#59-F-232 C
Hwy #81
AUSABLE R.
BRIDGE

ONTARIO

DEPARTMENT OF HIGHWAYS

Memo to Mr. A. M. Toye,
Bridge Engineer.

Date December 11, 1959.

Subject SOIL & STABILITY ANALYSIS by Geocon, Limited.

From Materials & Research Section.

Attention: Mr. S. McCombie.

Re: Ausa[illegible] River Bridge - Hwy. #81
Sta. 53+00 - Sta. 55+00
District #1 - Chatham, Ontario.

We are forwarding herewith, a copy of Geocon's report pertaining to the movements which have taken place at the above structure site.

Geocon's study has resolved that movements are due to either failure of the piles due to overload, or general slope instability. They have recommended that measurements of the hydrostatic pressures of the underlying soil types be made.

If piezometric levels are high, this will confirm that movements are the result of slope failure. Arrangements are being made for the piezometer units to be installed by the Materials and Research Section. Once these units are installed, observations will be carried out by the Regional Soils Engineer. We will advise you of these results when they are available.

Slope stability analyses will be carried out by our own Foundation Section to confirm the remedial measures required, and guarantee the stability of this structure slope.

If you have any queries regarding the contents of the Consultants' report, or our foregoing comments, please contact our office.

L. G. Soderman

L. G. Soderman,
PRINCIPAL SOILS & FOUNDATIONS ENGINEER

LGS/MdeF
Encl.
cc: Messrs. A. M. Toye (2)
H. A. Tregaskes
D. G. Ramsay
A. Gater
G. U. Howell
J. Roy
Foundation Section (2)
Gen. Files.

BA437A

GEOCON LTD

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Rexdale, Ontario,
November 20th, 1959.

Department of Highways, Ontario,
Parliament Buildings,
Toronto, Ontario.

W P. 179-57

Attention: Mr. A. Rutka, P. Eng.,
Materials and Research Engineer.

59-F-232

Re: Engineering Study,
Highway 81 Sta. 53+00 -
Sta. 55+00,
Ausable River Bridge.

Dear Sirs:

This letter accompanies our detailed report on an engineering study of observed movements at the existing Ausable River Bridge.

The study carried out included a detailed examination of the soil conditions at the site, available construction records, pile driving information and progressive readings of movement to date. It is considered that the cause of the movements of the south abutment of the bridge is either by overload of the back piles at the abutment or by instability of the south slope of the river induced by high piezometric pressures within the slope.

Measures to resolve one or other of the suggested mechanisms of movement are discussed. It is recommended that piezometers be installed in order to establish the present range of pore pressure.

We feel that this report will enable a definition of the integrity of the structure to be made once the piezometric observations are complete. We request that we be informed of the progress of these observations.

Yours very truly,
GEOCON LTD

M.A.J. Matich per V.

M. A. J. MATICH, P. Eng.,
Chief Engineer.

MAJM/dw
S6636

S6636

REPORT

TO

DEPARTMENT OF HIGHWAYS, ONTARIO

ON

ENGINEERING STUDY

HIGHWAY 81 STA. 53+00 - STA. 55+00

AUSABLE RIVER BRIDGE

Distribution:

10 copies - Department of Highways, Ontario,
Toronto, Ontario.

2 copies - Geocon Ltd,
Toronto, Ontario.

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INDEX

INTRODUCTION

Geocon Ltd has been retained by the Department of Highways, Ontario (proposal dated February 18th, 1958 and accepted by Official Work Order No. 7-5791, dated March 3rd, 1958) to carry out a soil investigation at the site of the bridge on Highway 81 where horizontal and vertical movements of the south abutment have been reported. The object of the investigation was to determine and interpret the soil conditions at the site, to make an analysis of the observed movement and their probable cause and to recommend remedial measures.

SUMMARIZED SOIL CONDITIONS

The site investigated is part of the south approach embankment which is up to 23 feet high and consists of a stiff brown clay fill obtained from local excavations. The present roadway and the previous roadway, which was encountered within the present embankment, consist of loose pit run sand and gravel. The embankment rests on a stratum of stiff grey and brown clay, about 40 to 60 feet in thickness, which is underlain by about 40 feet of very dense silty sand. The sand stratum is underlain by about 45 feet of clay which overlies bedrock.

DISCUSSION

General

The bridge, which was constructed in the period January 1956 to October 1957, is a three span continuous plate girder structure with piers and abutments founded on 12 inch diameter Monotube piles.

Movement of the partially completed structure was first suspected in June 1957 when the structural steelwork did not fit properly on the rocker seats at the south abutment. It is recorded that various

General (continued)

setting out stakes had occasionally suffered movement. Check measurements and levels were taken following the suspected movement, but no records of these measures are available prior to October 1957. At this time displacement of the completed road curbs and handrailing was definitely observed. This displacement became progressively worse and further settlements were recorded at the back ends of the wing walls of the south abutment. In February 1958 this settlement had been recorded as approximately $7\frac{1}{2}$ inches. It should be noted, however, that the initial three inches of settlement can only be inferred from the abutment elevations given on design drawing D3529-1. Various check observations have been made regularly since February 1958 and the details of these are presented graphically on Drawing S6636-3, together with an illustration of the overall pattern of movement.

The general site and specific points of movement are shown on Figures 1, 2, and 3 in Appendix III.

The basic programme of construction at the south abutment has been compiled from existing field books and records. Pertinent notes and observations have been summarized and are tabulated in Appendix IV. Where the records conflict this is noted in the Appendix. The assumed general schedule of construction is shown graphically on Drawing S6636-3A.

From the recorded observations given in Appendix IV and on the drawing, it is considered important to summarize what are known to be concrete facts concerning possible movement. It is definitely established that:

<u>DISCUSSION</u> (continued)

<u>General</u> (continued)

1. A gap about 6 inches wide has occurred between the road surface on the south abutment and the concrete at the south-east corner of the deck - Figures 2 and 3, Appendix III.

2. A movement of apparent closure between the south abutment and the south-west corner of the deck has also taken place - Figure 3, Appendix III.

3. Settlement has taken place of the south ends of the wing walls on both sides of the south abutment.

   This settlement until May 1958 was definitely not less than 5 inches and may have been slightly greater than 8 inches. In the period May to August, 1958, no settlement took place - S6636-3F.

   Minor settlement has been observed at the north end of the north abutment.

4. The batter of the front face of the south abutment now exceeds that given on design drawing D3529-1 by approximately 11 inches, indicating a possible backward tilt of the top of the abutment. The batter at the north abutment has correspondingly increased about 1 inch.

DISCUSSION (continued)

General (continued)

5. The ballast wall at the back of the south abutment was re-constructed some 6 inches from the design position to enable the steelwork to be fitted into position. - Appendix IV.

Observations which cannot be definitely established are:

6. A reported horizontal movement northwards of the partially constructed south abutment. This is variously given as $4\frac{1}{2}$ to 7 inches - Appendix IV.

7. The measured increase in elevation of the whole south abutment and concrete deck since August 1958, with erratic variations in individual readings - S6636-3C and F.

8. The measured minor variations in the lateral distance between the south abutment and south pier - S6636-3E.

The significance of the points tabulated above in relation to analyses carried out to determine the possible reasons for movement and to establish the integrity of the structure, is discussed under separate headings below.

DISCUSSION (continued)

Piled Foundations

The piers and abutments are founded on concrete filled Monotube pipe piles, 12 inches diameter at the top and tapering to 8 inches diameter at the tip. The details of the piles at the south abutment are shown on Drawing S6636-4, together with pile driving data, compiled from construction records.

It may be noted that piles numbered 3 to 6, 10 to 12, and 15 to 23, although driven in the period March 16th to 23rd, 1956, were not concreted until over a month had passed. At that time the piles were filled with concrete in the same pour as the abutment footing. These piles are all located in the western corner of the abutment or at the area of apparent closure of points 5 and 6, Drawing S6636-3D. Since the piles are bonded approximately 2 feet into the abutment footing, it seemed possible that lateral movement of the abutment could have been caused by structural deformation of imperfectly filled piles, thus inducing a partial bond failure in the local area between the piles and the footing. Consequently an inspection was carried out at the pile/abutment base interface from test pits dug along the front face of the abutment. No signs of bond or concrete shear failure could be detected and the pile tops examined were in sound condition.

It may further be noted that the back vertical piles in the wings of the south abutment represent the shortest piles driven. These piles are numbered 1 to 3, 7 to 9, 13, 14, 49, 53, 54, and 58 to 60. If no local bond failure occurs, it can be assumed that settlement of the back of the abutment must be accompanied by corresponding settlement of the back piles. It is therefore of interest to examine the range of probable load on the piles.

Piled Foundations (continued)

It is understood that the design pile loads were computed to be of the order of 35 tons, based on an equivalent fluid earth pressure of 30 pounds per cubic foot and on the assumption that the pile group would act as an equivalent beam section in relation to applied vertical loads and moments. It is considered that this method of computation contains several inherent errors. Normal earth pressure theory indicates that the pressure behind abutments induced by saturated clay backfill may be of the order of 90 rather than 30 pounds per cubic foot. This is particularly the case when the backfill is jetted in place to induce compaction. It is further known that the principle of superposition when applied to computing loads on a pile group containing batter piles often gives an underestimation of individual pile loads of the order of 20 to 30 per cent. It is suggested that the Westergaard elastic centre method of determining pile load is more correct. The pile loads were re-computed, taking into account the factors discussed above, and also making allowance for additional vertical load on the back piles due to drag of the fill on the back of the abutment. The computations show that pile loads could be of the order of 50 tons or greater.

The probable ultimate capacity, in relation to the range of loads imposed, was examined. Computations of ultimate capacity, based on the driving resistance and using the Hiley formula, give an ultimate value of about 150 tons, thus indicating a possible factor of safety of 3. However, it is generally considered that dynamic pile driving formulae may be grossly in error, consequently the capacity has been computed on the basis of the soil strength parameters. It may be seen that the piles mobilize resistance through point resistance in a silt or silty sand stratum and/or skin friction in

Piled Foundations (continued)

the clay stratum. It is possible that the back piles did not fully penetrate the silt stratum. As discussed in Appendix V, the ultimate capacity of the piles is computed to be of the order of 100 tons, based on the average "N" value of 70 blows per foot, measured in borehole 5 at the location of the abutment.

The load on the back piles of the abutment could be increased above the computed value of 50+ tons by negative skin friction induced by consolidation of the clay under the weight of approach fill. Consolidation of the clay is computed to be about 2 to $2\frac{1}{2}$ inches. The time rate for 90 per cent consolidation, assuming double drainage of the stratum, is about 6 years: The time rate for 90 per cent consolidation, assuming three dimensional consolidation effects, is about 1 year. In relation to this consolidation settlement, it is pertinent to note that the time interval between placing the backfill and the first observed settlement is approximately 1 year. Negative skin friction within the clay is computed to increase the load on the back piles to approximately the ultimate capacity of the individual piles. This is detailed in Appendix V. Once negative skin friction is fully developed, rapid settlement of the pile tip into the silt stratum would result due to bearing capacity deformation. This settlement would be of the same order as the computed consolidation effect.

The ratio of the distances of the ends of the wing walls and the centre of the back piles to the front face of the abutment is about 3 to 1. Consequently, a settlement of the back piles of about $2\frac{1}{2}$ inches would result in a corresponding settlement of the ends of the wing walls of $7\frac{1}{2}$ inches. This checks approximately with the detailed observations

Piled Foundations (continued)

recorded. It is therefore considered that the mechanism discussed above is a possible cause of the abutment movement.

Stability

The stability of the approach embankment and river slope were examined.

Total stress analyses, based on the average undrained shear strength of the clay of 1600 pounds per square foot, gave a computed range in factor of safety of 1.6 to 2.0. No allowance was made in these analyses for piezometric pressures within the slope.

As seepage was observed beneath the south abutment in local areas, a sliding block analysis was made along the sand and gravel stratum shown in Section A-A, S6636-2. Assuming a piezometric level within this sand and gravel equivalent to 4 feet head of water, a factor of safety of 1.1 is computed. However, it is considered that this analysis form is included within the earth pressure effects computed above.

The most rational form of analysis for the conditions at the site is considered to be one based on effective stress parameters. The piezometric effects noted in boreholes and discussed in Appendix I were taken into account. It is suggested that the artesian effects recorded may be due to two possible causes:

(a) Artesian drainage of the silty sand stratum from hill "A" shown on Figure 2, Appendix III: this would induce a relatively horizontal piezometric elevation across the site.

Stability (continued)

(b) Piezometric pressures within the slope developed through variations in river level: this variation is known to exceed 20 feet.

Considering assumption (a) and using the average clay strength parameters c' = 150 pounds per square foot and Ø' = 22 degrees, as shown in Figure 6, Appendix II, effective stress analyses were carried out. These results are summarized on Drawing S6636-5. It may be seen that, provided piezometric surface was about elevation 688, the factor of safety immediately after placing the backfill was less than 1. Increased stability would only take place with a reduction in elevation of the piezometric surface. The variation in factor of safety with piezometric surface for both the excavated and completed slope is shown.

Considering assumption (b) the most radical change in piezometric pressures within the slope, would take place following a rapid drawdown of river level. This drawdown condition could occur immediately following spring flooding, as for example in March 1956. Piezometric surface was assumed to coincide with the face of the slope below HWL, taking $\bar{B}$ as 1. On this assumption, the factor of safety for a drawdown from HWL of 694 to LWL of 672.5 is computed to be 1.1.

It may be seen from the stability computations that the overall stability of the south abutment is critical for high piezometric pressures within the slope. It is therefore recommended that piezometers be installed to measure the variation in piezometric pressure at depth over a period of from 6 months to 1 year. Readings should be taken at regular intervals within this period of both rainfall and river level in order

Stability (continued)

to establish the validity of assumptions (a) or (b) above.

Piezometric readings should be taken in sets of three piezometers installed at each front corner of the south abutment and close to the toe of the existing slope. The suggested tip elevations for each set of three piezometers is shown on Drawing S6636-2.

CONCLUSIONS AND RECOMMENDATIONS

1. The existing south abutment and approach fill are underlain by a stratum of stiff grey and brown clay about 40 to 60 feet in thickness. This stratum overlies a thin stratum of silt, then about 40 feet of dense to very dense silty sand.

2. River level generally varies between about elevation 672 and 673. The reported high water level is at elevation 694.

3. Artesian effects were observed within the silty sand stratum in March 1955 prior to construction of the bridge. The general piezometric surface was at about elevation 688 or some 15 feet above present river level.

4. The cause of the observed movements of the south abutment are discussed in detail. It is concluded that the probable mechanisms of movements is EITHER by overload and consequent tip deformation of the back piles at the abutment accompanied by possible movement of structurally imperfect piles at the western corner OR instability of the south slope of the river induced by high piezometric pressures within the slope.

5. It is recommended that piezometers be installed to determine the degree of piezometric pressure within the clay and silt strata. The suggested locations of the piezometers are given in the report.

6. If it is established that the piezometric pressures are below the critical range, discussed in the report, it is then suggested that the deformation of the south abutment has been caused by initial pile overload and that the overall safety of the structure is adequate.

PERSONNEL

The field work was carried out under the supervision of Mr. A. Prior. The report was written by Mr. V. Milligan and reviewed by Mr. M.A.J. Matich. During the course of the work consultations were held with Dr. H.Q. Golder, Consultant.

REGISTERED PROFESSIONAL ENGINEER V. MILLIGAN PROVINCE OF ONTARIO

V. Milligan

V. Milligan, P. Eng.

VM/dw
S6636

APPENDIX I

Procedure

Site and Geology

Soil Conditions

Water Conditions

Office Reports on Soil Exploration

GEOCON

PROCEDURE

The field work was commenced on February 20th, 1958 and completed on February 27th, 1958. Two detailed exploratory boreholes with adjacent dynamic penetration tests were put down, using a skid-mounted machine drillrig. These boreholes were supplemented by four shallow hand auger borings. The locations of these borings and the inferred soil stratigraphy are shown on Drawing S6636-1 located in the pocket at the rear of this report. This drawing also shows the location of two of the borings the Department of Highways put down in a previous investigation in 1955. The information derived from these boreholes has been used to obtain the soil stratigraphy shown. A detailed log for each of the boreholes put down in this investigation is given on the Office Reports on Soil Exploration in this Appendix.

The soil testing was carried out in the Toronto Soil Mechanics Laboratory of Geocon Ltd and the results are plotted on the Office Reports and on the Figures in Appendix II. The soil samples remaining after testing will be stored until March, 1960 and will then be destroyed unless other instructions are received.

After completion of the field work, piezometers were installed in boreholes 1 and 2 and water level readings were taken at regular intervals by the Department of Highways, who also supplied the elevations. These elevations are referred to Geodetic Datum and were obtained from local D.H.O. bench marks.

SITE AND GEOLOGY

The site is located on the southern bank of the Ausable River on Highway 81, about 10 miles south of Parkhill, Ontario.

SITE AND GEOLOGY (continued)

From available geological information and from previous work in the area by the Department of Highways, it is known that the site is part of a groundmoraine clay plain which has been formed in a valley previously eroded in a basal till. The clay is underlain by sedimentary rocks of Devonian age.

SOIL CONDITIONS

The soil strata encountered by the borings are as follows:

Clay Fill

The embankment consists of brown clay fill. This fill has been obtained from local excavations and contains organic material and gravel throughout. The colour of the fill and the occasional occurrence of sand pockets suggest that it has been obtained from the weathered and oxidized crust of the surrounding clay plain.

A number of unconfined compression tests carried out on samples of the stratum gave an average unconfined shear strength of about 0.6 tons per square foot. The average wet unit weight obtained was 130 pounds per cubic foot at an average natural moisture content of 22 per cent. The stress-strain curve obtained for the unconfined compression tests are shown on Figure 1 of Appendix II.

Sand and Gravel Fill

The clay fill embankment is covered by about 3 feet of brown sand and gravel fill. This type of fill was also encountered within the embankment where it represents the roadway of the former highway and it is known to exist behind and over the full height of the abutment where it has been used to backfill the

Sand and Gravel Fill (continued)

abutment excavation to grade. The fill consists of fine gravel and sand in all grainsizes. It has a fairly high silt content as indicated by the grainsize distribution curve in Figure 2 of Appendix II.

The results of dynamic and standard penetration tests indicate that the sand and gravel fill is of loose relative density.

Varved Clay

Underlying the embankment is a stratum of varved clay about 60 feet in thickness. Dispersed throughout the clay subrounded gravel was encountered which was usually about 1/8 inch in size, although occasionally sizes of up to 2 inches were encountered.

Above about elevation 675 the clay is grey in colour and the individual varves, consisting of silty clay and silty layers, are so closely spaced that this part of the stratum gives the impression of a very finely horizontally laminated clay.

At about elevation 675 there is an abrupt change in structure. From elevation 675 to elevation 660, the varves are alternately composed of brown silty clay and grey-brown silt layers and grey silty clay and grey-brown silt layers. The varve spacing here is irregular between 1/16 and $1\frac{1}{2}$ inches. The upper few feet of this part of the stratum contain many silt pockets, 1/16 to 2 inches in size.

Below about elevation 660, the varve structure as described in the previous paragraph gradually becomes less distinct. The

Varved Clay (continued)

clay becomes grey-brown to grey in colour and varve spacing is so close, that again the clay gives the impression of a very finely horizontally laminated clay.

The results of the laboratory testing carried out on samples of the stratum are summarized below:

	Min.	Max.	Mean
Natural Moisture Content (per cent)	18	32	20
Wet Unit Weight (pounds per cubic foot)	124	137	135
Liquid Limit	25	43	29 *
Plasticity Index	10	22	17 *
Unconfined Shear Strength (pounds per square foot)	1200	4000	1600
Apparent Angle of Internal Friction from Consolidated Quick Triaxial Tests (degrees)			19
Effective Angle of Internal Friction from Slow Drained Triaxial Tests (degrees)			22
Effective cohesion (pounds per square foot)			150
Compression index C_c	0.11	0.2	0.16

* Including results in D.H.O. report, March 1955

Varved Clay (continued)

The results of the Atterberg limit tests indicate that the clay is inorganic and of low plasticity.

Typical stress-strain curves obtained from the unconfined compression tests are given on Figure 3 of Appendix II. The mean strain at failure for the triaxial tests was about 4 per cent.

A number of consolidated quick triaxial tests were carried out and the resulting Mohr stress circles are given on Figure 4. The apparent angle of shearing resistance obtained for the failure envelope is 19 degrees at an apparent cohesion of 1400 pounds per square foot.

Two series of slow drained triaxial tests were carried out. The initial series of tests is plotted on Figure 5, Appendix II. This series is combined with the following series on Figure 6, Appendix II. It is significant to note that an almost constant failure envelope to both series of tests is obtained, giving strength parameters C' of 150 pounds per square foot and Ø' of 22 degrees. The divergence from this relationship by 2 of the 7 tests is considered due to the radical variation in index properties of these samples. This variation is noted on the Figures.

Four consolidation tests were carried out on selected samples at regular intervals throughout the depth of the clay stratum. The resulting void ratio-log pressure curves are given on Figures 7 to 10 inclusive.

Varved Clay (continued)

The permeability 'k' of the clay, as computed from the results of the drained triaxial and consolidation tests, is about 5×10^{-8} centimeters per second.

Silty Sand

A stratum of grey silty sand underlies the clay. Borehole 1 was penetrated about 6 feet into this stratum, but from previous work by the Department of Highways, it is known that this stratum is about 40 feet in thickness. Standard penetration tests in excess of 100 blows per foot in this stratum indicate that the silty sand is extremely dense.

From previous work by the Department of Highways, it is known that the sand is underlain by about 45 feet of clay, having properties very similar to the overlying clay, underlain by bedrock.

WATER CONDITIONS

During the course of the Department of Highways initial site investigation in March, 1955, excess artesian effects were observed at depth within the lower silty sand stratum. The inferred piezometric levels from the available records made during drilling are summarized below:

WATER CONDITIONS (continued)

Borehole	Ground Level	Piezometric Level	Remarks
D.H.O.:1	677	680+	Observed below elevation 626
D.H.O.:2	682	682+	Observed below elevation 625
D.H.O.:4	669	688	Observed below elevation 623
D.H.O.:5	690	689	After completion of borehole to elevation 565

Boreholes 1 and 2 were on the north side of the river, boreholes 4 and 5 on the south side. River water level was at elevation 673.5 to 674 in March 1955.

It is significant to note that the highest piezometric levels were recorded on the south side of the river, adjacent to the present bridge abutment; consequently, during the course of this investigation in February 1958, particular attention was directed to the observation of possible artesian pressure at depth. No excess water pressures above river level were detected above elevation 628 during the boring and sampling period. Piezometers were then installed in boreholes 1 and 2 within the varved clay stratum at about elevations 660 and 672 respectively. The readings taken in the piezometers are shown on Drawing S6636-3. It is considered that these readings have been possibly influenced by groundwater seepage from the upper sand and gravel fill. The lowest piezometric level recorded in borehole 1 was at about elevation 700.

River water level was at elevation 672.5 in February 1958.

EXPLANATION OF THE FORM "OFFICE REPORT ON SOIL EXPLORATION"

The object of this form is to enable a comprehensive study of the soil to be made by combining on one sheet all of the information obtained from the boring. An explanation of the various columns of the report follows.

ELEVATION AND DEPTH

This column gives the elevation and depth of boundaries between the various soil strata. The elevation is referred to the datum shown in the general heading.

WATER CONDITIONS

In this column the water level in the casing at the time of boring or the water table in the ground, determined by a series of observations in a piezometer or standpipe, is indicated to scale by a horizontal line with the symbol W.L. or W.T. above the line. A notation of any complicated groundwater conditions will be made in this column.

DESCRIPTION

A description of the soil, using standard terminology, is contained in this column. The consistency of cohesive soils and the relative density of non-cohesive soils are described by the following terms:

Consistency	U-Strength Tons/sq. ft.	Relative Density	Standard Penetration Resistance. Blows/ft.
Very soft	0.03 to 0.25	Very loose	0 to 4
Soft	0.25 to 0.5	Loose	4 to 10
Firm	0.5 to 1.0	Compact	10 to 30
Stiff	1.0 to 2.0	Dense	30 to 50
Very stiff	2.0 to 4.0	Very dense	over 50
Hard	over 4.0		

STRATIGRAPHIC PLOT

The stratigraphic plot follows the standard symbols of the National Research Council, Canada.

ELEVATION SCALE

The information in all columns is plotted to a true elevation scale which is shown in this column.

GRAPHS

The main body of the report forms a graph which is used to plot to correct elevation the important soil properties which are obtained through field and laboratory tests. The scales and symbols for the plotting are shown at the head of the column.

OTHER TESTS

In this column are shown, by symbol, the other field or laboratory tests which have been performed on the soil and for which the results have not been plotted on the above graph.

SAMPLES

The first three columns describe the condition, type and number of each sample obtained from the boring. The location and extent of each sample is plotted to scale.

In the last column is shown the penetration resistance in blows of 4200 inch-pounds required to drive one foot of the sampler into the ground. When a 2 inch Drive Sampler is used the result obtained is termed the "Standard Penetration Resistance".

GEOCON

OFFICE REPORT ON SOIL EXPLORATION

CONTRACT S6636 BORING # 1 DATUM GEODETIC CASING HX
BORING DATE FEB. 20, 1958 REPORT DATE FEB. 28, 1958 COMPILED BY J.A. & M.V. CHECKED BY [illegible]
SAMPLER HAMMER WT. 140 LBS. DROP 30 INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN - LBS. ENERGY)

SAMPLE CONDITION

- DISTURBED
- FAIR
- GOOD
- LOST

SAMPLE TYPES

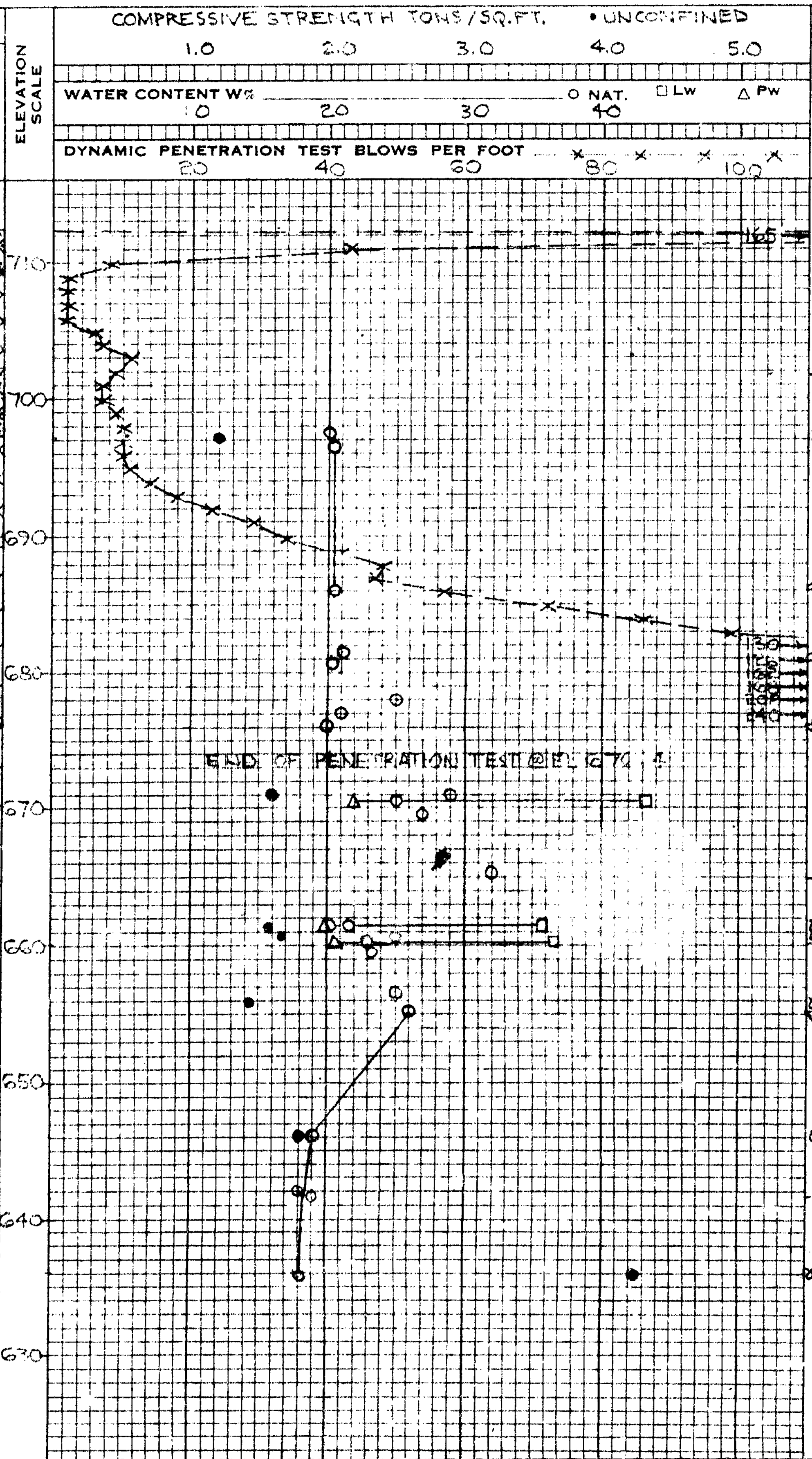
- A.S. - AUGER SAMPLE
- S.T. - SLOTTED TUBE
- W.S. - WASHED SAMPLE
- D.O. - DRIVE-OPEN
- D.F. - DRIVE-FOOT VALVE
- C.S. - CHUNK SAMPLE
- F.S. - FOIL SAMPLE
- S.O. - SLEEVE-OPEN
- S.F. - SLEEVE-FOOT VALVE
- T.O. - THIN WALLED OPEN
- R.C. - ROCK CORE

ABBREVIATIONS

- V - IN-SITU VANE TEST
- M - MECHANICAL ANALYSIS
- U - UNCONFINED COMPRESSION
- Qc - TRIAXIAL CONSOLIDATED QUICK
- Q - TRIAXIAL QUICK
- S - TRIAXIAL SLOW
- γ - WET UNIT WEIGHT
- K - PERMEABILITY
- C - CONSOLIDATION
- WL - WATER LEVEL IN CASING
- WT - WATER TABLE IN SOIL

SOIL PROFILE

ELEVN. / DEPTH	WATER CONDITIONS	DESCRIPTION
712.4		GROUND LEVEL
0.0		LOOSE BROWN SAND AND GRAVEL FILL
709.4 / 3.0		STIFF BROWN CLAY FILL
702.4 / 10.0		LOOSE BROWN SAND AND GRAVEL FILL
697.4 / 15.0		STIFF BROWN CLAY FILL
689.4 / 23.0		STIFF GREY VARVED SILTY CLAY
676.4 / 36.0		STIFF GREY AND BROWN BECOMING GREY VARVED SILTY CLAY
634.4 / 78.0		VERY DENSE SILT
630.0		
627.9 / 84.5		END OF HOLE
		VERY DENSE GREY SILTY FINE SAND



SAMPLES

OTHER TESTS	TYPE	NUMBER	PENETRATION RESISTANCE BLOWS/FT.
	2"SO	1	6
M	SO	2	9
γ=135	SO	3	9
	SO	4	11
C Qc γ=135	3"SO	5	29
Qc Qc	SO	6	39
Qc C	SO	7	36
	SO	8	28
C	SO	9	45
γ=128 Qc	SO	10	50
γ=129 Qc C	SO	11	66
	SO	12	88
	W.S.	13	—
γ=133	2"SO	14	33
Qc γ=136	SO	15	42
γ=136	SO	16	49
	SO	17	>100
	SO	18	>100

GEOCON

OFFICE REPORT ON SOIL EXPLORATION

CONTRACT S 5236 BORING # 2 DATUM GEODETIC CASING BX

BORING DATE FEB. 25, 19[illegible] REPORT DATE FEB. 29, 19[illegible] COMPILED BY J.A. & M.W. CHECKED BY [illegible]

SAMPLER HAMMER WT. 140 LBS. DROP 30 INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN - LBS. ENERGY)

SAMPLE CONDITION

- DISTURBED
- FAIR
- GOOD
- LOST

SAMPLE TYPES

- A.S. - AUGER SAMPLE
- S.T. - SLOTTED TUBE
- W.S. - WASHED SAMPLE
- D.O. - DRIVE-OPEN
- D.F. - DRIVE-FOOT VALVE
- C.S. - CHUNK SAMPLE
- F.S. - FOIL SAMPLE
- S.O. - SLEEVE-OPEN
- S.F. - SLEEVE-FOOT VALVE
- T.O. - THIN WALLED OPEN
- R.C. - ROCK CORE

ABBREVIATIONS

- V - IN-SITU VANE TEST
- M - MECHANICAL ANALYSIS
- U - UNCONFINED COMPRESSION
- QC - TRIAXIAL CONSOLIDATED QUICK
- Q - TRIAXIAL QUICK
- S - TRIAXIAL SLOW
- γ - WET UNIT WEIGHT
- K - PERMEABILITY
- C - CONSOLIDATION
- WL - WATER LEVEL IN CASING
- WT - WATER TABLE IN SOIL

SOIL PROFILE

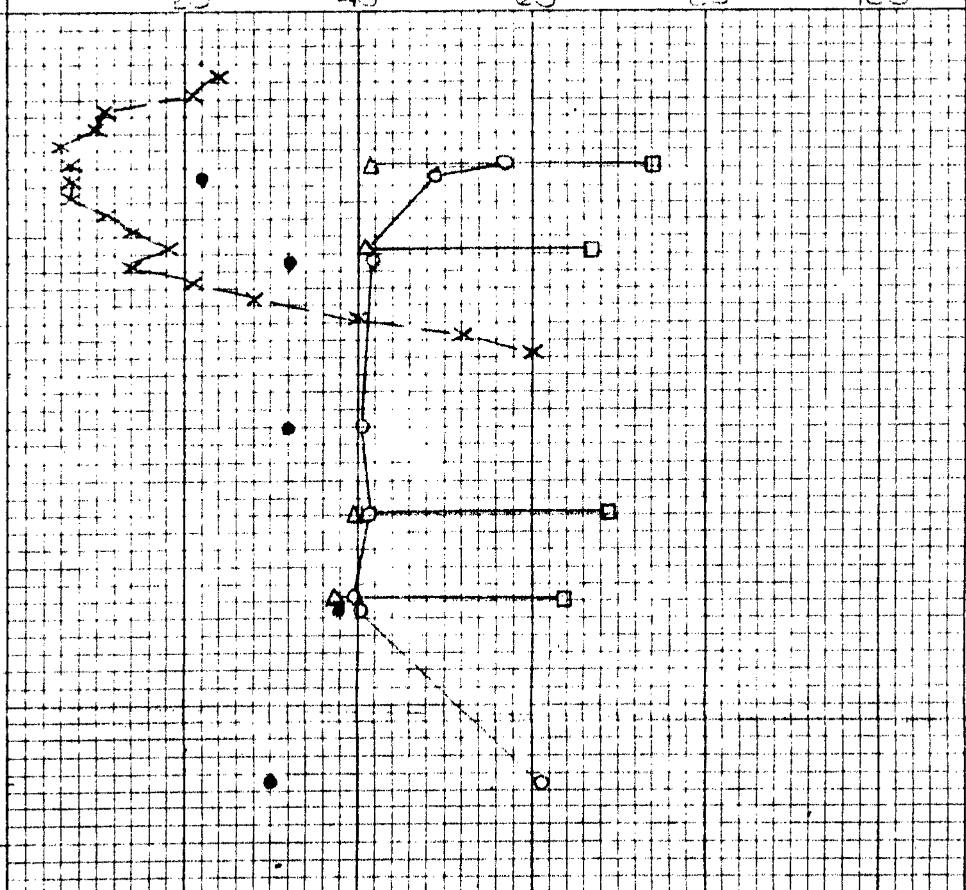
ELEVN. / DEPTH	WATER CONDITIONS	DESCRIPTION
714.8 / 0.0		GROUND LEVEL
		STIFF BROWN CLAY FILL
700.8 / 14.0		
		STIFF GREY VARVED SILTY CLAY
673.5 / 41.3		STIFF GREY AND BROWN VARVED SILTY CLAY
669.8 / 45.0		END OF HOLE

ELEVATION SCALE: 700, 690, 680, 670, 660

COMPRESSIVE STRENGTH TONS/SQ.FT. • UNCONFINED: 1.0, 2.0, 3.0, 4.0, 5.0

WATER CONTENT W%: ○ NAT, □ Lw, △ Pw: 10, 20, 30, 40, 50

DYNAMIC PENETRATION TEST BLOWS PER FOOT: 20, 40, 60, 80, 100



SAMPLES

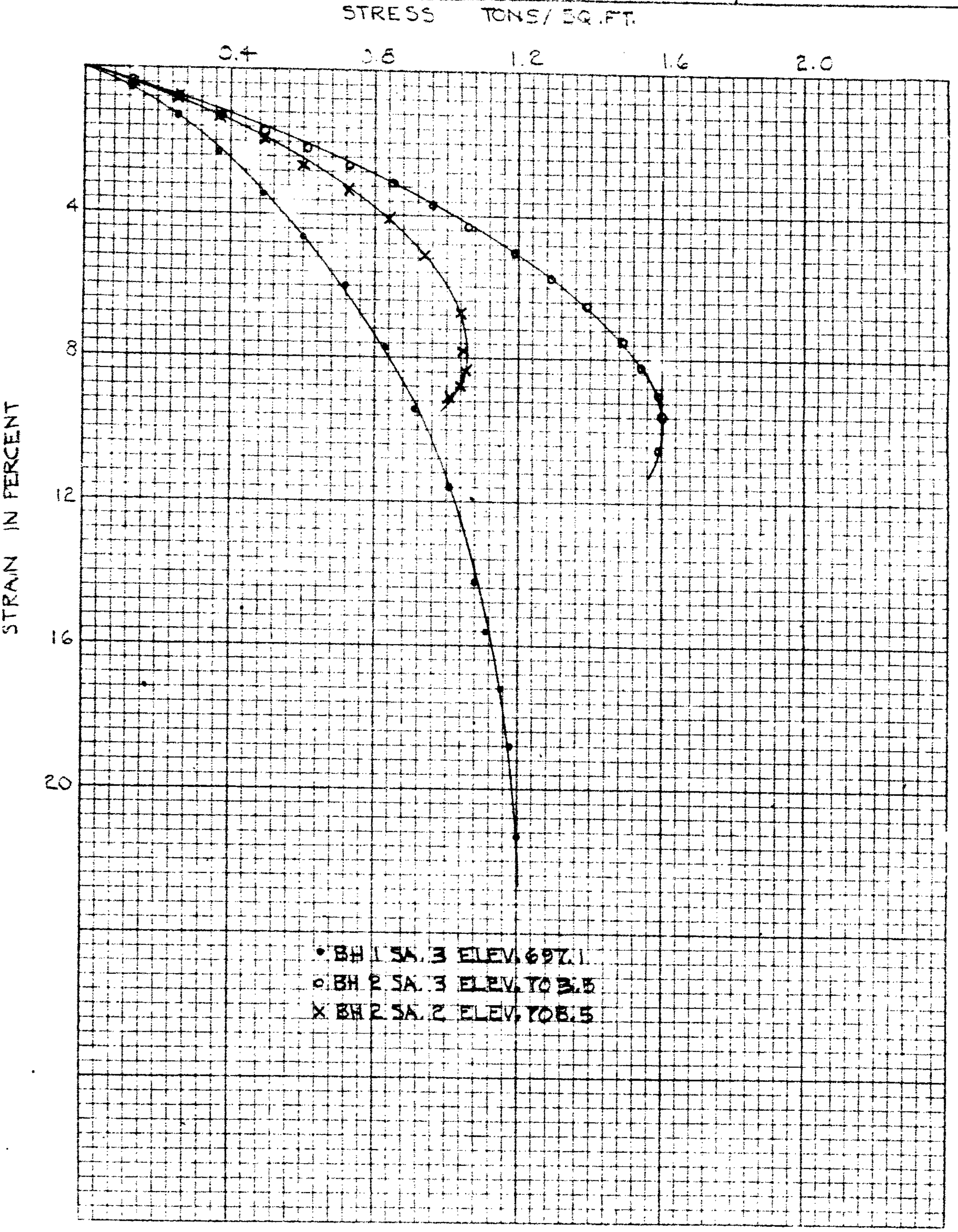
OTHER TESTS	CONDITION	TYPE	NUMBER	PENETRATION RESISTANCE BLOWS/FT.
		C.S	1	—
γ=130		2" SO	2	14
γ=124		SO	3	17
		SO	4	—
γ=135		SO	5	13
		SO	6	19
γ=137		SO	7	19
		SO	8	25
γ=124		SO	9	24
		W.S	10	—

APPENDIX II

FIGURES

UNCONFINED COMPRESSION TESTS
STIFF BROWN CLAY FILL
STRESS-STRAIN CURVES

APPENDIX II
FIGURE I
PROJECT 56636



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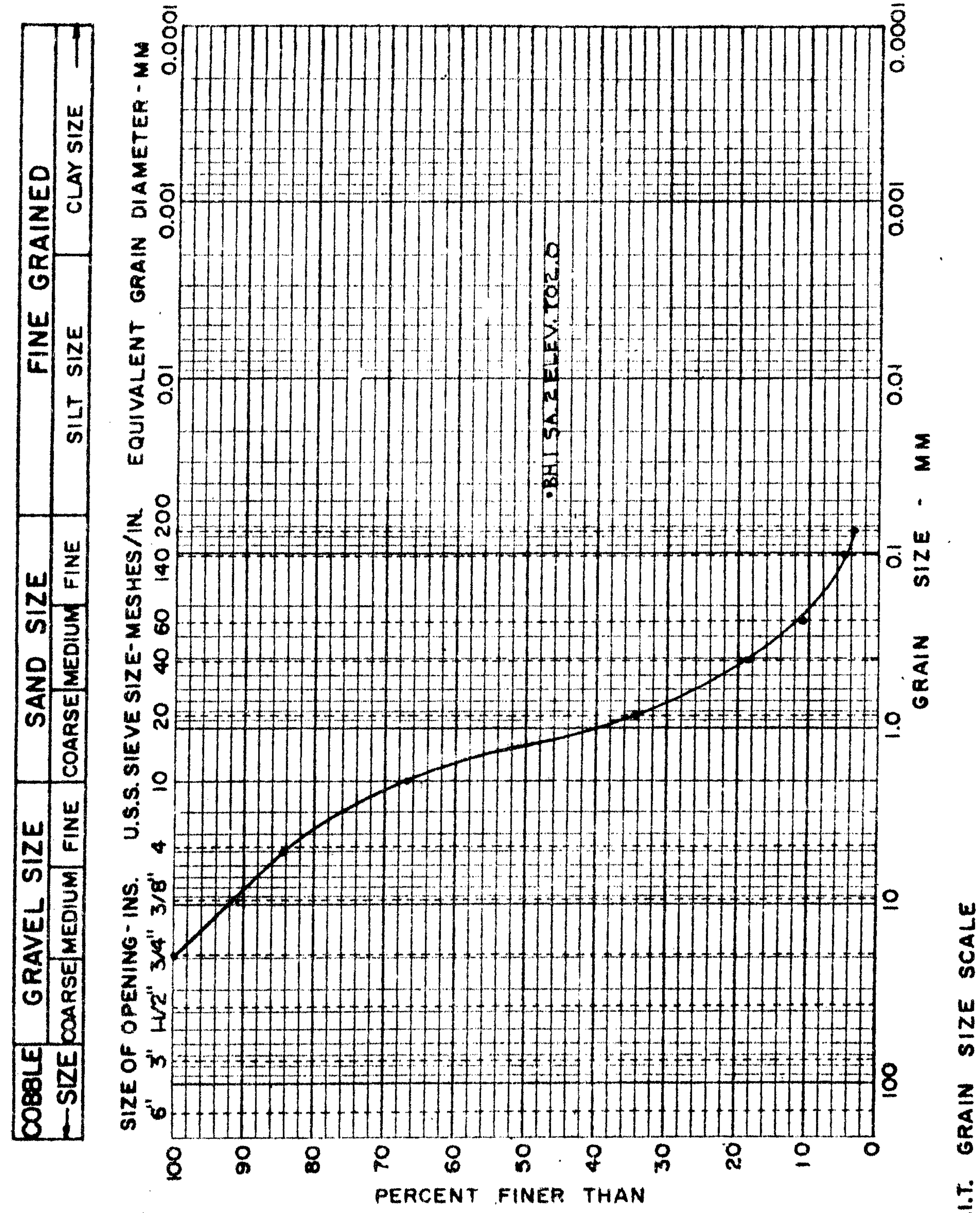
GRAIN SIZE DISTRIBUTION

APPENDIX II
FIGURE 2
PROJECT S6636



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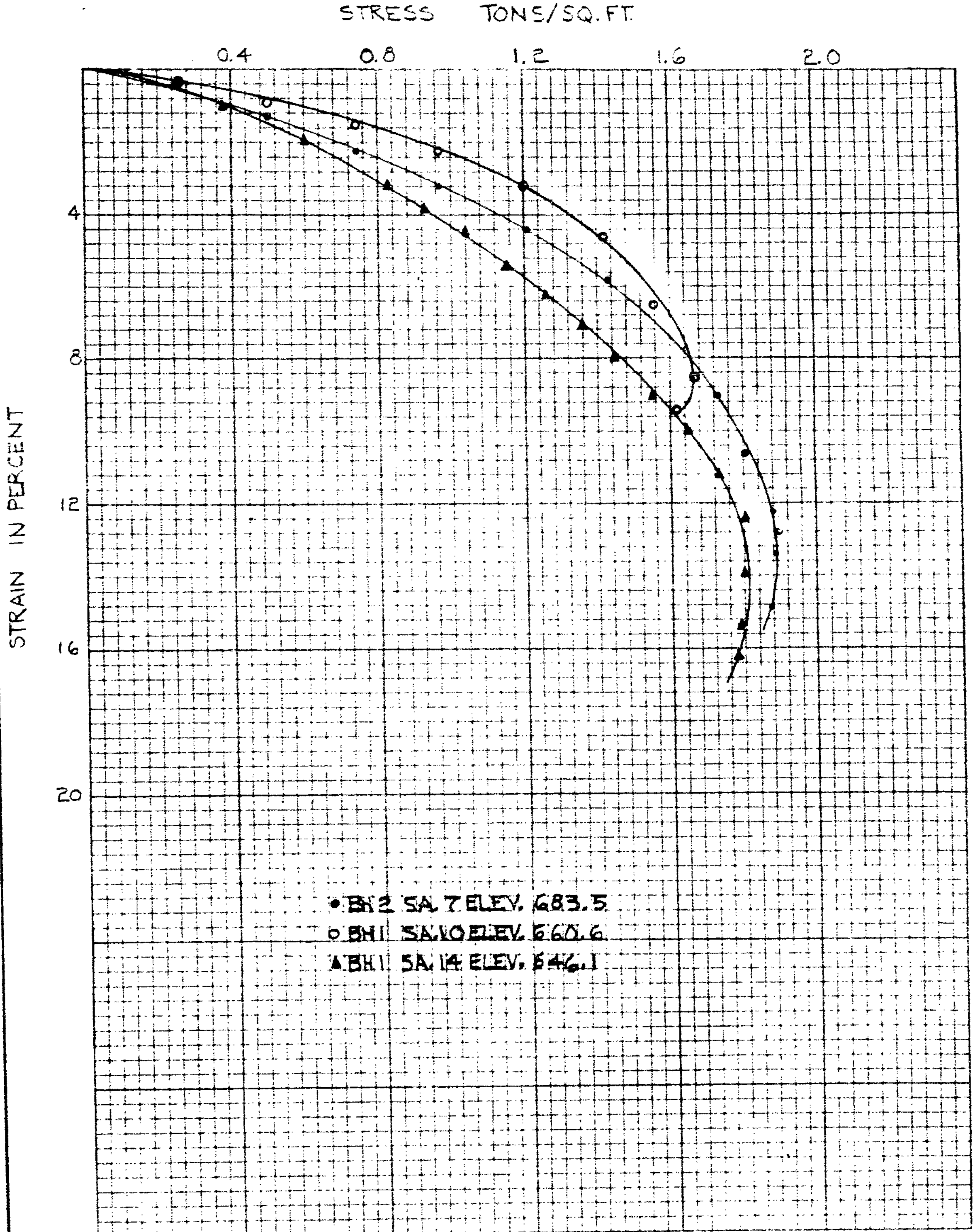
UNCONFINED COMPRESSION TESTS VARVED SILTY CLAY TYPICAL STRESS-STRAIN CURVES	APPENDIX II FIGURE 3 PROJECT 56636



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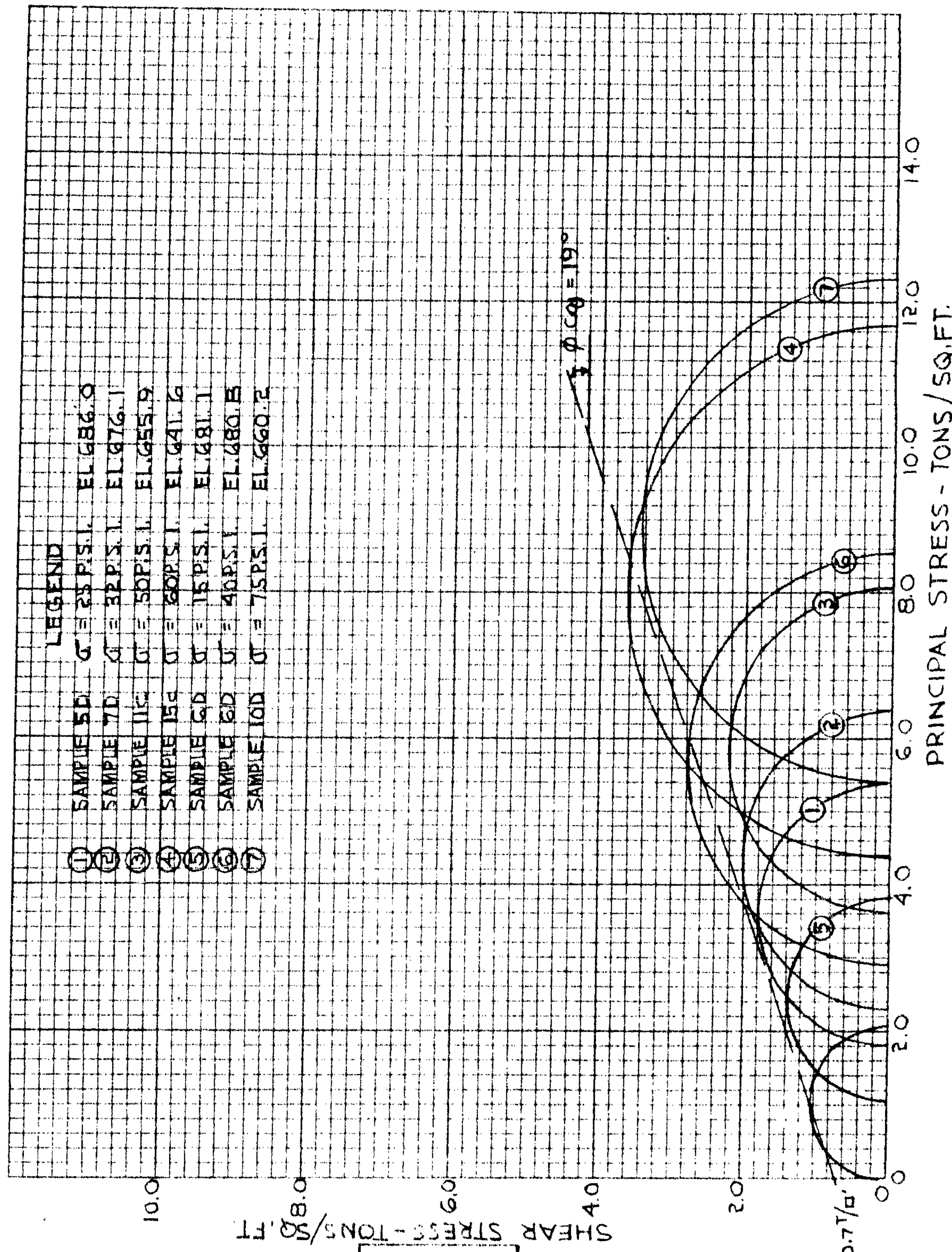
MOHRS CIRCLES
CONSOLIDATED QUICK TRIAXIAL TESTS
BOREHOLE I

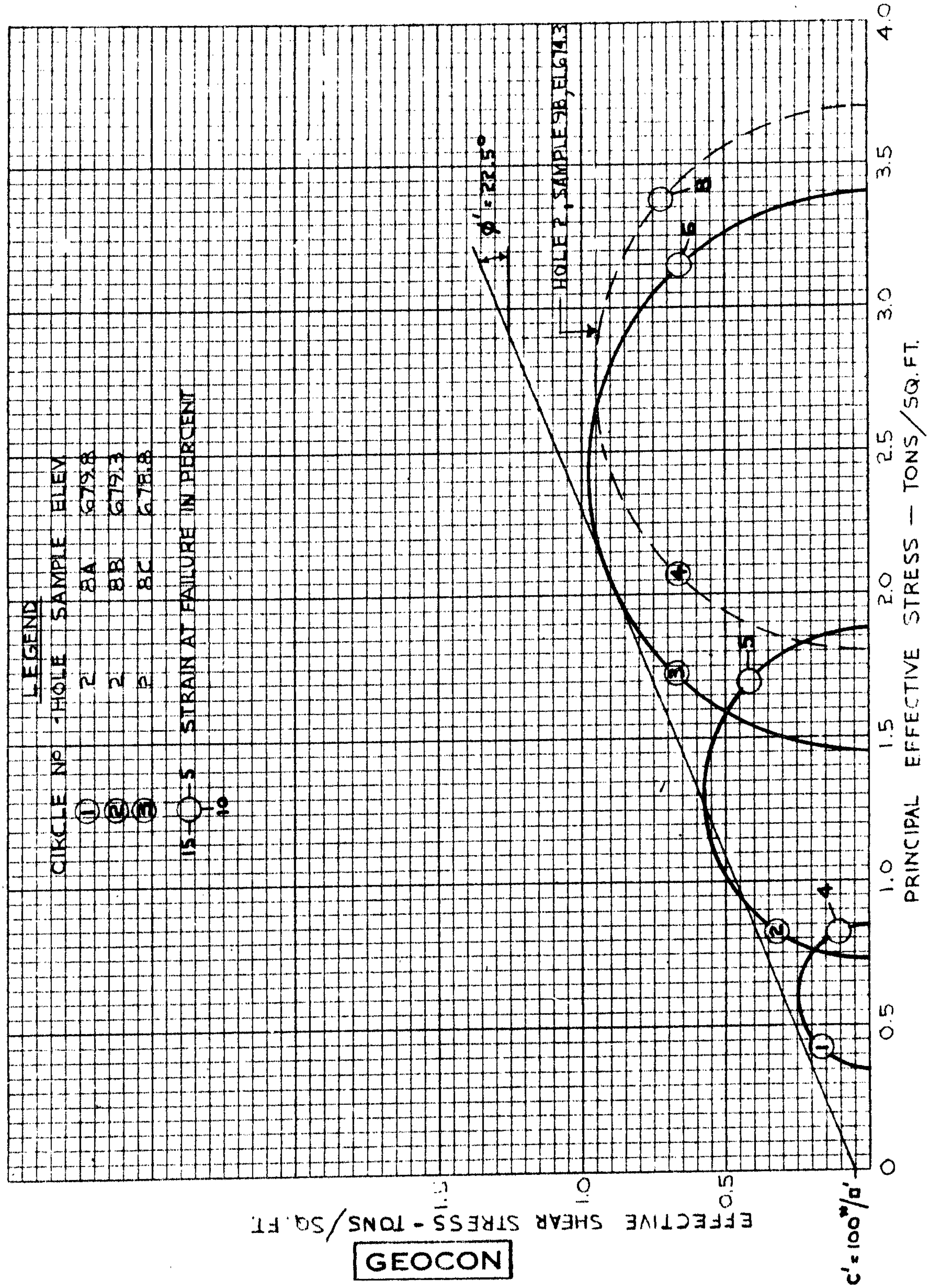
APPENDIX II
FIGURE 4
PROJECT 56636



GEOCON

MOHRS CIRCLES
SLOW DRAINED TRIAXIAL TESTS
VARVED SILTY CLAY

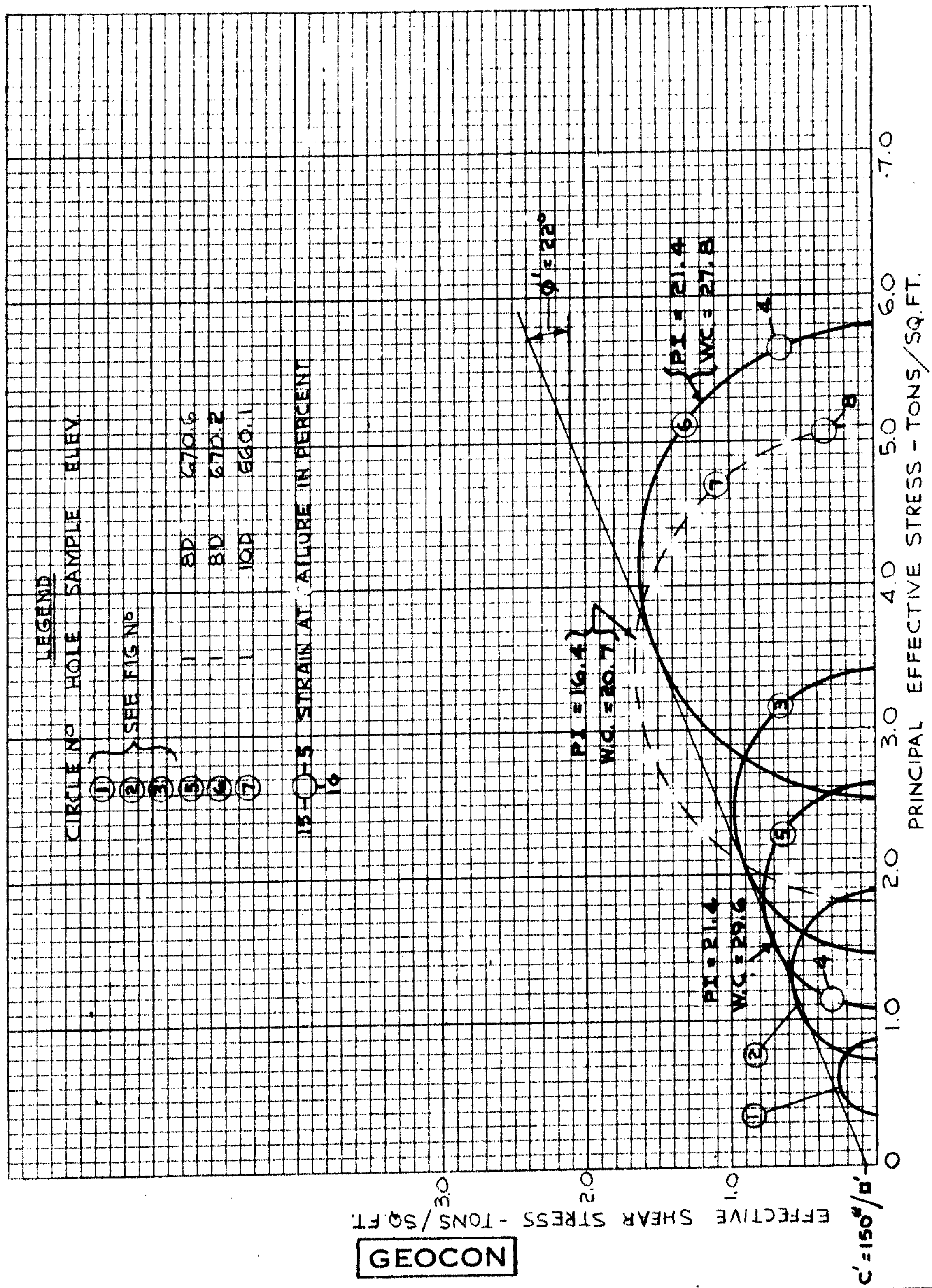
APPENDIX II
FIGURE 5
PROJECT S6636



GEOCON

MOHRS CIRCLES
SLOW DRAINED TRIAXIAL TESTS
VARVED SILTY CLAY

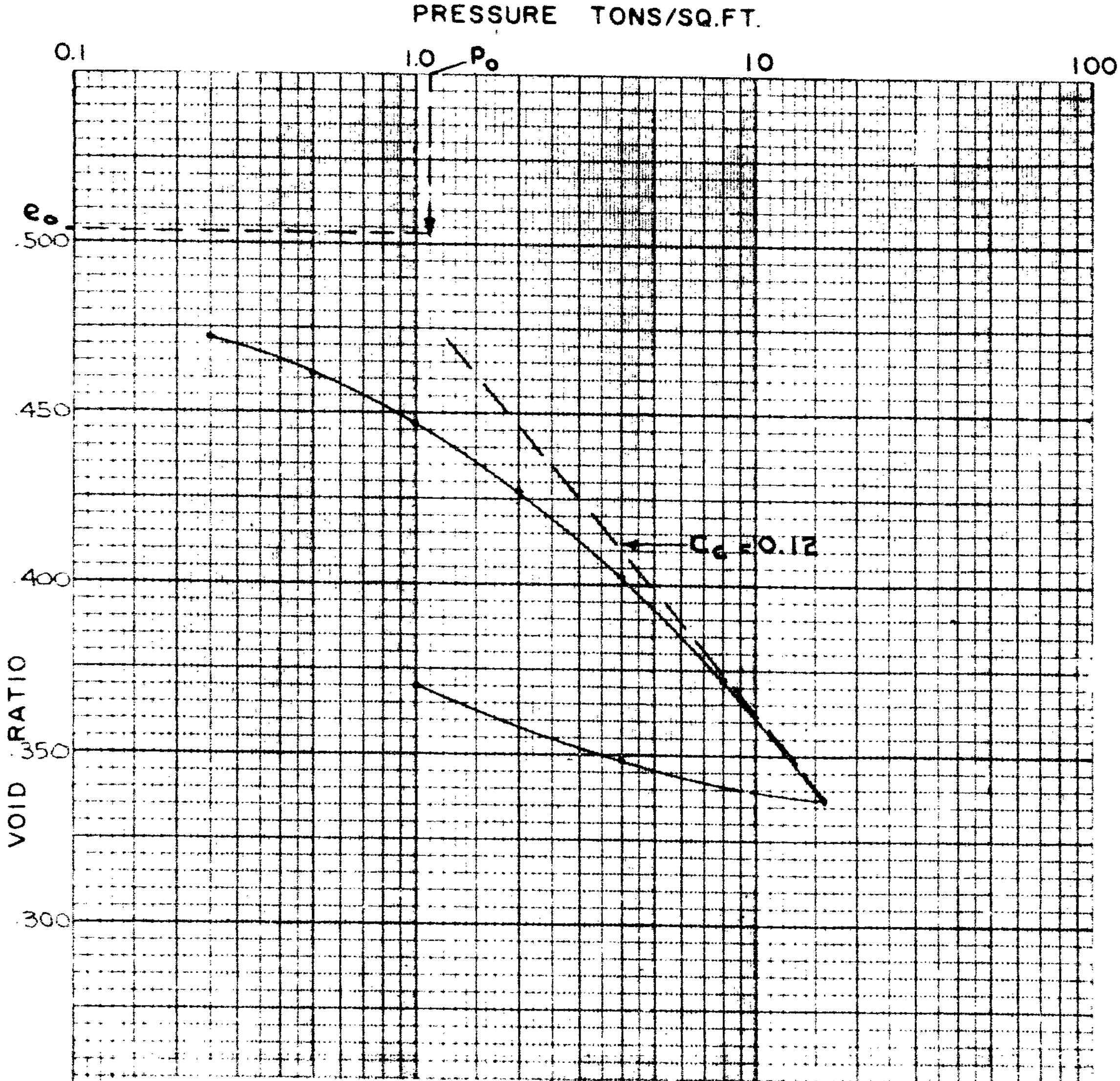
APPENDIX II
FIGURE 6
PROJECT S6636



GEOCON

VOID RATIO-PRESSURE CURVES
CONSOLIDATION TEST

APPENDIX II
FIGURE 7
PROJECT S6636



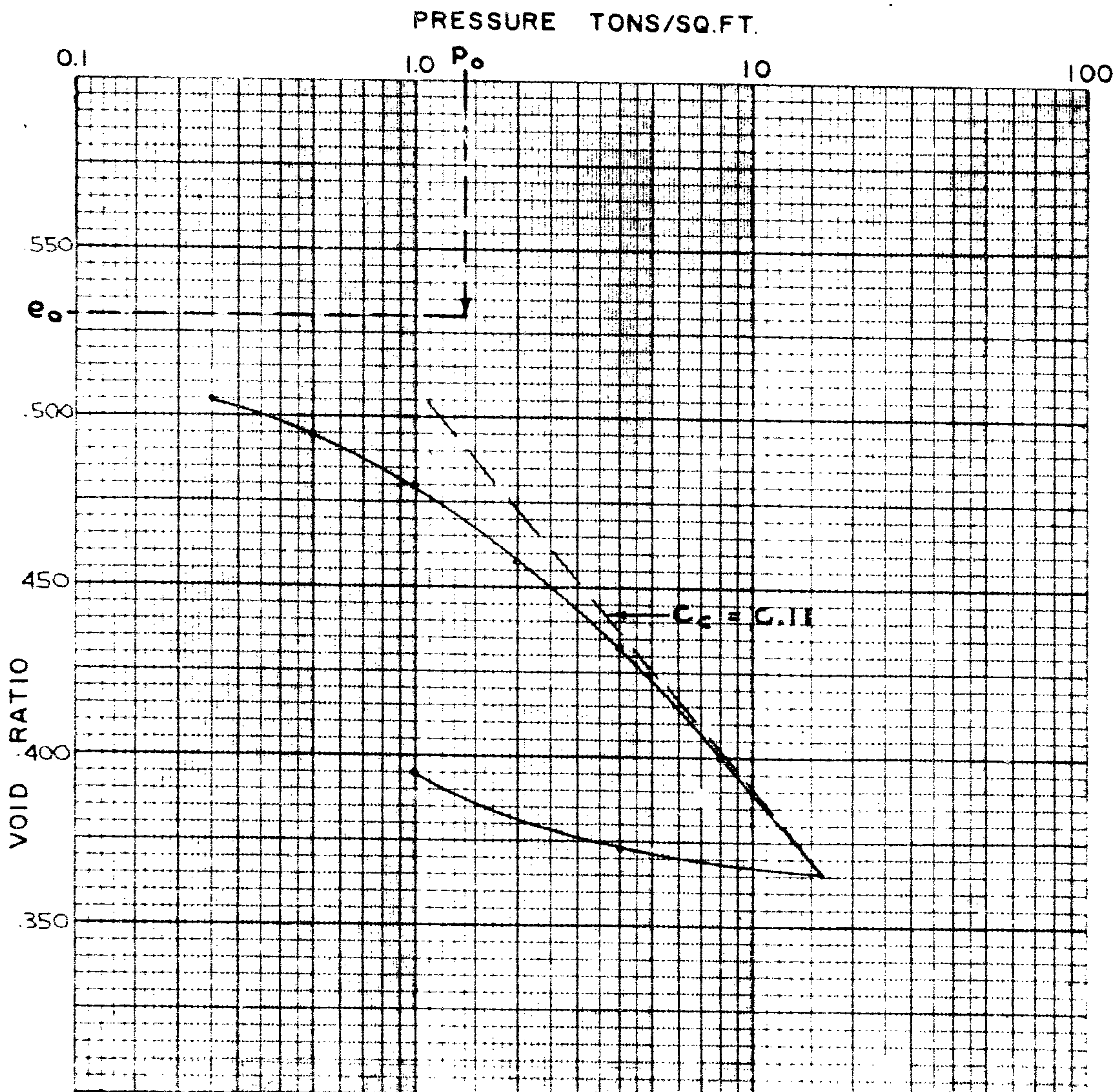
LEGEND

HOLE	SA.	ELEV.	MEASURED e_o	COMPUTED e_o	WATER CONTENT
1	5D	686.1	0.503	0.58	22.1%

GEOCON

VOID RATIO-PRESSURE CURVES
CONSOLIDATION TEST

APPENDIX 11
FIGURE 8
PROJECT S6636



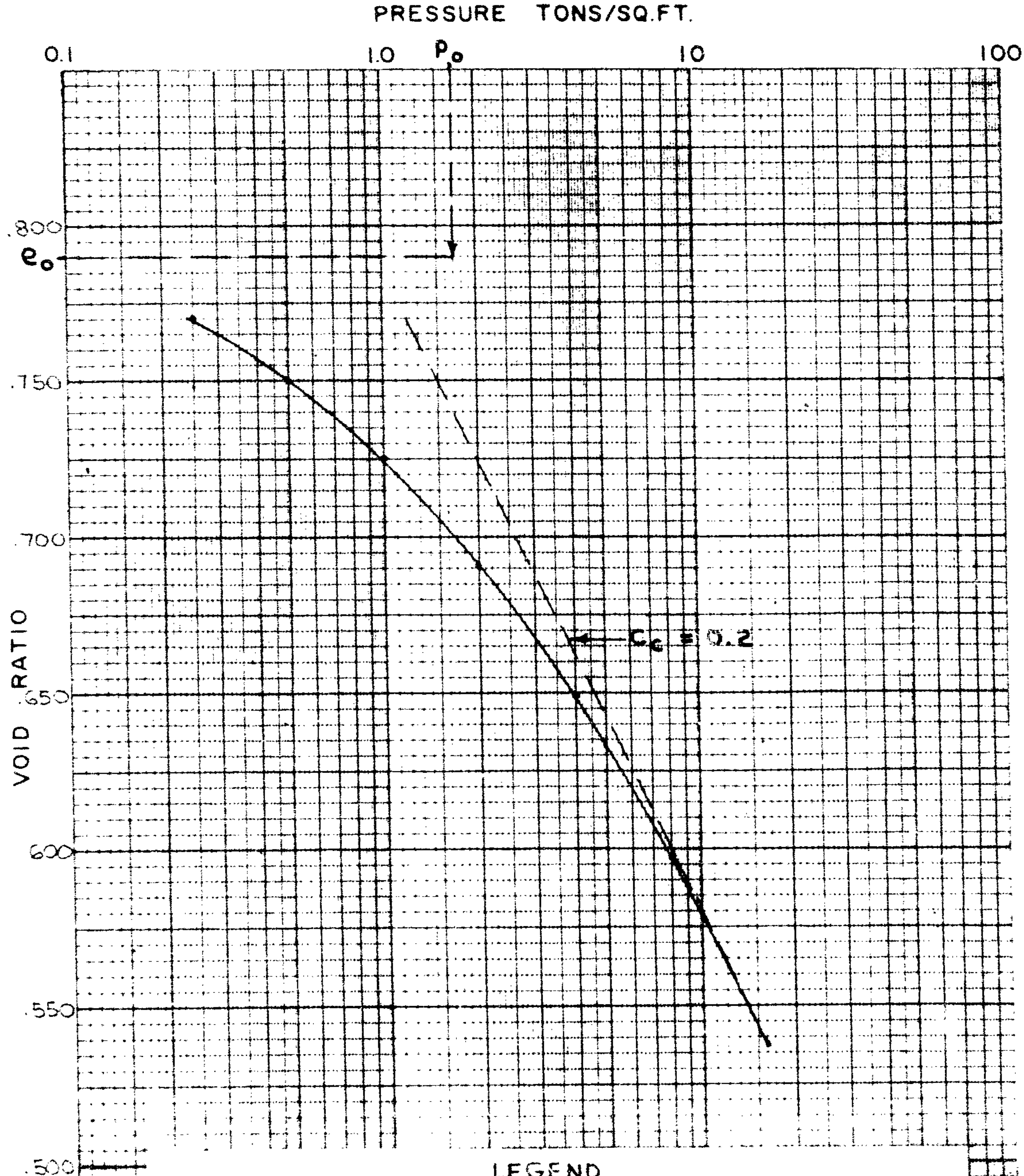
LEGEND

HOLE	SA.	ELEV.	MEASURED e_o	COMPUTED e_o	WATER CONTENT
1	7D	675.8	0.53	0.55	21.0%

GEOCON

VOID RATIO-PRESSURE CURVES
CONSOLIDATION TEST

APPENDIX 11
FIGURE 9
PROJECT S6636



LEGEND

HOLE	SA.	ELEV.	MEASURED e_o	COMPUTED e_o	WATER CONTENT
1	9D	664.9	0.79	0.82	31.2%

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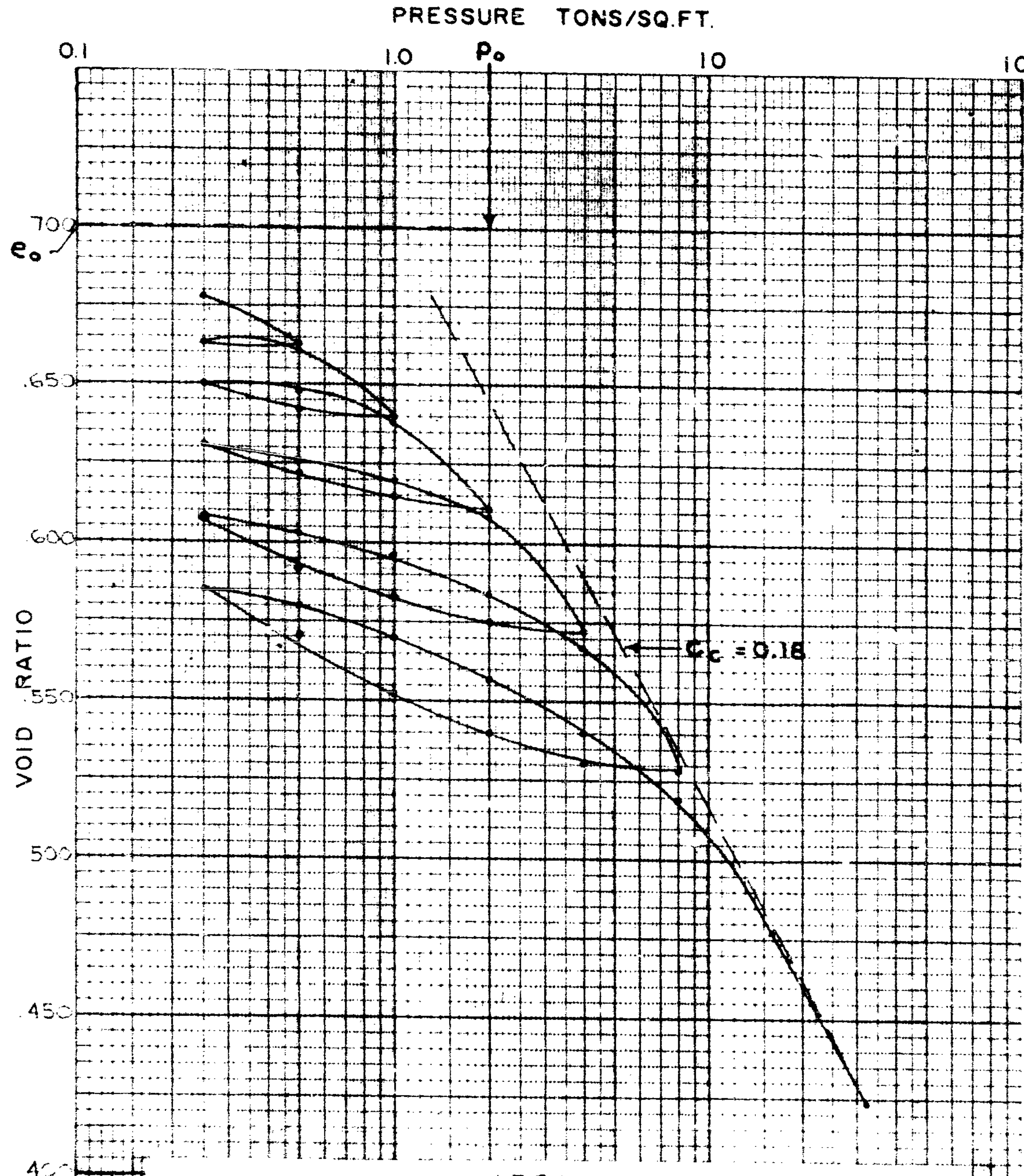
VOID RATIO-PRESSURE CURVES CONSOLIDATION TEST	APPENDIX II FIGURE 10 PROJECT S6636



LEGEND

HOLE	SA.	ELEV.	MEASURED e_o	COMPUTED e_o	WATER CONTENT
1	11D	654.9	0.70	0.71	26.7%

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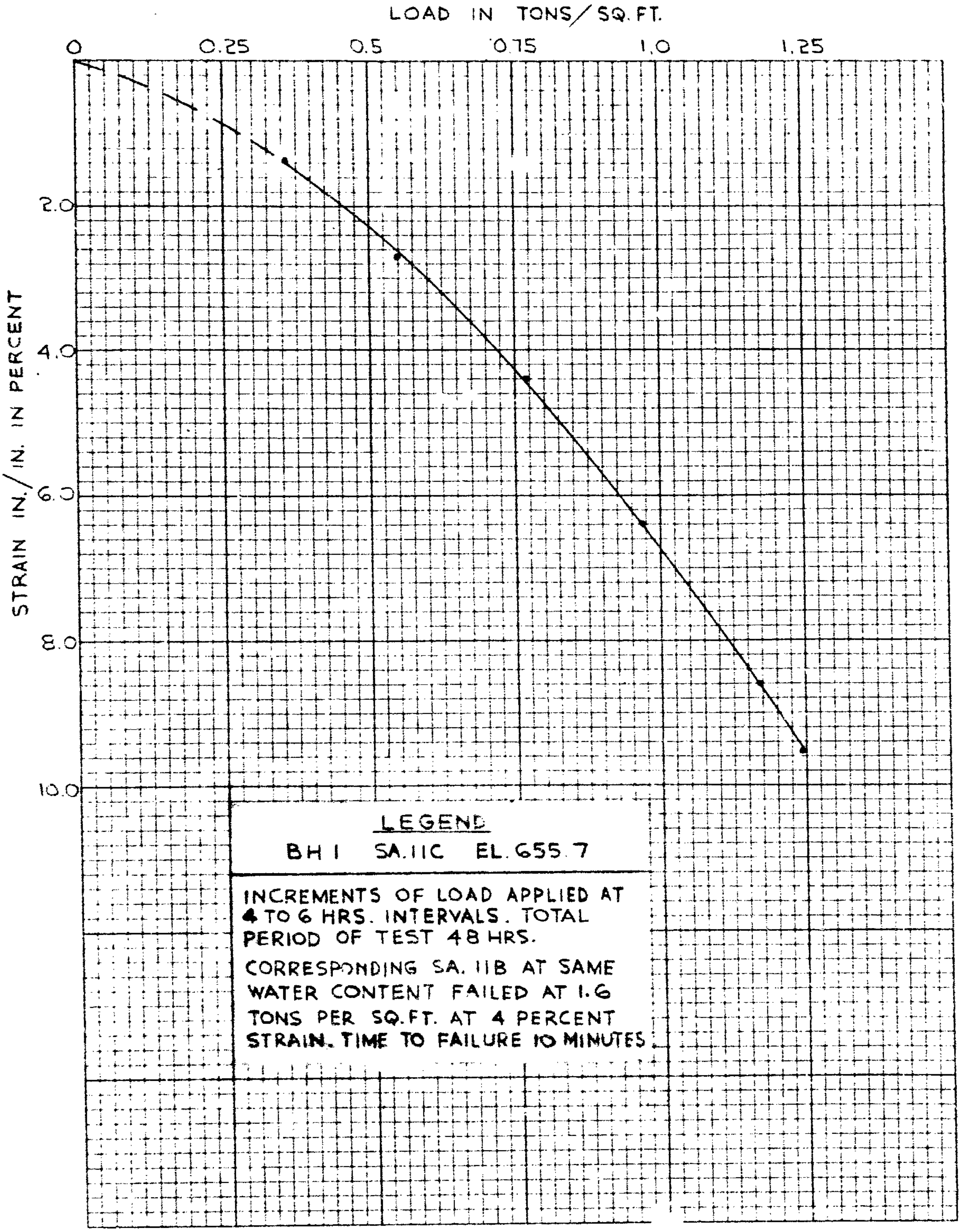
SLOW UNDRAINED TRIAXIAL TEST VARVED SILTY CLAY	APPENDIX II FIGURE 11 PROJECT S6636



LEGEND

BH 1 SA. 11C EL. 655.7

INCREMENTS OF LOAD APPLIED AT 4 TO 6 HRS. INTERVALS. TOTAL PERIOD OF TEST 48 HRS.

CORRESPONDING SA. 11B AT SAME WATER CONTENT FAILED AT 1.6 TONS PER SQ. FT. AT 4 PERCENT STRAIN. TIME TO FAILURE 10 MINUTES.

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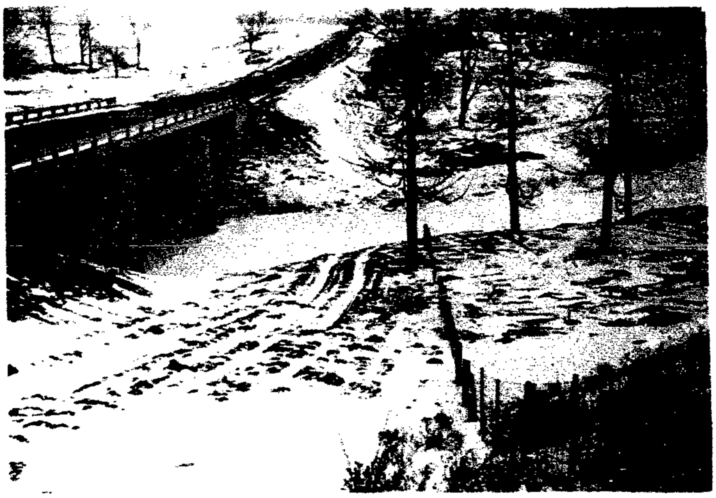
APPENDIX III

PHOTOGRAPHS, FIGURES 1, 2 AND 3

APPENDIX III
FIGURE I
PROJECT - S6636

General View of Bridge
Looking North



General View of Bridge
Looking North

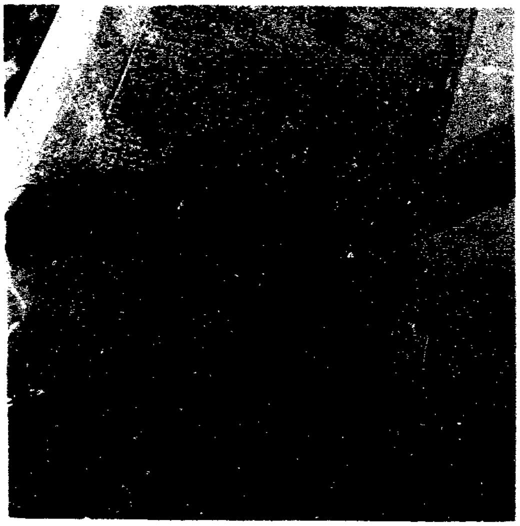
General View of Bridge
Looking East

South Abutment
Looking West

Hill "A"
Above South Approach Road

S.E. Curb
Present Condition

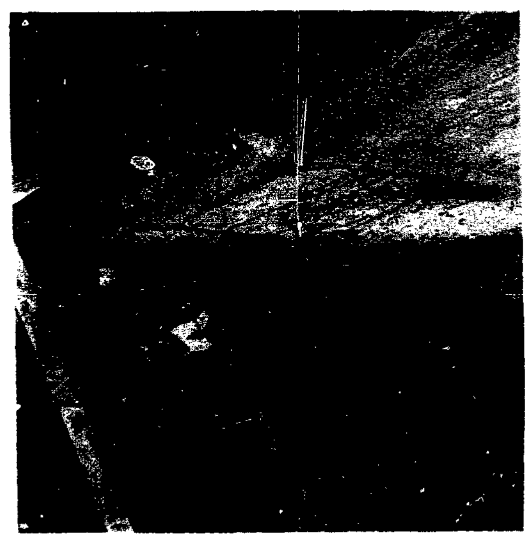
APPENDIX III
FIGURE 3
PROJECT - S6636



S.E. Curb & Expansion Joint
Primary Movement

S.W. Curb & Expansion Joint
Primary Movement

S.E. Curb & Construction Joint
Present Condition



Corner of W. Wing
and Face of S. Abutment

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APPENDIX IV

INFERRED CONSTRUCTION PROGRAMME

APPENDIX IV

INFERRED CONSTRUCTION PROGRAMME

Compiled for Field Books and Available Records

1956

February 3		Excavation north abutment commenced
"	8	Driving piles north abutment
"	15	Excavation south abutment commenced
"	20	Excavation south abutment completed
March	16	Pile driving south abutment commenced. Piles reported to drive easily for 30 feet going in almost under the weight of hammer. Hard driving for 1 foot then very easy driving for next 10 feet then refusal.
"	23	Pile driving south abutment completed
"	24	46 piles concreted in south abutment
"	26	Pile driving south pier commenced
"	27	From Field Book No. 5, Contract 55-274 "Piles #1 to #7, South Pier, off line due to movement of fresh fill towards river. This movement caused by vibration of pile drivers when pulling piles to site." High March floods reported by local residents.

1956

April	14	Pile driving completed south pier
"	25	South abutment footing poured, 16 piles remaining concreted.
May	24	Pouring south abutment wing walls, etc. to level of bridge seat.
July	17	Excavation south pier footing commenced
"	27	Excavation south pier footing completed
August	1	South pier footing poured
Sept.	26	Backfilling commenced south abutment, using water jet for compaction.
October	1	Backfilling completed

1957

April	8	Erosion of granular backfill, south abutment.
June	7	From Field Book No. 11, Contract 55-274

"Checking alignment of south abutment face with control point. It would appear from this check that the complete south abutment had moved approximately 7 inches in a northerly direction. This would account for the discrepancy between the bearing points on the abutment and girder rocker plates."

Other records quote this figure as $4\frac{1}{2}$ inches.

1957

June	14	Structural bridge steel placed.
"	26	Structural steel completed.
July	4	Ballast wall at south abutment amended. Possible 6 inch forward movement of abutment reported.
"	8	Control points established to measure south abutment movement.
"	24	Observed that south abutment wing walls are out of line with north abutment wing walls.
August	1	South abutment ballast wall poured. Constructed 6 inches further to south than shown on Drawing D3529-1.
	16	Deck poured

1958

January	13	1/4" lateral movement North west side north abutment measured. No movement north east side north abutment.

APPENDIX V

GENERAL CONCLUSIONS OF DR. H.Q. GOLDER, CONSULTANT

HUGH Q. GOLDER P.Eng.
D.Eng., M.I.C.E., M.E.I.C.
Consulting Civil Engineer

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TORONTO 12
RUssell 7-5711

806. REPORT ON MOVEMENT OF AUSABLE RIVER BRIDGE

Introduction

Geocon Ltd. are carrying out an investigation into the movement of the road bridge over the Ausable River for the Department of Highways, Ontario.

At the request of Geocon Ltd. I have examined all the available data and have visited the site with Mr. Milligan, with whom I have discussed the problem in detail.

The data are all given in Geocon's report and are not reproduced in detail in the present report.

Statement of Problem

At a late stage in construction movement of the south abutment of the bridge was noticed.

A system of measurements was started and it is thought that the south ends of the wing walls to the south abutment have settled about 8". On the north side the corresponding settlement is about 1". The increase of the batter of the abutment on the south side is about 11", and on the north side it is about 1".

A forward movement of the south abutment was reported at more than one stage in construction, the amount being variously given as $4\frac{1}{2}$" - 7".

The problem is to decide if these movements have really occurred, and if so what is their cause. Further it has to be decided whether or not the bridge structure is in danger, and if so what remedial measures should be taken.

Possible Explanations of Movement

1) An obvious possibility is that the movements observed are the first stages of a rotational slide. This can be checked by carrying out an analysis of the overall stability of the section.

2) The observed movements can also be explained by a settlement of the rear vertical piles and a rotation about the tops of the batter piles. If a settlement of the piles in the north-west corner of the south abutment is included the small observed horizontal rotation can also be accounted for. This mechanism can best be checked by a careful estimate of the loads on the piles and their probable ultimate carrying capacity.

These two possibilities are discussed below and the conclusion is reached that the second is the more likely.

Overall Stability of the Section

Drained triaxial compression tests on the clay give c' = 150 lb/sq.ft. ϕ' = 22°. The average undrained triaxial compressive strength is 3,200 lb/sq.ft.

Calculations carried out by Geocon show that the factor of safety for these values is 1.6 provided there is no excess pore pressure in the silt below a level of 630 ft.

However, the analysis is very sensitive to a rise in the water pressure in the silt layer, and a water level in this layer (stand-pipe level) of about 690 ft. reduces the factor of safety to unity. It is therefore very important that the water pressure in this silt should be known. At present this is not known, but there is some evidence which suggests that the water level may be higher than river level (672.5 ft.). High water level in the river during floods is 694 ft.

A normal circular failure due to overstress in the clay zone is not likely since such failures follow a slow plastic deformation. The movements which have been reported, if they have in fact occurred, are such that failure of this type would have resulted long ago.

It is possible however that these movements occurred when the water pressure in the silt was temporarily high, and ceased when the water pressure dropped again. This is typical of failures of this type. An abnormally high water pressure in the silt could result in complete failure. For this reason no one can say with certainty that the slope is stable until the water pressure in the silt is known. This means measuring it for at least one year, and preferably longer and correlating it with rainfall and river level over the same period. It is possible, and indeed likely, that such measurements will show quickly that there is no danger of a failure of this sort.

Settlement due to Overloading of the Piles

Geocon Ltd. have computed from the results of consolidation tests that settlement due to consolidation of the clay under the weight of applied fill will be 2" to 2½" on the south side.

This figure is computed allowing no swelling due to excavation.

The time for 90% of this settlement to take place would be 6 years with vertical drainage, or 1 year from triaxial tests, i.e. with horizontal drainage. In the practical case, as the loaded area is small the drainage will be three dimensional. All this means that the settlement will occur relatively quickly.

The downward movement of 8" at the wing wall on the south side corresponds with a movement of the vertical piles downwards of only 2" to 2½" if it is assumed that the abutment rotates about the tops of the batter piles. The increase in batter of the abutment of 11" at the top also agrees with this movement. It is therefore suggested that such a movement took place due to the penetration of the vertical piles under a load greater than their ultimate, and that this movement was limited to 2" or thereabouts.

As the clay under the fill settled due to the weight of the fill it would exert a drag on the vertical piles. This would be resisted only by the point resistance of the piles since there would temporarily be no skin friction in an upward direction.

The load on the piles would therefore be the structural load, i.e. about 50 tons, plus the negative skin friction.

Load on Pile Due to Negative Skin Friction

Frictional drag is computed for the top half of the pile only since in this region the settlement will be 1" to 2" i.e. sufficient to develop shear strength. This will give a lower limit for this force.

Length = 25ft. Average diameter = 10"

Embedded Area = $\dfrac{25 \times 10 \times \pi}{12}$ sq. ft. = 66 sq.ft.

Shear Strength of Clay = 0.8 ton/sq.ft. = 1,600 lb/sq.ft.

Load on pile = $\dfrac{1,600 \times 66}{2,000}$ tons = 53 tons.

Thus the total load on each pile could be about 53 + 50 = 103 tons.

Resistance of Piles to Penetration - Point Load Only

a) By Meyerhof's Method

N = blow count in standard penetration test.

Point Load = 4. N. A_p

A_p = area of point (diameter 8")

N = 70 for silty sand at level of toes of short vertical piles.

Therefore - Point Load = $\frac{4 \times 70 \times \pi \times 64}{4 \times 144}$ = 100 tons approximately

b) By Terzaghi's Method

Assume ϕ = 37°, and N_q for deep footing = 2 x surface value
Then N_q = 2 x 50 = 100
Point load = A_p x 100 x $\gamma . D$

$$= \frac{\pi \times 64 \times 100 \times 60 \times 50}{4 \times 144 \times 2,000} \text{ tons} = 52 \text{ tons.}$$

This is probably an underestimate.

Thus the load on the pile is of the same order as the load required to cause penetration.

If this is the actual mechanism of the movement it means that there is no danger of further movement since once the consolidation settlements have ceased the load on the piles drops to 50 tons, and the load the pile is capable of supporting is increased by the skin friction on the shaft of the pile.

Movement of North Abutment

On the north side the fill is lower and is farther from the piles. It would not be expected therefore that the same movements would occur here.

The slight movements which have occurred on this side are probably due to the same causes as above, combined with a normal settlement of the piles when they took up the applied load. The settlement and the slight rotation of the abutment are exactly what would be expected.

Conclusions and Recommendations

1) There is danger of instability, only if the pore water pressure in the silt layer at a level of 630 feet is above river level.

2) No one can say definitely that the bank and the abutment are safe until these water pressures are known. Piezometers must therefore be installed.

3) If high water pressures are discovered, they must be relieved by the installation of relief wells.

4) If the water pressures are shown to be below river level it can be concluded that the movements were due to penetration of the points of the vertical piles under loads greater than their ultimate loads. These loads were due to negative skin friction caused by consolidation settlements of the clay under the weight of the fill. In this case no further action need be taken.

Hugh Q Golder

H. Q. Golder

November 1959.

References

Terzaghi, K. "Theoretical Soil Mechanics" Wiley, 1943. Page 124.

Meyerhof, C.G. "Penetration Tests and Bearing Capacity of Cohesionless Soils" Paper No. 866, Jour. S.M. & Found. Eng. Div., Proc. A.S.C.E. Jan. 1956.

APPENDIX VI

Drawings	S6636-1	Site Plan
	S6636-2	Soil Stratigraphy
	S6636-3	Observations of Settlement - South Abutment - Piezometric Levels - Boreholes 1 and 2
	S6636-4	Pile Driving Data - South Abutment
	S6636-5	Stability Analyses

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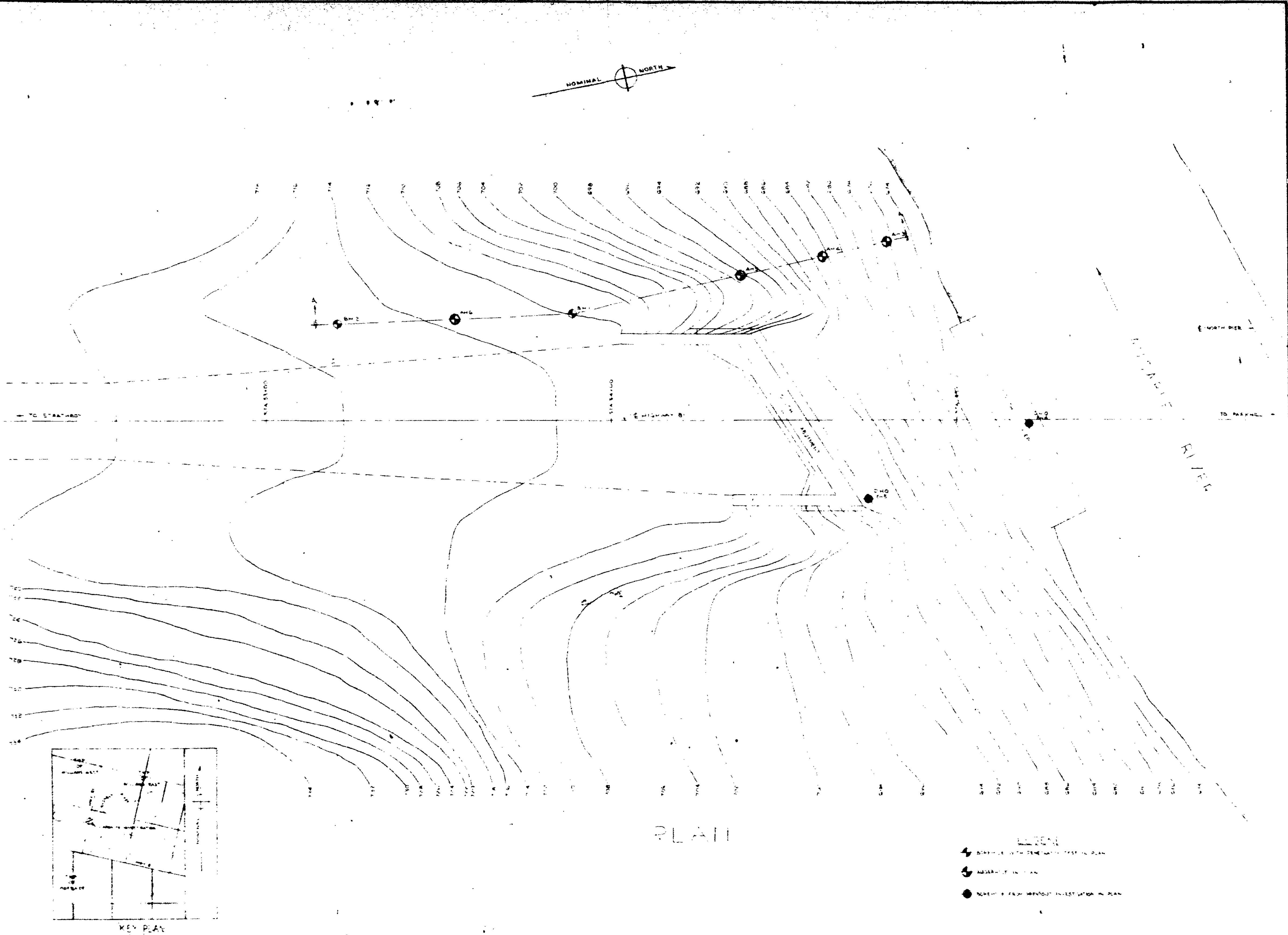


DEPARTMENT OF HIGHWAYS ONTARIO

SOIL INVESTIGATION & STABILITY ANALYSIS

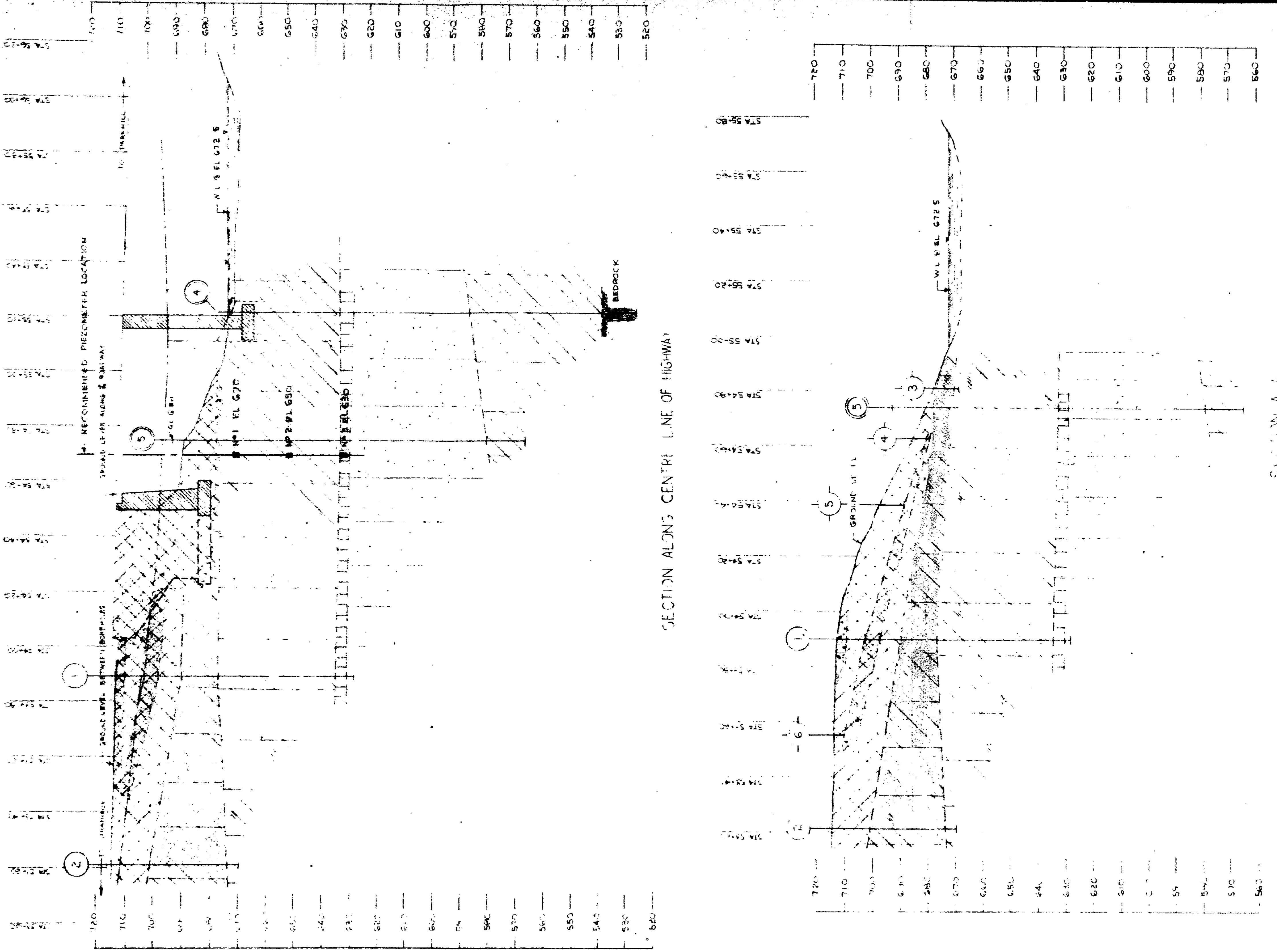
BORING PLAN

GEOCON LTD

BEFORE REDUCTION

J.A.

No. 56636-1



STRATIGRAPHY

- LOOSE BROWN SAND AND GRAVEL FILL
- STIFF BROWN CLAY FILL
- STIFF GREY VARVED SILTY CLAY
- STIFF GREY AND BROWN VARVED SILTY CLAY
- VERY DENSE SILT
- VERY DENSE GREY SILTY FINE SAND
- MEDIUM SILTY CLAY
- BEDROCK

LEGEND

- (1) BOREHOLE IN ELEVATION
- (3) AUGERHOLE IN ELEVATION
- (4) BOREHOLE FROM PREVIOUS INVESTIGATION IN ELEVATION

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

REVISIONS		
MARK	DATE	DESCRIPTION

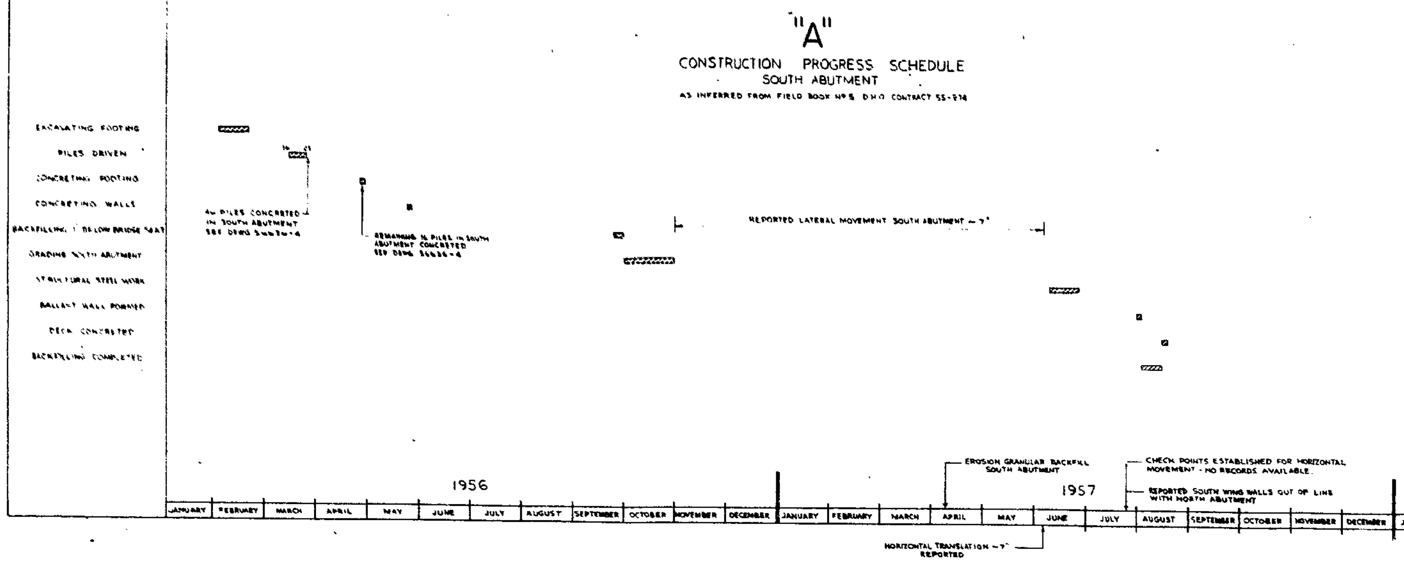
REFERENCE	
DWG NO	DESCRIPTION

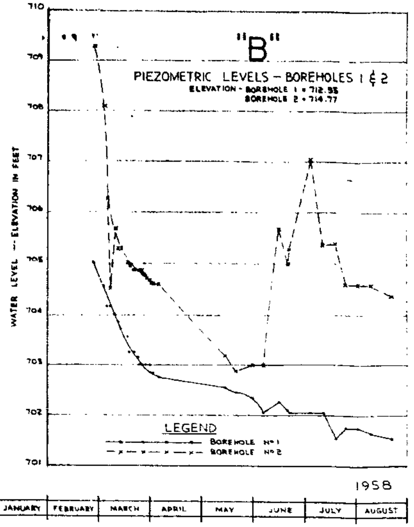
REFERENCE	
DWG NO	DESCRIPTION
–	CONTOUR PLAN – SUPPLIED BY DEPARTMENT OF HIGHWAYS, ONTARIO
D 3529	DEPARTMENT OF HIGHWAYS ONTARIO – GENERAL PLAN AND ELEVATION

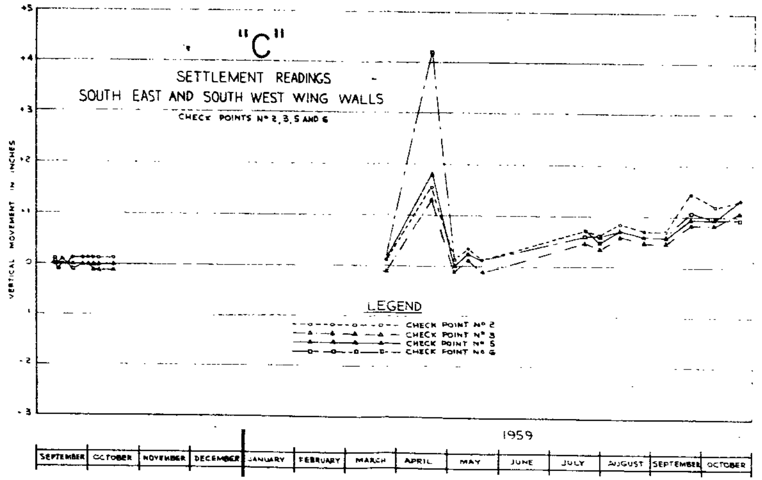
DEPARTMENT OF HIGHWAYS ONTARIO
TORONTO ONTARIO
SOIL INVESTIGATION
HIGHWAY B? STA 55+00-56+00
AUSABLE RIVER BRIDGE ONTARIO
SOIL STRATIGRAPHY

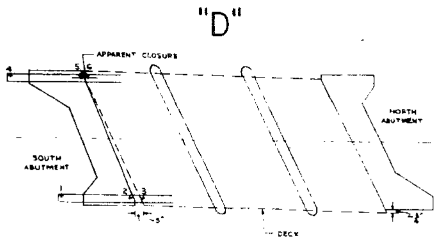
GEOCON LTD

DATE APRIL ? 1964 SCALE [illegible]
MADE J.A. CHKD. [illegible] APPD. [illegible] No. S [illegible]-2



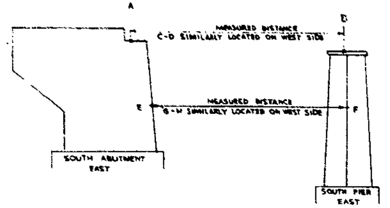




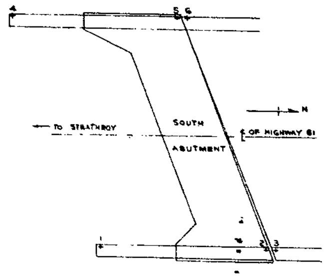


GENERAL PATTERN - OVERALL MOVEMENT

OVERALL VIEWS OF BRIDGE STRUCTURE - SEE FIG. APP. III
SETTLEMENT OF POINTS 1 AND 4 - SEE CHART "F"
SETTLEMENT OF POINTS 2, 3, 5 AND 6 - SEE CHART "C"
LATERAL MOVEMENT OF POINTS 2 AND 3 - SEE FIGS 2 AND 3, APP III
LATERAL MOVEMENT OF POINTS 5 AND 6 - SEE FIG 3, APP III



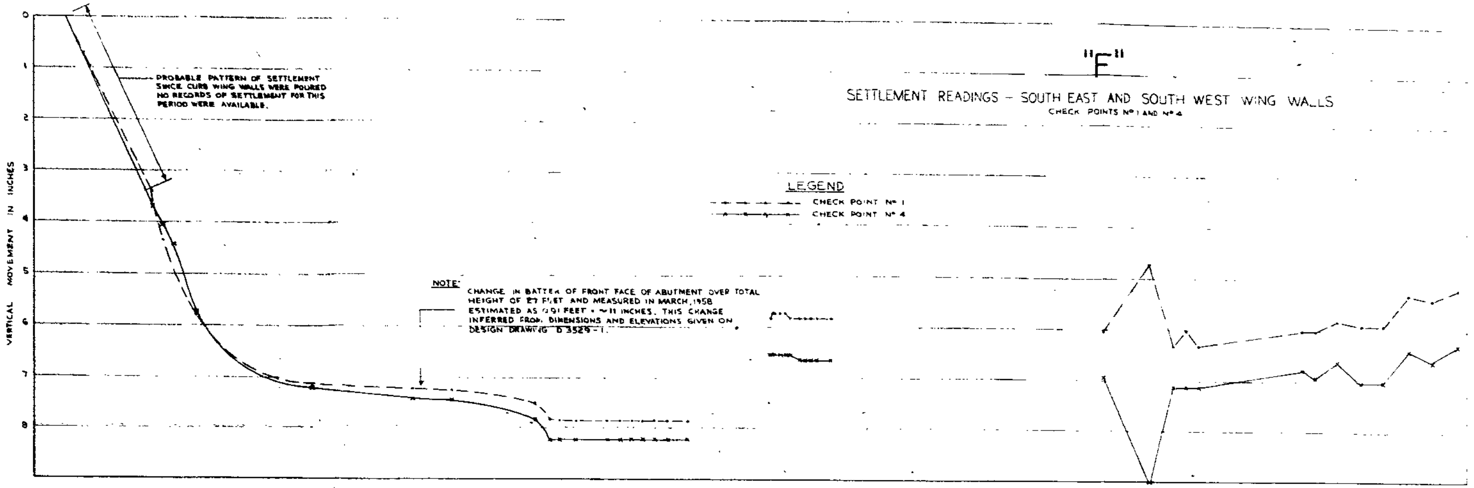
"E"



PLAN SHOWING LOCATION OF CHECK POINTS
N.T.S.

TABLE OF SPAN CHECKS

DATE	A B	C D	E F	G H	TEMP
SEPT 10/58	64.74'	64.74'			62°
" 11	64.74'	64.73'			58
" 12	64.74'	64.74'			56
" 15	64.73'	64.73'			76
" 19	64.74'	64.74'	63.57'	59.00'	60
" 21	64.74'	64.73'	63.57'	59.00'	56
" 26	64.74'	64.73'	63.57'	59.00'	71
" 30	64.74'	64.73'	63.57'	59.00'	68
OCT 3	64.73'	64.72'	63.57'	59.00'	54
" 7	64.73'	64.73'	63.57'	59.00'	64
" 15	64.73'	64.73'	63.57'	59.00'	58
MARCH 25/59	64.74'	64.74'			34
APRIL 21	64.77'	64.75'	63.55'	58.95'	43
MAY 5	64.78'	64.72'	63.54'	58.96'	48
" 15	64.77'	64.72'	63.55'	58.95'	66
" 20	64.76'	64.72'	63.54'	58.95'	77
JULY 22	64.76'	64.72'	63.53'	58.92'	78
" 29	64.76'	64.72'	63.53'	58.92'	82
AUG. 12	64.77'	64.72'	63.54'	58.92'	76
" 26	64.77'	64.72'	63.54'	58.92'	88
SEPT. 9	64.77'	64.72'	63.54'	58.92'	82
" 24	64.77'	64.72'	63.54'	58.92'	70
OCT 8	64.77'	64.72'	63.54'	58.92'	67
" 25	64.77'	64.72'	63.54'	58.92'	53

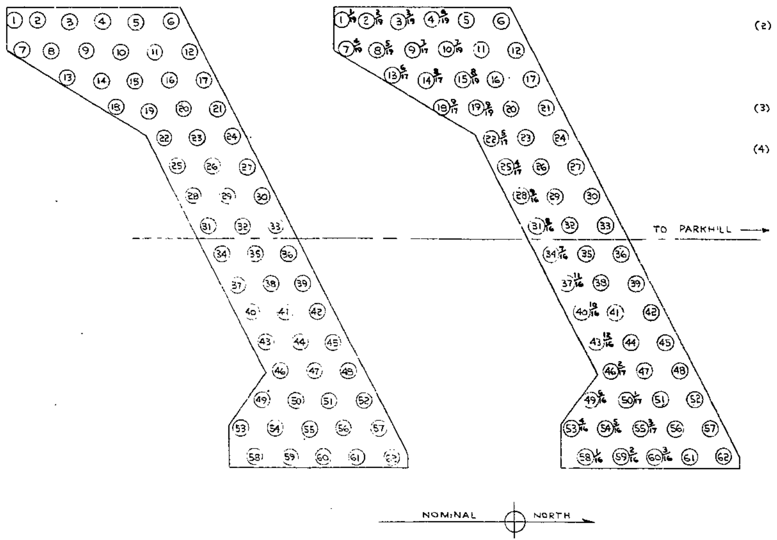


NOTE: ALL SETTLEMENT READINGS AND MEASUREMENTS MADE BY D.H.O. DISTRICT OFFICE - LONDON, ONT.

REFERENCE	
DWG. NO.	DESCRIPTION

DEPARTMENT OF HIGHWAYS, ONTARIO
TORONTO ONTARIO
AUSABLE RIVER BRIDGE
HIGHWAY 81
OBSERVATIONS OF SETTLEMENT AND LATERAL MOVEMENT - SOUTH ABUTMENT
PIEZOMETRIC LEVELS - BOREHOLES 1 AND 2

GEOCON LTD
DATE NOV 11, 1959 SCALE AS SHOWN
MADE J. A. CHECKED APPD. No. S6636 - 3



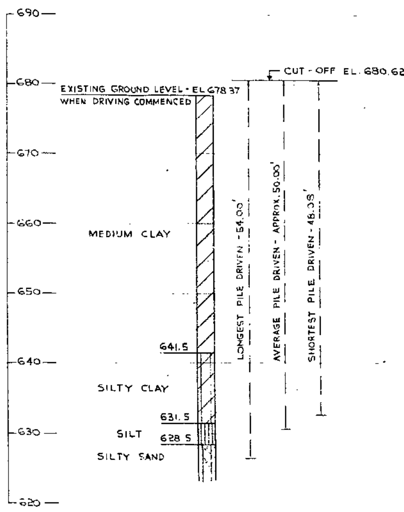
(1) PILES DRIVEN IN PERIOD MARCH 16–23 PILES MARKED AS DRIVEN EACH DAY e.g. $\frac{1}{19}$ FIRST PILE DRIVEN ON MARCH 19.

(2) ALL PILES FILLED WITH CONCRETE ON MARCH 24. WITH THE EXCEPTION OF THOSE PILES MARKED ◯ WHICH WERE FILLED ON APRIL 25, THE SAME DAY AS FOOTING. TEMP. RANGE 32° TO 40° F. WORK CEASED 1.00 A.M., TIME OF CONCRETE POUR 17 HRS.

(3) VERTICAL PILES ◯

BATTER PILES ◯

(4) PILE NUMBERS FROM D.H.O. RECORDS



NOTE:
SOIL CONDITIONS AS DESCRIBED IN DRIVING REPORT

PILE Nº	LENGTH DRIVEN
1	47.79'
2	48.48'
3	48.44'
4	50.39'
5	49.63'
6	49.75'
7	48.25'
8	48.17'
9	46.48'
10	50.14'
11	50.50'
12	52.50'
13	47.25'
14	48.77'
15	51.73'
16	53.17'
17	53.96'
18	50.95'
19	52.04'
20	53.48'
21	53.58'
22	51.75'
23	53.17'
24	52.73'
25	51.71'
26	53.37'
27	52.31'
28	50.55'
29	54.00'
30	52.17'
31	50.30'
32	53.04'
33	51.96'
34	50.44'
35	53.08'
36	52.31'
37	50.41'
38	52.83'
39	52.23'
40	50.32'
41	52.64'
42	52.46'
43	50.14'
44	52.21'
45	53.25'
46	51.22'
47	52.33'
48	50.42'
49	49.00'
50	50.32'
51	50.96'
52	50.25'
53	49.04'
54	48.67'
55	50.38'
56	50.49'
57	50.47'
58	48.50'
59	48.08'
60	48.83'
61	50.53'
62	51.42'

PILES DRIVEN BY BERMINGHAM PILE CO. HAMILTON – SUB CONTRACTOR –

AS NOTED IN D.H.O. CONTRACT RECORDS CONSIDERED INCORRECT

HEPBURN 50c HAMMER – 15,000 FT. LBS. 120 BLOWS PER MINUTE
STROKE 18" WEIGHT OF RAM 4,100 LBS.
WEIGHT OF HAMMER 14,000 LBS.

PROBABLY SUPER VULCAN DIFFERENTIAL ACTING STEAM HAMMER - 50C.
RATED ENERGY 15,100 FT. LBS. AT 95 BLOWS PER MINUTE

MONOTUBE PILES (STEEL TUBE CAISSON)
TYPE 7F 30' TAPERED SECTION #7 GAUGE
TYPE 9N 20' STRAIGHT SECTION #9 GAUGE

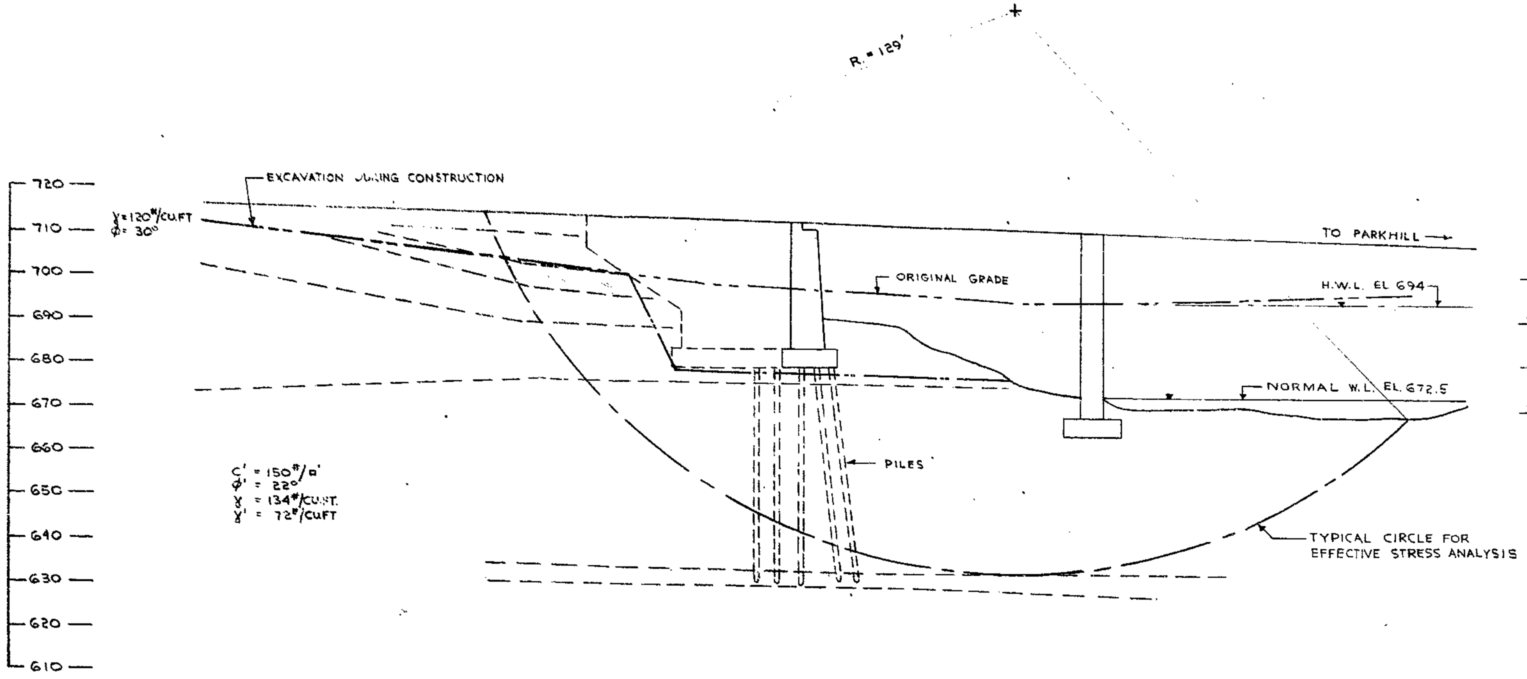
DRIVING RECORD-AVERAGE

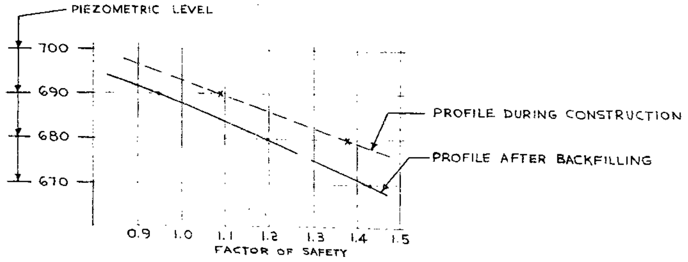
FIRST 5'	PILES PENETRATED	WEIGHT OF HAMMER
NEXT 15'	" "	6" TO 3" PER 120 BLOWS
NEXT 25'	" "	8" TO 6" PER 120 BLOWS
NE[illegible] 4'	" "	6" TO 3" PER 120 BLOWS
TO REFUSAL	" "	3" TO 1" PER 120 BLOWS

AVERAGE REFUSAL PER PILE 1" OR LESS PER 120 BLOWS

RE-DROVE PILES AFTER LEFT STANDING ~ 24 HRS.
PILE #19 – 60 BLOWS – PENETRATION $1\frac{3}{4}$"
PILE #60 – 60 BLOWS – " 2"

DEPARTMENT OF HIGHWAYS, ONTARIO TORONTO ONTARIO AUSABLE RIVER BRIDGE HIGHWAY 81 PILE DRIVING DATA – SOUTH ABUTMENT	GEOCON LTD DATE NOV. 17, 1959 SCALE AS SHOWN MADE J.A. CHKD. [illegible] APPD. [illegible] No. S6636 - 4





VARYING FACTORS OF SAFETY FOR TYPICAL CIRCLE SHOWN

(1) TOTAL STRESS ANALYSES GIVE RANGE OF FACTOR OF SAFETY 1.6 TO >2.0 DEPENDING ON RIVER WATER LEVEL.

(2) SLIDING PLANE ANALYSIS ABOVE SAND AND GRAVEL FILL ON SECTION A-A, DRAWING S6636-2 GIVES A FACTOR OF SAFETY = 1.1, IF PIEZOMETRIC HEAD OF 4 FEET EXISTS IN SAND AND GRAVEL.

(3) EFFECTIVE STRESS ANALYSIS OF SECTION SHOWN, ON CONDITIONS OF RAPID DRAWDOWN FROM H.W.L.-EL. 694 TO N.W.L.-EL. 672.5, GIVES FACTOR OF SAFETY = 1.1

FOR SOIL STRATIGRAPHY SEE DRWG. S6636-2

DEPARTMENT OF HIGHWAYS, ONTARIO TORONTO ONTARIO AUSABLE RIVER BRIDGE HIGHWAY 81 STABILITY ANALYSES	GEOCON LTD DATE NOV. 18, 1959 SCALE 1" = 20'-0" MADE J.A. CHKD. APPD. No. S6636-5