GROUP PILE LOADS IN PLASTIC SOILS

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SYNOPSIS

THIS PAPER presents the results of experimental tests made to determine the relationship between the load carried by a single pile and that carried by a group of piles driven into plastic soils. It also presents the load distribution, with respect to depth, and the load transfer from the piles to the surrounding soil and describes the instrumentation and the method of loading and establishes a relationship between the capacity, at failure, of a single pile and the pile when acting as a member of a group of piles.

This series of tests was prompted by a desire to obtain experimental data which could be used as a basis for judging the adequacy of existing pile-group formulas and design assumptions, and to correlate, if possible, soil constants derived from laboratory analysis of soil samples with soil stresses calculated from field-test data.

The data obtained from these tests indicate: (1) that the single pile carried its load by friction between the pile and the surrounding soil with the greater part of the pile load transferred from the pile to the soil in the upper half of the pile and with relatively high shear values existing in the soil for a short length of the pile; (2) that the load distribution to the piles in the group was fairly uniform, each pile, including the center pile, carrying a share of the load; (3) that the effect of the pile spacing was reflected by the drop in the shear values between the corner piles, side-center piles and the center pile of the group; and (4) that the load on the individual piles within the group was transferred to the soil by shear, with the maximum values occurring at slightly higher levels than for the single pile and these maximum values occurred over a much longer length of the pile.

• THE RELATIONSHIP between the bearing capacity of a group of piles as compared to that of a single pile when driven into plastic soils has been the subject of much conjecture. Several empirical formulas have been proposed and some have been adopted and are in use today. Until recently, however, very little experimental data or tests have been available to substantiate these formulas.

The American Association of Railroads very successfully carried out a series of loading tests for the Missouri Pacific Railroad and the Corps of Engineers. These tests were under the direction of E. J. Ruble, structural research engineer of the American Association of Railroads, and the results obtained in those tests were substantiated by the tests reported herein. The tests were conducted by the Bridge Design Section of the Nebraska Department of Roads and Irrigation in coöperation with the Bureau of Public Roads. They were carried on during the summer of 1950, at the time of construction of a viaduct in Omaha, Nebraska. The contract provided for the use of shell-type, cast-in-place concrete piles; the contractor's selection of the shell as manufactured by the Union Metal and Manufacturing Company permitted the conduct of these tests.

The testing procedure, tentatively proposed, was to install SR-4 strain gages on the interior walls of each of seven 55-ft. piles prior to driving. To determine the load distribution, as between piles and also with respect to depth, these gages were to be installed at several levels throughout the full length of these piles.

Before attempting any field installations, laboratory tests were carried out to develop and check the installation techniques and verify the accuracy of the measurement of load-carrying capacities by the proposed methods.

The laboratory specimen (Fig. 1) was prepared using a 6-ft. section of one of the pile shells. Each end of the section was capped with concrete. Strain gages of the type to be used in the field tests were attached to the exterior wall surface of this specimen. It was placed in a large testing machine, and a compressive load of 40,000 pounds was applied. The shell stresses were computed from the strains measured while the specimen was loaded. These calculations indicated a satisfactory accuracy by the proposed procedures.

In the field installations it was necessary to install the gages on the interior-wall surfaces of the piles. Hand holes were required to permit access to the interior of the pile shell. To determine the effect of the hand holes, a 5-in. diameter hole was cut in the shell of the laboratory specimen (Fig. 1). The specimen was reloaded and the effect of the hand hole was noted in the stress pattern in the pile shell. The portion of the shell taken out for the hand hole was subsequently filled back and the seam was butt welded. Some stress diversion was noticeable; however, it was evident that this diversion could be minimized by the proper location of the hand holes with respect to the gage lines.

The instruments used in both laboratory and field tests consisted of a portable strain indicator, an SR-4 calibration unit, and a switch panel. The SR-4 gages were unidirectional, 1-in. long, Type A-11, as manufactured by the Baldwin Locomotive Works (Fig. 2).

From the results of these preliminary tests it was apparent that the testing program previously outlined could be carried on successfully in the field. Gage installations were completed in the seven 55-ft. piles. Each of the seven piles consisted of one constant diameter top section 12 in. in diameter and 25 ft. long and a tapered lower section varying from 12 in. in diameter to 8 in. in diameter and approximately 30 ft. long. The small end of the pile, at the time of driving, was closed with a forged steel shoe. All gage installations were made at the Department of Roads and Irrigation Testing Laboratory at Lincoln, Nebraska, and the piles were transported to the viaduct site by truck. The gages were installed at 11 levels, each 5 ft. apart, throughout the full length of these piles. At the top and bottom levels, and at the two levels adjacent to the top and bottom levels, four active gages were used. Two active gages were used at all other levels (Figs. 3 and 5). Two inactive (temperature-compensating) gages were installed at each level. Thermocouples were distributed in the seven piles so that the temperature at each level could be determined. Two bundles of lead wires from the gages to the measuring instruments were installed to keep the leads from each gage to the cable bundle as short as possible. Two steel messenger cables were mounted and tensioned inside the pile and the two lead-wire cables from the gages were fastened to these cables by taping and tying.

Variations in temperatures within the body of the several piles were recorded by installing thermocouples in the piles. Temperatures were recorded using a 12-point Tag Celectray recording potentiometer having a range from 0 to 300 F. This instrument was manufactured by the C. J. Tagliabue Corporation of Newark, N. J.

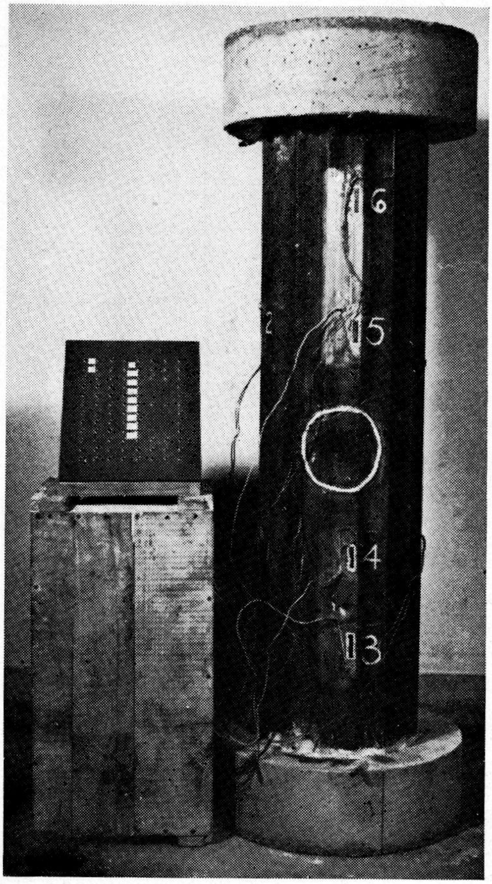


Figure 1. Preliminary test specimen showing gage installation in line with hand hole.

The footing design for the test-load group consisted of nine piles arranged in a square of three rows of three piles each. The piles were spaced 3 ft. 9 in. on centers.

Piles 1 thru 8 were 55 ft. in length while Pile 9 was 65 ft. in length. Piles 1 to 7 inclusive and Pile 9 had 7-gage shells while Pile 8 had a 9-gage shell thickness.

All piles were driven with a No. 1 Vulcan Steam Hammer delivering 15,000 ft.-lb. of energy per blow. Piles 1 thru 7 weighed 1,275 lb. each; Pile 8 weighed 1,070 lb. and Pile 9 weighed 1,620 lb. The driving cap weighed 650 lb. Piles 6, 7, 2, 8, 5, 4, and 3 were driven in that order. The pit for this test was excavated to Elevation 122.5 (about 6 ft. below

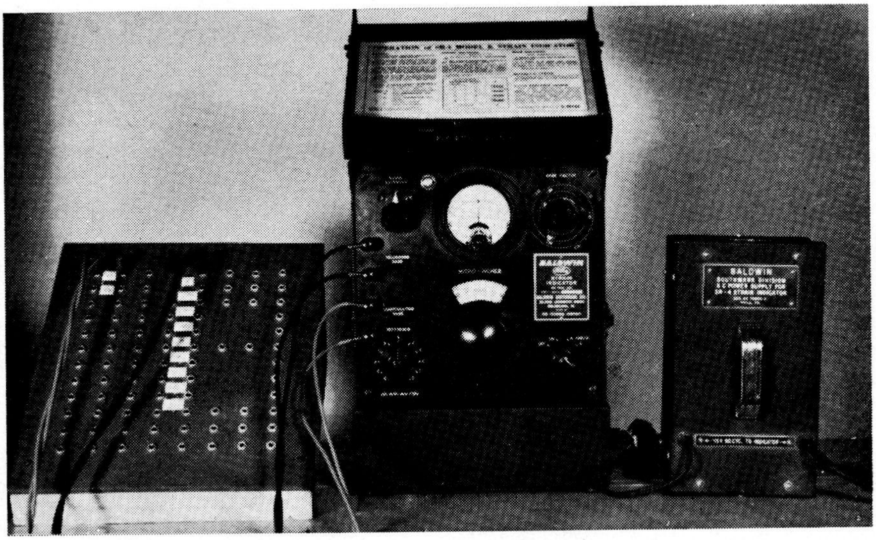


Figure 2. Instruments used in preliminary tests.

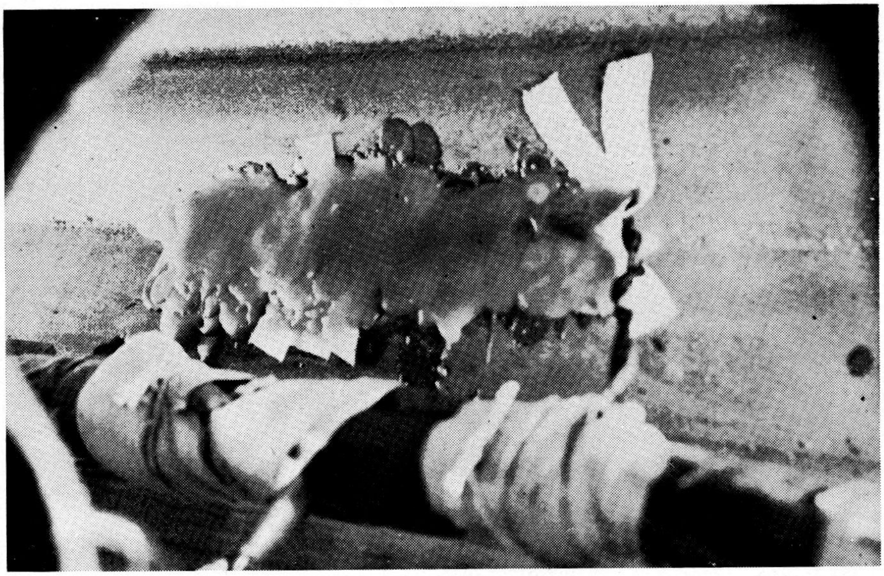


Figure 3. Waxed gage and lead wires.

the ground surface) prior to driving Piles 1 thru 8. A pilot hole 6 ft. deep was drilled from the bottom elevation of the pit and the piling were started in these pilot holes.

Pile 9 was driven as a test pile in the preliminary foundation exploration and was test loaded as a single pile to a maximum static load of 111.5 tons. The load-settlement curve indicated a yield point load of 80 tons. Since strain gages were not installed at the time of driving, it was necessary to install gages on the outside shell surface of Pile 9 and Pile 8 to determine the load transmitted to these piles when subjected to the group load test. These piles were filled with concrete to the lower level of the foundation pit.

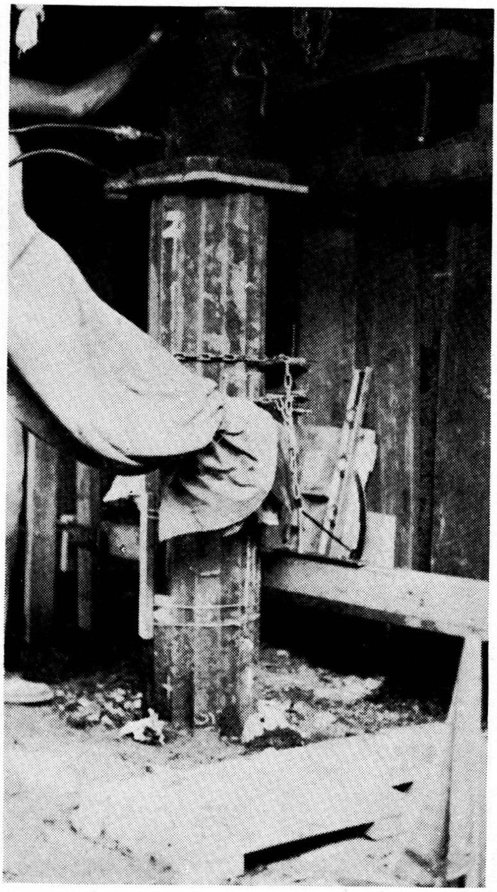


Figure 4. Method of measuring pile settlements and jack for loading, single-pile test.

The bearing values at final penetration for these piles, as computed by using the *Engineering News* Formula, were: Pile 1, 17.6 tons; Pile 2, 17.6 tons; Pile 3, 13.6 tons; Pile 4, 16.7 tons; Pile 5, 18.6 tons; Pile 6, 13.6 tons; Pile 7, 16.7 tons; Pile 8, 15.7 tons and Pile 9, 31.5 tons. The computed total bearing for the group: 162 tons.

SINGLE PILE TEST

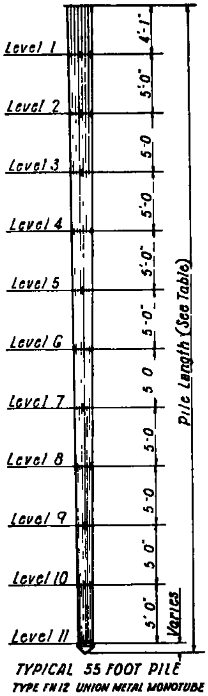
Pile 1 was driven in the southeast corner of the pit diagonally opposite Pile 9. It was the pile selected for loading as the single test pile. It was loaded by a hydraulic jack of 150-ton rated capacity, backed against a loading platform upon which was placed a dead load of 126 tons. This pile was loaded three times (Fig. 6). In the initial loading a maximum load of 79 tons was attained at which load the pile broke loose and settled approximately $1\frac{1}{2}$ in. The load was reduced to 64 tons and the pile came to rest. This 64-ton load was held for a period of 48 hr. during which time a further settlement of $\frac{1}{32}$ in. was noted. Upon release of the load a total permanent settlement of $1\frac{9}{32}$ in. was recorded.

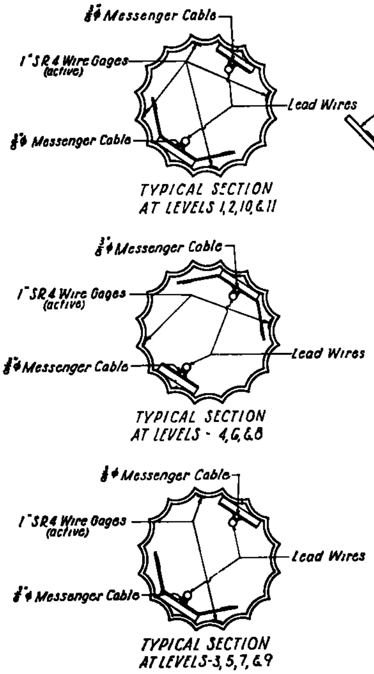
In the first reloading test the pile was loaded to a maximum of 70 tons for 48 hr. and unloaded. The maximum settlement was $\frac{1}{4}$ in. and the permanent settlement was $\frac{1}{64}$ in.

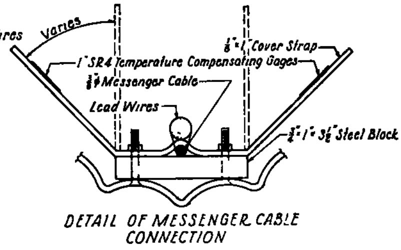
The second reloading was to a maximum load of 75 tons for 48 hr., after which time the pile was unloaded. In this instance the maximum observed settlement was $\frac{9}{32}$ in. and the permanent settlement was $\frac{1}{32}$ in.

The load settlement relationship for the loading of the single pile is shown in Figure 7. It is to be noted that the deviation of the load settlement curve from a straight line occurred at 70 tons.

From the values obtained from the gage readings, calculations were made to determine the load in the pile at each of the levels for the three applications of load (Fig. 8). It is to be noted that the gages of level one check very closely with the total load carried by the pile. This is the basis used in determining the loads in the several piles in the group load tests. Your attention is called to the shape of these load curves and particularly to the very small loads at the point of the pile. Only 2 tons of load were being carried at the point when the pile carried 64 tons, 2.7 tons at the 70-ton load and 3.0 tons at the 75-ton load.







PILE NO	LENGTH
1	55 - 7"
2	55 8¼"
3	55 - 6
4	55 - 5½"
5	55 6½"
6	55 6½"
7	55 - 6½"
8	55 - 1⅛"
9	65 - 1⅛"

Figure 5. Gage locations.

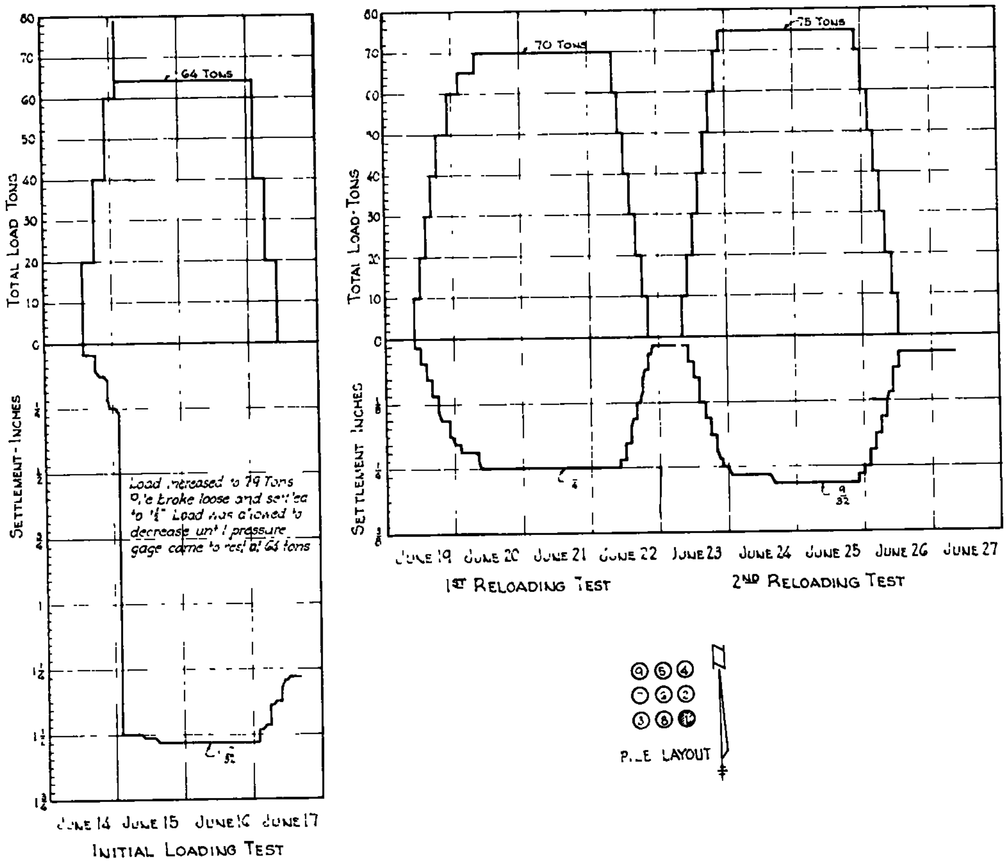


Figure 6. Load-settlement diagrams, single-pile test.

The difference in load between adjacent levels in the pile is equal to the load transferred to the soil in the length of pile between each pair of gage levels. From these successive differences the soil shear can be computed. These values are expressed in terms of lb. per sq. ft. and are shown in Figure 9 and Table 1. It is to be noted that the maximum shear value for the 64-ton load was 1,280 lb. per sq. ft.; for the 70-ton load it was 1,425 lb. per sq. ft., and for the 75-ton load: 1,675 lb. per sq. ft. These maximum values occur over a rather short length of pile and are in the tapered section of the pile, the first named stress occurring at 16 ft. above the point, the second at 26 ft. and the last named at 19 ft.

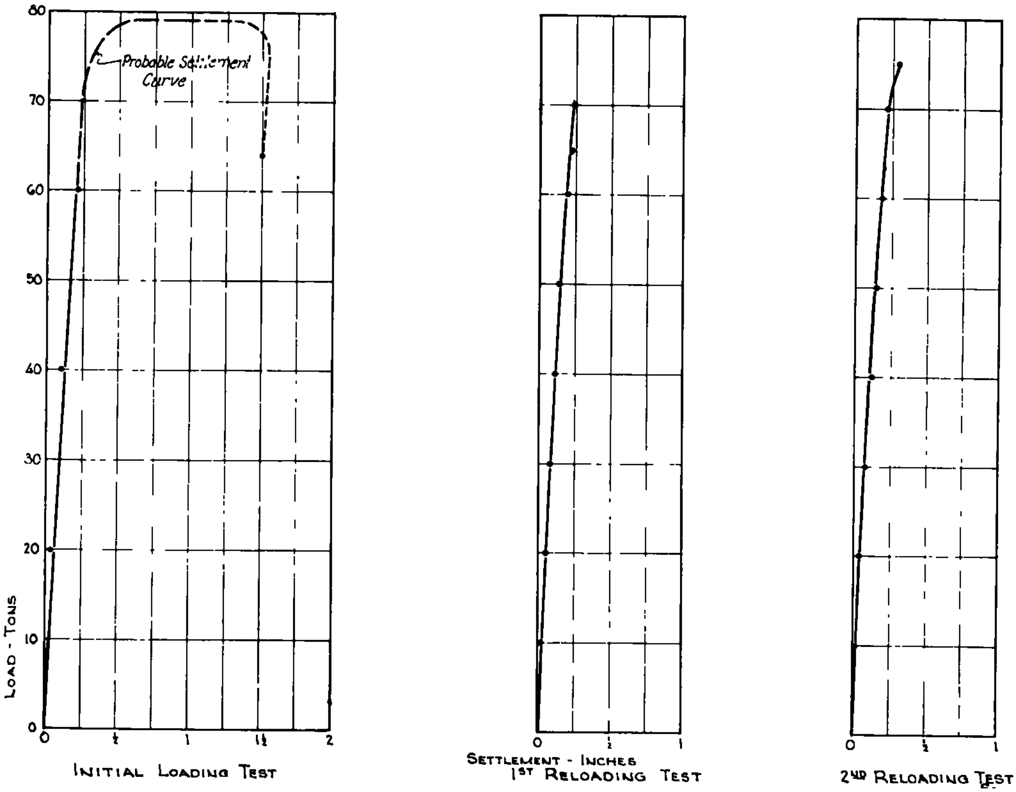


Figure 7. Load-settlement diagrams, single-pile test.

GROUP LOAD TEST

Following the completion of the single-pile test the remaining piles of the group were driven. A platform of structural steel was constructed on the pile group. It consisted of three tiers of beams, adequately cross braced, and spaced to provide a rigid cap.

Provision was made for obtaining the settlements of the several piling in the group by attaching $\frac{7}{8}$ in. vertical rods to each pile by welding. At the upper end of these rods was fastened a 6-in. rule graduated to $\frac{1}{16}$ in. Settlement readings were obtained by the use of an 18-in. dumpy surveyors' level, and a permanent bench mark located beyond the zone of influence of the pile group. The original increment of load was 254 tons (Fig. 10). This increment of load was held for 25 hr. The gross settlement for the first increment of load varied from $\frac{1}{32}$ in. for Pile 4 to $\frac{17}{64}$ in. for Pile 6. The second increment of 80 tons was allowed to remain in place for 42 hr. at which

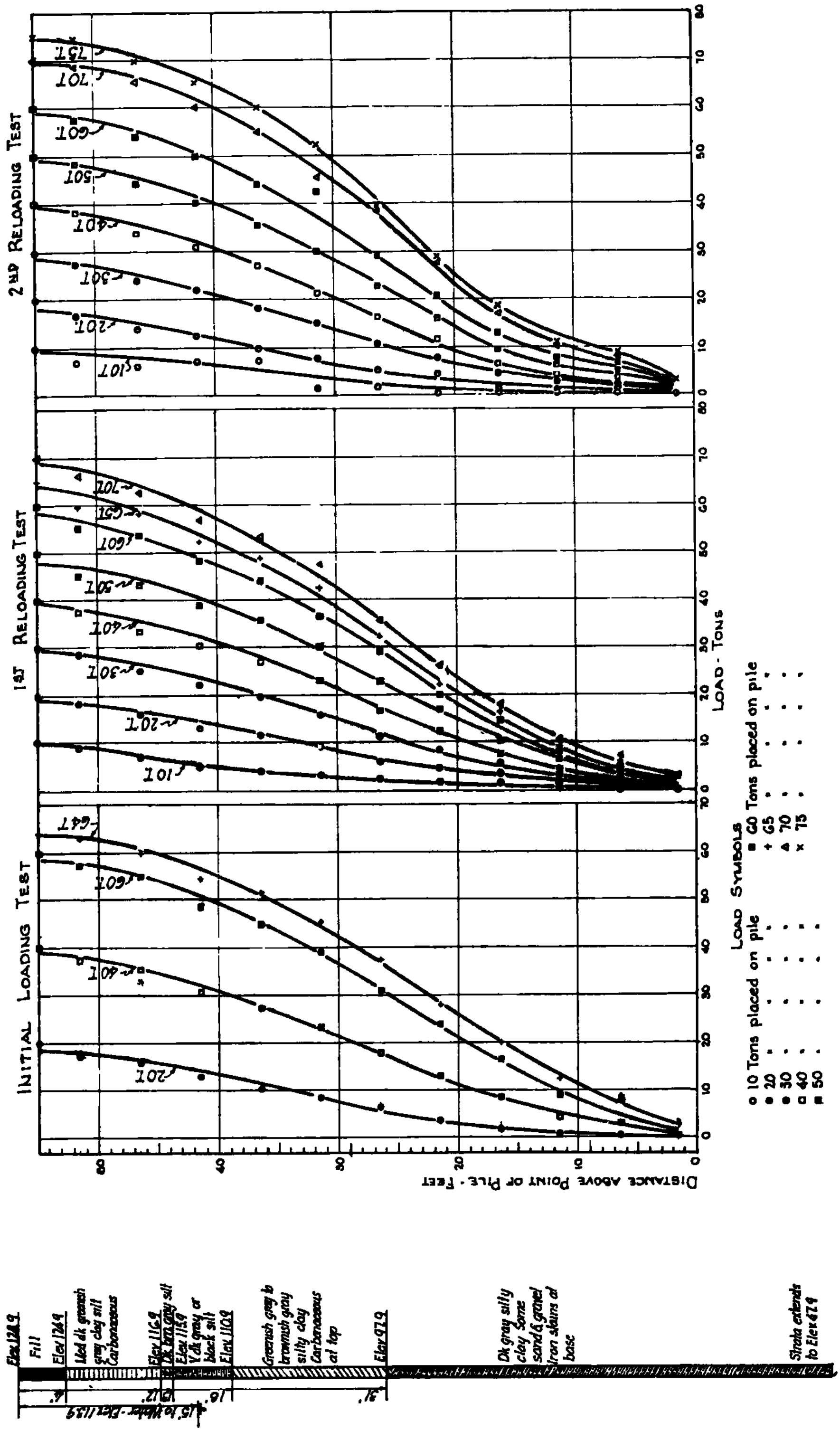


Figure 8. Loading reactions, single-pile test.

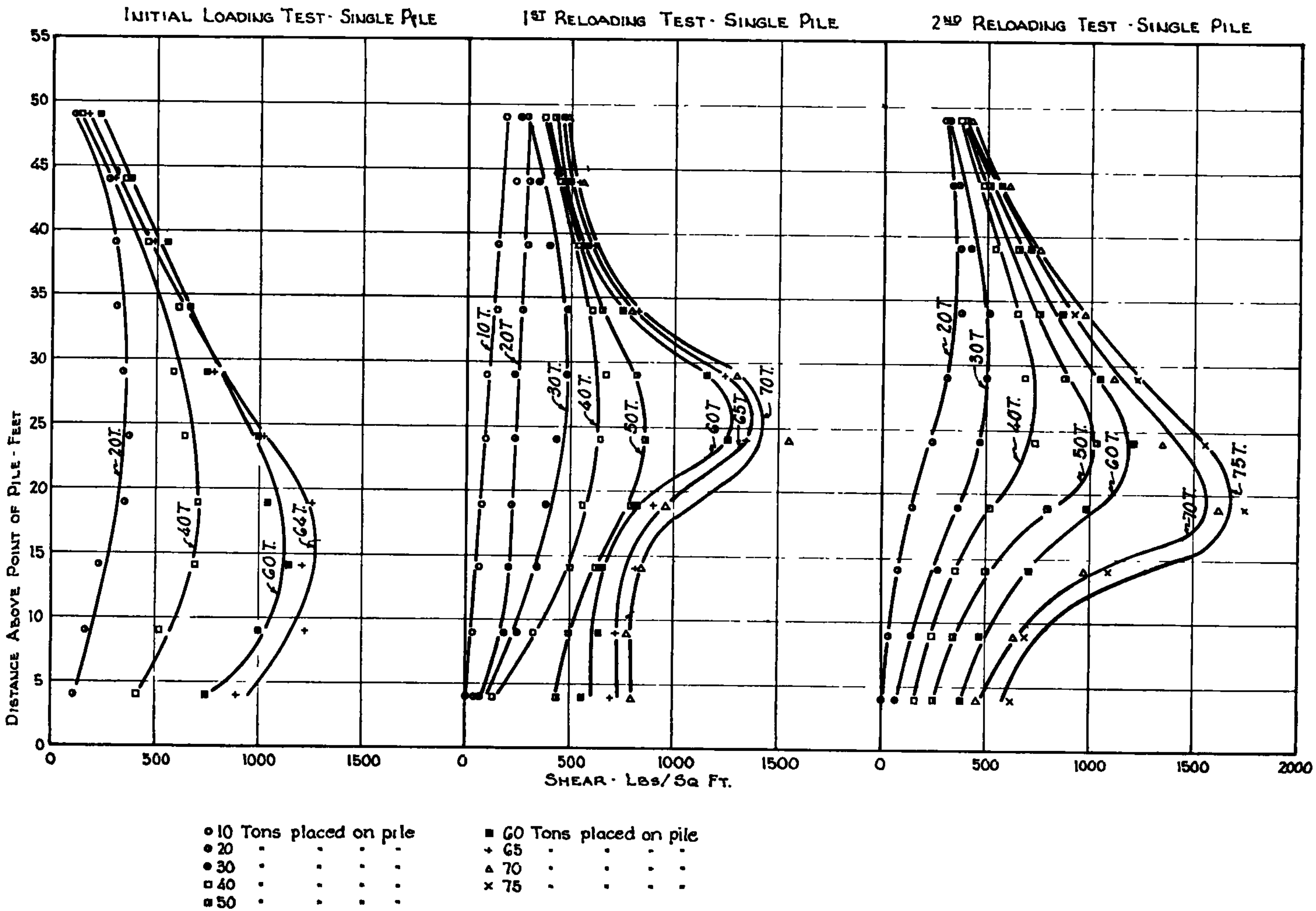


Figure 9. Shear diagrams, single-pile test.

time the settlement readings varied from $\frac{5}{32}$ in. for Pile 4 to $\frac{25}{64}$ in. for Pile 6.

Strain-gage readings were read and tabulated for all operating gages for each increment of load at intermediate 3-hr. periods. It first two increments of load, it was thought advisable to remove all the load then in place, and reëstablish the zero, or no-load, values on both the strain gages and the settlement rods. Accordingly, the 334-ton load was re-

TABLE 1
STRESSES IN PILE 1
SINGLE PILE TEST

Level	Area sq. in.	Gage	Initial Loading Test												
			Load = 20T			Load = 40T			Load = 60T			Load = 64T		
			Rdg.	KI	Tot. ld.	Rdg.	KI	Tot. ld.	Rdg.	KI	Tot. ld.	Rdg.	KI	Tot. ld.
1	7.29	N	165	5.0		335	10.1		508	15.2		546	16 4	
		E	166	5.0		367	11.0		587	17 6		639	19.2	
		S	140	4.2		332	10.0		526	15.8		580	17.4	
		W	177	5.3		337	10.1		498	14.9		537	16.1	
		Ave.	162	4.9	35.7	343	10.3	75.1	530	15.9	115 9	576	17 3	126.1
2	7.24	N	147	4.4		302	9.1		453	13 6		482	14.5	
		E	137	4.1		308	9 2		483	14 5		537	16.1	
		S	151	4.5		358	10.7		568	17 0		611	18.3	
		W	162	4.9		348	10.4		526	15.8		559	16.8	
		Ave.	147	4.4	31.9	329	9.9	71.7	508	15.2	110.0	547	16 4	118.7
3	7.19	N	147	4.4		274	8.2		435	13.1		475	14 3	
		S	126	3.8		290	8.7		463	13.9		528	15.8	
		Ave.	137	4.1	29 5	282	8.5	61.1	449	13.5	97 1	502	15.1	108 6
4	7.12	E	99	3.0		253	7.6		419	12.6		472	14 2	
		W	96	2.9		251	7.5		417	12.5		475	14.3	
		Ave.	98	2.9	20 1	252	7.6	54 1	418	12.5	89 0	474	14 2	101.1
5	7.05	N	82	2.5		217	6.5		363	10 9		414	12 4	
		S	70	2.1		213	6.4		366	11.0		421	12.6	
		Ave.	76	2.3	16.2	215	6.5	45.8	365	11.0	77.6	418	12.5	88.1
6	6.75	E	60	1.8		178	5.3		306	9.2		360	10.8	
		W	64	1 9		181	5.4		308	9.2		351	10 5	
		Ave.	62	1.9	12.8	180	5 4	36 4	307	9 2	62.1	356	10 7	72.2
7	6 36	N	41	1 2		144	4 3		234	7.0		264	7.9	
		S	48	1.4		148	4.4		281	8.4		324	9.7	
		Ave.	45	1.4	8 9	146	4.4	28.0	258	7.7	49 0	294	8.8	56.0
8	5 94	E	21	0.6		95	2.9		185	5 6		223	6 7	
		W	25	0.8		98	2.9		192	5.8		215	6.5	
		Ave.	23	0.7	4 2	97	2.9	17 2	189	5.7	33 9	219	6 6	39.2
9	5.49	N	21	0 6		59	1.8		112	3.4		154	4 6	
		S	14	0.4		57	1.7		111	3.3		140	4.2	
		Ave.	18	0.5	2 7	58	1.7	9 3	112	3 4	18 7	147	4.4	24.2
10	5 09	N	5	0.2		52	1.6		123	3 7		119	3 6	
		E	5	0.2		41	1.2		98	2.9		108	3.2	
		S	1			43	1.3		117	3.5		131	3.9	
		W												
		Ave	4	0.1	0 5	45	1.4	7.1	113	3 4		119	3.6	18.3
11	4.59	N												
		E	10	0 3		13	0.4		22	0.7		30	0.9	
		S												
		W												
		Ave.	10	0.3	1.4	13	0.4	1.8	22	0.7	3.2	30	0 9	4.1

required approximately $1\frac{1}{2}$ hr. to read and record the 231 values that constituted one complete set of strain gage readings.

Because of the variations in settlements of the several piling after the application of the moved in decrements of 80 tons and 254 tons. The first increment of load for the reloading of the pile group was 115 tons. After a 4-hr. waiting period the load was increased to 255 tons. Strain-gage readings and settlement read-

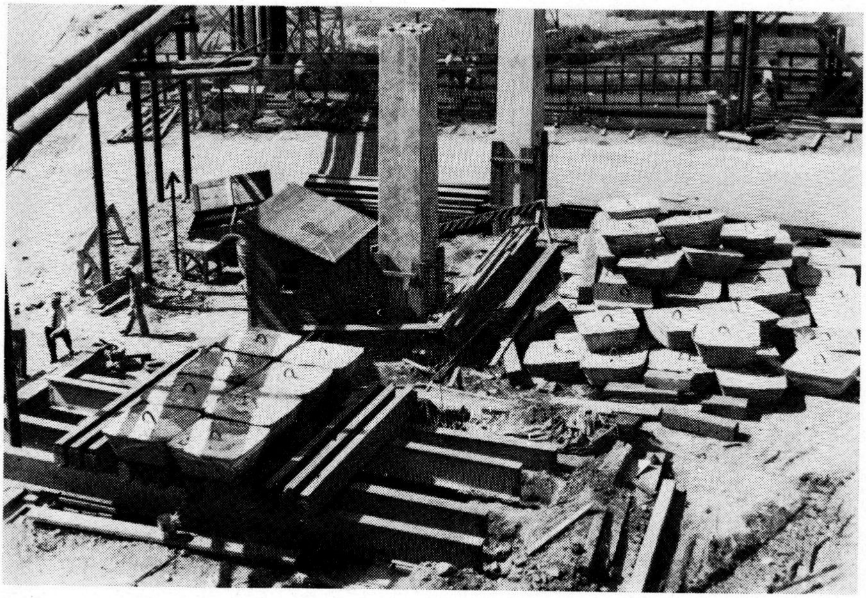


Figure 10. Placement of portion of original load increment, group-load test.

Figure 11. Maximum load in place, 569 tons; group-load test.

ings were read and recorded upon completion of the placement of each load increment, immediately prior to the placement of the next higher load, and at approximately 3-hr. intervals between the above stated times.

The load on the pile group was increased

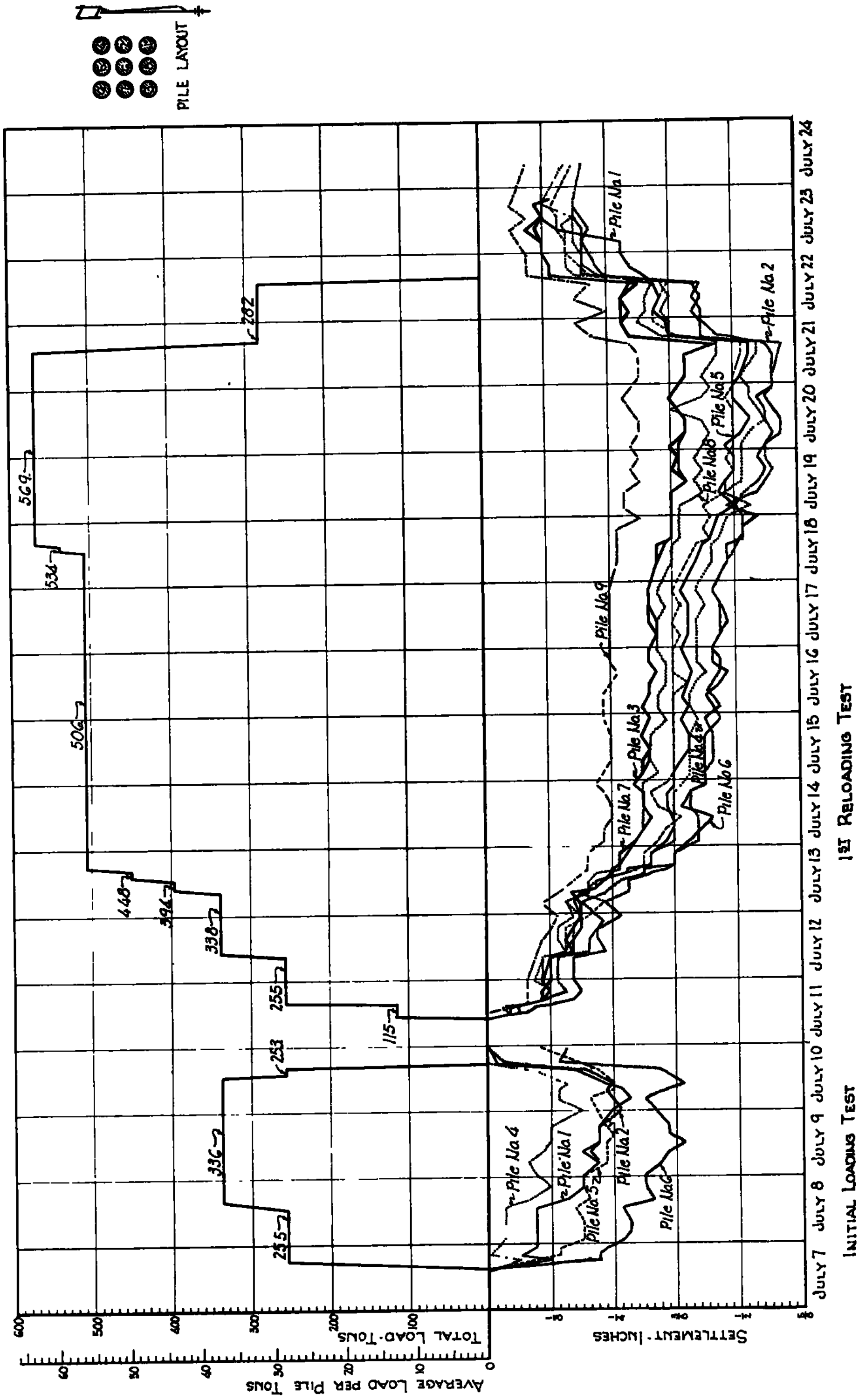


Figure 12. Load-settlement diagrams, group-load test.

by the application of 83-, 56-, 54- and 58-ton increments until a total load of 506 tons was in place. This represented an average load of 54.2 tons per pile. This load was held for 116 hr. During this time interval the pile group showed continued settlement during the early part of the period but became stabilized during the last 48 hr.

A review of the load-settlement curves (Fig. 13) indicated a nearly straight-line relationship between load and settlement. Since this would indicate that the pile group had not reached the failure point, an additional load increment of 28 tons was applied. At the end of a 2-hr. period this produced only a slight settlement of the pile group and the final increment of 35 tons was added. This brought the total load on the pile group to 569 tons, or an average of 62.6 tons per pile. This maximum load was in place for a total elapsed time of 71½ hr. (Figs. 11 & 12).

The maximum gross settlement and the net permanent settlement for each of the nine piles in the group are as follows:

Pile No.	Max. Gross Settlement	Net Permanent Settlment
1	17/32	1/8
2	19/32	13/64
3	15/32	9/64
4	25/64	11/64
5	23/64	3/16
6	19/32	7/32
7	15/32	9/64
8	15/32	13/64
9	19/64	3/32

Following the procedure established for the single pile test, charts were prepared showing the load at each gage level in each pile. Likewise charts were prepared indicating the values of the shear, or friction, between the pile surface and the earth.

It is interesting to note that the pile reactions and the shear curves for Pile 1 when test loaded as a single pile and as a member of the pile group indicated a marked difference in the manner in which the pile carried its load (Fig. 14). The maximum shear in the earth when this pile was loaded as a single pile was 1,675 lb. per sq. ft. and occurred over a very short length of the pile in the region of

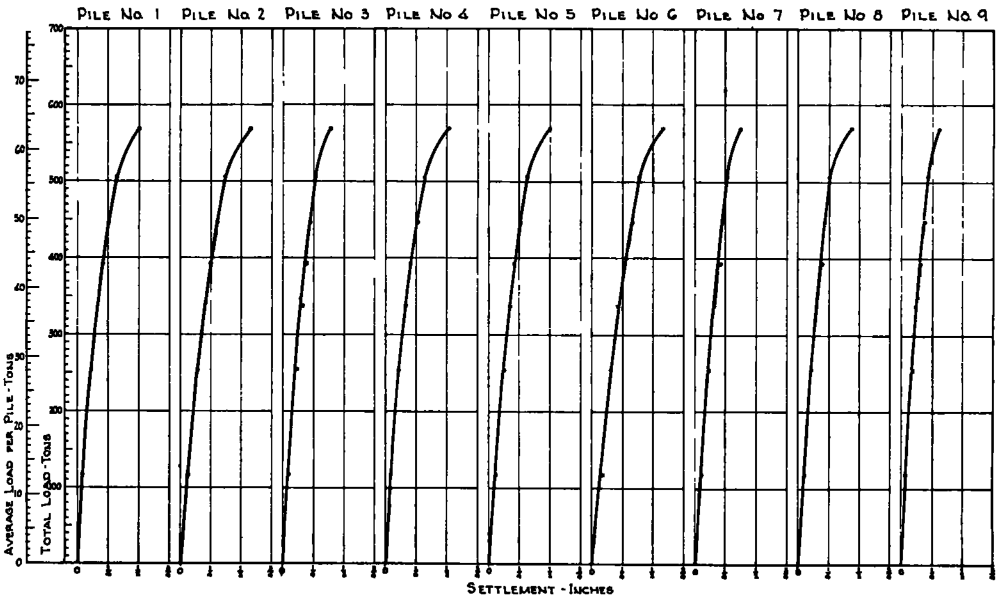


Figure 13. Load-settlement diagrams, group-load test.

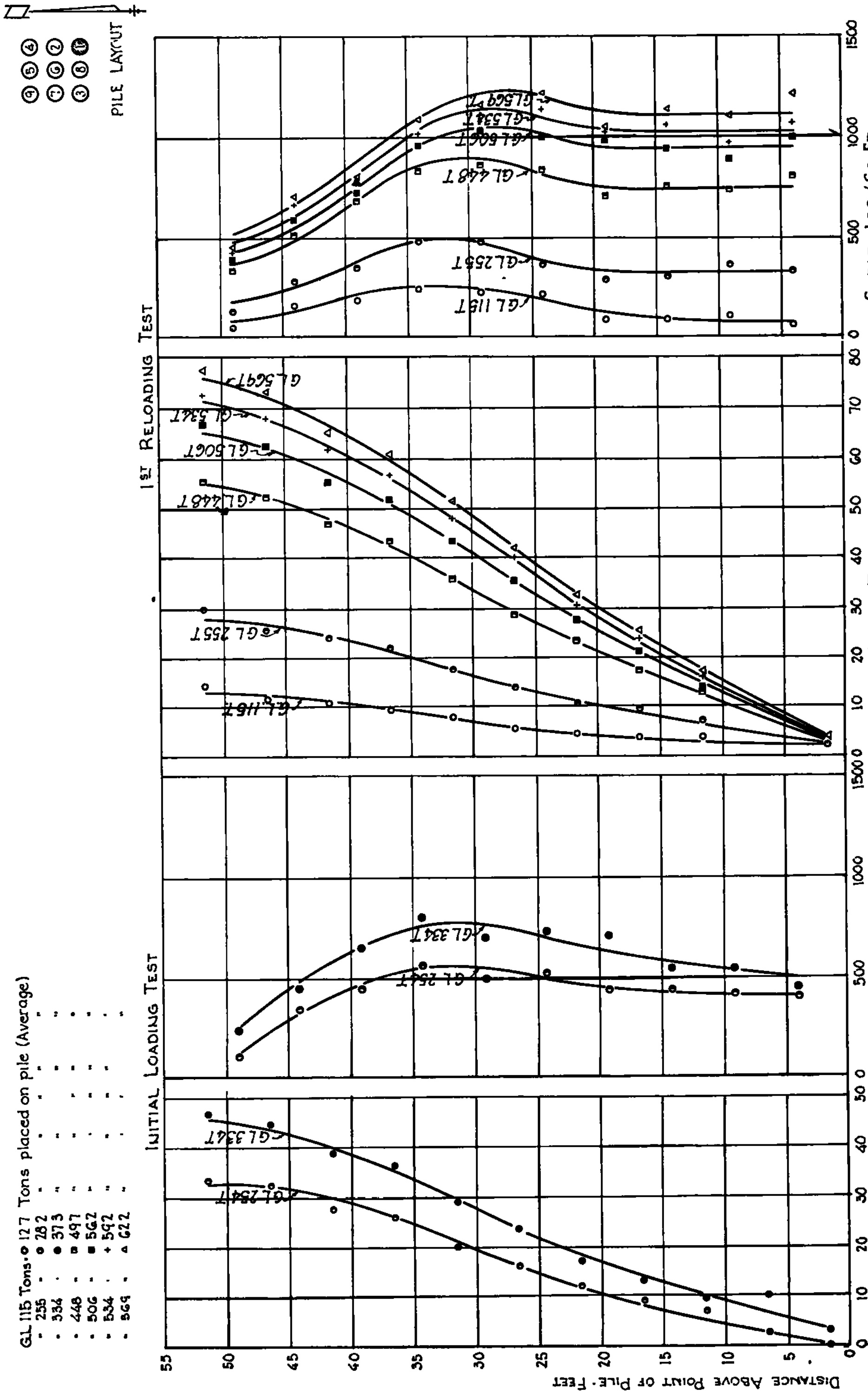


Figure 14. Loading reactions and shear diagrams; group-load test, Pile 1.

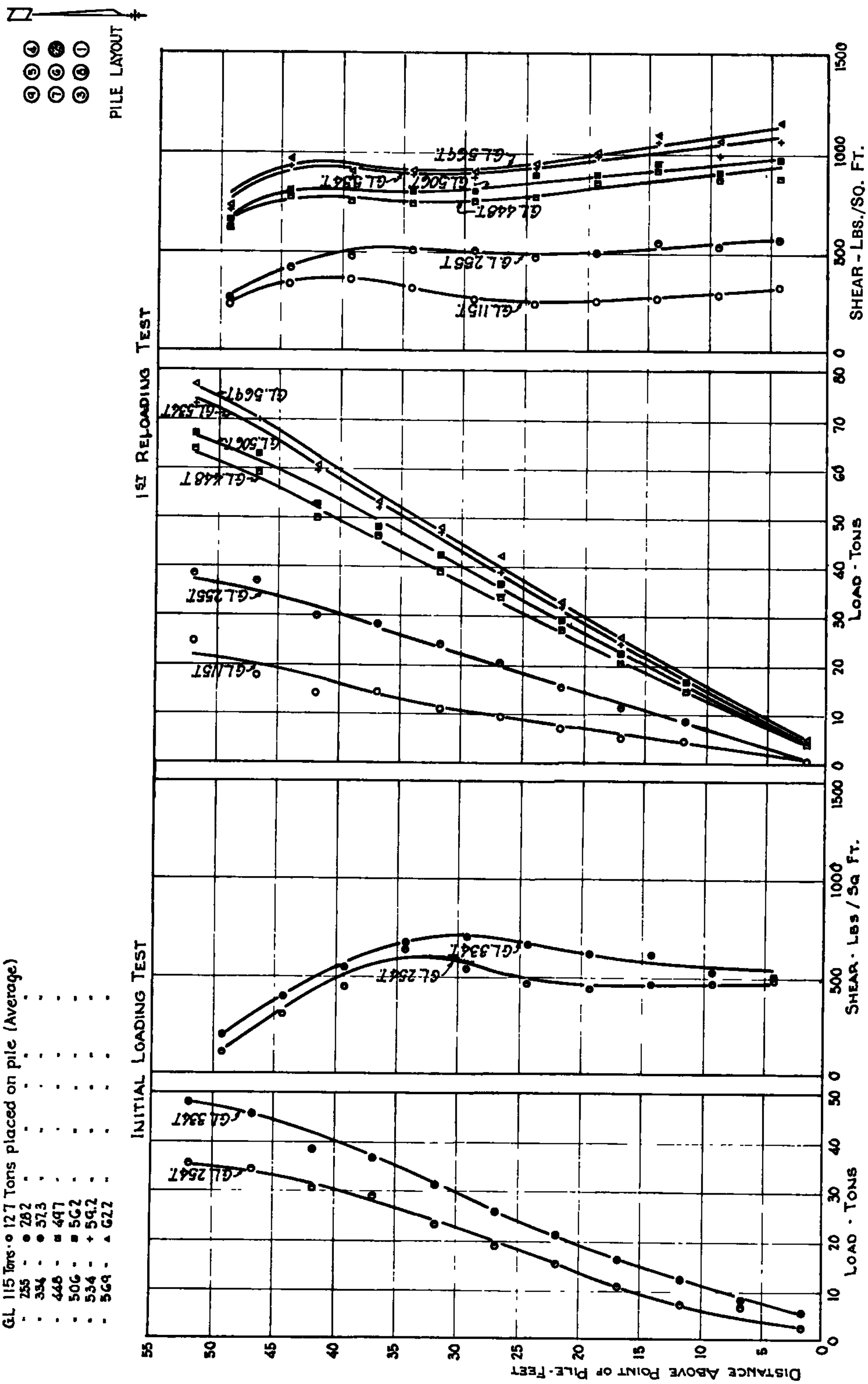


Figure 15. Loading reactions and shear diagrams; group-load test, Pile 2.

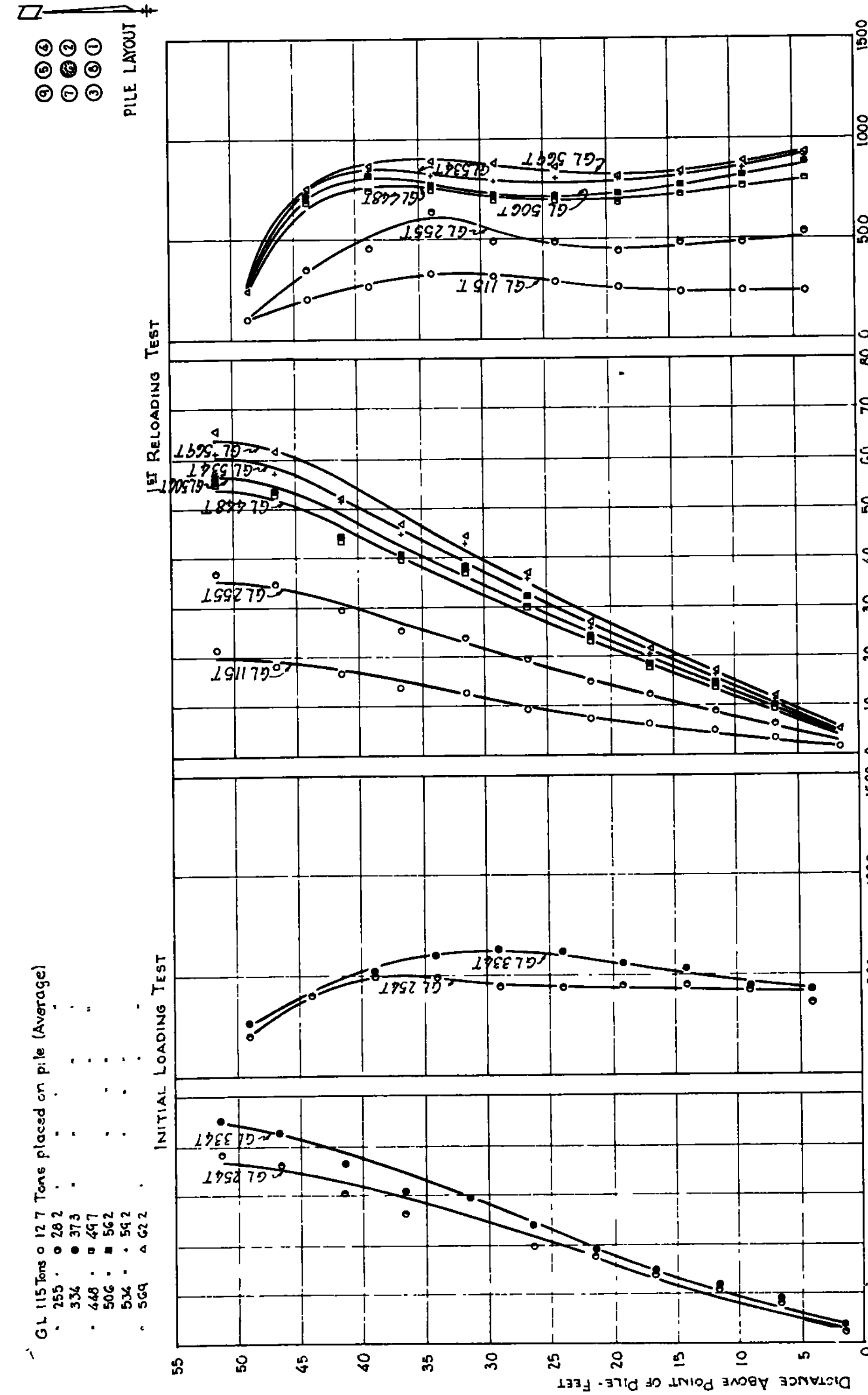
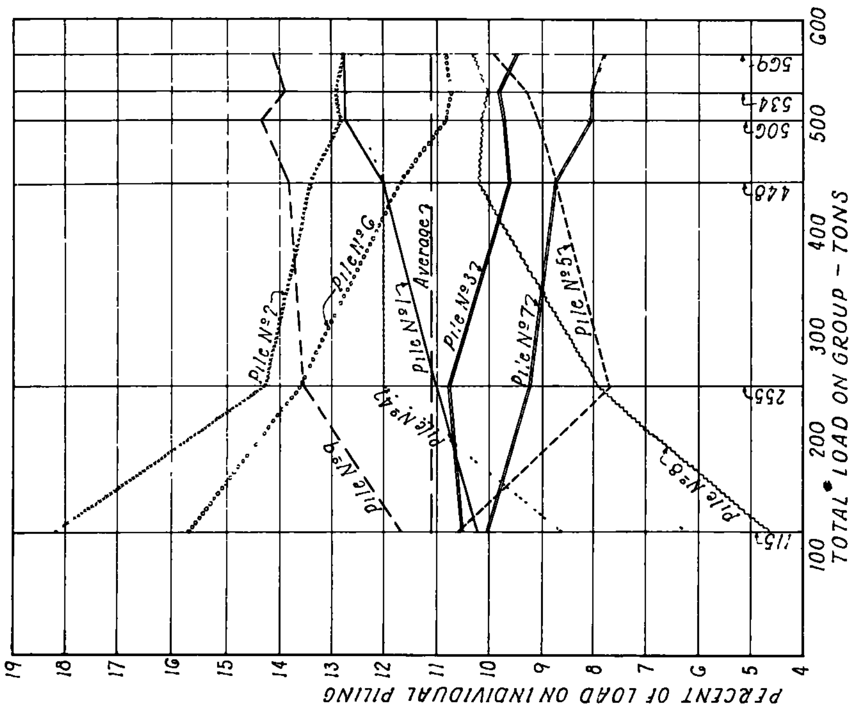


Figure 16. Loading reactions and shear diagrams; group-load test, Pile 6.



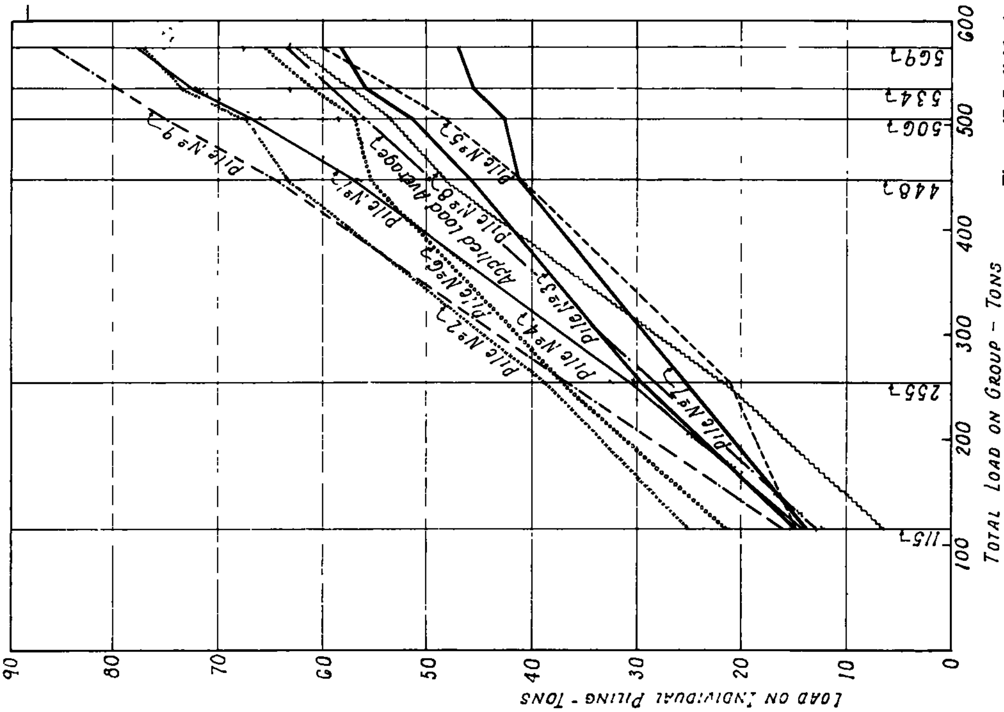


Figure 17. Individual-pile loads, group-load test.

TABLE 2
TRIAXIAL TESTS AND SOIL DATA

Level	Elevation Top of Sample	Lab. Ident.	Date Received	Percent Water	Dry Density Lbs./Cu Ft.	Shear Stress Lbs./Sq. Ft. Mohr	Shear Stress Lbs./Sq. Ft. Housel	Angle of Int. Friction, Degrees	Angle of Rupture, Degrees	Unconfined Compressive Strength[a]	Color	Texture	Specific Gravity[b]	Liquid Limit[b]	Plasticity Index[b]	Composition by Hydrometer Analysis[c] Sand	Silt	Clay	Colloids	Percent Retained on Screen No. 30	40	50	80	100	200	A A S H.O Classification
2	119.6	1000-R	9-23-50	30.5	87.4						Tan, black & gray fill, mottled	Old fill material; soil; cinders; etc.	2.69	41	17	12	70	18	6	3	3	4	4	4	5	A 7-6 (11)
3	113.7	1001-R	9-24-50	24.5							Tan, olive, & black fill; mottled	Soft tan; hard black; small stones														
	111.2	1002-R	9-26-50	31.5	87.4	562	662	14.6	37.7	1368	Tan & dk. green; black fill; mottled	Soft tan, gritty black	2.66	42	15	12	67	21	8			0	1	1	1	A 7-6 (10)
4	108.7	1003-R	9-26-50	30.2	89.9	720	850	10.0	40.0	1714	Dk. green; black; tan; mottled	Some gritty sections, slightly crumbly	2.69[d]	46	21	12	64	24	8		0	1	1	1	2	A 7-6 (13)
	106.2	1004-R	9-27-50	33.1	83.6	144	173	17.0	36.5	403	Dk gray, green; brown, mottled	Brown soft streaks, dark hard streaks	2.68	44	20	13	63	24	8					0	1	A 7-6 (13)
5	103.7	1005-R	9-27-50	31.1	95.5	734	821	5.2	42.4	1656	Dk. gray; green; brown; mottled	Fairly unif. texture; easily crumbled	2.68[d]	42	17	9	72	19	6				0	1	1	A 7-6 (11)
	101.2	1006-R	9-27-50	33.1	88.6	533	576	7.1	41.1	1195	Mottled gray; green; black; brown	Solid; easily carved, sticky	2.68[d]	43	18	11	67	22	7						0	A 7-6 (12)
6	98.7	1007-R	9-29-50	36.0	82.4	432	461	9.7	40.2	1008	Mottled gray; green; black	Solid; crumbly	2.69[d]	44	19	7	71	22	8						0	A 7-6 (13)
	96.2	1008-R	9-29-50	35.2	84.9	360	432	11.3	39.4	864	Mottled gray; green; black	Solid; crumbly	2.68[d]	43	18	6	70	24	8						0	A 7-6 (12)
7	93.7	1009-R	9-29-50	37.7	81.1	504	576	9.5	40.3	1181	Very dark gray; black; mottled	Hard; easily fractured	2.68[d]	47	22	4	69	27	9						0	A 7-6 (14)
8	88.7	1010-R	10-3-50	33.6	87.4	850	1008	11.9	39.1	2074	Dark gray, black; mottled	Hard; easily fractured	2.68[d]	47	22	4	65	31	12					0	1	A 7-6 (14)
	86.2	1011-R	10-4-50	33.6	86.1	605	749	13.0	38.5	1598	Black; green; blue; streaked	Solid; crumbly	2.69[d]	39	13	9	76	15	5						0	A-6 (9)
9	83.7	1012-R	10-4-50	33.8	88.0	533	706	20.5	34.8	1512	Dark green; gray, streaked	Solid, easily crumbled	2.67	42	17	7	71	22	8						0	A 7-6 (11)
	81.2	1013-R	10-4-50	35.2	86.7	590	720	16.7	36.7	1498	Dark green; gray; streaked	Solid; crumbly	2.66[d]	53	27	6	65	29	12						0	A 7-6 (18)
10	78.7	1014-R	10-4-50	37.8	81.7	504	605	10.3	40.0	1224	Dark green; black streaks	Very solid; easily crumbled	2.66	40	18	4	76	20	7					0	1	A-6 (11)
	76.2	1015-R	10-4-50	35.8	85.5						Dk. green; black; blue streaks	Soft; from standing in water	2.66[d]	37	13	6	73	21	7						0	A-6 (9)
11	73.7	1016-R	10-5-50	32.1	91.1	1080	1411	15.2	37.4	2808	Dark gray, green layers	Solid; easily carved	2.66[d]	37	13	10	72	18	6						0	A-6 (9)
	71.2	1017-R	10-5-50	31.8	85.5	1152	1368	9.9	40.0	2779	Olive, black; gray streaks	Soft and sticky; difficult to carve	2.66	35	9	10	79	11	7						0	A-4 (8)
	68.7	1018-R	10-5-50	31.2	91.1	720	907	13.9	38.1	1872	Dark layered gray; green	Solid; mellow; very easy to carve	2.66[d]	34	10	20	67	13	6					0	1	A-4 (8)
	66.2	1019-R	10-6-50	28.6	93.6	302	461	26.9	31.6	1022	Layers of dark gray, olive	Solid; sandy crumbly layers	2.66[d]	32	7	11	77	12	5	0	1	1	2	2	3	A-4 (8)
	63.7	1020-R	10-6-50	29.4	94.2	1325	1786	20.2	34.9	3758	Layers, black; green; gray	Sandy layers, mellow; easy to carve	2.66[d]	30	6	18	72	10	5	0	0	1	2	2	3	A-4 (8)

[a] Lbs./sq. ft., determined graphically. [b] Test on material passing #40 sieve. [c] Test on material passing #10 sieve. [d] Assumed.

NOTE: This test hole located 6'-3" North and 17'-5" East of center of group pile test.

the 20-ft. level. When loaded as a member of the group, the maximum shear was 1,230 lb. per sq. ft. In this latter case, however, the shear value was in excess of 1,100 lb. per sq. ft. from the point of the pile to the 34-ft. level with the above mentioned maximum occurring between the 25 ft. and 30 ft. levels. These shear values were obtained when the center piles varied from 1,140 lb. per sq. ft. to 800 lb. per sq. ft., and that of the center pile was 900 lb. per sq. ft. (Fig. 16). The average load carried by the 4 corner piles was 74 tons; by the side-center piles, 61 tons; and the center pile, 65 tons. These values are based on the strain-gage values, and their total exceeds the known load by 6.5 percent.

TABLE 3
SUMMARY OF TEST DATA
GROUP-LOAD TEST

Pile Number	Single	Group									
	1	1	2	3	4	5	6	7	8	9	Ave.
Diameter butt end—inches	12	12	12	12	12	12	12	12	12	12	12
Diameter 28′ from tip-inches	12	12	12	12	12	12	12	12	12	12	12
Diameter tip end-inches	8	8	8	8	8	8	8	8	8	8	8
Length of pile—feet	55.58	55.58	55 69	55.50	55.48	55 54	55.54	55.54	55.09	70.00[c]	57.33
Weight of pile—Kips	1 26	1 26	1.26	1.26	1.26	1 26	1.26	1.26	1 05	1.62[c]	1.27
Total penetration—feet	51.42	51.42	51.78	51 59	51 63	51.66	51.69	51.64	51.21	64.09	52 96
Total driving—blows[a]	460	460	342	300	333	401	357	365	299	1009	429
Blows last foot	16	16	16	12	15	17	12	15	14	32	16 5
Computed bearing—E.N.R. Formula—tons	17 6	17.6	17.6	13.6	16.7	18.6	13.6	16.7	15.7	31.5	18.0
Computed bearing—Modified Hiley Formula—tons	19.9	19.9	19.9	15.8	19.0	20.9	15.8	19.0	18.6	31.3	20.0
Penetration contact area—sq. ft.	154.5	154.5	155.7	155 1	155.2	155.3	155 4	155.3	153.9	196.6	159 7
Maximum test load—Kips	150	154 8	154.0	116 1	150 3	120.3	130.8	93.8	124.8	171 3	135.2
Duration maximum test load—hours	48	71	71	71	71	71	71	71	71	71	71
Gross settlement—64ths. of an inch	18[b]	34	38	30	35	33	38	30	30	19	32
Net (permanent) settlement—64ths of an inch	2[b]	8	13	9	11	12	14	9	13	6	10+
Maximum average skin friction—lbs./sq. ft.	971	1002	990	749	968	775	842	604	811	871	846
Maximum measured skin friction—lbs /sq. ft.	1675	1230	1140	1120	1400	910	900	800	—	—	1071
Percent of total load carried by each pile	—	12.7	12.7	9 5	12.3	9 9	10.8	7.7	10.3	14.1	11 1

[a] Driven with a No. 1 Vulcan Hammer operating at 55–58 blows per minute.
[b] 2nd reloading test.
[c] At time of driving.

loading at the top of the pile was 75 tons in the single test and 77.4 tons in the group test.

The loading reactions and shear diagrams for the other two corner piles show a somewhat different pattern than that of Pile 1. It is to be noted that these two piles carried loads of 58 and 75 tons and the maximum shear values occurred at elevations of 28 ft. and 27 ft. above the tip.

Of interest, also, is the variation in the manner in which the three corner piles carried their loads when compared to the three side-center piles and the center pile (Fig. 15). This may be influenced somewhat by the longer corner pile, Pile 9, but there is some indication of the effect of the spacing of the piles within the group in these stress patterns. The maximum shear stress in the three corner piles varied from 1,400 lb. per sq. ft. to 1,120 lb. per sq. ft., while that of the three side-center piles varied from ... The distribution of these loads under the several load increments is shown graphically in Figure 17 and in Table 3.

SOIL TESTS

A series of soil samples were taken for the purpose of laboratory study and for obtaining shear tests representative of the true strength of the soil in the undisturbed ground under the actual foundations. Two laboratory specimens were prepared from each field sample. These specimens were carved by hand from the 3⅜-in.-diameter field core. The specimens were 6 in. high by 2.80 in. in diameter. Approximately 15 min. were required to carve each sample and the triaxial tests were performed immediately after carving.

The data shown in this report were obtained from triaxial tests performed at a deformation speed of 0.1 in. per min. Lateral pressures of

5 psi. and 20 psi. were used. An open system was maintained (internal specimen pressure = atmospheric pressure). Perforated aluminum plates were placed on the top and bottom of the test specimen. These plates permitted moisture drainage during the testing of the specimen. Some drainage was observed.

The results of the soil analysis are shown in Table 2.

CONCLUSIONS

These tests afforded an opportunity to study the load distribution within a pile under static loads and to compare stresses in a single pile statically loaded with stresses in that pile when it was incorporated into a group of piles under static load.

The tests furnished information as to the efficiencies of individual piles when driven in a group into plastic soils and permitted a study of the variation in the soil stresses at various depths of penetration for the single pile as well as for the individual piles within a group.

While many interesting observations were made, it must be borne in mind that these tests were for a particular soil condition and for a short time of application of the load.

From the data obtained in these tests, the following conclusions may be drawn:

(1) The load on the single pile was carried by friction between the pile and the clay in the upper half of the pile. Relatively high shear values existed in the soil for a short length of the pile.

(2) The load on the individual piles within the group was transferred to the soil by shear, with the maximum values occurring at slightly higher levels than for the single pile. These maximum values were lower than those for the single pile and occurred over a much longer length of the piles.

(3) The effect of the pile spacing was reflected by the drop in the shear values between the corner piles, side-center piles, and the center pile of the group.

(4) The distribution of load to the various piles in the group was fairly uniform, each pile, including the center pile, carrying a portion of the load.

(5) The normal dynamic formulas are poor criteria for establishing bearing values for piles in plastic soils, as is the number of blows required for the final foot of penetration.

(6) The point at which failure is considered to occur in a test load provides the basic value for the assignment of a safe design load. Failure is considered to occur when the rate of movement begins to increase sharply in proportion to the increase in load, rather than when a pile has reached a specified permanent settlement.

DISCUSSION

PHILIP KEENE, *Connecticut Highway Department*—The author is to be complimented on his thorough presentation of these tests, which mark another step forward in the knowledge of the behavior of friction piles. Comparatively little is known of the behavior of piles and the soil surrrounding them, but interest in this highly complex subject is strong, as evidenced by the articles and papers in civil engineering literature in recent years. A pioneering step is the use of SR-4 strain gauges for measuring the loads in the piles. The magnitude of the work described by the author is evidenced by the fact that 231 such gauges were installed, under rather difficult conditions. Borings, including undisturbed soil sampling taken close to one or more piles after they were driven, would have been highly desirable to ascertain the effect on the soil by pile driving; apparently time did not permit taking such supplemental borings.

The author's six conclusions are clearly stated and appear to the writer to be correct, with the following exceptions:

Conclusion 1—The load on the single pile was carried by friction between the pile and the clay throughout the pile, and not along the upper half of the pile. Figures 8 and 9 show that about half the load was still in the pile at midheight.

Conclusion 6—A safe design load may be based on tolerable differential settlements for the structure to be supported, rather than on a certain percentage of the load at incipient failure of the piles.

As stated by the author, it is interesting to note that the pile reactions and shear curves for the single pile test show a marked difference from those of the group pile test.

Another feature is that the center pile, No. 6, settled more than the other piles. The following comments are offered as possible partial explanations of these phenomena:

When studying the pile load reactions for single-pile tests (Fig. 8) and for group-load tests (Figs. 15, 16, 17), the writer marked the points on the load curves at which 50 percent of the pile load was still carried in the pile. For the single-pile test, these "50 percent points" were at about 34 to 40 ft. above pile tip for the smallest load increment and gradually shifted down to about 25 ft. above pile tip for the maximum loads. In other words, as increments of load were added to the pile, the lower part of the pile took a large percentage of the load.

In the case of the group-load tests the 50 percent points were very close to 25 ft. above pile tip for all but one of the 23 load increments reported. There was only a very slight tendency for the 50 percent points to shift downward with increasing load increments.

Two of the important factors which influence the load distribution in the piles can be considered. One factor is the effect of displacement of the soil by the volume of the pile. The upper part of the pile is larger in diameter and hence causes more soil displacement. In a loose sand, this would cause a large densification, but in the soil at this site, it would cause much disturbance of the clay and possibly considerable loss of shearing strength. (Tests on remolded soil would indicate approximately the loss to be expected.) Hence the soil adjacent to the lower part of the pile may be stronger and stiffer and have a higher modulus of elasticity in shear than the upper clay. With each load increment and accompanying small settlement, the lower clay could mobilize a larger unit shearing resistance than the upper clay.

Another factor is the increase, as the pile settles under load increments, in the lateral (passive) pressure in the clay adjacent to the tapered portion of the pile. This increase does not occur in the upper clay, as the pile there is not tapered. Thus, as load increments are added, the upward resistance of the lower clay increases. This increase is equal to the vertical component of the pressure increase normal to the pile plus whatever increase in shearing resistance results from the increased compressive stress in the clay.

Both of these factors could explain why the lower clay takes a larger share of the load from the pile as load increments are increased.

In the case of the group-load tests, the effect on the soil around a pile due to the stresses imposed by an adjacent pile must be considered. The tapered portion of the adjacent pile, when loaded, causes further increase in the passive pressure around the lower half of the first pile. This tends to increase the shearing resistance of the lower half of the first pile as increments are added.

On the other hand, the vertical shearing stresses in the soil caused by the load on the adjacent pile are in the reverse direction to those caused by the first pile. In less technical language, the soil adjacent to the first pile is dragged downward while it is supporting the pile; when the adjacent pile is loaded, it tends to drag the same soil in the opposite direction. This reduces the shearing resistance which is supporting the first pile and results in a slight vertical yield or settlement. Since reduction is about proportional to stress, the greater reduction is in the lower clay.

Examination of the group-load-test data shows that the center pile, No. 6, settled more than the rest. Since it carried only about the average load per pile, its greater settlement is probably due to its being located in the center. Since it settled more than the other piles immediately upon loading, its greater settlement probably is not due to consolidation of the soil beneath its tip. Hence its greater settlement may be due to greater soil disturbance by pile-driving or to decrease in shearing resistance due to shearing stresses in the reverse direction as described in the preceding paragraph.

It is noted that in the group-load tests, Pile 1 settled about 40 percent more, for the same load imposed on it, as in the single-pile tests. This may be due chiefly to the reasons given above, or else chiefly to the added soil disturbance created after the single-pile test, when the remaining piles of the group were driven.

It can be assumed that the soil is rather homogeneous. In the boring log in Figure 8 is listed "some sand and gravel," but apparently this material is of small importance, as Figure 19 shows no sand and gravel and the undisturbed samples were taken at very close intervals.