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G.I.-30 SEPT. 1976

GEOCRES No. 3145-78

DIST. 9 REGION EASTERN

W.P. No. 10-69-09

CONT. No. N.A. PROJECT CANCELLED.

W. O. No. 71-F-123

STR. SITE No. _____

HWY. No. 417

LOCATION HWY. 417 ; HUNT CLUB
RD.

=====

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT. _____

REMARKS: _____

71-F-123	10-69--09	HWY. 417 & HUNT CLUB RD.	3105-78
W.O.	W.P.	LOCATION	GEOCRES NO.

● DATA ON FILE IN SOIL MECHANICS SECTION

REFER TO: W.P. FILE

Project Cancelled

REMARKS

GEOCRES

INDEXING CARD FOR REPORTS NOT MICROFILMED

GI-20 AUG. 74

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DEPARTMENT OF TRANSPORTATION AND COMMUNICATIONS

MEMORANDUM

TO: Mr. A. G. Stermac,
Principal Foundation Engineer,
Downsview, Ontario.

71-1153
FROM: Bridge Section,
Kingston, Ontario.

ATTENTION:

DATE: October 5, 1971.

OUR FILE REF.

IN REPLY TO

SUBJECT: Highway 417 - From Baseline Road, Ramsayville Northerly
to Ottawa Queensway 5.1 Miles, District 9 - Ottawa

At a meeting held in Kingston yesterday the scheduling of the above projects was discussed and the following dates were tentatively arranged:

Engineering Surveys Request	Nov. 1/71
Foundation Investigation Request	Nov. 8/71
Property Request	Nov. 15/71
E-Plans	Dec. 15/71
Foundation Reports	Mar. 29/72
Issue Structures to Design	Apr. 19/72

Since the extensive preliminary foundation investigations in this area were carried out by your Section there would appear to be many advantages in having your Section continue with the final investigations. However, I shall be glad if you will confirm whether you will be able to undertake the investigations, bearing in mind the above dates.

Fourteen structures are involved at present as listed in the Program Justification Report issued September 17, 1971. It is possible that other structures will be added. You will note that no E-Plans will be available at first and the location of bore hole sites will have to be established from the run centre line and the 200' = 1" photo mapping which should be available by mid November.

I shall be glad to have your comments on the above as soon as possible.

T. C. Kingsland

T. C. Kingsland
Regional Bridge Planning Engineer

TCK/hl
c.c. - P. D. Billings
A. J. Percy
R. J. Forrest
C. S. Grebski

MEMORANDUM

TO: Mr. A. G. Stermac,
Principal Foundation Engineer,
Downsview, Ontario.

FROM: Bridge Section,
Kingston, Ontario.

ATTENTION: Mr. M. Devata

DATE: October 22, 1971.

OUR FILE REF.

IN REPLY TO

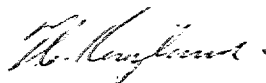
SUBJECT: W.P. 13-68 and W.P. 10-69,
Ramsayville Road to Ottawa Queensway,
Highway 417, District 9 - Ottawa

With reference to our recent discussions concerning the above project, I should be pleased if you will make arrangements for the foundation investigations to be carried out at the structure sites, the Foundation Report date for which is March 22, 1972. The structure sites to be investigated are as follows:

<u>W.P. No.</u>	<u>Location</u>
13-68-02	O.Q.W. Interchange Overpass - N.B.L.
13-68-03	-do- - S.B.L.
13-68-04	Cyrville Rd. Underpass (0.5 mi. S of OQW)
13-68-05	Innes Rd. Interchange Underpass (1.0 mi. South of OQW)
13-68-06	C.P.R. Overhead - S.B.L. (0.5 mi. South of Innes Rd.)
13-68-07	C.P. R. Overhead - N.B.L. -do-
10-69-02	Ridge Road Underpass (1.8 mi. South of Innes Rd.)
10-69-03	C.N.R. Overhead - S.B.L. (2.2 mi. South of Innes Rd.)
10-69-04	C.N.R. Overhead - N.B.L. -do-
10-69-05	Russell Road Underpass (2.8 mi. South of Innes Rd.)
10-69-06	Baseline Road Underpass (4.0 mi. South of Innes Rd.)
10-69-07	Ramsay Creek Bridge - W.B.L.
10-69-08	Walkley Rd. Interchange Underpass (1.5 mi. South of Innes Rd.)
71-11123 10-69-09	Hunt Club Rd. Interchange Underpass (3.1 mi. South of Innes Rd.)

As agreed at our recent meeting, the foundation investigations will be commenced on the basis of the 200' : 1" photo mosaic, two copies of which are enclosed for your use. Further information will be given to you as it becomes available.

I shall be glad to attend a field meeting as soon as it can be arranged with you and representatives of Regional Engineering Surveys Section to discuss the proposed methods of carrying out the investigations.



T. C. Kingsland
Regional Bridge Planning Engineer

TCK/hl
Encis.

c. c. -

M. M. Dillon & Company Ltd. (Attn. Mr. J. Kearney)
280 Metcalfe Street, Ottawa.

Mr. P. D. Billings
Mr. A. J. Percy
Mr. S. J. Markiewicz
Mr. E. R. Saint
Mr. L. Timson
Mr. A. G. Boucher
Mr. H. Aron
Mr. R. Forrest
Mr. C. S. Grebski

Gordon Carten, Q.C.

Structural Planning Office, Postal Bag 4000, Kingston, Ontario

March 8, 1972

M. M. Dillon Limited,
Consulting Engineers,
280 Metcalfe Street,
Ottawa 4, Ontario.

Attention: Mr. J. H. Kearney

Dear Mr. Kearney:

SUBJECT: W.P. 10-69-07, Ramsay Creek Bridge WBL, Site 3-308
W.P. 10-69-08, Walkley Road Int. U'Pass, Site 3-306 69-2-32
W.P. 10-69-09, Hunt Club Rd. Int. U'Pass, Site 3-307
Highway 417, District 9-Ottawa 71-11-123

Please find enclosed four copies each of Site Plans for the above structures as follows:

- E-5228-1, W.P. 10-69-08, Site 3-306, Walkley Rd. Int. U'Pass
- E-5229-1, W.P. 10-69-09, Site 3-307, Hunt Club Road Int. U'Pass
- E-5230-1, W.P. 10-69-07, Site 3-308, Ramsay Creek Bridge WBL

I shall be glad if you will put a preliminary grade on the profile and preliminary structure outlines showing probable foundation locations on each of these drawings and forward two copies each to Mr. M. Devata, Foundations Office, Department of Transportation and Communications, Central Building, Downsview 464, Ontario, to assist him in the preparation of the Foundation Reports for these structures. I would appreciate your sending me one of the marked-up copies for my records.

Yours truly,

T. C. Kingsland
Regional Structural Planning Engineer

TCK/hl
Encls. ✓

c.c. - M. Devata
C. S. Grebski

DEPARTMENT OF TRANSPORTATION AND COMMUNICATIONS

3165-78

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: March 20, 1972.

OUR FILE REF. IN REPLY TO APR 4 1972

SUBJECT:

3165-78
GEOCRES No.

FOUNDATION INVESTIGATION REPORT
For

Proposed Structure at the
Crossing of Hwy. #417 and Hunt Club Road
Regional Municipality of Ottawa-Carleton
District No. 9 (Ottawa)
W.O. 71-11123 -- W.P. 10-69-09

Cancelled Project
Site 3-57

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.

cc: Messrs. 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. Lillon
Foundations Files
Documents

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

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FOUNDATION INVESTIGATION REPORT
For
Proposed Structure at the
Crossing of Hwy. #417 and Hunt Club Road
Regional Municipality of Ottawa-Carleton
District No. 9 (Ottawa)
W.O. 71-11123 -- W.P. 10-69-09

1. INTRODUCTION:

The Foundation Office was requested to carry out a subsurface investigation at the crossing of proposed Hwy. #417 and the Hunt Club Road extension, 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 section to determine the subsoil, bedrock and groundwater conditions at the site.

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.

2. DESCRIPTION OF THE SITE AND GEOLOGY:

The area under investigation is located approximately 500 feet west of Russell Rd. and 300 feet south of Blake Rd., in the Township of Gloucester, Regional Municipality of Ottawa-Carleton. The terrain is flat to gently undulating in relief between about elevations 215 and 220. An abandoned railway track runs in a north-south direction approximately 400 feet west of the proposed site. A small creek meanders across the site generally from west to east. The land in this region is used exclusively for farming purposes.

The present physical features of the region are of a varied origin and are the results 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 the 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; the land gradually emerged from the sea. The area is now commonly referred to as 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 of the Collingwood and Gloucester formation.

3. FIELD AND LABORATORY WORK:

Eight sampled boreholes, four of which were accompanied by a dynamic cone penetration test, were put down at the site during the period of the field investigation, using conventional

.....3

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

**Chapman, L.V. and Putnam, D.F., "Physiography of Southern Ontario." University of Toronto Press, 1967.

diamond drill rigs adapted for soil sampling purposes. Two additional dynamic cone penetration tests were also conducted at the site.

Samples were obtained at required intervals in a 2-inch 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. This was supplemented by obtaining some 2-inch I.D. Shelby Tubes in the cohesive portions of the overburden. These tubes were manually pushed 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 3 of the boreholes by obtaining BX size rock core samples.

In order to establish the groundwater conditions in the area, one piezometer (Peaker Type) was installed within the overburden. This information was supplemented by recording the water levels in open holes at the remaining boring locations.

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 types encountered. The locations of all the borings, which were surveyed by the personnel from the Eastern Region Engineering Surveys Section, are shown on a Borehole Location Plan appended to this report.

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

- 4 -

The results of these tests are plotted on the Record of Borelog sheets and are summarized on Figures 1, 2 and 3, 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 soft to stiff silty clay to clay. The thickness of this cohesive stratum varies from 12.5 to 23 feet. Underlying this deposit is a 2.5 to 7 feet thick granular glacial till deposit. The glacial till is followed by shale bedrock.

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

4.2) Silty Clay to Clay:

Directly beneath a nominal cover of topsoil 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 12.5 to 23 feet. The material encountered is generally homogeneous with a few notable exceptions. At B.H.'s No. 1 and 8, for instance, a light grey and dark grey layering pattern is noticed. Occasional sea shells were encountered throughout the deposit. Grain-size distribution curves, for samples of the clay stratum, are shown on Figure No. 2 in Appendix I.

The engineering properties of the deposit, as determined by field and laboratory testing are presented in tabular form below:

TABLE I

Bulk Density (p.c.f.)	(γ)	99	-	125
Liquid Limit (%)	(W_L)	19	-	62
Plastic Limit (%)	(W_P)	12	-	29
Natural Moisture Content (%)	(W)	12	-	54

Standard Penetration Resistance ('N') (blows/ft.)	1 - 15
Initial Void Ratio (e_o)	0.832 - 1.546
Compression Index (C_c)	0.235 - 0.89
Degree of Reconsolidation Pressure ($P_c - P_b$) (t.s.f.)	1.09 - 3.05
Undrained Shear Strength (C_u) (p.s.f.)	
i) Field Vanes	400 - 1360
ii) Lab. Vanes	540 - 1955
iii) Lab. Testing	235 - 1525

The Atterberg Limit Tests, summarized above, are also plotted on the Plasticity Chart, Figure No. 1. These results indicate that, in general, the clay is inorganic and of high to intermediate plasticity.

Based on the Standard Penetration and the undrained shear strength tests carried out it is estimated that the consistency of the cohesive stratum varies from soft to stiff. There are some discrepancies in shear strengths between those obtained in the field and those in the laboratory. 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 seven laboratory oedometer tests, the results of which are shown on Void Ratio vs. Pressure Plots, on Figures No. 4 and 5. The results of this testing indicate that the clay is preconsolidated by about 2200 to 4500 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 sensitive silty clay to clay is a material of glacial origin consisting of a heterogeneous mixture of silt, sand and gravel, with a trace of clay. This granular glacial till deposit varies in thickness from 2.5 feet

(B.H. #5) to 7 feet (B.H. #1). Typical grain size distribution curves, for samples of the deposit obtained with 2" O.D. sampling equipment, are shown on Figure No. 3.

The Standard Penetration Tests gave 'N' values which ranged from 3 blows/ft. to 102 blows for 3 inches. Based on these values it is estimated that the relative density of 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 this zone has been subjected to some "reworking."

4.4) Shale Bedrock:

The granular glacial till deposit is directly underlain by bedrock, which was proven in 3 of the boreholes by obtaining up to 10 feet of BX size rock core samples. Over the site the bedrock surface was found to vary between elevations 192 to 198 which corresponds to depths below ground surface of from 20 to 26 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 samples were examined by Mr. K. Ingham, Geologist, Department of Transportation and Communications. Mr. Ingham presented the result of his bedrock examination in a memo to this Office, dated February 15, 1972, which is appended to this report.

5. GROUNDWATER CONDITIONS:

The groundwater level conditions across the site, during the period of the investigation (December 1971), were observed by taking readings in the open boreholes. In addition a piezometer (Peaker Type) was also installed in the overburden. The groundwater levels observed in the piezometer installation as well as the open boreholes are shown on the accompanying borelog sheets.

The observations indicated that the groundwater level in the upper cohesive stratum varies from 212 to 218,

which corresponds to ground surface at some locations to as much as 5 feet below existing ground surface at others.

Artesian groundwater pressure heads were observed at B.H.'s #3, 4, 5 & 7, once the borings penetrated into either the granular glacial stratum or the bedrock. Once this occurred the water rose instantaneously in the casing stabilizing itself between elevations 218 and 222, which corresponds to levels of from 1 foot to 3.5 feet above the existing ground surface. It is inferred that the lower portion of the glacial till and upper portion of the bedrock is acting as an aquifer which is being confined by the overlying relatively impervious cohesive stratum. This aquifer is probably being charged with groundwater from the surrounding higher terrain. The artesian flow was properly sealed, at the source, by conventional methods, following the completion of the boring operations.

6. DISCUSSIONS AND RECOMMENDATIONS:

6.1) General:

The design and construction of the rural portion of Hwy. #417, south-easterly of the Village of Ramsayville to the 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 remaining urban portion of Hwy. #417 in the Ottawa area. A preliminary Foundation Report, number W.O. 70-11115, for the various corridors of Hwy. #417, from Ramsayville northerly to the Ottawa Queensway (West of Blair Rd.), was submitted on March 23, 1971. The finalized alignment requires interchanges at Hunt Club Rd., the Walkley Rd. Extension, Innes Rd. and the Ottawa Queensway. In addition, structures will be required at the crossings of Baseline Rd., Canadian National and Canadian Pacific Railways, with the proposed Hwy. #417.

This report deals with the proposed underpass structure at the junction of Hunt Club Rd. and Hwy. #417.

Discussions with regard to other structures on this portion of the Freeway will be presented in separate foundation reports.

At the time of writing this report only preliminary information was available with regard to the specific location of the structure elements. In view of this we are not, at this time, preparing a detailed Foundation Drawing. This will be submitted when decisions regarding footing locations have been finalized. Included in the Foundation Report, however, is a drawing showing the borehole locations.

It is proposed to construct a two-span underpass structure at the crossing of the Hunt Club Rd. revision and Hwy. #417, in the Township of Gloucester, Regional Municipality of Ottawa-Carleton.

The profile grade of Hunt Club Rd., in the vicinity of the structure, will be between elevations 245 and 248. Correspondingly, the proposed profile grade of Hwy. #417, in this area, will be at elevation 226. The associated fills, therefore, will be about 22 and 30 feet in height in the longitudinal and transverse directions, respectively.

The predominant stratum across the site is composed of a 12.5 to 23 feet thick soft to stiff silty clay to clay. Underlying this cohesive deposit is a 2.5 to 7 feet thick granular glacial till, which in turn, is followed by shale bedrock.

6.2) Approach Fills:

6.2.1) Stability Considerations:

The critical condition for stability of an embankment on normally or slightly overconsolidated cohesive soils, as is the case at this site, generally occurs during or immediately after construction. This being the case, a total stress stability 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 characteristics of the foundation and embankment soils.

Analyses have been carried out, therefore, in terms of total stresses making use of the electronic computer, to determine the stability of the approaches.

The following assumptions were made:

Elevation	Soil	<u>SOIL PROPERTIES</u>		
		Bulk Density (p.c.f.)	Parameters Undrained Shear Strength (Cu. p.s.f.)	Effective Angle of Internal Friction (ϕ)
248 - 218	Embankment Fill (Slopes 2:1)	125	--	30°
218 - 214	Silty Clay	110	2,000	--
214 - 210	Silty Clay	100	875	--
210 - 195	Silty Clay	100	600	--
195 - 192	Glacial Till	125	--	40°

- Notes: 1) Approximate Groundwater Level - Elevation 216.
2) Tension Crack - 1/3 Height of Fill.

The results of the computations indicate the following:

i) Longitudinal or Forward Direction

Approaches of the clear height contemplated (21 to 22 feet maximum) will be stable ($FS \geq 1.3$), provided the fill is properly compacted and standard 2:1 slopes are employed, and

ii) Transverse Direction

Fills up to 30 feet in height will be placed. 23 feet high fills will be stable, provided 2:1 slopes are employed. If 2:1 slopes are maintained, mid-height berms will be required to stabilize the higher portion of the fill sections. The berm lengths recommended for various

heights are presented in tabular form.

<u>Height of Fill</u>	<u>Length of Mid-Height Berm Required</u>
23 feet	Nil
25 feet	15 feet
28 feet	30 feet
30 feet	40 feet

The berms should slope towards the toe of the fill at a grade of 20:1. Further, a smooth transition should be affected between the berm requirements for the various heights of fill.

6.2.2) Settlement Considerations:

The underlying compressible clay stratum will settle, over a long term period, due to the loading of the approach fills. The estimated consolidation settlements, due to embankment loading, are summarized in tabular form.

Consolidation Settlement Beneath Centre-Line of Approach
Fills (Max. Height 30 feet - In Place Bulk Unit Weight
125 p.c.f.)

<u>Time</u>	<u>Consolidation Settlement</u>
2 to 2½ years	5 to 6 inches
12 years	10 to 11 inches (max.)

Referring to the table it can be seen that a considerable percentage of the consolidation settlement will take place in the first few years after placement of the fills. In order to minimize post-construction maintenance costs, consideration should be given, if scheduling permits, to constructing the fills at least 18 to 24 months prior to the structure foundations. Further, final paving operations of the roadway should be delayed for as long a period as possible.

6.3) Structure Foundations:

The strength and compressibility characteristics of the clay deposit are such that it will be necessary to

support the structure abutments and pier foundations on end-bearing piles.

6.3.1) Pier (Refer to B.H.'s #3, 4 and 9):

The pier 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. 14BP74 steel H-pile may be designed for 95 tons/pile).

At least 4 feet of earth cover should be provided 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 field investigation. The excavations will be carried out in the relatively impervious cohesive stratum. Therefore, no major dewatering problems are anticipated. Any minor seepage or surface runoff into the excavation could be readily handled by using standard techniques, such as pumping from sumps.

As an alternative the pier columns can be founded directly on or within sound shale bedrock utilizing bored-in concrete caissons. In order to provide adequate lateral stability these caissons should be 'keyed' in to a minimum depth of 2 feet into shale bedrock. The allowable loads would depend on the size of caisson chosen. For example, a 30-inch diameter caisson may be designed for load of 250 tons.

6.3.2) Abutments (Refer to B.H.'s #1, 5, 7 and 8):

The abutments can be "perched" within the approach fills and supported on end-bearing piles driven to the sound shale bedrock surface. 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 strains imposed by the embankment loading, will tend to displace the piles laterally. In view of this, it would be desirable to reduce the design load by about 15% with respect to the maximum allowable load of the pile section chosen. For example 14BP74 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 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 for this project was carried out during the period of December 2 to 22, 1971, under the supervision of Mr. S. A. Ahmad, Project Foundation Engineer.

The drilling equipment was owned and operated by the F. E. Johnston Drilling Co. Ltd., Ottawa.

This report was written by Mr. S. A. Ahmad and reviewed by Mr. M. Devata, Supervising Foundation Engineer.

Shakeen Ahmad
S. A. Ahmad, P. Eng.

M. Devata
M. Devata, P. Eng.



SAA/ao
March 15, 1972.

APPENDIX I

DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 1

FOUNDATION SECTION

JOE 72-11123

LOCATION Co-ords. 493,428 N; 235,574 E.

ORIGINATED BY SA

W.P. 20-60-09

BORING DATE Nov. 9 & 10, 1971

COMPILED BY SO

DATUM Geodetic

BOREHOLE TYPE NX Washboring

CHECKED BY

SOIL PROFILE		SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION		RESISTANCE		LIQUID LIMIT		BULK DENSITY	REMARKS	
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE		BLOWS/FOOT	20	40	60	80	100			W _L
217.1	Ground Level					SHEAR STRENGTH P.S.F.					WATER CONTENT %			
0.0	Silty clay to clay		1	SS	8									
	Firm to Stiff		2	TW	14		210							
	Grey		3	TW	14									
201.9			4	SS	15									
12.5	Bot. mix. of silt, sand & gravel, trace of clay		5	SS	22	200								
	occ. seams of silty clay.		6	SS	40									
197.9			7	RX	100%									
19.5	Shale Bedrock		8	BY	100%	190								
	Sound													
	Grey													
187.9														
29.5	End of Borehole													

FOUNDATION SECTION

CHECKED BY 

[illegible]

FOUNDATION SECTION

CHECKED BY *AK*

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 ∇ Head
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT		SHEAR STRENGTH P.S.F.	WATER CONTENT % $w_p \quad w \quad w_L$		
							○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB. VANE 400 800 1200 1600 2000	15 30 45		
218.3	Ground Level									
0.0	Silty clay to clay trace of sand Soft to Stiff Grey		1	SS	10				114	
			2	TW	PM					
			3	TW	PM					
			4	TW	PM					
			5	TW	PM					
202.3			6	TW	PM					
16.0	Het. mix. of silt, sand & gravel, trace of clay.		7	SS	9				105	1
196.8	Glacial Till.		8	SS	22				106	15
21.5	Shale Bedrock		9	SS	50%					43
191.1	Sound Grey		10	BX	63%					Encountered
27.2	End of Borehole		11	BX	89%					

DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 4

FOUNDATION SECTION

JOB 71-11123

LOCATION Co-ords. 493,525 N; 235,673 E.

ORIGINATED BY MD

W.P. 10-69-09

BORING DATE Dec. 7, 1971

COMPILED BY SO

DATUM Geodetic

BOREHOLE TYPE NX Washboring

CHECKED 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							w_p — w — w_L				
							SHEAR STRENGTH P.S.F.					WATER CONTENT %				
216.8	Ground Level															
0.0	Silty clay to clay		1	SS	15	210									132	Head GR SA, SI, CL
	Firm to Stiff		2	TW	PM											
	Grey		3	TW	PM											
			4	TW	PM											
			5	SS	1											
12.5	Het. mix. of silt, sand & gravel, trace of clay.		6	SS	3	200									26 34 29 11 34 50 12 4	Encountered
189.3			7	SS	2											
17.5	Shale Bedrock		8	SS	13											
195.1	Sound		10	BX	55%	190										
21.7	End of Borehole															

FOUNDATION SECTION

CHECKED BY *[Signature]*

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE					LIQUID LIMIT ——— w_L			BULK DENSITY γ P.C.F.	REMARKS	
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT		BLOWS / FOOT					PLASTIC LIMIT ——— w_p					WATER CONTENT ——— w
							20	40	60	80	100	SHEAR STRENGTH P.S.F.					
							○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB. VANE					WATER CONTENT % w_p ——— w ——— w_L 15 30 45					
217.6	Ground Level						400	800	1200	1600	2000						
0.0	Silty clay to clay, trace of sand.		1	SS	9												
			2	TW	PM											111	
			3	TW	PM											109	0 4 50 46
			4	TW	PM												
			5	TW	PM											99	
			6	SS	1												
			7	SS	1												
194.6					8		SS	31									
23.0	Het. mix. of silt, sand & gravel, trace of clay.														33 2 21 1		
25.5	End of Borehole																
27.2	End of Cone Test					190											

DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 6

FOUNDATION SECTION

JOB 71-11123

LOCATION Co-ords. 493,592 N; 235,803 E.

ORIGINATED BY MD

W.P. 10-69-09

BORING DATE Dec. 6, 1971

COMPILED BY SO

DATUM Geodetic

BOREHOLE TYPE Cone Test Only

CHECKED BY *[Signature]*

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		20	40	60	80	100			w _p w w _L
							SHEAR STRENGTH P.S.F. ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB. VANE							WATER CONTENT %
216.8	Ground Level													
196.3						210								
20.5	End of Cone Test					200								
						190								

DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 7

FOUNDATION SECTION

JOB 71-11123

LOCATION Co-ords. 493,578 N; 235,885 E.

ORIGINATED BY M.D.

W.P. 10-69-09

BORING DATE Dec. 7, 1971

COMPILED BY SO

DATUM Geddetic

BOREHOLE TYPE NX Washboring

CHECKED BY

J.R.

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		20	40	60	80	100	w_p	w	w_L			
							SHEAR STRENGTH P.S.F.					WATER CONTENT %					
						○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB. VANE											
						400 800 1200 1600 2000					15 30 45						
217.6	Ground Level																
0.0	Silty clay to clay		1	SS	12	210											
	Soft to Stiff		2	TW	PM												
	Grey		3	TW	PM												
			4	TW	PM												
			5	TW	PM												
			6	TW	PM												
			7	TW	PM												
196.6		8	SS	11	200												
21.0	Het. mix. of silt, sand & gravel	9	SS	19													
193.1																	
24.5	End of Borehole					190											

Head
GR 5A, SI, CL111
101 0 7 43 50

98

15 12 36 7
Encountered

DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 8

FOUNDATION SECTION

JOB 71-11123

LOCATION Co-ords. 493,375 N; 235,588 E.

ORIGINATED BY JS

W.P. 10-69-09

BORING DATE Dec. 22, 1971

COMPILED BY SO

DATUM Geodetic

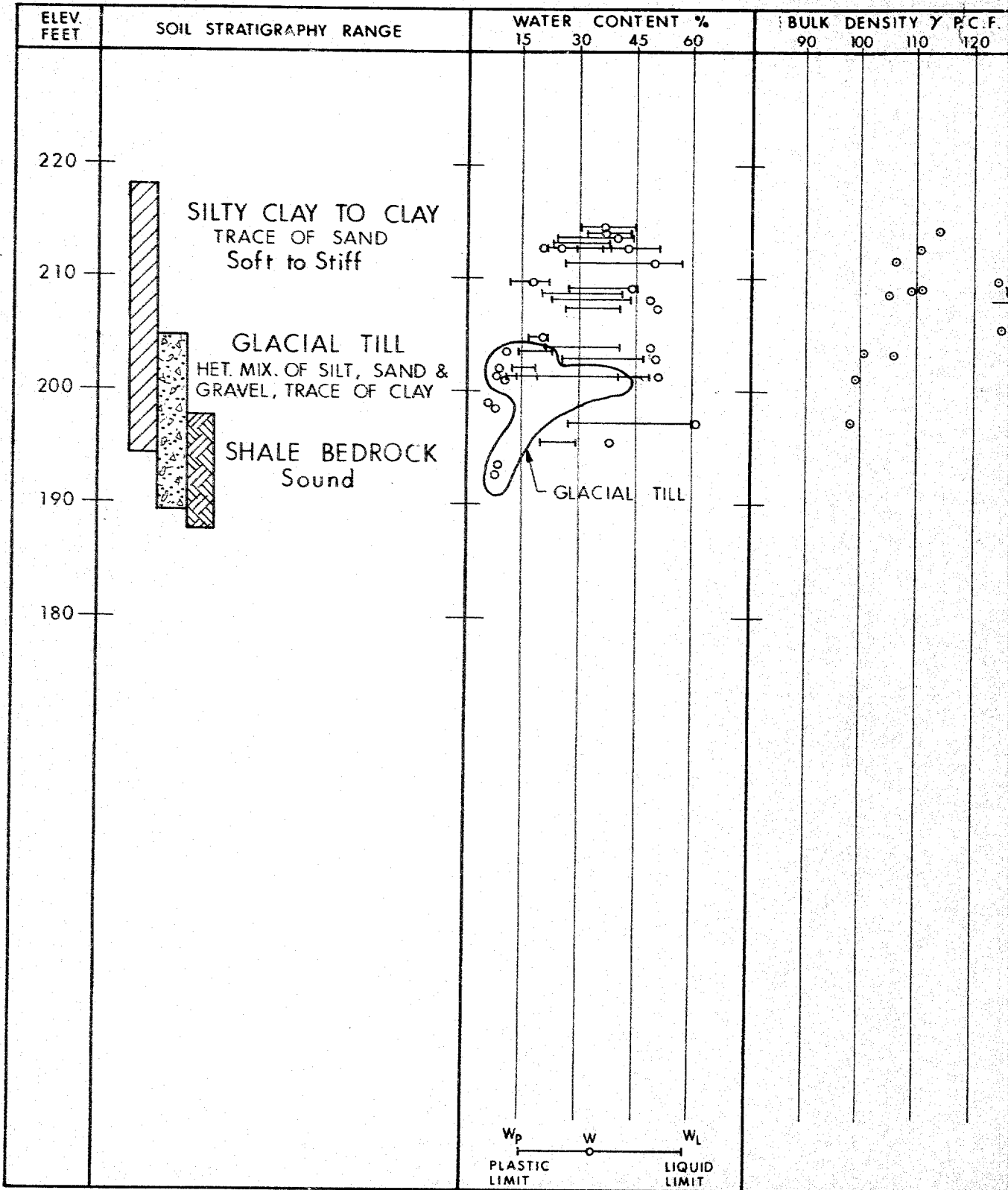
BOREHOLE TYPE NX Washboring

CHECKED BY *SK*

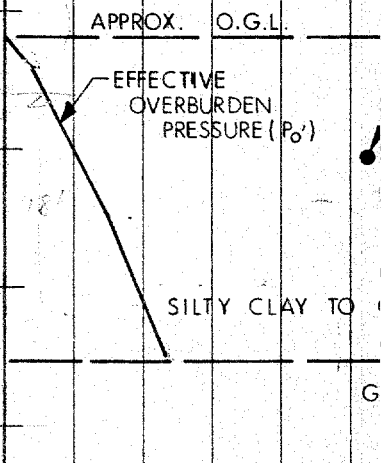
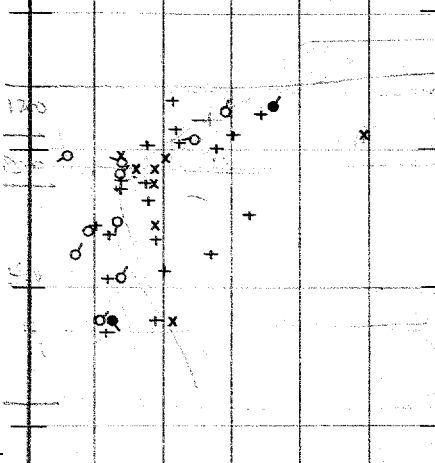
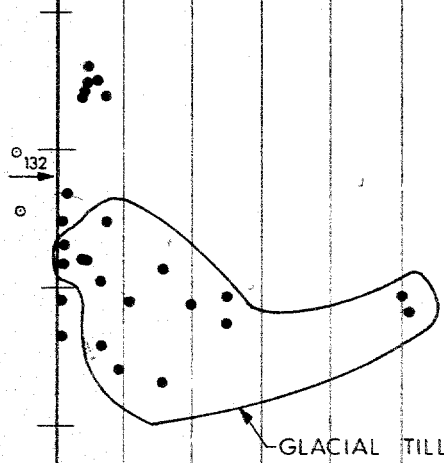
SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT					LIQUID LIMIT — w_L PLASTIC LIMIT — w_p WATER CONTENT — w			BULK DENSITY γ P.C.F.	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT		20	40	60	80	100	w_p	w	w_L		
217.9	Ground Level															
0.0	Silty clay to clay Firm to Stiff Grey		1		8											
			2	TW	PM					x					106.5	
			3	TW	PM											
204.4			4	TW	PM										125	
13.5	Het. mix. of silt, sand & gravel, trace of clay		5	SS	8											
			6	SS	100/4"											
198.1			7	SS	102/4"											
19.8	End of Borehole															

23 38 39 9
Piez. El. 201.1
17 40 33 10

[illegible]



BULK DENSITY γ P.C.F.					STAND. PEN. RESIST. 'N' BLOWS/FT.					SHEAR STRENGTH P.S.F.					EFFECTIVE OVERBURDEN PRESSURE (P _{o'})	
90	100	110	120	20	40	60	80	100	400	800	1200	1600	2000	1000	2000	

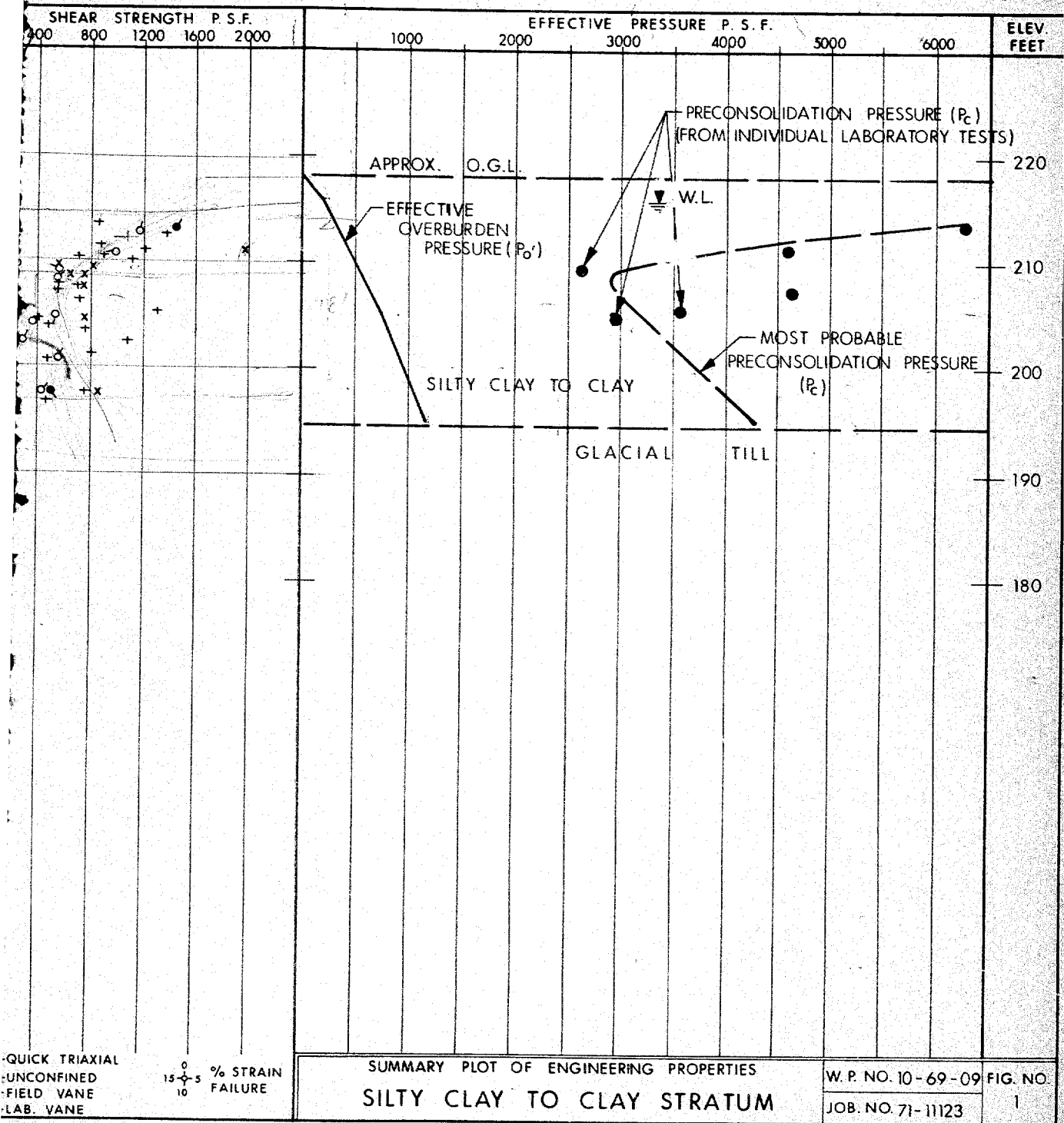


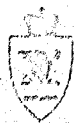
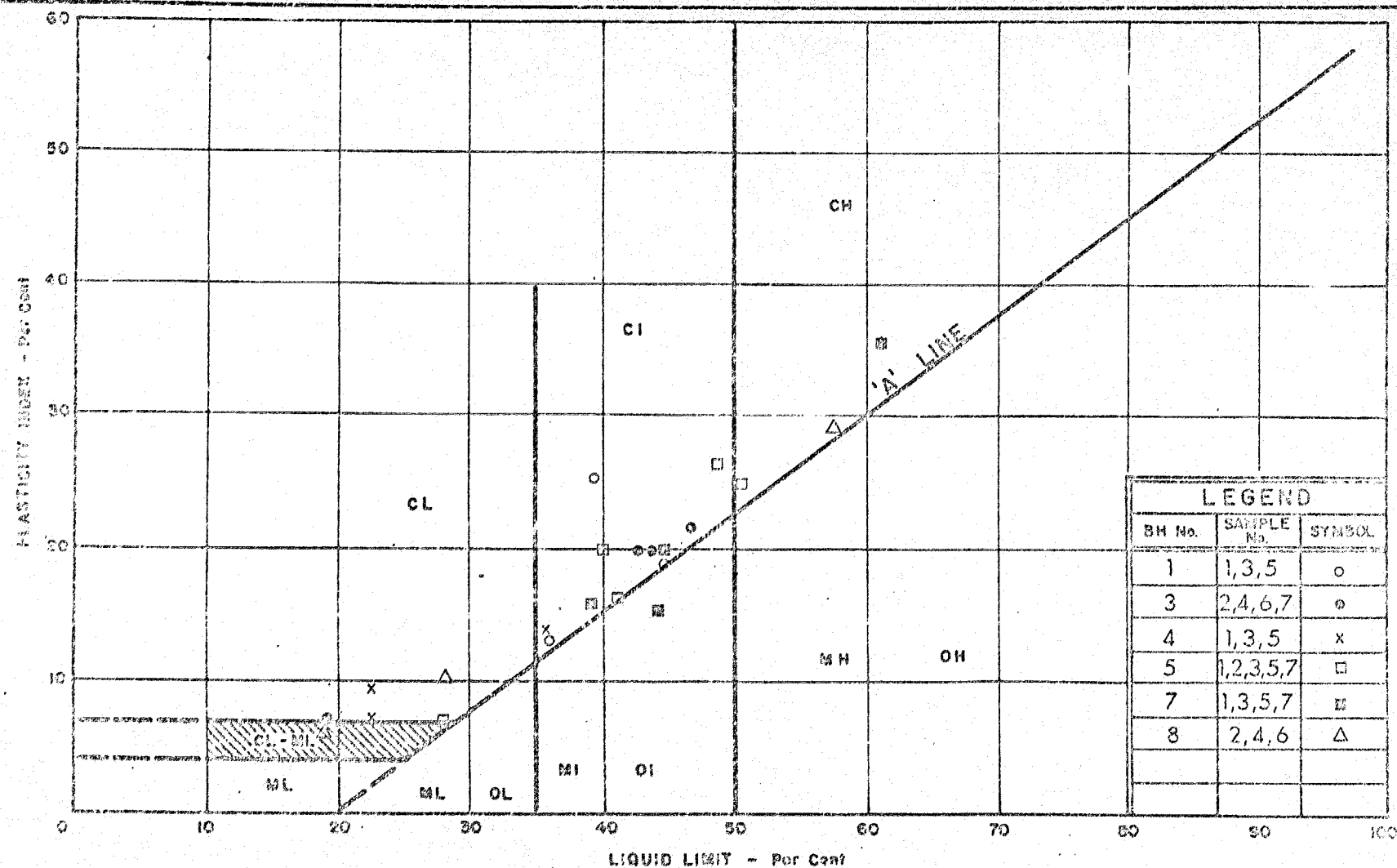
●-QUICK TRIAXIAL
 ○-UNCONFINED
 +-FIELD VANE
 *-LAB. VANE

0
 15 5 % STRAIN
 10 FAILURE

SUMMARY PLOT OF ENG

SILTY CLAY TO





DEPARTMENT OF HIGHWAYS
MATERIALS and
TESTING
DIVISION

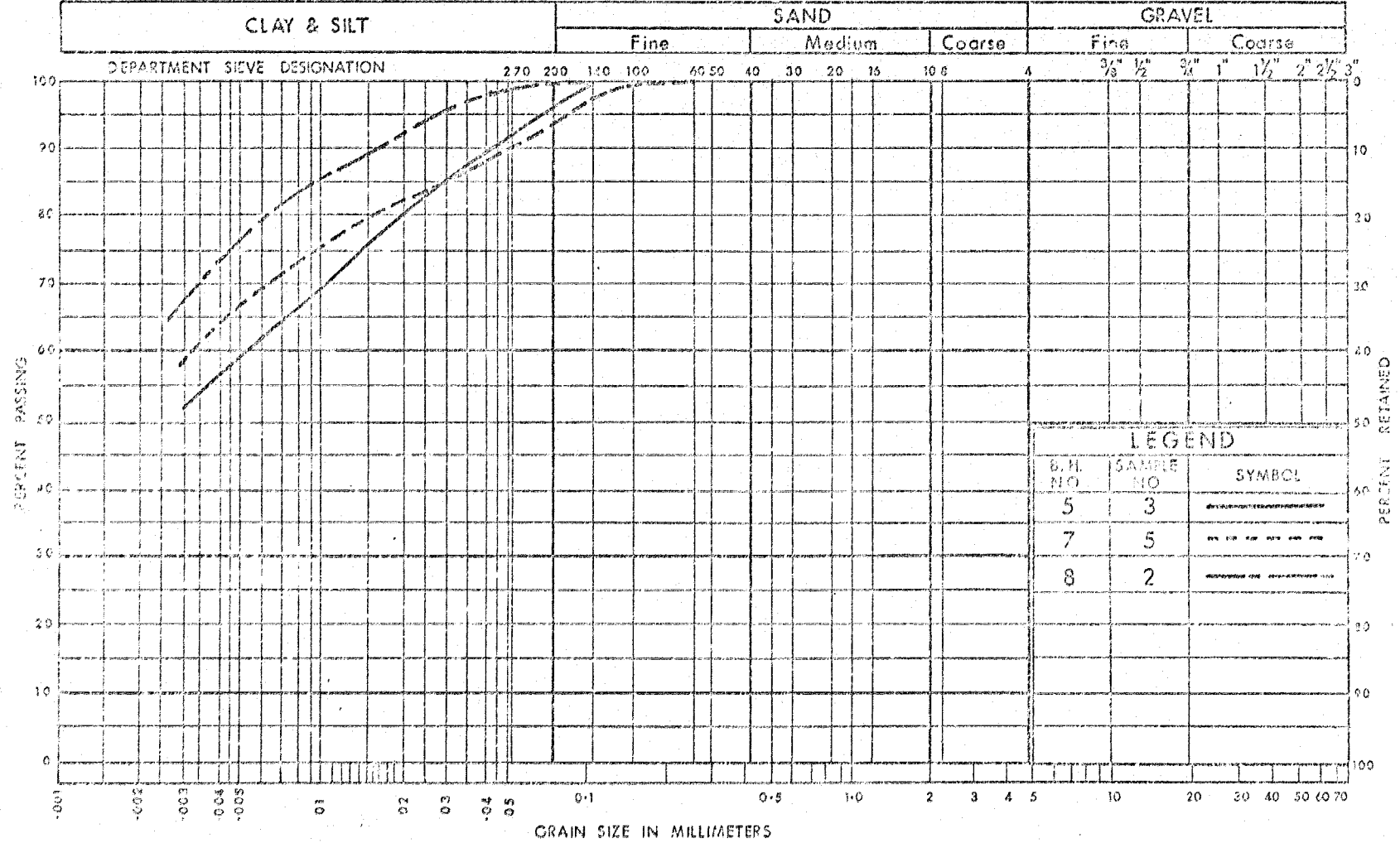
PLASTICITY CHART

WRP No. 10 - 69 - 09

JOS No. 71 - 11123

FIG. 2

UNIFIED SOIL CLASSIFICATION SYSTEM



DEPARTMENT
OF
TRANSPORTATION AND COMMUNICATIONS

DESIGN SERVICES
BRANCH

GRAIN SIZE DISTRIBUTION

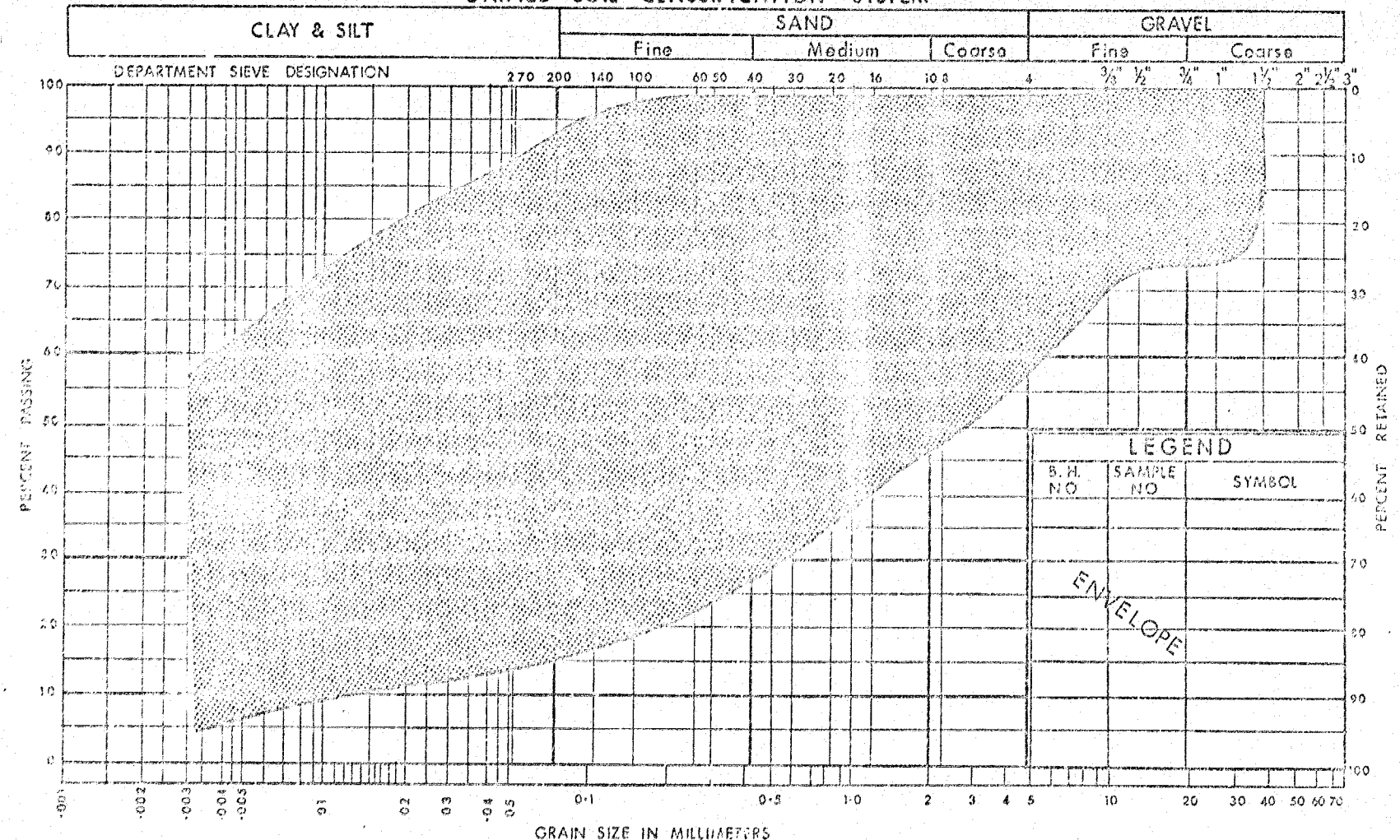
CLAY

W.R. No. 10 - 69 - 09

JOB No. 71 - 11123

FIG. 3

UNIFIED SOIL CLASSIFICATION SYSTEM



173
CHICAGO

DESIGN SERVICES
BRANCH

GRAIN SIZE DISTRIBUTION

GLACIAL TILL

W.P. No. 10 - 69 - 09

JOB No. 71-11123

FIG. 4

VOID RATIO - PRESSURE CURVES

JOB NO. 71 - 11123

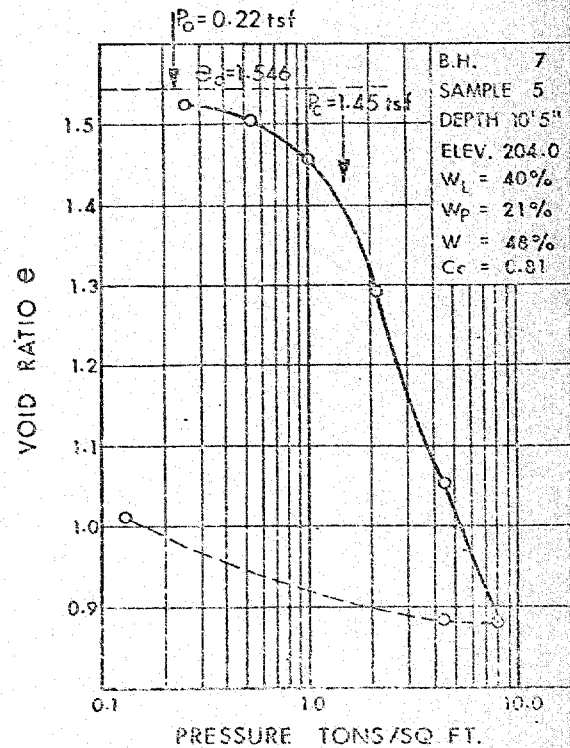
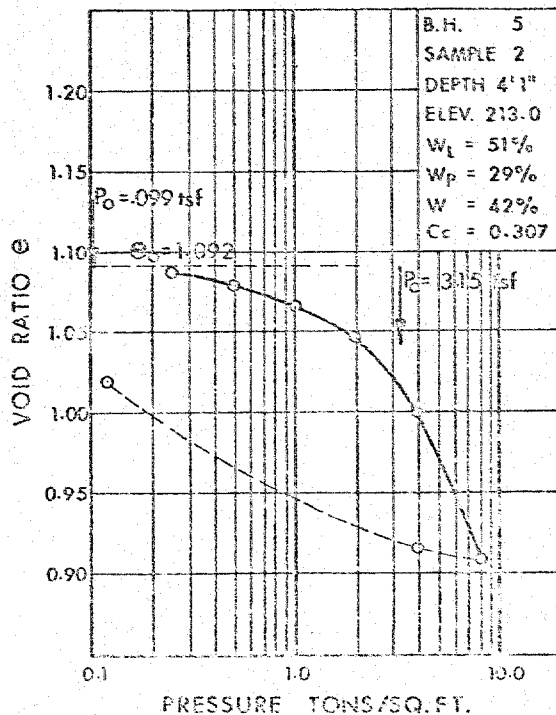
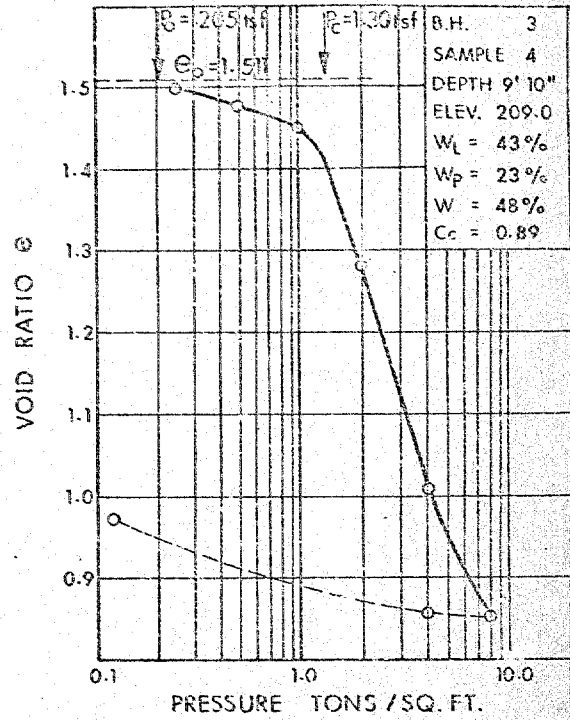
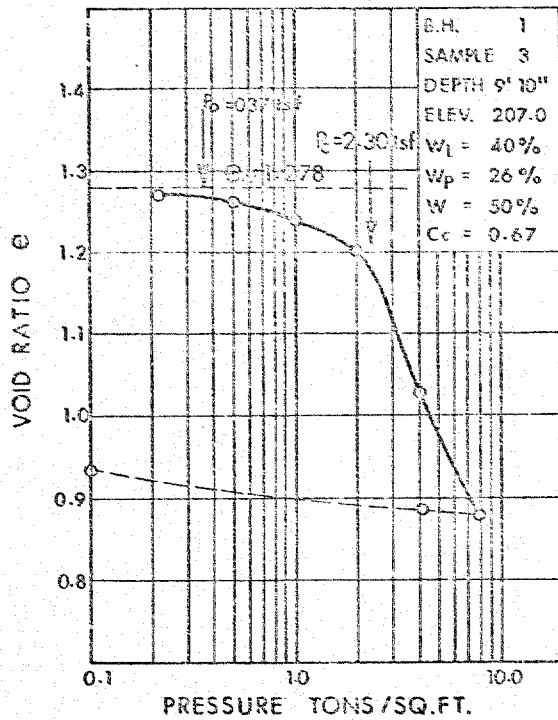


FIG. 5

VOID RATIO -PRESSURE CURVES

JOB NO. 71 - 11123

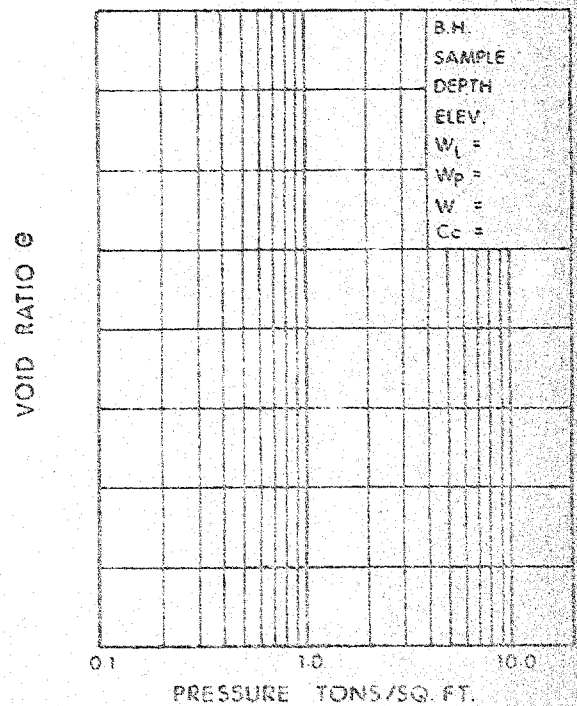
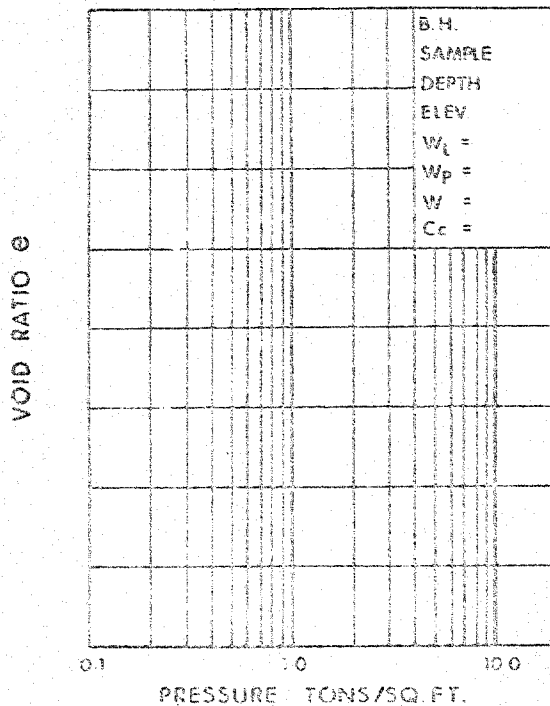
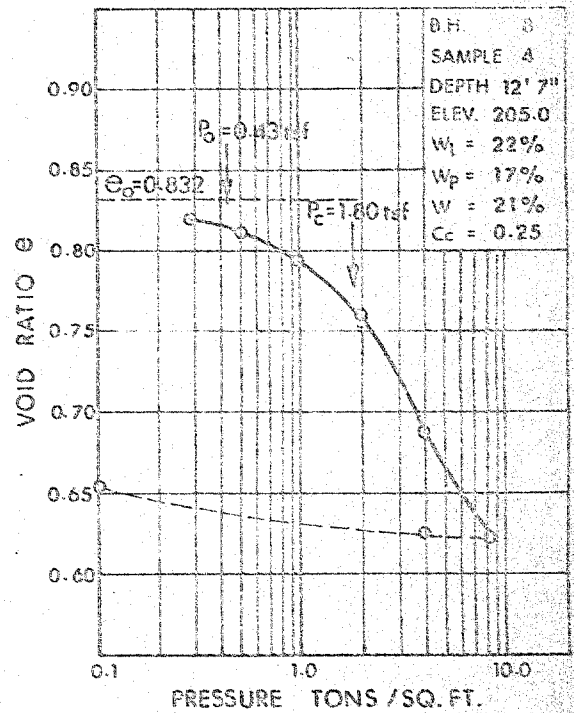
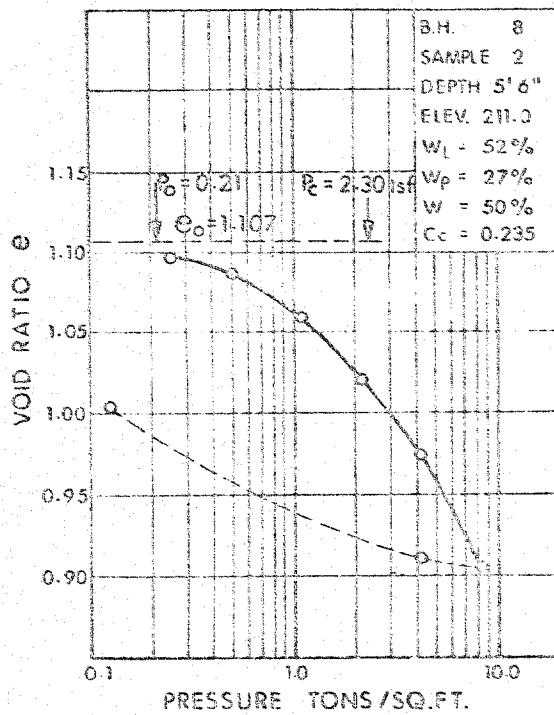


FIG. 5

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
$\log_e \sigma$ 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
$\bar{\sigma}$	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

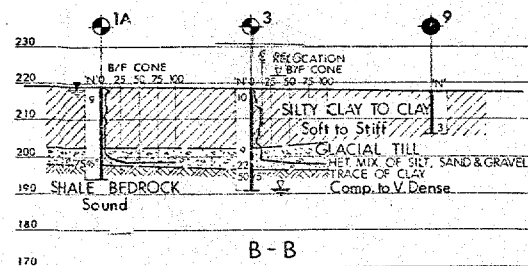
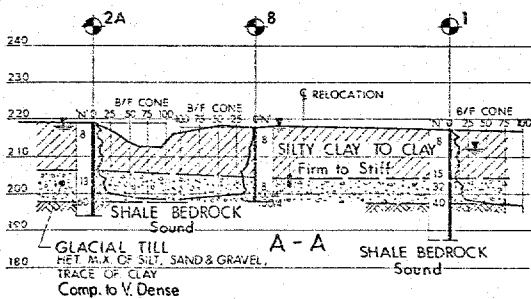
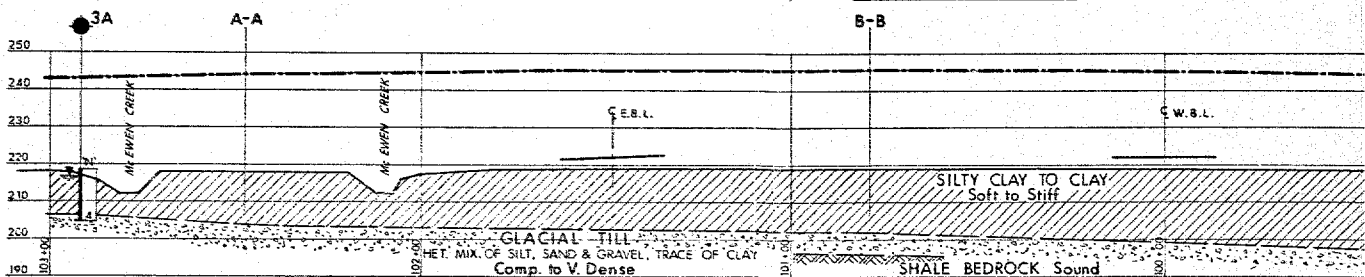
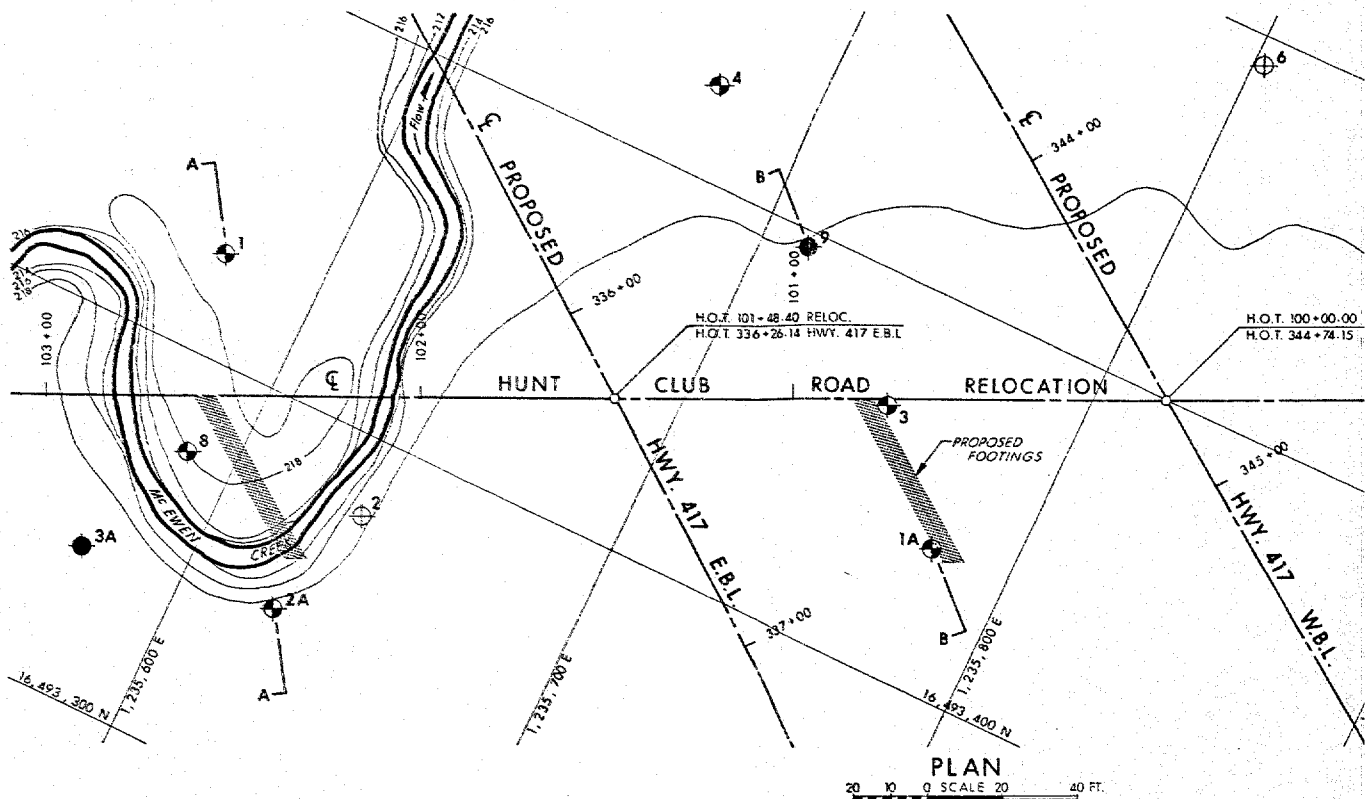
d	DISTANCE FROM TOP OF WALL TO POINT OF APPLICATION OF PRESSURE
δ	ANGLE OF WALL FRICTION
K	DIMENSIONLESS COEFFICIENT TO BE USED WITH VARIOUS SUFFIXES IN EXPRESSIONS REFERRING TO NORMAL STRESS ON WALLS
K_0	COEFFICIENT OF EARTH PRESSURE AT REST

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_c	MODULUS OF SUBGRADE REACTION

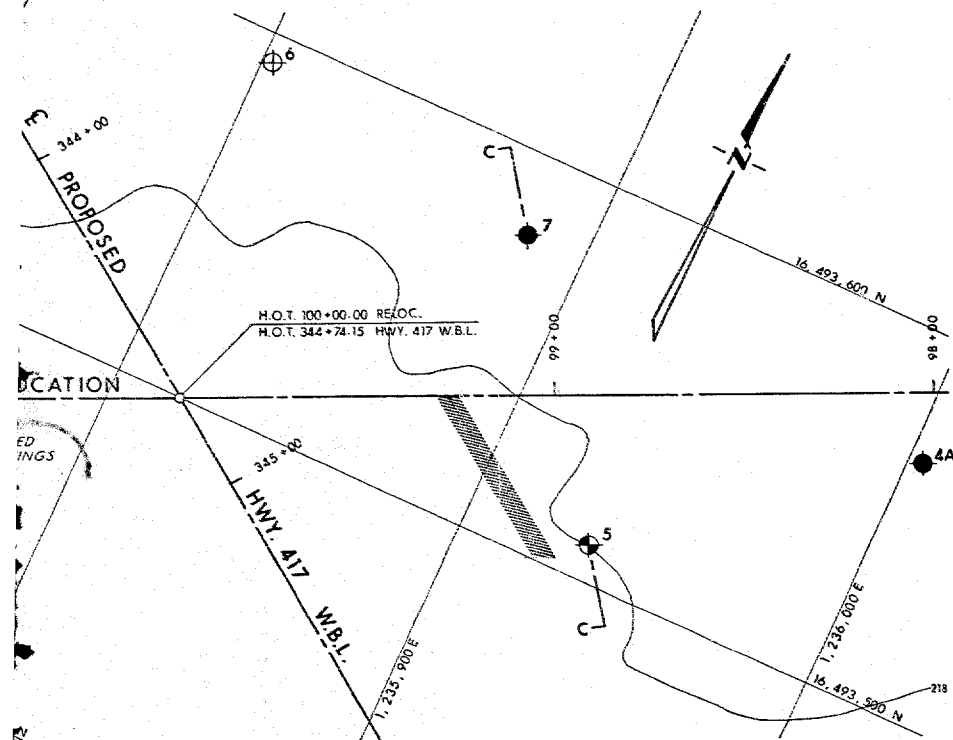
SLOPES

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

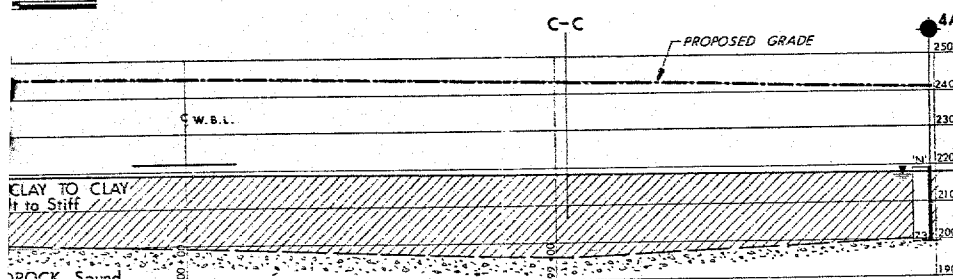


PROFILE & SECTIONS

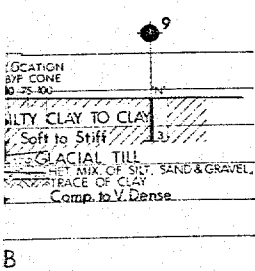




20 40 FT.

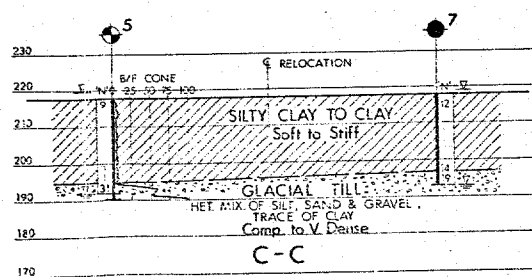


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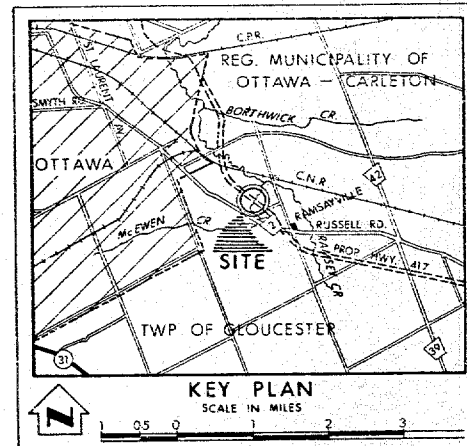


SECTIONS

20 40 FT.



REF. No: E-5229-1



LEGEND

- Bore Hole
- ⊕ Cone Penetration Test
- ⊕ Bore Hole & Cone Test
- Water Levels established at time of field investigation. Nov., Dec. 1971 & April 1972.
- Piezometer
- Head
- Artesian Water Levels
- Encountered

NO.	ELEVATION	CO - ORDINATES	
		NORTH	EAST
1	217.4	16,493,428	1,235,574
1A	219.1	16,493,437	1,235,779
2	216.8	16,493,380	1,235,637
2A	219.0	16,493,347	1,235,626
3	218.3	16,493,467	1,235,752
3A	218.5	16,493,340	1,235,572
4	216.8	16,493,525	1,235,673
4A	219.0	16,493,566	1,236,004
5	217.6	16,493,510	1,235,934
6	216.8	16,493,592	1,235,803
7	217.6	16,493,578	1,235,885
8	217.9	16,493,375	1,235,588
9	217.5	16,493,496	1,235,714

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

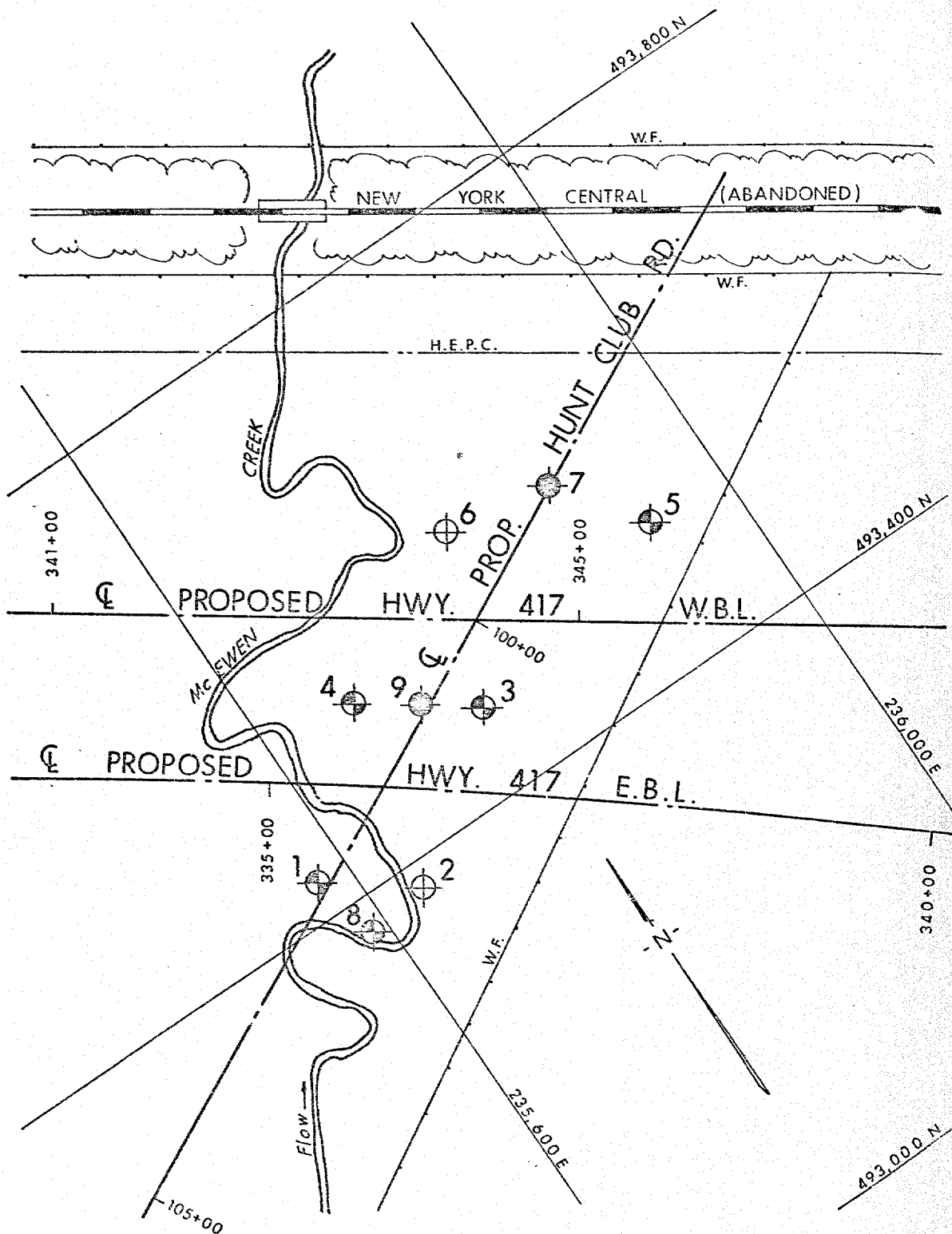
MINISTRY OF TRANSPORTATION & COMMUNICATIONS
DESIGN SERVICES BRANCH — FOUNDATION OFFICE

HUNT CLUB ROAD RELOCATION

HIGHWAY NO. 417 DIST. NO. 9
REG. MUNICIPALITY OF OTTAWA - CARLETON
TWP. GLOUCESTER LOT CON.

BORE HOLE LOCATIONS & SOIL STRATA

SUBMD. S.A.	CHECKED R.H.	W.P. NO. 10-69-09	DRAWING NO.
DRAWN S.R.	CHECKED	JOB NO. 71-11123	71-11123A
DATE MAY 23, 1972	SITE NO.	BRIDGE DRAWING NO.	
APPROVED	CONT. NO.		



BOREHOLE LOCATION PLAN

SCALE 1" = 100'

W.P. NO 10-69-09

JOB NO 71-11123

DEPARTMENT OF TRANSPORTATION AND COMMUNICATIONS

MEMORANDUM

TO: Mr. T. C. Kingsland, FROM: Foundations Office,
Regional Structural Planning Eng., Design Services Branch,
Eastern Region, Kingston. Central Bldg., Downsview.

ATTENTION: DATE: June 2, 1972.

OUR FILE REF. IN REPLY TO

SUBJECT: Proposed Structure at the Crossing of Hwy. #417 and
Hunt Club Rd., Regional Municipality of Ottawa-
Carleton, District No. 9 (Ottawa)
W.O. 71-11123 -- W.P. 10-69-09

Foundation investigation for the above-mentioned structure was carried out by this Office during November 1971, based on the information provided by M. M. Dillon & Co. Ltd., Consulting Engineers, Ottawa. A detailed report W.O. 71-11123 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.

Since the submission of the report changes in the alignment of the Hunt Club Rd. were made by the design consultants. Subsequently, this Office received an E-plan showing the revised structure footing details. As a result of this, the Foundations Office carried out additional borings during April 1972, to supplement the original field data.

The recent investigation consists of four boreholes, namely, 1A, 2A, 3A and 4A. These borings revealed that the subsoil conditions as well as strength and compressibility characteristics of the cohesive stratum, were similar to those borings carried out initially. A drawing (71-11123A) showing all the borings carried out at this site is included with this memo together with the Record of Borehole Sheets of the additional boreholes.

We conclude from our recent information that the recommendations contained in our Foundation Report W.O. 71-11123 are still applicable. This memo, together with log sheets and drawing should be included in corresponding Foundation Report.

Mr. T. C. Kingsland

- 2 -

June 2, 1972.

Should you require any further information with regard to this project, please contact this Office.

M. Devata

M. Devata,
SUPERVISING FOUNDATIONS ENGINEER.

ND/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

DEPARTMENT OF TRANSPORTATION AND COMMUNICATIONS

DESIGN SERVICES BRANCH

RECORD OF BOREHOLE No. 1A

FOUNDATION SECTION

JOB 71-11123

LOCATION Co-ords. 16,493,437 N. 1,235,779 E.

ORIGINATED BY S.A.A.

W.P. 10-69-09

BORING DATE April 26, 1972

COMPILED BY T.B.

DATUM Geodetic

BOREHOLE TYPE Auger & BX 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	REMARKS			
ELEV. DEPTH	DESCRIPTION	STRAT. PLT	NUMBER	TYPE	BLOWS / FOOT		20	40	60	80	100	SHEAR STRENGTH P.S.F. ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB. VANE 400 800 1200 1600 2000					W_P	W	W_L
219.1	Ground elevation.																		
0.0	Silty clay to clay. Firm to stiff.		1	SS	9														
			2	TW	PH														
			3	TW	PH														
			4	TW	PH														
202.6			5	TW	PH														
16.5	Het. Mix. of silt, sa. & gravel. Trace of clay.		6	SS	75/1														
198.1																			
21.0	Sound shale bedrock.		7	BX RC	Rec. 90%														
194.1	Grey.																		
25.0	End of borehole.																		

 EL. 218.0
 In open B.H.
 April 26/72

DEPARTMENT OF TRANSPORTATION AND COMMUNICATIONS

DESIGN SERVICES BRANCH

RECORD OF BOREHOLE No. 2A

FOUNDATION SECTION

JOB 71-11123

LOCATION Co-ords. 16,493,347 N. 1,235,626 E.

ORIGINATED BY S.A.A.

W.P. 10-69-09

BORING DATE April 26, 1972

COMPILED BY T.B.

DATUM Geodetic

BOREHOLE TYPE Auger & Rock Core

CHECKED BY *[Signature]*

[illegible]

DEPARTMENT OF TRANSPORTATION AND COMMUNICATIONS

DESIGN SERVICES BRANCH

RECORD OF BOREHOLE No. 3A

FOUNDATION SECTION

JOB 71-11123

LOCATION Co-ords. 16,493,340 N. 1,235,572 E.

ORIGINATED BY S.A.A.

W.P. 10-09-09

BORING DATE April 26, 1972.

COMPILED BY T.B.

DATUM Geodetic

BOREHOLE TYPE Auger

CHECKED BY *a.k.*

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE					LIQUID LIMIT — w_L		BULK DENSITY	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT		BLOWS / FOOT	20	40	60	80	100	PLASTIC LIMIT — w_p		
213.5	Ground elevation.														
0.0	Silty clay to clay. Firm to stiff. Grey.														
206.5						210									
205.0	Glacial till.		1	SS	4										
13.5	End of borehole.					200									

SHEAR STRENGTH P.S.F.

○ UNCONFINED + FIELD VANE
 ● QUICK TRIAXIAL x LAB. VANE

400 800 1200 1600 2000

w_p — w — w_L
 WATER CONTENT %

P.C.F. GR. SA. SI. CL.

El. 216.5
 in open B.H.
 April 26/72

DEPARTMENT OF TRANSPORTATION AND COMMUNICATIONS

DESIGN SERVICES BRANCH

RECORD OF BOREHOLE No. 4A

FOUNDATION SECTION

JOB 71-11123

LOCATION Co-ords. 16,493,566 N. 1,236,004 E.

ORIGINATED BY S.A.A.

W.P. 10-69-09

BORING DATE April 26, 1972

COMPILED BY T.B.

DATUM Geodetic

BOREHOLE TYPE Auger

CHECKED BY

[illegible]

Mr. F. R. Saint,
Regional Materials Engineer,
Eastern Region,
Kingston, Ontario.


Foundations Office,
Design Services Branch,
West Building, Downsview.

July 17, 1972.

Stability Analysis of Approach Fills for
Hwy. #417 and Hunt Club Rd.
W.O. 71-11123 --- W.P. 10-69-09
District No. 9 (Ottawa)

As per your request we have carried out stability analysis for the approach fills sections at the above-mentioned site, with different slopes, to find a safe slope without the use of berms. Based on the results of our analysis, it is concluded that the safe side slope for a fill height of 26 feet will be 3.5:1.

We hope that this data will be adequate for your requirements. If any further information is needed, please feel free to contact this Office.


Shaheen A. Ahmad,
Project Foundations Eng.,
M. Devata,
Supervising Foundations Eng.,

SAA/ao

For:

cc: T. C. Kingsland
M. M. Dillon & Co. Ltd. (Ottawa)

Foundations Files
Documents

MEMORANDUM

TO: Mr. E. R. Saint,
Regional Materials Engineer,
Eastern Region,
Kingston, Ontario.

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

ATTENTION: Mr. A. M. Batten,
Senior Soils Supervisor.

DATE: December 27, 1972.

OUR FILE REF. IN REPLY TO

SUBJECT:

*Stability of Approach Fills
Proposed Structure at the Crossing of
Hwy. #417 and Hunt Club Road
Regional Municipality of Ottawa-Carleton
District No. 9 (Ottawa)
W.O. 71-11123 -- W.P. 10-69-09*

According to your recent memo fill material consisting of shale bedrock or glacial till will be used in constructing the structure approach embankments in the above project. The unit weight of the compacted fill material will be of the order of 140 lb./cu.ft. instead of the 125 lb./cu.ft. which was assumed in the stability analyses presented in our Foundation Report.

We have carried out additional stability analyses using the revised unit weight as per your request. The revised lengths of the mid-height berm for various heights of fill are presented in tabular form below as well as on the graph appended to this letter.

Height of Fill Feet	Berm Length (At Mid-Height) Feet
30	50
25	35
20	10
17	Nil

- Notes: 1) Standard 2:1 slopes are used.
2) F.S. ≥ 1.3

December 27, 1972.

Should additional information be required, please do not hesitate to call this Office.



C. S. Poon,
Project Foundations Engineer,

For: M. Devata,
Supervising Foundations Engineer.

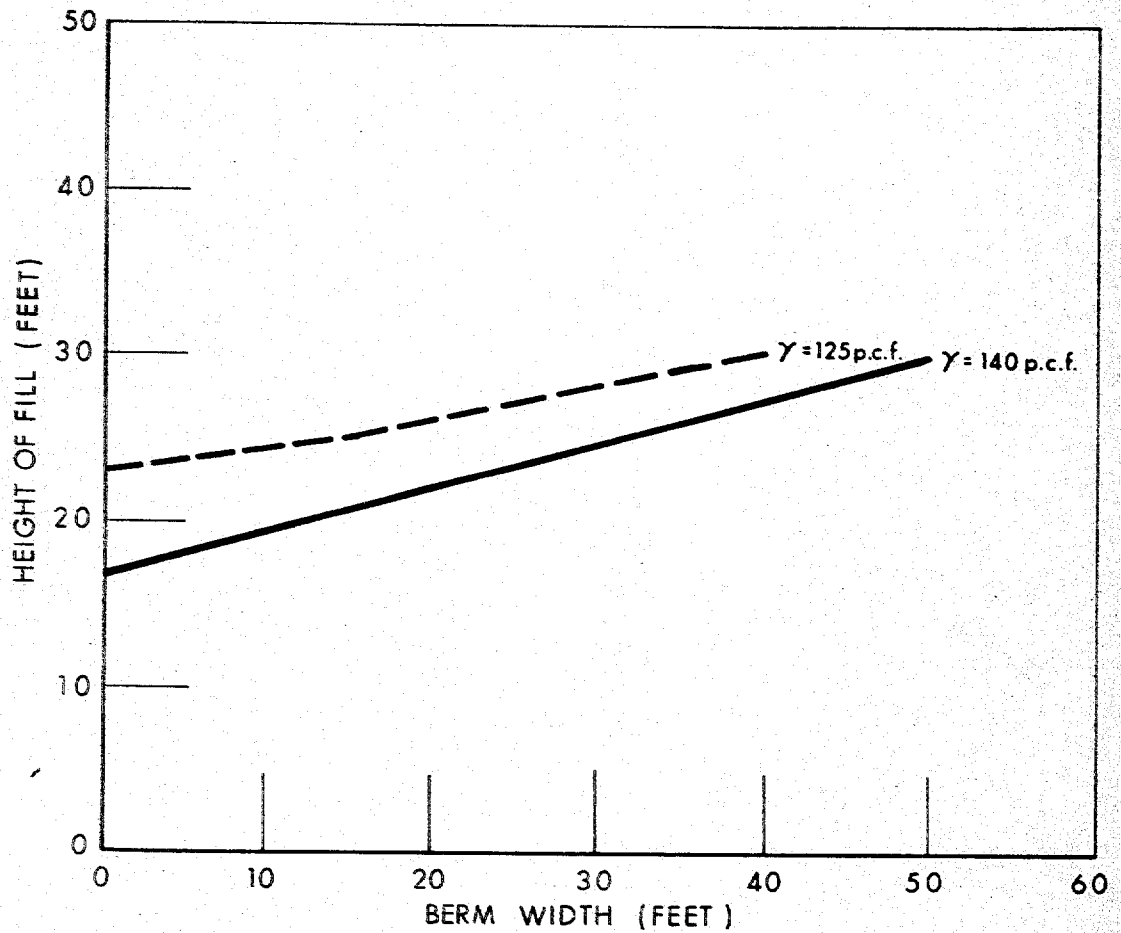
CSP/ck

c.c. A. Rutka
A. J. Percy
T. C. Kingsland
J. A. Cruickshank
G. A. Wrong
K. G. Bassi
M. M. Dillon & Co. Ltd.

Foundations Files ✓
Documents

W.O. No 71-11123

W.P. No 10-69-09



UNDERPASS STRUCTURE AT

HWY. # 417 and HUNT CLUB ROAD

(PRELIMINARY COMPUTATIONS BASED ON B.H. # 5) F.S. = 1.3

West Approach	SLOPES 3:1		SLOPES 2:1	
	Unsupported Ht.	Mid-Height Berm Require	Unsupported	Mid-Height Berm Requirements
Transverse Direction	20'	NIL	17'	NIL
	28' (max. Ht. Proposed)	30'	28' (max.)	45'
Longitudinal Direction	19'	NIL	16'	NIL
	22' (max.)	15'	22' (max.)	30'
<hr/>				
East Approach	(PRELIMINARY COMPUTATIONS BASED ON B.H. # 1)			
Transverse Direction	24'	NIL	20'	NIL
	27' (max.)	10'	27' (max.)	20'
Longitudinal Direction	20' (max.)	NIL	20' (max.)	NIL

DEPARTMENT OF TRANSPORTATION AND COMMUNICATIONS

MEMORANDUM

71-11125

TO: Mr. A. G. Stermac ,
Principal Foundation Engr.,
West Bldg., Downsview, Ont.

FROM: Materials & Testing Office,
Kingston, Ontario.

ATTENTION: M. Devata

DATE: October 24th, 1972 .

OUR FILE REF.

IN REPLY TO

SUBJECT:

RE: Hwy # 417, Innes Rd. to Ramsayville Structure Sites
W.P.'S 10-69-09, 10-69-04, 10-69-03, ~~10-69-08~~ ,
10-69-07, 10-69-13, 10-69-16, 10-69-14, 10-69-15 ,
10-69-01, 13-68-08 .

In your Foundation Reports for these projects you have assumed a unit weight of 125 lb/cu ft., for compacted fill in structure approach fills. Shale rock and glacial till may be used for fill purposes on these projects. Investigation in this regard is currently underway . The unit weight of these materials is approximately 150 lb/cu ft.

The Design Consultant should be advised of the additional berm requirements at these structure approach fills for the higher unit weight (150 lb/cu ft.

AMB/pab

A.M. Batten

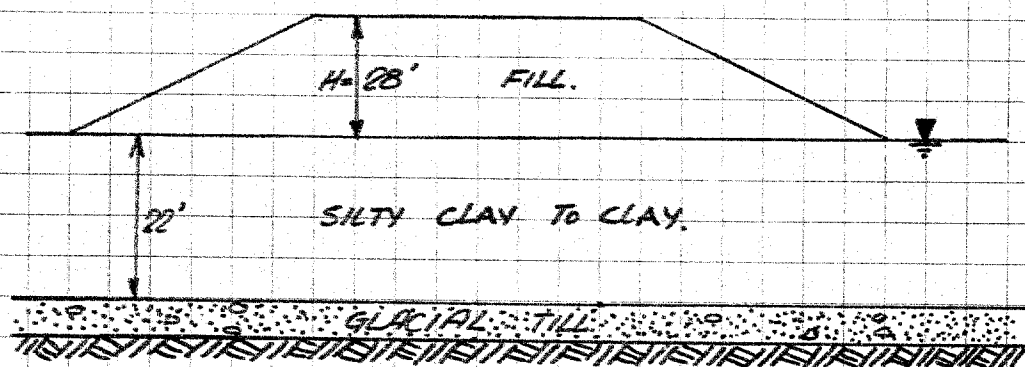
A.M. Batten ,
Senior Soils Supervisor .

c.c. P.D. Billings
A.J. Percy
J.E. Callaghan
T. Kingsland
A. Rutka
G. Wrong

CALCULATION FOR CONSOLIDATION SETTLEMENT: (71-11123)

UNDER PT. "B"

CASAGRANDE CONSTRUCTION:



LAYER	ELEV.	TEST	P_0	e_0	P_1	e_1	Δe	$S(\text{INS})$
218-208	213	I	0.125	1.092	1.875	1.050	0.042	2.4" 144
208-198	204	II	0.300	1.520	2.000	1.317	0.203	7.5" 5.6
198-196	198	III	0.500	1.504	2.130	1.293	0.211	3.0" 2.7

Total calculated settlement = 13 ins.
CORRECTED SETTLEMENT = 9.74 ~ 10"