

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 16, 1968

OUR FILE REF.

IN REPLY TO

APR 24 1968

SUBJECT:

FOUNDATION INVESTIGATION REPORT
For
Eastbound Lane and Westbound Lane
Structures at the Crossing of
Prop. Hwy. #417 and 7th Line Rev'n.
Twp. of Gloucester, Co. of Carleton
District No. 9 (Ottawa)
W.J. 67-F-113 -- W.P. 34-66-07

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

The governing soil at this site is a very thick layer of soft marine clay. Because of the relatively low shear strength of this layer, large stabilizing berms are required. Long-term settlements of up to eleven feet are computed. Within the first few years settlements of three to four feet can be expected. The settlement problem is compounded by the fact that, in order to maintain the grade, more material will constantly have to be added. Settlements of this magnitude create special problems with respect to foundations of abutments and piers within the berm areas.

In view of the very serious problems described above - and we feel that they are very real indeed - some other solutions should be given serious consideration.

Settlements and the detrimental effects on the structure can be greatly reduced if the structure is lengthened to the points where the approaches are only about 8 feet high. This alternative, however, would be much more expensive to start with, though not necessary in the long run, because of the high maintenance costs of the structure and approaches as analyzed in the report.

cont'd. /2 ...

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

Attn: Mr. S. McCombie

April 16, 1968

It would appear to us that probably the best solution for this crossing is an overpass structure, with the 7th Line Reclamation Rd. in a cut. A rigid culvert or a large multi-plate culvert could be designed. The details of this alternative would have to be analyzed and worked out. We feel that such a solution is practical, and should prove to be very economical as compared with either of the two aforementioned alternatives. The only problem we can foresee at this moment is the drainage of the "tunnel". However, this is certainly a very minor problem and there are ready solutions for it.

We would suggest that you acquaint yourself with the contents of this report and give serious consideration to our last proposal. Should there be any questions that you would like to discuss, please feel free to call on this Office.

AGS/MdeF
Attach.

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

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

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FOUNDATION INVESTIGATION REPORT
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Eastbound Lane and Westbound Lane
Structures at the Crossing of
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Twp. of Gloucester, Co. of Carleton
District No. 9 (Ottawa)
W.J. 67-F-113 -- W.P. 34-66-07

1. INTRODUCTION:

The Foundation Section was requested to carry out a subsurface investigation at the site of the crossing of the proposed Hwy. #417 and the 7th Line Revision, some 9 miles south-east 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 September 25, 1967. An investigation was subsequently carried out by this Section to determine the subsoil conditions at the site.

This report contains the results of the investigation, together with recommendations pertaining to the foundations of the proposed structures and the stability of the approach embankments.

2. DESCRIPTION OF THE SITE AND GEOLOGY:

The site is located about 150 feet north of the existing 7th Line Rd., approximately halfway between Anderson Rd. and Russell Rd. The terrain, at this site, is gently undulating in relief; oblong hummocks of sand, some 3 to 4 feet in height, are occasionally superimposed over the landscape. The immediate area at and surrounding the site is wooded.

Physiographically, the site is situated in "The Russell and Prescott Sand Plains". In the area, deep clay deposits are capped by a mantle of silts and fine sands, some 6 to 24 feet in thickness. The deep stratum of marine clay was deposited by the

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

Champlain Sea, which inundated the area during the post-glacial period following the Wisconsin Glacial Age. This clay deposit has become known as "Leda Clay". The clay stratum is underlain by a glacial till, which in turn, is underlain by grey to black calcareous shale of the Lorraine formation, Ordovician Period.

3. FIELD AND LABORATORY WORK:

Six sampled boreholes were put down during the course of the investigation, using conventional diamond drill rigs adapted for soil sampling purposes. Four of these borings were accompanied by a dynamic cone penetration test.

Samples were recovered at required depths in 2" and 3" I.D. Shelby tubes, which were manually pushed into the soil. In an effort to reduce the degree of disturbance, some Shelbies were advanced using a piston technique. In addition, samples of the surficial deposit and glacial till were obtained using 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. Field vane tests were carried out in the cohesive portion of the overburden, where possible, to determine the undrained shear strength of the stratum. Bedrock was proven at five of the boring locations by diamond core drilling in either AXT or BXL size.

The groundwater level conditions across the site were determined by installing sealed piezometers in three of the boreholes. This information was supplemented by recording the groundwater level in the open boreholes at the remaining boring locations.

The locations and elevations of all borings were surveyed by personnel from the Kingston Regional Engineering Surveys Section, and are shown on Drawings 67-F-113A (E.B.L.) and 67-F-113B (W.B.L.), together with the estimated stratigraphical profile at the respective crossings. All elevations given in the report are referenced to a Geodetic datum.

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

All 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 engineering properties of the overburden, namely:

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

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

4. SUBSOIL CONDITIONS:

4.1) General:

The surficial deposit across the site is basically composed of a loose to dense fine sand with traces of silt and gravel. This deposit is some 6 to 24 feet thick. Underlying the surficial deposit is a soft to stiff, highly plastic, sensitive marine clay stratum some 153 to 167 feet in thickness. Directly underlying the clay stratum is a deposit of glacial till up to 21 ft. thick, composed primarily of hard clayey silt with 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 Drawing 67-F-113A and 67-F-113B, is inferred from this data.

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

cont'd. /4 ...

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

4.2) Fine Sand with Traces to Some Silt - Surficial Deposit:

A surficial deposit, composed primarily of brown to grey fine sand with some to traces of silt and gravel, was encountered across the site. The thickness of this deposit varies from 6 to 24 ft. In B.H. #1 some pockets and layers of clay and silt are present below elevation 253. 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 sand, as determined from laboratory testing, ranges from 19% to 30%.

Standard penetration tests, carried out within the surficial deposit, are plotted on the borelog sheets as well as on Figure 1. The results of this testing gave 'N' values which range, in a random fashion, from 3 to 34 blows/ft. Based on these values, it is estimated that the relative density of the deposit ranges from loose to dense, being typically in the compact range.

4.3) Sensitive Clay:

The surficial deposit is underlain by the predominant overburden stratum across the site, a sensitive grey clay with occasional organic inclusions. The overall thickness of the clay stratum ranges from 153 feet to 167 feet. At B.H.'s 2, 3, 4 and 6 the upper 4 to 13 feet of the stratum is composed of alternate brown and grey clay layers. Numerous silt and sand seams, up to 1 inch thick, were encountered throughout this layered zone. The remainder of the stratum, as encountered at the boring locations, is quite homogeneous in nature, with a few notable exceptions. Namely, below elevation 90, at B.H.'s 1, 2 and 5, the stratum is again layered, with the individual layers composed of clay, clayey silt and silt. The thickness of these layers ranges from 1/4 to 1 inch. Grain-size distribution curves for samples of the clay stratum are shown on Figures 5 and 6 in Appendix I.

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

cont'd. /5 ...

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

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

		<u>Range</u>	<u>Average</u>
Bulk Density (p.c.f.)	(γ)	91 - 117	101
Liquid Limit (%)	(w_L)	28 - 82	61
Plastic Limit (%)	(w_P)	19 - 36	27
Natural Moisture Content (%)	(w)	32 - 96	64
Liquidity Index	(I_L)	0.5 - 2.1	1.1
Initial Void Ratio	(e_o)	1.6 - 2.2	
Compression Index	(C_c)	1.2 - 2.7	
Preconsolidation Pressure (p.s.f.)	(P_c)	1400 - 5200	
Undrained Shear Strength (p.s.f.)	(C_u)	<u>Range</u> (C_u)	<u>Range</u> Sensitivity (S)
i) Field Vanes		360 - 2000	2 - 15
ii) Lab. Vanes		380 - 2685	2 - 17
iii) Lab. Testing		300 - 2005	-

The Atterberg limit tests, summarized above, are also plotted on the Plasticity Chart, Figure 8. These results indicate that, in general, the clay is inorganic and of high to intermediate plasticity, with the natural water content greater than the liquid limit. Referring to Figure 1, it can be seen that the undrained shear strength increases in a linear fashion with depth as

cont'd. /6 ...

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

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

represented by a C_u/P_o ratio of about 0.4, where P_o is the effective overburden pressure. Based on these results, it is estimated that the consistency of the stratum varies from soft, immediately below the surficial deposit, increasing to very stiff with depth. The undrained shear strength values obtained from the laboratory testing, generally gave lower values than that obtained from the field vane tests. It is considered that this is primarily due to unavoidable sample disturbance caused by the field and laboratory handling and subsequent testing of the sensitive clay.

The consolidation characteristics of the stratum were determined by carrying out four laboratory tests, the results of which are shown as Void Ratio vs. Pressure plots, on Figures 10 to 13, inclusive. The results of this testing indicate that the clay is preconsolidated by about 400 to 2000 p.s.f. in excess of 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 or Silt with Sand and Gravel - Glacial Till:

This heterogeneous but generally cohesive deposit, which directly underlies the sensitive clay stratum across the site, is encountered between elevations 94 and 83. The thickness of the glacial till varies from 9 feet at B.H. #5 to 21 ft. at B.H. #4. At about elevation 80, occasional seams and layers of sand, silt and gravel were encountered throughout the glacial till. In addition, the deposit becomes very bouldery with depth, as indicated by the necessity of advancing the borings by diamond drilling techniques. The boulders encountered in this zone vary from 4 to 6 inches in size. Typical grain-size distribution curves, obtained from samples of the deposit, are shown on Figure 7.

cont'd. /7 ...

4. SUBSOIL CONDITIONS: (con't'd.) ...

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

The Atterberg limit tests, carried out on representative samples 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 16 to 22 and 13 to 15, respectively. The corresponding natural water content is generally below the plastic limit, represented by a liquidity index (I_L) between 0.3 and 1.1 with the average value being about 0.6.

The standard penetration resistance, or 'N' values, vary from 129 blows/ft. to as high as 150 blows/4 inches, indicating that the consistency of the deposit is hard. In B.H.'s 2 and 3 the upper 2 ft. of the glacial till layer is in a weathered and reworked condition as evidenced by the fact that a Shelby tube could be manually pushed into this zone. It is considered that this upper zone has a consistency in the stiff range.

4.5) Shale Bedrock:

Bedrock was proven in five of the borings, namely: B.H.'s #1 to 5, inclusive, by obtaining from 3 to 7 feet of either AXT or BXL rock core. The depth at which bedrock was encountered ranged from elevations 73 to 79 - i.e., from 181 to 187 feet below existing ground surface.

The bedrock is composed of a dark grey, calcareous shale interbedded with shaley limestone. The upper 2 to 3 feet of the bedrock is generally in a fractured or jointed condition; below this upper layer, however, 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 boreholes #1, 2 and 4, and 11) the open boreholes at the remaining locations. These observations which are recorded on the

cont'd. /8 ...

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

borehole logs and summarized on Drawings No. 67-F-113A and 67-F-113B, indicate that the groundwater level in the surficial deposit and sensitive clay stratum is about elev. 258 - i.e., some 2 ft. below ground level. The corresponding piezometric groundwater level within the lower portion of the glacial till, as determined at B.H.'s #1 and 4, was at elevation 248 and 246, respectively - i.e., some 13 feet below ground surface.

It is pertinent to note that the heterogeneous glacial till is more permeable than the overlying subsoil. Also, there are numerous very pervious sand and silt seams throughout the glacial till. It is, therefore, inferred that the groundwater level within this lower deposit may be at a lower elevation due to downward drainage, which occurs once the more pervious zones are intersected.

6. DISCUSSION AND RECOMMENDATIONS:

6.1) General:

It is proposed to construct two underpass structures to carry the 7th Line Rd. realignment over the East and Westbound lanes of proposed Highway #417. The Westbound lane structure (W.B.L.) will be located in line with the Eastbound lane structure (E.B.L.) with present proposals calling for a distance of 147 feet between the two structures. The underpasses are to be three-span structures (74'-92'-93' and 60'-99'-60' for the E.B.L. and W.B.L. structures, respectively).

The proposed profile grade of Hwy. #417, in the vicinity of the crossings, is elevation 267, some 3 and 8 ft. above the existing ground level for the E.B.L. and W.B.L., respectively. 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.

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

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

The maximum proposed profile grade of the revised Seventh Line Rd., in the vicinity of the crossings, is elevation 292. At this grade the associated approach embankments will have a maximum height of about 32 feet above ground surface. The embankments will have a crest width of 36 feet.

Underlying some 8 to 24 feet of fine sand is the predominant deposit across the site, a soft to very stiff, sensitive marine clay varying from 153 feet to 167 ft. in thickness. The clay is underlain by up to 21 feet of stiff to hard 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 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 and settlement of the approach fills are the major problems at this site, they will be discussed first.

6.2) Approach Embankments:

6.2.1) Stability Considerations:

The critical condition 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. For 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:

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:

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>
260 to 246	Surficial Deposit - Sand with some Clay and Silt. (W/L at elev. 256)	$\gamma = 125$ p.c.f., $\gamma' = 63$ p.c.f. $\phi = 30^\circ$
246 - 240	Sensitive Clay	$\gamma = 101$ p.c.f. $C_u = 500$ p.s.f.
240 - 230	" "	$\gamma' = 39$ p.c.f. $C_u = 630$ p.s.f.
230 - 220	" "	" $C_u = 780$ p.s.f.
220 - 210	" "	" $C_u = 950$ p.s.f.
210 - 195	" "	" $C_u = 1150$ p.s.f.
195 -	" "	" $C_u = 1500$ p.s.f.

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 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.) ...

	<u>Height Of Fill</u>	<u>Length of Berm Required at Mid-Height</u>
i) <u>W.B.L. Structure</u>		
Longitudinal Direction		
- West Approach	32'	40'
- East Approach	27'	20'
Transverse Direction		
- East Approach	0 - 16'	0'
" "	16 - 27'	0 - 45'
ii) <u>E.B.L. Structure</u>		
Longitudinal Direction		
- West Approach	27'	20'
- East Approach	32'	40'
Transverse Direction		
- West Approach	0 - 16'	0'
" "	16 - 27'	0 - 45'
iii) Embankment Between the Two Structures		
Transverse Direction	32'	65'

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

1) Fill less than 16 feet in height may be constructed with standard 2:1 side slopes.

2) Fills in excess of 16 feet should be constructed with a single berm at mid-height.

cont'd. /12 ...

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

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

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

3) At the proposed profile grade, berms will be required in both the longitudinal and transverse direction at all the approach fill locations. In order to accommodate berms in the longitudinal direction, the proposed spans of the structure may have to be modified.

4) It may be advantageous to minimize the longitudinal and transverse berm requirements by limiting the height of fill. This would necessitate a multi-span structure, but minimizing the heights of fill also has the advantage of reducing the settlements induced in the foundation subsoil, as discussed in Section 6.2.2 of this report.

5) The proposals discussed above are equally feasible with respect to stability of the approach fills. The ultimate choice, however, will be based on economic considerations.

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

6.2.2) Settlement Considerations:

The underlying highly compressible clay stratum will undergo excessive settlements 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 embankment between the two structures where the height of fill will be of the order of 32 feet above ground surface. The computations indicate that this settlement could be as much as 9.5 feet under the centre-line of the embankment. However, if the maximum fill height is maintained at 16 feet, the total consolidation settlement would be of the order of 3.5 feet.

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

6.3) Structure Foundations:

Because of the soft and compressible nature of the subsoil, the structure piers and abutments should be pile-supported. End-bearing piles or, alternatively, friction piles can be considered.

6.3.1) End-bearing Piles:

The piers and abutments can be supported on end-bearing piles driven to practical refusal within the lower portion of the competent glacial till deposit. For estimating purposes, it can be assumed that the piles would meet practical refusal between elevations 75 to 85. The allowable pile load would be dependent on the section chosen - for example, a closed-end 12-3/4" O.D. steel tubular pile, driven to practical refusal, could be designed to carry 75 tons/pile. End-bearing piles would be unusually long; they would, however, reduce the settlement of the structure components to a negligible amount. Continuous structures could, therefore, be employed.

Since settlement of the proposed roadway embankments will be excessive, considerable negative skin frictional loads may be imposed on the piles supporting the abutments and end-piers. It would be advisable to take precautions to prevent the mobilization of these large 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 this technique, an over-sized hole would be augered through the clay stratum to a depth of about 110 to 120 feet below ground surface. The closed-end pile would then be telescoped into the open hole to this depth, from where it would be driven to practical refusal. The annular space between the pile and the augered hole would then be backfilled with a bentonite slurry or a drilling mud.

In addition to the negative skin frictional forces, movement of the subsoil due to strain imposed by the embankment loading, will generally tend to displace the long slender piles

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

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

6.3.1) End-bearing Piles: (cont'd.) ...

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 of the abutments in the longitudinal direction. No bouldery or rock fill should be placed in areas where piles are to be driven.

6.3.2) Frictional Piles:

As an alternative to end-bearing piles, the abutments and piers can be founded on piles located within the clay stratum. Such piles would primarily derive their capacity from the adhesion between the foundation soil and the shaft of the pile. The allowable pile load would be dependent on the pile type and section chosen - for example, No. 14 timber piles, driven 45 feet into original ground, could be designed for an allowable pile capacity of 15 tons/pile. In addition to timber piles, it is considered that closed-end 12-3/4" O.D. tubular steel piles could be employed. The allowable capacities for this size of tubular pile, driven various lengths into natural ground, are given on Figure 3 in the Appendix.

The structure units, founded on friction piles, will undergo settlement due to the consolidation of the foundation soil under application of load. The actual magnitude of the settlement at the various locations will be dependent on a number of factors, including:

- i) Pile type and length.
- ii) Configuration of the pile group.
- iii) Applied load.
- iv) Influence of approach fills.

cont'd. /15 ...

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

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

A true settlement analysis, therefore, can only be carried out once the structure details have been finalized. A few qualitative points, however, can be made, namely:

a) The settlement of the piles at the abutment and the pier locations will be influenced by the approach fills. Differential settlements, therefore, can be expected between the abutments and end piers. This being the case, it is recommended that the end spans of the structures be simply supported.

b) If friction piles are employed to support the structural elements, then it is recommended that the allowable pile load be determined by carrying out full-scale pile loading tests at this site.

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 2 to 4 feet below ground surface. Because these excavations will be carried out mainly through a relatively pervious sand deposit, seepage will occur. This could 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 structures to carry the 7th Line Revision Rd. over the East and Westbound lanes of proposed Hwy. #417, in the Township of Gloucester, County of Carleton, is reported.

cont'd. /16 ...

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

Underlying between 6 and 24 feet of sand is the predominant overburden stratum across the site, composed of a soft to very stiff, sensitive marine clay varying from 153 to 167 feet in thickness. The clay is underlain by up to 21 feet of stiff to hard cohesive glacial till which, in turn, is followed by shale bedrock.

The groundwater level in the surficial sand deposit and underlying clay stratum was, at the time of the investigation, some 2 to 4 feet below ground surface.

The piers and abutments can be supported on end-bearing piles driven to practical refusal into the lower glacial till deposit; for estimating purposes it is considered that the pile tips will be between elevations 75 to 85. As an alternative, friction piles, founded within the extensive clay stratum, could be employed; timber or closed-end 12-3/4" O.D. steel tubular piles could be used for this purpose. The particular problems associated with each pile type, such as negative skin frictional forces, are discussed in detail in the 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 fills in excess of 16 feet in height. The berm requirements in the longitudinal direction necessitates that the proposed structure spans be modified.

Settlements up to 9.5 feet are estimated for a maximum fill height of 32 feet. In order to reduce the magnitude of the settlements, consideration should be given to constructing the approach fills some 18 to 24 months prior to the construction of the foundations for the structure.

Additional recommended construction procedures are presented in the report.

cont'd. /17 ...

8. MISCELLANEOUS:

The field work for this project was carried out during the period of February 2 to 29, 1968, under the supervision of Mr. W. G. Hutton, Project Foundation Engineer, who also prepared this report.

The investigation was carried out under the general supervision of Mr. B. T. Darch, Senior Foundation Engineer. The report was reviewed by Mr. M. Devata, Supervising Foundation Engineer.

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

April, 1968.

APPENDIX I

DEPARTMENT OF HIGHWAYS - ONTARIO

RECORD OF BOREHOLE NO. 1

FOUNDATION SECTION

MATERIALS & TESTING DIVISION

JOB 67-F-113

LOCATION Sta. 29 + 00 Ø Prop. 7th Line Revision

ORIGINATED BY WH

W.P. 34-66-07

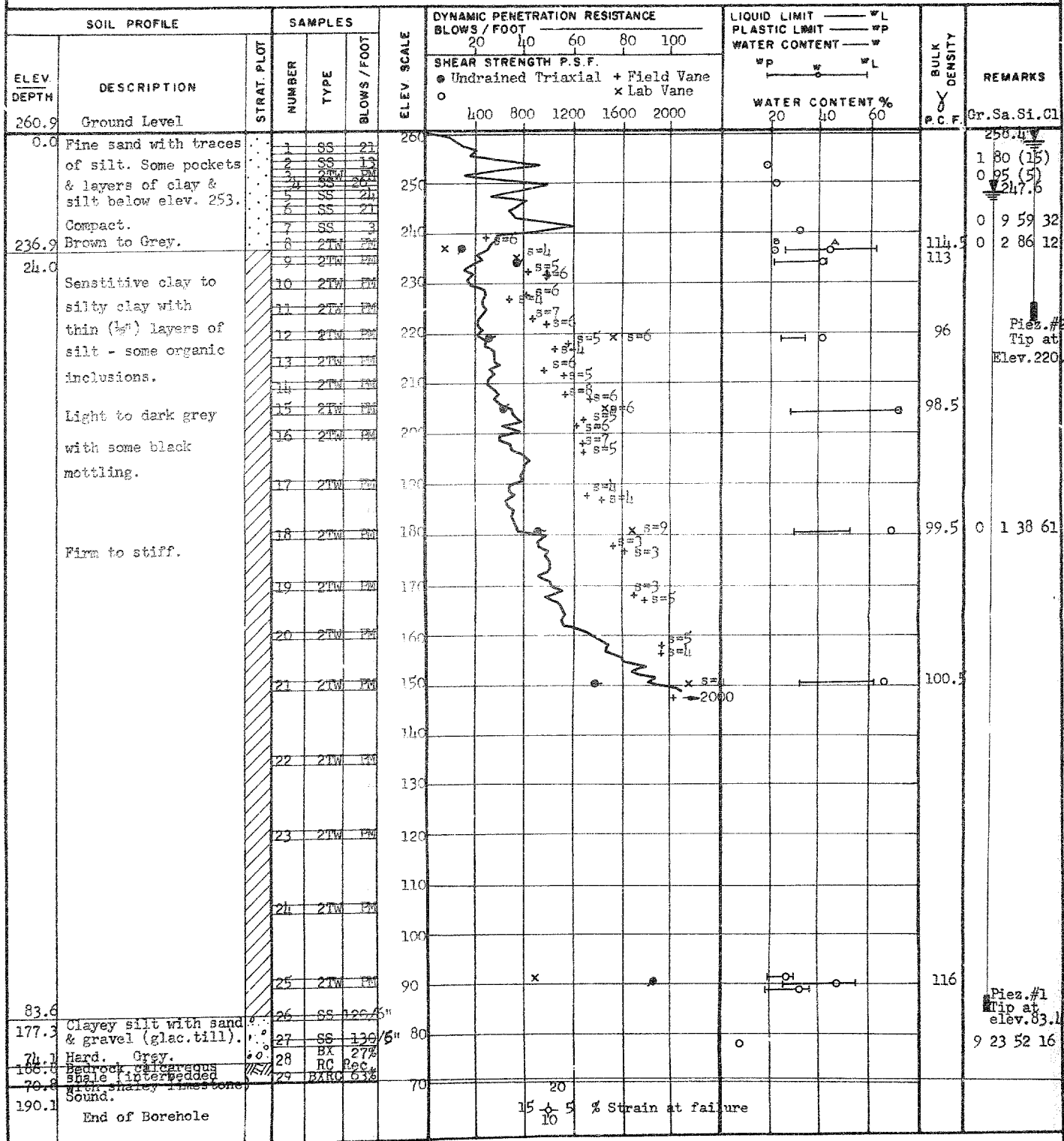
BORING DATE Feb. 6 - 15, 1968

COMPILED BY WH

DATUM Geodetic

BOREHOLE TYPE Diamond Drill - NX Casing

CHECKED BY



MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 2

FOUNDATION SECTION

JOB	67-F-113	LOCATION	Sta. 31 + 12 @ Prop. 7th Line Rev. o/s 6' Left	ORIGINATED BY	WH
W.P.	34-66-07	BORING DATE	Feb. 19 - 27, 1968	COMPILED BY	WH
DATUM	Geodetic	BOREHOLE TYPE	Diamond Drill - HM - BX Casing	CHECKED BY	<i>[Signature]</i>

[illegible]

DEPARTMENT OF HIGHWAYS - ONTARIO

RECORD OF BOREHOLE NO. 3

FOUNDATION SECTION

MATERIALS & TESTING DIVISION

JOB 67-F-113

LOCATION Sta. 33 + 36 & Prop. 7th Line Revision

ORIGINATED BY W.H.

W.P. 34-66-07

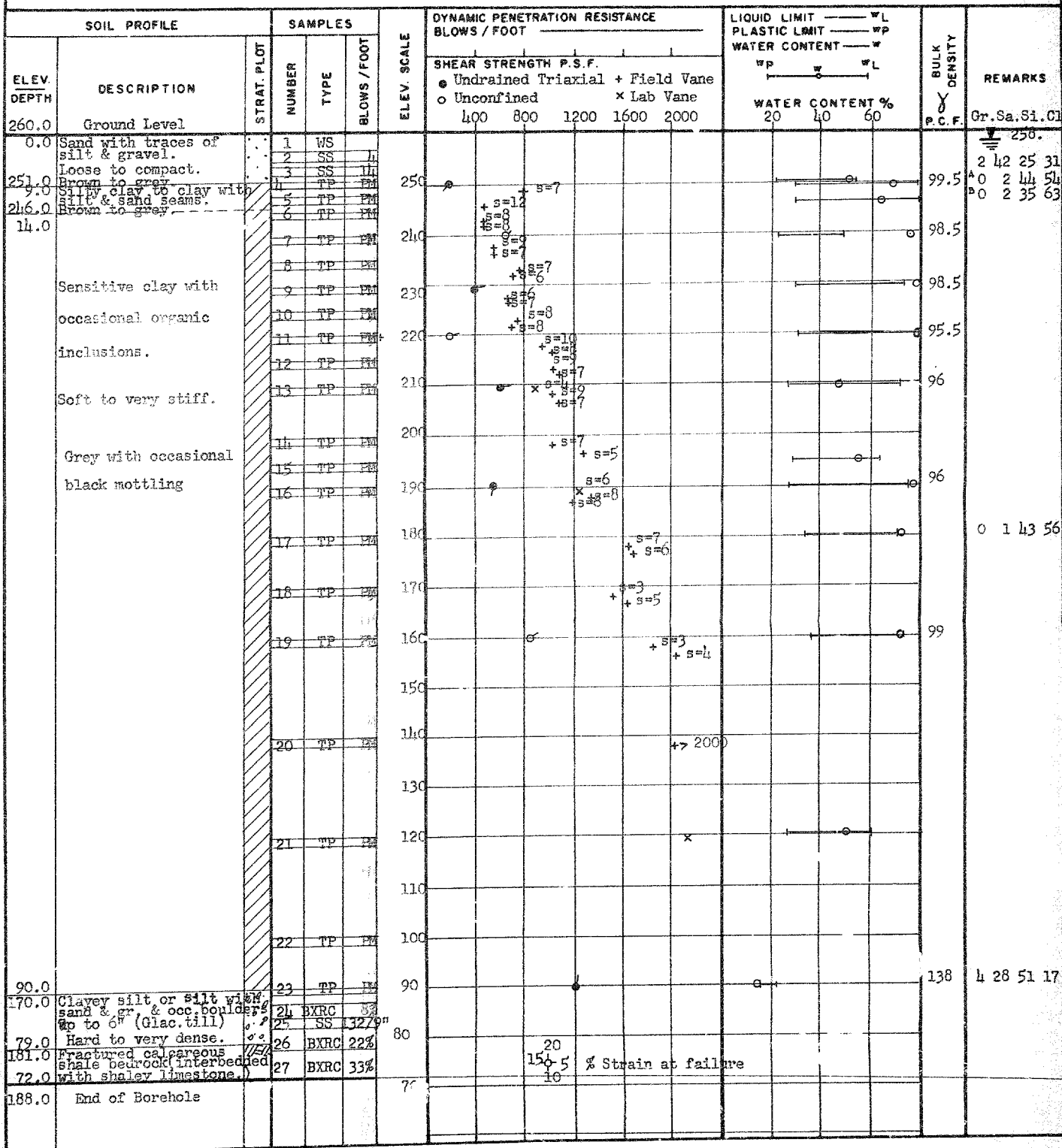
BORING DATE Feb. 19 = 24 1968

COMPILED BY WH

DATUM Geodetic

BOREHOLE TYPE Diamond Drill @ - NX - BX Casing

CHECKED BY



DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 4

FOUNDATION SECTION

JOB 67-E-113

LOCATION Sta. 35 + 46 @ 7th Line Revision (Ottawa)

ORIGINATED BY GEH

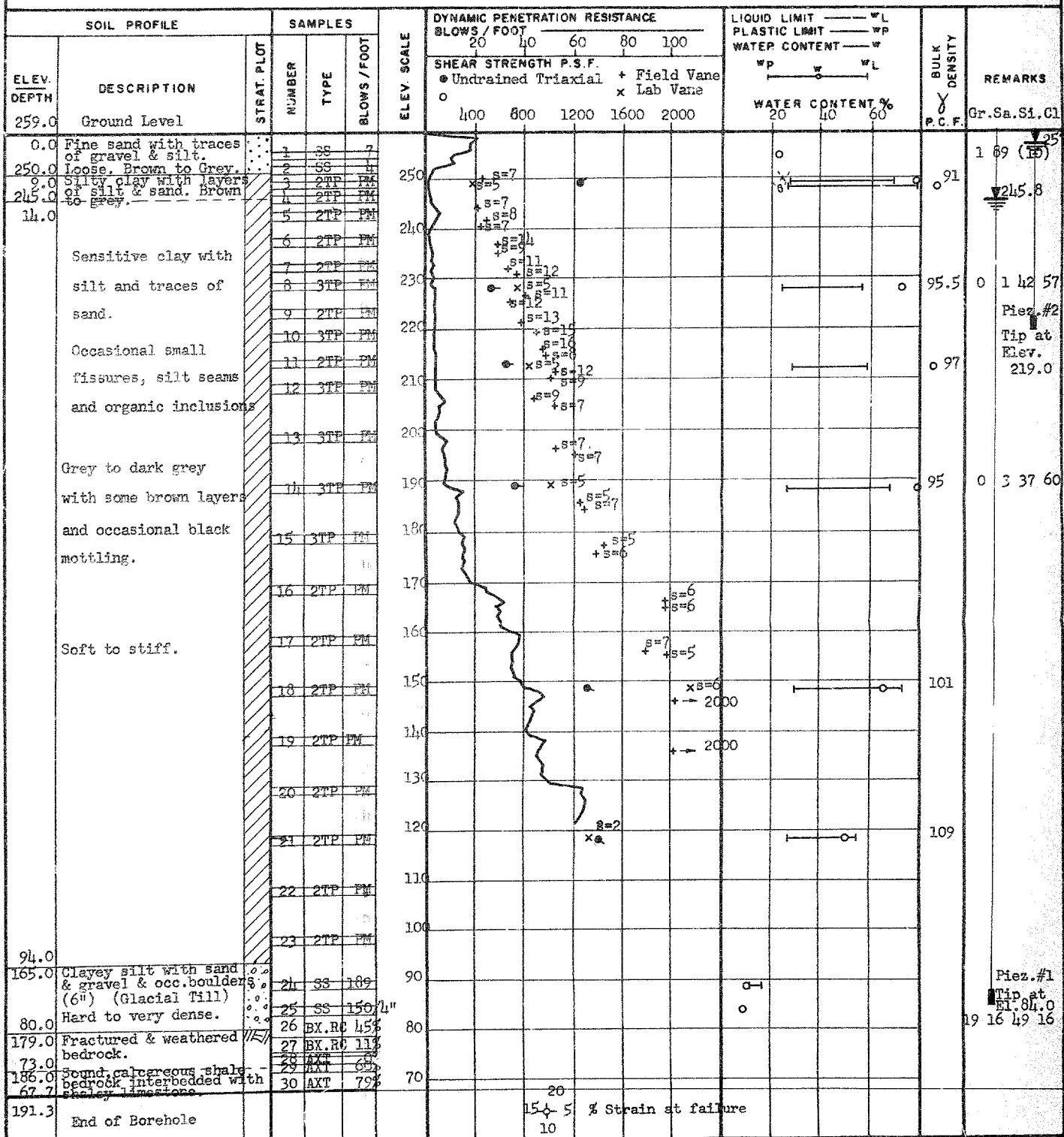
W.P. 34-66-07

BORING DATE Feb. 2 - 14, 1968

COMPILED BY WH

DATUM Geodetic

BOREHOLE TYPE Diamond Drill - HM BX Casing

CHECKED BY *SK*

DEPARTMENT OF HIGHWAYS - ONTARIO				RECORD OF BOREHOLE NO. 5				FOUNDATION SECTION				
MATERIALS & TESTING DIVISION												
JOB 67-F-113				LOCATION Sta. 27 + 00 @ Prop. 7th Line Revision				ORIGINATED BY WH				
W.P. 34-66-07				BORING DATE Feb. 7 - 15, 1968				COMPILED BY WH				
DATUM Geodetic				BOREHOLE TYPE Diamond Drill - NX Casing				CHECKED BY <i>HL</i>				
SOIL PROFILE		SAMPLES			DYNAMIC PENETRATION RESISTANCE				LIQUID LIMIT — WL PLASTIC LIMIT — WP WATER CONTENT — w		BULK DENSITY P.C.F.	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE	BLOWS / FOOT	20 40 60 80 100	20 40 60	WATER CONTENT %		
260.0	Ground Level							400 800 1200 1600 2000		20 40 60		
0.0	Fine sand with some silt. Loose to Dense.		1	SS	14							Gr. Sa. Si. Cl.
246.0	Brown to Grey.		2	SS	26							256.
14.0	Sensitive clay with occasional organic inclusions.		3	SS	34							Org. 0.37%
			4	SS	4							0.84 (16)
			5	SS	1							0.93 (7)
			6	SS	1							
			7	2TP	PM							
			8	2TP	PM							
			9	2TP	PM							
			10	2TP	PM							
			11	2TP	PM							
			12	2TP	PM							
			13	2TP	PM							
			14	2TP	PM							
			15	2TP	PM							
			16	2TP	PM							
			17	2TP	PM							
			18	2TP	PM							
			19	2TP	PM							
			20	2TP	PM							
			21	2TP	PM							
			22	2TP	PM							
			23	2TP	PM							
83.0	Silt seams up to 1" thick below Elev. 90		24	SS	70							
177.3	Clayey silt with sand & gravel (Glac. till) changing to shale fragments about elev. 75		25	AXT	RC 82							
74.2	Sound calcareous shale		26	SS	112							
185.8	Sound calcareous shale		27	AXT	112							
68.5	bedrock interbedded with clayey limestone		28	AXT	51							
191.5	End of Borehole											

15 20 5 Axial Strain at failure

RECORD OF BOREHOLE NO. 6

FOUNDATION SECTION

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & TESTING DIVISION

JOB 67-F-113

LOCATION Sta. 37 + 00 @ 7th Line Revision

ORIGINATED BY GEH

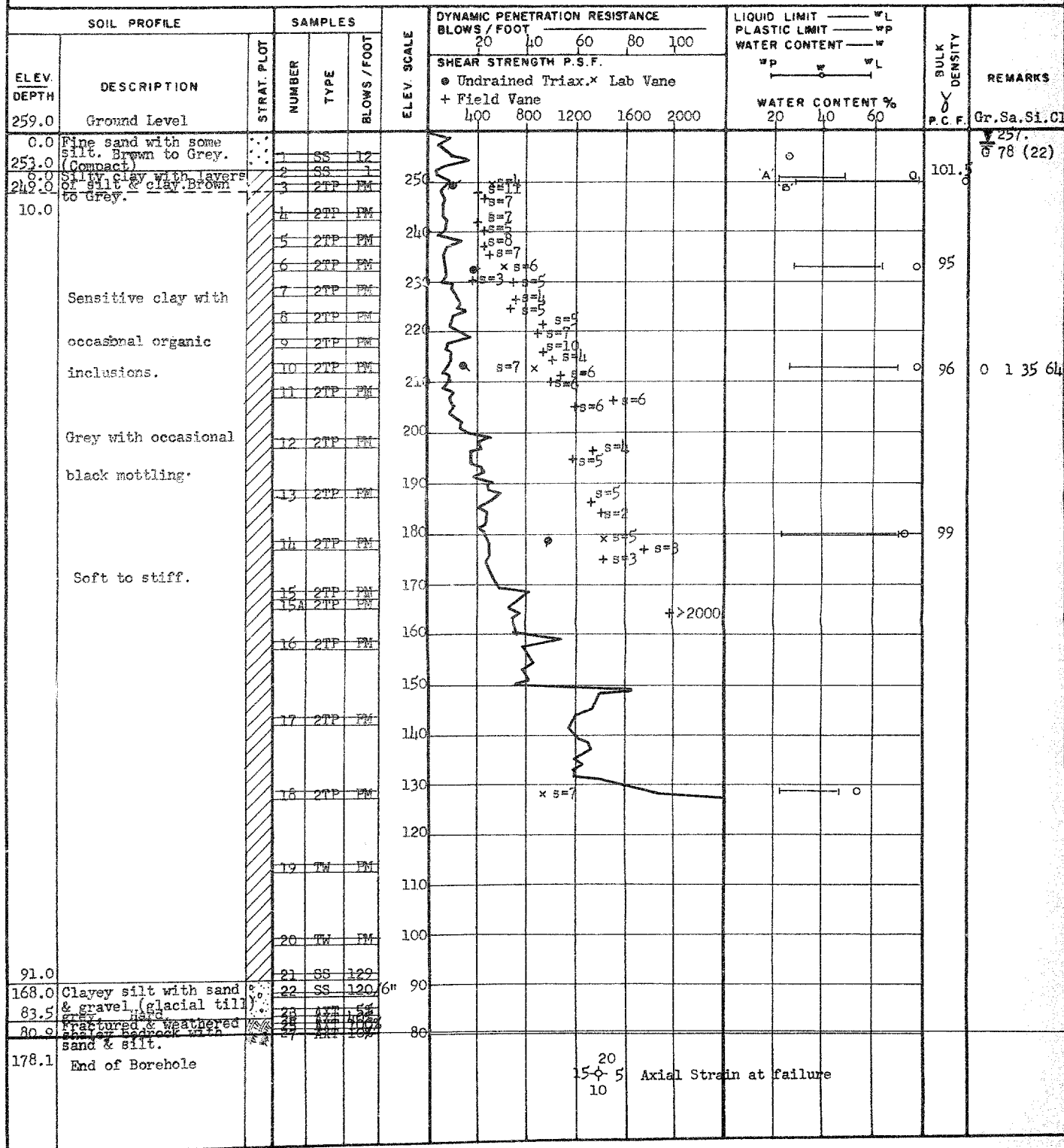
W.P. 34-66-07

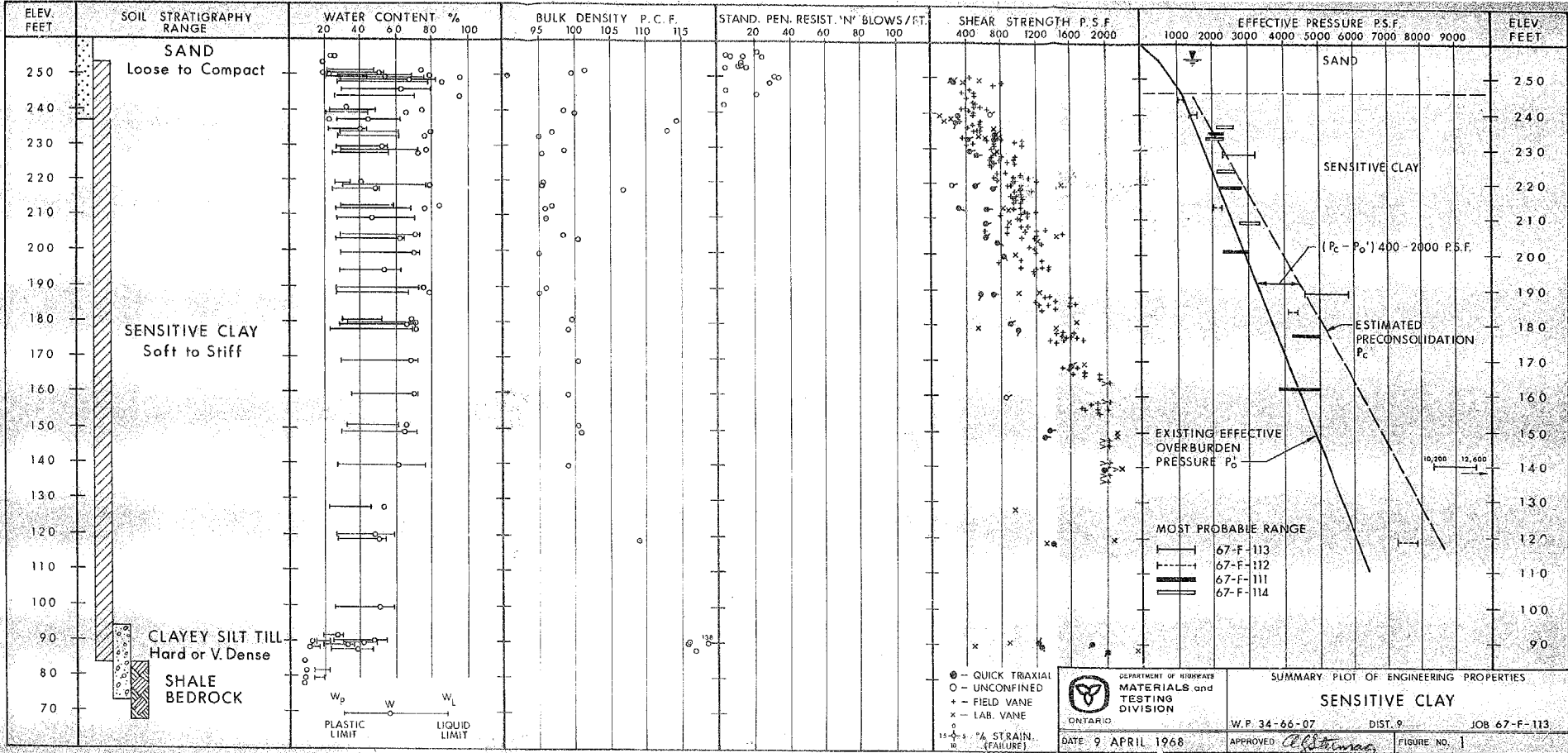
BORING DATE Feb. 7 - 23, 1968

COMPILED BY WH

DATUM Geodetic

BOREHOLE TYPE Diamond Drill - BX Casing

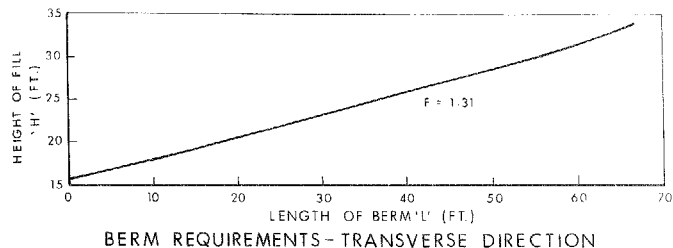
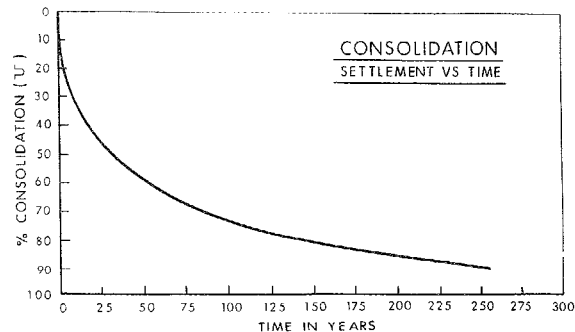
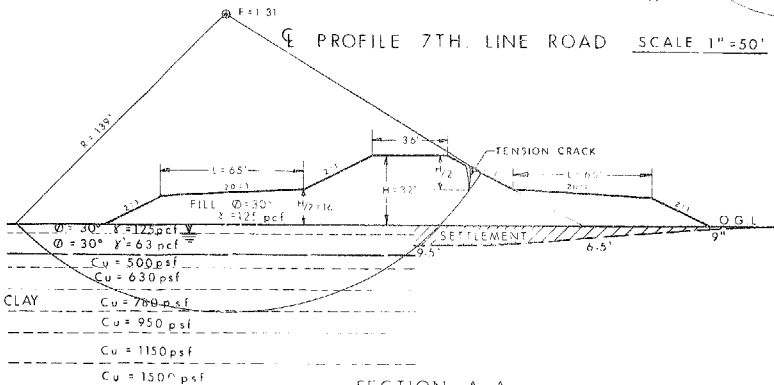
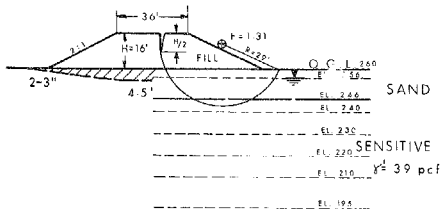
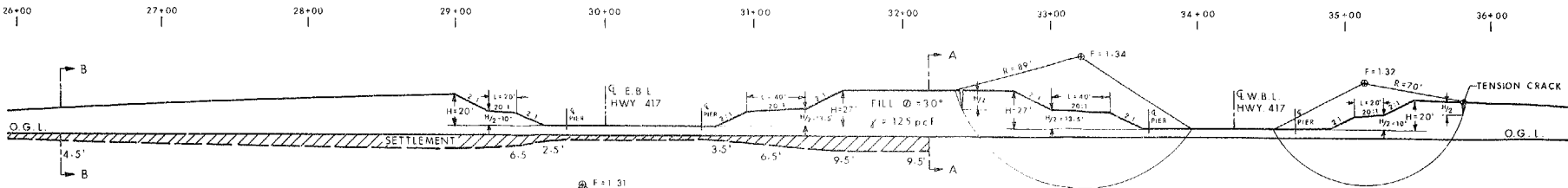
CHECKED BY *SL*




SUMMARY PLOT OF ENGINEERING PROPERTIES

SENSITIVE CLAY

W.P. 34-66-07 DIST. 9 JOB 67-F-113



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 ONTARIO	SUMMARIZED RESULTS OF		
	STABILITY & SETTLEMENT ANALYSES (APPROACH EMBANKMENTS)		
DATE APRIL 18, 1968	W.P. 34-66-07	DIST. 9	JOB 67-F-113
APPROVED <i>Alfama</i>	FIGURE NO. 2		

ALLOWABLE PILE LOADS

(CLOSED-END 12 $\frac{3}{4}$ " O.D. TUBULAR PILES)

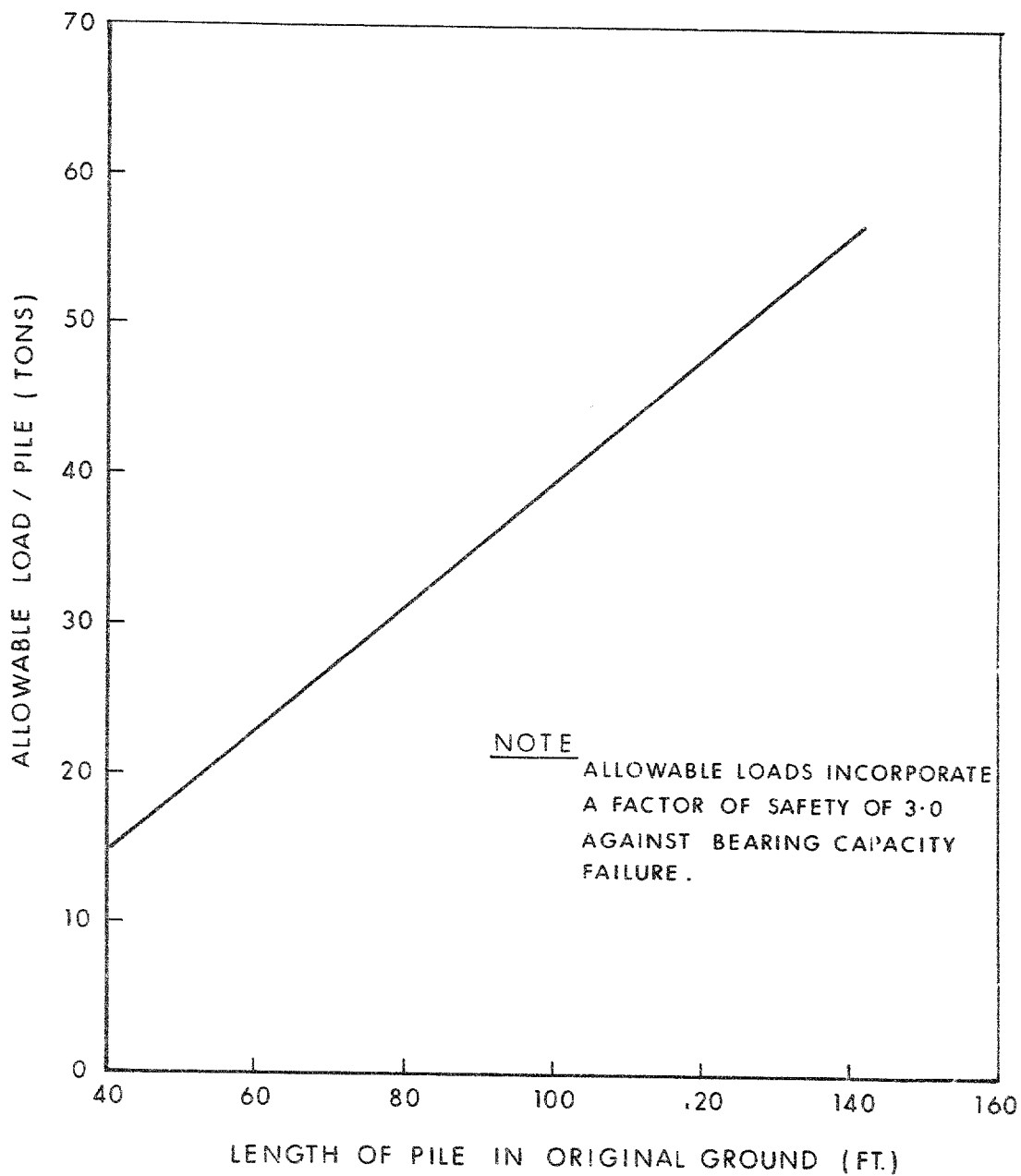
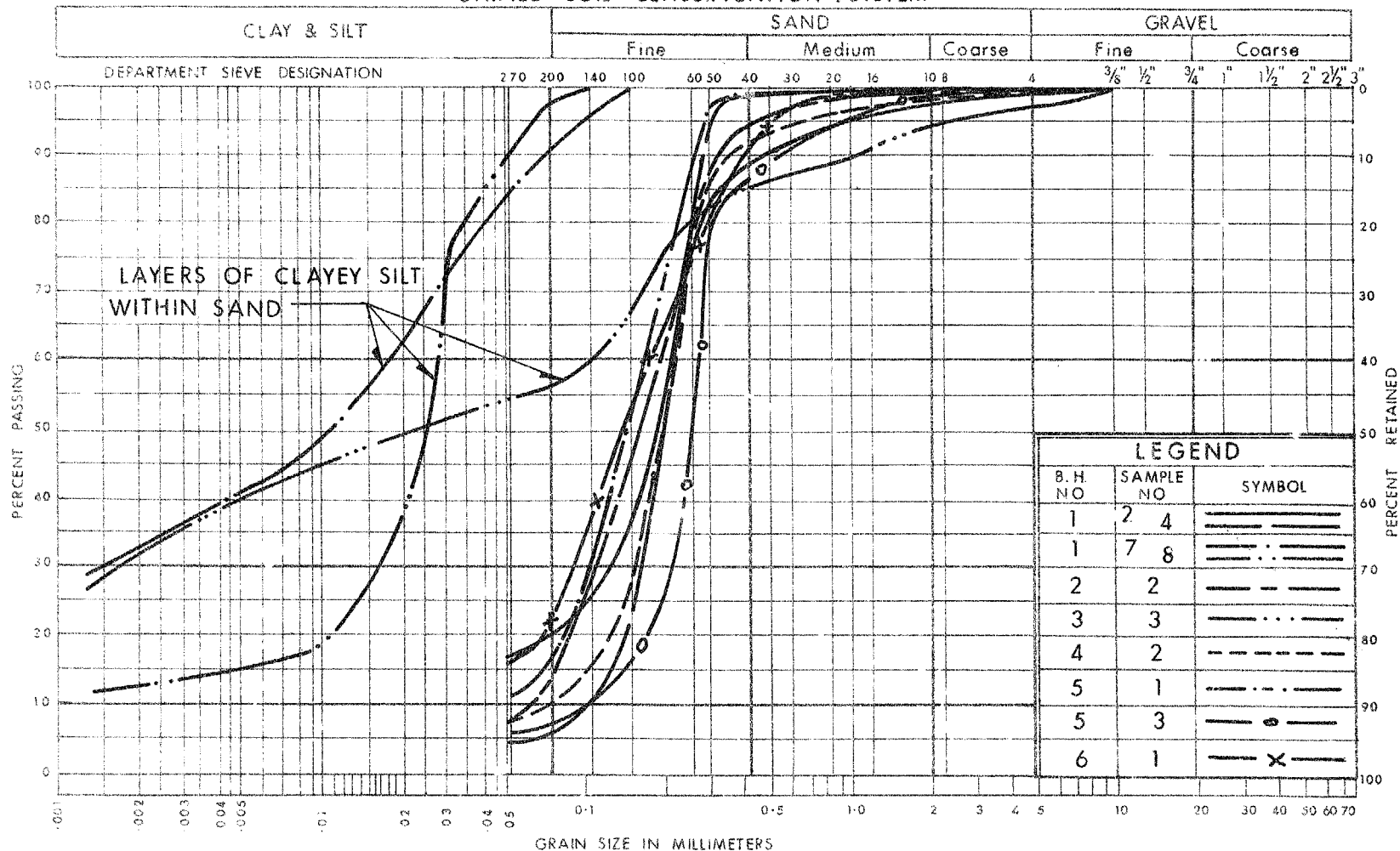


FIG. 3

UNIFIED SOIL CLASSIFICATION SYSTEM



DEPARTMENT OF HIGHWAYS
MATERIALS and
TESTING
DIVISION

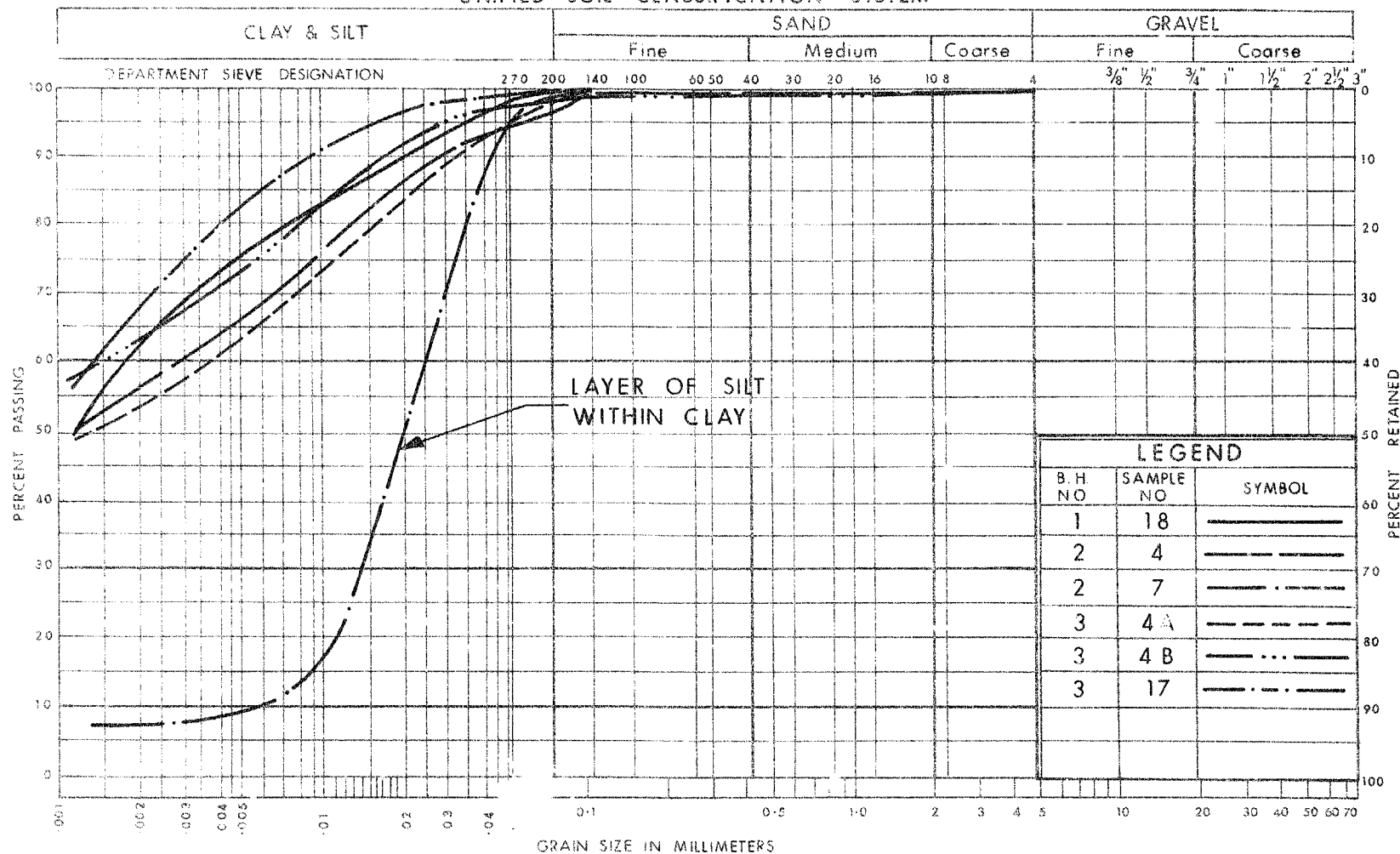
GRAIN SIZE DISTRIBUTION FINE SAND

W.P. No. 34-66-07

JOB No. 67-F-113

FIG. NO. 4

UNIFIED SOIL CLASSIFICATION SYSTEM



DEPARTMENT OF HIGHWAYS
MATERIALS and
TESTING
DIVISION

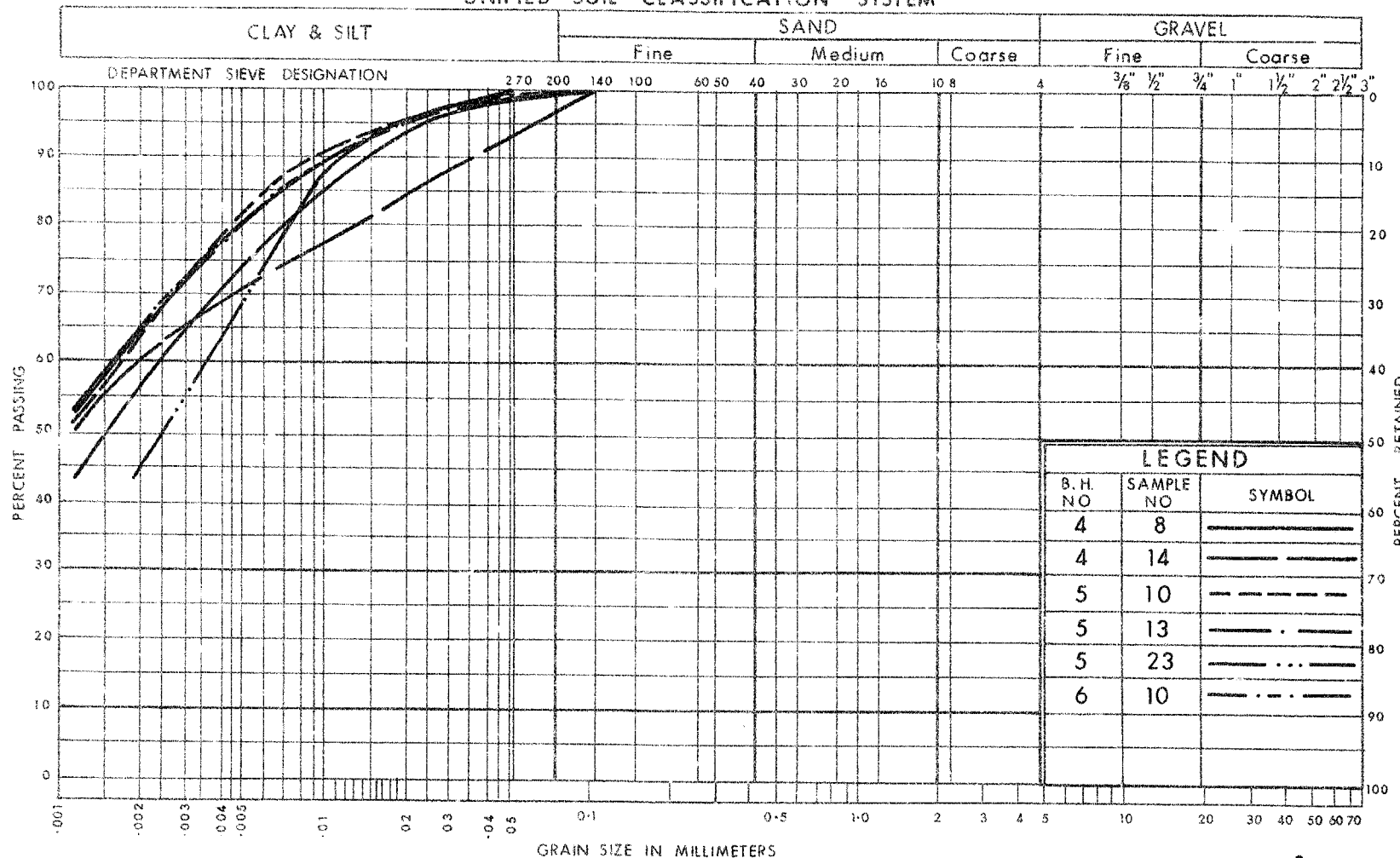
GRAIN SIZE DISTRIBUTION SENSITIVE CLAY

W.P. No. 34-66-07

JOB No. 67-F-113

FIG. NO. 5

UNIFIED SOIL CLASSIFICATION SYSTEM



DEPARTMENT OF HIGHWAYS
MATERIALS and
TESTING
DIVISION

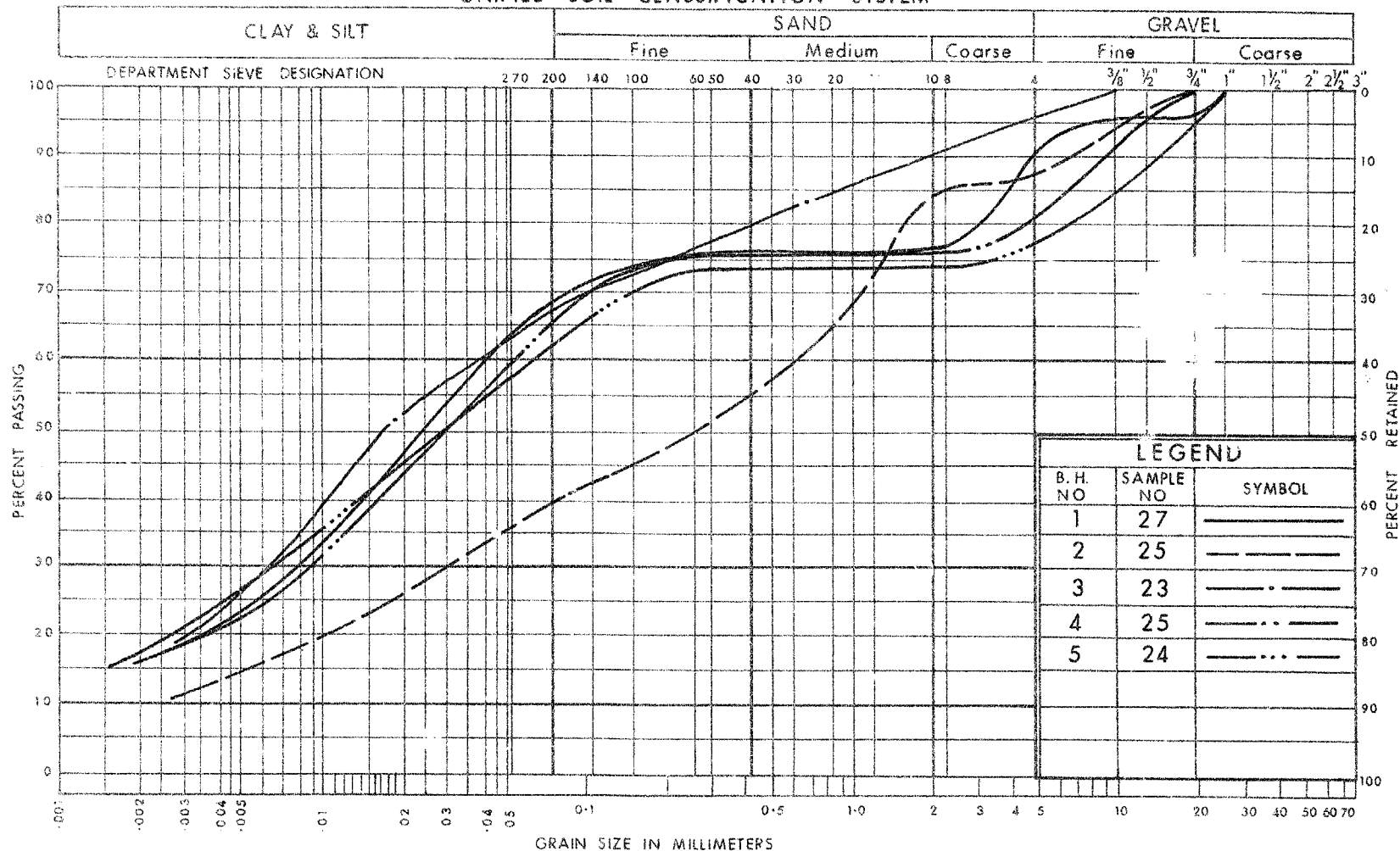
GRAIN SIZE DISTRIBUTION
SENSITIVE CLAY

W.P. No. 34-66-07

JOB No. 67-F-113

FIG. NO. 6

UNIFIED SOIL CLASSIFICATION SYSTEM



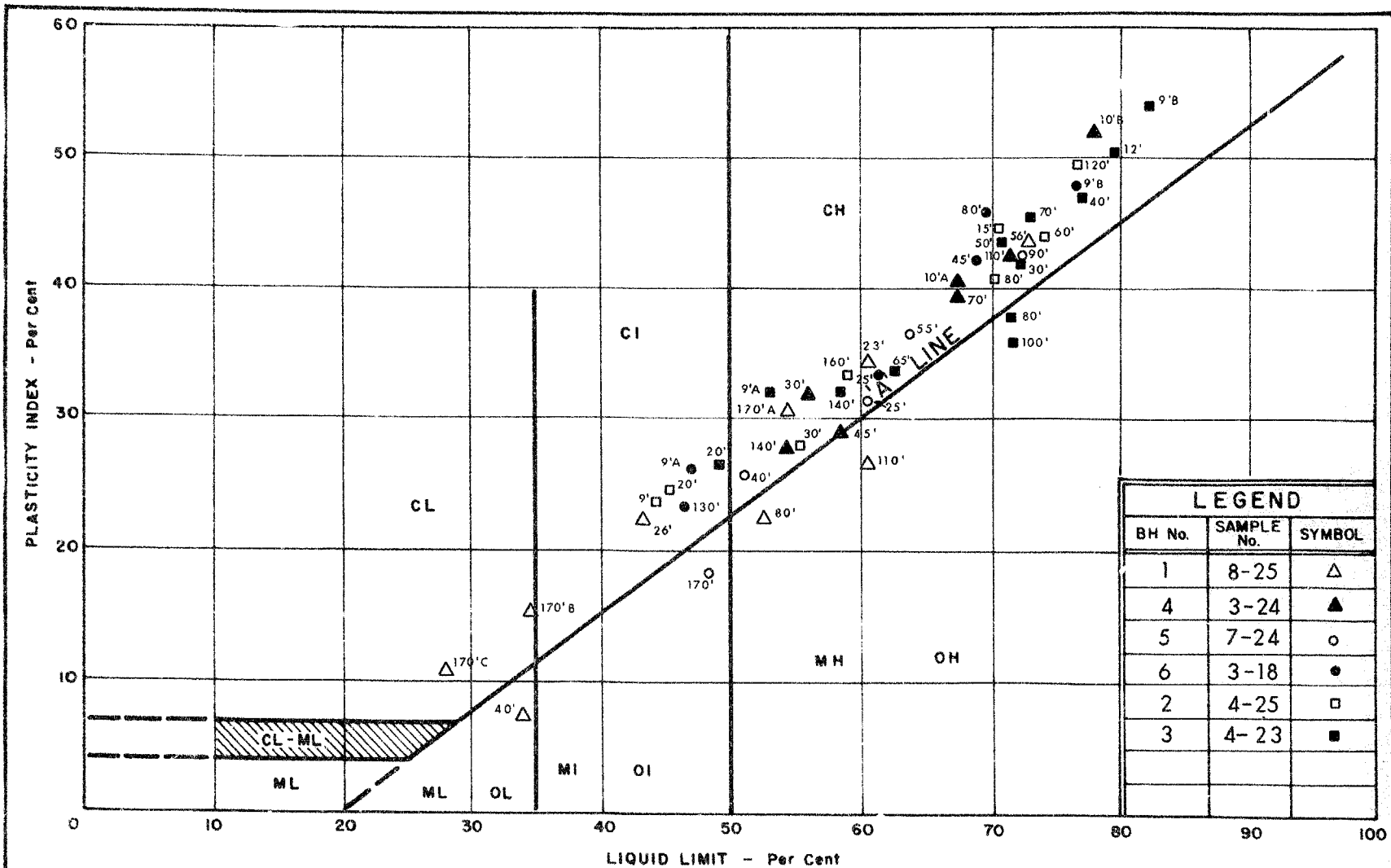
DEPARTMENT OF HIGHWAYS
MATERIALS and
TESTING
DIVISION

GRAIN SIZE DISTRIBUTION
CLAYEY SILT WITH SAND & GRAVEL
(Glacial Till)

W.P. No. 34-66-07

JOB No. 67-F-113

FIG. NO. 7



LEGEND		
BH No.	SAMPLE No.	SYMBOL
1	8-25	△
4	3-24	▲
5	7-24	○
6	3-18	●
2	4-25	□
3	4-23	■



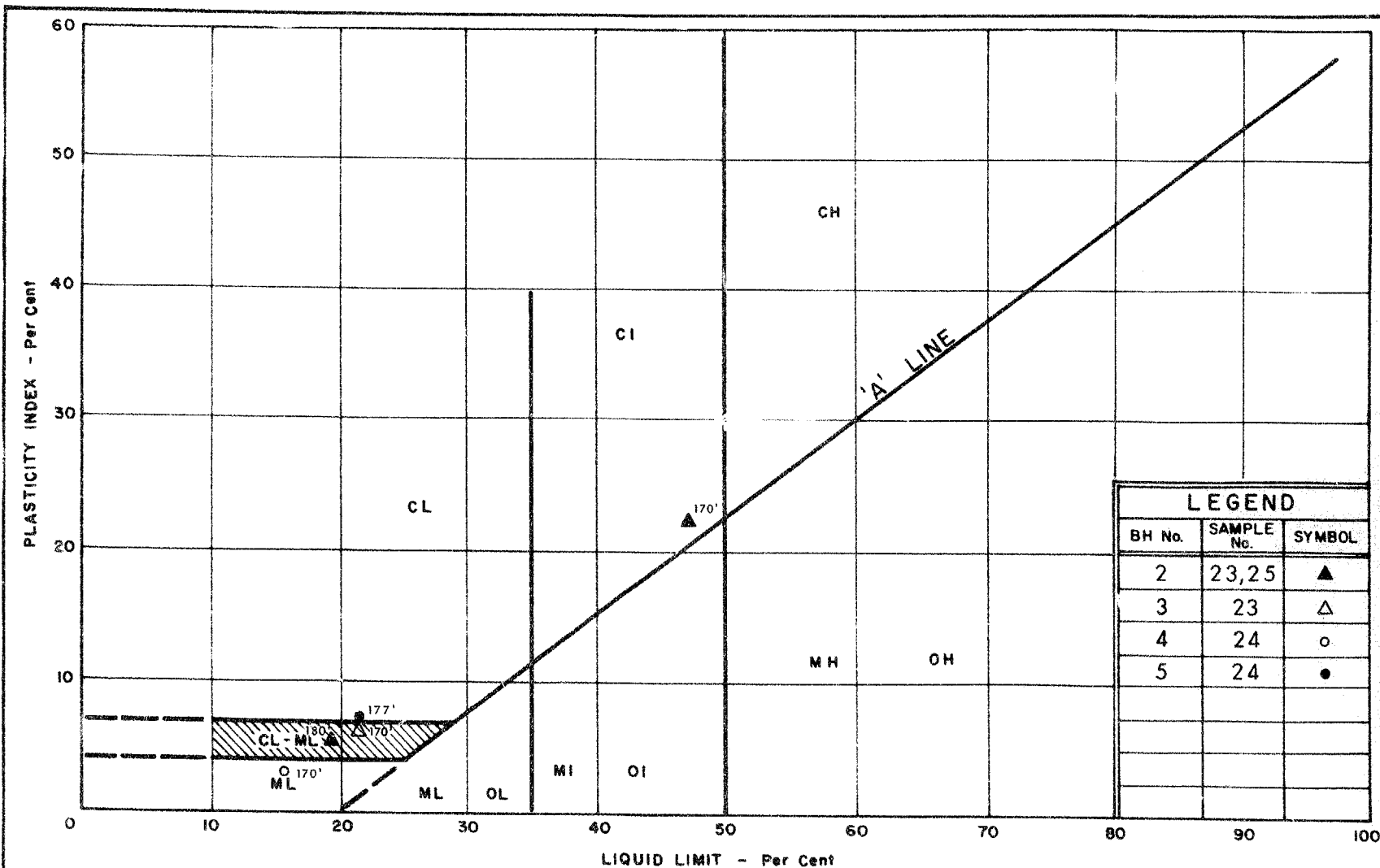
DEPARTMENT OF HIGHWAYS
MATERIALS and
TESTING
DIVISION

PLASTICITY CHART SENSITIVE CLAY

W.P. No. 34-66-07

JOB No. 67-F-113

FIG. NO. 8



DEPARTMENT OF HIGHWAYS
MATERIALS and
TESTING
DIVISION

PLASTICITY CHART CLAYEY SILT WITH SAND & GRAVEL (Glacial Till)

W.P. No. 34-66-07

JOB No. 67-F-113

FIG. NO. 9

VOID RATIO vs PRESSURE

$W_L = 45.6$
 $W_p = 20.8$
 $W = 65.1\%$
 $C_c = 1.36$

BORE HOLE 2
 SAMPLE 7
 DEPTH 21'-3"
 ELEV. 240

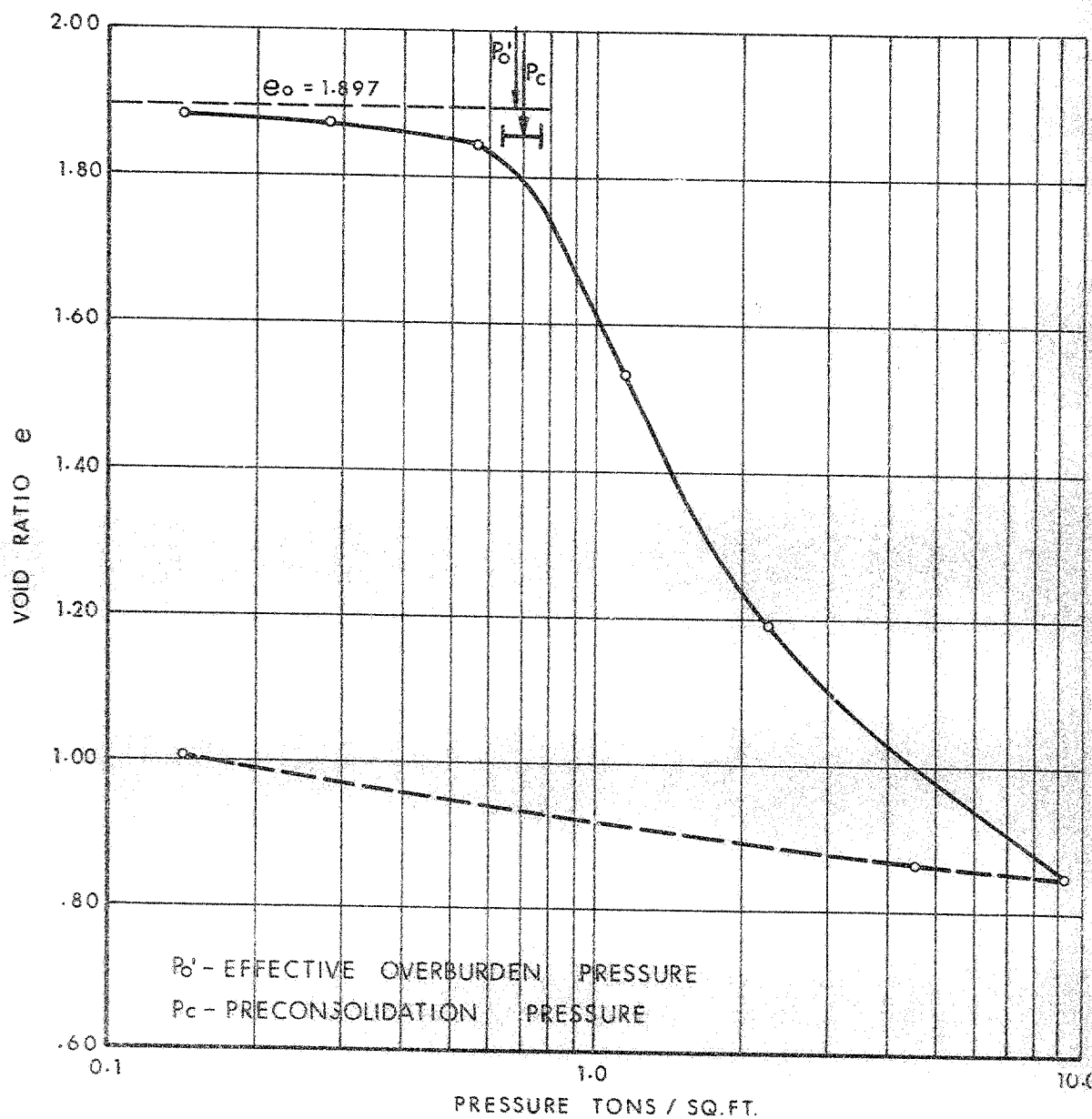


FIG. 10

VOID RATIO vs PRESSURE

$W_L = 56.0$
 $W_p = 24.3$
 $W = 71.4\%$
 $C_c = 1.29$

BORE HOLE 4
 SAMPLE 8
 DEPTH 31'-2"
 ELEV. 229

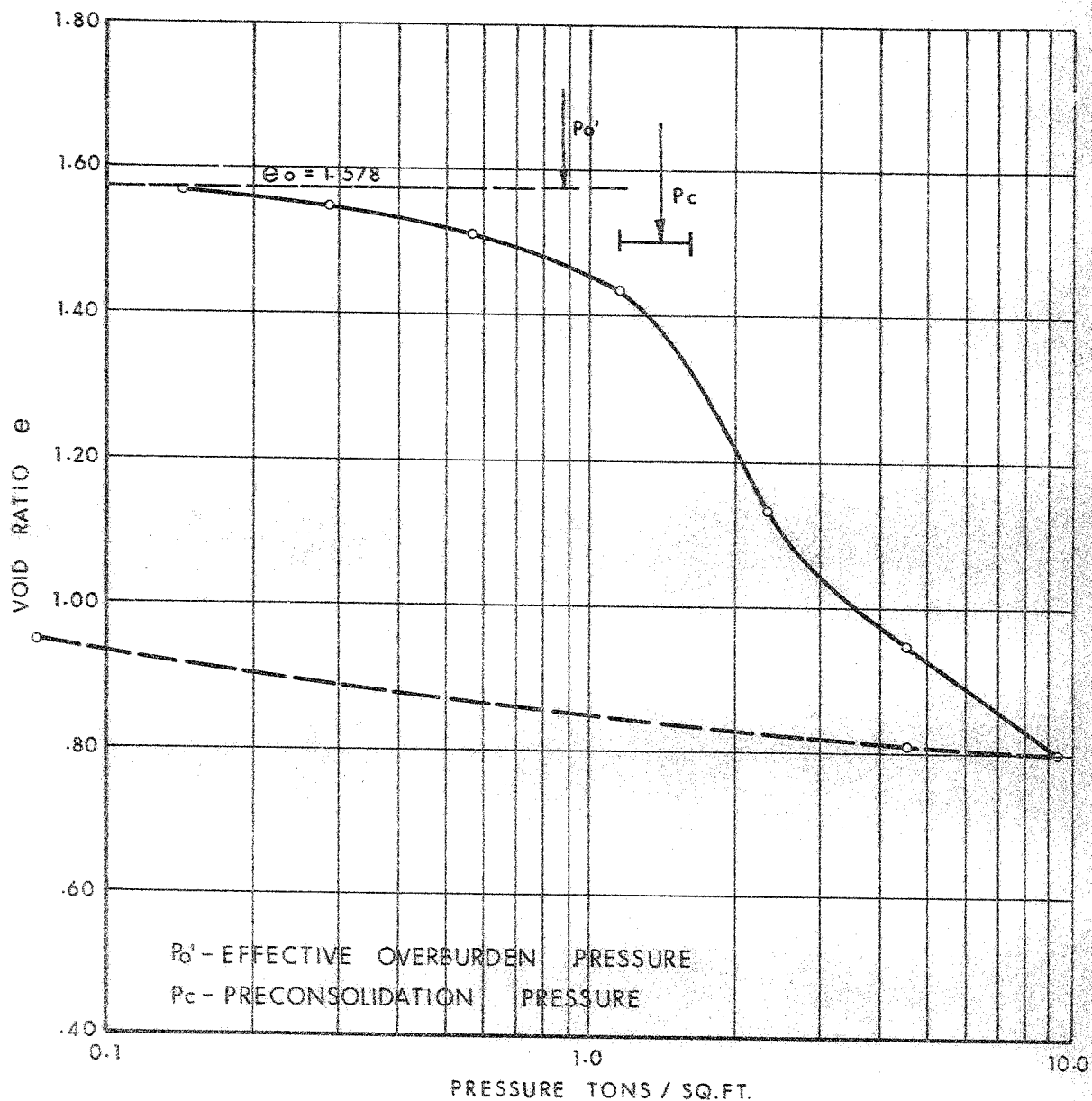


FIG. 11

VOID RATIO vs PRESSURE

$W_L = 67.1$

$W_p = 27.7$

$W = 79$

$C_c = 2.68$

BORE HOLE 4

SAMPLE 14

DEPTH 71'-3"

ELEV. 189

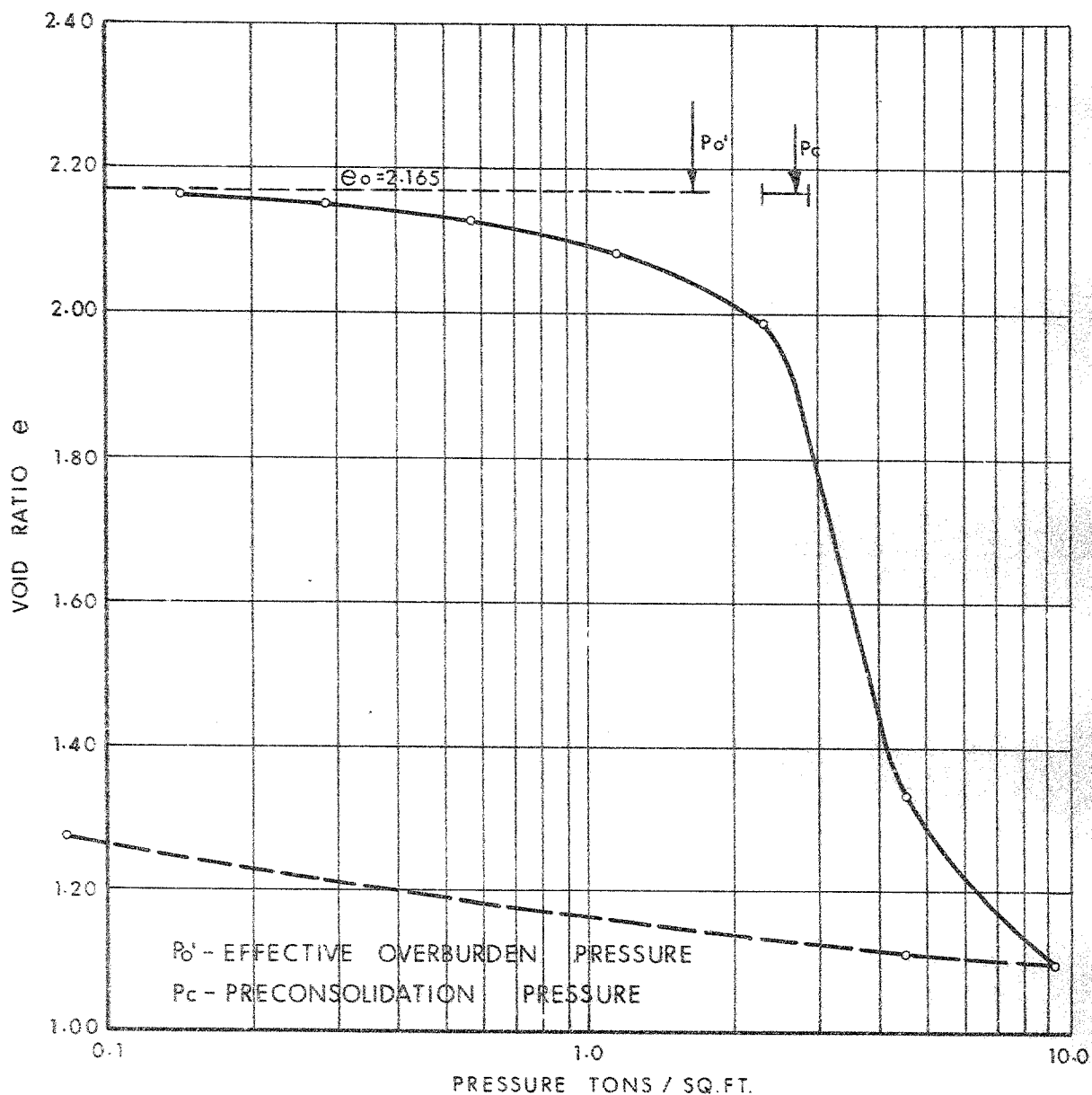


FIG. 12

VOID RATIO vs PRESSURE

$W_L = 76.5$
 $W_p = 27.2$
 $W = 61.5\%$
 $C_c = 1.96$

BORE HOLE 2
 SAMPLE 19
 DEPTH 120'-5"
 ELEV. 140

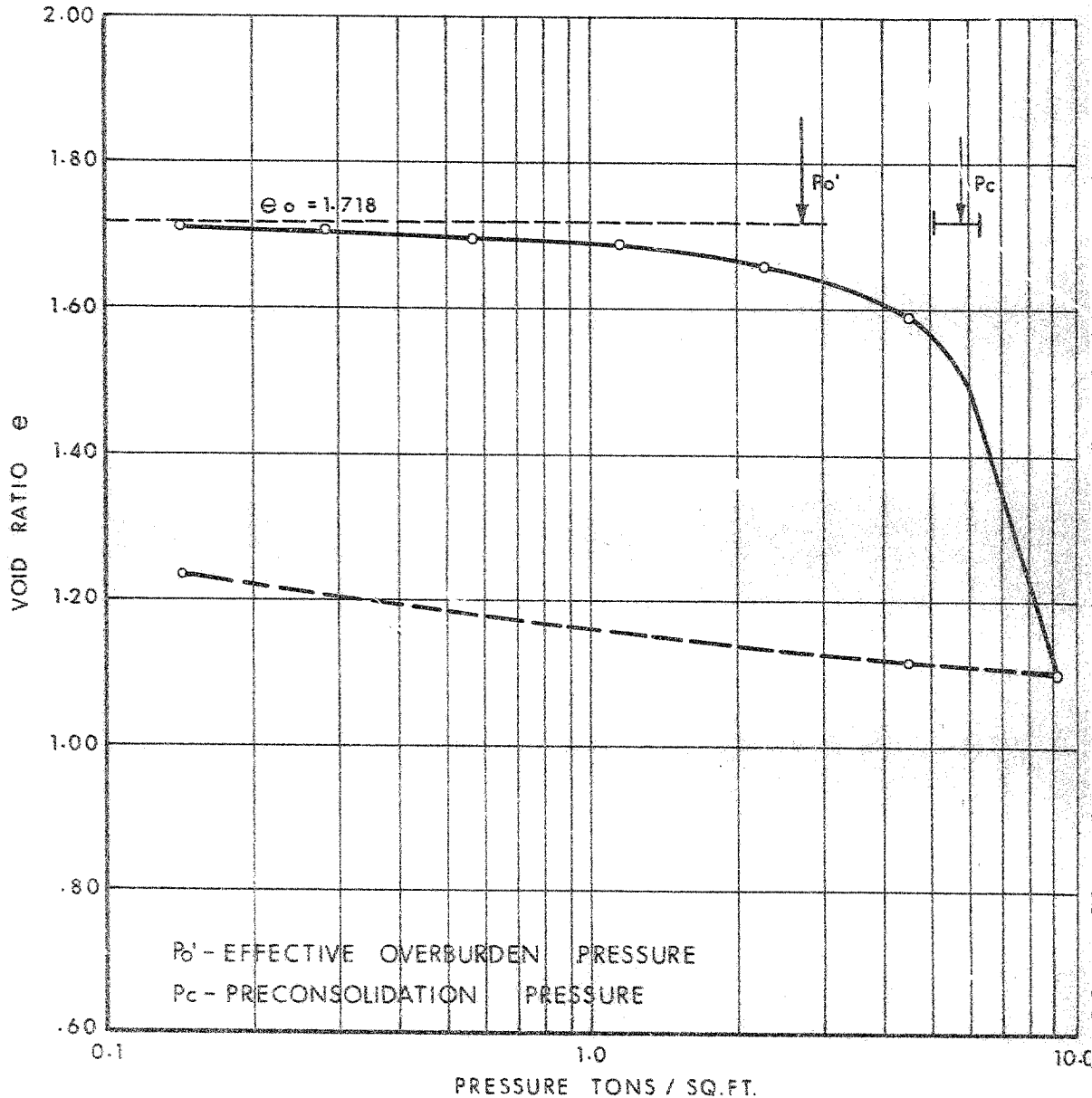


FIG. 13

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 _u	UNCONFINED COMPRESSION	L.V	LABORATORY VANE
Q	UNDRAINED TRIAXIAL	F.V	FIELD VANE
Q _{cu}	CONSOLIDATED UNDRAINED TRIAXIAL	C	CONSOLIDATION
Q _d	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
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_t	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
ρ	ANGLE OF SLOPE TO HORIZONTAL

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

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

April 25, 1969

-- 7th Line Road Underpass --
Highway 417, District #9 (Ottawa)
W.J. 67-F-113 -- W.P. 34-66-07

We have reviewed the preliminary bridge design drawings No. D-6486-1 to 4, inclusive, pertaining to the above structure; the following comments are submitted:

- 1) The end-bearing steel H-piles supporting the abutments will be subjected to some negative skin frictional forces due to the 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.
- 2) In our memo of November 14, 1968, we have suggested that the approach embankment in the immediate vicinity of the structure be surcharged. These details are not shown on the aforementioned drawings; however, we believe the Road Design drawings will include the necessary surcharge details.

AO/MieP

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

cc: Messrs. S. McCombie
G. Scott
J. E. Gruspler
S. J. Markiewicz
Foundations Files
Gen. Files

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

2.
November 14, 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.

In view of the significant change in alignment of the Seventh Line Rd. at the crossing of Hwy. #417, this Section will carry out additional boreholes prior to the design of the structure foundations.

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

Copy for the information of
MR. A. G. STERMAC

Mr. A. G. Stermac,
Principal Foundation Engineer,
Laboratory Building,
LOWNSVIEW, Ontario.

Bridge Division,
Kingston, Ontario.

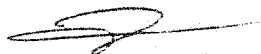
September 28, 1967.

W. P. 34-66-070, Site , Seventh Line Rd.
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-4653-1 for E. B. L. structure and E-4647-1 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.

Not approved fully for stability



J. A. Fisher

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

JAF/GS/NJ

Original letter & 1 copy of Plans E-4653-1, E-4647-1 to:
Bridge Office Files Section - (Mr. S. McCombie)

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

67-E-113

1967 OCT 3 AM 9:22

0002

DOWN KINR 1 OCT 3/67 9.25 AM

A G STERMAC PRINCIPAL FND ENGR

RE ANDERSON RD 7TH LINE RD

PROPOSED HWY NO. 417 DIST NO. 9

PRELIMINARY STRUCTURE SITE REPORT. PLEASE INVESTIGATE APPROACH
FILLS FOR STABILITY.

J A FISHER BRIDGE OFFICE

JS

67-113

T
E
L
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T
Y
P
E



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

Consulting Foundation Engineer
for A. J. Stelmach
Principal Foundation Engineer

cc: H. Koning
Foundation File (2) / 10
General File

MEMORANDUM

To: Mr. A. G. Stermac, P. Eng.,
Principal Foundation Engineer,
Laboratory Building,
Downsview, Ontario.

FROM: Bridge Division,
Kingston, Ontario.

DATE: February 20, 1968.

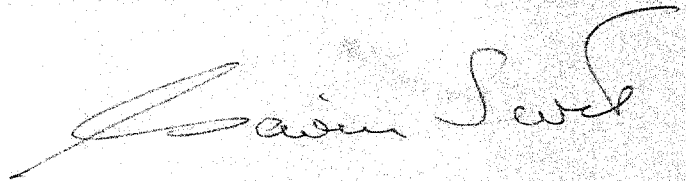
OUR FILE REF.

IN REPLY TO

SUBJECT: W.P. 34-66-070, Site 3-268, Seventh Line Road,
Proposed Highway 417, District 9

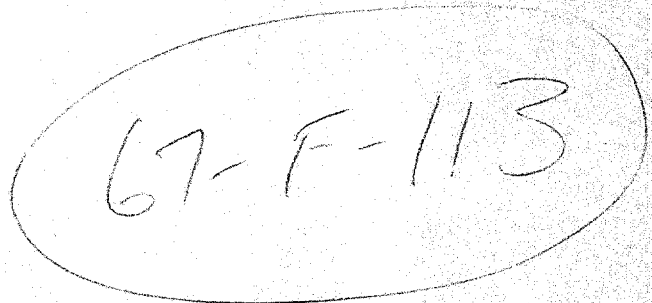
Further to our letter of September 25, 1967, we are sending to you herewith a print of Plan E-4647-1 giving the bridge site for the westbound lane structure.

Please note that this plan has been redrawn so as to give the north point towards the top of the sheet. We would be pleased if you will utilize this drawing in the preparation of your foundation report, which we anticipate receiving in due course.



Gavin Scott, P. Eng.
Regional Bridge Location Engineer

GS/hl
Encl.
c.c. (with copy of Plan E-4647-1)
Bridge Office Files Section



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

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

Attention: Mr. S. McComble

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.

WJS/mjsf

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

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

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

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

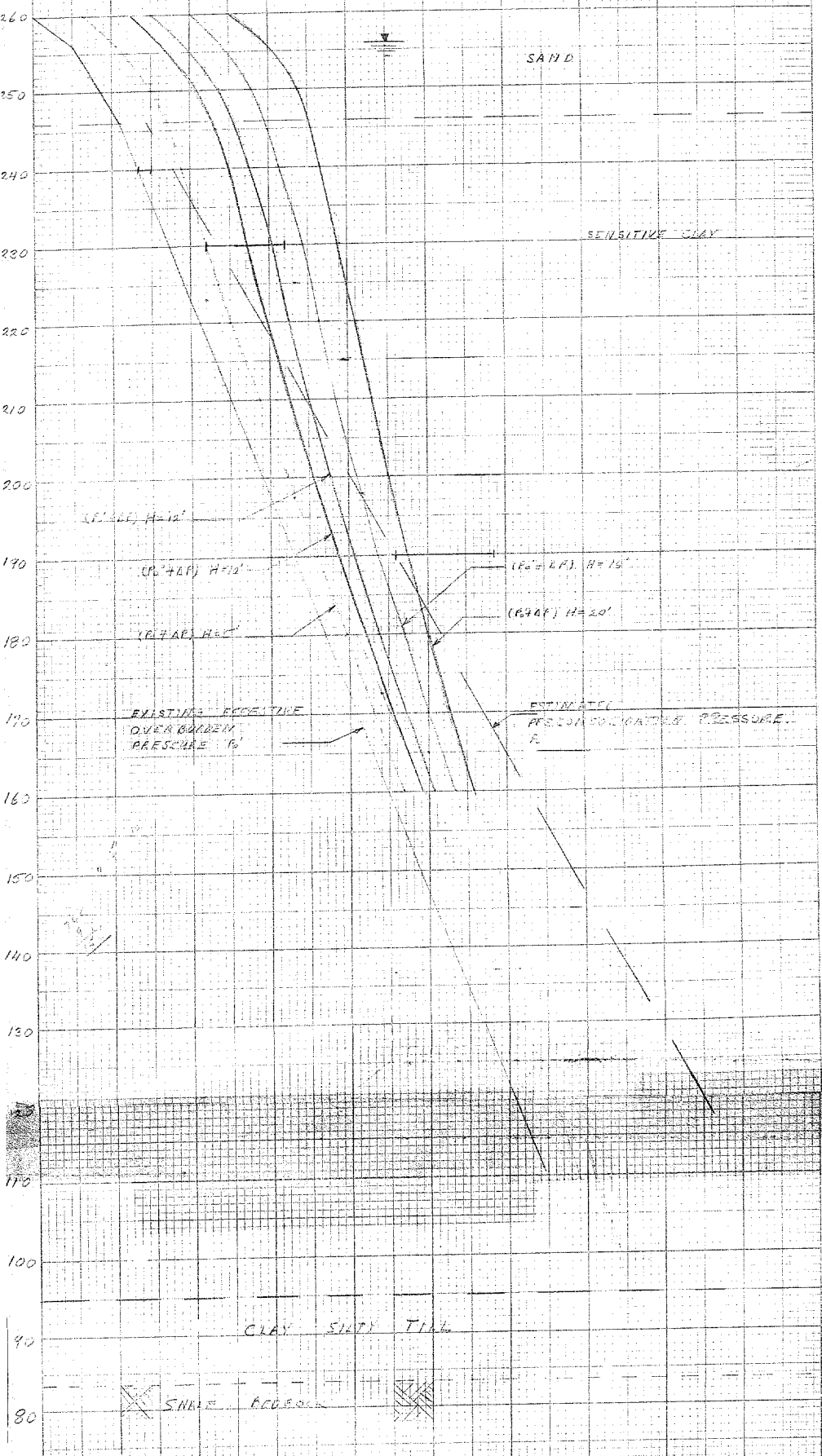
BTD/MdeF

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

cc: Gen. Files W.P. 34-66-07
 RE: Preliminary Drawing & Recommendations
 Lined U.R.S.S.

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 14, 1968

OUR FILE REF:

IN REPLY TO

SUBJECT:

7th Line Road Underpass - Site 3-268
 W.P. 34-66-07 -- W.J. 67-F-112
 Hwy. #417 -- District No. 9 (Ottawa)

We have reviewed the Preliminary Drawing D-6486-P1 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	--	14 ft.
Factor of safety against base failure	--	<u>1.4</u>
Factor of safety with a 6-ft. surcharge for the above case	--	<u>1.03</u>

Settlements:

Height of Fill	Total Settlement in Inches for Various Periods				
	2 Yrs.	7 Yrs.	15 Yrs.	25 Yrs.	50 Yrs.
14 ft. (Design height)	7"	9"	12"	17"	23"
20 ft. (Design height + 6-ft. surcharge)	10"	12"	19"	24"	31"

(Percentage Consolidation) -

15% 25% 38% 45% 60%

cont'd. /2 ...

W.P. 67-F-113
Hwy 417 AND
SEVENTH LINE RD.

EBL
CURVE DATA
- 3° 55' 30"
- 3° 12' 00"
- 0° 15' LT
- 229.12
- 1320.00
- 3.54
- 0° 15' 45"
- 250.00
- 910.5

ROGER GOYETTE
DENISE GOYETTE
JOINT TENANTS
INST. N° 72242

WILLI FALLER
INST. N° 73666

REGINA LAPLANTE
INST. N° 58022
HENK LUEI
INST. N° 64

OSCAR BARNABE
ROSE ANNA BARNABE
INST. N° 72382
RENÉ DESJARDINS
INST. N° 72192

RUDOLF POHL
INST. N° 64437

J.T. REJE
JEANNIN
INST. N°

JOHN F.C. BRAITHWAITE
INST. N° 73127

HAROLD STUART
INST. N° 77359

STR W.P. 34 64-67
E-4647-1 W.B.L.
E-4653-1 E.B.L.

MORRIS KATZ
GOPHIE KATZ

LLOYD KETTLES SHARP
INST. N° 78130

NELLIE SIMMER
INST. N° 73731

HENRY ARNOLD
ANN ARNOLD
JOINT TENANTS
INST. N° 64439

339+42.18
EBL

342+20 E.B.L.
20+100 temp. cong.

FR 67-377

PI 31+86.45
PROP. REV. N.L.

PI 31+75.25

PI 31+75.25

PI 31+75.25

PI 31+75.25

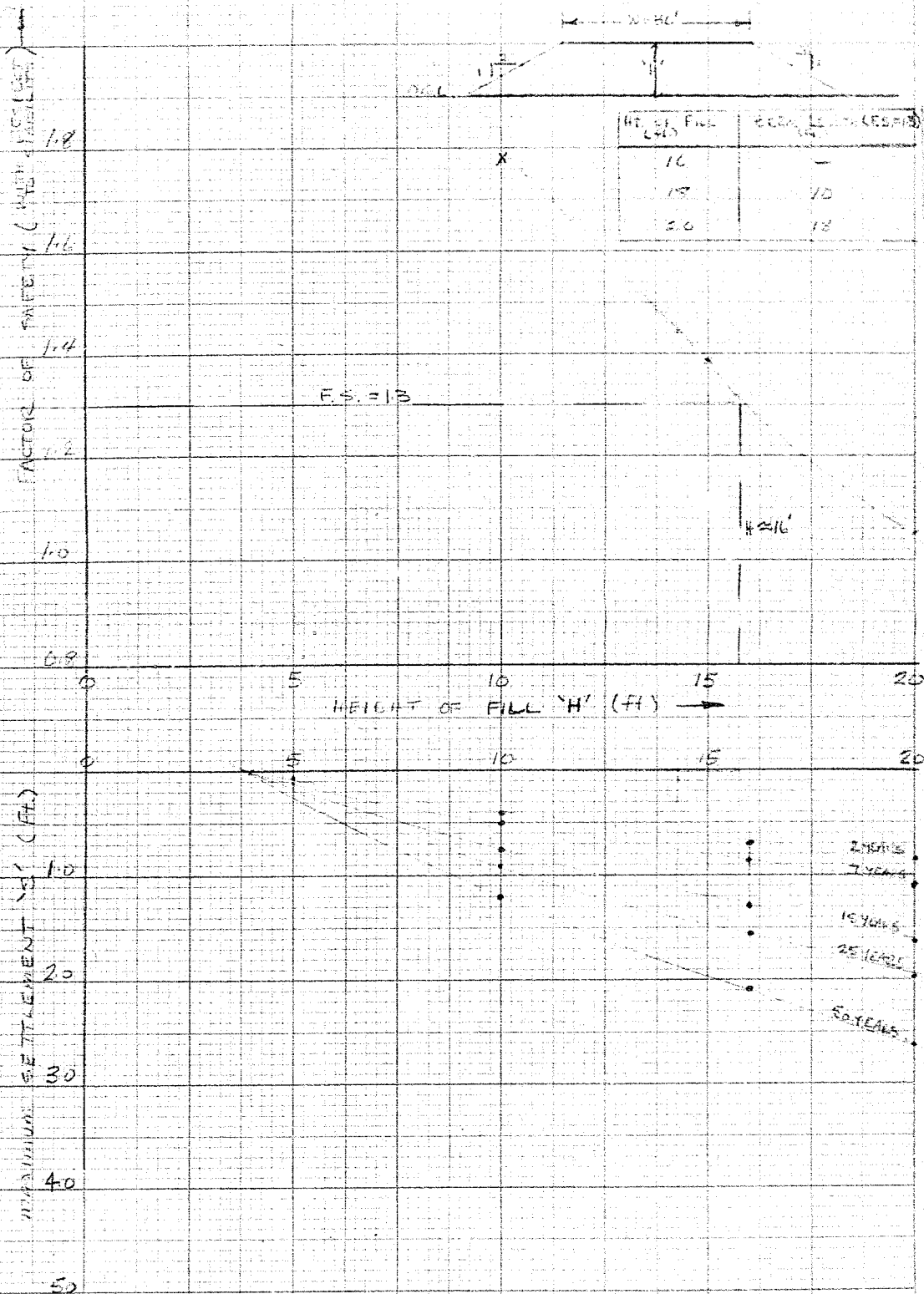
PI 31+75.25

PI 31+75.25

PI 31+75.25

PI 31+75.25

SUMMARIZED RESULTS OF STABILITY AND SETTLEMENT ANALYSES (APPROACH - LORAIN 1950)



Department of Highways Ontario

Copy for the information of
~~Mr. A. Stermac~~

~~G. Scott, Kingston Region~~
~~C.R. Robertson, Ottawa District~~
~~L. Forster, Kingston Region~~
~~S. Martkiewicz, Kingston Region~~

Bridge Division,
Downsview, Ontario

October 30, 1968

7th Line Road Underpass
H.P. 34-66-07, Site 3-268
Highway 417, District 9

67-F-118

Attached herewith are prints of the Preliminary Bridge Plan
Drawing B-6486-P1 for the above-mentioned structure.

The estimated cost of the proposed structure is \$462,000.
This cost includes tender, materials, engineering and sundry
construction.

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

CSG:rd

C.S. Grebaki,
Bridge Design Engineer

Attach.

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

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,
W.P. 34-66-07 - 7th Line Road Underpass, ✓ 67-F-113
W.P. 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

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 Rd. 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.

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. R. Robertson
J. L. Forster
C. S. Grebski
J. E. Gruspier
G. Scott

Foundations Files ✓
Gen. Files

MEMORANDUM

To: Mr. A. Stermac,
Principal Foundation Engineer,
DOWNSVIEW, 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. Gruspier and G. Scott

1968 FEB 7 AM 8:51

00021

K

KINR DOWN 1 FEB 7/68 8.42A VR
M J MACMASTER SR PROJECT DESIGN ENGR

FURTHER TO YOUR T T WE ARE GIVING THE SAFE NIGHTS 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

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1969 FEB 5 PM 1:59

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69-F-113

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 IT 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. Surf.
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	34-66-16	Bear Brook Tributary	18'	above valley floor elev of 229. Ht. ^{should} may be reduced if fill is placed on ground above the valley floor.

CM Feb 6/67

EQUATES TO THE

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-113 W.P. 34-66-06 - Carleton Cty. Rd. 27 Interchange *and also*
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 #17 Structures.

1. Eighth Line Lch - WP 34-66-08
 Drawg. D-6480-P1

a) Ht of Fill East Approach scales on Drawg 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

Ramsay Creek - WP 34-66-10
 Drawg D-6575-P1

NO Comments

3 Anderson Rd.

WP 34-66-06

Dwg D-6484-P1

67 f 71

- 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 road 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 $12\frac{3}{4}$ " 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.

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

Mr. A. Stermac

67-F-113

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
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

Foundation Section

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

C.S. Grebski,
Bridge Office

March 31, 1969

7th Line Road Underpass
W.P. 34-66-07, Site 3-263
Highway 417, District 9

67-1-113

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

C.S. Grebski,
Bridge Design Engineer

Attach.

c.c. Foundation Section

- 1) *Copy of the last 2, ~ 14 high (max).*
to be left as
2.0 (in 50 years).
the bridge is to be built prior to
construction
25% reduction
2) Bank Fills - 1 ft. short section skin friction loading
due to approach settlement caused by loading
approach on the bank.
Reduction - should be applied to the 2.0 (in 50 years) reduced from 2.0 to 1.0

Department of Highways Ontario

Copy for the information of

Foundation Section

Mr. A. Starnac,
Principal Foundation Engineer,
Room 107, Lab. Building

C.S. Grebski,
Bridge Office

April 9, 1969

7th Line Road Underpass
W.P. 34-66-07, Site 3-268
Highway 417, District 9

67-F-113

Attached herewith we are submitting the final bridge drawings which show the foundation design for this structure. The slope has been revised from 2:1 to 3:1.

Kindly give us your comments at your earliest convenience.

CSG:rd
Attach.

C.S. Grebski,
Bridge Design Engineer

c.c. Foundation Section

Alf

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

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

April 25, 1969

-- 7th Line Road Underpass --
Highway 417, District #9 (Ottawa)
W.J. 67-P-113 -- W.P. 34-66-07

We have reviewed the preliminary bridge design drawings No. D-6486-1 to 4, inclusive, pertaining to the above structure; the following comments are submitted:

- 1) The end-bearing steel H-piles supporting the abutments will be subjected to some negative skin frictional forces due to the 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.
- 2) In our memo of November 14, 1968, we have suggested that the approach embankment in the immediate vicinity of the structure be surcharged. These details are not shown on the aforementioned drawings; however, we believe the Road Design drawings will include the necessary surcharge details.

RD/MLP

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

cc: Messrs. S. McCombie
G. Scott
J. E. Gruspler
S. J. Markiewicz

Foundations Files
Gen. Files

MEMORANDUM

68-F-113

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:

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:

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
67F-113 Pipe at <u>7th Line</u> and Prop. Hwy. #417	Elev. 262.285
Pipe at 3th 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.

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

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

February 28th, 1974.

RE: Approach Slab Construction, ⁶⁷⁻¹¹⁻¹¹³
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-113

File Please

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

Structural Planning Office,
Kingston, Ontario.

Mr. K. Barai

4 March 1974.

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

Please find attached a copy of letter dated February 28, 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.

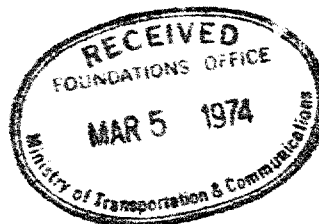
Please note that in the penultimate paragraph of Mr. Payer's letter he states the opinion that it may be satisfactory to incorporate approach slabs at the 7th Line Road structure but that approach slabs for the other two structures should be delayed. I therefore think that we should proceed on the basis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

TCR/hl
att.

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



DEFECTS IN NEGATIVE DUE TO
CONDITION OF ORIGINAL DOCUMENT

#67-F-113

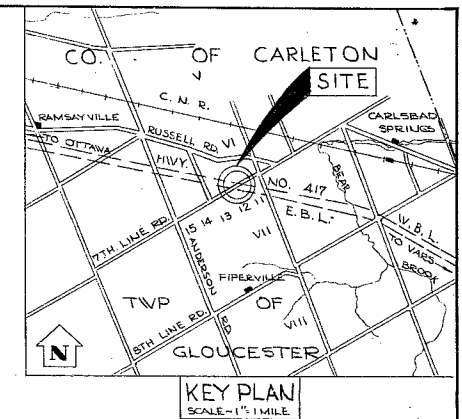
W.P. #34-66-07

HWY #417

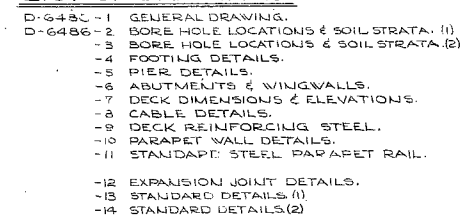
SEVENTH LINE

ROAD


REVISION



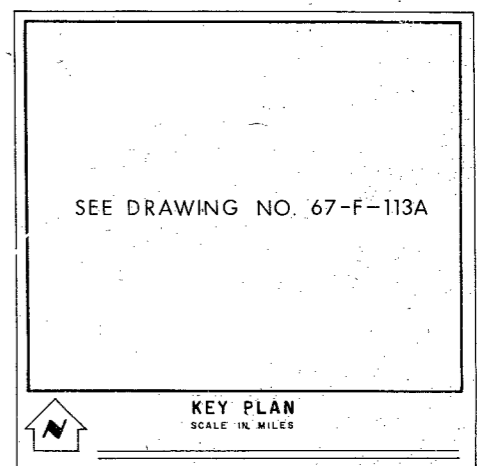
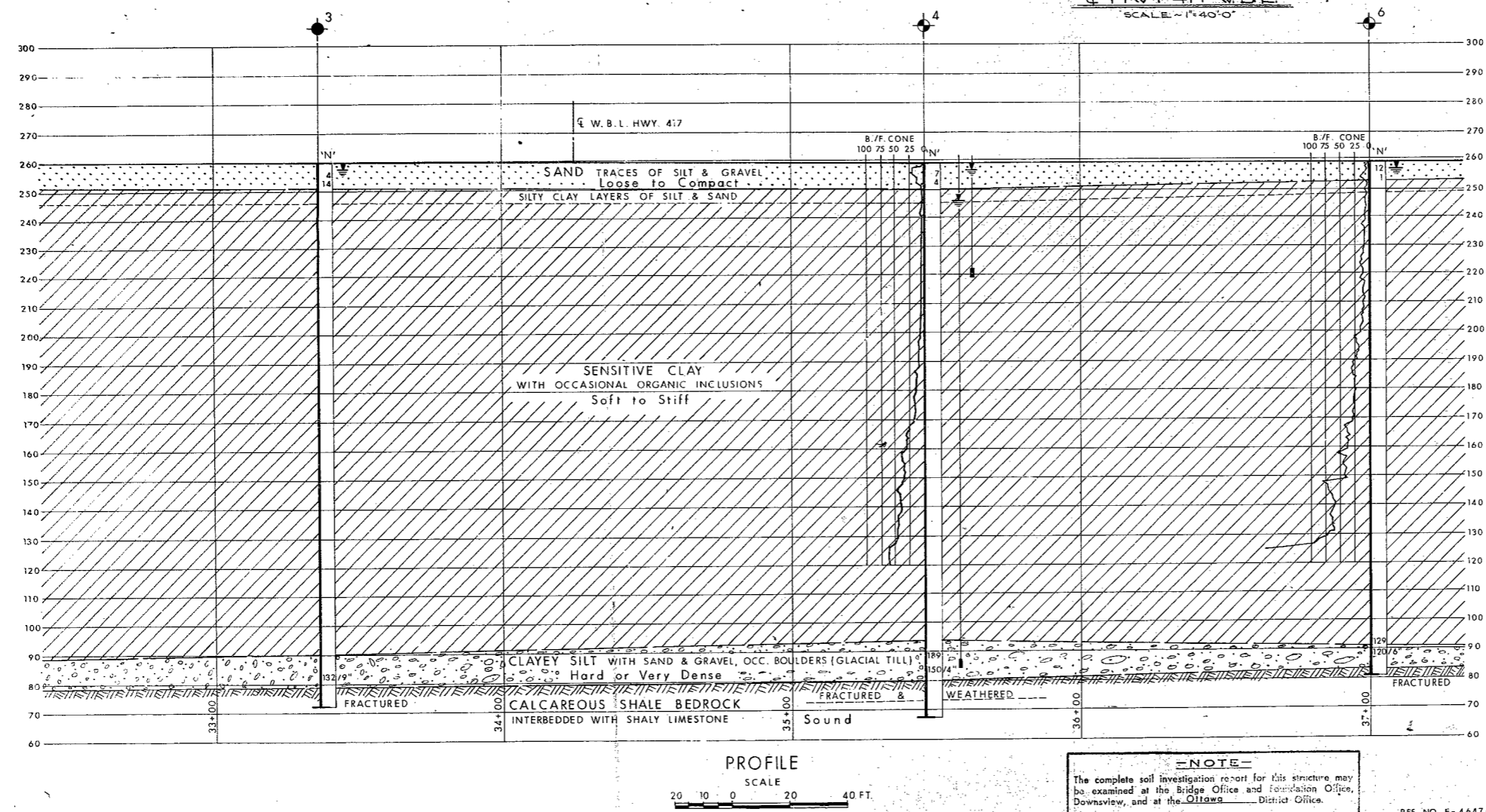
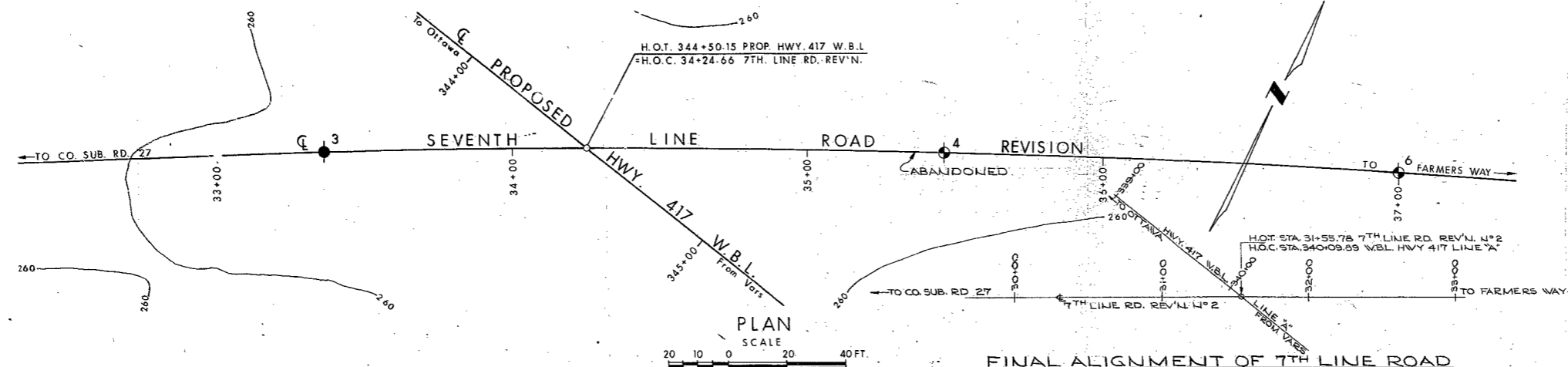
<u>CLEAR COVER ON REINFORCING STEEL</u>	
FOOTINGS, ABUTMENTS AND PIERS	- 3"
CURBS	- 2"
DECK	TOP-2", BOTTOM -1½"



DEPARTMENT OF HIGHWAYS ONTARIO BRIDGE DIVISION		
67-15-113		
7 TH LINE ROAD UNDERPASS		
KING'S HIGHWAY No. 417	DIST. No. 3	
CO. CARLETON	TWP. GLOUCESTER	
LOT 12	CON. <u>VI</u>	

APPROVED 				SITE No. 3-268		W.P. No. 34-66-07	
BRIDGE ENGINEER				CONTRACT No. <input type="text"/>			
DESIGN	R.K.	CHECK	D.R.G.				
DRAWING	D.H.B.	CHECK	R.K.	DRAWING No.		D-6486-1	
DATE	MARCH 60	LOADING	11520-44				

[illegible]



LEGEND

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

NO.	ELEVATION	STATION	OFFSET
3	260.0	33+36	€
4	259.0	35+46	€
6	259.0	37+00	€

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.

PRINT RECORD

NO.	FOR	DATE
1	OS	9/6/68

NOTE
The complete soil investigation report for this structure may be examined at the Bridge Office and Foundation Office, Downsview, and at the Ottawa District Office.

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

SEVENTH LINE ROAD REVISION

KING'S HIGHWAY NO. 417 W.B.L. DIST. NO. 9
CO. CARLETON
TWP. GLOUCESTER LOT 12 CON. VI

BORE HOLE LOCATIONS & SOIL STRATA

SUBM'D. B.D.	CHECKED <i>ML</i>	W.P. NO. 34-66-07	M.B.T. DRAWING NO.
DRAWN G.P.	CHECKED <i>ML</i>	JOB NO. 67-F-113	67-F-113B
DATE APRIL 8, 1968	SITE NO. 3-268	BRIDGE DRAWING NO.	D6486-3
APPROVED <i>AS Thomas</i>	CONT. NO.		

