

G.I.-30 SEPT. 1976

GEOCRES No. 3165-113DIST. 9 REGION W.P. No. 10-69-08CONT. No. 73-190W. O. No. STR. SITE No. HWY. No. 417LOCATION Walley Road ExtensionNo. of PAGES -=====OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT. REMARKS:

3165-113

DEPARTMENT OF TRANSPORTATION AND COMMUNICATIONS

MEMORANDUM

TO: Mr. T. C. Kingsland, (4)
Regional Bridge Planning Engineer,
Eastern Region,
Kingston, Ontario.

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

ATTENTION:

DATE: May 16, 1972.

OUR FILE REF.

IN REPLY TO

MAY 25 1972

SUBJECT:

FOUNDATION INVESTIGATION REPORT
For
Proposed Structure at the
Crossing of Hwy. #417 and Walkley Road
Extension
Regional Municipality of Ottawa-Carleton
District #9 (Ottawa)
W.O. 71-11125 -- W.P. 10-69-08

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

A. G. Stermac
A. G. Stermac,
PRINCIPAL FOUNDATIONS ENGINEER.

M. M. Dillon & Co. Ltd. (J. H. Kearney)

Foundations Files
Documents

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FOUNDATION INVESTIGATION REPORT
For
Proposed Structure
At the Crossing of Hwy. #417 and Walkley Road
Extension
Regional Municipality of Ottawa-Carleton
District #9 (Ottawa)
W.O. 71-11125 -- W.P. 10-69-08

1. INTRODUCTION:

The Foundations Office was requested to carry out a subsurface investigation at the crossing of proposed Hwy. #417 and the Walkley Rd. 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 Office to determine the subsoil, bedrock and groundwater conditions at the site, based on the available preliminary information. At a later date, a detailed scheme prepared by M. M. Dillon, Consulting Engineers, showing the proposed structure layout was submitted to this Office on E - plan (Drawing No. E-5228-1). The finalized proposal necessitated additional borings and as a result of this, further field investigation was carried out by this Office during April 1972.

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

2. DESCRIPTION OF SITE AND GEOLOGY:

The area under investigation is located approximately 1,200 feet west of the junction of Walkley Road and Sheffield Road along the projected extension of Walkley Road, in the Township of Gloucester, Regional Municipality of Ottawa-Carleton.

The terrain is flat to gently undulating in relief between about elevations 212 to 216. East of the structure a small stream meanders in a east-west direction. 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 bed-rock 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 retreat, the area was, in large part, below the sea level, so that as the ice retreated 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 retreated still further, uplift took place. The land gradually emerged from the sea. The area is now commonly referred to as "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 formations.

.....3

* Johnston, W.A.

"Pleistocene and Recent Deposits in the Vicinity of Ottawa, With a Description of the Soils." Geological Surveys #84. Department of Mines.

**Chapman, L.V.
and Putnam, D.F.

"Physiography of Southern Ontario."
University of Toronto Press, 1967.

3. FIELD AND LABORATORY WORK:

Five sampled boreholes, all of which were accompanied by dynamic cone penetrating tests, were put down at the site during the period of the field investigation, using conventional diamond drill rigs adapted for soil sampling purposes. In addition, two dynamic cone penetration tests were carried out 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 three of the boreholes by obtaining BX size rock core samples.

The soil, bedrock and groundwater conditions encountered at the boring locations are presented on the Record of Borelog sheets appended to this report. The locations and elevations of the various boreholes were provided by personnel from the Eastern Region Engineering Surveys Section. The elevations in this report are referenced to a geodetic datum, and the locations were subsequently referred to a coordinate system. The boring locations and elevations are shown on Drawing No. W.O. 71-11125A. Stratigraphical profiles, inferred from the boring data are also presented on the aforementioned drawing.

All the 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 a selected representative sample in order to determine the physical properties of the overburden, namely:

- Natural Moisture Content
- Atterberg Limits
- Bulk Density

Grain-Size Distribution
Undrained Shear Strength
Consolidation Characteristics

The results of these tests are plotted on the Record of Borelog sheets and are summarized on Figures 1, 2 and 3, in the Appendix to this report.

4. SUBSOIL AND BEDROCK CONDITIONS:

4.1) General:

The predominant stratum across the site is composed of a stiff to hard silty clay to clay. The thickness of this cohesive stratum varies from 16 to 21 feet. Underlying this deposit is an 11 to 17 foot thick granular glacial till deposit. 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. 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 (3") is the predominant stratum across the site, which is composed of silty clay to clay. The thickness of this sensitive cohesive deposit varied from 16 feet (B.H. #3A) to 21 feet (B.H.'s #2A and 5A). At B.H.'s #1A, 2, 3A & 5 a light and dark grey layering pattern was noticed. Occasional sandy silt seams, up to 1 inch thick, were encountered throughout the stratum. Grain-size distribution curves for samples of the clay stratum are shown in Figure No. 2 in the Appendix.

The properties of the cohesive stratum, as determined by field and laboratory testing, are plotted on the individual borelog sheets. A brief resume, presented in tabular form, follows:

	<u>Range</u>
Bulk Density (γ) (p.c.f.)	102 - 119
Liquid Limit (W_L) (%)	34 - 59
Plastic Limit (W_p) (%)	21 - 25
Natural Moisture Content (W) (%)	30 - 65
Standard Penetration Resistance ('N') (Blows/ft.)	9 - 16
Initial Void Ratio (e_o)	1.00 - 1.537
Compression Index (C_c)	0.16 - 0.98
Degree of Preconsolidation Pressure ($P_c - P_o$) (p.s.f.)	4760 - 5620
Undrained Shear Strength (C_u) (p.s.f.)	
i) Field Vanes	880 - >2,000
ii) Lab. Vanes	850 - 2520
iii) Lab. Testing	245 - 1610
Sensitivity	5 - 18

The Atterberg limit tests, summarized in Table I are also plotted on the plasticity Chart (Figure #1). These results indicate that, in general, the clay is inorganic with a plasticity in the intermediate 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 hard.

The consolidation characteristics of the stratum were determined by carrying out 4 laboratory tests. The results of this testing indicated that the clay is preconsolidated by 4,760 to 5,620 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 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 11 to 17 feet in thickness. Boulders up to 6 inches in size were encountered randomly throughout the glacial till deposit. Typical grain-size distribution curves, for samples of this deposit obtained with 2 inch O.D. sampling equipment, are shown on Figure No. 3.

The Standard Penetration Tests carried out gave 'N' values which ranged from 5 blows/ft. to 70 blows for 3 inches. Based on these values it is estimated that the relative density of the glacial till stratum varies from loose to very dense. The lower 'N' values are generally encountered in the upper portion of the till deposit. It is inferred that this zone has been subjected to "reworking".

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 7 feet of BX size rock core samples. Over the site the bedrock surface was found to vary between elevations 182 to 188, which corresponds to depths below ground surface of from 27.5 to 31 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.

5. GROUNDWATER CONDITIONS:

The groundwater level conditions across the site, during the period of investigation (November and December 1971), as well as in April 1972 investigation were observed by taking readings in the open boreholes. The results of the readings are plotted on the Record of borehole sheets.

The observations indicate that the groundwater level, in the overburden, is located between elevations 213 and 215, which corresponds to levels which range from ground surface to as much as 2 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 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 No. 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 the 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 report deals with the proposed underpass structure at the crossing of Walkley Rd. Extension and Hwy. #417, in the Township of Gloucester, Regional Municipality of Ottawa-Carleton. Discussions with regard to other structures on this portion of the Freeway will be presented in separate foundation reports.

It is proposed to construct a two-span (150' - 150') structure at the crossing of Hwy. #417 and Walkley Rd. Extension. The profile grade of Hwy. #417, in the vicinity of the structure, will be about elevations 216 to 218. The proposed profile grade of the Walkley Rd. Extension will be between elevation 238 and 240. At these grades the maximum height of the approach fills will be of the order of 24 feet.

The predominant stratum across the site is composed of grey firm to hard silty clay to clay having a thickness ranging from 16 to 21 feet. The cohesive deposit is underlain by 11 to 17 feet of granular glacial till stratum which, in turn, is followed by sound 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 in 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.)	PARAMETER UNDRAINED SHEAR STRENGTH (Cu (P.S.F.))	EFFECTIVE ANGLE OF INTERNAL FRICTION (ϕ)
239-215	Embankment	125	--	30°
	Fill (slope 2:1)			
215-205	Silty clay	112	1800	--
205-195	Silty clay	112	1000	--
195-185	Glacial till	130	--	35

Notes: 1) Approximate groundwater level - elevation 213.

2) Tension Crack - 7 feet.

The results of the computations indicate that the embankments constructed with granular type of fill material with standard 2:1 slopes will be inherently stable against a deep-seated rotational type of failure.

In the stability analysis, it was assumed that a granular type of material will be used for embankment construction. At a later date, if it is decided to utilize a different type of material for the embankment construction, the Foundation Office should be notified of the pertinent data, including the bulk

density of the fill material. Additional analyses will be carried out with regard to the stability of the approach fills and the recommendations for the revised conditions will be submitted by this Office.

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 will be of the order of 5 inches. It is estimated that these consolidation settlements will take place in a period of 5 years. It is expected that fifty percentage of the estimated consolidation settlement will be realized in a period of twelve months. In order to minimize post construction maintenance problems fills should be constructed and left in place for as long a period as possible prior to the paving operations, preferably twelve months if scheduling and other requirements permit.

6.3) Structure Foundations:

6.3.1) Pier (refer to B.H. #4 & 3A):

The pier can be supported on end-bearing piles driven to bedrock. The estimated pile tip elevation will be about elevation 186 to 188. 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 run-off into the excavation could be readily handled by using standard techniques, such as pumping from sumps.

6.3.2) Abutments (Refer B.H.'s 2A, 1A & 4A):

The "perched" abutments within the approach fills can be supported on end-bearing piles driven to bedrock. It should be noted that the piles may not reach bedrock in the case of east abutment in view of the presence of very dense glacial till deposit immediately above the bedrock surface. It is believed that the piles may not penetrate below elevation 187. Pile driving at this location should be controlled by means of the Hiley Dynamic Pile Driving Formula as per current M.T.C. standards, to obtain the safe allowable loads of the pile section chosen. The anticipated consolidation settlements at the approaches will induce negative skin frictional 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 will be desirable to reduce the allowable design load by about 15% with respect to the maximum safe 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 initial field work for this project was carried out during the period of December 2 to 22, 1971, and additional work was performed during April 22 to 28, 1972, under the supervision of Mr. S. A. Ahmad, Project Foundations Engineer.

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

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

S. A. Ahmad

S. A. Ahmad, P. Eng.



M. Devata

M. Devata, P. Eng.

SAA/ao
May 10/72

APPENDIX I

FOUNDATION SECTION

JOB	71-11125	LOCATION	Co-ords. 499,002 N; 233,025 E.	ORIGINATED BY	WH
W.P.	10-69-08	BORING DATE	Nov. 16 & 17, 1971	COMPILED BY	SO
DATUM	Geodetic	BOREHOLE TYPE	NX Washboring	CHECKED BY	<i>[Signature]</i>

[illegible]

FOUNDATION SECTION

SOIL PROFILE			SAMPLES			ELEV. SCALE ELEV. / FOOT	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		
215.0	Ground Level						SHEAR STRENGTH P.S.F. ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB. VANE	WATER CONTENT % $w_p \quad w \quad w_L$		
0.0						210				
						200				
196.0										
19.0	End of Cone					190				

FOUNDATION SECTION

[illegible]

FOUNDATION SECTION

[illegible]

FOUNDATION SECTION

JOB	71-11125	LOCATION	Co-ords. 498,840 N; 232,721 E.	ORIGINATED BY	
W.P.	10-69-08	BORING DATE	Dec. 16, 1971	COMPILED BY	30.
DATUM	Geodetic	BOREHOLE TYPE	NX Washboring	CHECKED BY	42

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION	RESISTANCE	LIQUID LIMIT ——— w_L	PLASTIC LIMIT ——— w_p	BULK DENSITY γ P.C.F.	REMARKS	
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT		BLOWS / FOOT			WATER CONTENT ——— w			
							20	40	60	80			100
							SHEAR STRENGTH P.S.F.		w_p ——— w ——— w_L				
							○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB. VANE		WATER CONTENT %				
							400 800 1200 1600 2000		15 30 45				
214.9	Ground Level												
0.0	Silty clay to Clay											GR. SA. SI. CL.	
	Stiff to Very Stiff		1	SS	13	210							
			2	TW	PM								
	Grey		3	TW	PM								
			4	TW	PM								
196.9			5	TW	PM	200					107		
			6	TW	PM								
18.0	Het. mix. of silt, sand & gravel, trace of clay.		7	SS	5								
	Glacial Till		8	SS	21								
184.8	Loose to Very Dense Probable Bedrock		9	SS	135/9"	190					115	6 18 54 22 17 25 40 18	
30.1	End of Borehole					180							

DEPARTMENT OF TRANSPORTATION AND COMMUNICATIONS

DESIGN SERVICES BRANCH

RECORD OF BOREHOLE No. 1A

FOUNDATION SECTION

JOB 71-11125

LOCATION Co-ords. 16,498,890 N; 1,233,074 E.

ORIGINATED BY SAA

W.P. 10-69-08

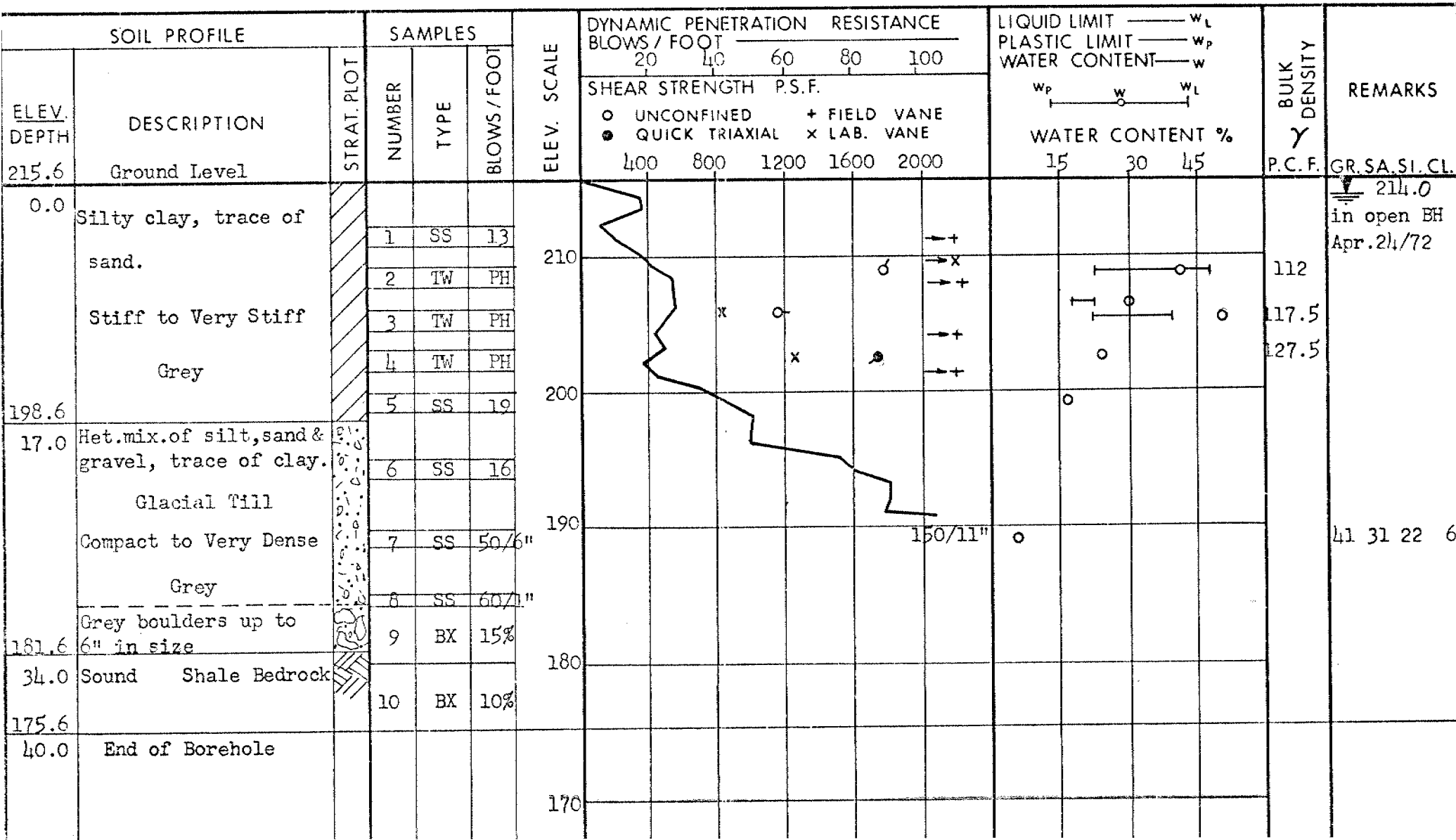
BORING DATE April 21, 1972

COMPILED BY SAA

DATUM Geodetic

BOREHOLE TYPE Auger and BX Rock Core

CHECKED BY



OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF TRANSPORTATION AND COMMUNICATIONS

DESIGN SERVICES BRANCH

JOB 71-11125

W.P. 10-69-08

DATUM Geodetic

LOCATION

Co-ords. 16,498,746 N; 1,232,806 E.

BORING DATE

April 24, 1972

BOREHOLE TYPE

Auger and BX Rock Core

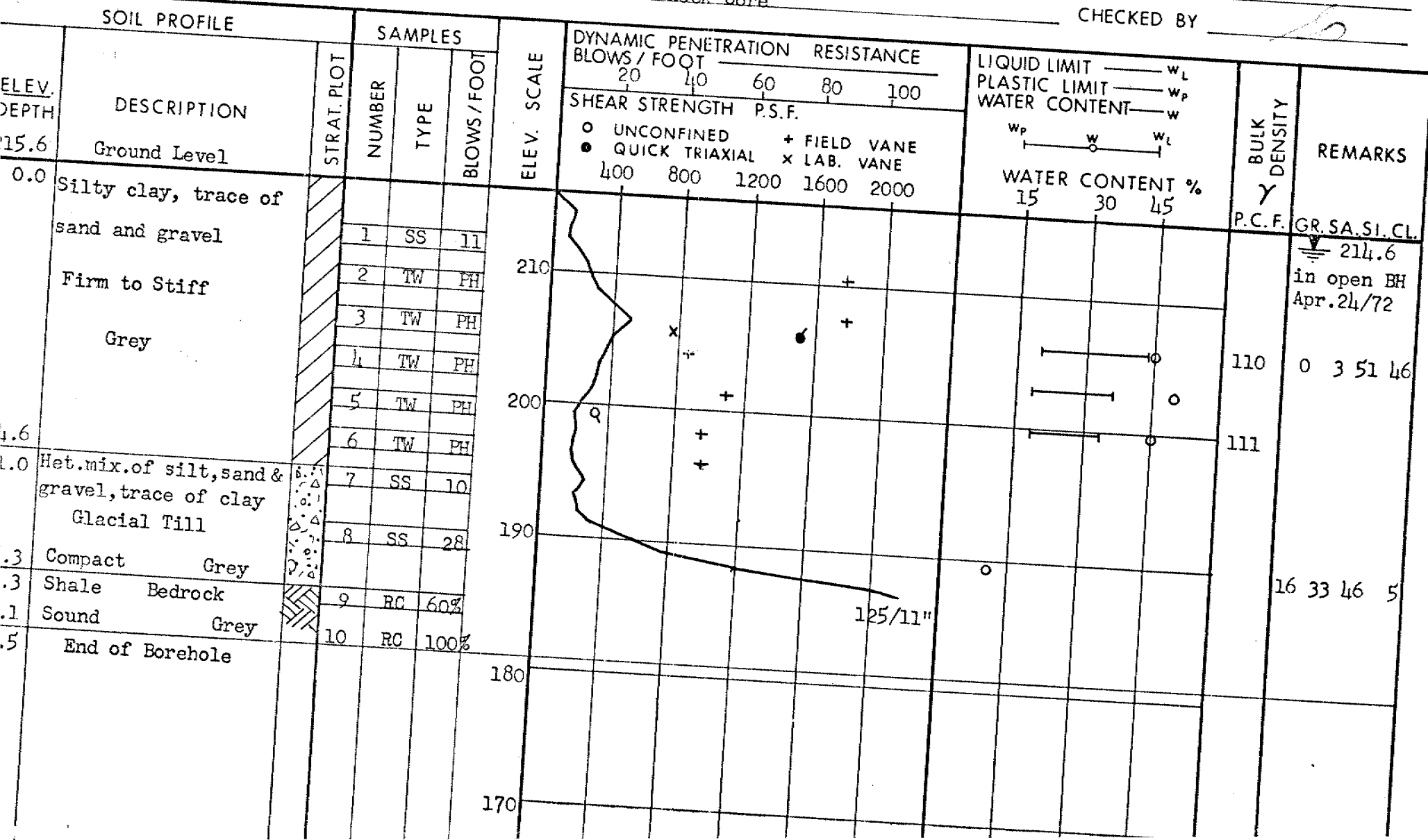
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ORIGINATED BY SAA

COMPILED BY SAA


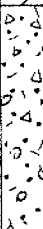


CHECKED BY

RECORD OF BOREHOLE No. 2A



FOUNDATION SECTION

CHECKED BY

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT					LIQUID LIMIT ——— w_L PLASTIC LIMIT ——— w_p WATER CONTENT ——— w			BULK DENSITY γ P.C.F.	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT		SHEAR STRENGTH P.S.F.					WATER CONTENT %				
							20	40	60	80	100	w_p	w	w_L		
215.5	Ground Surface															
0.0	Silty clay, trace of sand and gravel		1	SS	14											
	Firm to Very Stiff		2	TW	PH											
	Grey		3	TW	PH											
			4	TW	PH											
199.5			5	TW	PH											
16.0	Het. mix. of silt, sand & gravel, trace of clay.															
	Glacial Till		6	SS	34											
	Dense to Very Dense		7	SS	104											
186.0	Probable Bedrock															
29.5	End of Borehole															

DEPARTMENT OF TRANSPORTATION AND COMMUNICATIONS

RECORD OF BOREHOLE No. 4A

FOUNDATION SECTION

DESIGN SERVICES BRANCH

JOB 71-11125

LOCATION Co-ords. 16,498,955 N; 1,233,069 E.

ORIGINATED BY SAA

W.P. 10-69-08

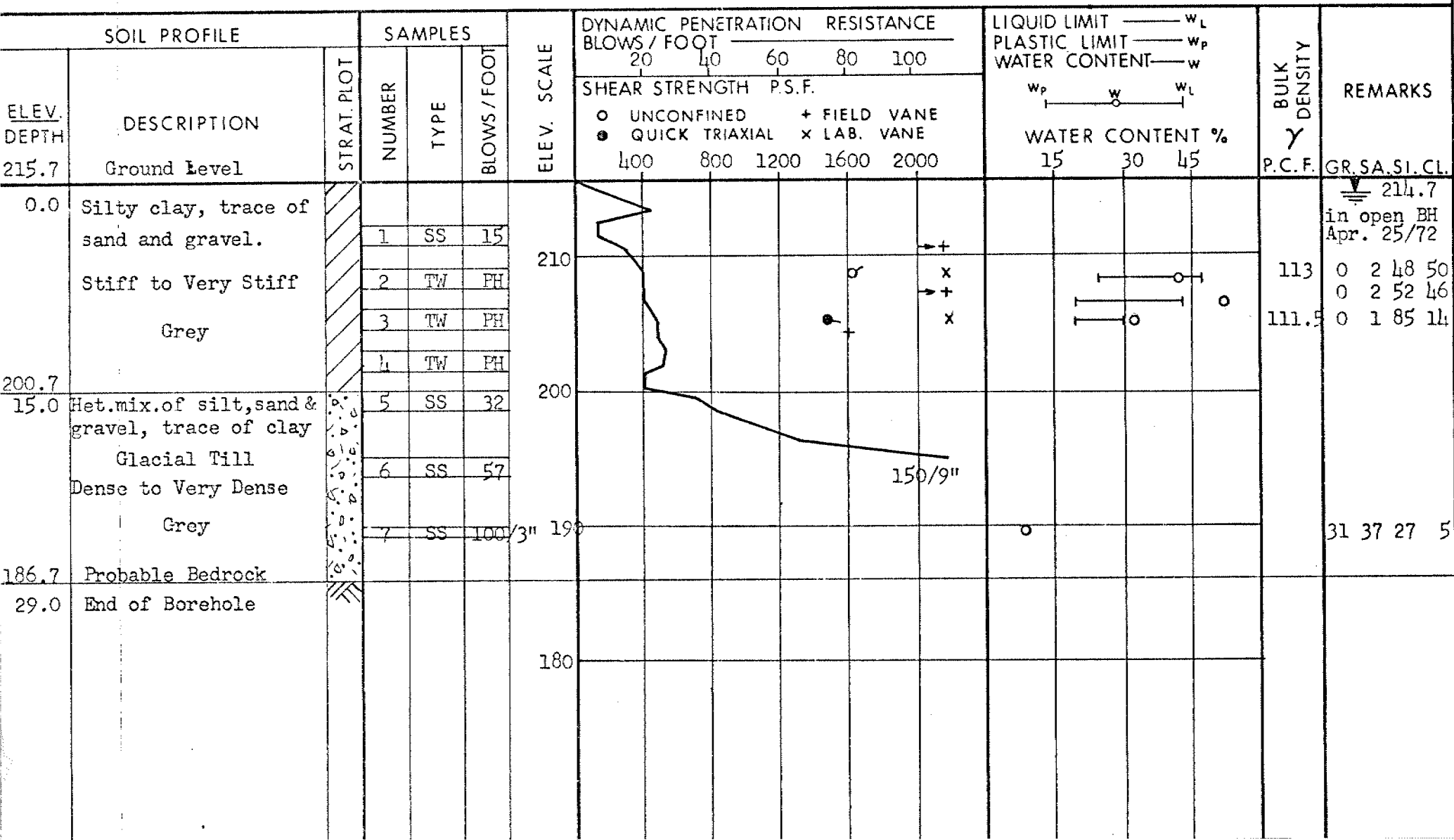
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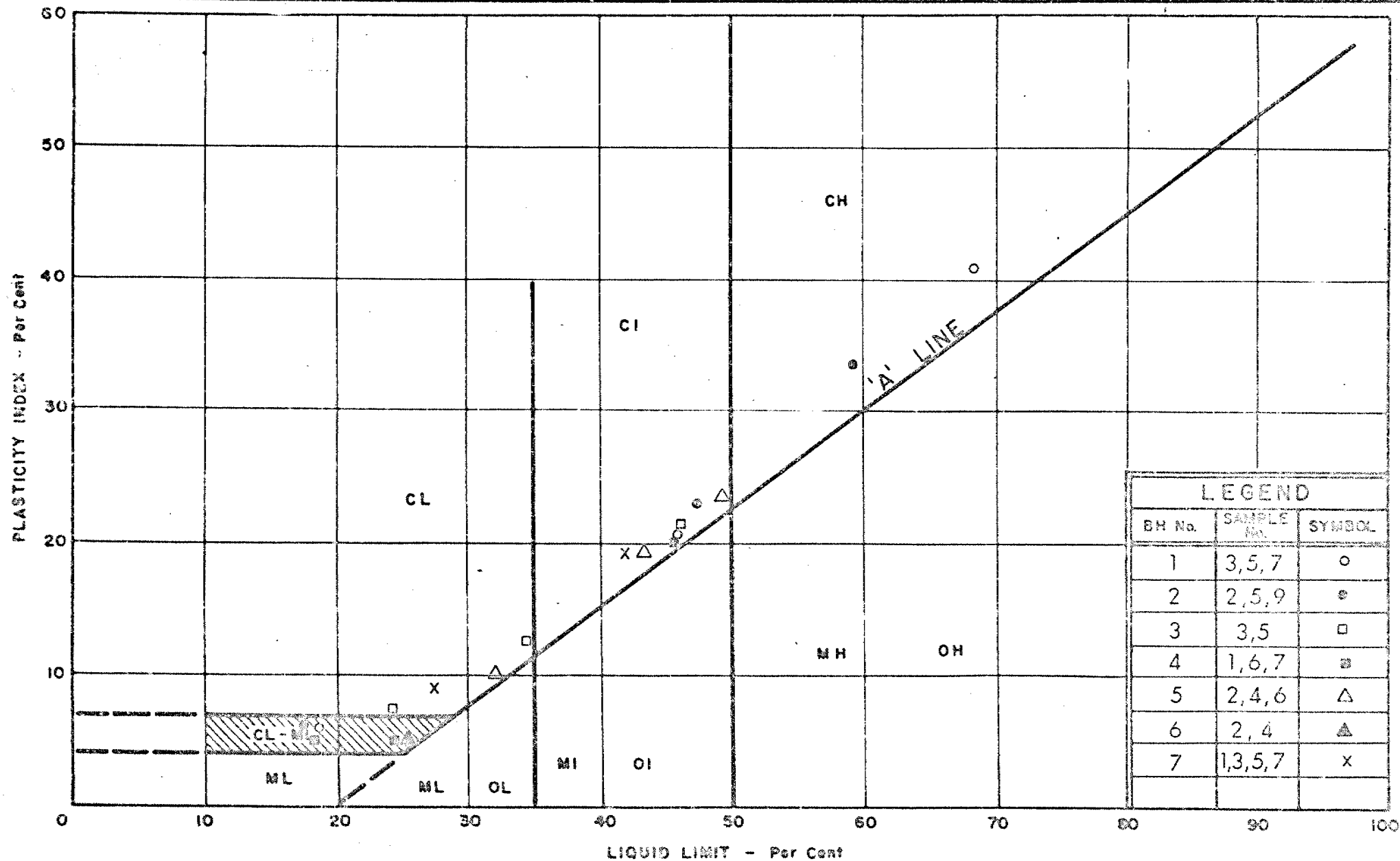
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DATUM Geodetic

BOREHOLE TYPE Auger - BX Rock Core

CHECKED BY





DEPARTMENT OF HIGHWAYS
MATERIALS and
TESTING
DIVISION

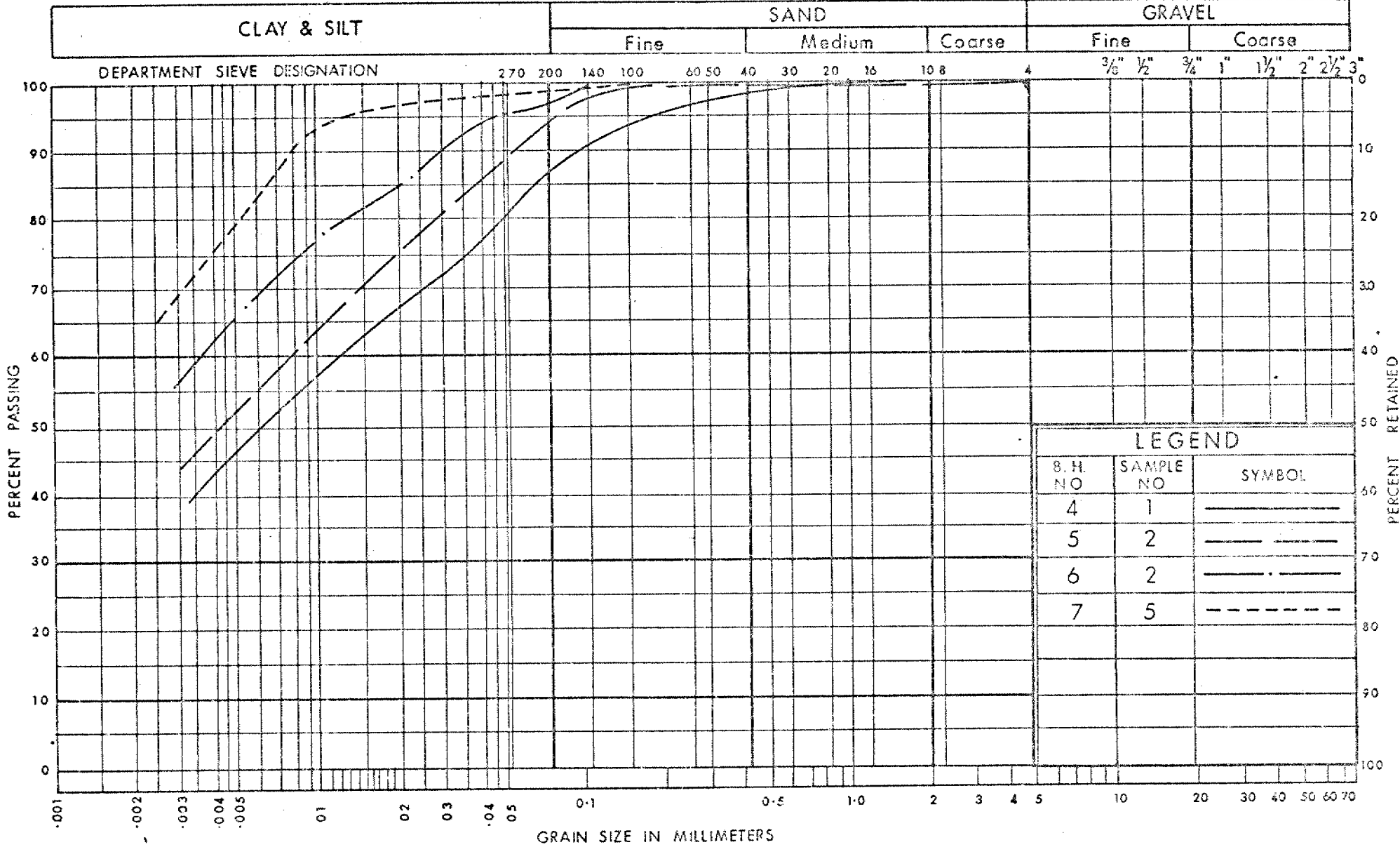
PLASTICITY CHART

W.P. No. 10-69-08

JOB No. 71-11125

FIG. 1

UNIFIED SOIL CLASSIFICATION SYSTEM



DEPARTMENT
OF
TRANSPORTATION AND COMMUNICATIONS

DESIGN SERVICES
BRANCH

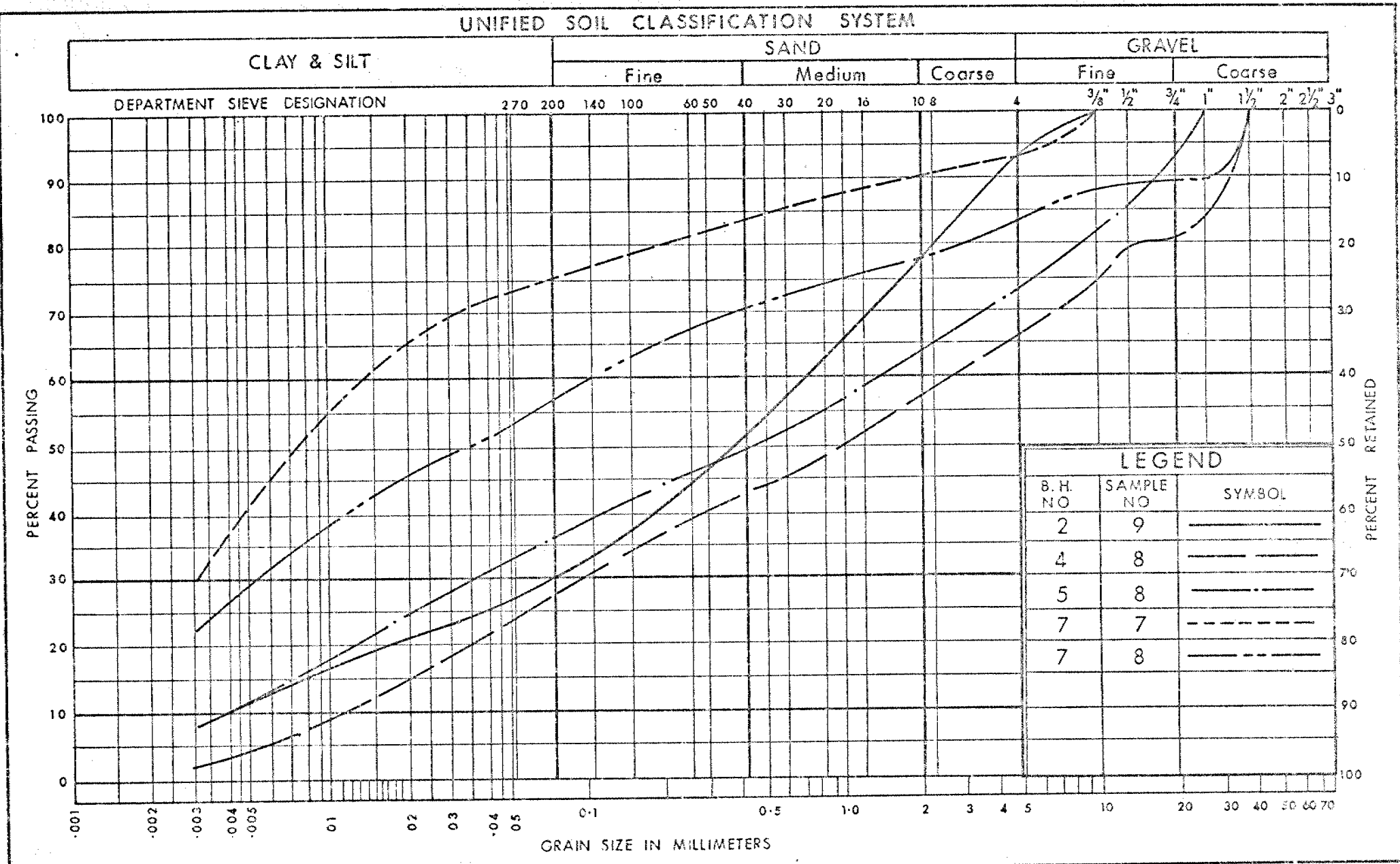
GRAIN SIZE DISTRIBUTION

SILTY CLAY TO CLAY

W.P. No. 10-69-08

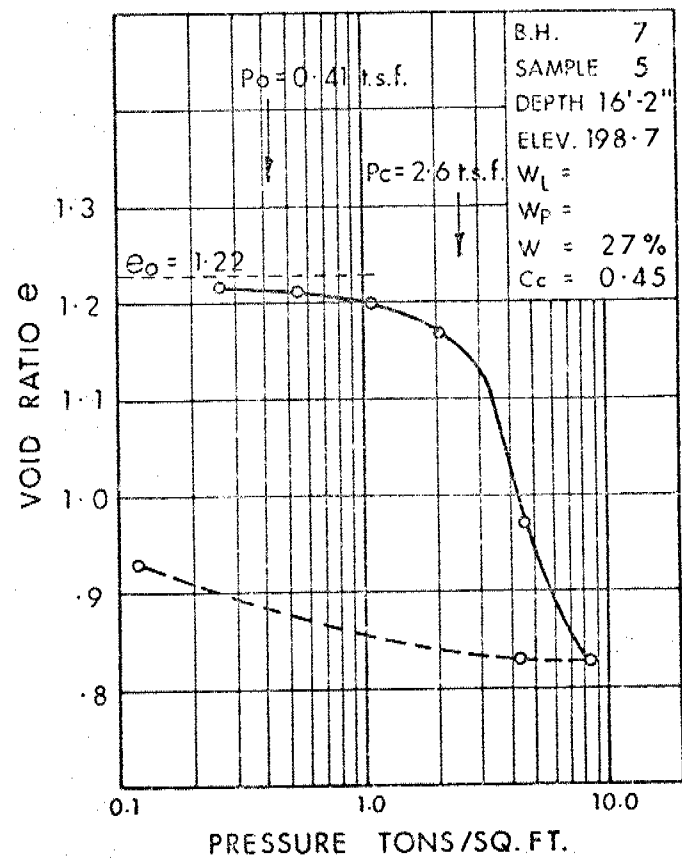
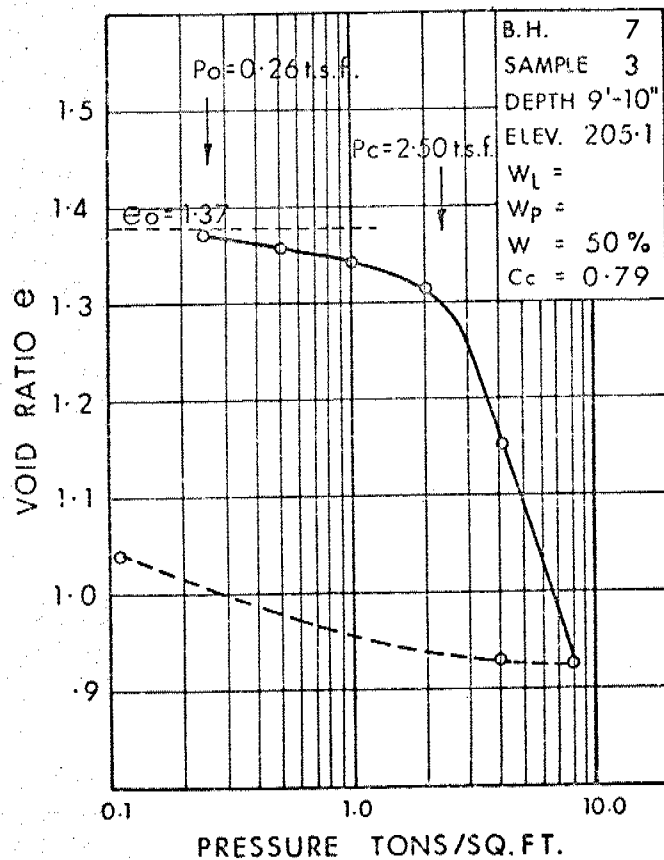
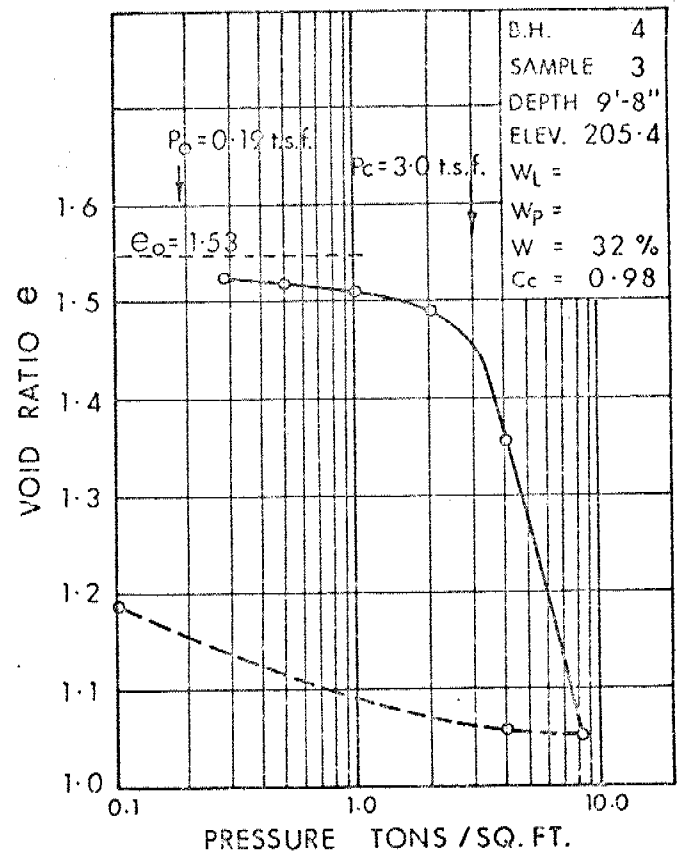
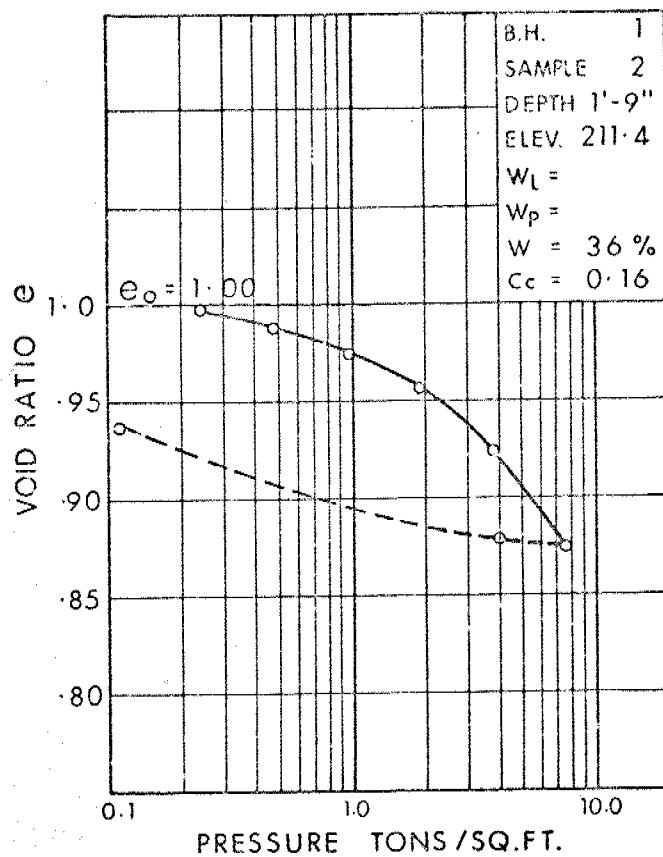
JOB No. 71-11125

FIG. 2



VOID RATIO - PRESSURE CURVES

JOB NO. 71-11125



ABBREVIATIONS USED IN THIS REPORT

PENETRATION RESISTANCE

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

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

DESCRIPTION OF SOIL

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

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

TYPE OF SAMPLE

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

SOIL TESTS

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

ABBREVIATIONS USED IN THIS REPORT

SOIL PROPERTIES

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

GENERAL

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

STRESS AND STRAIN

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

EARTH PRESSURE

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

FOUNDATIONS

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

SLOPES

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

35MM DRAWING

