# COMPARISON OF SINGLE- AND MULTIPLE-UNDERREAMED BORED PILES BASED ON LABORATORY AND FIELD EXPERIMENTS

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Bored piles are used extensively to meet foundation requirements in some soils. The maximum diameter of the belling tool for underreaming is limited to approximately 3 times the diameter of the shaft. This limitation causes an uneconomical design in some soils because of the resulting low stress in the concrete of the shaft. An increase in the area of that portion of the pile in point-bearing increases the bearing capacity of the pile. A method of increasing this effective area is to cut a bell at other locations along the shaft of the pile, in addition to the bell at the point. This paper describes the laboratory and field tests that were performed so that the load-carrying capacity of single- and multiple-underreamed bored piles could be determined. Bell spacings and failure planes in the soil of the model piles were studied extensively. The results of the model tests were used in the test design of the field piles. An analysis of all field test results was made by using the full area of both the bell in point-bearing and perimeter of the shaft in skin friction. Calculated load-deflection behavior using these assumptions compare reasonably well with the load-deflection values obtained by experimentation. Multiple belling of cast-in-place piles can result in economical designs for many soil conditions.

•STRUCTURAL foundation requirements for many soil conditions are satisfied by the use of cast-in-place concrete piles. Enlarging the bottom of the pile by underreaming or belling is one method of increasing the point resistance. The maximum diameter to which a bell can be cut is limited to approximately 3 times the diameter of the shaft because of size limitations of the belling tool. Many designers neglect the skin friction of belled piles; the resulting designs often involve shafts so large that the maximum stress in the concrete is less than 10 percent of the ultimate stress. This is uneconomical design because of the additional cost of excavation and concrete. Because it is nearly impossible to increase the shear strength of the soil, the alternative is to increase the relative areas of point to shaft of the pile. One method of accomplishing this is to underream at other elevations along the shaft in addition to the point as shown in Figure 1. This paper describes the laboratory and field tests that were performed so that the load-carrying capacity of various types of bored piles could be determined.

## LABORATORY TESTS

Laboratory tests were conducted on models to determine the relative bearing capacities of the single- and double-bell piles. The top bell of the model could be moved along the shaft so that bell spacing could be varied. The model pile consisted of a 1-in. diameter steel shaft with a 2½-in. diameter steel bell or bells. The material used for the soil was a mixture of ground, oven-dried Yazoo clay with 20 percent petroleum jelly

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added to replace the moisture and make the clay workable. This material remained consistent throughout repeated uses during the testing program. Static compaction yielded a material of constant density on which triaxial compression tests were run to determine the cohesion and friction angle. Load deflection curves were plotted from the data obtained from all model pile tests. The ultimate bearing capacity of each model pile was defined by using the tangent method. Table 1 gives the results of the average of 3 tests for each bell spacing.

Figure 2 shows a plot of the results given in Table 1. The double-bell pile produced a maximum increase of 48 percent in ultimate capacity over the single-bell pile. The optimum spacing between the bells of the double-bell pile corresponding to this maximum increase was found to be equal to, or greater than, 2 bell diameters.

After approximately 1-in. total deflection of the pile, the soil was carefully removed to allow visual observation of the failure planes that developed during the test. Figure 3 shows a typical failure surface obtained upon complete failure of the double-bell pile with 1.2- and 1.6-bell diameter spacings. It is evident that the increase in ultimate capacity of this pile is derived from the cylindrical shearing pattern that develops between the 2 bells. This cylinder is approximately equal in diameter to the diameter of the bells.

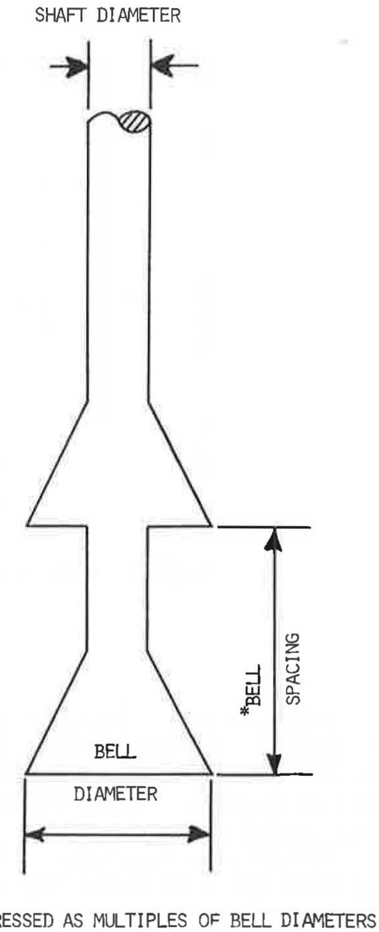


Figure 1. Double-bell pile.

Figure 4 shows a failure surface for one of the tests using a 2.0-bell diameter spacing. It appears that the 2.0-bell diameter spacing is the transition between the cylindrical shearing pattern and the pattern shown in Figure 5. The failure surfaces shown in Figures 5, 6, and 7 are typical of the bearing capacity failures derived by Prandtl, Terzaghi, and others. Therefore, the additional capacity of the double-bell pile using 2.4- and 2.8-bell diameter spacings is derived from the point-bearing capacity of the top bell.

The results obtained from the model tests indicated a sufficient increase in the capacity of a multiple-bell pile to justify conducting a series of full-sized pile tests.

TABLE 1

ULTIMATE CAPACITY OF MODEL TEST PILES

| No. of Bells | Bell Spacing (diameter) | Load (lb) | Percent Increase |
|---|---|---|---|
| 1 | – | 6,640 | – |
| 2 | 1.2 | 7,860 | 18.4 |
| 2 | 1.6 | 8,900 | 34.0 |
| 2 | 2.0 | 9,830 | 48.0 |
| 2 | 2.4 | 9,830 | 48.0 |
| 2 | 2.8 | 9,830 | 48.0 |

## FIELD LOAD TESTS

The site selected for the field tests was on the right-of-way of Interstate 220 in the northwest section of Jackson, Mississippi. Soil borings and subsequent laboratory tests indicated the following: the top 18-ft is a stiff, brown, silty clay; the next 12 ft is a stiff, tan, highly desiccated, weathered Yazoo clay; and the next 30 ft is a hard, blue, unweathered Yazoo clay. The soil investigation terminated

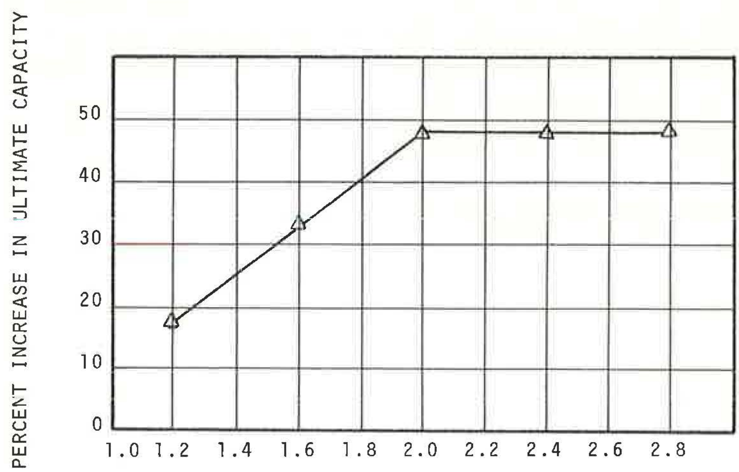


Figure 2. Increase in ultimate capacity of double-bell model piles over single-bell model piles.

at a depth of 60 ft. Figure 8 shows the plan view of the test site and the location of the test piles, the reaction piles, and the borings. Repetitive triaxial and direct shear tests were run on the undisturbed soil samples at approximately 5-ft intervals for the full 60-ft depth. Figure 9 shows a typical boring log on which cohesive shear strengths are graphically presented.

Fifteen piles were cast: 7 test piles and 8 reaction piles. Figure 8 shows that the geometrics of the layout were based on the maximum utilization of each reaction pile. The reinforcing steel in each reaction pile was arranged in such a manner that the piles could be used for 2 or more load tests. Each reaction pile was structurally designed to carry the predicted ultimate failure load of the test pile. All piles were cast of 5,000 psi concrete. The test piles had a shaft diameter of 12 in. and contained four No. 8 bars to their full depths except that pile 13 contained seven No. 11 bars to its full depth. Table 2 gives the physical properties of the test piles and the steel in each.

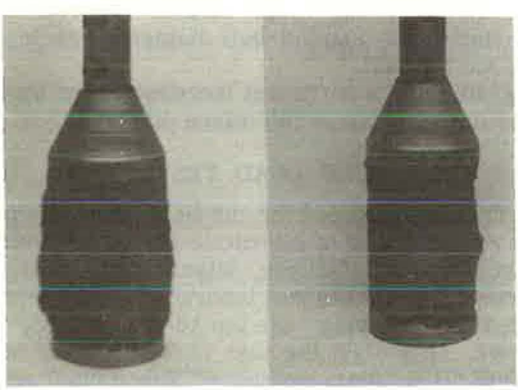

Figure 3. Failure patterns for double-bell piles with 1.2- and 1.6-bell diameter spacing.

Figure 4. Failure pattern for double-bell pile with a 2.0-bell diameter spacing.

Figure 5. Failure surface for 2.4-bell diameter spacing.

Figure 6. Failure surface for 2.8-bell diameter spacing.

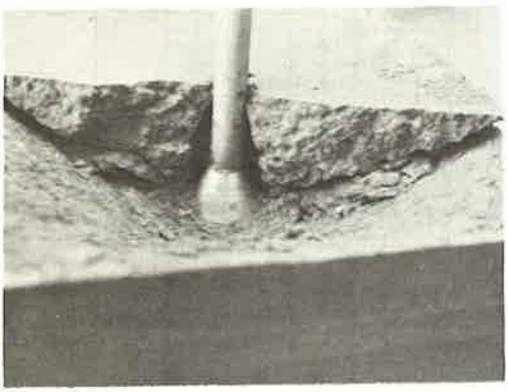

Figure 7. Failure plane for top bell for a 2.8-bell diameter spacing.

Each reaction pile as constructed with a 24-in. diameter shaft, 60-in. diameter bell, and 32-ft embedment. Figure 10 shows the profile of a typical loading arrangement.

Construction of the test piles began June 25, 1969, and was completed July 1, 1969. Construction of bored piles consisted of drilling the shaft with an auger to the desired tip elevation; the auger was then removed and the belling tool was lowered to form the bell. The cutting blades were activated by the weight and force of the drill stem that caused the blades to swing out and gradually cut the bell. The double-bell pile was constructed by advancing the shaft to the elevation of the base of the top bell; the belling tool was then used for con-

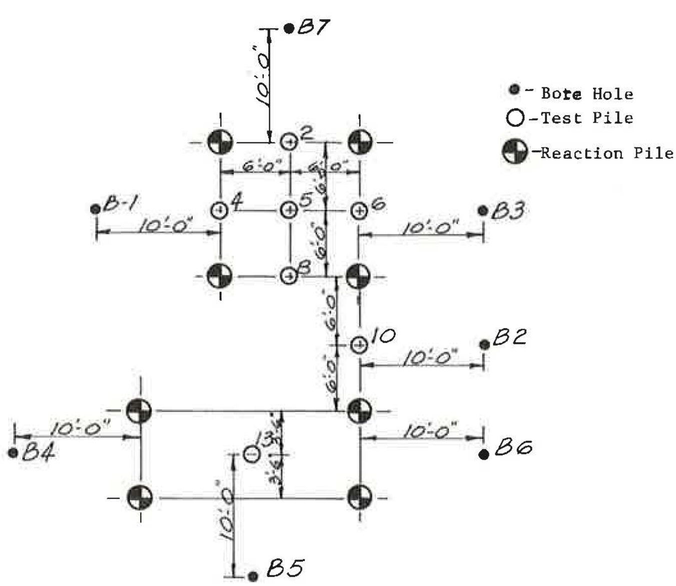


Figure 8. Test pile and boring location plan.

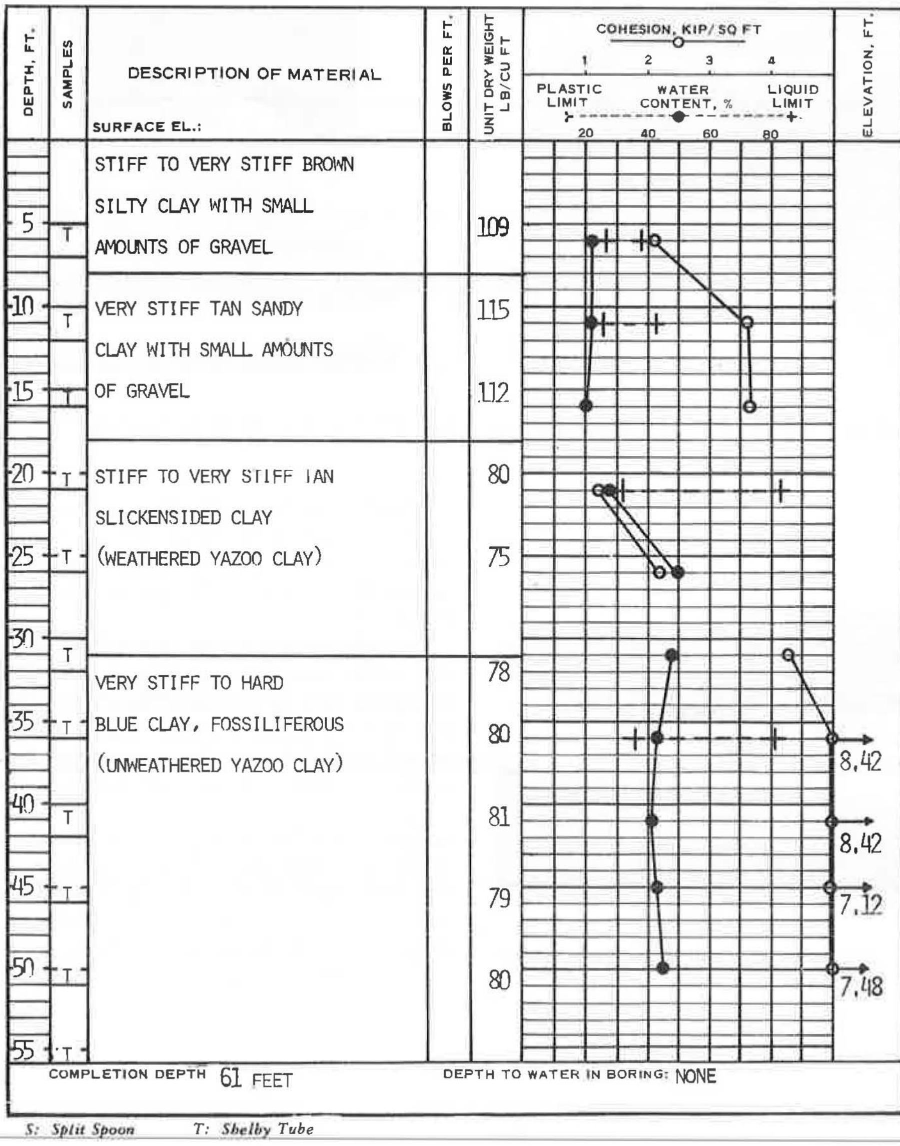


**Figure 9. Typical boring log.**

TABLE 2

SUMMARY OF TEST PILE DESIGNS

| Pile | Shaft Diameter (in.) | No. of Bells | Embedment Length (ft) | Steel Bars | Ties |
|---|---|---|---|---|---|
| 2 | 12 | 1, 36 in. | 39 | 4, No. 8 | No. 3 at 12 in. |
| 4 | 12 | 2, 24 in. at 4 ft | 39 | 4, No. 8 | No. 3 at 12 in. |
| 5 | 12 | No point | 37 | 4, No. 8 | No. 3 at 12 in. |
| 6 | 12 | 2, 24 in. at 8 ft | 39 | 4, No. 8 | No. 3 at 12 in. |
| 8 | 12 | Straight shaft | 39 | 4, No. 8 | No. 3 at 12 in. |
| 10 | 12 | 1, 24 in. | 39 | 4, No. 8 | No. 3 at 12 in. |
| 13 | 12 | 2, 36 in. at 4 ft | 38.5 | 7, No. 11 | No. 3 at 6 in. for top 6 ft and 3 at 12 in. lower 6 ft |

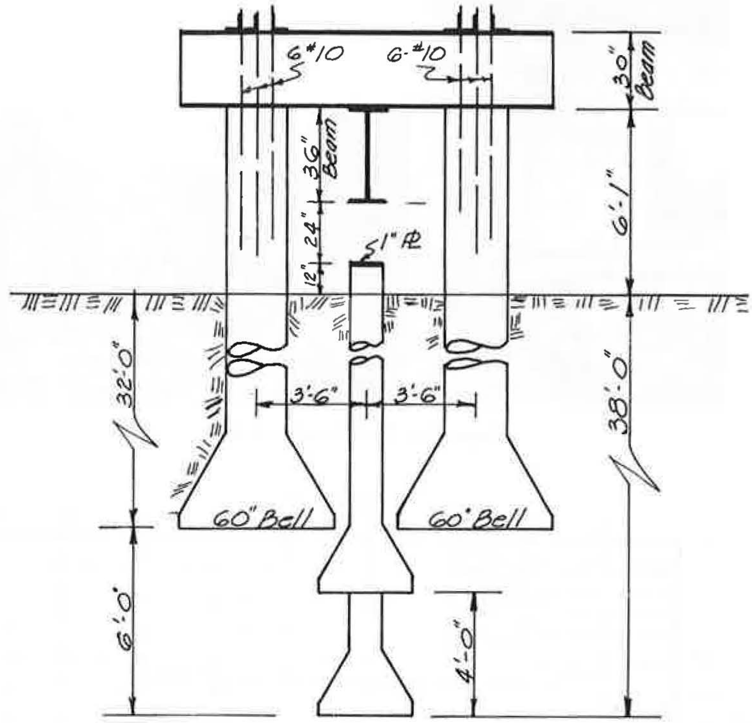


Figure 10. End profile of loading arrangement on pile 13.

struction of the top bell. The shaft was advanced to the elevation of the bottom bell, and then the bottom bell was reamed. The reinforcing steel then was placed, and the concrete was poured into the shaft, vibrated to eliminate voids, and finished at cutoff elevation.

The reference frame for measuring pile deflections was constructed of aluminum beams. Dial gages calibrated in 0.001 in. were used to measure the deflections. The loads were applied and maintained with a hydraulic jacking system. Figure 11 shows the load test on pile 13 in progress.

The approximate ultimate load capacity of each pile was computed, and the test loads were applied in increments of 25 or 50 kips, depending on the predicted ultimate load for each test pile. For each increment of loading, time-deflection readings were recorded at 0, 1, 2, 4, 8, and 15 and every 15 minutes thereafter for the duration of the loading increment. Each load was maintained until the deflection substantially ceased, or until at least 3 points formed a straight line in the lower portion of the time curve. Figure 12 shows the results of the time-loading data for pile 8. These results are typical for all piles tested.

Based on the time deflection data, equilibrium points for each loading increment were defined. These load deflection data for each test pile are given in Table 3.

## ANALYSIS OF DATA

All piles, regardless of pile material, size, or shape, depend on both point-bearing and skin friction for their load-carrying capacities. However, many designers sometimes neglect either one or the other of these 2 components, depending on soil conditions or pile type. For comparative analysis of the capacity of the piles in this investigation, both components were used.

The computed load deflection values were determined by an iteration process, which accounts for the deformation of the pile itself. For computational purposes, each test

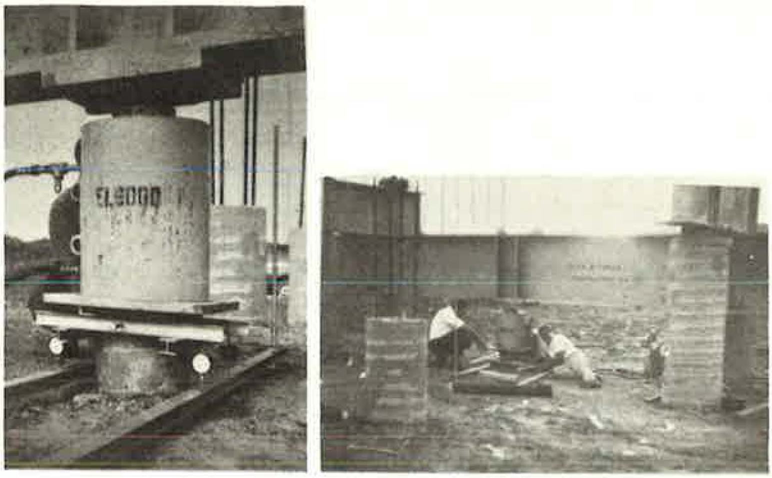


**Figure 11. Field loading test.**

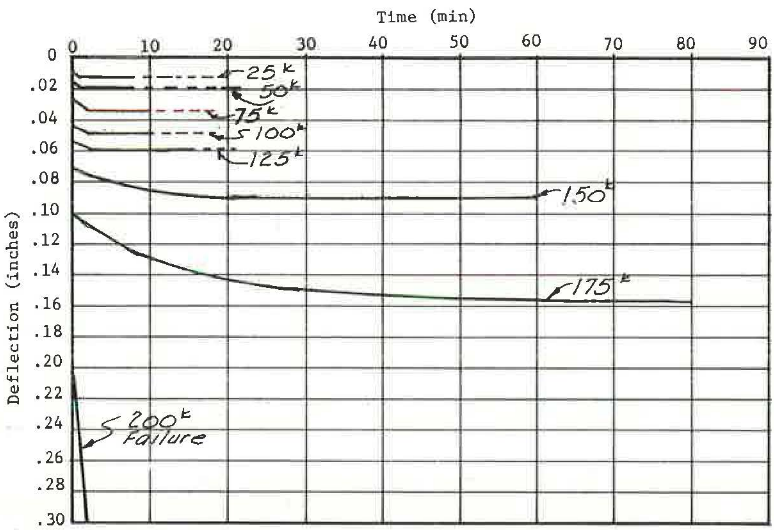


**Figure 12. Time-deflection values for pile 8.**

TABLE 3

TEST PILE DEFLECTIONS

| Load (kip) | Pile 2 | Pile 4 | Pile 5 | Pile 6 | Pile 8 | Pile 10 | Pile 13 |
|---|---|---|---|---|---|---|---|
| 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 |
| 25 | | | 0.011 | | 0.011 | 0.010 | |
| 50 | 0.017 | 0.016 | 0.026 | 0.018 | 0.018 | 0.020 | 0.019 |
| 75 | | | 0.039 | | 0.034 | 0.029 | |
| 100 | 0.041 | 0.039 | 0.062 | 0.058 | 0.046 | 0.038 | 0.039 |
| 125 | | 0.051 | 0.185 | 0.111 | 0.060 | 0.069 | |
| 150 | 0.072 | 0.110 | 1.809 | 0.491 | 0.088 | 0.149 | 0.069 |
| 175 | | 0.269 | | 1.143 | 0.159 | 0.329 | |
| 200 | 0.144 | 0.502 | | 1.964 | 0.962 | 0.542 | 0.135 |
| 225 | 0.218 | 0.751 | | | | 0.766 | |
| 250 | 0.351 | 1.005 | | | | 1.006 | 0.247 |
| 275 | 0.640 | | | | | | |
| 300 | 0.987 | | | | | | 0.419 |
| 325 | 1.343 | | | | | | |
| 350 | 1.678 | | | | | | 0.593 |
| 375 | | | | | | | |
| 400 | | | | | | | 1.049 |
| 425 | | | | | | | |
| 450 | | | | | | | 1.664 |
| 475 | | | | | | | 2.025 |
| 500 | | | | | | | 2.490 |

Note: Values are in inches.

pile, excluding pile 13, has a transformed section area of 128 sq in. and perimeter of 37.7 in. Pile 13 has a transformed section area of 168 sq in. The modulus of the concrete was taken as 1,000 $f_c'$ or 5,000,000 psi. This value is very close for the range of stresses applied to the top of the pile during testing.

The determination of the relationship between the point-bearing and point deflection utilized the undisturbed triaxial test results of the undisturbed sample nearest the pile tip. The confining pressure was taken as equal to the overburden pressure at the tip and the point stress as equal to the deviator stress. These values were substituted in the elastic deformation equation to obtain the final results. The relationship between skin friction force and deformation was computed by utilizing the remolded direct shear test results. The direct shear tests were run with the normal stress equal to the overburden stress at the location at which the sample was taken. The variation of shear stress on the face of the pile was assumed to be linear among soil sample locations. It was also assumed that the full shear strength of the soil was developed between the soil and the face of the pile, i.e., no reduction was made for adhesion.

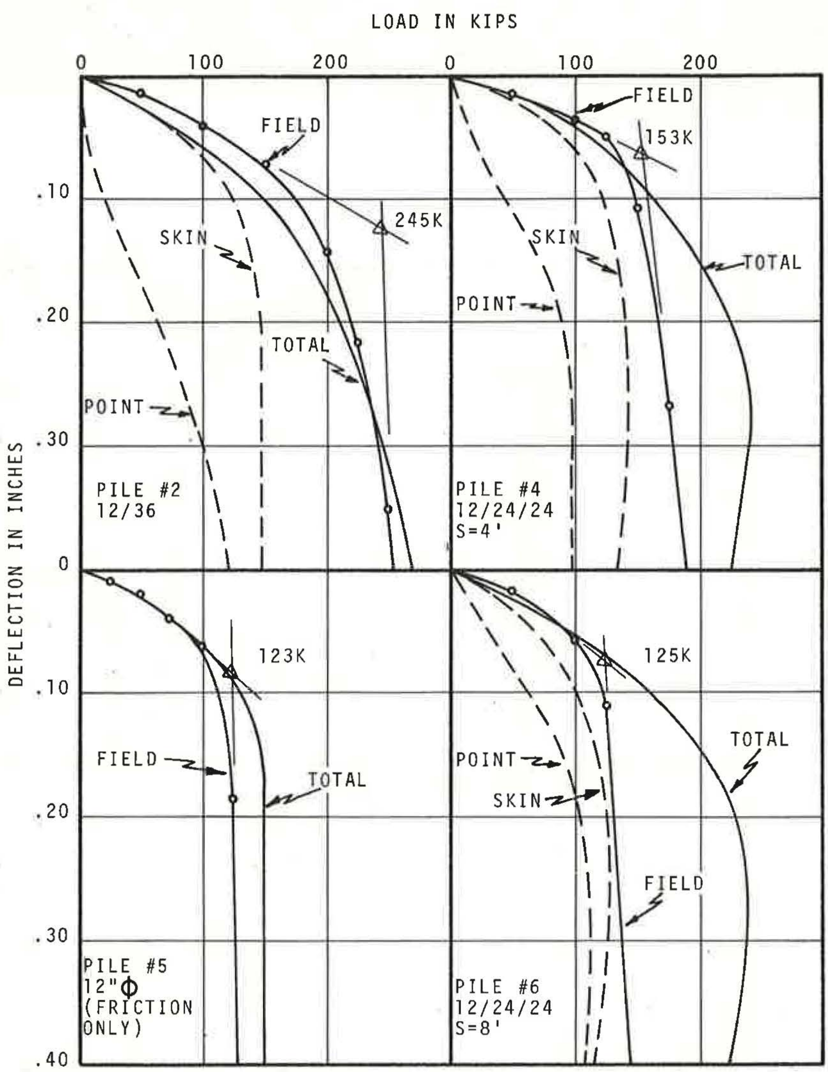


Figure 13. Computed and measured load deflection curves for piles 2, 4, 5, and 6.

Test pile 5 was cast on top of a 1-ft thick spacer that eliminated the point-bearing; therefore, only the skin friction was used to compute its bearing capacity. Test pile 8 was a straight shaft with end-bearing. The bearing capacity of this pile was computed by adding the point-bearing for the 12-in. diameter point to the skin friction for the entire length of the pile. The capacity of each single-bell pile, 2 and 10, was computed by using the area of the point and the skin friction from the top of the bell to the surface of the ground. The capacity for each double-bell pile was computed by using the point-bearing of both bells, the skin friction between the 2 bells, and the skin friction from the top of the upper bell to the surface of the ground. It is recognized that slightly different capacity values could be obtained with different assumptions.

Figures 13 and 14 show both the computed values and the field test results. On these figures, the computed values have been divided into the components of point-bearing and skin friction for study convenience. The dashed lines are the components, while the solid line is the sum of the 2 values. The field test results are shown as open points.

The ultimate capacity of each test pile was determined by the tangent method. These results are shown in Figures 13 and 14 as triangles.

The iteration method, which was used to determine the theoretical loading deflection curves, agreed well with the load test results throughout the elastic range of deflection. The theoretical and field curves began to diverge in the plastic range of deflection.

A comparison of the ultimate capacities of pile 2 (12/36) and test pile 13 (12/36/36) shows an increase of 47 percent. Therefore, a significant increase in ultimate capacity was realized by the addition of the second bell.

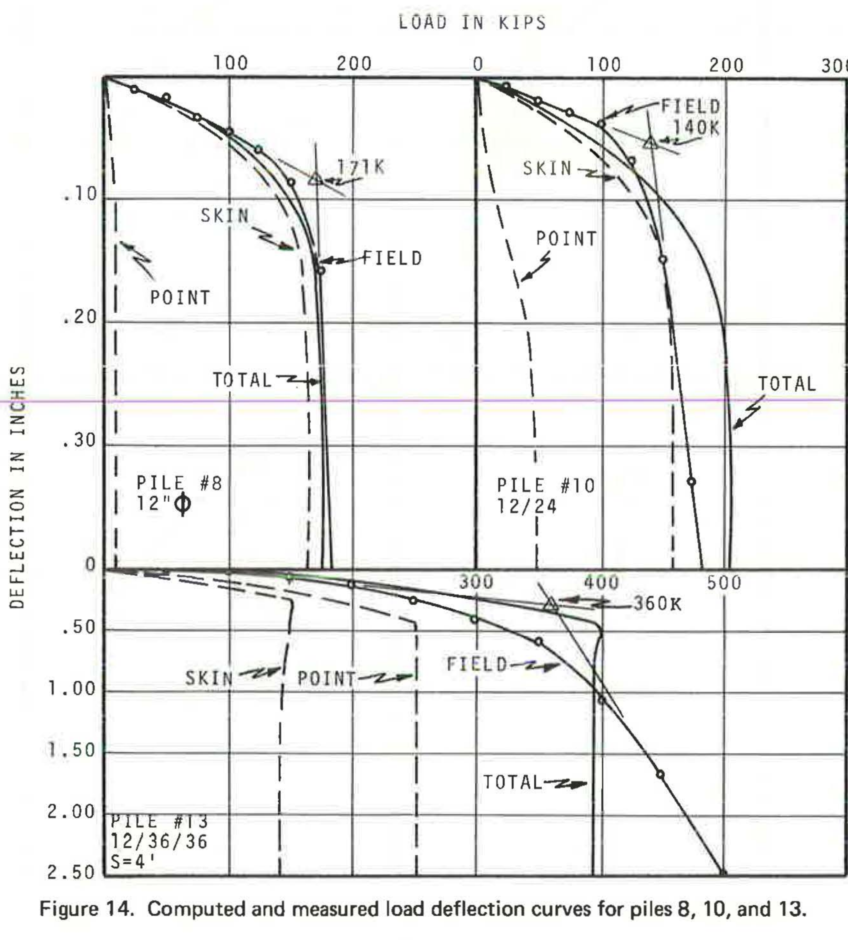


Figure 14. Computed and measured load deflection curves for piles 8, 10, and 13.

The ultimate capacities of all test piles having 24-in. bells fell in the range of 125 to 153 kips. These capacities are considerably lower than the 171-kip ultimate capacity of the straight shaft test pile (pile 8). It appears that the use of small bell-to-shaft ratios, in this case 2:1, may be detrimental to the ultimate capacity of the pile. Additional research must be conducted to properly evaluate this phenomenon.

## SUMMARY

Because of the scope of this pilot research project, no firm conclusions can be drawn at this time. From the foregoing presentation, however, the following observations are made:

1. The assumptions made in calculating the bearing capacity of the piles seem to be reasonable. However, the relative accuracy of computation of the straight shaft with point-bearing as compared to the bell piles indicates that a better relationship would result with different assumptions. It appears that the belling operation alters the effect of skin friction for some distance above the top of the bell. This effect could be the result of prestressing the soil above the bell by the hydrostatic force of the concrete prior to hydration.
2. The remolded strength of the clay should be used in determining the frictional component of cast-in-place piles as it is for other piles. This remolded strength allows for disturbance caused by the auger and the increase in moisture content of the soil in contact with the wet concrete.
3. The 24-in. diameter bell on the 12-in. shaft did little to improve the capacity; in fact, it appears that this small ratio of bell-to-shaft diameter may be detrimental to the ultimate capacity of the pile.
4. For the category of piles having a bell-to-shaft ratio of 3:1, an increase of 47 percent in ultimate capacity was realized by the addition of the second bell. This increase in capacity, considering the small cost of the additional bell, points out that the double-bell pile may be a very economical design. Based on the knowledge of the shear patterns that were developed during the model test phase of this project, it appears that the only limitation on pile capacity is the number of bells that could be placed along the pile shaft.
5. It is recommended that additional research be conducted to fully develop design criteria for this type of pile.

## ACKNOWLEDGMENTS

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