

#60-F-238

W.P. #903-60

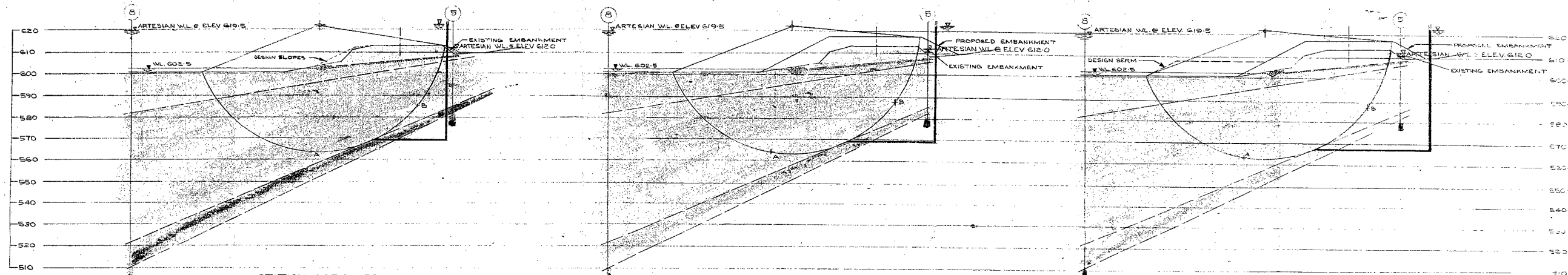
Hwy. #17

PROP. GRADE

ADJUSTMENT

SAULT STE.

MARIE



STATION 2054+50
EXISTING HIGHWAY EMBANKMENT

FACTORS OF SAFETY

SIMPLIFIED METHOD

RIGOROUS METHOD

FS: 1.04

FS: 1.04

FS: 1.04

STATION 2054+50
PROPOSED HIGHWAY EMBANKMENT

FACTORS OF SAFETY

CONSTRUCTION CASE (B=1)

LONG TERM CASE

FS: 1.02

FS: 0.79

FS: 0.77

FS: 1.01

STATION 2054+50
PROPOSED HIGHWAY EMBANKMENT PLUS BERM

FACTORS OF SAFETY

CONSTRUCTION CASE (B=1)

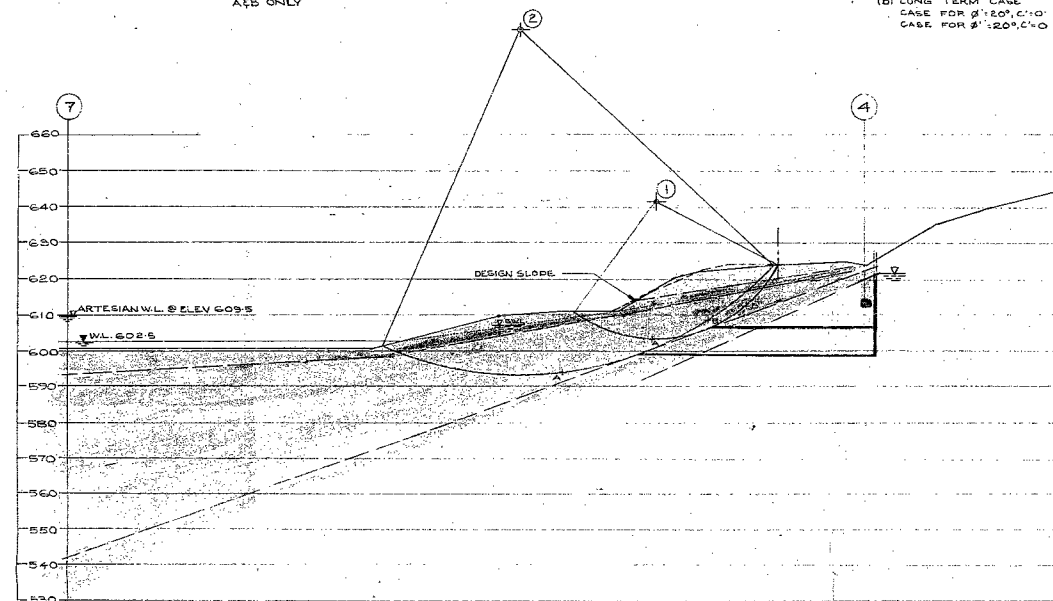
LONG TERM CASE

FS: 1.30

FS: 0.97

FS: 1.35

FS: 1.71



STATION 2056+60
EXISTING HIGHWAY EMBANKMENT

FACTORS OF SAFETY

CIRCLE 1

CIRCLE 2

FS: 0.61

FS: 1.09

FS: 1.10

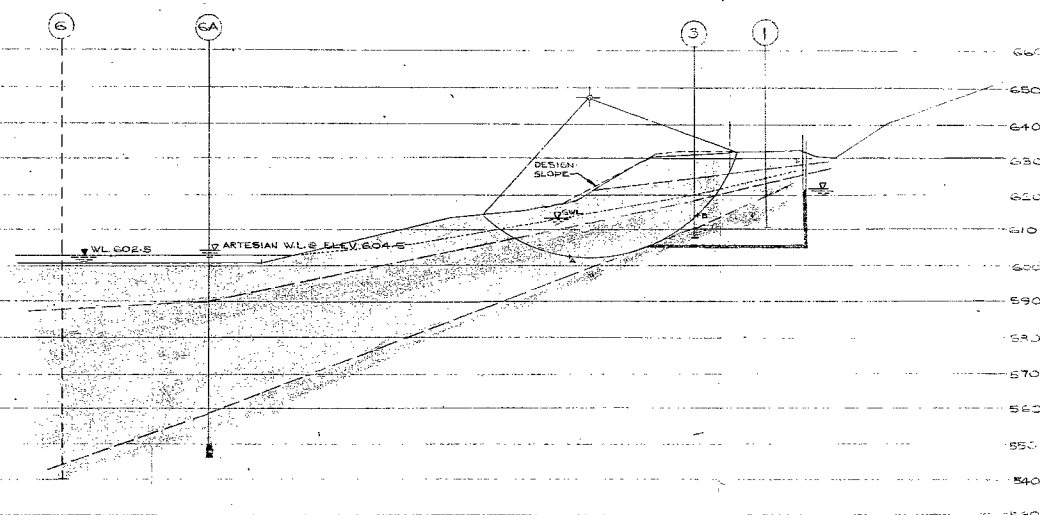
FS: 1.42

FS: 0.42

FS: 0.91

FS: 1.39

FS: 1.69



STATION 2058+00
EXISTING HIGHWAY EMBANKMENT

FACTORS OF SAFETY

CIRCLE 1

CIRCLE 2

FS: 1.06

FS: 1.25

FS: 1.37

FS: 1.80

DESIGN DATA

STRATA	γ	γ'	φ	c
LBS/CU.FT.	LBS/CU.FT.	DEGREE	LBS/CU.FT.	
GRANULAR FILL	130	67	30	—
CLAYEY SILT TO SILTY SAND WITH GRAVEL	120	57	25	—
VARVED SILTY CLAY	105	43	20	200
SANDY GRAVEL	135	73	35	—

LEGEND

- 3 BOREHOLE IN ELEVATION
- 6 PENETRATION TEST IN ELEVATION

STRATIGRAPHY

- COMPACT GRANULAR FILL
- VERY LOOSE TO LOOSE CLAYEY SILT TO COMPACT SILTY SAND AND GRAVEL
- SOFT TO STIFF REDDISH BROWN VARVED SILTY CLAY
- DENSE GRAY SANDY GRAVEL WITH BOULDERS
- BEDROCK

Mr. D. G. Ramsay,
Road Design Engineer.

Materials & Research Section.

December 21, 1960.

FOUNDATION INVESTIGATION REPORT

by: Geoccon, Limited.

Attention: Mr. H. D. McMillan.

Re: Soil Conditions and Engineering Study
Proposed Grade Adjustment, Hwy. #17,
Sault Ste. Marie, Ontario, W.P. 903-60.

Attached to this memo, we are forwarding to you the above mentioned report submitted by the Consultant, Geoccon, Ltd., Toronto. We have reviewed the report and believe that the presented data, discussion and recommendations will prove to be adequate for your future design work.

Should there, during your further work on this project, arise any questions you would like to discuss, please feel free to call on our Office.

L. G. Soderman,
PRINCIPAL FOUNDATIONS ENGR.

Per:



(A. G. Sternac,
FOUNDATIONS OFFICE ENGR.)

AGS/MBEF
Attach.

cc: Messrs. H. A. Tregaskes

A. Mantle

A. M. Tove

C. K. Hunter

D. P. Collins

H. R. Saint

A. Watt

Foundations Office ✓

Gen. Files.

GEOCON LTD

HEAD OFFICE

180 VALLÉE ST., MONTREAL 19, QUEBEC

TELEPHONE UN. 5-7632

Rexdale, Ontario,
December 9th, 1960.

DISTRICT OFFICES

14 HAAS ROAD
REXDALE, TORONTO, ONT.
TEL. CH. 4-8641

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

Department of Highways, Ontario,
Downsview, Ontario.

Attention: Mr. L. G. Soderman, P. Eng.,
Principal Foundation Engineer.

Re: Soil Conditions and Engineering Study,
Proposed Grade Adjustment, Highway 17,
Sault Ste. Marie, Ontario,
W.P. 918-59. 903-60

Dear Sirs:

This letter accompanies our detailed engineering report on the investigation and studies carried out at the above site for the proposed grade adjustment.

We find that the existing embankment is underlain by a thin stratum of silty sand and gravel which increases in thickness offshore to a depth of 18 feet and grades into a clayey silt. This stratum is underlain by firm to stiff varved silty clay which also increases in thickness offshore to a maximum of 60 feet. A two to fourteen foot thick stratum of sandy gravel underlies the clay and is in turn underlain by bedrock. Artesian pressures were measured in the lower gravel stratum, being a maximum of 17 feet above lake level in borehole 8.

The proposed embankment additions may only be constructed if berms are provided, to ensure an adequate factor of safety for long term stability. It is suggested that the construction of the berms and the raising of the grade be carried out as two separate operations, as discussed. An alternative solution for lowering the grade could be considered, where the highway would be cut beyond chainage 2059+00. Our studies show that the stability of the existing embankments up to chainage 2059+00 is adequate and this portion of the alignment need not be altered if the alternative involving cutting is adopted.

We believe that this report contains all the information required for safe and economical embankment design. If we can be of any further service, however, please do not hesitate to contact us.

Yours very truly,

GEOCON LTD


J. C. Osler, P. Eng.,
Division Engineer.

JCO/dw
S7127

ST. JOHN'S

HALIFAX

MONTREAL

TORONTO

VANCOUVER

S7127

REPORT

TO

DEPARTMENT OF HIGHWAYS, ONTARIO

ON

SOIL CONDITIONS AND ENGINEERING STUDY

EXISTING AND PROPOSED HIGHWAY 17

W.P. 918-59 903-68

SAULT STE. MARIE

ONTARIO

Distribution:

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

2 copies - Geocon Ltd,
Rexdale, Ontario.

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INDEX

	<u>Page</u>
Introduction	1
Summarized Soil Conditions	1
Discussion	2
Conclusions and Recommendations	6
Personnel	7
Appendix I	
Procedure	
Site and Geology	
Soil Conditions	
Water Conditions	
Office Reports on Soil Exploration	
Appendix II	
Figures - Laboratory Testing	
Appendix III	
Drawing S7127-3 Recommended Berm Sizes For Proposed Embankments	
Drawings in Pocket at Rear:	
S7127-1 Boring Plan and Soil Stratigraphy	
S7127-2 Summarized Stability Analyses, Existing and Proposed Embankments	

INTRODUCTION

Geocon Ltd has been retained by the Department of Highways, Ontario, by letter dated August 19th, 1960 to investigate and report on the soil conditions as they affect the stability of the existing and proposed highway embankments between stations 2054+00 and 2059+00 on Highway 17 adjacent to the shore of Batchewana Bay, near Sault Ste. Marie.

The object of the investigation was to determine the soil strata and interpret their properties, and to carry out detailed engineering studies in order to assess the stability of the existing embankments and to make recommendations for the proposed grade revisions.

A description of the procedure, site and geology, and detailed accounts of the soil and water conditions are given in Appendix I of this report. The results of the in-situ and laboratory testing are shown on the Office Reports on Soil Exploration in Appendix I and on the Figures in Appendix II. The locations of the boreholes and the inferred soil stratigraphy are given on Drawing S7127-1 at the rear of this report.

SUMMARIZED SOIL CONDITIONS

The existing highway embankment consists of a compact to dense granular fill. The embankment generally increases in thickness from north to south, the depth of fill ranging from 1 to 6.5 feet. The soil strata underlying the road grade dip to the south, and generally increase in thickness offshore. A stratum of loose clayey to sandy silt underlies the lake bed and has a maximum thickness of 18 feet. The stratum decreases in thickness to less than 1 foot, and contains a considerable amount of gravel, to the north of the site and underlying the highway embankment. A stratum of soft to firm varved silty clay underlies the clayey to sandy silt. The upper and lower boundaries of the

varved silty clay both dip to the south. The stratum does not exist beneath the north side of the embankment in the rock cut section west of station 2056+00. To the east of this point and to the south it increases in thickness to more than 60 feet at the boreholes located furthest offshore. A sandy gravel and boulder stratum which ranges in thickness from 2 to 14 feet underlies the above strata and overlies bedrock. An artesian pressure was encountered in the sandy gravel and boulder stratum, with the maximum head being at elevation 619.5 feet, or 17 feet above lake water level.

DISCUSSION

It is understood that it is proposed to reduce the grade of Highway 17 between stations 2052+00 and 2060+00 at Batchewana Bay, near Sault Ste. Marie. As planned, this would be effected by the placing of varying amounts of fill, with the maximum being about 4 feet between stations 2054+00 and 2056+00. The considered revisions would not alter the alignment of the highway.

The existing highway has been in service for approximately 15 years. During this period minor repairs have been required to service the highway; however, no major failures have been experienced. The section of the highway under consideration has recently been resurfaced, prior to this investigation.

A. General

The results of preliminary calculations were discussed at your office at a meeting in late October. These computations indicated that the embankments in the vicinity of station 2055+00 could not be raised as proposed without the addition of stabilizing berms. Because of this, alternative methods of reducing the grade were discussed. One such method would be to lower the highway surface beyond station 2059+00,

A. General (continued)

thereby not increasing the height of embankments around station 2055+00. From available information, it is inferred that some of the cut would be in rock. A second possibility discussed was a change in the alignment but it was agreed that this would be impractical at this site. The decision as to whether the grade is to be altered by cutting at the top or by filling at the bottom would be based on economic considerations which are beyond the scope of this report. In this report, consideration is only given to the alternative of placing fill at the bottom of the slope. The studies which were carried out included the assessment of the stability of the existing sections, together with the design of the proposed additions to the embankments, including the necessary berms.

The design values used in the stability analyses are shown on Drawing S7127-2 and are discussed in detail under "Soil Conditions". As discussed the significant stratum at the site is a varved silty clay. However, the clay is not distinctly varved, having only thin, irregular silt layers, and is of generally high plasticity. It is therefore considered that some effective cohesion could be assumed in effective stress analyses involving this stratum. The amount of cohesion which might be assumed in various analyses would depend on the geometry of the problem and on the time factor being considered in the design. The stratification of the clay is generally parallel to the lower stratum boundary. Thus, it is considered that a conservative assumption for employing cohesion in long term effective stress analyses would be to disregard cohesion along portions of the failure arcs which lie approximately parallel to the stratification, thus considering cohesion only where the circles cut across stratifications. This assumption was made in the calculations of the stability of the existing embankments, with the cohesion being omitted between points A and B as shown on the circles presented in Drawing S7127-2. No cohesion was assumed in the

A. General (continued)

computations of long term stability of the proposed embankment additions, for reasons which are discussed below.

The maximum artesian pressures at the site were encountered in boreholes 5 and 8, at station 2054+50. Here the maximum observed artesian head was at elevation 619. For the calculations, an artesian head 3 feet higher than this observed head was employed, to allow for possible higher pressures during periods of heavy runoff. Elsewhere at the site, the observed artesian pressures were variable and fell off almost to zero at borehole 6A. This fact was substantiated by an observed difference in the undrained shear strength profile, as discussed under "Soil Conditions". In the calculations, limiting conditions of the artesian pressures were therefore assumed for two analytical approaches, varying from a head at elevation 622 across the entire site, to the actual artesian head measured at boreholes 6A and 7. With this latter assumption and considering the upslope location of the critical circles at these sections, the effect of the artesian head on stability was almost entirely eliminated. An examination of the results obtained using these two limiting assumptions of artesian pressure at the typical sections at stations 2056+60 and 2058+00 indicate that the stability of the existing embankment under the maximum assumed artesian head would not be acceptable. Since the performance of the embankment over the past 15 years would indicate otherwise, the results of these calculations are considered as further evidence that the maximum artesian pressure observed at station 2054+50 is not effective across the entire site. Therefore, in the design of the proposed embankment additions, the artesian pressures have been assumed to vary across the site as observed in the boreholes. It is considered that the actual artesian pressures at the sections might fall somewhere between the limits mentioned above, but would probably be near the lower limit.

A. General (continued)

Consideration was given to the use of relief wells to reduce the artesian pressure in the critical areas. However, because of the possibility of silting up, it would be difficult to make such an installation permanently effective, and in any case, this solution would probably not be economic.

Pore water pressures will be set up in the varved silty clay stratum during the placing of additional fill for the embankments or berms. For design purposes, the ratio of the increase in pore pressure to the increase in effective major principal stress, \bar{B} , has been taken as 1. Therefore, any additional load or increase of effective pressure will set up a pore pressure equal to the applied load. The pore water pressures will dissipate with time, but for the short term or construction case of stability, the maximum pore pressures computed from the parameter \bar{B} , equal to 1, will be acting. However, due to the homogeneous nature of the clay, it is considered that up to the full effective cohesion could be used in assessing the stability for the after construction case by effective stress analyses.

B. Results

Calculations of the long term stability of the proposed embankments, using effective stress analyses show that the embankment would not be stable if plane side-slopes of 2 horizontal to 1 vertical were used. Thus stabilizing berms are necessary. The criterion used for design of the proposed highway embankments and berms was a factor of safety of 1.3, using no effective cohesion. This assumption of no cohesion in such a relatively homogeneous clay is considered to be conservative, but is offset by the uncertainties associated with the assumption of artesian conditions across the site, as discussed above. Three typical cross-sections showing recommended berms are given on Drawing S7127-3.

B. Results (continued)

Computations of the after construction stability, considering induced pore pressures and full effective cohesion show that the factors of safety of the berms necessary for long term stability, when constructed as a unit with the increased height of embankment, are in the order of 1.2 to 1.4. However, the factors of safety for this situation are only about 1.0 for the consideration of partial cohesion along the circular arcs, i.e. no cohesion between points A and B on circles on Drawing S7127-2. If this latter assumption is valid, as may well be the case, construction of the berm immediately prior to the raising of the embankment would involve some risk. The risk could be eliminated by enlarging the berms but a more practical approach would appear to be to allow a gap in the construction sequence, between the building of the berm and the raising of the embankment. The recommended length of time would be one construction season.

As can be seen from Drawing S7127-2, the stability of the three existing typical sections is adequate, even allowing for slope improvements to accepted standards. Thus if the alternative scheme of a cut section beyond station 2059+00 is adopted the existing embankment sections need not be altered. Disposal of the cut material should be away from any critical sections as defined herein.

CONCLUSIONS AND RECOMMENDATIONS

1. The critical stratum at the site is a firm to stiff varved clay which increases in thickness in an offshore direction. The clay is overlain and underlain by strata of clayey to sandy silt and sand and gravel.

CONCLUSIONS AND RECOMMENDATIONS (continued)

7.

2. A maximum artesian pressure at elevation 619.5 was observed while elsewhere the excess head was only several feet above lake level.

3. Stability calculations show that berms are required to ensure adequate long term stability of the proposed embankment additions.

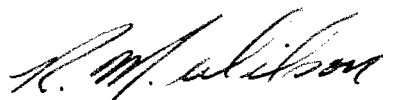
4. Because of pore pressures induced during construction, it is desirable to construct the berm separately from the raising of the roadway grade, as discussed in the report.

5. An alternative scheme to lower the grade by cutting beyond station 2059+00 could be adopted. Studies indicate the stability of the existing embankments is adequate and so they could be left unaltered with perhaps minor adjustments to the slopes to conform to accepted standards.

PERSONNEL

The field work was carried out under the supervision of Mr. R. M. Quigley and Mr. R. Gibson. This report was written by Mr. R. M. Wilson, checked by Mr. J. L. Seychuk and reviewed by Mr. J. C. Osler.

RMW/dw
S7127



R. M. Wilson,
Senior Soils Engineer

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APPENDIX I

Procedure

Site and Geology

Soil Conditions

Water Conditions

Office Reports on Soil Exploration

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PROCEDURE

The field work for this contract was carried out between August 29th and September 23rd, 1960 and included borings 4 to 8 inclusive. Boreholes 1 and 2 were put down prior to the above dates and were supervised by Department of Highways, Ontario personnel. A third borehole was put down by the Department of Highways in July, 1960 with this work being supervised by Geocon Ltd.

All boreholes were put down in NX and BX size using a skid-mounted machine drillrig supplied and operated by a local contractor working under the technical supervision of Geocon Ltd. Boreholes 6, 7, and 8 and the accompanying dynamic penetration tests, which are located offshore, were completed from a drum raft. Two inch Shelby tube samples were taken in the silty clay strata and two-inch drive samples were obtained in the granular strata. In-situ shear strength determinations of the silty clay were carried out in boreholes 5, 6A, 7 and 8, using a strain-controlled vane apparatus.

The locations of all boreholes and the inferred soil stratigraphy are given on Drawing S7127-1, contained in the pocket at the rear of this report. Detailed logs of the boreholes are given on the Office Reports on Soil Exploration in this Appendix.

The laboratory testing was carried out in the Soil Mechanics Laboratory of Geocon Ltd in Toronto. The test results are shown on the Office Reports in this Appendix and are given on the Figures in Appendix II. The samples remaining after testing will be stored until May 1st, 1961, at which time you will be contacted regarding their disposal.

All elevations given in this report are referred to Geodetic Datum and were determined from a Department of Highways, Ontario bench mark. The bench mark is located on the top of a 0.3 foot high maple stump, 32 feet left of centreline station 2049+39 with a given elevation

of 612.53 feet in accordance with Department of Highways, Ontario reference map No. C1595. A discrepancy of 2.0 feet was noted in comparison with the water level of Lake Superior during the period of the investigation and therefore 2.0 feet have been subtracted from all elevations obtained in the field.

SITE AND GEOLOGY

The site investigated is located between centreline stations 2054+00 and 2059+00 on Highway 17, approximately 50 miles northwest of Sault Ste. Marie, Ontario. The existing highway embankment is located immediately adjacent to the shore line of Batchewana Bay on the north shore of Lake Superior.

From available geological information and the results of previous work carried out in this area, it is known that the shore at the site is overlain by granular talus while the lake bottom is covered with silt, sand and gravel strata. These strata overlie a varved silty clay, which is followed by a sand and gravel stratum. Bedrock underlies the sand and gravel and consists of reddish granitic rock of the Archean age.

SOIL CONDITIONS

The principal soil strata encountered by the boreholes are as follows:

Topsoil

A thin layer of sandy clayey topsoil was encountered in borehole 5 for a depth of 1.3 feet. The topsoil contains roots and organic matter in a matrix of clayey sand. Topsoil was not encountered in the remaining onshore boreholes and has probably been removed during construction operations.

Fill

The fill of the existing highway embankment was penetrated by boreholes 1 and 3 only, which were put down on the shoulders of the highway. The fill is a grey-brown silty sandy gravel. The embankment thickness varies from 1 to 6.5 feet and generally increases in thickness from north to south, reflecting the offshore slope of the underlying natural strata.

Although the boreholes were put through the fill without sampling, the relative density is estimated as compact to dense. The wet unit weight and angle of internal friction have been assumed as 130 pounds per cubic foot and 30 degrees respectively for computation purposes.

Very Loose to Compact Clayey Silt to Silty Sand with Gravel

A stratum of loose to compact grey clayey silt to silty sand which has a variable gravel content underlies the topsoil and embankment fill and comprises the beach and the lake bottom at the site. The stratum was encountered in all boreholes except number 4 and varies in thickness from less than 1 foot where it occurs adjacent to the rock cut north of the highway embankment, to greater than 17 feet as it increases in thickness offshore. The composition of the stratum varies both horizontally and vertically. The sand and gravel content, which is most pronounced underlying and north of the highway, decreases in the offshore areas, where the stratum becomes a clayey silt. As encountered in borehole 3 the stratum is a compact silty sand and gravel and in borehole 5, it consists only of gravel and boulders, contained in a silt matrix. The high gravel content in these boreholes is probably due to the combination of talus deposits with the shoreline silt and sand deposits. In borehole 8 only, the upper 5 feet of the stratum consists entirely of fine to medium sand sizes.

Very Loose to Compact Clayey Silt to Silty Sand with Gravel (cont'd)

Atterberg limit determinations carried out on typical samples of the clayey silt show the liquid limit to have an average value of 36 percent and the plasticity index to range from 10 to 22, generally increasing with depth. The natural moisture content of the clayey to sandy silt varies from 32 to 42 percent. The above test results are shown on the Office Reports and are plotted on the plasticity chart on Figure 2 of Appendix II. The results of a grain size analysis carried out on a sample of clayey silt are plotted on Figure 1 of Appendix II and show this portion of the stratum to be composed of about 50 percent silt sizes and 30 percent clay sizes.

The results of a quick triaxial compression test carried out on a typical clayey silt sample gave a shear strength of 725 pounds per square foot.

Standard penetration resistances, or "N" values, varying from 1 to 16 blows per foot and generally less than 10 blows per foot were obtained in the stratum. Many of the samples were taken by levering the sampler down and therefore "N" values were not obtained. The penetration resistance of such samples has been shown as "push" on the Office Reports. It is believed that the higher "N" values obtained were due to the presence of gravel sizes and therefore the stratum is generally loose, except in boreholes 3 and 5 where it is compact to dense.

For computation purposes the stratum has been assumed to have saturated and submerged unit weights of 120 and 57 pounds per cubic foot respectively. An angle of internal friction of 25 degrees was considered for design.

Soft to Stiff Varved Silty Clay

A stratum of varved red-brown silty clay was encountered underlying the above strata in all boreholes. The stratum does not occur to the north of the highway embankment, to the west of station 2056+00, while to the south it increases to a maximum thickness of greater than 60 feet at offshore locations. The upper and lower strata boundaries dip to the south. The stratum has been deposited in successive layers approximately parallel to the lower boundary and at a slope of about 20 degrees to the horizontal. The clay layers are red-brown in colour and the colour of the silt layers is grey. The clay layers predominate, having a thickness of $1/4$ to $1/2$ inch, while the silt layers are generally less than $1/16$ inch in thickness and often are only several grains thick, being only noticeable after partial drying of the samples. Occasionally silt layers up to $1/4$ inch thick were encountered. Where the stratum is thinnest, near to and beneath the road embankment, the orientation of varves is more nearly horizontal. Here the varves are poorly developed, but the clay is still distinctly laminated. In some samples, extreme distortions of the varves were observed, immediately adjacent to uncontorted varves both above and below, suggesting the stratum has undergone movement in its past geologic history.

Atterberg limit determinations carried out on typical samples of the silty clay gave average liquid and plastic limits of 74 and 25 respectively. The mean plasticity index is 50. The liquidity index has an average value of 0.7. The results of the tests are given on the Office Reports and plotted on Figure 2 of Appendix II. Limits carried out on individual clay and silt laminae are also shown on the logs and the above figure. The liquid and plastic

Soft to Stiff Varved Silty Clay (continued)

limits and natural moisture content for the clay laminae averaged 80, 25 and 53 percent respectively. Similar values in the silt laminae were 30, 20 and 23 percent respectively.

Wet unit weight determinations for the stratum gave an average value of 105 pounds per cubic foot, based on a range of values from 97 to 116 pounds per cubic foot. The corresponding average submerged unit weight is 43 pounds per cubic foot. The natural moisture content varies from 32 to 67 percent with an average value of 53 percent, excluding the value of 23 obtained in a silt layer as noted above.

The shear strength of the stratum was determined by in-situ vane shear tests and undrained triaxial compression tests. Typical stress-strain curves obtained from three undrained quick triaxial tests are given on Figure 5 of Appendix II. The results of the testing showing the variation of shear strength with elevation are given on Figures 3 and 4 of Appendix II. The shear strengths obtained from the in-situ vane tests are generally considerably higher than those obtained in the laboratory. The results of remoulded vane tests gave values of sensitivity ranging from 3 to 15 and averaging 7. Thus the clay is sensitive and the lower strength values obtained from the triaxial tests is probably due to sample disturbance. The shear strength profile obtained from the vane tests is therefore considered to give a better indication of the undrained strength.

As shown on Figure 3 of Appendix II the shear strength increases from an average value of 800 pounds per square foot at elevation 585 to 1600 pounds per square foot at elevation 530. It is known from information obtained in previous investigations in

Soft to Stiff Varved Silty Clay (continued)

this area that the clay has been precompressed, with precompression loads of about 3 tons per square foot being measured above elevation 580. It is also known that the clays underlying Lake Superior show evidence of desiccation extending to about elevation 575, possibly explaining the large precompression loads measured above this level. On Figure 3, an approximate trend line has been inferred from the vane test results, using a value of $c/p = 0.3$, and assuming some desiccation effect to about elevation 585. This trend line is based largely on the results of borehole 8 and the value of overburden pressure used in computing the c/p ratio is not reduced to account for the artesian pressure observed at this hole. When the overburden pressure is reduced to account for a 20 foot artesian head at this point the resulting c/p ratio is 0.6, believed to be a representative value for a fresh water clay. The shear strength results of boreholes 6 and 7, also plotted on Figure 3, suggest a steeper trend line than the one shown, with the results of borehole 6 in particular suggesting a line of about twice the slope of that drawn. As noted under "Water Conditions", almost no artesian pressure was observed in this hole and it would appear that the greater buildup in strength with depth at this hole could result from this fact. Based on the complete range of shear strengths obtained, varying from 300 to 2000 pounds per square foot, the consistency of the stratum is described as soft to stiff; based on the above trend line it is firm to stiff.

Four consolidated undrained triaxial tests with pore pressure measurements were carried out on typical samples of the silty clay. The results of the tests are given on the effective stress circles on Figure 6 of Appendix II. The effective shear strength parameters, ϕ' and C' , obtained from the tests fall within a range of 16 to 23

Soft to Stiff Varved Silty Clay (continued)

degrees for the effective angle of shearing resistance and 150 to 400 pounds per square foot for the effective cohesion. From these results, the average parameters selected were 20 degrees for the effective angle of shearing resistance, ϕ' , and 200 pounds per square foot for the effective cohesion, C' . These values agree with other data obtained in the area. Also, based on experience, it is known that for an average plasticity index of 50, ϕ_d should be of the order of 21.5 degrees, which checks well with the measured ϕ' .

Dense Sandy Gravel

The silty clay is underlain by a stratum of grey sandy gravel which contains many boulders, up to 20 inches in size. The stratum which was encountered in all boreholes varies in thickness from 2 to 14 feet. Except for a two foot thick layer of sandy silt in borehole 8, the stratum contains almost no fines and the permeability is estimated as high.

Due to the high gravel and boulder content few samples or "N" values were obtained in the stratum; however the relative density of the stratum is estimated as dense.

An artesian water condition was encountered in this stratum in boreholes 5, 6A, 7 and 8. The flow of water obtained from the boreholes was considerable, probably due to the high permeability of the stratum.

For computation purposes the stratum was assumed to have saturated and submerged unit weights of 135 and 73 pounds per cubic foot respectively and an angle of internal friction of 35 degrees.

Bedrock

Bedrock was proved by core drilling in boreholes 4, 5, 6A and 8 for depths of 5 to 10 feet. In all cases the bedrock was a hard, sound reddish granitic rock of Archean age.

WATER CONDITIONS

The elevation of Lake Superior at the time of the investigation varied from 602 to 602.5 feet. An artesian pressure was encountered in the dense sandy gravel stratum which varied across the site from a head of 2 feet to 17 feet. The maximum heads were observed in boreholes 5 and 8 while lower heads were observed in boreholes 7 and 6A, which could be because of their increasing proximity to a rock outcrop and a possible discontinuity in the soil strata. The artesian pressure will probably be higher during the spring than that observed and a piezometric water level of 622 feet was assumed for stability calculations.

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:

<u>Consistency</u>	<u>U-Strength Tons/sq. ft.</u>	<u>Relative Density</u>	<u>Standard Penetration Resistance. Blows/ft.</u>
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

GEOCON

OFFICE REPORT ON SOIL EXPLORATION

CONTRACT 57127 BORING # 3 & 4 DATUM GEODETIC CASING BX & NX
 BORING DATE JULY 23 & SEPT. 1960 REPORT DATE SEPT. 27, 1960 COMPILED BY M.W. CHECKED BY [Signature]
 SAMPLER HAMMER WT. 140 LBS. DROP 30 INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN - LBS. ENERGY)

SAMPLE CONDITION

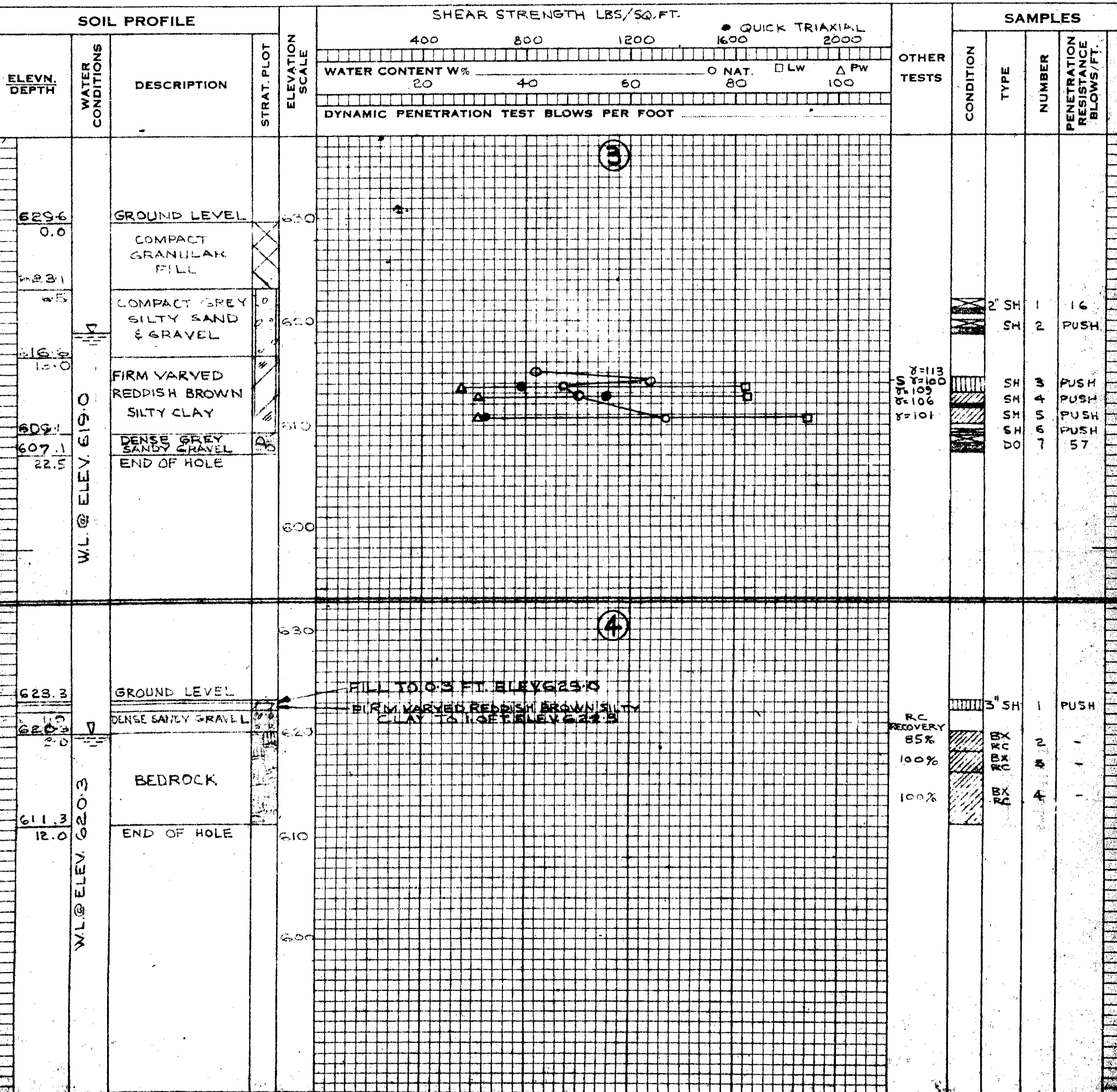


SAMPLE TYPES

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
 S.H. - SHELBY TUBE

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



GEOCON

OFFICE REPORT ON SOIL EXPLORATION

CONTRACT S7127 BORING # 5 DATUM GEODETIC CASING NX 5 BX
 BORING DATE SEPT. 20, 1960 REPORT DATE SEPT. 27, 1960 COMPILED BY M.W. CHECKED BY W.C.
 SAMPLER HAMMER WT. — LBS. DROP — INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN - LBS. ENERGY)

SAMPLE CONDITION



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

SAMPLE TYPES

F.S. - FOIL SAMPLE
 S.O. - SLEEVE-OPEN
 S.F. - SLEEVE-FOOT VALVE
 T.O. - THIN WALLED OPEN
 R.C. - ROCK CORE
 SH - SHELBY TUBE

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

SHEAR STRENGTH LBS./SQ. FT.

+ VANE TEST 400 800 1200 1600 2000
 • QUICK TRIAXIAL
 COMBINED ϕ Mc ϕ Lw ϕ Apw SILT ϕ Mc ϕ Lw ϕ Apw CLAY ϕ Mc ϕ Lw ϕ Apw IN PERCENT
 20 40 60 80 100
 DYNAMIC PENETRATION TEST BLOWS PER FOOT

SAMPLES

OTHER TESTS

CONDITION
 TYPE
 NUMBER
 PENETRATION RESISTANCE BLOWS/FT.

ELEV. DEPTH	WATER CONDITIONS	DESCRIPTION	STRAT. PLOT	ELEVATION SCALE
610.8		GROUND LEVEL		620
610.0		TOP SOIL		610
605.0		GRAVEL & BOULDERS		
585.0		SOFT TO STIFF VARVED REDDISH BROWN SILTY CLAY		600
580.0		DENSE GREY SANDY GRAVEL		590
577.8		BEDROCK		580
573.0		END OF HOLE		570

ARTESIAN WL @ ELEV. 612.0

γ=108

2" SH 1 PUSH

γ=103

SH 2 PUSH

γ=107

SH 3 PUSH

RC RECOVERY

SH 4 PUSH

83%

WS 5

EX RC 6

GEOCON

OFFICE REPORT ON SOIL EXPLORATION

CONTRACT S7127 PEN. TEST 6 DATUM GEODETIC CASING -
 BORING DATE SEPT. 12, 1960 REPORT DATE SEPT. 26, 1960 COMPILED BY M.W. CHECKED BY R.L.
 SAMPLER HAMMER WT. 149 LBS. DROP 30 INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN - LBS. ENERGY)

SAMPLE CONDITION



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

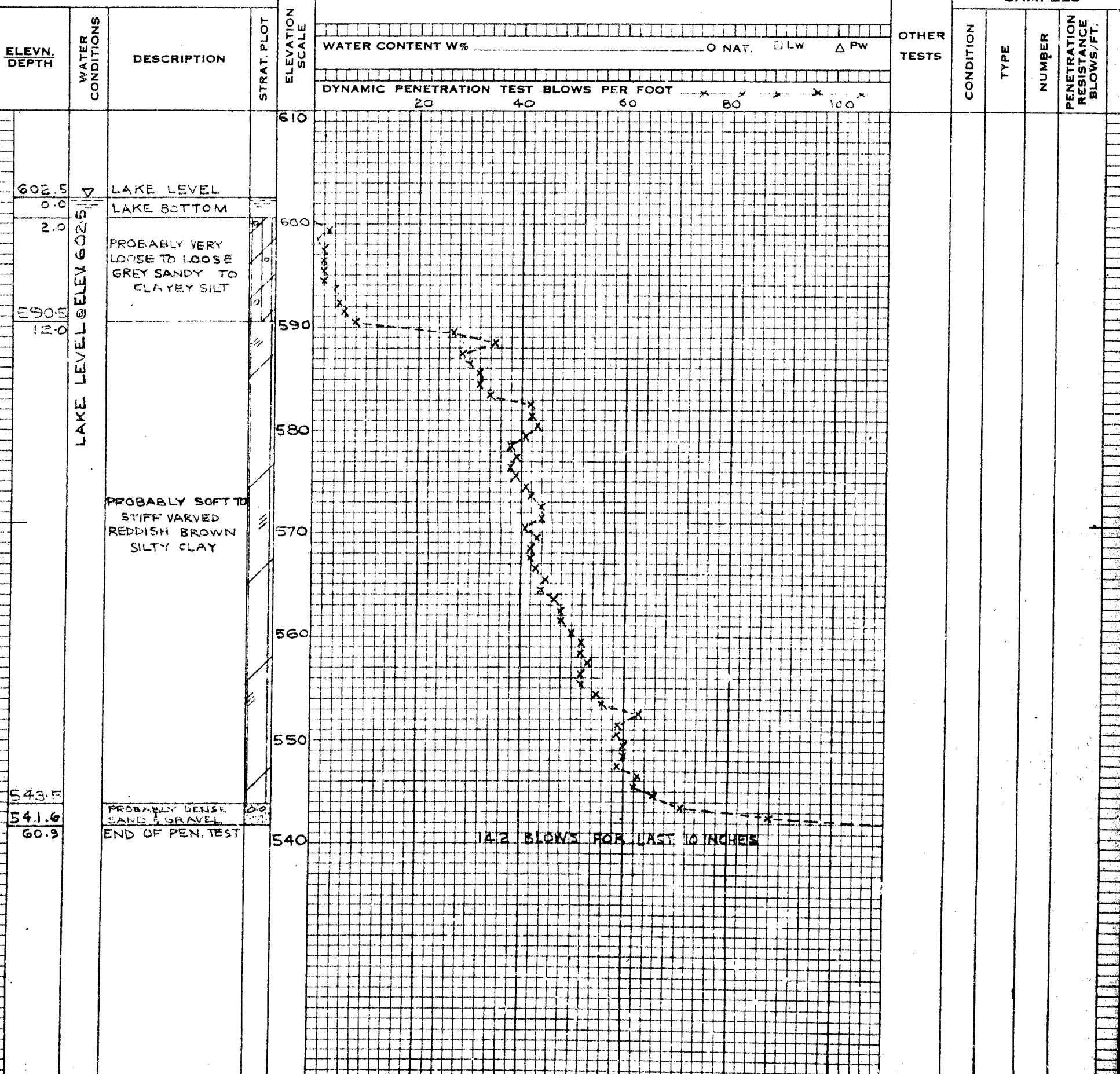
SAMPLE TYPES

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



GEOCON

OFFICE REPORT ON SOIL EXPLORATION

CONTRACT S7127 BORING # 6A DATUM GEODETIC CASING BX
 BORING DATE SEPT. 13, 1960 REPORT DATE SEPT. 26, 1960 COMPILED BY M.W. CHECKED BY MM
 SAMPLER HAMMER WT. 140 LBS. DROP 30 INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN - LBS. ENERGY)

SAMPLE CONDITION



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

SAMPLE TYPES

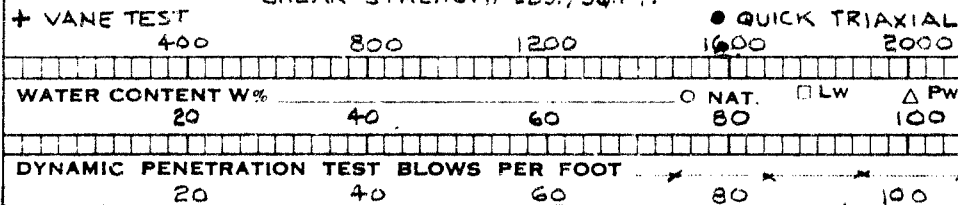
F.S. - FOIL SAMPLE
 S.O. - SLEEVE-OPEN
 S.F. - SLEEVE-FOOT VALVE
 T.O. - TWIN WALLED OPEN
 R.C. - ROCK CORE
 S.H. - SHELBY TUBE

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

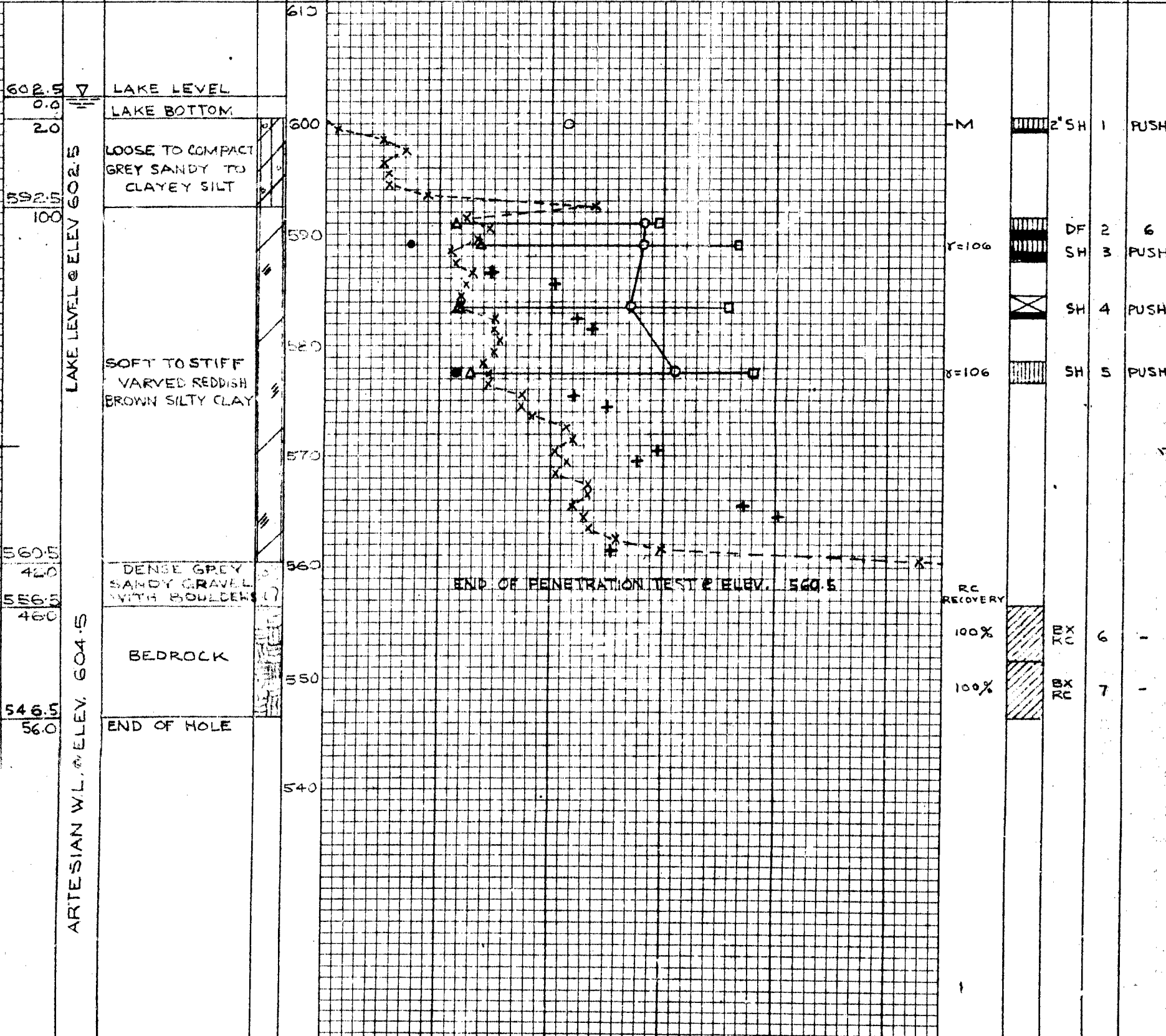
SHEAR STRENGTH LBS./SQ. FT.



OTHER TESTS

SAMPLES

CONDITION
 TYPE
 NUMBER
 PENETRATION RESISTANCE BLOWS/FT.



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OFFICE REPORT ON SOIL EXPLORATION

CONTRACT S 7127 BORING # 7 DATUM GEODETIC CASING NK & BX
 BORING DATE SEPT. 7, 1960 REPORT DATE SEPT. 26, 1960 COMPILED BY M.W. CHECKED BY W.C.
 SAMPLER HAMMER WT. 140 LBS. DROP 30 INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN - LBS. ENERGY)

SAMPLE CONDITION



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

SAMPLE TYPES

F.S. - FOIL SAMPLE
 S.O. - SLEEVE-OPEN
 S.F. - SLEEVE-FOOT VALVE
 T.O. - THIN WALLED OPEN
 R.C. - ROCK CORE
 S.H. - SHELBY TUBE

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

ELEV. DEPTH	WATER CONDITIONS	DESCRIPTION	STRAT. PLOT	ELEVATION SCALE
602.5	V	LAKE LEVEL		610
0.0		LAKE BOTTOM		600
2.0		VERY LOOSE TO LOOSE GREY SANDY TO CLAYEY SILT		590
592.5				580
10.0				570
		SOFT TO STIFF VARVED REDDISH BROWN SILTY CLAY		560
				550
				540
541.0		DENSE GREY SANDY GRAVEL AND BOULDERS		530
61.5				520
527.5		END OF HOLE		
75.0				

SHEAR STRENGTH LBS./SQ. FT.

+ VANE TEST				● QUICK TRIAXIAL			
400	800	1200	1600	2000			
COMBINED C Mc. Blw. ΔPw. SILT C Mc. Blw. ΔPw. CLAY C Mc. Blw. ΔPw. IN PERCENT							
20	40	60	80	100			
DYNAMIC PENETRATION TEST BLOWS PER FOOT							
20	40	60	80	100			

OTHER TESTS

SAMPLES

CONDITION	TYPE	NUMBER	PENETRATION RESISTANCE BLOWS/FT.
2" SH		1	PUSH
WS		2	-
2" DF		3	PUSH
WS		4	-
2" SH		5	PUSH
SH		6	PUSH
WS		7	-
WS		8	-
BX CORE		9	-

γ = 116

γ = 103

CORE RECOVERY

48%

END OF PENETRATION TEST @ ELEV. 541.5

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OFFICE REPORT ON SOIL EXPLORATION

CONTRACT S7127 BORING # B DATUM GEODETIC CASING NX & BX
 BORING DATE SEPT. 1, 1960 REPORT DATE SEPT. 26, 1960 COMPILED BY M.W. CHECKED BY [Signature]
 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
 S.H. - SHELBY TUBE

ABBREVIATIONS

∇ - 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

ELEV. DEPTH	WATER CONDITIONS	DESCRIPTION	STRAT. PLOT	ELEVATION SCALE
602.5 0.0	1"	LAKE LEVEL		610
600.2 2.3		LAKE BOTTOM		600
		VERY LOOSE TO LOOSE GREY SANDY TO CLAYEY SILT		590
580.5 20.5				580
		SOFT TO STIFF VARYED REDDISH BROWN SILTY CLAY		570
				560
				550
				540
				530
522.5 80.0		DENSE GREY SANDY GRAVEL		520
508.5 94.0		BEDROCK		510
503.5 99.0		END OF HOLE		500

SHEAR STRENGTH LBS./SQ. FT.

+ VANE TEST	400	800	1200	1600	2000
WATER CONTENT W%	20	40	60	80	100
DYNAMIC PENETRATION TEST BLOWS PER FOOT	20	40	60	80	100

● QUICK TRIAXIAL
 ○ NAT. □ LW △ PW

OTHER TESTS

SAMPLES

CONDITION	TYPE	NUMBER	PENETRATION RESISTANCE BLOWS/FT.
WS	1	-	
WS	2	-	
2" DF	3	2	
SH	4	PUSH	
DF	5	1	
DF	6	9	
WS	7	-	
2" SH	8	PUSH	
SH	9	PUSH	
SH	10	PUSH	
SH	11	PUSH	
SH	12	PUSH	
SH	13	PUSH	
WS	14	-	
WS	15	-	
WS	16	-	
EX RC	17	-	

END OF PENETRATION TEST @ ELEV. 514.5

RC RECOVERY
 100%

APPENDIX II

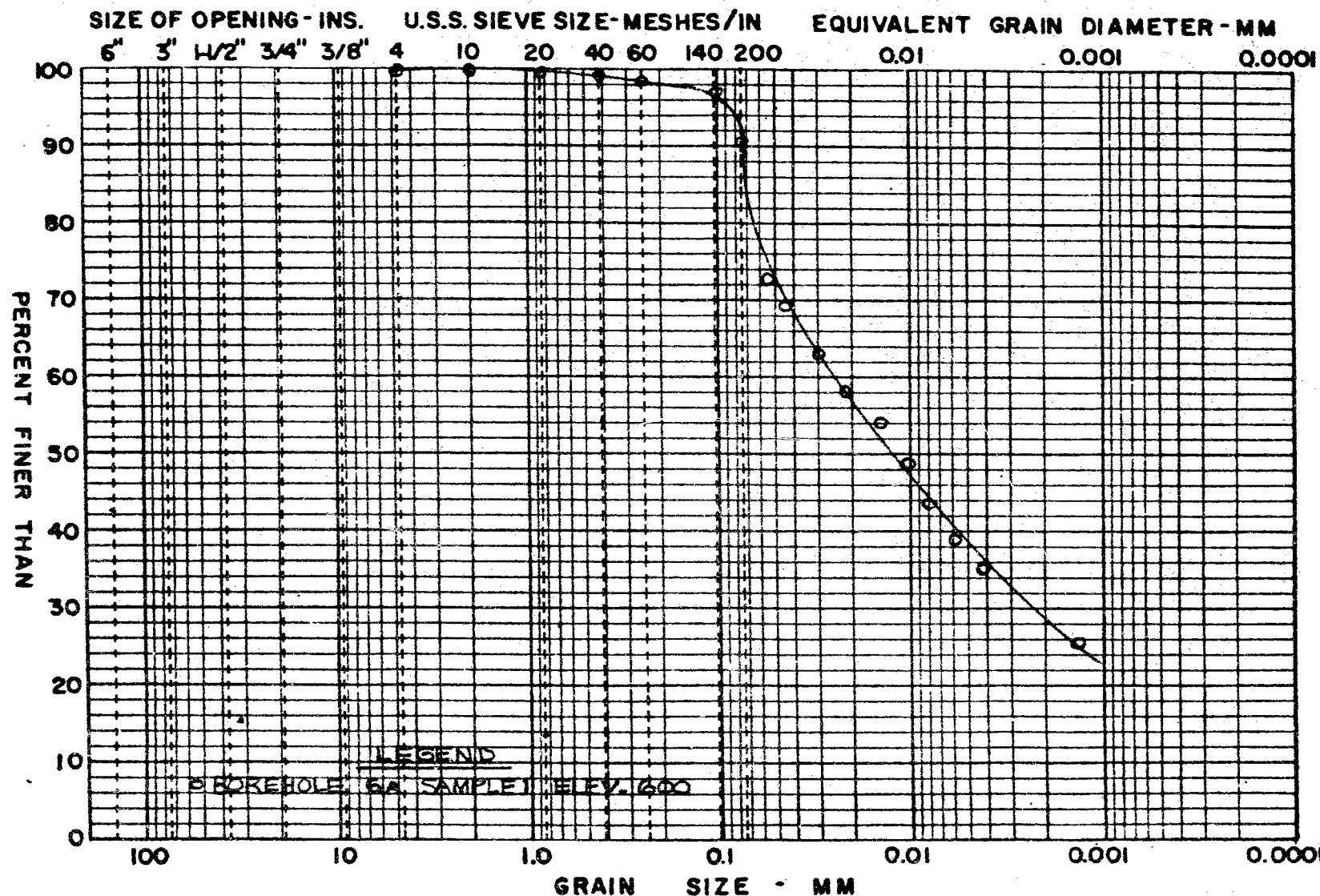
FIGURES - LABORATORY TESTING

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GRAIN SIZE DISTRIBUTION

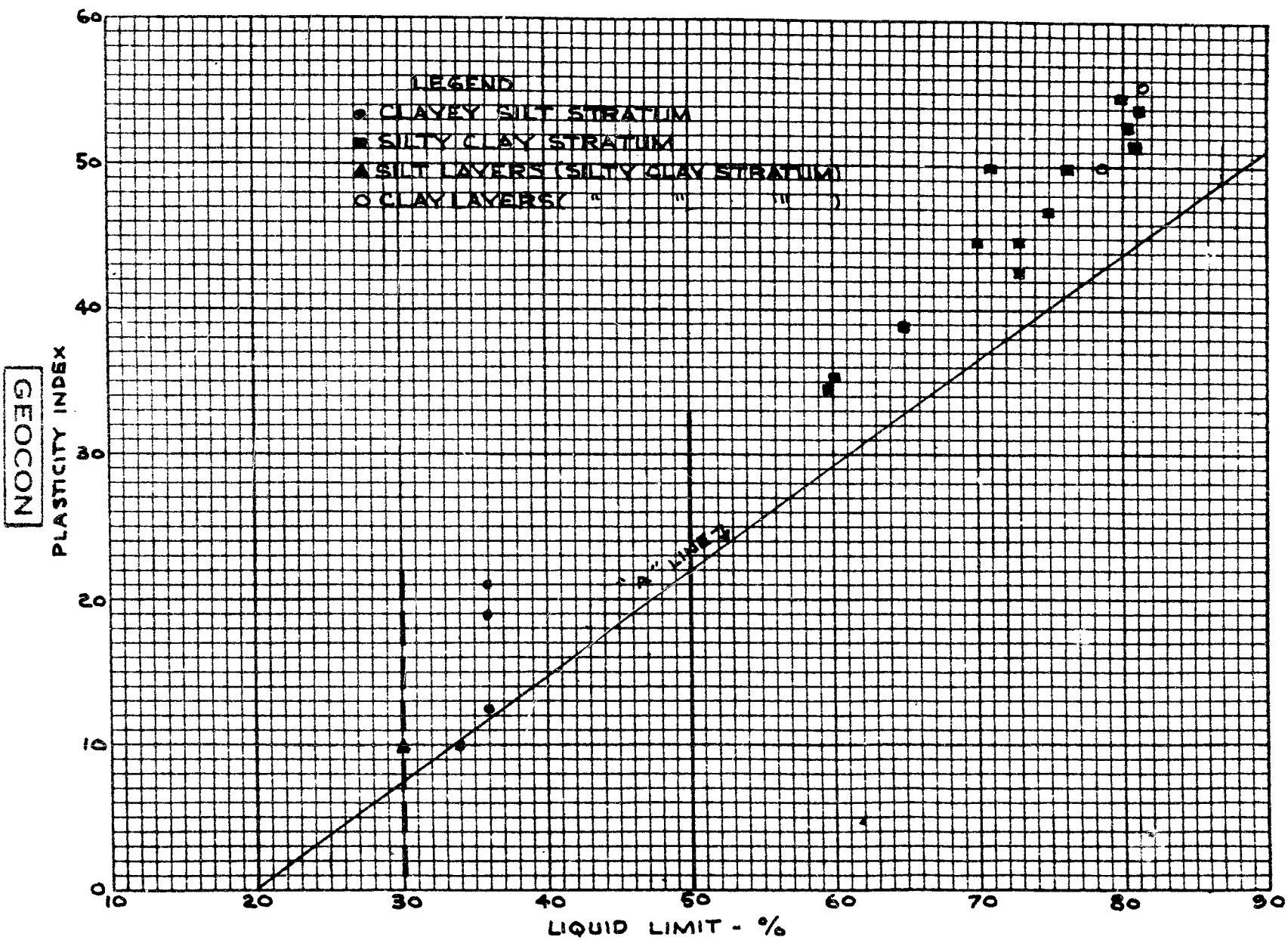
APPENDIX II
FIGURE 1
PROJECT S7127

COBBLE	GRAVEL SIZE			SAND SIZE			FINE GRAINED	
← SIZE	COARSE	MEDIUM	FINE	COARSE	MEDIUM	FINE	SILT SIZE	CLAY SIZE →



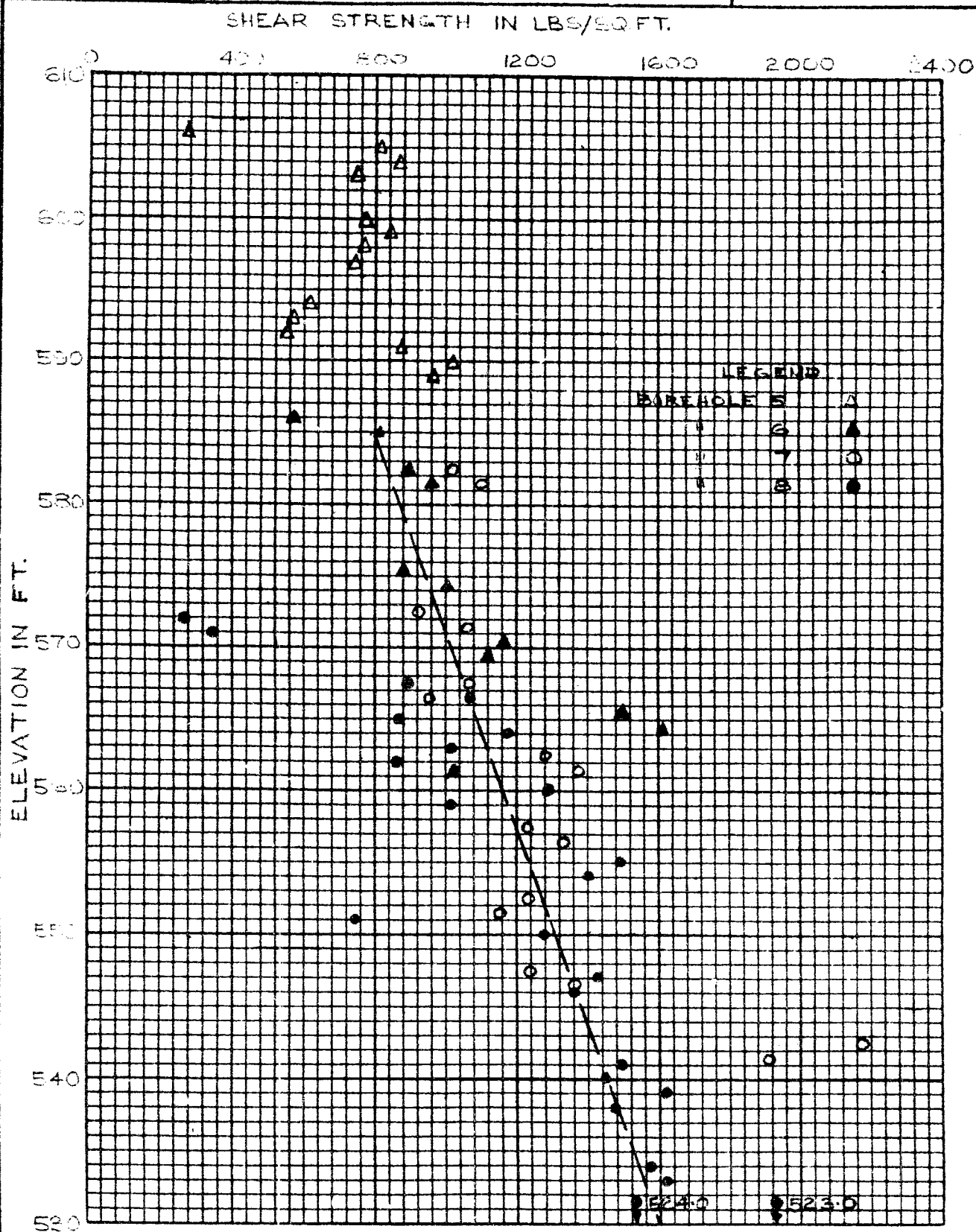
M.I.T. GRAIN SIZE SCALE

GEOCON



IN-SITU VANE SHEAR STRENGTH VS. ELEVATION

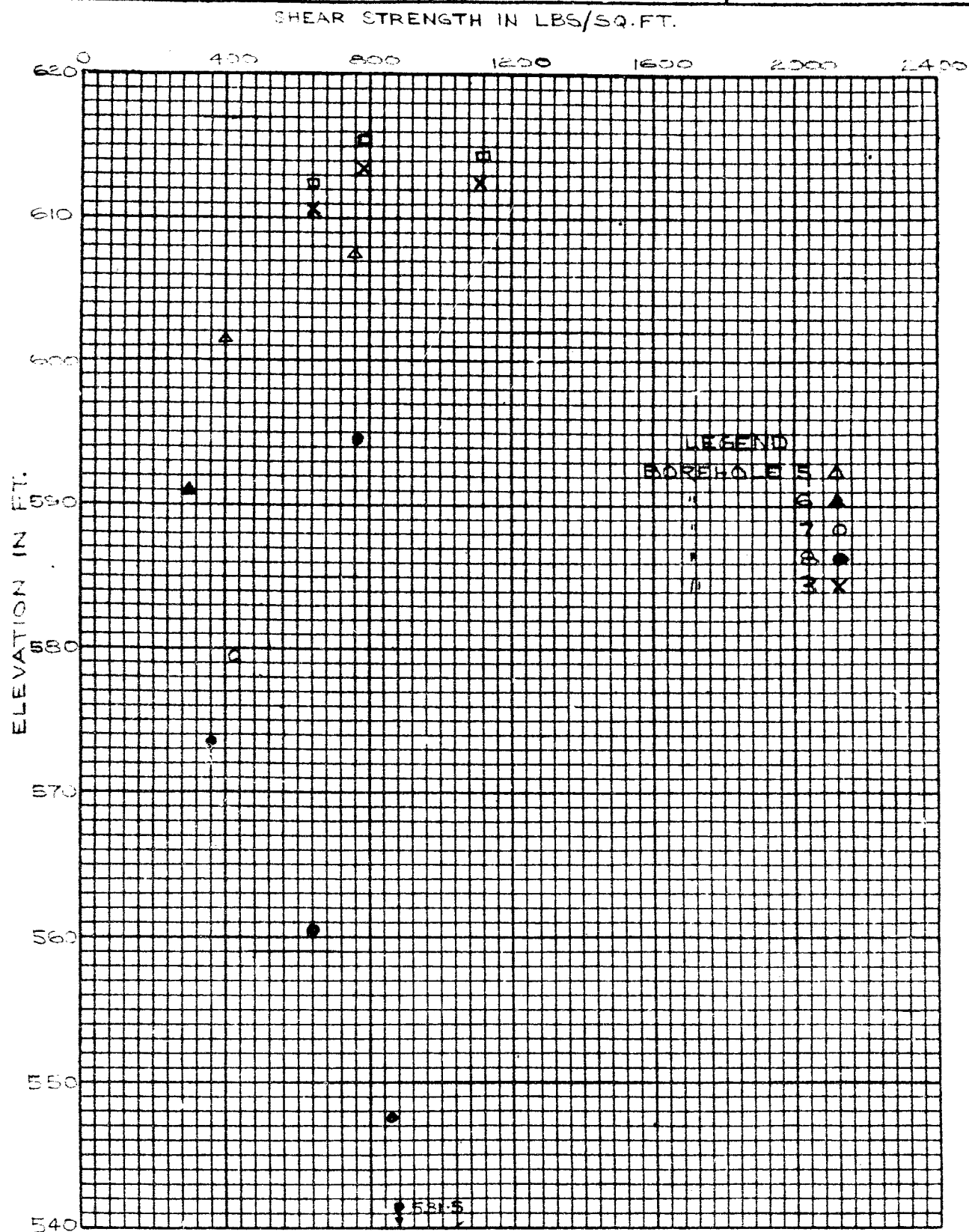
APPENDIX II
FIGURE 3
PROJECT S-7127



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SHEAR STRENGTH VS. ELEVATION QUICK TRIAXIAL TESTS

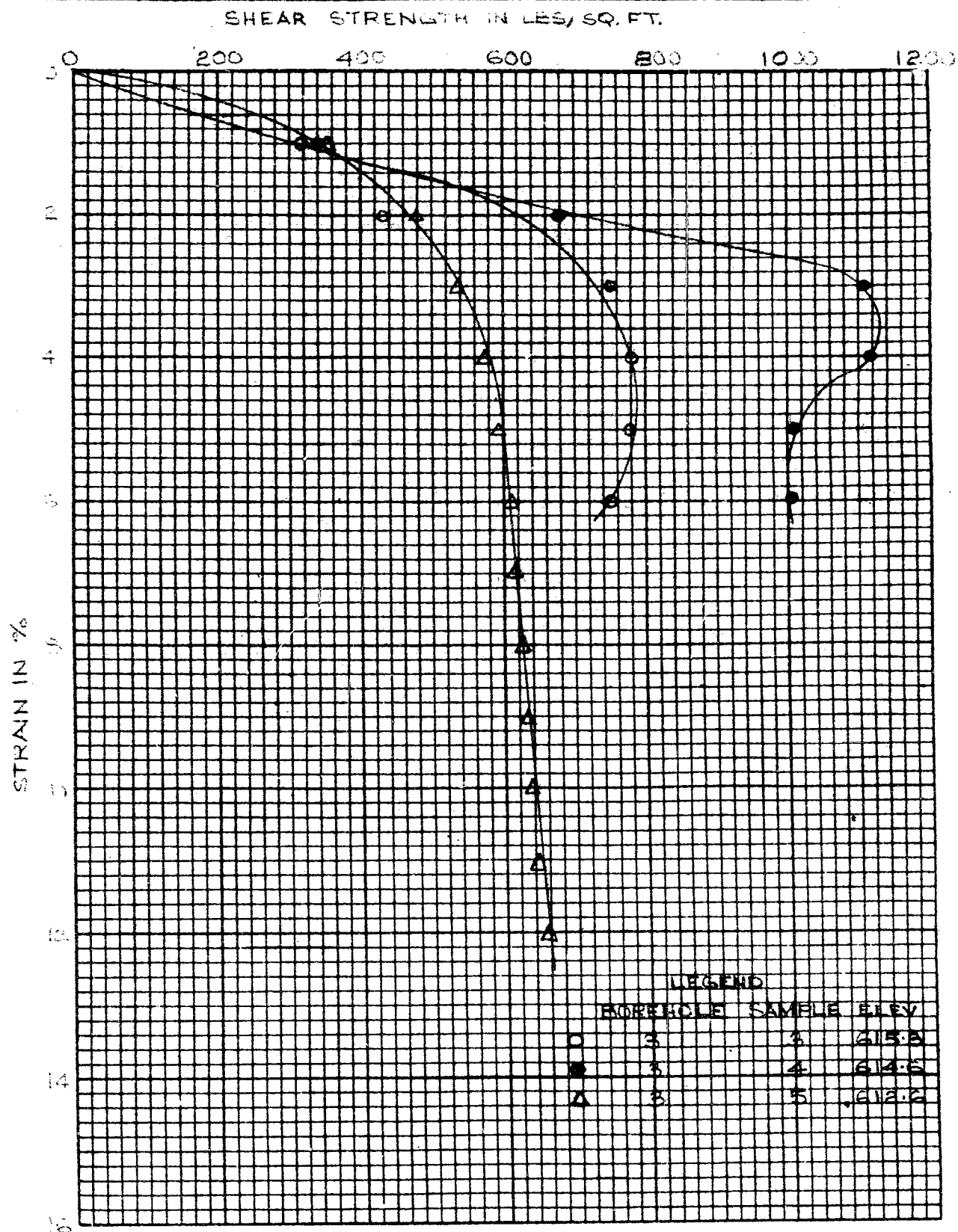
APPENDIX II
FIGURE 4
PROJECT S-7127



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QUICK TRIAXIAL COMPRESSION TESTS TYPICAL STRESS-STRAIN CURVES

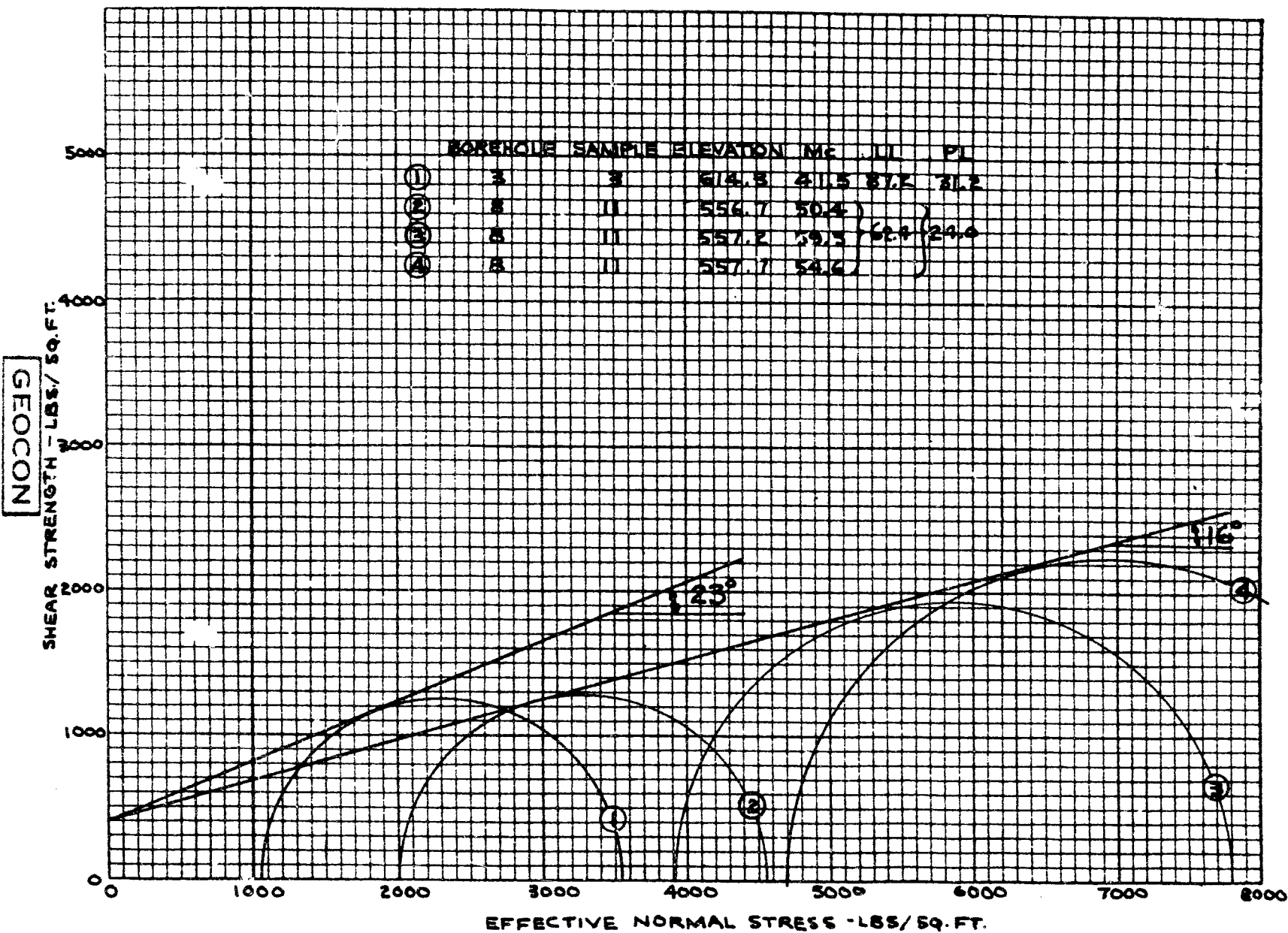
APPENDIX II
FIGURE 5
PROJECT S-7127



CONSOLIDATED UNDRAINED TRIAXIAL TESTS
WITH PORE PRESSURE MEASUREMENTS

MOHR'S CIRCLES

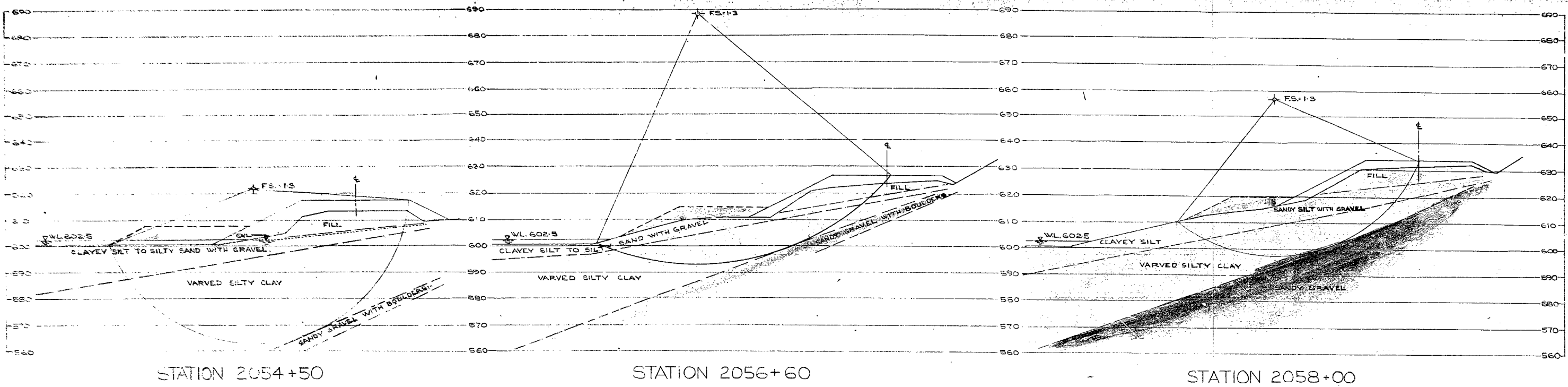
APPENDIX II
FIGURE 10
PROJECT S7127



APPENDIX III

DRAWING S7127-3 RECOMMENDED BERM SIZES FOR PROPOSED EMBANKMENTS

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NOTE

FACTORS OF SAFETY SHOWN
CONSIDER CASE FOR LONG TERM
STABILITY $\phi = 0$ IN VARVED
SILTY CLAY STRATUM.

LEGEND

- RECOMMENDED BERMS
- PROPOSED EMBANKMENTS

DWG. NO.	REFERENCE
S 7149-1	GEOCON LTD - BORING PLAN & SOIL STRATIGRAPHY
S 7127-2	GEOCON LTD - SUMMARIZED STABILITY ANALYSES, EXISTING & PROPOSED EMBANKMENTS

DEPARTMENT OF HIGHWAYS, ONTARIO
TORONTO
HIGHWAY 17, STA 2054-2059
SAULT STE. MARIE
ONTARIO

RECOMMENDED BERM SIZES
FOR PROPOSED EMBANKMENTS

GEOCON LTD

DATE DEC 9, 1960 SCALE 1" = 20'-0"

MADE 26% CHKD. 26% APPD. 26% No. S 7127-3

Mr. A. M. Toye, Bridge Engr.,
Bridge Office.

February 20, 1961.

ADDITIONAL STABILITY STUDIES -

Materials & Research Section.

by: Geoccon, Limited.

Attention: Mr. S. McCoable.

Re: Proposed Grade Reduction,
Existing Highway 17 Roadway Embankments
Near Sault Ste. Marie, Ont. - District #18.
W.P. 903-60.

Attached to this memo, we are sending you the Letter report for the above mentioned location, submitted by the Consultant, Geoccon, Ltd. This Letter report was preceded by a Report and an Addendum to this Report dated January 18th, 1961. A wrong W.P. number (W.P. 918-60) was given to this Addendum and therefore in the covering letter, dated January 27, 1961, it was stated that " in the meantime, the line has been moved out of the sliding zone." This mistake and misunderstanding was subsequently removed and clarified. This last Letter report provides the answer and solution to the question which was raised by the District Soils Engineer.

We would like to draw your attention to the assumptions of the stability analyses as well as to the recommendations based on these analyses. If this solution is adopted, part of the present road embankment, notably the shoulders, will have to be excavated and the material removed.

cont'd. /2 ...

We believe that now all the necessary information for your further design work is available. However, should there be any additional questions you would like to discuss, please feel free to call on our Office.

L. G. Soderman,
PRINCIPAL FOUNDATION ENGR.

Per:

Agstern

(A. G. Sternac,
SUPERVISING FOUNDATION ENGR.)

AGG/MdeF
Attach.

- cc: Messrs. H. A. Mantle
H. A. Tregaskes
H. D. McMillan
G. K. Hunter
D. P. Collins
E. R. Saint
A. Watt

Foundations Office ✓
Gen. Files.

S7187
LETTER REPORT
TO
DEPARTMENT OF HIGHWAYS, ONTARIO
ON
ADDITIONAL STABILITY STUDIES
PROPOSED GRADE REDUCTION
EXISTING HIGHWAY 17 ROADWAY EMBANKMENTS
NEAR SAULT STE. MARIE ONTARIO
W.P. 903-60

Distribution:

- 10 copies - Department of Highways, Ontario,
Downsview, Ontario.
- 2 copies - Geocon Ltd,
Rexdale, Ontario.

GEOCON

GEOCON LTD

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

DISTRICT OFFICES

14 HAAS ROAD
REXDALE, TORONTO, ONT.
TEL. CH. 4-8641

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

Rexdale, Ontario,
February 14th, 1961.

Department of Highways, Ontario,
Downsview, Ontario.

Attention: Mr. L. G. Soderman, P. Eng.,
Principal Foundation Engineer.

Re: Additional Stability Studies,
Existing Highway 17 Roadway Embankment,
Station 2055+00 to Station 2059+00,
Near Sault Ste. Marie, Ontario,
W.P. 903-60.

Dear Sirs:

This letter reports the results of the above additional studies carried out in accordance with verbal instructions given by your Mr. A. Stermac on January 31st, 1961. The results of the investigation and engineering studies for this site were presented in our report S7127, dated December 9th, 1960 and in an addendum to this report dated January 18th, 1961. As discussed in the addendum to the above report, the existing roadway embankment between stations 2055+00 and 2059+00, requires stabilizing berms which are shown for typical sections on Drawing S7127-4. The provision of these berms would necessitate removal of several cottages which are located between the existing embankment and the shoreline. In order to avoid removal of these cottages an alternative stabilizing scheme, which would require a reduction of the existing embankment height, has been considered. The additional stability analyses, the results of which are discussed below, were carried out to determine the grade reduction of the existing highway embankment necessary to obtain adequate stability.

Department of Highways, Ontario,
February 14th, 1961,
Page 2.

As discussed in the addendum to our previous report, it has been concluded that the artesian pressure in the underlying sand and gravel stratum could rise to elevation 622 in the spring and the remedial measures for embankment stability have therefore been designed for an artesian pressure at elevation 622.

In order to provide adequate stability for the existing embankments without the addition of berms it will be necessary to reduce the existing height. Stability analyses were carried out to determine this reduction in grade for three typical embankment sections and the results are summarized on Drawing S7187-1 attached to this letter. In these analyses it was assumed that the present roadway centreline would be maintained, that the maximum top roadway width would be 42 feet and that the side slopes of the embankment would be trimmed to 2 horizontal to 1 vertical.

The stability of the various sections was checked by an effective stress analysis, using the rigorous method. For long term stability, a computed factor of safety of 1.3 using the case of partial effective cohesion along the failure arc was considered adequate. In addition, all circles were checked to have a computed factor of safety of at least 1.0 when all effective cohesion is disregarded.

The stability computations indicate that a reduction in the embankment size would be required between station 2055+00 and 2059+00, with a lowering of the existing grade of 4.0 feet at station 2056+60 and 3.5 feet at station 2058+00. The existing embankments at a chainage of less than 2056+60 and greater than 2059+00 are considered, from visual examination of the roadway plan and profile, to have adequate stability.

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Department of Highways, Ontario,
February 14th, 1961,
Page 3.

It is recommended to minimize artesian affects, that a 3 foot deep graded ditch be provided between about stations 2055+00 and 2059+00 as shown on Drawing S7187-1. If for highway safety reasons this depth of open ditch is not acceptable, it may be backfilled with coarse gravel or rock fill.

Should it be decided to retain the existing grade or to increase the grade as originally planned, the berm sizes as discussed in the addendum to our report S7127, dated January 18th, 1961 would be required for adequate stability.

We believe that this letter contains all the information that you require. However if we can be of any further assistance, please give us a call.

Yours very truly,

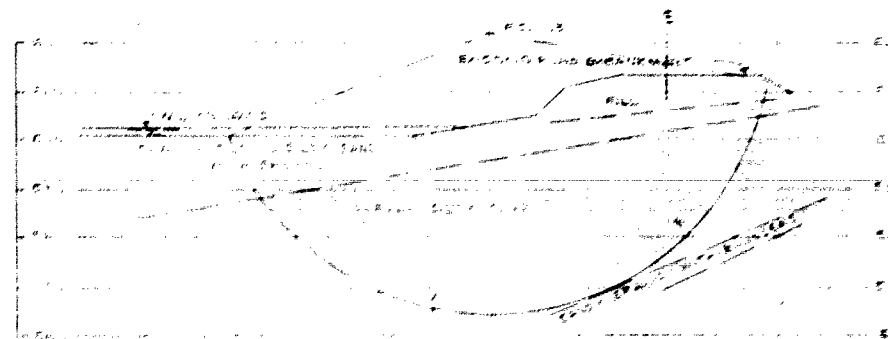
GEOCON LTD



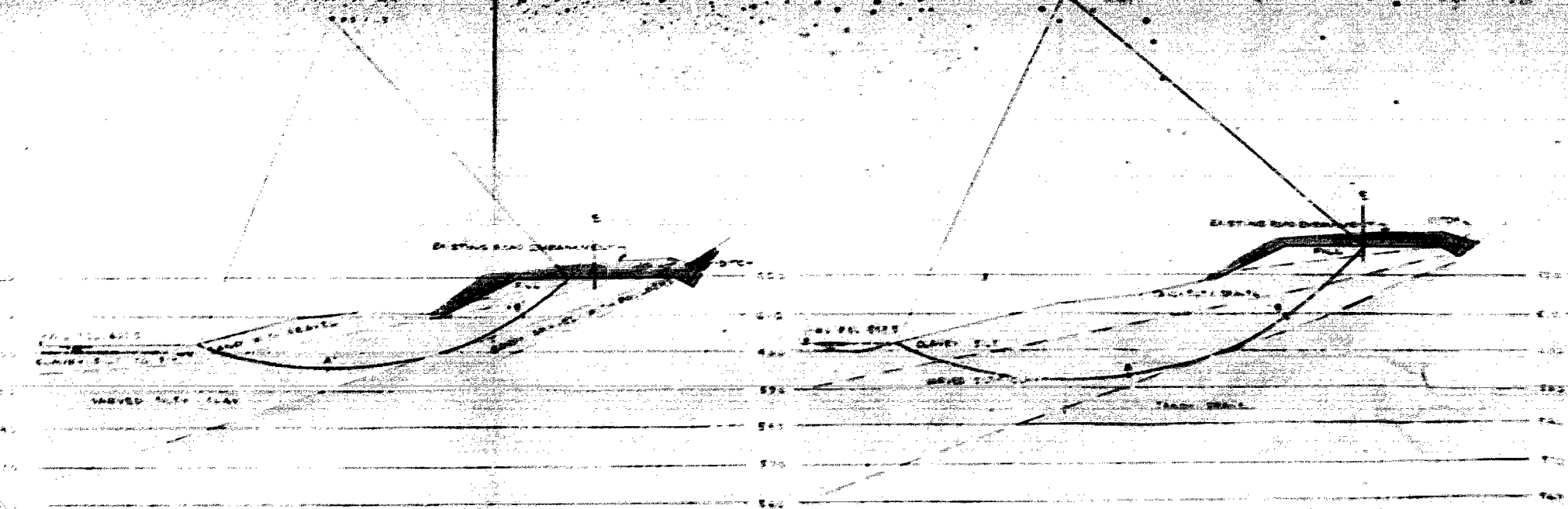
J. L. Seychuk, P. Eng.,
District Soils Engineer.

JLS/dw
S-7187

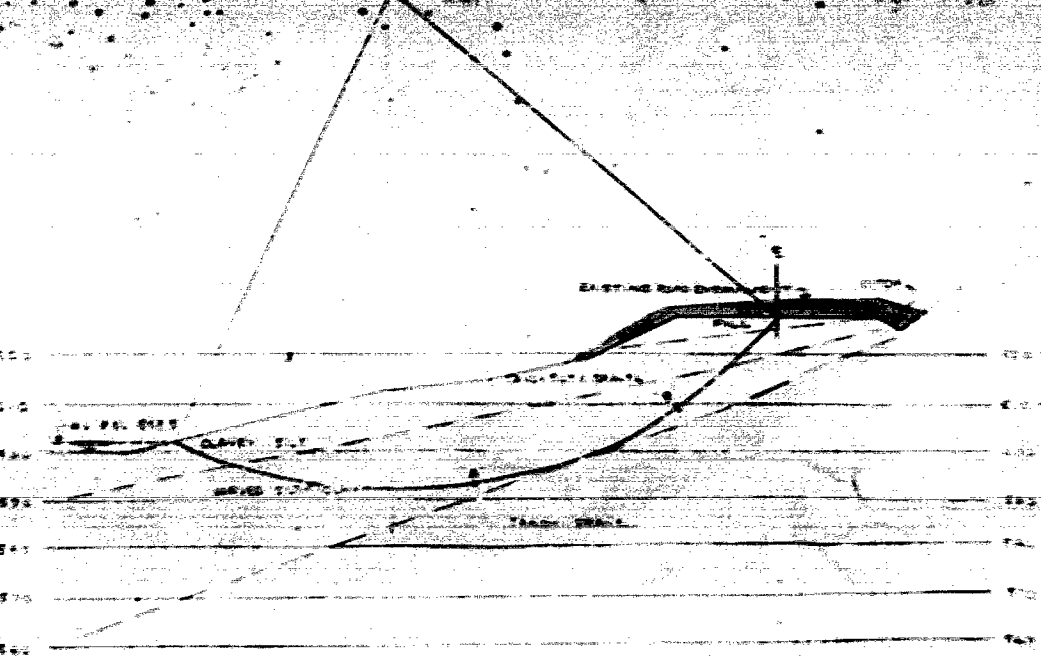
GEOCON



STATION 2054+50



STATION 2058+50



STATION 2059+00

NOTES: 1. THE ELEVATIONS OF THE ROAD SURFACE ARE BASED ON THE 1985 DATUM. 2. THE ELEVATIONS OF THE PROPOSED EMBANKMENT ARE BASED ON THE 1985 DATUM. 3. THE ELEVATIONS OF THE PROPOSED ROAD DRAINAGE ARE BASED ON THE 1985 DATUM. 4. THE ELEVATIONS OF THE PROPOSED ROAD DRAINAGE ARE BASED ON THE 1985 DATUM.

LEGEND

PROPOSED EMBANKMENT

STATION	DESCRIPTION
2054+50	SECTION 1: FROM THE EXISTING ROAD DRAINAGE TO THE PROPOSED EMBANKMENT.
2058+50	SECTION 2: FROM THE EXISTING ROAD DRAINAGE TO THE PROPOSED EMBANKMENT.
2059+00	SECTION 3: FROM THE EXISTING ROAD DRAINAGE TO THE PROPOSED EMBANKMENT.

DEPARTMENT OF HIGHWAYS, ONTARIO
HIGHWAY 17, STA 2054 TO 2059
SUMMARIZED STABILITY ANALYSIS
EXISTING EMBANKMENTS & RECOMMENDED CUTS

GEOCON LTD
DATE: FEB. 0, 1988 SCALE: 1" = 20'-0"
SHEET: 1 OF 1

January 27, 1961.

Mr. A. M. Toye,
Bridge Engineer,
Bridge Section.

FOUNDATION INVESTIGATION
REPORT

by - Geocon, Limited.

Attention: Mr. S. McCombie.

Re: Addendum to Report S7127 on Soil
Conditions and Engineering Study
Existing and Proposed Hwy. 17,
W.P. #18 Co., Sault Ste. Marie, Ont.
405-40 District #18.

Attached to this letter we are forwarding to you
the above-mentioned additional report. In the meantime the
line has been moved out of the sliding zone and therefore the
recommendations contained in this additional report do not
influence the chosen solution. However, they can be very
useful as future reference for the construction to be
carried out in this region.

L. G. Soderman,
PRINCIPAL FOUNDATIONS ENGINEER
Per:

A. Stermac

(A. Stermac
Senior Foundations Engineer

Attach.

AS/tt

c/c, Messrs. H. A. Mantle
H. A. Tregaskes
H. D. McMillan
A. Watt
C. K. Hunter
D. P. Collins
F. Saint
Foundations Office
General Files

ADDENDUM
TO
REPORT S7127
FOR
DEPARTMENT OF HIGHWAYS, ONTARIO
ON
SOIL CONDITIONS AND ENGINEERING STUDY
EXISTING AND PROPOSED HIGHWAY 17
908-60
W.P. 318-59
SAULT STE. MARIE **ONTARIO**

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Rexdale, Ontario,
January 18th, 1961.

Department of Highways, Ontario,
Downsview, Ontario.

Attention: Mr. L. G. Soderman, P. Eng.,
Principal Foundation Engineer.

Re: Embankment Stability,
Proposed Grade Adjustment, Highway 17,
Sault Ste. Marie, Ontario.
W.P. 910-59 403-60

Dear Sirs:

Our report S7127 covering the above investigation and engineering study was submitted on December 9th, 1960. As discussed in a telephone conversation on January 6th, 1961 between your Mr. L. G. Soderman and the writer, further information has been obtained which affects the conclusions and recommendations given in our original report. The necessary modifications are discussed below and are shown on the attached drawings S7127-4 and -5. Would you kindly consider the information and recommendations contained herein as supplementary to and superceding that given in our report dated December 9th, 1960.

It has now been ascertained that a minor failure of the existing roadway embankment between about stations 2055+50 and 2057+50 occurred during the spring of 1960. The maximum embankment slump was less than 6 inches and the surface grade was re-established by the placing of bitumen patching prior to this investigation. This resurfacing was referred to in the report. However it was not recognized as evidence of a failure, since during the period of the investigation, no other effects of the failure were apparent.

Department of Highways, Ontario,
January 18th, 1961,
Page 2.

The studies carried out in our original report showed that the existing embankment stability would be critical between about stations 2055+50 and 2058+00 should an artesian water head at elevation 622 be considered in the underlying sand and gravel stratum. The artesian pressures observed in the boreholes in this area during the field investigation in August and September were appreciably lower than elevation 622, being highest on shore and falling off appreciably at offshore locations. However, as a minor failure is now known to have occurred in this area, it is concluded that the artesian pressure could rise to elevation 622 in the spring and the remedial measures for embankment stability have now been modified accordingly for an artesian pressure at elevation 622. The changes involved are not excessive since in the original report certain contingencies were provided against the uncertainties in the artesian pressures assumed.

The existing embankments will require stabilizing berms between about stations 2055+00 and 2059+00. Drawing S7127-4, attached to this letter shows the required berm sizes at stations 2056+60 and 2058+00. The existing embankment at the other typical section investigated is considered to be stable. The stability of the various sections, shown on drawing S7127-4, was checked by an effective stress analysis, using the rigorous method. For long-term stability, a computed factor of safety of 1.3 using the case of partial effective cohesion along the failure arc was considered adequate. As discussed in the report, it was considered that cohesion need only be disregarded along that portion of the arc which was located approximately parallel to the stratification of the clay. In addition, all circles were checked to have a computed factor of safety of at least 1.0 when all effective cohesion is disregarded.

Department of Highways, Ontario,
January 18th, 1961,
Page 3.

For the case of stability during construction, \bar{B} was taken as 1.0 and full effective cohesion was assumed; a minimum computed factor of safety of 1.3 was considered desirable. The reasoning behind the assumption of the various soil properties is discussed in detail in the original report.

Drawing S7127-5, attached to this letter, shows the required berm sizes for the proposed grade revision. Stabilizing berms will be required between about stations 2053+00 and 2059+50. The design criteria used in determining the stability and required berm sizes for the proposed embankments are the same as those discussed above.

We believe that with this added information, this letter finalizes our report on the above investigation. However, should you require any further information or assistance in any other way, we would be pleased if you would give us a call.

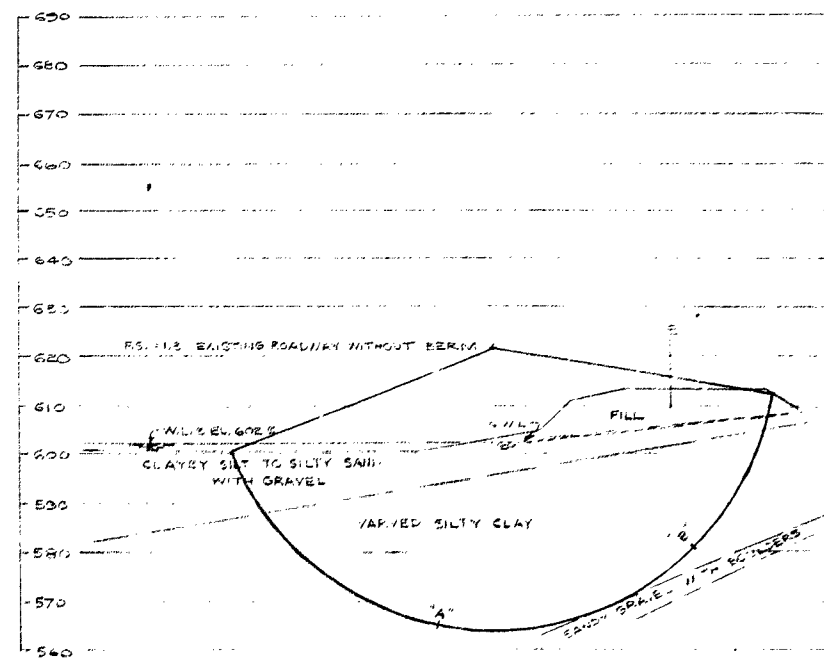
Yours very truly,

GEOCON LTD



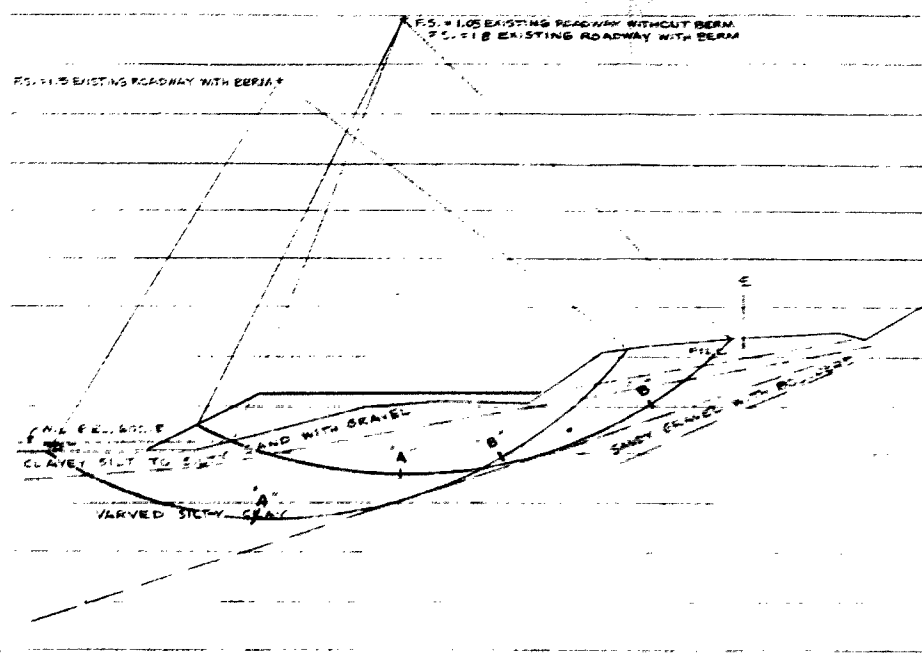
JLS/dw
S7127

J. L. Seychuk, P. Eng.,
District Soils Engineer.



STATION 2054+50

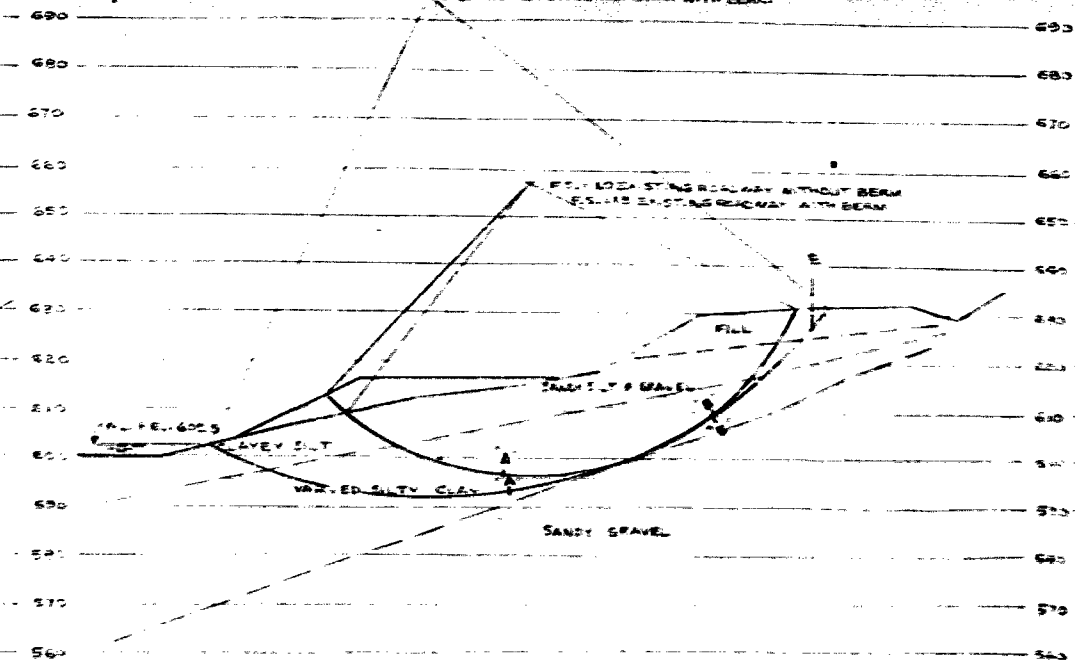
NOTE: THE FACTORS OF SAFETY SHOWN CONSIDER CASE FOR LONG TERM STABILITY WHERE:
 (1) $F = 1.00$ IN THE VARIED CLAY STRATUM
 (2) $F = 1.00$ IN THE VARIED SILTY CLAY STRATUM EXCEPT BETWEEN POINTS 'A' AND 'B' ON THE LEFT SIDE WHERE $F = 1.0$



STATION 2056+60

LEGEND

RECOMMENDED BERMS

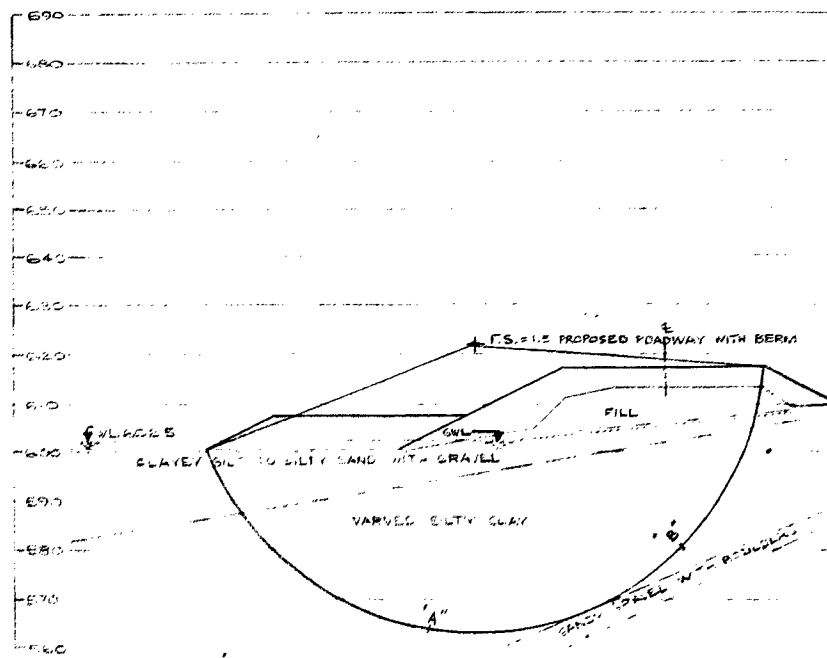


STATION 2058+00

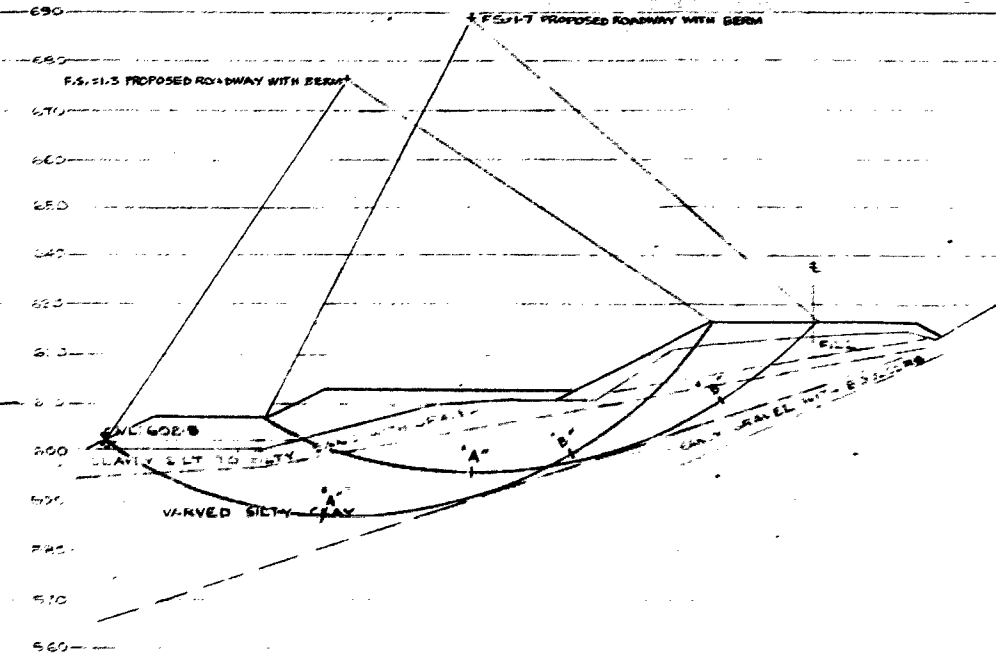
DWG. No.	REFERENCE
57127-1	SECTION LTD - DWG. OF HIGHWAY 17 STA. 2054 TO 2058 EXISTING PLAN AND SOIL STRATIGRAPHY

DEPARTMENT OF HIGHWAYS, ONTARIO
Highway 17, Sta. 2054 to 2058
SUMMARIZED STABILITY ANALYSES
EXISTING EMBANKMENTS & REQUIRED BERMS

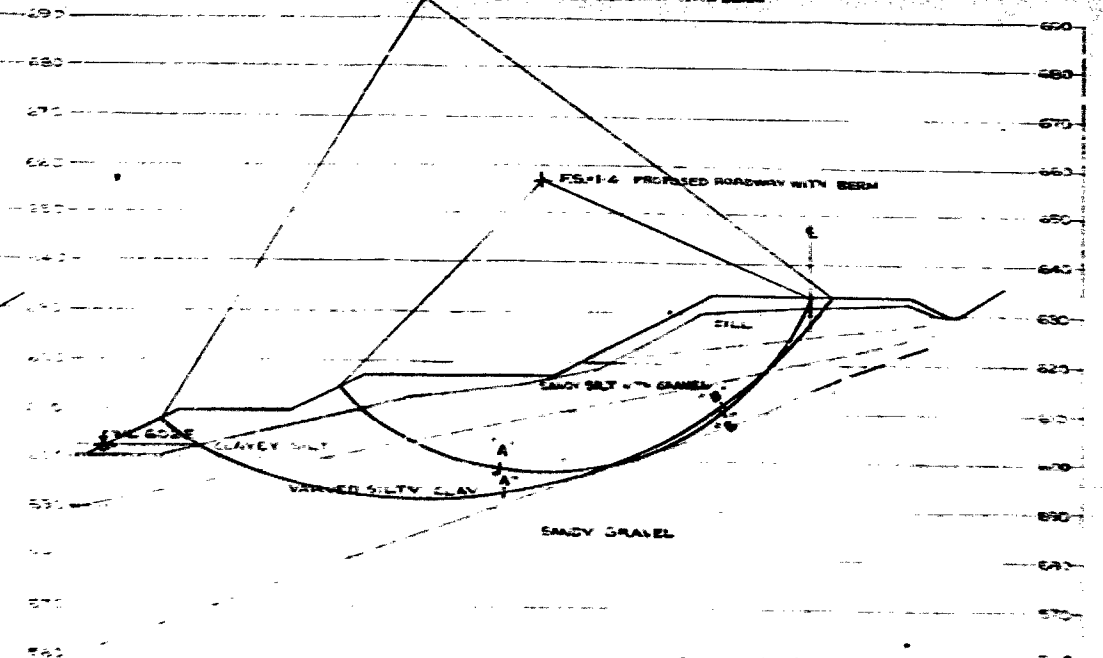
GEOCON LTD
DATE JAN 13, 1961 SCALE 1" = 20'-0"
No. S 7127-4



STATION 2054+50



STATION 2054+60



STATION 2055+00

NOTE: THE FACTORS OF SAFETY SHOWN FOR THE PROPOSED ROADWAY AND RECOMMENDED BERM SECTIONS CONSIDERS CASE FOR LONG TERM STABILITY WHERE:

- (i) $\phi = 20^\circ$ IN THE VARVED CLAY STRATUM
- (ii) $C = 200$ LBS/50 FT. IN THE VARVED CLAY STRATUM EXCEPT BETWEEN POINTS "A" AND "B" ON THE SUP CIRCLE WHERE $C = 0$

LEGEND

- RECOMMENDED BERMS
- PROPOSED EMBANKMENTS

DRAWING NO. S 7127-5	REFERENCE GEOTECHNICAL BORING PLAN & STRATIGRAPHY SAULT STE MARIE	DEPARTMENT OF HIGHWAYS ONTARIO TORONTO HIGHWAY 17 STA 2054-2059 SAULT STE MARIE ONTARIO SUMMARIZED STABILITY ANALYSES PROPOSED EMBANKMENTS & REQUIRED BERMS	GEOCON LTD DATE JAN 13, 1961 SCALE 1"=20'-0" MADE BY JZ CHECKED BY JZ APPR. BY JZ No S 7127-5
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41K-15

GEOCRE No.

BA 1180

41K-15

41K-15

GEOCRE No.

Mr. A. M. Teye,
Bridge Engineer,
Bridge Section.

January 27, 1961.
FOUNDATION INVESTIGATION
REPORT
by - Geocon, Limited.

Attention: Mr. S. McCombie.

Re: Addendum to Report S7127 on Soil
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Existing and Proposed Hwy. 17,
W.P. ~~918-59~~, Sault Ste. Marie, Ont.
703-60 District #18.

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ADDENDUM

TO

REPORT S7127

FOR

DEPARTMENT OF HIGHWAYS, ONTARIO

ON

SOIL CONDITIONS AND ENGINEERING STUDY

EXISTING AND PROPOSED HIGHWAY 17

W.P. 918-59 903-60

SAULT STE. MARIE

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Rexdale, Ontario,
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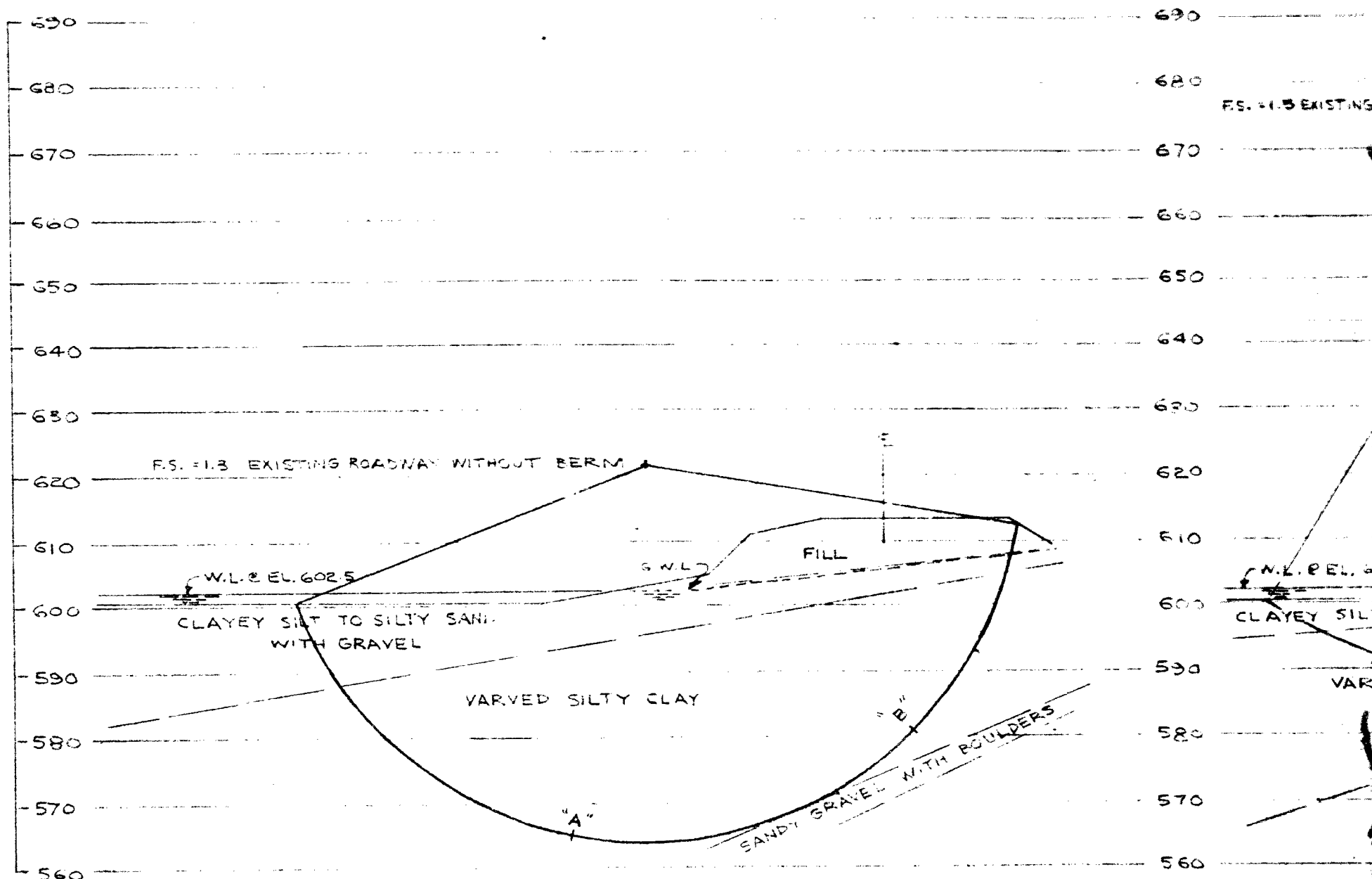
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GEOCON LTD



J. L. Seychuk, P. Eng.,
District Soils Engineer.

JLS/dw
S7127

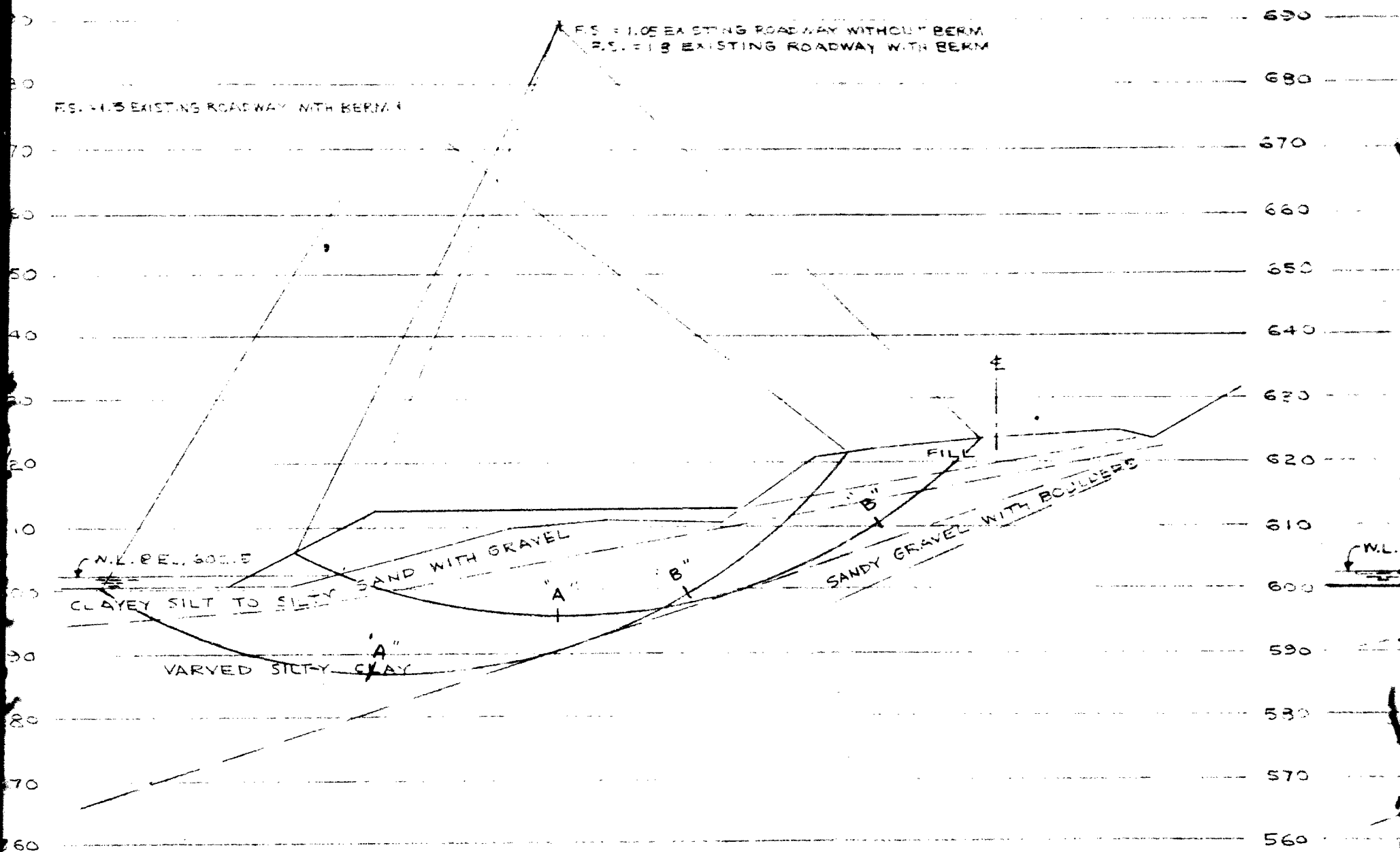


STATION 2054+50

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RE



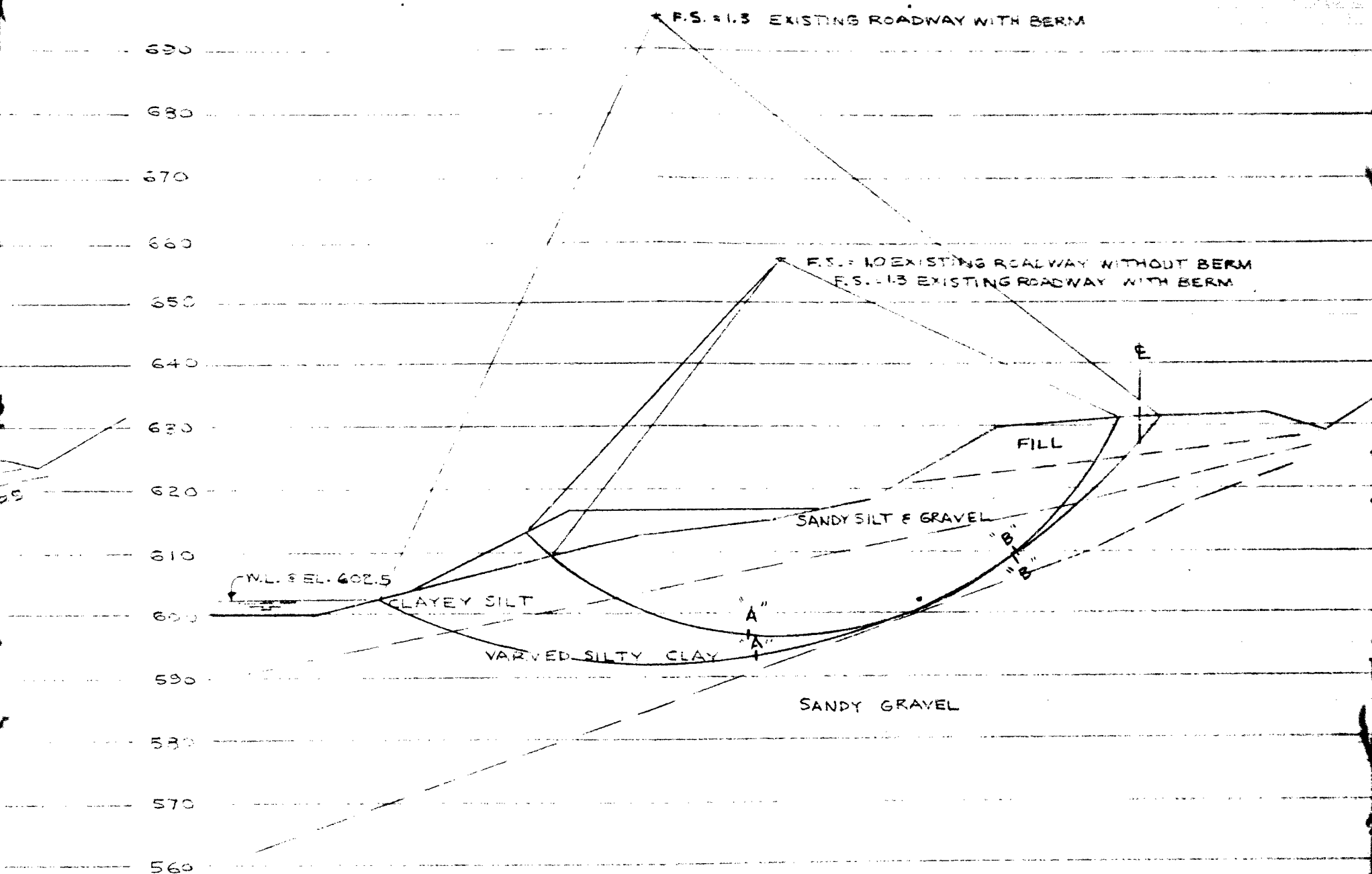
STATION 2056+60

LEGEND



RECOMMENDED BERMS

REFERENCE	
DWG. No.	DESCRIPTION
57127-1	GEOCON LTD - DWG. OF HIGHWAY 17, STATION 2059, BORING PLAN AND SOIL STRATIGRAPH



STATION 2058+00

41K
GEO

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DESCRIPTION
G. OF HIGHWAY 17, STA. 2054 TO
AN AND SOIL STRATIGRAPHY

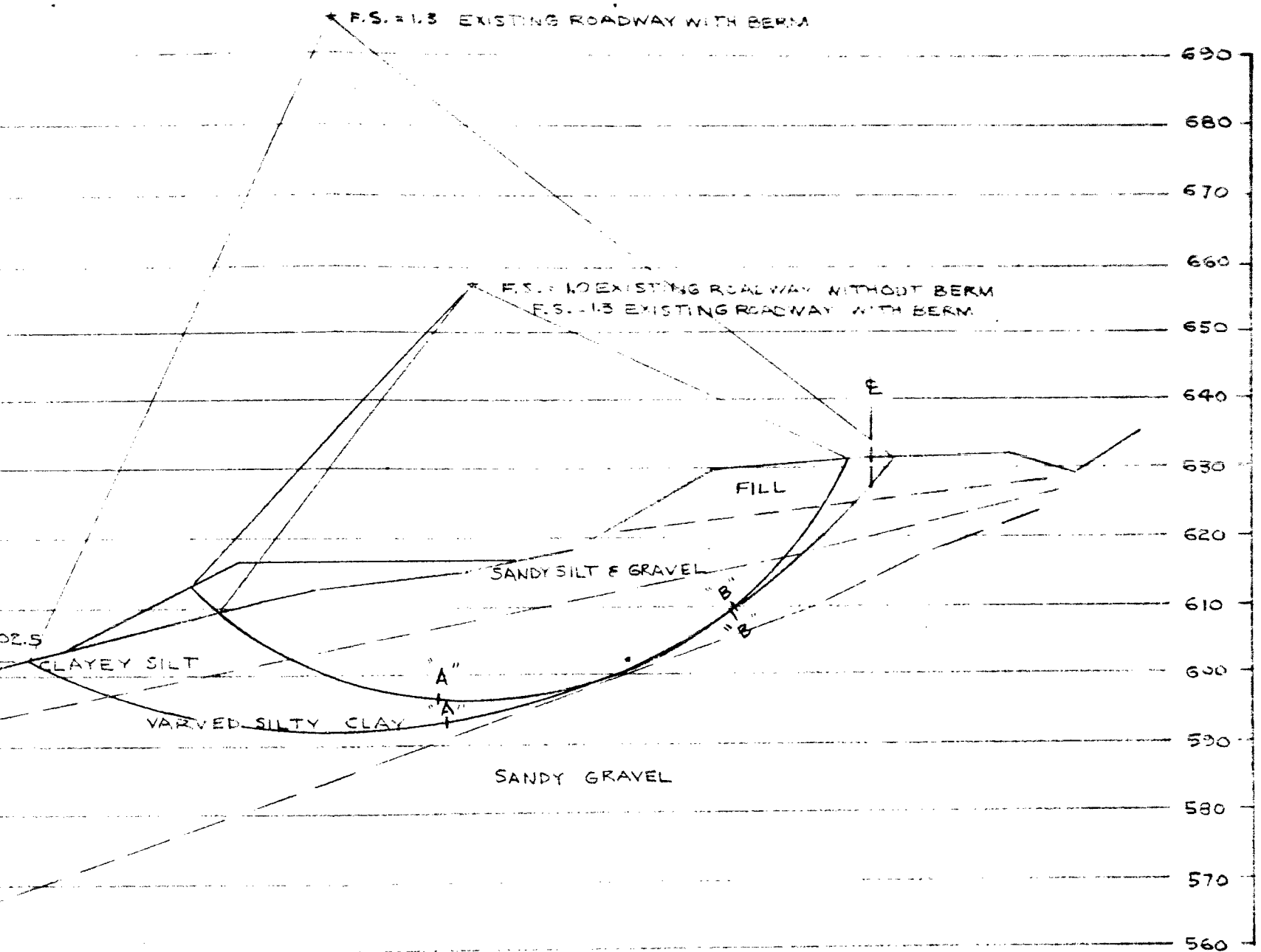
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TORONTO
SAULT STE. MARIE
HIGHWAY 17, STA. 2054 TO 2059
ONTARIO

SUMMARIZED STABILITY ANALYSES
EXISTING EMBANKMENTS & REQUIRED BERMS

GEOCON

DATE JAN. 13, 1961 SCALE

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GEOCRE No.

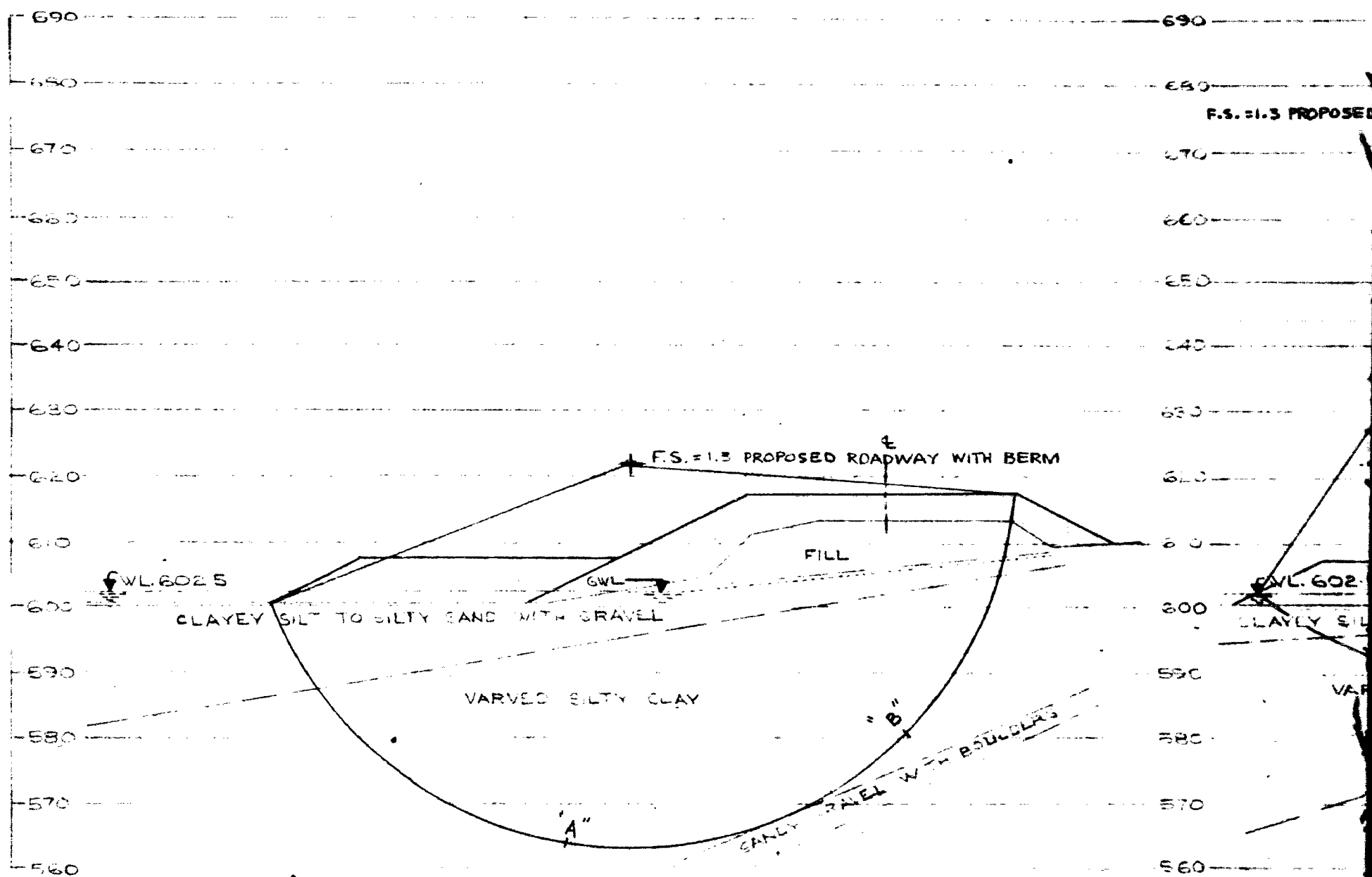
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TORONTO ONTARIO
HIGHWAY 17, STA. 2054 TO 2059
SAULT STE MARIE ONTARIO
SUMMARIZED STABILITY ANALYSES
EXISTING EMBANKMENTS & REQUIRED BERMS

GEOCON LTD

DATE JAN. 13, 1961 SCALE 1" = 20' - 0"

MADE M.W. CHKD. APPD.

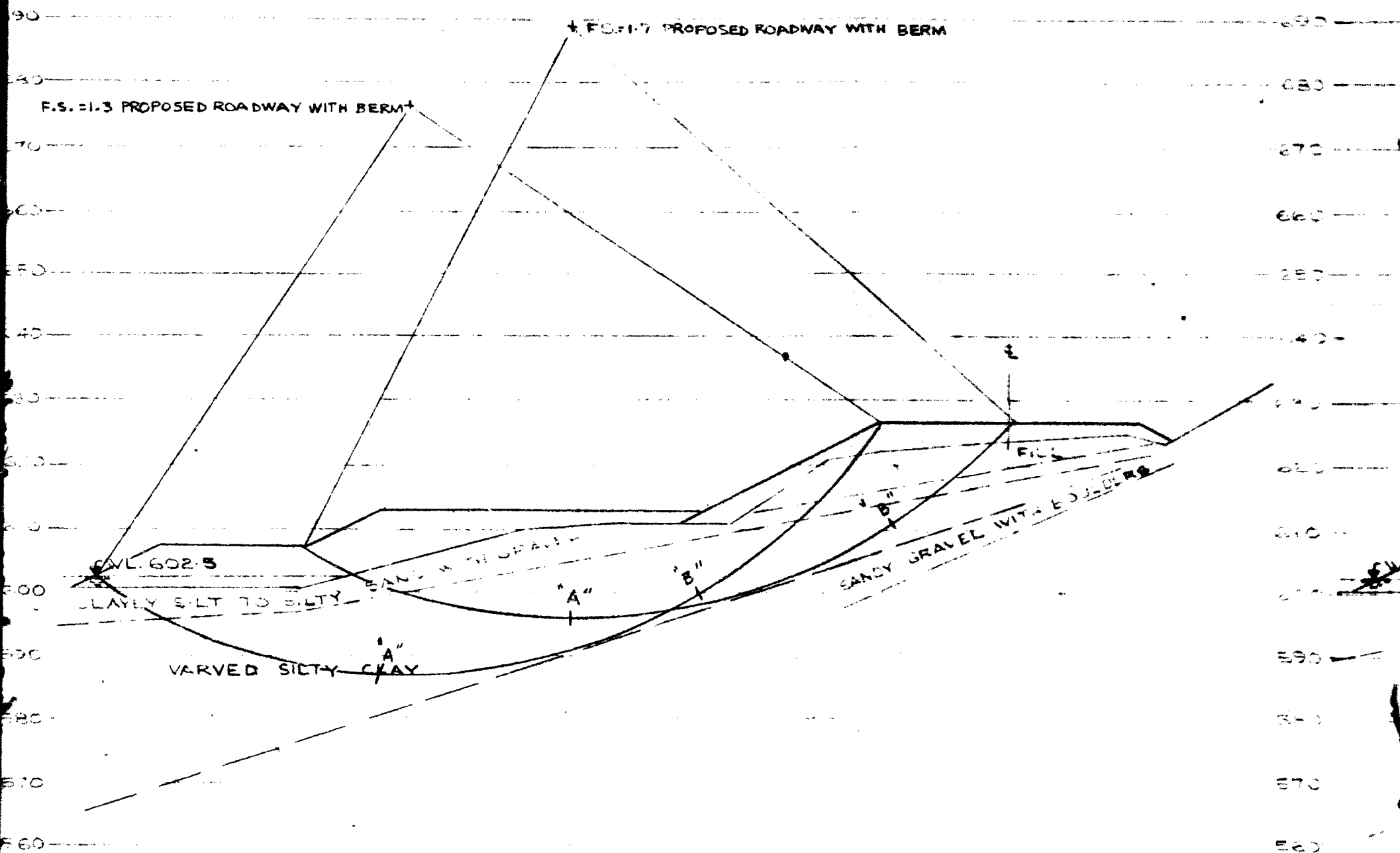
No. S 7127 - 4



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STATION 205+60

LEGEND

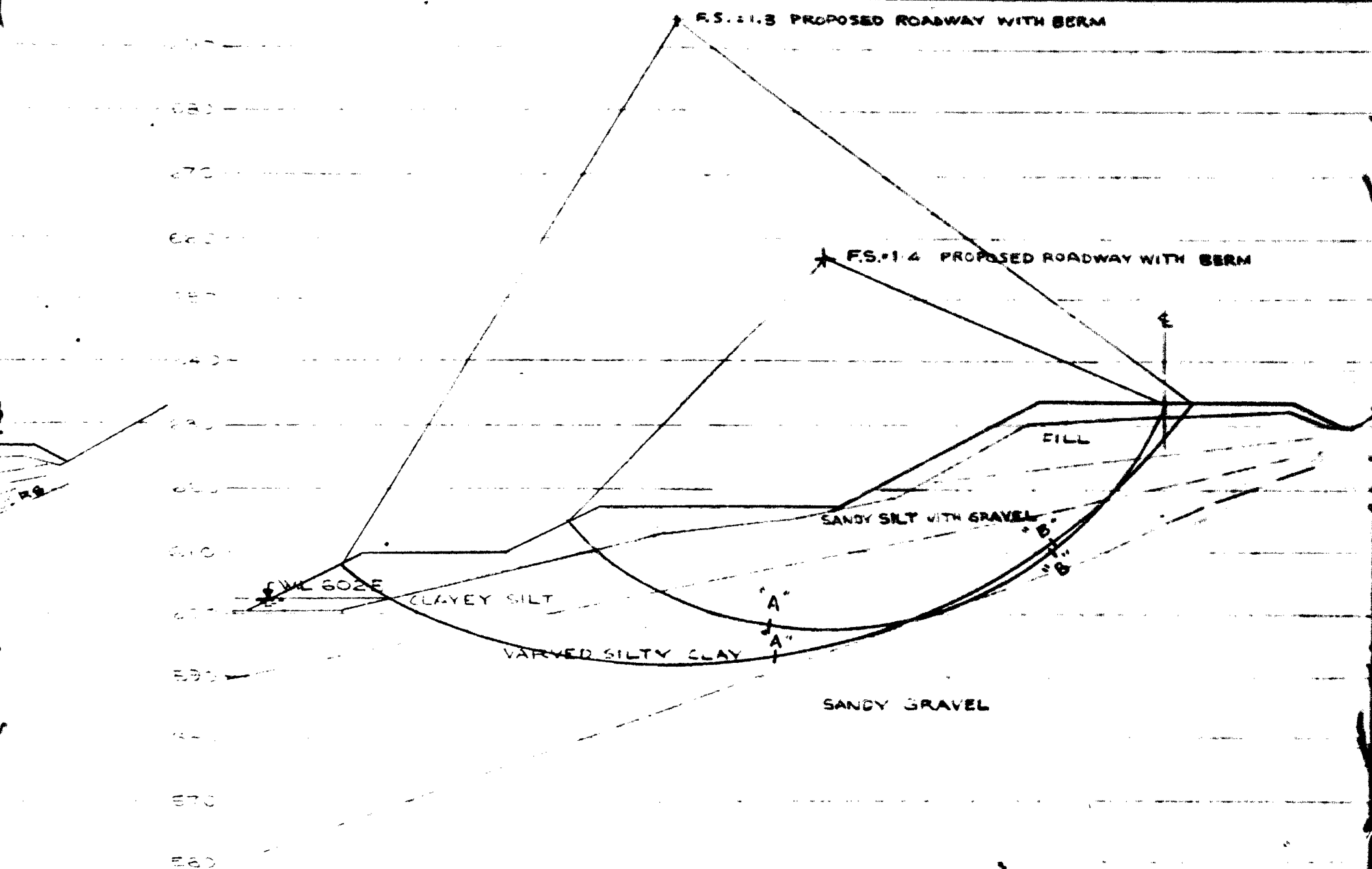


RECOMMENDED BERMS



PROPOSED EMBANKMENTS

DWG NO	REFERENCE
S 7127-1	DESCRIPTION
	GEOCON LTD BORING PLAN & SOIL STRAT



STATION 2058+00

REFERENCE
DESCRIPTION
DRIVING PLAN & SOIL STRATIGRAPHY

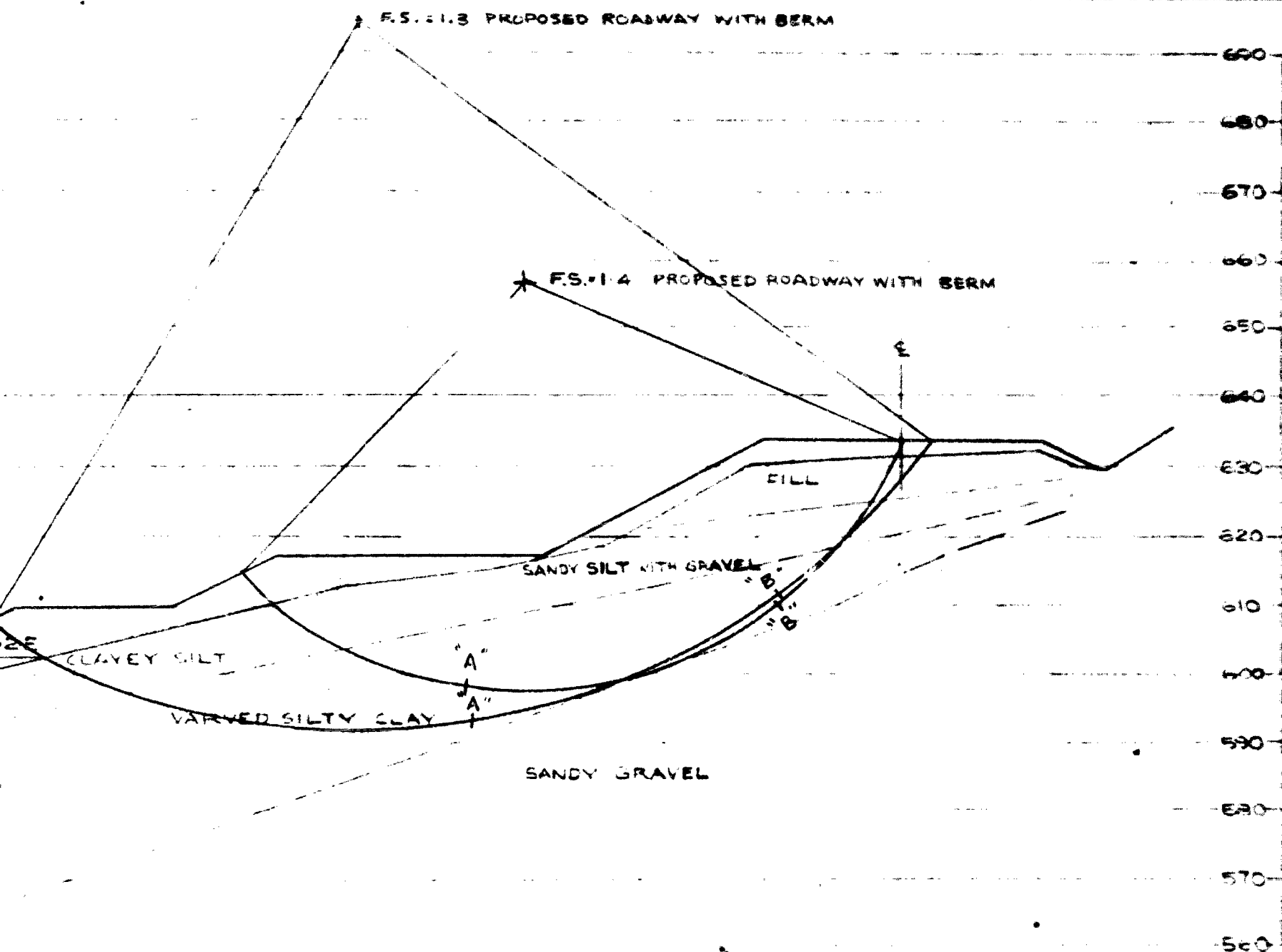
DEPARTMENT OF HIGHWAYS, ONTARIO
TORONTO
SAULT STE. MARIE
ONTARIO
HIGHWAY 17, STA 2054-2059

SUMMARIZED STABILITY ANALYSES
PROPOSED EMBANKMENTS & REQUIRED BERMS

GEOCON

DATE JAN. 13, 1961 SCALE

MADE DEC 12 1960
CHKD. 12 1960
APPD. 12 1960
No.



STATION 2058+00

41 K-15
GEOCRE No.

DEPARTMENT OF HIGHWAYS ONTARIO
TORONTO ONTARIO
— HWAY 17, STA 2054-2059
SAULT STE. MARIE ONTARIO
SUMMARIZED STABILITY ANALYSES
PROPOSED EMBANKMENTS & REQUIRED BERMS

GEOCON LTD

DATE JAN. 13, 1961 SCALE 1" = 20'-0"

MADE DEC 61 CHKD. 12 APPD. 210 No. S 7127-5