

#67-F-25

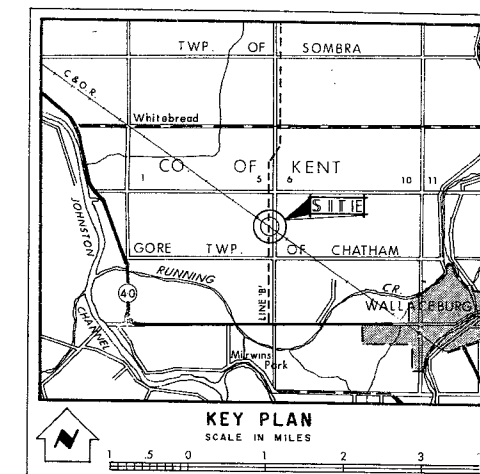
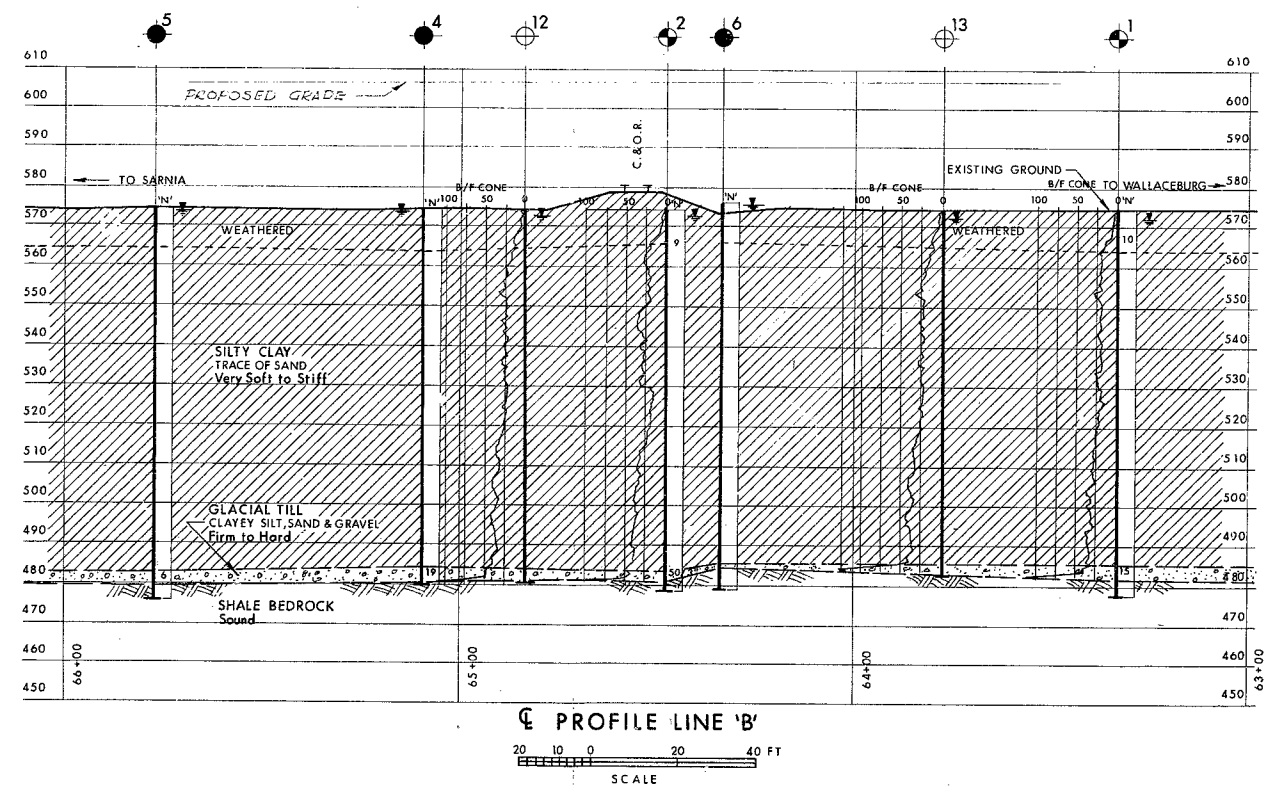
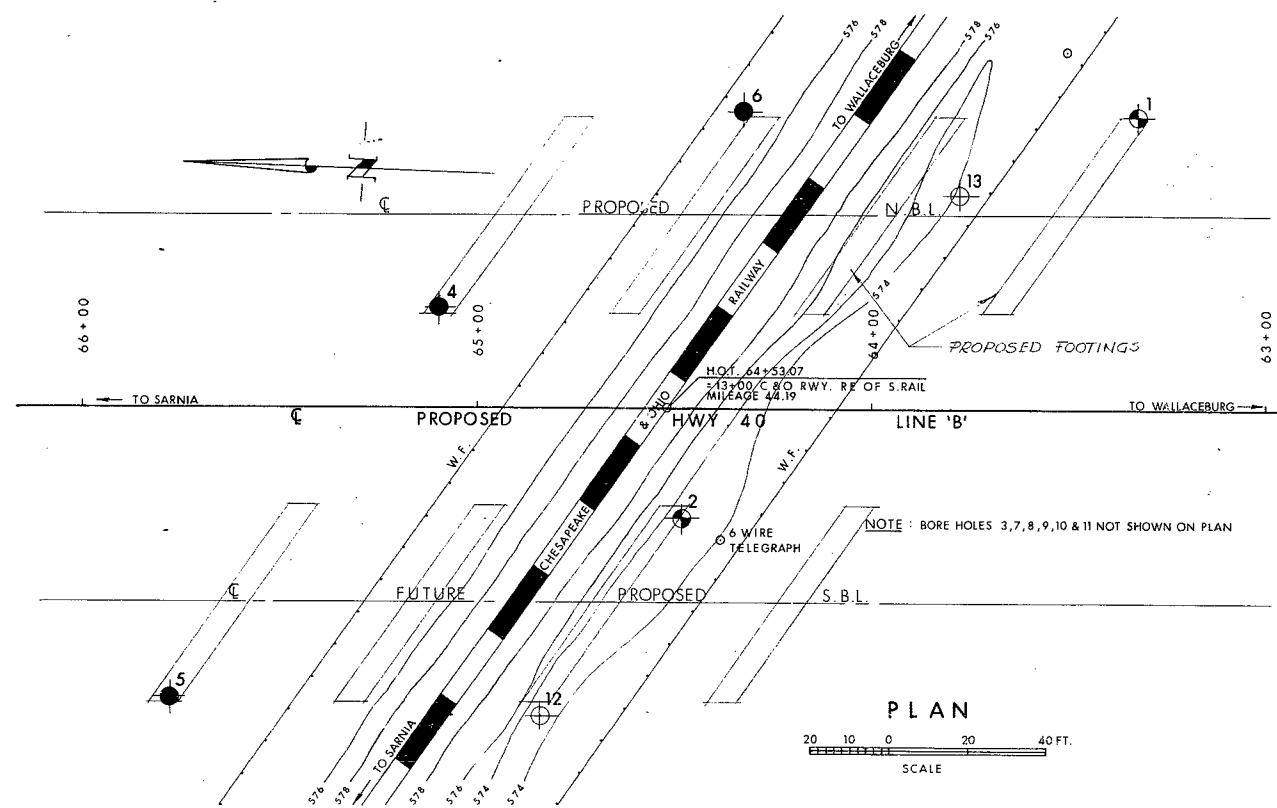
W.P. # 314-66

HWY # 40

CHESAPEAKE

& OHIO

RAILWAY



LEGEND			
	Bore Hole		
	Cone Penetration Hole		
	Bore & Cone Penetration Hole		
	Water Levels established at time of field investigation. APRIL 67.		

NO.	ELEVATION	STATION	OFFSET
1	574.4	63+34	74' RT.
2	574.5	64+49	28' LT.
3	574.5	67+10	388' LT.
4	572.2	65+11	25' RT.
5	574.2	65+79	74' LT.
6	576.0	64+34	75' RT.
7	574.2	68+97	536' LT.
8	574.0	70+00	0
9	574.5	58+00	0
10	574.5	65+89	456' LT.
11	574.5	68+56	394' LT.
12	574.5	64+85	79' LT.
13	574.0	63+79	54' RT.

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

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

CHESAPEAKE & OHIO RAILWAY

KING'S HIGHWAY NO. 40 LINE 'B' DIST. NO. 1
CO. KENT
TWP. CHATHAM GORE LOT 5 CON. 3

BORE HOLE LOCATIONS & SOIL STRATA

SUBMD. D.K. CHECKED <i>[initials]</i>	W.P. NO. 314-66	M.B.T. DRAWING NO.
DRAWN B.S. CHECKED <i>[initials]</i>	JOB NO. 67-F-25	67-F-25A
DATE 5 MAY 67	SITE NO.	BRIDGE DRAWING NO.
APPROVED <i>[signature]</i>	CONT. NO.	

PRINT RECORD	NO.	FOR	DATE

REF. E-4399-1

MEMORANDUM

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

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

Attention: Mr. S. McCombie

DATE: June 19, 1967

Our File Ref.

IN REPLY TO

JUN 22 1967

SUBJECT:

FOUNDATION INVESTIGATION REPORT
For

Proposed Overhead
New Hwy. #40, Line 'B' and
Chesapeake and Ohio Railway
District #1 (Chatham)

W.J. 67-F-25 -- W.P. 314-66

Attached, we are forwarding to you, our detailed foundation investigation report on the subsoil conditions existing at the above 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/ndef

Attach.

cc: Messrs. B. R. Davis (2)

H. A. Tregaskes

D. W. Farren

A. Gater

F. C. Brown

A. P. Watt

J. Roy

B. A. Singh

Foundations Office

Gen. Files ✓

M. D. Starnac
A. G. Starnac
PRINCIPAL FOUNDATION ENGINEER

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FOUNDATION INVESTIGATION REPORT
For

Proposed Overhead
New Hwy. #40, Line 'B' and
Chesapeake and Ohio Railway
District #1 (Chatham)

W.J. 67-F-25 -- W.P. 314-66

1. INTRODUCTION:

In a memo dated March 29, 1967, a request to carry out a foundation investigation, was received by this Section from the Regional Bridge Location Engineer (Mr. A. P. Watt).

Subsequently, an investigation was carried out at the proposed site to determine the subsoil conditions. Presented in this report are the results of field and laboratory work, together with discussion and recommendations pertaining to the design of the bridge foundations and stability of the approach embankments.

2. DESCRIPTION OF THE SITE AND GEOLOGY:

The site is located in the Township of Chatham, County of Kent, approximately 3.5 miles northwest of Wallaceburg. At the site the Chesapeake and Ohio Railway traverses in a northwest - southeast direction, and is constructed on a granular fill embankment which has a finished grade of 5 ft. above the surrounding ground surface.

Some 190 ft. east of the centre-line of the proposed highway, a drainage canal of bottom and top dimensions 37 ft. and 52 ft., respectively, traverses parallel to the proposed alignment. A five-span trestled timber bridge 62.5 ft. in length, supports the Chesapeake and Ohio railway tracks across the canal.

The site is located in the physiographic region known as the St. Clair Clay Plains which, at one time, was deeply covered by Glacial Lake Whittlesey and subsequently by Lake Warren.

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

The area is characterized by little relief and uniformity of the overburden, and is predominantly used for growing of cash crops.

The underlying bedrock was found to be dark grey shale.

3. DESCRIPTION OF FIELD AND LABORATORY WORK:

A total of 11 sampled boreholes and 5 dynamic cone penetration tests were carried out during the investigation. In the upper 20 ft. of the stratum, samples were recovered by means of 2-in. I.D. Shelby tubes advanced manually. Beyond 20-ft. depths the Osterberg Sampler was used to recover undisturbed samples.

The field work was carried out by means of a conventional diamond drill adapted for sampling purposes.

The Norwegian Vane was also used at one location to supplement the in situ undrained shear strengths obtained by the D.H.O. Vane between sampling depths.

Samples were visually examined in the field and subsequently identified in the laboratory. Laboratory tests were carried out on selected representative samples to determine, where applicable, Atterberg limits, bulk densities, grain-size distribution, natural moisture contents, undrained shear strength and consolidation characteristics.

Results of the laboratory and field tests, together with the locations and elevations of the boreholes, are presented in the appendix of this report.

4. SUBSOIL CONDITIONS:

4.1) General:

The subsoil at the site consisted predominantly of a very deep deposit of silty clay overlying a thin deposit of clayey silt with sand and gravel (till), and then shale bedrock. The various deposits as determined in the boreholes, are shown on the accompanying

cont'd. /3 ...

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

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

borelog sheets. The estimated stratigraphical profile contained on Drawing 67-F-25A, is based on this information. From ground level downwards, the various soil types encountered are described as follows:

4.2) Silty Clay with traces of Sand:

This deposit was intersected in all boreholes to depths ranging from 90 ft. in B.H. #1, to 94 ft. in B.H. #8. It consists of an extremely uniform deposit of grey silty clay. The upper 7 - 10 ft. is extensively weathered, forming a stiff upper crust. Below this, the deposit is very soft and then increases in strength with depth.

From the shear strength profiles presented in the Record of Borehole sheets, it will be noted that the field vane shear strengths are slightly higher than shear strengths obtained from laboratory quick triaxial or unconfined compression tests, a characteristic result of normal sample disturbance.

The physical properties as determined from field and laboratory tests, are summarized as follows:

Liquid Limit	36 - 54%
Plastic Limit	19 - 32%
Moisture Content	22 - 54%
Bulk Density	103 - 125 lb./ft. ³

A summary sheet which shows all measured values of shear strength plotted on a graph of shear strength versus depth, is given in the report appendix (Fig. 1). This graph also shows the shear strength profile assumed for stability calculations.

Also included is a drawing showing the relationship between in situ shear strength and depth for one borehole where the in situ strength was obtained using both the Norwegian and the D.H.O. vane (Fig. 2). The extremely close correspondence of these

cont'd. /4 ...

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

4.2) Silty Clay with traces of Sand: (cont'd.) ...

measurements show that the standard D.H.O. vane test results are as reliable as those obtained with more refined equipment normally used for research purposes (Norwegian Vane Apparatus).

A total of four consolidation tests was carried out on this material. The results of these tests are included in the report appendix.

4.3) Clayey Silt with Sand and Gravel (Till):

This deposit was intersected in all boreholes. It ranges in thickness from 1.5 ft. in B.H. #6, to 4.5 ft. in B.H. #7.

The 'N' values (No. of blows/ft. obtained in the Standard Penetration Test) ranged from 6 blows/ft. to 50 blows/3 inches, indicating a firm to hard consistency.

4.4) Shale Bedrock:

Bedrock consists of dark grey shale. The bedrock surface was found to vary from El. 476 to El. 484, or some 92 - 98.5 ft. below ground level.

Bedrock was proven by obtaining AXT-size core samples for a depth of 5 ft. in Boreholes No's 1, 3, 5, 6, 7, 8, and 10, for 1 ft. in B.H. #4, and 2 ft. in B.H. #2. Recovery ranged from 60% - 95%.

5. GROUNDWATER:

Groundwater level was observed to be between 0.5 ft. to 4.5 ft. below the ground surface. These water levels were very quickly established in the field, although the subsoil is relatively impermeable. This may have been due to the influence of the drains and the presence of the drainage canal.

The observed groundwater levels are shown on the Record of Borehole sheets and on Dwg. 67-P-25A, which are included in the report appendix.

cont'd. /5 ...

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

6.1) General:

It is proposed to construct an overhead structure for the crossing of new Hwy. #40, Line 'B' and the Chesapeake and Ohio Railway approximately 3.5 miles northwest of Wallaceburg. Present proposals call for a three-span (47'-47'-47') structure having approach fills generally 32 ft. higher than existing ground level.

Subsoil conditions consist of a very deep deposit of very soft to stiff silty clay overlying a thin deposit of clayey silt with sand and gravel (till) and then shale bedrock.

The silty clay deposit has a stiff upper crust approximately 7 - 10 ft. thick, but below this, it is of very soft consistency and then increases in strength with depth.

The presence of soft and very compressible clay at relatively shallow depth requires that steps must be taken to ensure overall stability of the approach embankments, and that the structure must be supported on piled foundations. As the stability of the approach fills is the major problem at this site, it will be discussed first.

6.2) Approach Fills:

Stability analyses in terms of total stresses, have been carried out using the following assumptions:

Fill Material - Granular

Bulk density	γ = 135 p.c.f.
Apparent cohesion in terms of effective stress	C' = 0
Angle of shearing resistance in terms of effective stress	ϕ' = 30°

Subsoil

Bulk density	γ = 120 p.c.f.
Undrained shear strength -	
Depth 0 - 10 ft.	= 1500 p.s.f.
10 - 15 ft.	= 600 p.s.f.
15 - 35 ft.	= 350 p.s.f.
35 - 50 ft.	= 500 p.s.f.
50 - 70 ft.	= 650 p.s.f.
70 - 95 ft.	= 800 p.s.f.

cont'd. /6 ...

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

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

From the stability analyses, the following conclusions have been drawn:

- 1) Fills less than 16 ft. in height may be constructed with 2:1 side slopes.
- 2) Fills in excess of 16 ft. but less than 28 ft. in height should be constructed with half-height berms. Fig. 3 contained in the report appendix shows the relationship between height of fill and berm length required. The surface of the berms should slope away from the fill at a gradient of 20:1 for drainage purposes.
- 3) Fills in excess of 28 ft. but less than 32 ft. can be constructed to conform to the design factor of safety using a double berm arrangement. A typical section for a 32-ft. high embankment having a total berm length of 215 ft., is shown in Fig. 4.

Smooth transitions between different berm requirements should be effected as the height of fill decreases from 32 ft. to 28 ft. and 28 to 16 ft.

- 4) The drainage ditch which runs parallel to the proposed alignment, in effect, increases the height of the fill. Stability calculations show that the toe of the 16-ft. embankment must be at least 90 ft. from the top of the slope of the drainage canal. For fills of 28 ft. and 32 ft., this distance should be at least 30 ft. - (Fig. 4). Hence, the alignment of the drainage ditch or of the proposed highway, will have to be revised.

The underlying soft silty clay will undergo large settlements due to consolidation over a long-term period under the weight of the approach embankments. A table of estimated settlements for various heights of approach fill is given in the report appendix (Fig. 5). It is estimated that the maximum settlement under 32 ft. height of embankment will be in the order of 7 feet of which about 40% will occur in the first 6 years.

cont'd. /7 ...

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

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

4) (cont'd.) ...

It is therefore recommended that the final paving be delayed for as long a period as is possible, and that flexible type pavement should be adopted. In any event, the immediate approach to the structure will present a permanent maintenance problem and will require re-paving from time to time.

6.3) Structure:

There are three alternatives for the structure according to the choice of fill height and the associated berm requirements. As discussed above, a 32-foot fill requires extensive berms; therefore, in order to accommodate the berms in the longitudinal direction, a multispans type of structure would be necessary.

As an alternative, the maximum fill height could be limited to 28 feet (to reduce the berm requirement) and the structure required would be a multispans bridge some 1030 feet long.

The other alternative would be to restrict the fill height to a maximum of 16 feet. This would necessitate a multispans structure some 1930 feet long.

The ultimate choice will be based on economic considerations. All three proposals are equally feasible from the foundation point of view; however, it should be pointed out that settlement problems hence, maintenance problems, are a direct function of the fill height.

6.4) Piers:

The bridge piers should be supported on steel H-piles driven to bedrock. The maximum allowable load for the particular pile section adopted, may be assumed for design purposes.

Pile caps should be founded at sufficient depth to ensure adequate frost protection.

cont'd. /8 ...

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

6.4) Piers: (cont'd.) ...

No major dewatering problems are anticipated. However, as excavations are required below groundwater level, seepage may occur and this can be dealt with by pumping from a sump.

6.5) Abutments:

The abutments should be constructed within the fills and supported on steel H-piles driven to bedrock. Since appreciable lateral movements of the subsoil beneath the embankment slope are anticipated, it is possible that due to the flexibility of H-piles approximately 110 ft. long, rotation of the abutments may occur. To prevent this, it is recommended that an additional pile should be used to support the end of each wingwall.

The maximum allowable load for the particular pile sections adopted may be assumed for design purposes.

Pile caps should be founded at sufficient depth to ensure adequate frost protection.

7. SUMMARY:

A foundation investigation at the site of the proposed crossing of Hwy. #40, Line 'B' and the Chesapeake and Ohio Railway approximately 3.5 miles northwest of Wallaceburg, is reported.

Subsoil at the site consists of a very deep deposit of very soft to stiff silty clay overlying a thin deposit of clayey silt with sand and gravel (till) and then shale bedrock.

It is recommended that the piers of the proposed structure be supported on steel H-piles driven to bedrock and the abutments should be constructed within the fills and also supported on steel H-piles driven to bedrock. Construction procedures have been outlined in this report.

cont'd. /9 ...

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

Detailed recommendations have been made regarding the procedures necessary to ensure stability of the approach fills. Berms will be required for fills in excess of 16 feet. An existing drainage canal will either need to be filled in and diverted, or else the proposed highway alignment should be revised.

In view of the excessive settlements which have been estimated, as much time as possible should be left between the construction of the approach embankments and completion of the final grade. There will be a continuing maintenance problem.

Flexible type pavement should be adopted.

8. MISCELLANEOUS:

The field work for this project was carried out during the period April 6 - May 15, 1967, under the supervision of Mr. D. Katauskas, Project Foundation Engineer. The equipment used was owned and operated by F. E. Johnston Drilling Co. Ltd.

This report was written by Mr. D. Katauskas and by Mr. A. C. Calder, Senior Foundation Engineer, and was reviewed by Mr. M. Devata, Supervising Foundation Engineer.

June 1967

APPENDIX I

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & TESTING DIVISION

67-F-25

314-66

Geodetic

DATUM Geodetic

RECORD OF BOREHOLE NO. 1

LOCATION Sta. 63 + 34 o/s 74th Rt.

BORING DATE Apr 11 13, 14 & 17, 1962

BOREHOLE TYPE Washboring

FOUNDATION SECTION

ORIGINATED BY DK

COMPILED BY

CHECKED BY

[illegible]

DEPARTMENT OF HIGHWAYS - ONTARIO

RECORD OF BOREHOLE NO. 3

FOUNDATION SECTION

MATERIALS & TESTING DIVISION

JOB 67-F-25

LOCATION Sta. 67 / 10 o/s 388' Left

ORIGINATED BY DK

W.P. 314-66

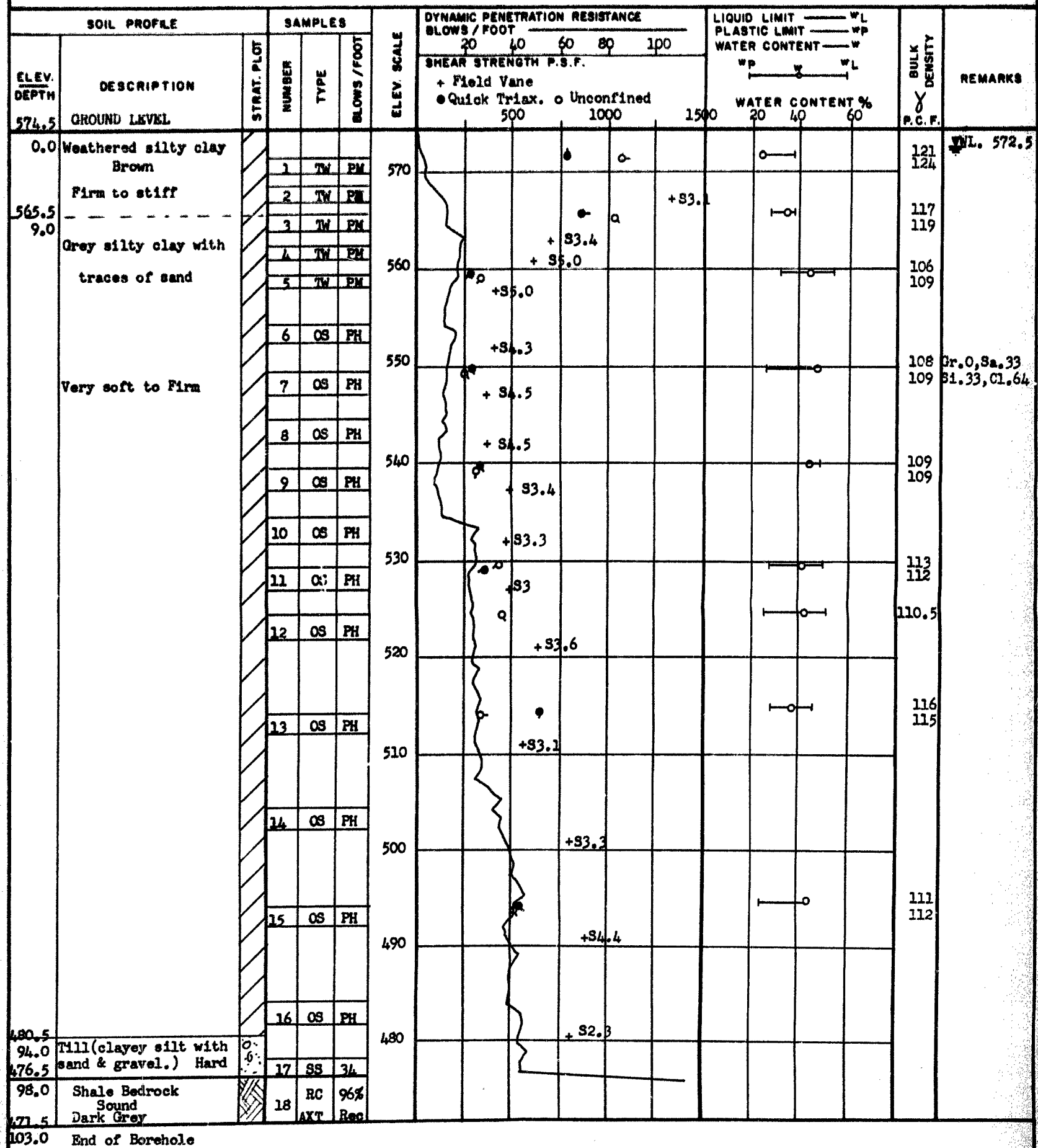
BORING DATE April 18-20, 1967

COMPILED BY DK, BQ

DATUM G eodetic

BOREHOLE TYPE Washboring

CHECKED BY



FOUNDATION SECTION

JOB 67-F-25 LOCATION Sta. 65 & 11 o/s 25' Rt. ORIGINATED BY DK
W.P. 314-66 BORING DATE April 20, 24, 25, 1967 COMPILED BY BO
DATUM Geodetic BOREHOLE TYPE Washboring CHECKED BY SR

[illegible]

FOUNDATION SECTION

CHECKED BY

[illegible]

FOUNDATION SECTION

CHECKED BY AK

[illegible]

FOUNDATION SECTION

JOB	67-2-25	LOCATION	68 & 97 n/a 536' Left	ORIGINATED BY	DK
W.P.	314-66	BORING DATE	March 1 & 2, 1967	COMPILED BY	DK
DATUM	Geodetic	BOREHOLE TYPE	Washboring	CHECKED BY	SK

[illegible]

FOUNDATION SECTION

JOB 67-F-25 LOCATION E 70 f 00 ORIGINATED BY DK
W.P. 314-56 BORING DATE May 4 & 5, 1967 COMPILED BY DK
DATUM Geodetic BOREHOLE TYPE Washboring CHECKED BY MR

[illegible]

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & TESTING DIVISION

JOB	67-E-25	LOCATION	4 58 6 00	ORIGINATED BY	DK
W. P.	324-66	BORING DATE	May 8 & 9, 1967	COMPILED BY	DK
DATUM	Geodetic	BOREHOLE TYPE	Washboring	CHECKED BY	<i>HL</i>

FOUNDATION SECTION

[illegible]

MATERIALS & TESTING DIVISION

JCB 67-25

LOCATION Sta. 65 / 89 o/s 456' Left

ORIGINATED BY _____ DK

W.P. 314-66

BORING DATE May 10, 1967

COMPILED BY _____ DK

DATUM Geodetic

BOREHOLE TYPE Washboring

CHECKED BY _____

[illegible]

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 12

FOUNDATION SECTION

JOB 67-F-25

LOCATION Sta. 64 + 85 o/s 79' Left

ORIGINATED BY DK

W.P. 314-66

BORING DATE April 12, 1967

COMPILED BY DK

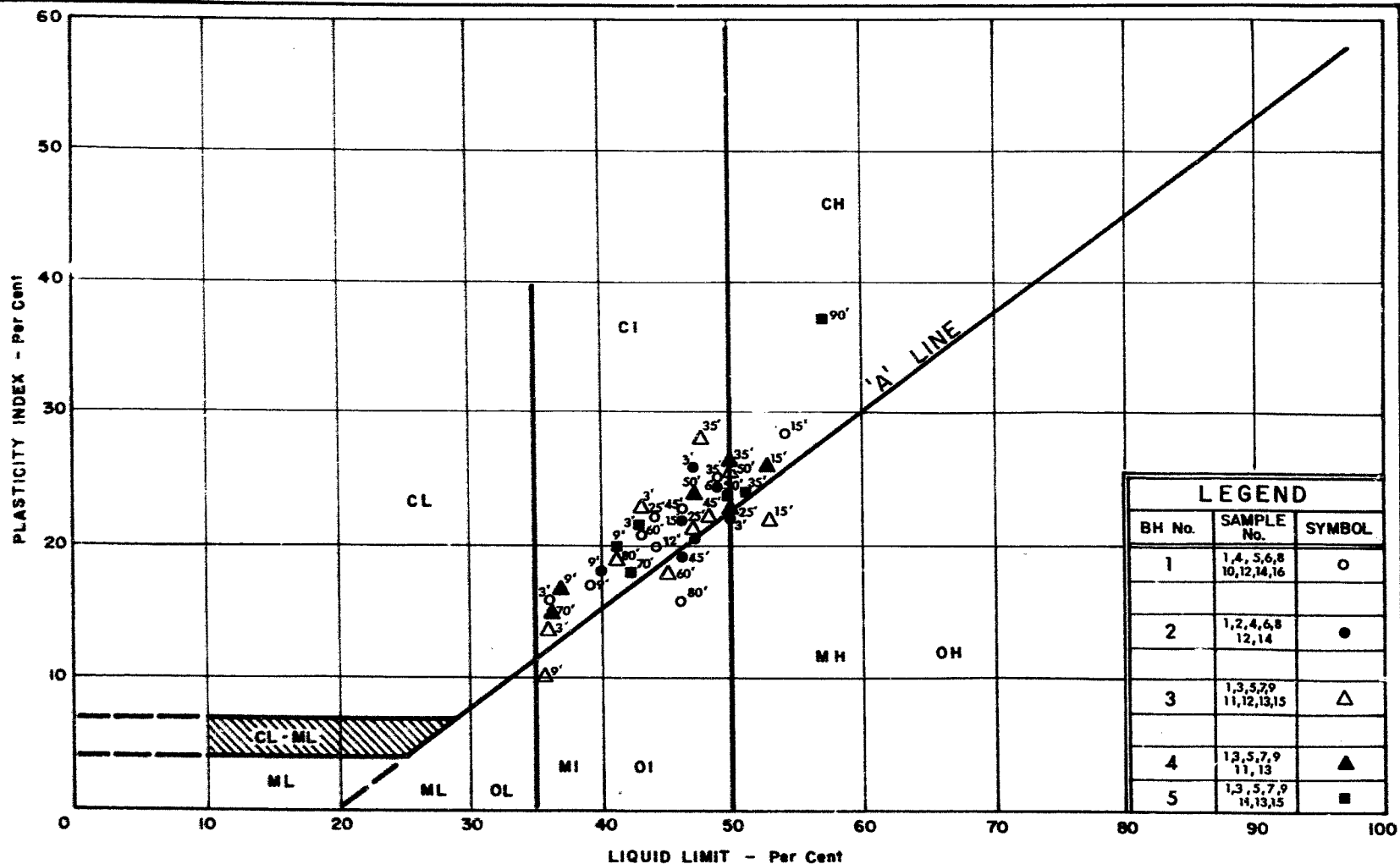
DATUM Geodetic

BOREHOLE TYPE Cone Penetration Test

CHECKED BY

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE					LIQUID LIMIT — WL		BULK DENSITY	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOW"/FOOT		BLOWS / FOOT	20 40 60 80 100		WEAR STRENGTH P.S.F.		PLASTIC LIMIT — WP	WATER CONTENT — W		
574.5	GROUND LEVEL											WP — WL			
												WATER CONTENT %			
															WL 572.5
							570								
							560								
							550								
							540								
							530								
							520								
							510								
							500								
							490								
480.7															
93.8	End of Cone Test						480								

[illegible]

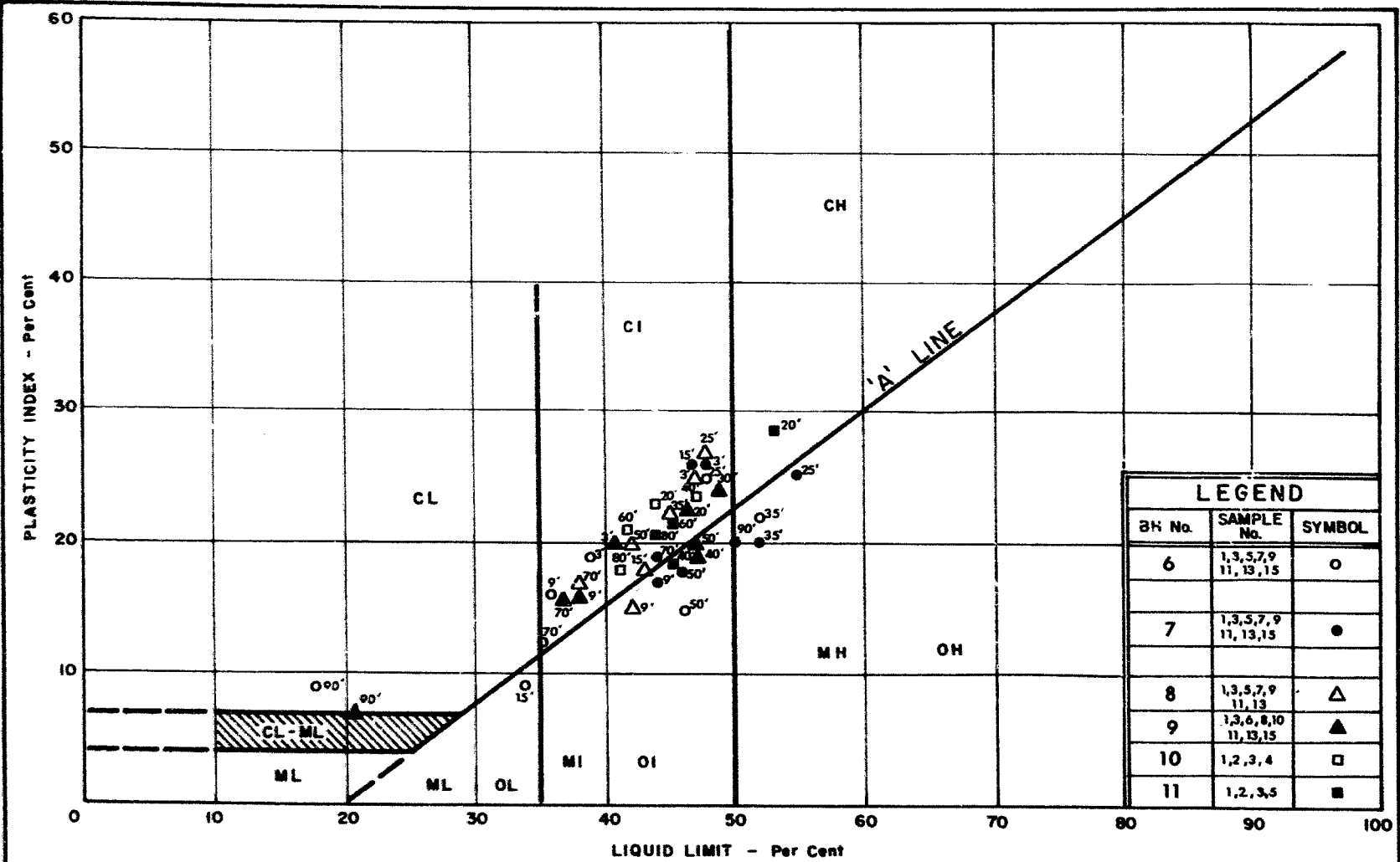


DEPARTMENT OF HIGHWAYS
**MATERIALS and
TESTING
DIVISION**

PLASTICITY CHART

WP. No. 314-66

JOB No. 67-F-25



LEGEND		
BH No.	SAMPLE No.	SYMBOL
6	1,3,5,7,9 11,13,15	○
7	1,3,5,7,9 11,13,15	●
8	1,3,5,7,9 11,13	△
9	1,3,6,8,10 11,13,15	▲
10	1,2,3,4	□
11	1,2,3,5	■

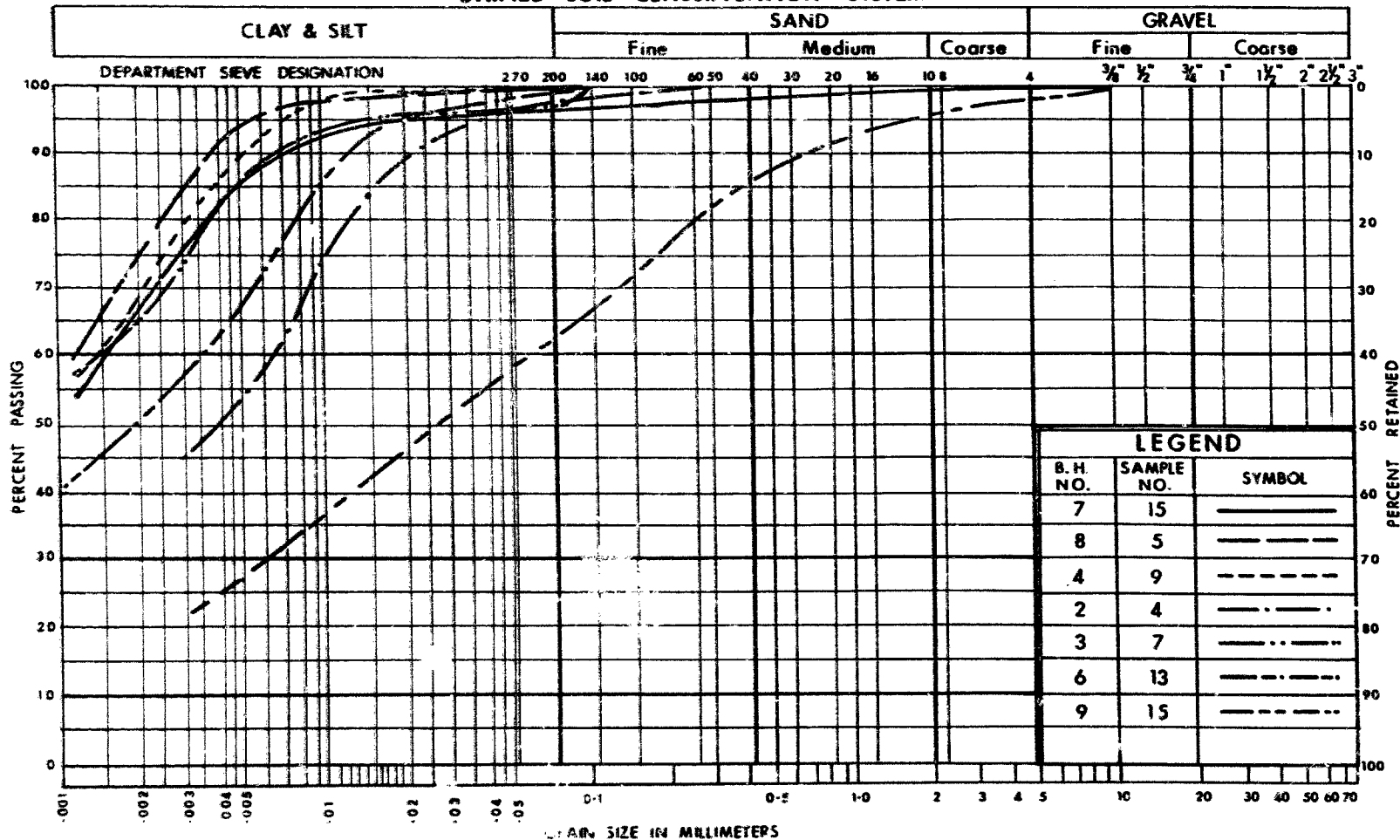


DEPARTMENT OF HIGHWAYS
MATERIALS and
TESTING
DIVISION

PLASTICITY CHART

WP No. 314 - 66
JOB No. 67 - F - 25

UNIFIED SOIL CLASSIFICATION SYSTEM

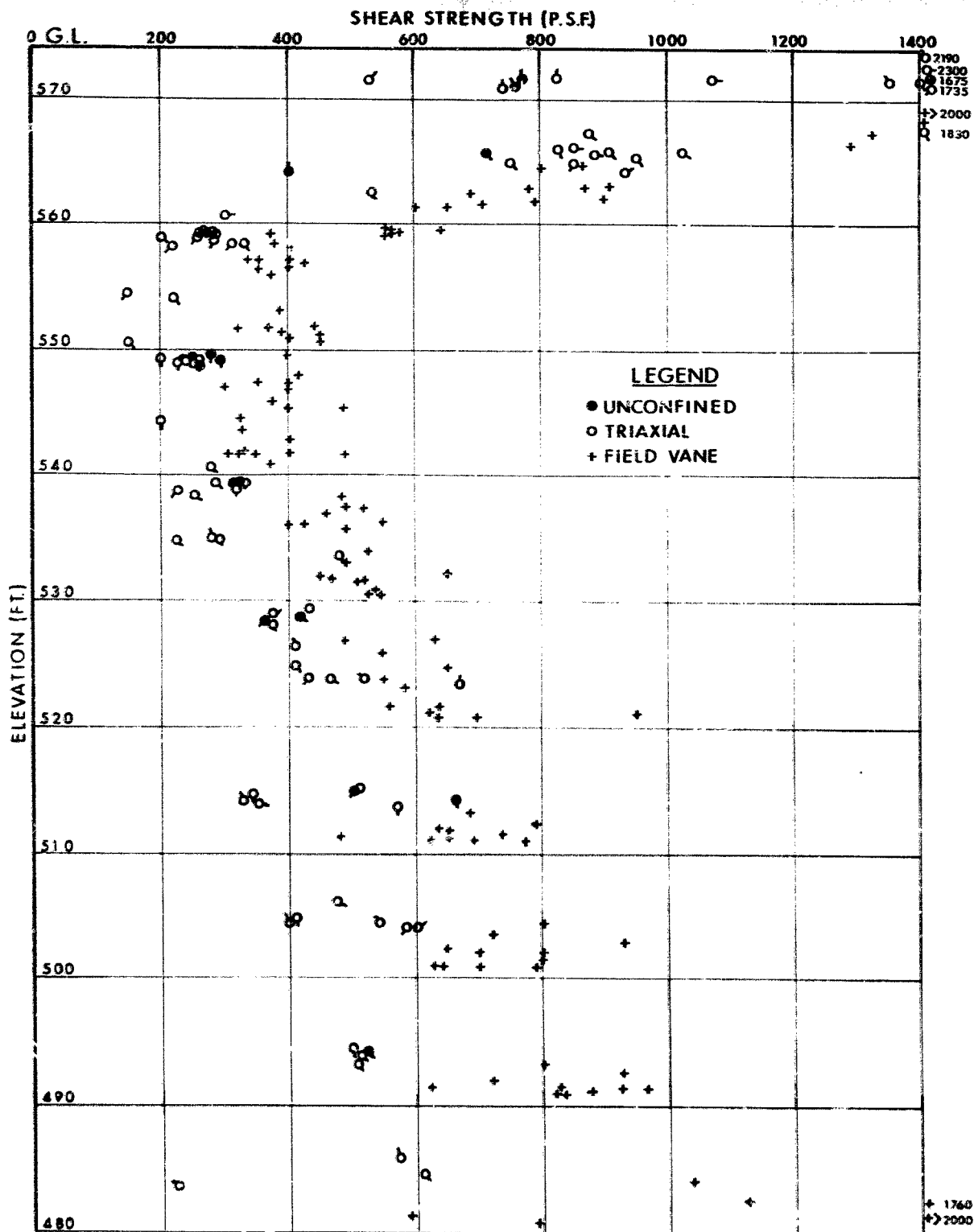


DEPARTMENT OF HIGHWAYS
MATERIALS and
TESTING
DIVISION

GRAIN SIZE DISTRIBUTION

W.P. No. 314-66

JOB No. 67-F-25



SHEAR STRENGTH PROFILE

FIG.1

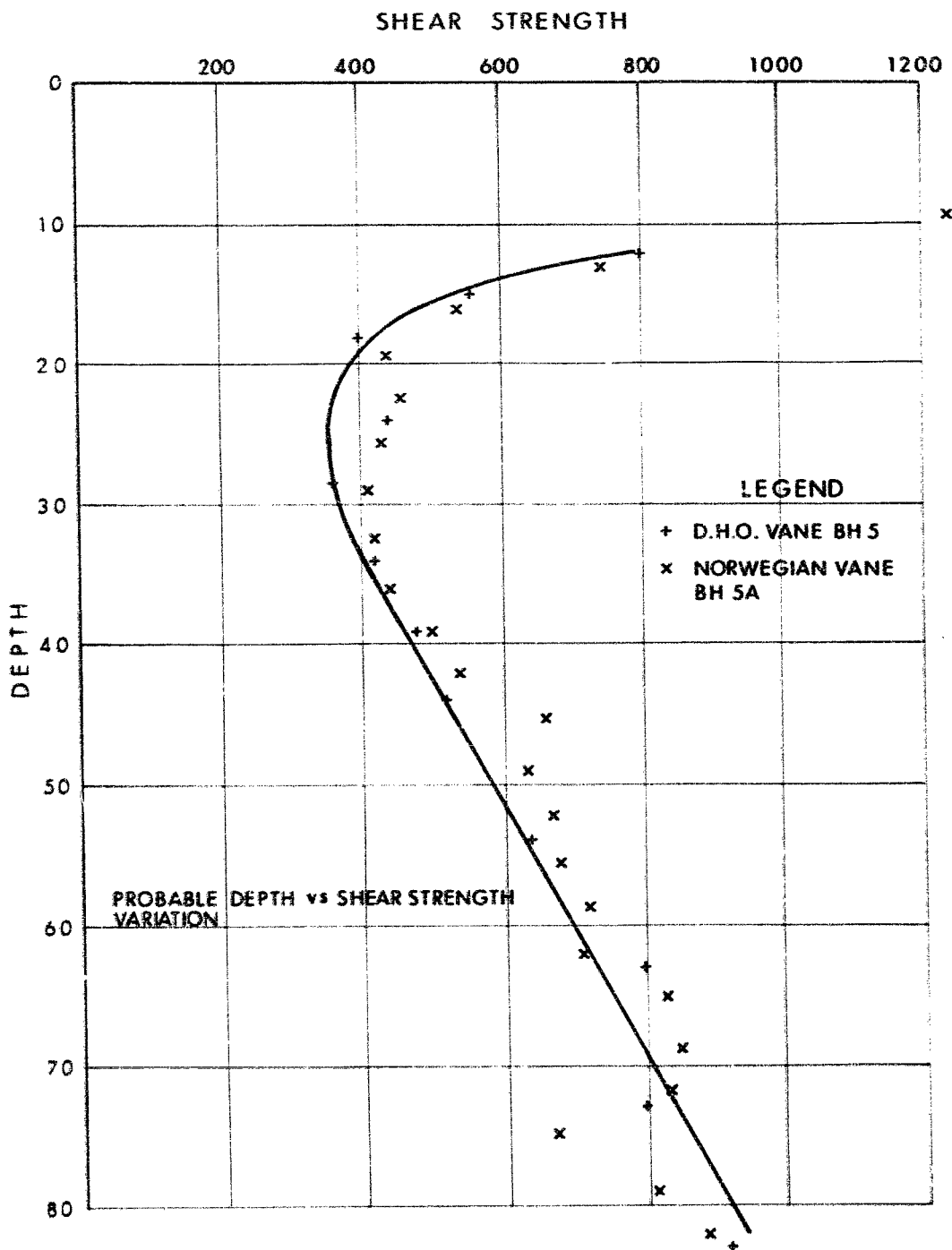


FIG. 2

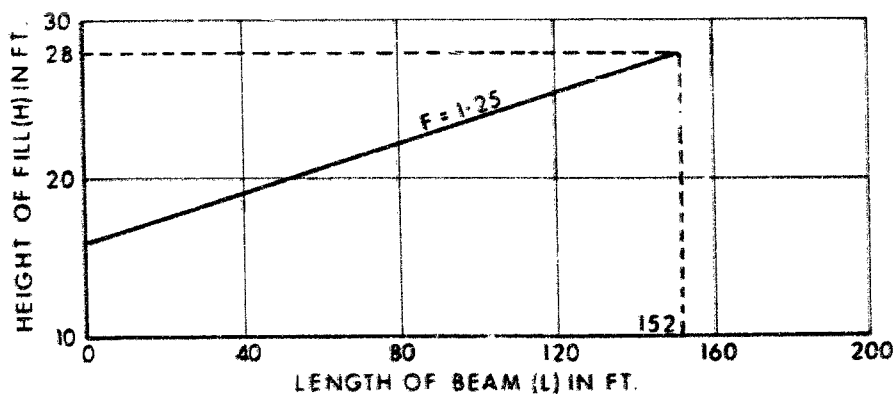
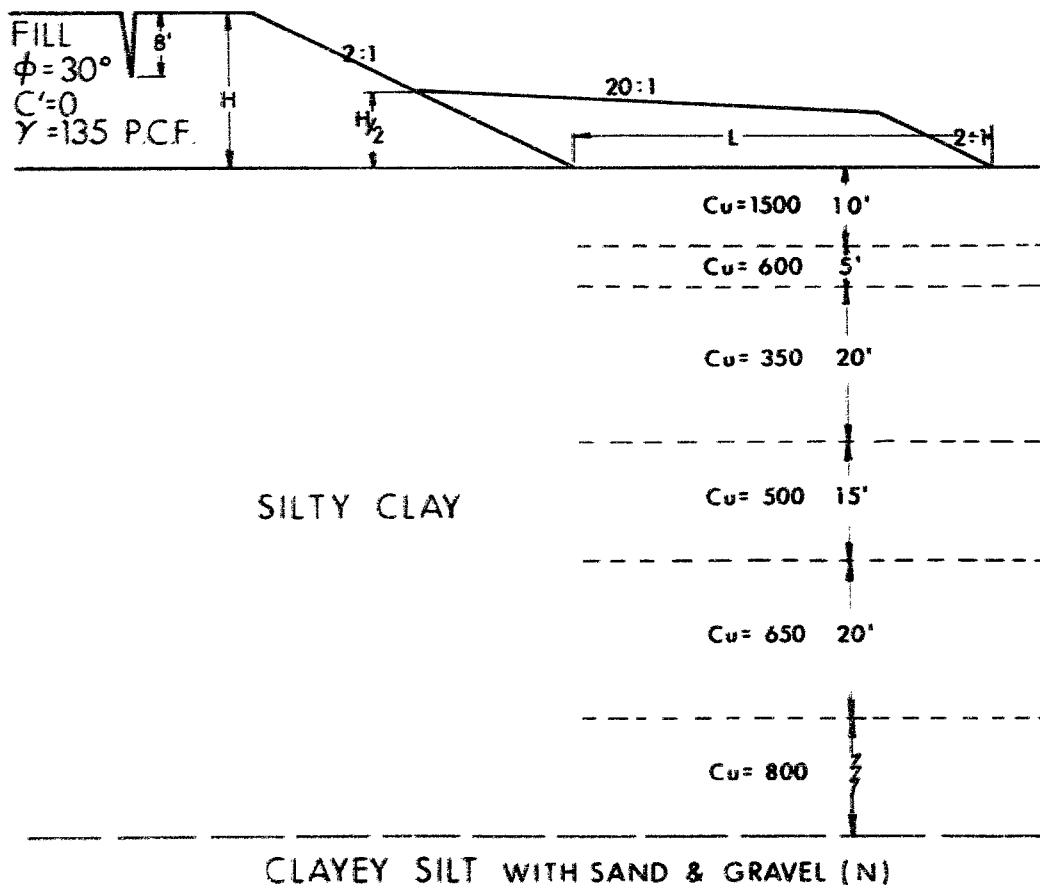
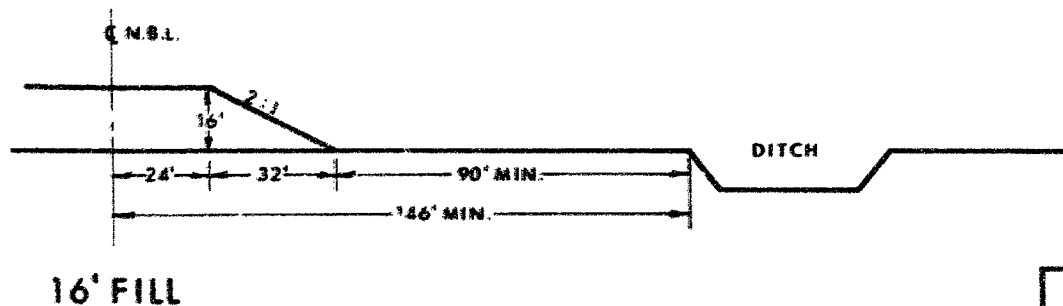
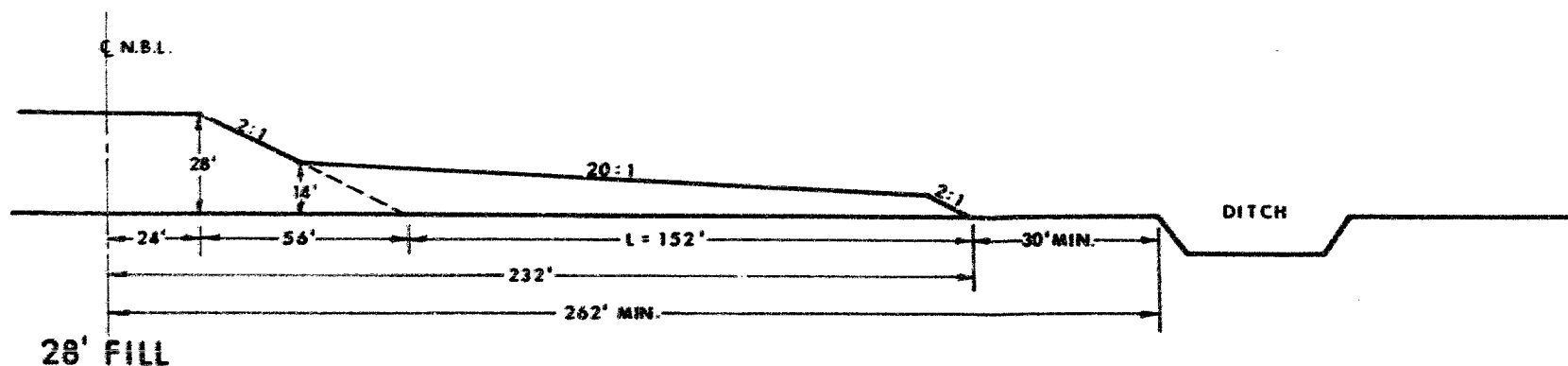
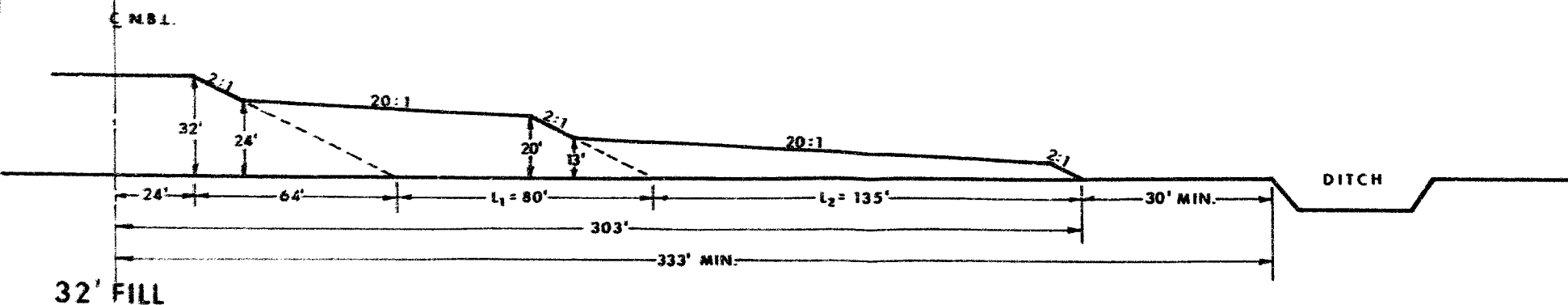



FIG. 3



SECTIONS
SCALE 1" = 40'

 ONTARIO	DEPARTMENT OF HIGHWAYS MATERIALS and TESTING DIVISION	HIGHWAY NO 40 LINE 'B' TYPICAL SECTIONS FOR 16', 28' & 32' FILLS	
	DATE 19 JUNE 1967	APPROVED <i>M. Levada</i>	WP 314-66
		DRAWING NO. FIG. 4	

SETTLEMENTS DUE TO EMBANKMENT LOADS

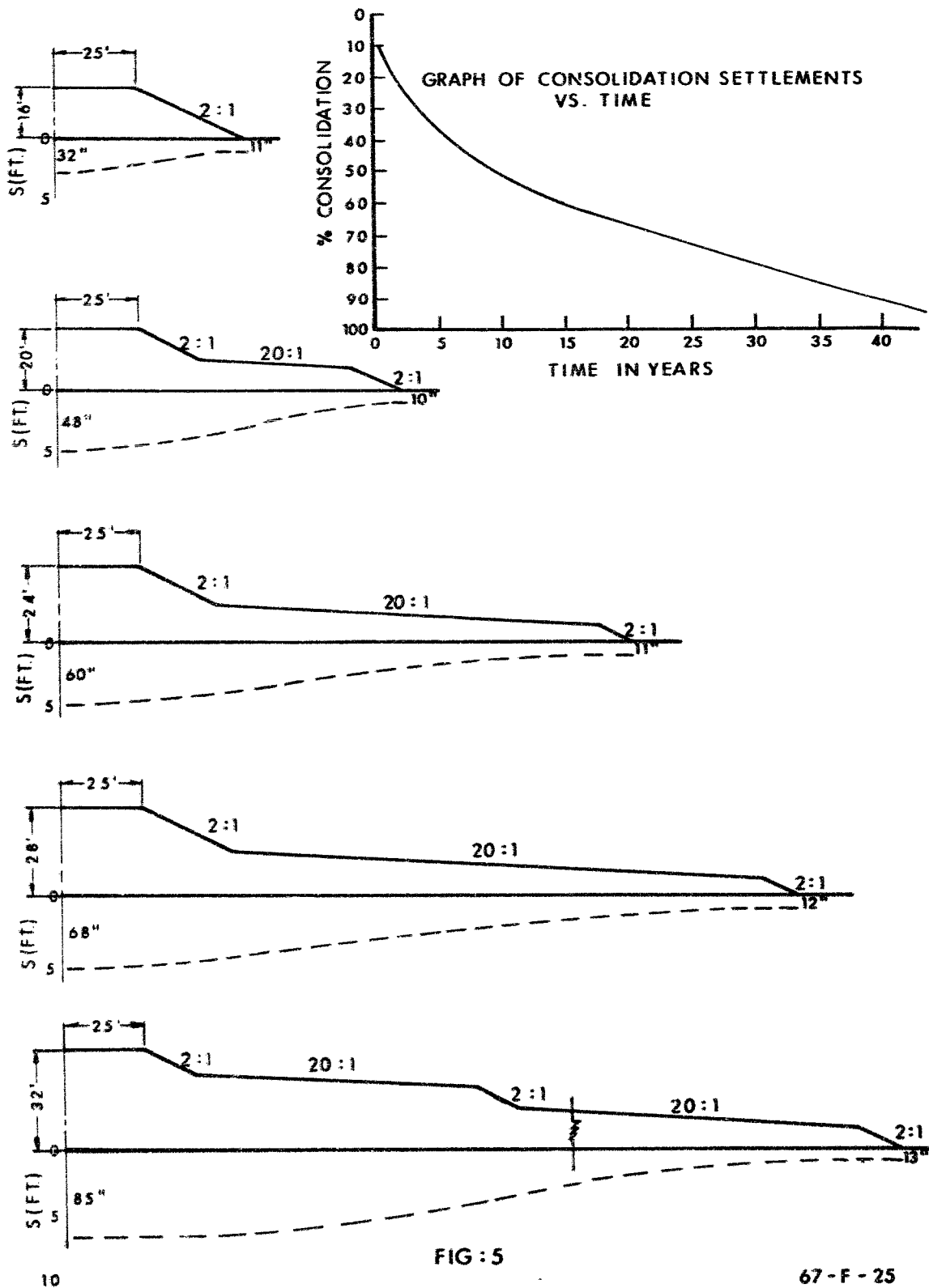
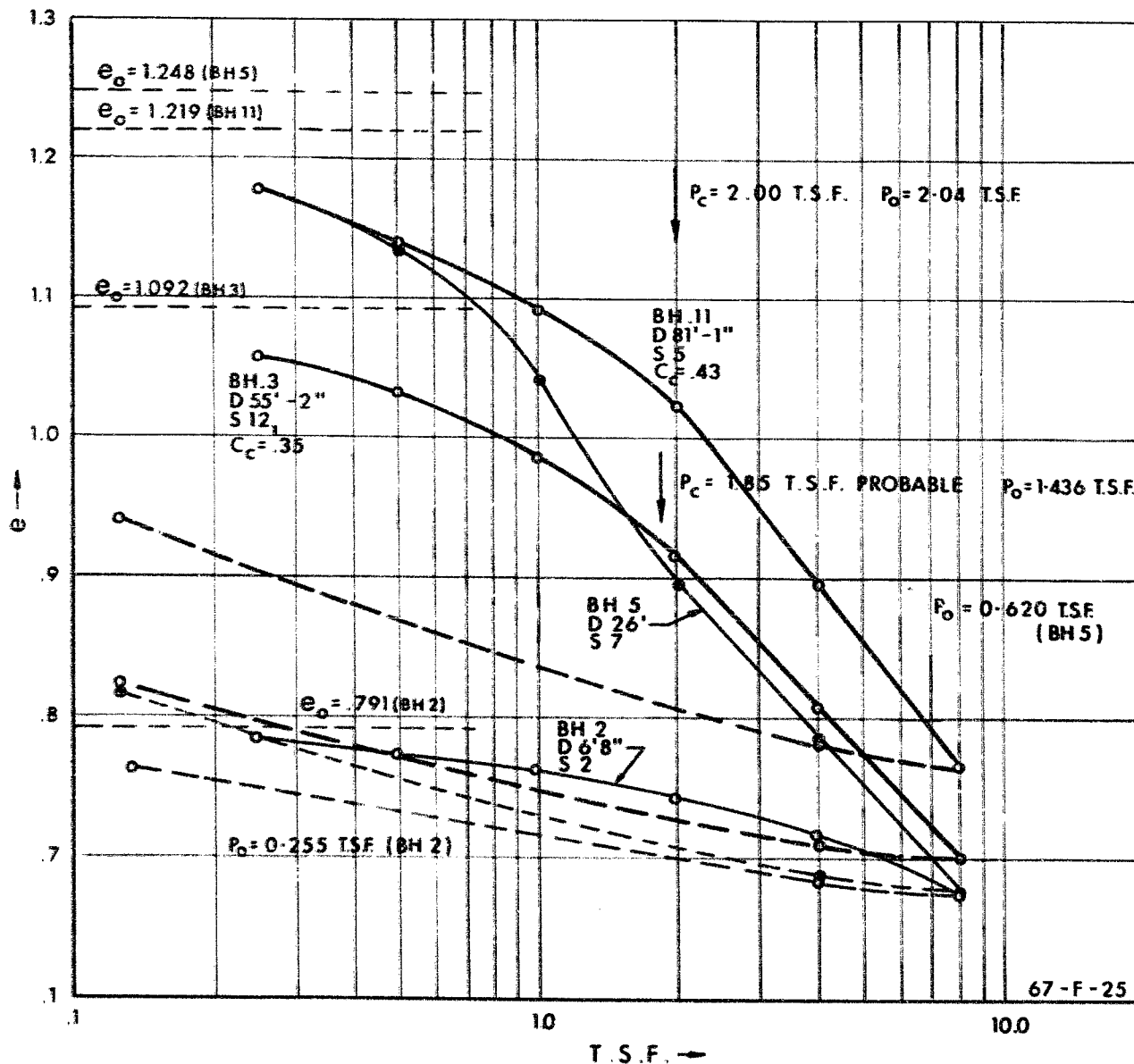
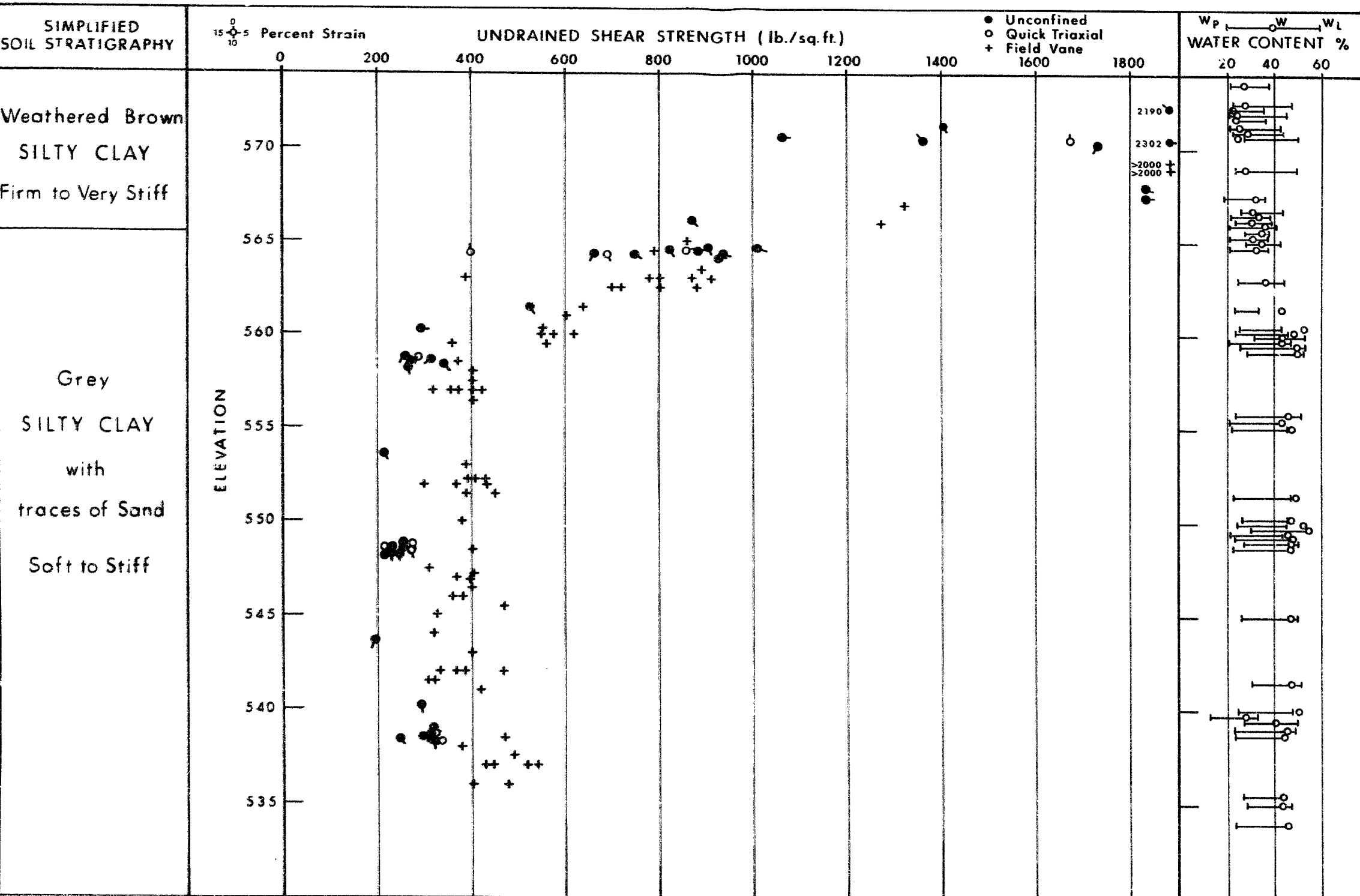


FIG : 5

FIG 6



NEW HIGHWAY No 40 LINE 'B' & CHESAPEAKE & OHIO RAILWAY
SUMMARY PLOT OF ENGINEERING PROPERTIES
ELEVATION 574 TO 535



ABBREVIATIONS USED IN THIS REPORT

PENETRATION RESISTANCE

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

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

DESCRIPTION OF SOIL

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

<u>CONSISTENCY</u>	<u>'N' BLOWS / FT.</u>	<u>LB. / 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	OSTERBERG SAMPLE
A.S	AUER SAMPLE	F.S	FOIL SAMPLE
C.S	CHUNK SAMPLE	R.C	ROCK CORE
S.T	SLOTTED TUBE SAMPLE		
	P.H	SAMPLE ADVANCED HYDRAULICALLY	
	P.H	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 = $\frac{w_p - w}{I_p}$
e_{max}	VOID RATIO IN LOODEST 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_1	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

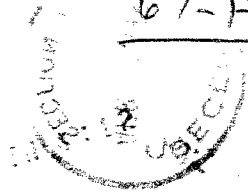
FOUNDATIONS

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_b	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

67-F-25



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CL

MX DOWN APRIL 4/67 400P VR

CHAT 3 F C BROWN DIST ENGR

LOND 6 J ROY MAT AND TESTING

RE WP314-66 BRIDGE SITE 13-296 CHESAPEAKE AND OHIO RAILWAY OVERHEAD

1.2 MILES NORTH OF EXISTING HWY 40 DIST 1 CHATHAM

FIELD INVESTIGATION WORK FOR THE ABOVE MENTIONED STRUCTURE

WILL COMMENCE ON APRIL 6/67. THIS IS FOR YOUR INFORMATION.

A DEVATA FOR A G STERMAC MAT AND TESTING

BB

67F-257

APR 5 11 2120

1960

OWN LOND 8 APR 5/67 2:30P

DE VATA LAB BUILDING

THE NECESSARY PERMISSIONS TO ENTER HAVE BEEN OBTAINED FOR FOUNDATION
SECTION TO GO IN FOR SOIL INVESTIGATION FOR THE OVERPASS ON THE C AND
RAILWAY ON THE SOUTH HALF AND NORTH HALF OF LOT 5 CONC 3 IN THE
TOWNSHIP OF CHATHAM GORE
D VEIGEL FOR D F WALTON REG SERV MGR

Meryhert Kun



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1967 JUN 2 AM 10:46

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DOWN LOND 7 JUNE 2/67

10.35A

A G STERMAC PRINCIPAL FOUNDATION ENGR

RE W P 314-66 BRIDGE SITE 13-296

CHESAPEAKE AND OHIO RAILWAY OVERHEAD

1.2 MILES NORTH OF EXISTING HWY 40 NEW HWY 40 DIST 1 CHATHAM

PLEASE ADVISE ME WHEN I CAN EXPECT THE PRELIMINARY MEMORANDUM GIVING

TH SIZE OF BERMS REQUIRED ALON WITH THE OFFSET FROM THE MUNICIPAL

DRAINING RUNNING PARALLEL TO THE PROPOSED ALIGNMENT OF NEW HWY 40.

THIS INFORMATION IS REQUIRED IN WRITING AS SOON AS POSSIBLE.

A P WATT BRIDGE PLANNING

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MEMORANDUM

To: Mr. A. Stermac,
Principal Foundation Engr.,
DOWNSVIEW, Ontario.

FROM: Mr. G. Baun,
Field Supvr.,
LONDON, Ontario.

Att'n: M. Devata

DATE: April 17, 1967.

Our File Ref.

IN REPLY TO

SUBJECT:

Location of boreholes at Clay Creek (W.P. 42-67)
and Chesapeake and Ohio Railway (W.P. 314-66). ✓ 67-F-25
New Hwy. 40, Twp. Chatham, County Lambton.
W.O.'s 9392-67-4, 9392-67-5, Dist. 1, Chatham

This is to inform you your requests at
Clay Creek (April 4, 1967) and Chesapeake and Ohio
railway (April 5, 1967) have been filled by M. Evans
and party April 5, 1967 and April 11, 1967 respectively.

All information obtained at these locations
has been forwarded to the engineers on the site upon
completion of the request.



G. Baun,
Field Supervisor

GB:CB

PRELIMINARY STRUCTURE SITE REPORT

67-F-25

HWY. 40 W.P. 314-66 STATION 64+55.96 ^{LINE 8'} DISTRICT 1 - CHATHAM
PLAN NO. B-102-20 PROFILE NO. C-102-16 SITE PLAN NO. E-4399-1

Purpose of Structure: River Crossing R.R.X ☒
Grade Separation Other ☐

Is Structure located on D.H.O. right-of-way? No. If not, who owns property and was permission obtained to carry out necessary exploration work? PERMISSION WAS NOT OBTAINED

Describe Soil Conditions at Site. This is to be determined chiefly from a visual observation and possibly a limited amount of hand exploration and should include the general geological formation, anticipated soil conditions, bedrock if visible, etc.

Is Structure Site readily accessible with Core Drill or Power Auger?

YES

Would preliminary borings by Power Auger be advantageous?

No

Is water available at the site? No If not, where is closest source?

RUNNING CREEK OR RESIDENTS

Should Approach Fills be investigated for stability?

YES

REMARKS:

DATE MARCH 28, 1967

ENGINEER A. P. WATT

Mr. S. E. Davis,
Bridge Engineer,
Bridge Division, Admin. Bldg.

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

Attention: Mr. S. McCombs,
Bridge Planning
Engr.

June 16, 1967

Chesapeake & Ohio Railway,
Box Highway 40, Line 'B',
District No. 1 (Guthrie).
W.P. 314-66 - S.J. 67-P-25

The field and laboratory investigations for the above site have disclosed that an approx. 90-ft. thick deposit of soft to firm clay overlies bedrock. The strength of the clay layer is of such an order that the construction of the approach embankments without extensive berming is not possible.

The stability analyses have shown that the maximum height of embankment without berm is 16 feet. Up to the height of 28 ft., a single berm of up to 150 ft. in length would be required, while for an embankment height of 32 ft. (maximum), double berms of combined length of 215 ft. would be necessary.

The analyses have also shown that the toe of a 16-ft. embankment (no berm) would have to be at least 90 ft. away from the edge of the top of the ditch slope. The toes of the berms would have to be only about 30 ft. away from the ditch slope.

The above described is shown on the attached sketch.

In addition to the stability problems, difficulties will also be experienced with settlements. Calculations have shown that settlements up to 7 feet can be expected, of which about 40% would occur within the first 4 - 5 years.

In connection with our findings, as outlined above, we would make the following suggestions:

(a) An embankment test section should be built some distance away from the railway crossing. The embankment should be built to its maximum height of 32 ft., but only one berm (150 ft. long) should be constructed. This proposal is based on our feeling that the stability calculations, although numerically correct, represent conditions which may never materialize in the field, for reasons yet unknown to us.

June 16, 1967

(b) A study should be undertaken to explore and establish the possibility of using fly-ash for the construction of the approach embankments. This material is lighter than earth and, therefore, its use reduces to varying degrees, the problems of stability and settlement. We realize that there are problems to be overcome, but we also believe that it is worthwhile exploring this proposal.

(c) In view of the magnitude of the problems encountered and, consequently, because of the very high costs that would result, we would suggest that it be investigated whether the level crossing could be maintained. Should this be possible, we would still suggest that the land required for the construction of the presently proposed crossing be reserved for any future eventuality.

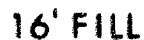
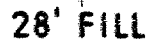
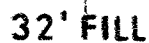
The report containing all the detailed information will be sent to you shortly.

ACS/ndf
Attach.

A. C. Starnes
A. C. Starnes
PRINCIPAL FOUNDATION ENGINEER

cc: Messrs. F. C. Brown
A. K. Watt
A. Gater
F. Harvey
J. Roy
A. Crowley

Foundations Files ✓
Gen. Files



SECTIONS

SCALE 1" = 40'



DEPARTMENT OF HIGHWAYS
MATERIALS and
TESTING
DIVISION

ONTARIO

DEPARTMENT OF HIGHWAYS
MATERIALS and
TESTING
DIVISION

HIGHWAY NO 40 LINE 'B'
TYPICAL SECTIONS
FOR 16', 28' & 32' FILLS

W.P. 314-66

APPROVED *[Signature]*

DRAWING NO. FIG. 4

Mr. B. A. Davis,
Bridge Engineer,
Bridge Division,
Admin. Bldg.

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

June 27, 1967

Attention: Mr. S. MacCubbin

-- Proposed Overhead --
New Bay. 40, Line 'B' and
Chesapeake and Ohio Railway
District No. 1 (Chatham)
A.J. 67-P-25 -- A.P. 314-66

In our Report A.J. 67-P-25, for the above mentioned crossing, we have outlined the main foundation problems and recommended ways of coping with them. No mention was made, however, of the problems connected with the possible future widening of the road into a four-lane, divided highway.

Because of the relatively weak subsoil, stability problems will arise, and also large settlements are expected. Therefore, there is no doubt that from the technical point of view, it would be desirable to build the approach embankments for their full width - i.e., as a divided, four-lane highway.

We understand that the widening of the highway may not be undertaken for another 20 to 30 years. This relatively long period of time would probably not warrant the building of the full width of the approach fills. However, in the light of past experience, the possibility that the need for highway widening may arise much sooner, cannot be entirely ruled out. This, of course, would introduce some problems with implications that could be quite serious. Here, we have in mind the lateral differential settlements which would be rather difficult to cope with should they be significant.

It is, therefore, our opinion that it would be very desirable that the approach embankments be built for their entire width, even though only one half would be used for the time being.

cont'd. /2 ...

Mr. B. S. Davis,
Bridge Engineer,
Attn: Mr. S. McCombie.

- 2 -

June 27, 1967

As an alternative, we would suggest that at least the end portions of the approach embankments be built for their entire widths, while the remainder could be built when the need arises. We would recommend a 100-ft. embankment section (in the longitudinal direction) plus all the necessary berms.

AGS/746P

A. G. Starnes
A. G. Starnes
PRINCIPAL FOUNDATION ENGINEER

cc: Messrs. F. C. Brown
A. P. Watt
R. C. Gaseoyne
A. Gater
J. Roy
A. Crowley

Foundations Files
Gen. Files

Materials and Testing Division

April 5, 1967

Johnston Drilling Co. Ltd.
378 Bering Street
Toronto, Ontario

Dear Sirs:

This is to confirm our request of April 4, 1967 for the supply of a Diamond Drill together with all necessary equipment, as specified under the terms of our Contract Agreement, at Wallaceburg, Ontario, on April 6, 1967 @ 11.30 a.m.

This project bears Job Number 67-F-25.

Yours truly,



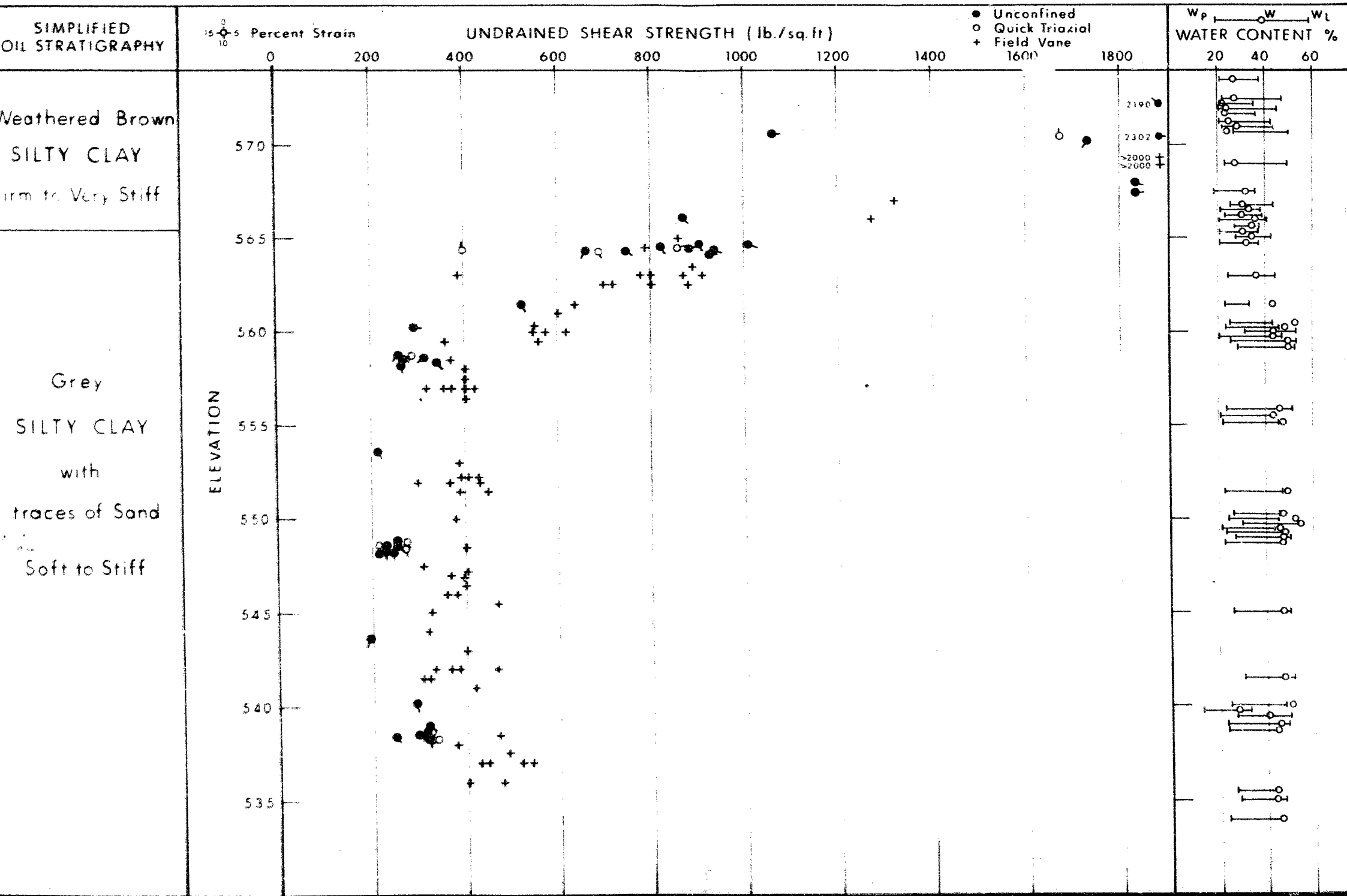
M. Devata
Supervising Foundation Engineer
for: A. G. Sternac
Principal Foundation Engineer

DMT

cc: M. Konings
H. Szymanski ✓

Foundation Files
General Files

NEW HIGHWAY No 40 LINE 'B' & CHESAPEAKE & OHIO RAILWAY
SUMMARY PLOT OF ENGINEERING PROPERTIES
ELEVATION 574 TO 535



Warnock Hersey Soil Investigations Ltd



Office Report Of Soil Exploration

Casing Diameter Elevn.
Casing Hammer Wt. Drop
Sample Hammer Wt. Drop

Client

Order Number
Borehole Number 5
Date

SAMPLE CONDITION & TYPE

ABBREVIATIONS

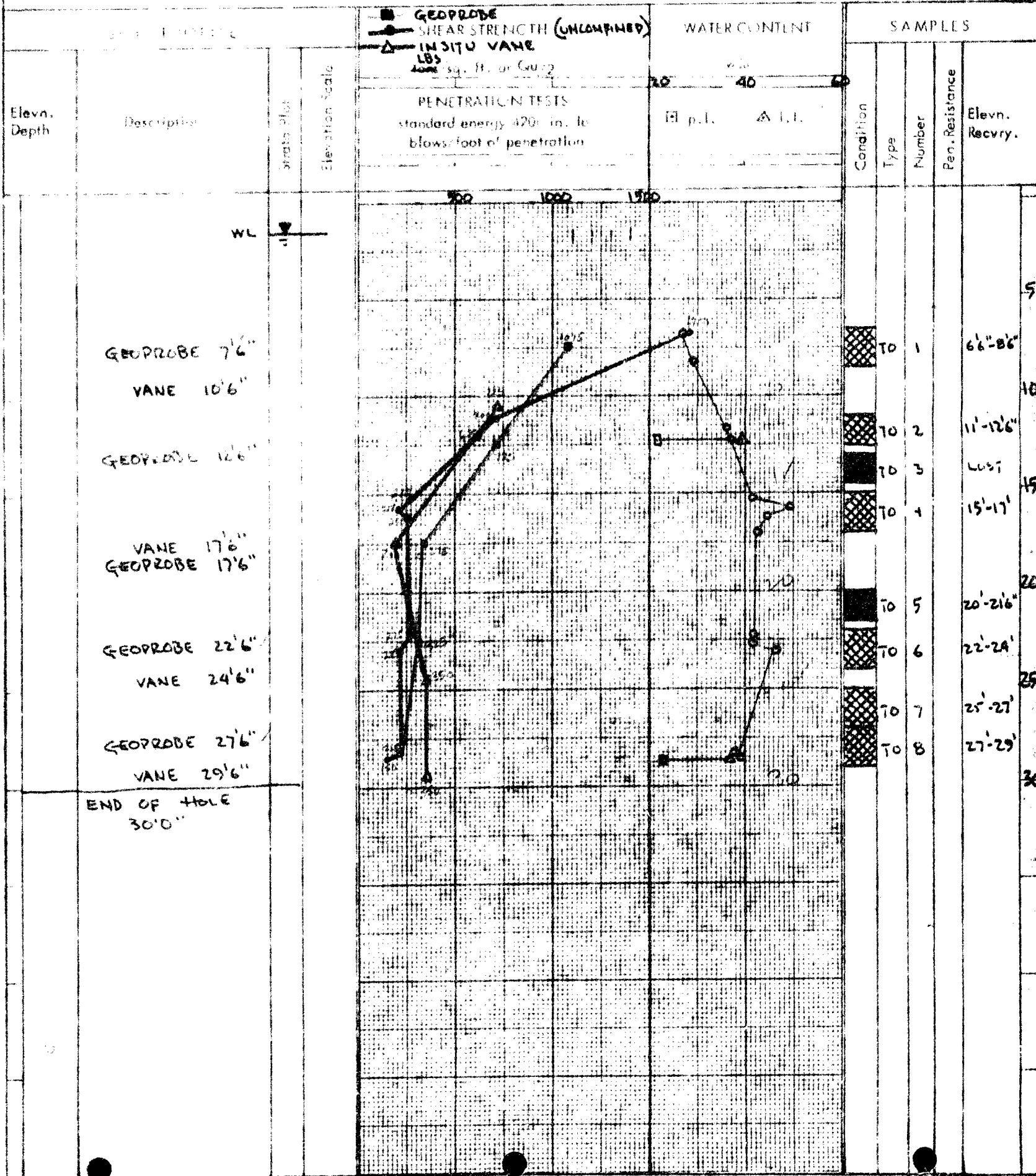


Disturbed
Good
Fair

CS - Chunk
DO - Drive Open
DF - Drive Foot Valve
TO - Thinwalled Open
CS - Cashed Sample
C - Rock Core

V - In Situ Vane Shear Test
M - Mechanical Analysis
U - Unconfined Compression
Qc - Triaxial Consolidated Quick
Q - Triaxial Quick
S - Triaxial Slow

Unit Weight
K - Permeability
C - Consolidation
CA - Casing
WL - Water Level in Casing
WT - Water Table in Soil



Warnock Hersey Soil Investigations Ltd

Office Report Of Soil Exploration

Casing
Casing Hammer
Sample Hammer

Diameter
Wt.
Wt.

Elevn.
Drop
Drop



Client

Order Number
Scribble Number
Date

6

SAMPLE CONDITION & TYPE



Disturbed
Good
Lost

CS - Churn
DO - Drive Open
DF - Drive Footvalve
TO - Unwalled Open
WS - Washed Sample
PC - Rock Core

ABBREVIATIONS

V - In-situ Vane Shear Test
M - Mechanism Analysis
U - Unconfined Compression
CC - Triaxial Consolidated Quick
C - Triaxial Quick
T - Triaxial Slow

U - Unit Weight
K - Permeability
C - Consolidation
CA - Casing
WL - Water Level in Casing
WS - Water Table in Soil

SOIL PROFILE

Elevn.
Depth

Description

Brand Plan

Elevation Scale

GEOPROBE SHEAR STRENGTH (UNCONFINED) IN SITU VANE

LBS
sq. ft. or Corp

PENETRATION TESTS

standard energy 4200 ft.-lb.
blows/foot of penetration

WATER CONTENT

Wt %

SAMPLES

Condition

Type

Number

Pen. Resistance

Elevn.
Recovery

GEOPROBE 7'6"

VANE 9'6"

GEOPROBE 12'6"

VANE 14'

GEOPROBE 18'6"

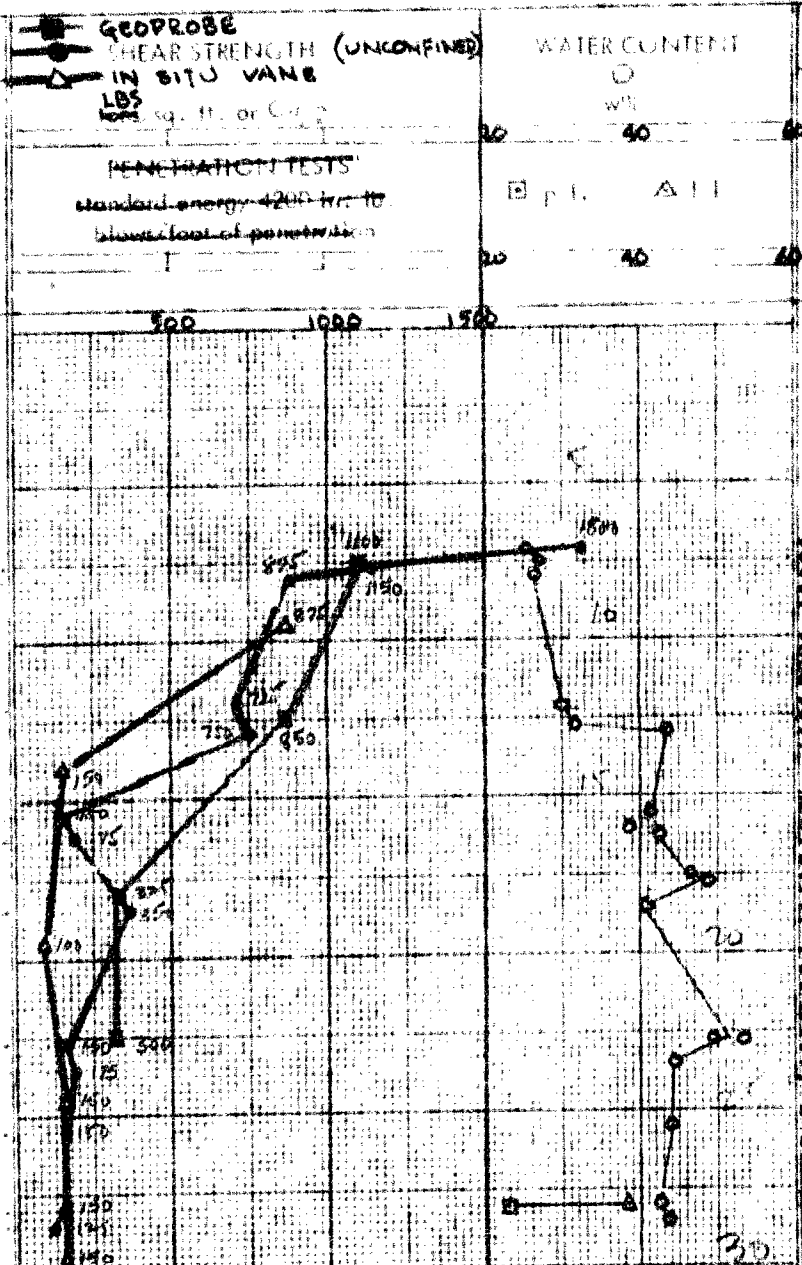
VANE 19'6"

GEOPROBE 27'6"

VANE 29'6"

VANE 29'6"

END OF HOLE
30'0"



Condition	Type	Number	Pen. Resistance	Elevn. Recovery
TO	1	7'-9'		5
TO	2	10'-11'6"		10
TO	3	12'-13'6"		15
TO	4	15'-15'6"		20
TO	5	17'-19'		25
TO	6	20'-22'		30
TO	7	22'-24'		35
TO	8	25'-27'		40
TO	9	27'-29'		45

Warnock Hersey Soil Investigations Ltd

Office Report Of Soil Exploration



Casing Diameter Elevn.
Casing Hammer Wt. Drop
Sample Hammer Wt. Drop

Client

Order Number
Borehole Number
Date

SAMPLE CONDITION & TYPE

ABBREVIATIONS



Disturbed
Good
Lost

CS - Chunk
DO - Drive Open
DF - Drive Footvalve
TO - Thinwalled Open
WS - Washed Sample
RC - Rock Core

V - Insitu Vane Shear Test
M - Mechanical Analysis
U - Unconfined Compression
Qc - Triaxial Consolidated Quick
Q - Triaxial Quick
S - Triaxial Slow

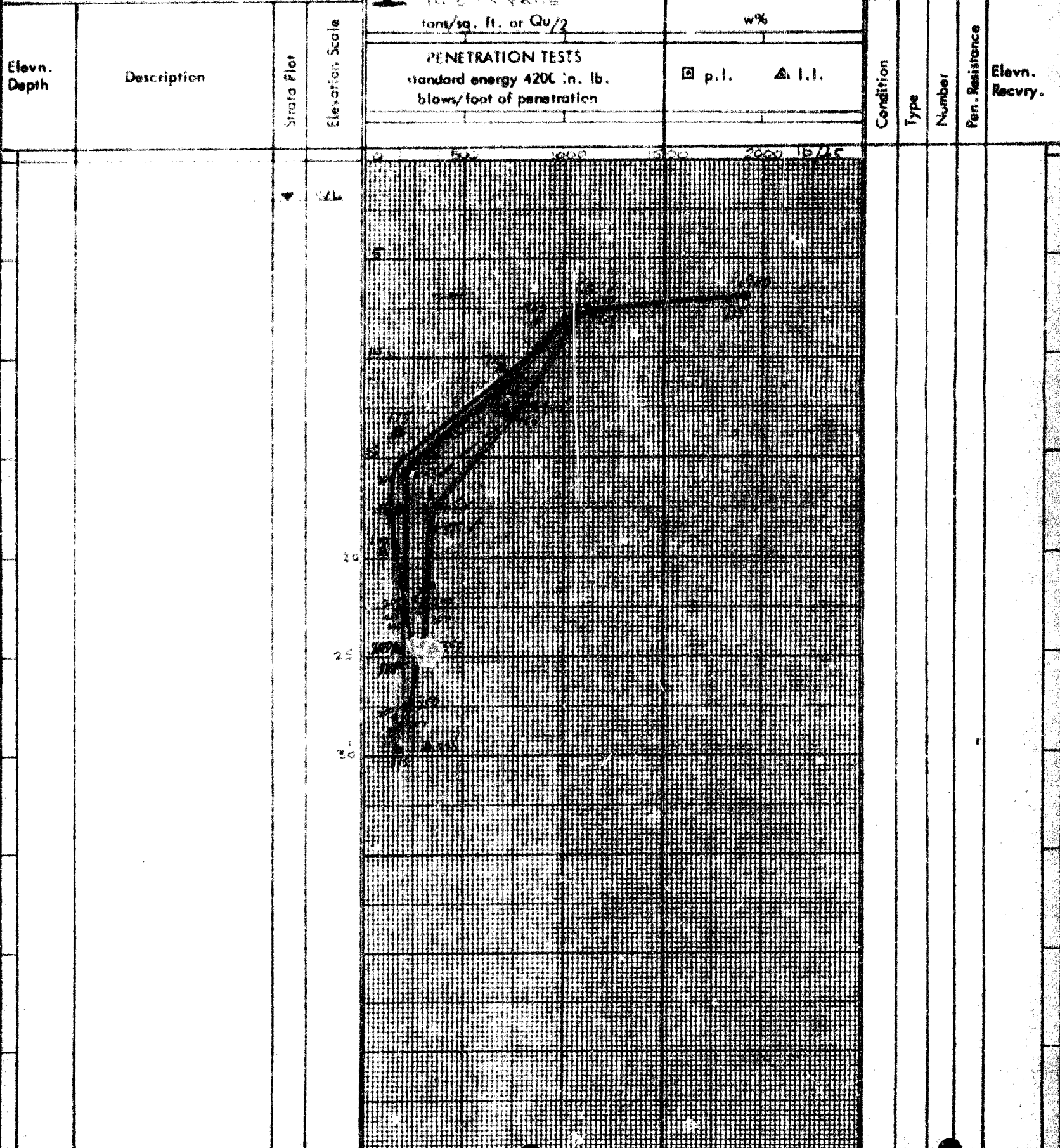
- Unit Weight
K - Permeability
C - Consolidation
CA - Casing
WL - Water Level in Casing
WT - Water Table in Soil

SOIL PROFILE

SHEAR STRENGTH
INSITU VANE
tons/sq. ft. or $Q_u/2$

WATER CONTENT

SAMPLES



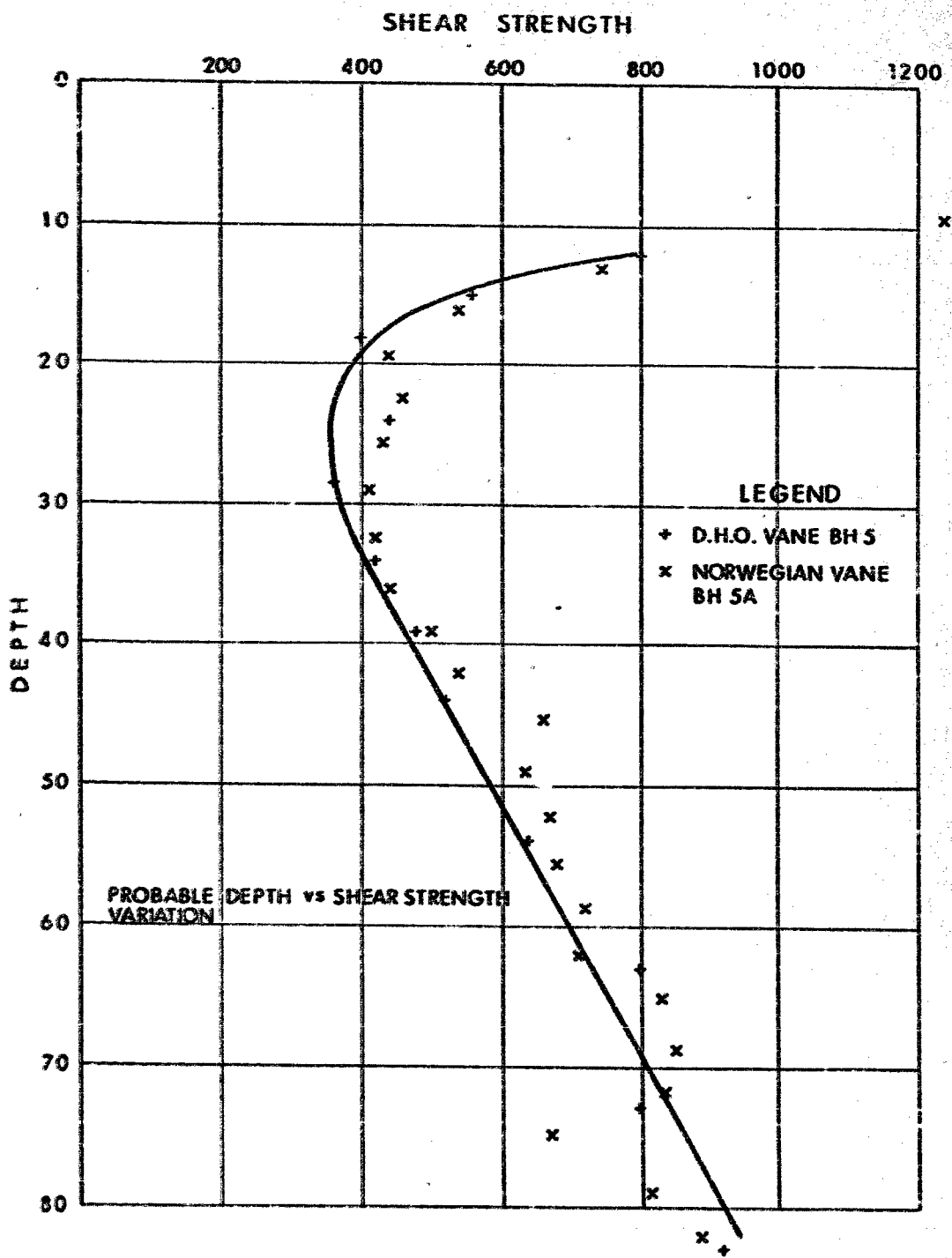


FIG. 2

57-F-25

NARNOLK HERSEY SOIL INVESTIGATIONS LTD

UNDRAINED SHEAR STRENGTH
(16/50, 4.)

wp 20 25

ELEVATION

570

565

560

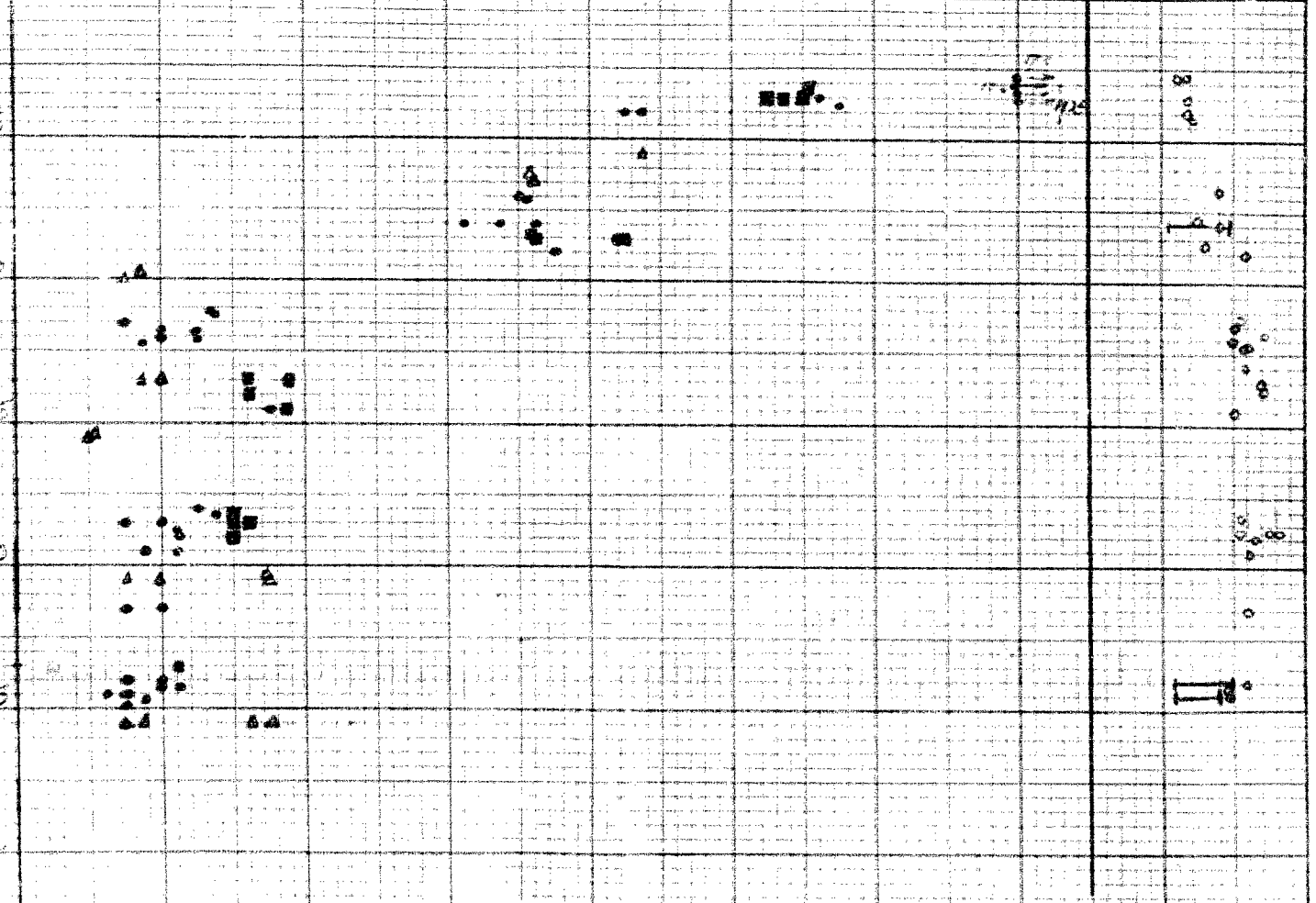
555

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0 200 400 600 800 1000 1200 1400 0 20 40 60



Mr. W. Ziemberg,
Regional Road Design Engineer,
London.

Att: Mr. D. D. Murray

Mr. W. G. Wigle,
Program Engineer.

November 17, 1967.

W. P. 314-86, C. & O. Railway Overhead Structure
(within W. P. 43-85-03)
New Highway 40, District 1 - Chatham

62F-25

I refer to your memorandum of November 2nd, concerning the
above.

This will confirm that the proposal to place an experimental test
fill section for the future railway overhead has been deferred indefinitely.

WGW/JMD/mt

c. c.

A. Stormac ✓
F. C. Brown
J. R. Ray

W. G. Wigle
W. G. Wigle,
Program Engineer.

MEMORANDUM

NOV 01 1967

To: Mr. W. Wigle,
Program Engineer,
Program Section,
DOWNSVIEW, Ontario.

FROM: Mr. D. D. Murray,
Sr. Project Design Engineer,
Road Design Division,
LONDON REGIONAL OFFICE.

DATE: November 2nd, 1967.

OUR FILE REF.

IN REPLY TO

SUBJECT:

Re: W.P. 314-66 Hwy. #40 New (W.P. 43-65-03;
C & O Railway Overhead Structure

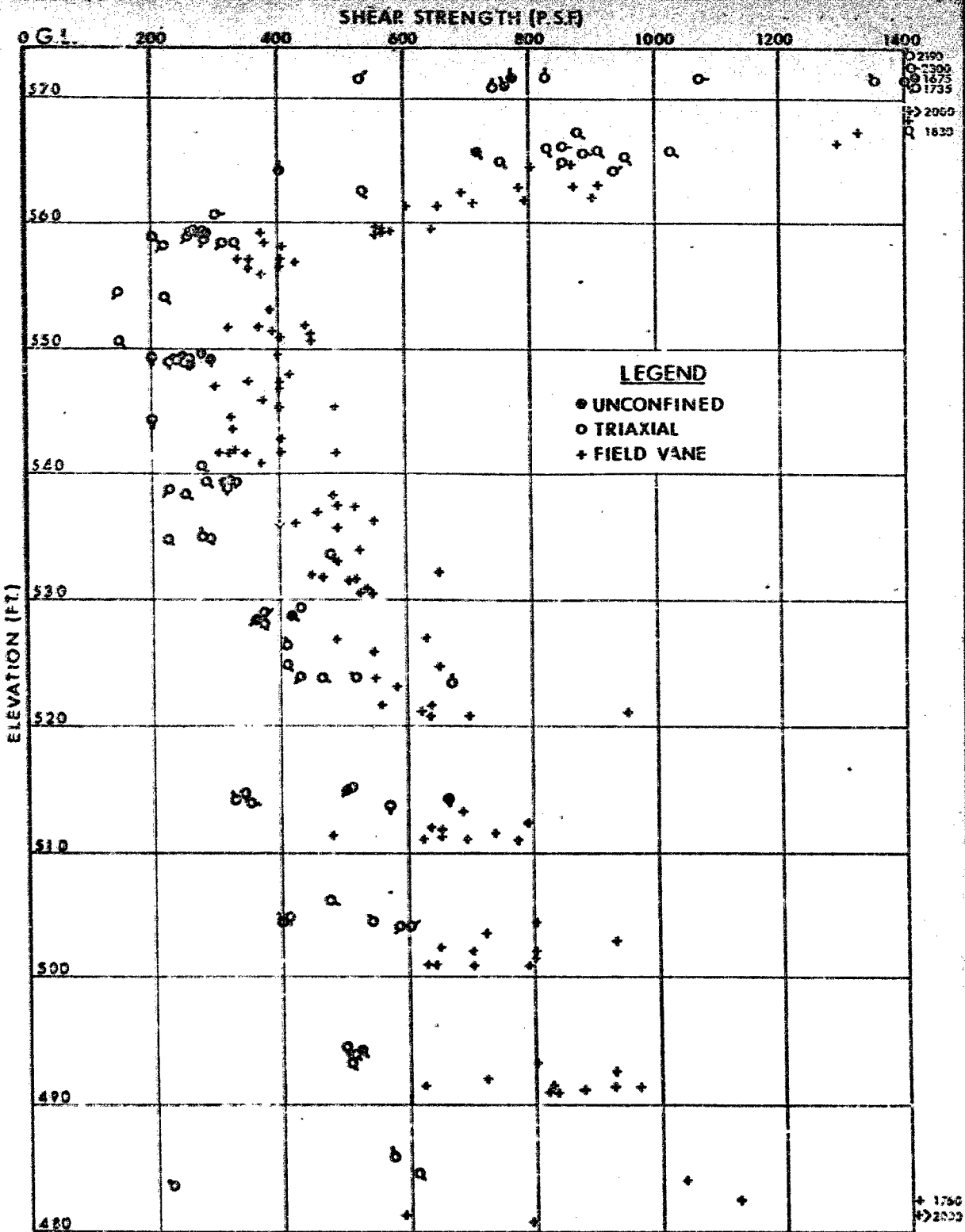
During the early stages of design it was proposed that an experimental test fill section be placed on south approach of the future railway overhead structure, using fly-ash as fill material.

It is understood that this proposal has been deferred and I would be glad to have confirmation of this.


D. D. Murray,
Sr. Project Design Engineer,

DDM/sl.

THIS IMPOSED DOCUMENT MAY
APPEAR AS MULTI-FEED ON FILM



SHEAR STRENGTH PROFILE

DEC. 7/73

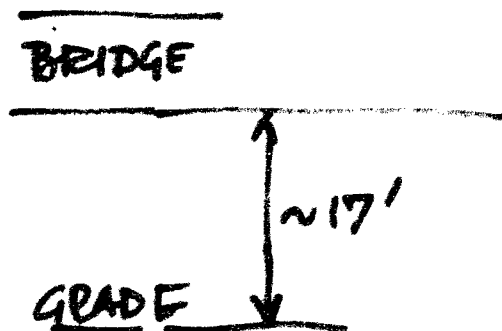
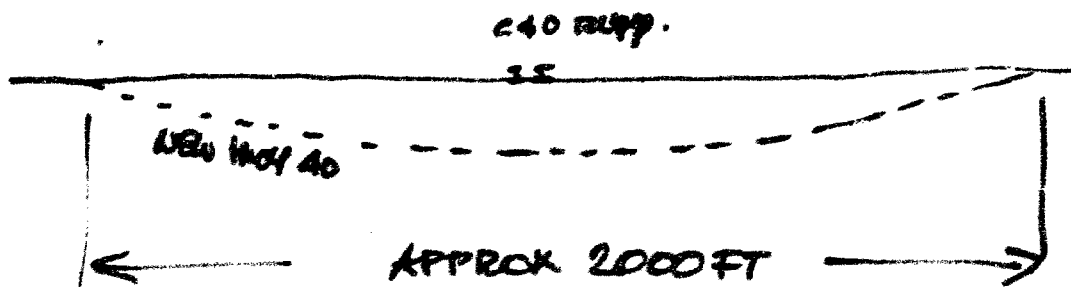
REQUEST (URGENT) BY BRIAN HUKENHA
STR. LOCATION - SOUTHWESTERN REGION

W.P. 314-66 W.J. 67-F-25

NEW HWY 40 & QH. RLY.

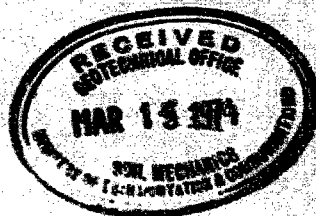
WHAT PROBLEMS CAN BE EXPECTED
IF THE HIGHWAY IS PUT UNDER
THE RAILWAY?

- CONSIDER PLOW FOUNDATIONS
(FLOATING FOUNDATIONS) FOR
ABUTMENTS AND PIERS
- SLOPE STABILITY. CUT SLOPES



DEFECTS IN NEGATIVE DUE TO
CONDITION OF ORIGINAL DOCUMENT

MAR 14 4 48 PM '74



0044 LMD 20 MARCH 14 1974 4:33 PM

W. H. DEVATA GEOTECHNICAL OFFICE

RE: HWY. NO. 40N. & C. 200. N. LY

W. O. 67-F-25 W P 314-86

FURTHER TO YOUR LETTER OF DECEMBER 13, 1973, PLEASE ADVISE IF AND WHEN
RESULTS ARE BEING SENT TO THIS OFFICE, AS MENTIONED IN THE LAST
PARAGRAPH.

RE A F WATT STRUCTURAL PLANNING OF REGION.

CS *Talked to Mr. Mike FHE March 7. Discussed at length.*
He will make all data and if he requires further
detailed information or calculation. We will call
APR 1974

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