

Mr. A. H. Toye,  
Bridge Engineer,  
Bridge Division.

Mr. A. G. Sternas,  
Principal Foundation Engr.,  
Foundation Section,  
Materials & Research Division.  
December 21, 1962.

Report - Soil Conditions and Engineering Study,  
Existing and Proposed Highway 17, Station 344+00  
to Station 353+00, Mattawa, Ontario.

Attached, we are sending you the above-mentioned report, submitted by Geocor Ltd., of Toronto. We have reviewed the report and believe that the contained information should prove adequate for your future work.

(Should there be any additional questions or problems that you would like to discuss, please feel free to call on our office.

*A. G. Sternas*

A. G. Sternas,  
PRINCIPAL FOUNDATION ENGINEER.

AGS:pa  
Attach.

cc: Messrs. A.H. Toye (2)  
H.A. Iregashan,  
H. D. McMillan,  
G. E. Hunter (2)  
~~E. Greenland, et al.~~  
T. J. Kovich,  
J. Roy,  
J. E. Graspier,  
L. E. Saint,  
P. Norman,  
A. Watt,  
Foundation Office, ✓  
Gen. Files.

Materials and Research Division

October 2, 1962.

Geoson, Limited,  
Consulting Engineers,  
14 Haas Road,  
Bendale, Ontario.

Attention: Mr. P. McFarlane.

Re: W.F. 69-60, Hwy. #17,  
Subsidence near Mattawa, Ont. 351\*

0.55 13

Dear Sir:-

Please consider this your authority to carry out an additional stability investigation at the above site. Plans and profiles were provided to your representative. This area was originally investigated by your Company, and the report was submitted in February, 1958. The alignment has since been changed, and additional information is required.

It is understood that a qualified Soils Engineer will be in charge of the field work at all times.

Fourteen copies of the completed report, plus one additional copy of the subsoil profile, should be submitted to the Foundation Section as soon as possible. Previous requirements as to preliminary borehole information, and laboratory testing program, should be followed.

Charges for the work performed will be in accordance with your Schedule of Rates, dated October, 1960, and invoices to be addressed to the attention of the undersigned.

Note:- As North Bay is the nearest recognized mobilization point, payment for mobilization will be from there, as discussed with your representative.

MRS/uef

Yours very truly,

cc: Messrs. S. McCombie  
H. McArthur  
J. D. Foster  
S. I. Saint  
E. D. Smith (2)  
Mrs. I. Tate  
Foundations Office  
Gen. Files (2)

*A. Rutka*  
Mr. A. Rutka,  
MATERIALS & RESEARCH ENGINEER

# GEOCON LTD

## HEAD OFFICE

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

Rexdale, Ontario,  
December 18th, 1962

## DISTRICT OFFICES

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

1425 WEST PENDER ST.  
VANCOUVER 5, B.C.  
TEL. MU. 1-6926

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

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

Re: Soil Conditions and Engineering Study,  
Existing and Proposed Highway 17,  
Sta. 344+00 to Sta. 353+00,  
Mattawa, Ontario.  
W.P. 69-60

Dear Sirs:

This letter accompanies our detailed report on the  
above soils engineering study.

We find that the soil conditions for the length of  
highway studied are in general similar to those encountered  
at Station 351+60 during our previous work at this site. We  
find further that the stability of the existing embankment is  
marginal along most of the section investigated and for the  
proposed and higher embankment, therefore, suitable stabilizing  
berms have been incorporated in the design. Removal of  
the peat and organic silt with sand layers is recommended.  
However, since deposits of sand also occur in the river, which  
might possibly be left in place, it is suggested that the  
lateral extent of the peat and organic silt within the proposed  
limits of the berm be checked prior to construction, as  
discussed in the report.

We believe that this report contains all the  
required information to enable the scheme for this proposed  
highway to be developed. However, if you should require  
further information, or if we can be of assistance otherwise,  
please contact us.

Yours very truly,

GEOCON LTD

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

MAJM/dw  
S7431

ST. JOHN'S

HALIFAX

MONTREAL

TORONTO

VANCOUVER

S7431  
REPORT  
TO  
DEPARTMENT OF HIGHWAYS, ONTARIO  
ON  
SOIL CONDITIONS AND ENGINEERING STUDY  
EXISTING AND PROPOSED HIGHWAY 17  
STA. 344+00 TO STA. 353+00,  
MATTAWA ONTARIO  
W.P. 69-60

Distribution:

- 14 copies - Department of Highways, Ontario,  
Downsview, Ontario.
- 3 copies - Geocon Ltd,  
Rexdale, Ontario.

**GEOCON**

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## INTRODUCTION

Geocon Ltd has been retained by the Department of Highways, Ontario, by letter dated October 2nd, 1962 to investigate in detail and report on the soil and stability conditions between Stations 344+00 and 353+00 on Highway 17 near Mattawa, Ontario.

There is an area of observed subsidence and cracking in the vicinity of Station 351+60. This particular area was the subject of a study carried out by us and described in our report S6531, dated February 17th, 1958. In this previous report, recommendations were given covering stabilizing measures for the immediate vicinity of Station 351+60. Also given were recommendations for stabilization of the section of the highway adjacent to the river and immediately west of Station 351+60. However, the latter recommendations were based on an extrapolation over the whole length of highway of the soil conditions encountered at Station 351+60. They were also based on a new roadway width of 38 feet and a centreline coincidental with the old centreline.

In this investigation, the soil conditions have been defined in detail between Stations 344+00 and 353+00. In addition, stability analyses have been carried out for the new roadway width of 48 feet, for a new centreline which is located up to 20 feet closer to the river than the existing highway and for a roadway grade raised slightly above the one previously proposed.

The object of the investigation was to determine and interpret the soil and water conditions as they affect the design of the new roadway embankment.

## SUMMARIZED SOIL CONDITIONS

The roadway embankment is constructed of sand and gravel fill which is generally wedge-shaped in cross section, and ranges in thickness from 2 feet at the shoulder adjacent to the railway embankment to 14 feet at the riverside shoulder. As reported previously, fill composed of rock, sand and gravel was encountered in the vicinity of Station 351+60. The roadway embankment is underlain by recent river deposits of silty clay, sandy silt and organic silt up to 20 feet in combined thickness. Underlying these recent deposits is a stratum of loose to compact varved silt ranging in thickness at the boreholes from 5 to 50 feet. A stratum of very loose to very dense sand, gravel and boulders, which was penetrated in a previous investigation for over 35 foot depth, underlies the varved silt. Bedrock occurs beneath the sand, gravel and boulder stratum.

## DISCUSSION

### General

The history of the known performance of the section of embankment under study is given in our report S6531. Pertinent information from the previous report is included herein.

From available information, it is known that the section of Highway 17 under investigation was constructed more than 25 years ago. It is further known that the roadway in the vicinity of Station 351+60 has been brought up to grade a number of times since construction because of local subsidence. During an examination of this area on May 17th, 1957, it was observed that cracks about 6 inches wide ran parallel to and about 5 feet north of the centreline of the road. These

## DISCUSSION (continued)

3.

### General (continued)

cracks extended for a length of about 20 feet and the area apparently affected by sliding appeared to be about 60 feet long. In October 1962, during a site visit, it was observed that recent patching of the critical section had been carried out. According to available information, no movement of the Canadian Pacific Railway embankment has ever been reported. In October, 1962, no cracking in the railway embankment was visible and the rails showed no disturbance.

### Slope Stability

It is understood that it is proposed to raise the roadway embankment by up to 10 feet along much of the section investigated. It is also proposed to relocate the roadway centreline towards the river in the western section investigated by up to 20 feet, as shown on Drawing S7431-1.

The observed pattern of cracking and subsidence of the roadway over 25 years suggests that the slope at Station 351+60 is in limiting equilibrium, that is, the existing factor of safety is never much above 1.0, and that from time to time a combination of forces causes temporary instability and some slippage results. Since progressive movement has been taking place over a long period of time, it appears that no change has occurred over the years in the potential instability of the slope.

Calculations carried out in the previous investigation indicate that the most important stratum from a stability point of view is the varved silt. Results of consolidated undrained triaxial tests on samples of the varved silt are



Slope Stability (continued)

shown on Figure 6 of Appendix II. They yield an effective cohesion,  $c'$ , of zero and an effective angle of shearing resistance,  $\phi'$ , of 38 degrees. The dilatancy of the samples during shearing and the high angle of shearing resistance,  $\phi'$ , suggests that the failure in the samples took place through the silt. This was confirmed by observation of the samples after test. However, in the field a significant portion of a failure arc could in fact take place through the clay layers in the varved silt and these would have a lower angle of shearing resistance. The clay layers in the samples from this site were too thin to isolate for testing. However, it is considered, based on experience with Northern Ontario glacial lake clays that the clay layers would have an effective angle of shearing resistance,  $\phi'$ , of 22 degrees. This value has accordingly been used in design with the assumption that the effective cohesion,  $c'$ , is zero. These properties may be conservative for the varved silt stratum as a whole.

Piezometers installed below the roadway in the varved silt or the underlying sand, gravel, and boulder stratum showed that the piezometric water level in October 1962 was only slightly in excess of the adjacent river level. In the previous investigation carried out in June and July, 1957 the piezometric water levels were significantly higher than the river level. The soil section at the site is such that considerably higher piezometric levels could and probably do exist at other times of the year. For design, the piezometer levels observed during the summer of 1957 were used for reasons discussed later.

Slope Stability (continued)

As mentioned in our previous report, stability computations carried out on the section of known instability at Station 351+60 give a factor of safety of about 1.0, based on an angle of internal friction of 22 degrees for the varved silt stratum and the piezometric conditions observed in the summer of 1957. Because of this check with the observed performance of the embankment, the piezometric conditions observed in the summer of 1957 have again been used in the present case in conjunction with an effective angle of shearing resistance,  $\phi'$ , of 22 degrees for the varved silt stratum. In reality, higher piezometric levels may possibly exist in the spring which would confirm that the effective angle of shearing resistance of the silt stratum as a whole may be higher than 22 degrees.

In the light of the above, two approaches have been used in the computations. The first, as already mentioned, is a combination of the piezometric levels as observed in the summer of 1957 and an effective angle of shearing resistance,  $\phi'$ , of 22 degrees for the varved silt. The second is that a high angle of shearing resistance taken as 35 degrees, as interpreted from the laboratory test results would apply for the varved silt stratum. In this case, piezometric levels higher than those observed during the summer of 1957, have been used; the actual pattern chosen being consistent with a factor of safety of 1.0 at Station 351+60. It was found that the first approach controlled design and the results of this approach only are given in the report. Because of the relatively high horizontal permeability of the varved silt and the continuity of this stratum, the same piezometric level

## DISCUSSION (continued)

6.

### Slope Stability (continued)

was assumed to apply for the full length of the section of highway investigated. The design values for the other strata are given under the heading "Design Data" on Drawing S7431-2.

The computed factor of safety of the critical cross section at Station 351+60 was about 0.9. The stability of the existing roadway was also checked at Station 352+70, 350+00, 348+00, 346+00, 344+40. The factors of safety obtained, as shown on Drawing S7431-2, range between about 0.9 and 1.25.

### Remedial Measures

The results of the stability calculations carried out on the existing roadway show that stabilizing measures are required along most of the roadway at its existing height. It follows that stabilizing measures would be required in order to accommodate the proposed increase in roadway elevation and widening. As it is understood that there is no objection to some widening of the embankment into the river, the most practical remedial measure appears to be the incorporation of a berm in the final design, to the north of the roadway. The size of berm required to give a factor of safety of at least 1.3 for the new roadway at several critical locations along the length of highway studied was computed and the results are shown on Drawing S7431-2. These berm sizes have governed the layout of the berm as a whole. The same minimum factor of safety has been used in checking the stability of the berms themselves. For this reason, and also because a 20 foot berm width has been adopted as a practical minimum the computed factor of safety is somewhat higher than 1.3 in some sections.

## DISCUSSION (continued)

7.

### Remedial Measures (continued)

Berm sizes at various stations have been obtained by extrapolation and several such cross sections are shown on the Drawing S7431-2. Side slopes of the roadway and berm have been taken as 2 horizontal to 1 vertical throughout. This is based on the assumption that the berms would be constructed of sand and gravel, which has a natural slope angle of about 30 degrees. Rock fill may also be used for the berms though it will tend to assume steeper angles of repose, 1-1/2 horizontal to 1 vertical or steeper. Because of this, special measures would be needed to flatten the slope of the berm to the slope shown on the drawing. It may be more practical to widen the top of the berm and allow the rock to assume its natural angle of repose. In this case, the toe of the berm would be in the same position as shown on Drawing S7431-2.

The berm required for the new roadway may be feathered out in plan at each end of the section of highway investigated. No berm is required east of Station 353+40 where rock outcrops or west of Station 344+00 where the surrounding ground is approximately equal to the roadway elevation.

In the previous investigation, the uppermost stratum in the river opposite Station 351+60 was found to consist predominantly of very soft dark brown peat. The present investigation has shown that this particular stratum is quite variable and that elsewhere it consists predominantly of organic silt with sand layers. In view of the most recent findings, this stratum as a whole has been described in this report as a very soft dark brown and grey organic silt with sand layers.

## DISCUSSION (continued)

8

### Remedial Measures (continued)

For stability of the berms themselves, and for the overall embankment, it is important that the peat and the organic silt stratum be removed prior to the construction of the berm as this stratum is not a reliable foundation stratum for the berm. Based on the assumption that the peat stratum extended for the full length of the roadway section under investigation, it was considered in our previous report that the peat could be displaced by the weight of the berm during construction. To affect this, a berm of rock fill appeared most suitable, constructed by end dumping in strips parallel to the road centreline as described in our previous report. However, based on the results of this investigation, it is now believed that the displacement method would not be wholly effective particularly in the parts of the organic silt stratum which contain an appreciable number of sand layers. The most suitable and positive method of removing the organic silt would appear to be to excavate it in narrow trenches perpendicular to the embankment centreline, and backfilling without delay with berm material. The probable excavation slope in the silt is shown on the cross sections, given on Drawing S7431-2. It is believed that in due course the space between this excavated slope and the face of the berm will be silted up to the former river bed level.

As shown on Drawing S7431-2, the berms will extend as far as 60 feet into the river beyond the present bank. They will thus extend beyond the limits of the available borehole information. In order to draw in the details of the base of the berm, it has been necessary to infer the soil conditions under the river bed in the vicinity of the toe of the

## DISCUSSION (continued)

9.

### Remedial Measures (continued)

berm. This is particularly true for the cross section shown at chainage 352+70. Although the river bottom immediately adjacent to the present bank is underlain by organic silt or peat, it was observed that further offshore, the river bottom consisted of sand. If this sand deposit occurs within the limits of the berm, it would not be necessary to excavate it prior to placing the fill for the berm. The boundaries of this underwater sand deposit has not been mapped and it is suggested that this be done prior to construction so as to keep the amount of excavation and berm material to a minimum.

The berm and side slope of the embankment will require protection against erosion by the river where they are located below the high water level. If rock fill is used for berm construction no special protection would be required for the berm itself although rip-rap backed by a suitable filter should be provided to the sand and gravel embankment side slope. If sand and gravel is used for berm construction then rip-rap protection will be required for the side slope of the berm, the top of the berm, and the embankment side slope where these are located below high water level. It should be noted that in connection with the selection of rip-rap that the most severe constriction of the river occurs at about Station 352+70.

As mentioned in our earlier report, it would be desirable to line the ditch formed between the existing railway embankment and the proposed highway to minimize saturation of the embankment fill due to this effect.

It is recommended that the roadway be designed in accordance with the procedures given in our previous report.

Remedial Measures (continued)

The proposed roadway in the section studied will traverse across a variety of soils with different compressibilities. For example at Station 350+00 it will be underlain mainly by granular material or varved silt. On the other hand at Station 346+00, it will be underlain by about 20 feet of organic silt, then the varved silt. There will thus be different settlements beneath the highway at these two locations. No consolidation tests were carried out on samples of the various strata encountered. However, it is believed that the settlement due to consolidation of the organic silt and the varved silt will be relatively rapid and would amount to an estimated total settlement of the highway of about 6 inches. To accommodate the anticipated settlement, it is suggested that final paving of the highway be deferred until the majority of the settlement has taken place. This could be checked in the field by installing a number of settlement plates at strategic locations under the fill during construction.

CONCLUSIONS AND RECOMMENDATIONS

1. The soil conditions under the proposed highway consists in general of granular fill, then varved silt beneath which is sand, gravel and boulders. Under the river bed and some sections of the highway there is a stratum of predominantly very soft organic silt with sand layers.

2. At the time of this investigation, the groundwater table was at approximately river level. For computation purposes, the higher position recorded during the summer of 1957 was used.

CONCLUSIONS AND RECOMMENDATIONS (continued)

11.

3. Analyses show that the stability of the existing highway is marginal for most of the length investigated. Further analyses show that the road can only be raised and relocated as proposed if stabilizing berms are incorporated in the embankment design.

4. It is recommended that the peat and organic silt be removed prior to berm construction as discussed in the report.

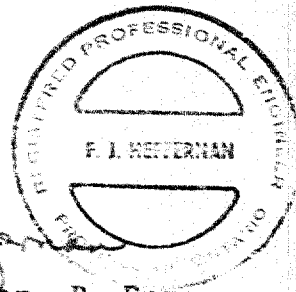
5. Comments with regard to settlements and other factors affecting roadway design are given in the report.

PERSONNEL

The field work for this investigation was carried out under the technical direction of Mr. J. F. Doherty. This report was written by Mr. F. J. Heffernan and reviewed by Mr. M. A. J. Matich.

FJH/dw  
S7431

  
F. J. Heffernan, P. Eng.





## APPENDIX I

Procedure

Site and Geology

Soil Conditions

Water Conditions

Office Reports on Soil Exploration

## PROCEDURE

The field work for this investigation was commenced on September 30th, 1962 and completed on October 23rd, 1962. A total of 12 boreholes, each with an accompanying dynamic penetration test, was put down using a standard machine drill rig. For the two overwater boreholes, the drill rig was mounted on a rented barge. The boreholes were put down in BX size and two inch Shelby tube samples and two inch sleeve samples were taken in the clay and varved silt strata. Two inch drive open samples were taken in the granular strata. In situ vane shear tests were carried out using a strain controlled vane tester. Piezometers were installed in seven of the boreholes, both in the varved silt stratum and the underlying sand, gravel and boulder stratum.

Detailed logs of the boreholes are presented on the Office Reports on Soil Exploration in this Appendix. The locations of the boreholes together with the inferred soil stratigraphy in this investigation and in the previous investigation carried out in 1957, are shown on Drawing S7431-1 located in the pocket at the rear of the report.

Drawing S7431-2 in the pocket at the rear of the report shows the stability analysis of the present conditions together with the stability analysis of the raised roadway and remedial berms. The design data used in the calculations is also given on this drawing.

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

## PROCEDURE (continued)

II.

All elevations given in this report are referred to Geodetic Datum. The Geodetic bench mark is located at chainage 362+52. The elevation of this bench mark is 608.40 as shown on your plan C1546-1.

## SITE AND GEOLOGY

The site comprises a section of Highway 17 between chainages 344+00 and 353+00, about 11 miles west of Mattawa, Ontario. The embankment of the Canadian Pacific Railway is directly south of and parallel to the highway, and the Amable du Fond River is directly to the north.

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

## SOIL CONDITIONS

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

### Loose to Compact Sand and Gravel Fill

Along this section of the highway, the ground level has been built up by a combination of rock, and sand and gravel fill. A brown sand and gravel fill was encountered at both shoulders of the roadway along the length investigated. The thickness of the fill as encountered ranged from 2 to 14 feet with an average thickness of 6 feet.

Loose to Compact Sand and Gravel Fill (continued)

A mechanical analysis was carried out on a drive open sample of the fill and the resulting grain size distribution curve is shown on Figure 1 of Appendix II. This sample was comprised of 75 percent sand sizes, 10 percent gravel sizes and 15 percent silt sizes.

Standard penetration tests carried out in the stratum gave "N" values ranging from 14 to 20 blows per foot. Based on these values and on the results of the dynamic penetration tests, the relative density of the fill is estimated to be loose to compact and generally compact.

Loose to Compact Rock, Sand and Gravel Fill

At the eastern end of the highway section investigated, the sand and gravel contains a large portion of rock in a sand and gravel matrix, and as such has been called a rock, sand and gravel fill. The thickness of this fill at borehole 7 was 13 feet.

Standard penetration tests carried out in the stratum in the present and previous investigations, gave "N" values of 5, 10 and 20 blows per foot. Based on these values, the relative density of the stratum is estimated to be loose to compact.

Topsoil

On the river bank at the location of boreholes 9 and 10, a layer of topsoil of about 1 foot thickness exists.

Very Soft to Stiff Silty Clay

A layer of brown and grey silty clay exists below the sand and gravel fill or the rock, sand and gravel below the river side of the roadway. This silty clay contains a variable amount of organic matter. The thickness of the clay layer ranged from 3 to 15 feet as encountered and is generally less than 5 feet thick.

Standard penetration tests carried out in the clay gave "N" values ranging from 2 to 15 blows per foot with an average value of 9 blows per foot. Two undrained triaxial tests carried out during the previous investigation gave an average compressive strength of 0.7 tons per square foot. Based on the above, the consistency of the clay is estimated to range from very soft to stiff.

Very Loose to Compact Sandy Silt

Layers of grey sandy silt were encountered in boreholes 4 and 6. These layers along with the organic silty clay and organic silt are believed to be geologically recent deposits laid down by the Amable du Fond River. The thickness of the layer at the above borehole locations was 7 and 6.5 feet respectively.

Standard penetration tests carried out in the layer gave "N" values of 3 and 9 blows per foot.

Very Soft Organic Silt

Underlying the topsoil on the river bank and the river bottom adjacent to the roadway, is a stratum of dark grey and brown organic silt. This stratum is

## SOIL CONDITIONS (continued)

V.

### Very Soft Organic Silt (continued)

variable in nature, having been laid down by the river, and contains numerous sand lenses and pockets. The thickness of the stratum where encountered by the boreholes ranged from 10 to 14 feet.

An Atterberg limit test carried out on a sample of the organic silt gave a liquid limit of 54 and a plastic limit of 45. The corresponding natural moisture content was 144 percent.

The results of grain size distribution tests are shown on Figure 2 of Appendix II. These results show a grading of 20 percent sand sizes with the remainder almost wholly comprised of silt sizes.

Standard penetration tests carried out in the stratum resulted in "N" values ranging from zero to 3 blows per foot. In many cases the sampler penetrated the stratum under the weight of the hammer only. Based on these values, the consistency of the material is estimated to be very soft.

### Very Loose to Compact Varved Silt

Underlying the recent river deposits is a stratum of brown to grey varved silt. This is the most significant deposit at the site and all boreholes penetrated this stratum completely. The varve laminae are made up of silt and silty clay. Typically, the silty clay layers are 1/2 inch in thickness and at about 3 inch spacing. Irregular silty fine sand seams up to 1 inch in thickness were also encountered.

Very Loose to Compact Varved Silt (continued)

Atterberg limit tests were carried out on the clay layers from two samples and the resulting liquid limits were 43 and 54 and plastic limits 17 and 25 respectively. The natural moisture content for the second sample was 48 percent. An Atterberg limit test was also carried out on a combined sample of silt and silty clay, and the results were a liquid limit of 32 and a plastic limit of 22. The natural moisture content of the clay and silt layers were 36 and 33 percent respectively. The natural moisture content in excess of the liquid limit suggests that the material is soft and has an unstable structure.

Grain size distribution tests were carried out on both the silt and clay layers of typical samples and the results of the tests on the silt and clay are shown on Figures 3 and 4 of Appendix II. The tests on the silt layers indicate that the material is composed basically of silt sizes with about 10 percent clay sizes. The tests in the clay layers indicate that the material is comprised of about 70 percent silt and fine sand sizes and about 30 percent clay sizes.

The shear strength of the stratum was determined by in situ field vane testing in boreholes 1 to 3. The measured shear strength ranged from 1000 to 2000 pounds per square foot. Because of the layered nature of the deposit, the predominance of the silt laminae, and the probability of drainage of the silt layers during shear, it is doubtful if the results of the field vane testing represent the actual undrained shear strength of the deposit.

Very Loose to Compact Varved Silt (continued)

The results do, however, suggest that the silt is in a loose condition at most of the locations where the vane testing was carried out.

Standard penetration tests were carried out in the varved silt and the resulting "N" values ranged from 1 to 26 blows per foot with an average of 12 blows per foot. Based on these values, the relative density of the varved silt is estimated to range from very loose to compact.

Three undrained triaxial compression tests with pore pressure measurements were carried out on samples of the varved silt and the results are shown on Figure 6 in Appendix II. For design purposes, the effective cohesion,  $c^1$ , has been taken as zero and the effective angle of shearing resistance,  $\phi^1$ , has been taken as 35 degrees. However, as the failure of the test specimen took place primarily through the silt layers which were predominant in these samples, it was hoped that a comparative set could be run on a sample in which the clay layers were predominant. No samples in which the clay layers were predominant could be found. However, testing of stiff silty clay from a nearby site and of similar plasticity characteristics was carried out and resulted in a  $c^1$  of 600 pounds per square foot and a  $\phi^1$  of 24 degrees.

Very Loose to Very Dense Sand, Gravel and Boulders

Underlying the varved silt stratum in all the boreholes except borehole 11 where it underlies the organic silt stratum is a stratum of grey and brown



Very Loose to Very Dense Sand, Gravel and Boulders (continued)

sand, gravel and boulders. The boreholes in this investigation were terminated in this stratum after penetrating it for about 5 to 10 feet. In the previous investigation this stratum was penetrated completely and was found to overlie bedrock directly. The thickness as found in the previous investigation was found to range from about 40 to 70 feet.

Occasional silty sand layers were encountered in the stratum in this and the previous investigation and large boulders up to 6 feet diameter were encountered previously. The results of two grain size distribution tests carried out on samples from the stratum are shown on Figure 5 in Appendix II. One sample was a fine sand while the other was a gravelly sand. Neither of these samples are considered typical as they do not reflect the gravel, cobble and boulder content which is known to be present.

Standard penetration tests carried out in the stratum gave "N" values ranging from 4 to greater than 100 blows per foot with an average of 34 blows per foot. Based on these values, the relative density of the stratum is estimated to range from very loose to very dense and to be generally compact to dense.

Bedrock

Bedrock was core drilled in the previous investigation for up to 15 feet. The bedrock is metamorphic and may be classified as a hard sound coarse grained gneiss.

## WATER CONDITIONS

IX.

Piezometers were installed in seven of the boreholes, both in the varved silt stratum and in the underlying sand, gravel, and boulder strata. Water levels were taken in these piezometers and in the open boreholes during the time of the investigation. The water levels are shown on the Office Reports in this Appendix.

The observed water levels ranged from elevation 548 to elevation 551. There was one high value of elevation 555 in an open borehole. The corresponding river level at the time of the investigation was 547. The water level below the roadway is therefore believed to have been up to 4 feet above river level at the time of the investigation, October, 1962.

It is pointed out, however, that the observed ground-water levels were higher during the investigation of July 1957 when readings between elevations 558 and 570 were obtained.

# EXPLANATION OF THE FORM "OFFICE REPORT ON SOIL EXPLORATION"

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

## ELEVATION AND DEPTH

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

## WATER CONDITIONS

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

## DESCRIPTION

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

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

## STRATIGRAPHIC PLOT

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

## ELEVATION SCALE

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

## GRAPHS

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

## OTHER TESTS

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

## SAMPLES

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

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

**GEOCON**



# GEOCON

## OFFICE REPORT ON SOIL EXPLORATION

CONTRACT S7431 BORING # 2 DATUM GEODETIC CASING 5X  
 BORING DATE OCT 14 1962 REPORT DATE OCT 9 1962 COMPILED BY AEL CHECKED BY FCH  
 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

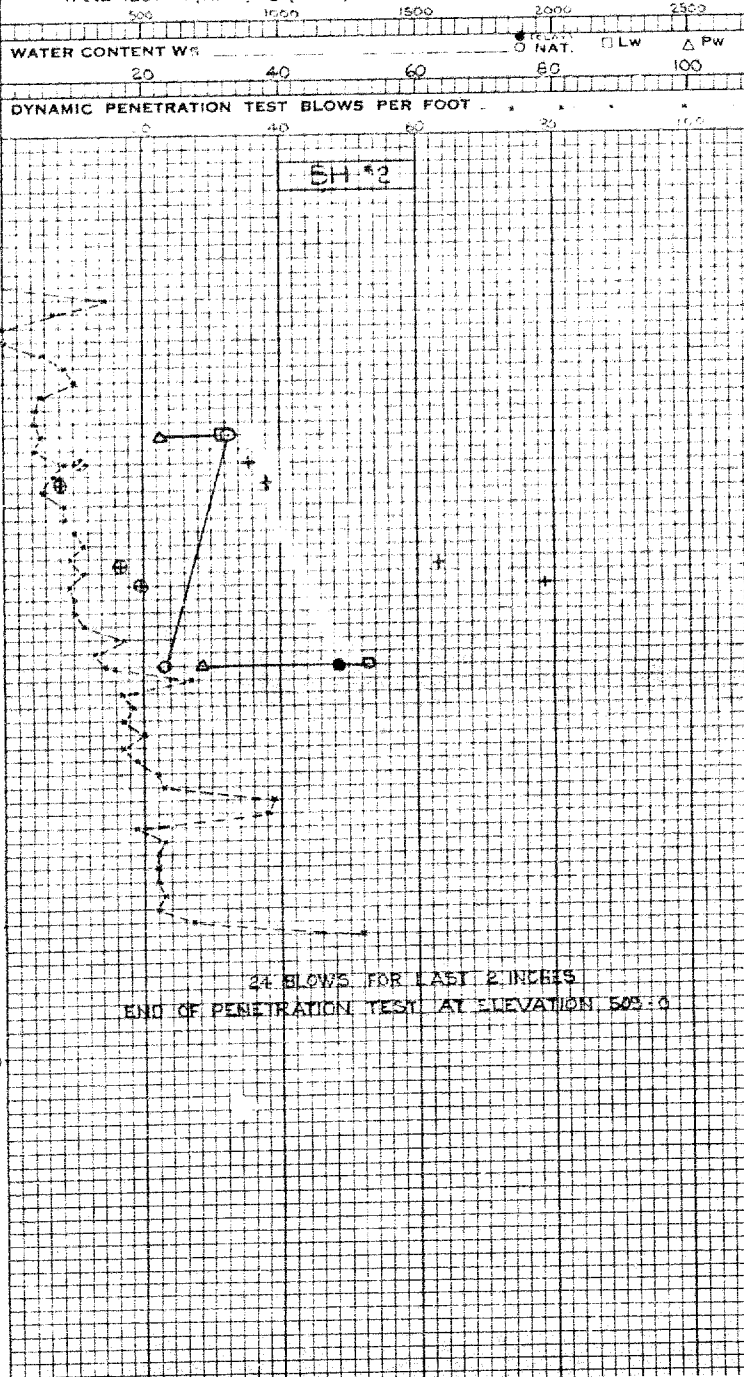
### ABBREVIATIONS

V - IN-SITU VANE TEST  
 M - MECHANICAL ANALYSIS  
 U - UNCONFINED COMPRESSION  
 QC - TRIAXIAL CONSOLIDATED QUICK  
 Q - TRIAXIAL QUICK  
 S - TRIAXIAL 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
556.8 0.0		GROUND LEVEL		560
552.8 4.0		LOOSE BROWN SAND AND GRAVEL FILL		550
		LOOSE TO COMPACT GREY VARVED SILT		540
				530
				520
518.8 38.0		LOOSE TO VERY DENSE GREY AND BROWN SAND, GRAVEL AND BOULDERS		510
505.3 51.5		END OF HOLE.		500

### SHEAR STRENGTH LBS./SQ. FT. VANE TEST + (NAT.) ⊕ (REM.)



### SAMPLES

CONDITION	TYPE	NUMBER	PENETRATION RESISTANCE BLOWS/FT.
DO	DO	1	8
SO	SO	2	3
SO	SO	3	7
SO	SO	4	7
SO	SO	5	12
DO	DO	6	22
DO	DO	7	14
DO	DO	8	4
DO	DO	9	12
DO	DO	10	52

# GEOCON

## OFFICE REPORT ON SOIL EXPLORATION

CONTRACT 01421 BORING # 2 DATUM 35' DEPTH CASING 3X  
 BORING DATE OCT. 5, 1962 REPORT DATE OCT. 12, 1962 COMPILED BY A.B.L. CHECKED BY F.J.H.  
 SAMPLER HAMMER WT 140 LBS. DROP 30 INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN. LBS. ENERGY)

### SAMPLE CONDITION

☐ DISTURBED  
☐ FAIR  
☐ GOOD  
☐ LOST

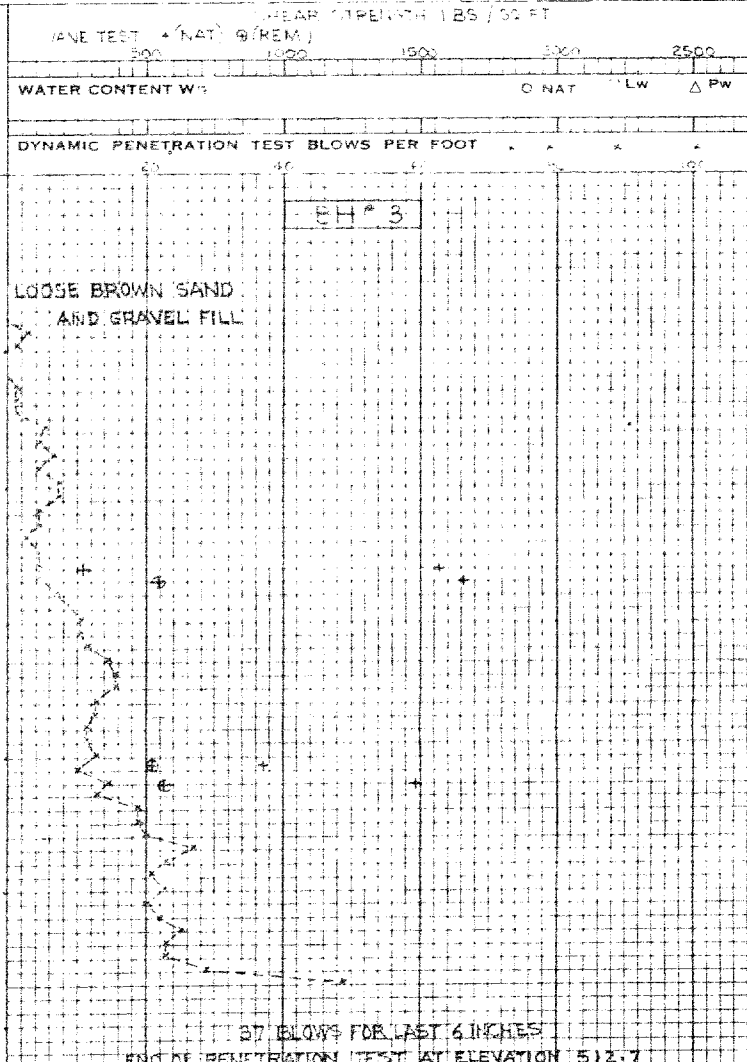
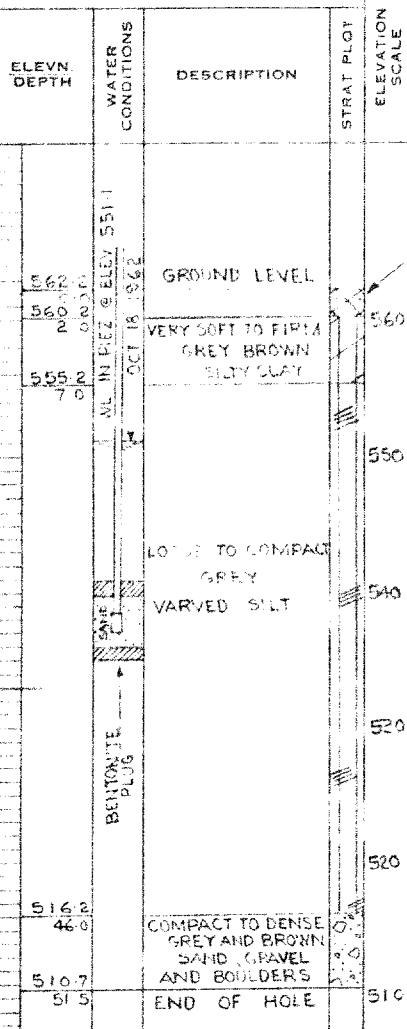
### SAMPLE TYPES

AS - AUGER SAMPLE  
 ST - SLOTTED TUBE  
 WS - WASHED SAMPLE  
 DO - DRIVE OPEN  
 DF - DRIVE FOOT VALVE  
 CS - CHUNK SAMPLE  
 FS - FOIL SAMPLE  
 SO - SLEEVE-OPEN  
 SF - SLEEVE-FOOT VALVE  
 TO - THIN WALLED OPEN  
 RC - ROCK CORE

### ABBREVIATIONS

V - IN-SITU VANE TEST  
 M - MECHANICAL ANALYSIS  
 U - UNCONSOLIDATED 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



### SAMPLES

OTHER TESTS

CONDITION	TYPE	NUMBER	PENETRATION RESISTANCE BLOWS/FT.
		1	3
		2	5
		3	PUSH
		4	7
		5	11
		6	10
		7	18
		8	20
		9	26
		10	43

## GEOCON

## OFFICE REPORT ON SOIL EXPLORATION

CONTRACT 27431 BORING # 4 DATUM GEODETIC CASING EX  
 BORING DATE OCT 9, 1962 REPORT DATE OCT 10, 1962 COMPILED BY A.L. CHECKED BY F.H.  
 SAMPLER HAMMER WT 140 LBS DROP 30 INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN - LBS. ENERGY)

## SAMPLE CONDITION

☐ DISTURBED  
☐ FAIR  
☐ GOOD  
☐ LOST

## SAMPLE TYPES

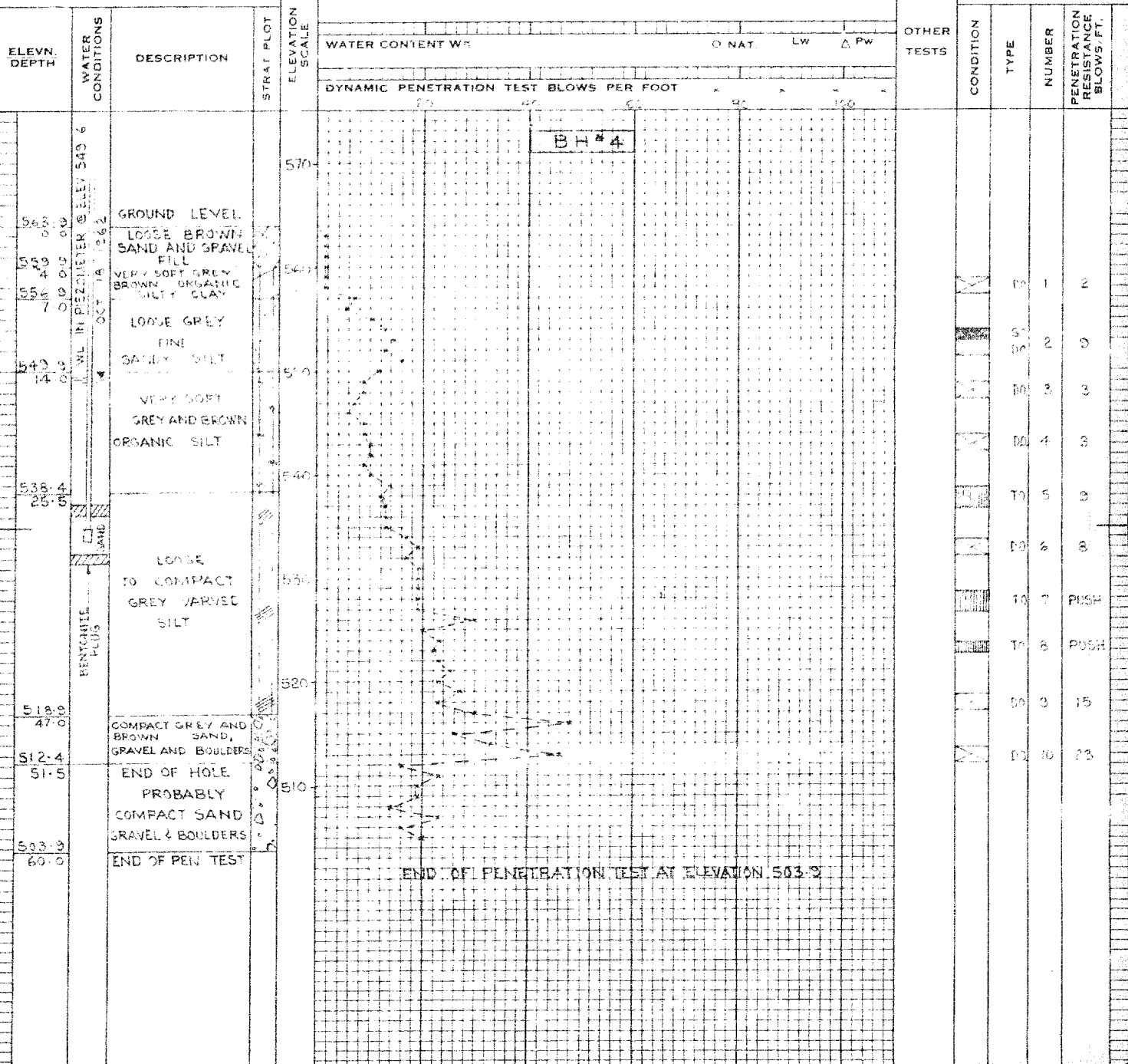
AS - AUGER SAMPLE  
 ST - SLOTTED TUBE  
 WS - WASHED SAMPLE  
 DO - DRIVE-OPEN  
 DF - DRIVE-FOOT VALVE  
 CS - CHUNK SAMPLE  
 FS - FOIL SAMPLE  
 SO - SLEEVE OPEN  
 SF - SLEEVE FOOT VALVE  
 TO - THIN WALLED OPEN  
 RC - ROCK CORE

## 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

## SAMPLES



# GEOCON

## OFFICE REPORT ON SOIL EXPLORATION

CONTRACT 07431 BORING # 5 DATUM GEOBETIC CASING BX  
 BORING DATE OCT. 10, 1962 REPORT DATE OCT. 15, 1962 COMPILED BY A.E.L. CHECKED BY F.S.H.  
 SAMPLER HAMMER WT. 140 LBS. DROP 30 INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN. LBS. ENERGY)

### SAMPLE CONDITION

☐ DISTURBED  
☐ FAIR  
☐ GOOD  
☒ LOST

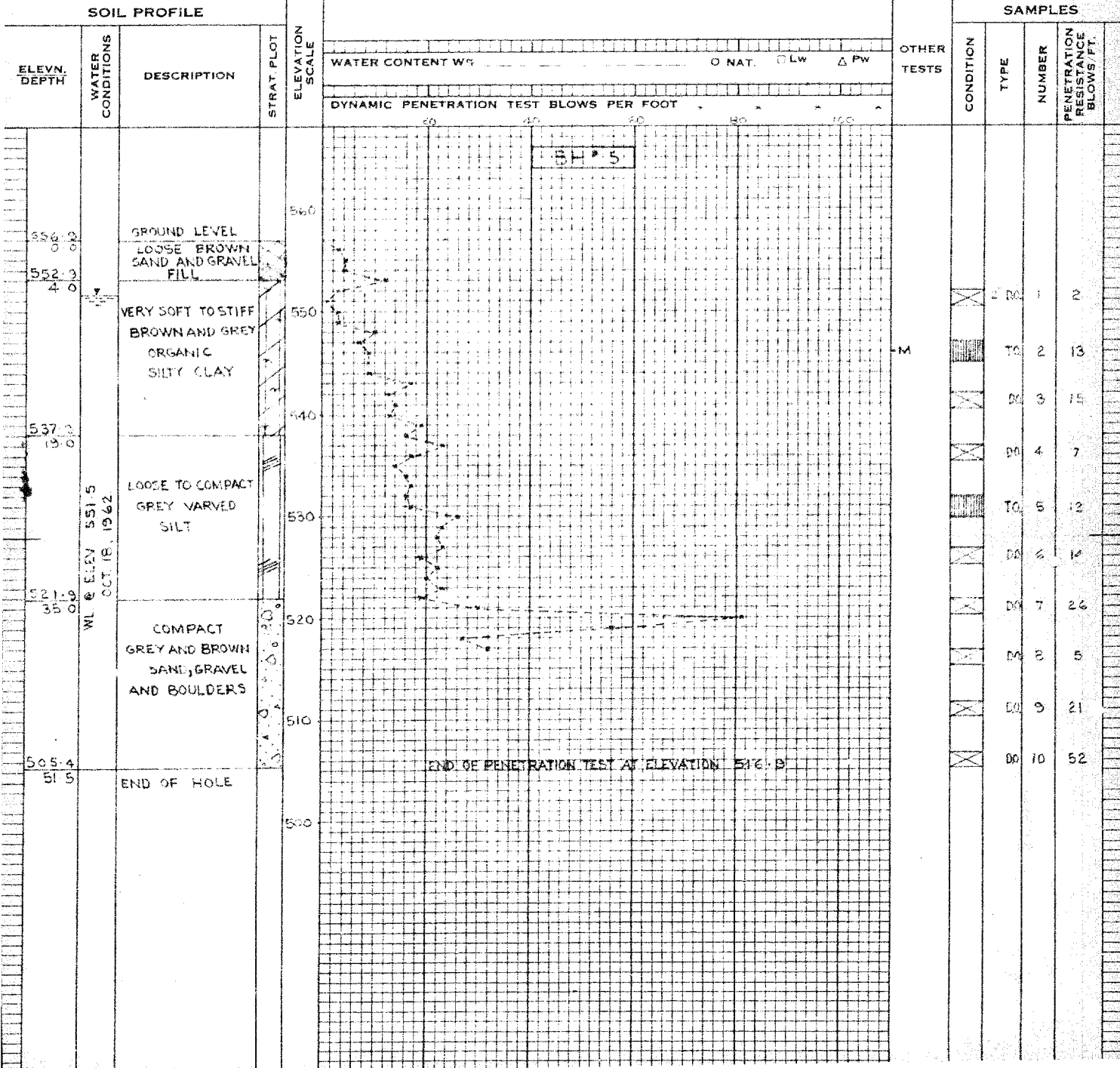
### SAMPLE TYPES

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

### ABBREVIATIONS

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

### SOIL PROFILE





## GEOCON

## OFFICE REPORT ON SOIL EXPLORATION

CONTRACT 131 BORING # 6 CATHUM GLEN Casing BX  
 BORING DATE OCT 11, 1961 REPORT DATE OCT 11, 1961 COMPILED BY ABL CHECKED BY F. J. H.  
 SAMPLER HAMMER WT 140 LBS DROP 30 INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN - LBS. ENERGY)

## SAMPLE CONDITION

## SAMPLE TYPES

## ABBREVIATIONS

☐ DISTURBED  
☐ FAIR  
☐ GOOD  
☐ LOST

AS AUGER SAMPLE  
 ST SLOTTED TUBE  
 WS WASHED SAMPLE  
 DO DRIVE OPEN  
 DF DRIVE FOOT VALVE  
 CS CHUNK SAMPLE

FS FOIL SAMPLE  
 SO SLEEVE-OPEN  
 SF SLEEVE-FOOT VALVE  
 TO THIN WALLED OPEN  
 RC ROCK CORE

V IN-SITU VANE TEST  
 M MECHANICAL ANALYSIS  
 U UNCONFINED COMPRESSION  
 Q 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

## SAMPLES

ELEVATION DEPTH	WATER CONDITIONS	DESCRIPTION	STRAT PLOT ELEVATION SCALE	WATER CONTENT Wt			OTHER TESTS	CONDITION	TYPE	NUMBER	PENETRATION RESISTANCE BLOWS/FT.
				O NAT	ILLW	Δ Pw					
				DYNAMIC PENETRATION TEST BLOWS PER FOOT							
546.1		GROUND LEVEL									
546.1		LOOSE TO COMPACT BROWN SAND AND GRAVEL FILL							DO	1	18
546.1		VERY LOOSE TO COMPACT BROWN FINE SANDY SILT							DO	2	22
541.5		LOOSE TO COMPACT GREY ORGANIC VARVED SILT							DO	3	3
541.5									DO	4	12
540.0									DO	5	PUSH
540.0									DO	6	PUSH
540.0		COMPACT TO VERY DENSE GREY BROWN SAND, GRAVEL AND BOULDERS							DO	7	19
540.0									DO	8	19
540.0									DO	9	20
540.0									DO	10	54
540.0		COMPACT GREY FINE SANDY SILT							DO	11	14
540.0		COMPACT GREY AND BROWN SAND GRAVEL AND BOULDERS							DO	12	10
540.0		END OF HOLE									

END OF PENETRATION TEST AT ELEVATION: 513.1

# GEOCON

## OFFICE REPORT ON SOIL EXPLORATION

CONTRACT S7431 BORING # 7 DATUM GEODETIC CASING EX  
 BORING DATE OCT 10, 1962 REPORT DATE OCT 20, 1962 COMPILED BY AEL CHECKED BY FTH  
 SAMPLER HAMMER WT. 140 LBS. DROP 30 INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN - LBS. ENERGY)

### SAMPLE CONDITION



AS - AUGER SAMPLE  
 ST - SLOTTED TUBE  
 WS - WASHED SAMPLE  
 DO - DRIVE-OPEN  
 DF - DRIVE-FOOT VALVE  
 CS - CHUNK SAMPLE

### SAMPLE TYPES

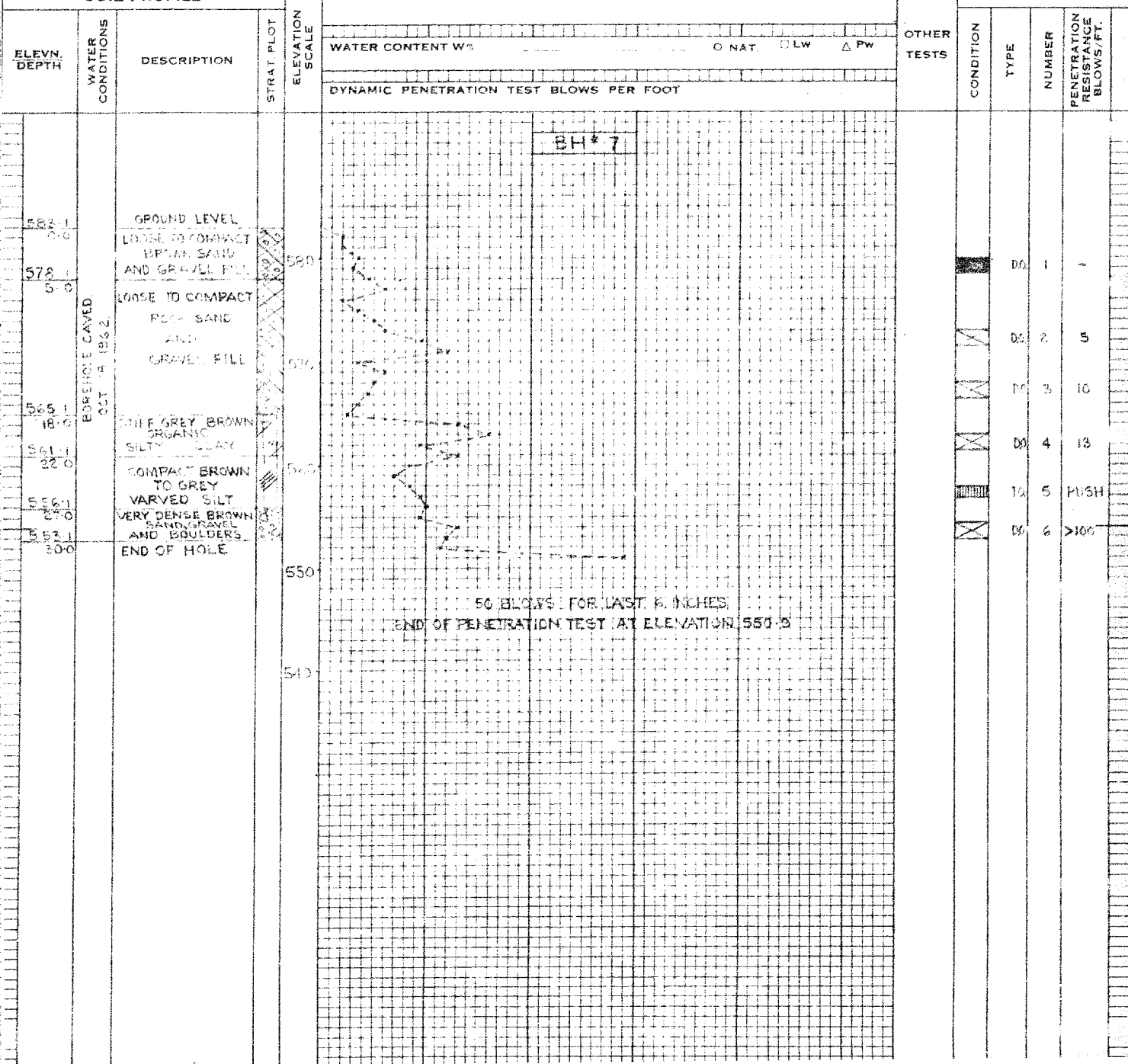
FS - FOIL SAMPLE  
 SO - SLEEVE-OPEN  
 SF - SLEEVE-FOOT VALVE  
 TO - THIN WALLED OPEN  
 RC - ROCK CORE

### 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

### SAMPLES



# GEOCON

## OFFICE REPORT ON SOIL EXPLORATION

CONTRACT 57431 BORING # 8 DATUM GEODETIC CASING 8X  
 BORING DATE OCT. 12, 1962 REPORT DATE OCT. 20, 1962 COMPILED BY AEL CHECKED BY F.C. HILL  
 SAMPLER HAMMER WT. 142 LBS DROP 30 INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN. LBS. ENERGY)

### SAMPLE CONDITION

☐ DISTURBED  
☐ FAIR  
☐ GOOD  
☐ LOST

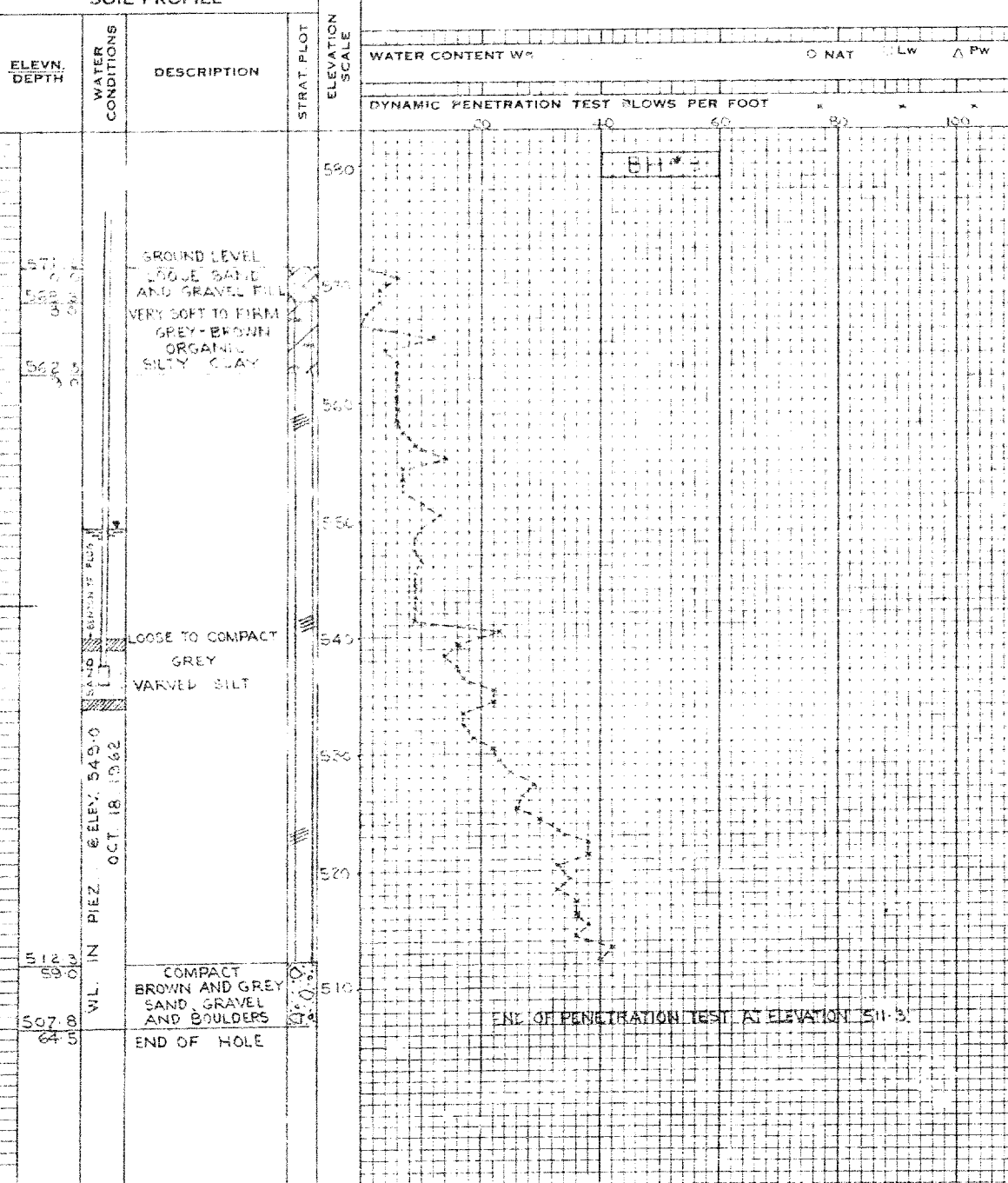
### SAMPLE TYPES

AS - AUGER SAMPLE  
 ST - SLOTTED TUBE  
 WS - WASHED SAMPLE  
 DO - DRIVE-OPEN  
 DF - DRIVE-FOOT VALVE  
 CS - CHUCK SAMPLE  
 FS - FOIL SAMPLE  
 SO - SLEEVE-OPEN  
 SF - SLEEVE-FOOT VALVE  
 TO - THIN WALLED OPEN  
 RC - 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



### SAMPLES

CONDITION	TYPE	NUMBER	PENETRATION RESISTANCE BLOWS FT.
DO	1	15	
DO	2	4	
T	3	PUGH	
TO	4		
TO	5		
DO	6	6	
DO	7	15	
DO	8	10	
T	9	21	
DO	10	10	
DO	11	16	
DO	12	26	
DO	13	19	

# GEOCON

## OFFICE REPORT ON SOIL EXPLORATION

CONTRACT S 7431 BORING # 3 DATUM GEOGETIC CASING BX.  
 BORING DATE OCT. 15, 1962 REPORT DATE OCT. 26, 1962 COMPILED BY A.E.L. CHECKED BY P.S.B.  
 SAMPLER HAMMER WT. 140 LBS DROP 30 INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN - LBS. ENERGY)

### SAMPLE CONDITION

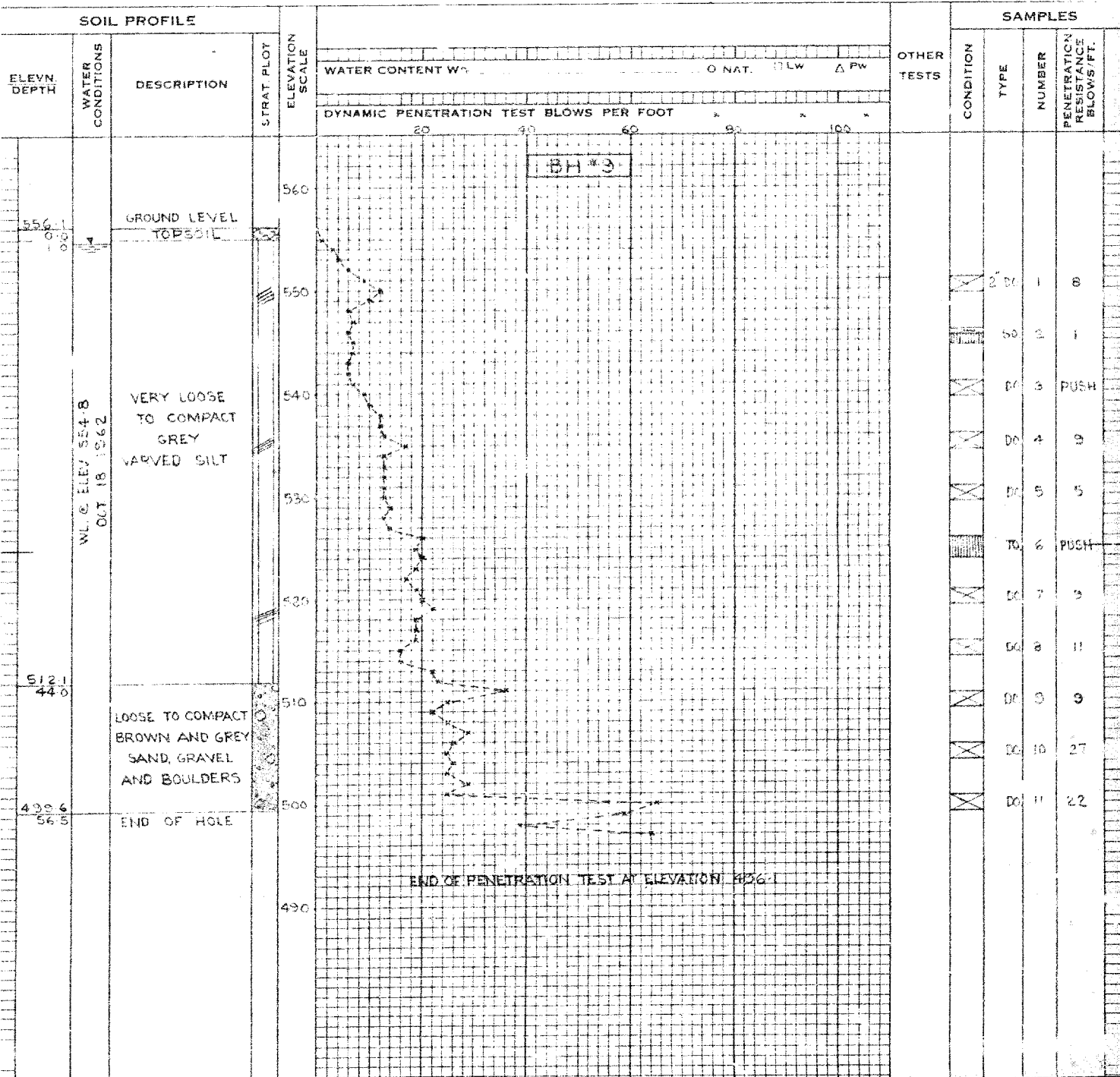


### SAMPLE TYPES

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

### ABBREVIATIONS

V - INSITU 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 37431 BORING # 10 DATUM GEODETIC CASING BX.  
 BORING DATE OCT 16, 1962 REPORT DATE OCT. 26, 1962 COMPILED BY AEL. CHECKED BY FCB.  
 SAMPLER HAMMER WT 140 LBS DROP 30 INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN - LBS. ENERGY)

### SAMPLE CONDITION

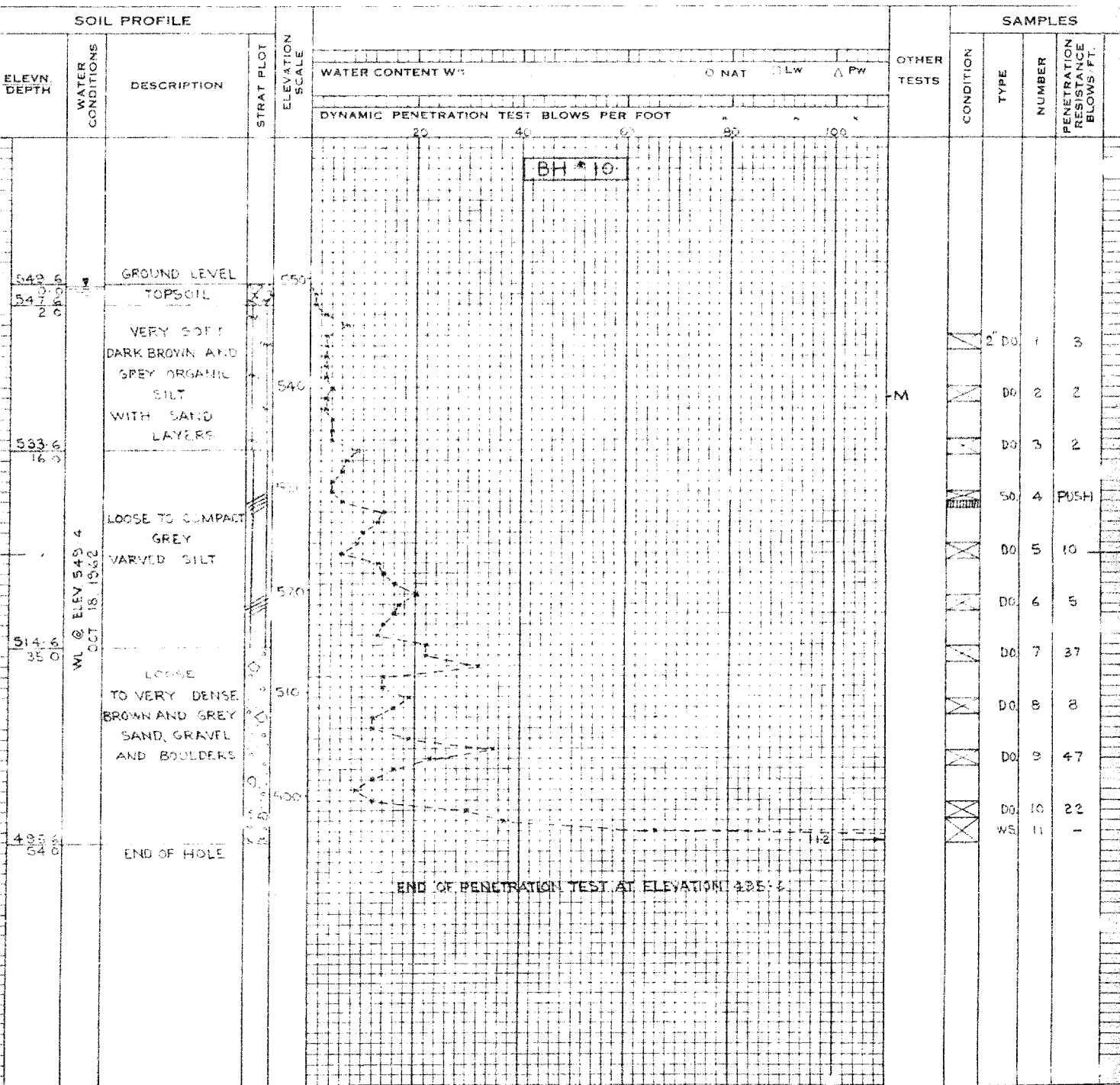


### SAMPLE TYPES

AS AUGER SAMPLE FS FOIL SAMPLE  
 ST SLOTTED TUBE SO SLEEVE-OPEN  
 WS WASHED SAMPLE SF SLEEVE-FOOT VALVE  
 DO DRIVE-OPEN TO THIN WALLED OPEN  
 DF DRIVE FOOT VALVE RC ROCK CORE  
 CS CHUNK SAMPLE

### 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



## OFFICE REPORT ON SOIL EXPLORATION

SAMPLE CONDITION

## SAMPLE TYPES

## ABBREVIATIONS

	DISTURBED
	FAIR
	GOOD
	LOST

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

FS - FOIL SAMPLE  
SO - SLEEVE OPEN  
SF - SLEEVE FOOT VALVE  
TO - THIN WALLED OPEN  
RC - ROCK CORE

V . IN-SITU VANE TEST  
 M . MECHANICAL ANALYSIS  
 U . UNCONFINED COMPRESSION  
 QU . TRIAXIAL CONSOLIDATED QUICK  
 Q . TRIAXIAL QUICK  
 S . TRIAXIAL SLOW

7. WET UNIT WEIGHT  
K. PERMEABILITY  
C. CONSOLIDATION

WL - WATER LEVEL IN CASING  
WT - WATER TABLE IN SOIL

## SOIL PROFILE

ELEV. 1000

### WATER CONDITIONS

### DESCRIPTION

### STRAT PLOT

ELEVATION  
SCALE

## WATER CONTENT WE

$$C_N = \frac{1}{N} \sum_{i=1}^N C_i$$

LW

PW

OTHER

## SAMPLES

CONDITIONS

TYPE

NUMBER

## INTEGRATION

DATE: 5-15-1

1. *Journal of the American Medical Association*, 1997; 277: 1033-1037.

VERY SOFT  
DARK BROWN  
ORGANIC SALT

VERY LOOSE  
TO VERY DENSE  
GREY AND BROWN  
SAND GRAVEL  
AND BOULDER

145 OF 150 F

END OF PENETRATION TEST AT ELEVATION 512.7

24 9 11

# GEOCON

## OFFICE REPORT ON SOIL EXPLORATION

CONTRACT 157431 BORING # 12 DATUM GEOLOGICAL CASING 8 X  
 BORING DATE OCT 20, 1962 REPORT DATE OCT 26, 1962 COMPILED BY AE-1 CHECKED BY  
 SAMPLER HAMMER WT. 140 LBS DROP 20 INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN. LBS ENERGY)

### SAMPLE CONDITION

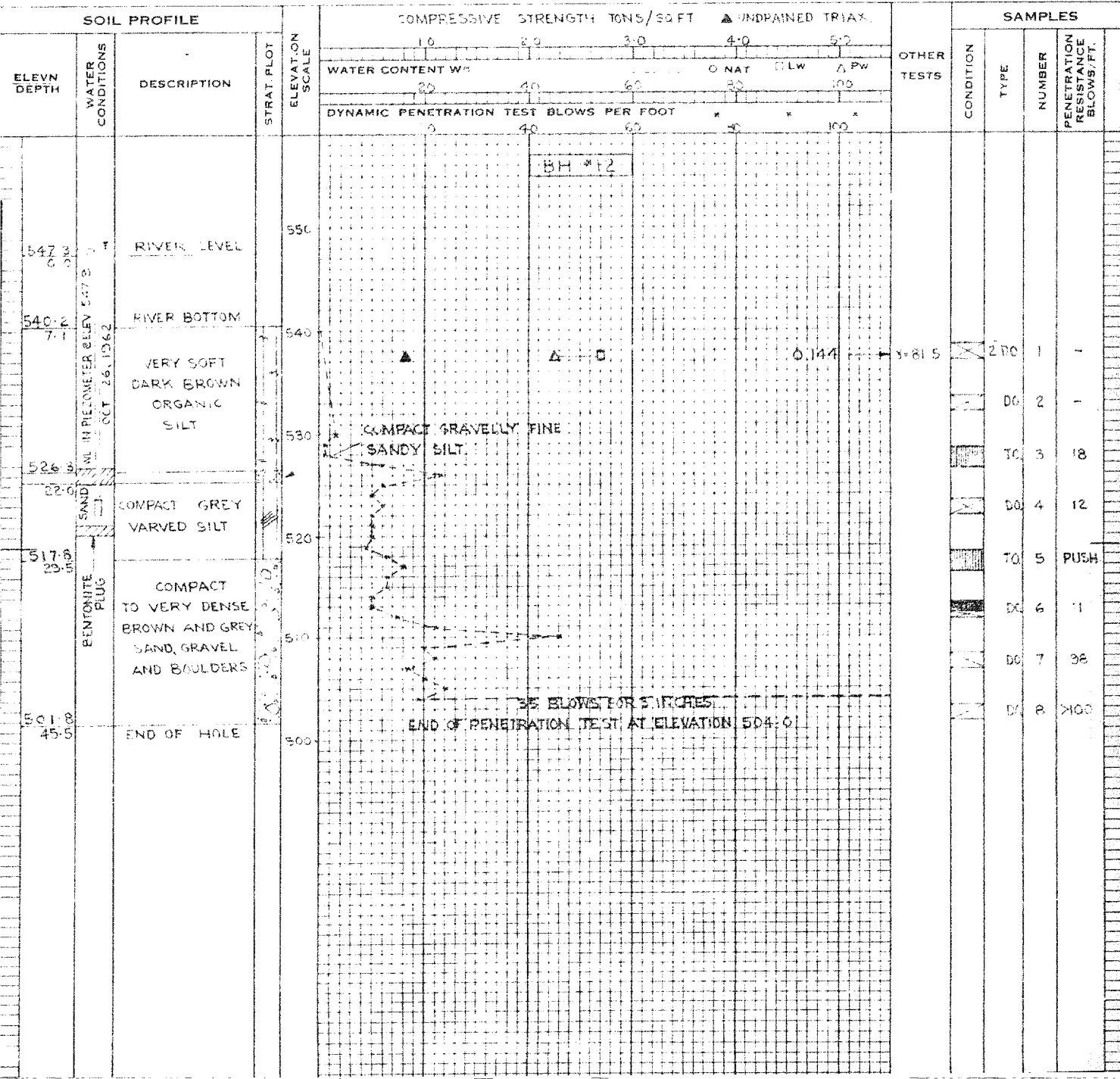


### SAMPLE TYPES

AS AUGER SAMPLE FS FOIL SAMPLE  
 ST SLOTTED TUBE SO SLEEVE OPEN  
 WS WASHED SAMPLE SF SLEEVE FOOT VALVE  
 DO DRIVE OPEN TO THIN WALLED OPEN  
 DF DRIVE-FOOT VALVE RC ROCK CORE  
 CS CHUNK SAMPLE

### ABBREVIATIONS

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



## APPENDIX II

Figures - Laboratory Testing

**GEOCON**



# GRAIN SIZE DISTRIBUTION

APPENDIX II

FIGURE 1

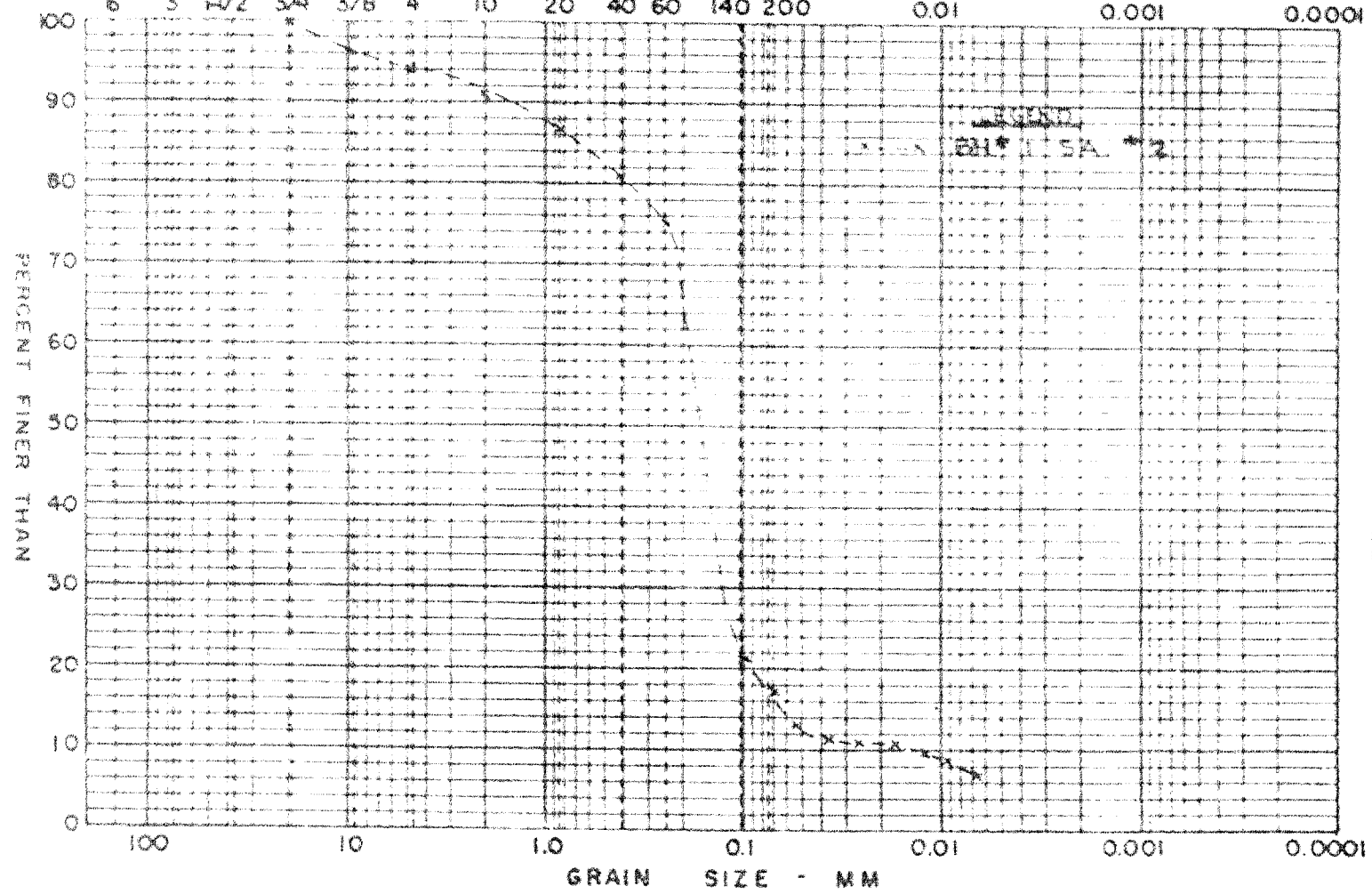
PROJECT 57431

ART. 101 GRAVEL FILL

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

SIZE OF OPENING - INS.    U.S.S. SIEVE SIZE - MESHES/IN    EQUIVALENT GRAIN DIAMETER - MM

6"   3"   1 1/2"   3/4"   3/8"   4   10   20   40   60   140   200   0.01   0.001   0.0001



GEOCON

# GRAIN SIZE DISTRIBUTION

APPENDIX 11

FIGURE 2

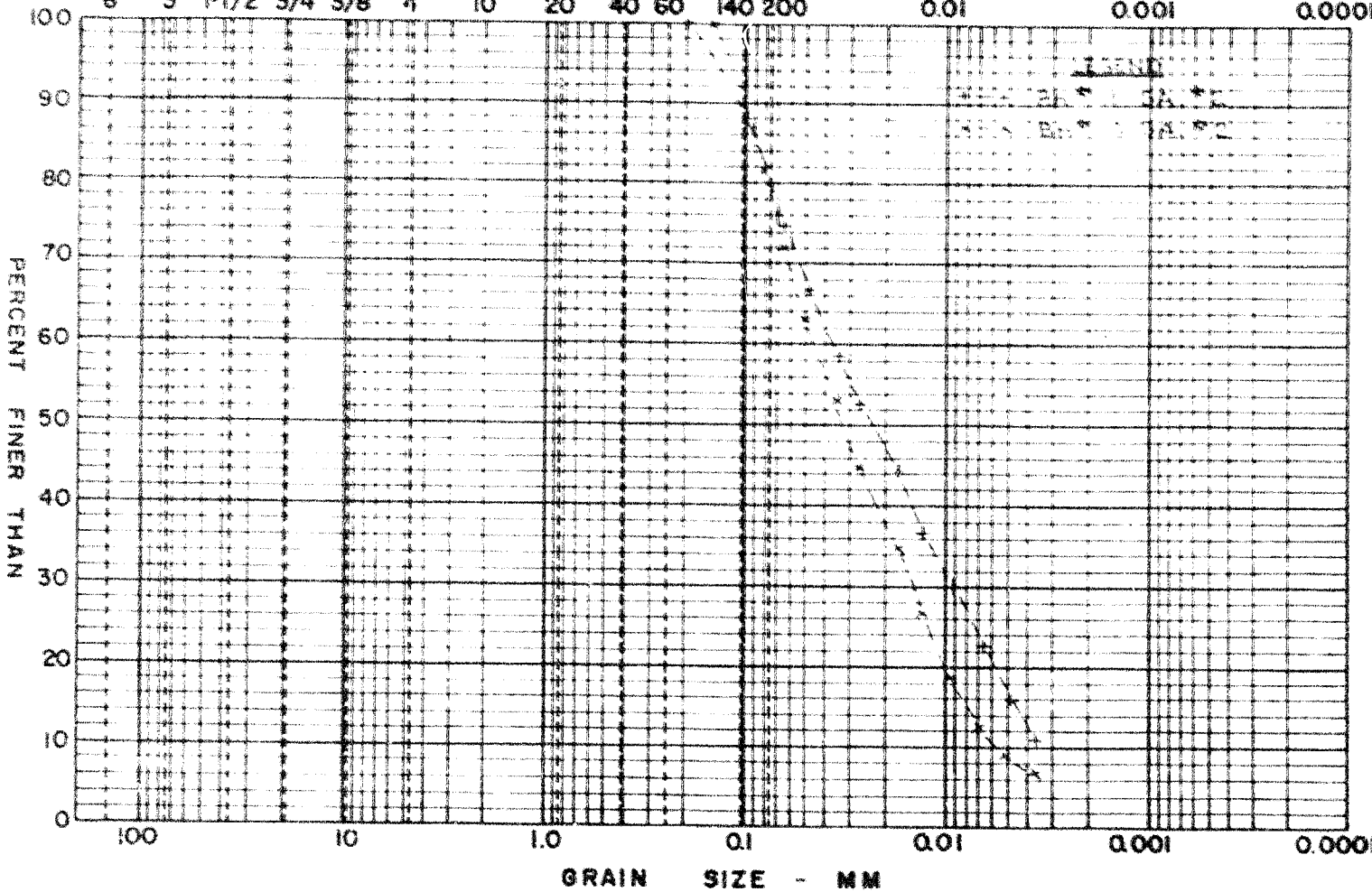
PROJECT 67461

ORGANIC SILT

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

SIZE OF OPENING - INS.      U.S.S. SIEVE SIZE - MESHES / IN.      EQUIVALENT GRAIN DIAMETER - MM

6"   3"   1-1/2"   3/4"   3/8"   1   10   20   40   60   140   200      0.01      0.001      0.0001

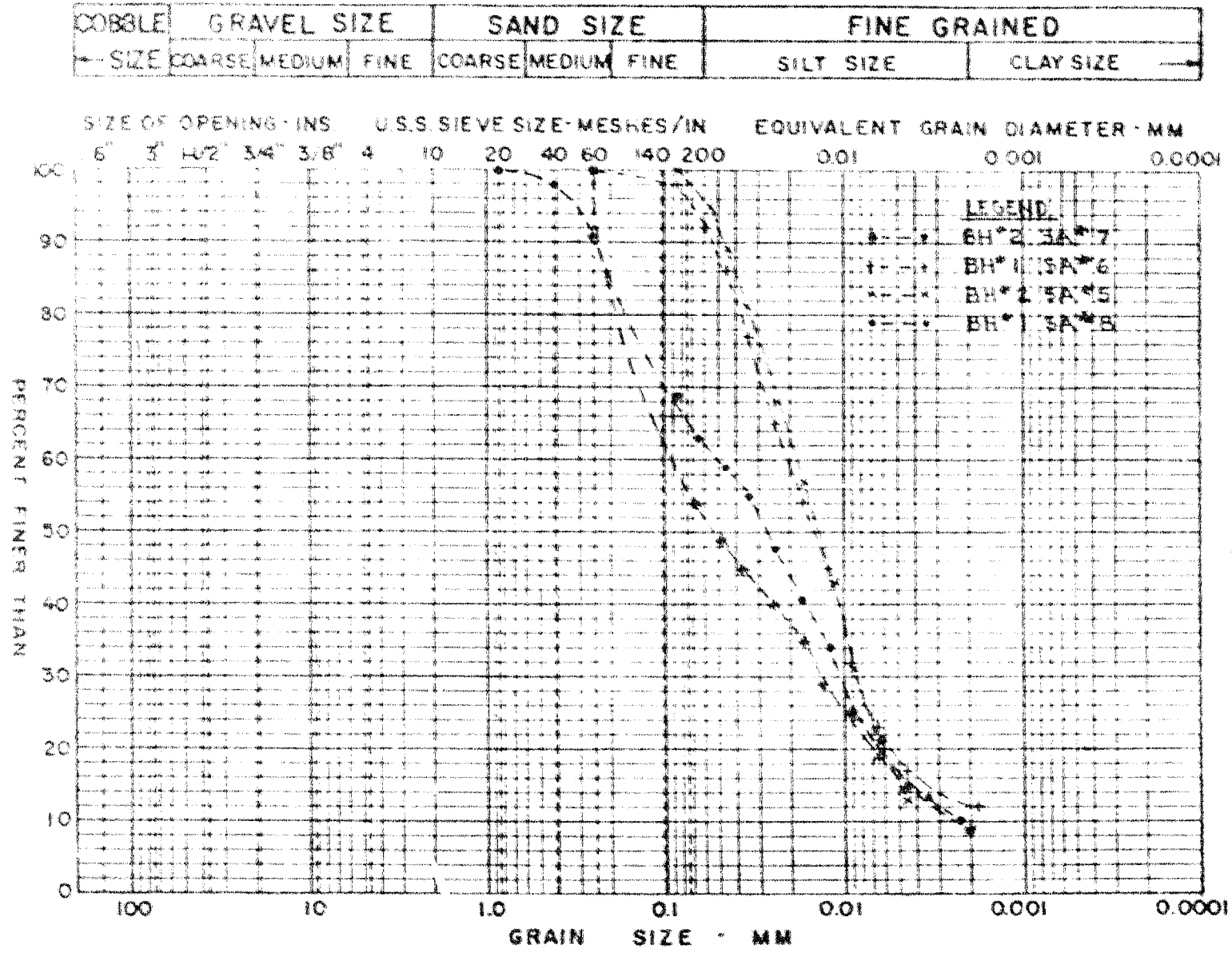


GEOCON

# GRAIN SIZE DISTRIBUTION

APPENDIX 11  
FIGURE 3  
PROJECT 57431

GRAIN SIZE DISTRIBUTION



# GRAIN SIZE DISTRIBUTION

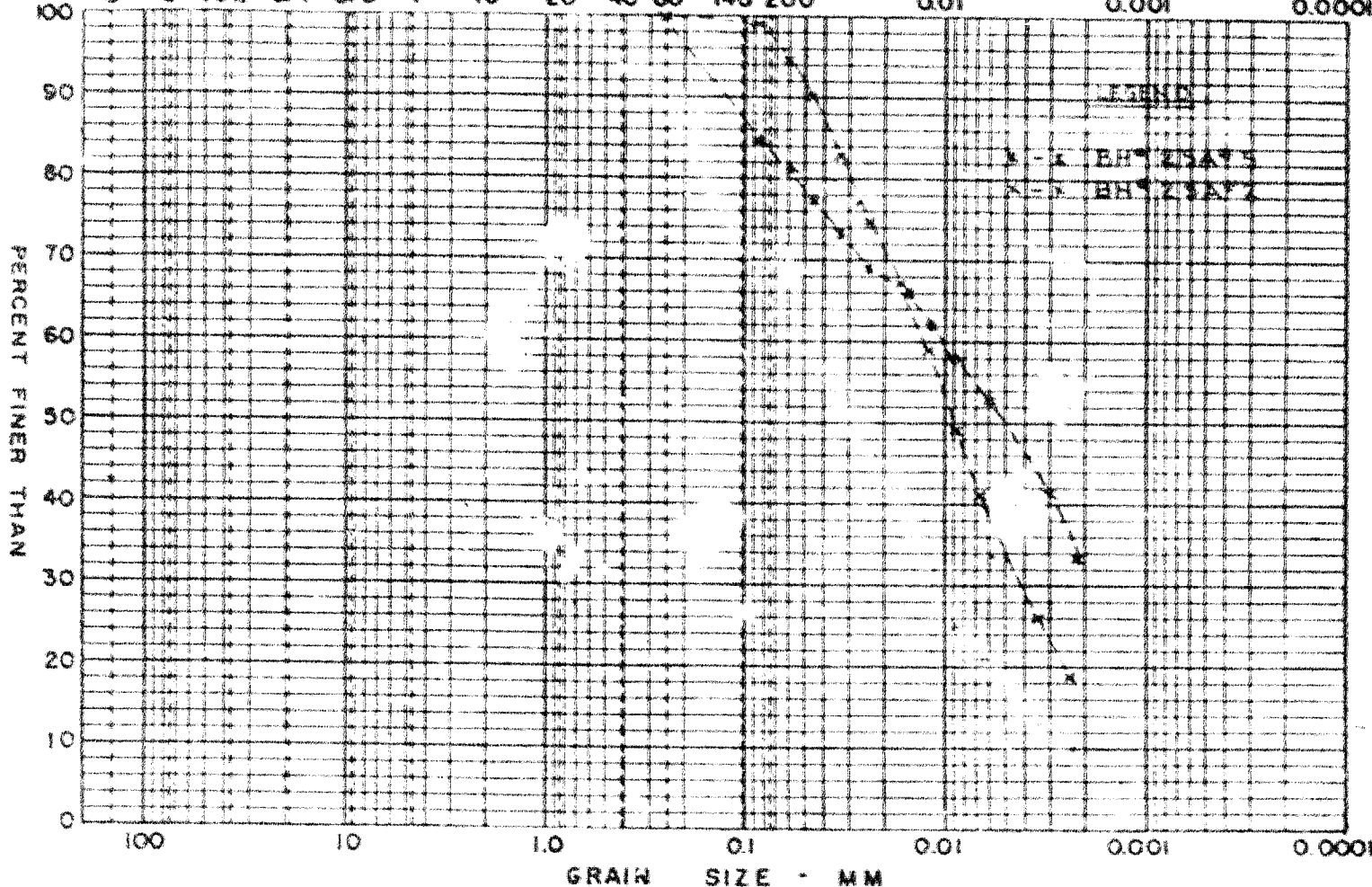
APPENDIX  
FIGURE 4  
PROJECT 07431

GRAVEL SILT (FINE LAYER)

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

SIZE OF OPENING - INS.    U.S.S. SIEVE SIZE - MESHES/IN.    EQUIVALENT GRAIN DIAMETER - MM

6"   3"   1 1/2"   3/4"   3/8"   4   10   20   40   60   140   200   0.01   0.001   0.0001



GEOCON

# GRAIN SIZE DISTRIBUTION

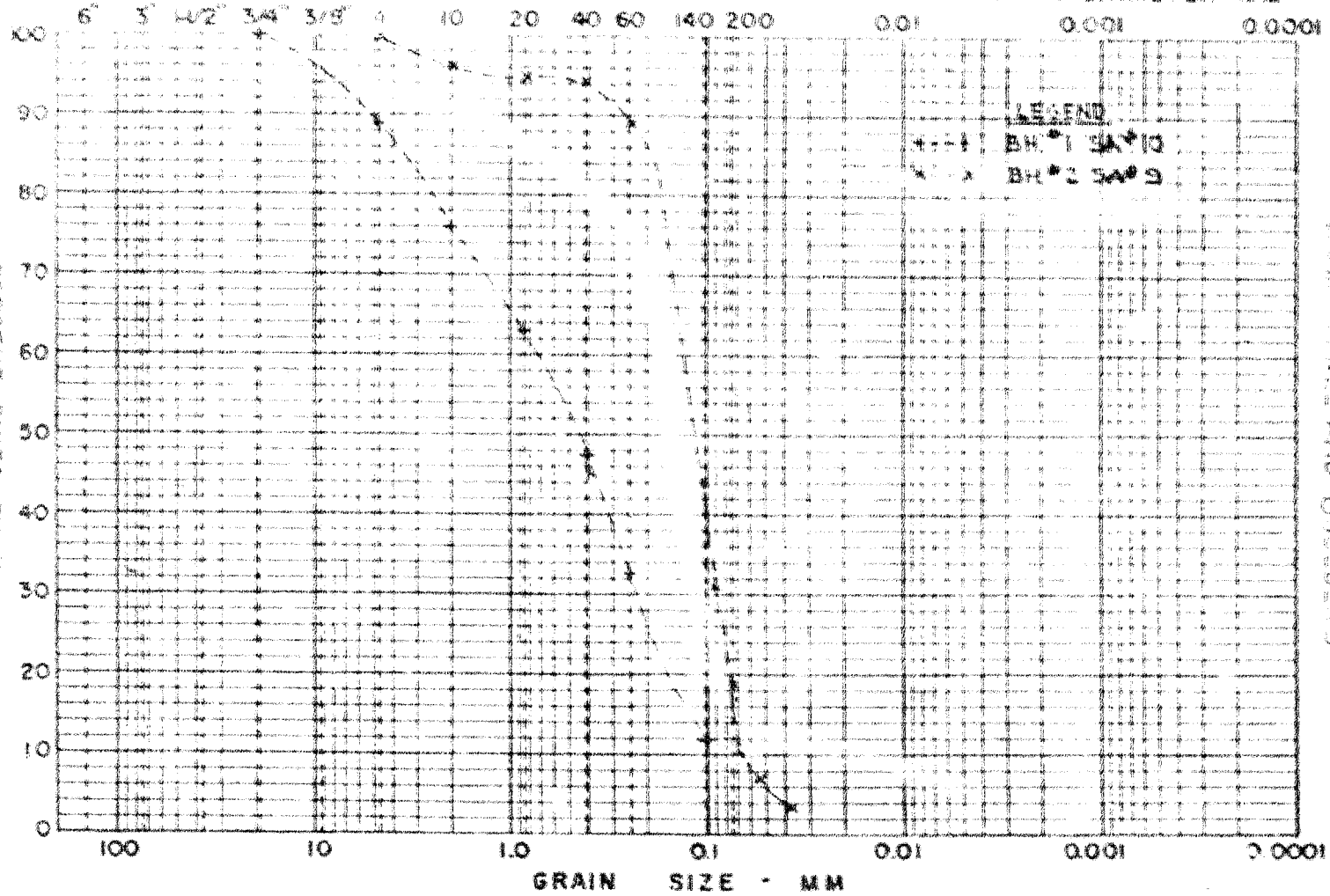
APPENDIX 11  
FIGURE 5

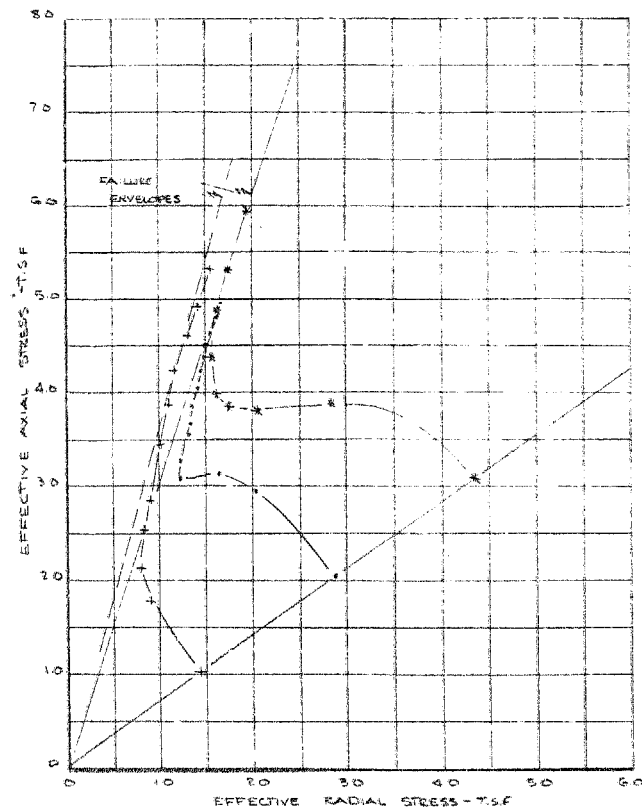
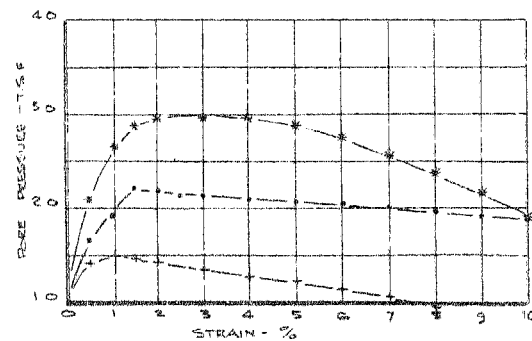
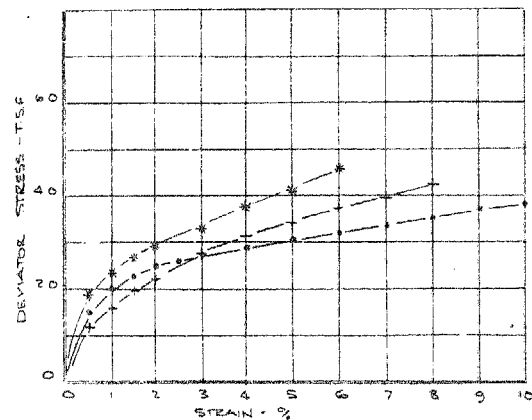
PROJECT 57431

SAND, GRAVEL AND BOULDERS

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

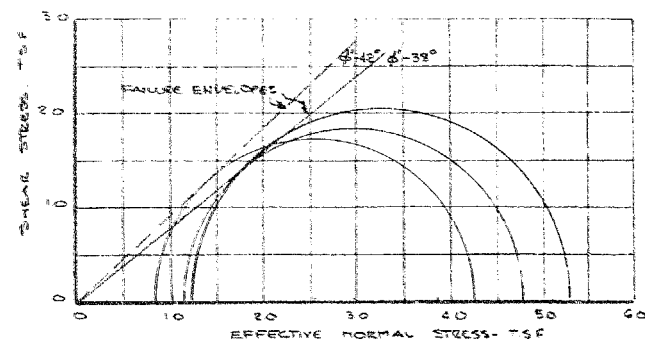
SIZE OF OPENING - INS.      U.S.S. SIEVE SIZE - MESHES/IN.      EQUIVALENT GRAIN DIAMETER - MM





TEST #	SYMBOL	BH #	SA #	DEPTH (FT)	WATER CONTENT (%)	$\gamma$ (PCF)	$G_s$ (T.S.F.)
1	+	3	2	32.0	27.5	123	1.02
2	•	13	2	32.5			2.04
3	*	2	5	40.0		114	3.06

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



# 62-F-218-C

W.P. # 69-60

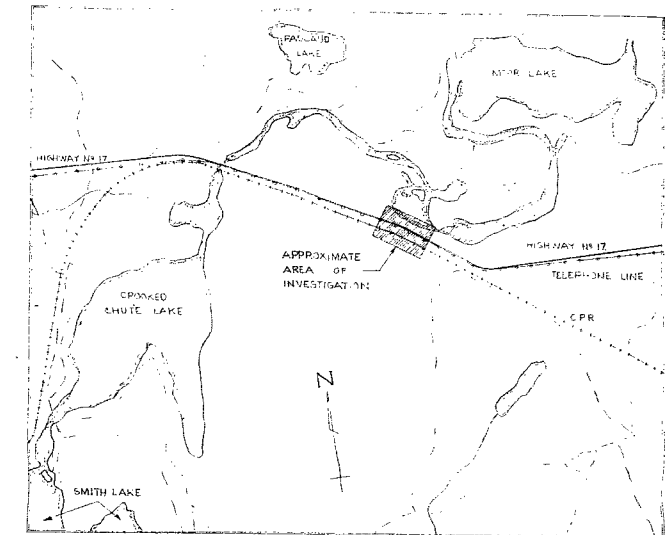
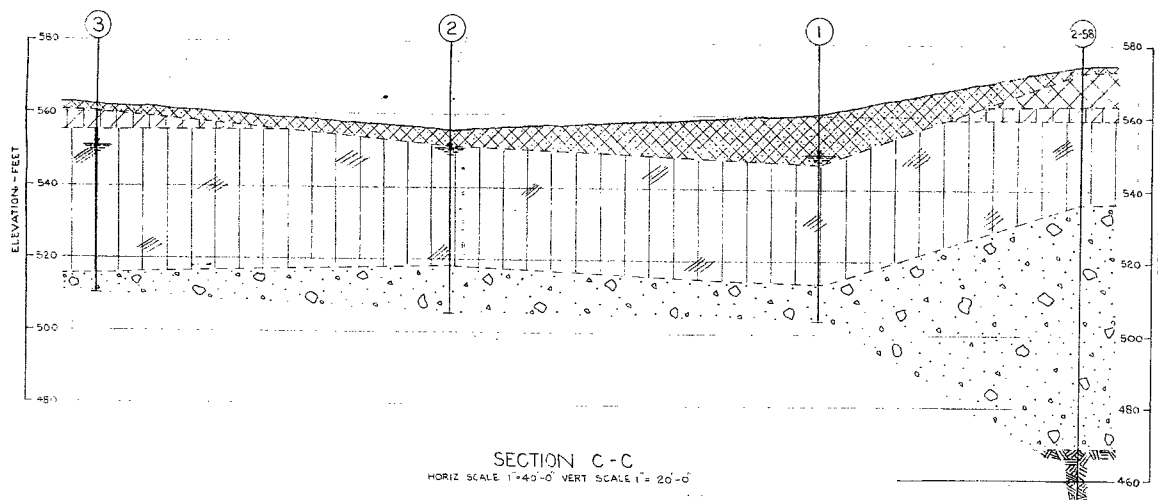
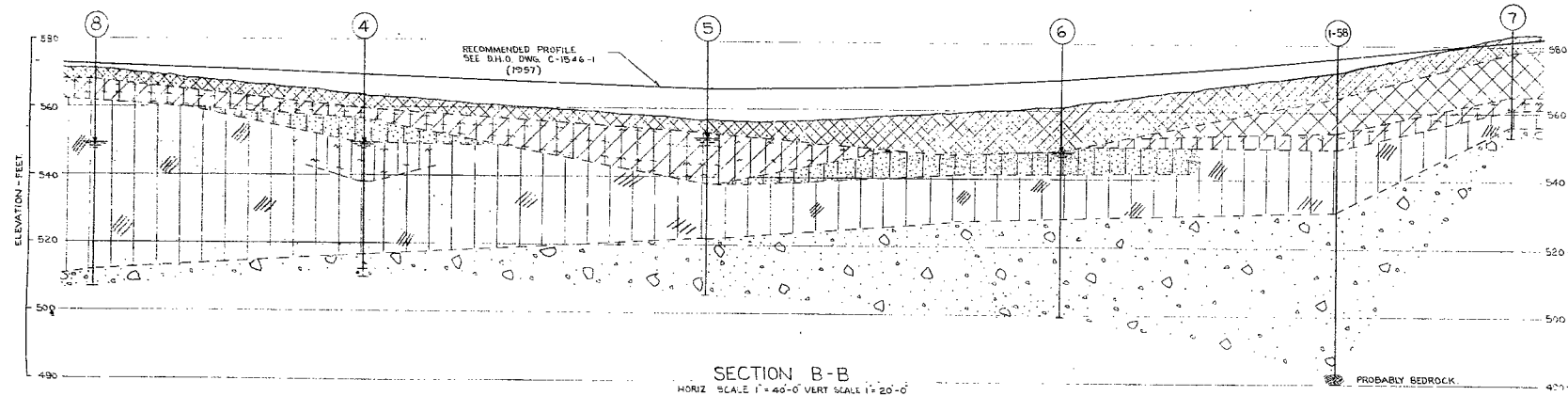
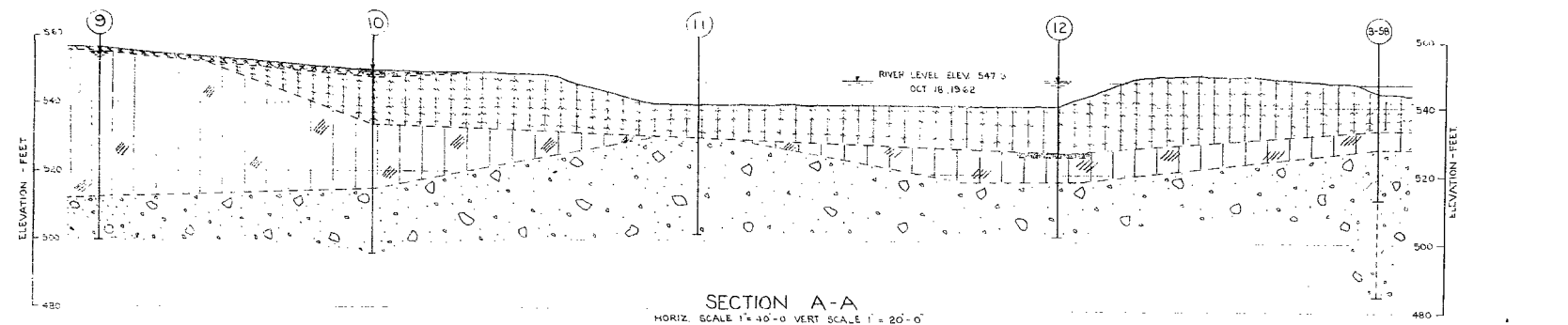
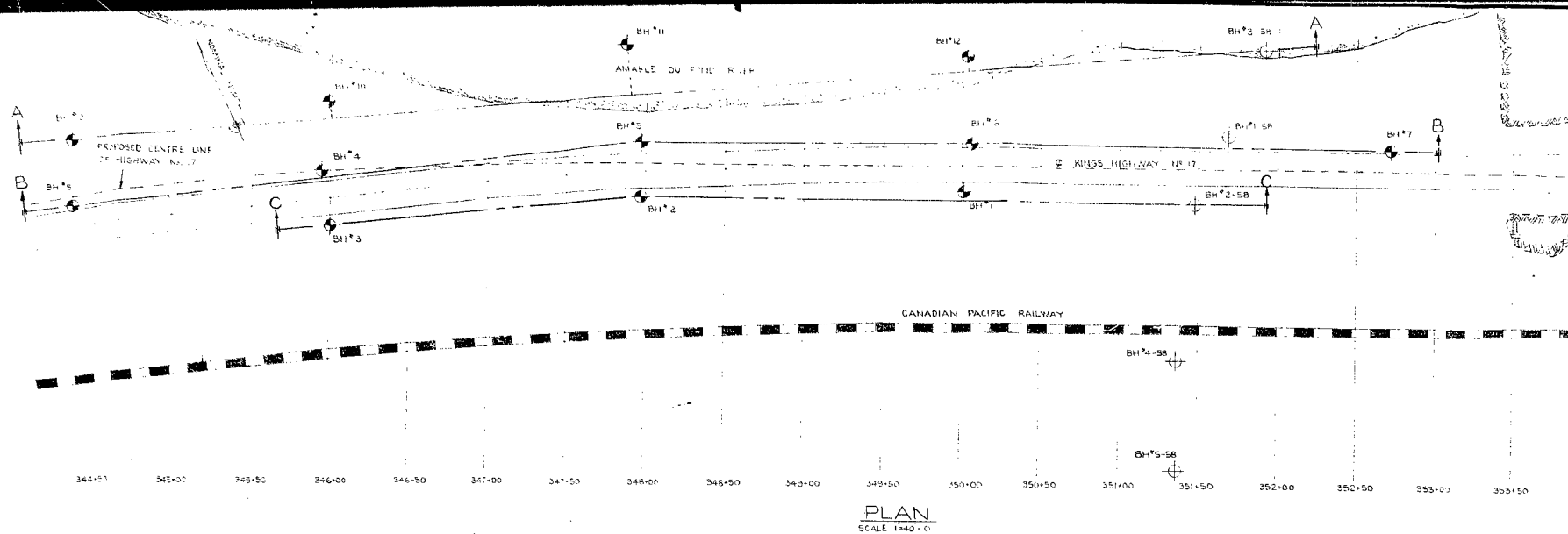
HWY. # 17

STA. 344 + 00

TO STA. 353 + 00

EMBANKMENT

NEAR MATTAWA



#### LEGEND

- BOREHOLE IN PLAN (PRESENT INVESTIGATION)
- BOREHOLE IN PLAN (PREVIOUS REPORT 1958)
- BOREHOLE IN ELEVATION (PRESENT INVESTIGATION)
- BOREHOLE WITH PENETRATION TEST IN ELEVATION
- BOREHOLE IN ELEVATION (PREVIOUS REPORT 1958)
- BOREHOLE WITH PENETRATION TEST IN ELEVATION
- WATER LEVEL IN ELEVATION OCTOBER 1962.
- ROCK OUTCROP

#### STRATIGRAPHY

- TOPSOIL
- LOOSE TO COMPACT BROWN SAND AND GRAVEL FILL
- LOOSE TO COMPACT ROCK, SAND AND GRAVEL FILL
- VERY SOFT TO STIFF BROWN TO GREY SILTY CLAY WITH TRACE OF ORGANIC MATTER
- VERY LOOSE TO COMPACT GREY SANDY SILT
- VERY SOFT DARK BROWN AND GREY ORGANIC SILT WITH SAND LAYERS
- VERY LOOSE TO COMPACT GREY AND BROWN VARVED SILT
- VERY LOOSE TO VERY DENSE GREY AND BROWN SAND GRAVEL AND BOULDERS
- GNEISS BEDROCK

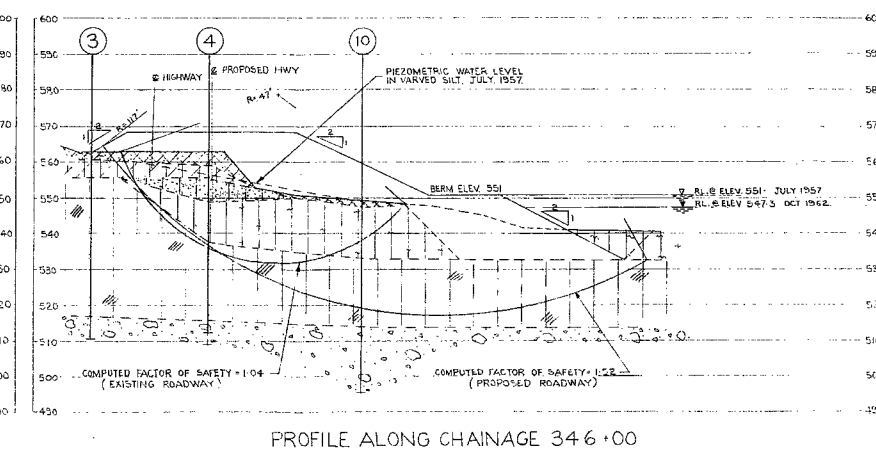
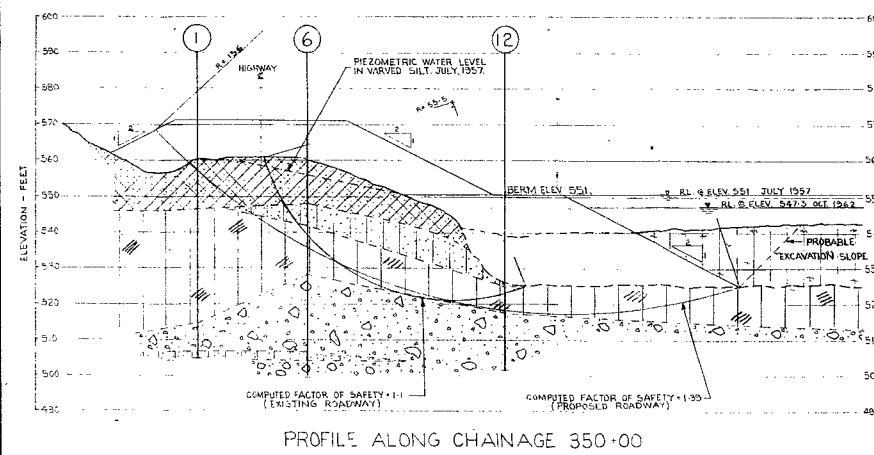
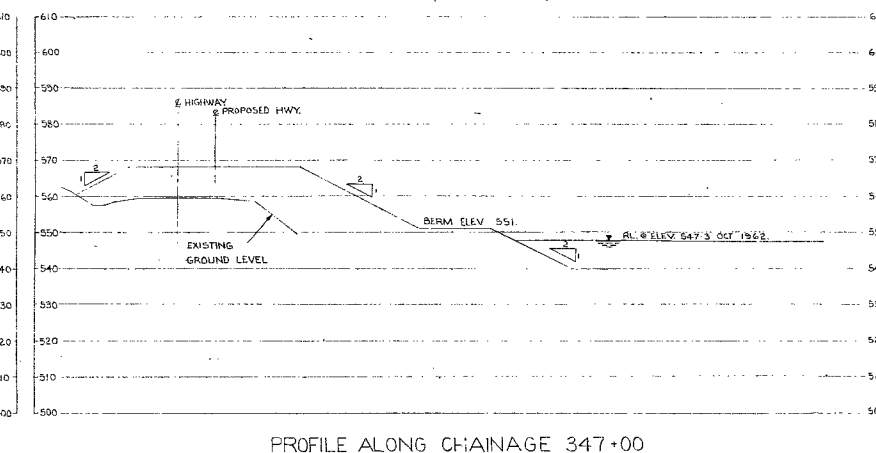
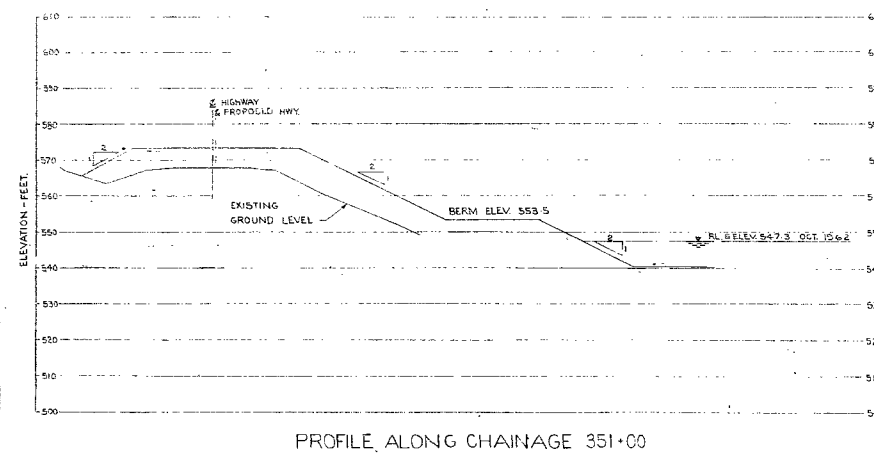
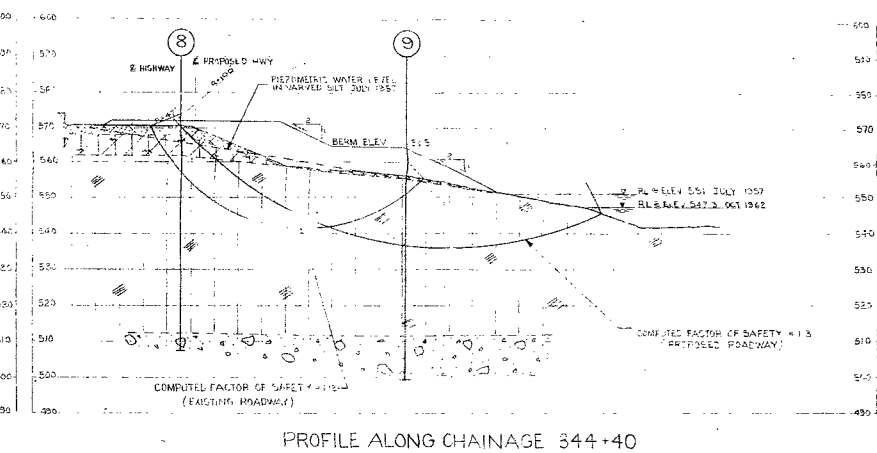
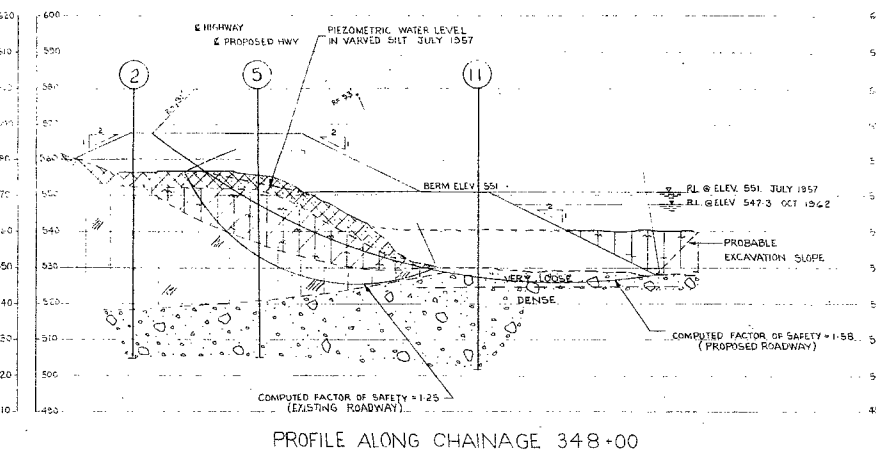
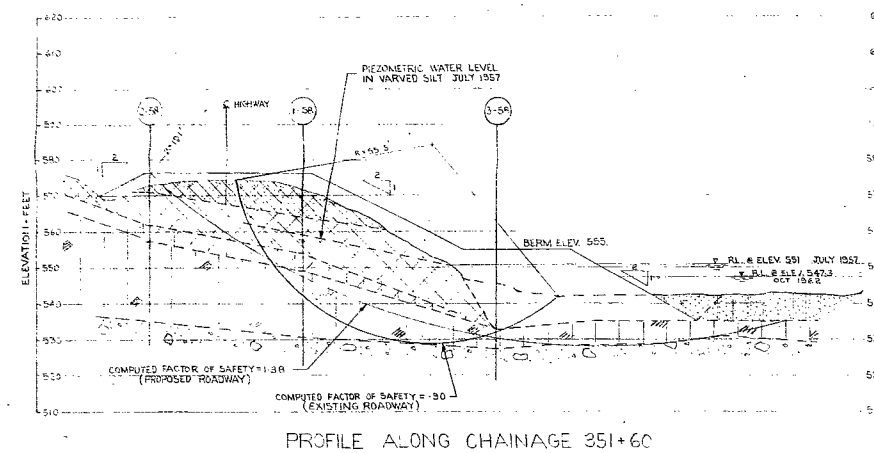
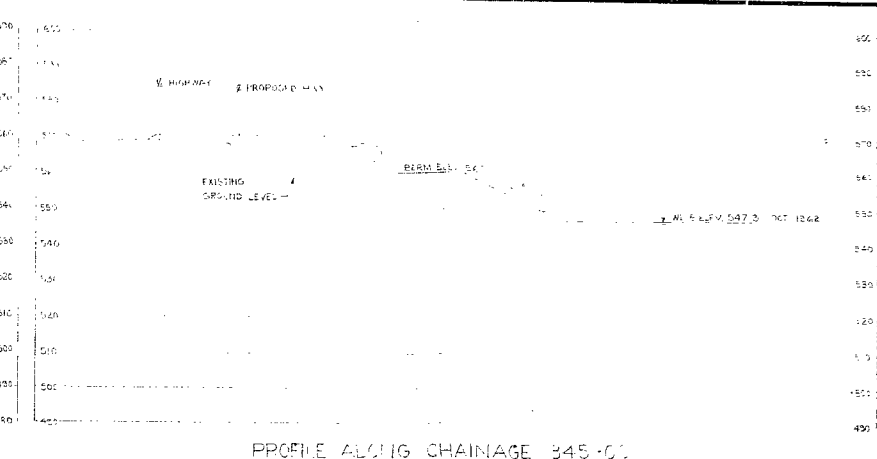
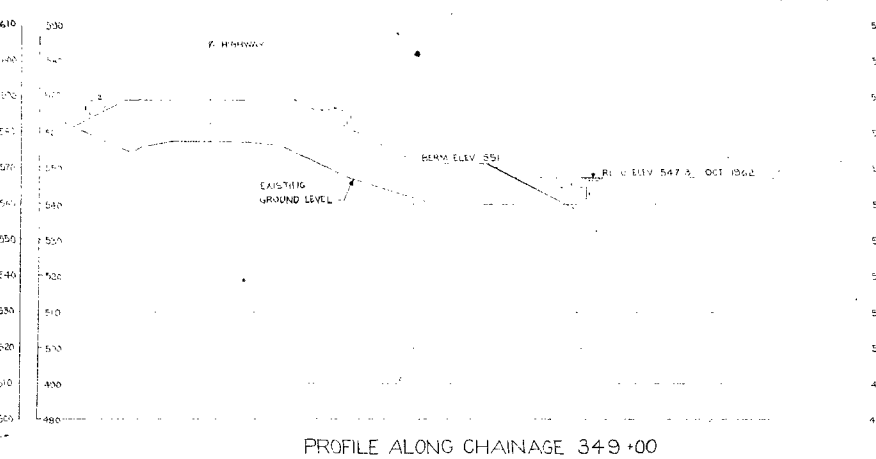
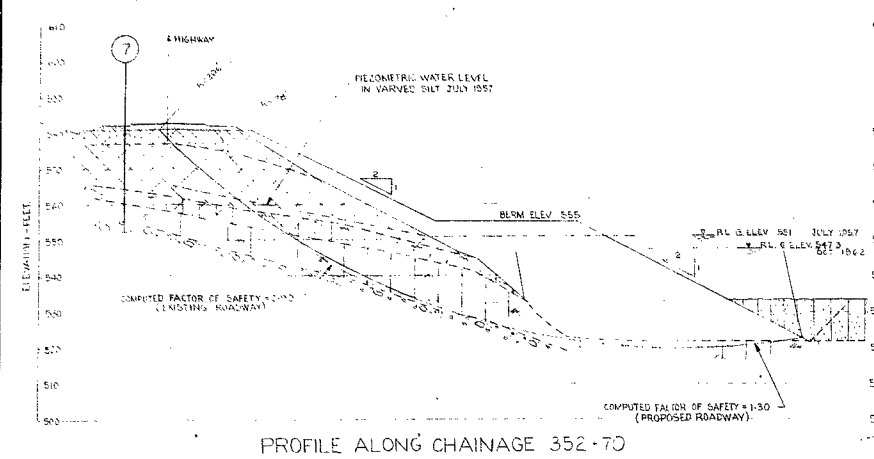
SPECIAL NOTE: DATA CONCERNING THE VARIOUS STRATA HAVE BEEN OBTAINED BY BOREHOLE LOCATIONS ONLY. THE SOIL STRATIGRAPHY BETWEEN BOREHOLES HAS BEEN INFERRED FROM GEOLOGICAL EVIDENCE AND SO MAY VARY FROM THAT SHOWN.

REVISIONS			REVISIONS			REFERENCE			REFERENCE		
MARK	DATE	DESCRIPTION	MARK	DATE	DESCRIPTION	DWG. NO.	DESCRIPTION	DWG. NO.	DESCRIPTION	DWG. NO.	DESCRIPTION
								WP-6560	DWG. NO. SUPPLIED		
								56531	GEOCON REPORT - HWY 1, STA 351+60 MATTAWA		
									ONT. DATED 1958		

DEPARTMENT OF HIGHWAYS, ONTARIO  
EXISTING AND PROPOSED HIGHWAY 17  
STA 344+00 TO STA 353+00  
MATTAWA, ONT.  
BORING PLAN AND SOIL STRATIGRAPHY

**GEOCON LTD**  
DATE NOV. 9, 1962 SCALE AS SHOWN  
MADE BY: AEL  
CHECKED BY: JCH  
APPROVED BY: JCH  
No. S 7431-1





# LEGEND

- BOREHOLE IN ELEVATION (PRESENT INVESTIGATION)
- BOREHOLE WITH PENETRATION TEST IN ELEVATION
- BOREHOLE IN ELEVATION (PREVIOUS REPORT 1958)
- RIVER LEVEL IN ELEVATION (PREVIOUS REPORT 1958)
- RIVER LEVEL IN ELEVATION (PRESENT INVESTIGATION)

DESIGN DATA		
STRATA	Y	Q' C' Ø'
SAND AND GRAVEL FILL	125.63	130°
ROCK SAND AND GRAVEL FILL	120.48	45°
ORGANIC SILT	72.15	125°
SILTY CLAY	118.53	22°
SANDY SILT	120.58	130°
VARVED SILT	120.58	12.2°
SAND GRAVEL AND BOULDERS	125.63	35°
ROCK FILL	120.48	45°

# STRATIGRAPHY

- TOPSOIL
- LOOSE TO COMPACT BROWN SAND AND GRAVEL FILL
- LOOSE TO COMPACT ROCK, SAND AND GRAVEL FILL
- VERY SOFT DARK BROWN AND GREY ORGANIC SILT WITH SAND LAYERS
- VERY SOFT TO STIFF, BROWN TO GREY SILTY CLAY WITH TRACE OF ORGANIC MATTER
- VERY LOOSE TO COMPACT, GREY SANDY SILT
- VERY LOOSE TO COMPACT, GREY AND BROWN VARVED SILT
- VERY LOOSE TO VERY DENSE GREY AND BROWN SAND GRAVEL AND BOULDERS

NOTE: SOIL BOUNDARIES OFF-SHORE FROM BOREHOLES HAVE BEEN OBTAINED BY ESTIMATION ONLY.

SPECIAL NOTE: DATA CONCERNING THE VARIOUS STRATA HAVE BEEN OBTAINED AT BOREHOLE LOCATIONS ONLY. THE SOIL STRATIGRAPHY BETWEEN BOREHOLES HAS BEEN INFERRED FROM GEOLOGICAL EVIDENCE AND SO MAY VARY FROM THAT SHOWN.

REVISIONS			REVISIONS			REFERENCE			REFERENCE		
MARK	DATE	DESCRIPTION	MARK	DATE	DESCRIPTION	NO.	DESCRIPTION	NO.	DESCRIPTION	NO.	DESCRIPTION

DEPARTMENT OF HIGHWAYS, ONTARIO  
TORONTO  
EXISTING AND PROPOSED HIGHWAY 17  
MATTAWA  
BERM DESIGN AND STABILITY ANALYSES

GEOCON LTD  
DATE NOV. 12, 1962 SCALE 1"=20'-0"  
MADE BY: [Signature]  
CHECKED BY: [Signature]  
APPROVED BY: [Signature]  
No. S 7431-2