

G.I.-30 SEPT. 1976

GEOCRES No. 3165-79DIST. 9 REGION W.P. No. 10-69-03/04CONT. No. 73-190W. O. No. STR. SITE No. HWY. No. 417LOCATION C.N.R. & Hwy 417No of PAGES -=====OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT. REMARKS:

DEPARTMENT OF TRANSPORTATION AND COMMUNICATIONS

MEMORANDUM

To: Mr. T. C. Kingsland, (2) FROM: Foundations Office,
Regional Bridge Planning Engineer, Design Services Branch,
Eastern Region, Central Bldg., Downsview.
Kingston, Ontario.

ATTENTION: DATE: February 28, 1972.

OUR FILE REF. IN REPLY TO MAR 7 1972

SUBJECT:

3165-79

FOUNDATION INVESTIGATION REPORT
For
Proposed Structures
At the Crossing of the C.N.R.
And Hwy. #417 (E.B.L. and W.B.L.)
Regional Municipality of Ottawa-Carleton
District No. 9 (Ottawa)
W.O. 71-11124 -- W.P. 10-69-03 (E.B.L.)
10-69-04 (W.B.L.)

SITE 3-301A
3-301B

Attached, we are forwarding to you our detailed foundation investigation report on the subsoil conditions existing at the above-mentioned 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/ao
Attach.

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

cc: Messrs. D. W. Farren
B. R. Davis
A. Rutka
S. J. Markiewicz
J. E. Callaghan
B. J. Giroux
E. H. Saint
G. A. Wong
B. A. Singh
H. W. Dillon & Co. Ltd., Ottawa. (J. Kearney)
Foundations Files
Documents

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For

At the Crossing of the C.N.R.

Regional Municipality of Ottawa-Carleton

W.O. 71-11124 -- W.P. 10-69-03 (E.B.L.)

10-69-04 (W.B.L.)

The Foundations Office was requested to carry out a subsurface investigation at the proposed eastbound and westbound lane crossings of Hwy. #417 and Canadian National Railway in the Township of Gloucester, Regional Municipality of Ottawa-Carleton. The request was contained in a memo from Mr. T. C. Kingsland, Regional Bridge Planning Engineer, Eastern Region, dated October 23, 1971. An investigation was subsequently carried out by this Office to determine the subsoil, bedrock and groundwater conditions at the sites.

The report contains the factual results obtained from the investigation, together with recommendations pertaining to the foundations of the proposed structure as well as the stability and settlement considerations associated with the approach fills.

The area under investigation is located approximately 1,000 feet north of Ridge Rd. along the C.N.R. tracks, in the Township of Gloucester, Regional Municipality of Ottawa-Carleton. The terrain is flat to gently undulating in relief between about elevation 210 and 220. The twin track C.N.R. line runs in a north-south direction. The land in this area is used primarily for farming purposes.

The present physical features of the region are of a varied origin and are the result of erosion and deposition by various agencies. During a long period of time, the region was above sea level*. During this time the major features of bedrock topography were formed by processes of weathering and stream erosion. During Pleistocene time the region was invaded by one or more ice sheets advancing from the north. The pre-glacial land surface was modified by glacial erosion and by deposition, in places, of material eroded by the ice sheet. Near the close of the Pleistocene time when the ice sheet began to retire, the area was, in large part, below the sea level, so that as the ice retired and melted back, the sea entered and overspread the Ottawa valley to a depth, in places, of several hundred feet. In this arm of the sea known as the Champlain Sea, thick deposits of sand, silt and clay were laid down. As the ice sheet still further retired, uplift took place and the land gradually emerged from the sea. The area is now commonly called "The Ottawa Valley Clay Plains." Here extensive sensitive clay deposits are interrupted by ridges of sand and/or bedrock. The clay is generally underlain by glacial till, which, in turn, is followed by shale bedrock of the Collingwood and Gloucester formation.

3. FIELD AND LABORATORY WORK:

Ten sampled boreholes, eight of which were accompanied by a dynamic cone penetration test, were put down at the site during the field investigation, using conventional diamond drill rigs adapted for soil sampling purposes. In addition an extra

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*Johnston, W.A. - "Pleistocene and Recent Deposits in the Vicinity of Ottawa, With a Description of the Soils."
Geological Surveys #84, Dept. of Mines.

dynamic cone penetration test was conducted at the site.

Samples of the overburden were obtained at required intervals in a 2-inch O.D. split-spoon sampler, which was hammered into the soil. The method of driving the split-spoon sampler conformed to the specifications for the Standard Penetration Test. The same method was used to advance the dynamic cone penetration tests. This was supplemented by obtaining some 2-inch I.D. Shelby Tubes in the cohesive portions of the overburden. These tubes were pushed manually into the soil. In situ vane tests were carried out, wherever possible, in the cohesive portions of the overburden, to determine the undrained shear strength characteristics of the strata. Bedrock was proven in three of the boreholes by obtaining BX size rock core samples.

The groundwater level conditions across the site were determined by recording the water levels in the open boreholes during the period of the investigation.

During sampling and drilling operations, detailed logs of the borings were made; these logs contain a record of the drilling and sampling techniques used, together with the soil and bedrock types encountered. The locations and elevations of all the borings, which were surveyed by the personnel from the Eastern Region Engineering Surveys Section, are shown on Drawing No. 71-11124A & B, together with the inferred stratigraphical profiles across the site.

All samples were subjected to a careful visual examination in the field and subsequently in the laboratory. Following this examination, laboratory testing was carried out on selected samples to determine the following physical properties of the subsoil.

- Bulk Density
- Natural Moisture Content
- Atterberg Limits
- Grain-Size Distribution
- Undrained Shear Strength
- Consolidation Characteristics

The results of these tests are plotted on the Record

of Borehole sheets and are summarized on Figures 1 to 5 inclusive, all contained in Appendix I of this report.

4. SUBSOIL AND BEDROCK CONDITIONS:

4.1) General:

The predominant stratum across the site is composed of a firm to very stiff silty clay to clay. The thickness of this cohesive stratum varies from 15 to 17.5 feet. Underlying this deposit is a 4 to 11.5 feet thick granular glacial till deposit composed of a heterogeneous mixture of silt, sand and gravel, with a trace of clay. The glacial till is followed by shale bedrock.

The boundaries of the various deposits, as determined in the boreholes, are shown on the accompanying Record of Borehole sheets. The stratigraphical profiles, shown on Drawings No. 71-11124 A & B, have been inferred from this data.

From ground surface downward the various soil and bedrock types encountered are as follows.

4.2) Silty Clay to Clay:

Directly beneath a nominal topsoil cover is the predominant stratum across the site, which is composed of a silty clay to clay. The thickness of this sensitive cohesive deposit varied from 15 to 17.5 feet. The material encountered is generally very homogeneous with a few notable exceptions. At B.H.'s # 8, 9, 11 and 12, for instance, a light and dark grey layering pattern was noticed. Occasional silt seams, up to 1 inch thick, were encountered throughout the stratum. Grain-size distribution curves for samples of the clay stratum are shown on Figure 2, in the Appendix.

The engineering properties of the deposit, as determined by field and laboratory testing, are presented in Table I.

TABLE I

		<u>Range</u>
Bulk Density (p.c.f.)	(γ)	103 - 118
Liquid Limit (%)	(W_L)	38 - 72
Plastic Limit (%)	(W_P)	21 - 32
Natural Moisture Content (W) (%)		32 - 62
Standard Penetration Resistance ('N') (Blows/ft.)		7 - 25
Initial Void Ratio	(e_o)	1.122 - 1.771
Compression Index	(C_c)	0.298 - 1.58
Degree of Preconsolidation Pressure (P_c) (t.s.f.)		1.23 - 3.1
Undrained Shear Strength (C_u) (p.s.f.)		
i) Field Vanes		720 - >2000
ii) Lab. Vanes		900 - 3,780
iii) Lab. Testing		295 - 3,050
Sensitivity		6 - 20

The Atterberg limit tests, summarized on Table I are also plotted on the Plasticity Chart, Fig. #1. These results indicate that, in general, the clay is inorganic with a plasticity in the intermediate to high range.

Based on the Standard Penetration and the undrained shear strength testing carried out it is estimated that the consistency of the cohesive stratum varies from firm to very stiff.

The consolidation characteristics of the stratum were determined by carrying out five laboratory tests, the results of which are shown as Void Ratio vs. Log of Pressure Plots, on Figures #4 and 5. The results of this testing indicate that

the clay is preconsolidated by 2500 to 6200 p.s.f. in excess of the existing overburden pressure.

4.3) Heterogeneous Mixture of Silt, Sand and Gravel, Trace of Clay - Glacial Till:

Underlying the cohesive sensitive silty clay to clay stratum is a deposit of glacial origin consisting of a heterogeneous mixture of silt, sand and gravel, with a trace of clay. This granular glacial till deposit varies from 4 feet to 11.5 feet in thickness. Typical grain size distribution curves, obtained from samples of this stratum, are shown on Figure No. 3.

The Standard Penetration Tests carried out gave 'N' values which ranged from 2 blows/ft. to 163 blows/ft. Based on these values it is estimated that the relative density of the glacial till stratum varies from very loose to very dense. The low 'N' values were encountered in the upper portion of the deposit. This would indicate that the upper zone has been reworked by weathering elements.

4.4) Shale Bedrock:

The granular glacial till deposit is directly underlain by bedrock, which was proven in five of the boreholes by obtaining up to 9 feet of BX size rock core samples. Over the site the bedrock surface was found to vary between elevations 186 and 195, which corresponds to depths below ground surface of from 20 to 30.5 feet. The bedrock is composed of a grey calcareous shale which is in a sound condition as evidenced by the high percentage of rock core recovered.

The bedrock core samples were examined by Mr. K. Ingham, Geologist, Department of Transportation and Communications. Mr. K. W. Ingham presented the result of his bedrock examination in a memo to this Office, dated February 25, 1972.

5. GROUNDWATER CONDITIONS:

The groundwater level conditions across the site, during the period of investigation (November 1971), were observed by

taking readings in the open boreholes. The results of the readings are summarized on Drawings No. 71-11124 A and B.

The observations indicate that the groundwater level is located between elevations 212 and 216, which corresponds to levels which range from ground surface to as much as 4 feet below existing ground surface.

6. DISCUSSIONS AND RECOMMENDATIONS:

6.1) General:

The design and construction of the rural portion of Hwy. #417, south-easterly of Ramseyville to Quebec border will be completed by 1974. The Department and the Regional Municipality of Ottawa-Carleton have undertaken a study to determine the most appropriate alignment for the urban portion of Hwy. #417 in the Ottawa area. A preliminary Foundation Report, W.O. 70-11115, for various corridors of Hwy. #417, from Ramseyville northerly to the Ottawa Queensway (west of Blair Rd.) was submitted on March 23, 1971. The finalized alignment requires interchanges at Hunt Club Rd., Walkley Rd. extension, Innes Rd. and Ottawa Queensway. In addition, structures will be required at the crossings of Baseline Rd., Canadian National Railway and Canadian Pacific Railway with the proposed Hwy. #417.

This discussion deals with the proposed overhead structures at the crossings of eastbound and westbound lanes of Hwy. #417 and Canadian National Railway in the Township of Gloucester, Regional Municipality of Ottawa-Carleton. Discussions with regard to other structures on this portion of the freeway will be covered under separate foundation reports. At the time of field investigation a simply supported, single span structure (clear span 40 ft.) with closed type abutments was being considered.

A letter, dated February 2, 1972, was submitted by this Office stating that the closed-type abutment scheme will not provide adequate overall stability unless the abutments foundations extended to the glacial till deposit. It was agreed by the

personnel from this Office and Structural Office to adopt the alternate scheme, consisting of a three-span structure with spill-through abutments. This change in scheme will necessitate additional borings, which will be carried out by this Office and the results will be submitted as an addendum to this Report at a later date.

The profile grades of the eastbound and westbound lanes of Hwy. #417 in the vicinity of the overhead structure locations will be about elevation 246 to 248. The C.N.R. track under the structure will remain at elevation 221, with the surrounding ground at about elevation 215. The associated approaches, therefore, will be 33 feet in height, both in the longitudinal and transverse directions.

The predominant stratum across the site is composed of a grey firm to very stiff silty clay to clay having a thickness ranging from 15 to 17.5 feet. The cohesive deposit is underlain by 4 to 11.5 feet of glacial till (heterogeneous mixture of silt, sand and gravel) which in turn is followed by sound shale bedrock.

The presence of firm to very stiff, compressible cohesive stratum, at a shallow depth below ground surface, is the governing factor from a foundation point of view, since it will be necessary to ensure that it is not over-stressed by either the embankment or structure loadings. These aspects will be discussed in detail in this section of the report.

6.2) Approach Embankments:

6.2.1) Stability Considerations:

The approach fills for the EBL and WBL structures, will have a maximum height of 33 feet, above the existing ground surface. The critical condition for stability of an embankment on slightly overconsolidated clays, as is the case with this clay stratum, generally occurs during or immediately after construction. This being the case, a total stress analysis ($\phi = 0$) provides a suitable means of assessing the stability of the embankment sections. For this method of analysis, stability is governed by

the applied loads and by the stress-strain and undrained shear strength characteristics of the foundation and embankment soils.

Analyses have been carried out, therefore, in terms of total stresses by the use of the electronic computer, to determine the stability of the fill sections. The following assumptions were made.

Soil Properties

Elevation	Soil Type	Angle of Shearing Resistance (Degrees)	Undrained Shear Strength (p.s.f.)	Unit Weight (p.s.f.)
248 - 215	Fill	30°	-	125 / 140
215 - 210	Silty Clay	-	2,000	125
210 - 207	Silty Clay	-	1,550	120
207 - 204	Silty Clay	-	900	120
204 - 197	Silty Clay	-	700	110
197 - 190	Glacial Till	37°	-	140

Notes: Ground Water at Elevation 212.

Tension Crack = $H/3$

Fill Slopes 2:1

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

From the stability analyses the following conclusions have been drawn:

- i) Fills less than 28 feet in height may be constructed with standard 2:1 side slopes.
- ii) Fills in excess of 28 feet should be constructed with a single berm at mid-height; e.g. for $H = 33$ feet a mid-height berm of 10 feet will be required.

- iii) Smooth transitions should be affected between full berm section at 33 feet in height to no berm section at 28 feet in height.

6.2.2) Settlement Considerations:

The underlying compressible silty clay to clay stratum will undergo settlements due to consolidation, over a long term period, under the weight of the approach embankments. Settlement computations were carried out for maximum fill height of 33 feet, with the required berm. The results indicated that settlements up to 4 inches can be anticipated under the centre-line of the embankment, over a maximum period of 5 years. Fifty percent of these settlements will be realized within the first 12 months after the placement of the fill.

6.3) Structure Foundations:

The strength and compressibility characteristics of the underlain clay deposit is such that it will be necessary to support the structure abutments and pier foundations on end-bearing piles.

6.3.1) Piers:

Piers can be supported on end-bearing piles driven to the sound shale bedrock surface. Allowable loads will depend on the pile type and section chosen (e.g. 14 H 74 steel H-piles may be designed for 95 tons/pile).

At least 4 feet of earth cover should be to the underside of the pile cap for frost protection purposes.

The base of the pile cap will be located below the groundwater level recorded during the period of the investigation. The excavations will be carried out in the relatively impervious cohesive stratum. Therefore, no major dewatering problems are anticipated. Any minor seepage of surficial runoff into the excavation could be readily handled by using standard techniques, such as pumping from sumps.

6.3.2) Abutments:

The abutments can be 'perched' within the approach fills and supported on end-bearing piles driven to the sound shale bedrock surface. The allowable loads will be dependent on the pile section chosen. The anticipated consolidation settlements at the approaches will induce negative skin friction forces for the abutment piles. These forces combined with creep movements within the subsoil due to stratum imposed by embankment loading, will tend to displace the piles laterally. In view of this, it would be desirable to reduce the design loads by 15% of the maximum allowable load of the pile section chosen. For example 14 HP 74 steel H-piles should be designed for a maximum load of 80 tons/pile rather than the usual 95 tons/pile. Considerations should also be given to supporting the extreme ends of the wingwalls on piles driven to bedrock to prevent any possible tilting of the abutments. The abutment and the wingwall should be designed to act as one unit.

No bouldery or rock fill should be placed in areas where piles are to be driven.

7. MISCELLANEOUS:

The field work, performed during the period of November 28 to December 12, 1971, under the immediate supervision of Mr. S. A. Ahmad, Project Foundation Engineer, who also prepared this report.

Equipment was owned and operated by F. E. Johnston Drilling Co. Ltd., Ottawa.

This project was under the general supervision of Mr. M. Devata, who also reviewed this report.

Shahen Ahmad
S. A. Ahmad, P. Eng.

M. Devata
M. Devata, P. Eng.

SAA/ao
February 14, 1972.



APPENDIX I

FOUNDATION SECTION

JOB	<u>71-11124</u>	LOCATION	<u>Co-ords. 495,769 N; 233,682 E.</u>	ORIGINATED BY	<u>WH</u>
W.P.	<u>10-69-03 & 04</u>	BORING DATE	<u>Nov. 10, 1971</u>	COMPILED BY	<u>SO</u>
DATUM	<u>Geodetic</u>	BOREHOLE TYPE	<u>NX Washboring</u>	CHECKED BY	<u>[Signature]</u>

[illegible]

FOUNDATION SECTION

ORIGINATED BY TF
COMPILED BY SO
CHECKED BY [Signature]

[illegible]

FOUNDATION SECTION

ORIGINATED BY JS

COMPILED BY SO

CHECKED BY *AK*

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION BLOWS / FOOT	RESISTANCE	LIQUID LIMIT ——— W _L	PLASTIC LIMIT ——— W _p	WATER CONTENT ——— W	BULK DENSITY γ	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT		20 40 60 80 100	SHEAR STRENGTH P.S.F.	W _p ——— W ——— W _L	WATER CONTENT % 15 30 45	P.C.F.		
216.5	Ground Level												
0.0	Silty clay to clay Firm to Very Stiff Grey		1	SS	25								
			2	TW	PH	210							
			3	TW	PH								
			4	TW	PH								
			5	TW	PH	200							
17.0	Het. mix. of silt, sand & gravel, trace of clay. Loose - Compact Grey		6	SS	9								
190.0			7	SS	12	190							
26.5	Shale Bedrock Sound Grey		8	BX	38%								
181.5			9	BX	98%								
35.0	End of Borehole					180							

FOUNDATION SECTION

[illegible]

FOUNDATION SECTION

CHECKED BY 

[illegible]

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DEPARTMENT OF TRANSPORTATION AND COMMUNICATIONS

DESIGN SERVICES BRANCH

RECORD OF BOREHOLE No. 3 A

FOUNDATION SECTION

JOB 71-11124

LOCATION Co-ord's 496,003 N. 233,610 E.

ORIGINATED BY S.A.A.

W.P. 10-69-4


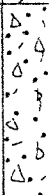


BORING DATE April 20, 1972

COMPILED BY A.T.

DATUM Geodetic

BOREHOLE TYPE Auger & RE Rock Core

CHECKED BY

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE					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		BLOWS / FOOT					SHEAR STRENGTH P.S.F.					WATER CONTENT %
							20	40	60	80	100	UNCONFINED + FIELD VANE QUICK TRIAXIAL x LAB. VANE					
							400	800	1200	1600	2000						
217.0	Ground surface.																
200.5	Silty clay to clay. Firm to very stiff. Grey.		1	SS	12	210											
			2	TW	PH												
			3	TW	PH												
			4	TW	PH												
			5	TW	PH												
16.5	Het. mix. of silt, sand and gravel. Trace of clay.		6	SS	6	200											
189.0	Probable bedrock.		7	SS	12	190											
28.0	End of borehole.					180											

DESIGN SERVICES BRANCH

FOUNDATION OFFICE

RECORD OF BOREHOLE NO 4A

JOB 71-11124LOCATION Co'ord's 496,069 N. 233,124 E.ORIGINATED BY S.A.A.W.P. 10-69-03BORING DATE June 16, 1972COMPILED BY S.A.A.DATUM GeodeticBOREHOLE TYPE Washboring - NX CasingCHECKED BY SK

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 P.S.F. ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE 100 800 1200 1600 2000					WATER CONTENT % W_P W W_L				
216.6	Ground surface.															
0.0	Silty clay, with trace of sand.		1	SS	11	210									Elev. ∇ 213.6 in open B.H. June 16/72	
	Firm to very stiff.		2	TW	PM											
	Grey.		3	TW	PM											
			4	TW	PM											
			5	TW	PM											
198.6						200										
18.0	Het. Mix. of silt, sand and gravel, trace of clay.		6	TW	PM	190										
	Glacial Till.		7	TW	60											
187.1	Very dense.					180										
29.5	Shale bedrock.		8	RC	Rec											
184.6	Sound - grey.		BX	100%												
32.0	End of borehole.															

DESIGN SERVICES BRANCH

FOUNDATION OFFICE

RECORD OF BOREHOLE NO 5A

JOB 71-11124

LOCATION Co-ord's 495,951 N. 233,170 E.

ORIGINATED BY S.A.A.

W.P. 10-69-03

BORING DATE June 19, 1972

COMPILED BY S.A.A.

DATUM Geodetic

BOREHOLE TYPE Washboring - NX Casing

CHECKED BY *AL*

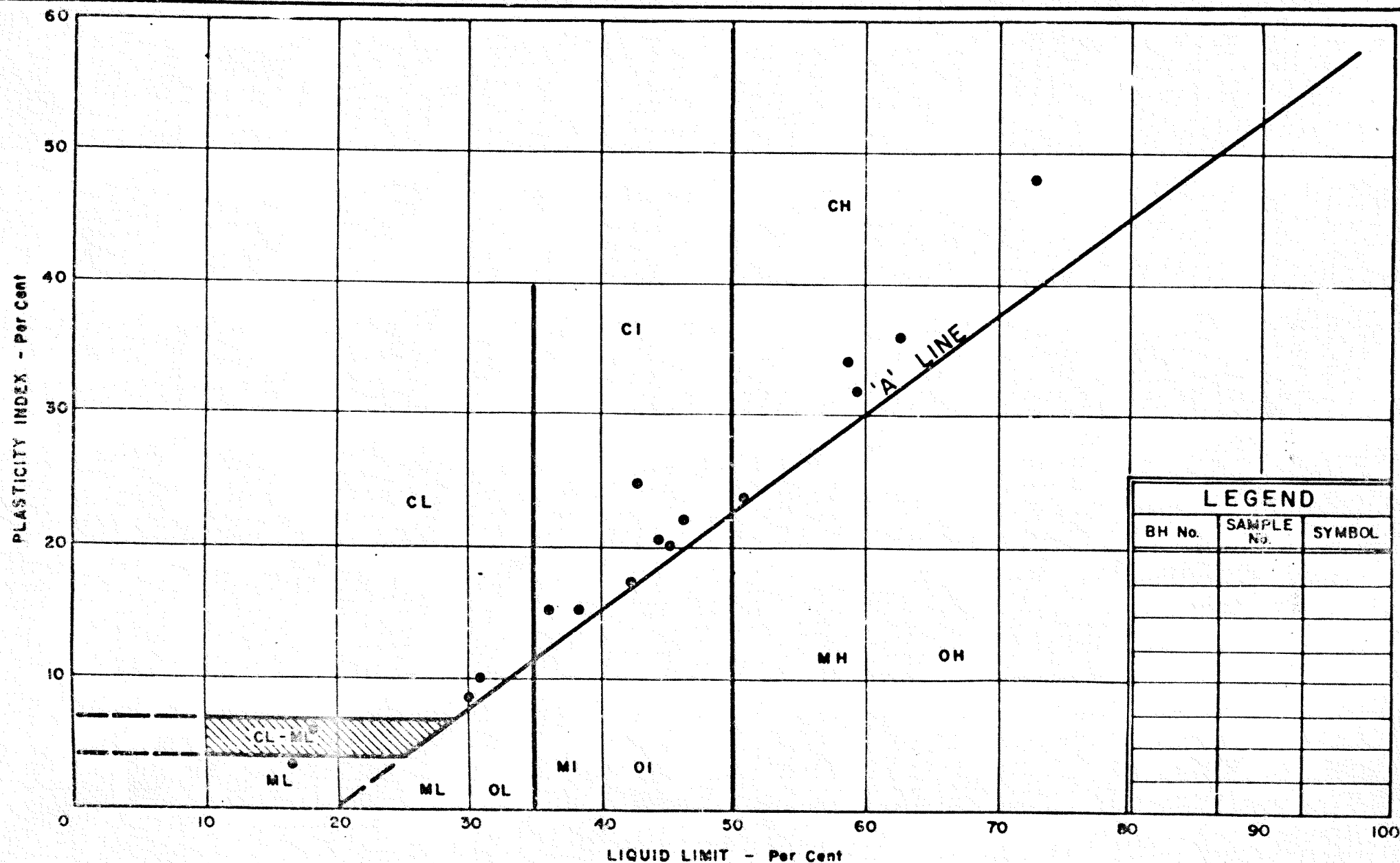
SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT						LIQUID LIMIT W_L PLASTIC LIMIT W_P WATER CONTENT W			BULK DENSITY γ P.C.F.	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FOOT		SHEAR STRENGTH P.S.F.						WATER CONTENT %				
						O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL X LAB VANE 400 800 1200 1600 2000						W_P — W — W_L WATER CONTENT %					
216.3	Ground surface.																
0.0	Silty clay, with trace of sand. Firm to very stiff. Grey.		1	SS	11												
			2	TW	PM	210											
			3	TW	PM												
			4	TW	PM												
199.3			5	TW	PM	200											
17.0	Het. mix. of silt, sand & gravel, trace of clay. Glacial Till		6	TW	PM												
	Very dense. Probable Bedrock.		7	SS	118	190											
188.3																	
28.0	End of borehole.					180											

FOUNDATION SECTION

JOB	71-11124	LOCATION	Co-ords. 495,992 N; 233,072 E.	ORIGINATED BY	JS
W.P.	10-69-03 & 04	BORING DATE	Dec. 10, 1971	COMPILED BY	SO
DATUM	Geodetic	BOREHOLE TYPE	NX Washboring	CHECKED BY	

[illegible]

DEPARTMENT OF HIGHWAYS - ONTARIO				RECORD OF BOREHOLE No. 11				FOUNDATION SECTION				
MATERIALS & TESTING OFFICE												
JOB 71-11124		LOCATION Co-ords. 495,866 N; 233,206 E.		ORIGINATED BY JS								
W.P. 10-69-03 & 04		BORING DATE Dec. 13, 1971		COMPILED BY SO								
DATUM Geodetic		BOREHOLE TYPE NX Washboring		CHECKED BY								
SOIL PROFILE		SAMPLES			DYNAMIC PENETRATION RESISTANCE		LIQUID LIMIT		BULK DENSITY		REMARKS	
					BLOWS / FOOT		PLASTIC LIMIT					
					20 40 60 80 100		W _L					
					SHEAR STRENGTH P.S.F.		W _p					
					400 800 1200 1600 2000		W					
							W _L					
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LEGEND		
BH No.	SAMPLE No.	SYMBOL



DEPARTMENT OF HIGHWAYS
 MATERIALS and
 TESTING
 DIVISION

PLASTICITY CHART SILTY CLAY TO CLAY

W.P. No. 10-69-03 & 04

JOB No. 71-11124

FIG. 1

UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT

SAND

GRAVEL

Fine

Medium

Coarse

Fine

Coarse

DEPARTMENT SIEVE DESIGNATION

274

200:

140

100

60.

0 - 4

•

0...

20

16

14

8.

4

3/

 $\frac{1}{2}$

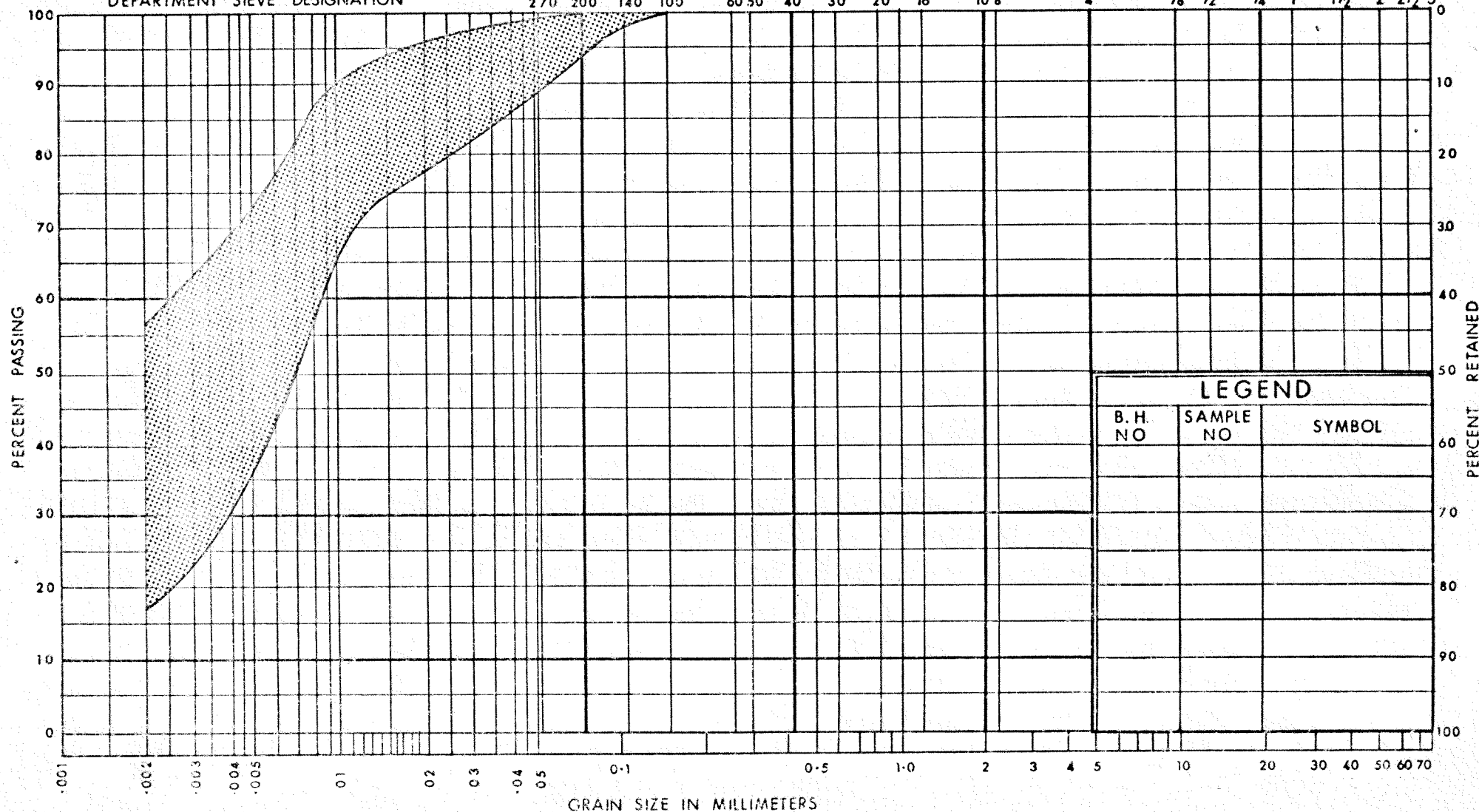
•

1

 $\frac{1}{2}$

2 25

3.



DEPARTMENT
OF
TRANSPORTATION AND COMMUNICATIONS



DESIGN SERVICES
BRANCH

GRAIN SIZE DISTRIBUTION
SILTY CLAY TO CLAY

W.P. No. 10-69-03 & 04

JOB No. 71 - 11124

FIG. 2

UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT

SAND

GRAVEL

Fine

Medium

Coarse

Fine

Coarse

DEPARTMENT SIEVE DESIGNATION

270

00

10

66

40

30.

14

10

4

...

 $\frac{1}{2}$

100

$$1\frac{1}{2}$$
 $2\frac{1}{2}$

3.

PERCENT PASSING

PERCENT RETAINED

LEGEND

B. H.
NO

SAMPLE NO

SYMBOL

GRAIN SIZE IN MILLIMETERS

DEPARTMENT
OF
TRANSPORTATION AND COMMUNICATIONS



DESIGN SERVICES
BRANCH

GRAIN SIZE DISTRIBUTION GLACIAL TILL

HET. MIXTURE OF SILT, SAND & GRAVEL, TRACE OF CLAY

W.P. No. 10-69-03 & 04

JOB No. 71-11124

FIG. 3

VOID RATIO-PRESSURE CURVES

JOB NO. 71-11124

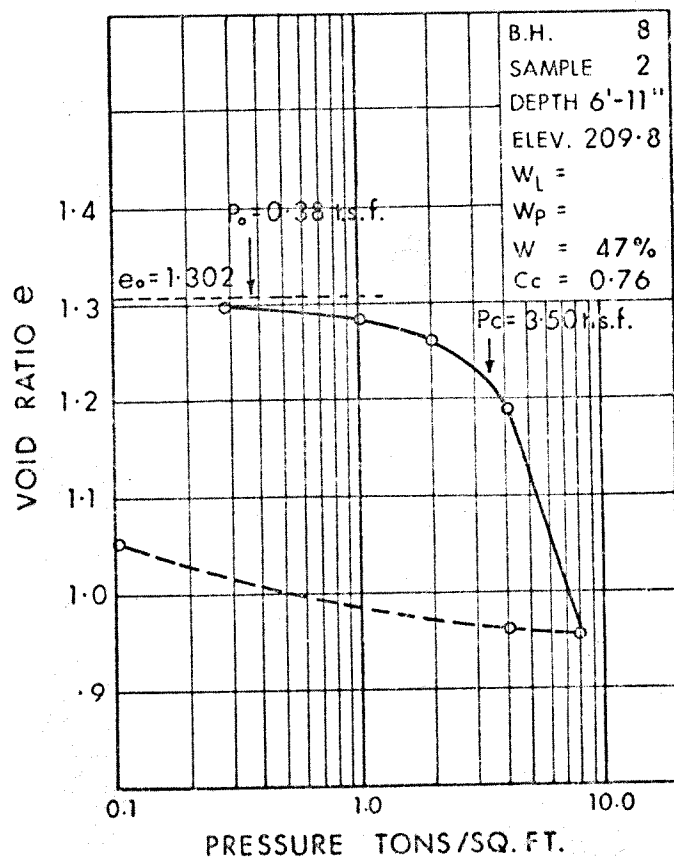
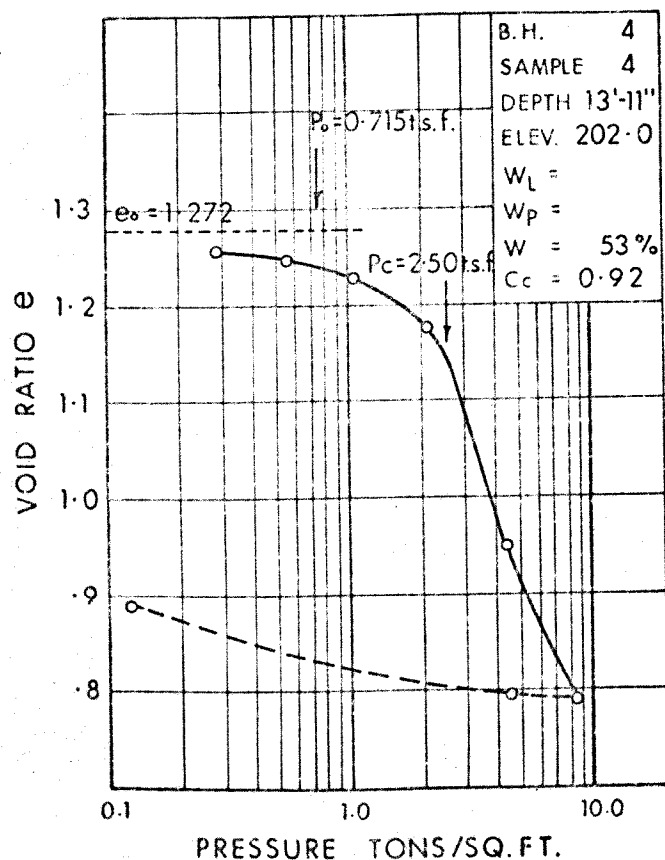
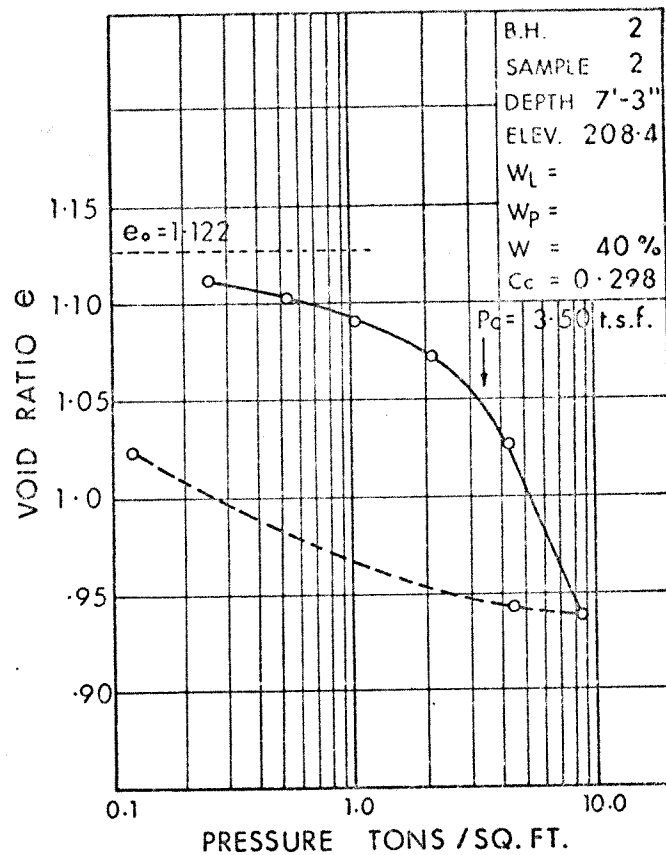
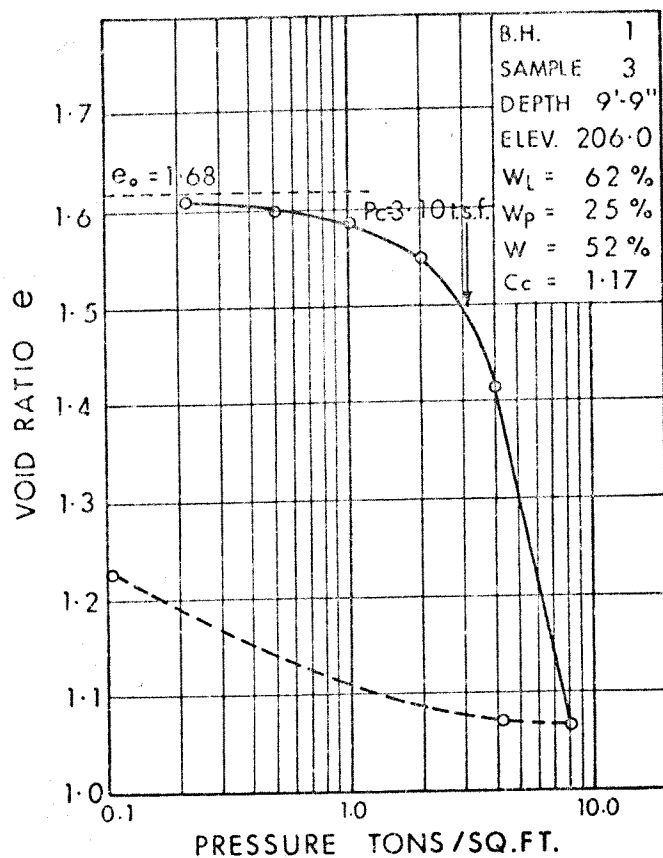
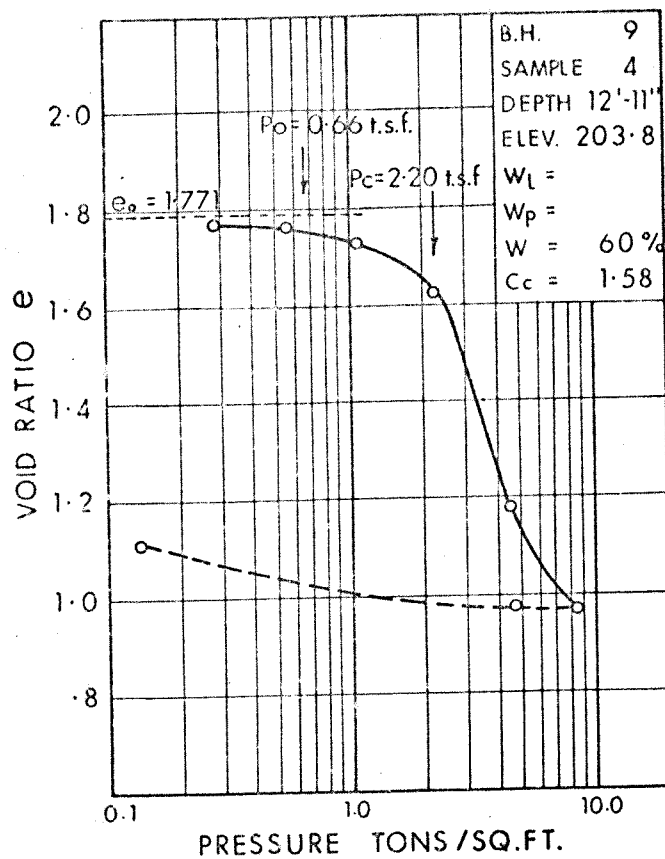


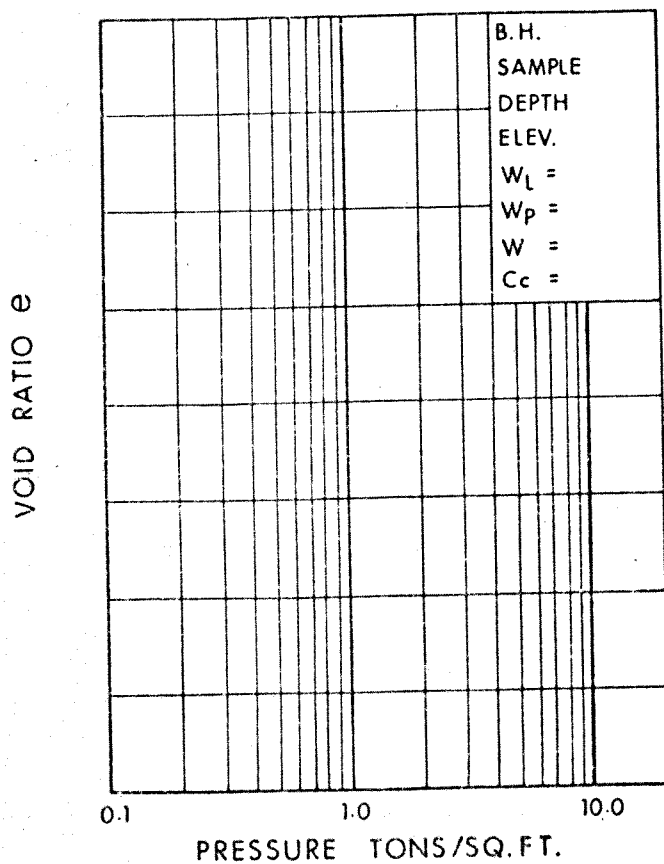
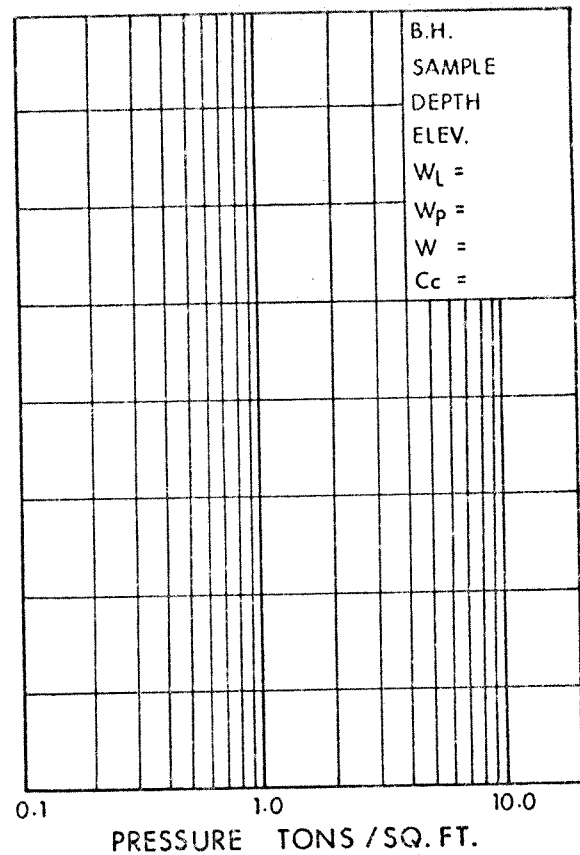
FIG. 4

VOID RATIO - PRESSURE CURVES

JOB NO. 71-11124



VOID RATIO e



VOID RATIO e

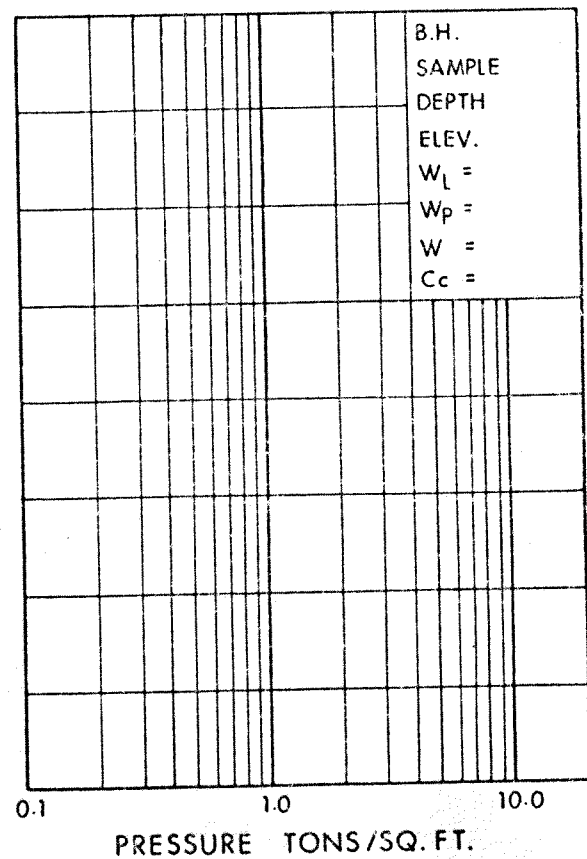


FIG. 5

ABBREVIATIONS USED IN THIS REPORT

PENETRATION RESISTANCE

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

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

DESCRIPTION OF SOIL

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

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

TYPE OF SAMPLE

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

SOIL TESTS

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

ABBREVIATIONS USED IN THIS REPORT

SOIL PROPERTIES

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

GENERAL

π	= 3.1416
e	BASE OF NATURAL LOGARITHMS 2.7183
$\log_e a$ OR $\ln a$	NATURAL LOGARITHM OF a
$\log_{10} a$ OR $\log a$	LOGARITHM OF a TO BASE 10
t	TIME
g	ACCELERATION DUE TO GRAVITY
V	VOLUME
W	WEIGHT
M	MOMENT
F	FACTOR OF SAFETY

STRESS AND STRAIN

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

EARTH PRESSURE

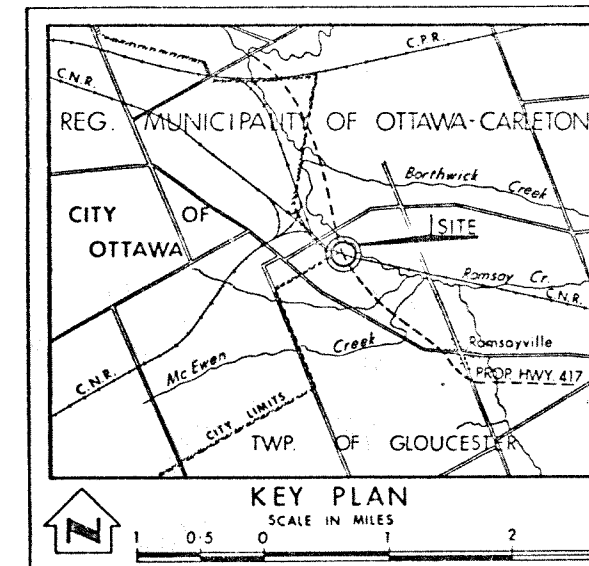
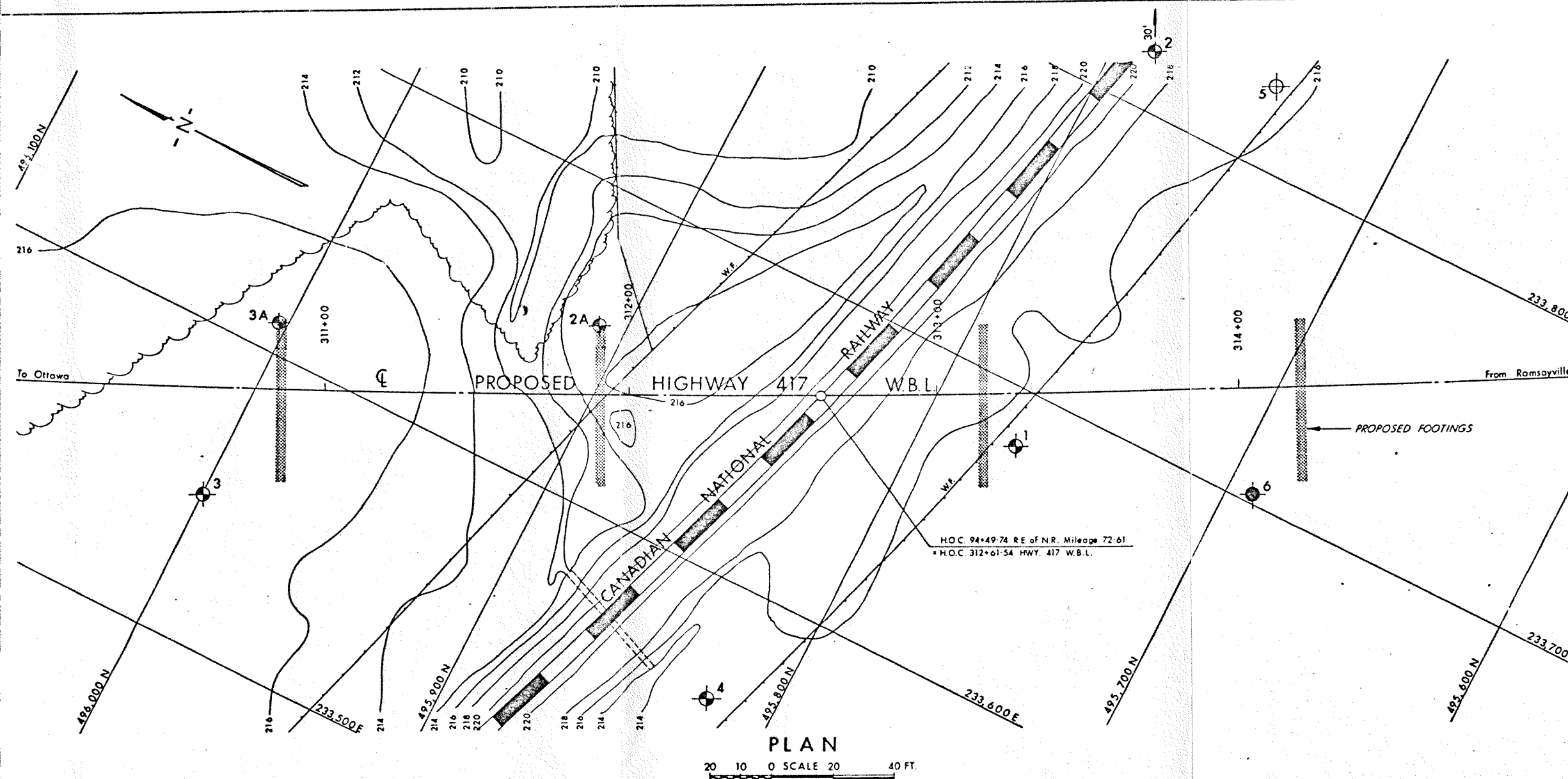
d	DISTANCE FROM TOP OF WALL TO POINT OF APPLICATION OF PRESSURE
δ	ANGLE OF WALL FRICTION
K	DIMENSIONLESS COEFFICIENT TO BE USED WITH VARIOUS SUFFIXES IN EXPRESSIONS REFERRING TO NORMAL STRESS ON WALLS
K_0	COEFFICIENT OF EARTH PRESSURE AT REST

FOUNDATIONS

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

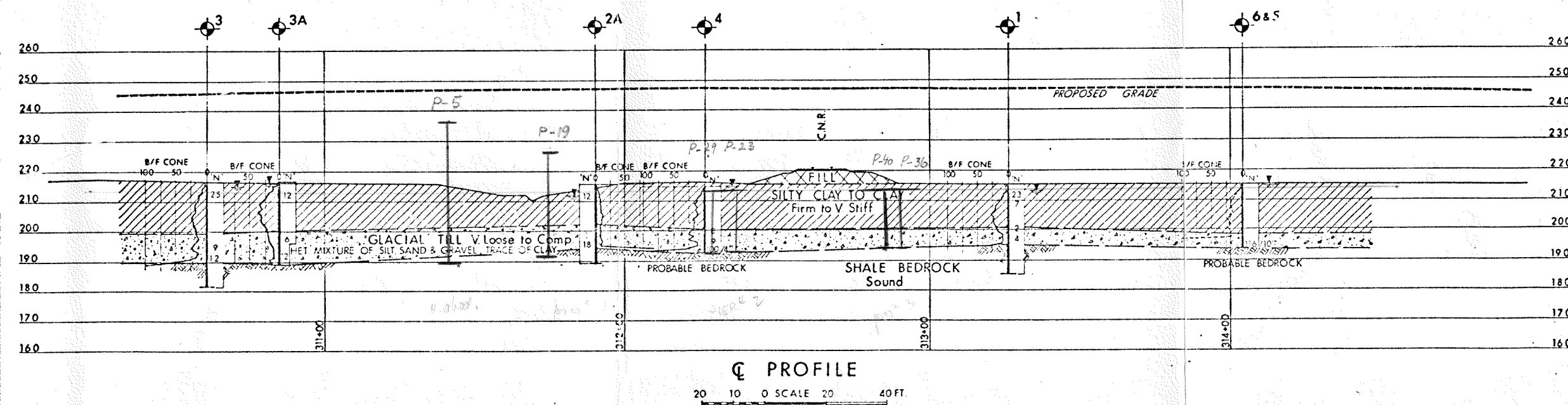
SLOPES

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



LEGEND				
	Bore Hole			
	Cone Penetration Test			
	Bore Hole & Cone Test			
	Water Levels established at time of field investigation. NOV. & DEC. 71 & APRIL 72.			
NO.	ELEVATION	CO-ORDINATES		
		NORTH	EAST	
1	215.7	495,769	233,682	
2	215.7	495,801	233,846	
3	216.5	495,999	233,548	
4	215.0	495,822	233,562	
5	215.9	495,746	233,826	
6	215.6	495,692	233,703	
2A	215.8	495,909	233,656	
3A	217.0	496,003	233,610	

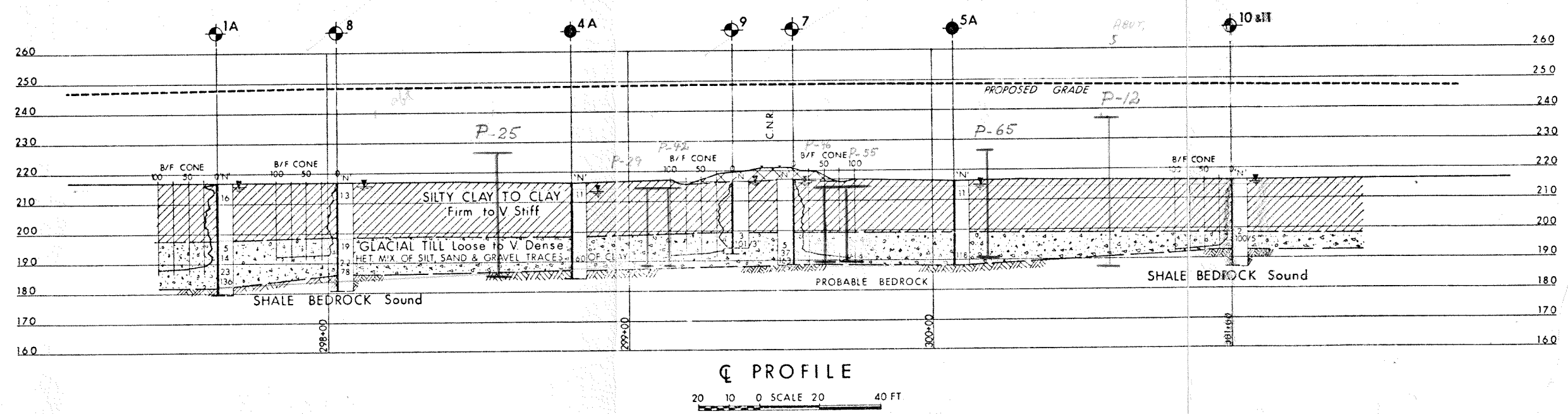
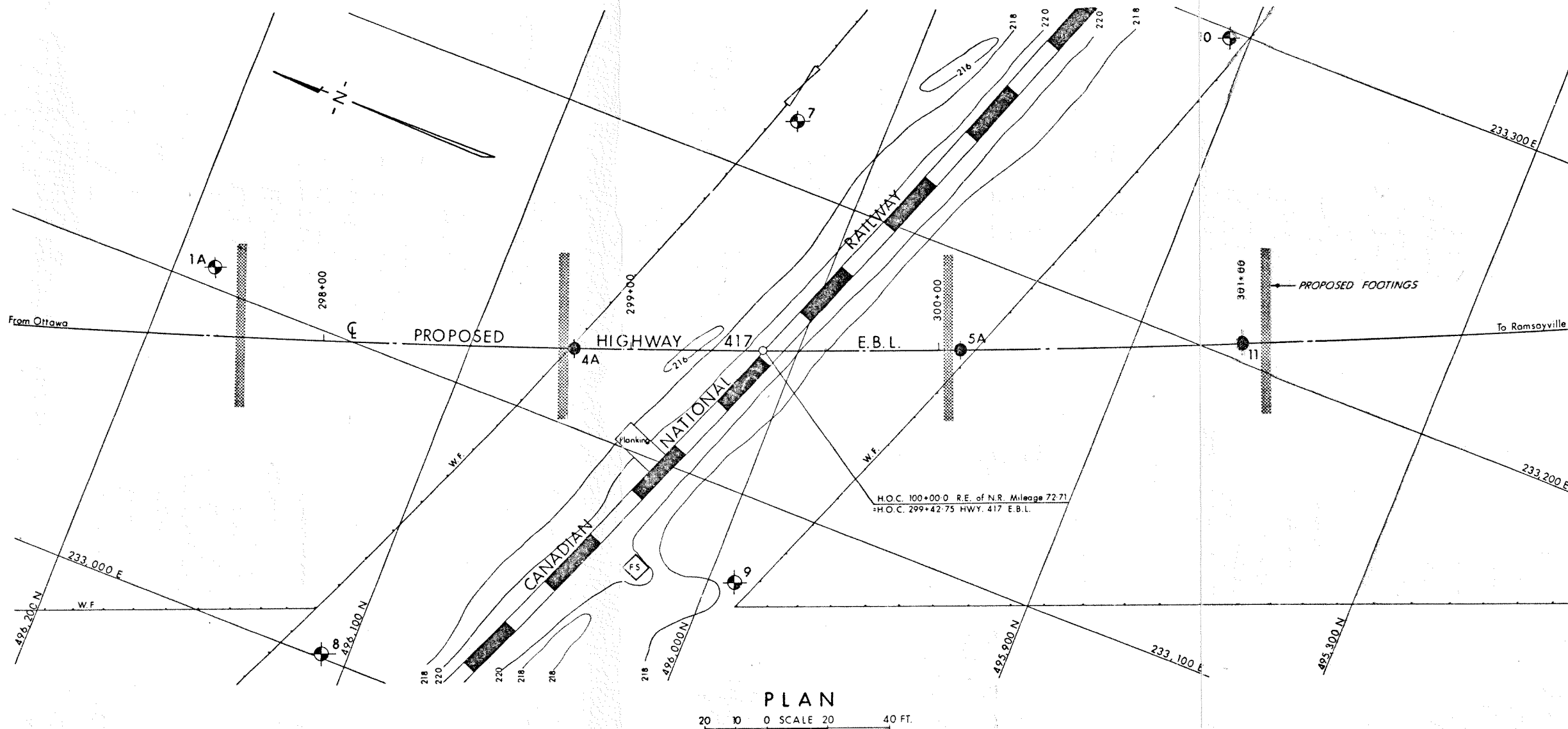
NOTE
The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence and may be subject to considerable error.



REVISIONS	DATE	BY	DESCRIPTION
JUNE 67	S.R.		BORE HOLES No. 2A & 3A WERE ADDED

MINISTRY OF TRANSPORTATION & COMMUNICATIONS DESIGN SERVICES BRANCH - FOUNDATIONS OFFICE			
CANADIAN NATIONAL RAILWAY (APPROX. 1.3 MILES N. OF RAMSAYVILLE)			
HIGHWAY NO. 417 W.B.L.	DIST. NO. 9		
CO. REG. MUN OTTAWA-CARLETON	TWP. GLOUCESTER		LOT 2 CON. 6
BORE HOLE LOCATIONS & SOIL STRATA			
SUBMD. S.A.	CHECKED <input checked="" type="checkbox"/>	WP NO. 10-69-04	DRAWING NO.
DRAWN S.O.	CHECKED <input checked="" type="checkbox"/>	JOB NO. 71-11124	71-11124 A
DATE 27 JAN 1972	SITE NO.		BRIDGE DRAWING NO.
APPROVED <i>[Signature]</i>	CONT. NO.		

REF NO. E-5224-1



SEE DRAWING NO. 71-11124A



KEY PLAN
SCALE IN MILES

LEGEND

- Bore Hole
- Cone Penetration Test
- Bore Hole & Cone Test
- Water Levels established at time of field investigation: NOV. & DEC. 71, APRIL 72, & JUNE 1972

NO.	ELEVATION	CO-ORDINATES	
		NORTH	EAST
7	216.6	496,028	233,220
8	216.7	496,108	233,001
9	216.7	495,992	233,072
10	216.0	495,907	233,297
11	216.1	495,866	233,206
1A	216.7	496,188	233,106
4A	216.6	496,069	233,124
5A	216.3	495,951	233,170

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
1	July 1972	G.P.	ADDED BORE HOLES NO. 4A & 5A ON PLAN & PROFILE
2	JUNE 67	S.R.	ADDED BORE HOLE NO. 1A

MINISTRY OF TRANSPORTATION & COMMUNICATIONS
DESIGN SERVICES BRANCH - FOUNDATIONS OFFICE

CANADIAN NATIONAL RAILWAY
(APPROX. 1.3 MILES N. OF RAMSAYVILLE)

HIGHWAY NO. 417 E.B.L. DIST. NO. 9
CO. REG. MUN. OTTAWA - CARLETON
TWP. GLOUCESTER LOT 2 CON. 6

BORE HOLE LOCATIONS & SOIL STRATA

SUBMD. S.A.	CHECKED	W.P. NO. 10-69-03	DRAWING NO. 71-11124 B
DRAWN S.O.	CHECKED	JOB NO. 71-11124	BRIDGE DRAWING NO.
DATE 28 JAN. 1972	SITE NO.	CONT. NO.	
APPROVED	PRINCIPAL FOUNDATION ENGINEER		

REF. NO. E-5225-1

DEPARTMENT OF TRANSPORTATION AND COMMUNICATIONS

MEMORANDUM

TO: Mr. T. C. Kingsland,
Regional Structural Planning Engineer,
Eastern Region,
Kingston, Ontario.

FROM: Foundations Office,
Design Services Branch,
West Bldg., Downsview.

ATTENTION:

DATE: August 24, 1972.

OUR FILE REF.

IN REPLY TO AUG 29 1972

SUBJECT: Proposed Structures at the Crossing of the C.N.R.
and Hwy. #417 (E.B.L. and W.B.L.), Regional
Municipality of Ottawa-Carleton, District No. 9
(Ottawa), W.O. 71-11124 -- W.P. 10-69-03
10-69-04

Foundation investigation for the above-mentioned structures was carried out by this Office during November 1971, based on the information provided by M. M. Dillons & Co. Ltd., Consulting Engineers, Ottawa. A detailed report W.O. 71-11124 was submitted by this Office with all the factual information, together with our recommendations pertaining to structure foundations and stability and settlement considerations of the approaches.

After the submission of our Report No. W.O. 71-11124 on March 7, 1972, this Office received two E-plans (No. E-5224-1 and E-5225-1), one for the E.B.L. structure and the other for the W.B.L. structure from the design consultants. These drawings indicate that either a single span structure or a three-span structure may be considered for E.B.L. and W.B.L. crossings with the C.N.R. tracks. As a result of this, additional borings (namely, B.H.'s No. 1A, 2A, 3A, 4A and 5A) were carried out in April and June 1972 to supplement the initial field data. The recent investigation revealed that the strength and compressibility characteristics of the subsoil were very similar to those encountered initially. Based on this information we conclude that the recommendations contained in our Report W.O. 71-11124 are still applicable. The attached Revised Drawings Nos. 71-11124A and B, and borehole log sheets, together with this memo should be included in our original Foundation Report.

Mr. T. C. Kingsland

- 2 -

August 24, 1972.

Should you require any additional information with regard to this project, please feel free to contact this Office.

M. Devata

M. Devata,

SUPERVISING FOUNDATIONS ENGINEER.

SAA/ao

cc: D. W. Farren
B. R. Davis
A. Rutka
S. J. Markiewicz
J. E. Callaghan
B. J. Giroux
E. R. Saint
G. A. Wrong
B. A. Singh
M. M. Dillon & Co. Ltd., Ottawa

Foundations Files
Documents

SUMMARY OF PILE DRIVING RECORDS

W.O. 71-11124 W.P. 10-69-04 CONT. 73-190 DIST. 9
SITE C.N.R. & Hwy # 417 (W.B.L. & E.B.L.)

DATE DRIVEN JUNE 28 - SEPT. 5, 1914 WEIGHT OF ANVIL 1500 lb

HAMMER TYPE DROP WEIGHT 3, 4 & 5 TON ENERGY DROP 5'

[illegible]