#63-F-203-C

W.P. #68-60-2

STABILITY HWY.

#17, STA. 183+

00, NEAR

MATTAWA

63-F-2030

GEOCON LTD

HEAD OFFICE
180 VALLÉE ST., MONTREAL 18, QUEBEC
TELEPHONE UN. 6-7632

DISTRICT OFFICES

14 HAAS ROAD
REXDALE, TORONTO, ONT.
TEL. 244-6476

1425 WEST PENDER ST.
VANCOUVER 5, B.C.
TEL. MU. 1-8926

Rexdale, Ontario,
January 30th, 1963.

Department of Highways, Ontario,
Materials and Research Section,
Downsview, Ontario.

Attention: Mr. A. G. Stermac, P. Eng.,
Principal Foundation Engineer.

Re: Soil Conditions and Slope Stability,
Highway No. 17, Station 183+00,
Mattawa, Ontario.

WP 68-60-2

W[redacted]

Dear Sirs:

This letter accompanies our report on the above soil investigation and stability study.

We find that the site in the vicinity of station 183+00 is covered by up to 20 feet of sand, gravel and rock fill, then about 8 feet of layered silty clay and clayey silt. This is followed by a stratum of silty sand, gravel and boulders then bedrock.

The availabl evidence, in our opinion, suggests that the failure of the embankment which took place during construction was of a deep seated type through the layered clay and silt stratum. The results of analyses of this past failure have been used in conjunction with information obtained in the current investigation to assess the stability of the proposed embankment. Our conclusion is that special measures will be required in the form of either stage construction or a stabilizing berm to enable the roadway elevation to be raised as proposed.

As discussed in the report, the outcome of the studies reported herein indicate that the section of the proposed roadway between stations 179+00 and 182+00 may also require the use of special measures to permit raising of the roadway as proposed. The requirements of this section of the roadway will have a bearing in the choice of stabilizing measure in the vicinity of station 183+00. It is therefore recommended that the section between stations 179+00 and 182+00 also be investigated prior to final design.

We believe that this report covers as far as possible the investigation of the particular area studied. However, please feel free to call us, should you require further information or if we can be of assistance to you in any further work which you may require at this site.

Yours very truly,

GEOCON LTD

M. A. J. Matich per D.B.O.

M. A. J. Matich, P. Eng.,
Vice-President and Chief Engineer.

MAJM/dw
S7437

S7437

REPORT

TO

DEPARTMENT OF HIGHWAYS, ONTARIO

ON

SOIL CONDITIONS AND SLOPE STABILITY

HIGHWAY NO. 17 - STATION 183+00

MATTAWA ONTARIO

Distribution:

14 copies - Department of Highways, Ontario, Downsview, Ontario.

3 copies - Geocon Ltd, Rexdale, Ontario.

GEOCON

INDEX

GEOCON

INTRODUCTION

Geocon Ltd has been retained by the Department of Highways, Ontario, to investigate the soil conditions at Station 183+00 on Highway 17 near Mattawa, Ontario.

The object of the investigation was to obtain the soil conditions at the location of a former embankment failure and to reappraise the stability of this section considering the proposed new horizontal and vertical alignment. This report contains all of the soils information obtained, together with an analysis of the stability of the proposed embankment.

SUMMARIZED SOIL CONDITIONS

Boreholes 1 and 2, located on the embankment of the present road, encountered approximately 20 feet of loose to very dense rock, sand, and gravel fill. The fill overlies a thin stratum of stiff grey-brown layered silty clay and clayey silt. At the location of boreholes 1 and 2 the stratum has a thickness of 10 feet, whereas a thickness of 3 to 4 feet was encountered at the toe of the embankment. The stiff silty clay and clayey silt is underlain by a stratum of very dense grey-brown sand, gravel and boulders, then bedrock. The thickness of this stratum was 13 feet at the location of borehole 1, whereas it was not fully penetrated at the auger hole locations at the toe of the slope.

DISCUSSION

It is understood that this section of Highway No. 17, west of Mattawa was constructed around the year 1933. Construction is said to have taken place during the winter months and an embankment failure in the vicinity of station 183+00 took place shortly thereafter. Available details of the failure are very scant and it is not known if the slide was of a surficial or deep seated nature.

The failure was remedied by shifting the centreline of the road approximately 30 feet due south and by unloading and flattening the embankment slopes as shown on Drawing S7437-1. These remedial measures were considered satisfactory since no further movements were observed.

It is understood that it is proposed to change the vertical and horizontal alignment of the present road. The grade requirements indicate that approximately 10 to 11 feet of fill would be required in the section of the former failure in order to raise the road surface to the desired profile. In view of the previous history, the stability of this area is to be reappraised. The following discussions applies to the section of the roadway in the vicinity of the previous failure, i.e. at about station 183+00.

The investigation has disclosed that approximately 20 feet of granular fill material overlies a thin stratum of stiff grey-brown layered silty clay and clayey silt. This stratum has a thickness of approximately 10 feet at the borehole locations on the crest of the embankment, and approximately 3 to 4 feet at the auger hole locations at the toe of the present embankment. In a sample obtained from the lower one half of the stratum there were clayey silt layers approximately 1-1/2 inches thick and silty clay layers about 1/4 inch thick. From visual examination of the samples, it is believed that the upper part of the stratum has been desiccated, but that the lower part is unweathered and probably normally consolidated.

The strength of the layered silt and clay stratum prior to the placement of the original fill is not known. The cause of the original failure cannot therefore be definitely attributed to a deep seated failure of this stratum, although the available evidence points to a failure of this type. Since the existing grade is to be raised by 10 to 11 feet and in view of the difficulties already encountered, it is necessary to examine the new fill from the point of view of both long and short term stability.

For long term stability, it is important to know the pore pressure situation in the clay phase of the stratum. This information, used in conjunction with the shear strength parameters for the material, enables the long term stability to be computed. For the stratigraphic picture at the site pore pressures can be developed in two ways. In the first instance, they will be developed in the clay phase during construction, due to the weight of the fill. This possibility is discussed in detail later. Based on the test data, it is believed that the critical condition will occur immediately upon loading. With time, dissipation of these pore pressures will take place and the stability will increase. The second case would be the possibility of a future build up of hydrostatic pressure in the sand, gravel and boulder stratum that underlies the clay. At borehole 1, the measured elevation of the groundwater level was below the base of the clay although at an elevation of about 10 feet above the clay at the bottom of the slope. Although there is no supporting evidence available, it is believed that higher groundwater levels will exist in the Spring, than those observed at the time of the investigation. It would be a good precaution therefore, to provide a shallow ditch at the bottom of the slope extending through the clay down to the sand and gravel. This ditch would be backfilled with a pervious filter such as sand and gravel, and would assist in relieving any excess water pressure that may develop in the sand and gravel stratum at other times of the year. For the same purpose, it is recommended that the ditch on the south side of the road be lined to facilitate run-off.

An effective stress stability calculation has been carried out for the proposed roadway, assuming that with the above provisions, the hydrostatic head in the sand and gravel would not be greater than ten feet above the observed water level in the slope. The resulting computed factor of safety for these conditions was 1.4.

As far as short term stability is concerned, the most unfavourable conditions would be the case of rapid construction and no change in water content in the stratum. The measured undrained shear strength of the clayey portion of the stratum as obtained from two unconsolidated undrained tests, are about 700 and 400 pounds per square foot, respectively. Both samples appear to be disturbed however. The results of the consolidated undrained test, consolidated to approximately the existing overburden pressure, suggests that the undrained shear strength may be higher than 700 pounds per square foo. In order to obtain a further indication of the undrained strength of the clay back calculations were made on the embankment profile as originally constructed assuming a failure surface in the clay. The calculations show that on first loading the average shear strength of the clay would have to have been about 450 pounds per square foot for limiting equilibrium. The shear strength of the clay has undoubtedly increased in the past 30 years due to the consolidating effect of the existing fill. Since the stratum is fairly thin and layered it is anticipated that full consolidation of the normally loaded part of the stratum has taken place. In this case the overall shear strength of the clay stratum would have increased by about 250 pounds per square foot. It would seem reasonable therefore to assume that the average shear strength of the clay stratum at the present time is about 700 pounds per square foot. On this basis the computed factor of safety of the raised embankment considering rapid loading would be about 1.1. This is considered too low under these circumstances. For short term stability therefore the embankment would have to be constructed in stages or provided with stabilizing berms.

Because of the low overall thickness of the layered silt and clay stratum and the relatively high horizontal permeability as a result of the layered structure it is believed that consolidation would for the most part be complete within about two or three months after loading. Raising of the embankment in stages would therefore appear to be practical.

The interval required would have to be determined from observed pore pressures as measured during construction in piezometers installed in the clay stratum. At least 50 percent dissipation of pore pressures should be allowed to take place under any stage of loading prior to construction of the next stage. As calculated by the effective stress analysis, with no excess pore pressures, the computed factor of safety is 1.7. At any stage of loading therefore the factor of safety would be intermediate between the factor of safety of 1.1 mentioned above and the ultimate figure of 1.7.

The alternative to stage construction would be the use of a stabilizing berm. Preliminary computations show that in the vicinity of station 183+00, a berm at elevation 745 and 50 feet wide would be required for a safety factor of about 1.3. The resulting berm is quite large as would be expected considering the height of the new embankment, the ground configuration, and the sloping soil stratigraphy. The required berm itself would need to be keyed down to the sand and gravel stratum because of the low strength of the clay at the bottom of the slope.

The investigation of the section in the vicinity of station 183+00 has shown that there may be soil conditio between chainages 179+00 and 182+00 which would also requ re the use of special stabilizing measures in order to raise the embankment in the latter area to the proposed elevation. It is therefore recommended that the remainder of the roadway between chainages 179+00 and 182+00 be investigated before the final choice is made between the use of stage constructio and berms. In view of this, the berm requirements in the vicinity of station 183+00 has not been finalized at this time.

PERSONNEL

The field work for this investigation was carried out under the technical direction of Mr. J. F. Doherty. The report was written by Messrs. D. B. Oates and L. S. Brzezinski, checked by Mr. F. J. Heffernan and reviewed by Mr. M. A. J. Matich.

D. B. Oates

D. B. Oates, P. Eng.

REGISTERED PROFESSIONAL ENGINEER
DAVID B. OATES
PROVINCE OF ONTARIO

DBO/dw
57437

APPENDIX I

PROCEDURE

The field work for this investigation was carried out between October 24th, 1962 and November 1st, 1962 and on December 21st, 1962. Two boreholes were put down using a diamond drill rig. The boreholes were put down in BX and NX size and two inch Shelby tube samples and two inch sleeve samples were taken in the layered silty clay and clayey silt stratum. Two inch drive open samples were taken in the granular strata. Additional information on the soil conditions north of the highway was obtained from three test pits and two auger holes, the latter being used to prove the depth of the silty clay and clayey silt stratum at the toe of the embankment slope. Bedrock was proved in one of the boreholes by drilling ten feet into bedrock. A single piezometer was installed in the sand, gravel and boulder stratum.

Detailed logs of the boreholes, test pits and auger holes are presented on the Office Reports on Soil Exploration in this appendix. The location of the boreholes, test pits and auger holes, together with the inferred soil stratigraphy are shown on Drawing S7437-1 located in the pocket at the rear of the report. Also shown on the above drawing is a profile showing the proposed roadway and the existing ground level along its centre line.

The laboratory testing of soil samples was carried out in the Soil Mechanics Laboratory of Geocon Ltd in Toronto. The results are plotted on the Office Reports in this Appendix and on the figures in Appendix II. The samples remaining after testing will be stored until August 1st, 1963, at which time you will be contacted for instructions regarding their disposal.

All elevations given in this report are referred to Geodetic Datum. The Geodetic bench mark is located 60 feet right of chainage 190+77. The bench mark consists of a nail driven into a 0.4' diameter birch stump. The elevation of the bench mark is given on your profile as 808.89.

SITE AND GEOLOGY

The site investigated comprises a section of Highway 17 at chainage 183+00, about 14 miles west of Mattawa, Ontario.

From available geological information, it is believed that the site is covered by post glacial silts and clays of variable thickness underlain by dense granular deposits, then Precambrian bedrock.

SOIL CONDITIONS

The principal soil strata encountered in the borings are as follows:

Loose to Compact Brown Sand and Gravel Fill

A surface stratum of sand and gravel fill was encountered on the north shoulder of the roadway and covering much of the embankment slope. The thickness of the fill as encountered by the borehole is 3 feet.

No standard penetration tests were carried out in this stratum; however, based on the resistance to driving the casing, the relative density is believed to be loose to compact.

Compact to Very Dense Rock, Sand and Gravel Fill

The boreholes and test pits No. 2 and 3 encountered a stratum containing a large portion of rock in a sand and gravel matrix. The thickness of this stratum as observed in the boreholes was 17 and 18.5 feet.

Standard penetration tests carried out in this stratum gave "N" values of 10, 46 and 52 blows per foot. Based on these values, the relative density of the stratum is estimated to be compact to very dense. However, the high incidence of boulders is believed responsible for the high "N" values.

For design purposes an angle of shearing resistance of 35 degrees should be used together with a unit weight of 120 pounds per cubic foot.

Organic Clay

Test pit No. 3 encountered a surface layer of organic clay of 1 foot thickness. Much of the area at the toe of the embankment slope is covered by this deposit. However, the organic content is variable and the surface layer at auger hole No. 2 is described as a dark fibrous peat.

Stiff Layered Silty Clay and Clayey Silt

A stratum of layered grey brown silty clay and clayey silt underlies the rock, sand and gravel fill and the organic clay. The thickness of this stratum as encountered beneath the shoulder of the roadway ranges from 9.5 to 10 feet. At the toe of the embankment slope the thickness is 3.5 feet as was observed in auger holes No. 1 and 2 which

Stiff Layered Silty Clay and Clayey Silt (continued)

penetrated through the stratum. It is believed that the occurrence of the clayey silt layers increases in the lower half of the stratum. Two samples taken from the upper half of this stratum were found to be silty clay. The samples were layered, generally at 45 degrees, and comprised of 1-1/2 inch thick brown silty clay layers and 1/4 inch reddish brown silty clay layers. A sample taken from the lower portion of this stratum showed the grey brown clayey silt layer to be about 1-1/2 inches thick and the reddish brown silty clay layers to be 1/4 inch thick. Again the layers were inclined at about 45 degrees.

An Atterberg limit test was carried out on a sample of the silty clay and the resulting liquid and plastic limits were 43 and 26 respectively at a corresponding natural moisture content of 39 percent.

Grain size distribution tests were carried out on two samples from the lower portion of the stratum and the results of the tests are shown on Figure 1 of Appendix II. The tests were carried out on the samples as a whole including both the silty clay and clayey silt layers. As can be seen, clay sizes amounted to 19 and 36 percent respectively.

The wet unit weight of the material was found to range from 116 to 120 pounds per cubic foot, with an average value of 118 pounds per cubic foot. The corresponding moisture contents ranged from 35 to 41 percent.

Stiff Layered Silty Clay and Clayey Silt (continued)

Two undrained triaxial compression tests were carried out on samples from this stratum and the results are shown on the Office Reports in this Appendix and on Figure 2 of Appendix II. Values of compressive strength of 0.7 and 0.4 tons per square foot were obtained. The high failure strains suggests that the samples were disturbed. Standard penetration tests carried out in this stratum gave "N" values of 18 and 10. On the basis of these "N" values and from information gained from the tests described below the consistency of this stratum is believed to be generally stiff.

Three undrained triaxial compression tests with pore pressure measurements were carried out on samples of the silty clay and the results are shown on Figure 3 of Appendix II. The results show that the silty clay has an effective angle of shearing resistance of 31 degrees and in the overconsolidated range it has an effective cohesion of 200 pounds per square foot. The information gained from the test at a cell pressure approximating to the in-situ confining pressure was used to describe the consistency of this stratum.

Very Dense Grey and Brown Sand, Gravel and Boulders

Underlying the silty clay and clayey silt stratum in the boreholes and auger holes is a stratum of grey and brown sand, gravel and boulders with some silt content. This stratum was penetrated completely by borehole 1 and found to overlie bedrock. The thickness was found to be 13 feet. No information is available on the thickness of this stratum in the area of the auger holes.

Very Dense Grey and Brown Sand, Gravel and Boulders (Cont'd)

Based on the time required for the piezometer to come to equilibrium and the estimated silt content of about 10 percent, the permeability of the stratum is believed to be about $1x10^{-3}$ centimeters per second.

A standard penetration test carried out in the stratum gave an "N" value greater than 100 blows per foot. The relative density of this stratum is therefore believed to be very dense.

Bedrock

Bedrock was core drilled in borehole 1 for a depth of 10 feet. The bedrock is metamorphic and may be classified as hard sound coarse grained gneiss.

WATER CONDITIONS

A piezometer was installed in the sand, gravel and boulder stratum in borehole 1 and observations taken in this and in the open borehole during the time of investigation. The water levels are shown on the Office Reports in this Appendix.

The observed water levels were at elevation 727.2 and elevation 744.8. The former is believed representative of the groundwater table at the time of investigation while the latter water level was taken shortly after drilling was completed and is believed to be influenced by wash water.

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.
Very soft	0.03 to 0.25
Soft	0.25 to 0.5
Firm	0.5 to 1.0
Stiff	1.0 to 2.0
Very stiff	2.0 to 4.0
Hard	over 4.0

Relative Density	Standard Penetration Resistance. Blows/ft.
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	over 50

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

GEOCON

OFFICE REPORT ON SOIL EXPLORATION

CONTRACT 57457 BORING # 1 DATUM GEODETIC CASING BX & NX.
BORING DATE OCT 26, 1962 REPORT DATE JAN. 4, 1963 COMPILED BY AEL CHECKED BY LSB
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

WATER CONTENT W% ○ NAT. □ Lw △ Pw

DYNAMIC PENETRATION TEST BLOWS PER FOOT

BH # 1

ELEVN. DEPTH	DESCRIPTION
760.5 / 0.0	GROUND [illegible] LOOSE TO [illegible] BROWN SAND AND GRAVEL FILL
757.5 / 3.0	DENSE TO VERY DENSE [illegible] SAND AND GRAVEL FILL
740.5 / 20.0	GREY BROWN LAYERED SILTY CLAY AND CLAYEY SILT
730.5 / 30.0	VERY DENSE GREY AND BROWN SAND, GRAVEL AND BOULDERS
717.5 / 43.0	GNEISS BEDROCK
707.5 / 53.0	END OF HOLE

WATER CONDITIONS: OBSERVATION PIPE; BENTONITE PLUG; WL IN PIEZOMETER @ ELEV 727.2 OCT 26, 1962

ELEVATION SCALE: 760, 755, 750, 745, 740, 735, 730, 725, 720, 715, 710

OTHER TESTS	TYPE	NUMBER	PENETRATION RESISTANCE BLOWS/FT.
R.C. RECOV. 50%	AXT. R.C.	1	-
	2" DO	2	52
	DO	3	46
	DO.	4	19
	TO.	5	-
R.C. RECOV. 0%	AXT. R.C.	6	-
	DO.	7	>100
R.C. RECOV. 100%	AXT. R.C.	8	-
R.C. RECOV. 100%	AXT. R.C.	9	-

GEOCON

OFFICE REPORT ON SOIL EXPLORATION

CONTRACT [illegible] BORING # 2 DATUM GEODETIC CASING BX & NX
BORING DATE OCT 30, 1962 REPORT DATE JAN. 4. 1963 COMPILED BY A.E.L. CHECKED BY F.J.H.
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

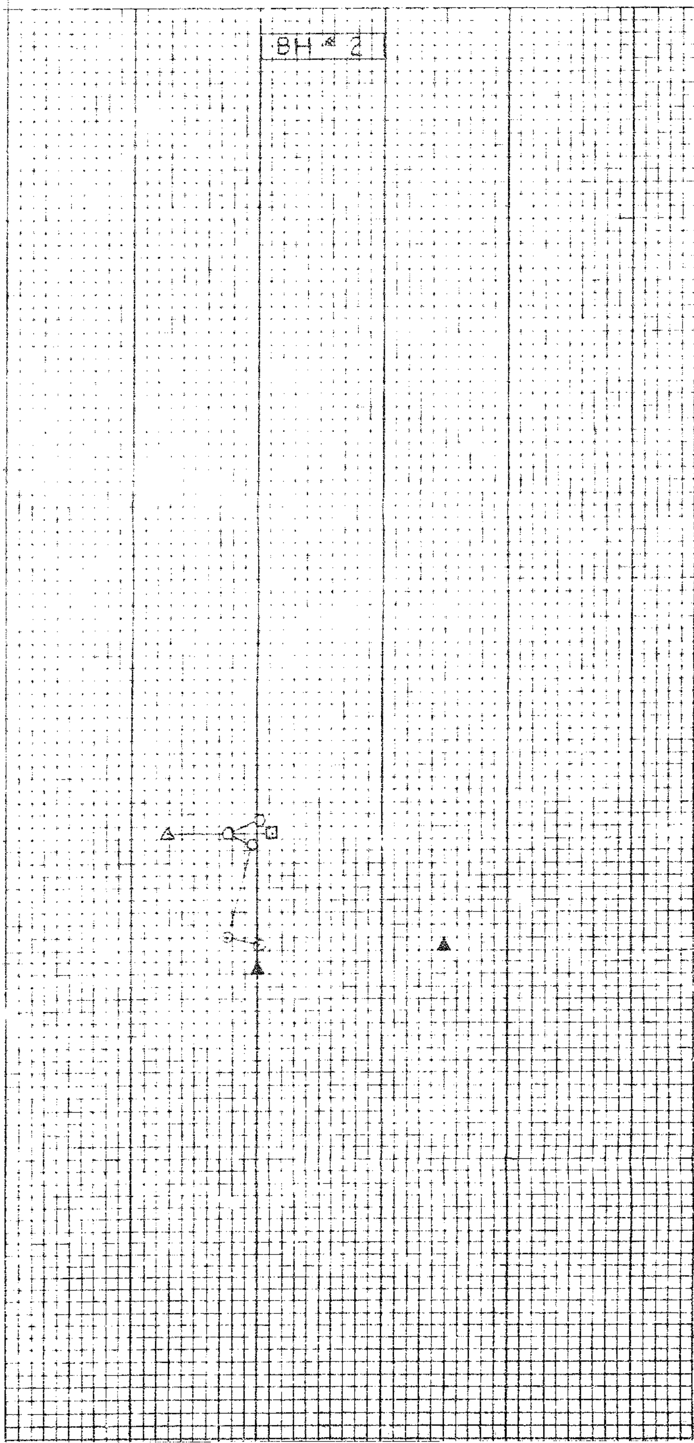
ABBREVIATIONS

- 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	ELEVATION SCALE
[illegible]		[illegible] LEVEL LOOSE TO COMPACT BROWN SAND AND GRAVEL FILL	765
[illegible] 3.5			760
		COMPACT TO DENSE BROWN SAND AND GRAVEL FILL	755
			750
744.6 21	▼		745
	W.L. ELEV. 744	GREY BROWN LAYERED SILTY CLAY AND CLAYEY SILT	740
[illegible] 31			735
		END OF HOLE VERY DENSE GREY AND BROWN SAND GRAVEL AND BOULDERS	

SHEAR STRENGTH ▲ UNDRAINED TRIAXIAL (200, 400, 600, 800, 1000)
WATER CONTENT W% ○ NAT ◻ Lw △ Pw (20, 40, 60, 80, 100)
DYNAMIC PENETRATION TEST BLOWS PER FOOT



SAMPLES

OTHER TESTS	CONDITION	TYPE	NUMBER	PENETRATION RESISTANCE BLOWS/FT.
RC RECOV. 17%		AXT RC	1	-
RC RECOV. 25%		AXT RC	2	-
		DO	3	10
		DO	4	30
		DO	5	-
Qc γ=116 Qc γ=117 Qc γ=117		TO	6	PUSH
M γ=117 γ=120		SO	7	18

GEOCON

OFFICE REPORT ON SOIL EXPLORATION

CONTRACT S 7437 [redacted] # AUGERHOLE 1 & 2. TEST PIT 1, 2 & 3. DATUM GEODETIC CASING –

BORING DATE DEC. 21, 1962 REPORT DATE JAN. 4, 1962. COMPILED BY A.E.L. CHECKED BY D.B.O.

SAMPLER HAMMER WT. – LBS. DROP – 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

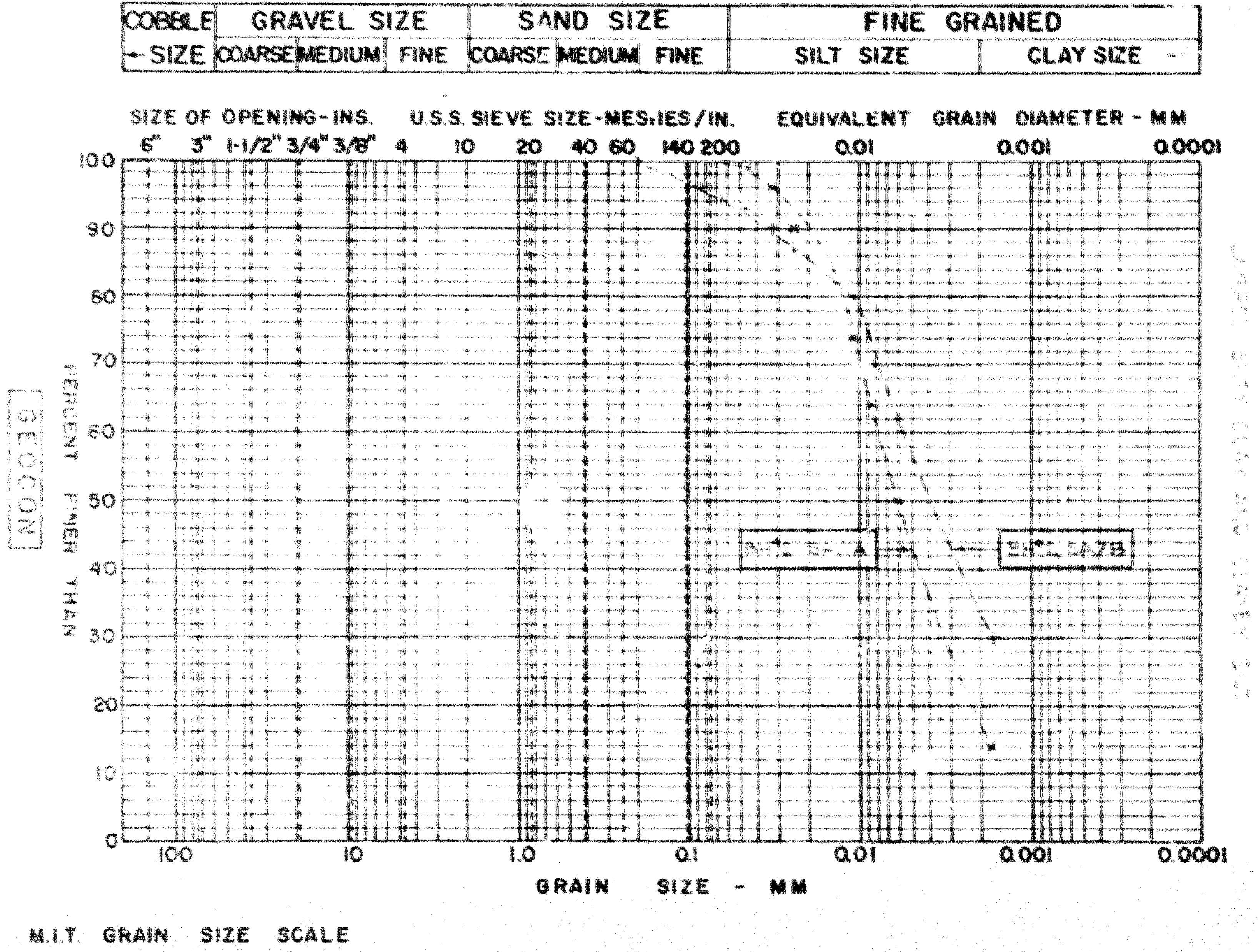
ABBREVIATIONS
- V - IN-SITU VANE TEST
- M - MECHANICAL ANALYSIS
- U - UNCONFINED COMPRESSION
- QC. - TRIAXIAL CONSOLIDATED QUICK
- Q - TRIAXIAL QUICK
- S - TRIAXIAL SLOW
- γ - WET UNIT WEIGHT
- K - PERMEABILITY
- C - CONSOLIDATION
- WL - WATER LEVEL IN CASING
- WT - WATER TABLE IN SOIL

SOIL PROFILE | WATER CONTENT W% ○ NAT. □ Lw △ Pw | DYNAMIC PENETRATION TEST BLOWS PER FOOT | OTHER TESTS | SAMPLES

ELEVN. / DEPTH	WATER CONDITIONS	DESCRIPTION	STRAT. PLOT	ELEVATION SCALE	OTHER TESTS	CONDITION	TYPE	NUMBER	PENETRATION RESISTANCE BLOWS/FT.
AH # 1									
				725					
717.5 / 0.0		GROUND LEVEL							
		GREY BROWN CLAYEY SILT		715		DISTURBED	AS.	1	–
714.0 / 3.5		REFUSAL TO AUGERING SAND, GRAVEL AND BOULDERS							
				710					
AH # 2									
715.0 / 0.0		GROUND LEVEL		715					
		DARK GREY FIBROUS PEAT							
714.0 / 1.0		GREY CLAYEY SILT				DISTURBED	AS.	1	–
710.5 / 4.5		REFUSAL TO AUGERING SAND, GRAVEL AND BOULDERS		710					
TP. # 1									
740.0 / 0.0		GROUND LEVEL		740					
		SAND AND GRAVEL FILL							
737.0 / 3.0		END OF TEST PIT.							
TP # 2				735					
733.3 / 0.0		GROUND LEVEL							
		ROCK, SAND AND GRAVEL FILL							
731.3 / 2.0		END OF TEST PIT.		730					
TP. # 3									
721.5 / 0.0		GROUND LEVEL							
		ORGANIC CLAY							
1.0		BROWN CLAYEY SILT		720					
718.5 / 3.0		END OF TEST PIT							
				715					

APPENDIX II

FIGURES - LABORATORY TESTING

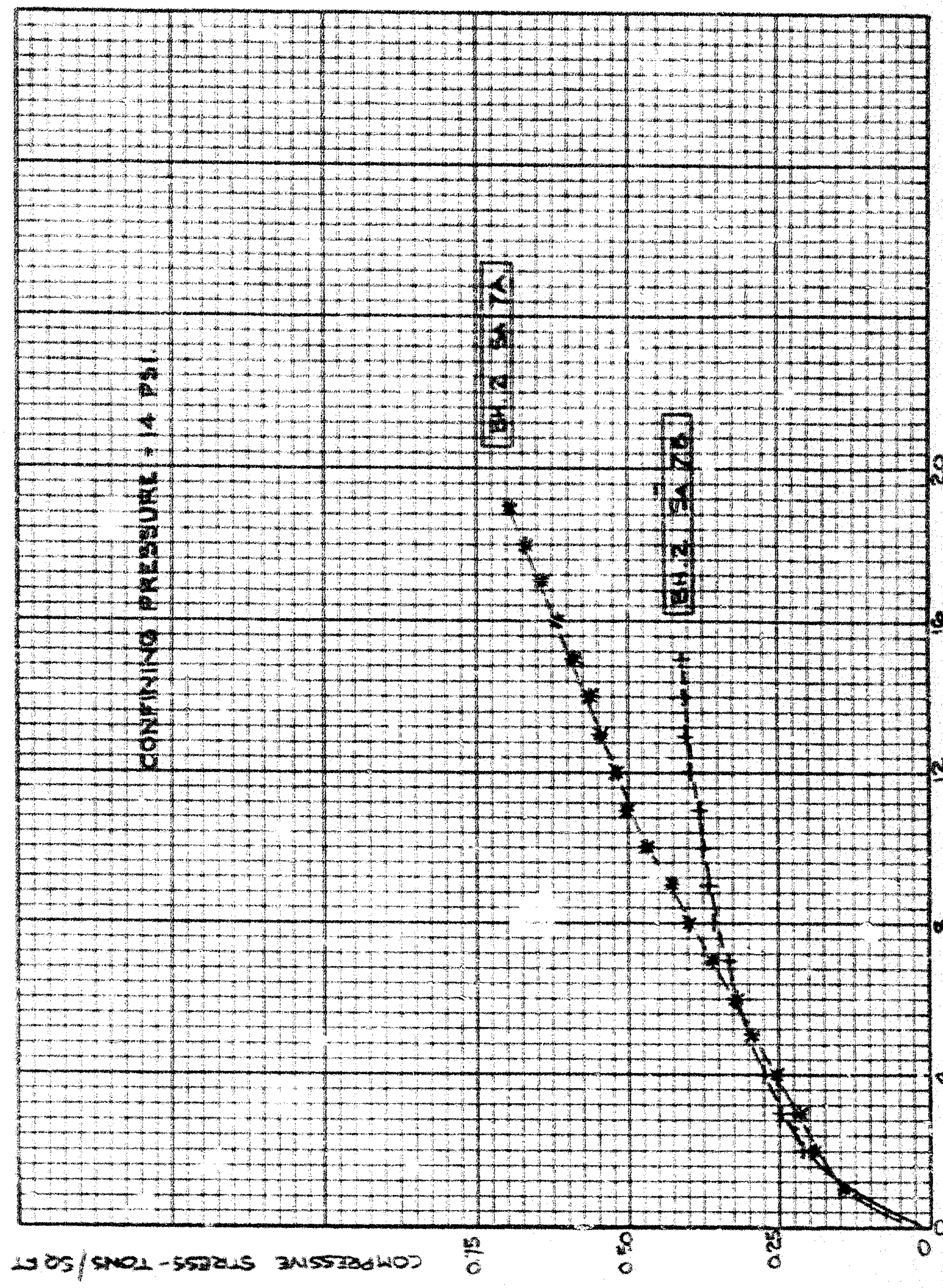


GRAIN SIZE DISTRIBUTION

APPENDIX II
FIGURE 1
PROJECT 5747

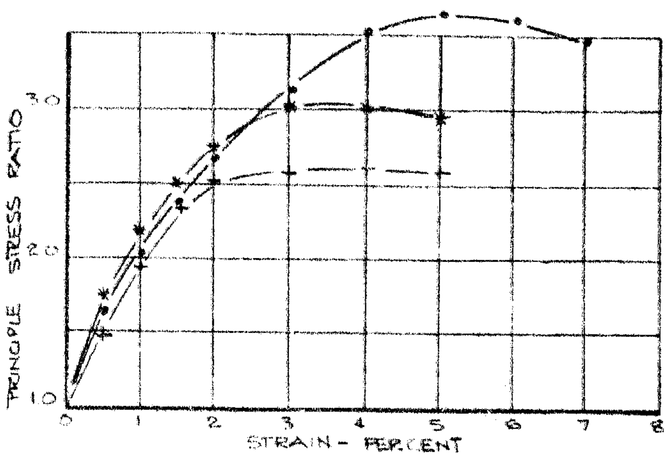
UNCONSOLIDATED UNDRAINED TRIAXIAL TESTS
STRESS - STRAIN CURVES.
GREY CLAYEY SILT WITH RED BROWN CLAY LAYERS.

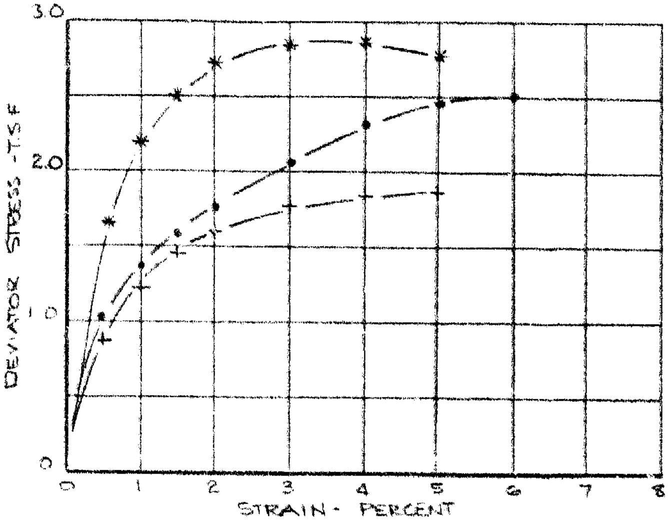
APPENDIX II
FIGURE 2
PROJECT S. 7437.

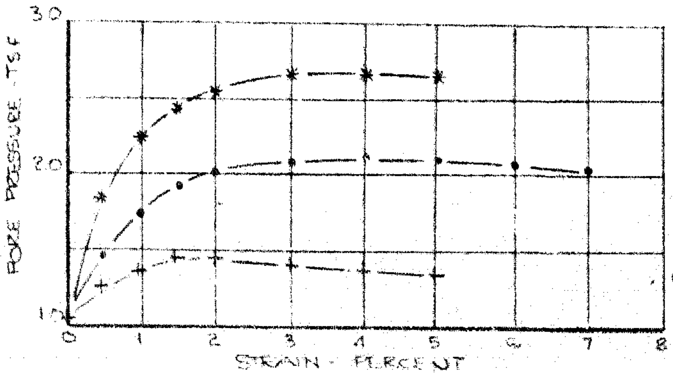


GEOCON

CONSOLIDATED UNDRAINED TESTS	PROJECT - S 7437
BROWN SILTY CLAY (BOREHOLE - 2 SAMPLE 6)	APPENDIX - II FIGURE - 3

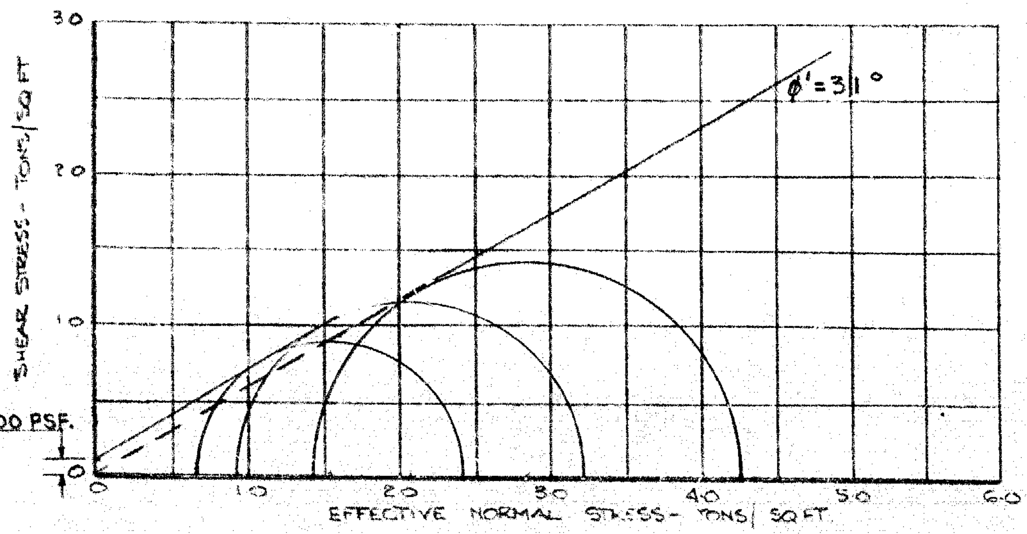


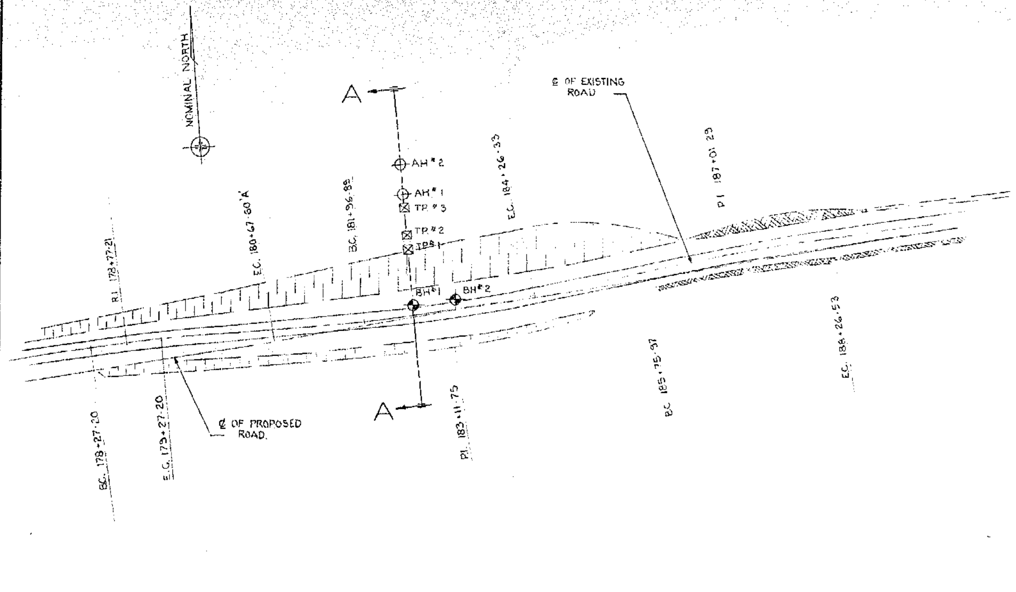




TEST #	SYMBOL	γ (PCF)	$\bar{\sigma}_3$ (TSF)	WATER CONTENT	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
4	+	117	1.02	39.3	42.6	25.9	16.7
5	•	117	2.04	35.2	-	-	-
6	*	116	3.06	40.4	-	-	-

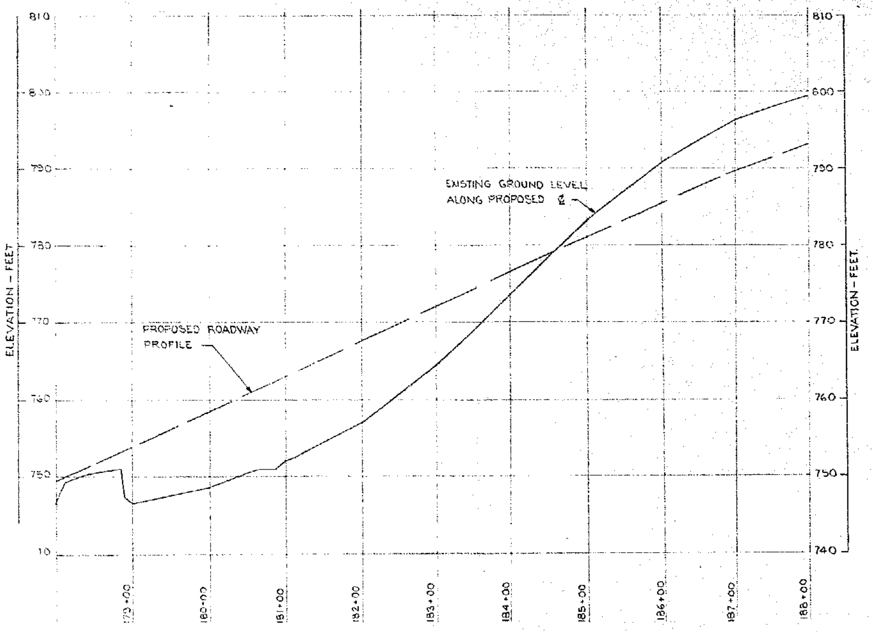
1. FILTER STRIPS - POROUS STONES TOP AND BOTTOM
2. RATE OF STRAIN - 2 % PER HOUR
3. BACK PRESSURE - 1.02 T.S.F





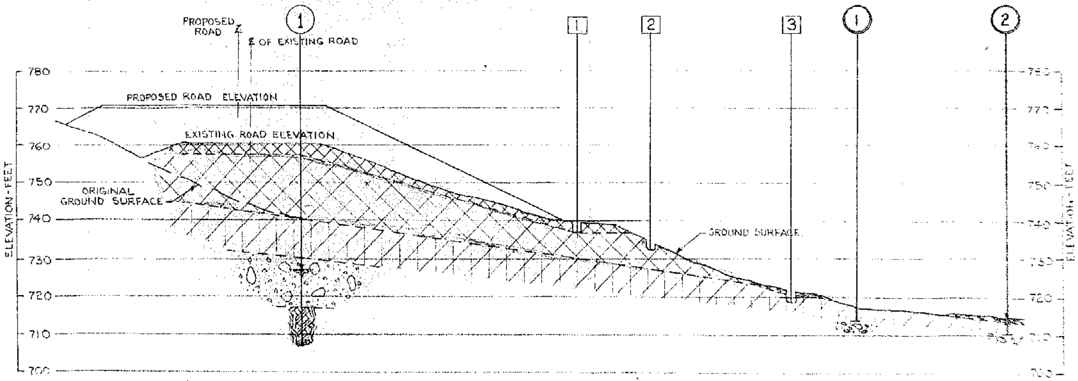
PLAN

SCALE. 1" = 100'-0"



PROFILE ALONG CENTRE LINE OF ROAD.

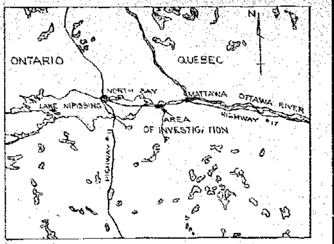
HORIZ SCALE 1" = 100'-0" — VERT. SCALE 1" = 10'-0"



SECTION A-A
STA. 182+65
HORIZ. & VERT. SCALE — 1" = 20'-0"

LEGEND

- BOREHOLE IN PLAN
- AUGERHOLE IN PLAN
- TEST PIT IN PLAN
- BOREHOLE IN ELEVATION
- AUGERHOLE IN ELEVATION
- TEST PIT IN ELEVATION
- WATER LEVEL OCT 1962



KEY PLAN
SCALE 1" = 32 M.

STRATIGRAPHY

- ORGANIC CLAY OR PEAT
- LOOSE TO COMPACT BROWN SAND AND GRAVEL FILL
- COMPACT TO VERY DENSE ROCK, SAND AND GRAVEL FILL
- GREY BROWN LAYERED SILTY CLAY AND CLAYEY SILT
- VERY DENSE GREY AND BROWN SAND, GRAVEL AND BOULDERS.
- GNEISS BEDROCK

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.

DWG. NO.	REFERENCE DESCRIPTION
—	D.H.O. DWGS. SUPPLIED
F-4075	LEVELS - GEODETIC ORIGIN ON PRESENT HWY No. 17

DEPARTMENT OF HIGHWAYS, ONTARIO
TORONTO ONTARIO
SLOPE STABILITY - HIGHWAY #17
STA. 183+00
MATTAWA ONTARIO
BORING PLAN AND SOIL STRATIGRAPHY

GEOCON LTD

DATE JAN. 11, 1963. SCALE AS SHOWN

MADE	CHKD.	APPD.	NO.
A.E.L.	F.J.H.	F.J.H.	S 7437 - 1

cc: Foundations Office (Rm. 110)

Mr. E. R. Saint,
Regional Soils Engr.,
North Bay, Ontario.

Mr. A. G. Stermac,
Principal Foundation Engr.,
Foundation Section,
Materials & Research Division.

February 4, 1963

FOUNDATION INVESTIGATION REPORT -
By Geocon, Limited, on Soil
Conditions and Slope Stability -
Hwy. #17 - Sta. 183+00, Mattawa,Ont.

Attached, we are sending you five (5) copies of the above-mentioned report for your use.

We have discussed the report with the Consultant and have advised him to get in touch with you so further action can be taken. It is our opinion that, although the investigation has answered a number of questions, it is still not conclusive, and that additional work will have to be carried out.

We are keeping the remaining copies in our files, because we believe that a distribution at this stage is undesirable.

AGS

A. G. Stermac,
PRINCIPAL FOUNDATION ENGINEER

AGS/MdeF
Encls. (5)

cc: Foundations Office
Gen. Files.

Note :- The additional work which was recommended in the final report was [illegible] [illegible] [illegible] a change of alignment [illegible] [illegible] [illegible] E. [illegible] [illegible]
March 17th 1964

[illegible]

Materials and Research Division

February 8, 1963

Geocon, Limited,
Consulting Engineers,
14 Haas Road,
Rexdale, Ontario.

Attention: Mr. F. Heffernan

Re: W.P. 68-60-1, Hwy. #17,
Rutherglen - Eau Claire, Sta. 183+00,
District #13, North Bay, Ontario.

Dear Sir:-

Please consider this as confirmation of our earlier verbal agreement, and as your formal authority to have carried out an embankment stability investigation at the above site.

Charges for the work performed will be in accordance with your Schedule of Rates, dated February 17, 1959.

Yours very truly,

A. Rutka,
MATERIALS & RESEARCH ENGINEER

ESS/McF

cc: Messrs. H. D. Smith (2)
E. R. Saint
Mrs. T. Tate
Foundations Office
Gen. Files (2)

Materials and Research Division

February 8, 1963

Geocon, Limited,
Consulting Engineers,
14 Haas Road,
Rexdale, Ontario.

Attention: Mr. F. Heffernan

Re: W.P. 68-60-1, B Hwy. #17,
Rutherglen - Eau Claire, Sta. 183+00,
District #13, North Bay, Ontario.

Dear Sir:-

Please consider this as confirmation of our earlier verbal agreement, and as your formal authority to have carried out an embankment stability investigation at the above site.

Charges for the work performed will be in accordance with your Schedule of Rates, dated February 17, 1959.

Yours very truly,

A. Rutka,
MATERIALS & RESEARCH ENGINEER

AR/MdeF

cc: Messrs. R. D. Smith (2)
E. F. Saint
Mrs. I. Tate
Foundations Office
Gen. Files (2)