The same pile was then screwed down to a depth of 50 feet below ground level, and the bottom of the tube plugged with concrete. Load was then added gradually up to a total of 110 tons, when a timber under the platform crushed, the settlement being $4\frac{1}{2}$ inches. On removal of the load the pile rose 0.24 inch. The load was then re-applied until at 130 tons a total settlement of 8.35 inches was observed. The test was then abandoned.

Pile Loading Tests. Loading tests which have been carried out in Shanghai from time to time, mostly on isolated piles, have shown the failure skin friction stress to vary from 400 pounds per square foot to 500 pounds per square foot, or more, for round tapered Oregon pine piles of from 40 to 60 feet long, though higher figures have been obtained from other types of pile. The lowest results so far obtained by the P.W.D. for piles of fair length are given in Fig. 5. A group of four piles was loaded with broken stone, contained in a timber box supported on steel beams, carried by four screw-jacks, one on the head of each pile. By means of these jacks it was possible to counteract the unequal settlement of the piles and keep the box vertical and clear of the guard trestles as the piles settled.

Test on a Composite Raymond Pile. This test was carried out by Messrs. The Hongkong Excavation, Pile Driving, and Construction Co., Ltd. and observed by the P.W.D. The results are given in Fig. 6. On three consecutive days during the test, the sand became soaked owing to heavy rainfall. The extra load due to this cause, has not been taken into consideration in the loading diagram.

Svagr Pile Test. This composite pile was driven and tested by Messrs. The Svagr Bros. Foundation Co., the authors witnessing the driving and checking the observations. The results are given in Fig. 7 together with a detail of the pile. The Svagr patent composite pile presents a novel principle in that a timber pile is driven down within a previously driven steel tube by means of a follower consisting of another steel tube of smaller diameter, until the butt of the timber pile is nearly down to the level of the base of the larger tube. Concrete is then poured down the larger tube and rammed by means of the inner tube until it forms a "bulb" enveloping the butt of the timber pile. Reinforcement is then placed in the larger tube and concrete is alternately poured and rammed, the ramming being carried out by means of the inner tube, both tubes being simultaneously and gradually withdrawn. The load was gradually applied over a period of 8 weeks. The maximum load reached was 400,000 pounds corresponding to a skin friction stress of approximately 600 pounds per square foot, at which load the test was terminated before the pile had come to rest. The pile head rose about 1/5 of an inch on removal of the load.

Takechi Pile Test. A "Takechi" pile 15 feet long of reinforced concrete, see Fig. 8, was driven and loaded under the authors' observation, by Mr. Shojiro Takechi, civil engineer of Osaka. 30 cubic feet of broken stone were forced into the ground surrounding the pile, by the pile itself, during driving. (The Takechi pile has been successfully used in the foundation of a large cotton mill in Shanghai.) The pile was loaded up to 75,400 pounds (consisting of a timber box filled with loose broken stone) not including the weight of the pile. The settlement remained constant at 0.297 inch for 10 days under this load. The load was then removed.

Additional Information. Further notes on Foundations on Shanghai Soil are contained in a paper by Mr. S. E. Faber read before the Local Association of the Institution of Civil Engineers in 1932.

Acknowledgements. The authors are indebted to the Commissioner of Public Works, Shanghai Municipal Council, for permission to publish the records and data contained herein, and to members of the staff for their assistance in preparing tables and curves.

No. I-5 LOAD TEST ON A WOOD PILE DRIVEN INTO THE GROUND

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In the autumn of 1929, a foundation layer was dumped at the mouth of the River Wien and the Danube Canal on a then existing stretch of land and secured on the water side by a pile work of larch-wood and banked up with a quarry stone pavement embedded in mortar. The former consists of a 180 m wall of round piles 4.5 m to 5 m in length placed side by side and earth anchorages spaced 5 m apart. The piles are about 30 cm in diameter and provided with steel shoes. Pile driving was done by means of a motor-driven rammer with a 600 kg drop hammer, the mean drop being about 3 metres. During these operations, a pile load test was carried out for scientific interest. Its arrangement will be seen from Fig. 1. One group of the eight piles 1 to 4 and 6 to 9 was fixed together by means of band steel loops along an I-steel girder, thus forming the tension resisting abutment for the pressure-die of a hydraulic press, the pressure cylinder of which exercised the pressure P acting downward on pile 5 situated in the centre. The intensity of the pressure could be determined from the readings on the pressure gauge. The equally strong counter-force - P acted in an extracting sense (mean lift h_m) on the first mentioned group of piles.

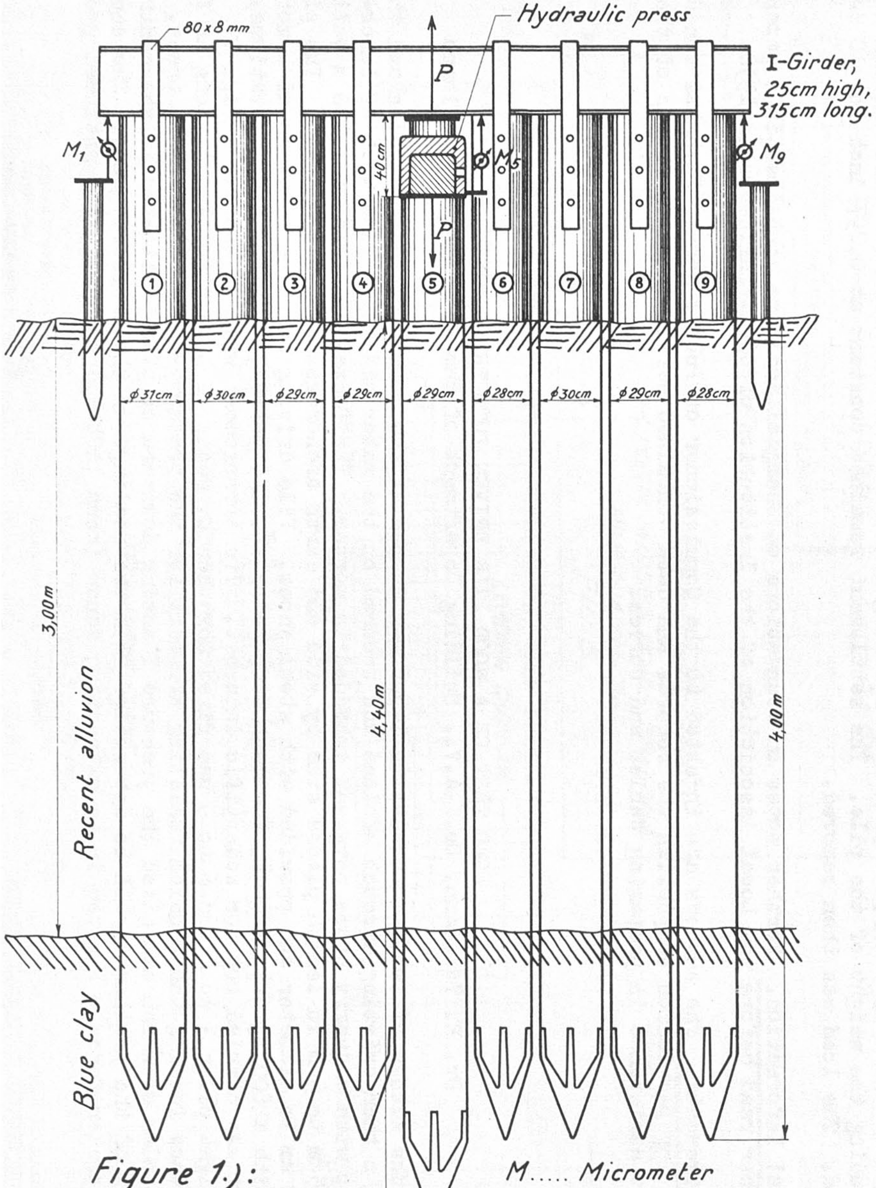

Figure 1.):
Arrangement of Test.

Nr	Loads and stresses of Piles: P $(t)^{x)}$	$\tau_{z,m}$ (t/m²)	$\tau_{d,m}$ (t/m²)	$\sigma_{z,m}$ (kg/cm²)	$\sigma_{d,m}$ (kg/cm²)	Lifting of Piles 1-4a. 6-9: h_1 (mm)	h_9 (mm)	h'_5 (mm)	h_m (mm)	Penetration of Pile 5: e'_5	$e_5 = e'_5 - h'_5$	Time (min.)	Remarks
1	20,46	0,79	5,10	3,78	31,0	0,00	0,00	0,00	0,00	0,00	0,00	0	——
2	"	"	"	"	"	1,20	2,00	1,68	1,57	7,50	5,82	12	Time approximatively observed.
3	34,10	1,31	8,50	6,30	51,7	1,63	3,50	2,53	2,50	12,50	9,97	25	
4	40,92	1,58	10,20	7,55	62,0	2,90	7,80	5,25	5,19	29,30	24,05	37	
5	"	"	"	"	"	3,43	12,20	7,64	7,52	32,50	24,86	45	——
6	"	"	"	"	"	3,52	14,30	8,69	8,55	36,50	27,81	51	——
7	34,10	1,31	8,50	6,30	51,7	3,52	14,30	9,02	8,80	36,50	27,81	80	Unloading
8	40,92	1,58	10,20	7,55	62,0	3,82	21,55	12,86	12,50	56,50	43,64	90	——
9	42,28	1,63	10,55	7,82	64,1	3,96	28,30	16,38	15,89	76,50	60,12	97	——
10	"	"	"	"	"	4,16	34,30	19,53	18,94	96,50	76,97	105	——
11	"	"	"	"	"	4,32	41,63	23,36	22,51	106,50	83,14	115	——
12	"	"	"	"	"	4,40	45,42	24,52	24,50	116,50	91,98	120	——

x) 1 t = 1000 kg.

Figure 2.): Table of Results.

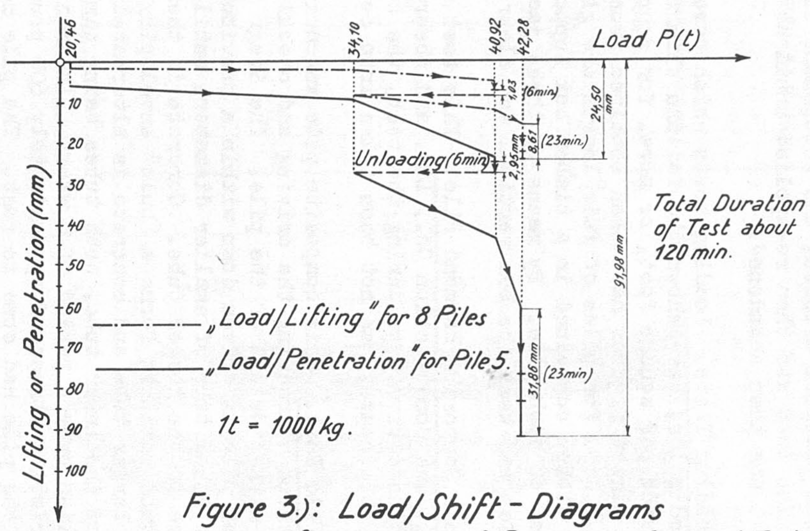

Figure 3.): Load/Shift - Diagrams
for a series of Piles in a Statical Test

The value h_m could be computed from observations registered by the micrometer at piles 1 and 9 (single lifts h_1 and h_9 respectively). The penetration of the actual test pile 5 was found to be $e_5 = \grave{e}_5 - h_5$, this being the difference between the micrometer observations $\grave{e}_5$ at this pile and the lift h_5, deducted from h_1 and h_9 of the above mentioned group of piles at place 5. All these vertical pile shiftings are represented in the table of Fig. 2, in connection with the pressure forces gradually increased in the course of two hours from 20.46 to 42.28 t. There will also be found the average frictional tensions τ_m caused by the pressure forces P on the side-area F of the piles (if the resistances at the pile shoes are ignored) and the longitudinal tensions σ (pull z, pressure d) related to the cross-sections f of the piles. For pile-group 1 to 4 and 6 to 9 $F_z = 259.500$ cm² = 25.95 m² and f_z = 5415 cm², while for pile 5 the analogous values are $F_d = 40100$ cm² = 4.01 m² and $f_d = 660$ cm². Then there is $\tau_{z_1 m} = \frac{P}{F_z}$, $\tau_{d_1 m} = \frac{P}{F_d}$, $\sigma_{z_1 m} = \frac{P}{f_z}$, $\sigma_{d_1 m} = \frac{P}{f_d}$. The above mentioned table also shows the progress of the test. The load was sometimes allowed to act unchanged for a longer period, and once the load was partially removed in order to ascertain a possible elastic behaviour. The piles of the aforesaid group are driven into the ground at a depth of four metres, whereas pile 5 is 4.4 metres deep. As several trial bores have shown, the piles are, for the upper three metres of their depth, in young alluvial soil, while their lower ends rest about 1.2 m deep in blue clay of the Pontium period. The first kind of soil was pedologically examined and, with its broken stone and sand contents of 46%, 52% Mo and 2% silt and clay, recognized to be an orange-yellow sand containing dust and clay, rich in deposits, dating from the latest Flysch-alluvions of the River Wien. The natural humidity of this soil was ascertained at $w = 29\%$, its degree of plasticity $B \sim 4\frac{1}{2}$; its shrinkage is $s = 2\%$; its cone flowing test specimen fell to pieces after having been immersed in water for eight days. This kind of soil alternates in layers of fine gray sand of greater plasticity ($B \sim 9$), with considerable clay contents.

The relations h_m/P and e_5/P described in the table (Fig. 2) are graphically represented as load/shift diagrams in Fig. 3. It will be seen that the pressure resistance of such a pile in this particular soil is exhausted when the yield point of the latter is reached, corresponding to a pile compression of 64.1 kg per square centimetre of the cross-section. The resistance of the pile against extraction appears to be limited at a maximum lifting power of 1.63 t/m² in relation to the pile side area. The soil showed an entirely plastic behaviour during this test.

No. I-6 PILE LOADING TESTS, BONNET CARRE FLOODWAY

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The projects, loading tests for which are covered in the following report, are all located in the Bonnet Carre Floodway, situated along the east bank of the Mississippi River about twenty-five miles above New Orleans, Louisiana. Sheet A-1.

Included in the structures for which pile test data were gathered and which will be discussed in order in the following paragraphs are: a. The Spillway Weir Structure; b. The Airline Highway Crossing; c. The L. & A. Railroad Crossing, and d. The I. C. Railroad Crossing.

a. The Spillway weir is a reinforced concrete structure, 29 feet wide, 7,7000 feet long divided into 22 foot bays and supported on untreated timber piles; with a fore apron 44 feet wide, 2½ feet thick and rear apron 20 to 30 feet wide, both bearing directly on the subsoil. Sheets A-2, 3.

Material encountered in the borings is typical of recent alluvium adjacent to stream banks; stiff clays with numerous lenses or pockets of packed sand which show very little, if any, continuity. Sheet A-5.

Piles under the main portion of the weir are untreated timber, 65 to 70 feet long, cut off to elevation +6 feet M.G.L., with natural ground surface at approximately +12 feet M.G.L., and spaced 3 feet centers along the length and 4 to 5 feet on the width. Sheet A-4.

Pile driving and loading tests at the weir site were of both round timber piles and Raymond concrete piles. Wood piles of untreated long leaf yellow pine, 6-8 inch tip, 15-16 inch butt, with penetrations of 65 to 70 feet below the bottom of a 5 foot excavation, and driven with a No. 1 Vulcan hammer, 5000 # weight, 3 foot drop.

Concrete piles, precast reinforced piles, 8 inch tip, 22 inch butt, with penetrations of 30 to 35 feet below surface of a 5 foot excavation, were driven with the same Vulcan hammer as above. Sheet A-6.

Pile driving records were kept on all piles, and bearing capacities computed by the Engineering News formula indicated safe loads of 11 to 24 tons for timber, and 13 to 28 tons for concrete piles. Loading tests, concrete blocks and pig iron on a loading platform, conducted on these same piles shortly after driving, show maximum loads supported with settlement under ½ inch of from 25 to 50 tons for timber and 50 to 100 tons for concrete. Sheet A-7a, b. These test loads were from 2 to 3 times those indicated as safe by the Engineering News formula.

A marked difference in supporting power (skin friction) is noted between timber and concrete piles; timber varying from 250 to 500#/ sq ft, and concrete from 600 to 1200#/sq ft. This difference is probably due to the fact that for a skinned timber pile friction is between wood and soil; whereas for concrete the shear surface is a minute distance from the roughened pile surface, the friction being of soil against soil.