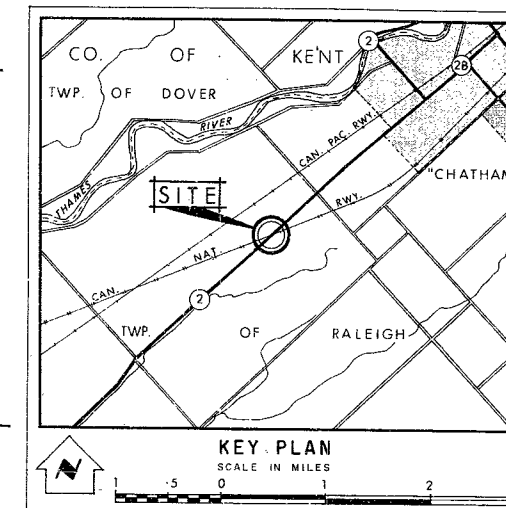
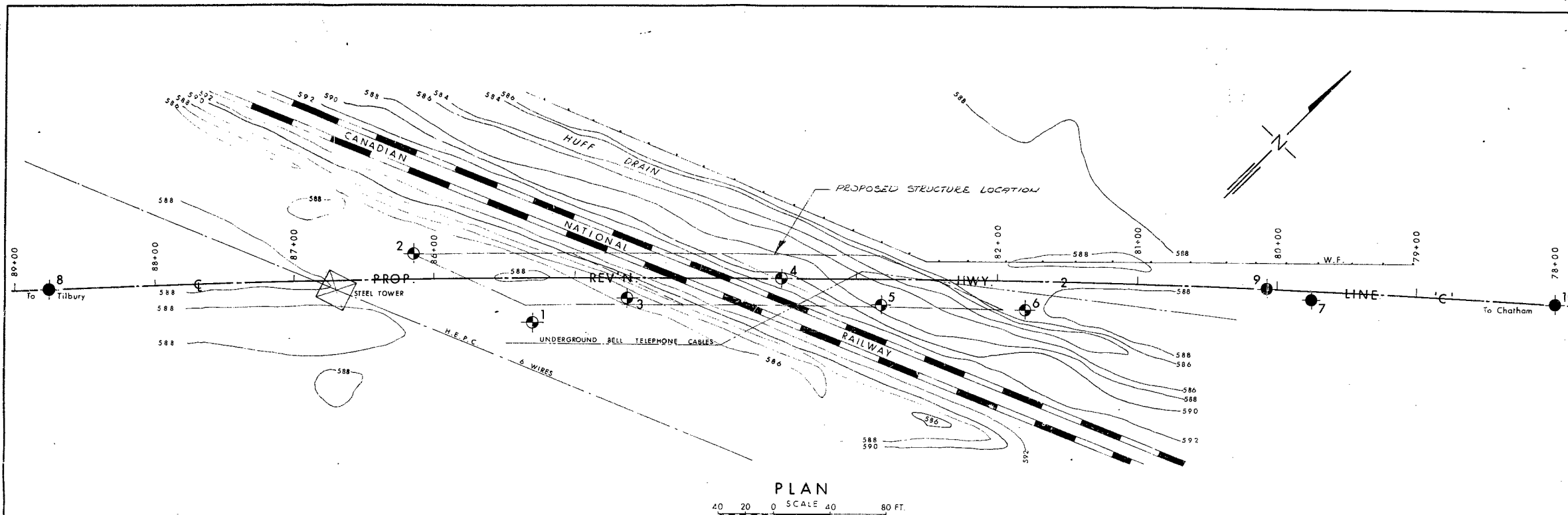


#66-F-76

W.P.# 13-63

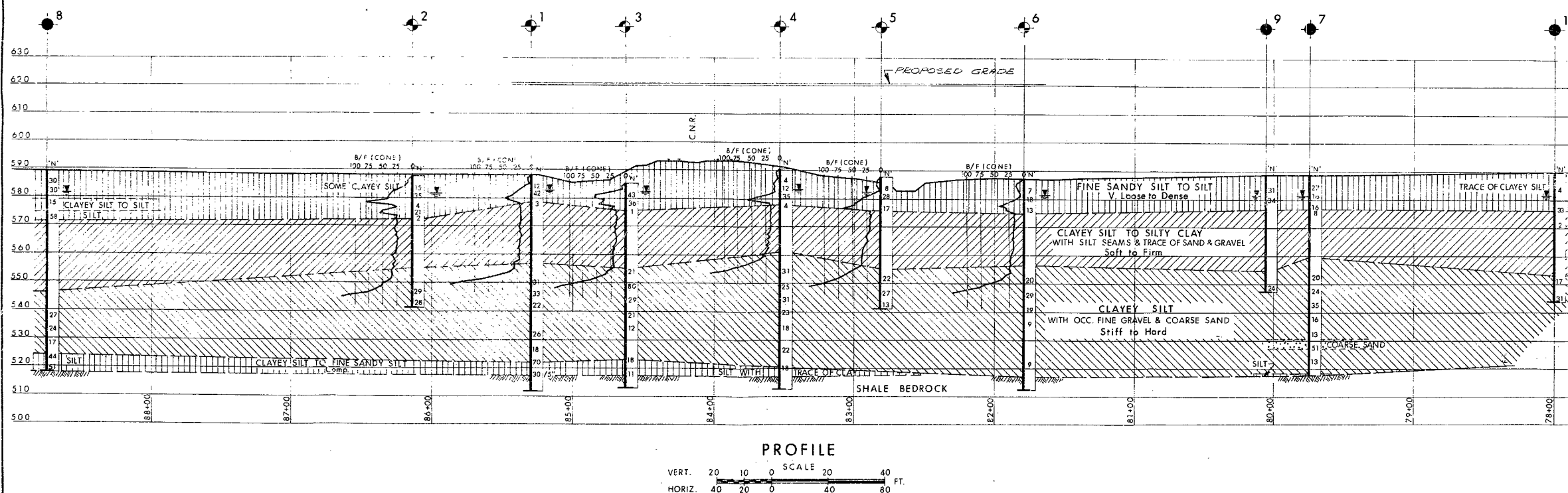
Hwy. #2 :

C.N.R.



LEGEND			
	Bore Hole		
	Cone Penetration Hole		
	Bore & Cone Penetration Hole		
	Water Levels established at time of field investigation. SEPT. 1966		

NO.	ELEVATION	STATION	OFFSET
1	587.7	85+30	32' LT.
2	587.7	86+14	18' RT.
3	585.1	84+63	14' LT.
4	590.5	83+53	CL
5	587.6	82+82	18' LT.
6	587.5	81+80	20' LT.
7	588.8	79+75	CL
8	589.5	88+75	CL
9	588.7	80+07	CL
10	590.0	78+00	CL



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.

DATE	BY	DESCRIPTION

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

CANADIAN NATIONAL RAILWAY

KING'S HIGHWAY NO. 2 LINE 'C' REVISION DIST. NO. 1
CO. KENT 1 1/2 MI. WEST OF CHATHAM
TWP. RALEIGH LOT 17 CON. IV

BORE HOLE LOCATIONS & SOIL STRATA

SUBM'D. A.S. CHECKED	W.P. NO. 13-63	M.B.T. DRAWING NO.
DRAWN S.O. CHECKED	JOB NO. 66-F-76	66-F-76 A
DATE 4 NOV. 1966	SITE NO.	BRIDGE DRAWING NO.
APPROVED <i>Althamas</i>	CONT. NO.	

REF. NO. E-4386-1

CC: 66-115

W.P. 13-63

DEPARTMENT OF HIGHWAYS ONTARIO

MEMORANDUM

TO: Mr. B. R. Davis,
Bridge Engineer,
Bridge Division.

Attention: Mr. S. McCombie

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

DATE: November 23, 1966

OUR FILE REF.

IN REPLY TO: **NOV 28 1966**

SUBJECT:

FOUNDATION INVESTIGATION REPORT
For
C.N.R. & Hwy. #2, Line 'C' Revision,
District #1 (Chatham)
W.J. 66-F-76 -- W.P. 13-63

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

We believe that the factual data and recommendations contained therein, will prove adequate for your design requirements. Should you require additional information, please feel free to contact our Office.

AGS/MdeF
Attach.

cc: Messrs. B. R. Davis (2)
H. A. Tregaskes
D. W. Farren
A. Gater
F. C. Brown
J. Roy
A. P. Watt
A. Watt

Foundations Office
Gen. Files ✓

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

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 - 5.3) Clayey Silt to Silty Clay.
 - 5.4) Clayey Silt.
 - 5.5) Bedrock.
6. GROUNDWATER.
7. DISCUSSION AND RECOMMENDATIONS:
 - 7.1) Overhead Structure and Approaches.
 - 7.2) Subway Structure and Approaches.
8. SUMMARY.
9. MISCELLANEOUS.

FOUNDATION INVESTIGATION REPORT
For
C.N.R. & Hwy. #2, Line 'C' Revision,
District #1 (Chatham)
W.J. 66-F-76 -- W.P. 13-63

1. INTRODUCTION:

The Foundation Section was requested to carry out an investigation at the site of the proposed crossing of the C.N.R. and Hwy. #2, Revision Line 'C', at Chatham, Ontario. The request was contained in a memo from Mr. A. P. Watt, Regional Bridge Location Engineer, dated July 18, 1966. An investigation was subsequently carried out by this Section to determine the subsoil conditions existing at the site. This report contains the results of the investigation, together with recommendations pertaining to the foundations of the new structure and the stability of the approaches.

2. DESCRIPTION OF SITE:

The site is located some 0.5 miles west of the west limits of the City of Chatham. At this location the existing Hwy. #2 crosses the C.N.R. tracks at a level crossing. The surrounding area is generally flat and consists mostly of cultivated farmland. The railway is built on a small embankment some 5 feet higher than the average original ground level. The centre-line of the proposed Revision Line 'C' is located on the north side of the present Hwy. #2.

Physiographically, the site is located in the region referred to as the St. Clair Clay Plains. The clay deposit at the actual site location is referred to as the Chatham Flats where the main clay layer is overlain by a shallow sand layer and is underlain by a clay till.

3. FIELD WORK:

Ten boreholes and six dynamic cone penetration tests were carried out during the course of the field work. Boring was achieved by means of a conventional diamond drill adapted for soil sampling purposes. Samples were recovered at required depths in 2-inch O.D. split-spoon samplers which were hammered into the soil, or in 2-inch I.D. Shelby tubes which were manually pushed into the soil. The method of driving the split-spoon samplers conformed to the requirements of the Standard Penetration Test. The same method was used to advance the cone in the dynamic cone penetration tests. Where possible, field vane tests were carried out at various depth intervals in order to determine the undrained shear strength of the cohesive strata. Bedrock was proven in four borings by obtaining AXT size rock core samples. In two other borings bedrock was assumed to be the level at which the borehole casing met refusal whilst being driven by means of a 350-lb. hammer. During sampling and drilling operations, detailed logs of the borings were made which described drilling and sampling techniques, soil types encountered, and groundwater observations. All samples were visually examined in the field prior to being transported to the laboratory.

The locations and elevations of all borings were surveyed in the field by personnel from London Region Engineering Surveys Section, and are shown on Drawing #66-F-76A, together with the estimated stratigraphical profile. This drawing is included in the Appendix of the report.

4. LABORATORY TESTING:

All samples were subjected to a careful visual inspection in the laboratory prior to any tests being carried out. Following this inspection, tests were carried out on certain samples to

cont'd. /3 ...

4. LABORATORY TESTING: (cont'd.) ...

determine the following physical properties of the various soil types:

- Grain-size Distribution
- Natural Moisture Content
- Bulk Density
- Liquid Limit
- Plastic Limit
- Undrained Shear Strength

The results of these tests are summarized and plotted on the Record of Borelog sheets contained in the Appendix of the report.

On completion of laboratory testing the various soil samples were classified as to type and consistency, or relative density, in general according to the Unified Soil Classification System (October 1963).

5. SOIL TYPES AND SOIL CONDITIONS:

5.1) General:

Generally uniform conditions were found to prevail over the site area. Subsoil consists of deposits of loose to dense sandy silt to silt, soft to firm clayey silt to silty clay and stiff to hard clayey silt overlying shale bedrock. The boundaries between the various soil strata are shown on the Record of Borelog sheets contained in the Appendix of the report. The estimated stratigraphical profile shown on Drawing #66-F-76A, is based upon this information. From ground level downwards, the different soil types are described in detail as follows:

cont'd. /4 ...

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

5.2) Sandy Silt to Silt:

From the ground surface to depths ranging from 9 to about 16 feet, the subsoil consists of a granular deposit of sandy silt to silt. Traces of clay were also evident. Standard Penetration tests carried out in the field, gave 'N' values ranging from 1 - 43 blows/ft., indicating a very loose to dense relative density; the average condition, however, is compact. The average moisture content is in the order of 20%.

5.3) Clayey Silt to Silty Clay:

This deposit was observed in all borings and underlies the sandy silt to silt stratum. It extends for depths ranging from 16 feet in B.H.'s 4 and 7, to 25 feet in B.H. #8. The material consists of clayey silt to silty clay with thin seams and pockets of silt and contains also, traces of fine gravel and coarse sand. Some variation in physical properties was observed to occur in a random fashion. These physical properties, as determined from field and laboratory tests, are summarized below:

	<u>Min.</u>	<u>Max.</u>	<u>Average</u>
Liquid Limit	19%	55%	32%
Plastic Limit	16%	27%	20%
Moisture Content	13%	53%	30%
Bulk Density	103 p.c.f.	128 p.c.f.	115 p.c.f.

Undrained Shear Strength

	<u>Min.</u>	<u>Max.</u>
Unconfined Comp. Test	219 p.s.f.	817 p.s.f.
Triaxial Comp. Test	300	920
Field Vane Test	320	960
Lab. Vane Test	282	974
Sensitivity	4	8

Plots of the shear strength test results are shown in the Appendix of this report. It is estimated that the

cont'd. /5 ...

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

5.3) Clayey Silt to Silty Clay: (cont'd.) ...

average undrained shear strength of the overall deposit is in the order of 500 - 600 p.s.f. For design purposes, a value of 500 p.s.f. is recommended.

Based on the foregoing, the deposit is classified as soft to firm clayey silt to silty clay.

5.4) Clayey Silt:

Underlying the clayey silt to silty clay stratum, a deposit of clayey silt with sand and traces of gravel was observed to extend for depths ranging from 22 feet in B.H. 8 to 41 feet in B.H.'s 4 and 7. The material consists essentially of cohesive clayey silt, and has a generally stiff to hard consistency. Standard Penetration tests gave values which ranged from 9 to 35 blows/ft. Field vane tests were possible in very few instances only, and the average undrained shear strength is estimated to be well in excess of 2000 p.s.f.

Physical properties of the material as determined from field and laboratory tests, are summarized as follows:

Liquid Limit	18% - 48%
Plastic Limit	16% - 24%
Moisture Content	13% - 40%
Bulk Density	125 p.c.f.- 136 p.c.f.

Mechanical analyses of samples from the deposit indicated the following average grain-size distribution: gravel 3%, sand 18%, silt 44%, and clay 35%.

In B.H.'s 1 and 7 a layer of dark-coloured, coarse sand some 1 - 3 feet thick, was encountered near the bottom of the main deposit. This layer was found to contain natural gas under a fairly high pressure. On being intersected by the boreholes, however, the gas was released and the pressure dissipated within a few minutes.

cont'd. /6 ...

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

5.5) Bedrock:

Below the stiff to hard clayey silt stratum and parted from it by 2 - 3 feet of dense silt, is shale bedrock. Rock core samples obtained show the rock to be generally sound right from the surface.

6. GROUNDWATER:

Observations carried out during and after drilling operations, indicated the groundwater level to lie at elevations ranging from 3 to 8 feet below the ground surface (i.e., approx. el. 582.0).

7. DISCUSSION AND RECOMMENDATIONS:

It is proposed to construct a bridge at this site to carry Hwy. #2 across the C.N.R. tracks. At this time, no decision has been made as to whether the new crossing will be an overhead structure or a subway. In the event that an overhead structure is constructed, approach fills having a maximum height of about 35 feet will be required. In the case of a subway, a cut section some 21 feet deep will be required. Both types of structure are discussed below:

7.1) Overhead Structure and Approaches:

The field investigation has revealed the presence of a soft to firm layer of cohesive soil some 8 feet below the ground surface. The maximum thickness of this layer is about 30 feet and the average undrained shear strength is about 500 p.s.f. The presence of such a layer precludes the possibility of a conventional spread footing type foundation: the proposed structure must therefore be supported on a piled foundation. It is recommended that a piled foundation utilizing steel H-piles driven to bedrock be constructed and that the maximum allowable load for the particular pile section be assumed for design purposes. Piles should

cont'd. /7 ...

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

7.1) Overhead Structure and Approaches: (cont'd.) ...

be designed to resist all lateral forces induced by the earth fill embankments on the structure. Excavations carried out in the sandy silt subsoil which extends from the ground surface may be subject to excessive water seepage due to the relatively high groundwater level. The soil itself, is susceptible to conditions of unbalanced hydrostatic head and is likely to 'boil' under such conditions. Dewatering may be carried out efficiently by driving interlocking steel sheeting into the underlying clay layers thus sealing the excavations.

The proposed approach embankments which have a maximum height of about 35 feet are the major problems of this scheme. Due to the existence of the soft clay layers, fills in excess of 15 feet in height constructed with standard 2:1 slopes, will not be stable. Stability analyses in terms of total stresses have been carried out with the following assumptions:

FILL MATERIAL - COHESIVE EARTH FILL ($\phi = 0$)

Bulk Density	γ = 135 p.c.f.
Undrained Shear Strength	C = 700 p.s.f. (Partial mobilization only)

SUBSOIL - SANDY SILT (el. 585 - el. 577)

Bulk Density	γ = 130 p.c.f.
Angle of Friction	ϕ = 30°

SUBSOIL - CLAYEY SILT TO SILTY CLAY (el. 577 - el. 546)

Bulk Density	γ = 120 p.c.f.
Undrained Shear Strength	C = 500 p.s.f.

SUBSOIL - CLAYEY SILT (el. 546 and below)

Bulk Density	γ = 130 p.c.f.
Undrained Shear Strength	C = 2000 p.s.f.

With regard to the fill material, it should be noted that the assumed value of 700 p.s.f. for the undrained shear strength represents the strength mobilized at the particular strain when the underlying sensitive soft subsoil layers are undergoing shear failure. The fill material, properly constructed, should

cont'd. /8 ...

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

7.1) Overhead Structure and Approaches: (cont'd.) ...

probably have a low sensitivity, and an undrained shear strength in excess of 1500 p.s.f.

As a result of the stability analyses, the following conclusions have been drawn:

(1) Fills less than 15.0 feet in height may be constructed with 2:1 side slopes.

(2) Fills in excess of 15.0 feet in height should be constructed with half-height berms. For the maximum fill height of 35.0 feet, a berm 120 feet long will be required. For fills between 15 and 35 feet, the required berm length may be calculated by interpolation. The surface of the berms should slope away from the fill at a gradient of 30:1 for drainage purposes. Details of the berms are shown on Fig. 1 of the Appendix.

(3) Due to the fact that the height of the fill in the forward direction is about 30 feet, berms of length about 90 feet will be required in front of the abutments. This will mean that the length of the bridge will be some 180 feet greater than was originally proposed.

The underlying soft, clayey silt to silty clay layers will undergo settlement due to consolidation over a long-term period under the weight of the approach embankments. It is estimated that the magnitude of this settlement will be in the order of 24 to 30 inches. It is therefore recommended that the final paving be delayed for as long a period as is possible, and that a flexible type pavement be constructed. In any event, the immediate approach to the structure will present a permanent maintenance problem and will require re-paving from time to time.

cont'd. /9 ...

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

7.2) Subway Structure and Approaches:

As an alternative to the overhead structure, the possible construction of a subway has also been considered. This solution has some advantages in that a cut of maximum depth of only 21 feet is required: however, other problems manifest themselves - namely, the question of dewatering the excavations and keeping them permanently drained to ensure stability. A railway detour will be required.

As for the overhead structure, the abutments and piers of the subway should be supported on steel H-piles driven to bedrock.

Stability analyses in terms of total stresses, have been carried out in order to assess the end-of-construction case. The long-term stability has also been considered. It is concluded that in order to permanently stabilize the approach cuts, the following steps must be taken:

(1) The sides of the subway approaches must be constructed with benches of minimum width 30 feet where the depth of cut is 21 feet.

(2) Cuts less than 10 feet in depth may be constructed with 2:1 slopes and without benches.

(3) Where the depth of the cut is between 10 and 21 feet the width of the berm may be calculated by interpolation. Details of the benches are shown on the attached Fig. 2, included in the Appendix.

(4) Great care must be taken to ensure that groundwater is prevented from seeping through the surface of the slopes and benches both during and after construction. For the latter purpose it will be necessary to construct permanent filters some 2 feet thick, and to provide an adequate drainage system. If the foregoing precautions are not taken, the slopes are likely to become unstable.

cont'd. /10 ...

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

7.2) Subway Structure and Approaches: (cont'd.) ...

The final preparation of the roadway in the cut is likely to present some problems. The surface of the exposed clay will be very easily churned up by construction equipment. A minimum thickness of 2.0 feet of granular base is recommended. For the foregoing reasons, however, the final thickness may well exceed 2.0 feet.

Due to the fact that both of the above mentioned schemes are quite expensive, it is suggested that another site be found if at all possible, where the subsoil conditions are more favourable.

8. SUMMARY:

A foundation investigation at the site of the proposed crossing of the C.N.R. and Hwy. #2, Revision Line 'C' near Chatham, Ontario, is reported.

Subsoil at the consists generally of about 8 feet of loose to dense silty sand followed by up to 30 feet of soft to firm clayey silt to silty clay followed by up to 30 feet of very stiff clayey silt followed by shale bedrock.

For the new structure, two schemes (1) an overhead; and (2) a subway, have been considered.

In both schemes a piled foundation for the structure is recommended.

In the case of the subway, benched cuts will be necessary.

In the case of the overhead, berms will be required for the embankments.

cont'd. /11 ...

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

Because of the expensive measures necessitated by the presence of the soft clay layers in either of the two schemes, it is recommended that another site for the crossing be found if at all possible.

9. MISCELLANEOUS:

The field investigation was carried out during the period September 7 - 26, 1966. Equipment used was owned and operated by Master Soil Investigation Ltd. Field supervision was carried out by Mr. A. Seppala, Project Foundation Engineer who also prepared this report, under the general supervision of Mr. K. G. Selby, Supervising Foundation Engineer.

November 1966

APPENDIX I

FOUNDATION SECTION

CHECKED BY 94K

[illegible]

DEPARTMENT OF HIGHWAYS - ONTARIO

RECORD OF BOREHOLE NO. 3

FOUNDATION SECTION

MATERIALS & TESTING DIVISION

JOB 66 - F - 76

LOCATION Station 84 + 63 and 14' o/s to Lt.

ORIGINATED BY A.M.S.

W.P. 13 - 63

BORING DATE September 13 & 14, 1966

COMPILED BY A.M.S.

DATUM Geodetic

BOREHOLE TYPE NX & BX Casing Wash Bore & Rock Core.

CHECKED BY

[illegible]

FOUNDATION SECTION

MATERIALS & TESTING DIVISION

LOCATION Station 83 + 53 on E

ORIGINATED BY A.M.S.

BORING DATE September 14 & 15, 1966

COMPILED BY A.M.S.

BOREHOLE TYPE NX & BX Casing, Wash Bore & Rock Core.

CHECKED BY AK

SOIL PROFILE			SAMPLES			DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT					LIQUID LIMIT ——— WL PLASTIC LIMIT ——— WP WATER CONTENT ——— W			BULK DENSITY P.C.F.	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT	25	50	75	100	125	WATER CONTENT % 20 40 60				
590.5	Ground Level														
0	Black fill (Cinders)														
	Fine sandy silt to silt		1	SS	4										
			2	SS	12										
	Loose to dense		3	SS	35										
578.0			4	SS	4										
12.5	Clayey silt to Silty clay		5	TW	PM									122	
	with		6	TW	PM									124	
	Layers of clay and silt.		7	TW	PM									118	
	Soft to firm.		8	TW	PM									122	
561.5			9	TW	PM									104	0 3 27 70
29.0			10	TW	PM									132	5 24 45 26
	Clayey silt with occasional fine gravel and coarse sand.		11	SS	31										4 31 47 18
	Very stiff to hard.		12	SS	25										
			13	SS	31										
			14	ss	23										
			15	SS	18										
			16	TW	PM									127	2 17 44 37
			17	tw	pm 9"										
			18	SS	22									134	11 28 34 27
520.5			19	SS	18										
70.0	Silt with trace of clay (Compact).		20	RC	AXT 100% Rec.										
517.4	Shale Bedrock (Sound)														
73.1															
512.4															
78.1															

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & TESTING DIVISION

JOB 66 - F - 76

w.p. 13 - 63

DATUM Geodetic

LOCATION Station 82 / 82 and 18' o/s to Lt.

BORING DATE September 16 & 19, 1966

BOREHOLE TYPE NX Casing & Wash Bore

FOUNDATION SECTION

ORIGINATED BY A.M.S.

COMPILED BY A.M.S.

CHECKED BY

[illegible]

FOUNDATION SECTION

CHECKED BY [Signature]

[illegible]

MATERIALS & TESTING DIVISION

FOUNDATION SECTION

ORIGINATED BY A.M.S.

COMPILED BY A.M.S.

CHECKED BY HL

[illegible]

FOUNDATION SECTION

CHECKED BY

[illegible]

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 9

FOUNDATION SECTION

JOB 66 - F - 76 LOCATION Station 80 + 07 on E ORIGINATED BY A.M.S.
W.P. 13 - 63 BORING DATE September 22 & 23, 1966 COMPILED BY A.M.S.
DATUM Geodetic BOREHOLE TYPE NX Casing & Wash Bore CHECKED BY HL

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT					LIQUID LIMIT ——— w _L PLASTIC LIMIT ——— w _p WATER CONTENT ——— w			BULK DENSITY Y P.C.F.	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT		SHEAR STRENGTH P.S.F.					w _p ——— w ——— w _L				
							• Triaxial	• Lab. Vane	○ Unconfined	+ Field Vane						
588.7	Ground Level					500	1000	1500	2000	2500	20	40	60			
0	Fine sandy silt to silt.		1	SS	31	580									W.L. Elev. 581.5	
	Dense		2	SS	34											
576.7			3	TW	PM	570	•	•							124	
12.0	Clayey silt to silty clay, with silt seams trace of sand and gravel.	4	TW	PM												
			5	TW	PM	570		•	•						115	
			6	TW	PM											
			7	TW	PM	560		•	•						116	
	Soft to firm.		8	TW	PM											
			9	TW	PM	560		•							118	
			10	TW	PM											
554.8			11	SS	24	550	•	•							103	
33.9	Clayey silt with occasional fine gravel and coarse sand.					550									109	
547.2	Very stiff															
41.5	End of Borehole.														127	

W.L.
Elev. 581.5

7

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & TESTING DIVISION

108 66 - P - 76

W P 13 - 63

DATUM Geodetic

RECORD OF BOREHOLE NO. 10

LOCATION Station 78 4 00 E

BORING DATE Sept. 23 & 25, 1966.

BOREHOLE TYPE NX Casing & Wash Bore

FOUNDATION SECTION

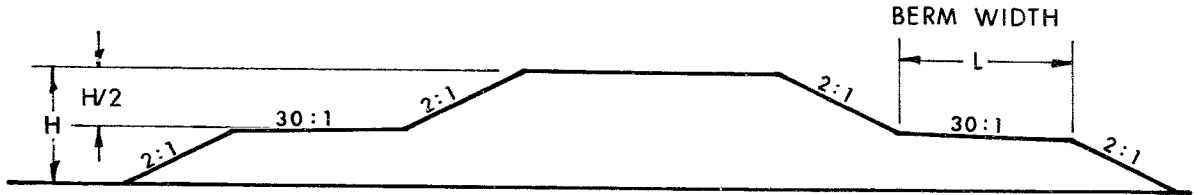
ORIGINATED BY A.M.S.

COMPILED BY **A.M.S.**

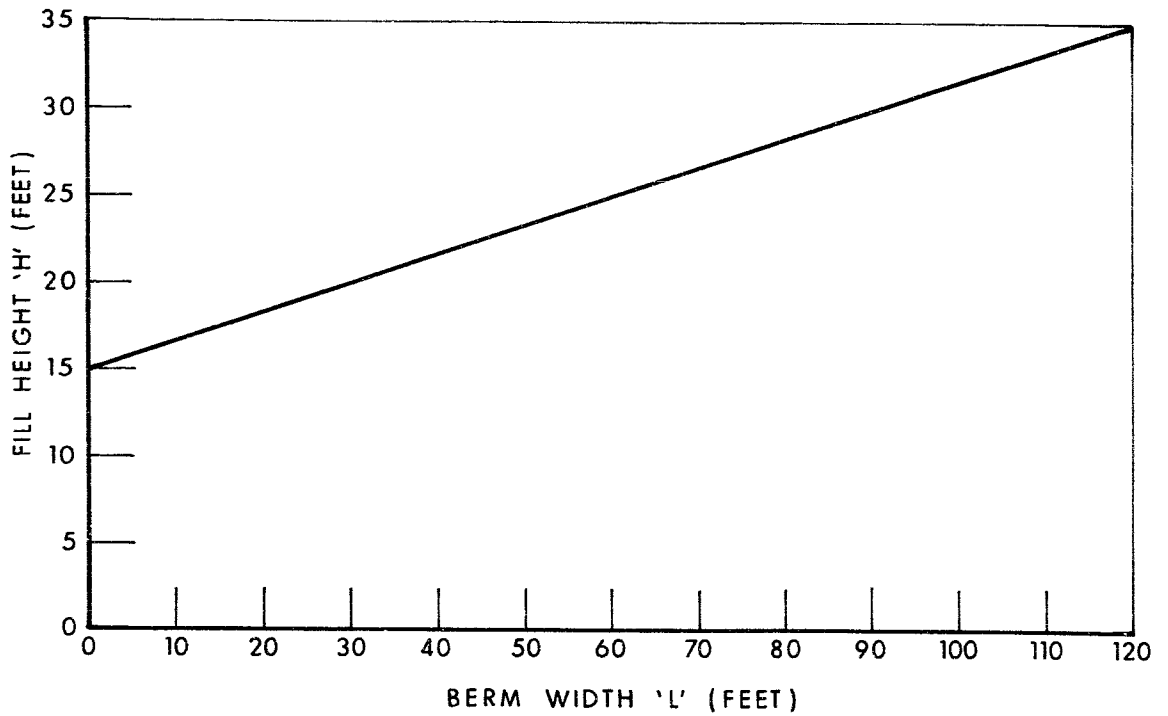
CHECKED BY

SOIL PROFILE			SAMPLES			DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT						LIQUID LIMIT ——— WL PLASTIC LIMIT ——— WP WATER CONTENT ——— W			BULK DENSITY	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT	SHEAR STRENGTH P.S.F.										
						• Triaxial • Lab. Vane ○ Unconfined + Field Vane 500 1000 1500 2000 2500						<p>WP ————— W ————— WL 20 40 60</p> <p>WATER CONTENT %</p>			γ	P.C.F. Gr. Sa. Si. Cl.
590.0	Ground Level.															
0	Fine sandy silt to silt, trace of clayey silt.		1	SS	4											
576.5			2	SS	33											
13.5	Clayey silt to silty clay, with silt seams and trace of sand and gravel		3	SS	2											
			4	TW	PM											
			5	TW	PM											
	Soft to firm.		6	TW	PM											
			7	TW	PM											
			8	TW	PM											
553.5			9	TW	PM											
36.5	Clayey silt with occasional fine gravel & coarse sand		10	SS	17											
543.5	Very stiff		11	SS	31											
46.5	End of Borehole															

FIG. 1

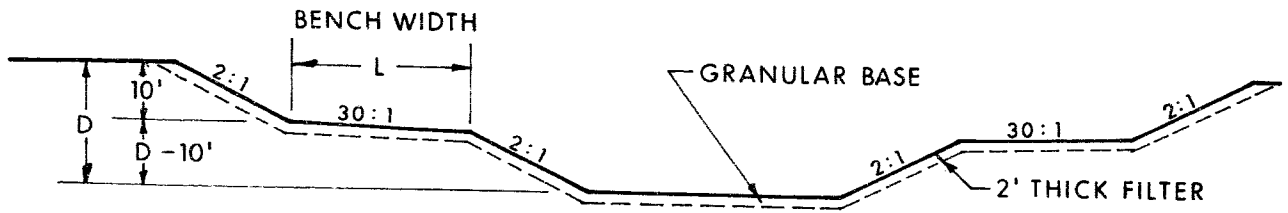


TYPICAL SECTION FOR FILLS MORE THAN 15' HIGH (ie. $H > 15'$)

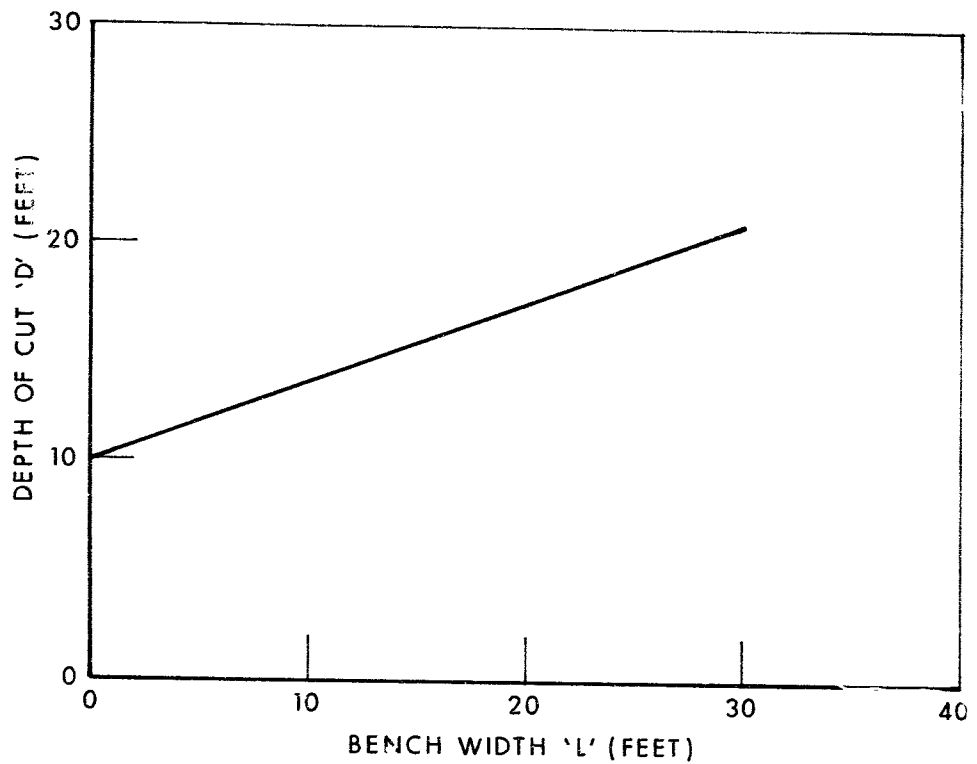


BERM WIDTH VS FILL HEIGHT

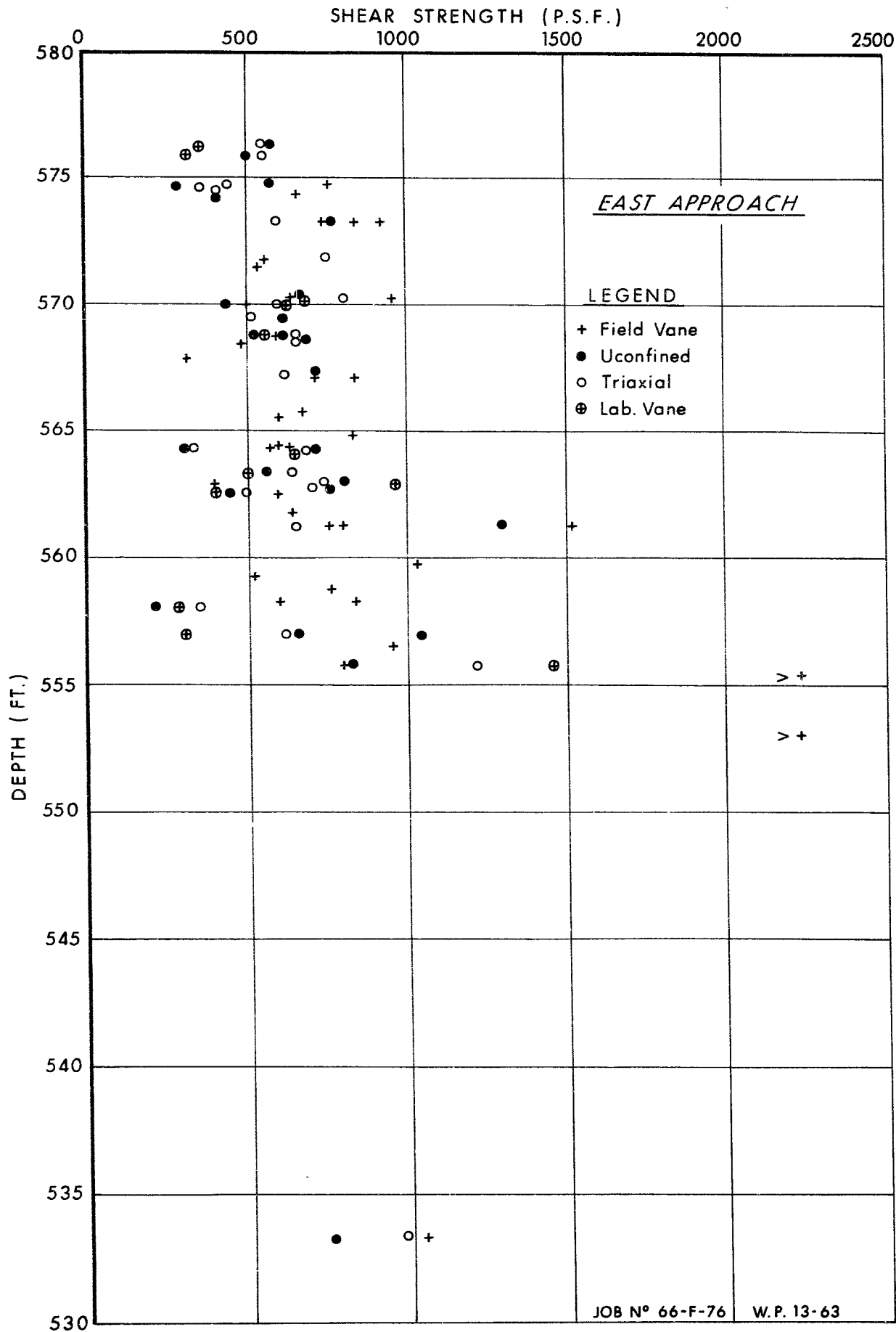
FIG. 2

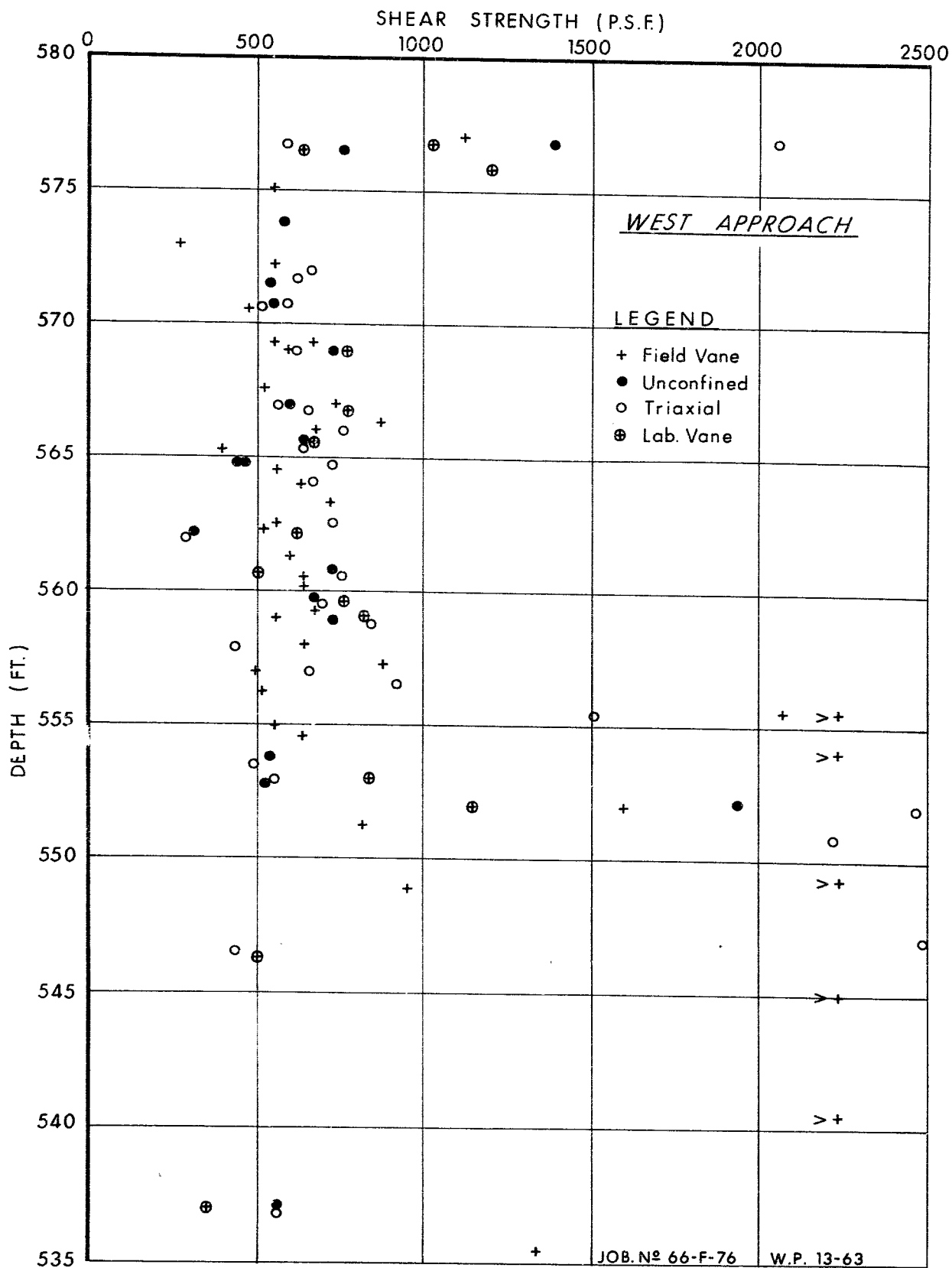


TYPICAL SECTION FOR CUTS MORE THAN 10' DEEP (i.e. $D > 10'$)

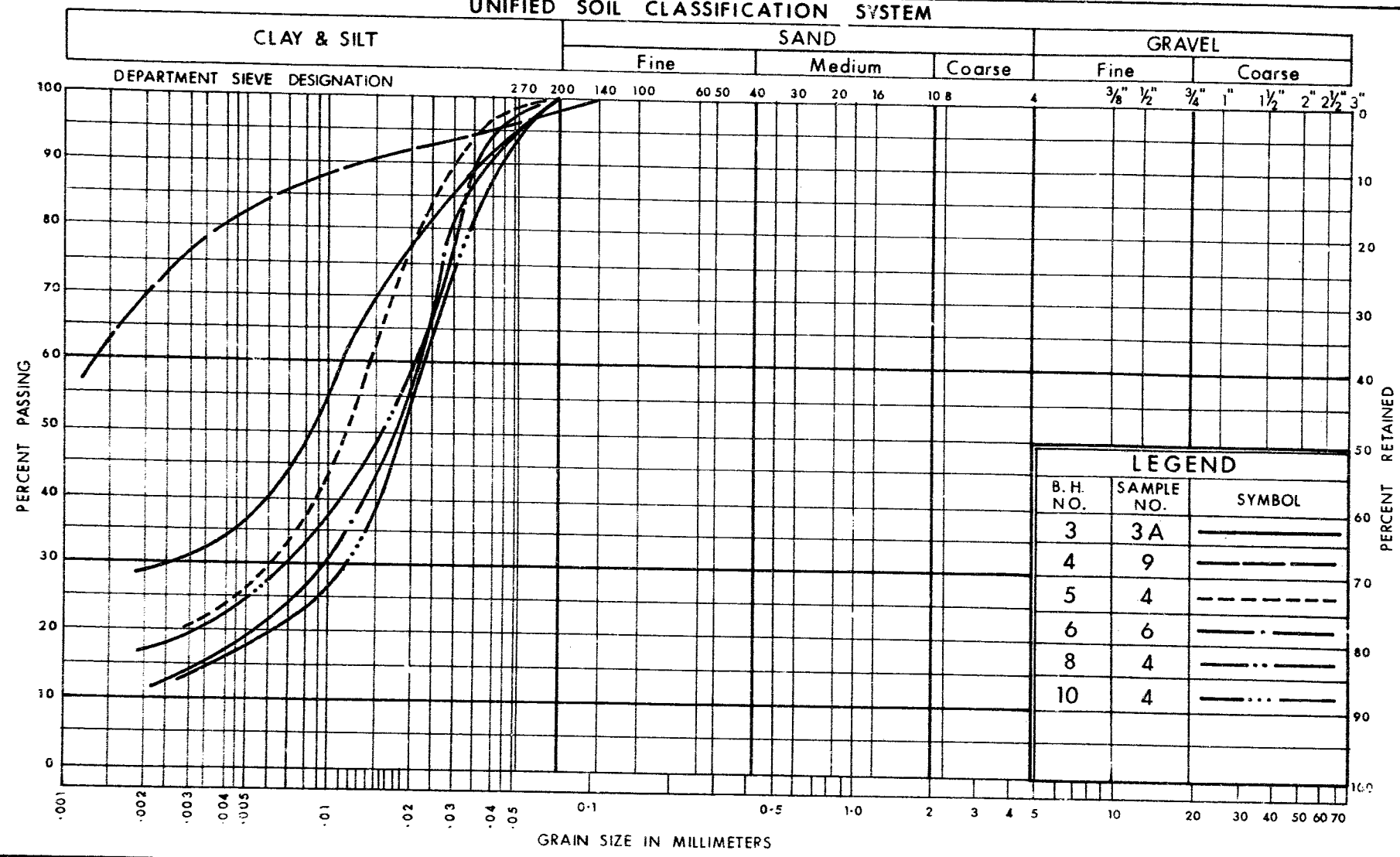


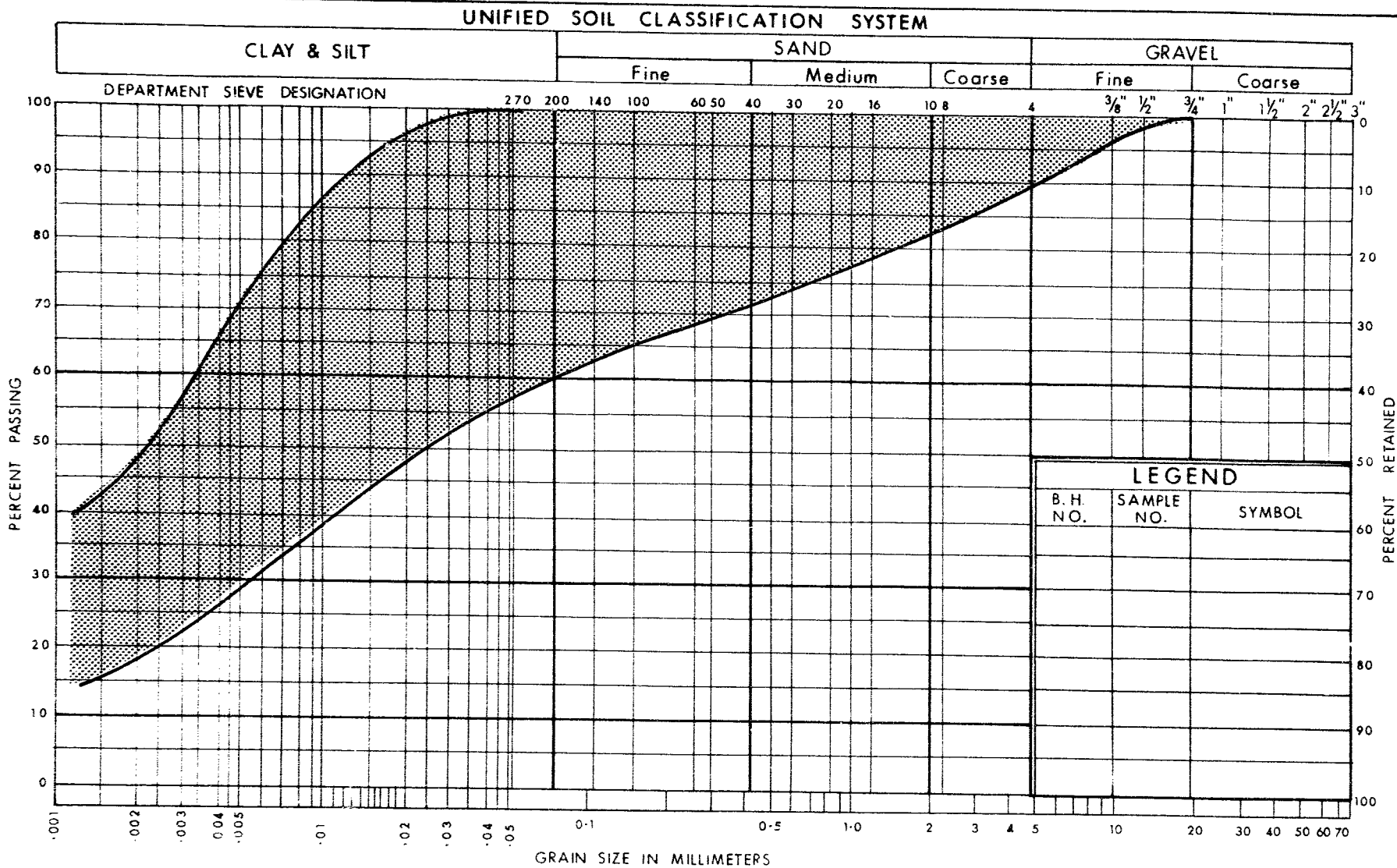
BENCH WIDTH VS DEPTH OF CUT





UNIFIED SOIL CLASSIFICATION SYSTEM





ONTARIO

DEPARTMENT OF HIGHWAYS
MATERIALS and
TESTING
DIVISION

GRAIN SIZE DISTRIBUTION

ENVELOPE OF SAMPLES

CLAYEY SILT with occ. fine Gravel & coarse Sand

W.P. No. 13 - 63

JOB No. 66-F-76

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

IN TERMS OF
EFFECTIVE STRESS
 $\tau_f = c' + \sigma' \tan \phi'$

IN TERMS OF
TOTAL STRESS
 $\tau_f = c_u + \sigma \tan \phi$

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