

MEMORANDUM

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

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

ATTENTION: Mr. S. McCombie

DATE: July 17, 1969

OUR FILE REF:

IN REPLY TO

SUBJECT:

FOUNDATION INVESTIGATION REPORT

For

Proposed Underpass Structure at the
Crossing of the Carillon Park Road
and Hwy. #417 (E.B.L. and W.B.L.)
E. Hawkesbury Twp., Prescott County,
District No. 9 (Ottawa)

W.J. 69-P-37 -- W.P. 37-66-13 (Str.)

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

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

AGS/RdeP

Attach.

cc: Messrs. B. R. Davis (2)
H. A. Tregaskes
D. W. Parren
S. J. Markiewicz
C. R. Robertson
T. C. Kingsland
J. E. Gruspier
B. A. Singh

A. G. Sternac

A. G. Sternac

PRINCIPAL FOUNDATION ENGINEER

Foundations Files
Gen. Files

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FOUNDATION INVESTIGATION REPORT
For
Proposed Underpass Structure at the
Crossing of the Carillon Park Road
and Hwy. #417 (E.B.L. and W.B.L.)
P. Hawkesbury Twp., Prescott County
District No. 9 (Ottawa)
W.J. 69-P-37 -- W.P. 37-66-13 (Str.)

1. INTRODUCTION:

The Foundation Section was requested to carry out an investigation at the proposed crossing of the Carillon Park Rd. and Hwy. #417, in the Township of E. Hawkesbury, County of Prescott. The request was contained in a memo from the Kingston Bridge Location Section (Mr. C. Scott, Regional Bridge Location Engineer), dated May 15, 1969. An investigation was subsequently carried out by this Section to determine the subsoil and ground-water conditions at this site.

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

2. DESCRIPTION OF THE SITE AND GEOLOGY:

The site is situated on existing Hwy. #17, approximately 13 miles east of the Town of Hawkesbury, Ontario. Highway #17 is a 2-lane paved roadway about 22 feet wide, with the grade varying between elevations 163 to 167. The surrounding terrain is flat lying to gently undulating in relief. South of the existing intersection the land is brush-covered and being used for farming purposes. North of the intersection, however, the area is densely wooded. The major drainage in this region is provided by the Ottawa River, which is located approximately 1 mile north of the highway.

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

Physiographically, the site is situated on an arm of the area known as "The Russell and Prescott Sand Plains". In this area a sand mantle, some 5 to 10 feet in thickness, overlies a deposit of sensitive 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 in the Champlain Sea. In the area the clay is between 10 and 25 feet in thickness. The clay stratum is underlain by a glacial till which, in turn, is underlain by limestone, dolomite and shale bedrock of the Chazy Group, Ordovician Period.

3. FIELD AND LABORATORY WORK:

Eight sampled boreholes, 6 of which were accompanied by a dynamic cone penetration test, were put down during the course of the recent field investigation. In addition, a borehole (B.H. #3), put down as part of the preliminary investigation for this portion of the route (our Report W.J. 68-P-91), has been incorporated into this report. The borings were advanced by means of a conventional diamond drill rig adapted for soil sampling purposes.

Samples of the surficial sand and 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 portion of the overburden was sampled with 2" I.D. Shelby tubes, which were manually pushed into the soil. In addition, field vane tests were carried out to determine the undrained shear strength of the clay stratum. Bedrock was proven in 7 of the borings by obtaining either AXT or BXT size rock core samples.

3. FIELD AND LABORATORY WORK:

The groundwater level conditions across the site were determined by installing sealed piezometers in three of the boreholes. This information was supplemented by recording the 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 Drawing No. 69-F-37A, 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).

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. SUBSOIL CONDITIONS:

4.1) General:

The natural surficial deposit across the site is composed of a loose brown sand with a trace of silt between 2 and 5 feet in

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

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

thickness; along the existing gravel access road this deposit is overlain by granular roadway fill. The sand is underlain by the predominant overburden stratum across the site, composed of a firm to stiff sensitive clay, varying from 9 to 25 feet in thickness.

Directly underlying the clay stratum is a deposit of glacial till composed primarily of stiff to hard clayey silt with some sand and gravel. This deposit, in turn, is underlain by fractured, becoming sound with depth, shaley to sandy dolomite; the bedrock surface was encountered at depths of between 46 and 54 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 Drawing 69-P-37A, is based on this information.

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

4.2) Surficial Deposits - Sand and Roadway Fill:

Roadway fill was encountered in those boreholes, namely: No. 1, 2 and 3, put down in close proximity to the existing roadways. This fill, which was between 2 and 4 feet in thickness, is composed of a compact to dense brown sand and gravel.

Underlying the roadway fill, at the aforementioned borehole locations and a topsoil cover at the other borings, is a loose to compact ('N' values between 5 and 28 blows/ft.) grey sand with a trace of silt. The thickness of the surficial sand deposit varies from 2 to 5 feet. Grain-size distribution curves for samples of the sand deposit are shown on Figure #2 in the Appendix of this report.

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

4.3) 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 9 feet (B.H. #8) to 25 feet (B.H. #7), being typically about 18 feet. It would appear that the thickness decreases in a northerly direction - i.e., towards the Ottawa River. The cohesive deposit is relatively uniform in composition; there are, however, localized zones where silt seams up to 1/4 inch thick are present. Grain-size distribution curves for samples of the clay are shown on Figure #3.

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

		<u>Range</u>	<u>Average</u>
Bulk Density (p.c.f.)	(γ)	95 - 109	98
Liquid Limit (%)	(W_L)	65 - 92	73
Plastic Limit (%)	(W_P)	24 - 33	27
Natural Moisture Content (%)	(W)	60 - 91	76
Liquidity Index	(IL)	0.6 - 1.4	1.0
		<u>B.H. #1, Sa. 5</u>	<u>B.H. #3, Sa. 4</u>
Initial Void Ratio	(e_o)	2.27	1.8
Compression Index	(C_c)	1.38	1.09
Degree of Preconsolidation	($P_c - P_o'$)	1,300	3,500
		2 Tests	
Undrained Shear Strength (p.s.f.)	(C_u)	<u>Range</u> (C_u)	<u>Range</u> Sensitivity (S)
1) Field Vanes		650 - 1,400	4 - 22
2) Lab. Vanes		750 - 1,650	4 - 10
3) Lab. Tests		600 - 1,050	-

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

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

The foregoing summarized Atterberg limit tests, are also plotted on the Plasticity Chart, Figure #4. These results indicate that the clay is of high plasticity and, in general, inorganic. The natural water content is consistently at or above the liquid limit as exemplified by the high liquidity indices; this indicates that the clay is indeed sensitive.

The consistency of the stratum, as determined from the undrained shear strength testing, increases from firm, immediately below the surficial deposits, to stiff with depth. At B.H.'s 4 and 4A, however, the minimum recorded undrained shear strength was 400 p.s.f., which would be indicative of a consistency in the soft range. It is inferred from this boring programme that the soft zone is restricted to this localized area. The increase in undrained shear strength mentioned above, is represented by an average C_u/P_o ratio of 0.4 for the overall deposit. 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 two laboratory consolidation tests, the results of which are shown as Void Ratio vs. Pressure plots, on Figures #5 and #6. One of the samples tested had a liquidity index of 1.2 (refer to Figure #5), while the other (Figure #6), taken from the lower more highly preconsolidated portion of the stratum, had a liquidity index of 0.5. The former was preconsolidated by approximately 1,300 p.s.f. in excess of the existing overburden pressure, while the latter was preconsolidated by about 3,500 p.s.f. It is inferred that the major portion of the stratum is preconsolidated by something of the order of 1,500 p.s.f., with

4. SUBSOIL CONDITIONS: (cont'd.)

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

only the lower 4 to 5 feet of the stratum having a degree of preconsolidation in the higher range. The relatively high values given for the initial void ratio (e_0) 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) Glacial Till - Heterogeneous Mixture of Clay, Silt, Sand and Gravel:

The clay is underlain by a grey glacial till deposit composed of a heterogeneous mixture of all grain sizes. The overall thickness of the glacial till varies from 16 feet at B.H. #7 to 32.5 feet at B.H. #4 - i.e., it tends to increase in a northerly direction. At some of the boring locations the upper 5 to 10 feet of the deposit is in a 'reworked' and softened condition. The matrix of the deposit is predominantly composed of a clayey silt binding sand and gravel sizes. There are, however, numerous localized zones throughout where the matrix is basically granular in nature (sand and gravel); on a weighted basis, it is inferred such zones could occupy as much as 20% of the deposit by volume. Occasional silt seams up to 4" in thickness are located randomly throughout the deposit. In addition, 1-1/2 to 2 feet thick sand and gravel layers were encountered within the glacial till at B.H. #4. At those locations where the glacial till is extensive, such as at B.H.'s #3, 4 and 6, boulders up to 8 inches in size were encountered within a 4 to 7 foot zone located immediately above the bedrock surface. Grain-size distribution curves, carried out on samples obtained with a 2" O.D. split-barrel sampler, are plotted on Figures #7 and #8.

Atterberg limit tests, carried out on the more cohesive portions of the glacial till, are summarized on the Plasticity Chart shown on Figure #9. The results of the testing indicate that such areas are representative of an inorganic clayey silt of low

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

4.4) Glacial Till - Heterogeneous Mixture of Clay, Silt, Sand and Gravel: (cont'd.) ...

plasticity. The natural water content of the stratum is consistently 1 to 3 percent less than the plastic limit. The more granular portions of the glacial till are, of course, non-plastic.

Standard penetration resistance tests, the results of which are summarized on Figure #1, were carried out within the deposit. This testing gave 'N' values which range from 5 to 8 blows/ft. in the upper 'reworked' zone of the glacial till; below this zone the values range from 10 to 107 blows/ft. Based on these results, it is estimated that the consistency of the 'reworked' zone is basically stiff, while the remainder of the deposit ranges from very stiff to hard with depth. The granular areas throughout the glacial till are inferred to have a relative density in the compact to dense range.

4.5) Dolomite Bedrock:

Bedrock was proven in 7 of the borholes by obtaining between 3 and 11 feet of either AXT or BXT rock core. The surface of the bedrock was found to vary from elevations 110 to 121 with the lower elevation being encountered in the northern portion of the site.

The bedrock is composed of a gray sandy to shaley dolomite. In general, bedrock is sound throughout; however, some signs of fracturing and jointing were observed in the upper 2 to 5 feet at some of the boring locations.

5. GROUNDWATER CONDITIONS:

Groundwater level observations have been carried out, during the period of the investigation, in 1) sealed piezometers installed in B.H.'s #1, 7 and 8, and 11) the open holes at the remaining boring locations. The observations are recorded on the borelog sheets and summarized on Drawing 69-P-37A. The results

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

of the measurements indicate that the piezometric groundwater level, within the surficial deposits and underlying clay stratum, is between elevation 156 and 160 - i.e., some 3 to 8 feet below ground surface. The corresponding piezometric groundwater level within the lower portion of the glacial till was, however, found to vary from elevation 147 to 153 - i.e., some 11 to 19 feet below ground surface.

It is pertinent to note that the heterogeneous glacial till is more permeable than the overlying cohesive 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 an underpass structure to carry the Carillon Park Road over the East and Westbound lanes of proposed Highway #417. Present proposals call for a five-span structure (36'-80'-59'-80'-30'). The proposed profile grade of the Carillon Park Road, in the vicinity of the crossing, is elevation 188. At this grade the south and north approach embankments will have a maximum height of about 22 and 20 feet above ground surface, respectively. The embankments will have a crest width of 46 feet.

The East and Westbound lanes of Highway #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 and a median. Existing Hwy. #17 will be incorporated into the E.B.L. The finished grade will be at or a few feet above the surrounding ground level - i.e., it will be between elevation 165 and 168 in the vicinity of the crossing.

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

6.1) General:

Underlying between 2 and 7 feet of sand, and in some places roadway fill, is the predominant stratum across the site, composed of a firm to stiff sensitive marine clay; this stratum varies from 9 to 25 feet in thickness. The clay is underlain by up to 32 feet of stiff to hard cohesive glacial till which, in turn, is followed by shaley to sandy dolomite bedrock.

The presence of soft to firm highly compressible clay at a relatively shallow depth below ground surface is the controlling factor as far as foundation considerations are concerned. The implications are that: 1) considerable consolidation settlement is expected beneath the approach fills, and 11) the structure elements must be supported on piled foundations. These factors will be elaborated upon in the sub-sections to follow.

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 ($\sigma = 0$) provides a suitable means of assessing the stability of the embankment sections. In this method of analysis, stability is governed by the applied loads and by the stress-strain and undrained shear strength properties of the foundation and embankment soils.

Analyses have been carried out, therefore, in terms of total stresses, both manually and by the use of the electronic computer, to determine the stability of the fill sections.

The following assumptions were made:

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

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

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

Soil Properties (South Approach Embankment)

<u>Elevation</u>	<u>Soil</u>	<u>Density</u> <u>(p.c.f.)</u>	<u>Strength Parameters</u>	
			<u>Cu(p.s.f.)</u>	<u>Ø (°)</u>
187 - 165	Embankment Fill 2:1 Slopes)	125	-	30°
165 - 160	Surficial Deposit - Sand	110	-	30°
160 - 155	Clay	γ = 100 p.c.f. γ = 40 p.c.f.	700	-
155 - 150	"	"	800	-
150 - 135	"	"	900	-

The stability computations indicate that, an embankment 22 feet high with 2:1 side slopes, would have a factor of safety of approximately 1.3 with respect to the overall stability of the section. It is considered that this is adequate and that an embankment of this height could be constructed at this site.

6.2.2) Settlement Considerations:

The underlying highly compressible clay stratum will undergo considerable settlement due to consolidation, over a long-term period, under the weight of the approach embankments. In addition, some settlement will take place in the underlying stiff to hard glacial till. Because of the relative competence of this deposit, however, this settlement will be of a recompression nature - i.e., take place during or immediately following the construction

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

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

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

period. Further, the magnitude of this settlement will be small when compared to the consolidation settlement occurring within the overlying clay stratum. The results of the settlement computations carried out are summarized in the paragraphs to follow:

The maximum consolidation settlement will occur under the centre-line of the south approach embankment where 1) the fill height is highest (approx. 22 feet above ground surface), and 11) the thickness of the compressible stratum is most extensive (of the order of 25 feet). The consolidation settlement beneath the north approach embankment will be less. As discussed previously, the settlement will take place over an extended period of time - i.e., the settlement is time dependent. An estimate of the magnitude of the settlement expected at various stages following construction of the approaches is presented in tabular form below:

Consolidation Settlement Beneath Embankments

<u>Time</u>	<u>South Approach</u> (Total)	<u>North Approach</u> (Total)
1 year	8 inches	6 inches
3 years	12 inches	9 inches
5 years (max.)	16 inches	11 inches

Referring to the above table, it can be seen that a high percentage of the total predicted consolidation settlement will take place in a relatively short period of time - e.g., approximately 50 percent within 1 year. In order to minimize post-construction maintenance costs, consideration should be given to constructing

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

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

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

the fills first and, by so doing, induce a significant portion of the settlement prior to installing the structure - i.e., employing stage construction. If scheduling will allow for this contingency, a 12 to 18 month period is recommended. In any event, final paving of the roadway fills should be delayed as long as possible.

6.3) Structure Foundations:

Because of the soft to firm and compressible nature of the subsoil, the structure piers and abutments should be supported on end-bearing piles driven to bedrock. For estimating purposes, the pile tips will be at approximately elevation 120 at the location of the two most southerly piers and south abutment, while it will be at elevation 110 at the location of the most northerly piers and north abutment - i.e., the pile lengths will vary from about 42 to 50 feet. Allowable loads will depend on the pile section chosen (e.g., 12 BP 74 steel H-piles may be designed for 90 tons per pile).

Since settlement of the proposed roadway embankments will be relatively large, considerable negative skin frictional loads may be imposed on the piles supporting the abutments. Taking this into consideration, it may be advisable to reduce the design load of the abutment piles from 90 tons to 70 tons/pile.

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 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

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

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

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

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

No major dewatering problems are anticipated. Excavations for the pier caps may, however, be carried out below the groundwater level, which is about 3 to 8 feet below ground surface. Because these excavations will be carried out mainly through relatively pervious sand deposits, seepage may occur. This could be dealt with by pumping from sumps or, alternatively, by excavating from within closed sheeting.

7. SUMMARY:

A foundation investigation at the site of the proposed underpass structure to carry the Carillon Park Road over the East and Westbound lanes of proposed Hwy. #417, in the Twp. of E. Hawkesbury, County of Prescott, is reported.

Underlying between 2 and 7 feet of sand, is the predominant overburden stratum across the site, composed of a firm to stiff sensitive marine clay varying from 9 to 15 feet in thickness. The clay is underlain by up to 32 feet of stiff to hard cohesive glacial till, which, in turn, is followed by shaley to sandy dolomite bedrock. The groundwater level in the surficial sand deposit and underlying clay stratum was, at the time of the investigation, some 3 to 8 feet below ground surface.

The piers and abutments can be supported on end-bearing piles driven to bedrock. Other aspects regarding the pile support of the structure elements, such as negative skin frictional forces acting at the abutment locations, are discussed in detail in the report.

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

Approach embankments of the height contemplated (22 feet max.) should be stable with respect to a deep-seated foundation failure. Long-term consolidation settlement will be induced in the foundation subsoil due to the embankment surcharge loading. Computations carried out indicate that the settlement could amount to as much as 16 inches beneath the south approach. In order to reduce the magnitude of the post-construction settlements, consideration should be given to constructing the approach fills some 12 to 18 months prior to installation of the structure proper.

8. MISCELLANEOUS:

The field work for this project was carried out during the period June 9 to 17, 1969, under the supervision of Messrs. D. Phelps and E. K. Kwan.

This project was under the immediate supervision of Mr. B. T. Darch, Senior Foundation Engineer, who also wrote this report.

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

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

July 1969.

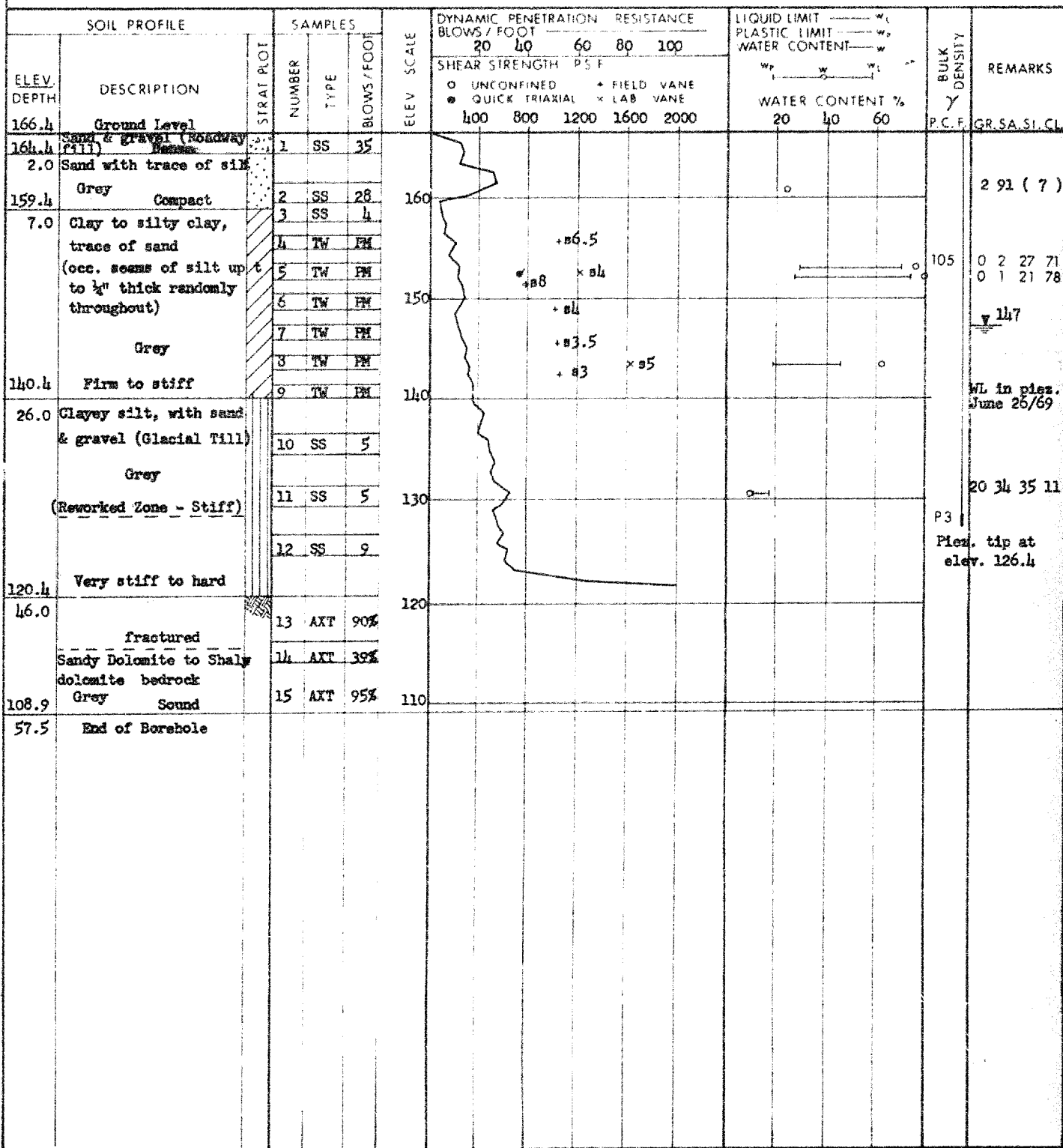
APPENDIX I

DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 1

FOUNDATION SECTION

JOB 69-F-37 LOCATION Sta. 29 + 88 o/s 28⁺ Rt. ORIGINATED BY KKK
 W.P. 37-66-13 BORING DATE June 12, 13 & 16, 1969 COMPILED BY BTB
 DATUM Geodetic BOREHOLE TYPE Washboring-NX, BX Casing, AXT Rockcore, Cone CHECKED BY



DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 2

FOUNDATION SECTION

JOB 69-F-37 LOCATION Sta. 29 + 62 o/s 22' Lt. ORIGINATED BY DP
 W.P. 37-66-13 BORING DATE June 12, 13 & 16, 1969 COMPILED BY BYD
 DATUM Geodetic BOREHOLE TYPE Washboring-NX, BX Casing - AXT Rock Core, Cone CHECKED BY

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION BLOWS / FOOT 20 40 60 80 100	RESISTANCE SHEAR STRENGTH P.S.F. ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB. VANE 400 800 1200 1600 2000	LIQUID LIMIT — w_L PLASTIC LIMIT — w_p WATER CONTENT — w w_p — w — w_L WATER CONTENT % 20 40 60	BULK DENSITY γ P.C.F.	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLT.	NUMBER	TYPE	BLOWS / FOOT						
167.5	Ground Level										
165.5	Sand with some gravel (Roadway Fill) Compact		1	SS	22						9 81 (13)
2.0	Sand, trace of silt, trace of gravel. Brown		2	SS	41						
160.0	Compact to dense		3	SS	15						
7.5	Clay to silty clay trace of sand		4	TW	PM						
			5	TW	PM						
			6	TW	PM						
	Gray		7	TW	PM						
			8	TW	PM						
137.5	Firm to stiff										
30.0	Clayey silt, with some sand and gravel (Glacial Till)		9	SS	13						
	Gray		10	SS	14						
			11	SS	18						
118.8	Very stiff		12	SS	12						
48.7	fractured Sandy Dolomite to Shaly Dolomite Bedrock		13	AXT	70%						
108.8	Sound Gray		14	AXT	79%						
58.7	End of Borehole										

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 3

FOUNDATION SECTION

JOB 69-F-37 LOCATION Sta. 30 + 33 o/s 6' Rt. ORIGINATED BY BTD
W.P. 37-66-13 BORING DATE Jan. 17, 20 & 21, 1969 COMPILED BY VK
DATUM Geodetic BOREHOLE TYPE Washboring - NX BX & AX Casing - AXT Rock Core CHECKED BY

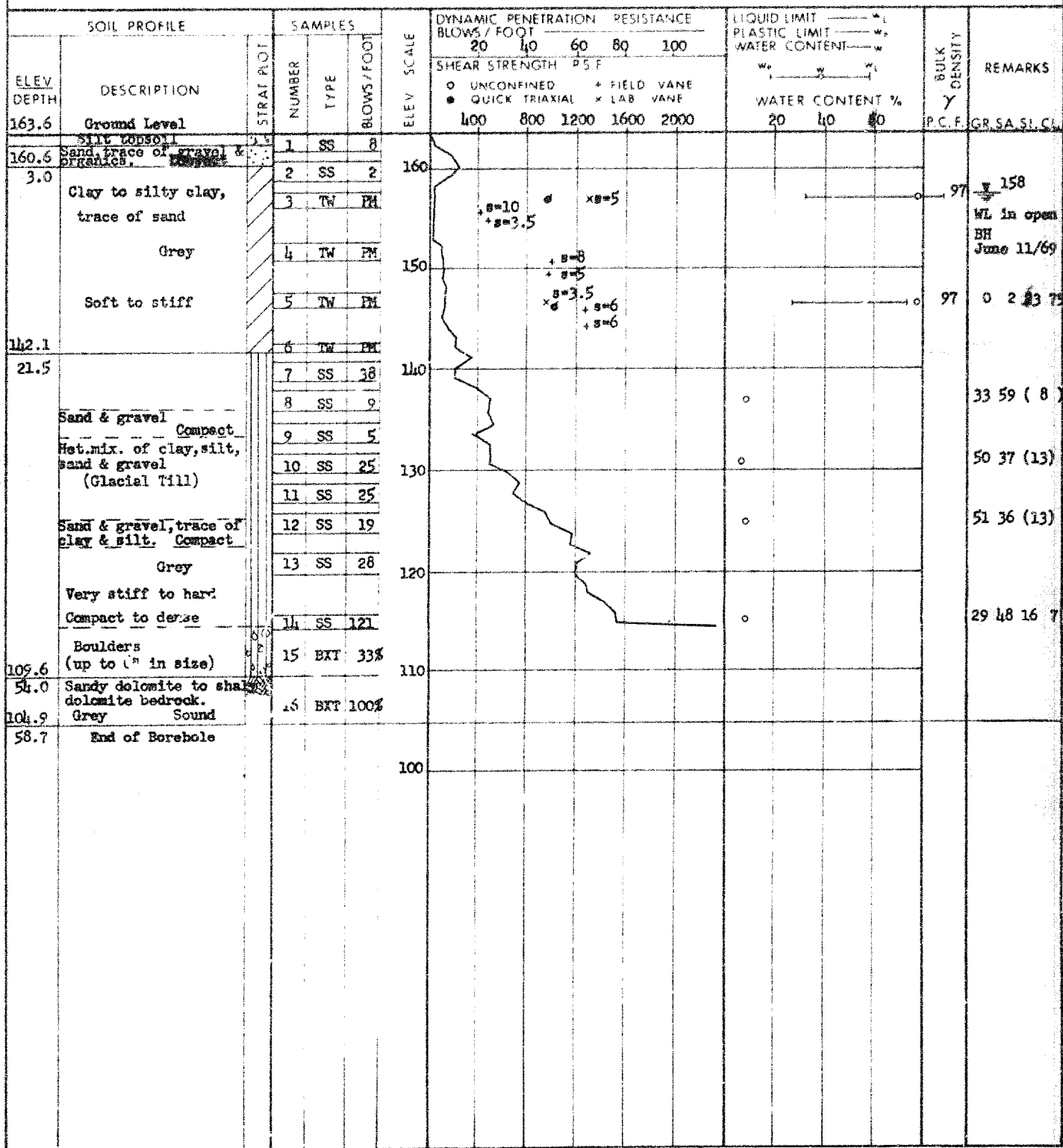
SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT		LIQUID LIMIT — w_L PLASTIC LIMIT — w_p WATER CONTENT — w		BULK DENSITY γ P.C.F.	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FOOT		SHEAR STRENGTH PSF		w_p — w — w_L WATER CONTENT % 20 40 60			
							○ UNCONFINED ● QUICK TRIAXIAL	+ FIELD VANE x LAB. VANE				
167.5	Ground Level					400	800	1200	1600	2000		
0.0	Silty sand & gravel (Roadway Fill) Brown											2 72 21 5 160.5
163.5	Compact											
162.5	Sandy silt topsoil											
159.5	Silty sand with trace of clay. Grey. Loose		1	SS	8							WL in open BH
8.0	Clay to silty clay, trace of sand (occ. seams of silt up to 1/2" thick randomly throughout)		2	SS	4							
	Grey		3	TW	PM						92	0 0 29 7
113.5	Firm to stiff		4	TW	PM						109.5	0 5 28 6
24.0	Reworked zone		5	SS	8							
	Clayey silt with sand & gravel (Glacial Till) (occ. silty layers up to 1/2" thick through- out) boulders 6" in size below elev. 127)		6	SS	28							21 34 37 8
	Grey		7	SS	20							21 33 34 12
			8	SS	52							
121.0	Very stiff to hard		9	SS	62							
46.5	Sandy dolomite to shaly dolomite Bedrock		10	BXL	90%							
	Grey											
112.0	Fractured to sound at elev. 118		11	BXL	90%							
55.5	End of Borehole											

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RECORD OF BOREHOLE No. 4

FOUNDATION SECTION

JOB 69-F-37 LOCATION Sta. 30 + 98 o/s 18' Lt. ORIGINATED BY KKK
 W.P. 37-66-13 BORING DATE June 10, 11 and 12, 1969 COMPILED BY BTB
 DATUM Geodetic BOREHOLE TYPE Washboring - NX Casing - BXT Rock Core; Cone CHECKED BY



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MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 11A

FOUNDATION SECTION

JOB 69-F-37

LOCATION

Sta. 31 + 12 o/s 15' Lt. (12' N. of BR#4)

ORIGINATED BY KKK

W.P. 37-66-13

BORING DATE

June 12, 1969

COMPILED BY RTD

DATUM Geodetic

BOREHOLE TYPE

Washboring - NX Casing

CHECKED BY 20

SOIL PROFILE			SAMPLES			ELEV SCALE	DYNAMIC PENETRATION RESISTANCE	RESISTANCE	LIQUID LIMIT	PLASTIC LIMIT	WATER CONTENT	BULK DENSITY	REMARKS			
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT		SHEAR STRENGTH P.S.F.							WATER CONTENT %		
163.6	Ground Level						400	800	1200	1600	2000	20	40	60	P.C.F.	GR. SA. SI. CL.
160.6	Silt topsoil Sand, trace of clay & org. matter.					160										
3.0	Clay to silty clay, trace of sand		1	SS	-											
	Grey		2	TW	PM											
			3	TW	PM	150										
			4	TW	PM											
141.9	Soft to stiff		5	TW	PM											
139.9	Clayey silt, some sand & gr. (G.I. Mill) Grey. V. Stiff		6	SS	13	140										
23.7	End of Borehole															

JOB 69-F-37 LOCATION Sta. 31 + 69 o/s 31' Rte. ORIGINATED BY KKK
W.P. 37-66-13 BORING DATE June 10 & 11, 1969 COMPILED BY BTD
DATUM Geodetic BOREHOLE TYPE Washboring-NX,BX,AX Casing, AXT Rock Core; Cons. CHECKED BY /O

SOIL PROFILE			SAMPLES			DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT					LIQUID LIMIT — w _L PLASTIC LIMIT — w _P WATER CONTENT — w			BULK DENSITY Y	REMARKS	
ELEV. DEPTH	DESCRIPTION	STRAT. PLT	NUMBER	TYPE	BLOWS / FOOT	20	40	60	80	100	SHEAR STRENGTH P.S.F.					WATER CONTENT %
											○ UNCONFINED	* FIELD VANE				
											● QUICK TRIAXIAL	x LAB VANE				
							400	800	1200	1600	2000			20	40	60
162.2	Ground Level															
0.0	Silt Topsoil		1	SS	7											
159.2	Silt with sand, trace of organics.					160										159
3.0	Clay to silty clay, trace of sand (occ. partings of silt up to 1/4" thick throughout)		2	TW	PM											Wt. in open BH June 11/69
	Grey		3	TW	PM	150										
	Firm to stiff		4	TW	PM											
143.2																
19.0	Clayey silt with sand & gravel (Glacial Till) (occ. seams of sand & gravel up to 1" thick below elev. 130)		5	SS	35	140										17 37 37 9
	Grey		6	SS	6	130										
	Very stiff to hard		7	SS	13											
			8	SS	107	120										22 41 27 10
			9	SS	26											
			10	SS	16											
113.3	fractured		11	AXT	88%	110										
107.3	sandy dolomite to shaly dolomite. Grey. Sound		12	AXT	73%											
54.9	End of Borehole															

RECORD OF BOREHOLE No. 6

FOUNDATION SECTION

JOB 69-F-37 LOCATION Sta. 32 + 21 o/s 13' Lt. E ORIGINATED BY DP
W.P. 37466-13 BORING DATE June 9, 10 & 11, 1969 COMPILED BY BTD
DATUM Geodetic BOREHOLE TYPE Washboring-NX, BX Casing-AXT Rock Core; Cone CHECKED BY

SOIL PROFILE			SAMPLES			ELEV SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT					LIQUID LIMIT w_L PLASTIC LIMIT w_p WATER CONTENT w			BULK DENSITY γ P.C.F.	REMARKS								
ELEV DEPTH	DESCRIPTION	STRAT. PLT	NUMBER	TYPE	BLOWS/FOOT		20	40	60	80	100	w_p w_L	WATER CONTENT % 20 40 60											
							SHEAR STRENGTH P.S.F.																	
							○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB. VANE																	
163.6	Ground Level						400	800	1200	1600	2000		20 40 60											
0.5	Sandy silt topsoil		1	SS	2																			
159.6	Silty sand Loose Brown		2	SS	6																			
4.0	Clay to silty clay, trace of sand		3	TW	PM																			
	Grey		4	TW	PM																			
	Firm to stiff		5	TW	PM																			
141.1			6	TW	PM																			
22.5	Clayey silt with sand and gravel (Glacial Till)		7	SS	9																			
	Grey		8	SS	22																			
	Very stiff		9	SS	13																			
			10	SS	22																			
			11	SS	23																			
			12	SS	10																			
109.6	Boulders up to 6" in size		13	AXT	20%																			
51.0	fractured Sandy dolomite to shaly dolomite bedrock.		14	AXT	48%																			
	Grey		15	AXT	62%																			
97.7	Sound		16	AXT	100%																			
65.9	End of Borehole																							

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RECORD OF BOREHOLE No 7

FOUNDATION SECTION

JOB 69-F-37

LOCATION

Sta. 27 + 76 14' Lt. 8

ORIGINATED BY

DP

W.P. 37-66-13

BORING DATE

June 12, 13, 16 & 17, 1969

COMPILED BY

BTD

DATUM Geodetic

BOREHOLE TYPE

Washboring-NX,BX Casing - AXT Rock Core

CHECKED BY

SOIL PROFILE		STRAT. PLOT	SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT		LIQUID LIMIT ——— w_L PLASTIC LIMIT ——— w_P WATER CONTENT ——— w		BULK DENSITY γ P.C.F.	REMARKS
ELEV. DEPTH	DESCRIPTION		NUMBER	TYPE	BLOWS / FOOT		SHEAR STRENGTH ψ (PSI)		WATER CONTENT % 20 40 60			
165.1	Ground Level											
160.6	Clayey silt topsoil		1	SS	5							
160.6	Silty sand		2	SS	3	160						
145.5	Loose Brown		3	TW	PM							
	Clay to silty clay, trace of sand		4	TW	PM							
	(occ. partings & seams of silt up to 1/4" thick throughout)		5	TW	PM							
			6	TW	PM	150						
			7	TW	PM							
	Grey		8	TW	PM	110						
			9	TW	PM							
135.1	Firm to stiff		10	SS	8							
30.0	Clayey silt with sand & gravel (Glacial Till) (occ. seams of sand & gravel up to 1/4" thick Below elev. 12h)		11	SS	7	130						
	Grey		12	SS	15							
119.2	Very stiff		13	AXT	91	120						
115.9	Shaly dolomite to sandy											
116.1	dolomite bedrock. Sound											
119.0	End of Borehole											

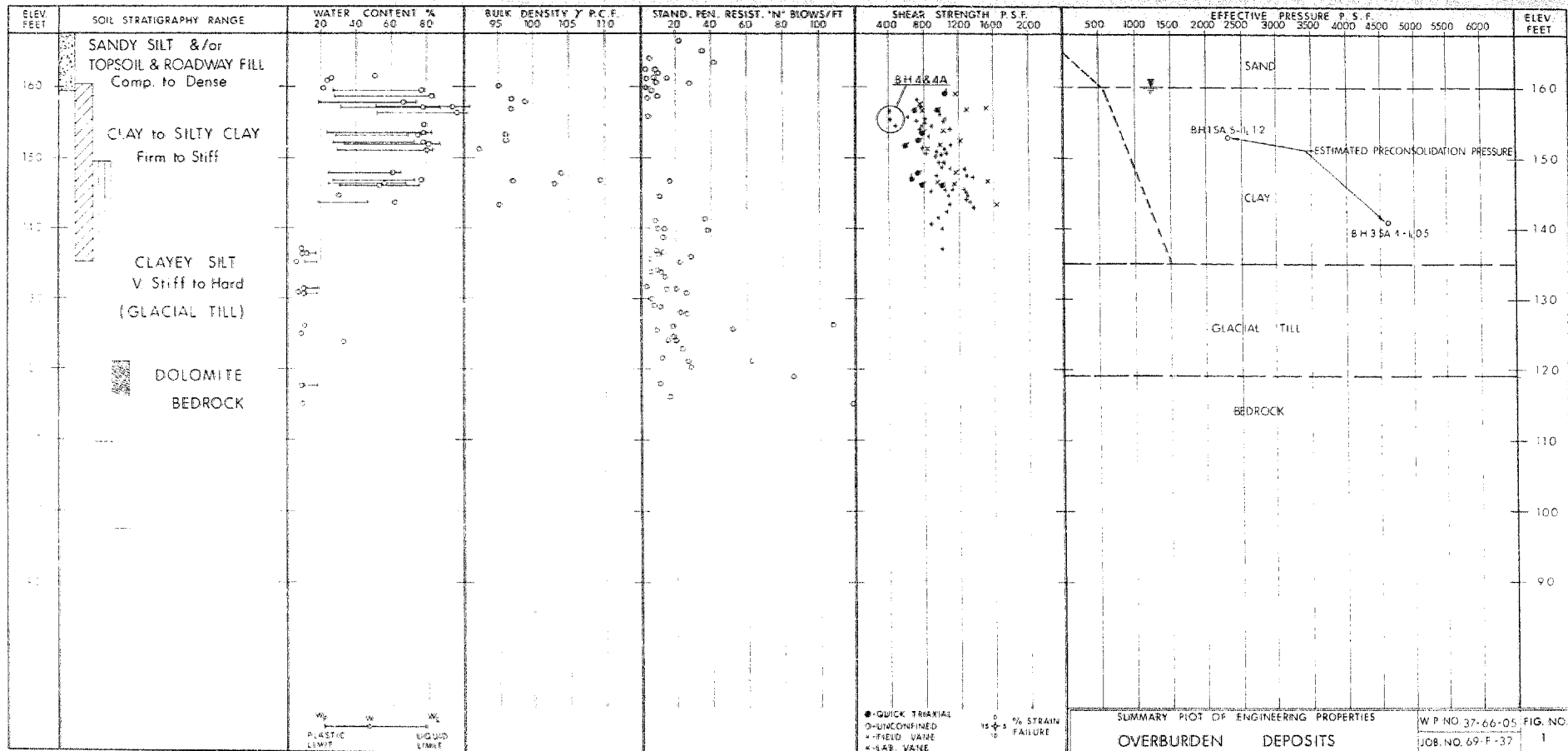
DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING OFFICE

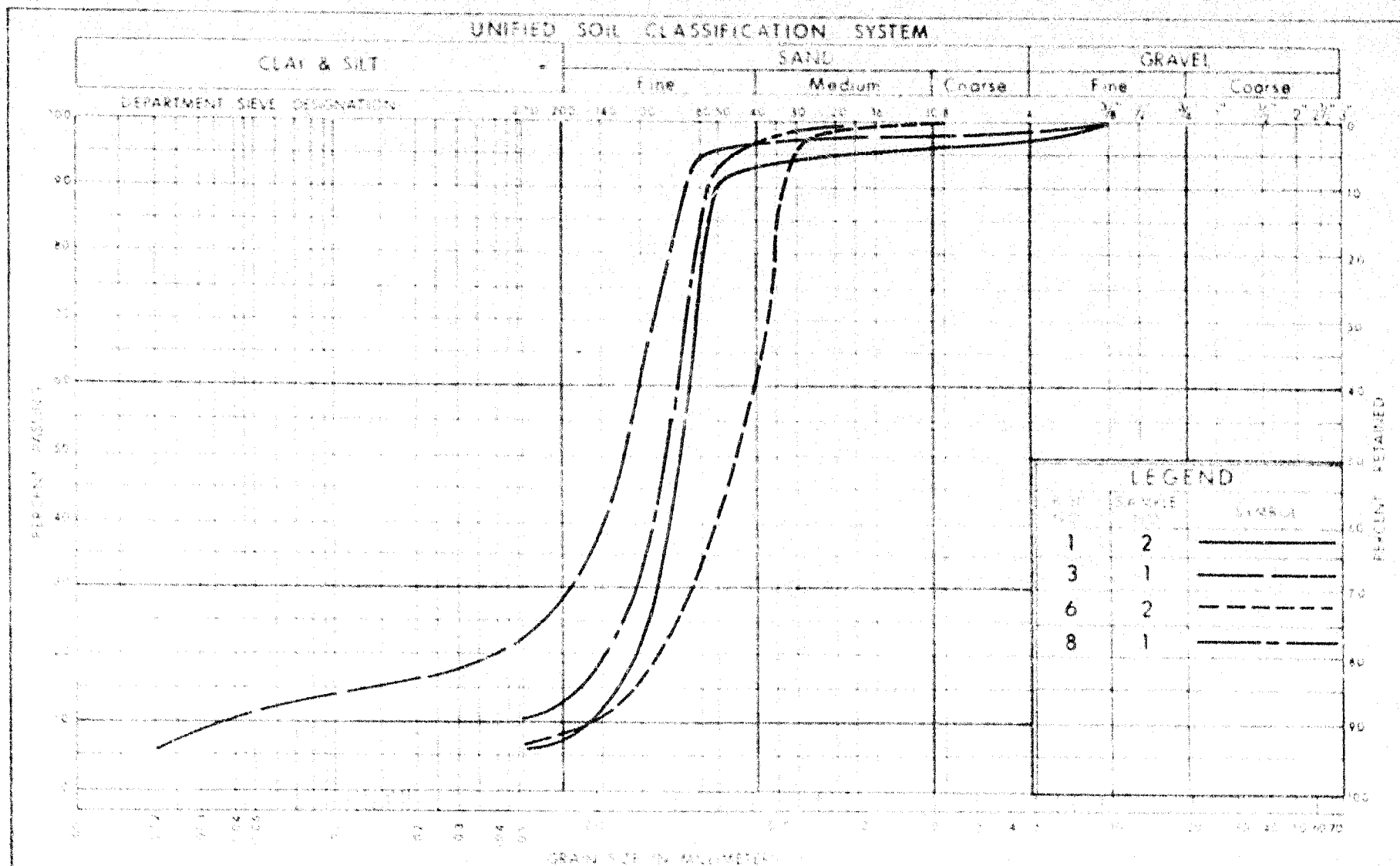
RECORD OF BOREHOLE No 8

FOUNDATION SECTION

JOB 69-F-37 LOCATION Sta. 33 + 11 1' Rt. 0
W.P. 37-66--13 BORING DATE June 10 & 11, 1969
DATUM Geodetic BOREHOLE TYPE Washboring-NX, BX Casing; Cone
ORIGINATED BY KKK
COMPILED BY BTD
CHECKED BY

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT		LIQUID LIMIT — % PLASTIC LIMIT — % WATER CONTENT — %		BULK DENSITY Y P.C.F.	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLT.	NUMBER	TYPE	BLOWS / FOOT		SHEAR STRENGTH P.S.F.		WATER CONTENT %			
							20 40 60 80 100	500 800 1200 1600 2000	20 40 60			
162.8	Ground Level											
0.5	Silty sand		1	SS	9							GR 5A SI CL
158.8	Loose Brown		2	SS	9							0 88 (12)
4.0	Clay to silty clay trace of sand		3	TW	PM							158.5
	Grey		4	TW	PM							153.
149.8	Firm		5	TW	PM							P5
13.0	Clayey silt, with some sand & gravel (Glacial Till)		6	SS	16							Tip elev. 150.8
	(occ. random granular zones)		7	SS	10							46 40 (14)
	Grey		8	SS	13							
			9	SS	10							
	Very stiff to hard		10	SS	11							P4
			11	SS	20							Tip elev. 127.8
117.8			12	SS	86							20 49 26 5
45.0	End of Borehole											WL's in piez June 26/69





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GRAIN SIZE DISTRIBUTION SAND

W.F. No. 37-66-05

JOB No. 69-F-37

FIG. 2

UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT

SAND

GRAVEL

Fine

Medium

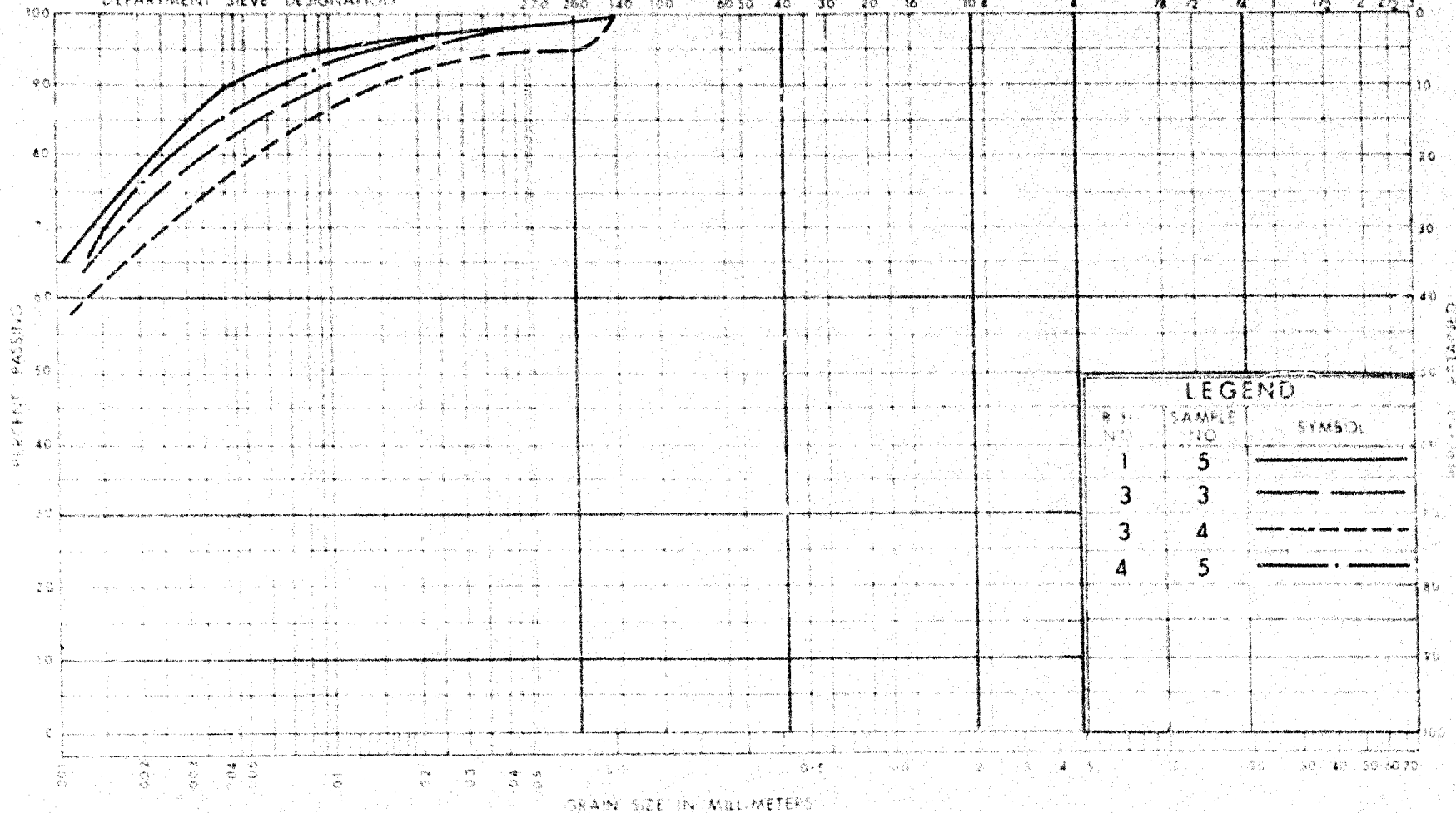
Coarse

Fine

Coarse

DEPARTMENT SIEVE DESIGNATION

270 200 140 100 60 50 40 30 20 16 10 8 4 3/8 1/2 3/4 1 1 1/2 2 2 1/2 3"



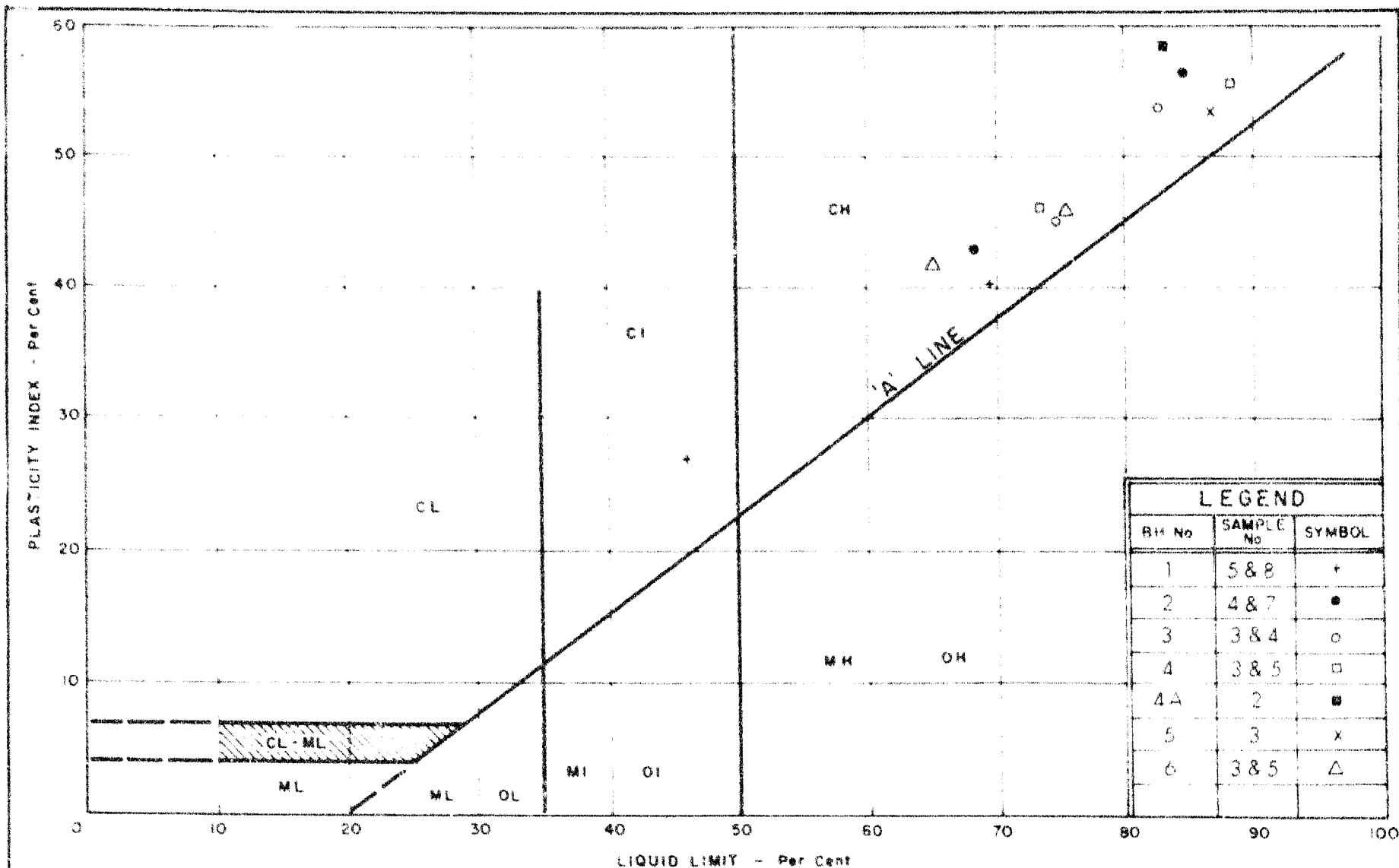
DEPARTMENT OF HIGHWAYS
MATERIALS and
TESTING
DIVISION

GRAIN SIZE DISTRIBUTION
CLAY TO SILTY CLAY
(SENSITIVE)

WP No 37-66-05

JOB No 69-F-37

FIG. 3



DEPARTMENT OF HIGHWAYS
MATERIALS and
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DIVISION

PLASTICITY CHART CLAY TO SILTY CLAY (SENSITIVE)

WP No. 37-66-05

JOB No. 69-F-37

FIG. 4

VOID RATIO vs PRESSURE

$W_L = 69.5$

$W_p = 29.2$

$W = 76.5 \%$

$C_c = 1.38$

BORE HOLE 1

SAMPLE 5

DEPTH 14'-3"

ELEV. 153.0

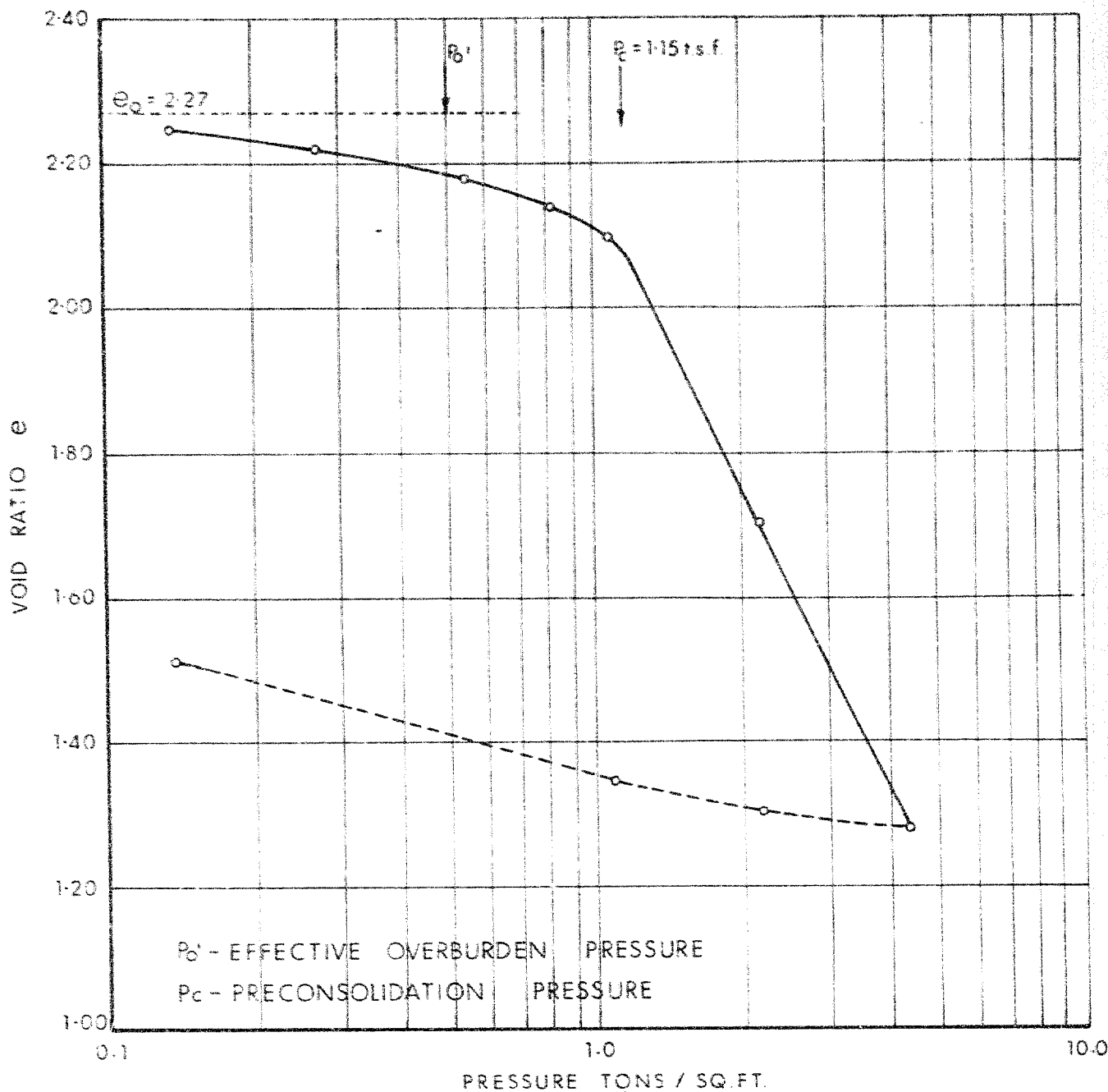
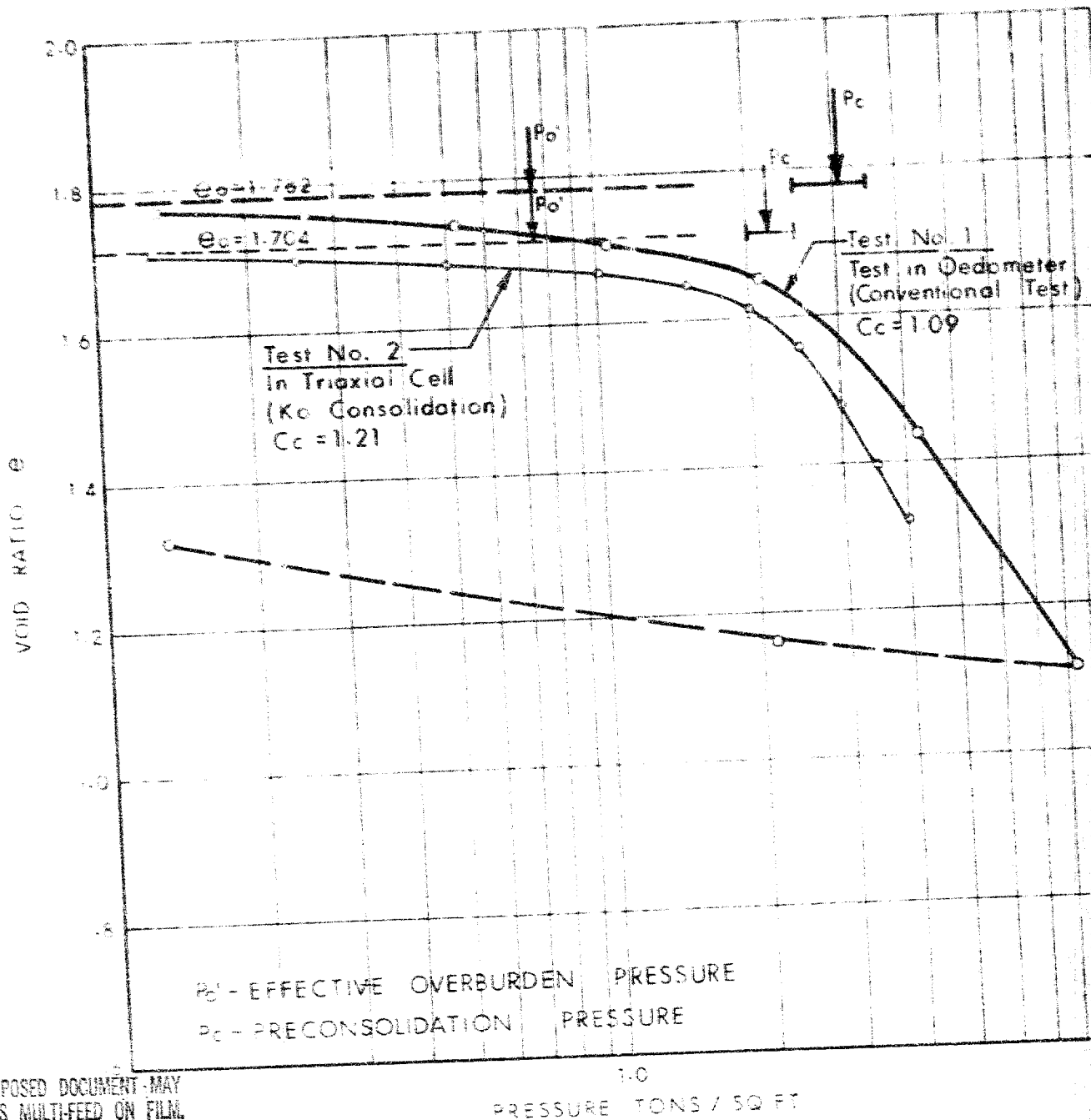


FIG. 5

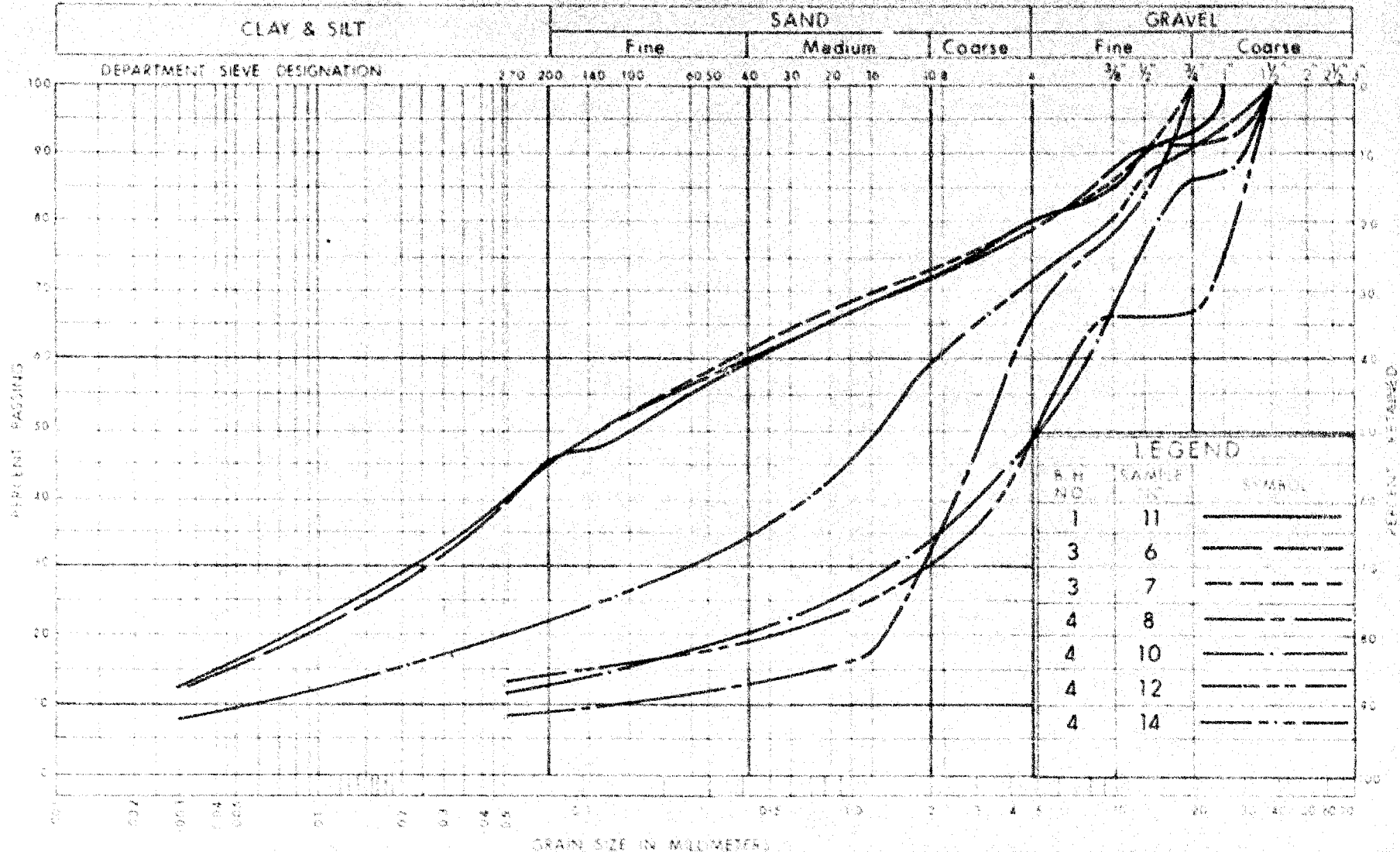
VOID RATIO VS PRESSURE

$W_L = 74.5$
 $W_p = 29.6$
 $W = 53.0 \%$

BORE HOLE 3
 SAMPLE 4
 DEPTH 20'-22'
 ELEV. 147.5-145.5



UNIFIED SOIL CLASSIFICATION SYSTEM



DEPARTMENT OF HIGHWAYS
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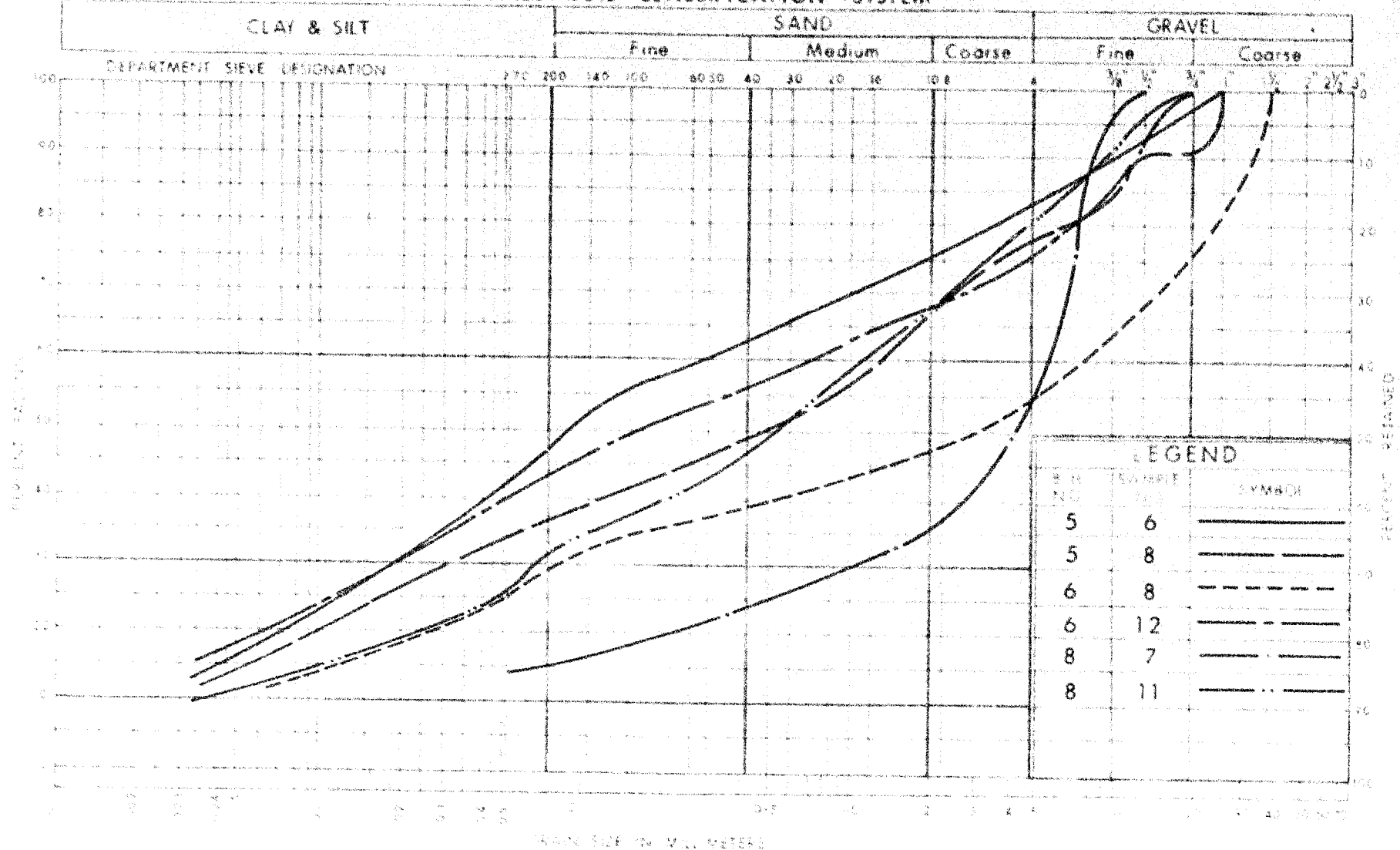
GRAIN SIZE DISTRIBUTION GLACIAL TILL

W.P. No. 37-66-05

JOB No. 69-F-37

FIG. 7

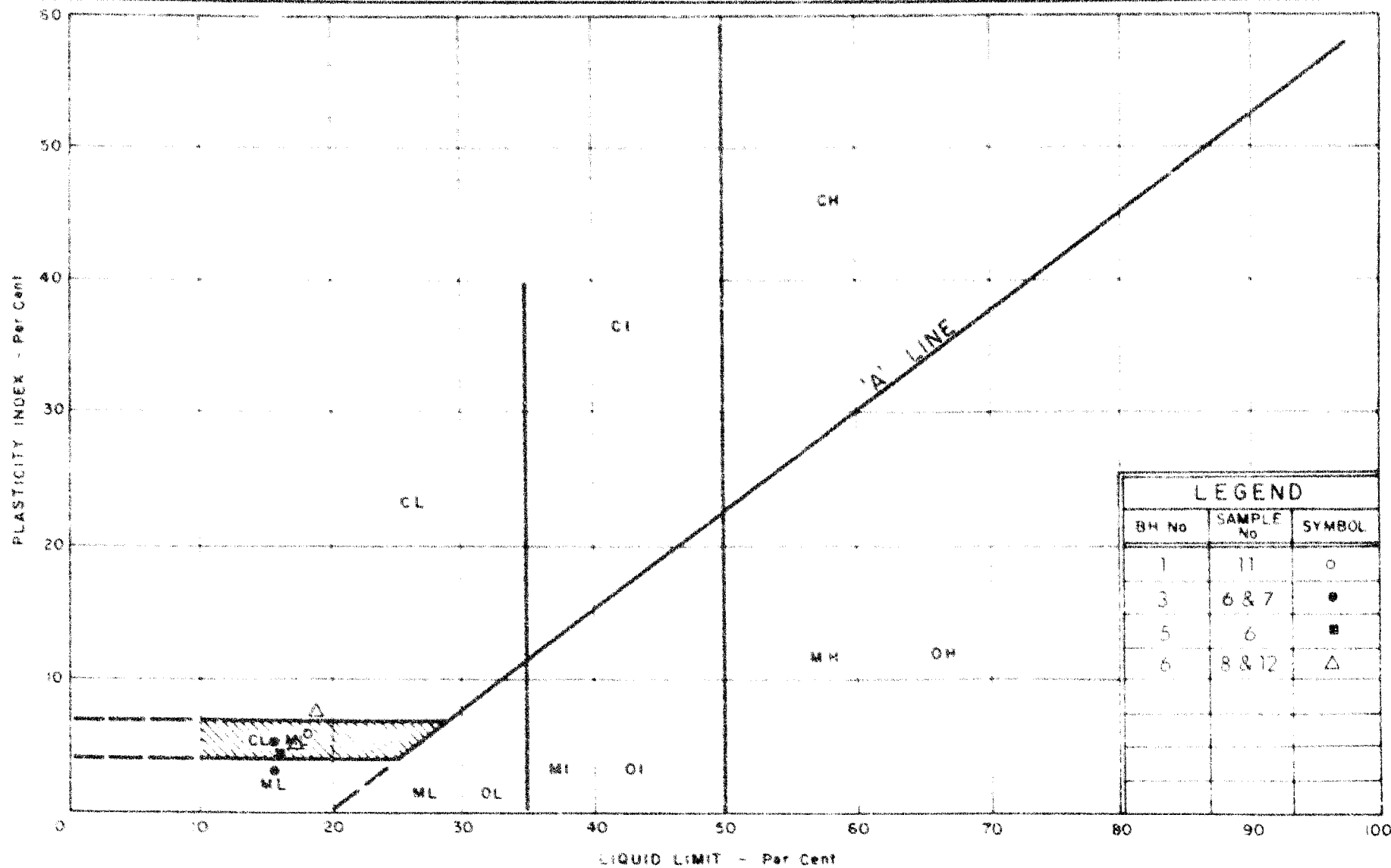
UNIFIED SOIL CLASSIFICATION SYSTEM



DEPARTMENT OF HIGHWAYS
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GRAIN SIZE DISTRIBUTION GLACIAL TILL

W.P. No. 37-66-05
JOB No. 69-F-37
FIG. 8



DEPARTMENT OF HIGHWAYS
MATERIALS and
TESTING
DIVISION

PLASTICITY CHART GLACIAL TILL

WP No 37-66-05

JOB No. 69-F-37

FIG. 9

ABBREVIATIONS USED IN THIS REPORT

PENETRATION RESISTANCE

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

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

DESCRIPTION OF SOIL

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

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

TYPE OF SAMPLE

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

SOIL TESTS

Q _u	UNCONFINED COMPRESSION	L.V	LABORATORY VANE
Q	UNDRAINED TRIAXIAL	F.V	FIELD VANE
Q _{cu}	CONSOLIDATED UNDRAINED TRIAXIAL	C	CONSOLIDATION
Q _d	DRAINED TRIAXIAL	S	SENSITIVITY

ABBREVIATIONS USED IN THIS REPORT

SOIL PROPERTIES

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

GENERAL

π	= 3.1416
e	BASE OF NATURAL LOGARITHMS 2.7183
$\log_e \sigma$ OR $\ln \sigma$	NATURAL LOGARITHM OF σ
$\log_{10} \sigma$ OR $\log \sigma$	LOGARITHM OF σ 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
e	LINEAR STRAIN
γ	SHEAR STRAIN
ν	POISSON'S RATIO (μ IS ALSO USED)
E	MODULUS OF LINEAR DEFORMATION (YOUNG'S MODULUS)
G	MODULUS OF SHEAR DEFORMATION
K	MODULUS OF COMPRESSIBILITY
η	COEFFICIENT OF VISCOSITY

EARTH PRESSURE

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

FOUNDATIONS

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

SLOPES

H	VERTICAL HEIGHT OF SLOPE
D	DEPTH BELOW TOE OF SLOPE TO HARD STRATUM
β	ANGLE OF SLOPE TO HORIZONTAL

SUMMARY OF PILE DRIVING RECORDS

W.O. 69-11037 W.P. 37-66-13 CONT. 71-66 DIST. 9
 SITE CARILLON PARK RD INTERCHANGE (CO. RD. #14)
 DATE DRIVEN AUG 10 - SEPT 11/71 WEIGHT OF ANVIL 800 LB
 HAMMER TYPE DROP HAMMER WEIGHT 7000 LB ENERGY _____

LOCATION OF PILES	PILE				ESTIMATED TIP EL. (ft.)	DIFFERENCE Longer(+) Shorter(-) Than Estimated (ft.)	REMARKS
	TYPE	NO.	LENGTH (ft.)	TIP EL. (ft.)			
NORTH ABUT. FTG.	12 BP53	1	21.5	135.0	110.0	- 25.0	
---	---	2	19.7	138.0	---	- 28.0	
---	---	3	22.3	134.5	---	- 24.5	
---	---	4	22.3	134.5	---	- 24.5	
---	---	5	20.6	136.1	---	- 26.1	
---	---	6	20.6	135.9	---	- 25.9	
---	---	7	27.3	130.0	---	- 20.0	
---	---	8	26.7	130.6	---	- 20.6	
---	---	9	28.9	128.5	---	- 18.5	
---	---	10	21.3	135.8	---	- 25.8	
---	---	11	28.3	129.1	---	- 19.1	
---	---	12	26.8	130.5	---	- 20.5	
---	---	13	27.7	129.7	---	- 19.7	
---	---	14	28.1	129.3	---	- 19.3	
SOUTH ABUT. FTG.	---	15	18.5	144.1	120.0	- 24.1	
---	---	16	19.4	143.2	---	- 23.2	
---	---	17	18.1	144.4	---	- 24.4	
---	---	18	18.3	143.3	---	- 23.3	
---	---	19	16.1	146.4	---	- 26.4	
---	---	20	14.3	148.1	---	- 28.1	
---	---	21	16.7	145.8	---	- 25.8	
---	---	22	15.8	146.7	---	- 26.7	
---	---	23	19.1	142.9	---	- 22.9	
---	---						

TILL
a) GLENN
DRIVEN TO

DRIVEN TO CLAY
TO CLAY CLAY
OVERBURY

SUMMARY OF PILE DRIVING RECORDS

W.O. 69-11037 W.P. 37-66-13 CONT. 7-66 DIST. 9

SITE CARILLON PARK RD INTERCHANGE (Co Rd #14)

DATE DRIVEN Aug 10 - SEP 7/71 WEIGHT OF ANVIL 500 LB

HAMMER TYPE DROP HAMMER WEIGHT 7000 lb ENERGY

[illegible]

MEMORANDUM

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

From: C.S. Grebski,
Bridge Office

ATTENTION:

DATE: March 31, 1970

OUR FILE REF.

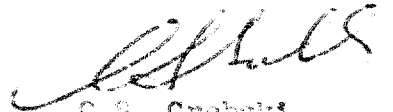
IN REPLY TO

SUBJECT: Carillon Park Rd. Interchange (Co. Rd. #14)
S.P. 37-55-13, Site 27-200
Highway 417, District No. 2

69-F-37

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

CCG:rv

Attach.

c.c. Foundation Office

We assume that the following comments have been incorporated in your design drawings:

- i) Pilement piles should be designed for a safe load of 70 tons taking into account for negative skin frictional forces
- ii) Reinforced concrete will be required for piles in view of the presence of boulders in the subsoil.

Mr. Sternac

April 14th 1970



69-F-37

MEMORANDUM

To: Mr. S. J. Markiewicz,
Regional Road Design Engineer,
Road Design Office,
Kingston, Ontario.

From: Materials and Testing Office,
Kingston, Ontario.

ATTENTION

Date: December 8, 1966.

OUR FILE REF.

IN REPLY TO:

SUBJECT:

Re: W.P. 37-66-06,
Interchange of Highway 417
and Carillon Park Road

The accompanying soils profile (M/K4-3-2) of the interchange legs indicate the recommended cut treatments for the gradeline shown. The clay cut treatment indicated on page 5 of the original Soils Design Report has been revised to provide for 6" G.B.C. Class 'A' over 30" Sand Cushion.

It is pointed out that all of the cut material is considered unsuitable for use in fills. It is therefore recommended that the gradeline in cuts be raised as much as possible to reduce the excavation quantities particularly on the leg in quadrant 'B'.

It has been assumed that the foundation conditions under the fills (up to 20' high) in quadrant 'C' are similar to that under Carillon Park Road approach fills to the structure. In the foundation investigation report for Carillon Park Road structure and approach fills, settlement up to 16 inches is predicted due to consolidation of the foundation soils. It has been indicated in the foundation investigation report that approximately 50 percent of the consolidation will take place within a year. It would therefore be desirable to build the fills in quadrant 'C' under the grading project on Hwy. 417 (W.P. 37-66-5) or as one of the initial operations of the follow up paving project (W.P. 37-66-06).

E. A. Meyer
E. A. Meyer,

for: J. E. Gruspier,
Regional Materials Engineer.

HAM/hl

c.c. A. G. Stermac	C. H. Robertson
D. W. Ferren	T. Kingsland
H. A. Tregaskes	M. Stoyanoff
T. C. Muir	B. M. Ernesaks
W. Wagle	J. D. Katona
G. A. Wrong	C. Prosser

MEMORANDUM

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

ATTENTION:

FROM: Bridge Section,
Kingston, Ontario.

DATE: May 15, 1969.

OUR FILE REF.

IN REPLY TO

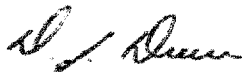
SUBJECT:

W.P. 37-66-13, Site 27-200,
Carillon Park Road Interchange,
Highway 417, District 9

69-F-37

We are sending to you herewith two prints of Bridge Site Plan E-4671-1 on which we have marked the proposed location of the subject structure. Also enclosed are two copies of your Field Reconnaissance Report.

We would be pleased if you will make arrangements for the necessary foundation investigation and to have your report, the scheduled date for which is July 9, 1969.



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

DJD/GS/hl
Encls.
c.c. (with encl.)
Bridge Office Files Section

#69-F-37

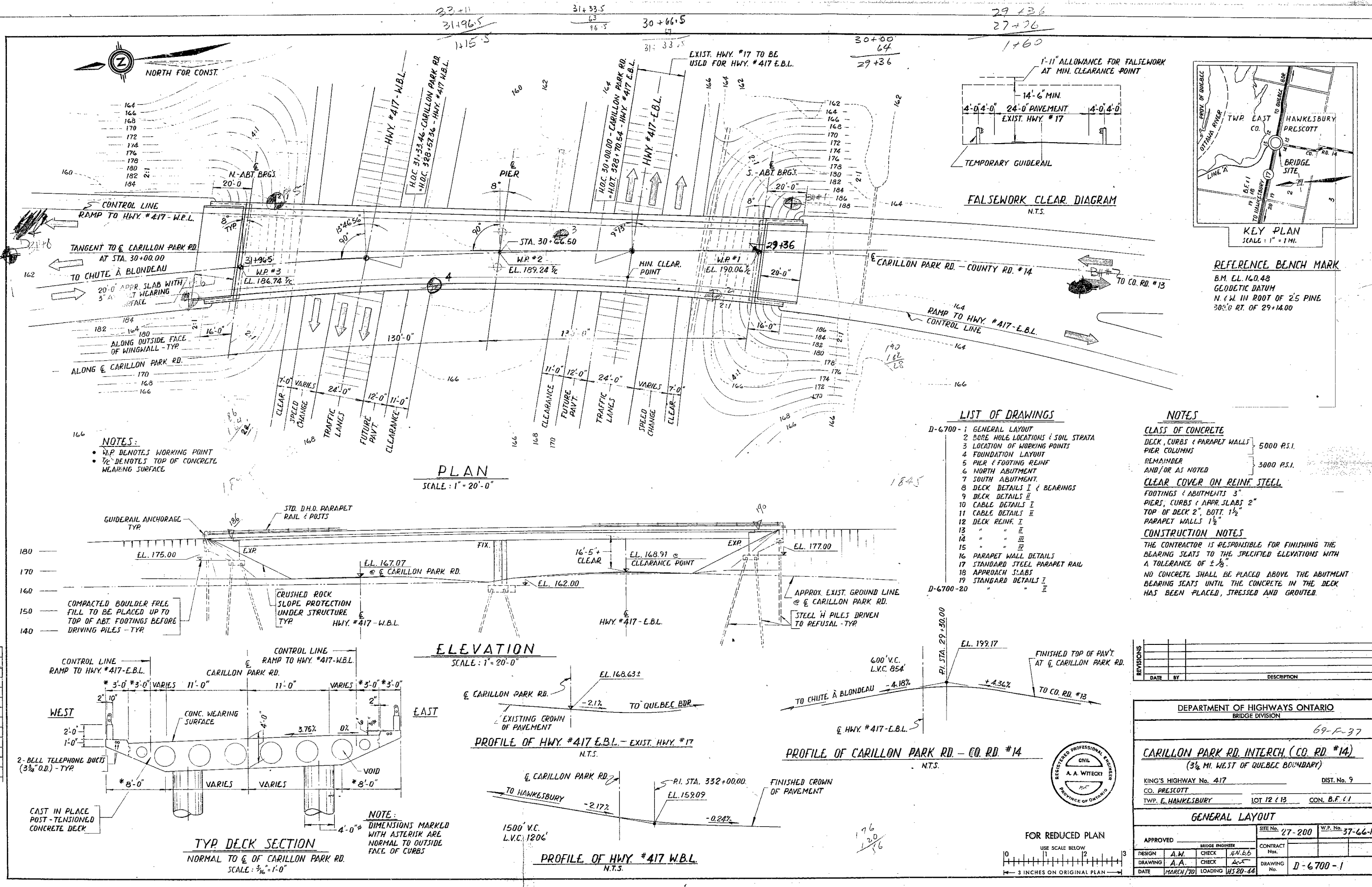
W.P. 37-66-13 (STR.)

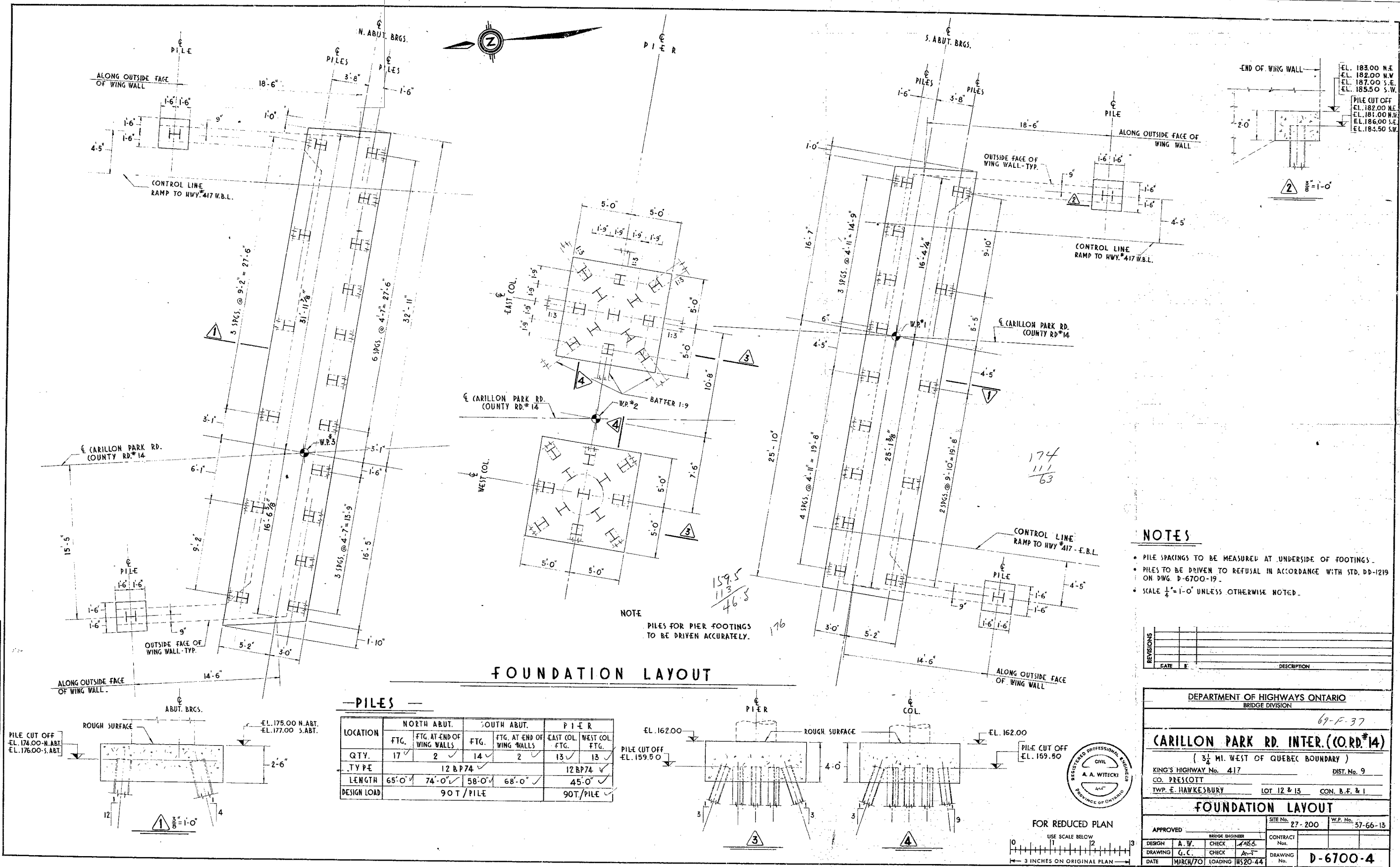
HWY #417 (E.B.L. AND W.B.L.)

CARILLON PARK ROAD

LINE 'A'

UNDERPASS STRUCTURE



[illegible]

— PILES —					
LOCATION	NORTH ABUT.		SOUTH ABUT.		P I E R
	FTG.	FTG. AT END OF WING WALLS	FTG.	FTG. AT END OF WING WALLS	EAST COL. FTG. WEST COL. FTG.
QTY.	17 ✓	2	14 ✓	2 ✓	13 ✓ 13 ✓
TYPE	12 BP 74				
LENGTH	65'-0" ✓	74'-0" ✓	58'-0" ✓	68'-0" ✓	45'-0" ✓
DESIGN LOAD	90 T / PILE				90 T / PILE ✓

REVISIONS <div style="border: 1px solid black; height: 100px; width: 100%;"></div>	DATE <div style="border: 1px solid black; height: 100px; width: 100%;"></div>	DESCRIPTION <div style="border: 1px solid black; height: 100px; width: 100%;"></div>
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DEPARTMENT OF HIGHWAYS ONTARIO BRIDGE DIVISION		
<div style="font-size: 1.5em; margin-bottom: 10px;">69-F-37</div> <div style="font-size: 1.5em; margin-bottom: 10px;">(CARILLON PARK RD. INTER. (C.O.R.D.#14))</div> <div style="font-size: 1.2em; margin-bottom: 10px;">(3 ¹/₄ MI. WEST OF QUEBEC BOUNDARY)</div> <div style="display: flex; justify-content: space-between;"> KING'S HIGHWAY No. <u>417</u> Dist. No. <u>9</u> </div> <div style="display: flex; justify-content: space-between;"> CO. <u>PRESOTT</u> </div> <div style="display: flex; justify-content: space-between;"> TWP. <u>E. HAWKESBURY</u> LOT <u>12 & 13</u> CON. <u>B.F. & I</u> </div>		
<div style="font-size: 2em; margin-bottom: 10px;">FOUNDATION LAYOUT</div>		
APPROVED <div style="border: 1px solid black; height: 40px; width: 100%;"></div>	SITE No. <div style="border: 1px solid black; height: 40px; width: 100%; text-align: center; font-weight: bold;">27-200</div>	W.P. No. <div style="border: 1px solid black; height: 40px; width: 100%; text-align: center; font-weight: bold;">57-66-13</div>
<div style="display: flex; justify-content: space-between;"> <div style="width: 40%;"> BRIDGE ENGINEER <div style="border: 1px solid black; height: 40px; width: 100%;"></div> </div> <div style="width: 60%;"> CONTRACT No. <div style="border: 1px solid black; height: 40px; width: 100%;"></div> </div> </div>		
DESIGN <div style="border: 1px solid black; height: 40px; width: 100%; text-align: center;">A.W. CHECK <i>AWS.</i></div>	DRAWING <div style="border: 1px solid black; height: 40px; width: 100%; text-align: center;">G.C. CHECK <i>AS</i></div>	
DATE <div style="border: 1px solid black; height: 40px; width: 100%; text-align: center;">MARCH/70</div>	LOADING <div style="border: 1px solid black; height: 40px; width: 100%; text-align: center;">HS20-44</div>	DRAWING No. <div style="border: 1px solid black; height: 40px; width: 100%; text-align: center; font-weight: bold; font-size: 1.2em;">D-6700-4</div>