Pile-Head Behavior of Rigidly Capped Pile Group

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The results of a series of vertical load tests on a full-scale group of nine rigidly capped piles and two control piles driven into stiff, saturated clay are described. The scope of the paper is limited primarily to a description of the performance of the piles at their heads; that is, load-settlement and load-distribution behavior and apparent mode of failure. Results from the study that are of practical engineering significance, including group efficiency, settlement ratio, and distribution of loads among the piles, are described, followed by a discussion of test procedures and magnitudes of inherent test errors.

It is generally understood that installing several piles in close proximity to one another alters the stress state and fabric of the supporting soil in a manner different from that produced by installing a single pile, where its synthesized or measured load-settlement response often forms the basis for prediction of foundation performance. Stress overlaps that result from loading the group of piles (mechanical interaction) further tends to produce differences in group and single pile behavior.

Model tests have been used extensively in the past to investigate the relative effects of spacing, penetration, soil properties, and other parameters on the behavior of pile groups. However, physical models fail to replicate effective stress states in the soil that impact on efficiency, settlement, and distribution of load among piles. Full-scale testing eliminates this problem, but the obvious expense of full-scale tests makes them a generally impractical means of conducting parameter studies. Therefore, it is important that maximum use be made of the limited body of full-scale data that does exist. This paper is presented for the purpose of adding to these data.

In the past, full-scale tests have been employed on a limited basis to investigate the effects on group performance of sand density and pile spacing (1), disturbance of sensitive clays (2), pile spacing and group size in soft clays (3,4), combined loads (5), and other phenomena. A recent study sponsored by the Federal Highway Administration (FHWA) (6) undertook the investigation of the behavior of a vertically loaded full-scale group of nominally vertical piles in stiff, insensitive, strain-softening, overconsolidated clay in southeastern Houston, Texas, that contained zones of slickensides, fissures, and sand partings. This paper describes the overall results of load tests on a nine-pile (3x3x3-diameter spacing) group. The test piles were 264-mm (10.75-in) outside diameter by 9.27-mm (0.365-in) wall thickness steel pipes, which were all instrumented and driven closed-ended to a depth of 13.1 m (43 ft) with flush boot plates. Subgroups of five (center and middle edge) and four (middle edge only) piles within the main group (6) were also tested after testing the main group. The piles were connected by a rigid cap suspended off the soil.

Three sets of tests were conducted to failure on the nine-pile group at 20, 82, and 110 days after the piles were driven. These tests were preceded (four or five days) by tests on two single control piles near the test group, which served as references for assessing group efficiency and settlement ratio. Details of the soil profile and testing procedures will be found later in the paper.

LOAD-SETTLEMENT BEHAVIOR: CONTROL PILES AND NINE-PILE GROUP

Load tests were first conducted on the two control piles (piles 1 and 11) 15 days after they were driven. These piles exhibited similar apparent load-settlement behavior, both at the butts and the tips, as shown in Figure 1, except that pile 11 failed suddenly at a lower load than pile 1. The term "apparent" refers to the fact that the tip loads were assumed to be zero prior to loading for purposes of plotting Figure 1. The piles were largely, but not exclusively, friction piles, and sudden plunging occurred after near-linear response.

All of the test piles in the study were driven in 200-mm (8-in) diameter pilot holes 3.1 m (10 ft) deep to assist in maintaining alignment. The pilot hole for pile 1 was dry at the time of driving, while that for pile 11 was partly filled with water. Piezometers on pile 1 indicated that essentially all excess pore-water pressure generated by driving had dissipated by the time of the first load test. Pile 11 had no piezometers, but it is speculated that its low capacity may have been due to undissipated pore pressures associated with driving the pile in a wet hole. Three piles of the main group were also driven in wet holes, so that the average performance of the two control piles is believed to be an appropriate reference for assessing group efficiency and settlement ratio.

By the second test, pile 11 had developed a capacity nearly equal to that of pile 1, which had also developed a slightly higher capacity. The apparent setup between tests 1 and 2 for pile 1 was due almost entirely to an increased tip capacity brought about by the effects of load cycling and of residual tip loads remaining after removal of the earlier test load. Freeze (increased shaft capacity due to pore-pressure dissipation) apparently occurred in the time interval between tests 1 and 2 in pile 11. No further freeze occurred, however, as revealed by uplift testing conducted after completion of compression testing (6).

The strain-softening nature of the soil (relaxation of load during plunging) is evident in Figure 1, as loads, which were reduced after each pile, were pushed beyond the settlement at which peak resistance occurred.

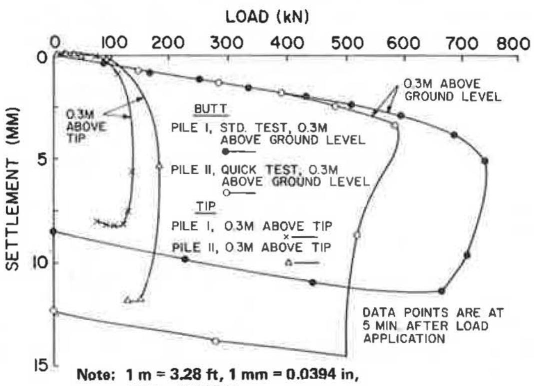
The pile numbering scheme and alignments for the piles in the nine-pile group are shown in Figure 2. (The pile tops were located at the top of the cap.)

Load-settlement behavior of the group during the first load test is shown in Figure 3. This figure does not include the dead weight of the pile cap [254 kN (57 kips)] that was supported by the piles prior to the test. It is observed from Figure 3 that

1. Load-settlement curves based on data taken 5, 30, and 55 min after application of a load increment are essentially linear and coincident to approximately 75 percent of the maximum load, which indicates very low creep rates at working load levels;
2. Settlement across the pile cap was essentially uniform up to about 60 percent of the maximum load, whereafter considerable tilting occurred; and
3. The load-settlement curve, unlike the curves for the control piles, plunged without relaxing.

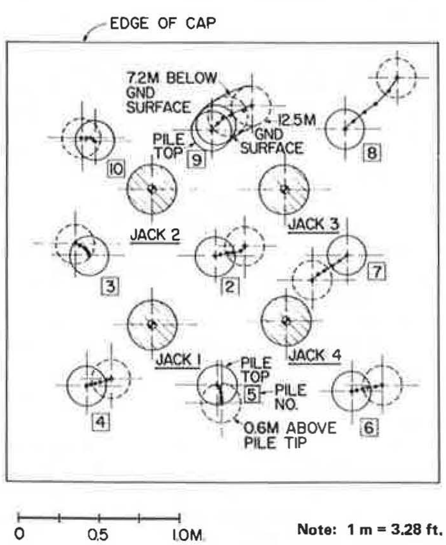
Item 2 is further illustrated by Figure 4, which shows the measured attitude of the cap at the maximum load as measured by triaxial dial gages mounted at the lower corners of the cap. These instruments confirmed rigid cap behavior. The cap pitched toward the north as the articulated beams used as a jacking reaction for the test translated slightly in that direction during the course of the test. The northeastward batter of piles 8 and 9 also caused a clockwise yaw to develop. The pitching was magnified by the fact that pile 8, which by virtue of its position in the laterally translating group and its slight batter attracting more than the average pile load, failed one load increment before the other piles failed.

Figure 1. Butt and tip load-settlement curves for control piles (test 1).



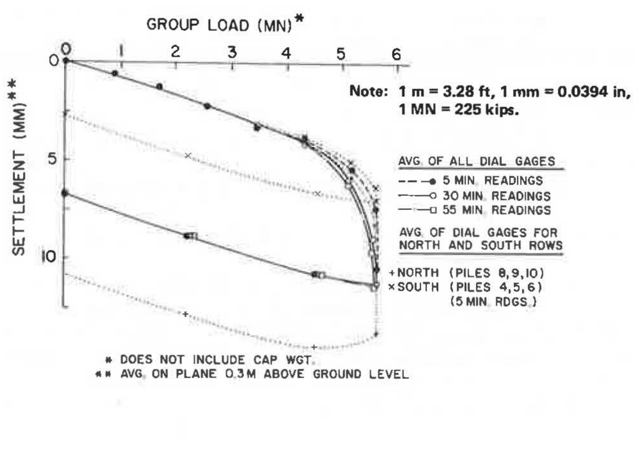
Note: 1 m = 3.28 ft, 1 mm = 0.0394 in, 1 KN = 0.225 kip.

Figure 2. Pile numbering and alignment.



Note: 1 m = 3.28 ft.

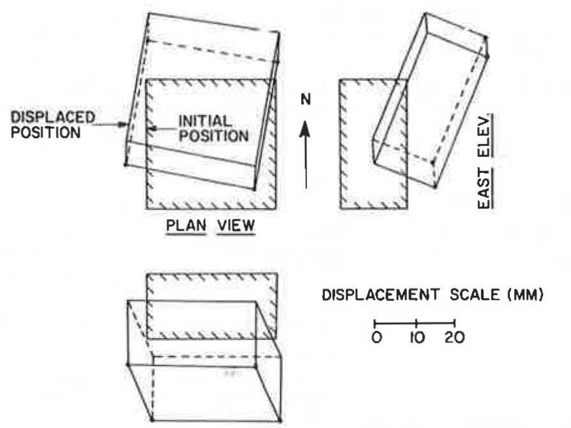
Figure 3. Load-settlement relations for nine-pile group (test 1).



Note: 1 m = 3.28 ft, 1 mm = 0.0394 in, 1 MN = 225 kips.

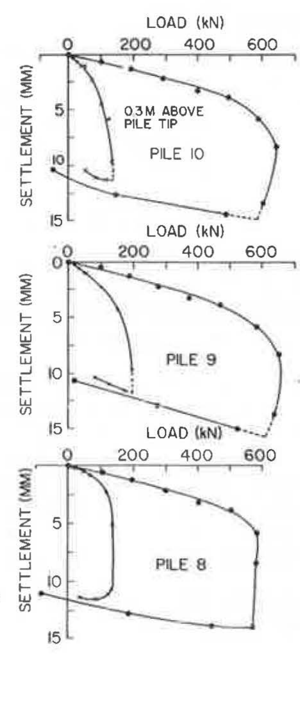
Item 3 can be explained qualitatively by considering the load-settlement curves of the individual piles in the group; typical curves are shown in Figure 5. First, all of the group piles had about the same distribution of side and end loads at failure as the control piles and, like the control piles, failed by plunging. No block action was observed (6). Second, because of the rotation of the cap, the piles on the north row plunged prior to those on the center or south rows. The north row piles then relaxed as the remaining piles continued to attract load; the net effect was that the overall group load-settlement curve became vertical. Piles 4 and 5 did not plunge during this test. Loading was stopped before those piles plunged because the group itself had plunged and because flexural stresses at the pile heads had reached allowable values. These observations are of practical significance because group failure in a strain-softening soil appears to be associated with the load at which the first pile fails and hence with both the symmetry of the piles in the system and the concentricity of the applied load.

Figure 4. Displacement of cap at maximum load (nine-pile group, test 1).



Note: 1 mm = 0.0394 in.

Figure 5. Load-settlement curves for north row of piles.



Note: 1 m = 3.28 ft, 1 mm = 0.0394 in, 1 KN = 0.225 kip.

A more fundamental representation of failure can be seen in Figure 6, in which failed zones along each pile are shown by vertical bars at various loads. In addition to the early failure of pile 8, Figure 6 also shows the manner in which failure (development of peak side load transfer) began and progressed along each of the piles. Progression of failure downward from the surface began at load 5 (83 percent of maximum group load) in most piles, and failure progressing upward from the tips began at loads 6 or 7 (92 or 99 percent of maximum group load). Downward-progressing failure is due to the flexibility of the piles relative to the soil, which causes maximum load transfer to be achieved at the pile tops first. Upward-progressing failure is produced by a shear stress concentration at the pile tips, followed by relaxation of that shear stress as further movement occurs. It is our hypothesis that the point of onset of upward-progressing failure in the first pile (pile 8) marks the limit of the load that could be sustained indefinitely by the group without plunging failure in the strain-softening soil. This load (load 6) is 92 percent of the short-term plunging capacity for this test. Side failure patterns in the control piles were similar to those shown in Figure 6.

The group appeared to gain capacity between tests. This phenomenon, which was also observed for the control piles, was primarily due to increased tip capacity produced by cycling the tip load and not to side resistance setup, as demonstrated by pore-pressure measurements and by tension tests conducted after the compression test (6). Essentially all excess pore-water pressures produced by installation had been dissipated against group piles 2, 3, 4, and 5 (those instrumented for this effect) and in the soil mass around those piles prior to test 1, and load testing produced insignificant pore pressures both in the soil and at the pile and soil interfaces.

INTERPRETATION OF FAILURE

At this point it is appropriate to address the subject of interpreting failure in tests on groups of piles. Several methods of assessing failure loads are illustrated in Figure 7. Curve 1 represents a group that plunges, the vertical tangent to which is the plunging load (P). This load may be an appropriate definition of failure in soils of the type described here if sufficient load can be applied in a test to affect plunging. Curve 2 represents the type of failure that might be expected for piles in granular soils. Plunging is not achieved, but a point is reached on the gross load-settlement curve beyond which a terminally linear branch is observed. This point (TL) can be interpreted as failure in such soils (1).

A rational method that is suggested if neither plunging nor terminal linearity is achieved is construction leading to the group offset (GO) load shown in the upper part of Figure 7. This construction is similar to that suggested for single piles by Davisson (7) but includes a tacit postulation that settlement of pile tips in a group is equal to the square root of the ratio of the width of the group to the width of a single pile times the tip settlement for a single pile. In Figure 7, $\zeta = 0.6$ for a friction pile group and 1.0 for an end-bearing group. The latter term is not a part of the original method proposed by Davisson. Failure of groups may also be defined by traditional methods, such as the point at which 13-mm (0.50-in) net settlement (NS) is realized (curve 3), since group settlements in excess of the settlement of a single pile under the same average load may be largely elastic.

The plunging and group offset loads are identical for this test, and they represent the short-term capacity of the group. Because plunging followed by relaxation occurred, the terminal-linearity method is inappropriate for this test. Load was not cycled, so the net settlement failure load was not obtained. By the former criteria (taking into account the cap weight), the efficiency of the group was 0.98.

An additional method that may be of use in groups of the type tested, in which progressive failure occurs both along and among piles in the group, is to define failure as the load at the accelerated creep point (C) shown in Figure 8, which was proposed by Housel (8) for single piles. Creep settlement (settlement in the last half of each load increment) versus load for test 1 on the nine-pile group is shown in Figure 8. The accelerated creep point falls at 92 percent of the plunging load, which (from Figure 6) is the load at which upward-progressive failure began.

Figure 6. Progressive failure in individual piles (nine-pile group, test 1).

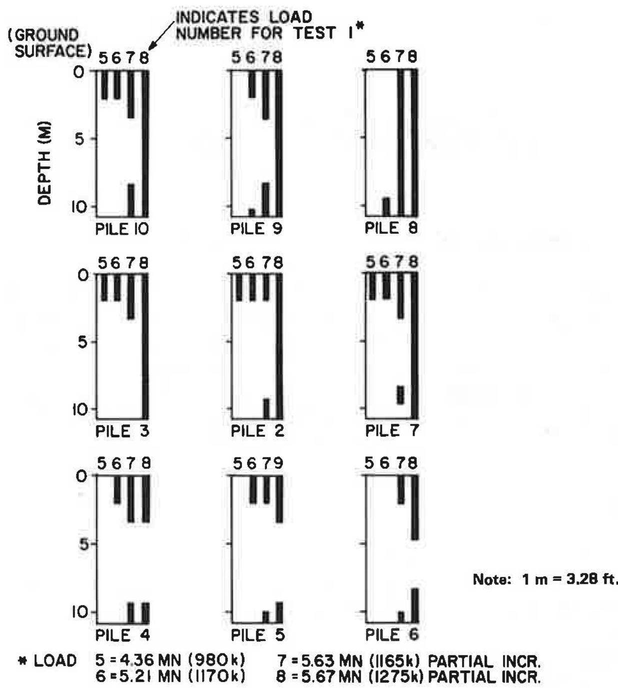


Figure 7. Methods of interpreting group failure.

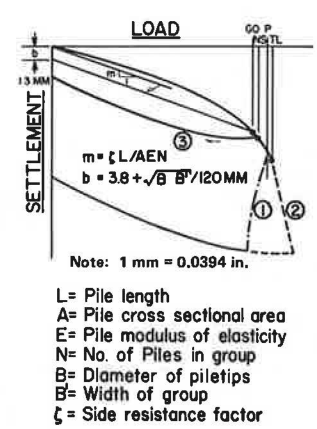
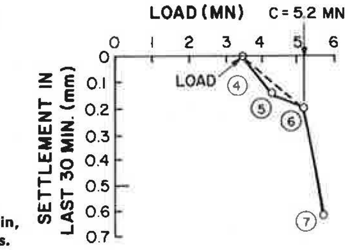


Figure 8. Failure load of nine-pile group (test 1) by creep method.



Note: 1 mm = 0.0394 in, 1 MN = 225 kips.

Figure 9. Load-settlement curves for subgroup tests.

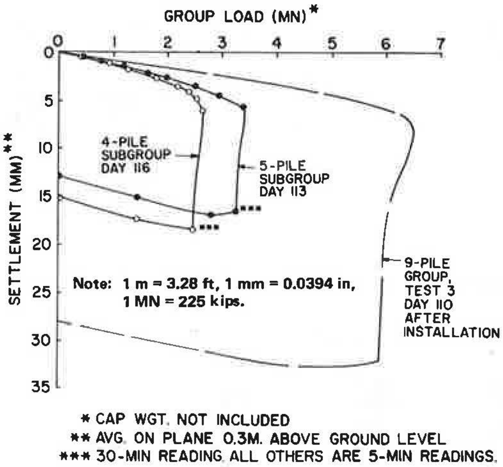


Figure 10. Settlement ratios.

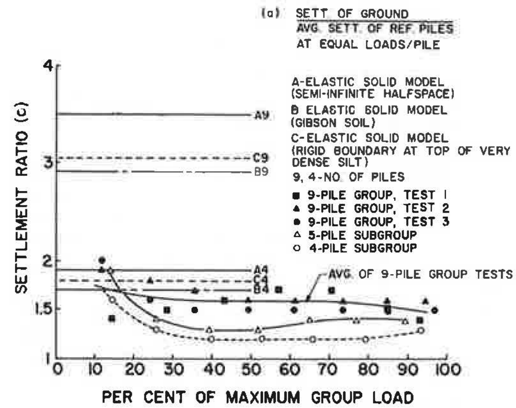
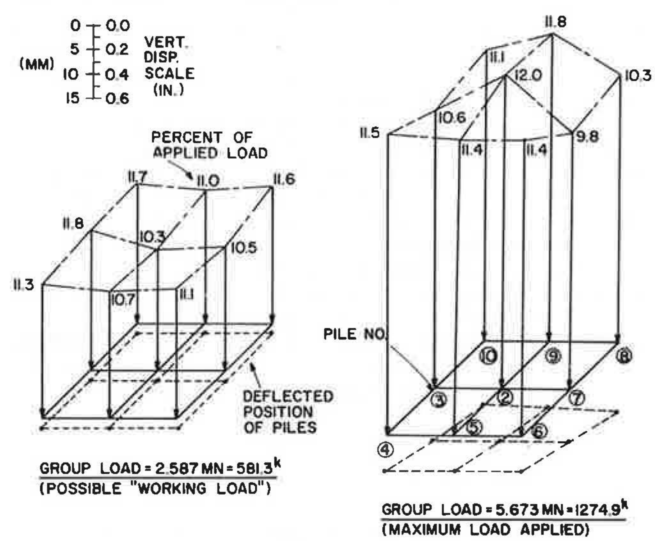


Figure 11. Distribution of loads to piles.



LOAD-SETTLEMENT BEHAVIOR: SUBGROUPS

Immediately following the third load test on the nine-pile group, the subgroup tests were conducted. The load-settlement curves for the final nine-pile test and the two subgroup tests are shown in Figure 9. The average capacity per pile varied little among the various tests, which is evidence that the failure mode was by plunging of individual piles in all tests.

SETTLEMENT RATIOS

The ratios of settlement of the pile group to average settlement of the control piles at a common average load per pile are shown in Figure 10. Figure 10 also shows settlement ratios for these groups, as computed by methods proposed by Poulos and Davis (9) [halfspace and rigid boundary at the top of a layer of very dense silt 20.5 m (67 ft) below grade] and by Banerjee and Davies (10) for a Gibson soil (zero modulus at the surface increasing linearly with depth). These methods all overpredict the settlement ratio; the Gibson soil and rigid boundary models yield results closest to those measured. The differences in computed and observed settlement ratios are believed to be due to the inability of the mathematical models to consider the stiffening effect of the piles on the soil and, to a lesser extent, to errors associated with settlement measurements, which will be described later.

DISTRIBUTION OF LOADS TO PILES

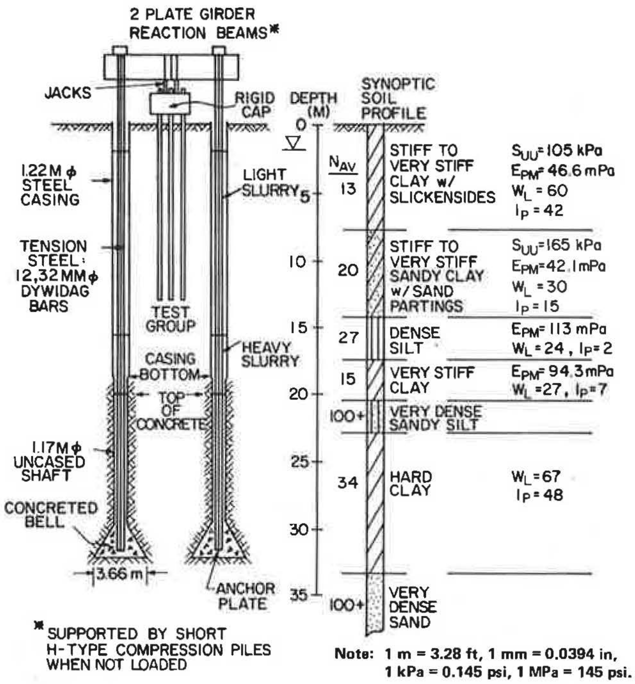
Figure 11 shows the distribution of axial loads to the pile heads and the deflections of the piles at two values of applied load for the first nine-pile test. The pattern of relative load among piles at the subfailure load is basically as predicted by elastic theory (9,10): highest loads at the corners, next highest at the middle edge piles, and lowest at the center pile. However, the measured variation in load magnitude is much lower than predicted by such solutions. The pattern at the failure load is apparently reversed, but these load values do not necessarily represent failure loads for each pile, since the piles failed progressively (not simultaneously) and relaxed thereafter. Similar observations were made in the subgroup tests.

SOIL PROFILE AND TEST ARRANGEMENT

A section that shows the main pile group, soil properties, and reaction system is depicted in Figure 12. The soil properties listed are average undrained shear strength from UU triaxial compression tests (s_{uu}), average Young's modulus from a self-boring pressuremeter (E_{PM}), and Atterberg limits. Average standard penetration test values for the various layers (N_{AV}) are also tabulated. The soil had been overconsolidated by desiccation in the geologic past and then reinundated. The overconsolidation ratio ranged from about 7 at a depth of 5 m (16 ft) to about 4 at a depth of 15 m (48 ft).

Within the context of group action, the most significant soil properties are believed to be (a) increasing elastic stiffness with depth; (b) the presence of a secondary structure network in the strata

Figure 12. East-west section of test group, reaction system, and soil.



Note: 1 m = 3.28 ft, 1 mm = 0.0394 in, 1 kPa = 0.145 psi, 1 MPa = 145 psi.

where the test piles were placed, thereby allowing rapid pore-pressure dissipation; and (c) soil insensitivity. The first property influenced settlement ratios, and the last two influenced efficiencies.

The cap that connected the group piles was a rigid concrete block 1.30 m (4.25 ft) thick and 2.75 m (9 ft) square in plan and was suspended 0.92 m (3.0 ft) off the ground. The test piles were instrumented at their tops for measurement of axial load and deflection with precalibrated strain-gage circuits and dial gages, respectively. The dial gages were suspended from steel reference beams supported on shallow piles 6.1 m (20 ft) from the center of the cap in a direction perpendicular to the section shown in Figure 12. In order to minimize thermally induced movements of the reference system, the test piles were covered by an opaque shroud. Independent measurements of cap deformations in three dimensions were made with 12 dial gages suspended from the reference beams and mounted in three orthogonal directions at the four lower corners of the cap and by microhead survey techniques that use a benchmark outside the zone of influence of the piles and the reaction anchor system.

The pile group was loaded by four hydraulic jacks (Figure 2) acting through load cells and reacting against a plate girder system that was anchored by two deep concrete caissons, each situated laterally approximately 3.66 m (12 ft) from the center of the group. The caissons consisted of 3.66-m-diameter bells and 1.17-m (3.83-ft) diameter shafts of concrete. The concrete was terminated 20 m (66 ft) below grade to restrict stresses produced by the anchor caissons in the zone of soil below the pile tips. The anchor holes were cased above that depth, except for a small gap that was provided to prevent the concrete from engaging the casings and producing shear stresses around the casings.

The caissons were connected to the reaction beams by means of tension bars placed through tubes in the concrete to prevent bonding and anchored at the bases of the bells to introduce load at the bottoms of the anchors. This detail further served to restrict anchor system stresses in the soil around the test group.

The jacks were positioned so their centroid was as near the anticipated center of reaction from the piles as possible, and the reaction beams were set over the jacks for the first load test. This caused the anchor bars to be slightly out of plumb (possibly due to minor misplacements of the anchor plates in the bells), so that when the group was loaded, the reaction beams tended to translate to the north, as described earlier.

The control piles were located about 4 m (13 ft) on either side (north and south) of the main group and were each loaded with single hydraulic jacks reacting against beams anchored by a system of four H-piles embedded 7.6 m (25 ft); each was situated 3.5 m (11.5 ft) symmetrically from the test pile. Settlements were measured, as with the group, by using the group reference system.

TESTING PROTOCOL

The testing program consisted of the following tests:

1. Simultaneous load tests of the two control piles 15 days after installation. Pile 1 was loaded by using the standard procedure described below. Pile 11 was loaded in increments of about 130 kN (29 kips) every 2.5 min until failure was achieved and then unloaded in two decrements.
2. Test of the nine-pile group 20 days after installation by using the standard procedure.
3. Simultaneous load tests of the two control piles 78 days after installation, followed in 4 days with a second test on the nine-pile group. Standard loading procedures were used in this and the following steps.
4. Repeat of 3 above at 105 and 110 days, respectively.
5. Test of the five piles, which consist of piles 2, 4, 6, 8, and 10 of the original group, after detaching the heads of the other piles from the cap, at 113 days after installation.
6. Test of the four piles, which consist of piles 4, 6, 8, and 10, after detaching pile 2 at 116 days after installation.

The standard procedure for loading consisted of the application of increments of about 12 percent of the anticipated failure load every hour until failure occurred, followed by a 1-h hold, followed by unloading in three decrements. Instruments were read at 5, 30, and 55 min after application of the load increment. All electronic strain-gage circuits and load cells were read and processed into engineering units in real time by a microcomputer data-acquisition system that was capable of scanning all circuits, including 99 circuits in the piles, in about 60 s. All dial gages and survey instruments were read manually.

SETTLEMENT ERROR ANALYSIS

The two primary sources of error in the measurement of settlement were thermally induced strains in the reference system and displacements of the test piles and reference beam supports produced by loading the anchors and the piles. The first problem was studied experimentally (6, Appendix E), and it was determined that the maximum displacement error excursion during the approximately 12-h period of a test was 0.13 mm (0.005 in). The use of the shroud over the test piles and the restriction of testing to low-temperature-differential overcast days helped to minimize this effect.

The effects of anchor and pile loads were ad-

Table 1. Summary of test results.

Test[a]	Failure Mode[b]	Plunging Load[c] (MN)	Efficiency			Settlement Ratio
			Shaft[d]	Tip[d]	Overall	
Control 1 (15 days)	P	0.747				
Control 11 (15 days)	P	0.591				
Nine pile (20 days)	PI	5.92	0.98	≈1.04	0.99	1.62
Control 1 (78 days)	P	0.831				
Control 11 (78 days)	P	0.756				
Nine pile (82 days)	PI	6.80	0.99	1.33	0.98	1.54
Control 1 (105 days)	P	0.787				
Control 11 (105 days)	P	0.804				
Nine pile (110 days)	PI	6.85	0.90	1.40	0.96	1.48
Five pile (113 days)	PI	3.70	0.89	1.15	0.93	1.31
Four pile (116 days)	PI	2.94	0.94	0.81	0.92	1.19

Note: 1 MN = 225 kips.

[a] After driving.
[b] P = plunging and PI = plunging of individual piles in groups (no block failure).
[c] Includes weight of cap and loading accessories.
[d] Apparent.

dressed analytically and experimentally (6, Appendix E). Four separate phenomena were considered analytically:

1. Upward movement of the pile tips (assumed equal to the movement of the pile tops) due to soil stresses produced by the group anchor reactions,
2. Upward movement of the reference beam supports [5-m (16.4-ft) deep H-piles 6.1 m (20 ft) north and south of the center of the test group] due to group anchor reactions,
3. Upward movement of the reference beam supports due to soil strains induced by H-pile anchors during the control pile tests, and
4. Downward movement of the reference anchors due to loads from the test piles themselves.

The first three phenomena were approached by using Mindlin's equation for the case of upward-directed point loads in a semi-infinite elastic mass (11). The mean depth of load transfer in the group anchors was assumed to be at 27.5 m (90 ft) below grade, the elastic modulus of the elastic mass (soil) was taken to be 103 MPa (15 000 psi) based on deep-seated pressuremeter test results, and the soil was assumed to be incompressible. Similar conditions were assumed for phenomenon 3, except that the mean depth of load transfer was assumed to be 4.6 m (15 ft) below grade in the H-pile anchors. The net effect of phenomena 1 and 2 yielded theoretical settlements of the piles with respect to the reference beams in the nine-pile group that were 6 percent low at one-half of the group plunging load.

Phenomenon 4 was approached by first computing the downward movement of the small piles that supported the reference beams by employing the elastic solid model proposed by Poulos and Davis (9), which uses the soil properties described above. (The same model was also used to compute the downward movements of the unloaded piles during the subgroup tests, which were measured relative to the displacements of the loaded piles.) The computed reference beam support settlements were then multiplied by the ratio of the observed to computed settlements of the unloaded piles to arrive at a corrected reference beam support settlement for both the group and control pile tests. This correction was considered necessary because the model that was used overpredicted the relative settlements of the unloaded piles in the subgroup tests.

The addition of the effect of phenomenon 4 to phenomena 1 and 2 for the group yielded a net error of 19 percent in settlement measurement in the nine-pile group (on the low side) at one-half of the maximum group load. Independent survey measurements made on the pile cap indicated an error of 9 percent on the low side with a statistical probable error of ±25 percent in the survey data. (The low quality of the survey data is associated with rainy weather conditions during the group load tests.)

The addition of phenomena 3 and 4 for the control pile tests resulted in essentially zero error for those piles. Therefore, it appears that the settlement ratios in the nine-pile group tests may actually be as much as about 20 percent higher than reported, with lesser errors for the subgroup tests.

CONCLUSIONS

Table 1 summarizes the test results in terms relevant to the designer, i.e., efficiencies and settlement ratios, the latter at one-half of the average plunging load for the control piles in a given test. Shaft and tip efficiencies for all tests are also given. Note that overall efficiencies are not weighted averages of shaft and tip efficiencies, since shaft and tip failure was not simultaneous.

The following observations are made:

1. The efficiencies of the group and subgroups were essentially 1.0. The most significant reasons for this fact are that the piles failed as individual piles (no block failure occurred) and that the soil was insensitive and contained a secondary structure network that allowed dissipation of excess pore pressures within a few days after the group piles were driven.
2. The settlement ratios were lower than those predicted by elastic theory, possibly because of the effect of pile reinforcement on the soil.
3. The distribution of load was generally uniform among the piles, although at about one-half of the maximum load interior piles carried slightly less load than the corner piles.
4. Failure was progressive. This fact suggests that long-term group capacity under concentric or eccentric loading can be evaluated from short-term tests by using the creep failure method suggested in Figure 7.

ACKNOWLEDGMENT

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Pile Foundation—From Preliminary Borings to Production Driving

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Foundation design for a major bridge structure requires extensive field and office investigation. The design process undertaken for the Arrowhead Bridge, which carries US-2 between Superior, Wisconsin, and Duluth, Minnesota, over the St. Louis Bay, is presented. Subsurface investigation results, geologic studies, pile load tests, wave-equation analysis, and dynamic pile testing are presented. Results of the geotechnical investigations allowed the use of high-capacity piles in soil for the bridge foundation. Subsurface conditions consisted of soft lacustrine and glacial clay deposits over dense glacial outwash sands. Depths to the underlying dense strata ranged from 130 to 260 ft (39.6-79.2 m) across the site. Six load tests were performed on steel H-piles and cast-in-place type piles. Maximum loads of 344 tons-force (3060 kN) were applied by using both maintained-load (ML) and constant-rate-of-penetration (CRP) methods. Load test results are presented by using five interpretative techniques, and comparisons between ML and CRP methods are shown. Wave-equation analyses were performed by using the WEAP computer program, and results are compared with driving records. Dynamic pile analysis was done by using the Goble-Case Western pile driving analyzer (PDA), and pile capacity predictions are compared with load test results. The PDA was also used on production piling, and experiences while analyzing the very long piles for capacity and damage are discussed.

The design of a foundation for a major bridge project requires a progression through various stages of literature review, field investigations, and office interpretation and evaluation. This paper presents details of the design process and resulting construction experience for the Arrowhead Bridge in Superior, Wisconsin.

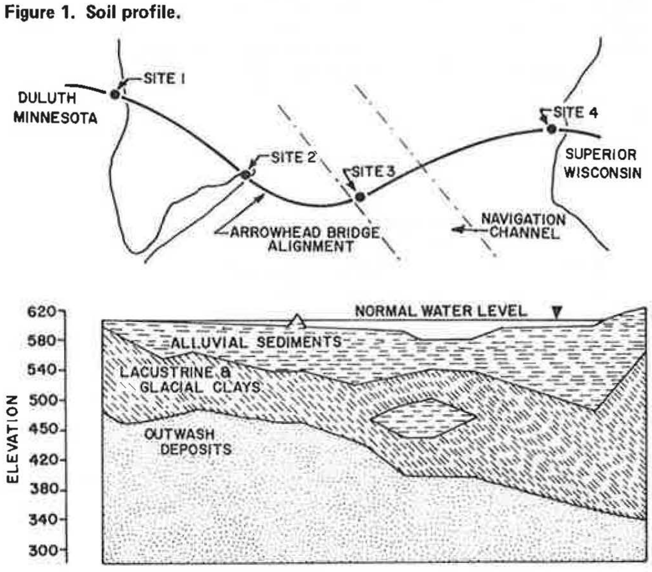
The new Arrowhead Bridge is to be some 8400 ft (2.56 km) in length and will carry US-2 between Superior, Wisconsin, and Duluth, Minnesota. Located at the western tip of Lake Superior, the new high-level structure will span St. Louis Bay, the harbor shipping channel, a number of railroad tracks, and Interstate 35 in Duluth. The curved and skewed alignment, as shown in Figure 1, was necessary to provide navigational clearances for a harbor bend down channel yet meet desired connection points in Superior and Duluth. The bridge will provide a horizontal clearance of 500 ft (152.4 m) and a vertical clearance of 120 ft (36.6 m) at the channel.

The geotechnical investigation consisted of a literature review, three separate and progressive phases of subsurface investigation, and a pile load test program. The final foundation design was determined from results of these investigations, and construction was started. Foundation work on three of the four substructure contracts is essentially complete; so far, no major problems have developed.

SITE CONDITIONS

The foundation investigation for the structure began

Figure 1. Soil profile.



BRIDGE ALIGNMENT AND GENERALIZED SOIL PROFILE