

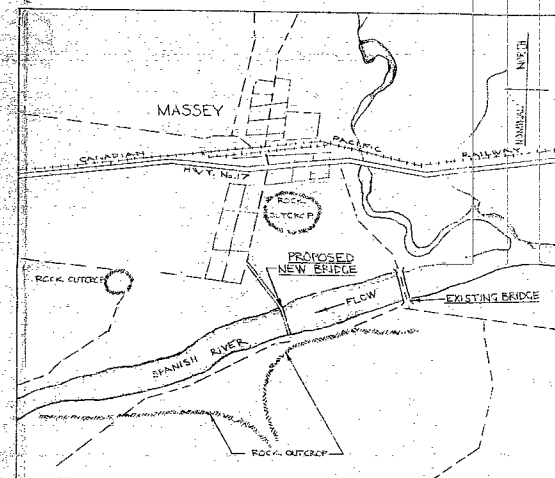
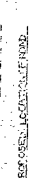
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W.P. # 25-57

SPANISH R.

CROSSING

MASSEY



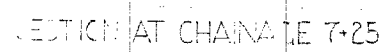
SEDROCK PROFILE INFERRED FROM  
RESISTIVITY SURVEY

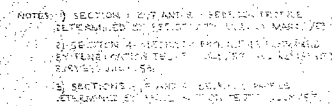
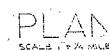
DEPARTMENT OF HIGHWAYS, ONTARIO  
TORONTO ONTARIO  
PROPOSED SPANISH RIVER BRIDGE  
WASSEY ONTARIO  
BEDROCK DETERMINATION BY RESISTIVITY

**GEOCON LTD**

DATE AUG. 15, 1958 SCALE AS SHOWN

MADE	CHKD.	APPD.	No. S 6508-1
M.W.	E.B.N.	V.M.	

HUGHES OWENS Co., No. 170H — TEACING PART



SOME DEFECTS IN NEGATIVE DUE  
TO CONDITION OF ORIGINAL DOCUMENTS

REVISIONS			REVISIONS			REFERENCE		REFERENCE	
MARK	DATE	DESCRIPTION	MARK	DATE	DESCRIPTION	DWG. NO.	DESCRIPTION	DWG. NO.	DESCRIPTION
						5653	REPORT BY GEOCON LTD./1957 - PROPOSED SPANISH RIVER BRIDGE - MASSEY, ONTARIO		
						609 657-674	AERIAL PHOTOGRAPHS OF MASSEY AREA SUPPLIED BY D.H.O. MASSEY		

DEPARTMENT OF HIGHWAYS, ONTARIO  
TORONTO

PROPOSED LOCATIONS - SPANISH RIVER BRIDGE  
MASSEY, ONTARIO

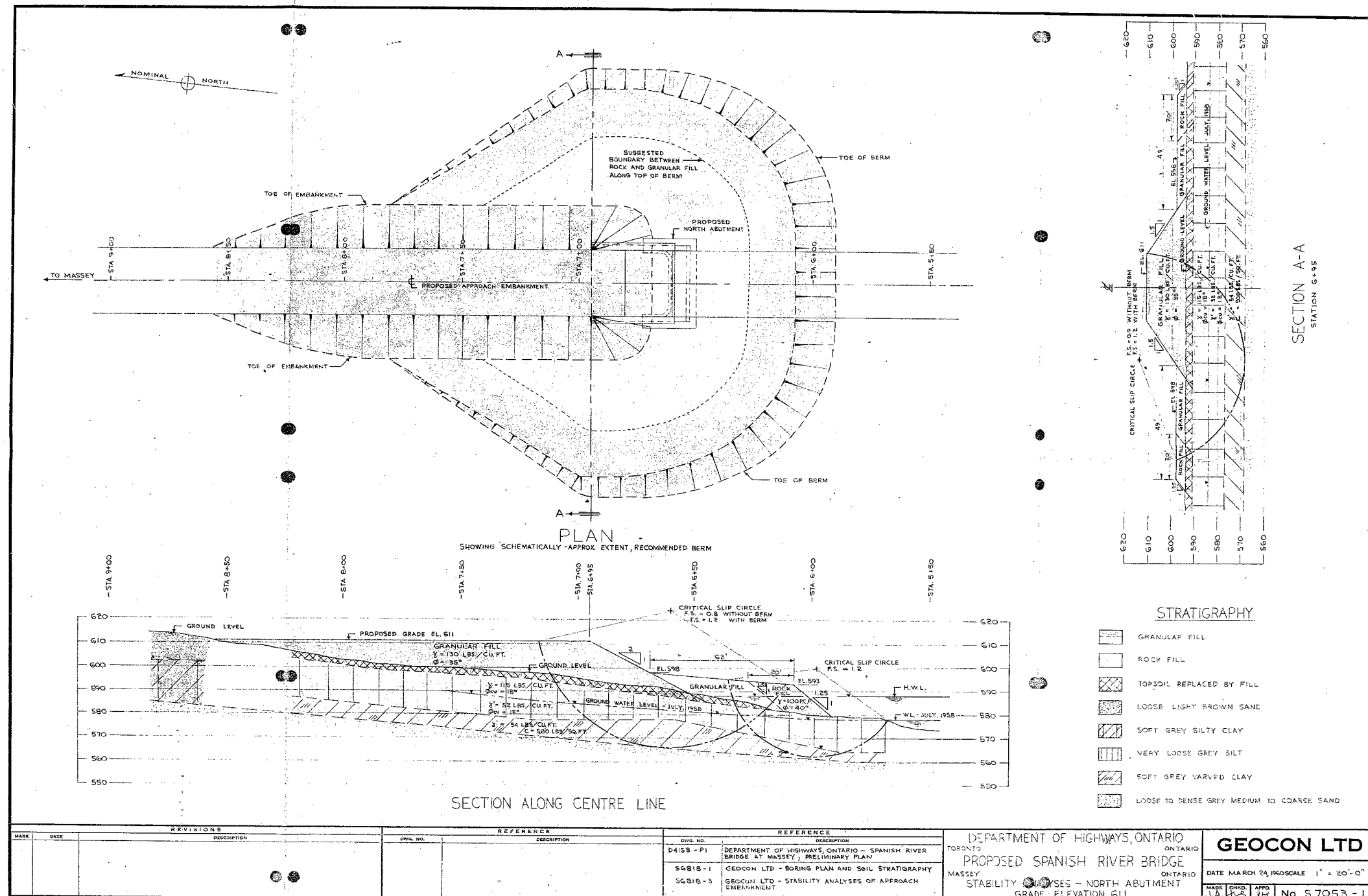
BEDROCK DETERMINATION BY RESISTIVITY

**GEOCON LTD**

DATE: MARCH 20, 1958 SCALE: AS SHOWN

MADE: *[Signature]* CHECKED: *[Signature]* APPR.: *[Signature]*

No. 5623-1



Mr. A. M. Toye,  
Bridge Engineer.  
Materials & Research Section.

206-198.  
April 19, 1960.

STABILITY ANALYSES -- by  
Geocon, Limited.

Attention: Mr. J. McCombie.

Re: M.P. 25-57 - District 18,  
North Approach Embankment,  
Proposed Spanish River  
Crossing, Massey, Ontario.

This memo accompanies a brief report by Geocon, Ltd., confirming berm requirements at the above structure location. You will recall that the initial proposal at this site showed the grade line above elevation 611' and, because of this, a large excavation of the soft subsoil underlying the North approach embankment was considered necessary.

Upon review of the design criteria, we were advised that a grade reduction to Elev. 611' was possible. Because of this grade reduction, we have determined that the subexcavation is not necessary, and that the approach embankment can be built to this elevation with minor berm requirements. The abutment location for the berm requirements shown in Geocon's report, will be 6 + 57.5

Detailed boring results at the specific abutment and pier locations have been forwarded to you in a separate report which was submitted to us by Franki of Canada, Ltd.

If you have any questions with respect to the construction procedures or berm requirements for this North approach, we would be pleased to discuss these with you.

L.C./MER

Attach.

cc: Messrs. A. M. Toye (2)

L. G. Soderman,

PRINCIPAL SOILS & FOUNDATIONS ENGINEER

H. A. Tregaskes  
D. C. Ramsay  
C. E. Hunter  
D. F. Collins  
E. F. Saint  
A. Watt

Foundations Office  
Gen. Office.

Mr. A. M. Toye,  
Bridge Engineer.  
Materials & Research Section,  
(Foundations Office).

August 4, 1961.

Re: Spanish River Bridge  
at Massey.  
W.P.25-57 - District #18.

Attention: Mr. S. McCombie.

With regard to our memo Aug. 2nd 1961, we would like to clarify what we mean by "borderline case". Normally for stability analyses we consider a factor of safety 1.2 as a minimum acceptable value.

In the particular case in question we estimated a factor safety of 1.25 assuming a total load of crane and girder of 65 Tons. We feel that factors such as vibration and impact may increase this value beyond 65 Tons, hence there is a possibility that the safety factor may become less than 1.2 and consequently less than our minimum acceptable value. For these reasons we would describe the situation as a borderline case.

L. G. Soderman,  
Principal Foundation Engr.  
Per:

MD/tt

c.c. Mr. C. S. Grebski  
Foundations Office  
Gen. Files.

(M. Devata,  
Project Foundation Engr.)

Mr. A. M. Teye,  
Bridge Engineer.  
Materials & Research Section,  
(Foundations Office).

August 2, 1961.

Re: Spanish River Bridge at  
Massey.  
W.F. 25-57 - Dist. #18.

Attention: Mr. S. McCombie.

In response to a request from your Mr. C. S. Grebski, we have reviewed the above project to ascertain whether or not a crane could be operated on the edge of the constructed berm. It is understood from the structural steel contractor that the total load of the crane including the weight of the girder, is 65 tons. The stability analyses indicate that this will be a borderline case of factor of safety. Therefore, we recommend that if the contractor takes full responsibility for any possible damages and delay in work, we would have no objections. In our opinion, the loading of the crane on the berm should not cause any damages to the abutment.

L. G. Soderman,  
PRINCIPAL FOUNDATION ENGR.  
Per:

MD/udeF

*M. Devata*  
(M. Devata,  
PROJECT FOUNDATION ENGR.)

cc: Mr. C. S. Grebski  
Foundations Office  
Gen. Files.



Mr. S. McCombie,

3rd June, 1959.

Bridge Engineer.

Re: Interim Report -  
Proposed Spanish River  
Crossing, Massey, Ont.

Materials & Research Section.

The interim report prepared by Geocon has been reviewed by the Foundation Office. Presented in this report are the results of recent and previously determined bed rock profiles at suggested river crossings. Geocon favour the bridge location at section 1, 2 or 8; however, a crossing at section 7 would seem feasible if traffic requirements warrant this location. A detailed foundation investigation will be undertaken when the foundation section is advised of the selected location.

L. Soderman  
Principal Soils & Foundation Engineer.

*K. Peaker*  
Per: K. Peaker  
Field Supervising Engineer.

KP:LG

c.c. A.M. Toye

A. Mantle

H. McMillan

W.S. Cole

E. Saint

File - L.G.S.✓

26/5/60

Mr. A. M. Toye,  
Bridge Engineer.  
Materials & Research Section.

May 18, 1960.

STABILITY ANALYSES -- by  
Geocon, Limited.  
(Additional Work)

Attention: Mr. D. McCombie.

Re: W.P. 25-57 - District 18,  
North Approach Embankment,  
Proposed Spanish River  
Crossing, Massey, Ontario.

Attached, is a brief letter submitted by Geocon, Ltd., dealing with the stability of the approach embankment at the above site. The initial report on this site which was submitted to you April 19, 1960, was reviewed in detail, by this Section. As a result of this review, it became apparent that Geocon's analyses had not taken into consideration, the weight of backfill within the abutment and above the 2 horizontal to 1 vertical slope shown on their drawing. Because of this, we asked Geocon to carry out additional analyses, taking this mass of soil into account. The results of this additional work are covered in the attached letter.

The summarizing conclusion from the work carried out, is that the slopes with berms, as defined by Geocon, should be followed in design.

LGS/PdeW  
Attach.

  
L. C. Boderman,  
PRINCIPAL SOILS & FOUNDATIONS ENGINEER

cc: Messrs. A. M. Toye (2)  
H. A. Tregaskes  
D. G. Ramsay  
C. E. Hunter  
D. P. Collins  
E. N. Saint  
A. Watt  
Foundations Office  
Gen. Files.

# GEOCON LTD

## HEAD OFFICE

180 VALLÉE ST., MONTREAL 18, QUEBEC  
TELEPHONE UN. 6-7692

## DISTRICT OFFICES

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

1425 WEST PENDER ST.  
VANCOUVER 8, B.C.  
TEL. MU. 1-8926

Rexdale, Ontario,  
May 16th, 1960.

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

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

Re: W.P. 25-57 - District 18,  
Stability Analyses,  
North Approach Embankment,  
Proposed Spanish River Crossing,  
Massey, Ontario.

Dear Sirs:

This letter confirms our discussions following the presentation of our report (S7053, dated April 11th, 1960) on the above work.

The stability analyses for the proposed approach embankment to the north abutment, shown on Drawing S7053-1 in the above report, were carried out for the abutment centreline of bearing at chainage 6+57.5. In the stability computations for the end slope of the embankment, the weight of the backfill within the abutment and above the 2 horizontal to 1 vertical slope shown on the drawing was ignored. Further computations carried out, including the total weight of backfill within the abutment, gave a minimum factor of safety of 1.1, against a circular failure through the end embankment and berm section.

It is considered however, that the major portion of the fill load within the abutment would be transmitted to the abutment piles. If this supporting effect of the piles is taken into account, the minimum factor of safety would be about 1.2, as given on the drawing.

Department of Highways, Ontario,  
May 16th, 1960,  
Page 2.

As stated in our report, to obtain a minimum factor of safety of 1.2 for the embankment end slope, a berm of the size shown on the drawing must be provided. The minimum factor of safety against a circular failure through the berm section alone has also been computed to be 1.2. Because of the proximity of the toe of the berm to the river channel, any increase in size of the berm would decrease the factor of safety below 1.2. To maintain a minimum factor of safety of 1.2 for stability, the construction of the berm to the size indicated on the drawing should be carefully controlled.

We believe that this letter, together with our previous report, gives all the information necessary to finalize the foundation design of the proposed bridge. If we can be of any further assistance, however, please give us a call.

Yours very truly,

GEOCON LTD



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

JLS/dw  
S7053

**GEOCON**

Mr. A. M. Teye,  
Bridge Engineer.  
Materials & Research Section.

May 13, 1960.

STABILITY ANALYSES -- by  
Gecon, Limited.  
(Additional Work)


Attention: Mr. S. McCombie.

Re: W.P. 25-57 - District 18,  
North Approach Embankment,  
Proposed Spanish River  
Crossing, Massey, Ontario.

Attached, is a brief letter submitted by Gecon, Ltd., dealing with the stability of the approach embankment at the above site. The initial report on this site which was submitted to you April 19, 1960, was reviewed in detail, by this Section. As a result of this review, it became apparent that Gecon's analyses had not taken into consideration, the weight of backfill within the abutment and above the 2 horizontal to 1 vertical slope shown on their drawing. Because of this, we asked Gecon to carry out additional analyses, taking this mass of soil into account. The results of this additional work are covered in the attached letter.

The summarizing conclusion from the work carried out, is that the slopes with berms, as defined by Gecon, should be followed in design.

LGE/MGEF  
attach.

  
L. G. Goderman,  
PRINCIPAL SOILS & FOUNDATIONS ENGINEER

cc: Messrs. A. M. Teye (2)  
B. A. Tregaskes  
D. G. Ramsay  
G. F. Hunter  
D. P. Collins  
E. J. Saint  
A. Watt  
Foundations Office  
Gen. Files.

# GEOCON LTD

HEAD OFFICE  
180 VALLÉE ST., MONTREAL 18, QUEBEC  
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REXDALE, TORONTO, ONT.  
TEL. CH. 4-8641

1425 WEST PENDER ST.  
VANCOUVER 8, B.C.  
TEL. MU. 1-8826

Rexdale, Ontario,  
May 16th, 1960.

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

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

Re: W.P. 25-57 - District 18,  
Stability Analyses,  
North Approach Embankment,  
Proposed Spanish River Crossing,  
Massey, Ontario.

Dear Sirs:

This letter confirms our discussions following the presentation of our report (S7053, dated April 11th, 1960) on the above work.

The stability analyses for the proposed approach embankment to the north abutment, shown on Drawing S7053-1 in the above report, were carried out for the abutment centreline of bearing at chainage 5+57.5. In the stability computations for the end slope of the embankment, the weight of the backfill within the abutment and above the 2 horizontal to 1 vertical slope shown on the drawing was ignored. Further computations carried out, including the total weight of backfill within the abutment, gave a minimum factor of safety of 1.1, against a circular failure through the end embankment and berm section.

It is considered however, that the major portion of the fill load within the abutment would be transmitted to the abutment piles. If this supporting effect of the piles is taken into account, the minimum factor of safety would be about 1.2, as given on the drawing.

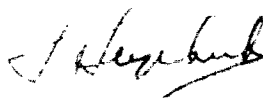
Department of Highways, Ontario,  
May 16th, 1960,  
Page 2.

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We believe that this letter, together with our previous report, gives all the information necessary to finalize the foundation design of the proposed bridge. If we can be of any further assistance, however, please give us a call.

Yours very truly,

GEOCON LTD



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

JLS/dw  
S7053

GEOCON

Mr. A. M. Toye,

July 5, 1960.

Bridge Engineer.

Spanish River Bridge at Massey  
Development Road,  
W.P. 25-57 -- District #18.

Materials & Research Section.

Attention: Mr. S. McCombie.

In response to a request from your Mr. C. S. Grebski, we have reviewed the General Layout drawings of the above structure.

We have found that, in your design, you have shortened the berms in a direction parallel to the road centre line, by a distance of 14 feet. The stability analyses which have been carried out, indicate that a berm length of 62 feet must be maintained in order to keep the safety factor equal to 1.2. With the berm length as shown on your drawings, the stability of the slope is not assured.

In order to maintain a safety factor of 1.2 with respect to approach embankment failure, it will be necessary to increase the berms shown on your drawings so that they agree with the dimensions, as given on Geocon's Drawing S 7053-1.

Drawings returned herewith.

If you have any further questions with respect to this project, please contact our Office.

*L. G. Soderman*

LGS/MdeF

L. G. Soderman,  
PRINCIPAL FOUNDATIONS ENGINEER

cc: Foundations Office  
Gen. Files



Toronto 5,  
October 22, 1958.

MEMORANDUM TO:

Mr. A. Rutka,  
Acting Materials and Research  
Engineer,  
Department of Highways,  
Room 103,  
Downsview, Ontario.

RE: W.P. 25-57  
B.A. 643-B, Spanish  
River Bridge, Salter  
Twp. Rd., south of  
Massey, District #18.

Attached please find further soil  
report BA 643-B for the above structure.

*J.C. McAllister*

JCMc:CP  
Attach.

J.C. McAllister,  
for S. McCombie,  
Bridge Planning Engineer.

*Handwritten notes:*  
1. ...  
2. ...  
3. ...

*Handwritten notes:*  
24/1/60  
200

23-60-278.  
**GEOCON LTD**

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

Rexdale, Ontario,  
October 20th, 1958.

BA643-B  
DISTRICT OFFICES  
14 HAAS ROAD  
REXDALE, TORONTO, ONT.  
TEL. CH. 4-8641

3355 WEST BROADWAY AVE.  
VANCOUVER 8, B.C.  
TEL. CH. 5810

Department of Highways, Ontario,  
Parliament Buildings,  
Toronto 5, Ontario.

Attention: Mr. A. M. Teye, P. Eng.,  
Bridge Engineer.

Re: Soil Investigation,  
North Bank,  
Proposed Spanish River Bridge,  
Massay, Ontario.

58F208C

Dear Sirs:

This letter accompanies our detailed report on the above soil investigation together with our recommendations for foundation design.

We find that the north bank of the Spanish River at the site is covered by about 25 feet of loose silt and soft clay underlain by a very thick stratum of loose to compact coarse sand which becomes very dense with depth.

Based upon our detailed study, it is considered that the piers and north abutment should be founded on steel H piles driven to bedrock or to refusal in the very dense sand stratum at depth. As the stability of the north approach embankment is controlled by the thin upper loose silt and soft clay strata, it is recommended that stabilizing measures be taken. These are discussed in detail in the report. Of the alternatives suggested, it is considered that excavation and backfilling in the local area of the proposed abutment location would be the most economical solution.

We feel that this report gives all the information necessary for safe and economical foundation design. If we can be of any further service, please do not hesitate to contact us.

Yours very truly,

GEOCON LTD

*V. Milligan*  
V. Milligan, P. Eng.,  
District Engineer.

VM/dw  
S6818



ST. JOHN'S

HALIFAX

QUEBEC

MONTREAL

TORONTO

LONDON

VANCOUVER



36816  
REPORT  
TO  
DEPARTMENT OF HIGHWAYS, ONTARIO  
ON  
SOIL CONDITIONS AND FOUNDATIONS  
NORTH BANK  
PROPOSED SPANISH RIVER BRIDGE  
MASSEY                      ONTARIO

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## INTRODUCTION

Gecon Ltd has been retained by the Department of Highways, Ontario (proposal dated July 30th, 1958 and accepted August 18th, 1958) to carry out a soils investigation at the site of the north abutment of the proposed Spanish River Bridge, Massey, Ontario.

The object of the investigation was to interpret the soil conditions as they effect the stability of the proposed north embankment and the design of the north abutment.

## SUMMARIZED SOIL CONDITIONS

The upper 25 feet of soil at the site consists chiefly of very loose grey silt underlain by soft grey varved clay. A very thick stratum of medium to coarse sand underlies the varved clay to the depth explored by all of the borings and probably extends down to bedrock. The sand is loose to compact down to a depth of about 90 feet, below which it becomes compact to dense with very dense silty layers. Bedrock below the north bank, as determined by a resistivity survey carried out in conjunction with the soils investigation, varies in elevation from about 505 to less than 430.

## DISCUSSION

It is understood that the proposed bridge will be a three span simply supported steel truss structure. It is tentatively planned to found the piers and abutments on end-bearing steel H piles, save for the south abutment which will be founded directly on bedrock. The proposed location of the piers and abutments is shown on Drawing S6815-2. Bedrock elevation was determined by a previous geophysical survey and the scope of this present study was specifically to examine the foundations of the north abutment and approach embankment.

Based on a study of the general soil conditions, it is considered that the north abutment and bridge piers should be carried on a piled foundation, as proposed. Computations were carried out to determine the variation of ultimate pile capacity with depth for various pile types, and the results are summarized on Figure 5 of Appendix II. It may be seen that various types

may be used acting as friction piles, but because of the comparatively low increase in capacity with depth due to skin frictional resistance, the most effective piled foundation using the proposed pile type may be obtained by driving the piles to refusal in the dense silt and sand stratum or to bedrock. If this is done and taking a 12 inch 53 lb. H pile for example, it is estimated that the ultimate pile capacity will be of the order of 160 tons, and therefore that a load of 50 tons may be used for design purposes. However, irrespective of the pile type chosen, it is recommended that the design load be confirmed in the field by pile loading tests. It is also recommended that at least 5 feet of earth cover be provided at the base of the abutment for adequate frost protection. } N3

Stability analyses were carried out for the approach embankment for a maximum height of 25 feet at the initial proposed location of the north abutment at chainage 6+55.5, and the results are shown on Drawing S6818-3. They show that the critical height of embankments in this area is controlled by the thickness and strength properties of the loose silt and soft clay strata immediately overlying compact sand at about elevation 570. It was determined that under the above conditions the factors of safety against rotational failure of both the end slope and the side slopes of the embankment were below 1.0, and that it was not practical to increase the factor of safety to the minimum desired value of 1.3 by either flattening the side slopes or with the use of berms. However, an adequate factor of safety against sliding may be obtained by moving the abutment about 100 feet further from the river to chainage 7+58, where the height of the approach embankment is reduced to about 15 feet. With the abutment at this location, an additional span to the proposed bridge would be required. In addition, provision would have to be made in design to accommodate settlement of the embankment behind the abutment of the order of 2 inches and also minor horizontal translatory movement due to creep of the soft clay deposit under the weight of the embankment. Due to this effect, an allowance should be made in design for a possible 3 or 4 inches of horizontal movement at the top of the abutment.

Because the relatively thin upper loose silt and soft clay strata control the critical height of embankments, the replacement of these deposits by granular fill was studied, in order to retain the abutment in the original location and minimize the effects of creep. It was found that such a scheme was practical and it is recommended that the arrangement shown on Drawing S6818-4 be adopted. This scheme involves the excavation and backfilling of about 12,000 cubic yards of material and is probably more economical than relocation of the abutment. In order to ensure a minimum factor of safety of 1.3, the backfill material and the berms specified should consist of coarse well graded granular fill. Further, the berms shown should be well compacted during placing, in lifts not greater than 1 foot in thickness.

It is further recommended that all organic topsoil be stripped from the areas of construction of embankments and berms before placing backfill material.

It should be noted that the cut slopes of 30 degrees shown on the drawing are stable for temporary construction provided that the cut is kept filled with water above river level during excavation, and that the cut is backfilled immediately following removal of the silt and clay.

With the above procedure, consolidation settlement of the embankment adjacent to the abutment will be small. Settlement will reach a maximum computed value of about 4 inches just clear of the excavated area at about chainage 7+60.

#### CONCLUSIONS AND RECOMMENDATIONS

1. The site of the north abutment is covered by about 15 feet of very loose silt, then 10 feet of soft clay followed by over 70 feet of compact coarse sand overlying further dense sand deposits then bedrock.

2. The water level in the river at the time of the investigation was at elevation 578. Groundwater level was generally within 10 feet of ground surface.

CONCLUSIONS AND RECOMMENDATIONS (continued)

4.

3. Stability analyses show that the north abutment may only be retained in the proposed location provided that stabilising measures are taken. The recommended measures involve excavating and backfilling in the local area of the abutment and the use of side berms to the approach embankment as shown in the suggested arrangement on Drawing S6818-4.

4. Alternatively, for adequate stability, the north abutment could be relocated approximately 100 feet further from the river, at chainage 7+58.

5. It is considered that the most satisfactory foundation type for the bridge piers and north abutment would be end bearing piles, as proposed.

PERSONNEL

The field work was carried out under the supervision of Mr. R. M. Quigley. The report was written by Mr. Quigley, checked by Mr. V. Milligan and reviewed by Mr. M. A. J. Matich.

VW/dw  
36818





## APPENDIX I

Procedure

Site and Geology

Soil Conditions

Water Conditions

Office Reports on Soil Exploration

## PROCEDURE

The field work was carried out between July 28th and August 6th, 1958 inclusive. Three boreholes, two with accompanying dynamic penetration tests and one additional penetration test were put down using a skid-mounted machine drillrig to depths varying between 51 and 97 feet. The locations of the borings and a section of the inferred soil stratigraphy are shown on Drawing S6318-1 located in the pocket at the rear of this report. Detailed logs of the borings are given on the Office Reports on Soil Exploration in this Appendix.

The soil testing was carried out in the Toronto Soils Laboratory of Geocon Ltd and the results are plotted on the Office Reports and on the Figures in Appendix II. The soil samples remaining after testing will be stored until March 1st, 1959 and then destroyed unless other instructions are received.

Elevations of the boreholes are Geodetic and are referred to the elevations of various stations along the centre line of the proposed road as determined by Department of Highways surveyors at the site prior to the start of drilling operations.

## SITE AND GEOLOGY

The site investigated is located on the north bank of the Spanish River at the south edge of the town of Massey. The proposed bridge location follows "section 3" of a previous Geocon preliminary investigation, S6524, carried out to determine the soil conditions along various possible bridge locations.

From previous investigation and available geological information, it is known that silts and sands of great thickness fill the valleys and depressions between rugged steep sided outcrops in the area. Bedrock in the vicinity of the site consists primarily of Precambrian diorite or diabase of intrusive origin.

## SOIL CONDITIONS

II.

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

### Topsoil

A few inches to 3 feet of very loose dark brown to black organic sandy to silty topsoil covers the whole site except where removed by wave action along the shore of the river.

### Gray Silt

A stratum of very loose gray sandy to clayey silt varying in thickness from about 9 to 15 feet was encountered beneath the topsoil in all borings. The silt was wet and stratification was generally indistinct. A small amount of finely disseminated organic material is present in the upper 3 or 4 feet of the stratum in the vicinity of borehole 4.

Standard penetration resistance or "N" values averaging 2 blows per foot indicate that the silt is very loose in relative density.

Unconfined compression tests carried out on two undisturbed samples of the silt gave compressive strengths of 0.5 and 0.6 tons per square foot. Three consolidated quick triaxial tests were also performed on samples of the silt. The stress strain curves for these tests are plotted on Figure 1 in Appendix II. The consolidated undrained angle of shearing resistance,  $\phi_{cu}$ , was found to be about 18 degrees. The Mohr circles and the corresponding shear strength envelope are given on Figure 2 in Appendix II.

An average wet unit weight of 115 pounds per cubic foot and an average moisture content of 40 percent were obtained for the stratum. For design purposes, a submerged unit weight of 52 pounds per cubic foot was used. Two liquid limits of 36 and 27 and two plastic limits of 29 and 23 were obtained for two silt samples having natural moisture contents of 43 and 25 percent respectively.

Varved Clay

A stratum of grey varved clay varying in thickness from 6 to 13 feet was encountered beneath the silt stratum in all borings. The individual silt and clay layers average about 1/4 inches in thickness.

One standard penetration resistance value of 3 blows per foot was obtained for the clay. Three unconfined compressive strengths of 0.3 and 0.5 and 6.55 tons per square foot were obtained for undisturbed samples of the clay indicating that it is soft to firm in consistency. Three consolidated quick triaxial tests were also run on samples of the clay. The stress strain curves for these tests are shown on Figure 3 of Appendix II. The Mohr circles and corresponding strength envelope are plotted on Figure 4 of Appendix II. The consolidated undrained angle of shearing resistance  $\phi_{cu}$ , was found to be about 15 degrees.

An average wet unit weight of 118 pounds per cubic foot and an average moisture content of about 36 percent were obtained for the stratum. Liquid and plastic limits of about 31 and 22 respectively were obtained for a varved clay sample having a natural moisture content of about 32 percent.

Coarse to Medium Sand

A stratum of grey coarse to medium sand was encountered beneath the varved clay to the depth explored in all boreholes. The sand is generally well sorted as shown by the grain size curve given on Figure 5 in Appendix II. The individual grains are generally subrounded to subangular, although many well rounded grains are present. The grains consist chiefly of quartz, feldspar and amphibole. In borehole 1, at a depth of about 89 feet, layers of grey silt, 2 or 3 inches in thickness, were encountered within the sand stratum.

Coarse to Medium Sand (continued)

The standard penetration resistance or "N" values obtained in the sand are very erratic and in some cases possibly unreliable due to the presence of very coarse grain sizes. Binding between the sampler and casing due to the displaced sand during driving was frequently encountered. Based on the dynamic penetration tests and the lowest "N" values obtained, it is considered that the sand is loose to compact in relative density down to a depth of about 90 feet in borehole 1. For design purposes an average "N" value of 15 blows per foot was used for the coarse sand above an elevation of about 500. When corrected for overburden pressure an average "N" value of between 20 and 30 blows per foot is obtained. In borehole 1, below a depth of about 90 feet, the sand stratum becomes dense to very dense, apparently due to the presence of grey silt layers. Below the depth explored by the borings, the stratum probably consists of compact to dense coarse sand with very dense finer layers.

For design purposes, it is recommended that saturated and submerged unit weights of 125 and 62 pounds per cubic foot respectively be used for the sand stratum. The sand is estimated to have an angle of internal friction of 35 degrees down to about elevation 500 and 40 degrees below this elevation.

WATER CONDITIONS

Groundwater level at the site during the investigation was found to occur between 2 and 10 feet below ground surface sloping from north to south down to river level elevation. River water level at the time of the investigation was approximately at elevation 578. The silt stratum is considered to be 100 percent saturated by capillarity where the groundwater level is below the top of the stratum.

# 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**

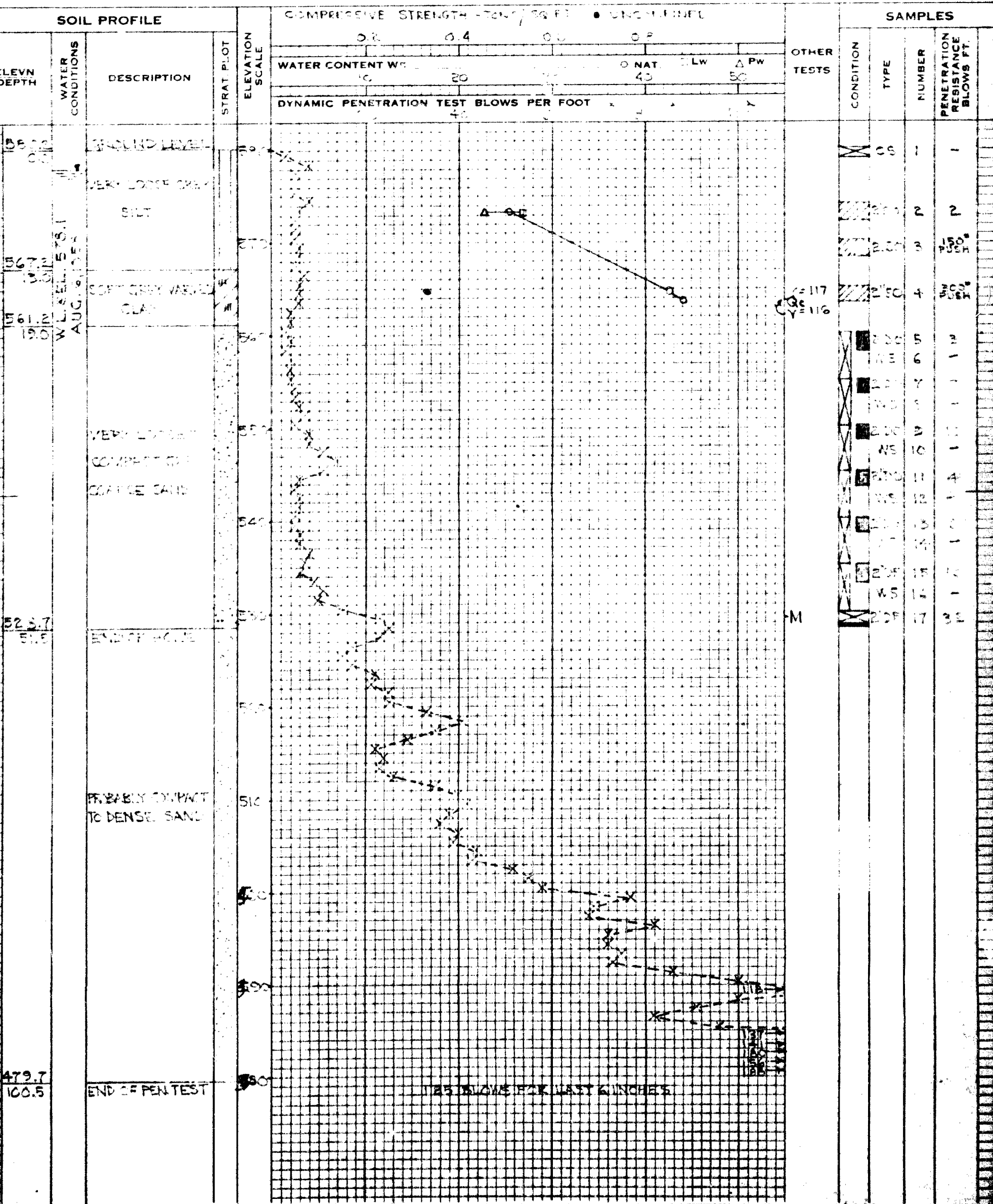


# GEOCON

## OFFICE REPORT ON SOIL EXPLORATION

CONTRACT: 0081 BORING #: 2 DATUM: GEOMETRIC CASING: 2A  
 BORING DATE: AUGUST 1962 REPORT DATE: AUGUST 1962 COMPILED BY: JMA CHECKED BY: J.A.  
 SAMPLER HAMMER WT. 100 LBS DROP 30 INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN - LBS. ENERGY)

SAMPLE CONDITION		SAMPLE TYPES		ABBREVIATIONS	
	DISTURBED	A.S. - AUGER SAMPLE	F.S. - FOIL SAMPLE	V. - IN-SITU VANE TEST	γ - WET UNIT WEIGHT
	FAIR	ST - SLOTTED TUBE	SO - SLEEVE-OPEN	M - MECHANICAL ANALYSIS	K - PERMEABILITY
	GOOD	W.S. - WASHED SAMPLE	SF - SLEEVE-FOOT VALVE	U - UNCONFINED COMPRESSION	C - CONSOLIDATION
	LOST	DO - DRIVE-OPEN	TO - THIN WALLED OPEN	QC - TRIAXIAL CONSOLIDATED QUICK	WL - WATER LEVEL IN CASING
		DF - DRIVE-FOOT VALVE	R.C. - ROCK CORE	Q - TRIAXIAL QUICK	WT - WATER TABLE IN SOIL
		CS - CHUNK SAMPLE		S - TRIAXIAL SLOW	



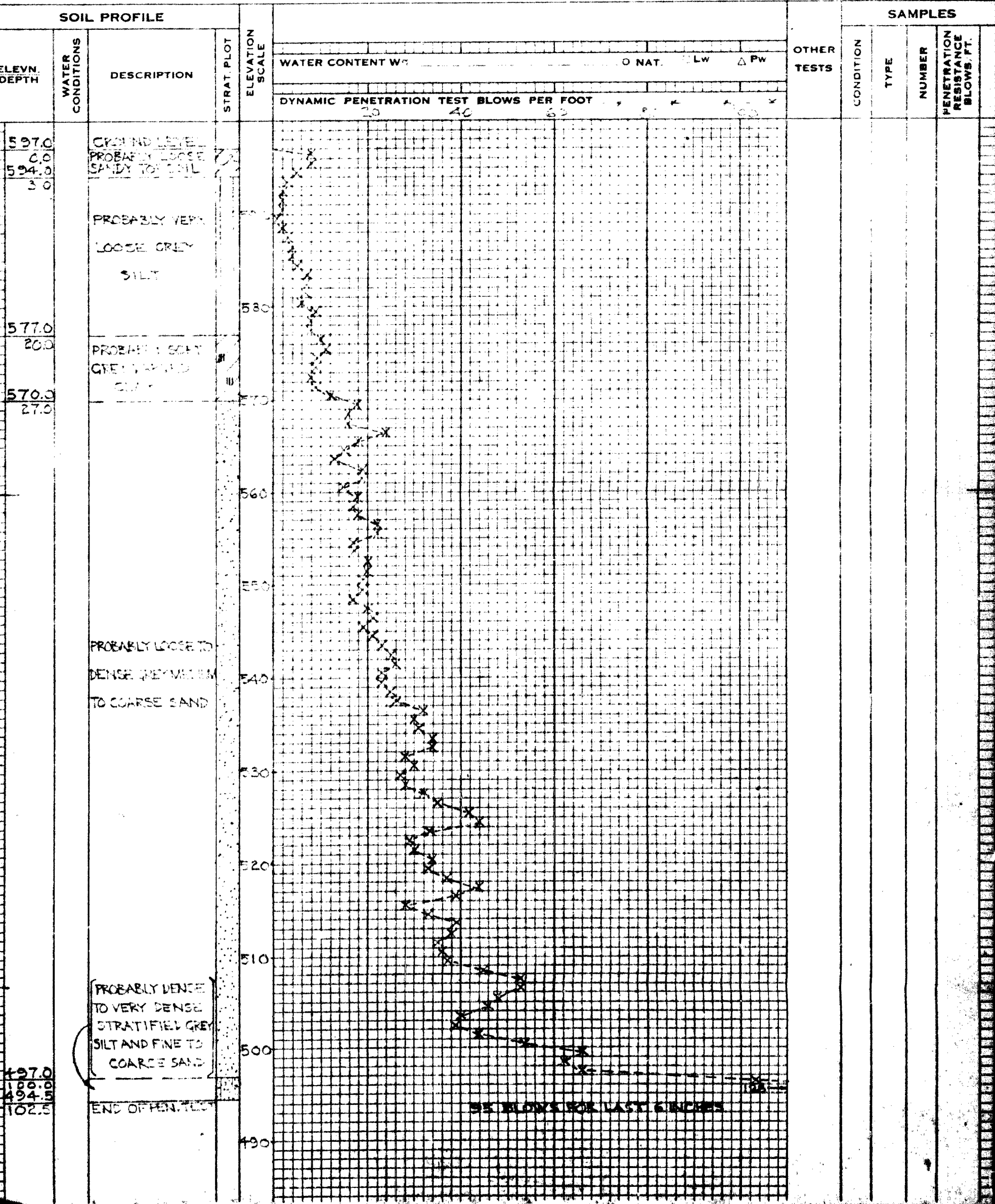


GEOCON

OFFICE REPORT ON SOIL EXPLORATION

CONTRACT 02213 PENETRATION TEST 3 BORING 3 DATUM GEOMETIC CASING  
BORING DATE AUG. 4, 1953 REPORT DATE AUG. 11, 1953 COMPILED BY MMV CHECKED BY J.A.  
SAMPLER HAMMER WT. 140 LBS. DROP 30 INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN - LBS. ENERGY)

SAMPLE CONDITION SAMPLE TYPES ABBREVIATIONS  
DISTURBED A.S. - AUGER SAMPLE F.S. - FOIL SAMPLE V - IN-SITU VANE TEST  
FAIR S.T. - SLOTTED TUBE S.O. - SLEEVE-OPEN M - MECHANICAL ANALYSIS  
GOOD W.S. - WASHED SAMPLE S.F. - SLEEVE-FOOT VALVE U - UNCONFINED COMPRESSION  
LOST D.O. - DRIVE-OPEN T.O. - THIN WALLED OPEN Qc - TRIAXIAL CONSOLIDATED QUICK  
D.F. - DRIVE-FOOT VALVE R.C. - ROCK CORE Q - TRIAXIAL QUICK  
C.S. - CHUNK SAMPLE WL - WATER LEVEL IN CASING  
WT - WATER TABLE IN SOIL



# GEOCON

## OFFICE REPORT ON SOIL EXPLORATION

CONTRACT 34816 BORING # 4 DATUM GEODETIC CASING BK  
 BORING DATE AUG. 5, 1958 REPORT DATE AUG. 13, 1958 COMPILED BY MW CHECKED BY JA  
 SAMPLER HAMMER WT. 140 LBS. DROP 30 INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN. LBS. ENERGY)

### SAMPLE CONDITION



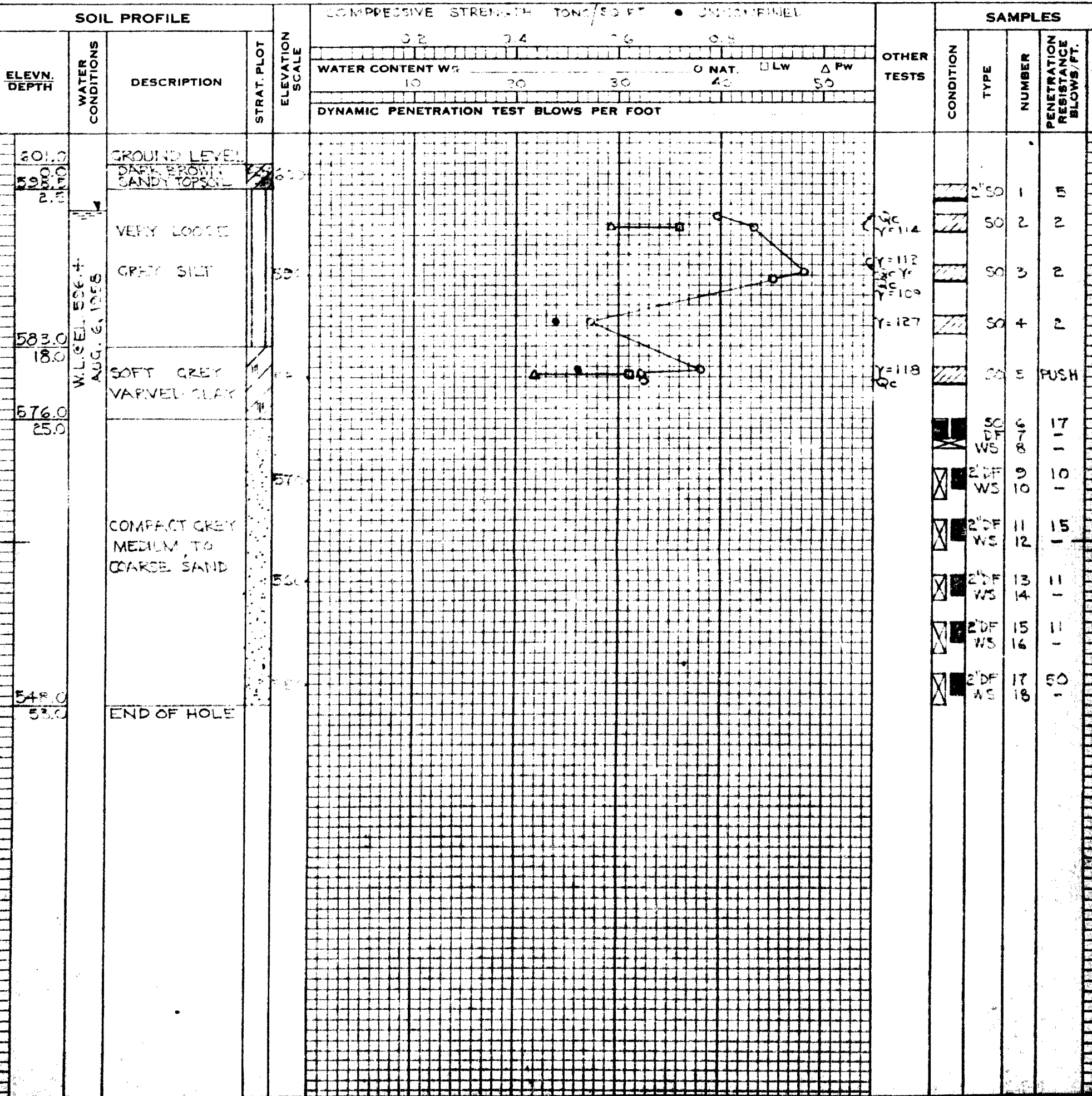
A.S. - AUGER SAMPLE  
 S.T. - SLOTTED TUBE  
 W.S. - WASHED SAMPLE  
 D.O. - DRIVE-OPEN  
 D.F. - DRIVE-FOOT VALVE  
 C.S. - CHUNK SAMPLE

### SAMPLE TYPES

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

### ABBREVIATIONS

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



**APPENDIX II**

**FIGURES - LABORATORY TESTING**

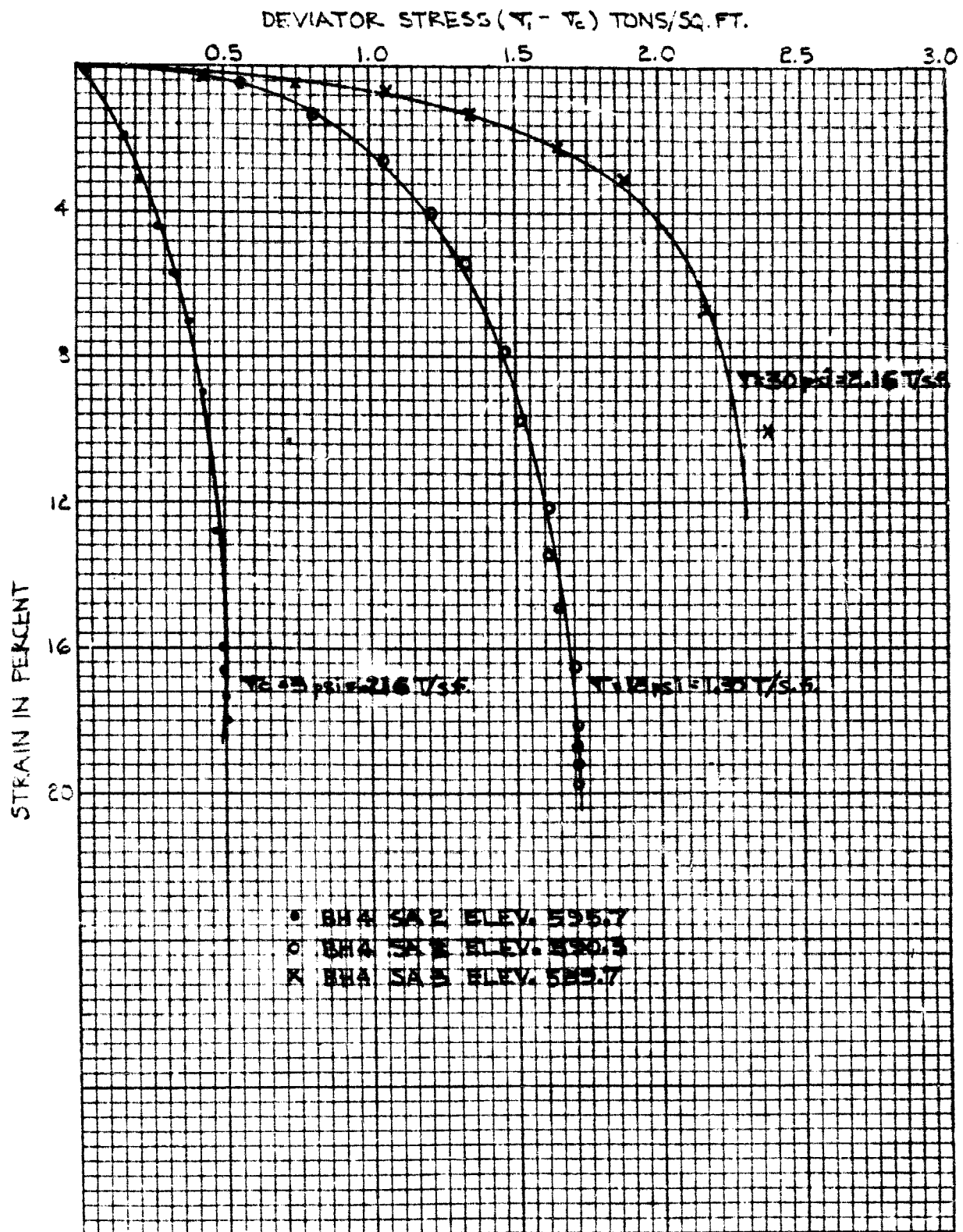
**GEOCON**

# CONSOLIDATED QUICK TRIAXIAL TESTS

## STRESS STRAIN CURVES

### VERY LOOSE GREY SILT

APPENDIX II  
FIGURE 1  
PROJECT 56818



# MOHR CIRCLES

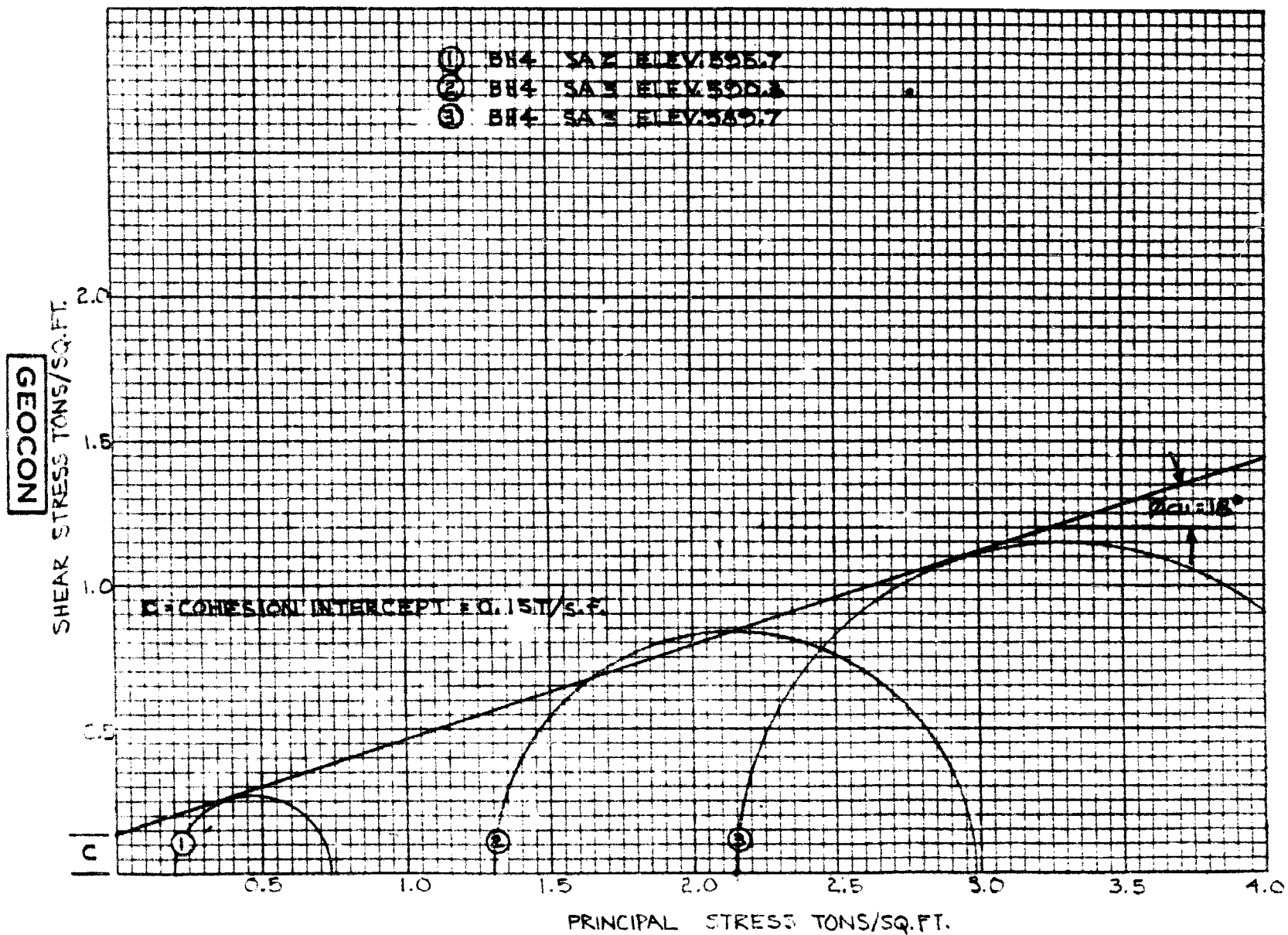
APPENDIX II

## CONSOLIDATED QUICK TRIAXIAL TESTS

FIGURE 2

VERY LOOSE GREY SILT

PROJECT 56818



# CONSOLIDATED QUICK TRIAXIAL TESTS

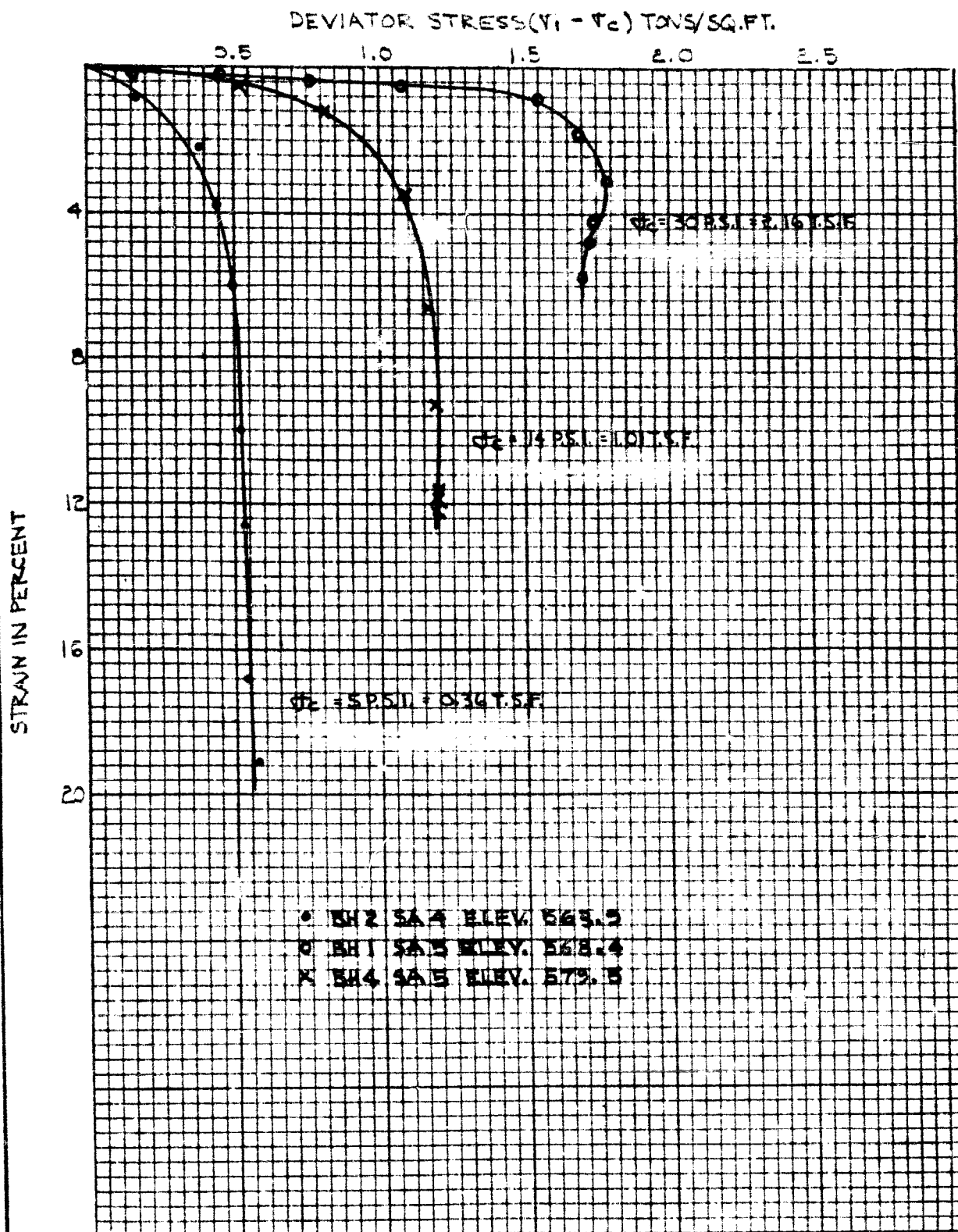
## STRESS STRAIN CURVES

SOFT GREY VARVED CLAY

APPENDIX II

FIGURE 3

PROJECT 56818

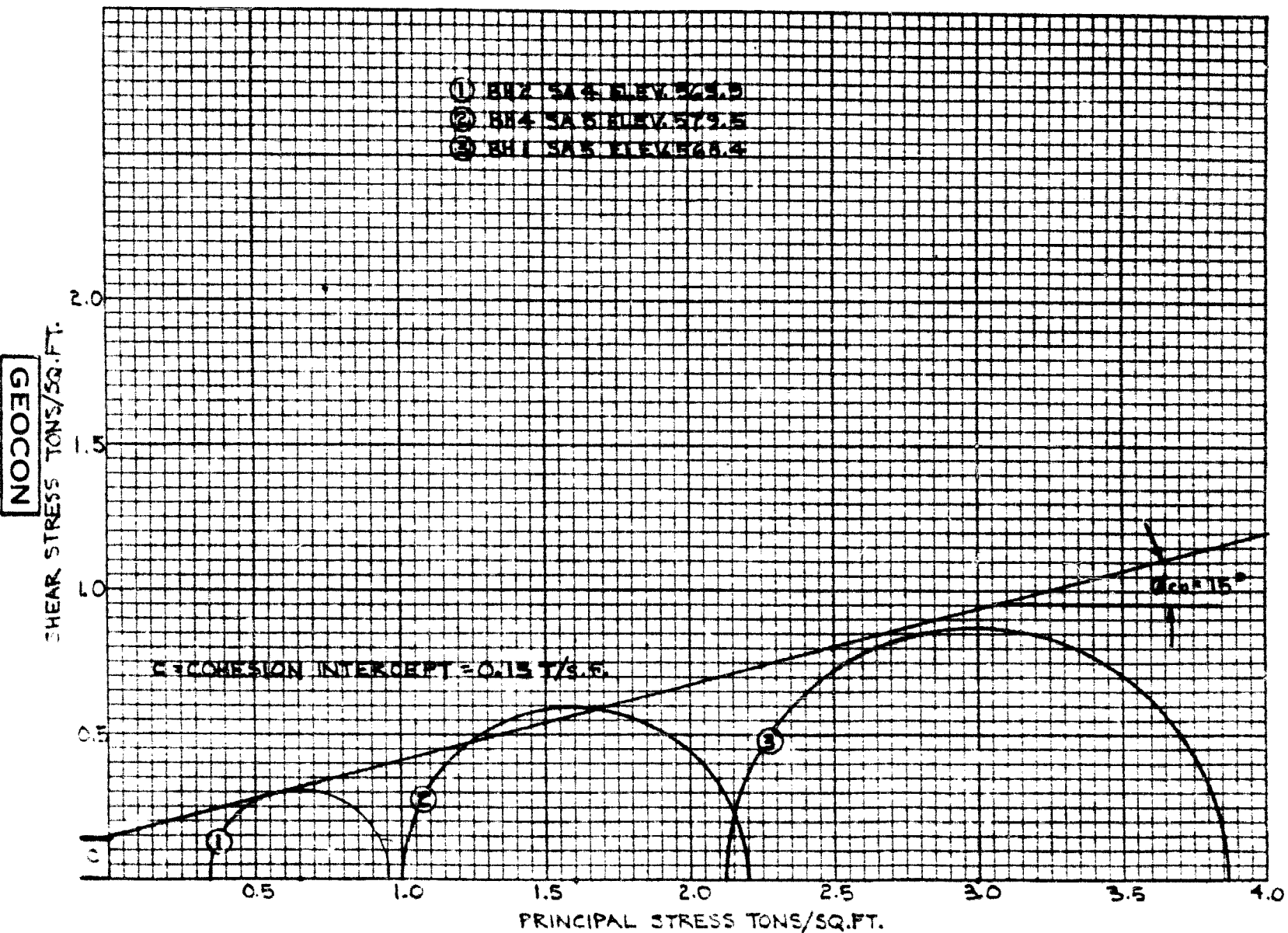




# MOHR CIRCLES CONSOLIDATED QUICK TRIAXIAL TESTS SOFT GREY VARVED CLAY

SOFT GREY VARVED CLAY

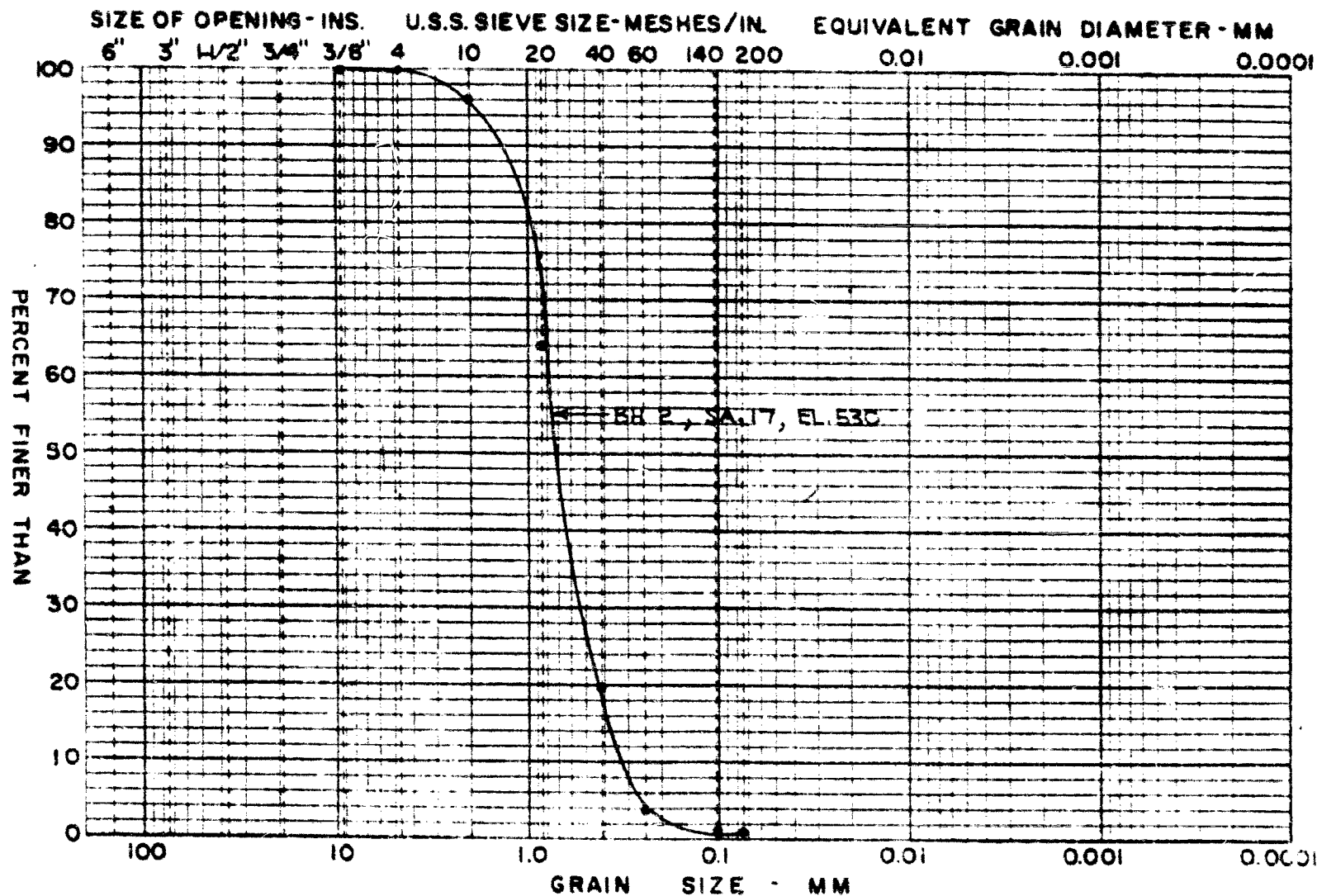
APPENDIX II  
FIGURE 4  
PROJECT 56818



# GRAIN SIZE DISTRIBUTION

APPENDIX II  
FIGURE 5  
PROJECT 56518

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



GEOCON

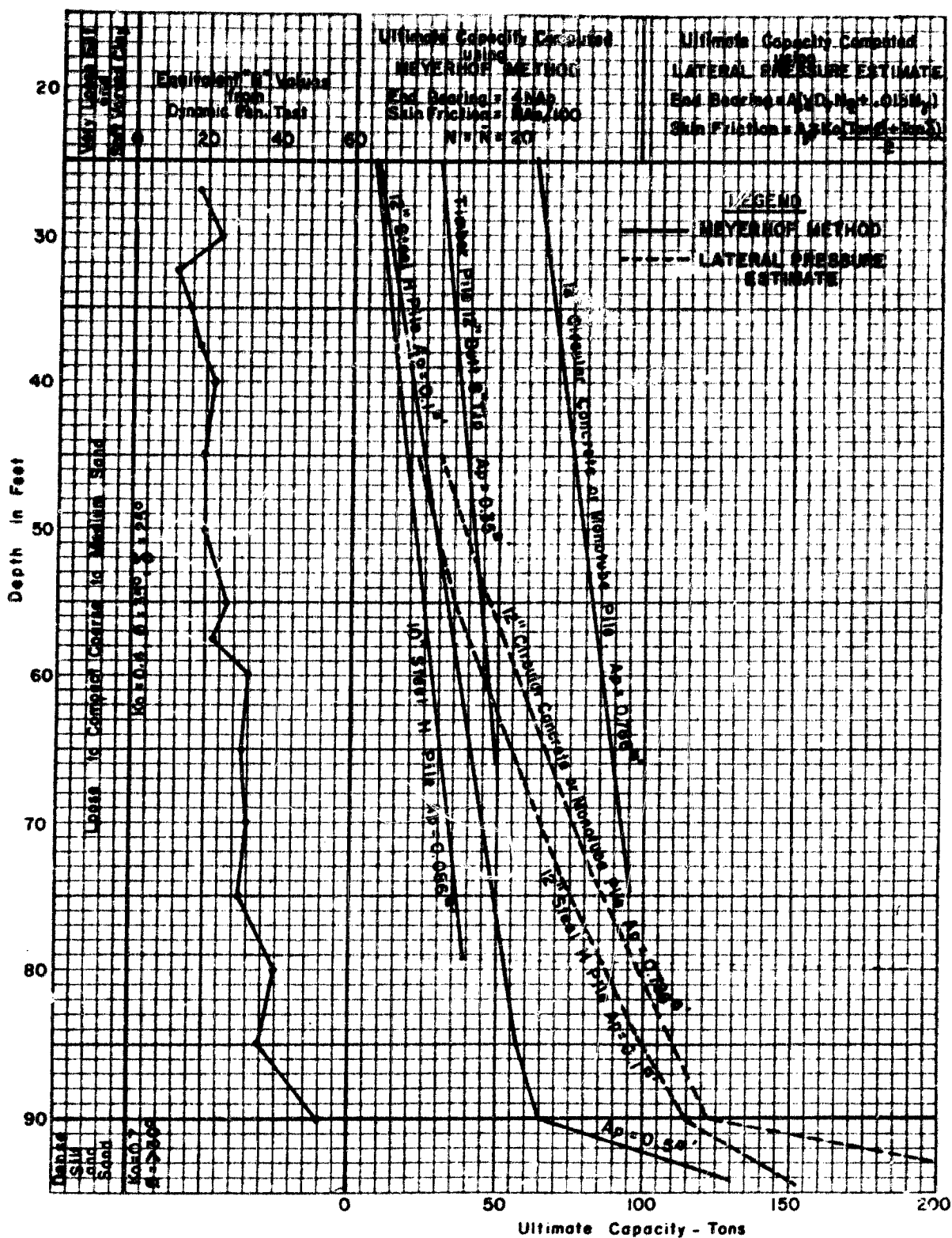
M.I.T. GRAIN SIZE SCALE



# COMPUTED PILE CAPACITY VS DEPTH

EFFECT OF UPPER 25 FEET NEGLECTED

APPENDIX II  
FIGURE 6  
PROJECT S 6818



S6894

INTERIM REPORT

TO

DEPARTMENT OF HIGHWAYS, ONTARIO

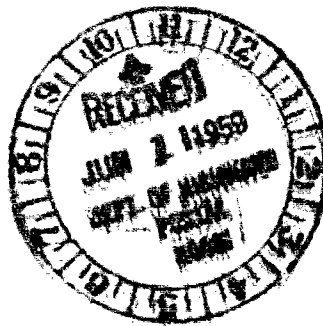
ON

RESISTIVITY SURVEY

PROPOSED SPANISH RIVER CROSSING

MASSEY

ONTARIO



Distribution:

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Toronto, Ontario.
- 2 copies - Geocon Ltd,  
Toronto, Ontario.

May 29th, 1959

**GEOCON**

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1425 WEST PENDER ST.  
VANCOUVER 8, B.C.  
TEL. MU. 1-8926

Rexdale, Ontario,  
May 29th, 1959.

Department of Highways, Ontario,  
Parliament Buildings,  
Toronto 5, Ontario.

Attention: Mr. A. M. Toye, P. Eng.,  
Bridge Engineer.

Re: Resistivity Survey,  
Proposed Spanish River Crossing,  
Massey, Ontario.

Dear Sirs:

This letter reports the results of the above preliminary investigation carried out in accordance with our proposal dated March 5th, 1959 and accepted March 16th, 1959. The purpose of the investigation was to determine and interpret, by the resistivity method, the probable depth to bedrock at various locations along the Spanish River.

### SITE AND GEOLOGY

The Town of Massey is situated about 60 miles west of Sudbury on Highway 17. The Spanish River flows in a westerly direction and is located approximately one-half mile south of the main highway. Four alternative sites were investigated, these being spread over about 5 miles of the river.

The river along this stretch varies from 250 feet to 600 feet in width and varies in depth from 7 feet to 40 feet. Rock outcrops are located at various points along the banks of the river. Irregularly distributed outcrops of bedrock occur in the sand plain some distance away from the river banks.

SITE AND GEOLOGY (continued)

From previous investigations and available information, it is known that silts and sands of considerable thickness fill the valleys and depressions between rugged steep sided outcrops in the area. Bedrock of the area is of the Precambrian age and consists primarily of diorites or diabase of intrusive origin.

PROCEDURE

The field work was carried out between March 2nd and March 13th, 1959. Fifty-two electrical resistivity depth determinations were made using a 115 volt, 60 cycle A.C. generator and a sensitive vacuum tube voltmeter capable of measuring to 0.01 millivolts.

Four sites were investigated using the electrical resistivity method. The locations of these sites, and those of the previous investigations, are shown on the accompanying Drawing S6894-1, together with the inferred bedrock profiles of each of the eight sections.

SUMMARY OF RESULTS

A previous preliminary site investigation conducted by Geocon Ltd in July 1957 indicated that the overburden on the banks of the Spanish River to be in the order of 100 feet. Four possible sites were checked during this investigation, a single borehole was put down at each section. From these boreholes it was found that in general, beneath a shallow layer of topsoil a stratum of sand of variable thickness was found. Beneath this sand stratum lies a layer of grey silty clay 10 to 30 feet thick. Underlying the clay is a

SUMMARY OF RESULTS (continued)

stratum of sand which extends down to the bedrock. This sand layer is of considerable thickness and becomes compact to dense with depth.

The inferred bedrock profiles of these four crossings are shown on the accompanying Drawing S6894-1 and are designated by Sections 3, 4, 5, and 6.

From the results obtained during the above mentioned investigation, it was decided to carry out a more detailed investigation of Section 4. This work was completed in August 1958.

In March 1959, Geocon Ltd carried out a third investigation of the Spanish River. A five mile stretch of the river, from Cole's Rapids, 3 miles west of Massey to a point 2 miles east of the town, was studied. After a preliminary study of the river banks, four sites were picked out for further investigation. As poor ice conditions existing on the river and deep snow covered the land areas, it was decided after discussion with you to conduct the boring programme at a later date when conditions were more favourable and when traffic studies for the proposed structure had been completed. The present investigation was therefore restricted to a resistivity survey.

The four alternative sites were investigated by means of a resistivity survey, the results of which are shown on Sections 1, 2, 7 and 8 on the accompanying Drawing S6894-1.

Station 1

This crossing, of approximately 310 feet, is the most easterly of the four sites investigated and is located approximately 2 miles east of the Town of Massey. The river, at this point, is

Section 1 (continued)

exceptionally deep, having a maximum depth of 43 feet. The bedrock profile as inferred from the resistivity results dips from the north bank where it outcrops to a depth of 65 feet below ground surface on the south bank of the river.

Section 2

This location gives a crossing of 480 feet in width, with the bedrock outcropping on the two small islands located close to the north bank. The resistivity results indicate that the bedrock is to be found at a depth of 65 and 70 feet on the north and south banks of the river.

The river is fairly shallow at this crossing, having a maximum depth of 15 feet.

Section 7

The results obtained from the resistivity survey indicated the bedrock to be at a depth of 70 to 80 feet at this location. No bedrock was found outcropping in the vicinity of this crossing, which has an approximate width of 320 feet.

Soundings made of the river indicated a maximum depth of 22 feet near the north bank.

Section 8

Located some 3 miles west of Massey, at Cole's Rapids, this site has a crossing width of 300 feet. The resistivity survey indicated that bedrock was at a depth of 65 feet on the north bank, but followed the river bed and outcropped on the south bank. The maximum depth of water encountered at this crossing was 19 feet.

Department of Highways, Ontario,  
May 29th, 1959,  
Page 5.

Section 8 (continued)

Due to the great depth of sand that is known to overlie the bedrock in this area, it is possible that the inferred profiles of bedrock may indicate the depth where the sand stratum becomes very dense. If this is the case, the bedrock would be at a slightly greater depth, the high resistivity of the dense sand stratum making it impossible to differentiate it from the bedrock. However, sand of the resistivity measured and consequently of high relative density may be considered to act as an adequate bearing stratum for end bearing piles.

Of the sites investigated, those shown by Sections 1, 2 and 8 are considered to represent the most feasible locations for the proposed bridge from a foundation standpoint. The inferred bedrock profiles have previously been supplied to you. It is suggested that when the traffic requirements for the proposed structure have been studied, that the detailed subsoil conditions at the final site be investigated. If we can be of any further service, we would be pleased if you would call us.

Yours very truly,

GEOCON LTD



V. Milligan, P. Eng.,  
District Engineer.

VM/dw  
S6894

**GEOCON**

S7053  
REPORT  
TO  
DEPARTMENT OF HIGHWAYS, ONTARIO  
ON  
STABILITY ANALYSES - REDUCTION OF GRADE  
NORTH APPROACH EMBANKMENT  
PROPOSED SPANISH RIVER CROSSING  
MASSEY ONTARIO

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TEL. CH. 4-8641

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

Rexdale, Ontario,  
April 11th, 1960.

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

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

Re: W.P. 25-57 - District 18,  
Stability Analyses,  
North Approach Embankment,  
Proposed Spanish River Crossing,  
Massey, Ontario.

Dear Sirs:

Further to your letter of authorization, dated February 16th, 1960, to carry out additional stability analyses for the above, we herewith present the results of our computations.

The soil conditions at the above site are described in our report No. S6818, dated October 20th, 1958. Recommendations given in the report for adequate stability of the north approach embankment were based on a profile grade line at elevation 618. It is understood that the grade line of the approach embankment at the north abutment has now been reduced to elevation 611.

Stability analyses were carried out for the north approach embankment at elevation 611 and the results of the analyses are presented on Drawing S7053-1 together with the engineering design values used. In the analyses, it was assumed that the embankment would be constructed of granular material, well compacted in place and that the topsoil and organic upper portion of the silt stratum would be removed prior to placement of the fill. For the end slope of the proposed

Department of Highways, Ontario,  
April 11th, 1960,  
Page 2.

approach embankment a factor of safety of about 0.8 against a rotational type failure was obtained. With provision of a berm of the dimensions shown on the drawing, computations gave a factor of safety of 1.2 against a circular failure through the end embankment and berm section and also through the berm section alone. Thus to give a minimum factor of safety of 1.2 for the embankment end slope, it is recommended that the berm shown be constructed.

For the side slopes of the approach embankment at the proposed abutment location, a factor of safety of about 0.9 against a rotational type failure was computed. Similarly, as above, to increase the factor of safety to at least a minimum value of 1.2, a berm of the size shown on the drawing should be constructed. The approximate extent of the recommended berm in plan is shown schematically on the drawing.

The recommended berm to the embankment will slope down towards the river channel to about elevation 580. Under river high water level conditions, the berm section would be subject to severe scour. To prevent erosion and scour of the granular berm, it is recommended that at least the outer 20 feet of the berm section be constructed of rock fill. If rock fill is readily available and comparative in cost to granular fill at the site, it is suggested that consideration be given to making the berm section of rock fill throughout.

Department of Highways, Ontario,  
April 11th, 1960,  
Page 3.

We believe that this letter report, together with our previous report, gives all the foundation information necessary to finalize the design of the proposed bridge at a grade elevation of 611. If we can be of any further assistance, however, please give us a call.

Yours very truly,

GEOCON LTD

JLS/dw  
S7053

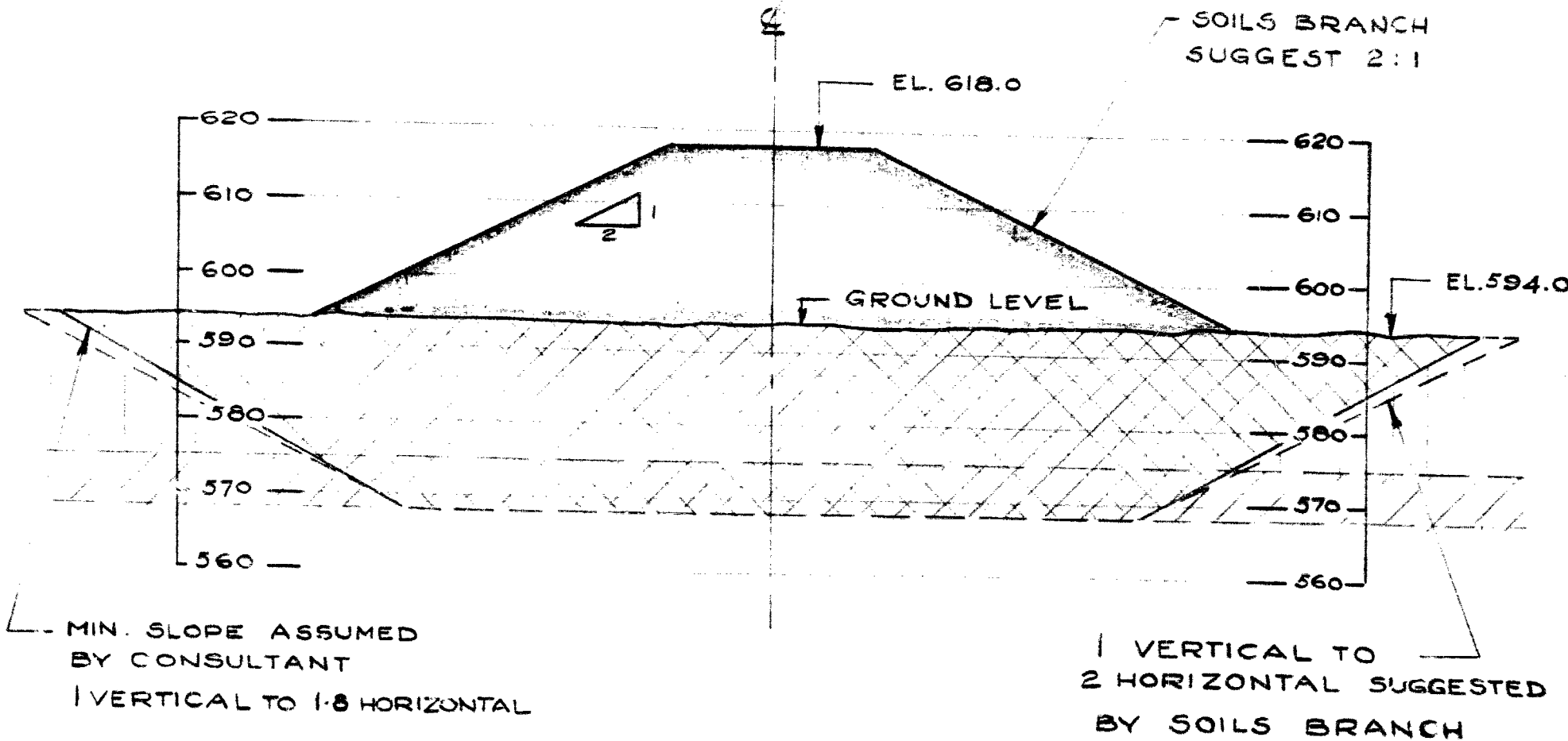
J. L. Seychuk, P. Eng.,  
Senior Soils Engineer.

**GEOCON**

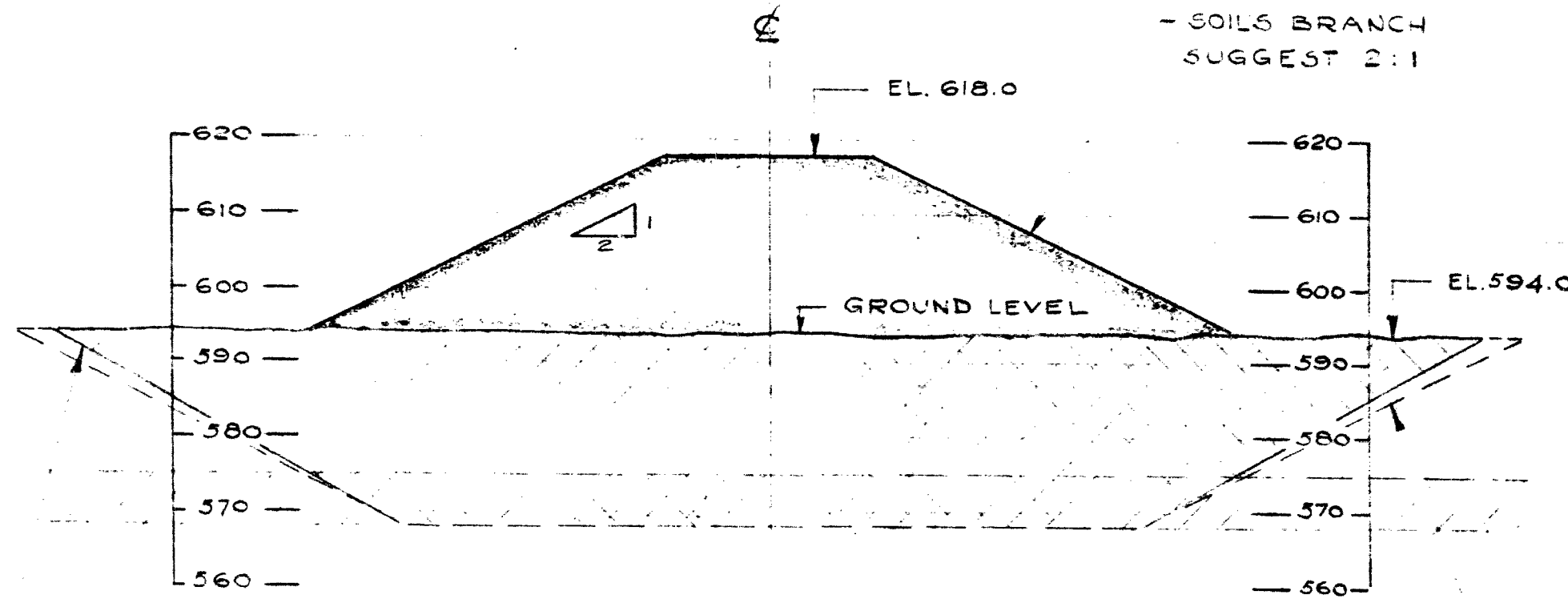
Spanish River Bridge & Massing  
Devel. Road.  
L.P. 25-57. Dist # 18.

The finished grade line has been  
reduced to Elev. 611 at this structure  
location. The original proposed grade line  
was at Elev. 618'. Because of this reduction  
in grade, the original proposal of Subexcavation  
was not considered necessary. Additional  
stability analyses were carried out by Geoscon  
LTD. indicate that the approach abutment  
at the north abutment can be built to  
the elevation (611) with minor beam requirements.  
The recommended beam section to be constructed  
is shown in Drawing No. S7053-1 of Geoscon's  
Report dated the 11th of April, 1960. A  
memo by L. G. Soderman on the 19th of  
April, 1960 was sent out to all the  
parties concerned together with this report  
of Geoscon's. In view of the fact that  
Geoscon's analyses had not been taken into  
consideration, the height of the backfill  
between the abutment and ~~the~~ above the 2  
horizontal to 1 vertical slope shown on  
this drawing, further additional analyses

was made by Crocon, taking this mass of  
rock into account. Crocon submitted their  
additional analyses on the 16th of May, 1960.  
This brief letter was sent out together  
with a memo by L. G. Sodeman on the  
18th of May to all the parties concerned.  
Results of the analyses have shown that  
the recommended berm section to be constructed  
contained in Crocon's Drawing No. 57053-1  
would be satisfactory. Due to the proximity  
of the toe of the berm to the river  
channel, in order to maintain a minimum factor  
of safety of 1.2 for stability, the size of  
the berm should be carefully controlled  
during construction.



SECTION B-B



- SOILS BRANCH  
SUGGEST 2:1

EL. 618.0

GROUND LEVEL

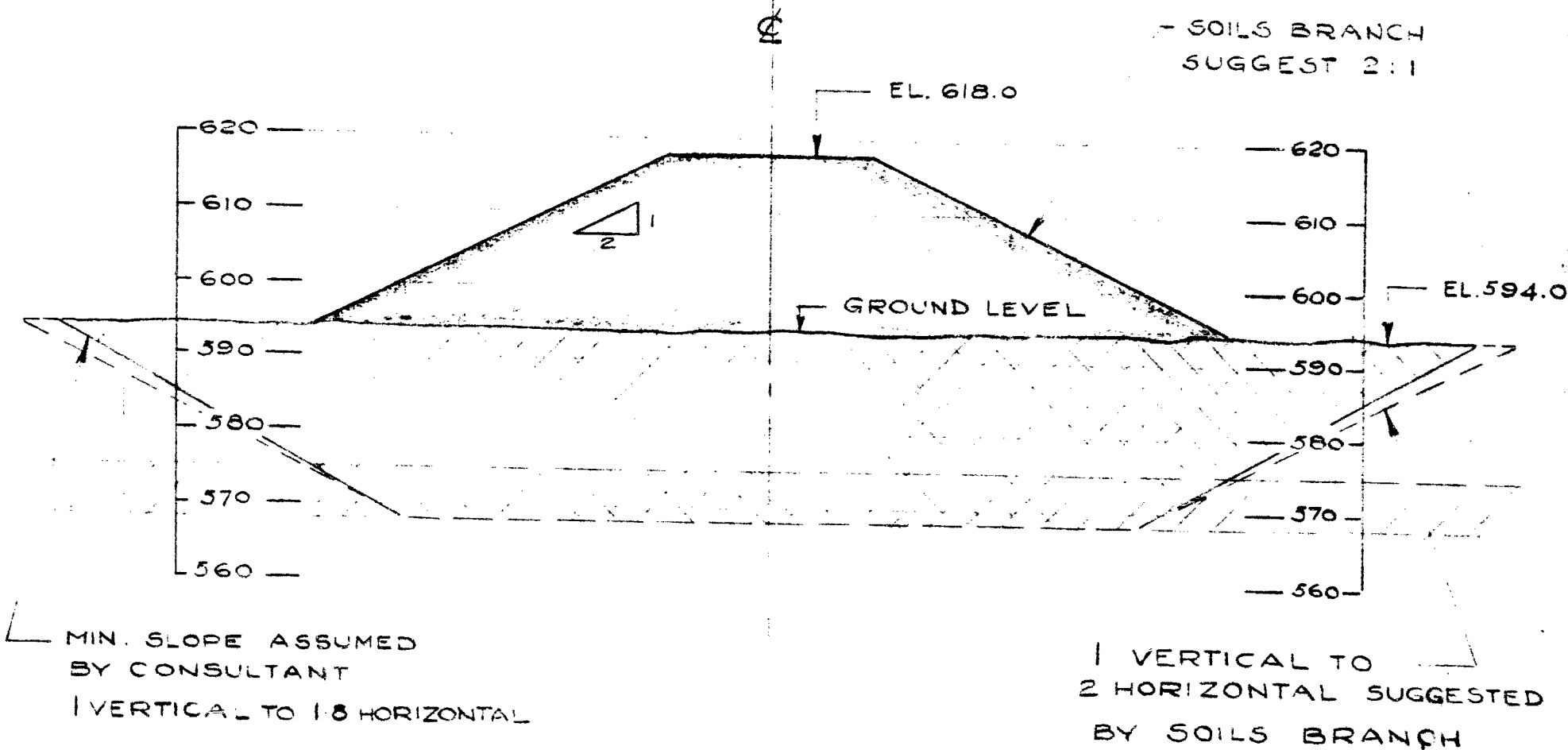
EL. 594.0

MIN. SLOPE ASSUMED  
BY CONSULTANT

1 VERTICAL TO 1.8 HORIZONTAL

1 VERTICAL TO  
2 HORIZONTAL SUGGESTED  
BY SOILS BRANCH

SECTION B-B



SECTION B-B