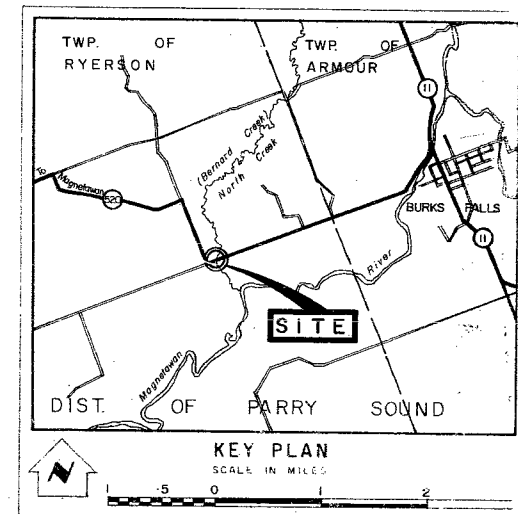
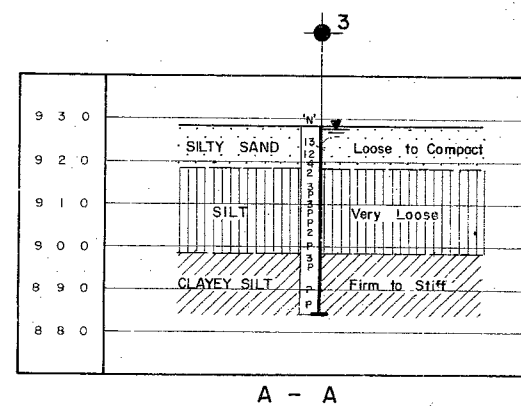
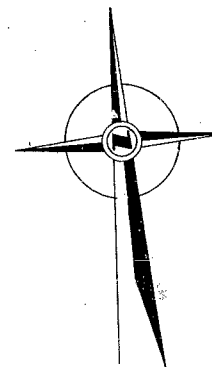
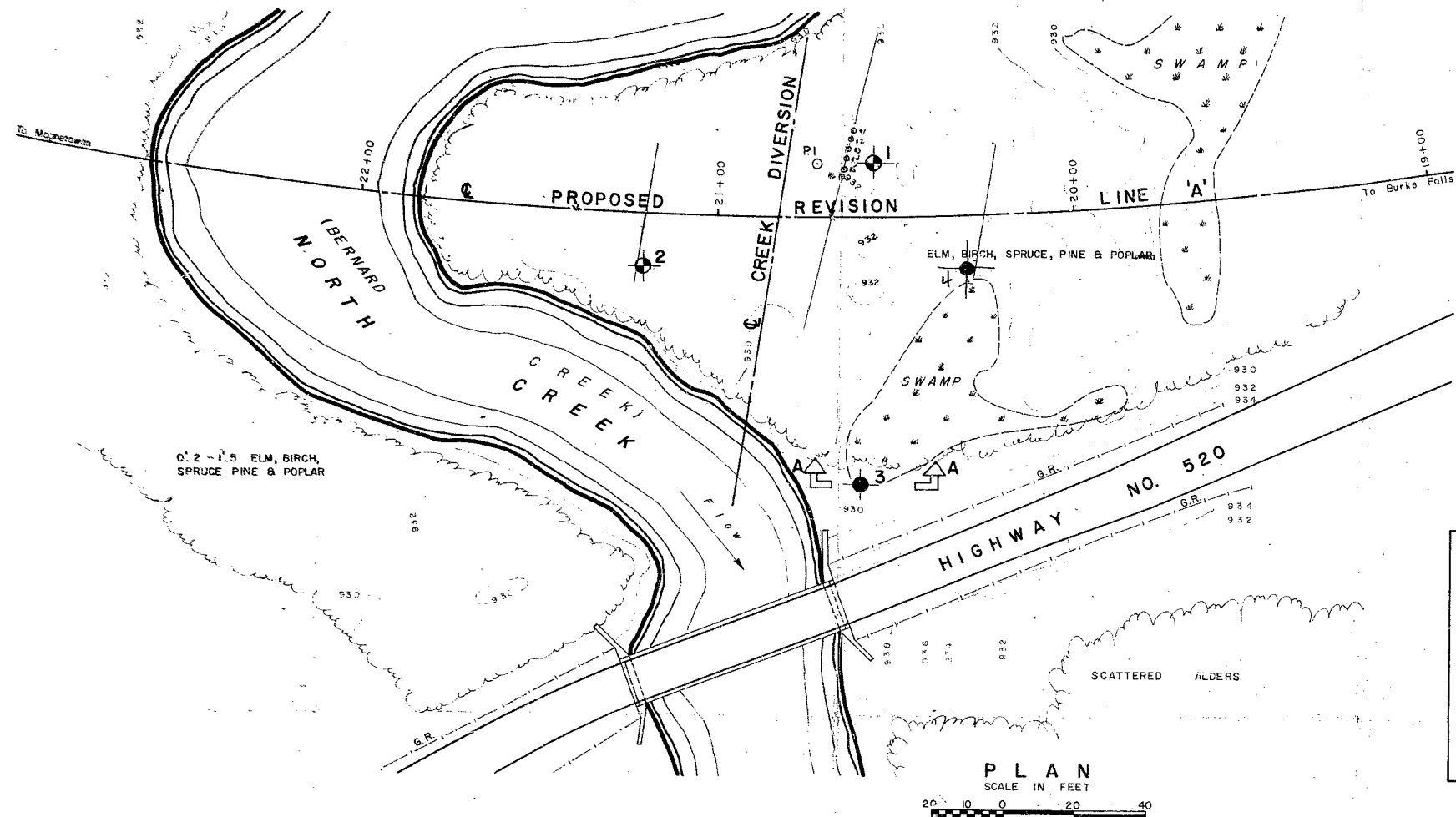


#63-F-80

W.P. #167-62

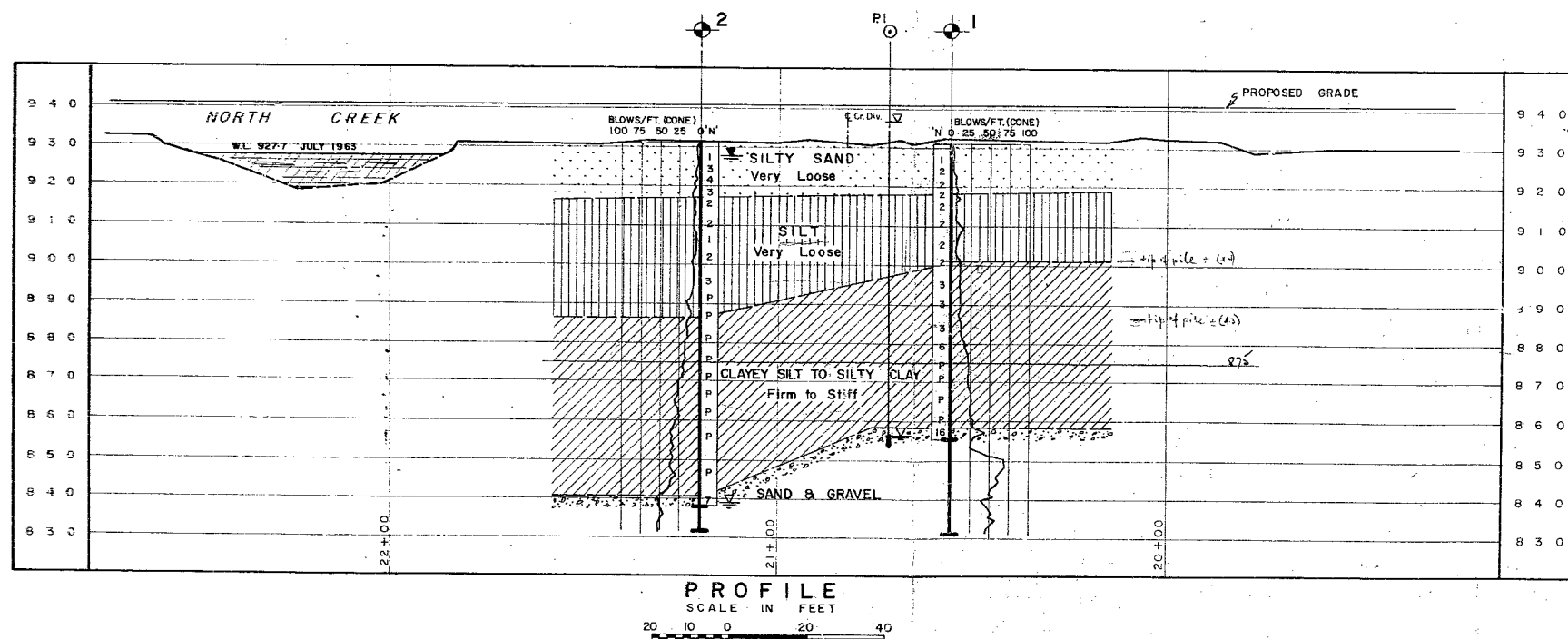
SEC. Hwy. #520

E' NORTH CR.



LEGEND			
	Bore Hole		
	Cone Penetration Hole		
	Bore & Cone Penetration Hole		
	Water Levels established at time of field investigation (July 1963)		
	Piezometer		
NO.	ELEVATION	STATION	OFFSET
1	931.1	20+56	15' RT.
2	931.4	21+20	15' LT.
3	928.6	20+60	75' LT.
TIP ELEV. OF PIEZOMETER			
P1	853.1	20+72	15' 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.



NOTE
ARTESIAN HEAD
ARTESIAN CONDITION ENCOUNTER

NO.	DATE	BY	DESCRIPTION
1			
2			
3			
4			
5			
6			
7			
8			
9			
10			

DEPARTMENT OF HIGHWAYS - ONTARIO			
MATERIALS & RESEARCH DIVISION - FOUNDATION SECTION			
NORTH CREEK (BERNARD CREEK)			
KING'S HIGHWAY NO. 520 PROP. REV. LINE 'A' DIST. NO. 11			
DIST. PARRY SOUND			
TWP. RYERSON LOT 5 CON. IX			
BORE HOLE LOCATIONS & SOIL STRATA			
SUBM.D. R.M.	CHECKED	W.P. NO. 167-62	M.B.R. DRAWING NO.
DRAWN D.M.	CHECKED	JOB NO. 63-F-80	63-F-80 A
DATE 14 AUGUST 1963	SITE NO.	BRIDGE DRAWING NO.	
APPROVED	CONT. NO.		

REF. NO. E-4202-1

Mr. A. M. Towe,
Bridge Engineer,
Bridge Division.

Attention: Mr. S. McCombie

23-64-331
Mr. A. G. Stermac,
Principal Foundation Engr.,
Foundation Section,
Materials & Research Division.

September 5, 1963

FOUNDATION INVESTIGATION REPORT

For

Proposed North Creek Structure,
Sec. Hwy. 520, Line 'A'
District #11, Huntsville

W. J. 63-F-80 -- W.P. 167-62

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

We believe that you will find the factual data
and recommendations contained therein, adequate for your future
design work. Should additional information be required, please
feel free to contact our Office.

KYL/MdeF
Attach.

cc: Messrs. A. M. Towe (2)
H. A. Tregaskes
H. D. McMillan
H. McArthur
E. H. Jones
T. ...
A. ...

KYL
K. Y. Lo,
SUPERVISING FOUNDATION ENGR.
For:
A. G. Stermac,
PRINCIPAL FOUNDATION ENGR.

Foundations Office
SEP 11 1963

TABLE OF CONTENTS

1. INTRODUCTION.
 2. DESCRIPTION OF THE SITE.
 3. FIELD INVESTIGATION PROCEDURE.
 4. LABORATORY TESTS.
 5. SUBSOIL CONDITIONS:
 - 5.1) General.
 - 5.2) Silty Sand.
 - 5.3) Silt.
 - 5.4) Clayey Silt to Silty Clay.
 - 5.5) Sand and Gravel.
 6. GROUND WATER CONDITIONS.
 7. DISCUSSION & RECOMMENDATIONS.
 8. SUMMARY.
 9. MISCELLANEOUS.
-

FOUNDATION INVESTIGATION

For

Proposed North Creek Structure,
Sec. Hwy. 520, Line 'A'
District #11, Huntsville
W.J. 63-F-80 -- W.P. 167-62

1. INTRODUCTION:

A request dated July 10, 1963, for a foundation investigation at the site of the proposed crossing of Sec. Hwy. 520 . (Line 'A') and North Creek, was received by the Foundation Section.

A field investigation was subsequently carried out by this Section during July 1963, to determine the subsoil conditions at the site of the proposed structure. Presented in this report are the results of this investigation, together with recommendations pertaining to the design of structure foundations and approach embankments.

2. DESCRIPTION OF THE SITE:

The site is located approximately 3 miles West of Burk's Falls. The surrounding area is gently undulating with no sharp protuberances.

Physiographically, the site is located in the region referred to as the "Canadian Shield".

3. FIELD INVESTIGATION PROCEDURE:

A total of 3 boreholes and 2 dynamic cone penetration tests were carried out during the course of the field investigation.

cont'd. /2 ...

3. FIELD INVESTIGATION PROCEDURE: (cont'd.) ...

Borehole 3 was performed in an attempt to assess the subsoil conditions of the existing structure. Boring was achieved by means of conventional diamond drilling equipment adapted for soil sampling purposes. Undisturbed soil samples were obtained by means of 2-inch I.D. Shelby tubes, which were pushed manually into the soil. Disturbed samples were recovered by means of a standard 2" O.D. split-spoon sampler. In-situ vane tests were carried out wherever possible, in cohesive deposits. A Norwegian piezometer was installed at 78 ft. in the sand and gravel in order to observe the water level of the stratum. The location and the details of the piezometer are shown on Dwg. No. 63-F-80A.

The locations and elevations of all boreholes are shown on Dwg. No. 63-F-80A which accompanies this report. All elevations are referred to a G.B.M. located on the abutment wall of the existing structure.

4. LABORATORY TESTS:

Samples were visually examined and identified in the laboratory as well as in the field. Tests were carried out in the laboratory for the determination of Atterberg limits, moisture contents, grain size distributions and shear strength measurements.

The laboratory test results have been summarized and are included in this report in Appendix I.

5. SUBSOIL CONDITIONS:

5.1) General:

The subsoil conditions at the site were found to be generally uniform, with minor variations, only. Detailed

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

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

descriptions of various soil types encountered in each boring are given in Appendix I of this report. The estimated stratigraphical profile of Dwg. No. 63-F-80A is based upon this information.

From ground level downwards, the various soil types encountered are as follows:

5.2) Silty Sand:

This layer was found immediately below the ground level in all boreholes. The depth of the layer varied between 13 ft. and 14 ft. in boreholes 1 and 2. Standard Penetration test or 'N' values ranged from 1 to 4 blows/ft., indicating a relative density of very loose to loose. In borehole 3, the depth of the stratum was 14 ft., with 'N' values ranging from 4 to 13, corresponding to a relative density of loose to compact.

5.3) Silt:

Extending downwards from the silty sand and found in all boreholes, is a stratum of silt. The thickness of the layer was found to vary from 17 ft. in B.H. 1 to 31 ft. in B.H. 2. Standard penetration resistances for this material vary from 2 to 3 blows/ft., indicating that the relative density is very loose.

Atterberg limits and moisture contents have been determined on the comparatively cohesive parts of the deposit. The range of the liquid and plastic limits is 20% - 23% and 16% - 24%, respectively. It has to be noted that the moisture content may be slightly too high since wash boring procedure was employed. The order of magnitude of the liquidity index is, however, considered to be representative.

cont'd. /4 ...

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

5.4) Clayey Silt to Silty Clay:

Immediately beneath the silt, the stratum of clayey silt to silty clay was observed. The stratum was proved to its maximum depth in B.H.'s 1 and 2, only, where its thickness was found to be 43 ft. and 46 ft., respectively. Generally, the first 10 ft. of the stratum is clayey silt, whereas the remainder is silty clay.

Comparative laboratory results of Atterberg limits, moisture contents, and bulk densities for the clayey silt and silty clay portions, are given below:

Clayey Silt -

Liquid Limit	--	26% - 33%
Plastic Limit	--	17% - 23%
Moisture Content	--	30% - 36%
Bulk Density	--	111 p.c.f. - 121 p.c.f.

Silty Clay -

Liquid Limit	--	36% - 57%
Plastic Limit	--	20% - 27%
Moisture Content	--	33% - 52%
Bulk Density	--	107 p.c.f. - 120 p.c.f.

The shear strength and 'N' values for this deposit were determined to be as follows:

In-situ Vane Test	--	1040 p.s.f. - > 2000 p.s.f.
Undrained Shear Strength	--	935 p.s.f. - 1810 p.s.f.
'N' Values	--	3 - 6

Based on these values, the consistency of the stratum may be described as firm to stiff.

cont'd. /5 ...

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

5.5) Sand and Gravel:

This stratum was found immediately below the deposit of clayey silt to silty clay. It was encountered in B.H.'s 1 and 2, 73 ft. and 91 ft. below the ground level, respectively. Due to an artesian condition encountered in this material, the 'N' values obtained are considered unreliable.

6. GROUND WATER CONDITIONS:

During the foundation investigation, ground water observations were carried out. These indicate that the ground water elevation varies very slightly over the site, generally being at approximate elevation 928.

During the period of observation, an artesian head corresponding to an elev. 937.8 in the sand and gravel stratum was observed.

The exact water levels observed during the investigation as well as the locations and depths of the piezometers, are shown on Dwg. No. 63-F-80A.

7. DISCUSSION & RECOMMENDATIONS:

It is proposed to realign Hwy. 520 to cross the diversion of North Creek some 100 ft. North of the present bridge. The Centre Line of the proposed Hwy. 520 and the North Creek diversion are shown on Dwg. No. 63-F-80A of this report. The present structure is a 60-ft. single span, Warren truss type bridge resting on concrete abutments which are supported on timber piles. At the present time,

cont'd. /6 ...

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

this bridge is in good condition, showing no signs of movement.

A single span, 60-ft. long, 30-ft. wide structure is proposed at the crossing of proposed revision, Line 'A', Hwy. 520 and relocated North Creek (Bernard Creek). Subsoil at this site consists of 30 ft. to 45 ft. of very loose silty sand and silt. Underlying the silt and overlying the stratum of sand and gravel, is a deposit of firm to stiff clayey silt and silty clay.

Because of the presence of loose silty sand, followed by very loose silt, adequate bearing capacity for spread footing support cannot be achieved in the sand or silt strata.

Therefore, the structure should be supported on piled foundations. Very long piles will be required if the design is based on conventional theories using the Standard Penetration resistances or 'N' values obtained in the sand and silt deposits.

During the time of the field investigation, the existing bridge was inspected and found in satisfactory condition. No tilting of the abutments nor excessive settlements at the abutment locations could be observed. It was also ascertained from local sources that the existing structure is supported on 40-ft. timber piles. In order to determine whether the subsoil conditions may vary at the existing structure as compared with the proposed structure location, a boring was carried out close to the Northeast corner of the existing structure. This boring revealed that the subsoil conditions are practically identical to those at the proposed structure location. At the crossing of Sec. Hwy. 605 and the Magnetawan River, some 10 miles Northeast of the present site,

cont'd. /7 ...

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

similar conditions were encountered. At that location, a pile loading test was carried out (W.J. 61-F-93) in order to determine the safe bearing capacity as well as the length of pile required. The loading tests proved that a safe load of 20 tons/pile can be used by driving 45' long, 12" Ø class 'B' timber piles.

In view of the above-mentioned reasons, it is recommended that pile loading tests be carried out at the proposed structure site in order to determine the safe bearing capacity as well as the required length of the piles. This loading test should be carried out under the supervision of the Foundation Section, with all the necessary details prepared by this Section.

No stability problems are anticipated for the approach fills, provided standard 2:1 slopes are used.

8. SUMMARY:

Subsoil at the site consists of sands, silts, clayey silts, silty clays and sand and gravel.

A piled foundation for the proposed structure is recommended. In order to determine the safe bearing capacity as well as the required lengths of piles, a pile loading test at the site, under the general supervision of the Foundation Section, should be carried out.

No embankment stability problems are anticipated.

cont'd. /8 ...

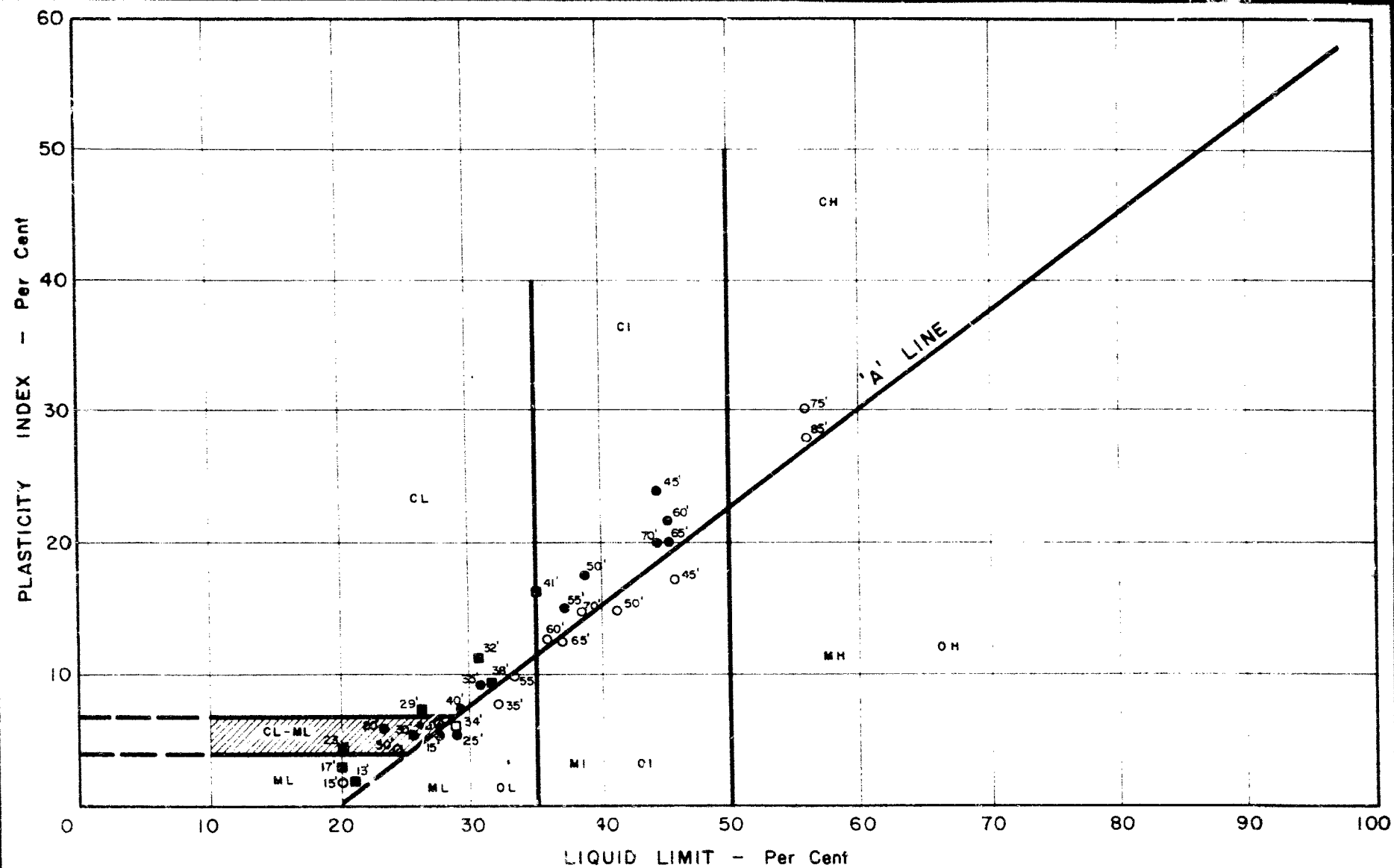
9. MISCELLANEOUS:

The field work, performed during July, 1963, together with the preparation of this report, was undertaken by Mr. R. Magi, Project Foundation Engineer. The investigation was carried out under the general supervision of Mr. M. Devata, Senior Foundation Engineer, who also reviewed this report.

Equipment was owned and operated by Johnston Drilling Co. Ltd. of Ottawa.

September 1963

APPENDIX 1.



NOTES	Bore	Hole	No. 1	—	●
	"	"	" 2	—	○
	"	"	" 3	—	■

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & RESEARCH DIVISION
PLASTICITY CHART

Job No. 63 - F - 80

W.P. No. 167 - 62

Location BURKS FALLS (NORTH CREEK & HWY. 520)

ABBREVIATIONS USED IN THIS REPORT

PENETRATION RESISTANCE

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

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

DESCRIPTION OF SOIL

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

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

TYPE OF SAMPLE

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

SOIL TESTS

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

ABBREVIATIONS USED IN THIS REPORT

SOIL PROPERTIES

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

GENERAL

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

STRESS AND STRAIN

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

EARTH PRESSURE

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

FOUNDATIONS

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

SLOPES

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

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & RESEARCH DIVISION

RECORD OF BOREHOLE NO. 1

FOUNDATION SECTION

JOB 63-F-80

LOCATION 15' Rt., Sta. 20456

ORIGINATED BY R.M.

W.P. 167-62

BORING DATE July 19, 22 & 23, 1963.

COMPILED BY R.M.

DATUM Geodetic

BOREHOLE TYPE Dynamic Cone Penetration & Washboring

CHECKED BY M.D.

SOIL PROFILE			SAMPLES			ELEV SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT					LIQUID LIMIT — WL PLASTIC LIMIT — WP WATER CONTENT — W			BULK DENSITY P.C.F.	ELEV. Art. Head
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT		20	40	60	80	100	WP	WL	W		
931.1 0	Groundlevel					930										ELEV. 937.0
	Silty sand.		1	SS	1											ELEV. 928.1
			2	SS	2											Sa 54%
			3	SS	2	920										Si 42%
918.1 13.0	Very loose.		4	SS	2											Cl 4%
	Silt.		5	SS	2											
			6	SS	2	910										
			7	SS	2											
901.1 30.0	Very loose		8	SS	2	900										Sa 14%
	Clayey silt to silty clay.		9	SS	3											Si 73%
			10	SS	3	890										Cl 13%
			11	SS	3											
			12	SS	6	880										
			13	TW	P											
			14	TW	P	870										
			15	TW	P											
	Firm to stiff		16	TW	P	860										
858.1 73.0	Sand and gravel		17	SS	16											ELEV. 856.5
855.6 75.5	End of borehole.															Art. cond'n observed.
						850										
						840										
						830										

END OF CONE TEST

(ELEV. 831.1)

JOB 63-F-80

LOCATION 15' Lt. Sta. 21/20

ORIGINATED BY R.M.

W. P. 167-62

BORING DATE July 23, 24, 25 & 31/63

COMPILED BY R.M.

DATUM Geodetic

BOREHOLE TYPE Dynamic Penetration & Washboring

CHECKED BY M.D.

SOIL PROFILE			SAMPLES			ELEV. SCALE	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		SHEAR STRENGTH P.S.F. + Field Vane o Unconfined Compression • Triaxial					WATER CONTENT %							
						20	40	60	80	100	500	1000	1500	2000	20	40	60		
931.4	Groundlevel																		
0	Silty Sand.		1	SS	1	930													▽ 928.0
			2	SS	3														Sa 84%
	Very loose.		3	SS	4	920													Si 16%
917.4			4	SS	3														
14.0	Silt		5	SS	2														Sa 13%
			6	SS	2	910													Si 82%
			7	SS	2														Cl 5%
			8	SS	2	900													
			9	SS	3														
	Very loose		10	TW	P	890													
886.4			11	TW	P														113
45.0	Clayey silt to silty clay.		12	TW	P	880													114
			13	TW	P														119
			14	TW	P	870													116
			15	TW	P														117
			16	TW	P	860													116
			17	TW	P														110
						850													
	Firm to Stiff.		18	TW	P														107
840.4						840													▽ 839.0
91.0	Sand and gravel		19	SS	7														Art. cond'n observed.
837.9																			
93.5	End of borehole.					830													

TO: Mr. A. Stermac,
Principal Foundation Eng.,
Lab. Bldg.

FROM: J. C. McAllister

DATE: July 10, 1963.

OUR FILE REF.

IN REPLY TO

SUBJECT: W.P. 167-62,
North Creek (Bernard Creek),
Sec. Road 520,
District #11

Attached please find two prints of site plan E-4202-1 and composite plan and profile B-1053-2, showing the location of a proposed bridge and stream diversion, for which a foundation investigation is required.

A single 60' span structure is proposed here.

J. C. McAllister

JCMcA/ah

J. C. McAllister,
for S. McCombie,
Bridge Planning Engineer.

c.c. N. D. Smith
R. Fitzgibbon

Mr. A. M. Toye,
Bridge Engineer,
Bridge Division.

Attention: Mr. S. McCombie

Mr. A. G. Stermac,
Principal Foundation Engr.,
Foundation Section,
Materials & Research Division.

January 8, 1964

File Load Tests - Hwy. 520 & Bernard (North) Creek,
2 Miles West of Burks Falls, Ontario, District #11.
W.P. 167-62 -- W.J. 62-F-80

Two pile loading tests have recently been completed at the above-mentioned site. It is intended to carry out additional load tests on the same piles and, subsequently, a detailed report will be submitted describing all the test results.

We have reviewed the recent load test results and our recommendations are as follows:

The proposed structure should be supported by untreated #14 timber piles with 35-ft. embedment. Cut-off elevation should be a minimum of 5 ft. below final grade elevation. The maximum allowable load per pile is 15 tons.

It is pointed out that a piled foundation as such, will not eliminate settlements and therefore, a simply supported structure should be considered.

AGS/MdeF

cc: Foundations Office /
Gen. Files

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

Mr. A. M. Toye,
Bridge Engineer,
Bridge Division.

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

Attention: Mr. S. McCombie

April 24, 1964

Hwy. 520 and Bernard (North) Creek,
2 Miles West of Burk's Falls, Ontario,
Dist. #11, W.P. 167-62 -- W.J. 62-F-80.

63-F-80.

Additional load tests on the piles driven at the above-mentioned site have now been completed. The results of these tests do not warrant any change in our original recommendations contained in our memo dated January 8, 1964.

If you have any further queries in connection with this project, please contact this Office.

MD/MdeF

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

cc: Foundations Office ✓
Gen. Files

Re. North Creek and Hwy 520 (secondary)

WP 167-62 Dist # 11 Job No 63-F-80

The following information was given by Mr. Stermac to Mr. Hewson of the Bridge Office on May 4th 1964:-

At the above mentioned site the timber piles should be driven to approximate elev 535 (i.e. 25 ft. embedment below natural ground surface).

M. Devata

May 4th 1964

Materials and Research Division

May 7, 1964

McNamara Engineering Limited,
2200 Yonge Street,
Toronto, Ontario.

Attention: Mr. E. Boddaert

Re: North Creek and Secondary Hwy. 520,
District 11, W.P. 167-62, W.J. 63-F-80.

Dear Mr. Boddaert:

Following our discussion on May 6, 1964, on the foundation design of the above proposed structure, we have reviewed the results of the pile loading tests and soil conditions.

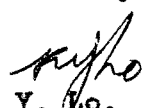
For 45 ft. (embedded length in natural undisturbed soil), #14 timber piles, a safe load of 25 T/pile may be used.

Yours very truly,

KYL/MdeF

cc: Mr. F. I. Hewson

Foundations Office
Gen. Files


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

(Installation of piles, reaction beams and anchorage system)

GENERAL DESCRIPTION OF WORK:

The work consists of the following:

(1) The driving of two class 'A' (Size No. 14) untreated timber piles 50 feet and 35 feet long to a depth of 45 feet \pm , and 30 feet \pm respectively below ground surface.

(2) The fixing of reaction beams and anchor piles of 50 feet length to form an anchorage system for each of the two piles mentioned in (1) capable of providing a maximum load of 75 tons to each pile under test.

(3) The installation of reference beams for each of the two test piles and the fixing of steel bearing plates and attachments for gauges to the pile butts.

(4) An alternative scheme utilizing a loading box to provide the dead load reaction may be submitted by the contractor subject to the approval of the D.H.C. Foundation Section.

MATERIALS SUPPLIED BY D.H.C.

The following materials and equipment will be supplied by D.H.C. and delivered to the site:

(1) H beams for reaction beams.

(2) One 50 feet, one 35 feet long class 'A' (Size No. 14) timber piles.

(3) All timber piles necessary for the anchorage system.

(4) Loading jack and all gauges necessary for the loading test.

All other materials necessary for the above work must be supplied by the contractor.

Note: If the Department decides to carry out the test using loading box, the contractor should supply the necessary box and reaction beams and assemble it at the site and dismantle and remove after the test is completed.

DRAWINGS:

The layout of the piles and anchors is shown on Dwg. No. 63-F-80 (B). The contractor must include with his quotations, a drawing showing the details of the anchorage system, reaction beams and reference beams.

QUOTATION:

The contractor should submit a lump sum quotation for carrying out the above work, such quotation to include the provision of all equipment and materials necessary except as provided under "Materials Supplied by D.H.C.".

LOADING TEST:

The contractor will not be required to carry out any loading tests. D.H.C. personnel will perform the necessary tests.

SITE:

The site is located approximately 2 miles west of Burks Falls and is approximately 200 feet north of the existing Hwy. No. 520 and North Creek structure. The Department will build an access road to the proposed test area to provide access for pile driving equipment.

M. Devata,
Sr. Foundation Engineer.

North Creek Structure and Sec Hwy 520 (Line A)

Dist # 11, W-J 63-F-80, W.P. 167-62

Contacted Mr. Burnhart, District Construction Engineer (Dist #11) by phone on 22nd Dec 64 regarding pile driving and advised as follows:-

1) Piles should be driven 45 ft into the natural ground and no attempt should be made to drive these piles for a particular blow/ft or controlling by means of any formula. Blows/ft during pile ^{driving} may not mean anything however pile driving record sheets should be filled in and ~~f~~ submit them to foundation section.

2

M. Devatu
Dec 21st 64

*Structure design
North Creek & Sec. Hwy. 520*

Materials and Research Division

May 7, 1964

McNamara Engineering Limited,
2200 Yonge Street,
Toronto, Ontario.

Attention: Mr. E. Boddaert

Re: North Creek and Secondary Hwy. 520,
District 11, W.P. 167-62, W.J. 63-P-80.

Dear Mr. Boddaert:

Following our discussion on May 6, 1964, on the foundation design of the above proposed structure, we have reviewed the results of the pile loading tests and soil conditions.

For 45 ft. (embedded length in natural undisturbed soil), #14 timber piles, a safe load of 25 T/pile may be used.

Yours very truly,

KYL
K. Y. Lo,
SUPERVISING FOUNDATION ENGR.
For:
A. G. Stermac,
PRINCIPAL FOUNDATION ENGR.

KYL/MdeF

cc: Mr. F. I. Hewson

Foundations Office
Gen. Files

CP

Pile Loading Tests at the
proposed crossing of Hwy. #520 over the
realigned North (Bernard) Creek
District of Parry Sound Dist. #11
W.O. 63-F-80 W.P. 167-62

TABLE OF CONTENTS

1. INTRODUCTION.
 2. DESCRIPTION OF THE SITE AND SUBSOIL CONDITIONS.
 3. PILE DETAILS, DRIVING DATA AND TEST ARRANGEMENTS.
 4. PILE LOADING TESTS.
 5. PIEZOMETER INSTALLATIONS.
 6. MISCELLANEOUS.
-

*Pile Loading Tests at the
proposed crossing of Hwy. #520 over the
realigned North (Bernard) Creek
District of Parry Sound Dist. #11
W.O. 63-P-80 W.P. 167-62*

1. INTRODUCTION:

Pile loading tests were carried out at the proposed crossing of Hwy. #520 over the realigned North Creek, some two miles west of Burks Falls. The purpose of the tests was to obtain load bearing values of timber piles driven through layers of very loose silty sands and silts and into firm to stiff cohesive clayey silts. In order to study the pore pressure response during pile driving, five piezometers were installed around one test pile prior to pile driving.

The original foundation investigation for the proposed crossing was carried out in July 1963, piezometers were installed and piles driven in November 1963. All these functions were supervised by the Foundation Section.

Although all relevant data of installations, drivings and load testings were kept on records no report was produced at the time of completion of the tests.

In the following paragraphs therefore a summary is given containing factual information as to installations and test results. This summary is being compiled in 1974 as part of a PPI project, devoted to the collection and critical review of all pile tests carried out by this office in the past 15 years.

2. DESCRIPTION OF THE SITE AND SUBSOIL CONDITIONS:

Detailed description on the site and subsoil conditions were given on the original foundation investigation, stored on microfiche under W.O. No. 63-P-80 in the Geocres system. A site plan, showing the location of the load tests together with a stratigraphical profile is attached to this summary. (See drawing 63-P80A)

To avoid redundancy subsoil conditions will not be reported here. Stratigraphy at the test site can simply be stated to consist of approx. 15-16 ft. (4.57-4.83 m) of very loose surficial silty sand, followed by a 15 ft. (4.57 m) thick layer of very loose silt, which in turn is underlain by firm to stiff clayey silt to silty clay. The silty sand and silt are basically non cohesive materials, and having average Standard penetration resistances of 2 blows/ft, (0.3 m). The clayey silt and silty clay are cohesive strata, the undrained shear strengths of which vary between 600 PSF (28.73 kN/m²) and 1700 PSF (81.4 kN/m²).

Several additional borings and dynamic core penetration tests were placed near the piles in September 1964 in order to observe changes if any of the subsoil conditions. The locations of these holes are shown on Drawing #63-F-80P accompanying this report. On figures Nos. 1, 2 and 3 the results of dynamic cone and Standard Penetration resistances are plotted obtained during the original Foundation investigation in July 1963, as compared with the ones obtained in September 1964 some 10 months after pile driving. On Figure #1 a considerable increase of the dynamic penetration resistances may be noted in BH #7, which was placed 6 inches (152 mm) from pile #4, suggesting compaction of the very loose granular material due to pile driving. Penetration resistances of BH #1 (July 1963) and BH's #8 and #9 (September 1964) are fairly similar indicating that the effect of pile driving on granular material does not extend beyond a circular area of 24 inches, (610 mm).

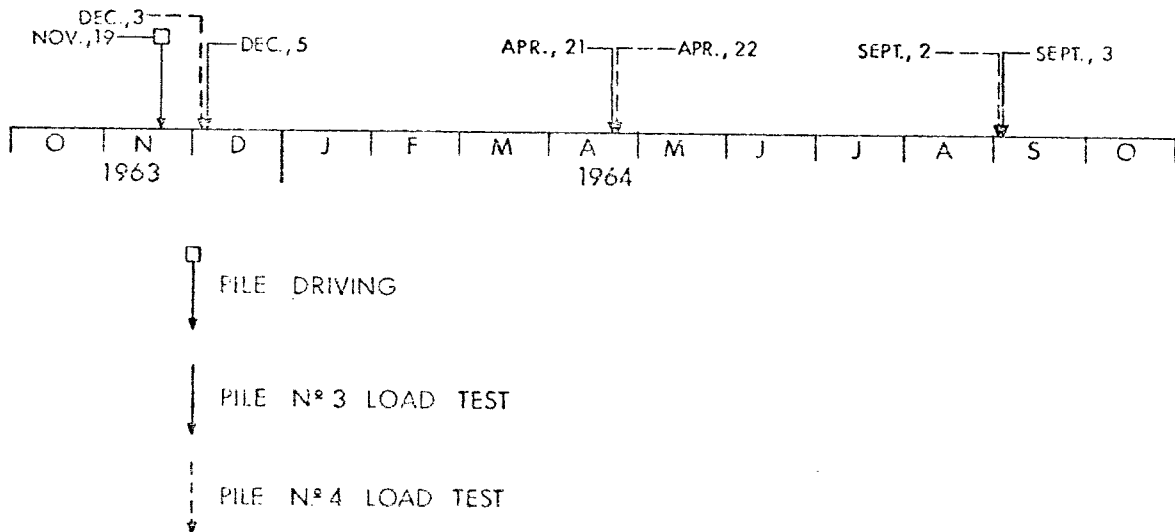
Figure # 2 presents a comparison between dynamic penetration resistances in the granular layers before and 10 months after pile driving within the zone of driving effect. Again some increase of penetration resistances may be observed especially within the uppermost 15-20 feet (4.57-6.10 m). On Figure #3 the "N" values of standard penetration tests before and after pile drivings are compared. The higher relative densities near the piles are quite considerable.

3. PILE DETAILS, DRIVING DATA AND TEST ARRANGEMENT:

Two test piles and four anchor piles were driven with a D-12 type hammer on November 19th, 1963. All the piles were No. 14 untreated timber piles. The embedded lengths of the anchor piles were 46 feet (14.02 m), and they were installed with a reaction beam, against which the loads were applied with a hydraulic jack. The test piles were numbered 3 and 4, the embedded length of pile #3 being 45 feet (13.72 m) and that of #4, 29 feet (8.84 m). As it may be seen on Drawing #63-R-80P, pile #4 is entirely embedded in the very loose granular layers, while the lower 15 feet (4.54 m) of pile #3 lies in the cohesive clayey silt. On the same drawing the pile driving records are also shown adjacent to the piles. It is to be noted that the height of fall of the hammer was 3 feet (914 mm) generating an energy of 8250 ft/lbs (11.19 kJ).

4. PILE LOADING TESTS:

Three sets of loading tests were performed on each of the two test piles. The time sequence of driving and load testing of the piles are shown on the sketch below:



The results of the pile load tests are shown on Figures 4-9 inclusive by means of conventional load versus time and load and time versus displacement diagrams.

All the tests were carried out to failure. Failure load was considered to be the one under which the piles plunged into the soil. By examining the diagrams it may be seen that there was no appreciable increase of the load bearing properties of the piles during the ten months testing time. The estimated ultimate bearing capacities of the piles are listed below:

<u>Pile Number</u>	<u>December 1963</u>		<u>April 1964</u>		<u>September 1964</u>	
	<u>Estimated Ultimate Loads</u>					
	<u>Ton</u>	<u>kN</u>	<u>Ton</u>	<u>kN</u>	<u>Ton</u>	<u>kN</u>
3	65	578	68	605	65	578
4	33	294	34	302	36	321

As was mentioned earlier Pile #4 was embedded entirely in the silty sands and silts. The average standard penetration "N" values of these deposits were estimated to be 2 blows/foot, (0.3 m). By calculating the bearing capacity of Pile #4 based on N values (Mayerhof) we get:

$$Q_u = 4 \times N \times \text{Area of Tip} + \text{N} \times \text{Perimeter} = 6 \text{ Ton (53 kN)}$$

The mobilized friction⁵⁰ along the pile shaft was much higher than the calculated one resulting in an average ultimate capacity of 35 Ton, (311 kN). We can use the actual bearing capacity to back calculate the "N" values corresponding to the mobilized friction. In doing so we obtain N values of 15-16 blows/foot (0.3 m) as opposed to the actual measured values of 2 blows/foot (0.3 m).

The lower 16 feet (4.88 m) of pile #3 was embedded in the cohesive clayey silt. The average undrained shear strength of this material along the pile was assumed to be 1300 PSF (62.24 kN/m²). Using Tomlinson's static formula, and assuming full mobilization of undrained shear strength along the shaft we obtain:

$$Q_u = 9 \times C_u \times \text{Tip Area} + C_u \times \text{perimeter} = \frac{9 \times 1300 \times 0.349 + 1300 \times 16 \times 3.14}{2000} =$$

$$= 2.09 + 33 = 35 \text{ Ton. (311 kN)}$$

The total ultimate load on pile #3 based on the load tests was 65 Ton, (578 kN), 35 Ton, (311 kN) was mobilized within the granular strata, hence the capacity in the clayey silt was 30 Ton (267 kN). Substituting this value in the Tomlinson formula the value of mobilized adhesion would be 1100 lbs. (52.67 kN/m²) ie. 85% of the undrained shear strength.

5. PIEZOMETER INSTALLATIONS:

In order to observe pore pressure behaviour during pile driving, five Geonor type brass piezometers were installed adjacent to test pile #3. Piezometer # 1-4 inclusive were installed prior to driving the piles. Piezometer #5 was secured to pile #3 and was driven with the pile to reach a final depth of 43 feet (13.1 m) below groundlevel. Since the response of the latter piezometer was rather erratic it was postulated that some damage or leakage occurred during driving. Consequently this instrument was disregarded.

Figure #10 in the appendix depicts the excess pore pressure induced by pile driving operations. General and broad conclusions of the pore pressure behaviour may be drawn as follows:

- (a) Insignificant excess pore pressures were measured within the silty sand and silt layers, indicating a very fast dissipation of pore pressures. This is of course due to the high permeability of these strata.
- (b) Piezometers #3 and #4, installed in the cohesive clayey silt registered excess pore pressures in the order of 25 PSI (172 kN/m²) during pile driving.
- (c) The piezometer response was noted to be quite fast, registering pressure increases immediately upon the piles penetrated into the cohesive stratum.
- (d) The dissipation of excess pore pressures was practically completed less than 24 hours after pile driving.

6. MISCELLANEOUS:

The installation of piezometers, the pile driving and load tests were supervised by Mr. R. Magi, Project Engineer. The entire project was under the direction of Mr. M. Devata, Supervising Engineer. This summary was compiled by the undersigned.

A. K. Barsvary

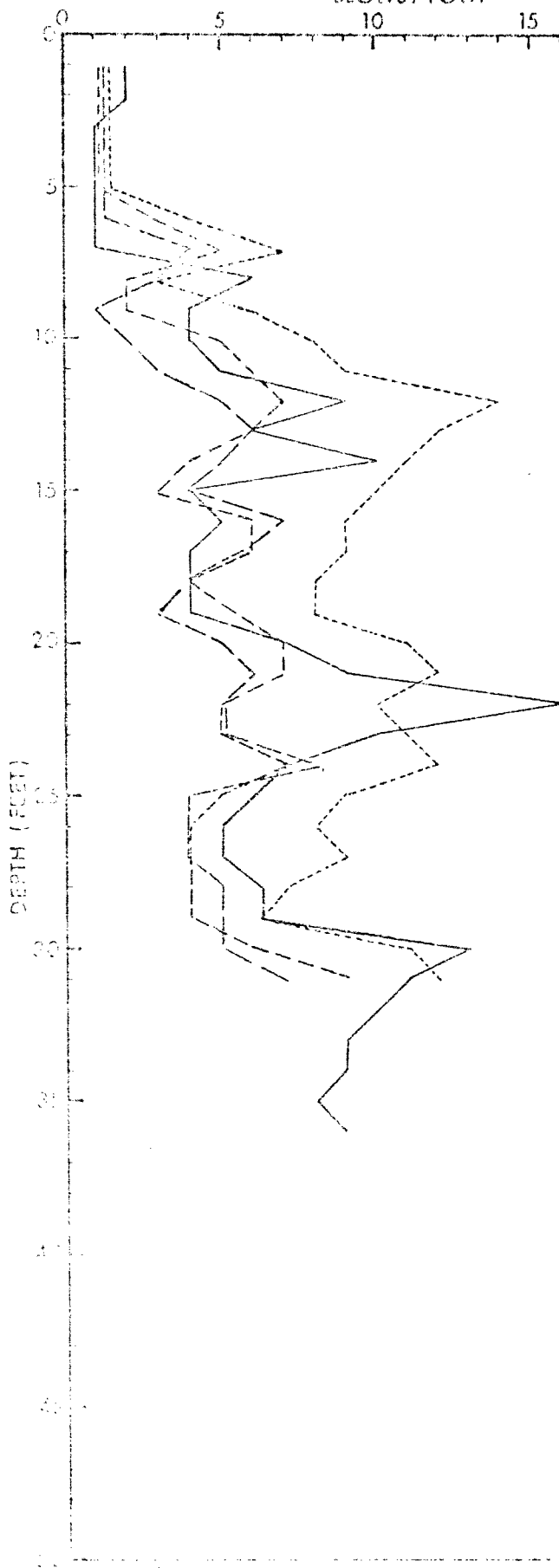
A.K. Barsvary, P. Eng.,
Head PPI Section

AKB/sh
January 31, 1974.



BLOWS / FOOT

GROUND LEVEL ELEV. 931.0



DYNAMIC CONE PENETRATION

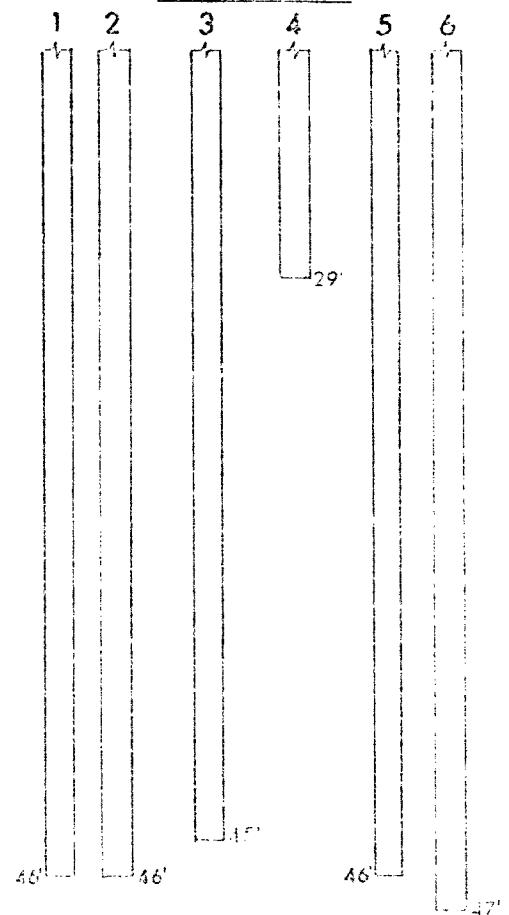
Vs.
DEPTH

W.O. 63-F-80

LEGEND:

JULY, 63	B.H.1	35"	FROM PILE 4(ORIG.)	_____
	B.H.7	6"	" " "	_____
SEPT. 64	B.H.8	24"	" " "	_____
	B.H.9	42"	" " "	_____

PILE LENGTHS



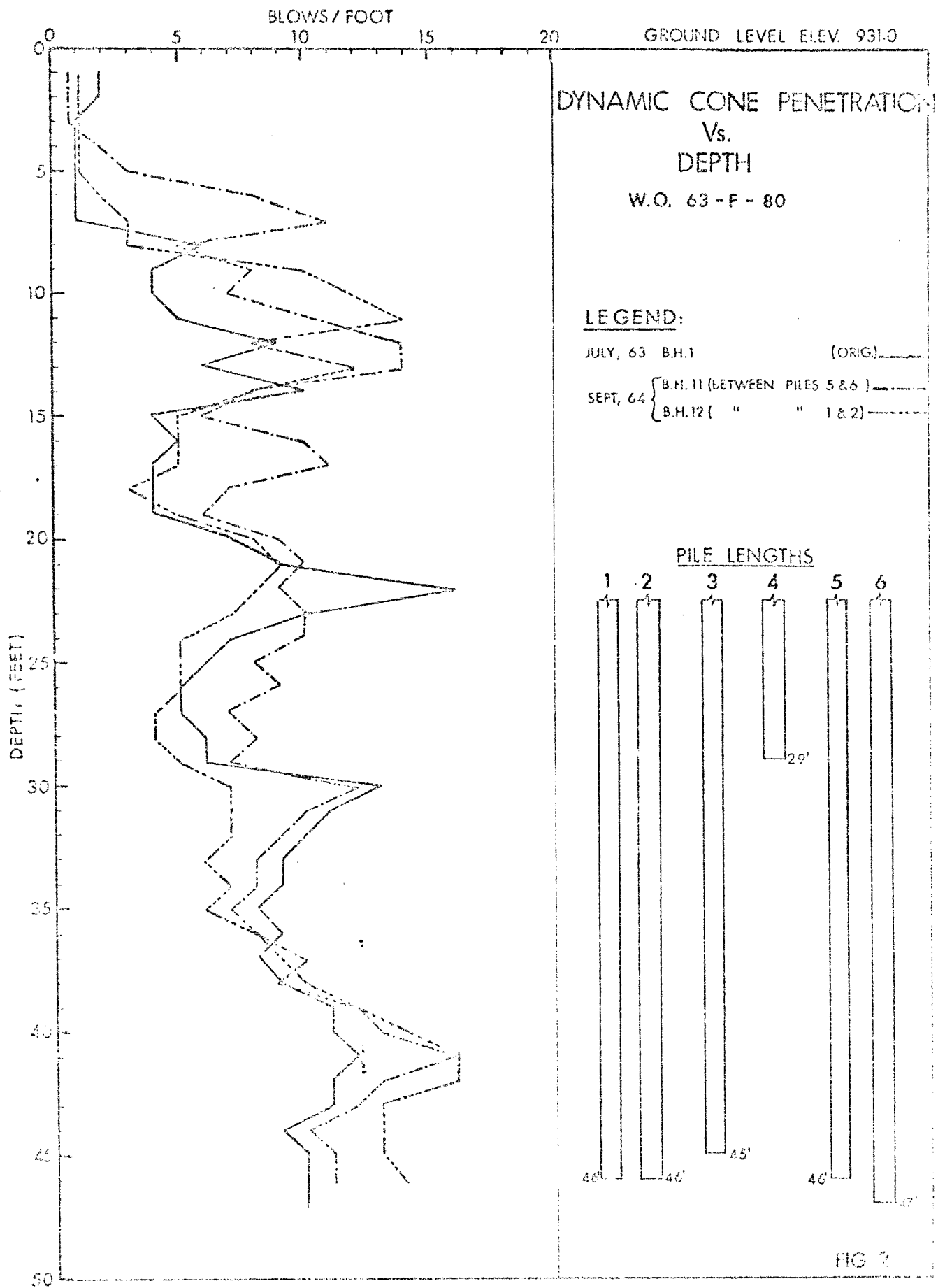
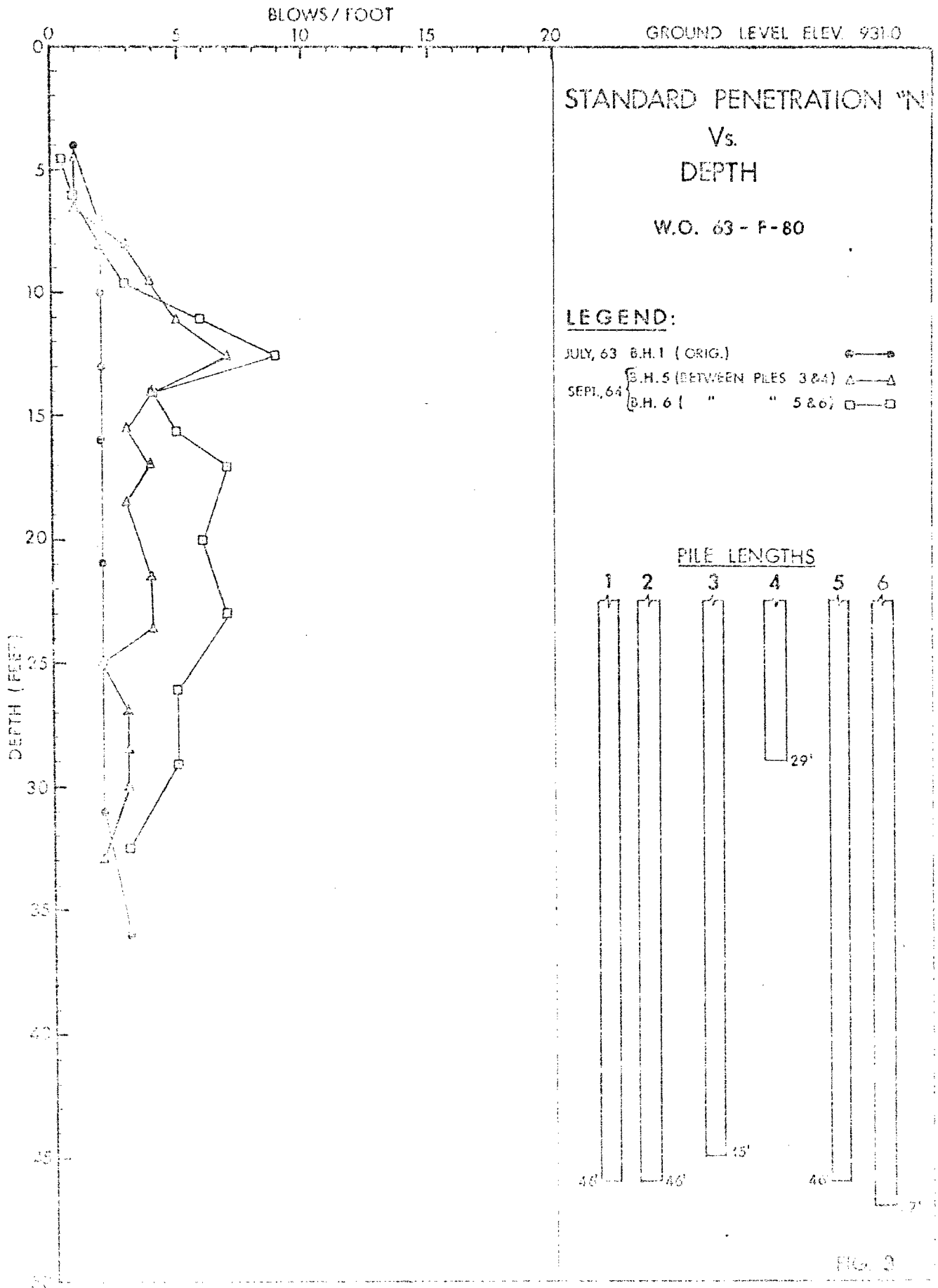


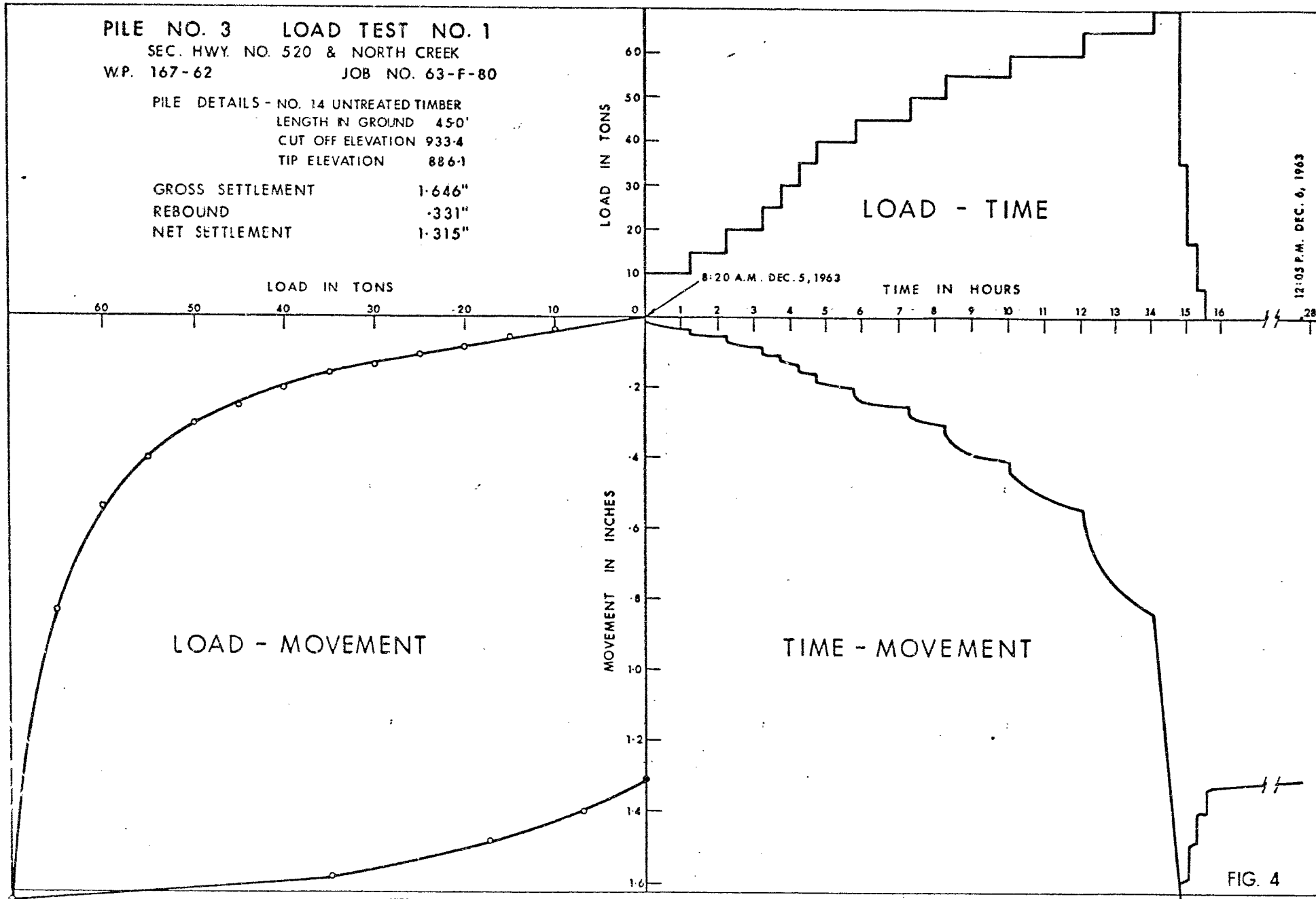
FIG 2



PILE NO. 3 LOAD TEST NO. 1
 SEC. HWY. NO. 520 & NORTH CREEK
 W.P. 167-62 JOB NO. 63-F-80

PILE DETAILS - NO. 14 UNTREATED TIMBER
 LENGTH IN GROUND 450'
 CUT OFF ELEVATION 933.4
 TIP ELEVATION 886.1

GROSS SETTLEMENT 1.646"
 REBOUND .331"
 NET SETTLEMENT 1.315"



PILE NO. 3 LOAD TEST NO. 2
 SEC. HWY. NO. 520 & NORTH CREEK
 W.P. 167-62 JOB NO. 63-F-80

PILE DETAILS - NO. 14 UNTREATED TIMBER
 LENGTH IN GROUND 45-0'
 CUT OFF ELEVATION 933-4
 TIP ELEVATION 886-1

GROSS SETTLEMENT 1-962"
 REBOUND .423"
 NET SETTLEMENT 1-539"

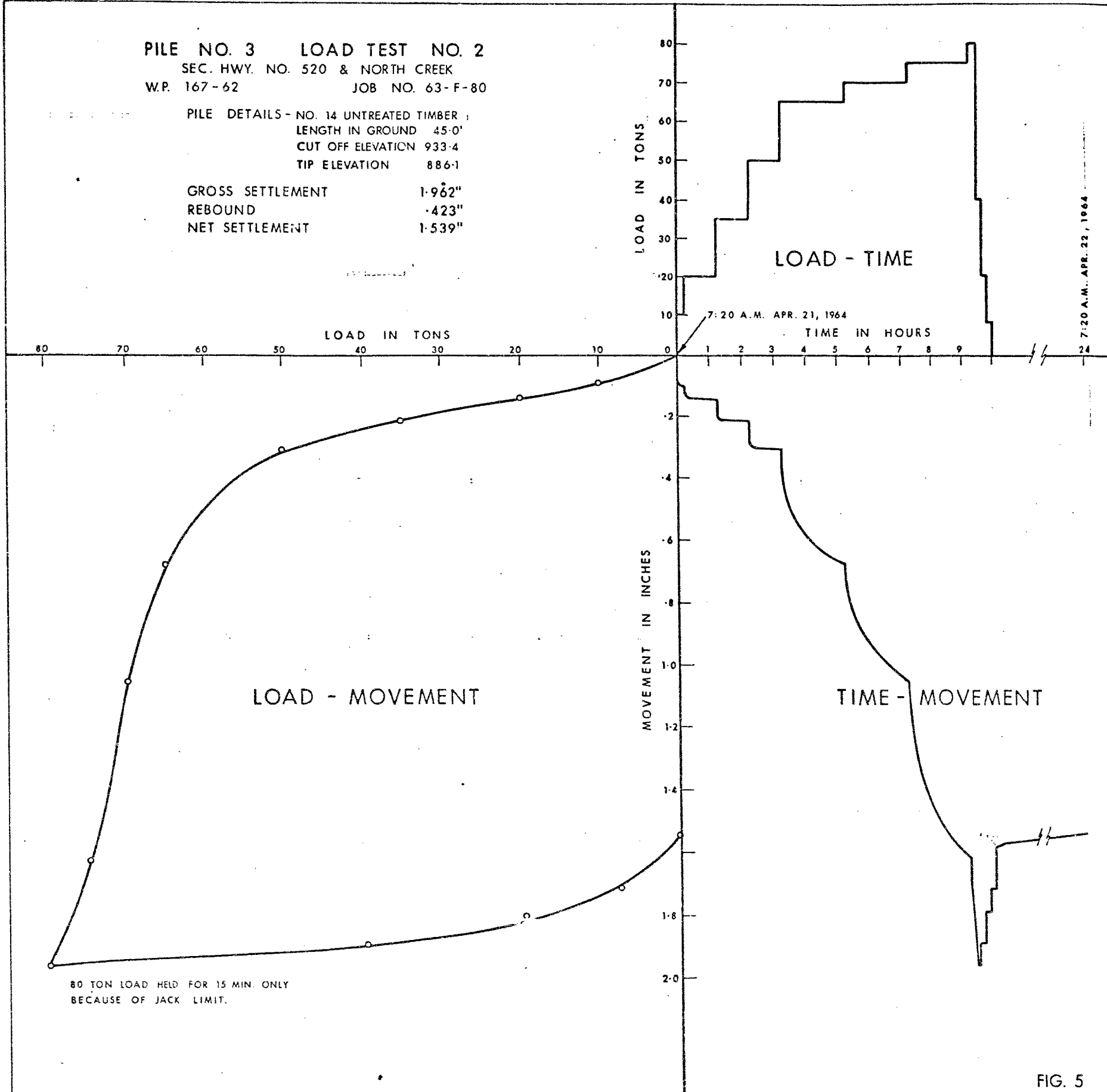


FIG. 5

PILE NO. 3 LOAD TEST NO. 3
 SEC. HWY. NO. 520 & NORTH CREEK
 W.P. 167-62 JOB NO. 63-F-80

PILE DETAILS - NO. 14 UNTREATED TIMBER
 LENGTH IN GROUND 45-0'
 CUT OFF ELEVATION 933-4
 TIP ELEVATION 886-1

GROSS SETTLEMENT 2.151"
 REBOUND .598"
 NET SETTLEMENT 1.553"

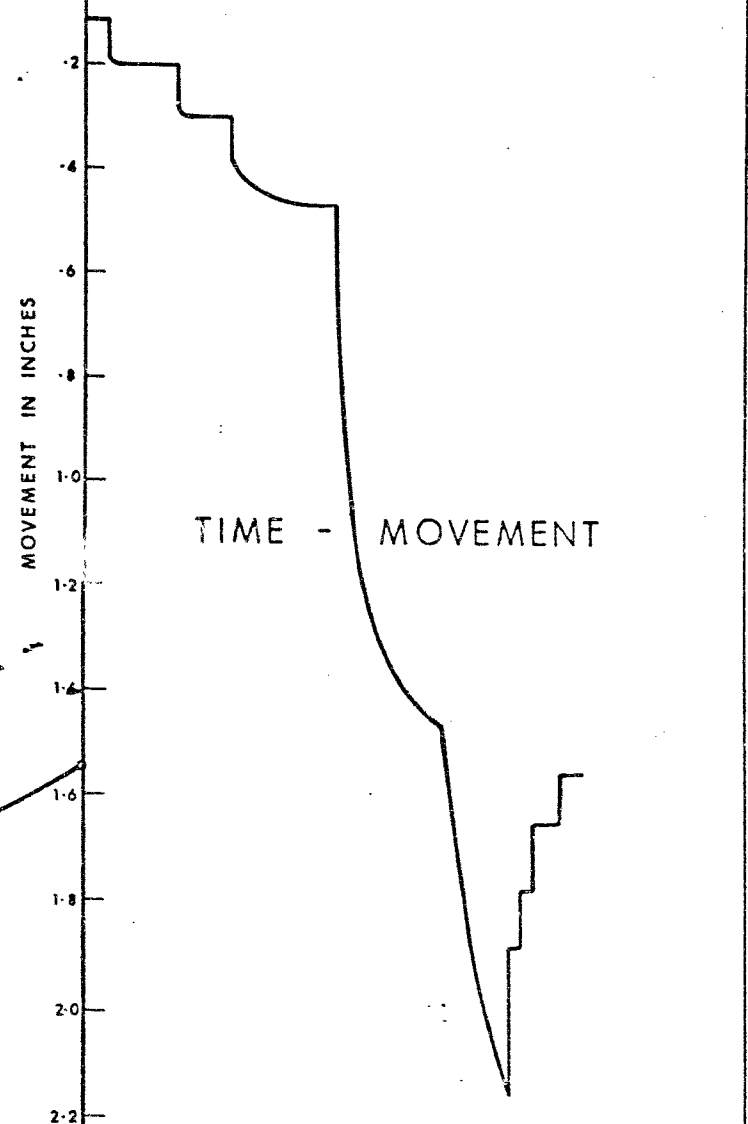
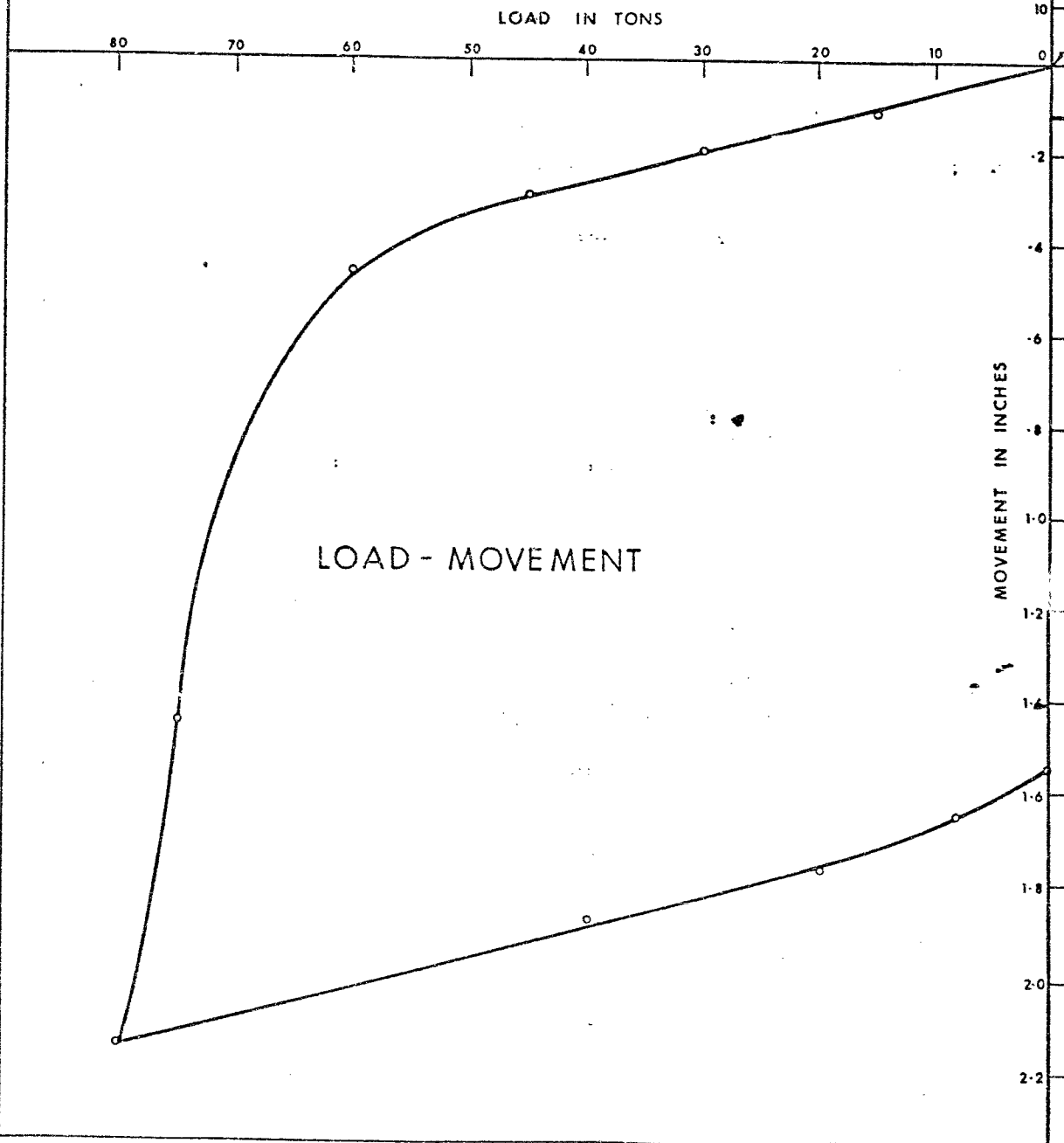
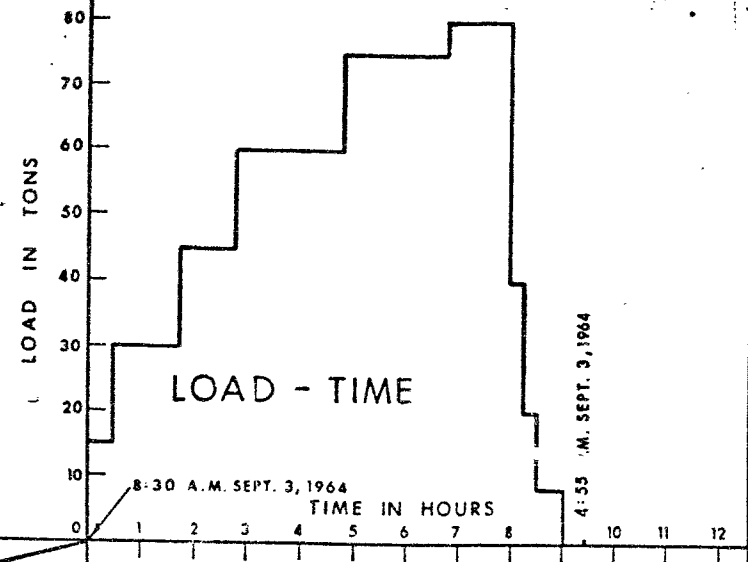


FIG. 6

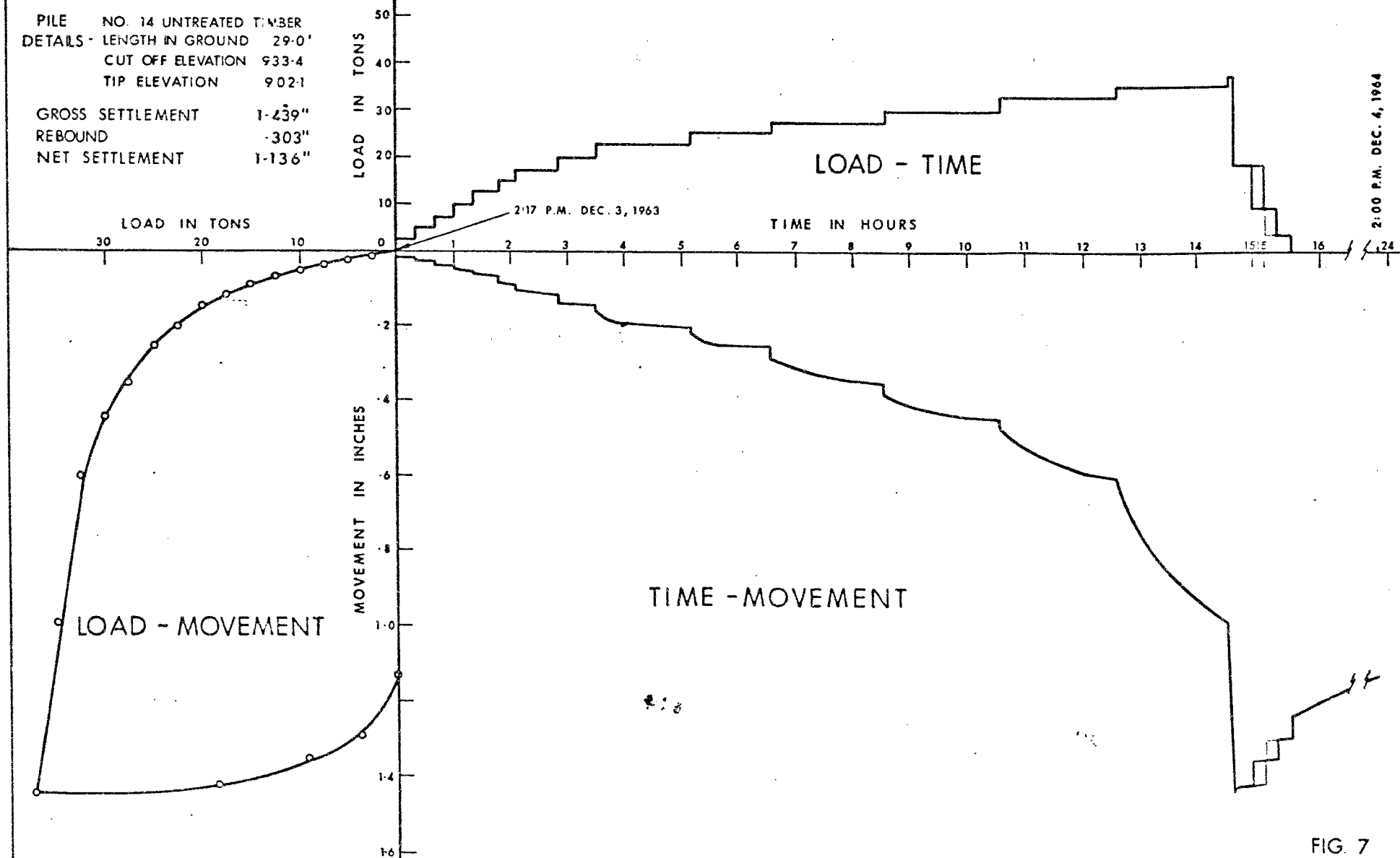
PILE 4 LOAD TEST 1

SEC. HWY. 520 & NORTH CREEK

W.P. 167-62 JOB 63-F-80

PILE NO. 14 UNTREATED TIMBER
DETAILS - LENGTH IN GROUND 29-0'
CUT OFF ELEVATION 933-4
TIP ELEVATION 902-1

GROSS SETTLEMENT 1-439"
REBOUND -303"
NET SETTLEMENT 1-136"



PILE NO. 4 LOAD TEST NO. 2
 SEC. HWY. NO. 520 & NORTH CREEK
 W.P. 167-62 JOB NO. 63-F-80

PILE DETAILS - NO. 14 UNTREATED TIMBER
 LENGTH IN GROUND 29'-0"
 CUT OFF ELEVATION 933.4
 TIP ELEVATION 902.1

GROSS SETTLEMENT 2.401"
 REBOUND .507"
 NET SETTLEMENT 1.894"

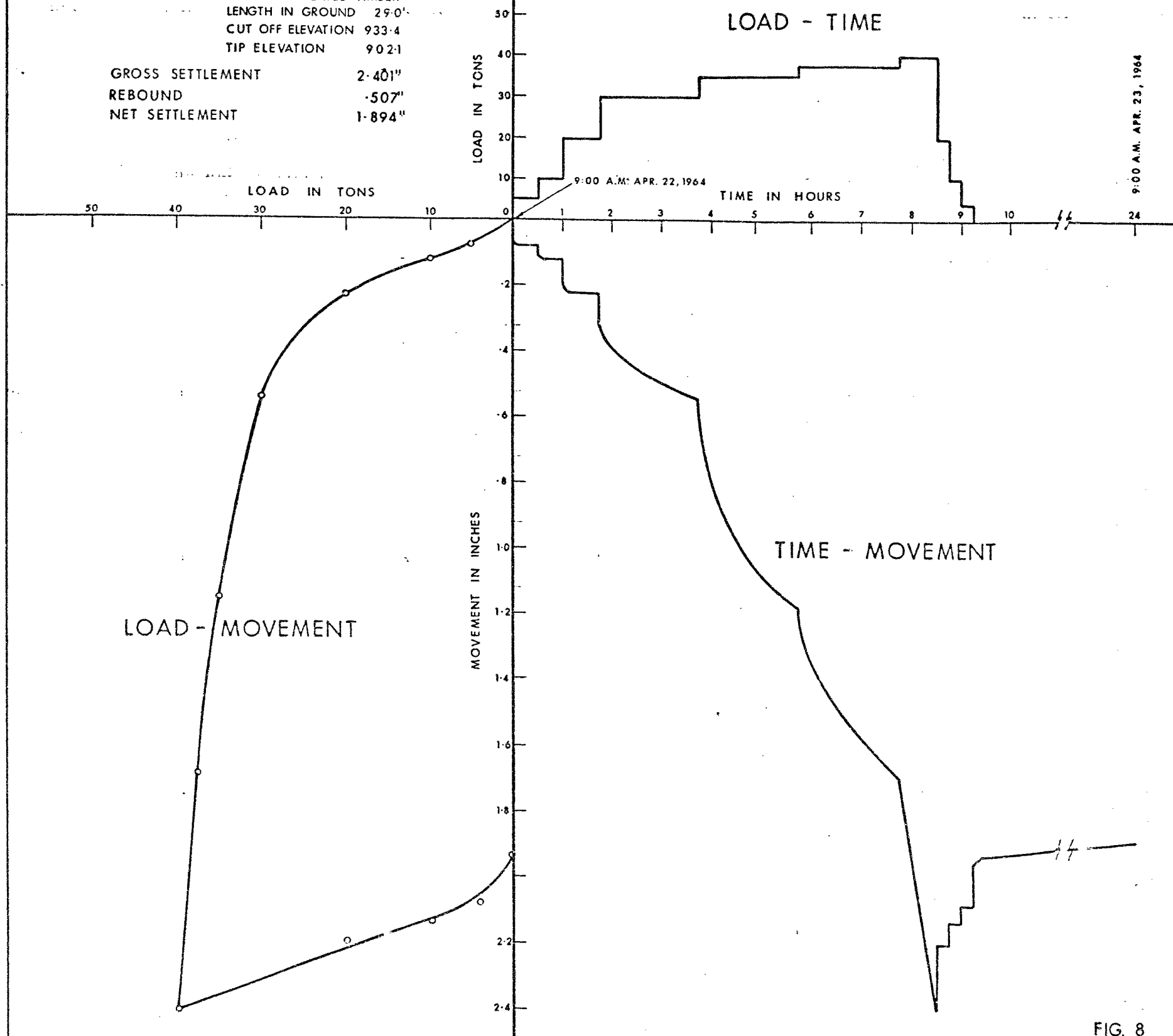


FIG. 8

PILE NO. 4 LOAD TEST NO. 3
 SEC. HWY. NO. 520 & NORTH CREEK
 W.P. 167-62 JOB NO. 63-F-80

PILE DETAILS - NO. 14 UNTREATED TIMBER
 LENGTH IN GROUND 29.0'
 CUT OFF ELEVATION 933.4
 TIP ELEVATION 902.1

GROSS SETTLEMENT 2.286"
 REBOUND .371"
 NET SETTLEMENT 1.915"

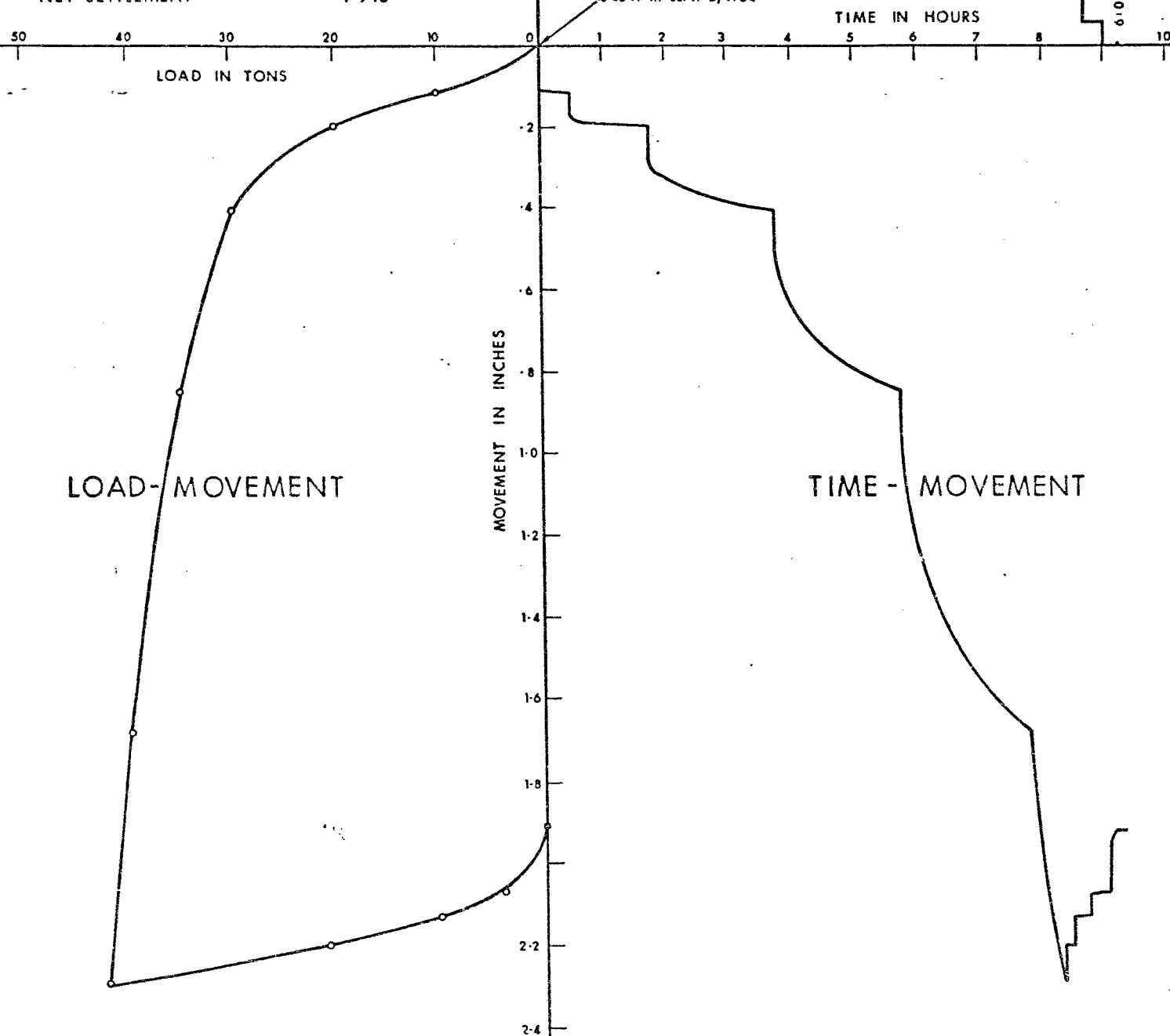
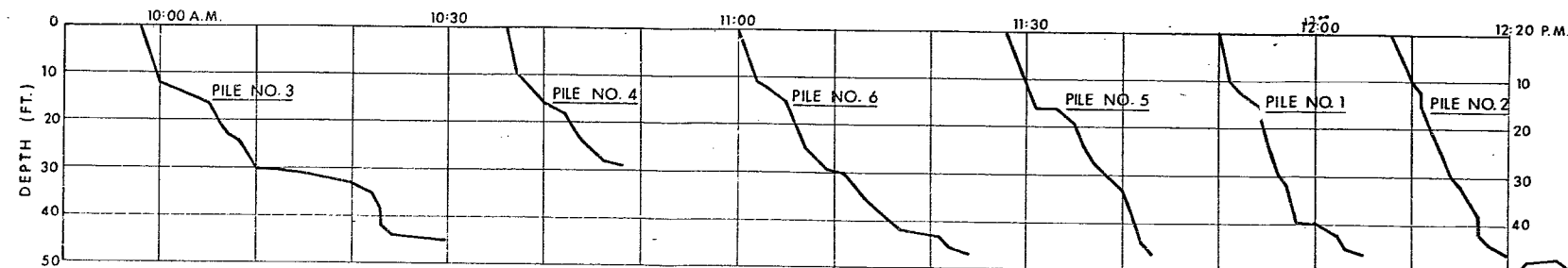
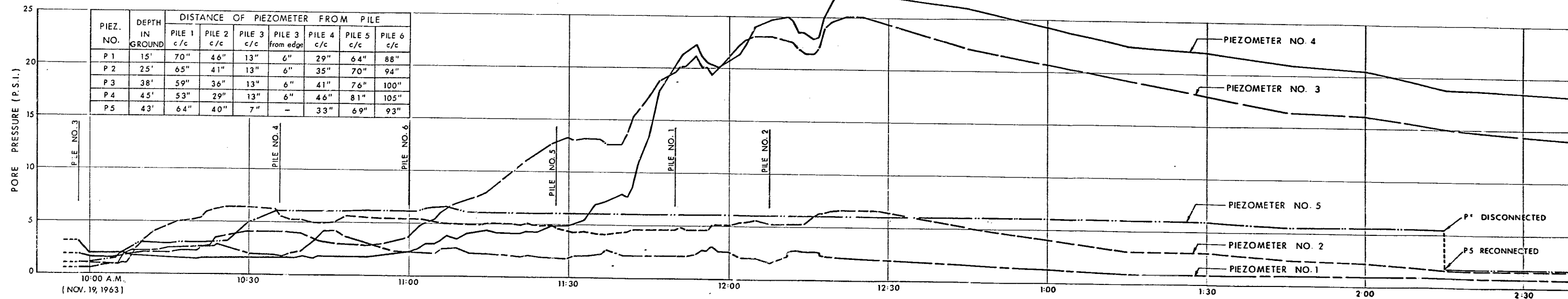


FIG. 9

PILE PENETRATION - TIME



PORE PRESSURE - TIME



12:20 P.M.

10
20
30
40

PORE PRESSURE - TIME

PILE PENETRATION
Vs.
TIME

PORE PRESSURE
Vs.
TIME

W.O. 63-F-80

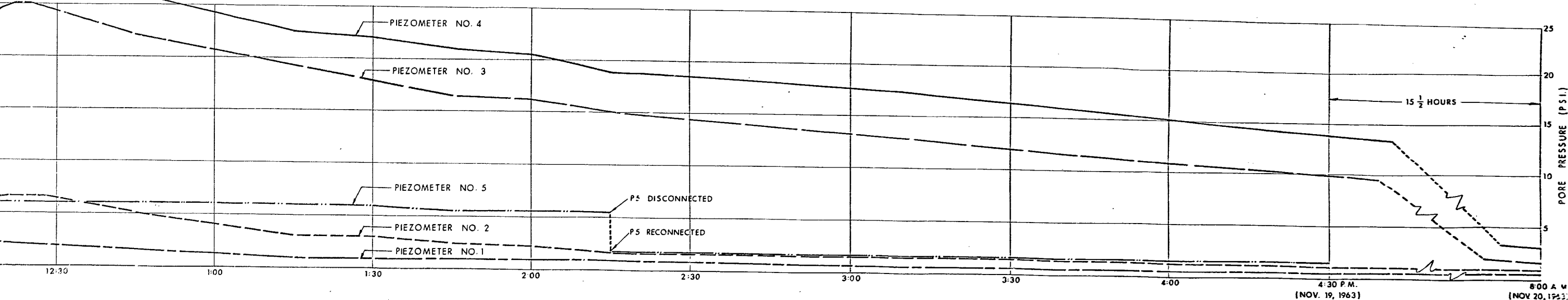
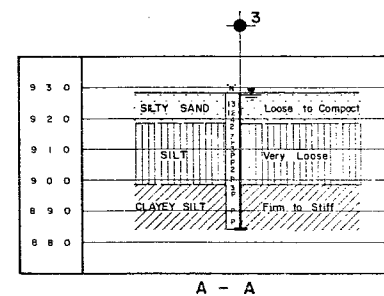
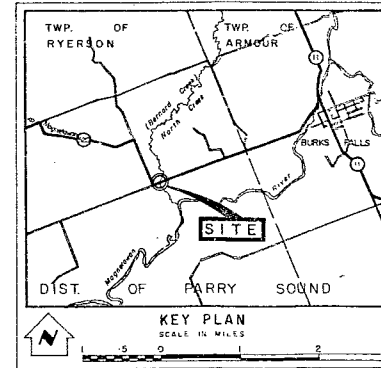
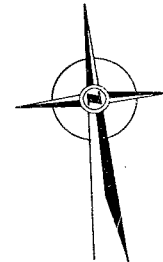
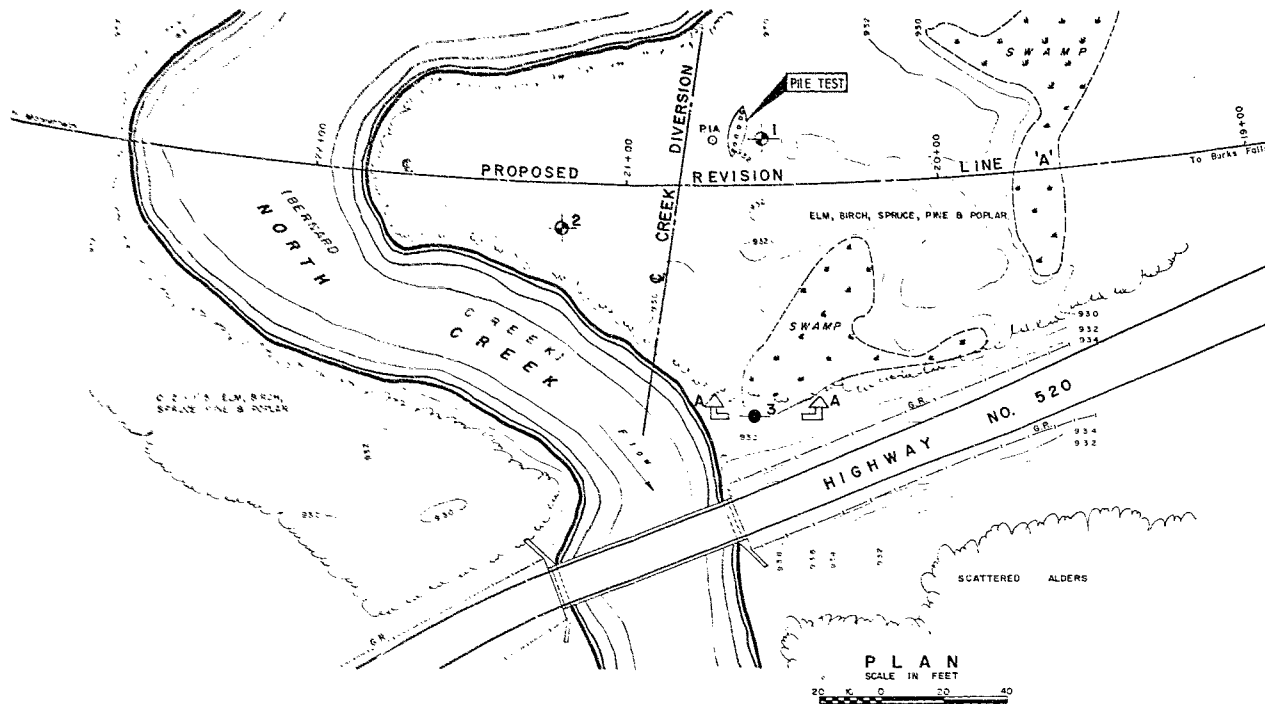
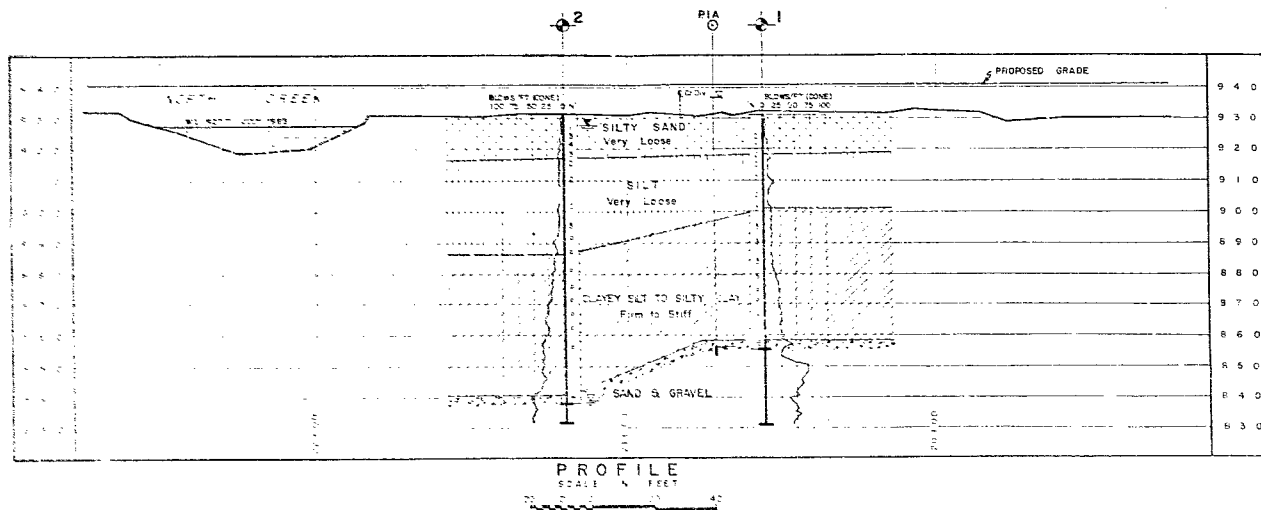


FIG. 10



LEGEND			
	Bore Hole		
	Cone Penetration Hole		
	Bore & Cone Penetration Hole		
	Water Levels established at time of field investigation (July 1963)		
	Piezometer		
NO.	ELEVATION	STATION	OFFSET
1	931.1	20+56	15' RT.
2	931.4	21+20	15' LT.
3	928.6	20+60	75' LT.
TIP ELEV. OF PIEZOMETER			
P1	853.1	20+72	15' RT.

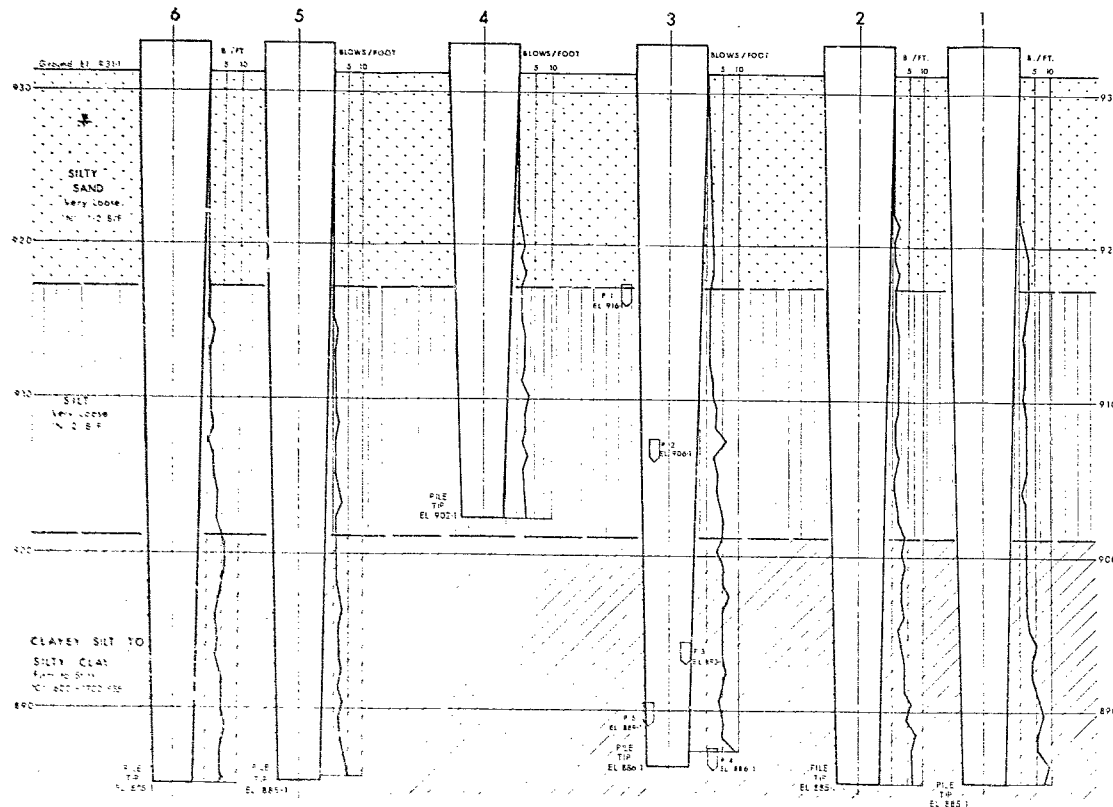
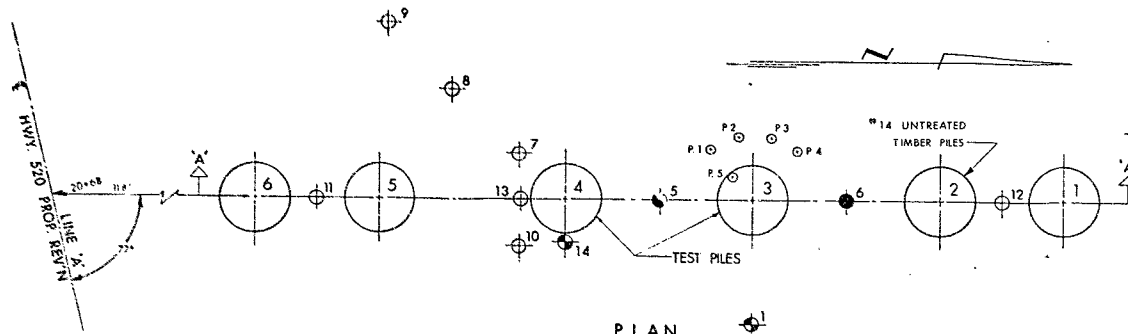
NOTE
The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence and may be subject to considerable error.



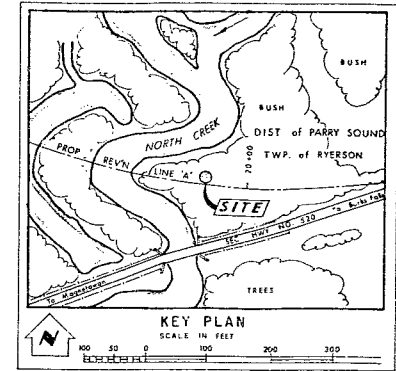
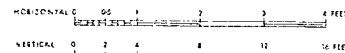
NOTE
ARTESIAN HEAD
ARTESIAN CONDITION ENCOUNTER

REVISIONS	DATE	BY	DESCRIPTION

DEPARTMENT OF HIGHWAYS - ONTARIO	
MATERIALS RESEARCH DIVISION	
NORTH CREEK (BERNARD CREEK)	
KING'S HIGHWAY NO. 520, PROP. REV. LINE 'A' DIST. NO. 11	
DIST. PARRY SOUND	
TWP. RYERSON	LOT 5 CON. 11
BORE HOLE LOCATIONS & SOIL STRATA	
DRILL NO. 100-63-F-62	DATE 14 AUGUST 1964
DRILLER D. M. [Signature]	63-F-80 A
DATE 14 AUGUST 1964	BY [Signature]
DATE 14 AUGUST 1964	BY [Signature]



SECTION A-A



LEGEND	
	Bore Hole
	Cone Penetration Hole
	Bore & Cone Penetration Hole
	Water Levels established at time of field investigation.
	P. Piezometer

NOTE: All piles were driven with a D-12 type hammer with a driving energy of 18,000 ft. lb.

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 & RESEARCH DIVISION - INVESTIGATION SECTION			
NORTH CREEK (BERNARD CREEK)			
KING'S HIGHWAY NO. 520 PROP. REV. LINE 'A'		DIST. NO. 11	
DIST. PARRY SOUND		TWP. RYERSON	
LOT 5		CON. IX	
TEST PILE & PIEZOMETER LOCATIONS & ELEVATIONS			
SURVEY NO.	DATE	BY	63-F-80 P
63-F-80	DECEMBER 1, 1964	J. E. G.	63-F-80 P
APPROVED BY: [Signature]			