt ranging in thickness from about 1/16" to 3/8" The strength of the clay layer was investigated by insitu field vane determinations and by laboratory unconfined triaxial compression testing. The results of these tests are summarized on figure 1 and shown in detail on the borehole logs and on the stress-strain charts in the Appendix. The insitu vane strengths werefound to be above 950 p.s.f. except for one determination in borehole 4 (which may represent disturbed soil). Moisture content and unit weight determinations were carried out on representative samples as indicated on the borehole logs. The moisture content of the varved clay was found to vary from 62.0 to 75.0 percent and the unit weight from 96.0 to 104.0 p.c.f. The average moisture content and unit weight was found to be 70.5 percent and 101.5 p.c.f. Consolidation testing was carried out on several samples from this layer as summarized in the following table.

Table Summarizing Results of
Consolidation Testing in Varved Clay and Silt Layer

Borehole No.	Sample No.	Elevation	Pre-consolidation Pressure T.S.F.	Existing over-burden Pressure T.S.F.	Net * Pre-consolidation Pressure T.S.F.
2	2		1.80	0.25	1.55
8	2	178	2.00	0.20	1.80
9	3	176	2.20	0.32	1.88
10	4	168	1.80	0.41	1.39

As can be seen on the soil profile, a medium dense silt with sand, some gravel, occasional cobble till textured stratum was encountered beneath the clay in all boreholes except No. 3. The penetration resistance of this stratum varied from 10 to 23 blows per foot with an average of 15 blows per foot.

The medium dense layer was in turn underlain by a very dense till textured stratum. The penetration resistance was found to vary from 33 to over 100 blows per foot in this soil. The average value of penetration resistance was found to be 70 blows per foot. A few unit weight and moisture content determinations were carried out on samples of this very dense till. The results of these tests are indicated on the borehole logs and found to range between 8.0 and 12.0 percent moisture and 148.5 p.c.f. for unit weight. This soil was not encountered in boreholes 1 and 2 which lie to the east of McConnell Avenue. Bedrock was found to underlie this very dense soil in the other boreholes.

3.4 Water Conditions

At the time of this investigation, the general ground water table was found to be at or near the topsoil level at natural ground surface. The exception to this was in the fill area north of the railway where the water table was found to be a few feet higher.

In order to accurately determine the elevation of the ground water in the medium dense to dense grey silt with sand, some gravel layer underlying the clay, four Geonor piezometers were driven as shown on the plan and soils profile in the Appendix and the water levels recorded over a period of time. The final water levels are shown on the soils profile in the Appendix. As can be seen on the profile the water level in the stratum below the clay has a head varying from elevation 185 to elevation 189.

3.5 Bedrock Conditions

Black limestone bedrock was encountered in all boreholes at a depth of about 33 feet. The upper surface of the bedrock was found to lie fairly flat as shown on the soils profile, The bedrock was found to be sound and free of weathering at the upper surface. The bearing capacity of the bedrock can be taken at 25 T. S. F.

SECTION 4

DISCUSSION OF OVERPASS STRUCTURE

4.1 General

Prior to commencing this soils investigation, the Consulting Engineers had prepared a preliminary layout of an overpass structure to replace the existing McConnell Avenue grade crossing. The bridge was laid out as a 3 span continuous structure centred about the C.N.R. right-of-way. The top of the existing railway embankment is about 5 feet above the surrounding ground level at elevation 191, thus in order to provide the required clearance, the approach fills to the bridge would be about 32 feet high for a 3 span structure.

As the initial field soils data became available, it was apparent that both stability and settlement problems could be expected with high earth fill approach embankments. In view of the complexity of calculations imposed by the variability, strength and depth of clay at the site, conservative assumptions of insitu soil strength and uniformity were made for the purpose of stability calculations. As a result, the earth fill dimensions presented in this report, while satisfactory for the determination of the most economical type of structure should be reviewed for the final arrangement of the structure.

4.2 Approach Fills

4.2.1 Stability

In view of the variable strength and depth of sensitive clay encountered at the site it was apparent that the use of stabilizing berms would be required for the originally contemplated approach fill embankments. In as much as the berms would be necessary between the abutments and the railway tracks as well as *perpendicular to the embankment centreline, the length of berm required to stabilize the fill must be considered by the bridge* designer prior to locating the piers and abutments of the structure. In view of the inter-relationship between the height of approach fill and length of span, a series of embankment design calculations were carried out to determine the necessary fill dimensions for various heights of fill.

The potential critical failure surface will either pass through the uniform clay layer or through the varved silt and clay stratum. In the uniform clay layer, base failure and failure by spreading must be considered. However, in the varved clay layer, because of the presence of horizontal thin sand seams, the danger of failure due to high excess pore pressures in the sand seams must be assessed in addition to base failure and failure by spreading.

The approach embankment design for base failure and failure by spreading was carried out using the charts and other data contained in "The Design of Embankments on Soft Clay" by B. Jakobson, (Geotechnique Vol. I). The shear strength of the clay strata was taken as 700 p. s. f. throughout the layers. This was consistent with the minimum envelope of field vane strengths for the uniform clay layer determined in the 8 boreholes. The increase in soil shear strength near the upper surface of the uniform clay layer was ignored because:

(a) it could not be accurately defined,

(b) it would have a small influence on the total shearing resistance and

(c) it was necessary for the simplified calculations.

The shearing resistance of the embankment fill was also ignored because the incompatibility of failure strain between the fill and clay foundation would not permit full shearing resistance mobilization. In view of the severe consequences of an embankment shear failure, a safety factor of 1. 50 was used for embankment design.

The results of the stability analyses for base failure or failure by spreading through the clay layers are indicated on Figure 2 overleaf. In summary this chart indicates the following:

1. Assuming embankment side slopes of 2 horizontal to 1 vertical, the maximum height of fill that can be built without stabilizing berms is 20 feet.

2. Assuming the potential failure surface to reach the bottom of the clay layer at its deepest point, the maximum height of fill that can be built with one berm is 38 feet.

3. The length and height of stabilizing berms required,varies with the depth of clay as shown on Figure 2. In general, as can be seen on the chart, shorter berms will be required for decreasing depths of clay.

CLIENT CITY OF CORNWALL

JOB. NO. 6335 LOCATION McCONNELL AT C.N.R.

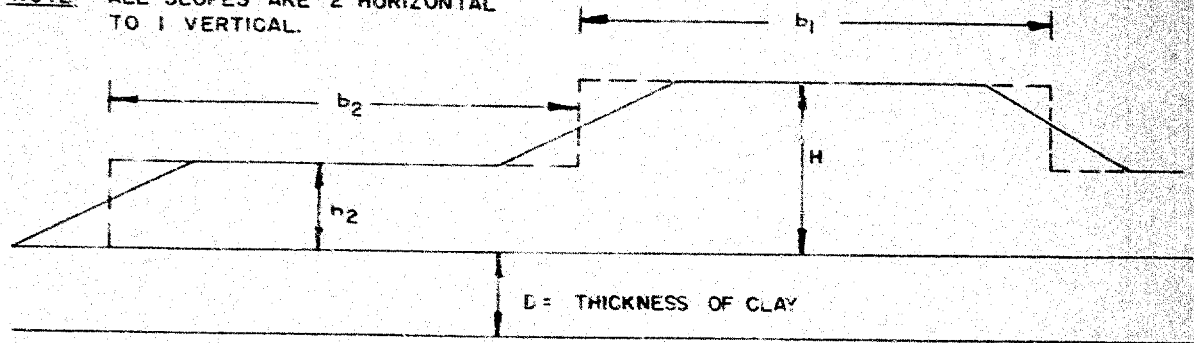
BOREHOLE NUMBER DEPTH

SAMPLE NUMBER DATE

ASSOCIATED GEOTECHNICAL SERVICES Limited

CHART SHOWING LENGTH AND HEIGHT OF BERMS REQUIRED FOR HEIGHT OF FILL.

NOTE: ALL SLOPES ARE 2 HORIZONTAL TO 1 VERTICAL.



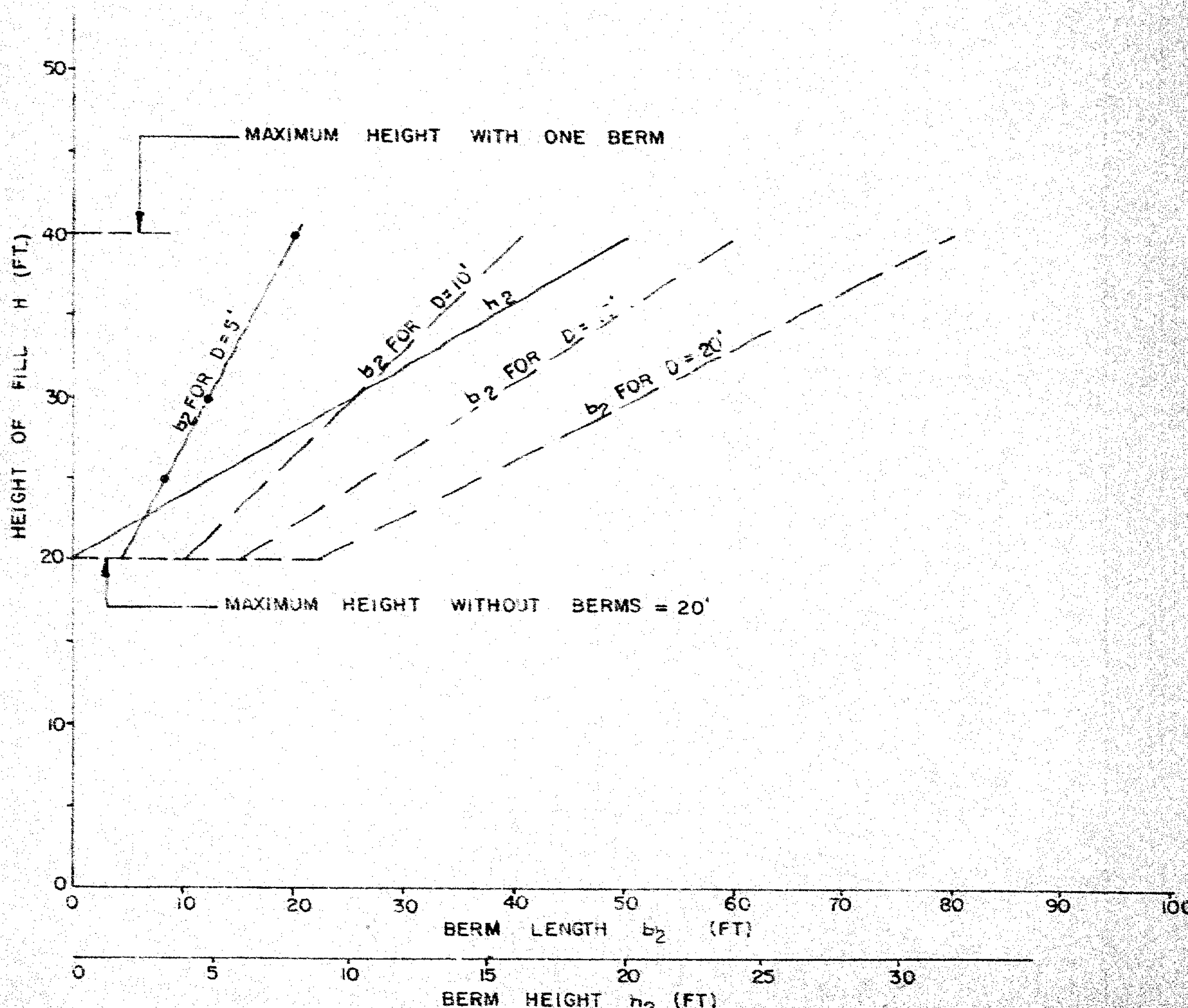


FIGURE 2

The possibility of approach fill failure due to excess hydrostatic pressures in the sand seams in the varved clay stratum must also be considered. However, this type of failure is almost impossible to predict by computation mainly because the magnitude of the potential pore pressures is difficult to predict. In order to guard against this type of failure, it is suggested that when the approach fill is built, special observations be made at the toe of the slope in order to detect heaving. In addition, a number of piezometers should be installed in the sand seam localities in order to detect rising pore pressures. In the event that the safety of the slope becomes impaired, the pore pressures may be reduced by constructing vertical drains near the toe of the slope. The slope may be further stabilized by the construction of berms. It is recommended that these features be designed and the installation supervised by a qualified soils engineer.

4.2.2 Settlement

High approach fill embankments will impose a stress on the soft clay subsoil that will result in settlement of the embankments. In order to determine the amount of this settlement, a number of calculations were carried out by conventional methods for varying conditions of fill height and clay depth. The calculations were based on the results of consolidation tests carried out on representative samples from each clay layer. The results of these calculations are summarized in the table below. The settlement to be expected for the various heights of approach embankments is illustrated on the soils profile in the Appendix.

Table showing Estimated Settlement of Approach Fill

Chainage	Fill Height	Estimated Settlement in inches
37 + 00 on ℄	11'	1.2"
38 + 00 on ℄	15'	1.6"
39 + 00 on ℄	20'	10.8"
40 + 00 on ℄	25'	20.3"
41 + 00 on ℄	31'	36.1"
46 + 00 on ℄	31'	21.6"
47 + 00 on ℄	25'	14.9"
48 + 00 on ℄	20'	13.9"
49 + 00 on ℄	17'	16.2"
50 + 00 on ℄	12'	10.2"

In order to calculate the settlement at any given point on the approach fill, it was first necessary to determine the stress change in the clay along a vertical line through this point. This was done by means of Boussinesq's theory, which assumes the clay to behave as a semi-infinite homogeneous, isotropic and elastic material. In our calculations, the method described by J. O. Osterberg in his paper "Influence Values for Vertical Stress in a Semi-Infinite Mass due to Embankment Loading" has been applied to this settlement.

The magnitude of preconsolidation loading of the clay was determined by the Casagrande method from the consolidation curves (see Appendix). For the purposes of these settlement calculations, the clay strata were assumed to have had a preconsolidation loading of 0.7 T.S.F. for the uniform clay and 1.80 for the varved clay.

The rate of settlement is normally computed on the basis of time curves plotted from the different loading increments in the consolidation tests. From these curves, the coefficient of consolidation C_v is obtained. On the basis of the C_v, the rate of settlement is computed. On this project, however, because of the small increments of loading necessary to prevent squeeze out of the sample in the consolidation tests, the C_v values obtained from the tests are too erratic to permit an accurate assessment of the rate of settlement for this approach fill. A much better guide is provided by local experience with embankments on similar soils. In this respect, the Department of Highways has instrumented the Brookdale Avenue approach fills to Highway 401 (about 3/4 mile north of the proposed Brookdale and C N. R. Overpass) where a 22 ft high embankment rests on similar clay. Settlement at this site has taken place rapidly. For a fill height of 22 ft, 2.15 ft of settlement has taken place within 72 weeks after construction and is still continuing. The fill at the McConnell Avenue - C.N.R. Overpass can be expected to settle in a similar manner, but with a larger magnitude.

Due to the long berms required, settlement of the railway track due to the stress imposed on the clay by the main approach embankments will not be significant.

The approach fill settlement studies indicate that a settlement of about three feet can be expected with a fill height of 32 feet. In order to reduce the maintenance associated with such settlements, it is conceivable that the Consulting Engineers may wish to lengthen the bridge.

4.3 Spread Footings

Considering the use of spread footings for the piers of this structure, it becomes immediately obvious that the low bearing capacity and high settlement of the clay would preclude the use of spread footings for this structure.

4.4 Piles

The presence of a deep layer of soft marine clay dictates the use of pile supported foundations for piers or abutments placed within several hundred feet of the railway tracks. For these locations, we recommend that low displacement end bearing piles, such as H-piles or pipe piles placed into the very dense soil or bedrock be employed for the foundation of this structure.

Considering the use of H-piles, it is expected that the bedrock or very dense till textured soil will be fully capable of supporting the maximum design loading (including negative skin friction) for H-piles, provided that the piles are driven to refusal. Refusal may be taken as penetration resistance of 12 blows or more to the inch using a D-12 Delmag hammer.

In view of the considerable settlements associated with high embankment fills at this site, an allowance for negative skin friction must be made when considering the load bearing capacity of the piles.

The negative skin friction force will be a function of the height of embankment fill and the depth of the clay at the pile cluster location. The negative skin friction due to the fill can be calculated from the following formula:

$$Q' = \frac{A}{n} \gamma H$$

where Q' = the load acting on each pile

A = the area included within the boundaries of the pile cluster.

n = the number of piles

H = the height of the fill

γ = the unit weight of the fill

The negative skin friction due to consolidation in the clay layer can be calculated as follows:

$$Q'' \max = \frac{LHS}{n}$$

where $Q'' \max$ = the maximum value of the skin friction drag

L = circumference of the pile cluster

H = thickness of the clay layer

S = the average shearing resistance of the clay

n = number of piles in the cluster

The total negative skin friction can then be computed as $Q = Q' + Q''$.

SECTION 5

DISCUSSION OF SUBWAY STRUCTURE

In view of the stability and settlement problems associated with high approach fill embankments, the Consulting Engineers have requested us to outline the soils problems that would be involved with the construction of a subway structure.

A profile of the road underpassing the railway is shown on the Soils Profile in the Appendix. As can be seen on the profile, the grades for the road would reach a maximum depth of 22 feet below the track elevation or 17 feet below ground level. Assuming the excavation for the road base courses would involve a further 4 feet of soil, the depth of excavation during construction would be about 21 feet below present ground level (i. e. down to elevation 172).

The excavation for the subway roadway will involve an excess hydrostatic water pressure problem. When the excavation reaches a depth of 21 feet, i. e. about elevation 172, water will be acting on the base of the clay layer at elevation 167 with an uplift pressure of 1375 p. s. f. Assuming the clay to have a unit weight of 100 lbs/c. ft., the downward pressure of the clay will be 500 p. s. f. leaving a net uplift pressure of 875 p. s. f. Thus in order to prevent uplift of the bottom of the excavation during construction it will be necessary to lower the uplift pressure to a safe level. In our opinion, this could only be done effectively through a deep well pumping system.

With the excavation backfilled up to pavement grade, i. e. elevation 176 the excess hydrostatic pressure will be only partially reduced. Therefore, in order to provide an adequate margin of safety against upheaval of the roadway, it will be necessary to maintain full-time artesian water pressure control during the life of the structure. In our opinion, this could best be accomplished by continuous pumping from deep wells.

Another major problem to be encountered with a subway structure would be slope stability of the subway cut. The slopes would have to be made through the Leda Clay, which because of its high sensitivity, has a reputation for sudden earth flows whereby an apparently stable slope is rapidly transferred into a semi-fluid mass. These earth flows, it has been found, are usually triggered by stream erosion, vibration or construction work. In our opinion, prior to finalizing the side slopes, it will be necessary to carry out vibration and pore water pressure studies.

APPENDIX

ASSOCIATED GEOTECHNICAL SERVICES Limited

OFFICE BOREHOLE LOG
BOREHOLE NO. 1

CLIENT: CITY OF CORNWALL
JOB NO.: 6335 LOCATION: McCONNELL & C.N.R.
COORDINATES: 45+51 60' EAST OF EXISTING RD.
ELEVATION (SURFACE): 192.7 (COLLAR): DATUM:
DATE (STARTED): 16/12/63 (FINISHED): 17/12/63 COMPILED: J.K.
RIG NO.: 2 TYPE: B.B.S-1 FIELD SUP.: R.J.G.

SYMBOLS: SILT, CLAY, SAND, GRAVEL, PEAT, FILL
▲ - VANE SHEAR (NATURAL)
○ - VANE SHEAR (REMOLDED)
● - STANDARD PENETRATION
+ UNCONFINED COMPRESSION
⊕ TRIAXIAL

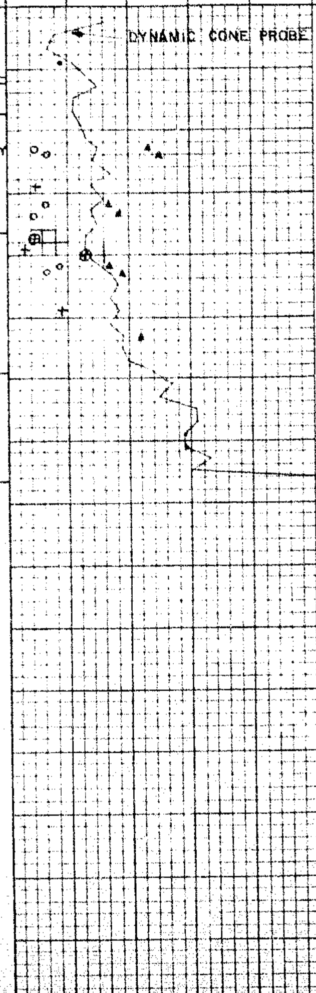
ABBREVIATIONS: UNDISTURBED; DISTURBED BUT REPRESENTATIVE; FAIR; LOST
SS - SPLIT SPOON; ST - SHELBY TUBE; TWP. - THIN WALLED PISTON; DB - DIAMOND BIT
C - CONSOLIDATION TEST; M - MECHANICAL ANALYSIS; T - TRIAXIAL COMPRESSION; K - PERMEABILITY; U - UNCONFINED COMP.; PCF - POUNDS PER CUBIC FOOT; WN - NATURAL WATER CONTENT

BORING LOG

DEPTH (FEET)	ELEV. (FEET)	DESCRIPTION
		SAND AND GARBAGE FILL
5.5	187.2	TOPSOIL
6.0	186.7	BROWN FINE SAND
8.6	184.1	STIFF TO MEDIUM GREY CLAY FREQUENT BLACK NODULES
18.0	174.1	MEDIUM GREY VARVED CLAY AND SILT OCCASIONAL THIN SAND LAYER
29.5	163.2	MEDIUM DENSE SILT WITH SAND SOME GRAVEL
38.1	154.6	BLACK - GREY LIMESTONE BEDROCK
48.7	144.0	END OF BOREHOLE

FIELD TESTS

SHEAR STRENGTH (TONS PER SQUARE FOOT): .25 .50 .75 1.00
STANDARD PENETRATION TEST (BLOWS PER FOOT): 10 20 30 40



SAMPLING

PENETRATION RESISTANCE (BLOWS PER FOOT)	SAMPLE NUMBER	DEPTH FROM (FEET)	DEPTH TO (FEET)	TYPE	RECOVERY LENGTH REC./DIST. DRIV.	REMARKS
9	1	4.0	5.5	SS	5/18	
	2	9.0	11.0	ST	9/24	
	3	14.0	15.5	ST	18/18	
	4	19.0	20.5	ST	18/18	
	5	24.0	26.0	ST	24/24	
	6	29.0	31.0	ST	0/24	SAMPLE RECOVERED WITH SPLIT SPOON
PUSH	7	34.0	35.5	SS	1/18	SAMPLE RECOVERED WITH SAND TRAP
150/4"						
	8	38.1	43.7	D.B	60/66	
	9	43.7	48.7	D.B.	63/60	

LABORATORY TESTS

UNIT WEIGHT PCF: 100 110
ATTERBERG LIMITS: WP ● WL; 80 WN 90

CLIENT CITY OF CORNWALL
JOB NO. 6335 LOCATION MCONNELL & C.N.R.
CO-ORDINATES 48+41 60' EAST OF EXISTING RD.
ELEVATION (SURFACE) 190.0 (COLLAR) DATUM
DATE (STARTED) 16/12/63 (FINISHED) 18/12/63 (COMPILED) J.K.
RIG. NO. 1 TYPE BBS I FIELD SUP. R.J.G.

SYMBOLS: SILT, GRAVEL, CLAY, PEAT, SAND, FILL
▲ - VANE SHEAR (NATURAL)
○ - VANE SHEAR (REMOLDED)
● - STANDARD PENETRATION
+ UNCONFINED COMPRESSION
⊕ TRIAXIAL

ABBREVIATIONS: UNDISTURBED; DISTURBED BUT REPRESENTATIVE; FAIR; LOST
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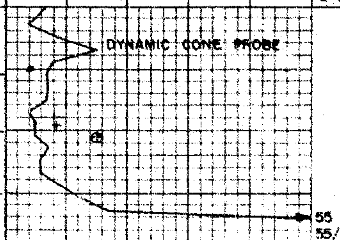
C - CONSOLIDATION TEST; M - MECHANICAL ANALYSIS; T - TRIAXIAL COMPRESSION; K - PERMEABILITY; U - UNCONFINED COMP.; PCF - POUNDS PER CUBIC FOOT; WN - NATURAL WATER CONTENT

ASSOCIATED GEOTECHNICAL SERVICES Limited

OFFICE BOREHOLE LOG

BOREHOLE NO. 2

BORING LOG | FIELD TESTS (SHEAR STRENGTH, TONS PER SQUARE FOOT: .25 .50 .75 1.00; STANDARD PENETRATION TEST, BLOWS PER FOOT: 10 20 30 40) | SAMPLING | LABORATORY (UNIT WEIGHT PCF: 90 100; ATTERBERG LIMITS WP WN WL: 80 90) | TESTS | REMARKS



SCALE FEET	DEPTH FEET	ELEV. FEET	DESCRIPTION	PENETRATION RESISTANCE (BLOWS PER FOOT)	SAMPLE NUMBER	FROM FEET	TO FEET	TYPE	RECOVERY LENGTH REC./DIST. DRIV.	TESTS	REMARKS
			BROWN FINE SAND								
5	5.2	184.8	GREY CLAY WITH FREQUENT BLACK NODULES	4	1	4.0	5.5	SS	18/18		MOIST. CONTENT FROM 4.0' TO 5.2'=24.4%; MOIST " " 5.2' TO 5.5=36.8%
10	10.0	180.0	MEDIUM GREY VARVED SILT AND CLAY		2	9.0	11.0	TWP	22/24	C	OCCASIONAL THIN SAND LAYER IN CLAY LAYERS
15	15.0	175.0	MEDIUM DENSE SILT WITH SAND SOME GRAVEL TILL TEXTURE OCCASIONAL COBBLE		3	14.0	15.0	TWP	12/12		CONE PROBABLY REFUSED ON COBBLE
20				20	4	19.0	20.5	S.S.	14/18		MOIST. CONTENT FROM 19' TO 20'=10.9%
25				19	5	24.0	25.5	S.S.	3/18		MOIST CONTENT FROM 24.0 TO 25.5 = 11.2%
30											UNABLE TO DRIVE SPLIT SPOON
35	32.1	157.9	BLACK LIMESTONE BEDROCK		6	32.1	37.4	D.B.	64/64		
40	37.4	152.6	END OF BOREHOLE								
45											

CLIENT CITY OF CORNWALL
JOB NO. 6335 LOCATION M^C CONNELL & C.N.R.
CO-ORDINATES 47+41 140' WEST OF EXISTING RD.
ELEVATION SURFACE 190.8 COLLAR DATUM
DATE STARTED 17/12/63 FINISHED 19/12/63 COMPILED J.K.
RIG NO. 2 TYPE BBS-1 FIELD SUPV. R.J.G.

SYMBOLS: SILT, GRAVEL, CLAY, PEAT, SAND, FILL

▲ - VANE SHEAR (NATURAL)
O - VANE SHEAR (REMOULDED)
● - STANDARD PENETRATION
+ UNCONFINED COMPRESSION
⊕ TRIAXIAL

ABBREVIATIONS

UNDISTURBED
DISTURBED BUT REPRESENTATIVE
FAIR
LOST

SS - SPLIT SPOON
ST - SHELBY TUBE
TWP - THIN WALLED PISTON
DB - DIAMOND BIT

C - CONSOLIDATION TEST
M - MECHANICAL ANALYSIS
T - TRIAXIAL COMPRESSION
K - PERMEABILITY
U - UNCONFINED COMP.
PCF - POUNDS PER CUBIC FOOT
WN - NATURAL WATER CONTENT

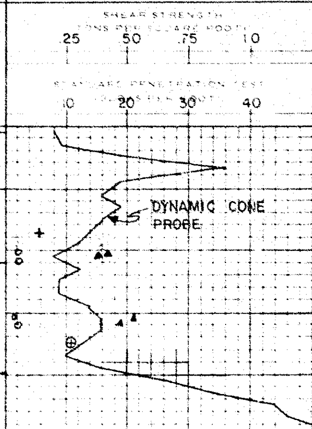
ASSOCIATED GEOTECHNICAL SERVICES Limited

OFFICE BOREHOLE LOG

BOREHOLE NO. 3

DEPTH (FEET)	ELEV. (FEET)	DESCRIPTION
0.3	190.5	TOPSOIL
		BROWN FINE SAND
5.0	185.8	MEDIUM GREY CLAY
11.0	179.8	MEDIUM TO STIFF VARVED CLAY & SILT NUMEROUS THIN SAND LAYERS
19.7	171.1	VERY DENSE SILT WITH SAND SOME GRAVEL TILL TEXTURE OCCASIONAL PEBBLE
36.1	154.7	BLACK LIMESTONE BEDROCK
41.2	149.6	END OF BOREHOLE

FIELD TESTS



SAMPLING

PENETRATION RESISTANCE	SAMPLE NUMBER	DEPTH FROM (FEET)	DEPTH TO (FEET)	TYPE	RECOVERED LENGTH / REC. DIST.
3	1	4.0	5.5	SS	18/18
	2	7.8	9.8	ST	21/24
	3	13.0	15.0	ST	17/24
	4	18.0	19.7	ST	14/20
120/6	5	23.0	23.5	SS	0/6
59	6	25.0	26.5	SS	12/18
33	7	30.0	31.5	SS	18/18
93	8	35.0	36.1	SS	12/13
	9	36.1	41.2	D.B	61/61

LABORATORY TESTS

UNIT WEIGHT PCF: 100 110
ATTERBERG LIMITS: 80 90

REMARKS

W_N : 55.2 %

ABSOLUTE REFUSAL AT 24.9 WITH CONE PROBE

MOIST. CONTENT FROM 30.0' TO 31.5' = 8.1%
UNIT WEIGHT " " " = 148.5
MOIST. CONTENT FROM 35.0' TO 36.1' = 9.7%

ASSOCIATED GEOTECHNICAL SERVICES Limited

OFFICE BOREHOLE LOG
BOREHOLE NO. 4

CLIENT: CITY OF CORNWALL
JOB NO. 6335 LOCATION McCONNELL & C.N.R.
CO-ORDINATES 44+51 140' WEST OF EXISTING RD.
ELEVATION (SURFACE) 191.6 (COLLAR) DATUM
DATE (STARTED) 18/12/63 (FINISHED) 19/12/63 (COMPILED) J.K.
RIG. NO. 1 TYPE BBS-1 FIELD SUP. R.J.G.

SYMBOLS: SILT, CLAY, SAND, GRAVEL, PEAT, FILL

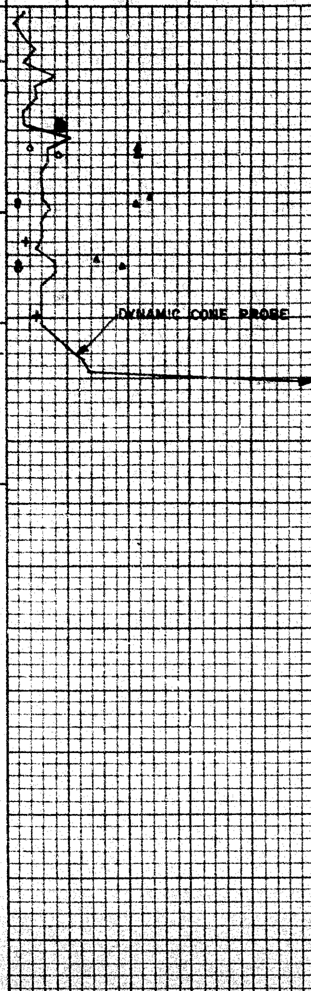
▲ - VANE SHEAR (NATURAL)
○ - VANE SHEAR (REMOLDED)
● - STANDARD PENETRATION
\+ UNCONFINED COMPRESSION
● TRIAXIAL

ABBREVIATIONS:
UNDISTURBED; DISTURBED BUT REPRESENTATIVE; FAIR; LOST
SS - SPLIT SPOON
ST - SHELBY TUBE
TWP - THIN WALLED PISTON
DB - DIAMOND BIT
C - CONSOLIDATION TEST
M - MECHANICAL ANALYSIS
T - TRIAXIAL COMPRESSION
K - PERMEABILITY
U - UNCONFINED COMP.
PCF - POUNDS PER CUBIC FOOT
WN - NATURAL WATER CONTENT

BORING LOG / FIELD TESTS / SAMPLING / LABORATORY TESTS

FIELD TESTS: SHEAR STRENGTH (TONS PER SQUARE FOOT) .25 .50 .75 1.00; STANDARD PENETRATION TEST (BLOWS PER FOOT) 10 20 30 40

LABORATORY TESTS: UNIT WEIGHT PCF 90 100; ATTERBERG LIMITS WP × — ● WN — ○ WL 70 80



DEPTH FEET	ELEV. FEET	DESCRIPTION	PENETRATION RESISTANCE (BLOWS PER FOOT)	SAMPLE NUMBER	FROM FEET	TO FEET	TYPE	RECOVERY LENGTH REC. / DIST. DRIV.
		BOULDERY ORGANIC MATERIAL						
4.5	187.1	BROWN FINE SAND	6	1	4.0	5.5	SS	18/18
6.0	185.6	STIFF TO MEDIUM GREY CLAY		2	9.0	11.0	TWP	8/24
				3	13.0	15.0	TWP	15/24
16.5	175.1	MEDIUM GREY VARVED CLAY AND SILT OCCASIONAL SAND SEAM		4	18.0	20.0	TWP	21/24
				5	23.0	25.0	TWP	19/24
25.4	166.2	MEDIUM DENSE TO						
28.0		VERY DENSE GREY SILT WITH SAND SOME GRAVEL TILL TEXTURE	62	6	28.0	29.5	SS	14/18
			6	7	33.0	34.5	SS	18/18
38.5	153.1	BLACK LIMESTONE BEDROCK	28/4"	8	38.0	38.4	SS	0/4
				9	38.5	43.5	D.B.	54/60
43.5	148.1	END OF BOREHOLE						

REMARKS:
ABSOLUTE REFUSAL AT 29.8 WITH CONE PROBE
MOIST. CONTENT FROM 33.0 TO 34.5 = 11.7%

CLIENT	CITY OF CORNWALL
JOB NO.	6335
LOCATION	McCONNELL AVE
CO-ORDINATES	37+69 65' WEST OF EXISTING RD.
ELEVATION (SURFACE)	189.0
(COLLAR)	
DATUM	
DATE (STARTED)	15 FEB. 64
(FINISHED)	16 FEB 64
COMPILED	
RIG. NO.	2
TYPE	B.B.S. I
FIELD SUP.	G. J.

SYMBOLS: SILT, GRAVEL, CLAY, PEAT, SAND, FILL

▲ - VANE SHEAR (NATURAL)
○ - VANE SHEAR (REMOLDED)
● - STANDARD PENETRATION

ABBREVIATIONS

UNDISTURBED; DISTURBED BUT REPRESENTATIVE; FAIR; LOST

SS - SPLIT SPOON
ST - SHELBY TUBE
TWP. - THIN WALLED PISTON
DB - DIAMOND BIT

C - CONSOLIDATION TEST
M - MECHANICAL ANALYSIS
T - TRIAXIAL COMPRESSION
K - PERMEABILITY
U - UNCONFINED COMP.
PCF - POUNDS PER CUBIC FOOT
WN - NATURAL WATER CONTENT

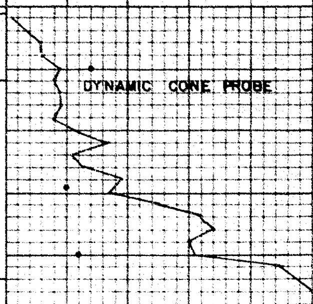
ASSOCIATED GEOTECHNICAL SERVICES Limited

OFFICE BOREHOLE LOG

BOREHOLE NO. 7

DEPTH FEET	ELEV. FEET	DESCRIPTION	PENETRATION RESISTANCE (BLOWS PER FOOT)	SAMPLE NUMBER	FROM FEET	TO FEET	TYPE	RECOVERY LENGTH REC. / DIST. DRIV.	REMARKS
0.5	188.5	TOPSOIL							
6.0	183.0	GREY CLAY SOME SILT OCC. SAND POCKET	14	1	4.0	5.5	T.W.P.	10/18	
		MEDIUM DENSE SILT WITH SAND SOME GRAVEL TRACE CLAY		2	9.0	10.5	T.W.P.	NIL	ATTEMPTED T.W.P. & S.S. NO RECOVERY WASH SAMPLE TAKEN FROM 10.0 TO 14.0
			10	3	14.0	15.5	S.S.	3/18	
22.0	167.0		12	4	19.0	20.5	S.S.	12/18	
		VERY DENSE SAND WITH SILT SOME GRAVEL TILL TEXTURE	100 FOR 6" 83	5	24.0	25.5	S.S.	6/18	
30.0	159.0		127	6	29.0	31.0	S.S.	3/12	
32.3	156.7	GREY SILT		7	32.3	34.9	R.C.		RAN CASING, RECOVERED BOULDER CORE
36.5	152.5	SAND AND GRAVEL SOME BOULDERS		8	36.5	41.0	R.C.	50/54	
41.0	148.0	BLACK LIMESTONE BEDROCK							
		END OF BOREHOLE							

FIELD TESTS — SHEAR STRENGTH (TONS PER SQUARE FOOT); STANDARD PENETRATION TEST (BLOWS PER FOOT) 10 20 30 40



LABORATORY — UNIT WEIGHT PCF; ATTERBERG LIMITS WP WN WL

TESTS

CLIENT CITY OF CORNWALL
JOB NO. 6335 LOCATION McCONNELL AVE
CO-ORDINATES 39+42 60' WEST OF EXISTING RD.
ELEVATION (SURFACE) 188.4 (COLLAR) DATUM
DATE (STARTED) 15 FEB. 64 (FINISHED) 16 FEB 64 (COMPILED)
RIG NO. 1 TYPE BBS. 1 FIELD SUPV. G.J.

SYMBOLS: SILT, GRAVEL, CLAY, PEAT, SAND, FILL
▲ - VANE SHEAR (NATURAL)
O - VANE SHEAR (REMOLDED)
● - STANDARD PENETRATION

ABBREVIATIONS: UNDISTURBED; DISTURBED BUT REPRESENTATIVE; FAIR; LOST
SS - SPLIT SPOON
ST - SHELBY TUBE
TWP. - THIN WALLED PISTON
DB - DIAMOND BIT
C - CONSOLIDATION TEST
M - MECHANICAL ANALYSIS
T - TRIAXIAL COMPRESSION
K - PERMEABILITY
U - UNCONFINED COMP.
PCF - POUNDS PER CUBIC FOOT
WN - NATURAL WATER CONTENT

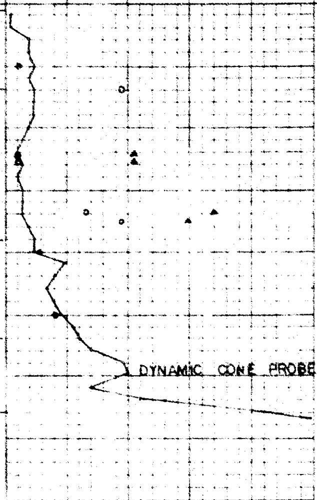
ASSOCIATED GEOTECHNICAL SERVICES Limited

OFFICE BOREHOLE LOG
BOREHOLE NO. 8

BORING LOG

DEPTH FEET	ELEV. FEET	DESCRIPTION
0.5	187.9	TOPSOIL
7.0	181.4	STIFF UNIFORM GREY CLAY
19.0	169.4	STIFF VARVED GREY CLAY SOME SILT OCCASIONAL SAND SEAM
27.0	161.4	MEDIUM DENSE TO
33.0	155.4	DENSE GREY SILT WITH SAND SOME GRAVEL TILL TEXTURE
37.5	150.9	BLACK LIMESTONE BEDROCK
		END OF BOREHOLE

FIELD TESTS — SHEAR STRENGTH (TONS PER SQUARE FOOT) .25 .50 .75 1.00; STANDARD PENETRATION TEST (BLOWS PER FOOT) 10 20 30 40



SAMPLING / LABORATORY TESTS

PENETRATION RESISTANCE (BLOWS PER FOOT)	SAMPLE NUMBER	DEPTH FROM FEET	DEPTH TO FEET	TYPE	RECOVERY LENGTH REC./DIST. DRIV.	TESTS	REMARKS
4	1	4.0	5.5	S.S.	12/18		VANE AT 7'0 = 1.37 T.S.F. REMOLDED = 0.42 T.S.F.
	2	9.0	10.5	T.W.P.	18/18	C	
	3	14.0	15.5	T.W.P.	9/18		
10	4	19.0	20.5	S.S.	2/18		
16	5	24.0	25.5	S.S.	12/18		
103	6	29.0	30.0	S.S.	8/12		
100 FOR 8"	7	32.0	33.5	R.C.	11/18		
	8	33.5	37.5	R.C.	44/48		

UNIT WEIGHT PCF 90 100; ATTERBERG LIMITS WP WN WL 70 80

CLIENT CITY OF CORNWALL
JOB NO. 6335 LOCATION MC CONNELL AVENUE
CO-ORDINATES 47+49 65' WEST EXISTING RD.
ELEVATION SURFACE 191.7 COLLAR DATUM
DATE STARTED 18 FEB.1964 FINISHED 19 FEB 64 COMPLETE
2 BBS.1 G.J.

SYMBOLS: SILT, CLAY, SAND, GRAVEL, PEAT, FILL
▲ - VANE SHEAR NATURAL
○ - VANE SHEAR (REMOLDED)
● - STANDARD PENETRATION

ABBREVIATIONS: UNDISTURBED; DISTURBED BUT REPRESENTATIVE; FAIR; LOST
SS - SPLIT SPOON; ST - SHELBY TUBE; TWP - THIN WALLED PISTON; DB - DIAMOND BIT
C - CONSOLIDATION TEST; M - MECHANICAL ANALYSIS; T - TRIAXIAL COMPRESSION; K - PERMEABILITY; U - UNCONFINED COMP.; PCF - POUNDS PER CUBIC FOOT; WN - NATURAL WATER CONTENT

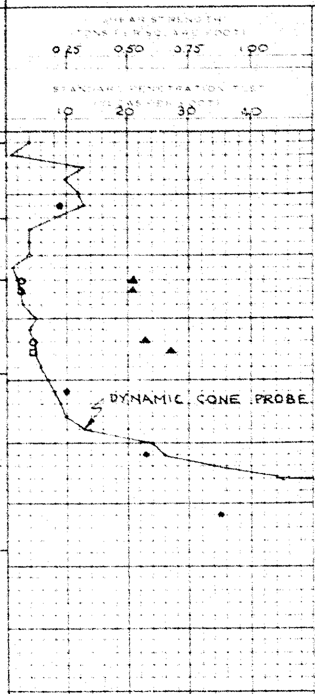
ASSOCIATED GEOTECHNICAL SERVICES Limited

OFFICE BOREHOLE LOG

BOREHOLE NO. 9

Depth	Elev.	Description	Penetration resistance (blows per foot)	Sample number	From (feet)	To (feet)	Type	Recovery	Lab tests
1.0	190.7	TOPSOIL							
7.0	184.7	BROWN FINE SAND	9	1	5.0	6.5	S.S.	15/18	
12.0	179.7	STIFF UNIFORM GREY CLAY		2	10.0	11.5	T.W.P.	18/18	C
19.5	172.2	STIFF GREY VARVED CLAY AND SILT		3	15.0	16.5	T.W.P.	18/18	C
27.0	164.7	MEDIUM DENSE GREY SAND WITH SILT OCCASIONAL COBBLE TILL TEXTURE	10	4	20.0	21.5	S.S.	18/18	
			23	5	25.0	26.5	S.S.	15/18	
33.8	157.9	VERY DENSE GREY SAND WITH SILT, OCCASIONAL COBBLE TILL TEXTURE	35	6	30.0	31.5	S.S.	15/18	
39.3	152.4	BLACK LIMESTONE BEDROCK		7	33.8	39.3	R.C.	67/67	

END OF BOREHOLE



LABORATORY: UNIT WEIGHT PCF 90 100; ATTERBERG LIMITS 70 80

REMARKS

CITY OF CORNWALL
6335 — Mc CONNELL AVENUE
49 + 64 65' WEST EXISTING RD.
1894
17 FEB 64 — 18 FEB 64
1 — BBS — G J

SYMBOLS: SILT, CLAY, SAND, GRAVEL, PEAT, FILL
▲ - VANE SHEAR NATURAL
○ - VANE SHEAR REMOULDED
● - STANDARD PENETRATION

ABBREVIATIONS
UNDISTURBED
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U - UNCONFINED COMP.
PCF - POUNDS PER CUBIC FOOT
WN - NATURAL WATER CONTENT

ASSOCIATED GEOTECHNICAL SERVICES Limited

OFFICE BOREHOLE LOG
BOREHOLE NO. 10

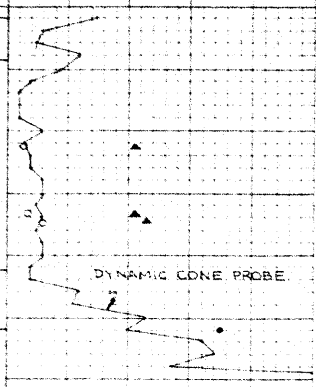
Depth	Elev.	Description
1.0	188.4	TOPSOIL
4.5	184.9	LOOSE BROWN SAND
13.0	176.4	UNIFORM GREY CLAY
21.5	167.9	STIFF VARVED GREY SILT AND CLAY
26.0	163.4	MEDIUM DENSE GREY SAND WITH SILT, OCCASIONAL COBBLE, TILL TEXTURE
35.5	153.9	VERY DENSE GREY SAND WITH SILT, OCCASIONAL COBBLE, TILL TEXTURE
41.0	148.4	BLACK LIMESTONE BEDROCK

END OF BOREHOLE

FIELD TESTS

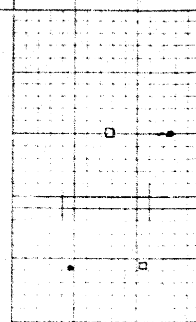
SHEAR STRENGTH: 0.25 0.50 0.75 1.00

STANDARD PENETRATION TEST: 10 20 30 40



N	Sample No.	Condition	From	To	Type	Recovery
	1	LOST	5.0	6.5	TWP	0/18
	2	UNDISTURBED	10.0	11.5	TWP	18/18
	3	UNDISTURBED	15.0	16.5	TWP	18/18
	4	UNDISTURBED	20.0	21.5	ST	18/18
35	5	DISTURBED BUT REPRESENTATIVE	25.0	26.5	SS	18/18
	6	UNDISTURBED	35.5	37.5	RC	24/24
	7	UNDISTURBED	38.5	41.0	RC	30/30

LABORATORY: 90 100; 70 80



TESTS: C (sample 4)

REMARKS: SAMPLE RECOVERED WITH SPLIT SPOON (sample 1)

CITY OF CORNWALL

6335 — M^c^CONNELL AT C.N.R.

190.3

14/12/63 — 14/12/63

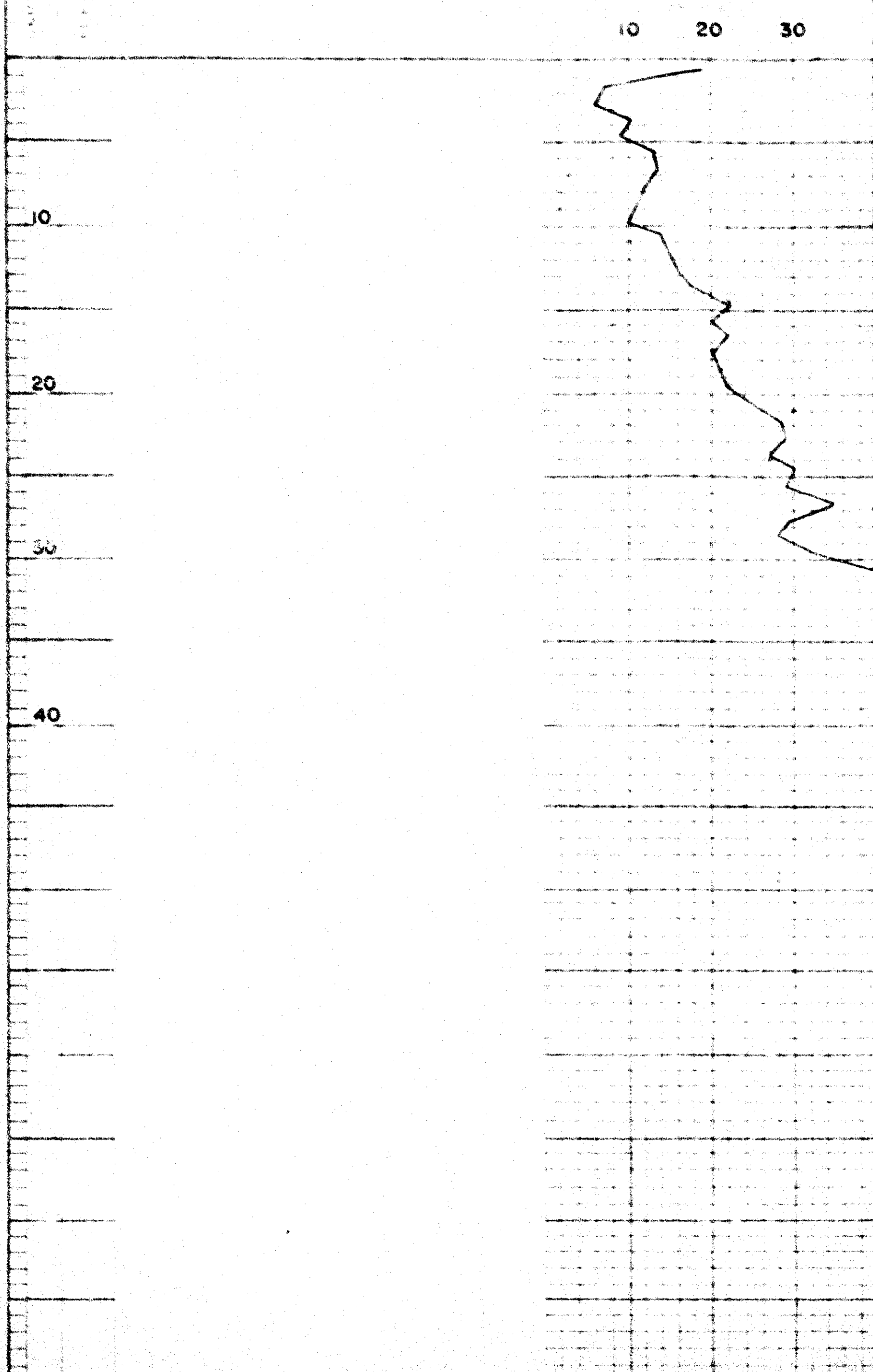
2 — BBS-1 — R.J.G

ASSOCIATED GEOTECHNICAL SERVICES Limited

OFFICE BOREHOLE LOG

DYNAMIC CONE PROBE #5

BOREHOLE NO.



43
41
50
44
99
104
120 FOR 4"

REFUSAL AT 37'4"

CLIENT CITY OF CORNWALL

JOB NO. 6335 LOCATION M^cCONNELL & C.N.R.

CO-ORDINATES

ELEVATION SURFACE 190.0 COLLAR DATUM

DATE STARTED 14/12/63 FINISHED 14/12/63

RIG NO. 2 TYPE BBS-1 SUP. R.J.G.

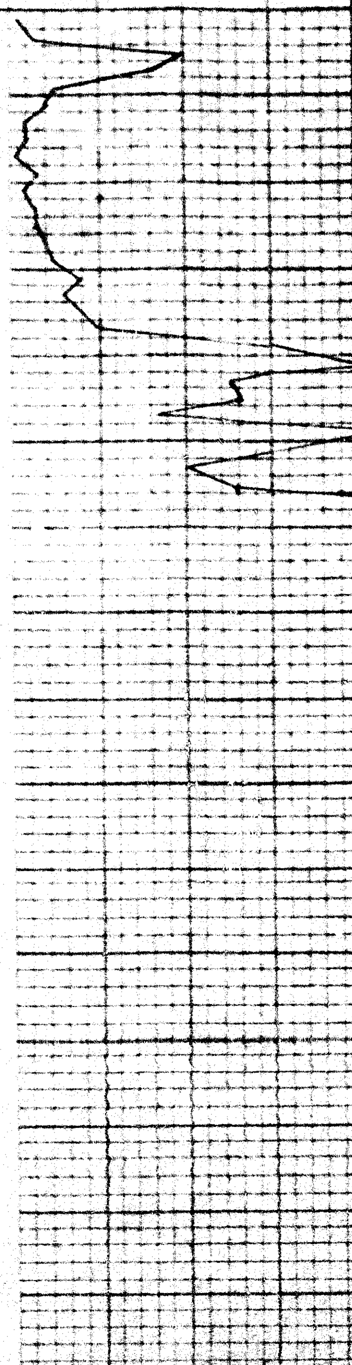
ASSOCIATED GEOTECHNICAL SERVICES Limited

OFFICE BOREHOLE LOG

DYNAMIC CONE PROBE # 8

BOREHOLE NO.

DEPTH FEET	ELEV. FEET	DESCRIPTION	LOG	SHEAR STRENGTH TONS PER SQ. FT. / STANDARD PENETRATION BLOWS PER FOOT 10 20 30	SAMPLE NO.	CONDITION	REMARKS
5							
10							
15							
20					45		
25					66		
					46		
30					65		
					134		160 FOR 7"
							REFUSAL AT 31' 7"
35							
40							



DEFECTS IN NEGATIVE DUE TO CONDITION OF ORIGINAL DOCUMENT

CITY OF CORNWALL
6345 — M^C CONNELL AT C N R
STA 45+64, 70' WEST OF EXISTING ROAD
190.5
FEB 3/64 — FEB 3/64 — W.K.
B.B.S. — G.J.

SYMBOLS: GRAVEL; PEAT; ROCK

▲ = VANE SHEAR (NATURAL)
○ = VANE SHEAR (REMOULDED)
● = STANDARD PENETRATION

ABBREVIATIONS

UNDISTURBED; DISTURBED BUT REPRESENTATIVE; WASH; LOST

SS = SPLIT SPOON
ST = SHELBY TUBE
TWP = THIN WALLED PISTON
DB = DIAMOND BIT

C = CONSOLIDATION TEST
M = MECHANICAL ANALYSIS
T = TRIAXIAL COMPRESSION
K = PERMEABILITY
U = UNCONFINED COMP.
PCF = POUNDS PER CUBIC FOOT
WN = NATURAL WATER CONTENT

ASSOCIATED GEOTECHNICAL SERVICES
Limited

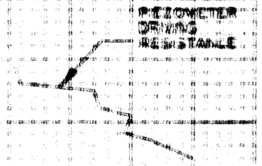
PIEZOMETER #1

FIELD TESTS — SAMPLING — LABORATORY — TESTS

10 20 30 40

5
10
15
20

190.5 172.5



BASE OF PIEZOMETER

DROVE BULL PLUG 10' TO CLEAR FROST AND POSSIBLE BOULDERS

DROVE PIEZOMETER TO 18.0'

ASSOCIATED GEOTECHNICAL SERVICES
Limited

PIEZOMETER #2

CITY OF CORNWALL

6335 — M^cCONNELL AT C.N.R.

STA 43 + 34, 65' WEST OF EXISTING ROAD

190.7

FEB 3/64 — FEB 3/64 — W.K.

B.B.S.I. — G.J.

SYMBOLS: SILT, GRAVEL, CLAY, PEAT, SAND, FILL

▲ – VANE SHEAR (NATURAL)
○ – VANE SHEAR (REMOULDED)
● – STANDARD PENETRATION

ABBREVIATIONS

UNDISTURBED
DISTURBED BUT REPRESENTATIVE
FAIR
LOST

SS – SPLIT SPOON
ST – SHELBY TUBE
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DB – DIAMOND BIT

C – CONSOLIDATION TEST
M – MECHANICAL ANALYSIS
T – TRIAXIAL COMPRESSION
K – PERMEABILITY
U – UNCONFINED COMP.
PCF – POUNDS PER CUBIC FOOT
WN – NATURAL WATER CONTENT

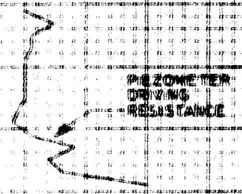
FIELD TESTS — SAMPLING — LABORATORY TESTS — REMARKS

STANDARD PENETRATION TEST: 10 20 30 40

FROM FEET — TO FEET — TYPE — RECOVERY LENGTH

ATTERBERG LIMITS: WP — WN — WL

Depth	Elev.		Description
5			
10			
15			
20			
25	25.0'	165.7	BASE OF PIEZOMETER
30			



DROVE BULL PLUG 10' TO CLEAR FROST AND POSSIBLE BOULDERS.

DROVE PIEZOMETER TO 25.0'

NOTE – LAST BLOW AT 25.0' SOUNDED AS IF ON ROCK.

ASSOCIATED GEOTECHNICAL SERVICES Limited

PIEZOMETER #3

CITY OF CORNWALL

6335 — McCONNELL AT C.N.R.

STA. 40+14, 66' WEST OF EXISTING ROAD

189.7

FEB 13/64 — FEB 13/64 — W.K.

B.B.S.1 — G.J.

SYMBOLS: GRAVEL, PEAT — ● VANE SHEAR (NATURAL) — ○ VANE SHEAR (REMOULDED) — ● STANDARD PENETRATION

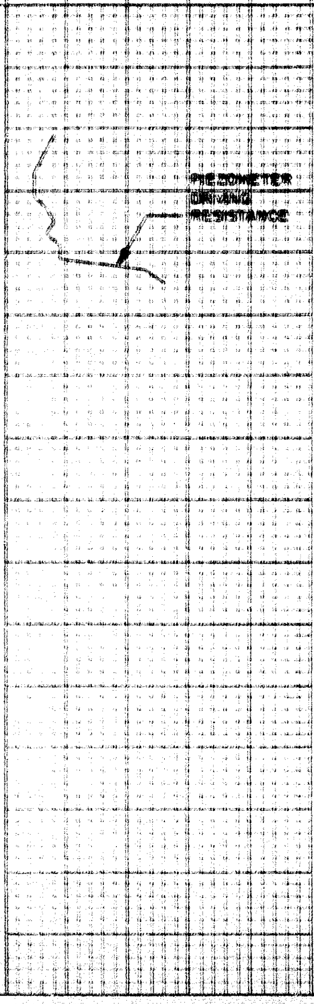
ABBREVIATIONS: UNDISTURBED, DISTURBED BUT REPRESENTATIVE, FAIR, LOST — SS = SPLIT SPOON, ST = SHELBY TUBE, TWP = THIN WALLED PISTON, DB = DIAMOND BIT — C = CONSOLIDATION TEST, M = MECHANICAL ANALYSIS, T = TRIAXIAL COMPRESSION, K = PERMEABILITY, U = UNCONFINED COMP., PCF = POUNDS PER CUBIC FOOT, W = NATURAL WATER CONTENT

FIELD TESTS — SAMPLING — LABORATORY TESTS — REMARKS

STANDARD PENETRATION TEST: 10, 20, 30, 40

ATTENBERG LIMITS: WP, WN, WL

Depth	Elevation	Description	Remarks
5			DROVE 10' LENGTH OF A.X. ROD WITHDREW A.X. ROD.
10			DROVE PIEZOMETER TO 23.0'
15			
20			
23.0	166.7	BASE OF PIEZOMETER	
25			
30			



ASSOCIATED GEOTECHNICAL SERVICES Limited

PIEZOMETER #4

CITY OF CORNWALL

6335 McCONNELL AT C.N.R.

STA 44+61, 69' WEST OF EXISTING ROAD

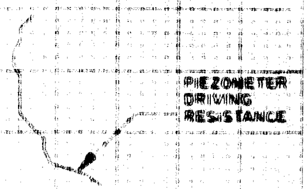
190.4

FEB 13/64 FEB 13/64 W.K.

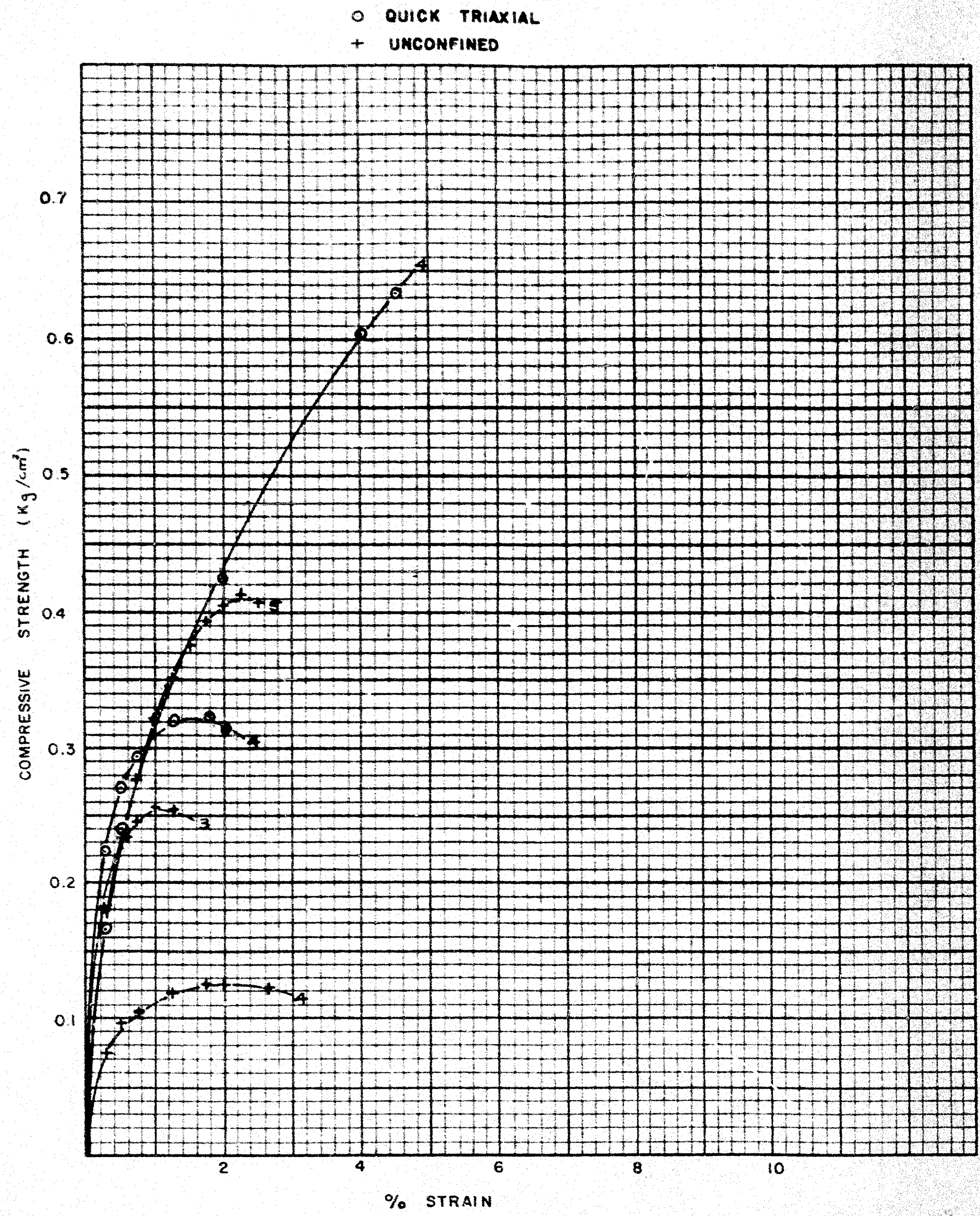
G.J.

5	
10	DROVE 10' LENGTH OF A.X. ROD WITHDREW A.X. ROD
15	DROVE PIEZOMETER TO 25.0'
20	
25	165.4
30	BASE OF PIEZOMETER

10 20 30 40



CLIENT CITY OF CORNWALL	ASSOCIATED GEOTECHNICAL SERVICES Limited
JOB NO. 6335 LOCATION M^{C}CONNELL AVE	
BOREHOLE NUMBER 1 DATE	SOIL MECHANICS LABORATORY
SAMPLE NUMBER DEPTH	STRENGTH TESTS



CLIENT CITY OF CORNWALL

JOB NO. 6335 LOCATION M^C^CONNELL AVE

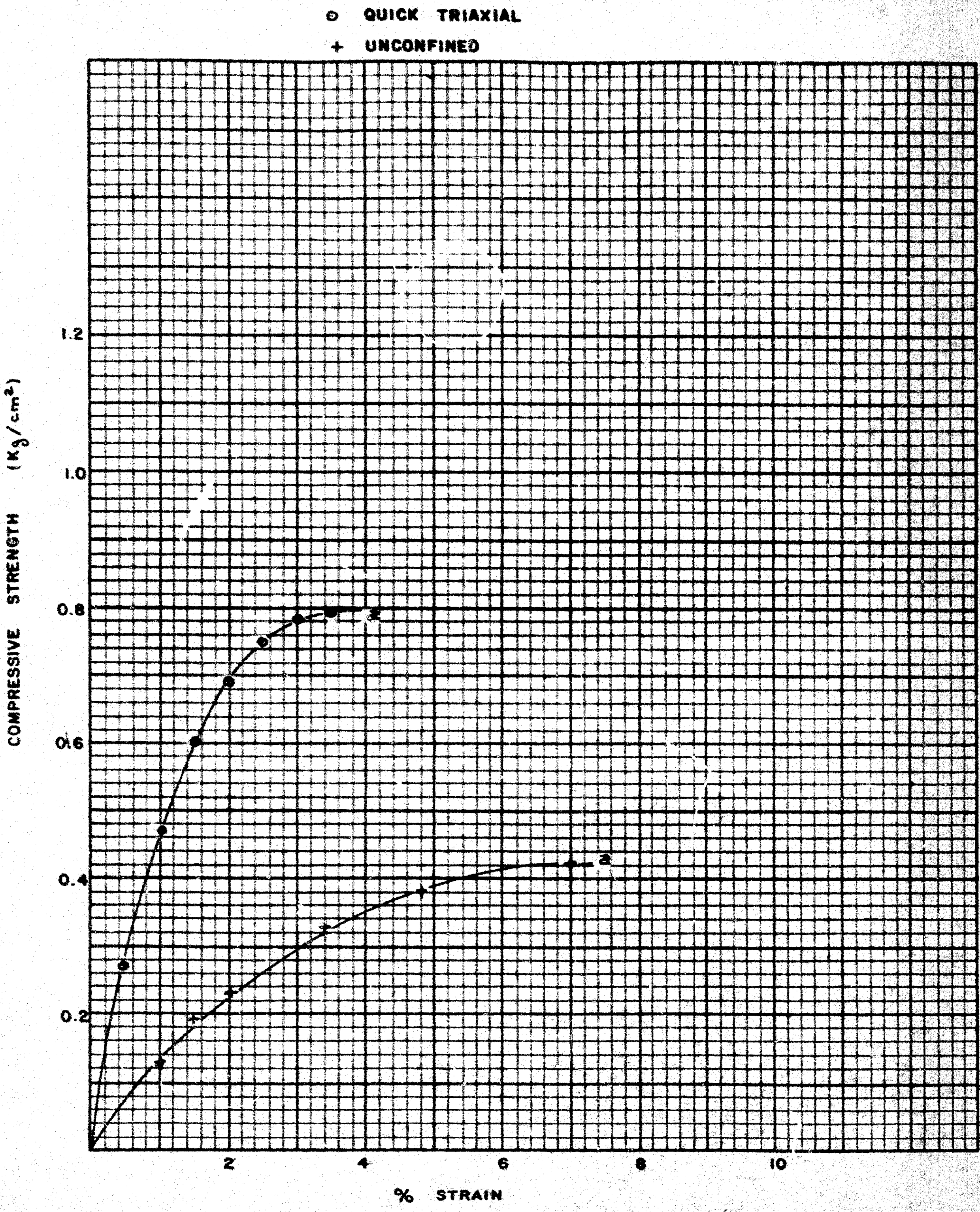
BOREHOLE NUMBER 2 DATE

SAMPLE NUMBER DEPTH

ASSOCIATED GEOTECHNICAL SERVICES Limited

SOIL MECHANICS LABORATORY

STRENGTH TESTS

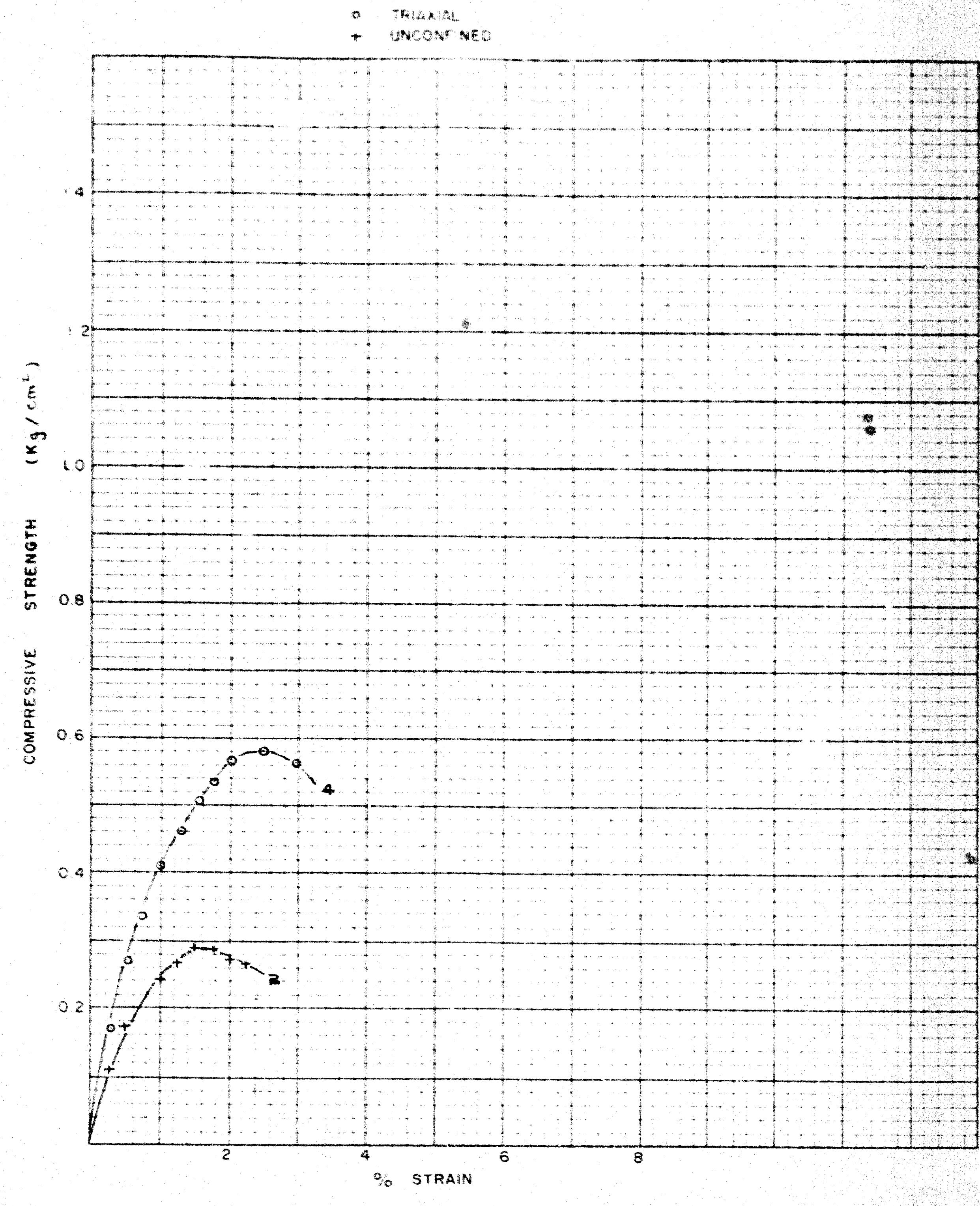


CLIENT	CITY OF CORNWALL	
JOB NO.	6335	M^c CONNELL AVE
BOREHOLE NUMBER	3	DATE
SAMPLE NUMBER		DEPTH

ASSOCIATED GEOTECHNICAL SERVICES Limited

SOIL MECHANICS LABORATORY

STRENGTH TESTS



CLIENT CITY OF CORNWALL

JOB NO. 6335 LOCATION M^cCONNELL AVE

BOREHOLE NUMBER 4 DATE

SAMPLE NUMBER DEPTH

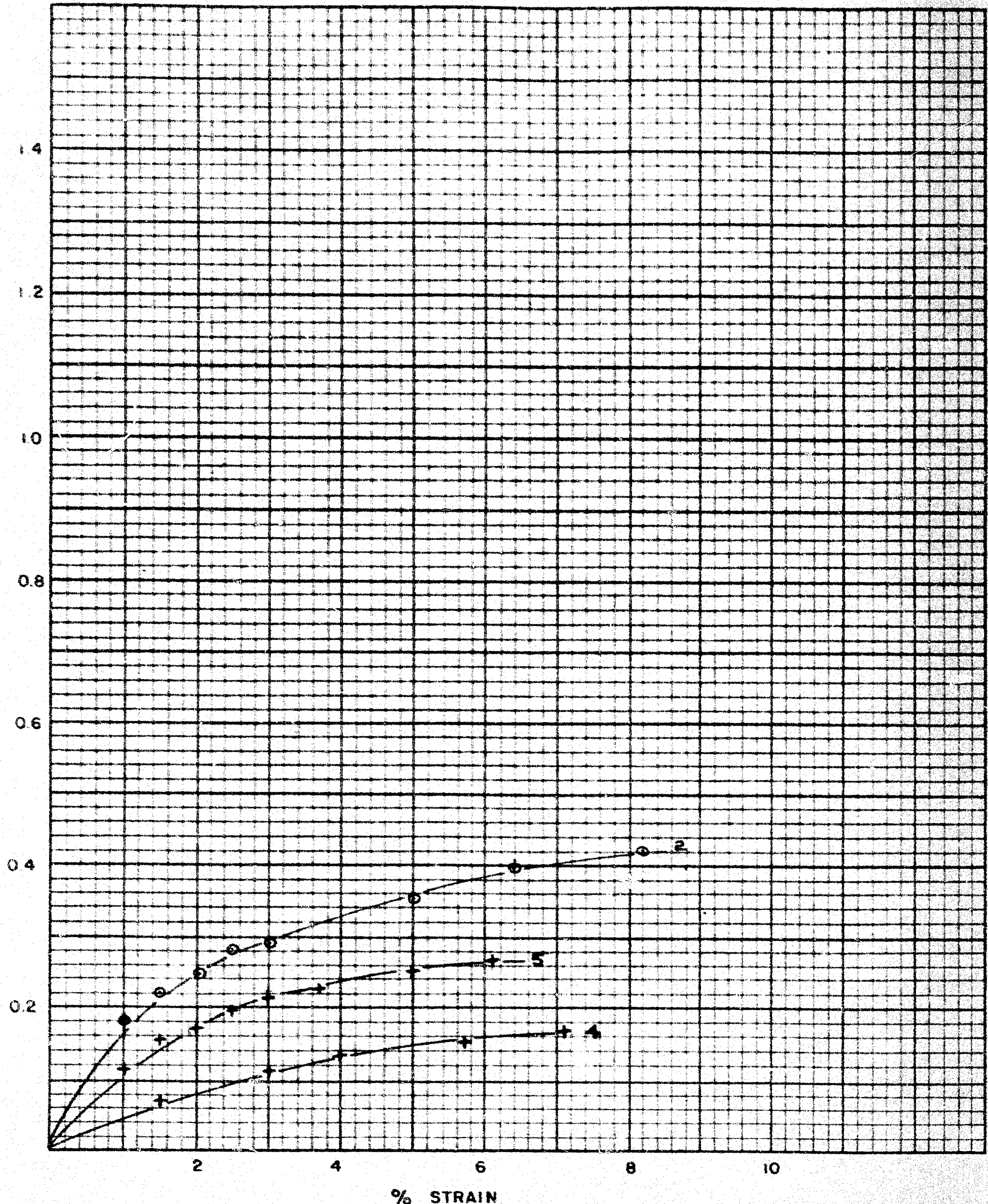
ASSOCIATED GEOTECHNICAL SERVICES
Limited

SOIL MECHANICS LABORATORY
STRENGTH TESTS

o QUICK TRIAXIAL

+ UNCONFINED

NOTE: SAMPLES APPARENTLY FROZEN



CLIENT: CITY OF CORNWALL

JOB NO.: 6335 LOCATION: M^c CONNELL AT C.N.R

BOREHOLE NUMBER: 2 DEPTH: 10.0'

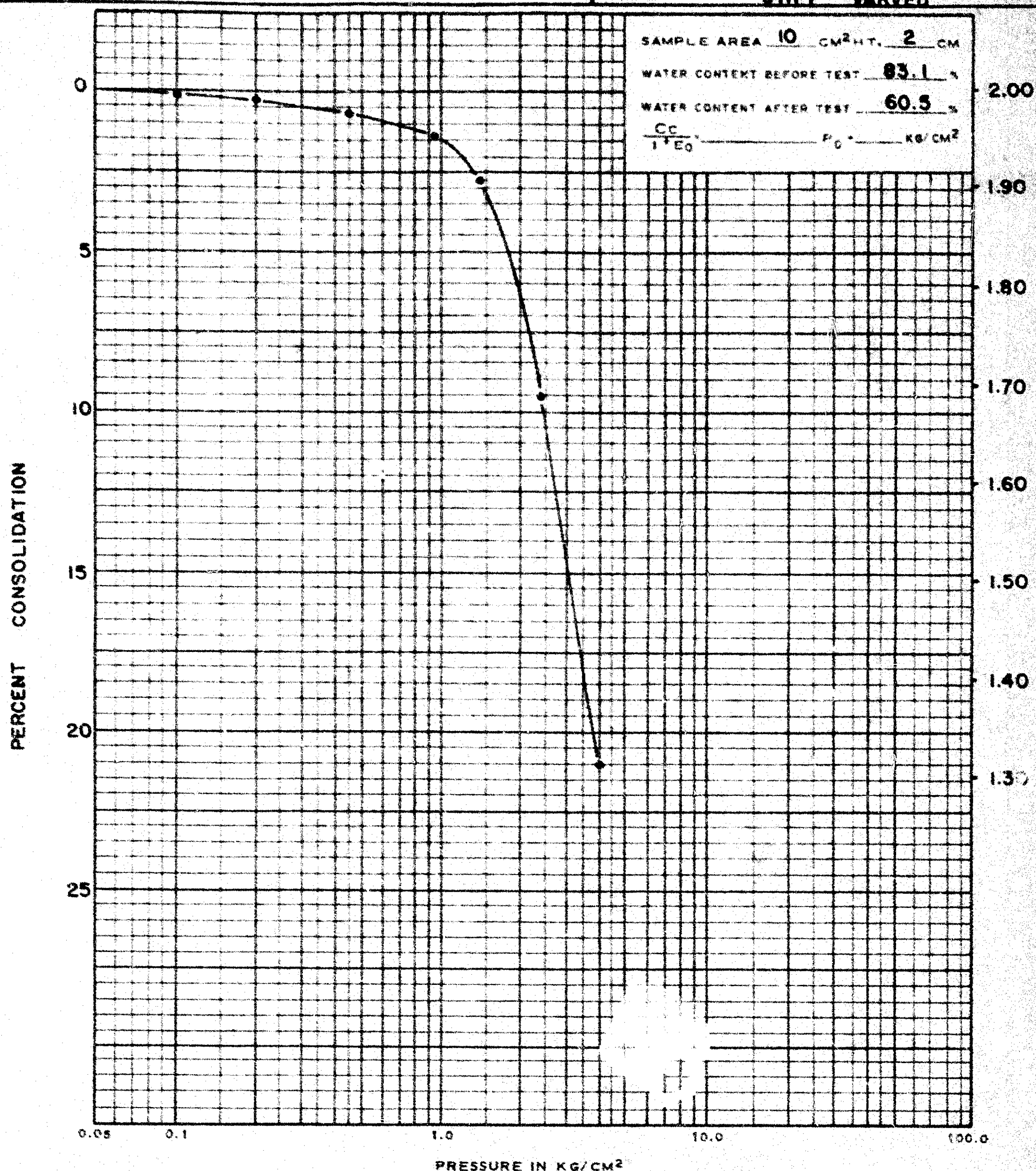
SAMPLE NUMBER: 2 DATE:

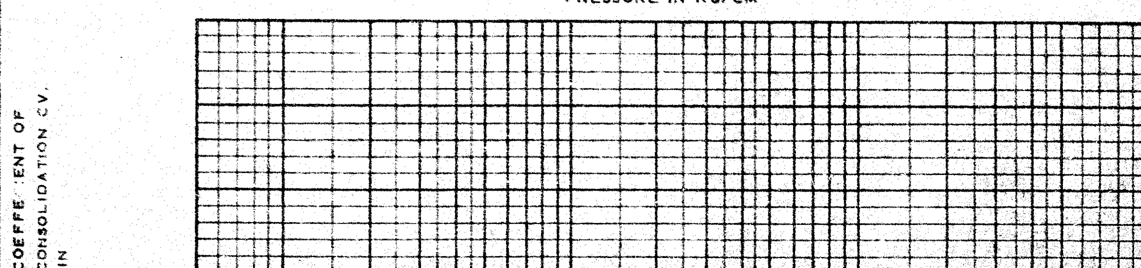
ASSOCIATED GEOTECHNICAL SERVICES Limited

SOIL MECHANICS LABORATORY

CONSOLIDATION TEST

STIFF VARVED





CLIENT CITY OF CORNWALL

JOB NO. 6335 LOCATION McCONNELL AT C.N.R.

BOREHOLE NUMBER 8 DEPTH 10.0'

SAMPLE NUMBER 2 DATE

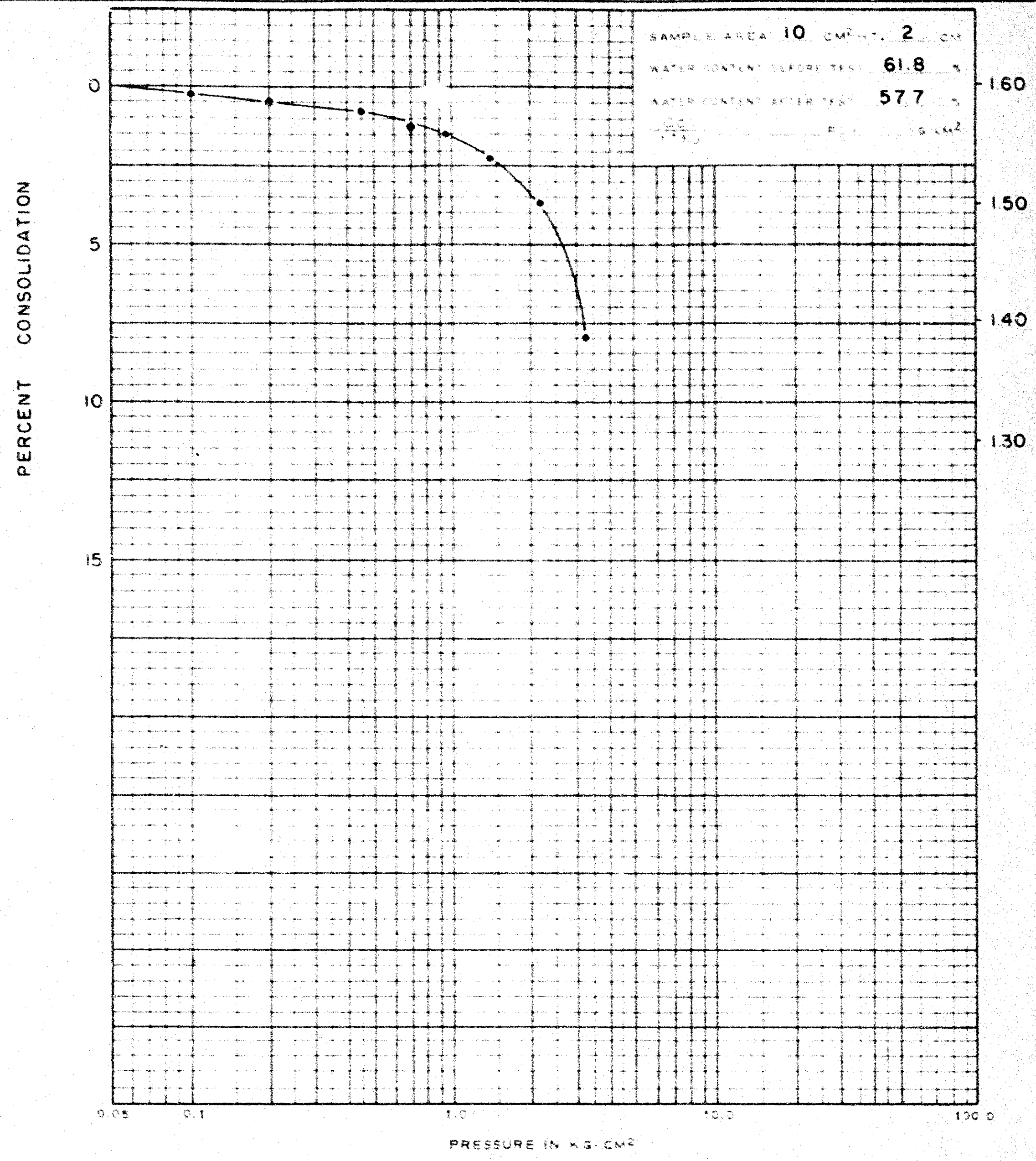
ASSOCIATED GEOTECHNICAL SERVICES

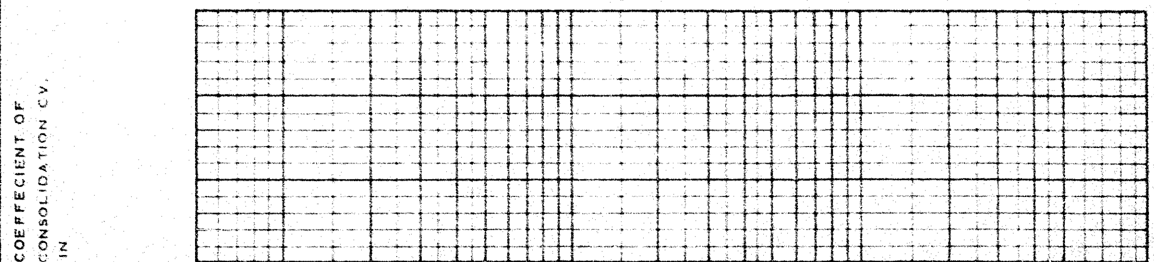
Limited

SOIL MECHANICS LABORATORY

CONSOLIDATION TEST

STIFF VARVED





CLIENT CITY OF CORNWALL

JOB NO. 6335 LOCATION McCONNELL AT C.N.R.

BOREHOLE NUMBER 9 DEPTH 10.0' – 11.5'

SAMPLE NUMBER 2 DATE

ASSOCIATED GEOTECHNICAL SERVICES Limited

SOIL MECHANICS LABORATORY

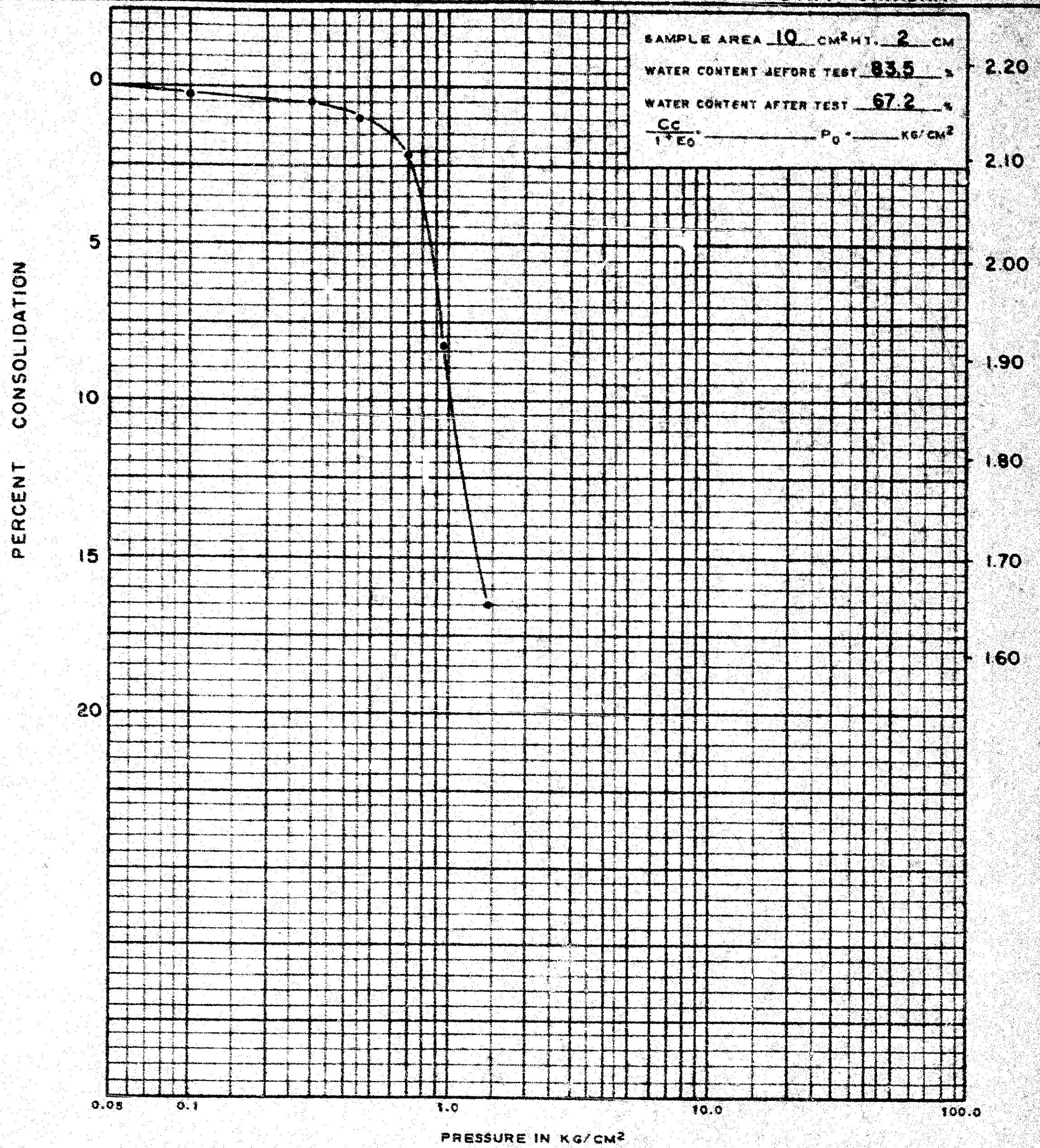
CONSOLIDATION TEST
STIFF UNIFORM

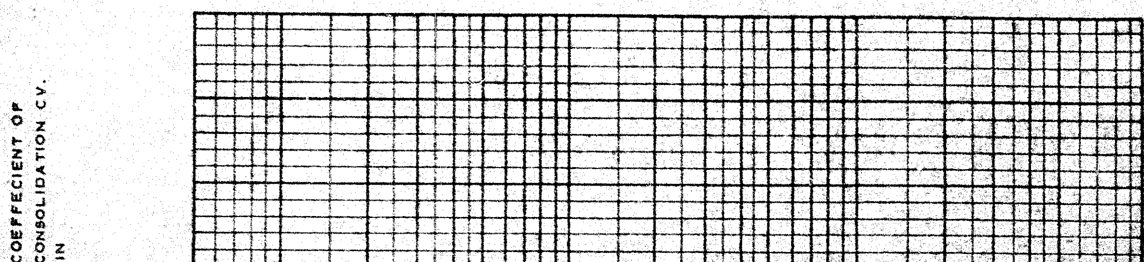
SAMPLE AREA 10 CM² HT. 2 CM

WATER CONTENT BEFORE TEST 83.5 %

WATER CONTENT AFTER TEST 67.2 %

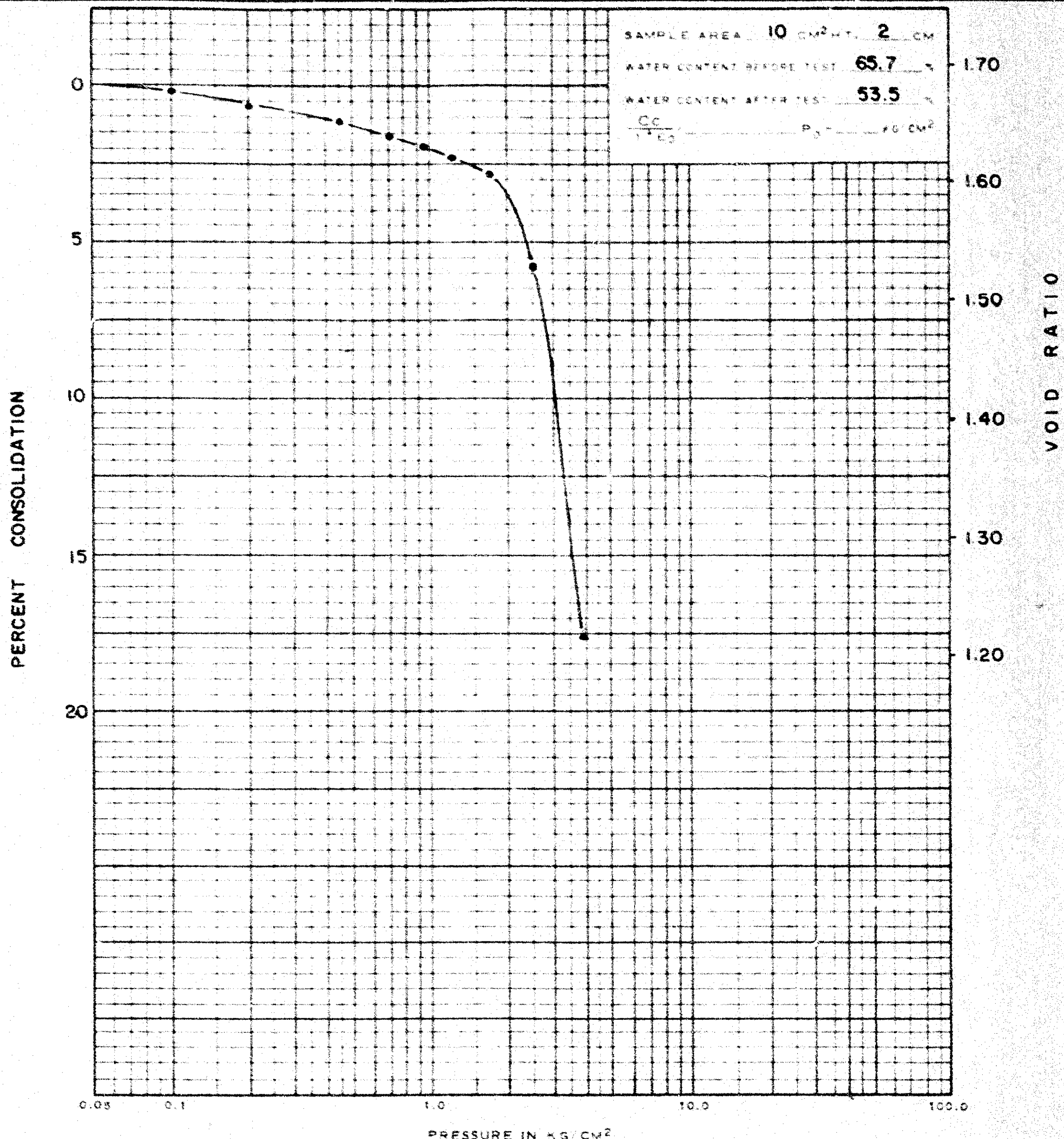
$\frac{Cc}{1+E_0}$ = ______ P_0 = ______ KG/CM²

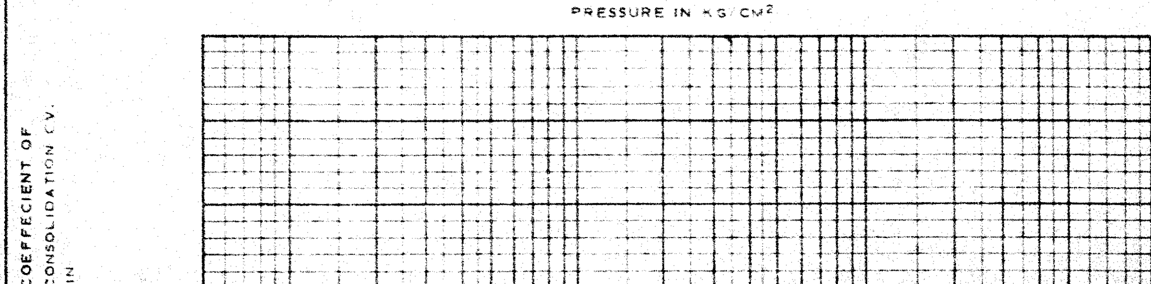




CLIENT: CITY OF CORNWALL	**ASSOCIATED GEOTECHNICAL SERVICES** Limited
JOB NO.: 6335 — LOCATION: M^c^CONNELL AT C.N.R.	
BOREHOLE NUMBER: 9 — DEPTH: 15.0' – 16.5'	**SOIL MECHANICS LABORATORY**
SAMPLE NUMBER: 3 — DATE:	CONSOLIDATION TEST **STIFF VARVED**

SAMPLE AREA 10 CM^2 HT. 2 CM
WATER CONTENT BEFORE TEST 65.7 %
WATER CONTENT AFTER TEST 53.5 %
$\frac{Cc}{1+e_0}$ .... P_0 = ... KG/CM^2





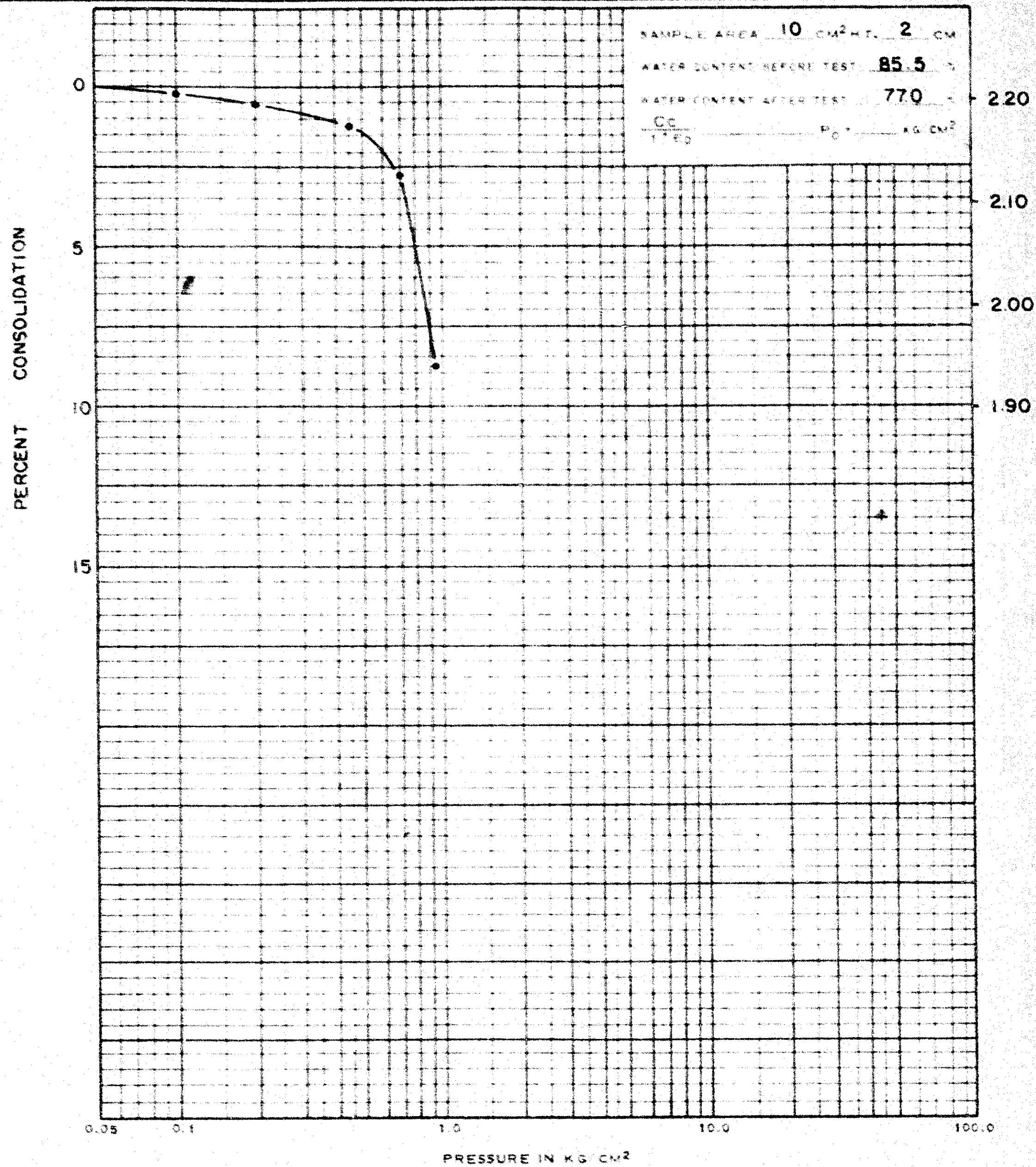
CLIENT CITY OF CORNWALL	**ASSOCIATED GEOTECHNICAL SERVICES** Limited
JOB NO. 6335 LOCATION M^{c}CONNELL AT C.N.R.	
BOREHOLE NUMBER 10 DEPTH 10.0' – 11.5'	**SOIL MECHANICS LABORATORY**
SAMPLE NUMBER 2 DATE	CONSOLIDATION TEST STIFF UNIFORM

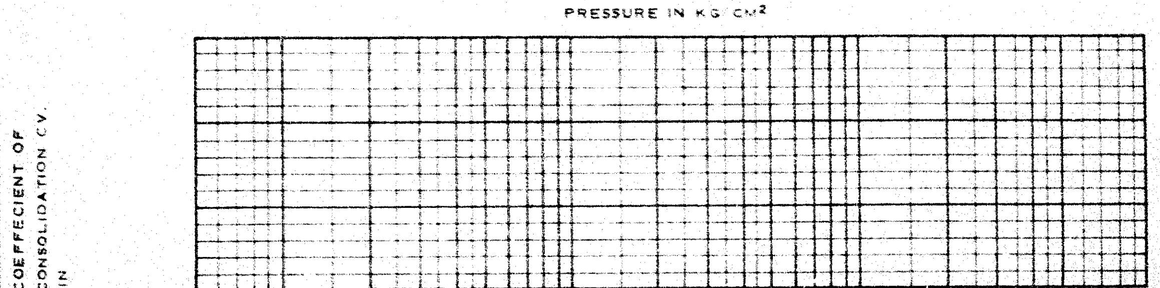
SAMPLE AREA 10 CM² HT. 2 CM

WATER CONTENT BEFORE TEST 85.5 %

WATER CONTENT AFTER TEST 77.0 %

Cc/1+Eo ______ Pc = ______ KG/CM²





CLIENT CITY OF CORNWALL

JOB NO. 6335 LOCATION M^C CONNELL AT C.N.R

BOREHOLE NUMBER 10 DEPTH 21.0'

SAMPLE NUMBER 4 DATE

ASSOCIATED GEOTECHNICAL SERVICES Limited

SOIL MECHANICS LABORATORY

CONSOLIDATION TEST

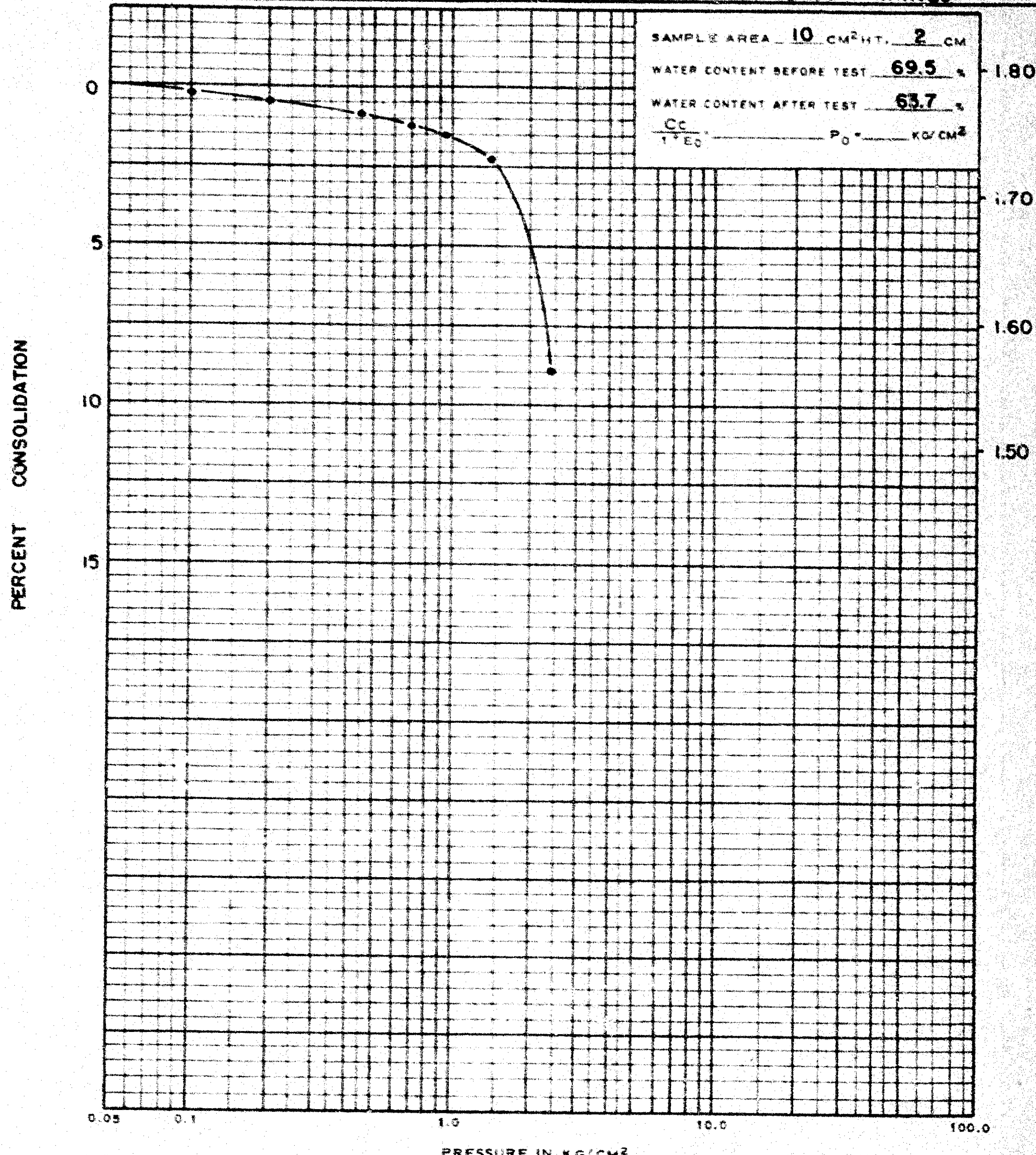
STIFF VARVED

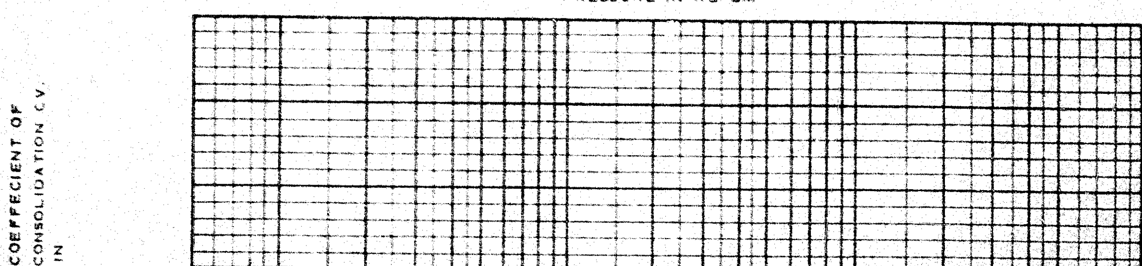
SAMPLE AREA 10 CM^2 HT. 2 CM

WATER CONTENT BEFORE TEST 69.5 %

WATER CONTENT AFTER TEST 63.7 %

$\frac{Cc}{1+E_0}$ = ____ P_0 = ____ KG/CM^2





SOIL CLASSIFICATION SYSTEM

The following system was used to describe the various soils encountered at the site as determined by visual field examination and test. It was also used to classify those soils upon which a laboratory grain size determination had been made.

Soil Components	Particle Size			
Clay	$<$ .002	mm		
Silt	$>$ .002	mm	$<$ .06	mm
Sand	$>$ .06	mm	$<$ 2.0	mm
Gravel	$>$ 2.0	mm	$<$ 2	in.
Cobbles	$>$ 2	in.	$<$ 6	in.
Boulders	$>$ 6	in.		

Descriptive Terms	Range of Proportions
and	greater than 40%
with	25% to 40%
some	10% to 25%
trace	less than 10%

Example

1. Silt (predominant type) with (25% - 40%) sand.
2. Sand and silt (predominant types), some (10% - 25%) gravel, trace ($<$ 10%) clay.

STANDARD PENETRATION CLASSIFICATION

Relative Density of Sands

as determined by Standard Penetration Tests

N	D_d	Designation on Borehole Log
0 - 4	0 - 0.2	Very Loose
4 - 10	0.2 - 0.4	Loose
10 - 30	0.4 - 0.6	Medium Dense
30 - 50	0.6 - 0.8	Dense
Over 50	0.8 - 1.0	Very dense

Shear Strengths of Clays

as determined by Standard Penetration Tests

N	s psf	Designation on Borehole Log
2	250	Very soft
2 - 4	250 - 500	Soft
4 - 8	500 - 1000	Medium
8 - 15	1000 - 2000	Stiff
15 - 30	2000 - 4000	Very Stiff
30	4000	Hard

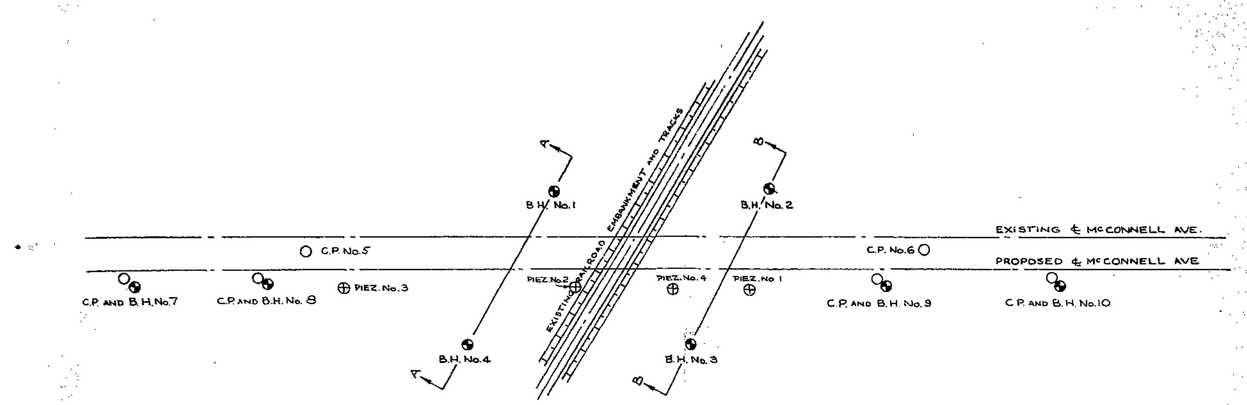
#64-F-283m

MC CONNELL

AVENUE

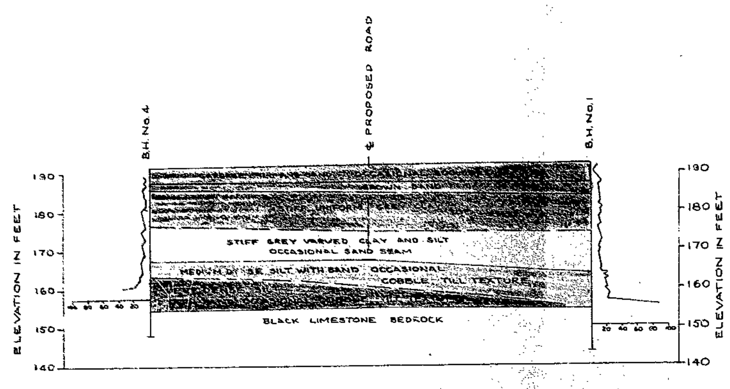
C.N.R. CROSSING

CORNWALL.



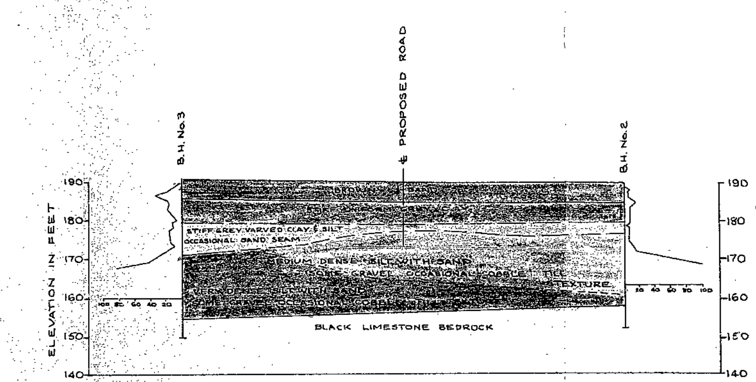
PLAN SHOWING LOCATION OF BOREHOLES AND CONE PROBES

SCALE - 1" TO 100'



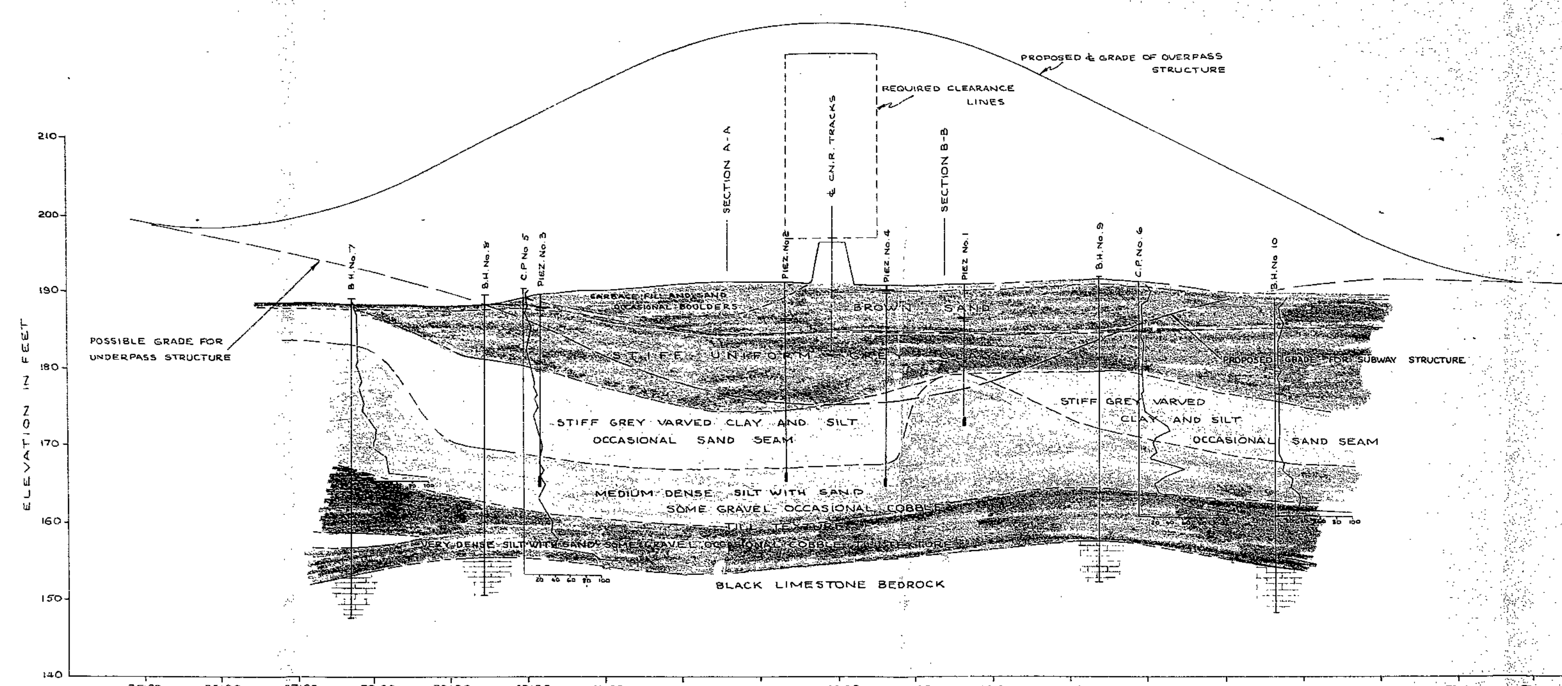
SECTION A-A

HORIZONTAL SCALE 1" TO 40'
VERTICAL SCALE 1" TO 20'



SECTION B-B

HORIZONTAL SCALE 1" TO 40'
VERTICAL SCALE 1" TO 20'



PROBABLE SOILS PROFILE ALONG ℄ PROPOSED ROAD

SCALE :- HORIZONTAL 1" TO 100'
VERTICAL 1" TO 10'

CITY OF CORNWALL

McCONNELL AVE AND C.N.R. CROSSING

PLAN, SECTION AND ℄ PROFILE

DWG. No. 1