

G.I.-30 SEPT. 1976

GEOCRES No. 40J16-47DIST. 1 REGION W.P. No. 43-66-17CONT. No. 75-27W. O. No. STR. SITE No. 14-371HWY. No. 402LOCATION Airport Rd. & Telfer
Channel BridgeNo. of PAGES -

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT. REMARKS:

40 J-110

GEOCRES No.

40J16-42

GEOCRES No.

PILE LOADING TESTS AT
THE PROPOSED CROSSING OF HWY. #402 AND
BLACKWELL ROAD, COUNTY OF LAMBTON
DISTRICT #1 (CHATHAM)
W.O. 70-11049 -- W.P. 43-66-05
CONT. 75-27

TABLE OF CONTENTS

1. INTRODUCTION.
 2. SUBSOIL CONDITIONS.
 3. PILE DETAILS, DRIVING DATA AND TEST ARRANGEMENT.
 4. DISCUSSIONS AND EVALUATION OF TEST RESULTS.
 5. MISCELLANEOUS.
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PILE LOADING TESTS AT
THE PROPOSED CROSSING OF HWY. #402 AND
BLACKWELL ROAD, COUNTY OF LAMBTON
DISTRICT #1 (CHATHAM)
W.O. 70-11049 -- W.P. 43-66-05

1. INTRODUCTION:

Pile loading and extraction tests were carried out at the location of the proposed crossing of Hwy. #402 and Blackwell Side Road in order to obtain data on load bearing properties of various piles. The original foundation investigation was carried out in 1969, the results of which were reported under W.O. Number 69-F-91. One of the recommendations given in above report called for driving friction piles through the fill and into - but not below - the desiccated crust. It was estimated that 12-3/4" O.D. steel tubes, driven according to above, will support safe loads of 25 ton/pile. Pile loading tests were suggested to confirm the recommendations.

The pile load tests were performed in 1971, the results of which have been kept on file without being summarized in a report.

In the following paragraphs the factual information concerning soil properties, pile driving and testing data and estimated loads are given.

This summary is being compiled in 1974 as part of a PPI project, devoted to the collection and critical review of all the pile tests carried out by this Office in the past 15 years.

2. SUBSOIL CONDITIONS:

During the original foundation investigation in 1969 a thorough drilling program was implemented, and soil properties were fully described in the report (69-F-91). Prior to the installation of piles three borings were placed adjacent to the test location,

and very close to each other (Borings designated as P, P-1 & P-2). In one boring continuous vane tests were performed, in the second continuous Shelby tube samples and in the third continuous split-spoon samples were taken. The borelogs of above holes - partially combined - are presented in the Appendix.

Soil stratigraphy consists of an approximately 48 ft. (14.6 m) thick deposit of clayey silt with some sand and traces of gravel, underlain by silty clay with sand and traces of gravel, extending to approximately 120 ft. (36.7 m) below ground, corresponding to elevation 467 ft. (142 m). The surficial 6 - 7 ft. (1.8 - 2.1 m) of the clayey silt is desiccated, having undrained shear strengths of 2000 p.s.f. (95.8 kN/m²) and over. Standard penetration N values within the crust were noted to range from 23 blows/ft. (0.3 m) to 14 blows/ft. (0.3 m). Below the crust the consistency of the clayey silts is generally firm to stiff and the silty clays is stiff to hard. The average undrained shear strength below the crust was measured to be 600 p.s.f. (28.7 kN/m²) increasing to approximately 1500 p.s.f. (71.8 kN/m²) around elevation 550 ft. (168 m). The maximum pile penetration was 60 ft. (18.3 m) below the top of fill (approximate elevation 545 ft. or 166 m).

"Geoprobe" pressuremeter tests were performed prior to pile driving in one boring which was drilled through the test fill in November 1970. The evaluation of these test results is outside the scope of this summary, notwithstanding a report of the pressuremeter tests are attached to the original foundation report (W.O. 69-F-91).

3. PILE DETAILS, DRIVING DATA AND TEST ARRANGEMENT:

A total of nine test piles and three anchor piles was driven between the 15th and 18th of December 1970. The particulars of the test piles are as follows:

Piles #1, 2 & 3: 12 BP @ 53 steel H piles, 20 ft., 60 ft. and 40 ft. (6.1 m, 18.3 m and 12.2 m) long respectively.

Piles #4, 5 & 6: 12" (0.305 m) Herkules precast concrete piles (Type #800), 40 ft. 60 ft. and 20 ft. (12.2 m, 18.3 m and 6.1 m) long respectively.

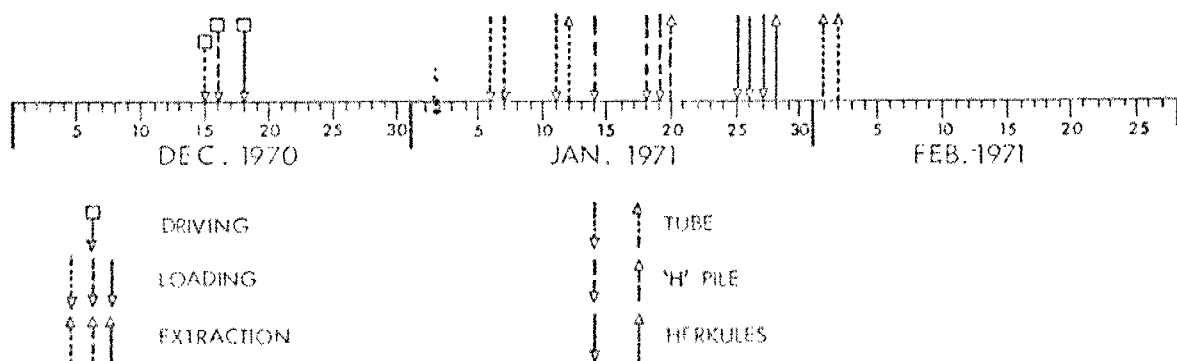
Piles #7, 8 & 9: 12-3/4 O.D. x 0.25" steel tube piles, 20 ft., 60 ft., and 40 ft. (6.1 m, 18.5 m and 12.2 m) long respectively.

The steel tubes were fitted with drive shoes and filled with concrete, upon driving. The embedded lengths of the piles were approximately 1 ft. less than the overall lengths of the piles.

Prior to driving the piles a test area was constructed, consisting of a fill of 12 ft. (3.67 m) high, about 50 ft. (15.2 m) long and 40 ft. (12.2 m) wide, to simulate the actual design conditions. Piles were driven through the fill. The pile test location and the testing arrangement are shown on Fig. #1 and 2 in the Appendix. In Fig. #3 the elevations of the test arrangement with the reaction beams, supported on anchor piles are given. Pile driving was carried out with a drop hammer, weighing 7000 lbs. (3175 kg), falling 4 ft. (1.219 m) and generating an energy of 28,000 ft.lbs./blow (37.963 kJ/blow). The weight of the pile cap was 500 lbs. (227 kg).

On Fig. #4 pile details, driving records and corresponding soil properties are shown.

Each test pile was subjected to one loading test. Upon completion of the load tests the three tube piles and one of each of the H and Herkules piles were further tested by extraction, in order to evaluate the frictional resistance along the pile shafts. The sequence of driving, loading and extraction tests are shown graphically below:



Loading and extraction tests were performed by conventional methods, the results of which are compiled as load versus time and displacement and time versus displacement diagrams in Figs. #5 - 13, inclusive.

4. DISCUSSIONS AND EVALUATION OF TEST RESULTS:

All the load and extraction tests were carried out to failure. To determine the ultimate capacity of a pile is relatively simple in those cases where the pile fails by slipping or plunging under a constant load. However, if a pile begins to experience very large settlements under small load increments, the ultimate capacity is subject to interpretation. In the latter cases, some arbitrary definition of failure load is necessary. In this summary, the suggestion by Terzaghi is adopted (Proc. A.S.C.E. 1942). According to this procedure, the ultimate load is considered to be that at which the settlement reaches a value equal to 1/10 the top diameter of the pile.

The estimated failure loads of the loading and extraction tests are tabulated below:

PILE		ESTIMATED FAILURE LOAD TON (kN)			Estimated End Bearing Factor N_c
No.	Type	Embedded Length Ft. (m)	Loading Test	Extraction Test	
1	12" @ 53 H	20 (6.1)	56 (498)		
2	12" @ 53 H	60 (18.3)	56 (498)	47 (418)	18
3	12" @ 53 H	40 (12.2)	64 (569)		
4	Herkules	40 (12.2)	50 (445)		
5	Herkules	60 (18.3)	70 (623)	66 (587)	9
6	Herkules	20 (6.1)	84 (747)		
7	12-3/4" Steel Tube	20 (6.1)	74 (658)	67 (596)	9
8	12-3/4" Steel Tube	60 (18.3)	80 (712)	57 (507)	52
9	12-3/4" Steel Tube	40 (12.2)	67 (596)	76 (676)	Inconclu- sive

The above results show considerable inconsistencies, most of which would be very difficult to explain. The principal purpose of the tests were to evaluate the load bearing capacities of the various piles driven into, but not below the crust. These are the piles with embedded lengths of 20 ft. (6.1 m). All the piles with longer embedment penetrated through the crust and into the weaker material. Indeed one unequivocal conclusion of the tests was that the load bearing capacities have not increased appreciably below the 20 ft. (6.1 m) depth. It appears that the increased mobilized adhesion due to the larger surface area of the longer piles, was accompanied by smaller end bearing values, on account of the smaller shear strength below the tip of the piles. Considering the 20 ft. (6.1 m) long piles only, the Herkules pile developed the largest ultimate load, equalling 84 ton, closely followed by the steel tube with 74 ton ultimate capacity.

Among the 40 ft. (12.2 m) and 60 ft. (18.3 m) long piles the tube piles proved to have the largest capacities.

If we assume that the ultimate pull load of the extraction tests gives indications of the mobilized adhesion along the pile shaft, than by using these values the mobilized unit adhesion as a percentage of the undrained shear strength may be calculated. By assigning average shear strength values of the soils the mobilized percentage adhesions are listed as follows:

No.	PILE		Ave. C_u Along Shaft PSF (kN/m ²)	Mobilized Adhesion PSF (kN/m ²)	Percentage Mobilized %
	Type	Length Ft. (m)			
2	12" H	60 (18.3)	1000 (47.88)	392 (18.77)	39
5	Herkules	60 (18.3)	1000 (47.88)	629 (30.12)	63
8	12-3/4" Tube	60 (18.3)	1000 (47.88)	570 (27.29)	57
7	12-3/4" Tube	20 (6.1)	2200 (105.34)	1994 (95.47)	91

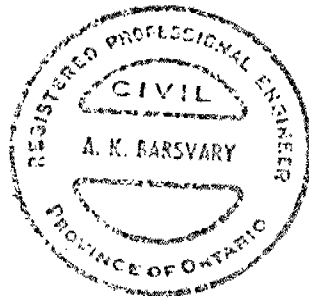
There appears to be quite a large scatter among the computed mobilized adhesion values. It should, however, be pointed out that some 12 ft. (3.67 m) length of the piles was embedded in the recently constructed fill, within which no strength values were available. This was estimated only, and the error introduced by this might be considerable.

5. MISCELLANEOUS:

The pile load tests were carried out by Mr. P. Payer. This summary was compiled by Mr. A. K. Barsvary. The entire load test was under the direction of Mr. K. G. Selby.

AKB/ao

A. K. Barsvary
A. K. Barsvary, P. Eng.



APPENDIX I

ORIGINATED BY P.P.

COMPILED BY H.D.R.

CHECKED BY OK

15 $\frac{20}{10}$ 5 % STRAIN AT FAILURE

RECORD OF BOREHOLE N^o P (Continued)

JOB 70-11049

LOCATION Hwy. 402 & Blackwell Road (Sarnia)

ORIGINATED BY P.P.

W.P. 43-66-05

BORING DATE July 28, 29 & 30, 1970

COMPILED BY H.D.R.

DATUM Geodetic

BOREHOLE TYPE

CHECKED BY C/K

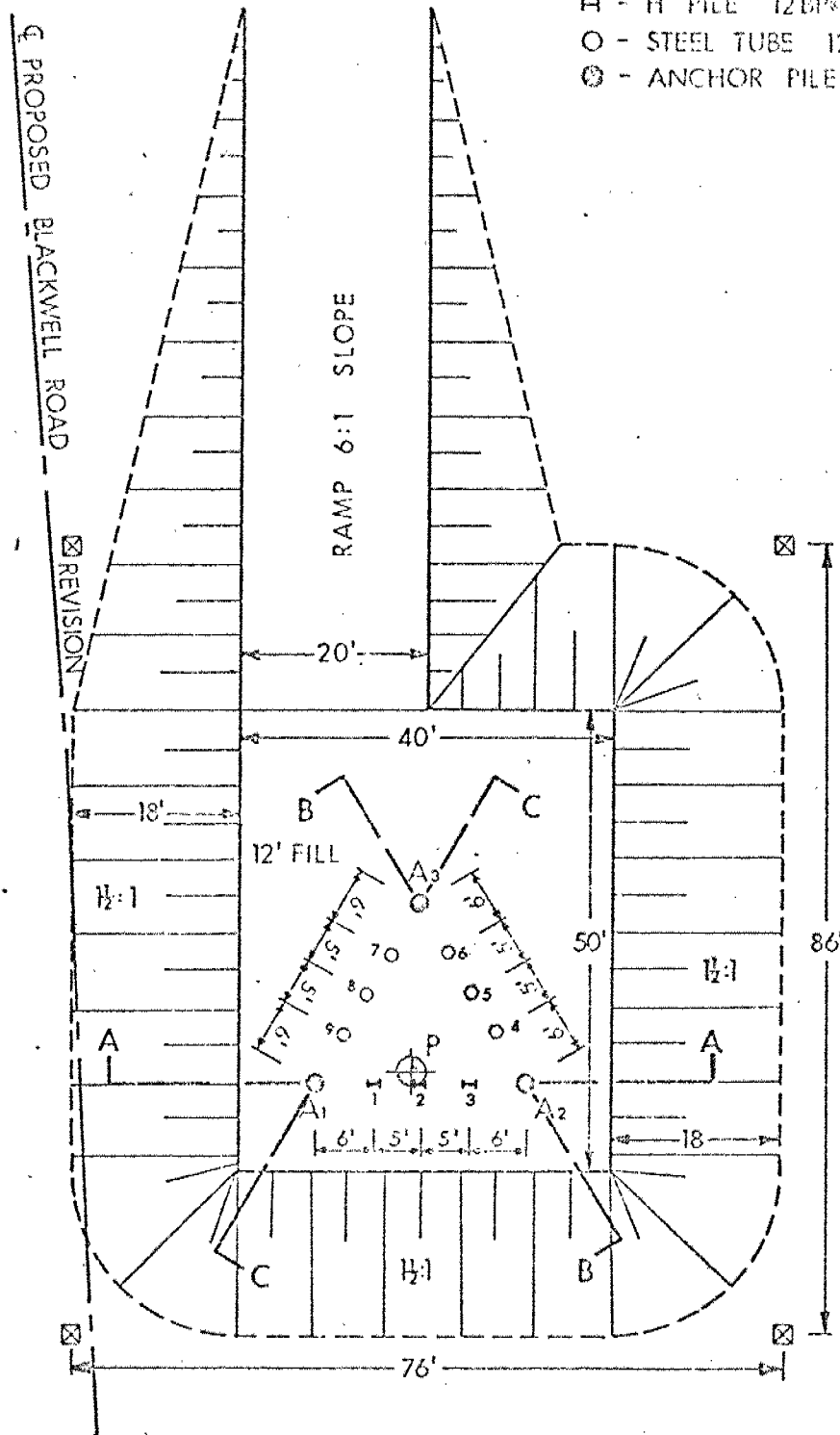
SOIL PROFILE				SAMPLES			DYNAMIC PENETRATION RESISTANCE			LIQUID LIMIT — w_L			BULK DENSITY γ P.C.F. T/m ³	REMARKS
	ELEV. DEPTH ft.	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FOOT (0.3m)	ELEV. SCALE ft./m.	BLOWS / FOOT	PLASTIC LIMIT — w_p	WATER CONTENT — w	WATER CONTENT %			
m.									SHEAR STRENGTH P.S.F.		WATER CONTENT %			
									O UNCONFINED + FIELD VANE		Wp — w — Wl			
									● QUICK TRIAXIAL x LAB VANE		10 20 30			
149.4	490.0								1000	2000				GR.SA.SI.CL
				40	TW	PM							128	
													2.05	
				41	TW	PM	480						127	
							146.3						2.03	
				42	TW	PM							122	1 7 40 52
													1.95	
143.4	470.5			43	TW	PM								
37.0	121.5	End of Borehole						0	25	50	75	100	kN/m ²	

OFFICE REPORT ON SOIL EXPLORATION

20
15 ϕ 5 % STRAIN AT FAILURE
10

PILE TYPES.

- - HERKULES (4, 5 & 6)
- H - H PILE 12BP053 (1, 2 & 3)
- - STEEL TUBE 12½ x ½ (7, 8 & 9)
- ⊗ - ANCHOR PILES (A1, A2 & A3)



PLAN
SCALE 1" = 20'

FIG. 2



DEPARTMENT OF HIGHWAYS
MATERIALS and
TESTING
DIVISION

ONTARIO

BLACKWELL ROAD & HWY. 402

PILE TEST ARRANGEMENT

W.P. 43 - 66 - 05

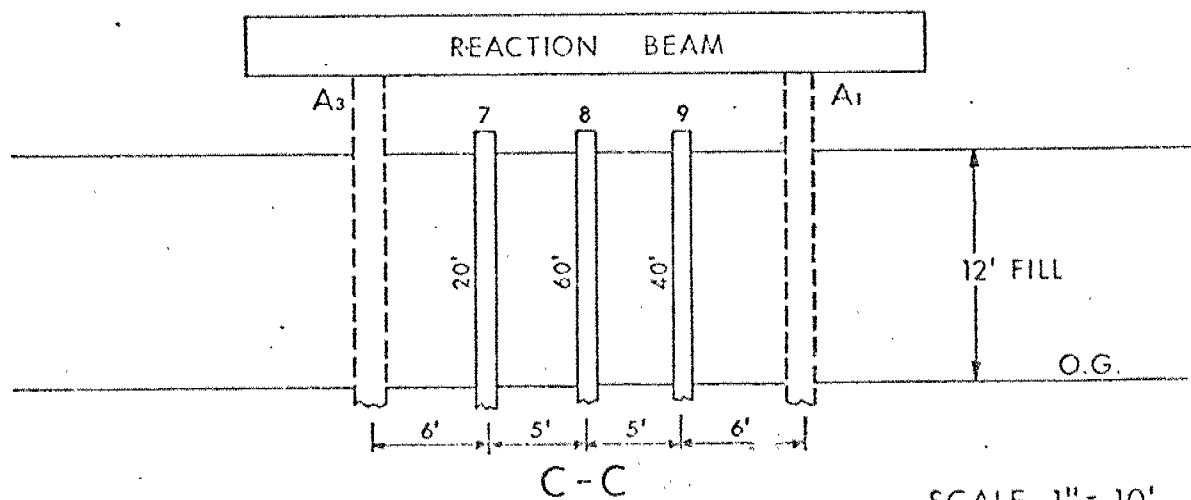
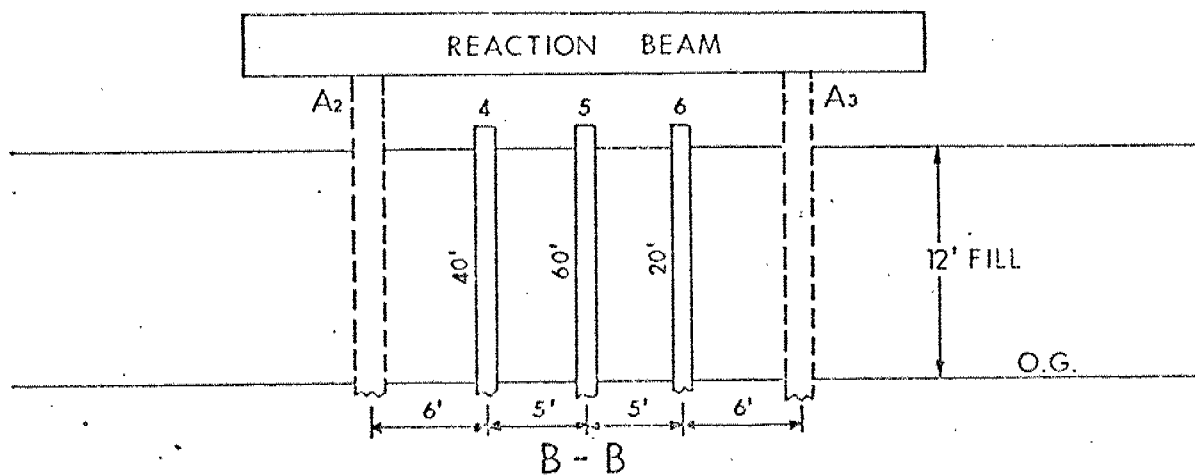
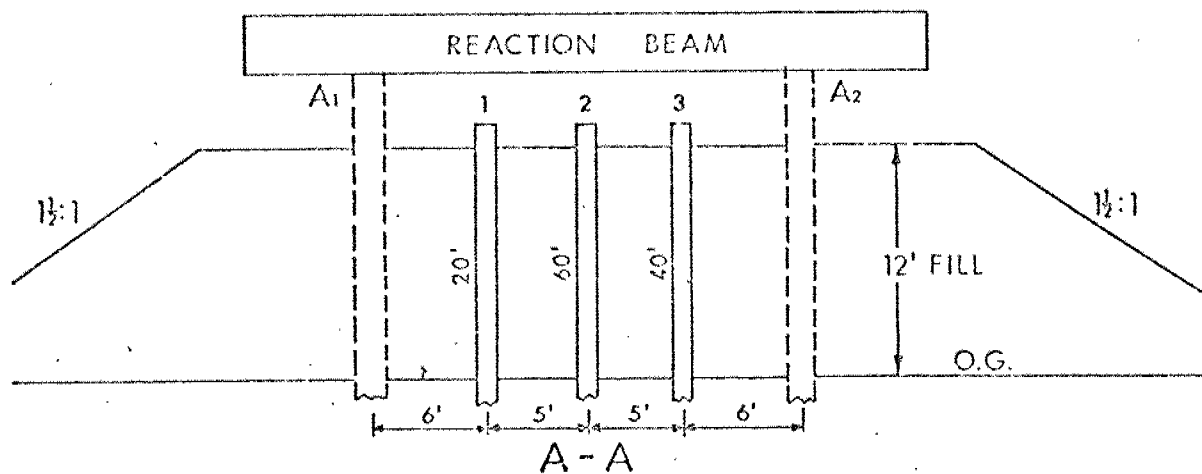
DIST. 1

JOB 70 - 11049

DATE 22 SEPT. 1970

APPROVED

DRAWING NO. 70 - 11049 B



SCALE 1" = 10'

FIG. 3



DEPARTMENT OF HIGHWAYS
MATERIALS and
TESTING
DIVISION

ONTARIO

BLACKWELL ROAD & HWY. 402
PILE TEST ARRANGEMENT - ELEVATION

W.P. 43 - 66 - 05

DIST. 1

JOB 70 - 11049

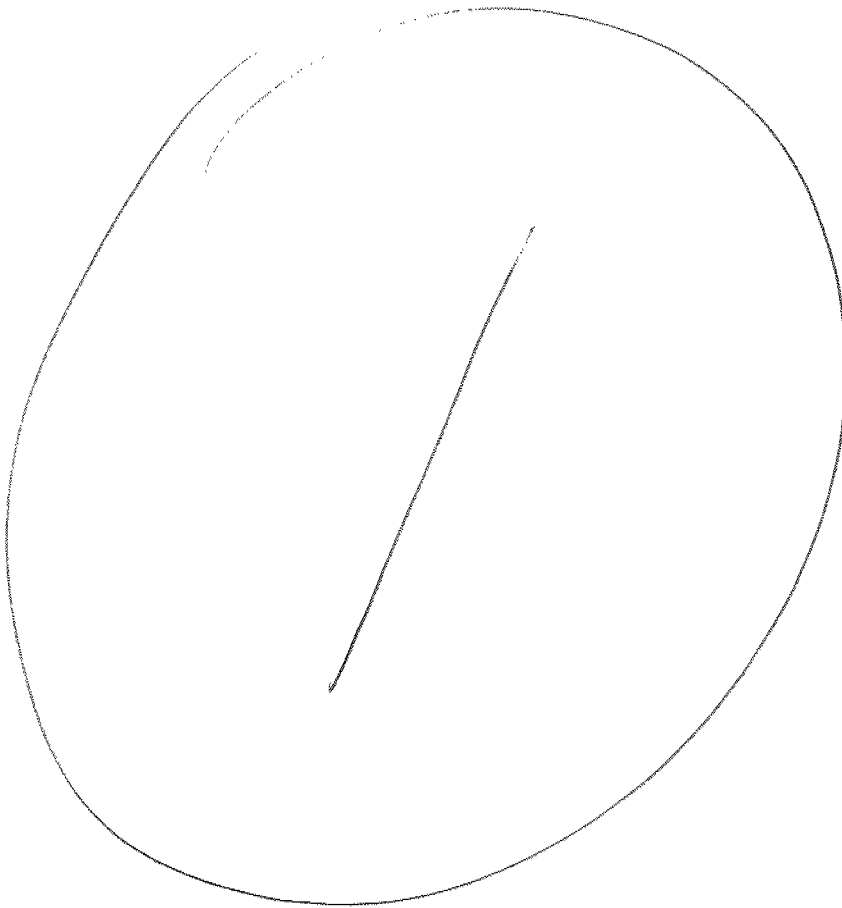
DATE 24 SEPT. 1970

APPROVED

DRAWING NO. 70 - 11049 C

35MM

DRAWING



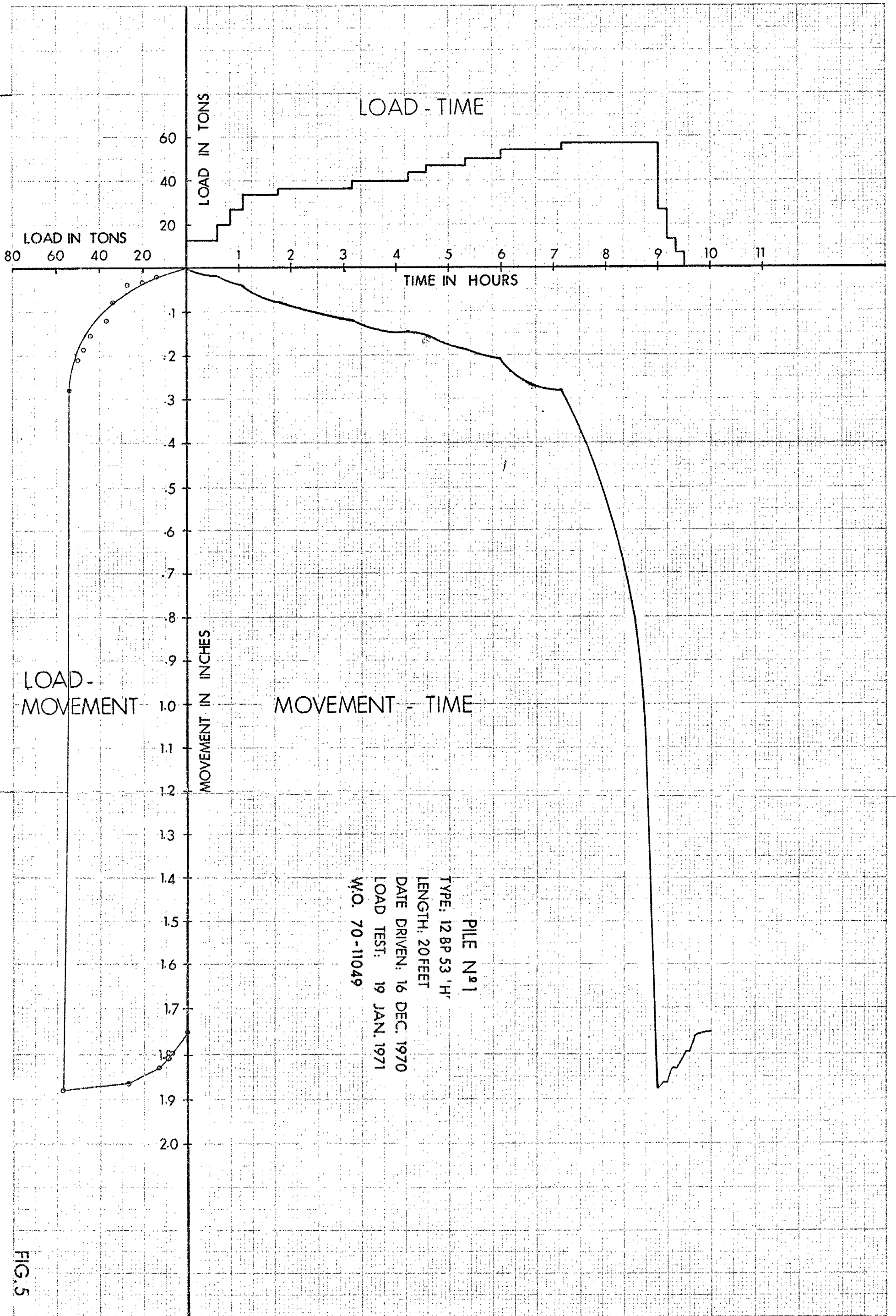


FIG. 5

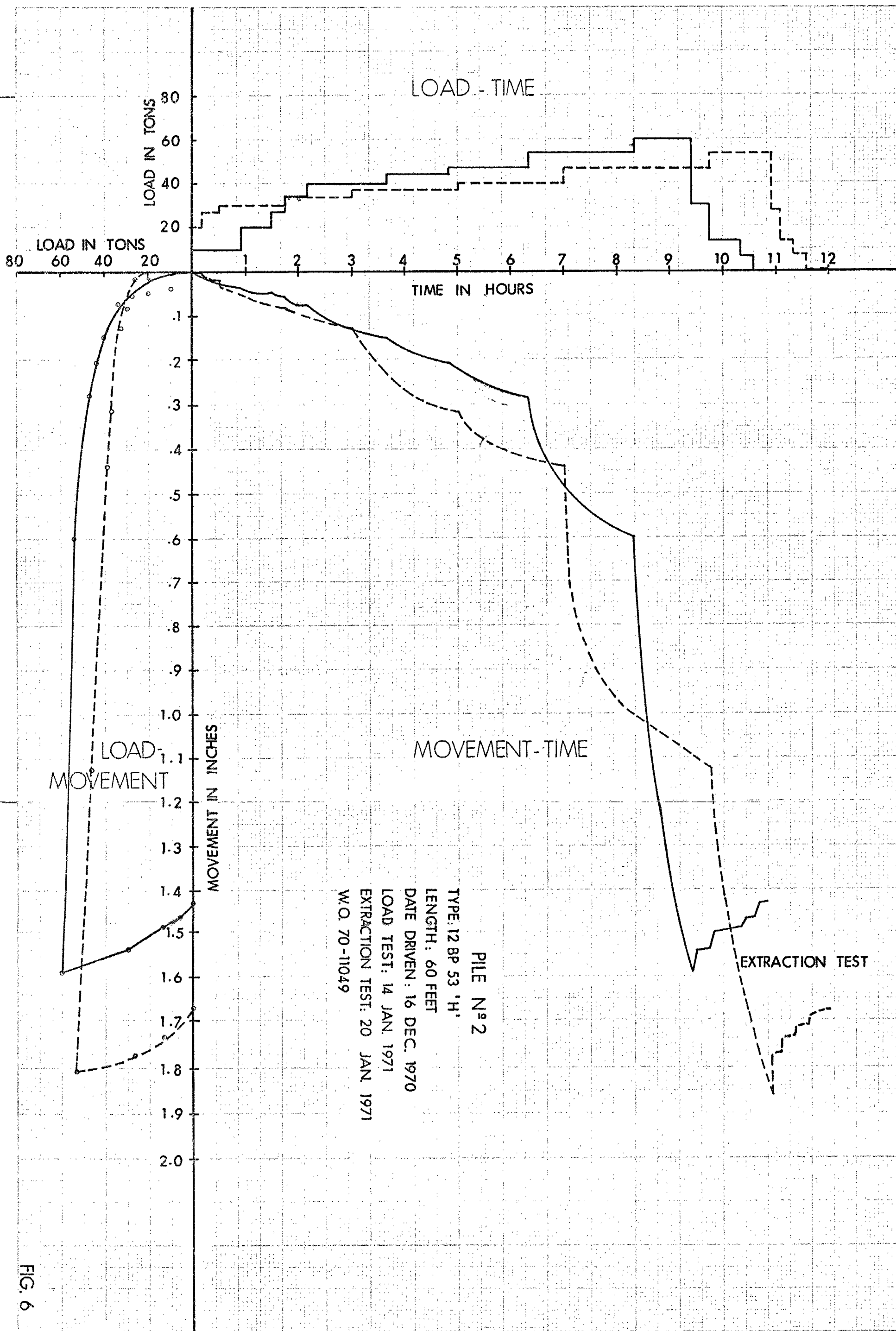


FIG. 6

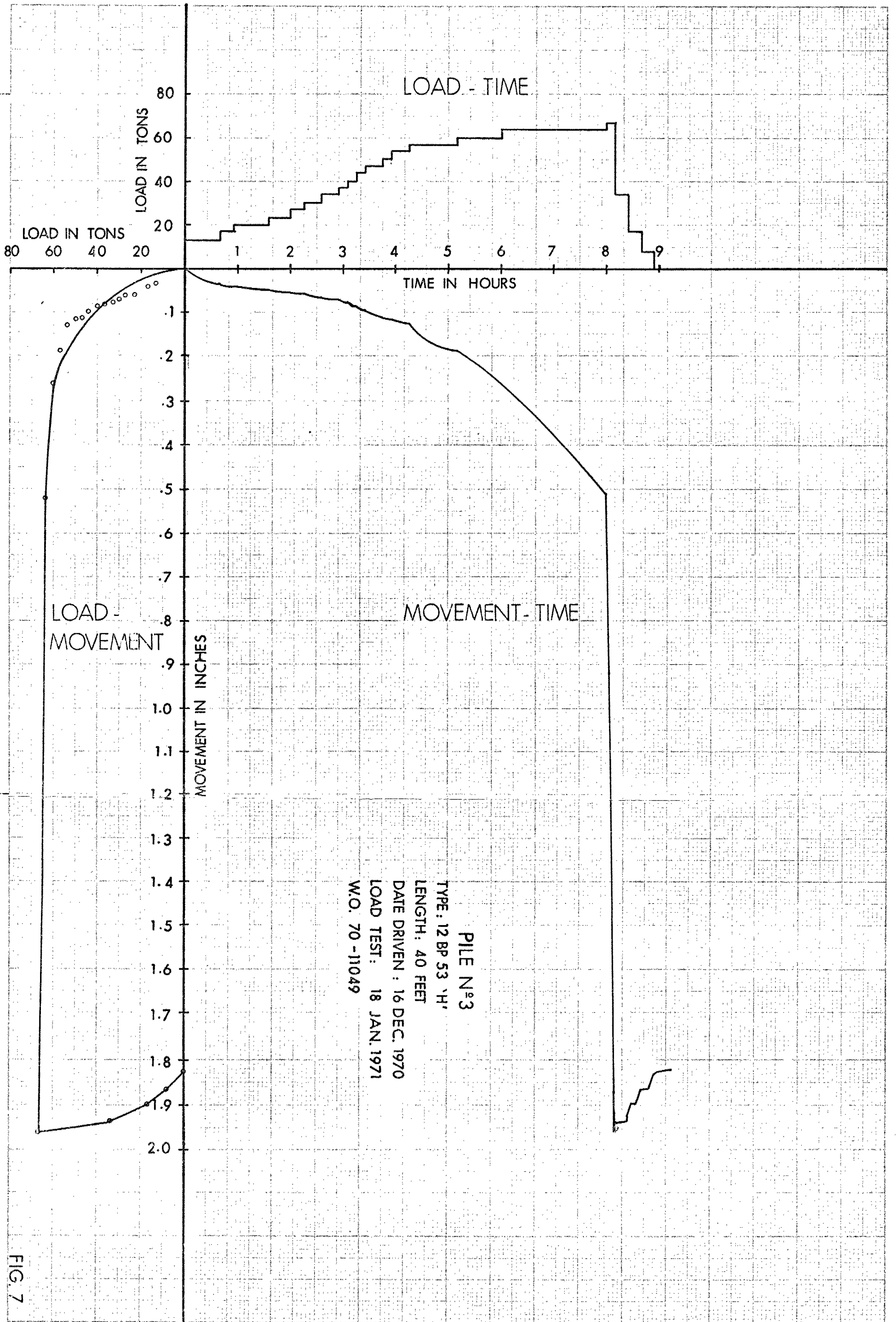


FIG. 7

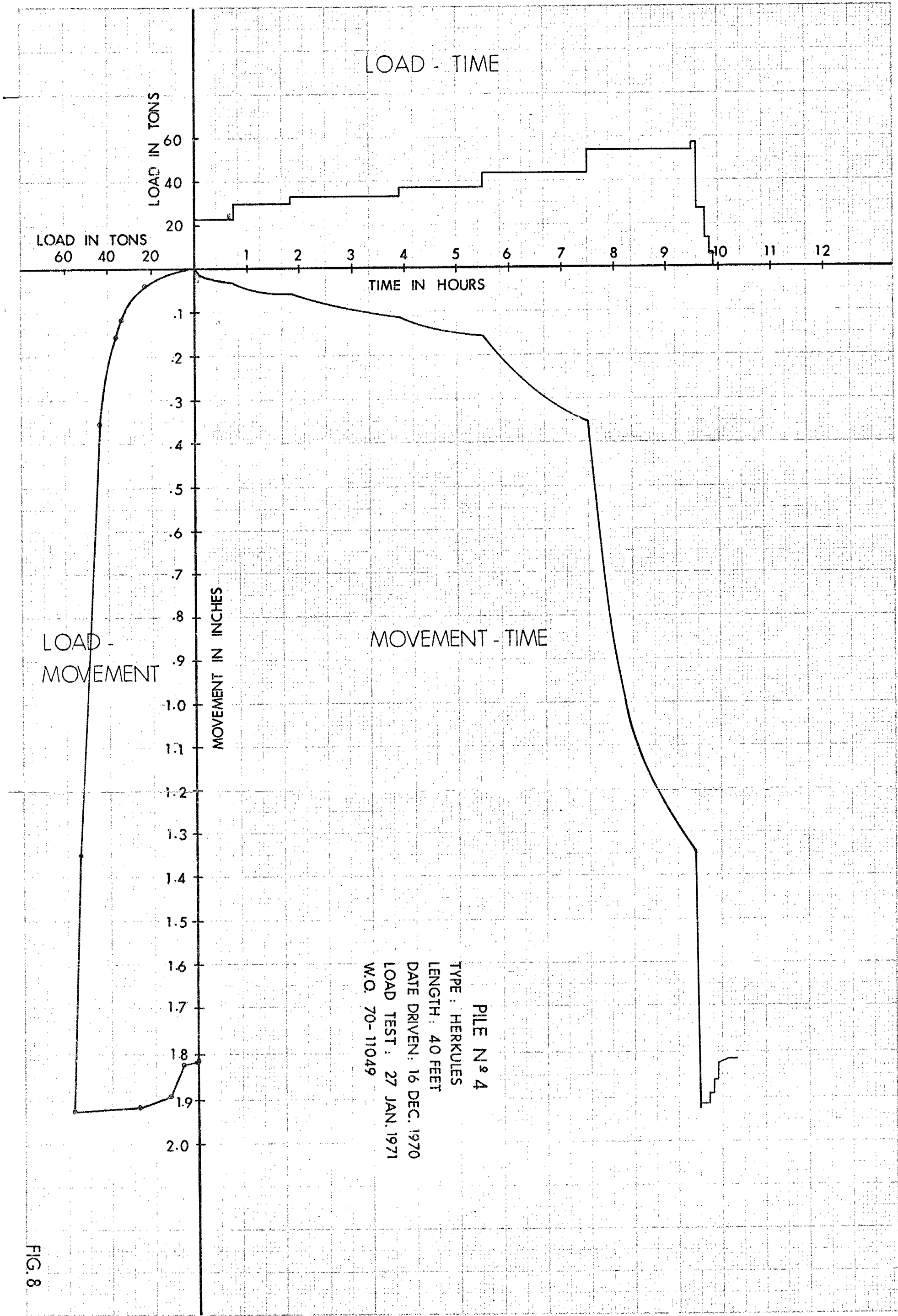


FIG. 8

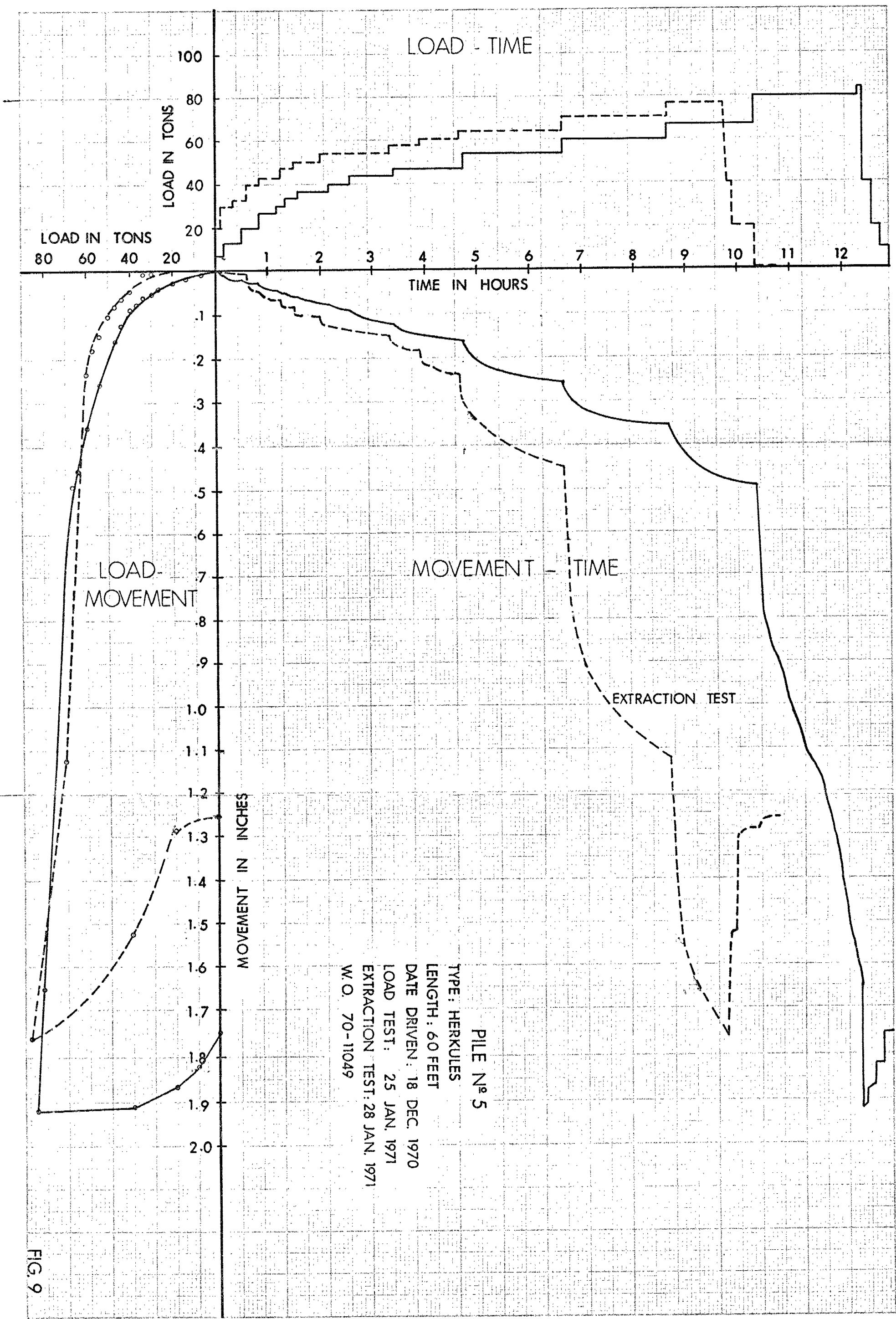


FIG. 9

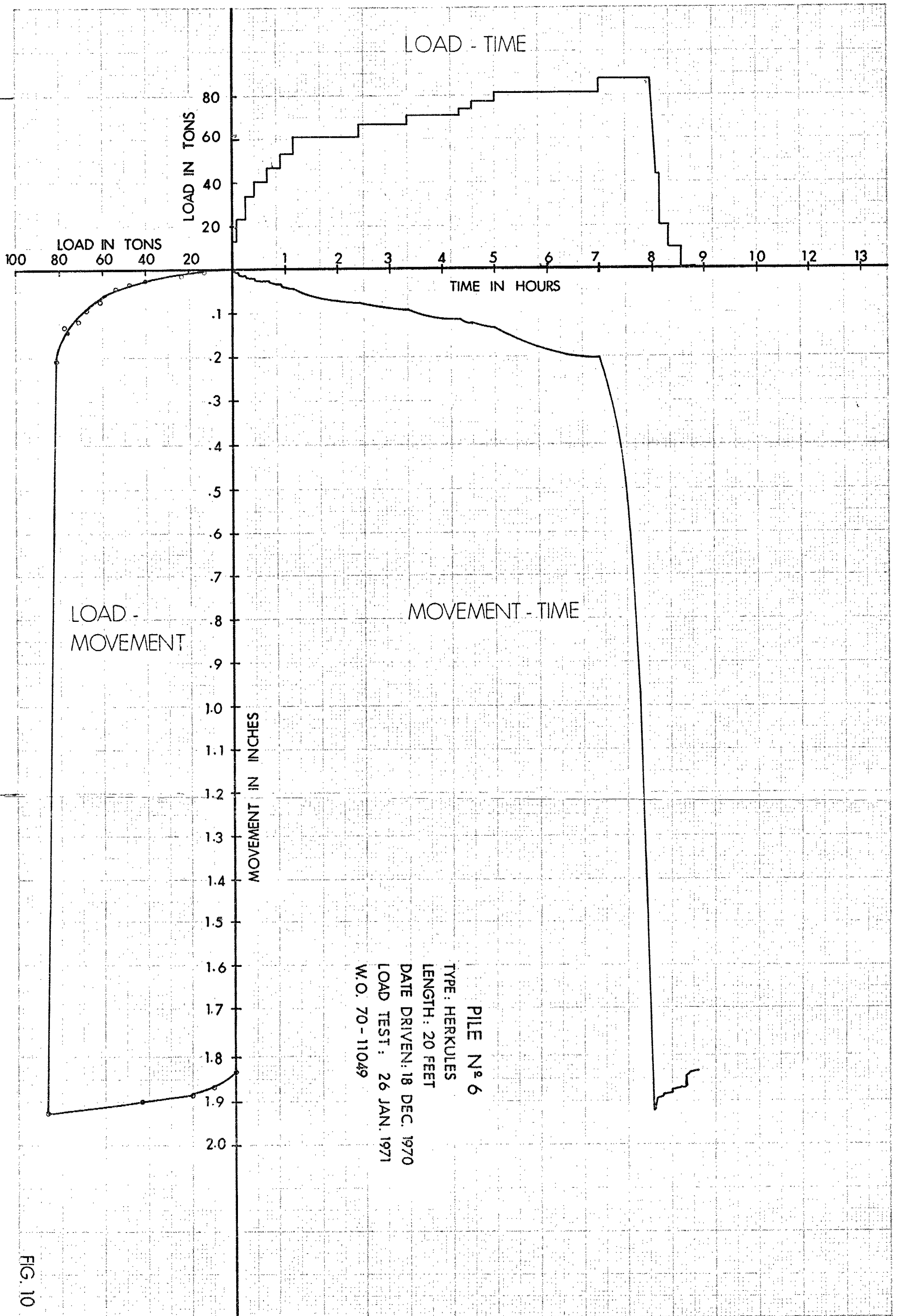


FIG. 10

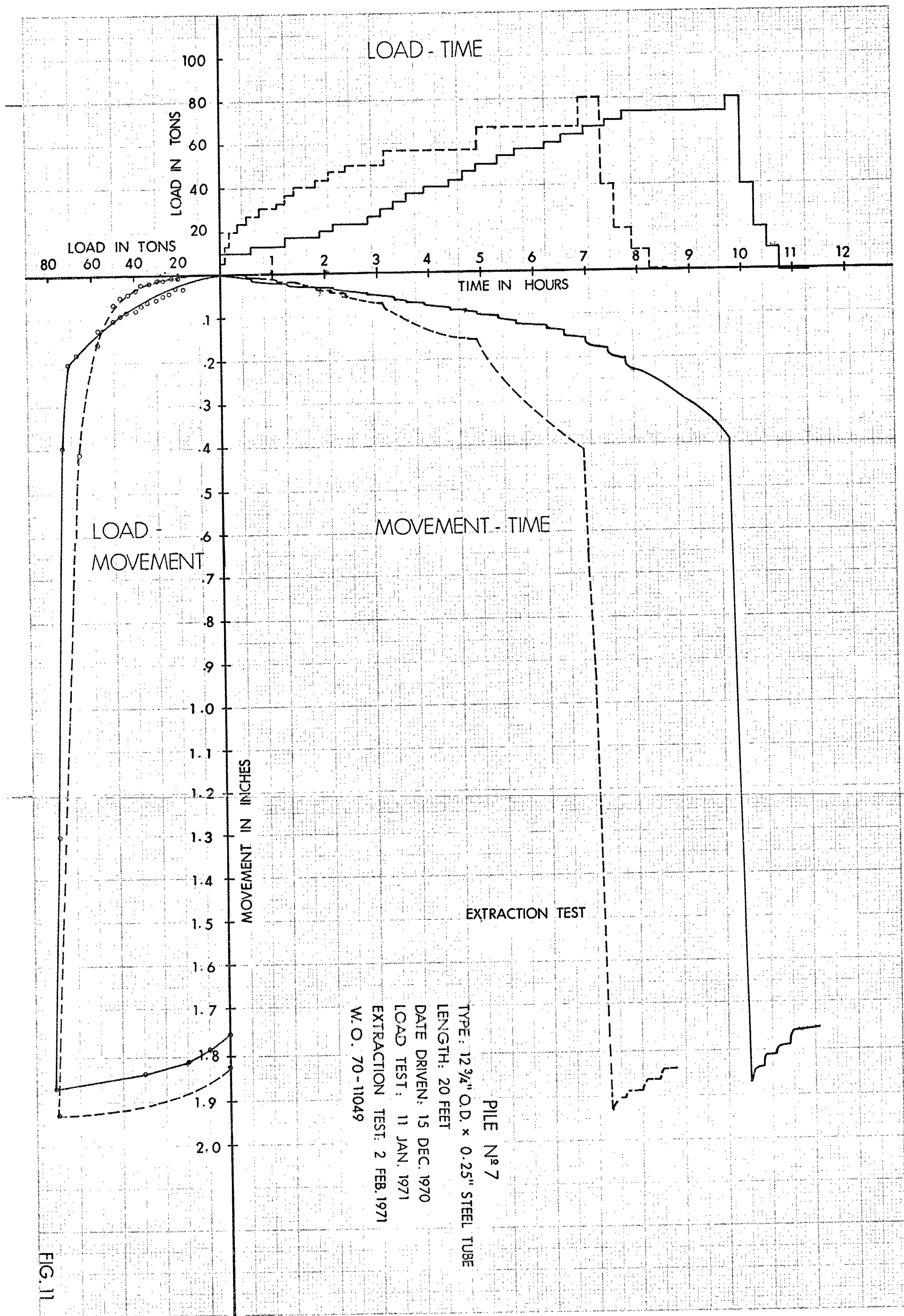


FIG. 11

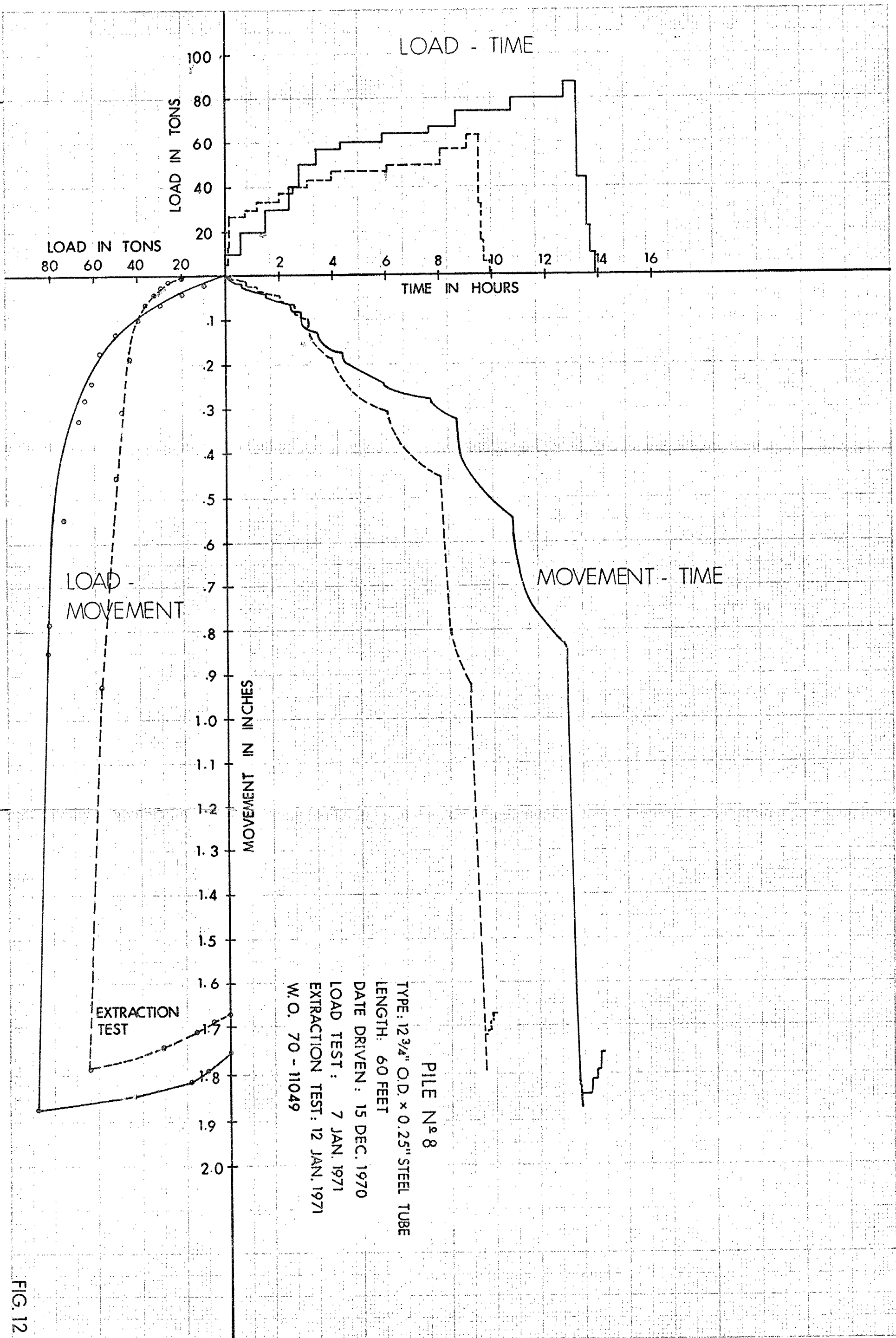


FIG. 12

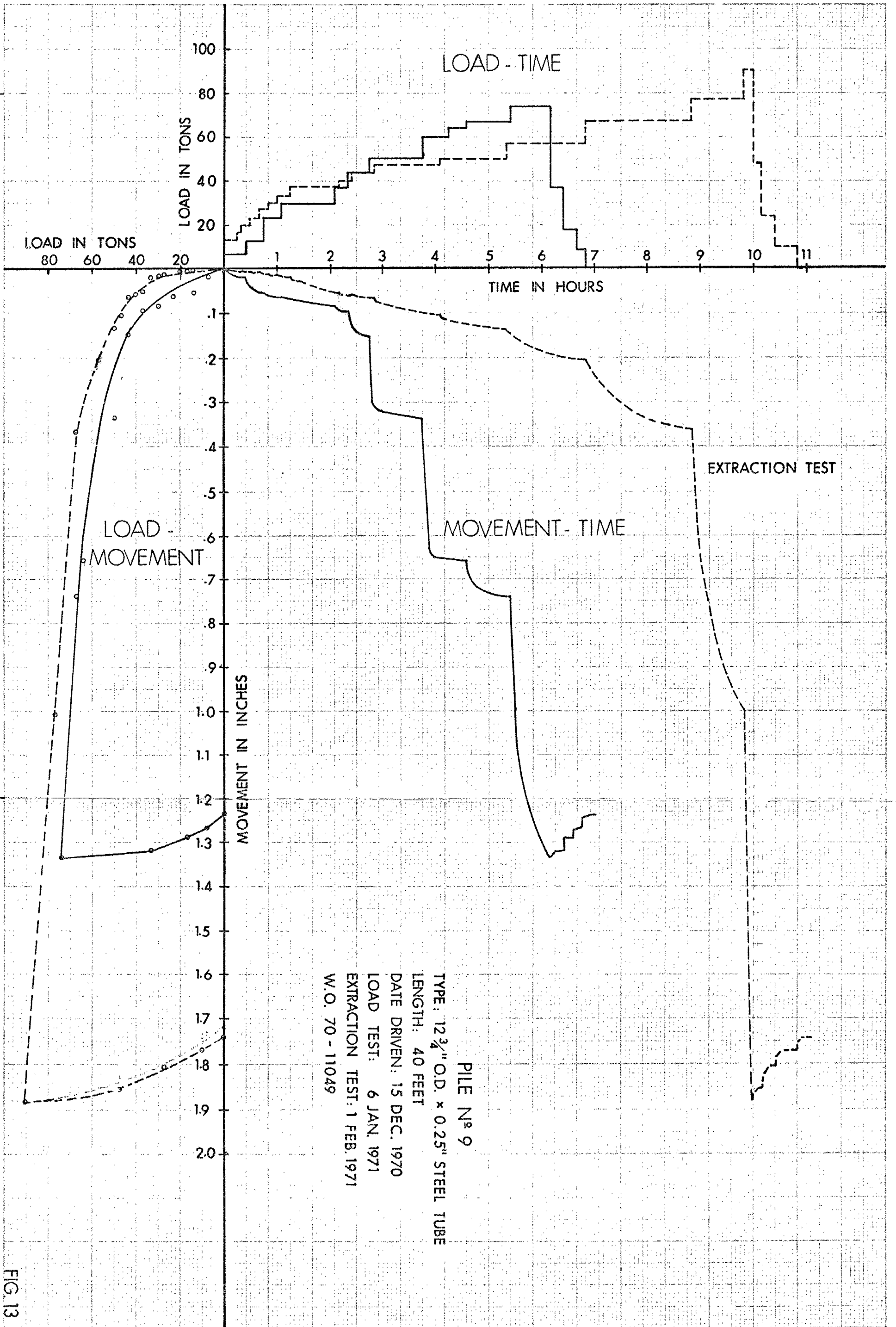


FIG 13

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS, ONTARIO

MEMORANDUM

40 J - 26
GEOCRES No.

TO: Mr. A. P. Watt, (2)
Regional Structural Planning Eng.,
Southwestern Region,
London, Ontario.

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

ATTENTION: Mr. S. Jants,
Structural Planning Technician

DATE: February 14, 1973.

OUR FILE REF. IN REPLY TO FEB 16 1973

SUBJECT:

40J16-47
GEOCRES No.

FOUNDATION INVESTIGATION REPORT
For
The Proposed Bridge Site 14-371
Airport Rd. and Telfer Channel
Diversion Approx. 0.5 Mi. South
of Hwy. 402
District #1, Chatham
W.O. 72-11125 -- W.P. 43-66-17
CONT. 75-27

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

We believe that the factual data and recommendations contained therein will prove adequate for your design requirements. Should additional information be required, please do not hesitate to contact our Office.

AGS/ao
Attach.

cc: E. J. Orr
B. R. Davis
A. Rutka
A. Wittenberg
F. C. Brown
B. J. Giroux
J. R. Roy
G. A. Wrong
B. A. Singh

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

Foundations Files
Documents

TABLE OF CONTENTS

1. INTRODUCTION.
 2. DESCRIPTION OF SITE.
 3. FIELD WORK AND LABORATORY INVESTIGATION.
 4. SUBSOIL CONDITIONS.
 - 4.1) General.
 - 4.2) Clay Silt, Some Sand, Traces of Gravel.
 - 4.3) Silty Clay, Some Sand, Traces of Gravel.
 - 4.4) Bedrock.
 5. GROUNDWATER CONDITIONS.
 6. DISCUSSION AND RECOMMENDATIONS.
 - 6.1) General.
 - 6.2) Foundations.
 - a) Spread Footings.
 - b) Pile Foundations.
 - 6.3) Channel Slopes.
 7. MISCELLANEOUS.
-

FOUNDATION INVESTIGATION REPORT
For
The Proposed Bridge Site 14-371
Airport Rd. and Telfer Channel
Diversion Approx. 0.5 Mi. South
of Hwy. 402
District #1, Chatham
W.O. 72-11125 -- W.P. 43-66-17

1. INTRODUCTION:

A foundation investigation was undertaken for the proposed Airport Road Bridge over Telfer Diversion Channel. The proposal consists of a two-lane, three-span structure. On receiving a request from Mr. S. Jants, Structural Planning Technician for the Southwestern Region, dated October 31, 1972, a field and laboratory investigation was undertaken by this Office so as to determine the existing subsoil and groundwater conditions at the proposed crossing. Presented in this report are the results of the investigation, together with recommendations concerning the structure foundations.

2. DESCRIPTION OF SITE:

At the proposed crossing the land is very flat. About 3/4 mile to the north is Sarnia airport. To the east of the site is an unused field and to the west is a trailer park.

Geologically, the site is part of the physiographic region known as the St. Clair Clay Plain. The region is one of little relief with a deep deposit of clay. At the site the clay is some 150 ft. deep. Most of Lambton County is essentially till plains smoothed by shallow deposits of lacustrine clay which settled in

the depressions while the knolls were being lowered by wave action.

3. FIELD WORK AND LABORATORY INVESTIGATION:

The field work consisted of three sampled boreholes and six dynamic cone tests, three of the cones being adjacent to the boreholes. The drilling was done by a Bombardier mounted C.M.E. equipped with hollow stem augers. The bedrock in B.H. #1 was cored by a second C.M.E. using NX casing and BX core barrel. Split-spoon samples were taken at regular intervals and standard penetration tests were conducted in driving the split spoon. The resulting penetration "N" values are recorded in the Appendix. Thin walled, 2 inch I.D. Shelby tube soil samples were obtained either by advancing the Shelby hydraulically or manually. In situ shear strength was measured using an M.T.C. vane. All field and laboratory test results are recorded on the accompanying borelog sheets.

Soil samples were identified in the field and again upon arrival in the laboratory. Laboratory tests to determine moisture content, grain size and Atterberg Limits were carried out on representative samples. The soil samples obtained from the Shelby tubes were subjected to unconfined compression, quick triaxial, and consolidation tests.

The groundwater levels across the site were determined by recording the water levels in the open boreholes over the period of the investigation.

The locations and elevations of the boreholes as well as a stratigraphical profile are plotted on Drawing 72-11125A attached at the end of this report. The surveying of the site was carried out by personnel from the Southwestern Engineering Surveys Section.

4. SUBSOIL CONDITIONS:

4.1) General:

Uniform subsoil conditions exist across the site. The

subsoil consists of a deep deposit of cohesive material overlying the bedrock. The first layer encountered was the roadbed material which is granular in nature and from 1 to 2 feet deep. This layer is followed by a deep deposit of clayey silt and then a layer of silty clay and finally bedrock. A summary of the main deposits is given below.

4.2) Clay Silt, Some Sand, Traces of Gravel:

The first layer encountered after the roadbed material was a deep deposit of clayey silt, some sand, traces of gravel. This deposit was found to extend some 74 ft. below ground level corresponding to elevation 523. Standard penetration "N" values within this layer varied between 13 and 48 blows per foot.

Laboratory grain size analyses yielded the following results.

Gravel	0 - 6%
Sand	6 - 28%
Silt	42 - 54%
Clay	28 - 45%

The following average physical properties were obtained from field and laboratory tests.

Natural Moisture Content (%)	12.5 - 23.5
Liquid Limit (%)	24 - 33
Plastic Limit (%)	14 - 22
Bulk Density (p.c.f.)	122 - 130

Undrained Shear Strength

Field Vane (p.s.f.)	800 - >2000
Unconfined Compression Test (p.s.f.)	545 - 1185
Triaxial Compression Test (p.s.f.)	570 - 1045

Based on the foregoing the consistency of this deposit is estimated to be firm to hard. A typical grain size envelope is included in the Appendix as Fig. 1.

4.3) Silty Clay, Some Sand, Traces of Gravel:

Following the clayey silt is a 90 foot deposit of silty

clay, some sand, traces of gravel. This deposit is between elevation 523 and 442 and extends to a depth of 154 ft. at which point the bedrock was encountered. Standard penetration "N" values range between 14 and 33 blows per foot which indicate that the consistency of the overall deposit varies from stiff to hard. Based on the obtained "N" values, the undrained shear strength is estimated to vary between 1000 p.s.f. and in the excess of 2000 p.s.f.

Laboratory grain size analyses yielded the following results:

Gravel	0%
Sand	8%
Silt	47%
Clay	45%

The following physical properties were obtained from laboratory tests of the subsoil samples.

Natural Moisture Content (%)	23 - 35
Liquid Limit (%)	39 - 41
Plastic Limit (%)	22 - 23

A typical grain size curve is included in the Appendix as Fig. 2.

4.5) Bedrock:

A brief description has been given below by Mr. K.W. Ingham, Geologist, for the borehole drilled to bedrock at this site, together with the appropriate bedrock elevation.

Hole No. 8	Bedrock at 442.7
442.7 - 443.4	Dark to medium grey calcareous shale; bedly weathered.
443.4 - 443.6	Dark grey calcareous shale.
443.6 - 448.5	Limestone; medium grey, fine grained, medium bedded, occasional thin dark grey shale seams
448.5 - 450.2	Dark grey calcareous shale; some thin interbeds of medium grey limestone.

5. GROUNDWATER CONDITIONS:

The following groundwater levels were observed during the field investigation.

B.H. #1	Elevation 578.8
B.H. #6	Elevation 583.3
B.H. #8	Elevation 577.1

The levels may not be representative of the actual groundwater levels due to the relative impermeable nature of the subsoil and the short duration of the field work.

6. DISCUSSION AND RECOMMENDATIONS:

6.1) General:

It is proposed to build a three-span (43.3' - 43.3' - 43.3') structure at Airport Rd. and Telfer Channel Diversion. The profile grade of Airport Rd. is to be raised by about 1 to 2 ft. to elevation 602. The base of the channel will be at elevation 576.

The subsoil at this site consists of a deep deposit of clayey silt and silty clay. The upper 9 to 12 ft. is a very stiff to hard desiccated surface crust. Below the crust the undrained shear strength of the material decreases until a minimum value of about 750 p.s.f. is reached, then increases again with depth, with some random variation.

6.2) Foundations:

a) Spread Footings:

The structure may be founded on spread footings. The abutment footings may be placed at or below elevation 590 with a safe net pressure of 1.8 t.s.f. For the piers the safe net pressure was calculated to be 1.5 t.s.f. at or below elevation 570. As the subsoil is susceptible to softening on contact with water it is recommended that the footing excavations be protected by a concrete working slab, immediately on exposure.

Settlement calculations indicate that the abutment and pier footings will settle in the order of 1.5 to 2.0 inches. A differential settlement of 1.0 inch between the pier and abutment footings should be allowed for in the design of the structure.

The foundations should be protected against the scour action of the water. The depth of scour may be obtained from the Hydrology Office.

b) Pile Foundations:

As an alternative the structure may be supported on timber piles driven to the elevation necessary to achieve the required pile capacity. It is recommended that the piles be treated if they are not completely below the groundwater level. In determining the safe capacity of a timber pile, the following equation may be used:

$$Q = 0.5 L \text{ (Tons)}$$

$$Q = \text{Safe capacity of one pile}$$

$$L = \text{Embedded length in original ground (ft.)}$$

Maximum settlements for the pile groups were calculated to be in the order of 1.5 inches. It is recommended that the structure be built to accommodate a 1.0 inch differential settlement between abutments and piers.

As a second alternative the structure may be supported on steel H-piles driven to bedrock utilizing the maximum allowable design load for the particular steel section adapted.

All footings and/or pile caps should be protected against frost action by at least 4 feet of earth cover.

No major dewatering problems are anticipated because of the relatively impervious nature of the subsoil.

6.3) Channel Slopes:

It is recommended that the proposed channel be constructed with 2:1 slopes and be protected against scour action in the vicinity of the new structure.

7. MISCELLANEOUS:

The field work was carried out from December 1 to December 13, 1972, and was supervised by Mr. P. Korgemagi, Project Foundations Engineer.

The equipment used was owned and operated by P.V.K. and Sons Drilling Ltd., Burford, Ontario.

This report was written by Mr. P. Korgemagi and reviewed by Mr. K. G. Selby, Supervising Foundations Engineer.

P. Korgemagi
P. Korgemagi, P. Eng.



K. G. Selby

PK/ao
Feb. 14, 1973.

K. G. Selby, P. Eng.

APPENDIX I

DESIGN SERVICES BRANCH

FOUNDATIONS OFFICE

RECORD OF BOREHOLE NO 1

JOB 72-11125

LOCATION Sta. 20 + 36 o/s 23' Rt. of Airport Rd.

ORIGINATED BY PK

W.P. 43-66-17

BORING DATE Dec. 4 to 8 & Dec. 1/72

COMPILED BY PK

DATUM Geodetic

BOREHOLE TYPE Hollowstem Auger & Washboring

CHECKED BY

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT				LIQUID LIMIT w_L PLASTIC LIMIT w_p WATER CONTENT w			BULK DENSITY γ	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FOOT		25	50	75	100	125	w_p	w	w_L	
596.9	Ground Level														
0.0	Gravelly sand														
594.4	Compact Fill														
2.5			1	SS	17										
			2	SS	37										
			3	SS	15										
585.9	Brown Very Stiff to Hard		4	SS	18										
11.0			5	SS	20										
			6	SS	18										
	Clayey silt some sand		7	SS	15										
			8	SS	18										
	traces of gravel		9	SS	21										
	Grey		10	TW	PH										
			11	SS	8 1/2"										
			12	SS	29										
	Stiff to Very Stiff		13	SS	13										
			14	TW	PH										
			15	SS	24										
522.9															
74.0			16	SS	33										
	Silty clay some sand, traces of gravel.														
			17	SS	25										
	Stiff to Hard														
	Grey		18	SS	19										

Continued

 20
15 5 % STRAIN AT FAILURE
10

OFFICE REPORT SOIL EXPLORATION

FOUNDATIONS OFFICE

JOB 72-11125

LOCATION Sta. 20 + 36 o/s 23rd Rt. @ Airport Rd.

ORIGINATED BY PK

W.P. 43-66-17

BORING DATE Dec. 4 to 8 & Dec. 1/72

COMPILED BY PK

DATUM Geodetic

BOREHOLE TYPE Hollowstem Auger & Washboring

CHECKED BY

20
15 ϕ 5 % STRAIN AT FAILURE
10

DESIGN SERVICES BRANCH

FOUNDATIONS OFFICE

RECORD OF BOREHOLE NO 3

JOB 72-11125

LOCATION Sta. 19+54 o/s 23' Rt. of Airport Rd.

ORIGINATED BY PK

W.P. 43-66-17

BORING DATE Dec. 1/72

COMPILED BY PK

DATUM Geodetic

BOREHOLE TYPE Cone Test

CHECKED BY

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT					LIQUID LIMIT — w_L PLASTIC LIMIT — w_p WATER CONTENT — w			BULK DENSITY γ	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FOOT		25	50	75	100	125	SHEAR STRENGTH P.S.F. ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE				
597.3	Ground Level															
0.0																
587.3						590										
10.0	End of Cone Test					580										

OFFICE REPORT SOIL EXPLORATION

DESIGN SERVICES BRANCH

FOUNDATIONS OFFICE

RECORD OF BOREHOLE N^o 4

JOB 72-11125

LOCATION Sta. 19 + 10 o/s 23' Rt. @ Airport Rd.

ORIGINATED BY PK

W.P. 43-66-17

BORING DATE Dec. 13/72

COMPILED BY PK

DATUM Geodetic

BOREHOLE TYPE Cone Test

CHECKED BY *[Signature]*

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE					LIQUID LIMIT W_L			BULK DENSITY	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FOOT		BLOWS / FOOT 25 50 75 100 125					PLASTIC LIMIT W_p WATER CONTENT W $W_p \rightarrow W \rightarrow W_L$				
597.5	Ground Level															
0.0																
587.7						590										
9.8	End of Cone Test															

OFFICE REPORT SOIL EXPLORATION

DESIGN SERVICES BRANCH

FOUNDATIONS OFFICE

RECORD OF BOREHOLE NO 5

JOB 72-11125

LOCATION Sta. 20 + 36 o/s 23' Lt. @ Airport Rd.

ORIGINATED BY PK

W.P. 43-66-17

BORING DATE Dec. 13/72

COMPILED BY PK

DATUM Geodetic

BOREHOLE TYPE Cone Test

CHECKED BY *[Signature]*

SOIL PROFILE			SAMPLES			DYNAMIC PENETRATION RESISTANCE					LIQUID LIMIT — w_L			BULK DENSITY	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FOOT	ELEV. SCALE	BLOWS / FOOT					PLASTIC LIMIT — w_p			
							25	50	75	100	125	WATER CONTENT — w			
							SHEAR STRENGTH P.S.F.					w_p — w — w_L WATER CONTENT %			
							○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE								
598.1	Ground Level														
0.0															
586.8						590									
12.0	End of Cone Test														

OFFICE REPORT SOIL EXPLORATION

DESIGN SERVICES BRANCH

FOUNDATIONS OFFICE

RECORD OF BOREHOLE NO 6

JOB 72-11125

LOCATION Sta. 19 + 97 o/s 23' Lt. @ Airport Rd.

ORIGINATED BY PK

W.P. 43-66-17

BORING DATE Dec. 6 and Dec. 13, 1972

COMPILED BY PK

DATUM Geodetic

BOREHOLE TYPE Hollow Stem Auger

CHECKED BY

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT					LIQUID LIMIT — W _L PLASTIC LIMIT — W _p WATER CONTENT — W			BULK DENSITY γ	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FOOT		25	50	75	100	125	W _p	W	W _L		
598.3	Ground Level															
0.0	Granular Roadway Fill															
596.3			1	SS	13											2 28 42 28
2.0			2	SS	34											
			3	SS	66	590										
			4	SS	35											
582.3	Brown Stiff to Hard		5	SS	29											
16.0			6	SS	21											
			7	SS	15	580										2 16 47 35
			8	SS	12											
	Clayey silt some sand, traces of gravel Grey		9	TW	PH	570									129	
			10	SS	23											2 14 45 39
			11	SS	22	560										
	Stiff to very stiff		12	TW	PH											
			13	SS	12	550										0 16 50 34
			14	TW	PH										124.5	
															123.5	
534.3			15	SS	15	540										
64.0	End of B.H.															

OFFICE REPORT OF SOIL EXPLORATION

DESIGN SERVICES BRANCH

FOUNDATIONS OFFICE

RECORD OF BOREHOLE NO 8

JOB 72-11125

LOCATION Sta. 19 + 10 o/s 23' Lt. @ Airport Rd.

ORIGINATED BY PK

W.P. 43-66-17

BORING DATE Dec. 1 & 7, 1972

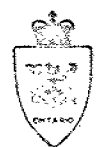
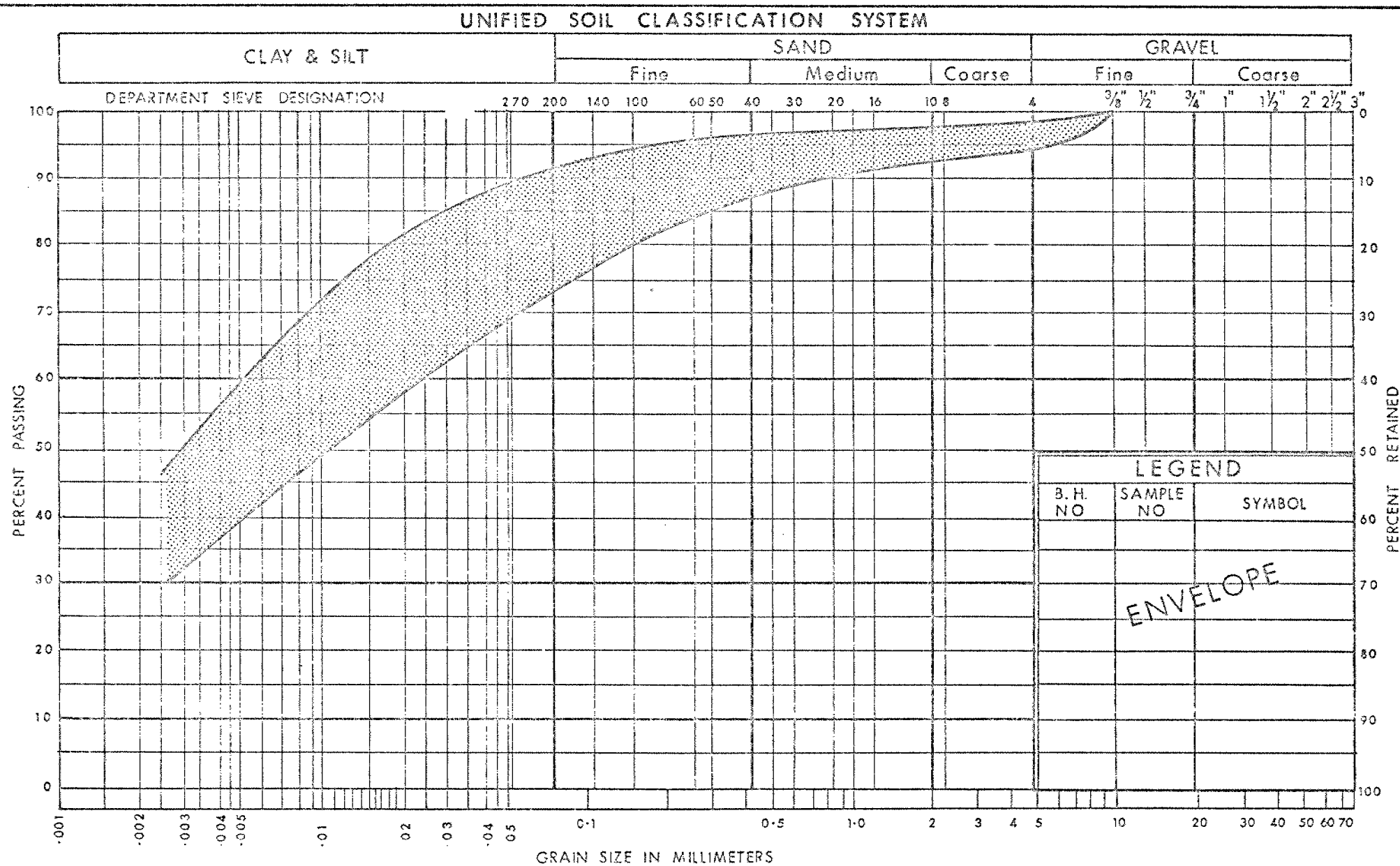
COMPILED BY PK

DATUM Geodetic

BOREHOLE TYPE Hollow Stem Auger

CHECKED BY

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT					LIQUID LIMIT W_L PLASTIC LIMIT W_P WATER CONTENT W			BULK DENSITY γ	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FOOT		25	50	75	100	125	W_P	W	W_L		
598.1	Ground Level															
0.0	Gravelly Fill															
0.5	Brown		1	SS	25											1 26 45 28
			2	SS	52											
588.1	Very Stiff to Hard		3	SS	61	590										
10.0	Clayey silt		4	SS	44											2 15 48 35
	some sand, traces of		5	SS	25											
	gravel		6	SS	22											
			7	SS	16	580										
	Grey		8	SS	14											
			9	TW	PH	570									129	
			10	TW	PH										129	3 14 47 36
	Stiff to Hard		11	SS	17	560									130	
			12	SS	13											
			13	TW	PH	550									126.5	
			14	SS	12	540									126	1 6 54 39
			15	SS	15	530										
519.1			16	SS	33	520										
79.0	End of Borehole															



DEPARTMENT OF HIGHWAYS
MATERIALS and
TESTING
DIVISION

GRAIN SIZE DISTRIBUTION

CLAYEY SILT

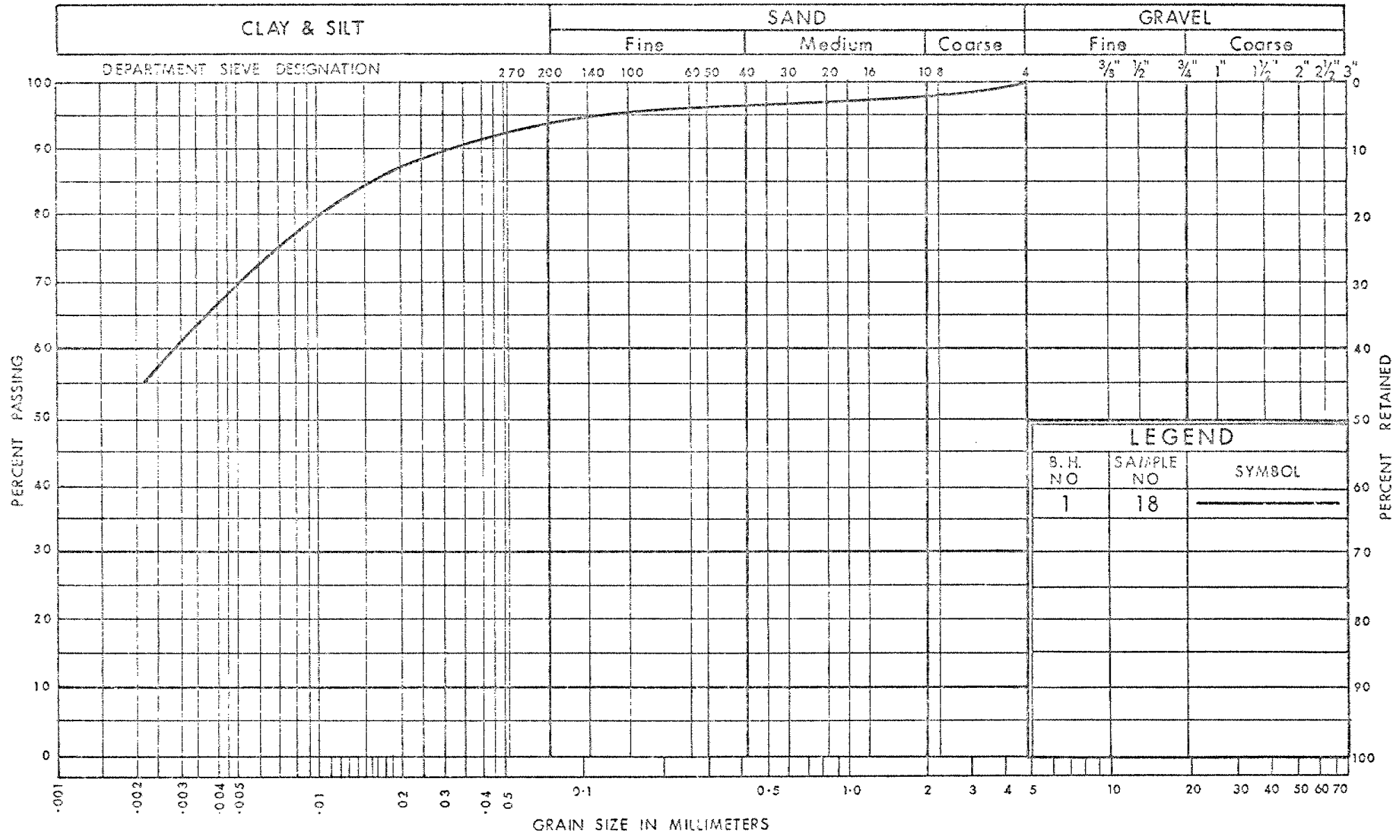
SOME SAND, TRACES OF GRAVEL

W.P. No. 43-66-17

JOB No. 72-11125

FIG. NO. 1

UNIFIED SOIL CLASSIFICATION SYSTEM



DEPARTMENT OF HIGHWAYS
MATERIALS and
TESTING
DIVISION

GRAIN SIZE DISTRIBUTION
SILTY CLAY
SOME SAND, TRACES OF GRAVEL

W.P. No. 43-66-17
JOB No. 72-11125
FIG. NO. 2

ABBREVIATIONS & SYMBOLS USED IN THIS REPORTPENETRATION RESISTANCE

'N'=STANDARD PENETRATION RESISTANCE : - 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>c LB./SQ. FT.</u>	<u>DENSENESS</u>	<u>'N' BLOWS / FT.</u>
VERY SOFT	0 - 250	VERY LOOSE	0 - 4
SOFT	250 - 500	LOOSE	4 - 10
FIRM	500 - 1000	COMPACT	10 - 30
STIFF	1000 - 2000	DENSE	30 - 50
VERY STIFF	2000 - 4000	VERY DENSE	> 50
HARD	> 4000		

TERMS TO BE USED IN DESCRIBING SOILS:-

TRACE < 10% , SOME 10-25% , WITH 25-40% , > 40% SILTY, SANDY, GRAVELLY, CLAYEY ETC.

TYPE OF SAMPLE

S.S.	SPLIT SPOON	T.W.	THINWALL OPEN
W.S.	WASHED SAMPLE	T.P.	THINWALL PISTON
S.T.	SLOTTED TUBE SAMPLE	O.S.	OESTERBERG SAMPLE
A.S.	AUGER SAMPLE	F.S.	FOIL SAMPLE
C.S.	CHUNK SAMPLE	R.C.	ROCK CORE

P.H. SAMPLE ADVANCED HYDRAULICALLY

P.M. SAMPLE ADVANCED MANUALLY

SOIL TESTS

U	UNCONFINED COMPRESSION	L.V.	LABORATORY VANE
UU	UNCONSOLIDATED UNDRAINED TRIAXIAL	F.V.	FIELD VANE
CU	CONSOLIDATED ISOTROPIC UNDRAINED TRIAXIAL	C	CONSOLIDATION
CID	" " DRAINED "	S	SENSITIVITY
CAU	" " ANISOTROPIC UNDRAINED "		
CAD	" " DRAINED "		

ABBREVIATIONS & SYMBOLS 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
w_s	SHRINKAGE LIMIT
I_L	LIQUIDITY INDEX $= \frac{w - w_p}{I_p}$
I_c	CONSISTENCY INDEX $= \frac{w_L - w}{I_p}$
e_{max}	VOID RATIO IN LOOSEST STATE
e_{min}	VOID RATIO IN DENSEST STATE
I_D	DENSITY INDEX $= \frac{e_{max} - e}{e_{max} - e_{min}}$
	RELATIVE DENSITY D_r IS ALSO USED
h	HYDRAULIC HEAD OR POTENTIAL
q	RATE OF DISCHARGE
v	VELOCITY OF FLOW
i	HYDRAULIC GRADIENT
k	COEFFICIENT OF PERMEABILITY
j	SEEPAGE FORCE PER UNIT VOLUME
m_v	COEFFICIENT OF VOLUME CHANGE $= \frac{-\Delta e}{(1+e)\Delta\sigma}$
c_v	COEFFICIENT OF CONSOLIDATION
C_c	COMPRESSION INDEX $= \frac{\Delta e}{\Delta \log_{10} \sigma}$
T_v	TIME FACTOR $= \frac{c_v t}{d^2}$ (d, DRAINAGE PATH)
U	DEGREE OF CONSOLIDATION
τ_f	SHEAR STRENGTH
c'	EFFECTIVE COHESION INTERCEPT
ϕ'	EFFECTIVE ANGLE OF SHEARING RESISTANCE, OR FRICTION
c_u	APPARENT COHESION
ϕ_u	APPARENT ANGLE OF SHEARING RESISTANCE, OR FRICTION
μ	COEFFICIENT OF FRICTION
S_t	SENSITIVITY

GENERAL

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

STRESS AND STRAIN

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

EARTH PRESSURE

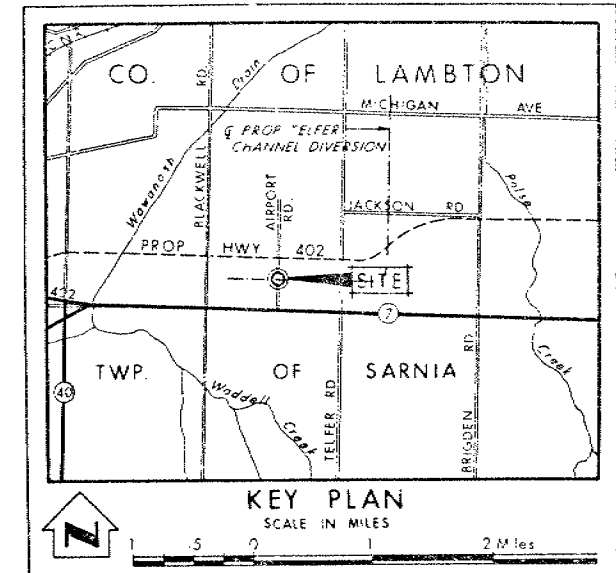
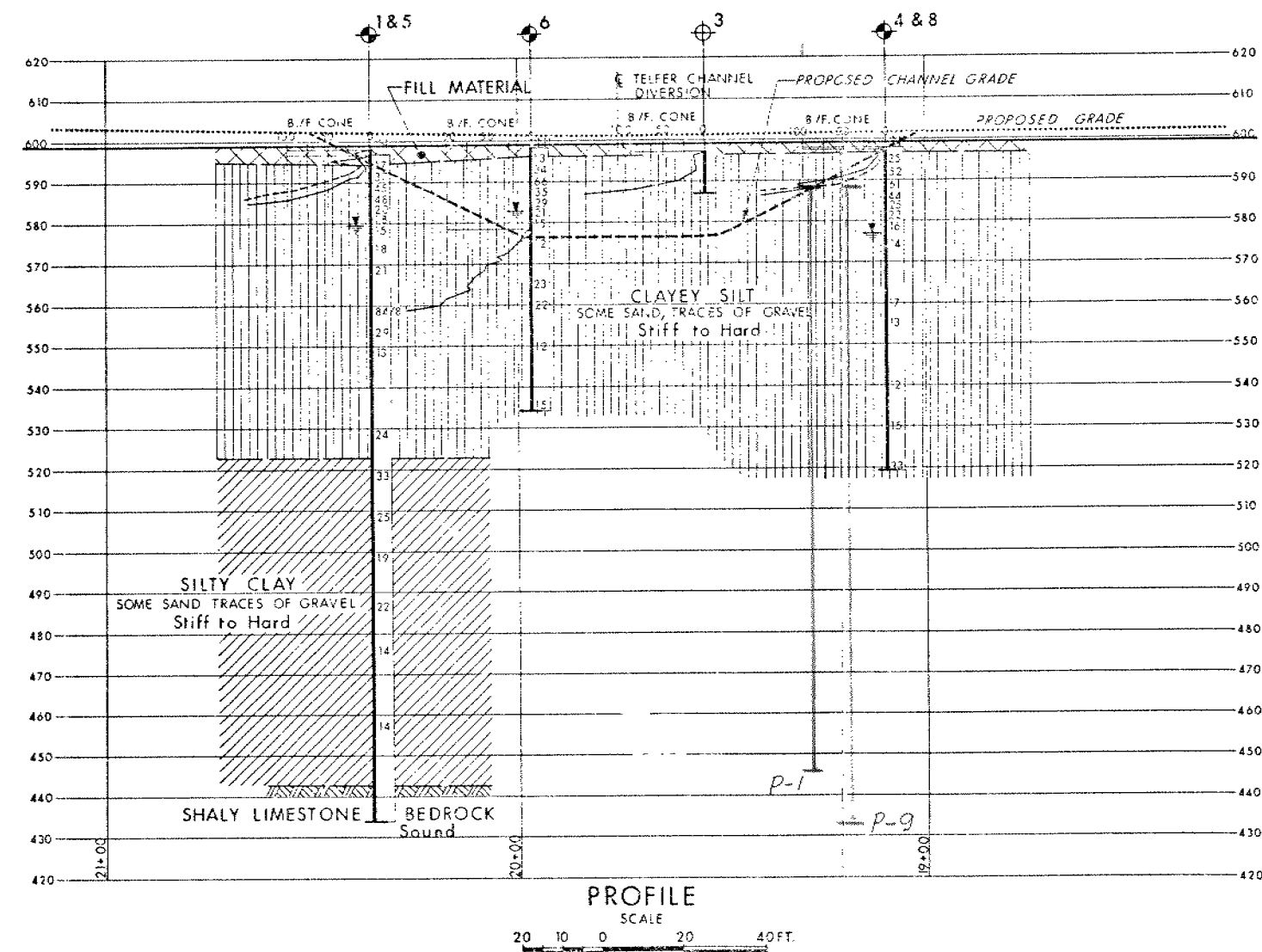
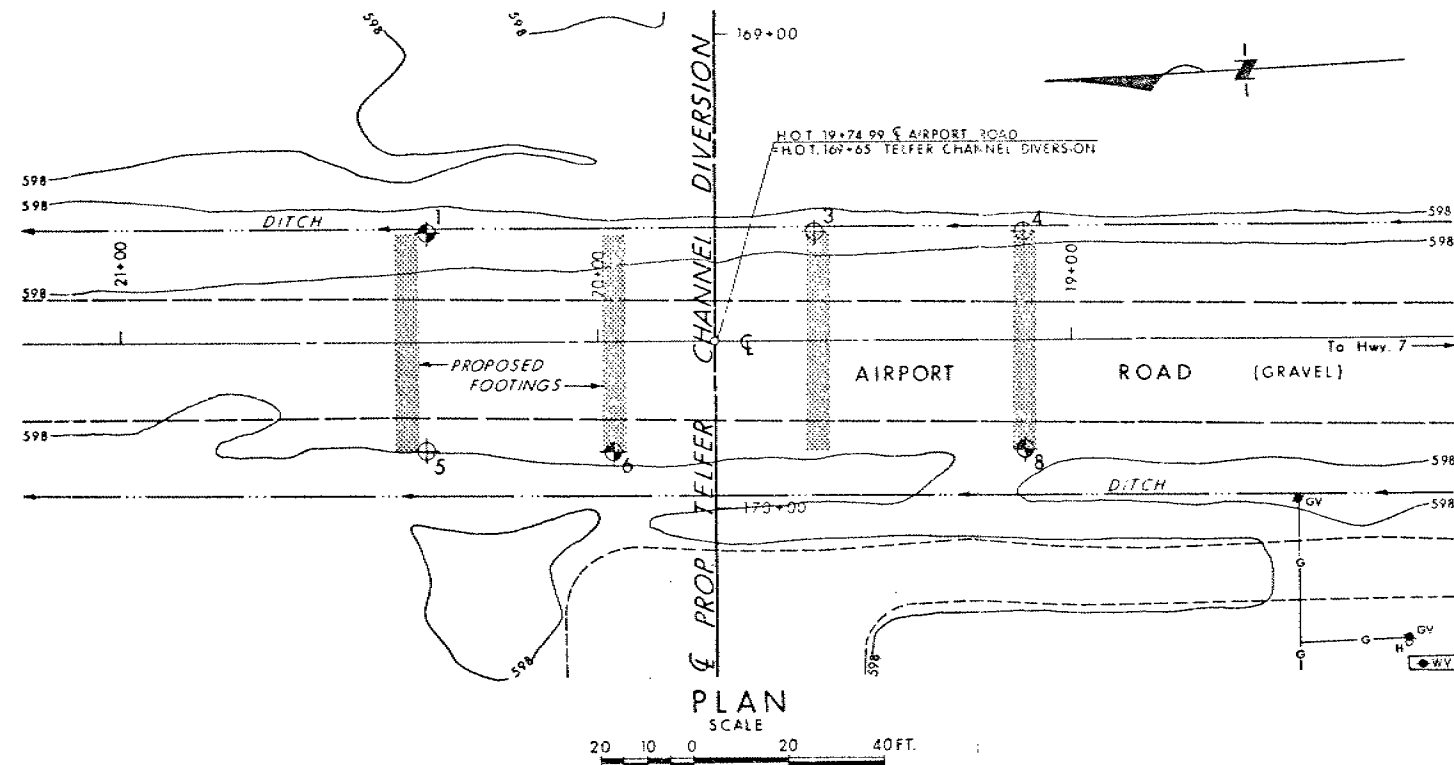
d	DISTANCE FROM TOP OF WALL TO POINT OF APPLICATION OF PRESSURE
δ	ANGLE OF WALL FRICTION
K	DIMENSIONLESS COEFFICIENT TO BE USED WITH VARIOUS SUFFIXES IN EXPRESSIONS REFERRING TO NORMAL STRESS ON WALLS
K_0	COEFFICIENT OF EARTH PRESSURE AT REST

FOUNDATIONS

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

SLOPES

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



LEGEND			
	Bore Hole		
	Cone Penetration Test		
	Bore Hole & Cone Test		
	Water Levels established at time of field investigation Dec 1972		
NO.	ELEVATION	STATION	OFFSET
1	596.9	20+36	23' RT.
3	597.3	19+54	23' RT.
4	597.5	19+10	23' RT.
5	598.1	20+36	23' LT.
6	598.3	19+97	23' LT.
8	598.1	19+10	23' LT.

NOTE
The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence.

REVISIONS	DATE	BY	DESCRIPTION



MINISTRY OF TRANSPORTATION AND COMMUNICATIONS - ONTARIO
DESIGN SERVICES BRANCH - FOUNDATIONS DIVISION

AIRPORT ROAD & TELFER CHANNEL DIVERSION

HIGHWAY NO. _____ DIST. NO. 1
CO. LAMBTON
TWP. SARNIA LOT 11 CON. VII

BORE HOLE LOCATIONS & SOIL STRATA

SUBMD P.K.	CHECKED: [initials]	WP NO. 43-66-17	DRAWING NO. 72-11125A
DRAWN: [initials]	CHECKED: [initials]	WD NO. 72-11125	BROGE DRAWING NO.
DATE Feb. 2, 1975	SITE NO. 14-371	CONT. NO.	

APPROVED: [signature] PROJECT ENGINEER

REF NO E-5333-1