

DEPARTMENT OF HIGHWAYS ONTARIO

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: September 6, 1967

OUR FILE REF.

IN REPLY TO SEP 11 1967

SUBJECT:

FOUNDATION INVESTIGATION REPORT
For
Proposed Crossing at Wahl Creek and
Hwy. #11B and #65, Twp. of Dymond
Lot 9, Con. 11, Dist. of Timiskaming
District No. 14 (New Liskeard)
W.J. 67-F-57 -- W.P. 188-66

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 additional information be required, please feel free to contact our Office.

AGS/MdeF
Attach.

cc: Messrs. B. R. Davis (2)
H. A. Tregaskes
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Foundations Files
Gen. Files ✓

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FOUNDATION INVESTIGATION REPORT
For
Proposed Crossing at Wabi Creek and
Hwy. #11B and #65, Twp. of Dymond
Lot 9, Con. 11, Dist. of Timiskaming
District No. 14 (New Liskeard)
W.J. 67-F-57 -- W.P. 188-66

1. INTRODUCTION:

A request to carry out a foundation investigation for the proposed new bridge to carry Hwy. #11B and #65 over the Wabi Creek, was received from Mr. J. B. Curtis, Regional Bridge Location Engineer, in a memo dated June 12, 1967.

An investigation was subsequently carried out by this Section to determine the subsoil conditions existing at the site of the proposed bridge.

This report contains the results of our field and laboratory investigation, together with our recommendations for the foundations of the new structure and the proposed detour.

2. DESCRIPTION OF SITE:

The new structure is proposed to be located at the same site as the existing one in the town of New Liskeard. The north pier of the existing bridge has settled and tilted, causing damage to the bridge. The proposed bridge is skewed at an angle to the centre-line. The proposed detour is located some 55 ft. downstream of the main structure. The north bank of the creek is 10 ft. higher than the south bank. The creek flows through the town of New Liskeard. Downstream of the proposed site, both the banks are bounded by roads, while upstream they are forested.

cont'd. /2 ...

3. FIELD AND LABORATORY WORK:

The field work at the proposed bridge location consisted of four sampled boreholes and six dynamic cone penetration tests. All holes were advanced using conventional diamond drilling equipment adapted for soil sampling purposes. A driving energy of 350 ft.-lbs. per blow was used for the dynamic cone penetration tests.

Disturbed samples were obtained using a 2-inch O.D. split-spoon sampler driven according to the specifications for the Standard Penetration Test. Undisturbed samples were obtained by means of 2-inch I.D. Shelby tubes which were pushed into the soil manually. Bedrock samples were obtained in boreholes 3 and 4 using BXT coring equipment. In boreholes 1 and 2, the bedrock was established when refusal to further drilling by means of chopping bit was reached.

In-situ vane tests were carried out wherever possible, at elevations 12 inches below various sample depths.

Samples were visually examined in the field and subsequently in the laboratory. The following tests were carried out on selected samples:

- 1) Grain-Size Distribution Curves
- 2) Atterberg Limits
- 3) Quick Triaxial Test
- 4) Unconfined Compression Test
- 5) Consolidated Undrained Shear Strength Test
- 6) Natural Moisture Content
- 7) Bulk Density

The water level in borehole 4 was observed at elevation 586.5, while the water level in the creek was 587.8. No artesian water conditions were encountered.

3. FIELD AND LABORATORY WORK: (cont'd.) ...

The results of field and laboratory tests are summarized in the Record of Borehole sheets, which are contained in the appendix to the report.

The locations and the elevations of boreholes are given on Drawing No. 67-F-57A, which is also contained in the appendix to this report.

The borehole elevations were provided by the New Liskeard District Office of the D.H.O.

4. SUBSOIL CONDITIONS:

4.1) General:

The site of the crossing is part of the region known as the "Little Clay Belt". In general, the subsoil consists of deep deposits of varved clay underlain by a layer of sandy silt to silty sand which in turn, overlies dolomitic bedrock.

The boundaries between the different deposits are shown on the attached Record of Borehole sheets. The estimated stratigraphical profiles shown on Drawing No. 67-F-57A, are based upon this information.

From ground level downwards, the different soil deposits are described as follows:

4.2) Sand and Gravel:

This material was found in borehole 4 only on the south bank, where it was 6.5 ft. in thickness. It is mainly sand and gravel which has been placed as fill material at the site of an old basement. The 'N' values indicate a compact denseness.

cont'd. /4 ...

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

4.3) Varved Clay:

This deposit occurred in all boreholes down to the following elevations:

B.H. #1	:	521.1
#2	:	520.9
#3	:	519.0
#4	:	513.0

The thickness of the varved clay stratum varies from 57.0 ft. (river bed) to 89.5 ft. (north bank).

The soil consists of alternate layers of silt to clayey silt and inorganic clay of high plasticity. The thickness of layers varies from 1/8" to 1-1/2" with no definite pattern of variation. The stratification is horizontal. The range of Atterberg limits for individual layers is as follows:

	<u>Plastic Limit</u>	<u>Liquid Limit</u>
Silt to clayey silt layer	19.2 - 25.7	22.3 - 32.6
Inorganic clay layer	22.0 - 36.9	48.5 - 70.7

Near the ground surface, the varves are very closely spaced so that it was not possible to separate them, in which case Atterberg limits have been determined from the bulk material, and which are as follows:

<u>Plastic Limit</u>	<u>Liquid Limit</u>
21.9 - 24.0	40.5 - 47.4

The natural moisture content of individual varves was found to be close to the liquid limit.

The undrained shear strength was determined in the field by vane tests and in the laboratory by means of undrained triaxial tests and unconfined compression tests. The shear strength as

cont'd. /5 ...

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

4.3) Varved Clay: (cont'd.) ...

determined from the field vane, ranged from 620 to 2000 lbs./sq.ft., while the results from the undrained triaxial tests, ranged from 230 to 1480 lbs./sq.ft., indicating a soft to stiff consistency. The values of the shear strength as obtained from unconfined compression tests performed on samples from borehole 1 only, have been discarded since they are, in general, very low and not in agreement with those obtained from triaxial or field vane tests, presumably because of the following reasons:

- 1) disturbance of sample because of the presence of silt layers.
- 2) expansion of layers in lateral direction because of the lack of support and consequent opening of vertical cleavage planes. This effect is obviously less pronounced when the sample is confined.

The shear strength values thus determined, have been plotted on the Record of Boreholes and are also shown in Fig. 1 as plotted against the elevation. From the shear strength profile it is evident, that, in general, the shear strength increases with depth. A layer composed of the top 10 ft. of the stratum on the banks, is desiccated as evidenced by higher values of shear strength as a result of which, no vane tests were possible in this zone. The upper portion of the deposit within the river bed up to a maximum of 10-ft. depth, is very much softer than the remainder of the deposit. The undrained shear strength of the soil in this zone varies randomly from about zero to about 600 p.s.f.

cont'd. /6 ...

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

4.3) Varved Clay: (cont'd.) ...

The consolidated undrained triaxial compression stage tests were performed in the laboratory to determine the effective stress parameters. The results obtained from these tests are:

Effective cohesion intercept $c' = 280 - 390$ p.s.f.
Effective angle of friction $\phi' = 27.0^\circ - 29.0^\circ$

For design purposes, the following values have been used:

$c' = 250$ p.s.f.
 $\phi' = 24^\circ$

The value of $\phi' = 24^\circ$ is somewhat lower than the laboratory test results and is based in part, on experience acquired at similar sites in the New Liskeard area.

4.4) Sandy Silt to Silty Sand with traces of Clay:

This deposit was encountered immediately below the varved clay layer and down to the bedrock. The thickness of the stratum varied from 23.0 ft. in borehole 4 to 42.2 ft. in borehole 3. The grain size analyses showed the following ranges:

Sand	:	10 - 79%
Silt	:	19 - 83%
Clay	:	2 - 10%

Standard Penetration Tests gave 'N' values indicating a compact to very dense denseness.

4.5) Dolomitic Bedrock:

The bedrock was proven to 5 ft. in borehole 3 and 10 ft. in borehole 4. In boreholes 1 and 2, it was established when refusal to further penetration by washboring was met. The bedrock

cont'd. /7 ...

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

4.5) Dolomitic Bedrock: (cont'd.) ...

core samples were examined by Mrs. Z. Koniuszy, Geologist, Materials and Testing Division, Department of Highways, whose report is as follows:

"The boreholes of the project No. 67-F-57 - New Liskeard, were drilled in Paleozoic (Silurian) Formation which is represented by heavily bedded porous and cavernous light bluish-grey to buff coloured, fine to medium crystalline, highly fossiliferous (large crinoid stems) dolomite. Densely but not evenly distributed vugs are often lined with small calcite crystals.

"Samples show very little variety - rock is sound with no signs of underground water erosion and in both the weathering zone is limited to about the top 2 inches of core."

The bedrock surface dips slightly from south to north. On the south bank, the depth to the bedrock is 108.0 ft., and on the north bank it is 131.7 ft.

5. GROUNDWATER:

The water level in the river at the time of investigation, was at elevation 587.8. Groundwater level in borehole 4 was found to be at 586.0. It may be assumed that the groundwater level in the vicinity of the river is equal to or slightly higher than the prevailing river water level.

6. DISCUSSION AND RECOMMENDATIONS:

It is proposed to replace the existing bridge with a new bridge at the same site. An approach fill of 1.5 ft. at the south abutment and of 4.0 ft. at the north abutment is required, resulting in a maximum approach height of about 46 ft. above the river bed.

cont'd. /8 ...

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

6.1) Main Bridge:

Because the varved clay layer is unable to provide adequate bearing capacity and, also, because of the undesirable settlements due to consolidation, a spread footing type foundation is considered to be unsuitable. Therefore, it is recommended that the entire structure be supported by means of end-bearing H-piles driven to the bedrock. Design loads to be used should be the maximum allowable for the particular pile section adopted and may be 100 tons in the case of 12 BP at 74.

6.2) Detour:

Since it is required to maintain traffic at all times on the highway, it will be necessary to provide a detour. The river will be temporarily crossed by means of a Bailey bridge during construction of the new bridge. It is recommended that the abutments of the Bailey bridge be supported on piles since spread footing support is impractical in the bed of the river. Depending on the economic factors involved, the pile foundation may be one of the following:

(1) Steel H-piles driven to bedrock using the maximum allowable design load.

(2) 12-3/4 inch O.D. steel tube piles end bearing in the sandy silt to silty sand stratum. For piles driven to approximate elevation 515.0, a safe capacity of 50 tons/pile should be achieved.

(3) No. 14 timber piles driven into the varved clay layers. These will be friction piles, and the safe load may be assumed to be 0.4 ton per ft. of penetration into the soil. The minimum penetration, however, must not be less than 30 ft. This latter requirement is very important because of the low shear strength of the soil in the upper portion of the stream bed.

cont'd. /9 ...

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

6.2) Detour: (cont'd.) ...

The deck level of the Bailey bridge should be lowered as much as possible to simplify and reduce the cost of construction.

6.3) Approaches, Dewatering and Excavation:

The stability of the approaches with the required 4-ft. fill, was investigated by means of:

- (1) total stress analysis.
- (2) effective stress analysis for the sudden drawdown case (water level drop from El. 595.0 - 581.0).

Soil properties assumed in the above analyses are as follows:

Undrained shear strength C (See Fig. 1).

Elevation 590.0 - 610.0	$C = 900$ p.s.f.
570.0 - 590.0	$C = 800$ p.s.f.
540.0 - 570.0	$C = 1000$ p.s.f.
520.0 - 540.0	$C = 1200$ p.s.f.
Bulk density (fill & subsoil)	$\gamma = 114$ p.c.f.
Effective cohesion intercept	$C' = 200$ p.s.f.
Effective angle of friction	$\phi' = 24^\circ$
Depth of tension crack	$= 8$ ft.

It was found that in order to ensure stability of the forward slopes for the proposed height of the approaches, it would be necessary to provide slopes of 3:1. The river banks must, therefore, be trimmed to this slope.

cont'd. /10 ...

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

6.3) Approaches, Dewatering and Excavation: (cont'd.) ...

Since the piers have to be founded under the water, it will be necessary to provide some dewatering scheme, which can be done by driving sheet piles into the varved clay stratum, or by some other suitable means.

Pile caps should be founded at sufficient depth to ensure frost protection. In the event that the pile caps for abutments are founded below the groundwater level, dewatering can be achieved by means of pumping the water out; the soil is relatively impermeable.

In forming pile cap bases, it should be borne in mind that the soil in the river bed is, in places, extremely soft and incapable of providing even temporary support. In such cases, a remedy would be to excavate below the pile cap bases and construct a granular pad of sufficient thickness (about 3.0 ft.⁺) to provide the necessary temporary support.

Scour protection should be provided.

7. SUMMARY:

A foundation investigation at the site of the proposed crossing of Hwy. #65 & #11B and the Wabi Creek in New Liskeard is reported.

Subsoil at the site consists of deep deposits of soft to stiff varved clay overlying a layer of compact to very dense sandy silt to silty sand. The bedrock is 108.0 to 131.7 ft. below the ground level.

It is proposed to construct a new bridge at this location. This will entail a raise in grade to a maximum of 4.0 ft. above the existing one. This will result in a total height of 46.0 ft. above the river bed.

cont'd. /11 ...

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

A detour located some 55 ft. east (downstream) of the existing bridge, is proposed to carry the traffic while the new bridge is being constructed.

For the new bridge, it is recommended that the entire structure be supported on steel H-piles driven to bedrock.

Forward slopes of 3:1 are recommended for the approaches of the proposed bridge.

Scour protection should be provided.

For the detour, it is recommended that the abutments of the Bailey bridge be supported on timber cribs and the piers on piles. The deck level of the Bailey bridge should be lowered as much as possible, to simplify and reduce the cost of construction.

Dewatering schemes will be necessary for excavations carried out below the stream bed or groundwater level. No major problems, such as 'boiling' of excavation bases, are anticipated.

8. MISCELLANEOUS:

The field work for this project was carried out during the period July 12 to August 1, 1967, under the supervision of Mr. A. Prakash, Project Foundation Engineer, who also prepared this report.

The equipment used was owned and operated by Master Soil Investigations Ltd.

This report was reviewed by Mr. K. G. Selby, Supervising Foundation Engineer.

September 1967

APPENDIX I

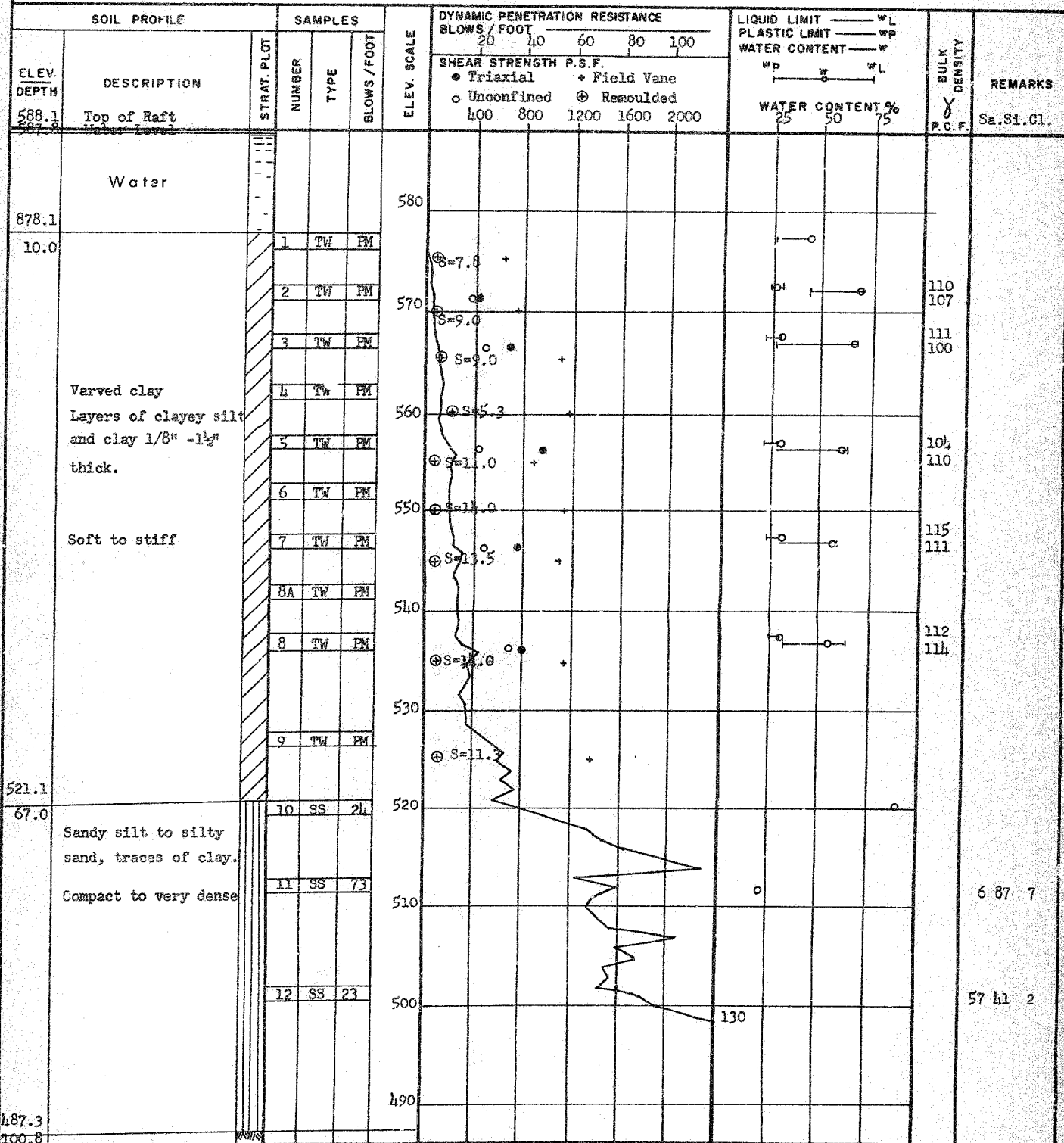
DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 1

FOUNDATION SECTION

JOB 67-F-57 LOCATION Sta. 77 + 34 33' Rt. ORIGINATED BY AP
W.P. 188-66 BORING DATE July 27, 29, 1967 COMPILED BY AP
DATUM Geodetic BOREHOLE TYPE Washboring NX & BX Casing, Cone CHECKED BY HR



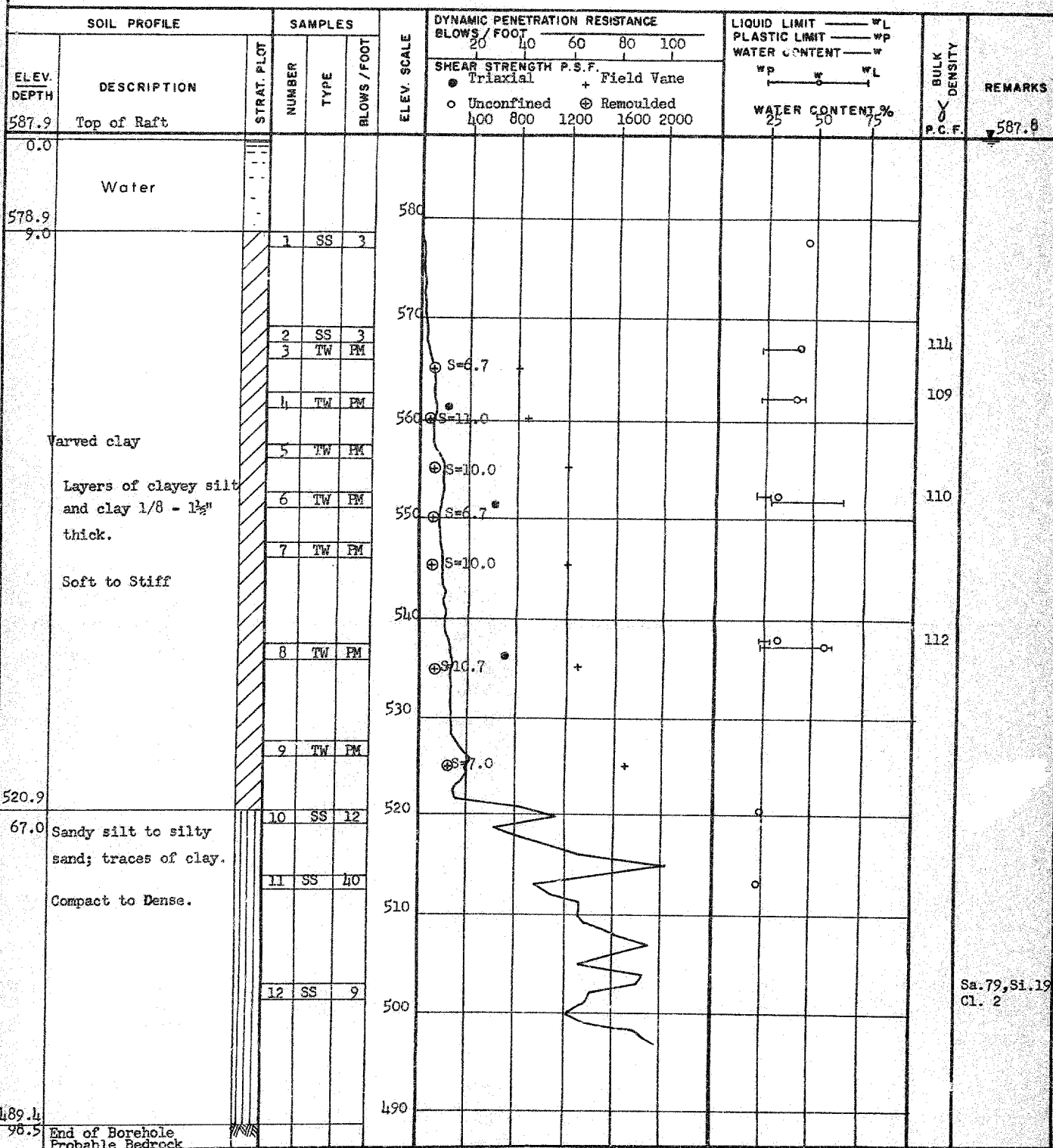
DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 2

FOUNDATION SECTION

JOB 67-F-57 LOCATION Sta. 76 + 54 23' Lt. ORIGINATED BY AP
W.P. 188-66 BORING DATE July 31 - Aug. 1, 1967 COMPILED BY AP
DATUM Geodetic BOREHOLE TYPE Washboring NX & BX Casing, Cone CHECKED BY HR



DEPARTMENT OF HIGHWAYS - ONTARIO		RECORD OF BOREHOLE NO. 3		FOUNDATION SECTION
MATERIALS & TESTING DIVISION				
JOB <u>67-F-57</u>	LOCATION <u>Sta. 78 + 27 h0' Rt.</u>	ORIGINATED BY <u>AP</u>		
W. P. <u>188-66</u>	BORING DATE <u>July 21 - July 25, 1967</u>	COMPILED BY <u>AP</u>		
DATUM <u>Geodetic</u>	BOREHOLE TYPE <u>Washboring NX & BX Casing, Cone</u>	CHECKED BY <u>SR</u>		

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.	Unconfined	+ Field Vane		P.C.F.
0.0	Ground Level					400 800 1200 1600 2000	o Unconfined	@ Remoulded	WATER CONTENT % 25 50 75	
0.0			1	SS	9					
			2	TW	FM					108
			3	TW	FM					109
			4	TW	FM					109 114.5
	Varved Clay Layers of clayey silt and clay 1/8 - 1 1/2" thick.		5	TW	FM					107 111
	Firm to stiff		6	TW	FM					112
			7	TW	FM					
			8	TW	FM					117
510.0										
89.5	Sandy silt, traces of clay.		9	SS	111					
	Dense to very dense/		10	Wash						
			11	Wash						
			12	SS	54					Sa. 19, Si. 75 Cl. 6
			13	SS	48					
476.8			14	SS	1/31					
131.7	Bedrock		15	RC	BIT					Sa. 40, Si. 50 Cl. 10
471.8	Dolomite				100%					
236.7	End of Borehole					470				

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 4

FOUNDATION SECTION

JOB 87-P-57

LOCATION Sta. 75 + 65 31' E Rt.

ORIGINATED BY AP

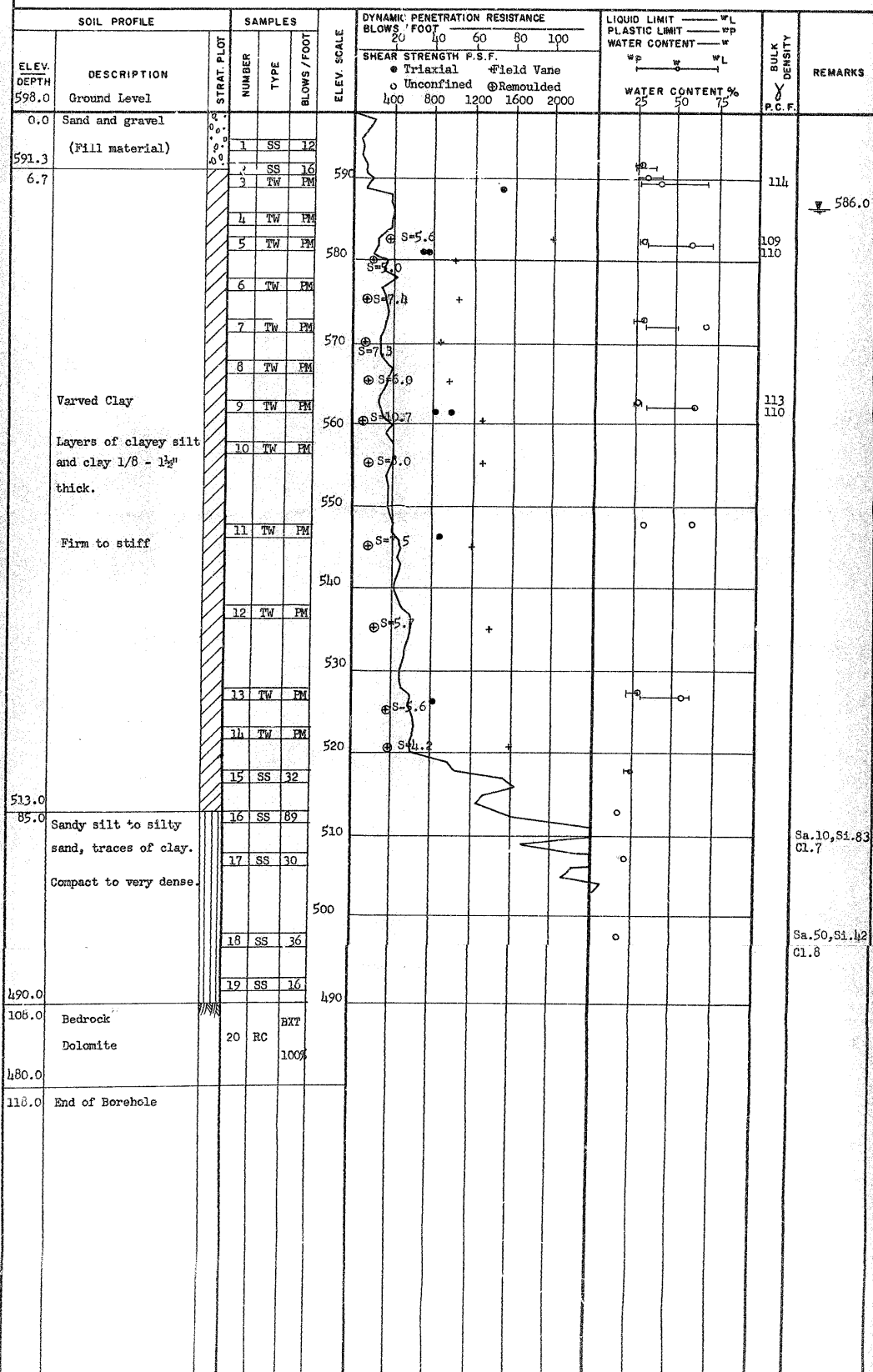
W. P. 188-66

BORING DATE July 12 - July 19, 1967

COMPILED BY AP

DATUM Geodetic

BOREHOLE TYPE Washboring, NX & BX Casing, Cone

CHECKED BY *SR*

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & TESTING DIVISION

67-F-57

LOCATION Sta. 76 + 04 32' Lt.

ORIGINATED BY AP

W. F. 188-66

BORING DATE July 20, 1967

COMPILED BY AP

DATUM Geodetic

BOREHOLE TYPE Dynamic Cone Penetration

CHECKED BY

[illegible]

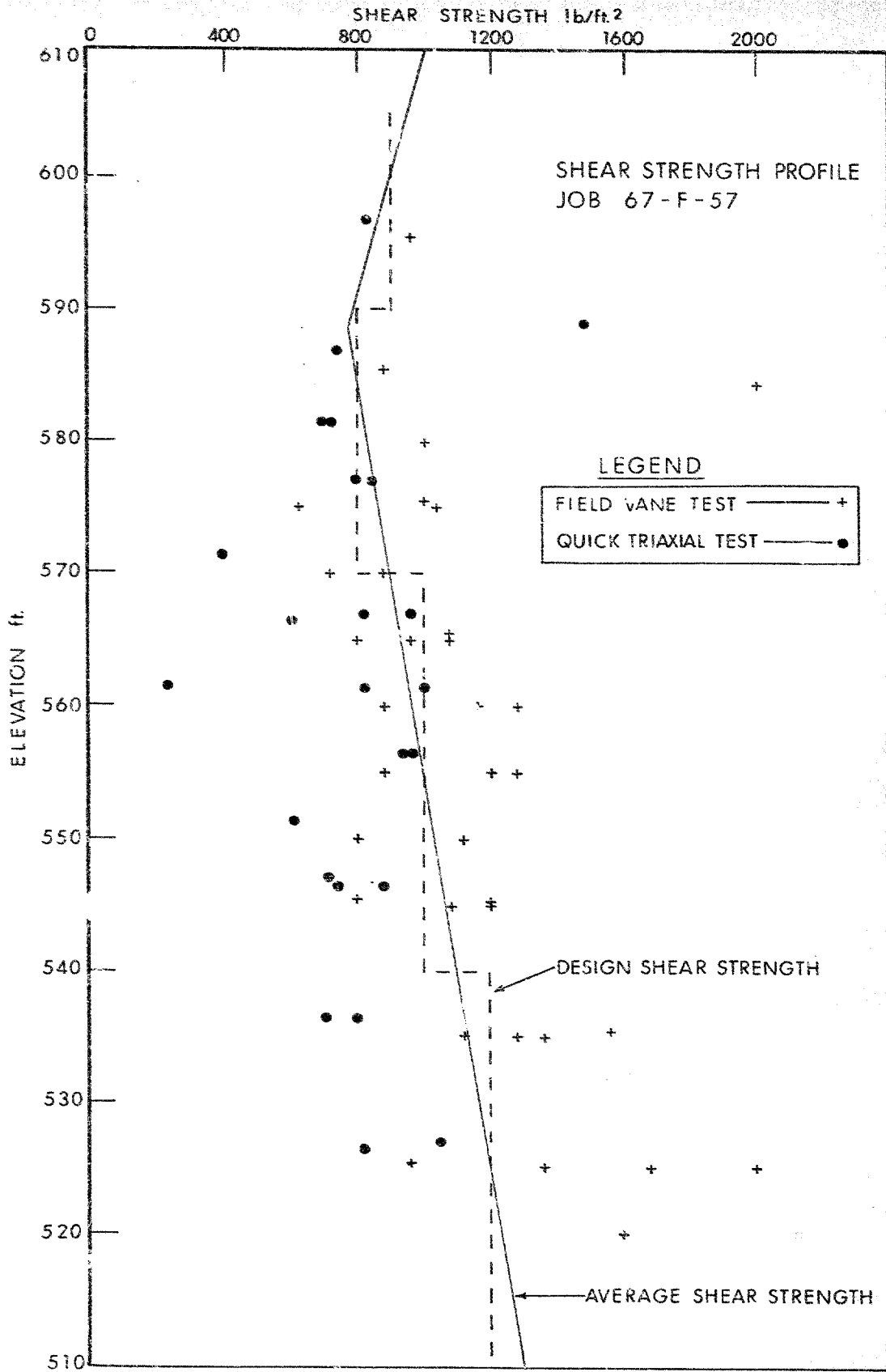
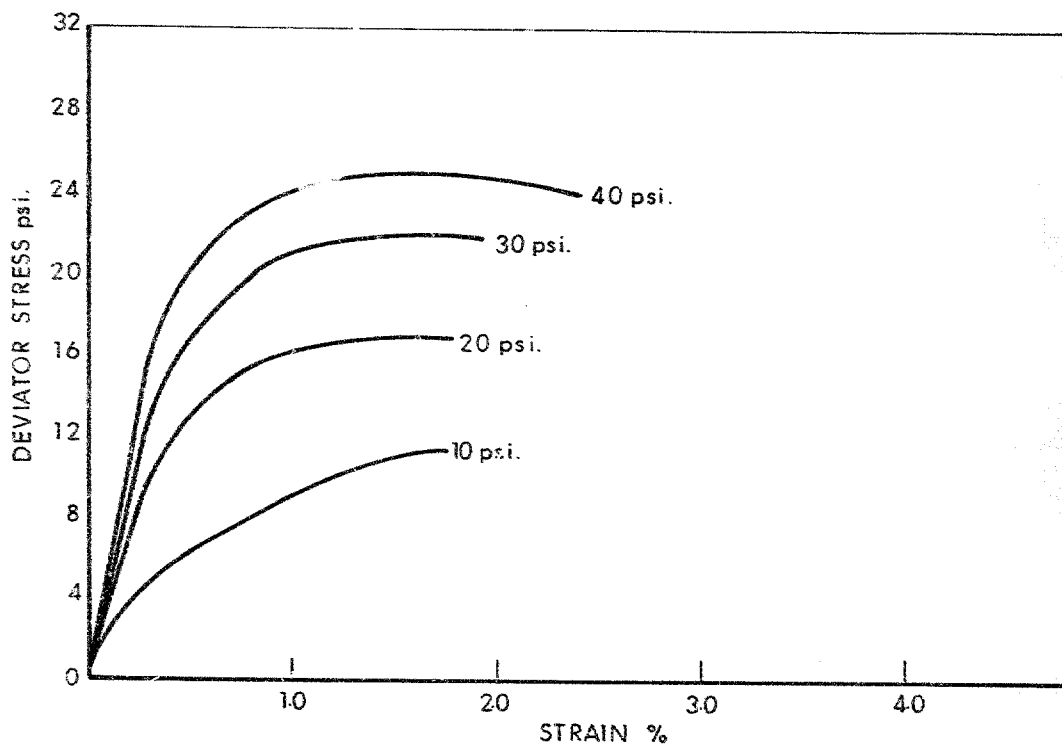
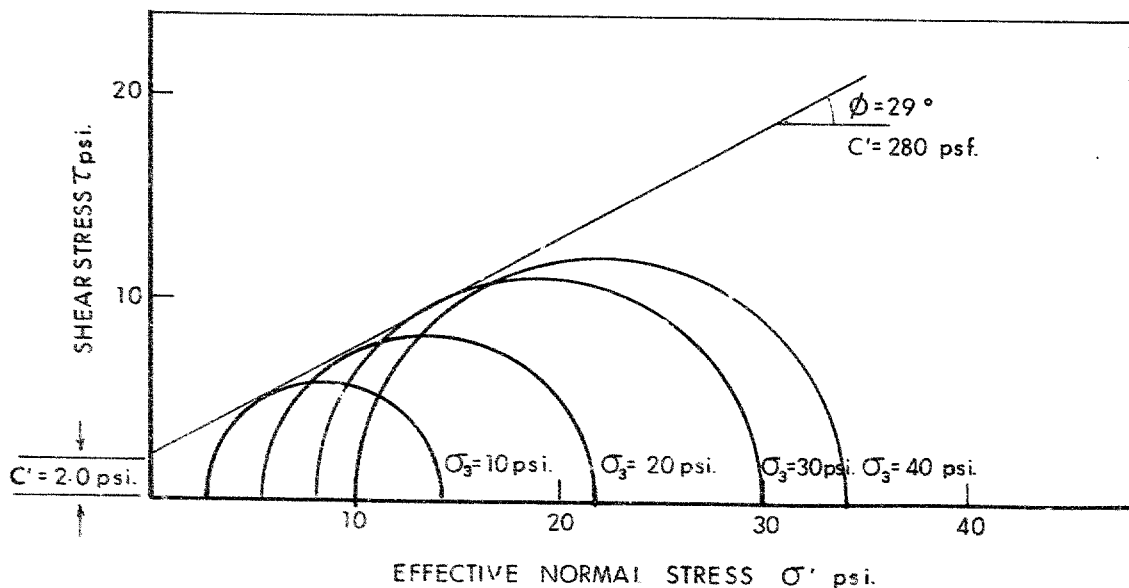


FIG. 1

CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION STAGE TESTS

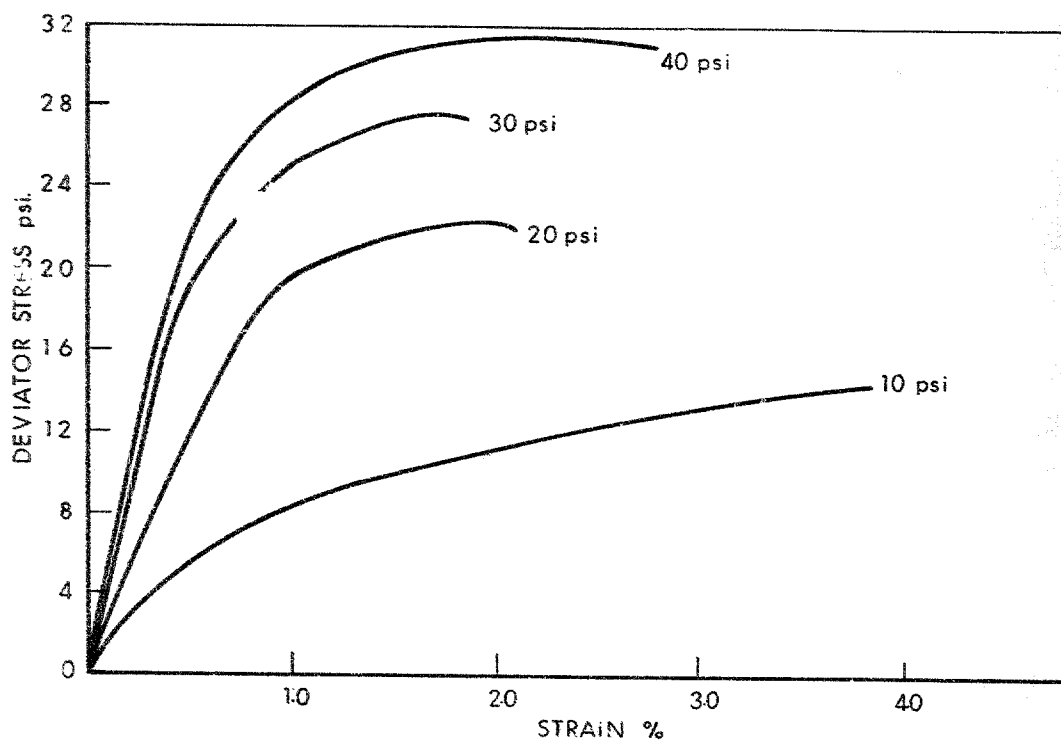
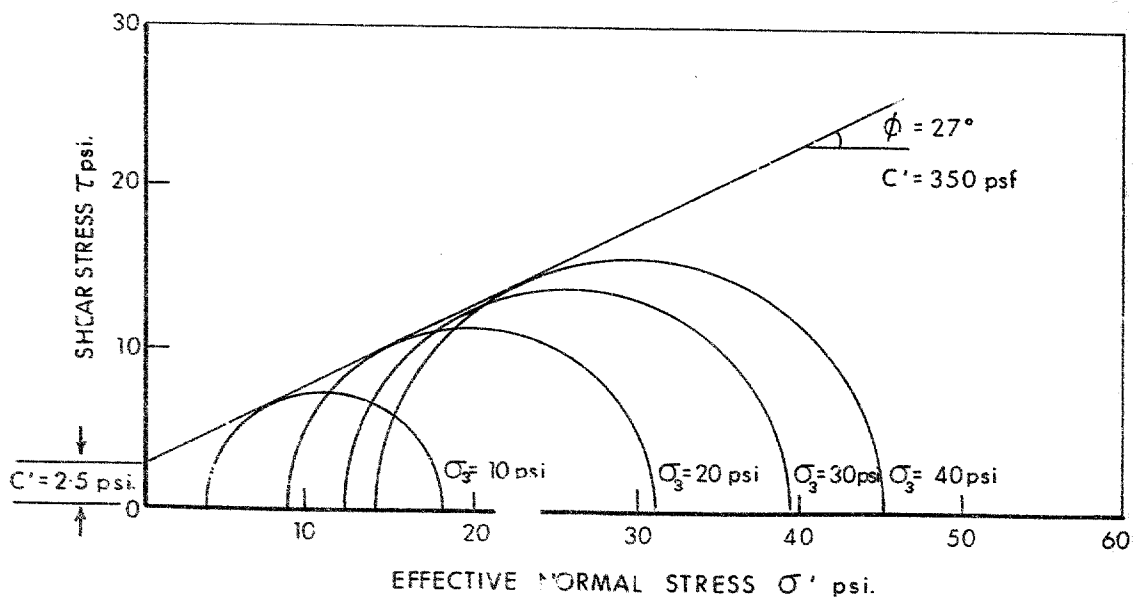


BORE HOLE _____ 1
 SAMPLE NO. _____ 4
 DEPTH _____ 26' 1"
 BULK DENSITY _____ 110.8 psf.

INITIAL MOISTURE CONTENT _____ 42.4 %
 FINAL MOISTURE CONTENT _____ 38.1 %
 σ_3 CONSTANT
 σ_1 INCREASING

FIG. 2a

CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION STAGE TESTS

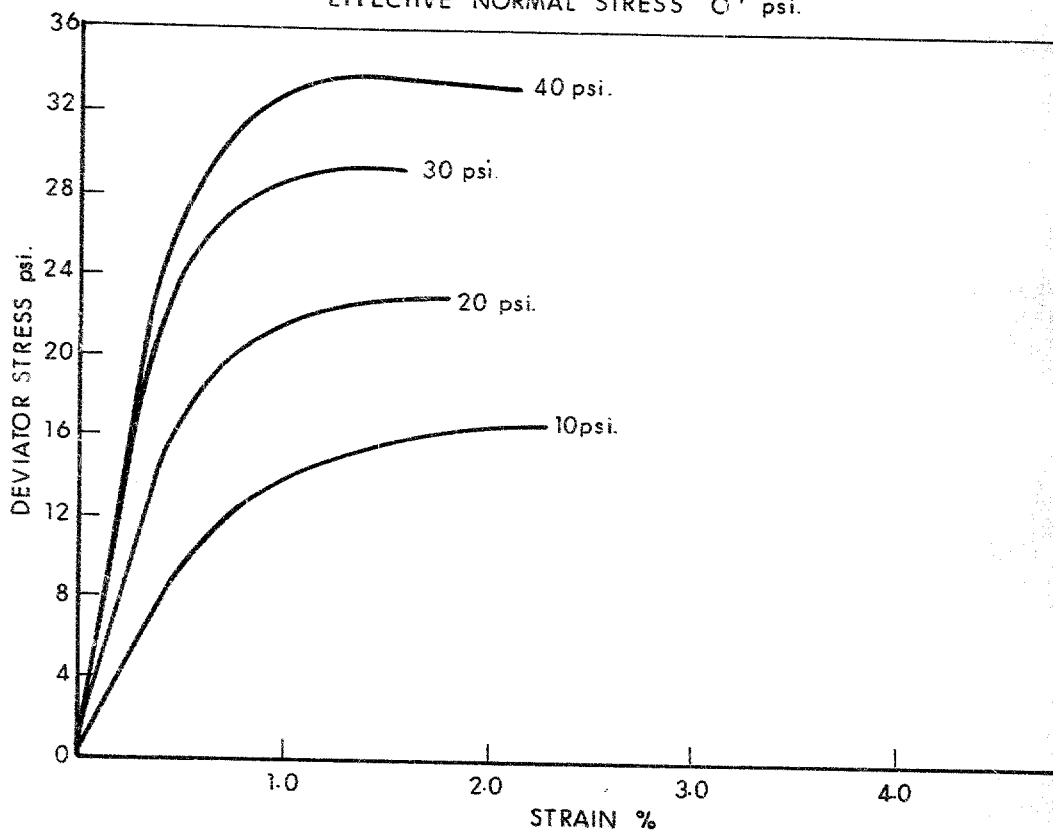
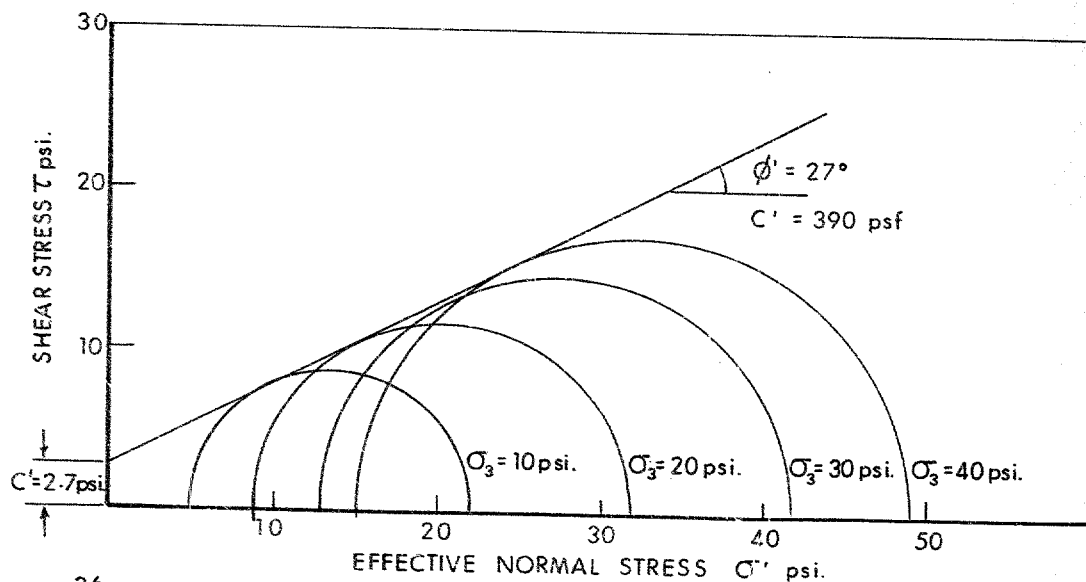


BORE HOLE _____ 1
 SAMPLE NO. _____ 6
 DEPTH _____ 35'8"
 BULK DENSITY _____ 109.9 psf

INITIAL MOISTURE CONTENT _____ 42.9 %
 FINAL MOISTURE CONTENT _____ 36.6 %
 σ_3 CONSTANT
 σ_1 INCREASING

FIG. 2b

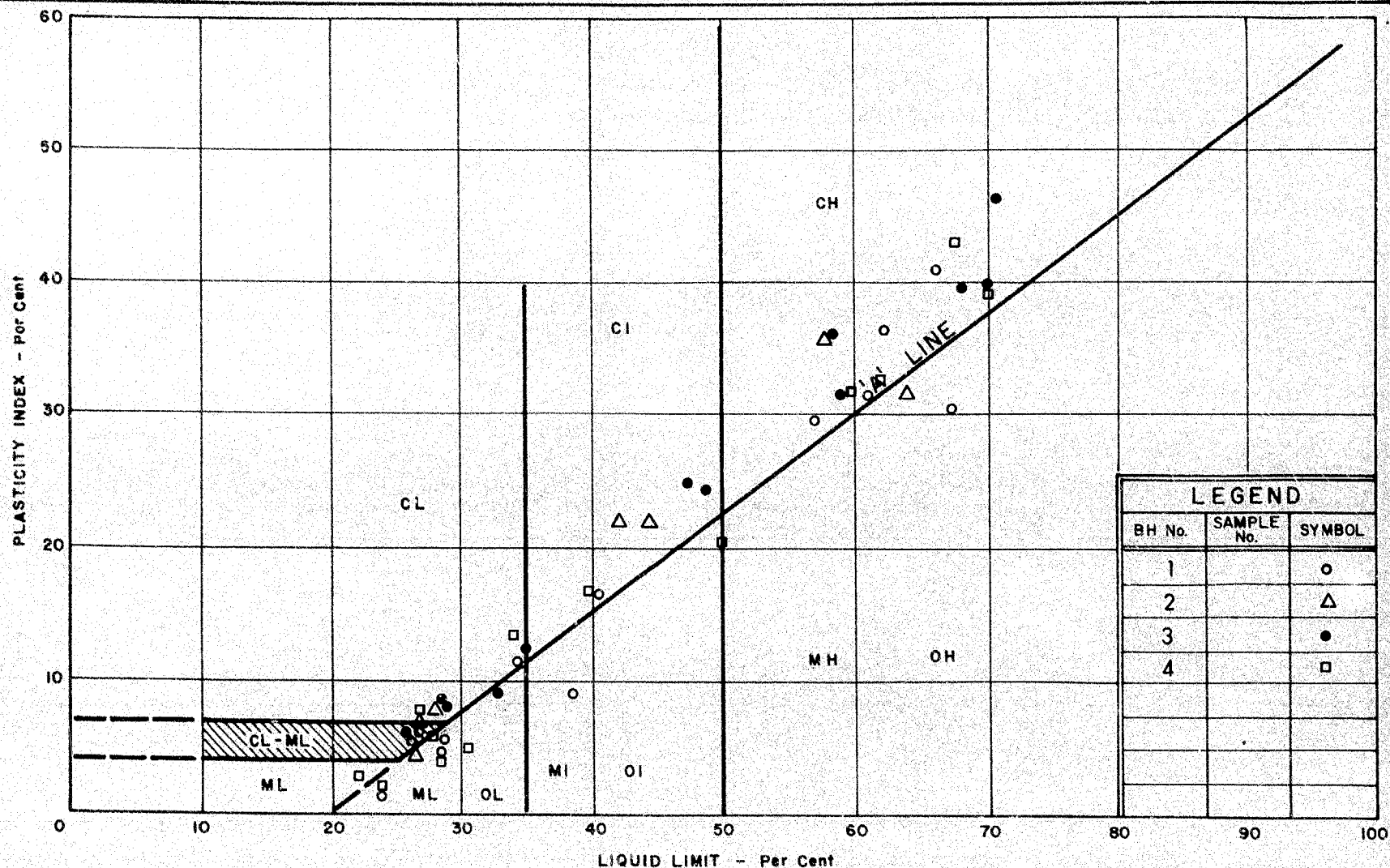
CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION STAGE TESTS



BORE HOLE _ _ _ _ 3
 SAMPLE NO. _ _ _ _ 7
 DEPTH _ _ _ _ _ 71'1"
 BULK DENSITY _ _ _ 111.4 psf.

INITIAL MOISTURE CONTENT _ 41.5 %
 FINAL MOISTURE CONTENT _ 37.3 %
 σ_3 CONSTANT
 σ_1 INCREASING

FIG. 2c



DEPARTMENT OF HIGHWAYS
 MATERIALS and
 TESTING
 DIVISION

PLASTICITY CHART

FIG. 3

W.P. No. 188-66

JOB No. 67-F-57

ABBREVIATIONS USED IN THIS REPORT

PENETRATION RESISTANCE

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

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

DESCRIPTION OF SOIL

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

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

TYPE OF SAMPLE

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

SOIL TESTS

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

ABBREVIATIONS USED IN THIS REPORT

SOIL PROPERTIES

γ	UNIT WEIGHT OF SOIL (BULK DENSITY)
	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	SHEAR STRENGTH
c	EFFECTIVE COHESION
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
$\bar{\sigma}$	NORMAL EFFECTIVE STRESS ($\bar{\sigma}$ IS ALSO USED)
τ	SHEAR STRESS
ϵ	LINEAR STRAIN
γ	SHEAR STRAIN
ν	POISSON'S RATIO (μ IS ALSO USED)
E	MODULUS OF LINEAR DEFORMATION (YOUNG'S MODULUS)
G	MODULUS OF SHEAR DEFORMATION
K	MODULUS OF COMPRESSIBILITY
η	COEFFICIENT OF VISCOSITY

EARTH PRESSURE

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

FOUNDATIONS

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

SLOPES

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

cc: G Files

W.P. 188-66

Re: Wabi Creek

Bridge

Mr. S. McCombie,
Bridge Planning Engineer,
Bridge Div., Admin. Bldg.,
Downsview.

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

Attn: Mr. J. B. Curtis,
Reg. Bridge Location Engr.,
North Bay.

December 1, 1967

Wabi Creek Bridge - New Liskeard
Highway #11B -- District No. 14
W.J. 67-F-57 -- W.P. 188-66

Re your memo of November 30, 1967, we consider it unlikely that additional borings will be required within the creek bed, in the event that the detour will be located some 80 to 100 feet upstream of the new bridge. If the abutments of the detour structure are to be founded on a rock-filled crib, it would be advisable, of course, to check the soil conditions at these locations. As you suggest, it will be wiser to wait until the preliminary bridge plan and the proposed footing locations of the detour are available before deciding whether additional information is required.

K. G. Selby

KGS/XdeF

K. G. Selby,
SUPERVISING FOUNDATION ENGR.
For:
A. G. Stermac,
PRINCIPAL FOUNDATION ENGR.

cc: Mr. S. McCombie

Foundations Files
Gen. Files ✓

ack

RECOMMENDATIONS
LAKESIDE CREEK
BRIDGE

Mr. J. B. Curtis,
Regional Bridge Location Engr.,
Regional Office,
NORTH BAY, Ont.

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

August 9, 1968

WABI CREEK BRIDGE
New Liskeard, Hwy. 118
District 14
W.P. 188-66, B.S. 47-53

With reference to your memorandum of July 31, 1968, to Mr. A. K. McKim, we wish to make the following comments:

It is very difficult to predict whether the pile driving will cause any damage to the buildings in the immediate proximity of the site. The soil (varved clay) is quite soft for a considerable distance below ground level and no resistance to pile driving is expected. However, when the piles reach the dense stratum overlying bedrock, in which they are supposed to penetrate, there might be vibrations due to pile driving. Whether these would be significant enough to cause any damage is difficult if not impossible to forecast.

In view of this and of the fact that Mr. H. T. Hutchinson has voiced concern, we would recommend that a careful and meticulous assessment of the conditions be carried out prior to the start of construction.

All the existing buildings in the proximity should be examined, and all cracks or similar signs of structural distress be noted and photographed.

During the pile driving operation, periodic checks will have to be carried out, and upon completion of the project, a final check should be made.

We feel that in case damage to private property has been caused by the necessary and unavoidable action of the contractor on this job, the Department will have to compensate the party or parties involved.

The proposed procedure would only prevent the Department from being held responsible for damages not caused by its action or actions.

We agree with Mr. Hutchinson that the soils in the area are bad, and we are not in the least surprised that he is experiencing problems along the river banks.

AGS/MdeF

cc: Messrs. A. E. McKim
D. A. O. White

Foundations Files
Gen. Files

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

AR

Re: Everglades
Flood Prevention
✓ Canal Creek B

Mr. A. McKim
Bridge Contr. Engineer
Admin. Bldg.

Foundation Section
Lab. Bldg. Room 107

March 5, 1960

Contract 88-174 - Oak Creek Bridge
at New Richmond - Highway 11 E
District 15
W.P. 182-66 - W.J. 87-F-71

Some discussions were held recently between the writer and Mr. A. Radkowski and Mr. Kingsland at the Bridge Office concerning the excavations which have to be carried out for the pier foundations at the above-mentioned contract. The contractor has advised Mr. Carney, District Construction Engineer, that in his opinion the steel sheet coffer dams shown on the drawings are not deep enough to provide against a possible base failure of the excavation.

Mr. Radkowski has advised us that the main purpose of the sheeting shown on the drawings is to act as formwork and to aid in dewatering after the concrete is poured. It is not intended to provide against the problem which concerns the contractor.

It is our opinion that in this case the temporary conditions which prevail during the course of construction are the responsibility of the contractor and it is up to him to take such steps as are necessary to ensure stability of the excavations. We feel, however, that it would be expedient at this time to advise the contractor at this early stage in the work of the Department's attitude in this matter. We also feel that it would be wise to advise him of the following:

1. The cofferdams must be braced laterally to prevent lateral earth movements. Sufficient earth must be placed around the sheeting to ensure that one side can be successfully braced against the other.
2. Under certain conditions which might occur during construction it is possible that a base failure in the form of an upward heave of the excavation base would take place. This type of failure can be prevented by advancing the sheeting to a greater depth or by removing some of the earth adjacent to the cofferdam. In any event it would be necessary to know the most unfavourable conditions which would prevail during construction in order to specify definite preventative measures.
3. Ground heave is likely to take place in the cofferdams due to displacement of soil as the steel H piles are driven. This type of heave should not be confused with the failure referred to in (2.). The deeper the sheeting is of course the greater will be the confining effect and hence the greater will be the ground heave.

W. G. Selby

RG:ant

cc: Messrs. A. Radkowski
R. Carney (2)
E. Bryson
D. Hopper

W. G. Selby
Supervising Foundation Engineer
Chief of Division

Mr. A. McKim
Bridge Control Engineer
Admin. Bldg.

Foundation Section
Lab. Bldg. Room 107

March 5, 1969

Contract 68-172 - Webb Creek Bridge
at New Lisheard - Highway 11 B
District 14
W.P. 100-66 - W.J. 67-2-37

Some discussions were held recently between the writer and Mr. A. Radkowski and Mr. Kingsland at the Bridge Office concerning the excavations which have to be carried out for the pier foundations at the above-mentioned contract. The contractor has advised Mr. Carney, District Construction Engineer, that in his opinion the steel sheet coffer dams shown on the drawings are not deep enough to provide against a possible base failure of the excavation.

Mr. Radkowski has advised us that the main purpose of the sheeting shown on the drawings is to act as formwork and to aid in centering after the concrete is poured. It is not intended to provide against the problem which concerns the contractor.

It is our opinion that in this case the temporary conditions which prevail during the course of construction are the responsibility of the contractor and it is up to him to take such steps as are necessary to ensure stability of the excavations. We feel, however, that it would be expedient at this time to advise the contractor at this early stage in the work of the Department's attitude in this matter. We also feel that it would be wise to advise him of the following:

1. The cofferdams must be braced laterally to prevent lateral earth movements. Sufficient earth must be placed around the sheeting to ensure that one side can be successfully braced against the other.
2. Under certain conditions which might occur during construction it is possible that a base failure in the form of an upward heave of the excavation base would take place. This type of failure can be prevented by advancing the sheeting to a greater depth or by removing some of the earth adjacent to the cofferdams. In any event it would be necessary to know the most unfavorable conditions which would prevail during construction in order to specify definite preventative measures.
3. Ground heave is likely to take place in the cofferdams due to displacement of soil as the steel H piles are driven. This type of heave should not be confused with the failure referred to in (2.). The deeper the sheeting is of course the greater will be the confining effect and hence the greater will be the ground heave.

Encsmt

cc: Messrs. A. Radkowski
R. Carney (2)
B. Tregubov
D. Harper

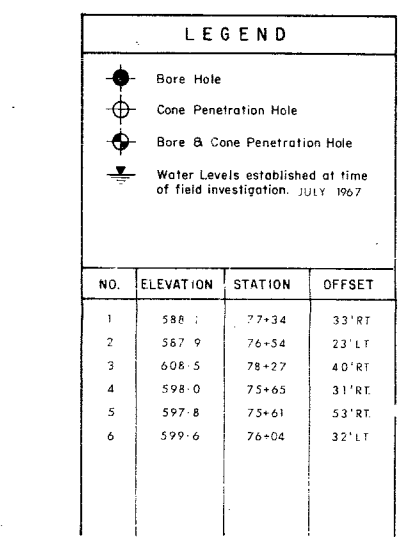
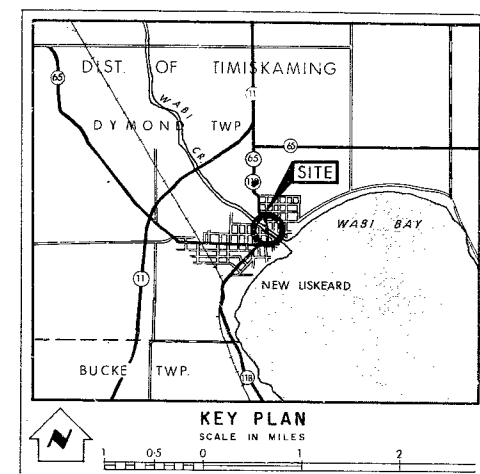
H. G. Solby
Supervising Foundation Engineer
Principal Foundation Engineer

#67-F-57

W.P. #188-66

HWY. #11B AND #65

WABI CREEK



- 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 B. TESTING DIVISION - FOUNDATION SECTION			
<h1>WABI CREEK</h1>			
KING'S HIGHWAY NO. 11B & 65		DIST. NO. 14	
DIST. TIMISKAMING		TOWN OF NEW LISKEARD	
TWP. DYMOND		LOT 9 CON. III	
<h2>BORE HOLE LOCATIONS & SOIL STRATA</h2>			
SUBMD. A.P. DRAWN MJD	CHECKED <input checked="" type="checkbox"/> CHECKED <input checked="" type="checkbox"/>	W.P. NO. 188-66 JOB NO. 67-F-57	M.B.T. DRAWING NO. <h1>67-F-57A</h1>
DATE 31 AUG 1967		SITE NO.	BRIDGE DRAWING NO.
APPROVED <i>A. J. Thomas</i> <small>PRINCIPAL FOUNDATION ENGINEER</small>		CONT. NO.	

[illegible]