Axial Capacity of Vibratory-Driven Piles versus Impact-Driven Piles

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In recent years, vibratory pile drivers have gained popularity with contractors due to the increased productivity that can be realized with their use. The driving time can be reduced by a factor of 10 to 20 over that of an impact-driven pile. This gain in productivity is very attractive and profitable, but questions exist as to whether vibratory driving has an effect on a pile's axial capacity when compared with an impact-driven pile. This paper will present and discuss the results of a study of three pile testing programs that make direct comparison between vibratory- and impact-driven piles. One of these testing programs has never before been reported in the literature. From the study of these testing programs, it was found that the vibratory-driven piles had a significant reduction in axial capacity when compared with the impact-driven piles. This reduction in capacity of the vibratory-driven piles was due to a lower tip resistance.

In recent years the use of vibratory pile driving hammers has gained popularity with contractors because of the increased productivity realized with their use. Driving piles with a vibratory hammer can reduce driving times by a factor of 10 to 20 over that of an impact-driven pile. This gain in productivity is very attractive and profitable when a large number of piles are being installed.

As the popularity of vibratory hammers has increased, so have the questions about the axial capacity of vibratory-driven piles as compared with impact-driven piles. Pile installation invariably results in altering the stresses in the soil surrounding the pile. Studies (*1,2,3*) have revealed that impact driving in granular soils causes compaction of the soil in the vicinity of the pile. Consequently, the stress levels are increased. Less is known about the changes that the soil undergoes in the vicinity of a vibratory-driven pile.

This paper presents the findings of an investigation that examined pile test data to determine if there were any significant differences between the axial capacity of piles driven by vibratory hammers and by impact hammers. The pile load test has long been recognized as the only true measure of axial capacity for a given site. A load test permits the direct measurement of pile capacity under the actual construction and soil conditions that prevail at the site. The investigation concentrated on test programs at sites where tests were performed on piles driven by both vibratory and impact hammers.

BACKGROUND

During the construction of Lock and Dam No. 1 for the Red River Waterway, a pile testing program was undertaken to verify the pile design for the dam. The piles at the site were driven with a vibratory hammer. The piles tested were H piles with lengths between 55 and 70 ft. The capacities of the piles tested were 40 to 70 percent less than the expected values. In an effort to discover if the reduced capacity was due to the driving of the piles with a vibratory hammer, the U.S. Army Engineer Division, Lower Mississippi Valley, initiated the investigation reported in this paper.

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FIELD PILE TESTS

A search of the literature and Corps of Engineer files was conducted to find as many pile test programs as possible that had direct comparisons of vibratory- and impact-driven piles. This paper presents three of the pile test programs discovered during the investigation (*4*). These three programs were selected because they were well documented and represented the most common use of vibratory hammers for pile driving. Brief descriptions of the site conditions, the pile tested, and the test results are presented here.

Arkansas River Lock and Dam No. 4

The pile testing program for Lock and Dam No. 4 was instituted as the primary source of information for the design and construction of the four locks and dams along the lower Arkansas River. In view of the magnitude of the projects and the lack of factual information regarding the drivability and capacities of piles in the lower Arkansas River Valley, a comprehensive pile testing program was conducted by the U.S. Army Engineer District, Little Rock. The purpose of the tests was to develop criteria for the design and construction of pile foundations for future locks and dams. The general objectives of the pile test program were to establish design and construction criteria for axially and laterally loaded piles and to determine the type and size of pile-driving hammers required for economical installation. The results of the pile test program were presented in a report for the Little Rock District prepared by Fruco and Associates (*5*).

Site Description

Soil conditions in the lower Arkansas River Valley are typical of an alluvial pastoral zone. In general, they consist of alluvial deposits of loose surface silts, sandy silts, and clays of variable thickness, underlain by a zone of medium to dense silty sand with a thickness ranging from 70 to 150 ft. This all overlies a stratum of deeply bedded Tertiary clays.

The test site was located on the east bank of the Arkansas River about 20 miles downstream from Pine Bluff, Ark., and 9 miles upstream from the future site of Lock and Dam No. 3 (Figure 1). The soil conditions at the pile testing site were determined by exploratory borings and laboratory tests made in connection with the foundation investigation for Lock and Dam No. 4, and further explorations were made specifically for the pile testing program. These explorations indicated that three major soil strata exist at the test site: a surface blanket of silts and clays, which extends about 15 ft below the ground surface; a deep stratum of relatively dense, fine to medium sand, which extends about 100 ft; and a basal stratum of Tertiary clay of undetermined thickness. Discontinuous thin seams of silt and clay were encountered in a sand stratum at depths between 30 and 50 ft.

The test area was prepared by excavating approximately 20 ft of silty surface soils, exposing the underlying stratum of sand. Post-excavation standard penetration resistances increased with depth, ranging from 20 to 40 blows/ft, with an average of about 27 blows/ft. The dry density of the sand ranged from 90 to 109 pcf, but showed no significant trend with depth. The groundwater level was held at 2 to 3 ft below the surface of the site. Figure 2 shows a generalized profile for the test site.

Description of Testing Program and Results

The basic pile investigation included field driving and load tests on a variety of pile types. Tests were performed on square prestressed concrete piles, steel pipe piles, and steel H piles that were driven with both a double-action steam and a Bodine sonic vibratory hammer. The field load tests included compression, tension, and lateral loading of single piles. Strain instruments were attached to steel piles to determine the distribution of stresses in the piles under compression, tension, and lateral loads.

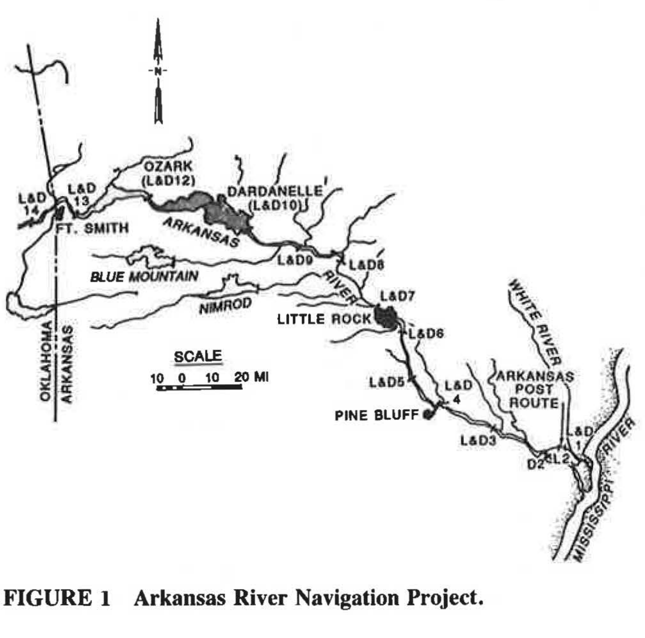

FIGURE 1 Arkansas River Navigation Project.

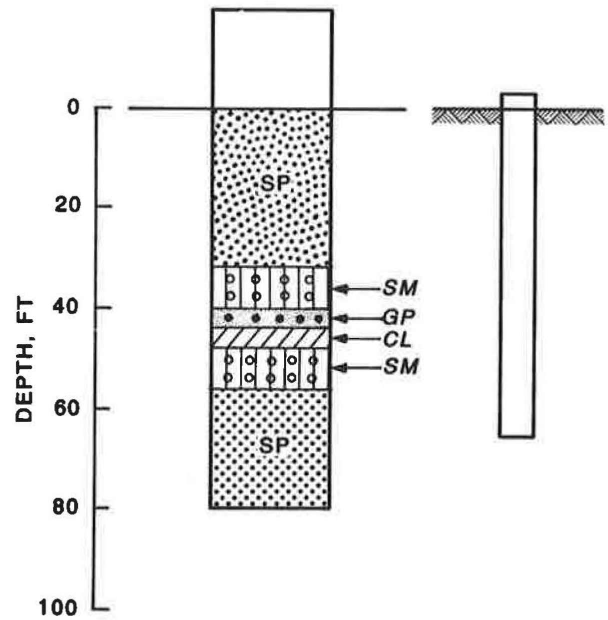

FIGURE 2 Generalized soil profile at Lock and Dam No. 4 pile test site.

Table 1 presents a summary of the axial load tests performed at the site. The table shows the type of pile tested, penetration, type of hammer used for installation, and reported average failures for compression and tension. Comparisons between impact- and vibratory-driven piles can be made with the 16 in. pipe piles, the H piles, and the 16 in. concrete piles.

In Table 2, the distribution of the load being carried by the side and the tip of the pile is given for the pipe sections. The total, tip, and side loads at failure are plotted in Figure 3 against the pile diameters for the various pipe piles tested. Piles 2 and 10, being spaced just 8 ft apart, provide a direct comparison between a pile installed by a Bodine sonic hammer (a high-frequency vibratory hammer) and a similar pile driven with a Vulcan steam impact hammer. Comparing the load carried by the tip and side for Piles 2 and 10, 16 in. pipe piles, shows that the impact-driven pile has significantly more capacity at the tip, 58 tons, than the vibratory-driven pile, 46 tons, while the side capacities differed only by 3 tons.

Table 3 presents the distribution of the load being carried by the side and tip for the H piles tested. Comparisons can be made between Piles 7 and 9. Pile 9, which was driven with the Bodine sonic hammer, had 20 tons greater capacity than Pile 7, which was impact driven. Examination of the distribution of the load in the piles reveals that the vibratory-driven pile, Pile 9, had 14 tons less tip capacity than the impact-driven pile, Pile 7, but had a substantially greater side capacity of 34 tons.

Arkansas River Lock and Dam No. 3

Exploration prior to construction showed a stratigraphy of the site typical of Arkansas River alluvial soils and comparable with that found at Lock and Dam No. 4. However, during the initial pile driving and load testing, it became apparent that the soil characteristics at the site were not as anticipated.

TABLE 1 SUMMARY OF ARKANSAS RIVER LOCK AND DAM NO. 4 PILE TESTS

TEST NO.	TYPE	PENETRATION Ft	HAMMER TYPE	AVERAGE PILE FAILURE LOAD, TONS COMPRESSION	TENSION
1	12 IN. PIPE	53.1	140C	140	70
2	16 IN. PIPE	52.8	140C	195	91
2X	16 IN. PIPE	52.8	140C	210	-
3	20 IN. PIPE	53.0	140C	215	90
4	16 IN. CONCRETE	40.2	140C	170	71
5	16 IN. CONCRETE	51.0	140C	240	-
6	14 BP 73	40.0	80C	140	-
7	14 BP 73	52.1	80C	190	45
8	TIMBER	38.6	65C	80	25
9	14 BP 73	53.2	BODINE	210	-
10	16 IN. PIPE	53.1	BODINE	180	87
11	16 IN CONCRETE	38.8	BODINE	150	-

TABLE 2 LOAD DISTRIBUTION IN PIPE PILES

TEST NO.	NOMINAL DIAMETER IN.	PENETRATION Ft	AVERAGE FAILURE LOAD TONS	TIP LOAD TONS	PERCENT	SKIN FRICTION TONS	PERCENT
1	12	53.1	140	34	24	106	76
2	16	52.8	195	58	30	137	70
2X	16	52.8	210	67	32	143	68
3	20	53.0	215	77	36	138	64
10	16	53.1	180	46	26	134	74

The initial compression and tension tests indicated that the design pile lengths would not carry the required loads with appropriate safety factors. Therefore, additional soil borings and field and laboratory tests were made. The results of these tests indicated that the removal of the overburden and/or scour during the cofferdam construction had relaxed the confining stresses within the soil mass, resulting in a significant reduction in the strength of the foundation. To determine the required pile lengths for the unexpected soil conditions, and to investigate the acceptability of the contractor's proposal to drive the bearing piles with a Foster vibratory hammer, the Little Rock District initiated a pile testing program.

Site Description

Arkansas River Lock and Dam No. 3 is located at Arkansas River navigational mile 49.3, approximately 30 miles down-

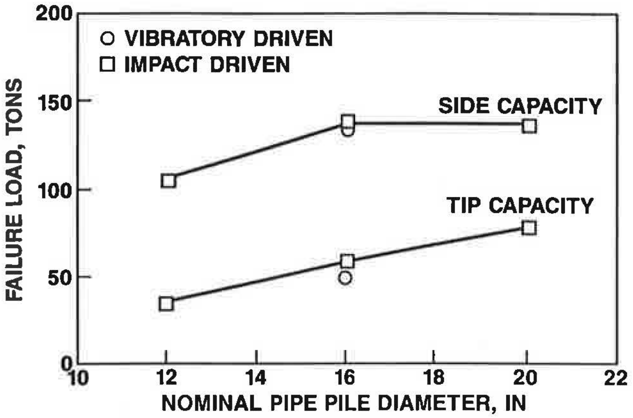

FIGURE 3 Failure load versus pipe pile diameter at Lock and Dam No. 4.

stream from Pine Bluff, Ark. (Figure 1). The geological conditions and stratigraphy are similar to those previously described for Lock and Dam No. 4.

The tests of interest were performed in the vicinity of the left river bank. The top stratum varied from 0 to 30 ft in thickness and consisted of erratically stratified silt and lean clay. These surface soils are underlain by 90 to 130 ft of sand, primarily gray and brown, clean, fine to medium sand with frequent lenses of clay, silt, and silty sand mixed with gravel lenses and occasional boulders. Below the sand deposit lies a Tertiary formation of stiff to hard, overconsolidated clay of low to high plasticity. The generalized soil profiles shown in Figure 4 were derived for the test area. Prior to pile driving, the test site was excavated into the thick sand stratum. Approximately 40 to 50 ft of the surface stratum was removed.

Description of Testing Program and Results

Load tests were performed to determine the axial capacity for different driving equipment, lateral capacity of piles, load versus length curves of the site, and water table correction factors for the submerged condition. For each of the piles tested, a cluster of at least nine piles was driven, with the center pile being the designated test pile. The test program consisted of 15 compression, 7 tension, and 10 lateral load tests (*6*).

Only the pile tests relevant to the comparisons of capacities between vibratory- and impact-driven piles were examined.

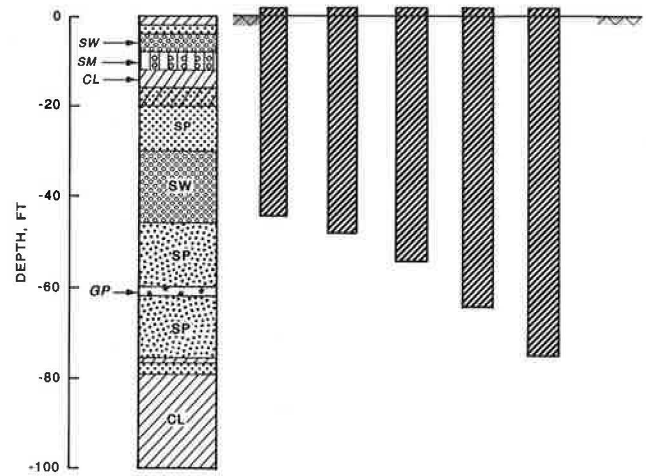

FIGURE 4 Generalized soil profile at Lock and Dam No. 3 pile test site.

The piles of interest were 14 BP 73, of lengths 45, 50, 55, 65, and 75 ft. Table 4 presents a summary of pile types, lengths, penetrations, hammer types, and failure loads for the piles that were examined for the investigation. Piles 1, 3, 3A, 3B, and 9 were driven by a Foster 2-50, low-frequency, vibratory hammer, and Piles 2, 2A, 5, 6, and 7 were driven by a Vulcan 140C steam hammer.

During the driving of the piles for these tests, the sand surrounding the piles loosened, and voids 5 to 10 ft deep formed between the flanges near the surface. Attempts were made to fill the voids and to compact the sand surrounding the piles. For Piles 3A and 3B, a concrete vibrator was used to place and increase the density of the sand in the flanges around the top of the pile. For Pile 2A, the voids in the flanges were filled with sand and water without compaction. For Pile 9, the voids were filled with sand and water, and the area surrounding the pile was compacted by vibroflotation. The vibroflotation compaction resulted in a significant increase in the capacity.

The main objectives of this portion of the testing program were to obtain data for determining the pile lengths needed for this lock and dam and to make a direct comparison between piles driven with a vibratory and an impact hammer. Figure 5 shows the failure loads versus depth of penetration for the compression tests. This figure shows that the impact-driven piles have a substantially higher capacity than the vibratory-driven piles, by an average of 32 tons. Figure 6 presents the

TABLE 3 LOAD DISTRIBUTION IN H PILES

TEST NO.	PENETRATION Ft.	AVERAGE FAILURE LOAD Tons	TIP LOAD TONS	TIP LOAD PERCENT	SKIN FRICTION TONS	SKIN FRICTION PERCENT
6	40.0	140	21	15	119	85
7	52.1	190	39	21	151	79
9	53.2	210	25	12	185	88

TABLE 4 SUMMARY OF ARKANSAS RIVER LOCK AND DAM NO. 3 H PIPE TESTS

TEST NO.	HAMMER *	PENETRATION FT	COMPRESSION FAILURE LOADS TONS		TENSION FAILURE LOADS, TONS	
			TESTED	ADJUSTED **	TESTED	ADJUSTED **
1	FR 2-50	42.3	85	71	25	22
2	VC 140C	42.8	134	104	34	27
2A	VC 140C	61.8	185	145	-	-
3	FR 2-50	46.7	105	80	-	-
3A	FR 2-50	46.7	120	92	31	23
3B	FR 2-50	61.8	145	117	-	-
5	VC 140C	52.8	150	128	39	32
6	VC 140C	63.0	175	155	51	44
7	VC 140C	73.0	215	190	-	-
9	FR 2-50	42.9	127	88	-	-

* FR 2-50 = FOSTER VIBRATORY HAMMER; VC 140C = VULCAN STEAM HAMMER.
** ADJUSTED FOR WATER LEVEL.

tension failure loads versus the depth of penetration. This figure reveals that the impact-driven piles have only a slightly greater capacity, 5 tons, than the vibratory-driven piles.

Crane Rail Tracks

During the construction of pile foundations for crane rail tracks for jib and gantry cranes, it was decided to investigate the use of a vibratory hammer for the pile driving instead of a drop hammer. It was believed that, for the subsoil conditions at the sites, the vibratory-driven piles would give the same bearing capacities as the impact-driven piles and would shorten the construction time. To substantiate this assumption, a series of pile load tests were conducted to make direct comparisons between piles driven with a vibratory hammer and with an impact hammer. Mazurkiewicz (7) reported the results and conclusions from the pile testing program. Site and pile descriptions and test results are summarized in the following paragraphs.

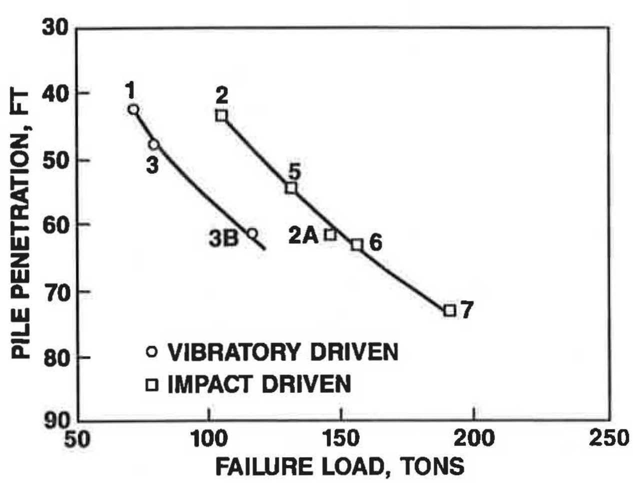

FIGURE 5 Failure load versus depth for compression tests at Lock and Dam No. 3.

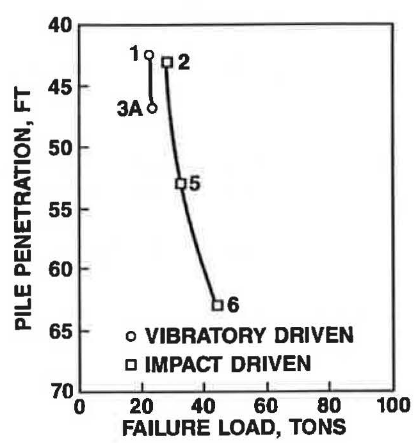

FIGURE 6 Failure load versus depth for tension tests at Lock and Dam No. 3.

In the vicinity of the construction sites and pile tests, the subsurface profile consists of a 3 to 5 ft layer of fill over a 3 to 6 ft layer of peat. The peat is underlain by layers of fine to medium sand and sandy gravels, which overlie a stratum of silty clay with silt. Penetration tests show a linear increase in resistance with depth through the sand and no trend in the clay. The stratification and representative penetration records are shown in Figure 7.

The load tests were performed on prestressed concrete piles with a diameter of 13.4 in. (34 mm) and lengths from 42.7 ft (13 m) to 88.6 ft (27 m). The comparison tests were made for 11 piles driven by a drop hammer and 11 piles driven by a vibratory pile driver. The distance between the two piles used for comparison (one impact driven and one vibratory driven) was 10 to 25 ft. Table 5 shows the failure loads obtained from the 11 sets of load tests. The ratios between the failure loads of the vibratory-driven and impact-driven piles are also given. Figure 8 is a plot of failure loads versus depth of penetrations for the 11 test sets. For each test set of piles, the impact-driven piles showed substantially higher failure load than the vibratory-driven piles.

The influence of time on the ratio of axial capacity of the vibratory- and impact-driven piles was also studied. It was found that, if the piles were tested after 4, 12, or 30 or more days, the difference in capacity between vibratory- and impact-driven piles remained unchanged. However, some increase in capacity with time did occur for both methods of installation.

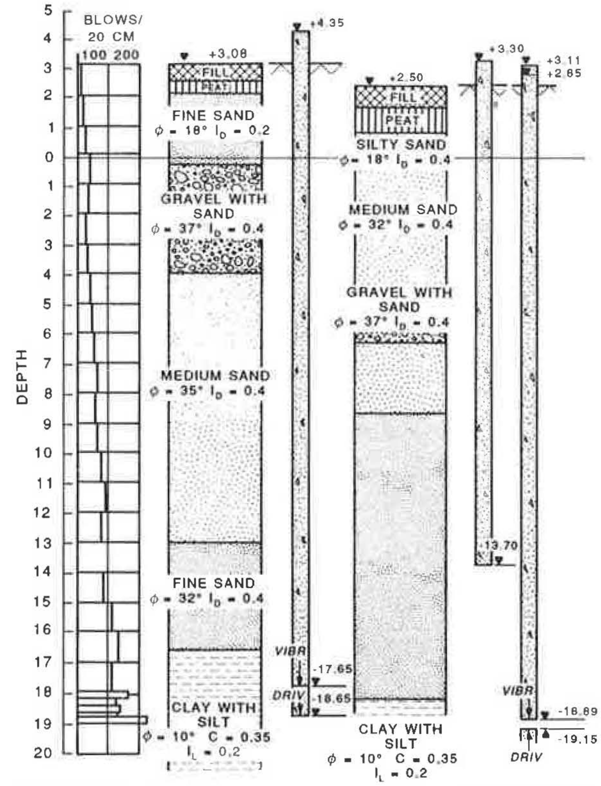

FIGURE 7 Stratification for the crane rail track test sites.

TABLE 5 SUMMARY OF CRANE RAIL TRACKS PILE TESTING PROGRAM

PILE TEST SET	LENGTH Ft (m)	FAILURE LOAD FOR VIBRATORY DRIVEN tons (mtons)	FAILURE LOAD FOR IMPACT DRIVEN tons (mtons)	RATIO
1	42.7 (13)	38.5 (35)	92.4 (84)	0.42
2	42.7 (13)	46.2 (42)	52.8 (48)	0.88
3	42.7 (13)	71.5 (65)	93.5 (85)	0.76
4	57.8 (17)	44.0 (40)	69.3 (63)	0.63
5	57.8 (17)	50.6 (46)	124.3 (113)	0.41
6	59.1 (18)	28.6 (26)	115.5 (105)	0.25
7	72.2 (22)	103.4 (94)	159.5 (145)	0.65
8	75.5 (23)	55.0 (55)	82.5 (75)	0.67
9	75.5 (23)	77.0 (70)	148.5 (135)	0.52
10	75.5 (23)	38.5 (35)	66.0 (60)	0.54
11	88.6 (27)	93.5 (85)	115.5 (105)	0.81

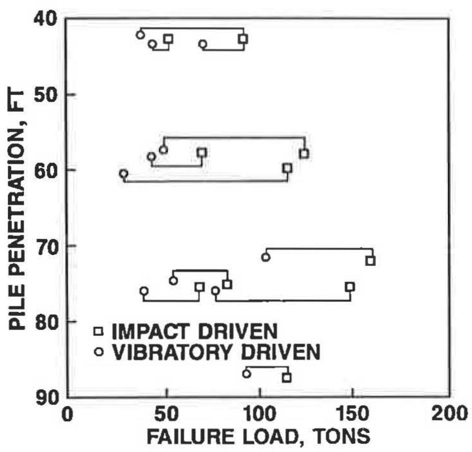

FIGURE 8 Failure load versus depth for the crane rail tracks pile testing program.

SUMMARY OF FIELD TESTS

The piles driven by the impact hammers had a significantly greater axial capacity than those driven with the vibratory hammers. In Figure 9, the failure load for the impact-driven piles is plotted versus the failure load of the vibratory-driven piles. The diagonal line in the plot represents a one-to-one correspondence between the impact and vibratory capacity; points below this line show a greater capacity for impact-driven piles, and points above this line show a greater capacity for vibratory-driven piles. This plot shows that, for the majority of the pile tests examined in this study, the vibratory-driven piles have less axial capacity than impact-driven piles.

REDUCED CAPACITY FOR VIBRATORY-DRIVEN PILES

A possible explanation for the reduced capacity for vibratory-driven piles is that the vibratory driving process results in less compaction at the pile tip, thus lowering the tip capacity. Hunter and Davisson (*8*), in their investigation of the Arkansas River Lock and Dam No. 4 pile tests, explained the difference in capacities by examining the driving process. They state that a vibratory hammer is very effective in overcoming the side resistance or skin friction along a pile in sand, but the very nature of the longitudinal pile vibration requires a small tip force. Therefore, the soil beneath the tip of a vibratory-driven pile remains relatively undisturbed compared with its state before driving. In comparison, Meyerhof (*1*) showed that impact driving in sand results in substantial compaction beneath the tip, which prestresses the surrounding soil mass.

Evidence of this can be found in the Arkansas River Lock and Dam No. 4 pile testing program. Figure 10 presents a plot of the tip load, skin friction, and total pile load at failure for impact-driven piles versus the vibratory-driven piles for Lock and Dam No. 4. The plot reveals that for each comparable set of piles the tip load at failure for the vibratory-driven piles is lower than the impact-driven piles. Even for the set of H piles, the load carried by the tip of the vibratory-driven pile was 14 tons less than the impact-driven pile, even though the vibratory-driven pile had a greater total load at failure.

RETESTS

Further supporting evidence that the tip capacity for vibratory-driven piles is less than for impact-driven piles can be found in the crane rail testing program. To investigate whether a pile previously vibrated into place and then driven a short distance by a drop hammer would achieve the same ultimate capacity as a pile completely driven with a drop hammer, some additional tests were performed in the crane rail testing program. These tests showed that vibratory driven piles with the last 9 ft of penetration driven by a drop hammer had the same failure load as purely impact-driven piles with the same penetration.

As previously mentioned, the axial capacity of the piles at Red River Lock and Dam No. 1 was significantly less than anticipated. In an attempt to investigate the reasons for the reduced capacity, several of the piles were retested. Figures 11, 12, and 13 show plots of the tip load versus displacement for three of the load tests and their retest at Red River Lock

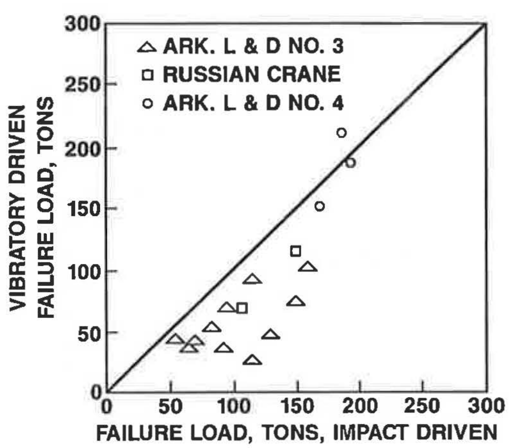

FIGURE 9 Comparison of impact- and vibratory-driven piles.

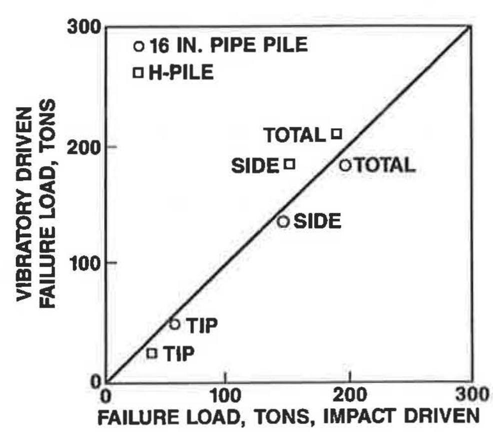

FIGURE 10 Comparison of load distribution of impact- and vibratory-driven piles for Lock and Dam No. 4.

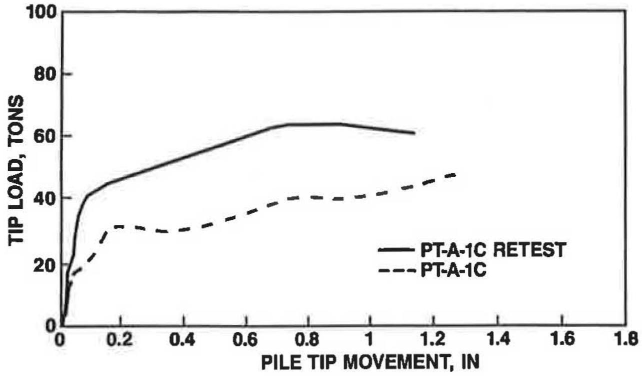

FIGURE 11 Tip load versus pile tip movement for Red River Lock and Dam No. 1 pile test PT-A-1C (*9*).

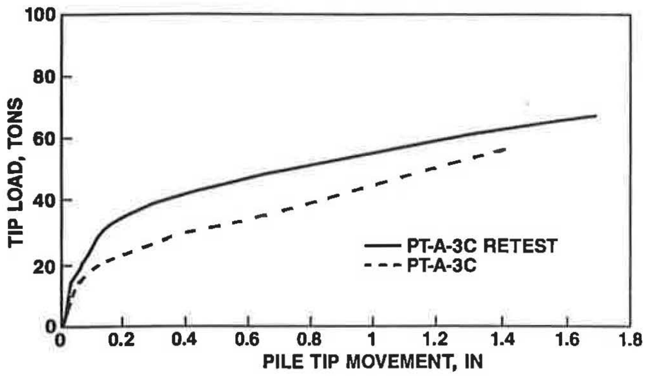

FIGURE 12 Tip load versus pile tip movement for Red River Lock and Dam No. 1 pile test PT-A-3C (*9*).

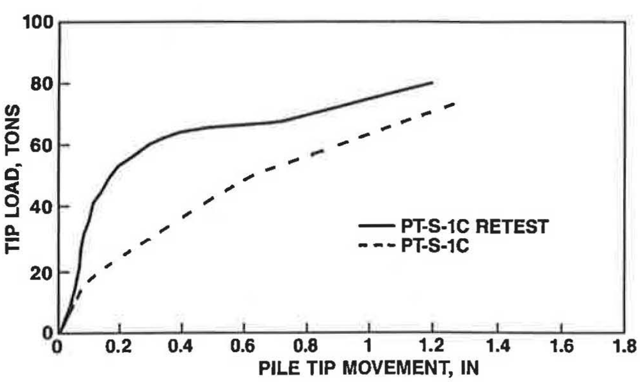

FIGURE 13 Tip load versus pile tip movement for Red River Lock and Dam No. 1 pile test PT-S-1C (*9*).

and Dam No. 1. In these plots, the retests have significantly greater load-carrying capacity than when previously tested. A large portion of the additional capacity exhibited by the retested piles resulted from the compaction of the soil surrounding the tip during the first load tests. The influence of time may also have had a small effect on the increased capacities.

CONCLUSIONS

The results of the field tests presented in this paper show that, for a significant majority of cases, the installation of piles in sand with a vibratory hammer of any type (high or low frequency) resulted in less axial capacity than impact-driven piles at the same site. Additional information was found showing that the influence of time affects piles driven by both methods equally and that additional driving by an impact hammer of a vibratory-driven pile causes an increase in its axial capacity compared with that of a pile driven totally by an impact hammer.

ACKNOWLEDGMENTS

The author would like to express his thanks to the U.S. Army Engineer Division, Lower Mississippi Valley, for the financial support provided for the original investigation. Permission was granted by the Chief of Engineers to publish this information.

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Publication of this paper sponsored by Committee on Foundations of Bridges and Other Structures.