

## MEMORANDUM

To: Mr. R. R. Davis,  
Bridge Engineer,  
Bridge Division,  
Admin. Bldg.

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

Attention: Mr. S. MacCombie

DATE: June 12, 1968

OUR FILE REF.

IN REPLY TO

JUN 19 1968

## SUBJECT:

FOUNDATION INVESTIGATION REPORT  
For  
Underpass Structure at the Crossing  
Of  
Proposed Hwy. #417 and Boundary Rd.  
Twp. of Gloucester, Co. of Carleton  
District No. 9 (Ottawa)  
W.J. 68-F-30 -- W.P. 34-66-09

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/MaeF

Attach.

cc: Messrs. B. R. Davis (2)  
H. A. Tregaskes  
D. W. Farren  
S. J. Markiewicz  
C. R. Robertson  
G. Scott  
J. E. Graspier  
J. L. Forster  
B. A. Singh

Foundations Files  
Gen. Files

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

## TABLE OF CONTENTS

1. INTRODUCTION.
  2. DESCRIPTION OF THE SITE AND GEOLOGY.
  3. FIELD AND LABORATORY WORK.
  4. SOIL TYPES AND SOIL CONDITIONS:
    - 4.1) General.
    - 4.2) Silty Sand to Sandy Silt - Surficial Deposit.
    - 4.3) Sensitive Clay to Clayey Silt.
    - 4.4) Clayey Silt with Fractured Shale and Limestone (Glacial Till Derived from Bedrock).
    - 4.5) Shale Bedrock.
  5. GROUNDWATER CONDITIONS.
  6. DISCUSSION AND RECOMMENDATIONS:
    - 6.1) General.
    - 6.2) Approach Embankments:
      - 6.2.1) Stability Considerations.
      - 6.2.2) Settlement Considerations.
    - 6.3) Structure Foundations.
  7. SUMMARY.
  8. MISCELLANEOUS.
-

FOUNDATION INVESTIGATION REPORT  
For  
Underpass Structure at the Crossing  
Of  
Proposed Hwy. #417 and Boundary Rd.  
Twp. of Gloucester, Co. of Carleton  
District No. 9 (Ottawa)  
W.J. 68-F-33    --    W.P. 34-66-09

1. INTRODUCTION:

The Foundation Section was requested to carry out an investigation at the proposed crossing of Boundary Rd. and Hwy. #417; the site is located southeast of Ottawa, in the Twp. of Gloucester, County of Carleton. The request was contained in a memo from the Bridge Division (Mr. G. Scott, Regional Bridge Location Engineer, Eastern Region), dated February 5, 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 mile south of Carlsbad Springs in the vicinity of the intersection of Ninth Line Rd. and Boundary Rd. Boundary Rd., in the vicinity of the site, is a gravel road approximately 22 ft. wide with the grade between elevation 253 and 255. Drainage ditches, 3 to 4 ft. deep and 10 to 15 ft. wide at the top, are located on either side of the road. The surrounding area is flat farmland, the surface of which varies between elevation 253 and 254.

Physiographically, the site is located within the "Russell and Prescott Sand Plains". In this area a sand mantle, varying in

cont'd. /2 ...

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

thickness from 20 to 30 feet in the north to less than 10 feet in the south, overlies a considerable thickness 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 Champlain Sea inundated the area. The underlying silty clay, known locally as "Leda Clay", was deposited by the Champlain Sea. In the area the base of the clay extends down below elevation 200. The clay stratum is underlain by a glacial till which, in turn, is underlain by grey to black shale bedrock of the Lorraine formation, Ordovician Period.

Most of the area lies within the drainage basin of the South Nation River. In general, the overburden deposits are poorly drained as evidenced by the occasional swampy and boggy area.

3. FIELD AND LABORATORY WORK:

Eight sampled boreholes, each with an accompanying dynamic cone penetration test, were carried out during 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 deposit and the glacial till deposit 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" and 3" I.D. Shelby tubes. In an effort to reduce the degree of disturbance, all the Shelby tubes were obtained by means of a piston sampler. In addition, field vane tests were carried out to determine the undrained shear strength of the clay stratum. Bedrock was proven in 6 boreholes by obtaining AXT size rock core samples.

The groundwater level conditions across the site were determined by installing sealed piezometers in two of the boreholes.

cont'd. /3 ...



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

This information was supplemented by recording the water level in the open holes at the remaining boring locations during the period of the investigation. Gas under very high pressure was encountered near the bedrock surface in the southernmost borehole (B.H. #1). The emission of gas was stopped by filling the borehole with quick-setting cement and sand.

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 Dwg. #68-P-33A, together with the estimated stratigraphical profile across the site.

All the 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 physical properties of the various soil types, namely:

- Bulk Density
- Natural Moisture Contents
- Atterberg Limits
- Grain-size Distributions
- Undrained Shear Strengths
- Consolidation Characteristics

On completion of laboratory testing, the various soil samples were classified as to type and consistency, or relative density in general, according to the Unified Soil Classification System (Oct. 1963).

The results of these tests are summarized and plotted on the Record of Borelog sheets contained in the Appendix of the report.

cont'd. /4 ...

#### 4. SOIL TYPES AND SOIL CONDITIONS:

##### 4.1) General:

The surficial deposit across the site is composed of a very loose to compact sand, some 6 to 9 ft. thick. The sand is underlain by a soft to firm sensitive clay to clayey silt approximately 64 to 67 ft. thick. The cohesive stratum is directly underlain by a 3 to 9 ft. thick deposit of glacial till consisting of clayey silt and fragments of shale and limestone. Sound bedrock was encountered at a depth of between 74 to 80 ft. below existing ground surface.

The boundaries between the various soil strata, as determined in the borings, are shown on the accompanying borehole log sheets. The stratigraphical profile, shown on Drawing #68-F-33A, is inferred from this data.

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

##### 4.2) Silty Sand to Sandy Silt - Surficial Deposit:

A surficial deposit of sand and silt was encountered at all the boring locations; the thickness of this deposit ranges from 6 ft. to 9 ft. Occasional pockets of organic matter were encountered in the upper 2 to 3 ft. of the stratum. Grain-size distribution curves for samples of the sand deposit are shown on Figure 4 in the Appendix of this report. The natural water content of the deposit, as determined from laboratory testing, varies from 20 to 38%; the higher values were obtained on samples taken from the organic pockets for which the organic matter content averaged 2%.

Standard Penetration Tests, carried out within the overall deposit, are plotted on the borelog sheets as well as on Figure 1. The results of this testing gave 'N' values which generally vary in a random fashion between 1 and 35 blows/ft. The higher values were obtained, however, in frozen soil and might not be representative of the deposit. It is, therefore, estimated that the relative density of the deposit varies randomly between very loose and compact.

cont'd. /5 ...

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

4.3) Sensitive Clay to Clayey Silt:

The surficial deposit is underlain by the predominant overburden stratum across the site, a sensitive marine clay to clayey silt with occasional inclusions of organic matter. The thickness of this stratum ranges from 60 to 64 feet. The upper and lower portions of the stratum contain layers and seams of silt up to 2 inches thick. The thicknesses of the upper and lower zones are respectively 13 to 15 ft. and 0 to 5 ft. Grain-size distribution curves for samples taken at different depths within this deposit are shown on Figure 5 in Appendix I.

The engineering properties of the stratum, as determined by field and laboratory testing, are summarized on Fig. 1; a brief resume, presented in tabular form, follows:

		<u>Range</u>	<u>Average</u>
Bulk Density (p.c.f.)	( $\gamma$ )	90.5 - 117.5	96
Liquid Limit (%)	( $W_L$ )	30 - 92	60
Plastic Limit (%)	( $W_P$ )	18 - 39	25
Natural Water Content (%)	( $W$ )	25 - 92	70
Liquidity Index	( $I_L$ )	0.5 - 3.0	0.8
Initial Void Ratio	( $e_o$ )	2.14 - 2.2	
Compression Index	( $C_c$ )	2.66 - 2.94	
Overconsolidation Ratio	$\left( \frac{P_c - P'_o}{P'_o} \right)$	0.3 - 0.7	
Undrained Shear Strength	( $C_u$ )	<u>Range</u>	<u>Sensitivity (S)</u>
		( $C_u$ )	
1) Field Vanes	: 280 - >2000	4 - 30	
2) Laboratory Vanes	: 160 - 1200	3 - >25	
3) Laboratory Testing	: 240 - 960	-	

cont'd. /6 .

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

4.3) Sensitive Clay to Clayey Silt: (cont'd.) ...

The Atterberg limits tests, summarized on the previous page, are also plotted on the Plasticity Chart, Figure 7. These results indicate that the clay is inorganic and of low to high plasticity. The natural water content, throughout the deposit, is, in general, at or above the liquid limit. The consistency of the stratum, as determined from the undrained shear strength testing, increases in a linear fashion from soft, immediately below the surface deposit, to stiff to very stiff with depth. This increase in undrained shear strength is represented by an average  $C_u/P_o$  ratio of about 0.6, where  $P_o$  is the effective overburden pressure. The undrained shear strength values obtained from the laboratory testing, gave consistently lower values than those 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 obtained by carrying out two laboratory consolidation tests, the results of which are shown as Void Ratio vs. Pressure plots on Figures 9 and 10. The results of this testing indicate that the clay is preconsolidated by approx. 400 to 600 p.s.f. with respect to the existing overburden pressure. 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".

4.4) Clayey Silt with Fractured Shale and Limestone (Glacial Till Derived from Bedrock):

This heterogeneous deposit was encountered immediately below the sensitive clay deposit. The total thickness of the deposit varies from 3.5 to 9 feet. This glacial till is composed of reworked material primarily derived from the underlying bedrock. Further, the composition of the deposit infers that it is a

cont'd. /7 ...

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

4.4) Clayey Silt with Fractured Shale and Limestone (Glacial Till Derived from Bedrock): (cont'd.) ...

transitional zone between the overlying sensitive clay and the underlying bedrock. Grain-size distribution curves for two composite samples of the deposit are shown on Figure 6.

Atterberg limit tests carried out on samples obtained from the more cohesive portions of the deposit are plotted on the Plasticity Chart, Figure 8. These tests gave values for the liquid and plastic limits that range from 22 to 25 and 16 to 18, respectively. The corresponding natural water content is typically 5 to 10% below the plastic limit.

The standard penetration tests carried out within this deposit, gave 'N' values ranging from 42 blows/ft. to 100 blows/5 inches. Based on these results, it is estimated that the consistency of the cohesive portions of this deposit is hard.

4.5) Shale Bedrock:

Grey calcareous shale bedrock with layers of shaley dolomite was encountered directly below the glacial till. The surface of the bedrock was some 74 to 79 ft. below ground surface - i.e., between elevations 176 and 179. The bedrock was penetrated for depths of between 5 and 8 ft. at six of the boring locations by diamond core drilling in AXT size. The bedrock is basically sound as evidenced by the high core recoveries (60 to 90%).

5. GROUNDWATER CONDITIONS:

Groundwater level observations have been carried out during the period of investigation in: i) sealed piezometers installed in B.H.'s #4 and #7, and ii) the open holes at the remaining boring locations. The observations are recorded on the borelog sheets and summarized on Drawing 63-P-33A. The results of the measurements indicate that the piezometric groundwater level within the surficial sand and underlying clay stratum is

cont'd. /8 ...

5. GROUNDWATER CONDITIONS: (cont'd.) ...

at about elevation 252 - i.e., some 3 ft. below ground surface. The corresponding piezometric groundwater level within the glacial till was between elevations 243 and 244 - i.e., some 10 to 11 ft. below the ground surface.

6. DISCUSSION AND RECOMMENDATIONS:

6.1) General:

It is proposed to construct an underpass structure to carry Boundary Rd. over Hwy. #417. Present proposals call for a 5-span (40'-83'-82'-81'-40') structure. The maximum proposed profile grade of Boundary Rd., in the vicinity of the crossing, is elevation 281. At this grade the associated approach embankments will have a maximum height of 25.5 ft. above ground surface. The embankments will have a crest width of 36 ft.

The East and Westbound lanes of Hwy. #417 will initially have three 12-ft. wide paved lanes (one a collector lane) with provision for a fourth lane; the roadway cross-section will also incorporate 11-foot wide shoulders. The finished grade will be elevated 4 to 5 ft. above surrounding ground level - i.e., it will be between elevation 258 and 259 in the vicinity of the crossing.

The subsoil conditions at the site consist of a surficial cover of sand, 6 to 9 ft. thick, followed by the predominant deposit across the site, a soft to stiff sensitive marine clay to clayey silt about 60 ft. thick. The clay is underlain by a glacial till deposit followed by shale bedrock.

The presence of the soft, highly compressible clay stratum at relatively shallow depth below ground surface requires that steps must be taken to ensure overall stability of the approach embankments, and that the structure must be supported on piled foundations. Since the stability and settlement of the approach fills are the major problems at this site, they will be discussed first.

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

6.2) Approach Embankments:

6.2.1) Stability Considerations:

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

cont'd. /10 ...

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

6.2) Approach Embankments: (cont'd.) ...

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

1) Soil Properties:

<u>Elevation</u>	<u>Soil</u>	<u>Density (p.c.f.)</u>	<u>Strength Parameters</u>	
			<u>C<sub>u</sub> (p.s.f.)</u>	<u>Ø Degrees</u>
-	Embankment Fill	125	-	30
254 - 246	Sand	$\gamma = 125, \gamma' = 62.6$	-	30
246 - 239	Layered Silt and Silty Clay	$\gamma = 96, \gamma' = 33.5$	550	-
239 - 226	Clay	" "	420	-
226 - 218	"	" "	520	-
218 - 211	"	" "	640	-
211 - 207	"	" "	750	-
207 - 199	"	" "	925	-
199 - 192	"	" "	1,000	-

2) All the berms required have been assumed to be at the mid-height of the section. The surface of all berms required should slope away from the fill at a gradient of 20:1 for drainage purposes.

The stability computations, which are summarized on Figure 2 in the Appendix, are given in the following Table. The requirements listed, provide a minimum factor of safety of 1.3 with respect to stability.

cont'd. /11 ...



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

6.2) Approach Embankments: (cont'd.) ...

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

--- STABILITY OF APPROACH EMBANKMENTS ---

<u>Direction</u>	<u>Height of Fill (H) - ft.</u>	<u>Berm Requirement (L) - ft.</u>
------------------	-------------------------------------	---------------------------------------

CASE A: Multispan

Structure ~ 457 ft. in length  
(Sta. 28+60 to Sta. 33+11)

Longitudinal	21	60
Transverse	0 - 13 13 - 25	- 0 - 80

CASE B: Multispan

Structure ~ 1260 ft. in length  
(Sta. 24+90 to Sta. 37+50)

Longitudinal	13	-
Transverse	0 - 13	-

cont'd. /12 ...

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

6.2) Approach Embankments: (cont'd.) ...

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

From the stability analyses, the following conclusions have been drawn:

1) Fills less than 13 ft. in height may be constructed with standard 2:1 side slopes.

2) Fills in excess of 13 feet in height should be constructed with a single berm as shown on Figure 2 in the Appendix of this report.

3) At the proposed profile grade, berms will be required in both the longitudinal and transverse directions at the approach fill locations. The berm requirements in the longitudinal direction will entail lengthening the structure from that proposed, namely, from 327 feet to 451 feet ( refer to Case A in Table).

4) It may be advantageous to minimize the longitudinal and transverse berm requirements by limiting the height of fill. For instance, if the height of fill is limited to 13 feet, then no berms will be required (refer to Case B). This, however, would necessitate a multispan structure some 1260 feet in length. It should be noted that minimizing the heights of fill has the added advantage of reducing the settlements induced in the foundation subsoil, as discussed in detail in Section 6.2.2) of this report.

5) All the proposals discussed previously, are equally feasible with respect to the stability of the approach fills. The ultimate choice, however, will be based on economic considerations presented in detail in sub-section 6.2.2.).

6) Smooth transitions between different berm requirements should be affected as the height of fill varies.

cont'd. /13 ...

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

6.2) Approach Embankments: (cont'd.) ...

6.2.2) Settlement Considerations:

The underlying highly compressible clay stratum will undergo excessive settlement due to consolidation over a long-term period under the weight of the approach embankments. Settlement computations were, therefore, carried out, the results of which are summarized on Figure 2 in the Appendix. The maximum consolidation settlement will occur under the 23-foot high approach embankments (Case A). The computations indicate that this settlement could be as much as 10 feet under the centre-line of the embankment. If the maximum fill height is maintained at 13 feet (Case B), the total consolidation settlement will be of the order of 4-1/2 feet.

The total amount of the consolidation settlement predicted will take place over an extended period of time, probably in excess of 150 years. However, about 30 to 40% should occur within the first 1 to 3 years after construction (see plot on Figure 2). It would be advantageous, therefore, to construct the embankments first and leave them in place for as long a period as possible prior to the construction of the structure. For example, if the construction of the structure is delayed 2 years, the maximum differential settlement between the approach fills and the structure foundations should not exceed 2-1/2 feet. Constructing the embankments prior to the structure will, therefore, tend to reduce the maintenance problems associated with the immediate approaches to the structure. Keeping this in mind, it is recommended that final paving be delayed for as long a period as possible.

Computations were also carried out to determine the consolidation settlement to be expected for various heights of fill between the two limiting conditions discussed above. The results of these computations are presented on Figure 3. The structure span-length required for each fill height is also plotted

cont'd. /14 ...

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

6.2) Approach Embankments: (cont'd.) ...

6.2.2) Settlement Considerations: (cont'd.) ...

on this figure. The most feasible fill height, and corresponding span-length employed in the finalized structural design scheme, should be based on economic considerations.

6.3) Structure Foundations:

The structure piers and abutments should be supported on end-bearing piles driven to practical refusal onto or within the bedrock. The allowable pile load would be dependent on the section chosen - for example, a 12 BP 74 steel H-pile, driven to practical refusal on bedrock, could be designed to carry 90 tons/pile.

Since settlement of the proposed roadway embankments will be excessive, considerable negative skin friction loads may be imposed on the piles supporting the abutments and end pier. This can be dealt with by subtracting the imposed skin friction load from the bearing capacity of the pile - i.e., net bearing capacity for a 12 BP 74 steel H-pile will be reduced to 75 tons. Precautions can also be taken to prevent the mobilization of negative skin frictional components. A pre-augering technique, has in the past, proved successful in reducing the negative skin friction in extensive deposits of "Leda Clay".

In addition to the negative skin frictional forces, movement of subsoil due to strain imposed by the embankment loading, will generally tend to displace the piles laterally and can cause rotation of the abutments. In view of this, we recommend that consideration be given to supporting the extreme ends of the wing walls on end-bearing piles founded as aforementioned. It is considered that this will improve the stability in the longitudinal direction. No bouldery or rock fill should be placed in areas where piles are to be driven.

cont'd. /15 ...

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

6.3) Structure Foundations: (cont'd.) ...

Pile caps should be founded at sufficient depth below finished grade so as to ensure adequate frost protection.

No major dewatering problems are anticipated. Excavations for the pier pile caps may, however, be carried out below the groundwater level, which is about 3 feet below ground surface. Because these excavations will be carried out through a relatively pervious sand deposit, seepage may occur. This can be dealt with by pumping from sumps or, alternatively, by excavating from within closed timber sheeting.

7. SUMMARY:

A foundation investigation at the site of the proposed underpass structure to carry Boundary Rd. over proposed Hwy. #417, in the Township of Gloucester, County of Carleton, is reported.

Underlying between 6 to 9 feet of sand, is the predominant overburden stratum across the site, a soft to very stiff, sensitive marine clay to silty clay 60 to 65 ft. thick. The clay is underlain by glacial till followed by shale bedrock 74 to 80 ft. below ground surface.

The groundwater level in the surficial sand deposit and underlying clay stratum was, at the time of the investigation, approximately 3 ft. below ground surface.

It is recommended that the piers and abutments be supported on end-bearing piles driven to practical refusal onto or within the bedrock. Construction procedures have been outlined in this report.

Detailed recommendations have been made regarding the procedures necessary to ensure stability of the approach fills. Berms will be required in both the longitudinal and transverse directions for fills in excess of 13 ft. in height. The berm requirements in the longitudinal direction will entail an increase

cont'd. /16 ...

7. SUMMARY: (cont'd.) ...

in the length of the structure over that proposed, as discussed in the report.

Settlements of up to 10 ft. are estimated for a maximum fill height of 23 ft. In order to reduce the differential settlements, consideration should be given to constructing the approach fills well in advance of the structure foundations, as discussed in the report.

8. MISCELLANEOUS:

The field work for this project was carried out during the period April 17 to 26, 1968, under the supervision of Mr. P. B. Schnabel, Project Foundation Engineer, who also wrote this report.

The equipment was owned and operated by F. E. Johnston Drilling Co. Ltd.

The investigation was carried out under the general supervision of Mr. M. Devata, Supervising Foundation Engineer, who also reviewed this report.

June, 1968.

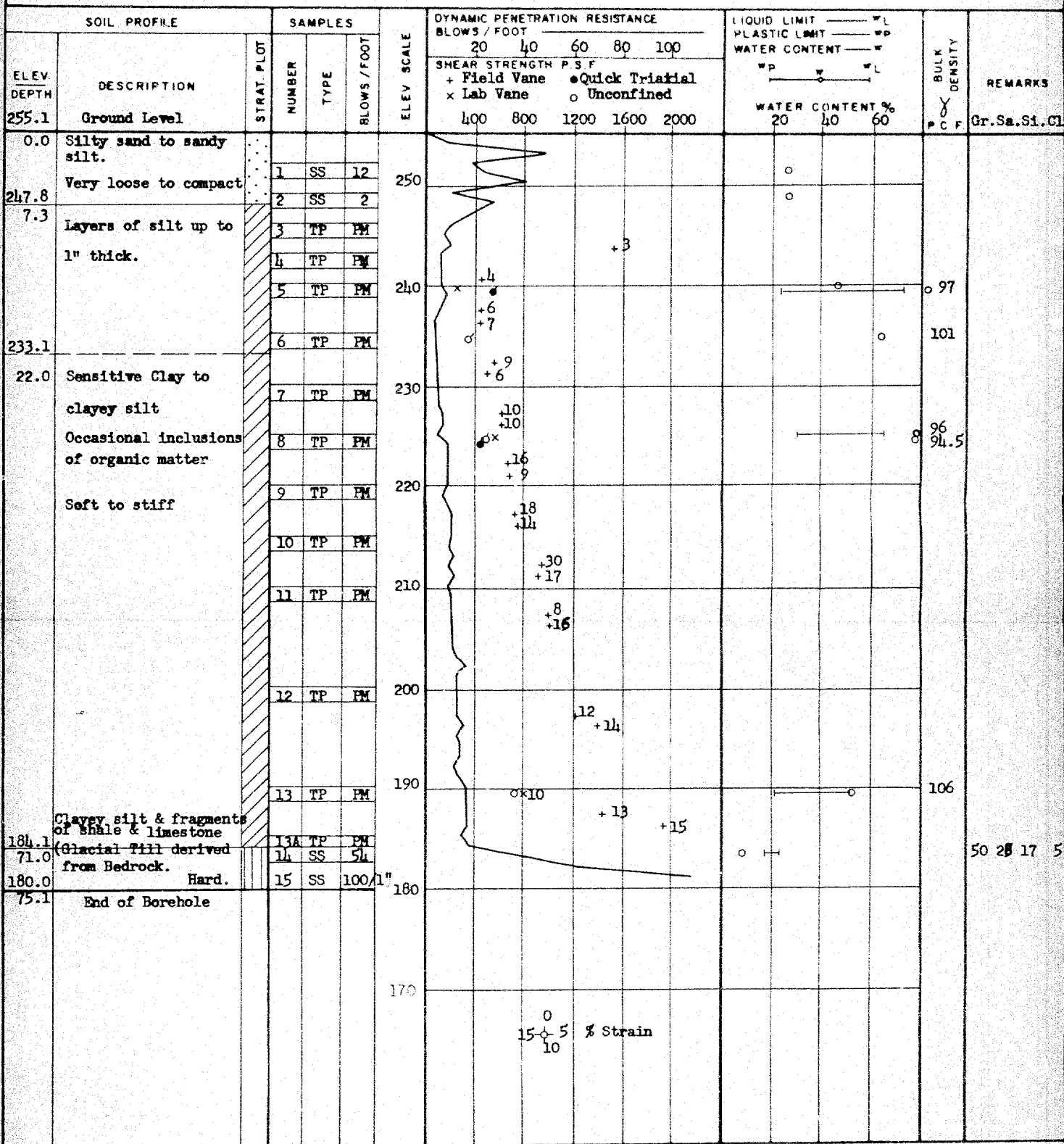
APPENDIX I

DEPARTMENT OF HIGHWAYS - ONTARIO  
MATERIALS & TESTING DIVISION

## RECORD OF BOREHOLE NO. 1

FOUNDATION SECTION

JOB 68-F-33 LOCATION Sta. 34 + 50 o/s 8' ORIGINATED BY FBS  
W P 34-66-09 BORING DATE April 18-19, 1968 COMPILED BY FBS  
DATUM Geodetic BOREHOLE TYPE Diamond Drill - NX BX Casing CHECKED BY [Signature]



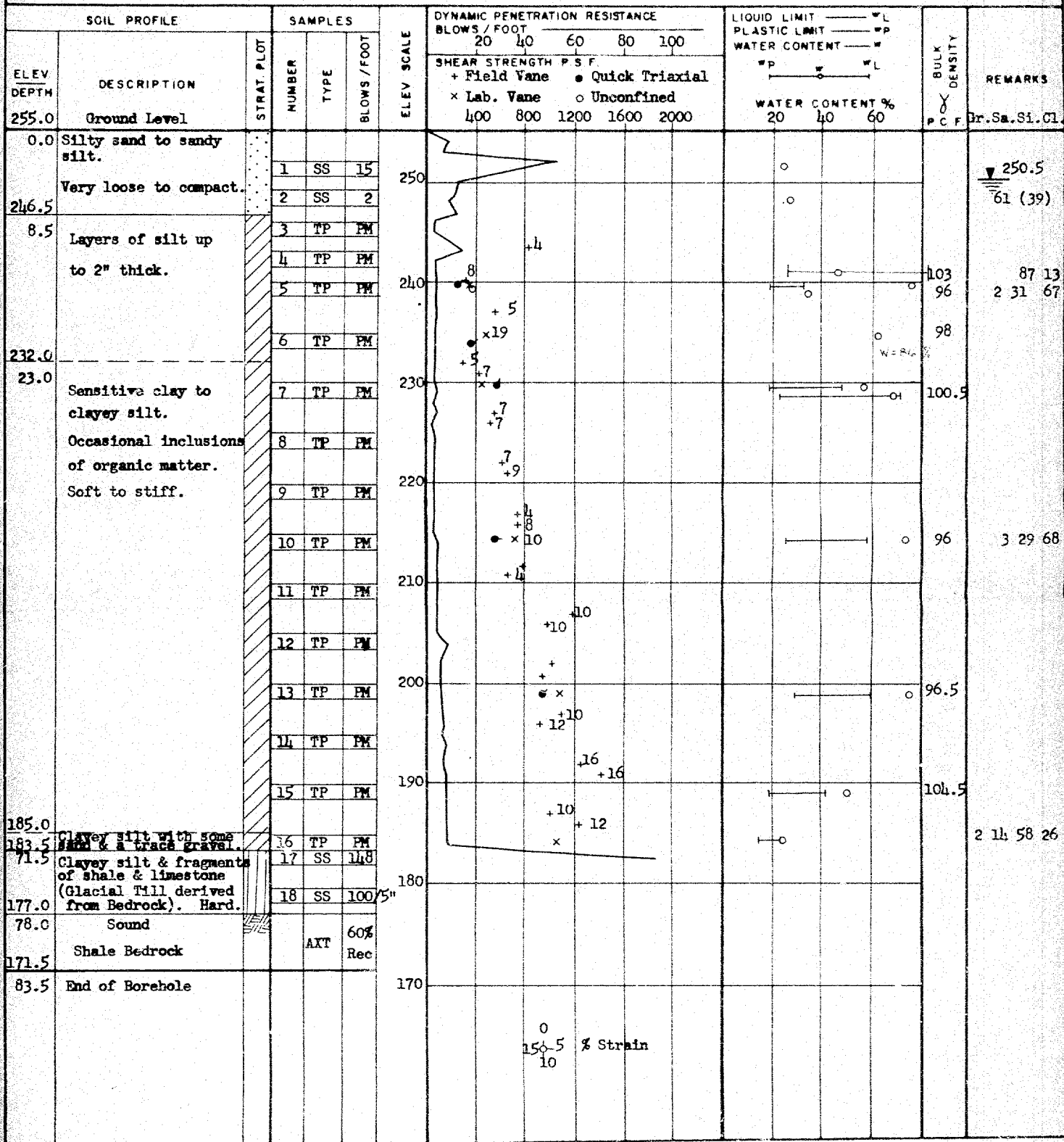


DEPARTMENT OF HIGHWAYS - ONTARIO  
MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 2

FOUNDATION SECTION

JOB 68-P-33 LOCATION Sta. 32 + 55 o/s 8' Lt. ORIGINATED BY PBS  
W.P. 34-65-09 BORING DATE April 17-18, 1968 COMPILED BY PBS  
DATUM Geodetic BOREHOLE TYPE Diamond Drill, HX & BX Casing, AXT Core CHECKED BY [Signature]



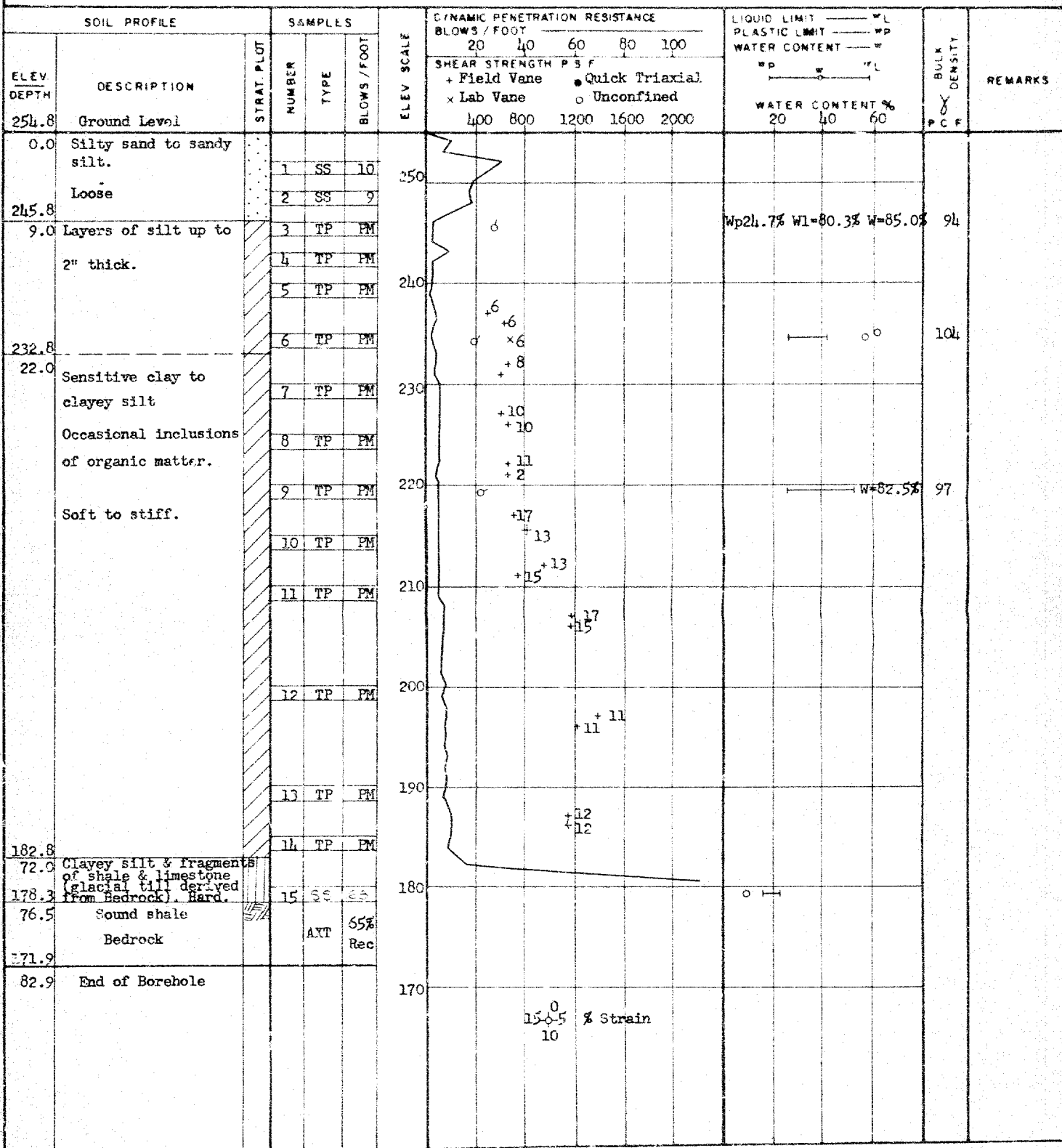
DEPARTMENT OF HIGHWAYS - ONTARIO

## MATERIALS &amp; TESTING DIVISION

## RECORD OF BOREHOLE NO. 3

FOUNDATION SECTION

JOB 68-F-33 LOCATION Sta. 31 + 33 o/s 8' Lt. ORIGINATED BY PBS  
 W P 34-66-09 BORING DATE April 24-25, 1968 COMPILED BY PBS  
 DATUM Geodetic BOREHOLE TYPE Diamond Drill - NX, BX Casing - AXT Core CHECKED BY LL



DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS &amp; TESTING DIVISION

## RECORD OF BOREHOLE NO. 4

FOUNDATION SECTION

JOB 68-F-33

LOCATION Sta. 30 + 39 o/s 8' Rt.

ORIGINATED BY PBS

W.P. 34-66-09

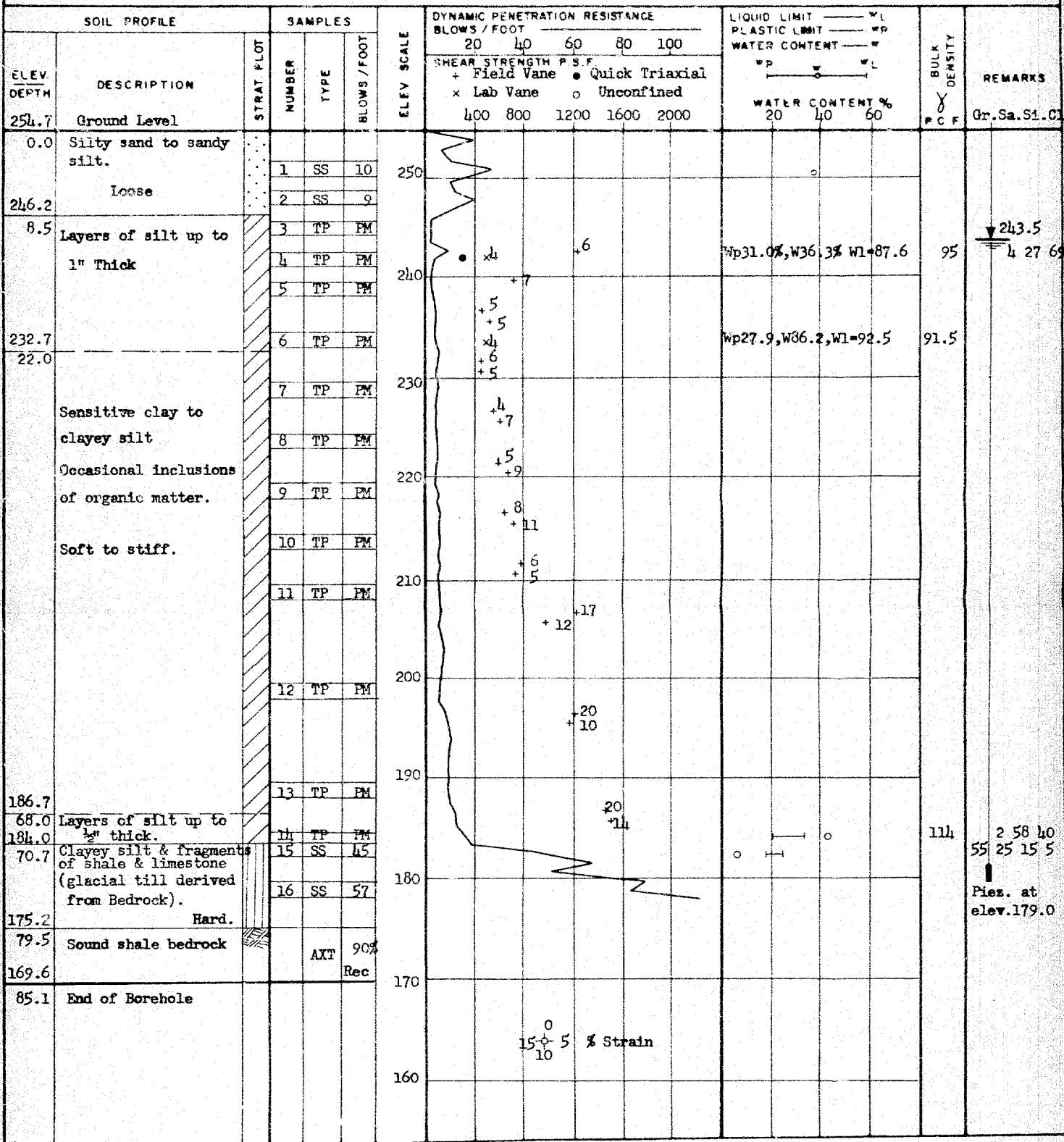
BORING DATE April 18-19, 1968

COMPILED BY PBS

DATUM Geodetic

BOREHOLE TYPE Diamond Drill - NX, BX Casing, AXT Core

CHECKED BY



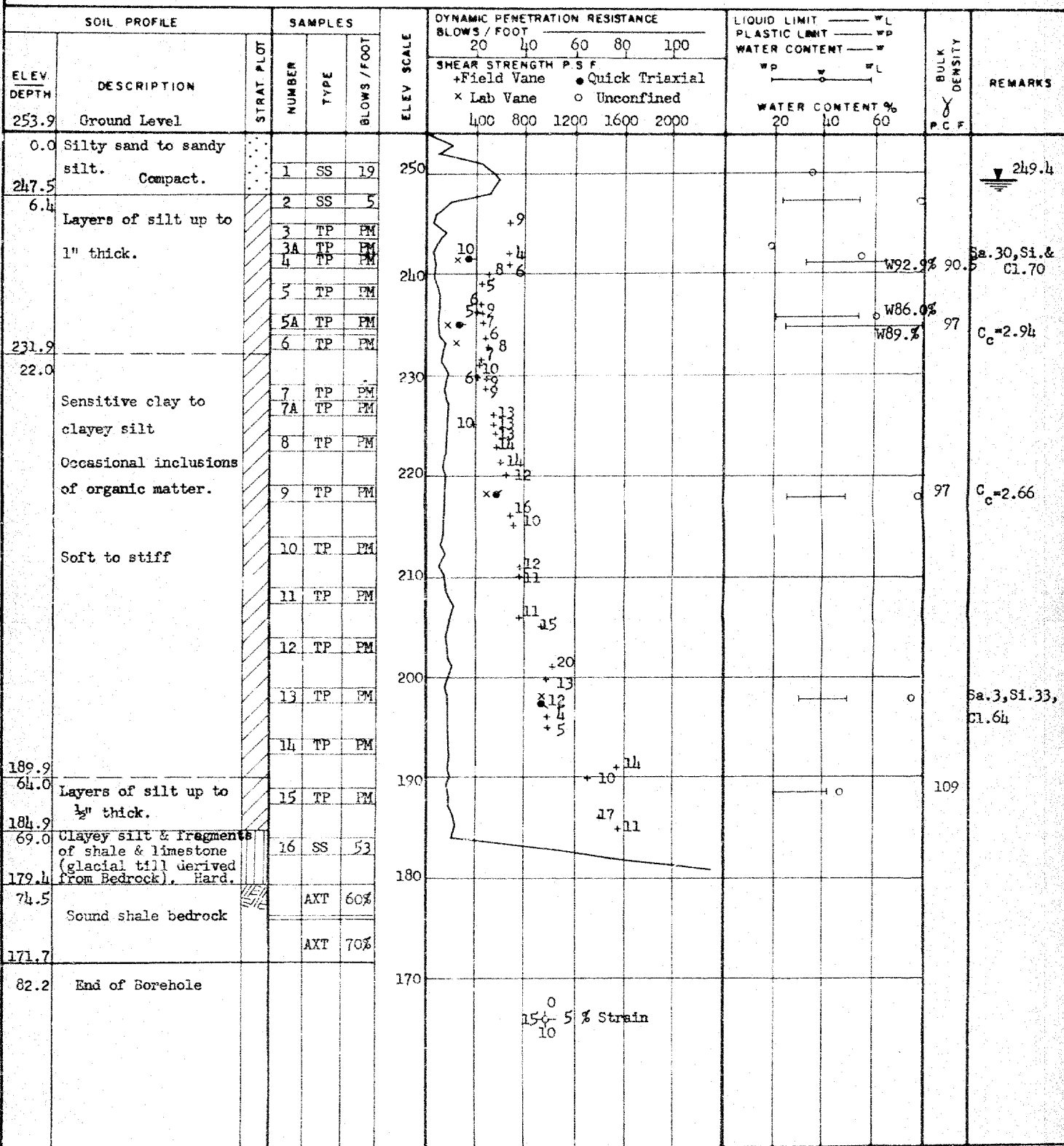
DEPARTMENT OF HIGHWAYS - ONTARIO

## MATERIALS &amp; TESTING DIVISION

## RECORD OF BOREHOLE NO. 5

FOUNDATION SECTION

JOB 68-F-33 LOCATION Sta. 29 + 10 o/s 8' Rt. ORIGINATED BY PBS  
W.P. 66-34-09 BORING DATE April 17-18, 1968 COMPILED BY PBS  
DATUM Geodetic BOREHOLE TYPE Diamond Drill - NX, BX Casing - AXT Core CHECKED BY JK



DEPARTMENT OF HIGHWAYS - ONTARIO

## MATERIALS &amp; TESTING DIVISION

## RECORD OF BOREHOLE NO. 6

FOUNDATION SECTION

JOB 68-F-33 LOCATION Sta. 27 + 20 o/s 8' Lt.

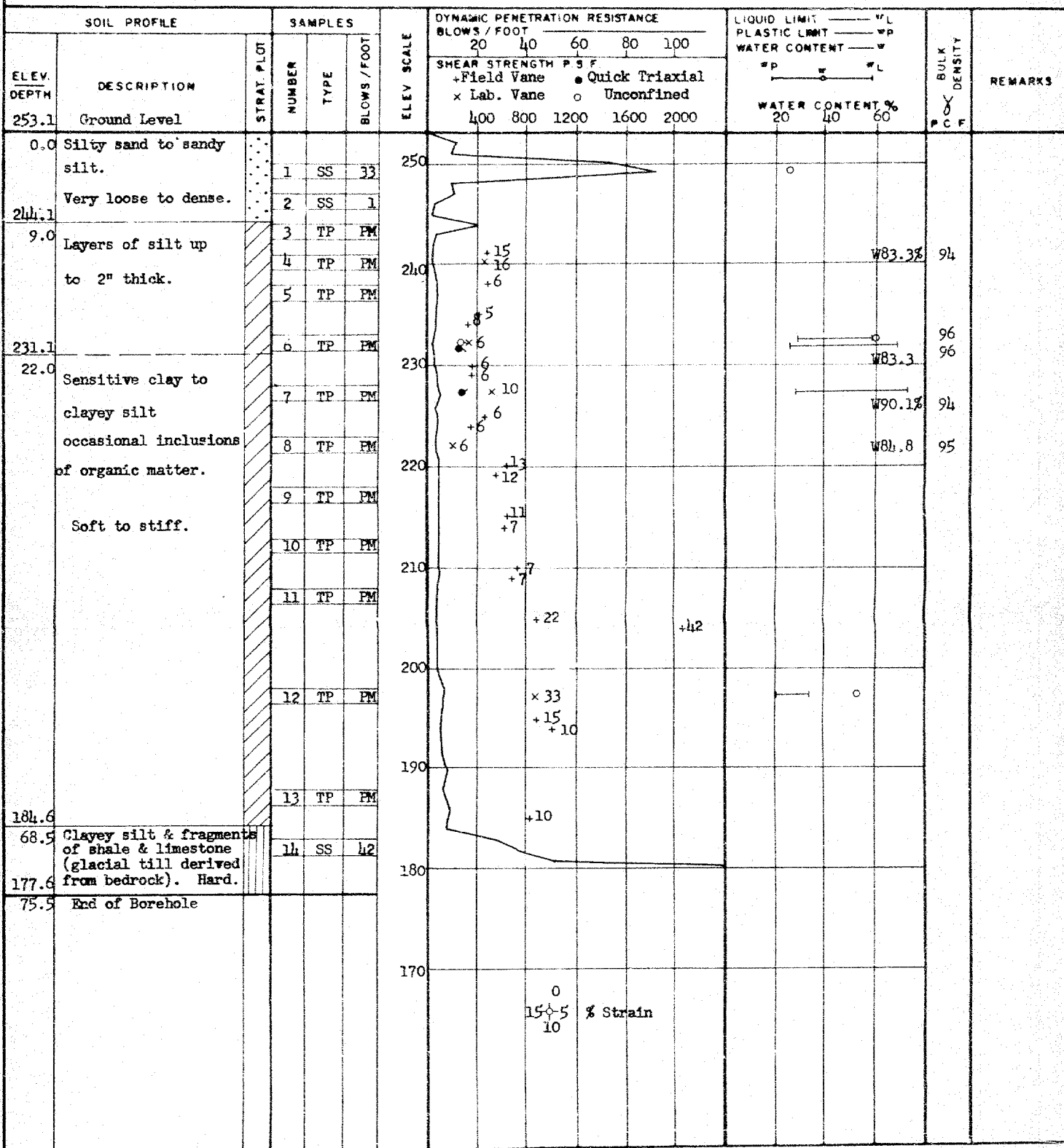
ORIGINATED BY PBS

W.P. 66-34-09 BORING DATE April 22-23, 1968

COMPILED BY PBS

DATUM Geodetic BOREHOLE TYPE Diamond Drill - HX, BX Casing

CHECKED BY



DEPARTMENT OF HIGHWAYS - ONTARIO

## MATERIALS &amp; TESTING DIVISION

## RECORD OF BOREHOLE NO. 7

FOUNDATION SECTION

JOB 68-F-33

LOCATION Sta. 32 + 04 o/s 8' Rt.

ORIGINATED BY PBS

W.P. 66-34-09

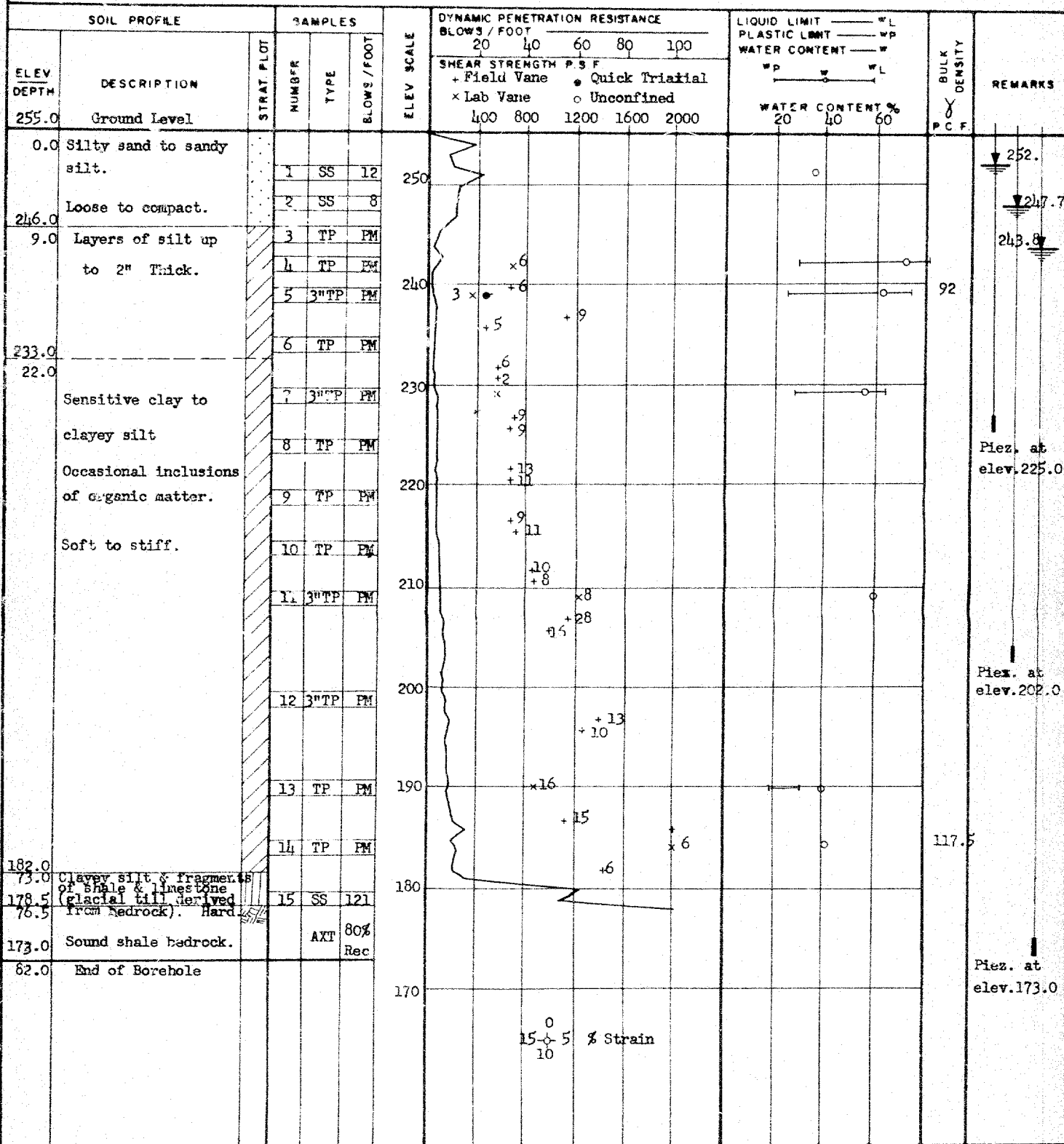
BORING DATE April 22-23, 1968

COMPILED BY PBS

DATUM Geodetic

BOREHOLE TYPE Diamond Drill - HX, BX Casing - AXT Core

CHECKED BY



DEPARTMENT OF HIGHWAYS - ONTARIO

## MATERIALS &amp; TESTING DIVISION

## RECORD OF BOREHOLE NO. 8

FOUNDATION SECTION

JOB 68-F-33

LOCATION Sta. 29 + 69 o/s 8' Lt.

ORIGINATED BY PBS

W.P. 66-34-09

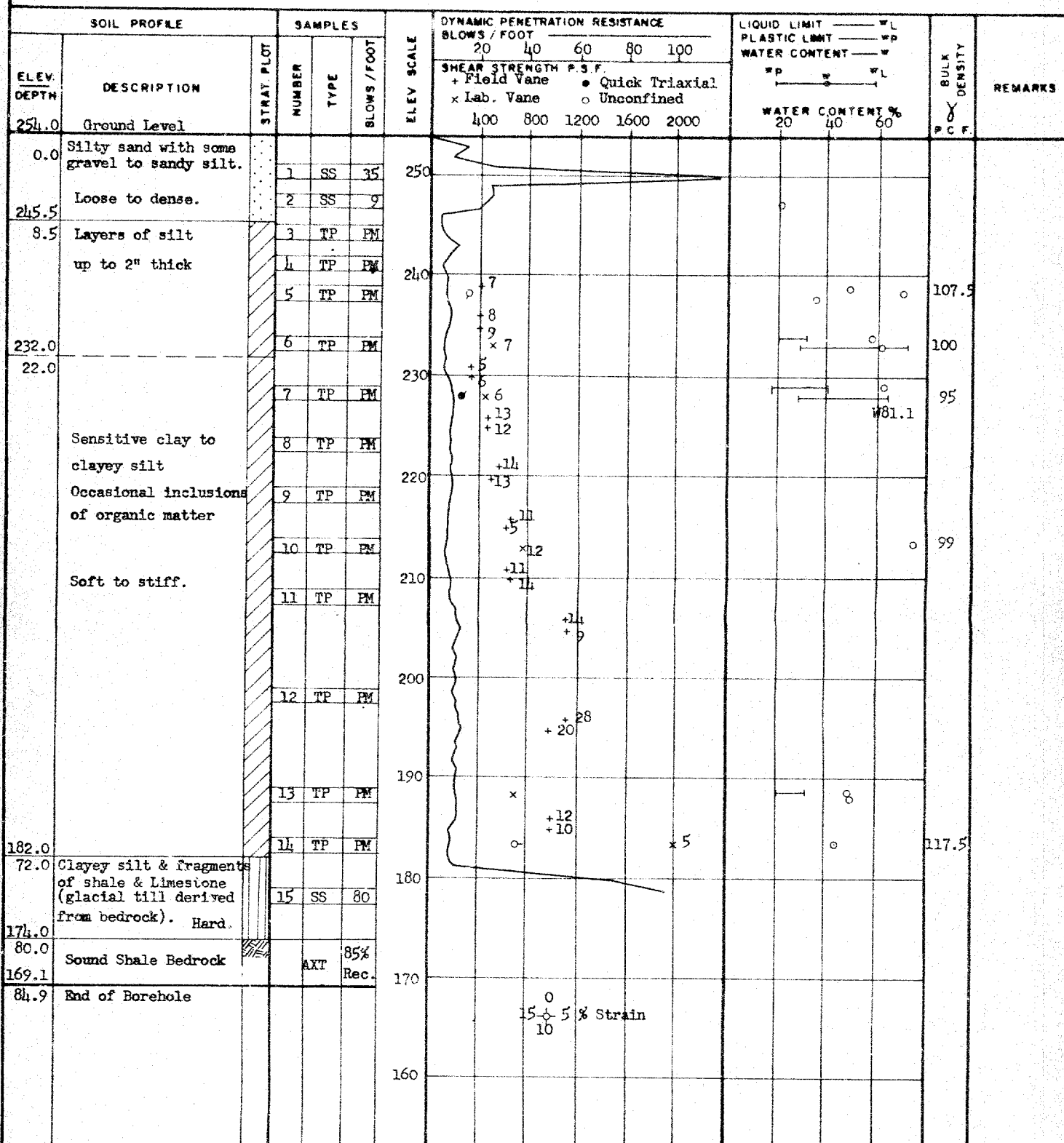
BORING DATE April 24 - 25, 1968

COMPILED BY PBS

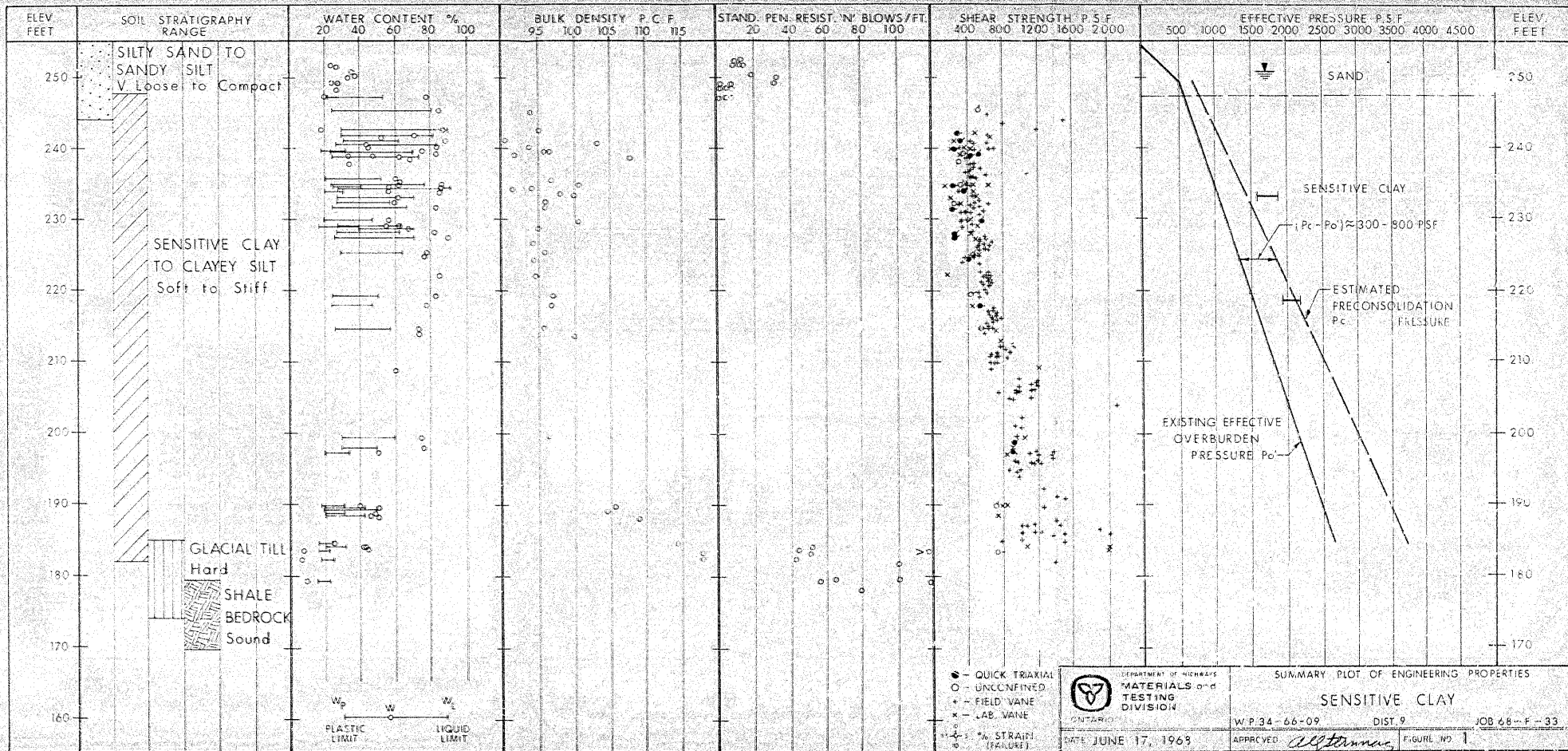
DATUM Geodetic

BOREHOLE TYPE Diamond Drill - HX, BX Casing

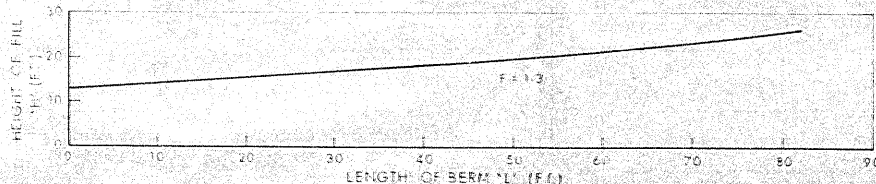
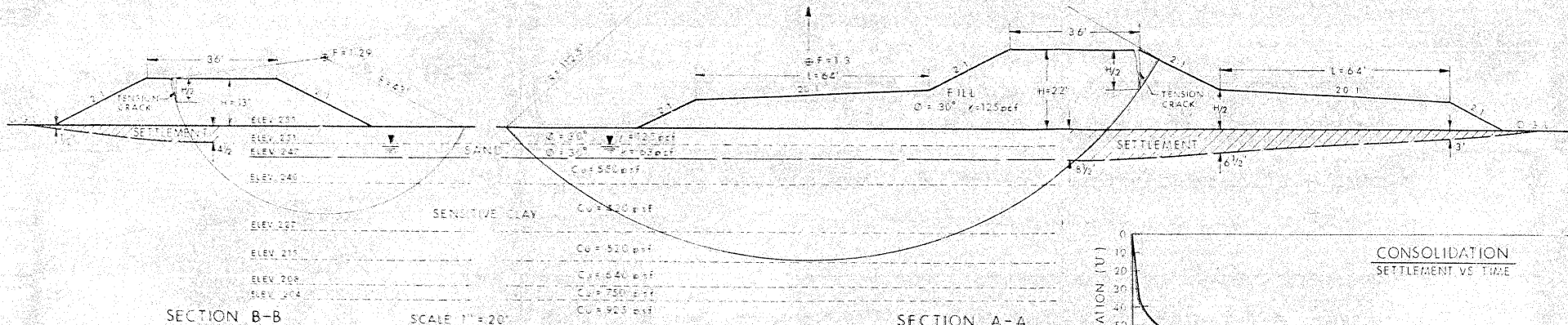
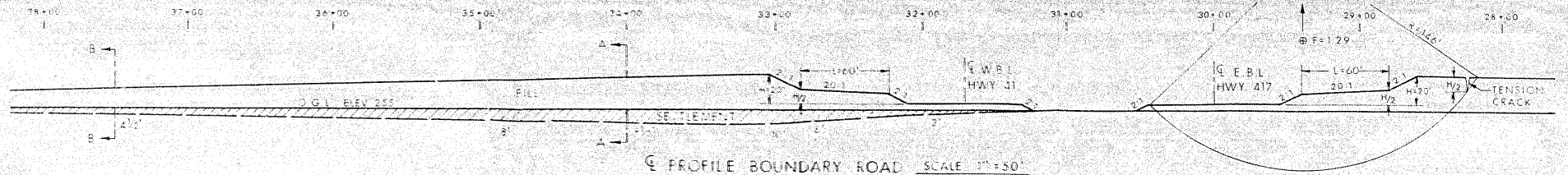
CHECKED BY







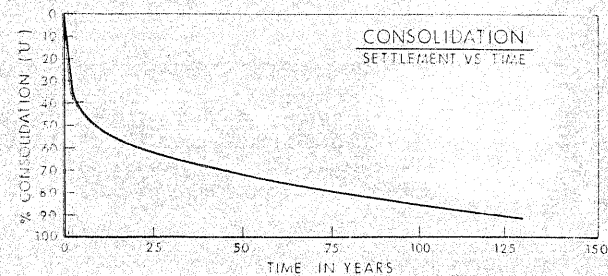




BERM REQUIREMENTS

#### LEGEND

- H - HEIGHT OF FILL (FT.)
- L - LENGTH OF BERM (FT.)
- F - FACTOR OF SAFETY
- ⊕ - CENTRE OF CRITICAL CIRCLE
- R - RADIUS OF CIRCLE (FT.)



DEPARTMENT OF HIGHWAYS  
MATERIALS AND  
TESTING  
DIVISION

DATE: JUNE 12, 1968

SUMMARIZED RESULTS OF  
STABILITY & SETTLEMENT ANALYSES  
(APPROACH EMBANKMENTS)

W.P. 34-66-09

DIST. 9

JOB 68-F-33

APPROVED: [Signature]

FIGURE NO. 2

# SUMMARIZED RESULTS OF SETTLEMENTS EXPECTED FOR VARIOUS HEIGHTS OF FILL

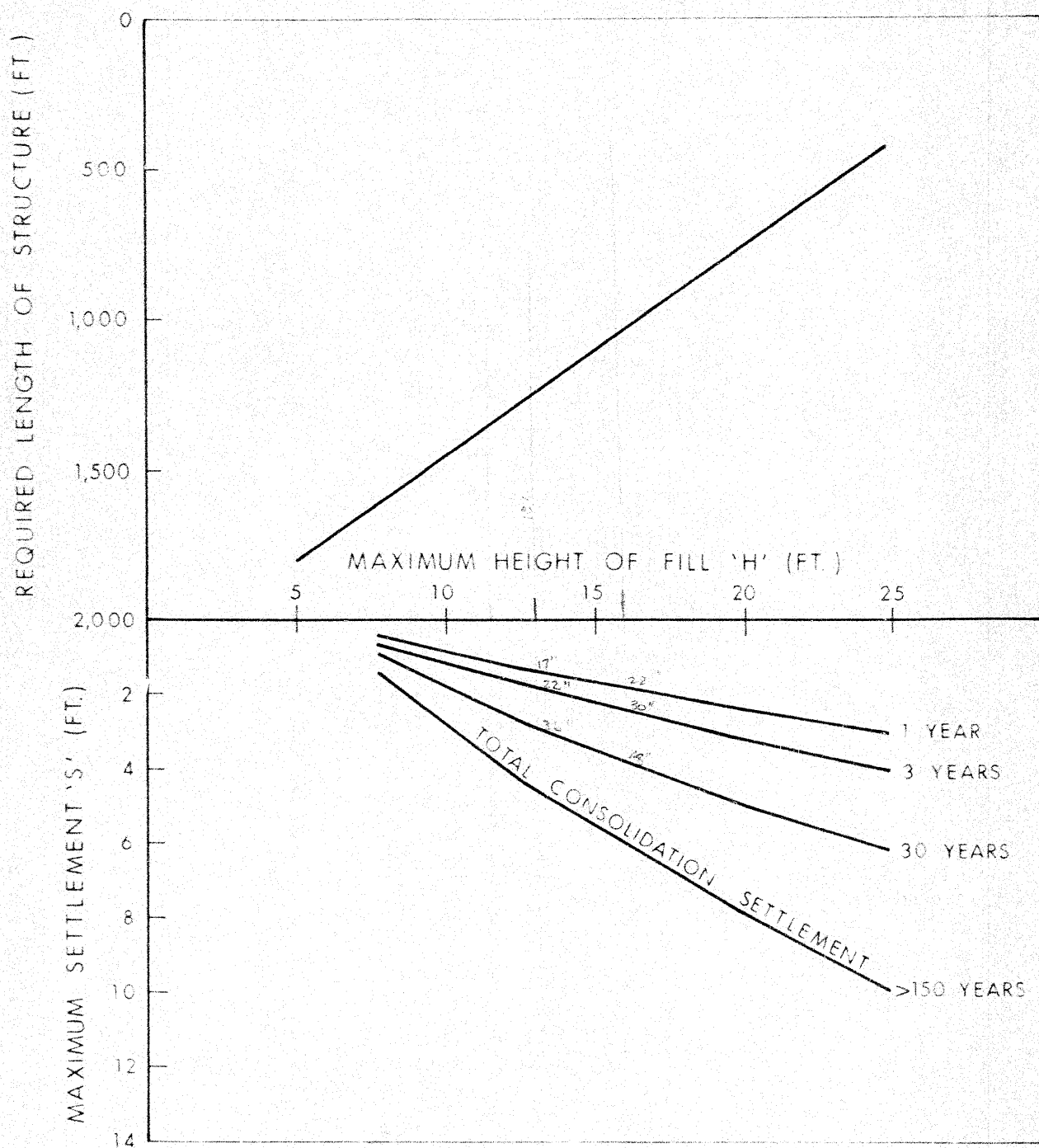
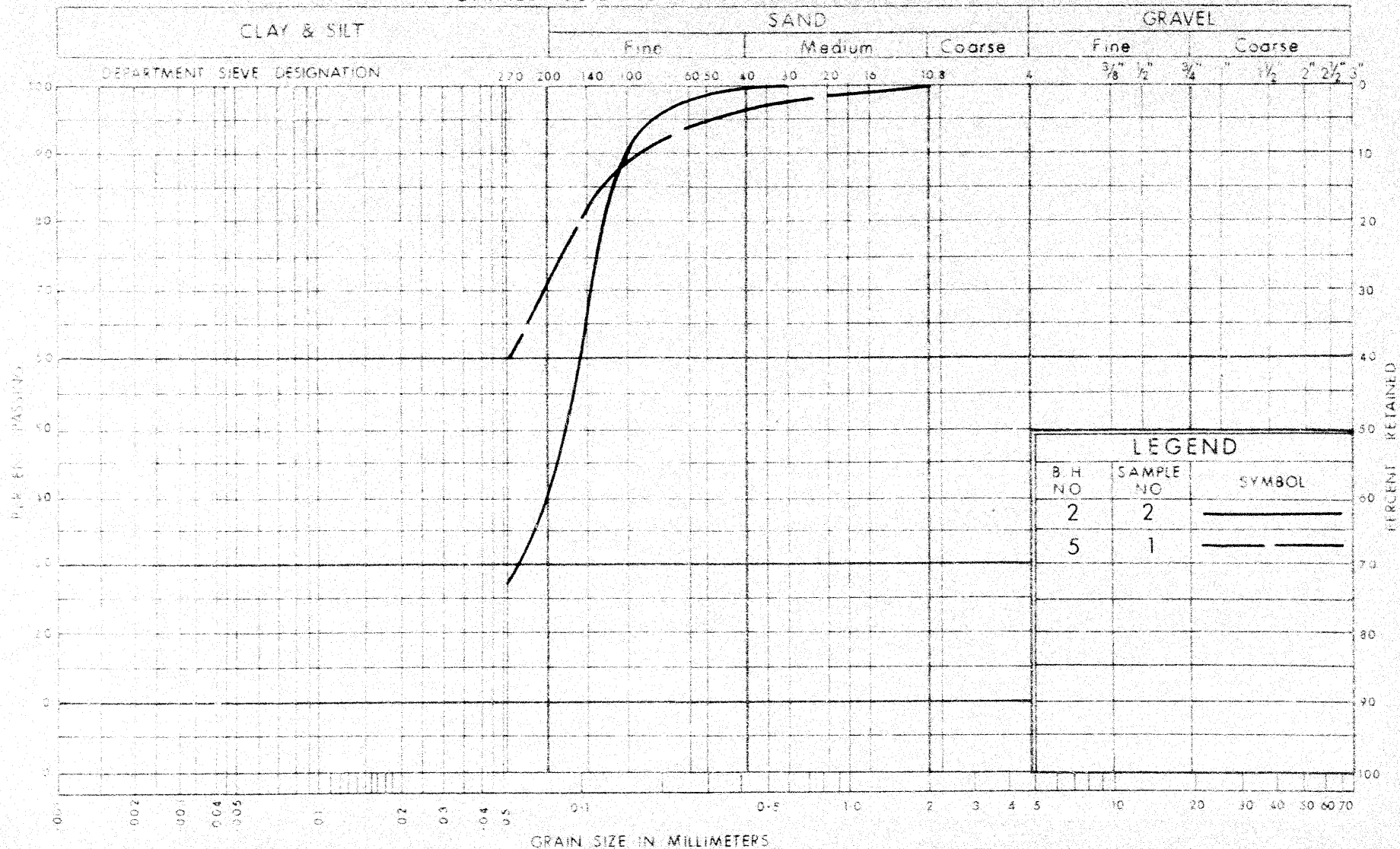


FIG. 3

# UNIFIED SOIL CLASSIFICATION SYSTEM



DEPARTMENT OF HIGHWAYS  
MATERIALS and  
TESTING  
DIVISION

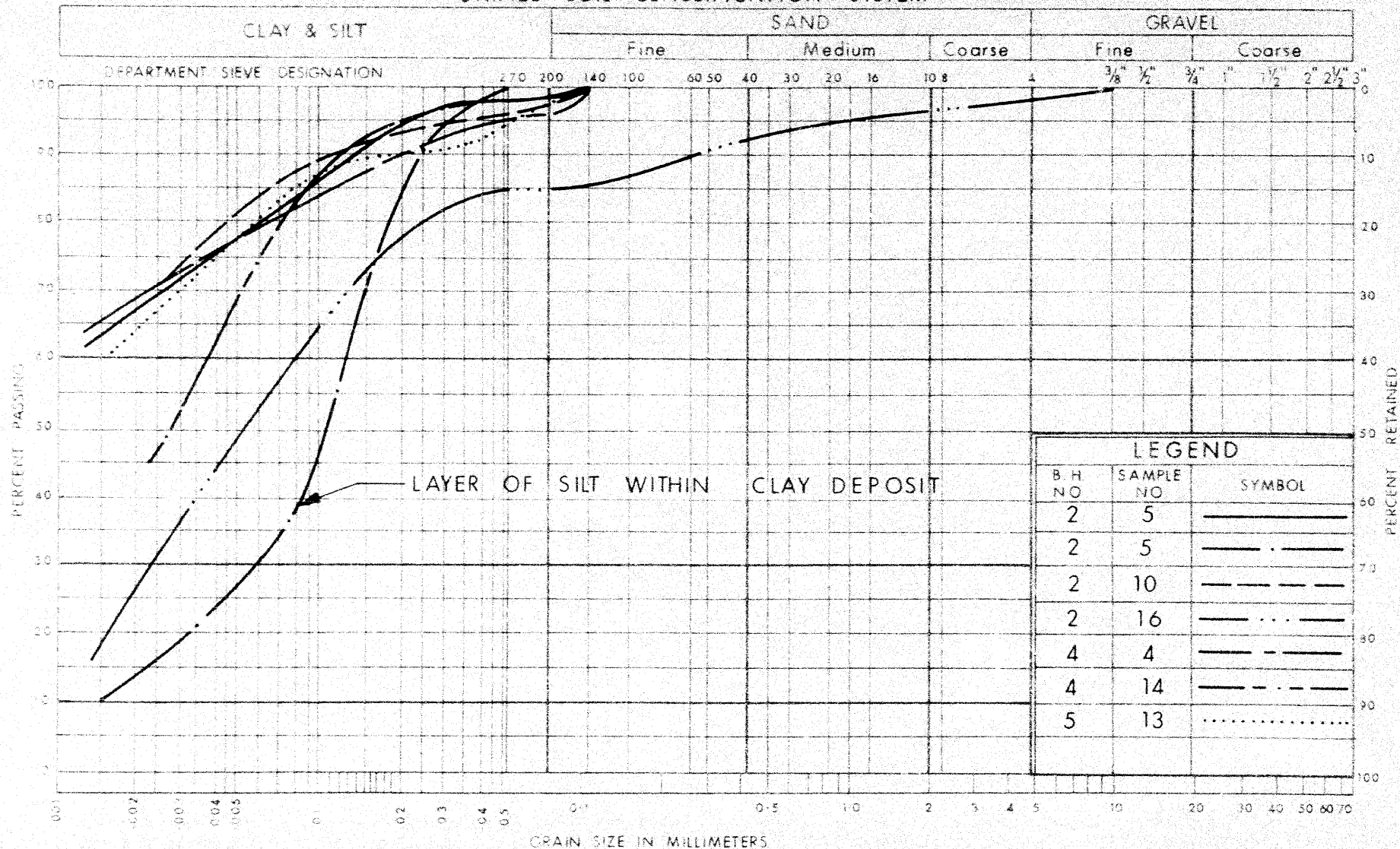
GRAIN SIZE DISTRIBUTION  
SILTY SAND TO SANDY SILT

W.P. No. 34-66-09

JOB No. 68-F-34

FIG. NO. 4

# UNIFIED SOIL CLASSIFICATION SYSTEM



DEPARTMENT OF HIGHWAYS  
MATERIALS and  
TESTING  
DIVISION

## GRAIN SIZE DISTRIBUTION SENSITIVE CLAY TO CLAYEY SILT

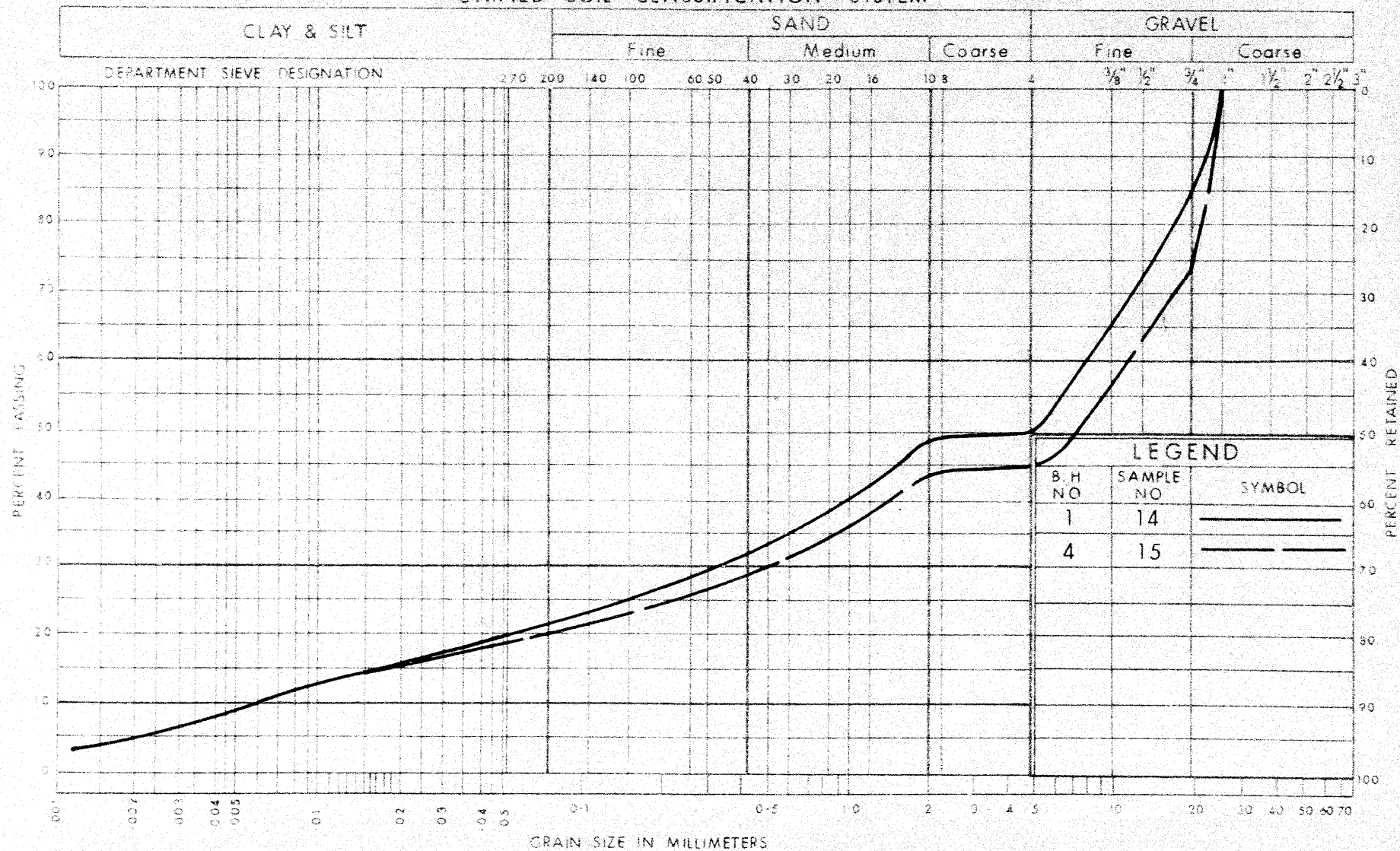
W.P. No. 34-66-09

JOB No. 68-F-33

FIG. NO. 5



# UNIFIED SOIL CLASSIFICATION SYSTEM



DEPARTMENT OF HIGHWAYS  
MATERIALS and  
TESTING  
DIVISION

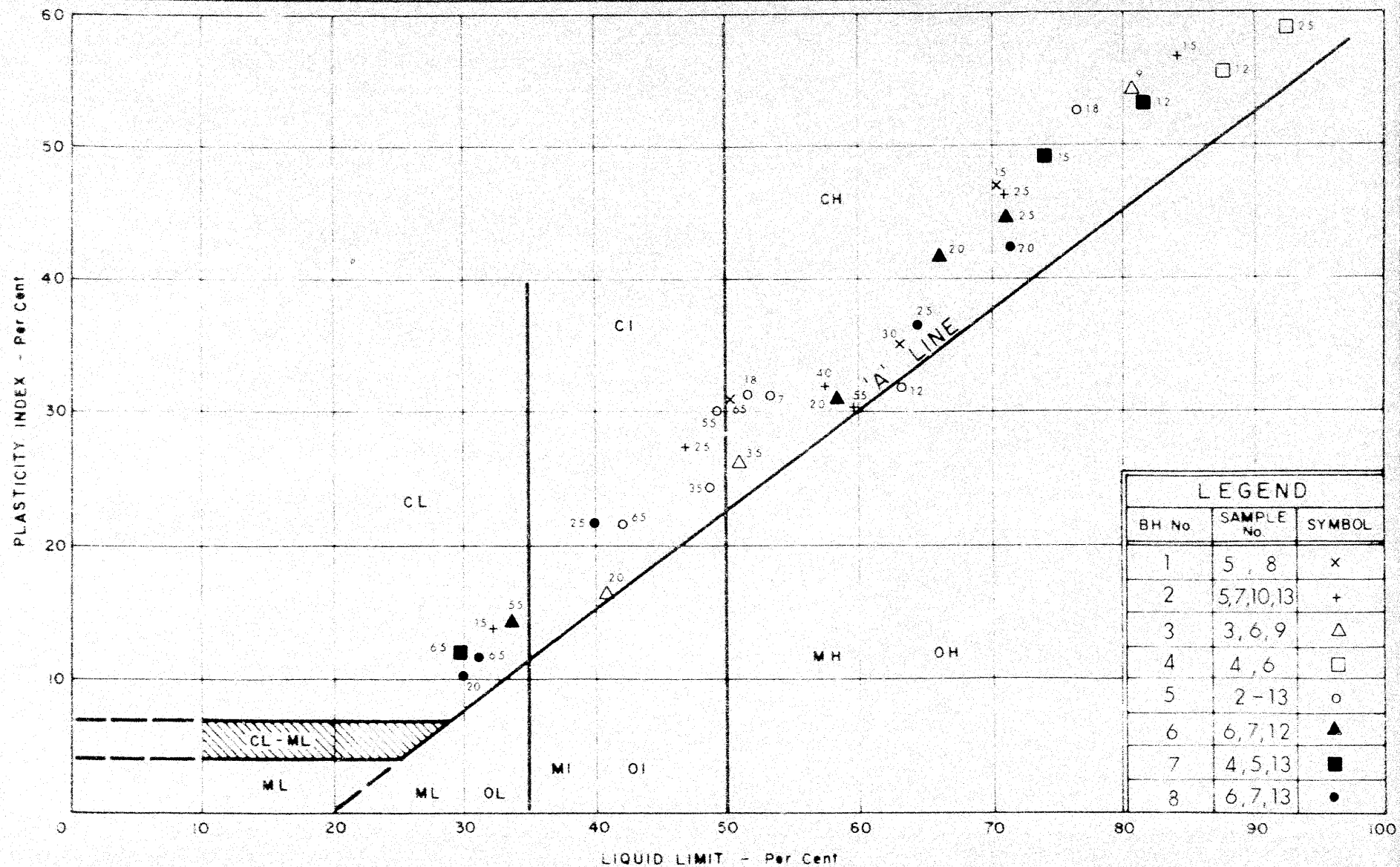
## GRAIN SIZE DISTRIBUTION GLACIAL TILL

CLAYEY SILT, FRAGMENTS OF SHALE AND LIMESTONE

W.P. No. 34-66-09

JOB No. 68-F-33

FIG. NO. 6



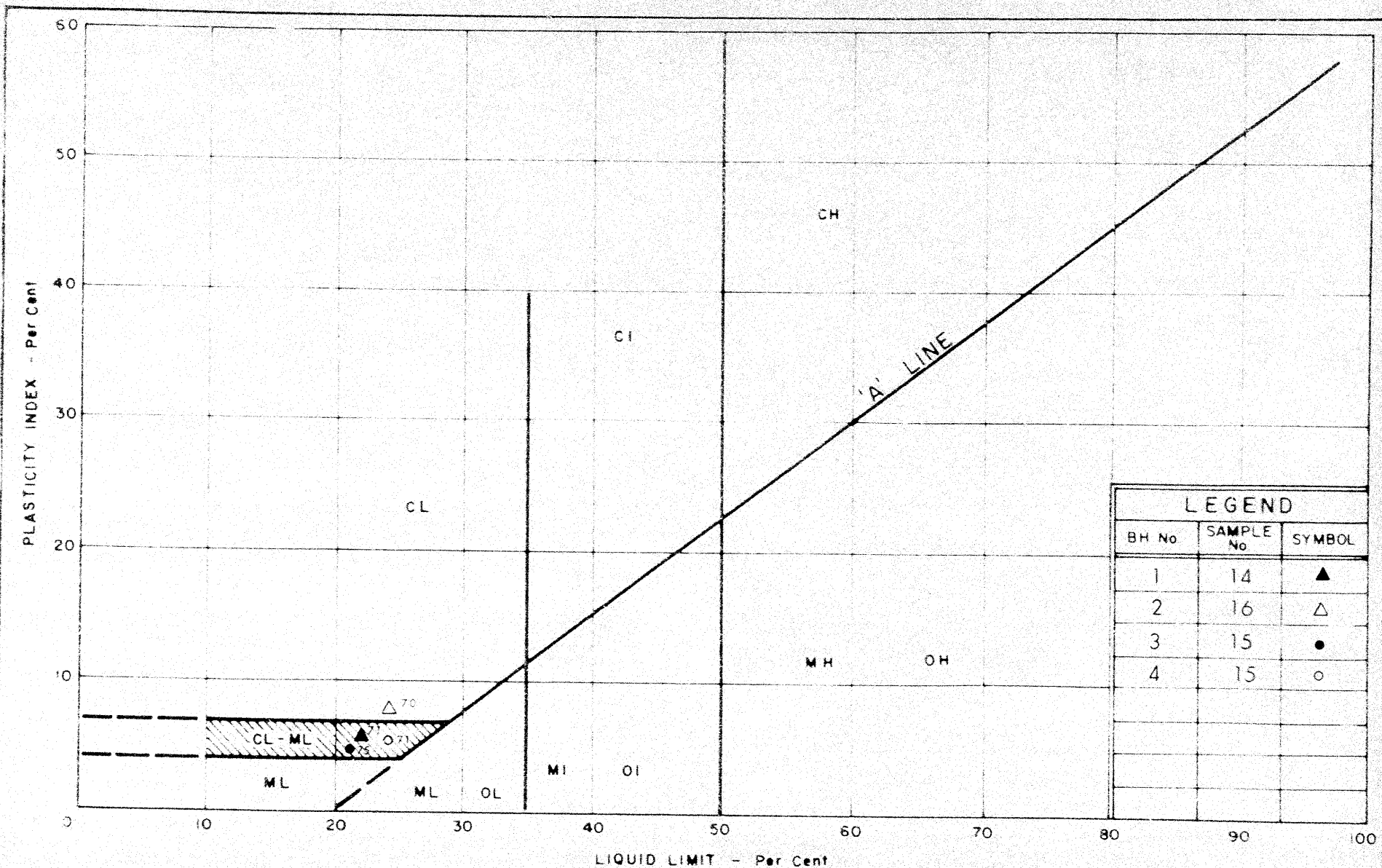
DEPARTMENT OF HIGHWAYS  
MATERIALS and  
TESTING  
DIVISION

# PLASTICITY CHART SENSITIVE CLAY TO CLAYEY SILT

WP No. 34-66-09

JOB No. 68-F-33

FIG NO. 7



DEPARTMENT OF HIGHWAYS  
MATERIALS and  
TESTING  
DIVISION

PLASTICITY CHART  
GLACIAL TILL  
CLAYEY SILT, FRAGMENTS OF SHALE & LIMESTONE

WP No. 34-66-09

JOB No. 68-F-33

FIG. NO. 8

# VOID RATIO vs PRESSURE

W = 89%

$C_c = 2.94$

BORE HOLE 5

SAMPLE 6

DEPTH 21'-3"

ELEV. 232.6

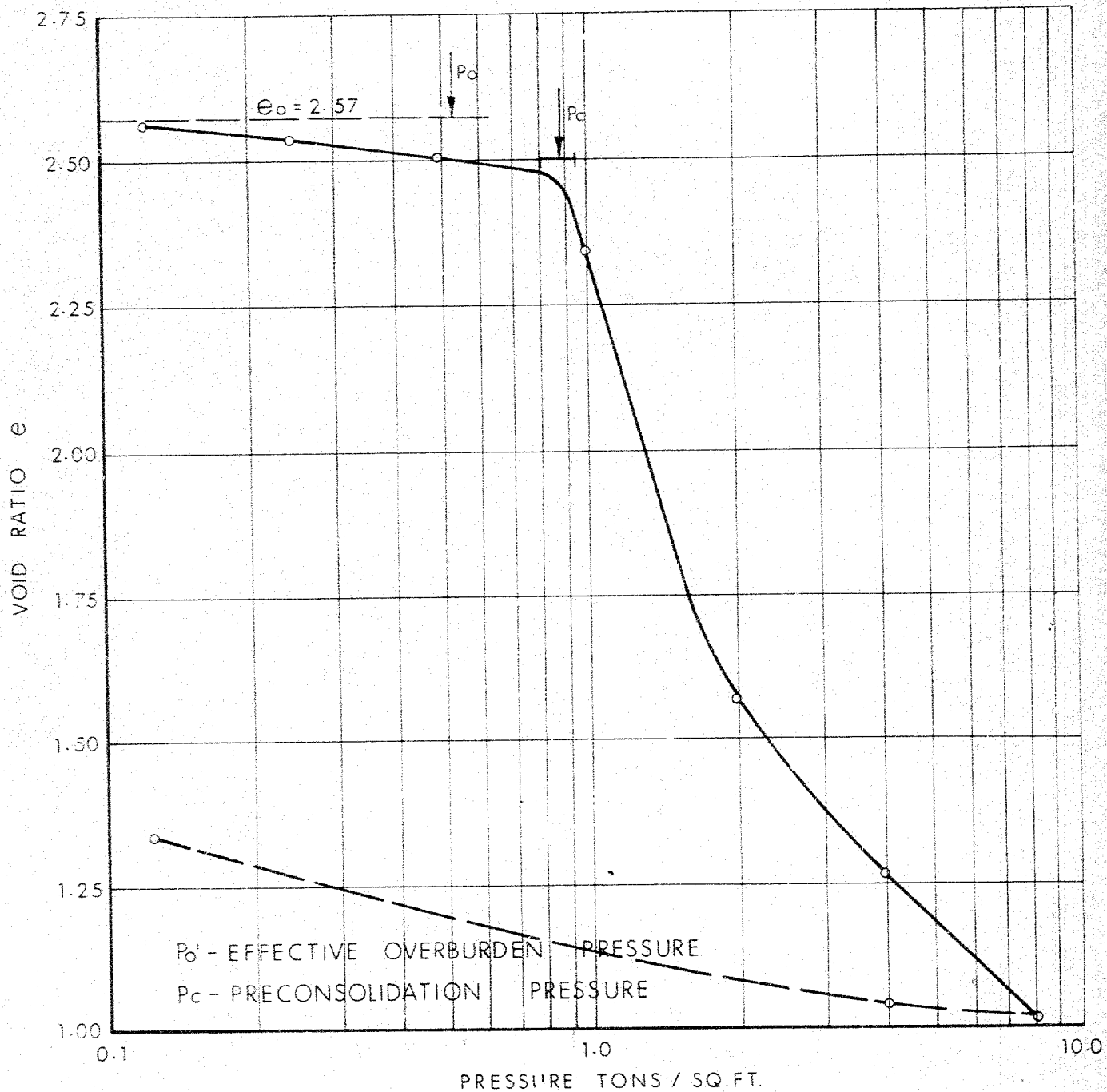


FIG. 9



# VOID RATIO vs PRESSURE

$W_L = 48.6 \%$

$W_p = 25.1 \%$

$W = 78.1 \%$

$C_c = 2.66$

BORE HOLE 5

SAMPLE 9

DEPTH 36'-0"

ELEV. 217.9

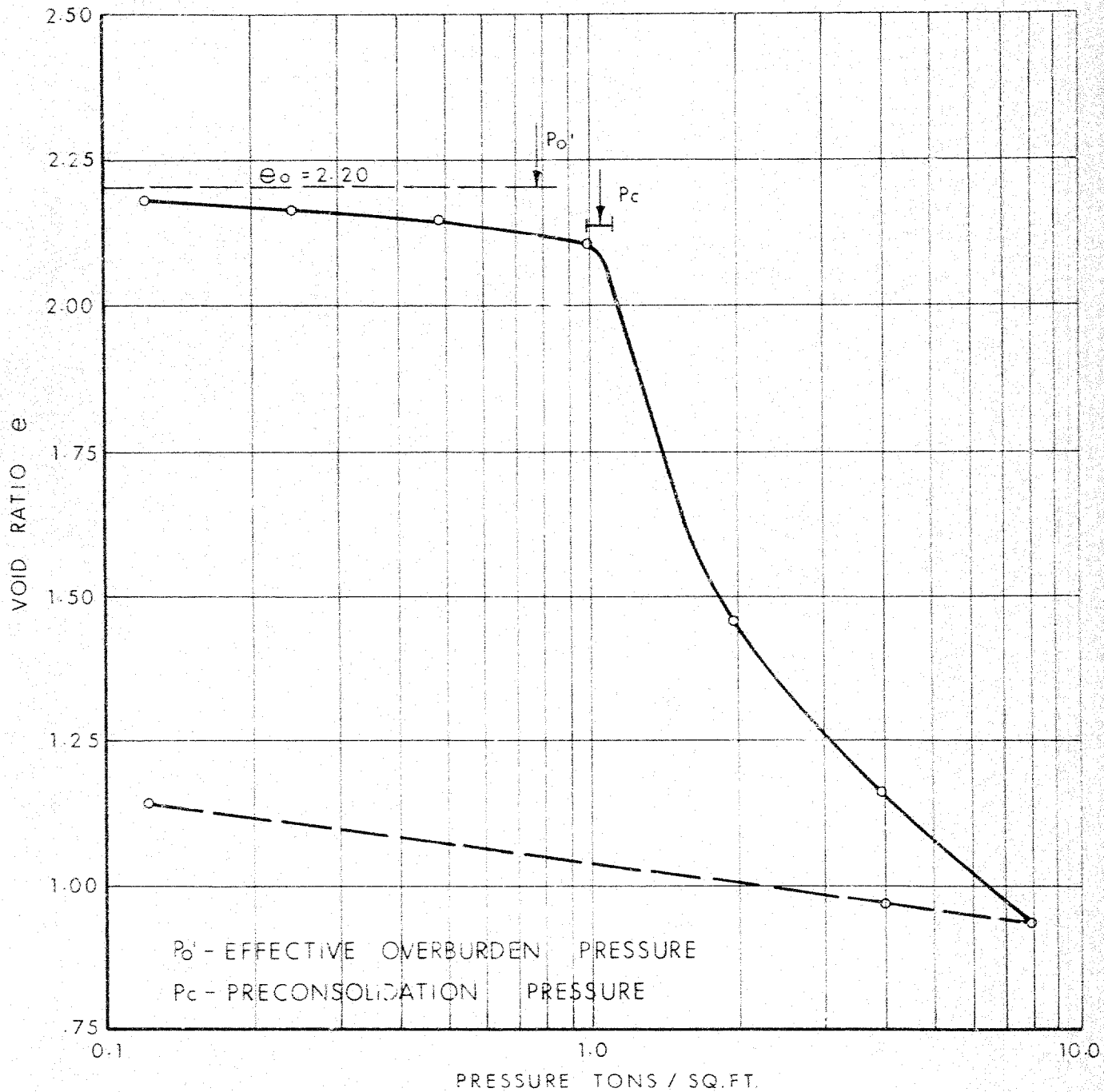


FIG 10

## 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 = $\frac{w_L - w}{I_p}$
$e_{max}$	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 INTERCEPT
$\phi'$	EFFECTIVE ANGLE OF SHEARING RESISTANCE, OR FRICTION
$c_u$	APPARENT COHESION
$\phi_u$	APPARENT ANGLE OF SHEARING RESISTANCE, OR FRICTION
$\mu$	COEFFICIENT OF FRICTION
$S_t$	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
$\sigma'$	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

Mr. E. R. Davis,  
Bridge Engineer,  
Bridge Division,  
Admin. Bldg.

Attention: Mr. J. McCombie

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

June 28, 1968

Proposed Structures - Hwy. #417  
District No. 9 (Ottawa)

--  
Anderson Rd. - W.F. 34-66-06, W.J. 67-F-112  
Seventh Line Rd. - W.F. 34-66-07, W.J. 67-F-113  
Eighth Line Rd. - W.F. 34-66-08, W.J. 67-F-114  
✓ Boundary Rd. - W.F. 34-66-09, W.J. 68-F- 33  
--

With reference to our memo of June 26, 1968, regarding the above subject, we wish to add the following comments:

On Anderson Rd. Overpass benches of 30 ft. length at elevation 257 are recommended. They are needed only in one direction, longitudinal or transverse, depending on the way they are described. For Hwy. #417 they would be transverse, while for Anderson Road they would be parallel or longitudinal. This explanation, we hope, removes any ambiguity that might be attached to the statement in our memo of June 26th.

On page 4 of the mentioned memo, it is stated that recommendations pertaining to structure foundations are similar to those discussed in our Foundation Report for underpass structures. This statement applies to abutments only, while the pier footings would most probably be founded on timber friction piles. Whether the same type of foundation could also be used for abutment footings would have to be looked into for each of the mentioned structures. For the Boundary Rd. Overpass it certainly looks very possible.

.../MdeP

*A. G. Sternas*  
A. G. Sternas  
PRINCIPAL FOUNDATION ENGINEER

cc: Messrs. E. R. Davis (2)  
G. Scott  
C. J. Markiewicz  
J. E. Graspier  
J. L. Forster

Foundations Files  
Gen. Files

## 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. McCombie

DATE: June 26, 1968

OUR FILE REF.

IN REPLY TO

## SUBJECT:

Proposed Structures - Hwy. #417  
District No. 9 (Ottawa)

--  
Anderson Rd. - W.P. 34-66-06, W.J. 67-F-112  
Seventh Line Rd. - W.P. 34-66-07, W.J. 67-F-113  
Eighth Line Rd. - W.P. 34-66-08, W.J. 67-F-114  
Boundary Rd. - W.P. 34-66-09, W.J. 68-F- 33  
--

Detailed subsurface investigations, at the proposed underpass locations, were carried out in late 1967 and early 1968. Reports, containing all the factual information obtained from the investigations, together with an engineering assessment of the stability and settlement of approach embankments and foundation design, have been submitted.

A surface layer of sand followed by an extensive deposit (70 feet or greater in thickness) of soft, highly compressible clay is located at all the sites. It was originally proposed to place approach fills of the order of 20 to 25 feet on this clay subsoil. Fills of this height would require berms both in the longitudinal and transverse direction. Further, settlements of the order of 7 to 11 feet would occur in the foundation subsoil located beneath the maximum fill heights. It was recommended that consideration be given to limiting the fill heights and by so doing, reduce the berm requirements and the induced consolidation settlements. This, however, will necessitate an increase in the span length of the structure.

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

2.

Attn: Mr. S. McCombie

June 26, 1968

Subsequently the Bridge Location Section requested that this office provide preliminary recommendations pertaining to an alternative proposal of carrying the Township roads under Hwy. #417 in cut. This proposal was submitted in a memo (from Mr. G. Scott, Regional Bridge Location Engineer), dated June 14, 1968.

Preliminary computations have been carried out for the proposed cut sections in terms of total and effective stress analyses. In these analyses it is assumed that the cut slopes will have standard 2:1 slopes. The results of these computations are summarized as follows:

1. Anderson Rd. Overpass:

	<u>Westbound Lane</u>	<u>Eastbound Lane</u>
Existing Ground Surface	Elev. 263	Elev. 263
Proposed Grade - Hwy. #417	" 265	" 266
Proposed Grade - Anderson Rd.	" 241	" 242

For the above mentioned scheme a bench, of the order of 30 feet in length, would be required at about elevation 257, both in the longitudinal and transverse direction.

cont'd. /3 ...

Mr. B. R. Davis,  
Bridge Engineer,  
Bridge Division,  
Admin. Bldg.  
Attn: Mr. S. McCombie

3.

June 26, 1968

2. Seventh Line Rd. Overpass:

	<u>Westbound Lane</u>	<u>Eastbound Lane</u>
Existing Ground Surface	Elev. 260	Elev. 260
Proposed Grade - Hwy. #417	" 267	" 267
Proposed Grade - Seventh Line Rd.	" 243	" 243

For this scheme a bench of the order of 25 feet in length would be required at about elevation 255.

3. Eighth Line Rd. Overpass:

	<u>Westbound Lane</u>	<u>Eastbound Lane</u>
Existing Ground Surface	Elev. 261	Elev. 261
Proposed Grade - Hwy. #417	" 268	" 268
Proposed Grade - Eighth Line Rd.	" 244	" 244

For this scheme a bench of the order of 20 feet in length would be required at about elevation 256.

4. Boundary Rd. Overpass:

	<u>Westbound Lane</u>	<u>Eastbound Lane</u>
Existing Ground Surface	Elev. 255	Elev. 255
Proposed Grade - Hwy. #417	" 258	" 258
Proposed Grade - Boundary Rd.	" 233	" 233

For this scheme a bench of the order of 35 feet in length would be required at about elevation 247.

cont'd. /4 ...

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

4.

Attn: Mr. S. McCombie

June 26, 1968

Our preliminary computations indicate that cuts up to a maximum depth of about 15 feet will be stable if standard 2:1 slopes are adopted. The bench requirements for the intermediate cut sections, as well as special treatment of the cut slopes, will be discussed when the final design details become available.

Recommendations pertaining to structure foundations are similar to those discussed in our Foundation Reports for underpass structures.

We trust that this memo presents the data required at the present time. If any of the aforementioned recommendations require clarification, or if additional design information is desired, please contact this office.

*B. T. Darch*

BTD/MdeF

cc: Messrs. B. R. Davis (2)  
G. Scott  
S. J. Markiewicz  
J. E. Gruspier  
J. L. Forster

*for* M. Devata,  
SUPERVISING FOUNDATION ENGR.  
For:  
A. G. Stermac,  
PRINCIPAL FOUNDATION ENGR.

Foundations Files ✓  
Gen. Files



## MEMORANDUM

To: Mr. C. S. Grebski,  
Bridge Design Engineer,  
Bridge Office,  
Admin. Bldg.

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

ATTENTION:

DATE: February 13, 1969

OUR FILE REF:

IN REPLY TO

SUBJECT:

Boundary Road Interchange Underpass  
4.9 Miles East of Carleton County Rd. 27  
Highway #417 -- District #9 (Ottawa)  
W.P. 34-66-09 -- W.J. 68-F-33

We have reviewed the Preliminary Bridge Drawing No. D-6520-P1 for the above mentioned project and submit the following comments:

Approach embankments of the height proposed (13 feet) will be inherently stable; they will, however, induce consolidation settlement within the extensive soft clay deposit encountered at the site. It would be advantageous, therefore, to construct the embankments prior to construction of the structure foundation. In addition, it would be beneficial to add a three-foot surcharge to the embankment, in order to accelerate the time rate of settlement. The magnitudes of the settlements predicted for: i) the proposed height of fill, and ii) the fill plus a nominal surcharge, are listed in tabular form below:

-- SUMMARY OF ESTIMATED SETTLEMENT --				
Height of Fill (Factor of Safety)	Total Settlements after Various Periods			
	1 Year	2 Years	3 Years	30 Years
13 ft. (Design Height) (F.S. = 1.3)	17"	20"	22"	36"
16 ft. (Design Height- + 3-ft. Surcharge) (F.S. = 1.1)	22"	28"	30"	

Mr. C. S. Grebski,  
Bridge Design Engineer,  
Bridge Office, Admin. Bldg.

2

February 13, 1969

Referring to the table, it can be seen that an additional 8 inches of settlement could be induced within the deposit if the surcharge was allowed to remain in place for approximately 2 years. If the surcharge loading was removed after this period, the incremental component of settlement expected, within the next 28 years, would be reduced from 16 to 8 inches.

Additional material may not be required to form this surcharge, provided that approach fills, less than 5 ft., are not constructed initially, the proposed surcharge being built instead. In such a case, the surcharge can be left in place for as long a period as possible. Following removal of the surcharge, the fill available could be utilized in constructing the approach fills less than 5 ft. in height, since nominal settlements at these lower fill height locations should not have any significant effect on the performance of Boundary Rd.

BTD/MdeF

*M. Devata*

M. Devata,  
SUPERVISING FOUNDATION ENGR.  
For:  
A. G. Stermac,  
PRINCIPAL FOUNDATION ENGR.

cc: Messrs. S. McCombie  
G. Scott  
S. Markiewicz  
J. Gruspier

Foundations Files  
Gen. Files

DEFECTS IN NEGATIVE DUE TO  
CONDITION OF ORIGINAL DOCUMENT

1969 FEB 7 AM 8:51

K

KINR DOWN 1 FEB 7/69 8.42A VR

M J MACMASTER SR PROJECT DESIGN ENGR

FURTHER TO YOUR T T WE ARE GIVING THE SAFE HIGHTS FOR STOCK PILES  
USING 2 TO 1 SLOPES AT THE VARIOUS LOCATIONS WHICH ARE AS FOLLOWS

WJ67-F-112

WP34-66-06 ANDERSON RD 13 FT

WJ67-F-113

QP34-66-07 SEVENTH LINE 17 FT

WJ67-F-114

WP34-66-08 EIGHTH 14 FT

WJ68-F-33

WP34-66-09 BOUNDARY RD 14 FT

WJ68-F-52

WP34-66-01 BASE LINE RD 20 FT

M DEVATA SUPVR FOUNDATION ENGR FOR

A G STERMAC PRINC FOUNDATION ENGR

AW

1969 FEB 5 PM 1:59

DOWN KINR 3 FEB 5/69 1.45 PM P R I O R I T Y

M DEVATA FOUNDATION OFFICE

RE WP 34-66-01 - HWY NO. 417

THE REGIONAL MATERIALS AND TESTING OFFICE HAS MADE THE FOLLOWING  
RECOMMENDATION: 2:1 STOCKPILING OF MATERIALS

"A SPECIAL SHOULD BE INSERTED IN THE CONTRACT DOCUMENTS TO LIMIT THE  
HEIGHT OF STOCKPILED MATERIAL DUE TO THE UNDERLYING WEAK CLAYS ALONG  
THE PROJECT.

STOCKPILES OF TOPSOIL, ETC., MAY HAVE VERY STEEP SLOPES AND THE CRITICAL  
HEIGHT MAY BE QUITE LOW. THE FOUNDATION SECTION SHOULD INDICATE  
THE SAFE HEIGHT FROM THEIR DATA AT STRUCTURE HEIGHTS".

IN ORDER THAT WE MAY COMPILE THE SPECIAL PROVISION PLEASE INFORM  
US BY RETURN TT OF THE CRITICAL HEIGHT FOR STOCKPILES WITHIN THE  
LIMITS OF THE PROJECT.

M J MACMASTER SR PROJECT DESIGN ENGR

JS

# SAFE HEIGHTS FOR FILLS @ 2:1 SIDE SLOPES.

W.P. 34-66.

HWY 417.

<u>W.J.#</u>	<u>W.P.#</u>	<u>Name</u>	<u>SAFE HT.(Ft)</u>	<u>Remarks</u>
67-F-111	<del>34-66-05, 4</del>	<del>Bear Brook</del>	<del>25'</del>	<del>above hor. Gr. Surface</del>
67-F-112	34-66-06	Anderson Rd.	13'	
67-F-113	34-66-07	Seventh Line	17'	
67-F-114	34-66-08	Eighth Line	14'	
68-F-33	34-66-09	Boundary Rd.	14'	
68-F-52	34-66-01	Baseline Rd.	20'	above hor. Gr. Surface
68-F-53	<del>34-66-01</del>	<del>McEwan Creek</del>	<del>20'</del>	away from Creek Banks
68-F-54	<del>34-66-10, 11</del>	<del>Ramsay Creek</del>	<del>12'</del>	" " "
68-F-57	<del>34-66-10</del>	<del>Bear Brook Tribut.</del>	<del>18'</del>	above valley floor Elev of 229 Ht. <sup>should</sup> <del>may</del> be reduced if fill is placed on ground above the valley floor.

DEFECTS IN NEGATIVE DUE TO  
CONDITION OF ORIGINAL DOCUMENT

CM Feb 6/67

(EQUATES TO 1/2 INCH)

## MEMORANDUM

68-F-33

To: Mr. A. G. Stermac,  
Principal Foundation Engineer,  
DOWNSVIEW, Ontario.

FROM: Road Design Division,  
KINGSTON, Ontario.

ATTENTION:

DATE: February 17, 1969.

OUR FILE REF:

IN REPLY TO

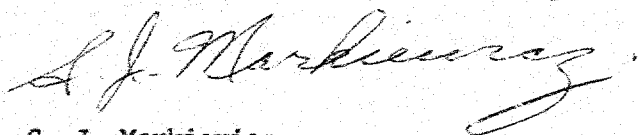
SUBJECT: W.P. 34-66-09 - Hwy. #417 - District #9 Ottawa - Boundary Road  
Interchange - 4.9 Miles East of Carleton County Road 27.

I would like to comment on your memo to Mr. C. S. Grebski dated February 13, 1969, in which you recommend a 3 foot surcharge.

I am not raising an objection but I cannot agree that the surcharge material can be used to construct the approach fills on Boundary Road less than 5 feet in height. The approach grades are 4%. This involves a length of 125 feet from a height of 5 feet to 0. From the 5 foot height of fill the grade could possibly be increased to 10% temporarily but not much more in order to permit effective equipment operation. This would reduce the length from a 5 foot fill to 0 by 75 feet to 50 feet. For an average depth of  $2\frac{1}{2}$  feet, this would hardly accommodate the surcharge material.

However, this is not my main concern. The pile cap for the abutments appears to be set at an elevation at about existing ground level. This will mean excavating into the front end of the approach fills for the pile caps. Our normal method of excavation is to excavate vertically and to pour the footings or pile caps neat, without forming. Since your stability calculations and factor of safety are based on 2 to 1 side slopes it will probably be necessary to remove the front portion of the approach fills, pour the pile caps and then replace the fill in front of the abutment.

Since this is an unusual method of operation it will be necessary to spell out in detail the manner in which the excavation is to be carried out and the quantities that will be paid for as footing excavation.



S. J. Markiewicz,  
REGIONAL ROAD DESIGN ENGINEER.

SJM/mac

c.c. - C. S. Grebski  
G. Scott

280

Mr. C. S. Grebski,  
Bridge Design Engineer,  
Bridge Office,  
Admin. Bldg.

Foundation Section,  
Materials & Testing Office,  
Room 107, Lab. Bldg.

February 24, 1969

C.N.R. Overhead, Hwy. #47,  
1.6 Miles North of Stouffville,  
Twp. of Uxbridge and Whitechurch,  
District No. 6 (Toronto),  
W.J. 68-P-4 -- W.P. 181-64-2

---

We have reviewed the Final Bridge Drawing No. O-6462-1, 2 and 3 for the above mentioned structure and submit the following comments:

In our Foundation Investigation Report we indicated the groundwater level at the time of investigation - (January 1968) to be at approximate elev. 998.0. It is possible, however, during the construction season, if this is in the summer, that the water levels will be at a lower elevation. Since the necessity for a dewatering scheme for the pier construction is dependent on the groundwater level, it will therefore be necessary to establish these at the time of construction in order to determine to what extent a dewatering scheme is required.

MD/MdeF

cc: Messrs. S. McCombie  
W.S. Melnychyn

Foundations Files ✓  
Gen. Files

M. Deveta

M. Deveta,  
SUPERVISING FOUNDATION ENGR.  
For:  
A. G. Sternac,  
PRINCIPAL FOUNDATION ENGR.



*Algo*

Mr. S. J. Markiewicz,  
Regional Road Design Engineer,  
Eastern Region,  
KINGSTON, Ontario.

Foundation Section,  
Materials & Testing Office,  
Room 107, Lab. Bldg.

February 26, 1969

Boundary Road Underpass - 4.5 Miles East of Carleton,  
County Road #27, Hwy. #417 - District No. 9 (Ottawa)  
W.P. 34-66-09                      --                      W.J. 68-F-33

---

We note the comments in your memo dated February 17, 1969, relating to the construction of approach fills less than 5 ft. in height, and the excavation for the abutment footing.

It would appear that the fills less than 5 ft. in height cannot provide all the surcharge material. Nevertheless, some saving can be effected by using as much of this as possible. The surcharge need only be placed for a limited distance behind the abutment, as in the case of the Anderson Road crossing.

With regard to the excavation for the abutment footing, sufficient fill material must be removed so as to accommodate the granular backfill which will be placed behind the abutment walls. This will involve a certain slope in the forward direction during construction. Footing excavation can be considered to be all material excavated below the level of the original ground surface. We do not foresee any stability problems insofar as 'base failures' are concerned, even if the fill is excavated with a vertical face. In our opinion, temporary slopes during construction, are the responsibility of the contractor.

MD/NJEF

cc: Messrs. C. S. Grebski  
C. Scott

Foundations Files  
Gen. Files

*M. Devata*

M. Devata,  
SUPERVISING FOUNDATION ENGR.  
For:  
A. G. Stermac,  
PRINCIPAL FOUNDATION ENGR.



MEMORANDUM

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

FROM: Bridge Office

ATTENTION:

DATE: May 13, 1969

OUR FILE REF.

IN REPLY TO

SUBJECT: Boundary Rd. Interchange Underpass  
4.9 Miles E. of Carleton Cty. Rd. 27  
W.P. 34-66-09, Site 3-270  
Highway 417, District 9

68-F-33

Attached herewith we are submitting the final  
bridge drawings which show the foundation design for  
this structure.

Kindly give us your comments at your earliest  
convenience.

CSG:rd

Attach.

c.c. Foundation Section

  
C.S. Grebski,  
Bridge Design Engineer

no comment

M. Levata

May 15/69

## MEMORANDUM

To: Mr. W. Wigle,  
Program Engineer,  
Downsview, Ontario.

From: Functional Planning Section,  
Kingston, Ontario.

ATTENTION:

DATE: July 29, 1969.

OUR FILE REF.

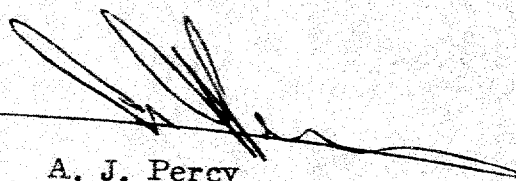
IN REPLY TO

SUBJECT:

W.P. 34-66-09,  
Boundary Road Interchange,  
Highway 417, District 9-Ottawa

With reference to the "Status Report & Pre-Engineering Schedule" dated July 10, 1969, for the above project, we would suggest that the structure not be constructed with W.P. 34-66-03 but that it be rescheduled for construction with the follow-up paving project W.P. 34-66-04. The structure approaches, however, should be constructed with W.P. 34-66-03.

This will permit construction of the approach fills in advance of the structure foundations in order to reduce the differential settlements.



A. J. Percy

For: M. R. Ernesaks

Regional Functional Planning Engineer

AJP/MRE/hl

c.c.

J. E. Gruspier  
T. C. Kingsland  
S. J. Markiewicz

C. R. Robertson

A. G. Stermac ✓

C. S. Grebski



Your File No. M43-2-48



Hwy. 401 & Keele St.,  
Downsview 464, Ontario,  
C A N A D A

Tel. 248-3232

(Area Code 416)

DEPARTMENT OF HIGHWAYS  
Materials and Testing Office

April 27, 1970

Mr. L. W. Gold,  
Head,  
Geotechnical Section,  
Division of Building Research,  
National Research Council,  
Ottawa 7, Ontario.

Re: Test Fill on Highway 417  
at Boundary Road

Dear Lorne:

Thank you for your letter of April 24, 1970 regarding the above subject.

After our telephone conversation, we were in touch with your office and have already answered some queries. The others will be discussed and resolved at the meeting we are organizing in the very near future - possibly this coming Friday.

Your representatives will have a chance to have all questions and problems resolved in discussion with the representative of the Department's Ottawa District Office and ourselves. Whatever is decided at this meeting will be carried out as accurately and reliably as possible.

Mr. Devata, from my Section, is arranging the meeting and he is maintaining close liaison with your Mr. Bozozuk.

Should there be any other queries or suggestions that you would have, please feel free to contact me.

Yours sincerely,

*Sony Stermac*  
A. G. Stermac

Principal Foundation Engineer

AGS/MdeF

cc: Foundations Files  
Gen. Files



CABLE ADDRESS  
ADRESSE TÉLÉGRAPHIQUE

"RESEARCH"

PLEASE QUOTE FILE NO  
NO DE DOSSIER À RAPPELER

M43-2-48

NATIONAL RESEARCH COUNCIL OF CANADA  
CONSEIL NATIONAL DE RECHERCHES DU CANADA

DIVISION OF BUILDING RESEARCH  
DIVISION DES RECHERCHES EN BATIMENT

OTTAWA 7.

24 April 1970

Mr. A.G. Stermac,  
Principal Foundations Engineer,  
Department of Highways of Ontario,  
Highway 401 and Keele Street,  
Downsview, Ontario.

Dear Tony:

Re: Test Fill on Highway 417  
at Boundary Road

Mike Bozozuk tried to reach you a couple of weeks ago concerning the status of the proposed test fill. We had heard that the contract for this stage of highway had been awarded to Dibblee Construction Company. Mr. M. Devata returned his call to you, and inform us that the contract was indeed signed and that the contractor planned to start construction of the fill in question by the end of April. In addition, he stated that the fill at the test section will be in three levels, and that we should be ready to install our field instruments prior to and during its construction.

In earlier correspondence we raised the matter of constructing a two level fill initially instead of a three level fill. The reason for suggesting this change was that there is no need for information concerning stresses and deformations under a four foot fill, and eliminating this section and extending the 14-foot and 9-foot sections would provide better conditions for the measurements that we wish to make. Although you considered this modification was possible at the time of our earlier correspondence, we gather from Mr. Devata that it was not possible to stipulate it in the contract. We would like to have this question resolved as soon as possible as it has some bearing as to how we will deploy our instrumentation.

Mr. A.G. Stermac,  
Downsview, Ontario.

- 2 -

24 April 1970

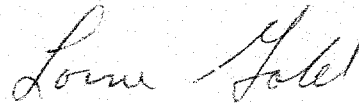
We wish also to establish what arrangements have been made with the contractor with regard to us instrumenting the fill and carrying out the subsequent observations. Was an agreement written into the contract, or has the contractor been informed in some other way of our arrangement with the Department of Highways to instrument the fill?

As you appreciate, scheduling of the installation of the instruments is very important. Most of instrumentation is ready, or can be ready within two weeks. The bench marks, settlement gauges and piezometers however cannot be installed until the existing roadway is removed and the detour road constructed. At least two working weeks, weather permitting, will be required to install these devices and to permit sufficient time for them to reach equilibrium with the conditions at the site. It is most important to properly establish the zero conditions before the construction of the embankment. We will also require sufficient time during the construction to install instruments such as the pressure cells and horizontal movement gauges and to measure the in place density of the fill.

We would like to have these questions resolved so that we can plan our activity so as to interfere as little as possible with the work of the contractor. Would it be possible for a representative of the contractor, the resident engineer and ourselves to get together with you at some convenient time either in Toronto or in Ottawa?

We look forward to hearing from you on this matter.

Yours sincerely,



L. W. Gold,  
Head,  
Geotechnical Section.

LWG:lg1

70-SI-186

68-F-33  
CABLE ADDRESS

ADRESSE TÉLÉGRAPHIQUE

"RESEARCH"

PLEASE QUOTE FILE NO M43-2-48

NO DE DOSSIER À RAPPELER

NATIONAL RESEARCH COUNCIL OF CANADA  
CONSEIL NATIONAL DE RECHERCHES DU CANADA

DIVISION OF BUILDING RESEARCH  
DIVISION DES RECHERCHES EN BATIMENT

OTTAWA 7.

14 May 1970

Mr. A.G. Stermac,  
Principal Foundation Engineer,  
Department of Highways of Ontario,  
Highway 401 and Keele Street,  
Downsview, Ontario.

Dear Tony:

Re: Test Embankment, Boundary  
Road, Highway 417.

Mike Bozozuk and Ken Burn met with Mr. Devata, Mr. Westerby, Mr. McShane, Mr. Mullins, DHO, and with Mr. Thompson, Dibblee Construction Company, on 1 May to discuss instrumentation of the proposed test embankment at Boundary Road. I expect that Mr. Devata has spoken to you about the meeting, but I thought I should confirm with you the results of it as recorded by Mike. These are as follows:

1. The north fill of Boundary Road will be constructed in two levels. The 14-foot high section will be 100 feet long and the 9-foot high section 60 feet long. DHO will supply NRC with design drawings required for planning the instrumentation. In turn, the NRC will supply plans of the instrumentation to DHO and to Dibblee Construction Co.
2. The contractor will start to build the detour road about the 25th of May. Since it will take about one week to build, the NRC will have from the 1st to the 15th of June, weather permitting, to install the instrumentation.
3. Dibblee Construction will remove the existing roadway from the location of the test fill before the instrumentation is installed.



Mr. A.G. Stermac,  
Downsview, Ontario.

- 2 -

14 May 1970

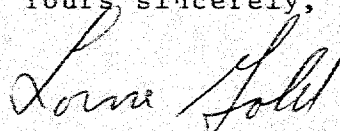
4. Dibblee will select the field material to be used in the construction. In all probability, this will be fine sand which is available in the surrounding area. DHO will supply samples of this material to NRC for lab testing.
5. The embankment will be constructed in 1 foot lifts and compacted with a vibratory compactor. The expected time for constructing the fill is about five days. NRC will be allowed sufficient time during construction to carry out in place density tests, conduct surveys, and install earth pressure cells.
6. The contractor will provide protection for instrumentation installed by NRC.
7. DHO can supply short lengths of corrugated metal culverts for use by NRC to house some instrumentation.
8. The location for the deep bench mark and reference piezometers will be arranged with DHO.

Our plans are proceeding well, and we expect to begin the installation of the instruments as soon as the existing roadway is removed.

I want to say how much we appreciate having the opportunity to instrument the fill at Boundary Road, and the willingness of DHO to modify the design of it so as to make the measurements more meaningful.

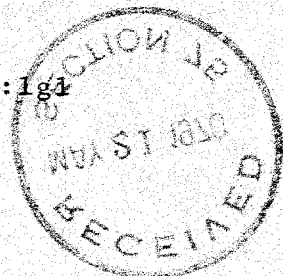
With kindest regards,

Yours sincerely,



L. W. Gold,  
Head,  
Geotechnical Section.

LWG:lgf



70-SI-207

INSTRUMENTATION OF NORTH APPROACH EMBANKMENT  
AT THE BOUNDARY ROAD UNDERPASS  
Highway #417 District No. 9 (Ottawa)  
W.J. 68-F-33, W.P. 34-66-03, Cont. 69-209

A meeting was held at D.H.O. Ottawa District Office on May 1, 1970, to discuss the proposed instrumentation at the above mentioned site by N.R.C. with the cooperation of the Department of Highways, Ontario.

Present were the following:

Mr. M. Bozozuk	National Research Council (D.B.R.) - Ottawa
Mr. K. Burns	" " " " "
Mr. K. Westerby	Department of Highways - Ottawa District
Mr. G. P. Mullins	" " " " "
Mr. W. McShane	" " " " "
Mr. M. Devata	" " " - Foundation Section Downsview
Mr. D. C. Thompson	Dibblee Construction - Ottawa

The Instrumentation Programme was discussed in detail by the members present and resulted in the following conclusions:

- 1) Details of Installations of Various Instruments by N.R.C.-(D.B.R.)
  - a) 30 Settlement Gauges
  - b) 2 Settlement Profilers
  - c) 10 Earth Pressure Cells
  - d) - Horizontal Movement Gauges
  - e) 1 Deep Bench Mark

All the materials, including field installations, will be carried out by N.R.C.-(D.B.R.). Detailed drawings of the instrumentation will be submitted to the D.H.O. Foundation Section and Ottawa District for their general approval. Necessary protection of the instruments at the site during construction, will be provided by the Contractor and the Ottawa District personnel will ensure this.



2) Instrumentation readings will be carried out by N.R.C.-(D.B.R.) personnel during construction. In addition, during embankment construction, compaction tests will be carried out by personnel from N.R.C.-(D.B.R.) and the Contractor will cooperate and coordinate his operations to carry out the required compaction tests. The tests are primarily for obtaining factual information only, and any control on compaction shall be controlled by D.H.O. Ottawa District.

3) All surveying for various installations and observations for the instrumentation project will be carried out by N.R.C.-(D.B.R.) personnel.

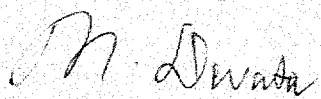
4) Instrumentation results will be periodically submitted to the Foundation Section of the Department by N.R.C.-(D.B.R.). Additional recommendations, if required for the stability of the embankment during construction, based on the instrumentation results, may be provided by the Foundation Section to the District personnel. At all times during construction, liaison will be maintained between N.R.C.-(D.B.R.) and Foundation Section personnel. No instructions will be given to the Contractor, either by Foundation Section or by N.R.C.-(D.B.R.) personnel. All communications to the Contractor will be carried out by Ottawa District.

5) A drawing showing the revised two-level earth fill section at the North approach embankment of the Boundary Road Interchange will be prepared by the Ottawa District Office. This drawing will include typical sections of earth fills for the two-level embankment.

6) Scheduling of the Instrumentation Work and Construction Operations by the Contractor:

- i) Construction of Service Road in this area will be commenced by May 25, 1970, by the Contractor.
- ii) N.R.C.-(D.B.R.) will complete the installation of various instruments by June 15, 1970. Any excavation equipment required for the installation of settlement profiler, D.H.O. Ottawa District Office will arrange rental of equipment, and the necessary payment will be made by N.R.C.-(D.B.R.).
- iii) Contractor may commence the embankment construction operations by mid-June, 1970.
- iv) The Foundation Section of D.H.O. may install some settlement plates at the original ground surface prior to the commencement of filling operations by the Contractor.

MD/MdeF

  
M. Devata  
Supervising Foundation Engineer



CABLE ADDRESS

ADRESSE TÉLÉGRAPHIQUE

"RESEARCH"

PLEASE QUOTE FILE NO M43-2-48

NO DE DOSSIER À RAPPELER

NATIONAL RESEARCH COUNCIL OF CANADA  
CONSEIL NATIONAL DE RECHERCHES DU CANADA

DIVISION OF BUILDING RESEARCH  
DIVISION DES RECHERCHES EN BATIMENT

OTTAWA 7.

27 October 1970

Mr. M. Devata,  
Supervising Foundation Engineer,  
Department of Highways of Ontario,  
Downsview, Ontario.

Dear Mr. Devata:

In answer to your telephone request of last week, I am sending you the vertical stress distribution under the centerline of the 14-foot high portion of the test embankment on Boundary Road (Purdue analysis). The two curves shown on the figure were prepared for Poissons ratios of 0.33 and 0.45. The vertical depth beneath the embankment is expressed in units of height of embankment  $Y/H$  which can easily be converted to feet. The vertical stresses are expressed in terms of the weight of embankment,  $\sigma_y/\gamma H$ , and once a value for  $\gamma$ , the average density of the fill is known, the actual stresses can be determined.

If it is assumed that the Poissons ratio of the fill material is 0.33, and that of the foundation clay 0.45, the actual vertical stress under the centerline will lie between those two curves.

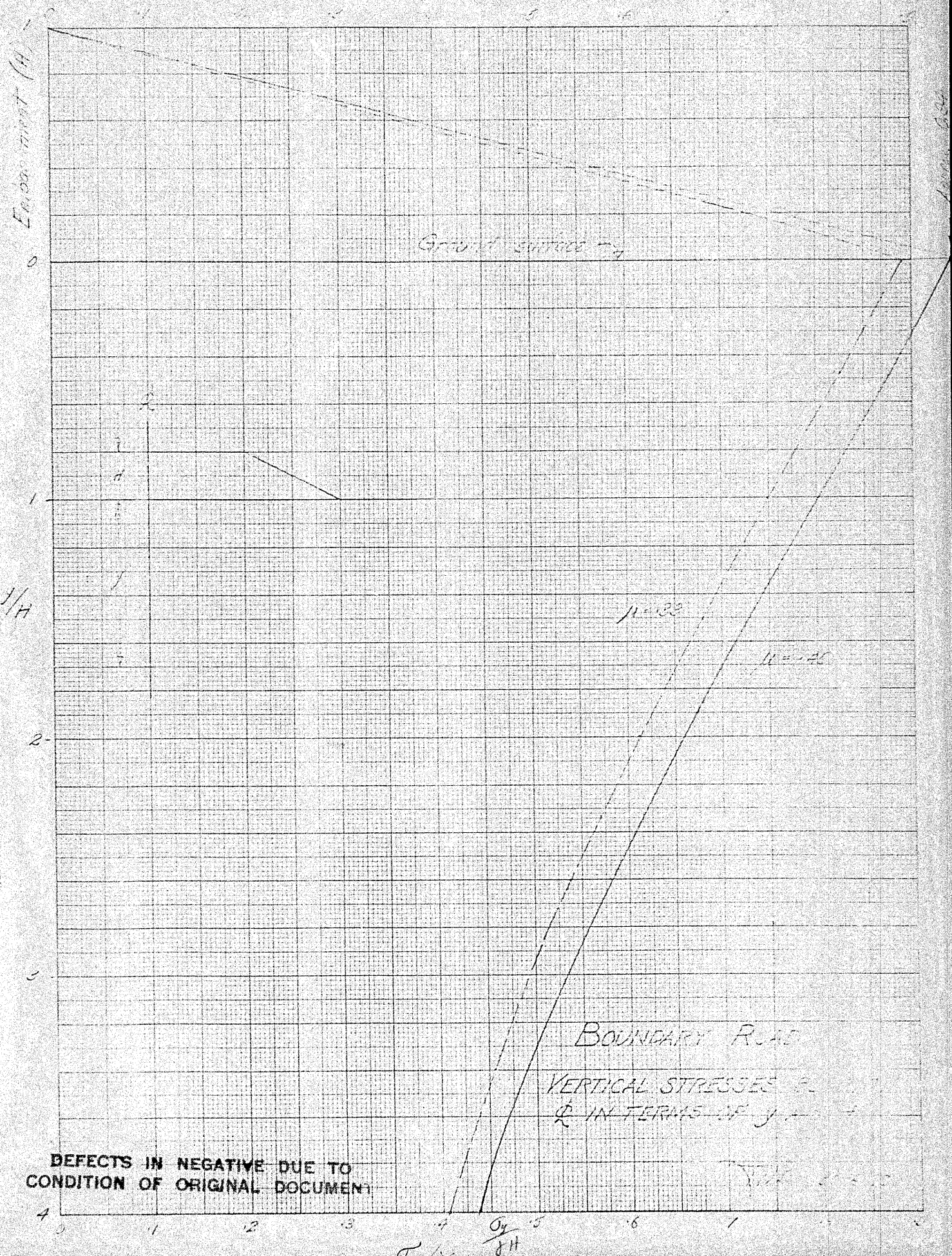
Yours sincerely,

M. Bozozuk,  
Geotechnical Section.

MB:lg1  
Enclosures

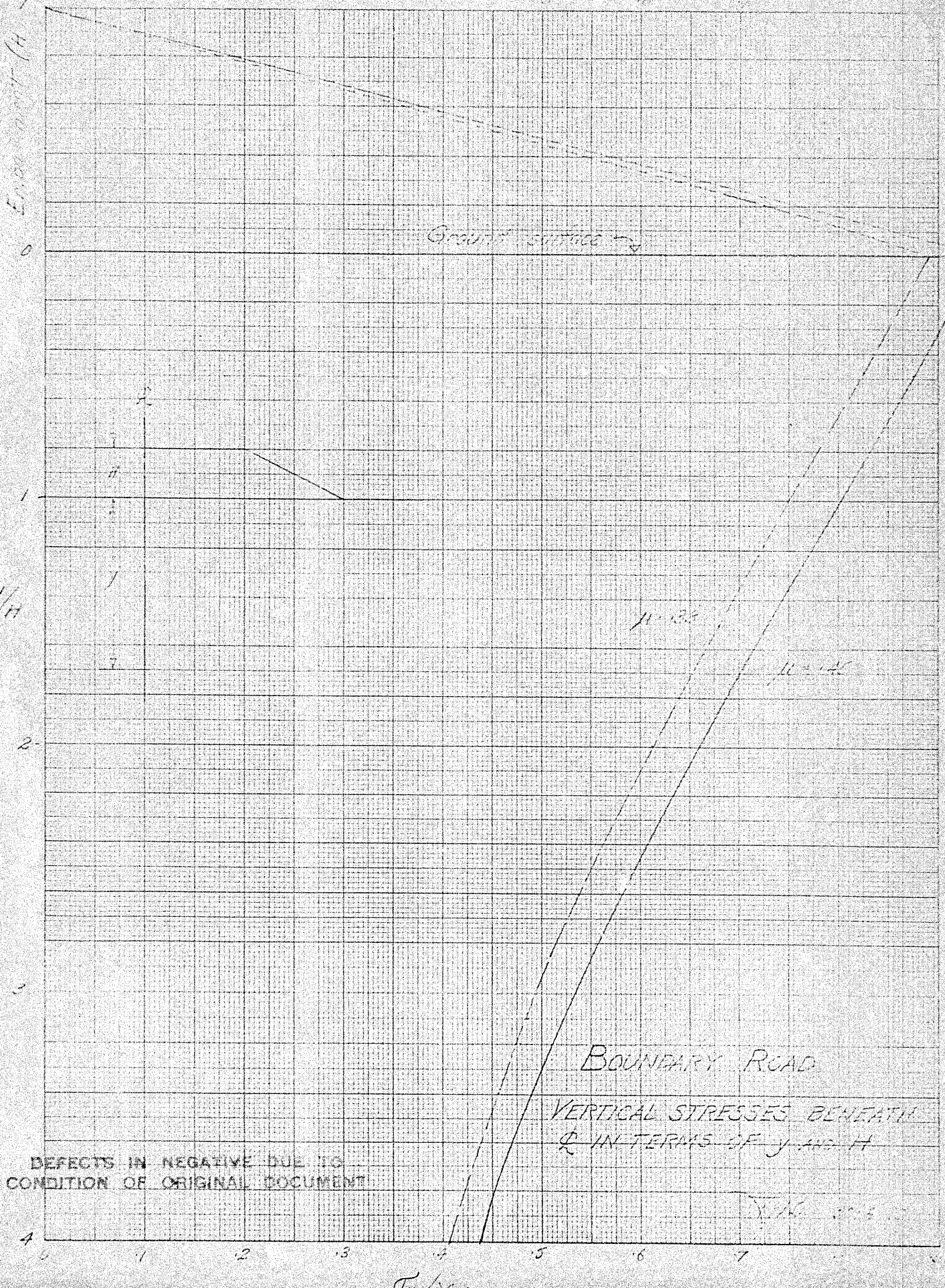
70-SM-176

c.c. Mr. A.G. Stermac,  
Materials & Testing Office,  
Department of Highways,  
Downsview, Ontario.





K&E 10 X 10 TO THE CENTIMETER 46 1510  
 10 X 2.5 CM.  
 KEUFFEL & ESSER CO.



DEFECTS IN NEGATIVE DUE TO  
 CONDITION OF ORIGINAL DOCUMENT

VERTICAL STRESSES BENEATH  
 & IN TERMS OF  $\gamma$  AND  $H$

MEMORANDUM

28-F-33

Boundary Rd.

To: Mr. A.G. Stermac,  
Principal Foundations Engr.  
Downsview.

FROM: District #9, Ottawa.

ATTENTION:

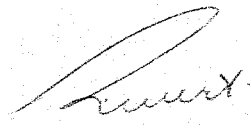
DATE: June 4th, 1970.

OUR FILE REF.

IN REPLY TO

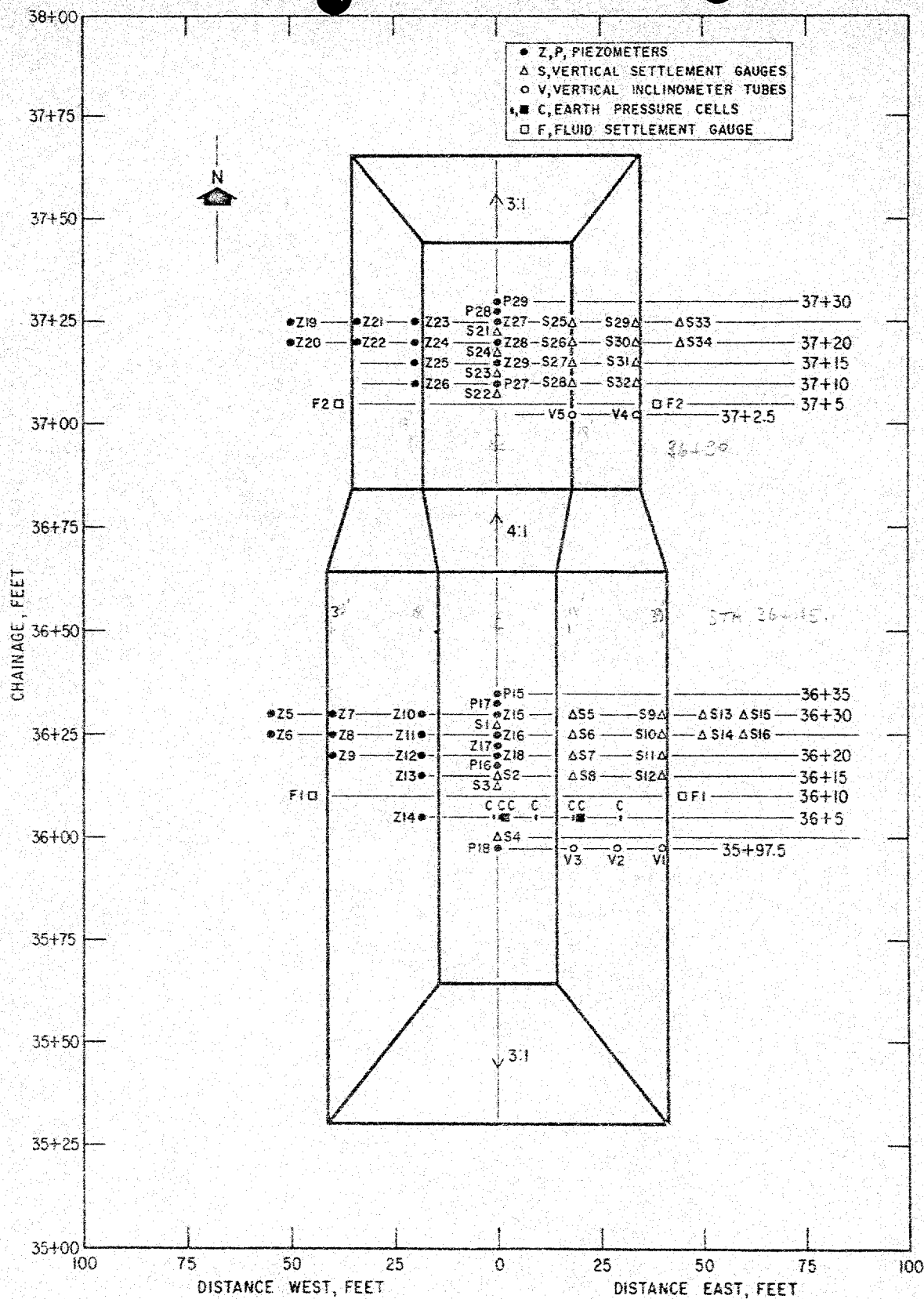
SUBJECT: Contract 69-209

Attached for your information is a plan of instrumentation of the Boundary Road test fill, together with a copy of a letter from Mr. Bozozuk of the Geotechnical Section of the National Research Council of Canada.



R. Wert,  
Eng. Off. Supvr.

RW/amcp  
Att'd  
C.C. Mr. W. McShane.





CABLE ADDRESS

ADRESSE TÉLÉGRAPHIQUE

"RESEARCH"

PLEASE QUOTE FILE NO. M43-4-5

NO DE DOSSIER À RAPPELER

NATIONAL RESEARCH COUNCIL OF CANADA  
CONSEIL NATIONAL DE RECHERCHES DU CANADA

DIVISION OF BUILDING RESEARCH  
DIVISION DES RECHERCHES EN BATIMENT

OTTAWA 7.  
29 May 1970

Mr. Ken Westerby,  
District Engineer,  
Department of Highways,  
530 Tremblay Road,  
Ottawa, Ontario.

Dear Ken:

We have just completed our plans for instrumentation for the Boundary Road test fill. As we agreed to at our meeting of the 1st of May in your office, I am enclosing six copies of these plans for you to distribute to your Field Office, Toronto and to Dibblee Construction Company. Not shown on these plans are the four reference piezometers and the deep bench mark. These, as you will recall, will be located in cooperation with your field office.

*R.W.  
please*

If there are any questions concerning these plans, please feel free to call me.

From a number of visits which we made to the site this past week, we have noticed that Dibblee Construction Company have not started on the detour road. When he does get started will you please let me know in order that we can plan our own field work for the earliest possible date.

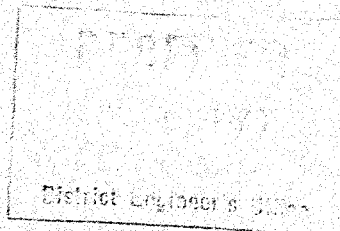
CONTR.	<input checked="" type="checkbox"/> NOTE & FILE	
MANRY.	DISCUSS WITH ME	
MUNICIP.	PLEASE ANSWER	
SERVICES	NOTE & PREPARE TO ME	
ACCOUNTS	INVESTIGATE & REPORT	
EQUIP.	TAKE APPROPRIATE ACTION	
TICKETS & PERMITS	ADVISE ME WHEN OBTAINED	

MB:lg1  
Enclosures

Yours sincerely,

*M. Bozozuk*

M. Bozozuk,  
Geotechnical Section.



70-SM-83



48-7-33

REPORT TO FOUNDATION OFFICE

INSTRUMENTATION PROGRAM ON HIGHWAY 417 IN THE OTTAWA AREA

In the period Dec. 6 - 9, 1971 I visited a number of bridge construction sites on Highway 417 south-east of Ottawa. The purpose of the visit was to review and discuss the progress of our Instrumentation program in this area, and to offer assistance on problems of equipment maintenance, readings and related records.

ANDERSON ROAD, 7TH LINE ROAD AND 8TH LINE ROAD - CROSSINGS  
W.J. 69-F-35, W.J. 67-F-112, W.J. 67-F-113, W.J. 67-F-114

On Dec. 6-7, 1971, I met with Mr. Ron Graham, Project Supervisor in charge of these three sites at his field office on 7th Line road. Bridge construction work has been completed at the sites and the Instrumentation program is continuing.

ITEMS DISCUSSED

- Mr. Graham will send us construction sequence data required for Anderson Road, 7th Line Road and 8th Line Road, structures and approach fills.
- He will also endeavour to find his copy of the Nuclear Density readings taken at the sites, and also suggests that we find the copy of this data which he sent to this office.
- We discussed the settlement equipment and the information collected to date.
- I also showed him our graphical presentations of settlement vs. time etc. which constitutes a permanent record of all our efforts in monitoring movements in this area.
- Mr. Graham stated that a number of settlement plate pipes had been damaged by construction activity, however he has subsequently had them repaired and restored to an operating condition.
- Ron Graham has recently transferred all instrumentation work at the three sites to Mr. Walter McShane whose field office is located at Carlsbad Springs on Boundary Road.
- I contacted Mr. McShane at his office and discussed his take over of the Instrumentation work at Anderson Road, 7th Line Road and 8th Line Road structure sites.
- Mr. McShane will send the October, November and December readings to this office when completed. His staff is presently taking the December readings at the three sites.
- I gave him two new reservoirs for permanent installation at one of the "Settlement Profile" sites. He already has another set to be installed.



When this work is completed, all three "Profiler" locations will have permanent equipment, thus preventing further damage to the threads of the original reservoirs through excessive coupling and uncoupling.

- Walter and Ron Graham intend to get some sort of flexible coupling or a 45° coupling so that the reservoirs will sit level on the end of the inclined P.V.C. pipe, as it protrudes from the ground. This is necessary, due to the fact that the manufacturers of the equipment, "Roctest" sent the last two sets of reservoirs with couplings at right angles rather than on a downward angle of approx. 45°
- Walter McShane will also get lids for the read-out end reservoir to prevent the entry of dust and sand when the equipment is not in use. These two points should be brought to the attention of the manufacturer, "Roctest of Montreal."
- Walter requires a supply of settlement profiler field sheets which will be sent to him shortly.
- On one set of readings which he had recently taken, he and his staff had done all the required calculations. I informed him that this was an unnecessary expenditure of his time as we can have it done quite quickly by computer at Head Office.

#### SLOPE INDICATOR - ANDERSON ROAD

- With the assistance of a member of Ron Graham's 7th Line field office staff, I took a set of readings at the Anderson Road-Slope Indicator installation. Under Mr. Graham's supervision, the installation of pipe was completed up to pavement level while the north abutment was being completed.
- The readings I took will be calculated and correlated with the initial readings taken in Oct. 1970. The compilation of this and data from future readings will hopefully define any pile bending movements beneath this end of the structure.
- Since the riveting gun we loaned them has served its purpose, I asked for and received it from Mr. Graham.

#### BOUNDARY ROAD CROSSING

W.J. 68-F-33

There are no problems at the Boundary Road settlement plate installation and Walter McShane and his staff have been sending us regular monthly readings.

#### COUNTY ROAD NO. 3 CROSSING

W.J. 70-F-2A, 2B

On December 7, 1971 I accompanied R. Graham to his field office at this site, where it crosses Hwy. 417.

#### ITEMS DISCUSSED

- Mr. Graham gave me the settlement plate locations at this site and also

the initial plate elevations and three sets of readings on the tops of the pipes as well as the exact pipe lengths as installed.

- Mr. Graham also gave me the construction sequence to date on the approach fills. More will follow as work progresses.
- Ron will also have Nuclear Density tests taken on the approach fills.

HIGHWAY 138 CROSSING

W.J. 70-F-11

I visited the field office for this project on Dec. 7, and met Mr. George Hambleton, Project Supervisor.

ITEMS DISCUSSED

- He gave me the plate locations and will send us a cross-section over the plates and a profile of the approach fill.
- Mr. Hambleton will also send the approach fill construction sequence.
- He has had the plate locations staked, at my request of Dec. 6/71, and will send this office the exact pipe lengths to plates as installed, and all subsequent settlement readings at one month intervals.
- Mr. Hambleton has also agreed to have Nuclear Density checks taken on the approach fills.
- I advised each of the aforementioned project supervisors that the resources of this office would be at their disposal, should any problem arise, regarding the Instrumentation program.

*William F. Deike*

William F. Deike

c.c. File - 69-F-35  
- 67-F-112  
- 67-F-113  
- 67-F-114  
- 68-F-33  
- 70-F-2A, 2B  
- 70-F-11  
-Instrumentation Records

#68-F-33

WP#34-66-09

HWY#417 8

BOUNDARY ROAD

