



ONTARIO

DEPARTMENT OF HIGHWAYS

Memo to Mr. A. M. Teye, Date December 10, 1959.
Bridge Engineer. Subject FOUNDATION REPORT by
From Materials & Research Section. Geocon, Limited.

Attention: Mr. S. McCombie.

Re: Stoney Creek Traffic Circle
Overpass and C.N.R. Overhead
W.P. 10-57 -- District #4.

Enclosed herewith is a report on a foundation investigation carried out by Geocon at the above site.

Geocon have incorporated some alternatives with respect to minimizing differential settlement between the structure and backfill. Their comments are based upon a discussion which we had with Dr. Casagrande while in Boston, in connection with the Homer Skyway.

When you have had an opportunity to review the comments contained in this report, we would be pleased to discuss any points with you in regard to either the embankment problem or means of footing support.

LGS/MdeF
Encl.

L. G. Soderman,
PRINCIPAL SOILS & FOUNDATIONS ENGINEER

cc: Messrs. A. M. Teye (2)
H. A. Tregaskes
D. G. Ramsay
D. W. Farren
R. E. Richardson
P. Weber

Foundation Section
Gen. Files.

GEOCON LTD

HEAD OFFICE

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DISTRICT OFFICES

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Rexdale, Ontario,
December 3rd, 1959.

59-E-231

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

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

Re: Soil Conditions and Foundations,
Stoney Creek Traffic Circle Overpass and
C.N.R. Overhead,
W.P. 10-57 - District No. 4.

Dear Sirs:

This letter accompanies our detailed report covering the
above soil investigation.

We find that the strata and strength characteristics previously defined in preliminary site investigations have been confirmed by this work.

In general the proposed overpass structure may be satisfactorily founded on displacement pile foundations as discussed in the report. Several alternative methods of minimizing differential movement at the abutments are further discussed. The specific choice between these alternatives is dependent upon the degree of maintenance which can be provided to the structure.

We believe that this report confirms all the information necessary for final structural design. If we can be of any further service on this project, we would be pleased if you would call us.

Yours very truly,

GEOCON LTD

V. Milligan
per AB

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

VM/dw
S6989

S6989

REPORT

TO

DEPARTMENT OF HIGHWAYS, ONTARIO

ON

SOIL CONDITIONS AND FOUNDATIONS

STONE CREEK TRAFFIC CIRCLE OVERPASS AND C.N.R. OVERHEAD

WENTWORTH COUNTY

ONTARIO

Distribution:

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

2 copies - Geocon Ltd,
Toronto, Ontario.

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	<u>Page</u>
Introduction	1
Summarized Soil Conditions	1
Discussion	2
Approach Embankments	2
Structural Foundations	3
Conclusions and Recommendations	5
Personnel	5
Appendix I	
Office Reports on Soil Exploration	
Appendix II	
Drawings S6989-1 Boring Plan and Soil Stratigraphy	
S6989-2 Alternative Abutment Foundations	

INTRODUCTION

Geocon Ltd has been retained by the Department of Highways, Ontario by letter of authorization dated September 23rd, 1959 and under the terms of our proposal dated September 29th, 1959 to define the soil conditions beyond the limits investigated for the proposed Queen Elizabeth Way overpass structure at the existing traffic circle in Wentworth County, Hamilton, Ontario. The purpose of the investigation was to extend the scope of the preliminary investigation carried out at the site in April 1958, to determine the depth to bedrock at specific footing locations and to provide detailed confirmation of the strength characteristics previously determined.

SUMMARIZED SOIL CONDITIONS

The results of this further investigation confirmed that the site is covered by from 25 to 45 feet of clay, the upper 4 to 16 feet of which have been weathered and desiccated to a stiff to hard consistency. At the proposed location of the east abutment, the clay stratum was found to be about 30 feet in thickness and to be of firm to very stiff consistency throughout. The clay stratum is underlain by about 10 feet of hard reddish brown clay till followed by sound shale bedrock. At the proposed location of the west abutment about 3 feet of loose brown sandy silt was encountered. This is underlain by about 40 feet of clay, the upper 12 feet of which is of soft to firm consistency becoming generally stiff with increasing depth. At this location, the clay stratum follows approximately 40 feet of hard clay till, underlain by shale bedrock. Between the abutment locations discussed above, the depth to bedrock generally ranges between 60 to 80 feet, with corresponding thicknesses of the hard till stratum of 20 to 40 feet respectively.

Detailed descriptions of individual soil strata have been presented in our report S6623, dated May 5th, 1958 and in report S6935, dated May 22nd, 1959. The detailed Office Reports on Soil Exploration which confirm the characteristics previously described are given in Appendix I of this report.

DISCUSSION

It is now understood that the overpass structure will be a continuous trestle placed on piled foundations and with abutments at station 68+60 and station 84+93.50 respectively. The approach embankment at the west abutment will be approximately 20 feet in height in accordance with previous recommendations. At the east abutment the approach embankment will be approximately 45 feet in height. The embankments will be constructed of compacted earth fill with the areas adjacent to the abutment locations being of select granular fill. The side slopes of the embankment will be at 2 horizontal to 1 vertical. It is further understood that in the initial structural design the end spans at each abutment location are continuous.

Approach Embankments

The stability of both approach embankments has been examined and it is computed that the factor of safety is greater than 1.3 for the heights proposed at the respective locations. Settlement computations show that under the full height of embankment total ultimate settlements may be expected to occur of the order of 3 to 5 inches for the west and east embankments respectively. This settlement is further computed to take place over a period of approximately 5 to 10 years. This pattern of settlement beneath the approach embankments is pertinent to the design of the structural foundations in this area; this is discussed below.

Structural Foundations

In view of the initial design necessity to maintain continuity and therefore to reduce structural settlement to a minimum, it is preferable to found the structure generally on piled foundations. It is considered that, because of the hard consistency and thickness of the underlying clay till, displacement piles may be adequately founded within this stratum. It is recommended that the intermediate structural piers or pile bents be founded on 12 inch diameter concrete-filled steel tube piles driven to practical refusal within the till. It is estimated that for a minimum steel pipe section of No. 7 gauge and a driving energy of 15,000 foot pounds per blow, a penetration of the order of 5 to 10 feet into the till stratum can be expected. On the basis of the measured compressive strength of the till the permissible design load for individual piles is computed to be 40 tons.

If the end spans of the structure are continuous, as initially proposed, then it would be necessary to found the abutments on piled foundations; however, in this instance, differential settlement between the approach embankment and the abutment of about 6 inches could then be expected. The load on the abutment piles would also be increased due to negative skin friction by consolidation of the clay stratum. Taking into account the computed time rate of consolidation, it is estimated that a major part of the expected differential movement would take place within a year after completion of the structure. To eliminate this undesirable factor, it is suggested that the end spans be designed to be simply supported and that the abutments be founded within the approach embankments on spread footings. This would ensure that settlements of the embankments and of the abutments would take place at the same rate. The end spans would therefore have to be designed for a

Structural Foundations (continued)

probable average change in grade of 4 inches between the piled pier foundations and the spread abutment foundation or a change of one-half percent.

To utilize this method of founding the abutments, highly compacted select granular fill should be placed at the abutment locations. If this be done, an allowable bearing pressure of 1 ton per square foot may be used in design. The suggested general pattern at the proposed abutment locations is shown on Drawing S6989-2.

The alternative to the procedure outlined above, is to provide periodic maintenance to the road surface at the ends of the structure. In this case the design load on individual piles should be reduced to 30 tons to allow for negative skin friction effects. An abrupt step at the abutment ballast wall, due to settlement of the embankment, may be minimized by constructing a heavily reinforced concrete road slab about 15 feet in length, supported at one end by the abutment ballast wall and at the other end by a shallow strip footing within the embankment.

For both the alternatives, it is recommended that the embankment fill be placed before construction of the overpass structure is commenced in order to minimize the effects discussed above.

For protection against frost action all structural foundations should be provided with at least 4 feet of earth cover.

CONCLUSIONS AND RECOMMENDATIONS

5.

1. The site is generally covered by from 25 to 45 feet of clay following hard clay till extending to bedrock. The upper 10 to 16 feet of the clay are desiccated and this crust is underlain by about 10 feet of lightly overconsolidated clay. The remainder of the stratum is of stiff to hard consistency.

2. The general soil characteristics as determined in the previous preliminary investigations were confirmed by this investigation.

3. Methods of founding the proposed overpass structure are discussed in the report. It is recommended that 12 inch diameter concrete-filled steel tube piles be used for intermediate piers or pile bents; permissible design loads and driving resistance are discussed.

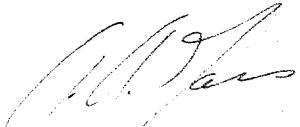
4. The stability of the proposed approach embankments is adequate; however, the effects of consolidation of the clay stratum under the embankments at the ends of the structure could be significant. It is suggested that the end spans be designed to be simply supported to provide a transition in final grade.

5. The alternative to redesign of the end spans is to provide periodic maintenance to the road surface at the ends of the structure. This is discussed in detail.

PERSONNEL

The field work was carried out under the supervision of Mr. R. Sorokoski. This report was written by Mr. A.A. Gass and checked by Mr. V. Milligan.

AAG/dw
S6989



A. A. Gass, P. Eng.

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APPENDIX I

OFFICE REPORTS ON SOIL EXPLORATION

EXPLANATION OF THE FORM "OFFICE REPORT ON SOIL EXPLORATION"

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

ELEVATION AND DEPTH

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

WATER CONDITIONS

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

DESCRIPTION

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

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

STRATIGRAPHIC PLOT

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

ELEVATION SCALE

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

GRAPHS

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

OTHER TESTS

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

SAMPLES

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

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

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OFFICE REPORT ON SOIL EXPLORATION

CONTRACT 54380 BORING # 1 DATUM 255.67 Casing 27
 BORING DATE 25.10.1959 REPORT DATE OCT. 14, 1959 COMPILED BY W. J. A. CHECKED BY
 SAMPLER HAMMER WT. 140 LBS. DROP 32 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. CHUCK 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
 GC. TRIAXIAL CONSOLIDATED QUICK
 Q. TRIAXIAL QUICK
 S. TRIAXIAL SLOW
 X. HYDRAULIC PUSH
 γ. WET UNIT WEIGHT
 K. PERMEABILITY
 C. CONSOLIDATION
 WL. WATER LEVEL IN CASING
 WT. WATER TABLE IN SOIL

SOIL PROFILE

SHEAR STRENGTH IN LBS./SQ. FT.

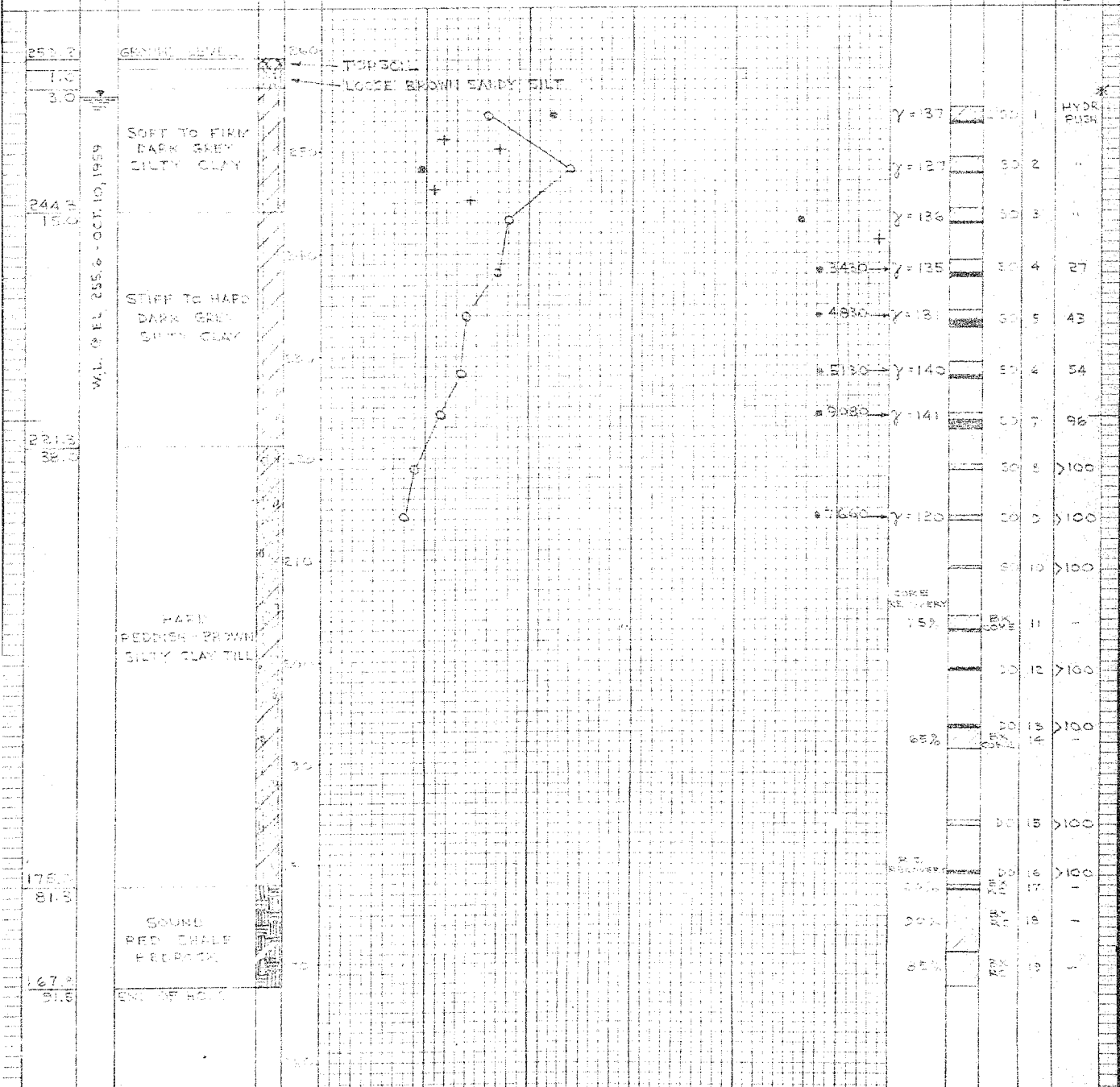
500 1000 1500 2000 2500

WATER CONTENT W% 10 20 30 40 50

DYNAMIC PENETRATION TEST BLOWS PER FOOT

SAMPLES

CONDITION
 TYPE
 NUMBER
 PENETRATION RESISTANCE BLOWS/FT.



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OFFICE REPORT ON SOIL EXPLORATION

CONTRACT 56380 BORING # 2 DATUM GEODETIC CASING BA
 BORING DATE SEPT. 20, 1959 REPORT DATE OCT. 14, 1959 COMPILED BY M. J. A. CHECKED BY J. J.
 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
 * - HYDRAULIC PUSH

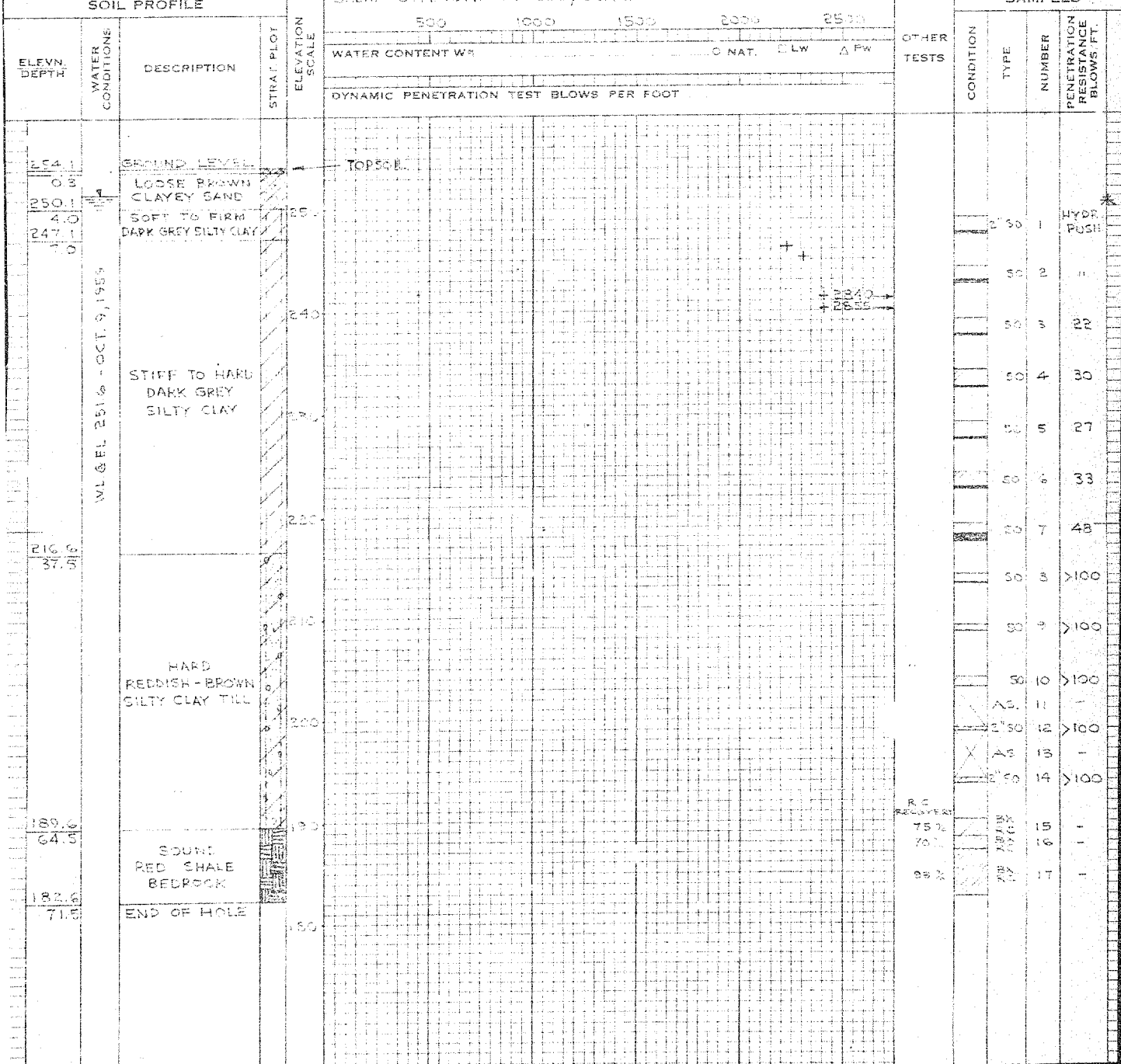
γ - WET UNIT WEIGHT
 K - PERMEABILITY
 C - CONSOLIDATION

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

SOIL PROFILE

SHEAR STRENGTH IN LBS./SQ. FT. - + VANE TESTS

SAMPLES



GEOCON

OFFICE REPORT ON SOIL EXPLORATION

CONTRACT 56280 BORING # 5 DATUM GEODETIC CASING BX
 BORING DATE OCT. 1, 1959 REPORT DATE OCT. 23, 1959 COMPILED BY M.W. & J.A. CHECKED BY
 SAMPLER HAMMER WT. 14.0 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

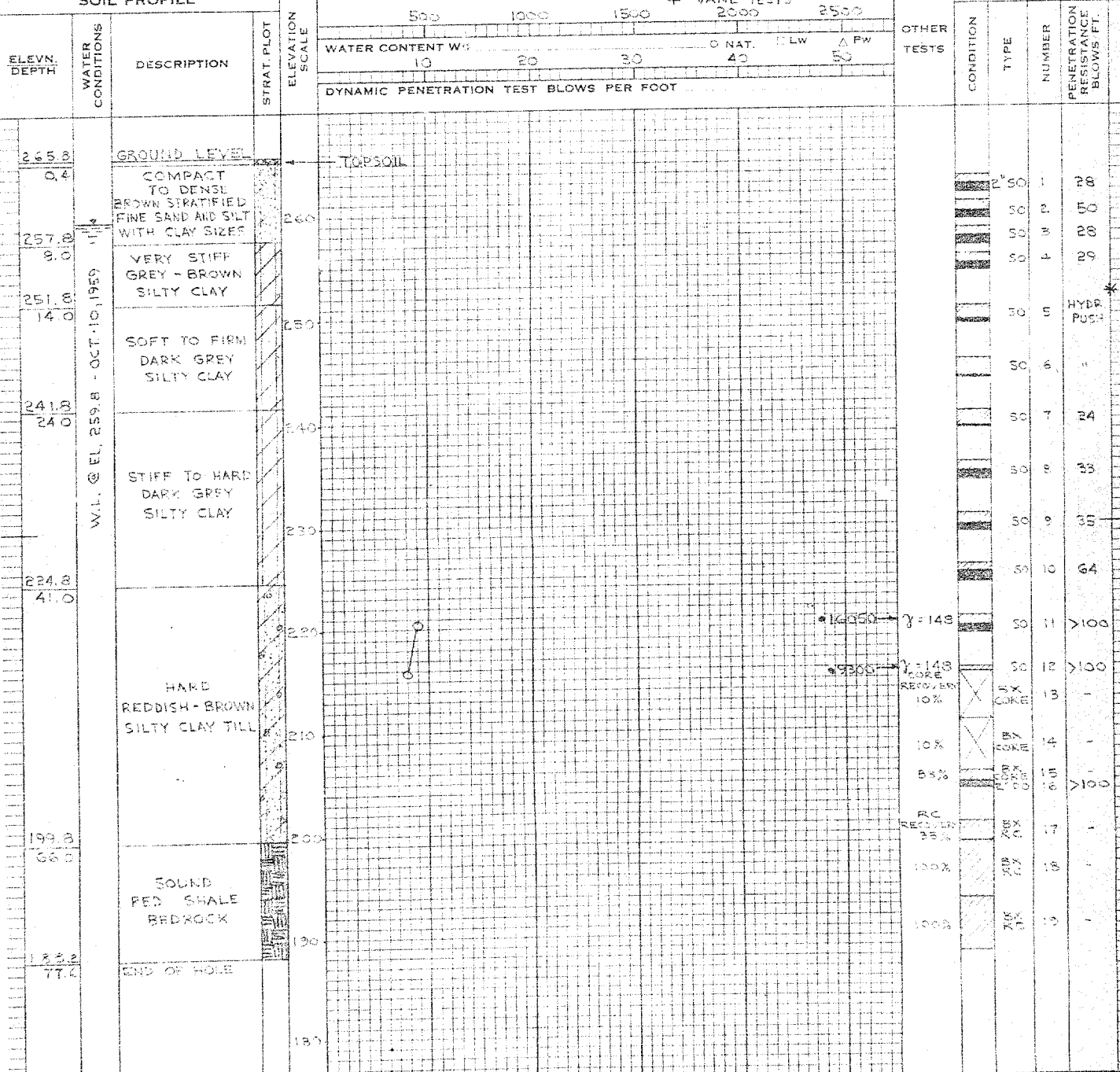
ABBREVIATIONS

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

SOIL PROFILE

SHEAR STRENGTH IN LBS/SQ FT - UNCONFINED + VANE TESTS

SAMPLES



GEOCON

OFFICE REPORT ON SOIL EXPLORATION

CONTRACT 36280 BORING # 4 DATUM GEOMETRIC CASING 8"
 BORING DATE OCT. 5, 1951 REPORT DATE OCT. 26, 1951 COMPILED BY MARY E. J. A. CHECKED BY [initials]
 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

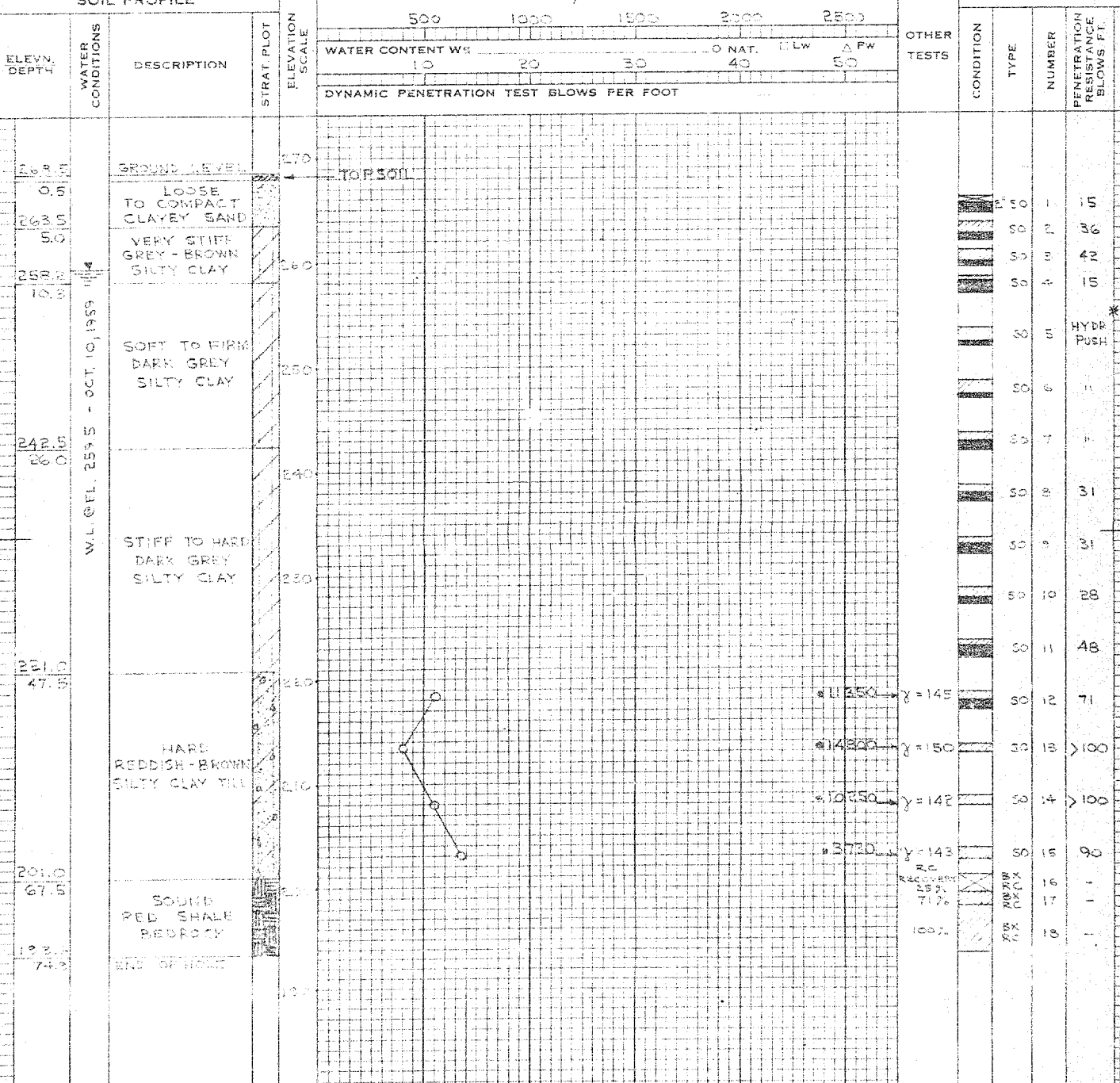
ABBREVIATIONS

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

SOIL PROFILE

SHEAR STRENGTH IN LBS/SQ. FT. - UNCONFINED

SAMPLES



GEOCON

OFFICE REPORT ON SOIL EXPLORATION

CONTRACT 56989 BORING # 5 DATUM GEODETTIC CASING 2X
 BORING DATE OCT. 8-9, 1959 REPORT DATE OCT. 28, 1959 COMPILED BY J.A. CHECKED BY W.E.
 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

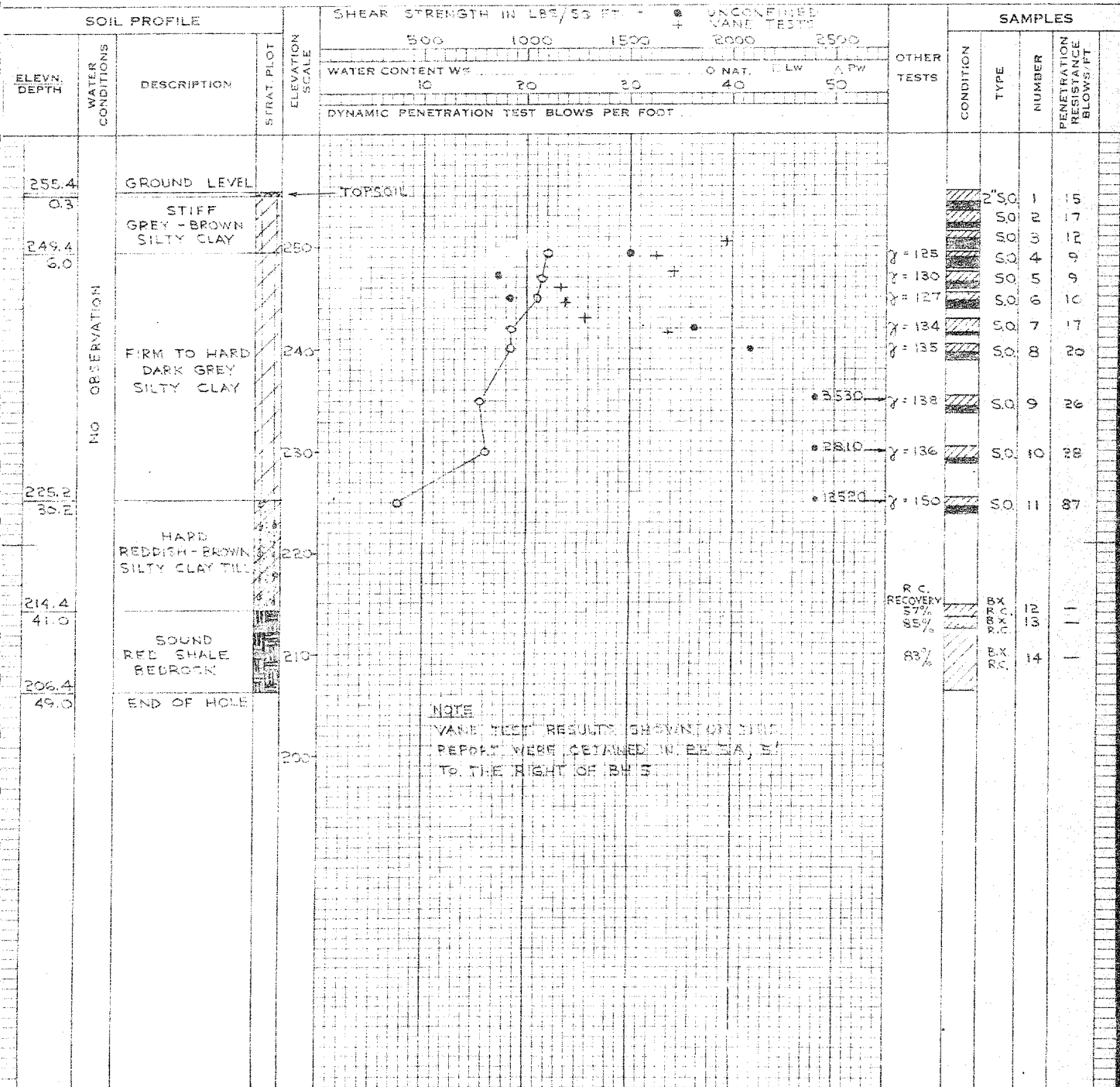
F.S. - FOIL SAMPLE
 S.O. - SLEEVE-OPEN
 S.F. - SLEEVE-FOOT VALVE
 T.O. - THIN WALLED OPEN
 R.C. - ROCK CORE

ABBREVIATIONS

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



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OFFICE REPORT ON SOIL EXPLORATION

CONTRACT 56989 BORING # 6 DATUM GEODETIC CASING _____
 BORING DATE OCT. 9, 1959 REPORT DATE OCT. 28, 1959 COMPILED BY J. A. CHECKED BY _____
 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

SHEAR STRENGTH IN LBS/SQ. FT. - UNCONFINED VANE TESTS

500 1000 1500 2000 2500

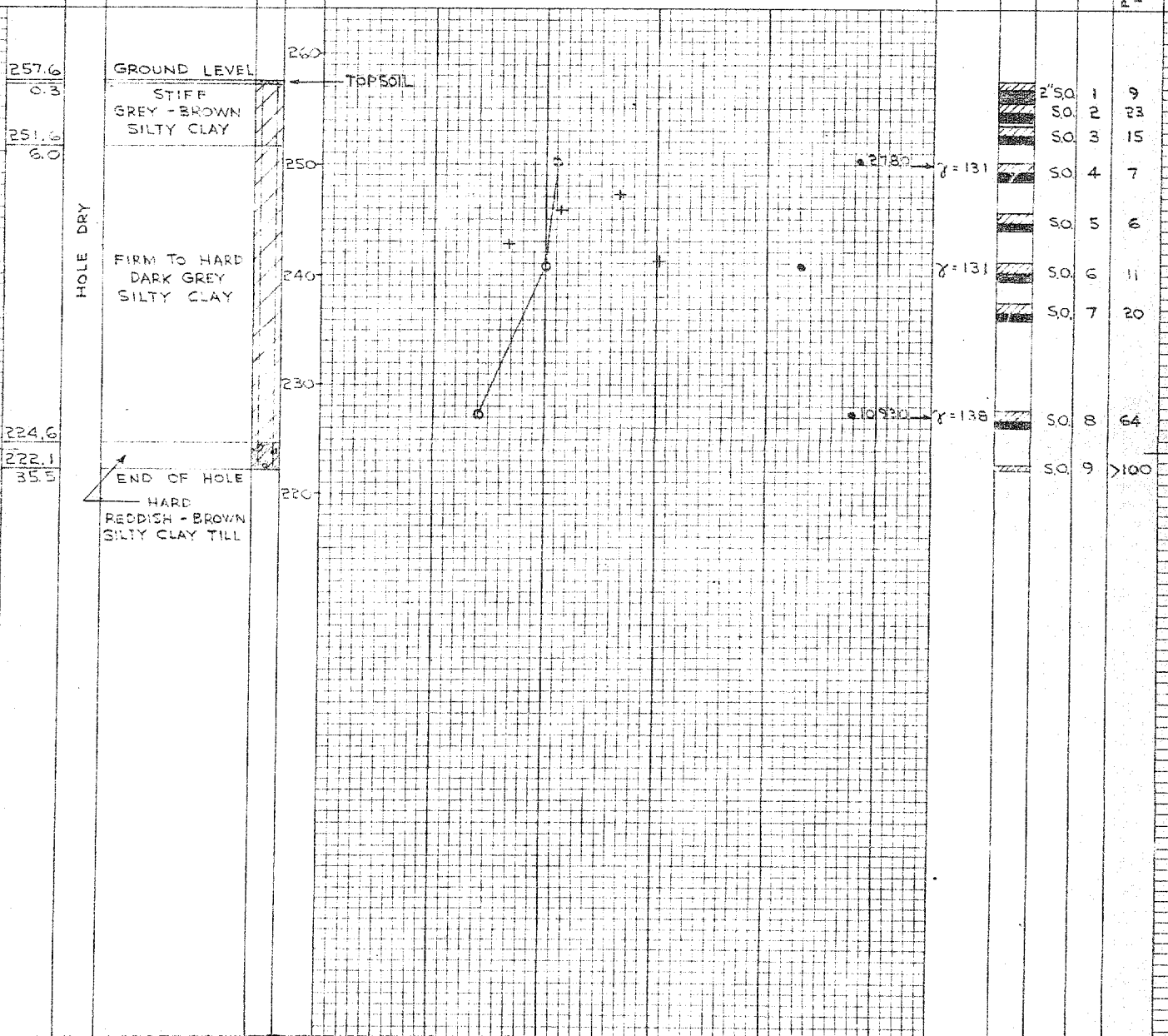
WATER CONTENT W% 10 20 30 40 50 NAT. LW Δ PW

DYNAMIC PENETRATION TEST BLOWS PER FOOT

SAMPLES

CONDITION
 TYPE
 NUMBER
 PENETRATION RESISTANCE BLOWS/FT.

OTHER TESTS

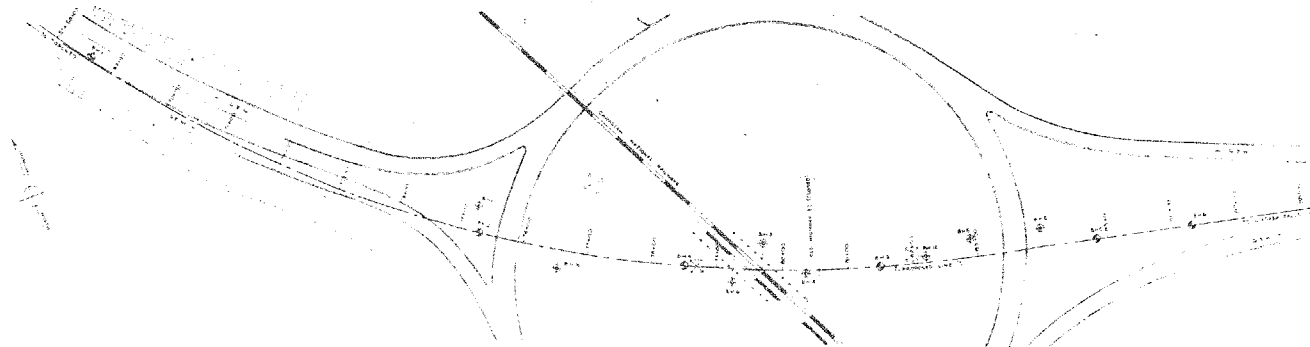


APPENDIX II

Drawings S6989-1 Boring Plan and Soil Stratigraphy
S6989-2 Alternative Abutment Foundations



KEY PLAN



PLAN

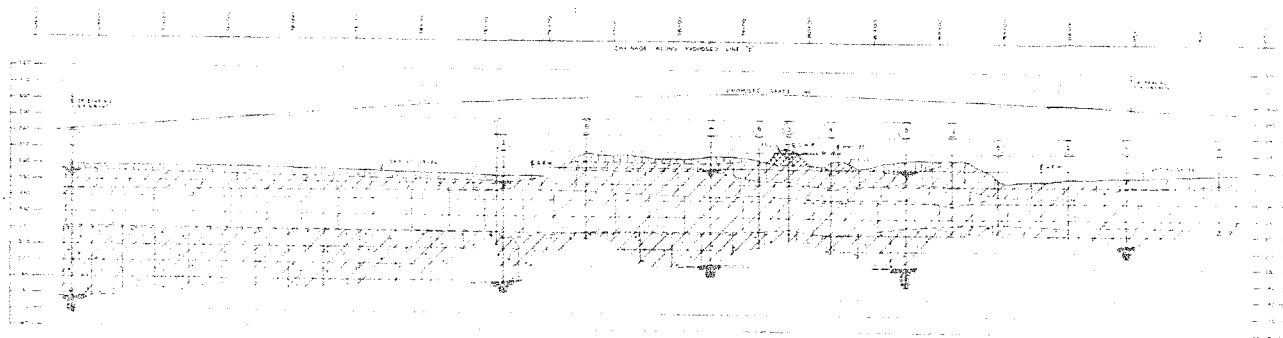
STRATIGRAPHY

- LOOSE BROWN CLAY - 10 TO 20 FT. DEEP
- STIFF TO VERY STIFF BROWN CLAY - 10 TO 20 FT. DEEP
- SOFT TO MEDIUM BROWN CLAY - 10 TO 20 FT. DEEP
- FIRM TO HARD BROWN CLAY - 10 TO 20 FT. DEEP
- HARD BROWN TO BROWN CLAY - 10 TO 20 FT. DEEP
- SAND AND GRAVEL - 10 TO 20 FT. DEEP

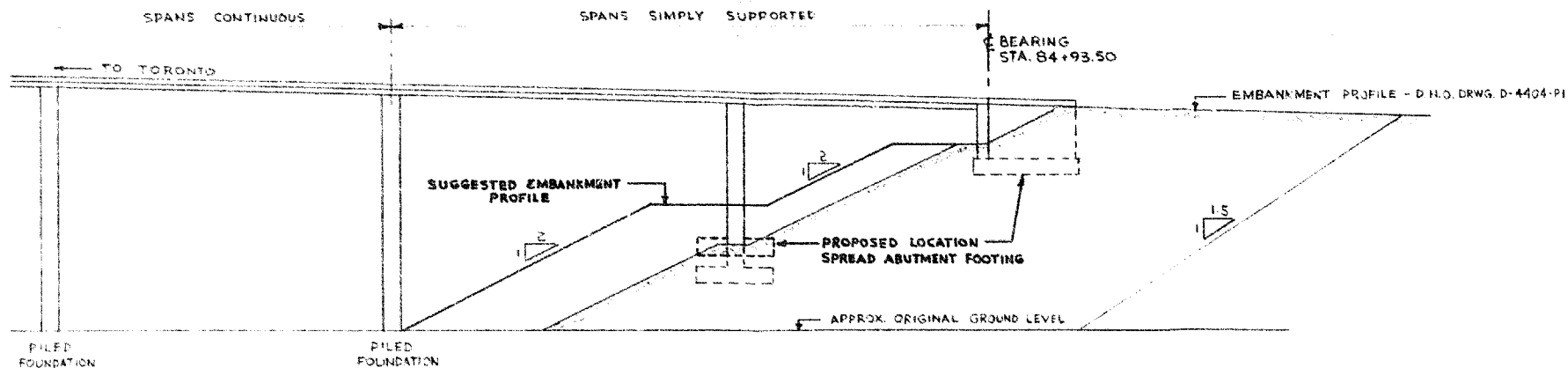
LEGEND

- PROPOSED LINE
- EXISTING LINE
- PROPOSED LINE WITH STATIONING
- EXISTING LINE WITH STATIONING
- WATER LEVEL IN WELL - 10 TO 20 FT. DEEP

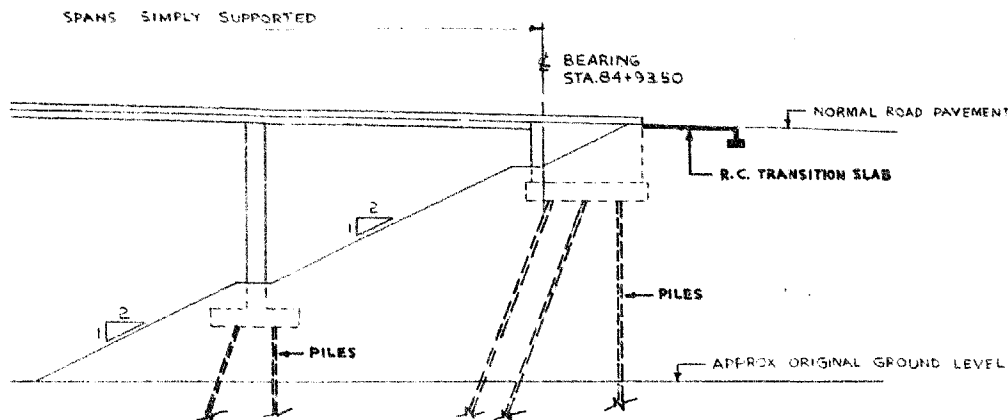
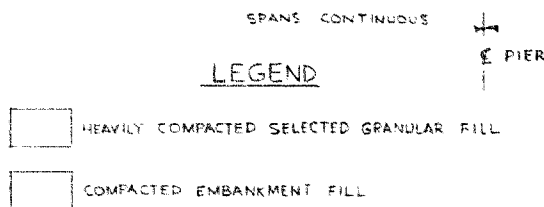
NOTES: 1. THE PROPOSED LINE IS SHOWN IN THE PLAN AND SECTION VIEWS. 2. THE EXISTING LINE IS SHOWN IN THE PLAN AND SECTION VIEWS. 3. THE WATER LEVEL IN THE WELL IS SHOWN IN THE SECTION VIEW.



SECTION ALONG PROPOSED LINE 'D'



SUGGESTED SPREAD FOUNDATIONS - TYPICAL ABUTMENT



PROPOSED PILED FOUNDATIONS (DRWG. D-4404-P1) - TYPICAL ABUTMENT

REFERENCE	
DWG. NO.	DESCRIPTION
D-4404-P1	D.H.O. - STONEY CREEK TRAFFIC CIRCLE OVERPASS AND C.N.R. OVERHEAD - PRELIMINARY PLAN

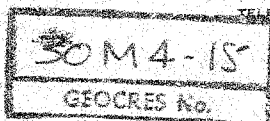
GEOCON LTD	
DEPARTMENT OF HIGHWAYS, ONTARIO	
TORONTO	
PROPOSED OVERPASS STRUCTURE	
QUEEN ELIZABETH WAY - STONEY CREEK, ONTARIO	
ALTERNATIVE ABUTMENT FOUNDATIONS	
DATE DEC 4, 1959 SCALE 1" = 20'-0"	
MADE J.A.	CHKD. M.R.
APPD. V.M.	No. S 6989-2

GEOCON LTD

HEAD OFFICE

180 VALLÉE ST., MONTREAL 18, QUEBEC

TELEPHONE UN. 3-7612



Rexdale, Ontario,
May 5th, 1958.

BR 735
DISTRICT OFFICES
14 HAAS ROAD
REXDALE, TORONTO, ONT.
TEL. CH. 4-8641
3355 WEST BROADWAY AVE.
VANCOUVER 8, B.C.
TEL. CH. 3810

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

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

Re: Soil Investigation,
Proposed Embankments and Structures,
Queen Elizabeth Way,
Wentworth County, Ontario.

Dear Sirs:

This letter accompanies our detailed report covering the above soil investigation.

We find that, from the point of view of foundation design, the site may be divided into three areas of different soil conditions. The soil conditions in these areas together with recommendations for preliminary design of foundations for the proposed embankments and structures are discussed in detail in the report.

In general, the embankments will have to be constructed in stages over a period of possibly as long as 2 years, and the overpass structures will have to be carried on piles. Economic and time considerations may show that a continuous trestle type structure founded as discussed, is more suitable for this site and it is recommended that this alternative be studied.

We believe that our report gives all the information required to enable preliminary design to proceed. However, should you require any further information, please give us a call.

Yours very truly,

GEOCON LTD

M. A. J. Matich

M. A. J. Matich, P. Eng.,
Chief Engineer.

MAJM/dw
S6623



ST. JOHN S

HALIFAX

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S6623
REPORT
TO
DEPARTMENT OF HIGHWAYS ONTARIO
ON
SOIL CONDITIONS AND FOUNDATIONS
PROPOSED OVERPASS STRUCTURES AND EMBANKMENTS
QUEEN ELIZABETH WAY
HAMILTON ONTARIO

Distribution:

- 4 copies - Department of Highways Ontario,
Toronto, Ontario.
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INDEX

	<u>Page</u>
Introduction	1
Summarized Soil Conditions	1
Discussion	1
Foundations	2
Conclusions and Recommendations	5
Personnel	6

Appendix I

Procedure

Site and Geology

Soil Conditions

Water Conditions

Office Reports on Soil Exploration

Appendix II

Figures

Drawing in Pocket at rear:

S6623-1 Boring Plan and Soil Stratigraphy

INTRODUCTION

Geocon Ltd has been retained by the Ontario Department of Highways (proposal dated February 3rd, 1958 and accepted February 12th, 1958) to investigate and report on the soil conditions at the site of the proposed Queen Elizabeth Way overpass at the existing traffic circle and C.N.R. track in Wentworth County, Hamilton, Ontario. The purpose of the investigation was to determine and interpret the general soil conditions and make recommendations for the preliminary design of foundations for the proposed overpass structures and embankments.

SUMMARIZED SOIL CONDITIONS

The site is covered by from 25 to 45 feet of clay, the upper 4 to 16 feet of which have been weathered and desiccated to a stiff to hard consistency. Loose silt, sand and peat pockets occur in places in the desiccated crust. With the exception of the extreme easterly portion of the site, about 10 feet of soft to firm dark grey silty clay underlies the gray-brown desiccated clay crust. The remainder of the clay stratum is of stiff to hard consistency and is underlain by hard reddish-brown clay till. From available geological information, it is known that the hard till is underlain by red shale bedrock.

DISCUSSION

It is understood that the alignment of the proposed extension of Queen Elizabeth Way is restricted by existing transportation facilities as shown on Drawing S6623-1.

Due to the variation of the upper soil strata over the site, the longitudinal section as shown on the drawing at the rear of this report has for purposes of analysis been divided into three areas designated "A", "B" and "C" which have different soil conditions and which will be considered separately in this discussion.

Area A, generally including that area from centre line station 74+00 to 77+00, has a very heterogeneous upper stratum of 10 to 15 feet of stiff clay with large areas of loose to compact silty sands and organic matter underlain by 10 feet of soft silty clay and 20 feet of stiff clay.

Area B, generally including that area from station 77+00 to 82+50, has a stiff desiccated clay crust for the upper 15 feet underlain by 10 feet of soft silty clay and 20 feet of stiff grey clay.

Area C, generally including that area from station 82+50 to 84+00, has an upper stiff crust of desiccated clay approximately 15 feet in thickness underlain by a stiff grey silty clay stratum approximately 20 feet thick.

FOUNDATIONS

Embankments

Preliminary studies of embankment design have been carried out for each of the three areas described above. The data used in the analyses is given on Figures 6, 7 and 8, Appendix II and on Drawing S6623-1. It has been assumed that the embankments will be constructed in granular fill, well compacted in layers not exceeding 9 inches in thickness.

Area A: Computations were carried out to determine the maximum height of embankment which could be constructed in this area. Because of the presence of the comparatively thin layer of soft clay at shallow depth, a spreading type failure was found to be critical. The results of the analyses showed that where the soft clay layer is overlain by loose sand and organic deposits, the recommended maximum initial height to which the embankment may be raised is 20 feet with side slopes of 1 vertical to 3 horizontal. Thus the final required height of 35 feet of embankment in this area could only be achieved by stage construction.

Embankments (continued)

Due to consolidation under the initial embankment loads, the shearing strength of the soft clay stratum will increase with time. Preliminary computations based on the results of consolidation tests and consolidated quick triaxial tests, summarized on Figures 3 and 8 respectively, show that the first increase in height of embankment should be restricted to a lift not greater than 5 feet after a period of at least 1 year. The final height of embankment could then be attained with further successive lifts not greater than 5 feet each, at time intervals of at least 6 months between each lift.

Area B: Similar to the discussion above, the critical height of embankment in this area is also governed by analysis of a spreading type failure. The presence of a stiff desiccated clay crust overlying the thin soft clay stratum increases the factor of safety against lateral spreading. Consequently, the proposed embankment may be raised initially to a recommended maximum height of 25 feet with side slopes of 1 vertical to $2\frac{1}{2}$ horizontal. After a period of at least 1 year, the embankment may be raised 5 feet and following a further 6 months completed to the assumed final height of 35 feet.

Area C: Computations show that, in this area, where the clay is stiff throughout, the embankment may be raised initially to the final desired height of about 35 feet with side slopes of 1 vertical to 2 horizontal.

In each of the cases above, the recommended heights have been computed on the basis of adequate factors of safety against sliding and against progressive failure due to elastic overstressing of the soft clay. It was found that the factor of safety was dependent, not only on the shear strength of the soft clay, but also upon the thickness of the stratum. For the conservative strength line assumed for the soft clay as shown on Figure 8, Appendix II, the factor of safety immediately after construction is computed to be 1.1. However, the thickness of the stratum is generally less than that assumed so that the actual factor of safety will be higher and is computed to have an average value of 1.3.

Embankments (continued)

It is considered that under the full height of embankment of 35 feet in Areas A and B, a total settlement of about 6 inches will take place below the centre of the embankment. By controlled settlement observations in the field and the installation of piezometers, it may be possible to shorten the time intervals between the successive stages of construction mentioned above. It is recommended that these field control measures be used.

Due to the variation of soil conditions, broadly described under Areas A, B and C, it is recommended that prior to final design of the embankments, the exact extent and thicknesses of the various surface deposits above about elevation 250 should be determined by short exploratory holes.

Structures

It is assumed that three overpass structures will be used to carry the highway over two parts of the existing traffic circle and also over the C.N.R. tracks as shown on Drawing S6623-1. Consequently, one structure will be located in each of the Areas A, B and C described above.

Because of the presence of the soft clay stratum and irregular loose surface deposits of sand or fill in Areas A and B, satisfactory bearing for foundations can only be reached below elevation 240. It is therefore recommended that short friction piles be used for overpass foundations. A number of pile types would be suitable, but because of the stiff character of the overlying desiccated crust, it is considered that timber piles would not be practical. Computations based on the measured shear strength of the clay show that 8 inch x 36 lbs. M. piles, for example, at a minimum spacing of 4 times the side dimension, would have an allowable bearing capacity of about 40 tons when driven to the hard clay till stratum.

Structures (continued)

In Area C occasional pockets of organic material were encountered to a depth of about 6 feet; below this depth and below about 5 feet elsewhere the clay has a compressive strength of 2 tons per square foot or greater. Spread footing foundations could therefore be used at a recommended allowable bearing value of 2 tons per square foot. However, before final design, it is considered advisable that the area of proposed footing locations be checked for the presence and depth of organic material.

To minimize negative skin friction on the piles and lateral pressures on the overpass structures due to horizontal consolidation of the soft clay stratum, it is recommended that the embankments be completed before any construction of the overpass structures is commenced.

For protection against frost action, all structural foundations should be provided with at least 4 feet of earth cover.

Possible Alternative Overpass Structure

Because of the volume of earthworks required for adequate stability in embankments of the assumed height of above 35 feet, it may prove uneconomical to use an alternative type of construction such as a continuous trestle type structure founded as discussed above under "Structures". The relative cost of such a continuous structure is beyond the scope of this preliminary investigation, but it is recommended that a comparative economic study be made considering this alternative.

CONCLUSIONS AND RECOMMENDATIONS

1. The site is generally covered by from 25 to 45 feet of clay overlying hard clay till extending to bedrock. The upper 10 to 16 feet of the clay are desiccated and this crust is underlain by about 10 feet of soft normally loaded clay. The remainder of the stratum is of stiff to hard consistency.

2. For preliminary design purposes, the soil conditions may be divided into three general areas which have been designated "A", "B" and "C", for discussion purposes.

3. From observations made during the investigation, the ground-water level is believed to be approximately 7 to 10 feet below existing ground surface.

4. Embankment design is dependent on the soil conditions in each general area. In Areas "A" and "B", it is restricted to stage construction due to stability requirements. The details of side slopes required and periods between successive stages of construction are given in the body of the report.

5. Recommended foundation treatment for the overpass structures is discussed in detail in the report.

6. It is recommended that a comparative economic study be made considering as an alternative, a trestle type structure continuous across the site.

PERSONNEL

The field work was carried out by Mr. J. N. Beckett. This report was written by Mr. R. M. Wilson, checked by Mr. V. Milligan and reviewed by Mr. M. A. J. Matich.

RMW/dw
S6623



R. M. Wilson

APPENDIX I

Procedure

Site and Geology

Soil Conditions

Water Conditions

Office Reports on Soil Exploration

GEOCON

PROCEDURE

The field work was carried out between February 17th and February 28th, 1958. Twelve boreholes were put down using a mobile power auger rig. Sampling was carried out in 2 inch and 3 $\frac{1}{2}$ inch sizes. The location of the borings and the inferred soil stratigraphy are shown on Drawing S6623-1, contained in a pocket at the rear of this report. A detailed log of each boring is given on the Office Reports on Soil Exploration in this Appendix.

The laboratory testing was carried out in the Toronto Soils Laboratory of Gecon Ltd. The laboratory results are plotted graphically on the Office Reports and are given in the Figures of Appendix II. The remaining soil samples will be stored until October 1st, 1958 and will then be destroyed unless instructions to the contrary are received.

Elevations given in this report are with reference to Geodetic Datum and were obtained from the elevation contours of the Ontario Department of Highways Plan E-3359-1.

SITE AND GEOLOGY

The site is located in the southeastern outskirts of the City of Hamilton in the County of Wentworth as shown on Gecon Drawing S6623-1 and the Ontario Department of Highways Drawing E-3359-1. The proposed revision of the Queen Elizabeth Way will pass over the existing traffic interchange of Highway No. 2 and the Queen Elizabeth Way and the existing Canadian National Railway track. The ground level varies from elevation 253 to 272 feet as a result of existing cut and fills while the original ground surface is generally at elevation 255 to 266 feet.

Available geological information indicates that glacial till overlies red shale bedrock. The till is overlain by lacustrine deposits of clay of the Lake Iroquois period, certain portions of which have been subjected to desiccation. In some areas recent heterogeneous silt, sand and organic deposits overlie the clay.

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

Topsoil

A few inches of topsoil cover the site. The topsoil is in a very loose condition and has a considerable organic content.

Grey-Brown Silty Clay

Underlying the topsoil a stratum consisting generally of grey-brown silty clay, ranging from 4 to 16 feet in thickness, was encountered.

The stratum contains local lenses and pockets of sand, silt and organic material. In boreholes 1, 7 and 8 the upper 3 feet of the stratum consist of brown clayey sand which is estimated to be loose. In borehole 4, fill composed of grey-brown silty clay, clayey silt, pockets of reddish-brown silty sand and organic matter was encountered in the upper 8 feet of the stratum. This fill is probably the roadbed of the old No. 20 highway. The relative density of the fill from the result of one standard penetration test of 3 blows per foot is estimated to be very loose to loose. In the upper 5 feet of the stratum in boreholes 2 and 10 the clay has a high organic content. An "N" value of 6 blows per foot obtained in this portion of the stratum indicates that the consistency is firm.

A layer of grey-brown clayey silt, about 5 feet in thickness, was encountered at the base of the stratum in boreholes 6 and 8, the relative density of which is compact as determined by an "N" value of 13 blows per foot.

The clay stratum generally has a horizontally laminated structure. The clay is generally sandy in the upper portion of the stratum becoming silty with depth. Occasional gravel sizes were encountered in the stratum. The colour and general structure of the stratum indicate that it has probably been subjected to weathering and desiccation. The colour of the clay is mottled grey and brown, becoming greyer with depth.

Grey-Brown Silty Clay (continued)

Unconfined compression tests carried out on samples taken from the stratum gave compressive strengths ranging from 1.4 tons per square foot to values greater than 4 tons per square foot. Standard penetration tests gave "N" values ranging from 13 to 34 blows per foot. The results of the strength and penetration tests indicate that the consistency of the stratum ranges from stiff to hard and is generally very stiff.

Figure 1, Appendix II shows typical stress strain curves for unconfined compression tests carried out on the grey brown silty clay.

The average wet unit weight obtained was about 130 pounds per cubic foot at a corresponding average moisture content of about 20 per cent.

A liquid and plastic limit determination gave values of 35 and 20 respectively, at a ^{moisture} moisture content of about 24 per cent.

Dark Grey Silty Clay

A stratum of dark grey silty clay, ranging in thickness from 14 feet in borehole 10 to 38 feet in borehole 8, was encountered beneath the weathered and desiccated grey-brown clay. The stratum contains a small percentage of gravel, up to $\frac{1}{2}$ inch in size, the gravel content generally increasing with depth.

In the lower 5 to 10 feet of the stratum, reddish-brown shale fragments were frequently encountered.

The upper 8 to 14 feet of the stratum, except in boreholes 2, 5, 10 and 11, have a soft to firm consistency, as indicated by the results of unconfined compression tests which gave compressive strengths varying from 0.4 to 0.9 tons per square foot. This is confirmed by the results of the standard penetration tests which gave "N" values ranging from a push resistance to 12 blows per foot.

Dark Grey Silty Clay (continued)

Figure 2, Appendix II shows typical stress strain curves for unconfined compression tests carried out on samples of the soft dark grey silty clay.

The results of one consolidation test carried out on a sample of the soft clay are presented on the pressure-void ratio curve in Figure 3, Appendix II.

The average wet unit weight obtained in this portion of the stratum was 127 pounds per cubic foot at a corresponding average moisture content of 25 per cent. A liquid and plastic limit determination gave values of 35 and 18 per cent respectively, at a natural moisture content of 32 per cent.

Unconfined compression tests carried out on samples taken from the remainder of the stratum gave compressive strengths ranging from 1.1 tons per square foot to values greater than 4 tons per square foot with an average value of 3.5 tons per square foot indicating that the consistency varies from stiff to hard and is generally very stiff. Standard penetration resistance or "N" values ranging from 14 to 45 blows per foot with an average value of 30 blows per foot were obtained confirming that the consistency is generally very stiff.

Typical stress strain curves for unconfined compression tests on the lower stiff portion of the stratum are shown on Figure 4, Appendix II.

The average wet unit weight obtained was 139 pounds per cubic foot at a corresponding average moisture content of 16 per cent. Liquid and plastic limit determinations gave values of 32 to 35 and 17 to 19 per cent respectively at a natural moisture content ranging from 19 to 22 per cent.

Dark Gray Silty Clay (continued)

The relationship of unconfined compressive strength to existing overburden pressure indicates that the lower portion of the dark grey silty clay stratum has been preconsolidated in excess of the existing overburden pressure.

Reddish-Brown Clay Till

A stratum of reddish-brown clay till was encountered beneath the very stiff dark grey clay and was penetrated for a maximum depth of 10 feet in borehole 4. The till consists of red shale fragments, limestone gravel and boulders in a matrix of reddish-brown silty clay. In boreholes 5 and 10 refusal was met on a boulder and in borehole 2 probably on red shale bedrock.

Three unconfined compression tests performed on samples of the till gave compressive strengths of 5.4, 7.0 and 8.7 tons per square foot. The stress strain curves for these tests are shown on Figure 5, Appendix II. The standard penetration resistance or "N" values obtained in the stratum ranged from 38 blows per foot to values in excess of 100 blows per foot. The results of the compression and penetration tests indicate that the consistency of the till stratum is hard.

Two wet unit weight determinations gave values of 146 and 143 pounds per cubic foot at moisture contents of 9 and 10 per cent.

WATER CONDITIONS

The groundwater level, after completion of the boring, was encountered about 7 feet below ground level in boreholes 3 and 4 and about 15 feet below ground level in boreholes 6, 7 and 8. The remaining boreholes were dry after completion of the boring. However, several days after completion of the borings, the groundwater level, in the majority of the boreholes, was at ground level due to surface water seepage. The true groundwater level is believed to be at the surface of the dark grey silty clay or approximately 7 to 10 feet below the existing ground surface.

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

OFFICE REPORT ON SOIL EXPLORATION

CONTRACT 56623 BORING # 1 DATUM D.H.O. CASING _____
BORING DATE FEB. 17, 1958 REPORT DATE FEB. 20, 1958 COMPILED BY MMW CHECKED BY Y.A.
SAMPLER HAMMER WT. 143 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	F.S. - FOIL SAMPLE
S.T. - SLOTTED TUBE	S.O. - SLEEVE-OPEN
W.S. - WASHED SAMPLE	S.F. - SLEEVE-FOOT VALVE
D.O. - DRIVE-OPEN	T.O. - THIN WALLED OPEN
D.F. - DRIVE-FOOT VALVE	R.C. - ROCK CORE
C.S. - CHUNK SAMPLE	

ABBREVIATIONS

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

SOIL PROFILE

SOIL PROFILE				COMPRESSIVE STRENGTH (LBS/SQ. FT.) - UNCONFINED						SAMPLES						
ELEV. DEPTH	WATER CONDITIONS	DESCRIPTION	STRAT. PLOT	ELEVATION SCALE	20406080100					OTHER TESTS	CONDITION	TYPE	NUMBER	PENETRATION RESISTANCE BLOWS/FT.		
					WATER CONTENT W%1020304050											
					DYNAMIC PENETRATION TEST BLOWS PER FOOT											
254.0 0.0	DRY AFTER COMPLETION OF HOLE FEB. 18, 1959 W.L. & GROUND LEVEL FEB. 28, 1955. DUE TO SURFACE WATER	02' TOP SOIL GROUND LEVEL		250						$\gamma = 132$		RA-1	1	-		
250.9 3.1		LOOSE BROWN CLAYEY SAND									$\gamma = 136$		SA-2	2	7	
244.9 9.1		SOFT TO FIRM DARK GREY SILTY CLAY										$\gamma = 138$		SA-3	3	1
												$\gamma = 139$		SA-4	4	14
												$\gamma = 139$		SA-5	5	31
				STIFF TO VERY STIFF DARK GREY SILTY CLAY								$\gamma = 140$		SA-6	6	11
												$\gamma = 140$		SA-7	7	54
220.0 34.0				HARD REDDISH-BROWN CLAY TILL		220						$\gamma = 148$		SA-8	8	35
212.3 41.7				END OF HOLE		210						$\gamma = 146$		SA-9	9	60
														SA-10	10	7100

GEOCON

OFFICE REPORT ON SOIL EXPLORATION

CONTRACT 54423 BORING # 2 DATUM Z.H.O. CASING 1
 BORING DATE FEB. 13, 1958 REPORT DATE FEB. 21, 1958 COMPILED BY M.W. CHECKED BY J.A.
 SAMPLER HAMMER WT. 14.5 LBS DROP 30 INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN. LBS. ENERGY)

SAMPLE CONDITION

☐ DISTURBED
☐ FAIR
☐ GOOD
☐ LOST

SAMPLE TYPES

A.S. - AURER 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

ELEV. DEPTH	WATER CONDITIONS	DESCRIPTION	STRAT. PLOT	ELEVATION SCALE
234.5		GROUND LEVEL		
240.0		STIFF TO VERY STIFF GRAY-BROWN SILTY CLAY		250
243.0		VERY STIFF TO HARD DARK GRAY SILTY CLAY		240
244.0		HARD REDDISH-BROWN CLAY TILL		230
213.0		END OF HOLE		210

COMPRESSION STRENGTH (TONS/SQ. FT.) UNCONFINED

20 40 60 80 100

WATER CONTENT W% 10 20 30 40 50

DYNAMIC PENETRATION TEST BLOWS PER FOOT

OTHER TESTS

SAMPLES

CONDITION	TYPE	NUMBER	PENETRATION RESISTANCE BLOWS/FT.
	A2	1	
	250	2	17
	30	3	32
	32	4	20
	32	5	32
	30	6	25
	30	7	27
	30	8	200

GEOCON

OFFICE REPORT ON SOIL EXPLORATION

CONTRACT S4423 BORING # 3 DATUM D.H.O. CASING
 BORING DATE FEB. 12, 1958 REPORT DATE FEB. 21, 1958 COMPILED BY W.W. CHECKED BY
 SAMPLER HAMMER WT. 143 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

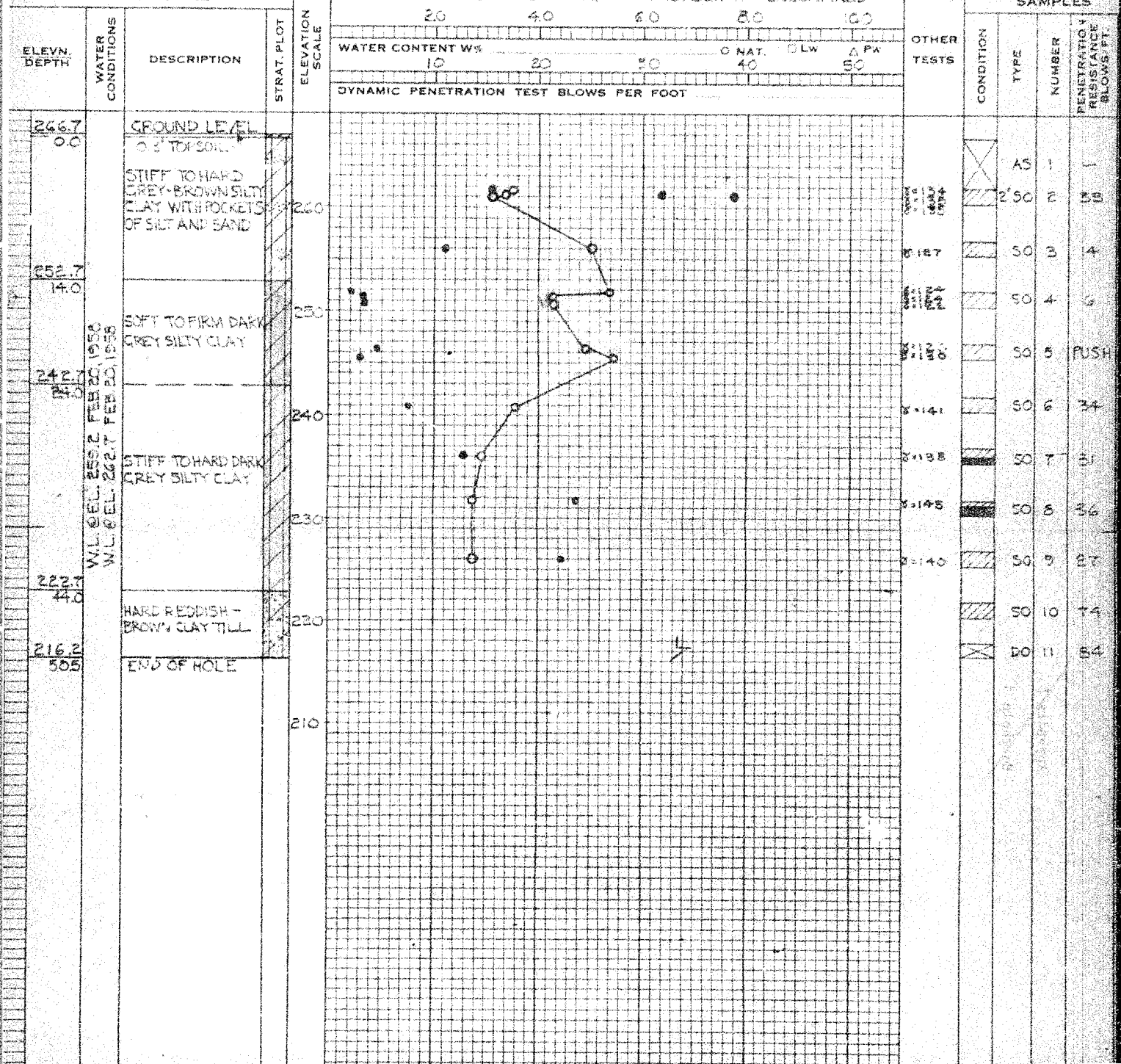
ABBREVIATIONS

V. - IN-SITU VANE TEST
 M. - MECHANICAL ANALYSIS
 U. - UNCONFINED COMPRESSION
 Qc. - TRIAXIAL CONSOLIDATED QUICK
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 K - PERMEABILITY
 C - CONSOLIDATION
 WL - WATER LEVEL IN CASING
 WT - WATER TABLE IN SOIL

SOIL PROFILE

COMPRESSIVE STRENGTH TONS/SG.FT. *UNCONFINED

SAMPLES



GEOCON

OFFICE REPORT ON SOIL EXPLORATION

CONTRACT 56623 BORING # 4 DATUM D.H.O. CASING
 BORING DATE FEB. 20, 1958 REPORT DATE FEB. 24, 1958 COMPILED BY M.W. CHECKED BY J.A.
 SAMPLER HAMMER WT. 145 LBS DROP 20 INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN. LBS. ENERGY)

SAMPLE CONDITION

☐ DISTURBED
☐ FAIR
☐ GOOD
☐ LOST

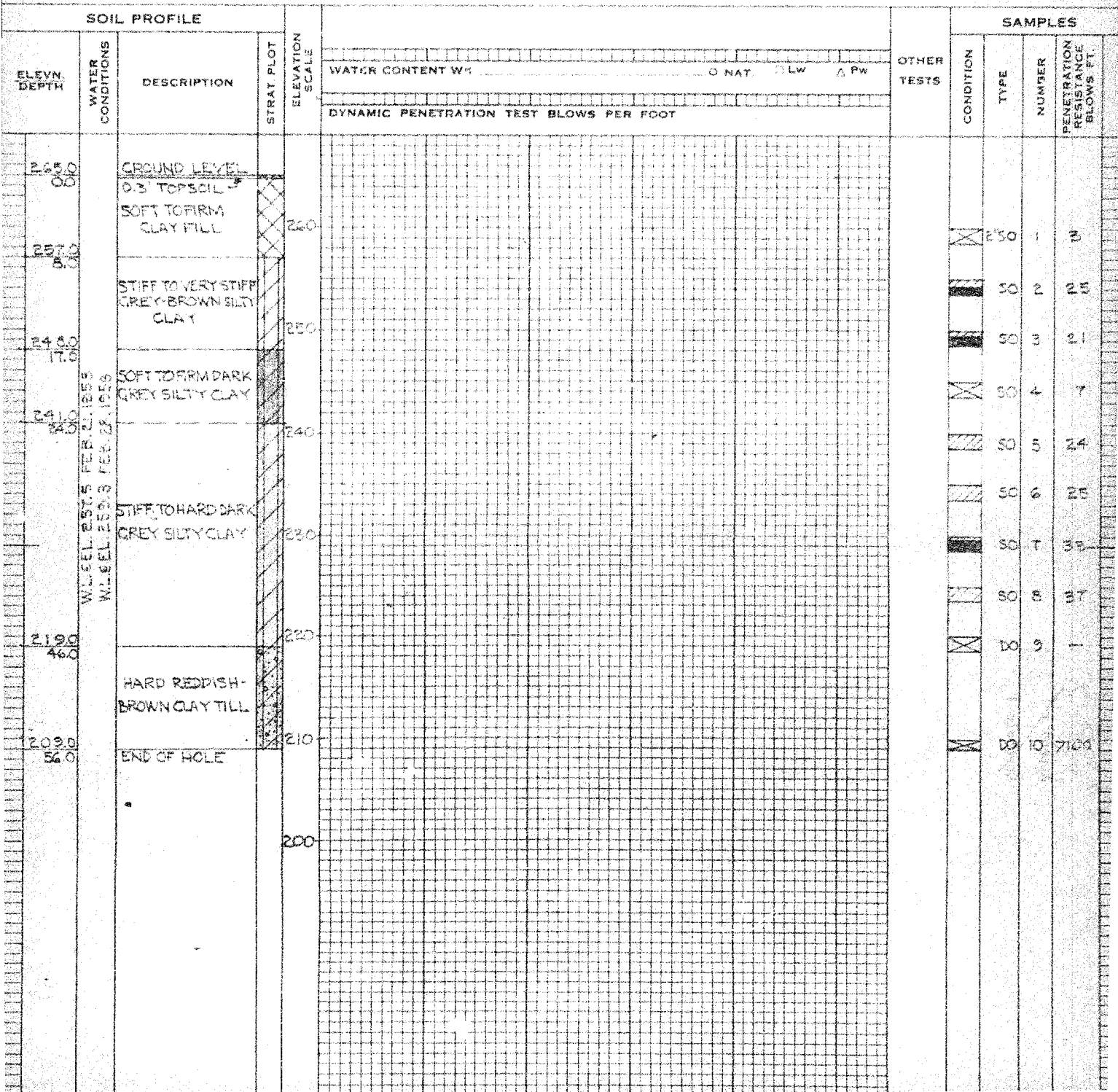
SAMPLE TYPES

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
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 M - MECHANICAL ANALYSIS
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 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 C-4285 BORING # 3 DATUM D.H.C. CASING
 BORING DATE FEB. 21, 1958 REPORT DATE FEB. 25, 1958 COMPILED BY W.M. CHECKED BY J.A.
 SAMPLER HAMMER WT. 145 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
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 QC - TRIAXIAL CONSOLIDATED QUICK
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 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. • UNCONFINED

SAMPLES

20 40 60 80 100
 WATER CONTENT W% 0 NAT. 100
 10 20 30 40 50
 DYNAMIC PENETRATION TEST BLOWS PER FOOT

OTHER TESTS

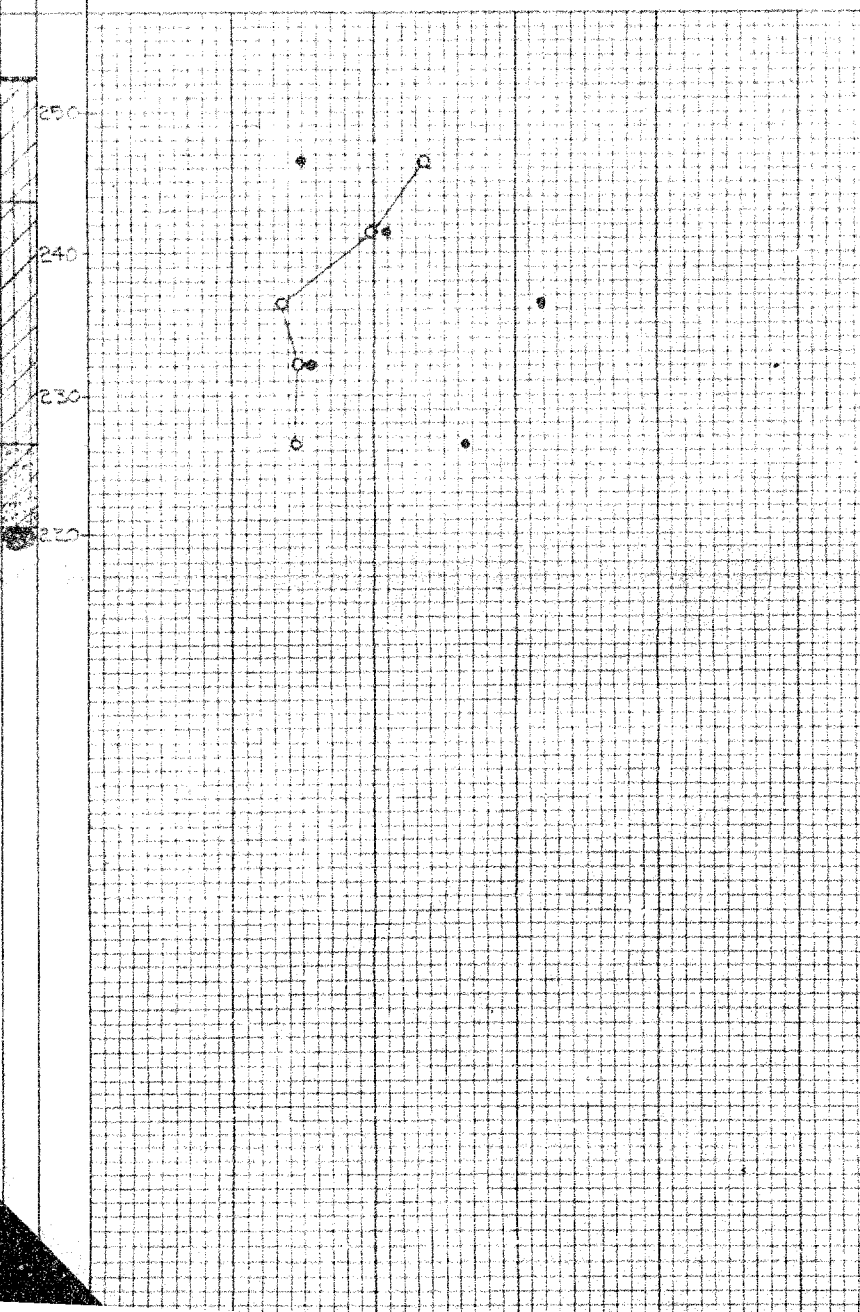
CONDITION
 TYPE
 NUMBER
 PENETRATION RESISTANCE BLOWS/FT.

ELEV. DEPTH
 WATER CONDITIONS
 DESCRIPTION
 STRAT. PLOT
 ELEVATION SCALE

2528 0.0
 243.0 9.0
 2260 26.0
 220.8 32.0

GROUND LEVEL
 CR. TOP SOIL
 STIFF TO VERY STIFF
 GREY BROWN SILTY
 CLAY
 VERY STIFF TO
 HARD DARK GREY
 SILTY CLAY
 DARK REDDISH-
 BROWN CLAY TILL
 END OF HOLE
 REFUSAL PROBABLY
 BOULDER

WATER AFTER COMPLETION OF HOLE FEB. 21, 1958
 DRY
 W.L. = GROUND LEVEL FEB. 28, 1958 DUE TO SURFACE WATER



γ = 12.8
 γ = 12.6
 γ = 14.2
 γ = 14.2

250
 50
 50
 50
 50
 DO

1 2
 2 3
 3 4
 4 5
 5 6

GEOCON

OFFICE REPORT ON SOIL EXPLORATION

CONTRACT 30002 BORING # 1 DATUM D.H.C. CASING 2
 BORING DATE FEB. 14, 1958 REPORT DATE FEB. 27, 1958 COMPILED BY M.S.C. CHECKED BY W.H.
 SAMPLER HAMMER WT. 145 LBS. DROP 30 INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN. LBS. ENERGY)

SAMPLE CONDITION

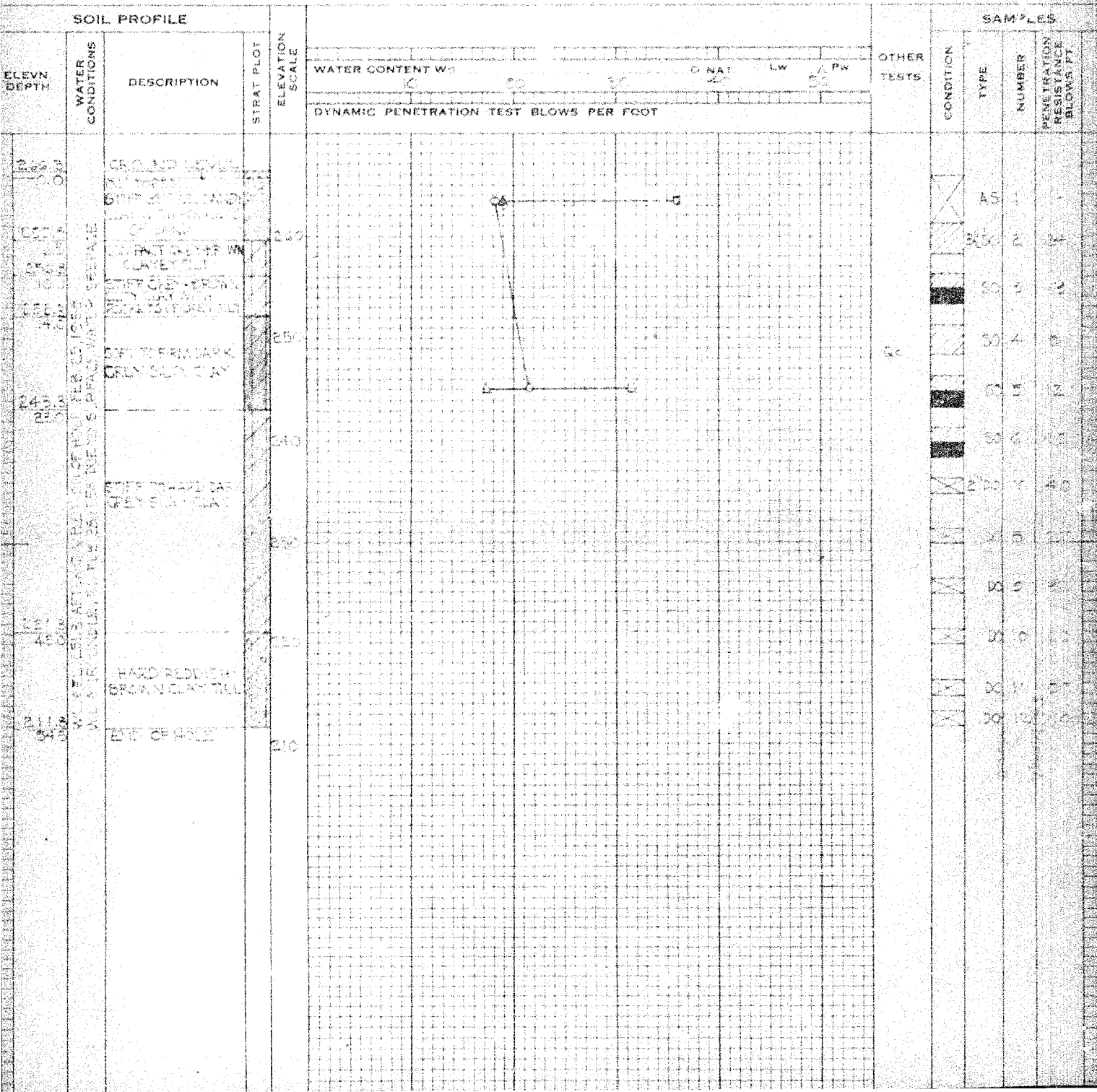
[] DISTURBED
 [] FAIR
 [] GOOD
 [] LOST

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
 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



GEOCON

OFFICE REPORT ON SOIL EXPLORATION

CONTRACT 56223 BORING # 7 DATUM D.H.O. CASING —
 BORING DATE FEB. 25, 1958 REPORT DATE MAR. 3, 1958 COMPILED BY M.W. CHECKED BY J.A.
 SAMPLER HAMMER WT. 145 LBS. DROP 30 INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN. LBS. ENERGY)

SAMPLE CONDITION



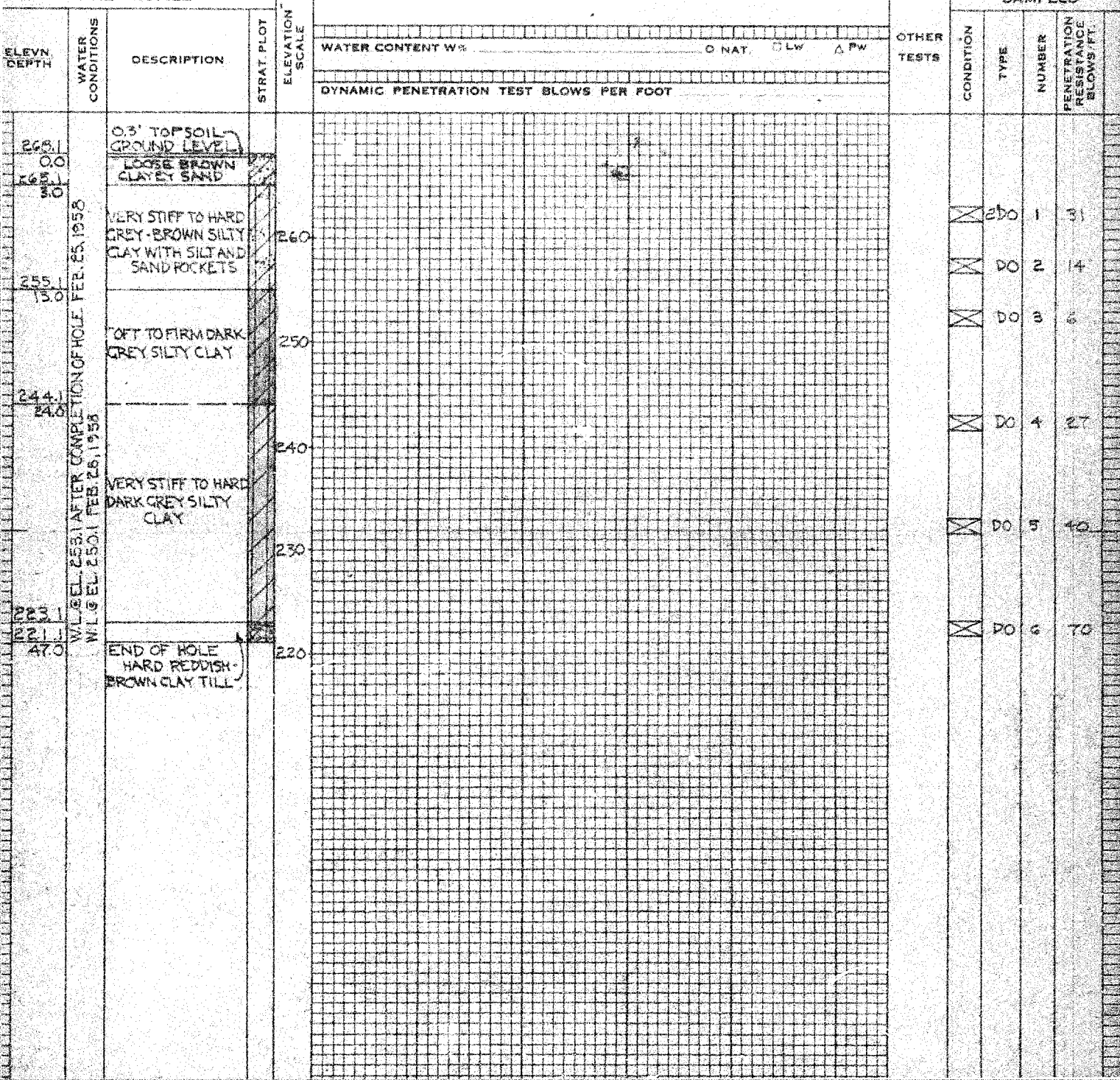
SAMPLE TYPES

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 - 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



OFFICE REPORT ON SOIL EXPLORATION

CONTRACT 54623 BORING # 8 DATUM D.H.Q. CASING
BORING DATE FEB. 25, 1958 REPORT DATE MAR. 3, 1958 COMPILED BY MSV CHECKED BY J.A.
SAMPLER HAMMER WT. 14.5 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	F.S. - FOIL SAMPLE
S.T. - SLOTTED TUBE	S.O. - SLEEVE-OPEN
W.S. - WASHED SAMPLE	S.F. - SLEEVE-FOOT VALVE
D.O. - DRIVE-OPEN	T.O. - THIN WALLED OPEN
D.F. - DRIVE-FOOT VALVE	R.C. - ROCK CORE
C.S. - CHUNK SAMPLE	

ABBREVIATIONS

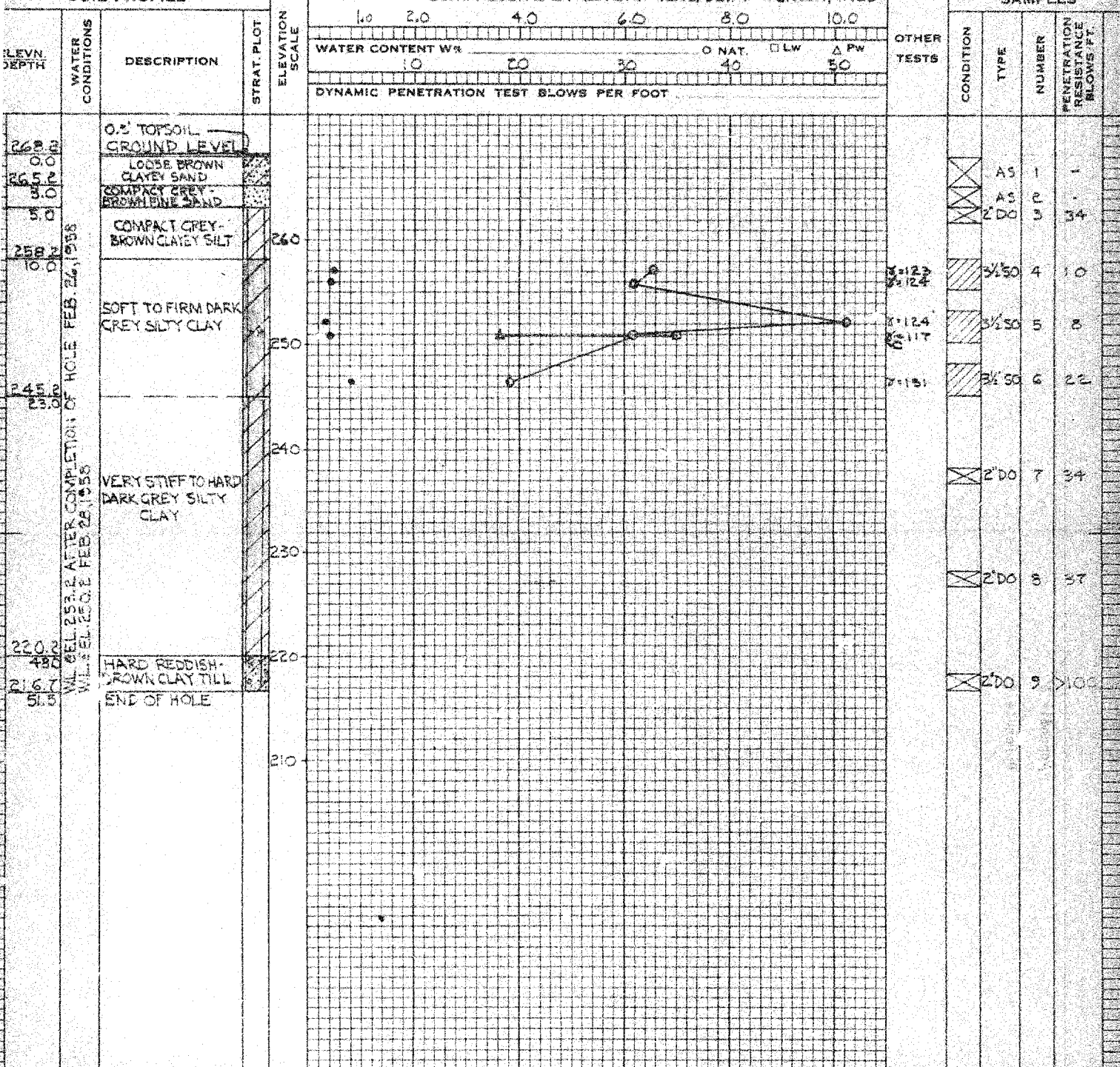
V	- IN-SITU VANE TEST	γ	- WET UNIT WEIGHT
M	- MECHANICAL ANALYSIS	K	- PERMEABILITY
U	- UNCONFINED COMPRESSION	C	- CONSOLIDATION
QC	- TRIAXIAL CONSOLIDATED QUICK		
Q	- TRIAXIAL QUICK	WL	- WATER LEVEL IN C
S	- TRIAXIAL SLOW	WT	- WATER TABLE IN S

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

SOIL PROFILE

COMPRESSIVE STRENGTH TONS/SQ. FT. • UNCONFINED


SAMPLES

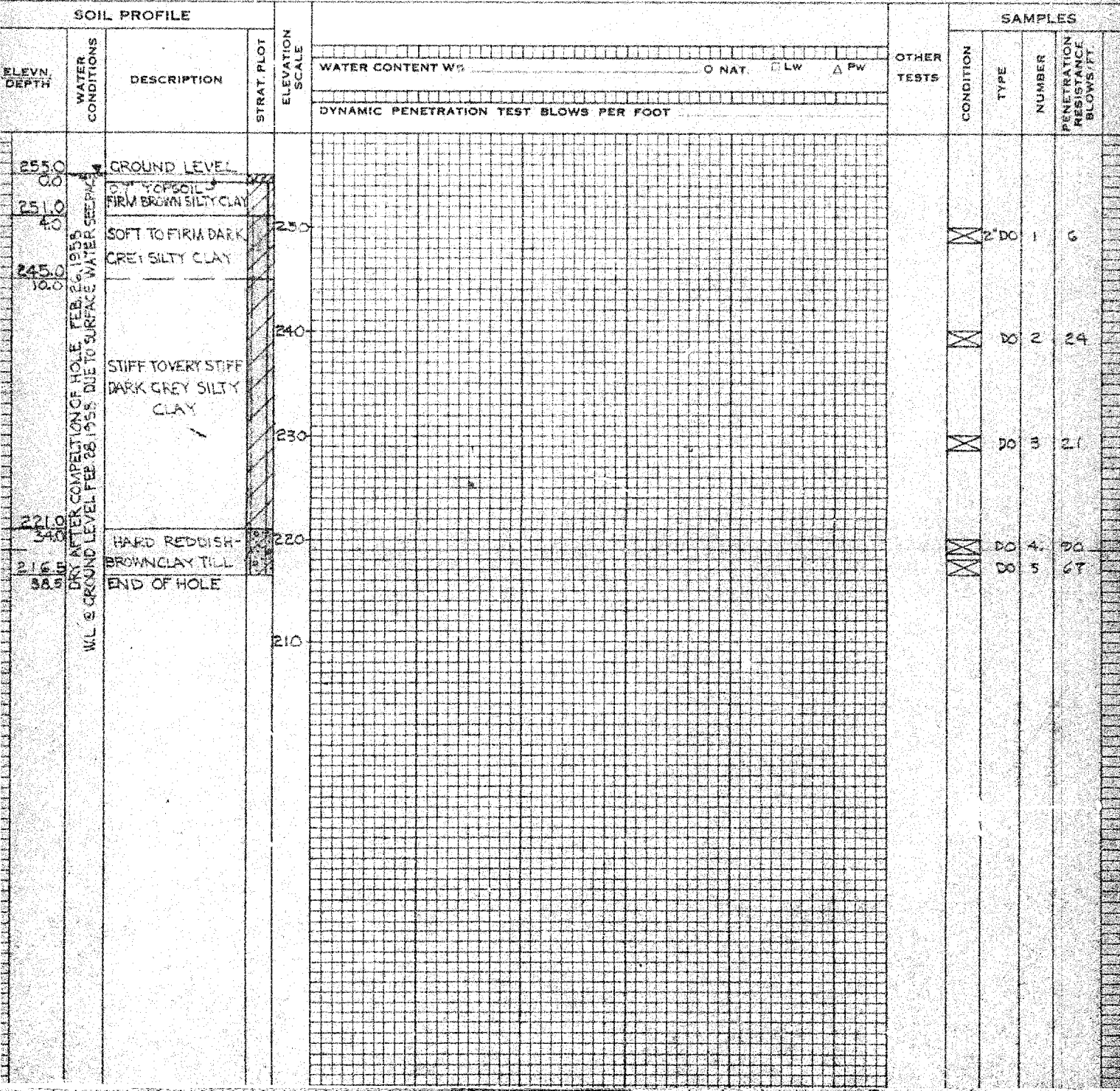


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OFFICE REPORT ON SOIL EXPLORATION

CONTRACT 56223 BORING # 2 DATUM D.H.O. CASING _____
 BORING DATE FEB. 26, 1958 REPORT DATE MAR. 3, 1958 COMPILED BY MAV CHECKED BY J.A.
 SAMPLER HAMMER WT. 145 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 CC. - 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. - CHUNK SAMPLE

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

V - IN-SITU VANE TEST
M - MECHANICAL ANALYSIS
U - UNCONFINED COMPRESSION
QC - 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				OTHER TESTS		SAMPLES			
LEVN. EPTH	WATER CONDITIONS	DESCRIPTION	STRAT. PLOT	ELEVATION SCALE	WATER CONTENT W% O NAT. □ LW ▲ PW	CONDITION	TYPE	NUMBER	PENETRATION RESISTANCE BLOWS/FT.
254.2	0.0	0.5 TOPSOIL GROUND LEVEL		250					
249.6	5.0	FIRM GREY-BROWN SILTY CLAY WITH BLACK PEAT AND SAND POCKETS		240		⊗	DO	1	6
242.2	12.0	STIFF TO VERY STIFF GREY-BROWN SILTY CLAY		230		⊗	DO	2	30
236.7	18.0	VERY STIFF TO HARD DARK GREY SILTY CLAY		220		⊗	DO	3	60
227.0	27.0	END OF HOLE HARD REDDISH-BROWN CLAY TILL REFUSAL PROBABLY BOULDER							

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OFFICE REPORT ON SOIL EXPLORATION

CONTRACT 56623 BORING # 11 DATUM D.H.O. CASING
 BORING DATE FEB. 27, 1958 REPORT DATE MARCH 3, 1958 COMPILED BY M.W. CHECKED BY J.A.
 SAMPLER HAMMER WT. 145 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

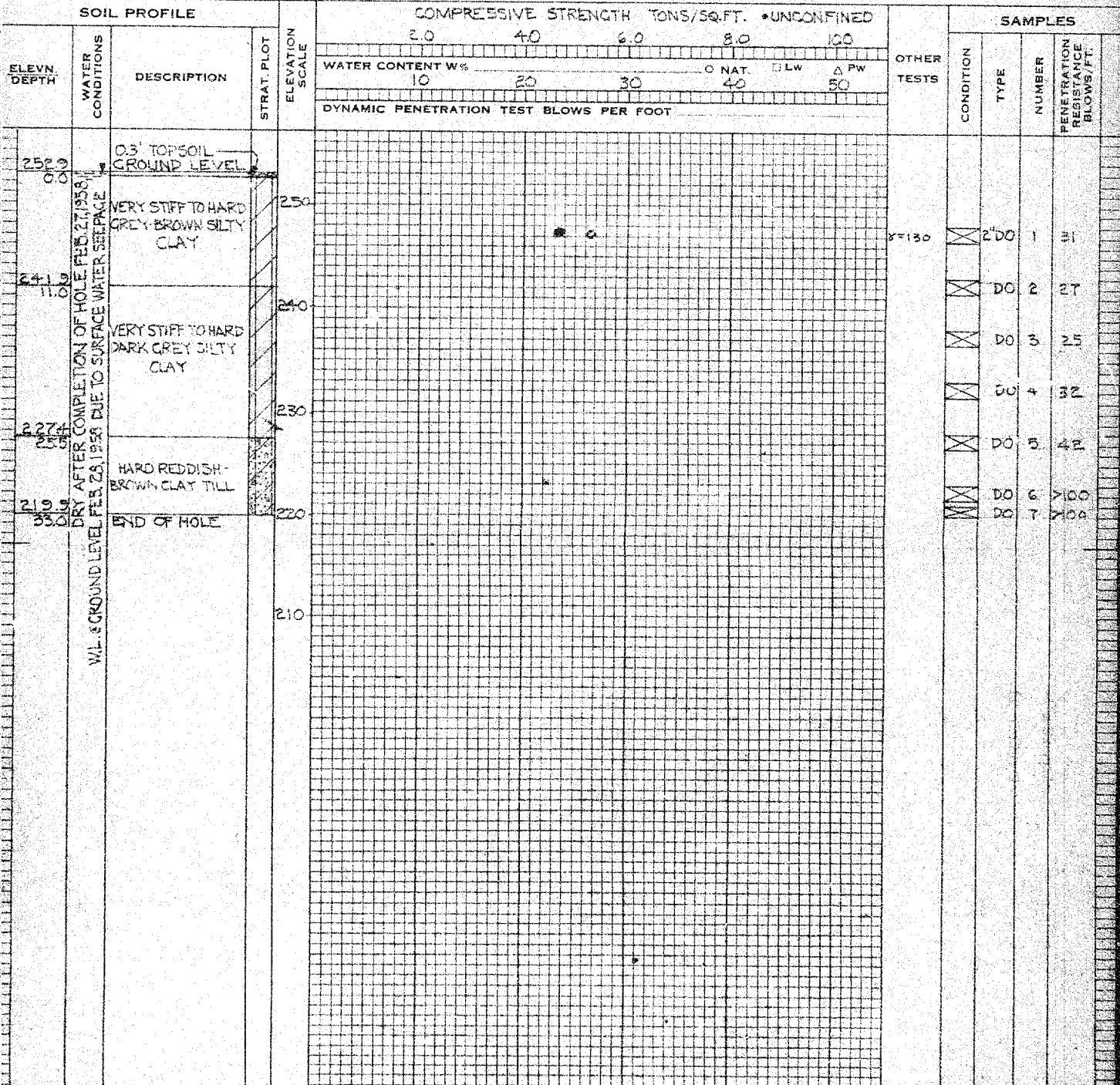
ABBREVIATIONS

V - 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



OFFICE REPORT ON SOIL EXPLORATION

SAMPLE CONDITION





SAMPLE TYPES

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

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

	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. - CHUNK SAMPLE

ABBREVIATIONS

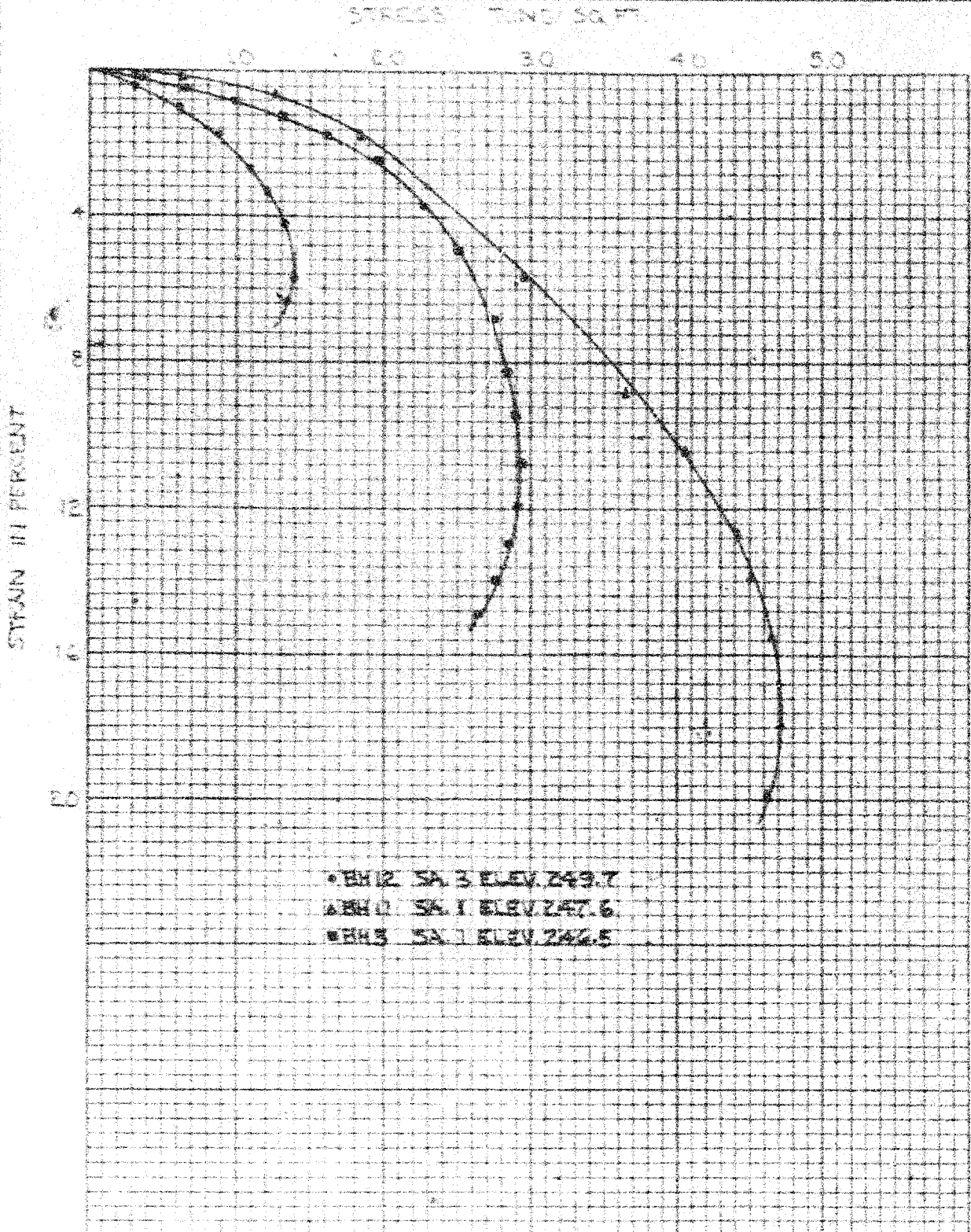
SOIL PROFILE				COMPRESSIVE STRENGTH TONS/SQ. FT. • UNCONFINED								SAMPLES			
ELEV. DEPTH	WATER CONDITIONS	DESCRIPTION	STRAT. PLOT ELEVATION SCALE	WATER CONTENT W%				OTHER TESTS	CONDITION	TYPE	NUMBER	PENETRATION RESISTANCE BLOWS/FT.			
				10	20	30	40						50	60	70
				DYNAMIC PENETRATION TEST BLOWS PER FOOT											
266.0 0.0		0.5 TOPSOIL GROUND LEVEL													
		STIFF TO VERY STIFF GREY BROWN SILTY CLAY	220												
250.0 16.0		SOFT TO FIRM DARK GREY SILTY CLAY	250					γ = 125							
232.0 34.0		STIFF TO VERY STIFF DARK GREY SILTY CLAY	240					γ = 132							
226.0 40.0		HARD REDDISH- BROWN CLAY TILL	230												
219.5 46.5		END OF HOLE	220												

APPENDIX II

FIGURES

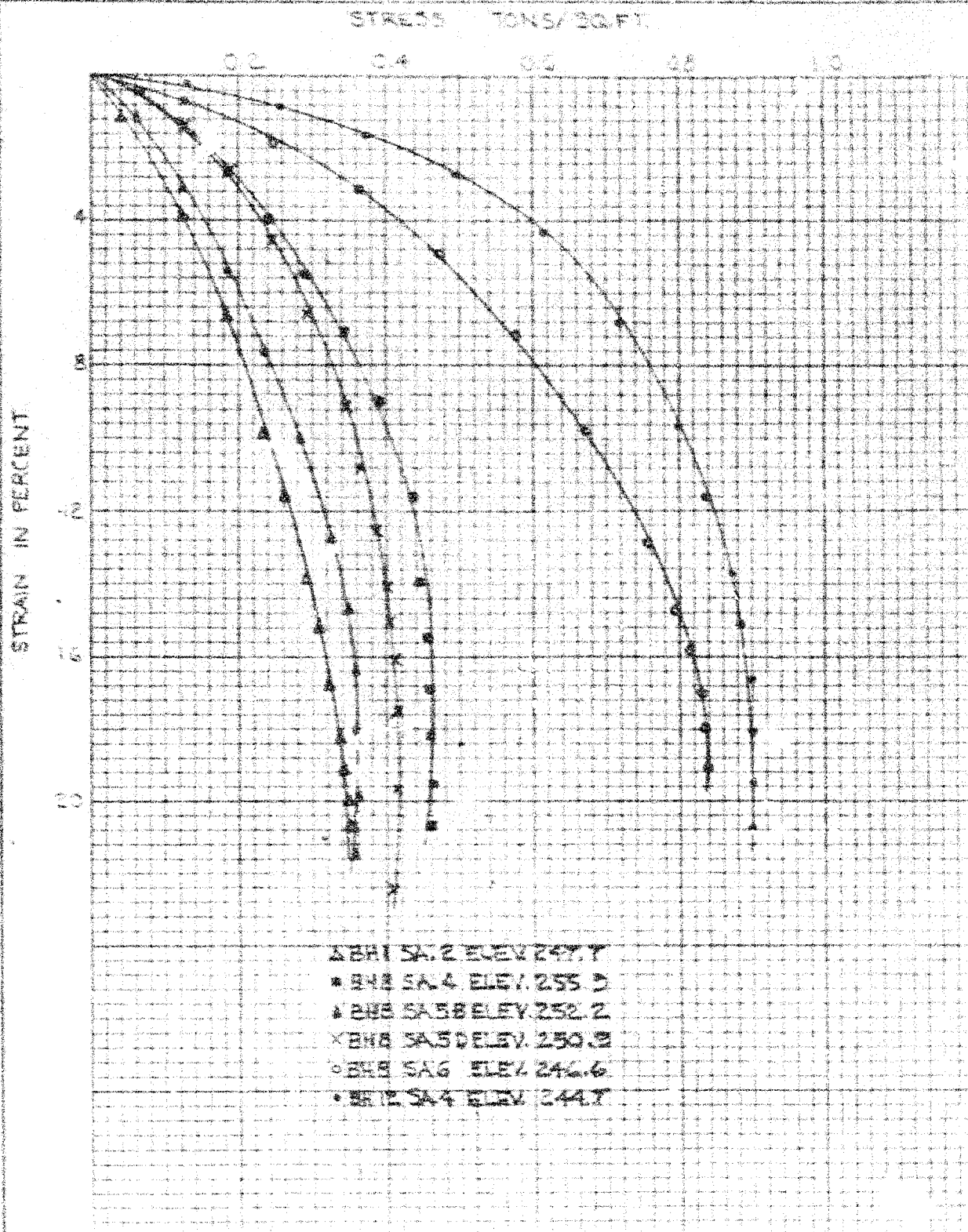
UNCONFINED COMPRESSION TESTS ON SILTY CLAY WITH SILT, SAND PEAT POCKETS TYPICAL STRESS-STRAIN CURVES

APPENDIX II
FIGURE 1
PROJECT SG623



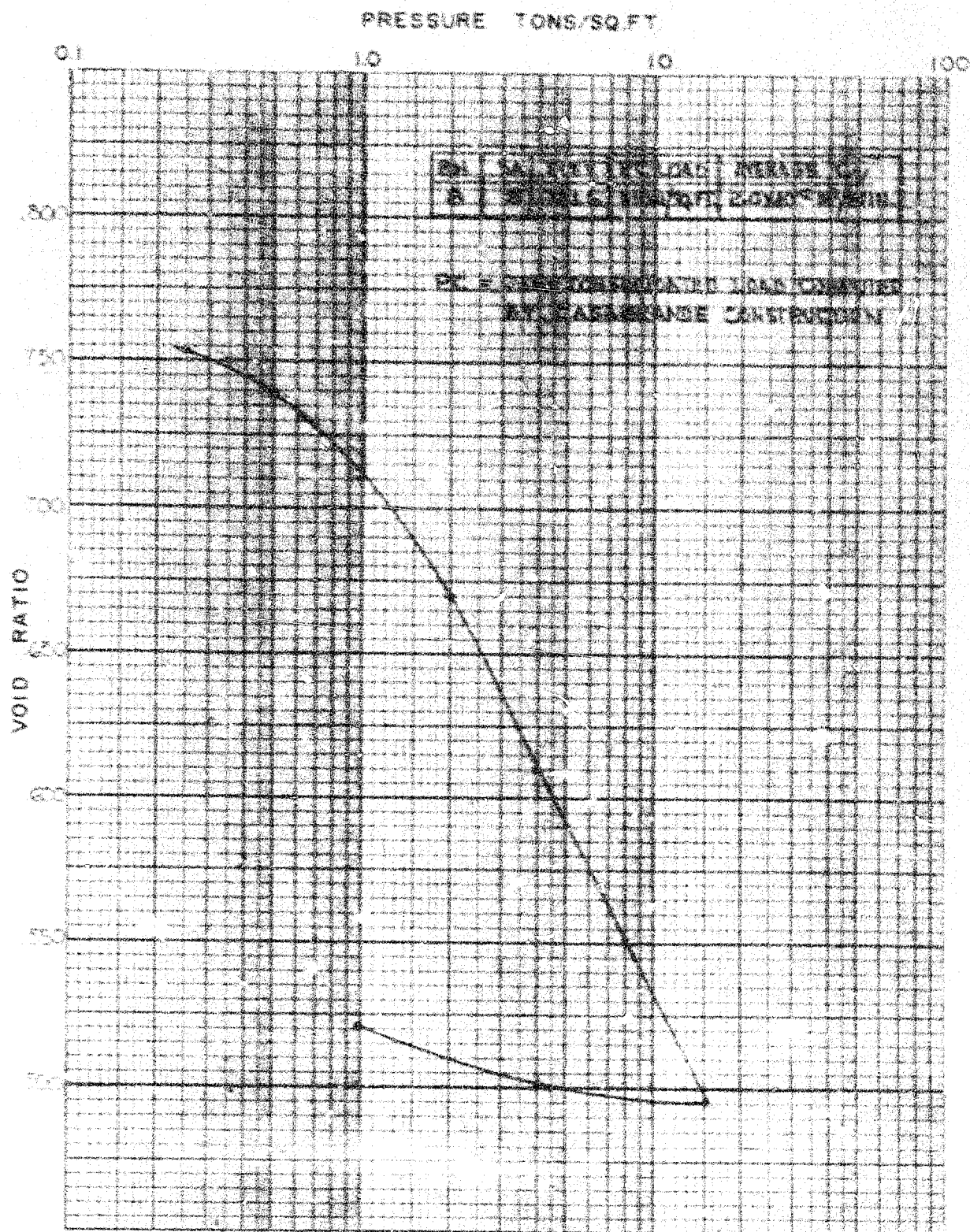
UNCONFINED COMPRESSION TESTS SOFT TO FIRM DARK GREY SILTY CLAY TYPICAL STRESS-STRAIN CURVES

APPENDIX II
FIGURE 2
PROJECT SC623



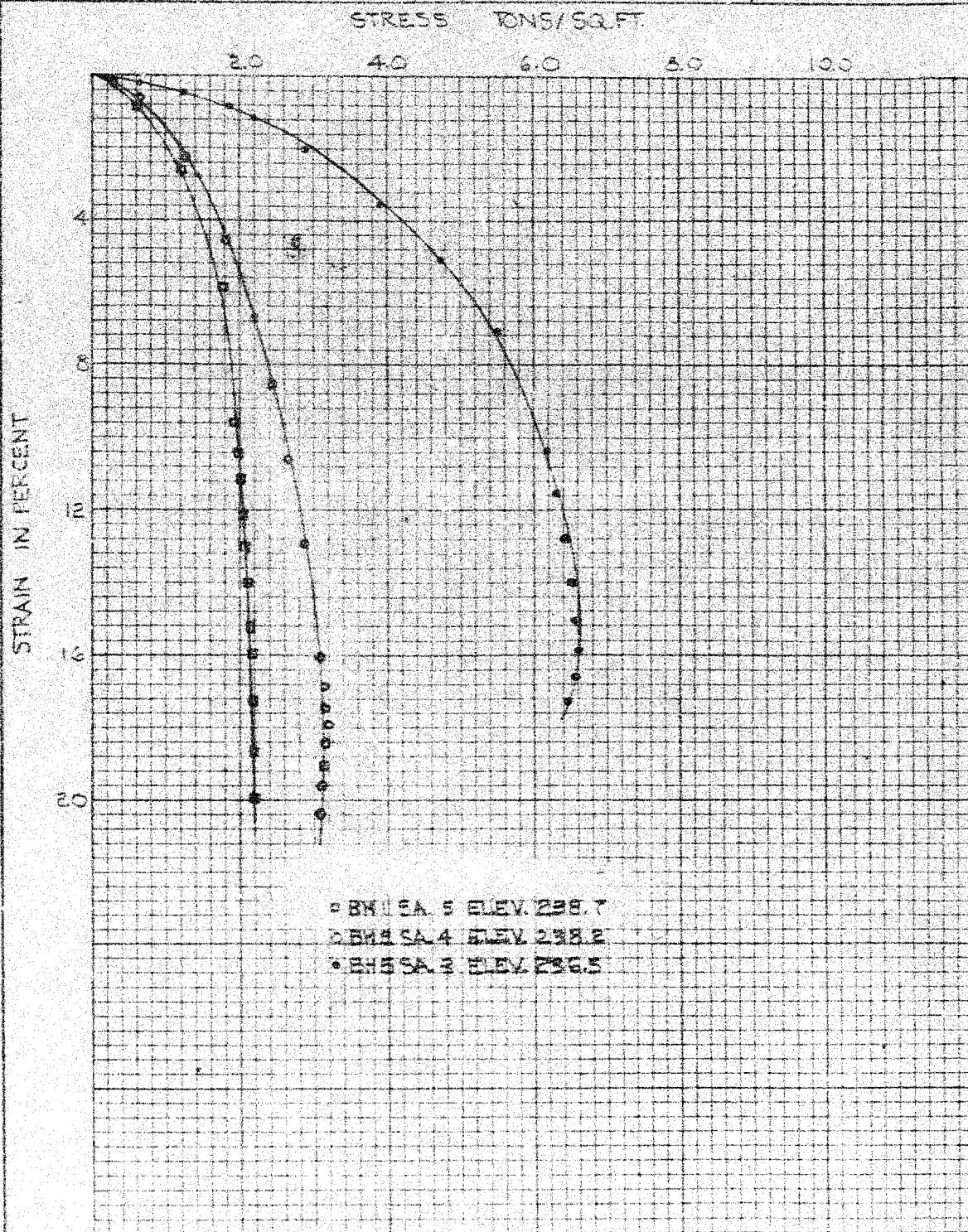
VOID RATIO-PRESSURE CURVES CONSOLIDATION TEST

APPENDIX II
FIGURE 3
PROJECT 54613



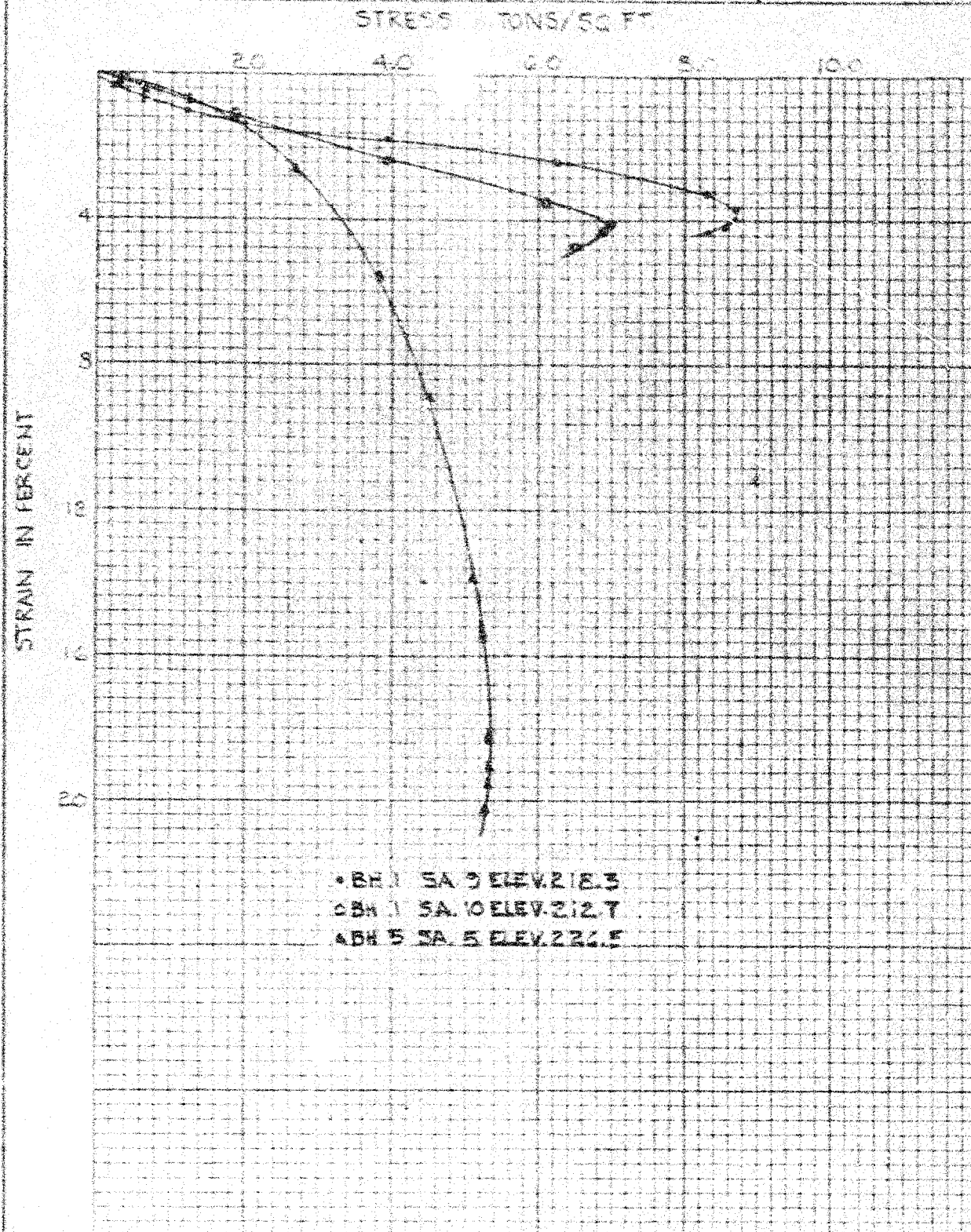
UNCONFINED COMPRESSION TESTS STIFF TO HARD DARK GREY SILTY CLAY TYPICAL STRESS-STRAIN CURVES

APPENDIX II
FIGURE 4
PROJECT S6623



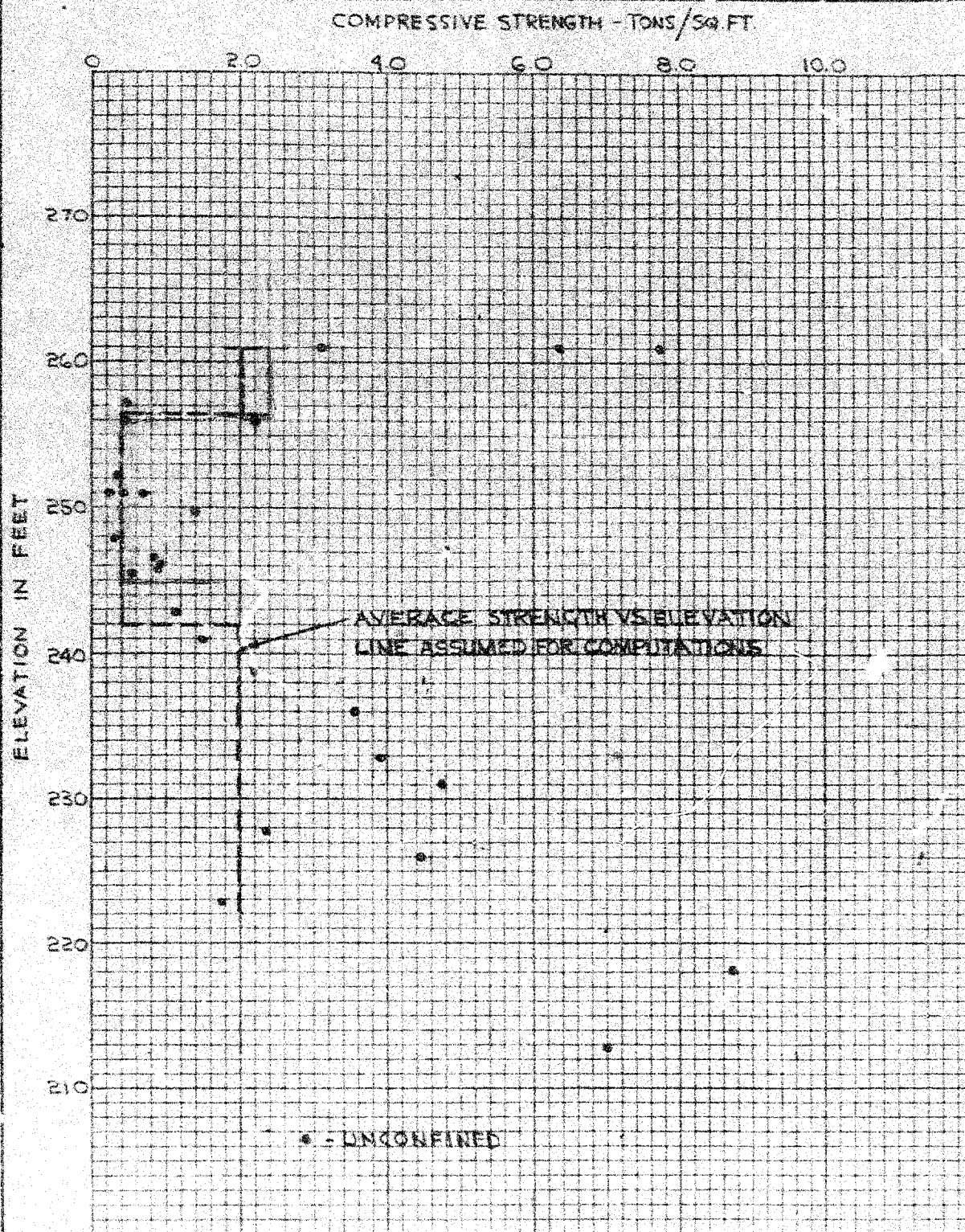
UNCONFINED COMPRESSION TESTS HARD REDDISH-BROWN CLAY TILL TYPICAL STRESS-STRAIN CURVES

APPENDIX II
FIGURE 5
PROJECT SCG23



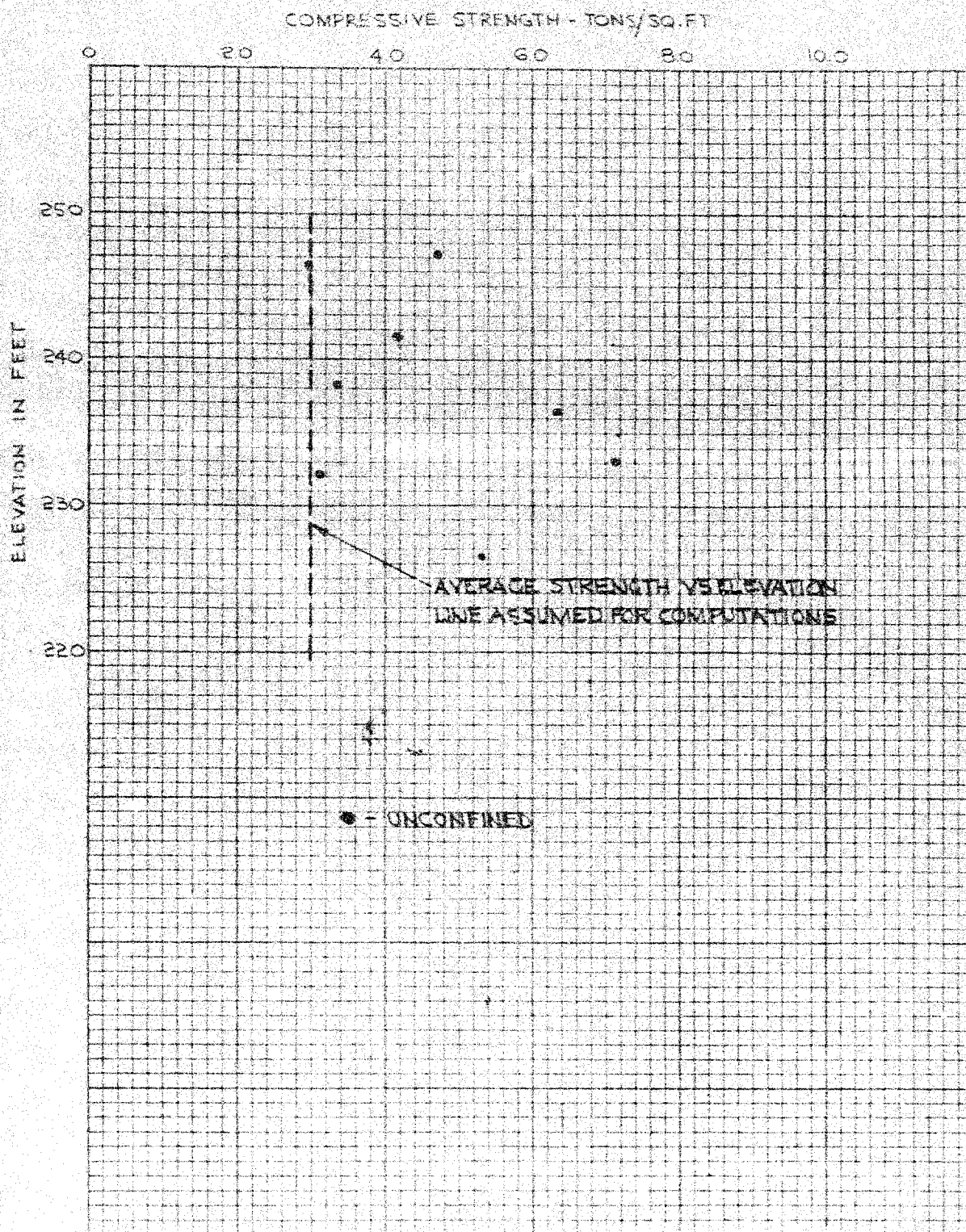
COMPRESSIVE STRENGTH VS ELEVATION
BOREHOLES 1, 8 AND 12 (AREA'S A, B)

APPENDIX II
FIGURE 6
PROJECT 56623



COMPRESSIVE STRENGTH VS ELEVATION BOREHOLES 2, 5 AND 11 (AREA C)

APPENDIX
FIGURE
PROJECT S6623



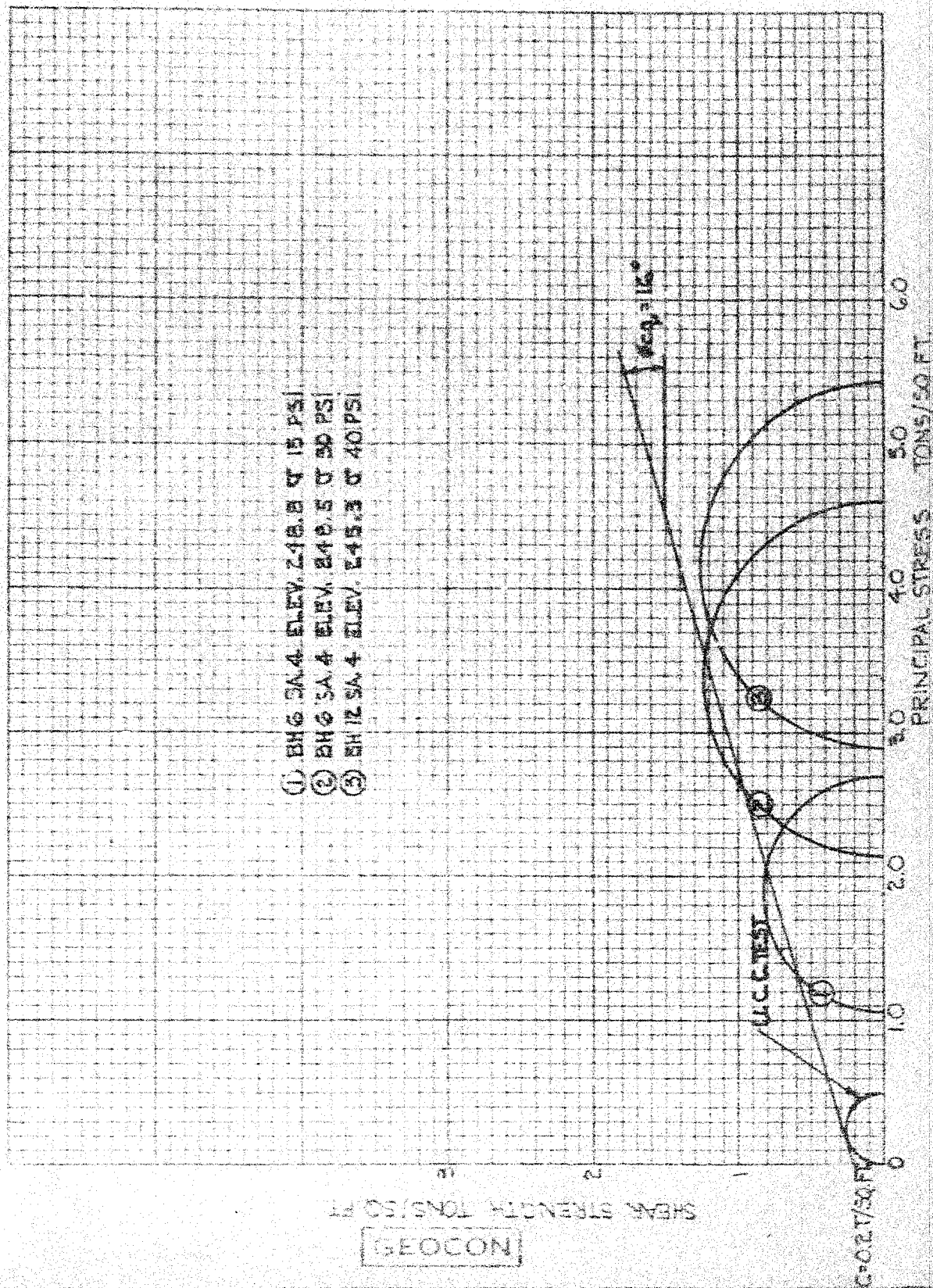
CONSOLIDATED QUICK TRIAXIAL TESTS

MOHR'S CIRCLES

APPENDIX II

FIGURE 5

PROJECT S6623



J. E. N. B. J.

QEW BE

S6935

REPORT

TO

DEPARTMENT OF HIGHWAYS, ONTARIO

TORONTO

ONTARIO

W.P. 10-57

ON

SOIL CONDITIONS

PROPOSED OVERPASS STRUCTURES

STONE CREEK ROTARY - QUEEN ELIZABETH WAY

HAMILTON

ONTARIO

Distribution:

- 4 copies - Department of Highways, Ontario,
Toronto, Ontario.
- 2 copies - Geocon Ltd,
Toronto, Ontario.

May 22nd, 1959

GEOCON

GEOCON LTD

HEAD OFFICE

180 VALLÉE ST., MONTREAL 18, QUEBEC

TELEPHONE UN. 6-7622

Rexdale, Ontario,
May 23rd, 1959.

DISTRICT OFFICES

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

1425 WEST PENDER ST.
VANCOUVER 5, B.C.
TEL. MU. 1-5926

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

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

Re: Additional Borings,
Proposed Overpass Structures,
Stoney Creek Rotary - Queen Elizabeth Way,
Hamilton, Ontario.

Dear Sirs:

This letter reports the results of the above work carried out in accordance with your verbal instructions and letter of authorization dated April 29th, 1959. The object of this work was to obtain additional information on the soil conditions previously outlined in our report 36623, dated May 5th, 1958.

The field work was carried out on May 6th and 7th, 1959 using power auger equipment. Three boreholes, 1A, 2A and 12A, were put down adjacent to the locations of boreholes 1, 3 and 12 from the previous investigation. The borings were carried through the soft to firm silty clay stratum and terminated in the lower stiff to hard silty clay stratum. Undisturbed soil samples were obtained and vane testing was carried out in the soft to firm clay stratum.

The results of the vane testing and laboratory triaxial tests are plotted on the detailed borehole logs in the Office Reports on Soil Exploration in Appendix I. The samples remaining after testing will be stored until December 1st, 1959 and will then be destroyed unless instructions are received to the contrary.

It may be seen that the results of these borings are in agreement with the results obtained in the previous investigation.

We believe that this letter report gives you the information that you requested. If, however, we can be of any further assistance, please do not hesitate to contact us.

Yours very truly,

GEOCON LTD



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

JLS/dw
36635

ST. JOHN'S

HALIFAX

MONTREAL

TORONTO

VANCOUVER

APPENDIX I

OFFICE REPORTS ON SOIL EXPLORATION

EXPLANATION OF THE FORM "OFFICE REPORT ON SOIL EXPLORATION"

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

ELEVATION AND DEPTH

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

WATER CONDITIONS

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

DESCRIPTION

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

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

STRATIGRAPHIC PLOT

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

ELEVATION SCALE

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

GRAPHS

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

OTHER TESTS

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

SAMPLES

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

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

GEOCON

GEOCON

OFFICE REPORT ON SOIL EXPLORATION

CONTRACT S 6335 BORING # 12 A DATUM D. H. O. CASING
 BORING DATE MAY 7, 1955 REPORT DATE MAY 8, 1955 COMPILED BY M.V. CHECKED BY J.H.
 SAMPLER HAMMEN WT. LBS. DROP INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN - LBS. ENERGY)

SAMPLE CONDITION



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

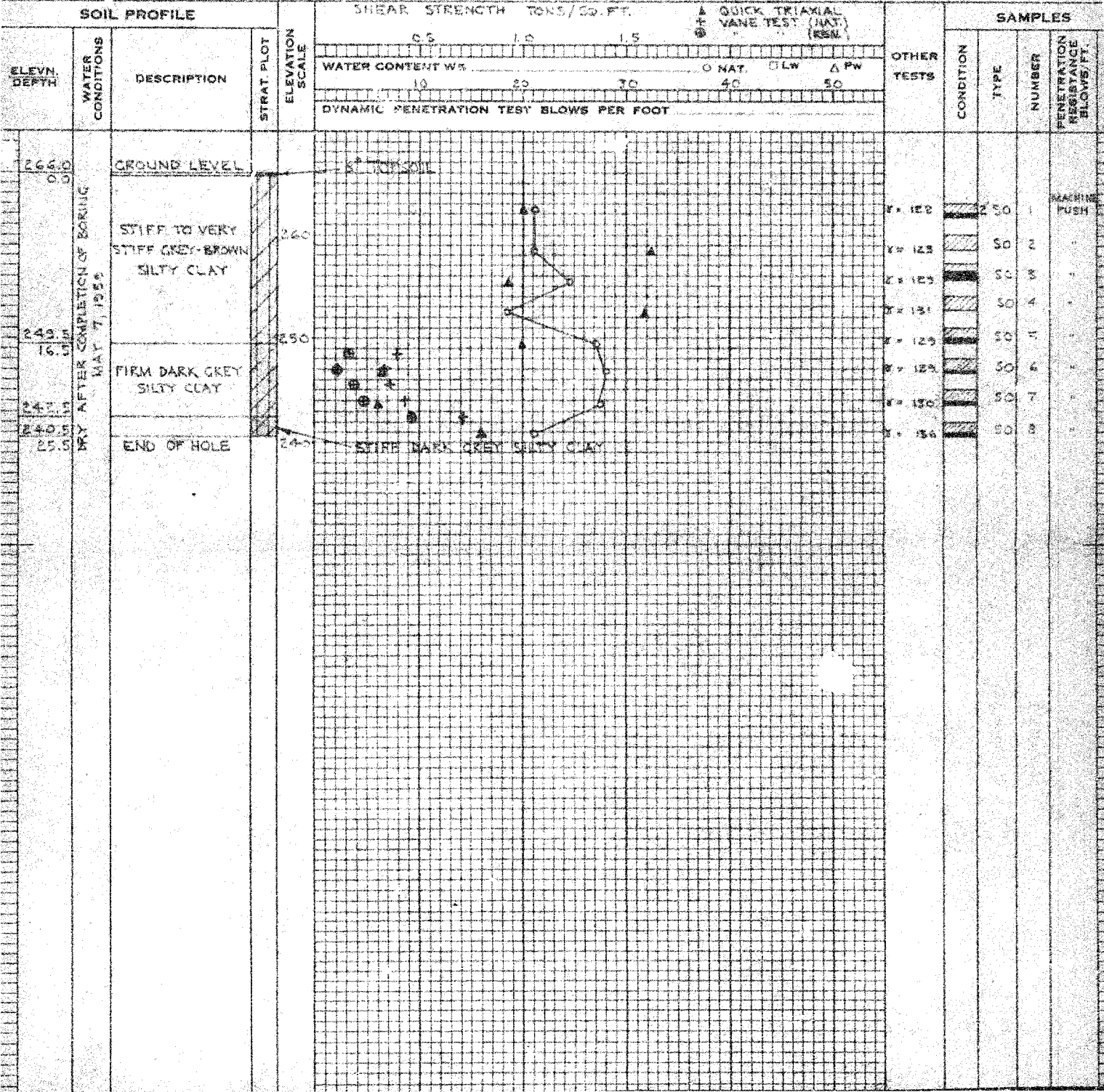
SAMPLE TYPES

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

ABBREVIATIONS

V - IN-SITU VANE TEST
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 Q - TRIAXIAL QUICK
 S - TRIAXIAL SLOW
 γ - WET UNIT WEIGHT
 K - PERMEABILITY
 C - CONSOLIDATION
 WL - WATER LEVEL IN CASIN
 WT - WATER TABLE IN SOIL

SOIL PROFILE



Geocros

30M4-15

STONE CREEK TRAFFIC CIRCLE

W.P. 10-57

LISTE 4

NOTE:

"Geocron" did the first investigation in 1958 and submitted a report (S 6623) dated May 5, 1958. Subsequently, in 1959 "Geocron" did an additional investigation to define the subsoil conditions beyond the limits investigated and reported in 1958. This additional report is dated Dec. 3, 1959 and carries no. S 6989.

During the time between the two above mentioned investigations "Geocron" has carried out three additional borings in order to check the shear strength of soft clay layer. This short ~~note~~ report (S 6935) is dated May 22, 1969.

All the above investigation refers to line 'C' or parried line 'D' of the A.E.W.

There doesn't seem to be any foundation investigation information available of the immediate surrounding area.

Jan 12, 1967

Alsterning



ONTARIO
DEPARTMENT OF HIGHWAYS

Toronto
September 24, 1959.

MEMORANDUM TO:

Mr. L. G. Soderman,
Principal Soils & Foundation Eng.,
Downsview, Ontario.

RE: W.P. 10-57,
Stoney Creek Traffic Circle,
Q.E.W., District #4.

This is to confirm a request for further soil investigation made by Mr. B. S. Richardson in a memo of September 18, 1959. The problems were set out in this memo and the investigation left to your discretion.

I am enclosing a preliminary print of the structure. The questions in connection with the 44' high abutment at the East end are urgent as the design is well under way.

J. C. McAllister

JCMcA:go

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

*Bridge office
sent 11-*



ONTARIO

DEPARTMENT OF HIGHWAYS

Memo to Mr. L. G. Soderman, Date Sept. 18th, 1959.
Room 121, Downsview Ave., Subject Stoney Creek Traffic Circle
O'Pass & C.N.R. J'Head,
From B.S. Richardson, Bridge Design Office W.P. 10-57. Soil Reports
No. BA 735 & BA 735A.

Herewith is the preliminary drawing of the above structure.

The original soils investigation BA735 was carried out by Geoson Ltd. in May 1958. A supplemental investigation BA735A was completed by them in May 1959 and your section reviewed their conclusions, according to your memo dated June 24th, 1959.

Your attention is directed to the following points and your observations on these points and on the preliminary details would be appreciated.

- 1) The height of the embankment at the East end of the structure is 44 ft. Mr. J.L. Seychuk of Geoson Ltd., during a telephone conversation, said that this would be stable with 2:1 slopes.
- 2) The height of the embankment at the West end of the structure is 20 ft. (aver.) This is in accordance with the recommendations of the original report.
- 3) The first and the last spans of the structure are continuous. This arrangement was adopted on the assumption that the abutments can be prevented from settling more than $\frac{1}{4}$ " relative to the first or last piers. The total negative skin friction load likely to be encountered on the abutment piles is required in order to facilitate the calculation of the available capacity.

If, in your opinion, the settlement of the abutments cannot be controlled, simply supported end spans will be used. This arrangement is to be avoided, if possible.

- 4) Although 12 B.P.53 piles have hitherto been assumed to be most suitable, it is possible that 12" diam. concrete filled steel tube piles would provide a more positive refusal in the till, due to the larger end area. The ends are normally closed with welded steel plates a little larger in diameter than the pile itself. These piles do not perform well when boulders are encountered but can be made to seat well on softer bedrocks.

- 5) In boreholes 2, 10 & 11, either bedrock or boulders were encountered. If bedrock, it would appear to slope up sharply at the East end of the structure. If you consider that bedrock could be at these elevations, perhaps it should be located more accurately for the full length of the bridge and if economic, all piles should be driven to bedrock.
- 6) Further boreholes have been requested through Mr. S. McCombie. The locations of these are left to your discretion.

B. S. Richardson

B. S. Richardson,
Bridge Design Office

BSR/r

Mr. A. M. Toye,

June 24, 1959.

Bridge Engineer.

Re: Stony Creek Rotary -
Overpass Structures -
Q.E.W. - W.P. 10-57

Materials & Research Section.

Attention: Mr. S. McCombie.

Enclosed herewith are two copies of a supplemental soils investigation recently completed by Gecon, Limited, in connection with the overpass structures at Stony Creek Rotary. This additional work was requested in order to confirm the results of a preliminary investigation previously carried out by Gecon. You will recall that the conclusions in their initial report were that stage construction would be necessary if embankment fill heights of the order of 28 feet were to be designed.

The conclusions of the initial analysis carried out by Gecon, have been reviewed by our Section, using the soil properties as given in their initial report, and the supplemental report attached. We hereby confirm the initial conclusion that if embankment fills are to be constructed, stage construction will have to be undertaken. A detailed instrumentation program and very close construction control would be required to assure successful embankment construction, using stage loading.

As we discussed with your Mr. McAllister, the alternative of a trestle structure appears to be a more desirable design in view of the difficulties that would be encountered with stage construction procedures. We will, upon your request, carry out a more detailed analysis, confirming stage heights and construction periods, if you require additional data on the stage construction proposal.

L. G. Soderman

LGS/WdeP
Encls. (2)

L. G. Soderman,
PRINCIPAL SOILS & FOUNDATIONS ENGINEER

cc: Mr. A. M. Toye
Mr. J. McAllister
Foundation Section
Gen. Files

GEOCOR LTD

HEAD OFFICE

180 VALLEE ST., MONTREAL 18, QUEBEC
TELEPHONE UN. 6-7501

DISTRICT OFFICES

14 HAAS ROAD
REXDALE, TORONTO, ONT.
TEL. CH. 4-8841

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

Rexdale, Ontario,
May 22nd, 1959.

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

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

RE: FOUNDATION INVESTIGATION
STONE CREEK ROTARY - Q.E.W.
W.P. 10-57 - DISTRICT NO. 4

Dear Sirs:

Further to the discussion between your Mr. L. G. Soderman and our Mr. Milligan, and your letter of April 23th, 1959 authorizing us to carry out additional work at the above site, we are pleased to confirm the arrangements made and to submit, as requested, our rates for the above work.

As specified in your letter, it is understood that three detailed boreholes are to be put down adjacent to boreholes 1, 8 and 12 put down in a previous investigation. Undisturbed soil samples are to be obtained at depth intervals not in excess of 3 feet and in-situ vane testing is to be carried out in the soft to firm silty clay stratum. The borings are to be terminated at the contact of the relatively stiff layer underlying the soft stratum of silty clay. Quick triaxial strength tests are to be carried out on the soil samples obtained.

It is further understood that the results of the above work are to be presented in a factual letter report.

We are pleased to carry out this work for you and our charges will be in accordance with the attached Schedule of Rates I and II. Please sign and return one copy of this proposal as our authority to invoice you accordingly.

Yours very truly,

GEOCOR LTD

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

JLS/dw
3-6935

ACCEPTED DATE.....1959

SCHEDULE OF RATES I

- Item 1. The provision and operation of a rented Pandrill power auger with all necessary tools and supplies and manned by a two-man crew, will be charged....AT COST PLUS TEN PERCENT
- Item 2. For the provision of soil sampling equipment in 2 inch size.....\$10.00 per calendar day of operation
- Item 3. For the provision of vane testing equipment.....\$10.00 per calendar day of operation
- Item 4. For soil testing carried out in our laboratory.
(See attached Schedule of Rates for Laboratory Tests)

TERMS OF PAYMENT

Progress invoices will be submitted monthly for the full amount of field and office work completed less a retention of 5% and payment will be made within 15 days of receipt of our invoice. Payment in full including all retentions will be made within 30 days of completion of our work.

NOTE

Under Terms of Payment, if the work is commenced and completed in the same month, a final invoice will be submitted at the end of that month.

Department of Highways, Ontario,
Foundation Investigation,
Stoney Creek Rotary - N.E.W.,
W.P. 10-57 - District No. 4.
5-6935

SCHEDULE OF RATES II

Item 1. For the provision of engineering services as required.

Senior Soils Engineer on report writing and
computations.....\$ 75.00 per day
Junior Soils Engineer on field supervision,
Technician or Draftsman.....\$ 50.00 per day

where the services of the Chief Engineer and/or
District Engineer are required on consultation
or to review the work, the fees for their services
will be as follows:

Chief Engineer.....\$125.00 per day
District Engineer.....\$100.00 per day

Item 2. All legitimate expenses incurred by our field
engineer, including transportation to and from
the site, daily living and travelling expenses,
long distance telephone calls and telegram charges
if required, will be charged.....AT COST

NOTE

Under Item 1, the daily fees for field supervision cover for a calendar working
day. The daily fees for office engineering services cover for a 7½ hour day.
Where only part of a day is worked or in excess of 7½ hours in any one day, the
fees will be pro-rated on an hourly basis.

Department of Highways, Ontario,
Foundation Investigation,
Stoney Creek Rotary - W.E.S.,
S.P. 10-57 - District No. 4.
S-6935

SCHEDULE OF RATES FOR LABORATORY TESTS

	<u>UNIT RATE</u>
EXTRUSION AND IDENTIFICATION:	
Extrusion of tube sample, labelling, and storing.	\$ 2.50
Visual identification and description.	2.50
CONSISTENCY LIMITS:	
Natural water content.	1.50
Plasticity index, liquid and plastic limits.	7.50
Shrinkage limit.	4.00
GRAIN SIZE ANALYSIS:	
Distribution curve included.	
a) Sieve.	5.00
b) Hydrometer.	15.00
c) Sieve and hydrometer.	20.00
STRENGTH AND COMPRESSIBILITY TESTS:	
Unconfined compression:	
a) Maximum stress only reported.	7.50
b) Stress-strain curve included.	10.00
Triaxial compression; stress-strain curve included:	
a) Quick.	15.00
b) Consolidated quick.	25.00
c) Slow.	30.00
Consolidation:	
a) Loading, initial cycle; pressure-void ratio curve included.	50.00
b) Rebound and reloading, per cycle.	10.00
UNIT WEIGHT:	
a) Sample of geometrical shape.	2.50
b) Sample of irregular shape.	7.50
PERMEABILITY:	
a) Falling head.	50.00
b) Constant head.	30.00
COMPACTION TEST:	
Moisture-density curve included.	50.00
RELATIVE DENSITY:	
Cohesionless soils.	10.00
SPECIFIC GRAVITY.	5.00

April 29, 1959.

Geoson, Limited,
14 Haas Road,
REEDALE, Ontario.

Attention: Mr. T. Milligan.

Re: Foundation Investigation -
Stanley Creek Rotary - Q.E.W.,
W.P. 10-57 -- District #4.

Dear Sir:-

Please consider this your authority to carry out additional field work at the above noted structure location. As outlined to you in our discussion of today's date regarding the necessity for this additional work, the scope of the investigation is to consist of:-

- (1) Three (3) detailed sampled borings carried out adjacent to borings numbered: 1, 8 and 12 (See Drawing S 6623-1 Geoson, Ltd.).
- (2) Undisturbed samples should be obtained at depth intervals not in excess of 3 feet. In-situ rotating vane tests are to be carried out in the soft silty clay stratum previously defined in your Foundation Report. If it is necessary to increase the sampling intervals within this stratum to allow the carrying out of vane tests, this can be done at your discretion.
- (3) Borings can be terminated at the contact of the relatively stiff layer underlying the previously noted soft stratum of silty clay.
- (4) Laboratory strength testing should be carried out using undrained unconsolidated triaxial tests with lateral pressures equal to existing total overburden pressures.

cont'd. /2 ...

- (5) If, after completion of the field work, you consider it necessary to supplement previously reported consolidation tests, we request that you discuss this with us prior to undertaking these, or other special tests.

As we pointed out in our discussion with you, we are presently employing Johnston Drilling Company for Pendrill work in the area of this structure. We are making arrangements for Johnston to have this drilling equipment available to you immediately we are finished with it. We expect this date to be Tuesday of next week. If you prefer to employ your own machine or another contractor, would you please advise us of this.

As you know, we do not have on file, a general outline indicating per diem rates which could apply to the work requested of you, in this letter. Perhaps you would be good enough to submit per diem rates which you would use for invoicing us for the field, laboratory and engineering services required to carry out this work. It is understood that the field work will be continuously supervised by a field engineer.

Yours very truly,

LGS/MieF

for CS
A. Rutka,
ACTING NAT'L. & RESEARCH ENGR.

cc: Messrs. E. McCombie
H. B. Richardson
W. F. Weber
Foundation Office ✓
N. D. Smith
File

Mr. A. H. Ioye,

April 29, 1959.

Bridge Engineer.

Re: Foundation Investigation -
Stoney Creek Rotary - S.E.W.
M.P. 10-57, District #4.

Materials & Research Section.

Attention: Mr. S. McCornie.

In response to your request, we have reviewed the subsoil conditions defined by Geoccon in their Foundation Report, carried out at this site. With reference to the contents of this report, they have defined a stratum of soft clay, generally 10 feet in thickness, which was encountered at depths from 10 to 20 feet below the existing ground surface. They have carried out stability analyses based on strength values obtained from the results of unconfined compression tests. A strength value of the order of 400 lbs. per sq. ft. was considered representative for this soft stratum, based on the limited number of strength tests which they carried out. In many of the samples obtained from this layer, the driving resistance noted on the borehole logs is not consistent with the shear strength values noted in the report. Furthermore, the stress-strain curves presented in their report indicate that the samples show a considerable degree of disturbance.

In view of the inconclusive data presented in this report and the consequences of accepting this data as a basis of design, we deem it advisable to carry out an additional three borings to accurately evaluate the strength properties of this reported layer.

We have discussed the contents of this report with Mr. Milligan of Geoccon, and he concurs with the necessity of obtaining more specific information from this site. We have discussed the extent of a sampling and testing program in detail with Mr. Milligan, and Geoccon have been retained by us to do the additional work required. It is expected that the field and lab. work will be completed in a period of 2 to 3 weeks, and immediately this data is available, we will forward it to your office.

The necessity of carrying out more detailed sampling and testing is readily apparent and one realizes that shear strength of 800 lbs. per sq. ft. would be sufficient for us to adopt single stage construction for the embankments required. The present value of shear strength reported as 400, does not allow this.

LGS/Wdof

cc: Messrs. R. Trepaskes

R. McMillan

R. E. Richardson

P. F. Weber

W. D. Smith

- Files

L. G. Soderman,

PRINCIPAL SOILS & FOUNDATION ENGINEER.

L. G. Soderman

Mr. A. M. Toye,

April 29, 1959.

Bridge Engineer.

Re: Foundation Investigation -
Stanley Creek Rotary - Q.B.W.
W.P. 10-57, District #4.

Materials & Research Section.

Attention: Mr. S. McCombie.

In response to your request, we have reviewed the subsoil conditions defined by Geeson in their Foundation Report, carried out at this site. With reference to the contents of this report, they have defined a stratum of soft clay, generally 10 feet in thickness, which was encountered at depths from 10 to 20 feet below the existing ground surface. They have carried out stability analyses based on strength values obtained from the results of unconfined compression tests. A strength value of the order of 400 lbs. per sq. ft. was considered representative for this soft stratum, based on the limited number of strength tests which they carried out. In many of the samples obtained from this layer, the driving resistance noted on the borehole logs is not consistent with the shear strength values noted in the report. Furthermore, the stress-strain curves presented in their report indicate that the samples show a considerable degree of disturbance.

In view of the inconclusive data presented in this report and the consequences of accepting this data as a basis of design, we deem it advisable to carry out an additional three borings to accurately evaluate the strength properties of this reported layer.

We have discussed the contents of this report with Mr. Milligan of Geeson, and he concurs with the necessity of obtaining more specific information from this site. We have discussed the extent of a sampling and testing program in detail with Mr. Milligan, and Geeson have been retained by us to do the additional work required. It is expected that the field and lab. work will be completed in a period of 2 to 3 weeks, and immediately this data is available, we will forward it to your office.

The necessity of carrying out more detailed sampling and testing is readily apparent and one realizes that shear strength of 300 lbs. per sq. ft. would be sufficient for us to adopt single stage construction for the embankments required. The present value of shear strength reported as 400, does not allow this.

LCS/Miaf

cc: Messrs. H. Fregaskes

H. McMillan

R. E. Richardson

P. F. Weber

E. D. Smith - Files

L. C. Soderman,

PRINCIPAL SOILS & FOUNDATION ENGINEER.

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

December 10, 1959.
FOUNDATION REPORT by
Gecon, Limited.

Attention: Mr. S. McCombie.

Re: Stoney Creek Traffic Circle
Overpass and C.R.R. Overhead
W.P. 10-57 -- District #4.

Enclosed herewith is a report on a foundation investigation carried out by Gecon at the above site.

Gecon have incorporated some alternatives with respect to minimizing differential settlement between the structure and backfill. Their comments are based upon a discussion which we had with Dr. Casagrande while in Boston, in connection with the Homer Skyway.

When you have had an opportunity to review the comments contained in this report, we would be pleased to discuss any points with you in regard to either the embankment problem or means of footing support.

L. G. Soderman

LGS/MCef
Encl.

L. G. Soderman,
PRINCIPAL SOILS & FOUNDATIONS ENGINEER

cc: Messrs. A. M. Toye (2)
E. A. Tregarves
D. C. Lamsay
H. W. Farren
R. H. Richardson
R. Weber

Foundation Section
Gen. Files.

59-F-231

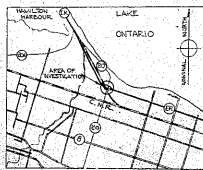
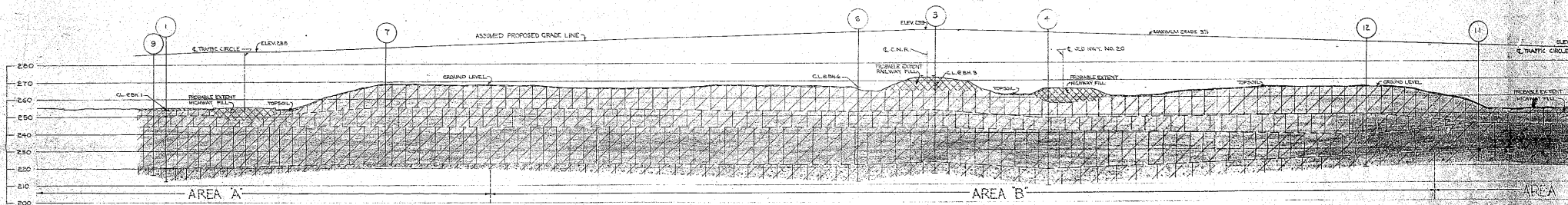
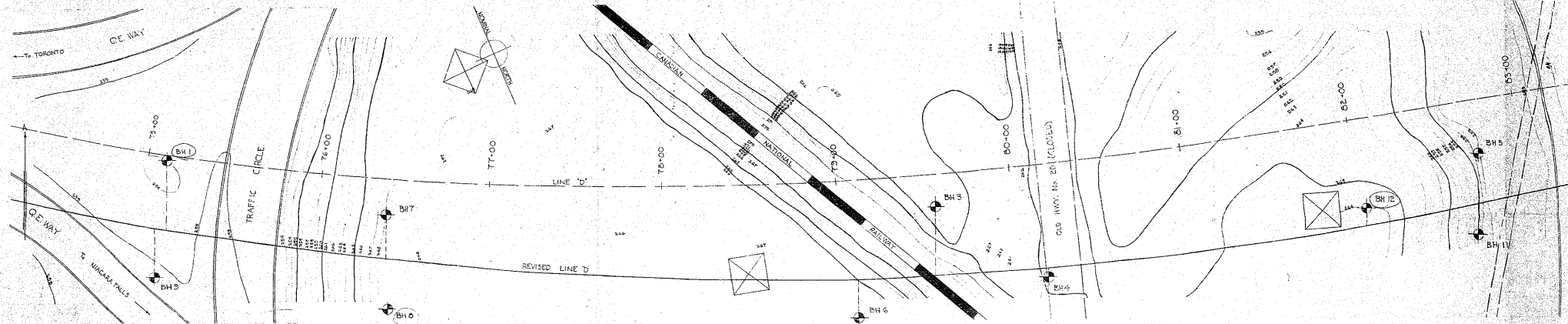
W.P. 10-57

C.N.R. & STONEY CR.

TRAFFIC CIRCLE

30M4-15

PLAN
SCALE 1" = 500'



DESIGN DATA					
SOIL	STRATIGRAPHY	DEPTH C-DEGREES	CONVERSION C-DEGREES	NET WEIGHT C-DEGREES	EMBEDDED NET WEIGHT C-DEGREES
COMPACT GRANULAR FILL		15 ASSUMED	150 ASSUMED		
STIFF TO HARD GREY BROWN SILTY CLAY		—	8000	150	6.5
LOOSE TO COMPACT SILTY SAND		30	—	110	4.8
SOFT TO FIRM DARK GREY SILTY CLAY		—	400	127	6.5
STIFF TO HARD DARK GREY SILTY CLAY		—	8000	135	7.7
HARD REDDISH BROWN CLAY TILL		—	2000	14.5	5.9

SECTION A-A
SCALE 1" = 20'

STRATIGRAPHY

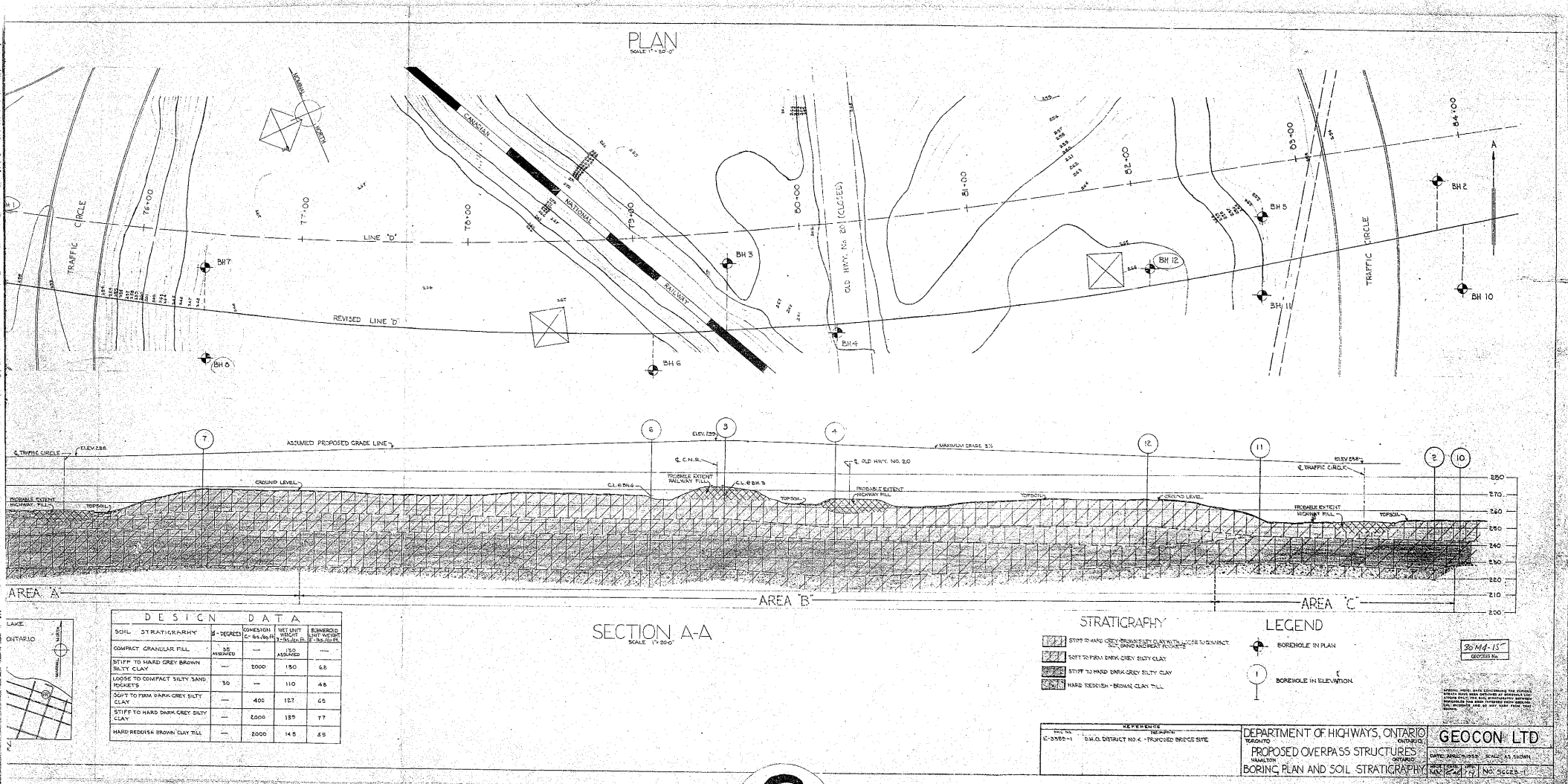
- STIFF TO HARD GREY BROWN SILTY CLAY WITH LOTS OF SAND
- SOFT TO FIRM DARK GREY SILTY CLAY
- STIFF TO HARD DARK GREY SILTY CLAY
- HARD REDDISH BROWN CLAY TILL

LEGEND

- BORING IN PLAN
- WATER TABLE ELEVATION

DEPARTMENT OF HIGHWAYS
PROPOSED OVERPASS
BORING PLAN AND SECTION

1



2