

# GEOCON LTD

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Rexdale, Ontario,

June 19th, 1964.

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Department of Highways, Ontario,  
Materials and Research Division,  
Highway 401 & Keele Street,  
Downsview, Ontario.

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

Re: Soil Conditions and Foundations,  
Proposed C.P.R. Underpass,  
Highway #15 & #7,  
Ottawa, Ontario. W.P. 907-64

Dear Sirs:

This letter accompanies our engineering report on the above investigation.

We find that the natural soil strata at the site consist of a total thickness of about 20 feet of overburden comprising topsoil, very stiff to firm silty clay, and loose to very dense sandy till in this order. Bedrock is a sandstone. The actual soil and groundwater conditions encountered are detailed in the report.

After consideration of the proposed highway grades and the soil conditions at this proposed underpass site, it is recommended that the foundation for this structure be carried directly to bedrock as discussed. A permanent drainage system and stabilizing measures during construction will be required as discussed. The trestle involved in the detour of the railway may be carried on end bearing piles and the detour embankments would be stable if side slopes of 1 vertical to 2 horizontal are used.

We believe that this report contains the information required from this investigation. However, should you require further information

Department of Highways, Ontario,  
Materials and Research Division,  
June 19th, 1964,  
Page 2.

or if we can be of assistance in the application of the findings to design,  
as the latter develops, we would be pleased if you would give us a call.

Yours very truly,

GEOCON LTD

*M. A. J. Matich per F. J. H.*

M. A. J. Matich, P. Eng.,  
President.

MAJM/reb

T7625  
REPORT  
TO  
DEPARTMENT OF HIGHWAYS, ONTARIO  
ON  
SOIL CONDITIONS AND FOUNDATIONS  
PROPOSED C.P.R. UNDERPASS  
HIGHWAYS #15 & #7  
OTTAWA ONTARIO

Distribution:

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

June 19th, 1964

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## INTRODUCTION

Geocon Ltd has been retained by the Department of Highways, Ontario, by letter dated May 11th, 1964 to investigate and report on the soil conditions east and west of Highway #15 and #7 in the vicinity of the existing C.P.R. underpass, 3.2 miles west of the Ottawa West city limits, District 9, Township of Nepean, County of Carleton.

The object of the investigation was to determine and interpret the soil and groundwater conditions as they effect the design of foundations for the proposed relocated subway abutments, centre pier and retaining walls necessitated by the widening of Highway #15 and #7 in this area. Also to be included in the investigation are an assessment of the foundation conditions for the proposed C.P.R. detour and Highway #15 and #7 detour necessitated by the construction of the proposed underpass.

## SUMMARIZED SOIL CONDITIONS

The site is covered by approximately 2 to 3 feet of topsoil. The topsoil is underlain by 7 to 13 feet of firm to very stiff silty clay which overlies loose to very dense sandy till. The till stratum which ranges in thickness from 3 to 10 feet, overlies a light grey sandstone bedrock directly; the bedrock surface varies in depth from 18.5 to 21.5 feet below existing ground level. The groundwater level at the time of the investigation ranged from 3 to 10 feet below ground level and slopes gently to the east. At one of the boreholes artesian pressure was encountered in the bedrock.

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General

It is understood that an underpass is to be constructed along Highway #15 and #7, over which the existing C.P.R. track will pass. The proposed underpass is to be located approximately 50 feet east of the existing underpass. The location of this structure along with the existing structure, as given to us, is shown on Drawing T7625-1 at the rear of this report. The existing underpass which is a steel girder one span bridge of 26 foot width is to be replaced by a two span structure of 75 foot width.

Because of the large skew angle between the existing railway embankment and the proposed underpass structure, it is understood that retaining walls will be required on both sides of the highway.

The proposed widening of Highway #15 and #7 will make the Highway a four lane divided highway with a median strip. The existing C.P.R. railway grade is 1.4 percent towards the east and is founded on an earth fill embankment approximately 14 feet high with side slopes being about 2 horizontal to 1 vertical. The base of the rail at the existing structure is at elevation 326.5.

It is understood that the finished Highway #15 and #7 grade has not been finalized at the time this report was being prepared; tentative grades, however, were available and are dependent on the type of underpass

General (continued)

superstructure employed. For a through type structure, the finished Highway #15 and #7 grade, at the proposed structure, will be approximately at elevation 300, while for a deck type it will be 293.

It is understood that during the construction of the underpass the C.P.R. railway and Highway #15 and #7 will be temporarily detoured. The C.P.R. detour will be located north of the proposed structure on a 5 degree horizontal curve with a 1.25 percent grade towards the east. The proposed C.P.R. grade is approximately 12 feet above the present ground surface level. The Highway #15 and #7 detour is to be located south and west of the proposed structure with the finished grade a maximum of 10 feet above the present ground surface level.

Foundations

The soil conditions are suitable for the use of spread footings for the proposed abutments and wing walls. For frost protection, it is recommended that all footings be provided with at least 5 feet of earth cover. For footings carried directly on sound bedrock, the frost cover may be reduced to 3 feet. For a through type underpass structure, the Highway elevation would be approximately 300 at the structure, and thus to meet frost protection requirements the approximate foundation eleva-

Foundations (continued)

tion would be 295, i. e. close to or in the compact to dense sandy till stratum. In view of the erratic and occasionally loose condition of the till, and the close proximity of bedrock, it is recommended that all foundations for the underpass structure be taken down to the sandstone bedrock, the surface of which is at an elevation varying from 289.5 to 294.5, at the borehole locations and generally being about 291.5. For footings founded directly on the bedrock the upper portion of which is fractured at some locations, an allowable bearing value of 10 tons per square foot could be used.

For a deck type highway structure, the highway elevation would be approximately 293 at the structure. The spread of footings would thus be carried on or within the sandstone bedrock which is generally at about elevation 291.5. An allowable bearing value of 10 tons per square foot could again be used for footings founded in the bedrock. It is pointed out that the bedrock elevation beneath part of the west retaining wall is actually higher than proposed highway grade at the structure.

It is recommended that the backfill immediately adjacent to the abutments, and retaining walls consist of at least 5 feet of clean free draining compacted nonfrost-susceptible granular material, and that this



Foundations (continued)

backfill be provided with positive drainage to avoid build-up of hydrostatic pressure behind the abutments, pier and retaining walls. If this is done, the abutment should be designed for a lateral earth pressure coefficient of 0.4 with due allowance for any surcharge loads that may be involved. If the abutments and the deck are constructed integrally as a rigid frame, as may be the case with through type construction, a lateral "at-rest" earth pressure coefficient of 0.5 is recommended. Because of the comparatively non-yielding nature of the retaining walls carried on bedrock, it is recommended that they be designed for a lateral earth pressure coefficient of 0.4.

Positive permanent subsurface drainage will be required since the proposed highway grade at the underpass location will be below the observed groundwater level. For subsurface drainage purposes, perforated or open jointed drainage pipes could be used if embedded in suitable filter material, and lead to a sump or sumps, from which accumulated water would have to be removed either by pumping, or gravity if this is practical. The actual details of the drainage system would depend on the catchment area and the details of natural or other forms of existing drainage such as storm sewers.

Foundations (continued)

The abutments, pier and retaining walls should be designed for a factor of safety of at least 1.5 against sliding based on a coefficient of friction of concrete to bedrock surface of 0.8. Additional resistance may be readily obtained by keying or dowelling into bedrock.

Because of the relatively impervious nature of the sandy till, it is believed that excavations can be dewatered by pumping from sumps within sheeted enclosures. Alternatively, where space permits, the excavations could be carried out in open cut using side slopes of 1 vertical to 2 horizontal in the clay and 1 vertical to 3 horizontal in the till. Steeper slopes would be permissible in the till if measures such as filter protection were provided to prevent sloughing due to seepage into the excavation.

It is pointed out that the least favourable conditions for embankment stability will exist during construction, if unsupported excavations are used, at which time an overall height of embankment of about 35 feet (measured from bedrock to top of existing C.P.R. fill) could exist. It is important therefore if the sides of the excavation within the present embankment are unsupported, that they be cut back to at least 1 vertical to 2 horizontal for reasons of stability against sliding in the clay. From practical considerations, it would probably be desirable to have a berm of say 20 foot width at present ground level to give access to the abutment excavations

Foundations (continued)

during construction. This feature would improve the stability of the end of the embankments. In this respect further, the stability of the embankment at the abutments should be checked once preliminary design data concerning a through type structure is available.

Railway Detour

It is understood that it is proposed to construct a railway detour north of the proposed structure during construction. The detour will span the existing highway and the highway detour by means of pile supported trestles, while the remainder of the detour will consist of a compacted fill embankment. The trestles will be constructed in the vicinity of boreholes 5 and 9, respectively.

These temporary structures would probably be best founded on piles end bearing on the bedrock surface or within the sandy till stratum. A variety of pile types would be suitable in either timber, concrete or steel. The choice would be dependent on economic considerations, and perhaps on the suitability of the piles for incorporation into the trestle structure itself.

Due to the low height of the embankment portion of the detour and the relatively high shear strength of the underlying clay stratum at the site no slope stability problem is anticipated, if side slopes of 2 horizontal

Railway Detour (continued)

to 1 vertical are used. This is confirmed by the fact that the present embankment is standing at a greater height on a 2 horizontal to 1 vertical slope without any visible sign of distress.

It is recommended that the final embankment be constructed of suitable compacted granular fill with grassed side slopes of 2 horizontal to 1 vertical.

The surcharge loading applied to the underlying soil strata by the embankment would be below the observed preconsolidation load of the clay. The settlements induced by the embankment would therefore be minor.

Highway Detour

It is understood that it is proposed to construct a highway detour south east of the proposed structure during construction of the underpass structure. Available information indicates that the highway will be carried on a compacted embankment whose maximum height is 12 feet. As for the railway detour, no stability or settlement problems are anticipated for embankments of this height.

- 1)           Aside from the railway embankment fill this site is covered by topsoil, then very stiff to firm silty clay, followed by loose to very dense sandy till, then sandstone bedrock. The total thickness of overburden is about 20 feet.
- 2)           The groundwater level at the time was within 10 feet of ground surface. Artesian pressure was encountered in the bedrock at one of the boreholes.
- 3)           In view of highway grade requirements and other factors mentioned in the report, it is recommended that the foundations for the underpass be carried to bedrock as discussed.
- 4)           In the construction of the underpass foundations is carried out in unsupported excavations, stabilizing measures should be adopted as discussed in the report to prevent instability due to the load of the existing C. P. R. embankment.
- 5)           The trestle structure for the railway detour could be readily carried on end bearing piles as discussed. No stability problems are envisaged for the proposed detour embankment.

The field work was carried out under the technical supervision of Mr. B. T. Darch. This report was written by Mr. Darch, checked by Mr. F. J. Heffernan, P. Eng. and reviewed by Mr. M. A. J. Matich, P. Eng.

BTD/reb

*B.T. Darch for F.J.H.*

B. T. Darch, P. Eng.,  
Soils Engineer.

APPENDIX I

PROCEDURE

SITE AND GEOLOGY

SOIL CONDITIONS

WATER CONDITIONS

OFFICE REPORTS ON SOIL EXPLORATION

## PROCEDURE

The field work was carried out between May 11th and May 26th, 1964, inclusive. A total of 9 boreholes, each with accompanying dynamic penetration tests, was put down in BX size using a rented diamond drill rig. Two inch split spoon samples, and where appropriate, 2 inch Shelby samples were taken in the overburden. The bedrock was proven in all boreholes by rock core drilling in AXT size for depths of 8 to 16 feet. In-situ vane strength tests were undertaken in the firm grey clay stratum of the overburden. They were also attempted in the stiff crust which proved too hard for testing by this means. Piezometers were installed in five of the boreholes.

Detailed logs of the boreholes are presented on the Office Reports on Soil Exploration in this Appendix. The location of the boreholes together with the inferred soil stratigraphy are shown on Drawing T 7625-1 located at the rear of this report.

The laboratory testing of selected soil samples was carried out in the Soil Mechanics Laboratory of Geocon Ltd in Toronto. The results of the laboratory tests and the in-situ vane shear strength tests are plotted on the Office Reports on Soil Exploration in Appendix II. The soil samples remaining after testing will be stored until July 1st, 1965 at which time you will be contacted for instructions regarding their disposal.



## PROCEDURE

II

All elevations given in this report are referred to Geodetic datum. The bench mark was a cut cross on the south-east corner of 2nd tier, north stone abutment wall of C.P.R. subway. The elevation of this bench mark was given as 314.17. The boreholes were located by our field personnel.

## SITE AND GEOLOGY

A proposed underpass to take the place of the existing C.P.R. underpass is located approximately 1 mile west of Bell's Corners, Ontario at Highways #15 and #7 in the Township of Nepean, County of Carleton. The ground surface is relatively flat on the east side of the Highway at the existing underpass and rises gradually on the west side. The C.P.R. railway track is presently located on approximately 14 foot embankment with side slopes of 2 horizontal to 1 vertical.

From available geological information it is known that the area is covered by glacial marine deposits laid down by the Champlain Sea in the Pleistocene epoch. The bedrock is sandstone of the Nepean formation, Ordovician period.

## SOIL CONDITIONS

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

Embankment Fill

The railway embankment which is about 14 feet in height consists of granular fill.

Topsoil

The surficial coverage of the site is a dark brown topsoil supporting coarse grass. The thickness of the topsoil generally varies between 2 and 3 feet.

Firm to Very Stiff Greyish Brown to Grey Silty Clay

Directly underlying the topsoil, in all boreholes is a stratum of greyish brown to grey silty clay. The stratum is laminated with laminations being approximately 1/2 of an inch thick and has occasional sand seams approximately 1/16 of an inch thick. The thickness of the stratum varies from 7 feet in borehole 9 to 13 feet in boreholes 1, 4 and 7 with an average of 11 feet. The silty clay is greyish brown at its surface and becomes grey with depth. For purposes of presentation the stratum is shown as two separate layers on the Office Reports on Soil Exploration and the stratigraphy on Drawing T 7625-1. However, the whole stratum has the same geological origin and the colour change is believed to be an indication of the oxidization and desiccation which has occurred since deposition. The average thickness of the greyish brown silty clay is 7-1/2 feet. From visual examination the greyish brown clay has a blocky and friable structure.

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

Five Atterberg limits were carried out on samples from the silty clay stratum and gave values of the liquid limit ranging from 58 to 77 with an average of 65. The plastic limit ranges from 24 to 32 with an average of 26. The corresponding natural moisture contents ranged from 29 to 58 percent with an average of 44 percent. The Unified Soil Classification system indicates that this material is an inorganic clay of high plasticity. The results are plotted on the Office Reports on Soil Exploration in this Appendix and on Figure 2 in Appendix II.

Six undrained triaxial compression tests and five unconfined compression tests were performed on representative samples from this stratum. The compressive strengths from the unconfined tests varied from 1.0 to 2.9 tons per square foot while the results of the quick triaxial tests varied from 0.5 to 2.2 tons per square foot. The failure strains as obtained from the triaxial compression tests varied from 4 to 11 percent with an average of 9 percent. Field in-situ vane shear strength tests were attempted in the stratum. In all cases the greyish brown crust was too stiff to carry out any shear strength tests with the vane tester; it was possible however, to carry out two vane tests in the grey clay. The results of these two tests gave shear strengths of 1200 and 1400 pounds per square foot. The results are shown on the Office Reports on Soil Exploration in this Appendix and on Figure 3 in Appendix II.

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

Figure 3, which is a plot of compressive strength versus depth shows some scatter of results. It is believed that some of the low compressive strength values are due to unavoidable sample disturbance during sampling and while preparing the samples of the sensitive clay for testing. In addition the strength results for the upper part of the stratum are probably on the low side due to the friable structure of the clay in this zone. Because of these observations it is believed that the vane strengths give the best indication of the shear strength of the grey clay.

Based on the results of the laboratory tests and the field shear strength tests, the consistency of the stratum is estimated to range from firm to very stiff and to be generally stiff. The sensitivity of the clay as measured from the undisturbed and remoulded tests was found to be 5 and 9 thus indicating that the clay is sensitive.

The wet unit weight as determined from eleven determinations varied from 105 to 118 pounds per cubic foot with an average of 111 pounds per cubic foot.

The results of a consolidation test carried out on a sample from the lower portion of the grey brown clay crust is shown on Figure 4 of Appendix II. The results of the test indicate that the sample is probably preconsolidated by a pressure in excess of 4.0 tons per square foot.

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

The compression index " $C_c$ " from the test results is 0.8 and the rebound compression index " $C_R$ " is 0.023. The coefficient of consolidation " $C_v$ " generally varies between 0.01 and 0.23 square inches per minute.

Seven standard penetration tests carried out in the clay gave " $N$ " values ranging from 7 to 16 blows per foot. Based on these " $N$ " values and the results of the dynamic penetration tests the consistency of the clay is estimated to be generally stiff to very stiff near the surface decreasing to firm with depth.

Loose to Very Dense Grey Sandy Till

Directly underlying the silty clay stratum in all boreholes is a stratum of grey sandy till with boulders. The thickness of the stratum varies from 3 feet in borehole 7 to 10 feet in borehole 9 with an average of 6 feet. The till has a matrix of silt and clay binding sand and gravel. Some boulders up to 6 inches in size and randomly spaced were encountered throughout the stratum; it was necessary to diamond drill them in two of the boreholes. Larger boulders may occur elsewhere in the stratum. Occasionally in the upper 2 feet of the stratum the till was in a loose condition and possessed a clayey nature. The gravel component of the till displayed angular and sub-angular shapes, characteristic of such glacial deposits.

Loose to Very Dense Grey Sandy Till (continued)

Mechanical analyses were carried out on three typical samples and the results are shown on Figure 1 of Appendix II. The grain size distribution curves indicate that the material consists of the following: 10 percent clay sizes, 17 to 22 percent silt sizes, 35 to 52 percent sand sizes and 21 to 35 percent gravel sizes.

Twelve standard penetration tests carried out in the stratum gave "N" values ranging from 3 to 57 blows per foot with an average of 18 blows per foot. The lower values were in a transition zone at the surface of the till. The nine dynamic penetration tests, 6 of which met practical refusal within the stratum gave values ranging from 10 to greater than 100 blows per foot with an average of approximately 26 blows per foot. The higher values indicated by the dynamic penetration tests were probably partially due to the skin friction developed on the rods in the clay stratum. Based on these values the relative density of the stratum below the transition zone is estimated to range from compact to very dense, and to be generally compact.

For design purposes the following parameters may be used, where appropriate:

Wet unit weight	140 pounds per cubic foot
Submerged unit weight	78 pounds per cubic foot
Angle of shearing resistance	35 degrees

Light Grey Sandstone Bedrock

Directly underlying the till stratum in all boreholes is a light grey sandstone bedrock. The bedrock was proven by diamond core drilling in AXT size for a depth generally greater than 10 feet, but in no case less than 8-1/2 feet. All the boreholes were terminated in the bedrock. It is estimated that on the west side of the highway at the existing underpass the top 5 to 6 feet of the bedrock is in a fractured and weathered condition while below this depth and on the east side of the highway the bedrock is sound as indicated by the high core recovery, and the appearance of the core.

The bedrock is composed mainly of a quartz mineral sand highly cemented and has a hardness of approximately 6 on the Moh's Hardness scale.

WATER CONDITIONS

The groundwater level was encountered at depths below ground surface ranging from 3 feet in borehole 5 to 10 feet in borehole 4 with an average depth of 6-1/2 feet. These readings were taken during the period of investigation in the 5 piezometer installations and in the open holes of the other four boreholes. The water level elevations range from elevation 312.5 in boreholes 5 to 300.0 in borehole 4. It is inferred from the water levels plotted on the stratigraphy on drawing T 7625-1 that the groundwater

level slopes gently to the east at the site.

At borehole 9, which is located at the intersection of the proposed highway detour and C.P.R. detour, artesian water pressure was encountered within the bedrock at a depth of between 25 and 30 feet below ground surface level. The flow encountered was continuous from the BX casing; on pulling back the casing the flow reached equilibrium with about 4 feet of BX casing extending above ground surface. The open borehole was observed 3 days after completion and it was found that the hole was still free flowing.

It is believed that the groundwater table in the overburden at the time was at or slightly above the contact of the greyish brown and the grey clay.



## 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".

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## OFFICE REPORT ON SOIL EXPLORATION

CONTRACT T7625 BORING # 1 AND 2 DATUM: GEODETIC CASING BX.  
 BORING DATE MAY 13-15, 1964 REPORT DATE JUNE 1, 1964 COMPILED BY AEL. CHECKED BY F.T.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

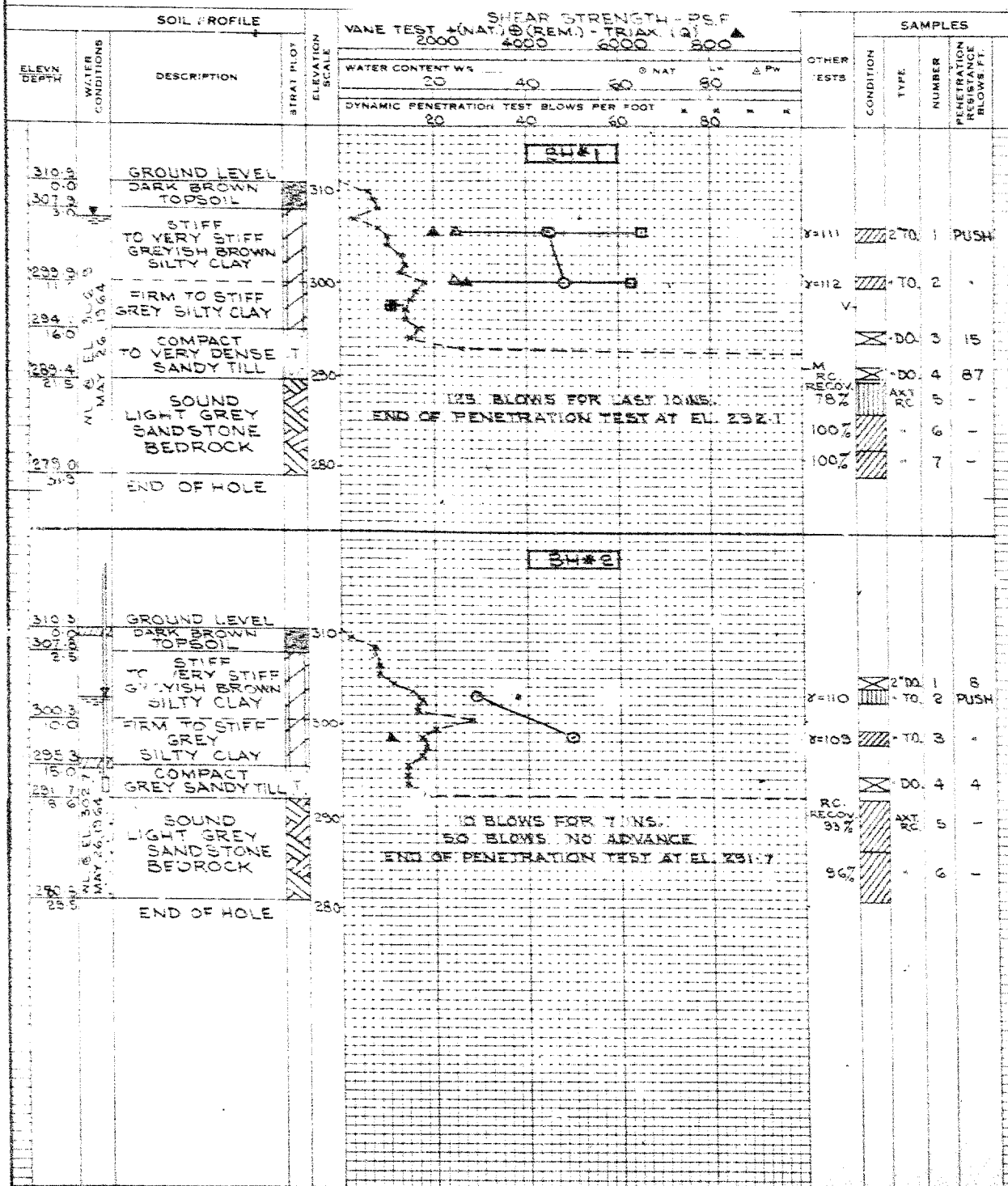
## SAMPLE TYPES

AS AUGER SAMPLE  
 ST SLOTTED TUBE  
 WS WASHED SAMPLE  
 DO DRIVE-OPEN  
 OF 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  
 OC TRIAXIAL CONSOLIDATED UNDRAINED  
 Q TRIAXIAL UNDRAINED  
 S TRIAXIAL DRAINED  
 W WET UNIT WEIGHT  
 K PERMEABILITY  
 C CONSOLIDATION  
 WL WATER LEVEL IN CASING  
 WT WATER TABLE IN SOIL



## GEOCON

## OFFICE REPORT ON SOIL EXPLORATION

CONTRACT T7625 BORING # 3 AND 4 DATUM GEODETIC CASING BX.  
 BORING DATE MAY 11-13, 1964 REPORT DATE JUNE 2, 1964 COMPILED BY AEL. CHECKED BY F. J. M.  
 SAMPLER HAMMER WT 140 LBS DROP 30 INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN. LBS ENERGY)

## SAMPLE CONDITION

☐ DISTURBED  
☐ FAIR  
☐ GOOD  
☐ LOST

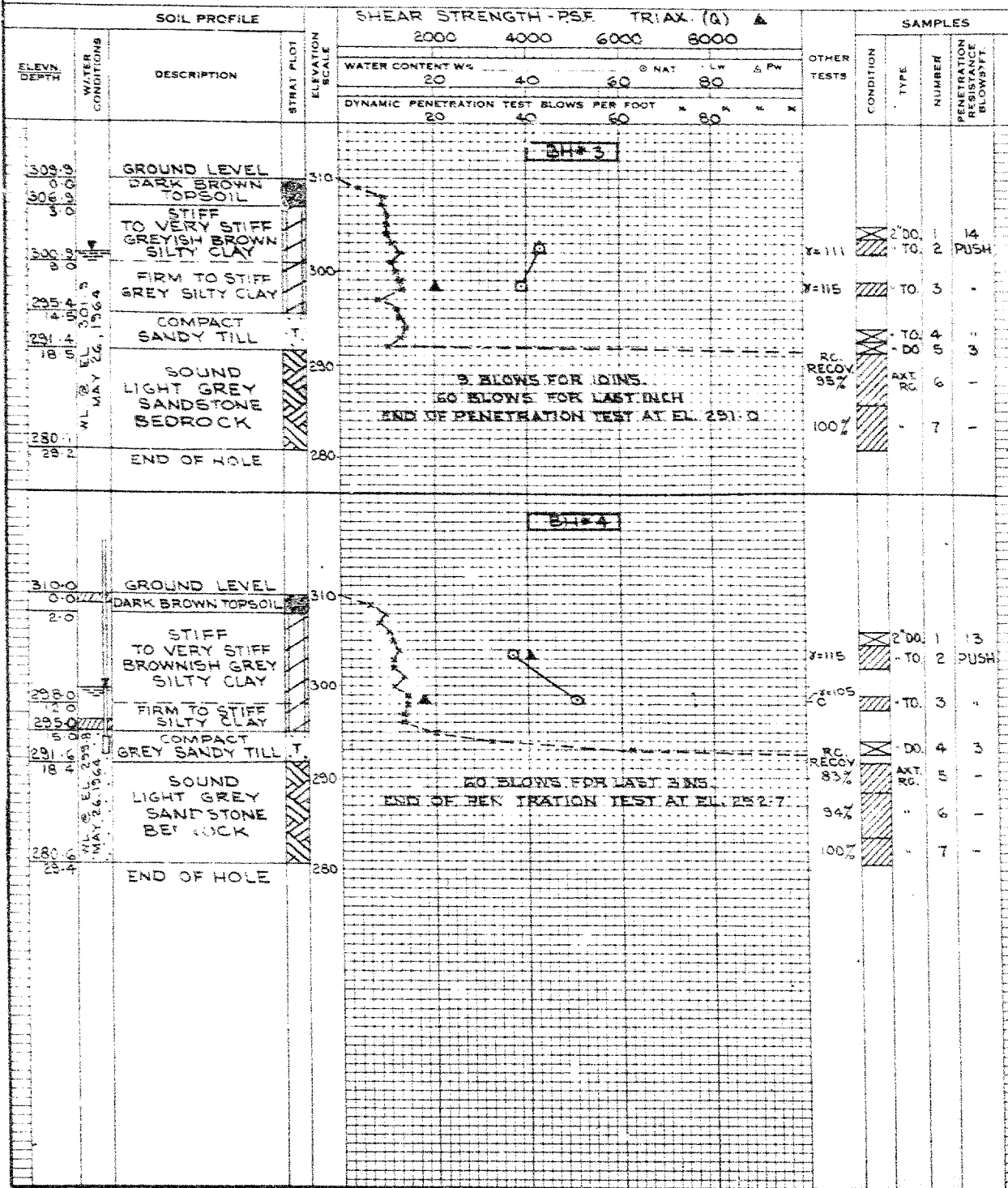
AS - AUGER SAMPLE  
 ST - SLOTTED TUBE  
 WS - WASHED SAMPLE  
 DO - DRIVE-OPEN  
 DF - DRIVE-FOOT VALVE  
 CS - CHUNK SAMPLE

## 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  
 CC - TRIAXIAL CONSOLIDATED UNDRAINED  
 Q - TRIAXIAL UNDRAINED  
 S - TRIAXIAL DRAINED  
 W - WET UNIT WEIGHT  
 K - PERMEABILITY  
 C - CONSOLIDATION  
 WL - WATER LEVEL IN CASING  
 WT - WATER TABLE IN SOIL






CONTRACT T7625 BORING # 5 AND 6 DATUM GEODETIC  
BORING DATE MAY 9 - 26, 1964 REPORT DATE JUNE 2, 1964 CASING  
SAMPLER HAMMER WT 140 LBS DROP 30 INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN. LBS. ENERGY) CHECKED BY E. J. H.

### SAMPLE CONDITION

## SAMPLE TYPES

### ABBREVIATIONS

	DISTURBED
	FAIR
	GOOD
	LOST

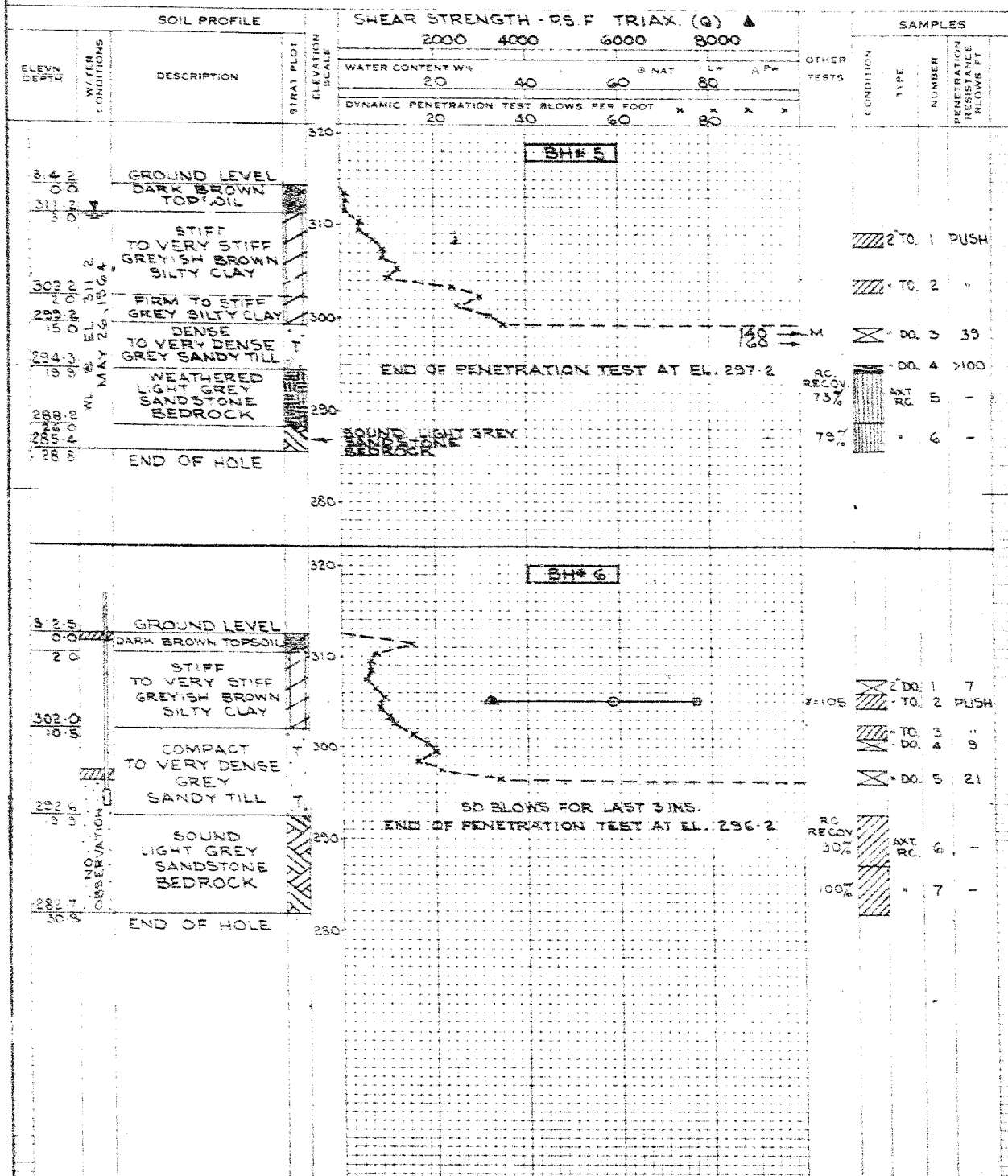
AS ALGER SAMPLE  
ST SLOTTED TUBE  
WS WASHED SAMPLE  
DO DRIVE-OPEN  
DF DRIVE-FOOT VALVE  
CS CRUNK SAMPLE

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

- V - IN-SITU VANE TEST
- M - MECHANICAL ANAL
- U - UNCONFINED COM
- QC - TRIAXIAL CONSOL
- Q - TRIAXIAL UNDRIN
- S - TRIAXIAL UNRAINED

### ABBREVIATIONS

S. WET UNIT WEIGHT  
 K. PERMEABILITY  
 C. CONSOLIDATION  
 WED  
 WL. WATER LEVEL IN CASING  
 WT. WATER TABLE IN SOIL

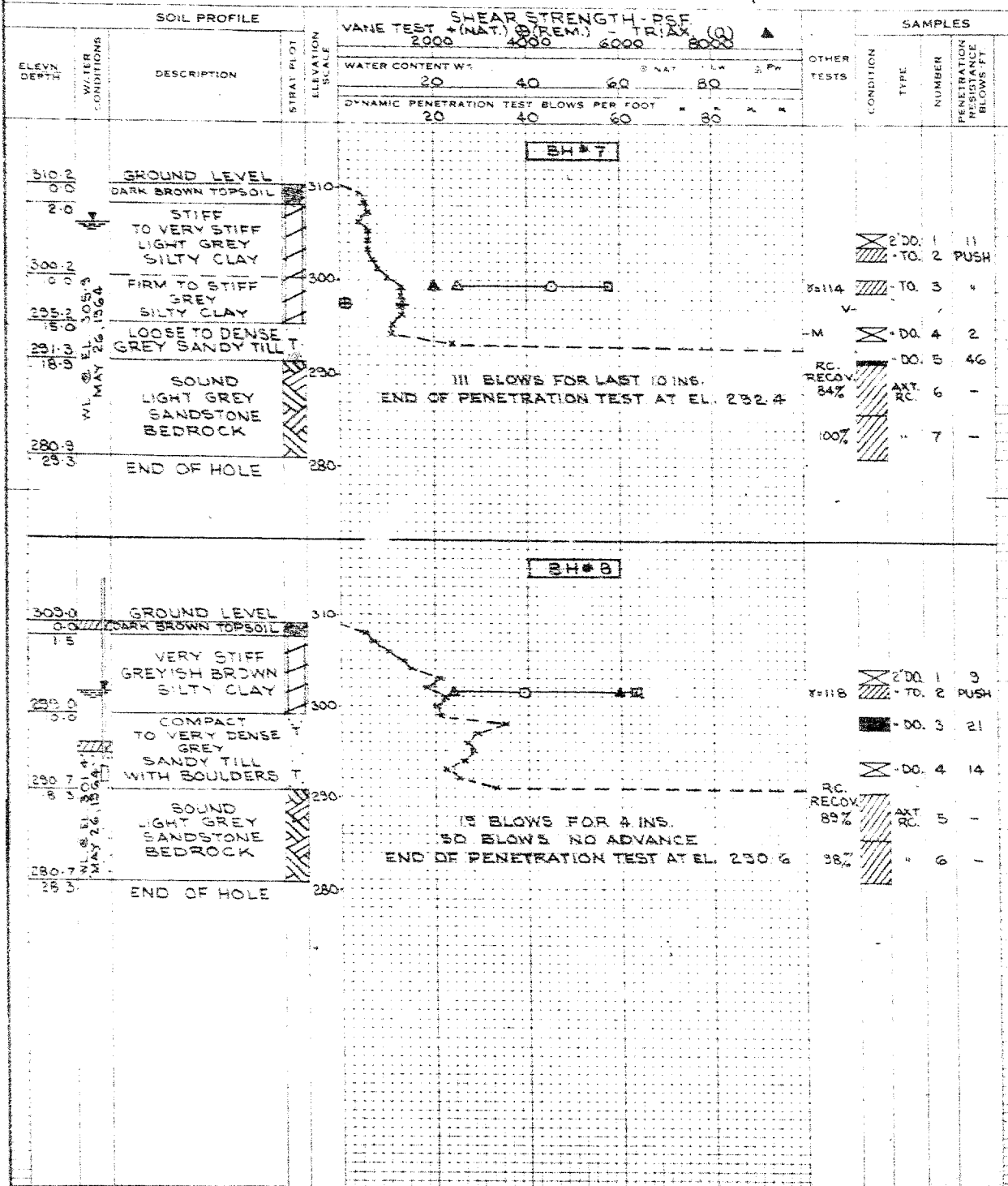


## GEOCON

## OFFICE REPORT ON SOIL EXPLORATION

CONTRACT T7625 BORING = 7 AND 8 DATUM GEODETIC CASING BX & AX.  
 BORING DATE MAY 21-25, 1964 REPORT DATE JUNE 2, 1964 COMPILED BY AEL. CHECKED BY F.J.M.  
 SAMPLER HAMMER WT 140 LBS DROP 30 INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN. LBS ENERGY)

SAMPLE CONDITION		SAMPLE TYPES		ABBREVIATIONS	
DISTURBED	AS AUGER SAMPLE	FS FOIL SAMPLE	V IN SITU VANE TEST	W UNIT WEIGHT	
FAIR	ST SLOTTED TUBE	SO SLEEVE-OPEN	M MECHANICAL ANALYSIS	K PERMEABILITY	
GOOD	WS WASHED SAMPLE	SF SLEEVE-FOOT VALVE	U UNCONFINED COMPRESSION	C CONSOLIDATION	
LOST	DO DRIVE-OPEN	TO THIN WALLED OPEN	GC TRIAXIAL CONSOLIDATED UNDRAINED		
	DF DRIVE-FOOT VALVE	RC ROCK CORE	CU TRIAXIAL UNDRAINED	WL WATER LEVEL IN CASING	
	CS CHUNK SAMPLE		S TRIAXIAL DRAINED	WT WATER TABLE IN SOIL	



## GEOCON

## OFFICE REPORT ON SOIL EXPLORATION

CONTRACT T7625 BORING # 3 DATUM GEODETIC CASING BX & AX.  
 BORING DATE MAY 20-21, 1964 REPORT DATE JUNE 2, 1964 COMPILED BY AEL. CHECKED BY F. H. M.  
 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

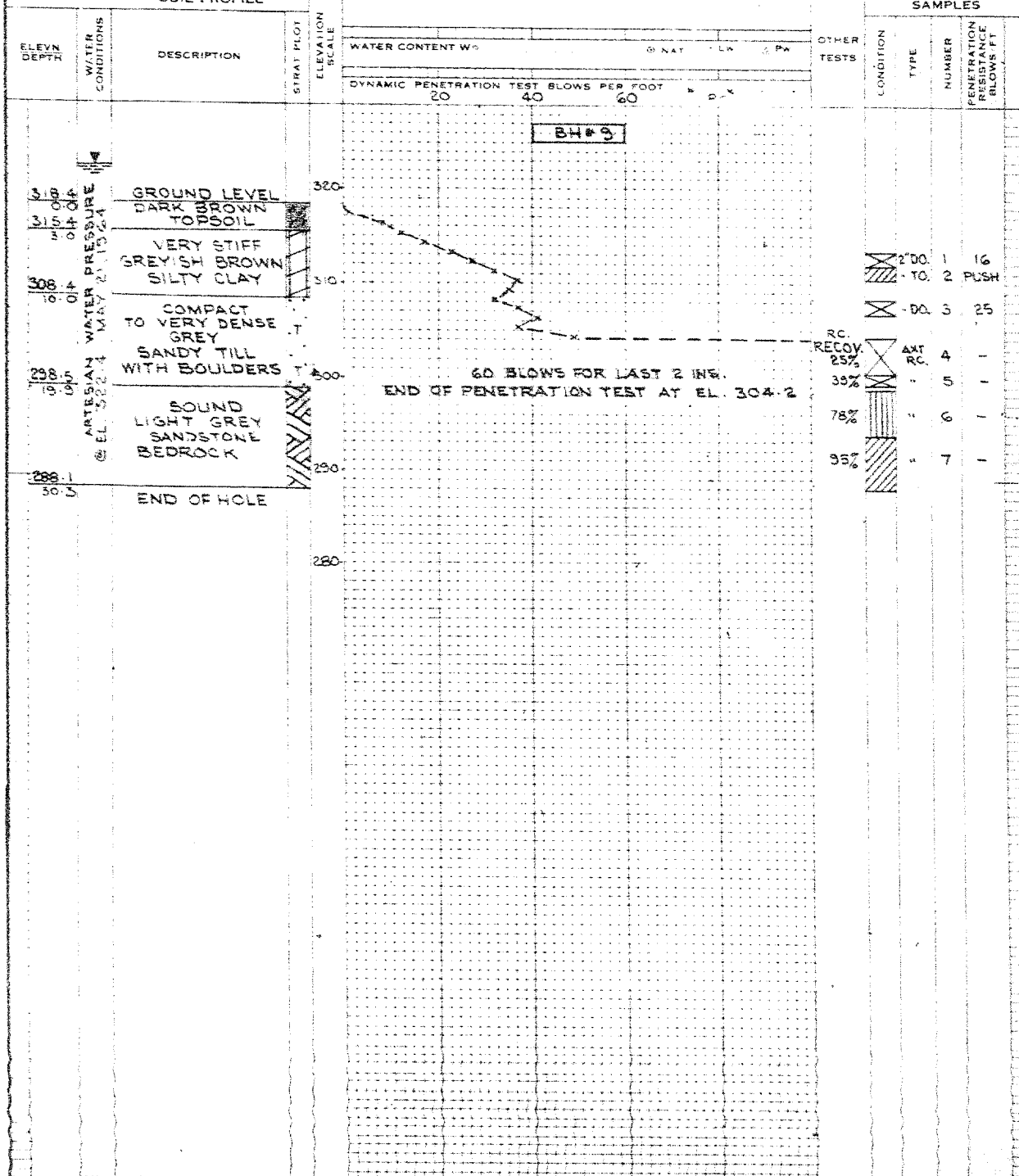
AS AUGER SAMPLE  
 ST SLOTTED TUBE  
 WS WASHED SAMPLE  
 DO DRIVE-OPEN  
 DF DRIVE-FOOT VALVE  
 CS CHUNK SAMPLE

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  
 CU TRIAXIAL CONSOLIDATED UNDRAINED  
 U TRIAXIAL UNDRAINED  
 S TRIAXIAL DRAINED  
 WET UNIT WEIGHT  
 K PERMEABILITY  
 C CONSOLIDATION  
 WL WATER LEVEL IN CASING  
 WT WATER TABLE IN SOIL

## SOIL PROFILE



APPENDIX II

FIGURES - LABORATORY TESTING

# GRAIN SIZE DISTRIBUTION

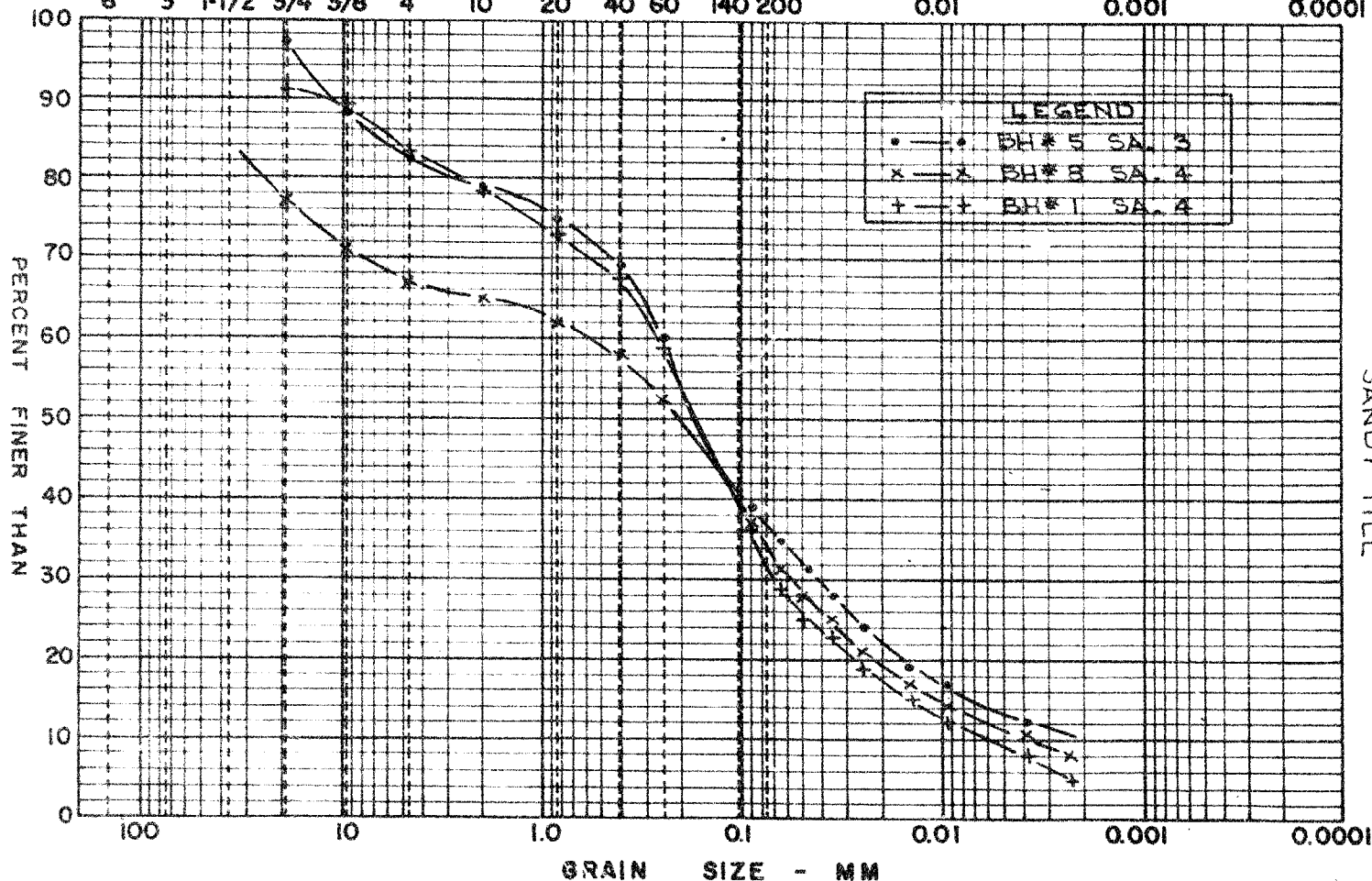
APPENDIX II  
FIGURE 1  
PROJECT T7625

GEOCON

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

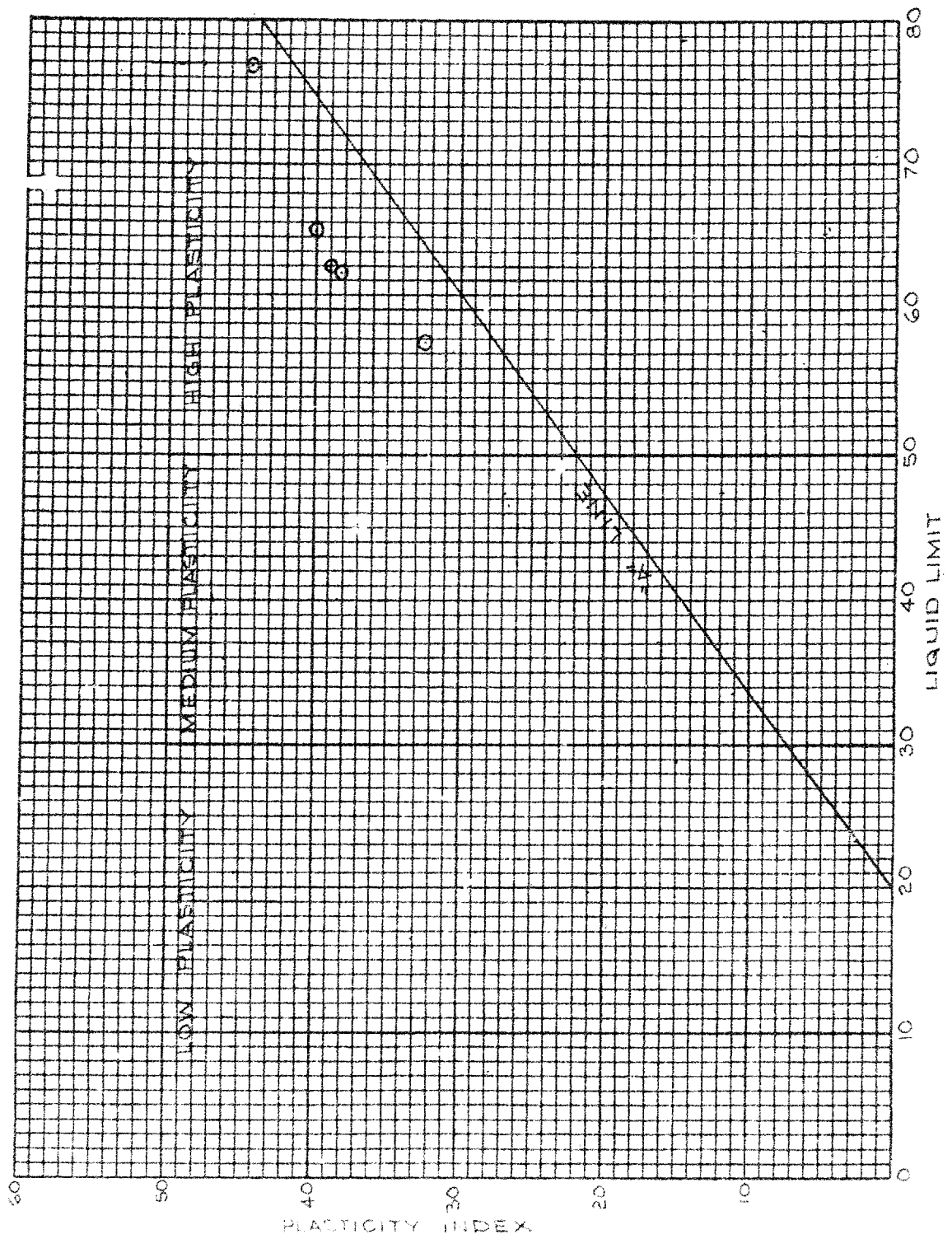


M.I.T. GRAIN SIZE SCALE



# PLASTICITY CHART

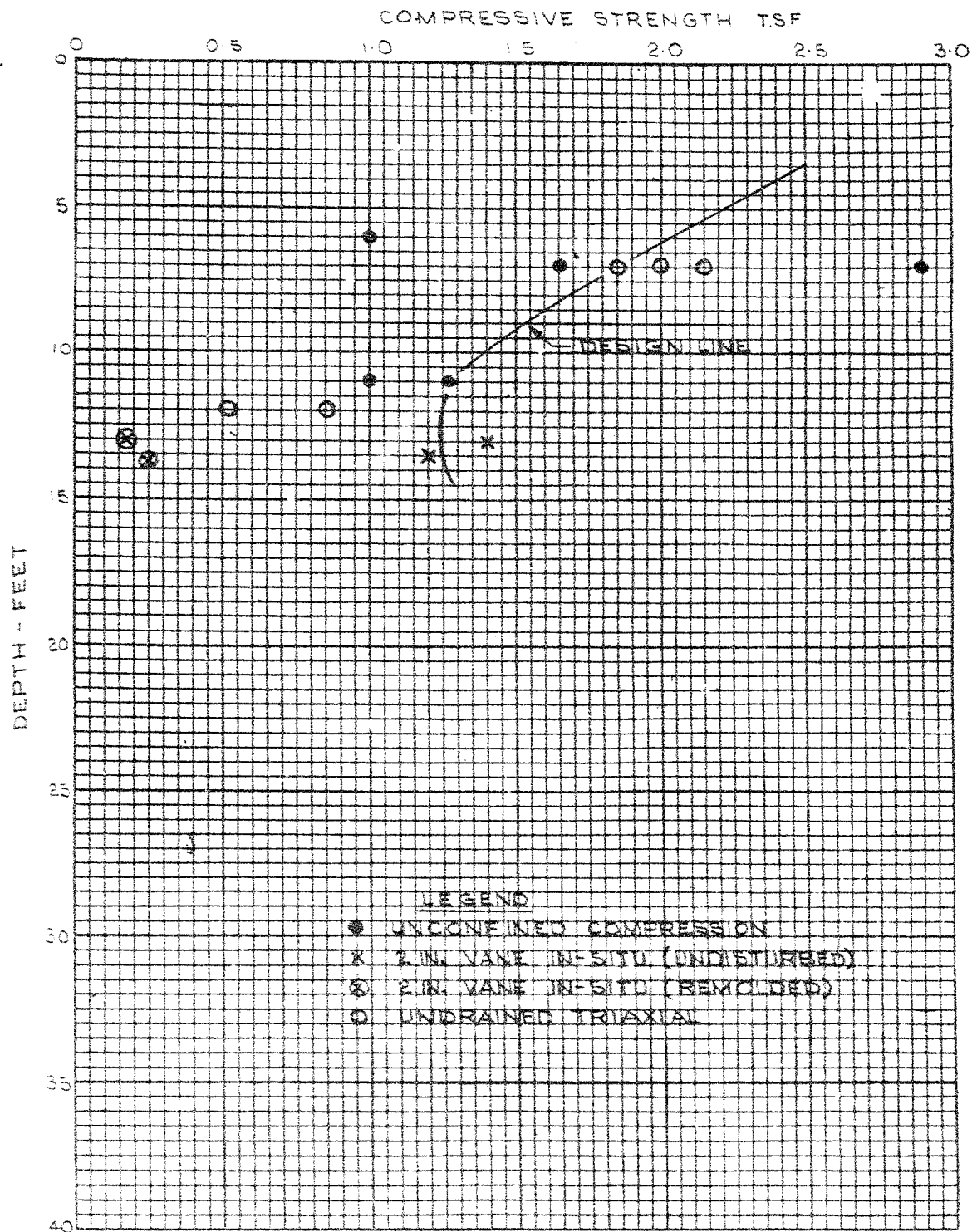
APPENDIX II  
FIGURE 2  
PROJECT T7625



GEOCON

# COMPRESSIVE STRENGTH vs. DEPTH

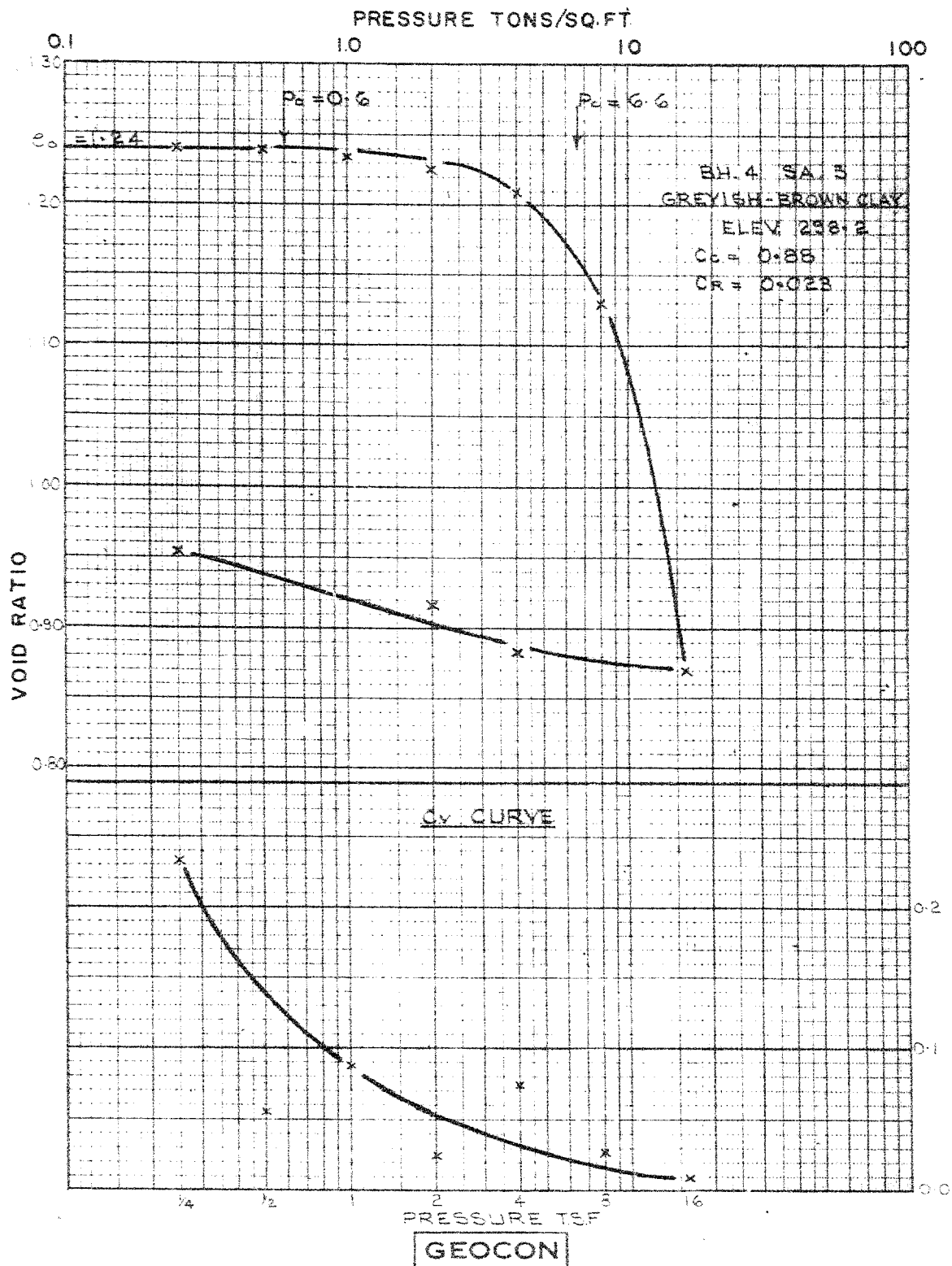
APPENDIX II  
FIGURE 3  
PROJECT T7625



GEOCON

# VOID RATIO-PRESSURE DIAGRAM CONSOLIDATION TEST

APPENDIX II  
FIGURE 4  
PROJECT T7625



*Letter of authorization, please  
AGS*

DEPARTMENT OF HIGHWAYS ONTARIO

MEMORANDUM

To: Mr. A. G. Stermac,  
Principal Foundation Eng.,  
Room 107, Lab. Bldg.

From: Bridge Division,  
Downsview, Ontario.

DATE: April 29, 1964.

OUR FILE REF.

IN REPLY TO:

SUBJECT: W.P. 907-64  
Bridge Site 3-37  
C.P.R. Subway  
3.2 Miles W. of Ottawa west limits  
Hwy. 15 and 7  
Dist. 9.

Would you kindly arrange to have a foundation investigation conducted at the above location. I have enclosed one copy of the site plan number E-4257-1 with the probable footing locations marked in red.

Because of the great skew angle between Hwy 15 and 7 and the railway, large retaining walls along the highway are anticipated as shown on the site plan, therefore would you also include them in your investigation.

There is also a possibility that the C.P.R. track maybe supported by a trestle during construction of the subway sub-structure, therefore would you kindly investigate the load carrying capacity of timber or steel piles for a trestle.

Enclosed please find a scheme of taking a C.P.R. Railway Detour to the west of the existing structure as shown on plate 13 and 14. Would you kindly investigate the stability of the approximate 13 foot fill along the detour.

*Apwatt*

APW/sp  
c.c. N.D. Smith  
R. Fitzgibbon

A. P. Watt,  
Bridge Location Engineer.

AS SOON AS POSSIBLE

JOB GIVEN TO "GEOCOR"  
MAY 8, 1964  
AGS

Materials and Research Division

May 11, 1964

Geoscon, Limited,  
Consulting Engineers,  
14 Hays Road,  
Berkdale, Ontario.

Attention: Mr. F.J. Heffernan

Re: W.F. 907-64, Hwy. 15 & 7, C.P.R. Subway,  
3.2 Mi. W. of Ottawa West Limits,  
District #9, Ottawa.

Dear Sir:

Please consider this your authority to carry out a foundation investigation at the above site. Plans and profiles were provided to your representative on May 8, 1964.

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

Ten copies of the completed foundation report, with one additional copy of each subsoil profile, should be submitted to the Foundation Section prior to June 24, 1964. Previous requirements as to preliminary borehole information and laboratory testing program, should be followed.

Because the drawings accompanying the foundation reports, showing the location of borings, the inferred subsoil conditions, etc., are to become contract drawings, you are requested to prepare them in accordance with the D.M.C. standards. To enable you to do this, we are supplying you with sample drawings with all the necessary explanations, together with linen sheets for your drawings. You are also requested to provide the D.M.C. with Cronaflex copies of the drawings.

Charges for the work performed will be in accordance with your Schedule of Rates, dated March 4, 1960, and invoice to be addressed to the attention of the undersigned.

NDs/adeP

Yours very truly,

cc: Messrs. S. McCombie

L.E. Walker

J. Ford

J.E. Graspier

H. Konings

H.D. Smith (2)

A. Butke

Foundations Office

MATERIALS & RESEARCH ENGINEER

(En. Files (2))

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

Foundation Section,  
Materials & Research Div.,  
Room 107, Lab. Bldg.

Attention: Mr. S. McCosbie

June 24, 1964

FOUNDATION INVESTIGATION REPORT BY:  
Geocon, Limited, Consulting Engineers.  
Proposed C.P.R. Underpass, Hwy. #15 & #7,  
District #9, Ottawa. -- M. P. 907-64

Attached, we are sending you the above-mentioned  
report submitted by Geocon, Limited of Toronto.

We have reviewed the report and have found the  
factual information well presented. We are also in agreement  
with the conclusions and recommendations contained therein.

We believe the information provided by the Consultant  
will be adequate for your future design work. However, should  
additional information be required, please do not hesitate to  
contact our Office.

KYL/MdeF  
Attach.

cc: Messrs. A. M. Toye (2)  
H. A. Tregaskes  
H. D. McMillan  
J. Ford  
L. E. Walker  
J. E. Gruspier  
A. Watt

*KYL*  
A. G. Sternac,  
PRINCIPAL FOUNDATION ENGINEER

Foundations Office ✓  
Gen. Files

Discussed with A.T. Watt

18-1 slope for C.P.R. detour. O.K.

*KYL* 27/7/64

MEMORANDUM

To: Mr. A. G. Stermac,  
Principal Foundation Engineer,  
Room 107, Lab. Bldg.

FROM: Bridge Division,  
Downsview, Ontario.

DATE: December 21, 1964.

OUR FILE REF.

IN REPLY TO

SUBJECT: W.P. 907-64 Site 3-37  
C.P.R. Subway  
3.2 Miles West of Ottawa  
Hwy. 7 and 15 - Dist. 9

Enclosed please find one copy of the preliminary plan D-5526-P9 and D-5526-P10 for the above noted structure.

Would you kindly review the bridge foundations proposed and inform me if they are satisfactory.



APW/sp

A. P. Watt,  
Regional Bridge Location Engineer.

cc: Foundations Office (Enr. 110)

Mr. S. McCombie,  
Bridge Planning Engr.,  
Bridge Division.

Materials & Testing Div.,  
Foundation Section,  
Rm. 107, Lab. Bldg.

Attention: Mr. A. P. Watt,  
Regional Bridge Location Engr.

December 29, 1964

C.P.R. Subway at Hwy. 7 & 15,  
3.2 Miles West of Ottawa,  
District #9 -- W.P. 907-64.  
(Report by Geocon, Limited,  
Consulting Engineers).

We have reviewed the Preliminary Drawings D-5526-P9 and P-10 for the above-mentioned structure, and submit the following comments:

We would like to point out that the Drawing shows 1½:1 side slopes for the Railway approach fill embankment, whereas the Foundation Report by Geocon recommends 2:1 side slopes for all the approach fill embankments. The Foundation Report also indicates that the existing side slopes of the approach embankments of the Railway are 2:1.

MD/MdeF

cc: Foundations Office  
Gen. Files

*M. Duvda*  
for A. G. Sternac,  
PRINCIPAL FOUNDATION ENGINEER



DE LEUW, CATHER & COMPANY  
OF CANADA LIMITED

CONSULTING ENGINEERS

TORONTO OTTAWA LONDON ST. JOHN'S

SUITE 206  
2277 RIVERSIDE DRIVE  
BILLINGS BRIDGE PLAZA  
OTTAWA 8, ONTARIO  
TELEPHONE 733-4160

Our Ref. C-277

January 4, 1965

Mr. F.I. Hewson,  
Consultant Liaison Engineer,  
Bridge Division,  
Department of Highways of Ontario,  
Downsview, Ontario.

Re: C.P.R. Subway at Hwy. 7 & 15  
3.2 Miles West of Ottawa -  
District #9 W.P. No. 907-64

Dear Sir:

With reference to Mr. Stermac's letter of December 29th,  $1\frac{1}{2}$ :1 side slopes for the Railway approach fill embankment was used adjacent to the structure in the preliminary design to reduce the extreme length of wing walls required for the closed abutment schemes. This was based on the fact that actual topo records showed an existing slope of about 1.6:1 and not 2:1 as shown in the Soils Report.

However, for the 4 span open abutment schemes a 2:1 slope was used to provide sufficient tail down span without uplift. As this type was finally adopted, we have used 2:1 slopes throughout in the final drawings modifying the stub abutments to suit.

In view of this no modifications to the final design drawings are required.

Yours very truly,

DE LEUW, CATHER & COMPANY OF CANADA LIMITED



Leon J. Marshall, P.Eng.  
Chief Bridge Engineer

LJM:ea

cc A.G. Stermac ✓

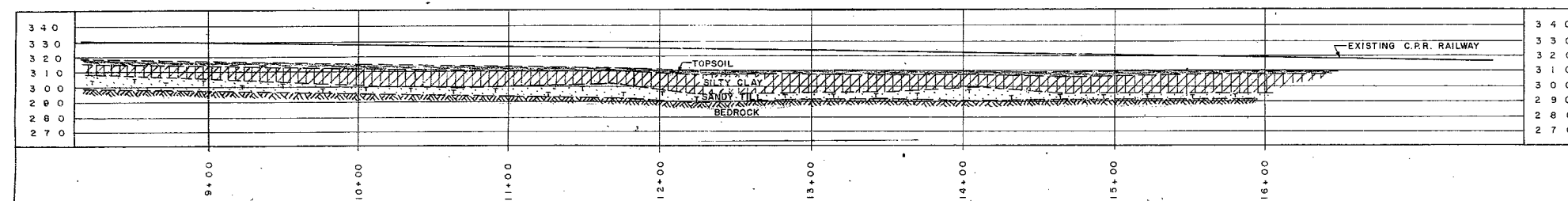
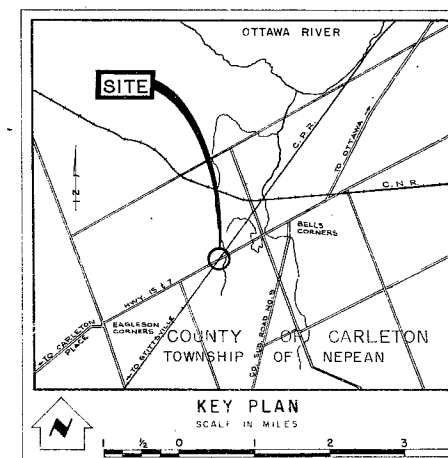
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W.P. #907-64

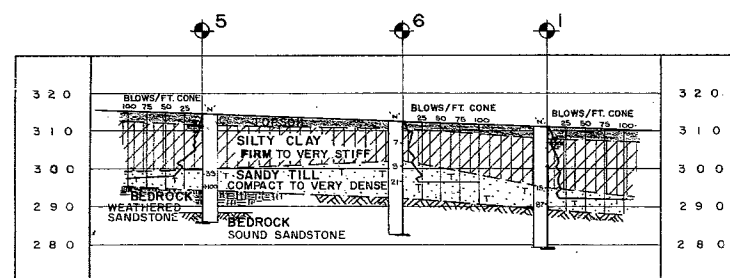
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C.P.R

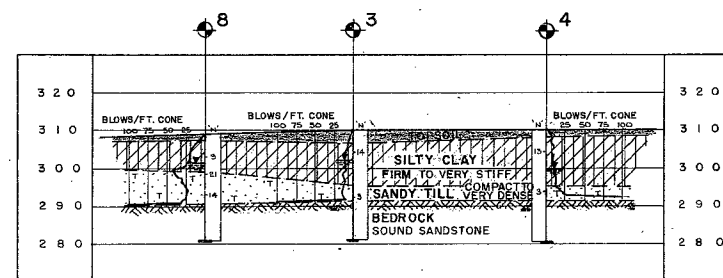
UNDERPASS



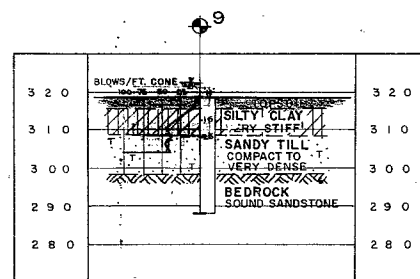
PROFILE ALONG C.P.R. TRACK



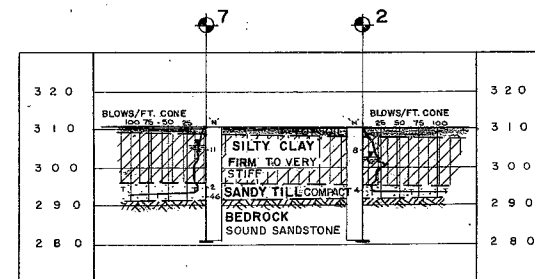
A- $\hat{A}$







B-B



C-C



D-D

LEGEND			
	Bare Hole		
	Cone Penetration Hole		
	Bore & Cone Penetration Hole		
	Water Levels established at time of field investigation. MAY, 1964		

- NOTE -

The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence and may be subject to considerable error.

[illegible]

DEPARTMENT OF HIGHWAYS - ONTARIO  
MATERIALS & RESEARCH DIVISION - FOUNDATION SECTION

PROPOSED HIGHWAY 7 & 15  
C.P.R. UNDERPASS

KING'S HIGHWAY NO. 7 & 15 DIST. NO. 9  
CO. CARLETON  
TWP. NEPEAN LOT 7 & 35 CON. II & V

BOREHOLE	LOCATIONS	&	SOIL	STRATA
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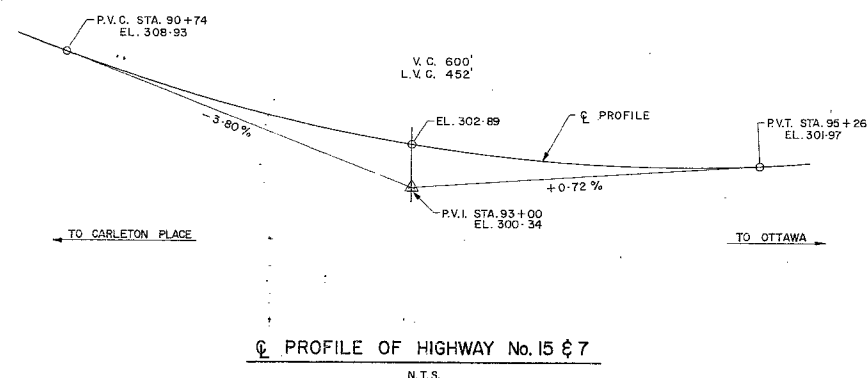
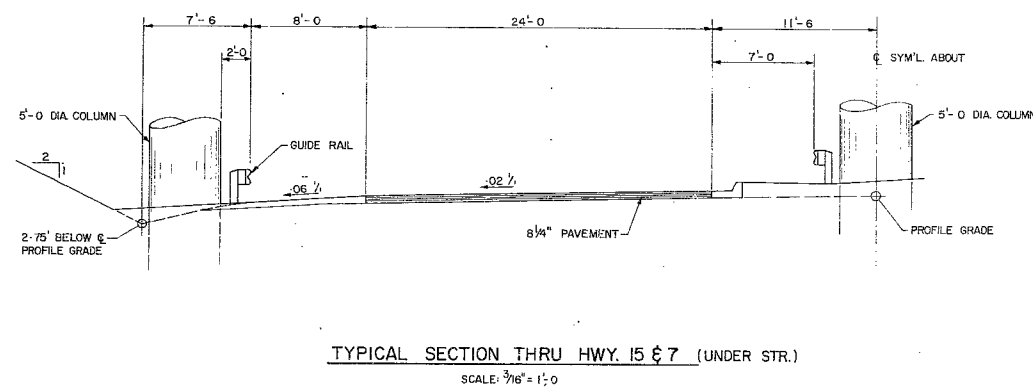
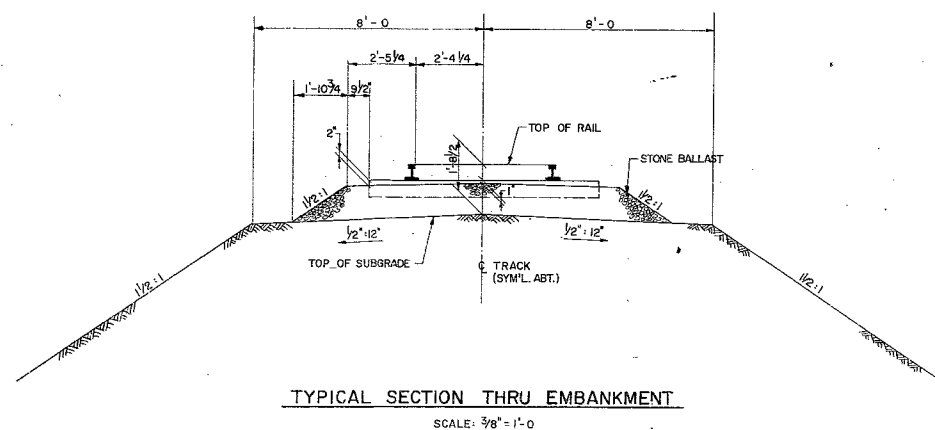
SUBWD	BTD.	CHECKED FJH.	W.P. NO.	907 - 64	M BR DRAWING NO
DRAWN	AEL.	CHECKED BTD.	JOB NO.		
DATE	JUNE 16, 1964		SITE NO	3 - 37	BRIDGE DRAWING NO
APPROVED	<i>[Signature]</i>		CONT NO		

**GEOCON LTD**

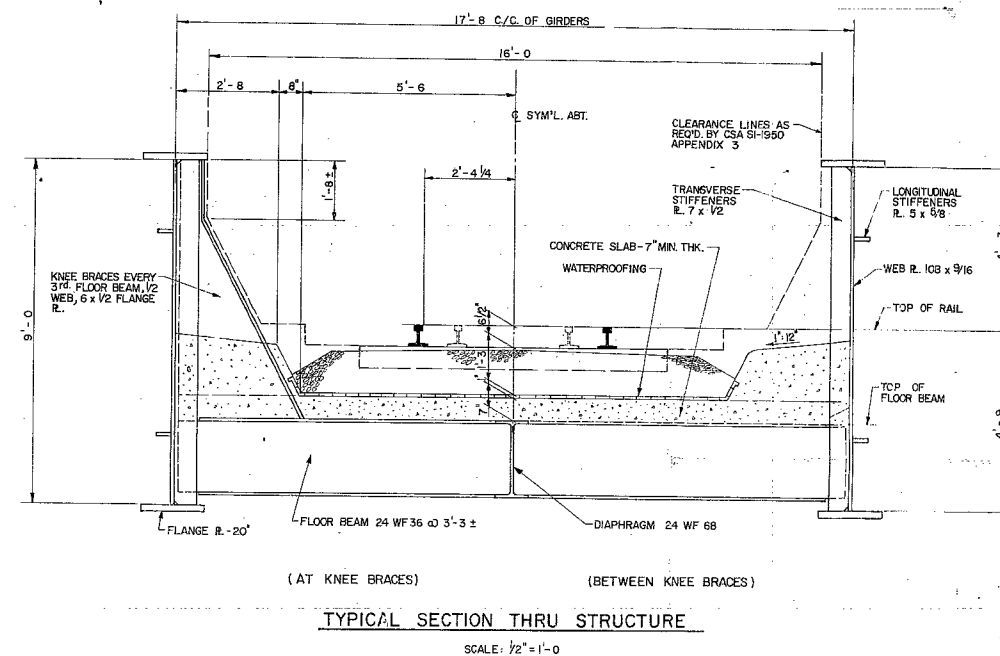
DRAWING No.

T 7625-1





NOTE:-PROFILE GRADE OF HWY. No. 15 & 7 IS 0.69' BELOW FINISHED GRADE AT THE CENTER LINE OF THE MEDIAN IN THE VICINITY OF THE STRUCTURE.



	B.H. 1	B.H. 2	B.H. 3	B.H. 4	B.H. 5	B.H. 6	B.H. 7	B.H. 8
310	 TOPSOIL	 TOPSOIL	 TOPSOIL	 TOPSOIL	 TOPSOIL	 TOPSOIL	 TOPSOIL	 TOPSOIL
300	 VERY STIFF TO FIRM SILTY CLAY	 VERY STIFF TO FIRM SILTY CLAY	 VERY STIFF TO FIRM SILTY CLAY	 VERY STIFF TO FIRM SILTY CLAY	 VERY STIFF TO FIRM SILTY CLAY	 VERY STIFF TO FIRM SILTY CLAY	 VERY STIFF TO FIRM SILTY CLAY	 VERY STIFF TO FIRM SILTY CLAY
290	 SANDY TILL	 SANDY TILL	 SANDY TILL	 SANDY TILL	 SANDY TILL	 SANDY TILL	 SANDY TILL	 SANDY TILL
280	 BEDROCK	 BEDROCK	 BEDROCK	 BEDROCK	 BEDROCK	 BEDROCK	 BEDROCK	 BEDROCK

**BOREHOLE LOG**  
VERT. SCALE: 1" = 10'  
(INFORMATION FROM SOILS REPORT No. BA1863  
PREPARED BY GEOCON LTD.)

REVISIONS				
1, 12, 64	A.G.Y.	THIS DRAWING TRACED FROM ORIGINAL DRAWING No. D5526-P10, DATED OCT. 64 WITH GENERAL REVISIONS FOR APPROVAL.		
DATE	BY	CHECKED BY		
DE LEUW CATHAR & COMPANY OF CANADA LIMITED CONSULTING PROFESSIONAL ENGINEERS				
<u>DEPARTMENT OF HIGHWAYS ONTARIO</u> BRIDGE DIVISION				
C. P. R. SUBWAY 3.2 MILES WEST OF OTTAWA				
KING'S HIGHWAY No. 15 & 7		DIST. No. 9		
CO. OF CARLETON		7	II OTTAWA FRONT	
TWP. OF NEPEAN		LOT 35	CON. V. RIDEAU FRONT	
PRELIMINARY BRIDGE PLAN - 2 of 2				
APPROVED		BRIDGE ENGINEER	SITE No.	W.P. No.
			3-37	907-64
DESIGN	G.S.S.	CHECK	L.J.M.	
DRAWING	P.T.	CHECK	G.S.S.	
DATE	DEC. 64	LOADING	E 70	
			DRAWING No.	D5526 - P 10