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ABSTRACT

The data measured in this study suggest that the increase in compressive pile capacity for a 42-ft long, HP14x102 pile, in a predominantly fine-grained Missouri River alluvium soil profile, increases by about 16 percent from days seven to forty-four after driving. It appears evident that Davisson's (1973) failure criteria seems to agree fairly well with the observed plunging failure of two compressive pile-load failure tests performed in Missouri River alluvium. By comparison of compressive proof and failure tests performed on day forty-four after driving, it appears that loading a pile to some degree prior to failure, and then reloading the pile, has almost no affect on the load-settlement relationship. Hence, proof loaded piles in Missouri River alluvium that pass should be allowed for use beneath the structure. Finally, comparisons of tension and compression pile load test data have lead to two possible conclusions. First, the estimation of tip load by tell-tale data may not be accurate, and may underestimate the amount of load actually transferred to the tip. And second, it seems viable that, at this site, the skin friction that can be counted on in design is perhaps 55 to 60 percent of that calculated for compression.

INTRODUCTION

Evidence of an increase in pile capacity with time is a well established and documented phenomenon that occurs primarily when piles are driven into fine-grained soils, but also, for less obvious reasons, when piles are driven into granular soils. However, the amount of increase, commonly termed "set-up" or "freeze", is not a well-understood parameter and is usually prohibitively expensive to attempt to quantify. The nature of soil set-up appears to be extremely site-specific and most certainly dependent upon the geologic origins of the local stratigraphy. This paper documents the measured increase in pile capacity with time as determined from axial static compressive load tests. This paper also documents the relationship in behavior between a pile exposed to compressive loading and an identical pile exposed to tensile loading.

Site Location and Project Description

Piles were driven on the south side of the city of Riverside, which is located in Platte County, Missouri. The United States Army Corps of Engineers is constructing a 6.2-mile long flood protection levee that includes 3 rolling gate, 4 stoplog gap, and 2 sandbag gap closure structures. The levee alignment is generally adjacent to the Missouri River, approximately 6 miles northwest of downtown Kansas City, in an area where the behavior of driven piles with time is very rarely documented, if at all. The site is approximately 1/8 of a mile east of the Missouri River bank.

The majority of the closure structures at this job site will be founded on deep foundations requiring the installation of a significant number of driven piles. It is of the utmost importance to establish a safe and economical pile design to support these critical structures. Therefore, the authors have attempted in this paper to quantify the increase in pile capacity with time at this site in order to better understand and estimate the actual capacity of the driven pile foundations during the life of the structure.

SUBSURFACE CONDITIONS

The general stratigraphy of the site consisted of five different soil stratums. Beginning at the existing ground surface, Stratum I is an approximately 10-ft thick, fine-grained, engineered fill. Stratum II is an approximately 13-ft thick layer of low-plasticity clay. Beneath Stratum II is an approximately 17-ft thick layer of high-plasticity clay that makes up Stratum III. Stratum IV consists of an approximately 6-ft layer of low-plasticity clay with intermittent sand seams. Stratum V is composed of silty sand. The range of soil properties measured in the respective stratums is tabulated on the following page in Table 1.

Table 1. Measured Soil Properties.

Stratum	Liquid Limit	Plastic Limit	Plasticity Index	-#200[A] (%)	ω^B (%)	N^C (blows/ft)
I	39	17	22	81	20	--
II	35-40	18-20	13-20	81	28-39	7-8 (8 average)
III	78-101	24-32	54-69	--	44-68	2-9 (5 average)
IV	26-38	21	5-17	72-90	32-38	3-13 (8 average)
V	--	--	--	--	--	24-29 (27 average)

Notes: [A] – Percent Finer than the #200 Sieve (0.074 mm);
[B] – In-Situ Moisture Content; and
[C] – Standard Penetration Test Blow Counts.

PILE DRIVING HISTORY

The performance, drivability, and to a certain degree, capacity, of driven piles are, among other things, dependent upon the pile type, hammer type, and driving resistance of the pile. These three unique aspects of pile design were identified for this particular job and are discussed in detail in the following sections.

Pile Type

The piles tested were steel, Grade A572-50, HP14x102 shapes. This type of H-pile has a cross-sectional area of 30.0 in.^{2}, a depth of 14.01 in., a width of 14.785 in., and a web and flange thickness of 0.705 in. Young's modulus of elasticity, E, of the steel was assumed to be 29,000 ksi in the analyses.

Hammer Type

The hammer used to drive the test piles and subsequent production piles was an open-ended, single-acting, diesel hammer (MKT DE33/30/20C). The ram weight was 3,300 lb and the maximum stroke distance was 10 ft. Accordingly, the hammer used to drive the piles had a rated energy of 33,000 ft-lb. However, at the time of driving, it was observed that the cycle of the hammer was only producing about an 8 ft stroke per blow.

Pile Driving Records

The two test piles investigated in this study were both driven on 23 September 2002. Both test piles were observed at the time of driving, and the hammer blows per foot of driving were recorded. The length of the pile before driving was measured, the ground surface elevations both before and after driving were determined, and the final pile cut-off elevation was determined after driving. This enabled the determination of embedded pile length and also allowed any heave to be identified. Both the compression and the tension piles had an embedded length of approximately 42 ft, and no heave was identified around either pile.

The final penetration rates of both the compression and tension test piles were approximately 5 blows/in. and 4 blows/in., respectively. The compression and tension test pile driving records are shown below in Fig. 1.

Because the same low-displacement pile type, driven with the same hammer, in the same soil profile, on the same day was used, comparison of the compression and tension pile driving records shows very little variation. It should be noted that the two test piles were driven approximately 12.5 ft apart, or, with a separation of approximately 10 pile diameters.

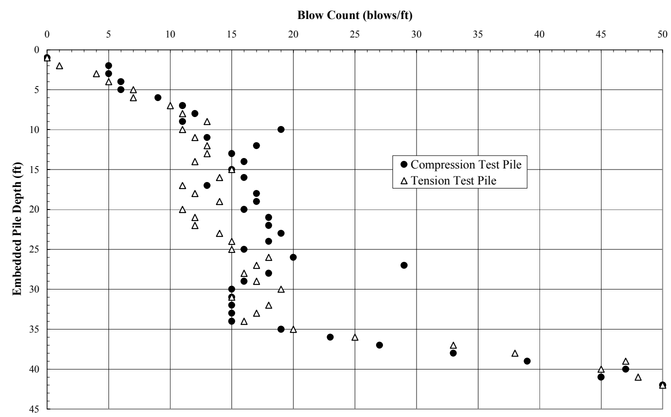

Fig. 1. Pile Driving Records for Compression and Tension Test Piles.

Note that at the completion of pile driving, a void was present on both sides of the pile between the flanges of the cross-section. The measured void extended from the ground surface to about 13 ft below the ground surface for the compression test pile and about 14 ft below ground surface for the tension test pile.

TEST SET-UP

The compressive and tensile load testing procedures and the respective test set-ups are discussed in following sections. A schematic plan view depicting the reaction systems and the test piles is shown in Fig. 2.

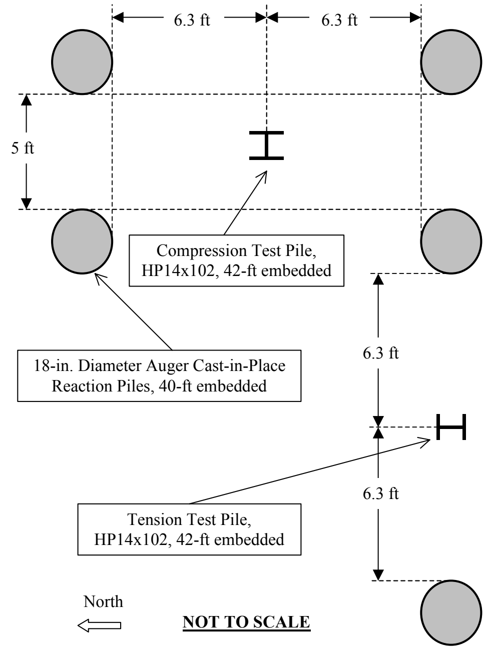

Fig. 2. Schematic Plan View of Test Piles and Reaction Piles

Fig. 3. General View of the Compression Test Apparatus and Set-Up (note that one additional dial gauge measuring pile head deflection and the tell-tale dial gauge are not visible in this photograph).

Compression Testing Procedures and Set-Up

The compression testing was performed in general accordance with ASTM D 1143, "Piles Under Static Axial Compressive Load", using Paragraph 5.6, *Quick Load Test Method for Individual Piles*. Essentially, for the proof tests, the load increments were held for a period of approximately 2.5 minutes up to 200 percent of the design load. The 200 percent design load increment, and all of the unloading increments were held for a period of approximately 5 minutes.

Pile head deflections were measured with four independently supported dial gauges that measured movements to 0.001 inch. The dial gauges measured deflection at each corner of the mounting plate on top of the test pile. Deflection of the tell-tale, which terminated near the pile tip, was measured with a single, separate 0.001 inch dial gauge. The four, 18-in. diameter, 40-ft long auger cast-in-place reaction piles were also monitored with single, separate 0.001-inch dial gauges. A photograph of the compression test set-up is shown in Fig. 3.

Compression proof tests were terminated either when continuous jacking was required to maintain the test load or upon achieving 200 percent of the design load. When continuous jacking was required to maintain the test load, the pile was assumed to be in a state of plunging failure. The compression failure test was also terminated when continuous jacking was required to maintain the test load.

On 30 September 2002, seven days after driving, the compression pile was proof tested to 200 percent of the design load. On this date, the pile experienced plunging failure when exposed to the 200 percent design load. The same pile was again proof tested on 6 November 2002, forty-four days after driving and thirty-seven days after the first compression test. The second proof test achieved the required 200 percent design load without failing, as determined by Davisson's (1973) method, and was unloaded. After unloading the test pile upon completing the second proof test, the pile was reloaded to plunging failure.

Tension Testing Procedures and Set-Up

The tension testing was performed in general accordance with ASTM D 3689, "Individual Piles Under Static Axial Tensile Load"; however, the loading increment time used was similar to that used for the aforementioned compression testing. For the tensile test, the load increments were held for a period of approximately 2.5 minutes up to 200 percent of the design load. The 200 percent design load increment, and all of the unloading increments were held for a period of approximately 5 minutes.

Pile head deflections were measured with four independently supported dial gauges that measured movements to 0.001 inch. The dial gauges measured deflection at each corner of the H-pile. The two, 18-in. diameter, 40-ft long, auger cast-in-place reaction piles were not monitored during the tension test because of the negligible movements measured during the compression test. Photographs of the tension test set-up are shown in Fig.s 4 and 5.

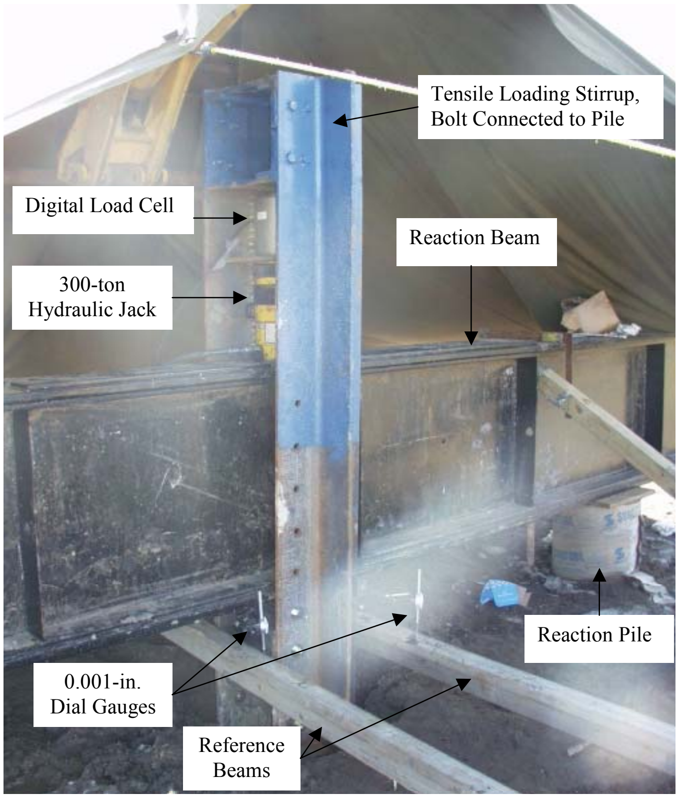

Fig. 4. General View of the Tension Test Apparatus

The tension test, which was a 200 percent design load proof test, was performed on 7 November 2002, forty-five days after driving. The test pile easily achieved 200% of the tensile design load, and was unloaded.

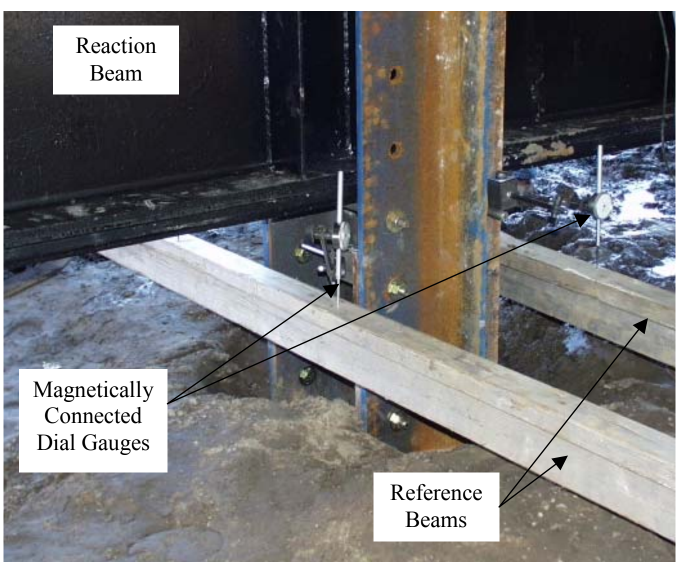

Fig. 5. Tension Loading Connection (note that two additional dial gauges are on the other side of the reaction beam and thus are not visible in this photograph).

RESULTS

The soil profile at the site consisted of approximately 46 ft of fine-grained soil over sand; the embedded pile lengths at this location were approximately 42 ft. One of the two test piles driven at the site was loaded in compression to plunging failure at 7 days. The same pile was then proof tested to 200 percent of the design load, unloaded, and then reloaded to plunging failure at 44 days. Another pile, driven directly adjacent to the aforementioned compression test pile was proof tested to 200 percent of the design load under tensile loading conditions. The loads that were applied to the test piles, as well as the criterion used to estimated failure are discussed in detail below, along with the results of both load tests.

Compressive and Tensile Design and Test Loads

The computed design load for the compression pile was 127 kips. Accordingly, the 200 percent proof test load was 254 kips. The computed design load for the tension pile was 34 kips. Accordingly, the 200 percent proof test load was 68 kips.

Failure Criterion

Davisson's (1973) failure criterion was used throughout this study. However, it should be noted that only half of the elastic compression term was used because the majority of the load was carried in side friction. The equation used to estimate failure for the compression tests is shown below in Equation 1.

$$s_f = 0.5(P_{HEAD}L/EA) + 0.15 \text{ in.} + (D_b/120) \qquad (1)$$

where: s_f = settlement at failure (in.),
P_{HEAD} = load imposed on the pile head (kips),
L = pile length (in.),
E = Young' modulus of elasticity (assumed to be 29,000 ksi for steel),
A = cross-sectional area of the pile (30 in^2.), and
D_b = the diameter of the pile tip (assumed to be 14 in.).

It was estimated from the tell-tale data that only about 10 percent of the load was transferred to the pile tip. The pile tip load was calculated assuming a constant unit skin friction using Equations 2 and 3 (Fellenius, 1969), which are shown below. The load carried in side friction was calculated using Equation 4 (Fellenius, 1969).

$$P_{AVG} = (\Delta L/L_o)EA \qquad (2)$$

where: P_{AVG} = average side friction transferred (kips),

ΔL = change in pile length (in.), and
L_o = original pile length (in.).

$$P_{TIP} = 2(P_{AVG}) - P_{HEAD} \tag{3}$$

where: P_{TIP} = load transferred to pile tip.

$$P_{SIDE} = P_{HEAD} - P_{TIP} \tag{4}$$

where: P_{SIDE} = load transferred in side friction.

It should be noted that less than 0.08 in. of deflection was measured during the tension proof test; hence, no failure criteria was identified or required because of the minimal movements. Failure as determined by Equation 1, minus the $D_b/120$ end-bearing term, would have required 0.17 in. of movement, more that twice what was measured.

Compression Test Results

It should be noted that the seven day compression test was originally planned to be a 200 percent proof test, but because of the plunging failure on the last loading increment, it was considered to be a failure test. The load-settlement curve for the seven day compression test is shown in Fig. 6.

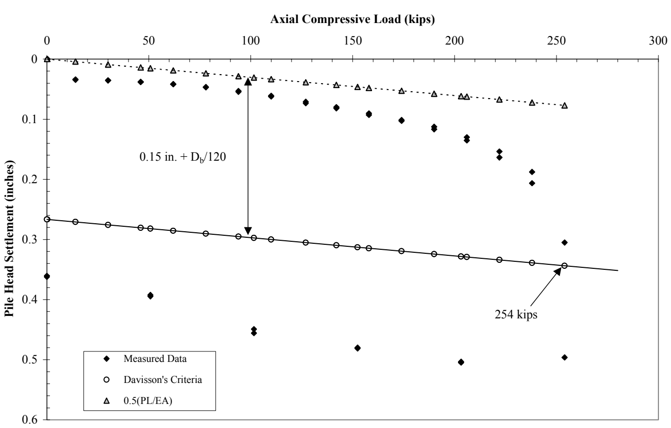

Fig. 6. Load-Settlement Curve for Seven Day Compression Test.

Using the formula presented in Equation 1, the pile failed at a load of 254 kips seven days after driving. Note that if the full elastic compression term, PL/EA, were used instead of half, 0.5(PL/EA), the computed failure load would still be 254 kips. This phenomenon exists because at that load, the pile was experiencing plunging failure.

Thirty-seven days after the first compression test, the same pile was again proof tested. The load-settlement curve for the proof test that was performed forty-four days after pile driving is shown in Fig. 7. Because this test was only a proof test, no failure load was computed. Hyperbolic methods to estimate pile capacity from proof test data were evaluated and found to be inaccurate when compared to the failure test data shown in Fig. 8.

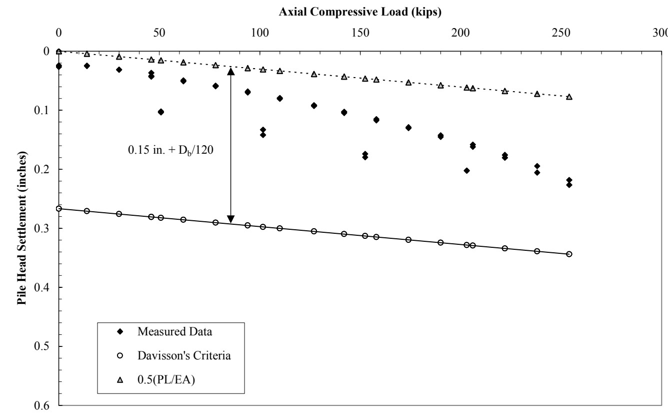

Fig. 7. Load-Settlement Curve for Forty-Four Day Compression Proof Test (note that failure was not achieved).

Upon completing the forty-four day proof test, shown in Fig. 7, the same pile was reloaded to failure as shown in Fig. 8. Using the formula presented in Equation 1, the pile failed at a load of 294 kips forty-four days after driving and thirty-seven days after the first failure test. Note that if the full elastic compression term, PL/EA, were used instead of half, 0.5(PL/EA), the computed failure load would still be 294 kips. Again, this phenomenon exists because at that load, the pile was experiencing plunging failure.

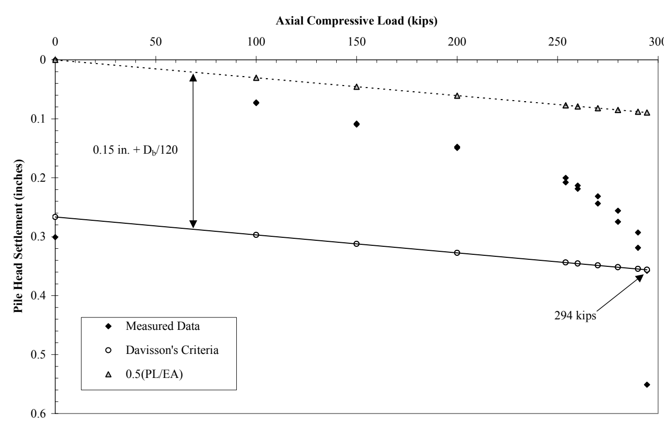

Fig. 8. Load-Settlement Curve for Forty-Four Day Compression Failure Test.

In Fig. 9, the two load tests that were performed on the same pile forty-four days after driving are compared by plotting both load-settlement curves on the same graph. There appears to be very little affect, if any at all, on the load-settlement relationship when the same pile is reloaded to failure the same day that it was proof tested to 200 percent of the design load.

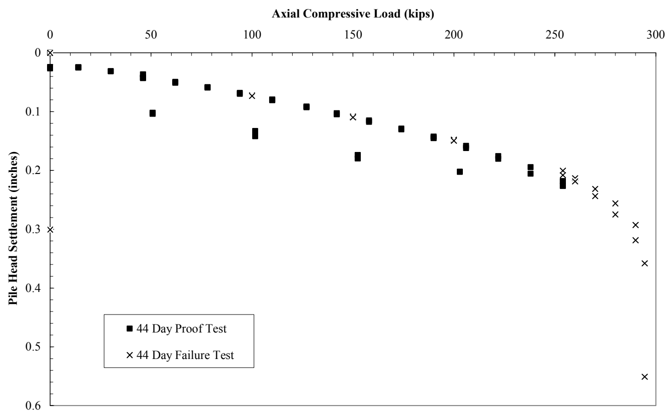

Fig. 9. Comparison of Pile Load Tests Performed Forty-Four Days after Driving.

The load-settlement curves for the two compression tests that were taken to failure, on the same pile, are shown together in Fig. 10. From days seven to forty-four after driving, the pile capacity increased from about 254 kips to 294 kips, an increase of about 16 percent. Because the soil profile is predominantly composed of fine-grained soils, with assumed low permeabilities, the capacity of the pile may continue to increase for quite some time as excess pore pressures generated during impact pile driving continue to dissipate. However, the increase in pile capacity between days seven and forty-four was initially expected to be greater than 16 percent.

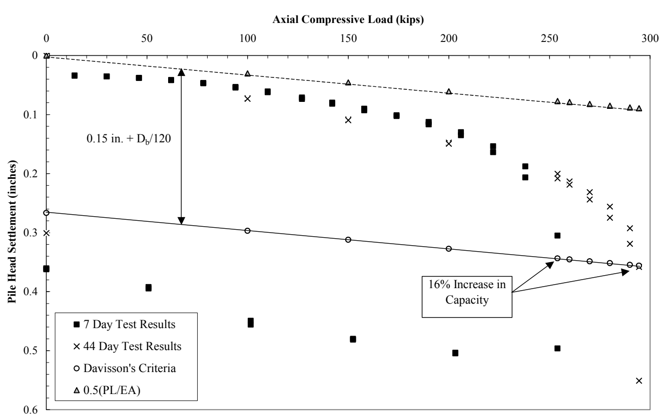

Fig. 10. Comparison of Compressive Pile Load Tests taken to Failure.

Tension Test Results

The 200 percent tension proof test, which equated to a test load of 68 kips, was performed on an identical pile to that which was tested in compression. The measured deflection of the tension pile when exposed to the 68 kip test load was 0.08 in. The load-deflection curve measured during the tension test is shown in Fig. 11, along with the load-settlement curve measured during the forty-four day compression proof test.

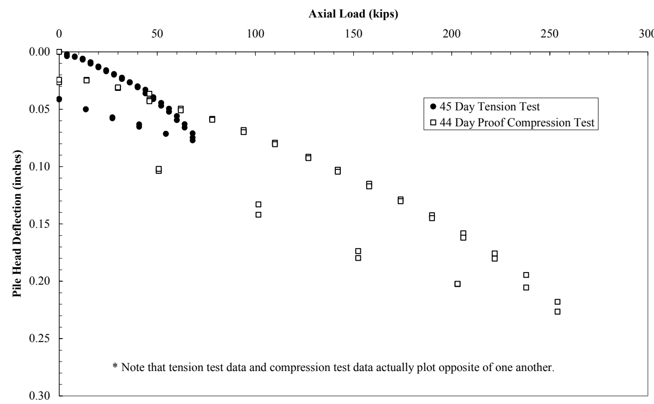

Fig. 11. Comparison of Tension Test Data and Forty-Four Day Compression Proof Test Data.

Comparison of the tensile and compressive load test data indicates that, as expected, the pile will have more capacity in compression than in tension. The obvious reason for this behavior is the end-bearing component, which contributes to the capacity of a pile when loaded in compression, but not in tension. However, the tell-tale data measured during the compressive load tests suggested that only about 10 percent of the load was being transferred at the tip of the pile, resulting in the majority of the load being supported by side friction. If the tensile load-deflection curve is aggressively extrapolated, the tensile capacity of the pile may be on the order of about 110 kips. If this is so, the tensile capacity is approximately 37 percent of the compressive capacity as measured by the forty-four day compressive failure test, which was 294 kips. Furthermore, since an estimated 10 percent of the compressive load was transferred at the tip, the amount transferred in load friction was about 265 kips. Hence, the skin friction capacity estimated from the tension test is approximately 58 percent of the skin friction capacity estimated from the compression test. This significant reduction in capacity seems to suggest that either the end-bearing load transfer of 10 percent, as estimated by the tell-tale data, is not accurate, or that a condition exists in the soil, such as anisotropy, that results in less soil shear strength dependent on the direction of loading.

CONCLUSION

The data measured in this study suggest that the increase in compressive pile capacity for a 42-ft long, HP14x102 pile, in a predominantly fine-grained soil profile, increases by about 16 percent from days seven to forty-four after driving.

It appears evident that Davisson's (1973) failure criteria seems to agree fairly well with the observed plunging failure of the two compressive failure tests. Furthermore, use of the entire elastic compression term from Davisson's (1973) failure criteria, as opposed to only half of the elastic compression term, has relatively no influence on the computed failure load because the pile was experiencing a state of plunging failure at that point in both compression failure tests.

By comparison of the compressive proof and failure tests performed on day forty-four after driving, it appears that loading the pile to some degree prior to failure, unloading, and then reloading the pile, has almost no affect on the load-settlement relationship. Hence, proof loaded piles should be allowed for use beneath the structure.

By comparison of the tension and compression test data, two possibilities have been identified. First, the estimation of tip load by tell-tale data may not be accurate, and may underestimate the amount of load actually transferred to the tip. Or, it seems viable that, at this site, the skin friction that can be counted on in design is perhaps 55 to 60 percent of that calculated for compression.

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