

MEMORANDUM

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

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

Attention: Mr. S. McComble

Date: March 25, 1968

Our File Ref.

In Reply To

MAR 29 1968

SUBJECT:

FOUNDATION INVESTIGATION REPORT
For
Eastbound Lane and Westbound Lane
Underpass Structures
At the Crossing of Eighth Line Road
And Proposed Hwy. #417
Twp. of Gloucester, Co. of Carleton
District No. 9 (Ottawa)
W.J. 67-F-114 -- W.P. 34-66-08

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 feel free to contact our Office.

AGS/MdeF
Attach.

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

Foundations Files
Gen. Files

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

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FOUNDATION INVESTIGATION REPORT
For
Eastbound Lane and Westbound Lane
Underpass Structures
At the Crossing of Eighth Line Road
And Proposed Hwy. #417
Twp. of Gloucester, Co. of Carleton
District No. 9 (Ottawa)
W.J. 67-F-114 -- W.P. 34-66-08

1. INTRODUCTION:

The Foundation Section was requested to carry out a subsurface investigation at the site of the proposed underpass structures; the site is located some 14 miles south-east of Ottawa, in the Township of Gloucester, County of Carleton. The request was contained in a memo from the Bridge Division - (Mr. G. Scott, Regional Bridge Location Engineer), dated October 6, 1967. An investigation was subsequently carried out by this Section to determine the subsoil and groundwater conditions at this site.

This report contains all the factual data obtained from the investigation, together with recommendations pertaining to the foundations of the proposed structures as well as the stability of the approach embankments.

2. DESCRIPTION OF THE SITE AND GEOLOGY:

The site is located about 1 mile south-west of Carlsbad Springs in the vicinity of the intersection of Eighth Line Rd. and Halls Rd. Eighth Line Rd., in the vicinity of the site, is a paved roadway approximately 22 feet wide with the grade at about elevation 261. Drainage ditches, some 4 to 5 feet deep and about 20 feet wide at the top, are located on either side of the roadway. The surrounding terrain is quite flat-lying with tree and brush cover.

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

Physiographically, the site is situated in the "Russell and Prescott Sand Plain". In this area a sand mantle, varying in thickness from 20 to 30 feet in the north to 10 feet or less in the south, overlies a considerable thickness of marine silty 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 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, as well as two additional dynamic cone tests, were put down at this site. The borings were advanced by using a conventional diamond drill rig adapted for soil sampling purposes. The majority of the dynamic cone penetration tests were, however, put down by a Penndrill auger machine.

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" and 3" I.D. Shelby tubes, which were manually pushed into the soil. In an

cont d. /3 ...

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

effort to reduce the degree of disturbance, some 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 5 boreholes by obtaining either BXL or AXT size rock core samples.

The groundwater level conditions across the site were determined by installing sealed piezometers in two of the boreholes. This information was supplemented by recording the water level in the open holes at the remaining boring locations during the period of the investigation.

The locations and elevations of all borings are shown on Drawings 67-F-114A (E.B.L.) and 67-F-114B (W.B.L.), 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 examination, laboratory testing was carried out on selected representative samples to determine the following engineering properties of the overburden:

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

The results of this testing are plotted on the Record of Borehole log sheets and summarized in the Figures contained in Appendix I of this report.

cont'd. /4 ...

4. SUBSOIL CONDITIONS:

4.1) General:

The surficial deposit across the site is composed of a compact to dense fine sand, which is typically about 14 feet thick. The sand is underlain by a firm to stiff, highly plastic sensitive marine clay stratum varying in thickness from 65 to 81 feet. Directly underlying the clay stratum is a deposit of glacial till, up to 13 feet thick, composed primarily of very stiff to hard clayey silt with some sand and gravel. The glacial till is, in turn, underlain by shale bedrock.

The boundaries between the various soil strata, as determined in the boreholes, are shown on the accompanying borehole log sheets. The stratigraphical profile, shown on Drawings 67-F-114A and 67-F-114B, is inferred from this data.

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

4.2) Fine Sand - Surficial Deposit:

A deposit of fine sand with some silt and a trace of clay was encountered at all of the boring locations. The thickness of this deposit varies from 12 feet (at B.H. #8) to 18 feet (at B.H. #10). The upper 4 to 6 feet of the fine sand is reddish-brown to brown in colour, while below this it is grey. It is inferred that the lighter colour of the upper zone is due to desiccation. Occasional pockets of organic matter are encountered in the upper 2 to 3 feet of the stratum. Grain-size distribution curves for samples of the fine 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 16% to 24%.

Standard penetration tests, carried out within the deposit, are plotted on the borelog sheets as well as on Figure 1.

cont'd. /5 ...

4. SUBSOIL CONDITIONS: (cont'd.) ...

4.2) Fine Sand - Surficial Deposit: (cont'd.) ...

The results of this testing gave 'N' values which generally vary in a random fashion from 5 blows/ft. to as many as 75 blows/ft. Based on these values, it is estimated that the relative density of the deposit varies from loose to very dense, being typically in the compact range.

4.3) Sensitive Clay to Silty Clay:

The fine sand deposit is underlain by the predominant overburden stratum across the site, a sensitive marine silty clay with occasional inclusions of organic matter. The thickness of this stratum ranges from 65 feet at B.H. #10 to 81 feet at B.H. #9. Alternate red-brown and grey layers, with the individual layers some 4 inches thick, were encountered in the upper 5 to 7 feet of the stratum. Further, occasional seams and layers of sand and silt up to 1/2 inch thick, are present below about elevation 200. Grain-size distribution curves for samples of the silty clay are shown on Figure 5 in Appendix I.

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:

cont'd. /5a...

4. SUBSOIL CONDITIONS: (cont'd.) ...

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

		<u>Range</u>	<u>Average</u>
Bulk Density (p.c.f.)	(γ)	94 - 111	98
Liquid Limit (%)	(W_L)	41 - 81	58
Plastic Limit (%)	(W_P)	18 - 32	26
Natural Moisture Content (%)	(W)	41 - 82	68
Liquidity Index	(I_L)	0.8 - 2.9	1.5
Initial Void Ratio	(e_o)	2.1 - 2.2	
Compression Index	(C_c)	1.4 - 2.0	
Preconsolidation Pressure (p.s.f.)	(P_o)	2,200 - 3,000	
		<u>Range</u> (C_u)	<u>Range</u> Sensitivity (S)
Undrained Shear Strength (p.s.f.)	(C_u)		
i) Field Vanes		350 - 1,600	3 - 13
ii) Lab. Vanes		300 - 1,440	-
iii) Lab. Testing		270 - 1,340	-

The Atterberg limit tests, summarized above, are also plotted on the Plasticity Chart, Figure 7. These results indicate that, in general, the silty clay is inorganic having an intermediate to high plasticity, with the natural water content exceeding the liquid limit by a significant degree. Referring to Figure 1, it

cont'd. /6 ...

4. SUBSOIL CONDITIONS: (cont'd.) ...

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

can be seen that the undrained shear strength increases in a linear fashion with depth, as represented by a C_u/P_o ratio of about 0.3, where P_o is the effective overburden pressure. Based on these results, it is estimated that the consistency of the stratum varies from firm, immediately below the sand deposit, increasing to stiff with depth. The undrained shear strength values obtained from the 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 silty clay.

The consolidation characteristics of the stratum were determined by carrying out four laboratory tests, the results of which are shown as Void Ratio vs Pressure plots, on Figures 9 to 12, inclusive. The results of this testing indicate that the clay is preconsolidated by about 400 to 700 p.s.f. in excess of 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 or Silt with some Sand and Gravel -
Glacial Till:

Directly underlying the clay stratum is a deposit of glacial till composed of a heterogeneous mixture of clayey silt, with some sand and gravel. The surface of this deposit was encountered at about elevation 180 at the eastern extremity of the site, decreasing to about elevation 163 to the west - i.e., it slopes towards the west. The thickness of the glacial till, where penetrated, varied from 7 to 13 feet. The upper 3 to 4 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.

cont'd. /7 ...

4. SUBSOIL CONDITIONS: (cont'd.) ...

4.4) Clayey Silt or Silt with some Sand and Gravel -
Glacial Till: (cont'd.) ...

Occasional seams of sand and silt, up to 1/2 inch thick, were encountered throughout the deposit. In addition, the 3 to 5 feet thick zone immediately above bedrock, is often very bouldery, as indicated by the necessity of advancing the borings by diamond drilling techniques. The boulders encountered in this zone were generally about 5 inches in size. Typical grain-size distribution curves, obtained from samples of the deposit, are shown on Figure 6.

The Atterberg limit tests carried out on representative samples of the cohesive portion of the glacial till, are plotted on the Plasticity Chart, Figure 9. These results gave values for the liquid limit and plastic limit that range from 18 to 25 and 13 to 16, respectively. The corresponding natural water content is typically about 4% below the plastic limit. Based on these values, it is estimated that the cohesive portion of the glacial till is inorganic and of low plasticity.

Standard penetration tests carried out in the deposit gave 'N' values which range from 11 to 21 blows/ft., in the upper weathered zone, increasing with depth to as many as 94 blows/ft. Based on these values, it is estimated that the cohesive portion of the glacial till varies from very stiff, in the upper weathered zone, to hard below this zone. The relative density of the non-cohesive portions of the deposit are estimated to range from compact, in the upper weathered zone, to very dense with depth.

4.5) Shale Bedrock:

Bedrock was established in five of the borings, namely: B.H.'s #4, 5, 8, 9 and 10, by obtaining from 4 to 18 feet of either AXT or BXL rock core. The depth at which bedrock was encountered ranged from about elevation 152 (B.H. #9) to 170 (B.H. #10) - i.e., from 80 to 98 feet below existing ground surface.

cont'd. /8 ...

4. SUBSOL CONDITIONS: (cont'd.) ...

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

The bedrock is composed of a grey, fossiliferous shale, the upper 3 to 8 feet of which is in a fractured and jointed condition. At B.H. #9 this fractured layer was overlain by a transitional bedrock zone some 9 feet thick. The shale in this zone has been highly weathered, with numerous silty clay seams and layers throughout. Below the upper weathered and fractured zones the bedrock is reasonably sound.

5. GROUNDWATER CONDITIONS:

Groundwater level observations were carried out during the period of the investigation in 1) sealed piezometers installed in B.H.'s #4 and 5, and 11) the open boreholes at the remaining locations. These observations, which are recorded on the borehole logs and summarized on Drawings 67-F-114A and 67-F-114B, indicate that the groundwater level in the surficial sand and sensitive clay stratum ranges from about elevation 253 to 255 - i.e., some 6 to 9 feet below ground surface.

The piezometers installed in the glacial till indicate that the piezometric groundwater level, within this deposit, is between elevations 238 and 240 - i.e., some 21 to 23 feet below ground surface. This is at a considerably lower level than the groundwater level encountered in the overlying deposits. It is pertinent to note that the heterogeneous glacial till is more permeable than the overlying subsoil. In addition, there are numerous very pervious sand and silt seams throughout the glacial till. It is, therefore, inferred that the groundwater level within the glacial till may be at a lower elevation due to downward drainage which occurs once the more pervious zones are intersected.

cont'd. /9 ...

6. DISCUSSION AND RECOMMENDATIONS:

6.1) General:

It is proposed to construct twin underpass structures to carry Eighth Line Road over the East and Westbound lanes of proposed Highway #417. The structure over the East and Westbound lanes of Hwy. #417, designated as the E.B.L. and W.B.L. structures, respectively, will be constructed end to end being about 220 feet apart. Present proposals call for three-span structures (62'-77'-54' and 57'-70'-47' for the E.B.L. and W.B.L. structures, respectively. The maximum proposed profile grade of Eighth Line Road, in the vicinity of the crossings, is elevation 288. At this grade the associated approach embankments will have a maximum height of about 27 feet above ground surface. The embankments will have a crest width of 36 feet.

The East and Westbound lanes of Hwy. #417 will initially have two 12-foot wide paved lanes with provision for a third lane; the roadway cross-section will also incorporate 11-foot wide shoulders. The finished grade will be elevated some 4 to 5 feet above surrounding ground level - i.e., it will be between elevations 264 to 265 in the vicinity of the crossings.

Underlying about 14 feet of sand is the predominant stratum across the site, composed of a firm to stiff, sensitive marine clay varying from 65 to 81 feet in thickness. The clay is underlain by up to 13 feet of stiff to hard cohesive 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 requires that steps must be taken to ensure overall stability of the approach embankments, and that the structures must be supported on piled foundations. As the stability of the approach fills is the major problem at this site, it will be discussed first.

cont'd. /10 ...

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

6.2) Approach Embankments:

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 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 sections. The following assumptions were made:

1) Soil Properties:

Fill Material

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

Foundation Subsoil

<u>Elev.</u>	<u>Subsoil</u>	<u>Parameters</u>
261 - 247	Sand - Surficial Deposit	$\gamma = 125$ p.c.f. $\gamma' = 63$ p.c.f. $\phi = 30^\circ$
247 - 230	Sensitive Clay -	$\gamma = 98$ p.c.f. $\gamma' = 33$ p.c.f. 500 p.s.f.
230 - 213	" "	800 p.s.f.
213 - 195	" "	1,000 p.s.f.
195 - 170	" "	1,300 p.s.f.

cont'd. /11 ...

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

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

2) All the berms required have been assumed to be at the mid-height of the section. The surface of the berms 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 paragraph. The requirements listed, provide a minimum factor of safety of 1.3 with respect to stability.

Stability of Approach Embankments

		Length of Berm Required At Mid-Height
i) <u>Longitudinal Direction</u>		
- { West Approach to Structure over E.B.L. East " " " " W.B.L. }	-	30 ft.
- { East Approach to Structure over E.B.L. West " " " " W.B.L. }	-	40 ft.
ii) <u>Transverse Direction</u>		
- All Approaches	-	0 - 60 ft.

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

1) At the proposed profile grade, berms will be required in both the longitudinal and transverse direction at all the approach fill locations.

cont'd. /12 ...

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

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

2) The berm requirements in the longitudinal direction will entail a lengthening of the structures from that proposed. The length of the structures over the E.B.L. and W.B.L. of Hwy. #417 will have to be increased by something of the order of 40 and 50 feet, respectively.

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

The underlying highly compressible clay stratum will undergo excessive settlements due to consolidation, over a long period, under the weight of the approach embankments. Settlement computations were, therefore, carried out, the results of which are summarized on Figure 3 in the Appendix. The maximum consolidation settlement will occur at some point midway between the two structures, where the maximum height of fill of 27 feet is realized. The computations indicate that this settlement could be as much as 7 feet under the centre-line of the embankment.

The total amount of the consolidation settlement should occur within a period of 65 years, while about 40% will occur in the first 7 years (refer to plot on Figure 3). It would be advantageous, therefore, to construct the embankments first and leave them in place for as long as possible prior to construction of the structures. In any event, the final paving should be delayed for as long a period as possible.

In conclusion, it is considered that the immediate approaches to the structures will present a permanent maintenance problem and will require re-paving from time to time.

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

6.3) 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, some negative skin frictional forces can be imposed on the end-bearing piles supporting the abutments. These forces, combined with movement of subsoil due to strain imposed by the embankment loading, will generally tend to displace the piles laterally; this can cause rotation of the abutments. Such movements have been noted in the past at the location of structures founded in a similar manner on extensive deposits of "Leda Clay". 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 of the abutment in the longitudinal direction. In addition, no bouldery or rock fill should be placed in areas where piles are to be driven.

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 6 to 9 feet below ground surface. Because these excavations will be carried out through a relatively pervious sand deposit, seepage may occur. This could be dealt with by pumping from sumps or alternatively, by excavating from within closed timber sheeting.

cont'd. /14 ...

7. SUMMARY:

A foundation investigation at the site of the proposed underpass structures to carry Eighth Line Road over the East and Westbound lanes of proposed Hwy. #417, in the Township of Gloucester, County of Carleton, is reported.

Underlying about 14 feet of sand is the predominant stratum across the site, composed of a firm to stiff, sensitive marine clay varying from 65 to 81 feet in thickness. The clay is underlain by up to 13 feet of stiff to hard cohesive glacial till which, in turn, is followed by shale bedrock. The ground-water level in the surficial sand deposit and underlying clay stratum was, at the time of the investigation, some 6 to 9 feet 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 direction for all the approaches. The berm requirements in the longitudinal direction will entail an increase in the length of both structures over that proposed, as discussed in the report.

In view of the excessive settlements which have been estimated, it would be advantageous to allow as much time as possible between the construction of the approach embankments and completion of the final grade. There will be a continuing maintenance problem.

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8. MISCELLANEOUS:

The field work for this project was carried out during the periods of December 5 and 15, 1967, and January 30 and February 7, 1968, under the supervision of Mr. B. T. Darch, Senior Foundation Engineer.

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

This report was written by Mr. Darch and was reviewed by Mr. M. Devata, Supervising Foundation Engineer.

March, 1968.

APPENDIX I

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 1

FOUNDATION SECTION

JOB 67-F-114

LOCATION Sta. 33 + 20 @ 8th Line Rd. o/s 18' Lt.

ORIGINATED BY BTD

W. P. 34-66-080

BORING DATE Dec. 15, 1967, Jan. 2, 1968

COMPILED BY _____ BTL

DATUM Geodetic

BOREHOLE TYPE Diamond Drill - HX, BX Casing

CHECKED BY

SOIL PROFILE			SAMPLES			DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT							LIQUID LIMIT — WL PLASTIC LIMIT — PL WATER CONTENT — WP			BULK DENSITY P.C.F.	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT	FLEV SCALE	SHEAR STRENGTH P.S.F.					WATER CONTENT %					
							• Quick Triaxial + Field Vane x Lab Vane					w p s L					
							400	800	1200	1600	2000	20 40 60					
261.5	Ground Level															Br.Sa.Si.Cl	
0.0	Fine sand with some silt & trace of clay (reddish-brown to brown). Compact.		1	SS	20	260										2 84 11 3	
257.0																0.84 14 2	
4.5	Fine sand with some silt & a trace of clay (grey)		2	SS	25											254.0	
			3	SS	41												
248.5	Compact to dense.		4	SS	33	250											
13.0	Layered clay (alternate grey & red-brown layers up to 1" thick).		5	SS	1		+6.5										
243.5																	
18.0	Sensitive clay to silty clay with occasional inclusions of organic matter.		6	3"TW	PM	240	+11.0	x4.5	x8.0	+15.0					94		
	(random silt seams & layers up to 1/2" thick below about elev. 190) (grey)		7	2"TP	PM												
			8	2"TP	PM	230	+7.0	x7.0	+5.0	+6.0					97		
			9	2"TP	PM												
			10	2"TP	PM	220	+12.0	+13.0							97		
			11	2"TP	PM		+7.0	+5.2									
	Firm to stiff.		12	2"TP	PM	210	x10.5	+8.0	+9.5								
			13	2"TP	PM	200	+8.0	+10.0									
			14	2"TP	PM	190	+10.0	+10.0									
			15	2"TP	PM	180	x4.0	+12.0							111		
176.0	Clayey silt or silt, some sand & gravel (glacial till)																
85.5	(occasional seams of silt up to 1/2" thick throughout) (grey)		16	SS	39											36 39 18 7	
170.2	Very stiff to hard or dense to very dense.		17	SS	91	170	200 blows for 5"										
91.3	End of Borehole						0 15 + 5 10	% Strain									

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 2

FOUNDATION SECTION

108 67-F-114

LOCATION Sta. 33 + 77 @ 8th Line Rd. o/s 18' Rt.

ORIGINATED BY BTD

W. P. 34-66-080

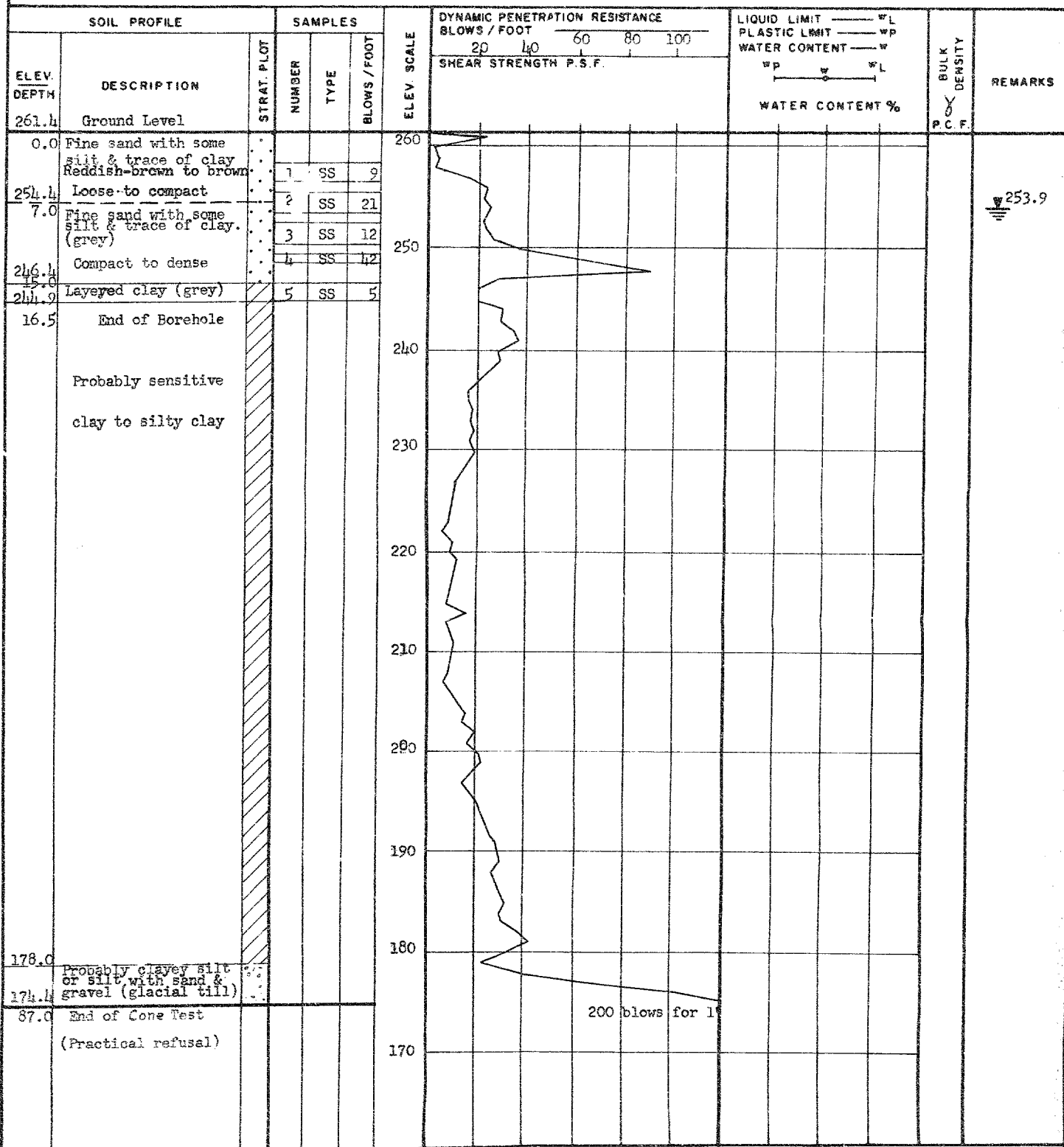
BORING DATE December 7, 1967

COMPILED BY _____ BTD

DATUM Geodetic

BOREHOLE TYPE Penn Drill

CHECKED BY



[illegible]

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 1

FOUNDATION SECTION

JOB 67-F-114

LOCATION Sta. 34 + 94 @ 8th Line Rd. o/s 18th Rt.

ORIGINATED BY BTB

W.P. 34-66-080

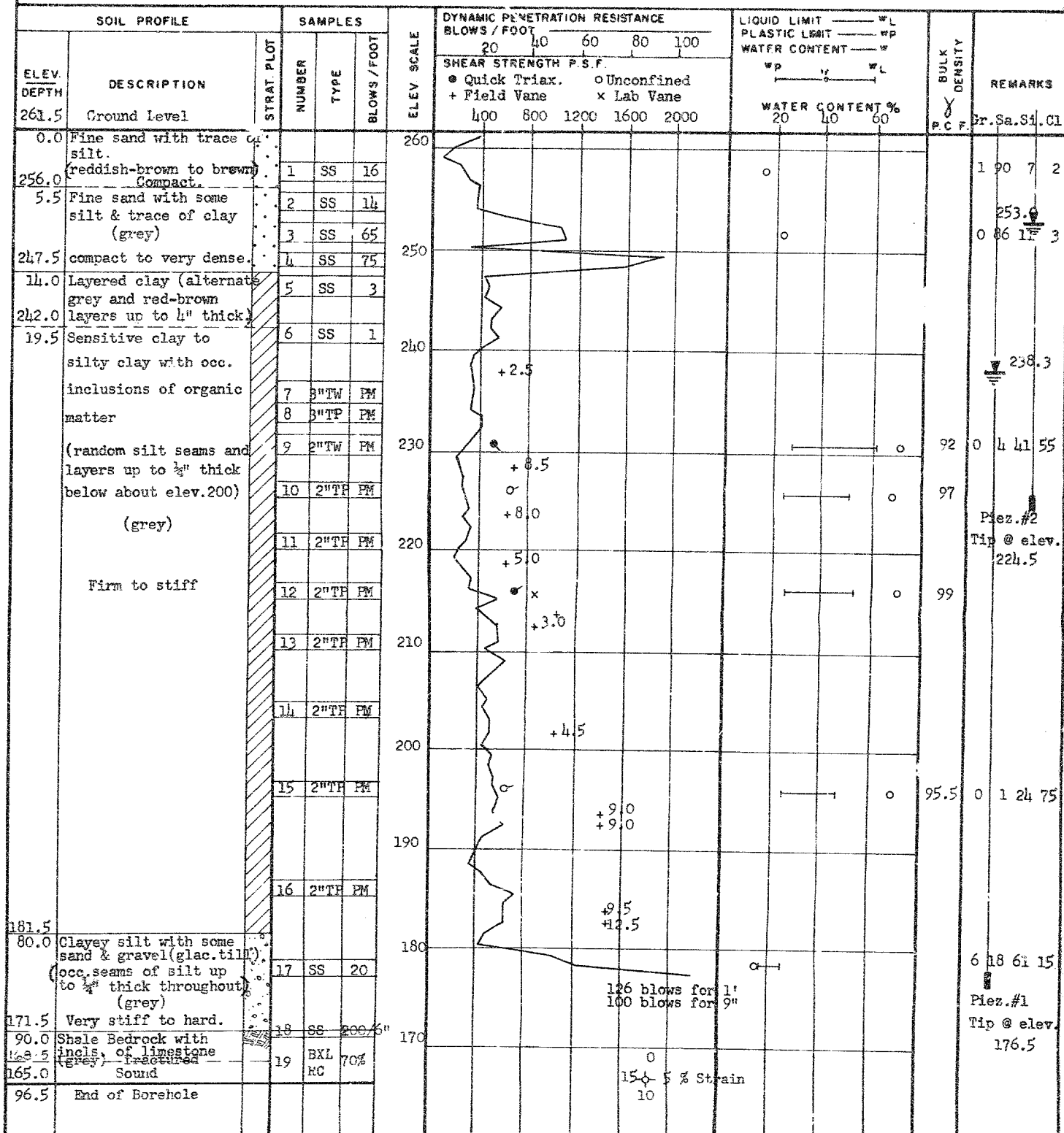
BORING DATE Dec. 8 to Dec. 15, 1967 - Jan. 2, 3 & 30, 1968

COMPILED BY BTB

DATUM Geodetic

BOREHOLE TYPE Penndrill, Diamond Drill - NX, BX Casing, BXL Core

CHECKED BY



DEPARTMENT OF HIGHWAYS - ONTARIO

RECORD OF BOREHOLE NO. 5

FOUNDATION SECTION

MATERIALS & TESTING DIVISION

JOB 67-F-1114

LOCATION Sta. 29 + 06 @ 8th Line Rd. o/s 18' Lt.

ORIGINATED BY LTD

W.P. 34-66-080

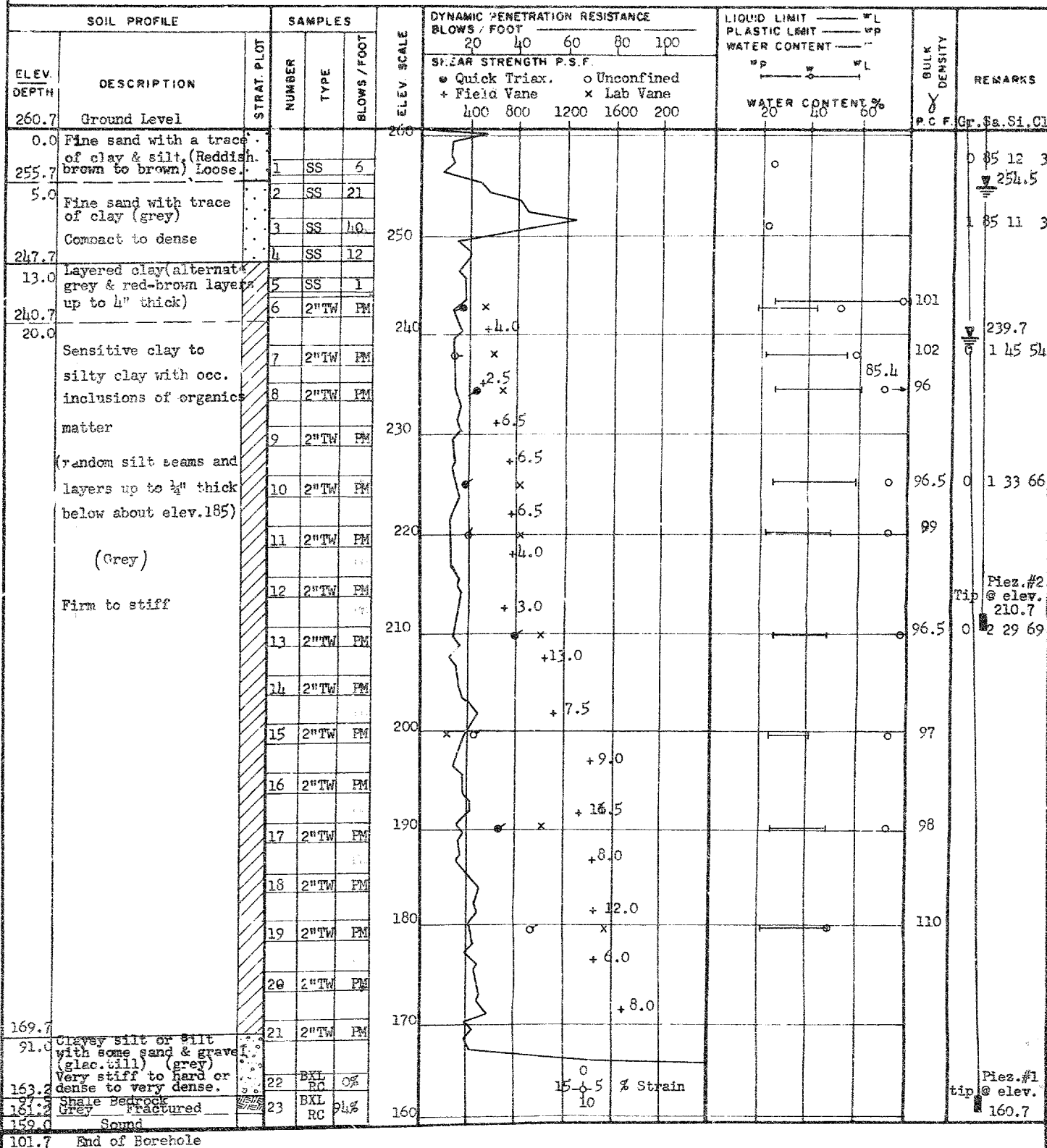
BORING DATE December 5 to Dec. 11, 1967

COMPILED BY LTD

DATUM Geodetic

BOREHOLE TYPE Diamond Drill, NX, BX Casing, BXL Core

CHECKED BY



MATERIALS & TESTING DIVISION

FOUNDATION SECTION

LOCATION Sta. 30 + 36 @ 8th Line Rd. o/s 18' Lt.

ORIGINATED BY BTD

BORING DATE December 5, 1967

COMPILED BY _____ BTD

BOREHOLE TYPE Penn Drill

CHECKED BY

[illegible]

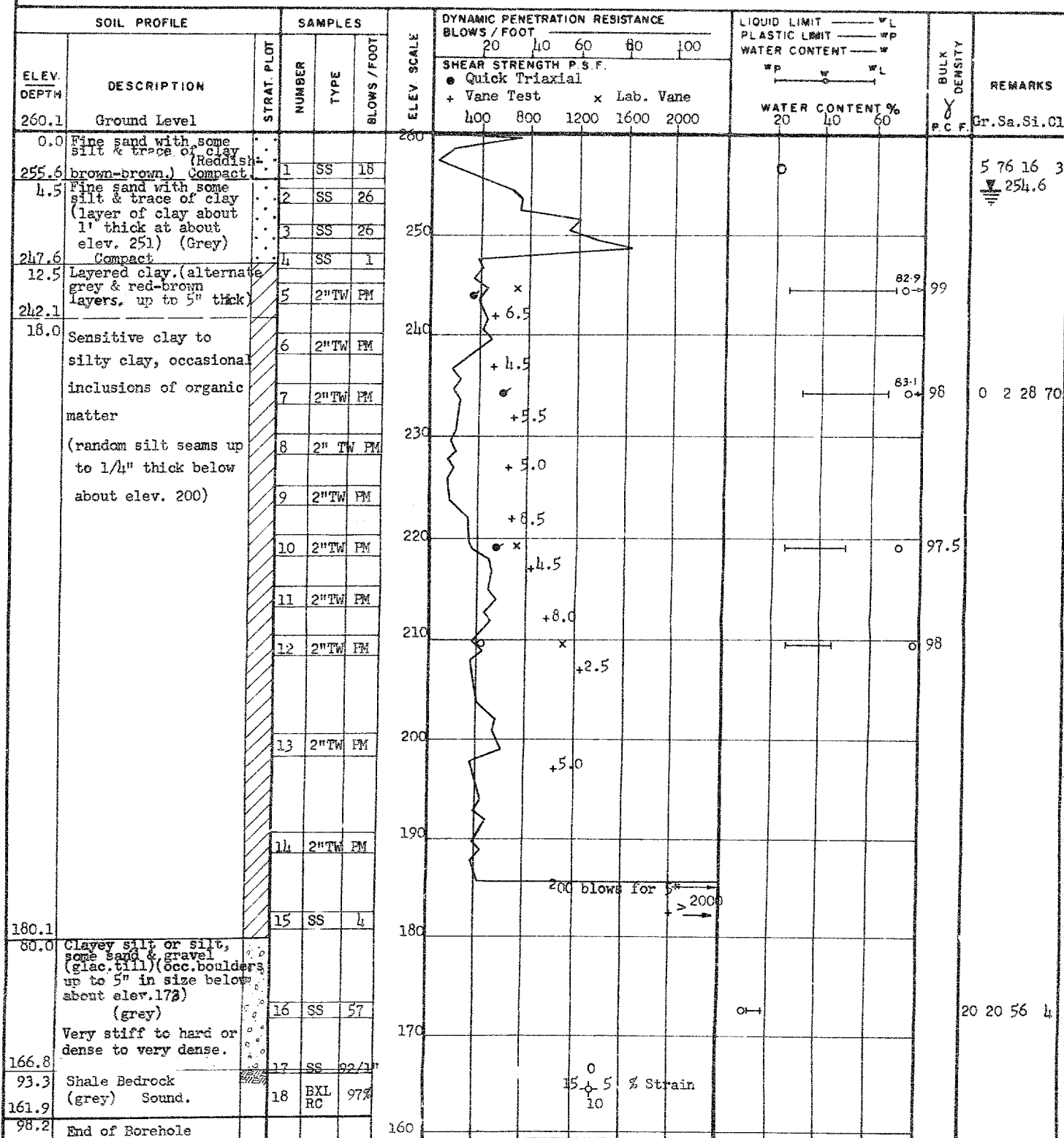
DEPARTMENT OF HIGHWAYS - ONTARIO

RECORD OF BOREHOLE NO. 8

FOUNDATION SECTION

MATERIALS & TESTING DIVISION

JOB 67-F-114 LOCATION Sta. 31 + 00 @ 8th Line Rd. o/s 18' Rt. ORIGINATED BY BTD
W.P. 34-66-080 BORING DATE Dec. 13 to 15, 1967 COMPILED BY BTD
DATUM Geodetic BOREHOLE TYPE Diamond Drill -NX,BX Casing - BXL Core CHECKED BY ALP



FOUNDATION SECTION

JOB 67-F-114 LOCATION Sta. 26 + 80 @ 8th Line Rd. o/s 18' Rt. ORIGINATED BY BTD
W.P. 31-66-08 BORING DATE Jan. 30 - Feb. 5, 1968 COMPILED BY BTD
DATUM Geodetic BOREHOLE TYPE Diamond Drill - NX, BX Casing, AXT Core CHECKED BY BR

SOIL PROFILE			SAMPLES			DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT			LIQUID LIMIT — L PLASTIC LIMIT — P WATER CONTENT — W			BULK DENSITY P C F	REMARKS	
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT	20	40	60	80	100	W P			W L
261.0	Ground Level					SHEAR STRENGTH P.S.F. + Field Vane x Lab Vane • Quick Triaxial			WATER CONTENT % 20 40 60					
						400	800	1200	1600	2000				
0.0	Fine sand with some silt and a trace of clay. Reddish-brown to brown Compact		1	SS	30	FROZEN								
254.5			2	SS	13									
6.5	Fine sand with some silt & trace of clay (occasional seams of clay up to 1/2" thick below about elev. 248)		3	SS	19									
244.0	Grey. Compact to dense		4	SS	36									
17.0	Layered clay (alternate grey & red-brown layers up to 1/2" thick)		5	SS	3									
241.0			6	2"TW	PM									
20.0	Sensitive clay to silty clay, inclusions of organic matter (random silt seams & layers up to 1/2" thick throughout)		7	2"TW	PM							104		
	(grey)		8	2"TW	PM									
			9	2"TW	PM							98		
			10	2"TW	PM									
			11	2"TW	PM							98		
	Firm to stiff		12	2"TW	PM									
			13	2"TW	PM									
			14	2"TW	PM							100		
			15	2"TW	PM									
			16	2"TW	PM									
			17	2"TW	PM							111		
			18	2"TW	PM									
163.0			19	SS	11									
98.0	Clayey silt to silt with sand & gravel (glac. till) (Grey) Very stiff to hard or compact to very dense		20	AXT RC	19%								26 32 30 10	
152.0			21	AXT RC	16%									
100.0	Fossiliferous Shale Bedrock (grey)		22	AXT RC	37%									
143.0	Highly weathered		23	AXT RC	100%									
118.0	fractured													
139.0														
122.0	sound													
134.0														
127.0	End of Borehole													

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 10

FOUNDATION SECTION

JOB 67-F-114 LOCATION Sta. 36 + 75 @ 8th Line Rd. o/s 15' Lt. ORIGINATED BY BTD
 W.P. 3h-56-08 BORING DATE Jan. 31 - Feb. 6, 1968 COMPILED BY BTD
 DATUM Geodetic BOREHOLE TYPE Diamond Drill - HX, BX Casing, AXT Core CHECKED BY 42

SOIL PROFILE			SAMPLES			ELEV SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT			LIQUID LIMIT — WL PLASTIC LIMIT — WP WATER CONTENT — W			BULK DENSITY P.C.F.	REMARKS	
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT		SHEAR STRENGTH P.S.F.					WATER CONTENT %			
							+ Quick Triaxial								
							+ Field Vane x Lab Vane								
261.5	Ground Level					400	800	1200	1600	1800	20	40	60		
0.0	Fine sand with some silt & trace of clay (reddish-brown to brown) Dense.		1	SS	38										
255.0			2	SS	20										
6.5	Fine sand with some silt and a trace of clay (Grey)		3	SS	27										
			4	SS	29										
243.2	Compact		5	SS	11										
18.3	Layered clay (alternate reddish-brown & grey layers up to 5" thick)		6	3"TP	PM										
239.5			7	2"TP	PM										
22.0	Sensitive clay to silty clay, occasional inclusions of organic matter		8	3"TP	PM										
			9	2"TP	PM										
	(random silt seams & layers up to 1/2" thick below about elev. 188)		10	3"TP	PM										
			11	2"TP	PM										
			12	3"TP	PM										
			13	2"TP	PM										
			14	2"TP	PM										
178.5			15	2"TP	PM										
83.0	Clayey silt or silt with some sand & gravel (glacial till) (grey)		16	SS	21										
170.3	Very stiff to hard or compact to very dense.														
91.2	Fossiliferous shale bedrock (grey) (fractured)		17	AXT RC	15%										
162.5			18	AXT RC	47%										
99.0	(sound)														
156.5															
102.0	End of Borehole														

200 blows/ft

End of Pen. Test
at elev. 180.2
(Practical refusal)

0
15 5 % Strain
10

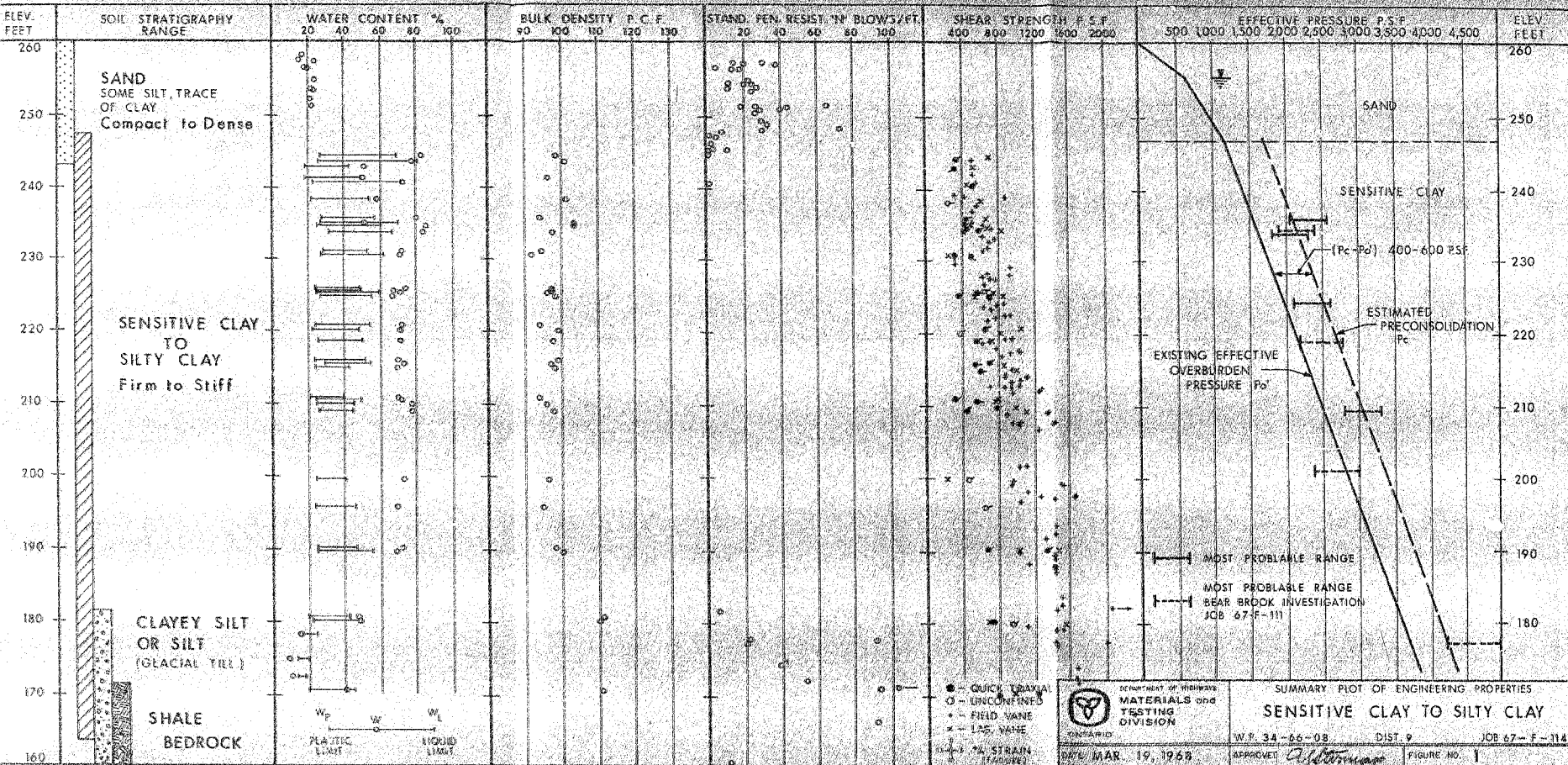
254.5
0 90 7 3

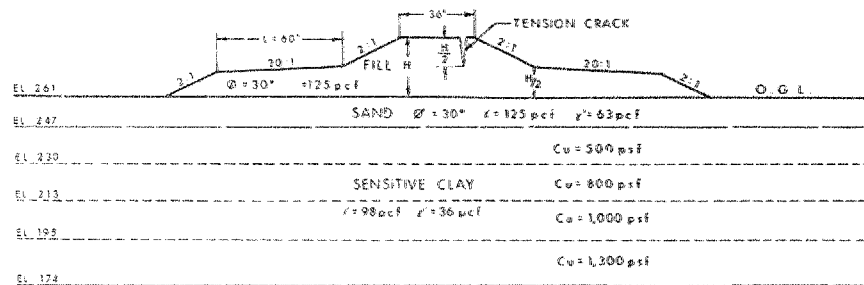
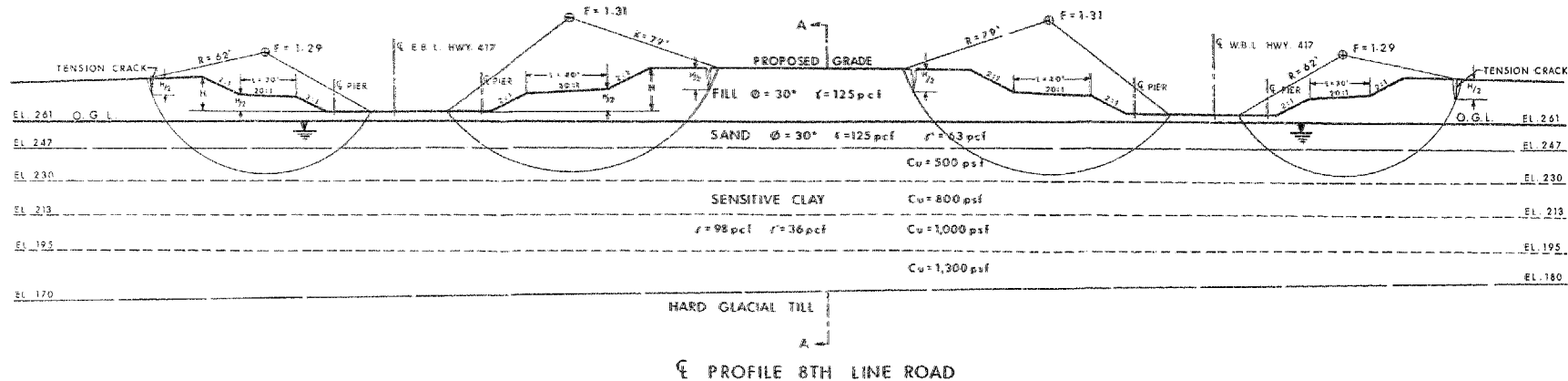
96 0 0 66 34
0 0 39 61

94.5

94

94



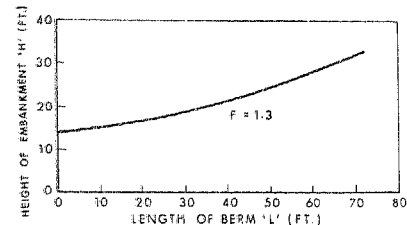


SECTION A-A

SCALE 1" = 40'

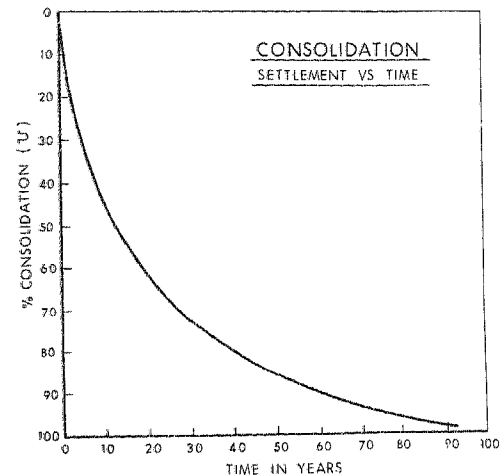
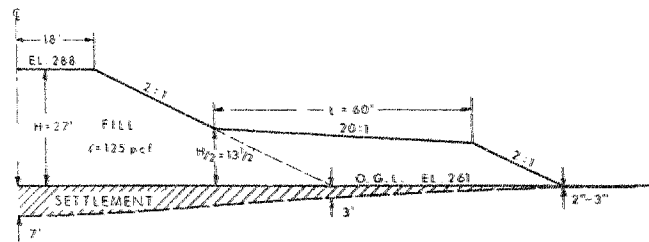
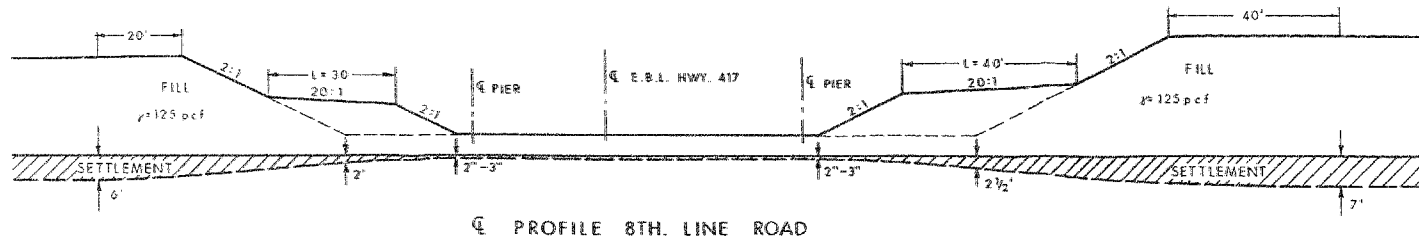
LEGEND


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



BERM REQUIREMENTS
(TRANSVERSE DIRECTION)

<p>ONTARIO</p>	<p>DEPARTMENT OF HIGHWAYS MATERIALS and TESTING DIVISION</p>	<p>SUMMARIZED RESULTS OF STABILITY ANALYSES APPROACH EMBANKMENTS (LONGITUDINAL & TRANSVERSE DIRECTIONS)</p>	
	<p>DATE MAR. 25, 1968</p>	<p>APPROVED <i>[Signature]</i></p>	<p>W.P. 34-66-08 DIST. 9 JOB 67-F-114</p>
		<p>FIGURE NO. 2</p>	



 ONTARIO DEPARTMENT OF HIGHWAYS MATERIALS and TESTING DIVISION	SUMMARIZED RESULTS OF SETTLEMENT COMPUTATIONS		
	APPROACH EMBANKMENTS		
DATE MARCH 26, 1968	APPROVED <i>alton</i>	W.P. 34-66-08	DIST. 9
			JOB 67-F-114
			FIGURE NO. 3

MEMORANDUM

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

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

ATTENTION:

DATE: November 5, 1968

OUR FILE REF:

IN REPLY TO

SUBJECT:

8th Line Road Underpass - Site 3-269
W.P. 34-66-08 -- W.J. 67-F-114
Hwy. #417 -- District No. 9 (Ottawa)

We have reviewed the Preliminary Drawing D-6480-P for the above mentioned project and submit the following comments:

It is understood that the structure approaches only, are contemplated at this stage and, therefore, we are not making any comments pertaining to structure foundations. Regarding the details of the approach embankments including the surcharge, it appears that the designer has complied with our verbal recommendations. These recommendations have been made based upon the following information:

Stability:

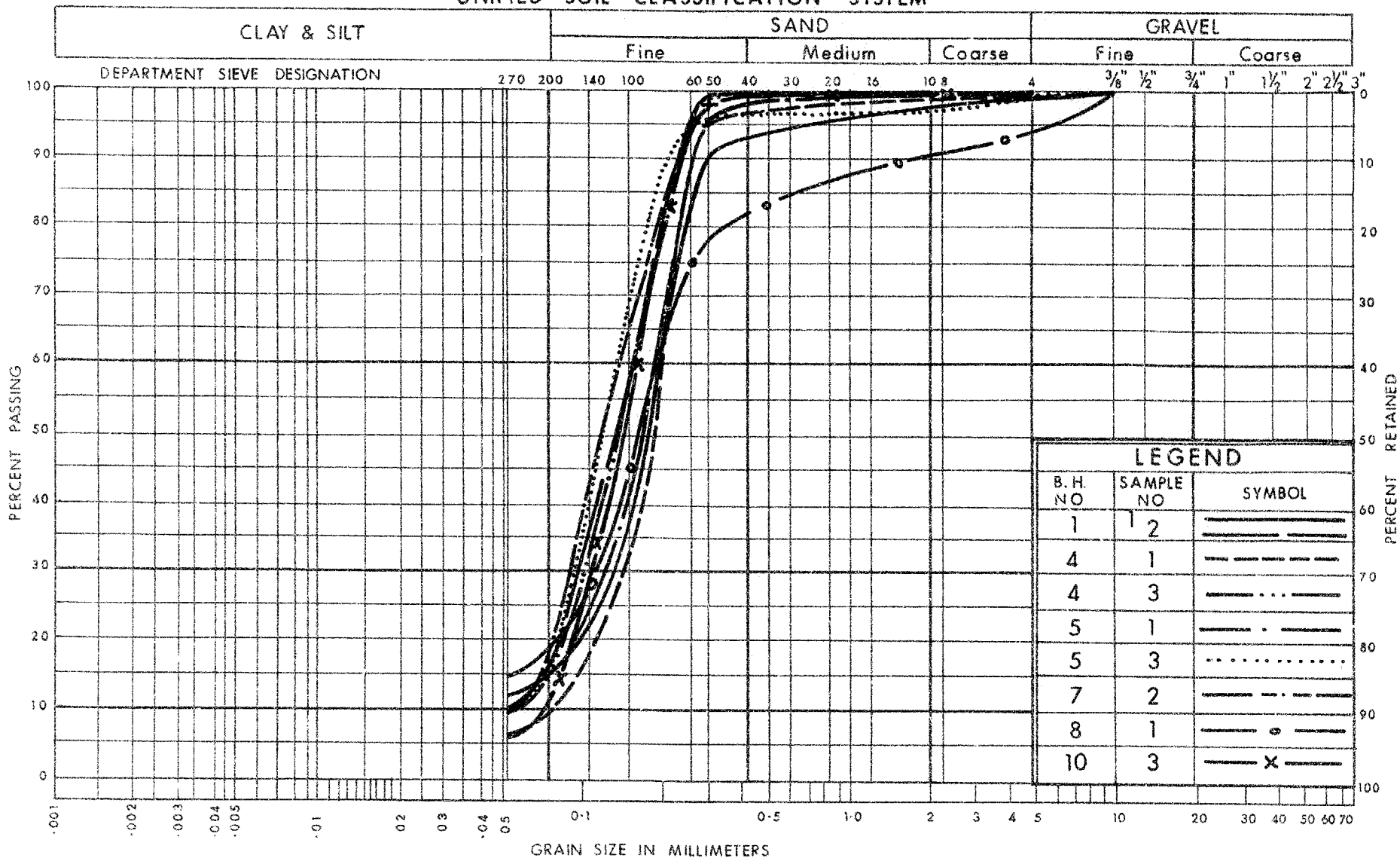
Limiting height of fill above original ground surface.	12 ft.
Minimum length of bench in the longitudinal direction at ground surface.	35 ft.
Depth of cut below original ground surface.	6 ft.
Factor of safety against base failure.	<u>1.33</u>
Factor of safety with a 4-ft. surcharge for the above case.	<u>1.04</u>

Settlements:

Height of Fill	Total Settlement in Inches for Various Periods				
	2 Yrs.	7 Yrs.	15 Yrs.	25 Yrs.	50 Yrs.
12 ft. (Design height)	6"	9"	14"	18"	26"
16 ft. (Design height + 4-ft. surcharge)	9"	12"	21"	27"	38"
(Percentage Consolidation) -	18%	27%	45%	58%	85%

cont'd. /2 ...

UNIFIED SOIL CLASSIFICATION SYSTEM

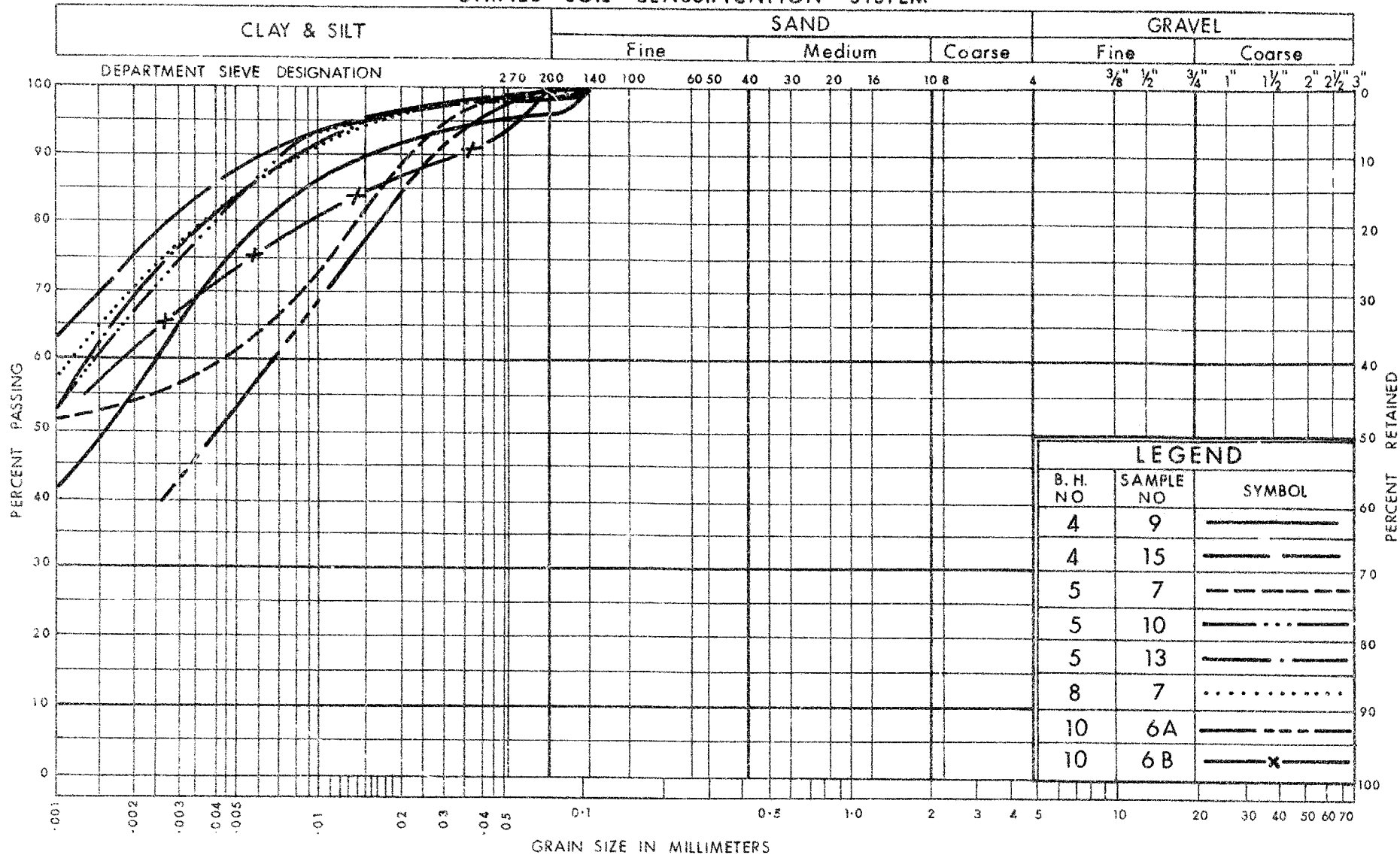


DEPARTMENT OF HIGHWAYS
MATERIALS and
TESTING
DIVISION

GRAIN SIZE DISTRIBUTION FINE SAND

W.P. No. 34 - 66 - 08
JOB No. 67 - F - 114
FIG. NO. 4

UNIFIED SOIL CLASSIFICATION SYSTEM

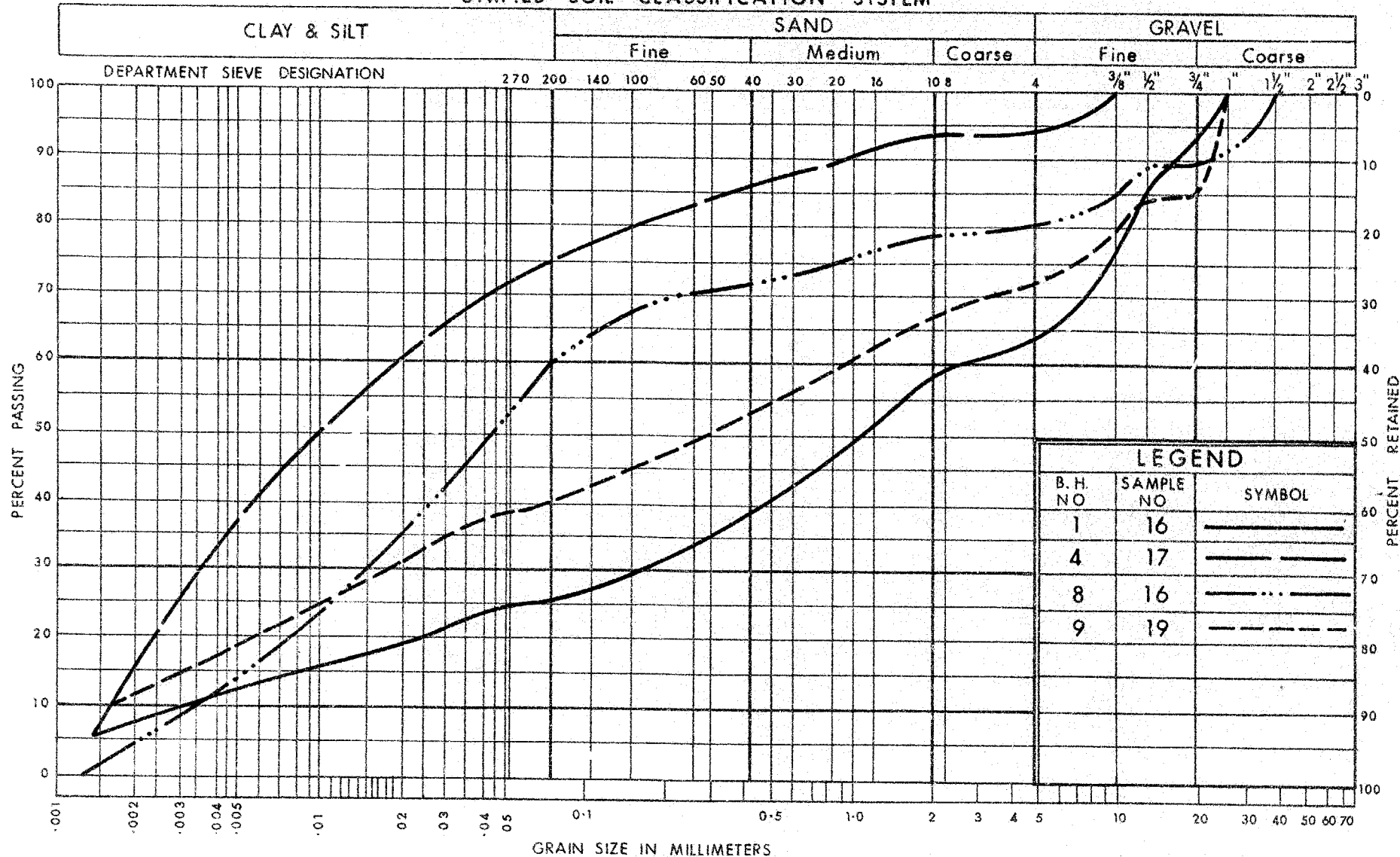


DEPARTMENT OF HIGHWAYS
MATERIALS and
TESTING
DIVISION

GRAIN SIZE DISTRIBUTION
SENSITIVE CLAY TO SILTY CLAY

W.P. No. 34-66-08
JOB No. 67-F-114
FIG. NO. 5

UNIFIED SOIL CLASSIFICATION SYSTEM



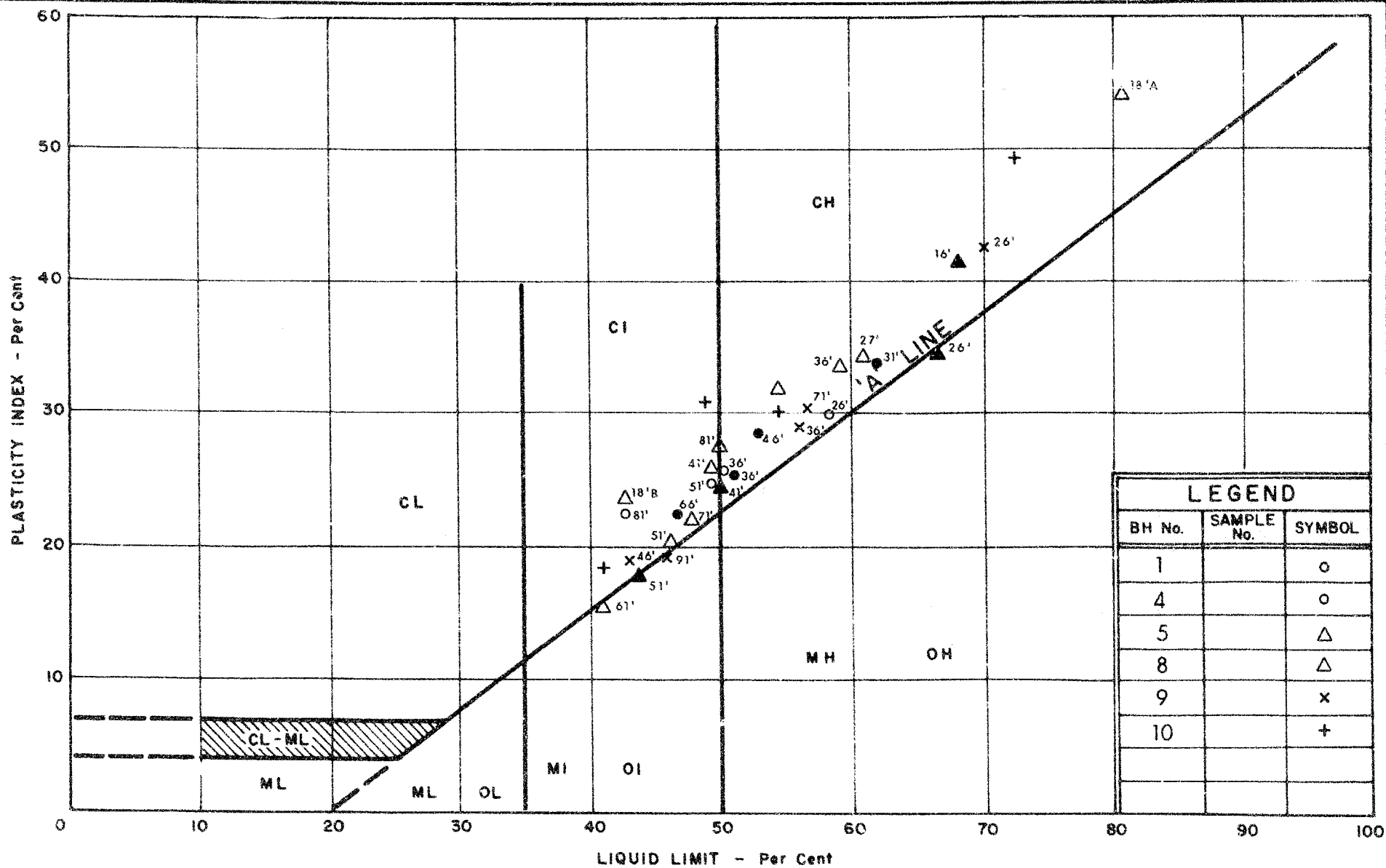
DEPARTMENT OF HIGHWAYS
MATERIALS and
TESTING
DIVISION

GRAIN SIZE DISTRIBUTION
CLAYEY SILT OR SILT, SOME SAND & GRAVEL
(Glacial Till)

W.P. No. 34-66-08

JOB No. 67-F-114

FIG. NO. 6

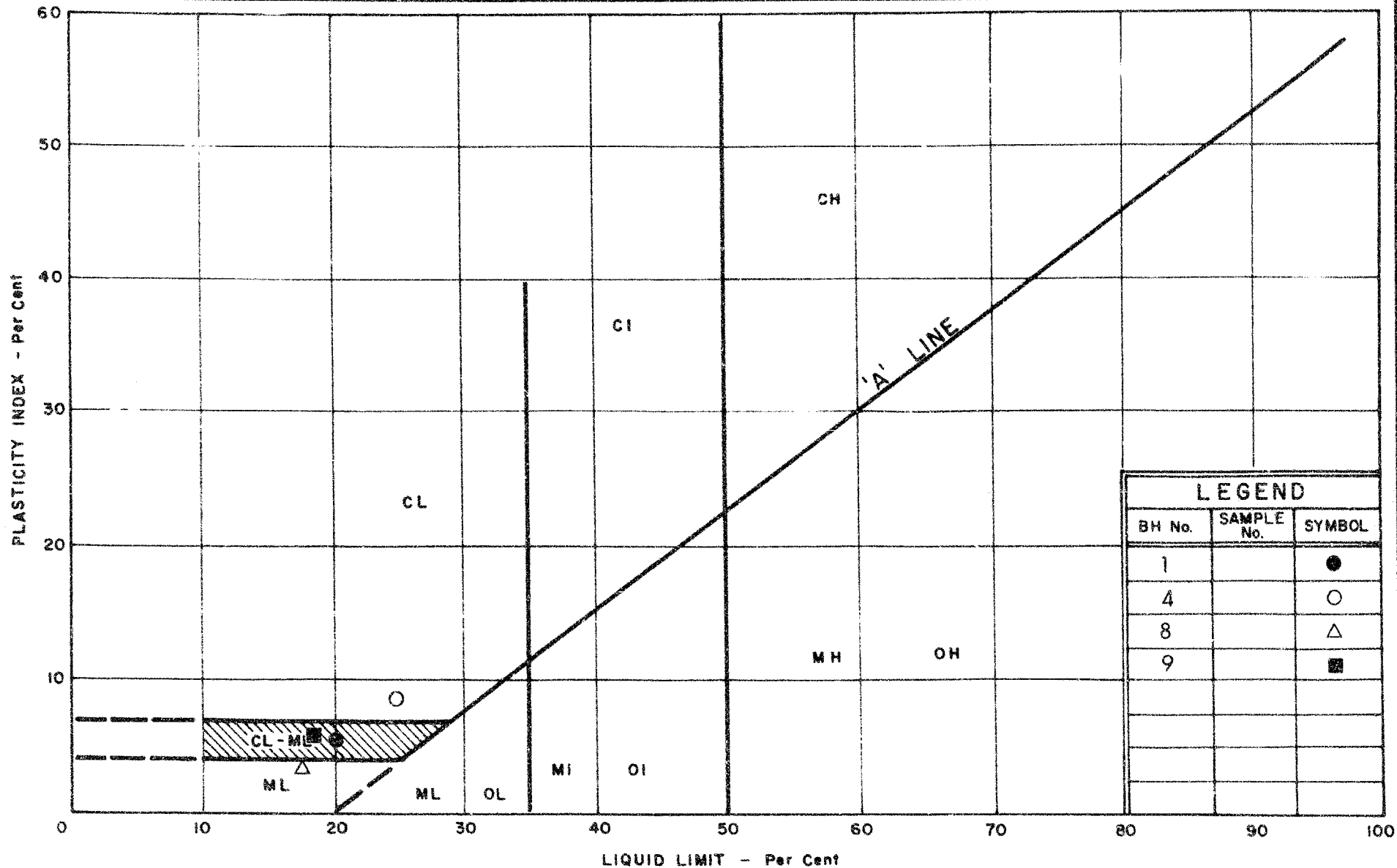


PLASTICITY CHART

SENSITIVE CLAY TO SILTY CLAY

JOB No. 67-F-114

FIG. NO. 7



DEPARTMENT OF HIGHWAYS
MATERIALS and
TESTING
DIVISION

PLASTICITY CHART CLAYEY SILT OR SILT SOME SAND & GRAVEL (GLACIAL TILL)

WP No. 34-66-08
JOB No. 67-F-114
FIG. NO. 8

VOID RATIO vs PRESSURE

$W_L = 54.5$

$W_p = 22.6$

$W = 58\%$

$C_c = 2.02$

BORE HOLE 5

SAMPLE 7

DEPTH 23'-8"

ELEV. 236.0

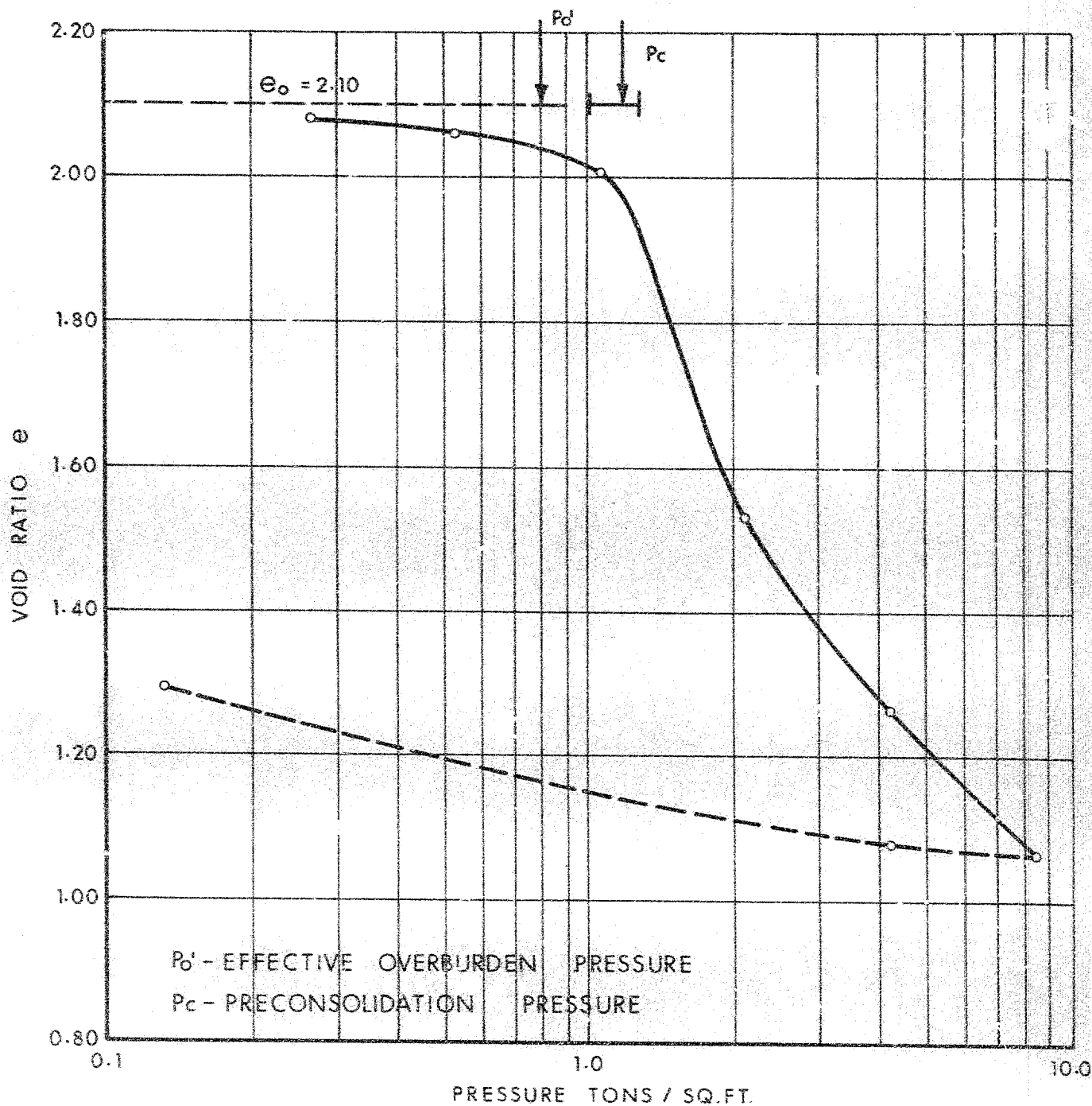


FIG. 9

VOID RATIO vs PRESSURE

$W_L = 59.0$
 $W_p = 25.6$
 $W = 71.2\%$
 $C_c = 1.41$

BORE HOLE 5
 SAMPLE 10
 DEPTH 36'-4"
 ELEV. 224.5

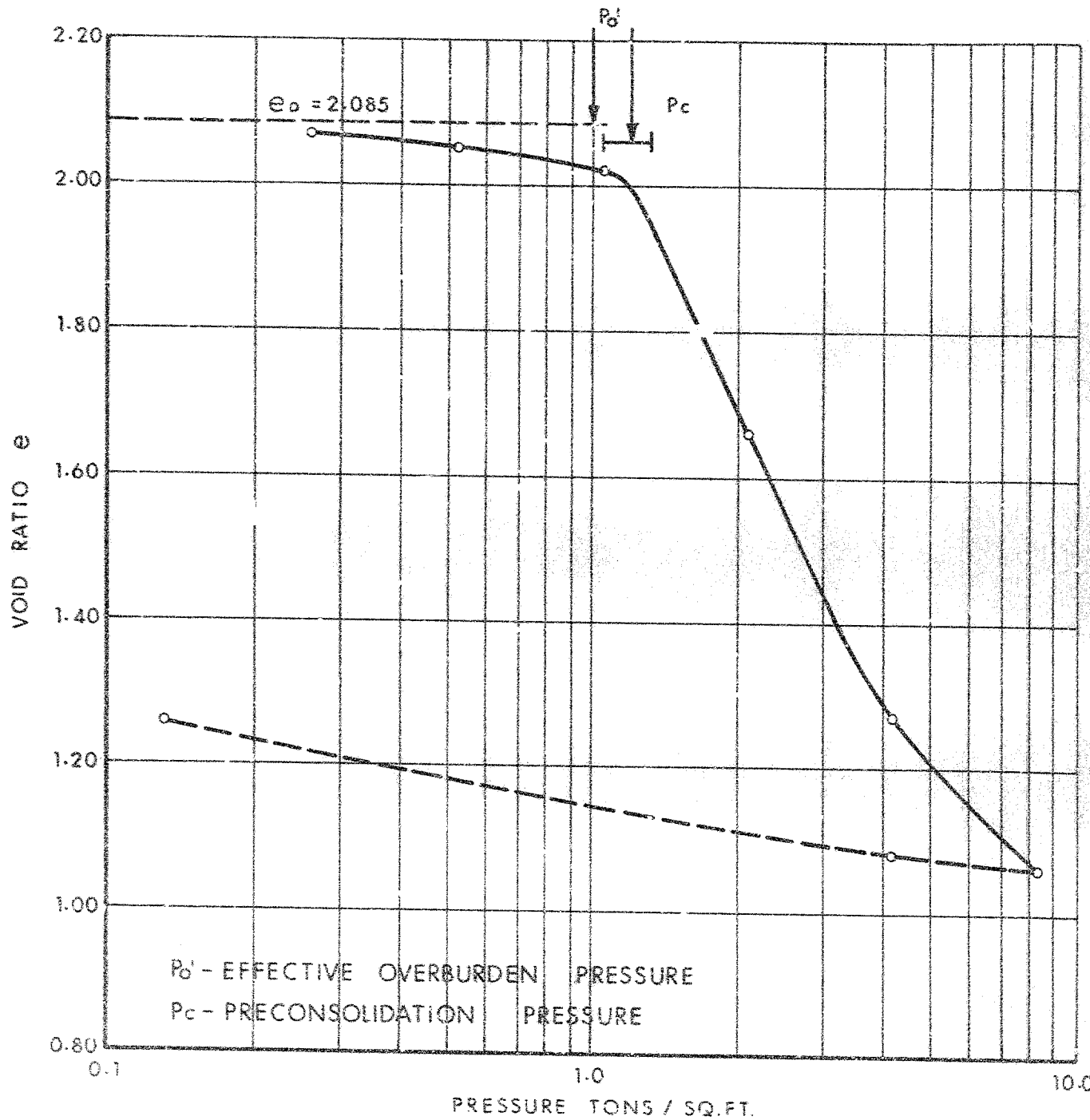


FIG. 10

VOID RATIO vs PRESSURE

$W_L = 46.1$
 $W_p = 25.7$
 $W = 78.4\%$
 $C_c = 1.67$

BORE HOLE 5
 SAMPLE 13
 DEPTH 51'-4"
 ELEV. 209.5

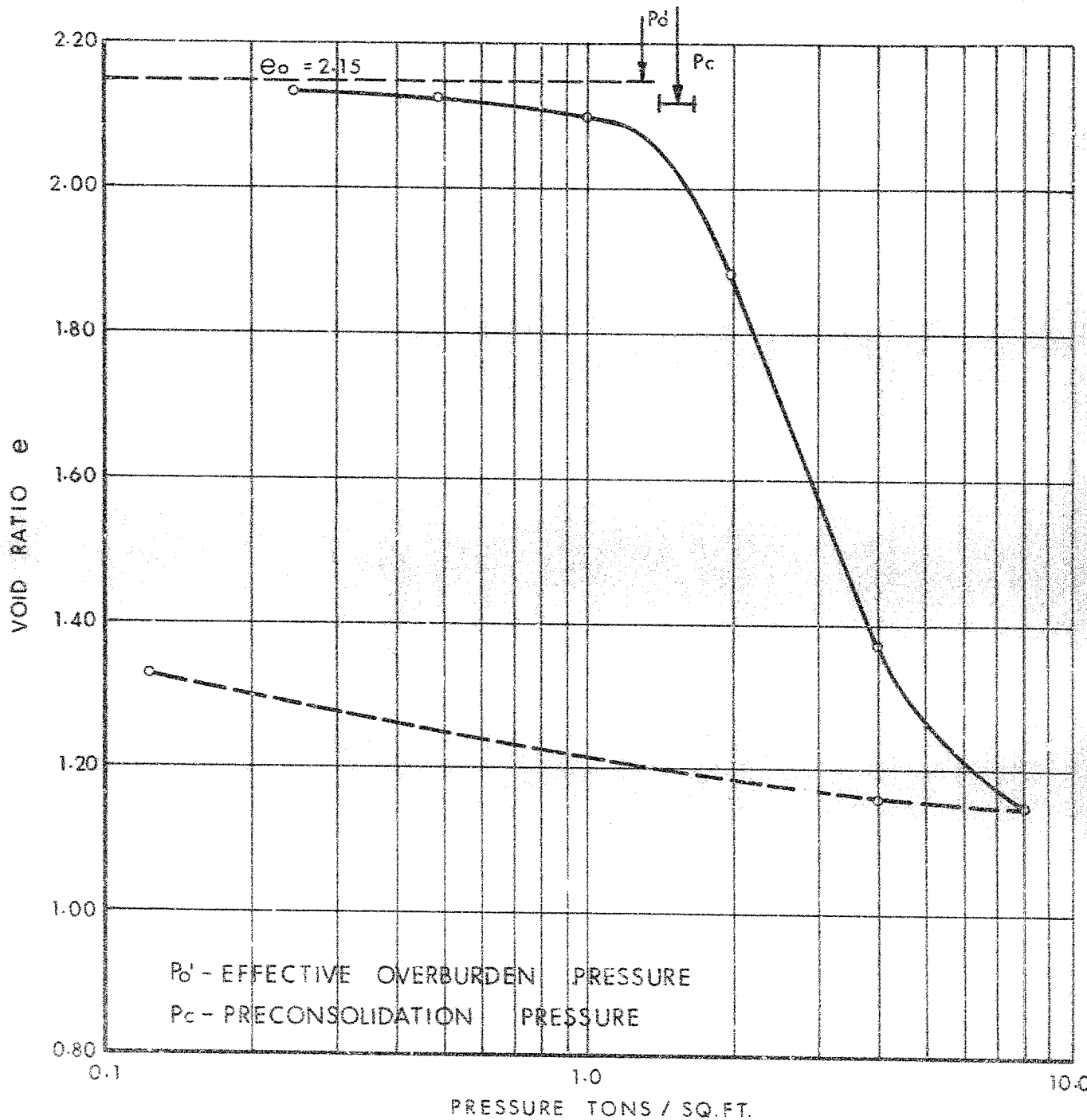


FIG. 11

VOID RATIO vs PRESSURE

$W_L = 66.5$
 $W_p = 31.8$
 $W = 83.1\%$
 $C_c = 1.89$

BORE HOLE 8
 SAMPLE 7
 DEPTH 26'-4"
 ELEV. 234.0

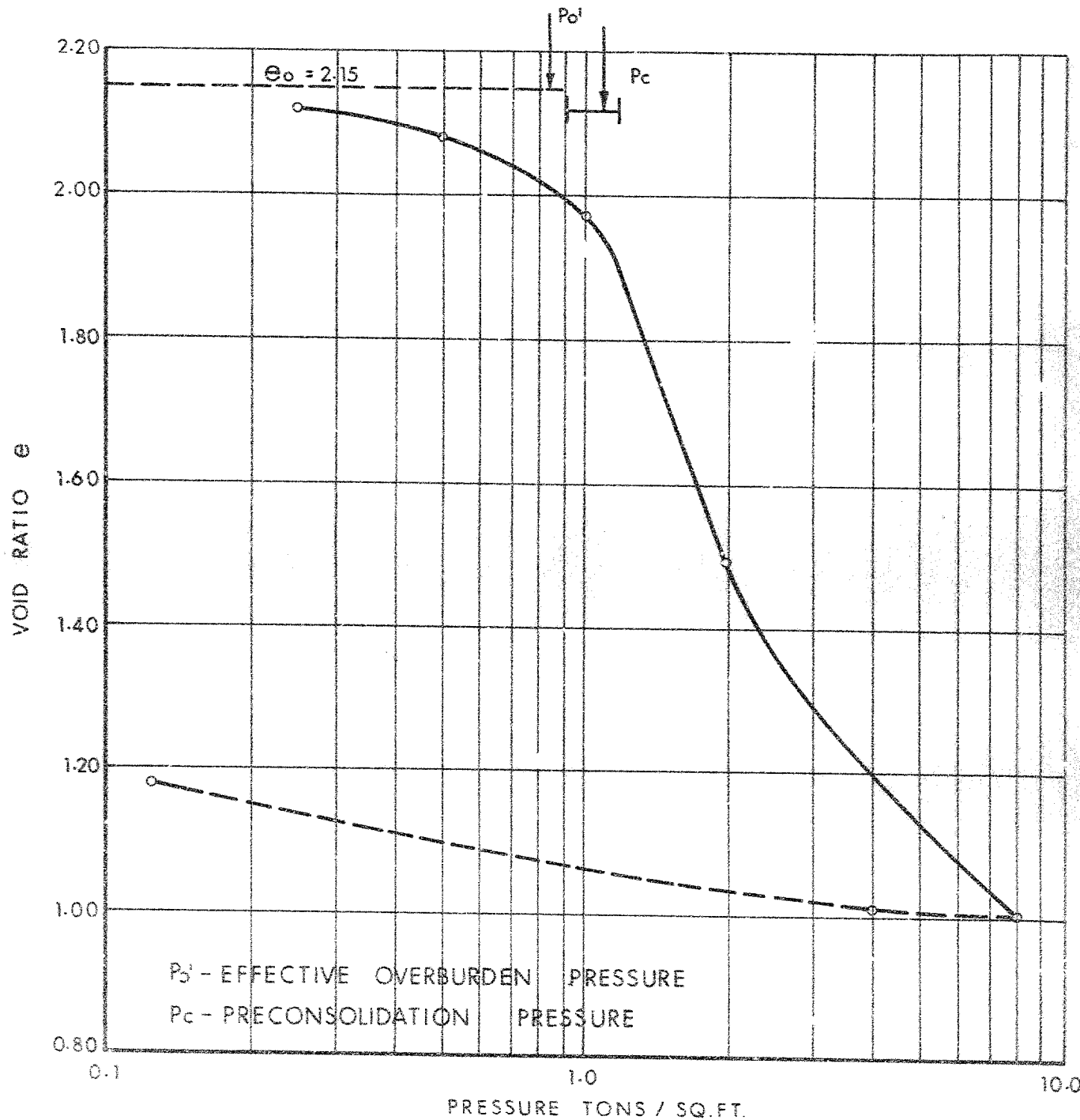


FIG. 12

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

Qu	UNCONFINED COMPRESSION	L.V.	LABORATORY VANE
Q	UNDRAINED TRIAXIAL	F.V.	FIELD VANE
Qcu	CONSOLIDATED UNDRAINED TRIAXIAL	C	CONSOLIDATION
Qd	DRAINED TRIAXIAL	S	SENSITIVITY

ABBREVIATIONS USED IN THIS REPORT

SOIL PROPERTIES

γ	UNIT WEIGHT OF SOIL (BULK DENSITY)
γ_s	UNIT WEIGHT OF SOLID PARTICLES
γ_w	UNIT WEIGHT OF WATER
γ_d	UNIT DRY WEIGHT OF SOIL (DRY DENSITY)
γ'	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
τ_f	SHEAR STRENGTH
c'	EFFECTIVE COHESION INTERCEPT
ϕ'	EFFECTIVE ANGLE OF SHEARING RESISTANCE, OR FRICTION
c_u	APPARENT COHESION
ϕ_u	APPARENT ANGLE OF SHEARING RESISTANCE, OR FRICTION
μ	COEFFICIENT OF FRICTION
S_i	SENSITIVITY

GENERAL

π	= 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
σ	NORMAL STRESS
σ'	NORMAL EFFECTIVE STRESS ($\bar{\sigma}$ IS ALSO USED)
τ	SHEAR STRESS
ϵ	LINEAR STRAIN
γ	SHEAR STRAIN
ν	POISSON'S RATIO (μ IS ALSO USED)
E	MODULUS OF LINEAR DEFORMATION (YOUNG'S MODULUS)
G	MODULUS OF SHEAR DEFORMATION
K	MODULUS OF COMPRESSIBILITY
η	COEFFICIENT OF VISCOSITY

EARTH PRESSURE

d	DISTANCE FROM TOP OF WALL TO POINT OF APPLICATION OF PRESSURE
δ	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
β	ANGLE OF SLOPE TO HORIZONTAL

MEMORANDUM

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

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

ATTENTION:

DATE: November 5, 1968

FILE REF:

IN REPLY TO

SUBJECT:

8th Line Road Underpass - Site 3-269
W.P. 34-66-08 -- W.J. 67-F-114
Hwy. #417 -- District No. 9 (Ottawa)

We have reviewed the Preliminary Drawing D-6480-P for the above mentioned project and submit the following comments:

It is understood that the structure approaches only, are contemplated at this stage and, therefore, we are not making any comments pertaining to structure foundations. Regarding the details of the approach embankments including the surcharge, it appears that the designer has complied with our verbal recommendations. These recommendations have been made based upon the following information:

Stability:

Limiting height of fill above original ground surface.	12 ft.
Minimum length of bench in the longitudinal direction at ground surface.	35 ft.
Depth of cut below original ground surface.	6 ft.
Factor of safety against base failure.	<u>1.33</u>
Factor of safety with a 4-ft. surcharge for the above case.	<u>1.04</u>

Settlements:

Height of Fill	Total Settlement in Inches for Various Periods				
	2 Yrs.	7 Yrs.	15 Yrs.	25 Yrs.	50 Yrs.
12 ft. (Design height)	6"	9"	14"	18"	26"
16 ft. (Design height + 4-ft. surcharge)	9"	12"	21"	27"	38"
(Percentage Consolidation) -	18%	27%	45%	58%	85%

cont'd. /2 ...

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

November 5, 1968

The Foundation Section would like to carry out settlement observations at this site and, therefore, a special note should be made on the Contract Documents to this effect, so that the District can advise this Section for the necessary installations prior to the commencement of the grading work.

MD/MdeF

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

cc: Messrs. S. McCombie
G. Scott
J. L. Forster
S. Markiewicz
C. R. Robertson
K. Westerby
Foundations Files
Gen. Files ✓

E-4662-1 EBL.
E-4663-1 WBL.
STRWP 34-66-05

WT 67-F-117
HWY 417 AND
BEAR BROOK

WBL 1'45"

ANISSE DAVID
INST. NO 775608
E-4649-1 WBL.
E-4654-1 EBL.
STRWP 34-66-08

EBL 1'30"

Copy information of

G. STERMAC

**Regional Engineer,
Highway Building,
KINGSTON, Ontario.**

**Bridge Division,
Kingston, Ontario.**

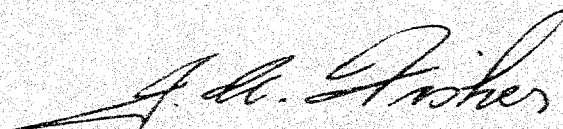
October 8, 1957.

3-269

**W.P. 34-35-000, Site 3-200, 3th Line Road,
E.B.L. & W.B.L. Structures, Proposed
Highway 417, District 9**

We are sending to you herewith 2 copies of Bridge Site Plan E-4554-1 for E.B.L. structure and E-4549-1 for W.B.L. structure together with Preliminary Structure Site Report. The proposed locations for the subject structures are marked in red on the plans.

We will be pleased to have you arrange for foundation investigation of this site and to receive your report in due course.


J. A. Fisher

For: **Gavin Scott, P. Eng.**
Regional Bridge Location Engineer

JAF/GS/N

Encls.

Original letter & 1 copy of Plans E-4554-1, E-4549-1 to:
Bridge Office Files Section - (Mr. S. McCombie)

c.c. & 2 copies of Plans with Site Report to:
Mr. A. G. Stermac

SUPER IMPOSED DOCUMENT MAY
APPEAR AS MULTI-FEED ON FILM.

67-F-114



401 & Keele Street
Downsview, Ontario



DEPARTMENT OF HIGHWAYS

December 5, 1967

Johnston Drilling Co. Ltd.
P.O. Box 4134
Postal Station 'E'
Ottawa 1, Ontario

Dear Sirs:

This is to confirm our request of November 30, 1967 for the supply of 2 Diamond Drills & 2 Penndrills together with all necessary equipment, as specified under the terms of our Contract Agreement, at Bearbrook and new Hwy 417, near Ottawa, Ontario.

This project bears Job Numbers 67-F-111; 67-F-112; 67-F-113 & 67-F-114.

Yours truly,

MD:mt

M. Devata

C. Consulting Foundation Engineer
for A. G. Stelmec
Principal Foundation Engineer

cc: H. Karpis
Foundation Files (2) //0
General File

DEFECTS IN NEGATIVE DUE TO
CONDITION OF ORIGINAL DOCUMENT

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: April 11, 1968

Our File Ref.

REPLY TO

SUBJECT:

Eastbound Lane and Westbound Lane
Underpass Structures
At the Crossing of Eighth Line Road
And Proposed Hwy. #417
Twp. of Gloucester, Co. of Carleton
District No. 9 (Ottawa)
W.J. 67-F-114 -- W.P. 34-66-08

On March 29, 1968, we have sent you the Foundation Investigation report for the above mentioned structure. In this report we have presented the predicted embankment settlements which will occur in the course of time. It should be mentioned that the settlements could be even larger due to the need to constantly add new fill in order to maintain the grade.

Settlements of such magnitude also present serious problems for the bridge foundations - especially for the abutments.

In view of the cost of the rather long structure and, also, because of the serious maintenance problems due to excessively large settlements, we would ask you to consider the following alternative solution.

The structure should be changed from underpass to overpass, and the Eighth Line Road should be placed in a cut at the crossing. Either a rigid box culvert or a multi-plate arch culvert should be used as structure.

We feel that such a solution is practical, and the minor problems that would arise, can easily be taken care of.

Should you wish to consider any aspect of this proposal, please feel free to contact this Section.

AGS/MdeF

cc: Messrs. B. R. Davis (2)
H. A. Tregaskes
D. W. Parren
S. J. Markiewicz
C. R. Robertson
G. Scott
J. E. Gruspier
B. A. Singh
Foundations Files ✓
Gen. Files

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

MEMORANDUM

To: Mr. J. L. Forster, P. Eng.,
Regional Functional Planning Engineer,
Kingston, Ontario.

FROM: Bridge Division,
Kingston, Ontario.

DATE: April 19, 1968.

OUR FILE REF.

IN REPLY TO

SUBJECT:

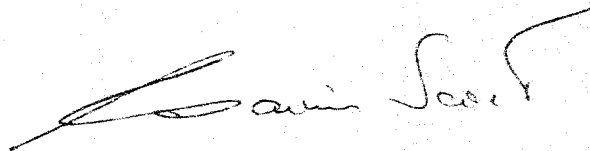
W.P. 34-66-08, Site 3-269, Eighth Line Road
Structure (3.1 Miles East of Carleton Cty. Rd. 27)
Highway 417, District 9

67-F-114

Attached herewith please find copy of letter dated April 11, 1968, which indicates that serious settlements of the approach fills will occur.

It is suggested that the structure might be changed to an overpass, with the Eighth Line Road in a cut at the crossing. Settlements of the order of 7 ft. and more are predicted for this structure and also at Bear Brook structure, and it is reasonable to suppose that conditions at the adjacent structures will be similar.

It would seem advisable that the grade for Highway 417 should be reconsidered.



Gavin Scott, P. Eng.
Regional Bridge Location Engineer

GS/hl

Encl.

c.c. (with encl.)

Mr. S. McCombie - Bridge Office Files Section

Mr. W. Lin

→ Mr. A. G. Stermac

Mr. S. J. Markiewicz

Mr. C. R. Robertson

Mr. J. E. Graspier

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: April 11, 1968

OUR FILE REF.

IN REPLY TO

SUBJECT:

Eastbound Lane and Westbound Lane
Underpass Structures
At the Crossing of Eighth Line Road
And Proposed Hwy. #417
Twp. of Gloucester, Co. of Carleton
District No. 9 (Ottawa)
W.J. 67-F-114 -- W.P. 34-66-08

On March 29, 1968, we have sent you the Foundation Investigation report for the above mentioned structure. In this report we have presented the predicted embankment settlements which will occur in the course of time. It should be mentioned that the settlements could be even larger due to the need to constantly add new fill in order to maintain the grade.

Settlements of such magnitude also present serious problems for the bridge foundations - especially for the abutments.

In view of the cost of the rather long structure and, also, because of the serious maintenance problems due to excessively large settlements, we would ask you to consider the following alternative solution.

The structure should be changed from underpass to overpass, and the Eighth Line Road should be placed in a cut at the crossing. Either a rigid box culvert or a multi-plate arch culvert should be used as structure.

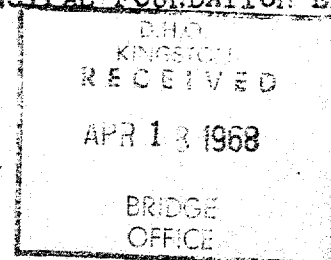
We feel that such a solution is practical, and the minor problems that would arise, can easily be taken care of.

Should you wish to consider any aspect of this proposal, please feel free to contact this Section.

AGS/MdeP

cc: Messrs. B. R. Davis (2)
H. A. Tregaskes
D. W. Farren
S. J. Markiewicz
C. R. Robertson
G. Scott
J. E. Gruspier
B. A. Singh
Foundations Files
Gen. Files

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



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

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

Attention: Mr. J. McCombie

June 28, 1968

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

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

AGS/ndef

A. G. Sternac
A. G. Sternac
PRINCIPAL FOUNDATION ENGINEER

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

Foundations File
Gen. Files

DEFECTS IN NEGATIVE DUE TO
CONDITION OF ORIGINAL DOCUMENT

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.

cont'd. /2 ...

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. B. 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

CONTRACT 67-F-114

JOB NO. 67-F-114

W. P. NO. 34-66-08

PIEZOMETER DATA	
PIEZOMETER NO.	<u>4</u> <i>SHALLOW</i>
LOCATION	<u>Sta 34+94, o/s 18' RT. - WBL</u>
ORIGINAL GROUND ELEVATION	<u>261.5</u>
TIP ELEVATION	<u>224.5</u>
FUTURE FILL HEIGHT	<u>176.5</u>
DATE INSTALLED	<u>Jan 30, 1968</u>

[illegible]

CONTRACT 67-F-114

W. P. NO. _____

PIEZOMETER DATA	
PIEZOMETER NO.	<u>4</u> DEEP (PRINTED RED)
LOCATION	<u>Sta 3A+94, 0/5 18' RT. - W.B.L.</u>
ORIGINAL GROUND ELEVATION	<u>261.5</u>
TIP ELEVATION	<u>176.5</u>
FUTURE FILL HEIGHT	
DATE INSTALLED	<u>Jan 30/68</u>

[illegible]

CONTRACT 67-E-114

JOB NO. 67-F-114

W. P. NO. _____

PIEZOMETER NO. 5 *DEED*

LOCATION Sta. 29+06, 0/5 18' RT. — E.B.L.

ORIGINAL GROUND ELEVATION 260.7

TIP ELEVATION 160.7

FUTURE FILL HEIGHT

DATE INSTALLED December 11/68.

[illegible]

CONTRACT 67-F-1K

W. P. NO. 34-66-08

PIEZOMETER NO. 4 SHALLOW

LOCATION _____

ORIGINAL GROUND ELEVATION

TIP ELEVATION _____

FUTURE FILL HEIGHT _____

DATE INSTALLED _____

[illegible]

CONTRACT 67-F-114

W. P. NO. _____

DATE INSTALLED _____

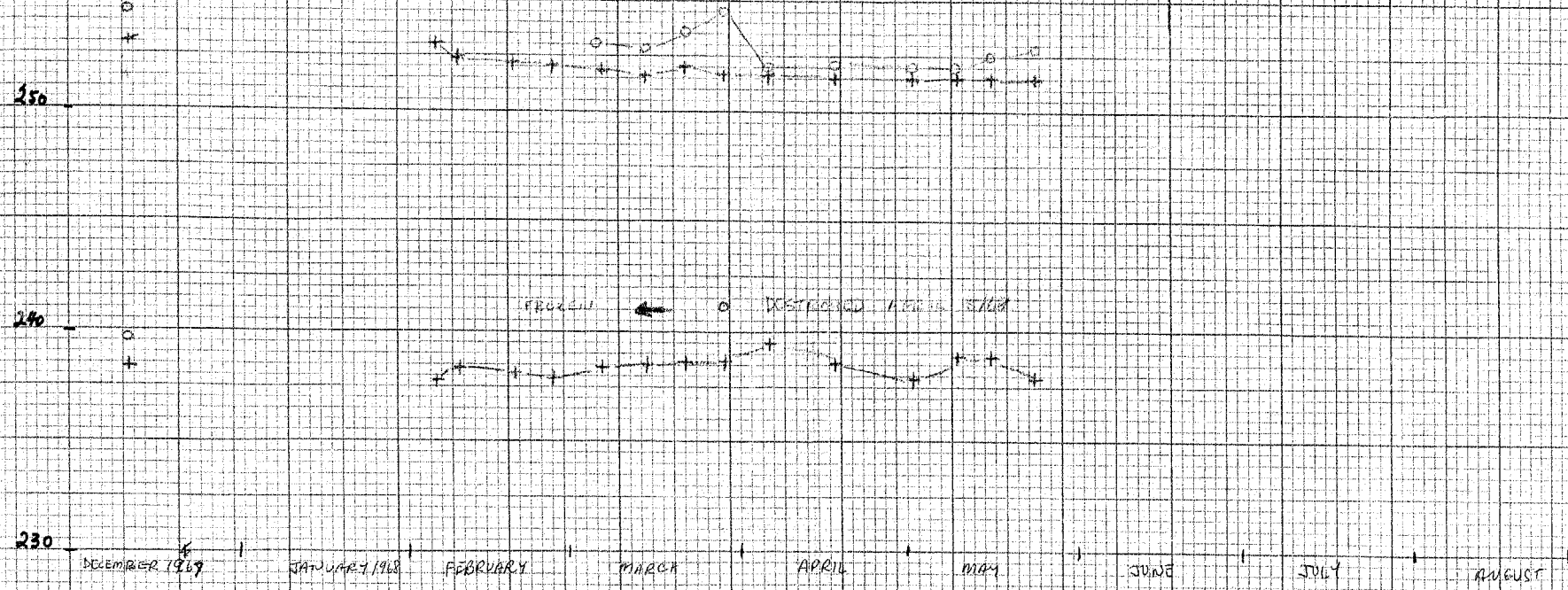
[illegible]

67-F-117. W.P. 34-66-08
 HWY. #417 CROSSING OF EIGHTH LND
 RECORD OF PIEZOMETRIC GROUND
 WATER LEVELS IN CLAY & GLACIAL
 TILL.

PIEZOMETER LEGEND

	PI#	LOCATION	GROUND SURFACE (ELEV.)	PIEZ. TIP (ELEV.)	PIEZ. LOCATED IN
+--+	4	STA. 34+44.518171 W.B.L.	261.5	224.5	CLAY.
+--+	4	"	"	176.5	GLACIAL TILL
○--○	5	STA. 39+00.0000 E.B.L.	260.7	210.7	SENSITIVE CLAY
○--○	5	"	"	2160.7	CLAYAL TILL & BEDROCK.

Piezometric Ground Water Level. (ft) →



Department of Highways Ontario

Copy for the information of
Mr. A. Stermac

67-F-114
file with the Report.

Re: -

G. Scott, Kingston Region
C.B. Robertson, Ottawa District
L. Forster, Kingston Region
S. Markiewicz, Kingston Region

Bridge Division,
Downsview, Ontario

October 30, 1968

8th Line Road Underpass
W.P. 34-66-02, Site 3-269
Highway 417, District No. 9

Attached herewith are prints of the Preliminary Bridge
Plan Drawing D-6480-F for the above-mentioned structure.

The estimated cost of the proposed structure is \$330,000.
This cost includes tender, materials, engineering and sundry
construction.

Any comments or revisions you may have should be submitted
within three weeks.

CSB:rd

C.B. Grebaki,
Bridge Design Engineer

Attach.

c.c. S. McCombie
A. Stermac (2)
D. Barr
J. Anderson

SUMMARIZED RESULTS OF STABILITY AND SETTLEMENT ANALYSES

HWY. 417 & 8th LINE RD
UNDERPASS STRUCTURES
CROSSING
GLOUCESTER TOWNSHIP

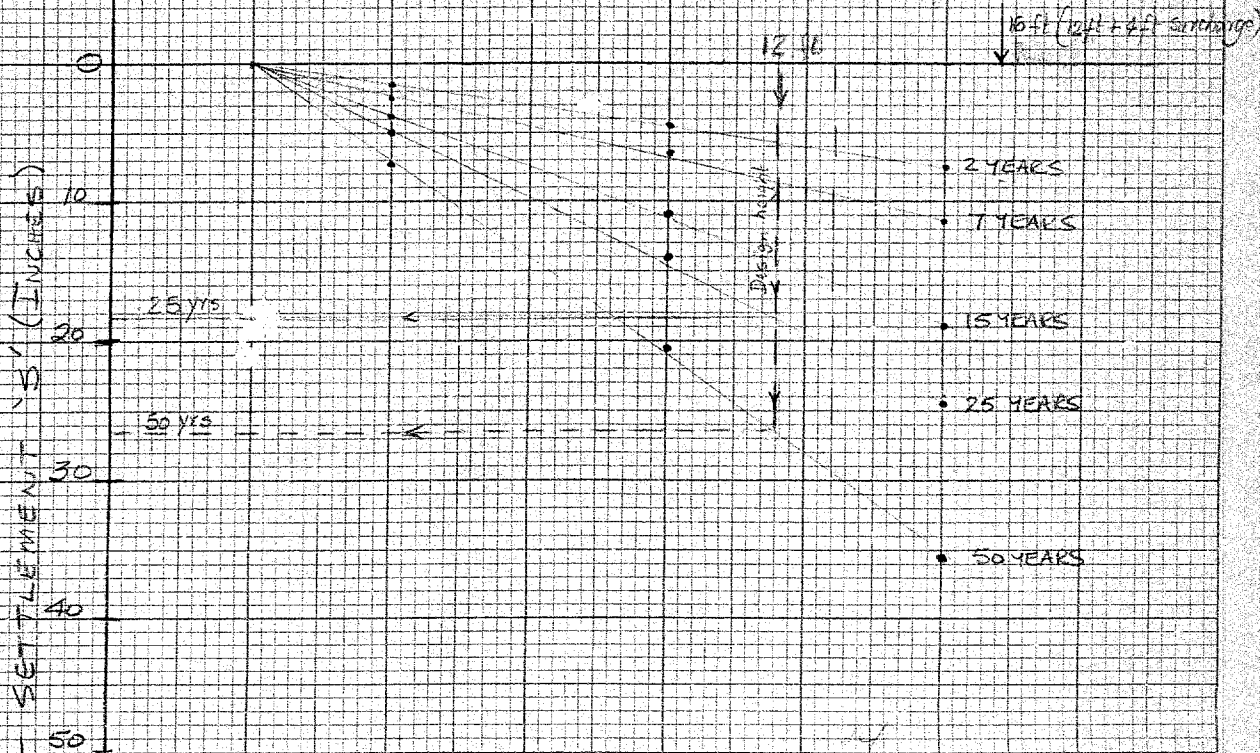
FACTOR OF SAFETY (WITH RESPECT TO
SPALLING)

1.5
1.4
1.3
1.2
1.1

HEIGHT OF FILL 'H' (FT.)

SETTLEMENT 'S' (INCHES)

0
10
20
30
40
50



af

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

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

July 4, 1969

8th Line Rd. Underpass --
Highway #417, District #9 (Ottawa)
W.J. 67-F-114 -- W.P. 34-66-08

We have reviewed the final bridge design drawings No. D-6480-1 to 4, inclusive, pertaining to the above structure; the following comment is submitted:

The end-bearing steel H-piles supporting the abutments, will be subjected to some negative skin frictional forces due to settlement of the surrounding subsoil, caused by the surcharge loading of the approach embankments. In view of this, it is recommended that the allowable pile capacities at the abutment locations, be reduced from 90 to 70 tons per pile. This provision was recommended at other structure locations in this area - (refer to our memo of May 6, 1969).

ND/MdeF

cc: Messrs. S. McCombie
T. C. Kingsland
Foundations Files
Gen. Files

M. Devata

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

MEMORANDUM

To: Mr. M. Devata,
Supervising Foundation Engineer,
Lab. Bldg.,
Downsview.

ATTENTION:

FROM: E. B. Fenner,
Field Surveys Superintendent,
Engineering Surveys Office,
Rexdale.

DATE: July 22nd, 1969.

OUR FILE REF:

IN REPLY TO

SUBJECT:

Foundation Test Pipes on Proposed Hwy. 417
Twp. Gloucester

Following is list of precise elevations obtained on your
test pipes:

Pipe at Anderson Rd. and Prop. Hwy.#417	Elev. 257.899
Pipe at 7th Line and Prop. Hwy.#417	Elev. 262.285
✓ Pipe at 8th Line and Prop. Hwy. #417	Elev. 258.184

These were established from our precise levels in the
area and confirmed.

E. B. Fenner
E. B. Fenner
Field Surveys Superintendent

EBF:WEG.

MEMORANDUM

To: Mr. M. Devata,
Supervising Foundation Engineer,
Lab. Bldg.,
Downsview.

ATTENTION

OUR FILE REF:

FROM: E. B. Fenner,
Field Surveys Superintendent,
Engineering Surveys Office,
Rexdale.

DATE: July 22nd, 1969.

IN REPLY TO

SUBJECT:

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Twp. Gloucester

Following is list of precise elevations obtained on your
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7-F-114 Pipe at <u>8th Line</u> and Prop. Hwy. #417	Elev. 258.184

These were established from our precise levels in the
area and confirmed.

E. B. Fenner
E. B. Fenner
Field Surveys Superintendent

EBF:WEG.

MEMORANDUM

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

FROM: C.S. Grebski,
Bridge Office

ATTENTION:

DATE: June 23, 1969

OUR FILE REF.

IN REPLY TO

SUBJECT: 8th Line Road Underpass
W.P. 34-66-08, Site 3-269
Highway 417, District No. 9

67-F-114

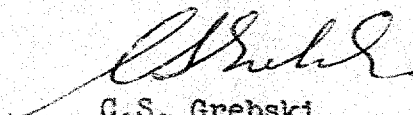
Attached herewith we are submitting the final bridge drawings which show the foundation design for this structure.

Kindly give us your comments at your earliest convenience.

CSG:rd

Attach.

c.c. Foundation Section



C.S. Grebski,
Bridge Design Engineer

DEPARTMENT OF HIGHWAYS ONTARIO

MEMORANDUM

To: Mr. J. MacMaster,
Sr. Project Design Engineer,
Road Design Division,
KINGSTON, Ontario.

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

ATTENTION:

DATE: November 26, 1968

OUR FILE REF:

IN REPLY TO

SUBJECT:

W.P. 34-66-08 - 8th Line Road Underpass, ✓ 68-F-1145
W.P. 34-66-07 - 7th Line Road Underpass,
W.F. 34-66-06 - Anderson Road Interchange Structure -
District #9 (Ottawa)

This is to acknowledge receipt of your memo dated Nov. 21, 1968, regarding the above mentioned subject.

In your memo you have posed a number of questions and also made certain statements. Questions require answers, statements require comments, and in the following paragraphs we will attempt, as best as possible, to provide both.

Most of the proposed alignment of the new Hwy. 417 crosses an area which could be considered from the foundation point of view, as possibly one of the most difficult in the Province of Ontario. This is due to the presence of a relatively thick layer of sensitive, soft to firm clay.

Two major problems have presented themselves at the various road and river crossings where approach embankments are required. Firstly, the stability of the approach embankments had to be ensured and, secondly, the amount of settlement had to be kept within certain reasonable limits.

As it turned out at a number of crossings, settlements proved to be the controlling factor and, to keep them within reasonable limits, fill heights had to be reduced. However, even with these reduced fill heights, the predicted settlements were not small by any means. We were therefore urged to study this problem further and suggest means and methods to reduce them.

At various meetings we had on a number of occasions with representatives of the Functional Planning, Road Design, Bridge Design, and Program Divisions, we were advised that there is no reason why the construction of the underpass structures could not be delayed for a number of years. Traffic along the incomplete Hwy. 417 is expected to be very light and level crossings of detour roads could easily be tolerated.

In view of the above, we have suggested surcharging.

cont'd. ... 2

Mr. J. MacMaster,
Sr. Project Design Engineer,
Road Design Division,
KINGSTON, Ontario.

November 26, 1968

At the crossings under consideration, a two-year surcharge period would result in settlements that would otherwise (under the normal fill height) take about 7 to 9 years. It was felt that this is indeed a very desirable aspect, and it was recommended as a design feature.

The very low factor of safety for surcharged fills is justified in our opinion because:

- a) the surcharge is only a temporary feature, and
- b) if a failure does occur, it would not affect any structure, nor would it greatly affect construction.

The lack of recommendations for the legs of the Anderson R² interchange is due to the fact that we were not given the design details of the interchange.

When considering settlements, their amount and rate, it should be borne in mind that this is an area where accuracy assumes a somewhat different meaning. We have outlined the limitations of our forecasts on a number of occasions and were left with the impression that this point was realized and appreciated.

In view of this, we have pointed out that it may be warranted and even desirable to change the presently prepared designs in the light of the information which will become available in the course of the next few years from the carefully instrumented and monitored approach fills.

As in all engineering projects, economics of the proposed design has to be considered. Once a technical solution is arrived at, the price of it has to be determined. If alternative solutions are prepared, it becomes a matter of a very careful study to determine the most appropriate, the most convenient and the most acceptable combination of technical excellence and least expenditure.

In the cases under discussion, it is neither simple nor easy to put a price tag on inches of settlement. It becomes a question of philosophy or opinion as to whether smaller or larger settlements can be tolerated. And the less settlements can be tolerated, the higher the acceptable expenditure becomes to reduce or prevent them.

To reach a decision of any kind, all facts and factors influencing it have to be known as accurately as possible. We are fully aware that, for the cases in question, we may not have or know all the pertinent facts, and have therefore not recommended surcharging as a "sine qua non", but rather as a desirable feature. Someone else who knows all the facts will have to make the final decision.

cont'd. ... 3

Mr. J. MacMaster,
Sr. Project Design Engineer,
Road Design Division,
KINGSTON, Ontario.

3.

November 26, 1968

In conclusion, we would like to add that, if only one year is available, surcharging should be dispensed with. On the other hand, if two or more years are available, surcharging should be given serious consideration. In any case, paving should be delayed until such time when settlement readings begin to show a definite trend towards a stable condition.

AGS/MdeF

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

cc: Messrs. D. Farren
S. McCombie
C. E. Robertson
J. L. Forster
C. S. Grebski
J. E. Gruspier
G. Scott

Foundations Files ✓ (3)
Gen. Files

MEMORANDUM

To: Mr. A. Stermac,
Principal Foundation Engineer,
DOWESVIEW, Ontario.

From: Road Design Division,
KINGSTON, Ontario.

ATT: Mr. M. Devata

DATE: November 21, 1968.

Our File Ref.

IN REPLY TO

SUBJECT: W.P. 34-66-08 - 8th Line Road Underpass,
W.P. 34-66-07 - 7th Line Road Underpass,
W.P. 34-66-06 - Anderson Road Interchange Structure -
District #9 Ottawa.

We recently received copies of your letters to Mr. C. S. Grebski commenting on the preliminary bridge drawings for the above project. We note that surcharges are recommended at all three locations and that the surcharges at the 7th and 8th lines will induce 3" more of settlement than the design height of fill if left in place for 2 years. Since the present programming will only allow the surcharge to be in place for approximately one year the additional induced settlement will presumably be about 1½". The cost to surcharge the above approach fills will be about \$16,000.00. In view of the cost therefore and the very limited benefit, we do not feel that surcharges are warranted at the above structure locations.

Also we note that the factor of safety with surcharge is in the area of 1.03 to 1.04 at the 7th and 8th lines and this does not seem to provide a sufficient margin of safety for construction purposes. Perhaps the District would comment on this aspect.

The Anderson road surcharge will require berms to be placed which are otherwise not required by the ~~maximum~~ design height of fill. The cost of the surcharge here, therefore will be \$16,000.00 exclusive of the interchange legs for which there appears to be no surcharge recommendation as yet. The additional induced settlement here will be 3½" and the factor of safety with berms is 1.14. In view of the above it would perhaps be better to defer paving at all three sideroad locations and provide prime and double surface treatment only. Final paving could then be carried out when the fills have reached a reasonable degree of stability based on actual observations.

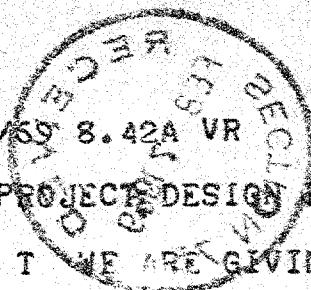
May we have your comments please.


M. J. MacMaster,
SR. PROJECT DESIGN ENGINEER.

MJM/mac

c.c. - D. Farren, S. McCombie, C. R. Robertson, J. L. Forster, C. S. Grebski,
J. E. Graspier and G. Scott

K



0019

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-3.

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

67-F-114

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 STOCKPIILING OF MATERIALS

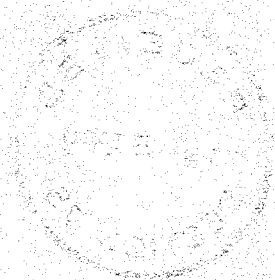
"A SPECIAL SHOULD BE INSERTED IN THE CONTRACT DOCUMENTS TO LIMIT THE
HEIGHT OF STOCKPILED MATERIAL DUE TO THE UNDERLYING WEAK CLAYS ALONG
THE PROJECT.

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	34-66-05, 14	Bear Brook	25'	above hor. Gr. Surface
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	34-66-01	McEwan Creek	20'	away from creek banks
68-F-54	34-66-10, 11	Ramsay Creek	12'	" " "
68-F-57	34-66-14	Bear Brook Tribut.	18'	above Valley floor. Elev of 229 Ht. ^{should} may be reduced if fill is placed on ground above the valley floor.

C.M. Feb. 6/69

SQUARES TO THE INCH

Department of Highways Ontario

Copy for the information of

Mr. A. Stermac

Mr. G. Scott,
Reg. Bridge Location Engineer,
Kingston Regional Office,
Kingston, Ontario

Bridge Division,
Downsview, Ontario

November 29, 1968

67-F-114 W.P. 34-66-06 - Carleton Cty. Rd. 27 Inte. change *Anderson*
W.P. 34-66-07 - 7th Line Rd. Underpass
W.P. 34-66-08 - 8th Line Rd. Underpass
W.P. 34-66-05 - Bear Brook Bridge (E.B.L.)
W.P. 34-66-10 - Ramsay Creek Bridge
W.P. 34-66-14 - Bear Brook Bridge (W.B.L.)
Highway 417, District No. 9

Please find attached copies of revised Preliminary
Drawings for the above-mentioned structures.

Please let me know if additional copies are required.

WL:rd

W. Lin,
Regional Bridge Project Engineer

Encls.

c.c. S. McCombie
A. Stermac (2)
J. Anderson

Review of "Preliminary" Design
Drawings for Hwy 417 Structures.

1 Eighth Line Rd - WP 34-66-08
Dwg. D-6480-P1

- a) Ht of Fill East Approach scales on Dwg to 14-15 ft. This ht. is 2-3' greater than recommended maximum of 12 ft.
- b). Surcharge of 4' should be applied only at crest and forward slope of fill. It is not necessary to provide the surcharge along the side slopes.
with surcharge of 4' $F=1.04$
therefore, very marginal.

2 Ramsay Creek - WP 34-66-10
Dwg D-6575-P1

NO

Comments

3 Anderson Rd.

WP 34-66-06

Dwg D-6484-P1

67F-114

- a) If piles are to be driven to refusal, consider use of steel 'H' piles rather than pipe piles as shown on Dwg.
- b) If pipe piles contemplated in friction, re-compute allowable capacity due to -ve skin friction arising from consolidation of clay beneath approaches and the load bed fills.

4. 7th Line Road

WP 34-66-07

Dwg D-6486-P1

- a) Use steel 'H' pile in preference to pipe pile if load is to be carried down to competent stratum. Reevaluate figure of 75 T/pile for $12\frac{3}{4}$ " ϕ pipe piles in view of -ve skin friction arising from consolidation settlement, if pipe piles used as friction piles.
- b) Surcharge only crest is forward slope of fill. F with surcharge = 1.03, marginal.

67F-114

Bear Brook - EBL

WP 34-66-05

Dwg D-6467-P1

- a) Protect pile cap for Pier #3 with
rip-rap cover on stream bank.

Bear Brook - WBL

WP 34-66-14

Dwg - D-6578-P1

No comments.

DEFECTS IN NEGATIVE DUE TO
CONDITION OF ORIGINAL DOCUMENT

Department of Highways Ontario

Copy for the information of

Mr. A. Stermac

67-114

Mr. G. Scott,
Reg. Bridge Location Engineer,
Kingston Regional Office,
Kingston, Ontario

Bridge Division,
Downsview, Ontario

November 29, 1968

W.P. 34-66-06 - Carleton Cty. Rd. 27 Interchange
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W.P. 34-66-10 - Ramsay Creek Bridge
W.P. 34-66-14 - Bear Brook Bridge (W.B.L.)
Highway 417, District No. 9

Please find attached copies of revised Preliminary
Drawings for the above-mentioned structures.

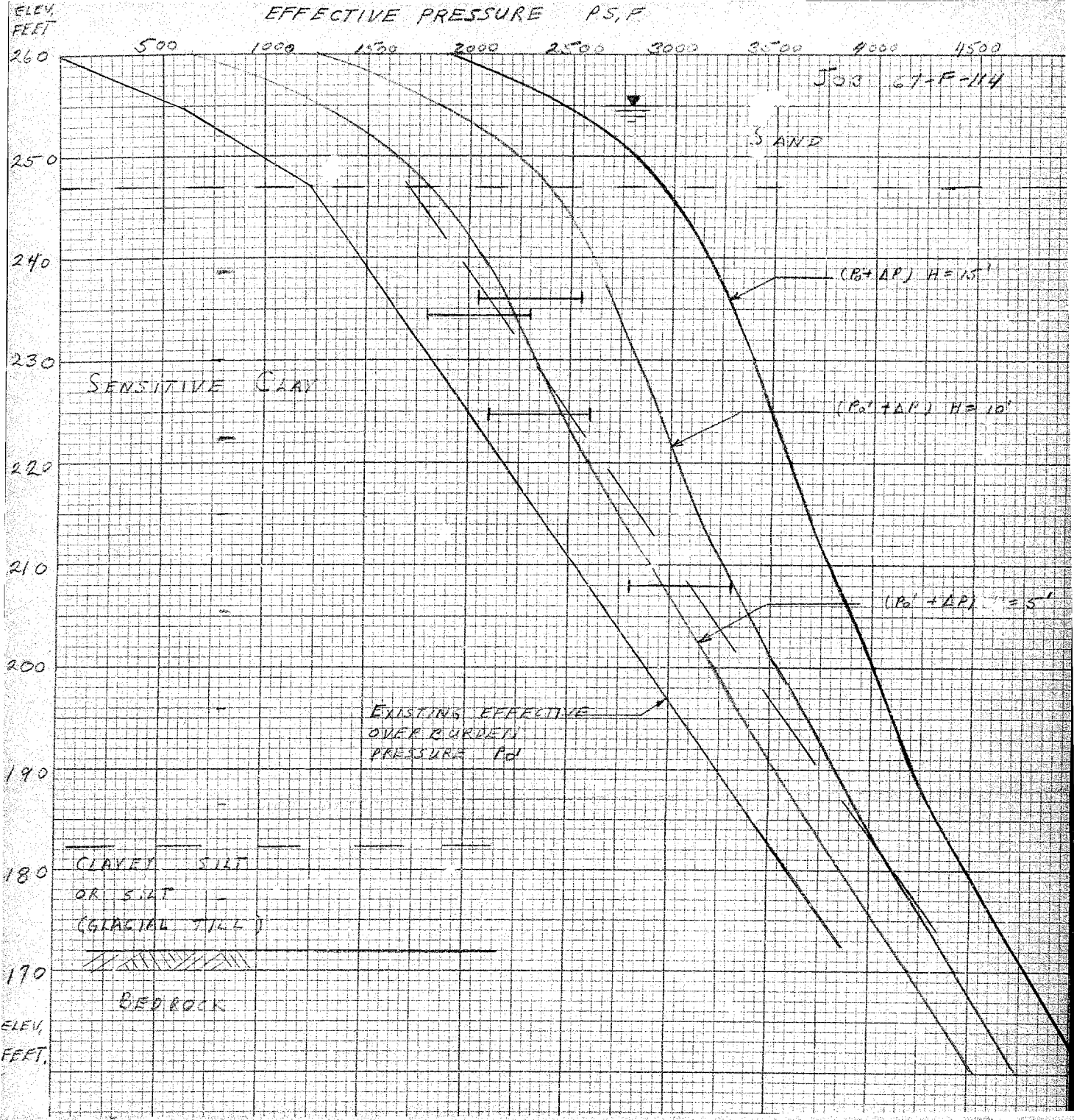
Please let me know if additional copies are required.

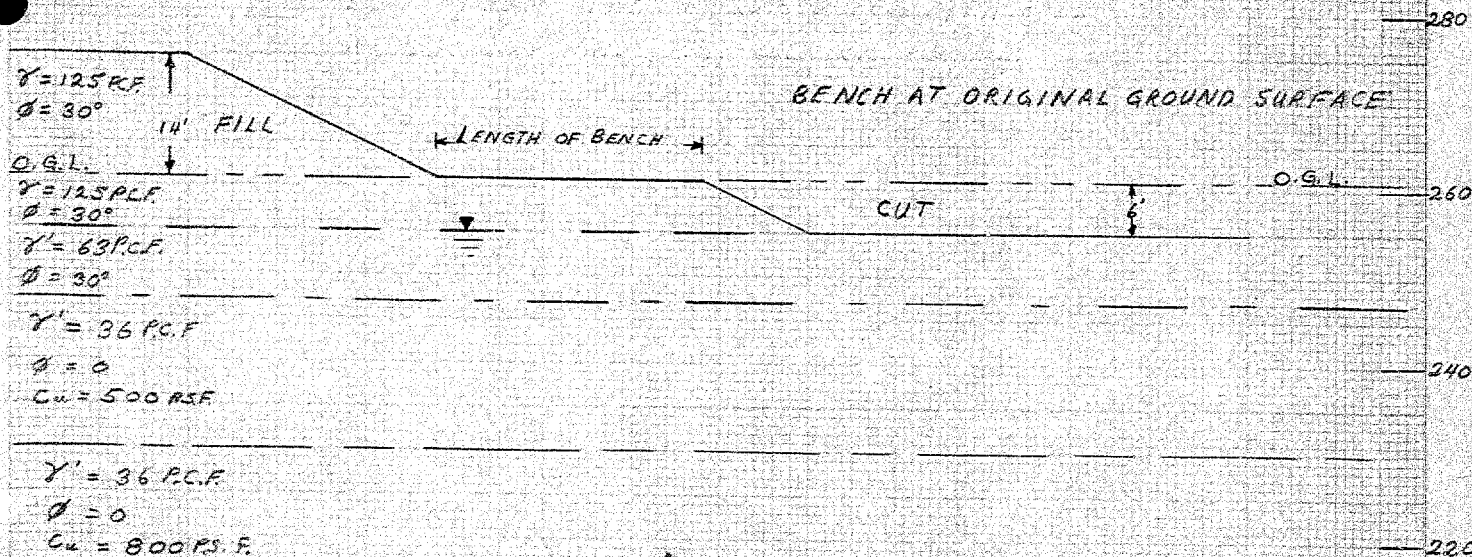
WL:rd

W. Lin,
Regional Bridge Project Engineer

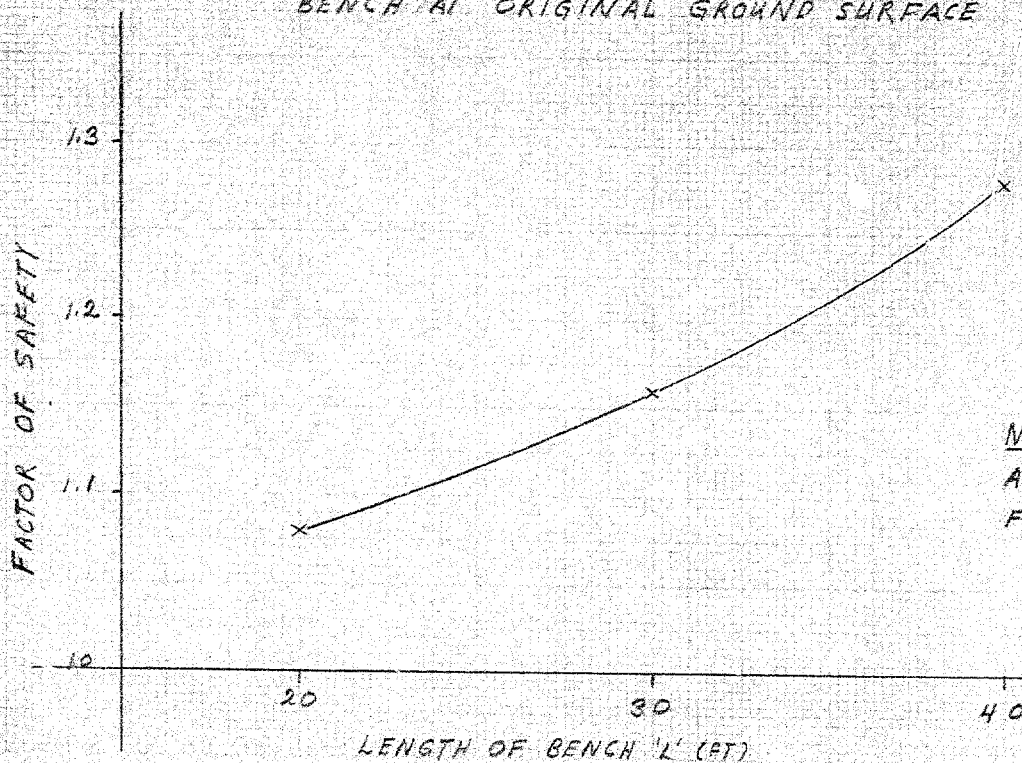
Encls.

c.c. S. McCombie
A. Stermac (2)
J. Anderson





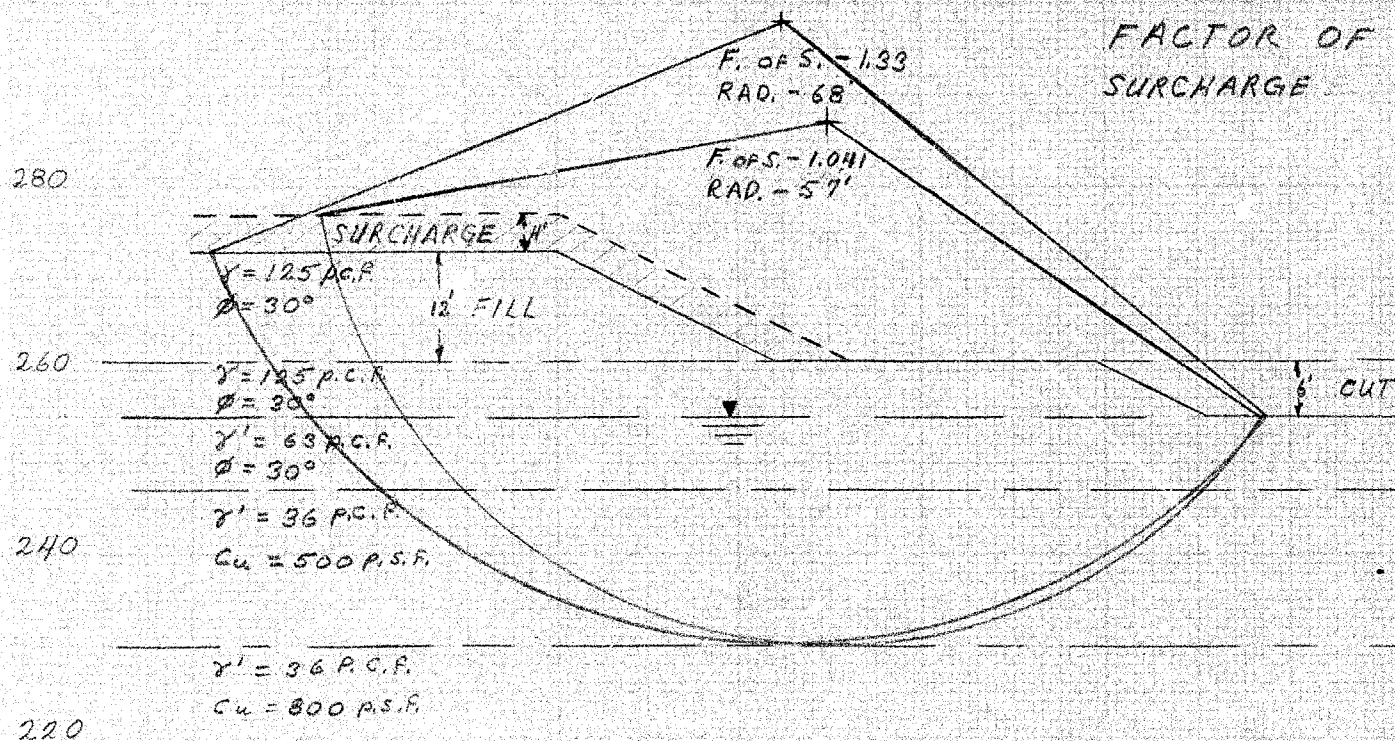
LENGTH OF BENCH REQUIRED FOR 14FT. FILL ABOVE
 GROUND LEVEL AND 6FT CUT BELOW GROUND LEVEL
 BENCH AT ORIGINAL GROUND SURFACE



NOTE: COMPUTER RESULTS
 ARE FILED IN THE
 FOUNDATION FILES.

FACTOR OF SAFETY WITH 12'
FILL ABOVE GROUND LEVEL AND
6' CUT BELOW GROUND LEVEL
IS 1.33

FACTOR OF SAFETY WITH 4'
SURCHARGE 1.041



NOTE: COMPUTER RESULTS ARE
FILED IN THE FOUNDATION FILES.

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

2.

November 5, 1968

The Foundation Section would like to carry out settlement observations at this site and, therefore, a special note should be made on the Contract Documents to this effect, so that the District can advise this Section for the necessary installations prior to the commencement of the grading work.

MD/MdeF

M. Devata

M. Devata,
SUPERVISING FOUNDATION ENGR.

For:

A. G. Stermac,
PRINCIPAL FOUNDATION ENGR.

cc: Messrs. S. McCombie
G. Scott
J. L. Forster
S. Markiewicz
C. R. Robertson
K. Westerby

Foundations Files ✓
Gen. Files

Re. REVIEWED

FINAL BRIDGE

DESIGN COMMENTS
3rd LINE RD.

HIPASS

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

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

July 4, 1969

8th Line Rd. Underpass --
Highway #417, District #9 (Ottawa)
W.J. 67-P-114 -- W.P. 34-66-08

We have reviewed the final bridge design drawings No. D-6480-1 to 4, inclusive, pertaining to the above structure; the following comment is submitted:

The end-bearing steel H-piles supporting the abutments, will be subjected to some negative skin frictional forces due to settlement of the surrounding subsoil, caused by the surcharge loading of the approach embankments. In view of this, it is recommended that the allowable pile capacities at the abutment locations, be reduced from 90 to 70 tons per pile. This provision was recommended at other structure locations in this area - (refer to our memo of May 6, 1969).

MD/ndef

cc: Messrs. S. McCombie
T. C. Kingsland
Foundations Files
Gen. Files

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

af

67-F-114

MS
File Please

Mr. C. S. Grebski,
Structural Design Engineer,
Downsview, Ontario.

Structural Planning Office,
Kingston, Ontario.

Mr. K. Bassi

4 March 1974.

W.P. 34-66-18 - Waterproofing & Paving of Bridge Decks
Ramsayville Easterly to Eighth Line Road
Highway 417, District 9 - Ottawa

Please find attached a copy of letter dated February 26, 1974 from Mr. P. Payer, Soil Mechanics Section, relating to the advisability of constructing approach slabs at the Anderson Road, 7th Line Road and 8th Line Road Underpasses as part of the above-mentioned project.

Please note that in the penultimate paragraph of Mr. Payer's letter he states the opinion that it may be satisfactory to incorporate approach slabs at the 7th Line Road structure but that approach slabs for the other two structures should be delayed. I therefore think that we should proceed on the basis of providing approach slabs at the 7th Line Road structure only.

I am expecting to hear from Mr. W. A. Stewart, Ottawa District Maintenance Engineer, whether repairs are required to any of the structures involved, and also as to the possibilities of closing off the underpass structures during the waterproofing and replacing of expansion joints.

T. C. Kingsland
Regional Structural Planning Engineer

TCK/hl
att.

c.c. J. Cruickshank
W. A. Stewart (+att.)
M. Devata - Att. P. Payer
E. R. Saint
A. J. Percy - Att. D. B. Thomas
R. Forrest



DEFECTS IN NEGATIVE DUE TO
CONDITION OF ORIGINAL DOCUMENT

Found. Files
67-11-114

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS, ONTARIO

MEMORANDUM

TO: Mr. T.C. Kingsland,
Regional Structural Planning Eng.,
Kingston, Ontario.

FROM: Soil Mechanics Section,
Geotechnical Office,
West Bldg., Downsview.

ATTENTION:

DATE: February 28th, 1974.

OUR FILE REF.

IN REPLY TO

SUBJECT: RE: Approach Slab Construction, 67-11-112
Anderson Road: W.P. 34-66-01; W.O. 69-11035
7th Line Road: W.P. 34-66-07; W.O. 67-11113
8th Line Road: W.P. 34-66-08, W.O. 67-11114.

Further to your memo dated February 21st, 1974, we have reviewed the settlement records for the approaches of the above-mentioned structures and submit the following comments:

Anderson Road: (W.P. 34-66-01.)

According to the settlements records, 0.9 ft. of settlements have occurred since the completion of the approach embankments. Based on theoretical settlement computations, approximately 5" to 6" of further settlements are anticipated in the next 10 years under the 9 ft. high approach embankments.

It should be noted that due to settlements, additional fill will be required to bring up to profile grade and this will induce additional stresses in the underlying soil and may increase the magnitude of the predicted settlements.

7th Line Road: (W.P. 34-66-07)

Settlement observations indicate that approximately 12" of settlement has been completed under the 11.5 ft. high embankments in the past 4 years, since the completion of the approach fills. It is estimated that up to 3" of additional settlements are expected in the next 10 years.

8th Line Road: (W.P. 34-66-08)

At this location settlement observations indicate that settlements of approximately 0.8 ft. have taken place since the completion of approach embankments of 10.5 ft. It is estimated that further settlements of 5" to 6" can be anticipated in the next 10 years.

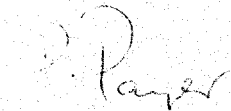
continued . . . /2

February 28th, 1974.

Mr. T.C. Kingsland - RE: Approach Slab Construction.

In our opinion it may be satisfactory to incorporate approach slabs in the forthcoming paving contract for the 7th Line Road approaches. However, it may be beneficial to delay the construction of concrete approaches for Anderson Road approaches and also 8th Line Road approaches in view of the anticipated future settlements.

Should we be of any further assistance with regard to the abovementioned projects, please contact our Office.



P. Payer,
Senior Engineer

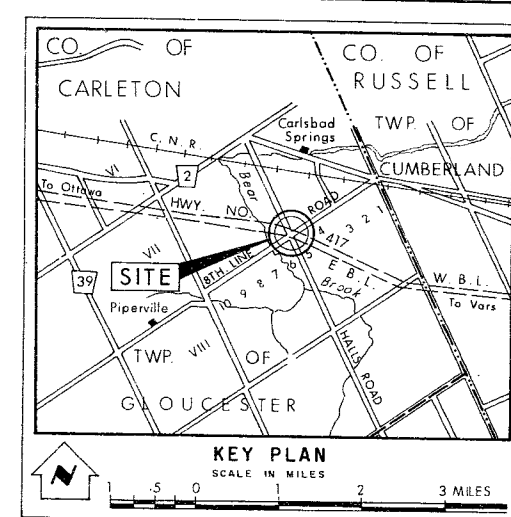
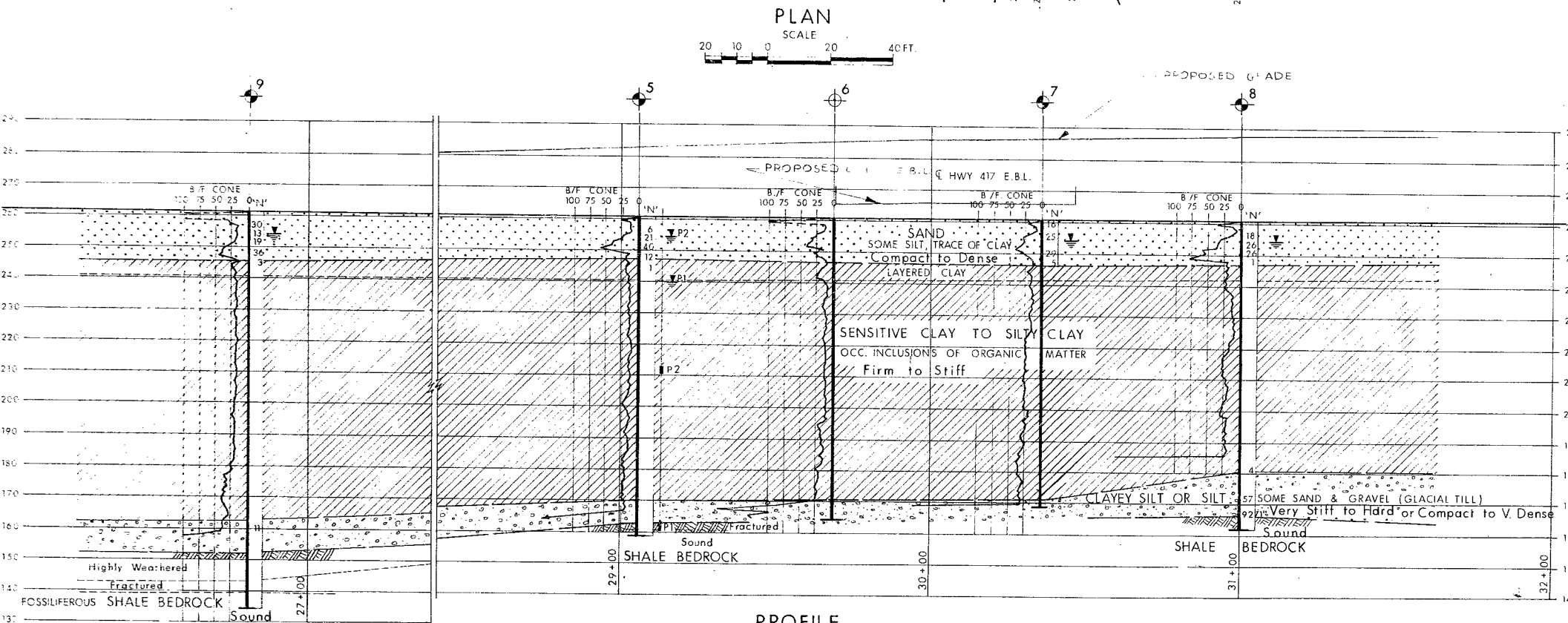
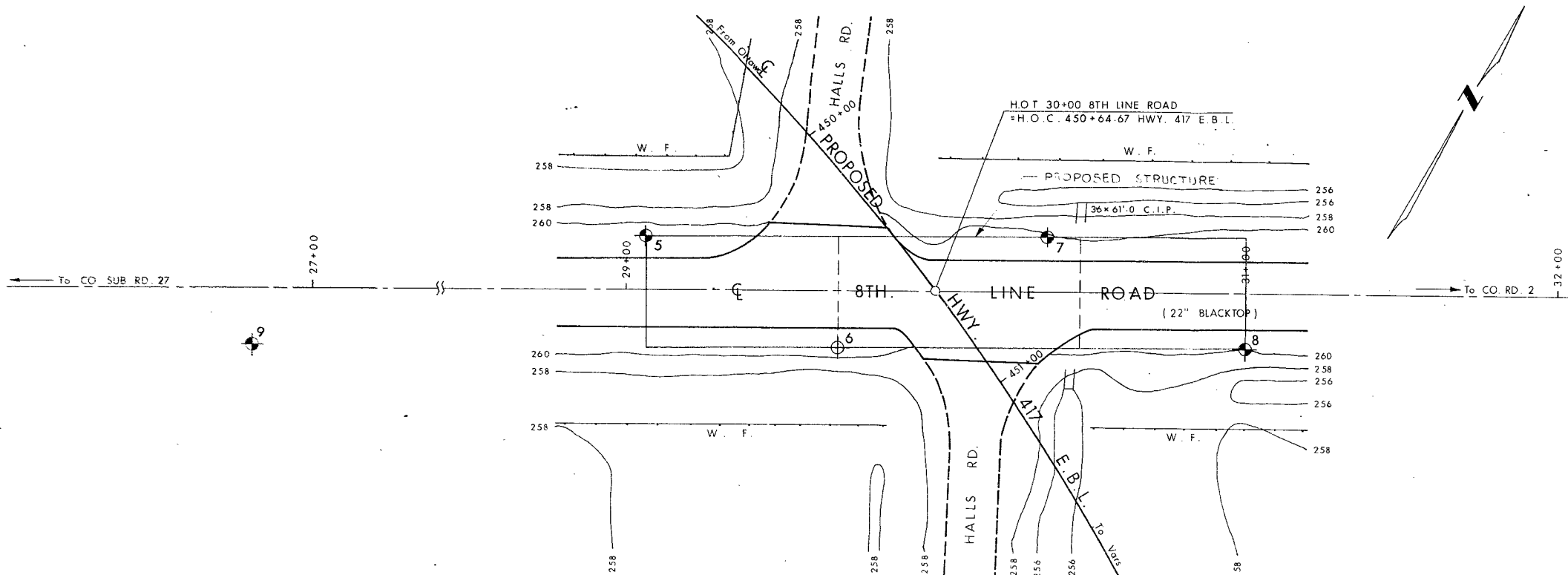
FOR: M. Devata,
Supervising Engineer.

PP/mj

c.c. E.V. Saint
A.J. Percy
J.A. Cruickshank

Foundations File (3)
Documents

#67-F-114
W.P. #34-66-08
HWY #417
EIGHTH
LINE ROAD



LEGEND

- Bore Hole
- Cone Penetration Hole
- Bore & Cone Penetration Hole
- Water Levels established at time of field investigation.
- Piezometer

NO.	ELEVATION	STATION	OFFSET
5	260.7	29+06	18' LT
6	260.5	29+68	18' RT
7	260.3	30+36	18' LT
8	260.1	31+00	18' RT
9	261.0	26+80	18' RT

NOTE

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

DATE	BY	DESCRIPTION

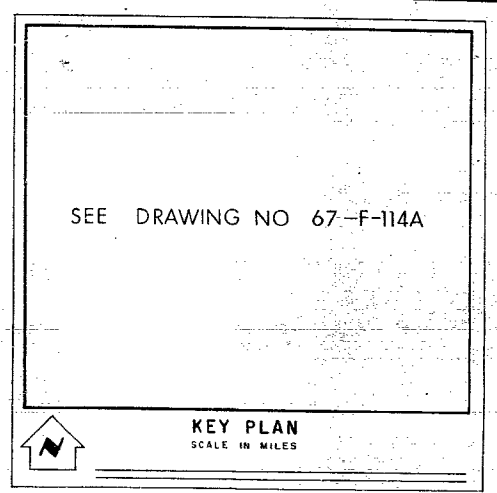
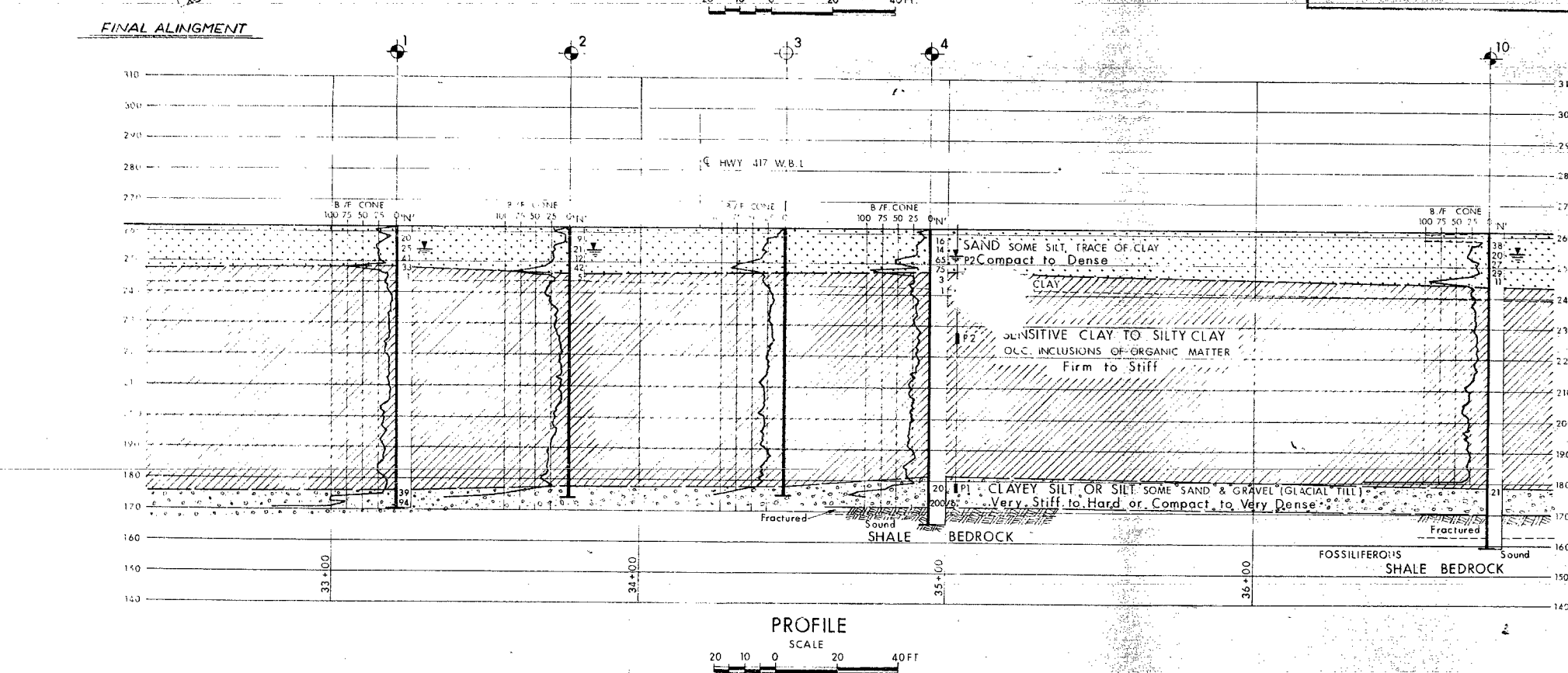
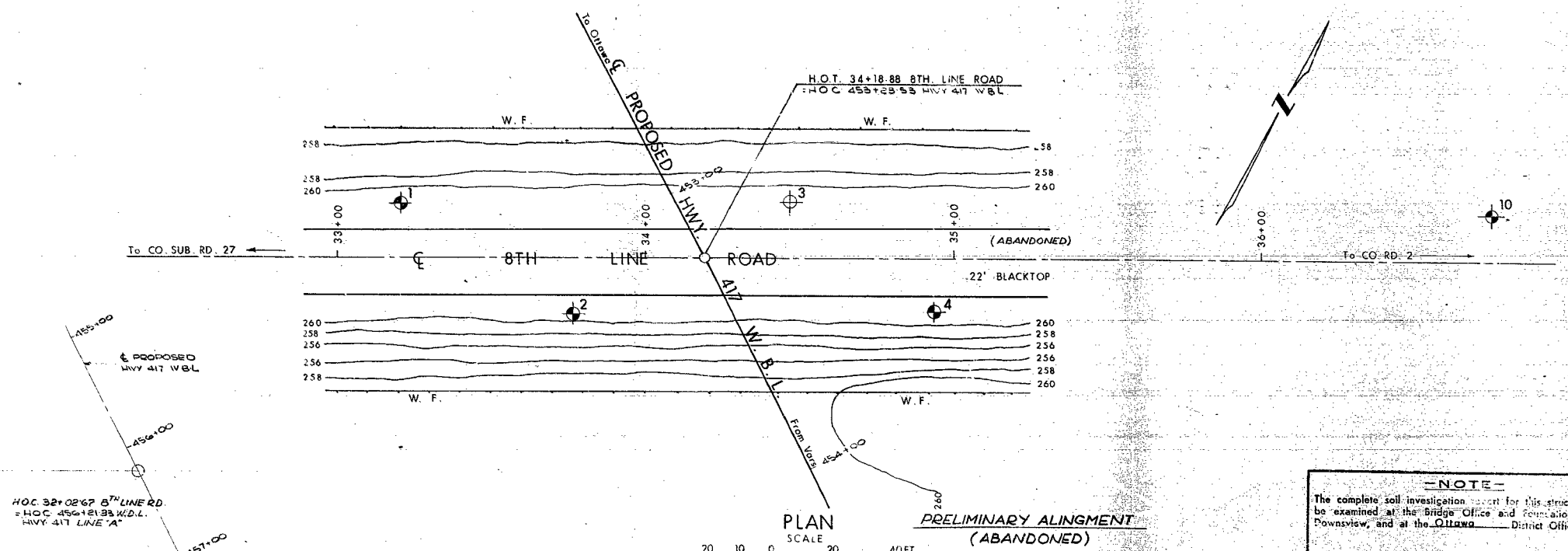
DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING DIVISION - FOUNDATION SECTION

EIGHTH LINE ROAD

KING'S HIGHWAY NO. 417 E.B.L. DIST. NO. 9
CO. CARLETON
TWP. GLOUCESTER LOT 5 & 6 CON. VII & VIII

BORE HOLE LOCATIONS & SOIL STRATA

SUBM'D B. D. CHECKED 2/77	W.P. NO. 34-66-08	M.B.T. DRAWING NO.
DRAWN G. P. CHECKED 2/77	JOB NO. 67-F-114	67-F-114A
DATE FEB. 20 1968	SITE NO.	BRIDGE DRAWING NO.
APPROVED [Signature]	CONT. NO.	



LEGEND

- Bore Hole
- Cone Penetration Hole
- Bore & Cone Penetration Hole
- Water Levels established at time of field investigation.
- Piezometer

NO.	ELEVATION	STATION	OFFSET
1	261.5	33+20	18' LT.
2	261.4	33+77	18' RT.
3	261.6	34+47	18' LT.
4	261.5	34+94	18' RT.
10	261.5	36+75	15' LT.

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

REVISIONS

NO.	DATE	BY	DESCRIPTION

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

EIGHTH LINE ROAD

KING'S HIGHWAY NO. 417 W.B.L. DIST. NO. 9
CO. CARLETON
TWP. GLOUCESTER LOT 5 CON. VII & VIII

BORE HOLE LOCATIONS & SOIL STRATA

SUBM'D. B. D.	CHECKED <i>[Signature]</i>	W.P. NO. 34-66-08	M.B.T. DRAWING NO.
DRAWN G. P.	CHECKED <i>[Signature]</i>	JOB NO. 67-F-114	67-F-114B
DATE, FEB. 20, 1968	SITE NO. 3-269	BRIDGE DRAWING NO.	16480-3
APPROVED <i>[Signature]</i>	PRINCIPAL, FOUNDATION ENG'G	CONT. NO.	

PRINT RECORD

NO.	FOR	DATE

