

MEMORANDUM

TO: Mr. B. R. Davis,  
Bridge Engineer,  
Bridge Division,  
Admin. Bldg.

FROM: Foundation Section,  
Materials & Testing Div.,  
Room 107, Lab. Bldg.

Attention: Mr. S. McComble

DATE: September 3, 1968

OUR FILE REF.

IN REPLY TO

SUBJECT:

FOUNDATION INVESTIGATION REPORT  
For  
Eastbound Lane and Westbound Lane  
Structure at the Crossing of  
McEwen Creek and Proposed Hwy. #417  
District No. 9 (Ottawa)  
W.J. 68-F-53 -- W.P. 34-66-01

Attached, we are forwarding to you, our detailed foundation investigation report on the subsoil conditions existing at the above structure site.

We believe that the factual data and recommendations contained therein, will prove adequate for your design requirements. Should additional information be required, please do not hesitate to contact our Office.

AGS/MdeF  
Attach.

*A. G. Stermac*  
A. G. Stermac  
PRINCIPAL FOUNDATION ENGINEER

cc: Messrs. B. R. Davis (2)  
H. A. Tregaskes  
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G. Scott  
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Foundations Files  
Gen. Files

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FOUNDATION INVESTIGATION REPORT  
For  
Eastbound Lane and Westbound Lane  
Structure at the Crossing of  
McEwen Creek and Proposed Hwy. #417  
District No. 9 (Ottawa)  
W.J. 68-P-53 -- W.P. 34-66-01

1. INTRODUCTION:

The Foundation Section was requested to carry out an investigation at the proposed crossing of McEwen Creek and Hwy. #417; the site is located about 9 miles southeast of Ottawa, in the Twp. of Gloucester, County of Carleton. The request was contained in a memo from the Kingston Bridge Location Section (Mr. G. Scott, Regional Bridge Location Engineer), dated June 14, 1968. An investigation was subsequently carried out by this Section to determine the subsoil and groundwater conditions at this site.

This report contains the results of the investigation, together with the recommendations pertaining to the foundations of the proposed structure as well as the stability of the approach embankments.

2. DESCRIPTION OF THE SITE AND GEOLOGY:

The site is located about one-half mile south of Russell Road. The closest settlement is Ramsayville, which is about 3/4 of a mile to the north. McEwen Creek, in the vicinity of the site, meanders along the floor of a broad flat valley. The valley is approximately 100 feet wide and 15 feet deep with the natural side slopes standing at between 2:1 and 3:1. The creek is about 3 to 4 feet deep with the water level being at approximately elevation 241. The terrain, which is grass-covered and in places lightly wooded, is being utilized as pasture land.

2. DESCRIPTION OF THE SITE AND GEOLOGY: (cont'd.) ...

Physiographically, the site is situated on the northern edge of the area known as "The Russell and Prescott Sand Plains". In this area a sand mantle, 10 to 15 feet in thickness, overlies an extensive deposit of marine clay. The sand is of deltaic origin built up by the Ottawa River and its northern tributaries during the geologic period when the area was inundated by the Champlain Sea. In the area the base of the clay often extends below elevation 180. The clay stratum is underlain by a glacial till which, in turn, is underlain by grey and black shale bedrock of the Lorraine formation, Ordovician period.

Most of the area lies within the drainage basin of the Ottawa River.

3. FIELD AND LABORATORY WORK:

Four sampled boreholes, with accompanying dynamic cone penetration tests, were put down during the course of the recent field investigation. The borings were advanced by means of a conventional diamond drill rig adapted for soil sampling purposes.

Samples of the surficial sand and the lower glacial till were obtained, at specified intervals, in a 2" O.D. split-spoon sampler, which was hammered into the soil in accordance with the specifications for the Standard Penetration Test. The same method was used to advance the dynamic cone penetration tests. The cohesive overburden was sampled with 2" I.D. Shelby tubes, which were pushed manually into the soil. In an effort to reduce the degree of disturbance, most of the Shelby tube samples were advanced by means of a piston technique. In addition, field vane tests were carried out to determine the undrained shear strength of the clay stratum. Bedrock was proven in 2 of the borings by obtaining AXT size rock core samples.

The groundwater level conditions across the site were determined by installing two sealed piezometers in one of the boreholes. This information was supplemented by recording the

cont'd. /3 ...

3. FIELD AND LABORATORY WORK: (cont'd.) ...

water level in the open holes at the remaining boring locations.

The locations and elevations of all the borings were surveyed in the field by personnel from the Kingston Regional Engineering Surveys Section, and are shown on Drawings 68-P-53A (E.B.L.) and 68-P-53B (W.B.L.), together with the estimated stratigraphical profile across the site.

All samples were subjected to a careful visual examination in the field and subsequently in the laboratory. Following this inspection, laboratory tests were carried out on certain samples to determine the engineering properties of the various soil types, namely:

Bulk Densities  
Natural Moisture Contents  
Grain-Size Distributions  
Atterberg Limits  
Undrained Shear Strengths  
Consolidation Characteristics

On completion of these tests, the various soil samples were classified as to type and consistency, or relative density in accordance with the Unified Soil Classification System - (Oct. 1963).

The results of the laboratory testing are plotted on the Record of Borelog sheets and summarized in the Figures, all contained in Appendix I of this report.

4. SOIL TYPES AND SOIL CONDITIONS:

4.1) General:

The valley floor is surficially covered by a layer of clayey silt about 3 feet thick, while on the valley banks the cover is a 1 to 4 ft. thick deposit of loose sand. The surficial cover is underlain by the predominant overburden stratum across the

cont'd. /4 ...

4. SOIL TYPES AND SOIL CONDITIONS: (cont'd.) ...

4.1) General: (cont'd.) ...

site, composed of a firm to very stiff sensitive clay to silty clay extending down to about elevation 180 to 187 - i.e., some 59 to 78 feet in depth. The clay stratum is underlain by a firm to hard glacial till composed of clayey silt to silt with sand and some gravel, which, in turn, is underlain by shale bedrock.

The boundaries between the various deposits, as determined at the boring locations, are shown on the accompanying Borehole sheets. The stratigraphical sections, inferred from this data, are shown on Drawings 68-F-53A and 68-F-53B.

From ground surface downwards, the various soil types encountered are described as follows:

4.2) Surficial Deposits:

A loose deposit of brown sand ranging from 1 to 4 feet in thickness was encountered in those borings put down through the valley bank (B.H.'s #3 and 4). A grain-size distribution curve for a sample of the sand is plotted on Figure #3, located in the Appendix of this report.

In the borings put down through the valley floor (B.H.'s #1 and 2), a surficial layer composed of soft grey-brown clayey silt with some sand was encountered. The thickness of this layer is approximately 3 feet.

4.3) Sensitive Clay to Silty Clay:

Directly underlying the surficial cover is the predominant overburden stratum across the site, a sensitive marine clay to silty clay, with occasional inclusions of organic matter. The thickness of this stratum ranges from 59 ft. to 78 ft. - i.e., it extends down to between elevation 180 and 187. In the lower 10 to 18 ft. of the stratum, random seams and layers of sand and silt, with the thickness of the individual layers varying from 1/2 inch to 6 inches, were encountered. The granular seams and layers were

cont'd. /4A...

4. SOIL TYPES AND SOIL CONDITIONS: (cont'd.) ...

4.3) Sensitive Clay to Silty Clay: (cont'd.) ...

not found to be continuous across the site; it is inferred, therefore, that they are localized pockets. Grain-size distribution curves for samples of the clay stratum are shown on Figure #4.

The engineering properties of the stratum, as determined by field and laboratory testing, are summarized on Figure #1. A brief resumé, presented in tabular form, follows:

		<u>Range</u>	<u>Average</u>
Bulk Density (p.c.f.)	( $\gamma$ )	95 - 100	97
Liquid Limit (%)	( $W_L$ )	48 - 84	74
Plastic Limit (%)	( $W_P$ )	22 - 31	27
Natural Moisture Content (%)	( $W$ )	55 - 81	73
Liquidity Index	( $I_L$ )	0.95 - 2.2	1.1
Initial Void Ratio	( $e_0$ )	1.78 and 2.35	
Compression Index	( $C_c$ )	1.36 and 2.72	
Undrained Shear Strength (p.s.f.)	( $C_u$ )	<u>Range</u> ( $C_u$ )	<u>Range</u> <u>Sensitivity (S)</u>
1.) Field Vanes		400 - 2,000	8 - 20
2.) Lab. Vanes		320 - 1,400	5 - 20
3.) Lab. Tests		570 - 1,080	

cont'd. /5 ...

4. SOIL TYPES AND SOIL CONDITIONS: (cont'd.) ...

4.3) Sensitive Clay to Silty Clay: (cont'd.) ...

The Atterberg limit tests, summarized on the foregoing page, are also plotted on the Plasticity Chart, Figure #6. These results indicate that the clay is of intermediate to high plasticity and, in general, inorganic. The natural water content is lower than the liquid limit in the upper part of the deposit, increasing with depth to values that are higher than the liquid limit. The consistency of the stratum, as determined from the undrained shear strength testing, increases from soft to firm immediately below the surface deposits, to stiff with depth. This increase in undrained shear strength is represented by an average  $C_u/P_o$  ratio of 0.35 for the overall deposit.  $P_o$  is the effective overburden pressure. The undrained shear strength values obtained from laboratory testing, gave consistently lower values than that obtained from the field vane tests. It is considered that this is primarily due to unavoidable sample disturbance caused by the field and laboratory handling and subsequent testing of the sensitive clay.

The consolidation characteristics of the stratum were determined by carrying out two laboratory consolidation tests, the results of which are shown as Void Ratio vs Pressure Plots, on Figures #8 and #9. The results of this testing indicate that the clay is preconsolidated by about 1,500 to 1,700 p.s.f. in excess of existing overburden pressure on the top of the valley banks. It is, however, preconsolidated by about 2,300 to 2,500 p.s.f. in excess of existing overburden pressure on the valley floor. The relatively high values given for the initial void ratio ( $e_o$ ) and the compression index ( $C_c$ ) are within the normal range for such values obtained from laboratory consolidation testing on sensitive "Leda Clay".

cont'd. /6 ...



4. SOIL TYPES AND SOIL CONDITIONS: (cont'd.) ...

4.4) Clayey Silt to Silt with Sand and Some Gravel -  
(Glacial Till):

This heterogeneous deposit was encountered immediately below the clay stratum, between elevations 180 and 188. The glacial till was fully penetrated only at B.H.'s #1 and 2; at these locations the deposit is about 31 feet thick. The upper 6 to 10 feet of the glacial till is transitional with respect to the overlying clay - i.e., in general, it is in a reworked and softened condition. Random seams and layers of sand and silt up to 1 foot thick, are located throughout the glacial till. Grain-size distribution curves for samples of the deposit are shown on Figure #5 in the Appendix of this report.

Three Atterberg limit tests carried out on representative samples of the basically cohesive glacial till, are plotted on the Plasticity Chart, Figure #7. These results gave values for the liquid and plastic limits that range from 17 to 28 and 12 to 16, respectively. The corresponding natural water content within the upper "reworked" zone of the deposit is generally above the liquid limit. Below this upper zone the natural water content ranges from a few percent above to a few percent below the plastic limit. Based on these results, it is estimated that the glacial till is inorganic and of low plasticity.

The standard penetration resistance or 'N' values vary from 4 to 13 blows/ft. in the upper "reworked" zone; below this zone the values range from 20 blows/ft., increasing with depth to as high as 100 blows/4 inches. Based on these results, it is estimated that the consistency of the glacial till varies from firm to stiff, in the upper "reworked" zone, while below this zone the deposit varies from very stiff to hard.

4.5) Shale Bedrock:

Bedrock was established in two of the borings, namely: B.H.'s #1 and 2, by obtaining AXT rock core samples. At these boring locations the bedrock was encountered at about elevation 149 and 152, respectively - i.e., 95 and 90 feet below existing ground surface.

cont'd. /7 ...

4. SOIL TYPES AND SOIL CONDITIONS: (cont'd.) ...

4.5) Shale Bedrock: (cont'd.) ...

The bedrock is composed of grey fossiliferous shale, the upper few feet of which is in a fractured and weathered condition.

5. GROUNDWATER CONDITIONS:

Groundwater level observations have been carried out, during the period of investigation, in 1) sealed piezometers installed in B.H. #2, and 11) the open holes at the remaining boring locations. The observations are recorded on the Borelog sheets and summarized on Drawings 68-F-53A and 68-F-53B. The results of the measurements indicate that the piezometric groundwater level within the surficial deposits and underlying sensitive clay stratum is at about elevation 241 - i.e., about 1 foot below the valley floor ground surface.

The piezometer installed in the lower portion of the glacial till indicated that the piezometric groundwater level, within this deposit, was at about elevation 245 - i.e., groundwater level approximately 2-1/2 feet above the valley floor. This represents an artesian pressure with respect to ground surface. Past experience in the area indicates that an artesian condition often exists in the lower portion of the glacial till and upper fractured zone of the bedrock. The factors which provide the favourable environment for this artesian condition are:

1) the glacial till and fractured bedrock are more pervious than the overlying cohesive overburden and underlying sound bedrock.

11) these zones are in hydraulic communication with groundwater from the surrounding higher terrain - i.e., the zones are continuously being charged and thus act as an aquifer.

cont'd. /8 ...

## 6. DISCUSSION AND RECOMMENDATIONS:

### 6.1) General:

Between Stations 193+50 and 196+00, proposed Hwy. #417 will cross McEwen Creek. The proposed profile grade of Hwy. #417 in this vicinity will be about elevation 260. The embankment proposed will vary from a few feet in height at the above stations to a maximum of 18 feet above the valley floor. The Eastbound lane and Westbound lane of Hwy. #417 will each incorporate three paved lanes with provision for a future paved lane, together with associated shoulders and medians.

A multi-plate culvert, approximately 12 feet in section, will be employed to carry McEwen Creek beneath the embankment.

Underlying between 1 and 4 feet of sand or clayey silt (surficial deposits) is the predominant stratum across the site, composed of a soft to firm, increasing with depth to stiff, sensitive marine clay. The thickness of this stratum ranges from 59 to 79 feet. The clay is underlain by up to 31 feet of competent glacial till which, in turn, is followed by shale bedrock.

The presence of an extensive deposit of soft and highly compressible clay at a relatively shallow depth below ground surface will mean that the stability of the embankment, as well as the settlement induced in the foundation subsoil, will be critical as far as the feasibility of the proposed scheme is concerned.

### 6.2) Embankment:

#### 6.2.1) Stability Considerations:

Prior to placing any fill on the valley floor, any organic matter associated with the existing channel should be sub-excavated in accordance with current D.H.O. practices.

The critical condition for stability of an embankment on normally or slightly overconsolidated clays, as is the case with this stratum, generally occurs during or immediately after construction. This being the case, a total stress analysis ( $\phi = 0$ )

cont'd. /9 ...

6. DISCUSSION AND RECOMMENDATIONS: (cont'd.) ...

6.2) Embankment: (cont'd.) ...

6.2.1) Stability Considerations: (cont'd.) ...

provides a suitable means of assessing the stability of the embankment sections. In this method of analysis, stability is governed by the applied loads and by the stress-strain and undrained shear strength properties of the foundation and embankment soils.

Analyses have been carried out, therefore, in terms of total stresses, both manually and by the use of the electronic computer, to determine the stability of the fill section. The following assumptions were made:

1) Soil Properties:

Fill Material

Bulk Density	$\gamma$ = 125 p.c.f.
Angle of Shearing Resistance	$\phi$ = 30°

Foundation Subsoil

242 to 240	Clayey Silt - Surficial Deposit	$\gamma$ = 110 p.c.f. $\phi$ = 30°
240 to 230	Sensitive Clay - $\gamma$ = 97 p.c.f. $\gamma'$ = 35 p.c.f.	$C_u$ = 750 p.s.f.
230 to 225	" "	$C_u$ = 900 p.s.f.
225 --	" "	$C_u$ = 1,100 p.s.f.

The stability computations carried out are summarized on Figure 2 in the Appendix of this report. These computations indicate that the embankment, whose maximum height is 18 feet, will be stable with respect to an overall deep-seated failure provided the side slopes are maintained at 2:1.

cont'd. /10 ...

6. DISCUSSION AND RECOMMENDATIONS: (cont'd.) ...

6.2) Embankment: (cont'd ) ...

6.2.2) Settlement Considerations:

The underlying compressible clay stratum will undergo settlements due to consolidation, over a period of time, under the weight of the embankment. Settlement computations were, therefore, carried out, the results of which are summarized on Figure 2. The induced stress increase within the foundation subsoil, due to a 18-foot surcharge loading, which is the maximum height of embankment fill, is less than the preconsolidation pressure of the stratum. The consolidation of the subsoil will, therefore, be primarily of a recompression nature - i.e., the settlement will be less than would occur if the loading had exceeded the preconsolidation pressure, in which case the settlement would be due to "virgin" compression. The computations indicate that the consolidation settlement across the valley would vary from 5 to 9 inches (see plots of Figure 2).

The total amount of this settlement should occur within a period of about 6 years, while about 50% will occur in the first 14 months (refer to plot on Figure 2). It would be advantageous, therefore, to construct the embankment first and leave it in place for as long as possible prior to final paving.

6.3) Multi-Plate Culvert:

All organic material should be removed within the plan limits of the proposed culvert. The sub-excavation so formed should be backfilled with suitable earth fill. The culvert should be seated on a pad composed of well compacted granular material; it is recommended that this pad be approximately 2 feet thick. The fill surrounding the culvert should be well compacted and brought up evenly on either side.

The culvert, located within the embankment, will settle differentially due to consolidation of the underlying foundation subsoil. It is recommended, therefore, that the culvert be provided with a 6-inch camber at the centre of the embankment.

cont'd. /11 ...

7. SUMMARY:

A foundation investigation at the crossing of Hwy. #417 and McEwen Creek, in the Township of Gloucester, County of Carleton, is reported.

The predominant deposit acrosss the site is a stratum of firm to very stiff sensitive clay to silty clay some 59 to 78 feet thick, overlying up to 31 feet of stiff to hard, basically cohesive glacial till. The glacial till is underlain by shale bedrock, the surface of which was encountered between elevations 149 and 152. The groundwater level in the surficial deposits and clay stratum was about 1 foot and 4 feet below ground surface on the valley floor and on top of the valley bank, respectively. An artesian water pressure head (about 2-1/2 feet above ground surface) was encountered in the lower portion of the glacial till and the upper fractured portion of the bedrock along the valley floor. The height of the embankment crossing McEwen Creek will vary from a few feet to up to 18 feet. Fills of this height will be stable provided 2:1 side slopes are employed. Computations carried out indicate that the long-term consolidation settlement induced in the foundation subsoil due to the surcharge loading will not exceed 9 inches.

The multi-plate culvert should be placed as discussed in the report.

8. MISCELLANEOUS:

The field work for this project was carried out during the period of July 2 to 5, 1968, under the supervision of Mr. P. B. Schnabel, Project Foundation Engineer. The equipment used was owned and operated by F. E. Johnston Drilling Co. Ltd.

This report was written by Mr. T. Card, Project Foundation Engineer, and Mr. B. T. Darch, Senior Foundation Engineer, and was reviewed by Mr. M. Devata, Supervising Foundation Engineer.

September, 1968.

APPENDIX I

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS &amp; TESTING DIVISION

## RECORD OF BOREHOLE NO. 1

FOUNDATION SECTION

JOB 68-F-53

LOCATION Hwy. 417 Sta. 196 + 89 EBL 10' Rt.

ORIGINATED BY PBS

W P 34-66-01

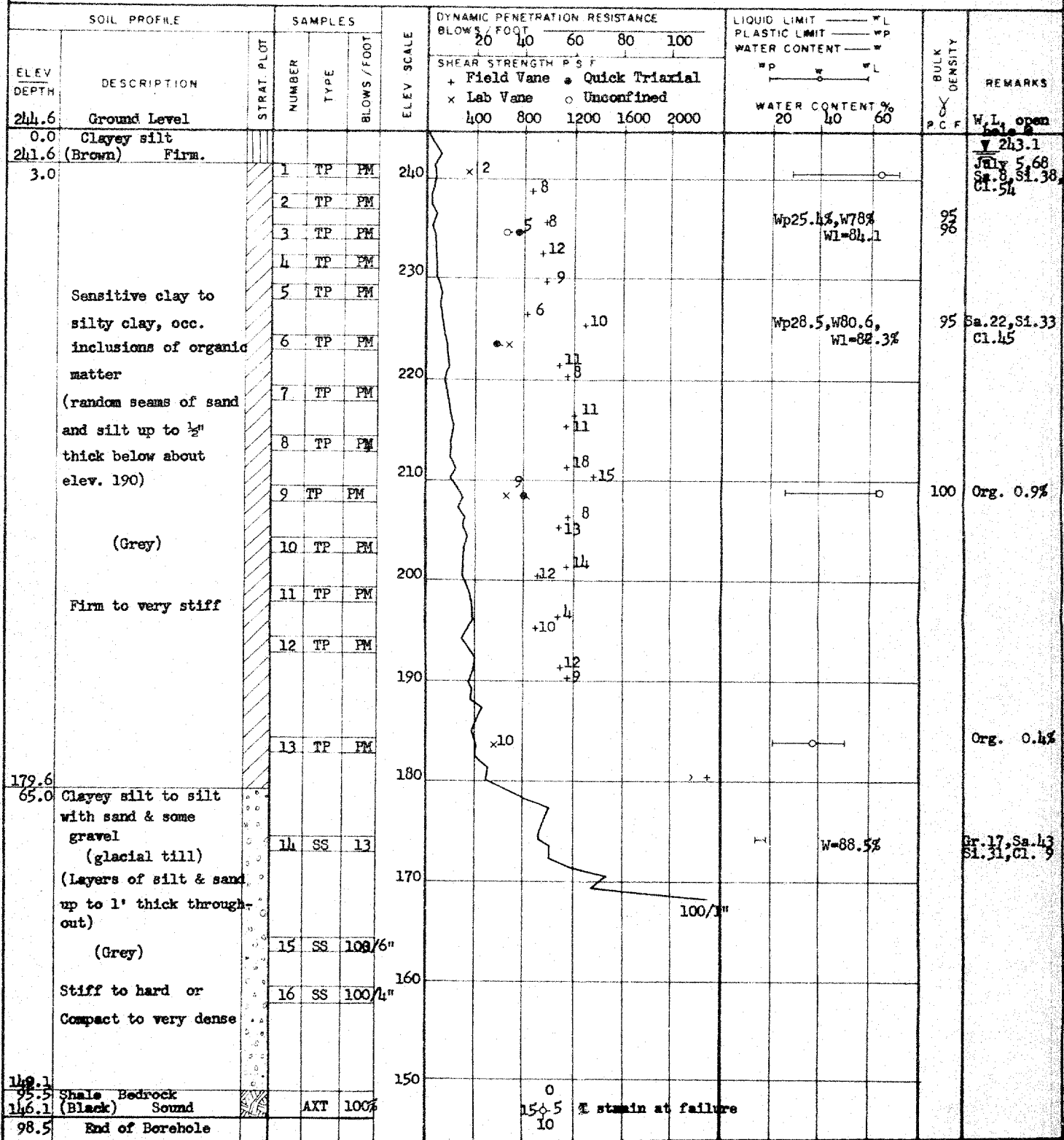
BORING DATE July 2 - 3, 1968

COMPILED BY PBS

DATUM Geodetic

BOREHOLE TYPE Diamond Drill NX - BX Casing, AXT Core

CHECKED BY





DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS &amp; TESTING DIVISION

## RECORD OF BOREHOLE NO. 2

FOUNDATION SECTION

JOB 68-F-53

LOCATION Hwy. 417 Sta. 194 + 91 WBL 2

ORIGINATED BY PBS

W.P. 34-66-01

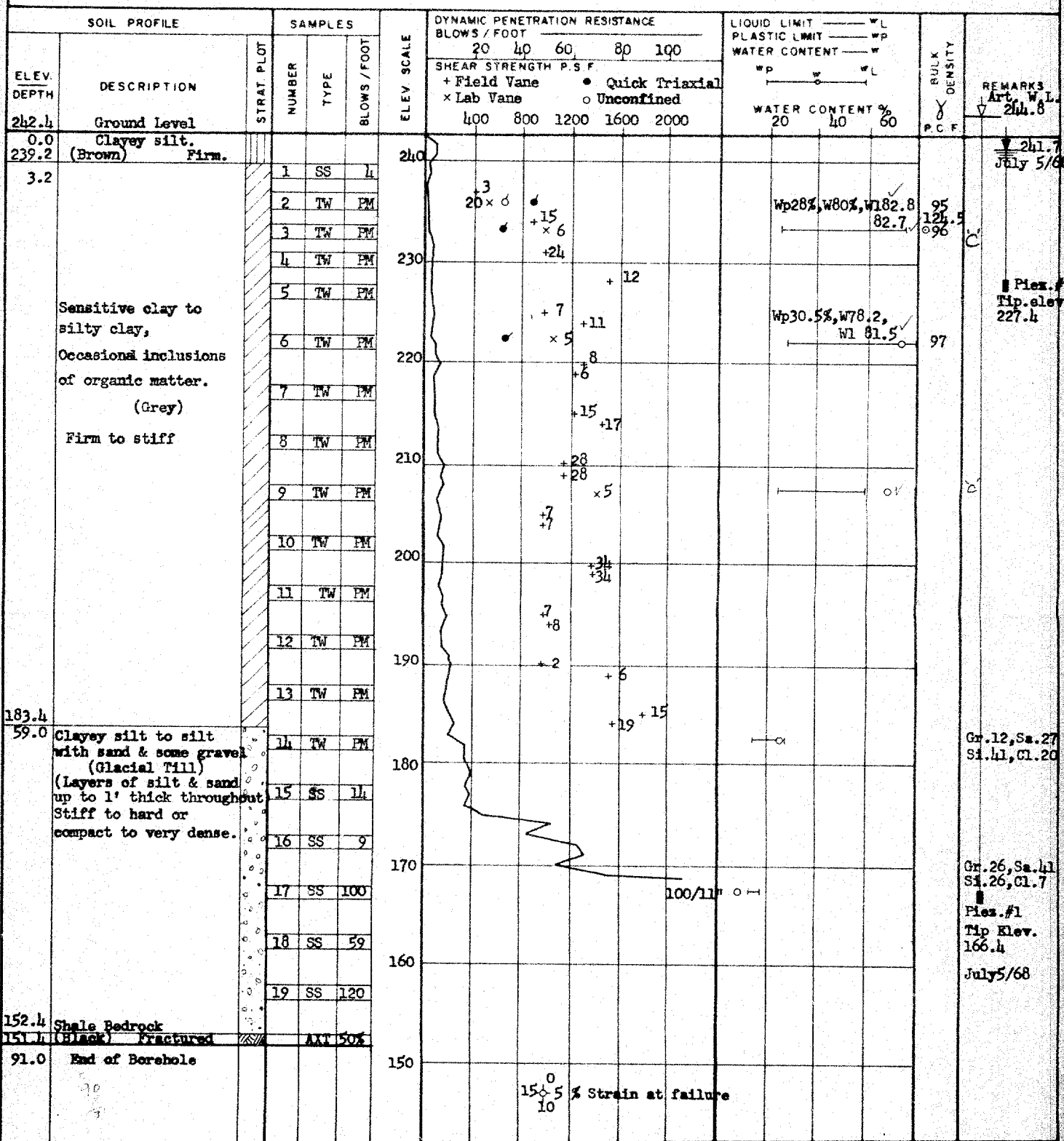
BORING DATE July 2, 1968

COMPILED BY PBS

DATUM Geodetic

BOREHOLE TYPE Diamond Drill, NX, BY, AX Casing, AXT Core

CHECKED BY



DEPARTMENT OF HIGHWAYS - ONTARIO

## RECORD OF BOREHOLE NO. 3

FOUNDATION SECTION

MATERIALS &amp; TESTING DIVISION

JOB 68-F-53

LOCATION Hwy. 417 Sta. 195 + 37 EBL 0

ORIGINATED BY PBS

W. P. 34-66-01

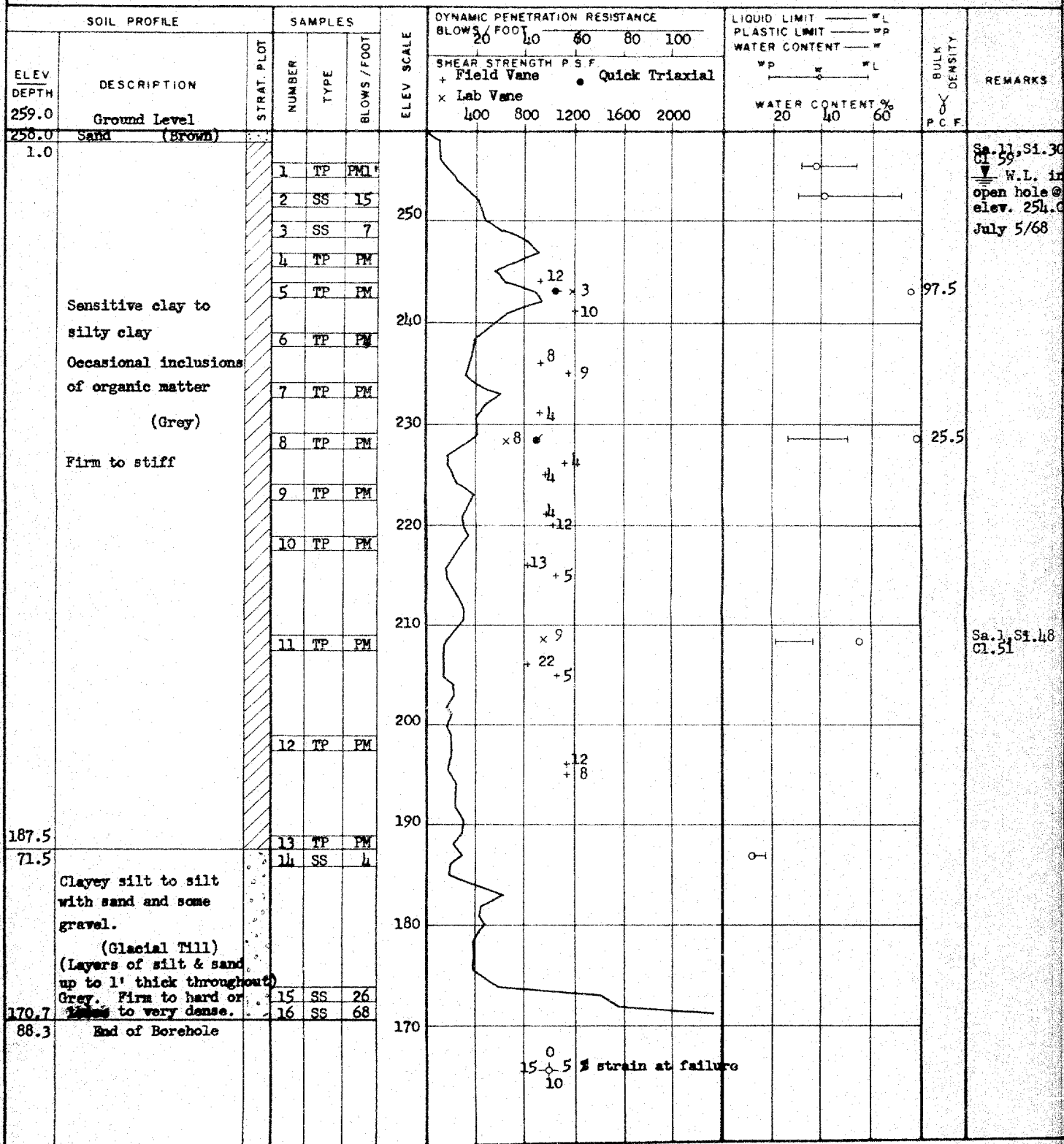
BORING DATE July 4-5, 1968

COMPILED BY PBS

DATUM Geodetic

BOREHOLE TYPE Diamond Drill NX Casing

CHECKED BY



DEPARTMENT OF HIGHWAYS - ONTARIO

## MATERIALS &amp; TESTING DIVISION

## RECORD OF BOREHOLE NO. 4

FOUNDATION SECTION

JOB 68-P-53

LOCATION Hwy. 417 Sta. 195 + 15 WBL 0

ORIGINATED BY PBS

W P 34-86-01

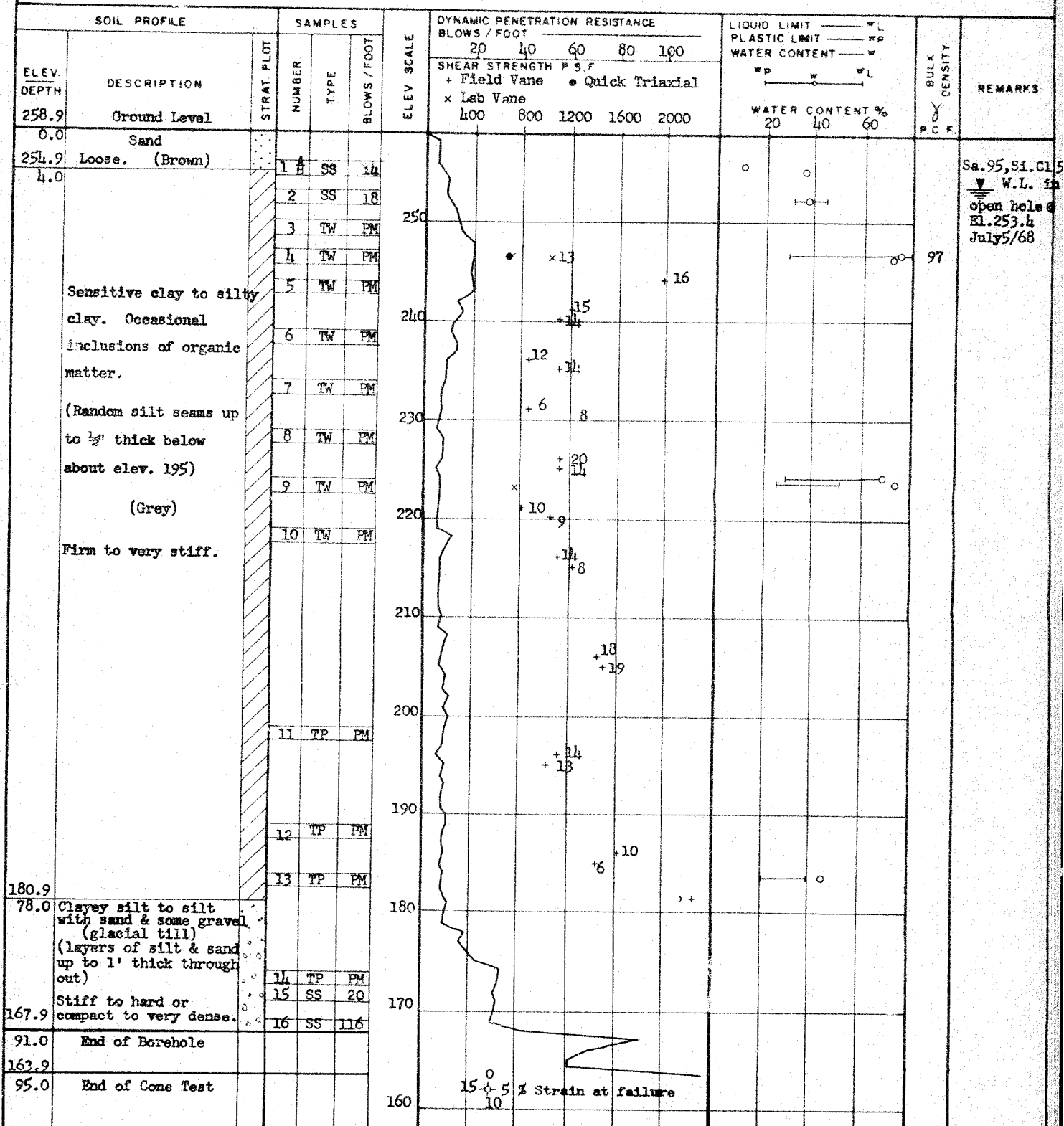
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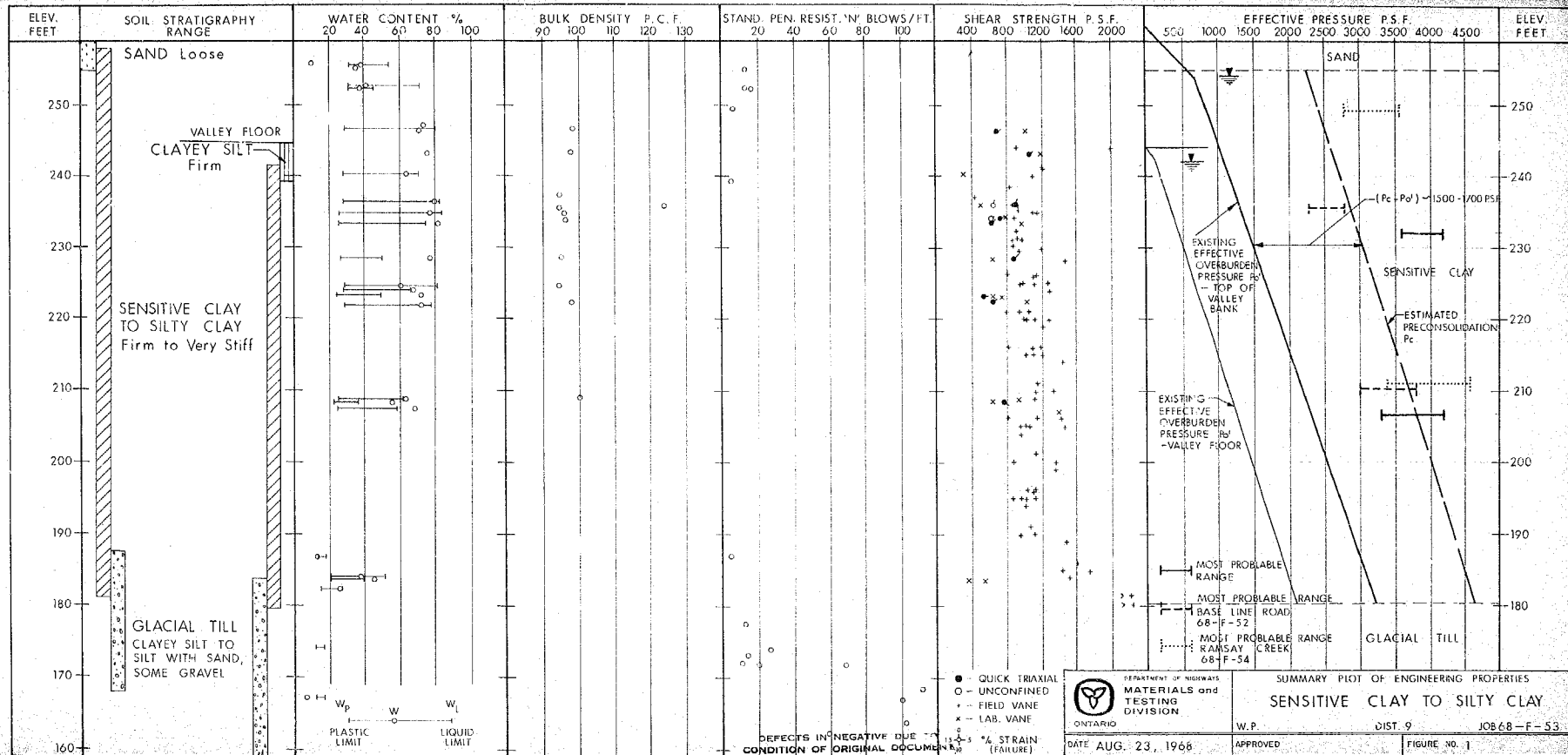
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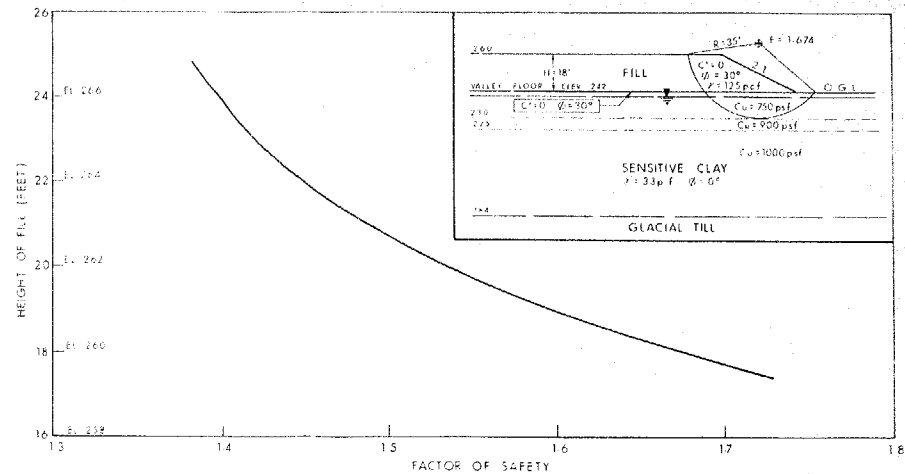
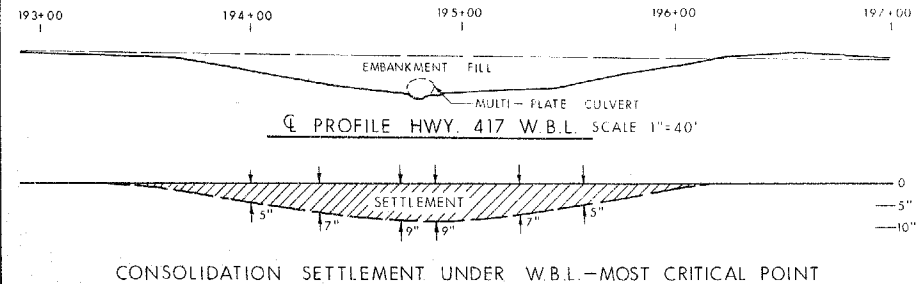
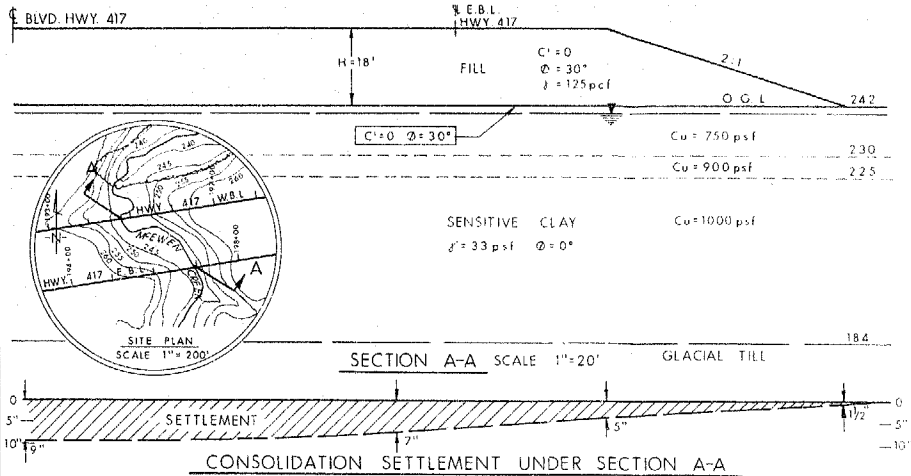
DATUM Geodetic

BOREHOLE TYPE Diamond Drill, NX Casing

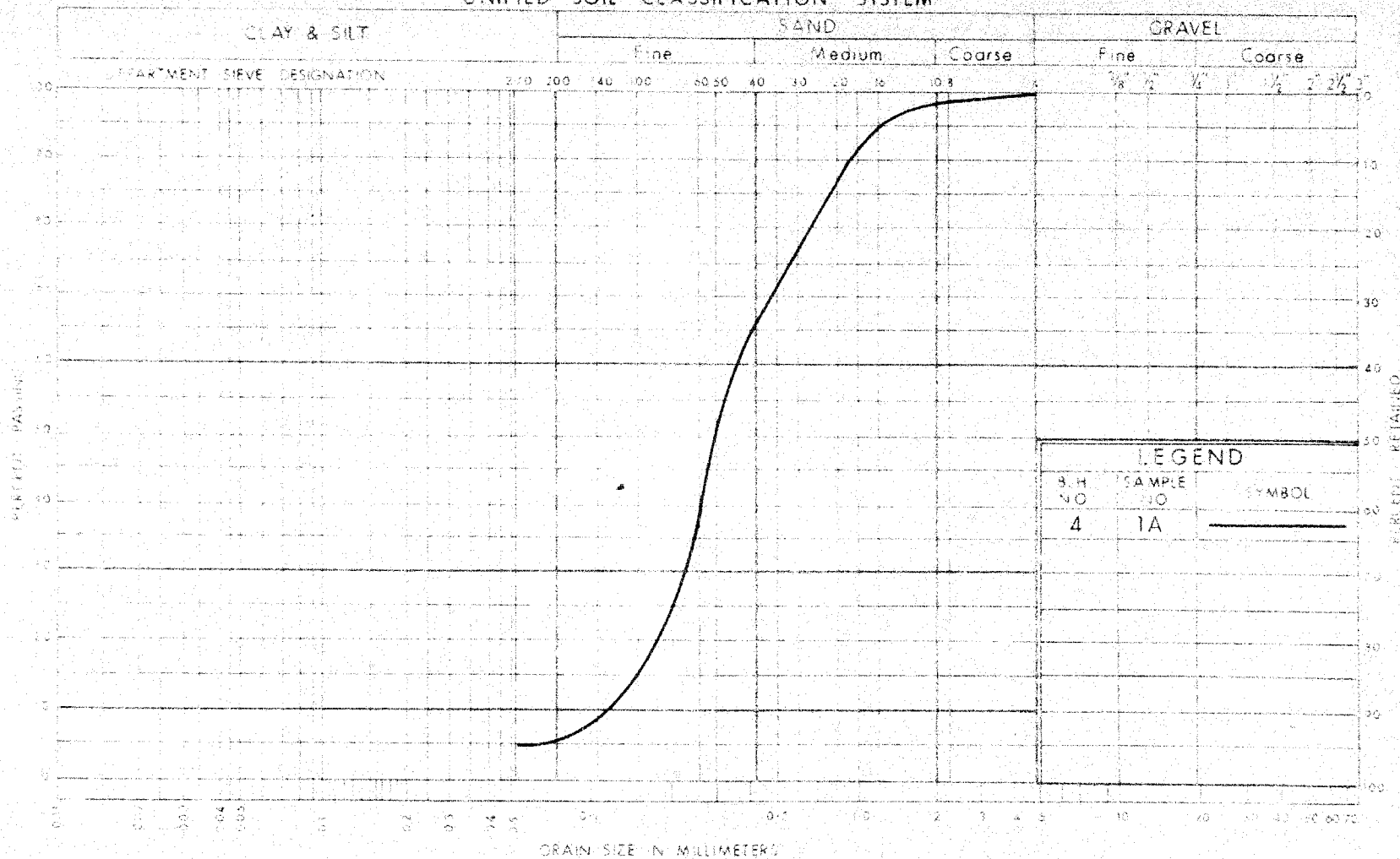
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## UNIFIED SOIL CLASSIFICATION SYSTEM



DEPARTMENT OF HIGHWAYS  
MATERIALS and  
TESTING  
DIVISION

# GRAIN SIZE DISTRIBUTION

## SAND

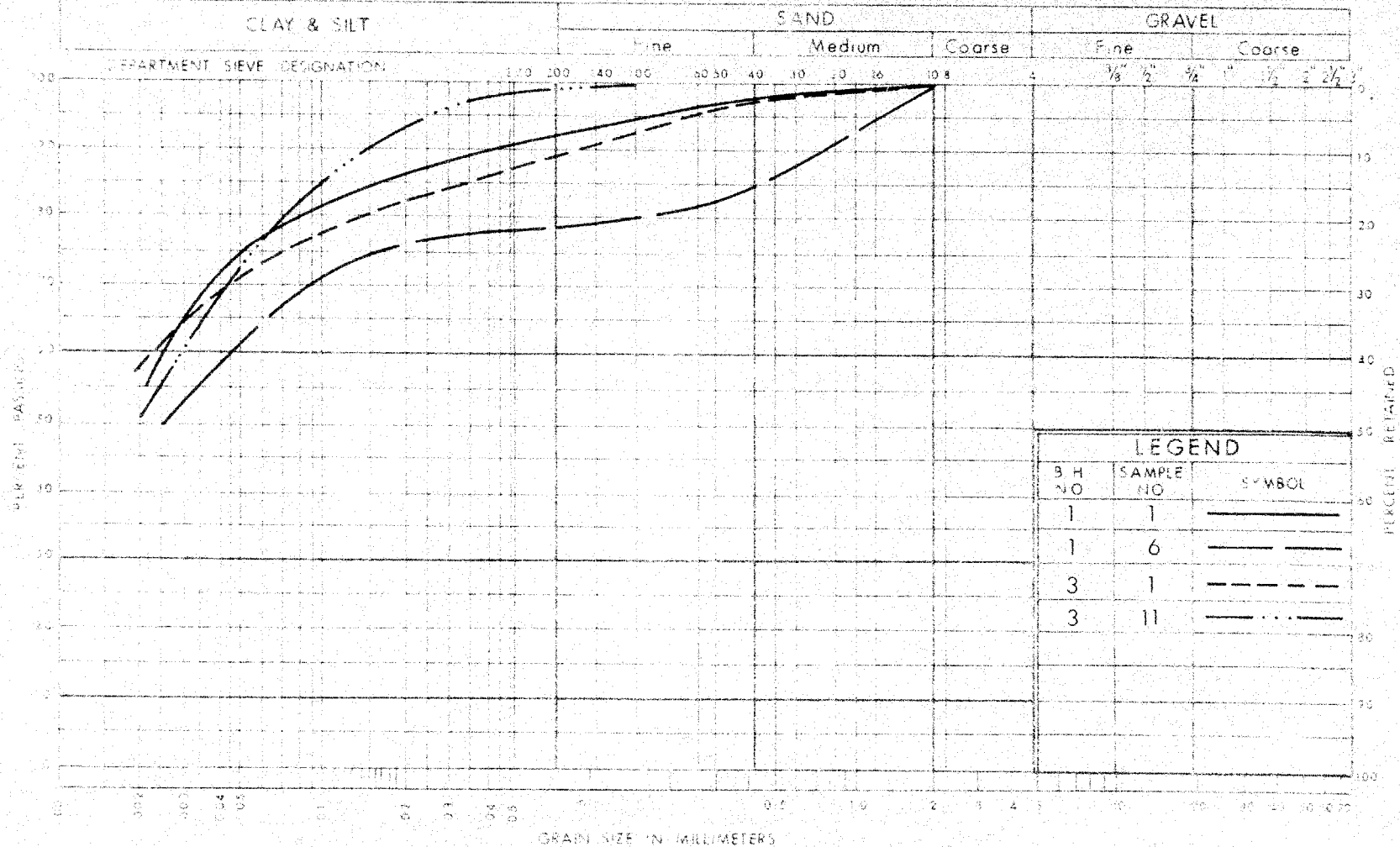
WP No. 34-66-01

JOB No. 68-F-53

FIG. NO. 3

DEFECTS IN NEGATIVE DUE TO  
CONDITION OF ORIGINAL DOCUMENT

# UNIFIED SOIL CLASSIFICATION SYSTEM



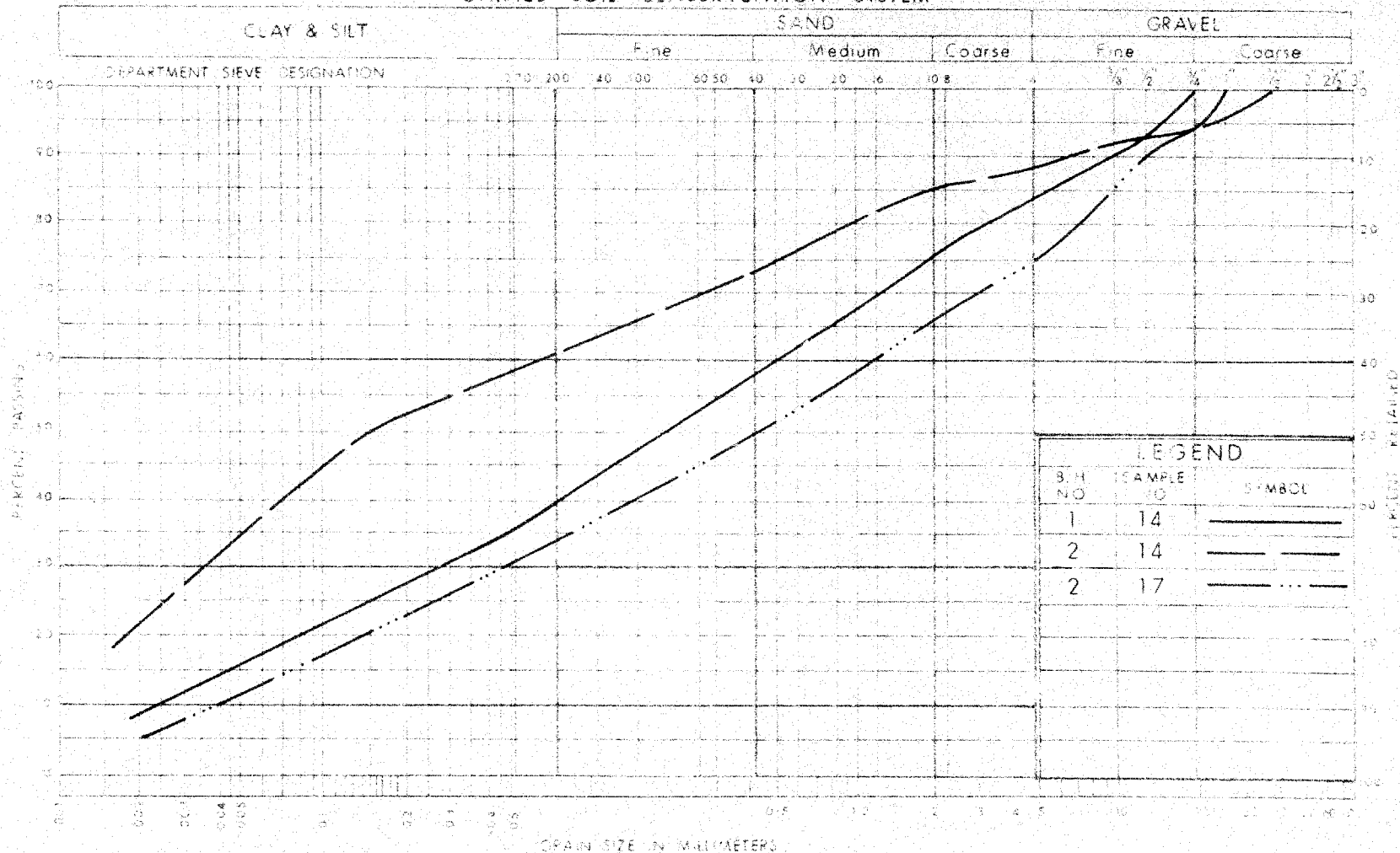
DEPARTMENT OF HIGHWAYS  
MATERIALS and  
TESTING  
DIVISION

GRAIN SIZE DISTRIBUTION  
SENSITIVE CLAY TO SILTY CLAY

WP No. 34-66-01  
JOB No. 68-F-53  
FIG. NO. 4

DEFECTS IN NEGATIVE DUE TO  
CONDITION OF ORIGINAL DOCUMENT

# UNIFIED SOIL CLASSIFICATION SYSTEM



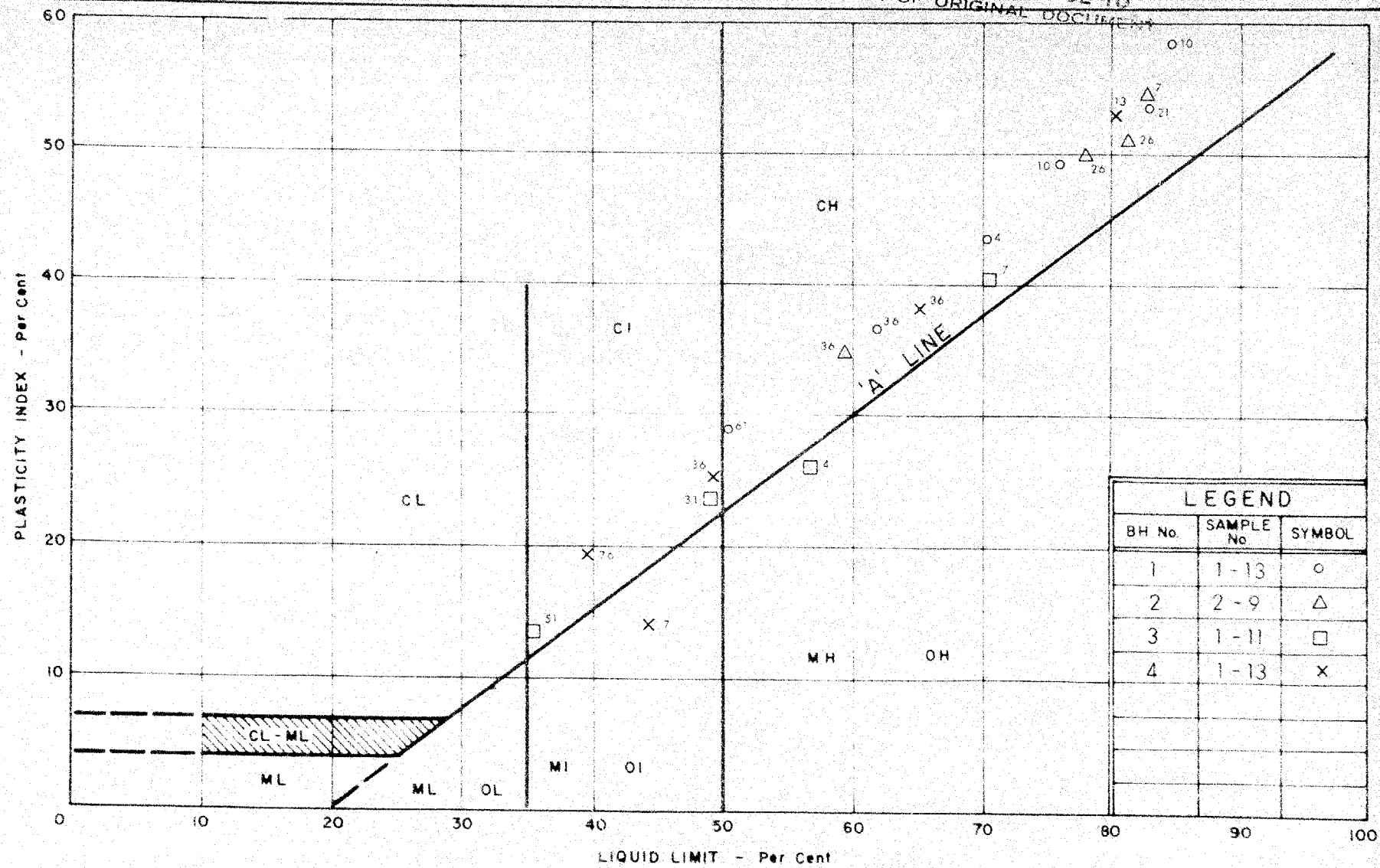
DEPARTMENT OF HIGHWAYS  
MATERIALS and  
TESTING  
DIVISION

GRAIN SIZE DISTRIBUTION  
GLACIAL TILL  
CLAYEY SILT TO SILT WITH SAND, SOME GRAVEL

W.P. No. 34-66-01  
JOB No. 68-F-53  
FIG NO 5



DEFECTS IN NEGATIVE DUE TO  
CONDITION OF ORIGINAL DOCUMENT



DEPARTMENT OF HIGHWAYS  
MATERIALS and  
TESTING  
DIVISION

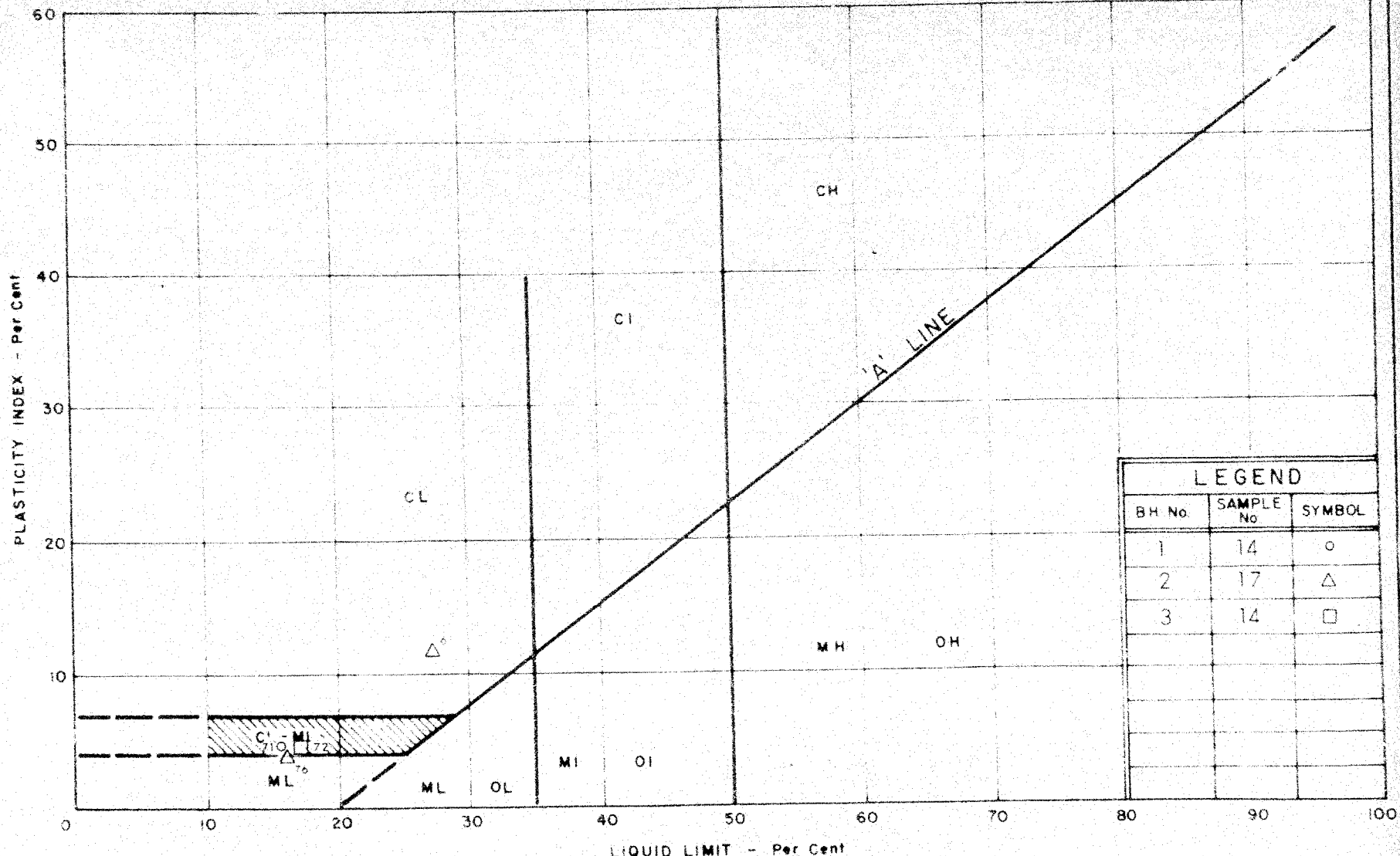
# PLASTICITY CHART SENSITIVE CLAY TO SILTY CLAY

WP No. 34-66-01

JOB No. 68-F-53

FIG. NO. 6

DEFECTS IN NEGATIVE DUE TO  
CONDITION OF ORIGINAL DOCUMENT



DEPARTMENT OF HIGHWAYS  
MATERIALS and  
TESTING  
DIVISION

PLASTICITY CHART  
GLACIAL TILL  
CLAYEY SILT TO SILT WITH SAND, SOME GRAVEL

W.P. No. 34-66-01

JOB No. 68 - F - 53

FIG. NO. 7

# VOID RATIO vs PRESSURE

$W_L = 75.5\%$

$W_p = 26.3\%$

$W = 82.7\%$

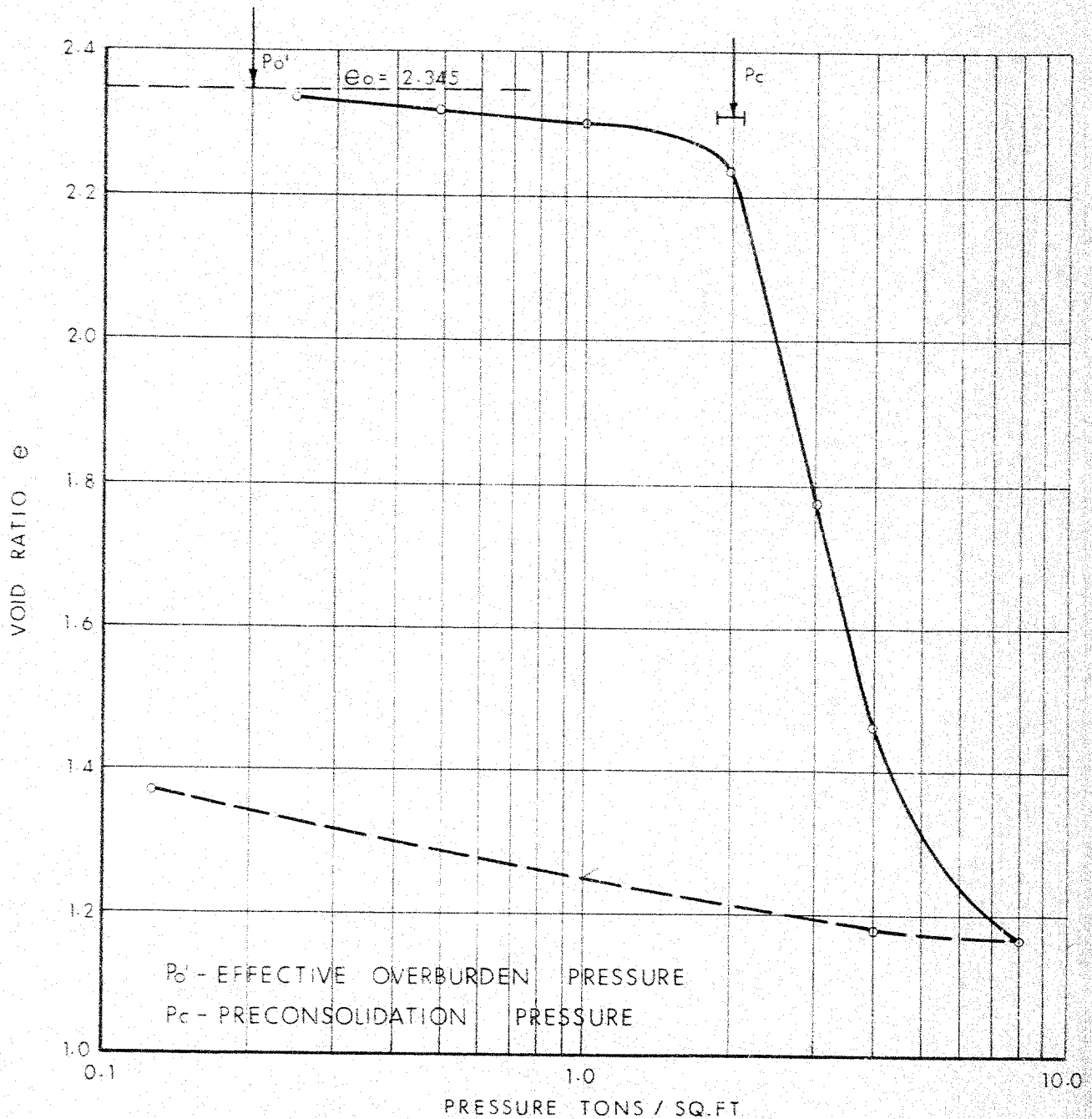
$C_c = 2.72$

BORE HOLE 2

SAMPLE 3

DEPTH 10'-5"

ELEV. 232.0



EFFECTS IN NEGATIVE DUE TO  
CONDITION OF ORIGINAL DOCUMENT

FIG 8

# VOID RATIO vs PRESSURE

$W_L = 59.1\%$

$W_p = 24.1\%$

$W = 68.7\%$

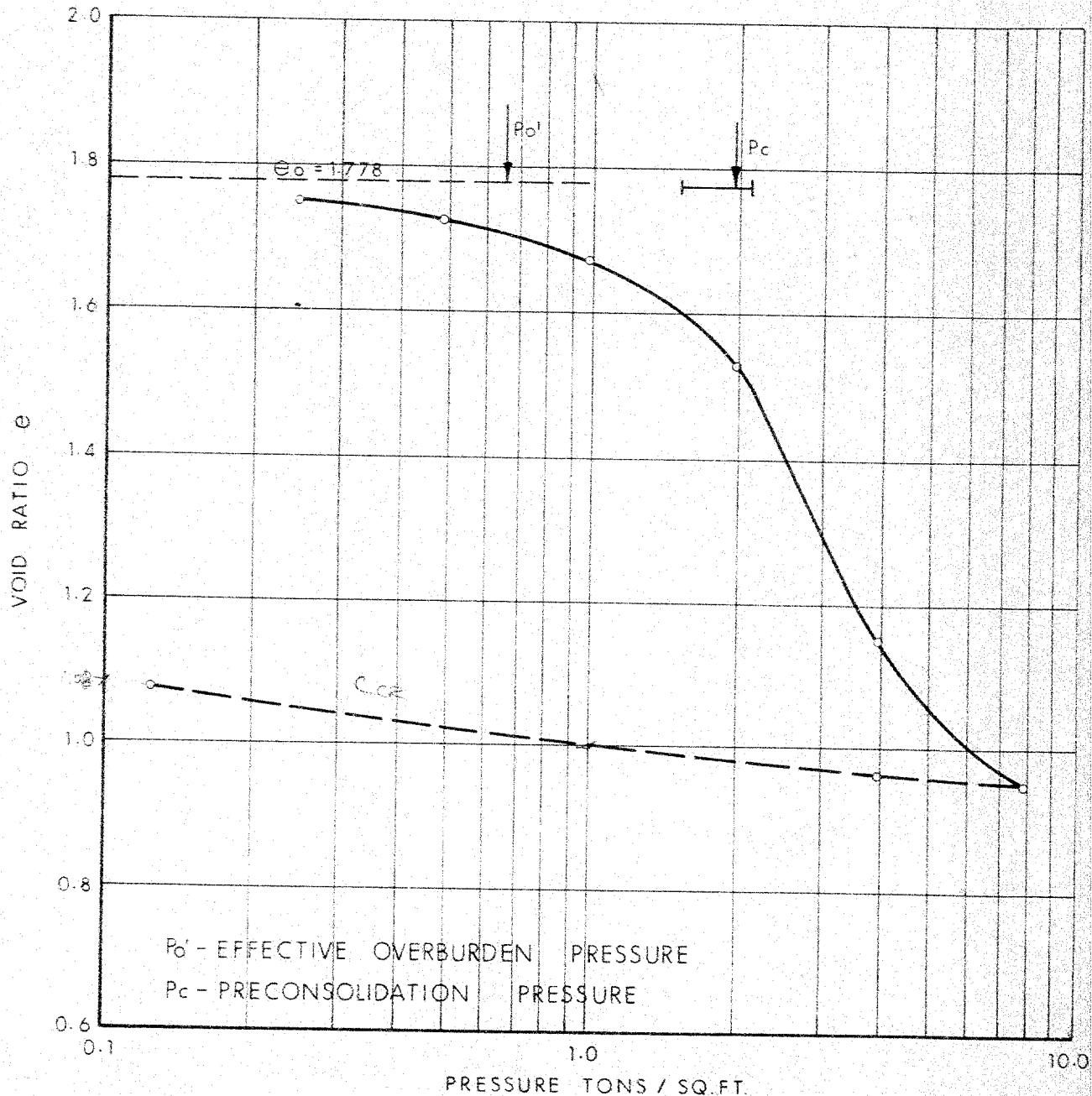
$C_c = 1.36$

BORE HOLE 2

SAMPLE 9

DEPTH 35'-8"

ELEV. 206.7



DEFECTS IN NEGATIVE DUE TO  
 CONDITION OF ORIGINAL DOCUMENT

FIG. 9

## ABBREVIATIONS USED IN THIS REPORT

### PENETRATION RESISTANCE

STANDARD PENETRATION RESISTANCE 'N' :- THE NUMBER OF BLOWS REQUIRED TO ADVANCE A STANDARD SPLIT SPOON SAMPLER 12 INCHES INTO THE SUBSOIL, DRIVEN BY MEANS OF A 140 POUND HAMMER FALLING FREELY A DISTANCE OF 30 INCHES.

DYNAMIC PENETRATION RESISTANCE :- THE NUMBER OF BLOWS REQUIRED TO ADVANCE A 2 INCH, 60 DEGREE CONE, FITTED TO THE END OF DRILL RODS, 12 INCHES INTO THE SUBSOIL, THE DRIVING ENERGY BEING 350 FOOT POUNDS PER BLOW.

### DESCRIPTION OF SOIL

THE CONSISTENCY OF COHESIVE SOILS AND THE RELATIVE DENSITY OR DENSENESS OF COHESIONLESS SOILS ARE DESCRIBED IN THE FOLLOWING TERMS :-

<u>CONSISTENCY</u>	<u>'N' BLOWS / FT.</u>	<u>c LB. / SQ. FT.</u>	<u>DENSENESS</u>	<u>'N' BLOWS / FT.</u>
VERY SOFT	0 - 2	0 - 250	VERY LOOSE	0 - 4
SOFT	2 - 4	250 - 500	LOOSE	4 - 10
FIRM	4 - 8	500 - 1000	COMPACT	10 - 30
STIFF	8 - 15	1000 - 2000	DENSE	30 - 50
VERY STIFF	15 - 30	2000 - 4000	VERY DENSE	> 50
HARD	> 30	> 4000		

### TYPE OF SAMPLE

S.S	SPLIT SPOON	T.W	THINWALL OPEN
W.S	WASHED SAMPLE	T.P	THINWALL PISTON
S.B	SCRAPER BUCKET SAMPLE	O.S	OESTERBERG SAMPLE
A.S	AUGER SAMPLE	F.S	FOIL SAMPLE
C.S	CHUNK SAMPLE	R.C	ROCK CORE
S.T	SLOTTED TUBE SAMPLE		
	P.H	SAMPLE ADVANCED HYDRAULICALLY	
	P.M	SAMPLE ADVANCED MANUALLY	

### SOIL TESTS

Q <sub>u</sub>	UNCONFINED COMPRESSION	L.V	LABORATORY VANE
Q	UNDRAINED TRIAXIAL	F.V	FIELD VANE
Q <sub>cu</sub>	CONSOLIDATED UNDRAINED TRIAXIAL	C	CONSOLIDATION
Q <sub>d</sub>	DRAINED TRIAXIAL	S	SENSITIVITY

## ABBREVIATIONS USED IN THIS REPORT

### SOIL PROPERTIES

$\gamma$	UNIT WEIGHT OF SOIL (BULK DENSITY)
$\gamma_s$	UNIT WEIGHT OF SOLID PARTICLES
$\gamma_w$	UNIT WEIGHT OF WATER
$\gamma_d$	UNIT DRY WEIGHT OF SOIL (DRY DENSITY)
$\gamma'$	UNIT WEIGHT OF SUBMERGED SOIL
G	SPECIFIC GRAVITY OF SOLID PARTICLES $G = \frac{\gamma_s}{\gamma_w}$
e	VOID RATIO
n	POROSITY
w	WATER CONTENT
$S_r$	DEGREE OF SATURATION
$w_L$	LIQUID LIMIT
$w_p$	PLASTIC LIMIT
$I_p$	PLASTICITY INDEX
s	SHRINKAGE LIMIT
$I_L$	LIQUIDITY INDEX $= \frac{w - w_p}{I_p}$
$I_C$	CONSISTENCY INDEX $= \frac{w_L - w_p}{I_p}$
$e_{max}$	VOID RATIO IN LOOSEST STATE
$e_{min}$	VOID RATIO IN DENSEST STATE
$I_D$	DENSITY INDEX $= \frac{e_{max} - e}{e_{max} - e_{min}}$
	RELATIVE DENSITY $D_r$ IS ALSO USED
h	HYDRAULIC HEAD OR POTENTIAL
q	RATE OF DISCHARGE
v	VELOCITY OF FLOW
i	HYDRAULIC GRADIENT
k	COEFFICIENT OF PERMEABILITY
j	SEEPAGE FORCE PER UNIT VOLUME
$m_v$	COEFFICIENT OF VOLUME CHANGE $= \frac{-\Delta e}{(1+e)\Delta\sigma}$
$C_v$	COEFFICIENT OF CONSOLIDATION
$C_c$	COMPRESSION INDEX $= \frac{\Delta e}{\Delta \log_{10} \sigma}$
$T_v$	TIME FACTOR $= \frac{C_v t}{d^2}$ (d, DRAINAGE PATH)
U	DEGREE OF CONSOLIDATION
$\tau_f$	SHEAR STRENGTH
$c'$	EFFECTIVE COHESION
$\phi'$	EFFECTIVE ANGLE OF SHEARING RESISTANCE, OR FRICTION
$c_u$	APPARENT COHESION
$\phi_u$	APPARENT ANGLE OF SHEARING RESISTANCE, OR FRICTION
$\mu$	COEFFICIENT OF FRICTION
$S_r$	SENSITIVITY

### GENERAL

$\pi$	= 3.1416
e	BASE OF NATURAL LOGARITHMS 2.7183
$\log_e a$ OR $\ln a$	NATURAL LOGARITHM OF a
$\log_{10} a$ OR $\log a$	LOGARITHM OF a TO BASE 10
t	TIME
g	ACCELERATION DUE TO GRAVITY
V	VOLUME
W	WEIGHT
M	MOMENT
F	FACTOR OF SAFETY

### STRESS AND STRAIN

u	PORE PRESSURE
$\sigma$	NORMAL STRESS
$\bar{\sigma}$	NORMAL EFFECTIVE STRESS ( $\bar{\sigma}$ IS ALSO USED)
$\tau$	SHEAR STRESS
$\epsilon$	LINEAR STRAIN
$\gamma$	SHEAR STRAIN
$\nu$	POISSON'S RATIO ( $\mu$ IS ALSO USED)
E	MODULUS OF LINEAR DEFORMATION (YOUNG'S MODULUS)
G	MODULUS OF SHEAR DEFORMATION
K	MODULUS OF COMPRESSIBILITY
$\eta$	COEFFICIENT OF VISCOSITY

### EARTH PRESSURE

d	DISTANCE FROM TOP OF WALL TO POINT OF APPLICATION OF PRESSURE
$\delta$	ANGLE OF WALL FRICTION
K	DIMENSIONLESS COEFFICIENT TO BE USED WITH VARIOUS SUFFIXES IN EXPRESSIONS REFERRING TO NORMAL STRESS ON WALLS
$K_0$	COEFFICIENT OF EARTH PRESSURE AT REST

### FOUNDATIONS

B	BREADTH OF FOUNDATION
L	LENGTH OF FOUNDATION
D	DEPTH OF FOUNDATION BENEATH GROUND
N	DIMENSIONLESS COEFFICIENT USED WITH A SUFFIX APPLYING TO SPECIFIC GRAVITY, DEPTH AND COHESION ETC. IN THE FORMULA FOR BEARING CAPACITY
$k_s$	MODULUS OF SUBGRADE REACTION

### SLOPES

H	VERTICAL HEIGHT OF SLOPE
D	DEPTH BELOW TOE OF SLOPE TO HARD STRATUM
$\beta$	ANGLE OF SLOPE TO HORIZONTAL

Oct 68 ①

McEwen Creek.

68-F-53

Settlement Due to 22 ft. embankment

Beneath  $\frac{1}{2}$  of Embankment

$B = 36'$ ,  $L = 100'$        $q = 2750$  psf

Depth	$\frac{B}{2}$	$\frac{L}{2}$	I	2I	$\Delta q$
0	$\infty$	$\infty$	0.5	1.0	2750
20	1.8	5	0.5	1.0	2750
40	0.9	2.5	0.492	0.984	2700
60	0.6	1.7	0.48	0.960	2640

$$\Delta H = \sum_{i=1}^n H_i \frac{\Delta e_i}{1+e_{i0}}$$

Zone #	$\Delta H$	elev. Range	$P_{avg}$	$P_0 + \Delta P$	$e_1$	$e_2$	$\Delta H$
1	20	202 to 222	475	3,225	2.34	2.30	2.9"
2	20	222 to 202	1,125	3,350	1.78	1.68	8.6"
3	18	202 to 184	1,730	4,400	2.21	2.10	<u>7.9"</u>

Total Predicted Settlement 18.9"Corrected Settlement  $\approx 16"$

# Baseline Rd.

Settlement Due to 10 ft embankment

Beneath  $\frac{1}{2}$  of Embankment

Depth	$\sigma' = 25'$ $\frac{1}{2}$	$b = 24'$ $\frac{1}{2}$	$I$	$2I$	$q = 2375 \text{ psf}$ $\Delta q \text{ (psf)}$
0	$\infty$	$\infty$	0.5	1.0	2375
20	1.4	1.2	0.47	0.94	2240
40	0.7	0.6	0.325	0.73	1880
60	0.47	0.4	0.32	0.64	1520
100	0.28	0.24	0.22	0.44	1050
120	0.23	0.2	0.19	0.38	908

Layer No	Depth	$P_0$	$P_0 + \Delta q$	$e_0$	$e_1$	$\Delta e$	$H \frac{\Delta e}{1 - e_0}$
1	40'	2550	3,150	2.91	2.28	0.13	18.3'
2	40'	2300	3,850	2.22	2.13	0.09	13.8
3	45'	3,750	4,750	1.66	1.65	0.01	<u>20'</u>

Total Consol. Settlement = 34.1"

say 2  $\frac{1}{2}$  ft. (30")

7  $\frac{1}{2}$ " in 4 months

14" in 1 year

19  $\frac{1}{2}$ " in 2 years

22  $\frac{1}{2}$ " in 3 years

Complete in 7  $\frac{1}{2}$  years



#68-F-53

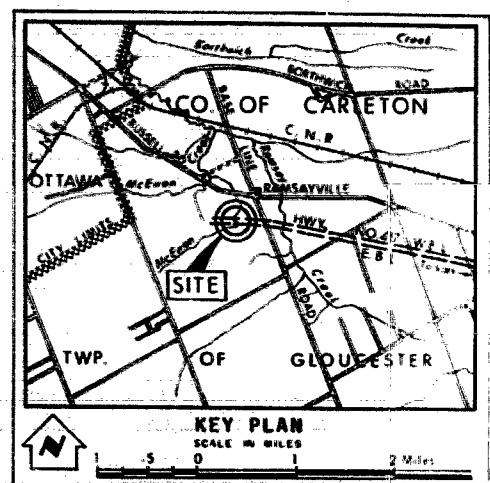
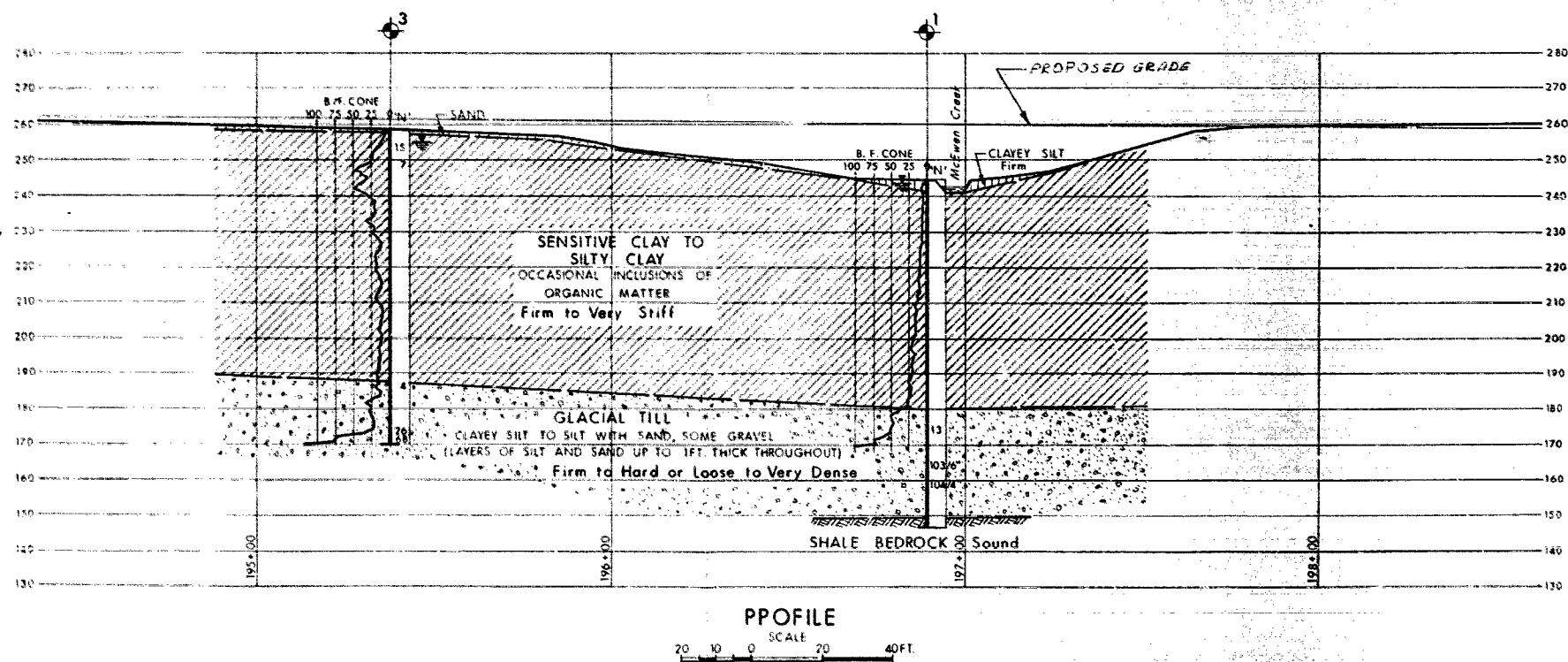
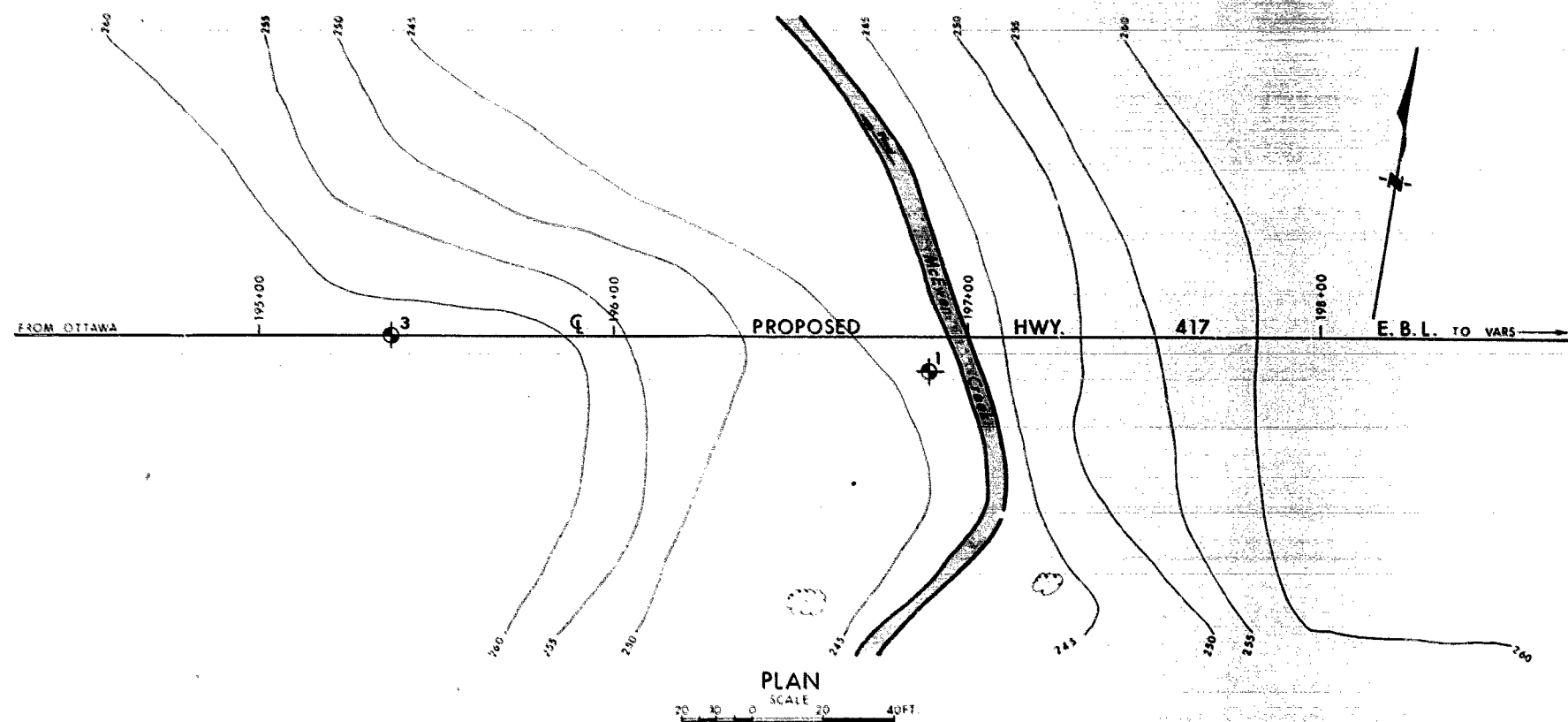
W.P. #34-66-01

HWY. #417





EAST AND WEST

BOUND LANES

McEWEN CREEK



**LEGEND**

 Bore Hole  
 Cone Penetration Hole  
 Bore & Cone Penetration Hole  
 Water Levels established at time of field investigation. July 1968

NO.	ELEVATION	STATION	OFFSET
1	244.6	196 + 89	RT RT
3	259.0	195 + 37	6

**- NOTE -**

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

<b>REVISIONS</b>				
	<b>DATE</b>	<b>BY</b>		<b>DESCRIPTION</b>

DEPARTMENT OF HIGHWAYS - ONTARIO MATERIALS & TESTING DIVISION - FOUNDATION SECTION			
<h1>McEWEN CREEK</h1>			
KING'S HIGHWAY NO. 417		E.B.L.	
CO. CARLETON		DIST. NO. 9	
TWP. GLOUCESTER		LOT 8	CON. VIRF.
<h2>BORE HOLE LOCATIONS &amp; SOIL STRATA</h2>			
SUBWD. S.T.D.	CHECKED <i>AT</i>	WP. NO. 34-66-01	W.B.T. DRAWING NO.
DRAWN G. P.	CHECKED	JOB NO. 68-F-53	68-F-53A
DATE AUG. 20, 1968		SITE NO.	BRIDGE DRAWING NO.
APPROVED <i>A.B. McNamee</i>	CONT. NO.		

