

# 62-F-222-C

W.P. # 189-62

Hwy. # 7 E

Hwy. # 402

PERCH CREEK

FOUNDATION OF CANADA ENGINEERING  
CORPORATION LIMITED

TELEPHONE  
WALnut 5-4371

8 SPADINA ROAD  
TORONTO 4

CABLE ADDRESS  
"FOUNDANENG" TORONTO

March 12, 1963

Department of Highways, Ontario  
Parliament Buildings  
Toronto 2, Ontario

Attention Mr. F. I. Hewson  
Consultant Liaison Engineer

Dear Sirs,

PROPOSED WAWANOSH DRAIN WIDENING  
HIGHWAY 7, SARNIA, ONTARIO  
W.P. 189-62

This will confirm our recent telephone conversation when we discussed the foundation design for the above rigid frame bridge extension.

We refer you to Geocon Report S-7390 (BA 1470), Page 11, Paragraph 2 of the Conclusions and Recommendations, which recommend a bearing pressure for spread footings of 1.0 tons per square foot, for a footing width of about 10 feet. Under these conditions, the expected settlement would be about 2.0 inches. Our final design calculations indicate that if this recommended soil pressure is used, the footing width will have to be 10 feet, and we wish to draw this to your attention at this stage as the footing of the existing bridge is only 3'-6" wide.

Accordingly, we would suggest that your Foundation Section should now review the problem to see if an increased bearing pressure can be recommended since there does not appear to be any report of the existing structure behaving unsatisfactorily.

Yours very truly  
FOUNDATION OF CANADA ENGINEERING  
CORPORATION LIMITED

*R. Temple*

R. Temple, P. Eng.  
SUPERVISING ENGINEER - BRIDGES

RT/sb  
2552

c.c. Mr. A. G. Stermac, D.H.O.

March 12, 1963

Department of Highways, Ontario  
Parliament Buildings  
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SUPERVISING ENGINEER - BRIDGES

RT/sb  
2552

c.c. Mr. A. G. Stermac, D.H.O. 

Rexdale, Ontario,  
April 5th, 1963.

Foundation of Canada Engineering Corporation Limited,  
Foundation House,  
2200 Yonge Street,  
Toronto, Ontario.

Attention: Mr. R. Temple, P. Eng.

Re: Allowable Bearing Value,  
Proposed Wawanosh Drain Widening,  
Highway #7,  
Sarnia, Ontario.

Dear Sirs:

On April 1st, 1963, the writer met with Mr. A. G. Stermac and Mr. K. Y. Lo of the Foundation section of the Department of Highways, Ontario to discuss the shear strength data found in our investigation at the above site.

On comparing the results with those from neighbouring sites at which the Department or ourselves had carried out work, we found the Wawanosh Drain strengths to be typical of the area.

I had, before this meeting, checked on our calculations of strength from the field and laboratory data, and found no errors in interpretation. Based on this, we believe that the strengths as plotted on Figure 3 of Appendix II are correct. However, a slightly higher design curve may be inferred from these results in the region of the footing elevation, i.e. elevation 576.

It is understood that the existing structure is founded on spread footings at a bearing pressure well in excess of what we recommended in our report. In fact it would appear that this bearing pressure approaches the ultimate bearing value for the soil. The structure appears to be performing satisfactorily and there is no known record of any excessive settlement. Based on the performance of this structure, we recommended that a reduced factor of safety may be employed in this case. We recommend that a factor of safety of 2 be used where dead plus live load is considered, and that a factor of safety of 3 be used where dead load only is considered.

Therefore, the allowable bearing value may be taken as 1.8 tons per square foot where dead plus live load is considered and 1.2 tons per square foot where dead load only is considered. These values are based on a design shear strength of 0.6 tons per square foot.

Foundation of Canada Engineering Corporation Limited,  
April 5th, 1963,  
Page 2.

The settlements under these design loads should be less than those which the present structure has undergone. These expected settlements should then be within tolerable limits for the structure.

We trust that this information is sufficient to allow design to proceed. Should you have any questions concerning this letter, please contact us.

Yours very truly,

GECCON LTD



F. J. Heffernan, P. Eng.,  
District Soils Engineer.

FJH/dw

c.c. Mr. A. G. Stermac

S-7390

REPORT

TO

DEPARTMENT OF HIGHWAYS, ONTARIO

ON

SOIL CONDITIONS AND FOUNDATIONS  
PROPOSED WAWANOSH DRAIN WIDENING  
HIGHWAY NO. 7

SARNIA

ONTARIO

W. P. 189-62

**Distribution:**

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

**GEOCON**

# GEOCON LTD

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

Rexdale, Ontario,  
July 26th, 1962.

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

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

Re: Soil Conditions and Foundations,  
Proposed Wawanosh Drain Widening,  
Highway 7, Sarnia, Ontario.  
W. P. 189 - 62.

Dear Sirs:

This letter reports the results of the above investigation carried out in accordance with your letter of authority dated June 29th, 1962. The object of this investigation was to determine and interpret the soil and water conditions at the site as they affect the foundation design of the proposed bridge extension necessitated by the proposed widening of Highway No. 7.

## PROCEDURE

The field work was commenced on July 8th, 1962 and completed on July 10th, 1962. A total of 3 boreholes was put down using a Penndrill power auger. Shelby tube samples, in two and three inch diameter sizes and two inch sleeve samples, were taken in the clay stratum. In-situ vane shear tests, using a strain controlled vane tester, were carried out in two of the boreholes. One piezometer was installed in borehole 1 and an

PROCEDURE (continued)

observation pipe was installed in borehole 2.

The location of the boreholes, together with the inferred soil stratigraphy, are shown on Drawing S-7390-1 attached. Detailed logs of the boreholes are presented on the Office Report on Soil Exploration in Appendix I.

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

All elevations given in this report are referred to Geodetic Datum. The Geodetic bench mark No. 2965 is located on a concrete box culvert under Highway No. 402 immediately west of the Canadian National Railway crossing. The elevation of this bench mark is 595.152 as shown on your plan drawing No. E-4115-1.

SITE AND GEOLOGY

The site investigated is located adjacent to the existing bridge along Highway 7 crossing Wawanosh Drain, which is also known as Perch Creek, near Sarnia, Ontario. In general, the bottom width of the creek is about 20 feet and the banks, having a slope of approximately 2 horizontal to 1 vertical, are about 20 feet in height. The existing two lane concrete bridge is about 50 feet wide and 50 feet long and the deck level is at about elevation 600 which is

approximately 20 feet above the creek bottom.

From available geological information and previous experience in the area, it is known that the area is covered by a thick stratum of silty clay with a hard, desiccated crust. The clay is believed to overlie a stratum of sand and gravel and is sometimes referred to as a clay till.

#### SOIL CONDITIONS

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

##### Miscellaneous Fill

A stratum of fill was encountered at the ground surface in all of the boreholes. From visual inspection, the fill consists of layers or pockets of gravel and clay. The fill also contains organic matter. The thickness of these layers ranged from a few inches to 2 feet for the gravel fill, 4 to 7 feet for the clay fill, and about 4 feet for the silty sand fill. Due to the lack of continuity and similarity of the fill material encountered in the borings, it is described as one stratum in the report. However, these layers are described in detail on the borehole logs.

From tactile examination and the results of standard penetration tests, the consistency of the clay fill is estimated to be stiff and the relative density of the granular fill is estimated to be loose.

Very Stiff to Firm Brown to Grey Silty Clay

Underlying the fill as encountered in all of the boreholes is a stratum of silty clay. The stratum is desiccated by weathering and the resulting crust extends to about elevation 570 which is approximately 10 feet below the creek bottom. At its natural moisture content, this desiccated crust has a brown colour for the upper portion and changes to brownish grey and finally to grey with depth. Some sand sizes and occasional gravel sizes up to 1/2 inch were encountered throughout the stratum.

A mechanical analysis was carried out on a sample from the crust and the results are shown on Figure 1 of Appendix II. The grain size distribution curve indicates that the sample contained 25% sand sizes, 35% silt sizes and 40% clay sizes.

Atterberg limits were performed on a sample from the lower portion of the desiccated crust and liquid limit and plastic limit values obtained were 31 and 16 respectively. The corresponding natural moisture content was 21 percent. These limit values indicate that the clay is inorganic and of medium plasticity.

Undrained triaxial compression tests were carried out on samples from the stratum and the maximum compressive strength was found to range from 6 tons per square foot at the upper portion of the stratum to 0.8 tons per square foot at the

Very Stiff to Firm Brown to Grey Silty Clay (continued)

base of the stratum. Typical stress strain curves are plotted on Figure 2 of Appendix II. The results of the laboratory tests and the field vane tests are plotted on Figure 3 of Appendix II. Based on these measured strength values, the consistency of the desiccated clay is estimated to be very stiff for the upper portion and decreasing to firm with depth.

One consolidation test was performed on a sample from the lower portion of the stratum and the pressure-void ratio curve is shown on Figure 4 of Appendix II. The value of compression index  $C_c$ , was found to be 0.14 and the rebound compression index,  $C_r$ , was 0.016.

Wet unit weight determinations were carried out and an average value of 135 pounds per cubic foot was obtained.

Firm to Stiff Grey Silty Clay

Underlying the desiccated crust is a thick stratum of grey silty clay. The thickness of the stratum was not determined as all of the borings were terminated within the stratum. However, in borings carried out nearby, the bedrock was encountered at a depth of about 120 feet. From visual examination, the material in this stratum is very similar to that of the desiccated crust.

Four undrained triaxial compression tests were carried out from representative samples of the stratum. The compressive strength appears from

Firm to Stiff Grey Silty Clay (continued)

these results, to increase with depth. Results of the compression tests, together with the results of the in-situ vane tests, are shown on Figure 3 of Appendix II. For design purposes, the average strength line as shown on Figure 3 of Appendix II is recommended. From the strength versus elevation plot, it can be seen that the shear strength is about 0.4 tons per square foot at elevation 570 increasing to about 0.7 tons per square foot at elevation 530. This corresponds to a c/p ratio of about 0.25 in this elevation range. The sensitivity of the clay as determined from natural and remoulded in-situ vane tests ranges from 1.5 to 3.5.

One consolidation test was carried out on a sample from this stratum and the results are shown on Figure 5 of Appendix II. The pressure-void ratio curve shows that the stratum is normally consolidated. The compression index,  $C_c$ , was found to be 0.09 and rebound compression index,  $C_r$ , was 0.01. Wet unit weight determination gave values ranging from 132 to 139 pounds per cubic foot with an average value of 135 pounds per cubic foot.

#### WATER CONDITIONS

During the time of the investigation, the creek level adjacent to the boreholes was at about elevation 582. A piezometer was installed in borehole 1 and an observation pipe was installed in borehole 2. Two weeks after the investigation, the water level was found to be at elevation 577 in the observation pipe and at elevation 574 in the piezometer.

#### DISCUSSION

It is understood that the existing single span two-lane bridge is a rigid frame structure founded on spread footings and spanning about 50 feet. From a visual inspection, the structure appears to be performing satisfactorily. It is further understood that water levels in the creek range from elevation 596 at flood stages to a low at elevation 582.

The significant stratum for foundation design is the firm to stiff grey silty clay. Based on the shear strength properties of this clay as measured in-situ by vane, and also by triaxial tests in the laboratory, it is considered that the proposed bridge extension may be also founded on spread footings. The structure may alternatively be carried on piles as discussed later.

For frost protection, footings or pile caps should be provided with at least 4 feet of earth cover. The amount of cover required for protection against scour should also be checked for the spread footing design case. In this report, it has been assumed that positive protection, such as rip rap, would be provided if required to prevent erosion of the soil directly in front of the foundations to the abutment wall.

DISCUSSION (continued)

Based on the design shear strength profile shown on Figure 3, Appendix II, the recommended allowable bearing value for footings 10 feet in width and founded at about elevation 576, (i.e. - 4 feet below the bottom of the creek) is 1.0 tons per square foot. It is recommended that the maximum edge pressure beneath the footing resulting from a combination of vertical loading from the structure and lateral earth pressure against the abutment wall should be limited at 1.2 tons per square foot. Under the above allowable bearing values, the estimated settlement below the footings would be about 2.0 inches. In the calculation of the above settlement value, the effect of filling to elevation 600 behind the abutment wall has been considered. It is believed that both of the abutments would settle by approximately the same amount and therefore that the structure could tolerate the anticipated amount of settlement.

The factor of safety against sliding along the base of the footing should be checked and a minimum value of 1.5 used in design. For the clay, a shear strength value of 800 lbs. per square foot may be used. For the added check for long term stability, the clay may be considered frictional with an angle of internal friction of  $25^{\circ}$ . A factor of safety of 1.2 against sliding in the clay should be used.

An alternate foundation solution would be the use of friction piles. A number of pile types would be suitable. Timber piles, if used, should be treated or else located below the low water level to prevent deterioration due to alternate wetting and drying. In the design of single piles, an allowable adhesion of 300 lbs. per square foot may be used. The piled

DISCUSSION (continued)

foundation as a whole should also be checked for group action. A minimum pile spacing of 2-1/2 diameters is recommended. Preliminary computations show that the anticipated settlement under a piled group of the size that will probably be required will be of the order of one half inch.

End bearing piles would virtually eliminate settlement. However, in view of the small anticipated settlements as above and the inferred depth to bedrock of greater than 100 feet, the use of end bearing piles would probably not be warranted.

To avoid frost action and the possible build up of hydrostatic pressures behind the abutment, it is recommended that a vertical layer of free draining non-frost susceptible granular material at least 5 feet in thickness be provided immediately behind the abutment wall. The most practical design for such a drainage layer, however, which would also minimize any possible effects due to swelling of the clay on wetting and provide a construction expedient, would be to excavate the existing banks on about a 1:1 slope for the construction of the footings or pile caps, and then backfill with suitable granular material. This granular drainage layer should be connected to toe drains discharging through or around the abutment wall.

Such a provision for drainage would also improve the stability of the proposed bank and abutment under rapid draw-down conditions, as discussed later. The granular backfill should be well compacted in horizontal lifts of about one foot in thickness. A coefficient of "at rest" earth pressure " $K_0$ " of 0.4 should be used in computing lateral pressures from

DISCUSSION (continued)

this backfill in the design of the abutment walls.

The end of construction stability of the embankment and abutment wall was checked using a total stress analysis. In the computations a shear strength of 2000 pounds per square foot was taken between elevations 580 and 592. Below elevation 580 an average shear strength of 1000 lbs. per square foot was used. The resulting factor of safety against overall sliding of the embankment and abutment was found to be 2.4 which is considered satisfactory. Preliminary computations were also carried out to check on long term stability using a  $\phi'$  value of  $25^\circ$  and a  $c'$  of 100 pounds per square foot for the clay and assuming rapid drawdown conditions from elevation 596 to 589.

It was found that if a wedge of granular backfill on a 1:1 slope is provided to the abutments as described above, the factor of safety against long term instability is about 1.0. However, because of the relatively impermeable nature of the clay and the short duration during which the high water level probably exists, it is not believed likely that the pore pressures in the clay bank will build up to that value considered in our calculations. Because of the severe assumptions used, the computed factor of safety of about 1.0, considering rapid draw down conditions, is believed to be conservative and therefore the long term stability of the embankments and the abutments is considered to be adequate.

In excavations for the pile caps or the footings, assuming these are carried out at the low water stage, the water from the creek may be diverted by means of a small clay dike. The water seeping through the dikes and the underlying clay

DISCUSSION (continued)

should be small and readily handled by pumping from sumps. It is recommended that the footing excavations be overdug for about 6 inches and brought back up to grade by filling with crushed stone. This will prevent disturbance of the bottom of the excavation due to subsequent construction operations and increase the frictional resistance to sliding along the base of the abutment footing.

CONCLUSIONS AND RECOMMENDATIONS

1. The site is covered by 7 to 14 feet of miscellaneous fill which overlies an extensive deposit of firm to stiff clay with a very stiff crust. The clay was penetrated to a total depth of 65 feet.
2. The allowable bearing value for spread footings founded at about elevation 576 may be taken as 1.0 tons per square foot for a footing width of about 10 feet. The total settlement under this loading and the embankment load would be about 2.0 inches.
3. An alternative foundation solution which would reduce the anticipated settlements would be the use of friction or end bearing piles.
4. Assuming that a wedge of granular material is placed behind the abutment wall, the stability of the abutment and bank is considered to be adequate.

Department of Highways, Ontario,  
July 26th, 1962,  
Page 12.

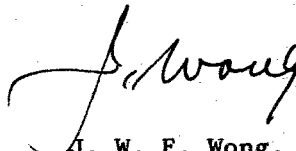
DISCUSSION (continued)

The abutment walls should be checked for stability against sliding as discussed in the report.

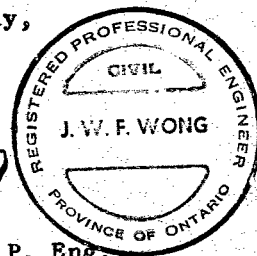
We trust that this letter, which was written by Messrs. J. Wong and F. J. Heffernan and reviewed by Mr. M. A. J. Matich, P. Eng., contains all the necessary information required for the design of foundations of the proposed structure. If you have any further questions, we would be pleased if you would contact us.

Yours very truly,

GEOCON LTD



J. W. F. Wong, P. Eng.,  
Soils Engineer.



JWFW:bc  
S-7390  
Attach.

**GEOCON**

APPENDIX I

OFFICE REPORTS ON SOIL EXPLORATION

GEOCON

## 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 57390 BORING # 1 DATUM GEODETIC CASING         
 BORING DATE JULY 8-9, 1962 REPORT DATE JULY 19, 1962 COMPILED BY M.S. CHECKED BY J.W.  
 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  
 L.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

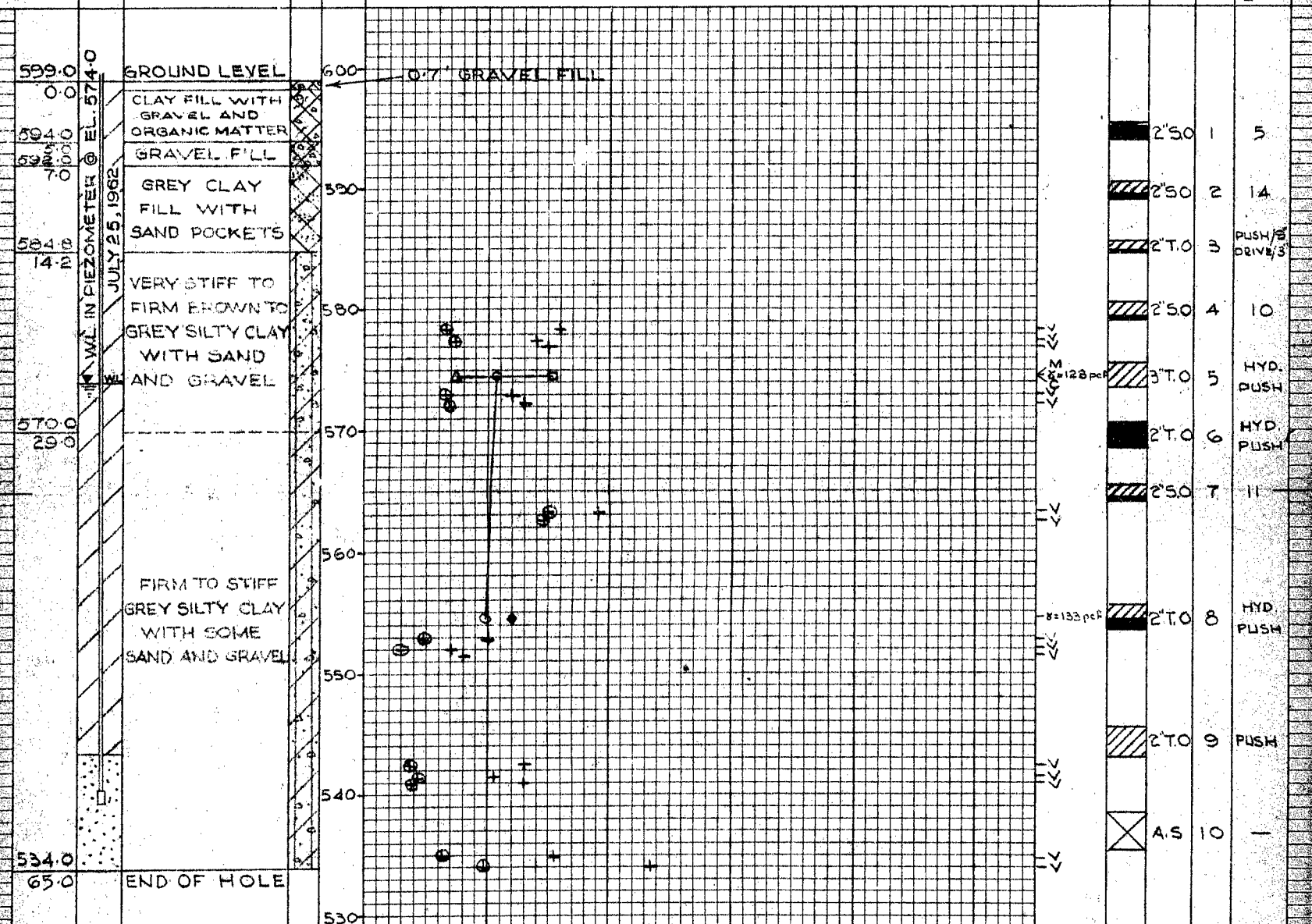
### SOIL PROFILE

COMPRESSIVE STRENGTH-TONS/SQ. FT. VANE TEST + NAT. ⊕ REM  
 • UNDRAINED TRIAXIAL

WATER CONTENT W% 0 NAT. □ LW Δ PW  
 DYNAMIC PENETRATION TEST BLOWS PER FOOT

### SAMPLES

CONDITION TYPE NUMBER PENETRATION RESISTANCE BLOWS/FT.



# GEOCON

## OFFICE REPORT ON SOIL EXPLORATION

CONTRACT S 7390 BORING # 2 DATUM GEODETIC CASING \_\_\_\_\_  
 BORING DATE JULY 9, 1962 REPORT DATE JULY 19, 1962 COMPILED BY M.S. CHECKED BY J.W.  
 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

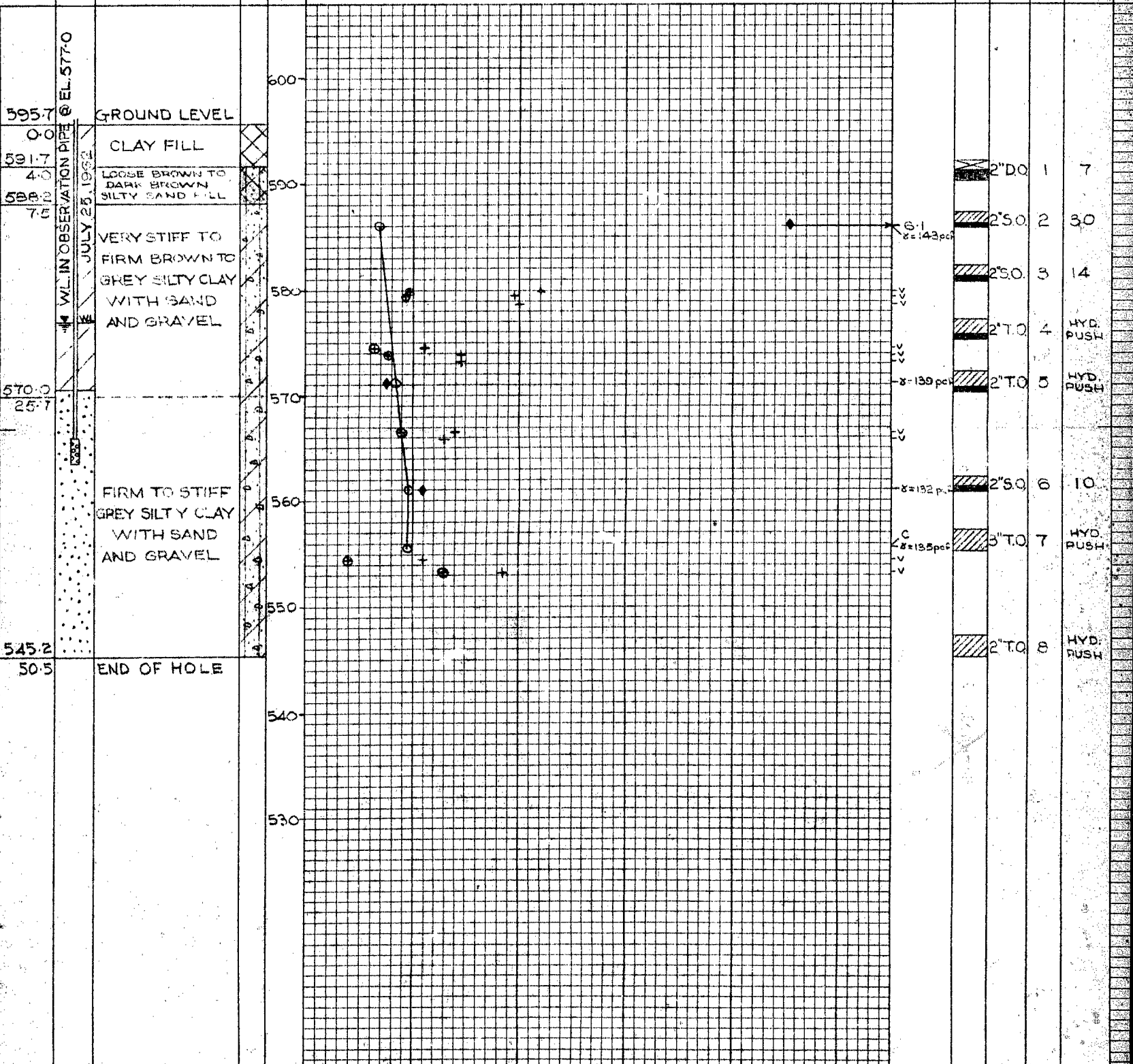
V - IN-SITU VANE TEST  
 M - MECHANICAL ANALYSIS  
 U - UNCONFINED COMPRESSION  
 Qc - TRIAXIAL CONSOLIDATED QUICK  
 Q - TRIAXIAL QUICK  
 S - TRIAXIAL SLOW  
 γ - WET UNIT WEIGHT  
 K - PERMEABILITY  
 C - CONSOLIDATION  
 WL - WATER LEVEL IN CASING  
 WT - WATER TABLE IN SOIL

### SOIL PROFILE

COMPRESSIVE STRENGTH TONS/SQ.FT.  
 VANE TEST + (NAT.) ⊕ (REM)    ♦ UNDRAINED TRIAXIAL  
 1.0    2.0    3.0    4.0    5.0  
 WATER CONTENT W% \_\_\_\_\_ 0 NAT. □ LW Δ Pw  
 20    40    60    80    100  
 DYNAMIC PENETRATION TEST BLOWS PER FOOT \_\_\_\_\_

### SAMPLES

OTHER TESTS  
 CONDITION  
 TYPE  
 NUMBER  
 PENETRATION RESISTANCE BLOWS/FT.



# GEOCON

## OFFICE REPORT ON SOIL EXPLORATION

CONTRACT S 7390 BORING # 3 DATUM GEODETIC CASING ---  
 BORING DATE JULY 10, 1962 REPORT DATE JULY 24, 1962 COMPILED BY A.I.B. CHECKED BY J.W.  
 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

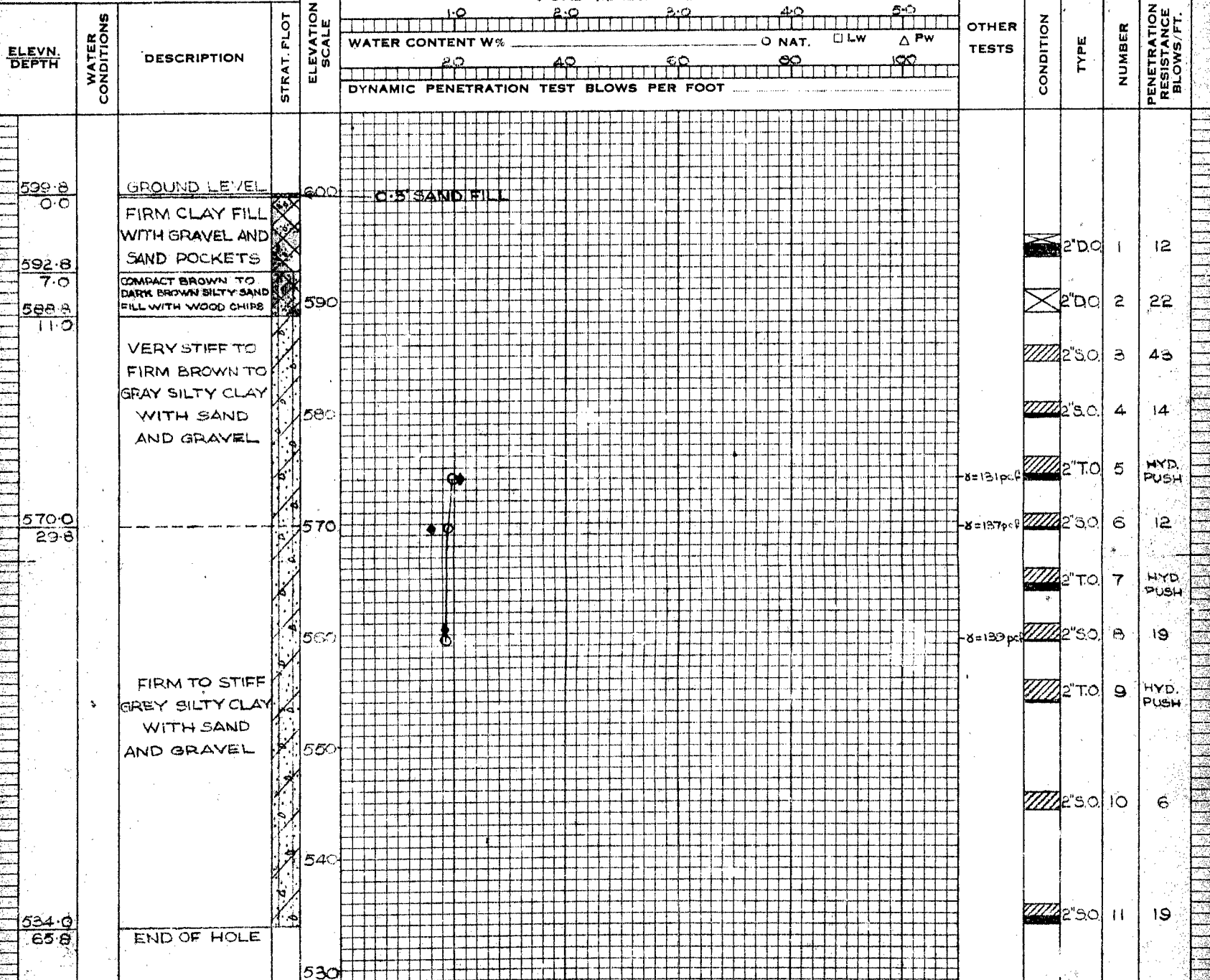
### ABBREVIATIONS

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

### COMPRESSIVE STRENGTH TONS/SQ.FT. ♦ UNDRAINED TRIAXIAL

### SAMPLES



APPENDIX II

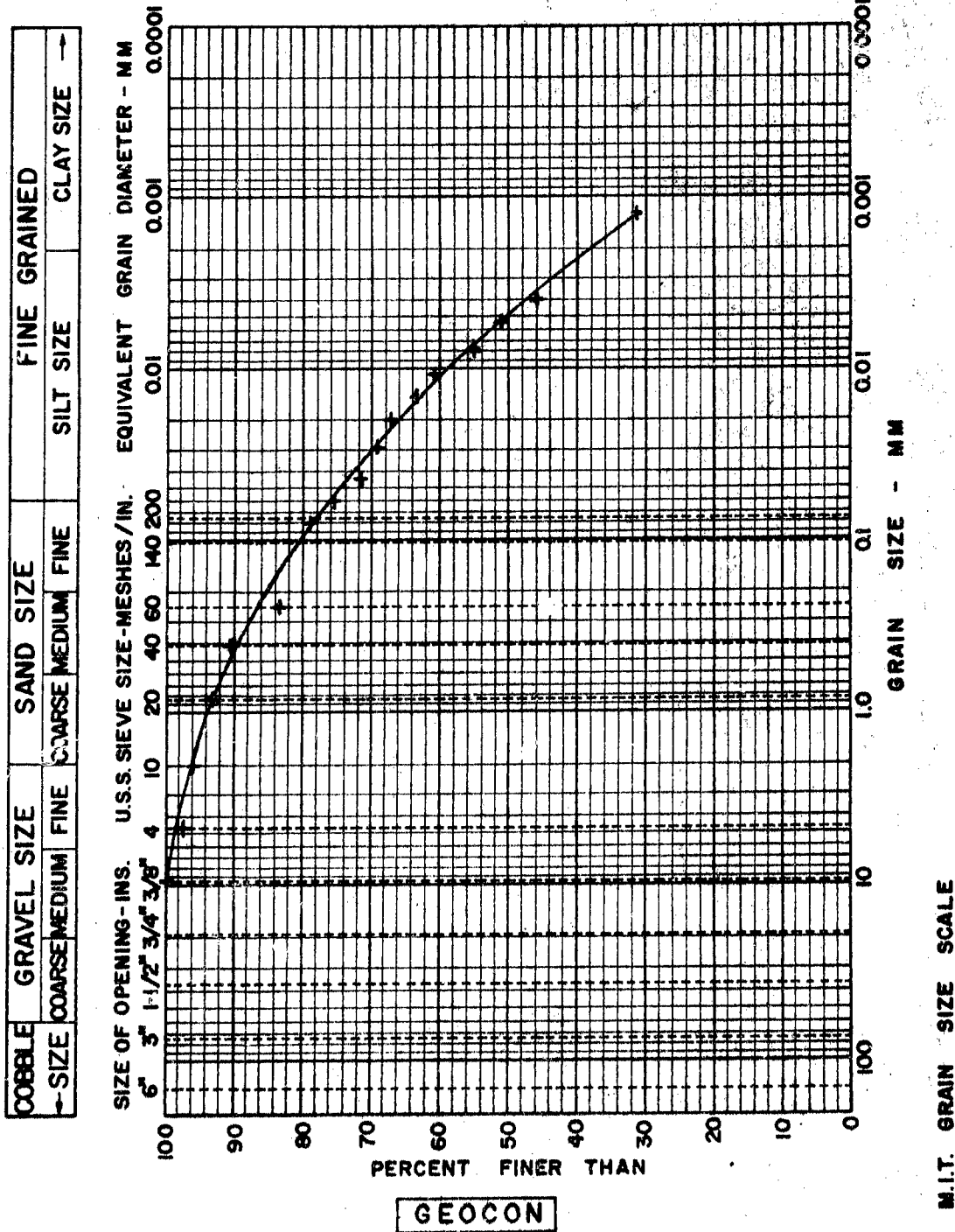
FIGURES - LABORATORY TESTING

**GEOCON**

# GRAIN SIZE DISTRIBUTION

## SILTY CLAY

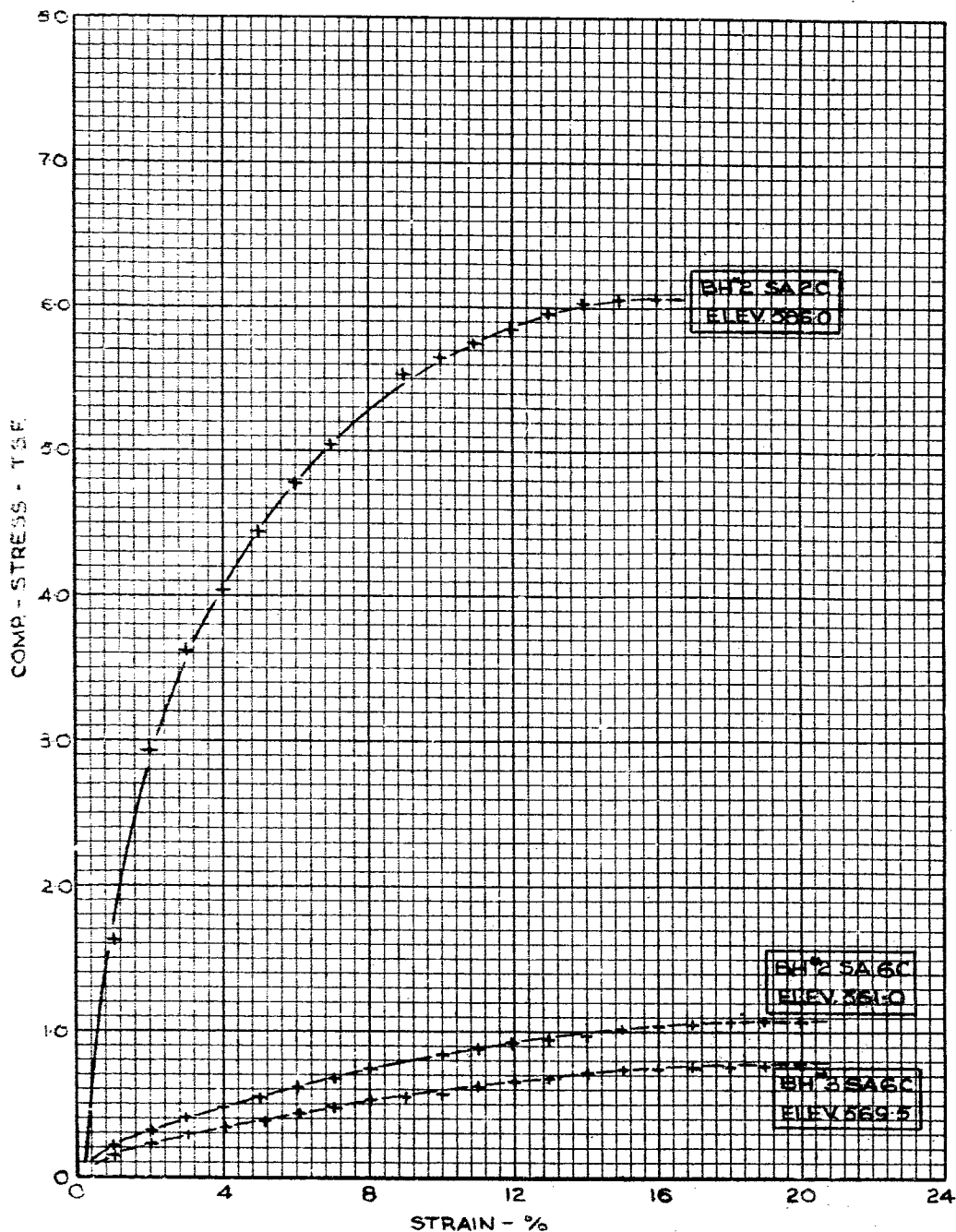
APPENDIX II  
FIGURE 11  
PROJECT S7390



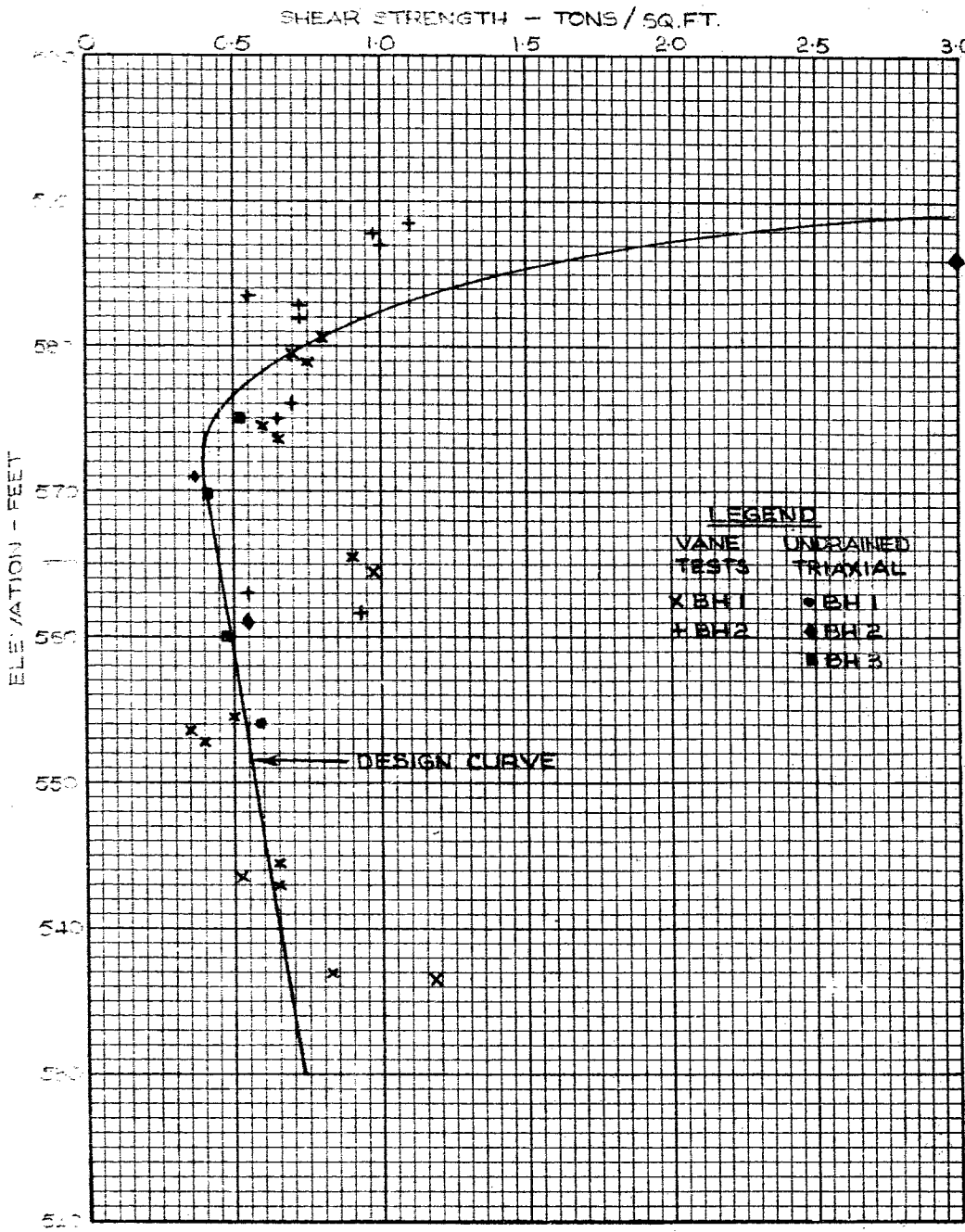
# STRESS - STRAIN CURVES

VERY STIFF TO FIRM SILTY CLAY

APPENDIX II  
FIGURE 2  
PROJECT S7390

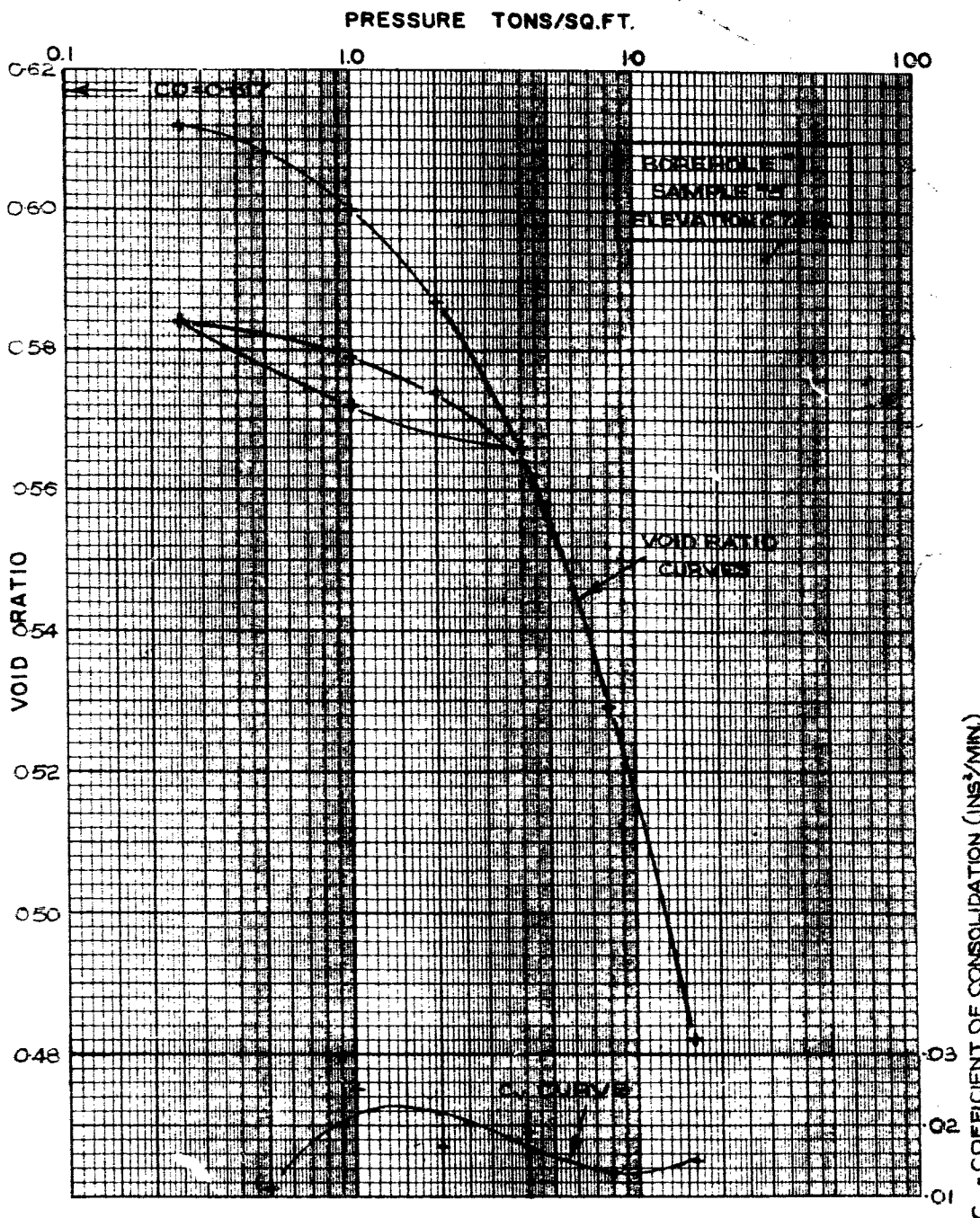


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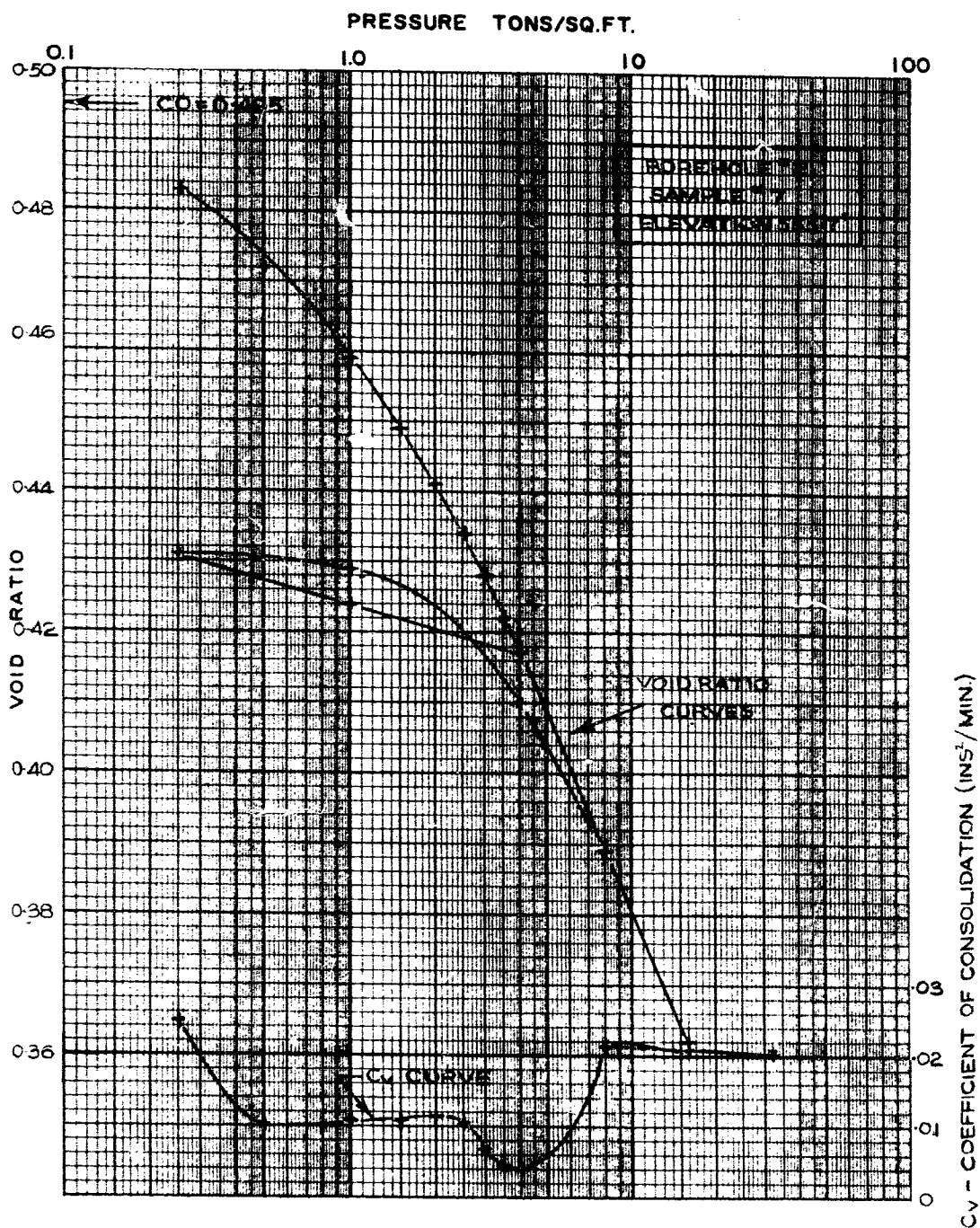
# VOID RATIO-PRESSURE CURVES CONSOLIDATION TEST

APPENDIX II  
FIGURE 4  
PROJECT S7390

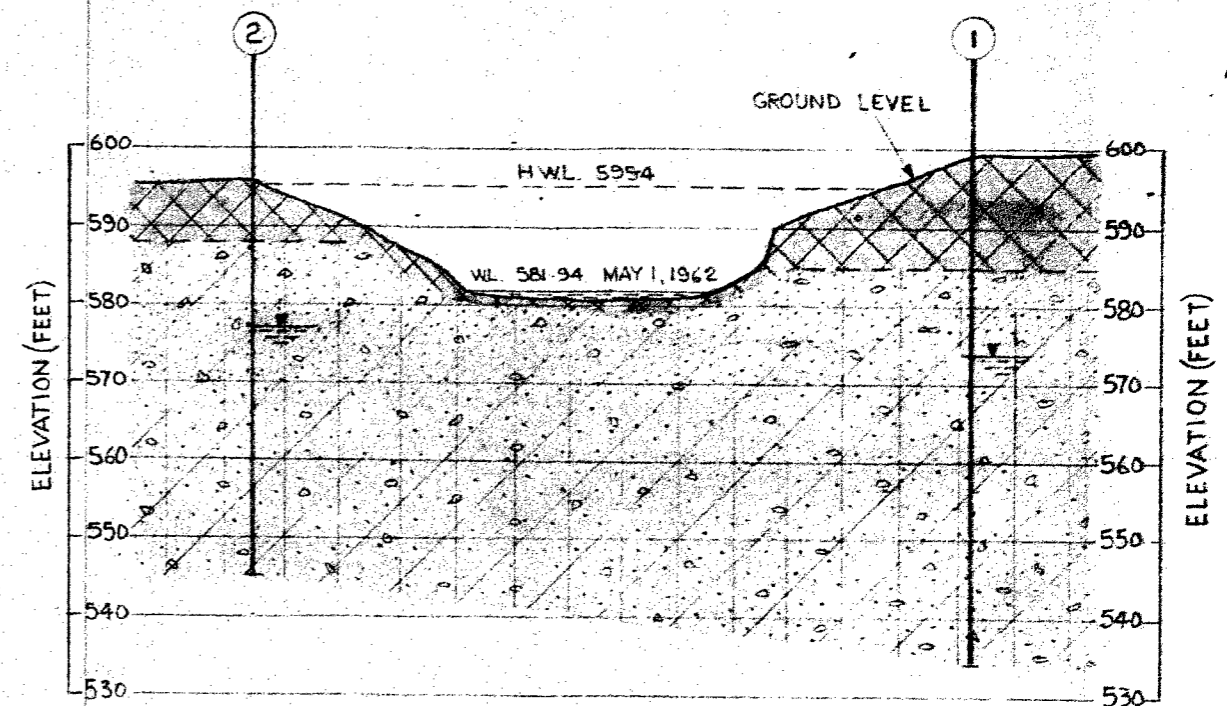
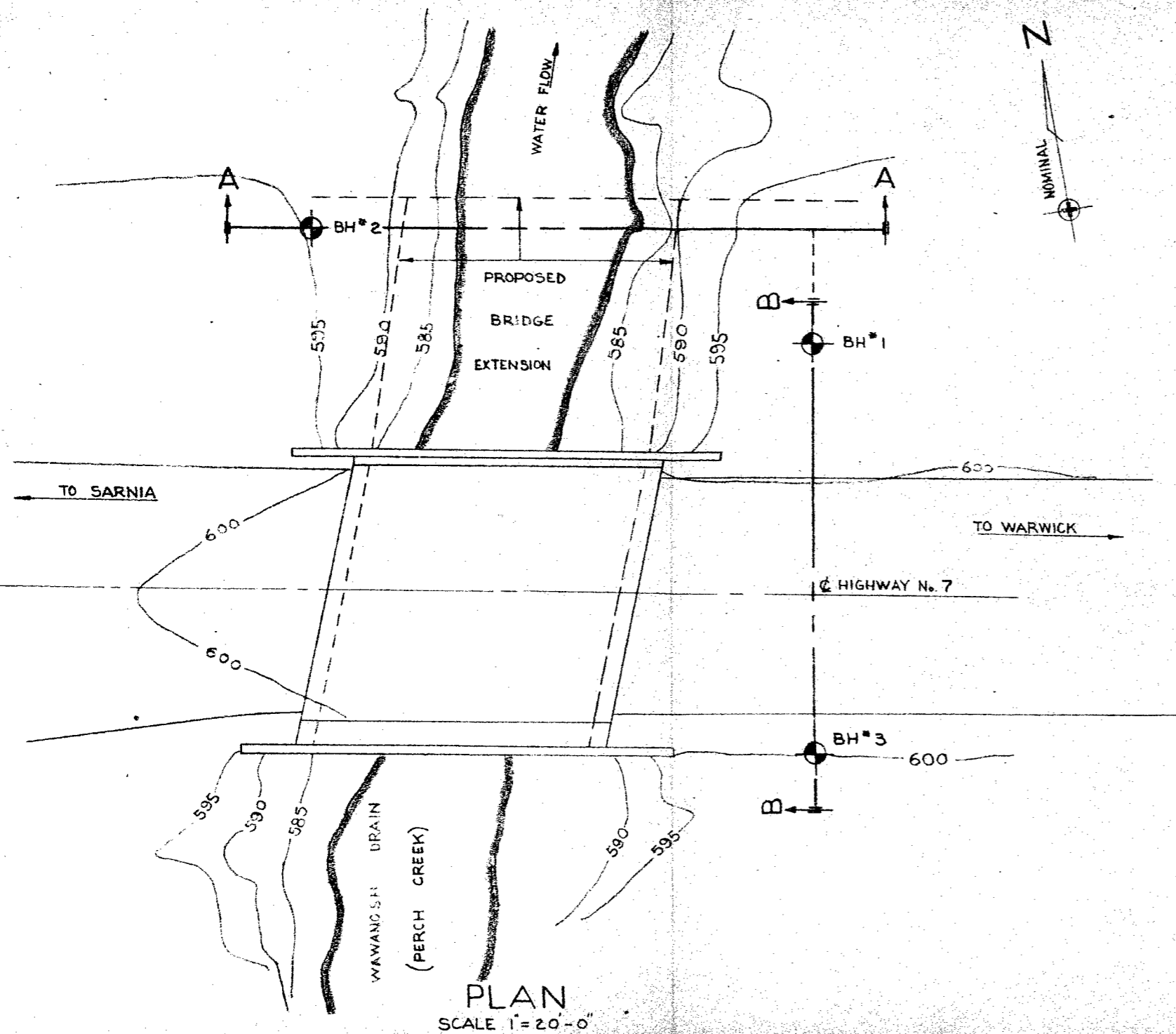


# VOID RATIO-PRESSURE CURVES CONSOLIDATION TEST

APPENDIX II  
FIGURE 5  
PROJECT S7390

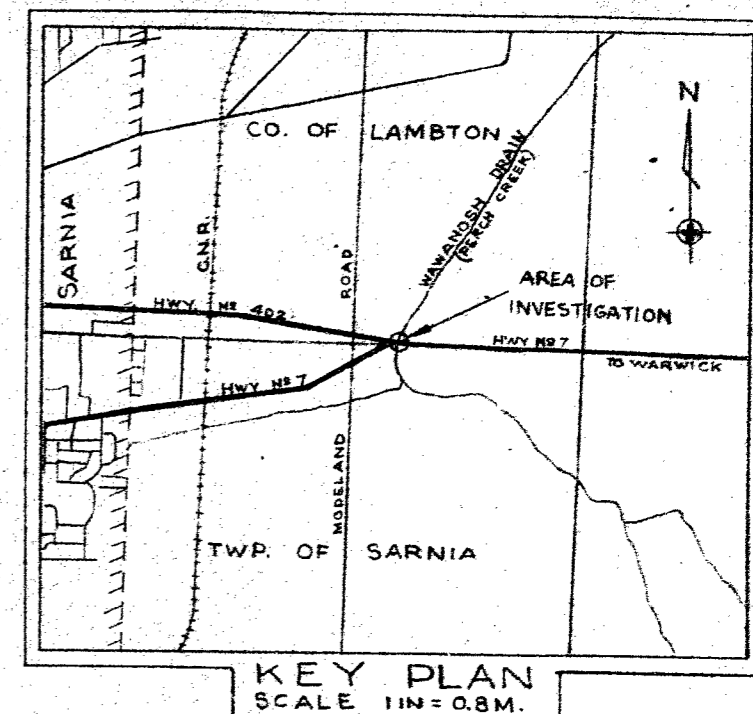
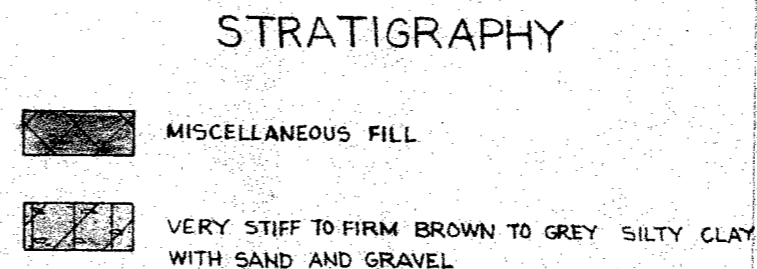
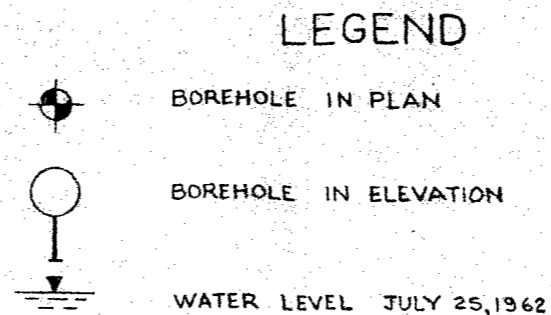
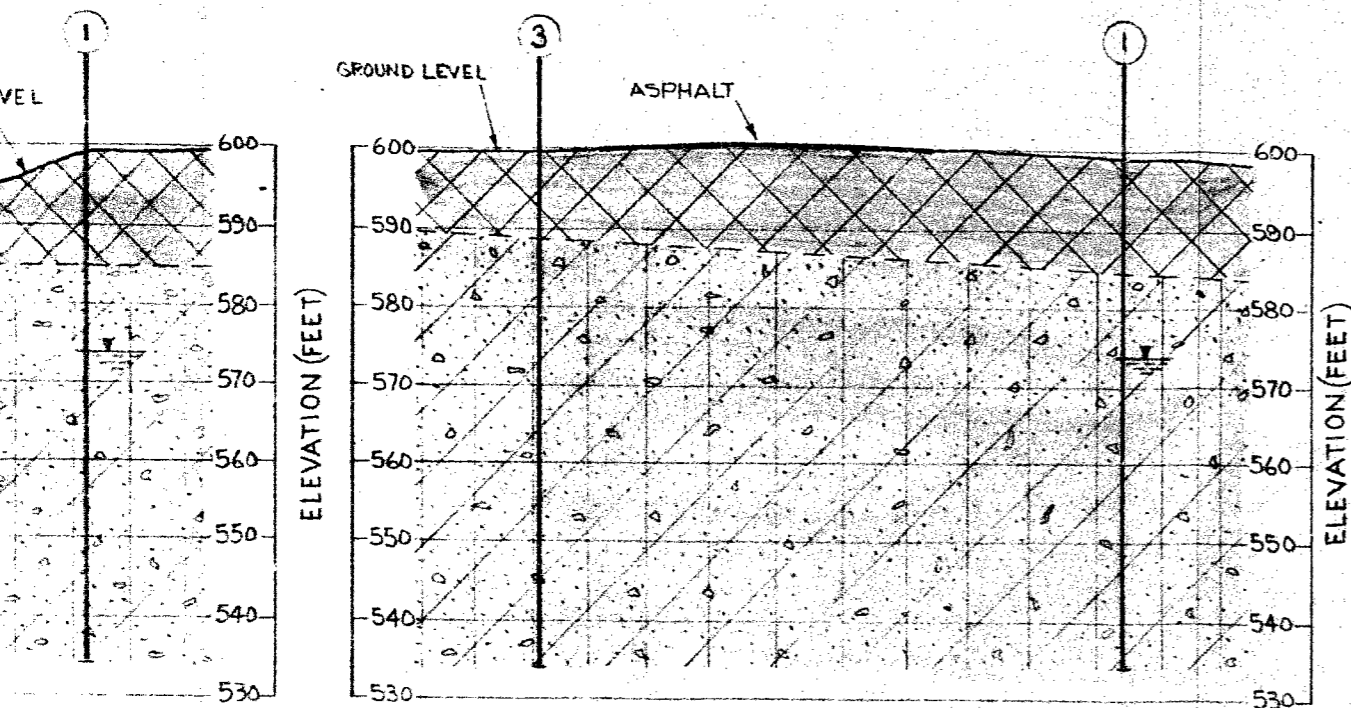


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SECTION A-A

HORIZONTAL AND VERTICAL



SECTION B-B  
HORIZONTAL AND VERTICAL SCALE 1"=20'-0"

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

DWG. NO.	REFERENCE DESCRIPTION
E-4115-1	D.H.O. PROPOSED CROSSING AT WAWANOSH DRAIN AND THE KING'S HIGHWAY No. 402 DATED - MAY 1962

DEPARTMENT OF HIGHWAYS ONTARIO  
TORONTO  
PROPOSED WAWANOSH DRAIN WIDENING  
SARNIA ONTARIO  
BORING PLAN AND SOIL STRATIGRAPHY

**GEOCON LTD**

DATE JULY 24 1962 SCALE 1"=20'-0"

MADE AEL CHKD. J.W. APPD. F.H. NO. 5-7390-1