GEN. FILES

DEPARTMENT OF HIGHWAYS ONTARIO

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MEMORANDUM

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

Attention Mr. S. McCombie

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

Date: April 11, 1968

Our File Ref.

In reply to APR 16 1968

Subject:

FOUNDATION INVESTIGATION REPORT
For
Eastbound Lane and Westbound Lane
Structures at the Crossing of
Anderson Rd. and Proposed Hwy. #417
District No. 9 (Ottawa)
W.J. 67-F-112 -- W.P. 34-66-06

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 ...

Attn: Mr. S. McCombie April 11, 1968

It would appear to us that probably the best solution for this crossing is an overpass structure, with Anderson Road 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.

AGStermac

A. G. Stermac
PRINCIPAL FOUNDATION ENGINEER

AGS/MdeF
Attach.

cc: Messrs. B. R. Davis (2)
H. A. Tregaskes
D. W. Farren
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C. R. Robertson
G. Scott
J. E. Gruspier
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TABLE OF CONTENTS

FOUNDATION INVESTIGATION REPORT
For
Eastbound Lane and Westbound Lane
Structures at the Crossing of
Anderson Rd. and Proposed Hwy. #417
District No. 9 (Ottawa)
W.J. 67-F-112 -- W.P. 34-66-06

1. INTRODUCTION:

The Foundation Section was requested to carry out an investigation at the proposed crossing of Anderson Rd. and Hwy. #417; the site is located some 11 miles south-east of Ottawa, in the Twp. of Gloucester, County of Carleton. The request was contained in a memo from the Kingston Bridge Location Section (Mr. G. Scott, Regional Bridge Location Engineer), dated September 28, 1967. An investigation was subsequently carried out by this Section to determine the subsoil and groundwater conditions at this site.

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

2. DESCRIPTION OF THE SITE AND GEOLOGY:

The site is located on Anderson Rd. about one-half mile south of Russell Rd. The closest settlement of any size is Carlsbad Springs, which is about 4 miles to the east. Anderson Rd., in the vicinity of the site, is a paved roadway approximately 22 feet wide with the grade between elevations 260 and 265. Drainage ditches, some 4 to 5 feet deep and about 12 feet wide at the top, are located on either side of the roadway. The surrounding area, which is wooded, is quite flat-lying terrain.

cont'd. /2 ...

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

Physiographically, the site is situated on the northern edge of the area known as "The Russell and Prescott Sand Plains". In this area a sand mantle, some 10 to 15 feet in thickness, overlies an extensive deposit of marine clay. The sand is of deltaic origin built up by the Ottawa River and its Northern tributaries during the geologic period when the Champlain Sea inundated the area. The underlying clay, known locally as "Leda Clay", was deposited by the Champlain Sea. In the area the base of the clay extends below elevation 100. The clay stratum is underlain by a glacial till which, in turn, is underlain by grey and black shale bedrock of the Lorraine formation, Ordovician Period.

Most of the area lies within the drainage basin of the 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:

Seven sampled boreholes, 5 of which were accompanied by a dynamic cone penetration test, as well as three additional cone tests, were carried out during the course of the recent field investigation. The borings were advanced by means of a conventional diamond drill rig adapted for soil sampling purposes. A continuous flight power auger (Penn Drill) was used to drive the majority of the dynamic cone penetration tests.

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 effort to reduce the degree of disturbance, some of the Shelby tube samples were

cont'd. /3 ...

3. <u>FIELD AND LABORATORY WORK</u>: (cont'd.) ...

advanced by means of a piston technique. In addition, field vane tests were carried out to determine the undrained shear strength of the clay stratum. Bedrock was proven in 2 of the borings by obtaining AXT size rock core samples.

The groundwater level conditions across the site were determined by installing 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.

The locations and elevations of all the borings were surveyed in the field by personnel from the Kingston Regional Engineering Surveys Section, and are shown on Dwgs. #67-F-112A (E.B.L.) and 67-F-112B (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 inspection, laboratory tests were carried out on certain samples to determine the engineering properties of the various soil types, namely:

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

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

cont'd. /4 ...

3. <u>FIELD AND LABORATORY WORK</u>: (cont'd.) ...

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

4. SOIL TYPES AND SOIL CONDITIONS:

4.1) General:

The surficial deposit across the site is composed of a sand with random layers of silt and clayey silt t ughout; this deposit is some 6 to 13 ft. thick. The sand is underlain by the predominant overburden stratum across the site, composed of a firm to stiff sensitive clay about 180 to 196 feet in thickness.

Directly underlying the clay stratum is a deposit of glacial till composed primarily of very stiff to hard clayey silt with some sand and a trace of gravel. This deposit, in turn, is underlain by sound shale bedrock which was encountered at a depth of about 212 to 224 feet below ground surface.

The boundaries between the various deposits, as determined in the boreholes, are shown on the accompanying borehole sheets. The stratigraphical profile, shown on Drawings 67-F-112A and 67-F-112B, is based on this information.

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

4.2) Sand - Surficial Deposit:

A surficial deposit of sand was encountered at all the boring locations; the thickness of this deposit ranges from 6 feet to 13 feet. Occasional layers of silt and clayey silt, up to 3 feet thick, are randomly spaced throughout the deposit. At B.H. #4 the surficial deposit consists of a 6-foot thick layer of soft clayey silt with occasional layers of sand. Grain-size distribution curves for samples of the sand deposit are shown on Figure 5 in the Appendix of this report.

cont'd. /5 ...

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

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

The Atterberg limit tests, carried out on representative samples from the cohesive layers within the sand deposit, are summarized on Figure 1. These results indicate that the cohesive layers are inorganic and of low plasticity.

Standard penetration tests, carried out within the overall deposit, are plotted on the borelog sheets as well as on Figure 1. The results of this testing gave 'N' values which generally vary in a random fashion between 2 and 18 blows/ft. Based on these values, it is estimated that the relative density of the deposit varies from very loose to compact. The consistency of the thicker cohesive layers is estimated to range from soft to firm.

4.3) Sensitive Clay:

Directly underlying the surficial cover is the predominant overburden stratum across the site, a sensitive marine clay with occasional inclusions of organic matter. The thickness of this stratum ranges from 180 to 196 feet. The lower 10 to 25 feet of the clay is composed of random seams and layers of sand, silt and clayey silt with the thickness of the individual layers varying from a fraction of an inch up to 1 foot in thickness. Further, a 1-foot thick layer of silt was encountered at elevation 229 in B.H. #5, while in B.H. #6, a 4-foot thick layer of sand was penetrated at elevation 196. These granular layers were not found to be continuous across the site; it is inferred, therefore, that they are localized pockets. Grain-size distribution curves for samples of the clay and the lower layered zone are shown on Figures 6 and 7, respectively.

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. /6 ...

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

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

		Range	Average
Bulk Density (p.c.f.)	(γ)	93 - 110	97
Liquid Limit (%)	(W_L)	48 - 88	60
Plastic Limit (%)	(W_P)	21 - 31	26
Natural Moisture Content (%)	(W)	38 - 90	75
Liquidity Index	(I_L)	0.7 - 2.4	1.5

Initial Void Ratio	(e_o)	1.58 - 2.35
Compression Index	(C_c)	1.21 - 2.24
Preconsolidation Pressure (p.s.f.)	(P_c)	1100 - 6900

Undrained Shear Strength (p.s.f.) (C_u)	Range (C_u)	Range Sensitivity (S)
1) Field Vanes	160 - >2000	2 - 30
2) Lab. Vanes	330 - 2240	3 - >50
3) Lab. Tests	140 - 1840	

The Atterberg limit tests, summarized above, are also plotted on the Plasticity charts, Figure 9 and 10 . These results indicate that the clay is of high plasticity and, in general, inorganic. The natural water content is greater than the liquid limit, in the upper part of the deposit, decreasing with depth to

cont'd. /7 ...

4. <u>SUBSOIL CONDITIONS</u>: (cont'd.) ...

4.3) <u>Sensitive Clay</u>: (cont'd.) ...

values that are at or slightly below the liquid limit. The consistency of the stratum, as determined from the undrained shear strength testing, increases from very soft to firm, immediately below the surface deposit, to very stiff to hard with depth. This increase in undrained shear strength is represented by an average C_u/P_o ratio of 0.4 for the overall deposit; in the upper 40 feet the C_u/P_o value, however, appears to be of the order of 0.45. P_o is the effective overburden pressure. 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 clay.

The consolidation characteristics of the stratum were determined by carrying out four laboratory consolidation tests, the results of which are shown as Void Ratio vs. Pressure plots, on Figures 11 to 14 , inclusive. The results of this testing indicate that the clay is normally consolidated in the upper 50 to 60 ft. of the stratum; below this the clay is preconsolidated with respect to existing overburden pressure, as represented by a P_c/P_o ratio of the order of 1.4. 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) <u>Clayey Silt with some Sand and a trace of Gravel - (Glacial Till)</u>:

This heterogeneous layered deposit was encountered immediately below the sensitive clay deposit, between elevations 188 and 202. The total thickness of the deposit varies from 21 to 26 feet. Random seams and layers of sand and silt, ranging in

cont'd. /8 ...

4. <u>SUBSOIL CONDITIONS</u>: (cont'd.) ...

4.4) <u>Clayey Silt with some Sand and a trace of Gravel - (Glacial Till)</u>: (cont'd.) ...

thickness from a fraction of an inch up to 1 foot, are located throughout the glacial till. A grain-size distribution curve, for a sample from the glacial till, is shown on Figure 8 in Appendix I. Based on visual examination and limited laboratory testing, it is estimated that the cohesive glacial till is basically inorganic and of low plasticity, with the corresponding natural moisture content consistently below the plastic limit. The granular layers throughout the deposit are, however, non-plastic.

The Standard Penetration Tests carried out in this deposit, gave 'N' values ranging from 14 blows/ft., immediately below the clay stratum, increasing with depth to as many as 142 blows/11 inches. Based on these results, it is estimated that the cohesive portion of the glacial till varies from very stiff, immediately below the clay stratum, increasing to hard with depth.

4.5) <u>Shale Bedrock</u>:

Bedrock was established by obtaining AXT rock core samples at B.H.'s #1 and #9. The depth at which bedrock was encountered, at these boring locations, ranged from elevations 37 to 50.

The bedrock is composed of a grey fossiliferous shale. The limited amount of core recovered was in a fractured and jointed condition.

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5. GROUNDWATER CONDITIONS:

Groundwater level observations have been carried out, during the period of the investigation, in i) sealed piezometers installed in B.H.'s #1 and #9, and ii) the open holes at the remaining boring locations. The observations are recorded on the borelog sheets and summarized on Drawings 67-F-112A and 67-F-112B. The results of the measurements indicate that the piezometric groundwater level within the surficial sand and underlying clay stratum is at about elevation 257 - i.e., some 3 to 7 feet below ground surface. The corresponding piezometric groundwater level within the lower portion of the till, as determined at B.H.'s #1 and 9, was at elevation 252 and 232, respectively - i.e., some 11 to 29 feet below ground surface, respectively.

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 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 twin underpass structures to carry Anderson Road over the East and Westbound lanes of proposed Highway #417. The structures are to be constructed end to end being about 145 feet apart. Present proposals call for three-span structures (53'-85'-63' and 58'-84'-66' for the W.B.L. and E.B.L. structures, respectively). The maximum proposed profile grade of Anderson Rd., in the vicinity of the crossings, is elevation 292. At this grade the associated approach embankments will have a maximum height of about 28 feet above ground surface. The embankments will have a crest width of 36 feet.

cont'd. /10...

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

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

The East and Westbound lanes of Hwy. #417 will initially have three 12-foot wide paved lanes (one a collector lane) with provision for a fourth lane; the roadway cross-section will also incorporate shoulders. The finished grade will be elevated some 4 to 5 feet above surrounding ground level - i.e., it will be between elevations 267 and 269 in the vicinity of the crossings.

Underlying between 6 and 13 feet of sand, is the predominant stratum across the site, composed of a very soft, increasing with depth, to very stiff sensitive marine clay; this stratum varies from 180 to 196 feet in thickness. The clay is underlain by up to 26 feet of very 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 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. 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.

cont'd. /11 ...

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

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

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

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:

Elevation	Soil	Density (p.c.f.)	Strength Parameters Cu (p.s.f.)	Ø(°)
-	Embankment Fill	125	-	30
261 - 255	Surficial Deposit - Sand	125	-	30
255 - 247.5	Clay	$\gamma = 95$ $\gamma' = 33$	300	-
247.5 - 235	"	"	500	-
235 - 230	"	"	550	-
230 - 210	"	"	800	-
210 - 200	"	"	950	
200 - 185	"	"	1100	-

2) The surface of all berms required should slope away from the fill at a gradient of 20:1 for drainage purposes.

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

cont'd. /12 ...

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

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

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

STABILITY OF APPROACH EMBANKMENTS

CASE A -

Approach	Direction	Height of Fill ('H')	Berm Requirements ('L')	Type and Length of Structure
South	Longitudinal	18 ft.	36 ft.*	Single Multi-Span Structure - ~ 630 ft. in length
North	"	18 ft.	36 ft.*	
South & North	Transverse	0 - 9 ft.	Nil	
	"	9 -17 ft.	0 - 30 ft.*	(Sta. 28+65 to Sta. 34+95)
	"	18 -23 ft.	20 - 35 ft.**	

CASE B -

South & North	Longitudinal	9 ft.	Nil	Single Multi-Span Structure - ~ 1500 ft. in length (Sta. 24+60 to Sta. 39+60)
South & North	Transverse	0 - 9 ft.	Nil	

* Single Berm - Mid-Height

** Double Berms - Equi-spaced on the slope - i.e., at the third points.

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

1) Fills less than 9 feet in height may be constructed with standard 2:1 side slopes.

2) Fills in excess of 9 feet but less than 18 feet in height should be constructed with a single berm, while fills in excess of 18 feet can be constructed if double berms are incorporated into the design, as shown on Figure 2 in the Appendix of the report.

cont'd. /13 ...

6. <u>DISCUSSION AND RECOMMENDATIONS</u>: (cont'd.) ...

6.2) <u>Approach Embankments</u>: (cont'd.) ...

6.2.1) <u>Stability Considerations</u>: (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. The berm requirements in the longitudinal direction will necessitate that the approach fill between the proposed twin structures be eliminated. This being the case, the proposed structures will have to be modified to a single multi-span structure spanning both the E.B.L. and W.B.L. (refer to Case A in Table). This structure will be of the order of 630 feet in length.

4) It may be advantageous to minimize the longitudinal and transverse berm requirements by limiting the height of fill. For instance, if the height of fill is limited to 9 feet, then no berms will be required (refer to Case B). This would, however, necessitate a multi-span structure some 1500 feet in length. It should be noted that minimizing the heights of fill has the added advantage of reducing the settlements induced in the foundation subsoil, as discussed in detail in Section 6.2.2) of this report.

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

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

6.2.2) <u>Settlement Considerations</u>:

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

cont'd. /14 ...

6. <u>DISCUSSION AND RECOMMENDATIONS</u>: (cont'd.) ...

6.2) <u>Approach Embankments</u>: (cont'd.) ...

6.2.2) <u>Settlement Considerations</u>: (cont'd.) ...

settlement will occur under the north approach embankment (Case A), where the height of fill will be of the order of 23 feet above ground surface. The computations indicate that this settlement could be as much as 11 feet under the centre-line of the embankment. If the maximum fill height is maintained at 9 feet (Case B), the total consolidation settlement would be of the order of 3 feet.

The total amount of the consolidation settlements predicted will take place over an extended period of time, probably in excess of 200 years. However, about 28% and 18% of the consolidation settlement should occur within 7 years and 18 months, respectively (see plot on Figure 2). In addition, it is considered that the estimated settlements may occur at a faster rate than that theoretically computed because of the presence of occasional permeable silt layers within the cohesive stratum, which would accelerate the drainage in the lateral direction. In view of this, it would, therefore, be advantageous to construct the embankments first and leave them in place for as long as possible prior to construction of the structure (say, for 18 to 24 months). This will tend to reduce the maintenance problems associated with the immediate approaches to the structure. Keeping this in mind, it is recommended that final paving be delayed for as long a period as possible.

Computations were also carried out to determine the consolidation settlement to be expected for various heights of fill between the two limiting conditions discussed above. The results of these computations are presented on Figure 3. The structure span-length required for each fill height is also plotted on this figure. The most feasible fill height, and corresponding span-length employed in the finalized structural design scheme, should be based on economic considerations.

cont'd. /15 ...

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

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

6.2.3) Light-Weight Fill:

To minimize the berm requirements and the excessive consolidation settlements expected, consideration could be given to using light-weight fill in place of standard fill. If it is deemed that this alternative is economically feasible, the Foundation Section could readily, assess the engineering aspects, including the stability and expected settlements.

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 at between elevations 50 and 60. 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. A continuous structure 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

cont'd. /16 ...

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

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

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

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

6.3.2) Friction 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 4 in the Appendix.

cont'd. /17 ...

6. <u>DISCUSSION AND RECOMMENDATIONS</u>: (cont'd.) ...

6.3) <u>Structure Foundations</u>: (cont'd.) ...

6.3.2) <u>Friction Piles</u>: (cont'd.) ...

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 pile group.
iii) Applied load.
iv) Influence of approach fills.

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

a) The intermediate piers should settle quite uniformly since, at these locations, the settlement will not be influenced by the approach fills. Therefore, the central spans of the structure can be continuous.

b) The settlement of the piles at the abutment and end-pier locations will, however, 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 structure be simply supported.

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

cont'd. /18 ...

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

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

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

No major dewatering problems are anticipated. Excavations for the pier pile caps may, however, be carried out below the groundwater level, which is about 3 to 7 feet below ground surface. Because these excavations will be carried out mainly 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.

7. SUMMARY:

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

Underlying between 6 and 13 feet of sand, is the predominant overburden stratum across the site, composed of a very soft to very stiff, sensitive marine clay varying from 180 to 196 feet in thickness. The clay is underlain by up to 26 feet of very 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 3 to 7 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 50 and 60. 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.

cont'd. /19 ...

7. <u>SUMMARY</u>: (cont'd.) ...

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 9 feet in height. The berm requirements in the longitudinal direction necessitates that the proposed twin structure scheme be modified to a single multi-span type of structure.

Settlements up to 11 feet are estimated for a maximum fill height of 23 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.

8. <u>MISCELLANEOUS</u>:

The field work for this project was carried out during the periods of December 5 to 15, 1967 and January 2 to 29, 1968. During the former and latter period the project was under the supervision of Messrs. W. Hutton and P. B. Schnabel, respectively, both Foundation Engineers. The equipment used was owned and operated by F. E. Johnston Drilling Co. Ltd.

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

April, 1968.

APPENDIX I

FORM OB-MT-126
66-4546

OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING DIVISION

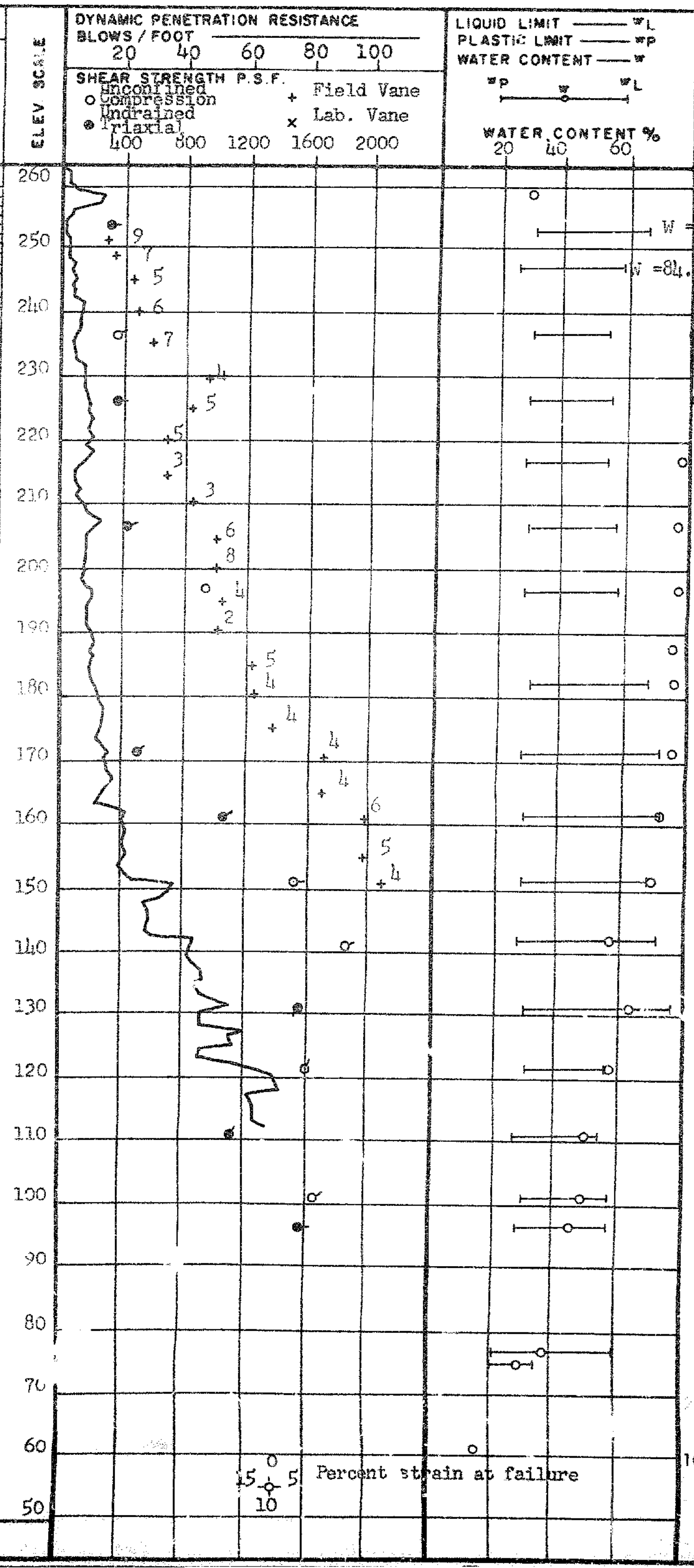
RECORD OF BOREHOLE NO. 1

FOUNDATION SECTION

JOB	67-F-112	LOCATION	Sta. 29+02 ℄ Anderson Rd. O/S 18' Left	ORIGINATED BY	W.H.
W.P.	34-66-06	BORING DATE	December 5 - 15, 1967	COMPILED BY	W.H.
DATUM	Geodetic	BOREHOLE TYPE	Diamond Drill - NX, BX, AX Casing-AXT, BXL Core.	CHECKED BY	[signature]

SOIL PROFILE / SAMPLES

ELEV. DEPTH	DESCRIPTION	NUMBER	TYPE	BLOWS/FOOT
262.8				
0.0	Sand - (Occasional layers of Silt and Silty Clay) - Loose.	1	SS	9
254.8		2	SS	6
8.0	Sensitive Clay - Occasional inclusions of organic matter. Soft to very stiff	3	TP	PM
		4	TP	PM
		5	TP	PM
		6	TP	PM
		7	TP	PM
		8	TP	PM
		9	TP	PM
		10	TP	PM
		11	TP	PM
		12	TP	PM
		13	TP	PM
		14	TP	PM
		15	TP	PM
		16	TP	PM
		17	TP	PM
		18	TP	PM
		19	TP	PM
		20	TP	PM
		21	TP	PM
		22	TP	PM
		23	TP	PM
		24	TP	PM
		25	TP	PM
		26	TP	PM
		27	TP	PM
		28	TP	PM
		29	TP	PM
		30	TP	PM
		31	TP	PM
		32	TP	PM
		33	TP	PM
		34	TP	PM
		35	TP	PM
85.8 / 177.0	Layers of Sand, Silt and Clayey Silt.	36	TP	PM
74.6 / 188.2	Clayey Silt with Sand and some Gravel - (Glacial Till), (Layers of Sand and Silt) Hard	37	TP	PM
		38	BXL RC	3% Rec.
		39	SS	75
49.7	Bedrock, Black Shale	40	AXT RC	8% Rec.
212.8	End of Borehole			



BULK DENSITY γ P.C.F.: γ=94, 91, 97, 95, 95.5, 96, 96, 97, 99, 103, 99, 109, 110, 110, 104.5

REMARKS:
- Org. 0.4%
- W.L. 257
- W.L. 252.2
- C_c = 1.43
- Sa.1%, Si.29%, Cl.70%
- C_c = 1.59
- Piezometer tip at 222.8
- C_c = 2.24
- Sa.7%, Si.34%, Cl.59%
- C_c = 1.21
- Si.78%, Cl.22%
- Piezometer Tip at 73.8

66-4546

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 2

FOUNDATION SECTION

JOB 67-F-112

W.P. 34-66-06

DATUM Geodetic

LOCATION Sta. 29+60 ℄ Anderson Rd. O/S 12' Rt.

BORING DATE December 6 - 7, 1967

BOREHOLE TYPE Penndrill

ORIGINATED BY W.H.

COMPILED BY W.H.

CHECKED BY [signature]

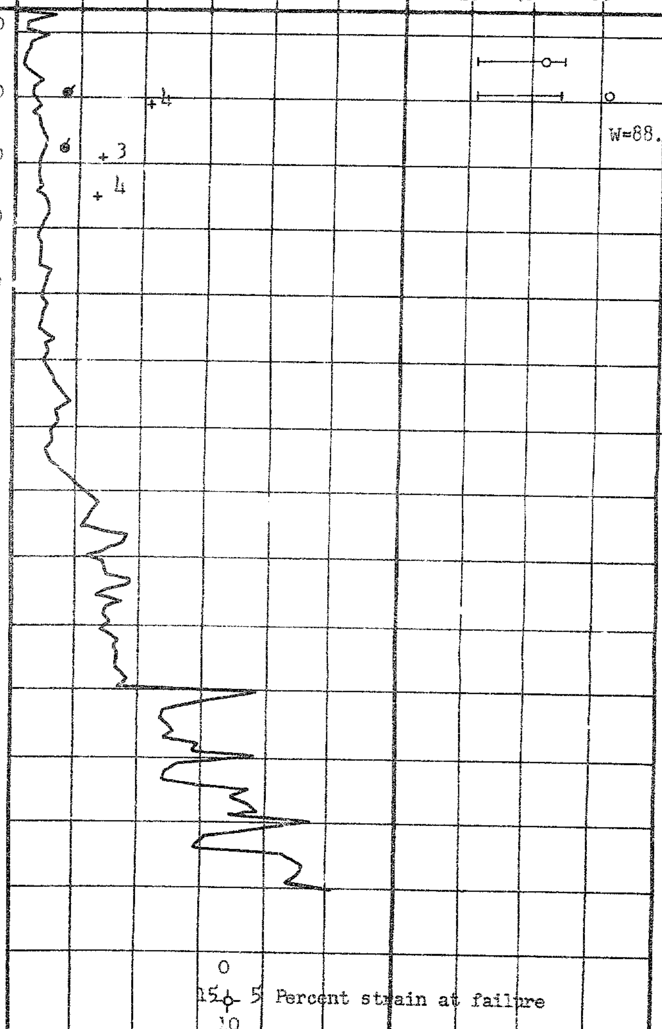
ELEV. / DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE	WATER CONTENT	BULK DENSITY γ P.C.F.	REMARKS
263.4									
0.0	Sand - (Layers of Silty Clay) - Loose		1	SS	2	260			
257.1			2	SS	4				
6.3	Sensitive Clay with Occasional inclusions of Organic Matter Soft to Firm		3	TW	PM				
			4	TP	PM	250		100	
			5	TP	PM				
			6	TW	PM	240	W=88.9	94	
235.9			7	TW	PM				
27.5	End of Borehole					230			
	Probably Clay					220 – 130			
129.2									
134.2	End of Cone Test					120			

DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT 20 40 60 80 100

SHEAR STRENGTH P.S.F. ○ Unconfined Compression ● Quick Triaxial + Field Vane × Lab. Vane 400 800 1200 1600 2000

LIQUID LIMIT — w_L PLASTIC LIMIT — w_P WATER CONTENT — w

WATER CONTENT % 20 40 60



FORM DB-MT-128
66-4546

OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING DIVISION

~~RECORD OF BOREHOLE NO.~~ PENETRATION TEST No. 3

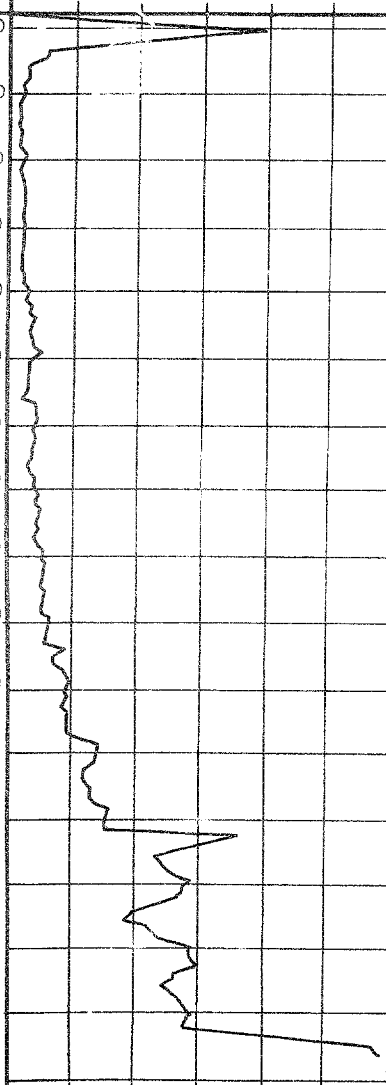
FOUNDATION SECTION

JOB 67-F-112
W.P. 34-66-06
DATUM Geodetic

LOCATION Sta. 30+44 ℄ Anderson Rd. 12' Lt.
BORING DATE January 29, 1968
BOREHOLE TYPE Diamond Drill

ORIGINATED BY P.B.S.
COMPILED BY P.B.S.
CHECKED BY [signature]

SOIL PROFILE			SAMPLES							
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT 20 40 60 80 100 — SHEAR STRENGTH P.S.F.	LIQUID LIMIT w_L PLASTIC LIMIT w_P WATER CONTENT w — WATER CONTENT %	BULK DENSITY γ P.C.F.	REMARKS
262.3										
0.0	Probably Sand Deposit									
254										
	Probably Clay									
105.3										
157.0	End of Cone Test									



FORM 08-MT-126
66-4546

OFFICE REPORT ON SOIL EXPLORATION

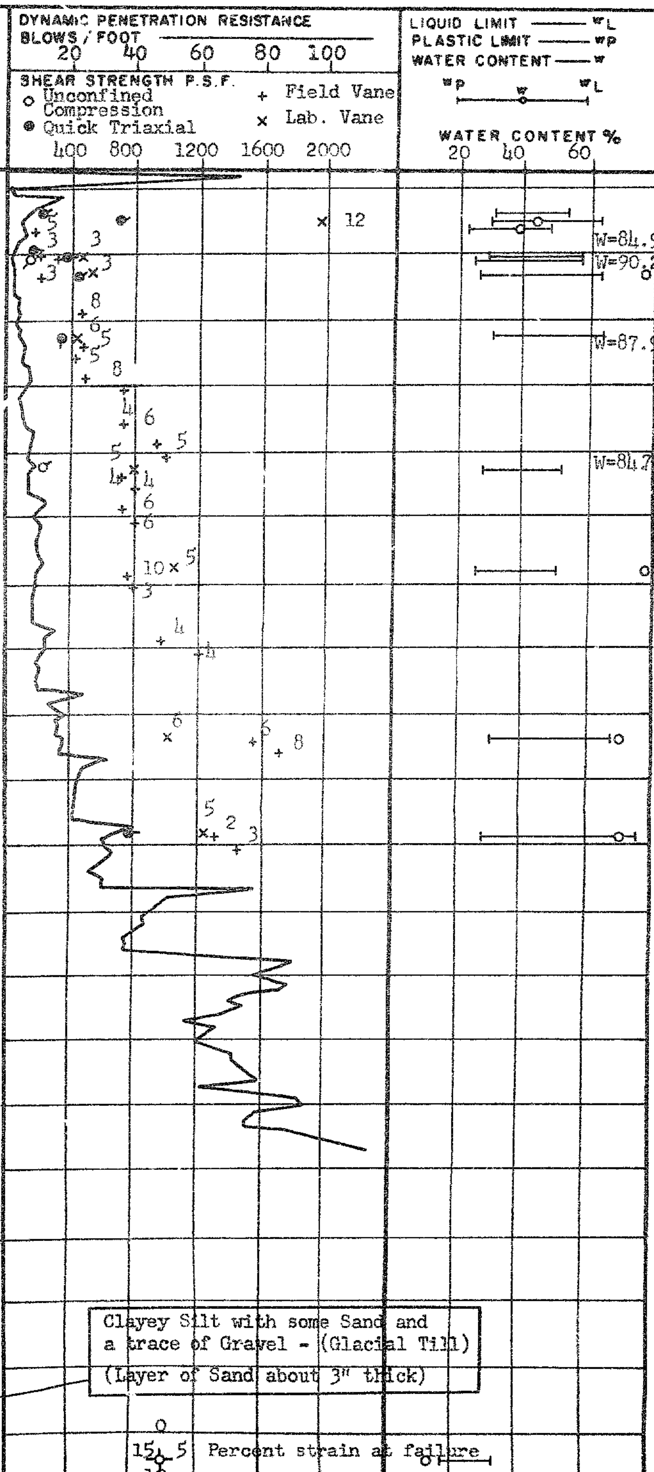
DEPARTMENT OF HIGHWAYS – ONTARIO
MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 4

FOUNDATION SECTION

JOB 67-F-112 LOCATION Sta. 31+10 ℄ Anderson Rd. O/S 12' Rt. ORIGINATED BY W.H. & P.S.
W.P. 34-66-06 BORING DATE December 5 - 6, 1967 January 4 - 15, 1968 COMPILED BY W.H.
DATUM Geodetic BOREHOLE TYPE Diamond Drill - NX Casing CHECKED BY [signature]

ELEV. DEPTH 262.8	SOIL PROFILE – DESCRIPTION	SAMPLES NUMBER	TYPE	BLOWS/FOOT
0.0 / 256.8	Clayey Silt (Layers of Silt and Sand) - Soft to Firm.	1	SS	2
6.0	Sensitive Clay with occasional inclusions of Organic Matter - Very Soft to Very Stiff	1A	TW	PM
		2	TW	PM
		3A	TW	PM
		4, 3A	TW	PM
		4A	TW	PM
		5	TW	PM
		6	TW	PM
		7	TW	PM
		8	TW	PM
		8A	TW	PM
		8B	TW	PM
		9	TW	PM
		10	TW	PM
		11	TW	PM
		12	TW	PM
		13	TW	PM
		14	TW	PM
76.8 / 186.0	Layers of Sand, Silt and Clayey Silt			
68.8 / 65.8		15	SS	106
197.0	End of Borehole			



BULK DENSITY γ P.C.F.	REMARKS
95	Gr. 0%, Sa. 5% Si. 66%, Cl. 29%
110	Org. 0·5 %
93	Org. 0·9 %
95.5	
94	
97	Sa. 1%, Si. 38% Cl. 61%
99	
98	
	Gr. 3%, Sa. 15% Si. 75%, Cl. 7%

FORM DB-MT-126
66-4546

OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS – ONTARIO
MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 5

FOUNDATION SECTION

JOB 67-F-112
W.P. 34-66-06
DATUM Geodetic

LOCATION Sta. 25+53 ℄ Anderson Rd. O/S 12' Rt.
BORING DATE January 24 – 29, 1968
BOREHOLE TYPE Diamond Drill – HX Casing

ORIGINATED BY P.B.S.
COMPILED BY P.B.S.
CHECKED BY

SOIL PROFILE

ELEV. DEPTH	DESCRIPTION
264.8	
0.0	Sand (Layers of Silt and Clayey Silt) – Loose to Compact
251.3 / 13.5	Sensitive Clay with occasional inclusions of Organic Matter (Layer of Silt – some 1-foot thick at about elev. 229) Firm to Very Stiff
78.8 / 186.0	Layers of Sand, Silt and Clayey Silt.
70.0 / 194.8	Clayey Silt with some Sand and a trace of Gravel – (Glacial Till) (Layer of Sandy Silt 2" thick)
197.0	End of Borehole

SAMPLES

NUMBER	TYPE	BLOWS / FOOT
1	SS	9
2	SS	11
3	SS	18
4	SS	6
5	TP	PM
6	TP	PM
7	TP	PM
8	TP	PM
9	TP	PM
10	TP	PM
11	TP	PM
12	TP	PM
13	TP	PM
14	TP	PM
15	TP	PM
16	TP	PM
17	SS	57

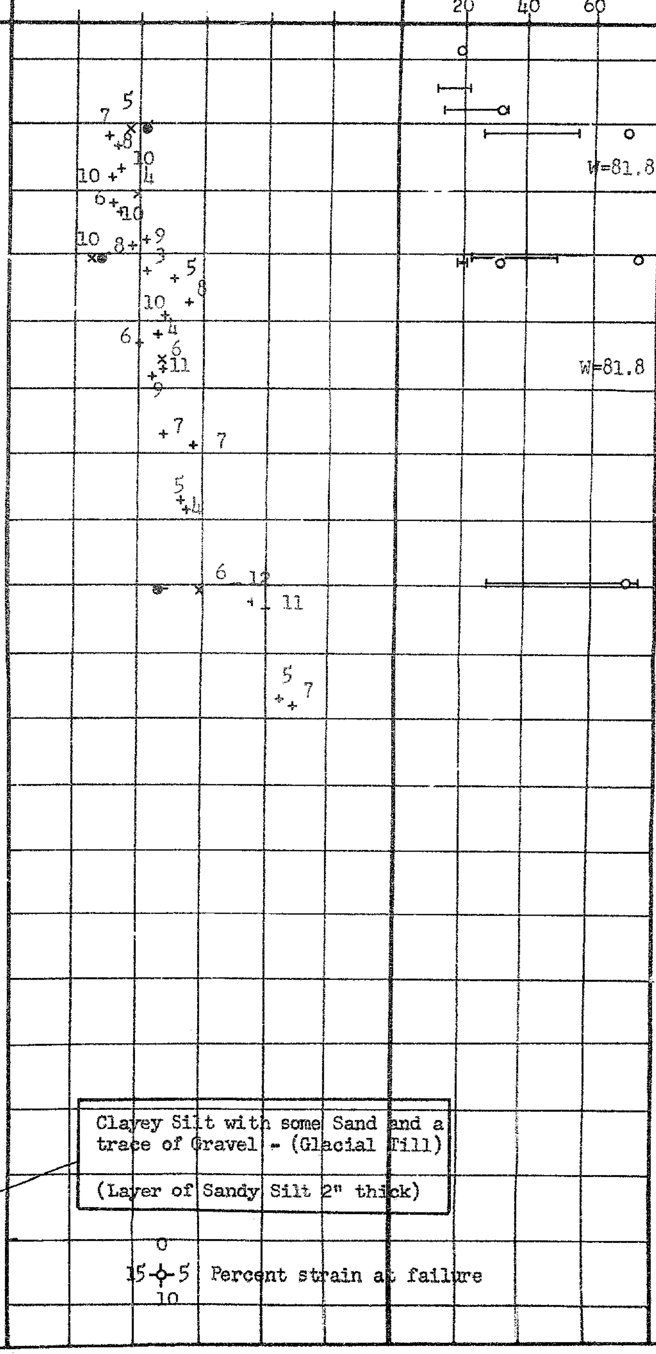
ELEV. SCALE: 260, 250, 240, 230, 220, 210, 200, 190, 180, 170, 160, 150, 140, 130, 120, 110, 100, 90, 80, 70

DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT: 20 40 60 80 100

SHEAR STRENGTH P.S.F.
○ Unconfined Compression
● Quick Triaxial
+ Field Vane
× Lab. Vane

LIQUID LIMIT — w_L
PLASTIC LIMIT — w_P
WATER CONTENT — w

WATER CONTENT %: 20 40 60



BULK DENSITY γ P.C.F.	REMARKS
98	
97	
98	Gr.0%, Sa.4% Si.85%, Cl.11%
98	

FORM DB-MT-126
66-4546

OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 6

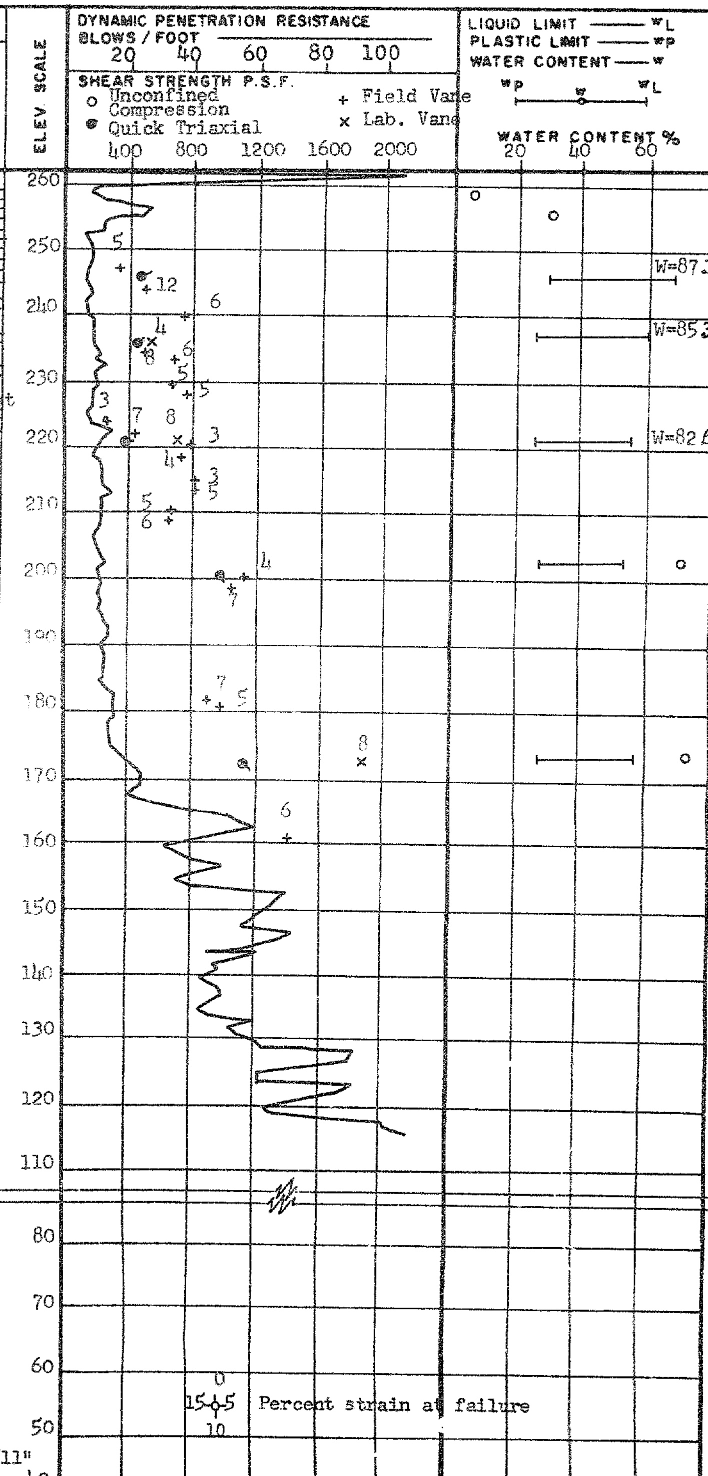
FOUNDATION SECTION

JOB 67-F-112
W.P. 34-66-06
DATUM Geodetic

LOCATION Sta. 32+55 ℄ Anderson Rd O/S 12' Left
BORING DATE December 14, 1967 January 16 - 29, 1968
BOREHOLE TYPE Diamond Drill - NX,BX,AX Casing

ORIGINATED BY W.H. & P.S.
COMPILED BY W.H.
CHECKED BY

ELEV. DEPTH (262.6)	DESCRIPTION	SAMPLE NUMBER	TYPE	BLOWS / FOOT	WATER CONTENT	BULK DENSITY γ P.C.F.	REMARKS
0.0	Sand - (Layers of Silt and Clayey Silt) Very Loose to Compact	1	SS	9			Sa.99%, Si.Cl.1%
		2	SS	12			Sa.37%, Si.37% Cl.26%
250.6		3A	SS	2			
12.0	Sensitive Clay with occasional inclusions of Organic Matter	3	TP	PM			
		4	TP	PM	W=87.1	94.5	
		5	TP	PM			
		6	TP	PM	W=85.3		Sa.2%, Si.53% Cl.45%
		7	TP	PM			
		8	TP TW	Lost			
		9	TW	PM	W=82.6	96	
		10	TW	PM			
		11	TW	PM			
		12	TW	PM		100	
196.6 / 66.0	Layer of Sand						
192.3 / 70.3		13	SS	14			
		14	TW	PM			
		15	TW	PM		99	
		16	TW	PM			
76.6 / 186.0	Layers of Sand, Silt and Clayey Silt						
67.6 / 195.0	Clayey Silt with Sand and some Gravel (Glacial Till) (Layers of Sand and Silt) Hard	17	SS	111			
		18	SS	112/11"			
41.0 / 221.6	End of Borehole						



66-4546

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & TESTING DIVISION

RECORD OF ~~BOREHOLE No.~~ PENETRATION TEST No. 7

FOUNDATION SECTION

JOB 67-F-112

W.P. 34-66-06

DATUM Geodetic

LOCATION Sta. 33+18 ℄ Anderson Rd. O/S 12' Rt.

BORING DATE December 13, 1967

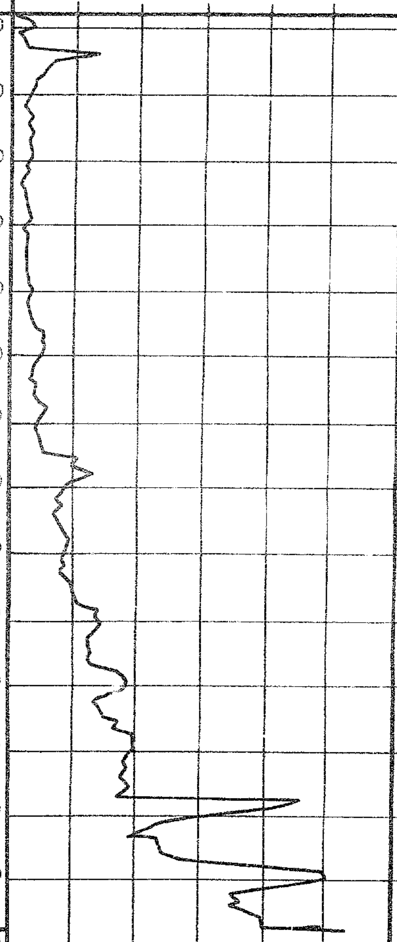
BOREHOLE TYPE Penndrill

ORIGINATED BY W.H.

COMPILED BY W.H.

CHECKED BY [signature]

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT 20 40 60 80 100 SHEAR STRENGTH P.S.F.	LIQUID LIMIT — w_L PLASTIC LIMIT — w_P WATER CONTENT — w WATER CONTENT %	BULK DENSITY γ P.C.F.	REMARKS
ELEV. DEPTH 262.1	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT					
0.0 / 252	Probably Sand Deposit					260 / 250				
122.1	Probably Clay					240 / 230 / 220 / 210 / 200 / 190 / 180 / 170 / 160 / 150 / 140 / 130				
140.0	End of Cone Test					120				



DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 9

FOUNDATION SECTION

JOB 67-F-112 LOCATION Sta. 34+56 @ Anderson Rd. O/S 12' Rt. ORIGINATED BY W.H.

W.P. 34-66-06 BORING DATE December 8, 1967 January 4 - 22, 1968 COMPILED BY W.H.

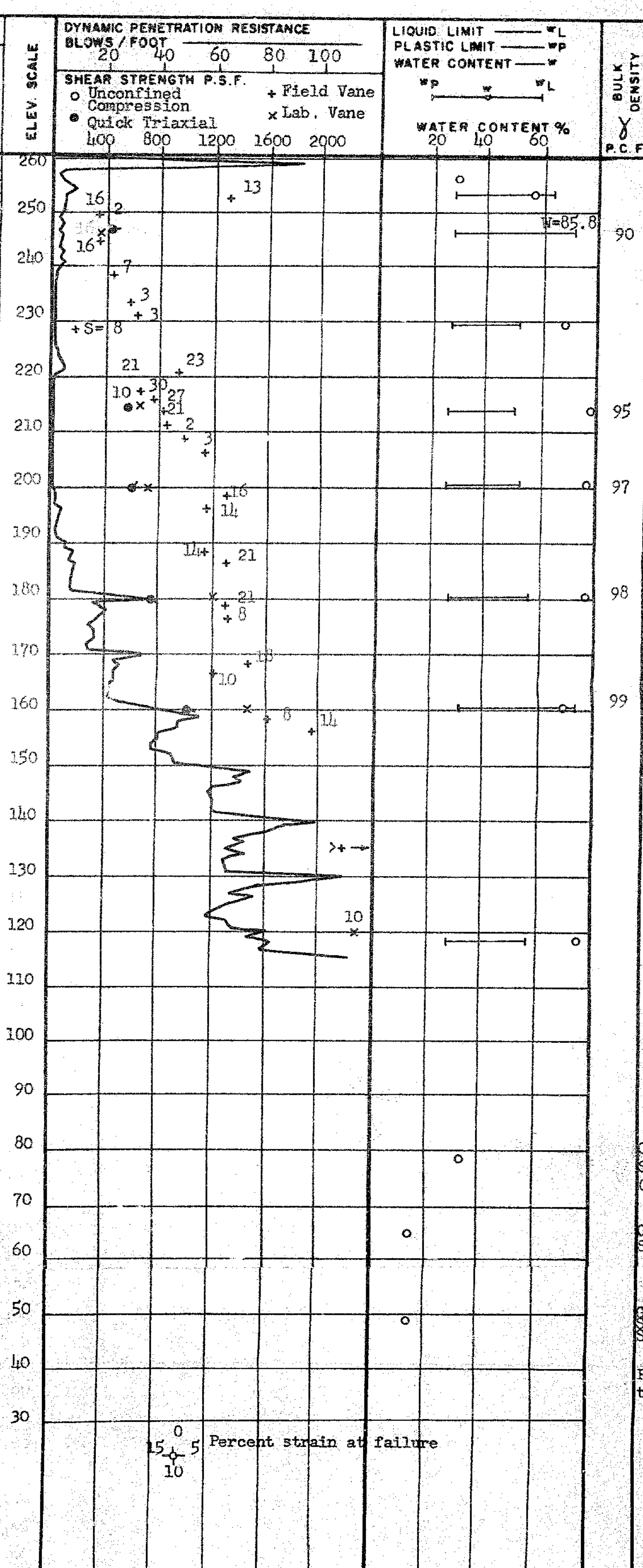
DATUM Geodetic BOREHOLE TYPE Diamond Drill - HX, BX, AX Casing-AXT Core CHECKED BY

SOIL PROFILE

ELEV. / DEPTH	DESCRIPTION
260.7	
0.0	Sand - (Layers of Silt and Silty Clay) Loose
254.5 / 6.2	Sensitive Clay with occasional inclusions of Organic Matter. Soft to Very Stiff
82.7 / 178.0	Layers of Sand, Silt and Clayey Silt. Compact to Very Dense or Very Stiff to Hard
58.7 / 202.0	Clayey Silt with Sand and a trace of Gravel (Glacial Till) - (Layers of Sand and Silt throughout) Very Stiff
36.8 / 223.9	Shale Bedrock Sound
31.4 / 229.3	End of Borehole

SAMPLES

NUMBER	TYPE	BLOWS / FOOT
1	SS	5
2,1A	TW	PM
2A	TW	PM
3A	3"TP	PM
4	TP	PM
5	TP	PM
6	TP	PM
7	TP	PM
8	TP	PM
9	3"TW	PM
10	TW	PM
11	3"TW	PM
12	TW	PM
13	3"TW	PM
14	TW	PM
15	TW	PM
15A	TW	PM
16	TW	PM
17	TW	PM
18	SS	18
19	SS	77
20	SS	62
21	SS	14
22	AXT	40%
23	AXT	70%



REMARKS

ELEV.	REMARKS
	W.L. 257.1
260–250	Org. 0.5%; Gr. 0%, Sa. 8%, Si. 41%, Cl. 51%
225	W.L.=231.9
205	Piezometer tip at 210.7
75	Gr. 1%, Sa. 2%, Si. 62%, Cl. 35%
62	Gr. 0%, Sa. 7%, Si. 78, Cl. 15%
47	Gr. 10%, Sa. 61%, Si. Cl. 29%
37	Piezometer tip at 40.7

FORM 08-MT-126
66-4546

OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS – ONTARIO
MATERIALS & TESTING DIVISION

RECORD OF ~~BOREHOLE NO.~~ PENETRATION TEST No. 8

FOUNDATION SECTION

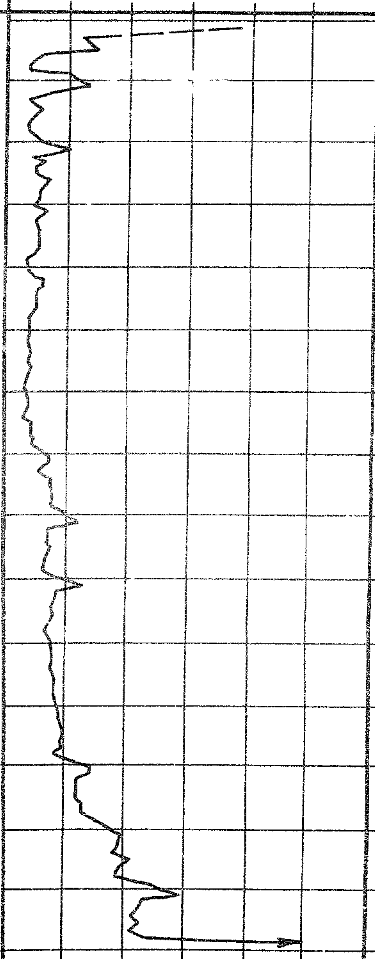
JOB 67-F-112
W.P. 34-66-06
DATUM Geodetic

LOCATION Sta. 34+03 ℄ Anderson Rd. O/S 13' Lt.
BORING DATE January 26, 1968
BOREHOLE TYPE Diamond Drill

ORIGINATED BY P.B.S.
COMPILED BY P.B.S.
CHECKED BY [signature]

SOIL PROFILE: ELEV / DEPTH	DESCRIPTION	STRAT. PLOT	SAMPLES: NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE
261.7						
254	Probably Sand Deposit					260
						250
						240
						230
						220
						210
						200
	Probably Clay					190
						180
						170
						160
						150
						140
						130
						120
109.7						110
152.0	End of Cone Test					

DYNAMIC PENETRATION RESISTANCE
BLOWS / FOOT 20 40 60 80 100
SHEAR STRENGTH P.S.F.



LIQUID LIMIT — w_L
PLASTIC LIMIT — w_P
WATER CONTENT — w
w_P w w_L
WATER CONTENT %

BULK DENSITY γ P.C.F.

REMARKS

FORM DB-MT-126
66-4546

OFFICE REPORT ON SOIL EXPLORATION

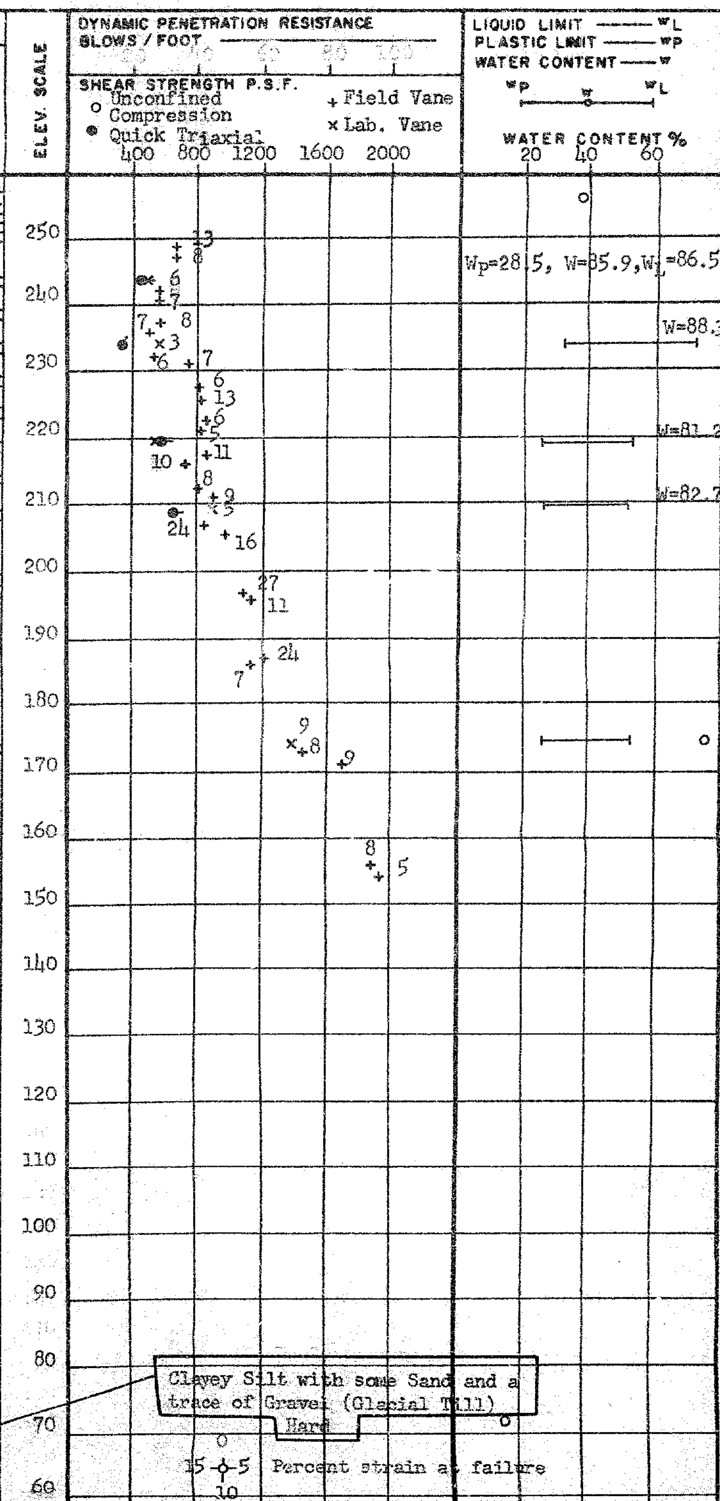
DEPARTMENT OF HIGHWAYS – ONTARIO
MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 10

FOUNDATION SECTION

JOB 67-F-112	LOCATION Sta. 36+00 ℄ Anderson Rd. O/S 10' Lt.	ORIGINATED BY P.B.S.
W.P. 34-66-06	BORING DATE January 19 - 23, 1968	COMPILED BY P.B.S.
DATUM Geodetic	BOREHOLE TYPE Diamond Drill	CHECKED BY

ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	SAMPLE NUMBER	TYPE	BLOWS / FOOT	BULK DENSITY γ P.C.F.	REMARKS
260.5							
0.0	Sand- Trace of Organic Matter in upper 5 ft. Loose		1	SS	8		Org. 4.9%
254.0 6.5			2	SS	9		
	Sensitive Clay with occasional inclusions of Organic Matter Soft to Very Stiff		3	SS	2		
			4	TP	FM	94	Si. 1%, Cl. 99%
			5	TP	FM		
			6	TP	FM	94	
			7	3"TP	FM		
			8	TP	FM		
			9	TP	FM	95	
			10	TP	FM	96	Sa. 1%, Si. 31% Cl. 68%
			11	TP	FM	94	
			12	TP	FM		
			13	TP	FM		
			14	TP	FM	99	
			15	TP	FM		
77.0							
182.5	Layers of Sand, Silt and Clayey Silt Very Dense or Hard		16	SS	83		
60.0 200.5			17	SS	54		
201.0	End of Borehole						



SUMMARIZED RESULTS OF
SETTLEMENTS EXPECTED FOR VARIOUS HEIGHTS OF FILL

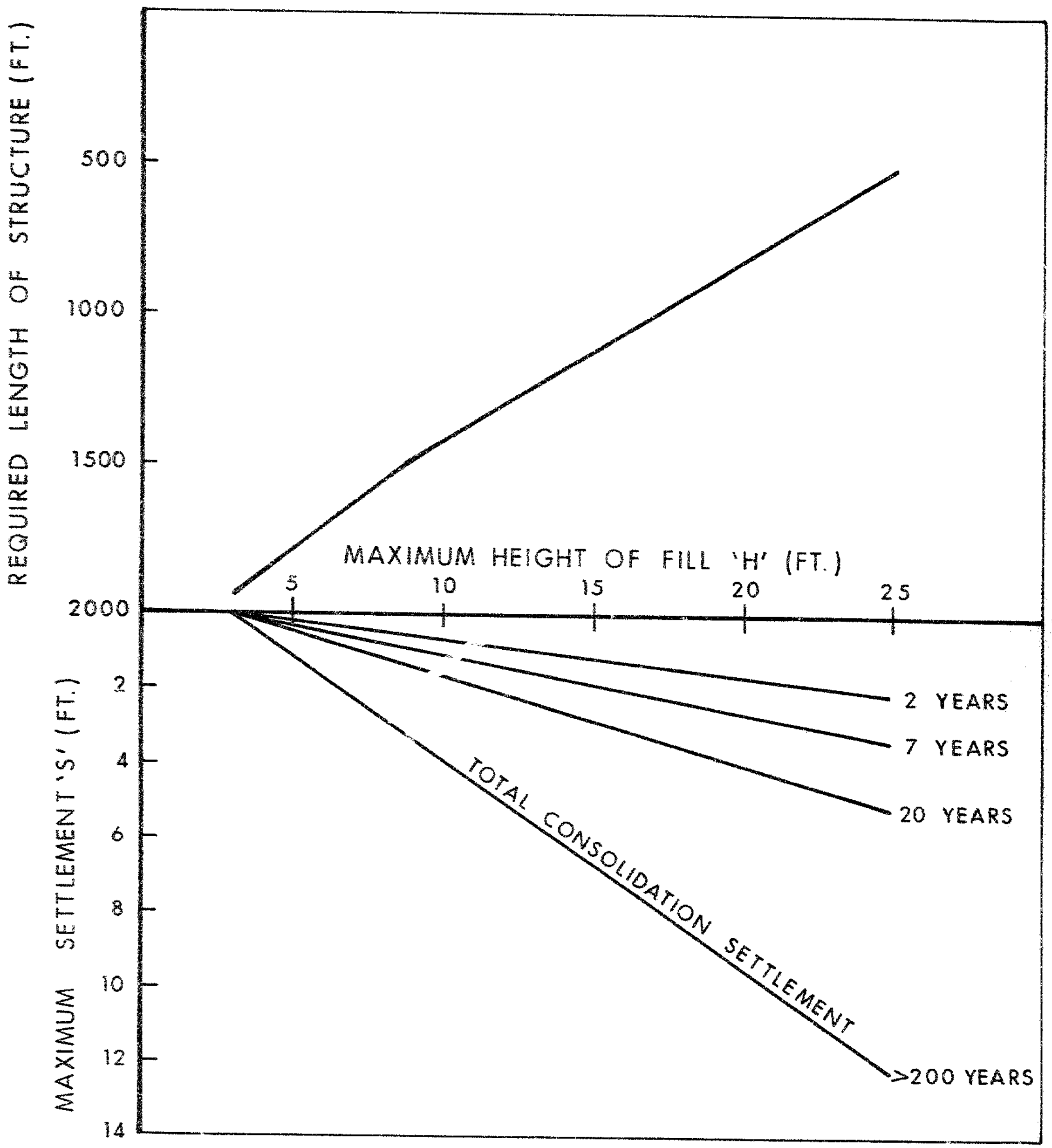


FIG. 3

ALLOWABLE PILE LOADS
(CLOSED-END 12 3/4" O.D. TUBULAR PILES)

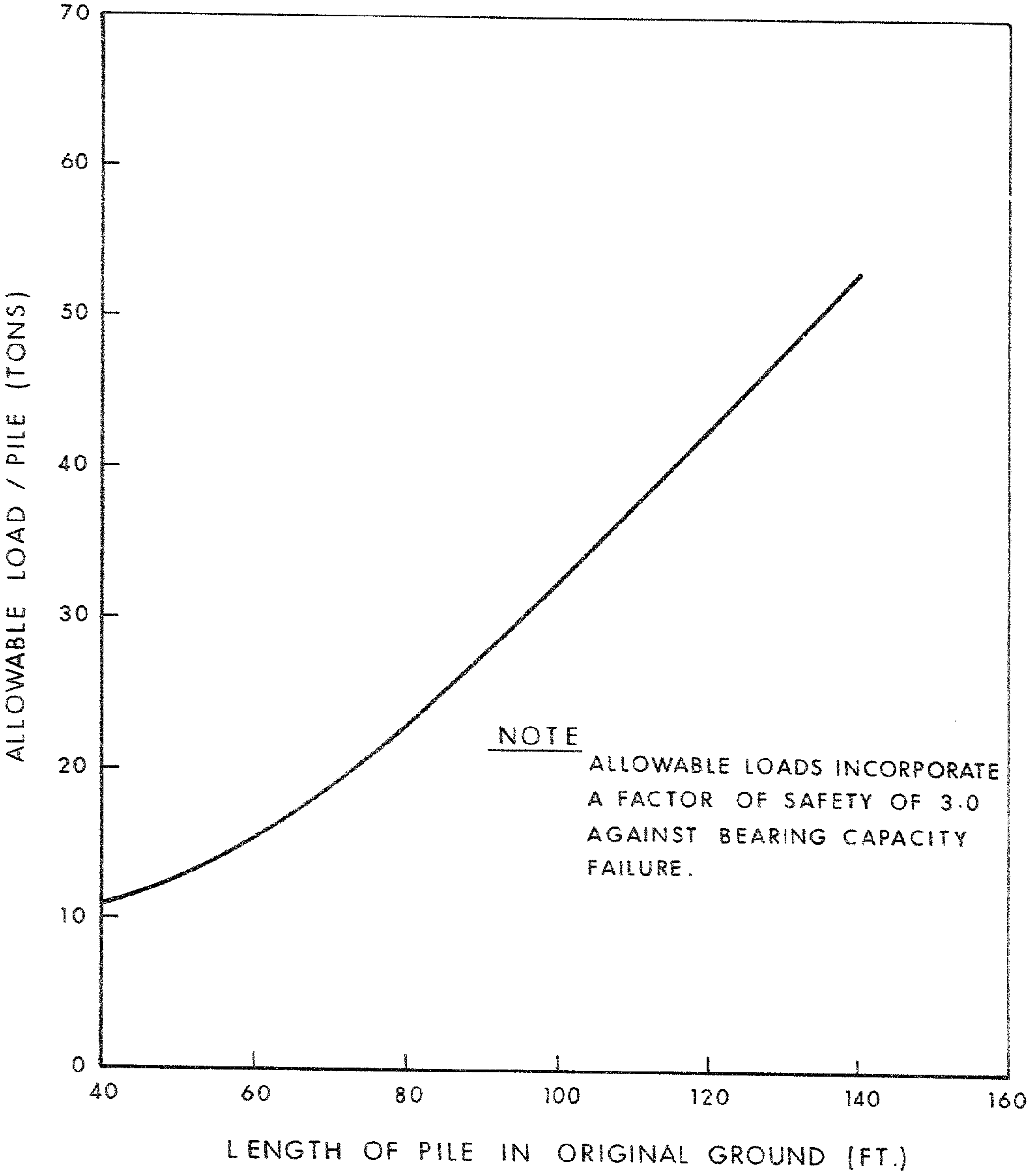
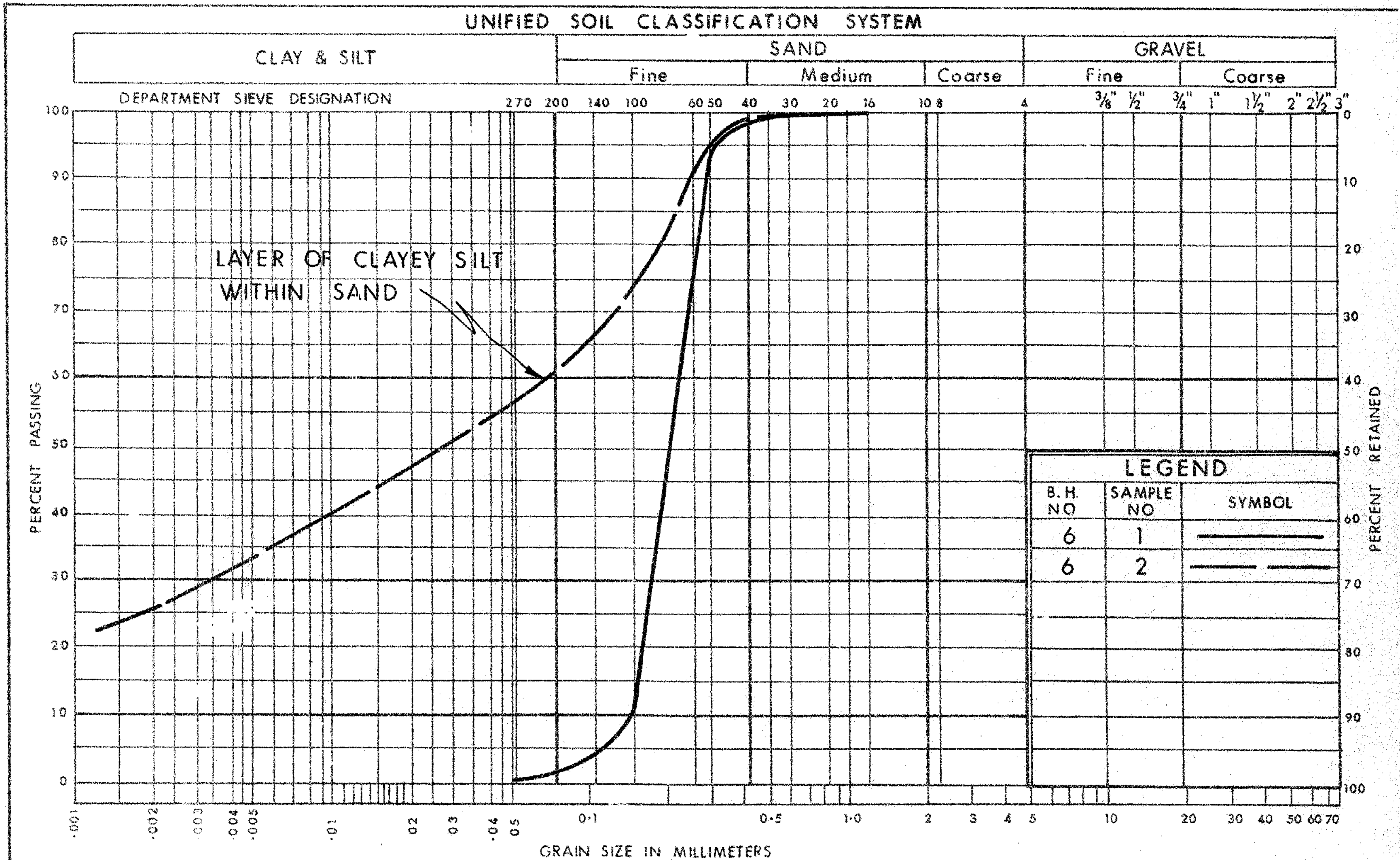


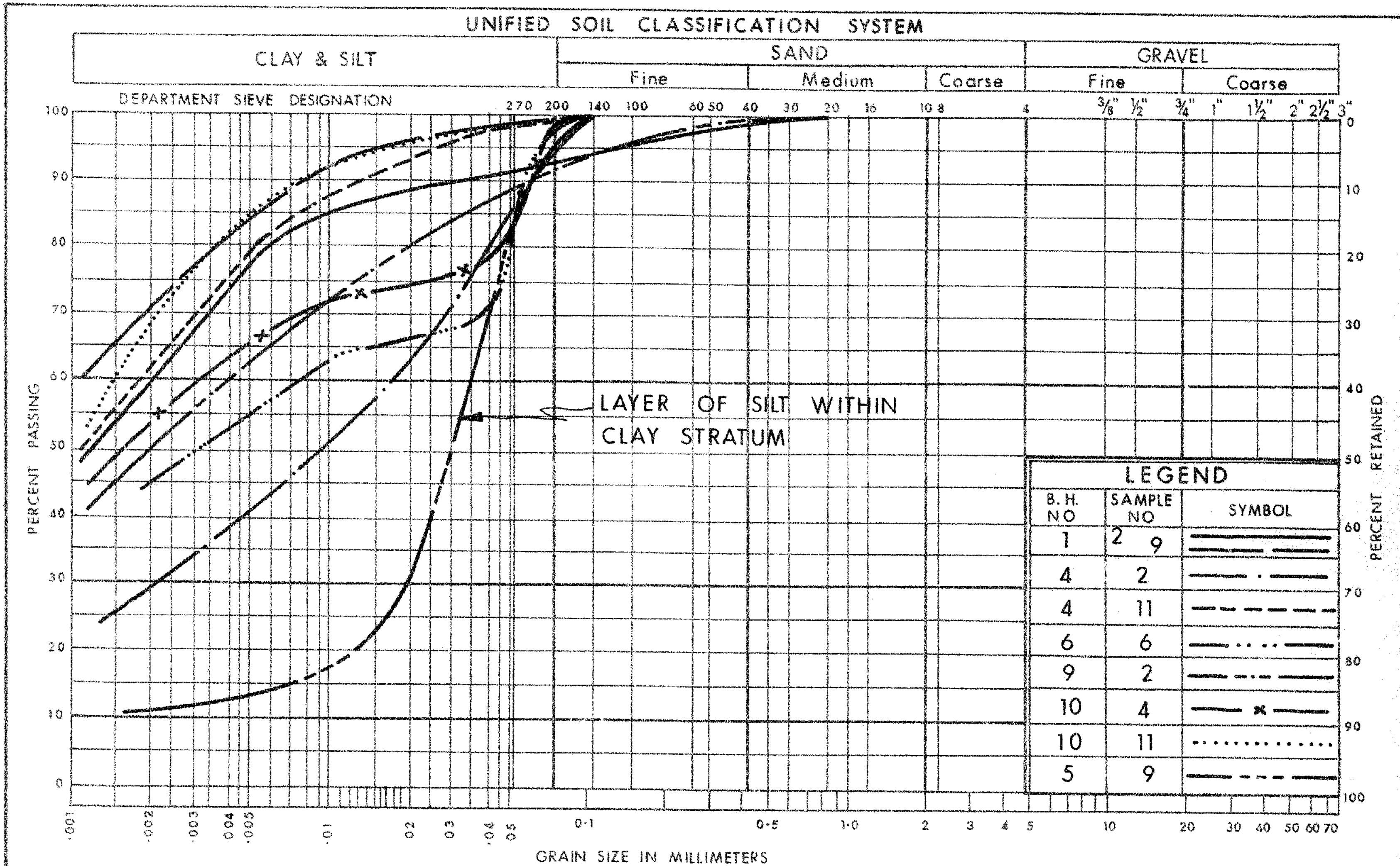
FIG. 4

FORM OB MT-17
REV. APR. 1965



DEPARTMENT OF HIGHWAYS MATERIALS and TESTING DIVISION	GRAIN SIZE DISTRIBUTION SAND	W.P. No. 34 – 66 – 06 JOB No. 67 – F – 112 FIG. NO. 5

FORM OB MT - 17
REV. APR. 1965



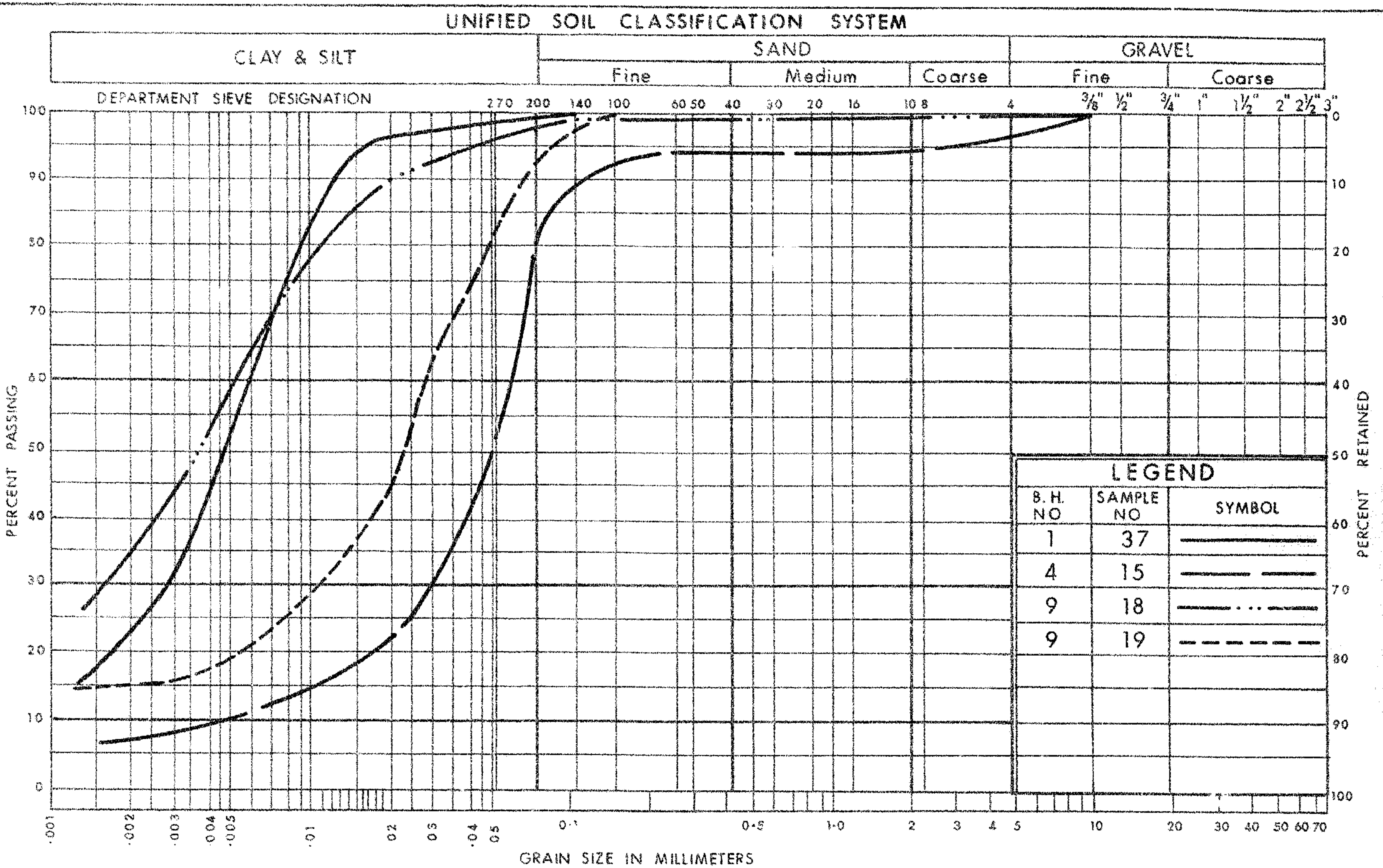
DEPARTMENT OF HIGHWAYS
MATERIALS and TESTING DIVISION

GRAIN SIZE DISTRIBUTION

SENSITIVE CLAY

W.P. No. 34 - 66 - 06
JOB No. 67 - F - 112
FIG. NO. 6

FORM OB MT- 17
REV. APR. 1965

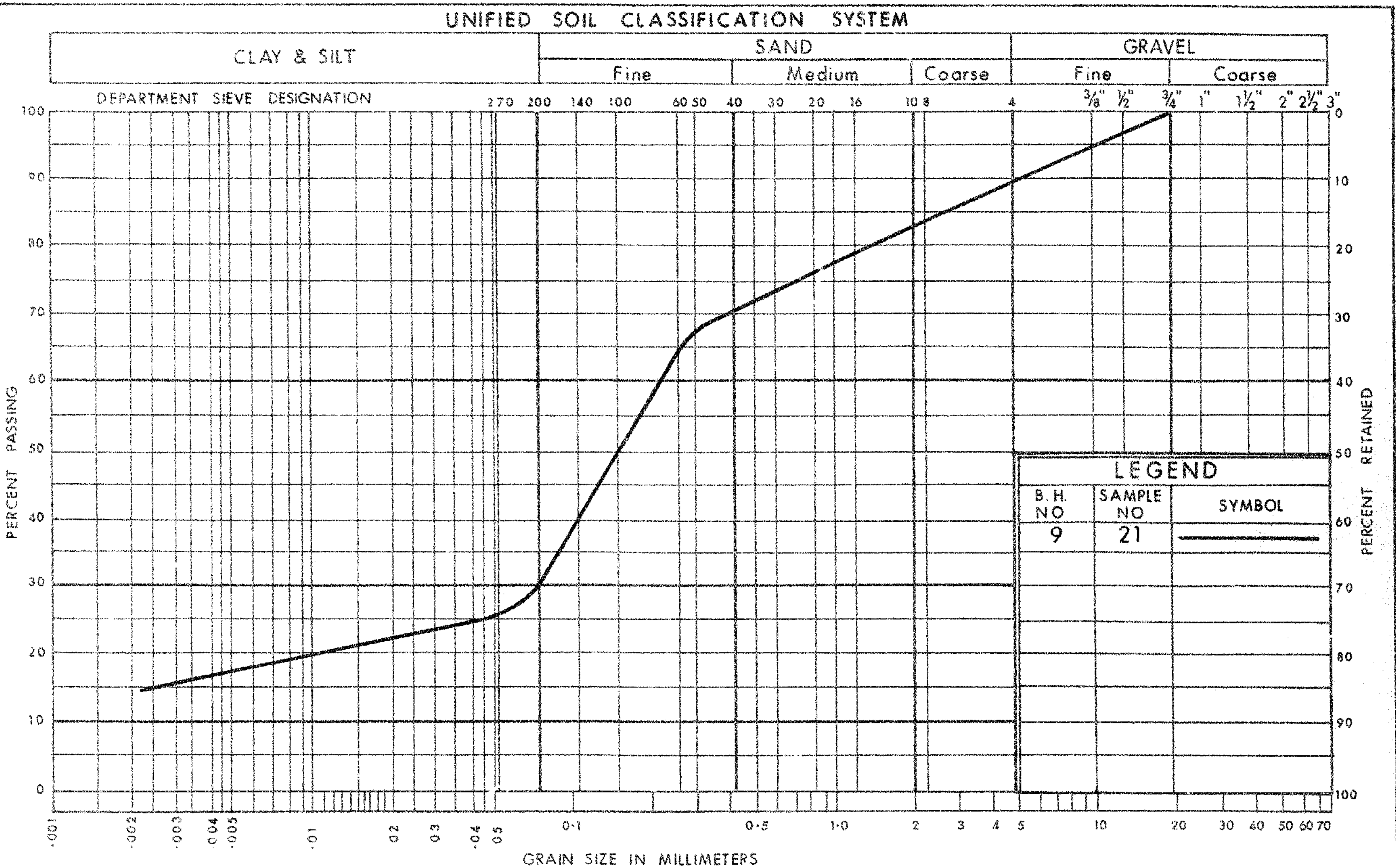


DEPARTMENT OF HIGHWAYS
MATERIALS and TESTING DIVISION

GRAIN SIZE DISTRIBUTION
LAYERED SAND, SILT AND CLAYEY SILT

W.P. No. 34 - 66 - 06
JOB No. 67 - F - 112
FIG. NO. 7

FORM OB MT-17
REV. APR. 1965



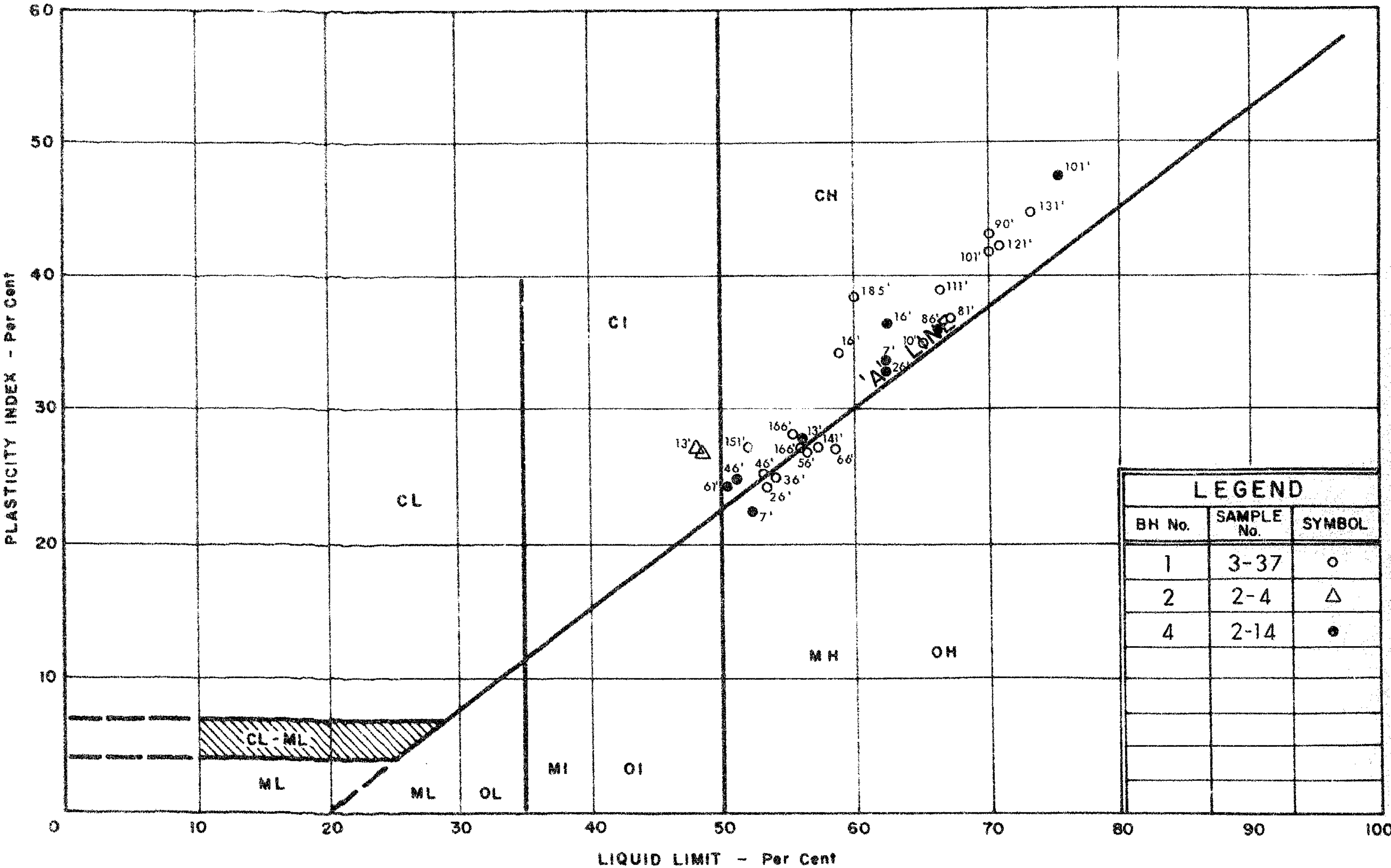
DEPARTMENT OF HIGHWAYS
MATERIALS and TESTING DIVISION

GRAIN SIZE DISTRIBUTION

CLAYEY SILT (GLACIAL TILL)

W.P. No. 34 – 66 - 06
JOB No. 67 – F – 112
FIG. NO. 8

FORM OB-MT-252
REVISED JULY 1966



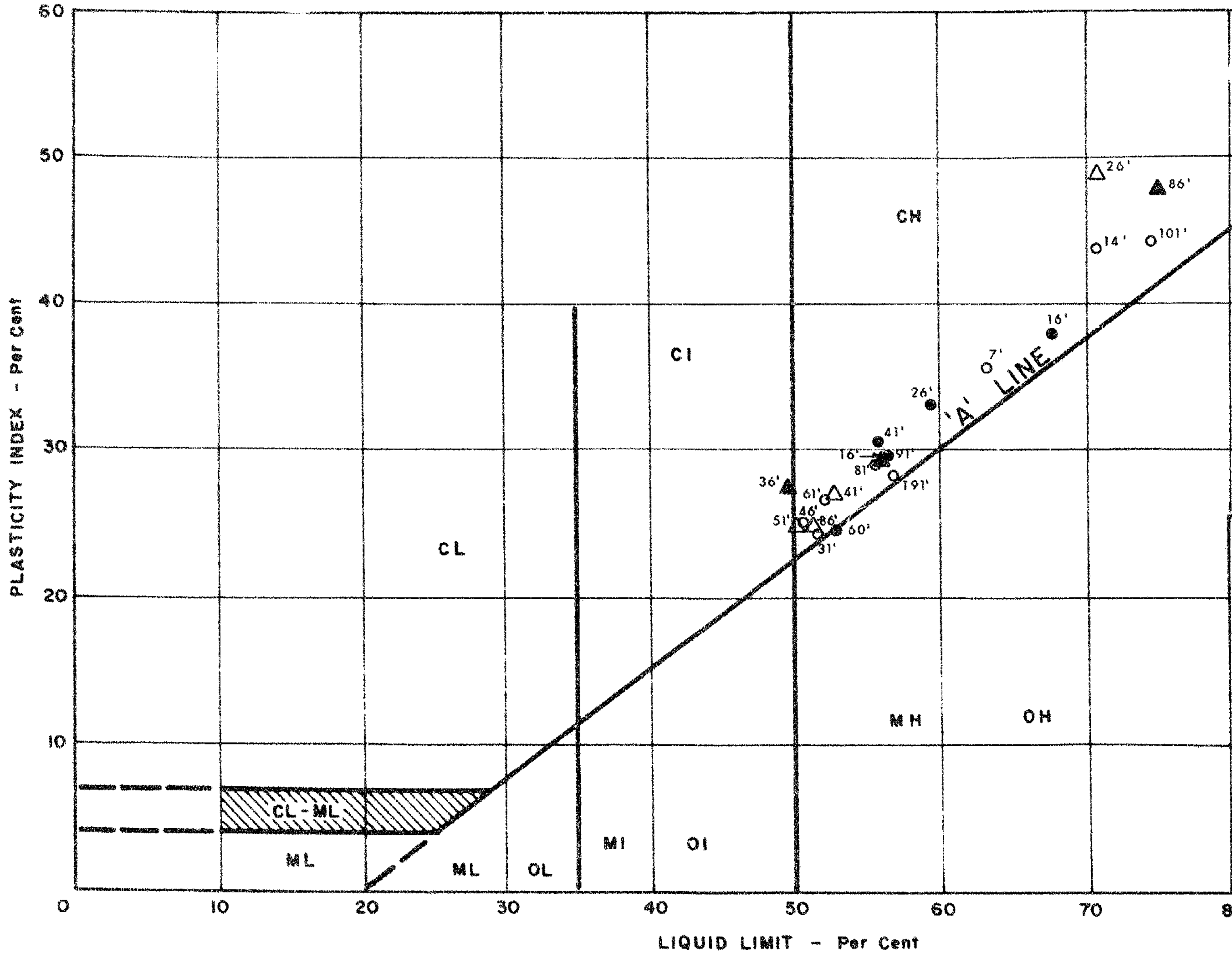
LEGEND		
BH No.	SAMPLE No.	SYMBOL
1	3-37	○
2	2-4	△
4	2-14	●

DEPARTMENT OF HIGHWAYS
MATERIALS and TESTING DIVISION

PLASTICITY CHART
SENSITIVE CLAY

W.P. No. 34-66-06
JOB No. 67-F-112
FIG. NO. 9

FORM OB-MT-252
REVISED JULY 1966



LEGEND

BH No.	SAMPLE No.	SYMBOL
5	5-15	▲
6	4-15	●
9	2-16	o
10	4-14	△

DEPARTMENT OF HIGHWAYS
MATERIALS and TESTING DIVISION

PLASTICITY CHART
SENSITIVE CLAY

W.P. No. 34-66-06
JOB No. 67-F-112
FIG. NO. 10

VOID RATIO vs PRESSURE

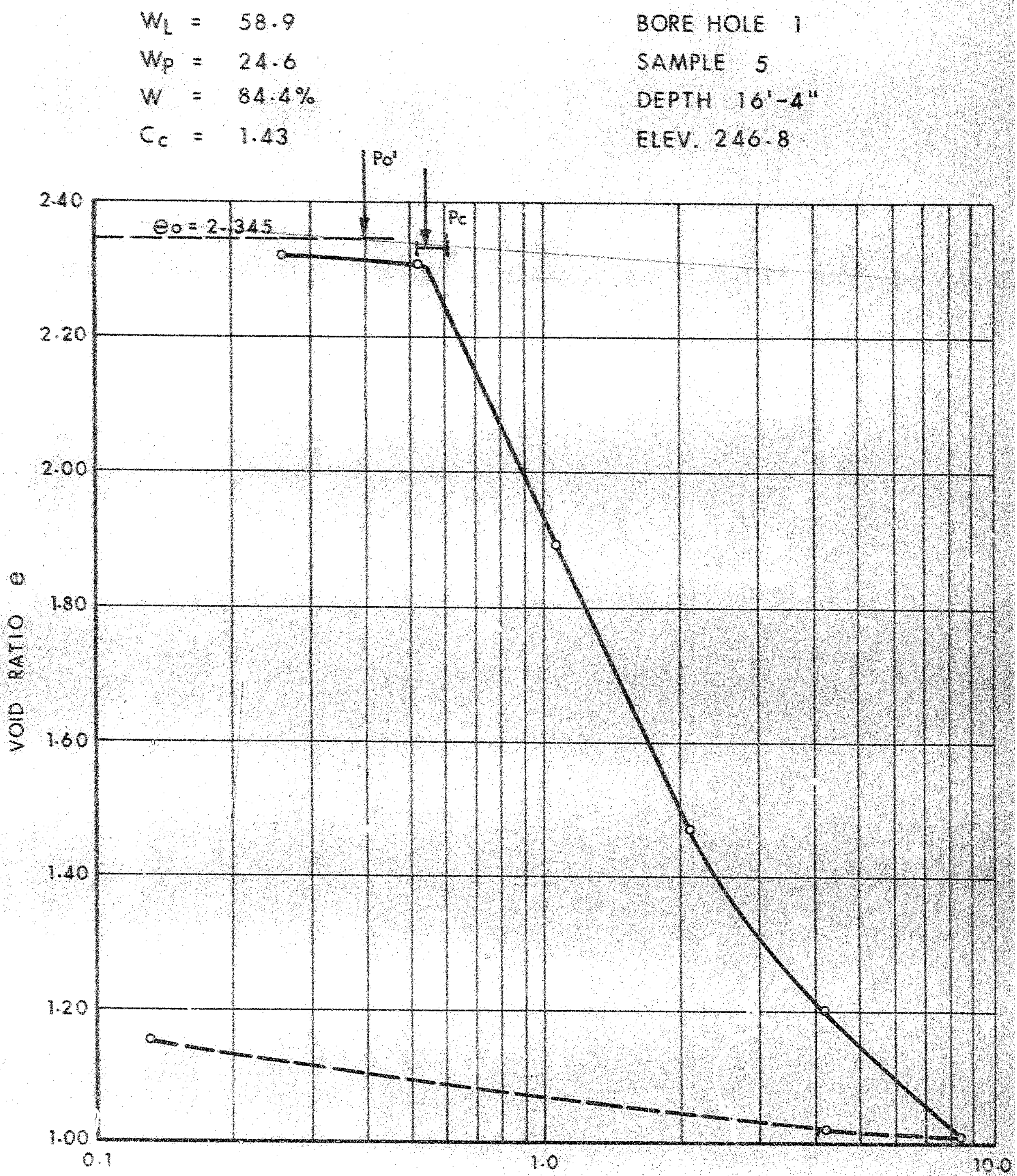


FIG. 11

VOID RATIO vs PRESSURE

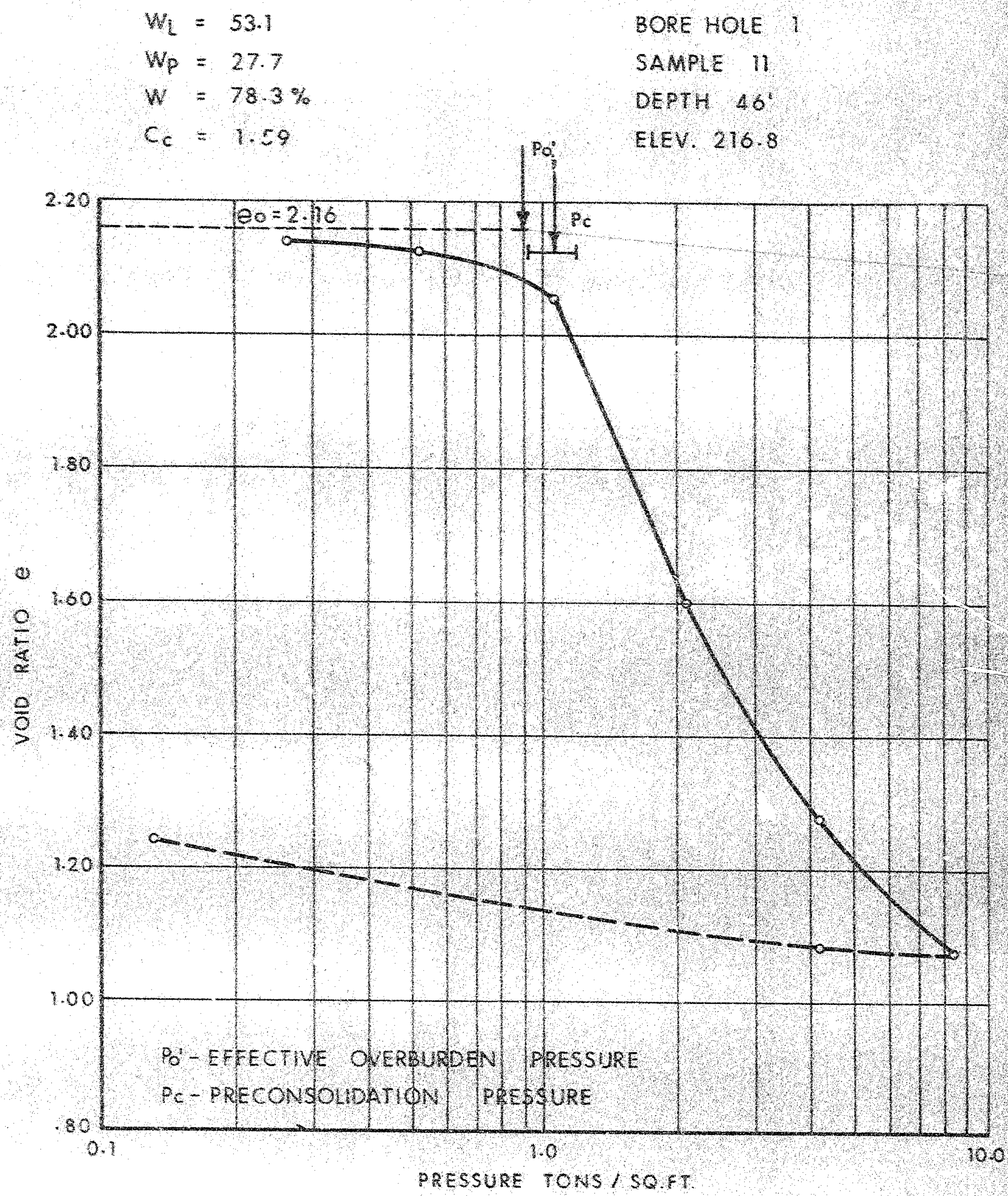


FIG. 2

VOID RATIO vs PRESSURE

W_L = 58.5
W_P = 29.6
W = 77.6%
C_c = 2.24

BORE HOLE 1
SAMPLE 17
DEPTH 73'-3"
ELEV. 186.8

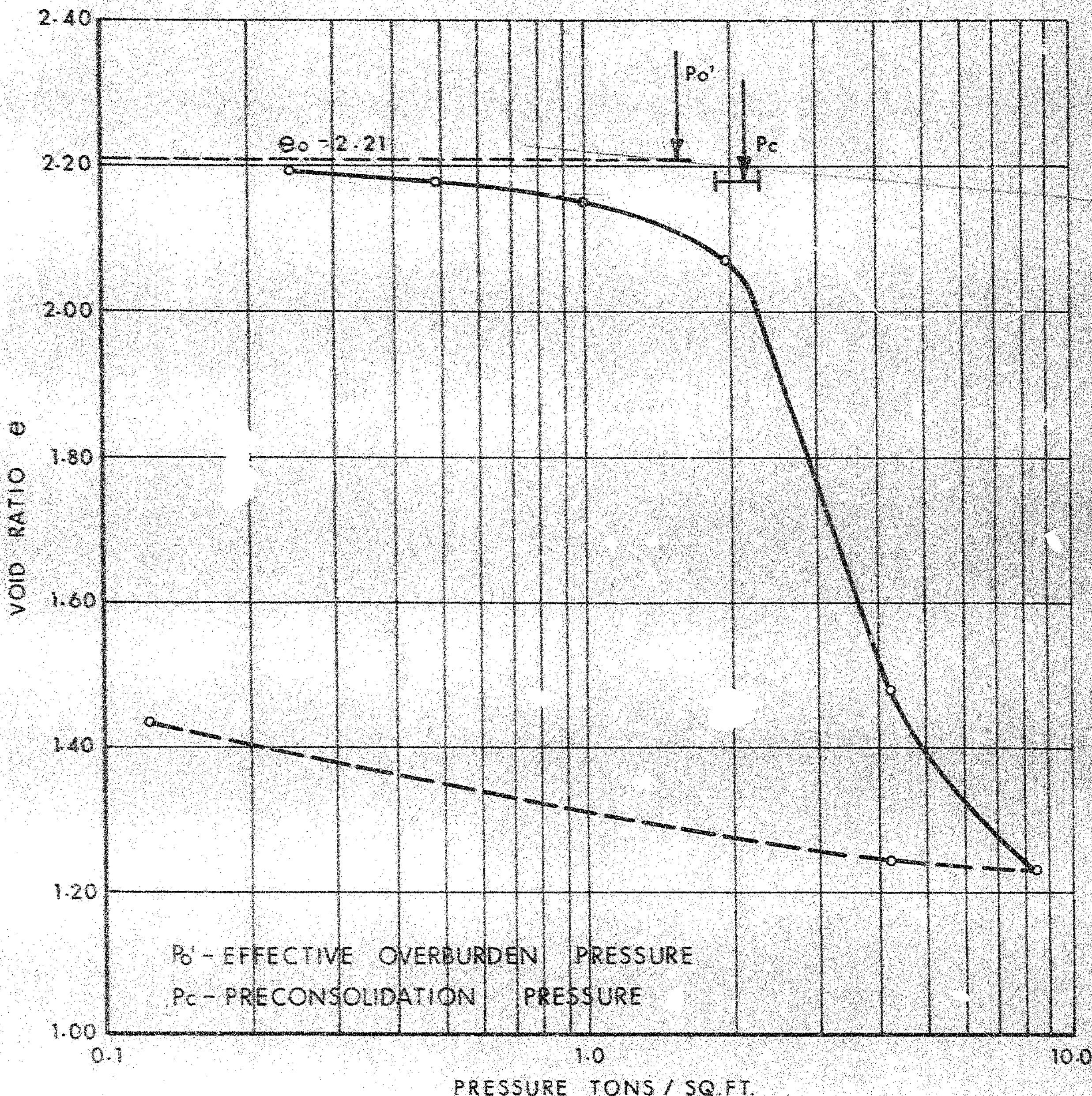


FIG. 13

VOID RATIO vs PRESSURE

W_L = 57.2
W_P = 29.8
W = 57.9
C_c = 1.21

BORE HOLE 1
SAMPLE 30
DEPTH 141'-0"
ELEV. 121.8

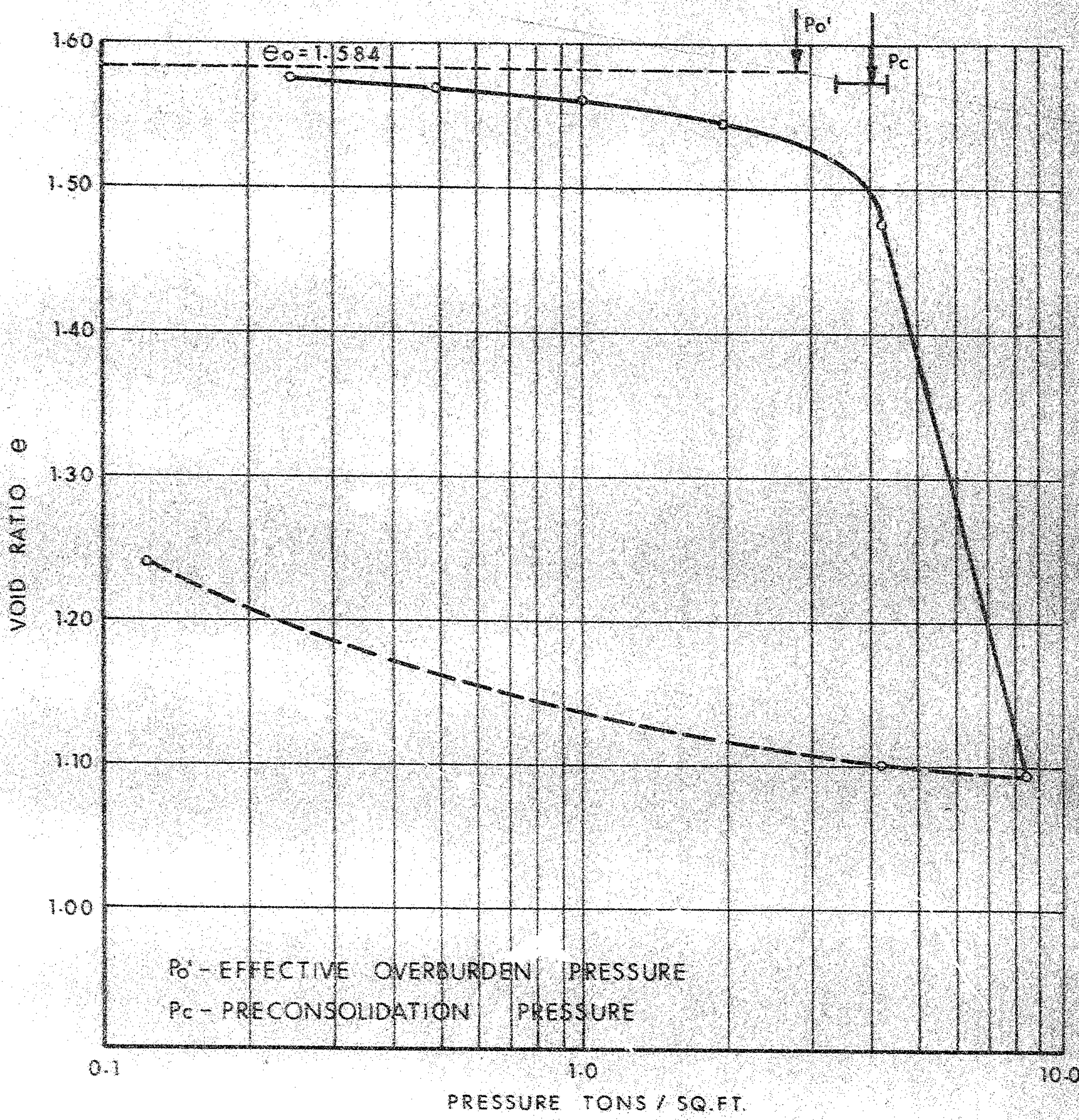


FIG. 14

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 :-

CONSISTENCY	'N' BLOWS / FT.	c LB. / SQ. FT.	DENSENESS	'N' BLOWS / FT.
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 — IN TERMS OF EFFECTIVE STRESS $\tau_f = c' + \sigma' \tan \phi'$
ϕ'	EFFECTIVE ANGLE OF SHEARING RESISTANCE, OR FRICTION — IN TERMS OF EFFECTIVE STRESS $\tau_f = c' + \sigma' \tan \phi'$
c_u	APPARENT COHESION — IN TERMS OF TOTAL STRESS $\tau_f = c_u + \sigma \tan \phi$
ϕ_u	APPARENT ANGLE OF SHEARING RESISTANCE, OR FRICTION — IN TERMS OF TOTAL STRESS $\tau_f = c_u + \sigma \tan \phi$
μ	COEFFICIENT OF FRICTION
S_t	SENSITIVITY

GENERAL

π	$= 3{\cdot}1416$
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 (YOUNGS 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_o	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

cc: Gen. Files

Re: Reviewed Drawings & Comments - Carleton County Rd #27 U/Pass Hwy 417

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

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

May 6, 1969

Carleton County Rd. 27 Underpass
Highway 417, District #9 (Ottawa)
W.J. 67-F-112 -- W.P. 34-66-06

We have reviewed the final bridge design drawings No. D-6824-1 to 6, 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 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.

M. Devata

MD/MdeF

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

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

Foundations Files
Gen. Files

cc: Gen. Files

Re Carleton Rd. #27 Interchange of Ottawa E. Comments on

DEPARTMENT OF HIGHWAYS ONTARIO

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 14, 1968

OUR FILE REF:

IN REPLY TO

SUBJECT: Carleton Co. Rd. #27 (Anderson Rd.) Interchange
3 Miles East of Ottawa East Limits - Site 3-267
W.P. 34-66-06 -- -- -- W.P. 67-F-112
Highway #417 -- District No. 9 (Ottawa)

We have reviewed the Preliminary Drawing D-6484-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	--	9 ft.
Factor of safety against base failure	--	1.30
Factor of safety with a 6-ft. surcharge for the above case (with a 22-ft. berm)	--	1.14

Settlements:

Height of Fill	Total Settlement in Inches for Various Periods				
	2 Yrs.	7 Yrs.	15 Yrs.	25 Yrs.	50 Yrs.
9 ft. (Design height)	6"	12"	15"	18"	24"
15 ft. (Design height + 6-ft. surcharge)	13"	24"	30"	36"	48"
(Percentage Consolidation) -	20%	28%	40%	48%	60%

cont'd. /2 ...

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

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.

M. Devata

MD/MdeF

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

cc: Gen. Files

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

Attention: Mr. S. McCombie

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

June 28, 1968

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

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/AdeP

AGStermac
A. G. Stermac
PRINCIPAL FOUNDATION ENGINEER

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

Foundations Files
Gen. Files

W.P. 34-66-6

RE: PROPOSED STR
SUBSURFACE
INVESTIGATIONS

DEPARTMENT OF HIGHWAYS ONTARIO

MEMORANDUM

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

Attention: Mr. S. McCombie

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

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 ...

ol

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

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:

	Westbound Lane	Eastbound Lane
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 ...

2. Seventh Line Rd. Overpass:

	Westbound Lane	Eastbound Lane
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:

	Westbound Lane	Eastbound Lane
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:

	Westbound Lane	Eastbound Lane
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.

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

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

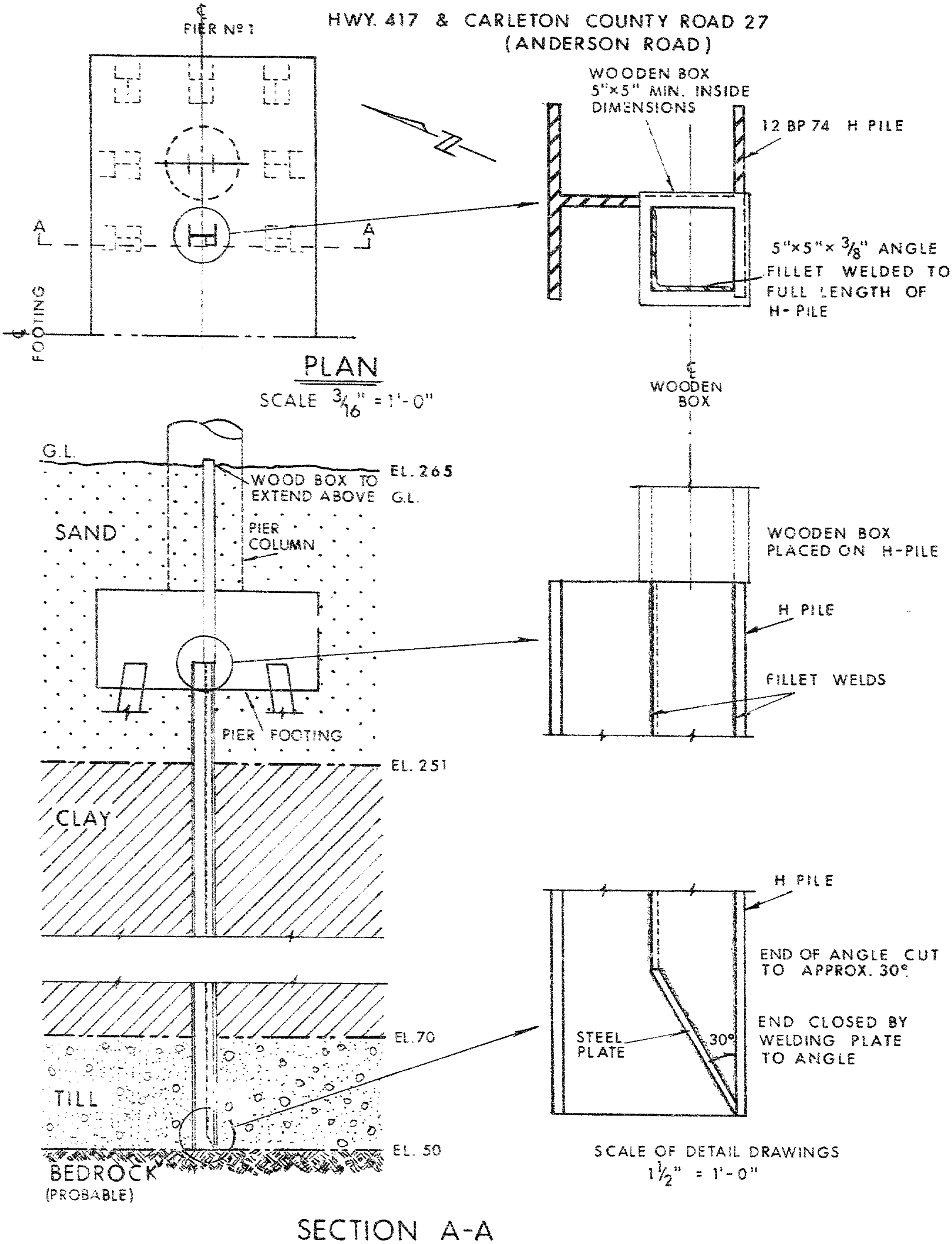
BTD/MdeF

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

Foundations Files
Gen. Files

SLOPE INDICATOR INSTALLATION DETAILS – PIER Nº 1

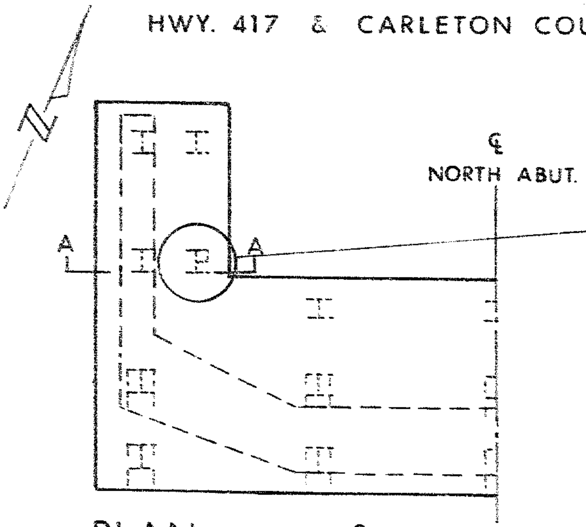
HWY. 417 & CARLETON COUNTY ROAD 27
(ANDERSON ROAD)



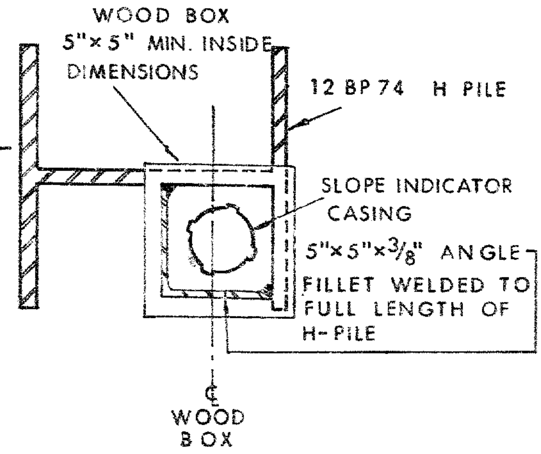
W.P. Nº. 34-66-06
CONT. Nº. 70-64
W.J. Nº. 67-F-112

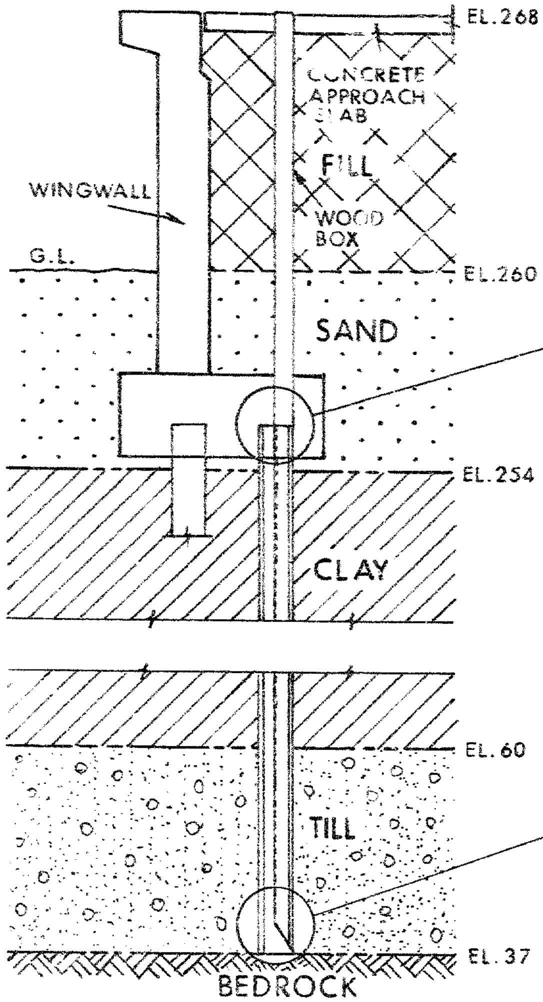
SLOPE INDICATOR INSTALLATION DETAILS - NORTH ABUTMENT

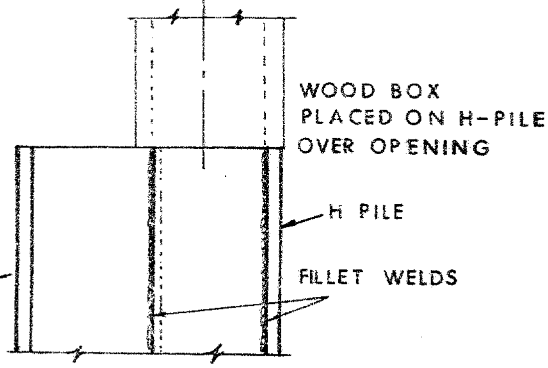
HWY. 417 & CARLETON COUNTY ROAD 27 (ANDERSON ROAD)

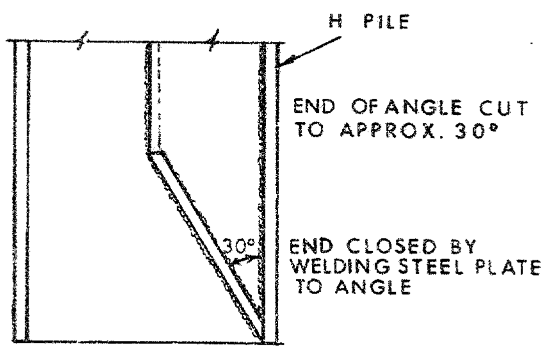


PLAN SCALE $3/16" = 1'-0"$









SECTION A-A SCALE $3/16" = 1'-0"$

SCALE OF DETAIL DRAWINGS
$1\frac{1}{2}" = 1'-0"$

W.P. No. 34-66-06
CONT. No. 70-64
W.J. No. 67-F-112

cc: Foundations Files (Rm. 110)

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

Attention: Mr. S. McCombie

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

June 28, 1968

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

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A G Stermac
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PRINCIPAL FOUNDATION ENGINEER

cc: Messrs. B. R. Davis (2)
G. Scott
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J. L. Forster

Foundations Files
Gen. Files

DEPARTMENT OF HIGHWAYS ONTARIO

MEMORANDUM

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

Attention: Mr. S. McCombie

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

Date: June 26, 1968

Our File Ref.

In reply to

Subject:

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

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cont'd. /3 ...

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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.

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	Westbound Lane	Eastbound Lane
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:

	Westbound Lane	Eastbound Lane
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.

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

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

BTD/MdeF

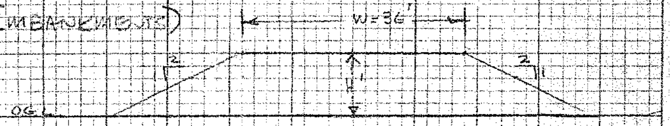
cc: Messrs. B. R. Davis (2)
G. Scott
S. J. Markiewicz
J. E. Gruspier
J. L. Forster

Foundations Files
Gen. Files

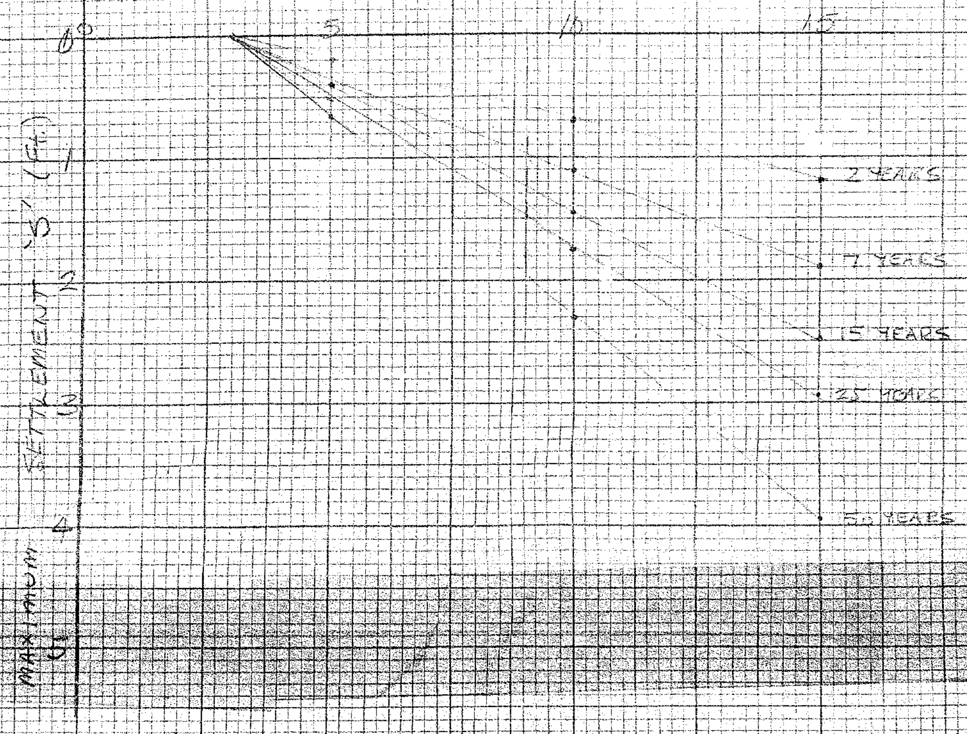
N.J. 57-F-112 W.P. 34-66-06

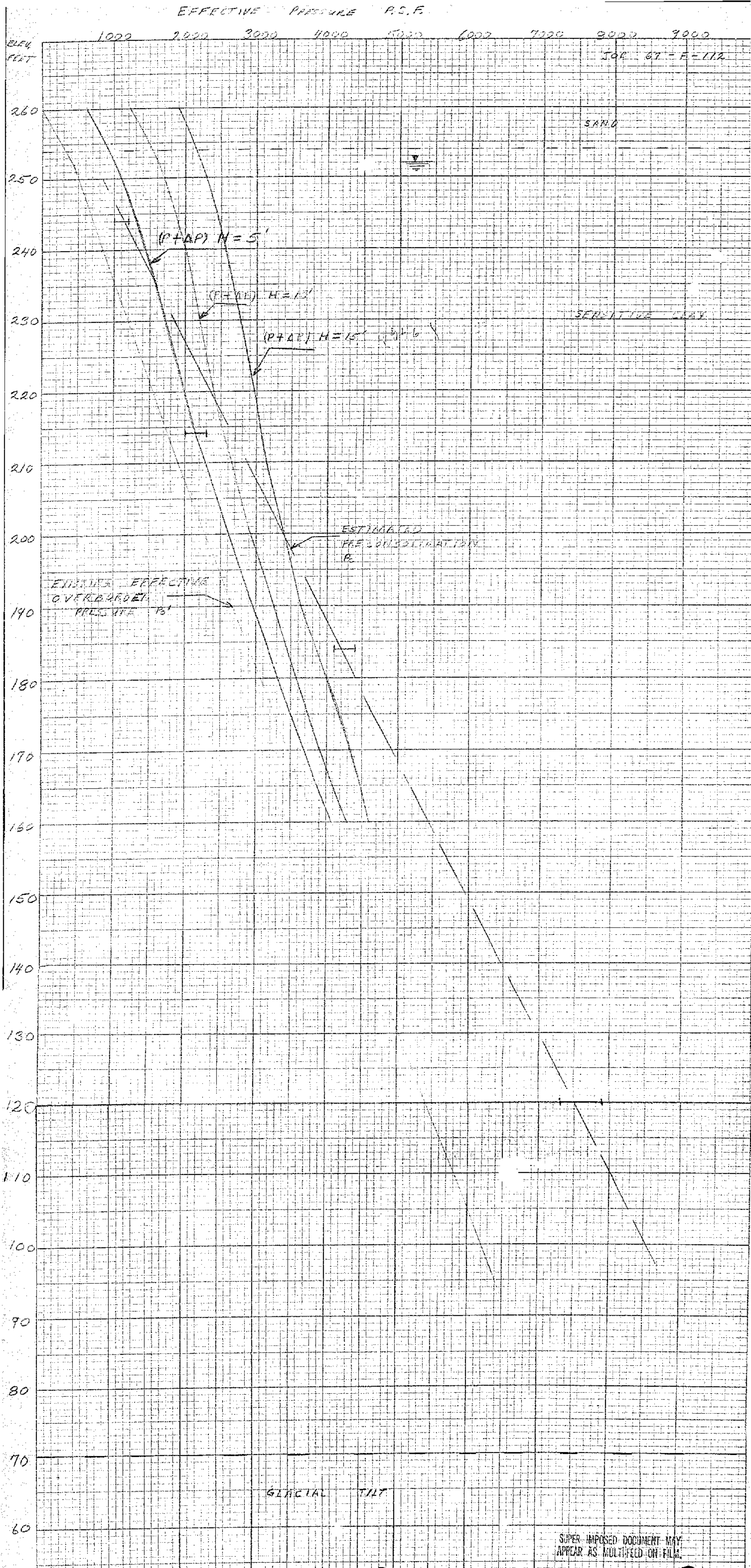
HWY. #417 & ANDERSON RD.
UNDERPASS CROSSING

SUMMARIZED RESULTS OF STABILITY AND SETTLEMENT ANALYSES (APPROACH EMBANKMENTS)



H of FILL (ft)	BERM LENGTH (FS=1.3) (ft)
9	—
12	10
15	22




EFFECTIVE PRESSURE P.S.F.
1000
2000
3000
4000
5000
6000
7000
8000
9000
ELEV. FEET
JOB 67-F-112
SAND
(P+ΔP) H=5'
(P+ΔP) H=10'
(P+ΔP) H=15'
SENSITIVE CLAY
ESTIMATED PRE CONSOLIDATION Pc
EXISTING EFFECTIVE OVERBURDEN PRESSURE Po'
GLACIAL TILL
260
250
240
230
220
210
200
190
180
170
160
150
140
130
120
110
100
90
80
70
60

67-F-112

Department of Highways Ontario
Copy for the information of
Mr. A. Stermac

~~Messrs.~~
G. Scott, Kingston Region,
C.S. Robertson, Ottawa District
L. Forster, Kingston Region
S. [illegible], Kingston Region

Bridge Division,
Downsview, Ontario

October 30, 1968

Carleton Co. Road 27 Interchange
3 Miles East of Ottawa East Limits
W.P. 34-56-06, Site 3-207
Highway 417, District No. 9

Attached herewith are prints of the Preliminary Bridge Plan Drawing D-6484-P1 for the above-mentioned structure.

The estimated cost of the proposed structure is $76[illegible],000. This cost includes tender, materials, engineering and sundry construction.

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

C.S. [illegible],
Bridge Design Engineer

CSG:rd

Attach.

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

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

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.

M. Devata

MD/MdeF

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: Foundations Offic. (Rm. 110)

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,
W.P. 34-66-06 - Anderson Road Interchange Structure - ✓67-F-113
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

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.

cont'd. ... 3

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

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.

AGStermac

AGS/MdeF

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 (2)
Gen. Files

DEPARTMENT OF HIGHWAYS ONTARIO

MEMORANDUM

TO: Mr. A. Stermac,
Principal Foundation Engineer,
DOWNSVIEW, Ontario.

ATT: Mr. M. Devata

FROM: Road Design Division,
KINGSTON, Ontario.

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. Crebski,
J. E. Gruspier and G. Scott

file a28

1968 JAN 3 PM 3 : 11

0293

K

KINR DOWN 2 JAN 3/68 302P VR F

FMARKIEWICZ RGN ROAD DESIGN

68-F-112

ATTN M J MACMASTER

RE HWY 417 RAMSAYVILLE EASTERLY DIST 9 OTTAWA

WP34-66-02

WE HAVE NO COMMENTS PERTAINING TO SURCHARGE DETAILED AT SEVENTH

LINE EIGHTH LINE AND ANDERSON ROAD CROSSING

M DEVATA MAT AND TEST FOR A G STERMAC

BB

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1969 FEB 7 AM 8:51

RECEIVED FEB 1969 SECTION

0012

K

KINR DOWN 1 FEB 7/69 8.42A VR

M J MACMASTER SR PROJECT DESIGN ENGR

FURTHER TO YOUR T T WE ARE GIVING THE SAFE HIGHTS FOR STOCK PILES

USING 2 TO 1 SLOPES AT THE VARIOUS LOCATIONS WHICH ARE AS FOLLOWS

WJ67-F-112

WP34-66-06 ANDERSON RD 13 FT

WJ67-F-113

QP34-66-07 SEVENTH LINE 17 FT

WJ67-F-114

WP34-66-08 EIGHTH 14 FT

WJ68-F-33

WP34-66-09 BOUNDARY RD 14 FT

WJ68-F-52

WP34-66-01 BASE LINE RD 20 FT

M DEVATA SUPVR FOUNDATION ENGR FOR

A G STERMAC PRINC FOUNDATION ENGR

AW

TELETYPE

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1969 FEB 5 PM 1:59 00192

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

M DEVATA FOUNDATION OFFICE

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

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

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

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

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

M J MACMASTER SR PROJECT DESIGN ENGR

JS

TELETYPE TELETYPE

SAFE HEIGHTS FOR FILLS @ 2:1 SIDE SLOPES.

W.P. 34-66.

HWY 417.

W.J. #	W.P. #	Name	SAFE HT. (Ft)	Remarks
~~67-F-111~~	~~34-66-05,14~~	~~Bear Brook~~	~~25'~~	~~above hor. Gr. Surf~~
67-F112	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'	
[illegible]-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. ~~may~~ should be reduced if fill is placed on ground above the valley floor.

GM Feb 6/69

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

69-F-112 W.P. 34-66-06 - Carleton Cty. Rd. 27 Interchange 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.)
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.

W. Lin,
Regional Bridge Project Engineer

WL:rd

Encls.

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

67-F112

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

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 12¾" φ pipe piles in view of -ve skin friction arising from consolidation settlement, if pipe piles used as friction piles.

b). Surcharge only crest & forward slope of fill. F with surcharge = 1.03 i.e. 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.

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

W.P. 34-66-06 - Carleton Cty. Rd. 27 Interchange 67-F-112
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.

W. Lin,
Regional Bridge Project Engineer

WL:rd

Encls.

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

DEPARTMENT OF HIGHWAYS ONTARIO

MEMORANDUM

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

From: C.S. Grebski,
Bridge Office

Attention:

Date: April 30, 1969

Our File Ref.

In reply to

Subject: Carleton Co. Rd. 27 Interchange
3 Miles East of Ottawa East Limits
W.P. 34-66-06, Site No. 3-267
Highway 417, District No. 9

67-F-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.

C.S. Grebski,
Bridge Design Engineer

CSG:rd

Attach.

c.c. Foundation Section

* Please note our comments of April 25th 1969.

~~April~~ May 5th 1969

Department of Highways Ontario
Copy for the information of

Foundation Section

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

C.S. Grebski,
Bridge Office

April 30, 1969

Carleton Co. Rd. 27 Interchange
3 Miles East of Ottawa East Limits
W.P. 34-66-06, Site No. 3-267
Highway 417, District No. 9

67-F-112

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.

C.S. Grebski,
Bridge Design Engineer

CSG:rd

Attach.

c.c. Foundation Section

* Please note our comments of April 25th 1969 regarding W.J. 67-F-113

M. Devata
~~April~~ May 5th 1969

67-F-112

CC235

1969 MAY 9 PM 2:09

DOWN OTTA 3 MAY 9/69 1:55 P

A G STERMAC PRINCIPAL FOUNDATION ENG

ATTN: M DEVATA

RE: CONT 69-28 HWY 417 RAMSAYVILLE EASTERLY 4.77 MI

WE EXPECT THE CONTRACTOR TO START WORK NEAR THE END OF MAY APPROACH FILLS ARE TO BE PLACED AT FOUR SITES WITHIN 30 WORKING DAYS TO ALLOW FOR MAXIMUM SETTLEMENT THE LOCATIONS ARE:

1) BEARBROOK E B L S.A 430 PLUS 10 TO 431 PLUS 70 (67-F-111)

2) ANDERSON RD STA 23 PLUS 80 TO 26 PLUS 60 AND STA 36 PLUS 10 TO 38 PLUS 80 67-F-112

3) 7TH LINE RD STA 24 PLUS 10 TO 27 PLUS 30 AND STA 34 PLUS 25 TO 36 PLUS 90 67-F-113

4) 8TH LINE RD STA 26 PLUS 00 TO 28 PLUS 50 AND STA 33 PLUS 45 TO 36 PLUS 35 67-F-114 (3)

K WESTERBY DIST CONST ENG

SECTION 12
MAY 13 1969
RECEIVED

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cc: Foundations Files (Rm. 110)

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

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

May 6, 1969

Carleton County Rd. 27 Underpass
Highway 417, District #9 (Ottawa)
W.J. 67-F-112 -- W.P. 34-66-06

We have reviewed the final bridge design drawings No. D-6484-1 to 6, 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 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.

M. Devata

MD/MdeF

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

cc: Messrs. S. McCombie
C. Scott
J. E. Gruspier
S. J. Markiewicz

Foundations Files
Gen. Files

69-F-112

DEPARTMENT OF HIGHWAYS ONTARIO

MEMORANDUM

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

From: Bridge Office,
Downsview, Ontario

Attention:

Date: January 21, 1969

Our File Ref.

In reply to

Subject:
W.P. 34-66-07 - 7th Line Road Underpass 460,000 68-F-112
W.P. 34-66-08 - 8th Line Road Underpass 330,000 " 113
W.P. 34-66-06 - Anderson Road Underpass 750,000 " 114
Highway 417, District 9 - Ottawa 1540,000

As end bearing piles will be used for the above-mentioned structures, the piles in some cases will reach 200 feet long.

In view of the poor soil conditions in this area and the unusual pile length, we feel that a pile test will provide us with some pertinent information for the design and construction of the structure foundation.

We estimated that the total cost of the foundation for the three structures will be approximately $300,000.

Wilson Lin

W. Lin,
Regional Bridge Project Engineer

for C.S. Grebski,
Bridge Design Engineer

WL:rd

6F-112

DEPARTMENT OF HIGHWAYS ONTARIO

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	W.P. 34-66-06
Pipe at 7th Line and Prop. Hwy.#417	Elev.	262.235	34-66-07
Pipe at 8th Line and Prop. Hwy. #417	Elev.	258.184	34-66-08

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

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

EBF:WEG.

AGS

DEPARTMENT OF HIGHWAYS ONTARIO

MEMORANDUM

To: Mr. J. E. Callaghan
District Engineer
District #9, Ottawa

From: Foundation Section
Materials and Testing Office
Room 107, Lab. Bldg.

Attention: Mr. K. Westerby
Constr [illegible] Engineer

Date: July 20, 1970

Our File Ref.

In reply to

Subject:

Carlton County Rd. # 27 Interchange
3 mi. east of Ottawa East Limits
Hwy. #417 Dist. #9 (Ottawa)
W.P. 34-66-06 W.J. 67-F-112, cont. 70-64

Further to our recent telephone conversation with regard to slope indicator installations at one of the pile locations of the north abutment footing of the above mentioned structure a drawing showing the details is enclosed. The Foundation Section will supply and supervise the installation of slope indicator casing and in view of this it is suggested that the District should advise this Section of the appropriate time of the driving of piles at the north abutment footing location. It is believed that the District will negotiate with the Contractor for the additional work with regard to installation such as welding of 5" x 5" x 3/8" angle iron to the steel 'H' pile etc. Since piling will be supplied by the Department to the Contractor, we request your office to make the necessary arrangements in obtaining the required length of 5" x 5" x 3/8" angle iron for the Contractor. Should you require any additional information with regard to the above mentioned project, please call our office.

M. Devata
M. Devata
SUPERVISING FOUNDATION ENGINEER

MD:lm

cc: Messrs. C. S. Grebski
A. McKim

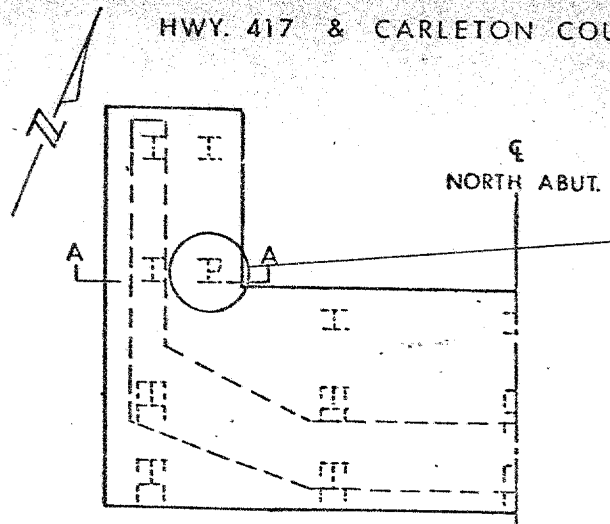
Foundation Files
Gen. Files

For:

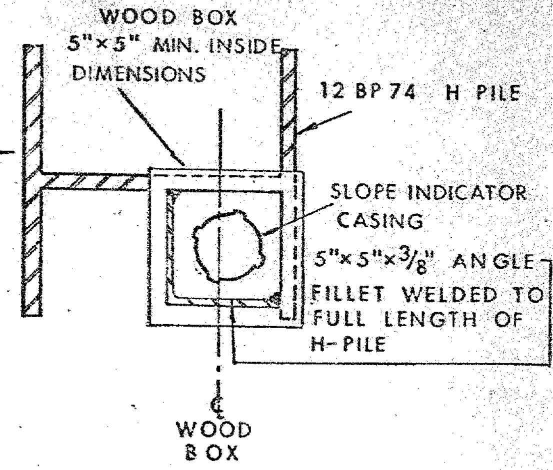
A. G. Stermac
PRINCIPAL FOUNDATION ENGINEER

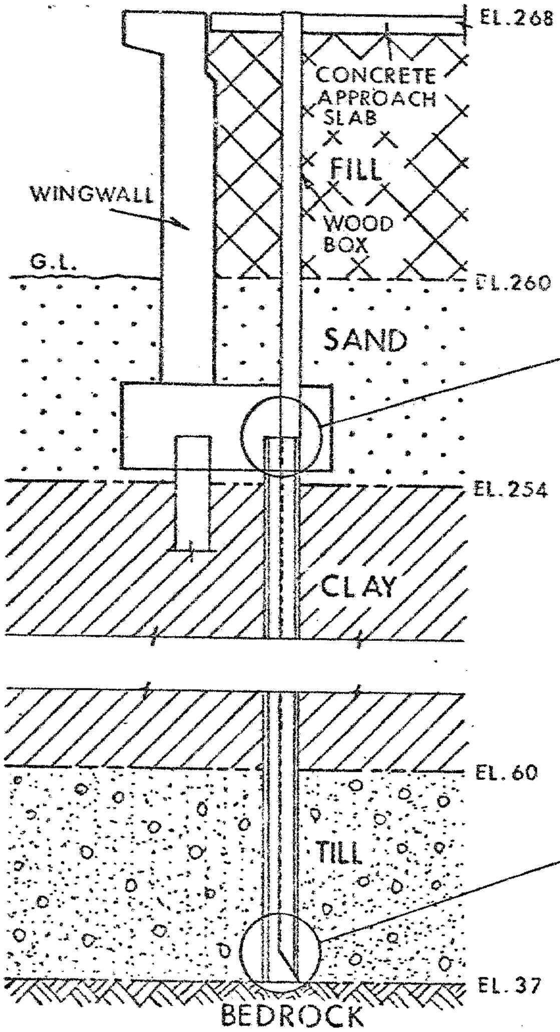
SLOPE INDICATOR INSTALLATION DETAILS - NORTH ABUTMENT

HWY. 417 & CARLETON COUNTY ROAD 27 (ANDERSON ROAD)

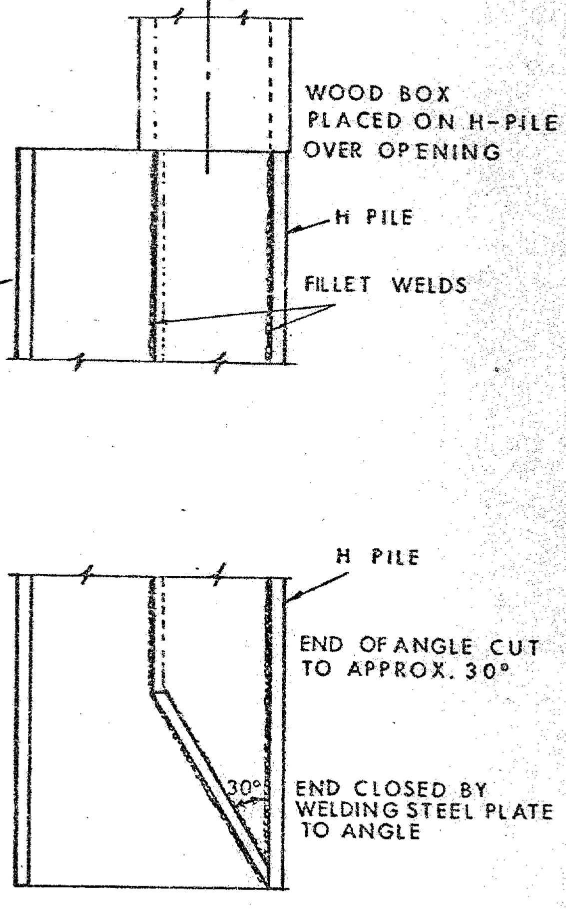


PLAN SCALE 3/16" = 1'-0"





SECTION A-A SCALE 3/16" = 1'-0"



SCALE OF DETAIL DRAWINGS 1½" = 1'-0"

W.P. No. 34-66-06
CONT. No. 70-64

DEPARTMENT OF HIGHWAYS ONTARIO

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:

67-F-112

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.

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

~~69-F-35~~

[redacted] W.P. 34-66-06 - Carleton Cty. Rd. 27 Interchange anderson 67-F-112
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.

W. Lis,
Regional Bridge Project Engineer

WL:rd

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 provid 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

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 12¾" ⌀ pipe piles in view of -ve skin friction arising from consolidation settlement, if pipe piles used as friction piles.

b). Surcharge only crest & forward slope of fill. F with surcharge = 1.03 ∴ marginal.

Bear Brook - EBL   WP 34-66-05
Drwg D-6467-P1

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

Bear Brook - WBL   WP 34-6[illegible]-[illegible]4
Drwg. D-6578-P1

No comments.

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

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.

W. Lin,
Regional Bridge Project Engineer

WL:rd

Encls.

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

67-11-112

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, 67-11-112
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

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-85~~

67-F-112

M.D. File Plan

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

Mr. K. Bassi

Structural Planning Office,
Kingston, Ontario.

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 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

TCK/hl
att.

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

RECEIVED
FOUNDATIONS OFFICE
MAR 5 1974
Ministry of Transportation & Communications

BA2812-A

DEPARTMENT OF HIGHWAYS ONTARIO

MEMORANDUM

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

Attn: Mr. S. McCombie

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

Date: June 25, 1968

Our File Ref.

In reply to

Subject:

FOUNDATION INVESTIGATION REPORT*
For
Eastbound Lane and Westbound Lane
Structures at the Crossing of
Anderson Rd. and Proposed Hwy. #417
District No. 9 (Ottawa)
W.J. 67-F-112 -- W.P. 34-66-06

* (Report distributed April, 1968)

REVISION:

Figure No. 2, contained in the Appendix of the above mentioned report, has been revised.

Would you, therefore, please delete and destroy the existing Figure No. 2 and insert the revision attached hereto.

Thank you.

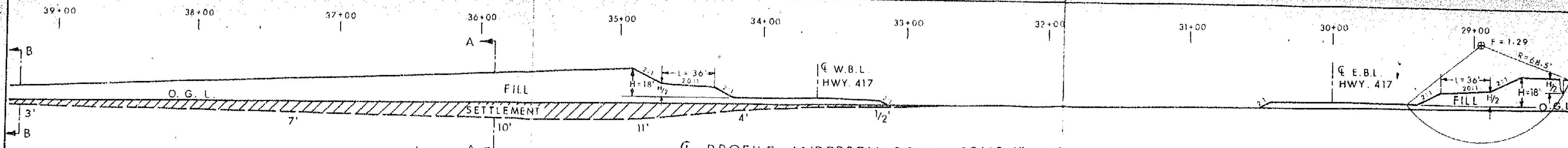
M. Devata

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

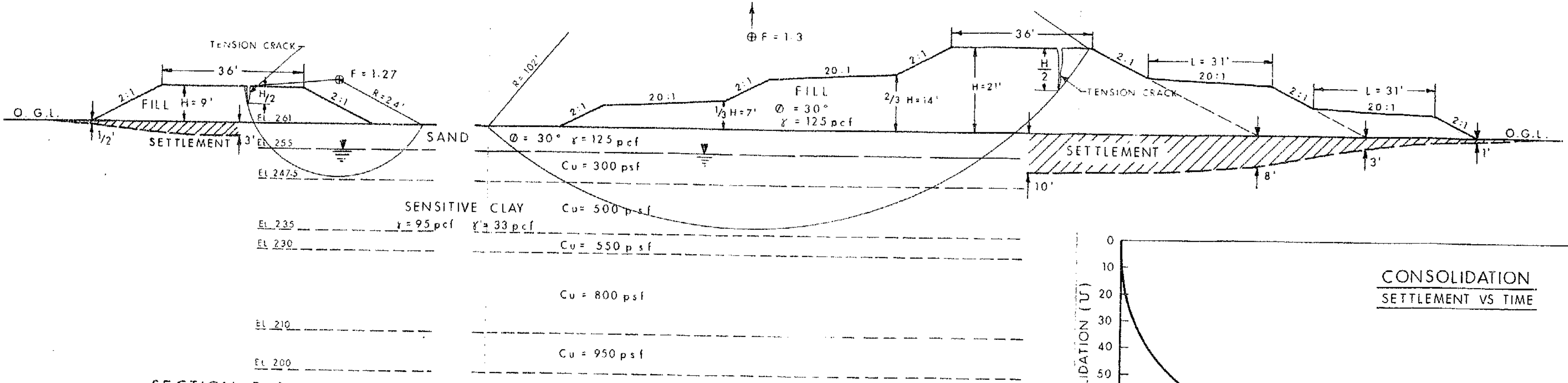
MD/MdeF
cc: Messrs. B. R. Davis (2)
H. A. Tregaskes
D. W. Farren
S. J. Markiewicz
C. R. Robertson
G. Scott
J. E. Gruspier
J. L. Forster
B. A. Singh

Foundations Files
Gen. Files

70-

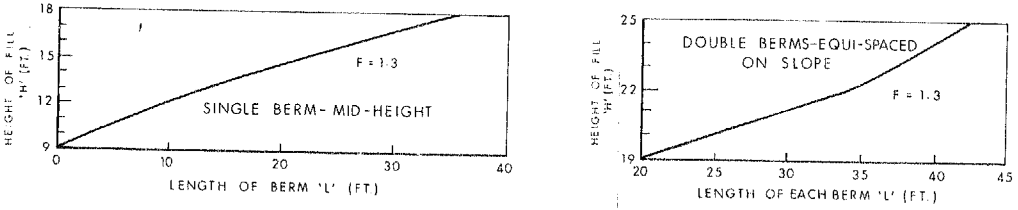


℄ PROFILE ANDERSON ROAD SCALE 1" = 50'

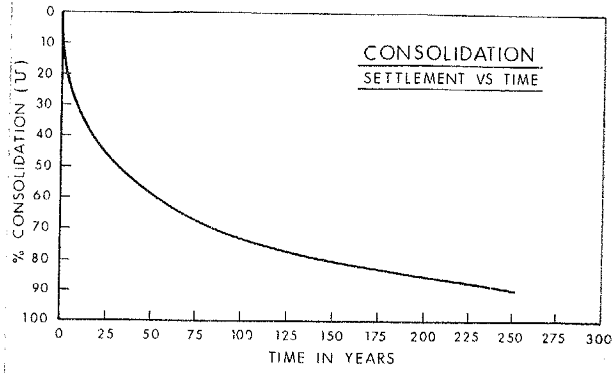


SECTION B-B SCALE 1" = 20'

SECTION A-A



BERM REQUIREMENTS - TRANSVERSE DIRECTION



LEGEND

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

ONTARIO DEPARTMENT OF HIGHWAYS MATERIALS and TESTING DIVISION	SUMMARIZED RESULTS OF STABILITY & SETTLEMENT ANALYSES (APPROACH EMBANKMENTS) W.P. 34-66-06 DIST. 9 JOB 67-F-112
DATE APRIL 4, 1968	APPROVED [signature] FIGURE NO. 2

BN 28/2

cc: Mr. S. McCombie

Mr. J. L. Forster,
Regional Functional Planning Engineer.

Regional Office,
KINGSTON, Ont.

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

June 13, 1968

BN-2153-266
2212-267
2213-268
2204-269

W.P. 34-66, Hwy. 417, Ramsayville to Vars
-- District No. 9 (Ottawa) --

With reference to your memos of May 22nd and June 10th, but without the information which you had requested on June 3rd from Mr. H. Aron, Regional Services Manager, we herewith submit our comments.

It is estimated that the quantity of water seeping from the surface granular layer into the excavation will be small. This estimate refers to the long-term problem. During construction, depending on the time, somewhat larger quantities could be encountered, but these conditions can very easily be coped with.

The quantity of water from rainfalls and snow melting will be much larger, and it will mainly govern the drainage and pumping facilities.

The above estimates are based on our own understanding of the problem area and, also, on the information gathered from the National Research Council. According to their knowledge, the sand layer is at times devoid of water. However, this information or finding may not apply to all our sites. The location of wells, their depths and water quantities in the vicinity of our structures will certainly shed more light on this problem.

Shortly we will submit our preliminary recommendations regarding the overpass alternatives.

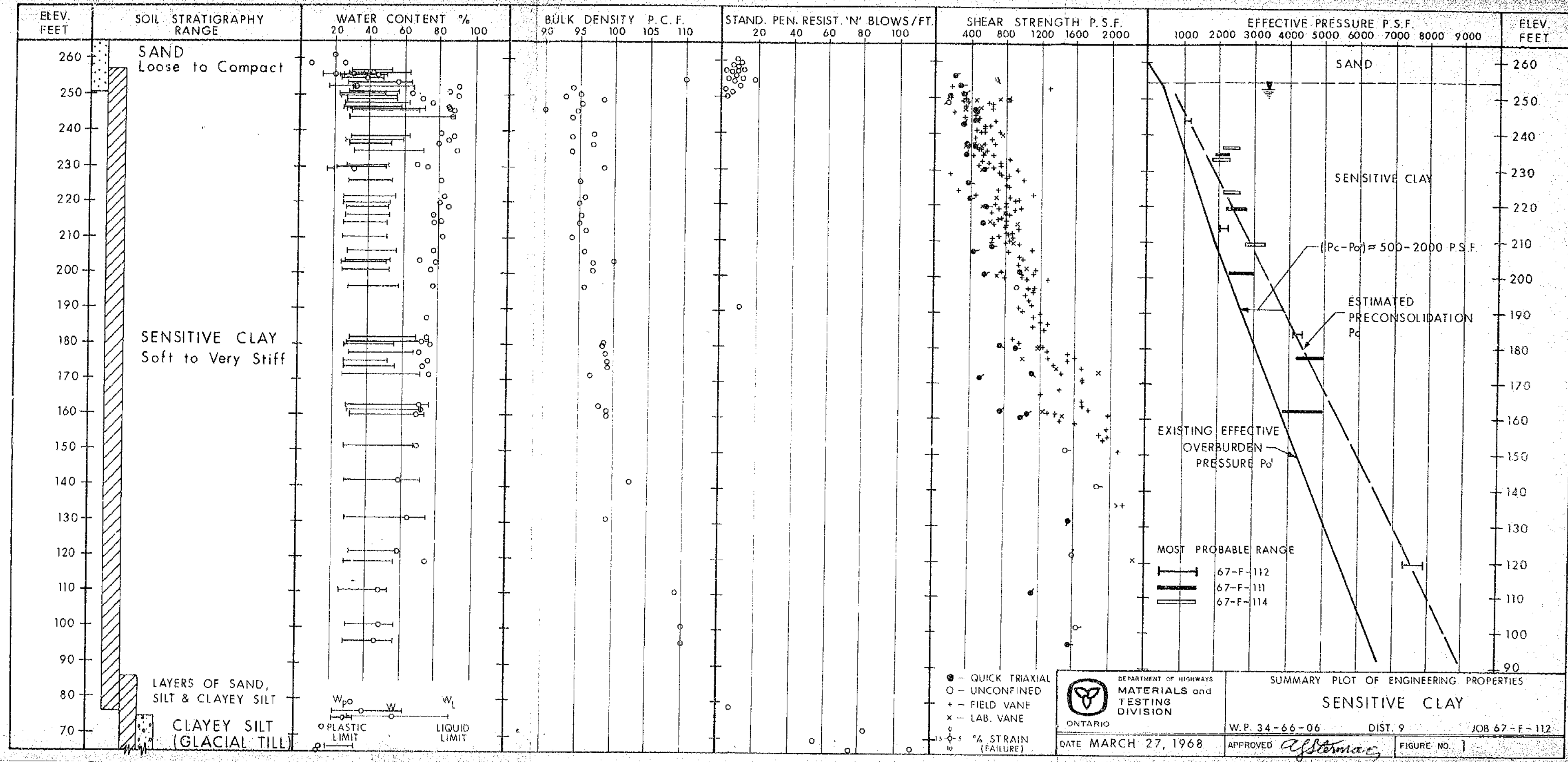
A. G. Stermac

A. G. Stermac,
PRINCIPAL FOUNDATION ENGINEER

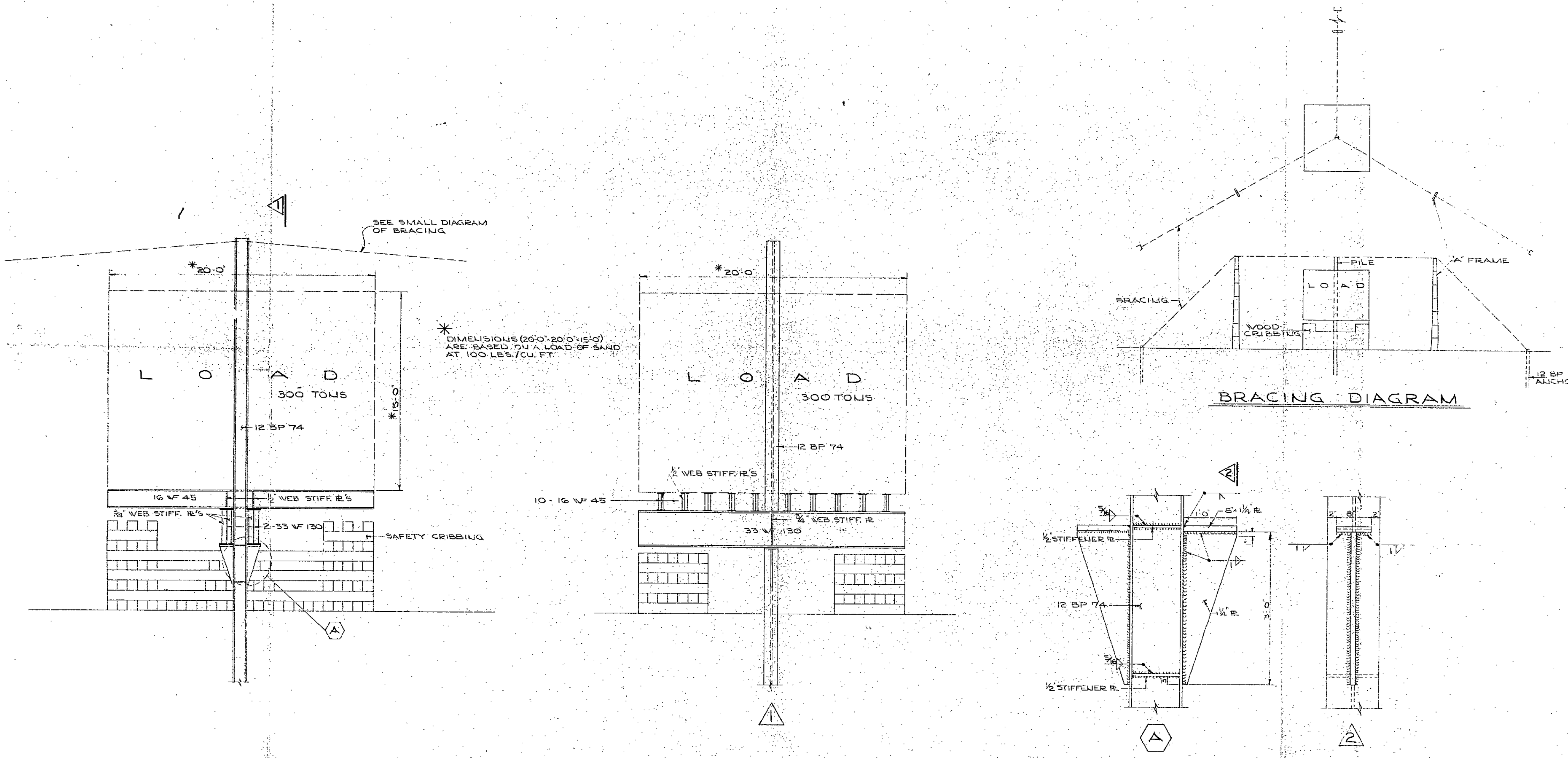
AGS/MdeF

cc: Messrs. C. Scott
J. E. Gruspier
C. S. Grebski
S. J. Markiewicz
S. McCombie
W. Wiglo

Foundations Files
Gen. Files


ELEV. FEET
SOIL STRATIGRAPHY RANGE
WATER CONTENT %
BULK DENSITY P.C.F.
STAND. PEN. RESIST. 'N' BLOWS/FT.
SHEAR STRENGTH P.S.F.
EFFECTIVE PRESSURE P.S.F.
SAND Loose to Compact
SENSITIVE CLAY Soft to Very Stiff
LAYERS OF SAND, SILT & CLAYEY SILT
CLAYEY SILT (GLACIAL TILL)
Wpo W WL
PLASTIC LIMIT
LIQUID LIMIT
QUICK TRIAXIAL
UNCONFINED
FIELD VANE
LAB. VANE
% STRAIN (FAILURE)
SAND
SENSITIVE CLAY
(Pc−Po') ≈ 500−2000 P.S.F.
ESTIMATED PRECONSOLIDATION Pc
EXISTING EFFECTIVE OVERBURDEN PRESSURE Po'
MOST PROBABLE RANGE
67-F-112
67-F-111
67-F-114
DEPARTMENT OF HIGHWAYS
MATERIALS and TESTING DIVISION
ONTARIO
SUMMARY PLOT OF ENGINEERING PROPERTIES
SENSITIVE CLAY
W.P. 34-66-06
DIST. 9
JOB 67-F-112
DATE MARCH 27, 1968
APPROVED
FIGURE NO. 1

#67-F-112
W.P. #34-66-06
Hwy #417
CARLETON CO.
RD. #27
INTERCHANGE



PRINT RECORD

No.	FOR	DATE

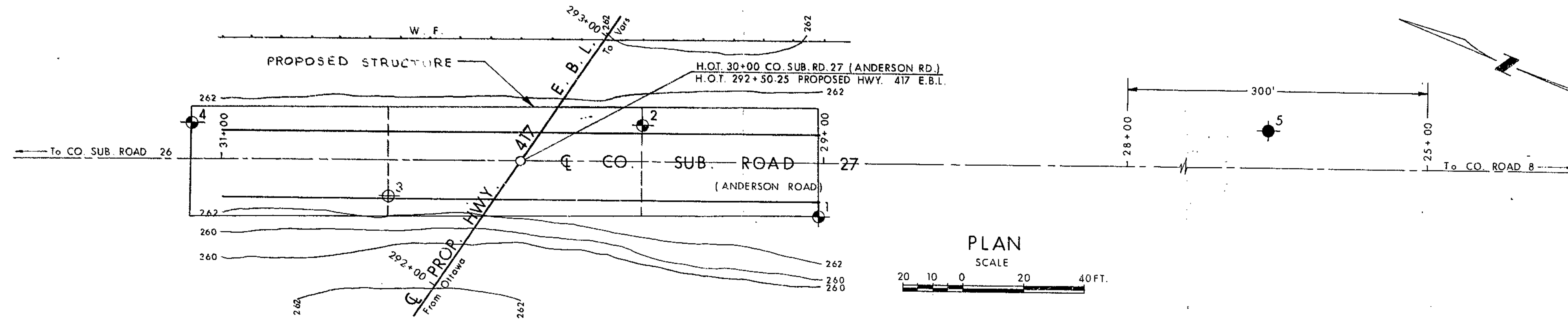
REVISIONS		
DATE	BY	DESCRIPTION

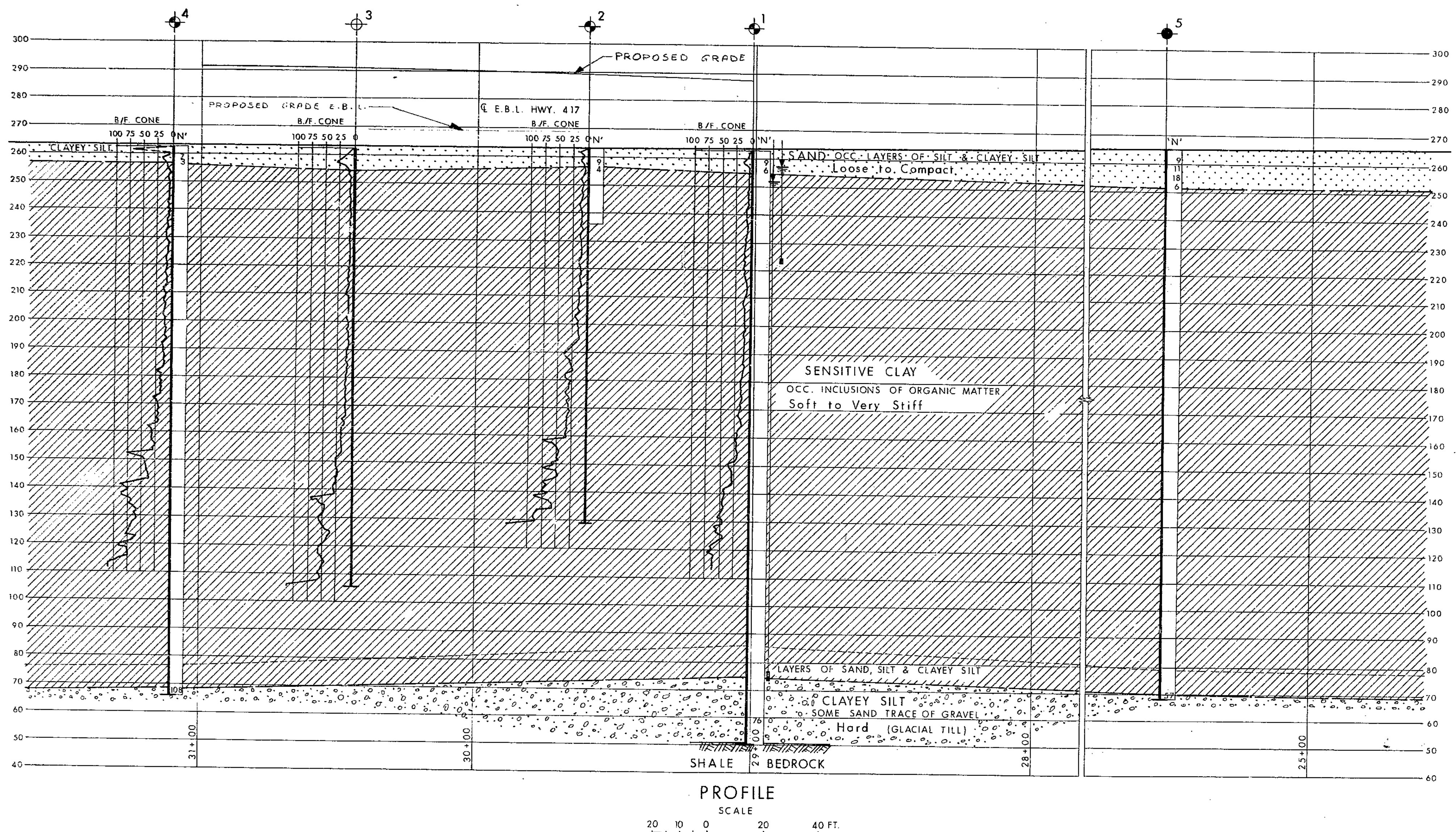
DEPARTMENT OF HIGHWAYS ONTARIO
BRIDGE DIVISION

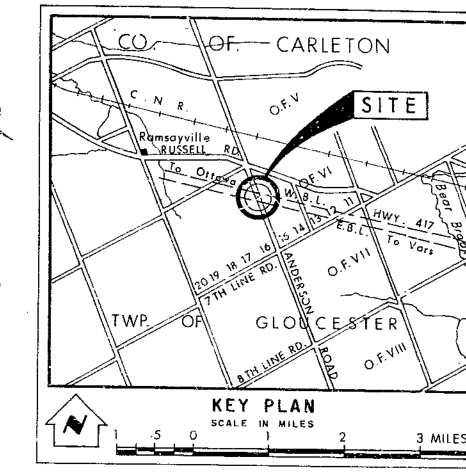
LOAD TEST OF "H" PILE

KING'S HIGHWAY No. DIST. No.
CO.
TWP. LOT CON.

APPROVED		SITE No.	W.P. No.
	BRIDGE ENGINEER	CONTRACT Nos.	
DESIGN	CHECK		
DRAWING	CHECK	DRAWING No.	
DATE	LOADING		







LEGEND

Bore Hole

Cone Penetration Hole

Bore & Cone Penetration Hole

Water Levels established at time of field investigation.

Piezometer

NO.	ELEVATION	STATION	OFFSET
1	262.8	29+02	18' LT.
2	263.4	29+60	12' RT.
3	262.3	30+44	12' LT.
4	262.8	31+10	12' RT.
5	264.8	25+[illegible]3	12' 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.

REVISIONS		
DATE	BY	DESCRIPTION

PRINT RECORD		
NO.	FOR	DATE

REF. NO. E-4648-1

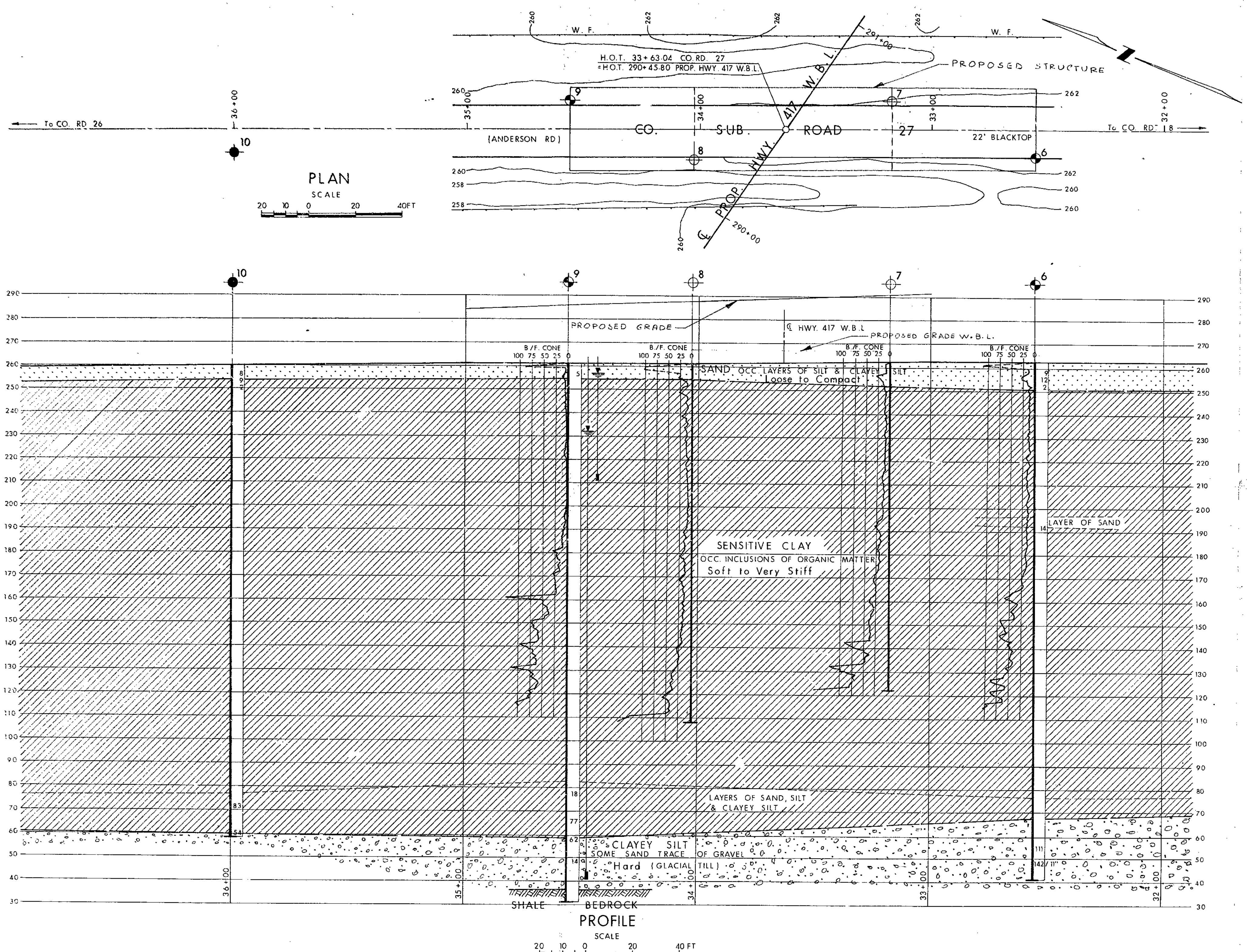
DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING DIVISION - FOUNDATION SECTION

COUNTY SUB. ROAD 27
(ANDERSON ROAD)

KING'S HIGHWAY NO. 417 E.B.L. DIST. NO. 9
CO. CARLETON
TWP. GLOUCESTER LOT 15 & 16 CON. VI

BORE HOLE LOCATIONS & SOIL STRATA

SUBM'D. B.D.	CHECKED PTD	W.P. NO. 34-66-06	M.&T. DRAWING NO. 67-F-112A
DRAWN G.P.	CHECKED	JOB NO. 67-F-112	
DATE MARCH 28, 1968		SITE NO	BRIDGE DRAWING NO.
APPROVED PRINCIPAL FOUNDATION ENGINEER		CONT NO	



REF. NO. E- 4646-1

SEE DRAWING NO. 67-F-112A

KEY PLAN
SCALE IN MILES

LEGEND

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

NO.	ELEVATION	STATION	OFFSET
6	262.6	32+55	12' LT.
7	262.1	33+18	12' RT.
8	261.7	34+03	13' LT.
9	260.7	34+56	12' RT.
10	260.5	36+00	10 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.

PRINT RECORD

NO	FOR	DATE

REVISIONS

DATE	BY	DESCRIPTION

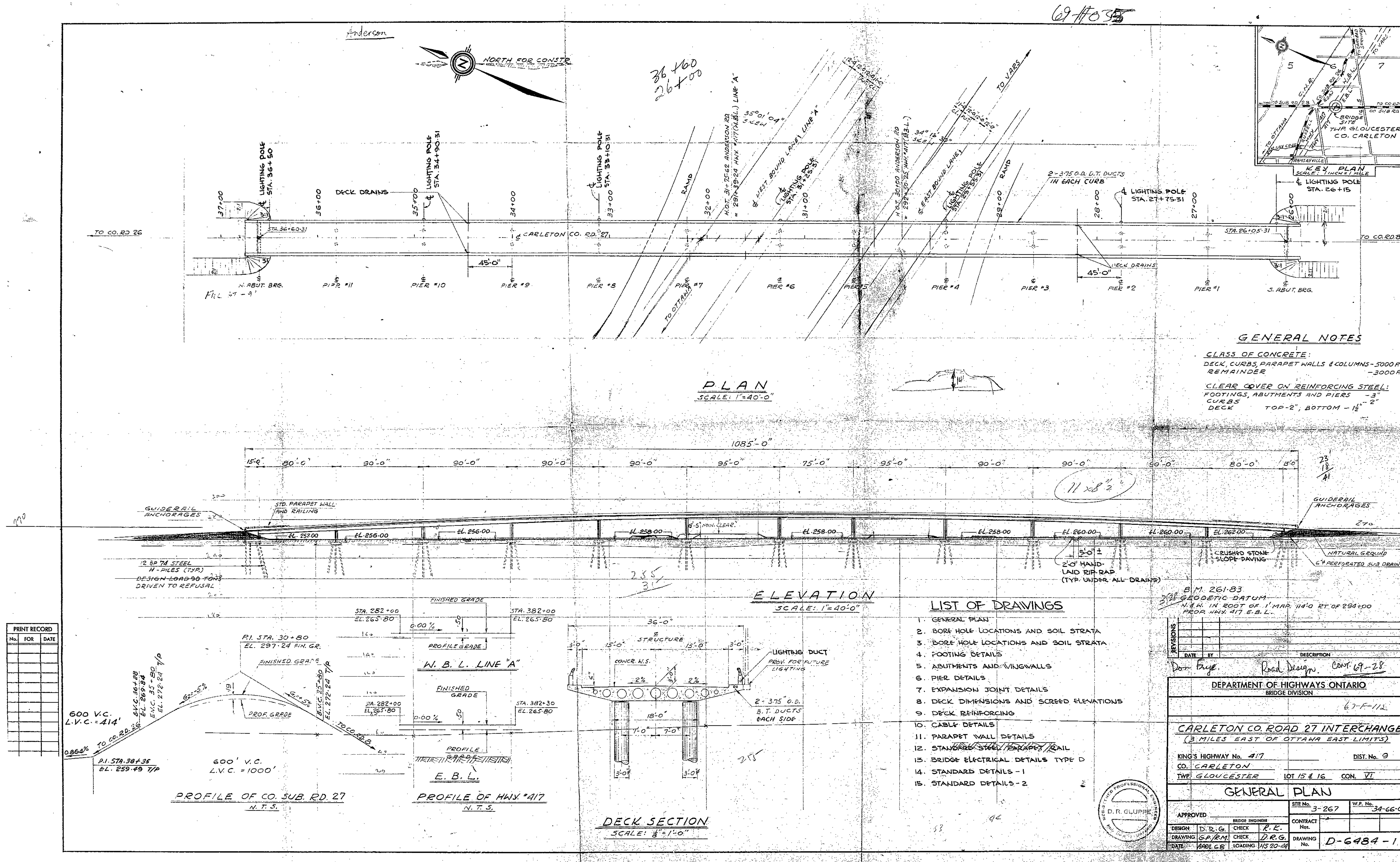
DEPARTMENT OF HIGHWAYS – ONTARIO
MATERIALS & TESTING DIVISION – FOUNDATION SECTION

COUNTY SUB. ROAD 27
(ANDERSON ROAD)

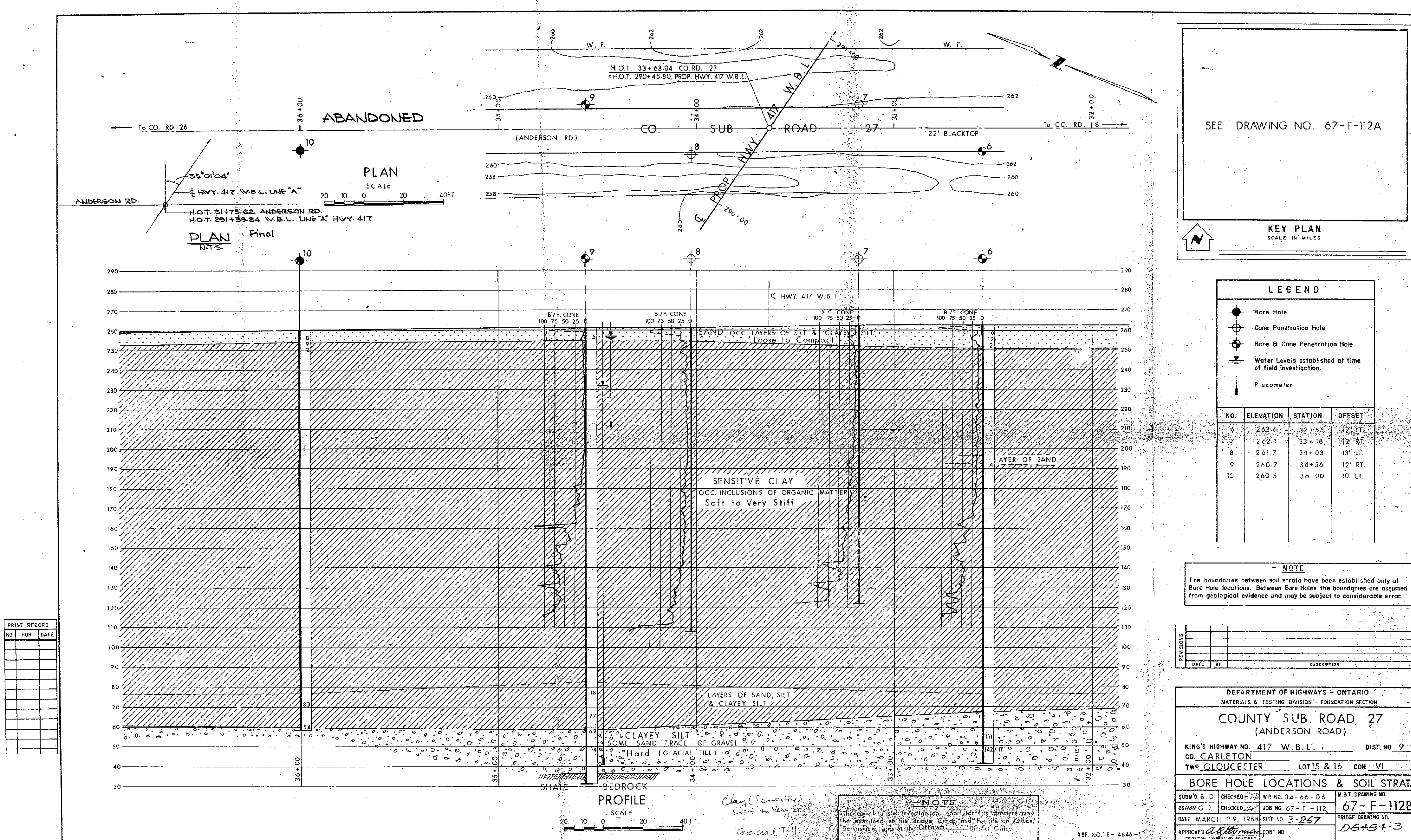
KING'S HIGHWAY NO. 417 W.B.L. DIST. NO. 9
CO. CARLETON
TWP. GLOUCESTER LOT 15 & 16 CON. VI

BORE HOLE LOCATIONS & SOIL STRATA

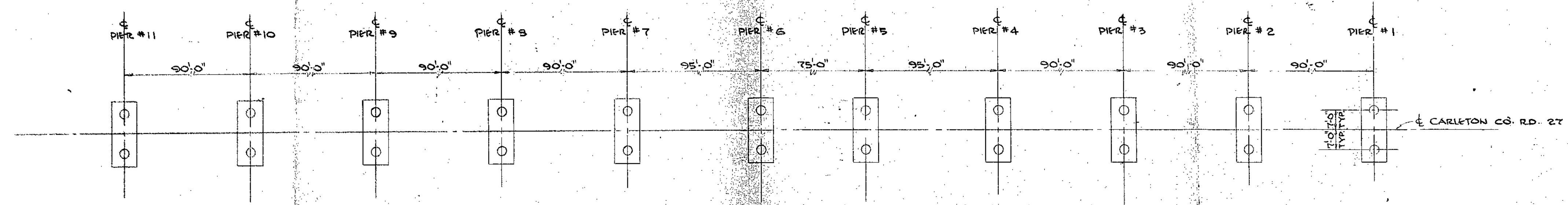
SUBM'D. B. D.	CHECKED E.T.D.	W.P. NO. 34-66-06	M.& T. DRAWING NO. 67-F-112B
DRAWN G. P.	CHECKED	JOB NO. 67-F-112	
DATE MARCH 29, 1968		SITE NO	BRIDGE DRAWING NO.
APPROVED (signature) PRINCIPAL FOUNDATION ENGINEER		CONT. NO.	



69-F-035
Anderson
NORTH FOR CONSTR.
KEY PLAN
SCALE: 1 INCH = 1 MILE
BRIDGE SITE
TWP. GLOUCESTER
CO. CARLETON
RAMSAYVILLE
LIGHTING POLE STA. 36+50
LIGHTING POLE STA. 34+90·31
LIGHTING POLE STA. 33+10·31
LIGHTING POLE STA. 27+75·31
LIGHTING POLE STA. 26+15
DECK DRAINS
2-3.75 O.D. U.T. DUCTS IN EACH CURB
TO CO. RD. 26
TO CO. RD. 8
STA. 36+60·31
STA. 26+05·31
CARLETON CO. RD. 27
45'-0"
N. ABUT. BRG.
PIER #11
PIER #10
PIER #9
PIER #8
PIER #7
PIER #6
PIER #5
PIER #4
PIER #3
PIER #2
PIER #1
S. ABUT. BRG.
TO OTTAWA
TO VARS
RAMP
WEST BOUND LANE LINE "A"
EAST BOUND LANE
PLAN
SCALE: 1"=40'-0"
GENERAL NOTES
CLASS OF CONCRETE:
DECK, CURBS, PARAPET WALLS & COLUMNS - 5000 P.S.I.
REMAINDER - 3000 P.S.I.
CLEAR COVER ON REINFORCING STEEL:
FOOTINGS, ABUTMENTS AND PIERS - 3"
CURBS - 2"
DECK TOP - 2", BOTTOM - 1½"
1085'-0"
15'-0" 80'-0" 90'-0" 90'-0" 90'-0" 90'-0" 95'-0" 75'-0" 95'-0" 90'-0" 90'-0" 90'-0" 80'-0" 15'-0"
GUIDERAIL ANCHORAGES
STD. PARAPET WALL AND RAILING
12 BP 74 STEEL H-PILES (TYP.)
DESIGN LOAD 90 TONS DRIVEN TO REFUSAL
EL. 257.00
EL. 256.00
EL. 258.00
EL. 260.00
EL. 262.00
16'-5" MIN. CLEAR.
5'-0" ±
2'-0" HAND-LAID RIP-RAP (TYP. UNDER ALL DRAINS)
CRUSHED STONE SLOPE PAVING
NATURAL GROUND
6" ⌀ PERFORATED SUB DRAIN
ELEVATION
SCALE: 1"=40'-0"
B.M. 261·83
GEODETIC DATUM
N. & W. IN ROOT OF 1' MAP. 114'-0 RT. OF 294+00 PROP. HWY. 417 E.B.L.
PRINT RECORD
No. FOR DATE
P.I. STA. 30+80
EL. 297·24 FIN. GR.
FINISHED GRADE
PROF. GRADE
600 V.C.
L.V.C. = 414'
600' V.C.
L.V.C. = 1000'
P.I. STA. 38+35
EL. 259·49 T/P
TO CO. RD. 26
TO CO. RD. 8
PROFILE OF CO. SUB. RD. 27
N.T.S.
STA. 282+00
EL. 265·80
STA. 382+00
EL. 265·80
FINISHED GRADE
PROFILE GRADE
W. B. L. LINE "A"
STA. 382+30
EL. 265·80
PROFILE GRADE
E. B. L.
PROFILE OF HWY. #417
N.T.S.
36'-0"
STRUCTURE
3'-0" 15'-0" 15'-0" 3'-0"
CONCR. W.S.
2%
18'-0"
7'-0" 7'-0"
3'-0"
LIGHTING DUCT
PROV. FOR FUTURE LIGHTING
2-3.75" O.D. B.T. DUCTS EACH SIDE
DECK SECTION
SCALE: ⅛"=1'-0"
LIST OF DRAWINGS
1. GENERAL PLAN
2. BORE HOLE LOCATIONS AND SOIL STRATA
3. BORE HOLE LOCATIONS AND SOIL STRATA
4. FOOTING DETAILS
5. ABUTMENTS AND WINGWALLS
6. PIER DETAILS
7. EXPANSION JOINT DETAILS
8. DECK DIMENSIONS AND SCREED ELEVATIONS
9. DECK REINFORCING
10. CABLE DETAILS
11. PARAPET WALL DETAILS
12. STANDARD ~~STEEL PARAPET~~ RAIL
13. BRIDGE ELECTRICAL DETAILS TYPE D
14. STANDARD DETAILS - 1
15. STANDARD DETAILS - 2
REGISTERED PROFESSIONAL ENGINEER
D. R. GLUPPE
REVISIONS
DATE BY DESCRIPTION
Don Frye. Road Design. CONT. 69-28
DEPARTMENT OF HIGHWAYS ONTARIO
BRIDGE DIVISION
67-F-112
CARLETON CO. ROAD 27 INTERCHANGE
(3 MILES EAST OF OTTAWA EAST LIMITS)
KING'S HIGHWAY No. 417
DIST. No. 9
CO. CARLETON
TWP. GLOUCESTER
LOT 15 & 16
CON. VI
GENERAL PLAN
SITE No. 3-267
W.P. No. 34-66-06
APPROVED
BRIDGE ENGINEER
CONTRACT Nos.
DESIGN D.R.G.
CHECK R.K.
DRAWING G.P./R.M.
CHECK D.R.G.
DATE APRIL 68
LOADING HS 20-44
DRAWING No. D-6484-1



NO.	ELEVATION	STATION	OFFSET
6	262.6	32+55	12' LT.
7	262.1	33+18	12' RT.
8	261.7	34+03	13' LT.
9	260.7	34+56	12' RT.
10	260.5	36+00	10 LT.

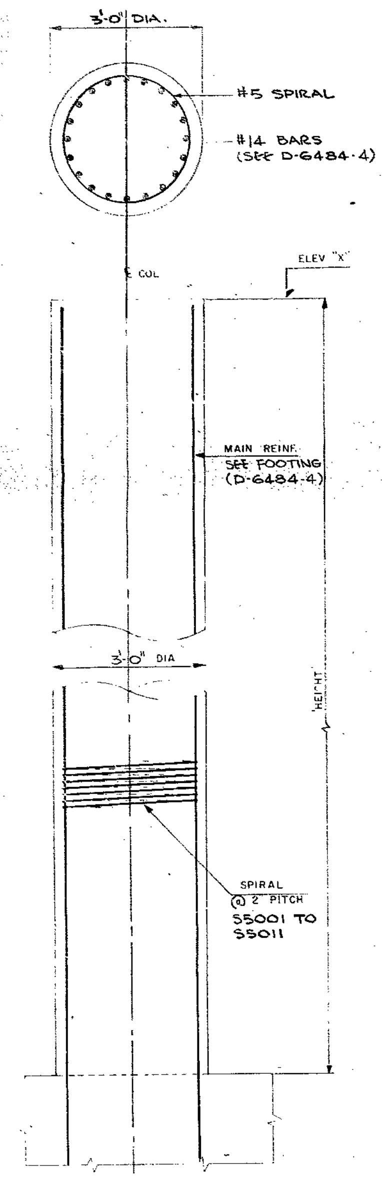


KEY PLAN
SCALE: 1" = 20'-0"

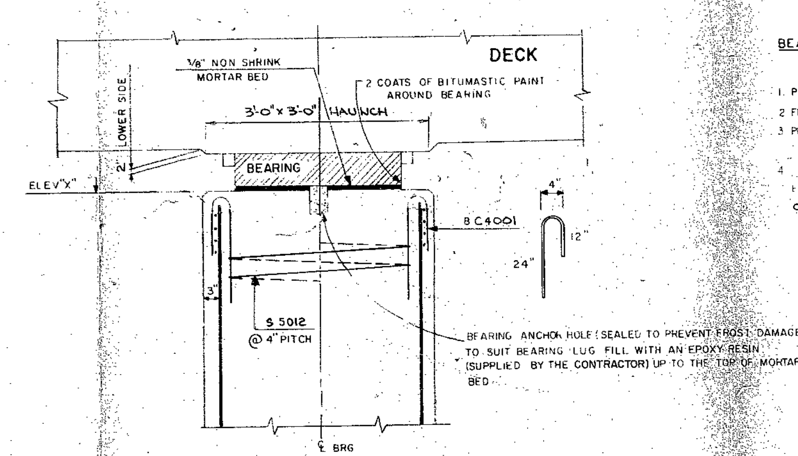
BEARING DESIGN DATA

DEAD LOAD	600 KIPS
DEAD LOAD AND LIVE LOAD	780 KIPS
LATERAL FORCE	75 KIPS
MAXIMUM MOVEMENT	± 4"

COLUMNS	ELEV "X"	HEIGHT	MAIN REINF.	SPIRALS	BEARING DETAILS	α
1	273.25	13'-3"	A1425	S5001	RF 350/80 OR APPR. EQUAL	
2	276.41	16'-4 7/8"	A1426	S5002	DO.	
3	278.77	18'-9 1/4"	A1427	S5003	DO.	
4	280.3[illegible]	22'-3 3/4"	A1428	S5004	DO.	
5	281.05	23'-0 5/8"	A1429	S5005	DO.	
6	280.99	22'-11 7/8"	A1430	S5006	DO.	
7	280.10	22'-1 1/4"	A1431	S5007	DO.	
8	278.42	20'-5"	A1432	S5008	DO.	
9	275.92	19'-11"	A1433	S5009	DO.	
10	272.62	16'-7 1/2"	A1434	S5010	DO.	
11	268.51	12'-6 1/8"	A1435	S5011	DO.	
1 TO 11				S5012		



PIER COLUMN



BEARING SETTING DETAIL

BEARING SETTING PROCEDURE

1. PLACE A 3/8" MORTAR BED.
2. FILL ANCHOR HOLE WITH EPOXY.
3. PLACE BEARING DEAD LEVEL.
4. ______ BEARING TO BE PLACED SUCH THAT EXPANSION TAKES PLACE IN THE DIRECTION OF TRAFFIC.

NOTE: BEARINGS TO BE SUPPLIED BY THE DEPARTMENT.
A TOTAL OF 22 BEARINGS ARE REQUIRED FOR PIERS.

DRAWING NOT TO BE SCALED

PRINT RECORD

No	FOR	DATE

REVISIONS

DATE	BY	DESCRIPTION

DEPARTMENT OF HIGHWAYS ONTARIO
BRIDGE DIVISION

CARLETON CO. RD. 27 INTERCHANGE
(3 MILES EAST OF OTTAWA EAST LIMITS)

KING'S HIGHWAY No. 417 DIST. No. 9
CO. CARLETON
TWP. GLOUCESTER LOT 15 & 16 CON. VI

PIER DETAILS

APPROVED	BRIDGE ENGINEER			SITE No. 3-267	W.P. No. 34-66-06
DESIGN	D.R.G.	CHECK	R.K.	CONTRACT Nos.	
DRAWING	R.M.	CHECK	D.R.G.		
DATE	APRIL 1969	LOADING	HS20-44	DRAWING No.	D-6484-6