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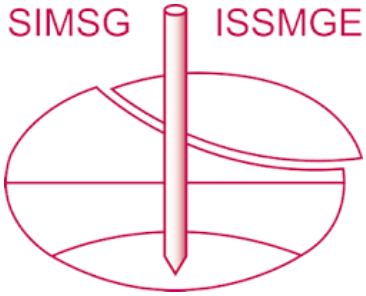

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the permanent set, δ', after removal of the load. These tests demonstrated that the piles could sustain the design loads with safety. The computations clearly show why this should be the case, as very little lateral motion could take place in the piles due to loading.

Another interesting test was performed at a bridge site in 1939. At the test location, the soil formation consisted of approximately 100 ft. soft clay and silt, 25 ft. soft silt and sand, and 53 ft. of firm materials such as sand and gravel, compact sand, and firm clay; underlying this formation was rock. The 14" H-102 lb. test pile, 188 ft. long, was driven to a final penetration of .60 in. per blow by a single-acting steam hammer with a 14,000-lb. ram falling 27 inches. This pile, fully braced to prevent lateral displacement, was loaded with a static load of 200 tons. Under this load, the settlement of the top of the pile was .924 inches, and upon removal of the load a final set of .180 inches was recorded.

Soil tests showed the average virgin branch of the pressure void-ratio diagram to be:

$$\varepsilon = 1.833 - .485 \log_e p$$

and the preconsolidation load to average .27 tons/sq.ft. or 12.0 lbs./sq.in.

From these data, the following were computed, based on $\alpha = L/400$ and E = 30,000,000 lbs./sq.in.

$$c = 20.5 \text{ lbs./sq.in.}$$

$$P_{cr} = 10{,}600{,}000 \text{ lbs.}$$

$$P = 304{,}000 \text{ lbs.}$$

In this case it can be concluded that either the initial distortion was less than assumed or the consolidation process had not fully taken place in the two days during which the load was being placed and the following four days that the load remained. Further, the computations make no allowance for the restraint provided by firm materials underlying the soft soils. The test demonstrates the lateral support provided by soft muds.

As a result of this study, we can draw the following conclusions:

1) Lateral stability provided by the soil has a marked influence on the strength of long piles.
2) Initial distortion of the piles contributes to loss of pile strength.
3) Ordinary column formulae do not provide a measure of strength of long piles.
4) In making load tests on piles, consideration should be given to time element of consolidation.
5) Where soil characteristics are known, a fair estimate of the ultimate strength of the pile may be determined by methods herein outlined.
6) Further study should be undertaken to extend these methods to other pile problems.
7) Further information should be secured on initial distortion of piles by field observation before driving and of open-ended pipe piles prior to filling.

REFERENCES.

1) Hjalmar Granholm
"On the elastic stability of piles surrounded by a supporting medium."
Stockholm, 1929.
2) M.A. Biot
"Bending of an infinite beam on an elastic foundation."
Journal of Applied Mechanics
March, 1937.
3) A.E.H. Love
"A treatise on the mathematical theory of elasticity."
Cambridge, 1934.
(pp. 185, 191)

ADDITIONAL REFERENCES

1) Cummings, A.E., "The Stability of Foundation Piles against Buckling Under Axial Load", Proc. Highway Research Board, December 1938, Vol. 38, No. II, p. 112.
2) Donnell, L.H., "On the Application of Southwell's Method for Analysis of Buckling Tests", "Stephen Timoshenko 60th Anniversary Volume", 1938, Macmillan Co., New York.
3) Hetenyi, M., "Beams on Elastic Foundations", 1946, University of Michigan Press, Ann Arbor.
4) McIntosh, W.T., "Extra Long Cylinder Piles Carry Column Loads to Rock", Engineering News Record, July 7, 1932, Vol. 109, p. 5.
5) "Long Piles and Floating Caissons for Potomac River Bridge Piers", Engineering News Record, Feb. 29, 1940, Vol. 124, p. 306.
6) Ciampa, Felix A., "Pier on 200-ft. Piles in N.Y. Harbor" Engineering News Record, Jan. 22, 1948, Vol. 140, v.p. 104.

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VII a 9 INFLUENCE OF CONSTRUCTION METHODS ON THE BEARING CAPACITY OF BOARD PILES

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INTRODUCTION.

In connection with the construction of foundations for important industrial buildings in the Milan suburbs, a series of test piles was set up in order to stress the influence of various construction methods upon the ultimate bearing capacity of the piles.

All tests were carried out on Rodio piles (bored piles), concreted under water by means of an automatic watertight bucket. The characteristics and various advantages of this type of pile may be found in various publications 1).

Ground conditions were particularly favorable for comparative tests. Fairly clean gravels and sands were found to be overlaid by a layer, 2.2 to 4 m thick, of clayey organic topsoil. Foundation explorations were done by means of borings and tests pits (with lowering of ground water level in the vicinity).

The testing program included the construction of 5 groups of 2 identical piles, each group having the following characteristics:

1) Two piles, down to a depth of 14 m below ground level, constructed in the same way as those on the neighbouring construction job, with reinforcing bars 2.50 m long in the upper proportion.

Test loading to 40 tons, unloading to 0 t, loading to 60 tons, unloading to 0 t, then loading to rupture of either pile or soil (if possible).

2) - Id. - , but pile bored while keeping the water level at least 50 cm above ground level.

Pile No.	Theoretical volume outside diameter casing shoe m3	Volume extracted m3	Concrete m3	Quantity of buckets	Volume of bucket m3
1	1.255	1.520	1.640	15 3/4	0.104
2	1.270	1.600	1.380	13 1/4	
3	1.280	———	1.370	13	
4	1.270	1.620	1.145	11	
5	1.200	1.640	1.510	27 1/2	0.055
6	1.200	1.560	1.290	23 1/2	
7	1.205	1.800	1.430	26	0.026
8	1.220	1.760	1.680	30 1/2	
9	0.540	0.400	0.675	26	
10	0.525	0.400	0.700	27	
11	0.525	0.350	0.780	30	

After the sinking of the casing, cleaning with water under pressure, the casing being sealed by means of a special head, until the water comes out clear. Concreting, reinforcing and loading test as under 1.-.

3) - Id. - , same as under 1), but setting up, before concreting, of 2 pipes for grouting the bottom of the pile immediately after completion of concrete operations.

4) - Id. - , same as under 3), but grouting 3 days after concreting (grout pipes were coated especially to render extraction possible).

5) Three piles, down to a depth of only 6 m below ground level, so as to penetrate at least 4 m into the gravels. Two of these piles to be grouted as indicated under 3) immediately after concreting; the third pile to be grouted 3 days later. All 3 piles reinforced along their whole length, in order to allow them to be pulled out after the tests.

The purpose of these tests was as follows:

a) to determine, when dealing with slightly clayey soils, the possible influence of sedimentation of fine grains at the bottom of the casing, and the influence of a preliminary washing out of the bore hole and chiefly of the washing of the subsoil at the base of the pile.

b) to determine the influence of grouting on the soil at the bottom of the pile. On account of the cost, piles of group 3) would have been more economical than those of group 1).

c) to determine whether it is advisable to grout the bearing soil before or after the concrete has set.

For further information, the outside diameter of the casing's shoe was 0.33 m (for a pile diameter of 0.30 m), the mix was 300 kg of portland cement per m^3 of concrete, and the cement was of type 500 in accordance with Italian standards.

Compressive strength of the concrete on 20 cm cubes, after curing under water for 41 days, was:

75, 71 and 73 kg/cm^2, therefore appreciably lower than the strength anticipated for concrete of such a mix (120 kg/cm^2) (year 1942)

All the construction procedure (boring, water levels, volume of excavated material during boring, progress) was carefully noted, see Fig. 1: Boring report for pile No. 1, Fig. 2: the corresponding graph, and Fig. 3: Boring graph for pile No. 3.

The volume of material excavated by boring was measured in order to give a clear idea of the difference between the theoretical and the actual volume. The following table allows comparison:

It is to be noted that the volume of excavated material is greater than the theoretical volume for deep piles No. 1 to 8, when it is smaller for short piles, No. 9 to 11.

The influence of the topsoil layer is naturally felt more acutely by short piles than by long ones, but it should be noted that, for the former, casings have been driven without preliminary excavation down to the depth given in the following table:

Pile No.	Depth of penetration of casing (m)	Total depth m	Depth excavated m
9	4.00	6.35	2.30
10	4.20	6.16	1.96
11	3.80	6.17	2.37

The result is that the proportion of boring carried out with the shell is relatively small.

LOADING TESTS.

The tests on all piles were carried out by means of a hydraulic jack bearing against a load placed above the ground level. All test results are summarized in the following table. Because the available static load did not exceed 100 tons, all the piles with a capacity above this value were submitted to a pulsation test, by varying the load 10 times between 50 and 100 tons.

The graphs show the detailed results of these tests.

It can be noted immediately that the behavior of short piles is not very different from that of long ones. All 3 short piles took successfully the 100 ton load and the 10 pulsations, when pile No. 2 withstood the 100 tons with a rather pronounced settlement but did not bear the 10 pulsations. Piles No. 7 and 8 have a greater settlement.

Pile No. 10, which withstood successfully the 100 ton test and the pulsations, was pulled out. Its stem was broken or at least partially destroyed or of inferior quality. It is a notable fact that the loading test did not indicate this; only the modulus of elasticity is low, but it is smaller yet for piles no. 7 and 8.

It is noted by comparing the settlements under the 40 ton load that the smaller ones apply to piles where grouting at the bottom was done immediately after concreting (No. 5 and 6 for long piles, and No. 9 and 10 for short piles), and that they are smaller for the 5.00 m piles (No. 9 and 10) than the ones

P I L E No 1 — B o r i n g R e c o r d — Date:

Base level of measurements : ml ..... Beginning deepening tubes : hours .....
Depth of casing shoe 1° driving : ml ..... Depth of ground level : ml .....

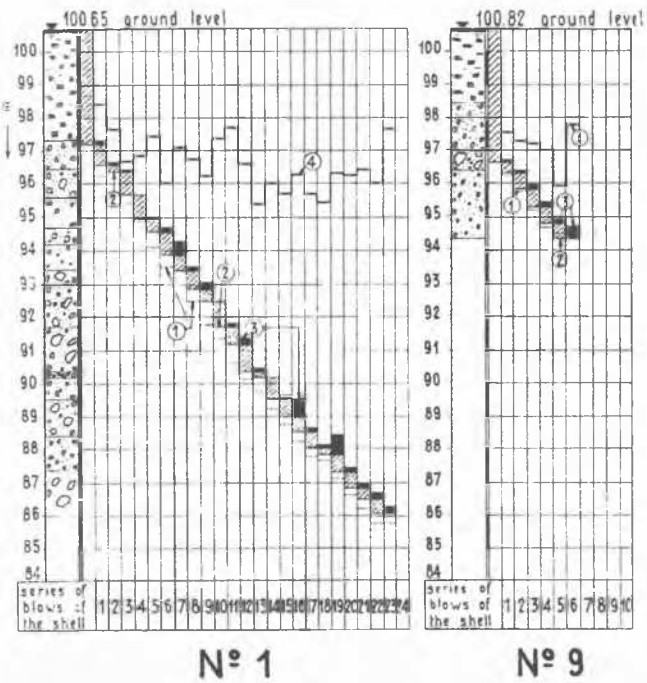
No of series	No of blows of the shell	Depth of water in the casing		Depth of the shoe of the shell after the last blow		Depth of casing shoe after driving		Depth of soil in casing tube found by the shell at the beginning of the successive series		Observations: a) type of soil b) colour of water c) various
		time	m	time	m	time	m	time	m	
1	60	8.40	2.27	8.42	3.45	8.45	2.00	8.58	3.35	Top soil black water
2	175	9.09	2.98	9.04	4.09	9.06	4.44	9.11	4.04	Gravel and sand" "
3	78	9.15	4.00	9.13	4.30	9.14	4.68	9.18	4.26	" " " " "
4	135	9.27	3.85	9.25	5.06	9.26	5.32	9.29	5.06	" , sand and pebbles x
5	210	9.36	3.17	9.33	5.73	9.34	5.79	9.41	5.69	" and sand" x
6	209	9.59	4.72	9.56	6.17	9.57	6.61	10.-	6.00	" " " x
7	150	10.05	3.57	10.03	6.86	10.04	7.21	10.12	6.47	" ,sand and pebbles x
8	162	10.19	3.93	10.16	7.31	10.16	7.71	10.20	7.20	" and sand x
9	146	10.25	4.47	10.22	7.85	10.23	8.26	10.40	7.64	" ,sand and pebbles x
10	158	10.44	3.32	10.43	8.25	10.43	9.01	10.45	8.25	" and sand x
11	58	10.48	2.97	10.46	9.05	10.47	9.42	10.51	8.93	" " " x
12	235	10.58	4.13	10.55	9.57	10.55 1/2	10.14	11.-	9.40	" ,sand and pebbles x
13	191	11.05 1/2	5.30	11.04	10.40	10.04e45"	10.63	11.24	10.30	" " " " x
14	236	11.30	4.65	11.27	10.54	11.28	10.98	11.31	10.54	Compact gravel x
15	247	11.37	5.00	11.36	11.23	11.36 1/2	11.55	11.38	11.09	" " and sand x
16	84	11.42	4.40	11.40	11.75	11.40 3/10	11.99	11.45	11.17	Gravel and sand x
17	90	11.50	5.00	11.48	12.23	11.49	12.49	11.50 1/2	12.12	Sand and pebbles x
18	134	11.56	5.20	11.55	12.68	11.55 1/2	12.95	11.56 1/2	12.59	" " " x
19	63	12.-	4.40	11.58 1/2	12.87	11.59	13.06	12.10	12.30	" " " x
20	138	12.15	4.42	12.13	13.43	12.14	13.67	12.15 1/2	13.32	Sand and gravel x
21	192	12.21 1/2	4.30	12.20	13.83	12.20 1/2	14.02	12.22	13.66	Sand, gravel and pebbles x
22	111	12.26 1/2	4.64	12.25	14.09	12.25 1/2	14.30	12.27	13.91	" " " " x
23	180	12.33	5.27	12.31	14.58	12.31 1/2	14.84	12.34	14.53	" " " " x
24		14.54	2.93	14.58	14.66			14.55	14.36	Clearing
										x = grey water
										All measurements are referred to the base level 100.65

Depth of water at end of boring . . . . . 5.27 time: 12.33
" " " after clearing 4.52
" " the bottom after cleaning. . . . . ml 14.66 " : 14.58
Height of depth of superior part of
the top of reinforcement . . . . . ml + 0.20 " : 15.06
The last casing tube over base level. . . . . ml 0.94
Cube of material extracted . . . . . (ml2.00 x ml 2.00 x ml 0.38 = mc 1.520

FIG. 1

Piles Nº 1 and 9

Deepening of the casing

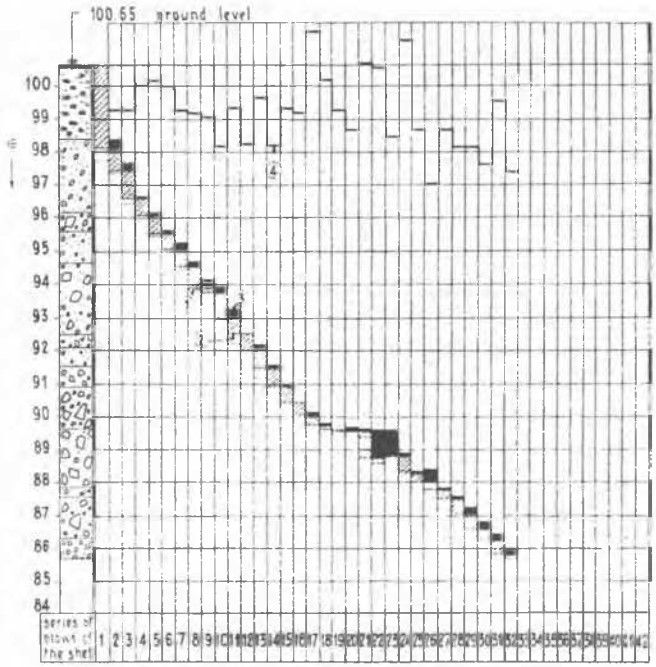


① Level of the casing shoe
② Level of the ground at the end of the series of blows
③ new level at the beginning of the successive series
④ ground water level in the casing

FIG. 2

Pile Nº 3

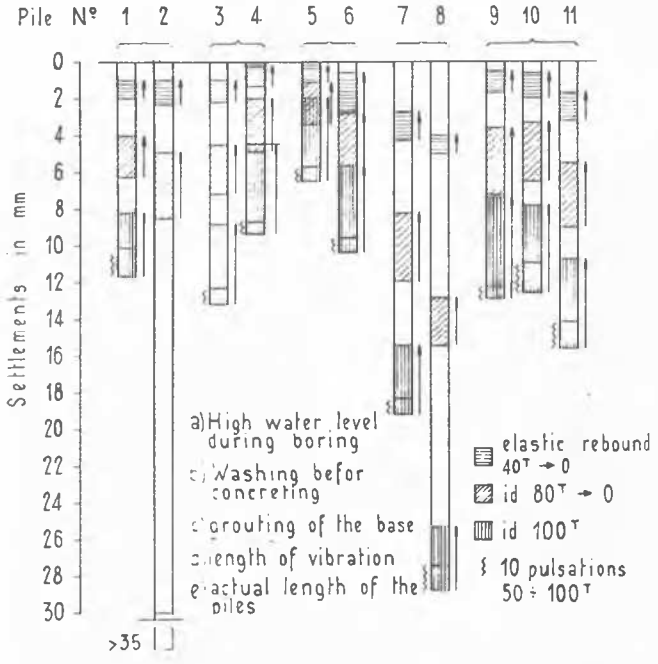
Deepening of the casing



① Level of the casing shoe
② Level of the ground at the end of the series of blows
③ new level at the beginning of the successive series
④ ground water level in the casing

FIG. 3

Load	Settlement in mm. of Pile No.9	Pile No.10	Pile No.11
10 t. instantaneous settlement	0.3	0.3	0.3
slow settlement	0	0.05	0.05
total settlement	0.3		
40 t. instantaneous settlement	1.55	1.75	2.80
slow settlement	0.25	0.20	0.40
total settlement	1.80	1.95	3.20
elastic settlement	1.30	1.15	1.5
80 t. instantaneous settlement	6.30	5.95	8.15
slow settlement	0.90	0.5	0.85
total settlement	7.20	6.25	9.00
elastic settlement	3.55	2.65	3.50
100 t. instantaneous settlement	10.1	9.3	12.20
slow settlement	2.15	1.75	1.95
total settlement	12.25	11.05	14.15
final elastic settlement after 10 pulsations	5.55	4.8	4.8
0 - 10 t. Modulus of elasticity - compression	kg/cm2 212'000	kg/cm2 178'000	kg/cm2 205'000
50 - 100 t. Modulus of elasticity, after 10 pulsations	235'000	240'000	224'000
40 t. (Compressive stress on)	57	57	57
80 " (core of 30 cm of)	114	114	114
100 " (diameter)	141	141	141
Compressive strength on cubes	75, 71 and 73		



a)	no			yes			no			no			no								
b)	no			yes			no			no			no								
c)	no			no			yes			3 days after concreting			immedia tel.			after 3 days				
d)																					
e)	13.61	4.45	13.95	4.45	13.15	4.35	13.90	4.35	13.15	4.20	13.16	4.20	13.15	4.30	13.05	3.95	5.45	4.35	5.31	4.35	5.27	4.30

FIG. 4

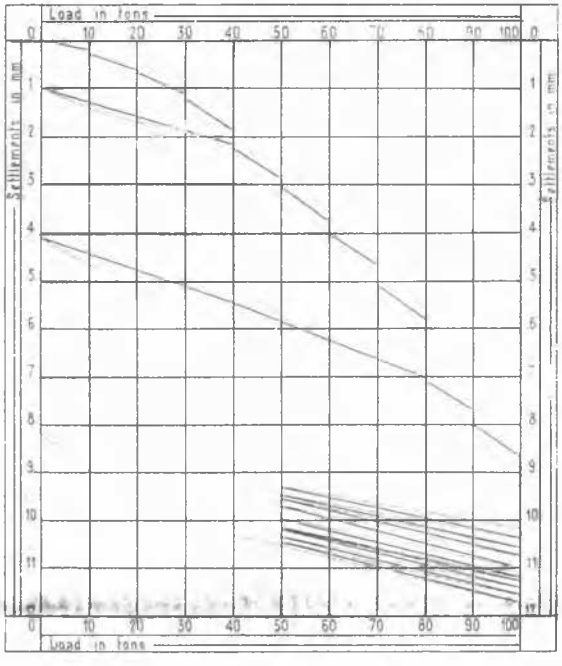


FIG. 5

obtained for long piles (No. 7 and 8).

The pile which gave the least settlement under 80 t. is also a pile grouted at the bottom (No. 5).

The piles grouted several days after concreting (No. 7, 8 and 11) showed worse results

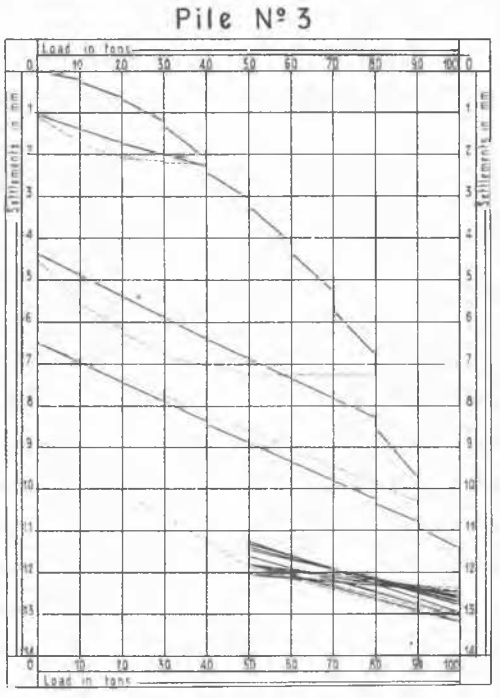


FIG. 6

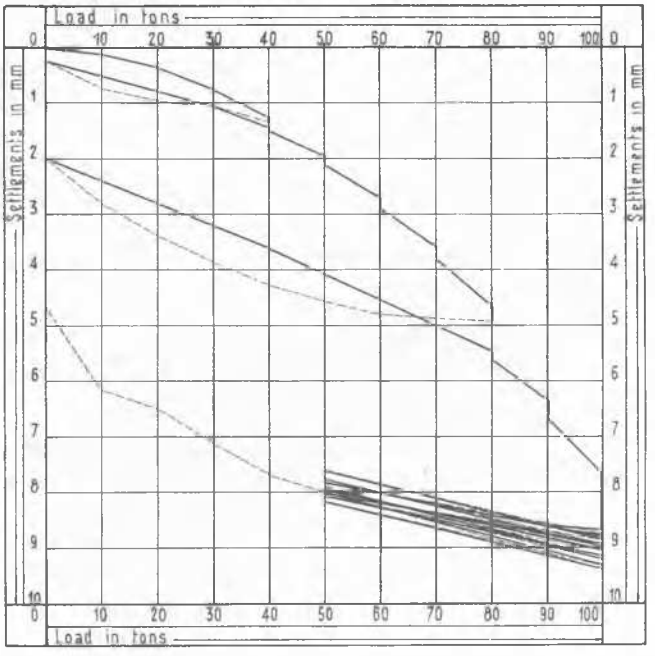


FIG. 8

under 40, 80 and 100 tons than the other types.

The settlement curves of piles No. 9 and 10 are very similar. (Cf. table pg. 91)

It can be concluded from these tests that the pile's concrete itself yielded and not the soil. The disintegrated zones of the extracted piles are due, on one hand to the bursting of the concrete out of the spiral reinforcement, and on the other hand, to localized crushing, and finally to the bending stresses during extraction and moving of the pile. This explains, in the writer's opinion, the ruptured zone of Pile No. 10 (see photograph).

The loads normally applied on a pile of 30 cm in diameter are usually limited to 40

Pile Nº 2

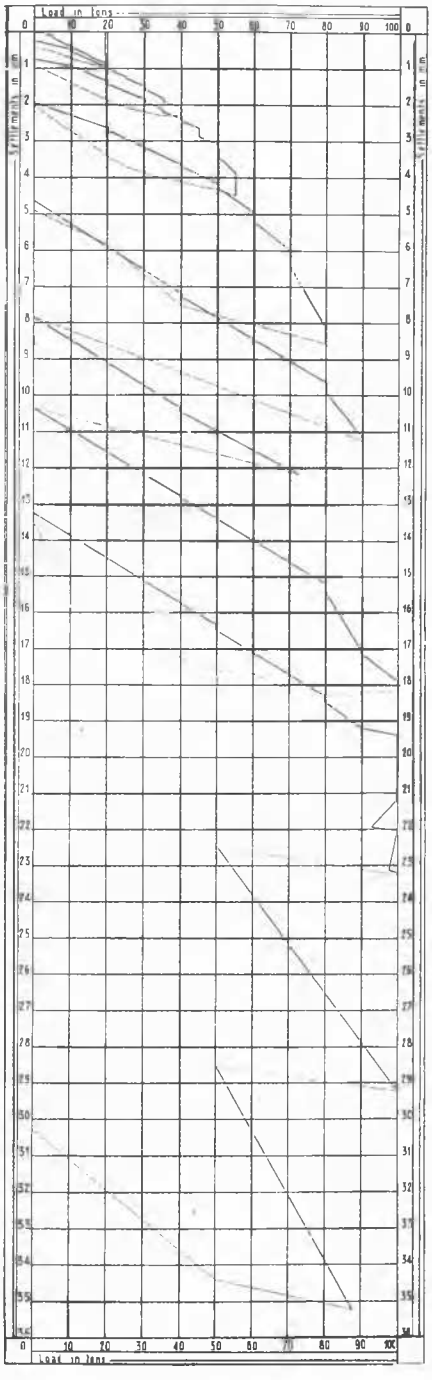


FIG. 7

tons, that is to a maximum stress in the concrete of about 57 kg/cm^2, but under 100 tons, the stress reaches 140 kg/cm^2.

It is therefore possible to state that the 3 short piles have withstood remarkably well the tests and that the rupture of pile No. 10 has been caused by the crushing of the concrete which occurred quite likely during the pulsation test between 50 and 100 tons (see graph). The concrete, mechanically destroyed, fell off later on account of the tension produced in the reinforcing bars while the pile was pulled out of the ground. A simple inspection of the results obtained for these 3 piles, under the 80 ton load, proves, as well as those under the 100 ton pulsations, that a pile of inferior quality could not have

Pile Nº 5

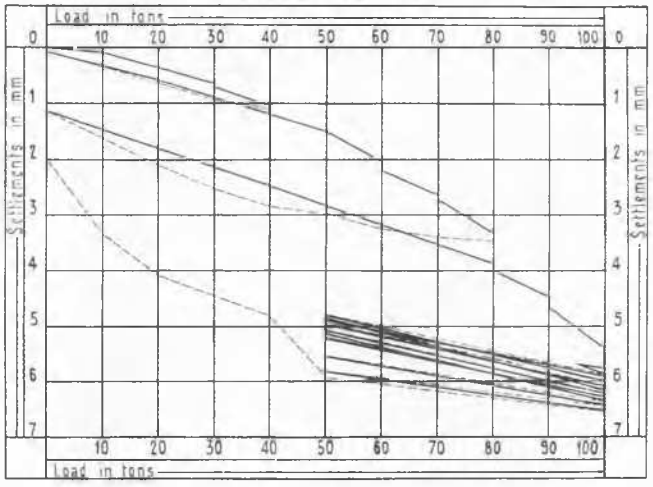


FIG. 9

Pile Nº 6

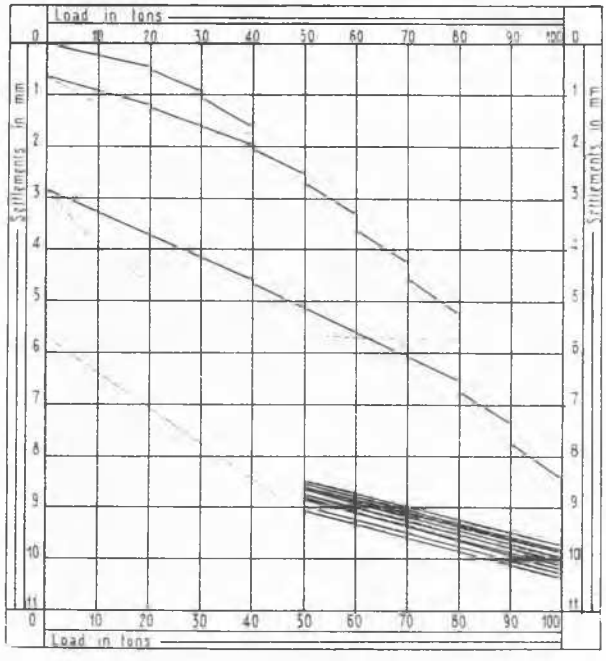


FIG. 10

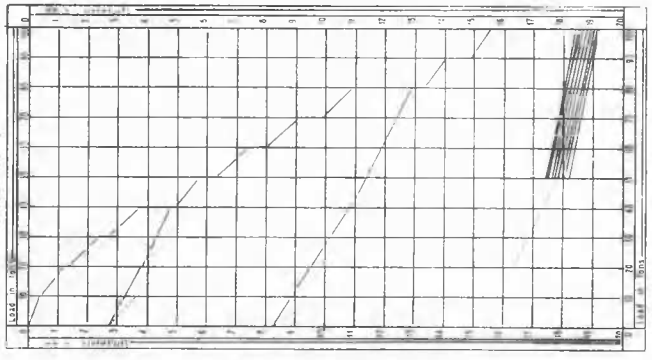


FIG. 11

withstood successfully such a test.

The same phenomenon explains why the bottom portion of the pile could not be extracted and why the reinforcing is visible at the low-

Pile Nº 8

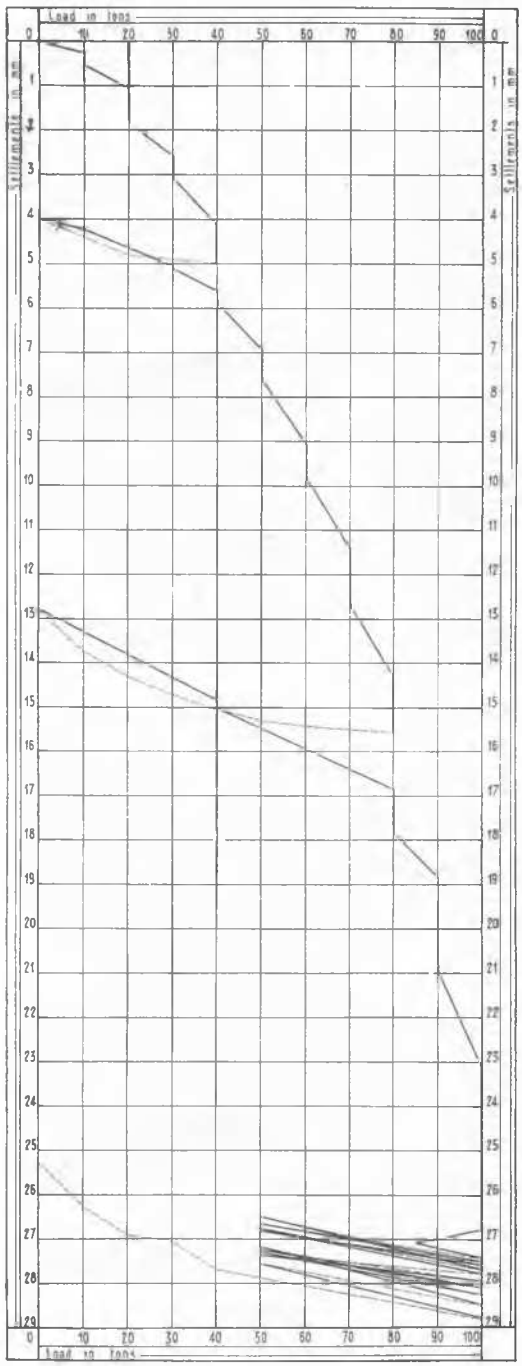


FIG.12

Pile Nº 9

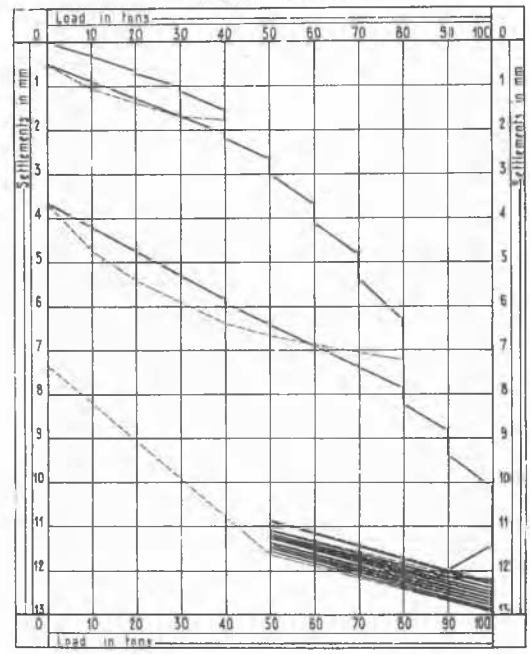


FIG. 13

Pile Nº 10

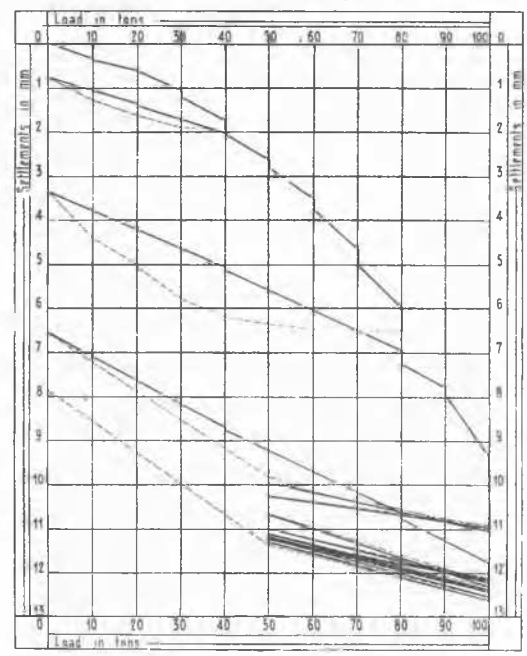


FIG. 14

est part of piles No. 9, 10 and 11.

With reference to the tests on piles of the "long" group, the following remarks can be made:

1) If the allowable permanent settlement had been set at 2 mm for the 40 t. load, only piles No. 7 and 8 would have been eliminated, but the "short" piles would have shown a very satisfactory behavior, and, from this standpoint, could perfectly well have been taken into consideration.

2) For double the load (80 t.), therefore a safety factor of 2, if the allowable permanent settlement had been set at 5 mm, only piles No. 7, 8 and 11 would have eliminated. All the others withstood fairly well 10 pulsations between 50 and 100 tons. piles No. 5 and 6 (grouted at the bottom) being particularly satisfactory in this respect.

3) The group 7 - 8 exhibited more appreciable settlements, notably exceeding those of short piles. It is strange that, having been constructed in the same fashion as pile No.11, they have acted less well. This fact is attributed to the boring which produced a decompression of the bearing zone of long piles, while this phenomenon was not felt so strongly for pile No. 11.

4) The improvement brought about by the washing of the bottom of the pile (No. 3 and 4), and the keeping up of an additional hydrostatic pressure during the boring operations, is very appreciable for pile No. 4, although not

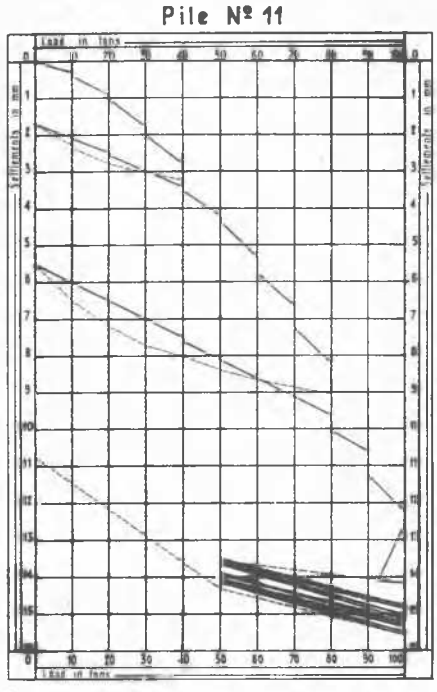


FIG. 15

Lower part of the stem of pile No. 11.

FIG. 17

Pile No. 11.

FIG. 16

General view of the extracted pile No. 9. Note the first enlargement.

FIG. 18

for No. 3, which is nevertheless much better than No. 2.

5) The inspection of the modulus of elasticity, computed on the basis of the tests, shows that piles No. 4, 5 and 6, in particular, are not stressed along their whole length at the 10 ton load.

6) The piles for which occurred the biggest sand rise during the boring operations were those which behaved least successfully.

Other observations can easily be obtained by inspection of tables or graphs of tests.

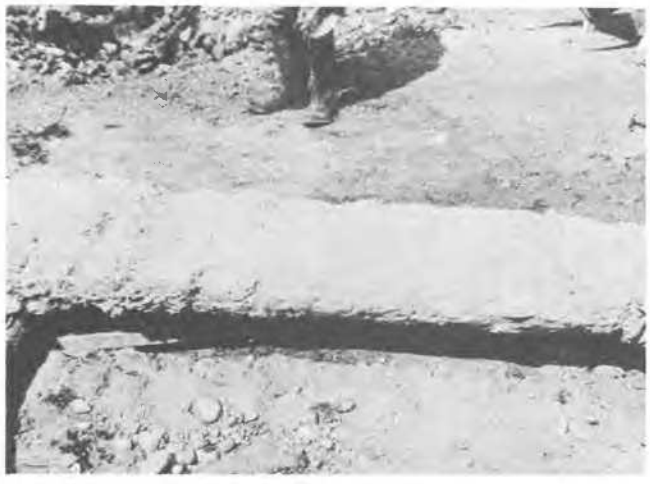
CONCLUSIONS.

1) The bearing capacity of a bored pile depends, to a large extent, upon the construction method, which must be adapted to the soil characteristics.

Pile No.	1	2	3	4	5	6	7	8	9	10	11
Sand rise (total) m	3.78	6.17	5.21	3.20	3.28	3.55	4.29	5.59	1.19	0.61	0.81
Max.settlement at 80 t.	6.35	8.55	7.25	4.9	3.45	5.7	11.95	15.55	7.2	6.5	9.0

Settlements in mm. for

Pile No.	Effective length meters	Depth of boring hole - meters	40 ton load: Settlement under load	40 ton load: Permanent settlement	40 ton load: Slow settlement under 40 tons	80 ton load: Total settlement	80 ton load: Permanent settlement	80 ton load: Slow settlement under 80 tons	100 ton load: Total settlement	100 ton load: Settlement under 100 tons	100 ton load: Permanent settlement	100 ton load: Settlement under 10 pulsations	100 tons Total settlement
1	13.81	14.66	2.05	1.0	0.15	6.35	4.1	0.55	11.7	8.65	8.2	1.5	10.15
2	13.95	14.85	2.3	0.95	0.2	8.55	4.9	0.35	>35.0	17.9	>30.0	part	-
3	14.15	15.00	2.2	1.05	0.15	7.25	4.5	0.5	13.2	11.4	8.8	0.75	12.35
4	13.90	14.83	1.35	0.2	0.1	4.9	2.0	0.3	9.4	7.7	4.7	0.65	8.7
5	13.15	14.08	1.1	0.1	0.06	3.45	1.1	0.15	6.5	5.35	2.0	0.7	5.75
6	13.16	14.01	2.7	0.65	0.15	5.7	2.85	0.45	10.4	8.45	5.7	0.7	9.65
7	13.15	14.15	4.3	2.7	0.5	12.95	8.25	1.1	19.2	15.55	15.4	0.9	18.3
8	13.05	14.30	5.0	4.0	0.8	15.55	12.8	1.25	28.8	23.1	25.3	1.4	27.4
9	5.45	6.30	1.75	0.5	0.55	7.2	3.65	0.9	12.9	10.1	7.35	2.8	12.2
10	5.31	6.16	1.75	0.75	0.2	6.5	3.35	0.5	12.6	9.3	7.85	3.3	10.9
11	5.27	6.17	3.2	1.7	0.4	9.0	5.3	0.85	15.6	12.2	10.7	1.9	14.15



Part of the stem of pile No. 9.

FIG.19

Base of extracted pile No. 9.

FIG.20

Extracted part of the base of pile No. 9.

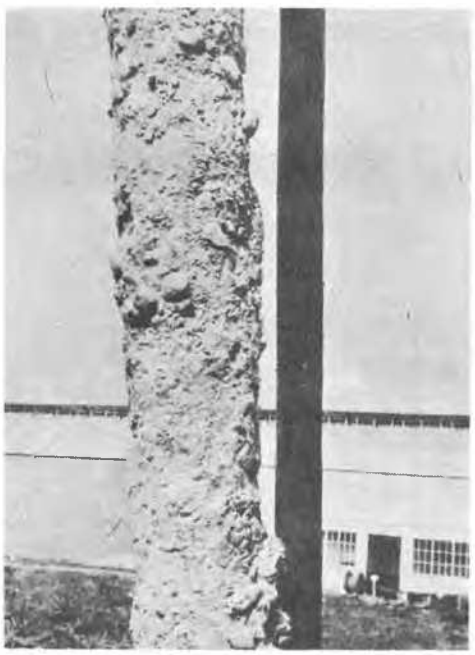
FIG. 21

2) The influence of base resistance is more important than that of side friction. Under a load of 40 tons, piles about 5.5 m long, grouted at the bottom, behave in the same manuer as piles 14 m long, without grouting.
3) In order to compare various types of piles, it is imperative to carry out several loading tests on several identical piles. Consequently, the conclusions drawn from a single test are unsafe and unreliable, especially if the pile was constructed as a test pile and was not chosen at random among several. Now, each time that testing apparatus, such as acoustic extensometers, are placed in the concrete of a pile, construction conditions are not identical to those of ordinary piles.
4) Grouting at the bottom of the piles is very effective if the piles are bearing upon or into a layer of gravels and sands.
5) Grouting performed after the concrete has set, is injurious.
6) If the pile is long, grouting at the bot-

Modulus of elasticity

kg/cm2

Pile No.	0 to 10 tons	0 to 40 tons reloading	0 to 80 tons reloading	50 to 100 tons - last loading
1	324'000	520'000	430'000	650'000
2	1'300'000	283'000 (for 3d.loading)	-	-
3	330'000	490'000	338'000 (not including slow settlement)	580'000
4	1'620'000	500'000	356'000	600'000
5	1'520'000	535'000	450'000	1'080'000
6	770'000	438'000	315'000	570'000
7	38'500	27'500	26'500 (not including slow settlement)	765'000
8	62'000	33'000	61'000 (not including slow settlement)	61'000
9	212'000	150'000	126'000	235'000
10	178'000	116'000	64'000	240'000
11	205'000	145'000	120'000	224'000



Stem of the pile No. 10. Note the perfect adhereuce with gravel and sand of the ground.

FIG. 22

General view of the extracted pile No. 10, with the rupture of the concrete under compression.

FIG. 23

tom is useless, as the pile is not stressed on its entire length (piles No. 5 and 6). This condition arises because a widening takes place, almost always, under a cohesive layer or within the capillary zone, In this case, strain measurements taken with extensometers, should be interpreted with caution, and are valid only when the pile is subsequently pulled out to obtain actual cross-section areas 2).

7) The customarily available methods of analysis do not allow to interpret the results obtained in the course of the above tests.

8) The pulsation test gives very interesting information with respect to the quality of the pile if it is not possible, for a test, to reach a clean rupture of either pile or ground.

9) It is to be hoped that other similar tests will be undertaken in typical soils, sands and clays, in order to furnish new data for

Base of the extracted pile No. 10.

FIG. 24

Base of the extracted pile No. 11. Note on the left the grouting pipe.

FIG. 25

the design of pile foundations.

REFERENCES.

1) a - Santarella "Tecnica delle fondazioni", Ed. Hoepli, Milan.
b - Catalogue "Pieux Rodio", de la société SEC, Paris
c - Note sur les travaux de réparation du Pont du Portail sur la Seille. Wahl et Dondin, 1937. Annales de la voirie, mai.
d - "La Gare Maritime d'Oran" revue "Chantiers" janv. mars 1936.
e - Electrification de la ligne du Mans. M. Laborde Science et Industrie - "Electricité" juin 1937
f - Parois de pieux jointifs, Travaux déc. 1941.
2) "Etude expérimentale de la force portante d'un pieu foré" H. Cambefort ("Travaux", juillet/août 1947).

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VII a 10

PERFORMANCE OF PILE FOUNDATIONS OF NAVIGATION LOCKS AND DAMS ON THE UPPER MISSISSIPPI RIVER

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SUMMARY

From 8 to 12 years have elapsed since completion of the navigation locks and dams on the upper Mississippi River, thereby affording an appreciable period for observing movement of those structures founded on piles. Records of such observations are presented for selected representative locks and dams, and comparisons are made with the results of pile driving and load tests made during construction.

Description of Project. The Congress of the United States, by an act approved by the President on July 3, 1930, authorized the improvement by the Corps of Engineers, United States Army, of some 650 miles of the upper Mississippi River to provide a navigable channel with minimum depth of 9 feet and width of 300 feet. The improvement was accomplished by construction of 23 locks and dams in addition to 3 previously completed. The last was placed in operation in 1940.

The locks are 110 ft. wide by 600 ft. long with provision for a future auxiliary lock at most projects. At Alton the auxiliary lock, 110 ft. by 360 ft., was provided initially. Most of the dams consist of various combinations of tainter and roller gates separated by concrete piers. Sills are at approximately river bed level.

Of the 26 locks and dams, 22 are founded in whole or in part on piles. The maximum lift for these 22 locks ranges from 5.5 to about 25 ft. (See Fig. 1)

Foundations. With few exceptions the pile foundations are in deep deposits of sand, vary-